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AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

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TECHNICAL NOTES

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DISCUSSION

Proc. Paper 14116

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I certify that the above statements made by me are correct and complete.—Richard R. Torrens, Editor of Technical Publications.

14133 LIQUEFACTION POTENTIAL OF HYDRAULIC FILLS

KEY WORDS: California; **Earthquake damage;** Earthquakes; **Harbors; Hydraulic fills;** Land use; **Liquefaction;** Sands; **Soil liquefaction**

ABSTRACT: The potential for future earthquake-induced liquefaction in the hydraulic fills of this important harbor area are examined with the aid of several recently proposed detailed and simplified procedures. In addition, the validity of these procedures is assessed by applying them in liquefaction analysis of the hydraulic fills existing in the harbor during the 1933 Long Beach Earthquake. The study suggests there is something less than a 50% chance of liquefaction occurring during the useful lifetime of these facilities. The consequences of liquefaction will depend on the nature of land use at the time.

REFERENCE: Pyke, Robert M., Knupple, Lee A., and Lee, Kenneth L., "Liquefaction Potential of Hydraulic Fills," *Journal of the Geotechnical Engineering Division,* ASCE, Vol. 104, No. GT11, **Proc. Paper 14133,** November, 1978, pp. 1335-1354

14141 DOWNDRAG ON BITUMEN-COATED PILES

KEY WORDS: Bitumens; **Bituminous coatings;** Coatings; Consolidation; **Downdrag;** Foundations; **Negative skin friction; Pile foundations;** Pile friction; Settlement records

ABSTRACT: The performance of bitumen-coated piles in a number of case histories shows that bitumen coatings can be used effectively to reduce negative skin friction on piles. The shearing behavior of coated steel piles subjected to downdrag loads is investigated by means of laboratory tests. Based on the test results, a method for predicting downdrag loads on bitumen-coated piles is developed, and the important factors controlling coating effectiveness are identified. Predictions are than compared to field measurements.

REFERENCE: Baligh, Mohsen M., Vivatrat, Vitoon, and Figi, Heinrich, "Downdrag on Bitumen-Coated Piles," *Journal of the Geotechnical Engineering Division,* ASCE, Vol. 104, No. GT11, **Proc. Paper 14141,** November, 1978, pp. 1355-1370

14170 STRESS HISTORY AND SAND DEFORMATION

KEY WORDS: **Cohesionless soils;** Cone penetrometers; **Modulus of deformation;** Prestressing; **Sands;** Shear strength; Soil mechanics; Stresses; **Stress-strain curves**

ABSTRACT: The prestraining induced by prestressing, without residual lateral stresses, was found to increase the deformation resistance to incrementally applied axial load by an order of magnitude. The stress level at which a prestressed sample's load-deformation behavior changed from the stiffer, essentially elastic reloading mode to that which was predominantly plastic and resembled the normally consolidated state depended upon the magnitude of the prestress (or prestrain). The relationship between initial modulus and confining pressure for anisotropically consolidated, non-prestressed samples was found to be essentially linear. Evaluation of the changes in stress-strain response due to prestressing was also attempted using a model cone penetrometer.

REFERENCE: Lambrechts, James R., and Leonards, Gerald A., "Effects of Stress History on Deformation of Sand," *Journal of the Geotechnical Engineering Division,* ASCE, Vol. 104, No. GT11, **Proc. Paper 14170,** November, 1978, pp. 1371-1387

14173 SITE DEPENDENT EARTHQUAKE MOTIONS

KEY WORDS: Computers; Dynamics; **Earthquake resistant structures;** Earthquakes; Geology; **Ground motion;** Powerplants; **Site selection;** Soil mechanics; **Structural analysis**

ABSTRACT: A statistical method for modeling time varying earthquake induced acceleration levels which will produce smooth response spectrums of the desired shape and amplitude levels is developed. Specific records are derived to simulate a mean spectrum developed from historically recorded rock motions and also for AEC Regulatory Guide spectrum. The synthetic rock motion is then used as the input motion to a number of site conditions simulating stiff, deep cohesionless and soft to medium clay and sand sites using the computer program SHAKE. The results are compared to mean spectra for similar sites derived from historically recorded motions and shown to provide reasonable engineering estimates of site motions.

REFERENCE: Romstad, Karl M., Bruce, John, and Hutchinson, James R., "Site Dependent Earthquake Motions," *Journal of the Geotechnical Engineering Division,* ASCE, Vol. 104, No. GT11, **Proc. Paper 14173,** November, 1978, pp. 1389–1400

JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

Liquefaction Potential of Hydraulic Fills

By Robert M. Pyke,[1] M. ASCE, Lee A. Knuppel,[2] A. M. ASCE, and Kenneth L. Lee,[3] M. ASCE

Introduction and Background

The combined ports of Los Angeles and Long Beach constitute one of the largest harbor facilities in the world with a total invested value exceeding $1.8 billion (1976). Together, the ports currently handle about 62,000,000 tons of cargo annually, carried by about 28,000 ships, netting some $468,000,000 for the Federal treasury collected by U.S. Customs.

Construction of the port facilities began about 1872, but major expansion involving hydraulic fills of dredged material began in the 1920's and has continued to the present. An aerial photograph showing the present harbor facilities is presented in Fig. 1. The port authorities plan to continue expansion of the harbor facilities by essentially the same hydraulic filling techniques as used in the past; dredging deeper shipping channels and creating additional land fill that will extend out to the present breakwater and, perhaps, even beyond.

The harbors are located in an area of known high seismic activity, and hydraulically filled sands are well known for their potential instability during earthquakes as a result of the phenomenon of liquefaction under cyclic loading. Thus, the various agencies involved in the development of the two harbors share a concern that the potential for liquefaction of these fills be evaluated realistically so that the risk involved in constructing new facilities can be better understood.

In this paper, data from three recent studies are combined in a general evaluation of the liquefaction potential of the harbor fills. The earliest of these studies was conducted by the second writer at the University of California, Los Angeles, Calif. (UCLA) (11). It included a compilation of previously obtained field data

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as well as original field, laboratory, and analytical studies. The second study was conducted in 1975 by Dames & Moore as part of an investigation of alternate design and construction procedures for a proposed new hydraulic fill in the Port of Los Angeles (9). At that time the first writer was a project engineer for Dames & Moore and directed the earthquake engineering portion of that study. The third writer served as thesis chairman for the UCLA study (11) and served as a consultant to Dames & Moore on the earthquake engineering portion of their studies (9). Additional work has been conducted jointly by the three writers following completion of the original studies, and some additional data are included from a third study conducted by Dames & Moore in late 1976 concerning the potential for liquefaction at a site on Pier J in Long Beach (6).

