International Conference on Case Histories in Geotechnical Engineering
(1984) - First International Conference on Case Histories in Geotechnical Engineering

09 May 1984, 9:00 am - 12:00 pm

General Report for Theme Five Case Histories in Geotechnical Earthquake Engineering

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There are 21 papers in this session. However, three listed papers, Nos. 513, 516 and 518, were not received in time for the general reporter's review, and thus the following comments relate to the 18 papers that were reviewed. These papers can be broken down into three categories, as listed below by paper number:

1. Case Histories Related to Actual Earthquakes:

Behavior of Pile Foundations	No. 503
Liquefaction in Level Ground	No. 504
Dam Failures	Nos. 517, 523
Intensity of Ground Shaking	No. 519
Rock Bursts	No. 522
Reservoir Induced Seismicity	No. 506

2. Case Histories of Soil Improvement for Seismic Design: No. 510

3. Description of Applications of Methods in Geotechnical Earthquake Engineering:

Seismic Stability Analysis of Dams	Nos. 501, 506,* 514,515
Liquefaction in Level Ground Sites	Nos. 505, 507 509, 502
Selection of Design Ground Motion	Nos. 512, 520
Seismic Zoning	No. 508

*Appears twice.

The first two categories represented by eight papers can be considered truly case histories which relate to observed field behavior. Category 3 includes more than half of the papers and deals with descriptions of seismic design or analysis for various purposes. We hope that more case histories in geotechnical earthquake engineering will become available in the future when earthquakes occur near earth structures or foundations for which there is a good background of prior geotechnical information, including predictions of their earthquake behavior. While we wait for such cases to occur, we must analyze failures after the fact with the danger that the analysis may be adjusted to "predict" a known occurrence. Naturally the tendency is to analyze failures even though much can also be learned from earthquake sites where structures behaved satisfactorily.

1. Case Histories on Earthquake Behavior

The case histories have served to confirm previous empirical or analytical knowledge as to how soils behave during earthquakes but have also revealed facts that do not reflect commonly accepted ideas.

Good behavior was shown by pile foundations supported by nonliquefiable soils, even when the overlying soils were loose and developed very high pore pressures during an earthquake, Paper No. 503 by Huishan and Taiping. An exception is when there is an opportunity for the loose soils to move horizontally and/or flow because of sloping ground or adjacent loads acting on the loose soils.

Observations of the behavior during earthquakes of saturated sands with a level ground surface range from no unsatisfactory behavior, to liquefaction; understood as a loss in strength evidenced by sinking of structures which apply a net downward load to soil or upward floating of structures which weigh less than the soil they displace. Other cases include limited shear deformations or some compression of soils leading to limited settlement of structures. In such cases, the deformations are small enough so that they do not decrease significantly the shear stresses that the structures apply to the soil, and thus one can infer that no loss in strength, i.e., liquefaction, has occurred. Sand boils are observed at the ground surface in almost all cases whenever loss in strength or deformations occur, indicating pore pressures at same depth. If no structures are present, one cannot ascertain whether sand soils are indicative of a loss in strength or of limited deformation. The term liquefaction will be used by this reporter to refer to cases in which loss of strength occurred. The term ground failure will be used to refer to all of the phenomena described above. Ground failures were associated only with sandy soils.

Paper 504 by Taiping et al. describes observations at 50 sand sites subjected to earthquakes. The Standard Penetration Test (SPT) is used as an index test for an empirical criteria for ground failure. The authors propose two modifications to similar existing criteria--namely, the effect of clay sizes in the sandy soil (defined as finer than 0.005 mm) and the introduction of a weight factor for the shallower sands that is used in analyzing a blowcount profile with low and high blowcounts. Specifically the soils deeper than 15 m are ignored in assessing the potential for ground failure.

A "liquefaction index" is defined as follows:

$$I = \int_0^{15} \left(1 - \frac{N}{N'}\right) W dz$$

where N = standard penetration test, blows/ft

z = depth in m

$$W = 10 - \frac{2}{3} z$$

$$N' = \overline{N}\,[1+0.125(d_s-3) -0.05(d_w-2)-0.07d_c]$$

$\overline{N}$ = 6 for Intensity 7
10 for Intensity 8
16 for Intensity 9

d_s = depth, m
d_w = depth of groundwater, m
d_c = clay content in %, but less than 10%

Sites are classified as follows in terms of liquefaction risk:

Low Risk	$I < 3$
Moderate Risk	$I = 3$ to 7
High Risk	$I > 7$

It has been commonly accepted that sand with fines are more resistant to earthquake loading than clean sands with the same blowcount. Seed and Idriss, 1981[1], following a study by Tokimatsu and Yoshimi separated sands into two groups based on the value of D_{50}, larger than 0.25 mm and smaller then 0.15 mm. Tokimatsu and Yoshimi, 1983[2], classified sands on the basis of percent finer than 0.074 mm (#200 sieve), larger than 10% and smaller than 5%. The work by Taiping et al., on the other hand, discriminated on the basis of percent finer than 0.005 mm.

