CGS Cross Canada Presentation Spring Lecture Tour 2013:

Case Studies in Soil Parameter Selections for Clay Foundations

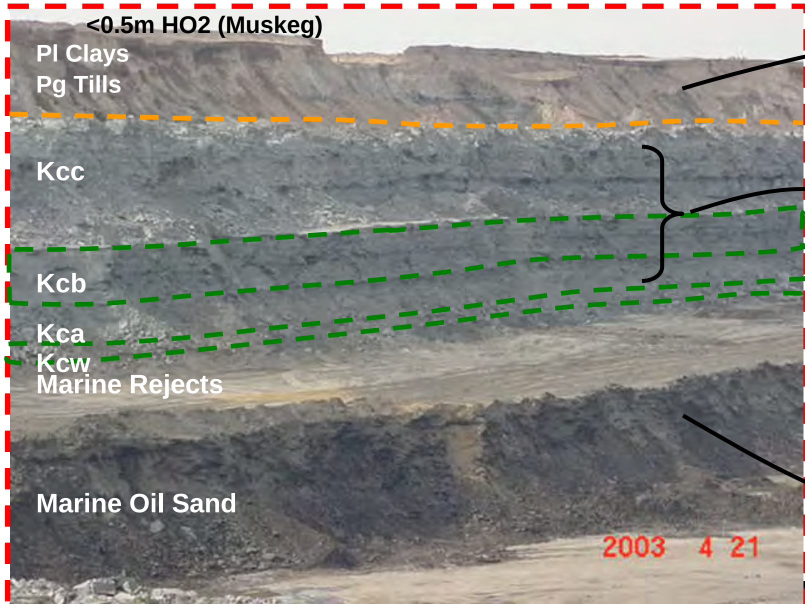
Bob Cameron, B.A.Sc., M.A.Sc., P. Eng.
Principal Geotechnical Engineer, Syncrude Canada Ltd.
Fort McMurray, Alberta

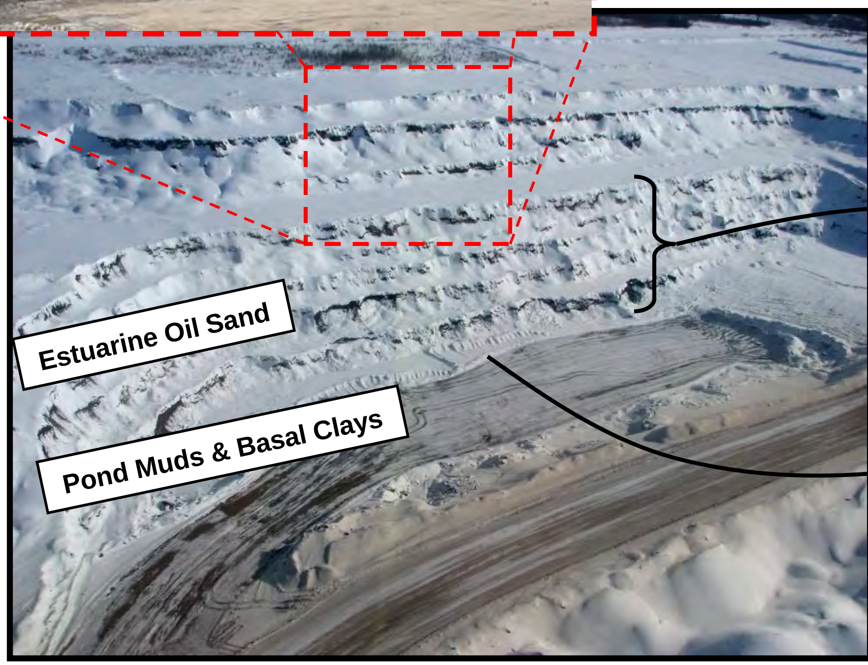
Prepared with assistance from Glen Miller, E.I.T.

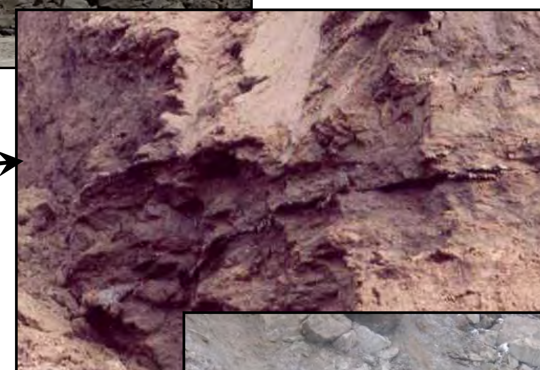
Introduction

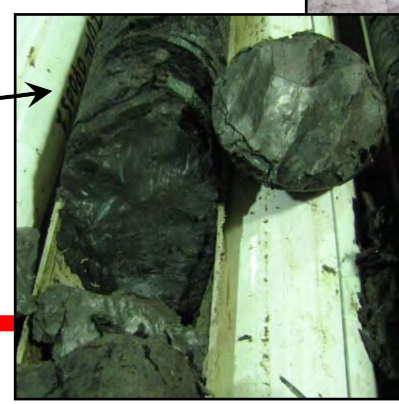
- The methodology used for choosing design shear strength and pore pressure parameters for the six (6) clay types found at Syncrude's Mildred Lake Mine site will be presented.
    1. Pl Clay - Glacially deposited lacustrine (lake) clay more kaolinite clay, at or wet of optimum, slightly overconsolidated soft (65 to 75 kPa) clay, low plasiticity
    2. Kc Clay (Kcc and Kca Clay – other Kc Clays not discussed) -- Deep Marine Smectitic Clay, heavily overconsolidated, well dry of plastic limit , high plasticity, stiff to hard (150 to 600 kPa) clay (clay shale)
    3. Marine Channel Clays -- Marine Clay near shore – kaolinite and degraded illite clay, some intercalated smectite, medium to medium-high plasticity, generally firm (75 to 150kPa) clays, usually dry of (to some close to) plastic limit
    4. Marine Near Shoreface Clays – Shallow marine clays of kaolinite and degraded illite, generally firm (75 kPa to 150 kPa) clays, dry of plastic limit to some close to plastic limit
    5. Estuarine Clays - at shore estuary clays mixed of kaolinite and illite, low plasticity, generally very stiff to hard clay (300 to 600 kPa) clays, dry of (with some layers just below) plastic limit
    6. Pond Muds/Basal Clays - Fluvial sands silt and muds – worse ones have degraded illite, medium to medium-high plastictiy, generally stiff to hard clay (150 kPa to 600 kPa) clays, well dry of (with some layer just below) plastic limit
- This methodology has been under development for the past 29 years (and longer) during open-pit oil sand mining and design and construction of almost all combinations of in-pit/out-of-pit, sand/overburden, waste dumps/tailings dams.
- Remember you need to look at your own samples in the field and in core and test your own samples for index testing comparisons and shear strengths and include historical impacts.

Syncrude Geotechnical Engineering









Introduction

- This presentation will demonstrate that:

1 – A given type of clay can have multiple pore water pressure design parameters, depending on surcharge loading and unloading;

2 – The selection of one shear strength value and one pore water pressure value for a single clay type is not always adequate;

3 – A given clay type can have high cross bedded shear strengths and a very low 'sliding' shear strengths along bedding planes and the use of 'hard layers', tested at multiple elevations is critical (as input/analyzed in slope stability programs);

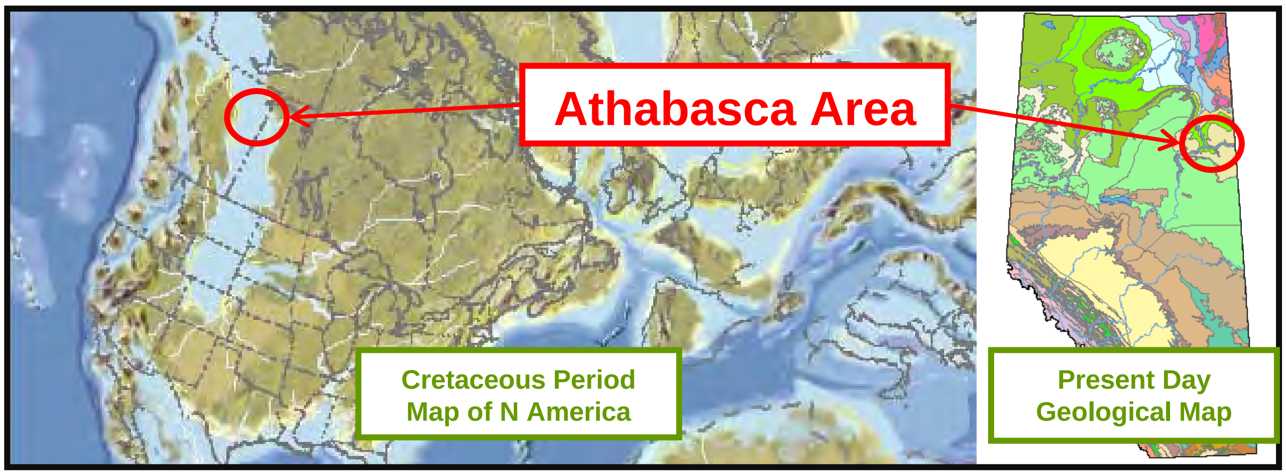
4 – Peak triaxial and peak and residual direct shear strength laboratory testing, total and effective stress considerations, and field pore pressure data can provide very useful, but often misleading or misinterpreted input parameters.

Contents

2. Site-Specific Geology – High Level Geological History

Need to know geological history to understand foundation characteristics:

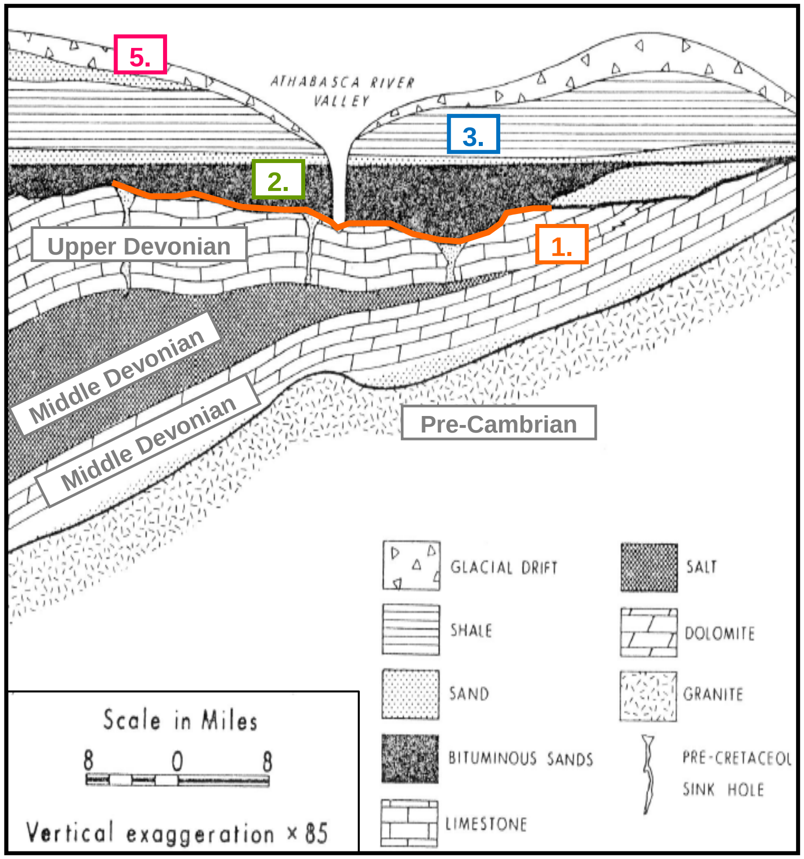
- Depositional Environments
  - Energy of deposition defines nature of sands and clays
- Erosional Events
  - Can change 'bad actor' clay location in strata when other units eroded away
- Overconsolidation Effects
  - Often increases cross-bedded strength of clays but not necessarily bedding/sliding strength
- Glaciation Effects
  - Erosion of units, some overconsolidation effects
  - Pre-shearing of uppermost units that were not eroded lowers their strength



2. Site-Specific Geology – High Level Geological History

Starting after deposition of Devonian Limestone:

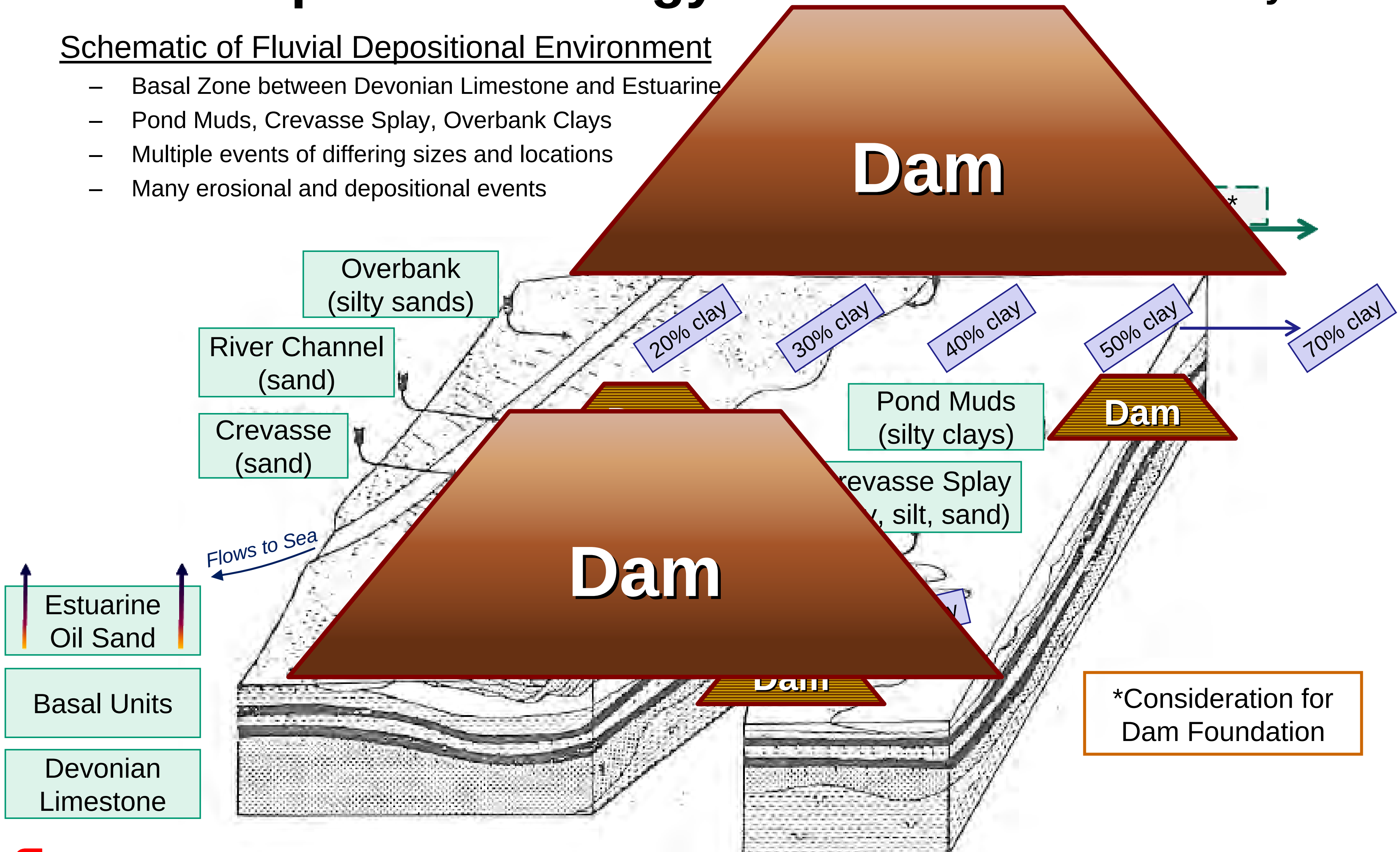
1. Large river (fluvial environment) through area (was not a predecessor to the Athabasca river)
   - Deposits Pond Muds, Overbank Clays, Crevasse Splay
2. Sea level rises, transitioning from fluvial to estuarine environment, to near shoreface shallow marine,
   - Deposits Estuarine Sand, then Marine Sand and Marine Clay Layers
3. As sea level rises further, depositional environment becomes less energetic to deep marine environment
   - Deposits Clearwater Clay units
4. Mountain building occurs to the west:
   - Leads to continental uplift which exposes Clearwater Clays
   - Forces hydrocarbons into the Estuarine and Marine Sand units = Oil Sand
5. Continental glaciation forms during Pleistocene glaciation period:
   - Erodes some of the upper Clearwater Clay units and pre-shears underlying units
   - Pleistocene Clays and tills deposited during glacial regression



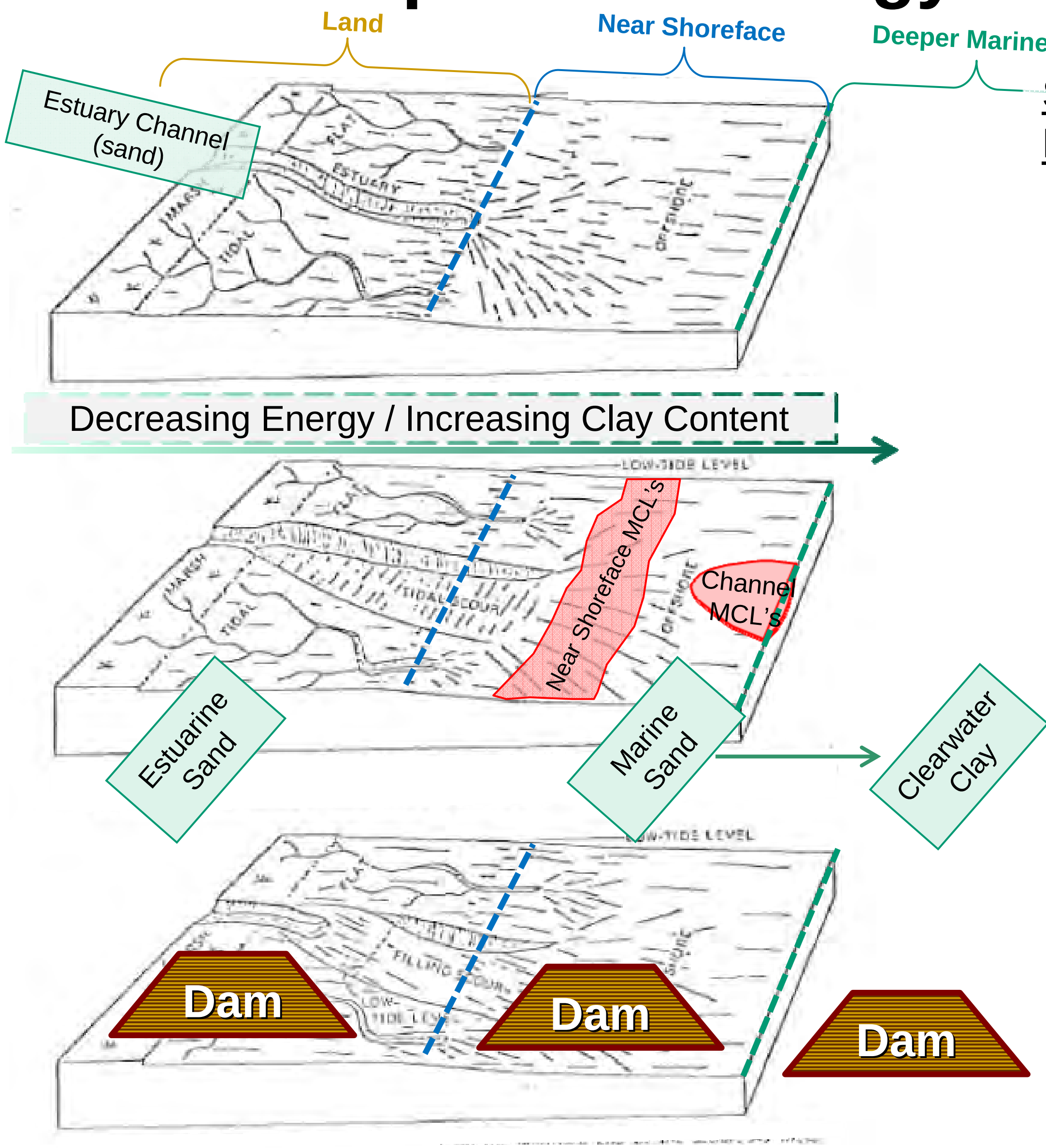
2. Site-Specific Geology – Pond Muds and Basal Clays

Schematic of Fluvial Depositional Environment

- Basal Zone between Devonian Limestone and Estuarine
- Pond Muds, Crevasse Splay, Overbank Clays
- Multiple events of differing sizes and locations
- Many erosional and depositional events



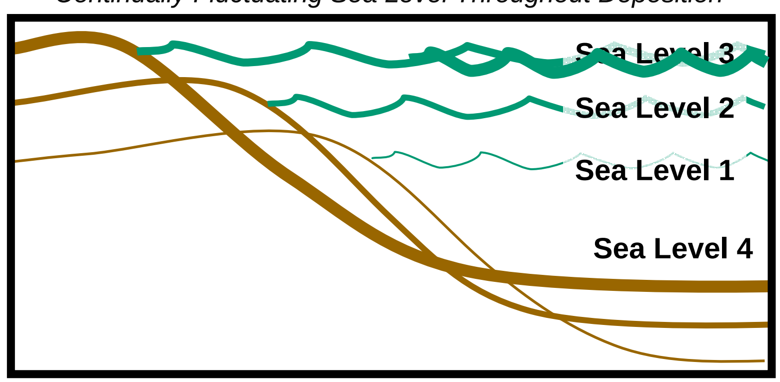
2. Site-Specific Geology - Estuarine and Marine Sand



Schematic of Estuarine and Marine Depositional Environments

- Estuarine and Marine Sand units deposited above Basal Clays
- Fining-upward sequence as sea levels rose and depositional energy decreased
- Sand with many truncating silt lenses, some semi-continuous clay layers in upper layers
- Marine Clay Layers (MCL's) deposited in lower energy Near Shoreface Marine and Marine Channel environments
  - Problematic thin clay layers which have caused a number of pitwall failures
- Multiple events of differing sizes and locations
- Many erosional and depositional events during rising sea levels

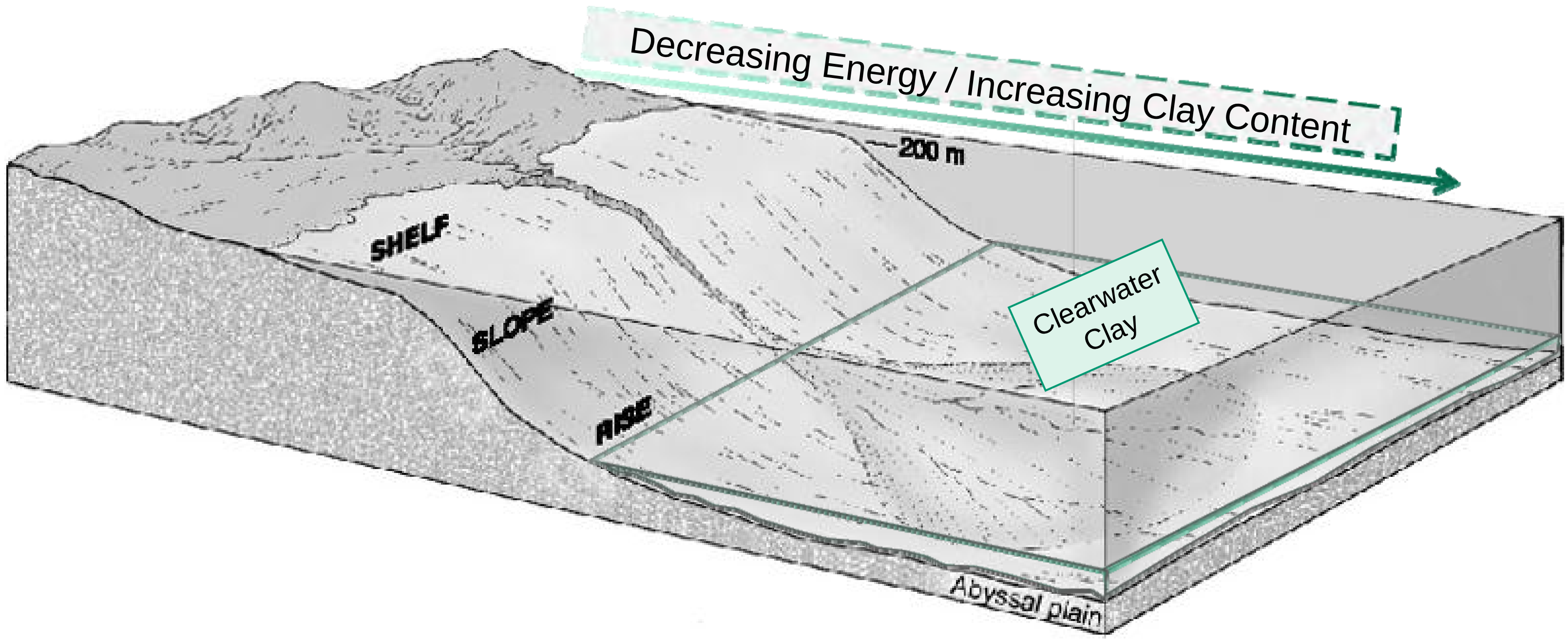
Continually Fluctuating Sea Level Throughout Deposition



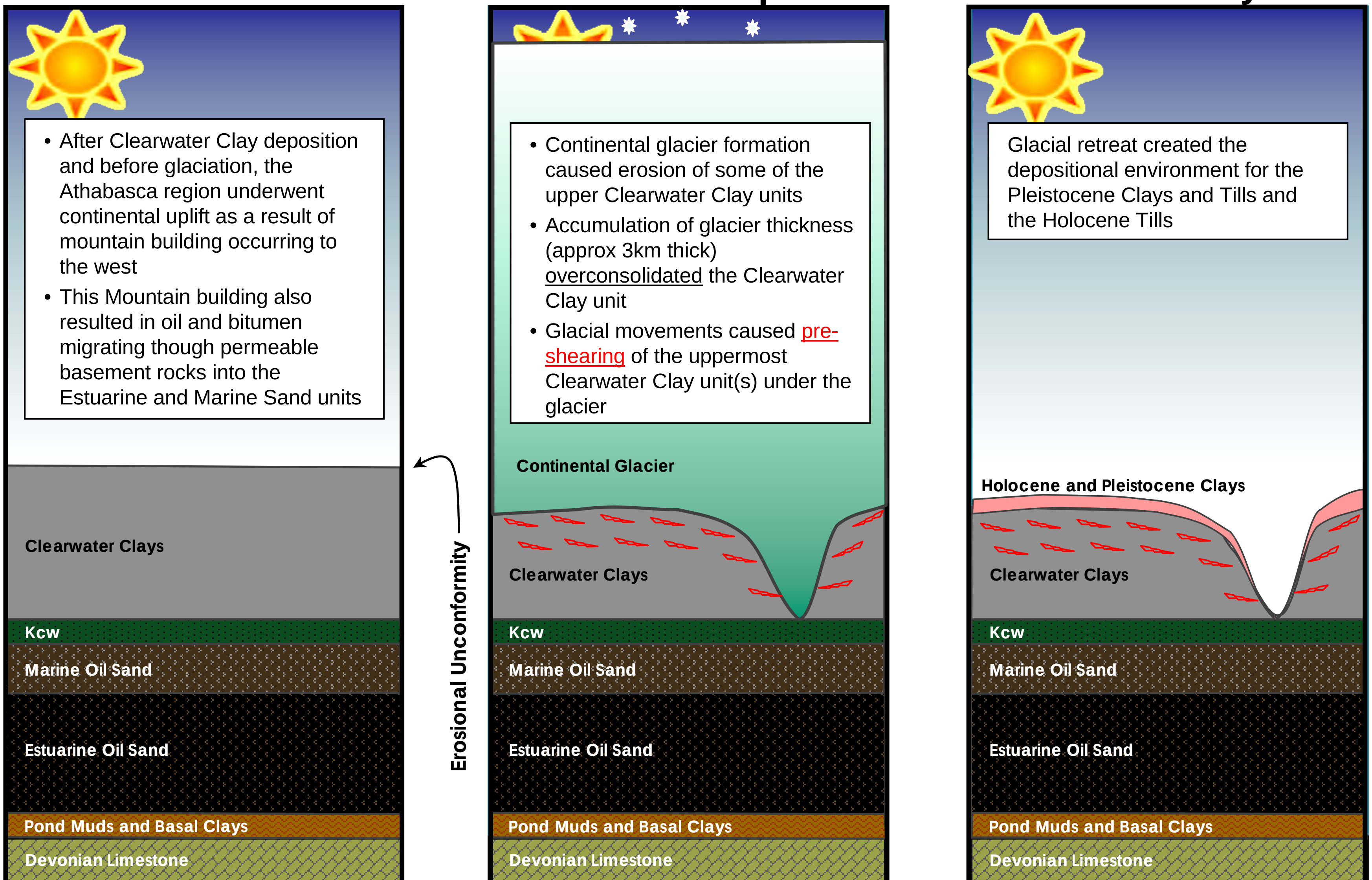
2. Site-Specific Geology – Clearwater Clays

Schematic of Deep Marine Depositional Environments

- Deep marine Clearwater Clays are deposited above Marine and Estuarine Sand units deposited
- Several well-defined massive clay units that require separate modelling
- Some variability of composition in uppermost units due to varying energy as sea levels began to lower during continental uplift (final deposition coarsens upwards
- After uplift, some overconsolidation occurs from draining

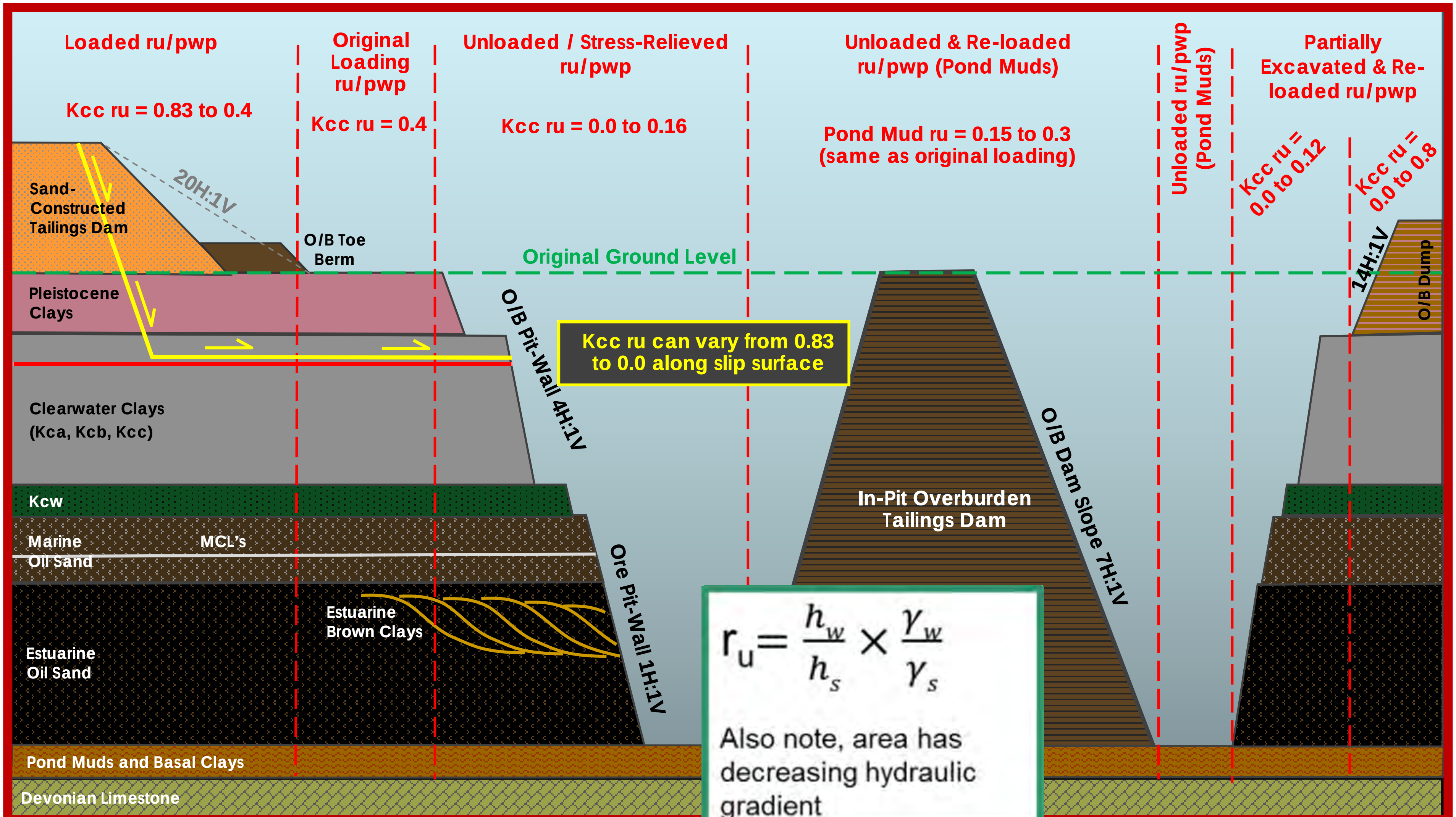


2. Site-Specific Geology – Glacial Effects on Clearwater Clay and Deposition of Pleistocene Clay



3. Site-Specific Excavation/ Back-Fill Geometries and Loading –

Variance of ru value with loading, unloading, and re-loading



$$r_u = \frac{h_w}{h_s} \times \frac{\gamma_w}{\gamma_s}$$

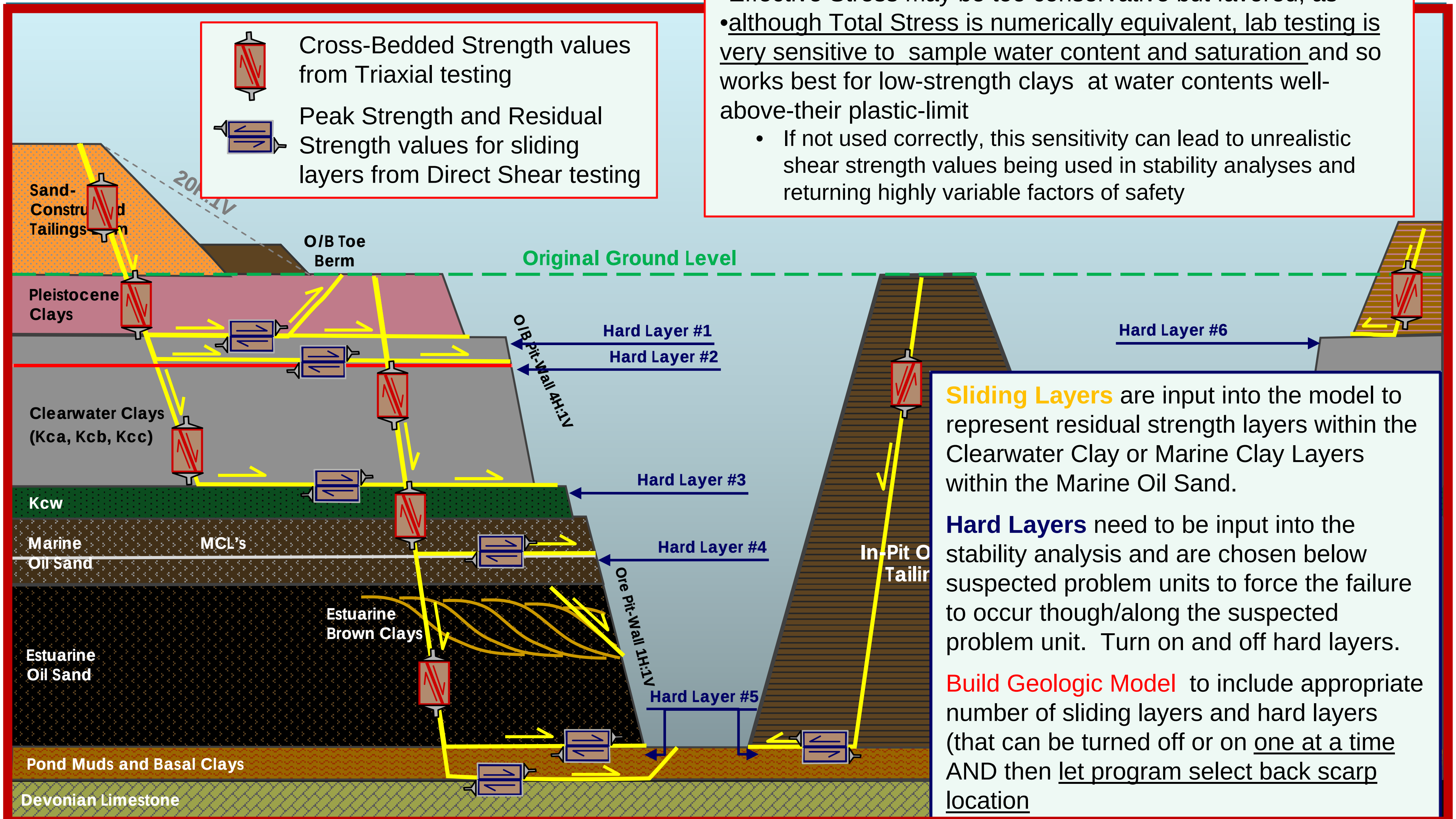
Also note, area has decreasing hydraulic gradient

3. Site-Specific Excavation/ Back-Fill Geometries and Loading –

Shear Strength and Slip Surfaces

Peak strengths from lab testing may be too high or too low:
•Effective Stress may be too conservative but favored, as
•<u>although Total Stress is numerically equivalent, lab testing is very sensitive to sample water content and saturation</u> and so works best for low-strength clays at water contents well-above-their plastic-limit

- If not used correctly, this sensitivity can lead to unrealistic shear strength values being used in stability analyses and returning highly variable factors of safety



Sliding Layers are input into the model to represent residual strength layers within the Clearwater Clay or Marine Clay Layers within the Marine Oil Sand.

