Correlations Between Soil Plasticity and Strength Parameters

USDA-NRCS Soils Maps

In the United States pedological information is contained in soil survey reports published by the U.S. Department of Agriculture (USDA). Around 1960, the SCS began publishing reports contain sections on engineering properties of the mapped soils with map scales of 1 inch to 2,000 feet (1:24,000), on aerial photograph mosaics (in black and white or gray-green). These "second generation" of reports also contain tabulations of test data and engineering classifications according to the American Association of State Highway Officials (AASHTO) and Unified Soil Classification System (USC) used by most consultants. The SCS has published and updated these reports from 1960 through the present. In 1971 SCS issued their *Guide for Interpreting Engineering Uses of Soils* (USDA, Soil Conservation Service, Washington, D.C., 86 p.), which lays forth the rationale by which engineering classifications of soil are tabulated in the individual County reports published by SCS.

In 1995 Congress reorganized the Department of Agriculture. The SCS is now called the Natural Resources Conservation Service (NRCS), and within each County the NRCS office is now co-located with a Consolidated Farm Service Agency (formerly the Agricultural Stabilization and Conservation Service) office. Many of the county soil survey reports contain soil plasticity information gleaned from nearby highway projects. These documents are given free to the public at any of the local NRCS offices. A copy of the NRCS soil survey report should always be checked to see if any useful engineering data, such as soil plasticity indices, are summarized and/or presented, to get a feel for the expected range in reported values. Soil surveys can provide a great deal of valuable geotechnical information in rural and agricultural areas.

Atterberg Limits

The Atterberg Limits (ASTM Test D-4318) define the ranges in moisture content that a soil will behave as a solid, plastic and liquid. The Liquid Limit (LL) of a soil is defined as the moisture content above which the soil behaves as a liquid, and the Plastic Limit (PL) is the moisture content above which the soil behaves plastically. The numerical difference between the Liquid Limit and Plastic Limit is termed the Plasticity Index (PI).

Example of How PI Correlates with Soil Strength

In the example case contained herein, we begin by summarizing published laboratory test results that were included in descriptions of the native soils on a hillside site in Santa Barbara, California. According to the Soil Survey (G. E. Shipman, 1981, *Soil Survey of Santa Barbara County, California, South Coastal Part*: U.S. Department of Agriculture, Soil Conservation Service and Forest Service, in Cooperation with University of California Agriculture Experiment Station, 144 p.), the native soils mantling the subject parcel are assigned to the Ayar Series, and have developed from the weathering of predominately soft calcareous shale and mudstone bedrock. These soils are generally described as dark grayish brown to very dark grayish brown clay, with a high shrink-swell potential. Published laboratory test results indicate that the Liquid Limit (LL) of such soils is 50-70, and the Plasticity Index (PI) ranges from 25-45

Atterberg Limits tests were performed in our laboratory to help classify and evaluate the expansion potential and drained shear strength (Ø) of the residual clayey soils mantling the slopes surrounding the site and estimate the residual shear strength ($Ø_r$) of the landslide slip surfaces(s). The residual shear strength is defined by the post-peak strength behavior of strain-softening materials, as sketched in Fig. 1.

Residual shear strength is generally assumed to occur when brittle overconsolidated materials, such as shale bedrock, undergoes excessive shear, such as would be expected during landslippage along the basal rupture surface (A.W. Skempton, 1970, First-Time Slides in Over-Consolidated Clays: *Geotechnique*, v. 20:3, pp320-24; and D.H. Trollope, 1973, Sequential Failure in Strain-Softening Soils: *Proc. 8th Int'l Conf. Soil Mechanics and Fdn Eng'g*, Moscow, v. 2, pt. 2, pp.227-32). Naturally-occurring soils tend to exhibit strain-hardening while lithified materials, such as weak rock, exhibit strain softening, as compared in Fig. 1. Landslide slip surfaces are normally associated with residual friction values ($Ø_r$).

