28 Apr 1981, 9:00 am - 12:30 pm

Site Analysis for Seismic Soil Liquefaction Potential

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Proceedings: First International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, April 26 - May 3, 1981, St. Louis, Missouri

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SYNOPSIS Field penetration data and cyclic laboratory test data are presented for evaluating the seismically induced soil liquefaction potential at a waterfront site. Dynamic split-spoon penetration recordings and three types of quasi-static friction cone probings were made. The friction cones were the standard mechanical cone, the electric cone, and the piezometric cone. The latter penetrometer has a porous element near the tip which permits measurement of pore water pressures generated during penetration. By interrupting penetration, the rate of pore pressure dissipation can be recorded, so an estimate of soil permeability can also be derived. Undisturbed samples were taken using an Osterberg piston sampler, and the specimens were subjected to cyclic triaxial testing in the laboratory. The field sounding techniques were in agreement; however, they indicated a recently deposited sensitive silty sand to have a liquefaction resistance approximately one-half that based upon cyclic triaxial testing. The question arises as to whether the liquefaction resistance as determined by laboratory testing is more appropriate herein, or whether behavior suggested by the penetration test results more realistically predicts the response of this type of soil to an actual earthquake.

INTRODUCTION

This paper outlines a field and laboratory investigation of seismic soil liquefaction potential. Major deficiencies exist in current liquefaction prediction technology (see Forrest and Ferritto, 1978), particularly in situations involving heterogeneous soils or complex loading conditions. Such diverse soil profiles and unique structures are particularly prevalent at the waterfront and are therefore of considerable interest to the Navy.

In order to investigate prediction capabilities, a specific site, located at a West Coast Naval facility, NAS, North Island, near San Diego, was selected for evaluation.

Field investigations included: standard wash borings; hollow stem auger borings; split-spoon penetration tests (ASTM D1586 except that a 2-1/2-inch split-spoon was used); and mechanical, electric, and piezometric friction cone soundings (see Schmertmann, 1978; and Wissa, Martin, and Garlanger, 1975). The cone penetration soundings conformed to ASTM D3441-75 T. In addition, undisturbed samples were taken using an Osterberg sampler and carefully transported to the laboratory for cyclic triaxial testing. Additional information on these investigations is presented by Forrest and Ferritto, 1976; Forrest and Ferritto, 1979; and Forrest, Ferritto, and Wu, 1979.

SITE DESCRIPTION

The test site encompasses an old filled bay channel formally known as the Spanish Bight (see Figure 1). It comprises several different soil types, including dredged hydraulic fills and naturally deposited shallow marine deposits. The area had been reclaimed about 30 years ago and now functions as a parking lot.

The soil profile is relatively complex, but a highly generalized soil profile is illustrated in Figure 2. Also shown on Figure 2 are the locations of borings, P1 through P5, initially used to define the site.

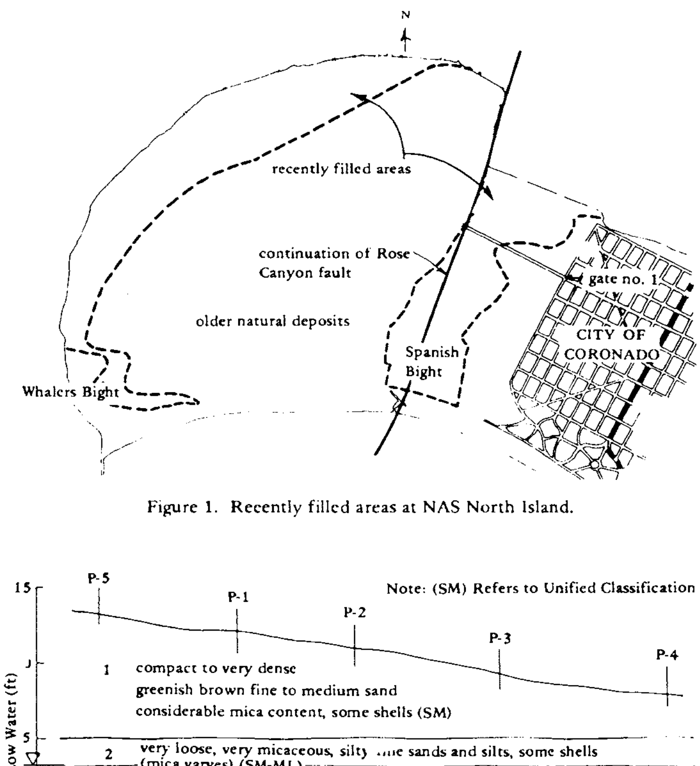

Figure 1. Recently filled areas at NAS North Island.

