SR-TR- 79-1283

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ADA078521

BLAST INDUCED LIQUEFACTION POTENTIAL

- Influence of Unsaturated Conditions -

Final Report

Air Force Office of Scientific Research

Grant No. AFOSR-78-3598

26 September 1979

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79 12 18 106

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER (19) (18) AFOSR-TR-79-1283
2. GOVT ACCESSION NO.
3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) (6) BLAST INDUCED LIQUEFACTION POTENTIAL. - INFLUENCE OF UNSATURATED CONDITIONS.
5. TYPE OF REPORT & PERIOD COVERED: Final Report 3/1/78 - 2/28/79
6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) (10) Wayne A. Charlie ~~and~~ Joseph P. Martin
8. CONTRACT OR GRANT NUMBER(s) (15) AFOSR-78-3598
9. PERFORMING ORGANIZATION NAME AND ADDRESS: Depart~~ment~~ of Civil Engineering - ~~Geotechnical Engineering Program~~ Colorado State Univ~~ersity~~, Ft. Collins, CO 80523
10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS: 61102F 230909 (16) (17) D9
11. CONTROLLING OFFICE NAME AND ADDRESS: U.S. Air Force Office of Scientific Research/NP Bolling AFB, D.C. 20332
12. REPORT DATE (11) 26 Sep~~tember 1~~979
13. NUMBER OF PAGES 68
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office) (12) 32
15. SECURITY CLASS. (of this report) unclassified
15a. DECLASSIFICATION/DOWNGRADING SCHEDULE
16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

(9) Final rept. 1 Mar 78-28 Feb 79,

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

BLASTS, cyclic loads, damage, effective stress, explosions, fatigue, groundmotion, ground shock, liquefaction, porewater pressure, soil, saturation, soil liquefaction, soil property changes, strength, structural stability, wave propagation.

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The phenomenon called blast induced liquefaction is examined. Blast induced ground motions which may cause liquefaction are categorized on the basis of wave form characteristics and motion types and compared to earthquake groundmotions. Prediction methods used in Russia, Europe and North America are reviewed. The influence of field tests on unsaturated subsurface soils in respect to liquefaction potential, porewater pressure increase, porewater pressure measurement, shock wave velocity and scaling factors are given. The occurrence, detection, → over

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and removal of entrapped gas are reviewed. Recent research has shown that the assumption of little or no soil property change may be non-conservative for certain saturated soil subjected to loadings from high explosive test events. Various indirect evidence indicates that blast induced liquefaction (a soil property change) may occur. Any attempt to correlate the results of blast-induced liquefaction with data obtained in conjunction with earthquake loadings must include the many differences between the two. However, important material property changes common to both seem to be pore-water pressures, shear strength, and relative density. The effect of soil type and magnitude and type of stress changes, induced by overpressure and direct induced ground motion, are also controlling factors. Until additional research is completed, loose saturated sands should be regarded as having a high blast-induced liquefaction potential.

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I. INTRODUCTION

Several military aspects are related to blast induced soil liquefaction.[1] Blast induced liquefaction may cause dams, dykes, embankments or natural slopes to fail by inducing sufficient excess porewater pressures to reduce the soil strength and allow gravity to fail the slopes. Liquefaction may also produce flotation, sinking or differential movements of military structures. An explosion, or multiple explosions, detonated in, on, or above a soil having a high liquefaction potential, could result in damages disproportionate to the energy released. An understanding of blast induced liquefaction potential of soils is therefore needed to evaluate existing military installations and for locating sites having a low blast induced liquefaction potential.

The general conditions necessary for soil liquefaction have been known for some time. Terzaghi was the first to describe the mechanics of flow failure due to liquefaction. Others amplified these considerations and introduced the concept that liquefaction may also occur as the result of several types of vibration, including earthquakes and explosions. Earthquake motions have received a great deal of attention by the research community (Seed and Idriss, 1967; Peck, 1979). However, comparatively very little attention has been given to liquefaction as a result of explosive loadings. Like earthquake induced liquefaction, its existence is site dependent and until recently escaped detection and concentrated study.

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[1] Definition: "Liquefaction - the act or process of transforming any substance into a liquid. In cohesionless soils, the transformation is from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress." (ASCE-GT, 1978)

II. BLAST AND SHOCK PHENOMENA

In discussing blast induced liquefaction and the possible mechanisms for creating it, it is helpful to categorize the ground motion field on the basis of waveform characteristics and motion types. Figure 1 illustrates the local and upstream components of ground motion by time of arrival contouring. The propagating velocity of the air blast is very high close-in, decaying to near acoustic velocities at distant ground ranges. Figure 2 illustrates various regions having similar ground motions caused by a surface explosion. Although particle velocities and accelerations are not shown, amplitudes will be greatest in Region I and lowest in Region IV with Regions II and III approximately equal. In Region I near the detonation point, the wave forms are dominated by the crater formation process, i.e., by the crater-related motions which are occurring at very low frequencies and which are, in large part, gravity-dominated. In addition a very intense vertical loading from the air-blast is observed which is followed by upward and outward motions creating material ejection, or the crater-related motions. Region II is where the motion is dominated by sperical expansion of the shock wave as would be in the case of buried or a near surface blast wherein the target point was sufficiently deep to avoid surface airblast effects. Region III is where the near-source energy has attenuated to a level where the motions are dominated by the local airslap. There will be upstream effects, but the general picture of Region III would include a very strong vertically downward directed