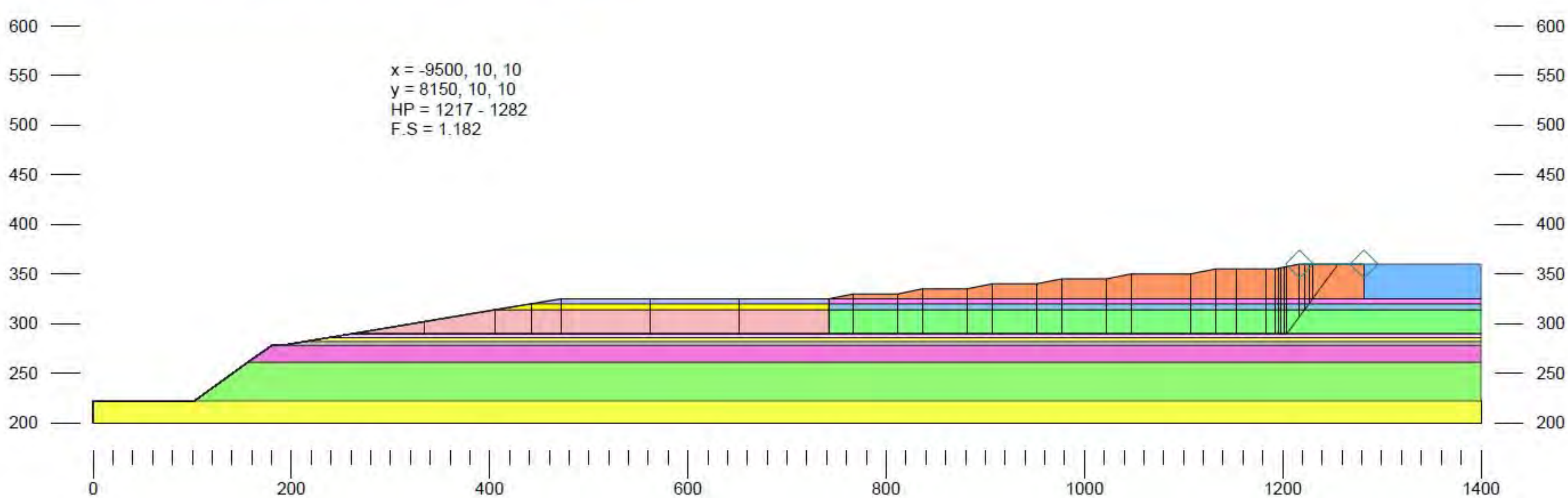
Hard Layers need to be input into the stability analysis and are chosen below suspected problem units to force the failure to occur though/along the suspected problem unit. Turn on and off hard layers.

Build Geologic Model to include appropriate number of sliding layers and hard layers (that can be turned off or on <u>one at a time</u> AND then <u>let program select back scarp location</u>

Example of adding sliding layers, multiple ru's & changing hard-layer locations

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.	Ru
Non Slope Area	18	0	20	0	.3
Core (Dump)	18	25	20	0	.2
Pug (not load)	20.2	0	25	0	.45
Pug (load)	20.2	0	25	0	.6
Kcd (not load)	19.6	25	20	0	.45
Kcd (load)	19.6	25	20	0	.55
Kcc (not load)	19.6	25	20	0	.45
Kcc (load)	19.6	25	20	0	.7
Kcc(Slid noload)	19.6	0	8	0	.45
Kcc(Slid load)	19.6	0	8	0	.7
Kcb	(Infinitely Strong)				
Kca	19.6	25	20	0	.38
Kcw	20.2	20	33	0	0
Marine	21.2	30	54	0	0
Estuarine	21.2	30	54	0	.19
Devonian	23	0	25	0	.19

Syncrude Canada Ltd. - Fort McMurray, AB
W4 Facility- Section N1-N1' West
6H:1V Overburden Slope from Top of Ore
Stability of W4 Sliding Along Bottom of Kcc
From 360m Bench
Mode 5Cb



4. Case Studies

a. 'S4 Dump' – Out-of-Pit Overburden Dump, 1989 to 1992 mvmt's

b. 'W1 Dump' – Out-of-Pit Overburden Dump, 2005 to 2009 mvmt's

c. 'MLSB' – Out-of-Pit Sand-Constructed Tailings Dam, 1980 to 1992 mvmt's

d. 'SWSS' – Out-of-Pit Sand-Constructed Tailings Dam, 2009 to 2012 piezo response

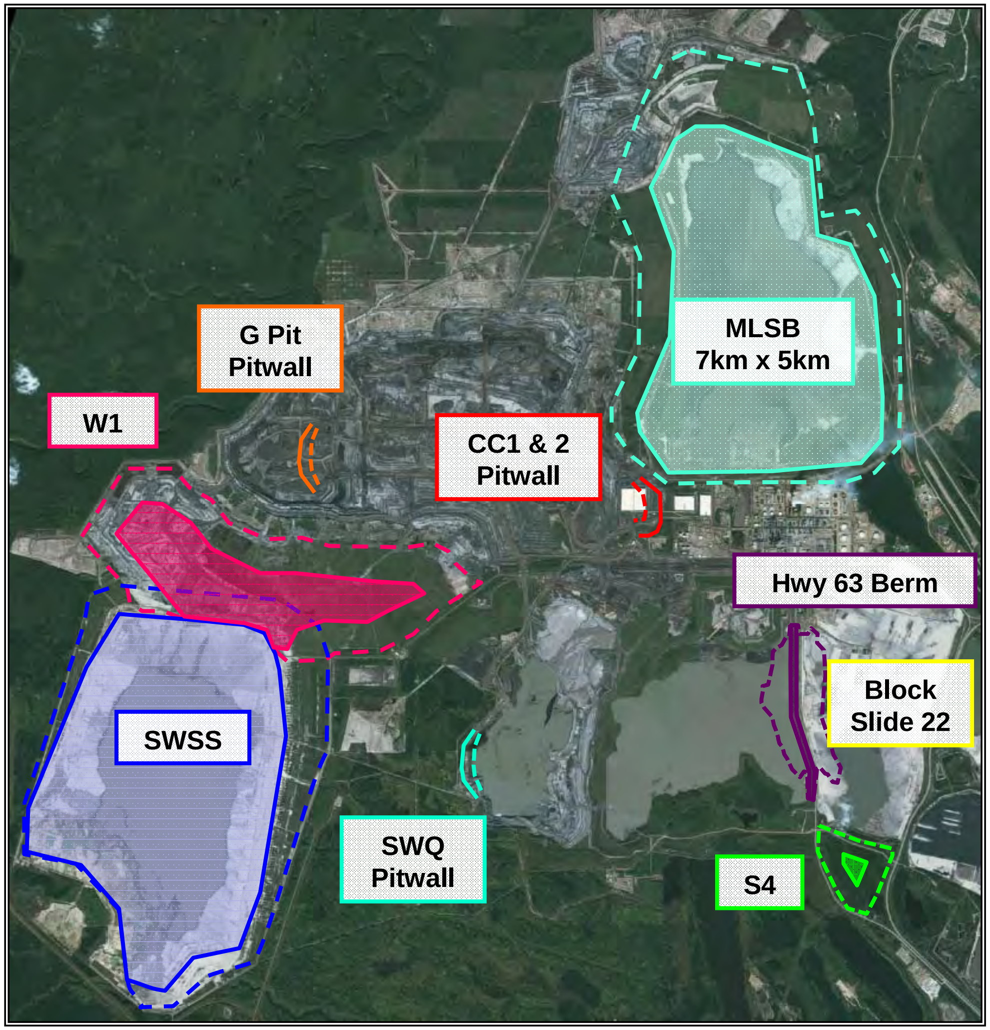
e. 'Hwy 63 Berm' – In-Pit Overburden Tailings Dam, 1992 to 1994 design

f. 'SWQ West Wall' – Steep Final Pitwall with No Movement in 2006,

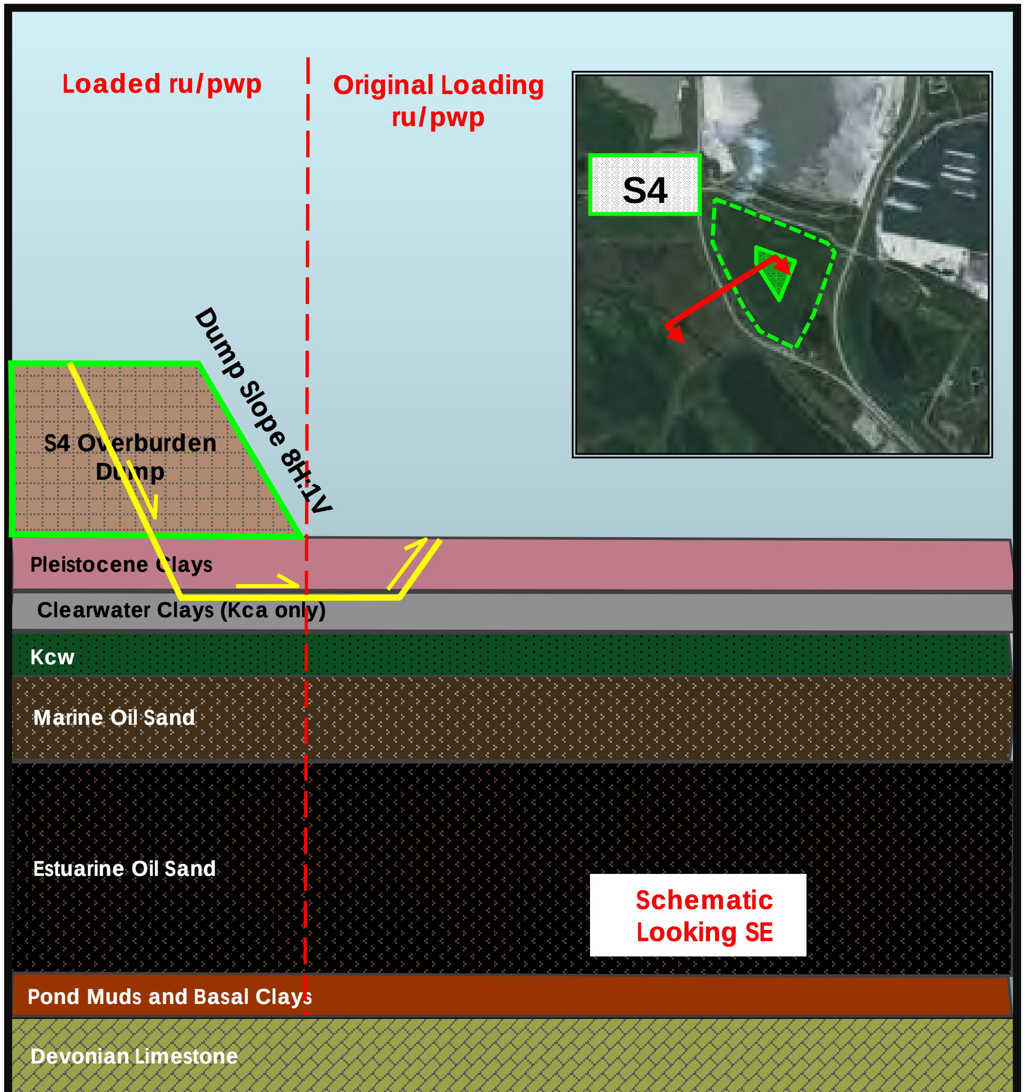
g. 'Block Slide #22' – Advancing Ore Pitwall with Movement in 1987

h. 'CC1 and CC2' – Final Pitwall with Movement in 2003

i. 'G-Pit' – Advancing O/B Pitwall with Movement/Slump in 2006



4a. S4 Dump-Out-of-Pit Overburden Dump, 1989 to 1992 mvmt's



Dump Characteristics:

•25Mm3 of overburden fill

•38m high above original topography with avg. 8H:1V side slopes (flattened from original design of 3 & 4H:1V)

Foundation Characteristics:

•Pleistocene Clays present but not a concern

•Most of the original 30m of Clearwater eroded away by glaciation

•Remaining 1 to 5m of Kca Clay unit was glacially-affected to residual strength

•Residual strength tested in tension in direct shear machines

Movement Characteristics:

•Internal deformation with some cracking observed at crest and 2m of vertical displacement on west side

•The two main movement sides are north and west

•Many sheared SI's though dump did not completely fail

•Slides along top of first weakest layer in the Clearwater clay unit

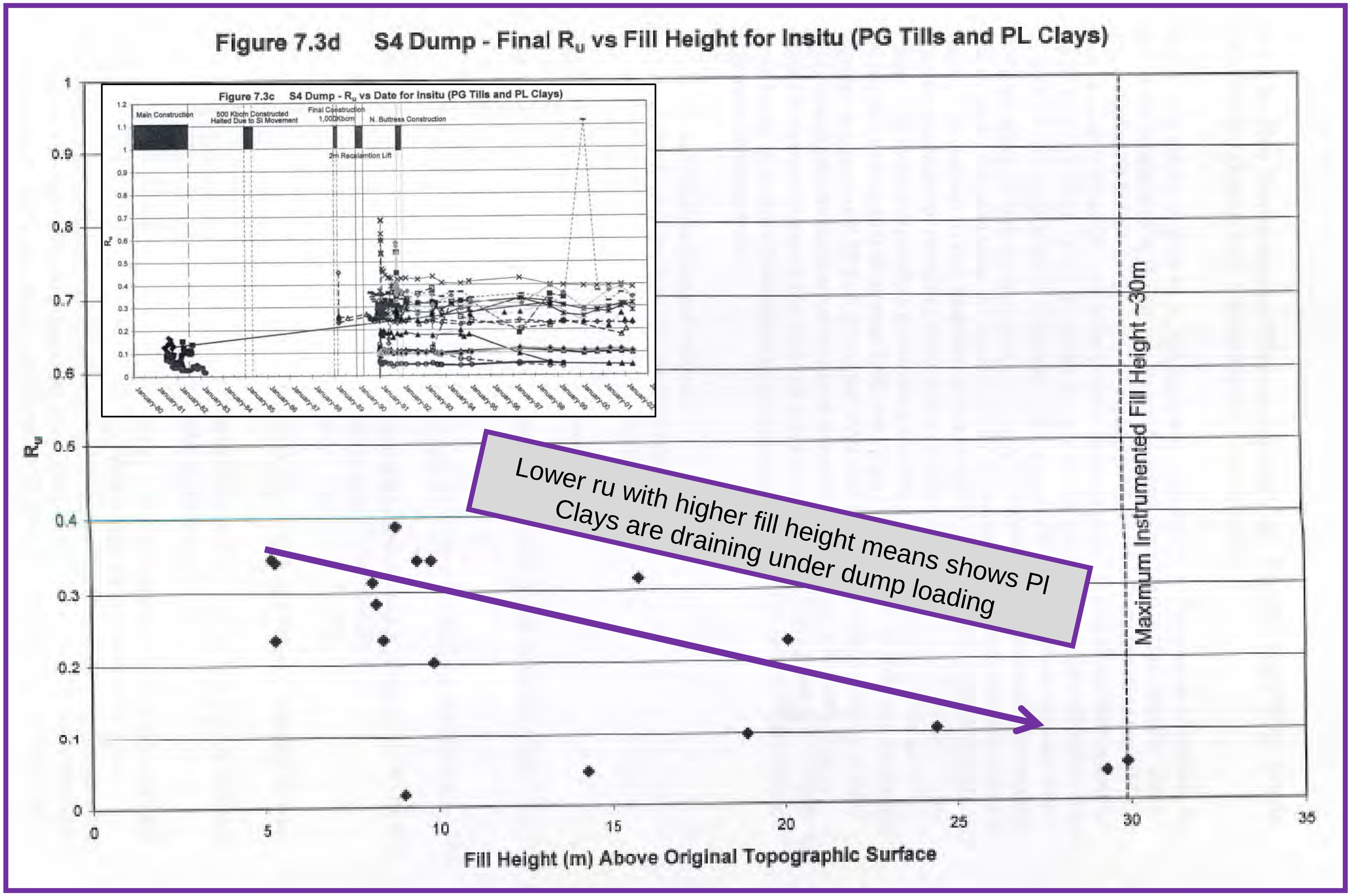
•Constructed a shear key/toe berm on north toe to allow pitwall to be mined closer and used observational approach to continue dump construction

•ru jumped up (and SI's moved) both when 1Mm3 additional fills added to crest of dump (5m thick) and when 1.5 to 2m of (reclamation) material was added close to area of previously high ru

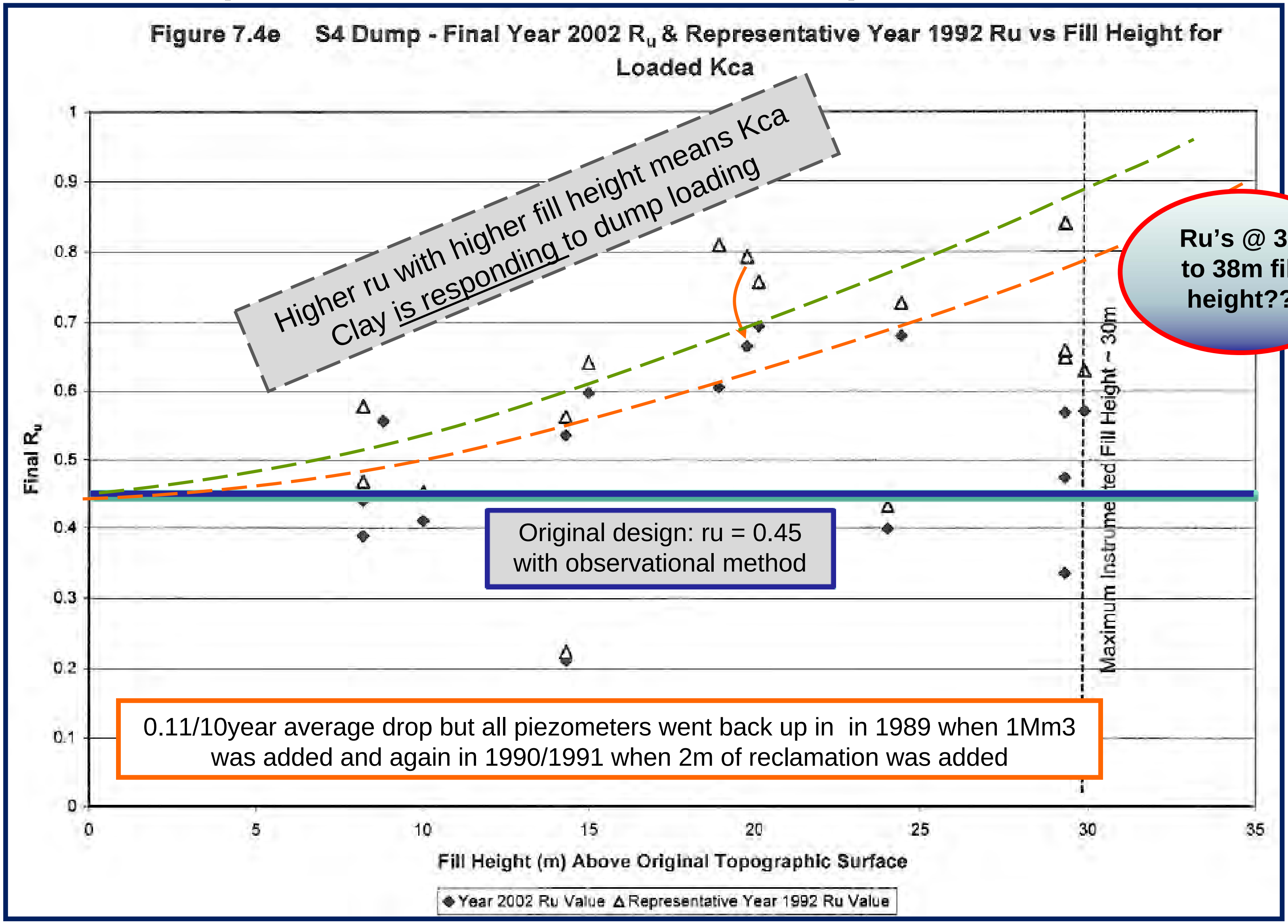
- SI's moved 50mm/year (from O/B fills) then slowed and stopped
- SI's moved again at 10mm/year (from reclamation fills) which shows this area has a low factor of safety and Kca is near plastic strain curve

Hard to get FS=1.3 or 1.5 without having dump slopes at 20H:1V to 35H:1V (F.S. in the order of 1.07 to long-term 1.18 for 8H:1V)

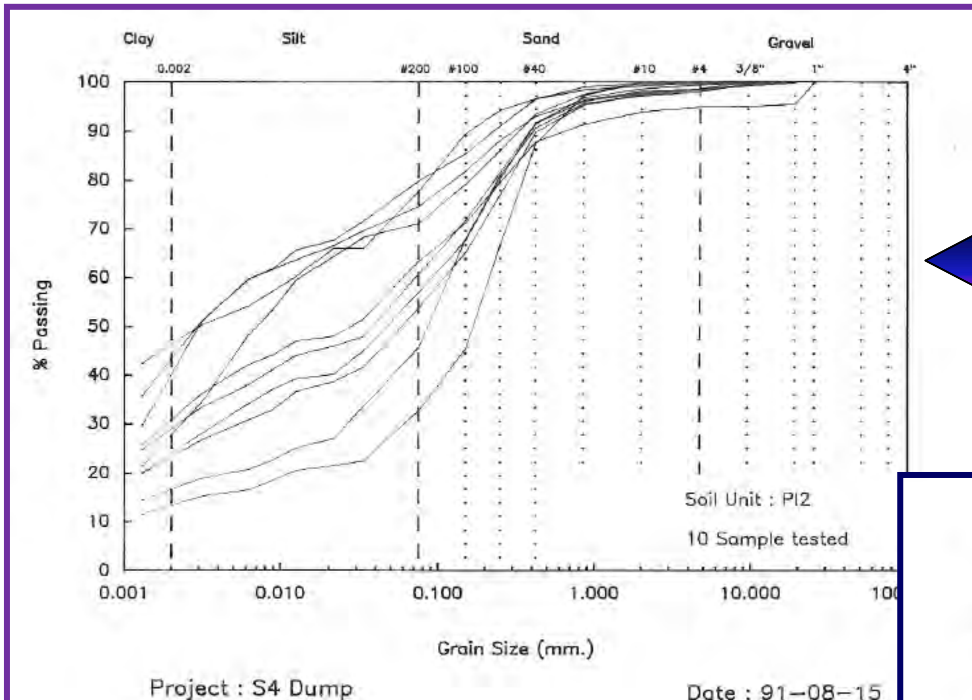
4a. S4 Dump – Out-of-Pit Overburden Dump, 1989 to 1992 mvmt's, Continued



4a. S4 Dump – Out-of-Pit Overburden Dump, 1989 to 1992 mvmts Continued



4a. S4 Dump – Out-of-Pit Overburden Dump, , 1989 to 1992 mvmt's Continued

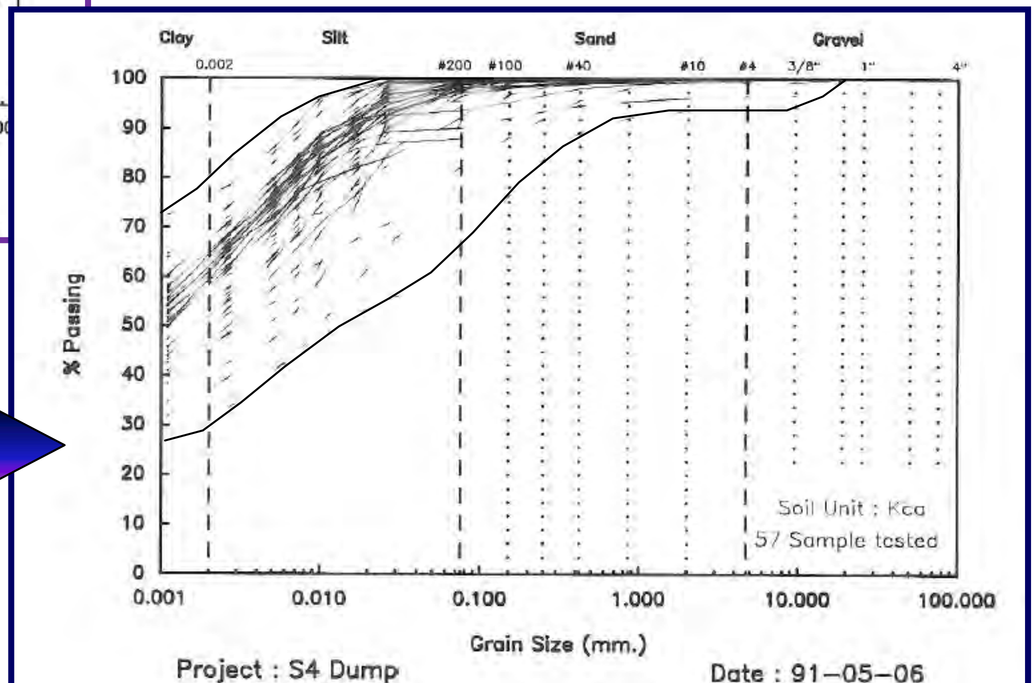


Pleistocene (Pl) Clay

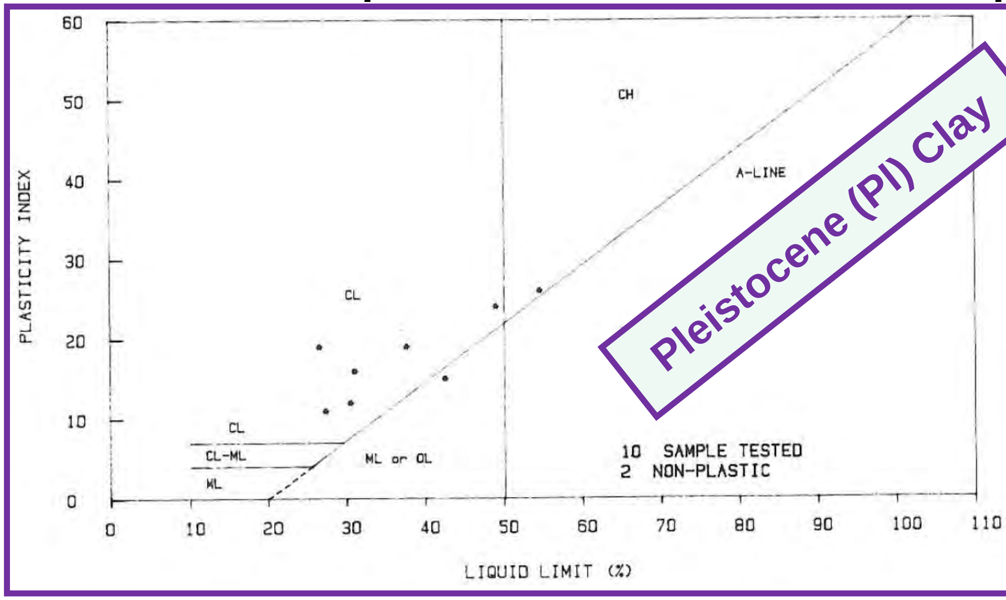
- Low to medium plastic clay,
- LL=38.5%, PL=20.3%, 16.6%
- Slightly over consolidated
- Optimum moisture content or wet-of-optimum moisture content
- Cross-bedded and Sliding Design: C'=0, Φ'=20°
- Measured r_u=0.05 to 0.4

Clearwater (Kca) Clay

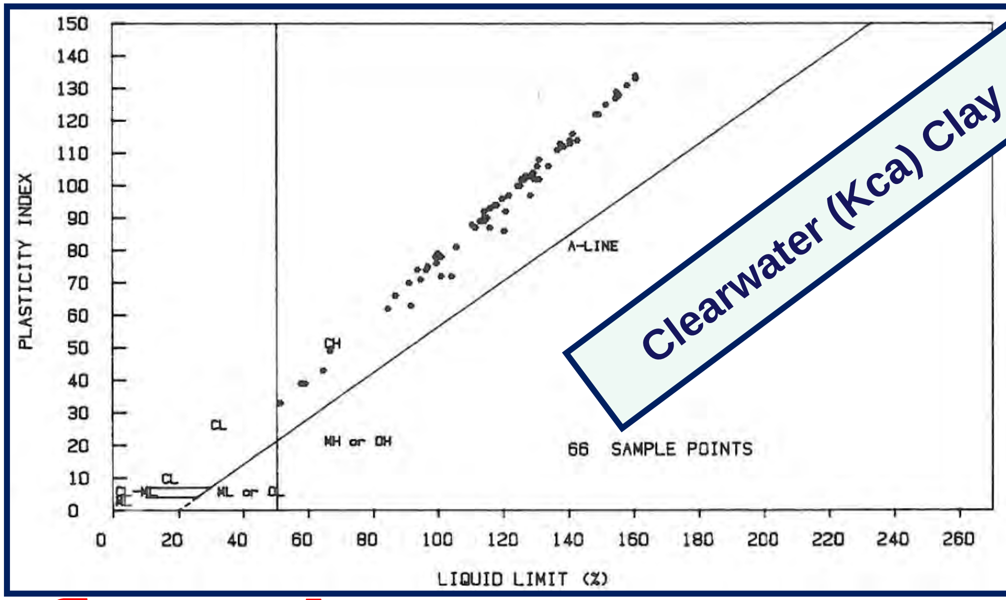
- High Plastic Clay,
- LL=117%, PL=24%, 26.2%
- Heavily eroded - missing units above Kca
- Greatly overconsolidated
- Heavily pre-sheared
- Sliding Design: C'=0, Φ'=8°
- Measured r_u=0.4 to 0.83



4a. S4 Dump – Out-of-Pit Overburden Dump, , 1989 to 1992 mvmt's



	TEST	n	Range	Avg	Median	Std Dev
	NATURAL MOISTURE CONTENT (%)	8	7.6 -28.2	16.6	16.2	6.5
ATTERBERG LIMITS	LIQUID LIMIT (%)	8	27.2 -54.5	38.5	37.5	9.4
	PLASTIC LIMIT (%)		14.4 -28	20.3	18.4	4.9
	PLASTICITY INDEX (%)		11 -26	17.8	19	5
	PLASTICITY		LO-M.HI	LOW	LOW	
PARTICLE SIZE	% GRAVEL	10	0 -5.1	1.3	1.4	1.4
	% SAND		12.6 -67.4	36.6	37.8	15.3
	% SILT		19.1 -35.8	29.8	31.8	5.6
	% CLAY		13.2 -52.2	32.2	31.4	12.7
	ACTIVITY		.43 -.82	.55	.51	.12

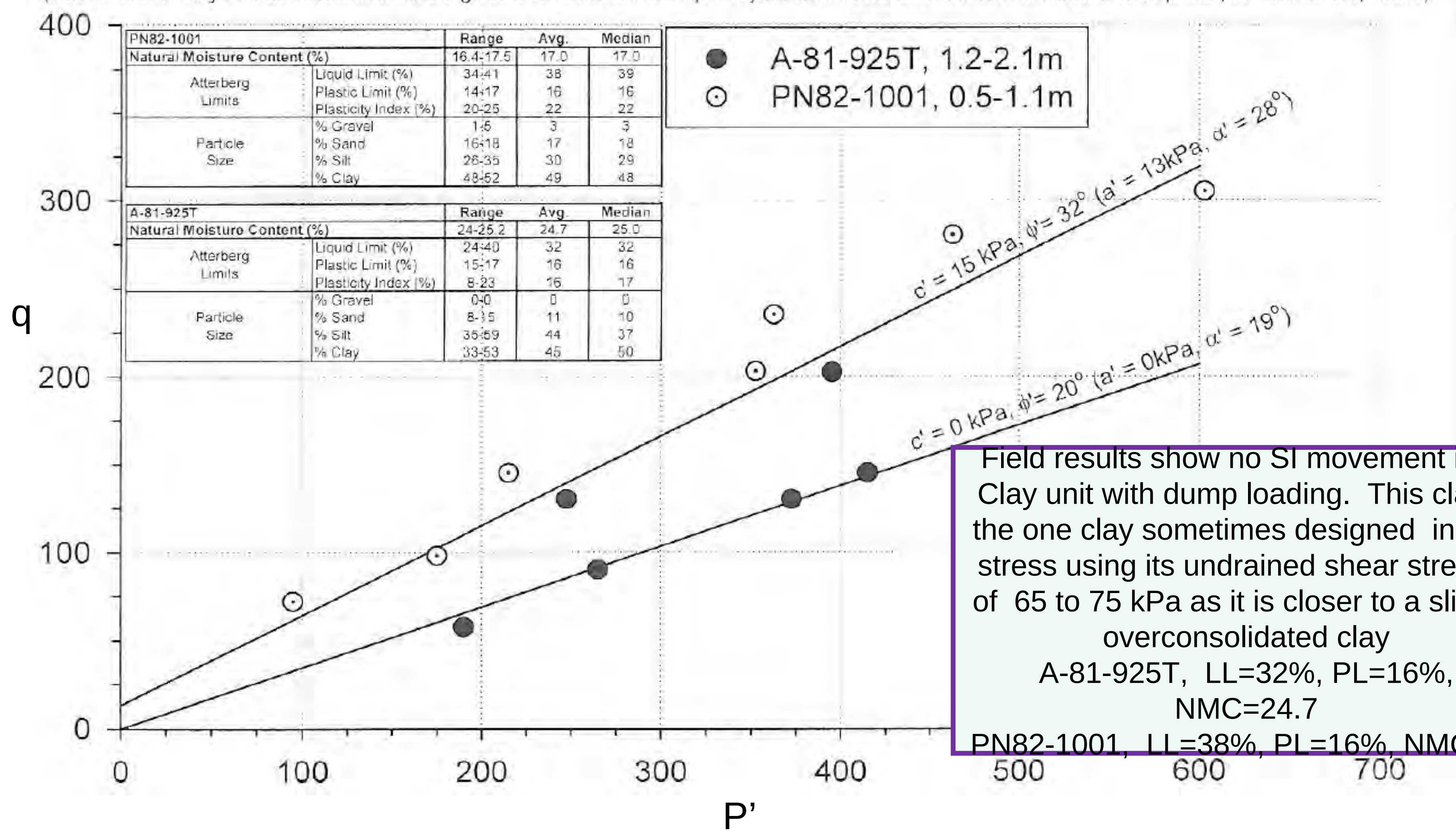


	TEST	n	Range	Avg	Median	Std Dev
	NATURAL MOISTURE CONTENT (%)	85	12.5 -43.9	26.2	26.2	6.4
ATTERBERG LIMITS	LIQUID LIMIT (%)	66	50.9 -160.4	117.2	119.4	25.4
	PLASTIC LIMIT (%)		16.4 -33.7	24	23.8	3.2
	PLASTICITY INDEX (%)		33 -134	92.7	94	23.5
	PLASTICITY		M.HI-HI	HI	HI	
PARTICLE SIZE	% GRAVEL	52	0 -6.2	.2	0	.9
	% SAND		.1 -45.2	3.2	.7	7
	% SILT		24.7 -61.7	39.8	38.8	8.1
	% CLAY		15.9 -74.6	56.7	59.2	10.3
	ACTIVITY		1 -2.13	1.6	1.66	.24

4a. S4 Dump – Out-of-Pit Overburden Dump, , 1989 to 1992 mvmt's Continued

C.2b In Situ PI_2 Clay CUP Triaxial q-p' Plot

(From: Summary of Geotechnical Testing for the 1990 S4 Dump Project for in situ overburden units and dump fills, B. Cameron, 1991)



PN82-1001		Range	Avg.	Median
Natural Moisture Content (%)		16.4-17.5	17.0	17.0
Atterberg Limits	Liquid Limit (%)	34-41	38	39
	Plastic Limit (%)	14-17	16	16
	Plasticity Index (%)	20-25	22	22
Particle Size	% Gravel	1-5	3	3
	% Sand	16-18	17	18
	% Silt	26-35	30	29
	% Clay	48-52	49	48