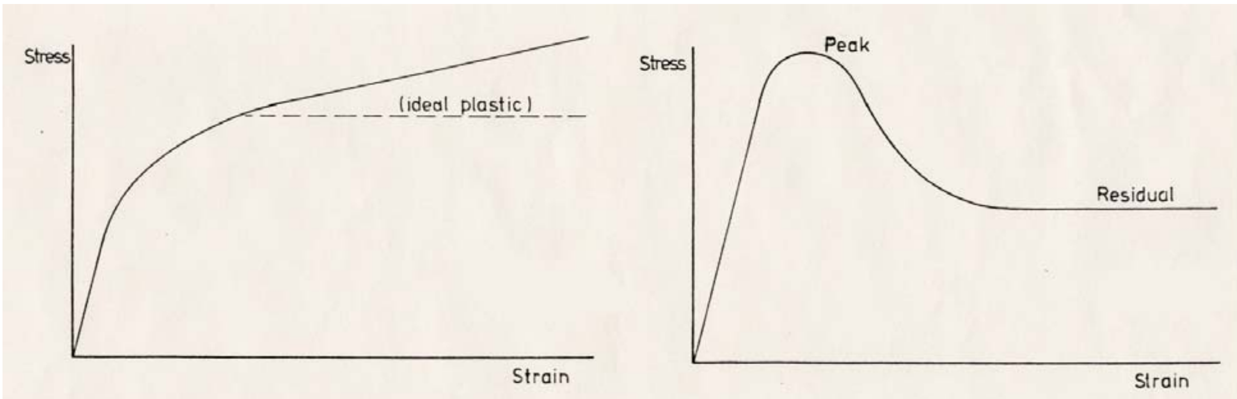
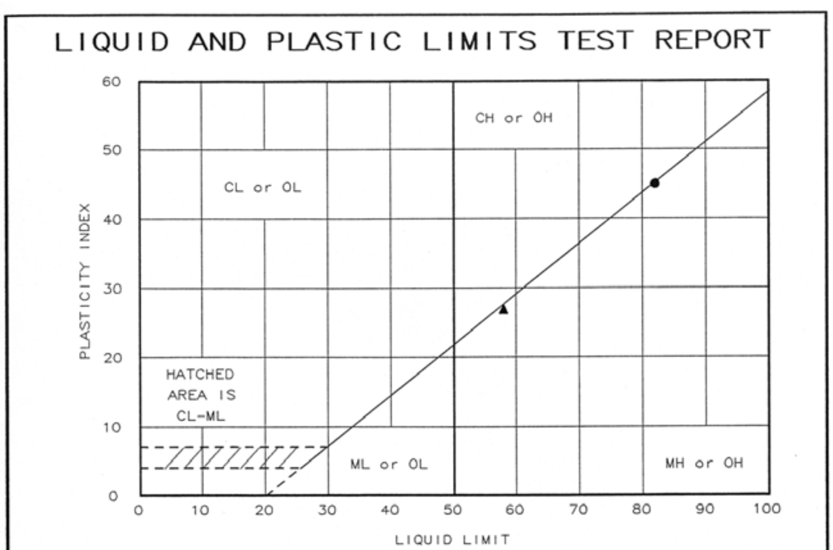


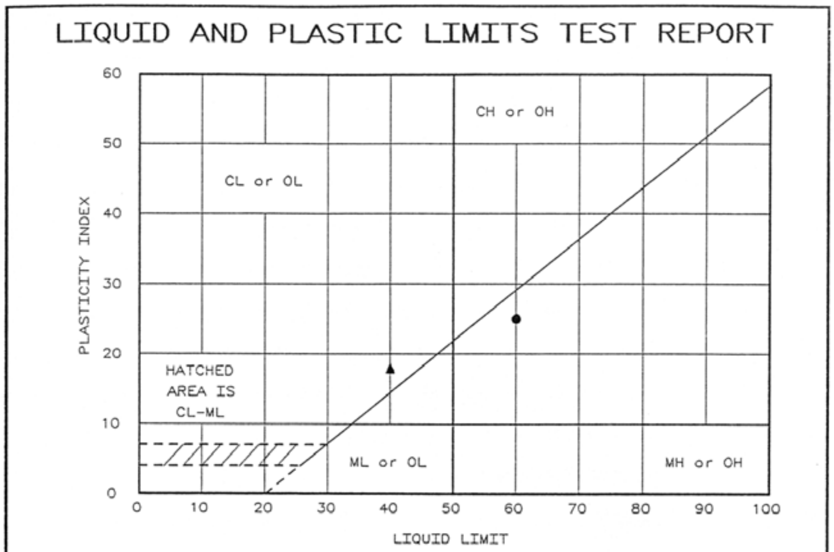
Fig. 1 presents the stress-strain behavior of two varying geologic materials involved in landslippage. The left diagram presents stress-strain behavior of a quasi-plastic, or *strain-hardening material*, such as normally consolidated clay. The right-hand plot shows the behavior exhibited by brittle materials, such as overconsolidated clay or bedrock, which tend to exhibit *strain-softening behavior*.

A number of representative samples were evaluated for soil plasticity on both this and the adjacent properties, presented herein as Figs. 2 and 3. These plots present the results of laboratory tests, determining representative values of the Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI = LL-PL).



	Location + Description	LL	PL	PI	-200	ASTM D 2487-90
●	B3, 5.0-5.5 FT. Yellowish brown soil.	82	37	45		
▲	B7, 5.0-5.5 FT. Yellowish brown soil.	58	31	27		

Figure 2 – *Atterberg Limits test data for soil samples recovered from exploratory borings on the subject parcel, a few hundred feet uphill of the Old Earth and Debris Flow Fan. Sample B3 came from a deep-seated slump-earthflow landslide.*



Location + Description	LL	PL	PI	-200	ASTM D 2487-90
● TP-4, upper 2 FT.	60	35	25		
▲ TP-7, upper 2 FT.	40	22	18		

Figure 3 – *Atterberg Limits test data for soil samples recovered from exploratory trenches on the subject parcel, a few hundred feet uphill of the Old Earth and Debris Flow Fan. These samples were representative of residual soils developed on the Rincon Shale.*

Soils taken from the upper two feet of the Monterey Shale exhibited PIs that averaged about 18 (medium expansion potential). Soils developed in the upper two feet near the tuffaceous facies of the Monterey shale exhibited Plasticity Indexes (PI's) averaging 25 (highly expansive). Near-surface samples of soils developed on the colluvium mantling the hillslopes showed PIs of around 27 (highly expansive). Samples of landslide debris taken from the headscarp areas of several of the mapped earthflows exhibited PIs that ranged as high as 45 (very high expansion potential). The range of these values seems appropriate to the observed conditions, which range from reasonably stable slopes to large deep-seated landslides.