The generalized results presented in this paper are not intended to replace the need for detailed foundation engineering studies for particular facilities, but rather to put the liquefaction problem in these hydraulic fills in perspective

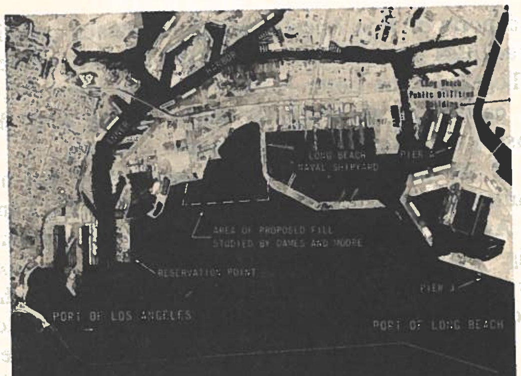

FIG. 1.—Aerial View of Los Angeles and Long Beach Harbors

using current procedures. In addition to an evaluation of the potential for liquefaction of present and proposed hydraulic fills, the paper also includes data on the performance of hydraulic fills that existed at the harbor during the 1933 Long Beach earthquake.

Typical Form of Construction

A form of construction that has commonly been used in the harbor area is shown in Fig. 2. In this form of construction, hydraulic fill is placed behind a containment dike composed of crushed rock material, known locally as quarry waste. In order to minimize the volume of the quarry waste, which is relatively more expensive than the hydraulic fill, the containment dike is normally brought up in several triangular shaped lifts. The hydraulic fill has been obtained in some instances by dredging existing channels and in other instances by dredging new shipping channels adjacent to the new fills. In the latter case, a dredged slope is formed in the natural soils that underlie the containment dike.

The new land formed by hydraulic filling is relatively weak and compressible but is nonetheless adequate for many harbor functions. When used to support special structures, the upper part of the hydraulic fill is either compacted or piles are driven to underlying firmer natural soils. Should, however, liquefaction of the hydraulic fill occur as a result of earthquake shaking, the resulting temporary loss of strength of the fill and subsequent settlement of the ground surface might cause significant damage.

The possibility that the containment dike and the underlying dredged slope might fail and block shipping channels should also be considered in an overall evaluation of this form of construction. Detailed consideration of the nature of the natural soils in the harbor area and the seismic stability of slopes dredged in these soils are beyond the scope of this paper but it is believed that the dredged slopes will generally be less subject to instability due to liquefaction under cyclic loading than the hydraulic fills (18). The quarry waste material used in the construction of containment dikes is relatively free draining and it is not expected to be susceptible to liquefaction. However, should liquefaction of the hydraulic fill occur, the upper lifts of quarry waste will tend to settle and the hydraulic fill will tend to spread laterally, possibly resulting in rupture of the containment dike.

FIG. 2.—Typical Section through Containment Dikes

It should be emphasized that the fact that liquefaction of these hydraulic fills could lead to significant damage does not necessarily mean that damage will occur. The probability of damage occurring is a function not only of the particular use of each fill but also of the probability that liquefaction will occur at all.

Description of Harbor Hydraulic Fills

Hydraulic fills are usually relatively loose and nonhomogenous. The existing hydraulic fills in the harbor area are no exception, containing sand, silt, and even some clay in combinations that vary both vertically and horizontally in a given fill.

Because of this variability and because much of the material is quite fine, it is difficult to determine the relative state of compactness of these materials. However, the various data available to the writers indicate an average relative density for the more sandy materials in the order of 55%–60%. These values are towards the upper bound but are not atypical of relative densities normally quoted for hydraulic fills (27,28).

Additional information on the fills existing at the time of the 1933 Long

Beach earthquake was obtained by drilling borings on Reservation Point and Pier A, Long Beach (18). These sites are shown in Fig. 1. A unique feature of the Pier A site is that it falls within the area subjected to subsidence as a result of production from the Wilmington Oil Field (15). As a result, some 20 ft (6 m) of additional fill now overlies the original hydraulic fill. The Pier A hydraulic fill consists primarily of silty sand, similar to that found in many of the more recent fills. By contrast, the Reservation Point fill is somewhat atypical, containing alternating layers of more clayey materials and more coarse-grained sands.

Laboratory Testing of Harbor Hydraulic Fills

A number of stress-controlled cyclic triaxial tests were conducted on both undisturbed and recompacted samples of hydraulic fill. The gradations of the

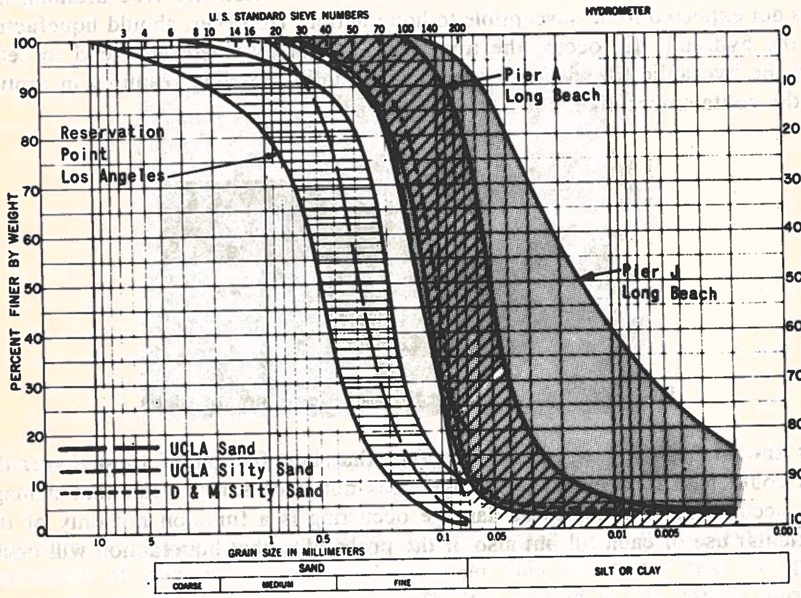

FIG. 3.—Gradations of Samples Selected for Cyclic Testing

materials for which results are presented in this paper are shown in Fig. 3, and it may be seen that they range from medium sands to clayey silts. All samples were isotropically consolidated, the undisturbed samples being reconsolidated under a stress equal to the estimated vertical effective stress in the field.

