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# Estimation of the Undrained Strength Characteristics of Mine Tailings Deposits Using Field Vane Shear Test Data

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## ABSTRACT

The Field Vane Shear Test (FVST) has been used in the past as a practical and economical means of measuring the in-situ undrained strength of cohesive soils. Values of peak, residual and remolded strengths can be measured with the FVST, providing insight into the material's behavior under various loading conditions and strain levels. The FVST is particularly well-suited to soils which are difficult to sample in-situ.

Since tailings deposits are generally in a loose to very loose state and seldom exhibit any true cohesion, the recovery of high-quality, reliable, undisturbed samples is often difficult or impossible. As such, a simpler and more cost-effective alternative to testing undisturbed samples in the laboratory would be of value. As part of the field exploration programs carried out in conjunction with the construction and expansion of mine tailings facilities in the USA, the authors have conducted various types of in-situ tests (SPT, CPT, etc.) including FVSTs, utilizing a strain-gage equipped electronic field vane to obtain an estimate of the undrained shear of various tailings deposits for use in both liquefaction and post-earthquake slope stability analyses.

Values of peak, residual and remolded shear strengths of the tailings materials have been obtained using the FVST. The residual and remolded strength values have been compared to residual strength values for sandy and silty soils proposed by various investigators (Seed and Harder, 1990; Stark and Mesri, 1992; and, Bur. Rec., 1989). The results of these studies indicate that the FVST provides an excellent tool for estimating the undrained shear strength characteristics of tailings deposits under various loading conditions, including post-earthquake behavior.

## INTRODUCTION

The proper soil strength for use in a post-earthquake stability analysis is, unfortunately, not well understood at this time. While a vast amount of research has been conducted on this subject the majority of the available data is for clean sands or silty sands. In addition, there are relatively few case histories from which one may assess the accuracy of any selected strength value.. The authors of the most commonly used methods for establishing this strength (Poulos, 1988; Seed, 1987) offer significant insight into the soil's behavior during or shortly after an earthquake. However, there are some apparent disagreements in the results which the two methods yield. An additional tool for estimating the remolded undrained shear strength of a cohesive soil or tailings deposits is the FVST. By including an additional means of estimating this post-earthquake strength, a higher level of confidence may be achieved, and therefore, a more meaningful analysis.

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The remolded undrained shear strength of a tailings deposit corresponds to its minimum strength at its in-situ void ratio. The peak strength of a tailings may be reduced to its residual or remolded strength through the introduction of any undrained loading which causes sufficient strain as to exceed the strain at peak strength. In order for this loss in strength to occur, the induced shear strains must be large enough to break down any initial structures which may exist in the soil, and transform it into steady-state conditions (Poulos, Robinsky and Keller, 1985). The liquefied strength of a soil is often referred to as the residual strength by Seed (1987), the steady-state strength by Poulos et al. (1985) and the undrained critical strength by Casagrande (1936). The term most often applied to the lowest strength which can obtained during shear vane testing is the remolded strength. The residual strength during an FVST corresponds to an intermediate strength between the peak and the remolded strengths where sufficient shear strains have taken place to exceed the peak value but not enough to reach the remolded strength value .

The remolded strength of a soil is an appropriate strength for use in the post-earthquake analysis of slope stability (Stark and Mesri, 1992). Seed (1987) has indicated that the determination of the residual strength of a liquefied soil for use in the evaluation of slope stability is useful for cases where the prevention of catastrophic failures, while allowing some limited slope displacement, is an acceptable solution. This approach may sometimes be applicable to the case of tailings dams where large deformations may be acceptable as long as contained materials are not released in an uncontrolled manner from the structure. In these cases, the determination of a remolded strength for the liquified tailings mass can be a major aspect in the evaluation of seismic stability (Seed, 1987).

