Performance of Driven Displacement Pile–Improved Ground in Controlled Blasting Field Tests
Tygh N. Gianella, A.M.ASCE1; and Armin W. Stuedlein, M.ASCE2

Abstract: Full-scale, controlled blasting field tests on driven displacement pile–improved ground were conducted to study the response of densified and reinforced ground to blast-induced excess pore pressures. In order to make appropriate comparisons to the baseline response of the native, unimproved ground, explosive charges sufficient to induce liquefaction were detonated in a control zone and the resulting postliquefaction settlements were measured. Excess pore pressures generated in the improved ground were observed to be significantly smaller than that in the unimproved ground, and resulted in settlements that were generally one-sixth to one-third of that measured in the unimproved ground. Piles tipped into a dense bearing layer settled significantly less than the surrounding soil and piles that were floated above the bearing layer. Importantly, measured excess pore pressures pointed to a change in soil response from contractive to dilative during blasting, indicating that the improved ground mobilized significant strength during blasting, similar to the response expected from cyclic mobility of dense soils. The energy of scaled ground motions developed from velocity measurements are used to relate the observed soil response to blasting to that expected from earthquake-induced ground motions. The paper concludes with a comparison of shear strains expected from shear strain compatibility (SSC) between the improved ground and the displacement piles to those implied by the measured pore pressures. The comparison indicated that some portions of the improved ground responded in an incompatible manner during the blast-induced ground motions and that the assumption of SSC may not be appropriate for design of some reinforcement-type ground improvements. DOI: 10.1061/(ASCE)GT.1943-5606.0001731. © 2017 American Society of Civil Engineers.

Introduction
Numerous and careful observations of the effects of earthquakes have pointed to the potential for liquefaction of relatively loose, saturated granular soils and resulting damage to civil infrastructure. However, cyclic stress-induced excess pore pressure magnitudes equal to the effective overburden pressures (i.e., liquefaction) are not required within a particular soil layer to initiate damaging magnitudes of settlement (Tokimatsu and Seed 1987; Ishihara and Yoshimine 1992). Based on the results of laboratory investigations, Lee and Albaisa (1974) determined that the primary factor governing postshaking volumetric strains (and therefore settlements) was the magnitude of the excess pore pressure ratio, \( r_p \), defined in their study as the ratio of cyclically-induced excess pore pressure to the isotropic consolidation stress. Among other factors, the pre-shaking relative density of the soil contributed to variations in the resulting magnitude of volumetric strains or postshaking compressibility, with increases in relative density leading to reductions in deformation for a given \( r_p \). Later, Seed et al. (1975) synthesized these observations using an analytical model that captured the general changes in compressibility, assessed using the modulus of volume compressibility, \( m_v \). Fig. 1 summarizes the measurements by Lee and Albaisa (1974) and analytical model by Seed et al. (1975). Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992) extended the previous findings to determine that the magnitude of cyclic shear strain imposed on liquefiable soils also governed the magnitude of postshaking volumetric strain, or settlement.

The aforementioned findings, among others, have led to accepted design protocols for the mitigation of liquefaction and its effects, such as setting a limiting maximum \( r_p \) to 50–60% or less (Schaefer et al. 1997). These protocols seem most appropriate for ground improvement techniques that reinforce loose soils in between stiffened elements, such as deep soil mix columns or panels, or jet grouted columns. However, densification-based ground improvement techniques increase the relative density of the soil, and subsequently lead to reduced settlements following the generation and dissipation of excess pore pressure (Fig. 1). Methods that both densify and reinforce, such as displacement piling, may offer redundancies that may provide optimal cyclic performance. Raising the maximum allowable \( r_p \) possible with such redundant methods may lead to improved cost-efficiency of ground treatment if good performance can be demonstrated to rightfully cautious engineers. Controlled blasting may be used to evaluate the in situ reduction in postshaking deformations that is possible with ground improvement (Ashford et al. 2000a, b). This paper describes the use of controlled blasting to compare the full-scale performance, including the generation of blast-induced excess pore pressures and subsequent deformations, of driven timber displacement pile–improved ground to that of the native, unimproved ground. First, the subsurface of the test site and program selected to evaluate the effect of various pile spacings and time elapsed since driving on densification of potentially liquefiable soils is summarized. Experiments conducted to evaluate the baseline response of unimproved ground to controlled blasting are described, and the blast-induced excess pore pressure response and postliquefaction ground settlements are presented. The pore pressure response and postblasting settlements in the improved ground resulting from the same blast pattern are then presented, and comparisons to the unimproved ground are made. Although peak residual excess pore pressures in excess of 60% of the effective overburden stress were observed in the