UCLA Tests on Recompacted Samples.—Tests were conducted at UCLA on two materials, a fine to medium sand and a silty sand. The gradations of these sands are shown in Fig. 3 and their maximum and minimum densities are shown in Fig. 4. Tests samples 2.8 in. (71 mm) in diameter and 6 in. (152 mm) high were prepared at a dry density of 95 pcf (1,520 kg/m^3), equivalent to 60% relative density, by sedimentation through water. A preweighed quantity of dry sand was saturated by boiling in a flask. After cooling, the soil was poured

under water into a membrane lined mold. Some silt would remain as residue in the flask, but this was later recovered and introduced into the water used to fill the mold for the next sample, so that the silt from the preceding sample always replaced the silt that was lost in preparing the current sample. The samples were brought to the required test density by vibrating the sides of the forming mold.

The results of the UCLA tests are summarized in Figs. 5(*a*) and 5(*b*) where the number of cycles of loading causing 5% single amplitude strain is shown as a function of the cyclic stress ratio, $\sigma_d\ 2\sigma_{3c}$, in which σ_d = cyclic deviator stress; and σ_{3c} = initial effective confining pressure. Tests were conducted at two or three different confining pressures for each sand and it may be seen that the cyclic stress ratio causing liquefaction in a given number of cycles decreased slightly as the confining pressure increased.

Dames & Moore Tests on Recompacted Samples.—Tests were conducted by Dames & Moore on a bulk sample obtained from the proposed dredge area

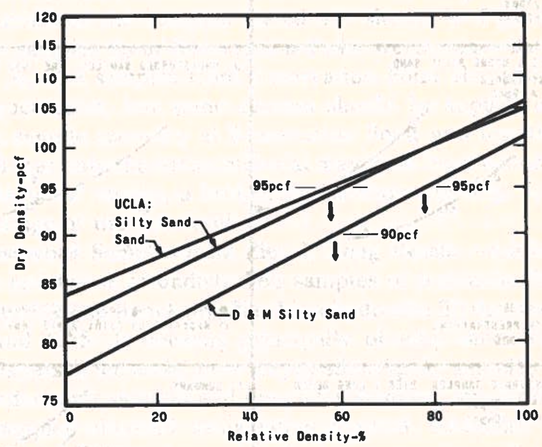

FIG. 4.—Relative Density Chart for Bulk Samples

in their study. Again, the gradation of this sample is shown in Fig. 3, and its maximum and minimum densities are shown in Fig. 4. Tests were conducted at dry densities of 90 pcf (1,440 kg/m^3) and 95 pcf (1,520 kg/m^3) corresponding to relative densities of 60% and 80%. The 60% relative density samples were 5 pcf (80 kg/m^3) lighter than the equivalent UCLA samples, although the relative densities were the same. The difference in the maximum and minimum densities of the two silty sands that have similar gradations is probably due to a higher mica content in the Dames & Moore material.

Test samples 2.4 in. (61 mm) in diameter and 5.6 in. (142 mm) high were prepared by a procedure that has come to be known as moist tamping. Following this procedure, moist sand was tamped into a membrane lined mold in six layers using a tamping rod with a diameter of 0.75 in. (19 mm).

The results of the Dames & Moore tests are shown in Fig. 5(*c*). Results are shown for three test series all conducted at the same confining pressure. In addition to the two basic series at 60% relative density and 80% relative density, a second series was conducted at 60% relative density in which the samples were prestrained by application of 20 load cycles with the cyclic stress ratio equal to 0.2. This was intended to simulate the effect of a smaller earthquake preceding one large enough to cause liquefaction. Based on previous work [e.g. Finn, et al. (8) and Lee and Focht (13)], it was expected that this would increase

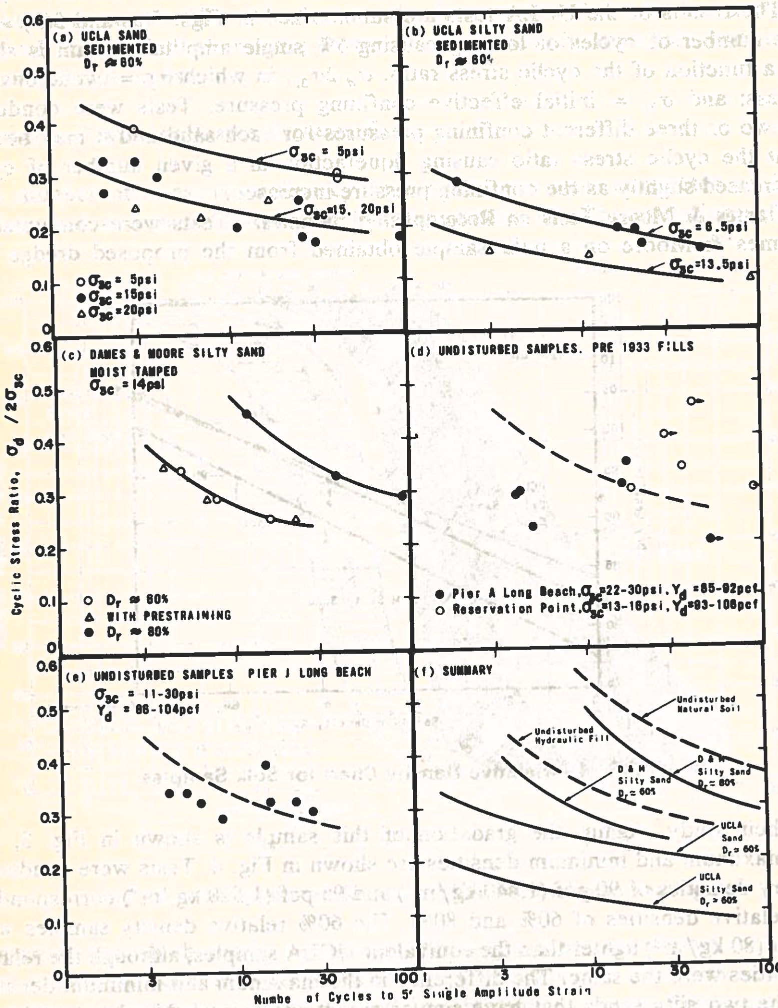

FIG. 5.—Results of Cyclic Triaxial Tests (1 psi = 6.89 kN/m^2; 1 psf = 16 kg/m^3)

the cyclic stress ratios required to cause liquefaction, but, as may be seen from Fig. 5(*c*), in this case the effect of prestraining was negligible.

It is believed that the lack of strength increase with prestraining results from a feature of the sample preparation method not yet explained, i.e., that a vacuum approaching 14 psi (97 kN/m^2) was applied to the samples while the forming

mold was removed and the triaxial chamber was assembled. It is customary to apply at least a small "stand-up vacuum" at this point, and it was found in preliminary testing that for this soil the resistance to liquefaction increased with the magnitude of the vacuum. It was then decided to use the maximum vacuum that could be applied in the remaining tests in the belief that this might simulate the increase in stiffness and liquefaction resistance that has been found to occur with the sustained application of consolidation pressure, even for sands (2,16). While the reasons for these increases in liquefaction resistance are not well understood, it appears that, for this soil at least, once the resistance has been artificially increased by application of a large vacuum, prestraining then causes no further increase. Subsequent tests actually showed that if a smaller stand-up vacuum was used, prestraining did increase the resistance to liquefaction.

