Residual shear strength direct shear testing in Clearwater Formation clays to model actual field conditions

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ABSTRACT
Historical performance of overburden dumps and tailings dams at Syncrude Canada Ltd. over the last 25 years has shown the field residual shear strengths for the high plasticity Clearwater Clays are consistently around 8 degrees and higher. Numerous back analyses with good pore water pressure information have confirmed this over a variety of conditions. In the past, difficulties have arisen in selecting design strength parameters for the Clearwater Formation clays due to conventional lab testing procedures for high plastic clays. Residual direct shear strength lab testing for these clays must in general, more representatively model the field pore fluid chemistry for the water used in the test bath.

RÉSUMÉ
Les performances historique des haldes à stérile et digues de rétention des résidus à Syncrude au cours des 25 dernières années ont démontrées que la résistance au cisaillement des glaises de la formation Clearwater à haute plasticité sont constamment autour de 8 degrés et plus. Plusieurs analyses, basées sur une sélection variée de gradient de pression interstitielle, ont confirmées ce fait. Par le passé, l'utilisation de méthodes analytiques conventionnelles pour les glaises à haute plasticité ont causées beaucoup de difficultés dans la sélection des paramètres de cisaillement des glaises de la formation Clearwater.

1 EFFECT OF PORE FLUID CHEMISTRY

The residual strength of overconsolidated clay-shales, especially marine environment deposition, is a function of the *in situ* pore fluid chemistry. It is well recognized that the residual angle is controlled by deformation and the development of a shear induced fabric. But the complementary effect of salt content is much less understood. Fundamental research on the importance of physico-chemical effects was clearly laid out by Chatterji (1973) and eventually published in Chatterji and Morgenstern (1990). Laboratory testing reporting the effect of salt content can be found in Kenney (1967), Moore (1991), DiMaio and Fenelli (1994), Anson and Hawkins (1998), and Tiwari et al (2005).

Physico-chemical forces of repulsion and attraction exist in swelling or active clay-water systems, and Chatterji (1773) extended the Terzaghi concept of effective stress to account for the net physico-chemical forces of repulsion and attraction (R-A) as follows.

[1] $\sigma^*_n = \sigma'_n - (R\text{-}A)$

where σ'_n is the apparent effective stress as defined by Terzaghi, and σ^*_n is the true effective stress accounting for physico-chemical effects. In an elegant series of experiments, Chatterji (1993) as well as Balasubramonian (1972) showed that this modified effective stress law could be applied to both the strength and the swelling characteristics of clay shales respectively. Chatterji's experiments showed that the residual strength not accounting for [R-A] effects was a variable quantity and that active clay minerals appear to possess a true residual angle which is unique and independent of pore fluid considerations. From a practical perspective, geotechnical practice is facilitated by using σ'_n the apparent effective stress as defined by Terzaghi as the difference between the calculated total stress and the measured pore pressure. It then follows that the angle of residual friction determined in a direct shear test is a variable depending on the pore fluid effect. Herein, the angle of residual friction consistent with conventional determination of effective stress is adopted, and it is expected that such a friction angle is a variable depending on pore fluid salt content.

1.1 Issues in Conventional Direct Shear Testing

Common direct shear testing practice ignores the pore fluid effect. Samples are usually flooded with distilled water, allowed to equilibrate, and then sheared. As the sample equilibrates during initial loading water is drawn into the swelling specimen. Additional water can then be drawn into the sample depending on how shearing progresses after peak is reached. In some laboratories the sample is sheared back and forward until a relatively constant residual strength is reached. It is often observed that the strength can be slowly reduced to a progressively lower value as shearing progresses. In some laboratories a thin wire is drawn through the sample as this is found to allow the sample to more quickly get to the “terminal” angle. Martens and Charron (2007) go further and recommend saw-cutting the sample, polishing the failure plane with a glass plate, and reassembling the sample. The wire-cut or glass polishing procedures maximize the ingress of distilled water to the failure plane and is considered a questionable approach. As will be shown, the mobilized residual angle of active clay minerals with a significant cation concentration is greater than the residual angle obtained with distilled water. Therefore, it follows that direct shear testing which promotes the ingress of distilled water into the failure plane will result in an artificial reduction in inferred residual depending on the *in situ* pore fluid chemistry. The search for the lowest or terminal residual angle in direct shear testing may in fact be an inappropriate search for the angle dominated by distilled water.

1.2 Strength of Pure Clay Minerals

From a geotechnical perspective the mineralogy of Clearwater clay-shales is dominated by smectite and mixed layer swellable clay minerals, see for example Dusseault and Cimonlini (1981). Other clay minerals are reported as kaolinite, illite, and chlorite in variable proportions and as reflected by a wide range of liquid limits from 30% to over 200%. The strength of pure clay minerals provides a benchmark against which to assess testing of natural samples. Table 1 lists the main clay minerals reported in the Clearwater as well as the specific surface. The specific surface is a measure of the relative size of the clay minerals and also provides an indication of the relative importance of physico-chemical effects.

