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REVIEW OF PRESENT PRACTICES USED IN PREDICTING THE EFFECTS OF BLASTING ON PORE PRESSURE

November 1985
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16. ABSTRACT

The potential hazards to earthfill dams from construction blast-induced pore-water pressure increases are examined. Numerous sources of information on the subject are cited, and recommendations for allowable peak particle velocity and compressive and shear strains are made.

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INTRODUCTION

It has long been recognized that earthquake-induced ground motions in loose, saturated silts and sands can lead to increases in pore-water pressures that may take hours or even days to dissipate. Increased pore-water pressures cause the shear strength of a soil to decrease. Less recognized is the fact that identical effects also have been triggered by explosion-induced ground motions. Unfortunately, little information is available on the performance and safety of earth slopes, earthfill dams, and other earth structures subjected to vibrations generated by construction blasting. Nevertheless, for the USBR (Bureau of Reclamation), the effects of blast-induced vibrations are important when blasting is performed:

- In or near an abutment of a dam
- In or near a dam
- To deliberately destroy a dam
- To compact a dam foundation
- To remove a cofferdam
- Near a canal, levee, dike, or saturated slope

The major concern of this report is the potential for blast-induced residual pore-water pressure increases that reduce the shear strength of a soil long enough to allow gravity to cause the failure of the slopes of earth structures. The objectives of this report are to develop recommendations for allowable peak particle velocity and for compressive and shear strains to limit or prevent residual pore-water pressure increases in earth structures.

BLASTING NEAR EARTHFILL DAMS

A number of studies have attempted to correlate ground-motion levels with observed damage to structures. It is generally agreed that the amount of blast damage correlates best to the peak particle velocity (how fast the ground moves). The blasting criteria for residential structures recommended by the U.S. Bureau of Mines is a peak particle velocity of 5 cm/s; although some structures may be damaged at lower velocities. An estimated safe peak particle velocity for tunnels and concrete structures is about 25 cm/s or less. The safe peak particle velocity for saturated earth slopes, earthfill dams, and other saturated earth structures is unknown.

Obermeyer [28]* measured no significant increase in residual pore-water pressures in a hydraulic fill uranium tailings dam subjected to blast-induced vibrations having particle velocities as high as 2 cm/s. However, significant blast-induced residual pore-water pressure increases have been reported in saturated, loose cohesionless deposits by Florin and Ivanov [14], Kummeneje and Eide [20], Ivanov [15], Langley et al. [21], Perry [31], Banister and Ellett [3], Yamamura and Koga [46], Rischbieter [35], Arya et al. [2], Kok [19], Marti [26], Charlie et al. [7], Studer and Kok [44], Long et al. [22], Prakash [32], and other researchers.

Terzaghi [48] reported that the upstream slope of SWIR III (SVIR III), an earthfill dam in Russia, failed in 1935, shortly after explosives were used to remove a cofferdam located approximately 200 m upstream. The failure originated in loose, saturated sand and rapidly spread along the upstream portion of the dam. Terzaghi attributed the failure to blast-generated ground motions. This and other examples indicate that blasting in or near earthfill dams can significantly increase residual pore-water pressure (excess pore-water pressure), reduce the stability of a dam, induce consolidation settlements, or otherwise damage an earthfill dam. Because an explosion could result in damage disproportionately greater than the energy released, the potential for blast-induced failure warrants serious examination.

GROUND MOTIONS CAUSED BY EXPLOSIONS

Ground motions caused by explosives produce localized peak accelerations that can be several orders of magnitude greater than earthquake accelerations. When a single contained (no surface crater) charge is detonated, the rapid release of energy generates a compression pulse that radiates away from the explosion and produces tensile hoop strains accompanied by intense radial compressive strains in the surrounding soil.

