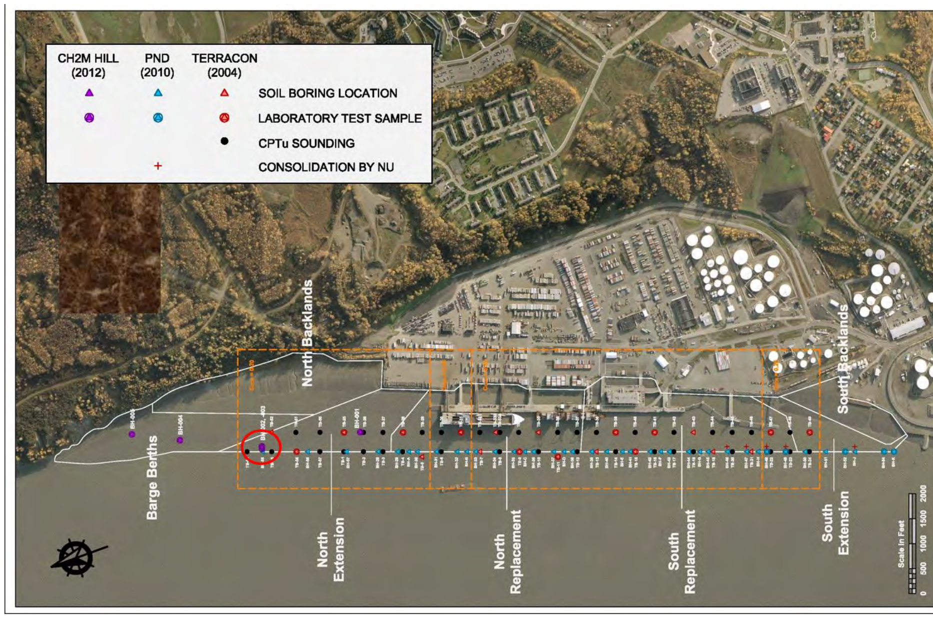
Attachment D

Ring Shear Testing Report (Dr. Timothy Stark)

PORT OF ANCHORAGE INTERMODAL EXPANSION:
Bootlegger Cove Clay Analysis and Testing
Anchorage, Alaska



(Figure by CH2M-HILL)

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Purpose and Scope of Study

At the request of CH2M HILL, Inc., this report presents the results and analysis of index property and torsional ring shear tests conducted on the Bootlegger Cove clay soil from the Port of Anchorage (POA) Intermodal Expansion Project in Anchorage, Alaska. In particular, the project site is the North Expansion of the POA located north of downtown Anchorage, Alaska. The center for the project site is at a longitude of approximately -149.883 degrees and at a latitude of approximately 61.249 degrees. At present, the POA expansion project makes use of the patented Open Cell Sheet Pile OCSP® system for the wharf and the berthing area. The OCSP® system was developed and patented by PND Engineers, Inc. (PND) of Anchorage, Alaska, in 2004 (Patent No. US 6,715,964 B2) and has been used since about 2000 to construct wharf and berthing structures for other projects. Torsional ring shear testing was requested by CH2M HILL, Inc. to investigate shear response of Bootlegger Cove clay at large shear displacements. It is anticipated that the results were used by CH2M HILL, Inc. in stability and permanent deformation evaluations of the OCSP system.

The table of contents above presents the structure of this report which presents the data and analysis for the Bootlegger Cove clay investigated during this study.

Soil Description

The index property and torsional ring shear testing was performed on Bootlegger Cove clay taken from Boring Number BH-002-12 Sample Tube ST-5. Sample Tube ST-5 was obtained from a depth of 99.0 to 101.0 feet and is located in the Fourth (IV) Facies of the Bootlegger Cove Formation (BCF). Figure 1 is an aerial view of the POA provided by CH2M HILL and identifies the location of Boring Number BH-002-12.

The cone penetration sounding (CPT-26) adjacent to soil boring BH-002-12 is shown in Figure 2. CPT-26 was conducted prior to construction of the OCSP structure. Soil boring BH-002-12 was drilled specifically to collect the nominal 5-inch diameter samples used for the ring shear testing described in this report. Figure 2 shows the depth of the fill/foundation interface at this location is 48 to 50 feet. The ground surface elevation, i.e. top of fill, is approximately 35 feet (MLLW). CPT-26 shows the two Bootlegger Cove clay facies and the depth of ST-5 at 99 to 101 feet. The elevation range of ST-5 from roughly EL -64 to EL -66 ft MLLW is shown by the red rectangle in Figure 2. Sample Tube ST-5 was obtained from Bootlegger Cove clay Facies IV (F.IV) as defined by Updike and Carpenter (1986). Updike and Carpenter (1986) use the following definitions for the various facies of the Bootlegger Cove clay that are shown in Figure 2:

- F.IV (upper layer): "Silty clay and clayey silt with silt and fine sand lenses".
- F.I (lower layer): "Clay with very minor amounts of silt and sand".

Figure 3 presents two photographs of the grey Bootlegger Cove clay prior to trimming of a constant volume ring shear specimen. Figure 3(a) shows a one inch thick sample that was cut from the end of the 5-inch diameter sample tube ST-5. The ring of the metal sample tube was also cut with a band saw. The metal ring surrounding the one inch thick sample was cut vertically to facilitate removal as shown in Figure 3(b). Figure 3(b) also shows the consistency and uniformity of the grey Bootlegger Cove clay. One of the Bootlegger Cove clay samples contained two light grey silt stringers, otherwise the one inch thick samples cut from the end of the 5-inch diameter sample tube ST-5 did not contain silt stringers and were similar to the material shown in Figure 3(b).

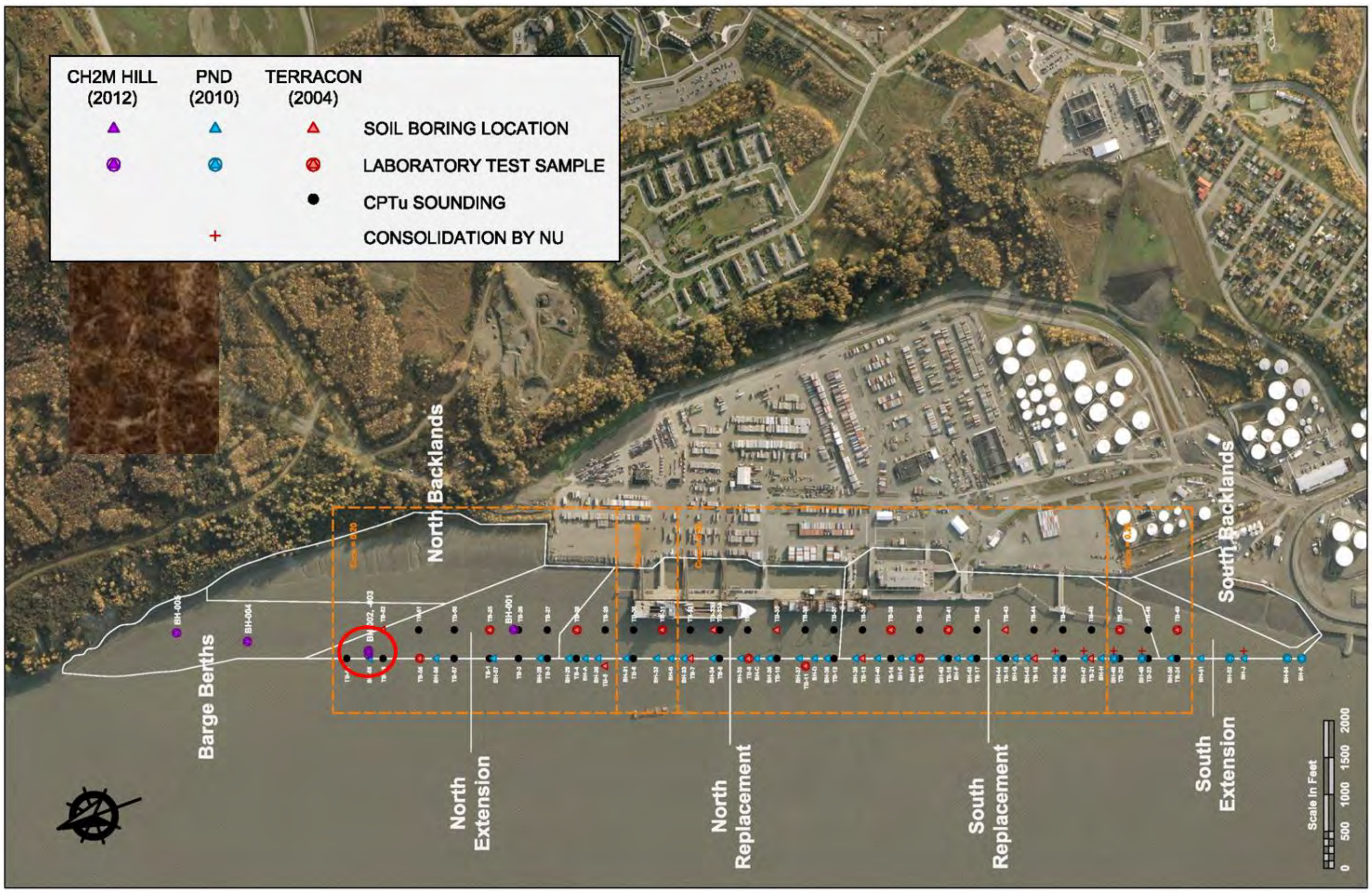

Figure 1. Aerial view of Port of Anchorage Intermodal Expansion Project and red circle indicates location of Boring Number BH-002-12 (figure prepared by CH2M HILL)

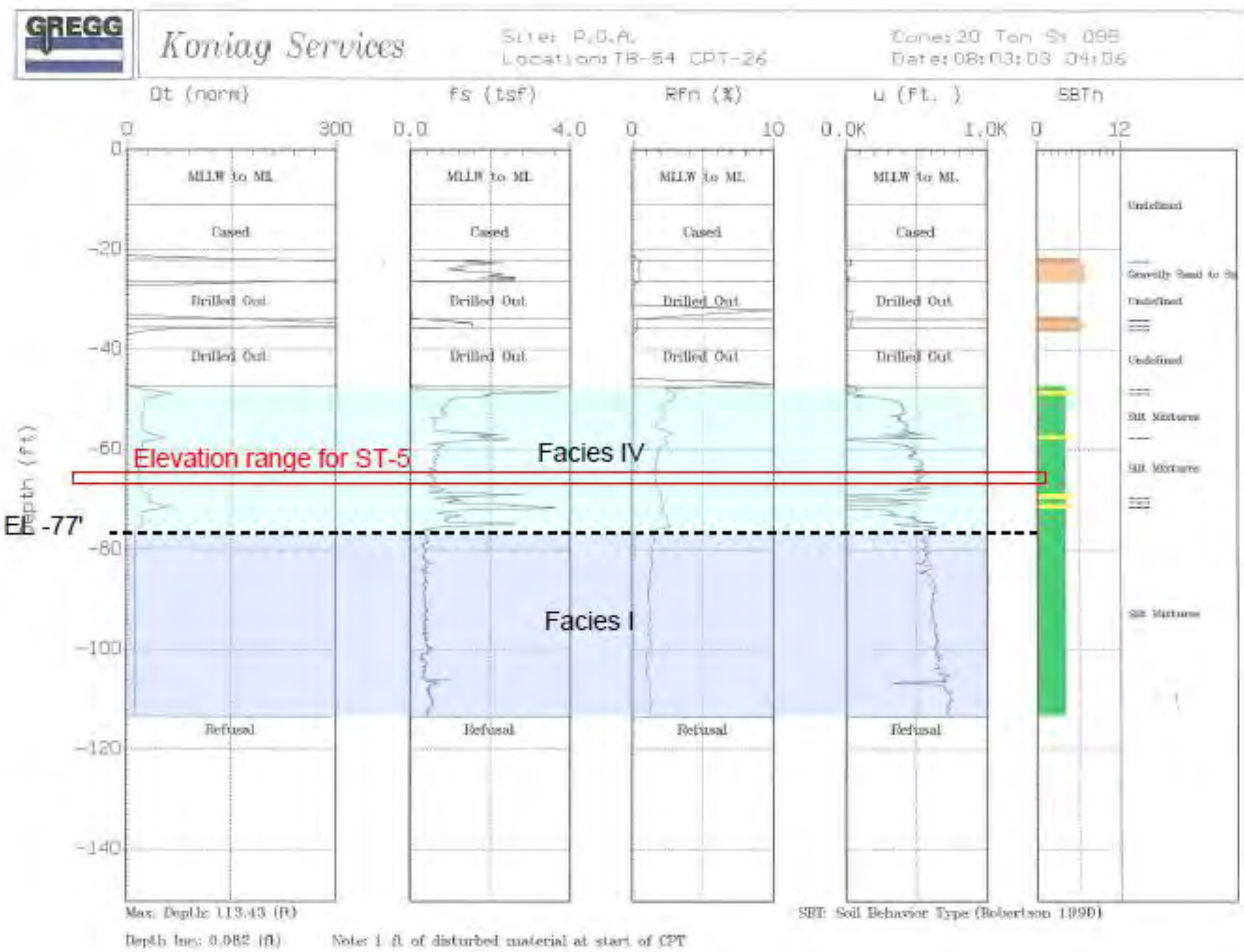

Figure 2. Cone penetration sounding CPT-26 adjacent to Boring Number BH-002-12 (figure prepared by CH2M HILL)

(a) (b)

Figure 3. View of one inch thick sample cut from ST-5 sample tube and close-up of grey Bootlegger Cove clay before trimming a constant volume ring shear specimen

Prior Bootlegger Cove Clay Testing

A number of landslides involving cohesive soils have occurred during earthquakes. The most notable examples are the Fourth Avenue, L-Street, Government Hill, and Turnagain Heights landslides in Anchorage that occurred during the 1964 Alaska earthquake (Seed 1968 and Idriss 1985). Figure 4 is a photograph of the Fourth Avenue landslide in which the Denali Theater dropped about one story so the theater awning is at the same level as Fourth Avenue. A comprehensive study of the Fourth Avenue landslide was conducted by Woodward-Clyde Consultants (1982) as part of an evaluation of the Fourth Avenue area for development of a major state office complex. This study concluded that the slide was not caused by liquefaction of sand seams but by a large undrained strength loss in the slightly overconsolidated clay of the Bootlegger Cove formation (Woodward-Clyde 1982; Idriss 1985; Updike et al. 1988). However, a laboratory or field apparatus that could measure the undrained post-peak strength loss of the cohesive soil was not available during these studies to confirm this hypothesis. Stark and Contreras (1996) developed a laboratory constant volume torsional ring shear apparatus that allows measurement of the magnitude and rate of undrained post-peak strength loss and the undrained residual shear strength in cohesive soils. Stark and Contreras (1998), see Appendix A, used this apparatus and the resulting data to re-evaluate the Fourth Avenue landslide and develop recommendations for evaluating the seismic stability of slopes in cohesive soils.

Figure 4. View of Denali Theater awning with Fourth Avenue on the right after the 1964 earthquake (photograph by the U.S. Geological Survey)

Stark and Contreras (1998) present a re-evaluation of the Fourth Avenue landslide in Anchorage that occurred during the 1964 Alaska earthquake. Laboratory constant volume ring shear tests were conducted on 5-inch diameter samples obtained using a 6 inch inside diameter hollow stem auger. These samples were obtained in a parking lot north of the businesses that front Fourth Avenue (see Figure 5). The results of these tests were compared to back-calculated shear strengths of the Bootlegger Cove clay to determine the mobilized strength of the clay. The comparison shows that slide blocks that moved less than 0.15 m mobilized at least 80 percent of the undrained peak shear strength. Slide blocks that moved between 0.15 to 2.5 m mobilized an undrained shear strength between the peak and residual shear strengths. Slide blocks that displaced more than 2.5 m mobilized the undrained residual strength. Figure 6 presents the decrease in undrained shear strength with block displacement described above.

(a) (b)

Figure 5. (a) Drill rig and (b) five inch diameter sample tubes in parking lot north of Fourth Avenue in Anchorage, Alaska and overlying the Fourth Avenue slide mass

Stark and Contreras (1998) also tested other cohesive soils to investigate the development of an undrained post-peak strength loss. Table 1 presents a summary of the natural soils tested by Stark and Contreras (1998) using the constant volume ring shear apparatus. It can be seen that these four natural soils exhibit a limited range of plasticity and clay size fraction, and thus similar peak and residual undrained strength ratios. In addition, the shear displacement required to mobilize the peak and residual strength conditions are in agreement.

Subsequently, Stark (2001) performed constant volume ring shear tests on the Bootlegger Cove clay obtained during the investigation and design for the development of the Port MacKenzie project. The main results of the Port MacKenzie testing are also shown in Table 1. Based on prior and current testing, data on the undrained shear behavior and post-peak strength loss is available for the Bootlegger Cove clay at three locations near Anchorage, Alaska. Figure 7 is a map that shows the location and proximity of these three Bootlegger Cove clay locations. Table 2 presents a list of the drained and undrained torsional ring shear tests performed during this study including the testing details such as, consolidation stress, drainage condition, shear displacement rate, and test termination. Table 1 also includes a summary of the constant volume ring shear tests on the Bootlegger Cove clay obtained from the Port of Anchorage. The Port of Anchorage values are shown in red and are in agreement with the values obtained from the Fourth Avenue landslide and Port MacKenzie investigations described herein.

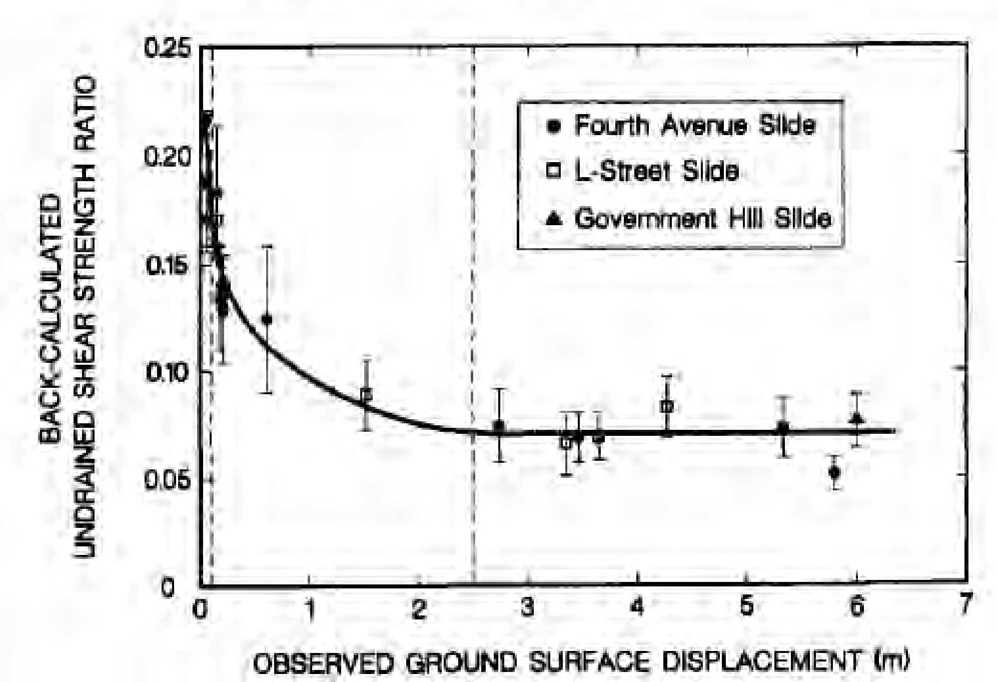

Figure 6. Variation of undrained shear strength ratio with ground surface displacement (figure from Stark and Contreras, 1998)

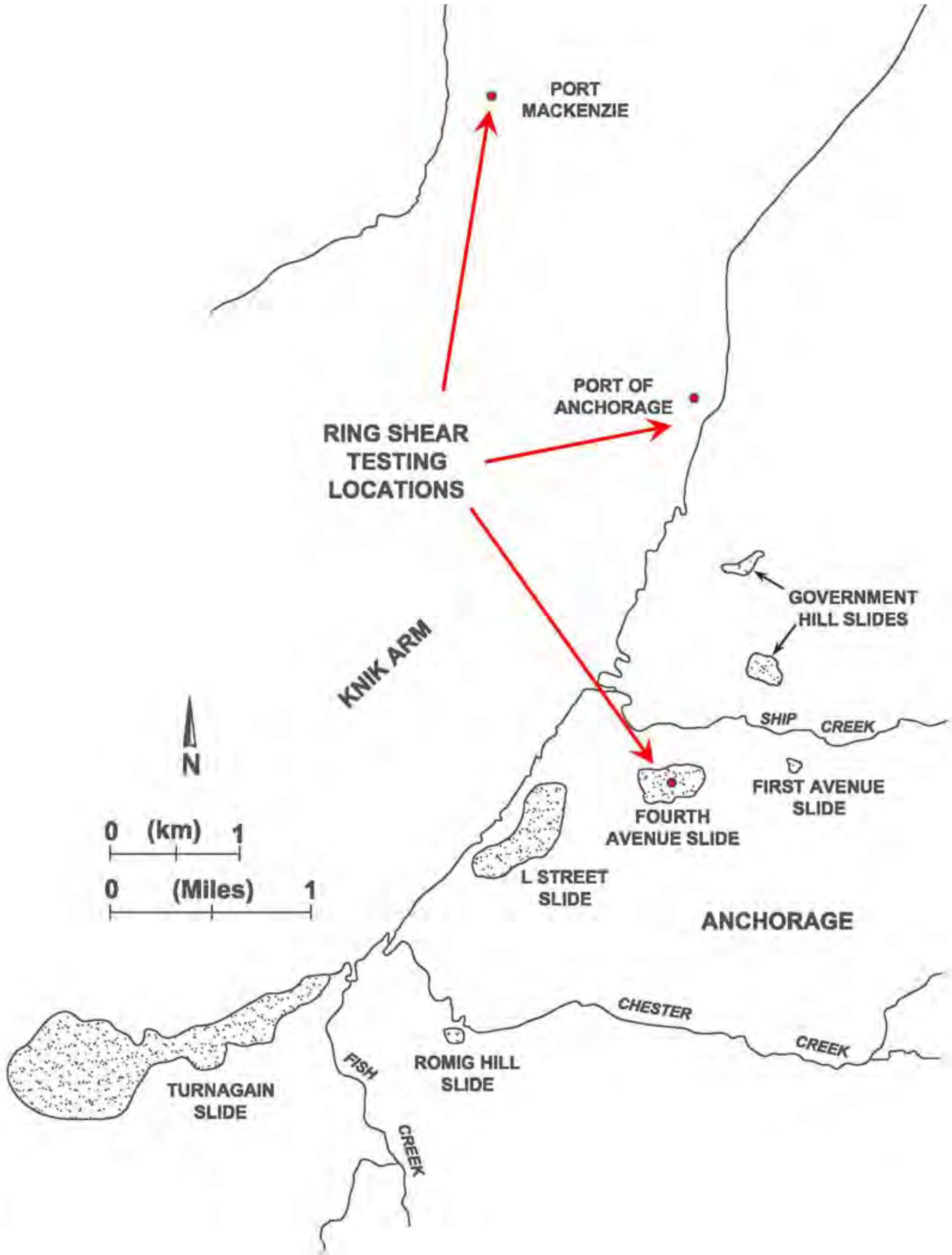

Figure 7. Map showing earthquake induced landsides during 1964 earthquake and three sites at which constant volume ring shear testing has been performed to investigate the undrained shear behavior of Bootlegger Cove clay (map prepared by CH2M HILL)

Table 1: Undrained ring shear test results on undisturbed Bootlegger Cove clay from three sites near Anchorage, Alaska and other cohesive soils

Soil deposit and location	Liquid limit (%)	Plastic limit (%)	Clay size fraction (%<0.002 m)	Ring Shear vertical consolidation stress σ'_{vc} (kPa)	Preconsolidation pressure σ'_{p} (kPa)	Undrained peak shear strength ratio s_u/σ'_{vc}	Shear displacement at peak strength ratio (mm)	Undrained residual strength ratio s_{ur}/σ'_{vc}	Shear displacement at residual strength ratio (mm)	Undrained residual strength ratio/ Undrained peak shear strength ratio
Bootlegger Cove clay, inside Fourth Ave. landslide, Anchorage, Alaska	40 34 36 38 39	20 19 21 21 20	59 57 56 55 62	100 230 300 400 500	280-320	0.28 0.28 0.24 0.23 0.23	1.2 1.1 1.3 1.8 1.8	0.07 0.07 0.06 0.06 0.06	55 75 75 120 130	0..25 0.25 0.25 0.26 0.26
Bootlegger Cove clay, outside Fourth Ave. landslide, Anchorage, Alaska	42 40 42 41	23 21 23 22	47 42 49 45	150 225 400 500	405	0.31 0.32 0.31 0.30	1.5 1.6 1.7 1.7	0.11 0.10 0.11 0.11	95 110 125 140	0.35 0.31 0.35 0.37
Bootlegger Cove clay, Port MacKenzie, Anchorage, Alaska	29 35 35	17 20 21	38 47 53	180.0 359.1 718.2	300 383 575	0.21 0.31 0.29	0.70 2.0 2.0	0.04 0.06 0.05	168 219 170	0.19 0.19 0.17
Bootlegger Cove clay, Port of Anchorage, Anchorage, Alaska	31 37	16 19	51 53	100 200 300 400 600	440	0.19 0.32 0.33 0.37 0.32	0.40 0.98 1.1 1.7 2.4	0.03 0.16 0.15 0.14 0.09	33 31 69 78 102	0.16 0.50 0.45 0.38 0.28
Drammen clay, Danvik-gate, Drammen, Norway	47 48 47	23 24 25	70 72 65	95 255 400	140	0.27 0.22 0.20	1.1 1.3 1.1	0.09 0.11 0.11	19 16 60	0.33 0.50 0.55
Cohesive alluvium, Enid Dam, Enid, Mississippi	30 28 23 25 30	22 22 19 22 22	19 20 17 20 20	95.8 147 191 287 383	122.4 138.9 81.4 143.5 134.5	0.19 0.27 0.24 0.23 0.23	2.2 1.1 1.1 1.2 1.2	0.10 0.05 0.07 0.07 0.06	52 77 70 72 75	0.52 0.19 0.29 0.30 0.26
Cohesive alluvium, Jackson, Alabama	59	31	51	51.8 79.4 100	75.8	0.21 0.23 0.23	0.50 0.35 0.37	0.13 0.16 0.14	36 50 38	0.62 0.69 0.61
Upper Bonneville clay, Salt Lake City, Utah	46	23	33	47.9 95.8 191.5 383	47.9 95.8 191.5 383	0.32 0.36 0.31 0.34	0.30 0.60 1.2 2.0	0.11 0.15 0.12 0.14	39 25 29 36	0.34 0.42 0.39 0.41

Table 2: Constant volume ring shear (CVRS) and drained ring shear (DRS) tests performed on undisturbed and remolded Bootlegger Cove clay from ST-5 sample tube for Port of Anchorage Intermodal Expansion Project (table prepared by CH2M-HILL)

Test ID No.	Boring ID No.	Sample Tube ID No.	Depth Range (ft)	Boot-legger Cove Clay Facies	Ring Shear Test Type	Test Overconsol-idation Ratio (OCR)	Ring Shear Applied Vertical Stresses: In Situ σ'_{vp} (psi)	Reconsol. σ'_{vm} (psi)	Shearing σ'_{vc} (psi)	Shearing σ'_{vc} (kPa)	Test Status
CVRS-01	BH-002-12	ST-5	99 to 101	F.IV	Constant Vol./Undrained	4.41	64.0	14.5	14.5	(100 kPa)	Done
CVRS-02	BH-002-12	ST-5	99 to 101	F.IV	Constant Vol./Undrained	2.21	64.0	29.0	29.0	(200 kPa)	Done
CVRS-03	BH-002-12	ST-5	99 to 101	F.IV	Constant Vol./Undrained	1.47	64.0	43.5	43.5	(300 kPa)	Done
CVRS-04	BH-002-12	ST-5	99 to 101	F.IV	Constant Vol./Undrained	1.10	64.0	58.0	58.0	(400 kPa)	Done
CVRS-06	BH-002-12	ST-5	99 to 101	F.IV	Constant Vol./Undrained	1.00	64.0	87.0	87.0	(600 kPa)	Done
DRS-08	BH-002-12	ST-5	99 to 101	F.IV	Drained/Constant Vert. Stress	4.41	64.0	14.5	14.5	(100 kPa)	Done
DRS-09	BH-002-12	ST-5	99 to 101	F.IV	Drained/Constant Vert. Stress	1.10	64.0	58.0	58.0	(400 kPa)	Done
DRS-10	BH-002-12	ST-5	99 to 101	F.IV	Drained/Constant Vert. Stress	1.00	64.0	87.0	87.0	(600 kPa)	Done
CVRS-13	BH-002-12	ST-5	99 to 101	F.IV	FASTER (0.18 mm/min) Constant Vol./Undrained	1.10	64.0	58.0	58.0	(400 kPa)	Done
CVRS-14	BH-002-12	ST-5	99 to 101	F.IV	HEALING Constant Vol./Undrained	1.10	64.0	58.0	58.0	(400 kPa)	Done
CVRS-15	BH-002-12	ST-5	99 to 101	F.IV	HEALING Constant Vol./Undrained-Remolded Specimen	1.10	64.0	58.0	58.0	(400 kPa)	Done
DRS-16	BH-002-12	ST-5	99 to 101	F.IV	MULTI-STAGE Drained/Constant Vert. Stress ASTM D6467	7.00	64.0	87.0	87.0	50 to 700 kPa	Done

Index Property Testing

Two sets of index property tests were conducted on Bootlegger Cove clay obtained from different portions of ST-5. The index property tests (Atterberg limits, particle size analysis, and hydrometer) were conducted in accordance with ASTM standards (D4318, D136, and D422, respectively). The results are shown in Tables 3 and 4 and show the Bootlegger Cove clay is fairly uniform. Based on the index property results in Tables 3 and 4, the Bootlegger Cove clay classifies as a low plasticity clay (CL) according to the Unified Soil Classification System.

Table 3: Index Property and Consolidation Data for Sample Tube ST-5 (Depth 99 to 101 feet) and Drained Ring Shear Sample #1 (Sample was obtained from bottom 2 to 3 inches of sample tube ST-5)

In-situ Natural Water Content	~19%
Liquid Limit	31%
Plastic Limit	16%
Plastic Index	15%
Clay Size Fraction (<0.002 mm)	51%
Preconsolidation Pressure from CH2M HILL	9,216 psf
Largest Particle Size	~6 mm
Percent Passing #4	~100%
Percent Passing #200	86%
Description Based On USCS	CL – low plastic inorganic clay, silty clay

Table 4: Index Property and Consolidation Data for Sample Tube ST-5 (Depth 99 to 101 feet) and Drained Ring Shear Sample #2 (Sample was obtained from bottom 3 to 4 inches of sample tube ST-5)

In-situ Natural Water Content	~20%
Liquid Limit	37%
Plastic Limit	19%
Plastic Index	18%
Clay Size Fraction (<0.002 mm)	53%
Preconsolidation Pressure from CH2M HILL	9,216 psf
Largest Particle Size	~6 mm
Percent Passing #4	~100%
Percent Passing #200	83%
Description Based On USCS	CL – low plastic inorganic clay, silty clay

Drained Torsional Ring Shear Testing

A modified Bromhead ring shear apparatus (Stark and Eid, 1993) and most of the procedure presented in ASTM D6467 were used to measure the drained residual shear strength of these three undisturbed samples. However, an undisturbed specimen was used for these three drained ring shear tests instead of a remolded specimen as allowed in ASTM D6467. The undisturbed ring shear specimens were trimmed from the Port of Anchorage sample tube ST-5 in a moist room using a specimen container fabricated to allow undisturbed annular specimens to be trimmed directly into the container. The new specimen container is described by Stark and Contreras (1996) and consists of eight parts that can be joined and disassembled for the trimming and shear phases of the test. For the trimming phase the inner cutter ring, inner ring, outer cutter ring, upper outer ring, and holder are assembled to create the trimming apparatus. After trimming, the specimen container is converted to the configuration used for shearing in the ring shear apparatus. For the shearing phase the inner and outer cutter rings and holder are removed. The porous disc, center core, and lower outer ring are added to create the shearing specimen container. Assemblage of the trimming apparatus and specimen container are illustrated in Stark and Contreras (1996).

After trimming of the annular specimen, the specimen is placed in the ring shear device and consolidated to the desired effective normal stress by increasing the normal load in small increments, i.e., load increment ratio of unity. Each load increment is maintained until the end of primary consolidation is achieved. Once the end of primary consolidation under the desired normal stress is achieved, the specimen is ready for drained shearing at a drained displacement rate of 0.018 mm/min. This shear displacement rate is the slowest rate possible in the Bromhead ring shear device and it has been successfully used to test soils that are much more plastic, i.e., lower permeability, than the Bootlegger Cove clay in sample tube ST-5.

Drained torsional ring shear tests were conducted on undisturbed specimens of the Bootlegger Cove clay from sample tube ST-5 at effective normal stresses of 2,088, 8,355, and 12,531 psf (100, 400, and 600 kPa). These tests were performed to measure the drained residual strength of the Bootlegger Cove clay for comparison with the undrained residual strengths described below. These tests are identified as DRS-08, DRS-09, and DRS-10.

The drained shear stress-displacement relationships at effective normal stresses of 2,088 psf (100 kPa), 8,352 psf (400 kPa), and 12,531 psf (600 kPa) are presented in Figure 8 and were used to obtain the drained peak and residual shear stresses for each specimen. Table 5 presents the values of drained peak and residual shear stress, and the corresponding drained residual friction angle, that were obtained from these measured shear stress-displacement relationships. These values of residual shear stress and the corresponding effective normal stresses were used to develop the drained peak and residual failure envelopes shown in Figure 9.

The measured peak and residual strength envelopes are slightly stress-dependent but could be reasonably modeled using a linear strength envelope. However, the measured drained peak and residual shear stresses measured for each specimen (see Table 5) can be used directly in a slope stability analysis to model the stress-dependent nature of the strength envelopes instead of a friction angle.