Tests on Undisturbed Samples from Pre-1933 Fills.—The results of tests conducted on undisturbed samples taken from Pier A, Long Beach, and Reservation Point are shown in Fig. 5(*d*). Samples were obtained with both the Dames & Moore underwater sampler and the Dames & Moore piston sampler. The ranges of gradations of the samples tested are shown in Fig. 3, and it may be seen that the samples from Reservation Point were somewhat coarser than those from Pier A. The samples from Reservation Point also showed a greater resistance to liquefaction, but some caution should be applied in interpreting these results, as sample recovery at Reservation Point was less than desirable, and it is believed that only the better material may have been tested. To facilitate comparisons with other results, a broken line is shown on Fig. 5(*d*) to indicate a reasonable average of the test results.

Tests on Undisturbed Samples from Pier J, Long Beach.—Additional data on the liquefaction resistance of undisturbed samples of hydraulic fill is available from a site on Pier J, Long Beach (6). Pier J is a hydraulic fill that was constructed between 1962 and 1965. It consists principally of silty sands or sandy silts, with occasional layers or lenses of more clayey silts. Again, samples were obtained with both the underwater sampler and the piston sampler, similar densities being measured for samples obtained using either method. Excellent recovery was obtained. A limited number of samples that were representative of the range of gradations and the average density of the fill were selected for testing.

The range of gradations of the samples that were tested are shown in Fig. 3, and the results of the tests are shown in Fig. 5(*e*). There is less scatter in these results than was the case for the pre-1933 fills, and it may be seen that the broken line that was shown in Fig. 5(*d*) for the pre-1933 fills also represents a reasonable average for the results shown in Fig. 5(*e*) for the Pier J fill. While the UCLA tests on recompacted samples showed a variation in the cyclic stress ratio causing liquefaction with confining pressure, no significant trend could be seen in the results of the tests on undisturbed samples, and they may be considered applicable over a range of depths from 20 ft–40 ft (6 m–12 m).

Summary of Cyclic Triaxial Test Results.—The results of all the cyclic triaxial tests are summarized in Fig. 5(*f*). There is some variation in the confining pressures for the tests that are represented, but most are in the order of 15 psi (104 kN/m^2).

For the recompacted samples, it may be seen that the UCLA sand was more resistant to liquefaction than the UCLA silty sand, but that the Dames & Moore silty sand was stronger than both the UCLA materials. In particular, the cyclic stress ratio causing liquefaction in a given number of cycles was more than twice as great for the Dames & Moore silty sand, for which test samples were prepared by moist tamping, than for the UCLA silty sand, for which samples were prepared by sedimenting through water. This difference is attributed to the effect of the two methods of sample preparation rather than to any significant difference in basic properties of the sands. Similar effects of sample preparation have been reported by Mulilis, et al. (17).

For undisturbed samples, the lower broken line shown in Fig. 5(*f*) is generally representative of the results of the tests on undisturbed samples that were shown in Figs. 5(*d*) and 5(*e*). Its relative position indicates a slightly greater resistance to liquefaction for the undisturbed samples than for the moist tamped silty sand.

The results of the Dames & Moore tests prepared by moist tamping to 80% relative density, as well as tests on undisturbed samples of the natural foundation soils with a similar gradation and density (9,18), are summarized in Fig. 5(*f*). Comparing these two sets of data, the undisturbed samples show a slightly higher resistance to liquefaction than the recompacted samples.

From the trend shown herein, i.e., that the cyclic strength of undisturbed soils is greater than that of recompacted samples at the same density, and from similar trends reported elsewhere (16) with regard to effects of sample disturbance on cyclic strength, it is possible that the long-term resistance to liquefaction of elements of soil in the field might be even greater than that of the undisturbed samples. On the other hand, the results obtained for the silty sand recompacted by moist tamping already include, to some extent at least, the effects of long-term consolidation and prestraining. Therefore, the long-term field strength was not considered to be significantly greater than that shown by the undisturbed samples. It might be noted, however, that the immediate strength of a newly placed hydraulic fill may be slightly less than is indicated by the laboratory test results on the undisturbed or moist tamped samples.

An interesting feature of these results is that strengths from the moist tamped samples were in better agreement with the results from the undisturbed samples than were the strengths from the sedimented samples. This observation is perhaps surprising since, at first thought, the process of sedimenting through water may appear to more closely approximate the field placement conditions that does the moist tamping procedure. However, the laboratory sedimentation procedure in a small mold is not necessarily a good representation of the field hydraulic fill placement conditions. In view of this observation, and in the absence of more data, caution should be exercised in using laboratory sedimentation for quantative representation of field conditions.

Correction of Cyclic Triaxial Test Results to Field Conditions

Although future practice may tend to change toward the use of cyclic simple shear tests or other techniques to evaluate liquefaction characteristics, the current state-of-the-art for evaluation of liquefaction potential relies principally on the use of cyclic triaxial test results. However, in recognition of the many factors that are different in the field and in the laboratory, a semi-empirical correction

factor, C_r, is used to convert cyclic triaxial data into equivalent field cyclic strengths:

$$\left(\frac{\tau_h}{\sigma_{vc}}\right)_{\text{field}} = C_r\left(\frac{\sigma_d}{2\sigma_{3c}}\right)_{\text{lab}} \quad \dots\dots\dots (1)$$

in which τ_h = the horizontal cyclic stress causing liquefaction; and σ_{vc} = the initial vertical effective stress. Available data (12,20,22) from comparison among back figured field case histories, large shaking table studies, cyclic simple shear, and cyclic triaxial test results indicate that for level surface field conditions and isotropic consolidated triaxial test results, the values of C_r range from about 0.5 to 1.0. The lowest values pertain to normally consolidated clean, freshly deposited sands while the higher values pertain to overconsolidated, silty or clayey soils, or older sand deposits that may have experienced some past low intensity seismic shaking. It was felt to be appropriate in this study to account for possible variations in the C_r factor by using a range of C_r = 0.5 to 0.8 applied to the average cyclic triaxial strength curve for the undisturbed samples of hydraulic fill.