Table 1. Main Clay Minerals in Clearwater Formation

Clay Mineral	Specific Surface[1] (m^2/g)	Residual Angle (degree)
Smectite	800	4 to 10[+]
Illite	80-100	10
Chlorite	5-50	11
Kaolinite	10-20	12

[1] Mitchell (1976)

There are several available data sets in the literature for the residual strength of pure smectite clays with distilled water as the pore fluid. The reported residual angles are summarized in Figure 1, and are shown with dashed lines. It can be seen that even for pure minerals there is a range in reported strength as a function of the effective normal stress. This may relate to different levels of Na or Ca absorbed on the clay and the degree to which the samples were washed with distilled water before testing.

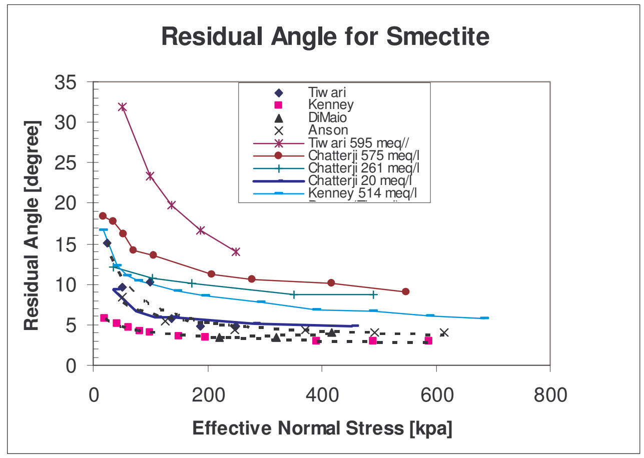

Figure 1. Pure Smectite Clay Residual Angles

Figure 1 also indicates an important trend especially with smectite clays where the residual angle is dependent on the apparent effective stress. At effective stresses above about 400 kPa the residual angle is convergent on from 3 to 4^{o} with distilled water. Figure 1 presents the results of Smectite for a range of salts added to the clay and for a range of cationic content measured by meq/l. The data set from Kenney (1967) at 514 meq/l is less than Chatterji (1973) at 261 meg/l. Tiwari et. al (2005) results for 595 meq/l is considerably greater than Chatterji 575 meq/l. This may be due to Tiwari’s use of representative Na/K/Ca/Mg salts whereas Kenney and Chatterji used a NaCl solution. The difference between the data sets of Kenney and Chatterji may also be due to the source of the clay mineral, and its purity.

It can be seen that an increase in residual strength results from the introduction of salts, and that the resulting strength also depends on effective stress.

At the low end of the specific surface spectrum, testing on kaolinite in many of the references cited indicate little effect from salt content. No research has been located on the salt effect on illite clay minerals, but given the specific surface it is possible that there may be some effect. This may have some impact on Clearwater strength, which can often have a high illitic clay percentage.

Based on these data sets, it can be deduced that residual strength angles of from 3^{o} to 4^{o} can only be expected with pure smectite and distilled water. This is a highly unlikely circumstance for natural clays like the Clearwater deposited in a marine environment.

Table 2. Methods used to Extract Pore Water from Overconsolidated Clay-shales

Soil	Extraction	Na	K	Ca	Mg	meq/l	Comments/References
Bearpaw Clay-shale	Squeezer at 5 Mpa	2667	53	274	215	149	Balasubramonian (1972)
Bearpaw weathered		1612	28	513	963	175	
Morden Clay-shale		4200	210	430	265	231	
Devon Clay-shale		314	18	184	55	28	
Clearwater	Centrifuge 180 Kg	6370	204	442	639	357	Issac & MacKinnon (1987)
Clearwater	Squeezer 2-3 days	7590	114	506	662	412	Dusseault & Cimolini (1981)
	at 70 Mpa	3190	28	222	569	197	followed by centrifuge
		6870	93	4589	769	594	reports Na content 2 to 4
		1000	296	466	424	109	times squeezer results
		6130	79	524	346	323	
This Study							
Kcc6	Centrifuge	1575	22	17	6	70	Dilution factor = 30.7
	5:1 extraction	5010	7630	280	62	432	
	Squeezer at 10 Mpa	1050	14	27	4	48	
Kcc2	Paste Extraction	9039	303	13	5	402	Dilution factor = 9.3
		3400	72	21	4	151	Dilution factor = 8.0
	Centrifuge	1480		87	48	73	
	5:1 extraction	4400	7800	700	50	430	Dilution factor = 31.4
	Squeezer at 10 Mpa	610	27	7	3	28	
	Squeezer at 20 Mpa	280	5	8	3	13	

1.3 Determination of *In situ* Pore Fluid

For natural clay-shales laid down in a marine environment it does seem highly unlikely that the residual strength would approach or equal that of pure smectite with a distilled water pore fluid. Determining a representative physico-chemical effect can be seen to be a necessary aspect of residual strength determination.