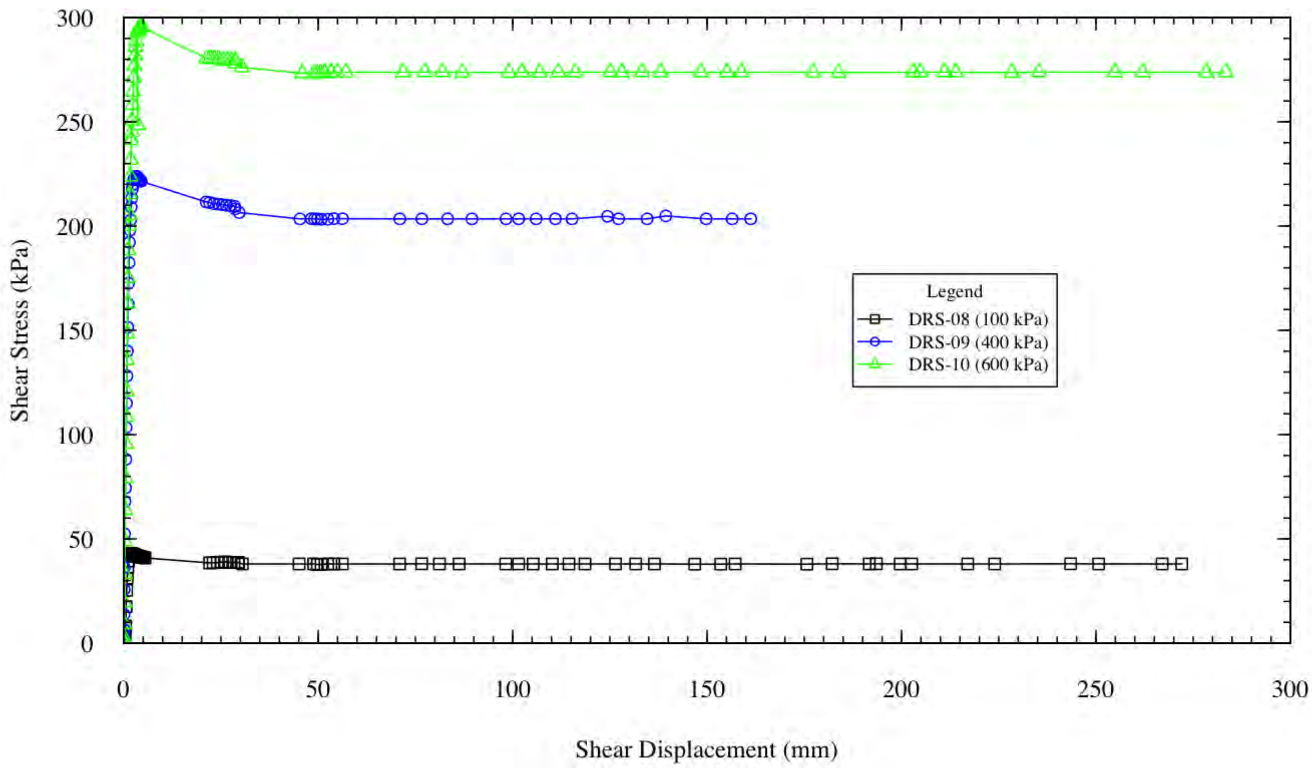

Figure 8. Drained shear stress-displacement relationships from ring shear tests on undisturbed Bootlegger Cove clay from sample tube ST-5

Table 5. Summary of Drained Ring Shear Test Results for Sample Tube ST-5 (Note: NA = Not Available)

Test No. in Table 2	**Drained Peak and Residual Shear Stress at each Effective Normal Stress (psf)**						**Drained Residual Friction Angle (degrees)**	
	1,044 psf	2,088 psf	4,177 psf	8,354 psf	12,530 psf	14,620 psf	**ST-5 Testing**	**Stark et al. (2005) Correlation**
DRS-08		898.2/ 798.0					20.9°	26.6°
DRS-09				4674.9/ 4249.2			27.0°	29.6°
DRS-10					6171.5/ 5717.9		24.5°	21.5°
DRS-16	NA/ 357.0	NA/ 502.9	NA/ 1039.5	NA/ 2177.8	NA/ NA	NA/ 4369.0	18.9-16.6°	23.8°

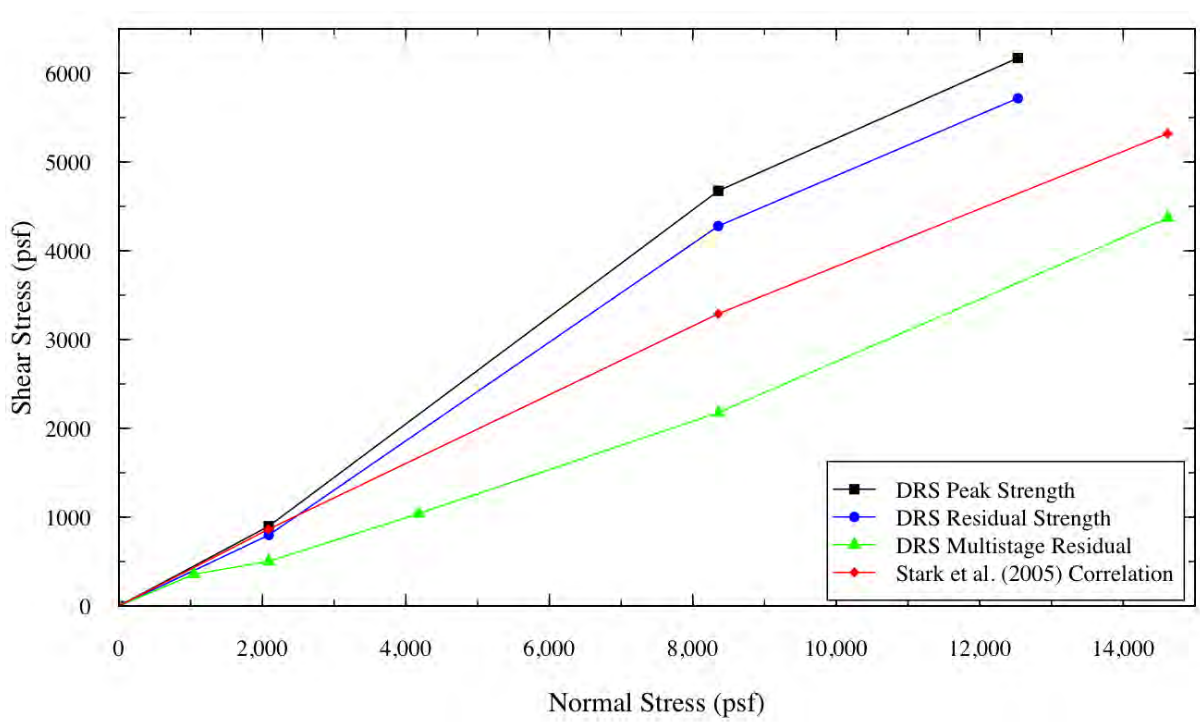

Figure 9. Drained peak and residual strength envelopes for undisturbed Bootlegger Cove clay from sample tube ST-5

To verify the drained residual strengths shown in Table 5 and the drained residual strength envelope in Figure 9, the drained residual strength of the soil used in the DRS-08, DRS-09, and DRS-10 tests was used to conduct a multi-stage residual strength ring shear test in accordance with ASTM D6467. In this test a remolded specimen is initially consolidated to 14,620 psf (700 kPa), unloaded to 2,088 psf (100 kPa), and then pre-sheared by rotating the ring shear base for at least one complete revolution using a shear displacement rate of 6.68 mm/min. After pre-shearing, the specimen is allowed to equilibrate before drained shearing. The specimen is then sheared at a drained displacement rate of 0.018 mm/min at the first effective normal stress of 2,088 psf (100 kPa). This is the slowest shear displacement rate possible in the Bromhead ring shear device and it has been successfully used to test soils that are more plastic, i.e., have lower permeability, than the Bootlegger Cove clay. After a drained residual strength condition is established at 2,088 psf (100 kPa), shearing is stopped and the normal stress increased to 200 kPa (4,177 psf). After consolidation at 200 kPa (4,177 psf), the specimen is sheared again until a drained residual condition is obtained. This procedure is then repeated for effective normal stresses of 8,352 psf (400 kPa) and 14,620 psf (700 kPa).

The remolded specimens were obtained by air-drying a representative soil sample, passing the soil through U.S. Standard Sieve No. 40, and then mixing the sample with distilled water until a water content near the liquid limit was obtained. The sample was then allowed to hydrate for at least 24 hours in a constant moisture room. A spatula was used to place the remolded soil paste into the annular specimen container. The ring shear specimen is annular with an inside diameter of 70 mm and an outside diameter of 100 mm and is 5 mm thick to facilitate consolidation. Annular bronze porous discs secured to the bottom of the specimen container and the top loading platen provides drainage during consolidation and shearing. Both porous discs have a machined or knurled surface to facilitate interlock of the soil specimen with the porous discs to prevent slippage at the specimen bottom and force shearing to occur in the soil at or just below the bottom of the knurled surface of the top porous disc.

The drained shear stress-displacement relationships at effective normal stresses of 1,044 psf (50 kPa), 2,088 psf (100 kPa), 4,177 psf (200 kPa), 8,352 psf (400 kPa), and 14,620 psf (700 kPa) are presented in Figure 10 and were used to obtain the drained residual shear stresses for each effective normal stress. These values of residual shear stress and the corresponding effective normal stresses are plotted in Figure 9.

Table 5 also shows the secant residual friction angles estimated from the empirical relationship proposed by Stark and Eid (1994) and updated by Stark et al. (2005). The empirical correlation uses the liquid limit, clay size fraction (% <0.002 mm), and effective normal stress to estimate the stress-dependent residual failure envelope. The liquid limit estimates the clay mineralogy, the clay size fraction quantifies the amount of the clay mineral, and the effective normal stress captures the stress dependent nature of the residual strength. These secant residual friction angles were estimated using a liquid limit of 34 and a clay-size fraction greater than 50%.

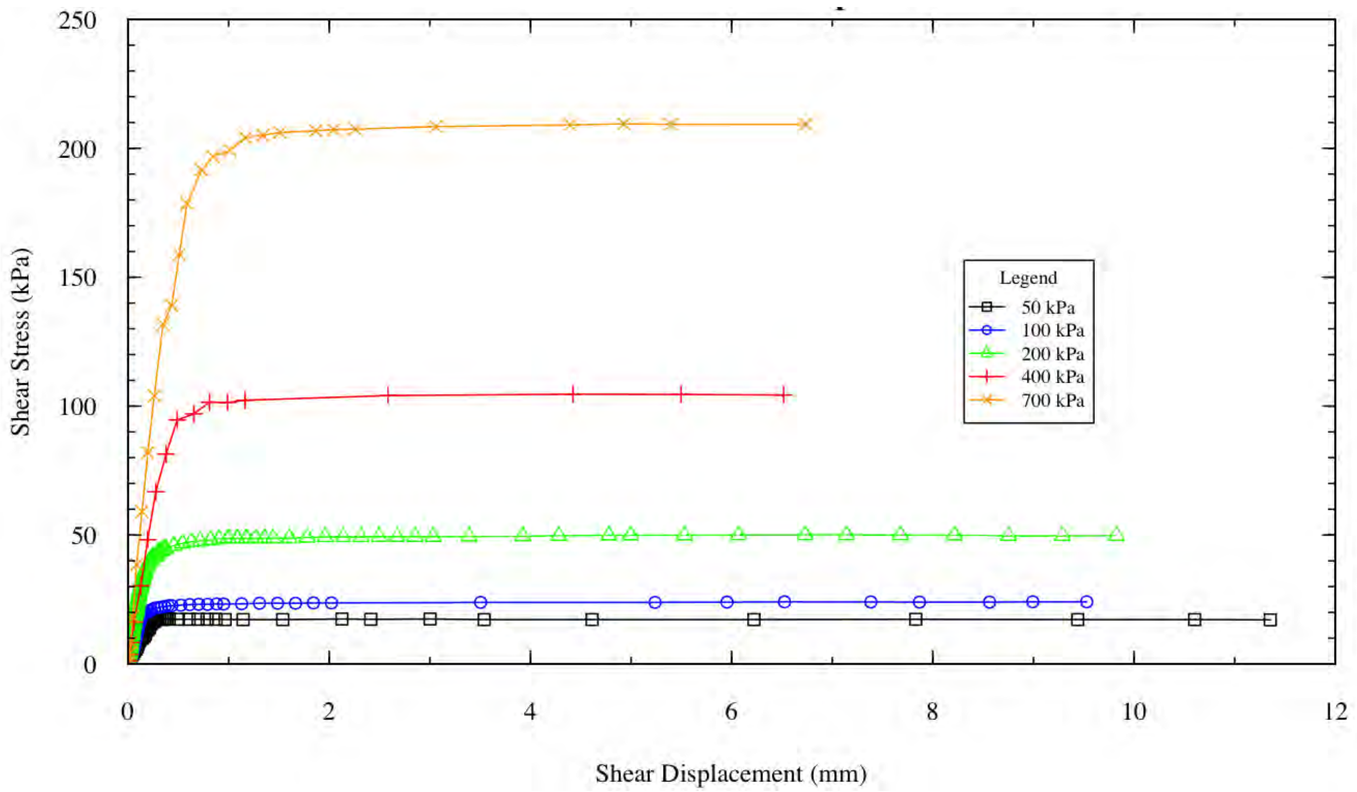

Figure 10. Drained shear stress-displacement relationships from multi-stage ring shear test on remolded Bootlegger Cove clay from sample tube ST-5 using ASTM D6467

Undrained Torsional Ring Shear Testing

(a) Constant Volume Ring Shear Apparatus

Taylor (1952) introduced the use of a constant volume shear test to measure the undrained peak shear strength. A modified direct shear apparatus was used by Taylor (1952) to perform constant volume tests on Boston Blue Clay. Bjerrum and Landva (1966) introduced the use of the direct simple shear apparatus to measure the undrained peak shear strength of Manglerud clay, which also maintains a constant volume. However, undrained triaxial, direct shear, and direct simple shear apparatuses cannot measure the undrained residual strength because only a limited amount of continuous shear displacement can be imposed along a failure surface in these devices. As a result, it was necessary for Stark and Contreras (1996) to develop a torsional ring shear apparatus to evaluate the undrained post-peak strength loss and residual strength of cohesive soils.

To measure the undrained peak and residual shear strengths, the original Bromhead (1979) ring shear apparatus was modified to conduct undrained (constant volume) tests (Stark and Contreras 1996). The modifications include a mechanism for adjusting the normal stress during shear, such that the volume change during shear is negligible and thus the specimen is undrained during shear. The reduction in normal stress is assumed to be equal to the shear induced pore-water pressures (Stark and Contreras 1996). A new specimen container was also fabricated to allow undisturbed specimens to be trimmed directly into the container. The ring shear specimen is annular with an inside diameter of 70 mm and an outside diameter of 100 mm. The specimen is confined radially by the specimen container, which is 10 mm deep. Annular porous discs are secured to the bottom of the specimen container and to the loading platen. The porous discs are serrated to prevent slippage at the porous stone/soil interfaces during shear.

The undisturbed ring shear specimens were trimmed from the Port of Anchorage sample tube ST-5 in a moist room. Remolded specimens were used for the index property testing, a multi-stage drained torsional ring shear test (DRS-16), and one of the constant volume ring shear tests (CVRS-15). Ring shear tests on remolded specimens were conducted to establish the normally consolidated peak and residual failure envelopes for comparison with the results of the undisturbed specimen tests and to assess the sensitivity of the Bootlegger Cove clay in ST-5. A remolded specimen was obtained by mixing undisturbed material with distilled water until the water content is at or near the liquid limit. Afterwards the sample was allowed to rehydrate for two days in a moist room. A spatula was used to place the remolded soil paste into the annular specimen container. The undrained (constant volume) ring shear tests were conducted at a shear displacement rate of 0.018 mm/min (Stark and Eid 1994).

In tests where the specimen was normally-consolidated, the normal stress was reduced during shear to maintain a constant volume/undrained condition. For tests where the specimen was over-consolidated, the normal stress was either reduced or increased during shear, depending on whether compression or dilation of the specimen was occurring, respectively, to maintain a constant volume/undrained condition. The reduction or increase in normal stress is assumed to be equal to the shear induced pore-water pressures.

(b) Constant Volume Ring Shear Test Procedure

The constant volume mechanism consists of a steel rod that is connected to the end of the horizontal beam that applies the normal stress to the top of the specimen. Attached to the steel rod is a load cell, which is shown in Figure 12 and has a wire leading to a digital readout device. A nut is threaded to the top of the steel rod and is adjusted during shear to reduce the amount of normal load transferred to the loading platen. The nut connected to the steel rod is used in combination with the vertical digital dial gauge to adjust the normal stress, such that the specimen thickness remains constant during shear. Adjustments are made by manually rotating the nut an amount required to maintain zero vertical displacement of the soil specimen. When the nut is rotated, a portion of the dead weight being applied to the horizontal beam is transmitted to the rod and the load cell indicates the magnitude of this load via the digital readout device. Additional details of the constant volume ring shear apparatus are presented by Stark and Contreras (1996).

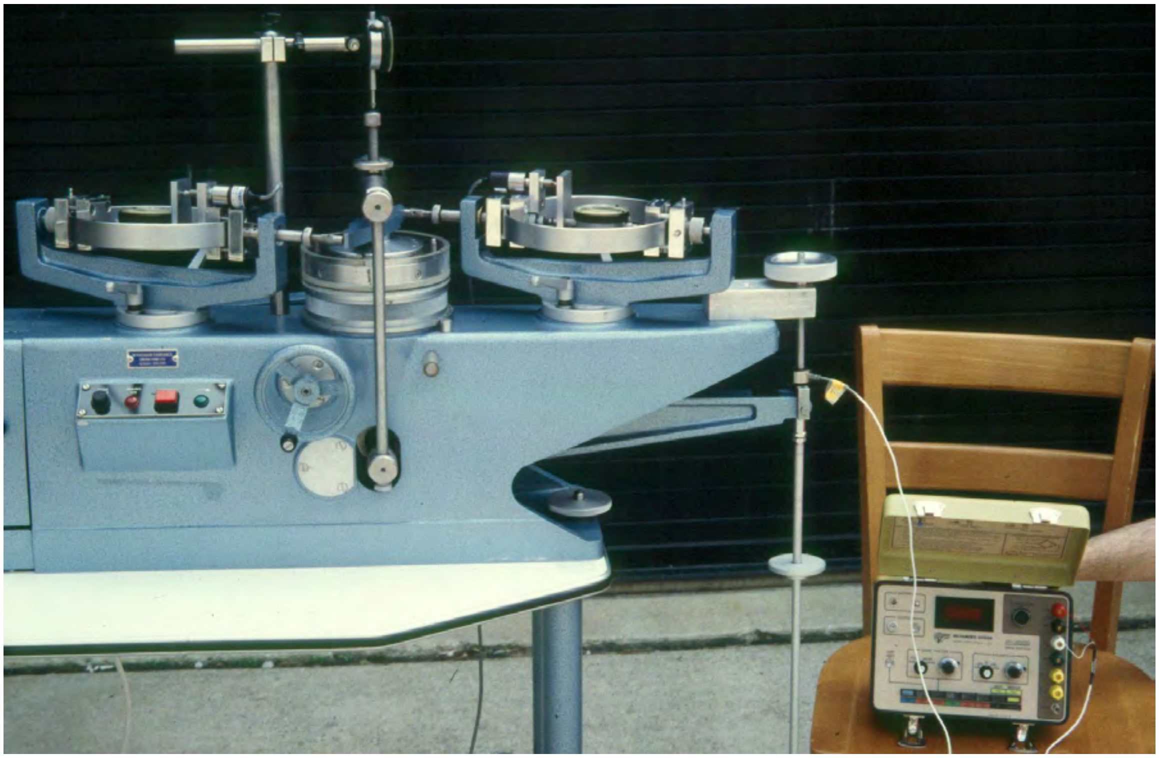

Figure 12. Photograph of constant volume ring shear system attached to modified Bromhead ring shear device

In the constant volume ring shear apparatus, the specimen is sheared by rotating the specimen or specimen container past the stationary upper loading platen at a drained constant rate less than or equal to 0.018 mm/minute. The procedure described by Gibson and Henkel (1954) was used to determine the shear displacement rate of 0.018 mm/minute so zero pore-water pressures were generated throughout the specimen. A drained shear displacement rate is used so that little, if any, excess pore-water pressure is induced during shear. The Bootlegger Cove clay from sample tube ST-5 was primarily contractive during shear because the applied effective normal stress resulted in an overconsolidation ratio (OCR) between one and four (see Table 2). Because the specimens were primarily contractive during shear, the normal stress was reduced during shear to maintain a constant specimen height or volume. It is assumed that the decrease in applied vertical stress during shear is equivalent to the increase in shear-induced pore-water pressure that would occur in an undrained test with constant vertical stress. The validity of this pore-water pressure assumption for constant volume tests was verified by Dyvik et al. (1987) for the direct simple shear apparatus and by Berre (1981) for the triaxial compression apparatus.

(c) <u>Constant Volume Ring Shear Test Results</u>

Figure 13 shows the undrained shear stress-shear displacement relationships from constant volume ring shear tests on undisturbed specimens of Bootlegger Cove clay (CVRS-01, 02, 03, 04, and 06). Vertical consolidation stresses of 2,088, 4177, 6,266, 8,352, and 12,531 psf (100, 200, 300, 400, and 600 kPa) were used in the tests on the Bootlegger Cove clay to simulate the effective stresses acting along the critical failure surface identified by CH2M-HILL. These vertical consolidation stresses were provided by CH2M-HILL.

The undrained peak shear strengths were reached after approximately 0.4 to 2.5 mm of shear displacement. After the undrained peak shear stress was mobilized, shear displacement continued along the shear surface and the measured shear stress decreased with increasing shear displacement. Displacement along the shear surface causes reorientation of soil particles parallel to the direction of shear and an increase in pore-water pressure if the material is contractive. If so, the normal stress was reduced to maintain a constant volume. This continues until the undrained residual strength is mobilized usually at a shear displacement of approximately 30 to 100 mm. At a shear displacement rate of 0.018 mm/min, a shear displacement of 30 to 100 mm required 2 to 4 days of shearing after several day of consolidating to the the desired effective normal stress.

The undrained peak shear stresses correspond to an undrained peak strength ratio of approximately 0.24 to 0.37. The undrained peak strength ratio estimated from the data in Figure 13 is defined as the undrained peak strength divided by the ring shear vertical consolidation stress even at stresses less than the preconsolidation pressure. The undrained residual shear stresses correspond to an undrained residual strength ratio of 0.04 to 0.15. The undrained residual strength ratio is also defined as the undrained residual strength divided by the ring shear vertical consolidation stress even at stresses less than the preconsolidation pressure. The ring shear vertical consolidation stress is used in the recompression and compression ranges because the effect of overconsolidation has been removed before the undrained residual condition has been reached. Based on these undrained strength ratios and review of the shear stress-shear displacement relationships in Figure 13, an undrained strength reduction of 50 to 80% occurred during these tests.

Table 1 presents a summary of the natural soils that have been tested using the constant volume ring shear apparatus. It can be seen that these natural soils exhibit a limited range of plasticity and clay size fraction, and thus similar peak and residual undrained shear strengths ratios as measured for the Bootlegger Cove clay from sample tube ST-5. In addition, the shear displacement required to mobilize the peak and residual strength conditions are in agreement. Therefore, the constant volume ring shear tests on the Bootlegger Cove clay from ST-5 appear to be consistent with other natural soils of similar plasticity and clay size fraction.

The undrained peak and residual shear stresses from constant volume ring shear tests CVRS-01, CVRS-02, CVRS-03, CVRS-04, and CVRS-06 are shown in Figure 14 and yield reasonable strength envelopes. Review of the data indicates that the undrained peak and residual shear stresses at an effective normal stress of 2,088 psf (100 kPa) are a little low and are skewing the strength envelope lower at low effective normal stresses. From Figure 14, the average undrained peak strength ratio is approximately 0.30 and is in agreement with prior testing shown in Table 1. The average residual undrained strength ratio from Figure 14 is approximately 0.10 and is in agreement with the value of 0.06 obtained by Stark and Contreras (1998) for the Bootlegger Cove clay obtained from the area of the Fourth Avenue Landslide.

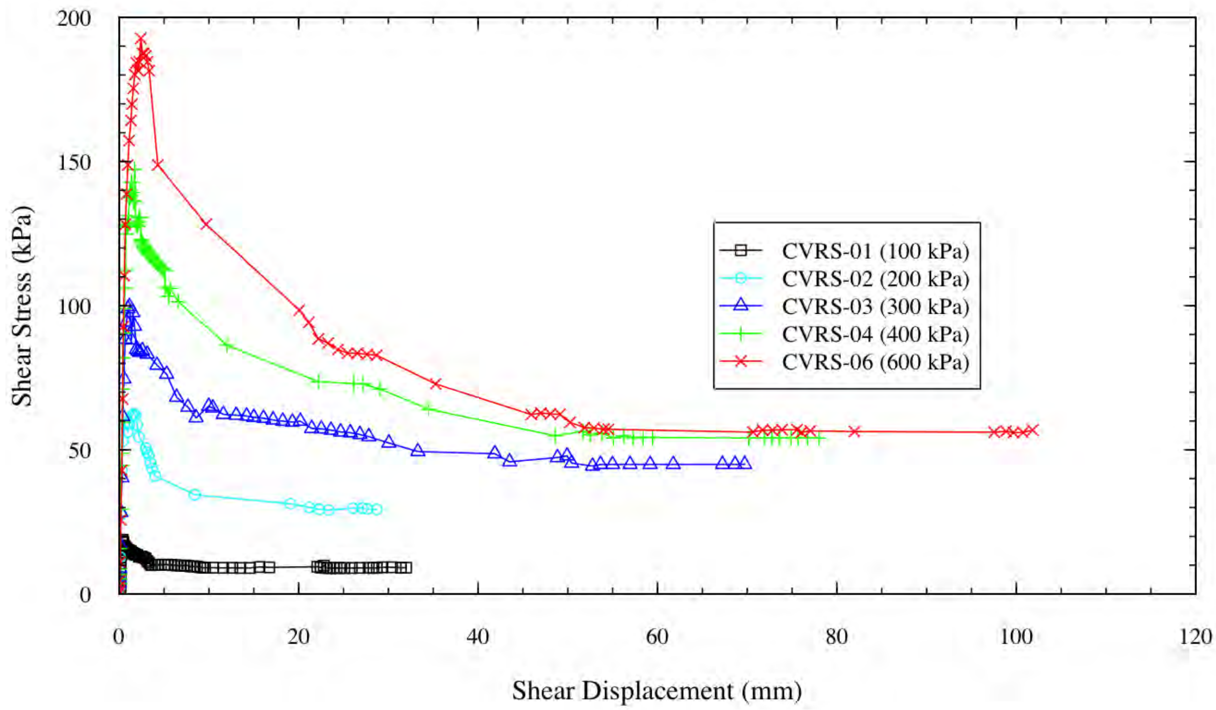

Figure 13. Undrained shear stress-displacement relationships from constant volume ring shear tests on undisturbed Bootlegger Cove clay from sample tube ST-5

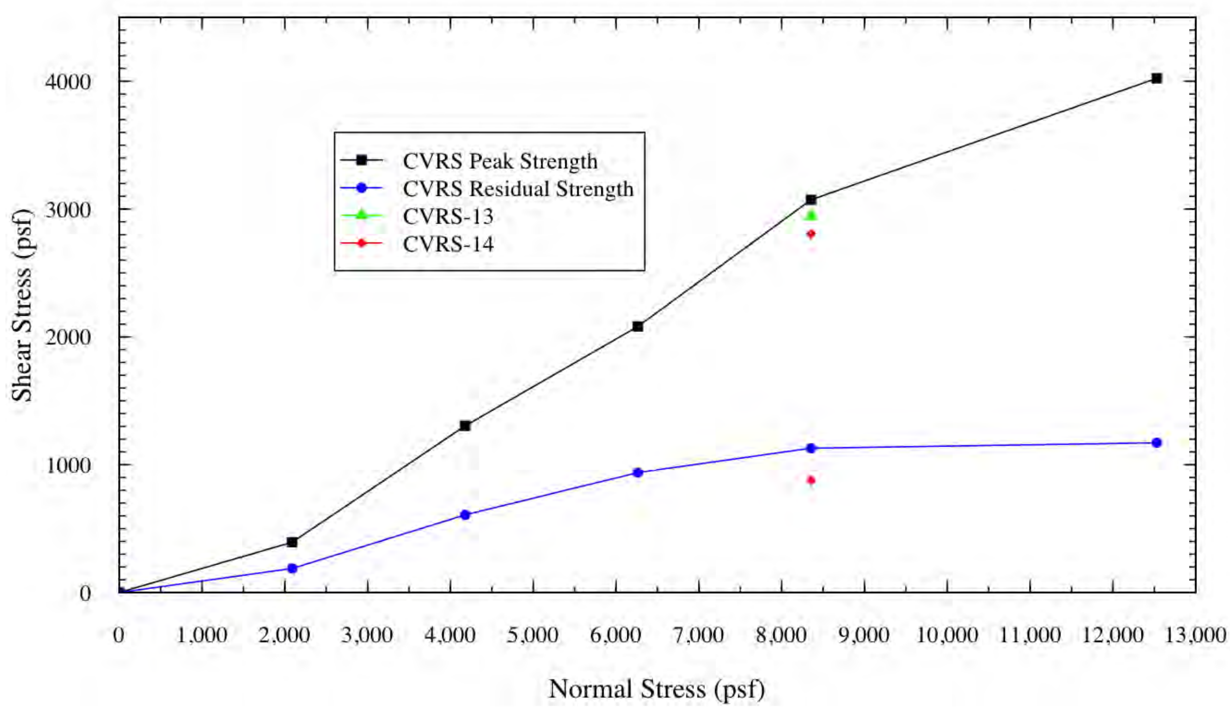

Figure 14. Undrained peak and residual strength envelopes for undisturbed and remolded Bootlegger Cove clay specimens from sample tube ST-5

(d) Rapid Constant Volume Ring Shear Test Results

Another series of constant volume ring shear tests were conducted to investigate the effect, if any, of a faster shear displacement rate on the response of undisturbed Bootlegger Cove clay from sample tube ST-5. A constant volume ring shear test was conducted at a shear displacement rate ten (10) times faster than the initial test at an effective normal stress of 8,352 psf (400 kPa). In other words, constant volume ring shear test CVRS-13 was conducted at a shear displacement rate of 0.18 mm/min for comparison with CVRS-04 which was conducted a 0.018 mm/min. A faster shear displacement rate was used to investigate whether or not additional strength loss, i.e., particle reorientation and/or pore-water pressure generation, would occur at a faster shear displacement rate. This test was performed to ensure that a different undrained strength would not occur in the field than observed in the laboratory. Both tests (CVRS-04 and CVRS-13) were conducted on undisturbed specimens and at an effective normal stress of 8,352 psf (400 kPa).

Figure 15 shows the undrained shear stress-shear displacement relationships from these constant volume ring shear tests at different shear displacement rates at a vertical consolidation stress of 8,352 psf (400 kPa). Figure 15 shows there is little difference in the undrained peak strengths (149 kPa versus 142 kPa) for these two tests. As a result, it was concluded that a shear displacement rate of 0.018 mm/min was appropriate and yielded a representative undrained response in the constant volume ring shear tests. The faster constant volume ring shear test (CVRS-13) was not continued to a residual strength condition because there is little particle reorientation, and thus pore-water pressure generation, at the residual condition so the constant volume ring shear test with a shear displacement rate of 0.18 mm/minute was stopped shortly after the peak shear stress was measured. There is little difference between the undrained peak strengths for the two different shear displacement rates so it was concluded that the peak strengths were not affected by shear displacement rate.

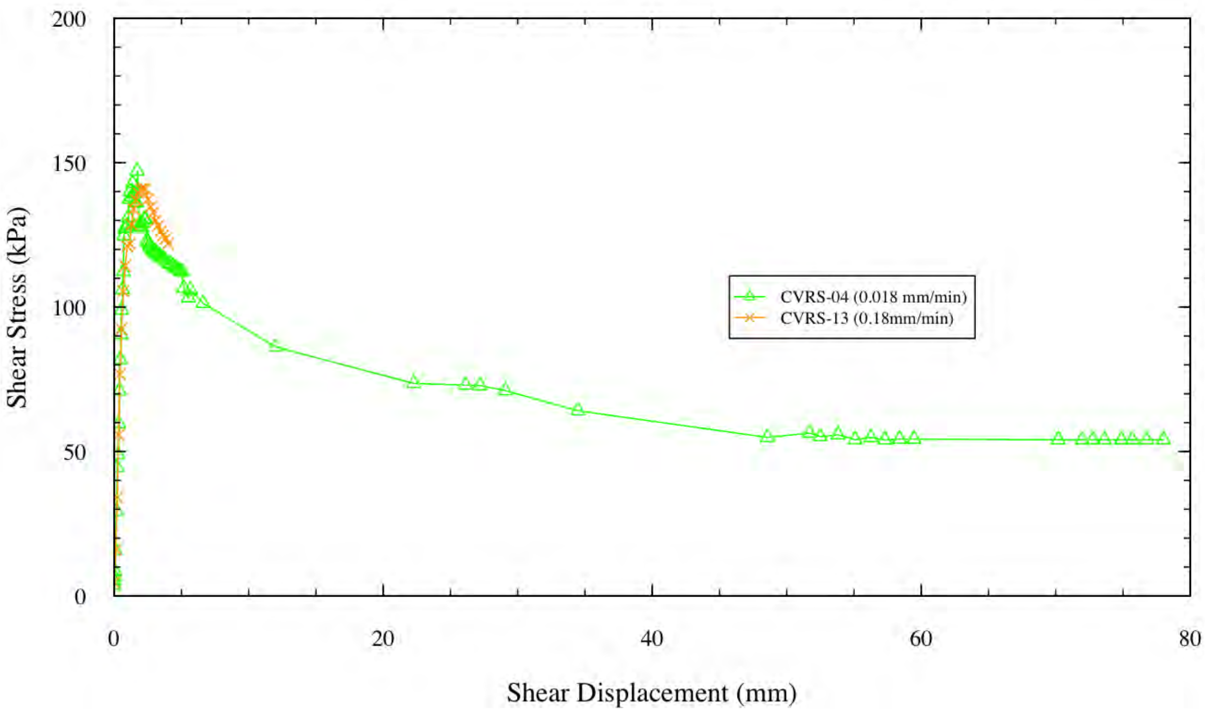

Figure 15. Undrained shear stress-displacement relationships from constant volume ring shear tests at different shear displacement rates on undisturbed Bootlegger Cove clay from sample tube ST-5

(e) Healed Constant Volume Ring Shear Test Results

Another series of constant volume ring shear tests were conducted to investigate the possibility of a preexisting shear surface in the Bootlegger Cove clay gaining strength with time as pore-water pressures dissipate and the soil "heals". This is important to evaluate the potential for and magnitude of earthquake induced displacements during subsequent ground motions or earthquakes. In other words, if a large earthquake does occur in the Anchorage area similar to the 1964 earthquake, the strength available to resist additional permanent displacements during aftershocks and subsequent earthquakes will be important.

A similar study was conducted by Stark and Hussain (2010) to investigate the possibility of a preexisting shear surface in landslides under static conditions. This study showed the recovered shear strength measured in drained laboratory ring shear tests is noticeably greater than the drained residual strength at effective normal stress of 100 kPa or less. The test results also show that the recovered strength even at effective normal stresses of 100 kPa or less is lost after a small shear displacement, i.e., slope movement. The brittle nature of the strength gain and an effective normal stress of 100 kPa corresponds to a depth of about 5 m indicates that this observed drained strength gain will have little, if any, impact on the analysis of deep landslides.

The constant volume ring shear test conducted herein differs from Stark and Hussain (2010) because it involves undrained strength gain with time and not drained strength. In this test, the undrained peak and residual strengths of the Bootlegger Cove clay from sample tube ST-5 were measured using the equipment and procedure described above. In other words, the test procedure used in CVRS-04 was used in CVRS-14 to measure the undrained peak and residual strengths of the Bootlegger Cove clay at an effective stress of 8,352 psf (400 kPa). Afterwards, the specimen was allowed to "heal". The healing process is accompanied by reapplication of the vertical stress to the specimen that was removed to maintain constant specimen volume and the level of excess pore-water pressures that could develop during seismic loading. After the specimen came to equilibrium under the re-applied normal stress to an original value of 8,352 psf (400 kPa).

Figure 16 shows the undrained shear stress-shear displacement relationships from these constant volume ring shear tests (CVRS-04 and CVRS-14) at a vertical consolidation stress of 8,352 psf (400 kPa). Figure 16 shows there is little difference in the undrained peak strengths (149 kPa versus 136 kPa) for these two tests, which suggests similar soil was trimmed from sample tube ST-5 for these two tests.

After reaching the undrained residual condition at a shear displacement of about 128 mm (see Figure 16), the pre-sheared specimen was allowed to dissipate excess pore-water pressures for about twenty-four hours. After about twenty-four hours, the normal stress that was removed to maintain a constant volume condition was reapplied to the specimen in four equal increments. After reapplying the increment of normal stress using the nut that is threaded to the top of the steel rod (see Figure 12), the specimen was allowed to equilibrate until no further vertical displacement was observed. This equilibration and reapplication of the vertical effective stress of 8,352 psf (400 kPa) required four to five days. Thus, the presheared specimen was healing during this four to five day time period. Afterwards, the pre-sheared specimen was sheared at the shear displacement rate of 0.018 mm/min and an effective normal stress of 8,352 psf (400 kPa) until the undrained residual shear strength was obtained again.

Figure 16 shows that upon restart of undrained shearing after the healing period, the specimen exhibited an undrained peak shear strength that is similar to the undisturbed undrained peak shear strength (136 kPa versus 137 kPa). After a shear displacement of about 164 mm, the "healed" specimen returned to about the same undrained residual strength as before the healing period.

The small difference in the healed and undisturbed peak shear strengths is significant because it suggests there is some healing of the preexisting shear surface in an undrained condition. This differs from the results of the drained healing ring shear tests reported by Stark and Hussain (2010) and may be beneficial for seismic performance during subsequent earthquakes.

To further investigate the undrained healing of the Bootlegger Cove from sample tube ST-5 and the sensitivity of the soil, another healing test was performed (CVRS-15) on a remolded specimen of the soil used for CVRS-14. The remolded specimen was obtained using the procedure described above and yielded an undrained peak strength of a normally consolidated specimen. As a result, the sensitivity of the Bootlegger Cove clay from sample tube ST-5 could be estimated by dividing the undrained peak shear strength (147 kPa) of the undisturbed specimen (CVRS-04) by the undrained peak shear strength (115 kPa) of the remolded specimen (CVRS-15). The undrained peak shear strengths of CVRS-04 and CVRS-15 were used to calculate a sensitivity of 1.3 which suggests a low sensitivity for this Facies IV, (F.IV) as defined by Updike and Carpenter (1986), of the Bootlegger Cove clay. Higher sensitivities have been reported for the Bootlegger Cove clay involved in the 1964 earthquake induced landslides, e.g., 3 to 11 for the Fourth Avenue landslide (Long and George, 1966 and Stark and Contreras, 1998).

Long and George (1966), Stark and Contreras (1998), Mitchell (1993), Terzaghi et al. (1996) define sensitivity as the undrained peak shear strength of an undisturbed specimen divided by the undrained peak shear strength of a remolded specimen. Stark and Contreras (1998) investigate the use of the field vane shear test to estimate the undrained peak and residual shear strengths for seismic stability evaluations because of the limited availability of the constant volume ring shear device. The undrained peak shear strength is estimated from the maximum torque generated at a vane shear rate of 0.1 degree/second (ASTM D2573; 2008). After measuring the maximum torque, the vane can be rotated a number of revolutions (3 to 25) to estimate the undrained residual strength. Stark and Contreras (1998) recommend ten (10) revolutions to measure the undrained residual strength and use this strength for analysis of the Fourth Avenue Landslide.

Figure 16 also shows the undrained shear stress-shear displacement relationship for CVRS-15 on a remolded specimen of the undisturbed material used for CVRS-14. The shear stress-shear displacement relationships are similar for CVRS-14 and CVRS-15 with CVRS-15 yielding a slightly lower undrained residual strength. The remolded specimen in CVRS-15 also yielded a lower healed peak strength (~115 versus ~130 kPa) after the same amount of healing (approximately four days). Regardless, both specimens indicate there is some healing of the preexisting shear surface in an undrained condition, which differs from the results of drained healing tests by Stark and Hussain (2010) and may be beneficial for seismic performance during subsequent earthquakes.

