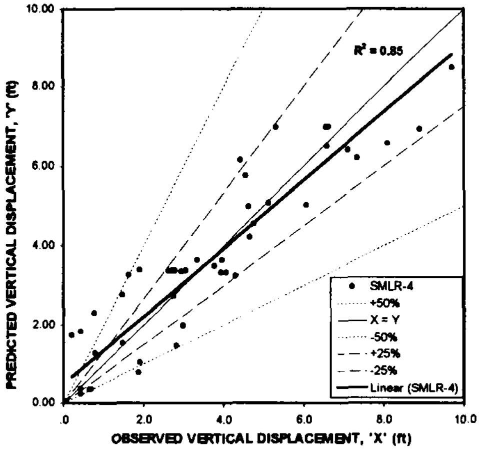

Figure 5. Comparison of Observed vs. Predicted Values for Model SMLR-4

$$SL = a_{max}/MSF \tag{8}$$

where MSF is the magnitude scaling factor defined below (Idriss, 1999):

$$MSF = 37.9 \bullet M_W^{-1.81} \quad \text{for } M_W \geq 5.75 \text{ and} \tag{9}$$

$$MSF = 1.625 \quad \text{for } M_W < 5.75 \tag{10}$$

With the addition of the variable SL, the following equation is obtained:

$$\Delta h = 0.943 + 0.145(T) + 0.439\,(\theta) - 0.151\,(N) + 0.00026\,(D) - 1.871\,(SL) \tag{11}$$

The final and highest R^2 obtained for this model, referred to herein as Model SMLR-5, is 0.854. The plot of the measured versus predicted values of vertical displacement is presented in Figure 6. Addition of the variable SL does not improve significantly the accuracy of the resulting model. This might be expected, as all data used in the analysis came from only two earthquakes.

A summary of the performance of the above regression models is presented in Table 2. In addition to R^2 values, the success rates, defined at two levels of accuracy, are shown. Model SMLR-5 is shown to be the best among all six models examined.

Plots of the residuals are presented in Figure 7 ('a' through 'e') for each of the input variables, respectively. The residuals plotted on the 'Y' axes are the difference between

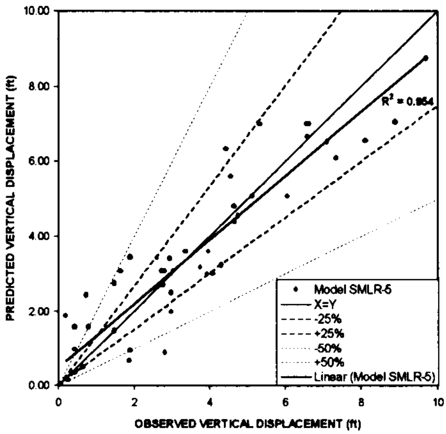

Figure 6. Comparison of Observed vs. Predicted Values for Model SMLR-5

the observed and the predicted values. The independent variables, thickness of the liquefied layer, ground slope, N-value, distance to the nearest water source, and seismic load, are plotted on the 'X' axes. A homoscedastic association is apparent in all of the scatter plots, since there is a random distribution of the residuals about zero.

Table 2. Performance of Regression Models for Predicting Vertical Displacement 'Δh'

Model	Inputs	R^2 for measured vs. predicted displacement	Success Rate (within ± 25% of measured value)	Success Rate (within ± 50% of measured value).
MLR	S = *f*(log(N), θ, T)	0.83	30%	60%
SMLR-1	S = *f*(T)	0.63	38%	78%
SMLR-2	S = *f*(T, θ)	0.78	48%	86%
SMLR-3	S = *f*(T, θ, N)	0.82	66%	86%
SMLR-4	S = *f*(T, θ, N, D)	0.85	70%	90%
SMLR-5	S = *f*(T, θ, N, D, SL)	0.854	76%	90%