Soil Residual Strength Parameters

The PI of a soil is commonly correlated with the expansive potential and residual angle of internal friction for drained field conditions, as proposed by R.E. Gibson, 1953, Experimental determination of true cohesion and true angle of internal friction in clays: *Proceedings of the 3rd International Conference on Soil Mechanics and Foundation Engineering* (Zurich), v. 1:126; T. C. Kenney, 1959, Discussion: *Journal of the Soil Mechanics and Foundation Engineering Division*, ASCE, v.85:SM3, p. 67-79; T.C. Kenney, 1967, The influence of mineral composition on the residual strength of natural soils: *Proc. Oslo Conf. on Shear Strength Properties of Natural Soils and Rocks*, v. 1, pp. 123-29; D.U. Deere and F.D. Patton, 1971, Slope Stability Analysis in Residual Soils: *Proc. of the 4th Panamerican Conf. on Soil Mech. and Fdn. Eng'g*, San Juan, PR, v. 1, pp. 144-45; B. Voight, 1973, Correlation between Atterberg plasticity limits and residual strength of natural soils: *Geotechnique*, v. 23, p. 265-267; P.K. De and B. Furdas, 1973, Discussion of correlation between Atterberg plasticity limits and residual shear strength of soils: *Geotechnique*, v. 23, pp. 600-601; J. K. Mitchell, 1976, *Fundamentals of Soil Behavior*: John Wiley, 422 p.; R.D. Holtz and W.D. Kovacs, 1985, *An Introduction to Geotechnical Engineering*: Prentice-Hall, p. 543-45; and A. W. Skempton, 1985, Residual strength of clays in landslides, folded strata and the laboratory: *Geotechnique*, v. 35:1 p. 3-18.

The Plasticity Indexes derived from Figs 2 and 3 were plotted on published charts relating drained shear strength (Ø') and residual shear strength ($Ø_r$), which are presented in Figs. 4 thru 7. Residual shear strength is generally assumed to occur when brittle overconsolidated materials, such as shale, undergoes excessive shear, as would be expected during landslippage (Skempton, 1970, op cit; Trollope, 1973, op cit). Naturally-occurring soils tend to exhibit strain-hardening while lithified materials, such as weak rock, exhibit strain softening, normally associated with residual friction values ($Ø_r$).

The aggregate plot of Kenny (1967, op cit) is presented in Fig. 4. This plot suggests that the natural soils on this site could be expected to exhibit average Ø values for drained shear conditions between 26^o and 29.5^o, as would be expected in naturally-occurring soils. The residual angle of internal friction ($Ø_r$) of a buried landslide slip surface with a PI of 45 would be expected to be much lower, something close to 16^o. This would be for the most highly expansive material tested (PI = 45).

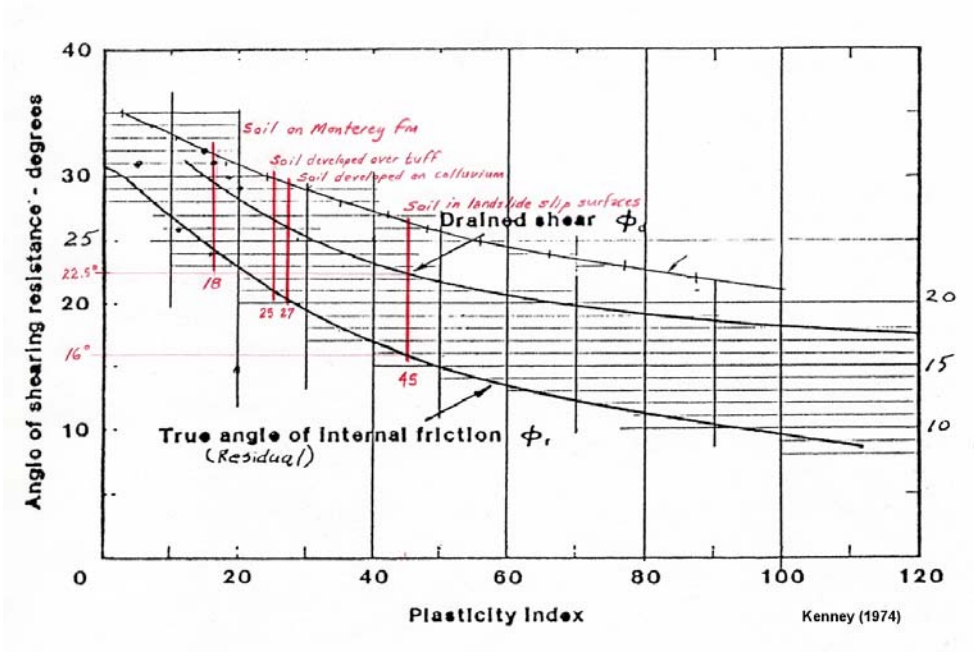


Figure 4 – *Plasticity data for residual soils and landslides developed on the Rincon Shale plotted in red on a chart comparing PI with residual angle of internal friction, $Ø_r$, taken from Kenney (1967). $Ø_r$ = 16 degrees appears representative of the largest bedrock slump-flow landslides developed in the weathered Rincon Shale.*

Fig. 5 presents similar data, from a relationship published by Holtz and Kovacs (1985, op cit). This plots suggests an average effective friction angle (Ø') of about 26^o for a PI = 45, with the lower plasticity materials exhibiting increasing friction, up to 31^o. These relationships were developed from laboratory tests initiating virgin rupture surfaces, not residual friction values ($Ø_r$), as would be expected in a dormant landslide.

