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ON THE USE OF EMPIRICAL CORRELATIONS FOR ESTIMATING THE RESIDUAL UNDRAINED SHEAR STRENGTH OF LIQUEFIED SOILS IN DAM FOUNDATIONS

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ABSTRACT

Current practice (2009) for seismic analysis of embankment dams relies heavily on empirical correlations with penetration resistance (standard penetration test or cone penetration test) to predict the residual undrained shear strength of liquefied foundation materials. At least six such relationships have been published for the SPT alone, in different “formats.” Some apply a fines adjustment to the SPT blowcounts, but others do not; some express the predicted strength as a ratio with pre-earthquake effective overburden stress, whereas others predict it directly, without explicit consideration of overburden. For the foundations of embankment dams, the difference between the strength-ratio approach and prediction of S_{ur} directly, from the SPT alone, can be important. In this paper, the underlying assumptions and data are reviewed critically, including the effects of different material types and different mechanisms governing the strength. Simplified statistical analyses were applied in attempt to determine the most appropriate format for a correlation and to obtain a new correlation that explicitly accounts for both overburden and blowcount.

INTRODUCTION

STATEMENT OF PROBLEM

Analysis of earthquake-induced deformations or post-earthquake stability of embankment dams often requires an estimate of the residual undrained shear strength, S_{ur}, of liquefied foundation or embankment materials. More precisely, what is needed is the amount of shearing resistance that can be mobilized without strains or deformations that impair the safety of the structure. In the case of an embankment dam with generous freeboard, displacements of a meter or even more are sometimes considered tolerable, provided that the embankment is stable after the shaking is over.

In a typical case, an existing compacted-fill embankment was constructed on an alluvial foundation, prior to the profession becoming fully cognizant of the potential for liquefaction in general, and the seismicity at that site in particular. Subsequently, *in situ* testing and new analyses have shown the presence of loose, granular, foundation material that is potentially liquefiable, and/or new seismologic studies have shown the area to be more active than previously thought. Hence, stability and deformation potential of the dam need to be reevaluated.

Under the current state of practice, the strength estimate for the foundation is most commonly obtained from empirical correlations between penetration resistance, and S_{ur} values back-calculated from case histories of instability or large deformations of slopes, plus one case history of bearing capacity failure under an apartment builiding. At least six correlations have been developed using the standard penetration test (SPT) blowcount as an index of soil density, beginning with the work by H. Seed (1987). Correlations have also been developed using the cone penetration test (CPT) as an index of density, although this paper includes only those based on the SPT. The principles involved are quite similar.

For development of the correlations, estimates of Sur were obtained originally from simple stability analyses of the pre-failure and post failure configurations, with the material properties varied until the factor of safety of 1.0 is obtained. The analysis of the pre-failure condition gives a firm upper bound, because the strength needed to maintain stability was obviously exceeded by the driving forces. If the properties of the materials are actually constant throughout the process, the post-failure condition should provide a lower bound, because a state of equilibrium has been reached, and the actual static factor of safety has to be at least 1.0. More detailed calculations can take into account the kinematics of the slope movement. However, the shearing resistance of materials without large fines contents may not be constant during the slope movement, because of drainage and dilation. The resistance that is available to bring the slide mass to a stop and then keep it stable could therefore be larger than the resistance available to prevent the movement from beginning or exceeding some tolerable small amount in the first place. (The latter value is, of course, what the analyst needs for evaluating stability or determining whether deformations of the dam would be within tolerable limits.)

There are other methods in use, but these correlations are very common in current practice for several reasons. First, penetration testing is far easier and less expensive than undisturbed sampling of loose granular materials for laboratory strength measurements. In that type of material, it is nearly impossible to prevent sampling disturbance, and the undrained strength is quite sensitive to minor changes in void ratio (Poulos *et al,* 1985). The measured shear strength is also very sensitive to the stress and strain boundary conditions (Riemer and R. Seed, 1997), and those in the ground are not easily replicated in the laboratory. Even the best laboratory testing cannot achieve the very high strains that may be needed to mobilize the residual undrained shear strength. Possibly more important is the fact that a laboratory test specimen can provide an estimate of shearing resistance at only a point (and even that is not a direct measurement), whereas an actual slope failure involves a large volume of soil, within which there is significant variation in density and behavior, even in relatively uniform soils. A laboratory test simply cannot capture the behavior of a large soil mass with varying properties.

Correlations with penetration resistance are far from a perfect solution, however, for a number of reasons, including the small, heterogeneous data set, uncertainty in the material properties and back-figured S_{ur} in the histories, and the potential for different mechanisms to govern the strength in different cases.

