

Simplified Seismic Design of Coal Refuse Disposal Facilities

By Barry Thacker, P.E.

INTRODUCTION: The updated *MSHA Engineering and Design Manual for Coal Refuse Disposal Facilities* contains new seismic design standards. The first step in the evaluation process is a three-page screening flow chart, which is used to assess if a facility can be designed using conservative default values or if a detailed seismic analysis is required, as shown below.

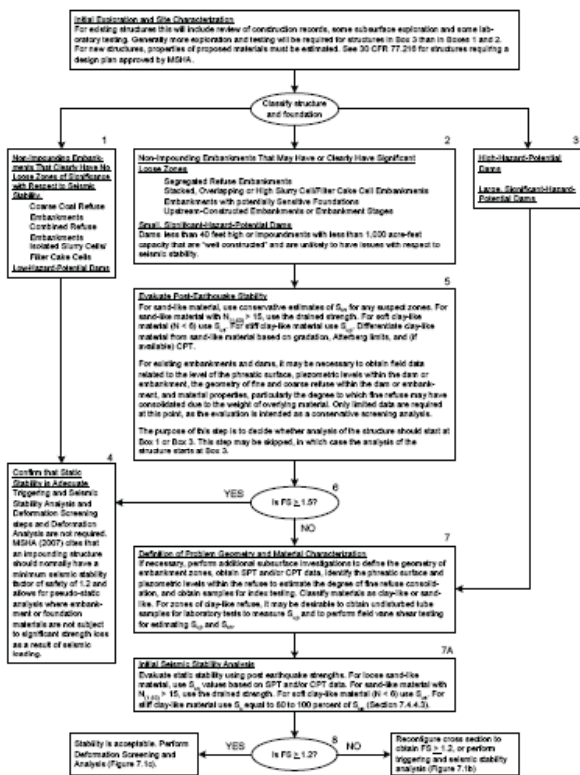


FIGURE 7.1a SEISMIC STABILITY SCREENING

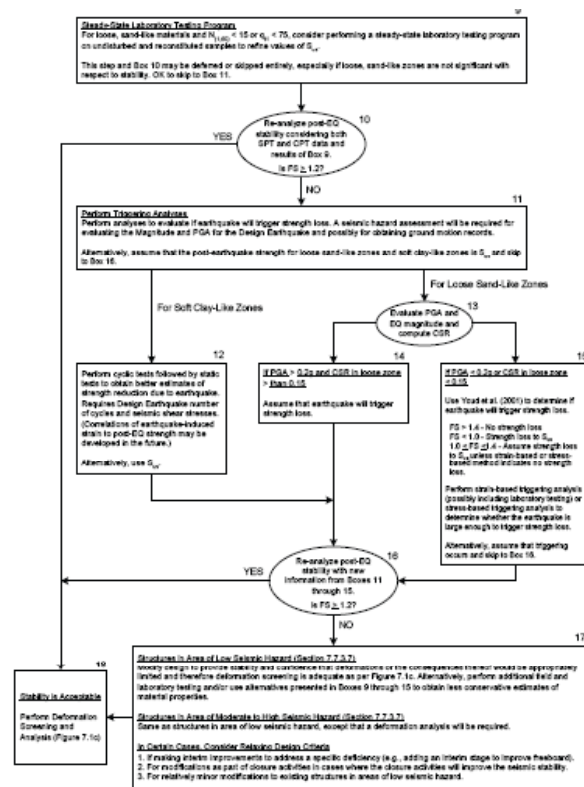


FIGURE 7.1b TRIGGERING AND SEISMIC-STABILITY ANALYSIS

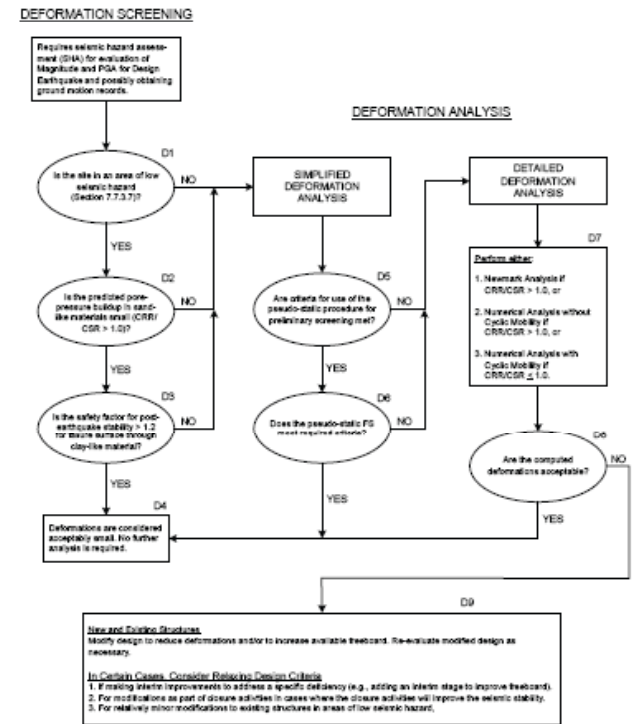
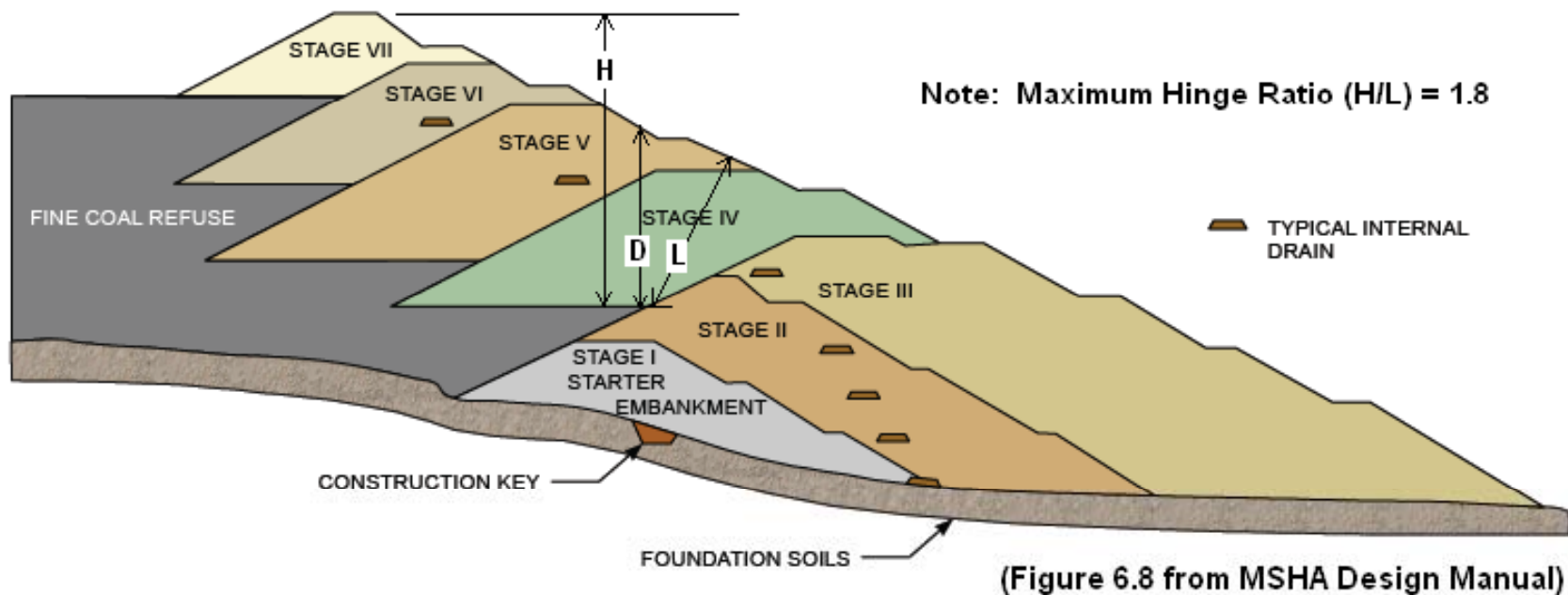


