THEORETICAL ANALYSIS OF DYNAMIC PRESSURE PROPAGATION IN THE PORE FLUIDS OF SOILS

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U. S. NAVAL CIVIL ENGINEERING LABORATORY
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ABSTRACT

A study has been made of the propagation of pressure waves through the pore fluid of a soil. Dry, saturated, and partially saturated cases are considered for chesive and cohesionless soils. Tentative recommendations are made concerning assumptions to be made and methods to be used in predicting the pore pressures resulting from impulsive loadings.

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INTRODUCTION

Although a great deal of effort has been devoted to an examination of stress wave propagation in dry sands and gravels (11, 15, 16, 17, 18, 19, 26, 28, 29, 30, 32, 33, 36, 37, 42, 43, 44, 45), very little information is available regarding the behavior of saturated and partially saturated soils under the influence of impulsive or blast loadings. In addition, only a very limited amount is known about the pore pressures developed in dry soils due to these loadings (10, 13, 14, 35).

Most actual designs for blast-resistant buried structures will probably be concerned with soils which are partially saturated or saturated. In those cases where the soils are dry, the pressures in the pore air may be of concern if the effective stresses in the design region are affected.

The study reported here was undertaken in order to establish a basis for design estimates and to define the needs for further investigations.

SCOPE OF THIS REPORT

This report presents a discussion of the mechanism of the shock transmission and recommends procedures for making estimates for design purposes. Various situations in which the pore pressures may be of concern are considered, and possible design solutions are suggested. Certain experimental work is recommended in order to verify or, if necessary, modify the conclusions arrived at in this study.

PORE FLUID PRESSURE EFFECTS

The effect of pressure in the pore fluid of the soil is to reduce the effective stress (i.e., grain to grain contact stress) in the soil mass. This, in turn, results in a reduction of shearing strength and the bearing capacity of the soil. On the other hand, the forces due to the flow of the pore fluids may have the effect of increasing or decreasing the stability of the soil mass depending upon the direction of flow. In the particular problems being considered, the initial flow would be downward and would tend to have a stabilizing influence. Thus, the two effects would tend to offset each other to some extent. It might be noted that under static conditions, a pore fluid pressure of 100 psi would be sufficient to neutralize the effective stress due to the normal overburden at a depth of 140 feet or more in an average soil.

The existence of pressures in the pore fluid will create problems other than lack of bearing. As the effective stress decreases, the coefficient of active pressure increases, while the coefficient of passive pressure decreases. Both of these coefficients attain the value of unity if the effective stress becomes zero. For flexible structures such as arches, which depend upon the passive pressure for part of their support, this results in increased loading with decreased support and might well lead to structural damage or failure. In addition, the increased pressure in the pore fluid might result in excessive leakage into the structure impairing or destroying its usefulness.

The pore pressures may be caused by propagation of impulsive loadings through the pore fluid directly, by compression of the soil mass, or by a combination of these effects.

PULSE PROPAGATION

There are many references which deal with the propagation of stress waves and shock loadings. Kolsky (21) has presented an excellent summary of the theory, as of 1954, of stress wave propagation in solids; and Binder (1), von Mises (40), Glasstone (12), Carrier (5, 6), and others have dealt with the transmission of shock waves in fluids. Clark (7, 8, 9) and Swatosh (38) have studied the characteristics of shock waves in tunnels of various configurations and with various angles of the entrance to the blast wave.

In general, the impulsive loadings studied consist of a compression phase followed by a tension or rarefaction phase. In the case of a shock wave in air, there is a sharp pressure rise followed by a forward facing rarefaction wave. As noted by Clark (9), the forward portion of the rarefaction wave moves at a greater velocity than the pressure jump thus continuously overtaking and decreasing the shock front pressure. A similar characteristic is noted by Salvadori et al (28) with regard to the linear-hysteretic soil model in which the slope of the unloading branch of the stress-strain curve is greater than that of the loading branch.

In the current study, the assumption is made that the problem is one dimensional in character. This is equivalent to assuming that a semi-infinite expanse is loaded simultaneously as in most analyses made for the stress distribution in dry cohesionless soils due to blast effects.

EFFECT OF BLAST ON PORE PRESSURES IN DRY SOILS

General

The pore air pressure in a dry soil subjected to blast loading can be attributed to several factors. The most important of these factors will be the transmission of blast pressures through the pores, the increase in pressures due to the compression of the pores of the soil skeleton, and temperature increases which may accompany the blast wave.

