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.

[1]Assoc. Prof. of Civ. Engrg., Newmark Civ. Engrg. Lab. MC-250, 205 N. Mathews Ave., Univ. of Illinois, Urbana, IL 61801-2352.

[2]Proj. Engr., GEOHIDRA Consultores, C.A., Caracas, Venezuela.

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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.

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

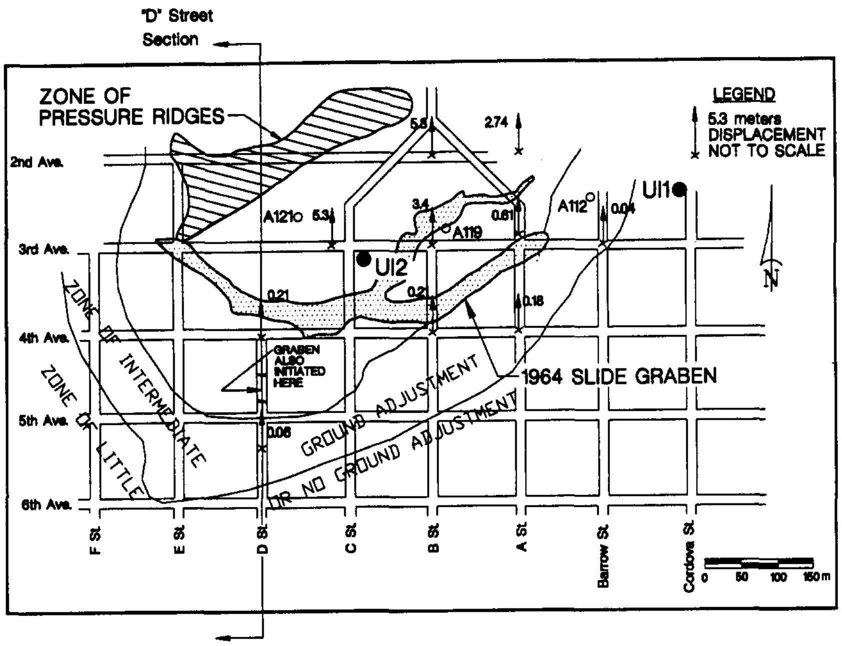

FIG. 2. Ground Breakage and Cross Section in Fourth Avenue Slide Area (After Shannon and Wilson 1964)

