Induced liquefaction experiment in relatively dense, clay-rich sand deposits

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[1] In this paper we report results from a controlled blast-induced liquefaction experiment at the field scale. The physical and mechanical properties of the materials at the subsurface are characterized by a suite of in situ and laboratory tests, including the Standard Penetration Test (SPT); downhole and cross-hole seismic velocity tests; density, porosity, and gradation tests; and direct shear tests. Since the blast experiment was performed above groundwater table, the subsurface was saturated by a sequence of controlled infiltration tests. A 50-kg TNT charge was detonated at a depth of 10 m, and seismic ground motions were recorded in a vertical geophone array positioned at a horizontal distance of 30 m from the blast borehole. Obtained liquefaction features include a water fountain that erupted from the blast borehole, prolonged bubbling of the water surface inside the infiltration trench (a process equivalent to “sand boils” typically observed at sites which have experienced liquefaction), lateral spreading, and surface settlement. We argue that in contrast to conventional predictions, liquefaction may be induced in relatively dense silty and clayey sands (shear wave velocity >300 m s\(^{-1}\); relative density = 63–89%) relatively rich in clays (fines content >30%) and that the driving mechanism should not necessarily be restricted to cyclic shear stress loading.


1. Introduction

[2] Soil liquefaction, perhaps the single most important source of earthquake damage around the world, has been assumed traditionally to be generated by the vertical propagation of shear waves through a saturated soil column following strong earthquakes [Seed and Idriss, 1970]. Liquefaction may be defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore water pressure and reduced effective stress [Marcuson, 1978]. It is commonly assumed that cyclic shear stresses which develop within the soil column during strong earthquakes induce shear modulus degradation [Seed, 1979, 1980], leading ultimately to structural collapse of the soil skeleton and rapid pore pressure increase typically manifesting in “sand boils” on the surface and irreversible settlement [Seed et al., 1981]. It was found experimentally that in relatively soft sediments that contain a certain amount of fines, and where the pore space is completely filled with water, the collapse of the soil skeleton will induce a positive increase in pore pressure, which will in turn reduce the effective stress, following conventional effective stress and pore pressure concepts [e.g., Skempton, 1954; Terzaghi, 1943]. When the effective stress is reduced to zero the soil is said to liquefy.

[3] Because of the assumed shear-based failure mechanism, shear modulus degradation and void ratio changes during cyclic shearing in saturated soils of various relative densities and clay contents have been the subject of experimental research during the past three decades [Boulanger and Idriss, 2007; Evans et al., 1992; Martin et al., 1978; Mulilis et al., 1977; Vucetic and Dobry, 1991]. Numerical codes have also been developed to compute one-dimensional shear wave propagation from an input motion at the base of the analyzed column and upward through the strata [e.g., Schnabel et al., 1972]. Such codes can be modified to incorporate experimentally obtained shear modulus degradation and material damping curves, and are used extensively throughout the world for one dimensional site response analysis and for liquefaction prediction. While theoretical work is constantly being published [Cristescu, 2000; de Groot et al., 2006; Sawicki and Swidzinski, 2007] liquefaction prediction is still largely empirical, and is based on international documentation of liquefaction events on one hand and on the material properties of the sediments at site on the other [e.g., Bray et al., 2004; Cetin et al., 2004a, 2004b; Chu et al., 2004].

[4] The relative density of the material in situ (Dr) is considered to be of prime importance for liquefaction prediction and is best estimated in the field using correlations with the in situ standard penetration test (SPT). Since the pioneering work of H. B. Seed on the correlation between liquefaction potential and SPT values [Seed and
a substantial amount of data has been accumulated and synthesized from liquefaction events from all over the world [Cetin et al., 2004a; Yould et al., 2001]. These correlations suggest that with increasing relative density, as scaled by the corrected SPT blow count value (i.e., the number of hammer strikes to advance 0.3 m in the soil, for strict definition of terms, see for example Idriss and Boulanger [2006]), the liquefaction potential of the material is decreased; a corrected SPT value of \( N_{1,60} \) seems to be the upper bound for liquefaction.

[5] The shear wave velocity of the material \( (V_s) \) is another measure of the relative density but is much better defined mechanically. A comprehensive compilation of case histories performed by Andrus and Stokoe [2000] suggests that liquefaction potential decreases with increasing shear wave velocity. An overburden stress-corrected shear wave velocity of \( V_{s1} = 300 \text{ m s}^{-1} \) seems to be an upper bound for liquefaction; \( V_{s1} \) is given by \( V_{s1} = V_s(P_o/\sigma_o')^{0.25} \), where \( P_o \) is atmospheric pressure and \( \sigma_o' \) is effective stress (for comprehensive definition of terms see Andrus and Stokoe [2000]).

[6] Finally, the fines content (FC), defined as the percent of total weight of grains smaller than 0.075 mm, plays an important role because it controls the permeability of the material and consequently its potential to develop undrained conditions in the short term, necessary for the build up of pore water pressure as a result of shaking. From a recent compilation [Cetin et al., 2004a] it is evident that liquefaction potential increases with increasing fines content; fines content of 5% seems to be the lower bound for the development of undrained conditions during and after shaking. The upper bound of FC is poorly defined although most documented failures are found in sediments with FC between 5% and 35%. Silts and sands with fines content greater than 35% are therefore considered less prone to liquefaction although liquefaction events in cohesive sediments with up to 48% [Miura et al., 1995], 70% [Kishida, 1969] and even 90% [Tohno and Yasuda, 1981] fines have been reported from documented case studies in Japan. In Norway liquefaction in clays has been studied experimentally [Andersen et al., 1980; VanEekelen and Potts, 1978].

[7] While an assumed shear based failure mechanism is currently the norm in liquefaction prediction, there is nevertheless ample evidence for pressure-induced liquefaction in granular materials. Florin and Ivanov [1961] showed experimentally that sands may transform into a liquid state by subjecting the soil column to different sources of pressure waves. In a series of papers, Charlie et al. [1985, 1995, 1996] and Charlie and Doehring [2007] discussed liquefaction in granular deposits owing to explosions of buried charges. They show that explosive-induced ground motions can alter well water levels and induce liquefaction in water-saturated cohesionless geological profiles. They further show that the relationship between blast-induced pore pressure rise and scaled distance to the source is linear over 11 orders of magnitude of energy (Joules) or trinitrotoluene (TNT) equivalent mass (kilograms). Regarding earthquake-induced liquefaction owing to \( P \) waves, Lin [1997] demonstrated convincingly that during the magnitude 7.2, 1995 southern Hyogo prefecture earthquake in Japan liquefaction features in the field were induced by \( P \) wave arrival because they formed during the first 3 to 5 s after the main shock; before \( S \) wave arrivals were recorded at the site. Finally, Bachrach et al. [2001] discussed pressure-induced liquefaction using concepts of dynamic poroelasticity and demonstrated that critical conditions for liquefaction may be achieved owing to a resonant mode of Biot’s type II wave, a mechanism which is compatible with \( P \) wave loading.