1Staff Engineer, GeoEngineers, Inc., 1200 NW Naito Pkwy. #180, Portland, OR 97209.
2Associate Professor, Oregon State Univ., 101 Kearney Hall, Corvallis, OR 97331 (corresponding author). E-mail: armin.stuedlein@oregonstate.edu

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improved ground, the blast pulses forced the pore pressure response from contractive to dilative, and indicated that the strength and stiffness of the densified ground was effectively mobilized, serving to limit the subsequent deformations. The site-specific attenuation characteristics of the blast-induced ground motions are presented, and used to estimate the free surface ground motions at the center of the displacement pile–treated area and are shown to correlate to the measured pore pressure responses. The paper concludes with comparisons of the observed and estimated performance to that expected assuming shear strain compatibility of the pile-reinforced soils. Comparisons indicate that the measured excess pore pressures and implied shear strains are inconsistent with shear strain compatible deformation for some portions of the pile-improved soil during the blasting-induced ground motions.

Subsurface Conditions and Test Pile Program

The investigation of soil response to controlled blasting was conducted at two separate areas separated 15 m apart, and included the unimproved or control zone and the pile-improved or treated zone. Gianella et al. (2015) and Stuedlein et al. (2016) describes the displacement pile ground improvement test program and postimprovement in situ tests in detail; a brief summary follows to provide an appropriate context for interpretation of the blast experiments. Fig. 2 presents the site and exploration plan along with the pile groups constructed to evaluate the magnitude of ground improvement possible with conventional and drained timber piles. Fig. 3 shows the subsurface profile and preimprovement and postimprovement in cone penetration test (CPT) corrected cone tip resistance, $q_t$. Treated Zones 1 and 2 consisted of timber piles fitted with prefabricated vertical drain (PVD) elements, and were installed in

**Fig. 1.** Variation of the modulus of volume compressibility, $m_v$, with peak excess pore pressure ratio, $r_u$, deduced from experiments and theoretical considerations

**Fig. 2.** Site and exploration plan including pile locations, initial in situ tests, and blast casing locations; note that the control zone was located more than 15 m northeast of Zone 5
five-by-five pile groups spaced at five and three pile head diameters ($D$, equal to 310 mm, on average), respectively. Pile groups in treated Zones 3, 4, 5A, and 5B were installed at 5D, 3D, 2D, and 4D, respectively.

The piles were installed through a 2–2.5 m thick layer of loose to medium dense, clayey and silty sand fill (SM and SC), soil that included residential housing debris (e.g., brick, wood, roofing shingles, etc.) generated following Hurricane Hugo in 1989. Pre-drilling and spudding of pile locations to depths of 2–3 m was required to allow the piles to penetrate the fill layer. Underlying the fill was a 8.5–9 m thick layer of loose to medium dense (prior to improvement), potentially liquefiable clean to silty sand (SP and SM), overlying a 1–1.5 m thick stratum of sandy clay (CH), and followed by a deposit of dense to very dense sand (SP). The baseline $q_t$, shown in Fig. 3, corresponds to the conditions at the center of each treated zone prior to improvement (Fig. 2). The stratigraphy across the site was relatively uniform; prior to improvement, $q_t$, and energy-corrected standard penetration testing (SPT) blow count, $N_{60}$ ranged between approximately 1 and 10 MPa and 1 to 10 blows per 0.3 m, respectively, within the potentially liquefiable soil layer. The groundwater table was approximately 2.13 m below the ground surface during the first blast event, and 2.15 m during the second blast event (described subsequently).

In general, $q_t$ measured 10 days following pile installation (Fig. 3) indicated that the improvement increased with reductions in pile spacing (Stuedlein et al. 2016). Approximately 8 months following installation, $q_t$ reduced across the test zones, with the greatest reductions associated with the greatest pile spacing. Zone 2, with drained piles spaced at 3D, yielded the largest magnitudes of $q_t$ 255 days following installation; however, it is not clear if this observation resulted from the presence of PVDs or the pile installation sequence (Stuedlein et al. 2016). In general, the relative density of the subsurface increased from 40–55% to 60–90% as measured 255 days following pile installation, depending on the pile head spacing and depth.