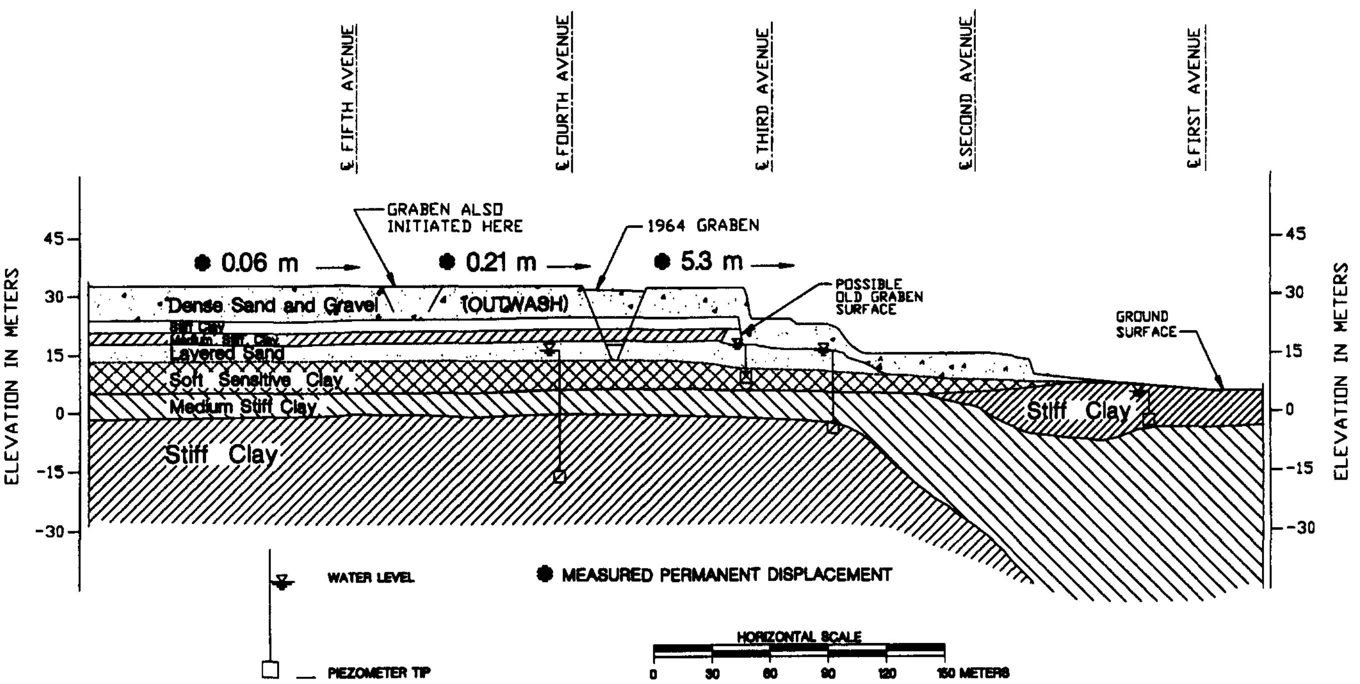

FIG. 3. Cross-section along D Street in Fourth Avenue Landslide (Shannon and Wilson 1964)

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

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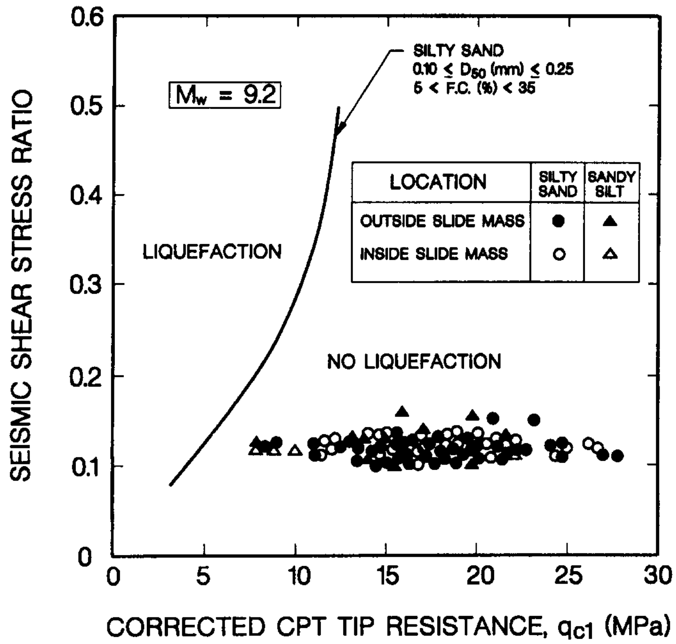

FIG. 4. Liquefaction Potential in Sandy Strata in Interbedded Zone during 1964 Alaska Earthquake

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 mod-

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

ified 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.