Failures of embankment and tailings dams are described in Paper Nos. 517 and 523. The descriptions of the failures described in these papers correspond to flow slide-type failures involving liquefaction. The loss in strength is evidenced by the substantially smaller shear stresses indicated by the after failure geometry of the failed mass.

Winshao, Paper No. 517, describes the failures of several dams in China during earthquakes in the last 20 years. Xigeer Dam failed due to liquefaction of a silt layer in the foundation. The upstream slope was 1V to 12H, and the downstream slope was of 1V to 2H, and it failed in the downstream direction where the dam imposed higher shear stresses in the foun dation. The subject of the effect of static shear stresses on liquefaction will be discussed later in this report. Two dams, Wangwee and Yeyuan, had sand shells that were constructed by dumping and thus were loose. Liquefaction failures occurred during filling of the reservoir and, after reconstruction by the same procedures, failed during an earthquake. The static and earthquake failures had similar characteristics.

Shiman and Baihe Dams contained saturated upstream zones of gravelly sands that liquefied during earthquakes, thus confirming previous information that gravelly sands can liquefy if sufficiently loose.

Ishihara in Paper 523 described a study of the earthquake-induced failures of two dikes that impounded tailings in an upstream construction-type configuration. Silt size tailings had blowcounts of zero to two throughout most of its depth, while the dikes built of local soils consisting of silts, sands, and gravel had blowcounts of 4 to 5. Thus it is not surprising that liquefaction failures occurred during the earthquake. It is, however, remarkable that one of the dikes Dike No. 2, failed about 24 hours after the earthquake, while Dike No. 1 failed either during or shortly after the earthquake. Ishihara proposed an explanation for the dela in the failure of Dike 2 based on a gradual rise in groundwater level in the outer dike caused by water migration from the liquefied tailings. His computations indicate that in order for enough water to have flowed into th outer part of the dike in the 24-hour period the permeability of the dike should have been enhanced by crack development. Longitudinal cracks on the slope were noticed increasing width within about 4 hours preceding the failure of Dike 2, suggesting a phenomenon occurring at an increasing rate prior to the failure. The author's analysis of the stability of Dike 2 indicates that the increased pressure of the liquefied tailing would not be sufficient to fail the dike on the basis of its drained strength. This reporter suggests that it is possible that as the shear deformations accelerated in the dike material, its behavior changed gradually from drained to undrained. Since the dike materials are loose, their undrained strength would be low leading to a progressive-type failure and an increasing rate of deformation until a sufficiently large zone behaved undrained so th the failure was possible.

Zhao and Fang, paper 519, present a large collection of response spectra obtained duri the Tangshan earthquake. The spectra confir that at firm sites with thin soil cover, the earthquake periods are low, while they are th highest for the soft deep soil sites. Structural damage occurred accordingly, e.g., long period structures were damaged the most in th soft ground sites. Of interest is the appearance of two peaks in the response spectra of the soft ground sites, one at about 0.2 sec and another at 0.7 to 1.2 secs.

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[1] Seed, H.B. and Idriss, I.M., "Evaluation of Liquefaction Potential of Sand Deposits Based on Observations of Performance in Previous Earthquakes," Proc. of Session No. 24 of ASCE National Convention, St. Louis, Mo., Oct. 1981.

[2] Tokimatsu, K. and Yoshimi, Y., "Empirical Correlation of Soil Liquefaction Based on SPT N Value and Fines Content," **Soils and Foundations**,. Vol. 23, No. 4, Dec. 1983.

Zhao et al., Paper 506, presents evidence of reservoir-induced seismicity as a result of construction of the Hutmo River Dam. More comments on their paper in Section 3.

Srivaslava in Paper 522 presents a description of ground motions resulting form rock bursts or fractures occurring from mining operations. The ground motions are reasonably predicted from Bonilla's relationships between earthquake magnitude and length and displacement of fault ruptures.

2. Case Histories of Ground Improvement in Seismic Design

Paper 510 by Bahoe et al. describes the vibroflotation treatment of a clayey sand with about 25% of fines (finer than 0.074 mm) and about 10% of clay sizes (fewer than 0.005 mm). The designers found that sufficient improvement was obtained using 80-cm-diameter columns of gravel formed by vibroflotations with a spacing of 160 cm. The soil improvement was shown by average blowcounts increasing from 2.9 to 6.2, cone penetration from 9.2 to 33.9 kg/cm^2, and shear wave velocities from 190 to 240 cm/sec. The characteristics of the improved ground were considered acceptable for a 0.1 g design earthquake.