Fig. 6 presents a similar relationship developed by Voight (1973, op cit). Voight's plot provides data developed from well-documented studies of large landslides developed in Montmorillonitic clay shales around the world, described by their back-analyzed residual friction values (μ_r = tan $Ø_r$). In this case, PI = 45 equates to a residual friction value of about 14.6^o , with the lower plasticity samples corresponding to residual friction values up to 23^o.

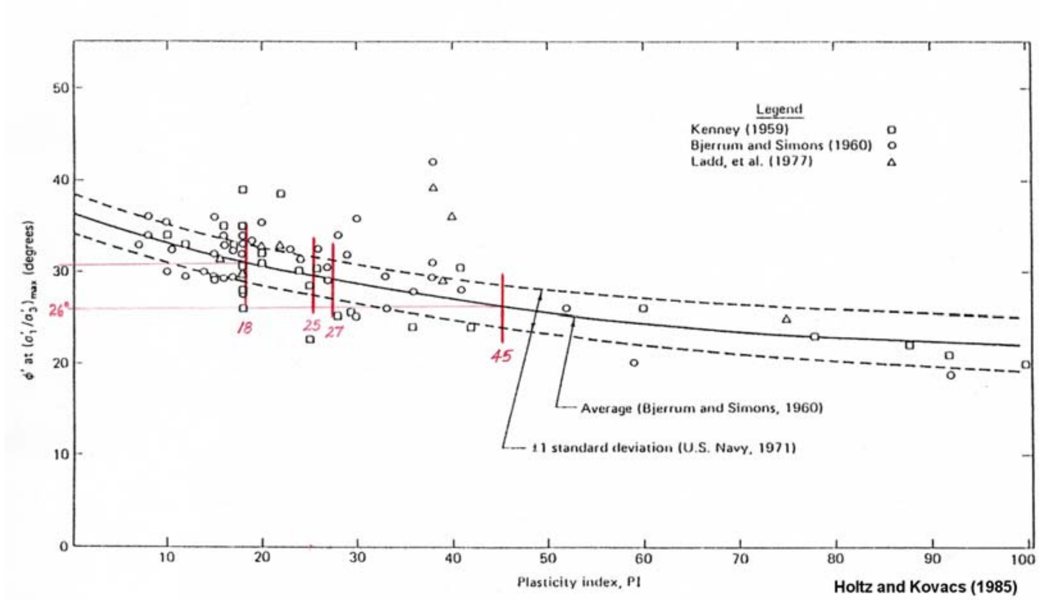


Figure 5 – *Plasticity data for residual soils and landslides developed on the Rincon Shale plotted in red on a chart comparing PI with residual angle of internal friction, $Ø_r$, taken from Holtz and Kovacs (1985). In this plot the median values range between $Ø_r$ = 26 and 31 degrees, which appear somewhat higher than might explain the observed sliding on this site.*

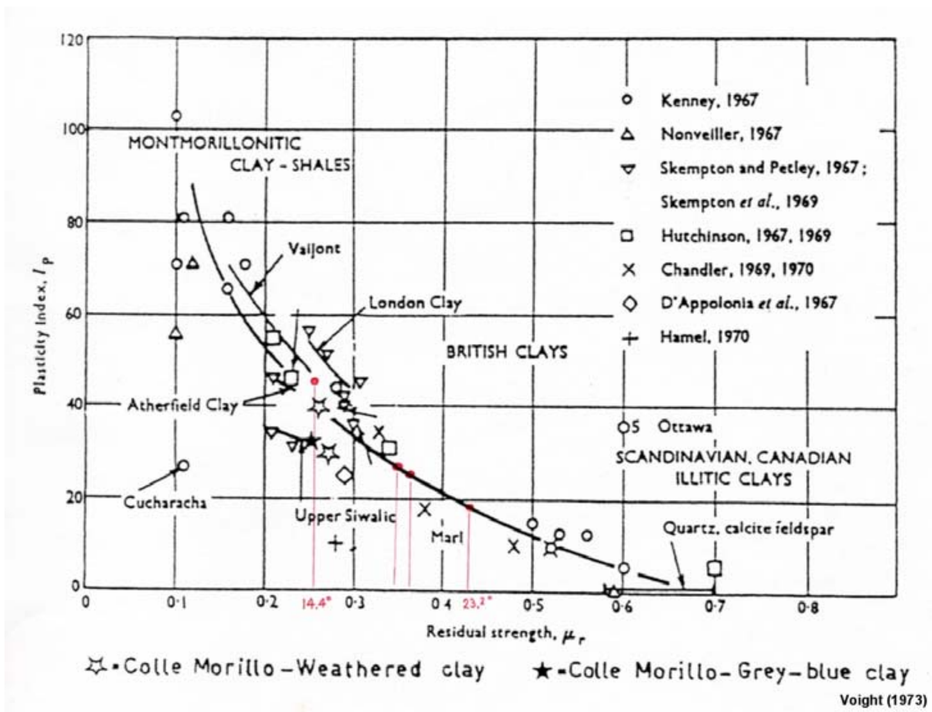


Figure 6 – *Plasticity data for residual soils and landslides developed on the Rincon Shale plotted in red on a chart comparing Plasticity Index with residual angle of internal friction, $Ø_r$, taken from Voight (1973). This plot is comprised of residual strength data from large landslides in Montmorillonitic clay shales. The median values range between $Ø_r$ = 14.4 and 23.2 degrees.*