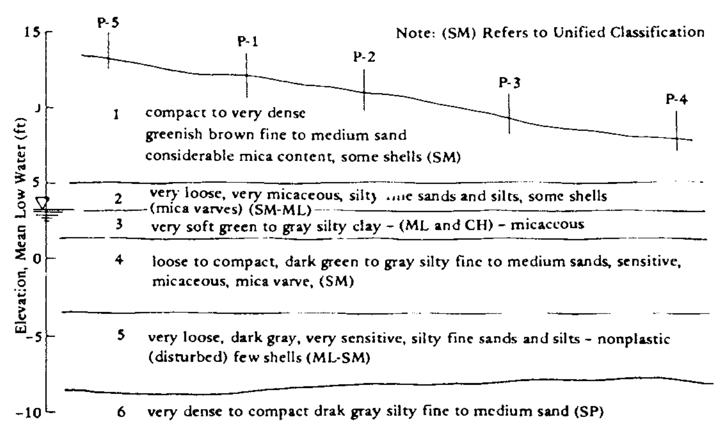

Figure 2. Generalized soil profile across former Spanish Bight.

The old former bay bottom, represented by layer 5, is a weakly cemented fine silty sand (see Figure 3) having a resilient cohesiveness in its undisturbed state.

The upper four layers include primarily dredged hydraulic fills with occasional lenses of plastic clays. The surface layer consists of a dense sand fill that has been compacted by heavy traffic including that of towed aircraft.

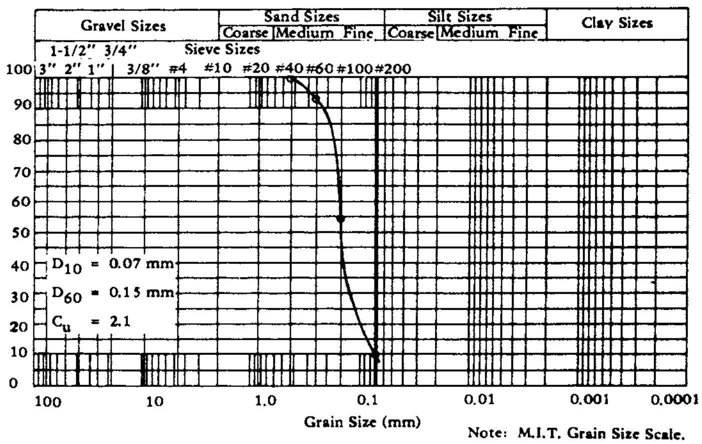

Figure 3. Grain size distribution curve for North Island weakly cemented sand.

FIELD INVESTIGATION

The results of split-spoon penetration and mechanical friction cone soundings conducted during the initial stages of the investigation (denoted as penetration holes P1 through P5 along the soil profile) are shown in Figure 4.

Friction cone soundings using an electric cone were conducted in the vicinity of test hole P3 on the soil profile of Figure 2. A schematic of the electric cone (owned by Fugro, Inc. of Long Beach, CA) is shown in Figure 5. A total of nine penetration cone soundings was carried out. Typical results are presented in Figure 6, which shows both point resistance and sleeve friction plotted in terms of kg/cm^2 versus depth. Also shown in Figure 6 is the ratio of sleeve friction to point resistance, plotted as a percent.

Piezometric cone soundings were also performed in the vicinity of holes P2 and P3. This work was accomplished by Fugro Gulf, Inc. of Houston, TX, using a truck-mounted electronic cone penetrometer system. The piezometric cone is shown in Figure 7. The cone tip had the same dimensions as the standard friction cone, however, a specially designed porous element is used to allow entry of pore water. Thus, both pore pressure and total tip resistance are measured. To simplify the instrumentation, sleeve friction is not measured with this penetrometer. Pore pressure may be recorded as a function of depth (and penetration rate) during penetration, and pore pressure dissipation may be recorded as a function of time when penetration is stopped.

In operation, special precautions were required to prevent air entry (water cavitation) into the porous element during cone passage through the 5-foot-thick soil zone above the water table. These precautions included predrilling and casing holes to below the water table. The piezometric cone element was then saturated and encased in a water-filled membrane which maintained tip saturation while the penetrometer was being lowered in the bored hole to beneath the water table. The membrane was subsequently punctured by the advancing tip when penetration commenced into the soil at the bottom of the predrilled hole.