PREVIOUS WORK

Prediction of residual undrained shear strength of liquefied soils by correlation with SPT blowcount was first proposed by H. Seed in 1987, based on twelve case histories that included both earthquakes and "static" liquefaction of two hydraulic fill dams under construction. It provided a prediction of S_{ur} as a function of the SPT blowcount adjusted for effective overburden stress, hammer energy, and fines content, $(N_1)_{60}$. The fines adjustment was included because of the observation that, for a given relative density or a given degree of resistance to generation of excess pore-water pressure, an increase in fines content causes a reduction in penetration resistance. This sort of adjustment is almost universally applied in analyses of liquefaction triggering. However, the adjustment proposed by H. Seed (1987) for S_{ur} is different from the earlier one proposed by H. Seed *et al* (1985) for use in liquefaction triggering analysis. The S_{ur} fines adjustment differs from the triggering fines adjustment in that it is significantly smaller for a given blowcount and fines content, and that it allows for additional benefit from fines contents greater than 35 percent. It ranges from zero for clean sands, to as much as 5 additional blows with 75 percent fines. At least five other correlations have been published for S_{ur} from the SPT, as shown in table 1.

Table 1. Published SPT- S_{ur} Correlations

Publication	**Format**	**Fines Adjustment**
H. Seed (1987)	S_{ur} Directly	Adds up to 6 blows, for up to 75 percent fines
R. Seed and Harder (1990)	S_{ur} Directly	Same as Seed (1987)
Baziar and Dobry (1995)	S_{ur} Directly	None
Stark and Mesri (1992)	S_{ur} / σ'$_{vo}$	Same as in Seed *et al* (1985) triggering analysis
Olson and Stark (2002)	S_{ur} / σ'$_{vo}$	None
Idriss and Boulanger (2008)	S_{ur} / σ'$_{vo}$	Same as Seed (1987)

Building on the work by H. Seed by reevaluating some of the case histories and including additional ones, R. Seed and Harder (1990) produced a similar correlation, using the same general format and the same fines adjustment. This correlation, shown in Fig. 1, is probably the one most widely used in the United States at present (2009). Not surprisingly, given the heterogeneous data set it is based on, the correlation is not particularly "tight," but it does show a clear trend of increasing S_{ur} with increasing adjusted SPT blowcount.

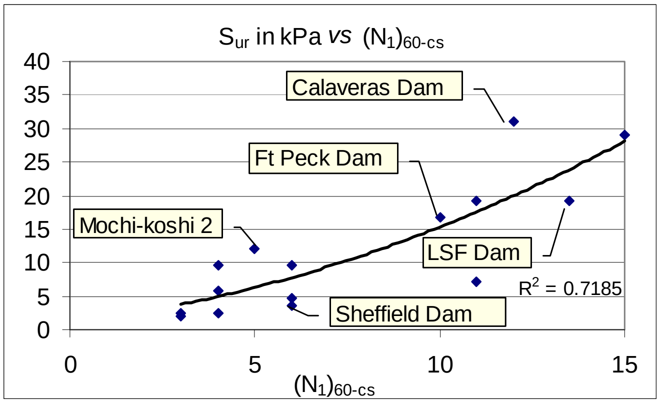

Fig. 1. R. Seed and Harder (1990) data for prediction of S_{ur} from SPT $(N_1)_{60\text{-cs}}$.replotted.

Baziar and Dobry (1995) replotted Seed and Harder's strength results and penetration data, but without the fines adjustment. The result was a somewhat tighter correlation, but they excluded the case history of the Mochi-koshi gold and silver tailings dams in Japan. That particular history is pivotal in establishing the need for and magnitude of the fines adjustment. The tailings were impounded by three upstream-method dams arranged around a natural low area (Okusa, *et al*, 1980). After filling behind the starter dikes (crushed weathered rock) was nearly complete, raise dikes were constructed on the surface of the tailings. The tailings were deposited as alternating thin layers of silt and sandy silt. At the time of the 1978 Izu-Oshima-Kinkai Earthquake (M 7.0), the three dams varied in height between 7 m and 16 m, and tailings placement was active. Within 10 seconds of the main shock, the highest of the three dams failed above its starter dike (which remained stable), and about 80,000 m^3 of slurried tailings were released. A day later, about 5 hours after a M 5.8 aftershock, the second highest dam failed, releasing about 3000 m^3 of tailings slurry. In post-failure investigations, standard penetration tests gave unadjusted SPT blowcounts as low as zero or one in the material that had liquefied, even though the behavior of the tailings was more consistent with material having somewhat higher blowcount. Thus, H. Seed (1987) proposed that the fines adjustment should add as much as 5 blows to $(N_1)_{60}$ for the "beneficial" effect of high fines content. Baziar and Dobry, citing Ishihara (1984), were concerned that the very low blowcounts resulted from the tailings dam being actively raised at the time of the earthquake, which was not long before the SPTs were performed, and from excess pore-water pressure remaining from the earthquake. They concluded that Mochi-koshi should not be included in the set of data for a correlation. (Later, Olson and Stark (2002) re-examined both of the failed Mochi-koshi dams, and assigned the tailings much lower strength estimates. These were fairly consistent with other cases with similar $(N_1)_{60}$ values (not adjusted for fines), suggesting that a fines adjustment may not be needed.)