Table 2 provides a summary of various methods used to extract pore water from overconsolidated clay-shales.

One of the first attempts at least within the geotechnical literature to sample pore fluid was reported by Balasubramonian (1972) using a cylindrical press or squeezer. Dusseault and Cimolini (1981) extended the use of this squeezer into the Clearwater clay-shales. Syncrude researchers Isaac and Mackinnon (1987) presented the results from a centrifuge test at 180,000g.

Table 2 also presents the results of a split sample on a Kcc6 facies (Kcc - Cretaceous Clearwater Clayshale) in which 3 different tests were done. Also shown are the results on Kcc2 facies with different methods. The solution extraction tests involve drying the sample, then adding a measured amount of water and extracting the water from the paste. In all cases reported in Table 2 the anionic content of the extraction sample is factored back to the intact core water content.

This and other testing indicate a wide range of salt contents depending on extraction method. Table 3 focuses on the 5 samples reported by Dusseault and Cimolini (1981) in which the same method was used for 5 samples, difference facies, and at varying elevations in the stratigraphical sequence. This test series compares anion content versus several index parameters. No clear correlation is indicated underscoring the difficulty in determination of representative salt content.

Table 3. Squeezer Extraction (Dusseault and Cimolini 1981)

Depth above Km (m)	meq/l	facies	Clay Smectite (%)	Smectite % of Total	w/c (%)	LL (%)	A
13.6	594	Kcb	30	26	29	147	1.22
18.0	412	Kcc	30	21	25	114	1.28
5.4	323	Kca[1]	<5	0	14	0	0
15.9	197	Kcc	10	5	12	54	0.81
11.5	109	Kcb	35	15	16	55	0.89

[1] described as sandy clay bitumen, glauconitic

1.4 Laboratory Testing

1.4.1 Limestone Clay

Consideration of the overall stability of major structures in the Fort McMurray area focused on the possible contribution of a highly overconsolidated layer of clay in the local Devonian limestone. Initial testing of two intact samples reported peak angles of 12.6^o and 13.4^o and residual angles of 7.4^o and 10.7^o, Hardy Associates (1982). The lower residual angle was of concern, but as the direct shear test had been run with distilled water, and given the likely pore fluid at the stratigraphical location of the clay, it was recognized that the test results might be over-conservative.

The chemistry and mineralogy was investigated by the Department of Soil Science at the University of Alberta and the results are reported in Table 4. The reported pore fluid chemistry was simulated by a mixed salt solution of 23 meq/l and two additional direct shear tests were run on remolded samples consolidated by a slurry. A control test was conducted with distilled water.

Table 4. Residual Shear Strength with Pore Fluid Consideration for Clay related to the Devonian Limestone

Fluid	Atterberg Limits		Direct Shear At 1245 kPa	
	w.	w_p	Peak	Residual
distilled	34%	16%	13.5	7.4
23 meq/l[1]	49%	18%	16	11.4
Mineralogy of 60% clay size	Kaolinite 55-65%	Mica 45-35%	Total CEC (me/100g) 14, 17, 19 on 3 samples	

[1] 23meq/l = Na @ 15 + K @3.5 + Ca @ 0.5 + Mg @ 4.0

Approximately 40% of the sample was sand and silt size consisting of siderite and quartz, with the 60% clay sizes being reported to be Kaolinite and Mica. Given the low cation exchange capacity CEC of 14 to 19, the testing laboratory considered that the clay sizes were dominated by non-expanding clay minerals. Difference in Atterberg Limit tests supported the influence of pore fluid effects. Therefore, while the low residual angle with distilled water was lower than expected given the mineralogy, the difference to design of 4^o was significant as it represents a 54% increase in strength. Of interest was the intact sample with distilled water in the bath yielded a range of residual results with the low value equal to remolded distilled water case.

1.4.2 Remolded Kca Testing

As part of the on-going risk assessments of Syncrude's Mildred Lake Settlement Basin [MLSB] which is partly founded on Clearwater clay shales, an exploratory series of Atterberg Limit and direct shear tests were undertaken in 1987 on a remolded sample prepared by consolidation from a slurry. Three sets of samples were prepared with a Na dominated solution as reported in Table 5, that is distilled water, pore fluid and sea water. The effect on liquid limit was substantial suggesting a strength effect.