### Use and Limitations of Controlled Blasting

Controlled blasting was conducted to compare the effectiveness of the driven displacement pile–improved ground to reduce excess pore pressures and deformations to that of the unimproved control zone. Although explosives cannot replicate the ground motions associated with earthquakes, they can be used to evaluate effects associated with earthquakes, such as liquefaction and the subsequent deformations (settlements). Hence, the use of controlled blasting in geotechnical experimentation has gained wide acceptance over the last 15 years. Experiments ranging from the performance of stone columns (Ashford et al. 2000a, b), axial (Rollins and Strand 2006) and lateral response of deep foundations (Ashford et al. 2004; Rollins et al. 2005, 2006; Weaver et al. 2005), and earthquake drains (Rollins et al. 2004) have been successfully conducted. Gohl et al. (2001) describe an experiment conducted to relate measured blast–induced shear strains to $r_u$, and showed that their blast pattern replicated the shear strain-pore pressure triggering curves developed in laboratory tests described by Dobry et al. (1982).

Nevertheless, significant differences in the characteristics of blast-induced and seismic ground motions exist, and a brief discussion of these differences is helpful for the interpretation of the results described in this study. Generally, earthquake-induced ground motions in a near-surface free-field produce vertically-propagating horizontal shear stresses as the dominant loading type (Seed 1979).
Conversely, detonation of explosives results in the generation of a shock wave that propagates radially from the charge (Dowding and Hryciw 1986). The resulting ground motions reflect the passage of the shock wave that is characterized by initial compressive followed by tensile hoop stresses (Narin van Court and Mitchell 1994), which upon unloading, induce shear stresses owing to the expanding conical shock wave geometry (Hryciw 1986). These stresses travel at the P wave velocity of the soil, as opposed to the S wave velocity associated with earthquake-induced cyclic shear stresses. The peak amplitude and frequency content of near-field acceleration time histories associated with blasting is significantly higher than those of earthquake time histories, and are characterized with high amplitude shear strain pulses (Gohl et al. 2001), implying higher shear strain rates than those of earthquakes. However, Gohl et al. (2001) noted that the ground velocity and displacement amplitudes developed from blasting over the test volume are comparable to those generated by earthquake motions; this development is attributed to the high frequency nature of the accelerations (Kramer 1996; Dowding and Duplaine 2004). Pore pressures that are generated from blasting result from a combination of changes in total mean stress during the blast pulse, transient mean effective stresses imposed on the soil skeleton, and unloading-type shear strains; of these, the major contributor to residual excess pore pressures is from the shear strains (Gohl et al. 2010). Thus, correlation of the development of liquefaction and subsequent effects from blasting to that from earthquake motions is possible.

Controlled Blasting Experiments

Instrumentation

Various instruments were used to observe the effectiveness of the driven displacement pile–improved zones and the control zone such as pore pressure transducers and ground surface settlement monitoring points for optical leveling. Additionally, regulations in South Carolina required that ground vibrations associated with the detonation of explosives be monitored given the proximity of several privately-owned structures. Velocity time histories of each blast event were monitored at the ground surface of five structures adjacent to the test site using Minimate Plus seismographs manufactured with a standard triaxial geophone. Velocity amplitudes of 0.5 mm/s triggered the seismographs, which then initiated sampling at 333 Hz. Unfortunately, geophones were not set above each of the blasted zones, and therefore no measurements of the ground motions at these areas are available. However, measurements at each of the five structure locations allowed the development of site-specific attenuation relationships that can be used to estimate ground motions at the ground surface at the center of the control and treated zones, as described subsequently.

Pore pressure transducers (PPTs) were installed within boreholes drilled in the center of the control zone (B-1) and treated zones (B-3, B-5, B-7, and B-9) in order to observe the excess pore pressures developed as a result of the detonation of explosive charges. Druck model UNIK 5000 PPTs capable of measuring pressures of up to 5.2 MPa, and withstanding blast pressures of up to 20.7 MPa were used. A sampling rate of 5 Hz was used because the focus of this investigation was on the peak residual excess pore pressures (i.e., those generated following passage of the shock wave); accordingly, peak blasting pressures may not have been captured. The PPTs were individually calibrated prior to insertion within weighted, protective acrylic housings fabricated similar to that described by Cox et al. (2009) and grouted within the boreholes at nominal target depths of 4.6, 6.1, 7.6, and 9.1 m. The actual depth of PPT installation varied from borehole to borehole, as described subsequently. A low-strength cement-bentonite grout was used to seal the PPTs and replace excavated soil over the entire length of the borehole and prevent communication of pore pressures within the borehole.