Consideration of Seismic Hazard

A detailed review of the geology and seismicity of Southern California is beyond the scope of this paper, but such reviews indicate that earthquakes producing combinations of acceleration and duration sufficient to cause liquefaction in the harbor area would most likely be generated on the central portion of the San Andreas fault or on the Newport-Inglewood fault. The San Andreas fault is some 50 miles (80 km) from the harbor area, but must be considered because of the potentially long duration of shaking that would be caused by a Magnitude 8 or greater earthquake. Such an earthquake occurred on the central portion of the fault in 1857, and this portion has been seismically quiet since that time, suggesting that the strain energy that accumulates along it is released in an occasional large earthquake rather than a number of smaller ones. The Newport-Inglewood fault zone is a somewhat complex feature that passes about 5 miles (8 km) from the harbor area. It was the source of the Magnitude 6.3 Long Beach earthquake of 1933. The total fault zone has a length of over 40 miles (65 km), yet because of its fragmented nature, it is generally considered that a maximum credible earthquake of no more than 7.0 should be assigned to this fault.

While the total seismic hazard in the harbor area is more complicated than is indicated by consideration of just these two sources, it is believed that for the purposes of this study, only the potential earthquakes generated by these two faults need be considered to represent the range of earthquake effects likely to occur at this site. That is, consideration was given to the occurrence of only a Magnitude 8+ earthquake on the San Andreas fault and to a Magnitude 6.0, 6.5, or 7.0 earthquake on the Newport-Inglewood fault.

The peak accelerations and durations that might be caused in the harbor area should these events actually occur may be estimated by means of various empirical relationships, and one set of estimates is shown in Table 1. The values given for the peak accelerations in rock were obtained by use of the relationship presented by Donovan and Bornstein (7), assuming a 10-km focal depth. These values were then reduced to give the peak acceleration at the surface by use of the relationship presented by Seed, et al. (24) between accelerations in rock and accelerations on deep cohesionless soil deposits. The durations are expressed in terms of the equivalent number of uniform cycles of motion corresponding to the various magnitudes (23). The procedure by which this equivalent number of cycles is computed is compatible to the use of average cyclic shear stresses as computed in the following section.

In using the earthquake parameters presented in Table 1, it should be recognized that these values are based on empirical data correlations from recorded ground motions and that there is significant scatter in the recorded ground motions from which these values have been derived. It is therefore unrealistic to attempt precise predictions of the motion that will result even if a specific event occurs.

TABLE 1.—Most Probable Values of Ground Motion Parameters

Event (1)	Peak acceleration in rock, in *g* (2)	Peak acceleration at surface, in *g* (3)	Duration, as equivalent number of uniform cycles (4)
	(*a*) San Andreas		
M = 8+	0.19	0.16	24
	(*b*) Newport-Inglewood		
M = 7.0	0.40	0.28	10
M = 6.5	0.31	0.22	6
M = 6.0	0.24	0.18	4

Thus, Table 1 is captioned "most probable values," signifying the use of average values from the empirical data correlations.

Computation of Cyclic Shear Stresses Induced by Earthquakes

For a given peak acceleration, a_p, at the surface expressed as a fraction of gravity, an average cyclic shear stress, τ_{av}, can be estimated at any depth in the soil profile by use of the following expression (after Ref. 22):

$$\frac{\tau_{av}}{\sigma_{vc}} = R\frac{\gamma h}{\sigma_{vc}} r_d a_p \quad (2)$$

in which $R = \tau_{av}/\tau_{max}$, an arbitrary factor usually chosen to be 0.65; γ = total weight of soil; h = depth below surface; and r_d = depth reduction factor.

The factor R is used in the conversion of any irregular time history of peak intensity τ_{max} to an equivalent number of uniform cycles, N_{eq}, of intensity τ_{av} (1,23). The N_{eq} values used herein correspond to $R = 0.65$. The depth reduction factor, r_d, accounts for the difference between the shear stresses computed by assuming that the soil is rigid and the shear stresses are computed

by one-dimensional response analyses that take the flexibility of the soil column into account.

For the purposes of this study, it was convenient to rearrange Eq. 2 as follows:

$$\frac{\tau_{av}/\sigma_{vc}}{a_p} = \frac{0.65\,\gamma h\, r_d}{\sigma_{vc}} \quad (3)$$

Assuming typical values for the soil densities at these sites and using the mean values of r_d given by Seed and Idriss (22), Eq. 3 may be solved to give a

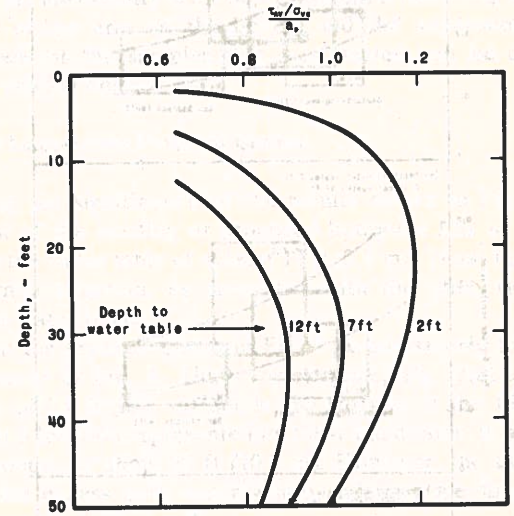

FIG. 6.—Variation of Induced Shear Stresses with Depth (1 ft = 0.305 m)

variation in cyclic stress ratio/acceleration versus depth as shown in Fig. 6 for several values of the depth to the water table.

Construction of Charts Showing Liquefaction Potential

An assessment of the liquefaction potential at the site may be made by comparing the cyclic strengths shown in Fig. 5 with the earthquake-induced stresses shown in Fig. 6. This comparison may be presented in several ways, but a form that makes it possible to indicate some of the uncertainties involved in the evaluations is shown in Fig. 7.

Each of the three separate charts in Fig. 7 represent a comparison of cyclic stresses to soil strength for one depth and one water-table condition. The depths for which the charts are constructed correspond to the critical depths, or depth to the greatest cyclic stress ratio/acceleration for each water-table location as indicated in Fig. 6. The range of expected cyclic strengths for the hydraulic fill is shown by the shaded zone and the range of induced earthquake stresses for the four postulated events listed in Table 1 is shown by the rectangular boxes.