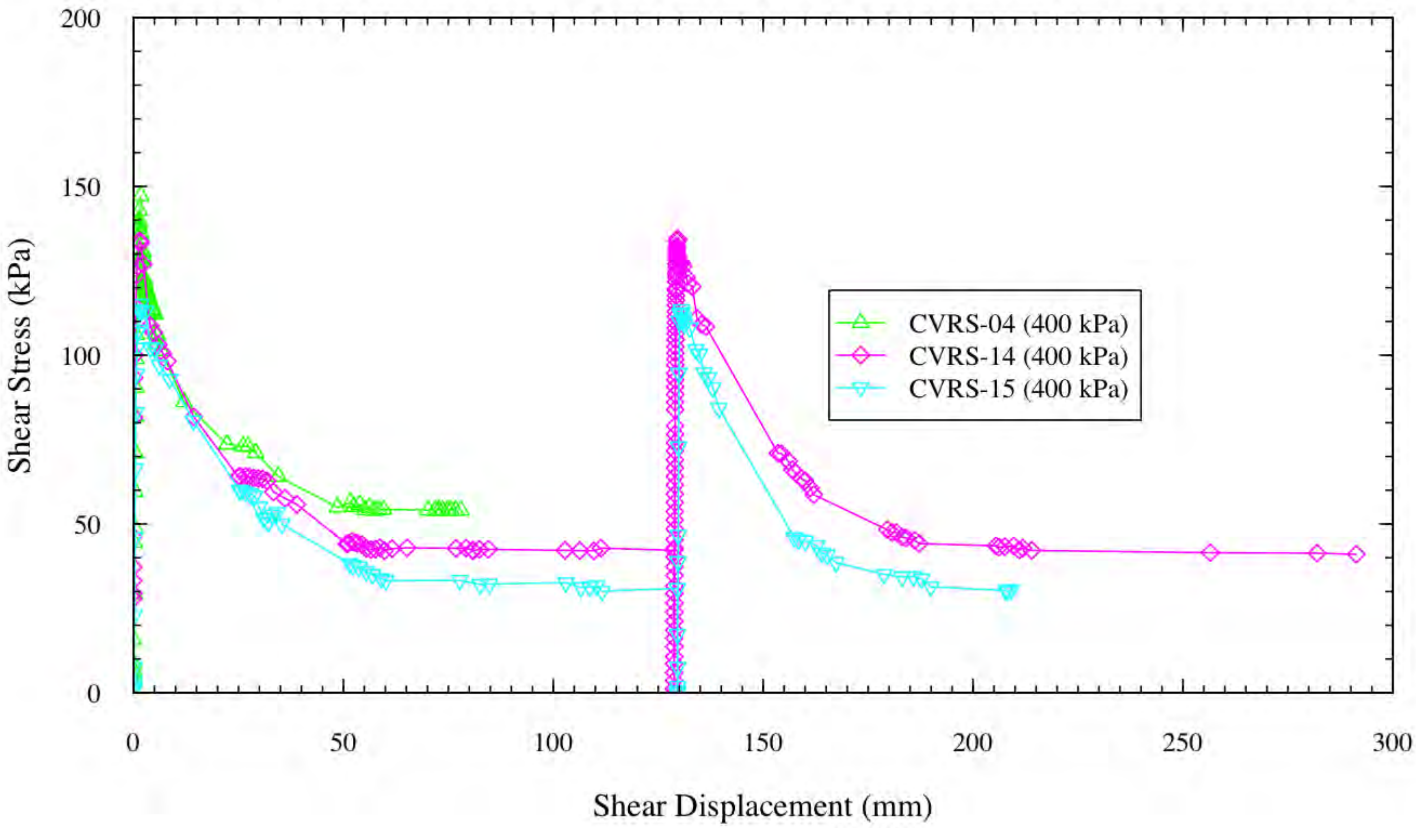

Figure 16. Undrained shear stress-displacement relationships from "healed" and "unhealed" constant volume ring shear tests on undisturbed and remolded Bootlegger Cove clay from sample tube ST-5

Summary

This report presents the results and analysis of index property and torsional ring shear tests conducted on the Bootlegger Cove clay from the Port of Anchorage (POA) Intermodal Expansion Project near Anchorage, Alaska. In particular, the results of index property and torsional ring shear testing performed on Bootlegger Cove clay taken from Boring Number BH-002-12 Sample Tube ST-5 are presented. Sample Tube ST-5 was obtained from a depth of 99.0 to 101.0 feet and is located in the Fourth (IV) Facies of the Bootlegger Cove clay.

The ring shear test results for Sample Tube ST-5 show this Bootlegger Cove clay is susceptible to a large undrained strength loss and development of an undrained residual strength condition under undrained shearing. This is in agreement with prior testing of Bootlegger Cove clay (Stark and Contreras, 1998; Stark, 2001) and other similar cohesive soils (see Table 1). The results indicate about a 20% reduction in undrained peak shear strength at a small displacement and about a 50 to 80% reduction if large shear displacements are induced. The relationship presented in Figure 6 can be used to estimate the percentage of the undrained peak shear strength that should be used to estimate the permanent lateral displacement. If permanent deformation exceeds 1.0 m, the undrained residual shear strength should be used for analysis purposes.

Limitations

The report has been prepared for CH2M-HILL (CH2M) for the static and seismic geotechnical evaluation of the Port of Anchorage Intermodal Expansion Project near Anchorage, Alaska as described above. This report is intended for the sole use of CH2M and its client. The scope of work performed during this study by CH2M was developed for purposes specifically intended by CH2M and may not satisfy other user's requirements or projects. Use of this report or the findings, conclusions or recommendations by others will be at the sole risk of the user.

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Sincerely yours,

TD Stark

Timothy D. Stark, Ph.D., P.E., F.ASCE, D.GE

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Appendix A – Stark and Contreras (1998)

FOURTH AVENUE LANDSLIDE DURING 1964 ALASKAN EARTHQUAKE

By Timothy D. Stark[1] and Iván A. Contreras[2]

ABSTRACT: This paper presents a reevaluation of the Fourth Avenue landslide in Anchorage that occurred during the 1964 Alaskan earthquake. Laboratory constant volume ring shear and field vane shear tests were used to measure the undrained peak and residual shear strength of the Bootlegger Cove clay. The results of these tests are presented and compared to back-calculated shear strengths of the Bootlegger Cove clay. The comparison shows that slide blocks that moved less than 0.15 m mobilized at least 80% of the undrained peak shear strength. Slide blocks that moved between 0.15 to 2.5 m mobilized an undrained shear strength between the peak and residual shear strengths. Slide blocks that displaced more than 2.5 m mobilized the undrained residual strength.

INTRODUCTION

A number of landslides involving cohesive soils have occurred during earthquakes. Some of the earthquakes in which landslides have occurred are the New Madrid earthquake of 1811, the Chilean earthquake of 1960, the Alaskan earthquake of 1964, and most recently the Saguenay earthquake of 1988. The most notable landslides are the Fourth Avenue, L-Street, Government Hill, and Turnagain Heights in Anchorage that were caused by the 1964 Alaska earthquake (Seed 1968; Idriss 1985) and the Sainte-Thècle and Saint-Adelphe landslides that occurred during the 1988 Saguenay earthquake (Lefebvre et al. 1992). It is anticipated that some of these slides were caused by an undrained failure and a postpeak strength loss in the cohesive soil involved in the slides. As a result, these landslides have lead to an interest in the seismic stability of cohesive soil slopes and therefore in the undrained peak and residual shear strength of cohesive soils.

FOURTH AVENUE LANDSLIDE

The Fourth Avenue landslide occurred during the great 1964 Alaska earthquake, which occurred at 5:36 p.m. local time on Friday, March 27, 1964. This earthquake had an epicenter approximately 130 km east of Anchorage. The earthquake was estimated to have a surface wave magnitude, M_s, and moment magnitude, M_w, of 8.5 and 9.2, respectively. The intensity in the Anchorage area was approximately VIII on the modified Mercalli scale. Based on patterns of damage to structures and their contents, the ground motion levels at Anchorage were estimated to be 0.15–0.20 g (Newmark 1965; Housner and Jennings 1964; Shannon and Wilson 1964). However, no accelerograms of the earthquake shaking were obtained. The duration of the ground motion in Anchorage was reported to range from four to seven minutes, with potentially damaging shaking lasting approximately two to three minutes (Housner and Jennings 1964). Ground fissuring and numerous small slope failures were reported, and five large translatory slides occurred in the Anchorage area. Although not the largest, the Fourth Avenue landslide is representative of the translatory failure type and drew considerable attention because of its downtown location (Fig. 1).

The Fourth Avenue slide was 487 m long and 275 m wide between Fourth and First Avenues and between E Street and slightly east of A Street (Fig. 2). The slide mechanism was primarily horizontal translation, which is characterized by lateral spreading and graben development. Two grabens were created east of C Street and another graben had begun to form between Fourth and Fifth Avenues at D and E Streets (Wilson 1967). The greatest damage to structures developed within and adjacent to the grabens, and along the pressure ridge that developed at the toe of the slide between First and Second Avenues. In contrast, little damage was suffered by buildings and streets that were located on the sliding mass.

This area of Anchorage was known, even before the 1964 earthquake, as an area in which large landslides occurred (Shannon and Wilson 1964). However, the triggering mechanism of these pre-1964 slides is not known. Miller and Dobrovolny (1959) had previously described the Bootlegger Cove clay in this area as susceptible to failure during earthquakes.

Based on the investigation of the landslide conducted shortly after the earthquake, the zone of shearing was estimated to occur between elevations +13.7 and +10.6 m, according to the graben rule (Hansen 1965). The graben rule consists of equating the cross-sectional area of the graben trough to the cross-sectional area of the space voided behind the block as the block moves outward.

This elevation is at or near the interface of discontinuous sandy layers and the underlying slightly overconsolidated clay of the Bootlegger Cove formation. At the time of the slide it was not clear whether the slide occurred as a result of liquefaction of the sandy layers or undrained failure of the slightly overconsolidated Bootlegger Cove clay. This uncertainty regarding the failure mechanism was in part caused by the relatively limited data, particularly on the discontinuous sandy soils, available at that time. As a result, two failure mechanisms were proposed in the resulting literature: liquefaction of sand seams (e.g., Seed 1968) and undrained failure of the slightly overconsolidated clay (e.g., Long and George 1966; Bjerrum 1964, unpublished manuscript).

A comprehensive study of the landslide was conducted by Woodward-Clyde Consultants (1982) as part of an evaluation of the Fourth Avenue area for development of a major state office complex. These researchers concluded that the slide was not caused by liquefaction of sand seams but by a large undrained strength loss in the slightly overconsolidated clay of the Bootlegger Cove formation (Woodward-Clyde 1982; Idriss 1985; Updike et al. 1988). However, a laboratory or field apparatus that could measure the undrained postpeak strength loss of the cohesive soil was not available during these studies to confirm this hypothesis.

Recently, the authors (Stark and Contreras 1996) developed a laboratory constant volume ring shear apparatus that allows the measurement of the magnitude and rate of undrained postpeak strength loss and the undrained residual shear strength in cohesive soils. This paper presents a reevaluation of the Fourth Avenue slide based on laboratory ring shear and field vane shear test results to determine the mobilized undrained shear strength and recommendations for evaluating the seismic stability of slopes in sensitive cohesive soils.

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FIG. 1. View of Damage and Scarp in Fourth Avenue Landslide

SUBSURFACE CONDITIONS IN FOURTH AVENUE AREA

The subsurface conditions in the Fourth Avenue slide area are illustrated in Fig. 3, which is a cross section along D Street (Fig. 2) where the largest lateral movement occurred in 1964. Typical standard penetration test (SPT) blow count and cone penetration test (CPT) profiles in the Fourth Avenue area are presented by Idriss (1985). The geologic profile along D Street can be summarized as follows:

(1) The upper deposit consists of very dense sands and gravels (Naptowne Outwash). The thickness of the Naptowne Outwash generally ranges from 7.5 to 12.0 m. The uncorrected SPT blow counts and CPT tip resistance in the outwash are typically greater than 70 blows and 40 MPa, respectively.

(2) Underlying the Naptowne Outwash are deposits of stiff clay and layered sand to a thickness of approximately 10 m. The layered sand in this zone is dense to very dense silty fine sand with alternating layers of clay and sandy silt. During this investigation, these dense layers caused crushing of three 125 mm diameter Shelby tubes during sampling. The median grain size of the silty fine sand and sandy silt ranges from 0.1 to 0.3 mm and 0.06 to 0.12 mm, respectively (Idriss 1985). The uncorrected SPT blow counts and CPT tip resistance measured after the earthquake range from 29 to 90 blows and 20 to 42 MPa, respectively. The thin, discontinuous, cohesionless seams found in this zone are probably the result of transport and reworking of the sediments within the glacial lake leading to the deposition of fine grained sands and silts. This type of transport and depositional environment can lead to dense packing and thus to relatively high densities in the cohesionless strata (Idriss 1985). The clays in this zone are stiff to very stiff, probably because of desiccation (Updike et al. 1988). An overconsolidation ratio (OCR)—the preconsolidation pressure (σ'_p) divided by the effective vertical overburden pressure (σ'_{vo})—of 3 to 4 was estimated for these stiff clays (Shannon and Wilson 1964; Woodward-Clyde 1982).

(3) Below the layered sand is a slightly overconsolidated, sensitive clay of nearly uniform texture, which displays planar bedding. The clay belongs to the Bootlegger Cove clay formation and contains extremely thin, discontinuous seams of silty fine sand. The sensitivity of the clay in the Fourth Avenue area ranges from 3 to 11. This slightly overconsolidated clay exhibits an OCR of 1.2 and 1.6 inside and outside of the slide mass, respectively. The clay exhibits a plasticity index between 7 and 22 with an average of 14 and a plastic limit between 20 and 30 with an average of 25.

LIQUEFACTION ASSESSMENT

Since the zone of shearing was located near the contact between the layered zone and the slightly overconsolidated, sensitive Bootlegger Cove clay, the liquefaction potential of the cohesionless material at this depth was evaluated using SPT and CPT results. In both cases, the average seismic shear stress ratio was computed using the water table and average subsurface conditions (Fig. 3) in the slide area and a peak ground surface acceleration of 0.2 g.

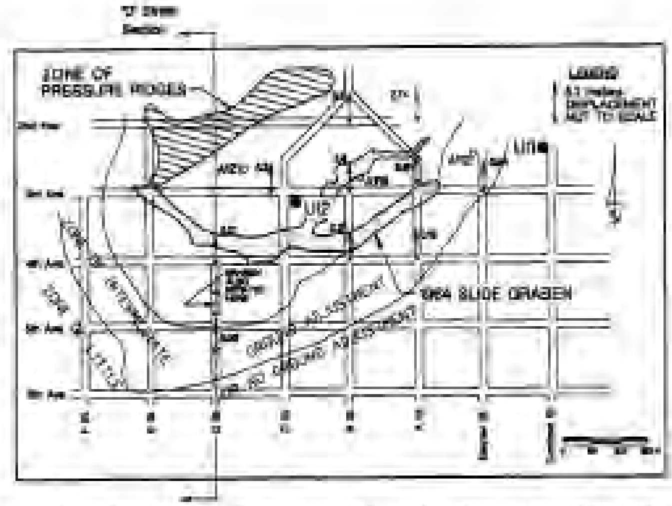

FIG. 2. Ground Breakage and Cross Section in Fourth Avenue Slide Area (After Shannon and Wilson 1964)

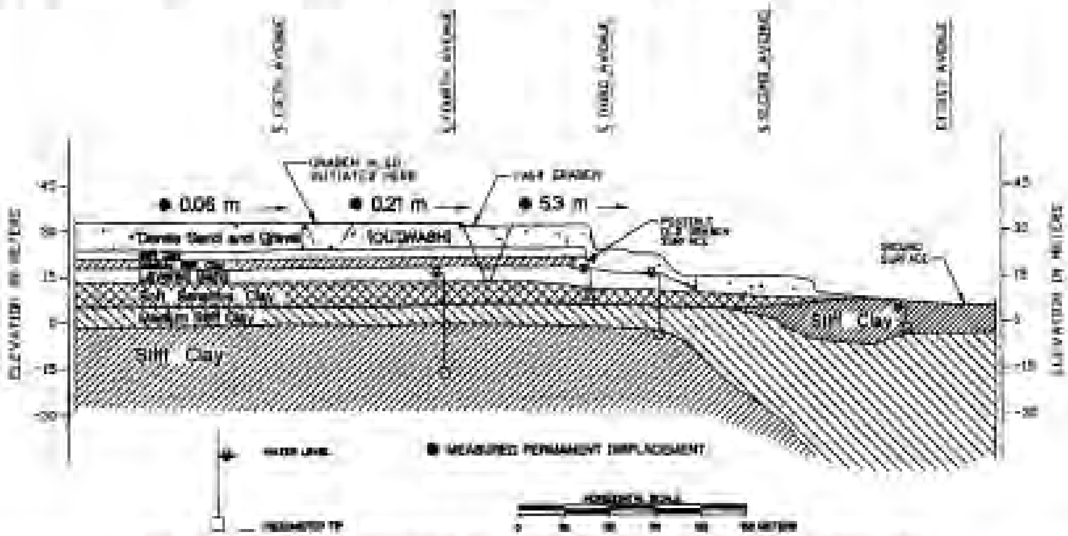

FIG. 3. Cross-section along D Street in Fourth Avenue Landslide (Shannon and Wilson 1964)

Liquefaction Assessment Using SPT

Based on SPT data, one of the main findings of the Woodward-Clyde (1982) study was that liquefaction of the sand and silt layers was probably not the cause of the Fourth Avenue landslide. The 1982 data suggest that the cohesionless materials are not liquefiable and that they exhibit a minimum factor of safety against liquefaction of approximately 1.6 (Idriss 1985).

Liquefaction Assessment Using CPT

The CPT data presented by Woodward-Clyde (1982) and the procedure developed by Stark and Olson (1995) were used to evaluate liquefaction potential of the sand and silt layers. Fig. 4 shows that the values of q_{c1} lie to the right of the boundary line that separates liquefied and nonliquefied sites for an earthquake moment magnitude of 9.2. These data were used to estimate a minimum factor of safety against liquefaction of approximately 1.5. This agrees with the liquefaction assessment using SPT blow count data. As a result, it was concluded that the sandy silt or silty sand layers in the layered sand deposit did not liquefy during the 1964 earthquake and therefore are not the cause of the Fourth Avenue slide. This reinforces the conclusion presented by Woodward-Clyde (1982) and Idriss (1985) based on SPT results.

Generation of Excess Pore-Water Pressure in Cohesionless Materials

Since liquefaction was unlikely in the layered sand zone, the possibility of excess pore-water pressures developing in the silt and sand seams due to earthquake shaking was evaluated during the present study. Positive excess pore-water

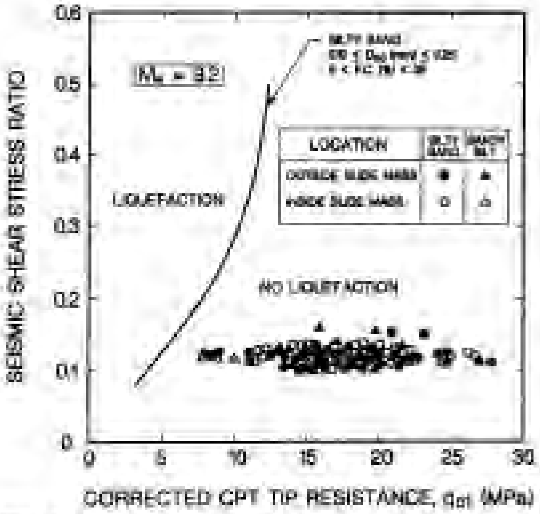

FIG. 4. Liquefaction Potential in Sandy Strata in Interbedded Zone during 1964 Alaska Earthquake

pressures could have aided the landslide by reducing the effective stress at the interface with the Bootlegger Cove clay. The computed excess pore-water pressure ratios necessary to cause failure of the sliding blocks along the bluff varies from 0.62 to 0.93.

Ishihara (1985) and Seed and Harder (1990) present relationships between the excess pore-water pressure ratio and the factor of safety against liquefaction. These relationships and a minimum factor of safety of 1.5–1.6, were used to estimate a maximum excess pore-water pressure ratio of 0.1 to 0.2. This maximum excess pore-water pressure ratio is significantly lower than the computed values of 0.62–0.93 necessary to cause failure. It is anticipated that this maximum excess pore-water pressure ratio did not cause a significant reduction in the effective stress at the sand/Bootlegger Cove clay interface and thus is not believed to have contributed significantly to the failure. Therefore, the cause of the slide appears to be an undrained failure of the soft Bootlegger Cove clay.

Bjerrum (1964), Kerr and Drew (1965), Hansen (1965), and Long and George (1966) also associated the failure of the 1964 slides with an undrained failure of the slightly overconsolidated Bootlegger Cove clay. For example, Bjerrum (1964) wrote in reference to the Fourth Avenue failure mechanism, "Such a movement, occurring on a nearly horizontal sliding surface, cannot be visualized if the sliding surface was located in sand. Very similar movements are however known from the Scandinavian slides where the sliding surface is positively known to be located in sensitive clay."

UNDRAINED PEAK SHEAR STRENGTH OF BOOTLEGGER COVE CLAY

Triaxial compression (TC) tests conducted by Shannon and Wilson (1964) indicate that the undrained peak shear strength ratio in the Bootlegger Cove clay ranges from 0.26 to 0.37. The undrained peak shear strength ratio is defined as the undrained shear strength, s_u, divided by the vertical consolidation stress, σ'_{vc}. The value of σ'_{vc} is used in the normalization process because the specimens were tested in the normally consolidated range. Similar undrained peak shear strength ratios, 0.27–0.28, were measured in the triaxial compression tests conducted by Woodward-Clyde (1982). These data agree with the average relationships between undrained peak shear strength ratio and plasticity index proposed by Jamiolkowski et al. (1985) and most recently Terzaghi et al. (1996).

As indicated earlier, the slide mechanism was primarily horizontal translation. As a result, the direct simple shear (DSS) apparatus probably allows the closest laboratory simulation of the stresses and deformations imposed by the earthquake shaking and translational sliding on soil elements in the field. The undrained peak shear strength ratio from the DSS tests conducted by Woodward-Clyde (1982) varies from 0.18 to 0.24 with an average of 0.20. These undrained peak shear strength data were obtained from monotonic tests at standard shear rates, and therefore are appropriate for static undrained loading conditions. Changes to this undrained shear strength can result from excess pore-water pressures induced by cyclic loading and/or large translatory displacements during slide movement.

Undrained Strength Loss Caused by Cyclic Loading

The potential for undrained strength loss caused by excess pore-water pressures induced by cyclic loading on the Bootlegger Cove clay was investigated by Woodward-Clyde (1982) using cyclic direct simple shear tests followed by a post-cyclic static test. These data indicate that even when high excess pore-water pressures are induced in the specimen by cyclic loading imposed by an earthquake moment magnitude of 9.2, the post-cyclic undrained shear strength is greater than 80% of the pre-cyclic peak shear strength.

Lade et al. (1988) also conducted cyclic triaxial compression tests on undisturbed specimens of the Bootlegger Cove clay and showed that the ratio of cyclic to static undrained shear strength is greater than unity. As a result, it is concluded that cyclic loading and generation of excess pore-water pressures does not significantly reduce the shear strength of the Bootlegger Cove clay. In fact, lengthy cyclic loading appears to result in a larger reduction in shear modulus than undrained peak shear strength (Vucetic and Dobry 1991).

Undrained Strength Loss Caused by Translatory Displacement

Undrained postpeak strength loss can occur in slightly overconsolidated clays because of large shear deformation. The deformation causes a collapse of the soil structure, which is accompanied by generation of excess pore-water pressures and orientation of some clay particles parallel to the direction of shear. This generation of excess pore-water pressure results in a decrease in the effective stress. If sufficient deformation occurs during an earthquake, a residual shear strength condition may be achieved under undrained conditions (Idriss 1985).

CONSTANT VOLUME RING SHEAR TEST

Taylor (1952) introduced the use of a constant volume shear test to measure the undrained peak shear strength. A modified direct shear apparatus was used by Taylor (1952) to perform constant volume tests on Boston Blue Clay. Bjerrum and Landva (1966) introduced the use of the DSS apparatus to measure the undrained peak shear strength of Manglerud clay. However, undrained triaxial, direct shear, and direct simple shear apparatuses are not suitable to estimating the undrained residual strength because only a limited amount of continuous shear displacement can be imposed along a failure surface. As a result, it was necessary to develop a torsional ring shear apparatus to evaluate the undrained postpeak strength loss and residual strength.

Constant Volume Ring Shear Apparatus

To measure the undrained peak and residual shear strengths, the original Bromhead (1979) ring shear apparatus was modified to conduct constant volume tests. The modifications include: a mechanism for adjusting the normal stress during shear, such that the volume change is negligible during shear, and fabricating a new specimen container to allow undisturbed specimens to be trimmed directly into the container. The ring shear specimen is annular with an inside diameter of 70 mm and an outside diameter of 100 mm. The specimen is confined radially by the specimen container, which is 10 mm deep. The porous stone is serrated to prevent slippage at the loading platen/soil interface during shear.

The constant volume mechanism consists of a steel rod that is connected to the end of the horizontal beam that applies the normal stress to the top of the specimen. Attached to the steel rod is a load cell. A nut is threaded to the top of the steel rod and is adjusted during shear to reduce the amount of normal load transferred to the loading platen. The nut connected to the steel rod is used in combination with the vertical dial gauge to adjust the normal stress, such that the specimen thickness remains constant during shear. Adjustments are made by manually rotating the nut as far as is required to maintain zero vertical displacement of the soil specimen. When the nut is rotated, a portion of the dead weight applied to the horizontal beam is transmitted to the rod and the load cell indicates the magnitude of this load. Additional details of the constant volume ring shear apparatus are presented in another paper by the authors (Stark and Contreras 1996).

Constant Volume Ring Shear Test Procedure

In the constant volume ring shear apparatus, the specimen is sheared by rotating the specimen or specimen container past the stationary loading platen at a drained constant rate less than or equal to 0.018 mm/min. The procedure described by Gibson and Henkel (1954) is used to determine the shear displacement rate that results in zero pore-water pressure throughout the specimen. A drained shear displacement rate is used so that little, if any, excess pore-water pressure is induced during shear. The Bootlegger Cove clay involved in the Fourth Avenue slide is contractive during shear, and therefore the normal stress is reduced during shear to maintain a constant specimen height or volume. It is assumed that the decrease in applied vertical stress during shear is equivalent to the increase in shear-induced pore-water pressure that would occur in an undrained test with constant vertical stress. The validity of this pore-water pressure assumption for constant volume tests was verified by Dyvik et al. (1987) for the direct simple shear apparatus and by Berre (1981) for the triaxial compression apparatus.

Sampling during This Investigation

Undisturbed samples of Bootlegger Cove clay were obtained in and adjacent to the Fourth Avenue slide mass during the 1991 summer. The samples were obtained using 125 mm diameter thin-walled Shelby tubes. One boring was located in the area where the largest movement occurred (UI2) and another boring (UI1) was located outside of the slide mass (Fig. 2).

A total of four samples were obtained from inside the slide mass (boring UI2) between elevations +13.7 and 10.1 m. These elevations are at or near the bottom of the layered sand zone and the top of the slightly overconsolidated Bootlegger Cove clay, which includes the elevation of the 1964 sliding surface (+13.7 to +10.6 m). The natural water content of the Bootlegger Cove clay at the depth of sliding ranges from 28 to 38%. The corresponding liquid limit, plasticity index, and clay size fraction are 38, 18, and 55%, respectively. The preconsolidation pressure of the clay from a series of oedometer tests was estimated to range from 280 to 320 kPa. The best estimate of effective overburden pressure from the boring log is 230 kPa. Therefore, the OCR of the Bootlegger Cove clay inside the slide mass is approximately 1.2 to 1.4. This OCR is within the range of values (1.2 to 1.5) reported by Shannon and Wilson (1964) and Woodward-Clyde (1982).

Constant Volume Ring Shear Test Results

Fig. 5 shows the shear stress-shear displacement relationships from constant volume ring shear tests on undisturbed specimens of Bootlegger Cove clay. Vertical consolidation stresses of 100, 230, 300, 400, and 500 kPa were used in the tests on the Bootlegger Cove clay from inside the slide mass (Boring UI2) at the depth of the 1964 sliding surface.

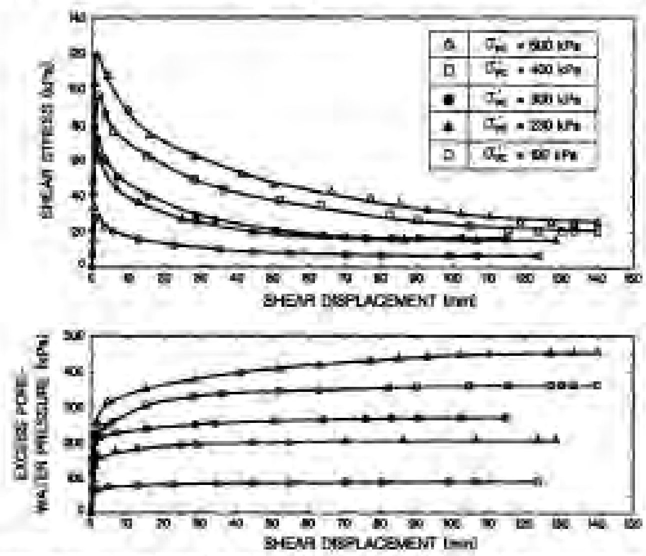

FIG. 5. Constant Volume Ring Shear Tests on Bootlegger Cove Clay from Inside Fourth Avenue Landslide

The peak shear strength was reached after approximately 1–2 mm of shear displacement. After the undrained peak strength was mobilized, shear displacement continued along the failure surface and the measured shear stress decreased with increasing displacement. Shear displacement along the failure surface causes an increase in pore-water pressure and presumably some orientation of soil particles parallel to the direction of shear. As a result, the normal stress is reduced to maintain a constant volume. This continues until the undrained residual strength is mobilized, usually at a shear displacement of approximately 80–100 mm. The undrained residual shear stresses correspond to an undrained residual strength ratio of approximately 0.06. The undrained residual strength ratio estimated from laboratory data is defined as the undrained residual strength divided by the vertical consolidation stress even at stresses less than the preconsolidation pressure. The vertical consolidation stress is used in the recompression and compression ranges because the effect of overconsolidation has been removed before the undrained residual condition has been reached.

Fig. 5 also presents the excess pore-water pressure-shear displacement relationships measured during the constant volume ring shear tests on the undisturbed Bootlegger Cove clay. The excess pore-water pressure is assumed to be equal to the decrease in normal stress. The excess pore-water pressure increases and then becomes essentially constant at the undrained residual condition.

Table 1 presents a summary of the natural soils that have been tested using the constant volume ring shear apparatus. These natural soils exhibit a limited range of plasticity and clay size fraction and therefore similar peak and residual undrained shear strengths ratios. (Remolded specimens of Upper Bonneville clay were used and therefore the vertical consolidation stresses equal the preconsolidation pressures.) In addition, the shear displacement required to mobilize the peak and residual strength conditions are in agreement. Therefore, the constant volume ring shear tests on the Bootlegger Cove clay appear to be consistent with other natural soils of similar plasticity and clay size fraction.

Fig. 6 presents the variation of undrained peak and residual shear strength from Fig. 5 with vertical consolidation pressure for samples inside of the slide mass. It can be seen from Fig. 6 that the undrained peak shear strength increases with consolidation stress. In the recompression range from σ'_{vo} to σ'_p, the undrained peak shear stress increases slightly, whereas in the normally consolidated range, beyond σ'_p, the undrained peak shear strength increases linearly with σ'_{vc}.

The undrained peak shear strength ratio is defined as the undrained peak shear strength divided by the in situ preconsolidation pressure at stress levels less than the preconsolidation pressure and divided by the laboratory vertical consolidation pressure at stress levels greater than the in situ preconsolidation pressure (i.e., in the normally consolidated range). In the recompression or overconsolidated range, the undrained peak shear strength ratio, s_u/σ'_p, increases slightly from 0.17 to 0.19 (see Fig. 6). The undrained peak shear strength ratio, s_u/σ'_{vc}, in the normally consolidated range is 0.23. Therefore, the ring shear data exhibits an undrained peak strength ratio in the recompression, s_u/σ'_p, and compression, s_u/σ'_{vc}, ranges from 0.17 to 0.23. This is similar to the range of undrained peak strength ratio measured using normally consolidated specimens, 0.18–0.24, in DSS tests conducted by Woodward-Clyde (1982).

BACK ANALYSIS OF FOURTH AVENUE LANDSLIDE

Regressive analyses of the Fourth Avenue slide were performed to estimate the mobilized undrained strength ratios that correspond to the permanent displacements observed after the earthquake. A similar approach was used by Woodward-Clyde (1982) and Idriss (1985). The method is based on Newmark's (1965) sliding block model as augmented by Makdisi and Seed (1978) and involves a rigid block acted upon by the following forces: (1) driving force due to earthquake inertia, F_{EQ}; (2)

TABLE 1. Summary of Constant Volume Ring Shear Test Results

Soil deposit and location (1)	Liquid limit (%) (2)	Plastic limit (%) (3)	Clay size fraction (% < 0.002 m) (4)	Vertical consolidation stress σ'_{vc} (kPa) (5)	Preconsolidation pressure σ'_p (kPa) (6)	Undrained peak shear strength ratio s_u/σ'_{vc} (7)	Shear displacement at peak strength ratio (mm) (8)	Undrained residual strength ratio s_{ur}/σ'_{vc} (9)	Shear displacement at residual strength ratio (mm) (10)
Bootlegger Cove clay,	40	20	39	100	230–330	0.28	3.2	0.07	55
inside Fourth Ave.	34	19	37	200		0.28	3.1	0.07	75
landslide, Anchorage,	39	21	36	300		0.24	3.3	0.06	75
Alaska	38	21	35	400		0.23	3.6	0.06	120
	39	20	42	500		0.23	3.6	0.06	130
Drammen clay,	47	23	70	95	140	0.27	1.1	0.09	19
Danvikgate,	48	24	72	235		0.22	1.3	0.11	16
Drammen, Norway	47	25	65	400		0.20	1.1	0.11	50
Bootlegger Cove clay,	42	23	47	150	405	0.31	1.5	0.11	45
outside Fourth Ave.	40	21	46	225		0.32	1.6	0.10	110
landslide, Anchorage,	42	23	49	400		0.31	1.7	0.11	125
Alaska	41	22	45	500		0.30	1.7	0.13	140
Cohesive alluvium,	30	22	19	95.8	122.4	0.19	2.2	0.10	32
Enid Dam,	28	22	20	147	138.9	0.37	1.1	0.05	77
Enid, Mississippi	23	19	17	151	81.4	0.24	1.1	0.07	75
	25	22	20	287	143.5	0.23	1.2	0.07	72
	30	22	20	383	134.5	0.23	1.3	0.06	75
Cohesive alluvium,	59	31	51	51.8	75.8	0.24	0.50	0.13	38
Jackson, Alabama				79.4		0.22	0.35	0.16	50
				100		0.25	0.37	0.18	38
Upper Bonneville clay,	46	23	33	47.9	47.9	0.32	0.30	0.11	33
Salt Lake City, Utah				95.8	95.8	0.36	0.60	0.15	25
				191.5	191.5	0.31	1.3	0.12	29
				383	383	0.34	2.6	0.14	35

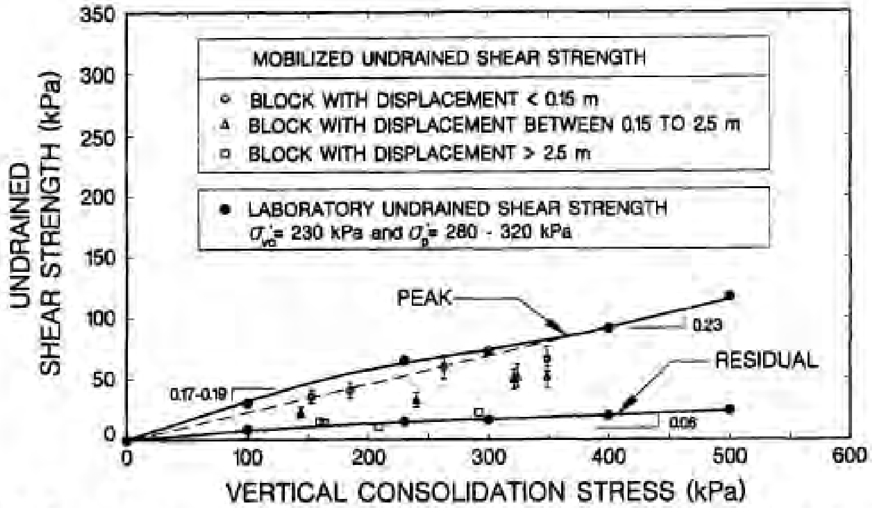

FIG. 6. Comparison between Laboratory and Back-Calculated Undrained Shear Strength for Bootlegger Cove Clay

resisting force due to soil shear strength, F_{RS}; (3) active driving force due to active soil pressure, F_{DA}; and (4) driving, F_{DG}, or resisting, F_{RG}, force due to the presence of a graben (Fig. 7).

The earthquake inertia force acting on the block is calculated by multiplying the total weight of the soil block, W, by the maximum seismic coefficient. The maximum seismic coefficient, K_{max}, is the peak ground acceleration at the base of the soil block. The resisting force, F_{RS}, corresponds to the soil shear strength acting along the bottom of the soil block and is computed by multiplying the length of the soil block, L, by the average undrained shear strength of the soil involved. The active soil force is computed using the unit weight of soil, height of the soil block, and the dynamic active earth pressure coefficient developed by Okabe (1926).