Seed (1988) noted that the best method for the determination of liquefied strengths of sands would be through the measurement of remolded strengths on truly undisturbed samples. This establishment of a relationship between the remolded strength of a soil, as determined by field performance studies, and some in-situ soil property may provide the most practical method for evaluating the liquified strength of a soil (Seed, 1987). For clean sands, triaxial tests have been used to estimate the residual strength. Or, for that matter, any suitable test method may be used, including rotational shear, direct simple shear or the vane test (Poulos, 1988).

When applying the FVST to mine tailings, the rotation of the vane must be rapid enough to allow for minimal drainage to take place within the sample (Poulos, Robinsky and Keller, 1985). The use of the FVST to directly determine undrained shear strength of tailings under in-situ conditions has been found to yield realistic results when drainage is limited within the test sample (Vick, 1990).

## BACKGROUND

There are presently two main schools of thought in regard to the estimation of the remolded strength of a soil. The first method was established by Casagrande (1936), and utilizes a steady-state concept in which the minimum soil strength is established from laboratory shear tests. The second method was proposed by Seed (1987), through which the lower-bound strength is estimated from case histories.

The use of laboratory test methods for the determination of the steady-state strength of a soil has been established using the results of consolidated-undrained triaxial compression tests on undisturbed samples. A steady-state strength of the soil, at which the soil deforms continuously without a change to the resistance to deformation, is determined after applying corrections to compensate for densification of the sample (Poulos et al., 1985). It has been found, however, that this method will often lead to strength values which are significantly higher than those calculated from actual field performance studies. This may be due to a number of reasons, including the assumption that a constant void ratio persists in a soil even after it has liquified. It is possible that there may be a redistribution of water in a liquefied soil, and that the lowest strength of the liquefied soil may be that of a loosened zone of sand where the void ratio during liquefaction is higher than the initial value (Seed, 1987).

Through the study of field performance cases of slopes which have undergone liquefaction failure, Seed (1987) was able to estimate the remolded shear strength of several soils. Through back-calculation using limit equilibrium analysis, the final geometry of the slide mass and different slide surfaces, the minimum soil strength which would have been necessary to cause the slope failure was determined. SPT blow counts, corrected to a clean sand value, were determined (or estimated in some cases) and a useful relationship between the SPT values and remolded strength based on case histories was derived (Stark and Mesri, 1992).

Other researchers question the accuracy of this approach. It has been noted that no case histories have been reported where blow count data was obtained prior to failure. In addition, blow counts, being very sensitive to the procedure used, may be regarded by some as a crude index test. As such, correlations between SPT blow counts and back-calculated strengths may be subject to interpretation (Davis et al., 1988). Although one might expect a reasonable correlation between the SPT blow count and the remolded shear strength of a soil, the fact that several unknown or estimated factors were used in back-calculated case histories, along with possible deviations from the standards during SPT testing, the use of these correlations may not always present accurate results. Additionally, blow counts are relatively insensitive at low values, the range where this information would be most useful (Poulos, 1988).

Stark and Mesri (1992) used laboratory test results and back-analysis in an attempt to refine the approach first presented by Seed. After difficulties were encountered in determining appropriate SPT blow count values for this approach, it was determined that the use of the relationship correlating cyclic stress ratio and SPT blow count (from Seed et al. 1985) would be appropriate. Cyclic stress ratios at 15 equivalent cycles in a cyclic simple shear test and an earthquake magnitude of 7.5 were used for this analysis. The slope configuration prior to failure was used in the back-analysis, and therefore it was assumed that the shear strength determined represented an undrained, constant volume residual strength, and that possible drainage conditions which may occur in the soil sometime after the failure began would therefore be discounted. In the results of this study, it was determined that the undrained shear strength for use in a post-earthquake stability analysis could be significantly less than that which was first presented by Seed.

Research conducted by the U.S. Bureau of Reclamation (1989) suggests that a relationship between the steady-state strength of a soil, as determined through laboratory testing, and the SPT blow count does not lend itself to a good correlation. Significant scatter of the data was obtained in the test results. Lateral variability of subsurface conditions and disturbance to samples during the sampling and retrieval process were sited as possible factors which led to this observation.