In a dry soil, the soil skeleton is much stiffer than the pore fluid and will transmit the blast wave by intergranular contact. In order for the blast wave to move through the pores of the soil, there must be a flow of air through the voids. The distance to which this pressure will penetrate must, therefore, be closely related to the pore size, shape, and distribution in the soil. It must be a function of the permeability of the soil. The general relationships between grain size and shape, void ratio, permeability, and density are extremely complex. In general, it is not possible to establish a valid and easily used mathematical model to inter-relate these quantities. Laboratory tests are available for determining each of the quantities, and specific standard tests have been established for most of them. The permeability normally reported in soil mechanics literature is the constant in Darcy's law and represents the "physical permeability" divided by the dynamic viscosity of the fluid. It would, therefore, seem possible to obtain the air permeability from the water permeability by multiplying by the ratio of water viscosity to air viscosity. However, it is worth noting that Brooks and Corey (4) found that air permeabilities measured before saturating a particular dry soil were often less than those measured after the soil had been saturated and subsequently dried. It would thus seem that the past history of the soil might have an appreciable effect upon this quantity. In particular, soil deposits which are frequently subjected to wetting and drying may experience increases in permeability after construction.

Cohesionless Soils

The only extensive tests concerning pore air pressures to be reported to date are those of Hampton (13, 14). These tests involved pressure pulses of very short duration, but they do give an indication of what might be expected under blast loadings.

These tests were run on three separate soils--a pea gravel, 20-30 Ottawa sand, and a silty sand. The results of the tests on silty sand showed practically no penetration. Tests were run with the samples in both head-on and side-on positions (see Figure 1) with the incident pressure on the sample and the pore pressures within the samples reported. The results are presented in Figure 1 as plots of the ratio of maximum gage pressure, p, in the pore to maximum incident gage pressure p_o, versus the ratio of distance penetrated, X, to the cube root of the diameter of the particle considered representative of the sample. The particle diameter used in this study is that particle which represents the upper limit of the finest 10 percent, d_{10}, expressed in feet.

The duration of the positive phase of the pressure wave used in the tests was relatively short and particularly so in the head-on tests. This would appear to be the probable reason that the pressure ratios observed in the head-on tests were generally slightly lower than those observed in the side-on tests. It, therefore, seems reasonable and proper to regard the side-on values as a lower limit although there is more uncertainty involved in the measurement of the incident pressures used in this case.

The results shown in Figure 1 could be predicted reasonably well by means of one of the two curves shown. The values obtained by using the equation

$$\frac{p}{p_o} = e^{-0.20(X/d_{10}^{1/3})} \tag{1}$$

would be a very good representation except for values of $X/d_{10}^{1/3}$ greater than 10. Even in that region, the equation would give values within approximately 8 psi of the measured values. It is quite possible that the load pressure readings corresponding to $X/d_{10}^{1/3}$ greater than 10 are subject to larger percentage variations due to extraneous effects such as temperature and normal experimental scatter. These same readings might be a function of irregular pore spacing and sizes which would allow slightly higher pore pressures to develop than would be the case with uniform distribution of grain sizes. Under the circumstances, it would seem advisable to use the curve given by

$$\frac{p}{p_o} = 0.94\, e^{-0.154(X/d_{10}^{1/3})} \tag{2}$$

for making design estimates until further evidence can be collected regarding this point. In applying these equations, one should obviously exercise a considerable degree of judgement in deciding whether or not the diameter of the 10-percent size appears to be the representative particle diameter for the soil being considered. It is always possible