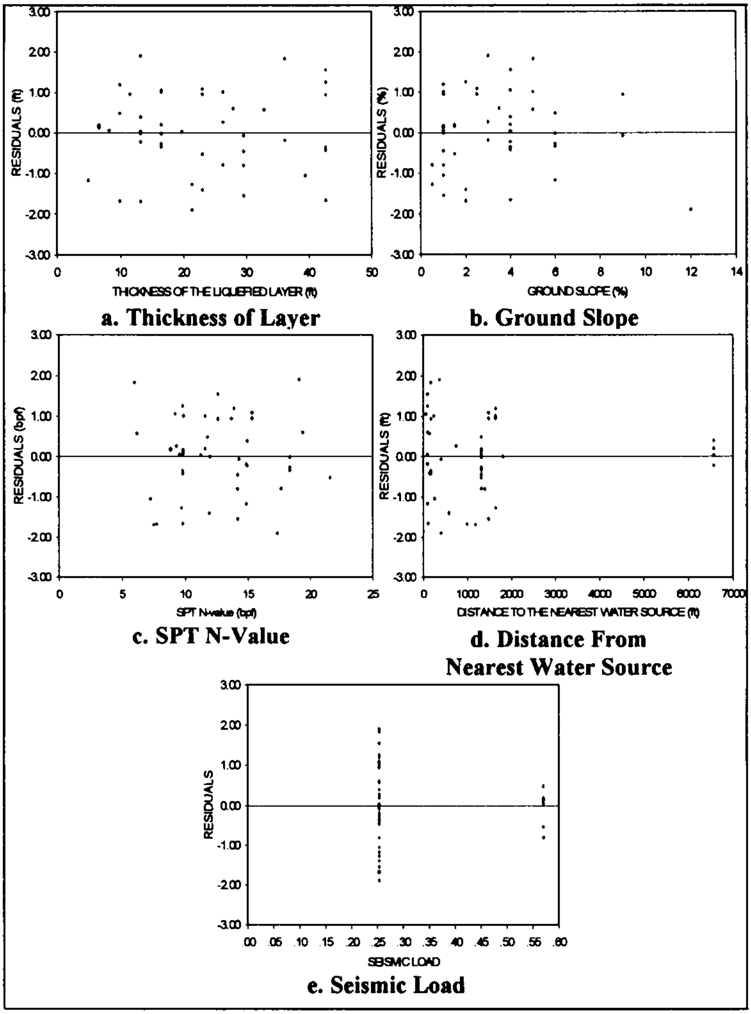

Figure 7 (a-e). Residual Plots of Input Variables For Regression Model 2

The residual plot of the independent variable, seismic load is presented in Figure 7e. Because the data were obtained from two earthquake sites (Niigata, 1964 and San Fernando, 1971), only one value of seismic load was obtained at each of the sites. Consequently, there are only two values along the 'X' axis where the distribution of residuals about zero is significant. Nonetheless, there is an approximately even distribution of residuals about the 'zero' intercept, which indicates that this relationship also is homoscedastic, and should therefore provide adequate results in future predictions.

It is recommended that Equation (11) (Model SMLR-5) be used with the following ranges of input variables:

T	= 0 to 50 (ft),
θ	= 0 to 12 (%),
N	= 0 to 30 (bpf),
D	= 0 to 1500 (ft), and
SL	= 0.20 to 0.6.

Equation (11) may be used to predict the total amount of vertical displacement at sites where the expected lateral spreads are potentially large (i.e., sites with a large free face ratio, ground slope, and/or near a water source).

CONCLUSION

The empirical MLR model presented in Equation (11) provides a good predictive capability (R^2 = 0.854) and success rate (90% of the cases fall within ± 50% of the observed values) based on the limited data examined. Further validation of the developed MLR model using additional database, particularly using data from earthquakes with various SL values, is warranted.

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POST-LIQUEFACTION FLOW DEFORMATIONS

W.D. Liam Finn[1]

ABSTRACT

The state of the art for evaluating post liquefaction flow deformations of earth structures is presented and the main parameters that affect the deformations are reviewed. The primary focus is on the estimation of residual strength and large displacement methods for analyzing the consequences of liquefaction.