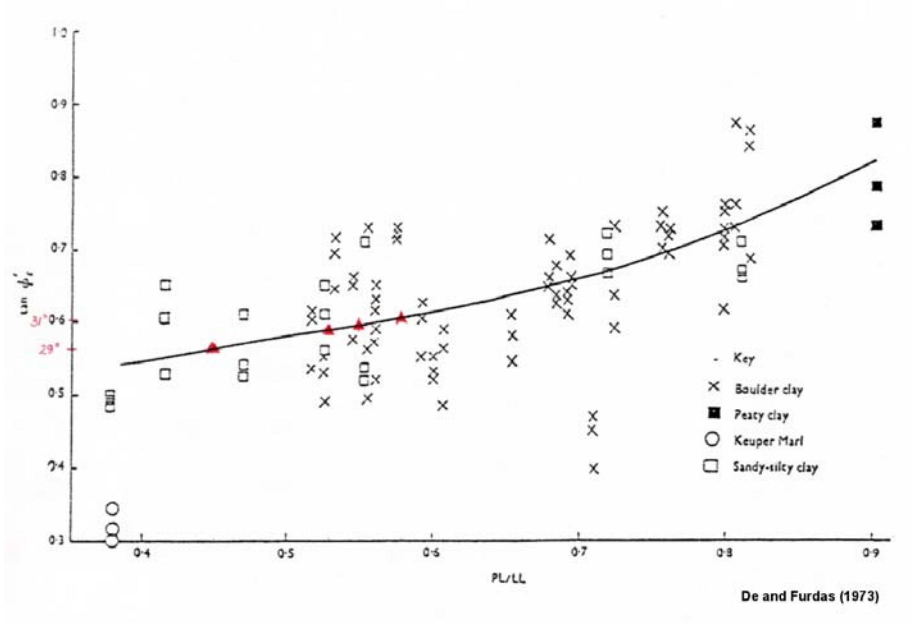


Figure 7 – *Plasticity data for residual soils and landslides developed on the Rincon Shale plotted in red on a chart comparing PlasticLimit/Liquid Limit with residual angle of internal friction, $Ø_r$, taken from De and Furdas (1973). This plot came from data on small to medium scale slumps developed in highly overconsolidated shales in South Wales, Great Britain. In this plot the data scatter is quite large, but the median values vary between $Ø_r$ = 29 and 31 degrees.*

Fig. 7 presents a slightly different plot similar relationships developed by others. De and Furdas (1973, op cit) plotted their residual friction angle data against the ratio the Plastic Limit to the Liquid Limit (PL/LL) instead of the Plasticity Index (PI). These data were derived from failures in brittle overconsolidated clay slopes in South Wales. The data spread is significant. The Las Positas plasticity data was plotted on the least mean squares line for comparison purposes, which would suggest a residual shear angle ($Ø_r$) between 29^o and 31^o, which appears much too high to explain the observed landslides.

Taken by itself, a $Ø_r$ value of 16^o would be too low to support the existing slope above Lots 1 thru 6, absent some addition strength contribution from soil cohesion. Old landslides tend to develop increasing cohesion with resident time (J.K. Mitchell, 1976, *Fundamentals of Soil Behavior*: John Wiley & Son, 422 p.). Absent actual tests on the buried landslide slip surface, we would recommend assuming that the strength of these horizons is something close to $Ø_r = 16^o$ with a cohesion of 250 pounds per square foot (psf), based on an assumed Plasticity Index of about 45.

Preliminary “Guestimates”:

Buried Landslide slip surface: $Ø_r = 16^o$ with a cohesion of 250 pounds per square foot along the slide plane. PI = 45 on slide plane γ_d = 93 pcf If disturbed , cohesion will drop by as much as 2/3, or down to 85 psf.

Remainder of shale slide debris: $Ø = 20^o$ with a cohesion of 250 pounds per square foot along the slide plane. PI = 25 to 30 and γ_d = 93 pcf

Weathered shale beneath the dormant landslide: $Ø = 26^o$ with a cohesion of 350 pounds per square foot along the slide plane. PI = 18 and γ_d = 107 pcf

Impact of Clay Mineralogy on Residual Shear Strength

The published charts relating soil plasticity to drained and residual frictional shear strength can be misleading because:

1) They are based on average values;
2) They mix frictional strength component for peak shear strength under drained conditions (Ø‘) and residual shear strength ($Ø_r$);
3) With the exception of Voight’s 1973 plot, they combine strength data for differing clay mineralogy. If the soil is very highly expansive (PI > 35), then the charts suggest median values that may be much higher than actually exists.

