Hydraulic fracture considerations in oil sand overburden dams

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ABSTRACT
There is potential for reservoir fluid to hydraulically fracture through a dam if the minimum stress within the dam is less than the reservoir pressure. Stress and strain conditions are affected by pore pressures, varying levels of compaction in adjacent fill and by underlying pit floor and abutment conditions. In the oil sand industry, the use of well dry-of-optimum fills in temporary dams is driven by the fill materials available and the use of thicker lifts, which are compacted by the 400-Ton payload haul trucks. Three case histories of suspected hydraulic fracture are noted.

RÉSUMÉ
Le fluide contenu dans un réservoir a le potentiel de produise une fracture hydraulique dans une digue si le stress minimum de ce dernier est plus petit que la pression du réservoir. Les conditions de stress sont affectées par la pression interstitielle, le niveau de compaction dans les zones adjacentes, par les conditions du fond de la fosse et du contact entre la digue et les murs de la fausse. Dans les exploitations de sable bitumineux, la construction des digues temporaires doit s'effectuer avec le matériel disponible, souvent de qualité inégale, et aussi composer avec l'étalement de couches épaisses du à la taille de l'équipement minier.

1 INTRODUCTION

At Syncrude Canada Ltd., the use of 400-Ton Payload haultrucks and well dry of optimum fills to build quasi-homogeneous earthen dams for temporary fluid containment results in a higher risk of hydraulic fracture, which needs consideration. The dry of optimum fills are used instead of traditional wet of optimum clay cores. The situation in some cases involves highly dispersive clays that have a higher piping potential than other silty sandy clay fills. Such events of hydraulic fracture in 14 dams were written up by Sherard, Decker and Ryker in 1972 titled "Hydraulic Fracturing in Low Dams of Dispersive Clays", where in their opinion differential settlement at abutments led to low stress zones that hydraulically fractured. Traditional approaches to dams of narrow wetted clay cores combined with non-cohesive, non-crackable sand filters mitigate many, but not all, risks of hydraulic fracture as noted by Kjaernsli and Torblaa (1968) in their paper on leakage through horizontal cracks in the narrow core of Hyttejuvet Dam. In this case, the narrow vertical and near vertical portion of the core arched onto the supporting shells. More settlement occurred in the middle of the core and less occurred at the vertical edges where the outer shell held up the core and prevented it from settling. The arching created a low stress zone that allowed for horizontal hydraulic fracturing by the reservoir fluid. In consideration of this, today most wetted clay cores are sloped on both sides to prevent arching, and widest at the base, where reservoir pressures are highest.

Wetted clay cores of sufficient width and geometry provide significant resistance to hydraulic fracture as long as they are maintained wet-of-optimum during construction and any dried out layers are removed. Significant resistance comes from the pore water within the clay creating a high pore pressure ratio (r_u), which resists the hydraulic forces of a reservoir. Tailings fluids with higher densities of 1.2 to 1.3 times that of water need special consideration, as hydraulic forces are higher in those reservoirs. Historically the potential for hydraulic fracture due to higher density fluid has also been a concern in grouting of instrumentation holes for dams. Marion Houston (1986) documented this with practical procedures for stage grouting of instrumentation to avoid hydraulic fracturing of dams based on the r_u of the fills.

Chimney drain sand filters, which are cohesionless and therefore non-crackable, provide a second line of defence against hydraulic fracture as long as they are capable of quickly draining the considerable fluid volumes that can flow through a crack. Thinner sand filters with low outflow capacity can have considerable head build up within the filter. This fluid pressure build-up has the potential to hydraulically fracture the soil that surrounds the filter. Historically, sand widths have been intentionally generous to deal with the concentrated flows from potential defects. By default, these generous widths also include some degree of protection against hydraulic fracture. Consideration of hydraulic fracture and a crack analysis to size sand filter widths and pipe outflows are integral to filter design within dams of dry-of-optimum compacted fills.

"Overweighting" the dam can create a third or alternative line of defence against hydraulic fracture. Additional fill above the fluid height increases the stresses at depth within the dam to counterbalance hydraulic reservoir forces. Sloping of additional "overweighting" fill over abutment areas can help reduce the differential settlement at these critical locations. Widening the zone of compacted dry of optimum fills at abutments helps account for local irregularities that may occur.

The number and size of dams required to keep pace with the rapid production in the oil sands industry necessitates the use of 400-Ton heavy haulers in dam construction. These large trucks perform significantly faster and better on dry-of-optimum fills, which are also often the most common and readily available fill. The trucks are capable of compacting thick lifts, which would be difficult to effectively and uniformly mix with water.

Hydraulic fracture has the potential to occur if the reservoir fluid pressure is greater than the minimum stresses in a dam. In general, there is additional complexity to this simple balancing of stresses, with the most prominent being crack initiation pressure ("breakdown pressure") and crack propagation pressure. Breakdown pressure (P_b) is the additional pressure required to create a crack if settlement, tension zones or slope movements have not already created a crack. Propagation pressure (P_p) is the additional pressure required to continue propagating a crack through a dam. Both of these are discussed by Barlow et al (1998) with examples of potential pressures for water and heavier mature fine tailings (MFT) fluid. The combination of P_b and P_p affects, along with good construction practices and typical abutment shallowing and smoothing have help provide hydraulic fracture protection to many dams.