The number of cycles of loading applied in laboratory tests or the equivalent number of uniform cycles in the field are shown on the horizontal axes in Fig. 7. For simplicity in presentation, only the peak surface accelerations in the field are shown on the vertical axes in Fig. 7, but by means of Eq. 3

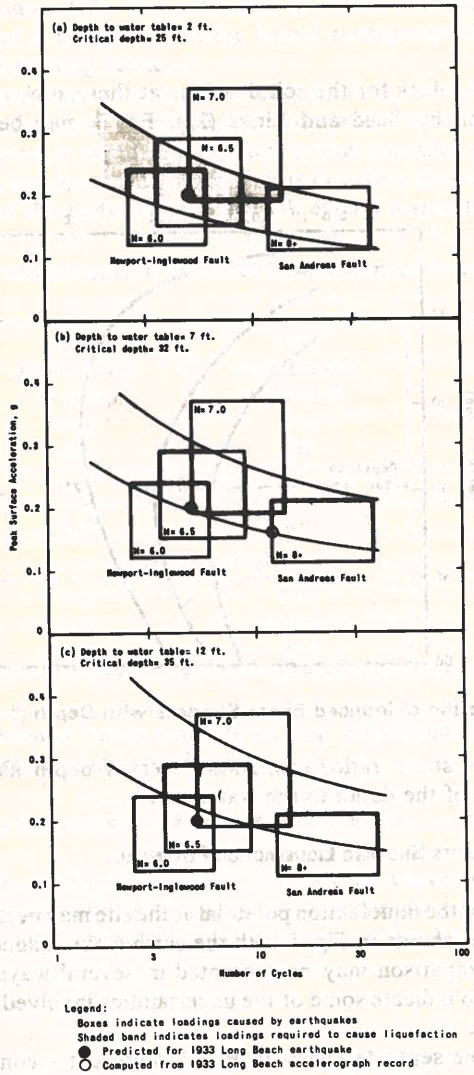

FIG. 7.—Evaluation of Liquefaction Potential (1 ft = 0.305 m)

or Fig. 6 the cyclic stress ratios, τ_h/σ_{vc} or τ_{av}/σ_{vc}, may be expressed in terms of the peak surface acceleration for any given depth. Thus, it is possible to convert the stress ratios estimated to cause liquefaction on the basis of laboratory tests to the equivalent peak surface accelerations that would cause liquefaction.

The shaded zones in Fig. 7 have been obtained using the broken curve in Fig. 5(*f*), which represents the average strength of undisturbed samples of hydraulic fill, and by using values of C_r = 0.5 and 0.8. The rectangular boxes in Fig. 7 are centered about the most probable values of peak surface acceleration and equivalent number of uniform cycles that are shown in Table 1, but a range corresponding to ± 1 standard deviation about these most probable values is shown.

There are, of course, uncertainties involved in the analysis other than those explicitly taken into account in constructing these charts, but it is believed that they are adequately represented by the size of the areas shown in Fig. 7. In particular, the uncertainty in the depth reduction factor, r_d, can be considered to be included in the area of the boxes, and the uncertainty regarding the representativeness of the samples that were tested can be considered to be included in the shaded zones.

Interpretation of Liquefaction Potential Charts

In interpreting the significance of the results shown in Fig. 7, it should be noted that most of the existing or proposed hydraulic fills in the harbor areas have a depth to the water table of at least 12 ft (3.7 m). Thus, Fig. 7(*c*) represents the most common situation, as compared with the other two charts having much shallower water tables.

Note also that the separate charts in Fig. 7 pertain strictly to the most critical depth, as indicated in Fig. 6. Thus, for example, Fig. 7(*c*) applies strictly to soil at a depth of 35 ft (10.7 m) below the surface of the fill. If liquefaction were to occur at a location represented by these conditions, it would be expected to initiate at a depth of about 35 ft (10.7 m). However, the subsequent upward dissipation of the excess pore water would endanger the strength and bearing capacity of the more shallow soils.

Finally, in order to make use of the results shown in Fig. 7, it is necessary to have at least some idea of the probability of occurrence of the four earthquakes during the useful life of the hydraulic fills. A detailed evaluation of this question is beyond the scope of this paper. However, it is prudent to expect that within the useful life of the harbor facilities there might be a Magnitude 8+ earthquake on the central portion of the San Andreas fault, or a Magnitude 6.0–6.5 earthquake on the Newport-Inglewood fault, or both. On the other hand, the occurrence of a Magnitude 7.0 earthquake on the Newport-Inglewood fault is most unlikely. Thus, for most practical purposes we need consider only the first three of these earthquakes in assessing the risk that liquefaction of the fills will occur.

It may be seen from Fig. 7(*c*) that the probability of liquefaction occurring is quite small for a Magnitude 6.0 earthquake on the Newport-Inglewood fault. Should a Magnitude 6.5 earthquake occur on this fault it is more probable that liquefaction will occur, but it must be remembered that for this source the chance that an event will occur at all diminishes as the magnitude increase. From a geologic standpoint, it is perhaps more likely that a Magnitude 8+ earthquake will occur on the San Andreas fault than a Magnitude 6.5 on the Newport-Inglewood fault. Thus, the San Andreas event may pose the greatest threat to the stability of the hydraulic fills. While the data do not lend itself to a precise evaluation of these probabilities, it might be concluded that, overall, the probability of liquefaction within the useful life of the harbor facilities is something less than 50%.

As noted previously, the risk of damage to structures, of economic loss, or of environmental distress further depends on the nature of any facilities constructed on the hydraulic fills. Thus, if the fill is to be used, e.g., for storing imported automobiles, the risk that liquefaction will occur may well be acceptable. If, however, a sensitive facility is to be constructed that has to be designed

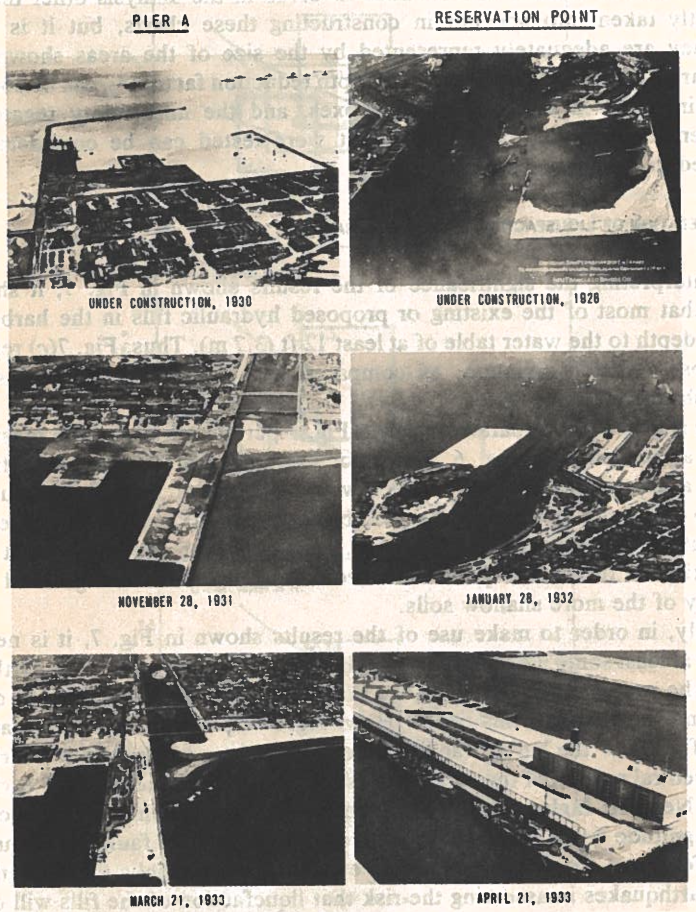

FIG. 8.—Photographs of Pre-1933 Hydraulic Fills

to withstand a Magnitude 7.0 earthquake on the Newport-Inglewood fault, then compaction or replacement of the hydraulic fill material would probably be required.