The detonation pressure of commercial explosives occurs almost instantaneously. It ranges from about 5×10^{8} to 1.5×10^{11} Pa [5]. A shock wave propagates away from each charge detonation. It, typically, has one sharp peak of acceleration with a duration of a few milliseconds for rock and tens of milliseconds for soil [43]. Kirt [18] (1982) noted that the ground-motion frequency from blasting in most soils ranges from 6 to 9 Hz (Hertz), but in loose, saturated sands and silts and soft

* Numbers in brackets refer to entries in the bibliography.

clays can be as low as 2 Hz. Figures 1 and 2 show typical particle velocity and acceleration time records for detonations in rock and soil. For a deeply buried charge, most of the wave energy in the surrounding material is in the form of a compressional stress wave. After the stress wave reaches interfaces, such as soil-rock boundaries, the water table, or the ground surface, reflections can produce compression or tension, shear, and surface waves. A blast that detonates in a few milliseconds can thus produce oscillatory ground motions lasting several seconds at locations several hundred meters away. Millisecond delays between detonations result in additional stress waves. In addition, high-frequency energy is damped out faster than low-frequency energy; therefore, the energy at great distances is in a lower frequency range.

The primary factors influencing ground-shock amplitude and frequency of ground motion are:

Charge weight per delay. In general, ground-shock amplitude increases and frequency decreases with increases in the quantity of charge detonated

Charge-delay patterns. The use of millisecond delays results in multiple ground strains

Depth of burial. A fully contained (no crater) explosion creates significantly greater ground vibrations than surface or near-surface explosions

Soil and rock properties. In general, peak particle velocity and frequency of ground motions are higher in rock than in soil

Local geology. Water tables and geologic interfaces between soil and rock influence ground motions and produce secondary oscillatory shear, tensile, and compression waves

Soil saturation. Explosions in saturated soils generally produce higher peak particle velocities than those in dry or partially saturated soils

Geometrical attenuation. At close ranges the energy from a concentrated charge radiates out from the source as a spherical stress wave; a line charge radiates as a cylindrical stress wave; and a long row of line charges radiates as a plane stress wave

ATTENUATION

A widely used scaling law that relates the quantity of the explosive charge and its distance to the peak particle velocity, V_{peak}, is

$$V_{peak} = C\left(\frac{R}{W^m}\right)^{-n} \tag{1}$$

where:

C = ground transmission constant (based on the confinement of charge and the local geology),
W = maximum explosive quantity per delay,
R = distance between the explosion and the recording site, and
m and n = empirical site constants.

Experience has shown that differences in vibration levels caused by different commercial explosives are usually small compared with the variations caused by other factors [12]. Because the constant m is approximately one-third to one-half, the scaling law is usually written in one of the following forms:

$$V_{peak} = C\left(\frac{R}{W^{1/2}}\right)^{-n} \text{ (square-root scaling)} \tag{2}$$

$$V_{peak} = C\left(\frac{R}{W^{1/3}}\right)^{-n} \text{ (cube-root scaling)} \tag{3}$$

The square-root scaling typically matches results from row charges, line charges, and near-surface charges generating surface waves. The cube-root scaling typically matches results from deeply buried point charges [40]. For construction blasting, Oriard [30] prefers square-root scaling, while Hendron [16] prefers cube-root scaling. For blasting in rock, DuPont [12] gives the following equations for a preliminary estimate (generally within a factor of 2) of the peak particle velocity:

$$V_{peak} = 1.1\left(\frac{R}{W^{1/2}}\right)^{-1.6} \text{ (m/s normal confinement)} \tag{4}$$

$$V_{peak} = 5.7\left(\frac{R}{W^{1/2}}\right)^{-1.6} \text{ (m/s heavy confinement)} \tag{5}$$

where:

V_{peak} = peak particle velocity (m/s),
R = distance between explosion and recording site (m), and
W = explosive quantity (kg per delay period).

Table 1 presents quantity-distance relationships for specific values of peak particle velocity as estimated by equation 4.