2 EARTH PRESSURE THEORY

Earth pressure theory relies on knowing the current state of the soil and whether it is at rest (K_o), in an active state (K_a) or passive state (K_p). Simple Rankine earth pressure theory would give:

$$K_a = \frac{1 - \sin\phi'}{1 + \sin\phi'} \qquad [1]$$

$$K_o = 1 - \sin\phi' \qquad [2]$$

To illustrate, effective and total horizontal stresses for K_a and K_o conditions are shown in Figure 1 for an unsaturated ($r_u = 0.0$) through to a saturated ($r_u = 0.5$) silty sandy clay with a peak shear strength of 33°, as determined by the secant method. Syncrude also uses another consideration of halfway between K_a and K_o called $K_{average}$. Figure 2 shows the horizontal stress calculations for multiple soil layers.

There is also a minimum top width of a dam required to assume full influence at depth, or a discounting of the height must be used. The minimum top width is suspected to relate to the size of two active wedges that would develop plus an additional width to provide a margin of factor of safety.

It is extremely difficult to know the stress state within the soil, but relatively easy to understand that lower stresses exist at locations like abutments (where fill thicknesses vary quickly), high foundation points on an otherwise flat base (creating differential settlement and potential tension zones), or areas with poorer quality fills (which are prone to more settlement). Engineering judgement is required to assess potential critical areas.

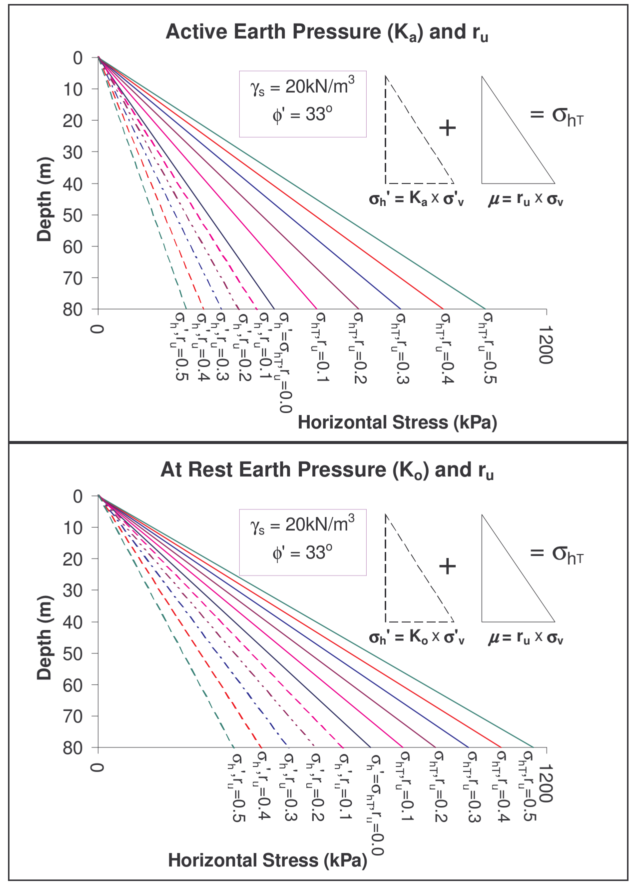
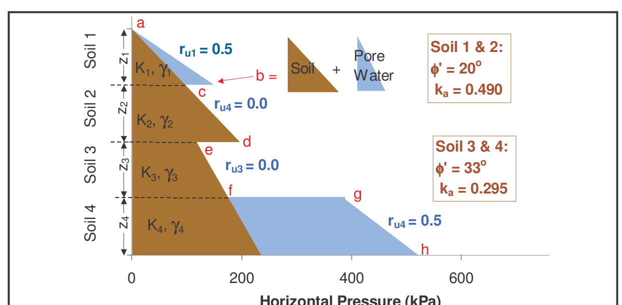


Figure 1. K_a and K_o Earth Pressure For Unsaturated, Partially Unsaturated and Saturated Soil



Total Horizontal Stress at Labelled Points:

$\sigma_H(a) = 0$

$\sigma_H(b) = k_1\gamma_1 z_1(1-r_{u1}) + \gamma_1 z_1 r_{u1}$

$\sigma_H(c) = k_2\gamma_1 z_1(1-r_{u2}) + \gamma_1 z_1 r_{u2}$

$\sigma_H(d) = \sigma_H(c) + k_2\gamma_2 z_2(1-r_{u2}) + \gamma_2 z_2 r_{u2}$

$\sigma_H(e) = k_3\gamma_1 z_1(1-r_{u3}) + \gamma_1 z_1 r_{u3} + k_3\gamma_2 z_2(1-r_{u3}) + \gamma_2 z_2 r_{u3}$

