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

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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 A method based on laboratory cyclic triaxial and torsional tests of undisturbed soil samples has been developed to predict the potential for liquefaction due to buried charges, such as those used in construction blasting. The results of a test blasting program conducted at a construction site are presented. The case history yielded data on particle velocities and blast induced porewater pressure changes.

INTRODUCTION

Information on liquefaction potential and corresponding soil response due to blasting has generally been restricted by the military to single, large, near-surface blasts; while the potential for liquefaction due to small, buried charges, detonated in rapid succession, such as those used in construction, has not received much attention. This paper presents a method to predict excess pore water pressures, particle velocities and soil strains induced by a compression pulse propagating from buried charges of various sizes. This analysis was performed for a construction site in southern Europe and utilized the results of dynamic triaxial and cyclic torsional tests performed on undisturbed soil samples.

The method uses empirical equations to predict the particle velocities, from which soil strains were derived. These soil strains, together with the results of the cyclic testing program, were then used to estimate pore water pressures generated by the multiple detonations.

In addition, a test blasting program was designed and carried out at the construction site. During the blasting program, up to six cycles of blasting were applied using charges varying from 0.25 kilograms to 44 kilograms at a depth of about 30 meters. Instrumentation at the site included piezometers, settlement gauges and velocity transducers.

SITE CONDITIONS

In order to dewater the construction site, a deep well system and a seepage cutoff wall were being installed. As shown in Table I, the site, whose natural groundwater table was within a few meters of the surface, consisted of 30 meters of sandy soils overlying a deep deposit of low permeability silty clay. The diaphragm wall was designed to surround the site, penetrate the high permeability sandy soils, and extend five meters into the silty clay. During construction of the diaphragm wall, progress was hindered due to the presence of a two to three meter thick layer of hard sandstone. Therefore prior to further excavation of the diaphragm trench, it was proposed that boreholes be drilled through the sandstone and explosives be detonated in an attempt to fracture the hard layer and allow faster progress.

TABLE I: Soil Layering and Properties

Depth (Meters)	Soil Type	Wave Velocity (m/sec) Shear	Compression
0-13	Silty Sand	400	1,700
13-23	Silty Sand	580	1,800
23-28	Sand	490	1,750
28-31	Sandstone	840	3,000
31-60	Silty Clay	350	1,800

Since blasting in the sandstone could lead to liquefaction in the overlying saturated sands, ground settlement, damage to already existing portions of the cutoff wall, and damage to site instrumentation (extensometers and electro-pneumatic piezometers), the potential hazards were analyzed and field tests were conducted before production blasting was allowed.

METHODOLOGY OF PREDICTING PORE PRESSURES

The methodology used to predict pore pressures involved the correlation of peak particle velocity with distance from the blast and the amount of explosive (charge weight). As no field measurements of particle velocity due to blasting had been made yet, the relationship presented by Duvall, et. al (1967) was used:

$$V_{part} = 121.1 \left(\frac{dist}{W^{0.5}} \right)^{-1.67} \qquad (1)$$

where:

V_{part} = the peak particle velocity (inches per second),
dist = the distance from charge (feet),
W = the charge weight (pounds)

To determine soil strains, the blast was considered to be a radial compression wave emanating from the shot point. The relationship presented by Stagg and Zienkiewicz (1968) was used to compute the longitudinal strains:

$$\varepsilon = V_{part}/V_p \tag{2}$$

where:

ε = the longitudinal strain,
V_{part} = the peak particle velocity from Equation 1, and
V_p = the compression wave velocity.

The values of compression wave velocity, measured in a cross-hole investigation at the site are presented in Table I. A velocity of 1,800 meters per second was considered to be the most appropriate value.

Thus, by using the empirical relationship to define peak particle velocities, and the measured value of compression wave velocity, the longitudinal strain could be determined as a function of the distance from the charge and the charge weight. In order to simplify the relationship between calculated values of longitudinal strain in the field, and measured values of axial strain in the laboratory tests, the two strains were assumed to be equivalent.

Since a cyclic triaxial and cyclic torsional testing program had been conducted on the sands above the sandstone layer, results relating the ratio of shear to normal stress (τ/σ) to number of cycles to liquefaction were available. In addition, since stress versus strain relationships were recorded for each cycle of loading, values of stress ratio (τ/σ) could be related to values of axial strain (ε) on the first quarter cycle of loading (Figure 1). Also, the pore pressure increase as a function of stress ratio, and number of cycles (Figure 2) could be determined from the results of the testing program. Thus, given a value of axial strain in the field, the value of stress level could be determined, and given a number of cycles, the excess pore pressures generated could be evaluated.