Plots of typical piezometric cone data are presented in Figure 8. On the left side, the generated pore pressures measured during cone penetration at an approximate rate of 2 cm/sec are shown. The right diagram depicts tip resistance in kg/cm^2 versus depth.

Undisturbed sampling was conducted to obtain specimens for cyclic triaxial testing. Samples of soil were obtained along the profile of Figure 2 using an Osterberg piston sampler.

This sampler utilizes a thin wall sampling tube which is 3 inches in outside diameter, 2.88 inches in inside diameter, and 36 inches long. Tube tips were designed to provide an inside clearance ratio of about 1 to 2% and an area ratio of about 10 to 15%.

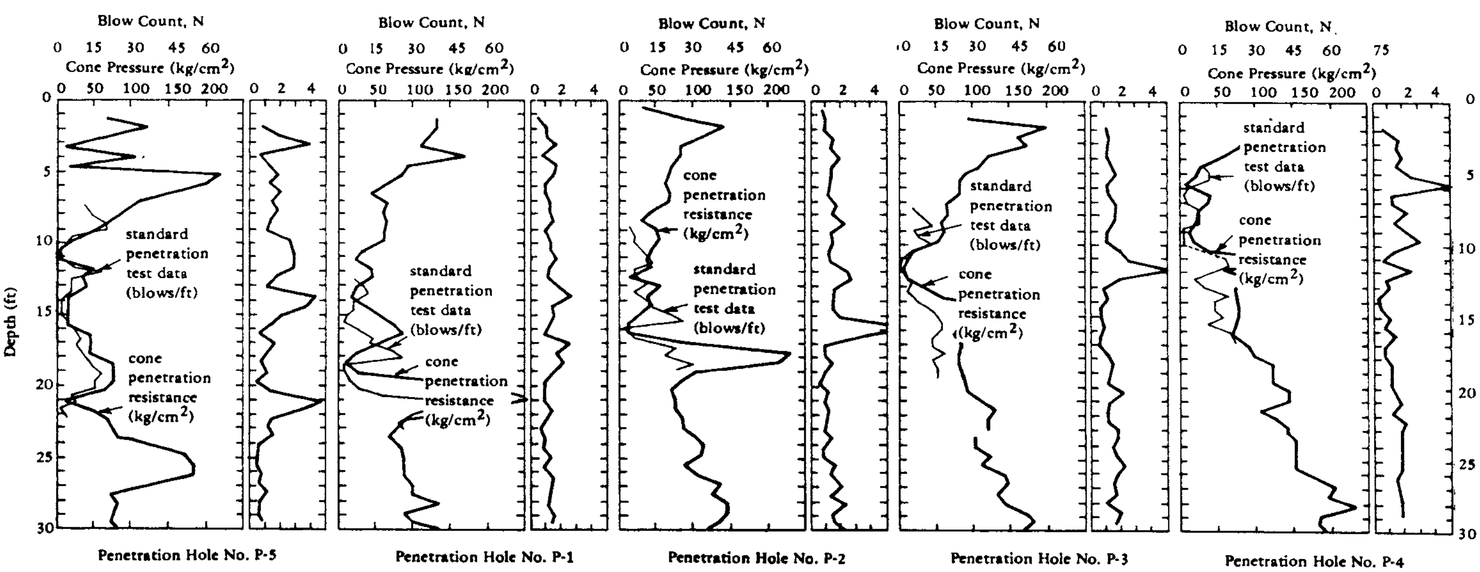

Figure 4. Dynamic split-spoon and mechanical cone penetration resistances.

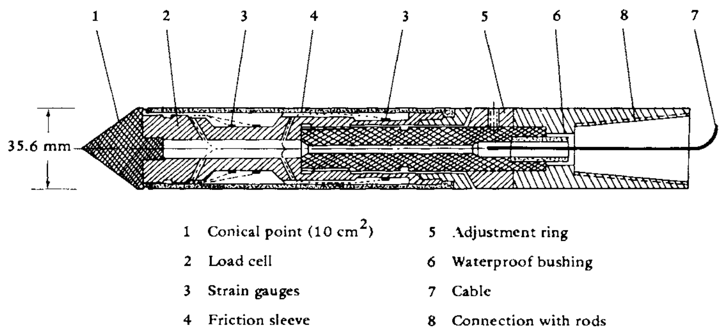

Figure 5. Cross section of friction cone penetration.