FIGURE 7.1c DEFORMATION SCREENING AND ANALYSIS

A simplified alternative to MSHA's screening flow chart is offered, which is based on 20 years of application at numerous facilities located throughout the Appalachian coal fields. I call it ...

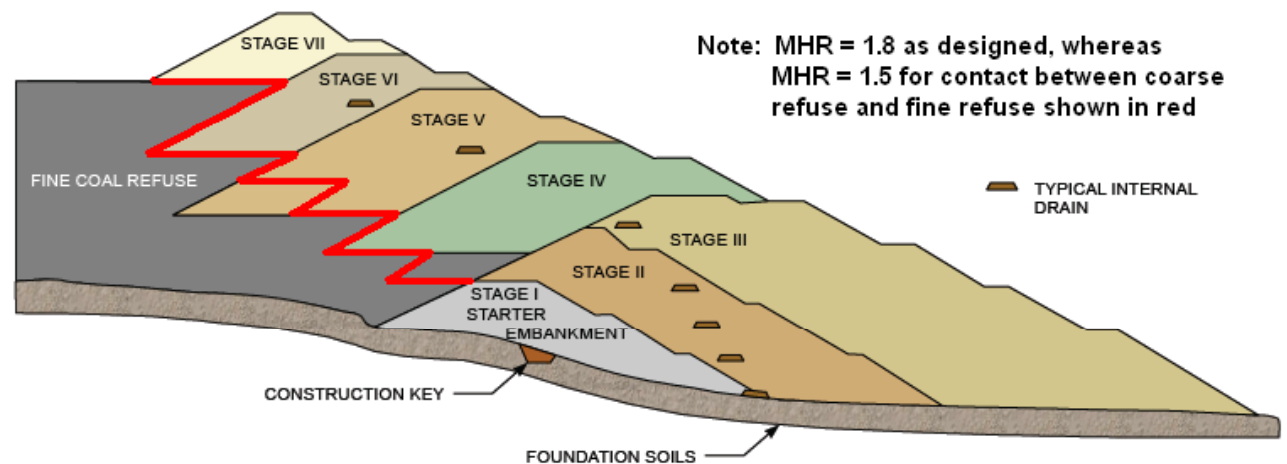
Old Indian Trick No. 1: Screen the hinge. This simplified seismic screening approach is based on the hinge ratio, defined as the elevation difference between a hinge and the crest (H) divided by the minimum lateral distance to the outslope (L). For Figure 6.8 shown below from the MSHA Manual, the Maximum Hinge Ratio (MHR) = 1.8.



Another important screening indicator is the minimum depth of cover at the hinge (D). A thicker cover results in more consolidation of the underlying fine refuse, provided excess construction pore water pressures have dissipated.

For the past 20 years, I have used a simplified design method with a default strength of “residual $\phi = 4$ degrees” for fine refuse and a minimum factor of safety = 1.0 in post-earthquake static analysis. With such an approach, I found that facilities needed to maintain MHR less than about 1.7; this was achieved by starting upstream construction sooner, rather than later, using a modified upstream construction method. For the example shown below, modified upstream construction would need to start after Stage II rather than after Stage III.