A-81-925T		Range	Avg.	Median
Natural Moisture Content (%)		24-25.2	24.7	25.0
Atterberg Limits	Liquid Limit (%)	24-40	32	32
	Plastic Limit (%)	15-17	16	16
	Plasticity Index (%)	8-23	16	17
Particle Size	% Gravel	0-0	0	0
	% Sand	8-15	11	10
	% Silt	36-59	44	37
	% Clay	33-53	45	50

Field results show no SI movement in PI Clay unit with dump loading. This clay is the one clay sometimes designed in total stress using its undrained shear strength of 65 to 75 kPa as it is closer to a slightly overconsolidated clay

A-81-925T, LL=32%, PL=16%, NMC=24.7

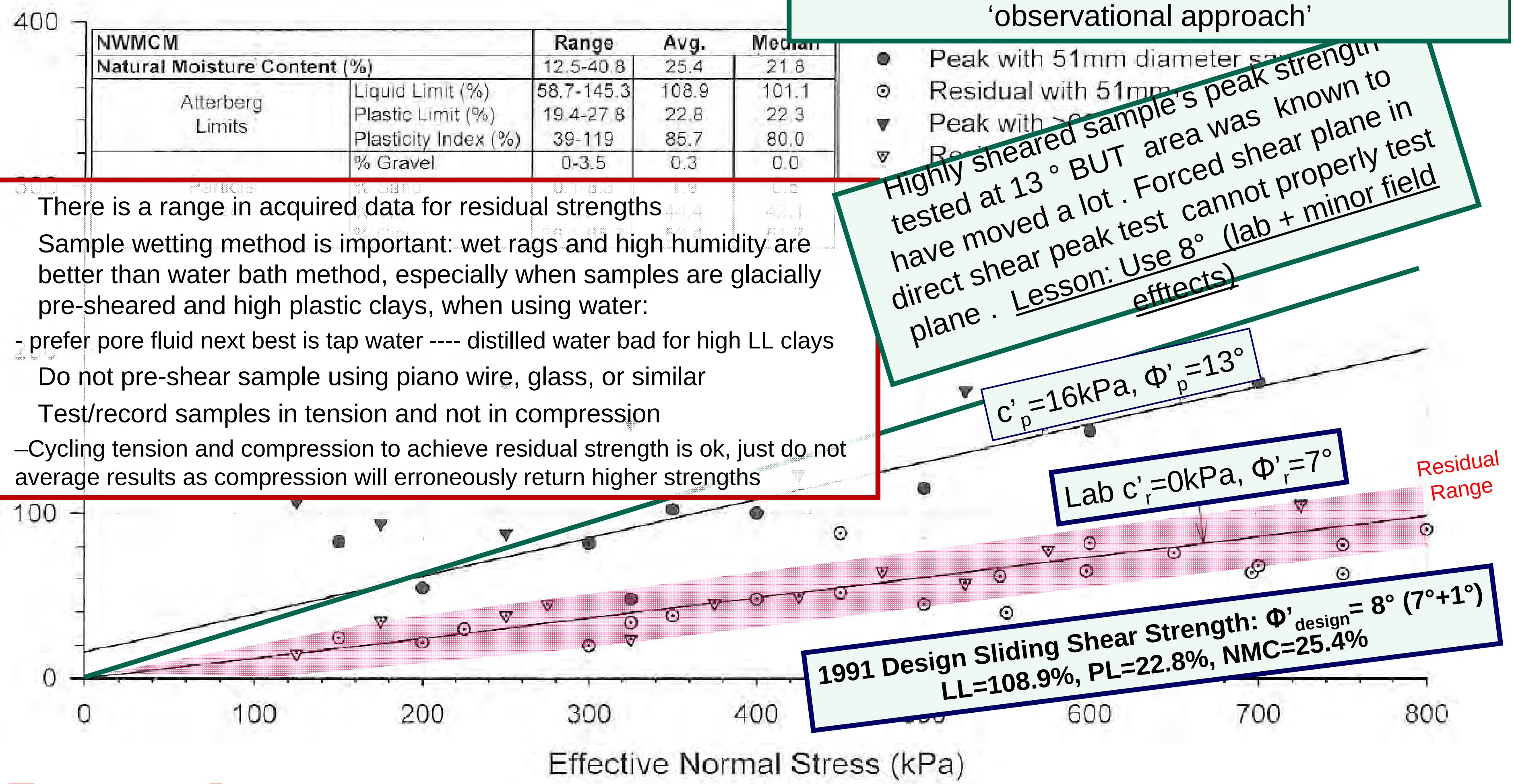
PN82-1001, LL=38%, PL=16%, NMC=17

4a. S4 Dump – Out-of-Pit Overburden Dump, , 1989 to 1992 mvmt's Continued

C.2d.6 Insitu Kca Direct Shear Testing
(From: Cameron, April 1992)

1980 design was C'_r=0kPa, Φ'_r=16° with use of 'observational approach'

NWMCM		Range	Avg.	Median
Natural Moisture Content (%)		12.5-40.8	25.4	21.8
Atterberg Limits	Liquid Limit (%)	58.7-145.3	108.9	101.1
	Plastic Limit (%)	19.4-27.8	22.8	22.3
	Plasticity Index (%)	39-119	85.7	80.0
	% Gravel	0-3.5	0.3	0.0



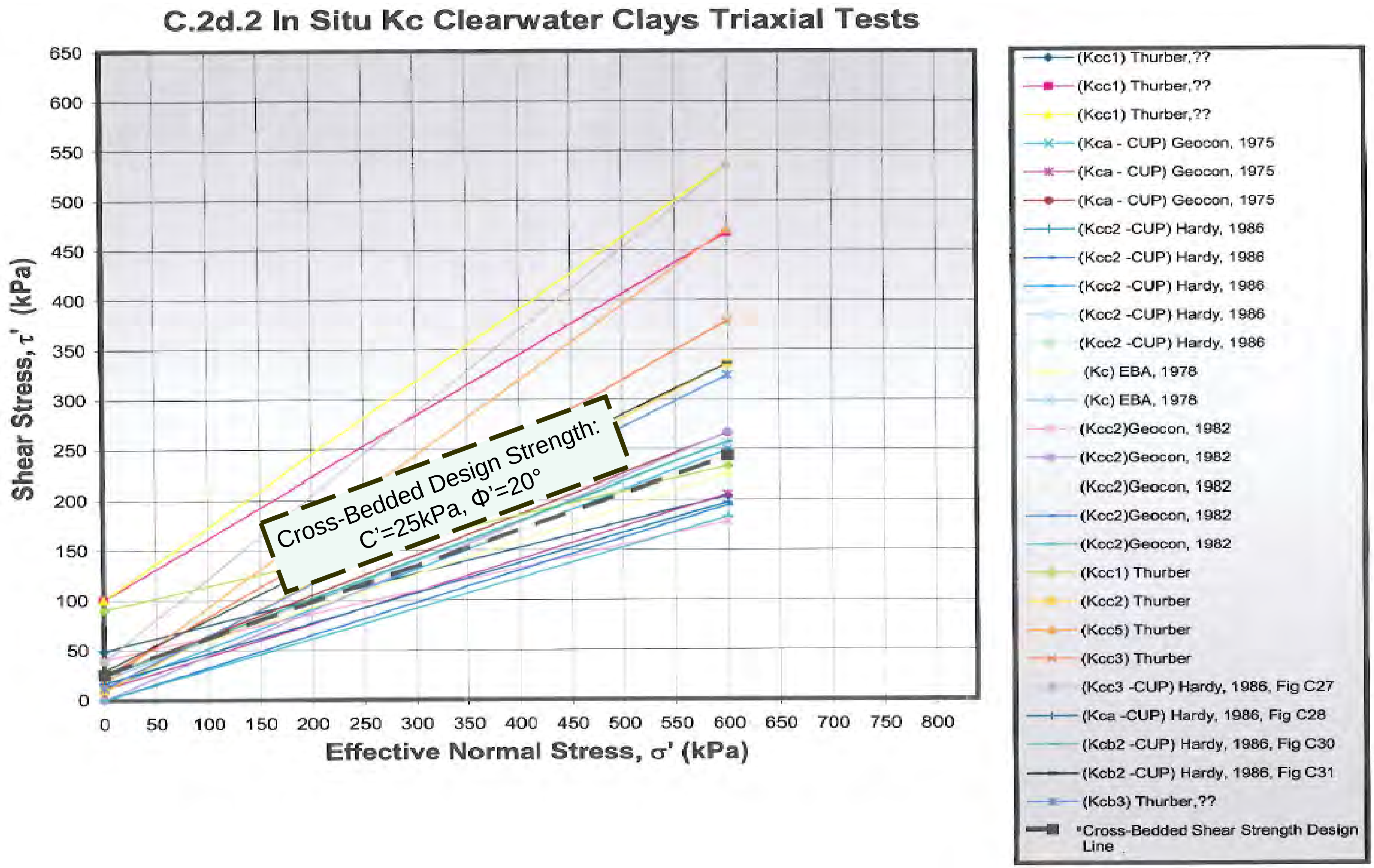
- There is a range in acquired data for residual strengths
- Sample wetting method is important: wet rags and high humidity are better than water bath method, especially when samples are glacially pre-sheared and high plastic clays, when using water:

- prefer pore fluid next best is tap water ---- distilled water bad for high LL clays

- Do not pre-shear sample using piano wire, glass, or similar
- Test/record samples in tension and not in compression

–Cycling tension and compression to achieve residual strength is ok, just do not average results as compression will erroneously return higher strengths

4a. S4 Dump – Out-of-Pit Overburden Dump, , 1989 to 1992 mvmt's Continued



4a. S4 Dump – Out-of-Pit Overburden Dump, , 1989 to 1992 mvmt's Continued

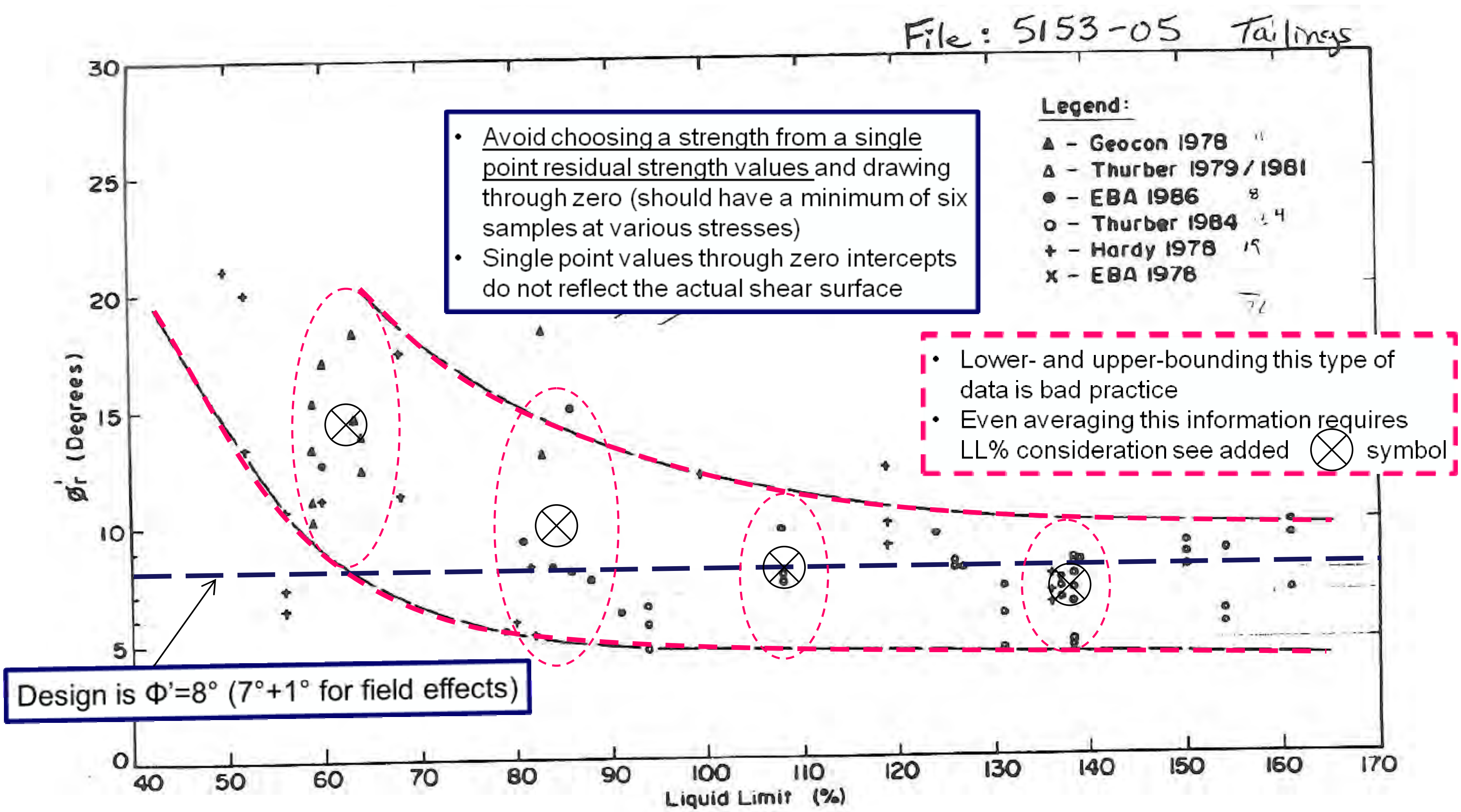
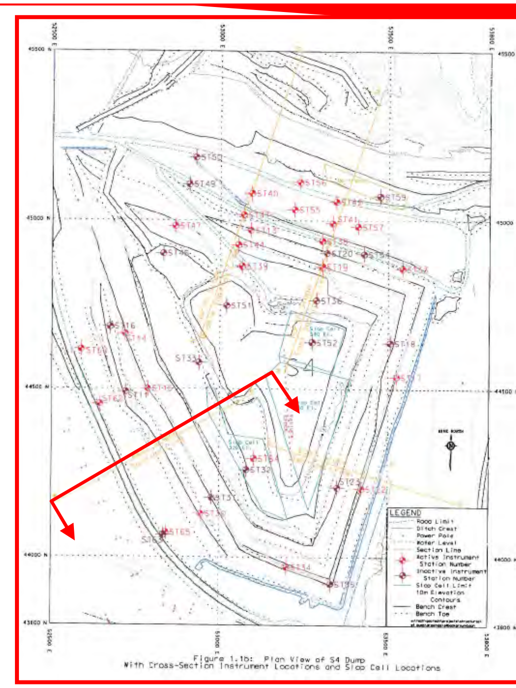


Table E2: Brief Summary of S4 Dump Loading History

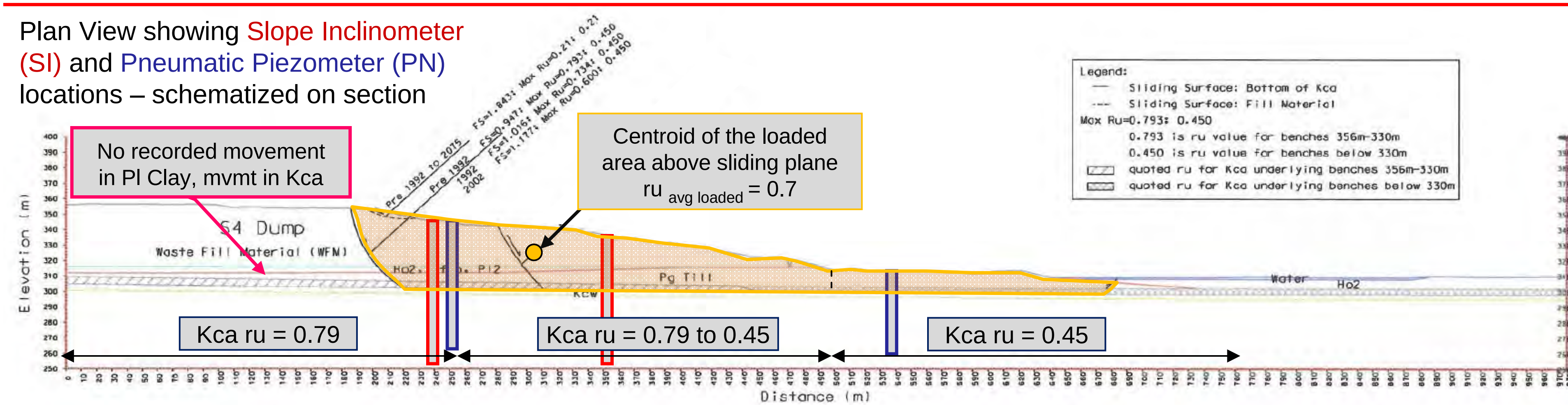
Slope	Section Line	Maximum Elevation of Crest of Dump (m)	Original Ground Elevation below the Crest of Dump (m)	Kca bottom elevation & depth below crest of dump: Elev. (m)	Depth (m)	Kca top elevation & depth at toe of dump: Elev. (m)	Depth (m)	Maximum fill thickness in Dump (m)	Crest to Toe Height (m)	Overall Slope from Dump Crest to Dump toe	Steepest Intermediate Slope > in Height	Total Recorded Movement ⊗⊗ (mm)
North	East Line	357m 347m	319m 318m	303.5m 303.5m	53.5m 43.5m	308m 308m	10 to 17m⊗ 10 to 17m⊗	38m 29m	34m 24m	7.4H:1V 6.2H:1V	4H:1V 4H:1V	>68mm >168mm
West	A-A	356m	318m	300m	56m	306m	8.5m-3m	39m	40m	7.7H:1V	6.7H:1V	>228mm
	B-B	356m	315m	304m	52m	305m	11.5m-8.5m	41m	40m	8H:1V	7.2H:1V	>352mm
	C-C	356m	317m	300m	56m	305m	9m-5m	39m	41m	8.9H:1V	6.1H:1V	>394mm
South	E-E	356m	317m	303m	53m	307m	7m-5m	39m	41m	7.4H:1V	6.6H:1V	24mm
East	"not reviewed"											

⊗ Kca under dump crest on the north slope is approximately 10m below the dump fill

⊗⊗ Actual Total Movement exceeds that recorded by the SI's, since replacement of new SI's did not always occur prior to pinching of the old. The total recorded movement is the sum of movement of many SI installations at one location.



Plan View showing Slope Inclinometer (SI) and Pneumatic Piezometer (PN) locations – schematized on section

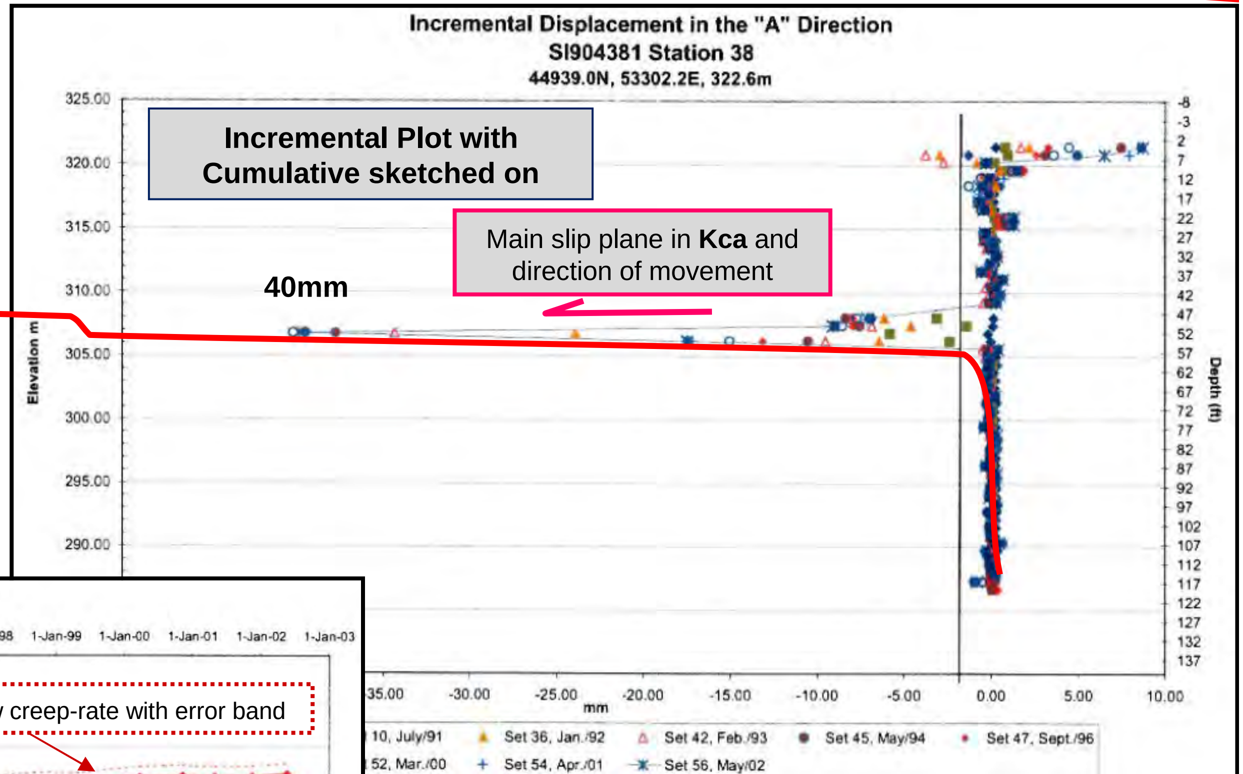


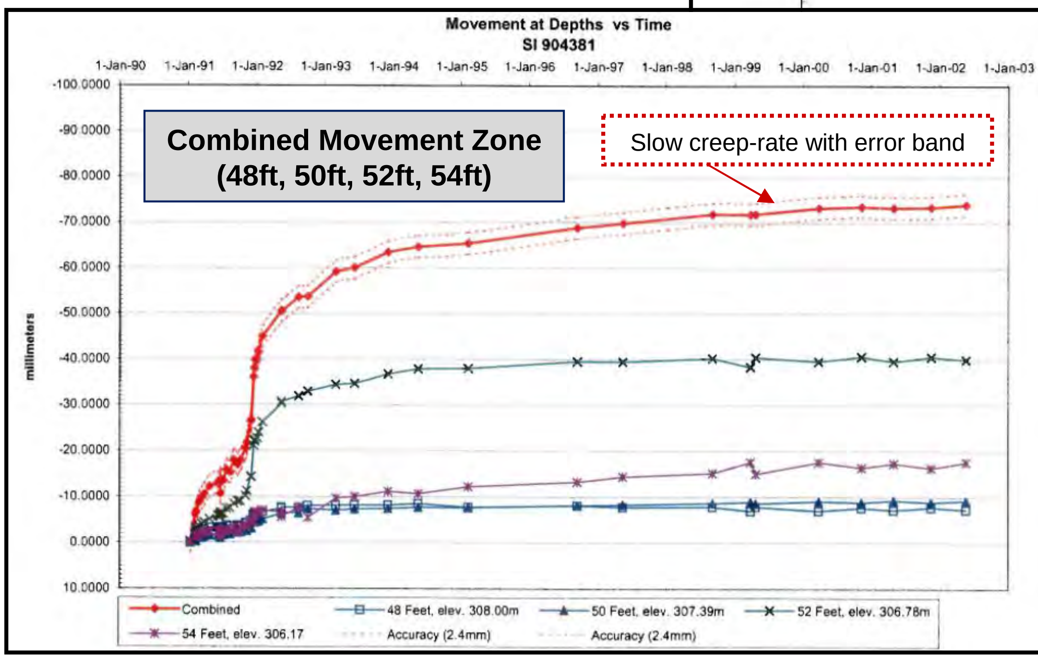
Back analysis used Φ'= 8 ° (7° lab residual strength + 1° for field effects) and either ru at centroid = 0.7 or multiple ru's under slope and got same FS = 1.0 for failure/mvmt

4a. S4 Dump – Out-of-Pit

Overburden Dump, 1989 to 1992 mvmt's

Sketch of Cumulative Down-slope Movement





<u>Measured movement:</u>

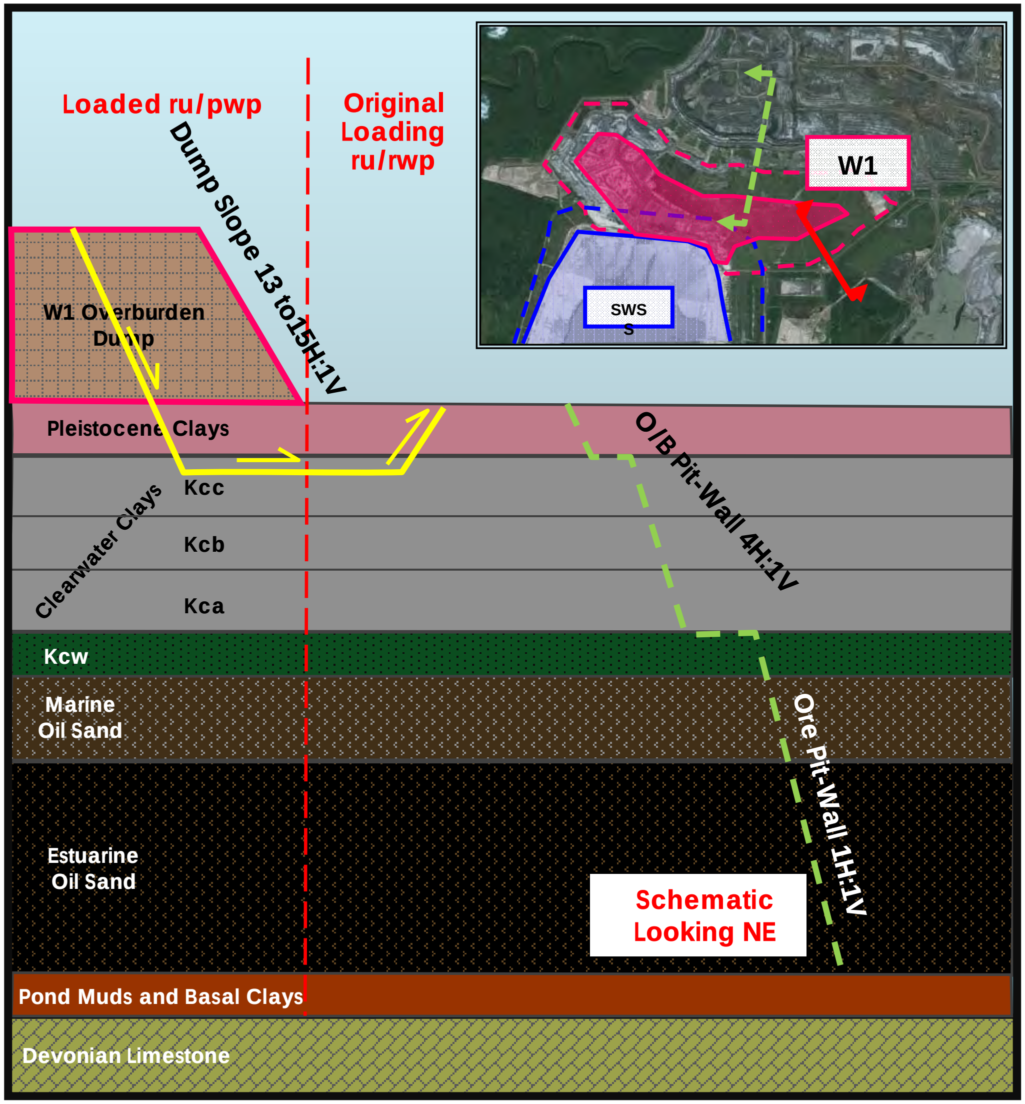
Station Locations show decreasing rate of movement

- 50mm/yr for 2yrs to;
- 10mm/yr for 2yrs to;
- 3mm/yr for 2yrs to;
- less than error bands

4a. S4 Dump – Out-of-Pit Overburden Dump, , 1989 to 1992 mvmt's Continued

- **Lessons Learned:**
1 – High plastic Kc Clay at residual strength is worse than Pl Clay – water content of both are close to optimum/ Plastic Limit
2 – **Ru increases for Kca** with increasing fill height (0.45 under dump toe, up to 0.793) – jumps when over 40m height. **Ru decreased for Pl Clay** with increasing fill height at S4 Dump
3 – Ru for Kca at centroid of dump gives same answer, but still need ru=0.45 at toe of dump and ru=0.0 by pitwall (**minimum 3 different ru's along slip plane, but using 5 or 6 may be better**)
4 – ru jumped up (and SI's moved) both when 1Mm3 additional fills added to crest of dump (5m thick) and when 1.5 to 2m of (reclamation) material was added close to area of previously high ru
    - SI's moved 50mm/year (from O/B fills) then slowed and stopped
    - SI's moved again at 10mm/year (from reclamation fills) which shows this area has a low factor of safety and Kca is near plastic strain curve
    - On the north slope SI movement was recorded at 2mm/day for 14 days but was relatively local and buttress was installed
5 – Φ'design = 8° (and back analyzed strength), lab residual strength testing tested in tension with tap water (not distilled water), tested at lab Φ'r=7°and added +1° for field effects giving Φ'= 8°
    - @ 4°, all slopes would fail,
6 – Slope deformation sheared many SI's – dump never completely failed, but caused construction to stop numerous times.
    - Stopping construction allows dump to 'settle down' allowing pore pressures to dissipate
    - Small ru changes send you along in plastic deformation at low FS – must allow time for dissipation/depressurization to occur
    - If construction must continue, can use flatter slopes with toe berms

4b. 'W1 Dump' – Out-of-Pit Overburden Dump, 2005 to 2009 mvmt's



Dump Characteristics:

- 117Mm3 of unengineered overburden fill
- 40m high above original topography with avg. 15H:1V side slopes to reduce amount of instrumentation required

Foundation Characteristics:

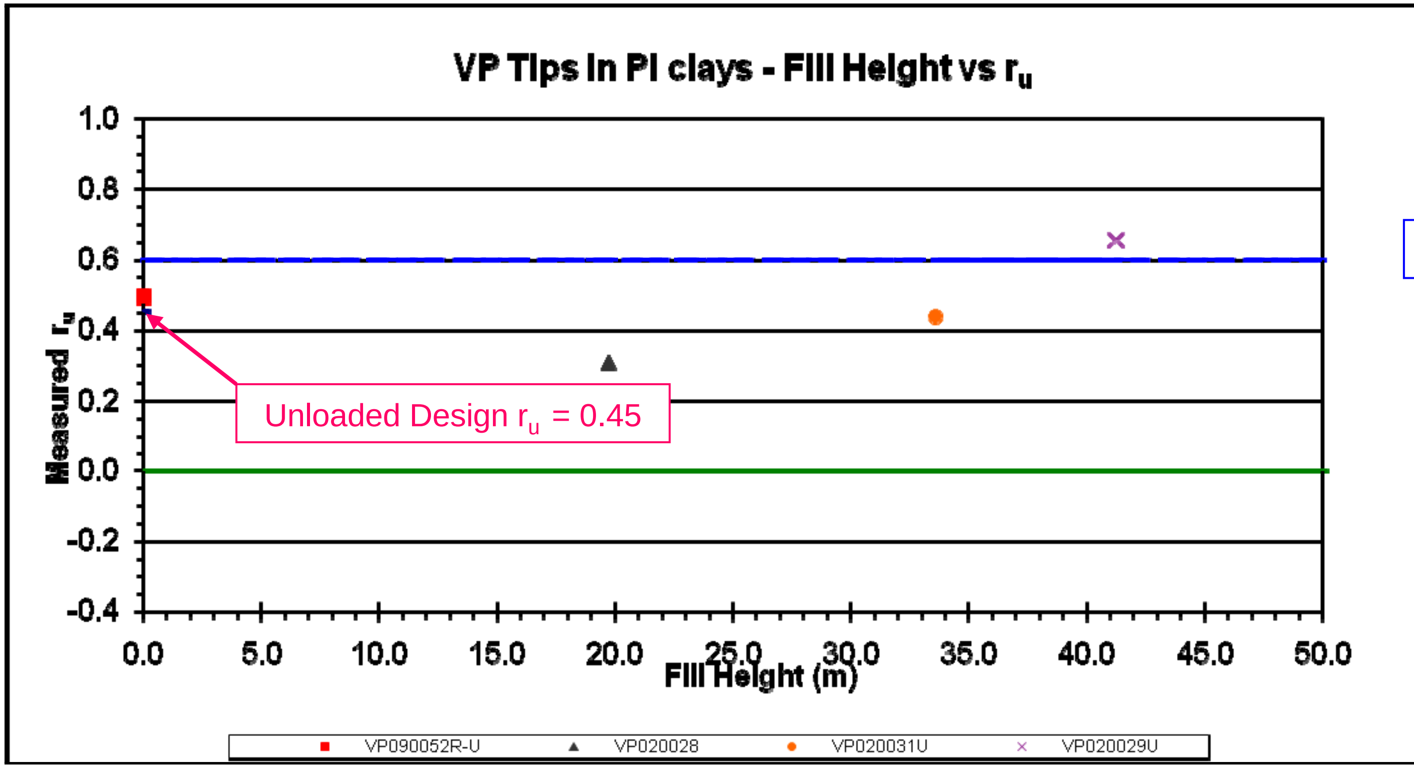
- Pleistocene Clays present but not a concern
- Approximately 30m thick Kc Clay unit underlies Pl unit
- Uppermost Kcc unit was glacially pre-sheared

Movement Characteristics:

- Internal deformation with no cracking or vertical displacement observed in area where overall dump slope ranged from 9H:1V to 13.4H:1V
- Movement area was on SE slope of dump (not towards pitwall)
- Slides along top of first weakest layer in the Clearwater clay unit (Kcc)
- Flattened slopes and delayed construction temporarily and used observational approach to continue dump construction

4b. 'W1 Dump' – Out-of-Pit Overburden Dump, , 2005 to 2009 mvmt's, con't

Vibrating Wire Piezometer data for In-situ Pl Clay under W1 dump fills

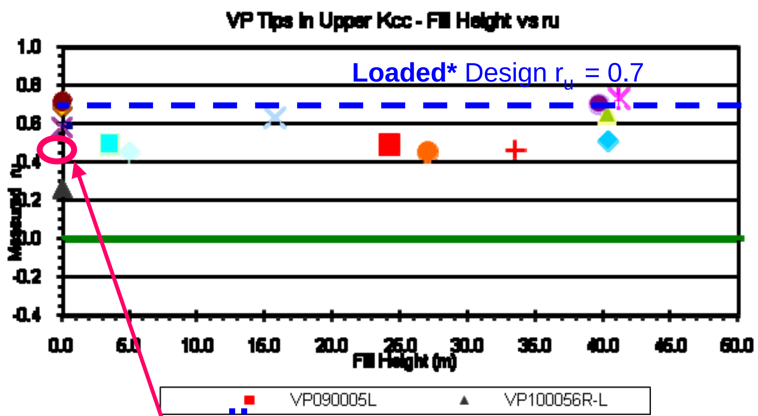


Loaded Design r_u = 0.6

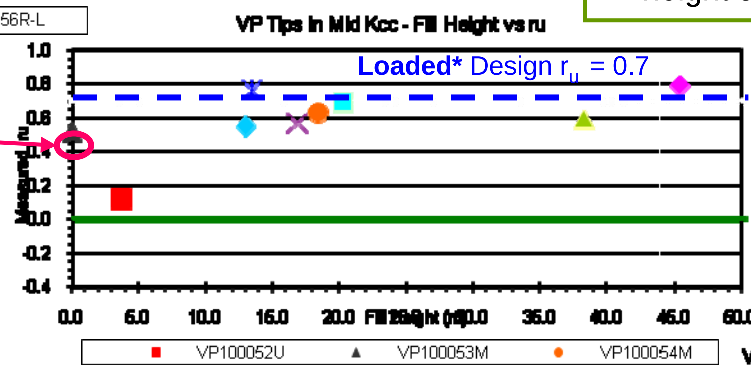
Note: ru in Pl clay staying the same or slightly increasing at W1 Dump unlike S4 Dump

4b. 'W1 Dump' – Out-of-Pit Overburden Dump, , 2005 to 2009 mvmt's, con't

- Multiple loaded design ru's can be applied to a range of fill thicknesses 5 to 40m or one average ru for the entire slope centroid after confirming fill height at centroid of loaded area
- This is due to the relationship between the failure mass geometry and the dump geometry centroids (future FLAC analysis need individual ru's for each height)
- These ru's generally increase with fill height small plots not showing it clearly