The results of the direct shear testing with distilled water and the pore fluid indicated up to 10^o or more difference, but are only tested up to 200 kPa normal stress.

Table 5 Summary of 1987 Tests on Remolded Kca

Water	Na me/l	Total me/l	w_l %	w_p %
Distilled Water (DW)	0	0	142	30
Pore Fluid	225	330	98	32
Sea Water	469	615	70	31

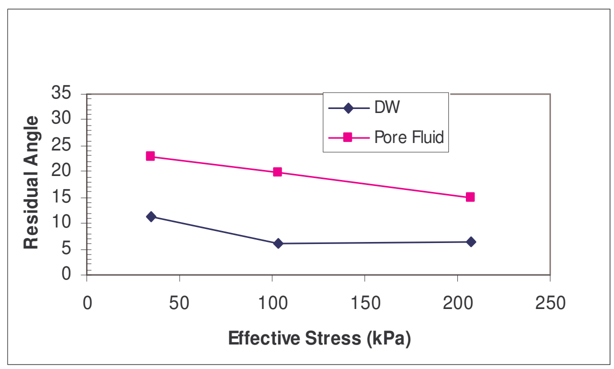

Figure 2. Residual tests on Remolded Kca

1.4.3 Intact Kc Samples

Several sets of comparisons with and without simulated pore fluids are presented. These tests have been undertaken as complementary components as part of testing programs in support of specific design objectives. One difficulty is obtaining a matched set of samples from available coreholes and reliance on a degree of consistency between clay shale facies types. Therefore some variability has to be expected as a consequence. Moreover, apart from flooding the sample with either distilled water or the simulated pore fluid no specific measures have been taken to ensure that the desired physico-chemical effect is induced into the failure plane, by for example leaching or splitting the sample after peak.

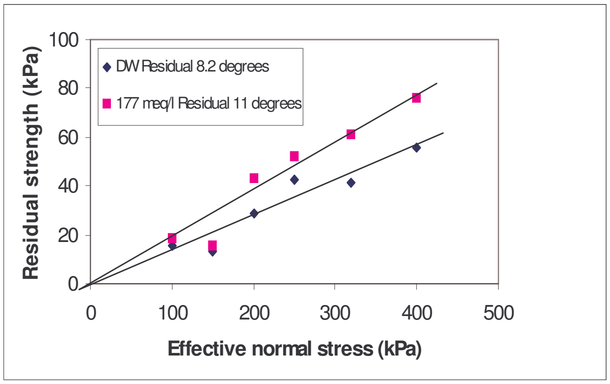

Figure 3. Kca Samples from Adjacent Coreholes

Figure 3 presents the results of a series of 6 samples of Kca taken from 2 adjacent core holes. The sample limits were wl from 111-121% and wp from 28-30%. The peak and the first three residual cycles were tested with a simulated pore fluid of Na dominated solution of Na-168 meq/l and total cation content of 177 meq/l. This pore fluid was based on paste extraction testing of the matched sample set. Then the shear box was flushed with distilled water and the samples allowed to soak for 2 to 3 days. The sample was moved back-and-forward to maximize distilled water intake and then 3 more residual strength measurements taken. The overall residual angle for the tests series is 8.2^o for distilled and 11^o for 177 meq/l.

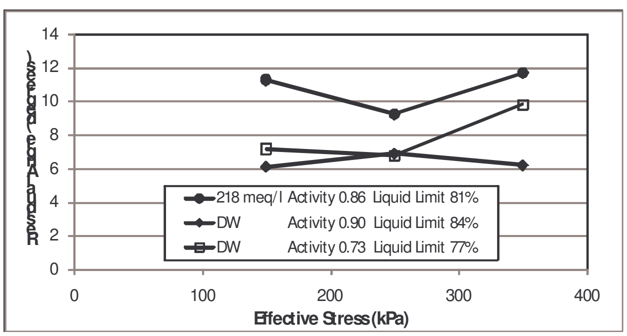

Figure 4. Kcg Residual Shear Strength

Figure 4 presents the results of 3 set of samples from the Kcg facies. The test sets were established by undertaking a set of tests with 218 meq/l . Then the total data set for other Kcg samples taken from the same vicinity was searched for similar Activity and liquid limit. All tests had been undertaken in the Syncrude laboratory using identical protocols, but with distilled water. The resulting matched set is considered as good as can be expected for natural samples of clay shales. It can be seen that even for samples potentially leached, historically an increase in direct shear strength occurs. It is also suspected that even lower distilled water residual shear strengths would have been reported had wire been used to cut the sample or the shear plane polished on glass and reassembled, methods not used at Syncrude's Geotechnical lab.