$\sigma_H(f) = \sigma_H(e) + k_3\gamma_3 z_3(1-r_{u3}) + \gamma_3 z_3 r_{u3}$

$\sigma_H(g) = k_4\gamma_1 z_1(1-r_{u4}) + \gamma_1 z_1 r_{u4} + k_4\gamma_2 z_2(1-r_{u4}) + \gamma_2 z_2 r_{u4} + k_4\gamma_3 z_3(1-r_{u4}) + \gamma_3 z_3 r_{u4}$

$\sigma_H(h) = \sigma_H(g) + k_4\gamma_4 z_4(1-r_{u4}) + \gamma_4 z_4 r_{u4}$

Figure 2: Horizontal Stress Calculations for Multiple Soil Layers

In Situ testing methods are not capable of testing all areas within a dam for potential design or construction weaknesses. Variations in soil type, compaction, water content, speed of construction, and weather during construction, all effect soil behavior. Assuming one can quantify the best and worse areas and make reasonable engineering judgements, it is then possible to consider the effects of fluid pressure from a reservoir on the potential minimum stresses in a dam.

3 FLUID PRESSURE COMPARED TO MINIMUM EARTH PRESSURES WITHIN A DAM

When considering water and MFT reservoirs against the simple example in Figure 1, Figure 3 shows that for soil in the K_a condition, r_u values of 0.25 and 0.45 are required to match water and MFT ($1.3\gamma_{water}$) reservoir pressures, respectively. For soil in the K_o condition, r_u values of 0.03 and 0.29 are required to match the same water and MFT reservoir pressures. This does not account for the added benefits that occur due to breakdown and propagation pressures, but provides a lower end comparison where one might consider a factor of safety of 1.0. Risk tolerance and probability of occurrence along with consequence of failure must be evaluated case by case, as well.

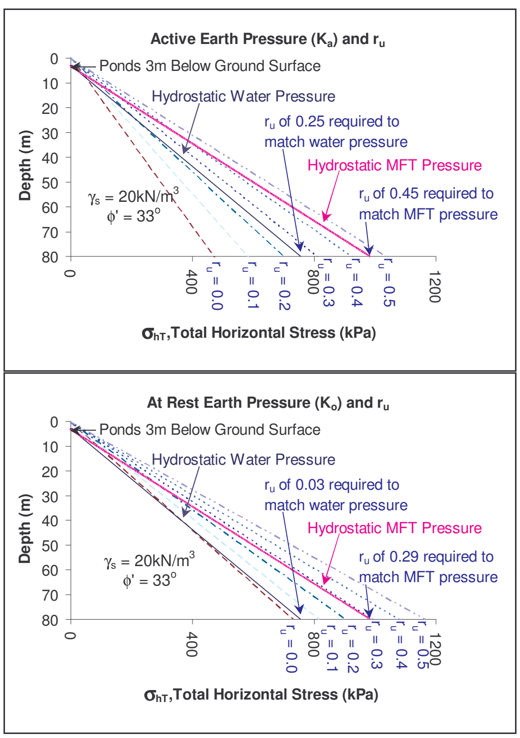


Figure 3: Water and MFT Reservoir Stresses Balanced By Varying Earth Pressure Conditions and r_u

If the soil r_u cannot be increased, as is the case for 0.75m to 1m thick lifts which are impossible to mix thoroughly with water and then compact with 400-Ton payload haulers, an alternative is to raise the height of the soil to allow for the same pond height, as shown in Figure 4. Only K_o soil conditions and a water reservoir are shown in Figure 4. If K_a soil conditions were shown with an MFT fluid reservoir, then a considerably higher soil height would be required to balance the reservoir pressures. Therefore it is important to prevent full K_a conditions from occurring by flattening abutment slopes and eliminating foundation irregularities wherever possible.

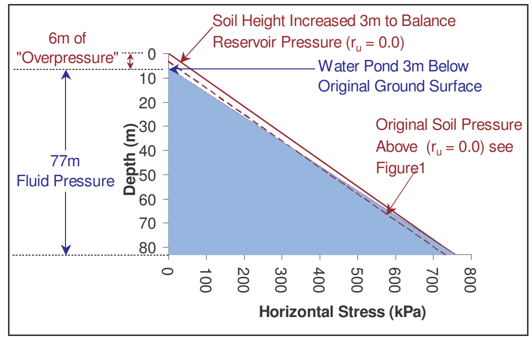


Figure 4: Increasing Unsaturated Soil Height to Balance Water Reservoir Pressure

4 SELECTION OF K_a, K_o, K_p AND *IN SITU* SOIL STRESSES

Engineering judgement in the selection of appropriate earth pressure coefficients requires quantitative understanding of the soil stress state, which revolves around the amount of soil strain that is likely to occur over time, after the soil has been placed and compacted. In some cases of additional loading, the minimum stress will increase, so continuing to build after initial reservoir infilling can be beneficial. However when construction is complete and no further stress is applied, the minimum stresses can begin to decrease over time. Engineers understand that abutment areas are most prone to tension cracking at the surface due to differential settlement related to consolidation of the rapidly changing fill thicknesses. However, Sherard's studies of 14 dams in 1972 showed that even without tension cracks fully developing, lower stress zones can set up and be lower than the reservoir pressure, making it possible for the reservoir to vertically hydraulically fracture through the lower stress zones. This scenario is shown in Figure 5.