Due to the presence of a graben, F_{DG} and F_{RG} are the driving and resisting forces, respectively. They represent the horizontal component of the shearing force developed between the sliding block and the graben when the graben descends in the void created by block movement. The shearing force between the block and the graben is computed using the graben inertia force, the weight of the graben, and the inclination of the shearing plane between the graben and the block.

The seismic yield coefficient, K_y, is the seismic coefficient, which, when multiplied by the total weight of the block, yields the minimum earthquake driving force required to fully mobilize the shear strength of the soil along the slide surface.

Once values of K_y and K_{max} are known, the displacement of the soil block can be computed. Makdisi and Seed (1978) present a graphical relationship between the magnitude of permanent displacement and the ratio of K_y and K_{max} for various earthquake surface wave magnitudes less than or equal to 8.25. Because an earthquake moment magnitude of 9.2 had to be considered in the reevaluation of the 1964 Fourth Avenue slide, Makdisi and Seed's (1978) results were extrapolated to a moment magnitude of 9.2 by Idriss (1985).

Cross sections along F, D, B, A, and Barrow Streets (Fig. 2) were analyzed during this study. The D Street cross section and values of permanent displacement observed during the 1964 event are shown in Fig. 3. In each cross section the permanent deformation was greatest for sliding blocks along the bluff and the deformation gradually decreased for sliding blocks away from the bluff line.

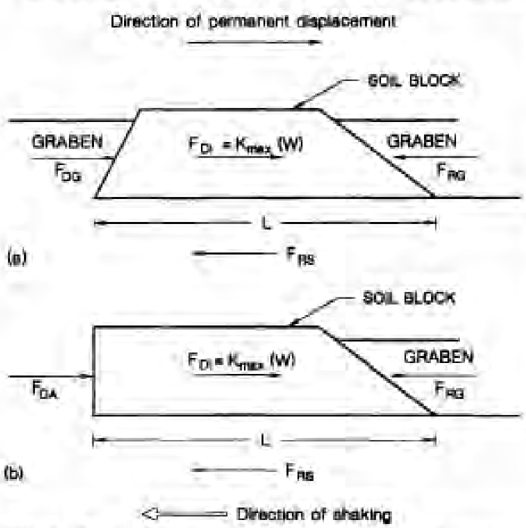

FIG. 7. Forces Used in Analysis for Calculating Permanent Lateral Displacement Due to Earthquake Ground Motions

Estimation of Mobilized Undrained Shear Strength

The process of calculating the mobilized undrained shear strength that corresponds to the observed displacements, s_u(mob), involves using K_{max} and the following expressions:

$$s_u(\text{mob}) = \frac{(K_y/K_{max}) \times W \times K_{max} + F_{DG} - F_{RG}}{L} \qquad (1)$$

$$s_u(\text{mob}) = \frac{(K_y/K_{max}) \times W \times K_{max} + F_{DA} - F_{RG}}{L} \qquad (2)$$

where (1) and (2) correspond to Figs. 7(a) and (b), respectively. These expressions yield a value of undrained shear strength per unit width of the slide mass. Because a range of

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TABLE 2. Summary of Regressive Displacement Analysis of Fourth Avenue Landslide

Cross section (1)	Observed surface displacement[a] (m) (2)	Block weight (kN) (3)	F_{DA} (kN) (4)	F_{DG} (kN) (5)	F_{RG} (kN) (6)	Block length (m) (7)	K_y/K_{max} (8)	σ'_{vo} (kPa) (9)	σ'_p (kPa) (10)	s_u(mob) (kPa) (11)	s_u(mob)/σ'_p (kPa) (12)
F-Street	0.06	38,981	3,065	N/G[b]	N/G	107	0.72–0.80	263	316	50–69	0.16–0.22
	0.15	29,374	1,068	N/G	N/G	110.6	0.62–0.72	185	222	34–47	0.15–0.21
	0.21	22,767	1,167	N/G	N/G	126	0.58–0.68	144	173	18–27	0.10–0.16
D-Street	0.06	43,624	1,372	N/G	N/G	102	0.72–0.80	349	411	56–76	0.14–0.19
	0.21	40,879	1,424	N/G	379	109	0.58–0.68	349	411	42–60	0.10–0.15
	5.3	39,944	N/A	1,485	N/G	217.6	0.17–0.27	161	193	11.4–17	0.06–0.08
B-Street	0.21	41,142	1,313	N/G	212	117.5	0.58–0.68	232	387	41–58	0.10–0.15
	3.4	31,798	N/A	1,462	90	119.5	0.21–0.32	292	350	20–28	0.06–0.06
	5.8	20,790	N/A	826	N/G	117.5	0.14–0.24	209	250	11–15	0.04–0.06
A-Street	0.18	20,118	1,413	N/G	319	117.8	0.58–0.69	324	388	43–62	0.11–0.16
	0.61	23,330	N/A	1,014	242	85.3	0.44–0.55	240	288	27–39	0.09–0.13
	2.74	16,877	N/A	352	N/G	89.3	0.25–0.36	160	192	11–17	0.057–0.09
Barrow Street	<0.04	25,315	692	N/G	N/G	114.3	0.72–0.80	153	183	30–41	0.16–0.22

[a]From Shannon and Wilson (1964).
[b]N/G = no graben present.

(K_y/K_{max}) values is used in (1) and (2), a range of mobilized undrained shear strength is computed. Table 2 summarizes the results of the back analysis of the Fourth Avenue slide. The preconsolidation pressures in Table 2 were estimated using a value of OCR equal to 1.2, which was obtained during this study and reported by Shannon and Wilson (1964) and Woodward-Clyde (1982), and average values of effective overburden pressure at the time of the slide. Table 2 shows that for slide blocks that displaced less than 0.15 m, the back-calculated undrained strength ratio ranges from 0.16 to 0.22. For blocks that moved between 0.15 and 2.5 m the back-calculated undrained shear strength ratio ranges from 0.19 to 0.15, respectively. For blocks that displaced greater than 2.5 m, the back-calculated undrained strength ratio ranges between 0.05 and 0.08 with an average of 0.06.

Comparison of Laboratory and Back-Calculated Undrained Shear Strengths

Fig. 6 presents a comparison of the laboratory and back-calculated undrained shear strengths. For blocks that were just initiating movement (less than 0.15 m), three of the four back-calculated undrained shear strengths are slightly lower (e.g., 10–20%) than the laboratory undrained peak shear strength measured in the constant volume ring shear apparatus. These three data points are in the recompression range, i.e., vertical stress less than 280–320 kPa. The fourth data point, at σ'_{vc} = 350 kPa, is in the normally consolidated range and is lower than an s_u/σ'_{vc} ratio equal to 0.23. These four data points also suggest that the shear strength reduction caused by cycling loading is small in the Bootlegger Cove clay.

For blocks that moved between 0.15 and 2.5 m, the range of back calculated undrained shear strength is between the undrained peak and residual shear strengths. This indicates that the undrained post-peak strength loss from the peak to residual shear strength, occurred within this range of observed ground surface displacement.

For blocks that moved greater than 2.5 m, the range of back-calculated undrained shear strength is in excellent agreement with the undrained residual shear strength measured using the constant volume ring shear apparatus. This indicates that for blocks that underwent large lateral movement, the shear strength was reduced to the undrained residual strength. As a result, if sliding is triggered by earthquake shaking in the Anchorage area and permanent ground surface displacement is estimated to exceed 2.5 m, the undrained residual shear strength may be mobilized.

A major unknown in analysis/design of slopes subjected to earthquake shaking is the magnitude of undrained shear strength that should be used to estimate the earthquake-induced permanent deformations. Data from Table 2 were used to develop Fig. 8, which provides some guidance for estimating the mobilized undrained shear strength ratio for the three Alaska landslides described in this study. The data in Table 2 were supplemented using data from the L-Street and Government Hill landslides, which are described subsequently. The L-Street slide exhibited a similar failure mechanism to the Fourth Avenue slide (Moriwaki et al. 1989) and the back-calculated undrained shear strengths agree with Fourth Avenue. The Government Hill slide also involved horizontal translation and thus exhibited a similar failure mechanism to the Fourth Avenue slide.

Fig. 8 shows that a strength ratio of approximately 0.18 is mobilized at a surface displacement of 0.15 m. It also can be seen that an undrained residual strength ratio of approximately 0.07 is mobilized at a surface displacement greater than 2.5 m. Unfortunately, there is only one data point between ground displacements of 0.6 and 2.5 m, which does not allow a better definition of the relationship in this range of displacement. However, this indicates that the transition from mobilized peak to residual occurs between a displacement of 0.6 and 2.5 m because mobilization of a residual strength condition resulted in a displacement greater than approximately 2.5 m. One reason for a lack of data between a displacement of 0.6 and 2.5

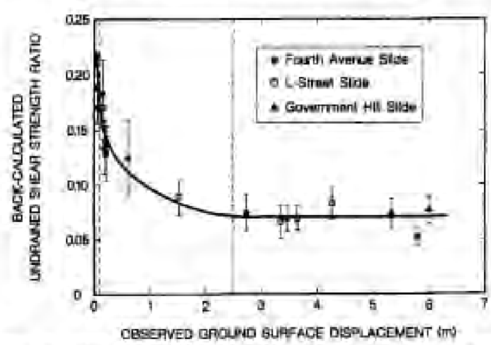

FIG. 6. Variation of Undrained Shear Strength Ratio with Ground Surface Displacement

J. Geotech. Geoenviron. Eng. 1998.124:99-109.

m may be that once a post-peak strength loss is initiated, progressive failure occurs rapidly. As a result, it seems prudent to design similar slopes so that a postpeak strength loss does not occur.

The use of ground surface displacement in Fig. 8 does not accurately represent the behavior along the failure surface. Unfortunately, deformation along the failure surface was not measured and the thickness of the shear zone is not known. For design purposes it would be more desirable to present Fig. 8 in terms of shear strain rather than displacement. The use of horizontal displacement in Fig. 8 is applicable to design if the thickness of the shear surface is small. For design purposes, it is assumed that a field displacement of 0.15 m occurred along a thin shear plane and Fig. 8 can be used to estimate the postpeak strength loss for other slopes in soils similar to Bootlegger Cove clay.

Fig. 9 presents the data in Fig. 8 in terms of the ratio of the back-calculated or mobilized undrained strength ratio (Table 2) to the laboratory peak undrained strength ratio obtained from Fig. 6. It is recognized that for laboratory tests, the value of undrained strength ratio, s_u/σ'_p, where s_u is the undrained peak shear strength measured at the field anisotropic stress condition, may be different from the value of undrained strength ratio, s_u/σ'_{vc}, where s_u is measured using an isotropic stress condition (Terzaghi et al. 1996). However, for design purposes and the present investigation this difference was assumed to be insignificant (Mesri 1989). It should be noted that the laboratory undrained peak shear strength ratio was calculated by dividing the undrained peak shear strength by the in situ preconsolidation pressure at stress levels less than the preconsolidation pressure and divided by the laboratory vertical consolidation pressure at stress levels greater than the preconsolidation pressure. Terzaghi et al. (1996) state that the undrained strength ratios s_u/σ'_p in the recompression range and s_u/σ'_{vc} in the compression range are similar.

It can be seen from Fig. 9 that a strength loss of approximately 20% occurs at a ground surface displacement of 0.15 m. Therefore, it may be concluded that 80% of the laboratory peak undrained strength can be used to conservatively estimate the permanent deformation of new or existing slopes. The use of 80% of the undrained strength of cohesive material to estimate the yield acceleration or permanent deformation was also proposed by Makdisi and Seed (1978). At a displacement greater than 2.5 m, a strength loss of approximately 70% is observed. Fig. 9 can be used to estimate the strength loss at displacements between 0.15 and 2.5 m in a permanent displacement analysis involving cohesive soils similar to Bootlegger Cove clay.

Since the ground displacement increases rapidly after 0.15

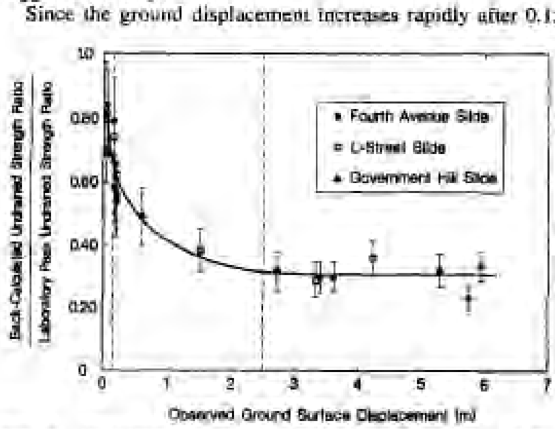

FIG. 9. Variation of Normalized Undrained Shear Strength Ratio with Surface Ground Displacement

m, it is suggested that an undrained residual strength be used for analysis if the estimated deformation exceeds 0.15 m. This was first proposed by Woodward-Clyde (1982) and Idriss (1985), who suggested that the transition from peak to residual occurs at a ground surface displacement of 0.15 m. In a permanent deformation analysis, the calculated displacement corresponds to the displacement along the failure surface and not the ground surface. Since soils are not rigid, it is likely that the failure surface displacement was greater than or equal to the ground surface displacement. Therefore, a residual strength should be used after a deformation of 0.15 m.

Displacement Causing Postpeak Behavior

It should be noted that the laboratory displacement (1–2 mm) required to mobilize the undrained peak strength is not equivalent to the field displacement. The laboratory displacement is usually less than the field value because the ring shear apparatus focuses the shear stresses on a thin shear plane whereas a shear zone probably develops in the field. The laboratory displacement required to mobilize an undrained residual strength condition (100 mm) is also probably less than the field deformation. Therefore, field deformations, and not laboratory displacements, should be used to estimate the transition from peak to postpeak behavior.

Using an undrained shear strength that corresponds to 80% of the peak value, instead of 100%, appears reasonable for a permanent deformation of 0.15 m along the failure surface. It should be noted that the observed deformations were measured at the ground surface and after earthquake shaking ceased. As a result, a ground surface deformation of 0.15 m probably does not correspond to the precise transition point from peak to postpeak behavior along the failure surface. The displacement that corresponds to this transition point in a range of cohesive soils is not known and warrants additional research. However, for analysis and design a peak-to-postpeak transition point may be assumed to occur at a calculated permanent deformation of 0.15 m.

USE OF FIELD VANE SHEAR TEST IN SEISMIC STABILITY ANALYSES

Since the constant volume ring shear apparatus is not readily available in practice, the use of the field vane shear test to estimate the undrained peak and residual shear strengths for seismic stability evaluations was also investigated. With the standard vane equipment (ASTM D2573) inserted in the undisturbed soil at the desired depth, a torque is applied to the vane at a rate of 0.1 deg/sec. The maximum torque is measured and used to estimate the undrained peak shear strength for the field vane mode of shear, $s_u(FV)$. This usually results in failure (peak shear strength) occurring in about one to five minutes. The height of the vane should be twice the diameter. In addition, the cohesive layer should be significantly larger than the vane so that drainage does not occur into adjacent soil layers.

The field vane can also be used to estimate the undrained residual strength. Apparatuses with geared drives allow intermediate values of torque to be recorded and thus the shear stress versus rotation angle relationship can be determined. In those cases, rotation of the vane can continue at a rate of 0.1 deg/sec and a decrease of shear stress with rotation can be observed. Rotation can be continued until the undrained residual strength is mobilized. Another vane test procedure for estimating the undrained residual strength that may be faster (ASTM D2573) involves rotating the vane rapidly for several revolutions after the undrained peak strength is measured. Afterward, the vane shear test is resumed at a rate of 0.1 deg/sec to measure the undrained residual strength. The number of

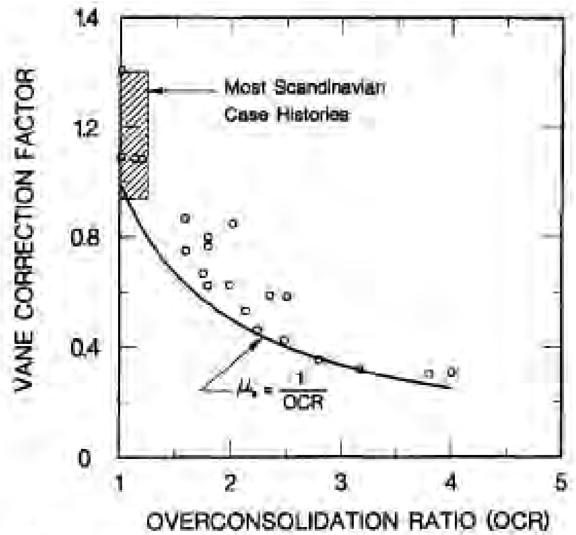

FIG. 10. Vane Shear Correction Factor for Seismic Stability Evaluations of Cohesive Soil Slopes (Terzaghi et al. 1996)

revolutions recommended to achieve an undrained residual strength condition include three (Pyles 1984), 10 (Skempton 1948; Arman et al. 1975), and 25 (Aas et al. 1986). Based on comparisons between field vane and constant volume ring shear data, it is here recommended that at least 10 revolutions be used to estimate the undrained residual strength. Ten revolutions is also recommended by ASTM D2573. It should be noted that the undrained residual strength determined using this procedure has been previously referred to as the remolded, ultimate, and minimum shear strength.

To use the undrained peak shear strength from the field vane test in seismic stability evaluations, it was anticipated that a correction factor would be required. The most desired approach to calibrating the undrained peak shear strength from a vane shear test is to use values back-calculated from field case histories as Bjerrum (1973) did to develop a correction factor for static loading. Leroueil et al. (1983) and Tavenas (1985) back-analyzed static failures of natural slopes in slightly overconsolidated clays and concluded that the shear strength mobilized at failure was lower than the peak value of $s_u(FV)$. By making a conservative interpretation of the data presented by Tavenas (1985), Terzaghi et al. (1996) developed a correction factor to use $s_u(FV)$ in static slope stability analyses (Fig. 10). It is assumed here that this correction factor is also applicable to seismic stability evaluations of cohesive soil slopes. The validity of this assumption is verified using the Fourth Avenue landslide. However, additional verification should be conducted as additional seismic case histories become available.

Fourth Avenue Field Vane Test Results

Field vane shear tests were performed at three Fourth Avenue locations during the 1964 investigation (Fig. 11); none were conducted during the 1982 investigation. The vane used in these tests had a height of 102 mm and diameter of 51 mm. The location of borings A112, A119, and A121 are indicated in Fig. 2. Fig. 11 shows that the undrained peak shear strength is approximately 35 kPa at elevation +10 m, then increases with depth to a value of 55 kPa at elevation +1 m. The undrained peak strength ratio from the vane test, $s_u(FV)/\sigma'_p$, is estimated to be 0.27. This value is typical for a slightly overconsolidated clay deposit of this plasticity (Terzaghi et al. 1996).

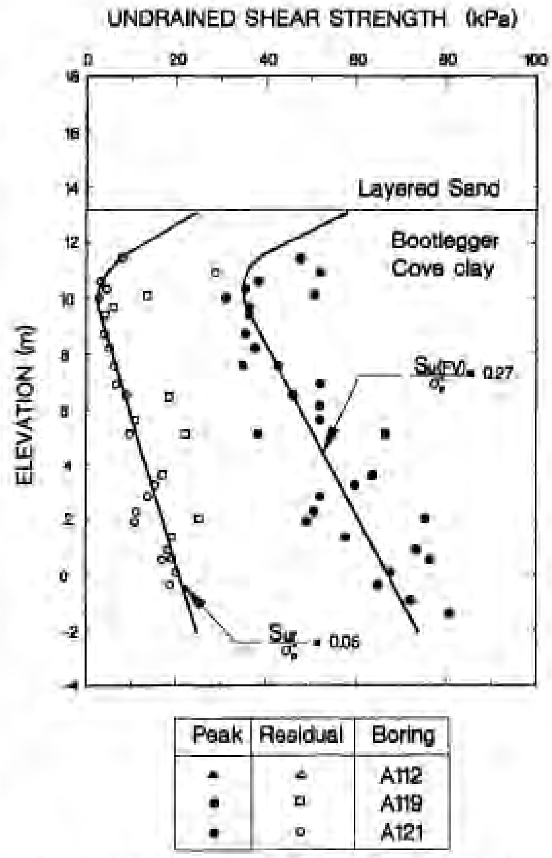

FIG. 11. Field Vane Shear Test Results from Fourth Avenue Slide Mass (data from Shannon and Wilson 1964)

As described previously, the Bootlegger Cove clay involved in the slide displays an OCR ranging between 1.2 and 1.4; therefore, a correction factor from Fig. 10 of 0.83–0.71 is applicable. Applying this correction factor to the measured $s_u(FV)/\sigma'_p$ value of 0.27 results in a mobilized value of undrained peak strength ratio of 0.19–0.22. This range of values agrees with the values back-calculated (0.14–0.22) for slide blocks that moved less than 0.15 m (Table 2).

The undrained residual shear strength measured in the field vane test is also shown in Fig. 11. The undrained residual strength is approximately 4 kPa at elevation +10 m, then increases with depth to a value of 19 kPa at elevation +1 m. The undrained residual strength ratio, $s_{ur}(FV)/\sigma'_p$, is estimated to be 0.05 from the linear portion of the data. This is in excellent agreement with the values measured in the ring shear apparatus (0.06) and back-calculated for slide blocks that moved more than 2.5 m (0.05–0.08). In summary, the constant volume ring shear apparatus and/or the proposed field vane shear test procedure and correction factor can be used to estimate the undrained peak and residual strengths for seismic slope stability analyses involving sensitive cohesive soils.

CONCLUSIONS

Reevaluation of the Fourth Avenue Landslide that occurred in Anchorage, Alaska, during the 1964 earthquake showed that the slide was caused by a large undrained strength loss and development of an undrained residual strength condition in the Bootlegger Cove clay. This failure mechanism was first proposed by Woodward-Clyde (1982) and Idriss (1985). The results of constant volume ring shear and field vane shear tests on the Bootlegger Cove clay are presented and compared with shear strengths back-calculated for the slide blocks. The comparison shows that the slide blocks that moved less than 0.15 m mobilized at least 80% of the undrained peak shear strength. Slide blocks that moved between 0.15 and 2.5 m mobilized an undrained shear strength between the peak and residual shear strengths. Slide blocks that moved more than 2.5 m mobilized the undrained residual strength. The results of this study suggest that 80% of the undrained peak shear strength should be used to conservatively evaluate the seismic stability of slopes in sensitive soils. If earthquake-induced sliding will be triggered with permanent deformation exceeding 0.15 m, the mobilized undrained shear strength will probably be less than the peak value. The relationship presented in Fig. 9 can be used to estimate the percentage of the undrained peak shear strength that should be used to estimate the permanent lateral displacement in cohesive soils similar to Bootlegger Cove clay. Otherwise, the undrained residual shear strength should be used for analysis purposes if the permanent deformation exceeds 0.15 m.

Since the constant volume ring shear apparatus is not readily available in practice, the field vane shear test procedure and correction factor proposed here can be used to estimate the undrained peak and residual shear strengths for seismic stability evaluations. If the entire shear stress-displacement relationship is desired, constant volume ring shear testing probably should be conducted.

ACKNOWLEDGMENTS

This study was performed as part of the United States Geological Survey, Grant No. INT14-08-001-G1953. The support of this agency is gratefully acknowledged. The first writer also acknowledges the support provided by the W. J. and E. F. Hall Scholar award. Gholamreza Mesri and Robert M. Ebeling provided extensive suggestions during the testing and analytical phases of the research and reviewed the manuscript. The writers also acknowledge Harold W. Olsen, Yoshi Moriwaki, Gabriel Fernández, and Randall G. Updike for their many valuable suggestions.

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Attachment E

Characterization of Bootlegger Cove Formation Clay Shear Behavior

Static and Cyclic Shear Strength of Bootlegger Cove Clay at Port of Anchorage

Port of Anchorage Intermodal Expansion Project

CH2MHILL

Suitability Study of OPEN CELL® Structure

Suitability Study

- Conduct independent analysis of the OPEN CELL foundation system at Port of Anchorage
- Scope of investigation:
  - Design criteria evaluation
  - Seismic hazard assessment
  - Hydrologic
  - Geotechnical **Focus of Presentation**
  - Structural
  - Construction

Bootlegger Cove Clay Investigation

- Additional study of the foundation material to support preliminary findings regarding behavior of the OPEN CELL structure in response to static and seismic loading conditions

Outline

Background

- Static and seismic global stability of OPEN CELL® structure
- Definitions and notation
- Previous characterization of Bootlegger Cove clay shear strength
- Due Diligence Investigation 2: study objectives and scope

Laboratory Soil Testing

- Exploratory drilling and soil sampling
- Stress history
- Peak effective-stress shear strength
- Peak undrained shear strength
- Cyclic and post-cyclic undrained shear strength
- Undrained residual strength

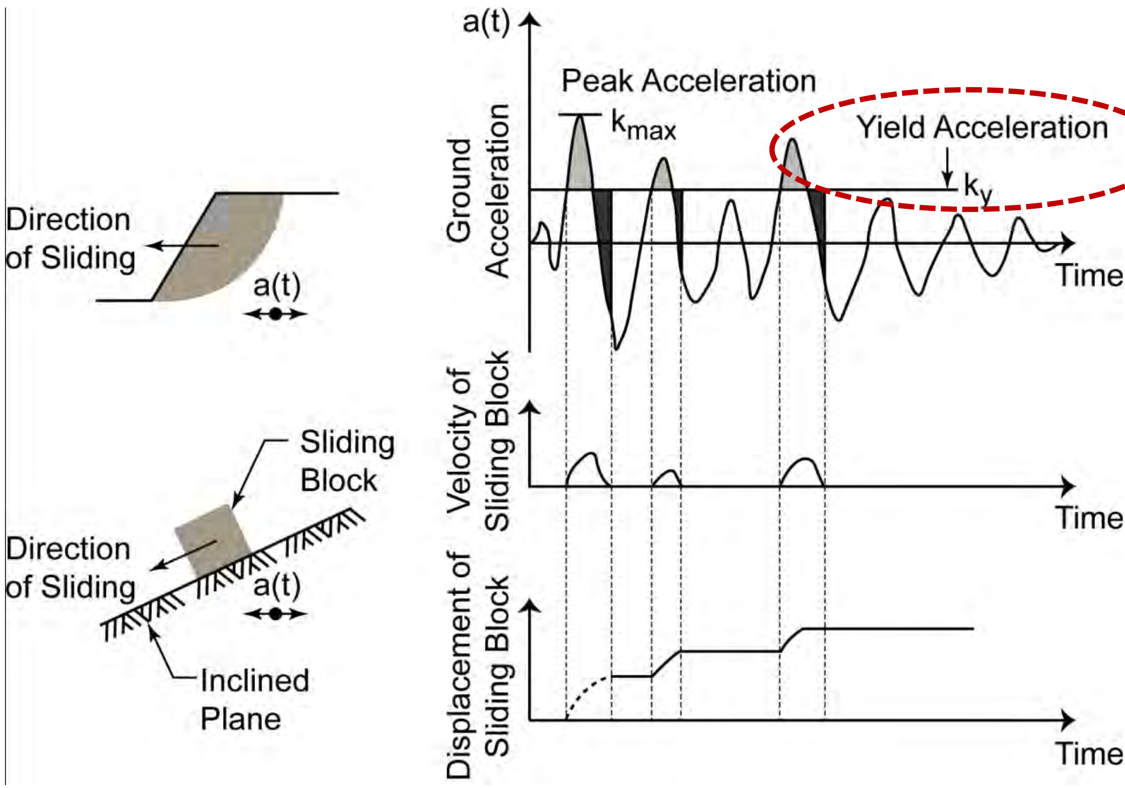
Conclusions

- Shear strength comparisons
- Project implications

Global Stability of OPEN CELL® Structure

Evaluation Methods

- Limit-equilibrium analyses
- Dynamic FLAC modeling
- Newmark analyses for seismic deformation



The yield acceleration indicates what magnitude of ground acceleration will cause permanent movement and is controlled by undrained shear strength along the failure surface

What pseudo-static seismic coefficient gives FS = 1.0?

Section 2-2:
k_y = 0.04 g

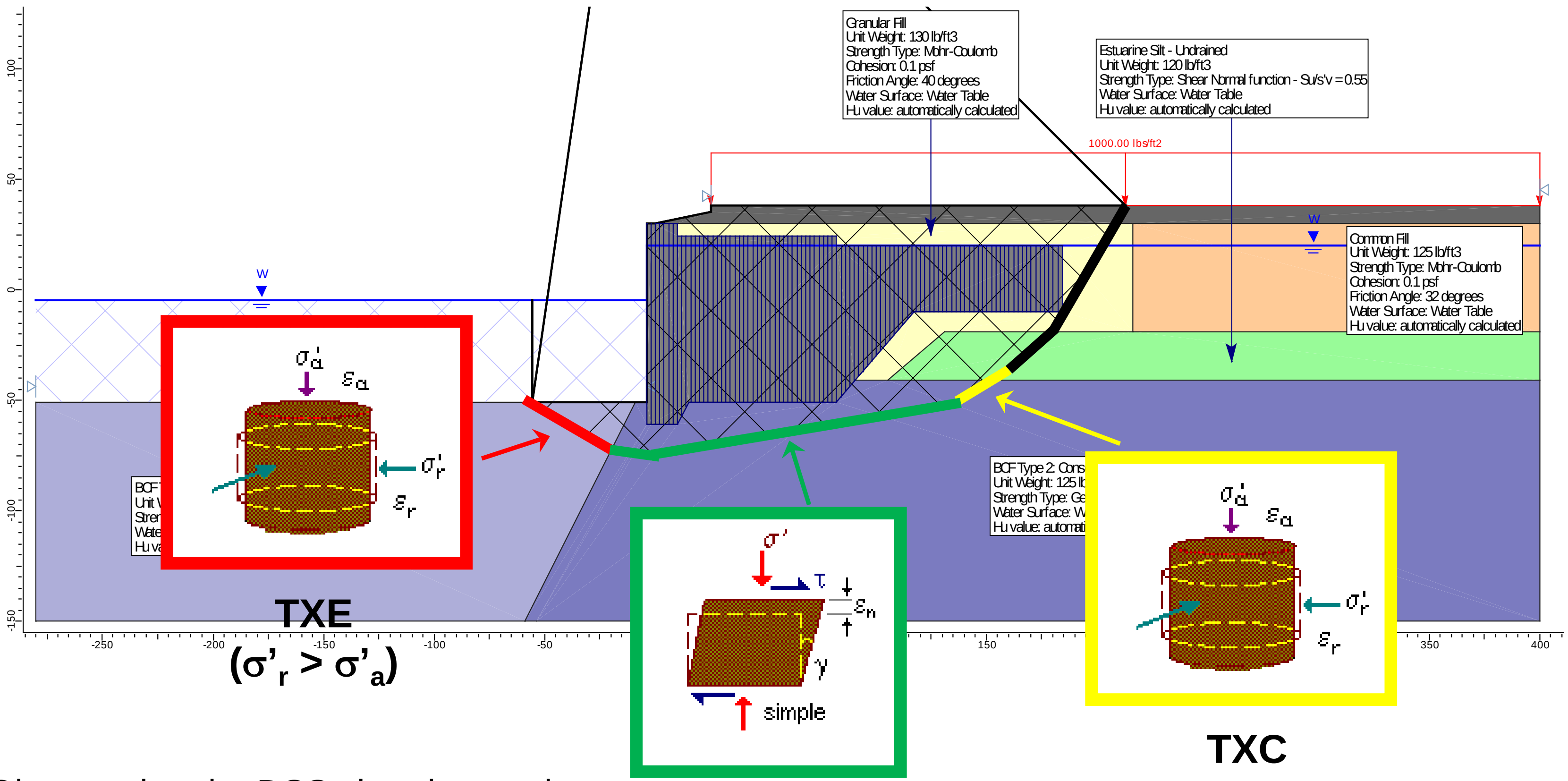
Section 3-3:
k_y = 0.03 g

(Extremely low "tolerance" for cyclic loading)

Global Stability of OPEN CELL® Structure

Evaluation Methods (cont.)

- Strength anisotropy



TXE

($\sigma'_r > \sigma'_a$)

DSS

TXC

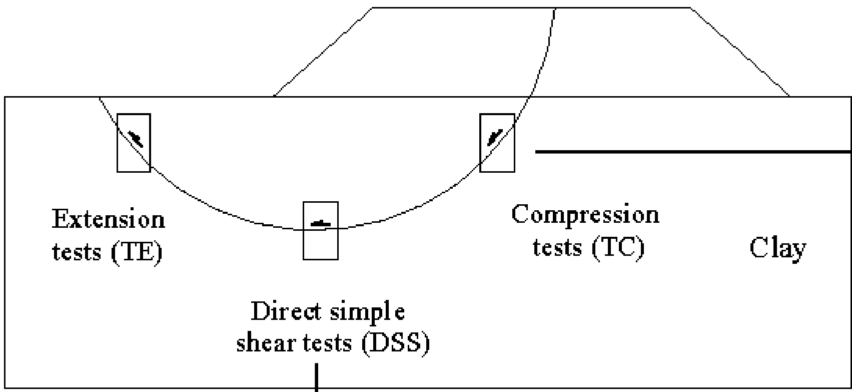
($\sigma'_a > \sigma'_r$)

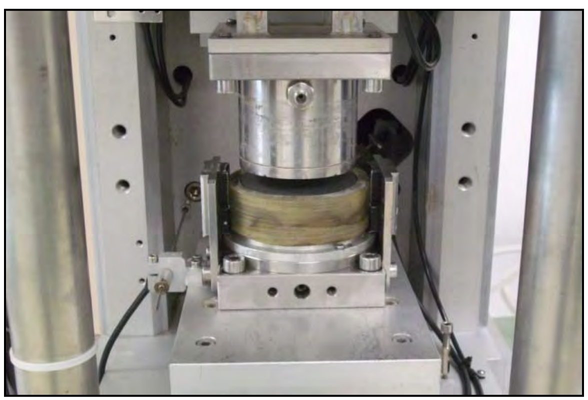
Observe that the DSS shearing mode has largest impact on stability

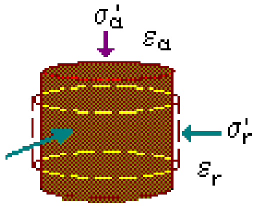
Global Stability of OPEN CELL® Structure

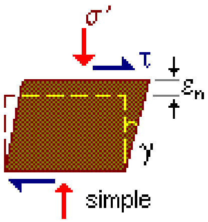
Evaluation Methods (cont.)