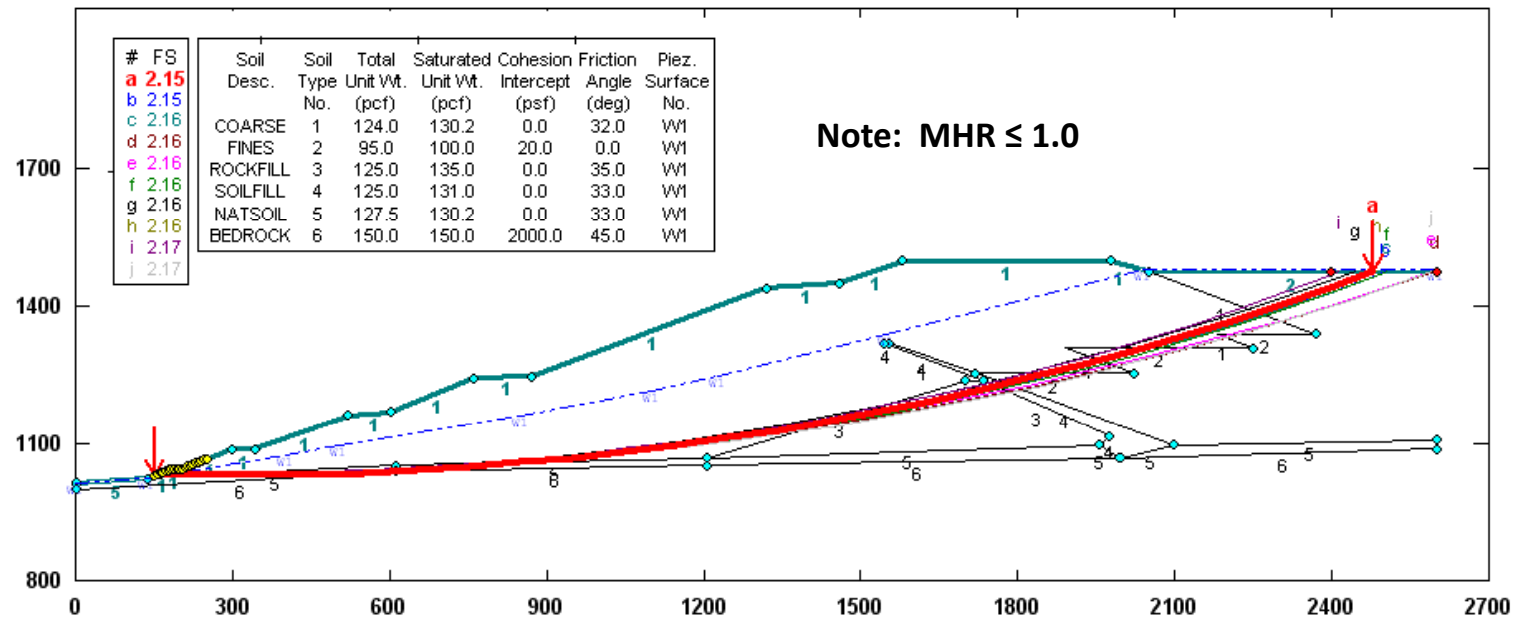
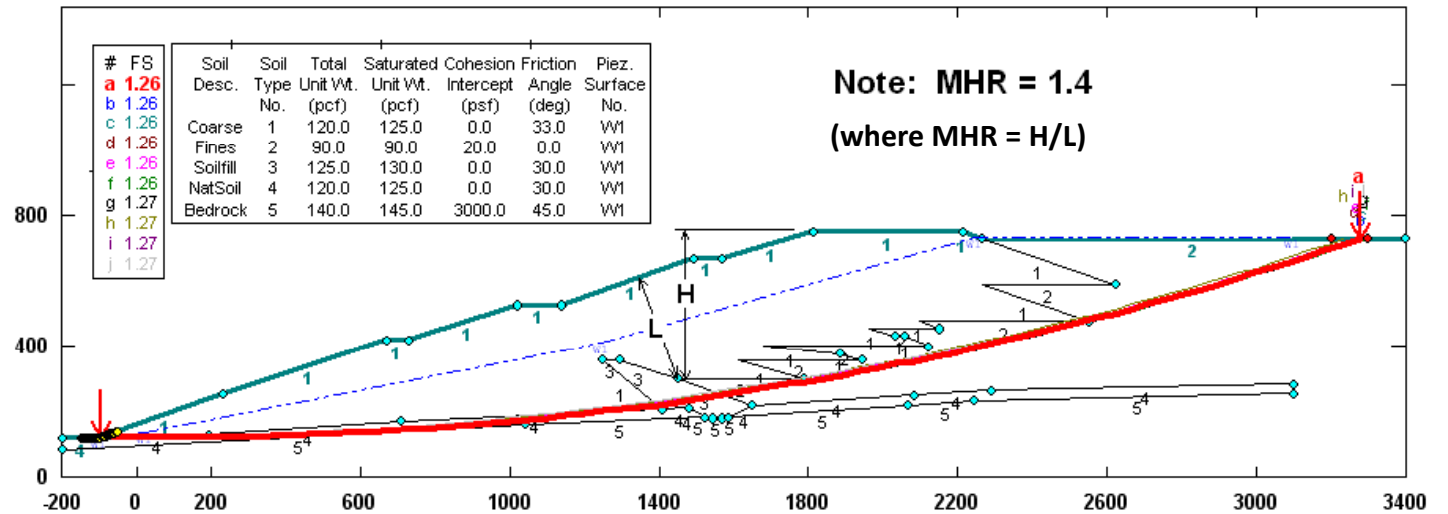
Even the minor change in the contact between coarse refuse and fine refuse provided using the modified upstream construction method (shown in the example at right) can have a major impact on MHR and a designer’s chances of being able to use a simplified seismic design method.



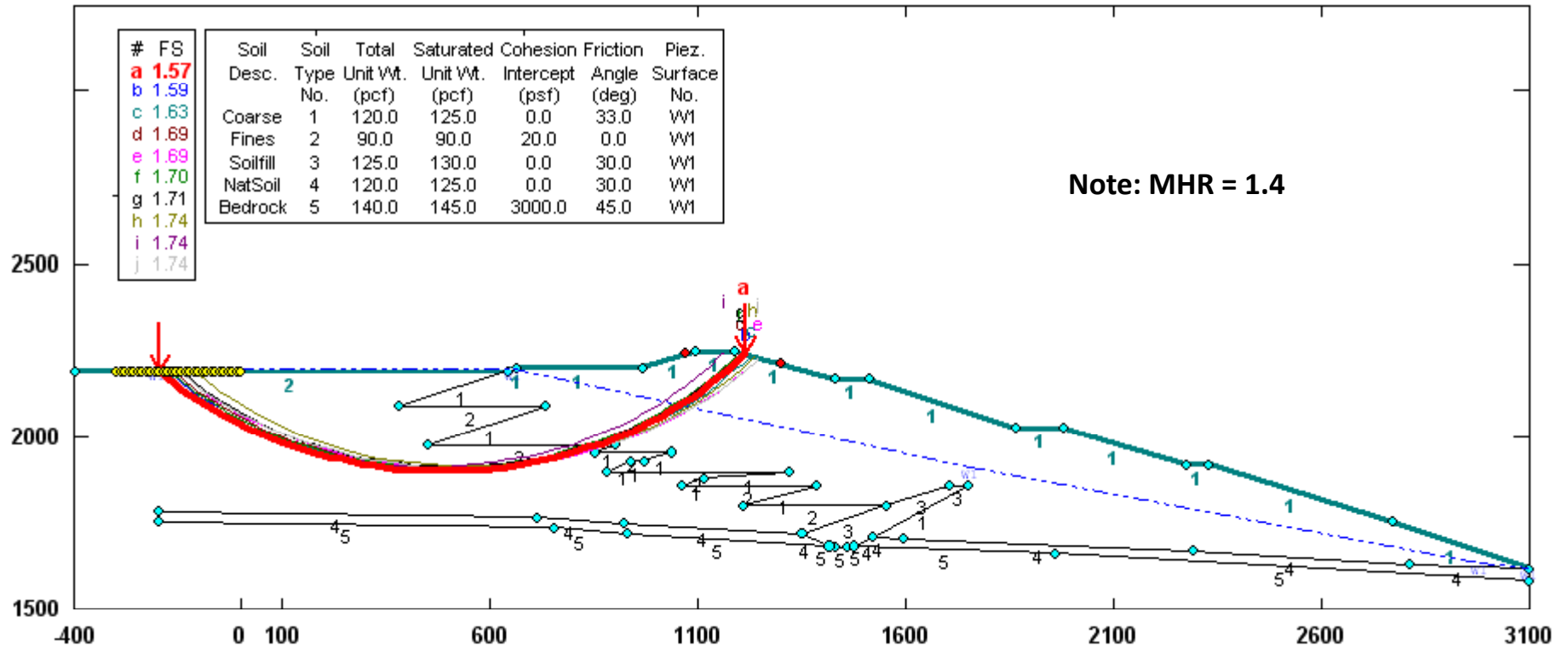
Sites with a higher hinge ratio and thinner cover are more susceptible to being impacted by high pore water pressures at the hinge than sites with a lower hinge ratio and thicker cover. High pore water pressures can be caused by rapid construction, lack of internal drains, and/or from earthquake motions, so providing a lower MHR and thicker cover are beneficial for all types of loading conditions.

The new MSHA Manual specifies a conservative default value for post-earthquake strength equal to 20 psf. The minimum factor of safety for seismic design is 1.2. *“At what MHR value can a facility likely be designed by the simplified method based on a post-earthquake strength = 20 psf and a minimum factor of safety = 1.2?”*

Case histories will be presented to illustrate the range of MHR values where stability can likely be verified in a downstream direction using a post-earthquake strength of 20 psf.