## FIELD VANE SHEAR TESTING IN MINE TAILINGS

The authors have carried out numerous intensive geotechnical site investigations at mine tailings facilities throughout the western U.S. in conjunction with the construction and expansion of these facilities. Through the site investigation activities, information and data pertaining to the static and dynamic behavior of the various contained materials were collected.

Mine tailings are mechanically broken particles in which the size is not dependent on the strength of the parent rock, but rather on the particular milling requirements. The product of this milling process is typically a fine silty/sand or sandy/silt which has little to no cohesion but contains significant amounts of fines usually in the range of 30 % to as high as 80 % passing the #200 sieve. The tailings are deposited in slurry form, and as such, generally settle to a loose to very loose state. The recovery of high-quality, reliable, undisturbed samples of this material is often difficult or impossible. As such, it is preferable to use in-situ testing methods whenever possible. In-situ testing techniques used at these sites typically include SPT, CPT and FVST.

The vane system used in these field tests consisted of two vane blades mounted on the end of an electronic load cell. The vane, designed by Acker, is described as a 5 cm blade (i.e., 5 cm across). The load cell, which is designed to measure the torque transmitted from the blade, is only 38 cm behind the blade, and hence the torque measured by the load cell represents the torque at the blade and is not influenced by rod friction. Torque measured during testing is transmitted to the surface by electrical cable, and is digitally recorded by computer for interpretation and analysis.

Typically, the surface of a tailings beach designated for in-situ testing is very soft and access to the area is limited, often even foot traffic is impossible. As such, an earth fill ramp-way is constructed to allow for the testing equipment to be moved into place. Initially, an auger hole is drilled through the fill material. The vane device is then loaded into the boring and pushed into position for testing. The rods to which the vane is mounted are then rotated, at the surface, through a 180° arc via the use of a large pipe wrench. Rotation speed during testing and measured torque are monitored by computer. The speed by which the vane is rotated is selected to prevent pore pressure drainage whenever possible. The maximum reading obtained during this phase of the test is noted to be the peak strength, while the value at 180° of rotation is taken as the residual strength. The total angular strain at 180° of rotation is approximately 50 percent.

Following the initial 180° rotation, the vane device is rotated through at least three full revolutions in order to completely remold the soil and break down its original structure. The second portion of the test is conducted in a similar manner to the first, and the remolded strength is noted to be the lowest value obtained during the 180° rotation. Subsequent tests may then be at greater depths by adding drill rods at the surface and advancing the vane to the desired depth.

An example of SPT and FVST data collected from a series of field tests at one particular site are presented herein to show results which have been found to be typical for mine tailings deposits. In Table 1 SPT data measured at seven different locations within the facility have been tabulated together with other pertinent information necessary to calculate the residual shear strength of the tailings deposits based on the Seed and Harder (1990) and the Stark and Mesri (1992) procedures. The measured SPT blow counts are plotted in Figure 1 and generally range between 0 and 5 with the majority of the points less than 3. This range of values is typical of the blow counts measured in loose to very loose tailings deposits and does not generally reflect any increase in blow counts with depth at shallow to moderate depths. In addition, the SPT does not provide any insight into the generally layered nature of tailings deposits which can be clearly identified from CPT and FVST data.