[8] In this paper we present results from a carefully laid out field experiment where a 50-kg trinitrotoluene (TNT) charge was detonated at a depth of 10 m below ground surface, in a saturated sequence of silty sands, clayey sands, and clays. Since the experiment was conducted within the vadose zone some 50 m above the groundwater table, saturation was achieved by controlled seepage from a trench that was excavated on the surface. Particle velocities were measured by a vertical array of geophones beneath the trench at a horizontal distance of 30 m from the blast. Following the blast, liquefaction features such as sand boils, lateral spreading, and irreversible settlement were observed.

[9] On the basis of the results of in situ seismic velocity and SPT tests conducted in the field along with fines content analysis conducted at the laboratory, we conclude that blast-induced liquefaction may develop in relatively dense sediments (\( Dr = 63–89% \)) with shear wave velocity greater than 300 m s\(^{-1}\), and with fines content between 30 and 60%. Such sediments are not considered “prone to liquefaction” in standard prediction procedures which assume a shear-based failure mechanism.

2. Site Investigations

2.1. Geological Setting

[10] The study was performed in the southwestern coastal plain of Israel where sands, silts, and clays are abundant. Massive deposition of sands along the shoreline of Israel started at the upper Pliocene [Menashe, 2003] and continued until late Holocene. This was associated with loess deposition, consisting largely of silty-clayey deposits [Crouvi et al., 2008; Enzel et al., 2008; Yaalon and Dan, 1974; Zilberman, 1992]. The soils in the study site area classified as sandy Regosols, arid Brown soils and sand dunes [Dan et al., 1976]. Shallow (0.5–1.5 m) young sandy deposits cover the entire landscape. Arid calcic Brown soils are found underneath the sand. In the northern part of the study site loessial calcic alluvial soils are either covered by or mixed with the sandy calcic Regosols. This association is typical to the southwestern part of the coastal plain of Israel, as well as of alluvial valleys in between the dunes of northern Negev and northern Sinai deserts [Zilberman et al., 2007].

2.2. In Situ Test Methods

[11] The test site was restricted to a representative area of 4 km\(^2\), in which six 14-cm-diameter boreholes were drilled to a depth of 20 m, to provide information on physical and mechanical properties of the material. The material was sampled every 1 m in all boreholes for soil classification tests and determination of index properties such as density, porosity, and fines content. The general layout of the field test program and site investigation campaign is shown in Figure 1.
In order to determine the liquefaction potential of the sediments at the site according to standard prediction methods, SPT and seismic velocity measurements were performed in situ, and density, porosity, and fines content for the different layers were determined in the laboratory.

In situ SPT tests were conducted at the test site when the material was in its natural water content condition, a moisture condition which will be referred to as “dry” throughout this paper. Some samples were dried at the lab to actual zero water content for porosity and dry density determination (Table 1), but the term “dry” will be used to discuss the natural moisture of the material at the site before controlled infiltration was conducted, unless stated otherwise.

In situ seismic velocity tests were performed below the infiltration trench (Figure 1) twice, before and after the first controlled infiltration test. The obtained velocities and elastic parameters are referred to as “dry” and “wet” parameters, to describe material moisture conditions before and after the first infiltration test, respectively. As will be discussed later, we believe 100% saturation was only reached after the second infiltration test, when the “wet” blast test was conducted.

Direct shear tests at the laboratory were performed on undisturbed samples at their natural water content and therefore the obtained shear strength parameters represent “dry” material.

SPT tests were performed following ASTM standard D1586 in boreholes 1, 2, 3, 4, and 6 (Figure 1), and corrected $N_{1,60}$ values [Seed et al., 1985] were calculated for every 2-m interval down to a depth of 20 m, in all boreholes. The SPT test involves driving a standard cylindrical sampler into the bottom of a borehole. The total number of hammer blows required to drive the sampler over

![Figure 1. Field test layout: (a) location of exploratory boreholes in test site, the wet blast experiment site with the infiltration trench and the dry blast experiment site and (b) vertical cross section through the infiltration trench used for the wet blast experiment. Boreholes B6, B6a, and B6b were used for dry and wet seismic velocity tests conducted before and after the first infiltration phase and prior to the “wet” blast experiment. (c) Plan view of infiltration trench.](image)

<table>
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<th>$\rho_{dry}$ (g/cm³)</th>
<th>$w_w$ (%)</th>
<th>$n$</th>
<th>$\rho_o$ (g/cm³)</th>
<th>$\rho_{sat}$ (g/cm³)</th>
<th>$N$</th>
<th>$N_{1,60}$</th>
<th>$D_r$ (%)</th>
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Table 1. Physical Properties of the Sediments Obtained at the Laboratory and in Situ

Abbreviations are as follows: USCS, Unified Soils Classification System; FC, fines content; $\rho_{dry}$, dry density; $w_w$, initial water content; $n$, bulk porosity; $\rho_o$, initial density at natural water content; $\rho_{sat}$, density at saturation; $N$, standard penetration test (SPT) blow count value; $N_{1,60}$, modified SPT blow count value; $D_r$, relative density, empirical correlation based on Gibbs and Holtz [1957].
the interval 150 to 450 mm gives the blow count \( N \) value, defined as the number of blows per foot. The \( N \) value is used universally as the primary index for liquefaction resistance of sandy deposits.

[17] Downhole and cross-hole seismic velocity tests were performed by Ezersky [2004] following ASTM and ISRM standards [Takahashi et al., 2006] to determine \( P \) and \( S \) wave velocities both in dry and wet subsurface conditions. The sequence of events was as follows. Seismic velocity tests were first performed below the infiltration trench before it was flooded to obtain the “dry” velocity data set representing the material at natural moisture conditions. At that stage all preparations for the “wet” velocity tests were completed by extending the inner PVC casings in the three boreholes above the anticipated water level in the trench when flooded. Drilling was performed using a wireline technique to ensure verticality with an accuracy of up to 2°. After completion of the “dry” velocity tests a 17-day-long flooding phase commenced while water content levels in the subsurface were continuously monitored using TDR probes. After the completion of the first flooding phase the seismic velocity tests were repeated to obtain the set of “wet” velocity values. As will be discussed below, it seems that at end of the first flooding phase some sections of the strata did not reach full saturation, and therefore the “wet” seismic velocity data set represents a material which is nearly, but possibly not 100%, saturated.

[18] Boreholes B6, B6A, and B6B were used to perform seismic velocity tests (Figure 1). The three boreholes were drilled inside the infiltration trench before it was flooded, down do a depth of 20 m at 3-m spacing, with individual borehole diameter of 76 mm.