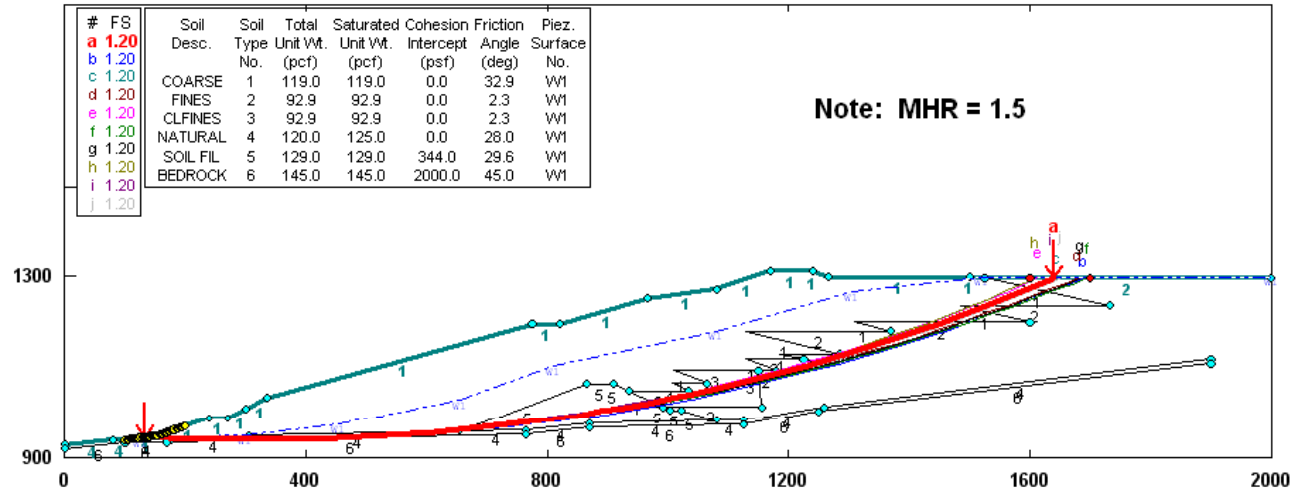
Furthermore, if a facility is appropriate for design using the simplified method in a downstream direction, Old Indian Tricks (OITs) will be presented which might enable a designer and operator to maintain stability in an upstream direction using the default standards stipulated in the MSHA Manual.



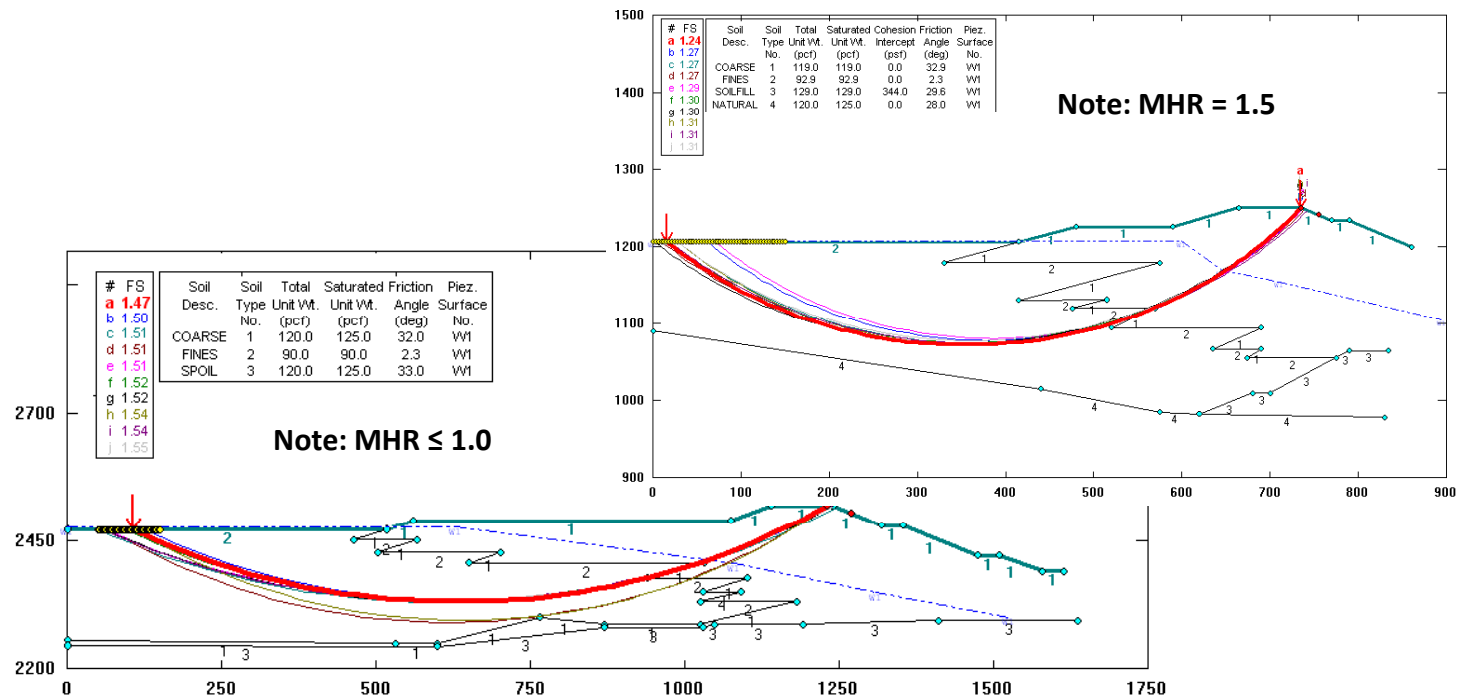
Even using multiple OITs, upstream stability is difficult to verify with a post-earthquake strength = 20 psf unless a consistently thick coarse refuse zone can be maintained over the fine refuse and the elevation of the fine refuse is kept relatively close to the elevation of the crest at all stages of construction.

If you can justify the slightly higher post-earthquake strength of $0.04 \sigma'_v$, your chances of using a simplified design

- increase some for verifying stability in a downstream direction,



- and increase substantially for verifying stability in an upstream direction.



The manual allows a default post-earthquake strength as high as $0.04 \sigma'_v$ to be justified using residual vane shear and/or Standard Penetration Test (SPT) data as shown in the table below.