The measured SPT values were then corrected for all of the usual factors including effective overburden pressure to arrive at the $(N_1)_{60}$ values which are tabulated in Table 1. These values were then furthered corrected for the effects of fines content to arrive at values of $(N_1)_{60\text{-}cs}$ for both procedures. Since the average fines content of the tailings deposits was on the order of 35 % a uniform fines correction of 3 was used in the Seed and Harder (1990) approach as recommended by the authors. In the Stark and Mesri approach the authors recommend the use of what they refer to as the yield strength fines correction which would be 7 for a 35 % fines content. Because of the low measured blow count values it was our opinion that this large an increase in the blow counts was unreasonable, and thus what we considered to be a more reasonable fines correction was applied as follows. For values of $(N_1)_{60}$ greater than 5 the maximum fines correction of 7 was used; for values of $(N_1)_{60}$ equal to or less than 1 a fines correction of 2 was applied; for values of $(N_1)_{60}$ between 1 and 5 intermediate fines corrections were used as shown in Table 1. Values of the corresponding clean sand residual shear strengths were then computed using both approaches. The average of the Seed and Harder (1990) relationship between clean sand blowcounts and undrained residual strength from case histories was used, as well as the relationship proposed by Stark and Mesri (1992); $S_u$ (critical) = 0.0055 x effective overburden pressure x $(N_1)_{60\text{-}cs}$, to compute values of residual shear strength. The values of residual shear computed using both approaches are tabulated on