2 Case Histories at Syncrude Canada Ltd.

Three case histories that support field friction angles of 8^o or higher for the Clearwater Formation clay shale at Syncrude's Mildred Lake mine site are discussed below.

2.1 S4 Overburden Dump

Syncrude's S4 overburden dump started construction in 1980 and consists of 25 Mm^3 of fill to a design height of 41m. Construction was temporarily halted in 1985 due to large slope inclinometer movements and again in 1989 when a crack opened on the crest of the west slope. Back analysis by Segleneiks and Van Wieren of the west slope gave a Factor of safety of 0.95 using a field shear strength of 8^o and a pore pressure ratio parameter r_u of 0.6. Pore pressure data gave a range of r_u's from 0.36 to 0.84 with an average of 0.62. Using the same parameters, the north and east slope back- analyzed to a factor of safety of 1.08 and 1.09, respectively. Therefore, the back-analyzed field shear strength friction angle of 8^o appears reasonable considering the performance of both the failed slope and the non-failed slopes that showed movement.

Further review, in 2002 by Danku and Cameron, re-evaluated the measured r_u responses to more accurately model the pore pressure affects under different fill heights and shown in Figure 5. This review determined that the maximum and typical r_u's just after construction stopped in 1989 were 0.793 and 0.65, and just after placement of reclamation in 1992 were 0.734 and 0.65, respectively for the west slope. This high pore water pressure response to only 1m to 1.5m of fill loading was unexpected and may be due to the relatively small volume change required in these tight clays to show a dissipation in the pore pressure and conversely, to re-establish them. After final buttressing and reclamation placement in 1992, the factor of safety for the west slope was analysed at 1.02 and by 2002 was 1.18 based on measured dissipated pore pressures. This factor of safety would explain the initial acceleration of movement from the placement of the reclamation fills and then subsequent slowing in the S4 west slope. This movement first accelerated to a maximum of 20mm/year, and then slowed to less than 4mm/year over the next 3 years, becoming negligible after 5 years. Overall slopes for the west, north and east slope were 8H:1V, 8H:1V and 6.5H:1V with steeper intermediate slopes. The east slope, though steeper, has the best overall conditions. This is because it has the deepest depth to sliding layer, a crest to toe fill height of 35m and may not have had Kca in places due the zero thickness Kca isopach contour being close to the east slope. Total recorded movement for the west, north and east slope were 136mm, 60mm (one at 175mm), and 11mm. However some movement was missed during the short period of time between a slope inclinometer shearing off and a replacement being installed. This is especially true for the west slope where a crack had occurred. Rapid pore pressure dissipation of the *in situ*

Kca Clearwater Formation clay during construction did not occur, with typical r_u's reducing only 0.01/year. However, this slow dissipation does lead to significantly higher factors of safety over the long term. In the 2002 review direct shear testing on the Kca clay in 1991 was also re-assessed. In 1991 it was recognized that using a distilled water bath versus using pore fluid could influence the peak and residual shear strength results, and so the samples were sheared under as close to 100% humidity

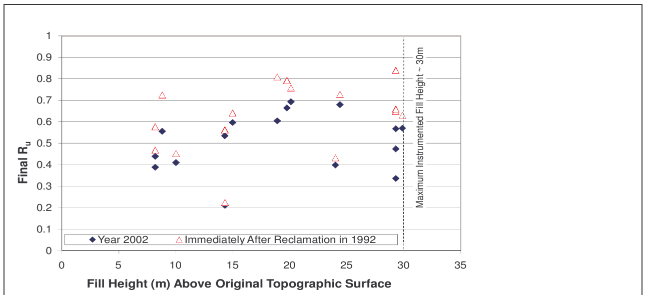

Figure 5. R_u after reclamation in 1992 and in 2002 versus Fill Height – S4 Dump

as possible. The samples were kept moist by surrounding the sample with saturated sponges and sealing it with plastic wrap during consolidation and shearing. This also kept the squeezed out consolidation pore water closest to the sample and given the initial moisture content of the lab samples and the consolidation done in the shear box, no negative pore pressures were anticipated to affect the residual strength tests. Shear box reversals were used to obtain the shear plane, with the residual strength measured only when the machine was pulling in tension, not pushing in compression. These field samples tested were heavily slickensided and interpretation of the direct shear results gave a best-fit lab residual shear strength of 7° in Figure 6, while avoiding complete flooding with distilled water.