Olson (1974) compiled a range of shear strength data from multiple tests on specimens of relatively pure kaolinite, illite and montmorillonite (R.E. Olson, 1974, Shearing Strength of Kaolinite, Illite and Montmorillonite: *Journal of the Geotechnical Division*, ASCE, v. 100:GT11, pp. 1215-1299). He found that the kaolinite was strongest, followed by the illite, with montmorillonite coming in very low. Mitchell (1976) overlaid Olson’s data on the plot developed by Kenney (1959), shown in Fig. 8. This figure helps to explain the data scatter observed in the other plots, because the weathering of shales is complex, uneven and non-linear, leading to noticeable variations in clay percentage and mineralogy. If montmorillonite is known to exist as a common weathering product in a particular area, Voight’s plot (Fig. 6) might be the most appropriate to use. But, if montmorillonite is absent, Voight’s plot will greatly underestimated the mobilized strength.

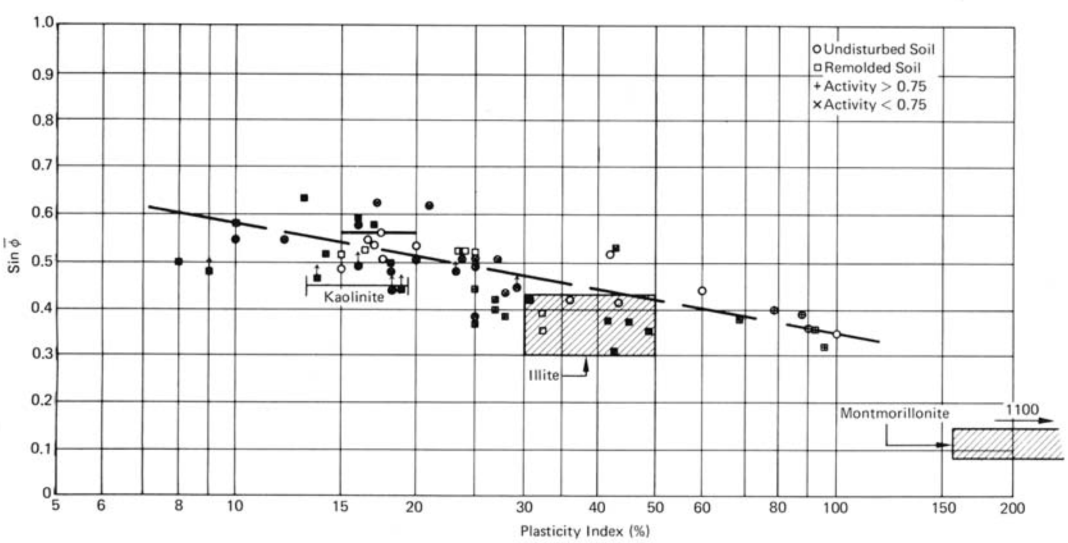


Figure 8 – *Relationship between sin Ø‘ and plasticity index for normally consolidated soils, using data from Kenney (1959) and Olson (1974). Note how montmorillonitic clay plots off the chart to right.*

Case Study

In 1998 a massive coalescing earthflow landslide complex in the hills above Fremont, CA reactivated, for the first time in the recorded history of the area (about 200 years). The slide was about a mile long and involved over 17 million cubic yards of debris. I served as a consultant to the City and directed an investigation of the failure, which included a program of conventional subsurface exploration using augured borings while recovering drive samples of the soils comprising the landslide.

10 representative samples of the slide materials were evaluated for soil plasticity indices. All of these samples exhibited moderate to high expansion potential, with a residual angles of internal friction between 17° and 20°.

Direct shear tests of remolded samples of slide materials were performed using quick undrained loading conditions in a direct shear testing device, in accordance with ASTM Test designation D-3080-72. The shear strength of the soil is commonly described by the two measurable components of shear strength: the angle of internal friction (phi) and its intrinsic cohesion (c). Both parameters can be measured by performing a series of direct shear tests on soil samples under varying levels of confinement. The friction angle was measured in degrees and defines the frictional strength characteristics mobilized by a soil under varying normal force (confinement), while applying sufficient lateral force to rupture the specimen. The cohesion of a soil is generally measured in pounds per square foot, and is, theoretically, independent of the applied load if the specimen is saturated.

The measured shear strength of the remolded slide materials (not actual slip surfaces) varied between 14 and 32 degrees, with cohesion intercepts between 0 and 320 psf. 10 of the 12 samples selected for testing had results that averaged 20.5°, a representative value of internal friction long associated with landslides in the Orinda formation, based on previous work (Duncan, J.M., and Stark, T.D., 1992, Soil Strengths from Back-Analysis of Slope Failures: in Seed and Boulanger, eds., Stability and Performance of Slopes and Embankments-II: *ASCE Geotechnical Special Publication No. 31*, v. 1, p. 890-904).

Most of the samples contained a mixture of fine grained silt and clay with entrained pieces of rock. The rock fragments occasionally created artificially high shear strength results at higher levels of strain and confinement, leading to artificially high friction (phi angle) and unrealistically low (negative) cohesion values, due to particle size influence factors (as described in N.D. Marachi, C. K. Chan and H. B. Seed, 1972, Evaluation of Properties of Rockfill Materials: *Journal of the Soil Mechanics and Foundations Division*, American Society of Civil Engineers, v.98:SM 1, p. 95-114). The width of the forced shear zone is typically around four times the mean particle size diameter for uniform (well graded or poorly sorted) gradation (K. H. Roscoe, 1953, An Apparatus for the Application of Simple Shear to Soil Samples: *Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering*, Zurich, v.1, p. 186-191; F. S. Tatsuoka, F., S. Nakamura, C.C. Huang and K. Tani, 1990, Strength Anisotropy and Shear Band Direction in Plane Strain Tests of Sand: *Soils and Foundations*, v. 30:1 (March), p. 35-54). These plots were adjusted with hand-drawn lines to better approximate the actual residual strength parameters of these remolded landslide materials.