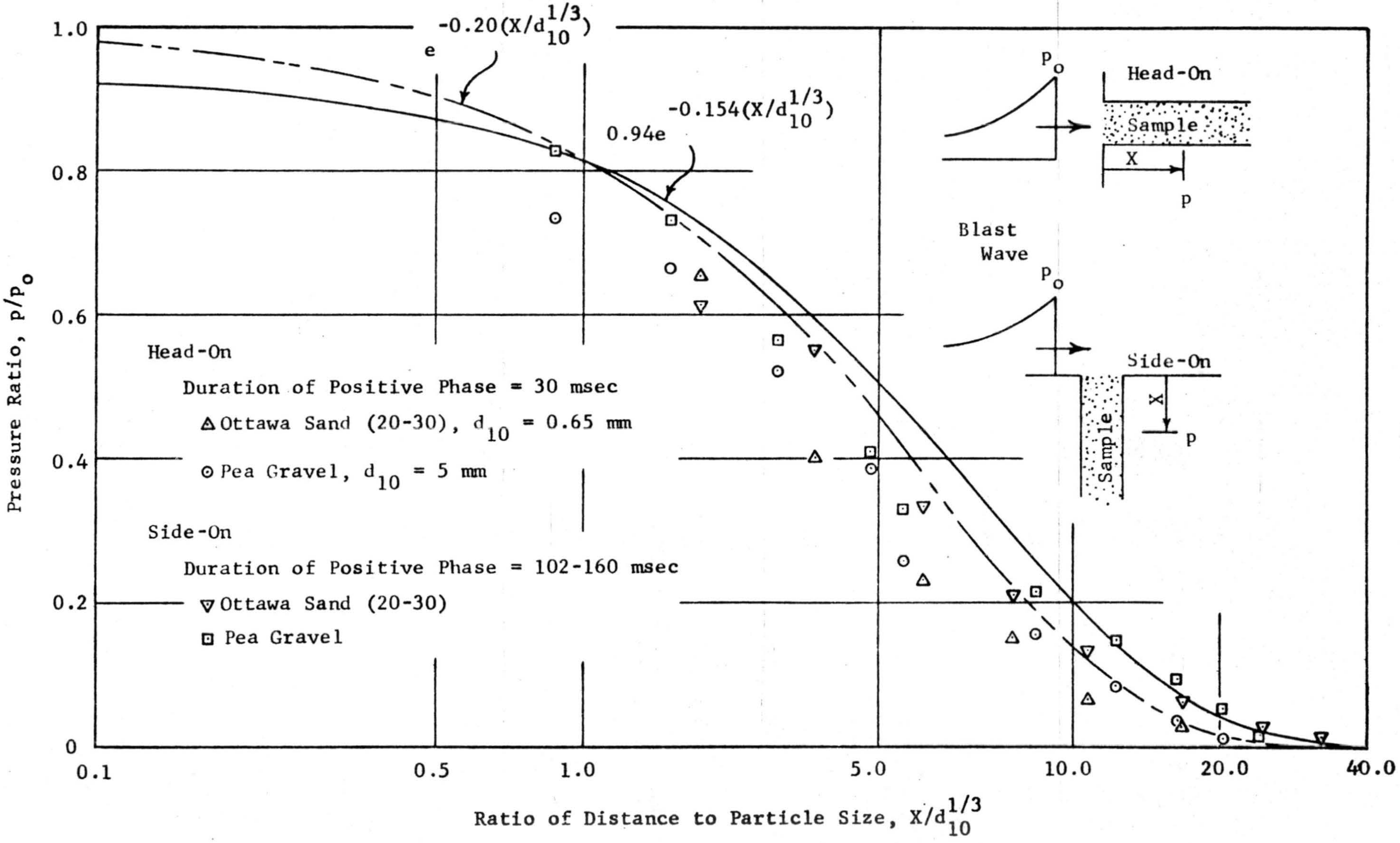

Figure 1. Relationship between pressure ratio and distance-particle size ratio.

that unsymmetrical grain-size gradations, unusual skip gradations, or other irregularities may make the choice of another diameter seem preferable.

Equation (2) will predict pressure ratios which are mostly on the safe side. Where the ratios are underestimated, the error is only about 3 psi. The maximum overestimate using this equation is approximately 25 percent. The general shape of these curves is similar to that obtained using the "hydraulic diameter" approach outlined for tunnels by Clark (9).

In order to verify the relationships discussed, it is desirable to conduct more conclusive tests as outlined in the Recommendations for Test Program section of this report.

Other correlations that have been investigated and discarded are with the ratios of X/d_{10} and $X/d_{10}^{1/2}$. The permeabilities of the samples reported by Hampton varied roughly as the first power of d_{10}.