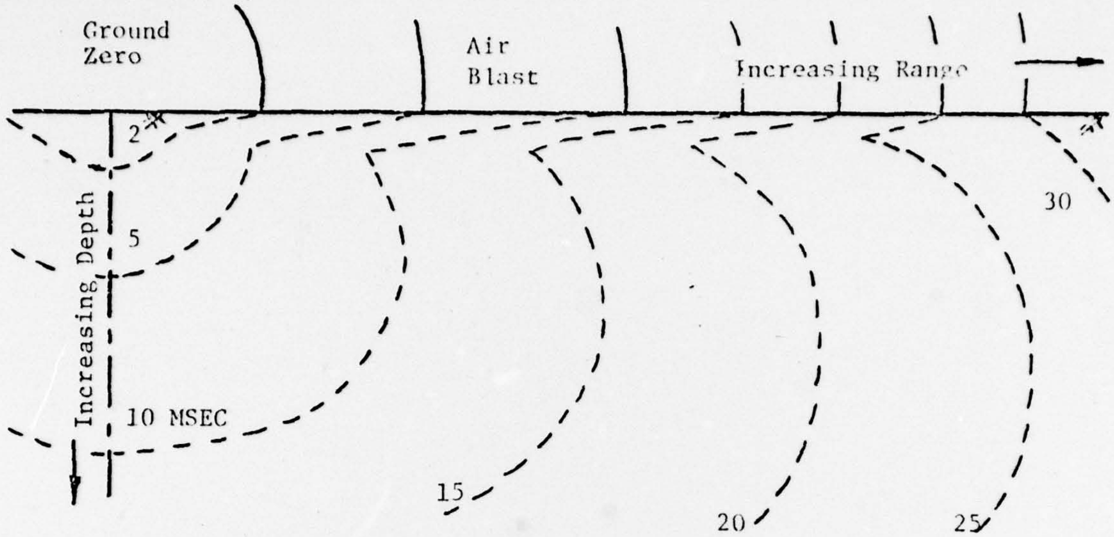
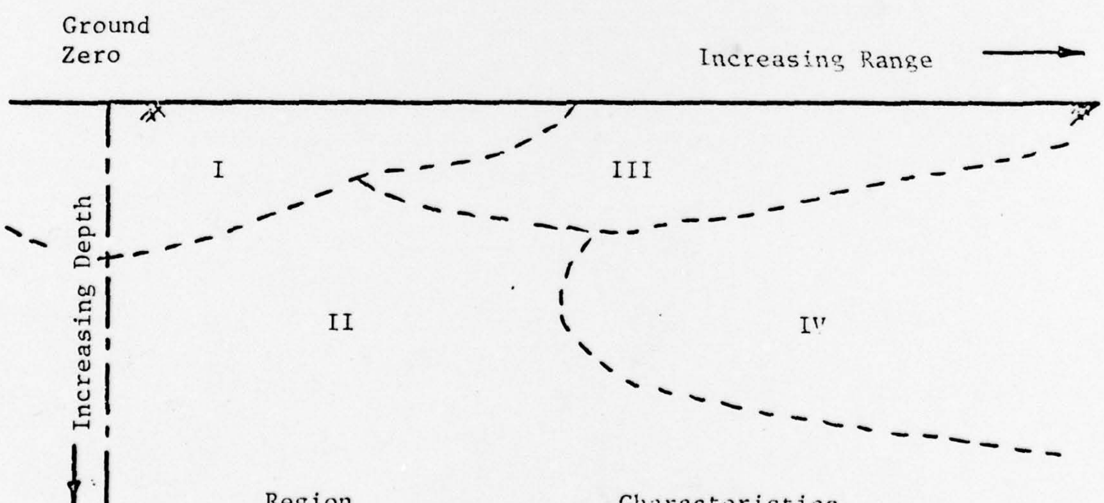


Figure 1. Local and Upstream Components of Groundmotion by Time of First Arrival Contouring.



Region	Characteristics
I	Crater Region
II	Spherically Diverging Motion
III	Local Airslap Motion Dominating
IV	Surface (Rayleigh) Wave Region

Figure 2. Regions of Similar Ground Motion.

airslap. Region IV, which is also very important for blast induced liquefaction studies, is where the motions are typically oscillatory, dominated by surface waves. Region IV goes from intermediate ground ranges (100 psi to 50 psi surface air over-pressure ranges) to where motions are no longer perceptible. Similarity between blast and earthquake wave forms are apparent only in Region IV where a large number of cycles of motion having low accelerations occur. The other regions differ in respect to accelerations, number of cycles of motion, and wave type. In addition, especially in Regions I and II, the point source geometry allows tensile hoop strains accompanied by intense radial and vertical compressive strains, a strain path quite unlike earthquake induced strains.

III. DESCRIPTION OF LIQUEFACTION

The almost universal nature of all materials, including soil, to lose strength in increasing number of repeated loadings is termed "fatigue". Research on many materials has shown that the number of cycles required to reach failure decreases with increasing stress intensity. When a loose dry sand is subjected to stresses sufficient to cause intergranular slip, the rearrangement of grains leads to volumetric compaction. However, for saturated sands, compaction is retarded because the water cannot drain instantaneously to accommodate the volume change. Therefore, the relaxing sand skeleton transfers some of its intergranular (effective) stress to the porewater. This increases the porewater pressure reduces the soil's shear strength.

IV. PREDICTION METHODS

The prediction of blast induced liquefaction has proceeded along two paths, the empirical methods which rely on experimental data and the mathematical methods which attempt to describe the observed phenomenon.