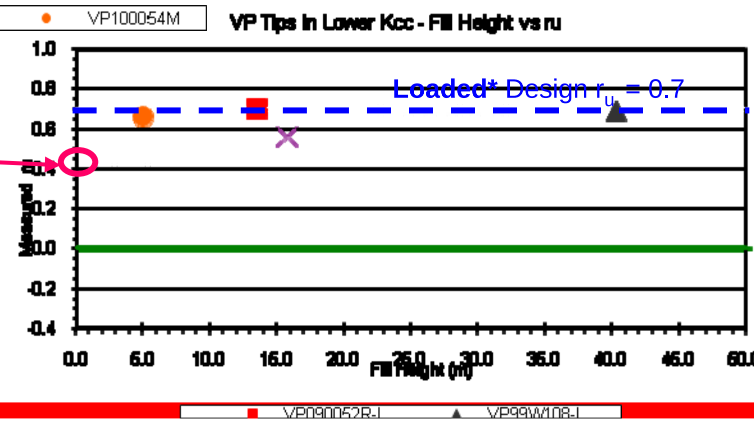


Unloaded Design r_u = 0.45



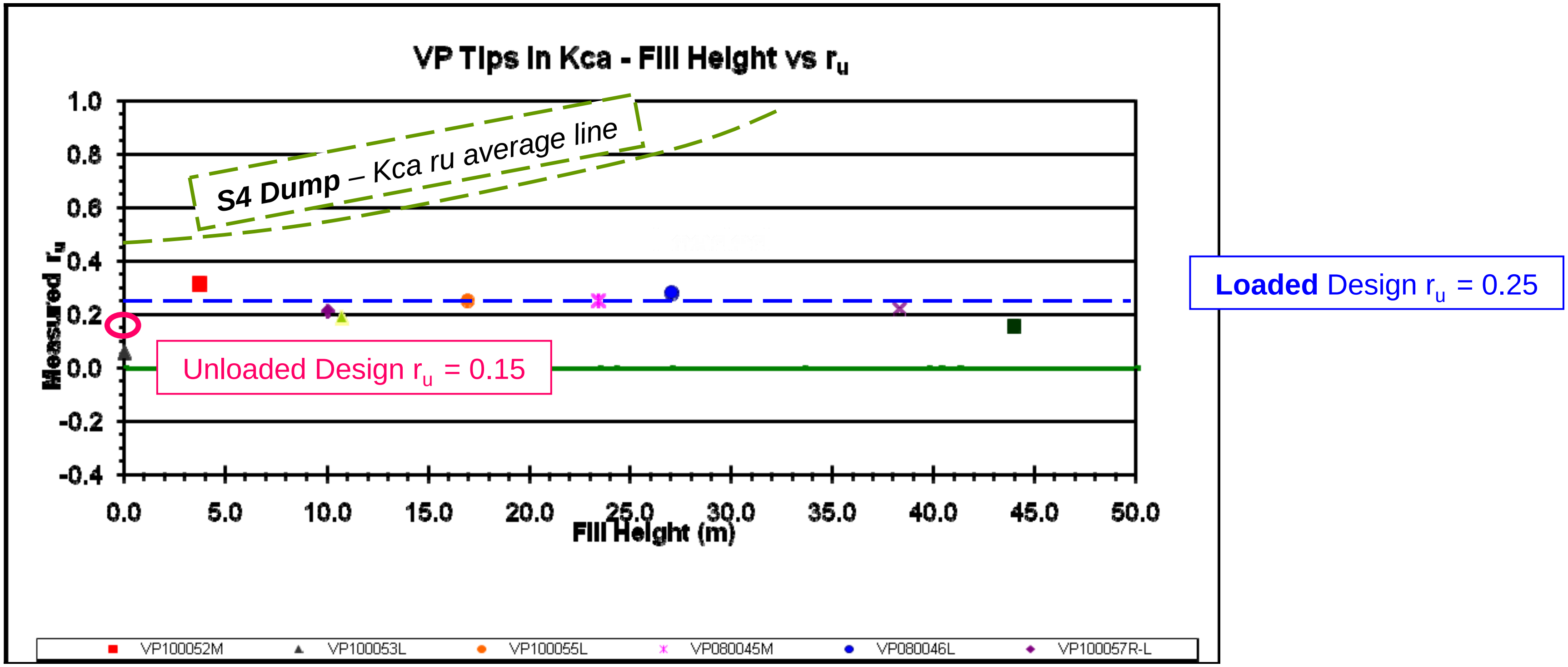
***S4 Dump** had Kca only (no Kcc or Kcb) underlying it as the upper Kc units were eroded away from that area*

Unloaded Design r_u = 0.45



4b. 'W1 Dump' – Out-of-Pit Overburden Dump, , 2005 to 2009 mvmt's, con't

- Kca deep under foundation (near 40m below top of original ground with thick, weaker Kcc units above) of W1 Dump ru=0.25 vs Kca under S4 Dump (approx. 6 to 17m below top of original ground for west section with all other Kc Clay units above eroded away) ru = 0.7
  - no or less glacial pre-shearing or glacial unloading as Kca units is nearly 40m below surface in this area

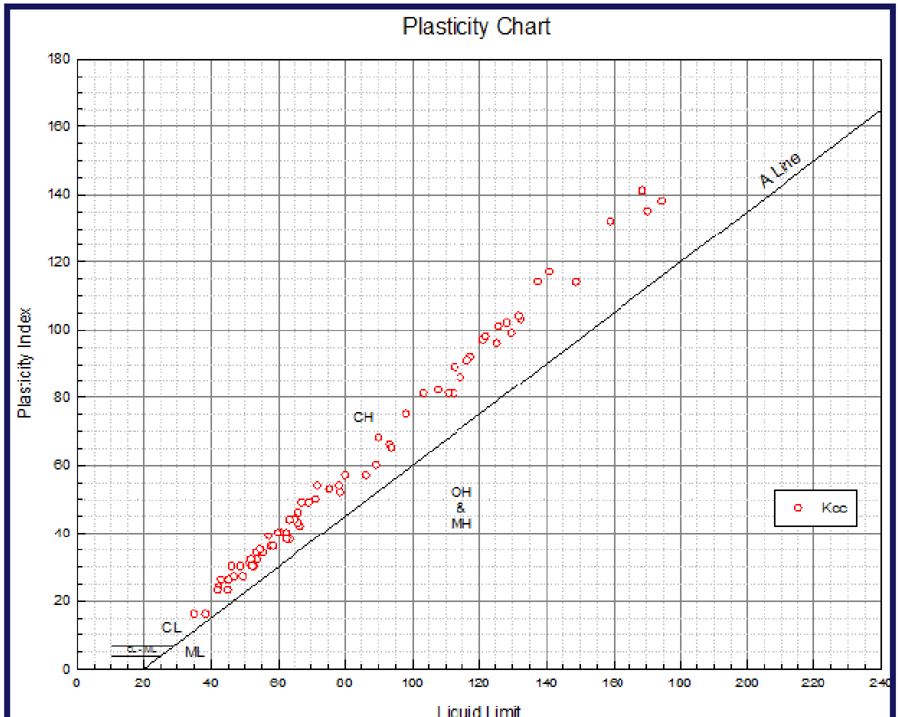


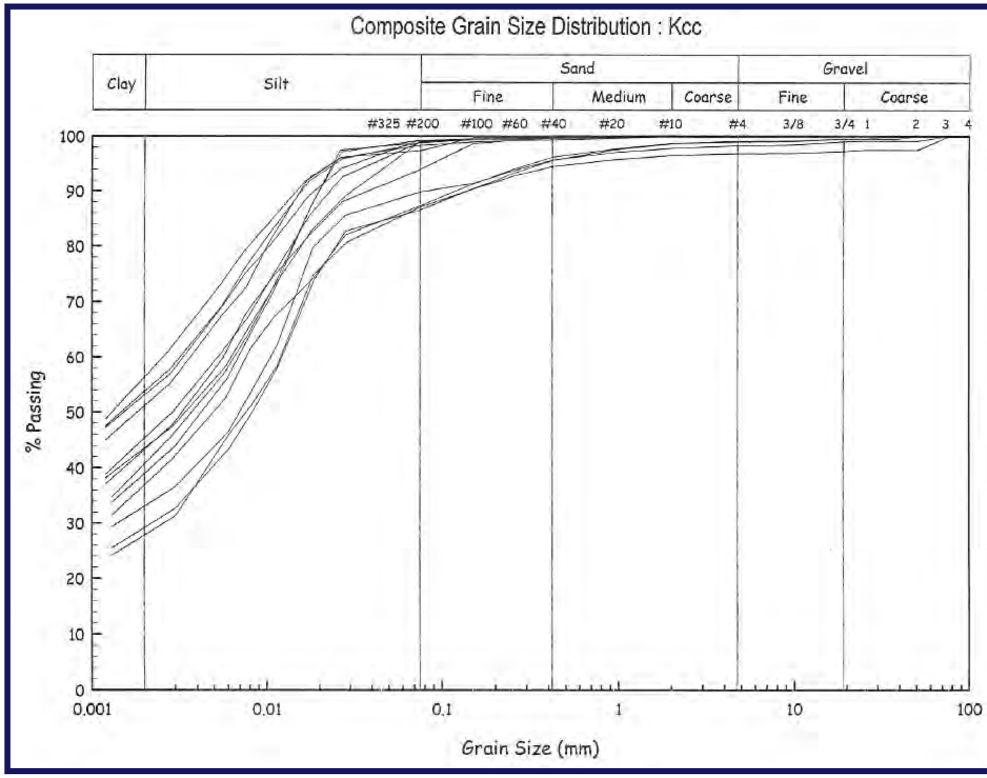
W1 Dump – Loaded ru's in Kca Clay

4b. 'W1 Dump' – Out-of-Pit Overburden Dump, , 2005 to 2009 mvmt's, con't

Clearwater (Kcc) Clay

- •High Plastic Clay LL=85.1%, PL=24.2% NMC=18.1
- •Greatly overconsolidated
- •Heavily pre-sheared
- •Measured $C'_r=0$, $\Phi'_r=8°$, Measured $r_u=0.45$ to 0.83
- •Cross-bedded design: $C'=25kPa$, $\Phi'=20°$, $r_u=0.7$
- •Sliding design: $C'=0kPa$, $\Phi'=8°$, $r_u=0.7$

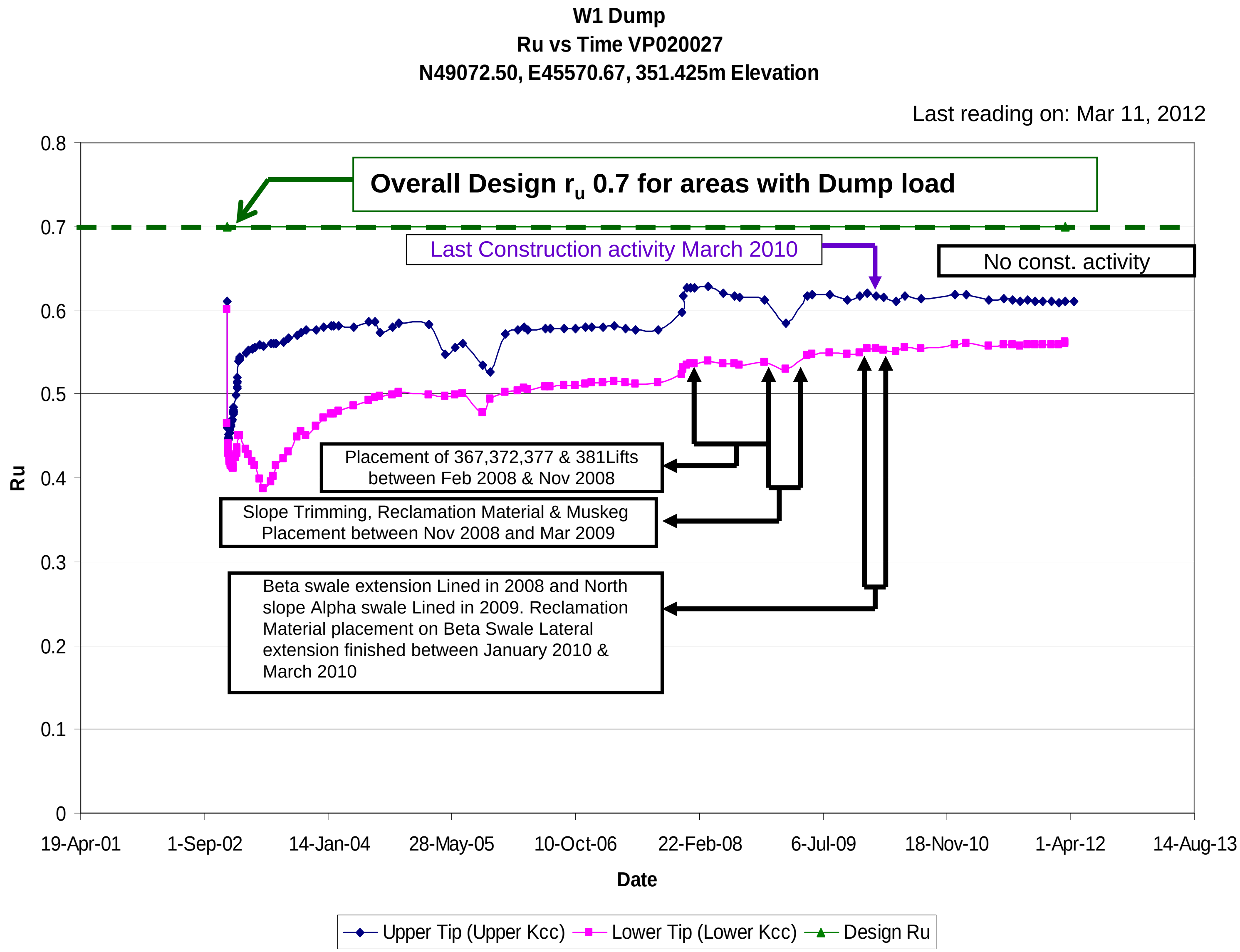




Summary of Test Results of Kcc Unit within W1 area

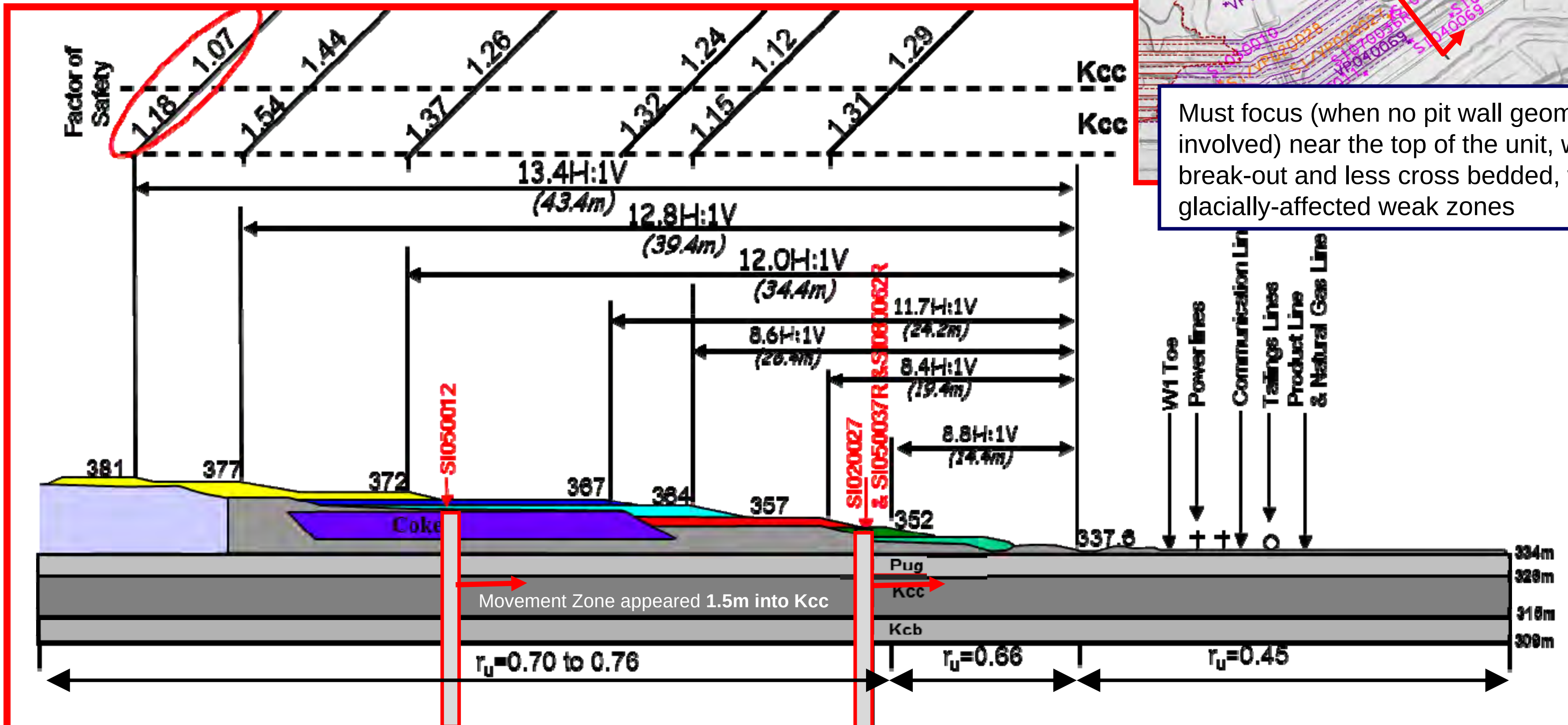
Test		n	Range	Average	Median	Std Dev.
Natural Moisture Content (%)		70	10.8-28.8	18.1	17.3	4.11
Atterberg Limit	Liquid Limit (%)	70	35.1-174.7	85.1	71.5	36.80
	Plastic Limit (%)	70	17.1-36.9	24.2	23.3	4.35
	Plasticity Index	70	16-141	61	50	33
	Plasticity	70	Low-High	High	Medium High	
Particle Size	% Gravel	63	0.0-0.0	0.0	0.0	0.00
	% Sand	63	0.0-23.8	1.0	0.3	3.02
	% Silt	63	22.4-81.0	54.1	55.7	13.87
	% Clay	63	18.7-73.3	44.9	42.8	13.19
Activity		63	0.60-2.76	1.27	1.18	0.44

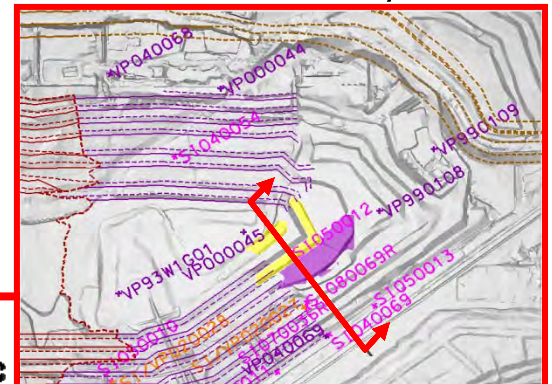
4b. 'W1 Dump' – Out-of-Pit Overburden Dump, , 2005 to 2009 mvmt's, con't



4b. 'W1 Dump' – Out-of-Pit Overburden Dump, , 2005 to 2009 mvmt's, con't

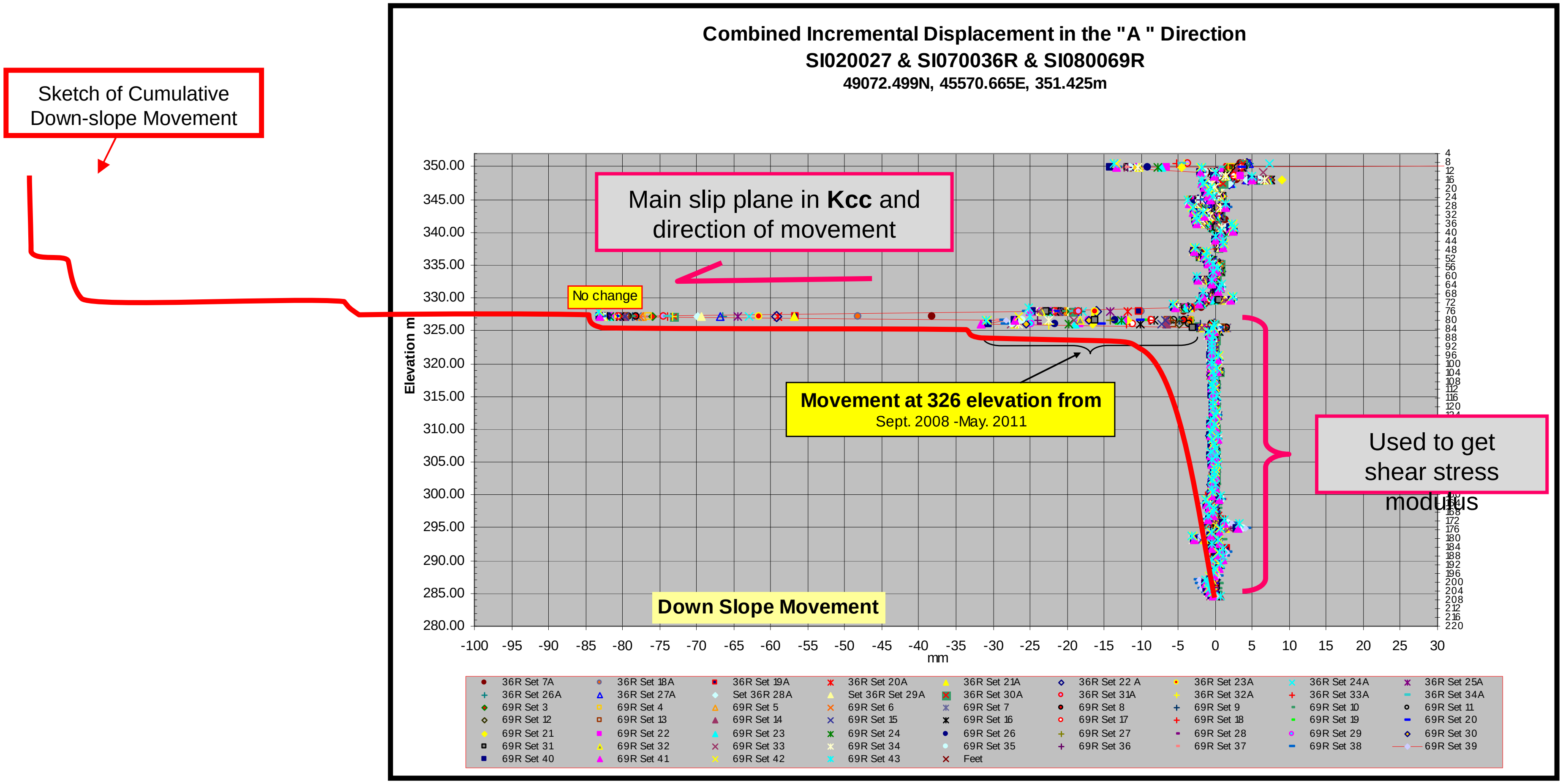
Controlled dump movement by stopping construction in 2007 for 2 months, then stepped each bench back flatter than previous





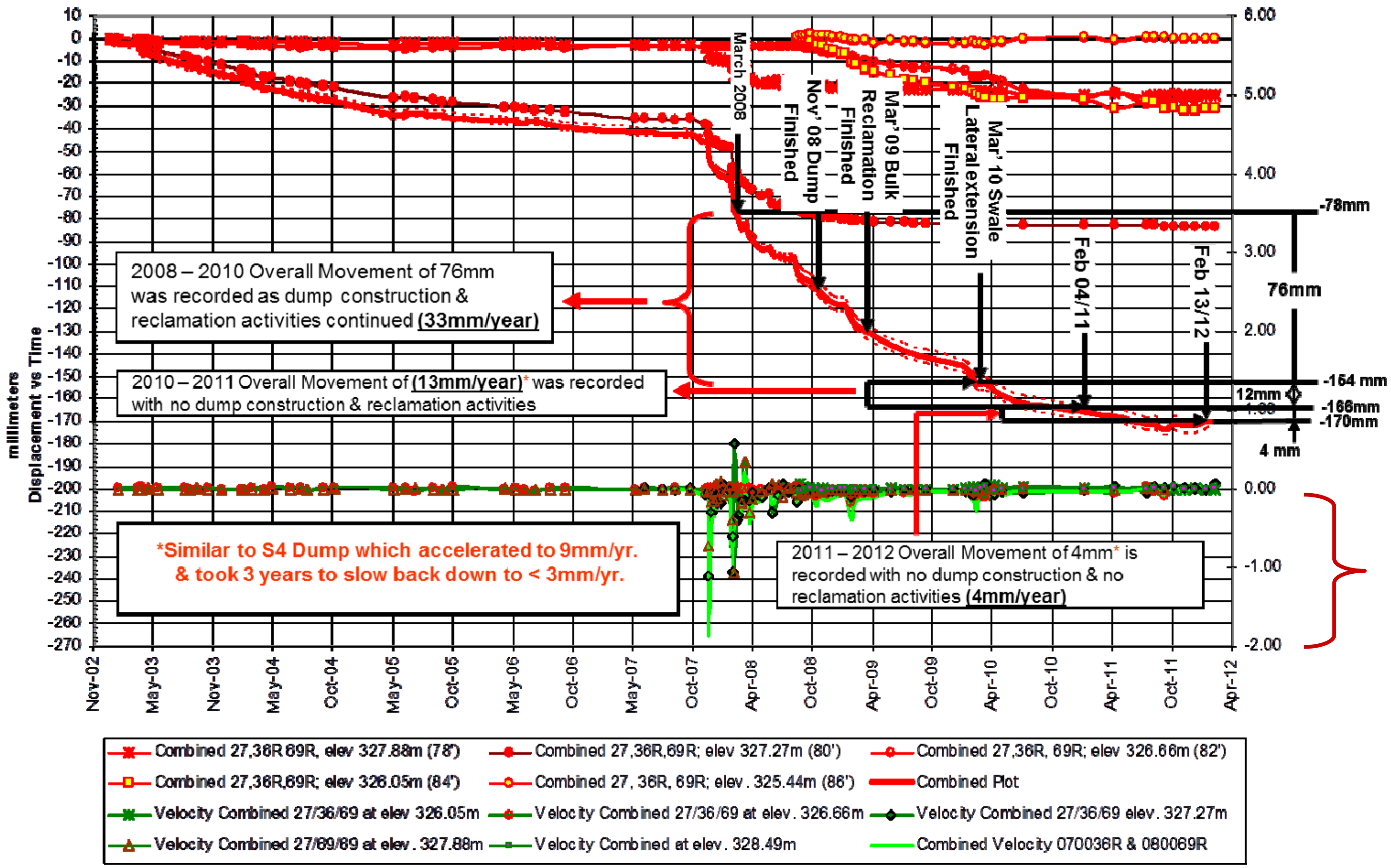
Must focus (when no pit wall geometry involved) near the top of the unit, where break-out and less cross bedded, where glacially-affected weak zones

4b. 'W1 Dump' – Out-of-Pit Overburden Dump, , 2005 to 2009 mvmt's, con't



4b. ‘W1 Dump’ – Out-of-Pit Overburden Dump, , 2005 to 2009 mvmt’s, con’t

Movement at Depth vs Time includes Combined Movement from 3 consecutively installed SI’s & solid red line is the Combination of Movement in the same zone
SI 020027 & 070036R & 080069R (351.425m Ground el.)



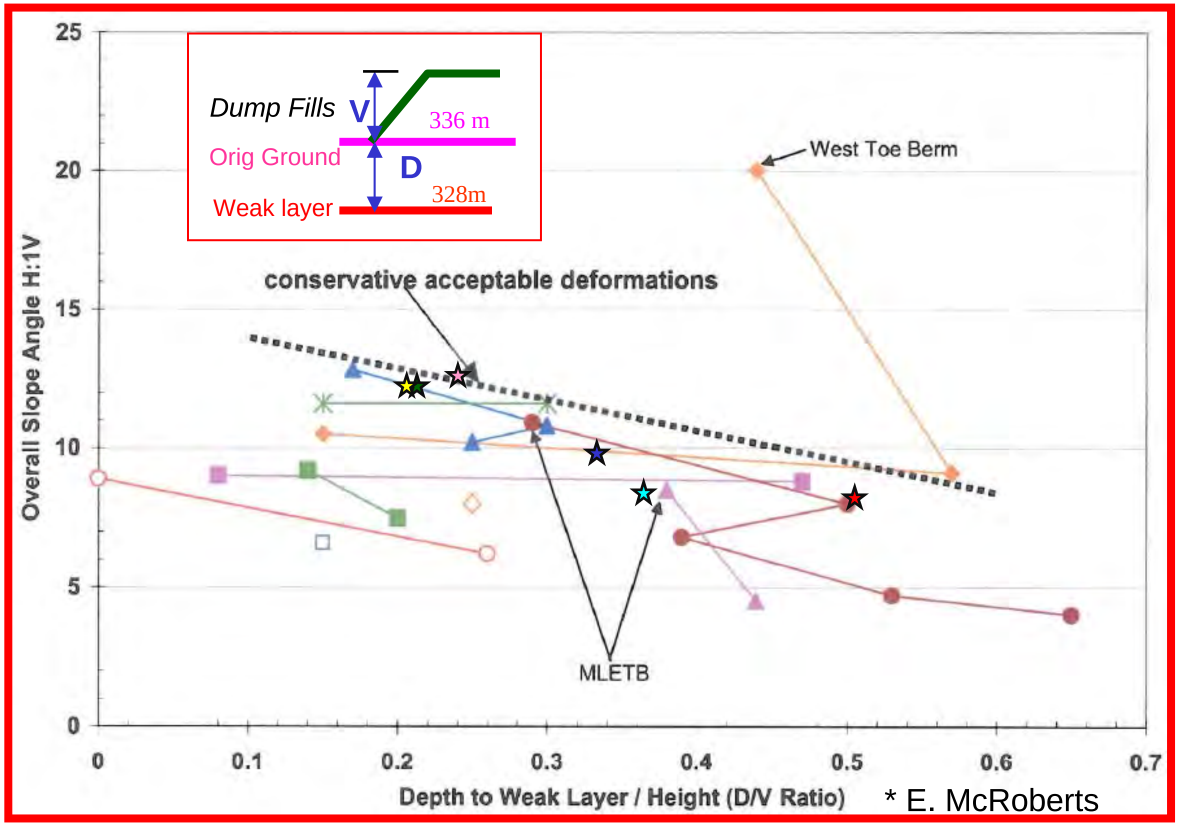
Approximately 178mm of total dump movement up to March 2013
(170mm shown up to April, 2012)

@ 1.0mm/day consider stopping placement

Velocity <2mm/day

@ 2.0mm/day must stop placement

4b. 'W1 Dump' – Out-of-Pit Overburden Dump, , 2005 to 2009 mvmt's, con't



- D/V deformation table shows deformation observed
- With acceptable movements defining the design, factor of safety could be 1.1 still
- FS=1.1 could lead to cracking and loss of serviceability
- FS 1.0 to 1.18 may also give movements above a given risk tolerance
- Modified from Ed McRoberts plot of serviceability limits for deformations

	Total Shear Zone Incremental A-dir Movement:	V=Height (m)
May 03/05 to Dec 05/07: Before any Buttressing	45mm	39.7m
Dec 17/07: After Placing the 347, 352, 357 Lifts	61.8mm	19.4m
Feb 29/08: After Placing the 364 Lift	70.5mm	26.4m
Mar 07/08: After Placing the 367 Lift	84.4mm	29.4m
Nov 17/08: After Placing the 381 Lift	111.5mm	43.4m
Feb 23/09: After Placing Reclamation material	124.5mm	44.9m

4b. 'W1 Dump' – Out-of-Pit Overburden Dump, , 2005 to 2009 mvmt's, con't

- **Lessons Learned:**
  1 – Dump never failed
  2 – During dump movement, more fill can be added, but must generally delay placement, then flatten overall dump slope as elevation increases (or construct toe berm, if area is available)
  3 – When factor of safety against slope instability is low, adding even minor additional loading can lead to plastic strain with an ru increase and could lead to failure sooner
  4 – Residual sliding shear strength along the foundation Kcc Clay unit is 8°- which is similar to S4 Dump but ru values measure:
    - W1 Dump **Kcc** (similar condition as S4 Dump **Kca**) ru=0.45 to 0.83
    - W1 Dump Kca ru=0.25
    - S4 Dump Kca ru=0.4 to 0.793
    - S4 Dump Pl clay ru decreased with fill height –drainage occurred and/or no response to loading
    - W1 Dump Pl clay ru stayed the same or slightly increased with fill height
  5 – @ 4° <u>all slopes</u> would fail, not all clay layers are 8°some can be 10 or 13°but this is based on lower Liquid Limits and lower clay contents.

4c. 'MLSB' – Out-of-Pit Sand-Constructed Tailings Dam, 1980 to 1992 mvmt's

Dam Characteristics:

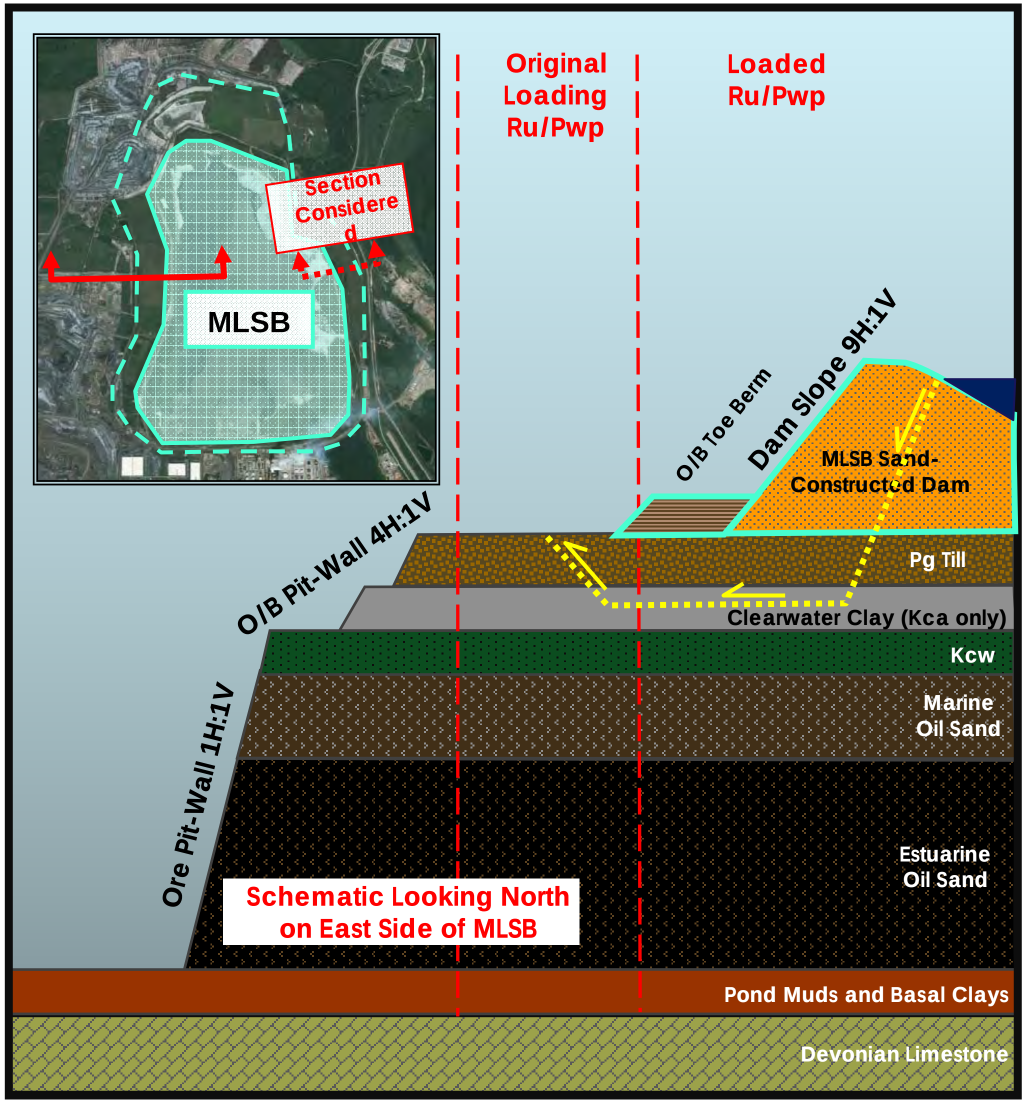
- 750Mm3 of engineered and unengineered sand
- 40m high above original topography with avg. 9H:1V side slopes (max. 4H:1V, min. 20H:1V)

Foundation Characteristics (for section considered):

- Pleistocene Clays present and not an issue
- Pg Tills and Kc Clay Tills present at/near surface
- Most of the original 30m of Clearwater eroded away by glaciation – similar to S4 Dump
- Remaining 1 to 5m of Kca Clay unit was glacially-affected to residual strength

Foundation Loading Characteristics:

- During early design stages, field strength of Kca was chosen at Φ'=12° and ru=0.3
- From more/better lab testing, design strength was reduced to Φ'=8°
- From increased loading of instrumented areas, design ru value was increased to 0.4, then 0.55, then 0.62, then finally to ru=0.7 to 0.75, but not over whole length of slip surface
- An overburden toe berm was added to increase factor of safety as dam went higher
- If Φ'=4°, slope failures would exist



4c. 'MLSB' – Out-of-Pit Sand-Constructed Tailings Dam, 1980 to 1992 mvmt's con't

Fig. A1. Spreadsheet model: Section I — dyke geometry, stratigraphy, and input variables.

