II. MEASUREMENT OF SEISMIC STRENGTH BY SIMPLIFIED METHODS:

When concerns began being raised about the seismic stability of coal refuse disposal facilities in the 1980s, several existing fine refuse deposits were assessed using results of laboratory cyclic triaxial testing and field cross-borehole shear wave velocity testing. The relationship shown below was developed from those studies, relating measured pore water pressure ratio (r_u) to effective consolidation pressure as a result of simulated earthquake loading.

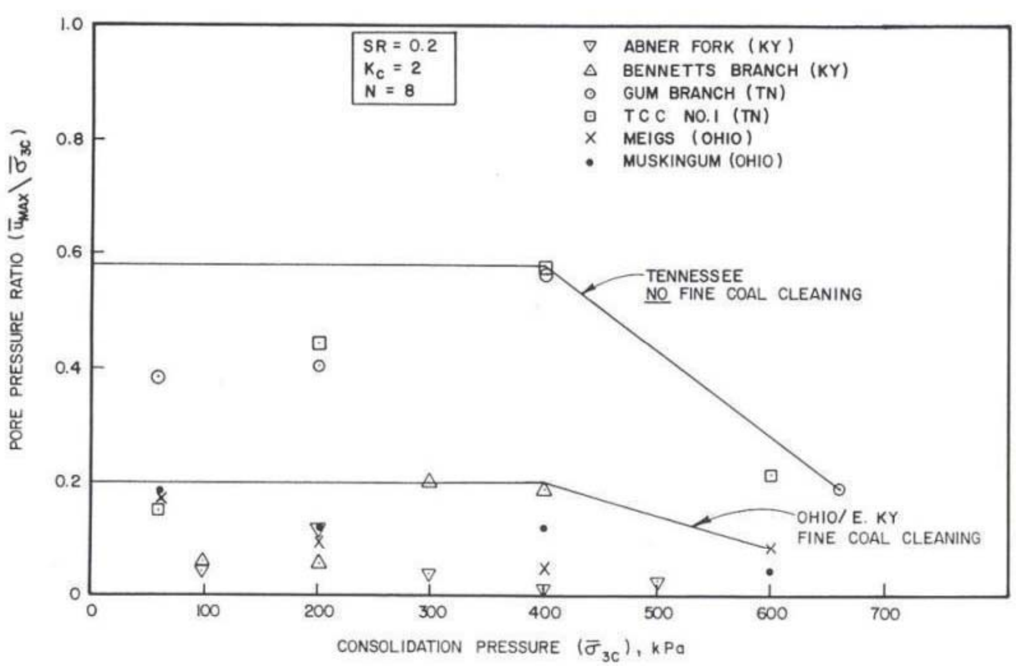


Fig. 22. Results of cyclic triaxial testing on fine refuse samples from the 1980s

A maximum r_u value = 0.6 was measured at sites without fine-coal cleaning circuits where the fine refuse was found to be sand-like. At sites with fine-coal cleaning circuits, the fine refuse was found to be clay-like with maximum r_u = 0.2.

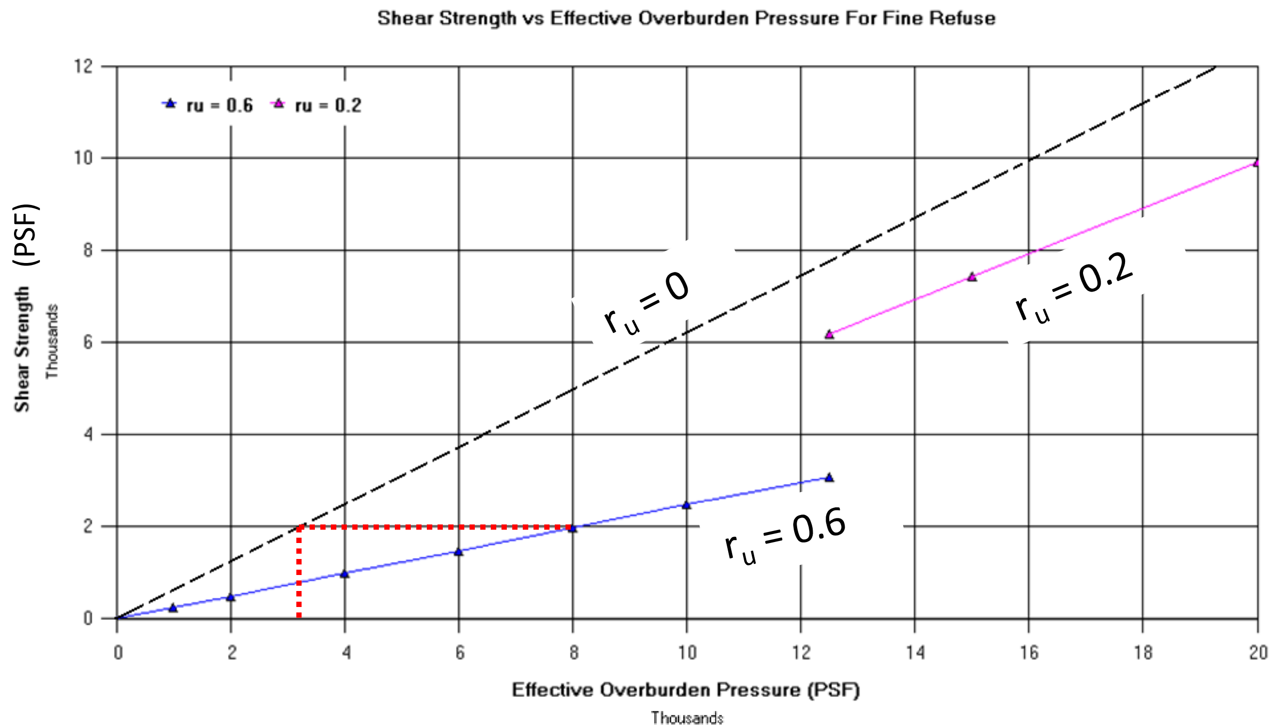
After achieving an effective consolidation pressure of about 600 kPa (i.e. 12,500 psf), sand-like and clay-like fine refuse from the study both have a maximum r_u on the order of 0.2.

An effective overburden pressure (σ'_v) = 12,500 psf is the vertical pressure exerted by about 100 feet of unsaturated coarse refuse. Lateral pressure is a percentage of the vertical pressure.

From the previous results, and site-specific field measurements, the upper bound post-earthquake shear strength of the fine refuse was calculated for use in the seismic assessment of sites included in the former study as shown below for fine refuse with an effective angle of internal friction (Φ') = 32^0 (i.e. $r_u = 0$). An increase in pore water pressure during earthquake loading reduces the effective overburden pressure and thus the shear strength of the fine refuse.

For example, an in-situ sample with $\sigma'_v = 8{,}000$ psf feels only $\sigma'_v = 3{,}200$ psf if an earthquake induces $r_u = 0.6$; thus, the shear strength reduces from 5,000 psf to 2,000 psf.