For buried detonations in a deep saturated-soil deposit, Drake and Ingram [13] presented measured peak particle velocities as a distance function of the following equations:

$$V_{peak} = C\left(\frac{R}{W^{1/2}}\right)^{-1.15} \quad \text{(m/s row charges)} \tag{6}$$

$$V_{peak} = C\left(\frac{R}{W^{1/3}}\right)^{-2.3} \quad \text{(m/s point charges)} \tag{7}$$

where:

C = 1.1 to 3.6

For blasting in saturated sands, Long et al. [22] obtained the following relationship from single buried dynamite charges:

$$V_{peak} = 0.6\left(\frac{R}{W^{1/2}}\right)^{-1.35} \quad \text{(m/s)} \tag{8}$$

Because of the variability in predicting peak particle velocity shown by equations 2 through 8, actual propagation relationships are generally established by field test blasts.

Nearly all experimental data on explosive-induced liquefaction consist of stress and pore-pressure measurments. Seldom have ground-motion measurements been taken in conjunction with pore-pressure measurements. Ivanov [15] states that the peak stresses generated by the detonation of explosives in water and in fully saturated cohesionless soils are essentially equal. Cole [10] and Lyakhov [23] give the following equation for peak stress from concentrated charges in water:

$$\sigma_{pw} = 5.23 \times 10^7 \left(\frac{R}{W^{1/3}}\right)^{-1.13} \text{ (Pa)} \tag{9}$$

where:

σ_{pw} is the peak compressive stress in water.

For saturated soil Lyakhov [23] states

$$\sigma_{pw} = 5.89 \times 10^7 \left(\frac{R}{W^{1/3}}\right)^{-1.05} \text{ (Pa)} \tag{10}$$

where:

σ_p is the peak compressive stress in a saturated soil.

The following approximate relationships can be used to relate the peak longitudinal strain and the peak compressive stress to the peak particle velocity [37 and 43]:

$$\epsilon_p = \frac{V_p}{V_c} \tag{11}$$

where:

ϵ_p = the peak compressive strain in the radial direction,
V_p = the peak radial particle velocity, and
V_c = the compressive wave velocity.

As an approximation, the peak radial particle velocity could be equated to the peak particle velocity, V_{peak}. For more details on determining the peak particle velocity, see Hendron [16] SME [42] and figure 1. The compression-wave velocity is approximately 1500 m/s for deaired water and saturated soil. Similarly, the peak shear strain can also be equated to the peak transverse particle velocity.

The peak compressive stress can be equated to the peak radial particle velocity as follows [37]:

$$\sigma_p = (\rho Vc)\ V_p \qquad (12)$$

where:

$\rho =$ mass density.

By use of equation 12, equations 9 and 10 can be related to peak particle velocity. Similarly, the peak shear stress can also be equated to the peak transverse particle velocity.

Equation 12 relates equation 1 to equation 11, which indicates that the peak compressive stress and peak longitudinal particle velocity scaling factors are related to each other by the mass density and by the compression-wave velocity of the media. Table 2 relates equations 11 and 12 to the peak radial particle velocity for loose saturated soils. As shown, a peak radial particle velocity of 15 cm/s corresponds to a peak compression strain of about 0.01 percent.

Although compressive strains predominate near the blast, shear strains may be important in generating pore-water pressure increases far from the charge. Where shear waves exist, similar equations can be used to obtain a preliminary estimate of the peak shear strain and stress.

PREDICTION METHODS FOR BLAST-INDUCED PORE-WATER PRESSURE INCREASES

The three stages of interest in studying pore-water pressure responses as a result of blasting are the transient response, directly associated with the passage of the stress wave; the residual response, after the passage of the stress wave; and the dissipation stage. For water-saturated soils subjected to blast-induced strains, a residual increase in pore-water pressure occurs when the fluid phase responds elastically while the soil skeleton responds plastically. When both phases respond elastically, no residual increase in pore-water pressure occurs. The residual increase in pore-water pressure is important for dam safety. Research has proceeded along two paths: the empirical methods, which rely on experimental data: and the mathematical methods, which attempt to describe observed phenomenon. The work perfomed in various nations are described in the following paragraphs.

U.S.S.R.