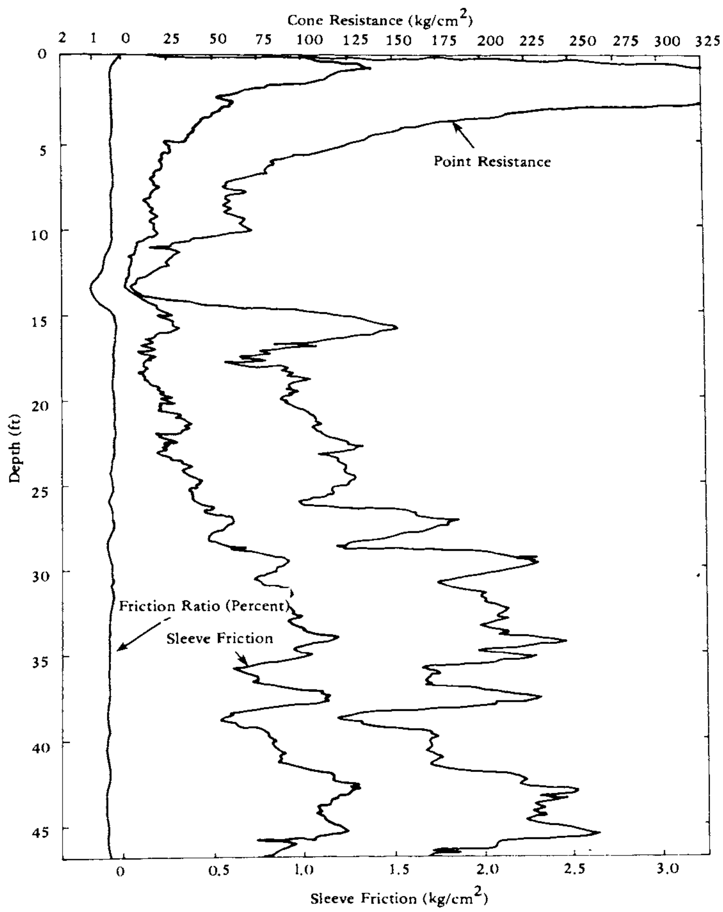

Figure 6. Electric cone results near location P-3.

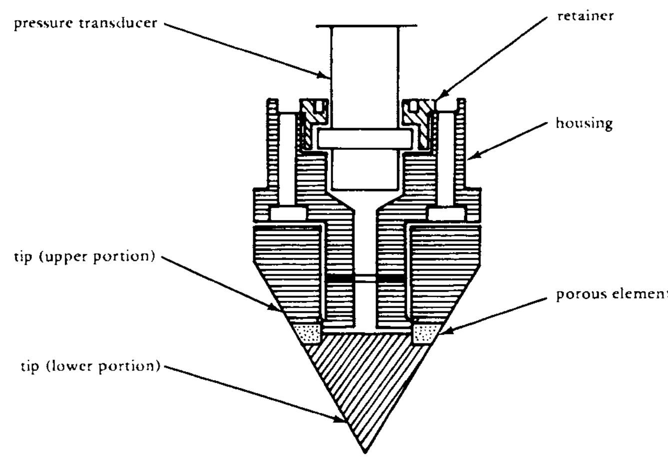

Figure 7. Cone tip assembly for piezometric cone.

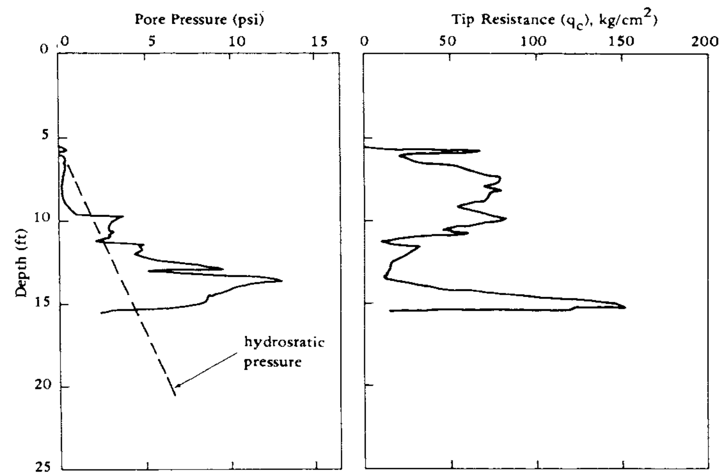

Figure 8. Piezometric cone record.

Immediately after sampling, each tube sample was subjected to a vacuum for approximately 30 to 60 minutes. Samples were then allowed to drain under gravity for an additional period of several hours. The purpose of this process was to drain off pore water, thereby increasing the strength of the material through capillary action. It was believed that this process would reduce the possibility of sample densification during transport to the laboratory. The maximum vacuum applied to the sample was less than two-thirds of the estimated effective overburden pressure. Samples were placed in foam padded boxes and shipped on automobile inner tubes which rested, in turn, on the bed of a pickup truck. The inner tubes were used as a cushioning mechanism to reduce vibrations during transport.