Another scenario, which has the potential to create low stress zones, exists when soil conditions vary rapidly across the base of a dam (from significantly more compressible fills to stiffer *In Situ*, for example). This can be bedrock sticking up higher in an area (e.g. a reject oil sand island with steep slopes, left behind and not mined), irregular fill lift thicknesses, or incorporating existing fills or dumps within a dam. These conditions are illustrated in Figure 6.

The worse case "low stress zone" scenario is a tension crack perpendicular to the dam axis with direct connection to the reservoir. There are many degrees of lower stresses that occur well before the tension crack state, starting from K_o at rest and working towards the active K_a state, and in the extreme case to below Ka in tension zones. This is where the quality of the fill and its compaction along with foundation and abutment conditions determine its resistance to differential strain. With higher quality compacted fills, level base preparation and shallow sloping abutments, K_o conditions would be expected. For these same conditions, it can be expected that if the reservoir fluid pressed against the sides of even a very small existing crack, the crack sides would allow very little strain and resist the reservoir pressure with additional stress, trending towards K_p as in the Figure 7. For less compacted fills, or fills at K_a close to an abutment, fluid in a crack would meet little additional resistance, especially if in an area of differential settlement allowing for further crack opening, crack propagation and more strain with little to no stress increase (see Figure 7).

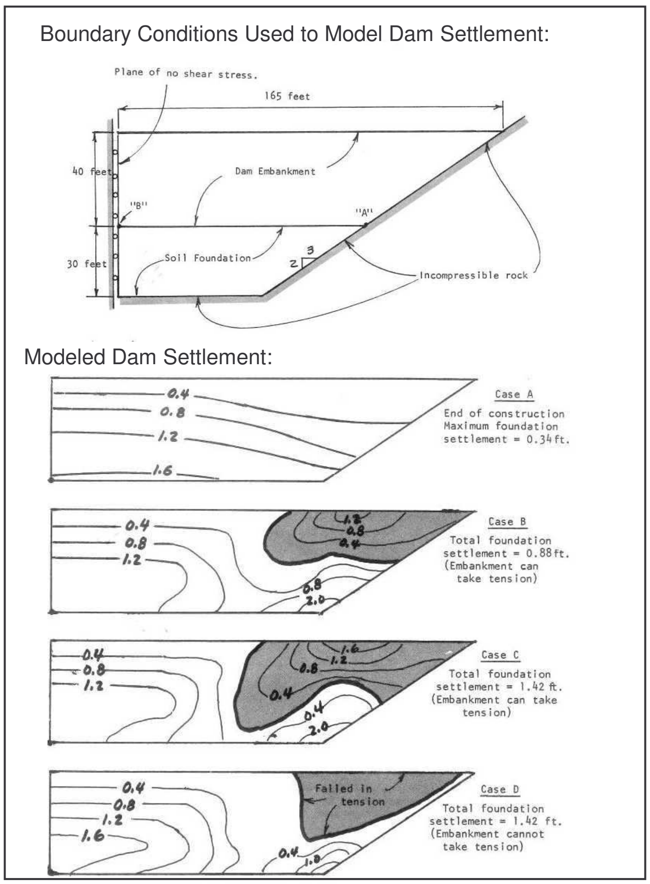


Figure 5: Modified from Sherard (1972) Finite element analysis on the affects of differential foundation settlement on internal dam stresses

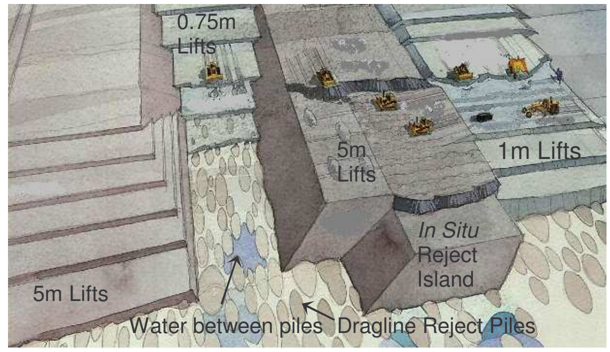


Figure 6: Illustration of varying soil conditions which can create low stress zones

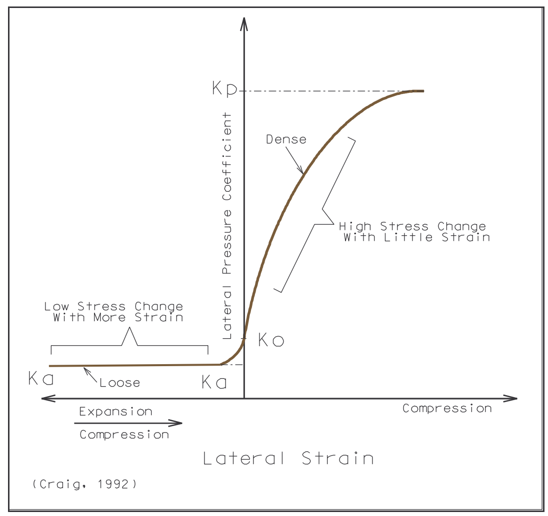


Figure 7: Relationship Between Earth Pressure Coefficients and Strain (After Craig, 1992)