Several references (Puchkov [34], Florin and Ivanov [14], and Ivanov [16]) indicate that the U.S.S.R. has analytical, laboratory, and field tests for predicting blast-induced soil property changes of saturated sand deposits. Researchers have studied blast-induced liquefaction where a complete loss of shear strength occurs as a consequence of reduced effective stress from increased residual pore-water pressure. Based on experimental laboratory and field studies of shock waves, Lyakhov [23] found that liquefaction did not occur in water-saturated sands with dry densities greater than 1.60 g/cm^3, and Puchkov [34] found that liquefaction did not occur in any saturated soil subjected to peak particle velocities less than 7 cm/s.

Russian work also includes empirical formulas used to predict the most effective methods of consolidating loose, saturated sands by using various configurations of explosives. Liquefaction (residual pore-water pressure increase equal to the initial vertical effective stress) is produced most effectively when the charge is contained (no crater). For a charge detonated below the groundwater table, the depth of possible liquefaction is approximately 1.5 times the depth of the charge. The radius of liquefaction for a single contained charge, R_{max} can be calculated by:

$$R_{max} = KW^{1/3} \tag{13}$$

where:

R_{max} = the radial distance from the charge (m),
W = the quantity of the explosive (kg), and
K = an empirical factor given in table 3.

Equation 13 was developed for Ammonite 9 and 10. For TNT, the radius of liquefaction should be multiplied by 1.1, because TNT is a more powerful explosive than ammonite (Marti [26]).

Russian field experiments also show that the extent of liquefaction for a given charge weight can be increased by subjecting the soil to multiple groundshocks by the use of multiple charges with delayed detonations.

India

Arya et al. [2] measured residual pore-water pressure increases in a loose, saturated sand out to a radial distance of 15 m, under a measured peak ground acceleration exceeding 0.1 *g* from a single 10 kg charge detonated 16 m below the ground surface (equation 4 yields a peak particle velocity of 9.1 cm/s at 15 m). Prakash [32] reported residual pore-water pressure increases out to a radial distance of approximately 20 m from a single 2-kg charge exploded at a depth of 6 m in a saturated, loose sand (equation 4 yields a peak particle velocity exceeding 1.6 cm/s at 20 m). The decay in residual pore-water pressure with distance is shown on figure 3.

Japan

Yamamura and Koga [46] measured dynamic and residual pore-water pressure increases in a series of field explosive tests. For a 1-kg charge placed at the bottom of a 6-m borehole at a distance of 10 m, a residual pore-water pressure increase of 10 percent of the initial effective stress was measured under an acceleration of 2 *g* (equation 4 yields a peak particle velocity of 2.8 cm/s at 10 m).

Europe

The Norwegian Geotechnical Institute (Kummeneje and Eide [20]) seems to be the first group outside of the Soviet Union to record blast-induced, residual pore-water pressures. They reported measurements of residual pore-water pressures to a radial distance of 20 m, from a 1.2-kg contained explosion in saturated sands (equation 4 yields a peak particle velocity of 1 cm/s at 20 m). Rischbieter [35], Kok [19], and Studer and Kok [44] reported field and laboratory blast experiments in which pore-water pressures and stresses were measured. Figure 4 presents the results of several contained single TNT explosive field tests on saturated, loose sands. From the figure, the factor of safety against liquefaction, defined as the initial effective overburden stress divided by the residual pore-water pressure, can be estimated as a function of charge weight and distance from the charge.

North America

Studies by Lyman [24] and Prugh [33] showed that blasting is an effective way to compact saturated, loose sands. The U.S. Air Force and the Army WES (Waterways Experiment Station) have conducted limited laboratory and field tests on blast-induced liquefaction (Perry [31] and Langley et al. [21]). Today these organizations are developing effective stress models to more accurately predict the behavior of saturated soils when a shock wave passes through them.