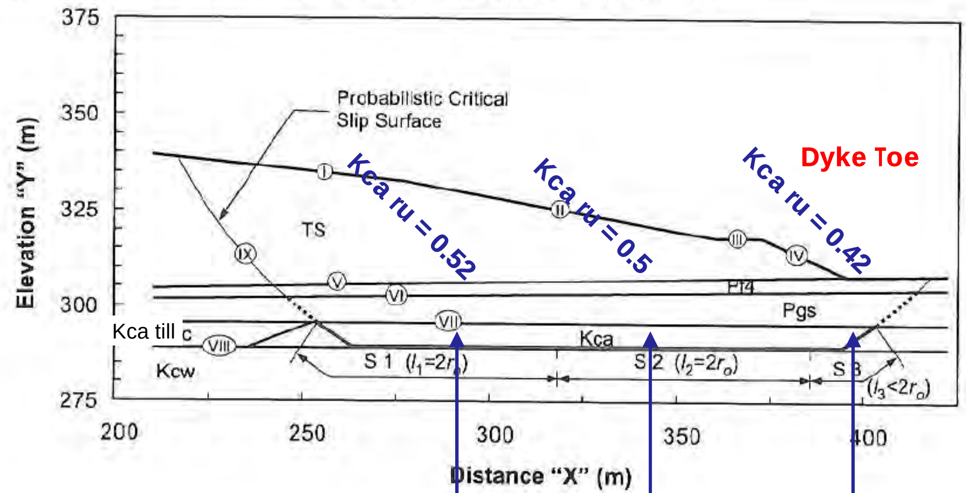
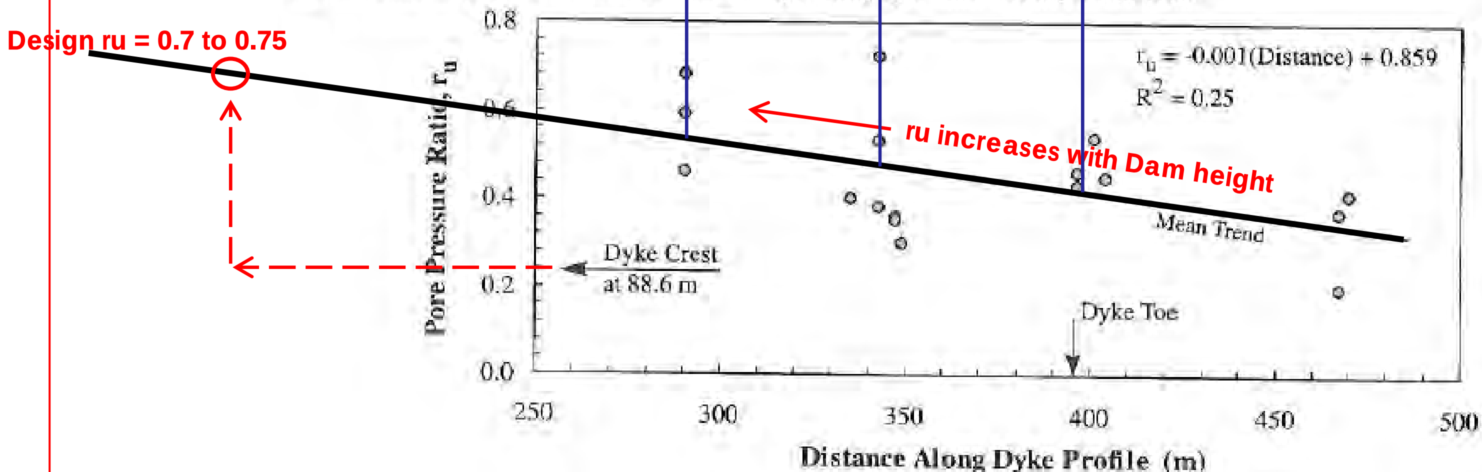
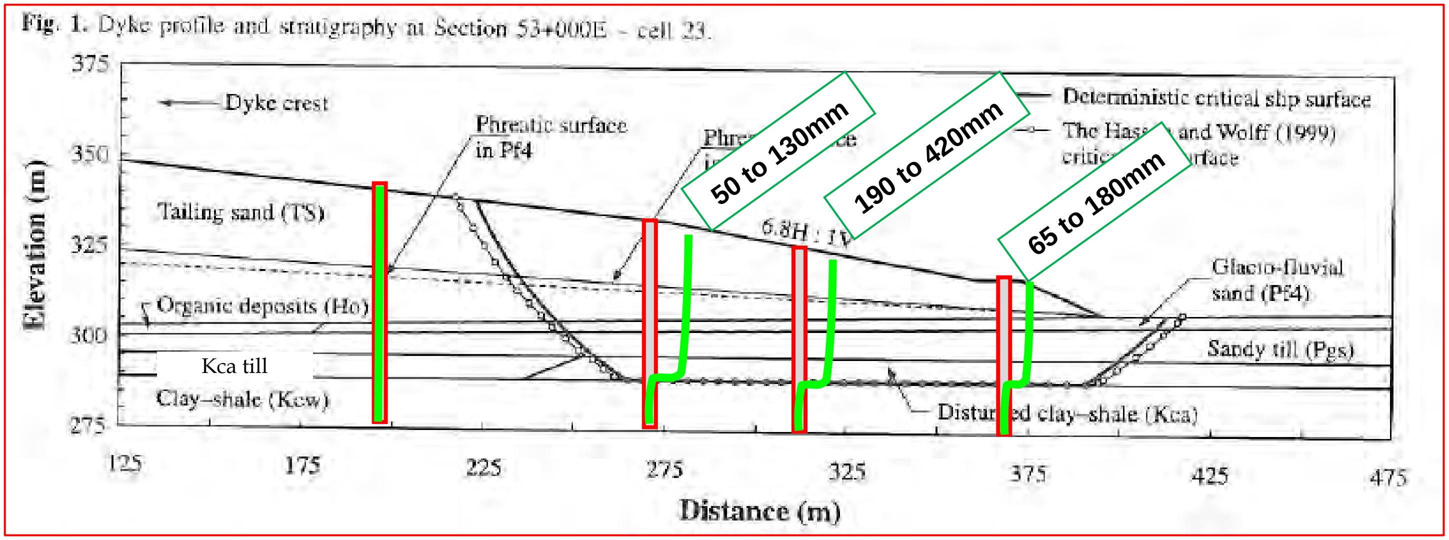


Fig. 4. Profile of pore pressure ratio in the Kca layer along dyke cross-section, March 1994.



MLSB – Analyzed Section and Loaded Kca ru Response

4c. 'MLSB' – Out-of-Pit Sand-Constructed Tailings Dam, 1980 to 1992 mvmt's con't



- Three SI's stations showed movement jumps of 50 to 130mm, 190 to 420mm, and 65 to 180mm
  - This movement event resulted in the construction of the overburden toe berm to flatten the overall dam slope to 20H:1V
  - Multiple SI's may be required at a given movement location (station) if slope movements exceed SI tube shear strength
- Even if SI's do not have the same total movement, if all three move 10mm together when you build upper lifts, that is trouble
- Generally safe from failure if SI at toe does not show similar movements to those mid-slope, as this is indicative of internal deformation

4c. 'MLSB' – Out-of-Pit Sand-Constructed Tailings Dam, 1980 to 1992 mvmt's con't

- Lessons Learned:

1 – Kca is pre-sheared in this area and has residual strength at 8°
  - Initial design strengths used ranged from 12° to 16° with use of an observational method for construction
  - At 4°, all slopes would have fallen down

2 – From increased loading of instrumented areas, design ru value was increased to 0.4, then 0.55, then 0.62, then finally to <u>ru=0.7 to 0.75</u>, but not over whole length of slip surface
  - Kca ru increases with fill height

3 – As Dam height increased and more accurate information was obtained for loading conditions, design slopes were shallowed:
  - 2.5 to 6.8H:1V to 5.5 to 9H:1V to 20H:1V

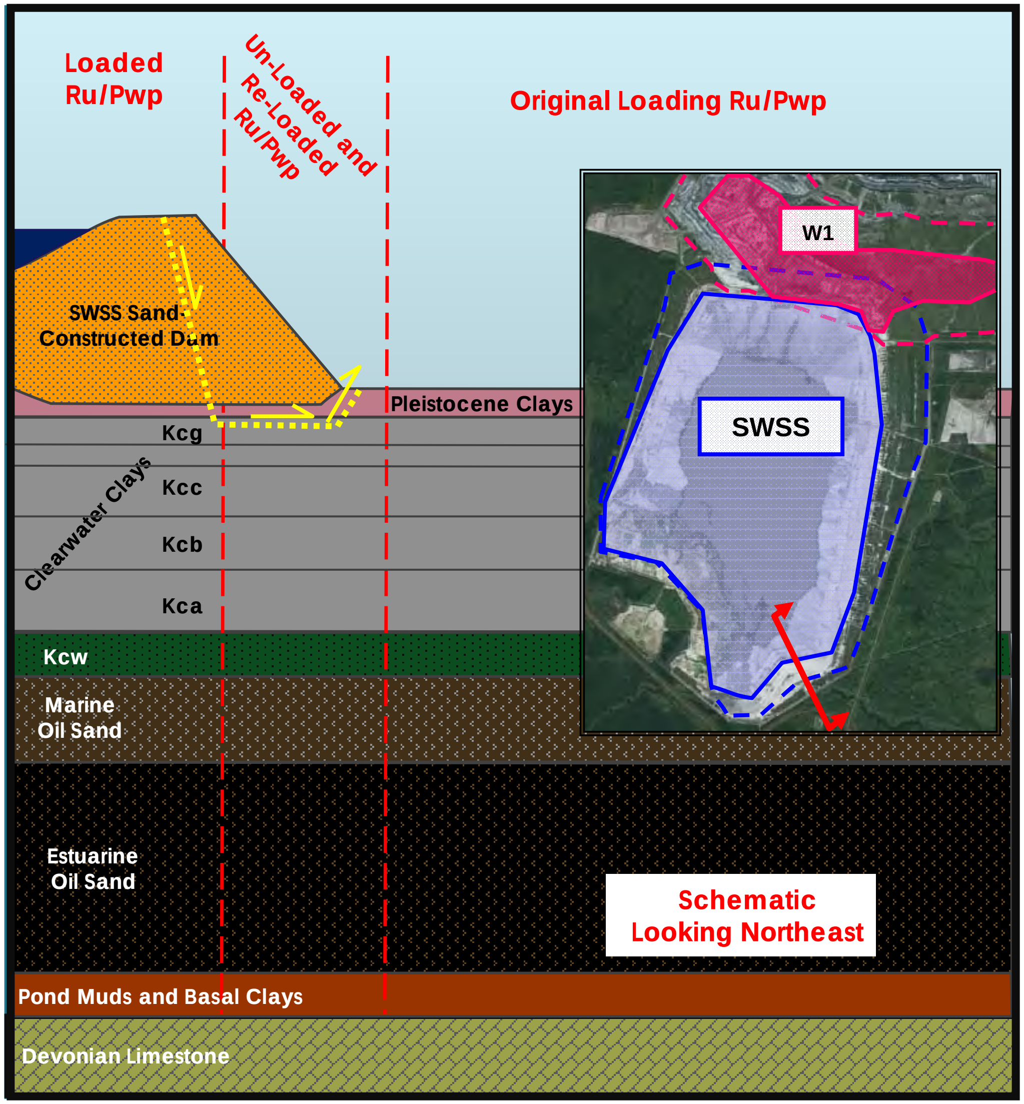
4 – 420mm of movement could occur without failure

5 – When SI's at the toe, middle, and upper slopes are moving at the same velocity, means large-scale failure is imminent – Toe berm can stop this

6 – Toe berms need to be relatively steep at lower heights and have benches to provide maximum toe weight/support – better to have local instabilities in the toe berm than in the containment structure/dam

7 – At low factor of safety, plastic strain characteristics were observed – this required caution in the form of:
  - Closer instrumentation monitoring
  - Flattening overall dam slopes
  - Use toe berm to take advantage of ru profiles

4d. 'SWSS' – Out-of-Pit Sand-Constructed Tailings Dam, 2009 to 2012 piezo response



Dam Characteristics:

- 750Mm3 of engineered and unengineered sand
- 40 to 50m high above original topography with avg. 20H:1V side slopes

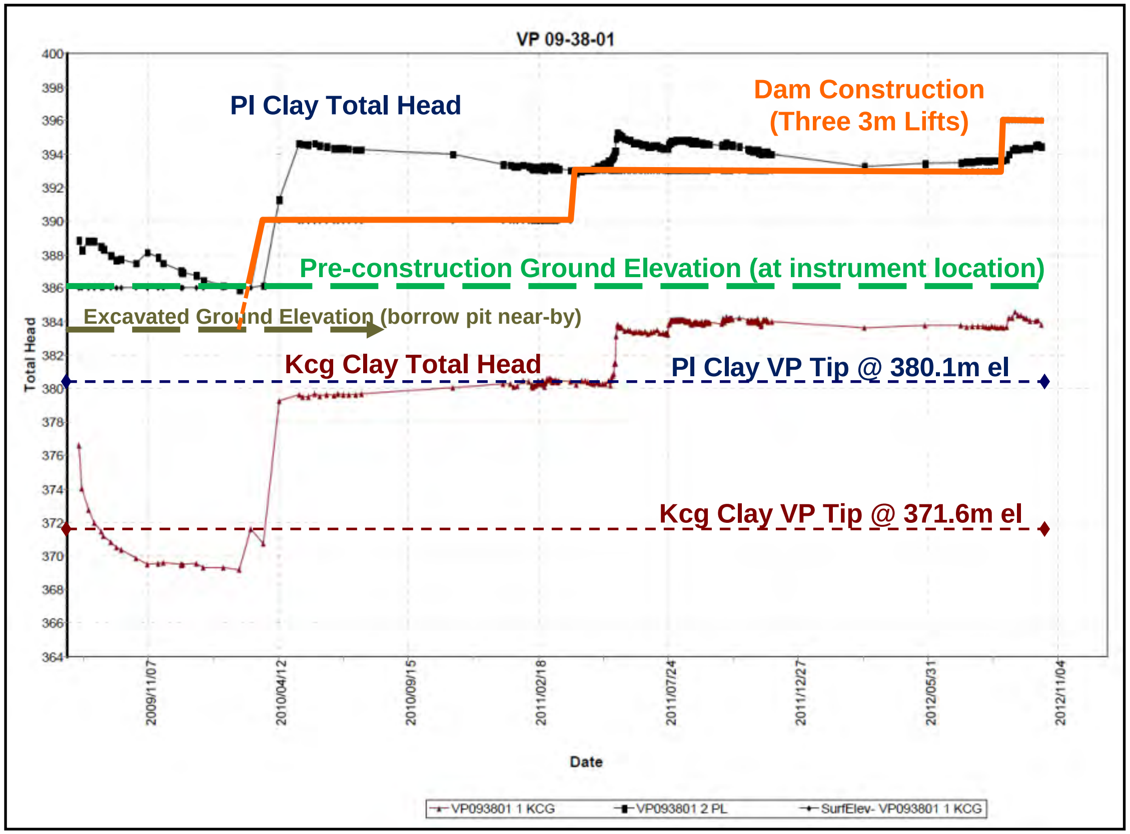
Foundation Characteristics (for section considered):

- Pleistocene Clays partially removed from a borrow pit in the section area
- Kcg unit present near surface
- Kcg in this area has high clay and moisture content and is glacially pre-sheared

Non-Movement Loading Characteristics:

- VP tip in Kcg Clay unit shows that the clay goes into suction as a result of unloading
- After the first 3m sand lift, rapid increase of Kcg pore pressure occurred
  - Bbar reading >1
  - ru reading of <0.5
- Factor of Safety against slope instability was verified to be acceptable and construction continued, with close monitoring
- Pore water pressure appeared to stabilize once construction was completed

4d. 'SWSS' – Out-of-Pit Sand-Constructed Tailings Dam, 2009 to 2012 piezo response con't



- Due to the saturated nature of the Pl and Kcg clay units, ru drops are observed in the initial stage of their readings, likely as a result of the near-by excavation
- The Kcg shows ru<0 which means the unit is in suction
- Once the first 3m sand lift is placed, both Pl and Kcg pore water pressures increase dramatically
    - In the Kcg unit, the ru jumps from approximately ru = -0.2 to ru = 0.6
    - Since Bbar only measures the change in pore water pressure, the Bbar reading was over 1

$$r_{u_{tip}} = \frac{h_w}{h_s} \times \frac{\gamma_w}{\gamma_s}$$

$$B_{bar} = \frac{\Delta hw}{\Delta hs} \times \frac{\gamma_w}{\gamma_s}$$

4d. 'SWSS' – Out-of-Pit Sand-Constructed Tailings Dam, 2009 to 2012 piezo response con't

Lessons Learned:

1 –Close-by excavation caused pore water pressure to drop in elevation by the approximate thickness of soil removed. Pore water pressure came right back when sand was placed above – in terms of ru, not an issue for stability and no SI movement as design was for 0.7 with additional FS

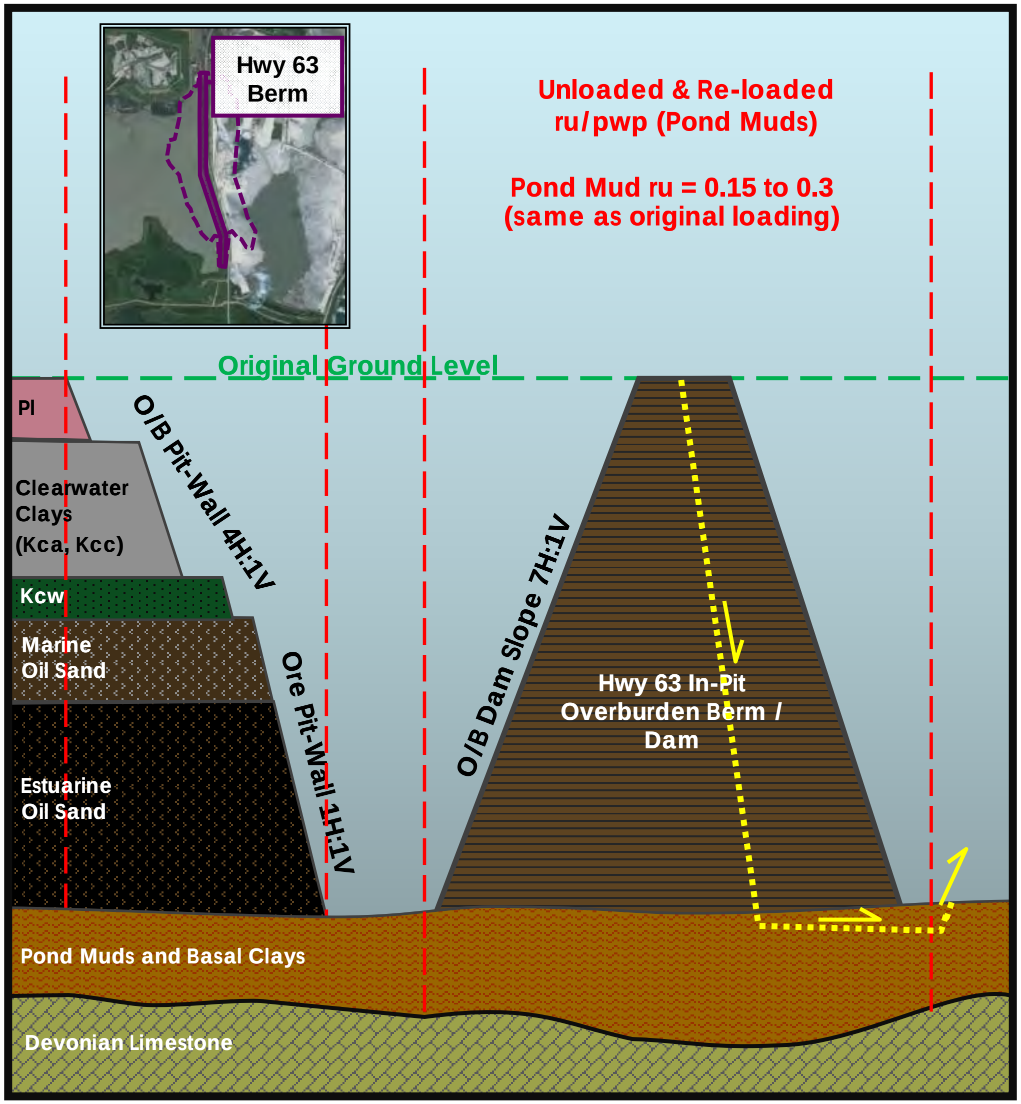
- Knowledge of geological history, as well as recent loading/unloading history is important for selecting design parameters
- Actually is a high real Bbar so must watch anyways but not as scary when you know a lot of the jump was a result of the previous excavation (this also occurs during excavation for shear keys)

2 –ru is what gives pore water pressure for stability analysis

3 –Instrumentation readings need proper interpretation and the use of ru over Bbar for non-sand fills/in situ is recommended for this.

–Piezometer plots need to include tip elevation and ground elevation on the same plot so one can properly interpret the results.

4e. 'Hwy 63 Berm'– In-Pit Overburden Tailings Dam, 1992 to 1994 design



Dam Characteristics:

- 42Mm3 of engineered and 40Mm3 uncontrolled O/B fill
- 65m high above mined-out pit floor with avg. 7H:1V side slopes

Foundation Characteristics:

- All overburden and ore removed from Dam foundation
- Undulating Pond Muds, Basal Clays, and Crevasse Splay exist under Dam, ranging from 10 to 20m thick
- Pond Mud was observed to be pre-sheared, both from core samples and from visual observation of pit floor

Non-Movement Loading Characteristics:

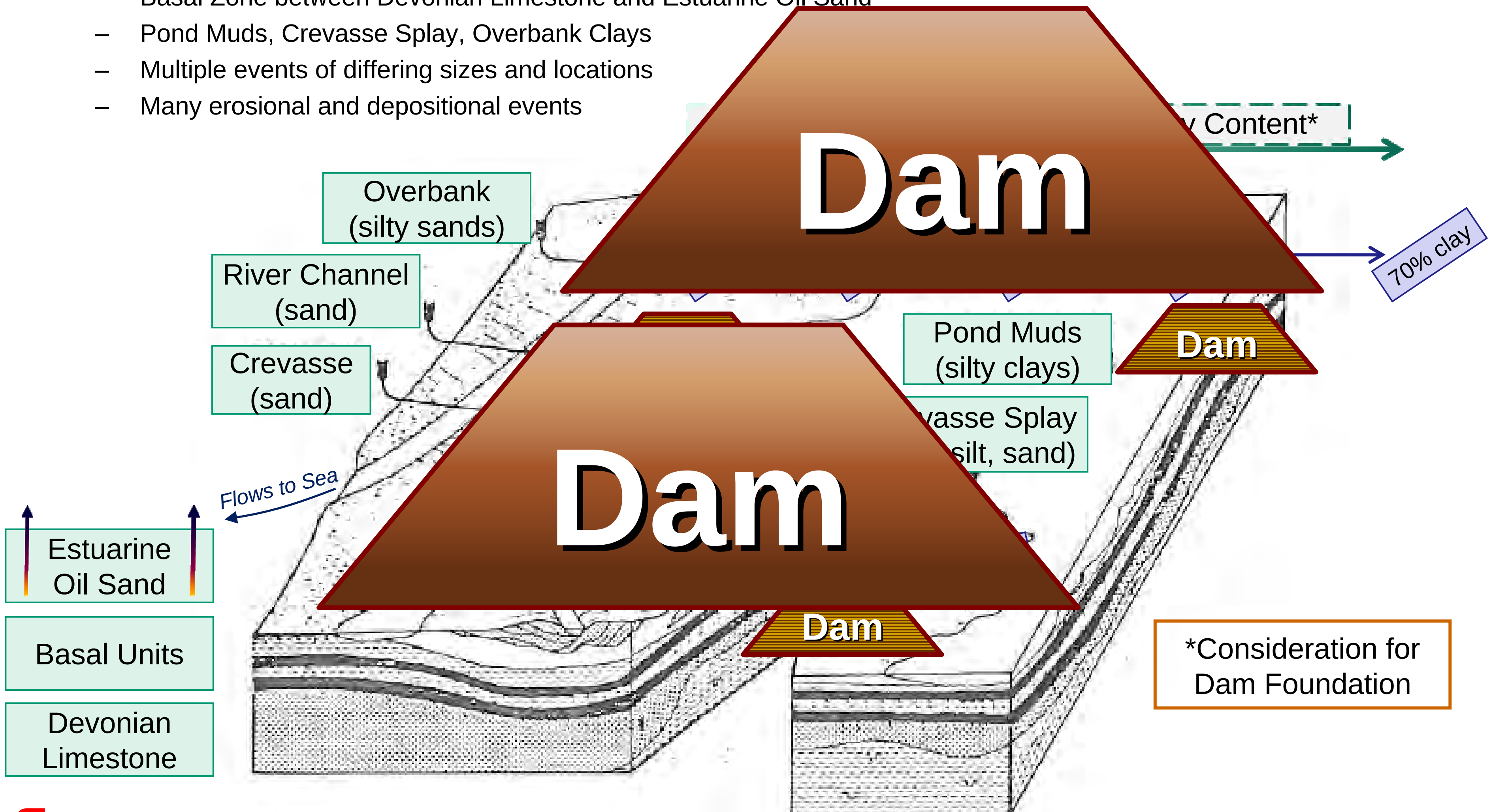
- Concern existed for fluctuating ru value due to water ingress into the porous basal units from the infilling of tailings fluids on both sides of the dam
- Design strength (residual) of Pond Mud was originally thought to be 6°
  - Questioned since Kca, a higher-plastic clay, is $\Phi_r = 8°$
- Further sampling/testing of these basal units returned higher but still variable strengths (Φ_r = 9° to 11°) that appeared to based on the samples' clay content
- Different design strengths could be assigned to certain areas of the basal units which, because of a good understanding of the geological depositional environments, were able to have field effects factored in

Important to note: 9° strength is 50% more resistant to movement than 6° and considering a dam stability analysis, 9° can support a FS=1.3 for a short term, where 6° at FS=1.3 would fail.

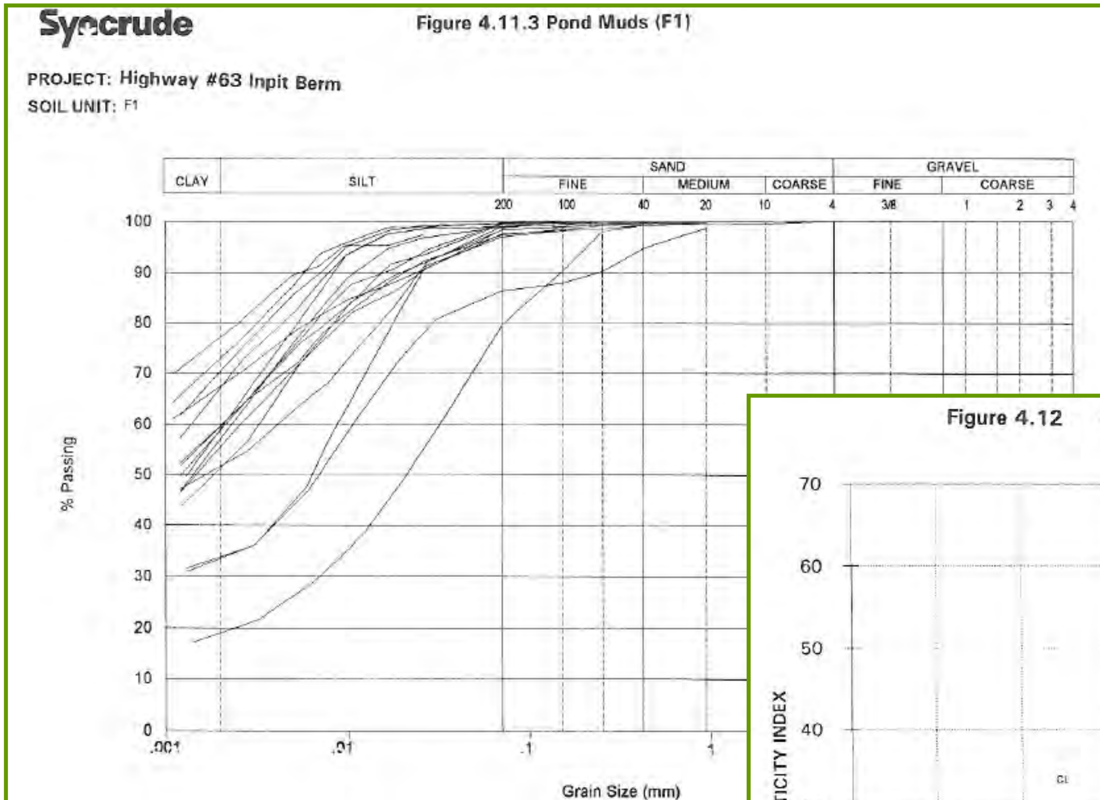
4e. 'Hwy 63 Berm' – In-Pit Overburden Tailings Dam, 1992 to 1994 design con't

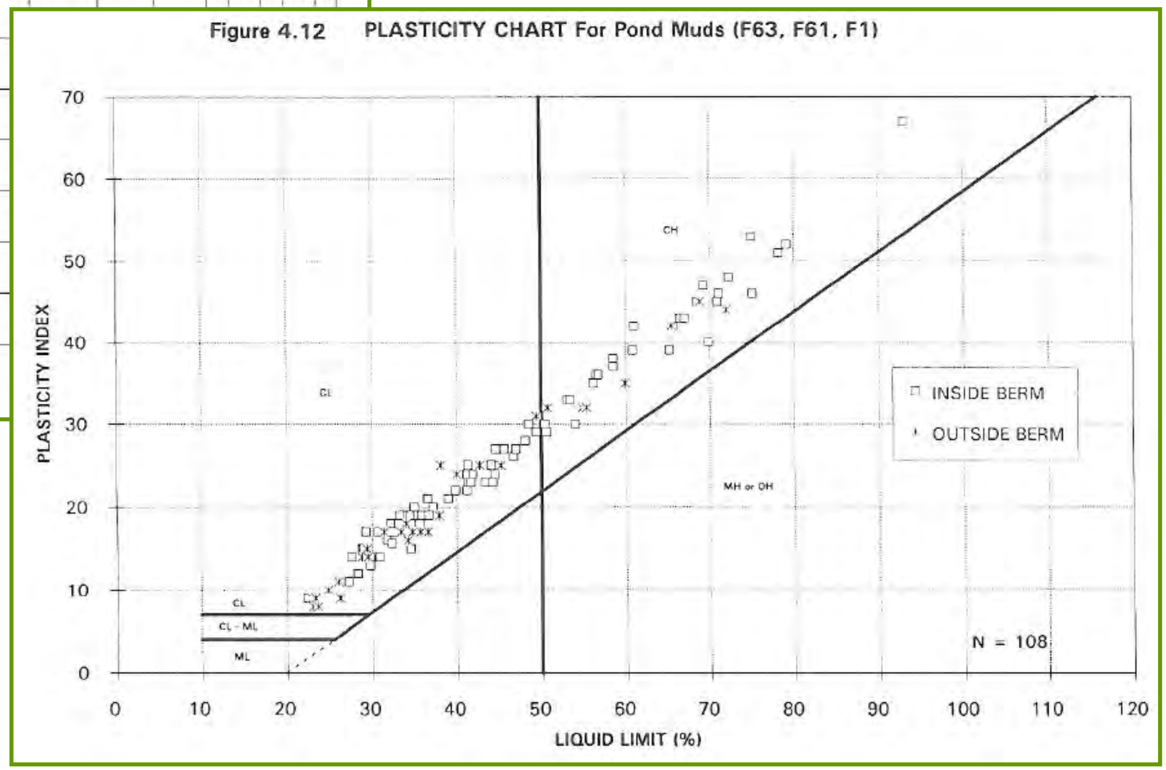
Schematic of Fluvial Depositional Environment (Repeated)

- Basal Zone between Devonian Limestone and Estuarine Oil Sand
- Pond Muds, Crevasse Splay, Overbank Clays
- Multiple events of differing sizes and locations
- Many erosional and depositional events



4e. 'Hwy 63 Berm' – In-Pit Overburden Tailings Dam, 1992 to 1994 design con't





4e. 'Hwy 63 Berm' – In-Pit Overburden Tailings Dam, 1992 to 1994 design con't

Table 4.1: Foundation *In situ* Soil Design Values

Soil Unit	Average Clay Content of Tested Sample (%)	Peak Direct Shear Strength (c_p'; ϕ_p')	Lab Residual Direct Shear Strength (c_r'; ϕ_r')	Design Clay Content (%)	Design Shear Strength Across Bedding (c'; ϕ')	Design Shear Strength Along Bedding (c'; ϕ')	Field Effects Required For Design Strength from lab Residual (c'; ϕ')	Average r_u Measured in the Field[3]	Design r_u
Estuarine Oilsands	N/A	30 kPa; 54°	34°	N/A	50°	50°	N/A	0.10	0.25
Pond Muds (f61, f63, f1)	33	26.6°	16°	70	20 kPa; 15°	9°	2.1°	0.14	0.2 to 0.3
	50	35 kPa; 20.4°	10°						
	75	35 kPa; 13°	6.4°						
	75	11°							
Overbank Mud (f3, f70, f71, f77)	12	Not enough data	27.8°	35	30 kPa; 20°	16°	0.5°[1]	0.08	0.3
	32	33 kPa; 29.4°	12.7kPa; 17°						
Crevasse Splay (F2)	67	13.8°	11.9°	25	30 kPa; 20°	16°	N/A	0.42	0.3
Watersands	N/A	33.7°[2]	29.8°	N/A	32°	N/A	N/A	0.19	0.25
Paleosol	34	100kPa; 20°	17.7° or 12.5°	40	N/A	9°	0.5° to 1°	None	0.2
	58	15.2°	7.5°						

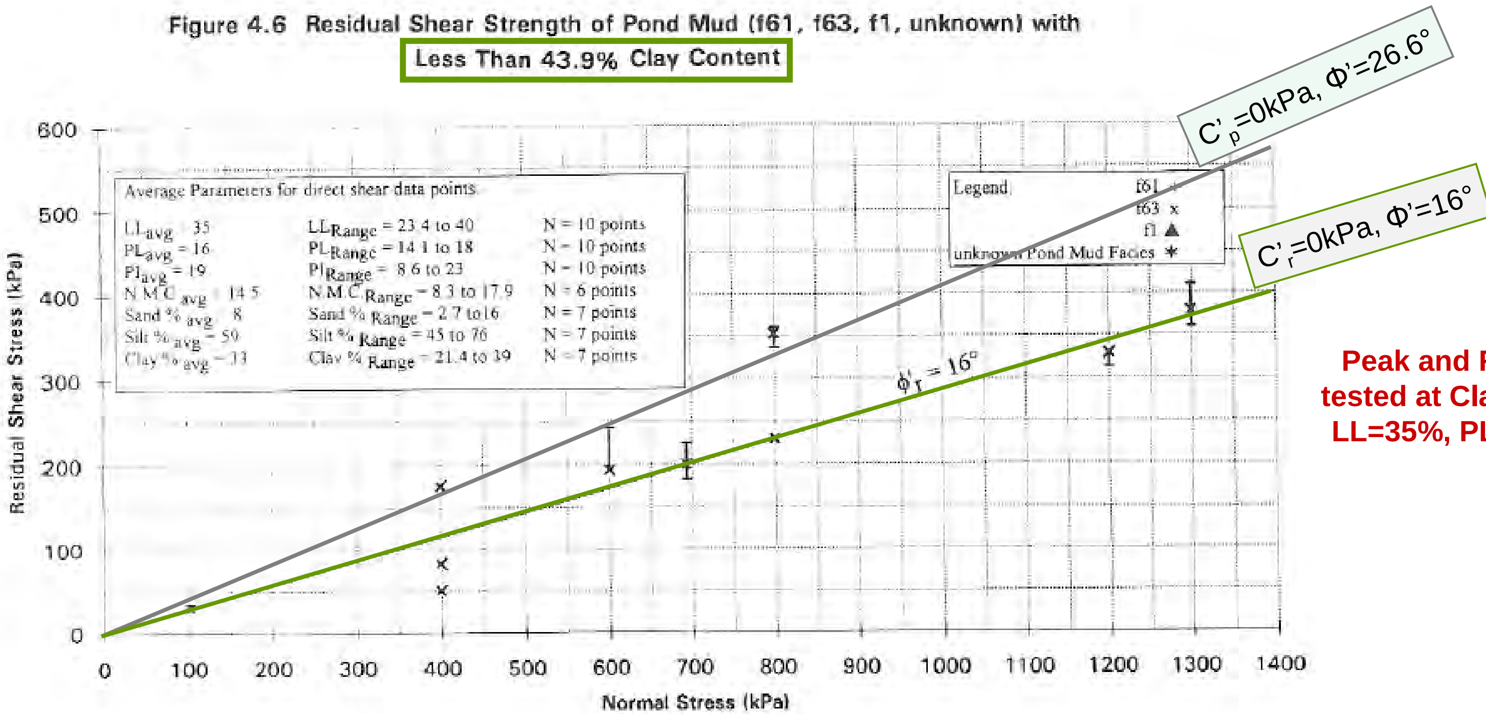
Notes:
1. From combined basal clay Residual Shear Strength Plot (Figure 4.2)
2. Triaxial Tests and direct Shear
3. Refer to Chapter 21.0 for field determined pore pressure ratios (r_u)