To fracture through well-compacted fills, a lower stress zone would have to develop in the fills near the abutment along the entire abutment (or other zone of weakness through the full width of the dam). Otherwise the higher stresses in the fills behind the initially fractured area would shut down the hydraulic fracture. Once partially cracked, it is possible to have softening of the crack tip and surrounding soil lead to potential crack closing. Barlow et al (1998) found that for shallow, thin lifts (1m) compacted with fully loaded 240T heavy haulers, the minimum soil stress direction was vertical. Still, Syncrude uses an earth pressure coefficient of only 0.7 for these fills to a depth equivalent to the truck pressure (approximately 30m), as long as the compacted lifts were not placed above deformable fill.

In determining the appropriate earth pressure coefficient to represent the existing and future dam stresses, the depth and confinement of soil are also important considerations. Syncrude generally considers K_o conditions for confined soil at depths greater than approximately 30m (even in areas of deformable foundations).

4.1 A Syncrude Example Of Construction Techniques Used To Increase Minimum Soil Stresses

The Highway Berm and SW Dam in Syncrude's Base Mine were both built on top of and around 15m high oil sand reject piles that existed prior to construction in an "egg carton" geometry. The size and magnitude of these piles made it impractical to remove them. It was more practical to overload these piles with large quantities of mine waste fills and rely on very wide soil widths. Generous widths allowed for more opportunity for hydraulic fracture to be shut down by the existence of better stress conditions due to varying soil types, r_u, or placement conditions. Sloppier fills with higher pore pressures offered more resistance against hydraulic fracture (see Figure 3) at the expense of increased deformation, instability and risk of internal erosion. Large widths of more compacted fills of similar density were used as continuous control elements because they are less likely to differentially settle along the dam axis, and therefore more likely to maintain "as-placed" stresses due to even strain. The earth pressure conditions of these large well compacted fill widths are more akin to K_o than K_a, which is more desirable (see Figure 3).

4.2 Calibrated Syncrude Model of Earth Pressure Coefficients

Syncrude has calibrated its model of earth pressure coefficients using piezometer response to reservoir filling (see Syncrude Case Study #2). When assessing piezometer response as input to a model of existing earth pressure coefficients, considerable care must be taken to ensure that minimum earth stresses are represented. At Syncrude, the minimum soil stress for deep, thin (1m), well-compacted lifts back-calculates to $1.08K_o$. The minimum soil stress for deep, looser, thicker lifts back-calculates to $0.9K_o$. These back-calculated values inherently include P_b and P_p pressures. It should be noted that the soils in Syncrude's Case Study #2 may actually have have existed in the K_a or $K_{average}$ stress state, and that P_p and P_b pressures brought the stresses up to the $1.08K_o$ and $0.9K_o$ values discussed above. These minimum soil stress values may be conservative for dams built above less deformable foundation conditions. Appropriate factors of safety above these minimum values are required; Syncrude considers a 1.15 minimum factor of safety.

5 SELECTION OF r_u

The previous section discussed the selection of the earth pressure coefficient that best represents the soil conditions. Of equal importance, is selecting the representative r_u value used in the design analysis. For slope stability analyses, a higher r_u value is considered more conservative. However, for hydraulic fracture analyses, lower r_u values are more conservative. It is very important to get this correct, as the driest fills are most likely to have the greatest risk of hydraulic fracture. It is also important to understand whether the fill has a typical lower bound r_u (usually the case for partially unsaturated fills) or whether it acts as a water table, like from a seepage front. Water slowly seeping into the dam fills and saturating helps resist hydraulic fracture forces, but should not be considered an r_u. Figure 8 shows that the misinterpretation of a water table as an r_u can lead to errors in the estimated pore pressure, and therefore in the estimated resistance to hydraulic fracture.

The idea that a lake does not fracture the underlying ground has been postulated as a natural comparison of fluid containment where hydraulic fracture does not occur. Natural lakes form over long periods of time with the ground below becoming saturated in the process. A piezometer tip in the ground below a lake would read the lake elevation (a piezometric elevation above ground). As Figure 3 shows, higher pore pressures increase resistance to hydraulic fracture. Figure 9 shows that the high pore pressures in the soil beneath a lake would balance the lake fluid pressures, and prevent hydraulic fracture regardless of P_b or P_p values. If the same fluid was impounded above or beside unsaturated lifts, the dry fill lifts would be more likely to hydraulically fracture.