Hampton's tests were conducted on samples of relatively small cross-sectional area held in rigid boundaries. His report includes a statement to the effect that he tested specimens with the pore fluid isolated from fluid inflow by a plastic membrane and that there was then no increase in pore fluid pressure. For soils in the natural state, longitudinal compression could be expected to cause a 0.3 to 0.8 atmosphere pressure increase in addition to those estimated from the foregoing equations. The actual increase is very much dependent upon the density and dynamic modulus of the soil, and the range quoted should be regarded only as a rough guide.

Hampton also reports difficulty in measuring the velocity of the pore fluid pressure front. He specifically cites an example in which he measured the velocity as 250 fps and dismisses this as obviously in error since he felt that it should at least be equal to the acoustic velocity. The resistance to flow in the small pores will be primarily due to the effects of viscosity, and the flow must take place before the pressure rise can occur. It would, therefore, seem quite probable that the velocity of pore fluid pressure propagation will be less than the acoustic velocity and that it might indeed approach values such as those measured by Hampton. It is thus probable, as Hampton concluded, that a considerable lag might occur between the passage of the soil stress wave and the pore fluid pressure wave. This might well mean that the maximum pore pressures would be more damaging to buried facilities than if they arrived simultaneously with the maximum soil stresses since the maximum effective stress (and hence the passive resistance) might be considerably less than maximum when the pore fluid pressure is a maximum. In addition, the pore fluid pressure might tend to aggravate structural damage already caused by the soil-stress wave.

Cohesive Soils

Dry cohesive soils have very low permeability, and it is unlikely that any appreciable flow of pore fluids can take place during the rapid loading of a blast. The increase in pore fluid pressure would thus be expected to be due almost entirely to compression of the void spaces, although temperature increases would also cause an increase in pore fluid pressure.

The pore fluid pressure increase due to the compression of the soil skeleton would be expected to be from 0.3 to 1.0 atmospheres due to a decrease in pore volume.

EFFECT OF BLAST ON PORE PRESSURES IN SATURATED SOILS

Cohesionless Soils

The commonly accepted value of bulk modulus of water (300,000 psi) is much higher than the value of the constrained modulus reported for most soil tests, e.g., Hendron (18, 19). Being relatively incompressible, the water will tend to absorb all of the blast overpressure and transmit the shock through the water. The result will be a temporary liquefaction of the soil. It might be expected that the stress would rise to a higher value in the soil-filled water than it would in water alone in much the same manner as reported by Carrier (4, 5) for a dust-filled gas.

The reduction of the soil to a liquid state would reduce the subgrade reaction on a structure to the value of the buoyant force and would lead to a condition of zero shear strength. This general condition has already been discussed in the section on Pore Fluid Pressure Effects.

In Figure 2 the depth, time, surface loading diagram is shown for a step loading applied to the surface of a saturated soil. The pressure rises suddenly to a value of p_o and stays there until time t_1 when it reduces suddenly to p_1 which remains on the system indefinitely.

Immediately upon loading, a pressure wave is propagated at a velocity c_o. This is shown in the diagram as a line through the origin having a slope of c_o. At time t_1, an unloading wave is propagated into the mass at a velocity dependent upon the bulk modulus. Assuming that the bulk modulus remains approximately constant, this velocity will also be c_o and the unloading wave can be represented in the X, t diagram by a line through $(0, t_1)$ having a slope of c_o. Under these conditions, the two wave propagation lines in Figure 2 are parallel and the unloading wave [illegible]