[19] The essential concept of the two methods is illustrated in Figure 2. For the downhole method (Figure 2a) a seismic source is set up at the ground surface and receivers are set in the borehole to measure seismic waves generated by the surface source. The cross-hole method (Figure 2b) uses three boreholes for setting up the source and two receivers to measure the travel time of seismic waves between these boreholes. While cross-hole tests, performed between boreholes B6 and B6a, provided a profile of \( S \) wave velocities only, both \( S \) and \( P \) wave velocities were obtained from two separate downhole tests performed inside these boreholes.

[20] The cross-hole test was performed in adherence to ASTM standard D4428. The energy source, designed to generate shear waves (SV type, manufactured by Soil Engineering and Geophysics Inc., USA.) was attached to borehole B6B using an air inflated, flexible flat jack. A cable was used to operate a hammer, the application of which triggered a seismic recording unit. The seismic waves were recorded in boreholes B6 and B6a using three component geophones (BHG-2 type, manufactured by GEO-STUFF, United States) which were attached to the walls of

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**Figure 2.** Schematic illustration of the in situ seismic velocity tests which were performed inside the infiltration test using boreholes 6, 6A, and 6B. (a) Downhole method. (b) Cross-hole method.
Table 2. Measured Mechanical Properties of the Sediments Before (Initial) and After the First Infiltration Phase Obtained From in Situ Seismic Velocity Tests

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$V_s$ (m/s)</th>
<th>$V_p$, $V_d$ (m/s)</th>
<th>$G_s$ (MPa)</th>
<th>$K_d$ (MPa)</th>
<th>At 100% Saturation</th>
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<td></td>
<td>$V_s$ (m/s)</td>
<td>$V_p$, $V_d$ (m/s)</td>
<td>$G_s$ (MPa)</td>
<td>$K_d$ (MPa)</td>
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<tr>
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<td>360 430 645 271 508 7.34</td>
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<tr>
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The values for 100% saturation were calculated. Abbreviations are as follows: $V_s$, shear wave velocity; $V_p$, over burden stress-corrected shear wave velocity; $G_s$, dynamic shear modulus; $K_d$, dynamic bulk modulus.

2.3. In Situ Test Results

The sediments at the test site can be classified into three major soil types according to ASTM unified soil classification system (ASTM D2487–00): (1) Nonplastic silty sand with fine content less than 20% (SM); (2) low-plasticity clayey sand with fine content greater than 33% (SC); and (3) low-plasticity clay with some amount of sand (CL). The clayey sand and the clays contain 3–5% carbonate concretions which are randomly distributed in the section. The variations in soil type, density, fines content, porosity, and in situ SPT values obtained at dry conditions in borehole 6 before the first infiltration phase are listed in Table 1. Seismic velocities obtained before and after the first infiltration phase are listed in Table 2 and plotted in Figure 3 for dry and wet subsurface conditions.

The density of the material in its natural water content ($\rho_o$) increases with depth, or with vertical overburden stress, from $\rho_o = 1.61$ g cm$^{-3}$ close to the surface to $\rho_o = 1.88$ g cm$^{-3}$ at a depth of 20 m (Table 1). Similarly, SPT values increase with overburden stress from $N = 16$ close to the surface to $N = 84$ at a depth of 20 m. This increase in $N$ value must reflect increase in the relative density of the material with increasing depth. Modified SPT values naturally do not reflect this trend because $N_{1,06}$ values are corrected for, among other parameters, the level of overburden stress [see Idriis and Boulanger, 2006; Seed et al., 1985]. $N_{1,06}$ values should, however, reflect material property variations. Indeed, $N_{1,06}$ values seem to be higher for the clayey sands and clays.

The seismic velocities obtained clearly increase with depth, supporting the previous conclusion that the relative density of the material increases with depth. This is best shown in Figure 3e where the shear wave velocities obtained in the Cross Hole survey are plotted versus depth for dry and wet subsurface conditions. Close to the surface the shear wave velocity for dry subsurface is $V_s = 402$ m s$^{-1}$ and it increases to $V_s = 547$ m s$^{-1}$ at a depth of 20 m (Figure 3e). $P$ wave velocities also increase with depth from $V_p = 690$ m s$^{-1}$ close to the surface to $V_p = 810$ m s$^{-1}$ at 20-m
Figure 3. Results of downhole and cross-hole seismic velocity tests before (solid line) and after (dashed line) first infiltration test: (a) columnar section below the infiltration trench; (b) $P$ wave velocity from downhole test in borehole 6; (c) $P$ wave velocity from downhole test in borehole 6a; (d) $S$ wave velocity from downhole test in borehole 6; (e) $S$ wave velocity from cross-hole tests performed between boreholes 6 and 6a; (f) $S$ wave velocity from downhole test in borehole 6a.
depth in dry subsurface conditions (Figures 3b and 3c). Overburden stress-corrected shear wave velocities [Andrus and Stokoe, 2000], the geophysical equivalent to the $N_{1,60}$ value, are also shown in Table 2.

[28] The dynamic elastic parameters for each layer are obtained from the following relationships:

$$\nu_d = \frac{\left(\frac{V_p}{V_s}\right)^2 - 2}{2 \left(\frac{V_p}{V_s}\right)^2 - 2}$$

(1)

$$G_d = \rho V_s^2$$

(2)

$$E_d = 2G_d(1 + \nu_d)$$

(3)

$$K_d = \frac{E_d}{(3 - 6\nu_d)},$$

(4)

where the subscript $d$ denotes “dynamic” and $\nu$, $G$, $E$, and $K$ are, respectively, the Poisson’s ratio, shear, Young’s, and bulk moduli. The calculated elastic moduli for dry and wet subsurface conditions are shown in Table 2 using the relevant velocities and densities for each layer.

[29] The elastic constants for wet subsurface conditions after the first infiltration phase (Table 2) were obtained using saturated densities ($\rho_{sat}$), namely the density of the material when the pore space is completely filled with water. The saturated density for each layer was obtained analytically using the measured values of dry density and porosity at the lab (Table 1). We use the saturated densities for each layer in our calculation of elastic parameters for wet conditions because the difference in material densities after the first infiltration phase and at 100% saturation is believed to be negligible.

2.4. Effect of Wetting on Seismic Velocities

[30] Interestingly, both measured $S$ and $P$ wave velocities decreased following the first, 17-day-long, infiltration phase. While theoretically the shear modulus should not be affected by saturation, the shear wave velocities should decrease upon wetting owing to the increased density of the material upon wetting. The expected reduction in shear wave velocity is readily given by the density correction factor (see equation (2)) if we assume that the shear modulus remains a constant: $V_{sat}/V_{dry} = \sqrt{\rho_{sat}/\rho_{dry}}$. Indeed, the average density correction factor for the 10 measured layers when using initial and saturated densities (Table 1) is 0.942 and the average ratio between the wet and dry shear wave velocities (Table 2) is 0.926. These values are very close suggesting that the change in total density owing to saturation is mainly responsible for the measured decrease in shear wave velocity. Note that the wet shear wave velocities obtained after the first infiltration phase (Table 2) although lower than in dry state, are still higher than the accepted upper bound for liquefaction that from Andrus and Stokoe’s [2000] compilation appears to be $V_{s1} = 300$ m s$^{-1}$.