**TABLE 1**
**Evaluation of Residual Shear Strength from Corrected "Clean Sand" Blow Counts**

| Boring Location | Depth Below Tailing Surface (ft) | Depth Below Top of Road (ft) | SPT Measured Blow Count | Corr. SPT Blow Count (w/o fines) | Estimated Fines Content | Ncorr (for fines content) | (N1)60-cs (Seed+Harder-90) | Residual Strength (Seed-90) (psf) | (N1)60-cs (Stark-92) | Effective Confining Pressure (psf) | Residual Strength (Stark-92) (psf) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| A*+40 | 1 | 9 | 4 | 5.4 | 35 | 3.0 | 8.4 | 220 | 12.4 | 108.0 | 7 |
| (Ramp = 8') | 6 | 14 | 2 | 2.9 | 35 | 3.0 | 5.9 | 148 | 7.9 | 486.0 | 21 |
| | 11 | 19 | 2 | 2.6 | 35 | 3.0 | 5.6 | 140 | 7.6 | 823.5 | 34 |
| | 16 | 24 | 1 | 1.2 | 35 | 3.0 | 4.2 | 105 | 4.2 | 1168.0 | 27 |
| | 21 | 29 | 3 | 3.2 | 35 | 3.0 | 6.2 | 155 | 8.2 | 1540.5 | 69 |
| | 26 | 34 | 1.5 | 1.5 | 35 | 3.0 | 4.5 | 113 | 5.5 | 1913.0 | 58 |
| | 31 | 39 | 1 | 0.9 | 35 | 3.0 | 3.9 | 97 | 2.9 | 2285.5 | 36 |
| | 36 | 44 | 0.3 | 0.3 | 35 | 3.0 | 3.3 | 77 | 2.3 | 2658.0 | 34 |
| | 41 | 49 | 2 | 1.6 | 35 | 3.0 | 4.6 | 115 | 5.6 | 3030.5 | 93 |
| A*+100 | 0 | 4 | 4 | 8.5 | 35 | 3.0 | 11.5 | 420 | 15.5 | 0.0 | 0 |
| (Ramp = 4') | 5 | 9 | 2 | 2.6 | 35 | 3.0 | 5.6 | 140 | 7.6 | 418.5 | 17 |
| | 10 | 14 | 2 | 2.9 | 35 | 3.0 | 5.9 | 148 | 7.9 | 756.0 | 33 |
| | 15 | 19 | 2 | 2.5 | 35 | 3.0 | 5.5 | 138 | 7.5 | 1093.5 | 45 |
| | 25 | 29 | 1 | 1.0 | 35 | 3.0 | 4.0 | 100 | 4.0 | 1838.5 | 40 |
| | 30 | 34 | 1 | 0.9 | 35 | 3.0 | 3.9 | 97 | 2.9 | 2211.0 | 35 |
| | 35 | 39 | 3 | 2.6 | 35 | 3.0 | 5.6 | 140 | 7.6 | 2583.5 | 108 |
| | 40 | 44 | 3 | 2.4 | 35 | 3.0 | 5.4 | 135 | 7.4 | 2956.0 | 120 |
| | 45 | 49 | 10 | 7.7 | 35 | 3.0 | 10.7 | 356 | 14.7 | 3328.5 | 269 |
| C*+10 | | 21 | 1 | 0.9 | 35 | 3.0 | 3.9 | 97 | 2.9 | 2277.0 | 36 |
| (Crest) | | 34 | 1 | 0.8 | 35 | 3.0 | 3.8 | 93 | 2.8 | 3310.5 | 51 |
| C*+40 | 1 | 9 | 2 | 2.7 | 35 | 3.0 | 5.7 | 143 | 7.7 | 108.0 | 5 |
| (Ramp = 8') | 6 | 14 | 5 | 7.3 | 35 | 3.0 | 10.3 | 324 | 14.3 | 486.0 | 38 |
| | 11 | 19 | 2 | 2.6 | 35 | 3.0 | 5.6 | 140 | 7.6 | 823.5 | 34 |
| | 16 | 24 | 3 | 3.5 | 35 | 3.0 | 6.5 | 163 | 8.5 | 1168.0 | 55 |
| | 21 | 29 | 1 | 1.1 | 35 | 3.0 | 4.1 | 103 | 4.1 | 1540.5 | 35 |
| | 26 | 34 | 1 | 1.0 | 35 | 3.0 | 4.0 | 100 | 4.0 | 1913.0 | 42 |
| | 31 | 39 | 2 | 1.8 | 35 | 3.0 | 4.8 | 120 | 5.8 | 2285.5 | 73 |
| | 36 | 44 | 2 | 1.7 | 35 | 3.0 | 4.7 | 118 | 5.7 | 2658.0 | 83 |
| | 41 | 49 | 16 | 12.7 | 35 | 3.0 | 15.7 | 750 | 19.7 | 3030.5 | 328 |
| | 46 | 54 | 5 | 3.8 | 35 | 3.0 | 6.8 | 170 | 9.8 | 3403.0 | 183 |
| C*+100 | 4 | 9 | 2 | 2.6 | 35 | 3.0 | 5.6 | 140 | 7.6 | 351.0 | 15 |
| (Ramp = 5') | 9 | 14 | 3 | 4.4 | 35 | 3.0 | 7.4 | 185 | 10.4 | 688.5 | 39 |
| | 14 | 19 | 2 | 2.5 | 35 | 3.0 | 5.5 | 138 | 7.5 | 1026.0 | 42 |
| | 19 | 24 | 2 | 2.3 | 35 | 3.0 | 5.3 | 133 | 6.3 | 1391.5 | 48 |
| | 24 | 29 | 1 | 1.0 | 35 | 3.0 | 4.0 | 100 | 4.0 | 1764.0 | 39 |
| | 29 | 34 | 1.5 | 1.4 | 35 | 3.0 | 4.4 | 110 | 4.4 | 2136.5 | 52 |
| | 34 | 39 | 1 | 0.9 | 35 | 3.0 | 3.9 | 97 | 2.9 | 2509.0 | 40 |
| | 39 | 44 | 0.7 | 0.6 | 35 | 3.0 | 3.6 | 87 | 2.6 | 2881.5 | 41 |
| | 44 | 49 | 0.7 | 0.5 | 35 | 3.0 | 3.5 | 83 | 2.5 | 3254.0 | 45 |
| | 49 | 54 | 3 | 2.2 | 35 | 3.0 | 5.2 | 130 | 6.2 | 3626.5 | 124 |
| | 54 | 59 | 14 | 9.8 | 35 | 3.0 | 12.8 | 524 | 16.8 | 3999.0 | 370 |
| | 59 | 64 | 6 | 4.0 | 35 | 3.0 | 7.0 | 175 | 10.0 | 4371.5 | 240 |
| | 64 | 69 | 14 | 9.0 | 35 | 3.0 | 12.0 | 460 | 16.0 | 4744.0 | 417 |
| E*+40 | 1 | 9 | 7 | 9.5 | 35 | 3.0 | 12.5 | 500 | 16.5 | 108.0 | 10 |
| (Ramp = 8') | 6 | 14 | 3 | 4.4 | 35 | 3.0 | 7.4 | 185 | 10.4 | 486.0 | 28 |
| | 11 | 19 | 2 | 2.6 | 35 | 3.0 | 5.6 | 140 | 7.6 | 823.5 | 34 |
| | 16 | 24 | 3 | 3.5 | 35 | 3.0 | 6.5 | 163 | 8.5 | 1175.0 | 55 |
| | 21 | 29 | 2 | 2.1 | 35 | 3.0 | 5.1 | 128 | 6.1 | 1582.5 | 53 |
| | 26 | 34 | 1 | 1.0 | 35 | 3.0 | 4.0 | 100 | 3.0 | 1990.0 | 33 |
| E*+100 | 1 | 4 | 4 | 7.8 | 35 | 3.0 | 10.8 | 364 | 14.8 | 108.0 | 9 |
| (Ramp = 3') | 6 | 9 | 4 | 5.2 | 35 | 3.0 | 8.2 | 210 | 12.2 | 486.0 | 33 |
| | 11 | 14 | 3 | 4.3 | 35 | 3.0 | 7.3 | 183 | 10.3 | 823.5 | 47 |
| | 16 | 19 | 6 | 7.5 | 35 | 3.0 | 10.5 | 340 | 14.5 | 1175.0 | 94 |
| | 21 | 24 | 5 | 5.5 | 35 | 3.0 | 8.5 | 225 | 12.5 | 1582.5 | 109 |
| | 26 | 29 | 6 | 5.9 | 35 | 3.0 | 8.9 | 245 | 12.9 | 1990.0 | 141 |
| | 31 | 34 | 3 | 2.7 | 35 | 3.0 | 5.7 | 143 | 7.7 | 2397.5 | 102 |
| | 36 | 39 | 3 | 2.5 | 35 | 3.0 | 5.5 | 138 | 7.5 | 2805.0 | 116 |