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)
Aterberg limits	$PI \leq 7$	$PI \geq 10$	$7 < PI < 10$	$7 < PI < 10$
% passing No. 40 sieve		≥ 35		≥ 35
% passing No. 200 sieve		≥ 20		≥ 20
Triaxial tests on undisturbed 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	$0.04 \sigma'_v$, if non-plastic, or if $LI \leq 1.0$	$0.04 \sigma'_v$, if $LI \leq 1.0$	$0.04 \sigma'_v$, if $LI \leq 1.0$	$0.04 \sigma'_v$, if $LI \leq 1.0$
	20 psf if $LI \geq 1.0$, but no higher than $0.04 \sigma'_v$	20 psf if $LI \geq 1.0$, but no higher than $0.04 \sigma'_v$	20 psf if $LI \geq 1.0$, but no higher than $0.04 \sigma'_v$	20 psf if $LI \geq 1.0$, but no higher than $0.04 \sigma'_v$
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_{uh}	Not Applicable	Applicable	Potentially Applicable if it can be demonstrated that the test is undrained (Section 6.4.3.8)	Potentially Applicable if it can be demonstrated that the test is undrained (Section 6.4.3.8)
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_{uh}	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 -

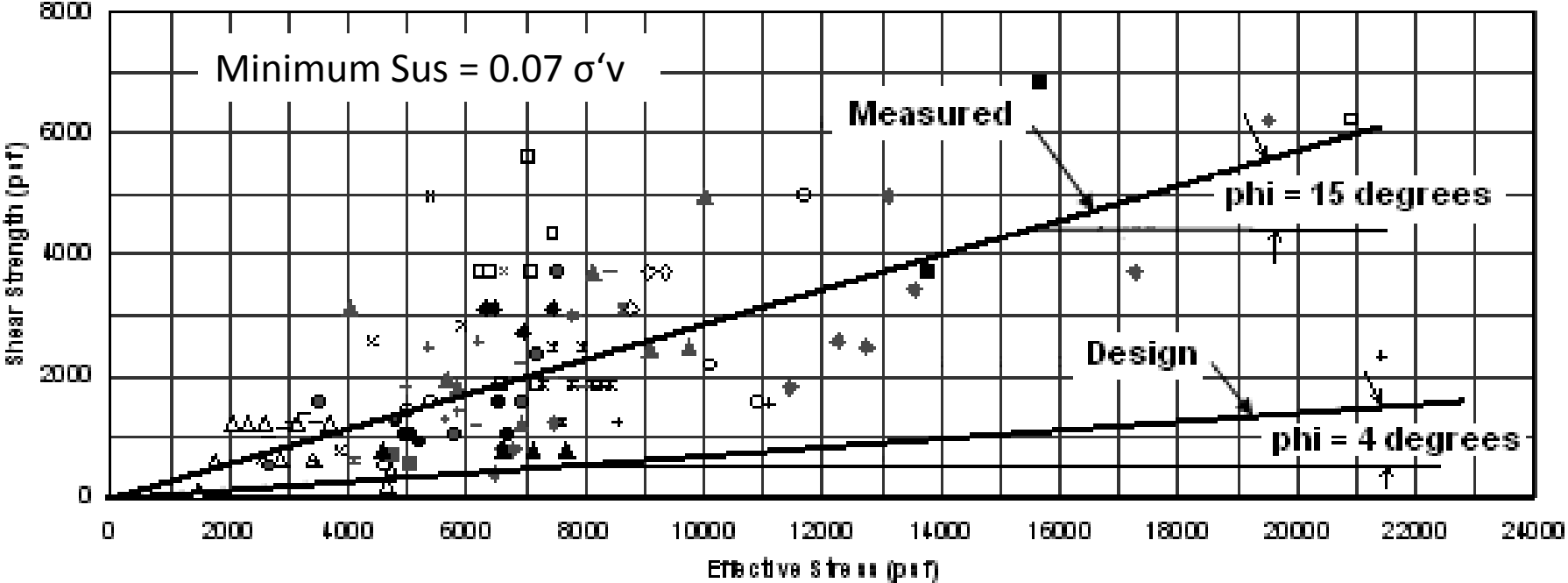
Question: So what is $0.04 \sigma'_v$?

Answer: The allowable post-earthquake strength = 0.04 times the effective overburden stress.

Question: What are my chances of being able to justify a post-earthquake strength of $0.04 \sigma'_v$ 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 modified upstream construction method), your chances are excellent.

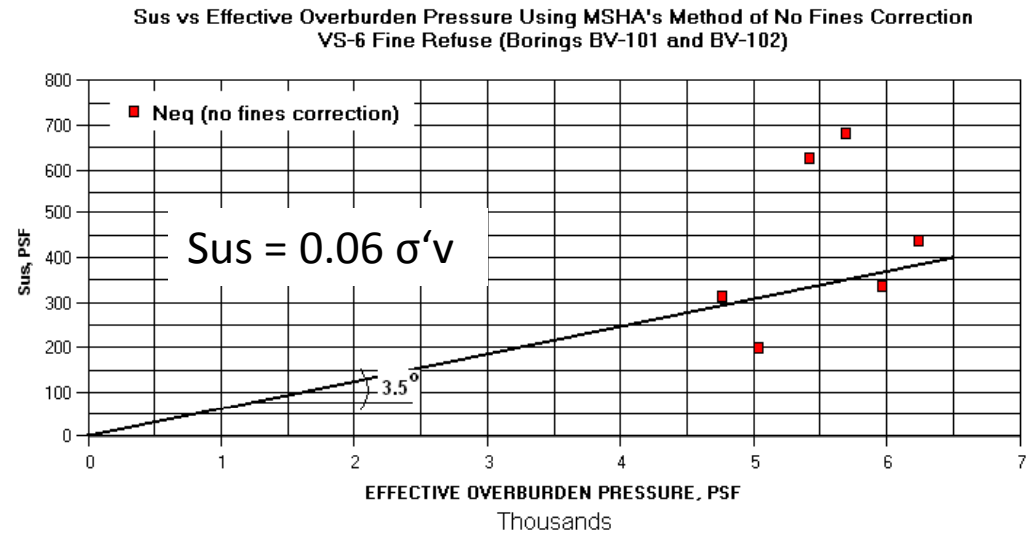
For facilities designed and built by the modified upstream construction method, I have yet to find a post-earthquake strength in fine refuse less than $0.04 \sigma'_v$ from residual vane shear testing as shown below.