The Plasticity Indexes were determined for each test specimen and compared with the measured shear strength, on published charts relating angle of shearing resistance versus using Plasticity Index and plots comparing residual angle of internal friction ($Ø_r$) for drained field conditions versus soil plasticity (PI). These data were plotted on charts prepared by Gibson (1953), Kenney (1959), Voight (1973), Deere (1974), Kenney (1977) and Skempton (1985). The distribution of measured Plasticity Indices test data with the published correlations are presented as Figs. 9 thru 12. The data plotted well below the means values listed on the published charts, with the exception of that prepared by Voight (1973), presented in Fig. 12. This is because Voight's chart was developed specifically with data taken from Montmorillonitic clay shales, which generally exhibit much lower shear strengths because of their high water content and plasticity (Fig. 8).

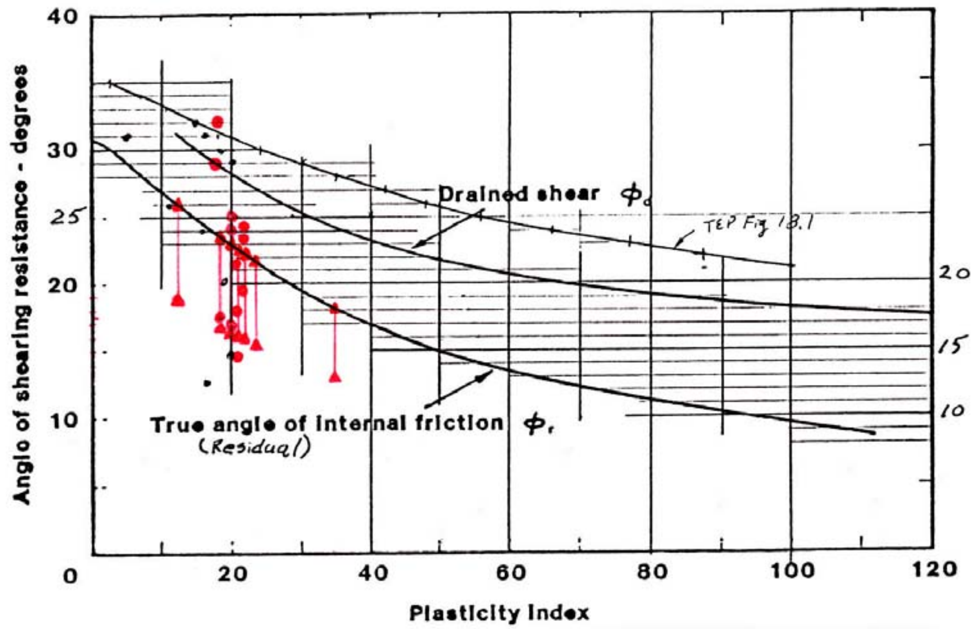


Fig. 9 – *Range of residual shear strengths from actual tests on materials sampled in the Pliocene-age Orinda formation in the Mission Peak Landslide. The lower thresholds are attributable to presence of smectite clays.*

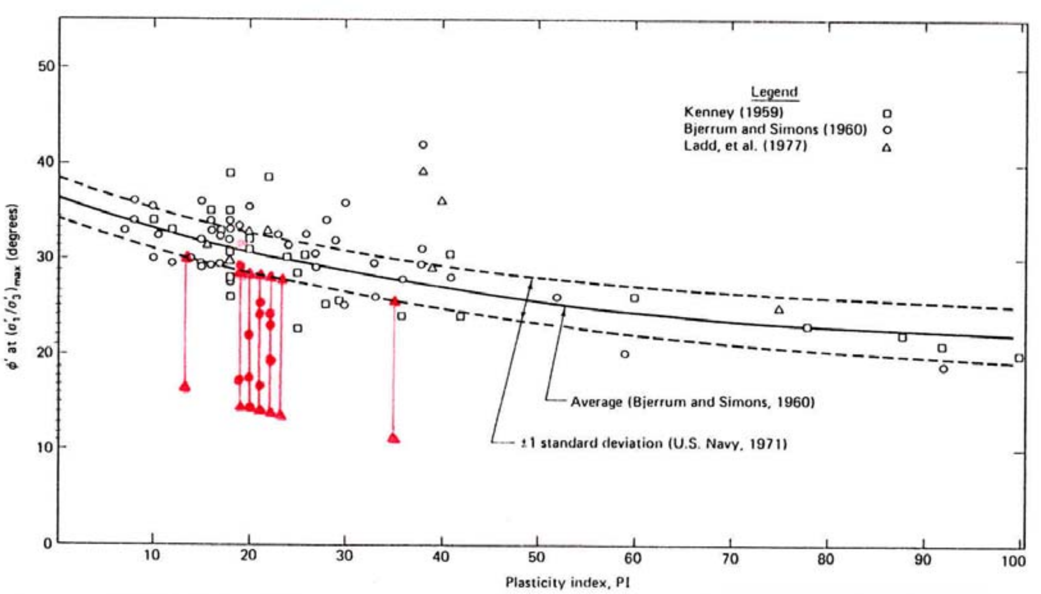


Fig. 10 — *Range of residual shear strengths from actual tests on materials sampled in the Pliocene-age Orinda formation in the Mission Peak Landslide. The lower thresholds are attributable to presence of smectite clays.*