Table 1. These will be discussed subsequently.

In Figure 2 values of peak, residual and remolded shear strengths obtained from FVSTs conducted at one of the seven site locations are plotted. The reduction in strength as one goes from peak to residual to remolded strength is readily apparent. It can be seen that the a nearly continuous profile of the tailings deposit can be obtained from the FVSTs. It is of interest to note that the randomness of the peak strength profile is very similar to the tip resistance versus depth profile obtained from a CPT test conducted at the same location, as would be expected for a highly layered soil deposit. The effects of layering become less recognizable for the remolded values since the initial structure of the material has been destroyed. Since the entire depth of the tailings deposit is essentially comprised of the same material (with alternating layers of coarser and finer materials) but subjected to varying levels of desiccation, it stands to reason that the remolded strength of the deposit would remain essentially constant with depth. This can be seen more clearly in Figure 3 where the mean values of shear strength obtained at all seven locations have been plotted versus depth. The strength profiles in this figure are much smoother since the data has been statistically averaged.

Values of residual shear strength shown in Table 1 and computed using both the Seed and Harder (1990) and Stark and Mesri (1992) procedures have been plotted in Figure 4 and are shown as the individual open and solid squares data points. It can be seen from this plot that the Stark and Mesri method is considerably more conservative than the Seed and Harder approach especially at shallow depths where the effective overburden pressure is low. In fact at these shallow depths it is probably overconservative since even for very high blow counts this procedure would generally lead to very low values of residual strength at shallow depths. Also plotted in Figure 4 are the mean and mean plus and minus one standard deviation values of FVST remolded strengths obtained at all seven locations. It can be seen that the range of the FVST data generally encompasses the other residual strength values. In general at depths less than 20 feet the mean FVST relationship falls between the values computed using the two other approaches. At greater depths it is somewhat to the right of the other data. Nevertheless, it is apparent that the remolded FVST does appear to provide a reasonable estimate of the residual undrained strength of tailings deposits.