Performance of Hydraulic Fills in 1933 Long Beach Earthquake

While the procedures used herein for evaluating liquefaction potential have been verified in a general way by checks against the historical occurrence of

liquefaction at other locations, it so happens that the Long Beach earthquake of March 10, 1933, provides a direct check on the prediction method for the fills in the harbor area. This Magnitude 6.3 earthquake occurred on the Newport-Inglewood fault. The epicenter was located near Newport Beach, some 15 miles (24 km) from the harbor area, but it is postulated that the fault ruptured to a point below Signal Hill, which is about 5 miles (8 km) from the harbor area. Rather extensive damage occurred as a result of the earthquake, particularly in Long Beach and in Compton. According to Wood (29), many of the most spectacular effects of the earthquake were seen in poorly drained areas where the water table was close to the surface, in loose natural alluvium, or in material that had been graded and filled to allow development. It is likely that today the settlement and lateral spreading that occurred would be attributed to liquefaction, and there is clear evidence that liquefaction occurred at several locations along the coast (4,29).

There was also some damage in the harbor area, but this was confined mostly to the Los Angeles inner harbor where there were minor movements of sheet pile walls and timber pilings. Of greater interest, however, is the fact that the hydraulic fills in the harbor area survived the earthquake with negligible damage. Photographs of the fills at Reservation Point and Pier A, Long Beach taken during construction, shortly before the earthquake and shortly after the earthquake, are shown in Fig. 8. No damage or cracks and sand boils, which usually result from liquefaction, are visible in these and other photographs. A U.S. Army Corps of Engineers report contains the following:

> Mr. Henning and Mr. Eaton were working at the Long Beach Harbor in connection with construction of Pier A at a point one-half mile south of Seaside Boulevard, Long Beach, California . . . Mr. Henning remembered that the construction was a 32-foot hydraulic fill retained by steel and wooden bulkheads and rock breakwater in water about 20-feet deep. Fill material was dredged to provide shipping channels adjacent to bulkheads and wharfs being constructed. It was deposited within the bulkhead area in such a manner as to place coarser materials next to the steel bulkheads and to float the fine material over a spillway and back to sea. Cursory examination of the hydraulic fills was made by Mr. Henning soon after the earthquake and indicated lowering of the grade by a negligible amount.

Evaluation of Liquefaction Potential Under 1933 Long Beach Earthquake

Use of the same relationships that were employed in constructing the values of peak acceleration and duration that were listed in Table 1 yields a most probable peak horizontal surface acceleration of 0.2 g and $N_{eq} = 5$ for $R = 0.65$ in the harbor area for a Magnitude 6.3 earthquake on the Newport-Inglewood fault. These values are shown by a solid dot in Fig. 7. The depth to the water table in the then existing fills is believed to have been in the order of 7 ft (2.1 m). Thus, from Fig. 7(*b*), the predicted loading from the earthquake falls at the lower bound of the shaded zone showing the cyclic stress conditions expected to cause liquefaction. Assuming that the resistance to liquefaction of the natural alluvium and filled areas inland from the harbor is the same as that of the hydraulic fill, the probability of liquefaction where the water table was close to the surface would have been higher, as shown by the relative position of the solid dot and the shaded strength zone in Fig. 7(*a*). While the various uncertainties involved in the analysis should still be recognized, it may be seen that the results of the analysis show good general agreement with the performance observed in the 1933 earthquake.

Reanalysis Using Long Beach Accelerograph Record

As an alternative to the simplified procedure used in the preceding analysis, the performance of Pier A, Long Beach in the 1933 earthquake has also been studied in more detail using the accelerograph record that was obtained nearby in the basement of the Public Utilities Building (see Fig. 1). This record is remarkable, not only for its proximity to the site of interest, but also because it was the first strong motion record ever obtained, the instrument having been installed only several months before the earthquake occurred.

There are some problems with this record. Unfortunately, because it was not known beforehand to what magnification scale the instrument should be set, a high gain was used and the records for the three components overlap badly. However, it has recently been digitized (26), and after baseline correction, the peak accelerations of the two horizontal components are 0.20 *g* and 0.16 *g*. The vertical component has the unusually high peak acceleration of 0.28 *g*, but this should not have any significant effect on the occurrence of liquefaction (25).

The north-south component of the record, which is the greater of the two horizontal components, was used to compute the induced shear stresses at Pier A by means of the computer program SHAKE (19) following the usual method for very deep soil sites. The recorded motion was input at a depth of 7 ft (2.1 m), basement level, to a model representing the site conditions at the Public Utilities Building and deconvoluted to obtain the motion at the base of the model, arbitrarily set at El. −200 ft (−60 m). This motion was then input at the same elevation base of a second model representing the site conditions at Pier A and propagated to the surface. It was assumed that both profiles had shear moduli and damping ratios as given by Seed and Idriss (21) for sands of 75% relative density, except for the hydraulic fill extending through the top 28 ft (8.5 m) at Pier A where properties corresponding to 50% relative density were used.

The peak acceleration computed at the surface of the hydraulic fill was 0.21 *g* but when the maximum value of the computed cyclic stress ratio in the hydraulic fill was converted to an equivalent peak surface acceleration by means of Fig. 6, a value of only 0.16 *g* was obtained. This difference results from the fact that the values of the depth reduction factor, r_d, that were computed in the SHAKE analysis were smaller than those assumed in constructing Figs. 6 and 7. The equivalent number of uniform cycles corresponding to the maximum cyclic stress ratio was found to be equal to 12, a value somewhat greater than might be expected for a Magnitude 6.3 earthquake according to the data presented by Seed, et al. (23). This difference probably results from the fact that the Long Beach accelerogram is rather richer in high frequency content than the bulk of the records studied by Seed, et al. However, even though the equivalent peak surface acceleration is a little lower than might have been expected and

the number of cycles is greater than might have been expected, the computed loading due to the stronger component of the 1933 earthquake plots at the lower bound of the conditions estimated to cause liquefaction, as shown by the open circle in Fig. 7(*b*). Again, this result is consistent with the observation that liquefaction did not occur during the 1933 earthquake, suggesting that the estimated resistance to liquefaction of the hydraulic fills that is shown in Fig. 7 is indeed reasonable.