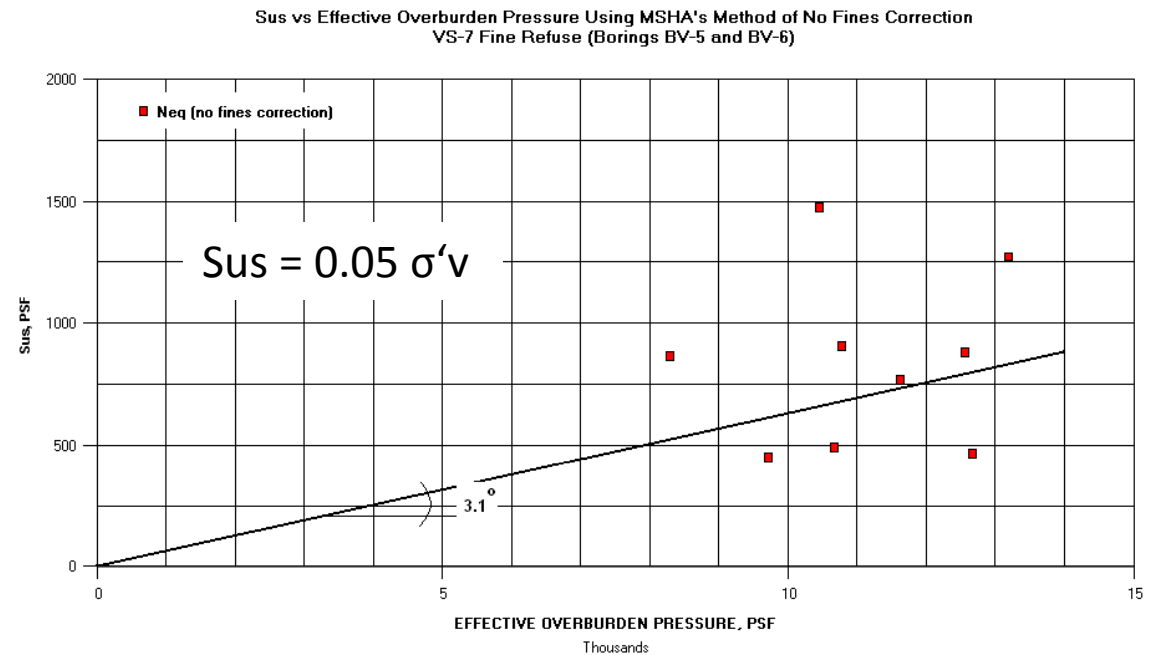
◆ ABNER FORK (KY)	▲ BEAR TREE (KY)	× ICG BIG BRANCH (KY)	± BRUSHY FORK (VA)
△ HARPER BRANCH (VA)	● WL. FORK HARPER'S BRANCH (VA)	□ JAKEGO RE (VA)	- KING BRANCH (KY)
= LONG FORK (KY)	■ MARION BRANCH (KY)	● MILLER COVE (VA)	- MOCCASIN HOLLOW (VA)
+ MOSS NO. 1 (VA)	+ POTCAMP FORK (VA)	○ SALLIES BRANCH (VA)	■ SCOTT'S BRANCH (KY)
◇ TRACIE BRANCH (VA)	× SIDNEY (KY)	▲ STEER BRANCH (VA)	◆ STONEGATE (VA)
Envelope points		— Linear (Envelope points)	

Similarly, I have yet to find less than $0.04 \sigma'_v$ based on SPT data even using the conservative correlations included in the manual, regardless of whether the fine refuse deposit was tested...

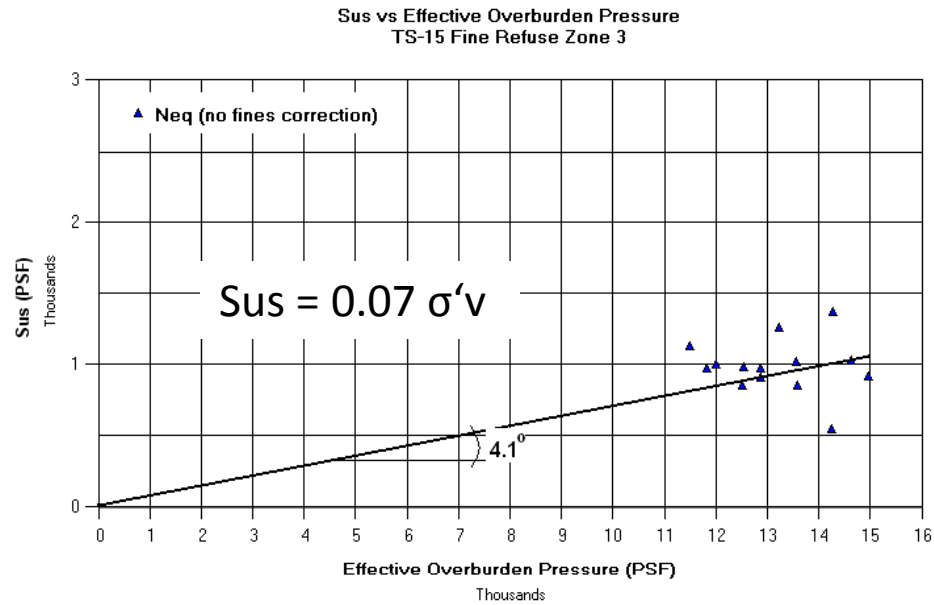
- immediately after installation of a pushout (i.e. with 40 feet of coarse refuse cover,



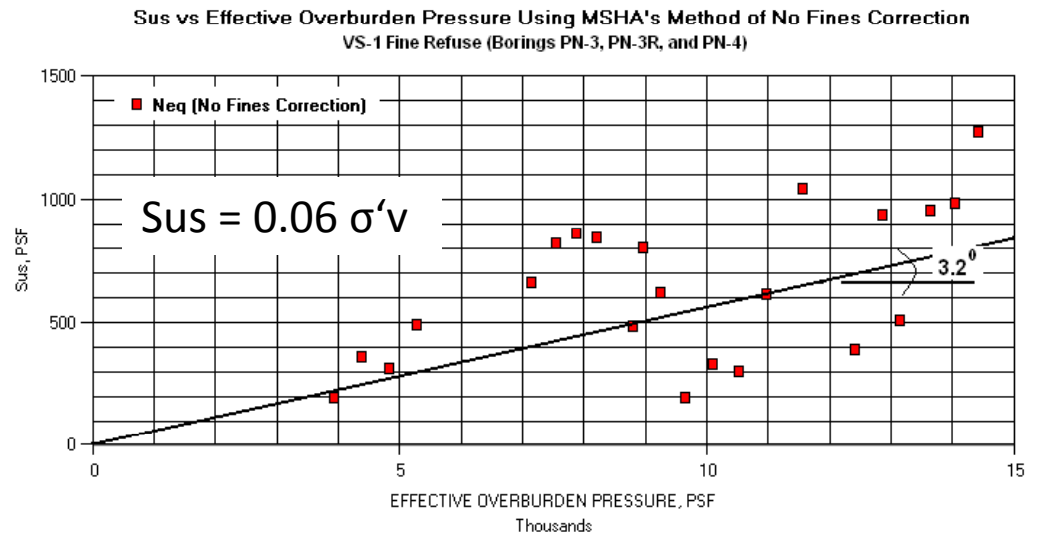
- after completion of the first upstream stage (i.e. with 80 feet of coarse refuse cover),