- Strength anisotropy









Isotropic CIUC: $\sigma'_r = \sigma'_a$

Anisotropic CAUC: $\sigma'_r = K\,\sigma'_a$

Relevance of laboratory strength tests to field shear conditions

Global Stability of OPEN CELL® Structure

Primary Design Concerns

- Stability and seismic deformations governed by Bootlegger Cove clay
- Inadequate safety factor (FS) for static-drained and static-undrained failure cases
  - Static-drained FS = 1.4 – 1.5
  - Static-undrained (end of construction, long term) FS = 1.1 – 1.2
- Excessive seismic deformations resulting from post-peak undrained strength loss
  - OLE deformations exceeding design/performance criterion
  - CLE, MCE deformations excessive (i.e., > 30 inches)
- Further evaluation of soil strength to determine if preliminary evaluation was too pessimistic
  - Re-assess previous strengths
  - Conduct more focused testing

Global Stability of OPEN CELL® Structure

Fundamental Link Between Static and Seismic Stability Analysis

- Static stability and seismic deformation analyses are not independent
- Static-case FS indirectly governs seismic deformations, because static strength dictates the yield acceleration, k_y, which is the seismic coefficient that, when exceeded during shaking, results in permanent displacement
- Low static-case FS = high seismic deformations!
- In high-seismicity areas, not uncommon for static-case FS to exceed about 3.0 or 4.0 for seismic deformation criteria to be achieved

Definitions and Notation

- **(Undrained) Intact strength, s_u**

  Peak undrained strength of an undisturbed, intact sample

- **(Undrained) Remolded/fully-softened strength, $s_{u,rem}$**

  Peak undrained strength of a remolded, fully softened, fully weathered, or fully deaggregated cohesive soil

  Example: Dallas levees subject to strength loss due to in-place weathering

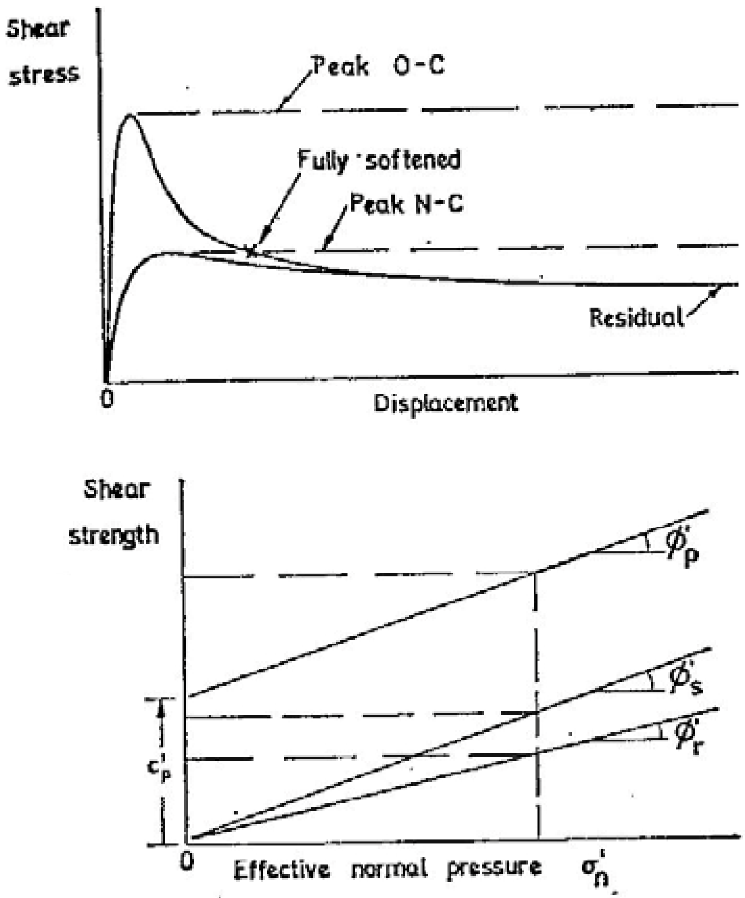
- **(Undrained) Residual strength, s_{ur}**

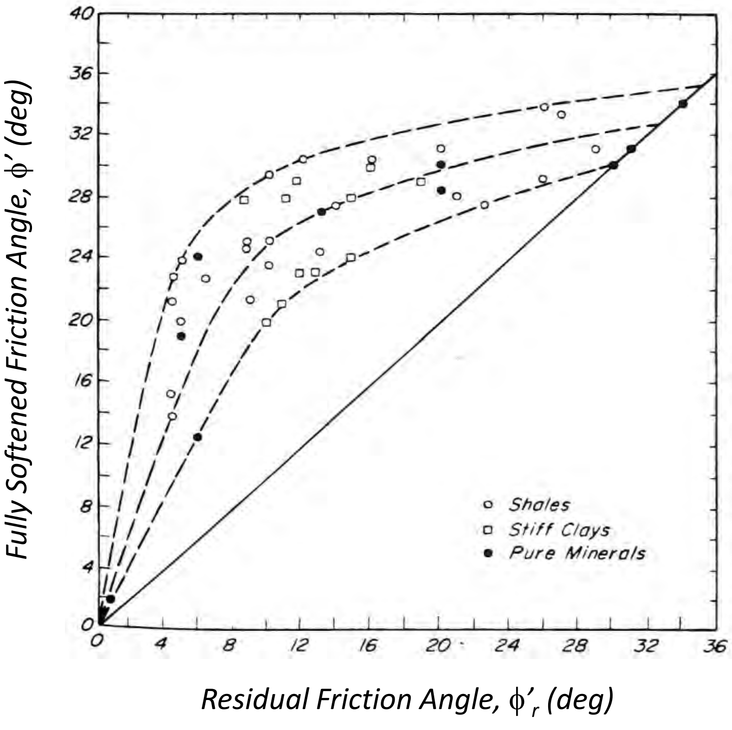
  *Minimum strength mobilized under **very large displacements** when shear-induced excess pore-water pressures and shear stresses are constant; under drained conditions, the soil arrives at same final water content and void ratio, regardless of initial state*

- **Sensitivity, S_t**

  Ratio of undisturbed, intact strength to remolded strength, both of which are peak strengths mobilized at low strains/displacements

Definitions and Notation (cont.)





Undrained residual strength
discussed more later

Definitions and Notation (cont.)

- **(Undrained) Cyclic strength, $s_{u,cyc}$**

  Undrained strength observed under cyclic load application (if failure occurs) or the monotonic strength observed immediately following cyclic loading (e.g., post-cyclic DSS test)

- **Cyclic degradation/softening**

  Undrained strength loss attributed to generation of excess pore-water pressures and strain induced by cyclic load application (typically under relatively small shear strain magnitudes)

- Cyclic degradation and mobilization of undrained residual strength of clay are different modes of strength loss, but mechanisms that may occur simultaneously during seismic shaking if permanent deformations are taking place

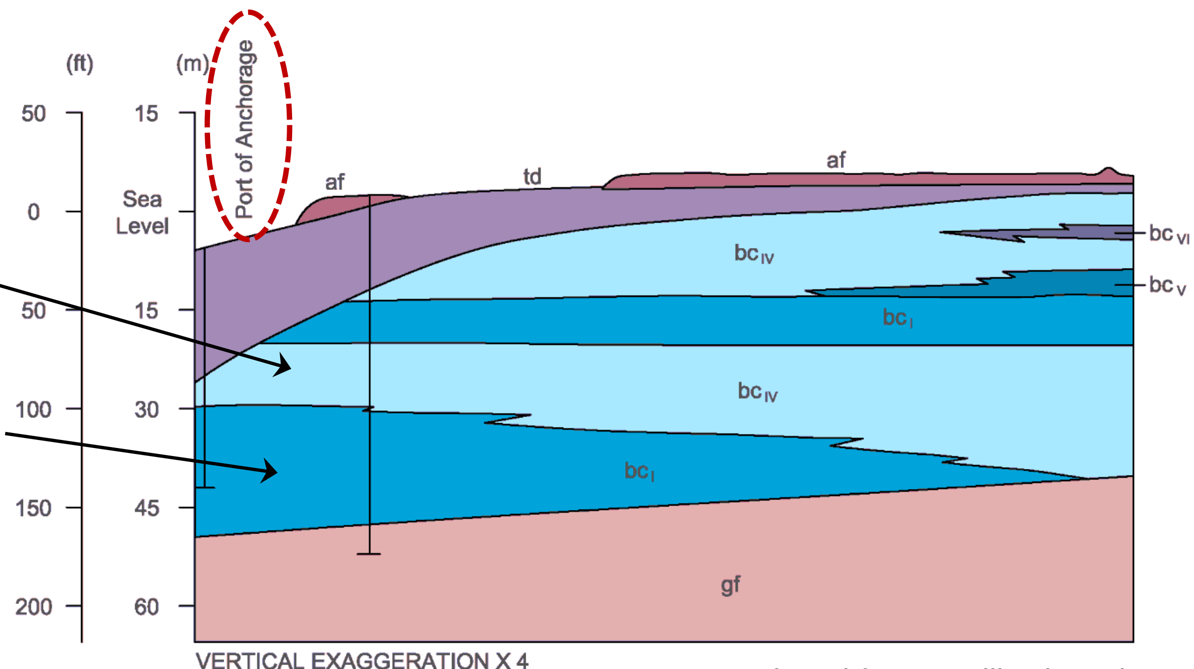
Previous Characterization

Ulery and Updike (1983), Updike and Carpenter (1986)

- Geologic characterization
  - Identification and definition of various facies of Bootlegger Cove Formation
  - Characterization of intrinsic soil properties (e.g., soil consistency)
  - Geologic maps/profiles

Facies F.IV:
"Silty clay and clayey silt with silt and fine sand lenses"

Facies F.I:
"Clay with very minor amounts of silt and sand"



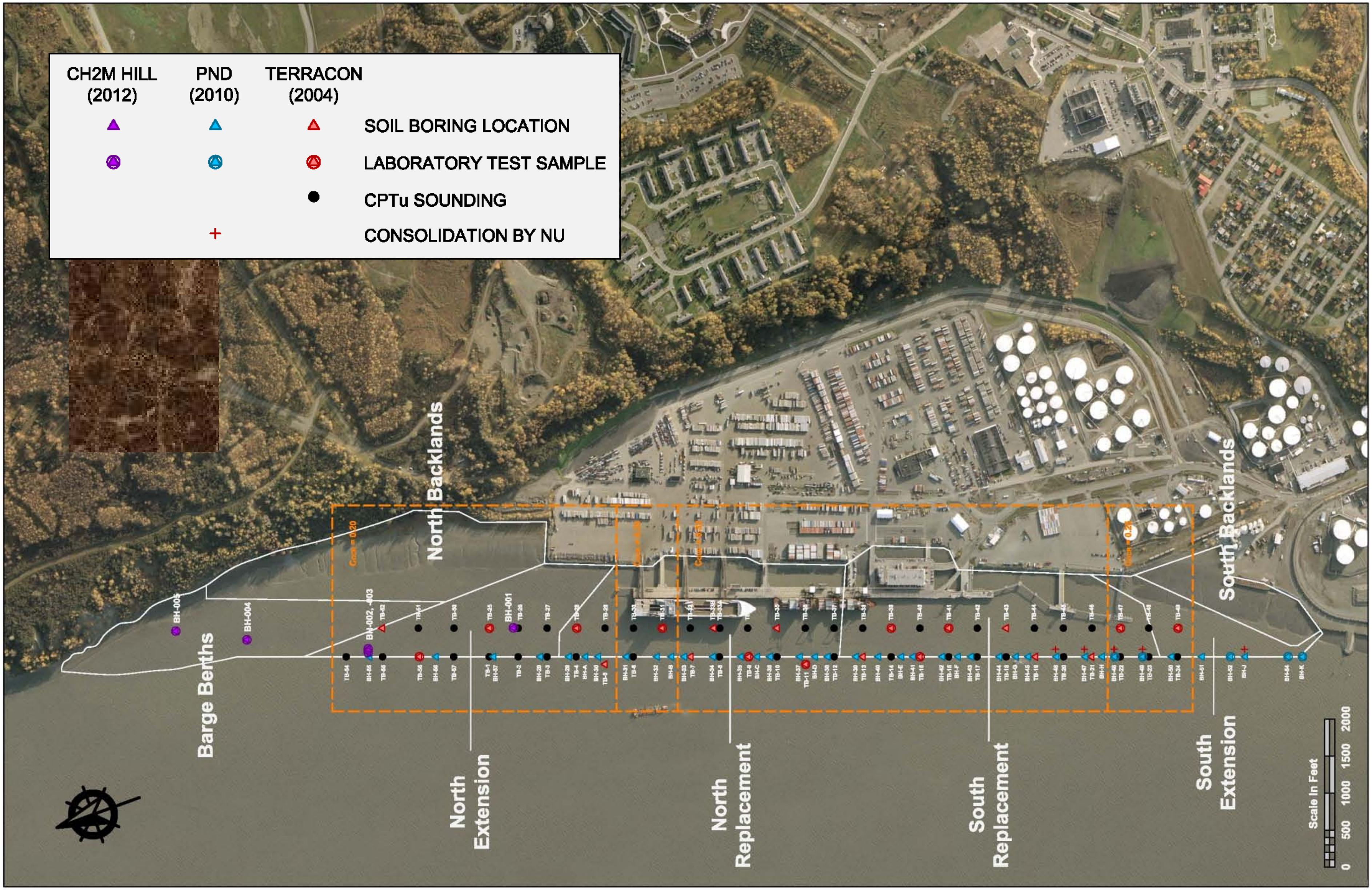
Reproduced from Updike (1986)

Previous Characterization (cont.)

Terracon (2004) Preliminary Engineering Site Investigation

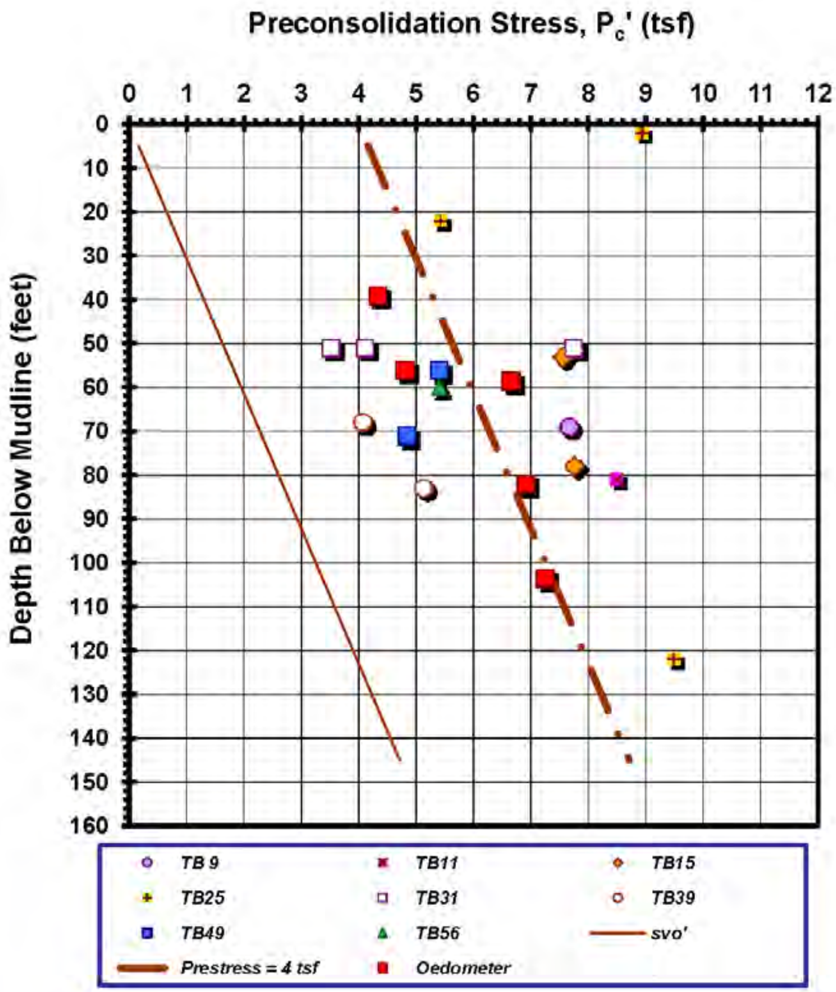
- Incremental load consolidation
- Stress History and Normalized Soil Engineering Properties (SHANSEP)
    - Direct simple shear (DSS)
    - Triaxial compression (CIUC)
- Cone penetration test (CPTu) soundings
    - Piezocone profiling of Bootlegger Cove clay for stress history and undrained strength characterization
- Prediction of undrained strength by modified Cam-Clay model (i.e., critical state soil mechanics)

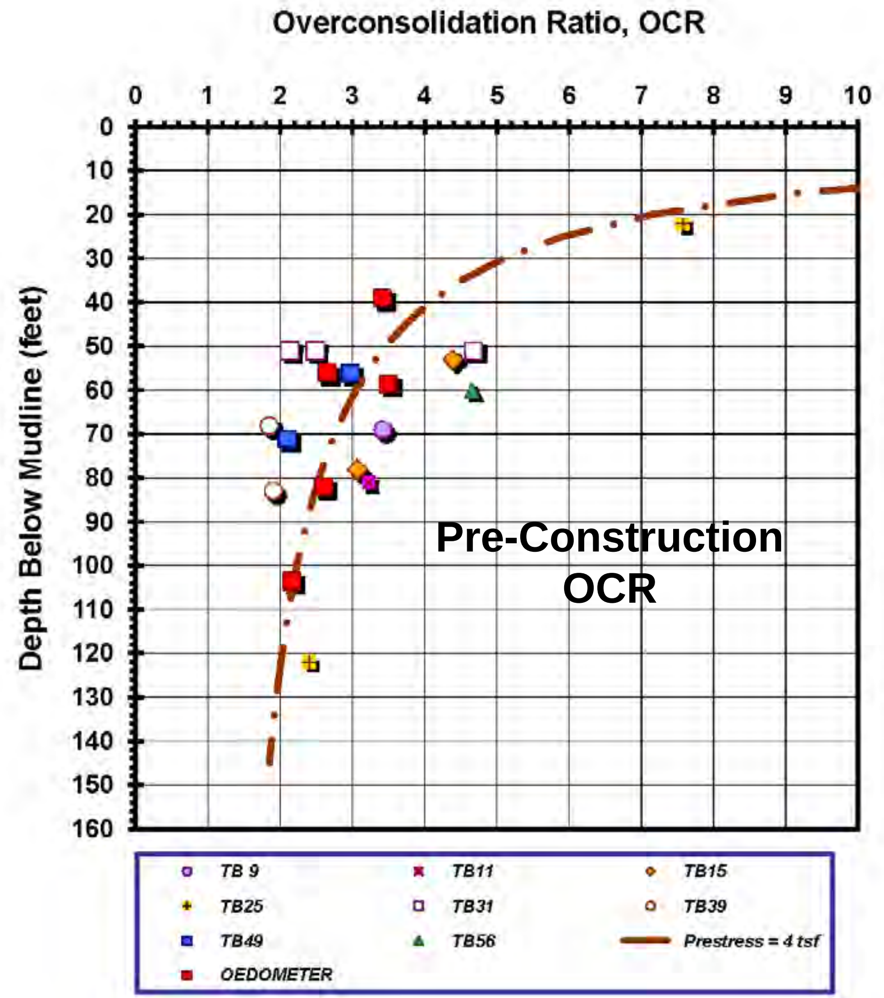
- **Primary limitations: limited test data in North Extension, no test data in Barge Berth**



Exploratory Soil Boring Locations in Barge Berths and North Extension Project Areas

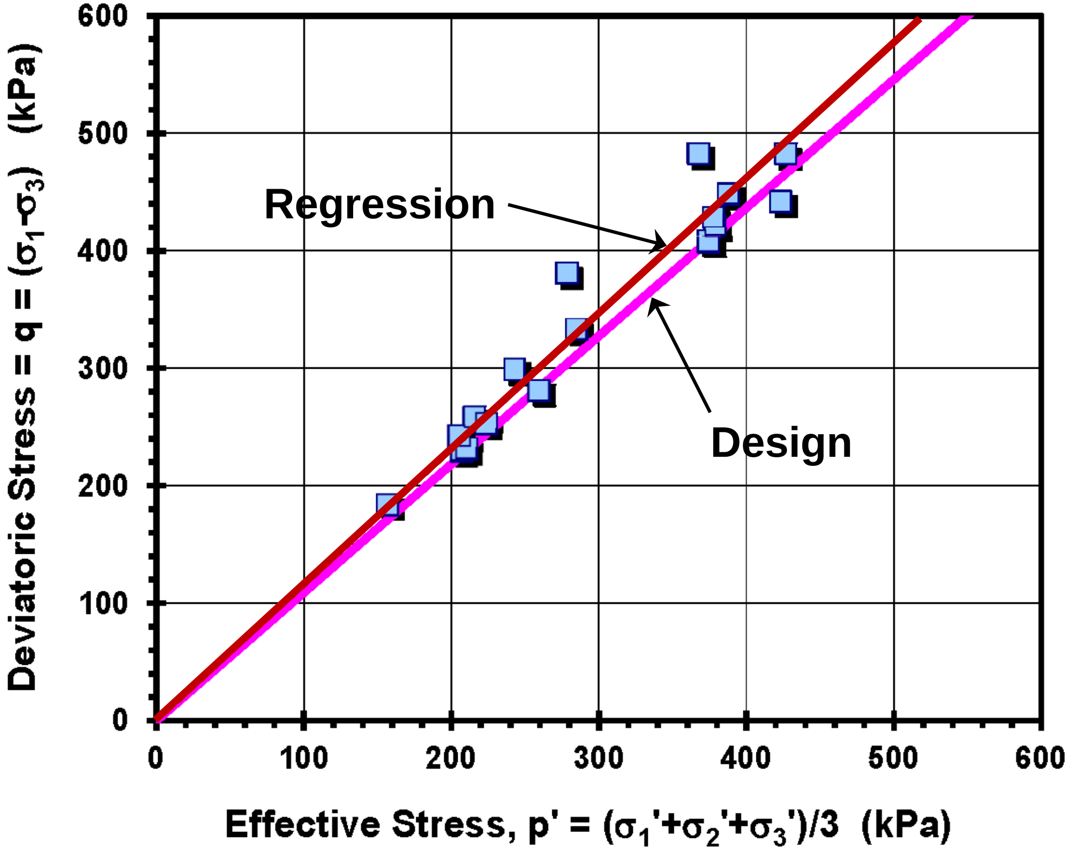
Previous Characterization (cont.)





Stress History Interpretation Based on IL Consolidation, Combined TXC-MCC

Previous Characterization (cont.)



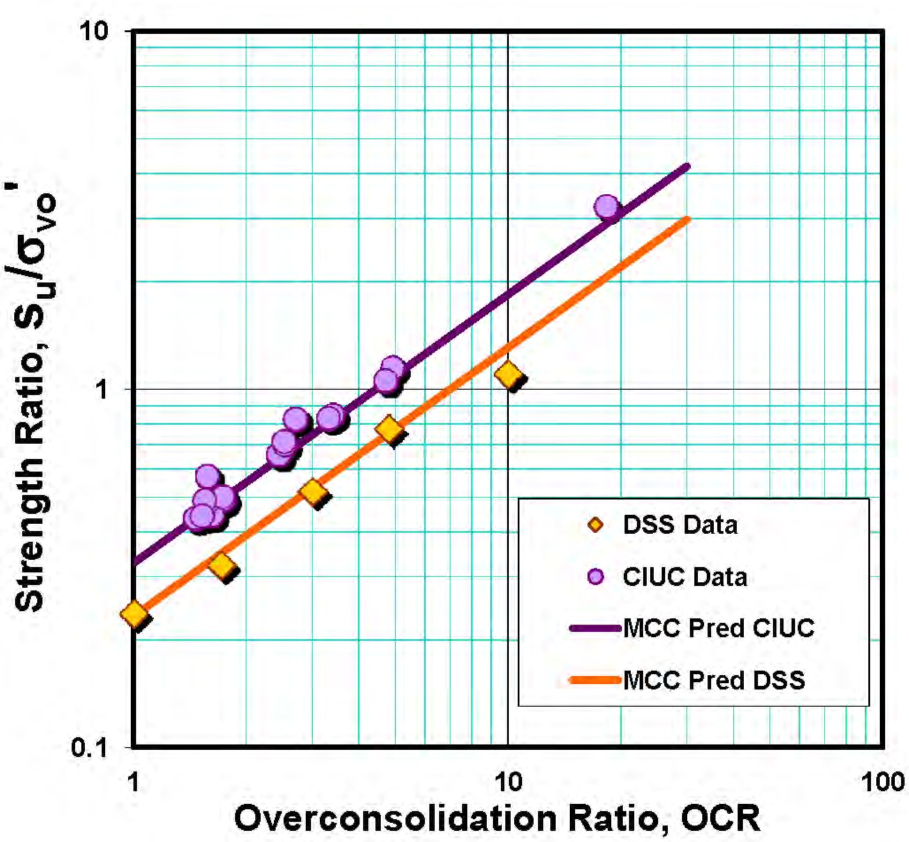
Peak Effective Friction Angle, ϕ'

Design: 27 degrees
Reasonable selection given agreement between measured DSS undrained strength ratios and MCC predictions (based on 27 deg); friction angle lower than other reported values

Regression: 29 degrees

Peak Effective Friction Angle Based on CIUC

Previous Characterization (cont.)



General Form:

$$\left(\frac{S_u}{\sigma'_v}\right)_{OC} = \left(\frac{S_u}{\sigma'_v}\right)_{NC} OCR^m$$

Direct Simple Shear (DSS):

$$\left(\frac{S_u}{\sigma'_v}\right)_{DSS} = 0.23\,OCR^{0.75}$$

(PND used m = 0.70, slightly more conservative)

Triaxial Compression (CIUC):

$$\left(\frac{S_u}{\sigma'_v}\right)_{CIUC} = 0.33\,OCR^{0.75}$$

MCC-predicted CIUC strength ratio of 0.33 is noted to correspond roughly to 29 degrees, not 27 degrees

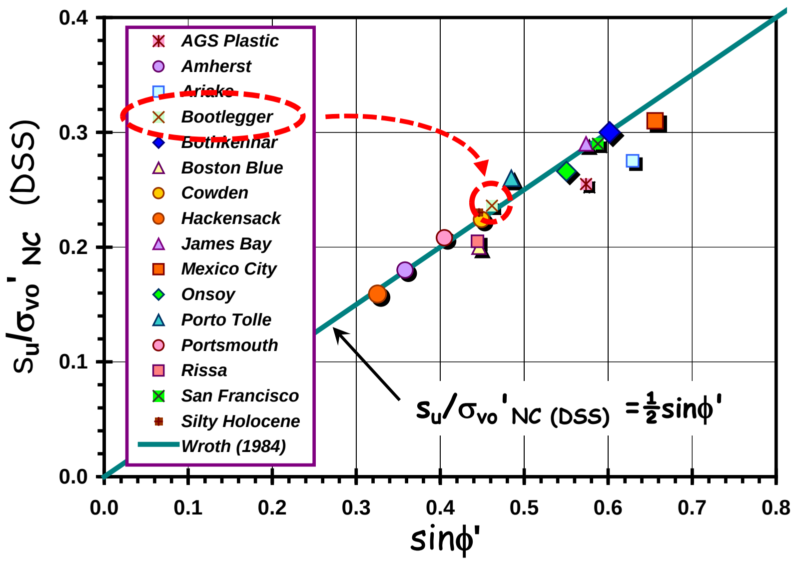
Undrained Strength Ratio with Modified Cam-Clay Predictions

Notes about Modified Cam-Clay Predictions

Direct Simple Shear (DSS):

$$\frac{s_u}{\sigma'_v}(DSS) = \frac{\sin\phi'}{2}\mathrm{OCR}^{\Lambda}$$

$$\text{where, } \Lambda = 1 - C_S / C_C$$

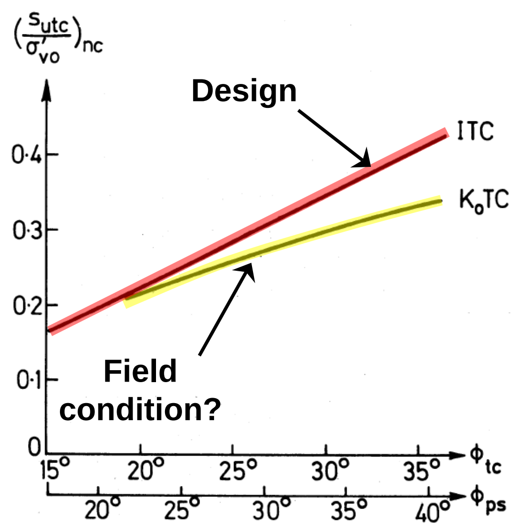


Data courtesy of Paul Mayne

Triaxial Compression (CIUC):

$$\frac{s_u}{\sigma'_v}(CIUC) = \frac{M_c}{2}\left(\frac{OCR}{2}\right)^{\Lambda}$$

$$\text{where, } M_c = \frac{6\sin\phi'}{3 - \sin\phi'}$$



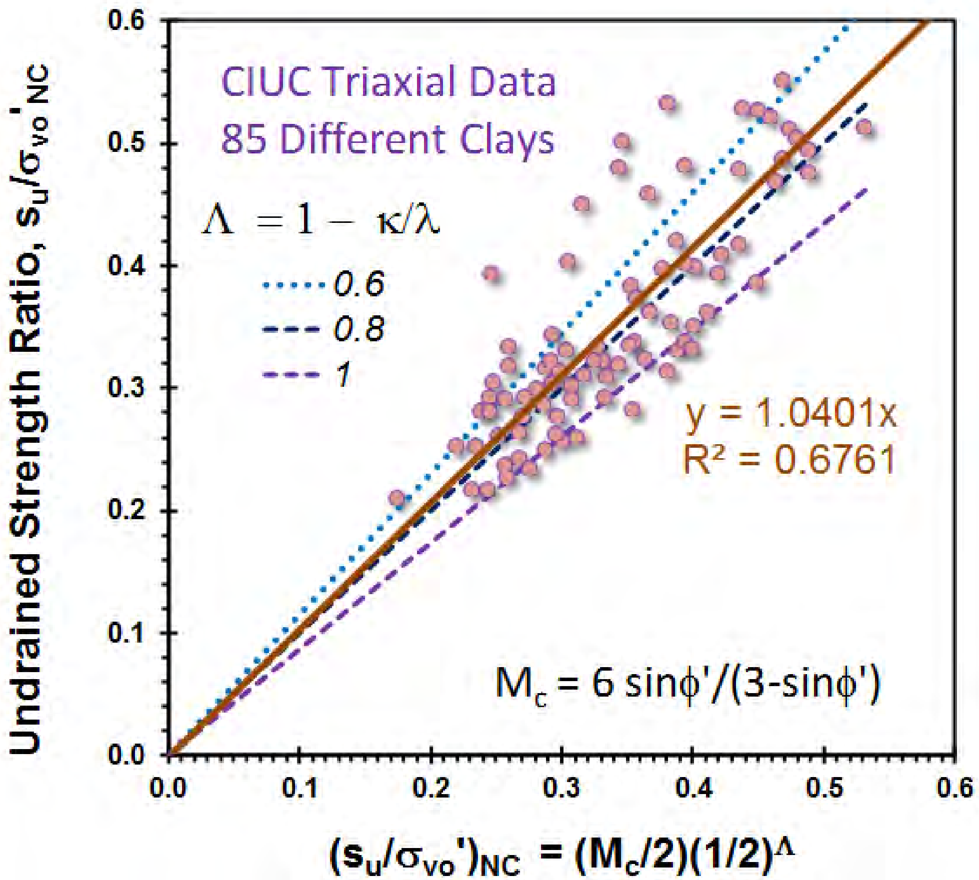
Wroth and Houlsby

Variation of undrained strength ratio with triaxial compression effective friction angle for isotropic and K_0 conditions

At ϕ' = 30 deg, difference is 16 percent

Discussed further later

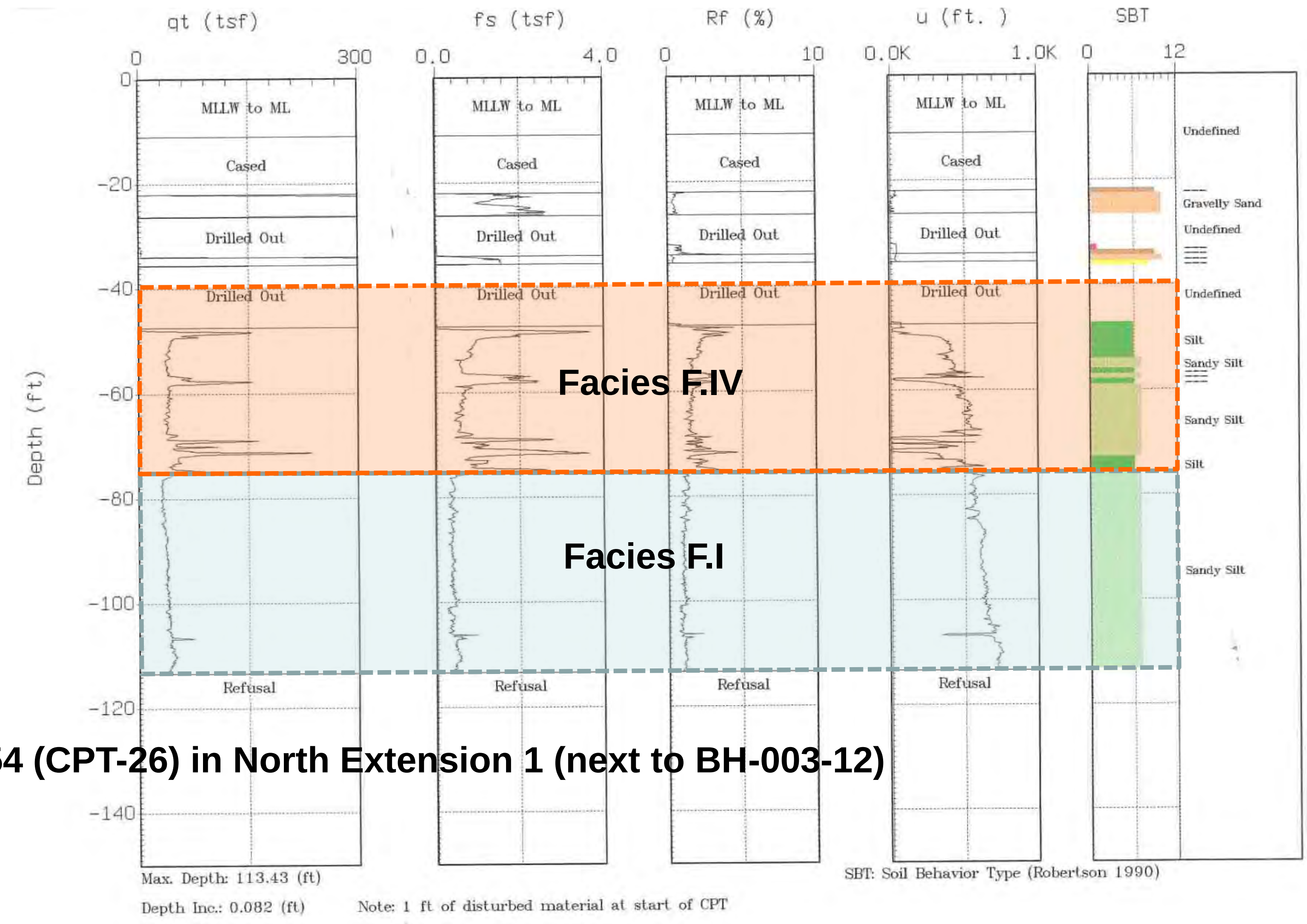
Notes about Modified Cam-Clay Predictions



Adequacy of CIUC MCC approach demonstrated by comparing undrained strength ratios (measured and predicted) for a wide range of different natural clays. Slope of comparison shows less than 5 percent difference.

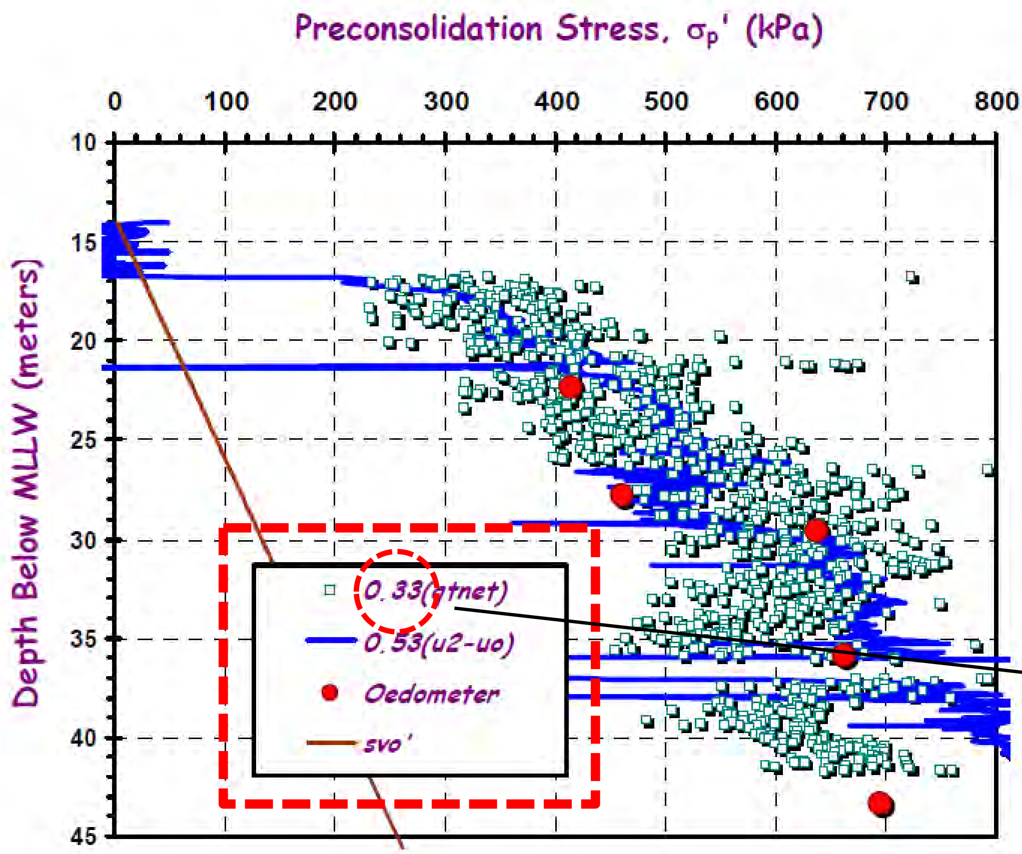
Data courtesy of Paul Mayne

Previous Characterization (cont.)



TB-54 (CPT-26) in North Extension 1 (next to BH-003-12)

Previous Characterization (cont.)



After converting preconsolidation stress to OCR, can input into SHANSEP equation(s) to estimate undrained strength for various shearing modes (e.g., DSS, TXC)

Note: 0.33 doesn't seem to apply to north end or very south end of site; lower value indicates higher qt

Previous Characterization (cont.)