INTRODUCTION

One of the most challenging problems facing geotechnical engineers is the seismic safety evaluation of soil structures such as embankment dams, which have potentially liquefiable soils in the structure itself or in the foundation. This problem poses three difficult questions;

- Will liquefaction be triggered
- If so what are the consequences
- What remediation measures should be adopted ensure satisfactory behaviour.

The triggering of liquefaction was reviewed in 1996 by a Committee appointed by the National Center for Earthquake Engineering at the University of Buffalo. The committee reviewed the state of the art of practice and research since a similar review was conducted in 1985 (NRC, 1985) and made several recommendations for improving the state of practice. The recommendations of the committee and the supporting documentation are reported in NCEER (1997). In view of this comprehensive study, the triggering of liquefaction will not be reviewed in this paper. Here the focus will be on issues related to evaluating the consequences of liquefaction.

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In the context of this review, liquefaction is synonymous with strain softening of relatively loose sands in undrained shear as illustrated by curve 1 in Fig. 1. When the sand is strained beyond the point of peak strength, the undrained strength drops to a value that is maintained constant over a large range in strain. This is conventionally called the undrained steady state or residual strength. If the strength increases after passing through a minimum value, the phenomenon is called limited or quasi-liquefaction and is illustrated by curve 2 in Fig. 1. Even limited liquefaction may result in significant deformations because of the strains necessary to develop the strength to restore stability.

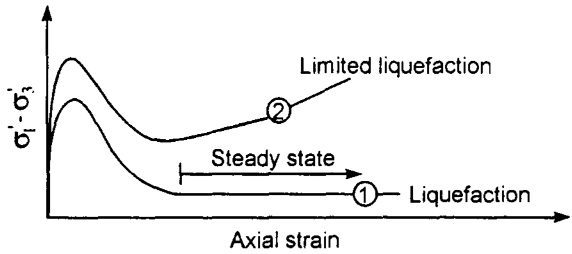

Fig. 1. Types of contractive deformation (Vaid et al., 1989).

The appropriate residual strength for design and analysis is a very controversial matter. This was clearly evident from the proceedings of a major workshop on the shear strength of liquefied soils which was held at the University of Illinois at Urbana in 1997 with broad representation from the research and engineering community (NSF, 1997) and of an international workshop at Johns Hopkins University in Baltimore in 1998 with emphasis on the physics and mechanics of soil liquefaction (Lade and Yamamuro, 1998). The two workshops make very significant contributions to understanding what controls the residual strength of soils. But they also demonstrate the sometimes widely divergent opinions that exist on even the most basic issues.

The focus of this paper is on engineering practice. Many interesting theoretical issues are ignored. They can be found in the proceedings of the two workshops cited above. The primary objective here is to provide a coherent framework of understanding of research findings and methods of analysis to the practicing engineer.

Very few field data have been available to validate our methods for analysing the consequences of liquefaction. The 1993 Kushiro and the 1994 Nansei earthquakes in Hokkaido, Japan caused widespread damage to flood protection dikes by liquefaction and provided a data base for validation of large displacement finite element analysis of post-liquefaction displacements. The results of a validation study for the Hokkaido Development Bureau will be described.

RESIDUAL STRENGTH FROM LABORATORY TESTS

Practice before 1988

Until the late 1980's, residual strength was determined using undrained triaxial compression tests on undisturbed samples from the field or on samples reconstituted to the field void ratio using moist tamping. This approach followed from the pioneering work of Castro (1969). Potentially liquefiable soils are very difficult to sample without disturbance. They are likely to densify during sampling, transportation, and during the process of setting up the samples for testing. Therefore, tests cannot be conducted at the field void ratio. Since the residual strength was considered to be a function of the void ratio only, a logical solution to the disturbance problem was to correct the laboratory residual strength for the effects of changes in void ratio. Poulos et al. (1985) developed such a procedure. However the corrections for disturbance can lead to order of magnitude changes in the measured residual strength. Such large corrections are a matter of concern.