Control Zone: Blast Program and Excess Pore Pressures

The blasting program for the unimproved control zone consisted of four separate blasts, of which the first two consisted of small charge weights to check the responsiveness of the PPTs and data acquisition system (termed BE1 and BE2; Mahvelati et al. 2016). The third and fourth blast events form the focus of the investigation described in this paper. Explosive charges were constructed of penterythritol tetranitrate (PETN), sized to an equivalent of 0.91 kg of trinitrotoluene (TNT) each. The experimental setup for the third blasting event (termed BE3) at the control zone consisted of six blast casings designated B-1E2 through B-6E2 installed within a circular arrangement (with radius of 3.81 m) as shown in Fig. 2, with four decks of charges in each casing. The decks were located at depths of 3.7, 5.3, 7.2, and 8.8 m below the ground surface, resulting in a total charge weight of 21.8 kg. This charge weight was selected based on the subsurface conditions from in situ tests at the test site and nearby blast-induced liquefaction studies reported by Camp et al. (2008). The intention was to verify that the charge weight necessary to induce liquefaction in the unimproved ground at the control zone was sufficient prior to blasting the displacement pile treated zone.

The blast sequence comprising BE3 was designed so that the center of the control zone would experience blast pulses from opposing directions in an effort to push and pull the ground as described by Gohl et al. (2001) and others (e.g., Rollins and Strand 2006; Ashford et al. 2004; Rollins et al. 2005, 2006; Weaver et al. 2005). The blast sequence started at the bottom deck, with charges detonated in approximate diametrically-opposed locations with detonations of one and one, then two and two charges (i.e., four detonations per deck for a total 16 detonations over 9 s) with a 600 ms delay between blasts. The blast sequence started at the bottom deck and worked upward toward the surface where the sequence was repeated. See Gianella (2015) for further details of the blast sequence. Unfortunately, BE3 was unable to be executed as intended, and the 24 charges were detonated over a relatively short time frame (i.e., approximately 1 s). New blast casings were installed in the control zone six months later, and the intended blasting sequence in the control zone, Blast Event 4 (i.e., BE4) was performed.

Fig. 4 presents the excess pore pressure ratio time history for each of the PPTs measured at the control zone during BE3 and BE4. The PPTs at elevations of 6.35, 7.39, and 8.60 m reached peak $r_u$ of 126, 140, and 152%, respectively, and peak residual $r_u$ ranged between 75 and 100% during BE3. Complete liquefaction was achieved at the deepest elevation (i.e., $r_u = 95–100\%$), and that near-complete liquefaction was achieved for the depth of 7.39 m. Fig. 4(b) presents the excess pore pressures measured following BE4. Each of the 16 individual detonations resulted in a slightly delayed peak in $r_u$, with the last peak in $r_u$ ending at approximately 9.5 s. The shallowest PPT responded slower to the blasting as a result of the charges being detonated in the deepest decks first. All of the PPTs demonstrated a contractive soil response with increases in $r_u$ following each blast, indicating that soil in the control zone consisted loose to medium dense, liquefiable sand. The PPTs at elevations of 5.25, 6.35, 7.39, and 8.60 m reached peak $r_u$ values of 105, 147, 133, and 148%, respectively, for BE4. The
two deepest PPTs sustained complete liquefaction with peak residual $r_u$ ranging from 95 to 105%, whereas the two shallow PPTs showed $r_u$ ranging from 75 to 85%. Sand boils were not observed in the control zone after blasting, likely because of the stiffness and relatively impermeability of the fill overlying the liquefiable layer.