-- *Note: shear strength =* $\sigma'_v * (1 - r_u) * \tan \Phi'$ --



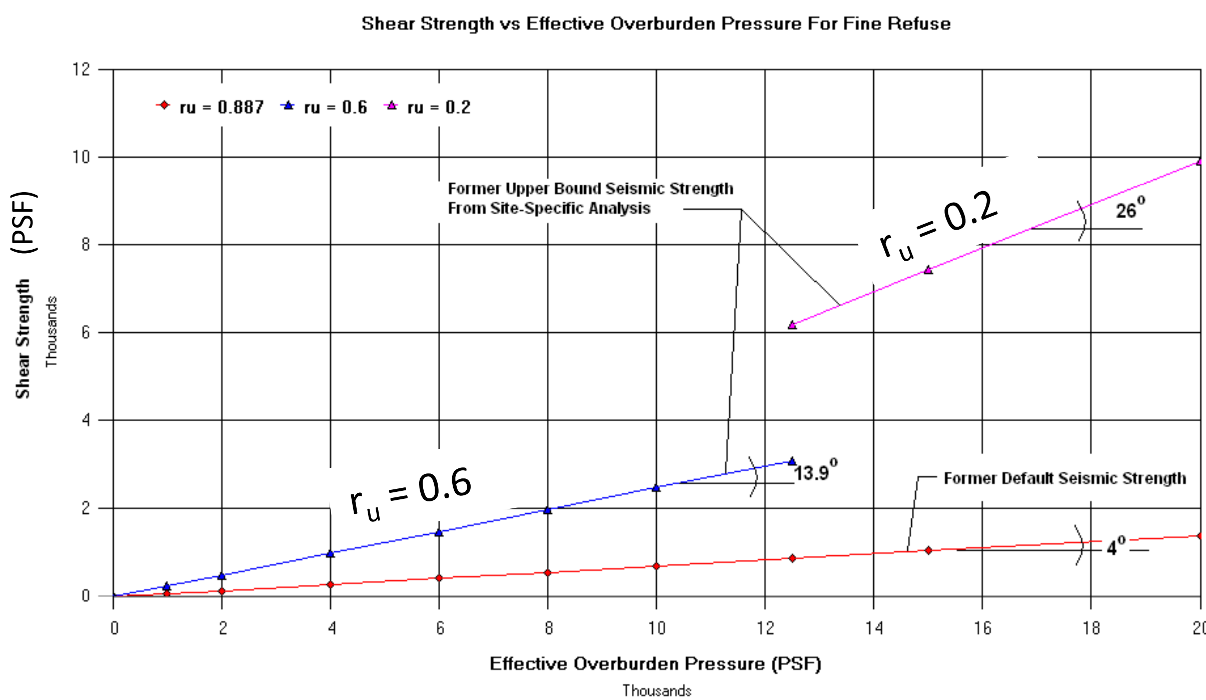
Pore pressure ratio is commonly used during upstream construction because it can be calculated using measurements from pneumatic or vibrating wire piezometers. Before the measured pore water pressures exceed the design pore water pressures, construction can be stopped to allow pressures to dissipate to safe levels. Unfortunately, pore water pressure measurements offer little verification and no early warning mechanism in seismic assessments because the earthquake occurs rapidly.

As part of the previous cyclic triaxial testing program, laboratory vane shear testing was performed and a minimum residual shear strength = 4 degrees was measured, and became the basis for the former simplified design method. Shear strength vs effective overburden pressure for fine refuse, measured in terms of r_u and the angle of the upper bound and default envelopes, is shown in the graph below.

The upper range is from cyclic triaxial data and site-specific field measurements, whereas the lower range is the default strength used in the former simplified seismic design method.

Prior to publication of the new MSHA Manual, the performance criteria that enabled use of peak undrained strength (i.e. $s_{up} = \sigma'_v \tan 26^0$, or $\Phi' = 32^0$* with $r_u = 0.2$*) in seismic stability analysis was to demonstrate average uncorrected N values ≥ 20 in the applicable fine refuse.

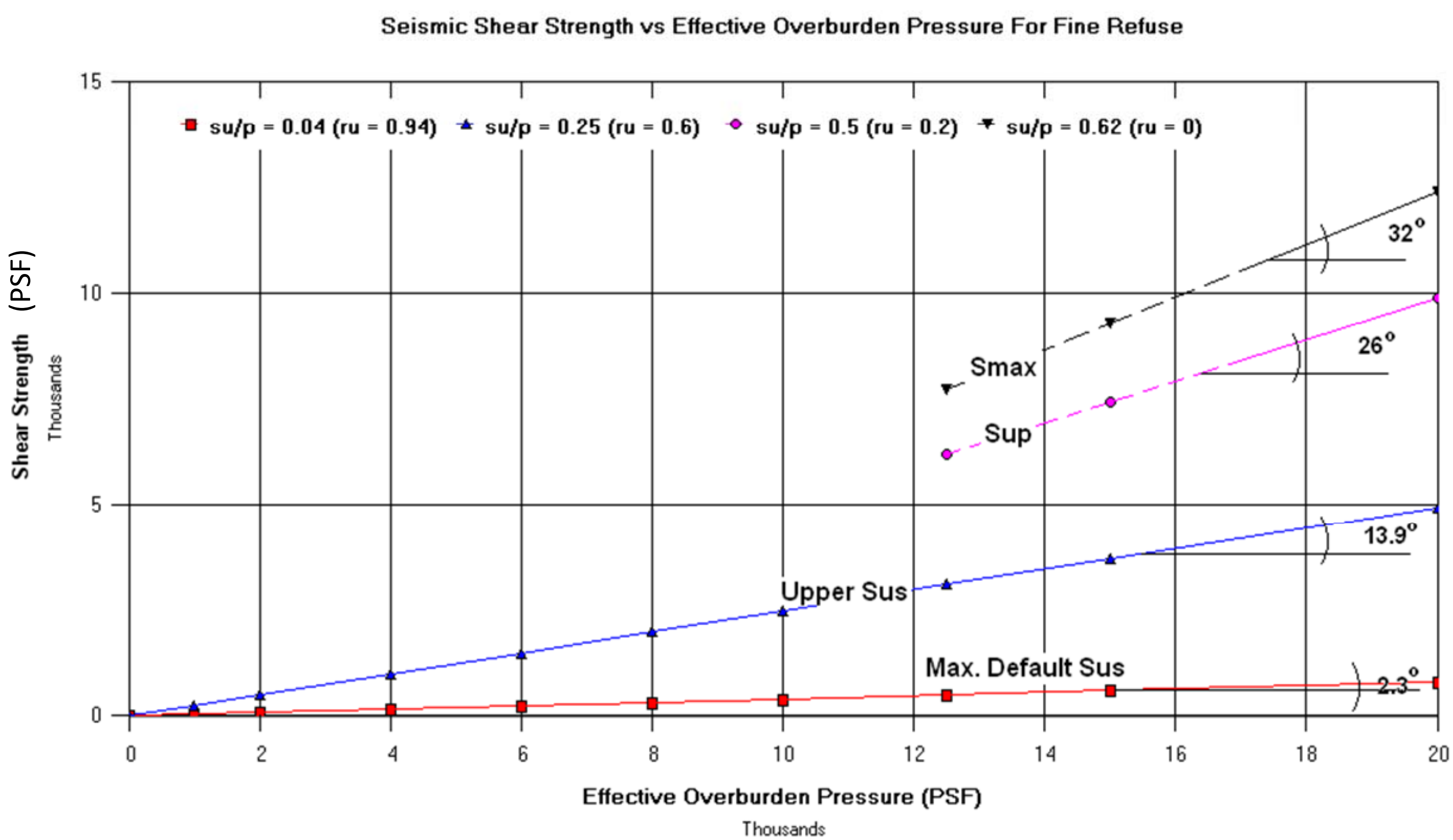
Due to the problems associated with verifying the seismic strength of fine refuse during construction, I began using the default residual strength = 4 degrees in design whenever possible. A factor of safety = 1.0 was used previously due to the conservatism built into using the default strength compared to actual measurements…



… with publication of the new MSHA Manual, this is all now ancient history – or is it?

The new MSHA Manual specifies a maximum default strength for the undrained steady-state residual strength (s_{us}) = 0.04 σ'_v …. which is the undrained strength (s_u) divided by the effective stress (σ'_v or p) = 0.04. Before higher strength can be used in seismic design, site specific data are needed.