Detailed field and laboratory shock tests are being planned. Obermeyer [28] measured no significant increase in pore-water pressures in a hydraulic fill uranium tailings dam subjected to particle velocities as high as 2 cm/s. Several Bureau of Reclamation earthfill dams have been subjected to blast-induced vibrations generated by subsurface nuclear tests. These include Navajo Dam, New Mexico (maximum particle velocity of 0.5 cm/s (toe) and 1.3 cm/s (crest)); Vega Dam, Colorado (maximum acceleration of 0.1 *g* (toe) and 0.2 *g* (crest)); and Rifle Gap Dam, Colorado (maximum particle velocity of 2.5 cm/s). Although the behavior of pore-water pressures in these dams was not recorded during the blast vibrations, measurements taken several hours after the test showed little or no increase in pore-water pressures (Rouse and Roehm [38], Rouse et al. [39] and Alberg et al. [1].

Long et al. [22] reported that blast-induced residual pore-water pressures occurred where the peak particle velocity exceeded 5 cm/s. Charlie et al. [8] have developed an effective stress finite element model using Biot's [4] theory to predict vibration-induced pore-water pressure increases. Charlie's model overestimated the pore-water pressure increases measured by Long et al. [22]. Banister and Ellett [3] recorded both transient and residual increases in pore-water pressures in the saturated clayey silt of a riverbed. The area was subjected to peak particle velocities exceeding 11 cm/s from a nearby underground nuclear detonation.

Charlie et al. [9] have developed laboratory techniques to evaluate the threshold particle velocity and strain required to induce residual pore-water pressure increases under blast loadings. Current testing on saturated, loose sands indicates residual pore-water pressures occur when compressive strains exceed 0.01 percent. Kirt [18] and Munson [27] suggest that for earthfill dams, the peak particle velocity should be limited to 2.5 cm/s below 40 Hz and 5 cm/s above 40 Hz. Marcuson [25] suggests that liquefaction should not occur where the peak particle velocity is less than 2.5 cm/s and the peak acceleration is less than 0.1 *g*. Sanders [40] and Seed [41] related earthquake-induced liquefaction to peak particle velocity. They indicated that a threshold particle velocity of 5 to 10 cm/s also holds true for blasts.

Hendron [17] suggests limiting strains to acceptable levels based on field information, including standard penetration and shear-wave velocity tests and field-calibrated peak particle velocity scaling factors. Hendron also suggests monitoring pore-water pressures and particle velocities during blasting operations. For hydraulic-fill dams, Oriard [29] and Obermeyer [28] suggest limiting the particle velocity within a dam to less than 2.5 cm/s. The *Blasting Review Team Report* [6] suggests that peak particle velocities be monitored when blasting occurs near a dam and that the peak particle velocities be less than 5 cm/s in the dam unless a dynamic analysis or field test shows that the structure can withstand greater velocities.

ASSESSMENT OF PREDICTION METHODS

The state of the art for assessing blast-induced residual pore-water pressure increases and liquefaction potential is limited at best. Theoretical approaches are almost nonexistent and have not been verified by experimental testing. Empirical scaling factors have been derived from a limited number of field tests. A logical approach at this time would be to determine possible threshold particle velocity, strain, and soil densities where blast-induced pore-water pressure increases should not occur. The only values reported in the literature that are based on a series of experimental field explosive tests are a peak particle velocity of 7 cm/s reported by Puchkov [34] for liquefaction induced by a single detonation and the peak particle velocities in field explosive tests (as low as 1 cm/s for an increase in pore-water pressures) reported in the previous section.