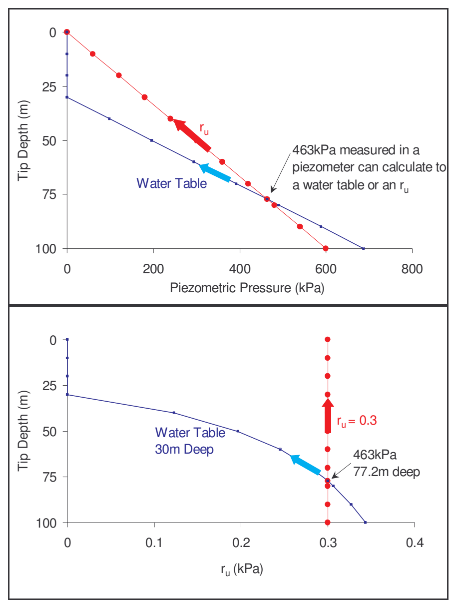


Figure 8: Comparing Piezometric Pressure With Depth for r_u and Water Tables

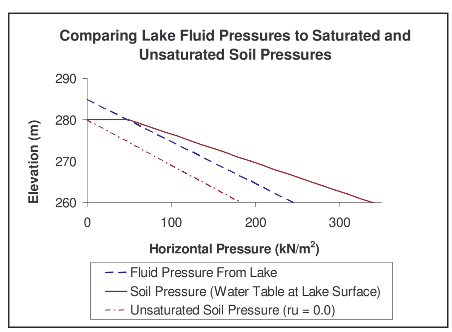


Figure 9: Comparing Lake Reservoir Pressures to Saturated and Unsaturated Soil Pressures

6 SYNCRUDE CASE STUDIES

Hydraulic fracture is generally observed as a piping failure, since that is what it progresses to. Hydraulic fracture also tends to find existing weaknesses, and is very difficult to distinguish as its own distinct failure mechanism.

6.1 Syncrude Case Study #1 at Coke Cell #5

The best example at Syncrude is the suspected hydraulic fracture of the south dyke of Coke Cell #5. This dyke had an imperfect upstream coke beach, which allowed for non-cohesive coke to infill any hydraulically fractured cracks, preventing outright piping failure. The original dyke was composed of thick lifts of non-uniform overburden and reject oil sand (not commonly done today) with partial coke beaching. During pouring of coke into the cell, it was noted by operations that the 250m by 150m by 4m deep pond, which typically required pumping, had disappeared. The south slope was inspected and evidence of 14 small seepage points were noted between approximately 293m to 300m elevation (see Figure 10). Although there was little to no visible flow, these seepage points remained wet for years, even under the driest weather conditions. Hydraulic fracture was suspected to have occurred at the elevation of these seeps, based on the existence of slightly lower than $K_{average}$ soil conditions (higher soil stresses than K_a, but not as high as K_o), see Figure 11. At 290m elevation, the water pressure in the coke back-calculates to an existing earth pressure of 79% of K_o, with no P_p or P_b.

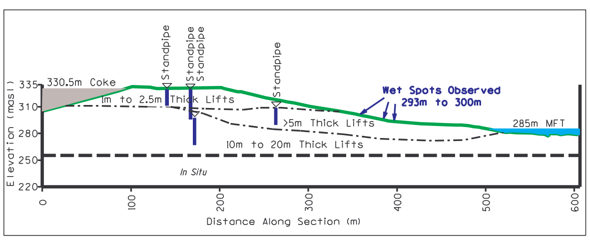


Figure 10: Coke Cell 5 Cross-Section

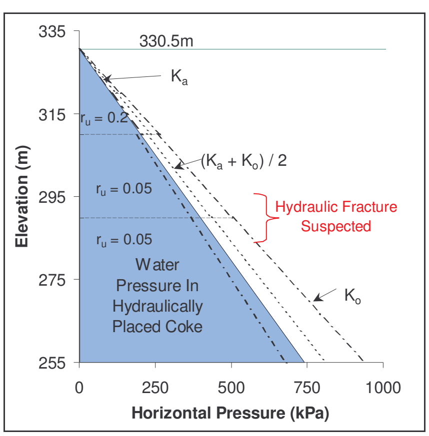


Figure 11: Coke Cell 5 Hydraulic Fracture Graph

In response to this occurrence, hydraulic placement of coke was stopped, and the coke cell was closed to any further coke placement. The coke cell is now being successfully dewatered for final decommissioning.

This example highlights the need to have continuous compacted control elements and stresses the important role of upstream beaching. For overburden dykes that have a risk of hydraulic fracture, large sand or fine coke upstream beaching in place of an internal sand filter have additional benefits to the overburden dyke, in terms of hydraulic fracture. The non-cohesive fine beached sand or fine coke, when wide enough, allow only water to touch the overburden fills and do not allow ingress of the heavier tailings fluids. This reduces the hydraulic forces on the dam by up to 20 to 30% in some cases. The upstream beach itself is non-crackable so has a controlling influence on the flow of water through it. The upstream beach, when thick enough, can provide infill material to be carried into a crack to help reduce the risk of an enlarging piping failure. Although the infilled crack has the ability to weep or leak, it becomes a manageable issue with greater time frames to remediate or resolve. Such beaching needs to be large and dimensionally wide enough to keep the tailings fluid reservoir sufficiently away from the overburden fills. Such upstream beaches also provide infilling potential for other defects and joint issues. However, they do not provide the same level of control of outflow from a dam as an internal filter.

Dams with continuous compacted overburden control zones with upstream beaches are cost effective approaches for mining areas where temporary dams are required and consequences of failure, ability to remediate and risk benefits warrant.

This example highlights another reason why loose uncompacted lifts should not be used as control elements in fluid retaining structures. The next example shows higher soil stresses, which result from better construction practices.