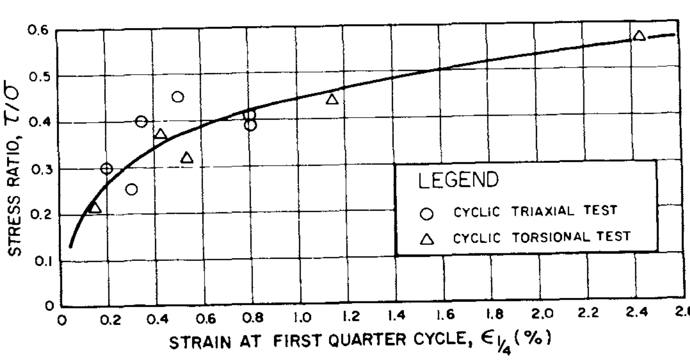

Fig. 1: Stress Ratio versus Strain at First Quarter Cycle of Loading

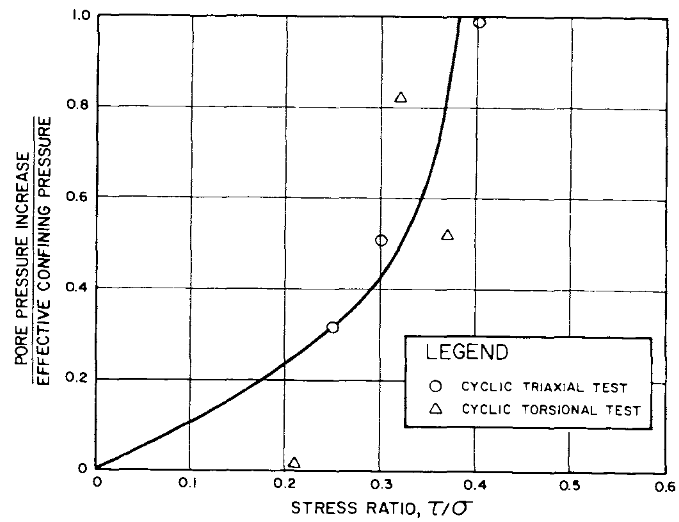

Fig. 2: Normalized Pore Pressure Increase for Six Cycles of Loading

FIELD TESTING PROGRAM

A field testing program was conducted to verify the predictions of excess pore pressure, and peak particle velocities and to provide a basis for design of the production blasting program for the site. Instrumentation used to monitor the effects of blasting consisted of geophones, piezometers, and some distant magnetic extensometers.

Because the borehole layout was for the purpose of constructing a diaphragm wall, all boreholes were in a linear arrangement (Figure 3). The explosives were distributed throughout the thickness of the sandstone layer, usually in three discrete charges. Detonation delay times between the blasts within one borehole were less than 0.06 seconds, therefore, the total charge weight within one borehole was considered as one blast. When multiple shot point borings were detonated, a delay of 0.1 seconds was used. Under these conditions, the detonations were considered analytically as separate blasts.

A total of eight detonations were conducted as summarized in Table II.

TABLE II: Test Blasting Sequence

Test Number	Shot Point Borings Used	Weight of Charge per Boring (Kilograms)	Delay between Blasts (seconds)
1	S1	0.25	-
2	S2	0.5	-
3	S3	2.0	-
4	S4	3.0	-
5	S10	3.0	-
6	S5, S6, S7 S8, S9, S11	2.0	0.1
7	A, B, C, D	6.4	0.1
8	E, F, G, H, I	9.0	0.1