Russian - Experimenters have studied blast induced liquefaction extensively and have developed empirical tests to determine the liquefaction potential of soils. A standard technique used to study the possibility of liquefaction of sand layers consists of detonation of a 5-kg charge at a depth of 4.5 meters. The criteria for liquefaction developed is that if the average settlement within a 5-m radius is less than 8-10 cm, there is no need for precautions against liquefaction (Florin and Ivanov, 1961; and Marti, 1978). Based on experimental laboratory and field studies of shock waves, Lyakhov (1961) found that liquefaction did not occur in compact sands with densities greater than 1.60 g/cm^3.

The Russian work up to 1967, the date of the most recent available publications, also included empirical formulas to predict the most effective methods to consolidate loose saturated sands with a wide variety of explosive configurations. The Russians have determined that the most effective means of producing liquefaction is when the charge is contained (no crater). This was found to occur when the relationship between the depth, h, in meters and the charge of weight, C, in kg, was:

$$C = 0.055\ h^3 \qquad \text{(Equation 1)}$$

This equation was developed for ammonite 9 and 10 (see Enclyclopedia of Explosive and Related Items - Picatanny Arsenal - Dover, N.M., USA).

For TNT, equations giving the weight, C, should be multiplied by 0.85. For a charge in intimate contact with the groundwater, the depth of possible liquefaction is approximately 1.5 times the depth of the charge. The radius of liquefaction, R_{max}, can be calculated by:

$$R_{max} = K_3 \sqrt[3]{C} \qquad \text{(Equation 2)}$$

where R_{max} is in meters, C in kg, and K_3 is an empirical factor (see Table 1).

TABLE 1. Empirical Factors to Calculate Liquefaction Radius (Marti, 1978)

Type of Soil	Relative Density	K_3	K_4
Fine Sand	0 - 0.2	25-15	5 - 4
Fine Sand	0.3 - 0.4	9 - 8	3
Fine Sand	more than 0.4	<7	<2.5
Medium Sand	0.3 - 0.4	8 - 7	3 - 2.5
Medium Sand	>0.4	<6	<2.5

To increase the extent of liquefaction, multiple charges have been laid out in a checker work pattern. For this configuration, the most effective horizontal spacing, S, between the charges is:

$$S = 2 \cdot K_4 \sqrt[3]{C} \qquad \text{(Equation 3)}$$

where S is in meters, C in kg, and K_4 is an empirical factor (see Table 1). Experiments have also been conducted where charges were placed at various depths in a sand layer, with the upper charges being detonated first and progressive detonation of the deeper charges. This procedure causes an increase in pore water pressure (a decrease in effective stresses), and along with the multiple ground motion, results in the depth of liquefaction to be increased (Marti, 1978).

European - The Tri-Nation Working Group (Swiss, Dutch and Germans) have expanded the Russian work to include a flow mechanism. Their work also includes experiments, both in the field and laboratory, in which pore pressure and stresses were measured. Several different pore pressure gages are used presently, with none of the gages being able to measure the initial high frequency portion of the waveform. The theoretical work utilizes the same theories as the Russians, except for unloading. The behavior of the soil mass is separated into the solid and fluid components of stress. As the fluid continues to recover strain, effective stress continues to drop to zero in a cohesionless soil. With time, consolidation of the soil relieves the excess pore pressures (Schapermeir, 1978). Although their work is promising, there is still a great deal of research to be completed.

North American - The United States Air Force and the Army Waterways Experiment Station are developing effective stress models to predict the dynamic portion of the shock wave. Field and laboratory shock tests are planned to study if factors known to be important in earthquake-induced soil liquefaction are also important in blast induced soil liquefaction. A review of earthquake-induced soil liquefaction shows the following significant factors:

a. Soil Type - Liquefaction initiates in saturated cohesionless (sandy) soils. Uniformly graded soils appear to be more prone to liquefaction that well graded soils (Lee and Fitton, 1969) and that for uniformly graded soils, fine sands tend to have a higher liquefaction potential than coarse sands, gravelly soils, silts or clays. Limited data appears to indicate that an increase in the clay fraction reduces the liquefaction potential (Seed, 1967).

b. Initial Relative Density - Other factors constant, the higher the relative density, the lower the liquefaction potential (Seed and Idriss, 1971).

c. Initial Effective Stress - Other factors constant, the higher the initial effective stress, the lower the liquefaction potential. The effect was shown in the field during the Niigata earthquake with soil under a 9-foot fill remaining stable, but similar soils surrounding the fill liquefied extensively (Seed and Idriss, 1967).

d. Intensity of Ground Shaking - Other factors constant, the higher the intensity of ground shaking (accelerations or stress changes), the higher the liquefaction potential (Seed and Idriss, 1971).

e. Duration of Ground Shaking - Other factors constant, the longer the duration (i.e., more strain cycles) the higher the liquefaction potential (Seed and Idriss, 1971). One large stress increase or many smaller cyclic stress increases may cause liquefaction under the right conditions.

f. Initial Shear Stress - Other factors constant, liquefaction will be induced more easily under level ground conditions than in sloping zones of a deposit (Seed, 1967).

g. Permeability - Liquefaction can only persist as long as high excess porewater pressures persist in a soil. Therefore, with other factors constant, the higher the permeability of the soil mass, the lower the liquefaction potential.

h. Degree of Saturation - Other factors constant, the higher the degree of saturation, the higher the liquefaction potential.