3. Descriptions of Application or Development of Methods in Geotechnical Earthquake Engineering

Analysis of seismic stability of dams are presented in four papers, Nos. 501, 506, 514, and 515.

In all cases the authors used some type of empirical blowcount correlation as one of the procedures for assessing liquefaction potential. It should be noted that the empirical criteria is based on experience with level ground sites. Two of the papers, 501 and 515, modify the criteria for sloping ground on the basis of the presence of shear stresses in the horizontal plane. The effect of the modification is to assume that the resistance to earthquake loading increases with the presence of shear stresses. To this reporters knowledge, there is no field evidence to indicate that such an assumption is correct, while in fact it would lead to the conclusion that a dam with steeper slopes would be safer. Apparent evidence to the contrary can be concluded from the case of Xigeer Dam in China, Paper 517, as discussed in a previous section.

In three of the papers, 501, 514, and 515, the authors rely on cyclic triaxial test results for determining liquefaction potential. The tests are used directly in the first two papers and indirectly in the last paper. The results of the triaxial tests are used to define failure or liquefaction as either 100% pore pressure or 5% strain. Since very often neither 100% pore pressure nor 5% strain result in a loss in shear strength, it is question-able whether the analyses relate to liquefaction failures of the flow slide type of which ample evidence has been presented in the first group of papers in this session. A loss in strength would be evidenced by the inability of the soil to withstand the applied load and would be triggered at various strains not necessarily 5%. Loss in strength is not indicated by the finding that 5% strain was reached in a particular cycle, since the soil will be able to support the next cycle of load regardless of the peak pore pressure in each cycle. The analysis methodology based on these test results addresses the question of how much deformation can occur **assuming** the soil has enough strength; and does not address the question as to whether failures of the type described in Paper 517 and 523 can happen. A differentiation of these two issues is crucial to the development of our ideas on how to analyze the seismic behavior of embankments and foundations.

Three papers on the analysis of the liquefaction potential of level ground sites (502, 507, and 509) deal with methods to predict pore pressure increases in a one-dimensional model of the soil. They consider soil compressibility, permeability, and the variations of moduli and damping with strain and effective stress. Soil properties for the model are obtained from shaking table tests, Gupta (502) while Oka (507) and Hyodo et al. (509) rely on cyclic triaxial tests.

Two papers, 512 and 520, deal with the selection of ground motion. The paper by Saragoni, 512, proposes a criteria for selection of earthquake motion for dynamic analysis of dams based on the "destructiveness potential factor":

$$P_D = 0.267\ g\ \frac{(\Delta t_s)\ [\ddot{u}_s(t)_{max}]}{\nu_o{}^2}$$

where $\ddot{u}_s(t)_{max}$ = peak ground surface acceleration
(Δts) = duration of strong notion
ν_o = zero crossing ratio

Of several possible earthquake motion records, Saragoni recommends to use those with the highest value of P_D and justifies the recommendation on the basis of the results of dynamic analysis of dams and soil deposits with various records, in which the highest P_D earthquake resulted in the largest earthquake stresses.

The paper by Nuttli and Herrman presents a good summary of various attenuation laws for the eastern and western U. S. and a discussion of the factors determining attenuation.

Seismic zoning is the subject of a paper by Ciuffi, 508, in which the geotechnical factors considered were slope stability, local seismic amplification factor and standard penetration of the soils.

The above discussion has focused on a few issues raised by the various papers that, in the opinion of this reporter, are important and timely for the engineer in the practice of designing earth structures for seismic loading. It is hoped that the general discussion that follows will deal with them and that a better understanding will emerge from the interchange of ideas.

Discussion by Pedro A. De Alba, Associate ›fessor of Civil Engineering, University of ∨ Hampshire, Durham, NH on: "Liquefaction ·ential Evaluation for Arcadia Dam" by J. gner; "Liquefaction Risk Evaluation During ·thquakes" by T. Qiao, C. Wang, L. Weng, and Liu; and "Liquefaction Potential of a Silty nd Site" by H. Dezfulian and N.D. Marachi.

The writer has chosen to discuss these pers together because all three deal with the oblem of evaluating the effect of fines connt on field liquefaction potential.