Furthermore, the manual provides allowable ranges of seismic strength for sand-like and clay-like fine refuse at various density and anisotropy states that must be verified by field measurements and testing.



Put it all together and you get the relationship shown above… Does it look familiar?

The more things change, the more they stay the same….

The manual allows a default post-earthquake strength as high as 0.04 σ'_v (i.e. $\sigma'_v \tan 2.3^0$) to be justified using the Liquidity Index, residual vane shear, or Standard Penetration Test (SPT) data as shown in the table below. All three are common testing methods used at refuse disposal sites.

TABLE 7.1 COMPARISON OF BASIC CRITERIA FOR SAND-LIKE, CLAY-LIKE AND BORDERLINE MATERIALS

	Sand-Like	Clay-Like	Borderline (Treat as Sand-Like)	Borderline (Treat as Clay-Like)
Atterberg limits % passing No. 40 sieve % passing No. 200 sieve	$PI \le 7$	$PI \ge 10$ ≥ 35 ≥ 20	$7 < PI < 10$	$7 < PI < 10$ ≥ 35 ≥ 20
Triaxial tests on un-disturbed samples to obtain stress-strain curve	Not Required	Not Required	Not Required	Shear strain at peak strength must exceed 5%; otherwise treat as sand-like
Lower-bound post-earthquake strength	0.04 σ'_v if non-plastic, or if LI is < 1.0 20 psf if $LI \ge 1.0$, but no higher than 0.04 σ'_v	0.04 σ'_v if LI is < 1.0 20 psf if $LI \ge 1.0$, but no higher than 0.04 σ'_v	0.04 σ'_v if LI is < 1.0 20 psf if $LI \ge 1.0$, but no higher than 0.04 σ'_v	0.04 σ'_v if LI is < 1.0 20 psf if $LI \ge 1.0$, but no higher than 0.04 σ'_v
Other methods to obtain post-earthquake strength	1. Correlations with SPT/CPT 2. Steady-state lab testing	1. Field vane shear or CPT 2. Cyclic followed by static lab testing	Correlations with SPT/CPT	1. Field vane shear or CPT 2. Cyclic followed by static lab testing
Field vane-shear testing for peak-undrained strength and S_{us}	Not Applicable	Applicable	Potentially Applicable if it can be demonstrated that the test is undrained	Potentially Applicable if it can be demonstrated that the test is undrained
CPT to help identify layering and to differentiate sand-like from clay-like	Recommended	Recommended	Recommended	Recommended
CPT to measure peak undrained strength and S_{us}	Not Applicable	Applicable	Potentially Applicable if it can be demonstrated that the test is undrained (Section 6.4.3.7)	Potentially Applicable if it can be demonstrated that the test is undrained (Section 6.4.3.7)

Note: 1. PI is the Plasticity Index.
2. LI is the Liquidity Index.

- Frequently Asked Questions -

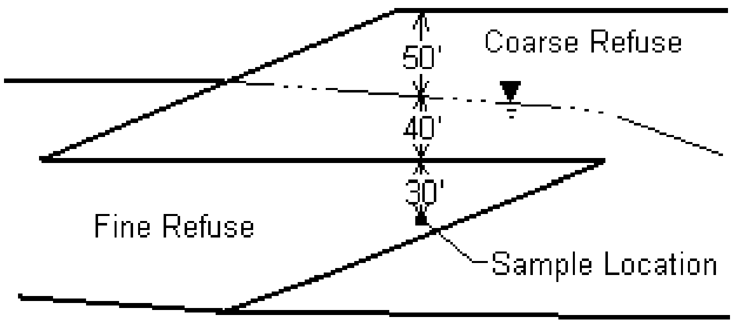
Question: So what are PI and LI?

Answer: You'll learn about the Plasticity Index and the Liquidity Index when we perform index testing of actual fine refuse samples.

Question: But what is 0.04 σ'_v?

Answer: The allowable post-earthquake strength = 0.04 times the effective overburden pressure as shown in the following example problem.

Effective overburden pressure and undrained steady-state residual strength calculations:



Coarse refuse moist unit weight = 120 pcf

Coarse refuse saturated unit weight = 125 pcf

Coarse refuse buoyant unit weight = 125 pcf – 62.4 pcf = 62.6 pcf

Fine refuse buoyant unit weight = 90 pcf – 62.4 pcf = 27.6 pcf

Effective overburden pressure (σ'_v) = (50' * 120 pcf) + (40' * 62.6 pcf) + (30' * 27.6 pcf) = 9,332 psf

Undrained steady-state residual strength (s_{us}) measured by residual vane shear testing at sample location = 2,050 psf (after correction as prescribed in the MSHA Manual)

s_{us}/σ'_v = 2050 psf/9332 psf = 0.22 or $s_{us} = 0.22\ \sigma'_v$, which exceeds the maximum default seismic strength = 0.04 σ'_v (Note: tan 12.4^0 = 0.22 and tan 2.3^0 = 0.04)

- Frequently Asked Questions (continued) -

Question: What are my chances of being able to justify a post-earthquake strength = 0.04 σ'_v and use a simplified design?

Answer: If you started upstream construction sooner rather than later, loaded the fine refuse deposit slowly, and have a substantial coarse refuse zone over the fine refuse (i.e. according to the Bubba Construction Method), your chances are excellent.

For facilities designed and built by the Bubba Construction Method, I have yet to find a post-earthquake strength in fine refuse less than 0.04 σ'_v from residual vane shear testing as shown below.

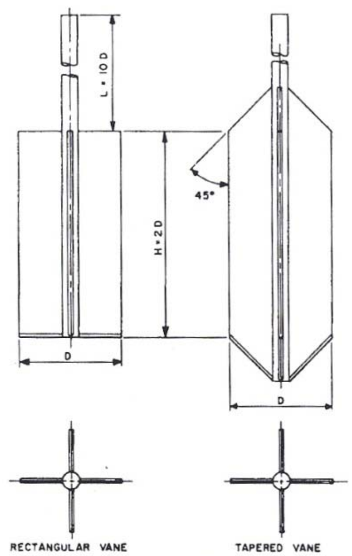
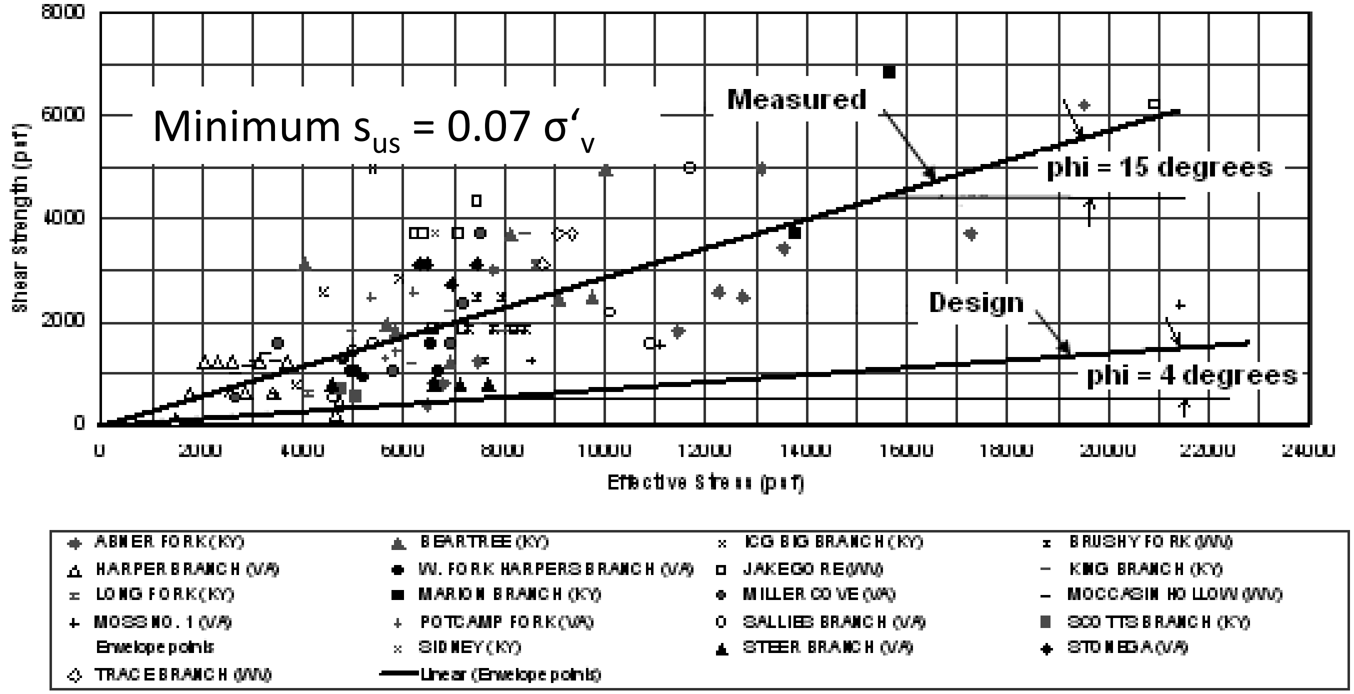