V. INFLUENCE OF UNSATURATED CONDITIONS

The degree of saturation is very important when conducting liquefaction tests. Since the degree of saturation may not be constant with depth, scaling factors may be unreliable unless the degree of saturation is accounted for. Tests have shown that liquefaction is very difficult, if not impossible, to achieve at initial saturations below 85 percent. Typical saturations are less than 80 percent for soils located above the capillary fringe. Martin, Finn and Seed (1978) noted that the number of cycles to cause liquefaction in a simple shear test increased by a factor of about 30 for 99 percent saturation relative to 100 percent saturation. In the laboratory, complete saturation is required for running "standard" earthquake liquefaction tests, since this leads to liquefaction in the least number of cycles. However, for field testing, complete saturation is rare near the ground surface.

Entrapped gas also attenuates and influences the velocity of compression (P) waves. In saturated zones, P wave velocities in soils cluster around the speed in pure water (4800 feet per sec) with reported values of 4000 to 7000 fps. Above the capillary fringe, where gas exists as interconnected voids at atmospheric pressure, P wave velocities are in the range of 600 to 2000 fps. Below the capillary fringe, other zones may exist which may be unsaturated with the gas in isolated bubbles. A saturation of 80 percent is generally regarded as a level above which most of the gas matrix exists in isolated bubbles. These isolated bubbles may occur in the capillary fringe and in the positive water pressure zone below the water table. Such bubbles in the soil may result from several processes, such as bacteria action, temperature changes, a changing of the groundwater level, or induced by man.

Air entrained in or around monitoring equipment, especially piezometers, influences the accuracy of field data, especially at high frequencies. Peaks and reversals of pressure actually occurring may be damped out or lost if piezometers have air bubbles nearby. This is not much of a problem when monitoring the late time porewater pressures after a blast. However, for studying porewater pressures and effective stress changes during blast loading of saturated soils, insuring saturation around the equipment is a must. A review of the literature indicates little research into high frequency dynamic behavior of soil containing entrapped gas.

Occurrence - Although customary geotechnical practice and laboratory liquefaction tests assume saturated voids below the water table, the physics of water content distribution often dictate that this is not the case for up to several meters below the phreatic surface. In preparing a sand bed for a blast tests, Drake (1978) found that six months after upflow wetting, only 85% saturation had been reached.

The degree of saturation in soils will distribute in a continuous manner during and after a change in the hydrologic conditions. At a given point in time after such a change, a typical profile is shown in Figure 3.

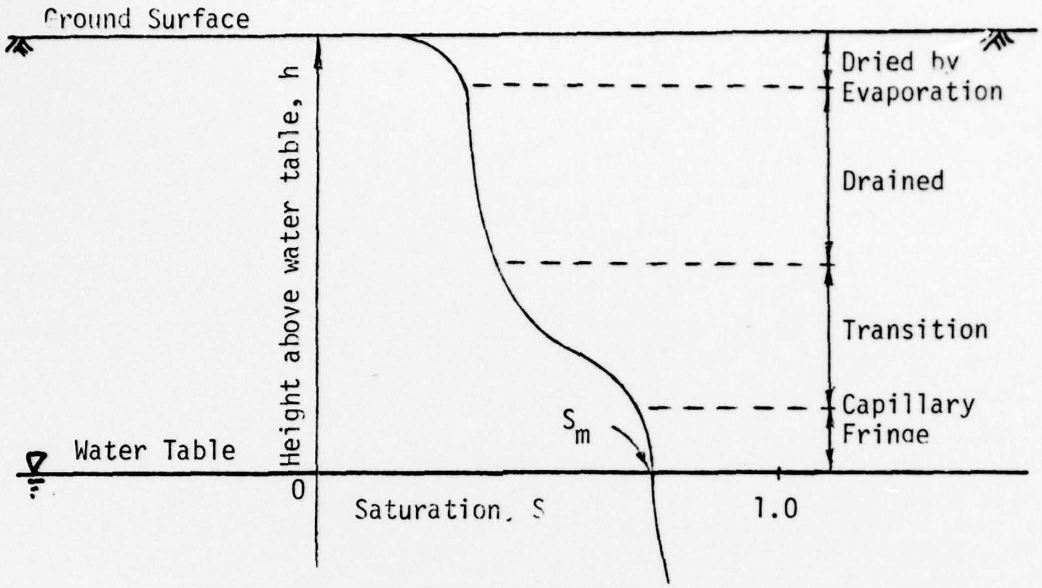


Figure 3. Distribution of degree of saturation in a homogeneous soil profile with water table. (Corey, 1978)

The maximum saturation, S_m, in the capillary fringe occurs just above the phreatic surface (h = 0). Continuity requires that if S_m is less than 1.0, a portion of positive pressure zone below the water table also be unsaturated, with air contents from 5 to 20% typical in the first few meters.

Over time the positive pressure gradient shown in Figure 4, and dissolution of entrapped gas would cause the zone below the water table to become fully saturated. Continuity now requires a portion of the negative pressure zone above the water table to also become saturated as shown in Figure 5. However, a finite amount of time is required for this de-airing, and disturbances of the water table tend to re-establish the unsaturated condition.