[31] The decrease in $P$ wave velocity following the first infiltration phase clearly suggests that the material did not reach 100% saturation at the end of that phase, because in fully saturated subsurface conditions the $P$ wave velocity should be greater than about 1500 m s$^{-1}$, the velocity in fresh water at 25 C [Clark, 1966]. The presence of trapped air bubbles in the sequence, amounting to as low as 1–2% of the void space, would be sufficient to keep the $P$ wave velocities low, as shown for example by Bachrach and Nur [1998] using poroelasticity. The reduction of the wet $P$ wave velocities below the dry $P$ wave values can be explained again by the increase in total density upon increased degree of saturation in the voids. This will hold true until the void space is completely filled with water, at which point the $P$ wave velocity will sharply increase to values greater than 1500 m s$^{-1}$, as illustrated by Bachrach and Nur [1998].

[32] It is also possible that in cohesive, weakly cemented, granular materials stiffness degradation may develop with increasing degree of saturation [e.g., Assalay et al., 1996; Dijkstra, 2001; Fener et al., 2005; Jefferson et al., 2004] thus reducing the values of both the bulk and shear moduli. This process, if encountered in the subsurface, may affect both $S$ and $P$ wave velocities.

2.5. Theoretical Determination of $P$ Wave Velocity for 100% Saturation

[33] For estimating the $P$ wave velocities of the tested materials for fully saturated conditions poroelasticity may be invoked. Let the effective $P$ wave velocity of the material be

$$V_p = \sqrt{\frac{K + \frac{4}{3}G}{\rho}}.$$  

(5)

To find the $P$ wave velocity for saturated sediments the values of $K_{sat}$, $G_{sat}$, and $\rho_{sat}$ must be obtained. Since theoretically $G_{sat} = G_{drained}$ [Gassmann, 1951] only the value of $K_{sat}$ must be resolved. Using the Biot-Gassmann equation [Bachrach and Nur, 1998] $K_{sat}$ may be obtained from

$$\frac{K_{sat}}{K_o - K_{sat}} = \frac{K_{dry}}{K_o - K_{dry}} + \frac{K_f}{n(K_o - K_f)},$$

(6)

where $K_o$ is the mineral bulk modulus, $K_{dry}$ is the dry bulk modulus obtained from the results of the dry seismic velocity tests, $K_f$ is the bulk modulus for the pore fluid, in our case water where $K_{water} = 2.29$ GPa, and $n$ is the porosity, the value of which is known for each layer (Table 1).

[34] The mineral bulk modulus may be estimated using weighted average of the quartz and clay fractions in the sediments given that $K_{o,qtz} = 38$ GPa and $K_{o,clay} = 20.9$ GPa, by assuming that the “finer” fraction determined from sieve analyses represents clays and the coarser fractions represent quartz; a reasonable assumption for loess deposits. The computed $K_{sat}$ and $V_p$ values for each layer are shown in right column of Table 2. We used the shear modulus obtained for wet conditions for these calculations as it is
assumed to better represent the elastic response of the material when fully saturated.

3. Laboratory Direct Shear Tests

3.1. Test Systems

[35] Direct shear tests were performed at the laboratory on undisturbed samples from the two dominant material types, namely the silty sands (SM) and the clayey sands (SC) at their natural water content conditions, to obtain the characteristic shear strength of the material.

[36] Two test systems were employed, a high-capacity hydraulic load frame with a maximum shear force of 300 kN originally designed for testing rock joints, and a low-capacity mechanical load frame with a maximum shear force of 5 kN, originally designed for testing soils. Cubic block samples were quarried out of an exploratory trench in the test site for testing in the high-capacity load frame; cylindrical samples were extracted from the sampling tube when drilling boreholes 6, for testing in the low-capacity load frame.

[37] The high-capacity load frame is a hydraulic, closed-loop servo-controlled system (TerraTek Systems model DS-4250) with a particularly large shear box with dimensions of 170 mm x 170 mm x 300 mm allowing for a large sample cross-sectional area of up to 225 cm² (Figure 4). Both normal and axial cylinders are operated hydraulically in this system under two possible control modes: load and displacement. The normal and horizontal load capacities are 1000 kN and 300 kN respectively, with load-cell linearity of 0.25% full scale. Two 50-mm-range LVDTs were used to measure horizontal shear displacement and four 50-mm-range LVDTs were used to record vertical motions during shear; the LVDTs were manufactured by Macro Sensors® (model GHSA-750-1000) with 0.25% linearity full scale. The low-capacity load frame (Wykham Farrance 31-WF25421) is a commercially available microprocessor controlled drive system operated with a high-precision step motor.

3.2. Direct Shear Test Results

[38] Shear displacement versus shear load curves obtained with the servo-controlled system are plotted in Figures 5a and 5b for the silty sands and clayey sands, respectively for different levels of normal stress. All tests were performed under an imposed constant normal stress boundary condition and under a constant shear displacement rate of 0.025 mm s⁻¹. Since the normal piston maintained constant normal stress under load control the material was free to dilate or contract during shear. The corresponding average vertical displacements recorded by the four vertical LVDTs during horizontal shear displacement are plotted in the lower panels in Figure 5 where as a convention dilation is positive.

[39] The tendency of the silty sands to dilate during shear is suppressed with increasing normal stress (Figure 5a). The clayey sands exhibit a more brittle deformation (Figure 5b) and the material dilates during shear under all levels of normal stress. The direct shear test results therefore suggest that the clayey sands are stiffer than the silty sands. This finding is supported by the obtained in situ $N_{10}$ values (Table 1). The seismic velocities measured in situ do not seem to be sensitive enough to detect such differences in material stiffness, probably because the difference between the wavelength of the in situ tests and the sample size tested at the laboratory.

[40] The resulting failure envelopes for the two materials in $\tau$–$\sigma$ space are shown in Figure 6 including test results obtained with the sensitive 5-kN system. Both materials seem to obey the linear Coulomb-Mohr failure criterion. The silty sands exhibit negligible cohesion and an effective friction angle of $32^\circ$, or a friction coefficient of $\mu = 0.62$ (Figure 6a). The clayey sands exhibit cohesion of approximately 50 kPa and an effective friction angle of $41.5^\circ$, or a friction coefficient of $\mu = 0.88$ (Figure 6b).