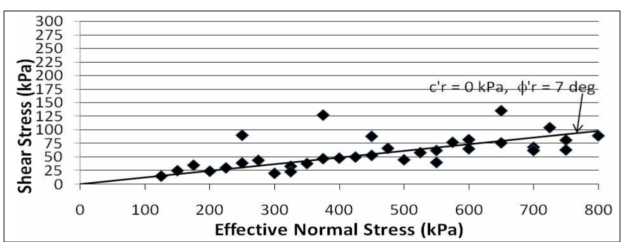

Figure 6 Residual Shear Strength on Kca – S4 Dump (Cameron 1991)

As a note, in the area of the S4 dump, the other Clearwater Formation clays above the Kca clay had been eroded off during glaciations, which would have allowed some of the pore fluid to be leached out of this Kca clay. It is important to get the correct pore fluid concentration as this will have a significant affect on shear strength, as shown previously. For slip planes close to the bottom of the Pleistocene clays (top of Kc units), as in this case of S4 Dump, the pore fluid concentration would be weaker than the for slip planes that daylighted out a mine pit wall deep within the Kc clays. Even with all the worst case factors of in situ glacial pre-shearing and the a slip plane just below the Pleistocene, the S4 Dump west slope instability still back-analyzed at a field shear strength of 8°. This represents the lab residual tested of 7° plus 1° where the lab testing as noted avoided complete flooding with distilled water, but still was not all pore fluid.

2.2 W1 Overburden Dump

Syncrude's W1 overburden dump started construction in 1998 and will contain an estimated 178 Mm^3 of overburden by completion in 2012. Almost 3 km of slopes have been built to the 40m design height with the dump founded on Clearwater Formation Kcc clays. The design of W1 Dump with a 40m fill height used a pore pressure ratio (r_u) of 0.7 for the slope, and to date this has been confirmed by the field measurements in Figure 7. A design r_u of 0.45 was used for the ground in front of the dump. Figure 7 shows a realistic maximum r_u of 0.83 but this occurs behind the potential slip plane. These piezometer results are for the top 15 to 20m of Kcc, which is the uppermost Clearwater unit below the Pleistocene in the area of the W1 Dump. It should be noted that below this Kcc the deeper clayey-portions of the Kcb and Kca had pore pressure response ratios well below an r_u of 0.7. The typical W1 dump slopes ranged from 13H:1V to14.8H:1V and had calculated factors of safety of 1.12

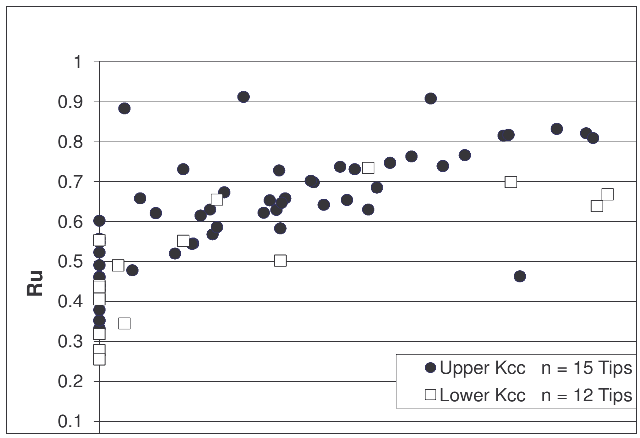

Figure 7. Historical Ru versus Fill Height – W1 Dump (Purhar and Cameron 2006)

to 1.35. Slope inclinometers have shown little to no movement of the slopes verifying that the field shear strength is close to the 8° design shear strength or higher given that the design r_u has been confirmed. Local over-steepening of one slope initially to 11.3H:1V has resulted in some movement and has analyzed at a Factor of Safety of 1.1. Successful further over-steepening of this area to 9H:1V for an intermediate height of 24m (not 40m), for final closure of a coke cell, has resulted in total movement of 89mm along the top of Kcc as measured by slope inclinometers. Analyses indicate that the field friction angle is around 8° for this local area when the r_u for the 24m height is correctly used. Above the 24m high slope, the benches are being successfully pulled back to flatter overall slopes due to the knowledge that the r_u is higher for increasing fill heights. This is leading to slower measured velocities in the slope inclinometers. Over time, the factor of safety of this area increases significantly as pore pressure dissipation occurs, as noted previously for S4 Dump.

2.3 Mildred Lake Settling Basin (MLSB)

Syncrude's Mildred Lake Settling Basin (MLSB) is the main tailings facility constructed of 1000Mm^3 of tailings sand and is founded on Clearwater Formation clays. Work in 1991 by Nicol on various aspects of this facility noted that the initial design shear strength of 12^o to 16^o were too high and the estimated pore pressure ratios (r_u) of 0.3 to 0.55 were too low for the 40m height. A summary of some of Nicol's work is discussed.