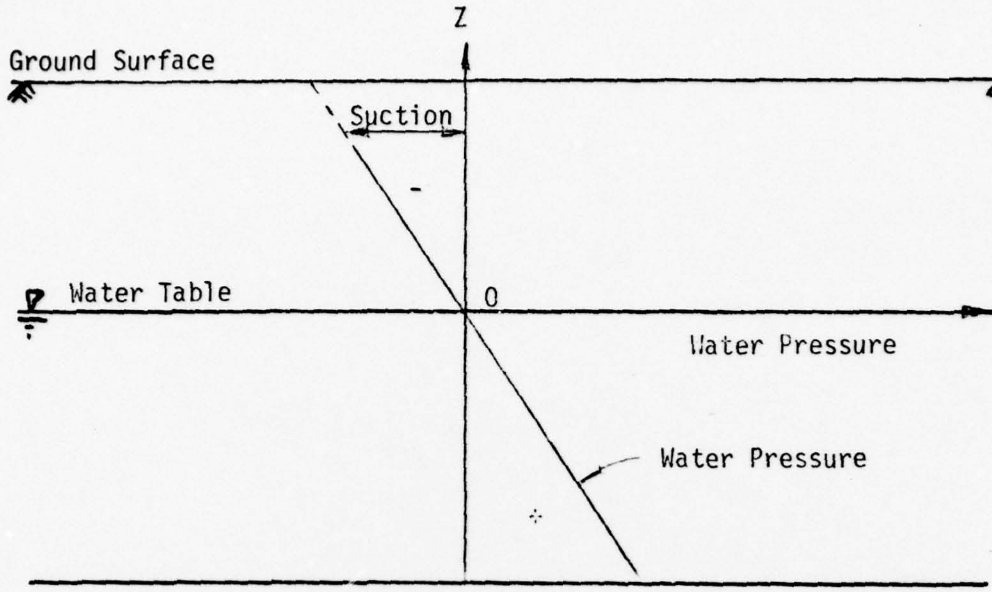


Figure 4. Distribution of water-pressure head in static subsurface water.

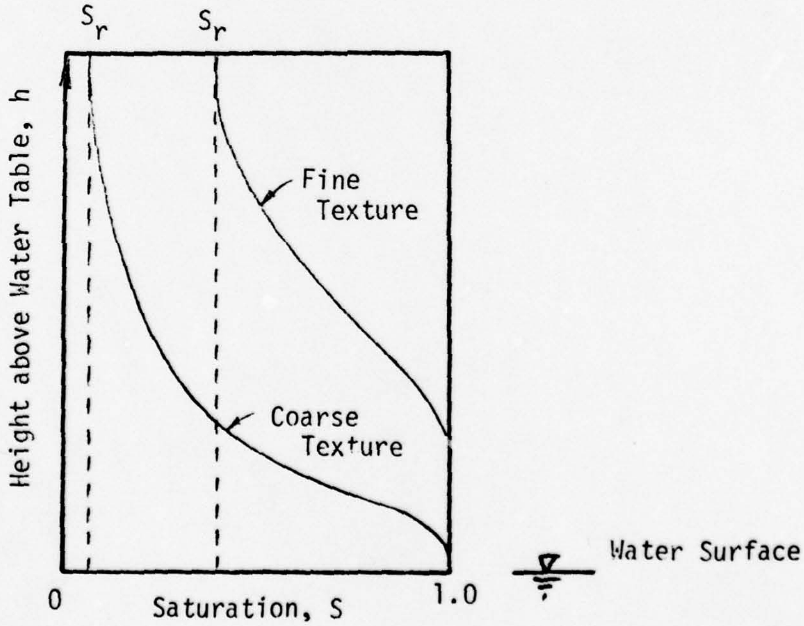


Figure 5. Typical water retention curves. (McWhorter and Sunada, 1977)

Hanson (1977) reviews the causes of entrapped gases. Air is usually entrapped in a rising water table. The prime influences in air entrapment below the water table are the soil pore size distribution (a function of grain size distribution and density), the rate and direction of advance of the water table or wetting front, and soil layering. Christiansen (1944) reports that wetting from above apparently results in higher air contents. Gupta and Swartzendruber (1964) showed that flow of water tends to reduce air entrainment. Lateral flow (such as stream recharge) or a rise in the water table from positive gradients tended to give higher saturations than inflow from irrigation or rainfall recharge. In addition, layering of fine and coarse soils (Figure 6) favors low degrees of saturation but also takes longer for air displacement after initial pressure establishment.

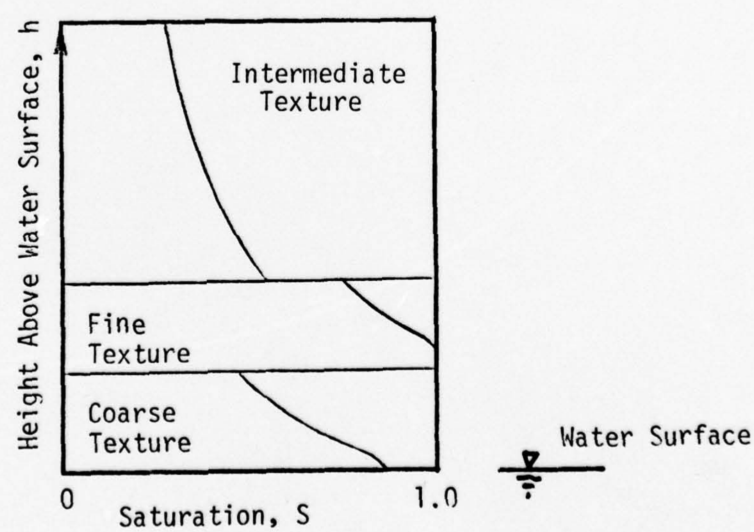


Figure 6. Equilibrium distribution of degree of saturation above a water table in stratified aquifer. (McWhorter and Sunada, 1977)