Table 4 compares predictions based on particle velocity, strain, stress, and Russian scaling factors. A shear or compression strain of less than 0.01 percent is generally considered small enough to preclude generation of residual pore-water pressures because the strains are in the elastic range (Dobry et al. [11] and Charlie et al. [9]). As shown in table 2, a compression strain of 0.01 percent in saturated soil corresponds to about 15 cm/s peak longitudinal particle velocity. Although they are not shown, shear strains of 0.01 percent correspond to a peak transverse particle velocity of about 1 to 3 cm/s for cohesionless soils having shear-wave velocities of 100 to 300 m/s, typical of saturated soils. For blasting with a single charge of 100 kg, table 4 indicates that residual pore-water pressures could occur out to approximately 50 to 100 m. High-speed tunneling and construction blasting typically use several charges detonated with millisecond delays, called a round. Under ideal conditions, up to eight rounds can be detonated within 24 hours. Based on the Russian research with multiple charges, unless complete dissipation of the residual pore-water pressure occurs between millisecond delays or between each round, the predicted maximum radius of residual pore-water pressure increases may be greater than those in table 4.

RECOMMENDATION FOR BLASTING NEAR EARTHFILL DAMS

Extensive evidence in the literature indicates that excess pore-water pressures in saturated soils can be generated by nearby blasting operations. Although considerable work still must be done to develop and evaluate methods to investigate increases in blast-induced residual pore-water pressures, the following recommendations relating to dam safety can be made.

- Blasting is not recommended near operating dams constructed of or having foundations consisting of saturated loose sand or silts that are sensitive to vibrations. This generally includes all hydraulic fill dams. If blasting is required, peak particle velocity and pore-water pressure should be monitored and evaluated at several locations in the dam, foundation soils, and abutments. Peak particle velocities should be kept below 2.5 cm/s. The results of the tests should be evaluated before the next shot. Time between shots should be long enough to allow dissipation of blast-induced excess pore-water pressures.

- For operating dams having medium dense sands or silts, the peak particle velocity and strain should be below the threshold values for the material. The recommended threshold peak particle velocity without laboratory or field tests is 5 cm/s. The pore-water pressure should be monitored at several locations in the dam and foundations. When significant increases in pore-water pressures occur, dissipation of excess pore-water pressures should be allowed before further charges are detonated, and smaller charges or greater time between rounds should be used.

- For operating dams not having materials in the dam or foundations sensitive to vibrations, a peak particle velocity of 10 cm/s is reasonable without detailed laboratory and field studies. Pore-water pressures should be monitored throughout blasting operations.

- Extraordinary situations may require more conservative criteria.

SUMMARY

The detonation of explosive charges releases large quantities of energy that can produce rock and soil deformations far from the detonation point. The potential for strain and damage is related to the energy of the shock wave. Extensive data are available on blasting in general and on the performance of certain structures subjected to blast vibrations. However, only limited information is available on the performance of earthfill dams and other hydraulic structures subjected to blasting. The limited data indicate that, whatever the cause, one or several cycles of repeated strains in earthfill dams may cause residual pore-water pressure increases. Limiting the peak particle velocity to less than 2.5 cm/s for operating dams constructed of or having foundations consisting of saturated, loose sands or silts and to less than 5 cm/s for other earthfill dams, should keep strains below the threshold strains required for residual pore-water pressure increases to occur. Pore-water pressures should be monitored throughout a blasting operation, and they should be allowed to dissipate, if necessary, before subsequent charges are detonated.

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Table 1. – Quantity-distance relationships for specified values of peak particle velocities under average field conditions.

Explosive quantity per delay period, kg	Radial distance for peak particle velocity[1] meter				
	1 cm/s	2.5 cm/s	5 cm/s	10 cm/s	15 cm/s
1	19	11	7	4	3
5	42	24	15	10	8
10	60	34	22	14	11
25	95	53	36	22	17
50	134	75	49	32	25
100	190	107	69	45	35
250	300	169	109	71	55

[1] Based on equation 4. Distances shown may be higher under some geologic and confinement conditions.

Table 2. – Predicted peak compression strains and peak compressive stresses vs. peak radial particle velocities for loose saturated sands.

Peak radial particle velocity		Peak compression[1] strain, percent	Peak compression[2] stress	
cm/s	in/s		kPa	lb/in^2
1.5	0.6	0.001	40	6
2.5	1	.002	70	10
5	2	.0035	140	20
10	4	.007	280	40
15	6	.01	400	60
25	10	.02	700	100
150	60	.1	4000	600

Assumptions: Loose saturated sand with void ratio equal to 1.0 $V_c = 1500$ m/s.