FIG. 1 Geometry of Field Vanes



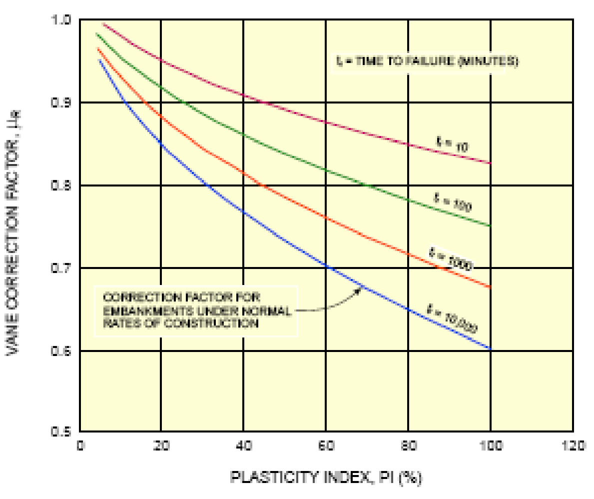


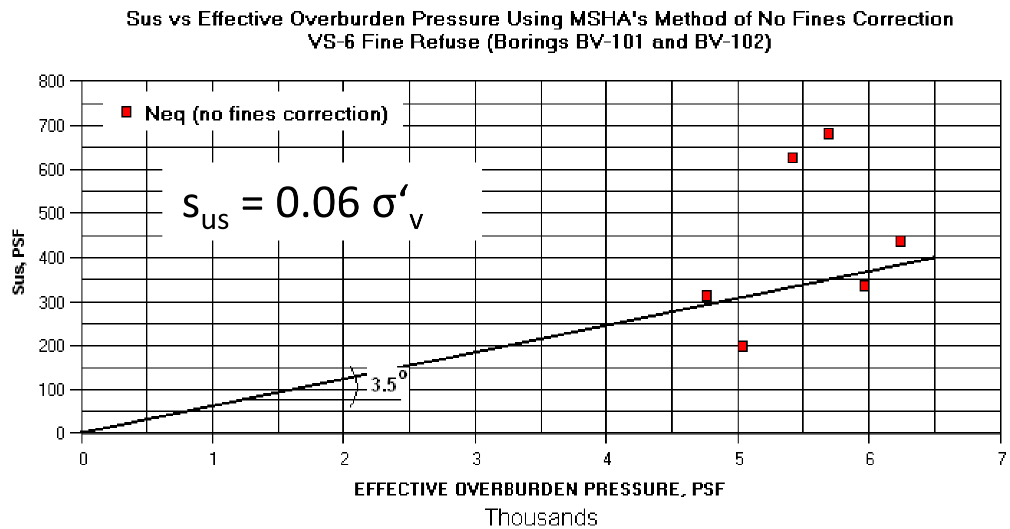
FIGURE 6.14 VANE CORRECTION FACTOR

NOTE: The manual recommends that failure envelopes be drawn where one-third of the data points lie below the line, whereas the "measured" envelope shown above is based on regression analysis.

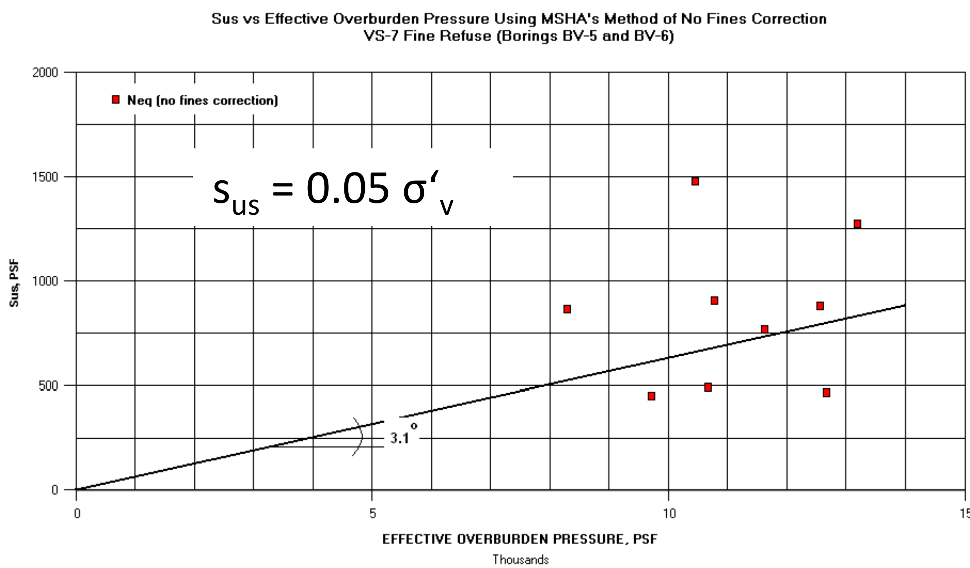
Note: The MSHA Manual includes a correction factor for vane shear data as shown at left.

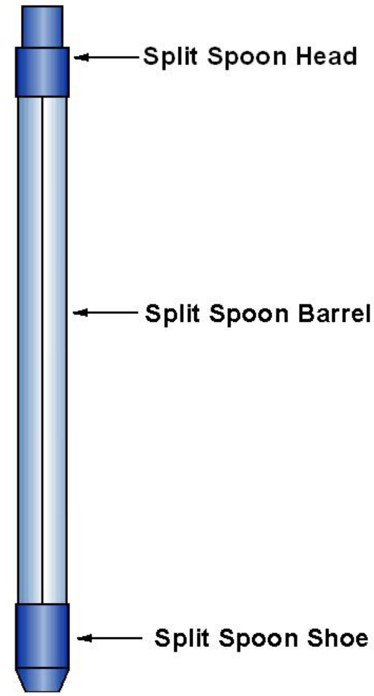
Similarly, I have yet to measure seismic strength less than 0.04 σ'_v for SPT data, -- even using the conservative correlations included in the manual, which require corrections for overburden pressure but not fines content -- regardless of whether the fine refuse deposit was tested…

- immediately after installation of a pushout (i.e. with 40 feet of coarse refuse cover,