In addition to the occurrence of entrapped gas from groundwater fluctuations, researchers have identified several other mechanisms. Sedimentation may result in initial air and organic entrapment. Smith et al (1960) noted this in stratified layers of silt and clay in Lake Mead deposits. Hoyt and Henry (1964) noted a similar phenomenon in beach sands. Moran (1936) noted that release of pressure on a sample allows evolution of gas from solution, primarily in soils with organic material. Methane, carbon dioxide and hydrogen sulfide are often involved from biological decomposition. Chaplow (1974) noted that some bubbles occur from sample disturbance, including pressure release, while others are the result of vibration, which may have occurred historically at a particular site. Hanson (1977) also noted air release with temperature changes.

Detection - Given that entrapped gas decreases liquefaction potential, interpretation of field tests requires knowledge of the extent of such conditions. Moreover, since even fractions of a percent of entrapped gas have a strong influence on compressibility of the pore fluid, the detection precision required is quite exact. Unfortunately, in-situ testing to the required precision is quite difficult, expensive, and subject to several levels of interpretations. Thus, in a site-search program, a progressive series of tests is advisable, starting with relatively quick analysis to determine if entrapped gas is likely to be present in significant quantities.

Qualitative Information - Thus, on a first analysis, full or nearly full saturation is likely below the water table given the following site characteristics:

a. Fairly stable water table elevations.

b. Continuous soil type below the water table, increasing in density with depth.

c. Evaporative upflow through the capillary zone.

d. Laterally recharge from distant surface recharge sites and minimal surface recharge.

e. No organic entrapment.

However, this type of analysis gives qualitative information only.

Quantitative Information - Computation of saturation as a function of P wave velocity is greatly influenced by the soil skeleton properties and the size and pressure of the bubbles. On a theoretical level, considering an air-water mixture only, a wide range of P wave velocity can be predicted depending upon the assumptions made. In general, the lower the compressibility, the higher the P wave velocity.

Neglecting the influence of surface tension, but accounting for Boyles Law and Henry's Law of Solubility, Koning (1963) postulated:

$$\frac{1}{Kw} = \frac{1}{Kwo}\left(\frac{1-Va}{V}\right) + \left(\frac{1}{Ka}\right)\left(\frac{Va}{V}\right) \qquad \text{(Equation 4)}$$

with Kw equalling the modulus of compressibility of the air-water mixture; Kwo equalling the modulus for pure, de-aired water; V equalling total initial volume, Vao + V_W; Vao equalling the initial air volume; Vw equalling the water volume; and Ka equalling the modulus of the air bubble. However, the value of Ka is variable with pressure and, therefore, this equation may not be appropriate for blast induced liquefaction studies in regions near the blast. A more comprehensive expression, such as presented by Schuurman (1966), may be more appropriate near the blast. However, using the basic compression wave velocity and Koning's expression, small changes in saturation has a considerable influence on the velocity, whereas small changes in soil density produce only small changes in velocity.

Domenico (1974) in laboratory tests with pure quartz sand at various saturations, noted a general increase of P wave velocity with increasing saturation, with a very sharp increase around 94% saturation. Researchers quoted by Johnson (1974) applied this concept of material identification by Vs and saturation by Vp. Bailey and Van Alstine used P and S waves in a downhole survey through sands and shales, detecting confined aquifers and unsaturated layers. At a saturated boundary, a sharp increase in the compressional velocity occurs, while the shear velocity remains virtually unchanged. For a solid rock boundary, one should expect increases in both compressional and shear velocities.

It can be seen that high resolution of the degree and depth of entrapped air is difficult from these surface tests. However, much useful information can be gained with a single boring at the site. The change of shear modulus with density, and indirectly, porosity can be calibrated. The borehole will also locate the phreatic surface on the P wave curve. Together, a rough approximation of saturation in both the capillary fringe and the upper water table zone can be calculated, giving sufficient data for decisions of site acceptability and/or further exploration.

Gravimetric analysis of saturation from standard tests is well developed for clays, and to some extent for silts, but very difficult for sands. The two relevant values are porosity and water content. Well conducted sampling with thin walled, hydraulically pushed tubes give a reasonably "undisturbed" clay sample for saturation determination. Thus, where the water table is in clays, preliminary tests can give a good saturation profile. Down hole nuclear density and water content probes give reasonable values. However, the soil sampled around the hole may be disturbed and the equipment's accuracy is less than the required degree of saturation accuracy. Equipment improvement and calibration in the future may make this method attractive.

A critical parameter in dynamic analysis is the relative density. Various investigators have researched the relation between the blow count and relative density, and a temptation is to derive porosity from this value, and compare it with the sample water content for saturation computation. Marcuson (1977) describes a series of tests at the Waterways Experiment Station. The article deals with not only correlation of the Standard Penetration Test (SPT) and relative density, but also undisturbed sampling of sand. Based on the actual density of the soil, its structure, grain size distribution and the in-situ stress conditions, the following conclusions were reached:

a. The SPT is not sufficiently accurate for final evaluation of either the absolute or relative density at a site unless site specific correlations are developed.

b. High quality undisturbed samples can be obtained using a fixed-piston sampler and drilling mud. This procedure yields very good samples of medium-dense sands, but tends to densify loose deposits and loosen dense deposits.