[1] Equation 11.
[2] Equation 12.

Table 3. – Empirical factors to calculate liquefaction radius [26].

Type of soil	Relative density[1] percent	K
Fine sand	0 to 20	25 to 15
Fine sand	30 to 40	9 to 8
Fine sand	more than 40	less than 7
Medium sand	30 to 40	8 to 7
Medium sand	more than 40	less than 6

[1] Relative density, $D_r = (e_{max} - e) / (e_{max} - e_{min})$

where:

e = measured void ratio of the soil in situ,
e_{max} = void ratio for the loosest state, and
e_{min} = void ratio for the densest state.

Table 4. – Predicted maximum radius of liquefaction and residual pore-water pressures for a single charge detonated in loose, saturated, cohesionless soils.

	Predicted maximum radius, meter						
	Liquefaction[1]				Pore-water pressure increase[2]		
Explosive quantity, kg	U.S.S.R.[3] peak particle velosity >7 cm/s	U.S.S.R.[4] $R_{max} = kW^{1/3}$ D_R 0%	U.S.S.R.[4] $R_{max} = kW^{1/3}$ D_R 40%	F.S.[5] against liq. of 1	Compressive[6] strain >0.01%	F.S.[7] against liq. of 10	Peak[8] particle velocity > 2.5 cm/s
1	6	25	8	3	3	<15	11
5	13	43	14	3	8	17	24
10	18	54	17	5	11	23	34
25	28	73	23	7	17	32	53
50	40	92	29	10	25	42	75
100	56	116	37	12	35	52	107
250	89	157	50	20	55	75	169

Notes: Predicted maximum radius may be higher under multiple detonations and some geologic and confinement conditions.

[1] Maximum radius for the residual pore-water pressure increase equal to the initial effective vertical stress.
[2] Maximum radius for some increase in residual pore-water pressure.
[3] Equation 4 and [34].
[4] Equation 13 and table 3.
[5] Figure 4 for factor of safety against liquefaction equal to 1.0.
[6] Equations 4 and 11 assuming a compression wave velocity of 1500 m/s.
[7] Figure 4 for factor of safety against liquefaction equal to 10.0.
[8] Equation 4.

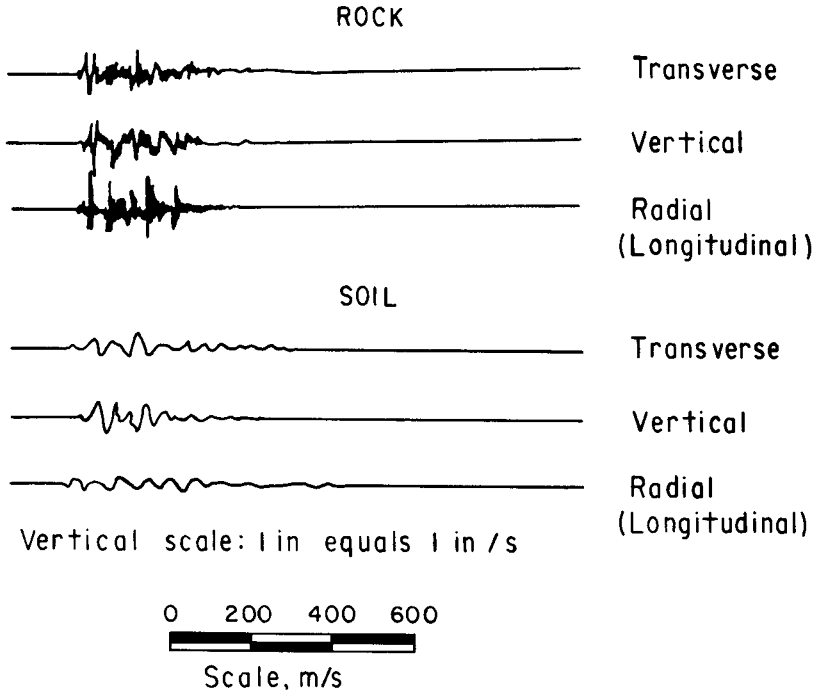

Figure 1. – Typical blast-induced particle velocities for rounds using millisecond delayed charges detonated in rock and soil [42].