In the laboratory, Shen, Vrymoed and Ueno 977) have shown that, for the same void ratio the clean sand matrix, increasing the fines ntent in the voids increases resistance to quefaction, yet at the same overall density creasing fines content implies higher sand trix void ratio and lower resistance to quefaction. Both calculation and experiment ive shown that a fines content of perhaps 15-)% is enough to completely fill the voids of a and matrix with fines; higher fines contents ould imply that the sand matrix is at a void atio higher than its maximum clean-sand value nd liquefaction behavior would thus be conrolled by the characteristics of the fines. n this respect, we know from other studies e.g. Lee and Fitton, 1969) that liquefaction esistance increases rapidly with the plastiity of the fines. On the other hand, we also now that in a general way, for materials at he same overall density, the SPT blowcount is educed by increasing fines content. This is specially important since the SPT continues to e a basic tool for evaluating site liquefacion potential.

Efforts at accounting for the effects of 'ines have concentrated on correcting the blowounts in such a way as to obtain a blowcount equivalent to that which would be observed at a lean sand site with the same liquefaction resistance. This number can then be compared with a limiting curve separating clean sand sites which did or did not liquefy under the same conditions of ground shaking, as for example the well-known Seed Idriss and Arango (1983) curves.

Wagner has made this correction for the Arcadia dam materials comparing the field SPT values with the results of laboratory cyclic tests on materials with the same fines content. This process requires selecting an equivalent number of cycles for the design earthquake, finding the liquefaction stress ratio at that number of cycles and converting it to a blowcount normalized to an effective vertical stress of 1 tsf (N_1) through the Seed et al. curves. The procedure is summarized in Fig. 9 of the Paper. For the Arcadia dam materials, this additive correction, which depends on the design earthquake, ranges from 0 at 21% fines to 7.5 blows at 50% fines or more.

Several features of Fig. 9 are noteworthy; first, the equivalent laboratory resistance plots are essentially a horizontal line between about 25% and 50% fines. Thus, although the median blowcount is decreasing rapidly with fines content, the soil resistance remains roughly constant. Based on the previous discussion of lab results, 15 to 20% fines represents the range in which fines begin to control behavior, and this effect is reflected in the sharp drop in SPT values. Based on Wagner's data points, it might be argued that at fines contents exceeding about 25%, the sand matrix no longer plays any role and both the liquefaction behavior and the SPT values are controlled exclusively by the fines. It might also be noted that, based on these results, the Arcadia dam blowcounts at 50 to 60% fines content were as low as one to two blows/ft, yet their resistance to liquefaction was equivalent to that of a clean sand with N_1 of perhaps 14 blows/ft or more.

It is interesting to compare the trend of SPT blowcount decrease of Fig. 9 with that presented in a recent paper by Tokimatsu and Yoshimi (1983) for silty sand sites, Fig. D.1. The Arcadia N-values have been corrected from automatic trip hammer to cathead and rope SPT values, but unfortunately the magnitude of the correction is not stated in the Paper. The Tokimatsu and Yoshimi values were also automatic hammer values, and their suggested correction factor of 1.4 has been used to compare with the Fig. 9 median. The trends are seen to be in very good agreement, for field data obtained in such different environments.

The site-dependent blowcount correction procedure described in the Paper may also be compared with the general correction proposed by Tokimatsu and Yoshimi on the basis of liquefaction observations at many different sites.

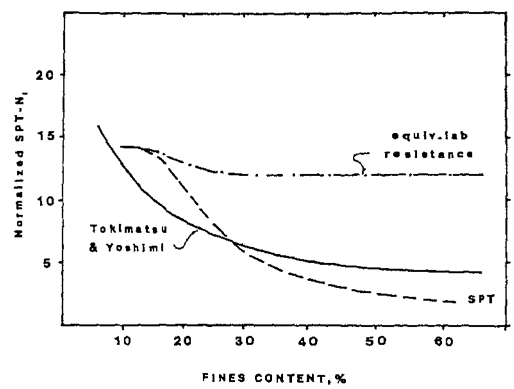

Fig. D-1. SPT-N_1 versus Fines Content (< #200 mesh)