4. Controlled Infiltration

[41] Two controlled infiltration experiments were performed. The first infiltration test lasted 17 days, then after a period of 3 months, the second 25-day-long infiltration test was performed. In both tests the excavated trench was flooded allowing downward water infiltration down to at least 20 m. The trench sidewalls were sealed so that infiltration took place only through its base. At the first experiment, after which the wet seismic velocity tests were performed, a constant head of 1.0 m above the trench base was maintained. During the first test the dimensions of the base of the trench were 17 m by 3 m, and the total amount of water used was 1400 m³. An important objective of the first experiment was to track the infiltration rate and its temporal and spatial patterns. Therefore, the original trench dimensions were relatively small. For the second infiltration experiment the trench was elongated by additional 31 m to provide infiltration area of 48 m by 3 m. Infiltration commenced for a period of 25 days during which a constant head of 1.25 m above the base of the trench was maintained. The total amount of water used during the second infiltration period was 10,300 m³. The propagation of the wetting front during that stage was faster, as expected.

[42] The temporal changes of water content were monitored at 28 points beneath the trench, using time domain reflectometry (TDR) probes placed in 4 boreholes, providing a two-dimensional vertical image below and across the trench. Detailed records of a 17-day period of wetting, followed by a 3-month period of drying, and finally,
followed by a second 25-day period of re-wetting, were obtained [Gvirtzman et al., 2008].

Time domain reflectometry (TDR) is a common method for measuring water content in unsaturated soils in the field. The TDR technique involves measurements of the propagation velocity of a high-frequency signal transmitted along a waveguide. The dielectric constant of water (80) is significantly higher than that of most minerals (2–7); therefore, propagation velocity measurements can be used to estimate the volumetric soil water content. Lately, monitoring deep vadose zone horizons (down to 25 m) has been made possible using flexible waveguides that are pressed against the borehole walls using a sleeve filled with liquid resin [Dahan et al., 2003].

In this study, we followed Dahan’s TDR setup and monitoring procedure. Four flexible polyethylene sleeves, each with 10-cm diameter and 22-m long, were prepared for the four boreholes. Eight TDR probes (waveguides) were glued to each sleeve, at a distance of about 2 m from each other. The probes were made of two stainless steel foil strips, each 30-cm long and 2-cm wide, glued parallel to each other, at a spacing of 5 cm. The sleeves were perfectly sealed to prevent future liquid leakage. Shielded coaxial cables were connected to the probes to allow in situ, real-time readings. All individual waveguides were connected to a digital cable tester (model 1502C, Tektronix, Beaverton, OR) through a multiplexer (model SDM50, Campbell Scientific, Inc. Logan, UT), and the entire system was connected to a data logger (model CR10X, Campbell Scientific, Inc. Logan, UT) for data collection. The probes were checked for TDR readings before the trench was flooded with water. Out of the 32 pairs of probes, 28 were functional after installation.

During the experiment, numerous TDR readings were collected, stored on a laptop, and the volumetric water content was calculated by the standard technique of waveform analysis. However, because of temporal and spatial changes in moisture salinity (owing to flooding and salt washing), the relationship between TDR signal and moisture content was not unique, and therefore the results have some inherent degree of uncertainty. Nevertheless, the wetting front propagation was reconstructed very precisely [see Gvirtzman et al., 2008].

Traced by the TDR probes, the wetting front was onion-shaped, and slowly propagated vertically and horizontally. Detailed description of the water flow in the

Figure 5. Shear stress (top panels) and dilation (bottom panels) versus shear displacement during servo-controlled direct-shear tests; dilation is positive. (a) Silty sands. (b) Clayey sands.
unsaturated zone is provided by Gvirtzman et al. [2008]. The spatial and temporal variations during the wetting, drying and re-wetting processes were reconstructed using a numerical code that solves the flow equation, under both saturated and unsaturated conditions. We divided the sediment column into a sequence of two fundamental sediment types, each of which was treated as an equivalent homogeneous medium. This approach transfers the heterogeneity problem within each of the units to a problem of identification of the effective parameters [see Wierenga et al., 1991; Wildenschild and Jensen, 1999]. The 2-D simulation domain was 35 m by 30 m in horizontal and vertical directions respectively, reflecting the actual size of the field experiment. This domain was discretized into 153 by 79 pairs of triangle elements (24,174 elements) with mean size of 0.2 m by 0.4 m. The bottom of the domain was assumed to be under a unit gradient condition; the left and right hand sides and the top of the domain, except for the center zone corresponding to the trench, were modeled as no-flow boundaries. A constant head boundary was assumed for the base of the trapezoid-shaped trench at the top boundary, reflecting the water head in the trench. The initial capillary tension distribution was assumed uniform at −3.0 m, reflecting the natural moisture content. [47] The retention curve, as well as the hydraulic conductivity versus moisture content curve for each unit, were plotted using the parameters of the Mualem–Van Genuchten model [Van Genuchten, 1980]. These curves were first estimated using field data and then adjusted by the numerical modeling results [see Gvirtzman et al., 2008]. In Figure 7 the temporal and spatial changes in moisture content during the second infiltration experiment, and the resulting pressure buildup, are shown graphically. The numerical simulation reconstructed the changes in the position of the onion-shaped wetting front detected in the field by the TDR probes. It is important to emphasize that sediment stratification, and variations in hydraulic conductivity under saturated conditions, result in differential flow rates throughout the profile and consequently pressure buildup inside the onion-shaped saturated zone. The results of the numerical simulations indicate that a head of up to 3.1 m developed in the saturated zone and in particular within the central silty sand unit, bounded in the field by the less-permeable sandy clay units form above and below.

5. Blast-Induced Liquefaaction Experiment

[48] To investigate induced liquefaction wet and dry blast tests were conducted in the field. Recall that the two
infiltration periods were followed by two different sets of tests. Before and after the first infiltration period seismic velocity tests were performed below the trench to obtain seismic velocities at dry and wet conditions. The infiltration trench was then let to drain without additional water inflow for a period of three months, during which it was elongated and preparations were made for the wet blast test. After the second 25-day-long infiltration period, the wet blast test was performed below the trench and after several hours the dry blast test was performed 300 m away from the trench on dry and leveled ground, with exactly the same subsurface composition as found below the infiltration trench.

Since the wet blast test was performed after the completion of the second infiltration phase we assume, on the basis of our hydrological model and TDR measurements, that the subsurface below the trench was 100% saturated at that time down to a depth of at least 14 m, before the charge was detonated.

5.1. Blast Test Setup

The layout of the blast tests is shown in Figure 1. A 50-kg trinitrotoluene (TNT) charge was placed and buried in a 40-cm diameter borehole that was drilled below one end of the infiltration trench to a depth of 10 m, the depth of detonation. After placing the charge the boreholes were back filled tightly with sand. At a horizontal distance of 30 m from the blast borehole along the trench axis a 20-cm-diameter borehole with 15.2-cm-diameter casing was drilled to a depth of 15 m for the placement of a vertical geophone array. The geophones (BMIII type three-component sensors, Instanell) measured particle velocities in longitudinal, transverse, and vertical directions. The triaxial geophones were triggered by the blast wave and data were stored 0.25 s before the trigger for a total acquisition time of 1.5 s at a sampling rate of 1024 samples per second. The range of the geophones was 254 mm s\(^{-1}\) with a resolution of 0.127 mm s\(^{-1}\). The natural frequency of the geophones was 10 Hz and the frequency range 2–250 Hz.