Postliquefaction Settlements at the Control Zone

Optical level measurements of the ground surface were conducted to observe the settlement resulting from postliquefaction consolidation. A baseline survey was performed prior to blasting at 29 individual points, distributed along three lines as shown in Fig. 5. Each line was spaced 60 degrees apart with the survey points spaced at 1.52 m intervals from the center of the control zone. Protocols were set to ensure that the points could be re-established following each blast event. Ground surface elevations were surveyed approximately 3 and 20 h after BE3; after 3 h, $r_u$ ranged from 2% in the shallow PPTs to 4% in the deeper PPTs. The elevations were surveyed again the following morning to determine if further settlement occurred, and on average, approximately 8 mm of additional settlement occurred between the 3 h and 20 h settlement surveys. Fig. 5 presents the ground surface settlements for the three survey lines 20 h after BE3, and indicates that the maximum settlement, equal to approximately 160 mm, occurred in the center of the control zone and decreased with increasing distance from the center of the control zone.

Another survey was performed along the A, B, and C lines 24 h after BE4. The ground surface settlements measured along these lines indicate the differences in settlement between events with significantly different shaking durations. The settlements observed following BE4 were approximately 25 mm larger, on average, than those measured from BE3, and a maximum settlement of approximately 200 mm was observed near the center. An additional survey was performed 48 h following BE4, but little to no additional settlement occurred. The cumulative maximum settlement for both

Fig. 4. Characterization, instrumentation, and blast performance at the unimproved control zone: (a) excess pore pressure generation and dissipation time histories for Blast Event 3; (b) excess pore pressure generation and dissipation time histories for Blast Event 4; (c) in situ tests and PPT locations.
Blasting events equaled approximately 350 mm (14 in.) in the center of the control zone.

**Improved Ground: Blast Program and Excess Pore Pressures**

The same charge weight and blasting sequence previously described for BE4 in the control zone was applied to the treated zones in order to make one-to-one comparisons of $r_u$ and settlement. This event is designated Blast Event 5 (i.e., BE5). Fig. 2 shows the location of the 18 blast casings with four decks per casing, set at the same elevations as those for the control zone. Each of the 72 explosive charges contained an equivalent of 0.91 kg of TNT resulting in a total charge weight of 65.5 kg. Owing to the presumed difficulty in interpreting the pore pressure and deformation response for Zones 5A and 5B, and due to the poor control of pile head spacing for the 2D piles detailed by Stuedlein et al. (2016), Zone 5 was not considered during the blast program.

Fig. 6 compares the generation of excess pore pressures in the treated zones at the nominal target PPT depths of 4.6, 6.1, 7.6, and 9.1 m (variations in depths developed during installation). In general, contractive pore pressures resulted from detonation of the explosive charges in the beginning of the blast sequence. However, continued detonation of charges result in the change in response of the improved ground. For example, the comparison of $r_u$ response for the shallowest PPTs in the control zone and Zone 3 is illustrated in Fig. 7. The seventh charge in the treated Zone 3 caused a transition from the contractive, positive pore pressure response-to-detonation to a dilative response, where pore pressures reduced in response to subsequent detonations. In between blasts and after approximately 3.7 s, $r_u$ equaled approximately 75%; the $r_u$ dropped a consistent (and absolute) 15 to 20% in response to subsequent blasts. In terms of cyclic stress paths, such a response indicates that the stress state has effectively crossed the phase transformation line and dilated (e.g., Ishihara et al. 1975; Zhang et al. 1997), and that significant soil strength had been mobilized in response to detonation of the charges.

In general, the excess pore pressure response was relatively similar between the various arrangements of displacement pile-improved ground. The drained piles, installed to investigate the improvement in densification due to draining driving–induced contractive pressures, not cyclic stress–induced pore pressures, did not perform noticeably better than the conventional piles. The shallowest PPTs exhibited the strongest dilative responses, owing to the development of the larger relative densities at these depths following pile installation (Fig. 3). In some instances (e.g., at a depth of ~7.5 m in Zones 3 and 4), the response changed from contractive to dilative repeatedly. Table 1 summarizes the peak residual $r_u$ observed at 12 s for all of the PPTs for ease of comparison. Peak residual $r_u$ in the treated zones were up to 22% (absolute) smaller than those measured in the control zone at similar depths. In general, the reductions in $r_u$ were approximately 2 to 10% (absolute) lower for piles spaced at 3D (i.e., Zones 2 and 4) than those at 5D (i.e., Zones 1 and 3). Although peak residual $r_u$ appeared to be high, liquefaction, as commonly defined (with residual $r_u$ > 95 to 100%) did not occur.