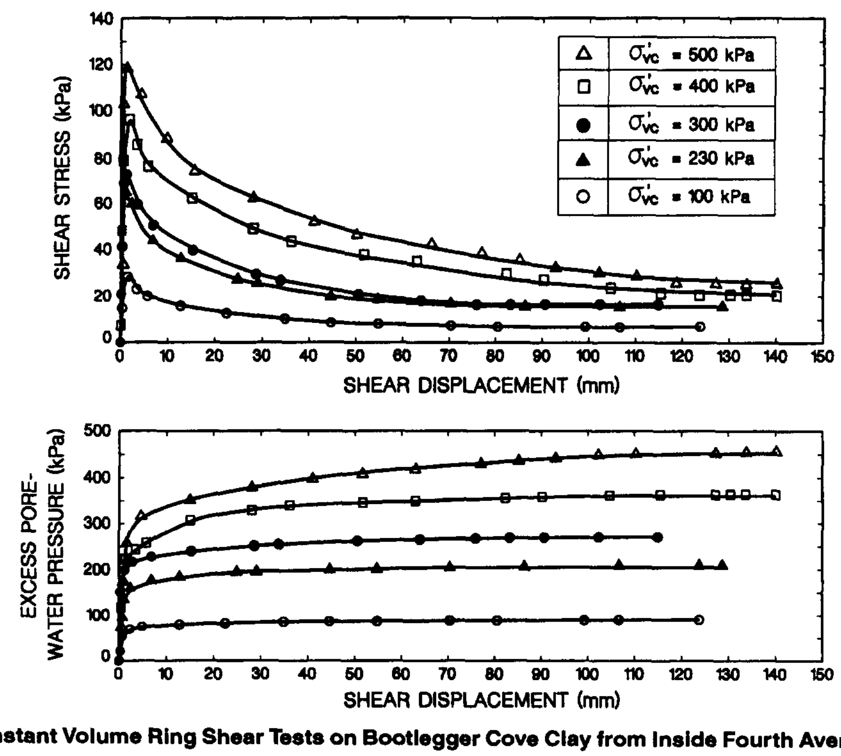

FIG. 5. Constant Volume Ring Shear Tests on Bootlegger Cove Clay from Inside Fourth Avenue Landslide

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

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_{DI}; (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	59	100	280–320	0.28	1.2	0.07	55
inside Fourth Ave.	34	19	57	230		0.28	1.1	0.07	75
landslide, Anchorage,	36	21	56	300		0.24	1.3	0.06	75
Alaska	38	21	55	400		0.23	1.8	0.06	120
	39	20	62	500		0.23	1.8	0.06	130
Drammen clay,	47	23	70	95	140	0.27	1.1	0.09	19
Danvik-gate,	48	24	72	255		0.22	1.3	0.11	16
Drammen, Norway	47	25	65	400		0.20	1.1	0.11	60
Bootlegger Cove clay,	42	23	47	150	405	0.31	1.5	0.11	95
outside Fourth Ave.	40	21	42	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	0.11	140
Cohesive alluvium,	30	22	19	95.8	122.4	0.19	2.2	0.10	52
Enid Dam,	28	22	20	147	138.9	0.27	1.1	0.05	77
Enid, Mississippi	23	19	17	191	81.4	0.24	1.1	0.07	70
	25	22	20	287	143.5	0.23	1.2	0.07	72
	30	22	20	383	134.5	0.23	1.2	0.06	75
Cohesive alluvium,	59	31	51	51.8	75.8	0.21	0.50	0.13	36
Jackson, Alabama				79.4		0.23	0.35	0.16	50
				100		0.23	0.37	0.14	38
Upper Bonneville clay,	46	23	33	47.9	47.9	0.32	0.30	0.11	39
Salt Lake City, Utah				95.8	95.8	0.36	0.60	0.15	25
				191.5	191.5	0.31	1.2	0.12	29
				383	383	0.34	2.0	0.14	36

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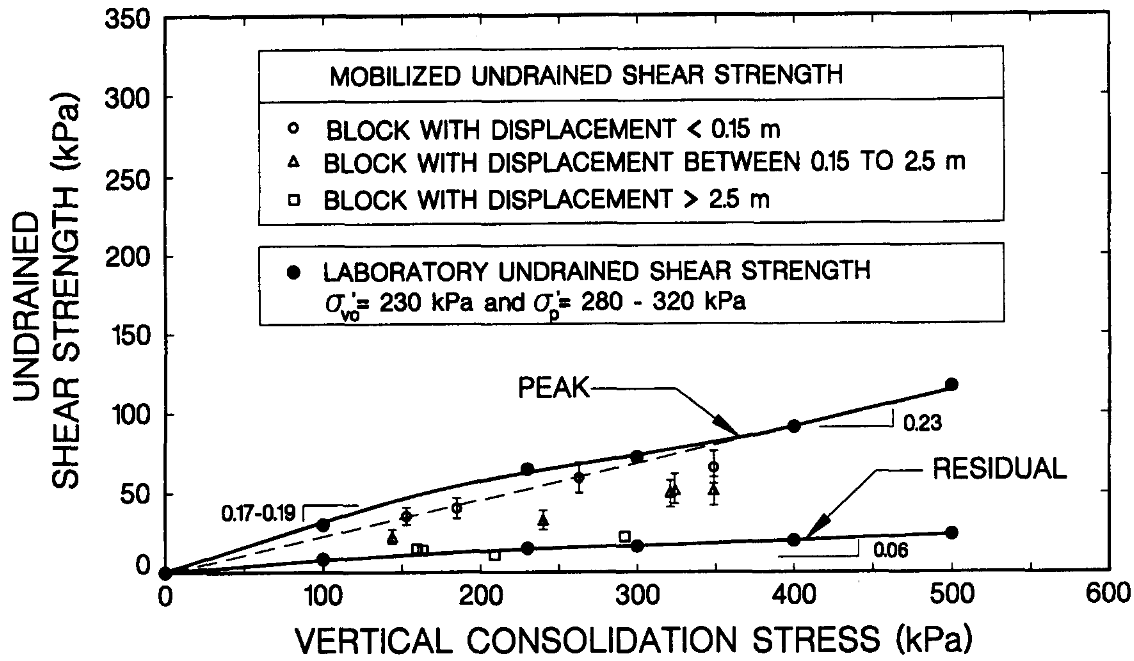