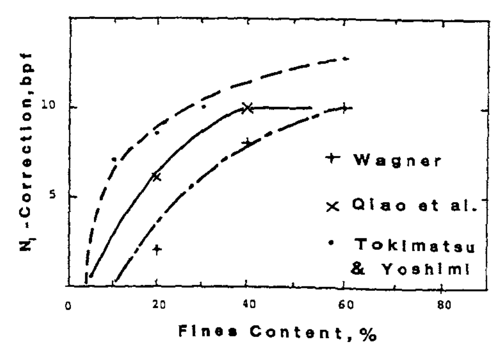

Fig. D-2. SPT correction ΔN_1 versus Fines Content

Since this correction is based on a M = 7.5 earthquake, the writer has obtained an equivalent laboratory resistance curve for this case, following the procedure described in the Paper, but suggesting that a curve of this type should converge with the field N_1-curve at low fines content, rather than pass underneath it as in Fig. 9 of the Paper. The lower laboratory resistance values measured at low fines content can be attributed to greater sample disturbance. When the blowcount corrections (ΔN_1) are compared on this basis with the Tokimatsu and Yoshimi values, Fig. D-2 shows that the Arcadia dam corrections are very conservative. It is especially noteworthy that collected field evidence would tend to show a dramatic increase in resistance between 5% and 10% fines content, leading to an additive correction of 7 blows/ft at 10% fines, which is not apparent in the Arcadia data. It might therefore be further speculated that both the laboratory and field curves of Fig. D-1 rise much more steeply at fines contents lower than about 20% and do not actually converge until the fines content is less than about 5%. If this assumption is correct, it would also imply that, even with the very careful sampling techniques described, it was not possible to obtain undisturbed samples of these medium dense to dense silty sands if fines content was less than about 20 to 25%.

Dezfulian and Marachi do not explain in their paper how, if at all, they corrected the normalized field N-values (N_1) for the effect of fines content before comparing them with limiting stress ratio vs. N_1 curves for clean sand. If no correction was applied, this analysis would be extremely conservative in view of the significant fines content of the materials involved. Further clarification by the authors is desirable.

In the paper by Qiao, Wang, Weng and Liu, a negative correction to the critical N-value for liquefaction of clean sand is used. This correction, in effect, shifts a limiting curve in terms of stress ratio vs. N_1 towards higher resistance for a given earthquake. It is based not on fines content but on clay-size content (d<0.005 mm) and becomes constant for clay content in excess of 10%.

In order to compare the proposed correction with those previously discussed, the writer has attempted to establish the variation of critical N_1 with stress ratio that would be predicted by this relationship for a M = 7.5 earthquake and an effective stress of 1 tsf. The equivalencing procedure is analogous to that suggested by Seed et al. (1983). It is assumed that the correction applies to rope-and-cathead SPT.

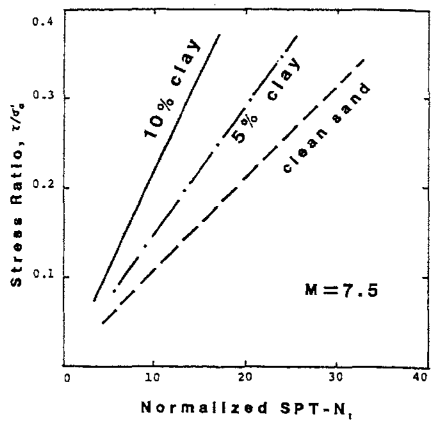

Fig. D-3. SPT-N_1 versus Limiting Stress Ratio.

Results are shown in Fig. D.3, for clean sands and for sands with 5% and 10% clay sizes. The equivalent blowcount correction for this method is shown in Fig. D.2, for a stress ratio of 0.2, considering that the range of stress ratios of greatest interest might vary from 0.15 to 0.25. It was further assumed that soils with 5% clay might have about 20% fines, and that soils with 10% clay would have at least 40% fines. Under these assumptions, it may be seen that the blowcount correction is of the same order as those previously discussed. It might also be observed that this approach, based exclusively on clay sizes, may give results which are overly conservative in soils with significant fines content but little clay-size material. A refinement to the critical N-formula presented might be to include the effect of fines coarser than clay size.

In general, the liquefaction index presented in this Paper seems a very rational way to integrate the liquefaction potential of different layers in the same profile. Field evidence at various sites indicates that it can discriminate between liquefying and non-liquefying areas at the same site. A case of special interest to the writer was that of silty sand liquefaction at the Shanggulin site in the Tangshan earthquake. The plasticity of the finer fraction is such that it would be classified as a CL (PI = 8.9; LL = 28.9) and the material exhibits an unconfined compressive strength equivalent to that of a medium clay; yet field evidence shows that it did liquefy.

In conclusion, it was very interesting to find that studies carried out in different locations seem to agree in a general way on the magnitude of correction to be applied to obtain the liquefaction resistance of materials containing fines. Since at fines contents above perhaps 20%, the liquefaction behavior is controlled by the characteristics of the fines, perhaps the plasticity index and the penetration resistance may be combined to produce a liquefaction indicator for these soils. In any case, the fines content will obviously remain an important and easily-determined parameter. Finally, the data suggests that careful conventional sampling may produce samples with minimum disturbance if the fines content is greater than about 30%. This raises the possibility of significant comparison between liquefaction indicators and laboratory cyclic resistance in these materials.