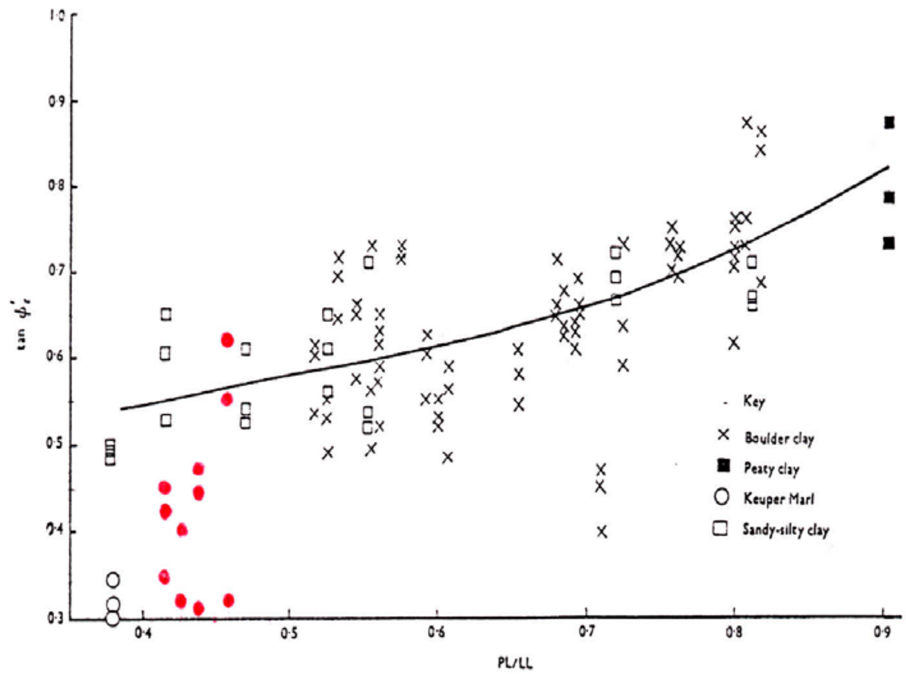


Fig. 11 — *Range of residual shear strengths from actual tests on materials sampled in the Pliocene-age Orinda formation in the Mission Peak Landslide. The lower thresholds are attributable to presence of smectite clays.*

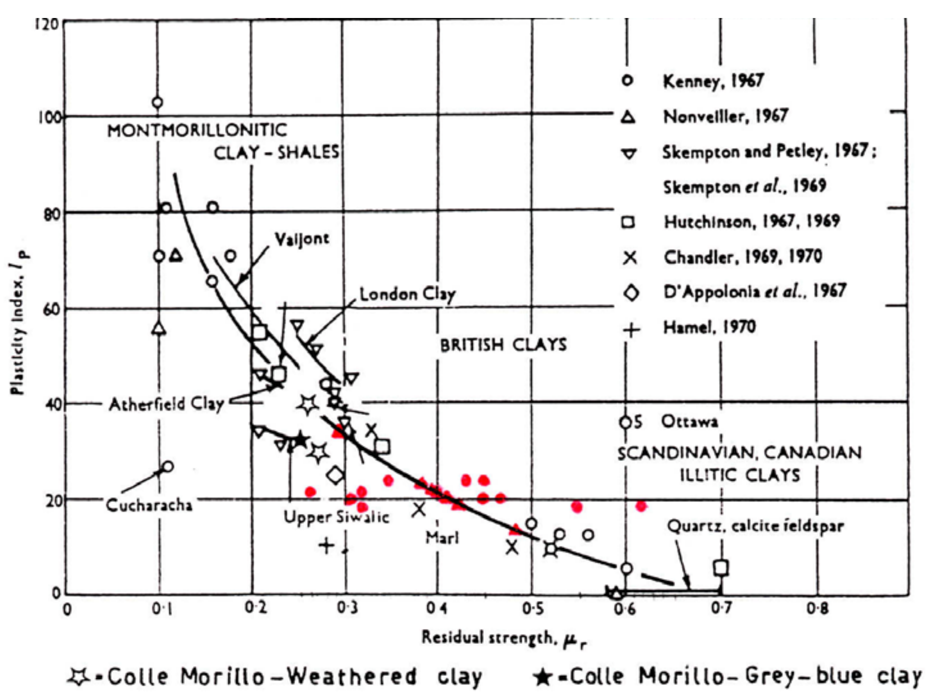


Fig.12 - *Range of residual shear strengths from actual tests on materials sampled in the Pliocene-age Orinda formation in the Mission Peak Landslide. These data plotted closer to the line of mean data because these data were derived from montmorillonitic clay shales, similar to the Orinda formation. .*