The implications are that sufficient data can be gathered by standard subsurface testing for site selection, but standard testing gives only a general indication of unsaturated conditions in the sands likely to liquefy, but can give reasonable estimates for soils elsewhere in the profile.

Removal of Entrapped Gas - After gas is entrapped below the water table due to imbibition or biological respiration or fluctuations in the phreatic surface, processes initiate to remove or dissolve gas in zones of positive water pressure. There are three general mechanisms for gas removal from pores. One process is dissolution of the gas phase into water, well known in chemical engineeing and water treatment technology. A second process is circulation of de-aired water. This process is customary in de-airing piezometers and laboratory samples. A third mechanism, diffusion due to positive and capillary pressure gradients, has recently been investigated in porous media studies for application in agricultural and hydrocarbon recovery activities (Adam, Bloomsburg, and Corey, 1969).

Dissolution of gas into fluids follows Henry's Law, Cs = (kg)(P) where C_s is the saturation concentration of the gas into water, kg is the coefficient of absorption, and P is the partial pressure of the gas in the gas phase. Over time, the rate of absorption is

$$\frac{dc}{dt} = kg(C_s - C_t) \qquad \text{(Equation 5)}$$

with C_t the concentration at any point in time.

All of the concentration parameters can be measured by sampling and standard water tests. However, the interest in this project is the total volume and distribution of bubbles in a porous media, so that evaluation of kg, which varies with time, is difficult. kg generally is a function of the area of interface per unit volume of liquid, temperature, pressure and fluid circulation. For similar degrees of saturation and fluid pressures, coarse soils have a smaller bubble suface area per volume ratio than fine soils. Thus, contrary to first expectations, dissolution proceeds at a slower rate in porous sands than in clays.

Circulation of the pore water includes convective removal of the bubbles, dispersion of dissolved gas in the fluid and disturbance and breakups of the bubbles to produce more interface area. This is a much more complex process in soils than open water, but is the most rapid mechanism of entrapped air or other gas removal. As per porous media theory, permeability increases with fluid saturation, making artifical circulation more efficient at high saturations (85% plus).

Diffusion under capillary pressure is a third de-airing mechanism. Recall that the differential pressure between air and water in a porous media is calculated from

$$\Delta p = \sigma\left(\frac{1}{r_1} + \frac{1}{r_2}\right) \qquad \text{(Equation 6)}$$

with σ the interfacial (surface) tension and r_1 and r_2 the principal radii of the subsurface pore air-water interface. In the discontinuous entrapped air phase below the capillary fringe, the excess Δp is a driving force. Even if the fluid is fully saturated with the dissolved gas according to Henry's Law, it has been shown that entrapped air can diffuse through it under the pressure gradient to the exposed continuous gas phase at the capillary fringe.

Adam (1967) presented the following diffusion relationship for unit volumes of media:

$$\frac{Vd}{Vt} = f \frac{C_o, D_e}{(1-S_o), \phi, L} \qquad \text{(Equation 7)}$$

with Vd the volume of gas diffused at any time, Vt the original gas volume, D_e the effective diffusion coefficient, L the distance to the surface, C_o the excess concentration, S_o the fluid saturation, and ϕ the porosity. D_e is a function of the pore size distribution, bubble size, pressure distribution, and other driving force parameters. Most of these parameters can be reasonably estimated from sampling. Experimental studies by Adam, Bloomsburg and Corey (1969) confirmed that diffusion, as well as dissolution, proceeds faster in fine soils than coarse.

The implications for blast implications are:

a. In general, fine grained soils de-air faster than coarse soils.

b. The method and rate of initial wetting, whether it fills large pores or smaller ones first affects the de-airing processes.

c. Loose fine sands, the soil type most likely to liquefy, will have average de-airing properties relative to coarse or finer soils.

d. The historical range of water table fluctuation determines the depth of partial saturation. De-airing is oppositely influenced both by the length of escape path and the positive pore fluid pressures.

e. Hydraulic conductivity increases with fluid saturation of the media, so circulation near piezometers is most effective at high initial saturation.

f. Even with relatively minor water table fluctuations, unsaturated conditions may vary seasonably, highest during spring vertical recharge periods, lower during upflow evaporative conditions in late summer.

g. Coarse layers below fine soils tend to be more unsaturated than homogeneous soils.

h. Biological activity, temperature changes, etc. may make achieving 100% saturation in the field very difficult.

VI. METHODS TO REDUCE LIQUEFACTION POTENTIAL

If a site is found to have a high potential for liquefaction, various methods are available to reduce the potential. The most direct approach is to relocate the structure to a site with a lower liquefaction potential. If this is not feasible, various methods of modifying the site are possible. Since a loose soil is more susceptible to liquefaction, increasing its density will decrease the liquefaction potential. Various methods have been used to increase the field density and include compacting the soil in-site by use of pile drivers, vibroflotation, blasting, dropping large weights, etc. (Janes and Anderson, 1976). Another method involves removing the soil with a high liquefaction potential and simply replacing it with a soil of low liquefaction potential or putting the original soil back in at a higher density. Modification of the soil by use of chemicals and grouting has also been used. Various methods to increase the effective stresses on the soil have been used to decrease the liquefaction potential. These include lowering the ground water level and/or placing a fill over and around the site (Seed and Idriss, 1967; Florin and Ivanov, 1961). Seed and Booker (1977) discuss stabilization of potentially liquefiable sand deposits using drains. If the soil with a high liquefaction potential is located near the surface, it may be possible to place the foundation at a depth below the soil which may liquefy. However, the use of long piles to accomplish this requires extensive investigations since the piles may buckle if the soil around them liquefies (Peck, 1977). Lyakhov (1961) recommended an artificial increase of the air content of a soil along with increasing the density to prevent blast induced liquefaction. For compact sands with densities greater than 1.60 g/cm^3, liquefaction was absent at any value of saturation. Below 1.50 g/cm^3, the presence of entrapped air did not exclude the possibility of blast induced liquefaction. However, the presence of entrapped air reduced the liquefaction potential, increased the soil damping and reduced both the amplitude and speed of the shock wave.