Taking a different approach, Stark and Mesri (1992) developed a correlation to predict, not S_{ur} itself, but the ratio of S_{ur} to the pre-earthquake effective overburden stress, σ'_{vo}. The strength-ratio approach would, if well supported by data, be advantageous for embankment dams, where the pre-earthquake effective stresses are generally much higher than in most of the case histories. The reasonable range of strengths for $(N_1)_{60\text{-cs}}$ equal to 12 would be about 8 kPa to 30 kPa from R. Seed and Harder's correlation. Under a large embankment, this range is not generally sufficient to maintain stable slopes. In contrast, the strength of this material under 40 m of embankment would be predicted as 80 kPa or more by Stark and Mesri's, which could make a significant difference in the post-earthquake static factor of safety. The selection of strength-ratio format for the correlation was based on observations of loose soils in undrained laboratory shear tests. This work was further distinguished from the R. Seed and Harder (1990) by the use of the fines adjustment usually associated with liquefaction *triggering* analysis (H. Seed *et al*, 1985) instead of H. Seed's 1987 fines adjustment for S_{ur}.

The strength-ratio approach was expanded upon by Olson and Stark (2002), who included a number of additional case histories, and more detailed back-analyses of S_{ur} that included kinetics. (Olson's 2001 dissertation provides a valuable summary of most of the available case histories.) A significant departure from Stark and Mesri's work was the elimination of the fines adjustment. Figure 2 is a plot of Olson and Stark's $(N_1)_{60}$ and S_{ur}/σ'_{vo} data; the correlation is less obvious than is the correlation with S_{ur} directly, shown in Figure 1. (Stronger correlation is seen if data with low σ'_{vo}, less than, say, 50 or 70 kPa, are removed from the set; this is discussed below.)

Considering that high pre-earthquake effective overburden stress must produce at least some increase in S_{ur}, Seed *et al* (2003) recommended using an average of the S_{ur} values from Seed and Harder (1990), weighted 80 percent, and from Olson and Stark (2002), weighted 20 percent. The effect of overburden is fairly minor in this relationship.

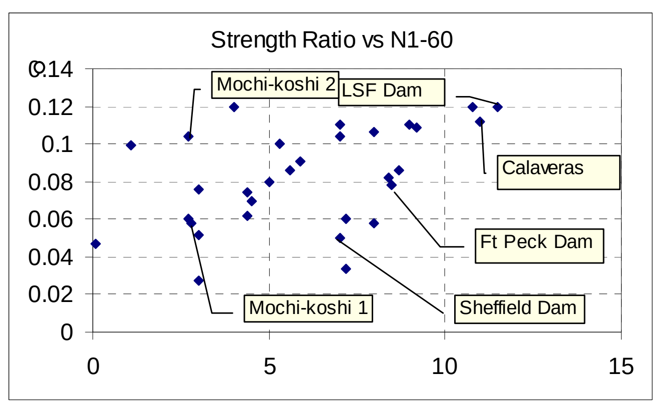

Fig. 2. Olson and Stark $(N_1)_{60}$ and S_{ur}/σ'_{vo} data replotted.

The strength-ratio "format" was later adopted by Idriss and Boulanger (2008), albeit WITH the H. Seed (1987) fines adjustment. Idriss and Boulanger used blowcounts and strength estimates both from R. Seed and Harder (1990), and from Olson and Stark (2002). From those data, they developed two "recommended" curves, one for use in situations with potential for upward migration of voids to create a fluidized zone just below a less-pervious layer, and one for situations without that potential. This is shown in Fig. 3. The importance of void migration is discussed in more detail later in this paper.