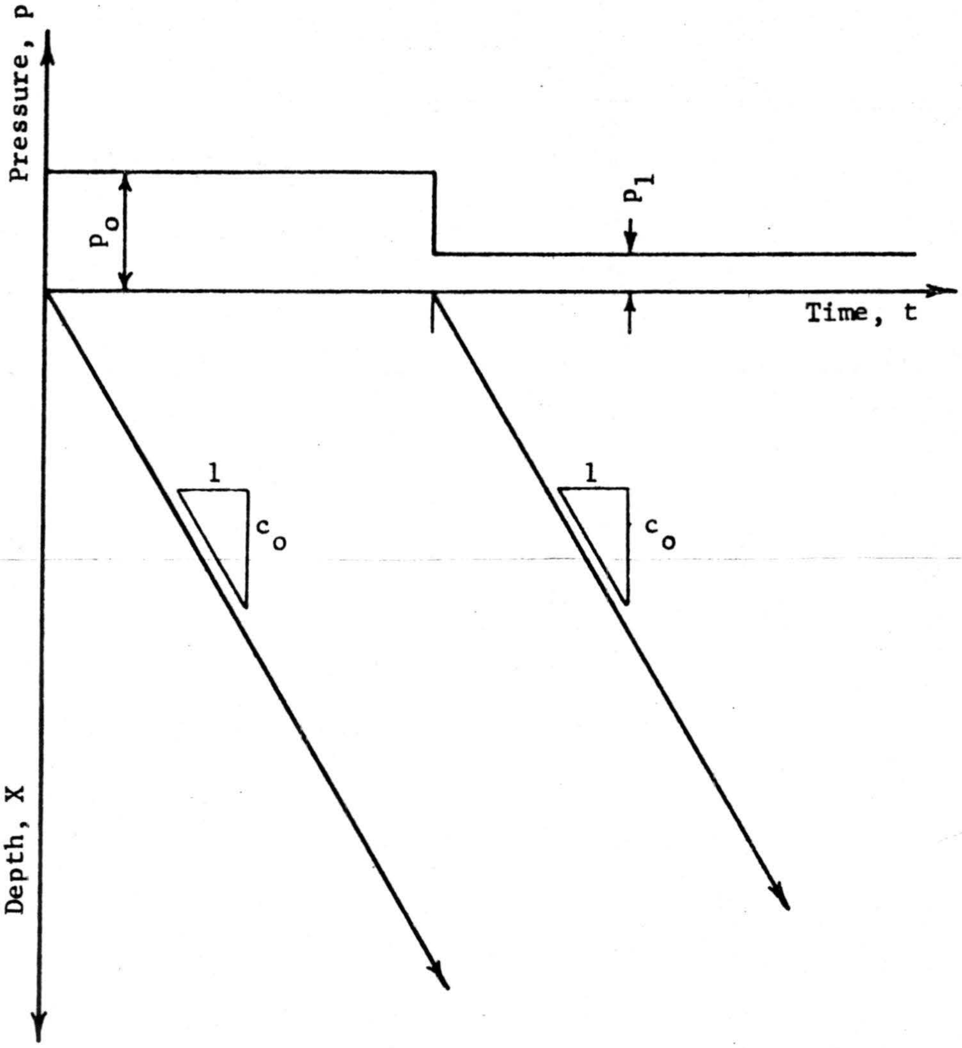

Figure 2. Depth, time, surface loading diagram for saturated soil.

will never overtake the loading wave to cause attenuation. In this instance, the intensity and duration of the pressure pulse at depth X will be the same as at the surface (X = 0).

The value of c_o is obtained from the equation

$$c_o = \sqrt{E/\rho} \tag{3}$$

where c_o = velocity of wave propagation, LT^{-1}

E = bulk modulus of the material, FL^{-2}

ρ = the mass density of the material, FT^2L^{-4}

For a sand having a void ratio, e_1, of 0.50 and a specific gravity, G, of 2.65, the value of c_o is computed in the following example:

$$e_1 = 0.50$$

$$G = 2.65$$

$$E = 300{,}000 \text{ psi} = 43{,}200{,}000 \text{ psf}$$

Unit weight of water, $\gamma_w = 62.5$ pcf

Saturated unit weight, $\gamma_{sat} = \left(\frac{G}{1 + e_1} + \frac{e_1}{1 + e_1}\right)\gamma_w = 131$ pcf

$$\rho = \frac{131}{32.2} \frac{\text{lb sec}^2}{\text{ft}^4}$$

$$c_o = \sqrt{\frac{43.2 \times 10^6}{\frac{131}{32.2}}} = 3260 \text{ fps}$$

It might be noted that the corresponding value for water alone is 4720 fps.

The differential equations given by Salvadori, et al (28) are valid for this system, thus:

$$\frac{\partial^2\sigma}{\partial X^2} = \frac{1}{c_o^2}\frac{\partial^2\sigma}{\partial t^2} \qquad (4)$$

and

$$\frac{\partial^2\dot{u}}{\partial X^2} = \frac{1}{c_o^2}\frac{\partial^2\dot{u}}{\partial t^2} \qquad (5)$$

where σ = stress
X = position
t = time
$\dot{u}$ = particle velocity

and the other symbols are as defined above. The solutions to these equations are

$$\sigma = f(X - c_ot) + g(X + c_ot)$$

$$\dot{u} = \frac{c_o}{E}\left[f(X - c_ot) + g(X + c_ot)\right]$$

If we assume an impulse loading at the surface of the form p_oe^{-t/T_o}, the boundary conditions are:

$$X = 0,\ t = 0 \quad , \quad \sigma = p_o$$

$$X = 0,\ t = t \quad , \quad \sigma = p_oe^{-t/T_o}$$

$$X = c_ot \quad , \quad \sigma = p_o$$

where T_o is a time constant with units of T. This yields the solution

$$\sigma = p_oe^{(X/c_o-t)1/T_o} \qquad (6)$$

and

$$\dot{u} = \frac{c_o}{E} p_o e^{(X/c_o - t)1/T_o} \qquad (7)$$

Then the acceleration,

$$\ddot{u} = -\frac{1}{T_o} \frac{c_o}{E} e^{(X/c_o - t)1/T_o} \qquad (8)$$

It is apparent that the particle velocity is dependent upon the magnitude of the maximum overpressure while the acceleration is dependent upon both the maximum magnitude and the rate of decay of the applied pulse.

These values are for the period following the application of pressure. The acceleration during the pressure jump would no doubt be more critical. This will be a function of the rise time of the pressure pulse.

Cohesive Soils

Most of what has been said for cohesionless soils might also be applied to cohesive soils. Under the intense pressures considered, it would be expected that any natural inter-particle forces would be overcome and that the soil would liquefy into a mud. However, it is interesting to note that Wilson and Sibley (44) quote results from a private report of Whitman showing constrained moduli approaching one million pounds per square inch for glacial till from North Dakota. It is possible that these materials can be treated as a very viscous fluid in somewhat the same manner as that used by Nielsen (25).

It should be pointed out that, if the grains of either the cohesionless or cohesive soils do remain in contact, there may be some attenuation of pressure due to friction. The procedures recommended here will be on the safe side and should be followed unless sufficient test data is developed to prove that attenuation does exist.

PARTIALLY SATURATED SOILS

Cohesionless Soils

The pore fluid pressure distribution in a partially saturated soil will be largely dependent upon the degree of saturation. It is well established in the literature on flow through porous media that the relative permeability of the non-wetting phase of the pore fluid (in this case air) falls off rapidly with increasing saturation. For example, Brooks and Corey (4) present charts showing this phenomenon for various unconsolidated porous materials. In all cases, the relative permeability of the non-wetting phase drops quite rapidly after some critical degree of saturation is attained. The relative permeability of the wetting phase of the pore fluid (in this case water) increases from zero at a degree of saturation which varies with the material but which is somewhere in the vicinity of 50 percent for the sandstones discussed by Brooks and Corey to a value of 1.0 at a degree of saturation of 100 percent.

In view of the foregoing, it would appear that the flow of air in partially saturated soils would very likely be impeded or prevented by water in the void spaces between grains. This could be expected to reduce the distance to which the pore air pressures due to blast would penetrate. On the other hand, the compression of the soil skeleton due to the blast wave may be sufficient to increase the degree of saturation to a point where the wave will propagate more in the nature of a saturated soil. The soil could not become fully saturated unless the pressure pulse duration was sufficient to allow the pore air to flow out of the soil or unless the pressure intensity was sufficient to drive the air into solution. For normal soils, it would seem probable that soils at 50 percent or lower saturation would develop less pore fluid pressure than a dry soil.

Cohesive Soils

The permeability of partially saturated cohesive soils is so low that one would expect the pressure to follow one of two patterns. Either the degree of saturation would be low enough that the pressure increase would be that due to the compression of the air in the voids or the amount of pore air would be sufficiently small that it would largely go into solution and the soil behavior would approach that of a saturated soil.

DISCUSSION OF RESULTS

The recommendations for prediction of pore air pressures shown in Figure 1 indicate that the pressures might be expected to penetrate the soil to a distance of thirty-five to forty times the cube root of the representative particle diameter, d_{10}. These values are predicated upon the assumption that the material will be in a relatively dense state. This would mean that a very coarse gravel with a representative particle size of 5 inches could be expected to show pore air pressure penetrations of approximately 30 feet while a silty sand, such as that tested by

Hampton, might be expected to show penetration to a maximum of approximately 1 foot. The pea gravel tested by Hampton would be expected to show overpressure penetration to approximately 10 feet. There are no test results available for the large diameter particles, but Hampton's tests show that some overpressure did penetrate the pores of the pea gravel beyond 6.13 feet which was his last gage station. For the highest incident overpressures used (136 psi), these values were still between 5 and 10 psi. For the silty sand, the data are very scattered and show very small pressures penetrating as far as 0.63 feet.