PND (2010)

- Incremental load consolidation (*results shown later*)
- Monotonic triaxial strength testing by DOWL
  - Loading rates much too fast (60 times ASTM standard) and therefore disregarded as having little value (general consensus in the record)
- Cyclic DSS testing by MEG Consulting

Zapata-Medina (2012)

- Doctoral dissertation published at Northwestern University
- Incremental load consolidation (*results shown later*)
- Stress-path testing
  - Stress paths based on stress conditions from numerical modeling
  - Focused on effects of construction-induced stresses on dynamic behavior of clay

Due Diligence Investigation 2

Objectives

- Characterize the undrained shear strength ratios for triaxial compression (CAUC) and direct simple shear (DSS) shearing modes
- Identify cyclic softening of Bootlegger Cove clay under cyclic load application
- Investigate post-peak strength reduction occurring at large shear displacements under constant volume conditions
- Identify potential spatial variation in geotechnical properties at Port site

Scope

- Collect intact samples of Bootlegger Cove clay
- Laboratory testing:
  - Consolidation
  - Monotonic triaxial compression (CAUC) and DSS testing
  - Cyclic and post-cyclic DSS testing
  - Drained and constant volume ring shear testing

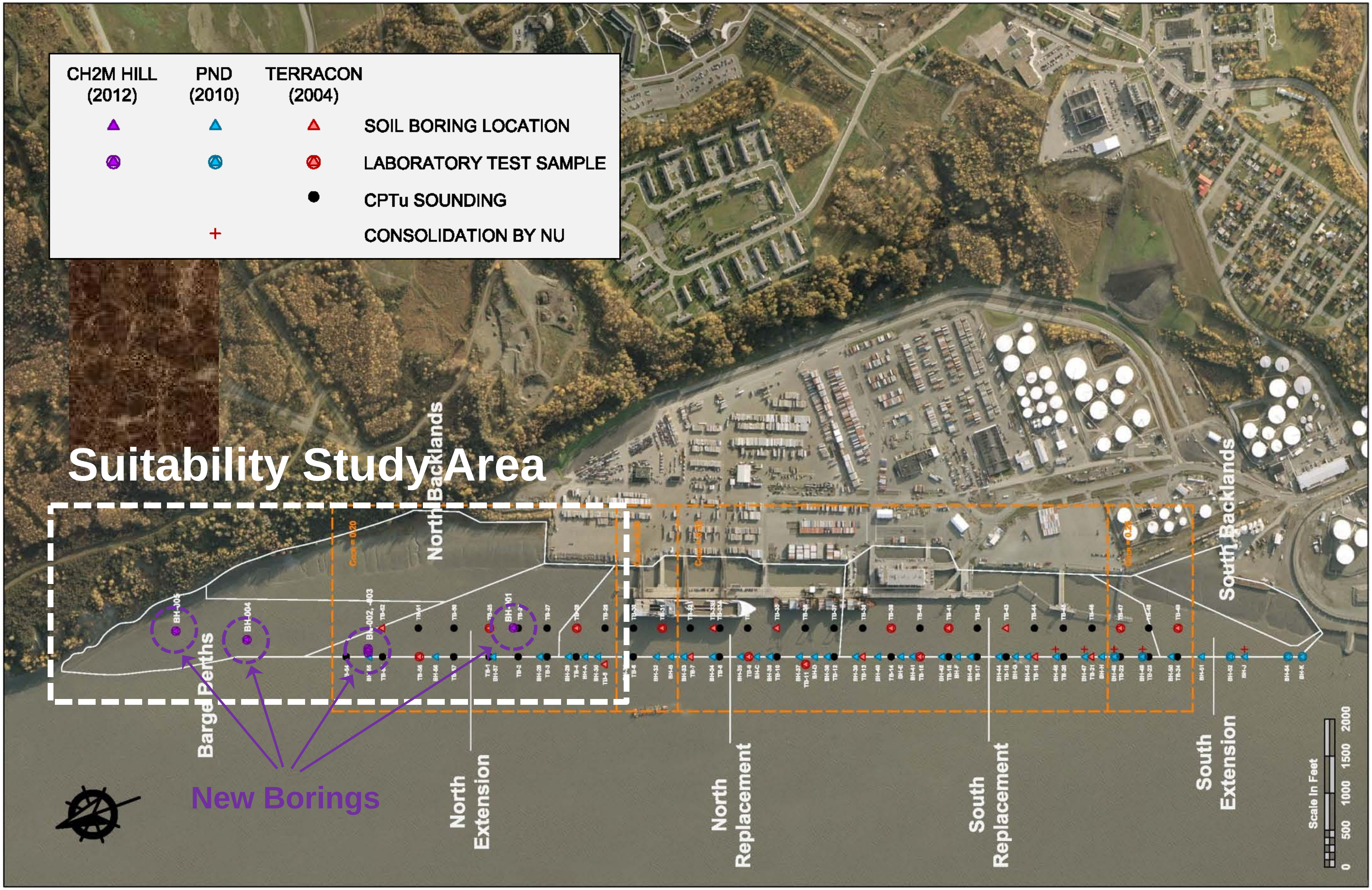
Exploratory Drilling and Soil Sampling

Exploratory Drilling

- Five soil borings in DBB, WBB, NE1, and NE2
- Principal objective to recover nominal 3- and 5-inch samples for laboratory testing

Undisturbed Soil Sampling

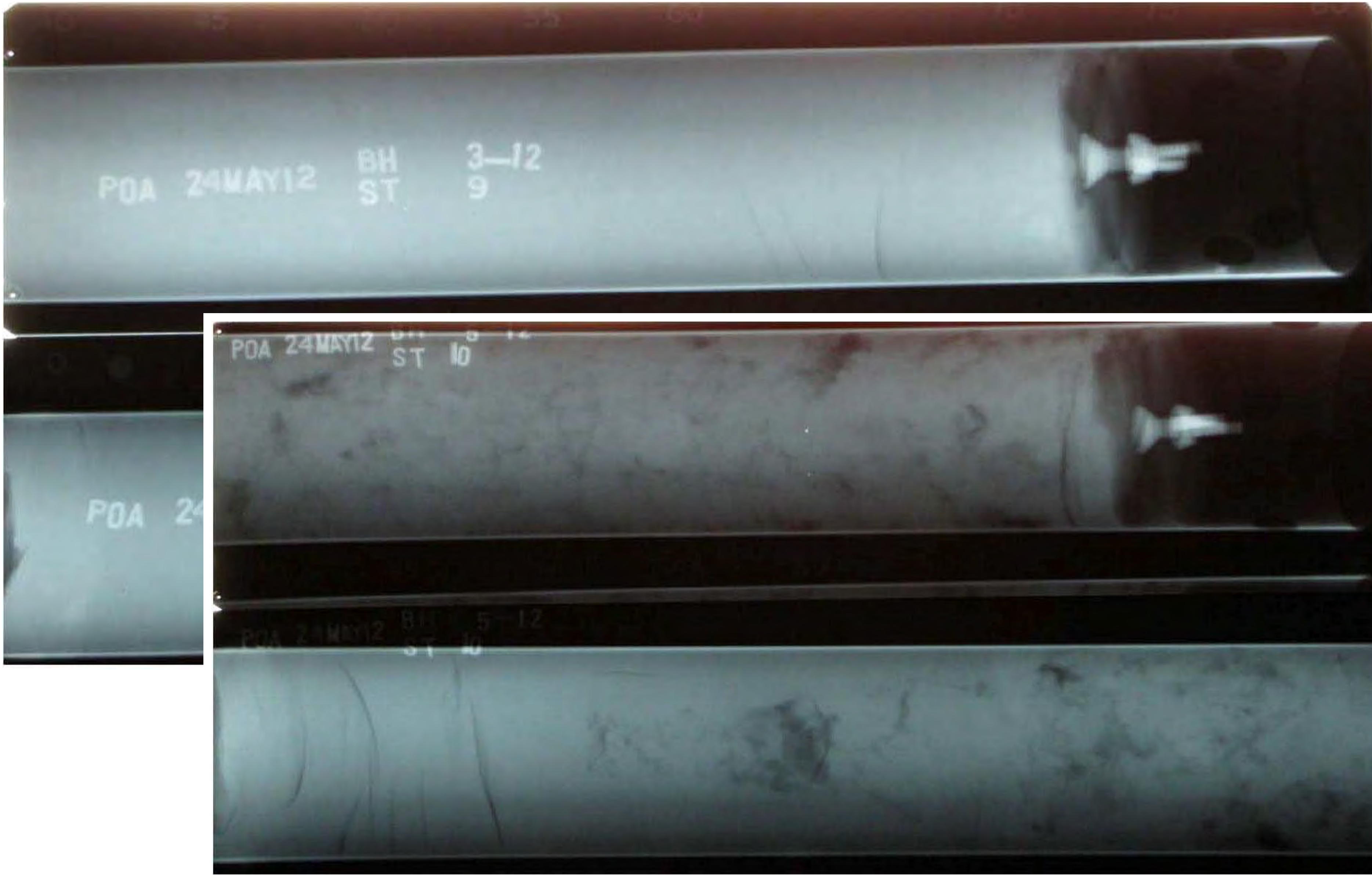
- Undisturbed sampling of very stiff clay is difficult
- Attempted to minimize disturbance
  - Mud rotary drilling methods
  - Gregory Undisturbed Samplers (GUS)
  - Modified Shelby tubes (zero inside clearance)



Exploratory Soil Boring Locations in Barge Berths and North Extension Project Areas

Exploratory Drilling at Port of Anchorage (Set-Up with Two Rigs in North Extension 1)

5-inch Shelby Tube Samples: Drilling Hole in
Thin-Walled Tube to Release Vacuum Created by GUS



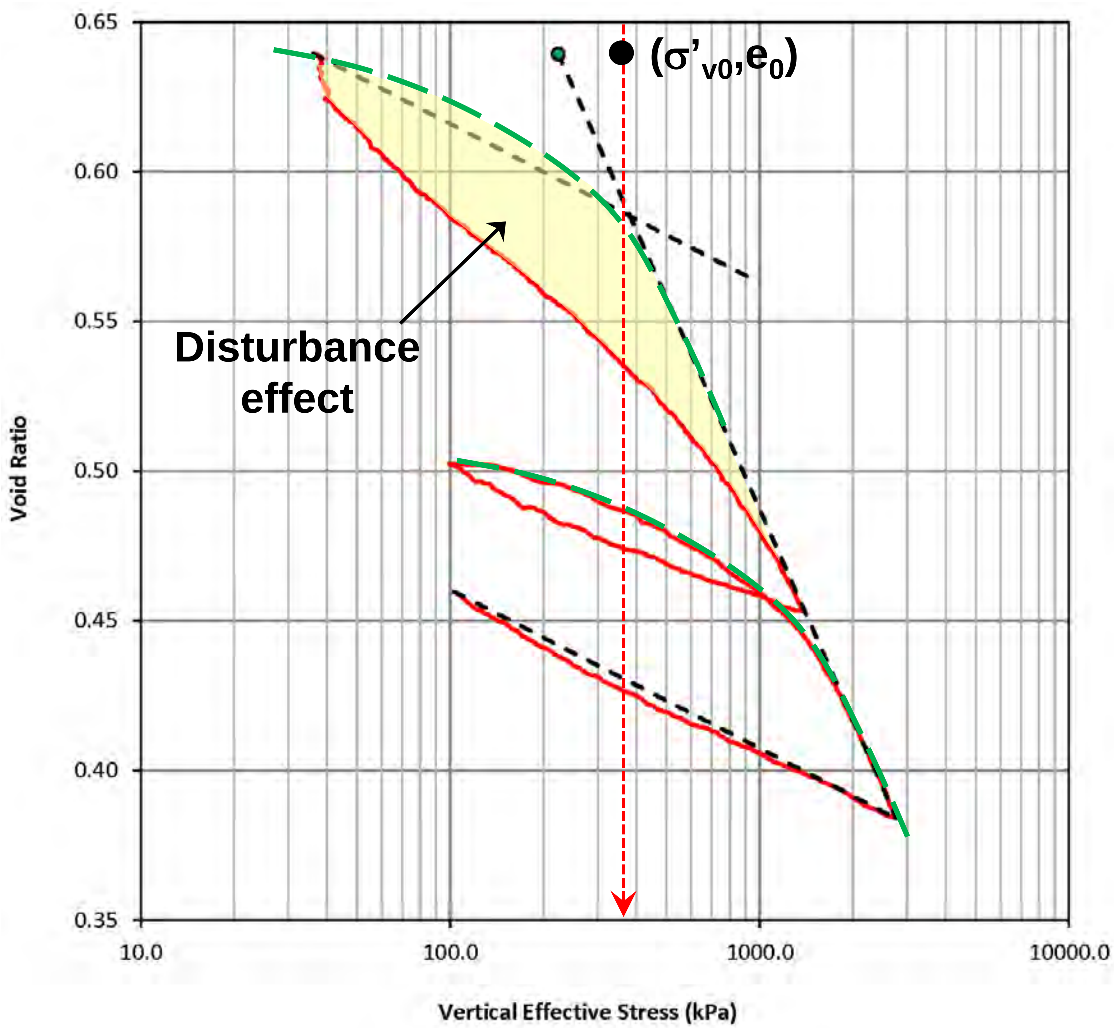
Radiography (Gamma-Ray Scans) to Investigate Sample Quality and Select Test Samples

Stress History

Estimation of Preconsolidation Stress

- Graphical techniques (e.g., Casagrande, Becker, Pacheco Silva) giving unreasonably-high values of preconsolidation stress
- Resolved to use Boone (2010) method
  - *"The initial part of the test curve represents only recompression of the swelling and disturbance associated with sampling, specimen preparation, saturation, and test set-up."*
  - *"Once the applied effective stress is equal to the in situ vertical stress, the void ratio should be representative of the in situ void ratio, e_{v0}, excluding the potential for significant sample disturbance."*
  - Preconsolidation determined as intersection of virgin compression line (VCL) and line drawn through point (σ'_{v0}, $e_{v0} = e_0$) with slope equal to C_S based on unload-reload curves

Stress History (cont.)



Example: BH-003, ST-3
σ'_{vp} = 360 kPa = σ'_{v0}

Note 1: when VCL is "left" of σ'_{v0} at e_0, σ'_{vp} assumed to be σ'_{v0} (normally consolidated)

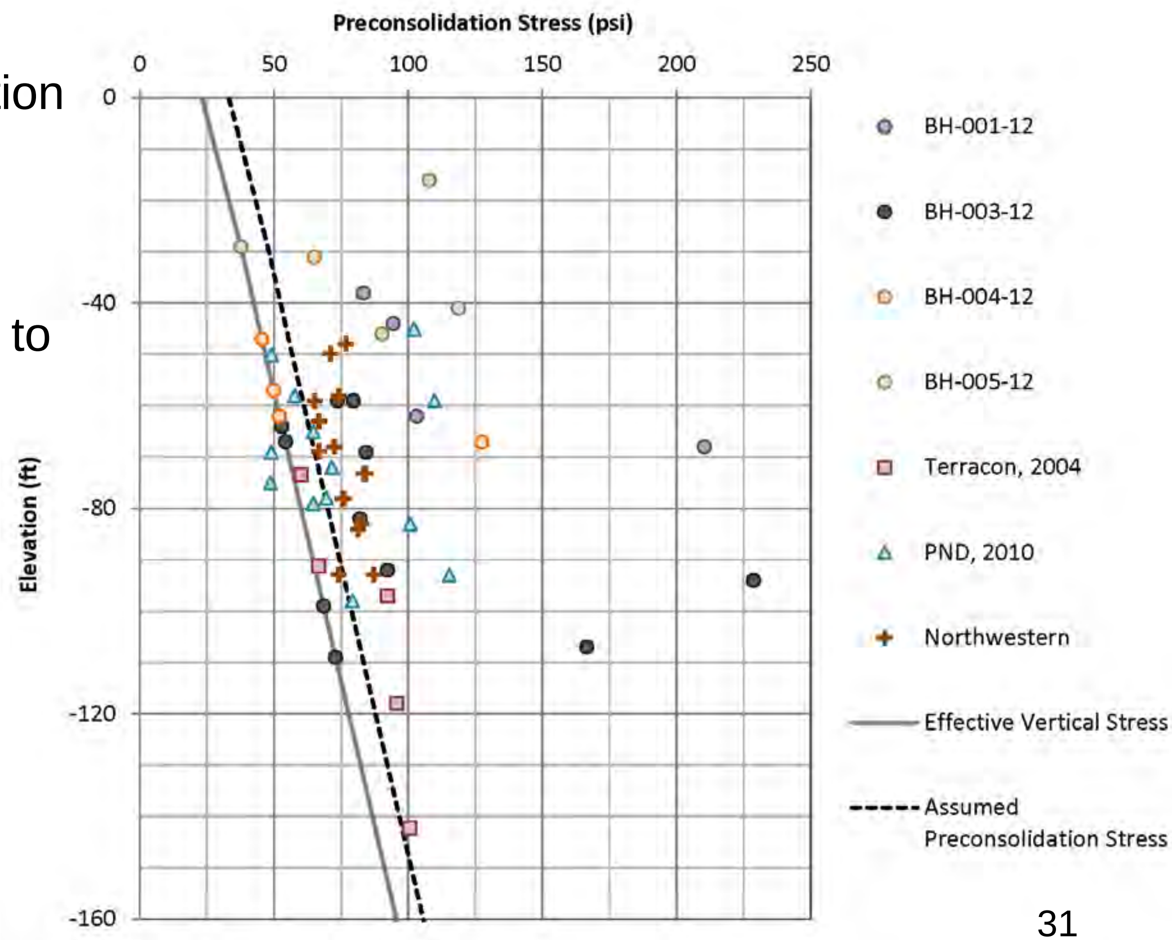
Casagrande construction technique, by inspection, would give preconsolidation stress around 1,000 kPa (145 psi)

Note 2: average Λ = 0.79

Stress History (cont.)

Characterization of Stress History

- New tests confirm previous preconsolidation stress profile is reasonable and was adopted for this study
- Stress history appears to be uniform across the Port site



Peak Effective-Stress Shear Strength

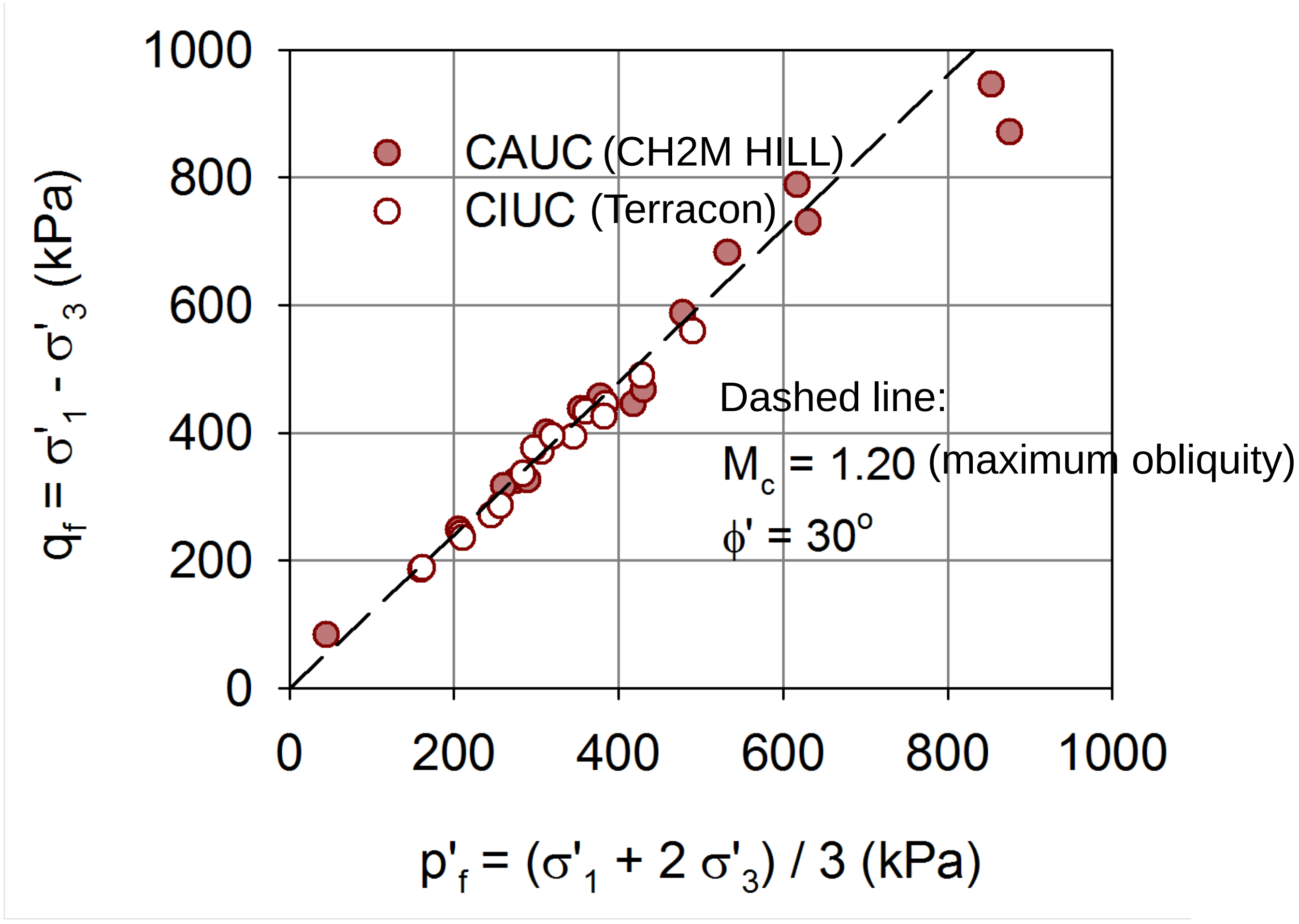
Peak Effective Friction Angle, ϕ', Based on CIUC and CAUC

- Recall other work:
  - Lade (1985) CIUC: 29 degrees
  - Terracon (2004) CIUC: 29 degrees (27 degrees used for design)
  - Zapata-Medina (2012): 28 to 31 degrees (30 degrees for modeling; match observed in PLAXIS simulation of laboratory tests)
  - **CH2M HILL (2012): 30 degrees**; data show around 29 degrees (higher value selected, in part, based on fit of MCC predictions of undrained strength ratios with measured strengths)

Effective Friction Angle, ϕ', Based on DSS

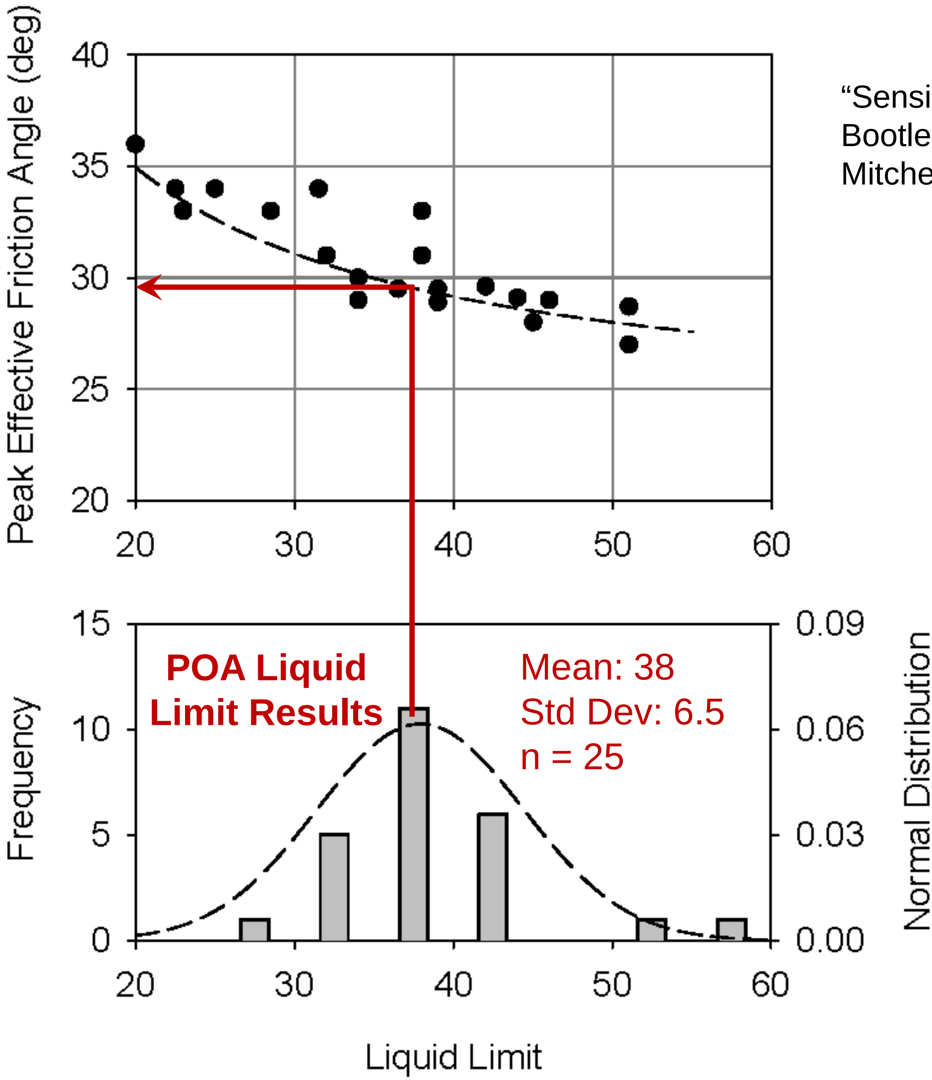
- Not recommended by Ladd (1980's), but…
- 30 degrees seems appropriate based on plots of shear stress vs. effective vertical stress (maximum obliquity)

Peak Effective-Stress Shear Strength (cont.)



Peak Effective Friction Angle Based on New CAUC

Peak Effective-Stress Shear Strength (cont.)



“Sensitivity and geotechnical properties of Bootlegger Cove clay”
Mitchell, Houston, and Yamane

CIUC Tests on BCC:
“Effective friction angle of shearing resistance ϕ’ was found to vary with liquid limit (LL) as shown. This decrease of ϕ’ with increase in LL is consistent with results for other clays (Bjerrum, 1961).”

Peak Undrained Shear Strength

General Approaches to Undrained Shear Strength Characterization

- Measure DSS and CAUC (TXC) undrained strength ratios at different OCRs
- Use effective friction angle to estimate DSS and CAUC undrained strength ratios based on modified Cam-Clay model
- Same methods as used by Terracon (2004)

Stress History and Normalized Soil Engineering Properties (SHANSEP)

- Samples “reconsolidated” past in situ preconsolidation stress and then unloaded to achieve overconsolidated condition
- Purpose of stress-path loading and unloading is to remove disturbance effects

Recompression Technique

- Samples “recompressed” to varying stress levels
- Sample OCR determined as in-situ σ'_{vp} / σ'_{vc}
- May provide higher strength/modulus (SHANSEP is typically conservative)
- Results for this method depend heavily on sample quality/disturbance!

Peak Static Undrained Shear Strength (cont.)

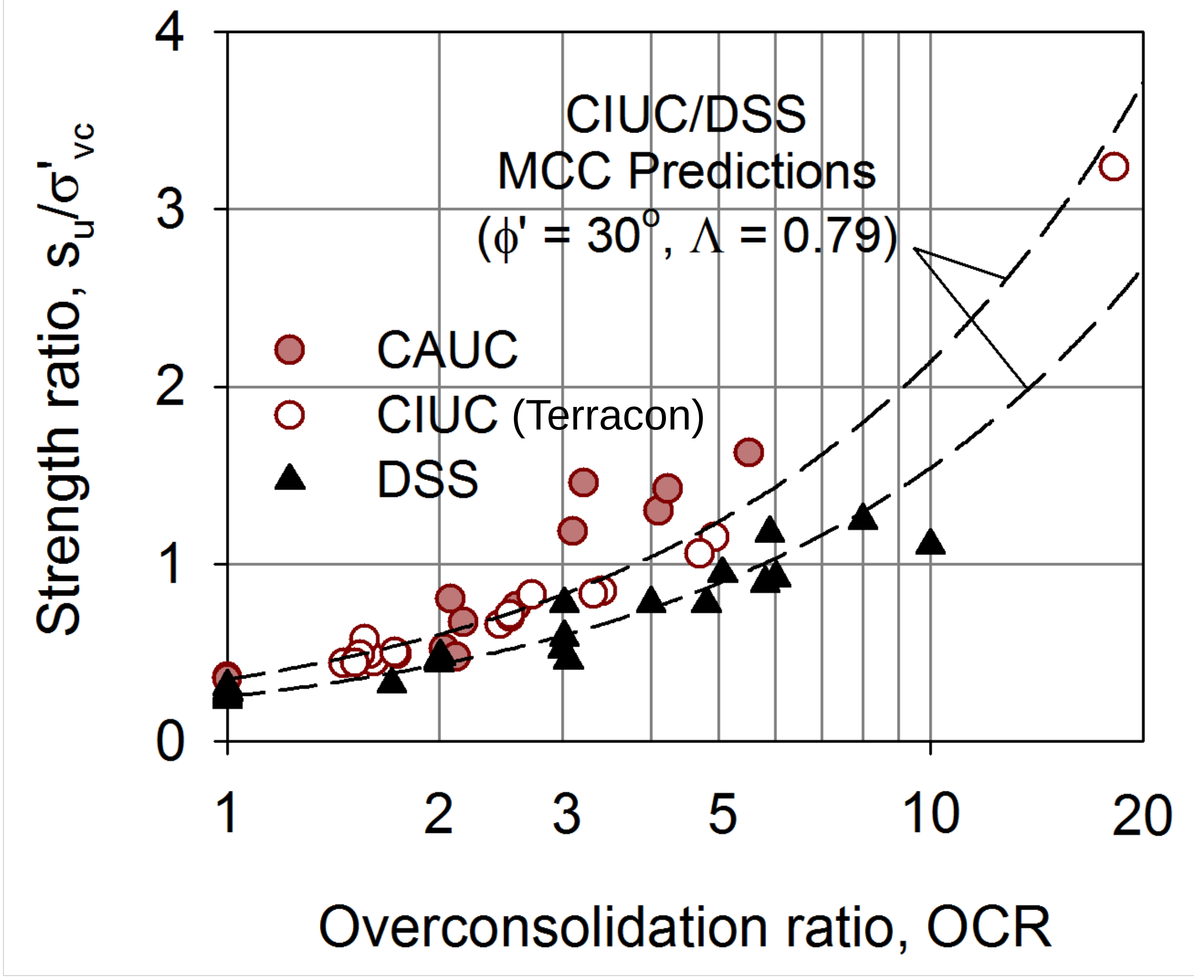
Direct Simple Shear (DSS) Testing

- 30 DSS tests conducted (15 SHANSEP, 15 Recompression)
- OCR ranges from 1 to about 8.5
- Average measured $(s_u/\sigma'_{vc})_{NC}$: 0.277
  (OCR = 1 tests only)
- MCC prediction based on 30 deg: **0.25**

Triaxial Compression (CAUC) Testing

- 19 CAUC tests conducted (15 SHANSEP, 4 Recompression)
- OCR ranges from 1 to about 9
- Average measured $(s_u/\sigma'_{vc})_{NC}$: 0.35
  (both OCR = 1 tests)
- [Wroth CK_0UC] MCC prediction based on 30 deg: 0.30, but…
- MCC prediction corrected to **0.34**, as discussed

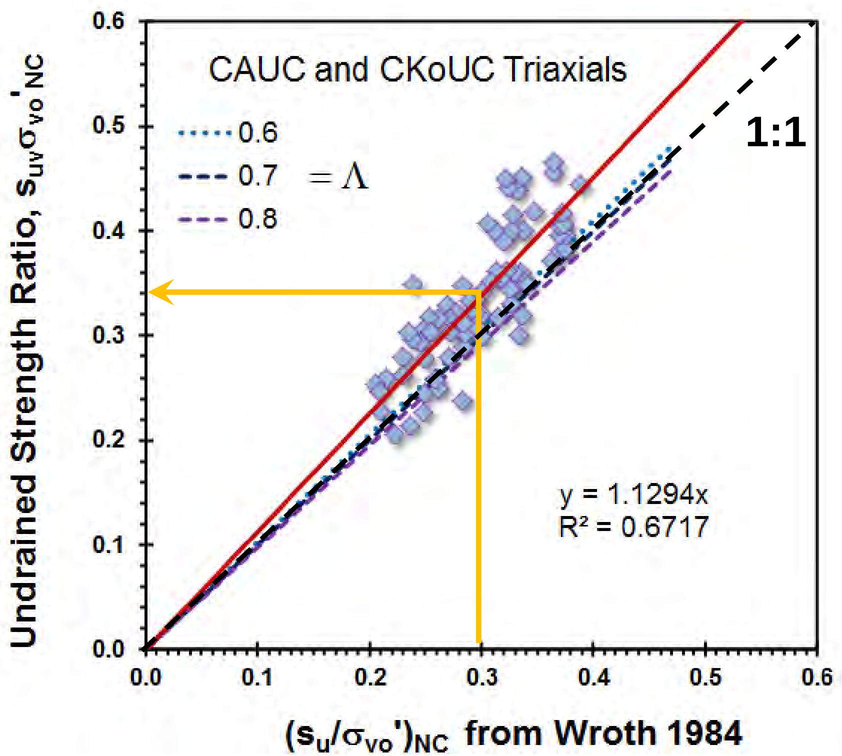
Peak Static Undrained Shear Strength (cont.)



Peak Static Undrained Shear Strength (cont.)

Modified Cam-Clay Prediction of CAUC Undrained Shear Strength

- For $\phi' = 30$ deg, $(s_u/\sigma'_{vc})_{NC}$ based on Wroth is 0.30 for CK_0UC
- Testing of other natural clays has shown this to be conservative (i.e., undrained strength ratio not very sensitive to ϕ')
- Correction of 1.1294 used to increase CAUC strength ratio from 0.30 to 0.34.
- Strength ratio of 0.34 compares with 0.35 for CIUC, just slightly more conservative.



Data courtesy of Paul Mayne

Cyclic Behavior

Bender Element Shear Wave Velocity

- Conducted on 12 of 16 samples after consolidation, before cyclic loading (V_s = 1150 to 1350 ft/s) – agree with field V_s measurements

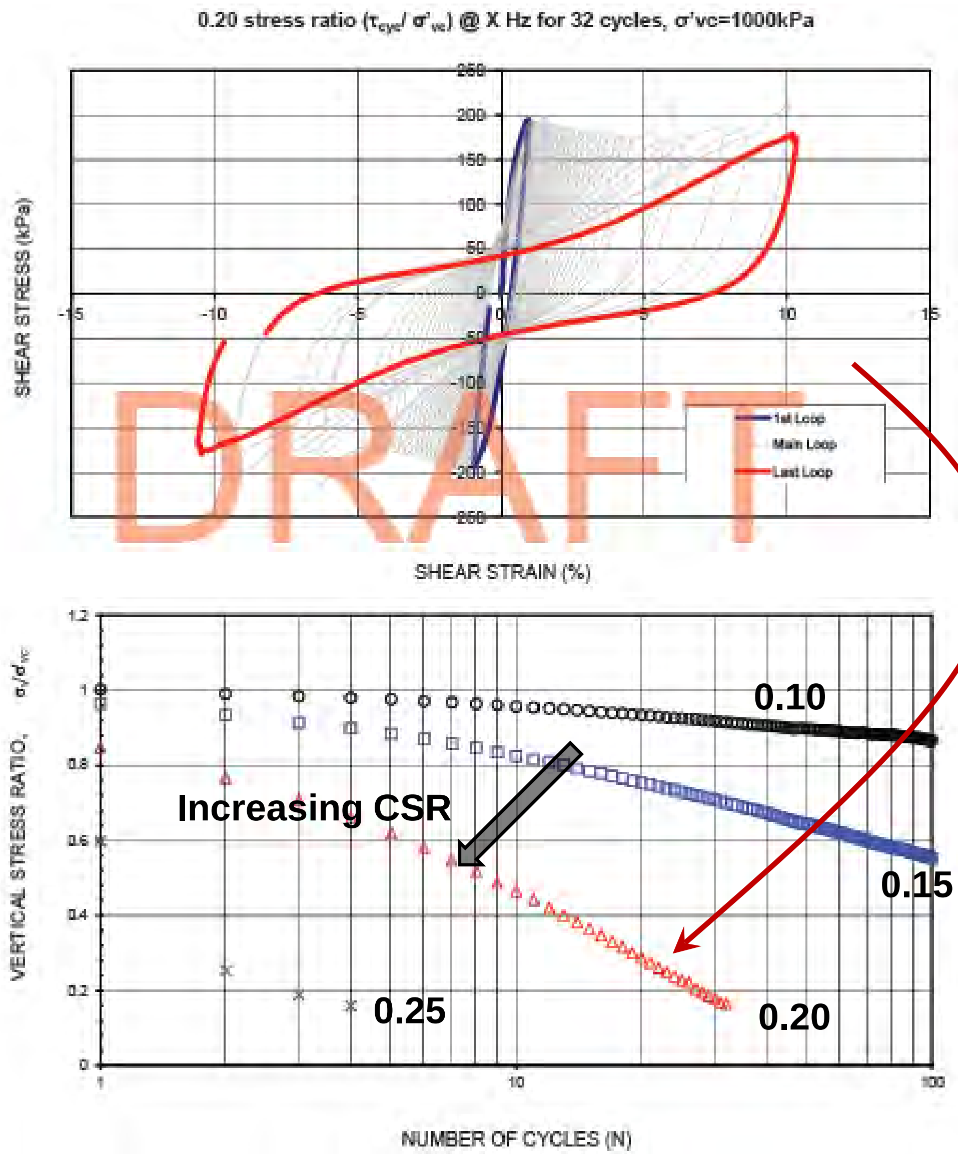
Cyclic DSS (CyDSS) Testing

- Stress-controlled tests
- Facies F.IV and F.I
- OCR = 1, 2
- Each of four test series including four CSR values
- One test series including a static shear bias

Post-cyclic DSS Testing

- Immediately after cyclic loading (i.e., no drainage allowed), each sample sheared monotonically

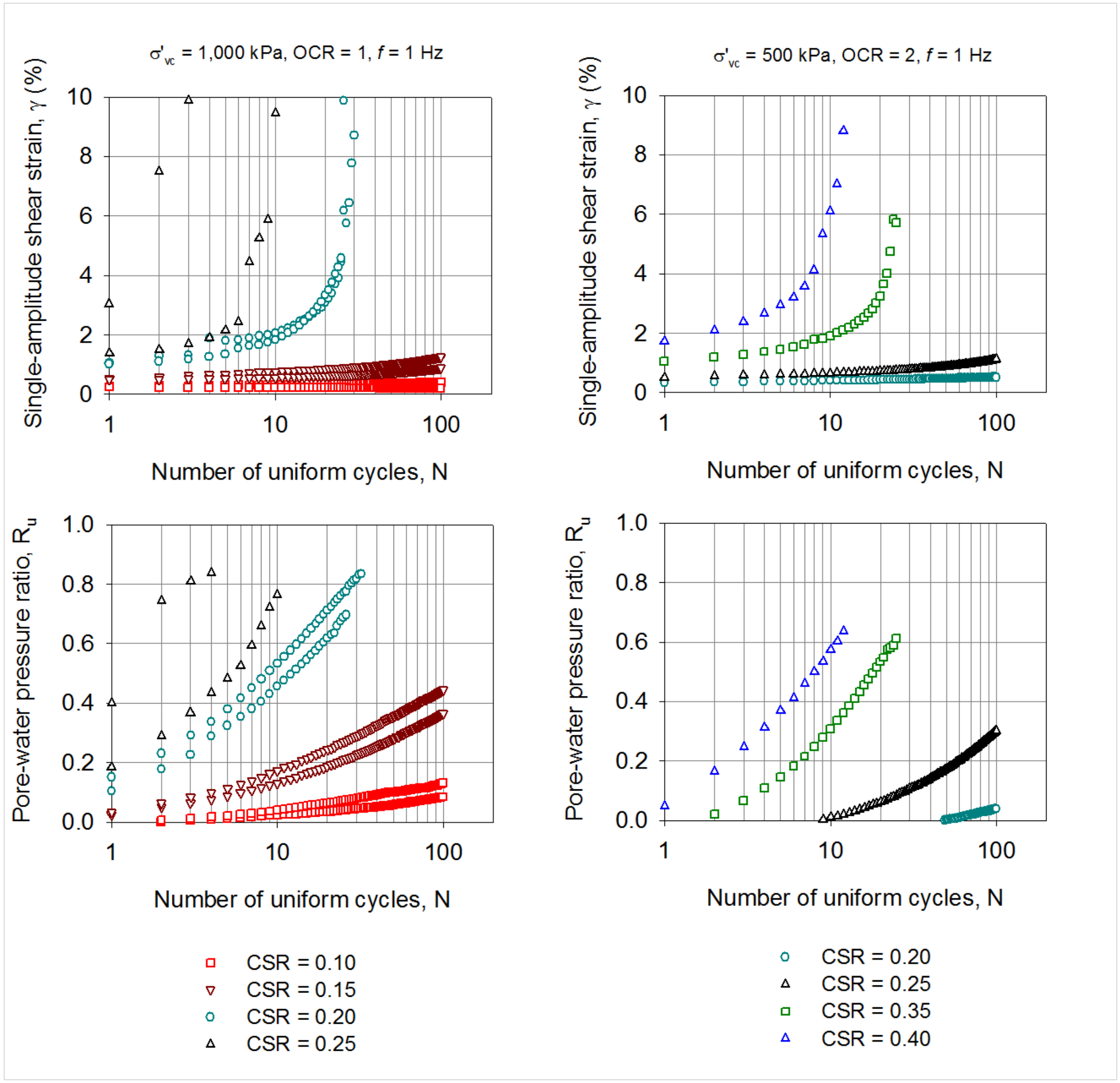
Cyclic Behavior (cont.)