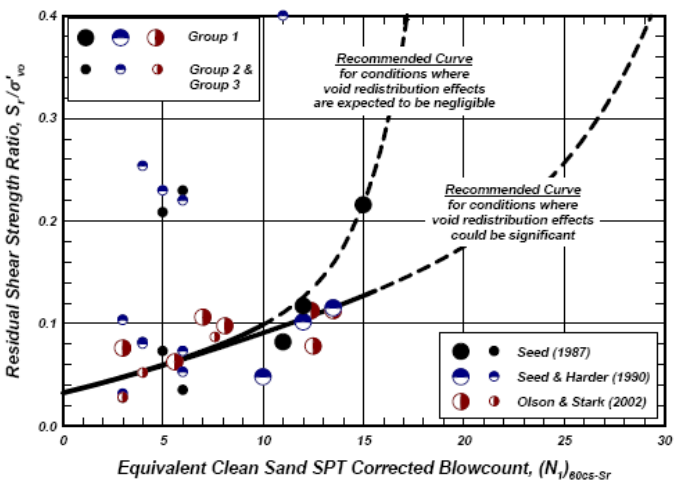

Fig. 3. Idriss and Boulanger (2008) recommended curves for estimating S_{ur}/σ'_{vo}. (Note that some case histories appear more than once, where different researchers have estimated different values.)

Discussions that follow refer to several specific case histories; a portion of those compiled by Olson and Stark (2002) are shown in Table 2. The table includes all of the embankment dam cases, and all cases with more than 50kPa of overburden, these being the ones most relevant to this study. With the exception of La Marquesa and La Palma Dams, they all have σ'_{vo} over 50 kPa. The values of N shown are average or "representative" values. In some cases, the SPT blowcount was not actually measured and had to be estimated from relative density or CPT. Seismic case histories in the table have been marked with asterisks. Considering only the flow slides actually caused by earthquakes would substantially diminish the data base, removing from it the construction-related failures of two large hydraulic fill dams, Fort Peck and Calaveras, which are the two cases with the highest effective overburden stresses.

Table 2. Selected Case Histories. Data from Olson and Stark (2002). (* indicates seismic case histories)

Site	**$(N_1)_{60}$, % Fines**	**σ'_{vo}, kPa**	**Est'd S_{ur}, kPa**
Wachusett Dam dumped fill	8 5 - 10	151.3	16
Calaveras Dam hydraulic fill	11 10 to >60	307.5	34.5
*Sheffield Dam alluvium	7 33 to 48	72.4	3.6
Ft. Peck Dam hydraulic fill	8.5 ~ 55	350.6	27.3
*Lake Merced bank dumped fill	10.8 1 - 4	55.3	6.7
*Kawagishi Cho Building hydraulic fill	4.4 <5	69.2	5
*Uetsu dumped fill	3 0 - 2	60.3	1.7
*Hokkaido tailings hydraulic fill	1.1 ~ 50	65.9	6.5
*Lower San Fernando Dam hydraulic fill	11.5 50 (5-90)	155.7	18.7
Tar Island hydraulic fill	8 10-15	205.9	12.0
*Mochi-koshi tailings no. 1 hydraulic fill	2.7 85	59.9	3.6
*Mochi-koshi tailings no. 1 hydraulic fill	2.7 85	52.2	5.5
Asele Road dumped fill	7 32 (23-38)	60.1	6.2
*Tajikistan loess	8.4 100	103.9	8.4
*La Marquesa Dam u/s alluvium	4.5 ~ 30	43.6	3.1
*La Marquesa Dam d/s alluvium	9 ~ 20	47.9	5.3
*La Palma Dam alluvium	4 ~ 15	37.8	4.8
Lake Ackerman dumped fill	3 ~ 0	51.5	3.9
*Chonan School dumped fill	5.9 18	53.6	4.8
*Nalband Railroad dumped fill	9.2 ~ 20	52.7	5.7

VARIOUS MECHANISMS THAT MAY GOVERN S_{ur} AT DIFFERENT SITES

Were one to consider *a priori* what governs S_{ur} and how it would vary with SPT blowcount, one might not even expect to see much correlation at all, because of the number of different mechanisms that govern S_{ur}, possibly all at once in different portions of the same deposit.

Liquefied soil is often thought of in terms of its critical state or steady state, the condition of constant volume and constant effective stress when the soil has been sheared to the point that all structure has been destroyed and the shearing resistance is governed solely by the void ratio (for a given soil).(Castro and Poulos, 1977). If this is the case, for a particular soil at a given void ratio, the residual undrained shear strength should be unaffected by the pre-earthquake overburden stress, and it should be at least somewhat predictable from the SPT blowcount. For liquefaction assessment, the "raw" SPT blowcount N is adjusted to a standard hammer efficiency of 60 percent and standard effective overburden stress to 1 ton/ft^2 (approximately 1 atmosphere or 100 kPa), to give $(N_1)_{60}$. As an index of density, the value of $(N_1)_{60}$ is independent of the overburden stress, and for a given soil, it is controlled solely by the void ratio or relative density. By this line of reasoning, one would expect the best prediction of S_{ur} to come from a direct correlation with $(N_1)_{60}$, like those proposed by H. Seed (1987) and R. Seed and Harder (1990). Indeed, a positive correlation exists. However, things are more complicated.