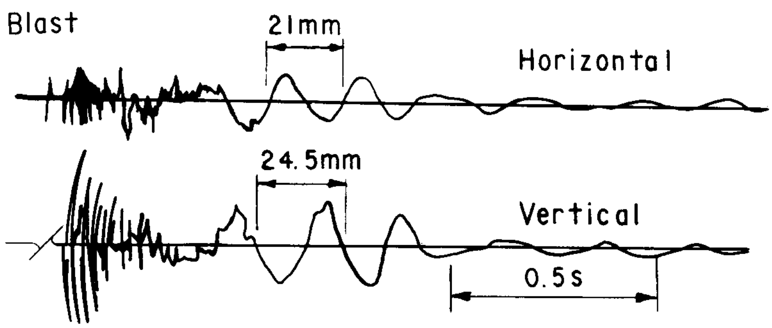

Figure 2. – Record of acceleration at a distance of 50 m from a 1-kg charge (60% gelatin) detonated in saturated sand at the Obra Dam site [32].

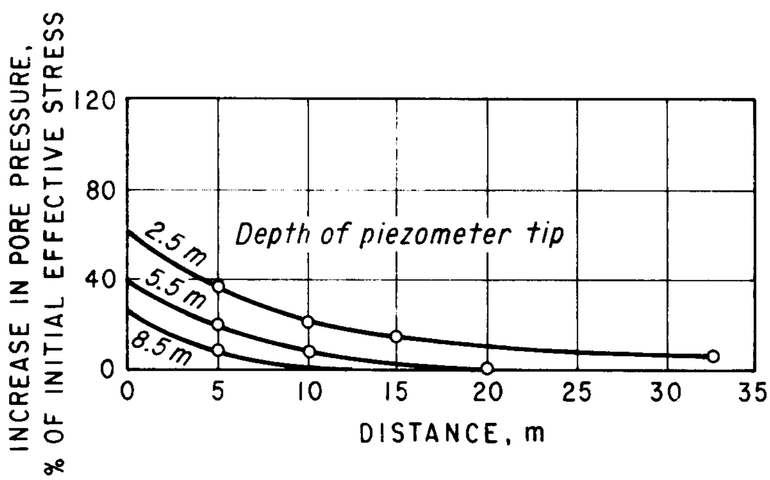

Figure 3. – Pore-water pressure vs. distance for a 2-kg charge (60% gelatin) at Obra Dam site [32].

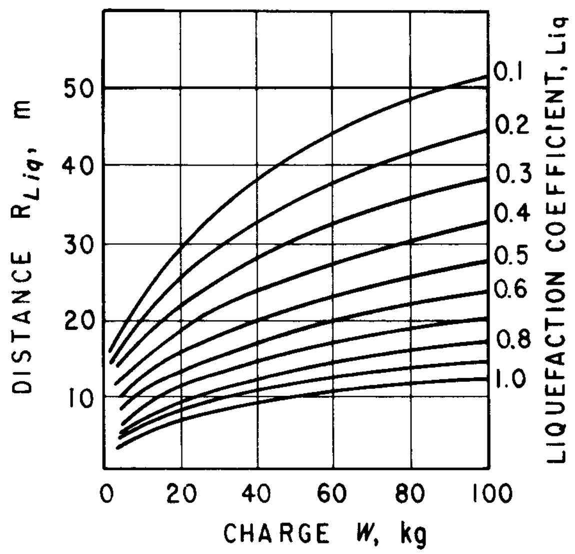

Figure 4. – Liquefaction coefficient for a single buried TNT explosive as a function of charge quantity and distance [44].

APPENDIX

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Mission of the Bureau of Reclamation

The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.