LABORATORY TESTS

The soil selected for cyclic testing (layer 5) was a very fine silty sand generally dark grey in color. This soil is relatively uniform (Figure 3) with a coefficient of uniformity of less than 4.0. Maximum and minimum dry densities were 108.2 pcf and 79.4 pcf, respectively. The density of undisturbed samples was approximately 93 pcf, which was equivalent to a relative density (D_r) of about 55%. Undisturbed test specimens were prepared for testing by horizontally extruding samples from the thin-wall sampling tubes into a tray. Specimens suitable for testing were then selected and trimmed to a length of 6.0 inches (15.2 cm). The initial wet density of each trimmed sample was determined on the basis of its final height, diameter, and weight.

After determining the initial wet density, each sample was enclosed in a latex rubber membrane (0.3175 mm thick), placed in a triaxial chamber, and subjected to an isotropic consolidation pressure.

Samples were saturated before testing. Sample saturation was accomplished by flushing the sample and drainage lines with deaired water and then applying a back pressure. The back pressure was determined by the amount of pressure necessary to achieve a very high degree of saturation. Liquefaction tests were performed by cyclically loading the cylindrical sample of soil in compression and extension. A pneumatic cyclic triaxial device utilizing a cyclic loader was used to impose the cyclic loads. A Gould strip chart recorder in combination with a Schaevitz pore pressure transducer and a Schaevitz LVDT were used to record pore water response and soil deformation.

The stress controlled, undrained cyclic triaxial tests were performed in the manner described by Silver, 1977. This procedure involved applying sinusoidal load with a constant maximum amplitude to the top of the specimen. The frequency of loading was 1.0 Hertz. Axial deformation, axial loads, and sample pore pressures were recorded for each cycle of load. Cycling was continued until either double amplitude axial strain was greater than 10% or the number of cycles of load without 10% strain exceeded 300.

The soil specimens were subjected to the same level of confinement as the estimated initial in-situ vertical effective stress, and then saturated to "B-values" greater than 0.97 prior to testing. Figure 9 presents the results of cyclic triaxial testing on eight soil specimens extracted from soil layer 5. Two criteria for soil failure were used. Figure 9A presents maximum shear stress to confining stress ratio versus the number of load cycles that result in an axial or vertical strain of 5%. Figure 9B presents the same type of information but is based upon the time at which measured pore water pressure in the specimen equals the triaxial confining or chamber pressure (rather than 5% strain level). Both criteria lead to the same result in this case.

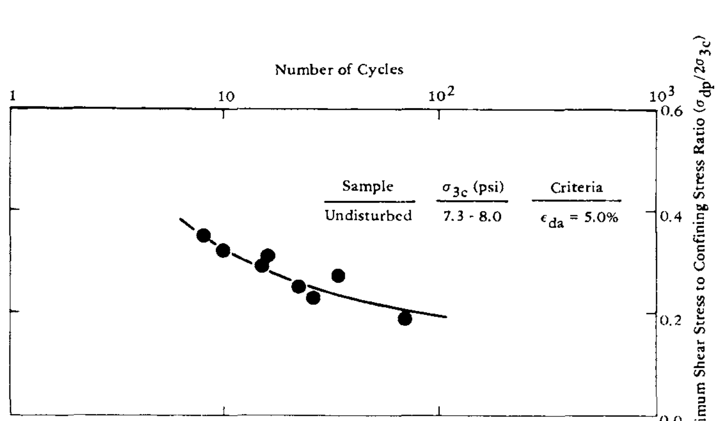

Figure 9a. Liquefaction strength curve based upon 5 percent vertical strain.

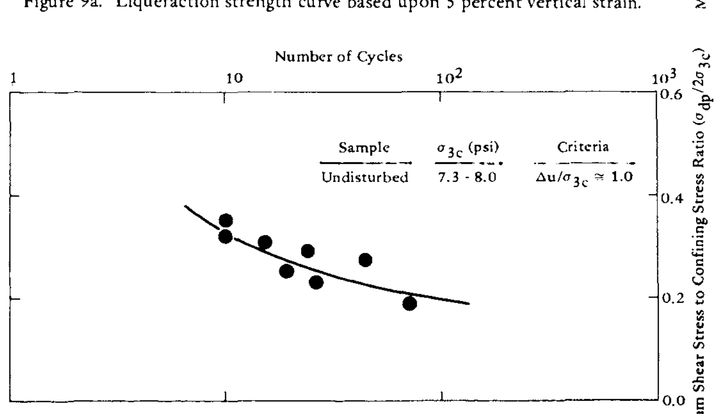

Figure 9b. Liquefaction strength curve based upon pore pressure.