One thing that is quite clear is that excessive pore air pressures will not penetrate dry granular soils of relatively low permeability to depths beyond 2 to 5 feet. For construction in which large diameter gravel backfill is necessary or desirable, the area could be effectively sealed by grading to a top layer of a few feet thickness of relatively fine material such as a sandy gravel. Since a very fine soil such as fine silt might retain large amounts of moisture and thus be near saturation, it might be undesirable as a backfill material but might be quite effective as an isolation layer to prevent pore air pressure penetration.

The relationship between the attenuation of pressure in the pores of the soil and ratio of $X/d_{10}^{1/3}$ is consistent with Hampton's conclusion that attenuation with distance is essentially independent of the magnitude of the overpressure. This would be true for any one soil at a given density.

It is also obvious that the stress transmission in saturated soils is apt to be very much more serious than in dry soils. This would indicate that, wherever possible, buried construction should be in a well-drained site and backfilled with a free-draining material in order to obtain as much natural attenuation from the soil as possible. Where this is not possible, provision must be made for the higher stresses and the water leakage problems that will be encountered as well as the less favorable soil strength conditions.

It is also quite probable that a coarse, well-drained backfill material should be employed in the vicinity of buried structures even in dry areas. This would help prevent the soil becoming temporarily saturated due to the percolation of precipitation. Unless so protected, buried structures would be especially vulnerable to attack during or after a storm. Such a situation would be particularly undesirable if weather modification techniques are perfected to the state where a potential enemy could coordinate widespread precipitation with an attack.

For partially saturated soils underlain by saturated soils, the shock wave will be transmitted, with normal attenuation, through the partially saturated soil to the zone of near saturation. At this stage the pressure wave will be transmitted to the pore water and will propagate in the same manner as in a saturated soil except that the incident overpressure on the saturated zone will be the attenuated pressure arriving through the partially saturated zone.

SUMMARY AND CONCLUSIONS

The study outlined in this report has considered the propagation of pressure waves due to blast loadings through the pore fluid of a soil. The cases discussed are for dry, saturated and partially saturated conditions and cohesionless and cohesive soils.

It is recommended that pressure attenuation in the pore air of dry cohesionless soils be predicted using the equation

$$\frac{p}{p_o} = e^{-0.154(X/d_{10}^{1/3})}$$

where p = maximum pore air pressure, F/L^2

p_o = maximum incident overpressure, F/L^2

X = distance from surface to which p_o is applied, L

d_{10} = the diameter of the 10 percent size of the soil or some other representative particle diameter (see text).

For the saturated case, the stress wave will propagate rapidly with essentially zero attenuation.

Partially saturated soils will have behavior which is strongly influenced by the degree of saturation. At lower ranges of saturation they can be expected to have a relative air permeability which is less than the dry soil air permeability, and thus they should exhibit more rapid pore air pressure attenuation. At the higher ranges of saturation, the soil will begin to behave very nearly as a completely saturated soil.

RECOMMENDATIONS FOR FUTURE WORK

Introduction

The prediction of pore fluid pressure effects due to blast on the basis of the very limited data is a precarious procedure. The fact that the test results reported to date are on a very limited range of sizes and on largely uniform sizes makes it difficult to extrapolate the results to other conditions without some qualms. It is, therefore, most desirable that a program of testing and evaluation be carried out to confirm or modify and extend the results of this study.

Particular attention should be paid to the correlation of results with the cube root of the diameter of the 10 percent size of the material. It is obvious that wide variations of grain-size distribution may occur and that situations may arise where the 10 percent size may not be the representative particle size for the material. It is quite probable that an additional correlation factor may be needed for a complete solution. In particular, the uniformity coefficient d_{60}/d_{10} or some similar quantity may be useful in evaluating the results of further tests.

Careful measurements should be made of the physical permeability of soils to be used in tests. These should be measured with respect to the flow of air through the soil rather than for the flow of water. One possible procedure for this is outlined in the work by Brooks and Corey (4). As noted earlier, the permeability of the soils used by Hampton varied roughly as the diameter of the 10 percent size. If this relationship were to alter drastically, it might be necessary to include permeability in the correlation, either directly or by relating it to representative particle size.