VII. POREWATER PRESSURE MEASUREMENT

Whereas the field measurement of pore pressure has developed over forty or fifty years, only recently has this work been extended beyond static or semi-dynamic applications. As noted by Vaughan (1973), most such measurements are used to check assumptions made in effective stress analysis or to monitor the effectiveness of drainage or dewatering works. Blast induced liquefaction following passage of blast waves falls into these categories, but measuring pressures during shock waves, which include high frequency, high magnitude stress reversals, requires new methods.

Besides the usual list of requirements such as durability, ease of operations, etc., the primary concern in static or semi-dynamic (low frequency) piezometer use are the compliance and response time to equalization. The following table from Vaughan illustrates the relative compliance and response time for common sensors used in static applications.

TABLE 2. Typical Piezometer Reponse Times

Piezometer System	Response Time (hours)
Borehold piezometer, sand pocket 60cm x 15cm dia. 1.5cm dia. standpipe	820
Ditto with 150m 3mm bore nylon tube & 3mm Hg manometer	1.6
Ditto with electric transducer in the piezometer cavity	0.002
Ditto with transducer with $100cm^3$ air in the cavity	19

Coefficient of permeability of soil, $k = 10^{-10}$ m/s
(from Vaughan)

However, the required response times decrease dramatically for dynamic blast tests as the desired reading time approaches the blast instant. Thus, the hydraulic or pneumatic piezometer used in civil works are not usable in field dynamic tests, except for post-blast late time data.

In the last decade, mechanical, electric and electronic transducers have been developed, including vibrating wire strain gauges, piezoresistive and piezoelectric types, with sophisticated readout and recording systems. Application in the laboratory on saturated samples has been satisfactory for cyclic stress, blasting and projective impact studies. One problem that has been encountered is that of installing the transducer without changing the soil saturation. The use of various fluids and soil de-airing techniques have reduced this problem. Besides pre-test transducer calibration, static calibration of the installation can be done before and after the test.

In field applications, however, all of this control is restricted, and a whole new host of new problems arise with isolation of instrumentation and soil-instrumentation interaction. Installation using either borehole ejection, driving or test pit methods, introduces some disturbance. Amplification for transmission over long distances, noise, and interference further add to the problems. Hence the problems with intergranular stress measurement are monumental. Fortunately, in saturated, unconsolidated deposits the pore water is stiffer than the soil, so that local disturbance of the soil matrix does not seriously disrupt pore pressure measurements. It must be remembered, however, that dense soils tend to dilate under stress, so recorded negative pore water pressures during stress pulses must be analyzed in the light of possible local dialation. Bassett (1978) noted that Dutch Cone driving affects the soil for 20, perhaps 40 diameters radially, and at the tip, the relative density reaches 90% regardless of initial void ratio.

The most serious error source with pore water pressure measurements during dynamic impulses is that of air inadvertently entrapped in or around the transducer. The previous section discussed the effects of entrapped bubbles on liquefaction potential and wave velocity and attenuation for the soil mass.

Results of gas entrapment on the accuracy of pore pressure measurements are as follows:

a. There will be a severe impedance mismatch between the undisturbed water saturated soil and the soil surrounding the stress gauge; only a fraction of the true dynamic stress will be transmitted to the gauge, because the stiffness of the soil in the vicinity of the stress gauge is much lower than that of the undisturbed soil.

b. Of the stress actually transmitted in the vicinity, the distribution will change between the pore fluid and grain matrix in favor of the grain matrix. Thus the stress indicated by a dynamic pore pressure gauge will be reduced, while the stress indicated by a gauge for grain matrix stress will be enhanced.

Of course, after the wave period of stress reversal, pore pressure equalization will occur locally, so that accurate readings can be made in time span presently regarded as that of potential liquefaction. Thus, deliberate bubble entrapment is used as a method to protect piezometers when high inertial stress conditions are expected.

Thus, the planning of piezometer installation starts with the decision as to which period after the blast is of interest. The technology and procedures are available for accurate detection of water pressures after the first few milliseconds after blast effect. Before that time measurement is difficult and accuracy is suspect. Research is needed in this area.

VIII. CONCLUSION

The phenomenon called blast induced liquefaction has been examined. Today's understanding of the phenomena has advanced only slightly beyond the point of recognition of its existence. Documented occurrence, although sketchy and often incomplete, is now in the open literature. For field blast tests, the degree of saturation influences the liquefaction potential and dynamic porewater pressure measurements. However, methods to accurately determine the degree of saturation of many field deposits are not available. Since the degree of saturation may vary with depth, with time (either by nature or man), small field tests and scaling factors may be unreliable unless the degree of saturation is accounted for. Considerable work remains in projecting this information to a comprehensive method of predicting liquefaction for actual or hypothetical future blasts. Until additional research is completed, loose saturated sands should be regarded as having a high blast induced liquefaction potential.

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APPENDIX

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