Another manner of presenting the FVST remolded strength data is shown in Figure 5. The solid lines shown on the figure represent the range of historical residual strength values calculated by Seed and Harder (1990). Also shown on this figure are the solid squares which are the result of research conducted by the Bureau of Reclamation (1989) relating the steady-state strength of a soil, as determined by laboratory testing and SPT blow count data. The third data set consisting of the open circles correspond to the FVST remolded strength values for this study obtained at locations where SPT data and corresponding $(N_1)_{60\text{-}cs}$ values were available. It can be seen that the FVST data set compares very favorably with the findings presented by Seed and Harder (1990), whereas the Bureau of Reclamation data are generally much higher.

## CONCLUSIONS

The results of this study indicate that the FVST provides an excellent tool for measuring the in-situ undrained strength characteristics of loose to very loose tailings deposits under various loading conditions. The measured strengths may be used in liquefaction or in post-earthquake stability analysis. Since these materials are extremely difficult or impossible to sample without major disturbance, laboratory tests results may not be meaningful. The remolded strength values obtained from FVSTs compare quite favorably with the range of residual strengths values computed from measured SPT blow counts in the tailings, using two different methodologies developed for clean sands which are considered the present state of practice. Values of steady-state strengths determined by the Bureau of Reclamation are, in general, much greater than those obtained using either FVST data or the residual strengths approaches.