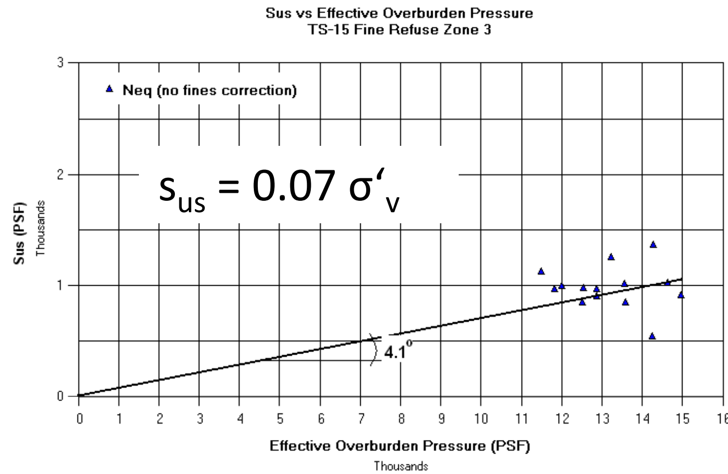


- after completion of the first upstream stage (i.e. with 80 feet of coarse refuse cover),

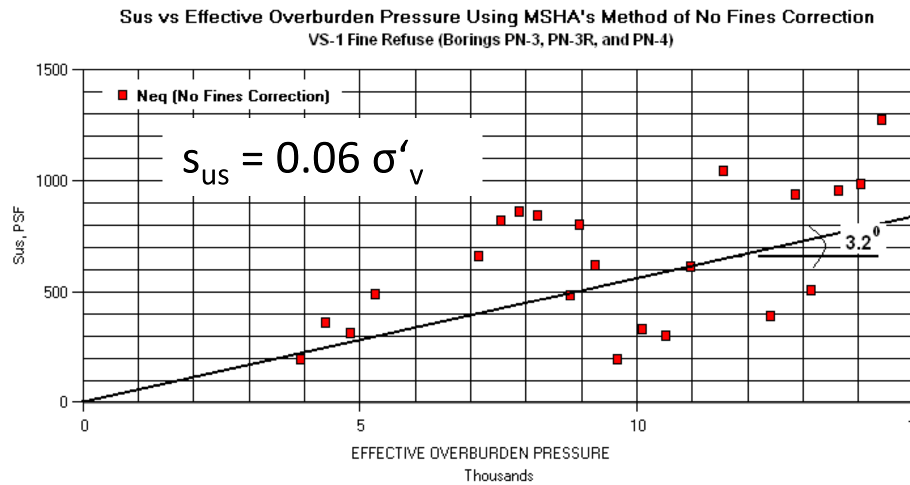




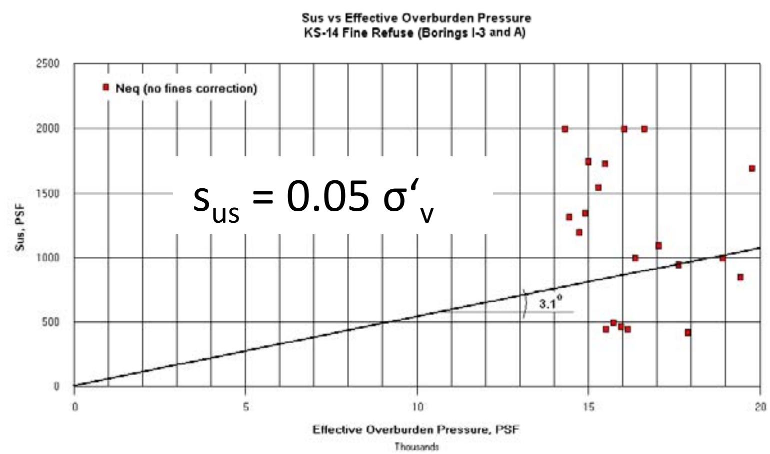
- after loading with multiple stages (i.e. with 120 feet of coarse refuse cover and remaining idle for 15 years),



- after multiple drilling and testing events (i.e. when the coarse refuse cover was 40 feet, 80 feet, and then 120 feet thick above the fine refuse deposit),



- and even after being gradually loaded with 150 feet of coarse refuse cover over a period of 18 years…



…SPT data routinely justifies a post-earthquake strength between 0.05 σ'_v and 0.1 σ'_v in fine refuse deposits using the conservative method recommended in the manual.

I asked my buddy Ray Bob to review the data on s_{us} correlated to SPT N values corrected for overburden pressure, but not fines content. Ray Bob is proud to describe himself as, "a snuff-dippin', coal-minin', 6-foot-6, 250-pound, American veteran"; and his friends call him "Tiny". Tiny observed:

"Wait just a dang minute - - - Will correlations using SPT data always yield seismic strength only marginally greater than the maximum default value?"

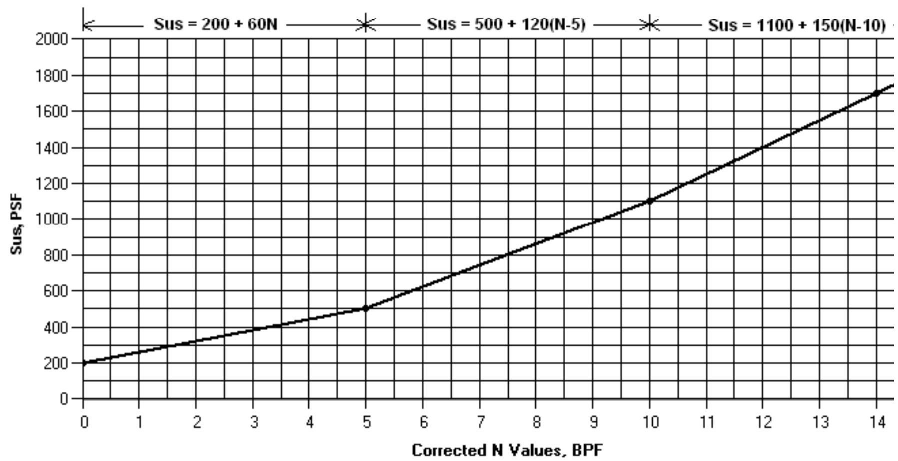
Probably. With the conservative protocol recommended in the manual of correcting for overburden pressure and not fines content, it's difficult to achieve corrected N values greater than 15, which is the level where the manual allows peak strength (i.e. on the order of 0.5 σ'_v) to be used.

"So how can a strength higher than the maximum default value be used?"

Unless you have clay-like or borderline fine refuse where residual vane shear data can be justified -- or N values corrected for overburden pressure and not fines content greater than 15 in sand-like fine refuse --, a more detailed seismic study with data from cone penetrometer testing (CPT) or steady-state lab testing will be needed.

SPT PROTOCOL FROM MSHA DESIGN MANUAL:

Upper-Bound Values of Sus vs. Corrected N Values
(From MSHA Design Manual)



Values of post-earthquake strength for various zones of the embankment should be selected as discussed in the following:

- Dense sand-like materials ($N_{1,60} > 15$ and $q_{t1} > 75$ tsf) such as compacted coarse refuse and dense sand-like natural soils tend to be dilative when they are sheared. That is, the undrained strength tends to be higher than the drained strength. Also, these materials do not experience strength loss due to earthquake shaking. For post-earthquake stability analysis, one cannot be certain whether the material will act as if it is drained or undrained, and the negative pore pressures required to mobilize a higher strength may not develop because they cause cavitation. Therefore, it is reasonable and conservative to use the drained strength for these materials, as discussed in Chapter 6.
- Stiff clay-like materials (SPT N > 6 and CPT data in Zone B) tend to have high shear strain up to the peak undrained strength and limited drop-off in shearing resistance after the peak, so they should not experience significant strength loss due to earthquake shaking. Unlike dense sand-like materials, stiff clay-like materials should act as undrained during the most critical earthquake and post-earthquake period. For these materials, 80 to 100 percent of the peak undrained strength should be used.