The testing program should be extensive enough to be statistically significant and should cover the conditions of moisture discussed in the report, i.e., dry, saturated, and partially saturated. All tests should cover a range of sizes. A minimum program would consist of at least three separate and distinct sizes of material which would be tested as uniform grain size materials and also blended to give different gradations of grain size.

Some of the significant variables which should be investigated in the test program are as follows:

1. The influence of grain size.
2. The influence of grain-size distribution.
3. The relationship to permeability.
4. The influence of moisture content.
5. The influence of the range of overpressure.
6. The influence of axial compressibility.
7. The influence of relative density.

It is apparent that even a minimum program will be quite extensive and difficult. For example, it will be very difficult to allow axial compression while maintaining zero or near zero lateral strain. Perhaps a device similar to the one reported by Stoll and Ebeido (36) might be useful here.

Dry Soil

The tests on dry soil may be run in a shock tube and can no doubt be run in the horizontal position. It would appear that instrumentation to measure incident pressure might be more reliable in a head-on configuration if the duration of blast pulse can be made sufficiently long. It is highly desirable to get to higher overpressures such as 300 psi or even 500 psi and durations of 1/2 to 1 second if possible. The effect of longitudinal compression in raising pore pressure should be evaluated. This might, in this case, be achieved in the same test by sealing the end of the sample with plastic so that no air can enter the pores and then running a series of tests.

Saturated Soil

It would appear that saturated soil tests could best be run in the vertical configuration. The same range of pressures and durations used in the dry soil tests would be appropriate here. As an incidental but highly interesting problem, it would be helpful to measure the movement of a soil inclusion under these conditions. Some small amount of axial deformation would have to take place, but the fluidity of the mixture might make rigid boundaries feasible.

Partially Saturated Soil

The pressure and duration relationships used in the other tests should also be applied to the partially saturated soils. In order to obtain any meaningful data, it would appear to be essential to allow axial deformation to occur.

General

All tests should be instrumented with a view to obtaining enough information to allow the observation or determination of:

1. The distance to which pore pressure overpressures penetrate the mass.

2. The amount by which pore pressures increase during the blast wave.

3. The velocity at which the pore pressure wave propagates.

4. The total stress in the soil.

5. The effective stress in the soil. (Note that this means that it will be necessary to know the pore fluid pressure and the total stress at the same time even if the maxima propagate at different velocities. The most desirable way to do this would be to record a pressure-time history for both quantities.)

Hampton's report (14) contains some useful data on instrumentation of the pore pressure portion of the test. There are also a number of suggestions in the papers of Session Three of the Proceedings of the Symposium on Soil-Structure Interaction published by the University of Arizona, Tucson, Arizona, September 1964 (NCEL Library No. 620.19 S989). Some of the techniques developed in NCEL soil mechanics research will also be of considerable value in these measurements.

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NOMENCLATURE

		Units
c_o	Velocity of wave propagation $= \sqrt{E/\rho}$	LT^{-1}
d_{10}	The maximum particle diameter of the smallest 10 percent, by weight, of the soil	L
e	Base of natural logarithms	None
e_1	Void ratio	None
E	Bulk modulus	FL^{-2}
f, g	Symbols representing "function of"	None
p	Maximum pore air pressure at position X	FL^{-2}
p_o	Maximum incident pressure at surface	FL^{-2}
p_1	Reduced incident pressure at surface	FL^{-2}
t	Time	T
t_1	Time at which p_o reduces to p_1	T
T_o	Time constant	T
$\dot{u}$	Particle velocity	LT^{-1}
$\ddot{u}$	Particle acceleration	LT^{-2}
X	Distance from surface upon which incident pressure acts	L
ρ	Density	FT^2L^{-4}
σ	Stress	FL^{-2}

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14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Soil Mechanics						
Porous materials						
Propagation						
Impulsive loadings						
Predictions						
Pressure						

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13. ABSTRACT

A study has been made of the propagation of pressure waves through the pore fluid of a soil. Dry, saturated, and partially saturated cases are considered for cohesive and cohesionless soils. Tentative recommendations are made concerning assumptions to be made and methods to be used in predicting the pore pressures resulting from impulsive loadings.

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