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

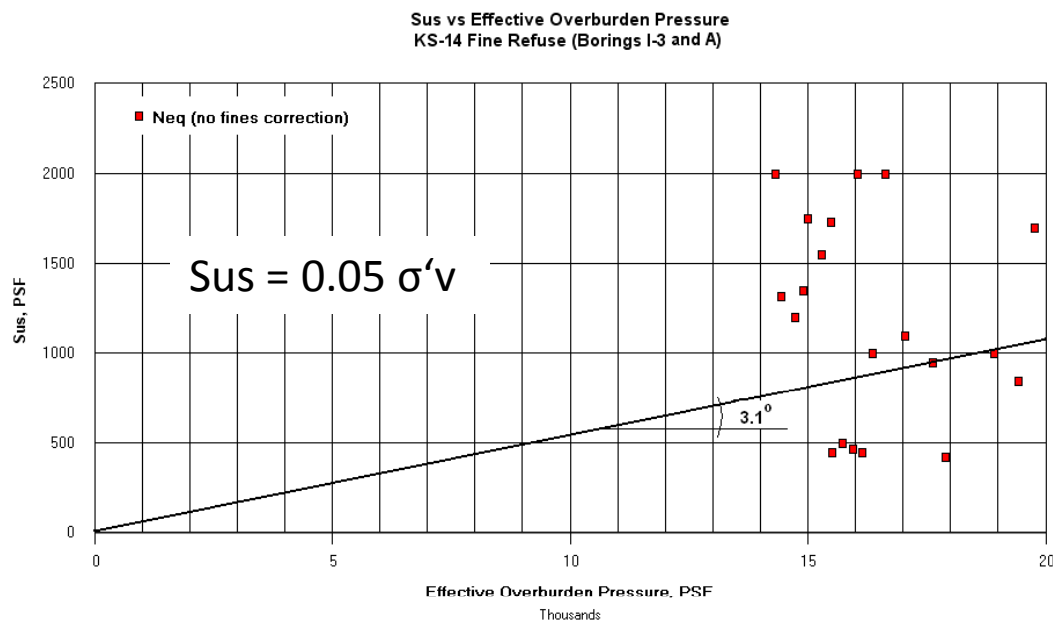


- after multiple drilling and testing events (i.e. when the coarse refuse cover was 40 feet, 80 feet, and then 120 feet thick above the fine refuse deposit),



- 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 $0.05 \sigma'v$ and $0.1 \sigma'v$ in fine refuse deposits using the conservative correlations included in the manual.



Wait just a dang minute - - - Will correlations using SPT data always yield seismic strength only marginally larger 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 to be used in a simplified analysis.

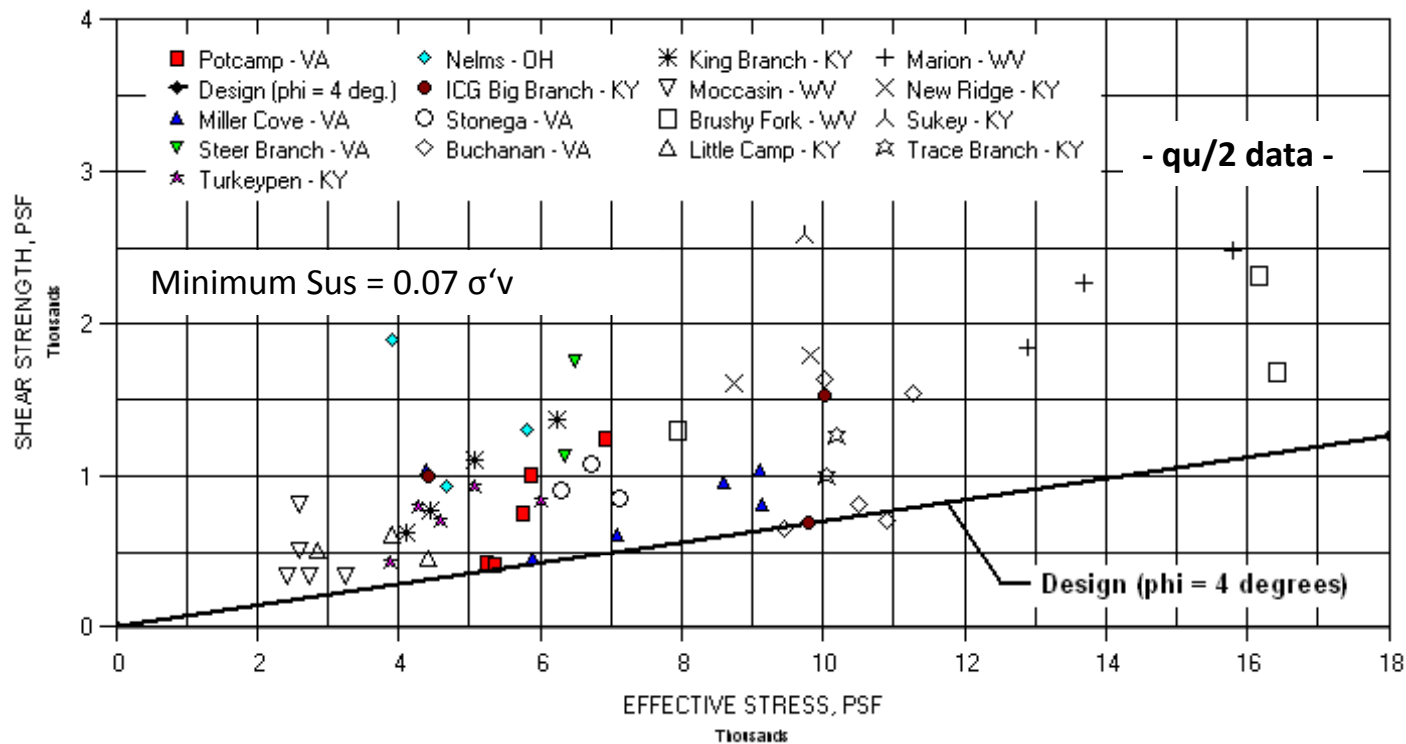
So how can a strength higher than the maximum default value be used?

Unless you have clay-like or borderline fine refuse where residual vane shear methods 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 other than SPT sources will be needed.

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 shaking 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 and highly standardized. Shear strength during such a test is one-half of the measured unconfined compressive strength or $qu/2$. For cohesionless sands, $qu/2 = 0$. For materials with some cohesion, $qu/2$ increases as a fine refuse deposit consolidates under load from a coarse refuse embankment.

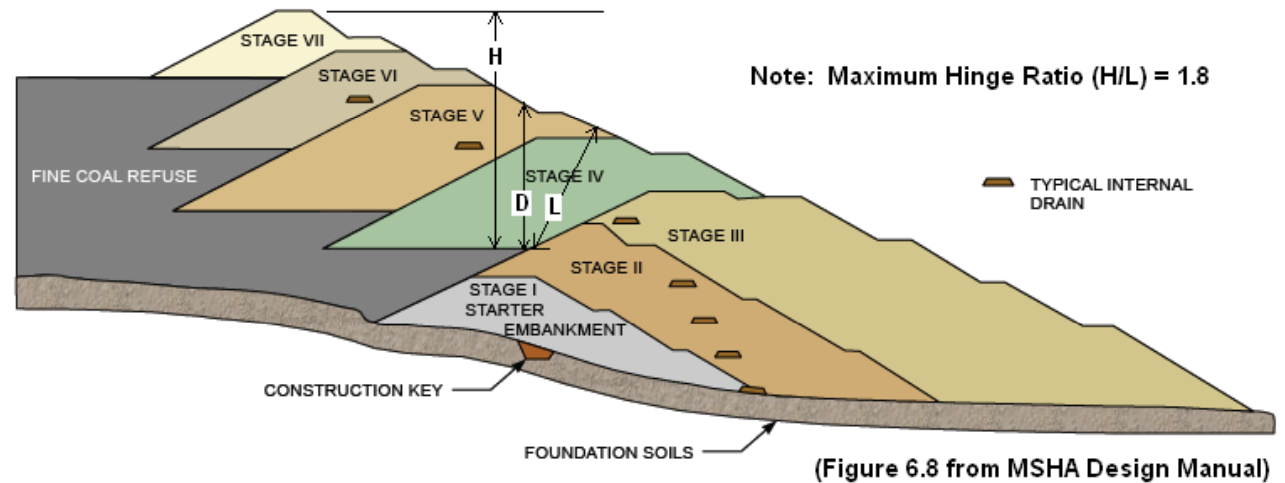
As shown below, the default seismic strength of $0.04 \sigma'v$ is also validated by $qu/2$ data.