N/A = not applicable

4e. 'Hwy 63 Berm' – In-Pit Overburden Tailings Dam, 1992 to 1994 design con't

- Pond Muds Lab Testing – **Low Clay Content**

Figure 4.6 Residual Shear Strength of Pond Mud (f61, f63, f1, unknown) with Less Than 43.9% Clay Content



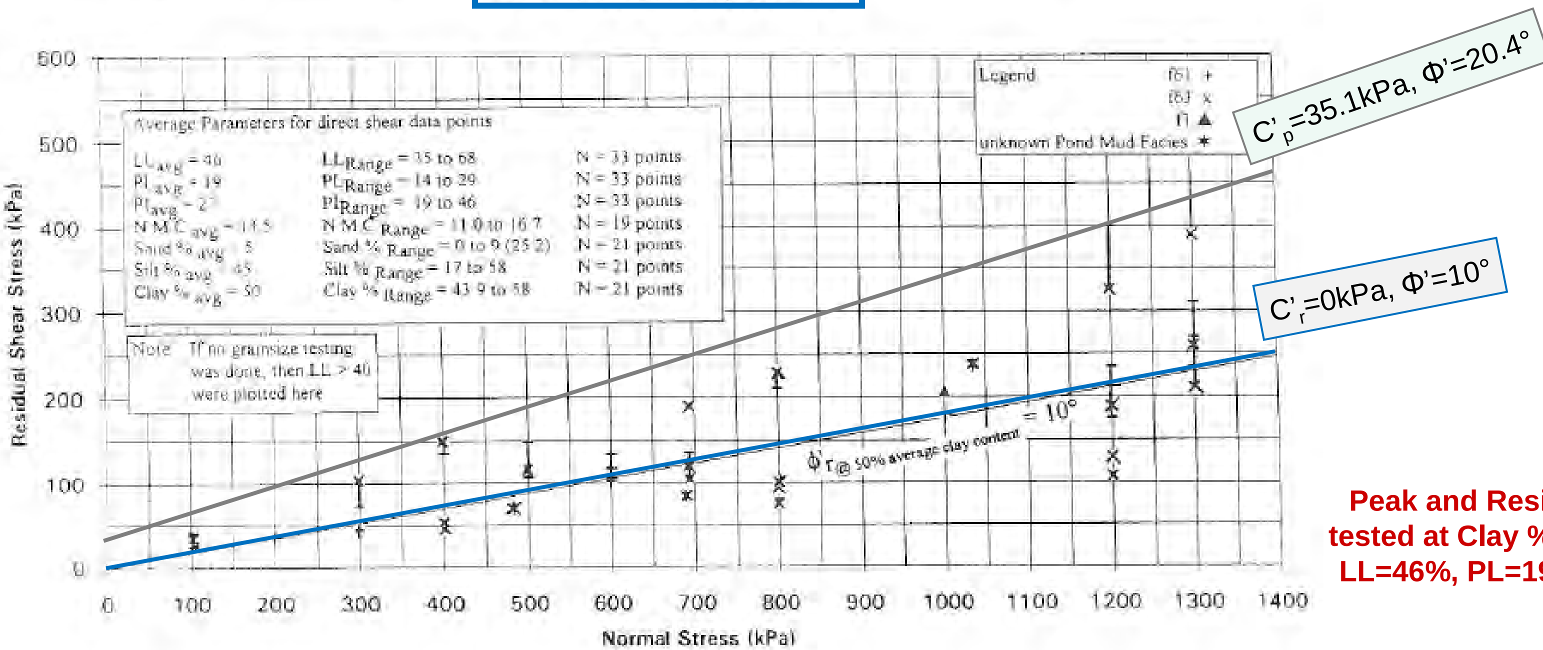
Peak and Residual Strengths tested at Clay % average of 33% LL=35%, PL=16%, NMC=14.5%

Direct Shear - All SCL, Hardy, Thurber, EBA, HBT Agra data - April 23, 1994 - B. Cameron
LL = Liquid Limit, PL = Plastic Limit, PI = Plasticity Index, N.M.C. = Natural Moisture Content, N = Sample Number

4e. 'Hwy 63 Berm' – In-Pit Overburden Tailings Dam, 1992 to 1994 design con't

- Pond Muds Lab Testing – **Medium Clay Content**

Figure 4.5 Residual Shear Strength of Pond Mud (f61, f63, f1, unknown) with 43.9 to 59.9% Clay Content



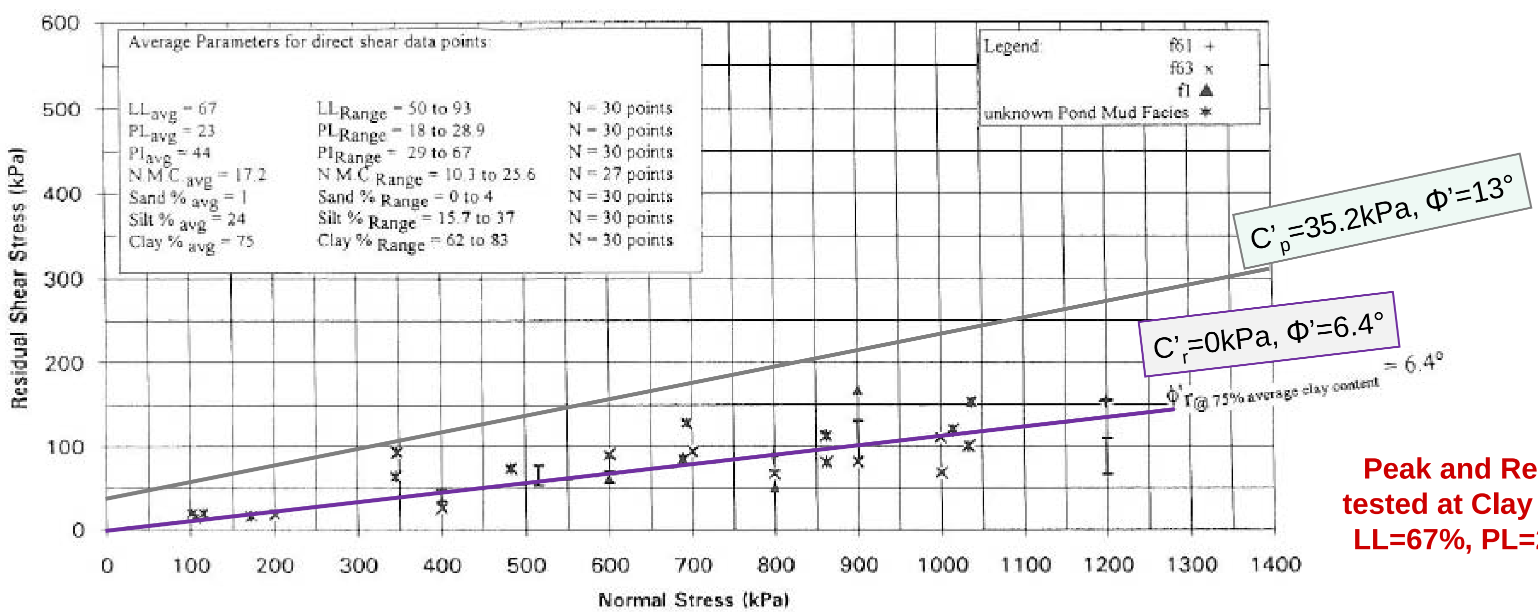
Peak and Residual Strengths tested at Clay % average of 50% LL=46%, PL=19%, NMC=13.5%

Direct Shear - All SCL, Hardy, Thurber, EBA, HBT Agra data - April 23, 1994 - B. Cameron
LL = Liquid Limit, PL = Plastic Limit, PI = Plasticity Index, N.M.C. = Natural Moisture Content, N = Sample Number

4e. 'Hwy 63 Berm' – In-Pit Overburden Tailings Dam, 1992 to 1994 design con't

- Pond Muds Lab Testing – **High Clay Content**

Figure 4.4 Residual Shear Strength of Pond Mud (f61, f63, f1, unknown) with 60 to 83% Clay Content

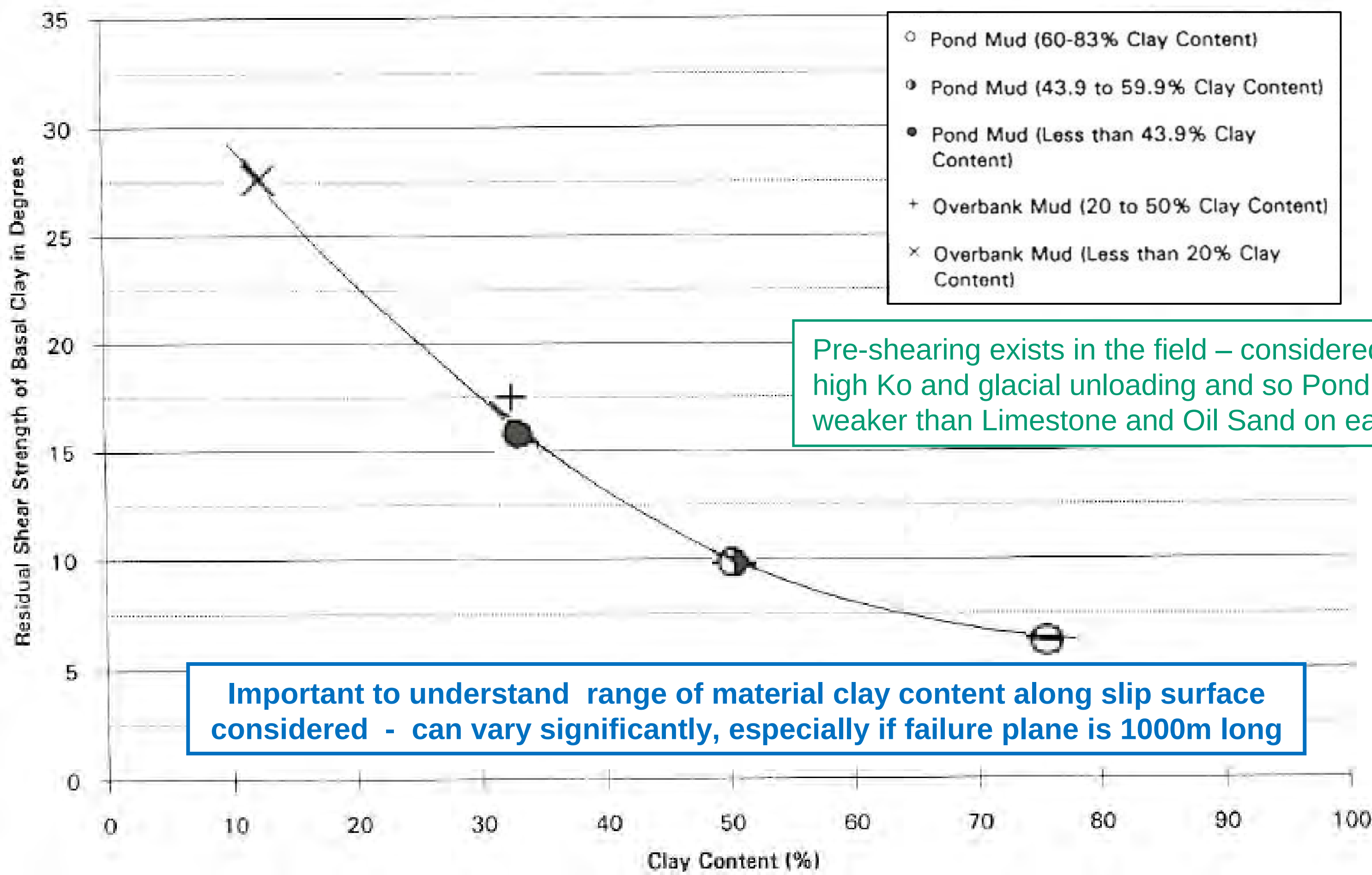


Peak and Residual Strengths tested at Clay % average of 75% LL=67%, PL=23%, NMC=17.2%

Direct Shear - All SCL, Hardy, Thurber, EBA, HBT Agra data - April 23, 1994 - B. Cameron
LL = Liquid Limit, PL = Plastic Limit, PI = Plasticity Index, N.M.C. = Natural Moisture Content, N = Sample Number

4e. 'Hwy 63 Berm' – In-Pit Overburden Tailings Dam, 1992 to 1994 design con't

Figure 4.2 Combined Direct Shear Strength Average Basal Clay Residual Shear Strength Plot for Pond Mud (f61, f63, f1, unknown), Overbank Mud (f3, f70, f71, f77), and Crevasse Splay (f2)



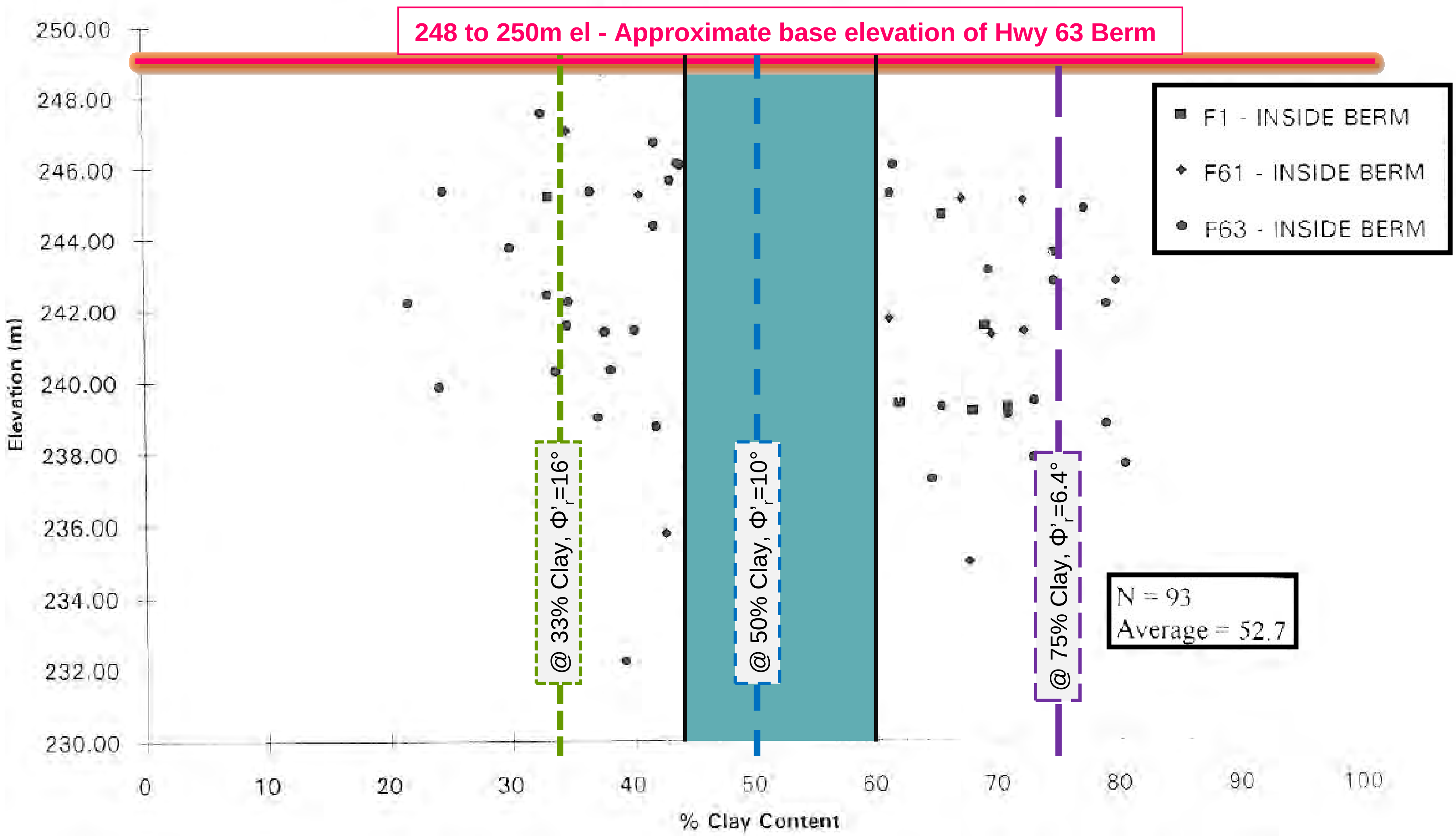
Pre-shearing exists in the field – considered a result of high Ko and glacial unloading and so Pond Mud is weaker than Limestone and Oil Sand on each side

Important to understand range of material clay content along slip surface considered - can vary significantly, especially if failure plane is 1000m long

Hwy 63 Berm–Pond Mud Summary of Residual Strength Tests

4e. 'Hwy 63 Berm' – In-Pit Overburden Tailings Dam, 1992 to 1994 design con't

Figure 4.13 Elevation vs. Clay Content for Pond Muds



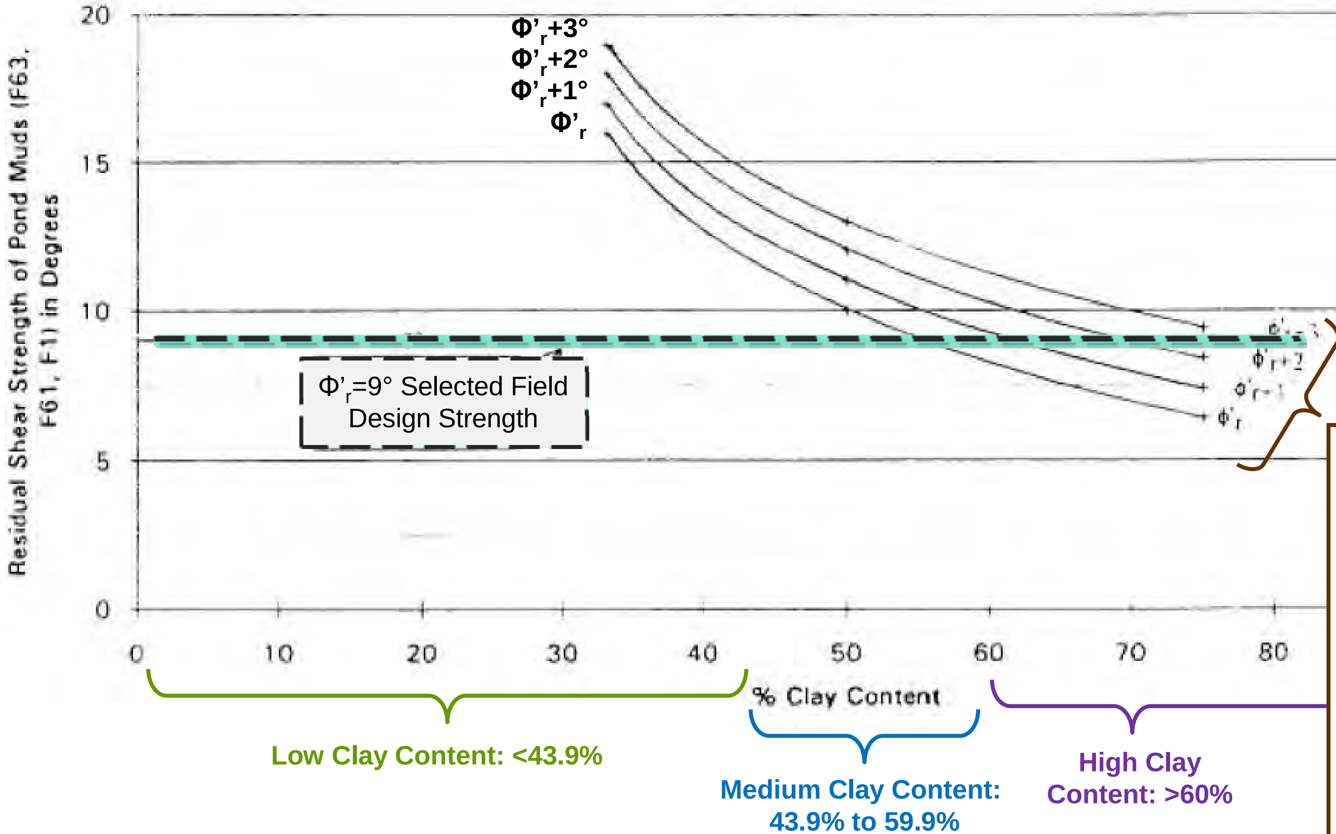
Low Clay Content: <43.9%

Medium Clay Content: 43.9% to 59.9%

High Clay Content: >60%

4e. 'Hwy 63 Berm' – In-Pit Overburden Tailings Dam, 1992 to 1994 design con't

Φ'_r=9° Selected Field Design Strength for Hwy 63 Berm from:
1.Average Strength
2.Lab strength of 60% Clay Content (6.9°) + 2.1° Field Effects = 9° Design Strength



'selected'

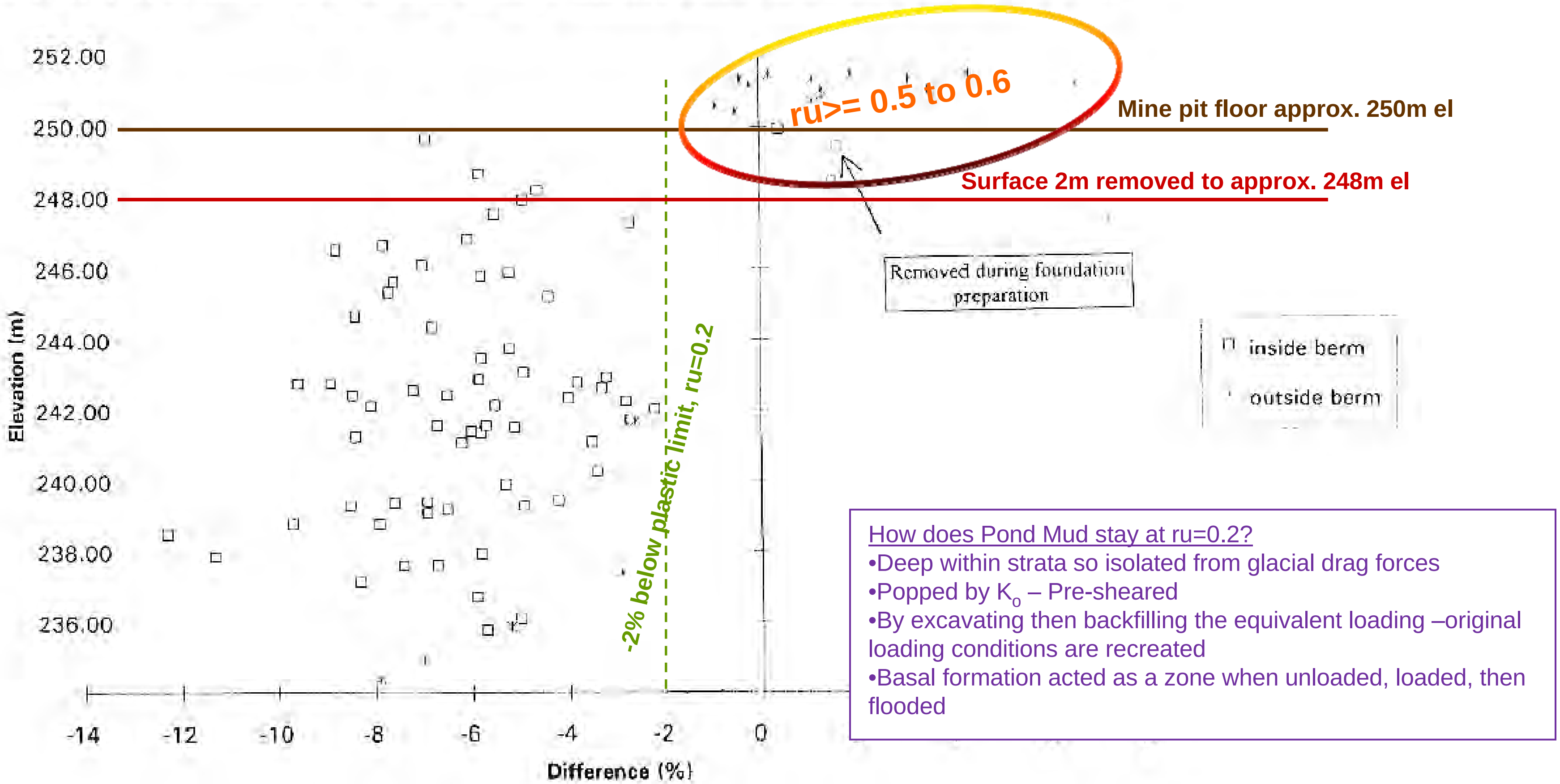
For slip surfaces with average clay contents along the entire slip surface the following is obtained for a 9° field design stength:

Average Clay Content	Lab Residual Strength	Required Field Effects
75%	6.4°	2.6°
70%	6.9°	2.1°
65%	7.5°	1.5°
60%	8°	1°
55%	9°	none
50%	10°	none

+1°, 2°, or 3° for additional equivalent strengths considered to occur in the field:
- Discontinuous distributions of clay and water content along slip surface
- Entire slip surface may not be at residual strength
- Slip surface has to shear through undulating/truncated strong/weak layers
- Only some pre-shearing observed is sub-horizontal, majority of pre-shearing is at high angles
- Horizontal pre-existing shears are unlikely to have clay minerals aligned in the same direction of shearing

4e. 'Hwy 63 Berm' – In-Pit Overburden Tailings Dam, 1992 to 1994 design con't

Figure 4.15 Plot of Elevation vs. Natural Moisture Content minus Plastic Limit (point pairs only)



How does Pond Mud stay at ru=0.2?

- Deep within strata so isolated from glacial drag forces
- Popped by K_o – Pre-sheared
- By excavating then backfilling the equivalent loading –original loading conditions are recreated
- Basal formation acted as a zone when unloaded, loaded, then flooded

Similar to Kc clay that goes to 0.7 ru once 40m of fill is constructed above

4e. ‘Hwy 63 Berm’ – In-Pit Overburden Tailings Dam, 1992 to 1994 design con’t

Lessons Learned:

1 –Pond mud ru is not sensitive to fill height up to original ground elevation and relates to:

- Natural moisture content below plastic limit
- Deep geological environment after deposition and downward hydraulic gradient so pwp low
- Overconsolidation of deep fills
- All in situ basal units (Pond Muds, Basal Clays, Watersands, Crevasse Splay) and in situ reject Oil Sand acted with the same ru
- Fluid loading led to minor ru increase but same fluid also buttressed the Dam

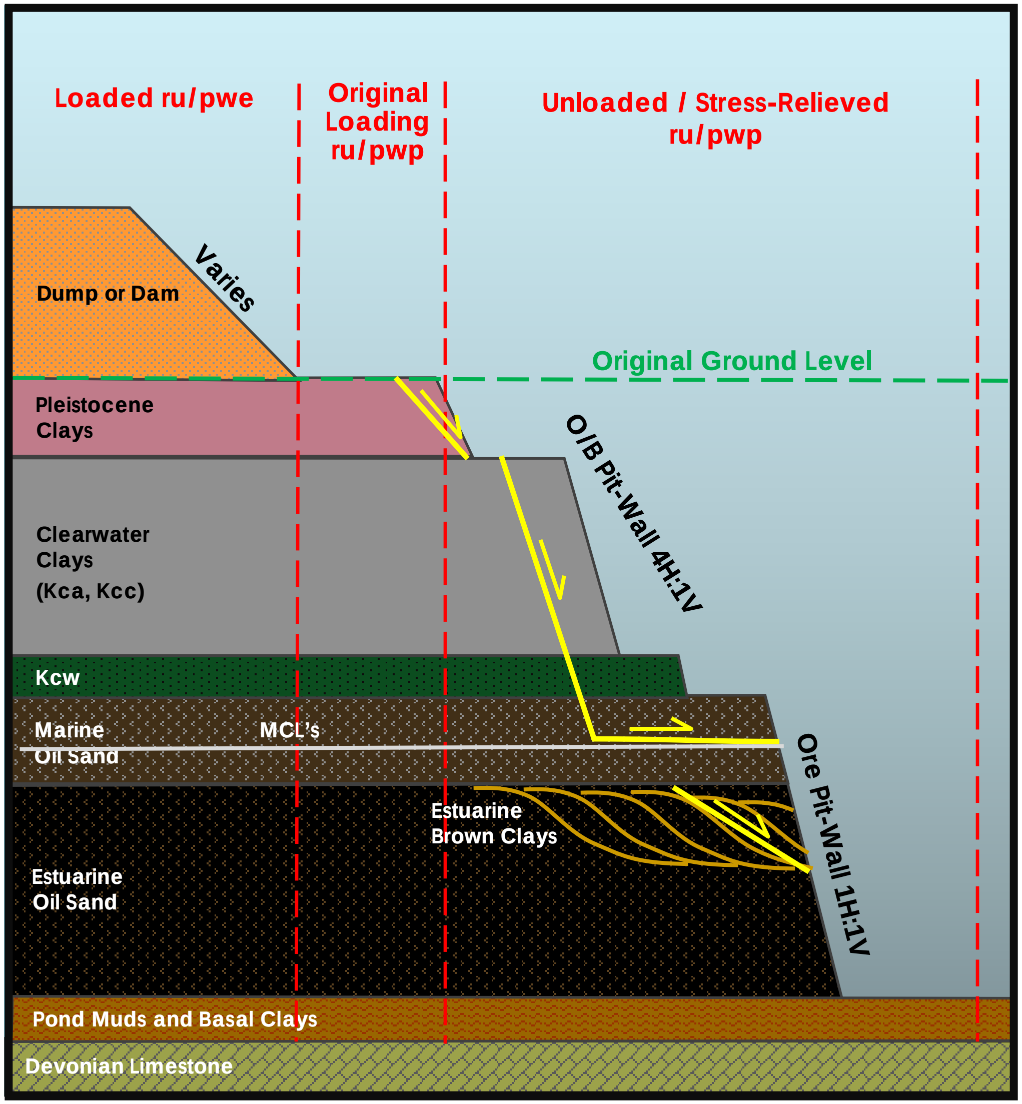
2 –Pond Muds were unloaded during ore mining, reloaded during Dam construction

- Excavated 63m of in situ ground and replaced with fill at overall slopes of 7.3H:1V on pond muds vs out-of-pit dumps, and dams 40m high at 13H:1V to 20H:1V side slopes on Kc clays

3 –Pond Mud sliding shear strength learning’s:

- **9° was correct vs 6° lower bound** (other areas of mine with less clay content we now use up to **10 ° and 11°**, therefore need you own index testing and some direct shears)
- High k_o leading to pre-shearing
- Clay content, liquid limit, and understanding geological depositional environment were important to determine the overall field strength

Excavated Pitwall Case Studies:



4f. 'SWQ West Wall' – Steep Pitwall with No Movement in 2006
4g. 'Block Slide #22' – Advancing Ore Pitwall with Movement in 1987
4h. 'CC1 and CC2' – Final Pitwall with Movement in 2003
4i. 'G-Pit' – Advancing O/B Pitwall with Movement in 2006

- Dump loading can affect pitwall stability and should be designed to be independent of pitwall stability
- Pitwall **benches mined at 72°**
- **Pl Clay bench often falls down to 3H:1V to 4H:1V**
- **Kc Clay bench locally falls down to 1.7H:1V**
- **Marine Clay Layers (MCL's) in Marine Oil Sand** can be problematic units as they can have a large range of sliding shear strength **(pwp design simple piezo level)**
  - **High sliding strength (19.5°)** when deeper and not subject to the effects of glacial drag
  - **Low sliding strength (9°)** when shallow and subjected to glacial drag or other pre-shearing forces
- **Estuarine Clays** in Oil Sand **were strong** but prone to block slides during dragline mining due to the **steep dip** orientation of local, discontinuous clay layers
- **ru always almost reduces due to stress relief or drains down as result of pitwall drainage** from original pore water pressure which helps overall stability

4f. 'SWQ West Wall' – Steep Final Pitwall with No Movement in 2006

Pitwall Characteristics:

▪Overburden removed in 2002, with un-mined ore left in front of the overburden benches (~175m from O/B toe to ore crest left from dragline mining)

▪Marine Oil Sand was daylighted and had Marine Oil Sand Clay layers present near top of unit (no clay layers present in deeper Marine Oil Sand)

▪No Pond Mud or paleosol present in area

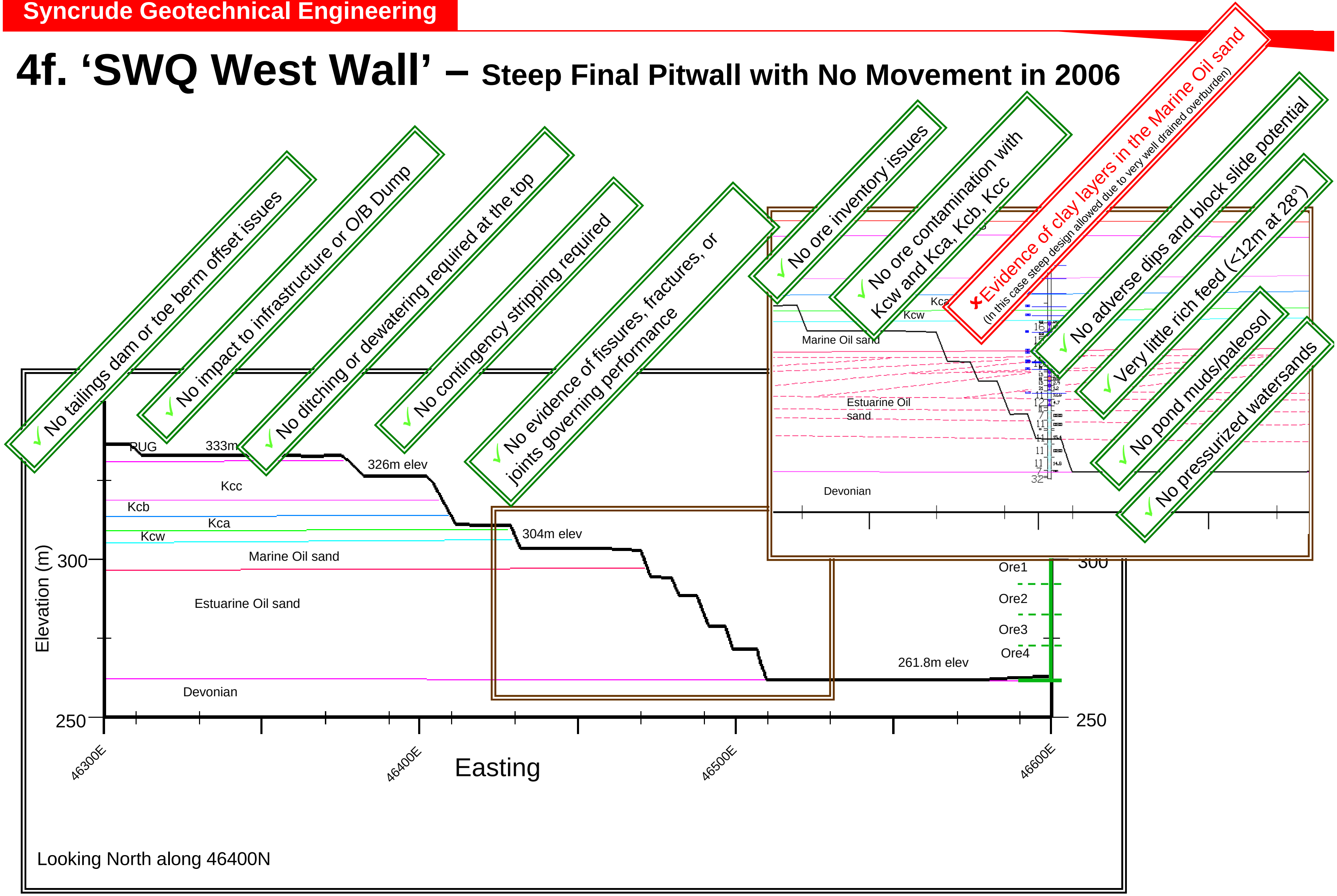
▪Overburden "staircase" stepping design and 4 year drainage period increased stability of the pitwall

▪Final dragline mining completed in March 2006

▪Mining of ore (including bottom Marine Oil Sand) was completed Feb. 2007, giving the ore an additional year of drainage (and gas exsolution) during mining due to the slower mining strategy (smaller shovels/smaller benches) applied to the SWQ

▪Aggressive advancing dig-limits for the ore since no evidence of instability was observed and because this was a low-risk area for testing a steeper design

4f. 'SWQ West Wall' – Steep Final Pitwall with No Movement in 2006



4g. 'Block Slide #22' – Advancing Ore Pitwall with Movement in 1987

Pitwall/Movement Characteristics:

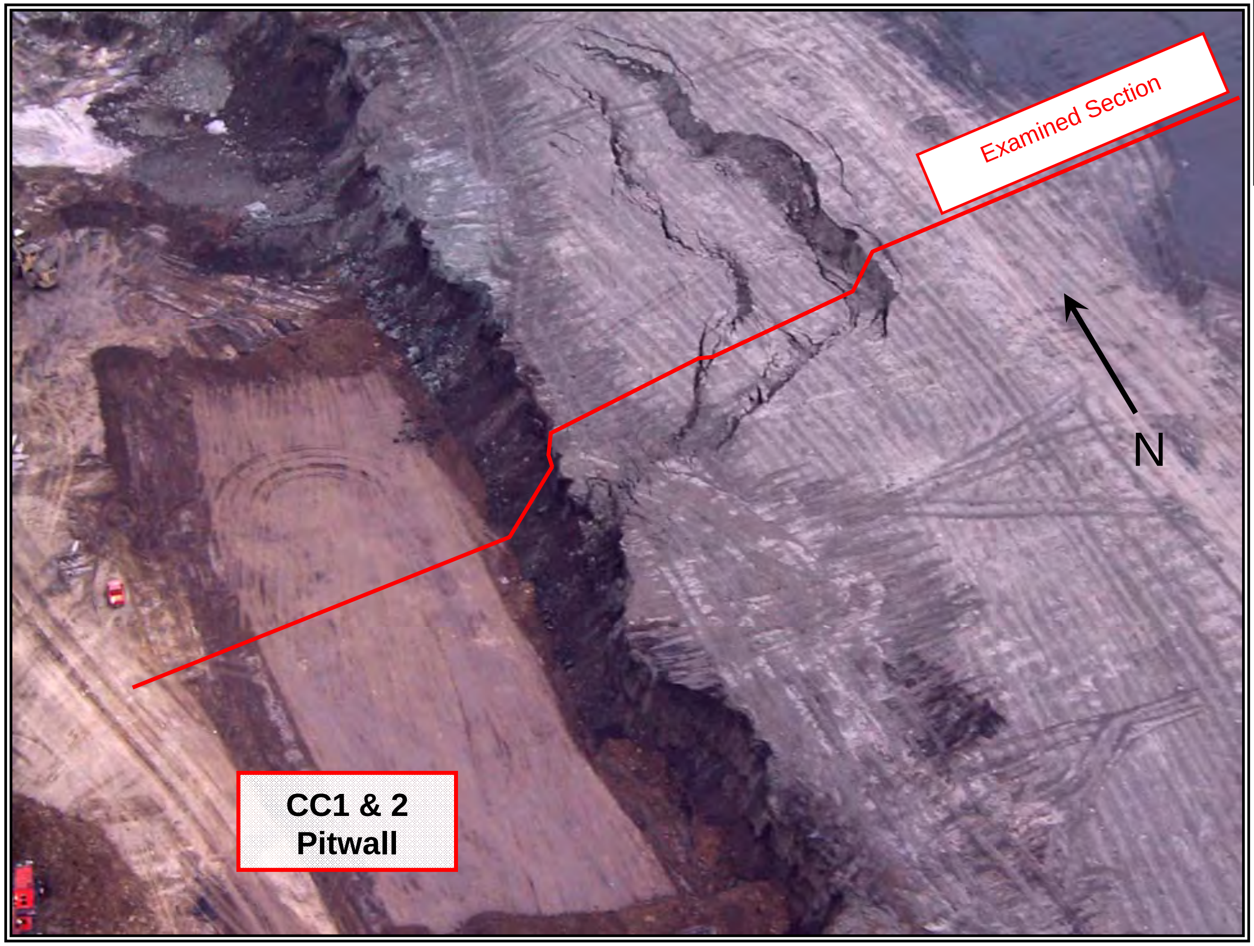
▪Estuarine Clays in Oil Sand were strong but prone to block slides during dragline mining due to the steep dip orientation s of local, discontinuous clay layers as clay drapes over old sand bars and the like,

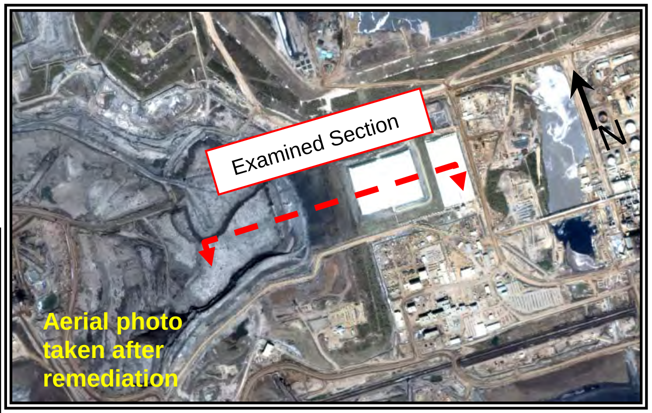
▪Block slide occurred during mining of pitwall

▪Estuarine dip angle ranges from 11° to 15° to steeper angles were problematic

▪This represents a steep geological environment problem for an otherwise strong clay

4h. ‘CC1 and CC2’ –
Final Pitwall with Movement in 2003





Pitwall/Movement Characteristics:

- The final east pitwall in North Mine in the area of the old Coke Cell #1 and #2 moved after some additional ore mining had occurred
- Slip surface extended through a recently constructed earth buttress, through the in-situ Marine Oil Sand below, and along a Marine Clay Layer within the Marine Oil Sand
- The dimensions of the displaced soil were **200m in length by 19m in height by 67m in width from scarp top crack to daylighting toe** at Marine Oil Sand Clay layer

4h. 'CC1 and CC2' – Final Pitwall with Movement in 2003, con't

Lab Analysis on F16 Marine Oil sand Sliding Clay Layers from CC 1&2

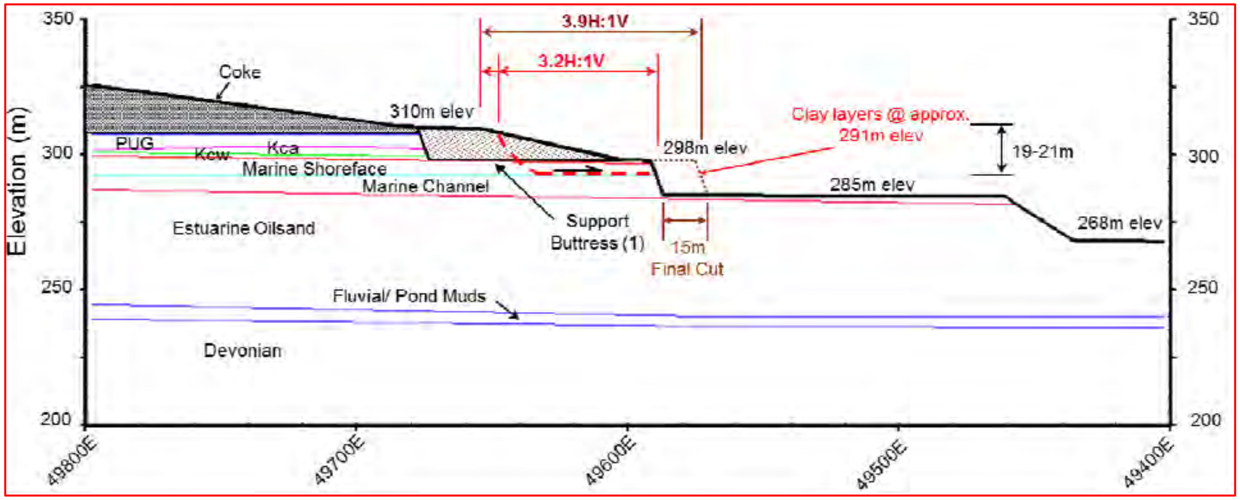
Note: Relatively low activity for Φr = 9° due to presence of degraded illite and intercalated layers of smectite within the illite.

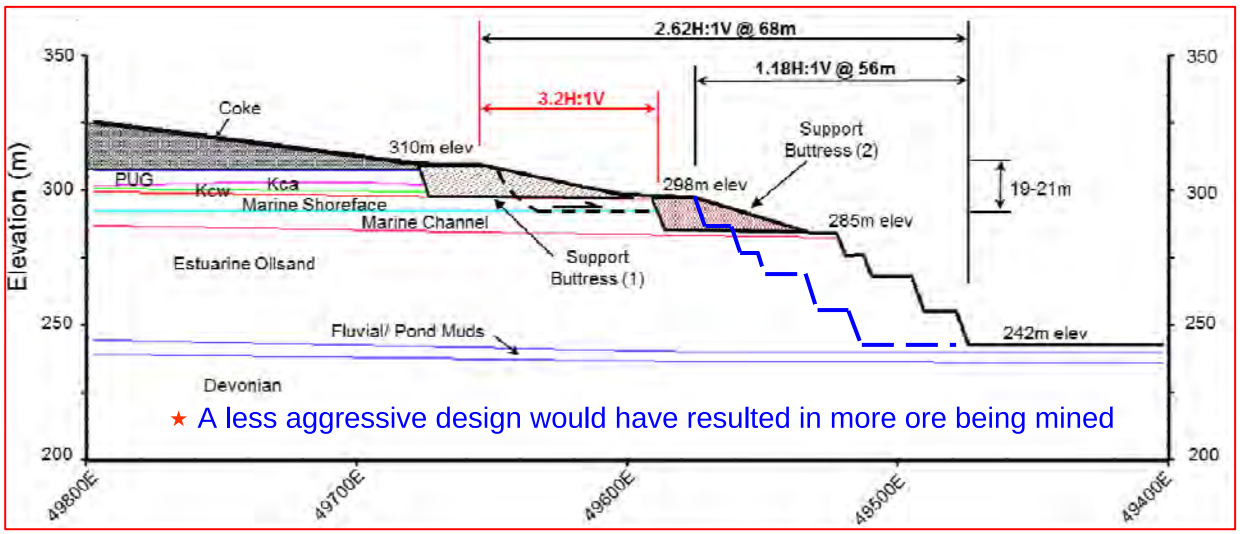
SAMPLE 1		SAMPLE 2
LL = 57		LL = 59
PL = 23	(NMC Range 1&2 = 20.9% to 28.5%)	PL = 24
PI = 34		PI = 35
Clay = 52		Clay = 57
Silt = 39		Silt = 36
Sand = 9		Sand = 7
Activity = 0.67		Activity = 0.63

Marine Clay Layer Sliding Shear Strength (From Back-Analysis)	Marine Clay Layer & Marine Oil sand Piezo	Marine Oil sand Cross-bedded Strength	Buttress Shear Strength	Buttress Piezo Condition Assumed	Best Back Analyzed Factor of Safety	Look of Failure Geometry as Compared to Failure in the Field	Comments
Φr = 9°*	304m draining to 298m elev at bench	c' = 30kPa & Φ = 54°	Φ = 33°	ru = 0.25	1.00	Similar	Good

* Distance from top of Overburden to sliding clay layers is 19-21m

4h. 'CC1 and CC2' – Final Pitwall with Movement in 2003, con't





<u>Lab Analysis on F16 Marine Oil sand Sliding Clay Layers from CC 1&2</u>

<u>SAMPLE 1</u>	<u>SAMPLE 2</u>
LL = 57	**LL = 59**
PL = 23	PL = 24
PI = 34	PI = 35
Clay = 52	**Clay = 57**
Silt = 39	Silt = 36
Sand = 9	Sand = 7
Activity = 0.67	Activity = 0.63

NMC Range for samples 1 & 2 = 20.9% to 28.5%

4h. 'CC1 and CC2' –

Final Pitwall with Movement in 2003, con't,

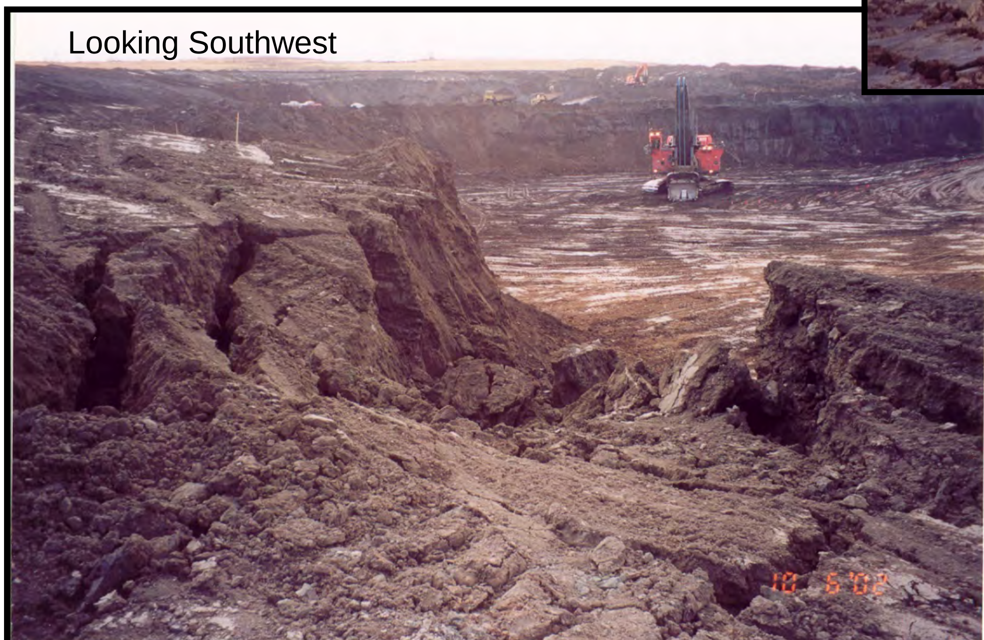
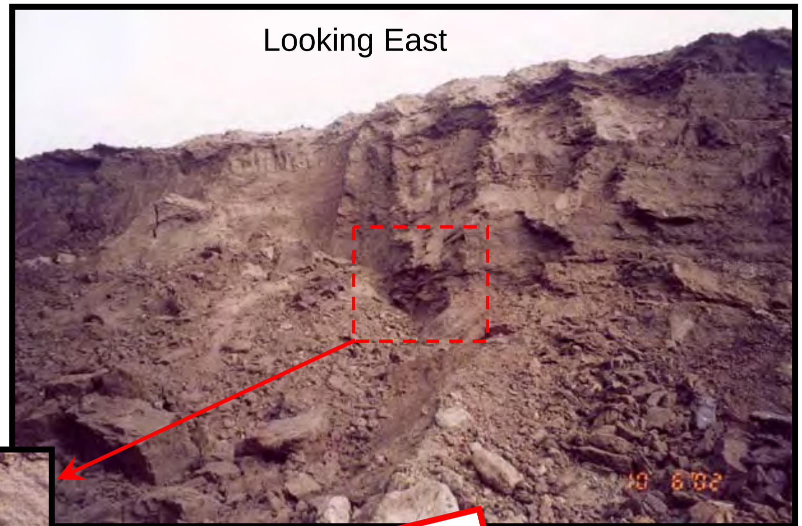


Photo looking up radial shear at head scarp

Photo looking down through radial shear

4h. 'CC1 and CC2' – Final Pitwall with Movement in 2003, con't



- Translational slide plane is made up of two weak Marine Oil sand clay layers, each approximately 35mm thick

- Photos show these layers being squeezed out by the weight of the buttress above (Buttress not shown in photos above)

4h. 'CC1 and CC2' – Final Pitwall with Movement in 2003, con't

Lessons Learned:

1 -Limited Kca and no Kcb and no Kcc

2 -This site had loading from CC 1&2 and then the buttress, and it is close to the Old Beaver Creek Escarpment

3 -Marine Clay layers in the O/B analyzed pre-sheared (at Residual Strength) for this maximum 21m O/B height, from topography to Marine Oil Sand Clay Layer(s)

4 -The failure resulted in pitwall re-designs that reduced the amount of ore available at that time and increased costs of operating to replace the bonus ore lost. Additional bonus ore loss occurred, as compared to having originally just left additional in-situ Oil Sand in-place. This was due to the need for earthen buttresses which, in this case, required more space to support the slope as they had to be built up from below the failure, This steep wall design was part of trying to obtain more ore than required on what was once a drier slope.

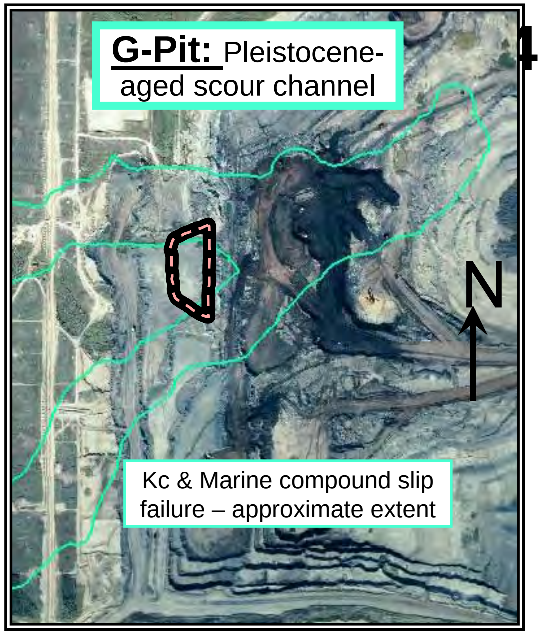
5 -The failure showed that for the "actual" field conditions occurring (wetter), the pitwall could not be cut this steeply to obtain all the additional bonus ore hoped for.

6 -This failure also indicates that either additional overburden should have been removed (not practical here) or additional Oil Sand left un-mined to shallow the slope to avoid failure

Marine Clay Layer Sliding Shear Strength (From Back-Analysis)	Marine Clay Layer & Marine Oil sand Piezo	Marine Oil sand Cross-bedded Strength	Buttress Shear Strength	Buttress Piezo Condition Assumed	Best Back Analyzed Factor of Safety	Look of Failure Geometry as Compared to Failure in the Field	Comments
Φr = 9°*	304m draining to 298m elev at bench	c' = 30kPa & Φ = 54°	Φ = 33°	ru = 0.25	1.00	Similar	Good

* Distance from top of Overburden to sliding clay layers is 19-21m

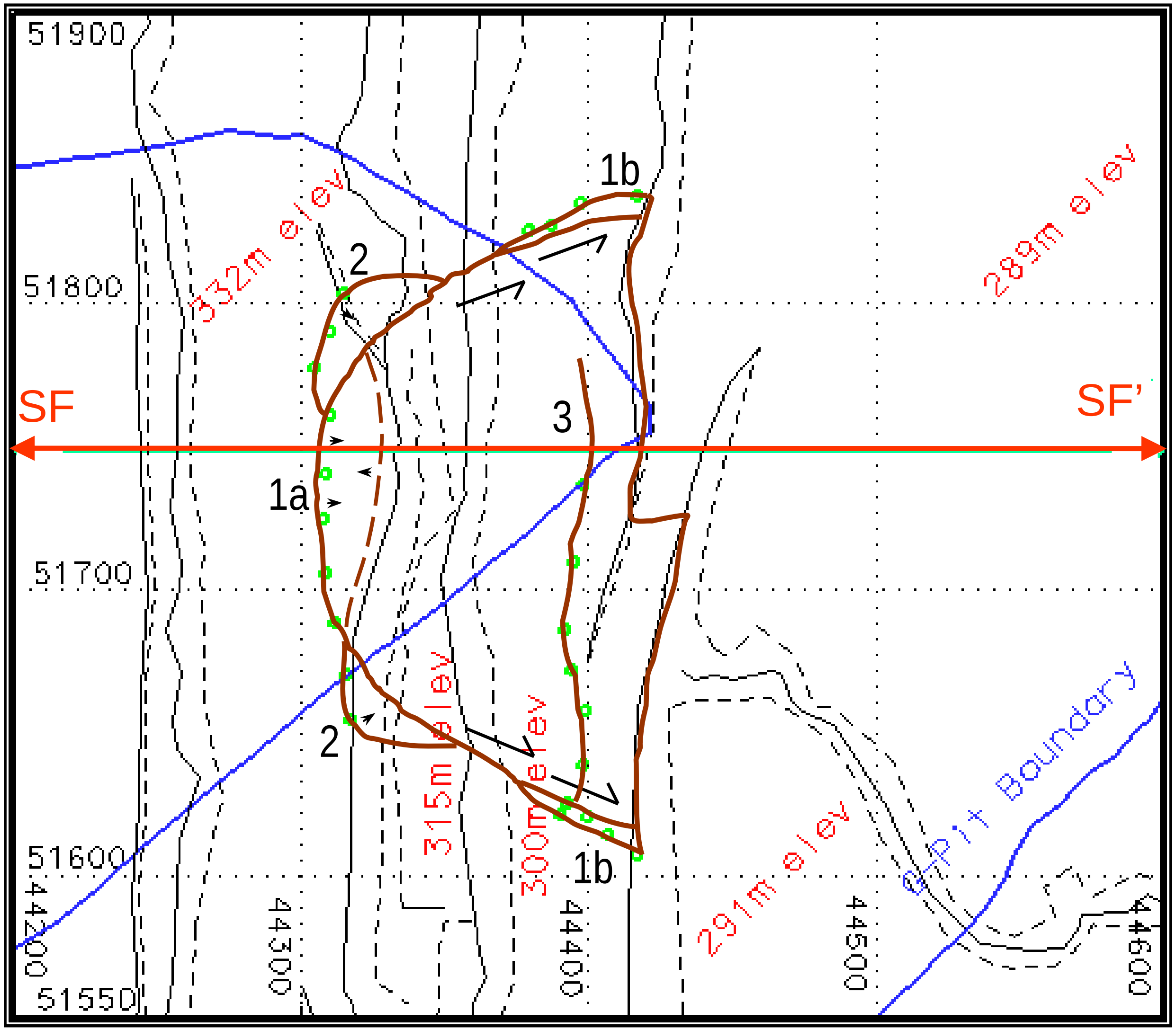
4i. 'G-Pit' – Advancing O/B Pitwall with Movement in 2006



Pitwall/Movement Characteristics:

- The west advancing O/B pitwall developed a slip surface on Marine Clay Layers (white dots on graben are 5 gallon pails)
- Pitwall movement in G-Pit area, but not influenced by G-Pit channel/gravels
- The dimensions of the displaced soil were **240m in length by 33m in height by 120m in width from scarp top crack to daylighting toe** at Marine Oil Sand Clay Layer

4i. ‘G-Pit’ – Advancing O/B Pitwall with Movement in 2006, con’t



June 8-14

1a Cracking occurs, Graben forms

June 8-14

1b Radial shearing & lipping occur

June 8-14

2 Flank failures develop

June 15-16

3 Toe heaves after translational slip progresses to rotational slump

4i. 'G-Pit' – Advancing O/B Pitwall with Movement in 2006, con't

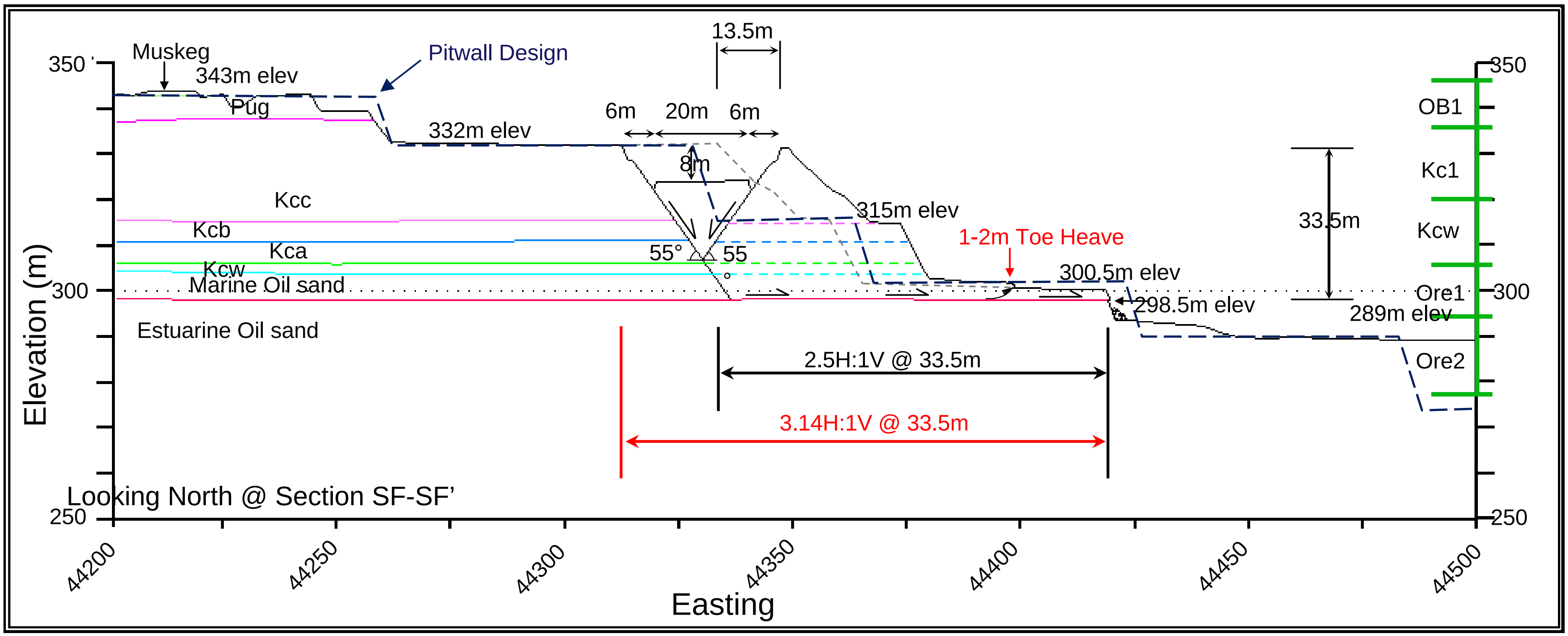
June 8-14: 1a Cracking occurs, Graben forms

June 8-14: 1b Radial shearing & lipping occur

June 8-14: 2 Flank failures develop

June 15-16: 3 Toe heaves after translational slip progresses to rotational slump

- Under-mining of Kc1 bench by an average of 7.5m added substantial driver
- Over-mining of Kcw bench by an average of 4.5m reduced toe support
- Daylighting of Marine Oil sand - Estuarine Oil sand geologic contact combined with the over digging Ore1 bench by an average of 5m



4i. 'G-Pit' – Advancing O/B Pitwall with Movement in 2006, con't

Lab Testing Results for Near Shoreface and Channel Marine Clay Layers - **Pit wall design had used fully softened peak strength at Φ'= 19.5 degrees** because Marine was more than 40m below ground so no expected glacially pre-shearing – back analyses of failed and non-failed slopes confirmed this was correct.

RESULTS:

Liquid Limit = 63.3%

Plastic Limit = 22.8%

Plasticity Index = 41%

Clay = 67.5%

Silt = 29.1%

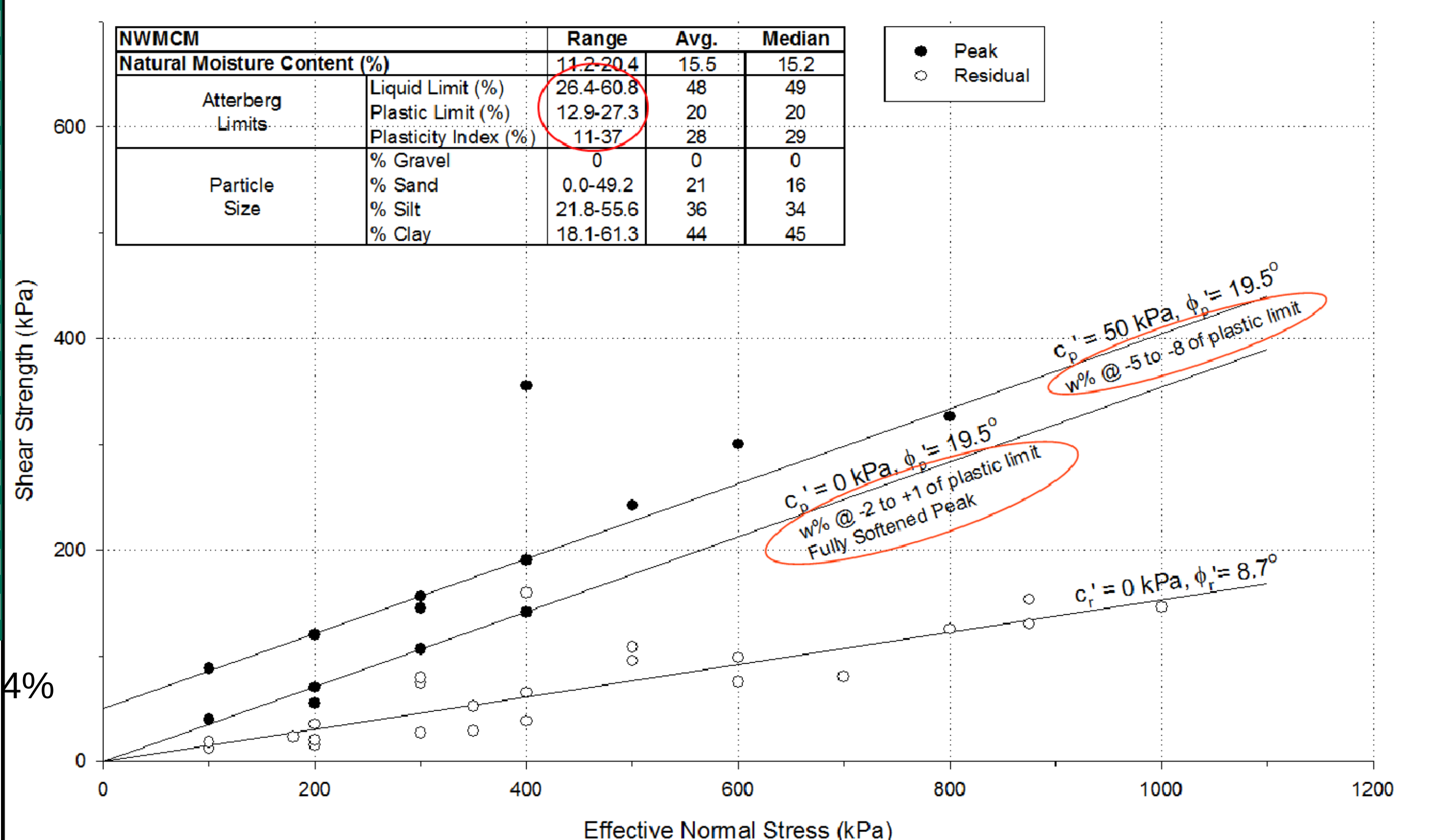
Sand = 3.4%

Activity = 0.61%

NMC range = 11.2 to 20.4%

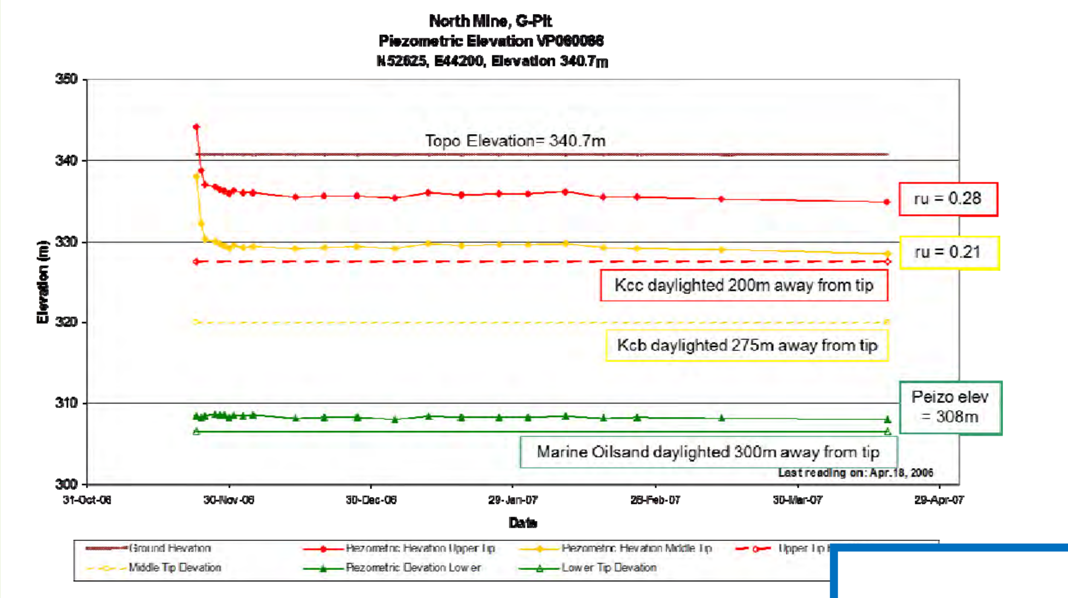
C.2g In Situ Northwest Marine Channel Clays in Marine Oil Sand Direct Shear Strength

(From: Northwest Marine Channel Investigation, G. McKenna, 1992)



NWMCM			Range	Avg.	Median
Natural Moisture Content (%)			11.2-20.4	15.5	15.2
Atterberg Limits	Liquid Limit (%)		26.4-60.8	48	49
	Plastic Limit (%)		12.9-27.3	20	20
	Plasticity Index (%)		11-37	28	29
Particle Size	% Gravel		0	0	0
	% Sand		0.0-49.2	21	16
	% Silt		21.8-55.6	36	34
	% Clay		18.1-61.3	44	45

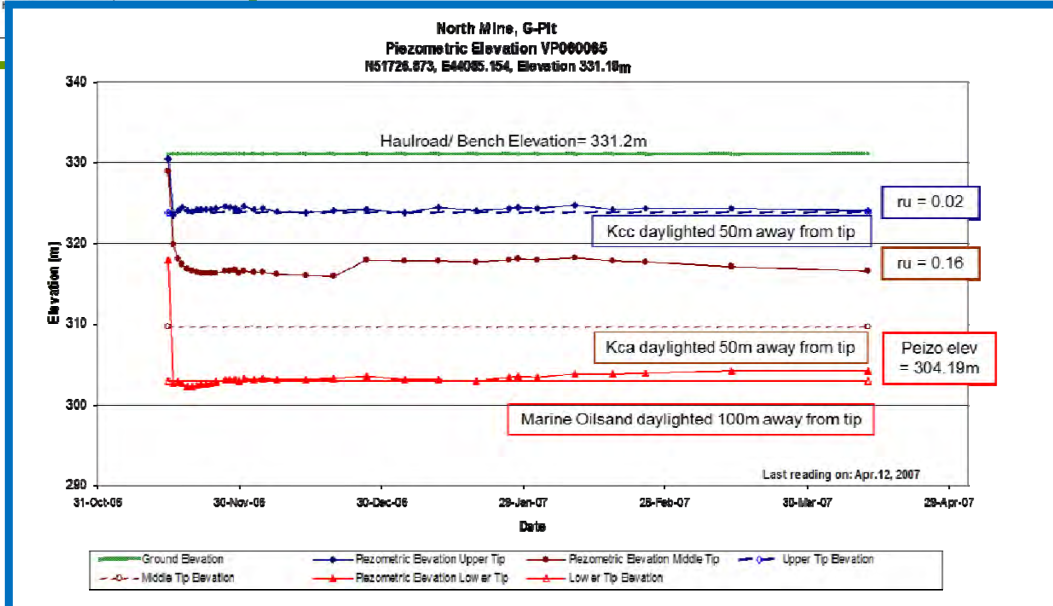
4i. 'G-Pit' – Advancing O/B Pitwall with Movement in 2006, con't



ru values in Kcc=0.28, Kcb=0.21 and Marine Oil Sand units at 308m piezo elevation for instrument installed **200 to 300m away from daylighted highwall**

- Stress relief in the clays causes the ru to reduce from normal original ground ru's, approximately 0.45
- Marine Oil Sand maintains close to original piezometric level with modest ru response to slight drainage

ru values in Kcc=0.02, Kca=0.16 and Marine Oil Sand units at 304m elevation for instrument installed **50 to 100m away from daylighted highwall**



Syncrude
Securing Canada's Energy Future

4i. 'G-Pit' – Advancing O/B Pitwall with Movement in 2006, con't

Syncrude Canada Ltd. - Fort McMurray, AB

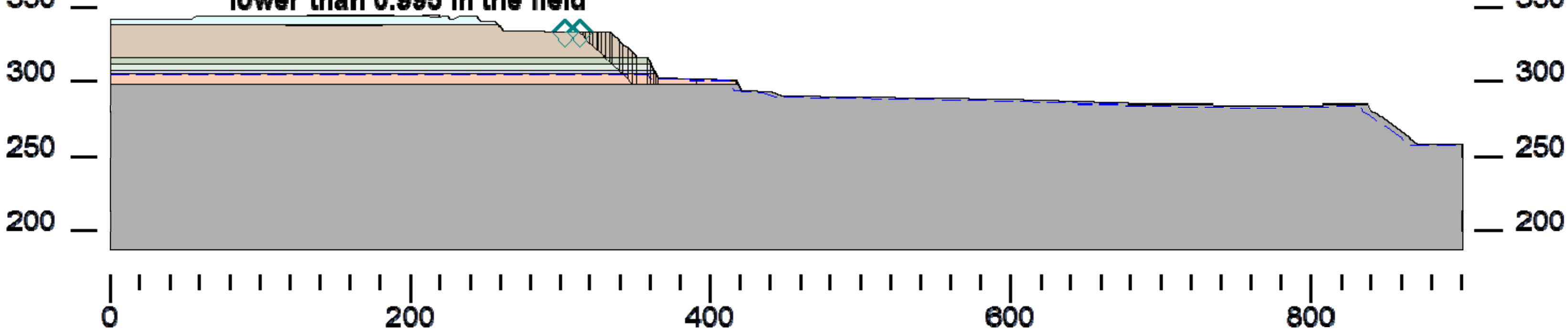
	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.	Ru
Muskeg	15.7	0	38	0	0
Pl Clay	20.2	0	25	0	0
G-Pit	21.75	0	33	0	0
Kcc	19.6	25	20	0	0 to .02
Kcb	19.6	25	20	0	0 to .21
Kca	19.6	25	20	0	0 to .16
Kcw	20.2	20	33	1	0
Marine	21.2	30	54	1	0
Marine Sliding	21.2	0	19.5	1	0
Estuarine	(Infinitely Strong)				