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Discussion by M. Hyodo
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on "Behavior of Some Earth Dams on Liquefiable Soil" by A. Popovici, M. Perlea and I. Corda

This paper presents interesting information on the characteristics of damages of levees for flood protection due to liquefaction of sand in foundation soil or in their body incurred by 1977 Vrancea Earthquake. Further presents the results of seismic analyses of earth dams underlain by liquefiable sand. The analyses are carried out in order to find the improvement points for preventing the damage. It is noteworthy that the effect of drainage blanket at the base of dam on liquefaction is discovered as a result of analyses.

However, the test data of material used in these analyses are not fully explained. Although there exist the initial static shear stresses in the elements of dams before earthquake, the values of which are obtained by the authors in the analyses, the cyclic shear test data under such stress states are not shown. The results of cyclic triaxial loading tests shown in Fig. 9 seem to be those performed under isotropic consolidation state. It is supposed that these analyses will need the data of residual pore water pressure and residual strain obtained from cyclic shear tests with initial static shear stresses.

As the other results of analyses, represented are the permanent displacements and the safety factors of slopes of dams. It is considered that the results will become more excellent if they are compared with the actual damages of slopes or dams occurred during the earthquake.

Discussion by H. Dezfulian,
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on "Seismic Response and Liquefaction by an Approximate Method" by M. Hyodo

The author presents a new simplified method of effective stress analysis. The dynamic response analysis as practiced today is a total stress method and does not take into account the effects of increasing pore water pressures on the shear stress response. As pore water pressures rise and nonlinearity becomes more pronounced, it becomes more desirable to consider nonlinear effective stress methods of analysis. Furthermore, deformations cannot be computed directly by equivalent linear methods but are inferred from laboratory data by the equivalent approach a procedure which results in a noncompatible distribution of deformations. The method developed by the author is certainly a welcome step toward developing new tools to obtain additional insight into the response of soils to earthquake loading.

The basic requirement of effective stress analysis is a reliable pore water pressure generation model. The author has made use of the Hardin-Drnevich model (1972) which is claimed to be "one of the fittest models with a few parameters." The procedure involves the performing of a total stress analysis in order to obtain shear moduli and damping factors corresponding to specified strains. These values become the initial constants of the effective stress analysis conducted subsequently. It is assumed that the degradation of the shear modulus due to an increase of the magnitude of shear strain and reduction of the effective stress are independent of one another. The validity of this assumption ought to be established.

A hypothetical saturated sand site is studied to determine how closely the proposed analysis procedure can approximate the nonlinear result. The results of the nonlinear and equivalent linear methods of analysis are shown to agree well regardless of the input acceleration. A comparison of the time history of pore water pressure in the top element of the ground indicates somewhat different rates of pore pressure buildups, although the two methods result in comparable pore water pressures after some time. A comparison of the surface acceleration computed by the two methods shows definite points of similarity, although the equivalent linear approximation does not reproduce the short-term components of motion present in the nonlinear solution.

In conclusion, it appears that the proposed method is certainly a step toward the development of nonlinear stress-strain relations for soils and effective stress methods of analysis. The single hypothetical rather than real-world case analyzed in the present study renders the conclusions reached by the author tentative and it would be interesting to see what the results of a parametric study would reveal.

Discussion by John P. Sully, Principal Geotechnical Engineer, INTEVEP, S.A., Venezuela on 'Liquefaction Potential Evaluation for Arcadia Dam' by J. Wagner and 'Assessment of Seismic Stability of Earth Dams by Comparative Methods' by R.J. Huang and M.L. Silver.

Both of the above papers use the method proposed by Seed (1979) for evaluation of liquefaction susceptibility of level ground and, using a modified form, apply this method to assess the susceptibility of sloping ground; the obvious difference between the two situations being the presence of initial static shear stresses beneath a slope.

Based on the methodology proposed by Seed (1983), the effect of initial static shear stress is taken into account by a factor whose value relates to the magnitude of initial shear stress; the larger the initial shear stress the larger the factor, which thus suggests an increase in resistance to liquefaction as the level of static shear stress increases.

Seed's initial results were obtained from analysis of the San Fernando Dam using cyclic load tests with reversal of shear stress. While this indeed may be the case at low stress levels, depending on the magnitude of ground shaking, it will not necessarily hold at higher stress levels where reversal may not occur. The effect is also dependent on wether the total shear stress (initial static shear stress plus cyclic shear stress) exceeds the undrained steady-state shear strength.

For more detailed aspects of the above the reader is referred to Vaid & Chern (1983) and Mohamad and Dobry (1983). The effect of static shear stresses on resistance to liquefaction can thus be summarised as:

- in contractive sand with stress reversal, increased resistance to liquefaction is obtained as the initial shear stress is increased, provided that the steady-state shear strength is not exceeded.
- in contractive sand with non-reversal of stress, resistance to liquefaction decreases as the initial shear stress increases.