TABLE 2. Corrections to SPT (Modified from Skempton 1986) as Listed by Robertson and Wride (1998)

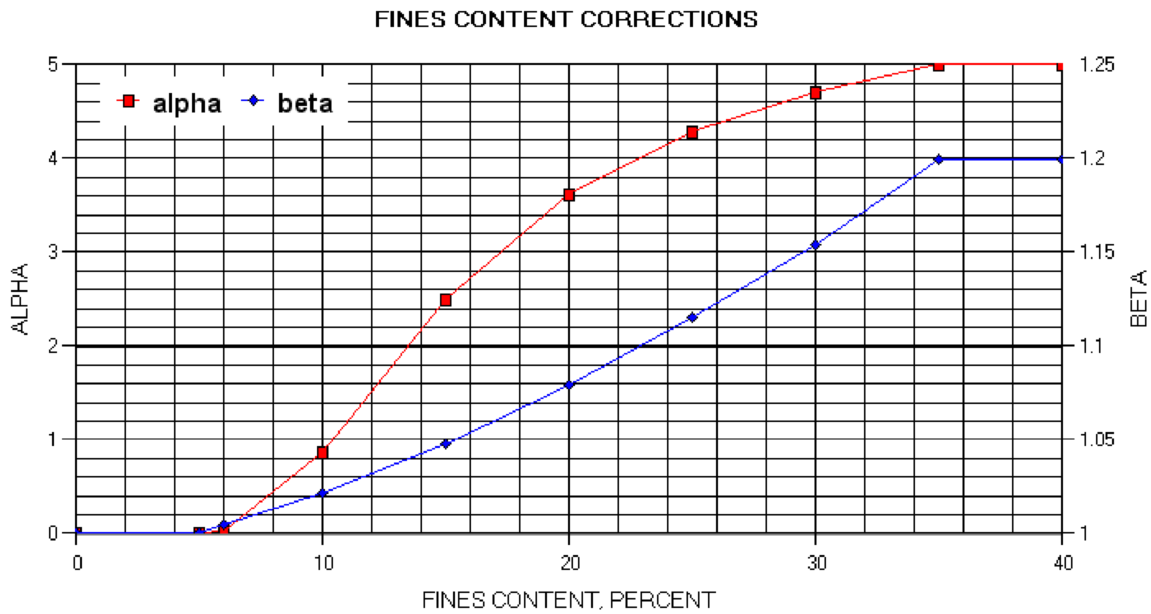
Factor	Equipment variable	Term	Correction
Overburden pressure	—	C_N	$(P_a/\sigma'_{vo})^{0.5}$
Overburden pressure	—	C_N	$C_N \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5–1.0
Energy ratio	Safety hammer	C_E	0.7–1.2
Energy ratio	Automatic-trip Donut-type hammer	C_E	0.8–1.3
Borehole diameter	65–115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	<3 m	C_R	0.75
Rod length	3–4 m	C_R	0.8
Rod length	4–6 m	C_R	0.85
Rod length	6–10 m	C_R	0.95
Rod length	10–30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
Sampling method	Sampler without liners	C_S	1.1–1.3

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \quad (8)$$

where N_m = measured standard penetration resistance; C_N = factor to normalize N_m to a common reference effective overburden stress; C_E = correction for hammer energy ratio (ER); C_B = correction factor for borehole diameter; C_R = correction factor for rod length; and C_S = correction for samplers with or without liners. **Note: Pa = atmospheric pressure = 14.7 psi**

NOTE: When using SPT data, the manual recommends that N values be corrected for overburden pressure and equipment efficiencies, but not fines content, before making correlations with seismic strength.

FINES CORRECTION FOR SPT DATA REFERENCED IN SEISMIC LITERATURE, BUT NOT RECOGNIZED IN THE MSHA DESIGN MANUAL:



NOTE: N corrected for fines content = α + β (N corrected for overburden pressure and equipment efficiencies)

For the past 2 years, I have been correcting SPT data for fines content using references cited in the draft MSHA Manual before correlating with seismic strength and found good agreement with other methods for estimating/measuring seismic strength.

"Based on the relatively low seismic strength estimated from SPT data in the previous examples when fine refuse deposits were consolidated with more than 100 feet of coarse refuse, are restrictions on the use of the fines correction in the manual appropriate? "

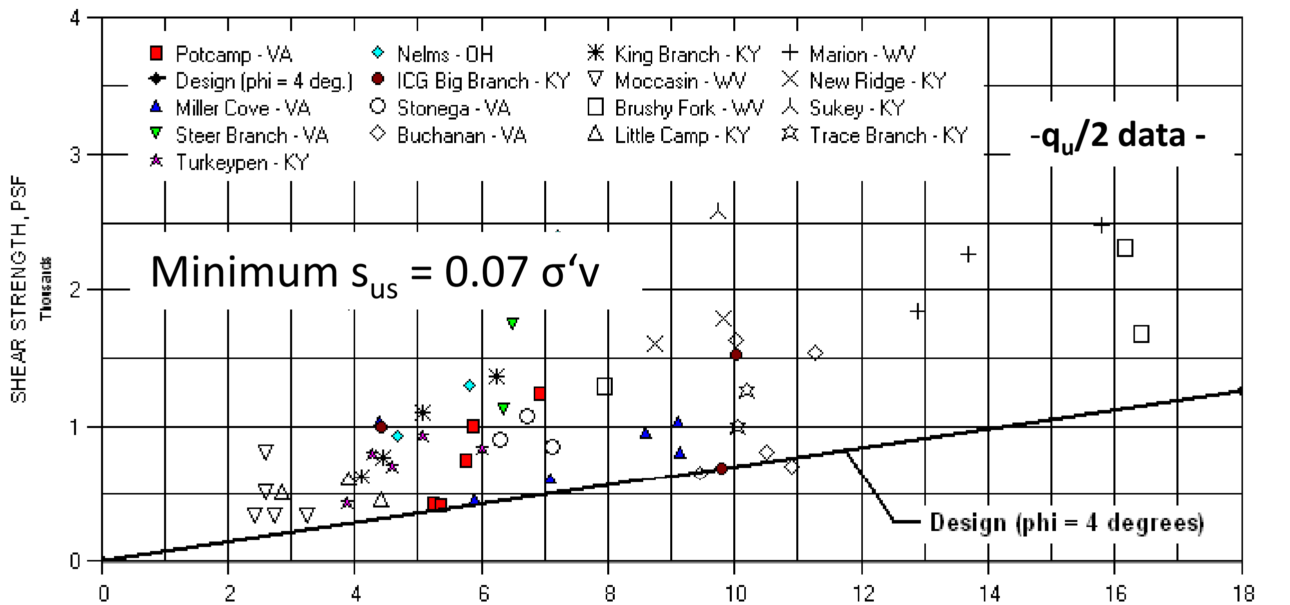
Good question. We'll examine the answer during the case history studies.

Although not recognized in the manual, a supplementary testing method I have found useful is the unconfined compression test.

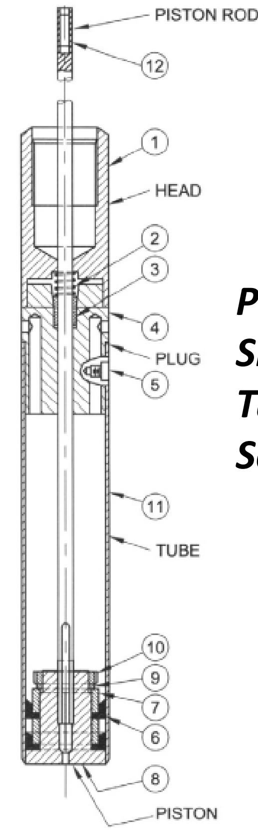
The "worst-case" seismic condition is when earthquake motion causes the pore water pressure to increase to where the effective overburden pressure becomes zero (i.e. the deposit liquefies). The condition of "no confining pressure" can be simulated in the laboratory by an unconfined compression test, which is simple, highly standardized, and the most common method used to measure undrained shear strength for soil in the engineering profession.