Construction of the MLSB north starter dyke started in 1977 with Cell 18, Cell 23 and Cells 4 to 17 following it. All had initial slopes ranging from 2.5H:1V to 6.8H:1V and were eventually considered too steep after movements were noted, especially those in the starter dyke which ranged from 300mm to 760mm. By 1980 as construction proceeded most of the slopes were flattened to 5.5H:1V to 9H:1V overall. Overall flatter slopes were achieved by pulling back the upper lifts while the lower part was left steeper. This type of flattening had more benefit than may have been realized at the time, as lower r_u's occur for lower fill heights, as has already been shown in Figure 7 for the W1 dump. After reaching 23m in height in 1984, movement was again noted. The applicability of limit equilibrium (LEM) analyses came under question so finite element methods (FEM) were employed. FEM tried to match models to current observed movements and then extrapolate these models to higher construction heights. The FEM movement predictions were too low and the approach was eventually abandoned as foundation movements continued to exceed predictions. Additional attempts at lab testing continued to show a large range in residual shear strength. Piezometer readings also continued to give a large range in r_u and B bar values. As a result of these difficulties, slope inclinometer movement and pore water pressure monitoring was adopted as the construction control. When the dyke showed displacements at the crest, mid slope and toe, ranging from 50 to 130mm, 190 to 420mm and 65 to 180mm respectively, and with some toe movements being close to crest movements, it was decided to flatten some slopes further to as low as 20H:1V by means of a toe berm. This successfully controlled the movements.

As noted by Nicol, the initial design used fully softened peak shear strengths of 12^o to 16^o derived from back analyses of a constructed slope and from a 17m high Kc clay spoil pile, both less than the 40m slope height required for the MLSB. Initial designs were checked against residual shear strengths of 8^o with factors of safety of 1.07 considered acceptable for these sensitivity analyses. Further back analyses of some natural slopes and some constructed slopes gave field shears strengths ranging from 7^o to 14^o. Additional lab testing on the residual shear strengths ranged from 4.5^o to 8^o. More lab testing in 1984 gave residual strengths ranging from 8^o to 14^o, but typically less than 10^o. Eventually design shear strengths were lowered to 9^o in 1980 and furthered lowered to 8^o in 1984 when test pitting revealed slickensided *in situ* Kc clay likely due to the action of glaciation. Therefore, Nicol reports initial designs started at 12^o to 16^o, well above the 8^o field shear strength considered more appropriate today. This caused the first part of the concern of whether limit equilibrium analyses could be relied on and correlated to MLSB performance.

Nicol reported that the pore pressure uncertainty was the second part of the difficulty of not being able to get parameters into the limit equilibrium analyses that <u>both</u> matched the MLSB performance and then could reliably predict future factors of safety. Pore pressure dissipation values initially were from the Pleistocene glacial Kc-rich clays and glacially rafted Kca that showed r_u's between 0.3 to 0.4. In 1980, the estimated pore pressures were considered too low and a more conservative r_u ranging from 0.5 to 0.55 was used in the design. The r_u of 0.55 worked until fill heights began to exceed 23m. Today, it is well known that r_u's are lower for fill heights less than 20m than for fill heights above 20m, for the Clearwater Formation clays, where r_u is not a soil property. For the clayiest Clearwater formations to date that occur within 15 to 20m of the Pleistocene interface, the "overall" slope r_u is more in the range of 0.7 than 0.55 when going up to 40m fill heights, or the r_u must be varied for each fill bench height.

A paper by El-Ramly, Morgenstern and Cruden (2003) on a probabilistic stability analysis of Syncrude's MLSB demonstrated the likely shear strength of the Kca clay is around 7.5^o based on a statistical distribution that included residual shear strength lab testing which also would have used distilled water flooding. A much tighter standard of deviation and/or higher values may be realized had pore fluid been used in the direct shear testing instead of distilled water. The same analyses determined that the r_u increased with increased fill height; a similar trend is noted in Figure 7 in this paper. The key to understanding the slope analyzed is that it is 23m in height and was flattened above to reach the 40m height. Had the 6.8H:1V slope been continued to 40m in height a higher r_u would have been be used, as is also noted in the their paper, and shows why this slope had to be flattened by pulling the top benches back.

2.4 Deformation Effects

Back analysis of case records using limit equilibrium methodology requires considerable judgment in understanding the relative effects of geometry, strain, displacement and construction sequence effects on mobilized strength, and the actual pattern of deformations within the case record. As major structures are strictly monitored and operated to maintain acceptable patterns of deformation, the actual mobilized strengths therefore have a degree of uncertainty. Back analysis of structures to determine mobilized strength at deformation levels potentially trending to unacceptable levels, and then applying factors of safety, is essentially a method for limiting deformations to very low levels. In the future there may be a role for undertaking coupled stability and deformation analysis as an additional aid to the design and operations of structures founded on the Clearwater.