The consequences of liquefaction were assessed primarily by limiting equilibrium analyses of stability. Levels of safety and remediation requirements were defined in terms of acceptable factors of safety. In some instances, displacement criteria were also used in addition to factors of safety. Displacements were estimated using the Newmark sliding block method of analysis. In applying this method, the residual strength was used in determining the yield acceleration. The Newmark method is not an appropriate method for analyzing structures with large volumes of liquefied material undergoing complicated internal distortions. It is best left for those situations envisaged by Newmark in which displacements are constrained to relatively narrow zones of concentrated shear. Large displacement finite element analysis is now being used in practice to determine post-liquefaction deformations in embankment dams. About 15 dams have been analysed in this way since 1989. This type of analysis will be discussed later.

Practice after 1988

In 1987, Harry Seed published the results of a study that changed drastically the state of practice (Seed, 1987). He determined representative values of residual strength by back-analyzing embankments which had undergone significant displacements during earthquakes. The materials yielding these strengths were characterized by corrected, normalized Standard Penetration Resistances, $(N_1)_{60}$. An updated version of his original correlation chart, developed by Seed and Harder (1990), is shown in Fig. 2.

There is no data beyond $(N_1)_{60}$ of 15. However the curve is often extrapolated beyond this range to provide values of residual strength at higher penetrations for safety evaluation and remediation studies. There is considerable scatter in the data and in the region near the lower bound the residual strengths are negligible for $(N_1)_{60}$

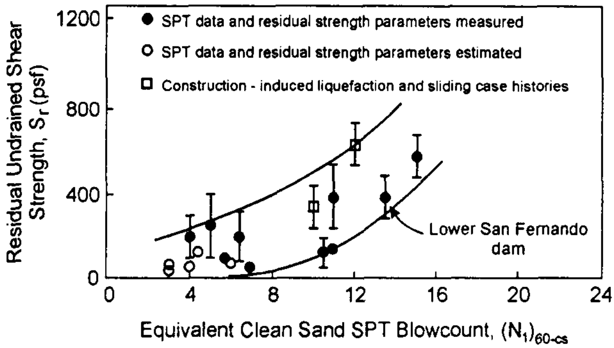

Fig. 2. Relationship between corrected "clean sand" blowcount $(N_1)_{60c\text{-}s}$ and undrained residual strength, s_r, from case studies (Seed and Harder, 1990).

less than 12. Most designers opted for strengths between the lower bound and the 33 percentile. Since these values were generally substantially less than would be given by triaxial compression tests, there was a considerable impact on seismic safety assessments and the extent of required remediation.

A re-evaluation of the liquefaction induced failure of the San Fernando dam which occurred during the 1971 San Fernando earthquake was undertaken by both Castro and Seed in 1986-1987 with the objective of resolving the uncertainties surrounding the determination of residual strength. Seed et al. (1989) reported that the average steady state strength of all samples tested in undrained compression was 5250 psf before correction for disturbance, and 800 psf after correction, a correction factor of about 6.5. The corrected average value did not allow the dam to fail by sliding instability in a static equilibrium analysis. The average residual strength obtained from back-analysis of the failed dam in the final configuration was 400±100 psf. The 35 percentile residual strength based on laboratory data would predict failure of the San Fernando dam. This suggests that, on the average, laboratory compression tests overestimate the residual strength. The San Fernando study did not resolve all the difficulties surrounding the determination of residual strength. However, use of the Seed (1987) chart for estimating reduced strength became widespread in engineering practice after this study.

FACTORS CONTROLLING RESIDUAL STRENGTH

Stress Path

Vaid and Chern (1985) showed that the residual strength measured in extension was much smaller than the strength in compression and that sands in a given state were much more contractive in extension than in compression. These differences are