Whitman (1985) identified redistribution of voids as the governing mechanism for the post-earthquake shearing resistance in some situations. If, during and after the earthquake, the liquefied soil settles under its own weight, pore water would be forced upward. If there is an impervious cap layer that traps the water, this can create a very loose zone or even a film of water at the top of the liquefied soil. The strength is still governed by the critical state, but it is the critical-state strength of much looser material, and therefore much lower than with the material at its original density. This may explain the delay between the end of an earthquake and the onset of slope instability, for example at Lower San Fernando Dam and at Mochi-koshi Tailings Dam No. 2. When the slope of Fort Peck Dam failed during construction, there was already upwelling of water from the hydraulic fill, consistent with the existence of zones with very high excess pore-water pressure within the stratified fill prior to the slide. The delayed development of the loosened layers suggests that one could use a higher value of Sur for analyzing dynamic deformation during the strong ground motion than for post-earthquake slope stability. (Depending upon how fast the excess pore pressure dissipates, that may not be valid for aftershocks.) Idriss and Boulanger (2008) included separate curves for cases where void migration is expected and those where it would not. Controlling factors would include the original void ratio and the thickness of the liquefied layer, which governs the volume of voids that are available to migrate upwards, in addition to the requirement for a less-pervious capping layer to confine the water.

The third major controlling mechanism is the tendency for materials of medium density, loose enough to be contractive initially when sheared but denser than their critical state, to begin to dilate after a few percent strain. They then recover some of their shearing resistance with further strain. This tendency could explain the absence from the data base of slope-failure case histories with $(N_1)_{60}$ greater than 12. In these cases, the strength-ratio model would quite likely be more correct.

LIMITATIONS OF THE AVAILABLE CASE-HISTORY DATA FOR EMBANKMENT DAM FOUNDATION ANALYSIS

The great majority of historic occurrences of liquefaction have occurred at fairly shallow depths, less than about 8 meters, and mostly with less than 70 kPa of effective overburden pressure (no more than 4 to 7 m below the ground surface). For flow liquefaction (as opposed to cyclic mobility), there have only been about eight cases with effective overburden more than 70 kPa, and all known cases of flow liquefaction occurred with effective overburden stress less than 400 kPa. This is in contrast to 1000 kPa or more under a large embankment dam.

Data are very few for $(N_1)_{60}$ values above 10, and completely nonexistent for values above 12, or for $(N_1)_{60\text{-}cs}$ values above 14, in the data sets of R. Seed and Harder (1990), and of Olson and Stark (2002). This probably results from one or both of two causes: First, there appears to be a qualitative difference in the post-earthquake behavior of soils slightly looser than $(N_1)_{60} = 12$ and those slightly denser, making the latter much less prone to flow after a few percent strain as mentioned before. It could also, at least in part, be the lack of suitable trials where slightly denser soils were liquefied but did not have sufficient driving forces on them to cause instability. If it were simply the latter, the historic absence of flow slides in materials with $(N_1)_{60}$ in the middle or upper teens would mean very little in evaluating sites with slightly higher densities.

While there might have been some effect from the small data set, lab tests on granular soils at comparable densities typically show a boundary between purely contractive behavior, and initially contractive behavior followed by dilation and recovery of strength at large strains (Ishihara, 1993; R. Seed *et al*, 2003). In cyclic undrained triaxial and direct simple-shear tests, this is seen as cyclic mobility, wherein, after several cycles of shearing, the effective stress and shearing resistance may be near zero for several percent strain, then abruptly increase as the soil begins to dilate; the same thing occurs when the direction of shearing is reversed. This is seen as "butterfly loops" in the stress path. Dilation has been observed in monotonic tests with relative densities as low as 18 percent (Ishihara, 1993). Robertson (in press) has identified a fines-adjusted normalized cone penetration tip resistance, $Q_{tn,cs}$, of 70 as the boundary between purely contractive and initially contractive behavior. This value is approximately equivalent to $(N_1)_{60\text{-}cs}$ equal to 14.

Only one of the cases of flow sliding with σ_v' above 50 kPa involved liquefaction of alluvium (Sheffield Dam,