Two blast tests were performed: the first below the infiltration trench after the material in the subsurface attained full saturation, and the second 3 h later and 300 m away from the infiltration trench at leveled ground in similar material but under natural moisture conditions. The same charge, distance, and vertical array configuration were used in both tests. Similarly, the same four geophone packages were placed in the vertical arrays in the wet and dry blast experiments: sensor BA8850 was placed on the surface; sensors BA8109, BA8835, and BA8836 were placed in the vertical borehole 6 m, 10 m, and 14 m below the surface, in both tests.

A single electric piezometer (model LV610, DINA electronics) was placed immediately below the vertical array (Figure 1) to enable direct measurements of excess pore water pressures during and after the wet blast experiment, should they develop at that location.

5.2. Blast Test Results

The acquired wave forms in the vertical arrays for the dry and wet blast tests are presented in Figure 8. Peak particle velocities measured in each geophone are tabulated in Table 3. The longitudinal (radial–horizontal) particle velocity output at depth of 6 m for the dry test was corrupted owing to an electrical problem and unfortunately cannot be used. Nevertheless, the obtained trend is quite clear. First, particle velocities at saturated subsurface conditions are much higher than particle velocities for dry subsurface, down to a depth of 14 m where the particle velocity outputs seem to be the same. Second, the vertical component is higher than the two horizontal components close to the surface but diminishes to almost zero at 14 m depth, for both subsurface conditions. Third, the longitudinal component is much higher than the transverse component in both subsurface conditions, as would be expected.

To compare between velocity readings at different depths and moisture levels the magnitude of the resultant peak particle velocity (PPV) is first discussed. In the wet blast test the surface geophone measured resultant peak particle velocity of PPV\(_{(h=0 \text{ m})}\) = 92 mm s\(^{-1}\). At 6 m and 10 m the results were identical and similar to the surface reading: PPV\(_{(h=6,10 \text{ m})}\) = 90 mm s\(^{-1}\). At 14 m depth the resultant peak particle velocity was much lower: PPV\(_{(h=14 \text{ m})}\) = 35 mm s\(^{-1}\). For comparison, in the dry blast test the surface geophone measured PPV\(_{(h=0 \text{ m})}\) = 42 mm s\(^{-1}\), at 10 m PPV\(_{(h=10 \text{ m})}\) = 28 mm s\(^{-1}\), and at 14 m PPV\(_{(h=14 \text{ m})}\) = 18 mm s\(^{-1}\). The results of the blast tests therefore clearly illustrate the effect of water on the particle velocity output.
Figure 7. Two-dimensional numerical solution of saturation degree (left panels) and pressure head (right panels) below the infiltration trench from day 0 to day 21 during the second infiltration test, demonstrating the process of water penetration and spreading beneath the flooded trench. The onion-shaped, solid black lines represent the measured wetting front.
Figure 8. Results of vertical geophone array measurements in wet and dry blast tests. The vertical array was positioned at a horizontal distance of 30 m from the source.
show that peak particle velocities decreased with decreasing degree of saturation, and with depth.

The acquired wave forms (Figure 8) do not permit reliable differentiation between the wave types that reached each geophone in the vertical array. While $P$ waves would be expected to be dominant owing to the nature of the energy source, possibly $S$ waves as well as surface waves could have been generated by the blast. The presence of surface waves for example is suggested by the observation that the vertical component was higher than the two horizontal components close to the surface but diminished to almost zero at 14-m depth, for both subsurface conditions.

Seven minutes after the wet blast test a water fountain erupted from the blast borehole to a height of several meters. After the “geyser” subsided (Figure 9a) the water that remained at the base of the trench continued to bubble (Figure 9b) for a period of about 30 min, to a horizontal distance of 15 m from the blast borehole along the trench axis. Approximately 20 min after the blast lateral spreading features were observed at opposite banks across from the blast borehole (Figure 9c). An accurate topographic survey performed immediately after the blast with continued ejection of a water fountain from the blast borehole for example does not necessarily imply that liquefaction was triggered in the adjacent saturated domain, as the immediate blast region is characterized by singularities in the generated pressure field typically resulting in mud fountains.

The bubbling of the water table inside the infiltration trench however (Figure 9b) does prove that liquefaction was triggered, as this is a common feature in sites undergoing instantaneous liquefaction following strong earthquakes, typically marked by “sand boils” that remain on the surface after the event [e.g., Fiegel and Kutter, 1994; Moretti et al., 1999; Rossetti, 1999; Yuan et al., 2004].

The breakage of the surface on two opposite banks of the infiltration trench (Figures 9c, 10b, and 10c) resembles lateral spreading features documented at sites that have experienced earthquake-induced liquefaction [e.g., Bardet et al., 2002; Boulanger et al., 1997; Cetin et al., 2004b; Rajendran et al., 2001]. The observed breakage of the ground surface may reflect brittle response of the dry upper layers to the liquefaction of underlying, saturated, layers (Figure 9c), as documented in many case studies reporting earthquake-induced lateral spreading phenomena [Rauch, 1997]. Volume changes leading to a more compact configuration in the underlying layers undergoing liquefaction could be followed by brittle breakage of the dry upper layers resulting in lateral spreading at the surface, as observed on both slopes (Figure 9c).

Finally, the measured ground settlement (Figure 10), while often reported from sites that have experienced blast-induced liquefaction [e.g., Cetin et al., 2002], is classic evidence for blast-induced liquefaction [Charlie et al., 2001; Gandhi et al., 1999]. The measured cone shaped settlement of the ground surface reflects a transformation of the deeper liquefied sand layers into a denser configuration. This process, although evidently time-dependent, should not be confused with consolidation, a very slow process the rate of which depends upon the hydraulic conductivity and the compressibility of the soils. The observed failure modes owing to the blast are further discussed below.