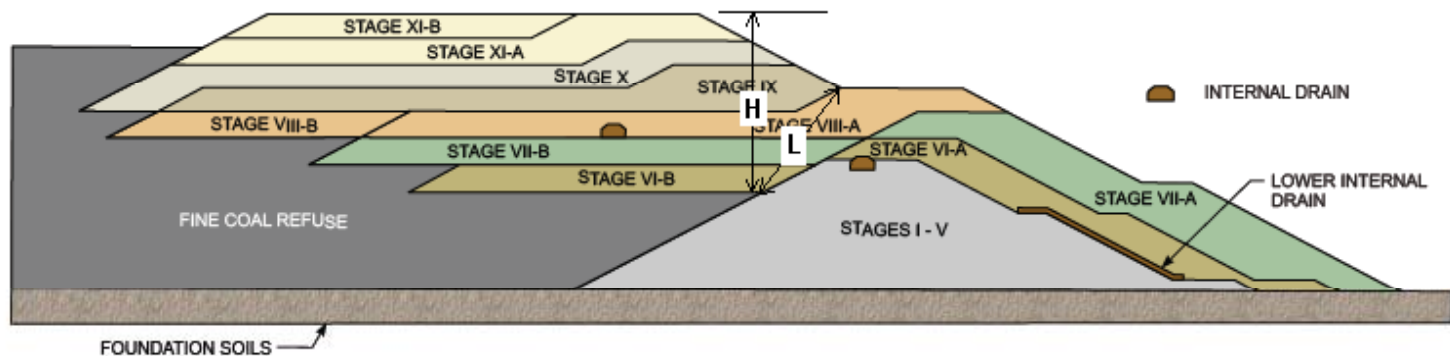
Note: Effective Stress = effective overburden pressure at the location where the piston Shelby tube sample was obtained within the fine refuse deposit for laboratory qu testing.

From what you will learn at the seminar, you will be able to recognize which of these two design profiles will likely have adequate stability in the downstream direction based on the minimum default parameters contained in the new MSHA Manual and which one will require more extensive site-specific study.

Advantages of using a conservative, simplified approach will become apparent in terms of assessing for both seismic stability and seismic deformation.

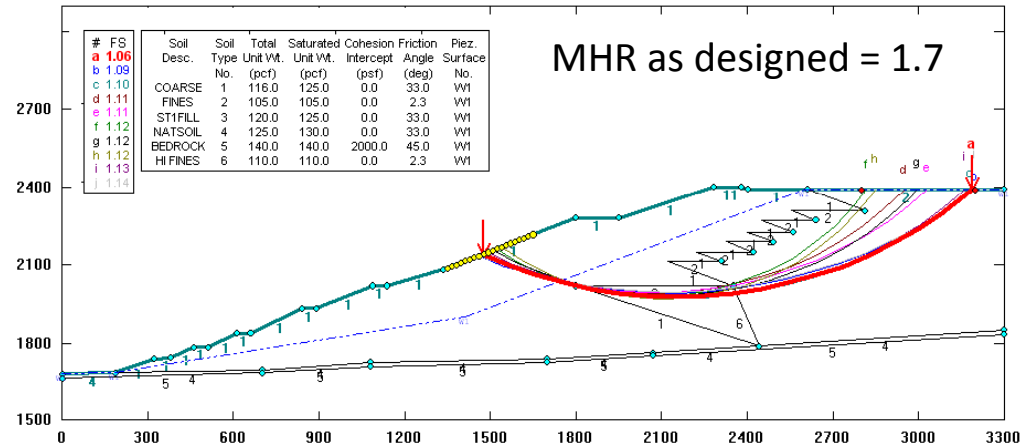


Maximum Hinge Ratio (H/L) = 1.3

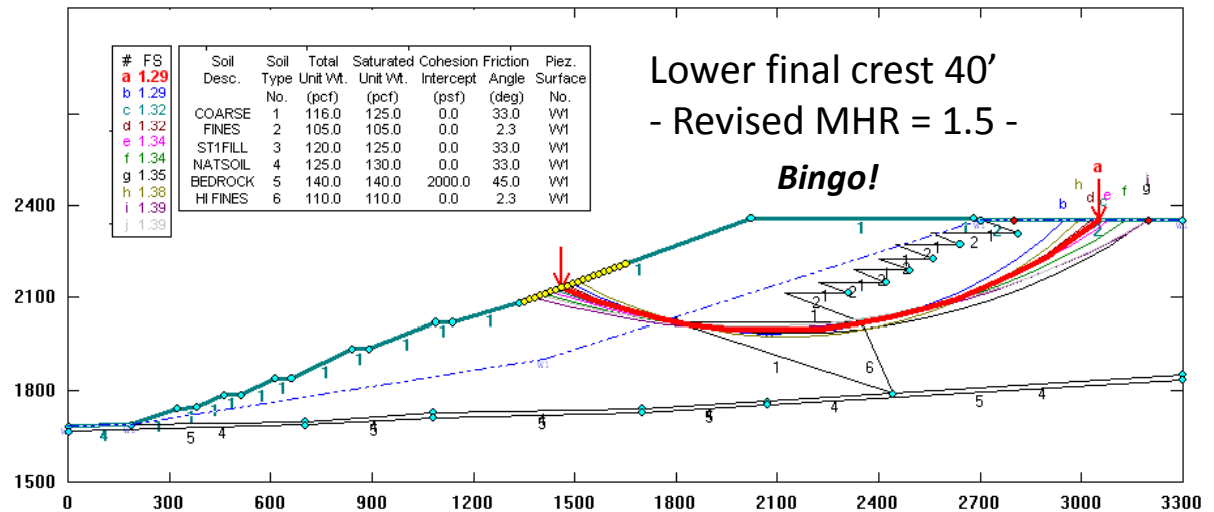


(Figure 6.9a from MSHA Design Manual)

Using the power of MHR, you will be able to perform preliminary seismic screening for any facility, provided you know the geometry and contact locations between coarse refuse and fine refuse.



“MHR is a valuable tool you can use throughout the planning, design, and construction process.”



Finally, If you are not completely satisfied with what you learn at this seminar, your registration fee will be refunded!