FS=0.995

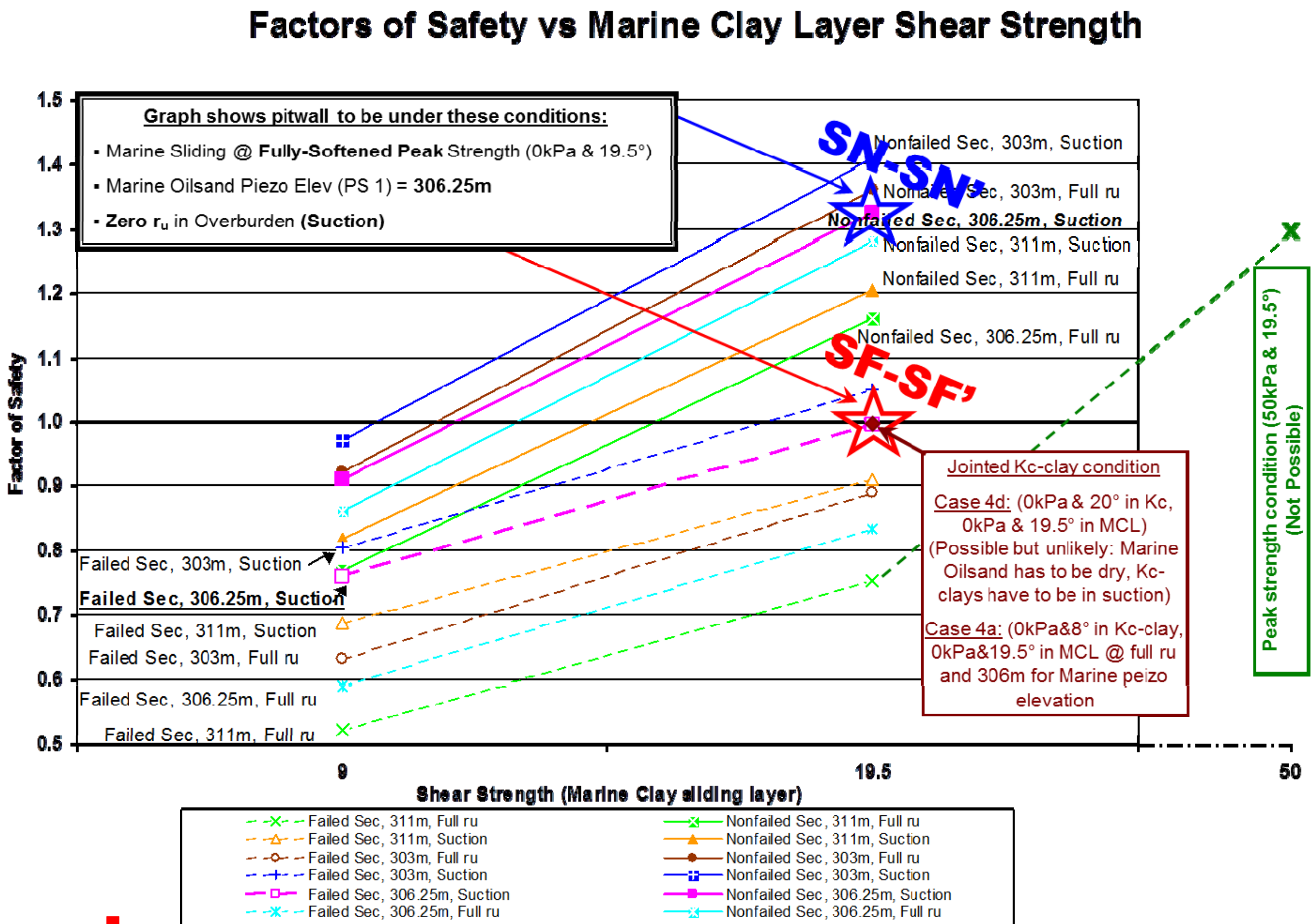
1.05 at back of actual graben

- Marine Sliding @ **Fully-Softened Peak** Strength (0kPa & 19.5°)
- Marine Oilsand Piezo Elev (PS 1) = **303m to 306m**
- r_u values for Kc-clays range from **zero** to values gained from instrumentation

- **Shows that Marine Sliding layer was at fully-softened peak strength (FS seems correct given that the wall failed shortly after being mined)**
- **Though this does not match the look, the 1.05 FS does match the look in 2D and 3D and it is suspected that there is a defect at the 1.05 location (partial joint) that makes the 1.05 lower than 0.995 in the field**



4i. 'G-Pit' – Advancing O/B Pitwall with Movement in 2006, con't
(Non-failed slope is SN-SN', and Failed Slope is SF-SF')



4i. 'G-Pit' – Advancing O/B Pitwall with Movement in 2006, con't

Lessons Learned

1 **Marine Clay Layer** in Marine Oil Sand in G-Pit area at **fully-softened peak strength c'= 0kPa & Φ'=19.5°**

2 Once failed **fully-softened peak strength goes to residual strength**

3 SF-SF' the failed section would not have failed if mined exactly to design dig limits, though FS would still have only been 1.19

4 Back scarp of failure was at 3.14H:1V (post-failure) with some cracking behind (3.2H:1V) which relates to residual strength. **Had the OB1 bench (the mining bench above) been within 3.2H:1V of this, it would have resulted in a substantially larger failure**.

5 Some areas to the north may be at full peak strength

6 Large failures may also occur if wetter, fully softened, areas are encountered during mining

7 In North Mine, Operations has the potential to encounter wide range of pore water pressure conditions

8 Pitwall between channel limbs can be analyzed to be jointed though field observations do not match GSlope modeling and photos do not show any continuous joints

9 Joints minor cross-bedded weaknesses in geology or water content, if present or minor, will control failure location so still did not input backscarp let program do that.

10 Kc-clays were analyzed at cross-bedded shear strengths and do not seem to be a factor in this failure mode – **Kc clays had low measured ru's by pit wall**

11 Problem 1 ---- Under-mined Kc1 bench to allow adequate room for haul road width

12 Problem 2 ---- Over-mined Kcw bench to allow adequate room for 2nd running surface

5. Retaining Wall/MSE Wall Construction Considerations for Marine Clays, Re-design

Case Studies:
5a. 2a/2c Conveyor Retaining Wall Re-Design, 1988
5b. MSE Wall with Shear Key for MLMR, 2011


2a/2c Conveyor Retaining Wall just prior to demolition in 2012

5a. 2a/2c Conveyor Retaining Wall Re-Design, 1988, con't

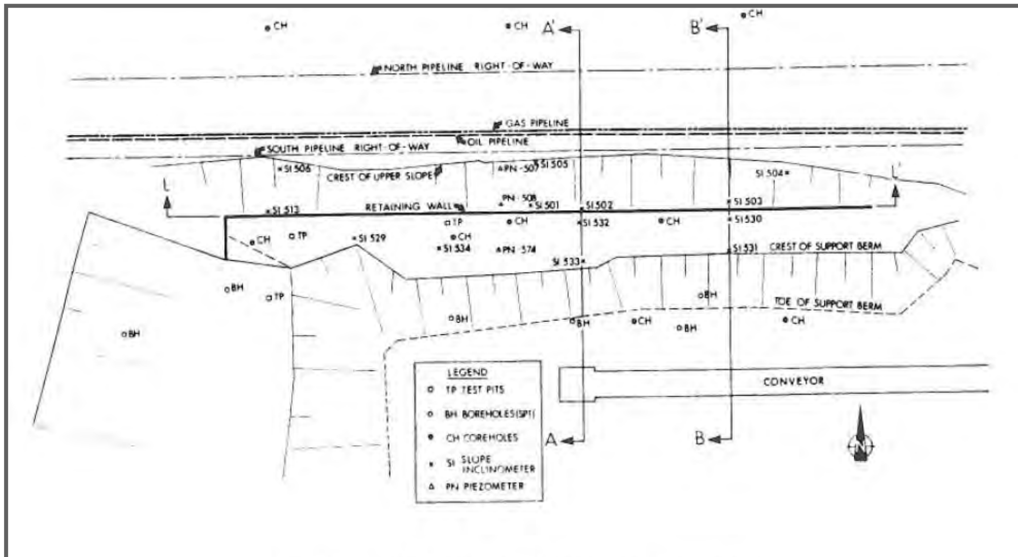


Fig. 1 Site Plan and Instrument Locations

Approach for re-design and monitoring of a retaining wall installed combined *in situ* / O/B fill areas with Marine Clay Layers

Design and monitoring critical since:

•Oil and Gas Pipelines existed near crest of slope

•Ore conveyors existed near toe of slope

•Marine Clay Layers existed near surface

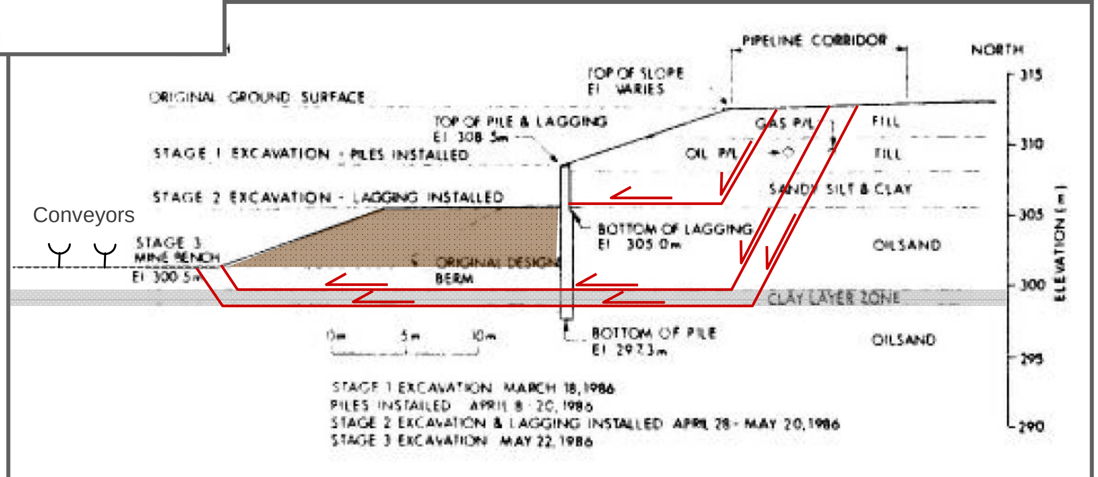


Fig. 2 Typical Cross-Section Through the Retaining Wall and Overburden Slope

5a. 2a/2c Conveyor Retaining Wall Re-Design, 1988, con't

Slope Inclinometers (SI's) and Pneumatic Piezometers (PN's) were installed in and around the soldier-pile wall to read soil movements and pore water pressure responses

•SI's read a maximum velocity of 0.3mm/day

•Majority of movement is suspected to be stress-relief from the excavation

•No significant piezometric elevation or ru changes occurred during or after construction

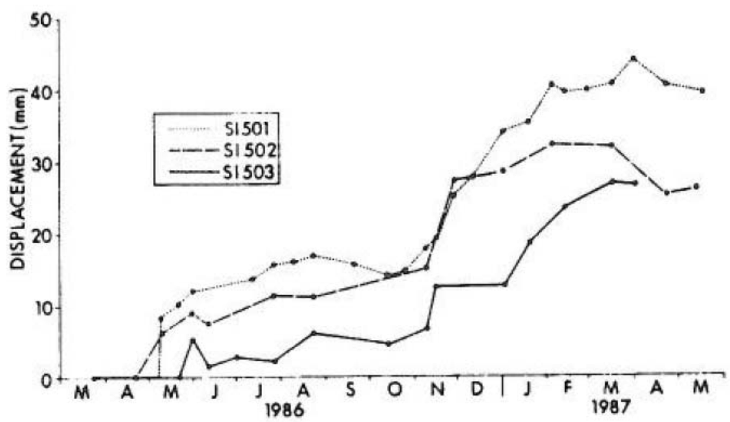


Fig. 11 Cumulative Displacement 1.2 Meters Below the Top of the Retaining Wall Measured by Slope Inclinometers

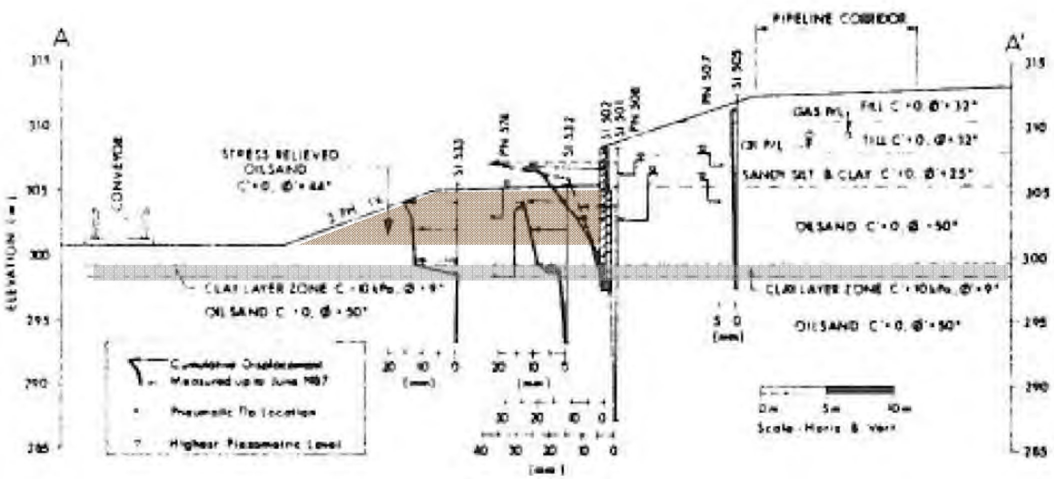


Fig. 8 Cross-Section A-A' Instrumentation Results

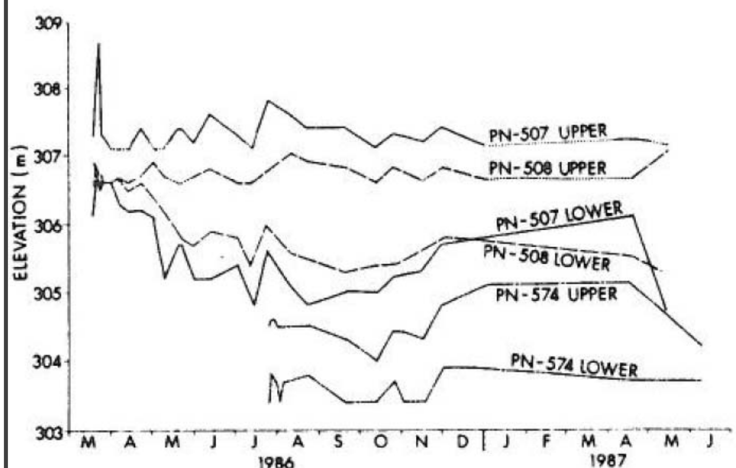


Fig. 14 Piezometric Levels

5a. 2a/2c Conveyor Retaining Wall Re-Design, 1988, con't

Evidence of pre-shearing in some clay samples required that residual shear strength parameters be used in the analysis due to near shore marine sand content and frequent sand filled burrows. Cohesion on residual due to frequents sand in-filled burrows through the clay and the cutting action of the sand grains into clay along a shear plane.

Establishing an instrumentation program and remedial stabilization plan allowed for a less conservative design strength to be used. Did not use zero cohesion in the design.

Three (3) design strength in Marine Clays Layers in Marine Oil Sand <u>for the Case Studies Presented</u>. Have used:

c'=0kPa, Φ'=9°, r_u= piezo level (when pre-sheared)

c'=0kPa, Φ'=19.5°, r_u= piezo level (not pre-sheared)

c'=10 kPa, Φ'=9°, r_u= piezo level(sandier, lower LL)

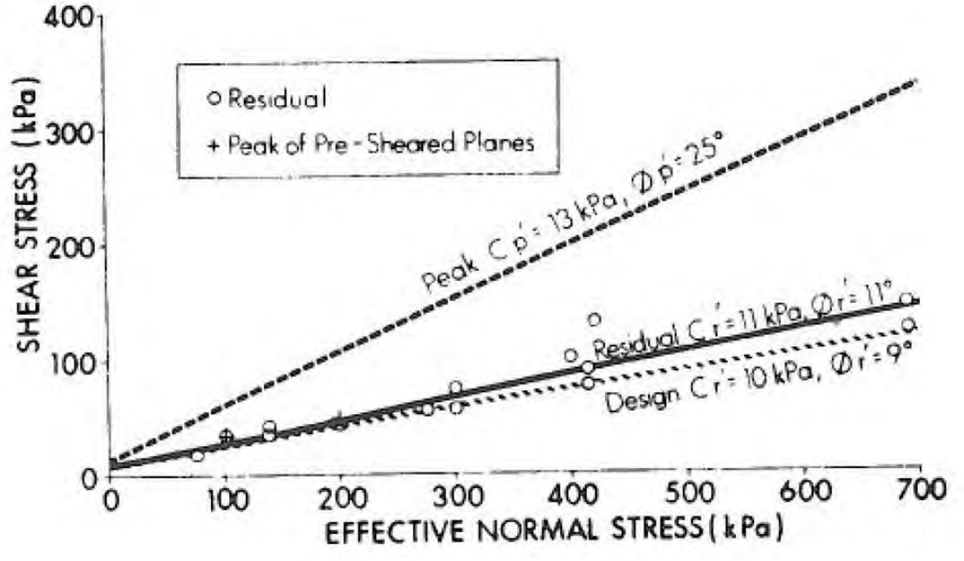


Fig. 6 Clay Layer Shear Strength Parameters

5b. MSE Wall with Shear Key for MLMR, 2011 design

Building the correct geologic model is important

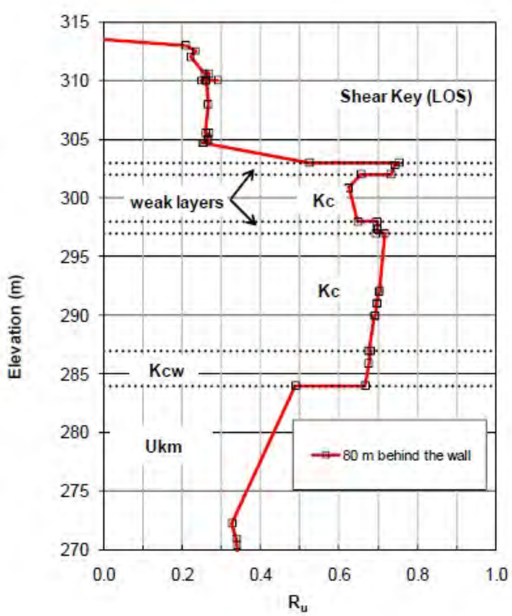
•Weak layers observed in core samples have to be built into the model

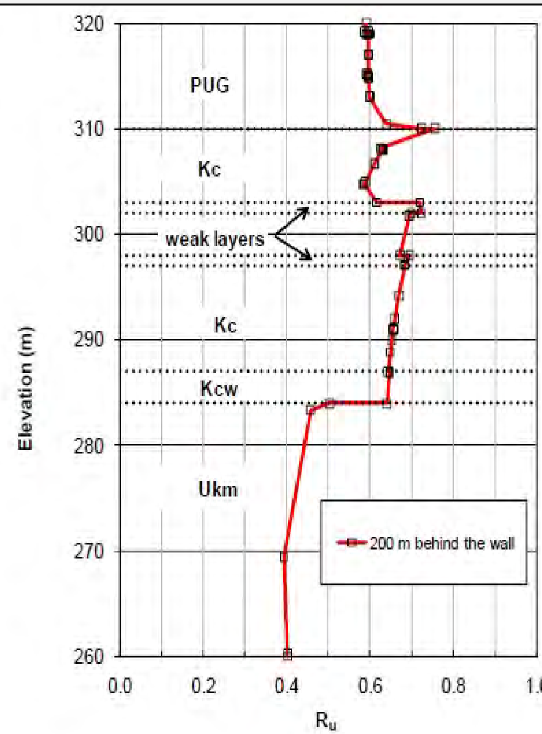
•Stronger cross-bedded layers

•Importance of inspection of core for low density, highest plastic zones, with emphasis on per-shearing

•**From experience, only one to two move**, but could be any of the one to two , depending on the shear key design, but maximum shown is two weak layers at a time i.e. only two move at a time, not all four at once. **Displacement programs will move all four if you allow the model to just run so this practical experience is required to be input.**

•Can get shear modulus of the material from SI's measuring cumulative movement over the thickness of stronger clay below the lowest moving sliding layer, if plain strain is assumed





5b. MSE Wall with Shear Key for MLMR, 2011 design con't

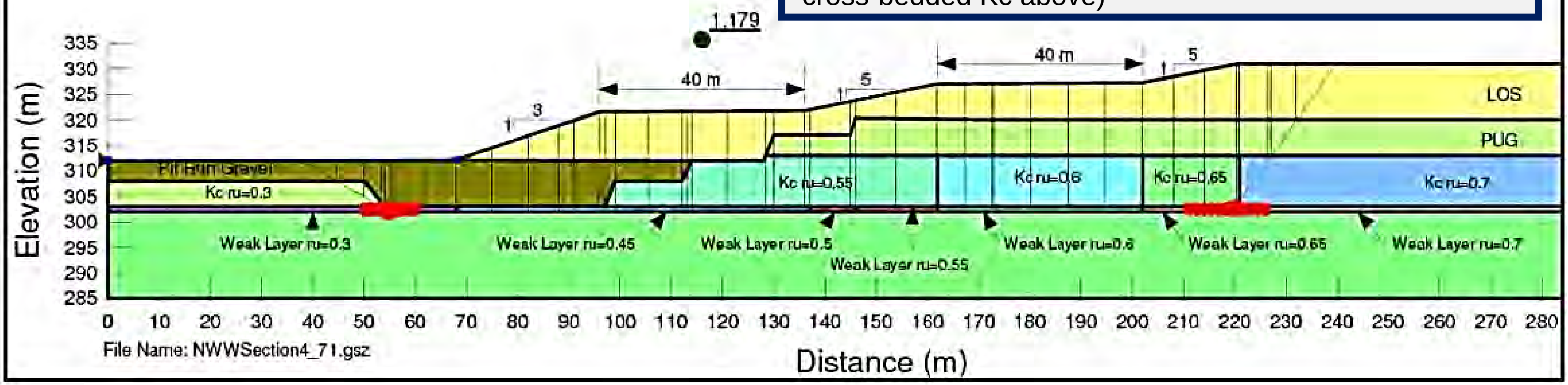
Name: LOS Unit Weight: 19.1 kN/m³ Cohesion: 0 kPa Phi: 33 ° Ru: 0.15 Include in PWP: Yes
Name: PUG Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 25 ° Ru: 0.6 Include in PWP: Yes
Name: Loaded Cross Bedded Kc (Ru=0.65) Unit Weight: 20 kN/m³ Cohesion: 25 kPa Phi: 20 ° Ru: 0.65 Include in PWP: Yes
Name: Loaded Weak Layer (Ru=0.6) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 8 ° Ru: 0.6 Include in PWP: Yes
Name: Unloaded Weak Layer Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 8 ° Ru: 0.45 Include in PWP: Yes
Name: Loaded Weak Layer (Ru=0.3) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 8 ° Ru: 0.3 Include in PWP: Yes
Name: Loaded Weak Layer (Ru=0.55) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 8 ° Ru: 0.55 Include in PWP: Yes
Name: Loaded Weak Layer (Ru=0.5) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 8 ° Ru: 0.5 Include in PWP: Yes
Name: Loaded Weak Layer (Ru=0.65) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 8 ° Ru: 0.65 Include in PWP: Yes
Name: Loaded Weak Layer (Ru=0.7) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 8 ° Ru: 0.7 Include in PWP: Yes
Name: Loaded Cross Bedded Kc (Ru=0.3) Unit Weight: 20 kN/m³ Cohesion: 25 kPa Phi: 20 ° Ru: 0.3 Include in PWP: Yes
Name: Loaded Cross Bedded Kc (Ru=0.55) Unit Weight: 20 kN/m³ Cohesion: 25 kPa Phi: 20 ° Ru: 0.55 Include in PWP: Yes
Name: Loaded Cross Bedded Kc (Ru=0.6) Unit Weight: 20 kN/m³ Cohesion: 25 kPa Phi: 20 ° Ru: 0.6 Include in PWP: Yes
Name: Loaded Cross Bedded Kc (Ru=0.7) Unit Weight: 20 kN/m³ Cohesion: 25 kPa Phi: 20 ° Ru: 0.7 Include in PWP: Yes
Name: Pit Run Gravel Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 38 ° Piezometric Line: 1 Include in PWP: No

Slope W Analysis Output

<u>Slope with Shear Key in area North of MSE wall:</u>

Displays how differ ru's for different loading conditions are considered in a stability analysis model

Weak sliding layer also varies in ru (more sensitive than cross-bedded Kc above)



5b. MSE Wall with Shear Key for MLMR, 2011 design, con't

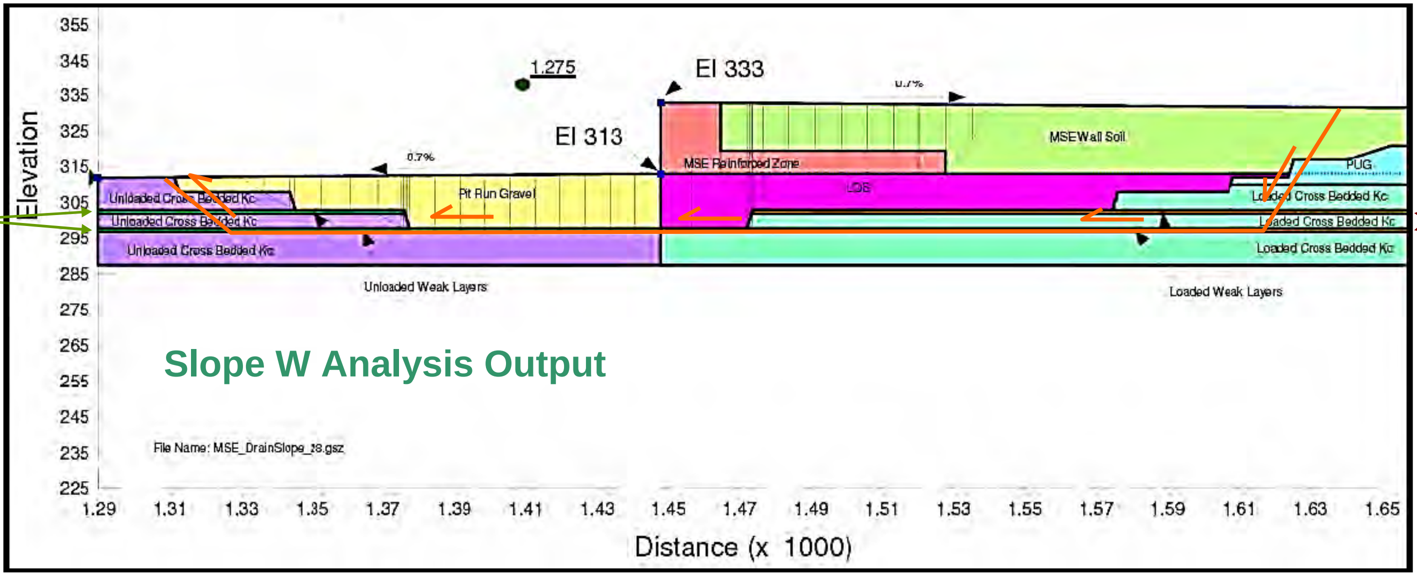
FLAC Analysis used to assess MSE wall with shear key :

•Must know how to introduce weak layers observed in core samples (only allowed two on at a time in FLAC)

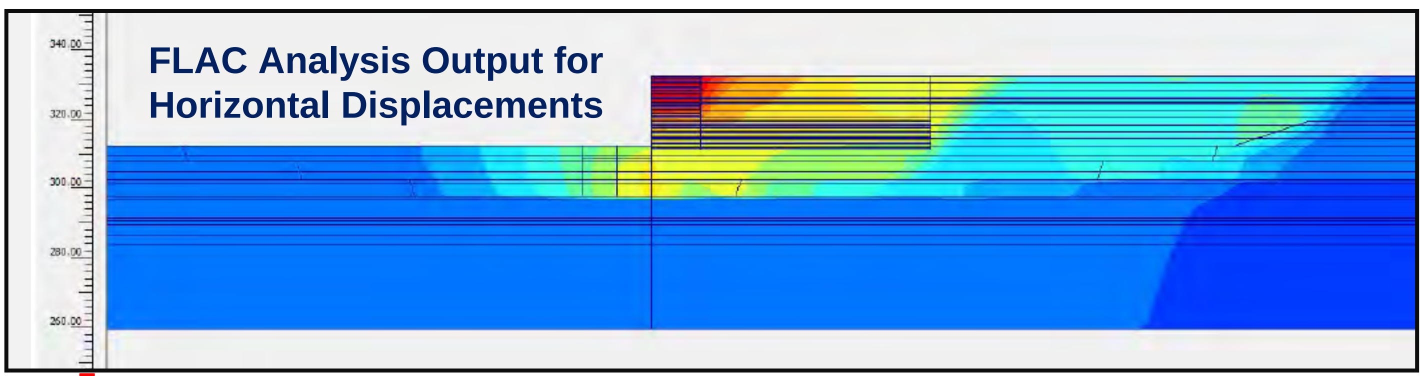
•Must model all clay layers (strong and weak)

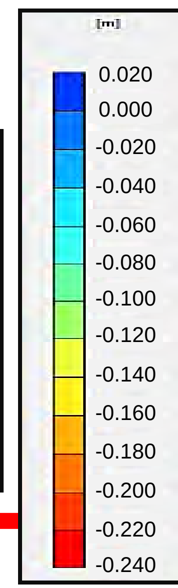
•Must have correct cross-bedded strengths, sliding strengths, and ru values for soil/loading condition

Unloaded sliding clay layers

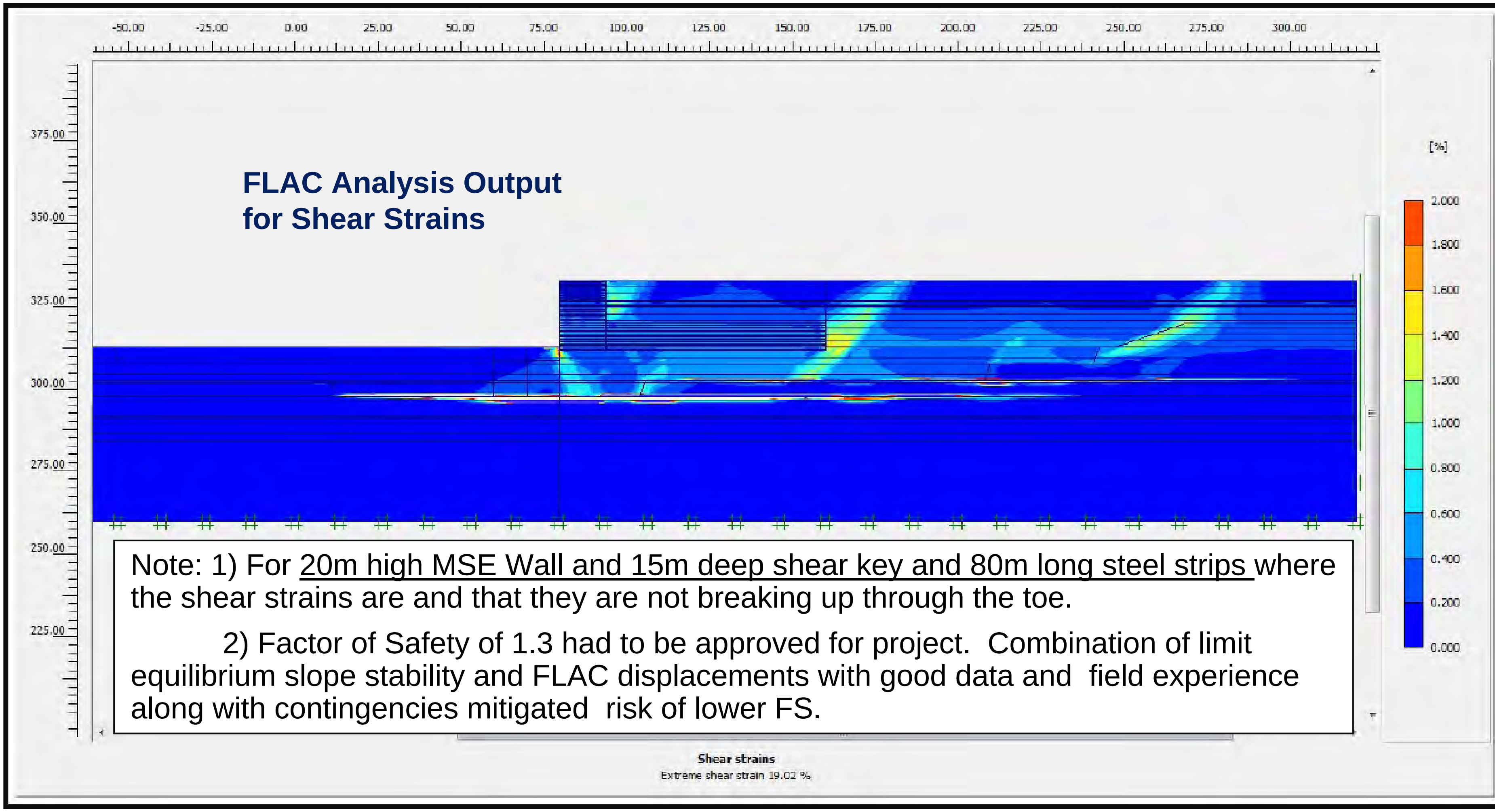


Loaded sliding clay layers





5b. MSE Wall with Shear Key for MLMR, 2011 design, con't



FLAC Analysis Output for Shear Strains

Note: 1) For <u>20m high MSE Wall and 15m deep shear key and 80m long steel strips </u>where the shear strains are and that they are not breaking up through the toe.

2) Factor of Safety of 1.3 had to be approved for project. Combination of limit equilibrium slope stability and FLAC displacements with good data and field experience along with contingencies mitigated risk of lower FS.

Conclusions

When choosing design ru and shear strength values:

1 Must know geological history, energy of depositional environment, any pre-shearing.

2 Must know recent history of excavations & backfilling.

3 Must know the extent of the planned loading and how much the ru will rise (or lower), and need separate ru's for loaded, non-loaded, unloaded and re-loaded areas.

4 Need to build the correct model. For stability analyses modeling need to:

- Draw slip surfaces & model sliding layers and hard layers to represent real failure conditions and pick strengths and variable ru's (pwp) along those slip surfaces to be analyzed one at a time,
- Have correct strength (cross-bedded vs sliding, peak vs residual) and ru parameters for material loading ,or unloading , or unloading and re-loading conditions . Use correct lab test and samples for that layer. Test results rarely include strain softening effects so careful of statistics on peak testing.

5 Need to evaluate parameters of each slip surface for field effects and design depth

6 Need real data from other projects and interpolations must be for the similar geologic conditions and conditions of loading/unloading and similar heights.

7 The same ru from a lower loading height for a higher loading height only works if the clay is not showing large changes in ru. Shallow Pl and Deep Kca vs Shallow Kcc & Shallow Kca.

8 Often need to understand displacements (from FLAC program) not just Factor of Safety (from limit equilibrium slope stability programs with large experience base), when designing slopes below FS=1.3, or for civil/soil interactions (like retaining walls& MSE walls)

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- • 2001 • Cameron, R., Mimura, W., Fong, V., Lussier, L., "Detailed Construction Procedures and
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- 1995 • Cameron R., Ashton, C., Fong, V., Strueby, B., "Syncrude's Highway Berm: Part 1 of 5 – Project Overview and Design Philosophy", 48th Canadian Geotechnical Conference, Canadian Geotechnical Society, Vancouver, B. C., 1995, pp. 789-798.
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