### 6. Discussion

#### 6.1. Failure Modes Triggered by the Wet Blast Experiment

Four different deformation patterns were observed at the test site following the wet blast experiment, all indicate excess pore pressure generation and all exhibit time dependency: (1) a water fountain that erupted from the blast borehole immediately after the blast with continued ejection of a water jet for a duration of $\sim$8 min (Figure 9a),

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Longitudinal Initial Sat.</th>
<th>Transverse Initial Sat.</th>
<th>Vertical Initial Sat.</th>
<th>Resultant Initial Sat.</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>32.8</td>
<td>51.9</td>
<td>4.06</td>
<td>31.4</td>
</tr>
<tr>
<td>6</td>
<td>...</td>
<td>89.0</td>
<td>24.6</td>
<td>18.3</td>
</tr>
<tr>
<td>20</td>
<td>27.7</td>
<td>89.5</td>
<td>3.05</td>
<td>24.3</td>
</tr>
<tr>
<td>30</td>
<td>16.8</td>
<td>33.5</td>
<td>15.6</td>
<td>19.0</td>
</tr>
</tbody>
</table>

*aInitial refers to moisture conditions at natural water content.*

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#### 6.2. Field Measurements of Excess Pore Water Pressure

The magnitude of excess pore pressure required for blast-induced liquefaction can be determined analytically, if the change in total stress owing to the blast wave ($\Delta p$) can be estimated. Let $P_o$, $P'_o$, and $u_o$ represent, respectively, initial mean total stress, initial mean effective stress, and initial pore water pressure, at a given point below the groundwater table that for the sake of this discussion, will be assumed to coincide with the leveled ground surface. The initial mean effective stress is then given by [$Terzaghi,
Figure 9. Failure modes associated with induced liquefaction as observed in the field following the “wet” blast experiments: (a) termination of water geyser eruption from the blast borehole, (b) bubbling of water surface inside the infiltration trench (sand boils), and (c) lateral spreading observed on the dry slopes of the infiltration trench.
During transient loading owing to the blast the mean effective stress becomes $P' = P - u = (P_o + \Delta p) - (u_o + \Delta u)$ where $\Delta p$ is the transient change in mean total stress owing to the blast and $\Delta u$ is the resulting excess pore water pressure, both of which are time-dependent. The limiting condition for liquefaction can be stated as the point in time when the mean effective stress diminishes:

$$P' = 0.\quad (7)$$

The excess pore pressure required for blast-induced liquefaction $\Delta u_c$ at that point in time is therefore

$$\Delta u_c = P'_o + \Delta p.\quad (8)$$
Finding $\Delta u_s$ by equation (8) requires accurate determination of $\Delta p$, a task which is beyond the scope of this paper. The limiting condition for liquefaction (equation (7)), if and where reached, will only be satisfied for a very short period of time. After the dynamic loading owing to the high-amplitude blast waves diminish, a process which may last $10^1$–$10^2$ ms (e.g., Figure 8), excess pore pressure dissipation will ensue at rate and form similar to experi-

Figure 10. (continued)
mental results as obtained for example by Ashford et al. [2004] and reproduced in Figure 11. The exact form of the pore pressure dissipation curve will depend upon the type and amplitude of the blast-induced waves, the compressibility of the soil skeleton, the hydraulic conductivity of the material, and the availability of drainage paths at the subsurface. [66] The excess pore pressure at any given time and location in the saturated domain is typically expressed by the excess pore pressure ratio \( R_u \), \( R_u = \Delta u / \sigma_v' \), where \( \sigma_v' \) is the initial vertical effective stress. A pore pressure ratio of \( R_u = 1.0 \) is generally defined as liquefaction in sandy soils, regardless of the loading mechanism. For blast-induced liquefaction existing reports suggest that it can be triggered at lower excess pore pressure ratios, with a lower bound at \( R_u = 0.8 \) [Kok and Trense, 1979]. The bubbling of the water surface inside the trench, as well as the water fountain, are surface manifestations of areas in the subsurface where the developed excess pore pressure ratio must have been within the range of 0.8 < \( R_u < 1.0 \).

[67] Since analytical determination of \( \Delta u \) owing to blasting requires a determination of the transient load \( \Delta p \), a magnitude of which is very difficult to obtain either experimentally or numerically, an empirical relationship between the scaled distance of the charge mass; the Hopkinson’s number \( \left( \sqrt{M/R}; \sqrt{\text{kg/m}} \right) \) and the resulting excess pore pressure has been proposed by several authors on the basis of field tests [e.g., Kok and Trense, 1979; Studer and Kok, 1980] and confirmed in recent studies [Ashford et al., 2004]:

\[
\frac{\Delta u}{\sigma_v'} = \begin{cases} 
0 & \text{for } \frac{\sqrt{W}}{R} < 0.12 \\
1.65 + 0.64 \cdot \ln \left( \frac{\sqrt{W}}{R} \right) & \text{for } \frac{\sqrt{W}}{R} \geq 0.12
\end{cases}
\]  

(9)

where \( M \) and \( R \) are the charge mass and the radial distance between the charge and the target, respectively. Equation (9) can be used to delineate the boundaries for blast-induced liquefaction at the subsurface on the basis of existing experience, given the charge mass and radial distance from the source point. This was done in Figure 12 for the scale and outline of the wet blast experiment. Indeed, the extent of the region where water bubbling (sand boils) was observed inside the infiltration trench coincides with the empirical \( Ru = 0.8 \) boundary.

[68] From the computed \( Ru \) boundaries plotted in Figure 12 it appears that the vertical array was drilled at the edge of the region where excess pore water pressure would be expected. This explains the lack of excess pore pressure readings in the piezometer during and after the wet blast test.

6.3. Ground Settlement

[69] Finally, it would be instructive to confirm that our measurements of ground settlement following the wet blast experiment (Figure 10) are consistent with reports from other sites that have experienced liquefaction under similar loading conditions. In our test site two liquefiable sandy layers are separated by an intermediate, nonliquefiable clayey layer: an upper 350-cm-thick layer of liquefiable sand is underlain by intermediate 490-cm-thick nonliquefiable clays, which overlay a lower 290-cm-thick liquefiable sand layer (Figure 12). In the preceding sections of this discussion we have argued that liquefaction must have developed in both liquefiable layers by virtue of excess pore pressure. Yet, the intermediate clay layer may absorb the deformations associated with liquefaction of the lower liquefiable layers, such that its failure will not have any surface manifestation. In such a case the amount of surface settlement would only reflect the thickness of the uppermost liquefiable layer.

[70] The thickness of an intermediate, nonliquefiable layer, required to absorb deformations associated with liquefaction of underlying liquefiable layers was investigated by Ishihara [1985] who compiled reports of ground failures triggered by earthquake-induced liquefaction at sites which have experienced strong ground motions (up to 0.2 g). Ishihara [1985] found that a nonliquefiable layer thickness of 3 m would be sufficient to absorb all deformations associated with liquefaction of underlying liquefiable layers. Ishihara’s study confirmed that with sufficient thickness of the intermediate, nonliquefiable layer, the liquefaction of underlying liquefiable layers will not have any surface manifestation, regardless of their thickness.

[71] In our test site the thickness of the intermediate nonliquefiable layer is 490 cm and therefore, on the basis of Ishihara’s study, it may be assumed that the liquefaction of the underlying liquefiable layers did not produce any surface manifestation. Consequently, all measured surface settlements should be associated with the uppermost liquefiable sandy layer.