FIG. 6. Comparison between Laboratory and Back-Calculated Undrained Shear Strength for Bootlegger Cove Clay

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.

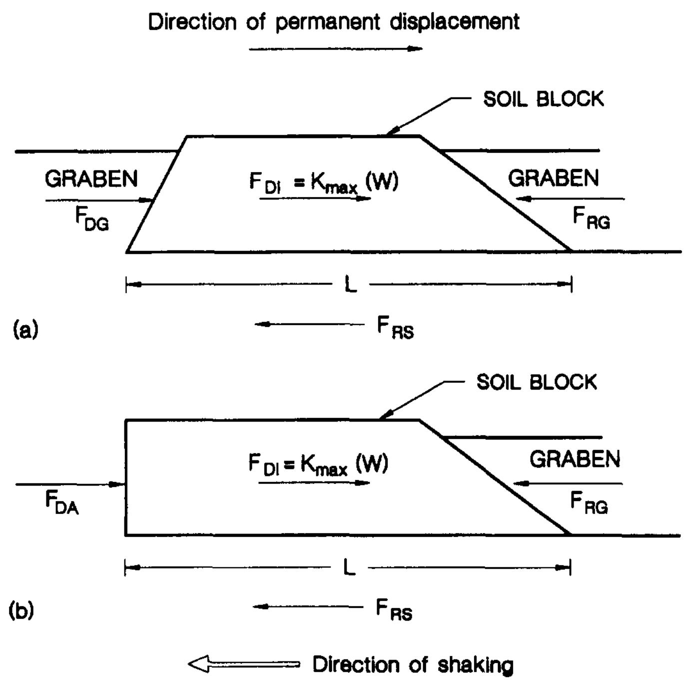

FIG. 7. Forces Used in Analysis for Calculating Permanent Lateral Displacement Due to Earthquake Ground Motions

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.

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:

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

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

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

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

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	0.06–0.08
	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

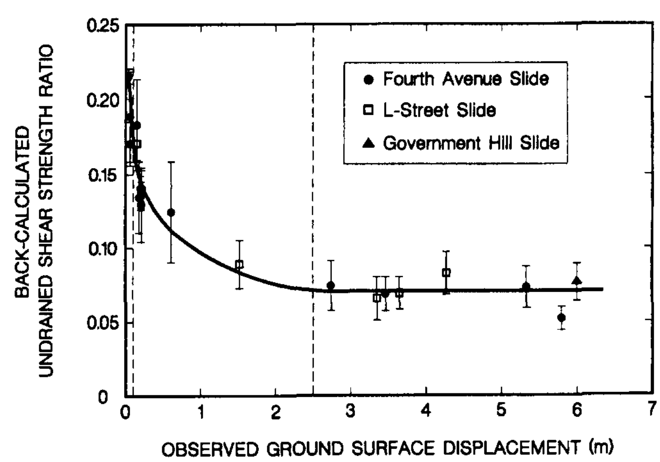

FIG. 8. Variation of Undrained Shear Strength Ratio with Ground Surface Displacement

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 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.

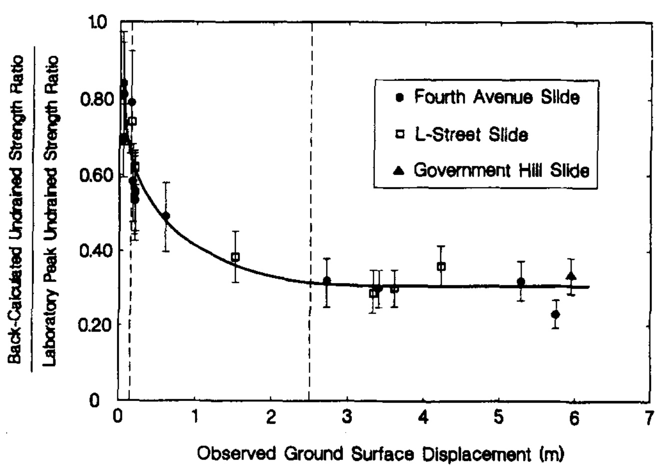

FIG. 9. Variation of Normalized Undrained Shear Strength Ratio with Surface Ground Displacement