6.2 Syncrude Case Study #2: SW Dam

In general, Syncrude's SW Dam continues to perform much better than expected in the original design analyses. The south abutment, which is thought to have hydraulically fractured, has afforded an opportunity to calibrate Syncrude's model of minimum earth pressure coefficients. The zone of well compacted, thinner lifts appears to have hydraulically fractured due to a pond that was 2m higher (38.4m total pond height) than originally predicted. The zone of looser thicker lifts appear to have hydraulically fractured earlier than originally predicted (2.8m earlier on a 38.4m pond).

The SW Dam is actually two dams in one to allow for initial pond infilling on the east side, while downstream mining continued, and eventual filling of the west side to a higher elevation than the initial eastside pond. The outer eastside compacted zone, was built to hold the initial interim eastside pond elevation, while the second higher clay rich compacted zone was being constructed to the west. This second higher western compacted zone was built to hold the final full height westside and eastside ponds. Between the 2 control elements, a zone of looser, thicker lifts was placed. This zoning was partly to decrease the risk of a continuous defect occurring through the full width of the dam. It was expected that the outer eastside compacted zone would hydraulically fracture at some point in time (dependent on actual P_b and P_p values), and in the interim, the second western control element would become the main line of defence for both ponds.

The SW Dam was built "overtop" unsaturated, medium-dense oil sand-reject piles, which existed in an "egg-carton" geometry on the pit floor. In general piezometers within the oil sand reject piles, behind the outer eastside compacted zone, currently read less than 15m of head, except for those piezometers within 50m of the south *In Situ* abutment. Piezometers within 50m of the south abutment currently trend towards the *In Situ* abutment piezometric elevation. The hydraulic fracture event was initially noted when a piezometer, within 50m of the abutment, abruptly changed from its gentle, upward seepage trend and went almost as high as the reservoir. The water inside the standpipe was blown out only to recharge immediately. The piezometer history is shown in Figure 12.

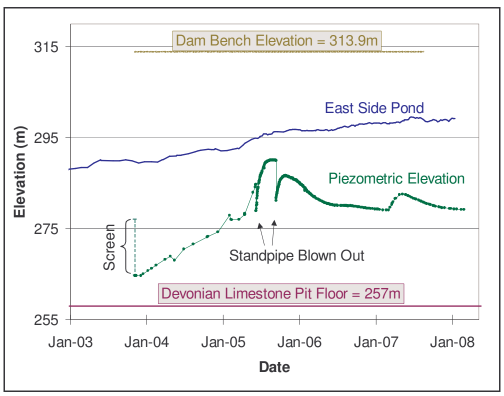


Figure 12: SW Dam piezometer response to hydraulic fracture

Piezometers installed along the abutment, after the hydraulic fracture event, read lower than the standpipe in Figure 12, ruling out defects along the abutment as the cause. It appears that a hydraulic fracture occurred through both the compacted eastside zone and the looser fills behind the compacted zone (see Figures 13 and 14). The hydraulic fracture occurred close to the abutment, pressuring up the oil sand rejects where the piezometer close to the abutment was able to read the higher pressure. The hydraulic fracture was not thought to have occurred at the piezometer or to have been caused by its installation because it was installed using dry reverse circulation drilling and staged grouting procedures. A downhole camera was sent down the piezometer to look for any evidence of casing breakage, casing leakage or surface water ingress and no evidence was obtained, even one year later. Photos of the area, at the time of the abrupt change in piezometric elevation, show dry surface conditions around the piezometer. The SW Dam design accounted for such an event and the higher loose fills to the west and the next compacted zone within the dam appeared to stop the fracture three days prior to the accelerated placement of fill on top of the dam to ensure sufficient factor of safety was maintained. Many additional piezometers were installed and it is striking how many have not responded to the 40m of tailings fluid head. It is suspected that the crack softened and closed, as eventually the area drained again to normal seepage trends. One subsequent event near this piezometer location may have occurred two years later, but not to the same degree. It is suspected more events will occur as the ponds are raised close to their final elevations, when K_o and P_p could be exceeded in the tertiary control elements, which are not relied on as the primary control zones.