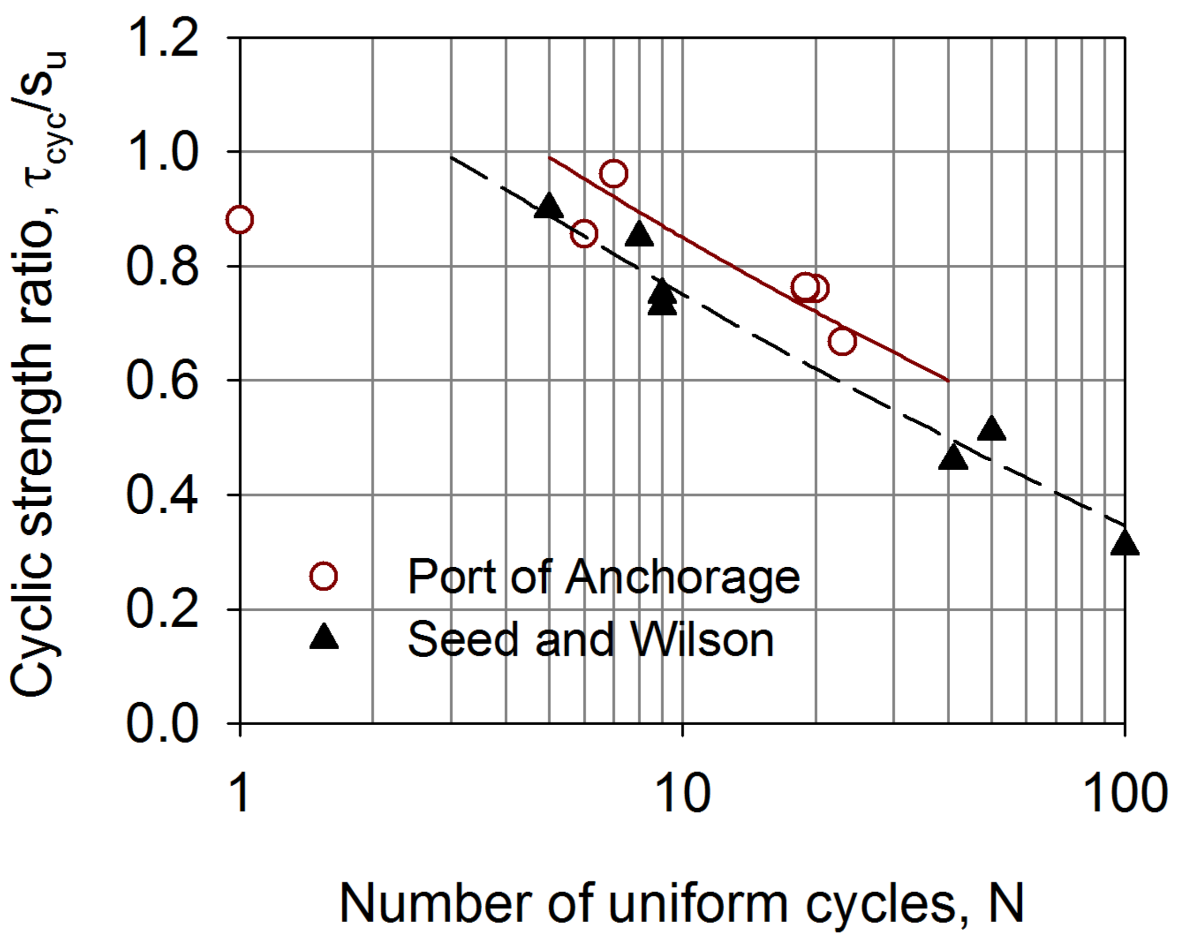
Example CyDSS data/results from most recent testing program

CyDSS test results in PND (2010) show similar response

Cyclic Behavior (cont.)



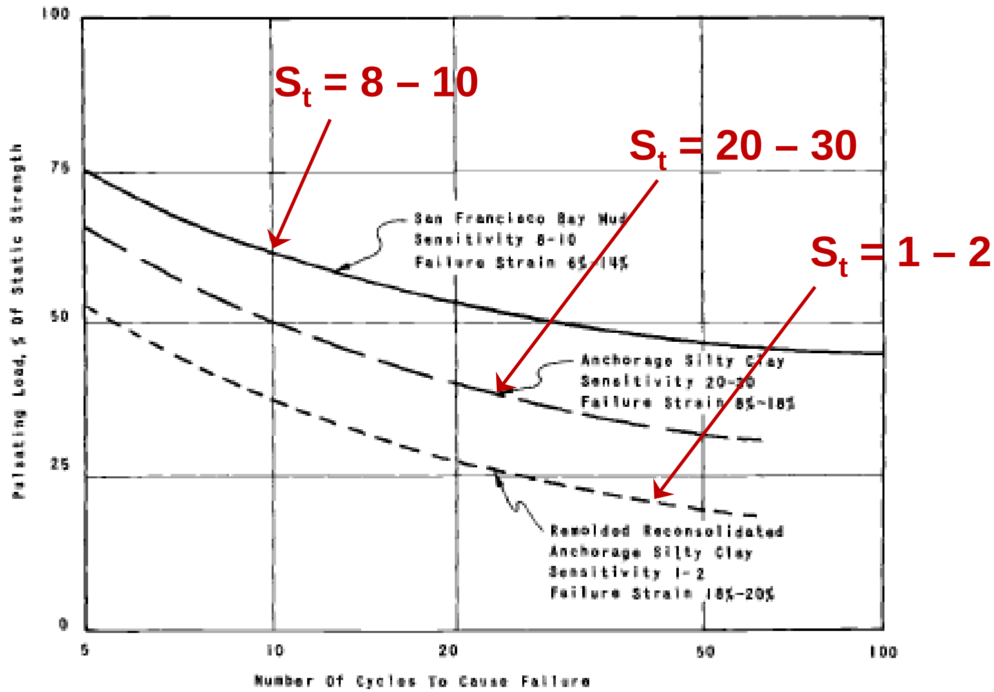
Cyclic Behavior (cont.)



Cycling Behavior (cont.)

*“It is evident that the **sensitivity** of a soil is **<u>not</u>** a good indicator of its susceptibility to strength loss under cyclic loading conditions.”*

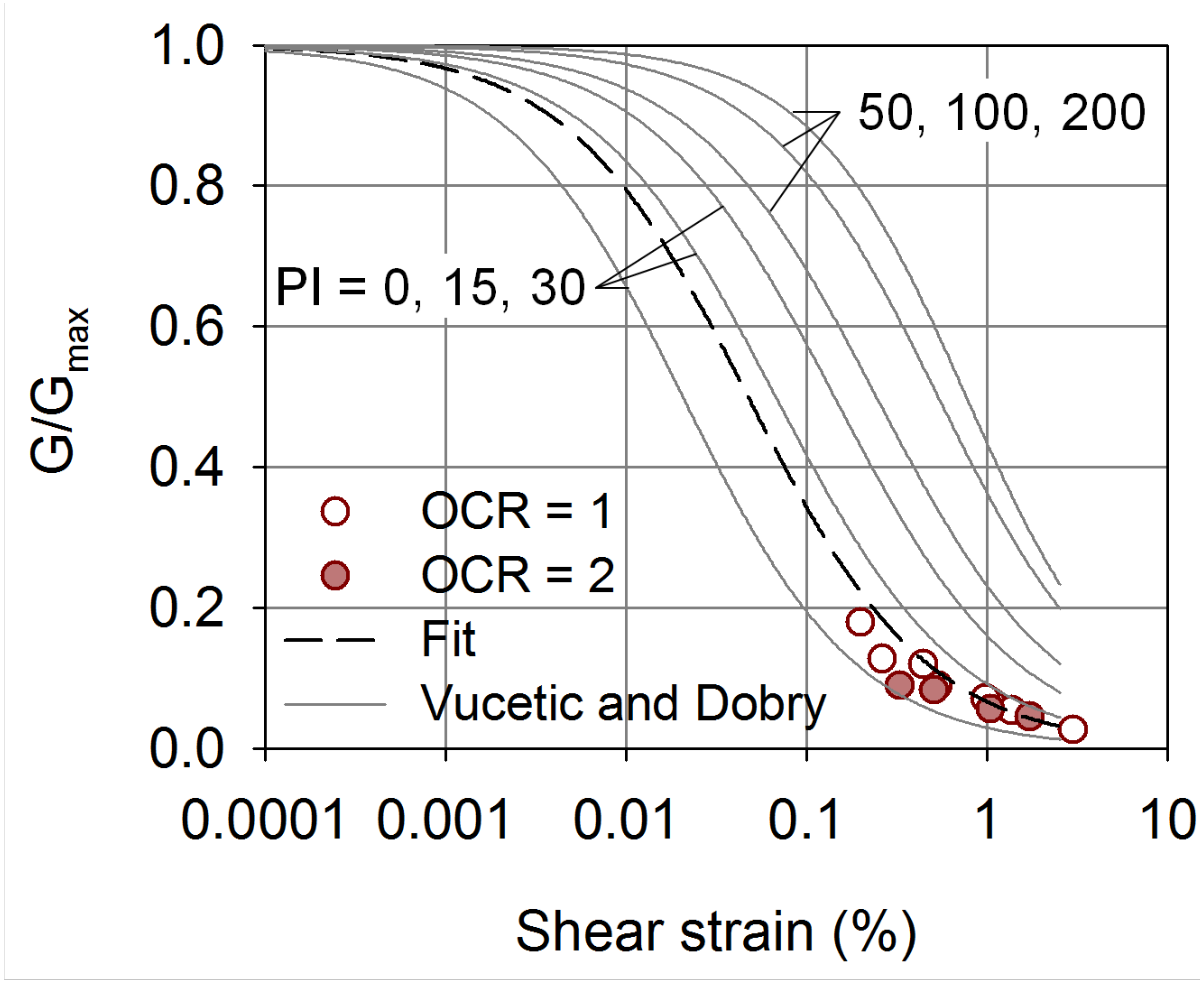
Thiers and Seed (1969). “Strength and stress-strain characteristics of clays subjected to seismic loading conditions.”



“Low” sensitivity of the Bootlegger Cove clay cannot be used as justification for ignoring potential strength loss during seismic event!

Consider effects of excess pore-water pressure and cyclic strain on cyclic strength

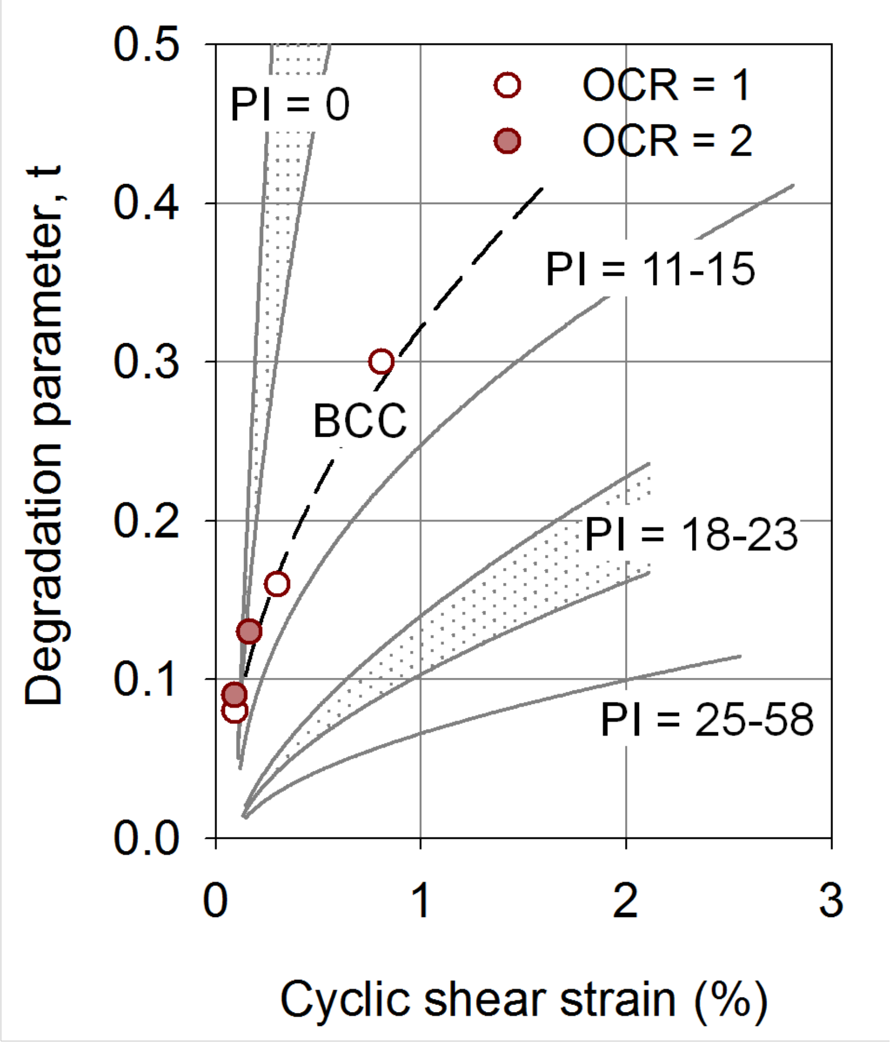
Cyclic Behavior (cont.)



Modulus reduction representative of a low-plasticity clay

G taken as secant modulus for first cycle of loading on each specimen; G_{max} from bender element shear wave velocity

Cyclic Behavior (cont.)



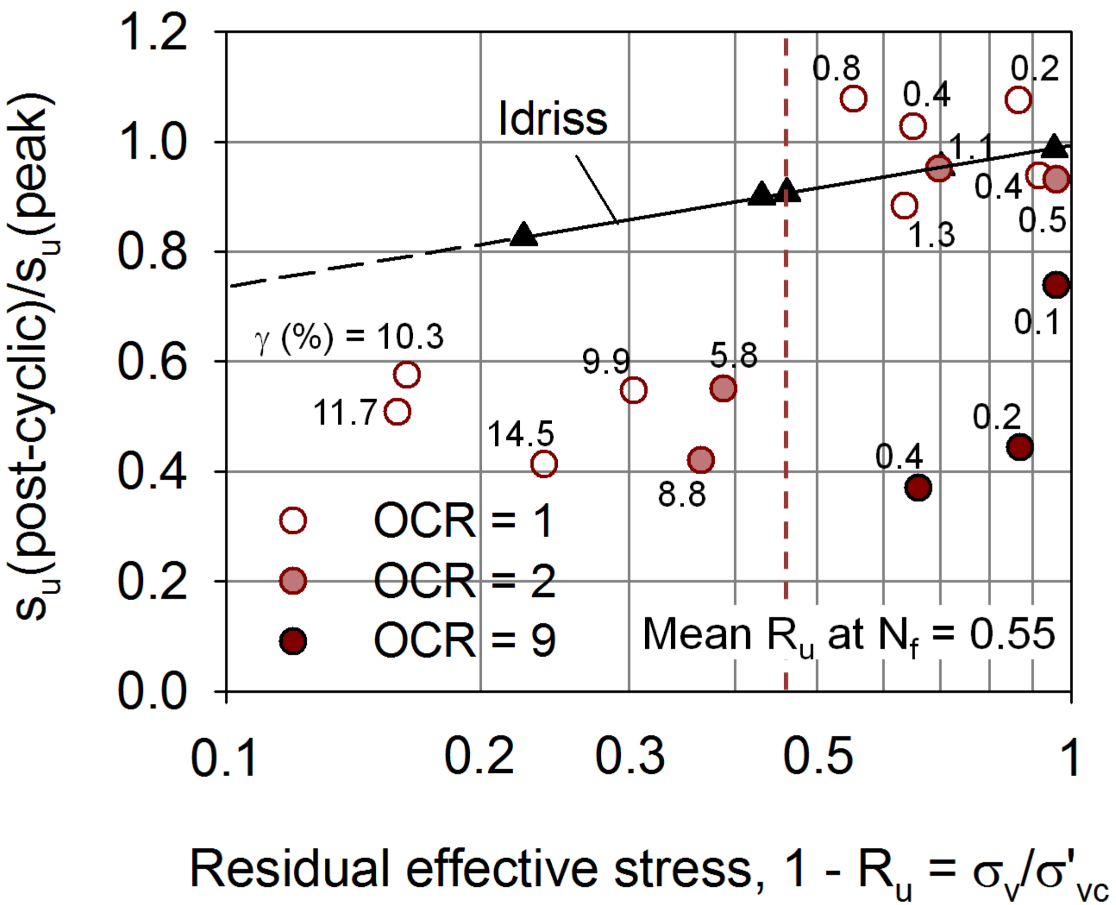
Cyclic stiffness degradation:

$$\delta = G / G_N = N^{-t}$$

where, t = degradation parameter

Degradation parameter also representative of a low-plasticity clay, referencing Tan and Vucetic (1989) and Vucetic (1994)

Post-Cyclic Shear Strength (cont.)



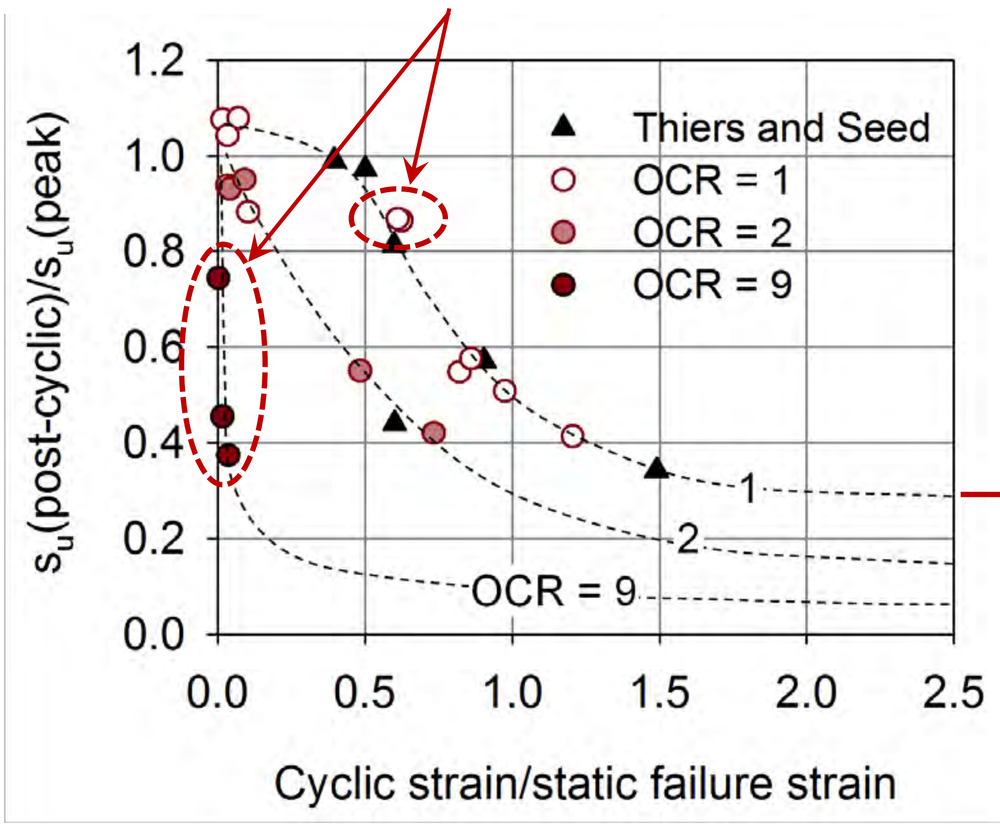
Idriss (1985). "Evaluating seismic risk in engineering practice."

Variation of post-cyclic undrained strength with excess pore-water pressure ratio induced during cyclic load application

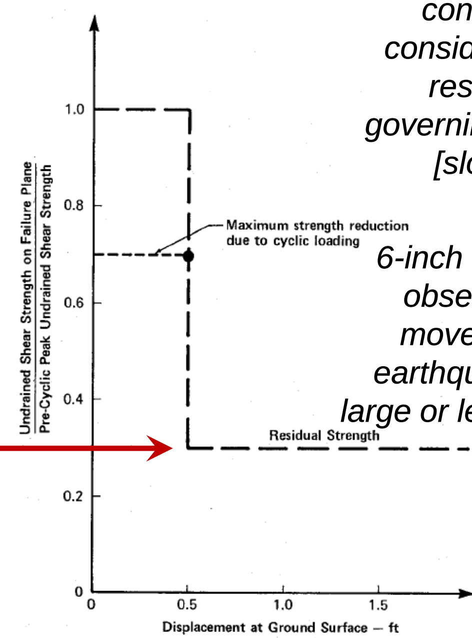
Li et al. (2011):
"Excess pore-water pressure continuously increases with the number of cycles, and no inflection appears, regardless of cyclic failure. Water pressure lag behaves seriously if the clay sample is at high frequency or large cyclic stress conditions. Thus, the **accumulative pore-water pressure is not suggested to estimate the cyclic failure for natural clays**."

Post-Cyclic Shear Strength (cont.)

Post-cyclic test data by MEG for PND (2010) design



Following Thiers and Seed (1969)



Idriss (1985), Moriwaki (1985), Idriss (2010)

Under high-displacement conditions, one must consider use of undrained residual strength as governing resistance against [slope] movements

6-inch delineation based on observation that ground movements during 1964 earthquake were either very large or less than about 6 inches

Undrained Residual Strength

Current Ring Shear Testing Program

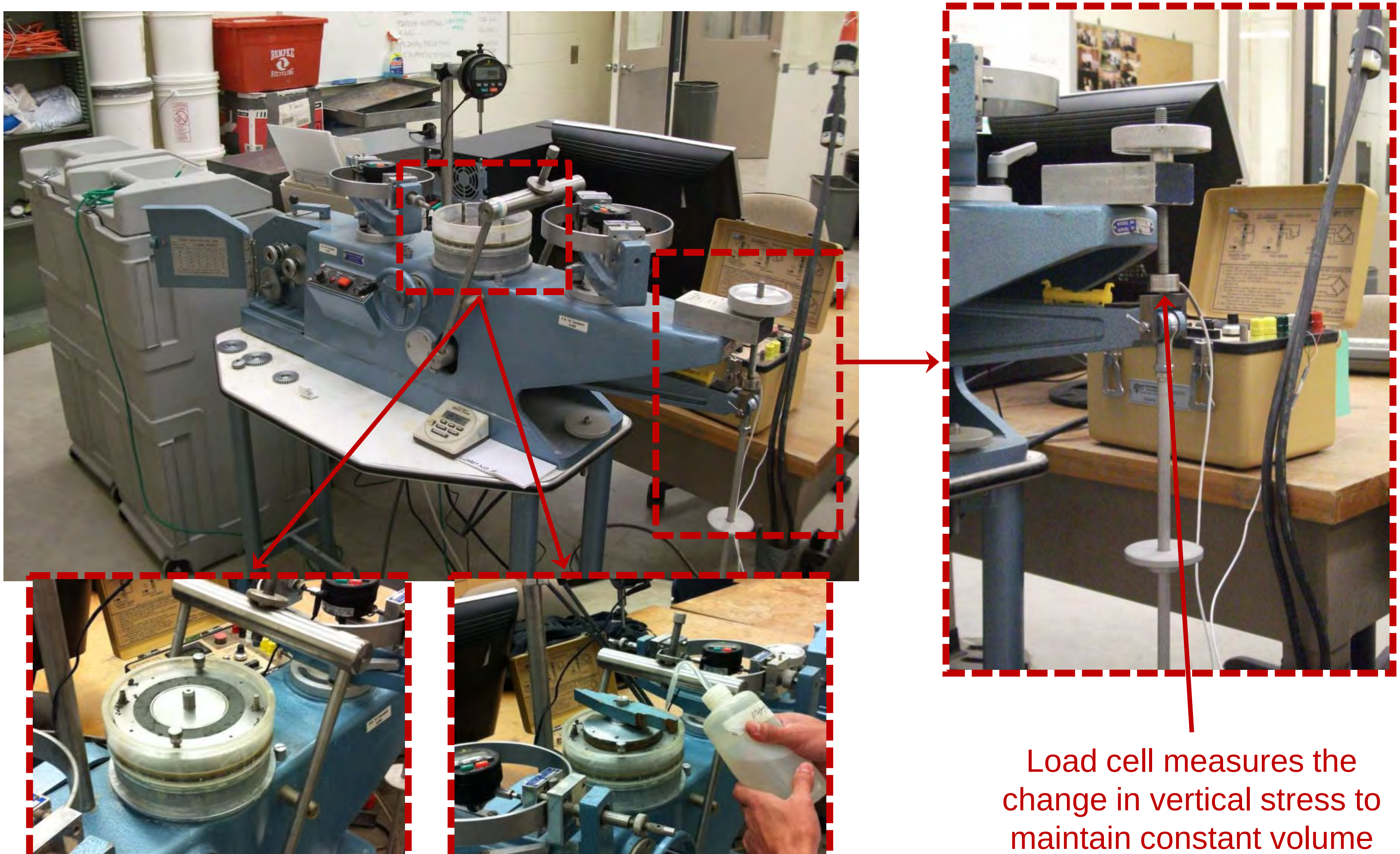
- Constant volume (i.e., undrained) tests at σ'_{vc} = 100, 200, 300, 400, 600 kPa (OCR from 1 to about 4.5)
- Constant stress (i.e., drained) tests at σ'_{vc} = 100, 400, 600 kPa; ASTM
- One "faster" test under constant volume conditions, σ'_{vc} = 400 kPa
- Two "healing" tests at σ'_{vc} = 400 kPa (undisturbed and remolded)

Other Ring Shear and Back-Analyses Involving Bootlegger Cove Clay

- Port MacKenzie constant volume ring shear tests on BCC
- Fourth Avenue landslide Bootlegger Cove clay ring shear tests
- Back-analyses
  - Fourth Avenue
  - L Street
  - Government Hill

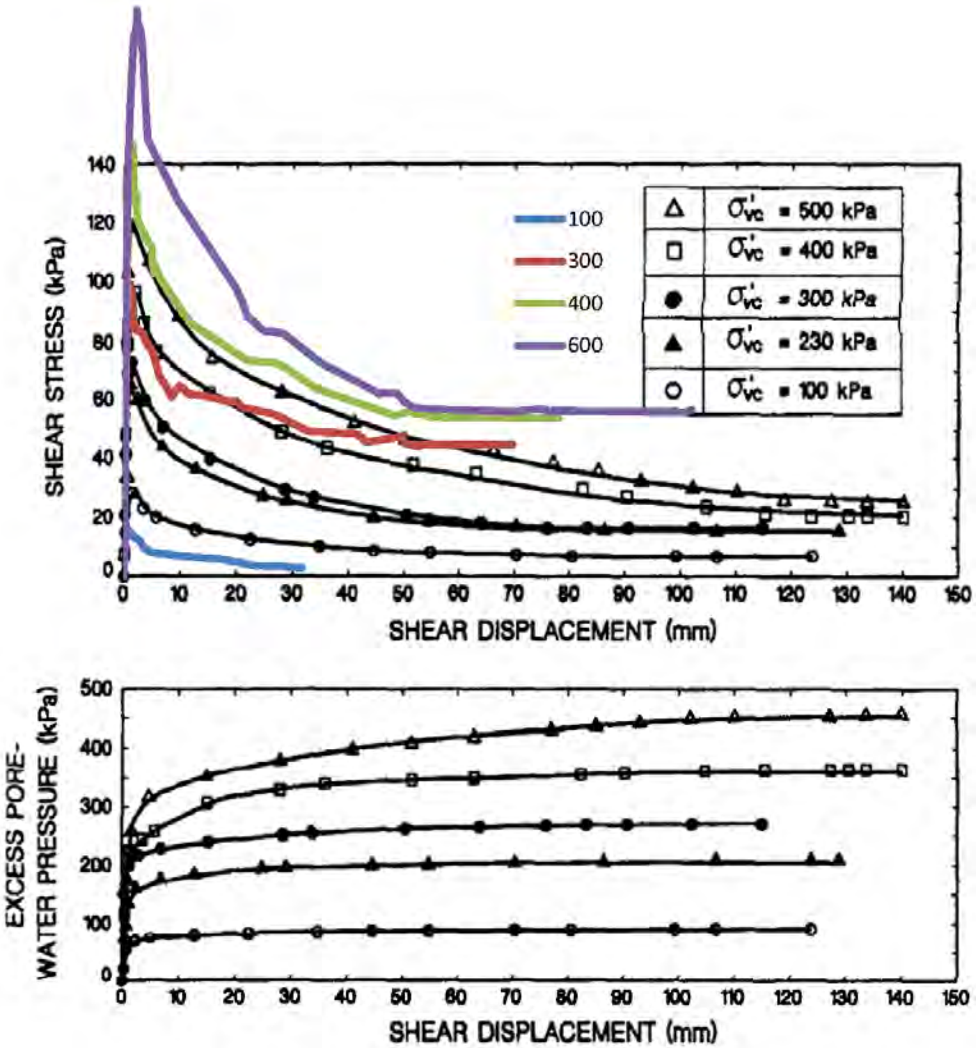
Ring shear device only apparatus to measure strength at very large strains

Undrained Residual Strength (cont.)



Constant Volume Ring Shear Test Apparatus

Undrained Residual Strength (cont.)



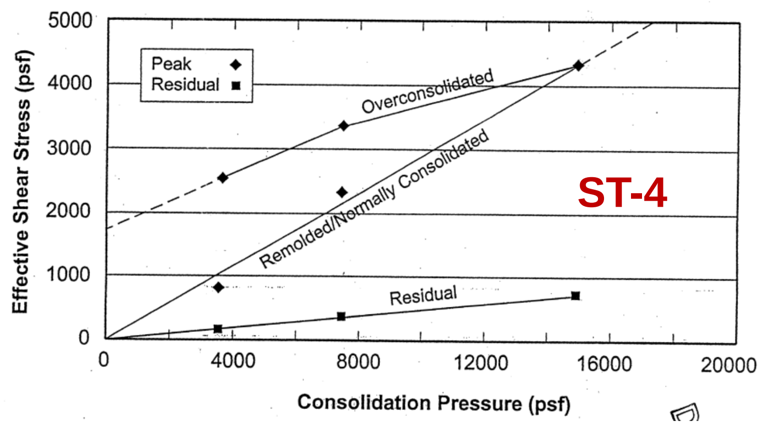
Port of Anchorage Bootlegger Cove clay select test results plotted over the test results from Fourth Avenue Bootlegger Cove clay

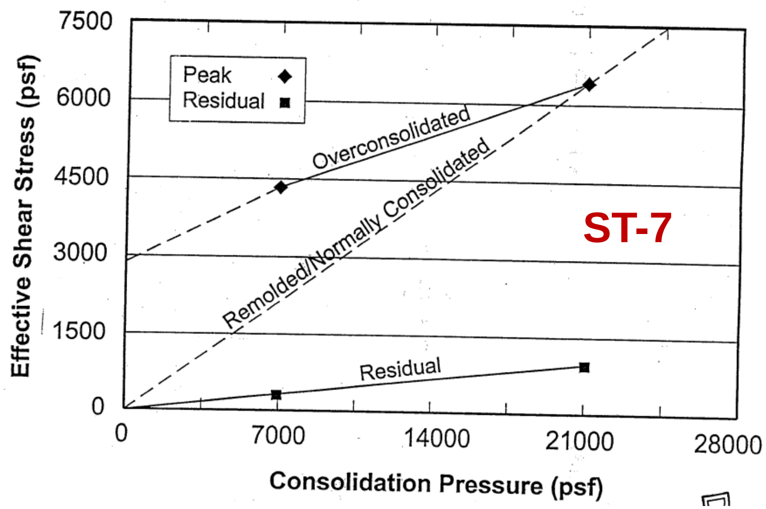
Under constant volume conditions, both materials show considerable post-peak strength reductions resulting from excess pore-water pressure "generation"

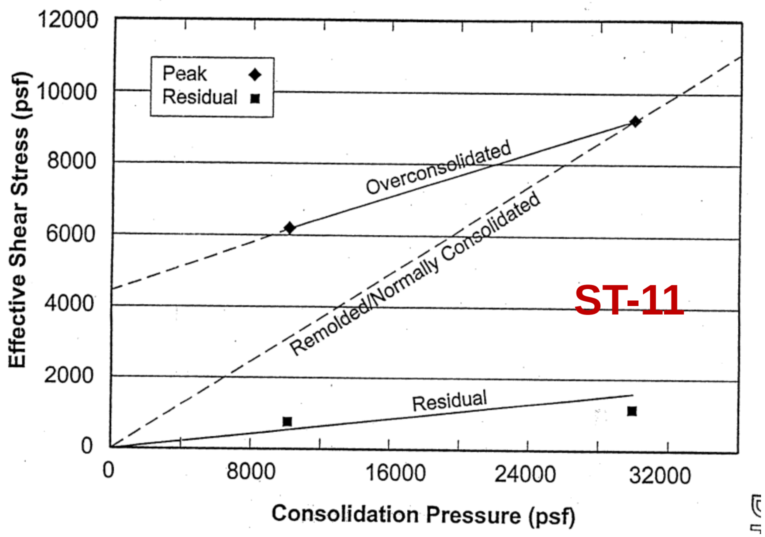
POA Constant Volume Ring Shear Test Results, Comparison with Fourth Avenue

Undrained Residual Strength (cont.)