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

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

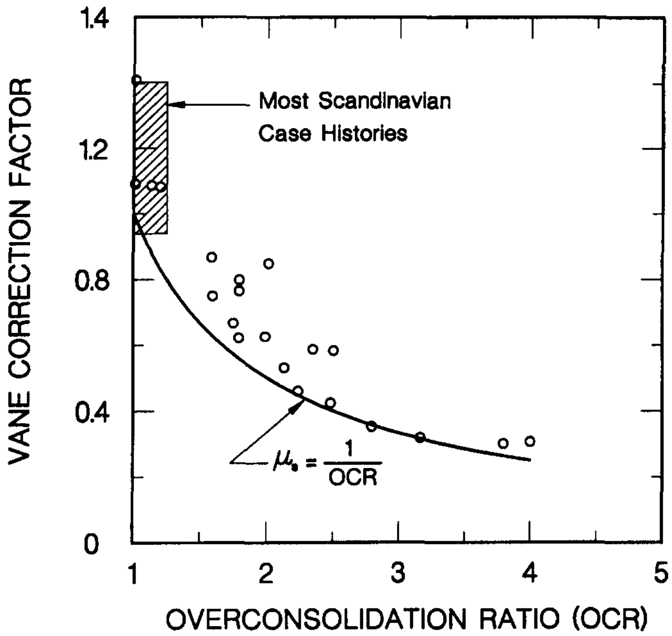

FIG. 10. Vane Shear Correction Factor for Seismic Stability Evaluations of Cohesive Soil Slopes (Terzaghi et al. 1996)

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).

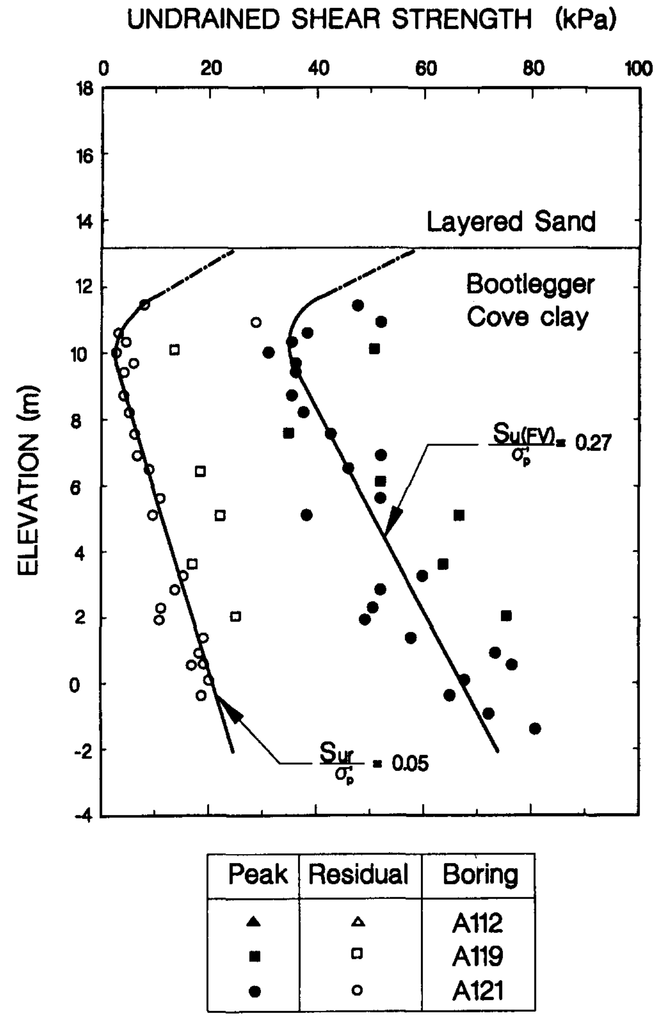

FIG. 11. Field Vane Shear Test Results from Fourth Avenue Slide Mass (data from Shannon and Wilson 1964)

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 pro-

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

posed 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.

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