Use of Standard Penetration Test Blowcounts to Evaluate Liquefaction Potential of Hydraulic Fills

As an alternative to the methods described previously, which use laboratory tests to evaluate the liquefaction characteristics of sands, several workers have

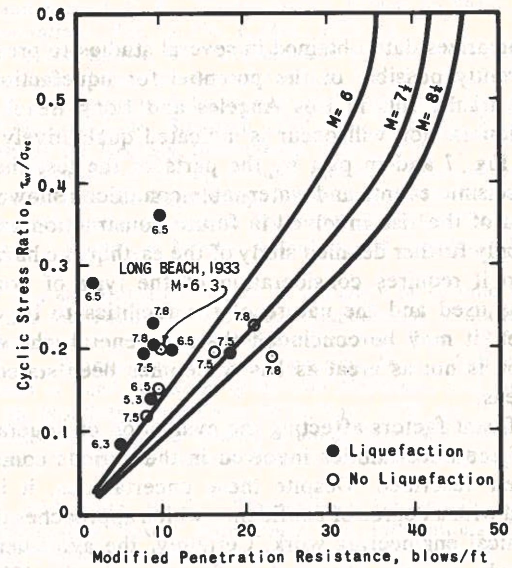

FIG. 9.—Correlation between Historical Occurrence of Liquefaction and Penetration Resistance, after Seed (1976)

suggested that more direct correlations with the observed occurrence of liquefaction can be made by using standard penetration test blowcounts. The correlation presented by Seed (20) is shown in Fig. 9. In this figure, the average cyclic stress ratio required to cause initial liquefaction is shown as a function of the modified penetration resistance and is intended to be computed by the simplified procedure included in Eq. 1. The modified penetration resistance is obtained by adjusting the measured blowcounts to those that would be obtained if the vertical effective stress was 1 tsf (96 kN/m^2) (10,14). The solid symbols indicate data from sites where liquefaction has occurred and the open symbols indicate data from sites that have been subjected to significant shaking but where liquefaction has not occurred. The three lines shown in Fig. 9 are lower bound relationships obtained from a combination of field and laboratory studies describing conditions causing liquefaction for three different magnitudes.

The modified penetration resistance obtained in the limited number of borings in Pier A, Long Beach, and Reservation Point and in an extensive set of borings in Pier J, average only about 10 blows/ft. Using Eq. 2 and the appropriate corresponding data described previously for the 1933 hydraulic fill leads to $\tau_{av}/\sigma_{vc} = 0.2$ at the critical depth. This information is plotted conspicuously in Fig. 9 and is located somewhat into the liquefaction zone.

This result is not inconsistent with the data on which Fig. 9 is based since, for the same relative density, the silty sands and sandy silts in the hydraulic fills would be expected to shown a lower standard penetration resistance than the generally cleaner sands on the figure is based. However, it does suggest that use of Fig. 9 may be rather conservative for more silty materials.

Conclusions

This paper summarizes data obtained in several studies to present as realistic a picture as currently possible of the potential for liquefaction under cyclic loading of the hydraulic fills in Los Angeles and Long Beach Harbors. The probability that liquefaction will occur is indicated qualitatively in part by the charts shown as Fig. 7 and in part by the parts of the test that describe the likelihood of the seismic events and water-table conditions shown on the charts. Full interpretation of the risk involved in future construction and use of these fills requires not only further detailed study of the earthquake hazard for specific locations but also it requires consideration of the type of containment dike construction to be used and the nature of the facilities to be constructed on the fills. However, it may be concluded that, in general, the risk of damage due to liquefaction is not as great as has sometimes been stated for hydraulic fills in seismic areas.

The more significant factors affecting the evaluation of liquefaction potential and the nature of the uncertainties involved in the various components of the analysis have been described. Despite these uncertainties, it is felt that the results can be used with a degree of confidence which approaches that appropriate in much geotechnical engineering work. Certainly, the agreement between the results of this study and the observed performance in the 1933 Long Beach earthquake is encouraging.

For the silty sands in the harbor hydraulic fills, it appears that the use of the full analytical procedure, along with laboratory tests, is appropriate since the simpler procedure that uses standard penetration test blowcounts to evaluate the cyclic strength may be unnecessarily conservative. This conservatism does not diminish the usefulness of the blow count procedure for a quick approximate analysis. But it does highlight the fact that this and all procedures are most meaningful when the basis of their construction and their limitations are understood.

Finally, it is emphasized that there is still ample room for improvement in our understanding of seismic induced soil liquefaction, including such factors as regional seismicity, stress calculations, effects of sampling disturbance, the corrections that should be applied to laboratory test results to obtain field behavior, and, perhaps most importantly, the consequences of liquefaction under cyclic loading for engineered facilities.

Acknowledgments

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Downdrag on Bitumen-Coated Piles

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Introduction

The settlement of the soil surrounding a pile foundation causes the shearing stress along the pile shaft to decrease and, eventually, to reverse its direction. This negative skin friction gives rise to downdrag forces on the pile and represents one of the major causes of pile failures (7,11,17,19). Numerous methods have been proposed to predict downdrag forces (5,11,20,24,27). Based on field measurements, Garlanger (15) and Baligh and Vivatrat (2) presented the β-method for predicting the maximum downdrag force on a single vertical pile.

Friction and batter piles are not recommended when downdrag might occur. On vertical end-bearing piles, small downdrag forces can be resisted by increasing the pile capacity, by providing additional piles, or by reducing the spacing between piles in a group. But when large downdrag forces are expected, as in the case of long piles exceeding 75 ft–100 ft, the reduction of downdrag loads by one of the following methods is necessary: (1) Eliminate the soil settlement that takes place after pile installation, e.g., use preloading; (2) use electro-osmosis to increase the pore water pressure around the cathode pile: this decreases the effective stress and, in turn, the shearing resistance between pile and soil (6,18); (3) use a casing to prevent direct contact between the pile and the soil (4); and (4) reduce negative skin friction by coating the pile with a friction reducer.

Coating the pile with bitumen often represents the most economical method for downdrag reduction (4,5,8,9,10,12,20,25,26). This article investigates the performance of bitumen-coated piles and develops a method for predicting

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