$(N_1)_{60} = 7$); the remainder were all hydraulic fill, dumped fill, or in one case, loess. Is there something different about alluvium? Considering the great abundance of alluvium the world over, the lack of flow-slide case histories from alluvium with blow counts above 7 suggests that alluvium is somehow different from hydraulic fills and loess. This may be a result of deposition by meandering streams, with alternating aggradation and degradation causing discontinuity in material properties over short horizontal distances, typically a few meters or less. Complete liquefaction would therefore be less likely to occur uniformly over a large area of the dam foundation, and there would be potential for drainage of excess pore pressure horizontally as well as vertically. Instability of a large embankment would require loss of strength to occur over a larger area of foundation than would instability of a small one. Point-to-point variation in foundation properties would therefore be likely to provide more benefit under high, short embankments on alluvium than under long low ones, or in hydraulic fills regardless of overburden. The use of data from hydraulic fills or uncompacted dumped fills to predict the behavior of alluvium could be misleading, possibly leading to unnecessarily conservative strength estimates. Nonuniformity could cause some portions of a deposit to be fully liquefied with pore-pressure ratios of nearly 100 percent, while others nearby still have grain-to-grain contact that can carry some portion of the original effective overburden stress. This would support the normalized strength models of Olson and Stark (2002) and Idriss and Boulanger (2008), particularly for alluvium.

From the 2001 Bhuj, India Earthquake, came well-documented case histories of several embankment dams that became unstable and were severely damaged due to liquefaction of their alluvial foundations, most notably Chang and Shivlakha Dams (Singh *et al*, 2005, Krinitskzsky and Hynes, 2002). Unfortunately, penetration data for Chang and Shivlakha Dams could not be located during the preparation of this paper, so they could not be added to the data base. The earthquake added no new information in the portions of the data base where it was most needed, σ'_{vo} greater than 70 kPa and/or $(N_1)_{60}$ greater than 8.

Singh *et al* and Krinitzsky and Hynes also describe several dams whose alluvial foundations were apparently liquefied by the Bhuj earthquake (in the sense of very high excess pore pressure and softening) without becoming unstable.

Kaswati Dam is a 12.9 m-high zoned earthfill embankment with a central cutoff trench. The alluvial foundation consists of sand-silt mixtures with raw blowcounts of 13 to 19, suggesting that typical fines-adjusted blowcounts $(N_1)_{60\text{-cs}}$ in the foundation would be no higher than the low or middle teens. (The word *suggesting* is used because drilling logs and laboratory data were not located in time for this publication.) The dam was subjected to an estimated peak horizontal ground acceleration (PHA) of 0.7 g. At the time of the earthquake, the reservoir was nearly empty, but the upstream foundation was saturated. In spite of the very severe loading and liquefaction of the upstream foundation, settlement of the crest was limited to roughly 1 m, and movements of the upstream slope were primarily translational. The translational movement produced longitudinal cracks, and a small heaved area at the upstream toe. It should be noted here that the geometry of the foundation and central cutoff trench would be conducive to instability with a low foundation strength such as would be predicted by the R. Seed and Harder (1990) correlation with $(N_1)_{60\text{-cs}}$ of 12 to 15.

Tapar Dam is also a zoned earthfill embankment with a central cutoff trench, having a height of 15.5 m., and is located only 10 km from the epicenter. There were sand boils at the upstream toe, indicative of liquefaction, and the embankment showed cracking and translational movements with scarps as high as 1 m in the upstream slope. Crest settlement was small, however, and the slopes remained stable at the end of the earthquake. The configuration of the dam, with a compacted clay blanket extending upstream to help control foundation seepage, is somewhat less prone to instability than that of Kaswati Dam.

While foundation density data have not been located for Tapar Dam, these two case histories show that foundation liquefaction is not a guarantee of instability, and they support the idea that medium-density alluvium can be liquefied and undergo loss of shearing resistance that is recovered at larger strains, enough to prevent slope instability.

IMPROVED CORRELATIONS WITH TWO INDEPENDENT VARIABLES

In an effort to both determine which format is more realistic and provide a better correlation that explicitly allows for improvement of S_{ur} with increasing overburden stress, the writer used strength and SPT blowcount estimates from R. Seed and Harder and from Olson and Stark to develop new correlations that make Sur a function of both blowcount and pre-earthquake effective overburden stress. Two different forms of the correlation were tried:

$$S_{ur} = a\,(N_1)_{60\text{-cs}}{}^{b} + c\,\sigma'_{v0}{}^{d} + e \tag{1}$$

$$S_{ur} = p\,(N_1)_{60\text{-cs}}{}^{q} \times \sigma'_{v0}{}^{r} + s \tag{2}$$

Eqn. 1 was intended to resemble somewhat the form of the recommendation of R. Seed *et al* (2003) for situations of high overburden stress, which is to use a strength that is a weighted average of the direct-strength and strength-ratio approaches. (Its two main components are a function of $\mathbf{(N_1)_{60\text{-cs}}}$ only and one of $\boldsymbol{\sigma'_{v0}}$ only, so it is not identical in form.) Equation 2 was intended to resemble the strength-ratio approach, consisting of the *product* of a function of the blowcount only and effective overburden stress raised to the power **r**. If the best fit of Equation 2 to the data is found with

r being approximately 1.0, that would support the validity of a simple strength-ratio approach.