3 Conclusions

The effect of pore fluid chemistry on direct shear testing has been shown. The difference in a few degrees is very significant for low residual shear strength soils as they represent upwards of 25% to 100% strength increases. Such increases could use up traditional factors of safety and so it is important to know that this is reliable. It is acknowledged that determination of the appropriate physico-chemical effect is difficult, but testing with distilled water can result in unrealistically low test results. The three case histories presented confirm that, when accounting for high r_u with proper fill height considerations and depth below the Pleistocene, the field shear strength of the clayiest Clearwater soil units is in alignment with strengths around 8^o and not the lower bound 3^o to 4^o that can be determined in a distilled water bath. Future direct shear lab testing on the Clearwater Formation should focus on: matching pore water chemistry, using only the tension cycle for strength determination and using only shear box reversals to create the residual shear planes. The pore water chemistry used must match the zone and location of the potential slip plane and not just be a random sample. Future designs must also use appropriate pore water pressure ratios corresponding to the height of the fill.

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5 References

Anson R.W.W. and Hawkins A.B. 1998. The Effects of Calcium Ions in Pore Water on the Residual Strength of Kaolinite and Sodium Montmorillonite, Geotechnique 48, 787-800

Balasubramonian B.I. 1972 Swelling of Compaction Shales Ph.D. Thesis University of Alberta.

Cameron R., 1991 Summary of Geotechnical Testing for the 1990 S4 Dump Project for In Situ Overburden Units and Dump Fills, Internal Report, Syncrude Canada Ltd.

Chatterji P.K. 1972. Residual Strength of Some Pure Clay Minerals, Ph.D. Thesis University of Alberta.

Chatterji P.K. and Morgenstern N.R. 1990 A Modified Shear Strength Formulation for Swelling Clay Soils. ASTM STP 1095 Physico-Chemical Aspects of Soil and Related Materials. ASTM. 118-135

Danku M. and Cameron, R. 2002. S4 Dump Final Geotechnical Performance Report, Internal Report, Syncrude Canada Ltd.

DiMaio C. and Fenelli G.B 1994. Residual Strength of Kaolin and Bentonite: the Influence of their Constituent Pore Fluids, Geotechnique 44, 217-226

Dusseault M and Cimolini P.F. 1981 The Dredge-ability of the Clearwater Formation, Geomechanical Evaluation. Report Agreement No 151 for AOSTRA / University of Alberta.

El-Ramly H., Morgenstern N.R., and Cruden D.M. 2003. Probabilistic Stability Analysis of a Tailings Dyke on Presheared Clay-shale, Canadian Geotechnical Journal Volume 40 : 192-208.

Hardy Associates 1982 Laboratory testing for GCOS, Internal Report Hardy Assoc 1978 Ltd

Isaac B. and MacKinnon M 1987. Hydraulic Transport and Disposal Impact of Clay Shale Pore Water Chemistry, Internal Research Report, Syncrude Canada Ltd.

Kenney T.C. 1967. The Influence of Mineral Composition on the Residual Strength of Natural Soils. Proceedings of the Geotechnical Conference on Shear Strength of Natural Soils Oslo, Vol 1 123-129

Martens S. and Charron R. 2007. Detailed Geotechnical Investigation of the Clearwater Clay Shale, 60th Canadian Geotechnical Conference OttawaGeo 2007, 1709-1716

Mitchell J.K. 1976. Fundamentals of Soil Behavior, John Wiley and Sons, Inc.

Moore R. 1991. The Chemical and Mineralogical Controls Upon the Residual Strength of Pure and Natural Clay Minerals, Geotechnique 42 151-153

Nicol, D. 1994. The Syncrude Mildred Lake Tailings Dyke Redesign. Transactions of the 18th International Congress on Large Dams, Durban, South Africa Vol. 3: 145-184.

Purhar, G. and Cameron, R. 2006. W1 Overburden Dump, Internal Report, Syncrude Canada Ltd.

Seglenieks, F. and Van Wieren, L 1989. The Past, Present and Future Stability of the S4 Dump, Internal Report, Syncrude Canada Ltd.

Nicol, D. 1994. The Syncrude Mildred Lake Tailings Dyke Redesign. Transactions of the 18th International Congress on Large Dams, Durban, South Africa Vol. 3: 145-184.

Purhar, G. and Cameron, R. 2006. W1 Overburden Dump, Internal Report, Syncrude Canada Ltd.

Tiwari B., Tuladhar G.R. and Marui H. 2005. Variation in Residual Strength of the Soil with the Salinity of Pore Fluid. Journal of Geotechnical and Geoenvironmental Engineering, December 2005, 1445 - 1456