DISCUSSION

The split-spoon penetration values shown on Figure 4 fall as low as one or two blows per foot near layer 5, suggesting resistance to liquefaction equivalent to that of sands having relative densities well below 50% (Bieganouski and Marcuson, 1976). The low friction cone penetration readings of 5-10 kg/cm² shown in Figure 4 also suggest relative densities of only 20 to 30% (Schmertmann, 1978). Very low friction ratios associated with both the mechanical cone soundings (Figure 4) and the electric cone (Figure 6) indicate that the material is generally cohesionless. The soils in layer 5 consisted of fairly sensitive silty sands and as such suggest a high liquefaction potential. This type of soil is frequently encountered under reclaimed areas along the southern California coast and therefore of special interest to the Navy. The electric cone penetration record in Figure 6 indicates soil with similar characteristics to those identified, using the split-spoon and mechanical friction cone data. The friction sleeve ratios for the electric cone were somewhat lower than those from the mechanical cone. Thus, although both devices provide sleeve ratios denoting cohesionless soils, there are some differences between mechanical and electrical cone friction ratios.

In Figure 10, a compilation of cone resistance versus depth for the three types of friction cones in the vicinity of hole P-3 (Figure 2) is shown. It is noted that despite the different cone surface textures and somewhat different penetration procedures, the records are quite similar.

The piezometric cone could not obtain any data throughout the first five feet of prebored hole. Penetration was discontinued at about a 15-foot depth where there was a

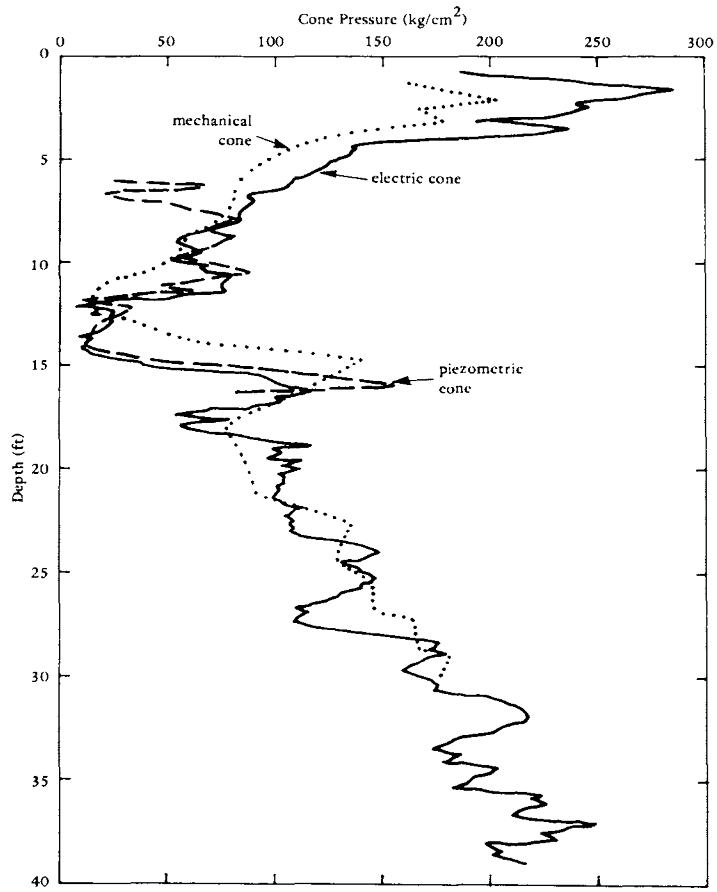

Figure 10. Comparison of point resistance from 3 types of cones.

danger that the dense in-situ material might damage the porous tip element. Thus, it appears that the three quasi-static cone penetrometers give very comparable data, at least with regard to point resistance. These data are also comparable with the split-spoon results.

The liquefaction potential of a soil is directly related to its volumetric change tendencies, more specifically to its volumetric-strain/shear-strain coupling.

Unfortunately, the penetration resistance of a soil is a function not only of its volume change characteristics but also of its strength, shear stiffness, and other deformational characteristics. Penetration resistance may be influenced by factors other than those directly influencing liquefaction potential. Therefore, it may be necessary to measure several types of response before accurate penetration test correlations become possible.