(For fitting Eqns. 1 and 2 to Olson and Stark's results, $(N_1)_{60}$ was substituted in the equations for $(N_1)_{60\text{-cs}}$.)

By simplified methods (primarily trial and error while tracking r^2), the parameters a through e, and p through s that gave the best fit were determined. A reasonable, though far from precise, fit of R. Seed and Harder's strengths and blowcounts is given by Eqn. 4:

$$S_{ur} = 0.64\,(N_1)_{60\text{-cs}}^{1.35} + 0.1\,\sigma'^{0.8}_{v0} - 2.3 \pm 6 \text{ kPa} \tag{4}$$

$(r^2=0.78)$

This correlation is only slightly better than a correlation using $(N_1)_{60\text{-cs}}$, with $r^2=0.72$. Better correlation is found using Olson and Stark's results:

$$S_{ur} = 0.28\,(N_1)_{60}^{1.30} + 0.16\,\sigma'^{0.88}_{v0} - 2.3 \pm 6 \text{ kPa} \tag{5}$$

$(r^2=0.90)$

No good fit was found using Eqn. 2 with the full set of data. However, by selecting only those cases with effective overburden stresses greater than 50 kPa (approximately 0.5 atm or 1000 lb/ft^2), much better results were obtained. This is shown in Equations 6 and 7 for Harder and Seed, and for Olson and Stark, respectively, especially using Olson and Stark's (Eqn. 7):

$$S_{ur} = 0.022\,(N_1)_{60\text{-cs}}^{1.0} \times \sigma'^{0.80}_{v0} + 1 \pm 5 \text{ kPa} \tag{6}$$

$(r^2=0.87)$

$$S_{ur} = 0.014\,(N_1)_{60}^{0.95} \times \sigma'^{0.95}_{v0} + 1 \pm 4 \text{ kPa} \tag{7}$$

$(r^2=0.94)$

The fact that the exponent on σ'_{v0} in Eqn. 7 is very close to 1.0 does support the idea of normalizing S_{ur}, but this only appears to work if the case histories with low overburden stresses are left out. Fig. 4 is similar to Fig. 2, except the data for $\sigma'_{\mathbf{v0}}$ less than 50 kPa have been left out. Doing so greatly reduces the number of data. One can see, however, that there is a more-visible trend of higher strength ratios with higher blowcounts than there is in Fig. 2, and plotting only those with more than 70 kPa. It must be recognized that the cases with high overburden also tend to have higher blowcounts, which may result from consolidation. For example, above 70 kPa, all cases have blowcounts $(N_1)_{60}$ between 7 and 12, and a review of data from other sites suggests that blowcounts less than 7 are not common with that much overburden (typically 4 to 7 m below the ground surface). Quite possibly, $(N_1)_{60}$ and σ'_{v0} aren't really independent variables as they were modeled here. At lower overburden stresses, Equation 7 tends to over-predict S_{ur}, suggesting that it is not appropriate to include ratios from the low-overburden cases in a correlation intended for use on dam foundations, where the effective overburden can exceed 1000 or even 2000 kPa.

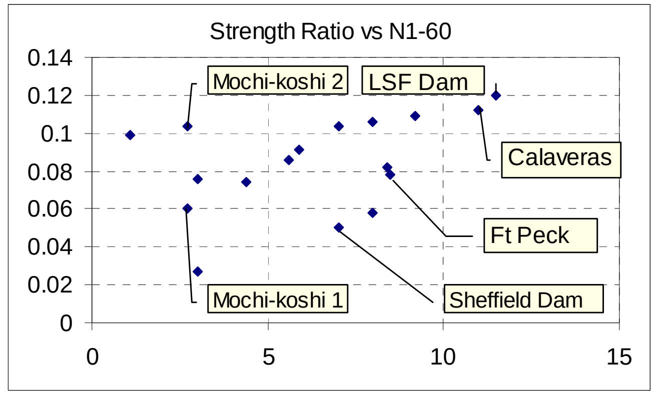

Fig. 4. Olson and Stark $(N_1)_{60}$ and S_{ur}/σ'_{vo} data replotted (cases with $\sigma'_{\mathbf{v0}}$ greater than 50 kPa only).