The effect of the two above conditions suggest that as the initial shear stress increases from zero under stress reversal, the resistance to liquefaction increases as the degree of reversal diminishes. A transition will then occur as stress reversal stops and the total shear stress begins to exceed the steady-state strength. Above this transition, the resistance to liquefaction will always decrease as the initial shear stress increases.

- in dilative soils, an increase in initial shear stress will almost always increase the resistance to liquefaction.

In view of the above it is thus apparent that using the authors' method for evaluation of seismic stability may give erroneous results for deep-seated failure surfaces where the steady-state undrained shear strength is overestimated by the assumption of increased resistance to liquefaction per se as the level of initial static shear stress increases.

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Discussion by H. Dezfulian, Department of Civil Engineering San Diego State University San Diego, California on "Liquefaction Risk Evaluation During Earthquakes" by Qiao Taiping, et al.

The authors have presented a very interesting paper in which a simplified method of liquefaction risk evaluation is discussed. The method attempts to include the effects of such factors as soil density, thickness and location of the liquefiable layers, and shear resistance of soils. Liquefaction index is defined as a function of depth, thickness, and SPT blow count for the liquefiable layer. The current Chinese Aseismic Design Code for Industrial and Civil Buildings is employed in which the maximum depth at which liquefaction is possible is considered to be 15 m. The correlation given by the Chinese Code for N_{crit}, the SPT resistance separating liquefiable conditions from non-liquefiable conditions, is somewhat different from that quoted by Seed, et al. (Journal of Geotechnical Engineering, ASCE, March 1983) in that a clay content factor is now added which appears to reflect the latest development in the Chinese Code. Based upon the investigations of structural damage induced by soil liquefaction during the Tangshan earthquake of 1976 (Magnitude 7.8), four categories for the evaluation of liquefaction risk is proposed.

A detailed table lists boiling conditions and structural damage at some 50 sites located in Tianjing and Tangshan counties. On the basis of the data presented, a classification system for liquefaction risk is proposed. Three of those sites are studied in detail.

The liquefaction index is an attempt to provide a preliminary estimate of the liquefaction risk at a site as well as the degree of structural damage. As noted in the paper, the problem of structural damage due to soil liquefaction is related to not only the soil conditions but the features of the structure and foundation as well. The paper is mainly concerned with soil conditions and further study considering structure and foundation features would be needed.

Discussion by H. Dezfulian,
Department of Civil Engineering
San Diego State University
San Diego, California
on "Liquefaction Potential Evaluation
for Arcadia Dam" by John Wagner

An interesting, detailed discussion of a case study involving the liquefaction potential of an earthfill dam on a sand foundation is presented. The author is to be commended for providing a clear, complete description of drilling, sampling, sample handling, and laboratory testing as well as an account of the decisions, engineering judgments and procedures used. Such detail is obviously essential to any complete account of a case study.

In applying the Seed-Idriss's simplified procedure for evaluation of liquefaction potential, two modifications were made. The first of these modifications concerns the initial horizontal shear stresses induced in the foundation by the dam embankment. A relationship was developed and used to correct the calculated stresses induced by the earthquake for the presence of initial static shear stresses. This is a valid correction and should certainly be considered for similar projects.

The second modification accounts for the high silt content of the potentially liquefiable soils. The modification was to increase the corrected SPT blow count by a value ranging from 0 for silt contents of less than 21 percent to a maximum of 7.5 for silt contents of 50 percent or more. The latest version of the Seed-Idriss's procedure (Journal of Geotechnical Engineering, ASCE, March 1983) recommends increasing the corrected blow count by 7.5 for silty sands and silts plotting below the A-line and with $D_{50} < 0.15$ mm. Chang, et al (3rd Microzonation Conference, Seattle, 1982) have shown that the cyclic shear resistance increases over that of the parent sand as the silt content increases and that the rate of this strength increase is greatly reduced as the silt content increases beyond 60 percent. This of course substantiates the nature of the modification discussed by the author.

In addition to the above, the writer wishes to make the following comments:

(1) It would be interesting to include a short discussion of how the results of the automatic drop hammer with a free falling weight used for the SPT program was corrected to estimate blow counts using the rope and cathead type of equipment.

(2) It is noted that the initial effective confining pressure which varied from 1 to 5 tsf had negligible effect on the laboratory cyclic strength. The writer's experience is that such effect could be significant for some sites.

(3) The results of the relative density tests are considered quite rightly by the author to be inconclusive as such results are not expected to be meaningful for soils similar to those found at the site. The SPT blow counts could possibly be used to estimate the range of the relative density of the site soils in a procedure similar to that developed at the Waterways Experiment Station (Marcuson and Bieganowski, Journal of Geotechnical Engineering, ASCE, June 1977).