[72] Ground settlement is typically discussed in terms of the ratio between the measured amount of surface settlement \( \Delta H \) and the thickness of the liquefied layer \( H \). For blast-induced liquefaction Kok [1981] proposed an empirical relationship between the observed ground settlement \( \Delta H / H \) and Hopkinson’s number for tests performed in saturated sands:

\[
\frac{\Delta H}{H} \%(\%) = \begin{cases} 
0 & \text{for } \frac{\Delta u}{\sigma_v'} < 0.8 \\
2.73 + 0.9 \cdot \ln \left( \frac{\sqrt{W}}{R} \right) & \text{for } \frac{\Delta u}{\sigma_v'} \geq 0.8
\end{cases}
\]  

(10)
where $\Delta H$ is the surface settlement as a result of blasting, $H$ is the initial thickness of the liquefiable layer, $W$ is the mass of the charge in equivalent kg of TNT (kg), and $R$ is the radial distance between the charge and the target (m). Application of equation (10) to this study (Figure 13a) suggests that settlement should be expected up to a horizontal distance of 11.20 m from the source, a prediction that is confirmed by our test results (see Figure 10e). Equation (10) is plotted graphically in Figure 13b along with our settlement measurements performed one day after the wet blast test. Our results are within the range of average settlements predicted by equation (10), but show better the obtained settlement cone around the blast borehole.

7. Summary and Conclusions

Results from a controlled blast-induced liquefaction experiment are reported in this paper. The experiment was performed above the groundwater table and therefore controlled infiltration tests, executed in two phases, preceded the blast test. The sediments at the site consist of alternating layers of silty sands, clayey sands, and clays. The mechanical behavior of the material was determined using in situ standard penetration (SPT) tests, in situ seismic velocity tests performed in both dry and wet subsurface conditions before and after the first infiltration phase, and laboratory direct shear tests. The physical properties of the materials including density, porosity, and fines content were also determined at the laboratory, for every 2-m interval.

The density of the material in its natural water content ($\rho_w$) increased from $\rho_w = 1.61$ g cm$^{-3}$ close to the surface to $\rho_w = 1.88$ g cm$^{-3}$ at a depth of 20 m reflecting material densification with increased overburden. Similarly, SPT values increased from $N = 16$ close to the surface to $N = 84$ at a depth of 20 m reflecting an increase in the relative density with depth. The fines content varied from 16 to 32% in the silty sands, 35% in the clayey sands, and from 56 to 68% in the clays.

Shear wave velocity for dry subsurface increased from $V_s = 402$ m s$^{-1}$ close to the surface to $V_s = 547$ m s$^{-1}$ at a depth of 20 m. $P$ wave velocities also increased from $V_p = 690$ m s$^{-1}$ close to the surface to $V_p = 810$ m s$^{-1}$ at 20-m depth in dry subsurface conditions. Interestingly, both measured $S$ and $P$ wave velocities decreased as a result of wetting, following the first infiltration phase. The reduced seismic velocities are explained by degraded material stiffness owing to wetting, a typical response of clay rich,
weakly cemented, granular soils to the presence of water in the voids. The decrease in $P$ wave velocity following the first infiltration phase suggests that the material did not reach 100% saturation at that time. $P$ wave velocities for 100% saturation are therefore determined analytically. Our numerical modeling of the groundwater flow below the infiltration trench clearly indicates that the material at the subsurface did reach 100% saturation down to a depth of 14 m after the second infiltration phase and before the wet blast test was executed.

Direct shear tests were performed at natural water content; the clays were not tested owing to sampling difficulties. It was found that the sandy materials at the site obey the Coulomb-Mohr failure criterion. The silty sands exhibit negligible cohesion and an effective friction angle of 32° ($\mu = 0.62$); the clayey sands exhibit cohesion of approximately 50 kPa and an effective friction angle of 41.5° ($\mu = 0.88$).

Blast tests were performed at both saturated and dry subsurface conditions, below the infiltration trench and 300 m away from it, respectively. Particle velocities owing to the blast experiments were measured using a vertical geophone array positioned 30 m away from the source. We found that particle velocities at saturated subsurface conditions were much higher than particle velocities at dry subsurface, down to a depth of 14 m where the particle velocity outputs of the two tests were the same. The longitudinal component was much higher than the transverse component in both subsurface conditions. The vertical component was higher than the two horizontal components close to the surface and diminished to almost zero at 14-m depth, for both subsurface conditions.

Four different types of liquefaction features were observed at the wet blast experiment site, whereas none were detected at the dry blast test site. Seven minutes after the wet blast test a water fountain (or a “geyser”) erupted from the blast borehole to a height of several meters. After the “geyser” subsided the water surface at the base of the trench continued to bubble for a period of about 30 min, to a horizontal distance of 15 m from the blast borehole along the trench axis. This procedure is assumed to be equivalent to “sand boils” typically observed on the surface of sites that have experienced liquefaction. Approximately 20 min after the blast lateral spreading features were observed at opposite banks of the trench across from the blast borehole. An accurate topographic survey performed immediately after the blast indicated that the ground subsided by about 85 mm close to the borehole and by up to 15 cm within a

Figure 13. Empirical determination of relative settlement [Kok, 1981] versus field measurements (this study) after wet blast test. (a) Geometrical layout. (b) Comparison between predicted (solid circles) and measured (solid diamonds) settlement owing to blast-induced liquefaction.


radius of 15 m from the borehole. An ultimate settlement of 150 cm was measured around the blast borehole 70 days after the experiment. The radius of the disturbed zone remained 15 m after 70 days.

The water fountain and bubbling of the water surface inside the infiltration trench indicate excess pore pressure generation at the subsurface. The lateral spreading observed on opposite banks of the infiltration trench could be a surface manifestation of material failure at the saturated subsurface, owing to diminished effective normal stress as a result of excess pore pressure build up. It may also reflect the brittle response of the dry material forming the banks of the infiltration trench to the settlement of underlying layers that have experienced liquefaction in the saturated zone. The measured settlement represents rearrangement of the material at the subsurface into a more compact configuration, after excess pore pressure has been dissipated. The extent of the area that has been subjected to excess pore pressures and the amount of surface settlement are in good agreement with published empirical relationships.

The major contribution of this paper is the detailed and comprehensive characterization of the materials at the site using a wide range of testing methods that cross the boundaries between established scientific and engineering disciplines, and the careful documentation of the controlled field experiments as well as the obtained failure modes in the field. We show that liquefaction can be induced when subjecting the saturated sediments to a variety of high-amplitude waves that in the particular case study reported here are emitted from a blast. We conclude that liquefaction can be induced in relatively dense sediments (relative density 63–89%) with shear wave velocities greater than 300 m s⁻¹, and with relatively high fines content. Such sediments are not considered prone to liquefaction in standard liquefaction prediction procedures.

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