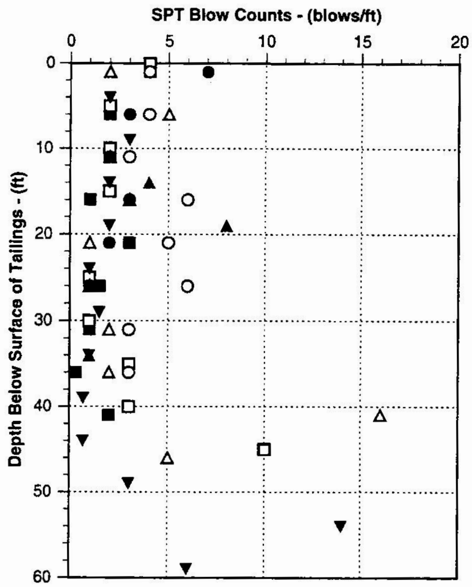



**Figure 1. Measured SPT Blow Counts**

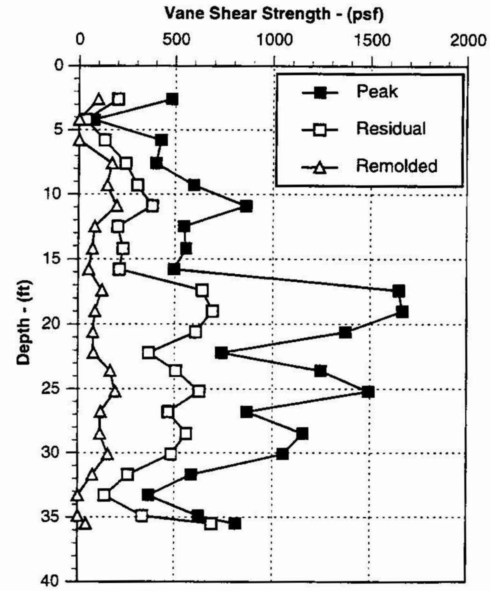



**Figure 2. Field Vane Shear Test (E" +100)**

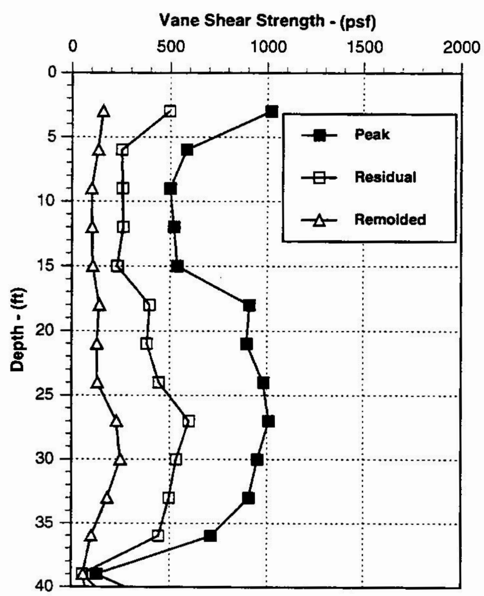
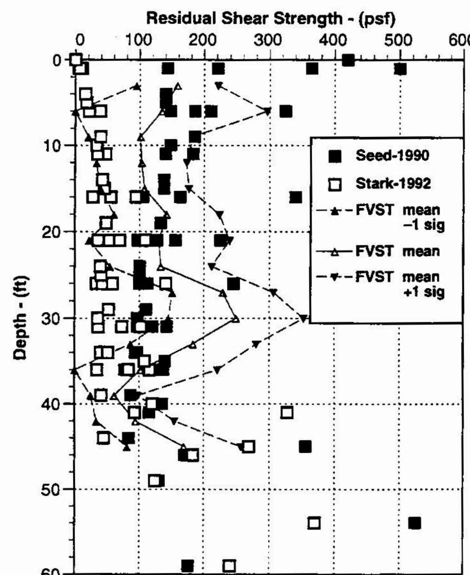



**Figure 3. Summary of Mean Vane Shear Strength Values From Seven Different Locations**




**Notes:**

1) Open and solid squares represents values of residual shear strengths based on corrected "clean sand" blow counts.
2) Mean and mean + and - one sigma lines correspond to range of remolded shear strengths obtained from in-situ FVST.

**Figure 4. Comparison of Residual Shear Strength Data and Remolded Strength from FVST**

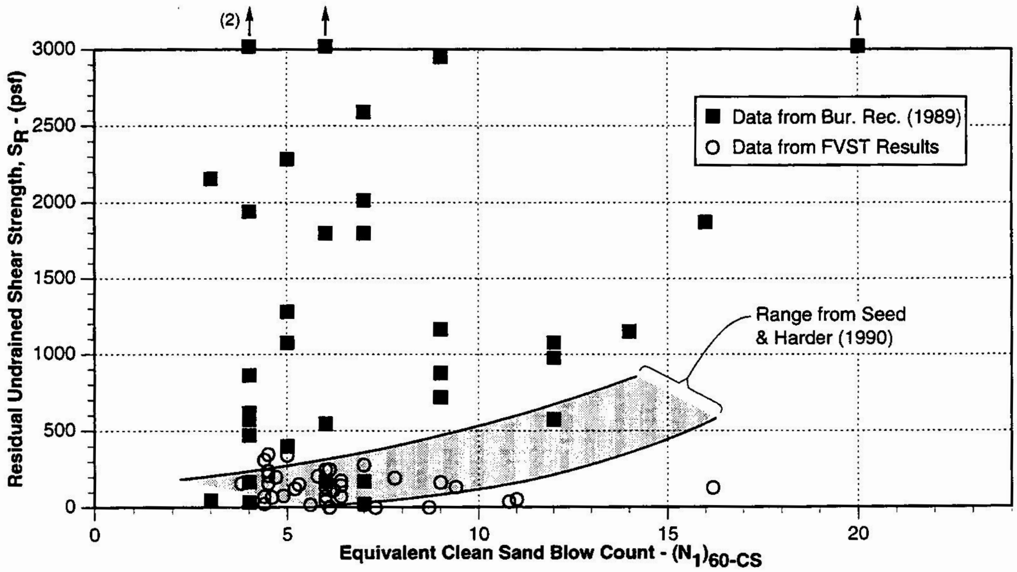



**Figure 5. Comparison of FVST Remolded Strength with Corrected SPT Blow Count**

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