The piezometer probe measures pore pressure near the tip of a penetrating cone. If, during cone penetration, positive (increased) pore water pressures are generated, then the effective stresses and, hence, the strength and resistance to penetration are reduced. Alternatively, if negative (reduced) pore water pressures occur, then the soil structure is dilating, and effective stresses are increased and so is penetration resistance. The measured incremental changes in pore pressure during penetration of saturated soils are directly related to volume change tendency. They are also a function of soil permeability and rate of penetration.

The rate at which pore water pressures reach equilibrium following cessation of penetration is a direct function of permeability. Thus, this device has the potential for correlation with both volume change characteristics and permeability. These two factors are the major determinants (along with nature of loading) which control the occurrence and severity of soil liquefaction.

The pore pressures during penetration at this site indicate both dilative (decreased pore pressure) behavior and compactive (pore pressure increase) behavior for the different in-situ soil strata.

Figure 8 indicates a medium-dense layer of soil to about a 10-foot depth, which tends to dilate due to passage of the cone and generates a reduction in pore pressures. Beyond a depth of 10 feet, the soil strength falls off, with attendant pore pressure generation. This corresponds to the very sensitive sands in the soil profile of Figure 2. A denser stratum is again encountered at a depth of about 15 feet (approaching layer 6) with a dramatic reduction in pore pressure generation. Tip resistance then falls off due to interruption of penetration.

Thus, it appears that the piezometric cone is sensitive to the type of soil response associated with liquefaction.

In Figure 11, the North Island cyclic triaxial data for soil layer 5 plotted on top of shake table data from DeAlba, Chan, and Seed, 1975, are shown. The numbers on the solid curves relate to relative density, D_r.

These data are plotted in terms of shear stress divided by effective confining stress (stress ratio) versus number of cycles to initial liquefaction. Initial liquefaction is defined here as the state at which pore pressure increase under cyclic shear loading equals the initial effective confining stress (100% pore pressure ratio).

The triaxial data have been corrected to be comparable with the stress ratio used for the shake table tests. The shake table test results show the stress ratio versus

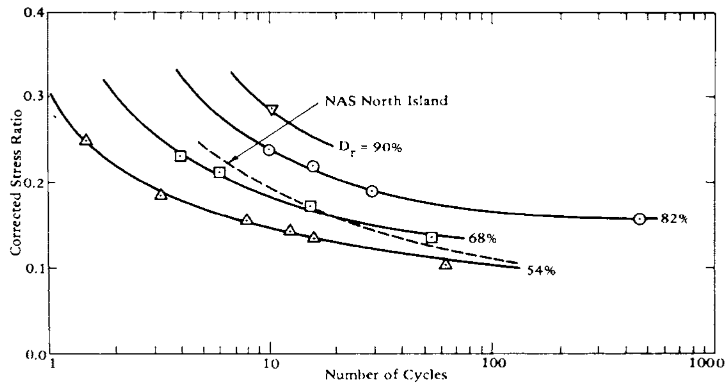

Figure 11. Stress ratio versus number of cycles for initial liquefaction.

the number of cycles leading to 100% pore pressure ratio for sands at four different relative densities (D_r). It is noted that the North Island data plot is somewhere above the D_r = 68% curve for the sand in the low cycle range and above the 54% relative density line for the higher cycle range. Thus, although the North Island soil appeared to have a D_r less than 35%, based upon quasi-static and dynamic penetration tests, it exhibits a resistance to liquefaction under cyclic triaxial conditions comparable to that of a denser soil.

This increased laboratory liquefaction resistance is possibly due to the distinct structure of the sensitive silty sandy materials in layer 5. This conclusion is further supported by the trend in the test data in Figure 11. It is noted that under higher stress levels (lower number of cycles to failure) the North Island soil performs like one with a considerably higher relative density. Under lower stress levels (higher number of cycles to failure) the indication of a higher relative density is not so marked. Perhaps the longer time to failure under the lower stress levels permits more disturbance of the initial soil structure and, hence, causes the soil to perform as if it had a lower relative density.

In Figure 12, some of the results of a compendium of soil data compiled from a number of commercial testing laboratories (Ferritto and Forrest, 1979) is portrayed. The data in Figure 12, for both uniformly graded (SP) and silty sands (SM), have been normalized in terms of D_r. (It has been observed that by dividing the stress ratio causing liquefaction at a particular number of load cycles by relative density, reasonable agreement could be obtained between the liquefaction resistance of similar soils at different densities, (see DeAlba, Chan, and Seed, 1975)). Also shown in Figure 12 is the best fit curve (Donovan, 1972) for data from a number of laboratory research programs.