Shear strength during such a test is one-half of the measured unconfined compressive strength or $q_u/2$. For cohesionless sand, which is the type of material most susceptible to strength loss during an earthquake, $q_u/2 = 0$. For materials with some cohesion, $q_u/2$ increases as a fine refuse deposit consolidates under load from a coarse refuse embankment or its own weight.

As shown below, the maximum default seismic strength = 0.04 σ'_v is also validated by $q_u/2$ data.



Effective overburden pressure, KSF, at the location where the piston Shelby tube sample was obtained within the fine refuse deposit for laboratory $q_u/2$ testing

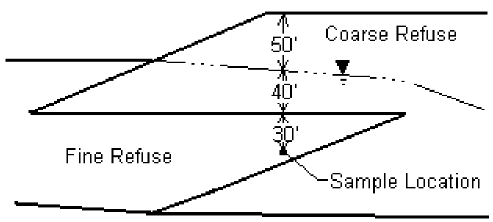


Piston Shelby Tube Sampler

"Let me get this straight. SPT and q_u/2 testing let you see the sample, which is one reason why these are the most commonly used testing methods for soil in Appalachia. Why are they red-headed step-younguns in the new manual? You need to get my Cousin Velmer workin' on the answer to that question."

As noted previously, Tiny's Cousin Velma is a simplified seismic testing expert. According to Tiny, she is a first cousin on his father's side of the family and a second cousin on his mother's side of the family. Tiny says she has an uncanny ability to get to the bottom of puzzling questions such as this. Velma tells it like it is, so don't be shocked with what she uncovers.

"Let's see if I can simplify this for you…. Soil and waste particles are as dumb as a rock. Water, on the other hand, is a genius and never makes a mistake. The shear strength of soil or waste particles is based on the principle of effective stresses; the shear strength is proportional to the effective overburden pressure the material feels, which is the total overburden pressure minus the water pressure.



In the previous example problem, let's say the fine refuse sample has an effective (phi) angle of internal friction = 32 degrees. If the upstream construction was performed slowly, such that construction pore water pressures dissipated, then the fine refuse particles feel the entire effective overburden pressure of 9332 psf and it's shear strength = σ'_v tan 32^0 = 9332 psf * 0.62 = 5800 psf.

*If the pushout is built quickly, then construction pore water pressures will develop, which will reduce the effective overburden pressure the fine refuse sample feels. For an excess construction pore water pressure = 5600 psf, as can be measured by pneumatic piezometers, the sample will feel only 9332 psf – 5600 psf = 3732 psf; thus, the shear strength = 3732 psf * tan 32^0 = 2332 psf.*

*Earthquake motion can also cause the pore water pressure to increase, which reduces the effective stress and thus the shear strength of the in-situ fine refuse. If the fine refuse is normally consolidated (i.e. construction pore water pressures have dissipated) and an earthquake induces a pore pressure ratio = 0.6, then the pore pressure increase during the earthquake = 9332 psf * 0.6 = 5600 psf. As shown above, the post-earthquake shear strength = 2332 psf due to pore water pressures induced by the earthquake."*

“Yea, but what happens if an earthquake occurs before construction pore water pressures have dissipated?”

“That’s when trying to predict the shear strength of an in-situ fine refuse deposit becomes extremely difficult, which explains why the seismic design aspects of the MSHA Manual are so complicated. Residual vane shear testing induces a pore water pressure to simulate earthquake loading effects, so the resulting shear strength can be measured. When performed in-situ, the residual vane shear results also account for construction pore water pressures that may be present.”

“So how is a residual shear strength estimated using SPT data and the Liquidity Index?”

“Seismic strength correlations are provided in the MSHA Manual for SPT data and the Liquidity Index based on published relationships. No coal refuse disposal facilities have ever failed due to earthquake loading, whereas failures of natural sand and hard-rock tailings deposits have occurred. Therefore, the published SPT relationships are primarily for non-plastic sands with a specific gravity on the order of 2.6 to 2.7, whereas fine refuse has a much lower specific gravity with varying levels of plasticity.”

According to the manual, if the Liquidity Index (LI) ≥ 1 (i.e. the sample has a natural moisture content greater than the liquid limit), then the minimum default seismic strength = 20 psf, but no higher than 0.04 σ'_v, if justified by other testing methods. If LI < 1, seismic strength = 0.04 σ'_v can be justified without testing by other methods. The definitions and published relationships shown below are for natural clays, which typically have specific gravities on the order of 2.6 to 2.7.

For fine refuse deposits, the liquid limit, plastic limit, and natural moisture content can change significantly over short distances due in large part to differences in specific gravity, which can vary between 1.6 and 2.5 within the same sample."

$$I_L = \frac{w - w_P}{w_L - w_P} = \frac{w - w_P}{I_P}$$

Soil-moisture Scale		Physical State (Remolded)	Consistency (Remolded)
		Liquid	Very soft
w_L	Liquid limit		Soft
I_P	Plasticity index, plastic range	Semisolid	Stiff
w_P	Plastic limit		Very stiff
w_S	Shrinkage limit		Extremely stiff
		Solid	Hard
	Air-dry / Oven-dry: Hygroscopic moisture		

FIGURE 1.11. Diagram of the soil-moisture scale showing Atterberg Limits, corresponding physical state, and approximate consistency of remolded soil.

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Table 45.2
Relation of Consistency of Clay, Number of Blows N on Sampling Spoon, and Unconfined Compressive Strength

	q_u in tons/ft²					
Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
N	<2	2–4	4–8	8–15	15–30	>30
q_u	<0.25	0.25–0.50	0.50–1.00	1.00–2.00	2.00–4.00	>4.00

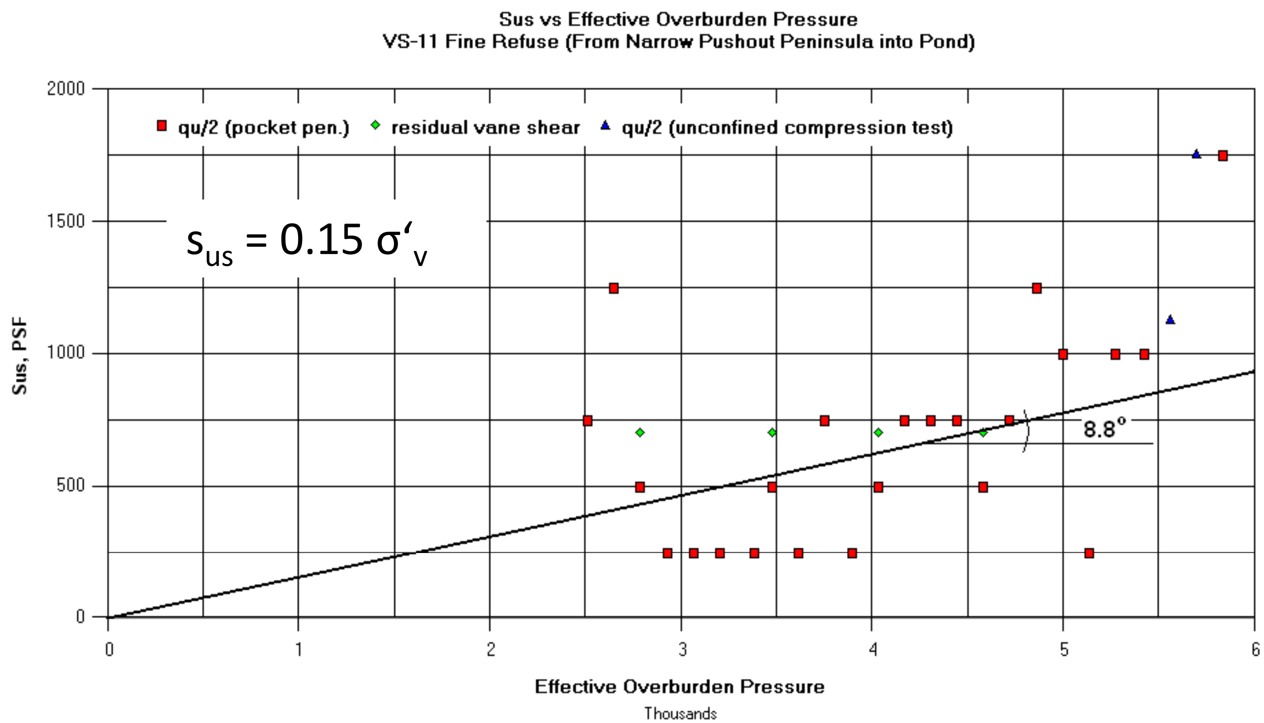
(After Terzaghi and Peck, 1967)