CONCLUSIONS

Simple statistical analysis of SPT blowcount and backfigured residual undrained shear strength did not produce a clear conclusion regarding the most appropriate "format" for correlations to predict residual undrained shear strength from SPT blowcounts. The analysis did, however, produce Eqns. 6 and 7, which the writer believes fit better with the existing data than either a strength ratio that is a function of $(N_1)_{60}$ or $(N_1)_{60\text{-cs}}$, or Seed and Harder's prediction of Sur directly from $(N_1)_{60\text{-cs}}$., which does not allow for any beneficial effect from higher overburden stress (except as it is modified in Seed *et al* (2003)). The strength-ratio approach appears to work better at higher effective overburden stresses, exceeding 50 to 70 kPa, than it does at lower ones. One must recognize, however, that the data are very few for those higher stresses, and that the foundation of an embankment dam can have overburden stresses twice as high as any of the case histories, or significantly more. For medium-density soils, those dense enough to dilate at larger strains after initial liquefaction, the strength ratio is thought to be the most realistic model, as the shearing resistance increases with larger strain and becomes a large fraction of the drained strength.

Very few of the case histories of flow sliding involve alluvium, and none of the ones with σ'_{v0} greater than 50 kPa or $(N_1)_{60}$ greater than 7 did, in spite of the abundance of alluvium worldwide. This suggests a qualitative difference between alluvium and other materials that have liquefied and allowed a flow slide historically. One possible explanation is discontinuities in material properties over relatively short distances due to alternate aggradation and degradation.

Foundations of embankment dams are commonly comprised of alluvium, and they generally have effective overburden stresses somewhat greater to many times greater than those in the case-history data base. In the writer's experience, values of $(N_1)_{60}$ below 8 or 10 are simply not very common in the foundations of large dams, whose weight may consolidate the foundations to some extent. This brings into question the applicability of a correlation based primarily on other materials, but it is also recognized that the data set is small so it is possible that there simply have not been the necessary earthquakes in the right location to "test" very many alluvial foundations with blowcounts in the low to middle teens and enough driving force to cause instability.

REFERENCES

Baziar, Mohammad H. and Ricardo Dobry (1995), "Residual Strength and Large-Deformation Potential of Loose Silty Sands", ASCE *Journal of Geotechnical Engineering*, vol. 121, no. 12, pp. 896-906.

Castro, G. and S.J. Poulos (1997), "Factors Affecting Liquefaction and Cyclic Mobility, *Journal of the Geotechnical Engineering Division,* ASCE, v. 103(6), 501-516.

Ishihara, K. (1993), "Liquefaction and flow failure during Earthquakes," 33rd Rankine Lecture, *Géotechnique*, Vol 43, No. 3, pp 351-415.

Krinitzsky E.L. and M.E. Hynes (2002), "The Bhuj, India Earthquake: Lessons Learned for Earthquake Safety of Dams on Alluvium," *Engineering Geology,* 66, 163-196.

Olson, S.M. (2001), *Liquefaction Analysis of Level and Sloping Ground Using Field Case Histories and Penetration Resistance,* PhD thesis, University of Illinois, Urbana-Champaigne.

Olson, S.M. and T.D. Stark (2002), "Liquefied Strength Ratio from Liquefaction Flow Case Histories," *Canadian Geotechnical Journal*, v. 39, 629-647

Poulos, S. J., Castro, G., and France, J. W., (1985), "Liquefaction Evaluation Procedure," *Journal of Geotechnical Engineering*, ASCE, v. 111(6), 772-792.

Riemer, Michael F. and Raymond B. Seed (1997), "Factors Affecting the Apparent Position of the Steady-State Line", ASCE *Journal of Geotechnical and Geoenvironmental Engineering*, 123(3), 281-288.

Robertson, P.K. (in press), "Evaluation of Flow Liquefaction and Liquefied Strength Using the CPT," submitted to *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE.

Seed, H. B. (1987), "Design Problems in Soil Liquefaction," *Journal of Geotechnical Engineering*, ASCE, Vol. 113(8), 827-845.

Seed, H.B., K. Tokimatsu, L.F. Harder, Jr. and R.M. Chung (1985), "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluaations," ASCE *Journal of Geotechnical Engineering,*, v. 111(12), 1425-1445.

Seed, R.B., K.O. Cetin, R.E.S. Moss, A.M. Kammerer, J. Wu, J.M. Pestana, M.F. Riemer, R.B. Sancio, J.D. Bray, R.E. Kayen and A. Faris (2003), "Recent Advances in Soil Liquefaction Engineering: a Unified and Consistent Framework," keynote presentation, 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Long Beach CA.

Singh, R., D. Roy, and S.K. Jain (2005), "Analysis of Dams Affected by the 2001 Bhuj Earthquake," *Engineering Geology,*80, 282-291.

Stark, Timothy D. and Gholamreza Mesri (1992), "Undrained Shear Strength of Liquefied Sands for Stability Analysis", ASCE *Journal Of Geotechnical Engineering*, v. 118(11), 1727-1747.

Whitman, R.V. (1985), "On Liquefaction," *Proceedings,* XIth International Conference on Soil Mechanics and Foundation Engineering, San Francisco CA.