Authors replies to discussion by M.Hyodo on "Behaviour of Some Earth Dams on Liquefiable Soil" by A.Popovici, V.Perlea and I.Corda

The writers appreciate Hyodo's comment on the disagreement between the expected stress state in the elements of dams before earthquake and the isotropic consolidation state applied to samples in laboratory tests.

This type of test has been adopted for two reasons: (1) elements in dam where computed initial static shear stress has important weight are to be built of unliquefiable material, either due to its grain size distribution, or due to a proper compaction; in perilous zones in foundation soil however, this weight is smaller; (2) in order to obtain liquefaction in laboratory tests, modelling reasonable close the field seismic load, it is convenient to perform cyclic triaxial tests on isotropic consolidated samples; the expected anisotropic stress state and initial shear stress on horizontal planes in the field may be taken into account by correction factors applied to laboratory results.

So, instead of performing distinct laboratory tests for stress condition in every element, the results obtained on samples isotropic consolidated have been corrected for the expected conditions according to the formula:

$$\frac{\tau_{max,\ell}}{\sigma'_v} = \frac{1 + 2(\sigma'_3/\sigma'_1)}{3} \cdot \frac{\sigma_{d\ell}}{2\sigma'_0}$$

where: $\tau_{max,\ell}/\sigma'_v$ is the maximum stress ratio in a soil element, $\sigma_{d\ell}/2\sigma'_0$ - the cyclic stress ratio at liquefaction in an isotropically consolidated sample, σ'_3 and σ'_1 - minimum and maximum principal effective stresses. This formula represents an extension, for taking into account the influence of initial shear induced by dam loading, of a formula recomanded by Ishihara et al (1977).

The procedure is, of course, a rough one, but has been considered acceptable as compared with other allowed approximations.

In fact, the problem of the influence of the initial static shear stress on liquefaction potential remains controversial. On the other hand, very sophisticated laboratory tests need complex equipment and may be subjected to experimental errors.

Another problem discussed by M.Hyodo is on the need of a comparison between computed permanent displacements and the actual damages of slopes of dams occured during the earthquake. It must be emphasized that the strong earthquake of March 4, 1977 induced damages to some embankments in the proximity of the analysed works site, but these works were not even projected at that time.

REFERENCE

Ishihara, K., S.Iwamoto, S.Yasuda and H.Takats (1977). "Liquefaction of anisotropically co solidated sand", Proc., 9-th International Conference on Soil Mechanics and Found.Engr Tokyo, vol.2, 261 - 264.

Closure by J.R. Wagner
Chief, Soil Mechanics Section
Tulsa District, Corps of Engineers
on "Liquefaction Potential Evaluation for Arcadia Dam"

The writer would like to thank the general reporte and the discussers for their valuable comments on the paper.

As pointed out, the magnitude of the N-value correction from automatic trip hammer to cathead and rope SPT values was not provided. The correction used for this study increased the blow count obtained with the automatic trip hammer by 30 percent. This factor was chosen based on a review of the literature and is conservative for this particular hammer.

The writer is especially grateful for the remainde of the comments which discussed the potential errors in the evaluation procedure as applied to Arcadia Dam. Th discussion topics included: the difficulty in obtainin undisturbed samples of clean sands; the appropriateness of the shear stress correction; and the definition of failure. Each of these subjects along with many others had to be considered during the evaluation. Since the state-of-the-art provided no finite answers, decisions based on judgement had to be made. The primary purpose of the paper was to discuss the various decisions required for the evaluation. The discussers have made a significant contribution to this purpose by pointing out additional uncertainties behind many of the decisions.

Reply by H. Dezfulian, Department of Civil Engineering, San Diego State University, San Diego, California, to the discussion by Pedro A. DeAlba on "Liquefaction Potential of a Silty Sand Site" by H. Dezfulian and N.D. Marachi.

The authors wish to thank Professor Pedro A. DeAlb for his discussion of their paper. In our paper we clearly note that the method of analysis used was the procedure advanced by Seed, et al. (1983) in which the effect of silt contents on the Standard Penetration Resistance is considered by increasing the corrected N-values in accordance with the following relation:

$$N_1 = (N_1)\ \text{measured} + 7.5$$

The authors are well aware of the increased resistance to liquefaction attributable to silt contents of a sandy material. This effect should be considered not only in empirical procedures such as the one employed i the present paper, but also in analytical-laboratory test approaches in which comparisons are made between the induced stress conditions from earthquakes and stres conditions causing liquefaction of the same soil in the laboratory.