It is noted that in spite of any differences in precision between research and commercial testing activities, the mean soil strength curves are in very good agreement.

Superimposed upon the data in Figure 12 are the North Island soil test data from Figure 11, with an assumed D_r = 60%. The agreement with the mean curves indicates that the North Island soil had a resistance to liquefaction roughly equivalent to that of a typical sand having a relative density of 60%. The lower D_r values estimated for this soil (35%) based upon the penetration readings could be explained in terms of the sensitivity of the soil structure.

It would appear that the soil structure is largely lost during the penetration tests in advance of the sounding device. Thus, the penetration tests, both with the split-spoon and with the fricton cones, do not appear to

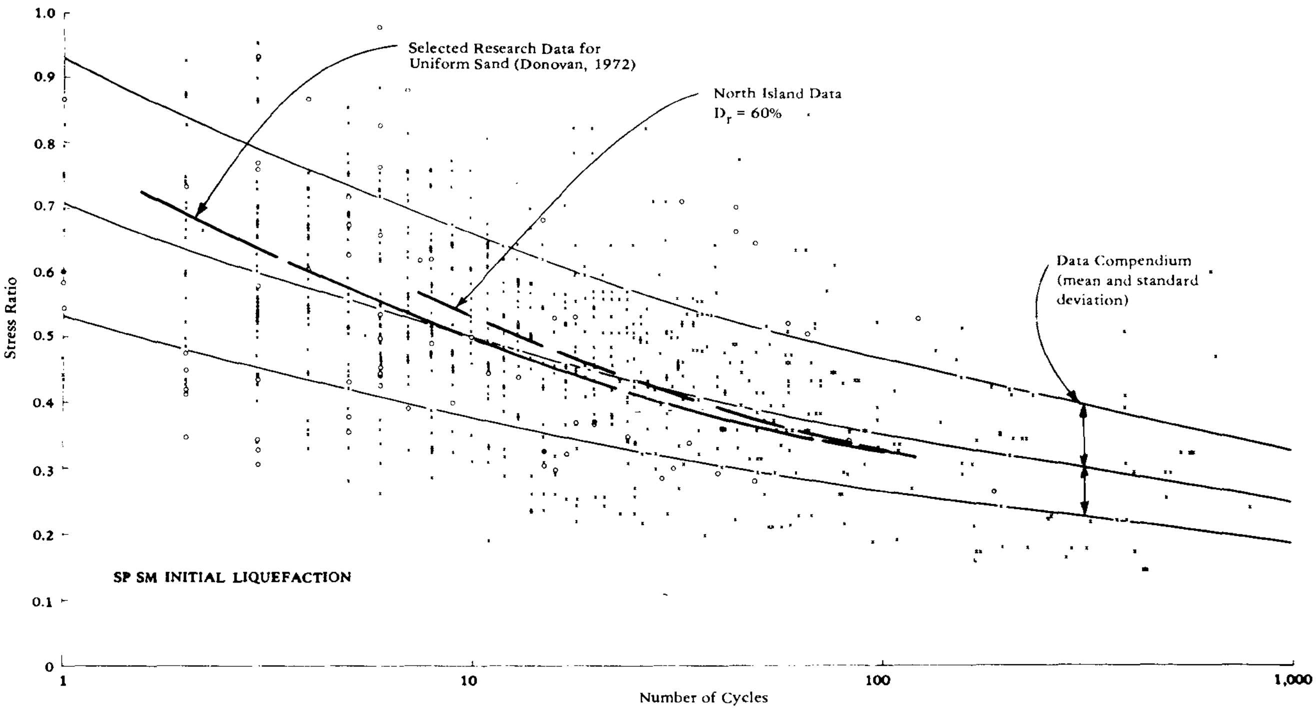

Figure 12. Normalized stress ratio versus number of cycles to initial liquefaction.

recognize the inherent resistance to cyclic liquefaction provided by the soil cementation. The piezometric cone also appears to be sensitive to the type of soil response that dictates liquefaction potential. Whether that method is precise enough to be used for reliable liquefaction potential assessments cannot be ascertained until further experience has been acquired.

The weakly cemented structure of the sensitive silty sand appeared to influence the results of the cyclic triaxial tests much more than those of the field penetration evaluations. Therefore, the most critical soil stratum in the profile might be characterized as one having a strength similar to that of a sand at a relative density of either 60% or 35%, depending upon whether cyclic triaxial or penetration tests are used. This represents a difference in liquefaction resistance of approximately two.

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