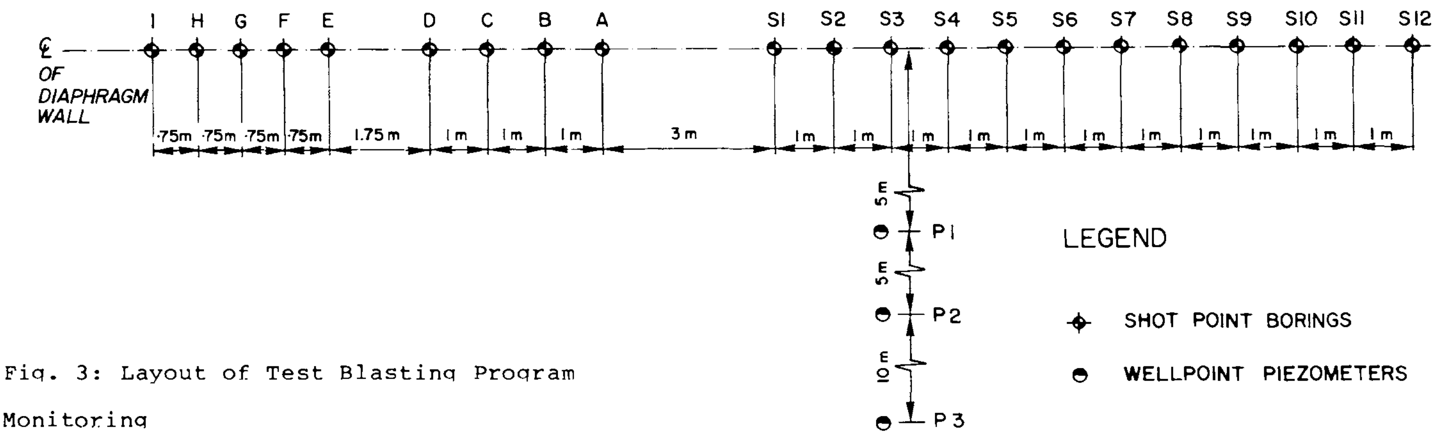

Fig. 3: Layout of Test Blasting Program

Monitoring

The instrumentation for the measurement of particle velocities consisted of six geophones, five of which were placed horizontally in the listening borings while the sixth was placed vertically. As is shown in Figure 4, the geophones were positioned to monitor particle velocities in the upper sand, the sandstone, and the underlying silty clay. The two sets of three geophones each were placed at variable distances from the shot point borings depending on the charge weight and the expected particle velocities.

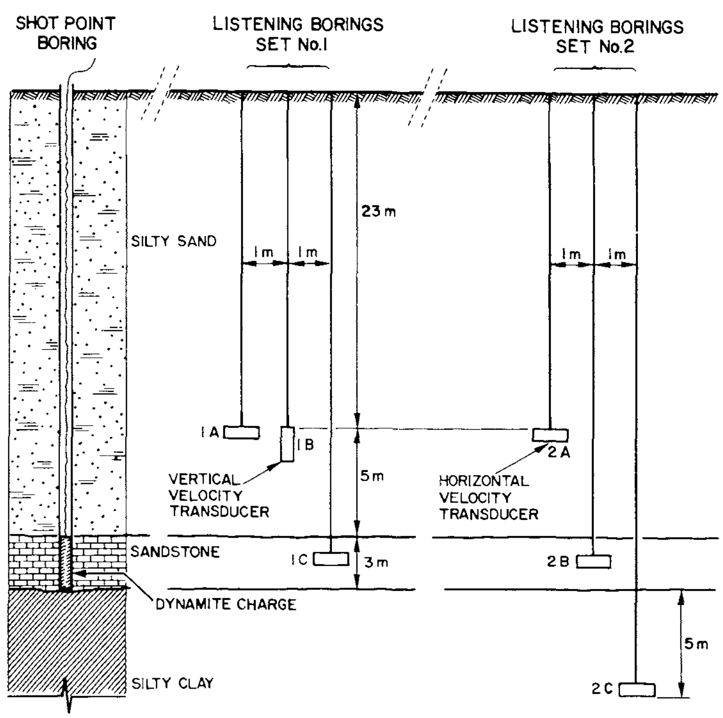

Fig. 4: Locations and Types of Geophones

Measured peak particle velocities in the upper sand layer are presented in Figure 5 as a function of charge weight and distance. The assumed relationship of Equation 1 did not fit the measured data exactly. A least squares fit line through the data is presented in the following equation and in Figure 5:

$$V_{part} = 69.2 \left| \frac{dist}{W^{0.5}} \right|^{-1.35} \qquad (3)$$

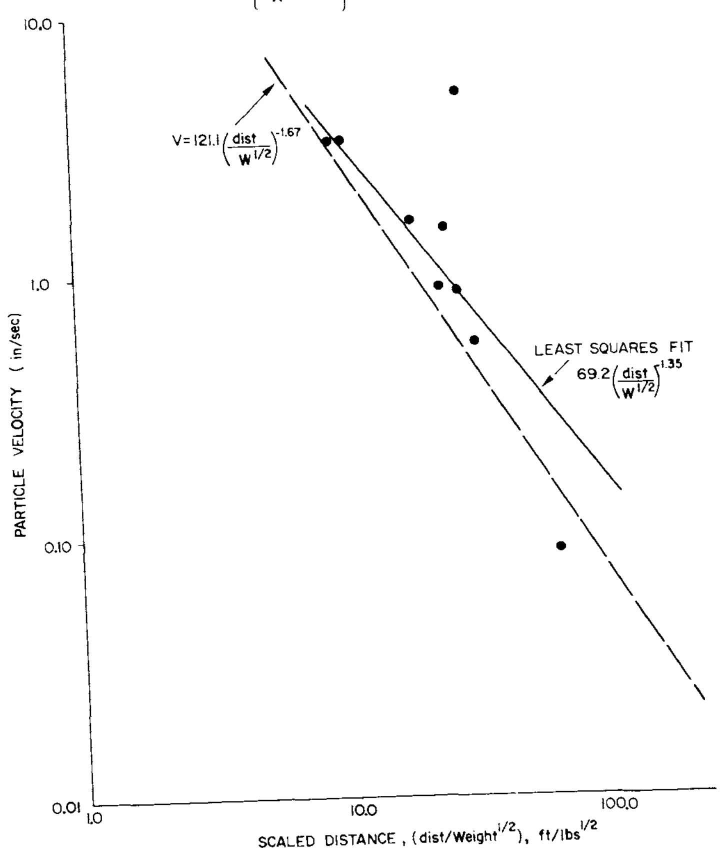

Fig. 5: Measured Peak Particle Velocities

In order to verify the predictions of pore pressures and to determine directly in the field the potential for liquefaction due to blasting, the increases in water table elevations were measured with three wellpoint piezometers. These piezometers were installed in a line perpendicular to the axis of the diaphragm at a depth of about 25 meters as is shown in Figure 3.

After blasting, the water level in the piezometers would rise and begin to fall too quickly for measurements of the peak water table to be performed. Therefore, before each blast, the electrical sounding device was secured at a specific level above the static groundwater table. Thus, when the blast was detonated, knowledge of the groundwater table was restricted to knowing if the groundwater table rose above or below the sounding device.

For some of the larger blasts, higher rises in the water table were anticipated. Therefore, the wellpoint piezometer tubes were extended above the ground surface with transparent tubing, and green dye was poured down the tube. This allowed the maximum rise in groundwater level to be recorded. These values are presented in Table III.

Table III. Peak Water Levels

Test Number	Piezometer Number	Measured Rise in Water Table (Meters)
3	1	> 0.32
	2	> 0.15
	3	> 0.13
4	1	> 0.76
	3	> 0.25
6	1	4.82
	2	> 2.00
	3	< 1.50
7	1	5.12
	2	0.85
	3	0.32
8	1	4.90
	2	< 2.94
	3	< 1.30

Note: The symbol ">" means the rise in water table was greater than this value.

The symbol "<" means the rise in water table was less than this value.

Groundwater level readings were taken at various time intervals after the blast, thus the time required for excess pore pressures to dissipate was recorded. An example of the dissipation rate is shown in Figure 6.

A magnetic extensometer which had previously been installed about 50 meters from the test blasting area was not affected by the blasting and did not indicate any settlment after blasting and dissipation of the excess pore pressures.

Comparison of Analysis with Measurements

Both the peak particle velocities and the maximum water level increases measured in the field were considerably higher than the predicted values. Similar results were found with a finite element model (Charlie, et al., 1981). However, even for the largest blasts, the field

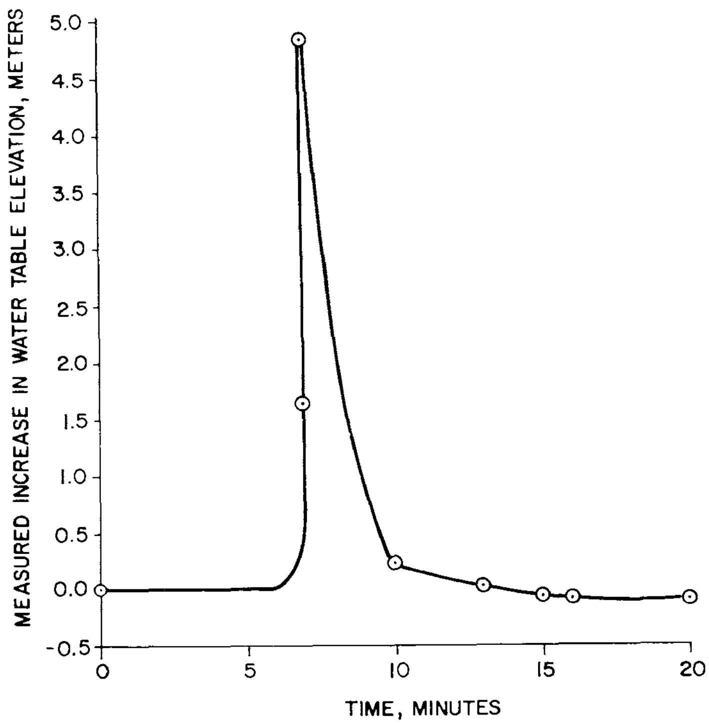

Fig. 6: Typical Variation in Water Table Test Blast

measurements showed that liquefaction was not a serious concern and the date provided a basis for design of the production blasting program.

SUMMARY

A simplified method of predicting the potential for liquefaction due to blasting has been presented. This method employs the results of cyclic soil testing and relationships between longitudinal strain and blast charge and distance. The results of a test blasting program are presented in the form of measured particle velocities and excess pore pressures. This test blasting program was used to define the production blasting performed at the site.

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