"While you review some site-specific data and case histories, I'll be examining Cousin Tiny's question regarding the role of SPT and qu/2 data in the MSHA Manual."

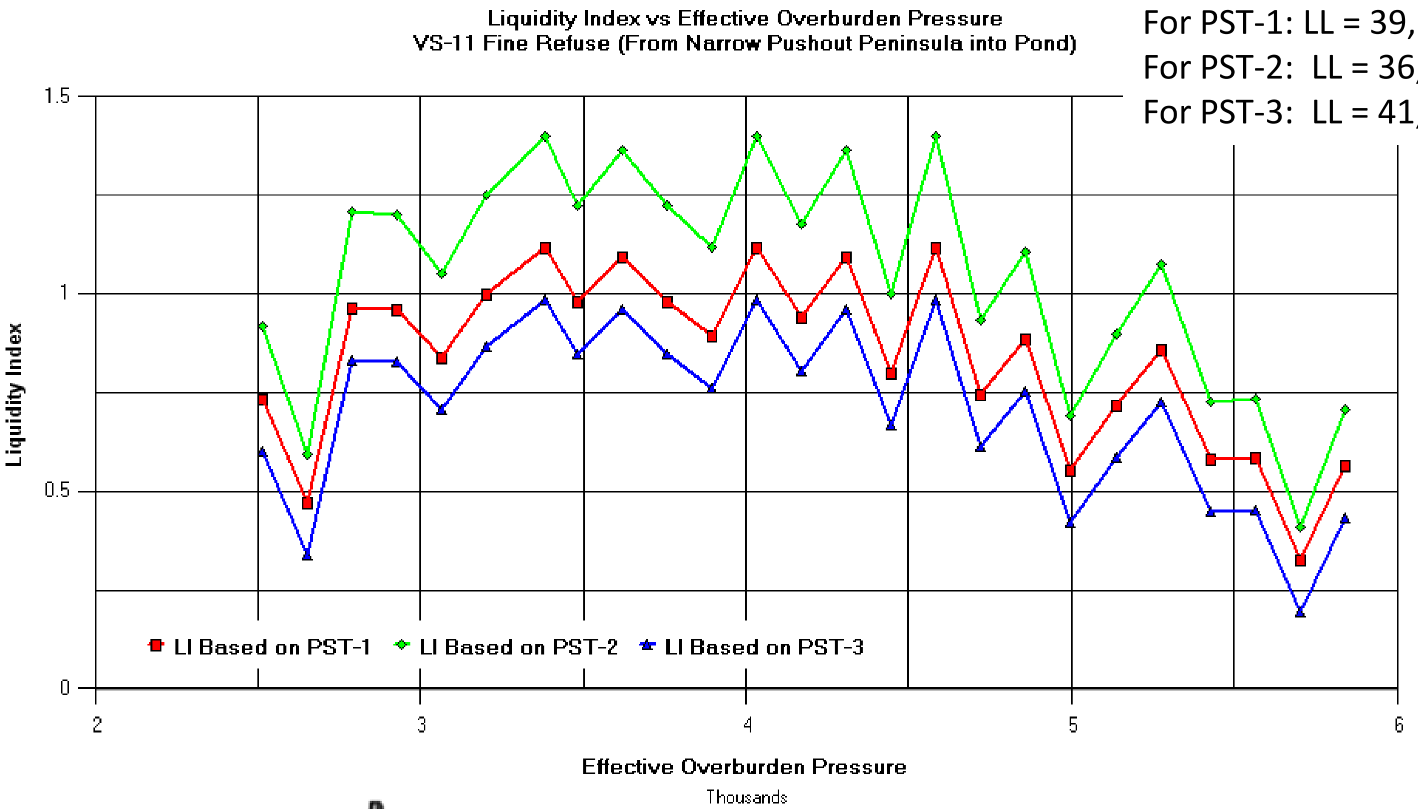
Recognizing that specific gravity can vary within an individual sample, and because seismic shear strength is the property of interest, I previously used a pocket penetrometer to estimate $q_u/2$ for each fine refuse sample obtained during drilling. I believe that the shear strength at no confining pressure (i.e. $q_u/2$) is a conservative estimate of s_{us} at the effective overburden pressure that exists where the particular sample was obtained based on comparison with residual vane shear testing, even though it has yet to be recognized in the manual.

For illustration purposes, a boring was drilled from a narrow pushout made into the center of the pond at Case History VS-11 in 2001. Not knowing at the time the importance of index testing in the new manual, moisture contents were measured for each SPT sample, but index testing was performed on only the undisturbed piston Shelby tube samples.

A pocket penetrometer was used as an indicator to estimate $q_u/2$ of each SPT sample to supplement residual vane shear and laboratory $q_u/2$ testing as shown at right. In this example, $q_u/2$ values obtained using a pocket penetrometer provide a slightly conservative (i.e. lower) estimate of s_{us} as compared to the results of residual vane shear testing.

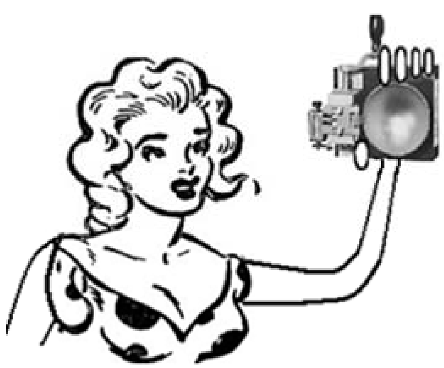


According to the new manual, liquidity index (which sounds a lot more sophisticated than $q_u/2$ data obtained with a pocket penetrometer) is recommended as a primary indicator.



Note:
For PST-1: LL = 39, PL = 24, PI = 15, LI = 0.6
For PST-2: LL = 36, PL = 24, PI = 12, LI = 0.7
For PST-3: LL = 41, PL = 26, PI = 15, LI = 0.2

As shown above, all the piston Shelby tube samples at VS-11 had LI <1; but as shown at left, you get completely different results, depending on which of those index test results you use to calculate LI for individual SPT samples.



"Based on its importance according to the manual, I recommend performing index testing on every sample where a natural moisture content is determined to avoid the confusion shown in the graph above.

"Let's perform index tests and q_u/2 tests with a pocket penetrometer on fine refuse samples so you can 'get your hands dirty' on the topic."

- By the way, can you guess which vehicle Bubba drives? -