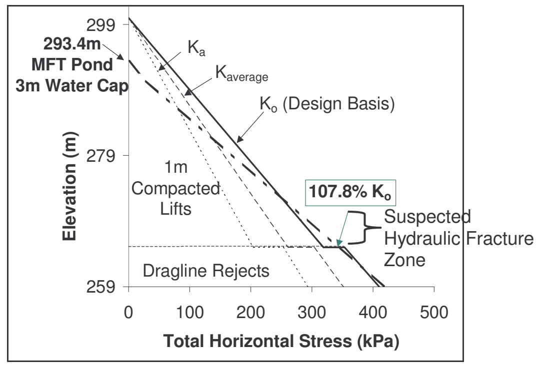


Figure 13: SW Dam Hydraulic Fracture Graph for the Outer Eastside Compacted Zone

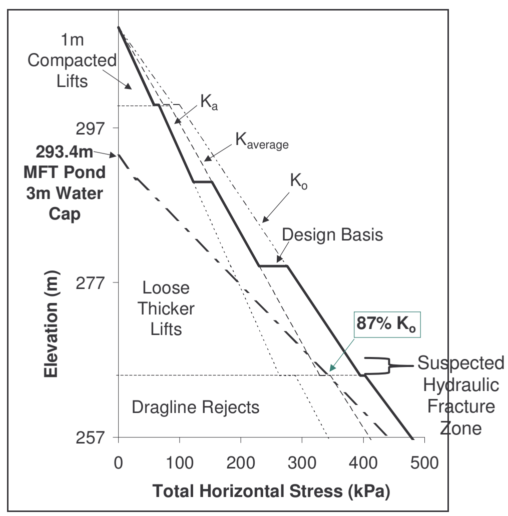


Figure 14: SW Dam Hydraulic Fracture Graph For the Loose Thicker Lifts Between the Two Compacted Control Elements

In order from worst to best construction practices, existing earth pressures at Syncrude have been back-calculated to $0.79K_o$ (Coke Cell 5 lifts, 5m to 20m thick) to $0.87K_o$ (SW Dam 5m lifts) to $1.08K_o$ (SW Dam 1m lifts), with better compaction but still above a deformable foundation. In terms of K_a, these values back-calculate to $1.08K_a$ to $1.30K_a$ to $1.68K_a$. The soils could be considered to be in the K_a condition with 8% to 30% to 68% P_p or P_b. P_b is considered unlikely to occur in these structures above deformable foundations, where some cracks or defects likely already exist. P_p could be expected to be in the range of 0 to 20% or 0 to 40% for hydraulic fracture by water or MFT respectively. (AGRA,1998).

6.3 Syncrude Case Study #3: Central Haulroad Berm Spillway

In another area in the Syncrude Base Mine, an interconnecting internal spillway between ponds failed, and was monitored for learnings. During the event, a very large erosion channel was created and monitored. Prior to fill placement to stop the flow over the spillway, in a single night, 25m of crest was lost within a canyon type geometry (see Figure 15). The fluid in the canal found and entered a crack. Although the three dimensional confinement was significant, fluid was able to enter a crack and push out the section of fill. Two days earlier, when 6m of crest loss occurred overnight, fluid had traveled in a crack and exited downstream at a lower elevation some distance away from the canyon. Where the fluid exited from the open crack further back, the fill between the crack and the channel did not fall (as would be predicted for a tension crack filled with fluid, but not for crack propagation). The exiting fluid and the intact soil above can be seen in the two photos with the failure scarp (waterfall) in the background in Figure 16. Though the failure mechanism is simple (fluid filling in a crack), it certainly highlights the forces these fluids exert to remove 25m back and 20m high overnight in a confined canyon and to be able to travel (and possibly propagate and possibly hydraulically fracture) through and out the side.

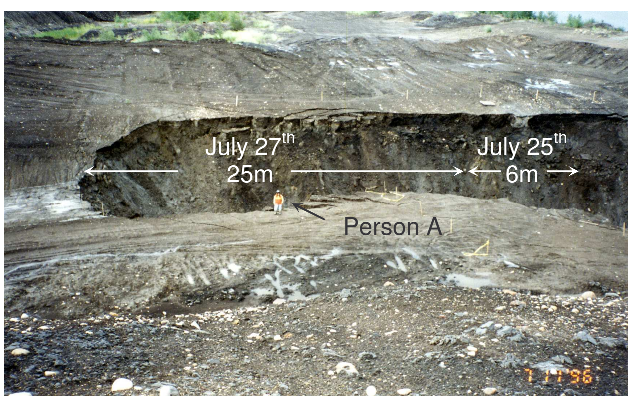


Figure 15: Spillway crest loss over two days

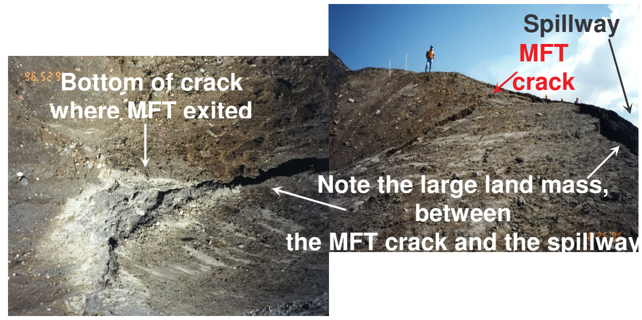


Figure 16: MFT exited from a crack in a confined area well away from the spillway channel

7 CONCLUSION

Given the large hydraulic forces and potential for catastrophic failure, hydraulic fracture failure modes need consideration in dam design. The issue is of particular importance when thin compacted lifts of dry of optimum fills replace wetted clay cores. Good construction practices and the elimination or shallowing of *In Situ* bedrock irregularities and abutments are important to reduce the potential of hydraulic fracture. Sufficient overpressure height, abutment sloping, sloping of fills above abutments, dams width and base conditions must reflect the importance of hydraulic fracture as a consideration. Upstream sand beaches or internal filters also offer a primary line of defence, pending risk benefit considerations, for dams with dry of optimum compacted control zones.

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