Port MacKenzie Bootlegger Cove Clay Ring Shear Test Results from 2001

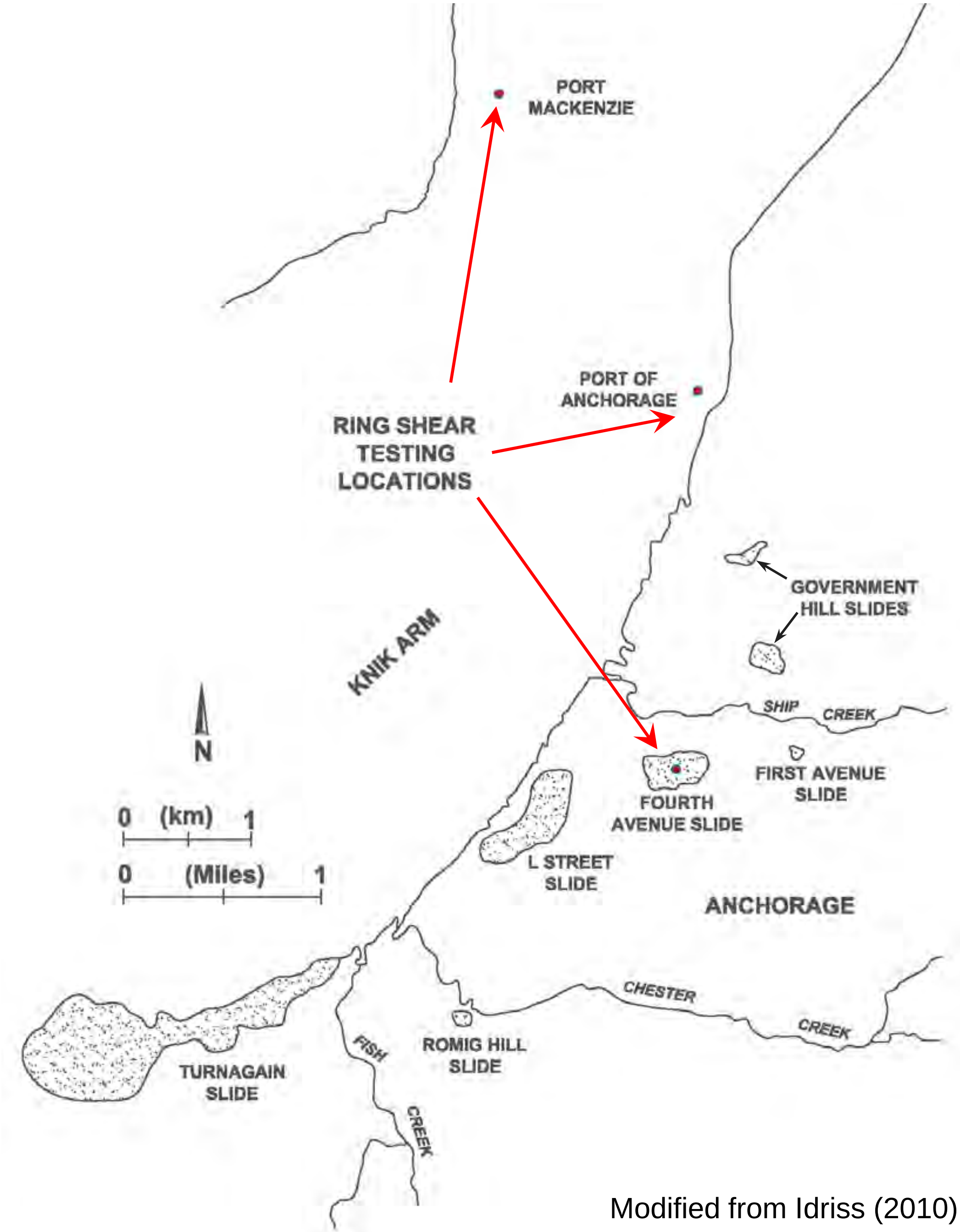






Undrained residual strength at large displacements about 10 to 20% of NC peak

Constant Volume Ring Shear Tests



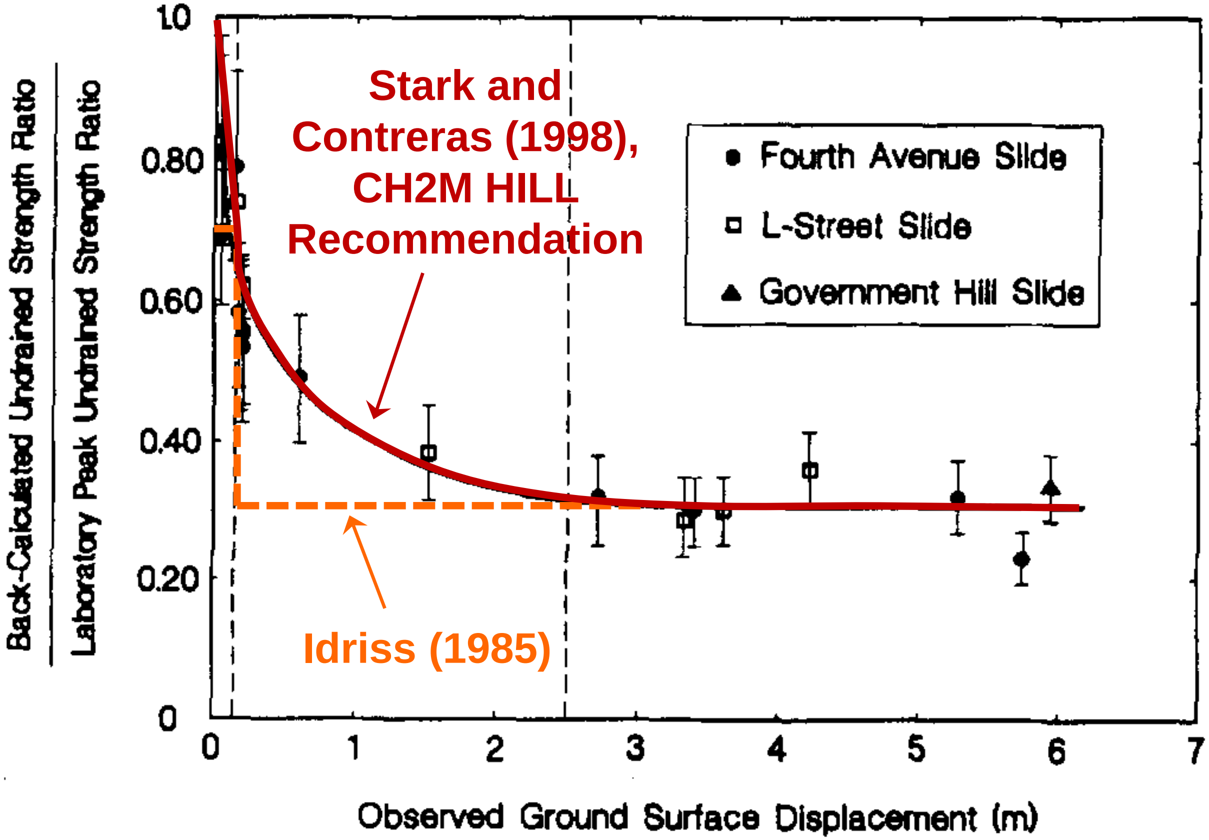
Modified from Idriss (2010)

Constant volume ring shear tests conducted on Bootlegger Cove clay from Fourth Avenue slide area (inside and outside slide mass), Port MacKenzie, and Port of Anchorage

Large-displacement, undrained shear behavior the same at all three locations!

Undrained Residual Strength (cont.)

How are ring shear results and back-analyses used?

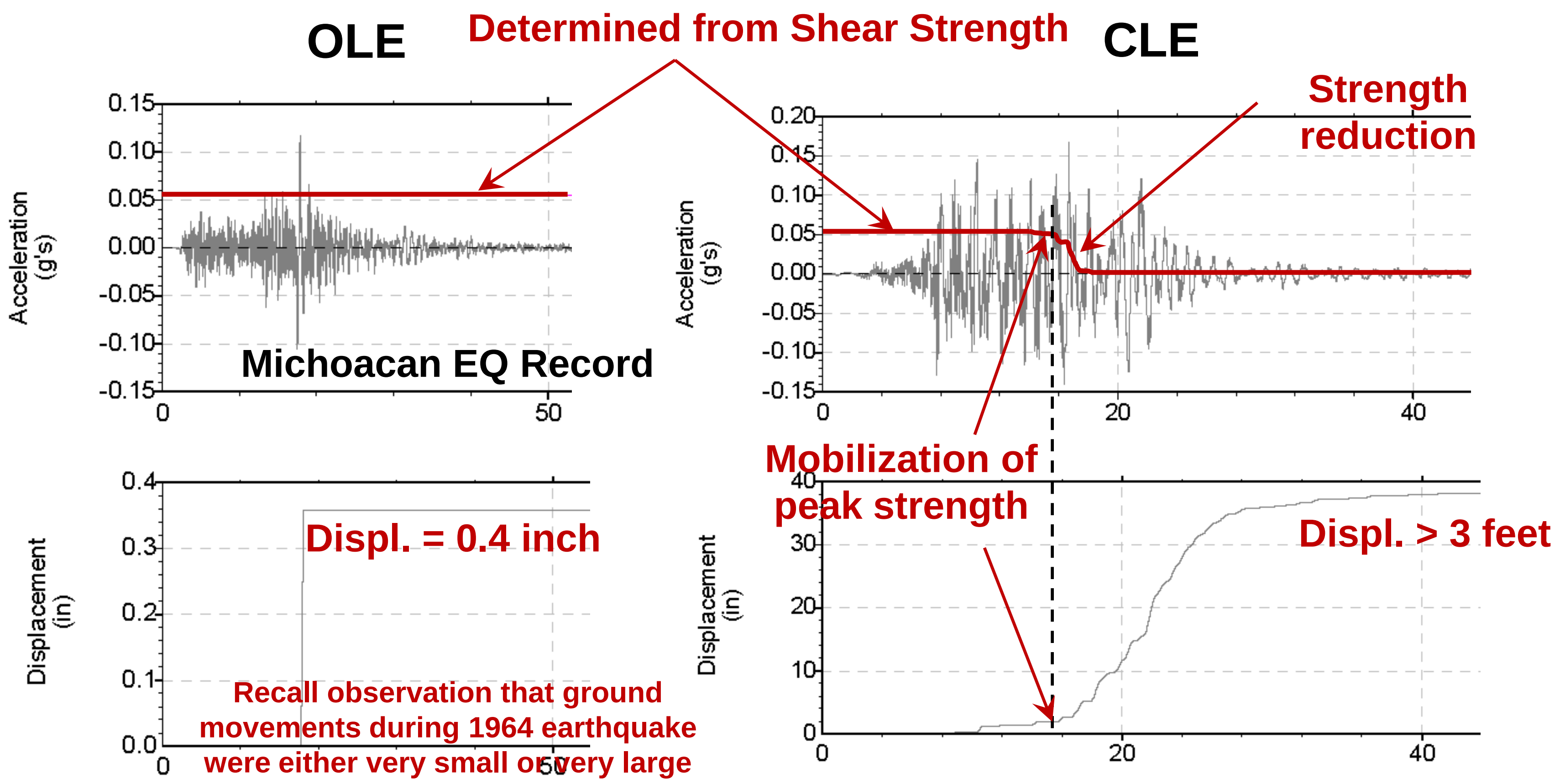


Variation of normalized undrained shear strength ratio with ground surface displacement

Based on back-analyses of landslides occurring during the 1964 earthquake: Fourth Avenue, L Street and Government Hill

Undrained Residual Strength (cont.)

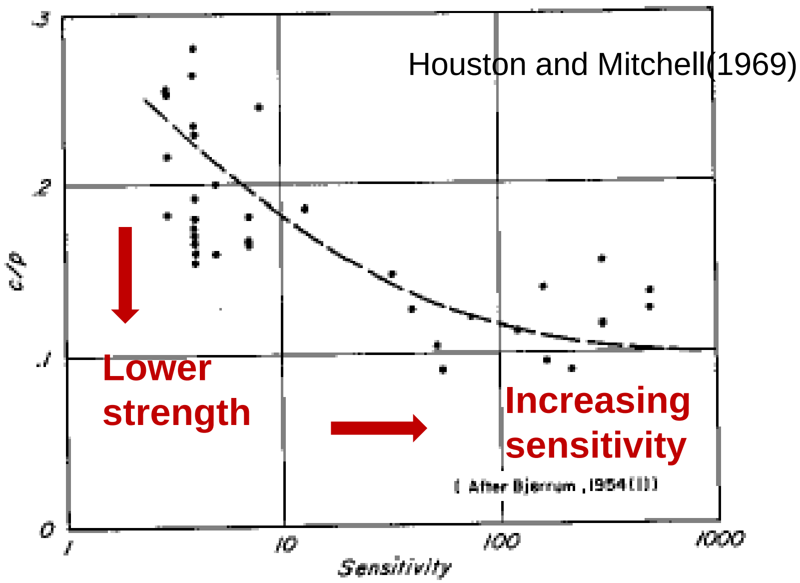
How are ring shear results and back-analyses used?

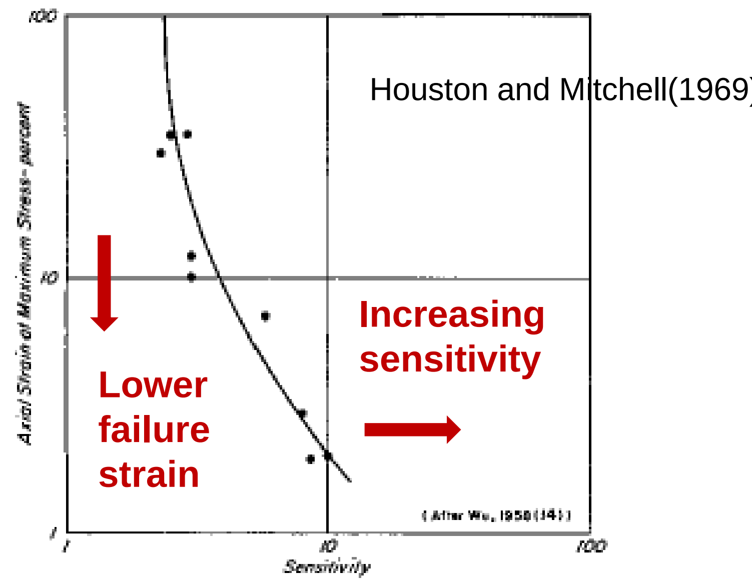


Example Newmark Seismic Displacement Analysis with Updated Shear Strength

Undrained Residual Strength (cont.)

Why are the 1964 landslides so commonly associated with the sensitive facies of BCF?





Reason 1:
Sensitive clays may exhibit **lower undrained strength ratios** and may, therefore, be more susceptible to ground failure

Reason 2:
Sensitive clays may mobilize peak strength at **lower strains or displacements** and, thus, post-peak strength reductions will occur sooner

A potential shear plane may more likely develop in a sensitive clay (if present) because of lower shear strength and more brittle behavior (i.e., "card-house" structure), not solely due to the sensitivity of the clay. **This observation does not preclude the same failures in "low sensitivity" clays of similar peak undrained shear strength and sufficiently high shear loading!**

Undrained Residual Strength (cont.)

Soil Deposit	LL	PL	Sensitivity	s_u/σ'_v	s_{ur}/σ'_v	s_{ur}/s_u
Bootlegger Cove clay, **inside** Fourth Ave. landslide, Anchorage, Alaska	40	20	3 – 11	0.28	0.07	0.25
	34	19		0.28	0.07	0.25
	36	21		0.24	0.06	0.25
	38	21		0.23	0.06	0.26
	39	20		0.23	0.06	0.26
Bootlegger Cove clay, **outside** Fourth Ave. landslide, Anchorage, Alaska	42	23	3 – 11	0.31	0.11	0.35
	40	21		0.32	0.10	0.31
	42	23		0.31	0.11	0.35
	41	22		0.30	0.11	0.37
Drammen clay, Danvik-gate, Drammen, Norway	47	23	4 – 10	0.27	0.09	0.33
	48	24		0.22	0.11	0.5
	47	25		0.20	0.11	0.55
Bootlegger Cove clay, Port of Anchorage, Anchorage, Alaska	34	20	1 – 3	0.25	0.03 – 0.16	0.16 – 0.50

Example Constant Volume Ring Shear Test Results on Different Clays

Undrained Residual Strength (cont.)

Soil Deposit	LL	PL	Sensitivity	s_u/σ'_v	s_{ur}/σ'_v	s_{ur}/s_u
Bootlegger Cove clay, **inside** Fourth Ave. landslide, Anchorage, Alaska	40	20	3 – **11**	0.28	0.07	0.25
	34	19		0.28	0.07	0.25
	36	21		0.24	0.06	0.25
	38	21		0.23	0.06	0.26
	39	20		0.23	0.06	0.26
Bootlegger Cove clay, **outside** Fourth Ave. landslide, Anchorage, Alaska	42	23	**3** – 11	0.31	0.11	0.35
	40	21		0.32		
	42	23		0.31		
	41	22		0.30	0.11	0.37
Drammen clay, Danvik-gate, Drammen, Norway	47	23	4 – 10	0.27	0.09	0.33
	48	24		0.22	0.11	0.5
	47	25		0.20	0.11	0.55
Bootlegger Cove clay, Port of Anchorage, Anchorage, Alaska	34	20	1 – 3	0.25	0.03 – 0.16	0.16 – 0.50

Stable during 1964 earthquake

Mobilization of peak strength the cause of seismic ground failure…no post-peak strength loss, if peak strength not mobilized

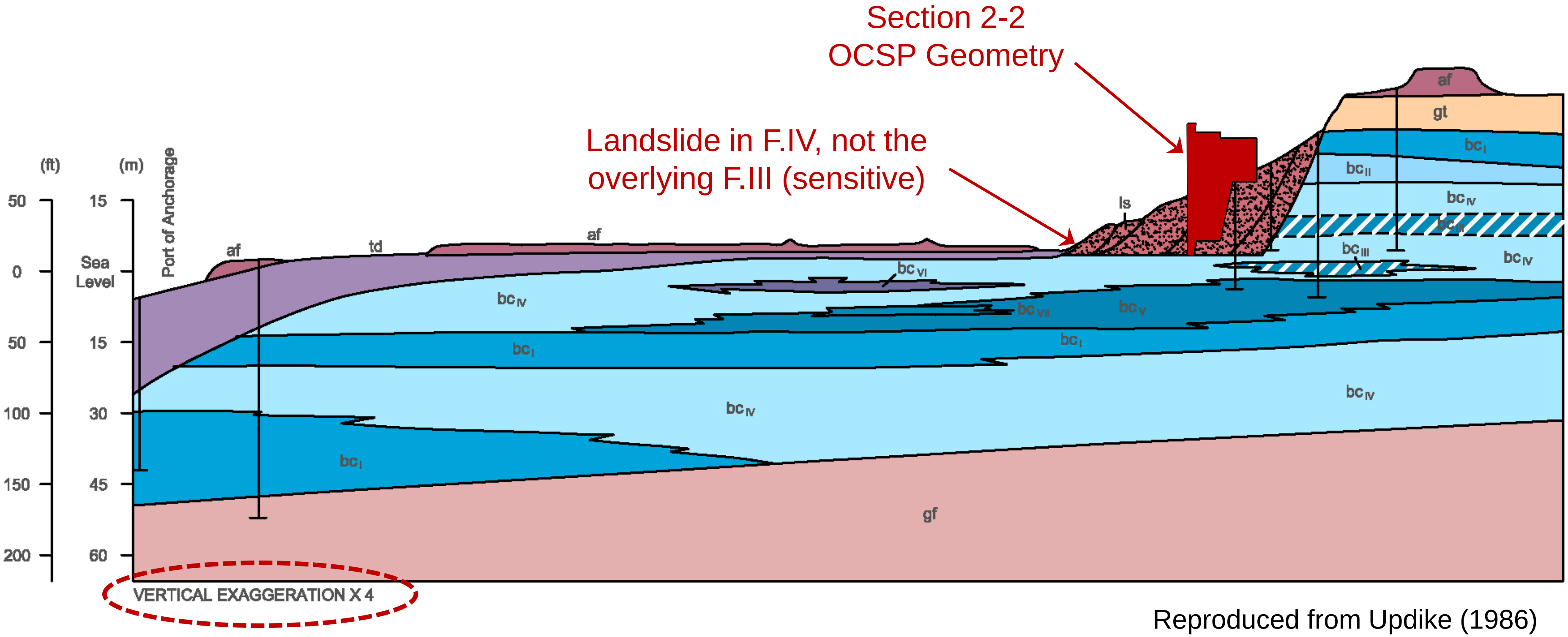
(DSS)

(CVRS)

Example Constant Volume Ring Shear Test Results on Different Clays

Undrained Residual Strength (cont.)

Could such a dramatic failure happen at Port of Anchorage?



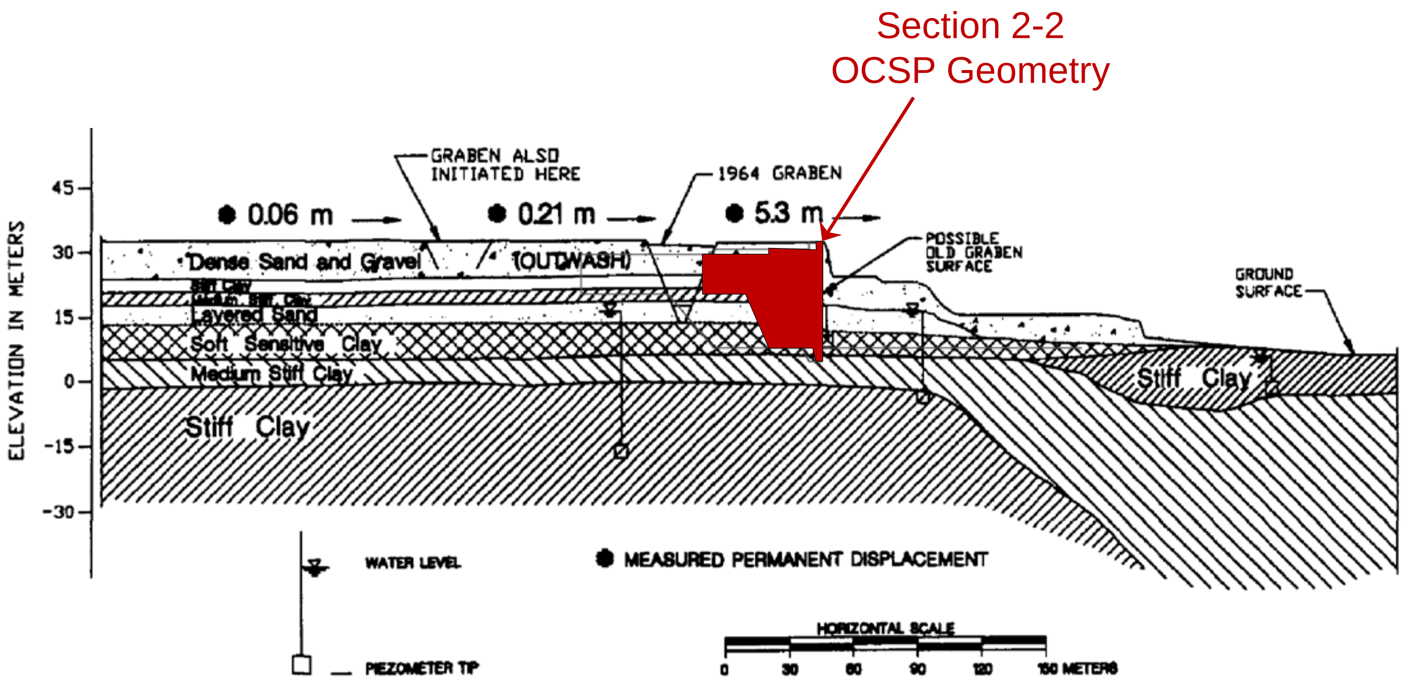
Reproduced from Updike (1986)

Yes, given similar soil loading (generation of unbalanced load) and seismic hazard:

PGA_{OLE} = 0.16 g < PGA_{1964} = 0.15-0.2 g < PGA_{CLE} = 0.29 g < PGA_{MCE} = 0.39 g

Undrained Residual Strength (cont.)

Could such a dramatic failure happen at Port of Anchorage?



Fourth Avenue failure profile from Stark and Contreras (1998)

Yes, given similar soil loading (generation of unbalanced load) and seismic hazard:

PGA_{OLE} = 0.16 g < PGA_{1964} = 0.15-0.2 g < PGA_{CLE} = 0.29 g < PGA_{MCE} = 0.39 g

Undrained Residual Strength (cont.)

Could such a dramatic failure happen at Port of Anchorage?

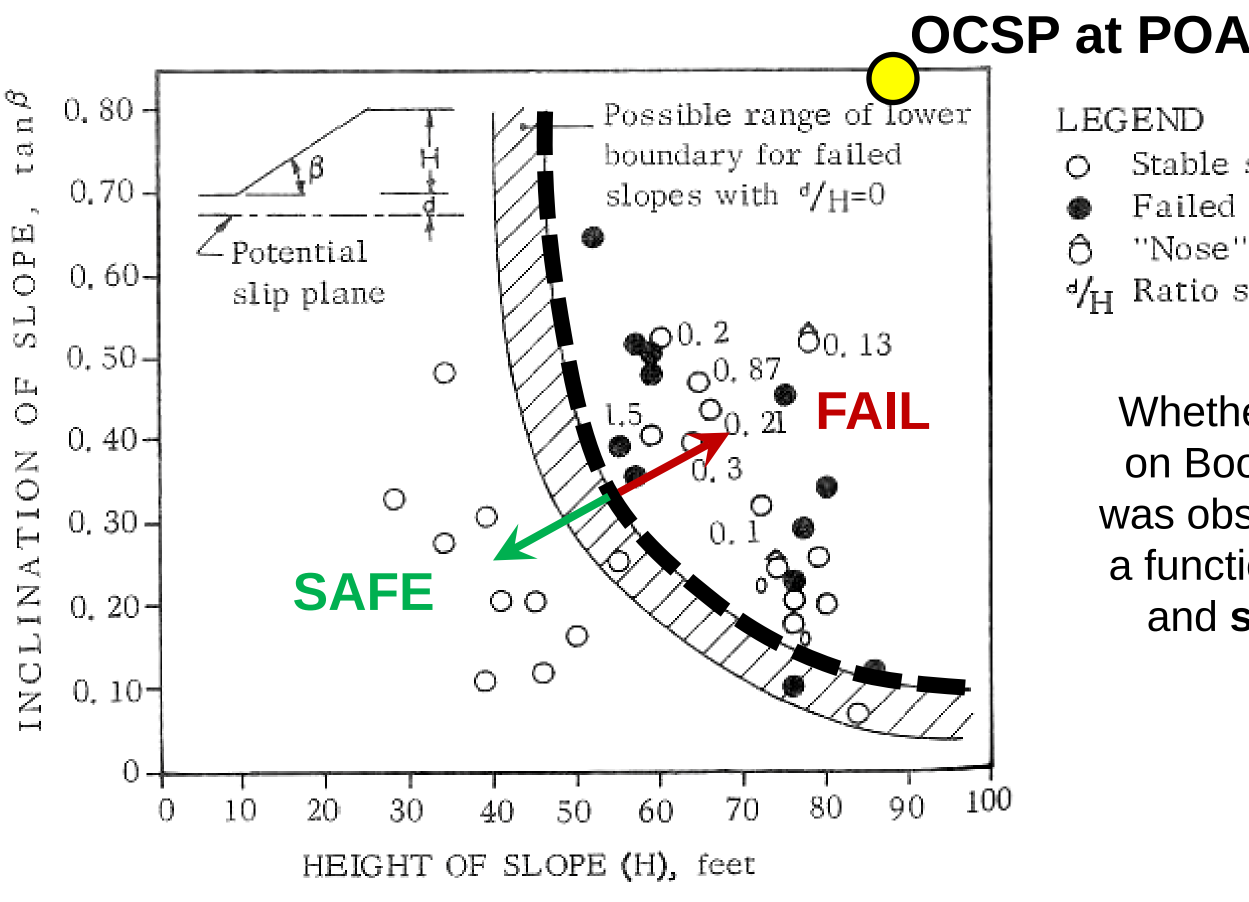


Chart from Wilson (1967)

Whether a slope founded on Bootlegger Cove clay was observed to move was a function of **slope height** and **slope inclination**

Shear Strength Comparisons

Terracon (2004), PND (2008)	CH2M HILL (2012)	
$\phi' = 27\,\text{deg}$	$\phi' = 30\,\text{deg}$	(⬆ 13%)
$\left(\dfrac{s_u}{\sigma'_v}\right)_{DSS} = 0.23\,OCR^{0.70}$	$\left(\dfrac{s_u}{\sigma'_v}\right)_{DSS} = 0.25\,OCR^{0.79}$	(⬆ 9%)
$\left(\dfrac{s_u}{\sigma'_v}\right)_{TXC} = 0.33\,OCR^{0.75}$	$\left(\dfrac{s_u}{\sigma'_v}\right)_{TXC} = 0.34\,OCR^{0.79}$	(⬆ 3%)

Major Difference:
Newmark analyses for estimating seismic deformation of OPEN CELL system at Port of Anchorage are considering the **post-peak strength loss (about 70%⬇)** due to excess pore-water pressures generated under large displacements (also **difference in limit-equilibrium critical failure surface shape/selection**)

Consequence:
Inadequate safety factor and significantly higher displacements estimated for CLE and MCE design events

Conclusions

Bootlegger Cove Clay Characterization

- **Stress history** interpretation similar to that assumed for design of North Expansion projects
- **Peak strengths** (effective-stress friction angle and undrained strength ratios) slightly higher than assumed for design
- **Cyclic degradation** due to cyclic load application at higher CSR values results in reduced shear modulus **and** post-cyclic undrained shear strength
- Very large shear displacements under constant volume conditions result in significant **post-peak undrained strength reductions**

Implications for Static and Seismic Stability

- Inadequate static-case FS and excessive seismic deformations still observed
- Shear behavior of Bootlegger Cove clay at Port of Anchorage similar to behavior of the material at Port MacKenzie and Fourth Avenue
- Analyses indicate instability due to BCF loading and seismic shaking (both similar to conditions at Fourth Avenue)

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Attachment F

Support from Dr. Paul Mayne

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02 November 2012

Mark Thompson
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Email: mark.thompson@ch2m.com

Re: Geotechnical Review: Laboratory Testing of Bootlegger Cove Formation at Port of Anchorage CH2M-Hill Project Number 427856.01.06; GTRC Project 2006T32

Dear Mark,

This final report relates to the recent series of laboratory tests conducted for the Port of Anchorage Expansion project. This is an updated version since our draft report that was issued on 30 August 2012. Since then, we have received most of the cyclic simple shear strength results and constant volume ring shear data for review.

LABORATORY TEST DATA

This final report documents the summary of our review concerning the laboratory testing and interpretation on the Bootlegger Cove Formation (BCF) clay for the Port of Anchorage (PoA) intermodal expansion suitability study. As outlined in the original Statement of Work, a new set of undisturbed tube samples of the clay were procured at the site using field rotary drilling operations. Selected samples were transported to MEG laboratories for testing which included: index, consolidation, monotonic triaxial compression tests, bender element measurements of shear wave velocity on specimens, and series of both static and cyclic direct simple shear testing (DSS). Also, selected samples were sent to the University of Illinois - Urbana - Champaign (UIUC) for performing constant volume (undrained) ring shear tests, as well as limited set of drained ring shear tests. A majority of the monotonic triaxial test series, related index tests, and CRS consolidation data were available in June and July of 2012. In September 2012, the majority of results from the cyclic simple shear test series by MEG labs were sent for review. We also received most of the data from the constant volume ring shear tests performed by UIUC in a 30-page report (dated 22 Oct 2012) .

Several documents pertaining to the analysis of lab data by CH2M were provided to us. These involved several spreadsheets with both raw and processed data, as well as two sets of Powerpoint slides: one entitled "Static and Dynamic Shear Strength of Bootlegger Cove Clay at Port of Anchorage" prepared by your office (dated 30 August 2012) and a second set of 19 slides (29 Oct 2012). Prior data collected in the preliminary 2004 study by Terracon and Gregg In-Situ were also considered, as well as later data obtained by GeoEngineers, PND, and DOWL (2008). Moreover, a recent dissertation (Zapata-Medina, August 2012) at Northwestern University also provided supplementary new data on BCF clay for consideration, including sets of triaxial compression and extension tests, CRS consolidation data, and cyclic strength testing.

Other documents supplied for consideration included portions of a Site Response Analysis Report prepared by Dr. Y. Hashash of UIUC (dated 14 July 2010); and an excerpt 10-slide PPT set (dated 02 Oct 2012) from CH2M regarding the calculated factors-of-safety (FS) concerning stability of the current open-cell sheet pile wall design that was under construction at the PoA. The Site Response Report gives expected cyclic stress ratios (CSR) at 0.12, 0.19, and 0.26 for operating level earthquake (OCE), credible level earthquake (CLE), and maximum credible earthquake (MCE), respectively, within the BCF clay layer.

SCOPE OF WORK

Our scope of services on this project involved four tasks: (1) review of the laboratory testing program; (2) review of laboratory results; (3) communication; and (4) summary review deliverables, including this report. Most of these tasks were accomplished using email memoranda, electronic attachments, and teleconferences. These are briefly discussed in subsequent paragraphs.

Task 1. For the laboratory testing program, GT reviewed and concurred with the outlined agenda that included:

- Collect intact samples of Bootlegger Cove clay
- Laboratory testing:
  - One-dimensional consolidation
  - Monotonic triaxial CAUC and DSS testing
  - Cyclic and post-cyclic DSS testing
  - Drained and constant volume ring shear testing

General discussions on these topics were made during emails and phone conversations in April and May of 2012.

Task 2. Review of laboratory test data analysis

Sampling: In earlier series of undisturbed sampling at PoA, common Shelby tubes and/or piston type samplers were used. Herein, CH2M-Hill utilized larger GUS samplers in an attempt to procure higher quality samples and mitigate effects of sample disturbance.

Consolidation: Previous series of consolidation tests involved incremental-load (IL) type with stepped loading. Herein, consolidation series of constant-rate-of-strain (CRS) with continuous loading were attempted in hopes of securing more pronounced and definitive evaluations of the effective yield stress (σ_p'), more commonly termed the preconsolidation stress. In fact, sample disturbance remained an issue during our discussions and CRS type testing appeared to exhibit some new difficulties in producing "expected" curves, thus directing CH2M-Hill to revamp the program slightly and substitute the some of the consolidation series with IL type in lieu of the CRS versions.

Shear Tests: For the triaxial compression (TXC) and direct simple shear (DSS) strength testing, the use of both SHANSEP procedures (Ladd 1991) and recompression techniques (e.g. Ladd & DeGroot 2003) were applied to series of specimens. While the reviewer originally recommended the latter procedure, it turns out that perhaps the unfortunate issues of sample disturbance prevailed at PoA and in hindsight, the SHANSEP approach proved to be a more reliable procedure in producing undrained shear strength data on the BCF clays. The evaluations shown in Mark Thompson's PPT indicate relationships between the normalized undrained strength ratios (s_u/σ_{vo}') and overconsolidation ratio (OCR) for both TXC (CAUC) and DSS series that are compatible with the earlier Terracon study (2004) and the recent NWU study (2012). The use of Modified Cam Clay (MCC) in organizing these lab data also seems validated. The NWU data also included series of triaxial extension tests (TXE) or CAUE mode.

The effective stress strength envelope was assessed in Mark Thompson's PPT with the following parameters: c' = 0 and $\phi' = 30°$. The earlier study by Terracon (2004) gave a more conservative set of values (c' = 0 and $\phi' = 27°$) while the NWU study suggested slightly higher values of around $\phi' = 28°$. Also, a much earlier study by Lade et al. (1985) indicated: c' = 0 and $\phi' = 28°$. It is noted that the interpretation of effective strength failure envelopes of soils can be made on the basis of several criteria, including: (a) maximum deviator stress; (b) maximum obliquity, i.e. (σ_1'/σ_3') ratio; and (c) values taken at 15% strain. The corresponding values of the effective cohesion intercept (c') and effective friction angle (ϕ') may be adjusted accordingly. Notably, criterion (b) often gives slightly larger values of ϕ' compared with criterion (a).

Cyclic Tests: The series of cyclic tests appear to agree with prior data collected on the BCF clays, as well as results from other clays tested under cyclic DSS (e.g., Azzouz et al. 1989; Pestana et al. 2000, Boulanger & Idriss, 2007). As the portion of the clay at the base behind the wall is heavily loaded due to large thickness of fill, the natural preconsolidation is forced towards a nearly normally-consolidated state, close to OCR = 1. The static undrained strength ratio for the DSS mode at this condition is $S = (s_u/\sigma_{vo}')_{NC} = 0.25$. At high levels of cyclic loading, the applied $CSR = (\tau_{cyclic}/\sigma_{vo}') = 0.19$ at the CRE is at 76% of static strength, thus significant positive porewater pressures can be generated and the accumulation of cyclic shear strains can occur at these magnitudes of loadings. At the higher MCE level, the CSR could force the clay through its peak strength, thus resulting in strain softening behavior. The level of sustained strength following peak would depend upon the numbers of cycles of loading (i.e. duration of the earthquake).

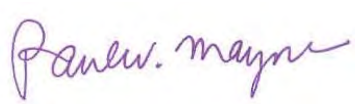
Ring Shear Tests: The use of constant volume ring shear tests is a specialized series of tests that were used to investigate post peak softening of the BCF clay. In these data, the values of peak undrained strengths normalized to effective overburden stress level appear comparable to those determined by static DSS test series. The subsequent large strain shearing after peak resulted in lower strengths, with magnitudes dependent upon the amount of displacement.

Task 3: Correspondence

Information between CH2M-Hill and GT was routinely provided using email memoranda with attachments provided in Excel, Powerpoint, Word, JPEG, and/or PDF formats, depending upon the specific documents. A number of phone conversations and/or teleconferences were held to discuss various matters at critical times during the project schedule. Of recent, a teleconference call was held with Don Anderson and myself on 12 Oct 2012 and another with both Drs. Anderson and Thompson of CH2M-Hill and myself on 31 Oct. 2012 to discuss the final aspects of this project.

Task 4: Deliverables. Several email memos were issued during planning and analysis stage of the project. In particular, this final report serves as a summary of information, interpretation, and conclusions made on the PoA project, based on the summary test data and synthesis of results.

Sincerely



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