Excess pore pressures could have been elevated in the treated zone as compared to the control zone because of the overlapping energies associated with four shared blast rings. Therefore, the blast energy experienced by the treated zones was probably larger than that experienced in the control zone, and therefore the measurements of pore pressure in the treated zone could have been higher than if a single treated zone was blasted with a single blast ring. However, the magnitude of residual pore pressure is of less concern than the consequences; these are described subsequently.

**Postblasting Settlements of Improved Ground and Piles**

A ground surface survey set using a square grid spaced at 1.52 m was conducted prior to and following blasting of the treated zones. Pile head elevations for piles that were tipped, as well as those that
were not tipped into the dense bearing layer (approximately 12.5–13 m below grade) were surveyed so as to understand the differences in settlement between the two toe bearing conditions. Following blasting, the settlement of the soil in the treated zones ranged from a minimum of about 15 mm to a maximum of 95 mm as shown in Fig. 8. The majority of observed settlements ranged between one-sixth to one-third of those observed at the control zone. Greater settlements were observed at the northern and southern ends of the test area, whereas smaller settlements were observed in the middle (Fig. 8). The variation in settlements correlates to the spatial distribution of the magnitude of silty fines throughout the test site, measured using split-spoon samples and estimated using kriging of fines content with calibrated variograms (Bong and Stuedlein 2017). Because the compressibility of silty sands increases with fines content (Bandini and Sathiskumar 2009), greater settlements are expected where the concentration of silty fines are larger.

The settlement response confirms the experimental observations by Lee and Albaisa (1974) and corresponding analytical model proposed by Seed et al. (1975; Fig. 1). Soils with relative density in the range of 70 to 80% such as those densified using driven displacement piles experience smaller increase in compressibility as a result of shaking-induced pore pressures, and therefore smaller postshaking reconsolidation settlement. Therefore, designers of densification-based ground improvement could allow larger peak magnitudes of $r_u$ than those associated with ground improvement methodologies that do not result in significant densification (e.g., deep soil mixing, jet grouting).

Optical level surveys showed that piles that were not tipped into the dense sand layer exhibited similar settlements as those of the surrounding soils. The magnitudes of pile head settlements for representative floating piles ranged from about 55 to 95 mm (Table 2). Adjacent piles that were embedded in dense bearing layer exhibited much lower settlements, ranging from about 0 to 45 mm. Pile compression following dissipation of excess pore pressure occurs as the downward soil movement relative to the pile shaft (i.e., downdrag) transfers load to the pile (i.e., dragload), and the movement that occurs is commensurate with the magnitude of pile toe resistance to downward movement (Fellenius and Siegel 2008; Wang and Brandenberg 2013). Some piles appeared to heave

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**Fig. 6.** Excess pore pressure generation and dissipation time histories for the improved ground at (a) Zone 1, 5DPVD; (b) Zone 2, 3DPVD; (c) Zone 3, 5D; (d) Zone 4, 3D
The effects of blasting on excess pore pressures and deformations have been quantified in the previous discussion, but have not been correlated to the ground motions triggered. On the free surface following detonation, a wave train consisting of a P wave, an S wave, and a Rayleigh wave are produced and spread out radially from the epicenter. Nearest to the epicenter, the body wave amplitudes are greatest and most distinguishable, but decay with distance so that the ground motions become dominated by Rayleigh waves. The displacement amplitude of body waves decay with the inverse of the radial distance squared, whereas surface waves decay in correspondence to the inverse of the square root of radial distance (Richart et al. 1970). Triaxial geophones used to observe the ground motions during blasting were positioned at epicentral distances ranging from 24 to 120 m from the center of the blasted areas, and allowed observation of longitudinal (P wave dominant), transverse (S wave dominant) and vertical motions. Comparison of the attenuation of displacement amplitudes determined by integrating velocity time histories to the epicentral distances from the center of the blast locations showed that the body wave–dominated ground motions transitioned to surface wave–dominated ground motions between approximately 25 and 38 m, similar to that reported by Dowding and Duplaine (2004). The measured near-field (e.g., 25 m) particle motions confirmed that the blast-induced ground motions were dominated by body waves, and therefore could be used to scale the measured velocity time histories to estimate those at the center of each blast area.

Scaling of ground motions is possible upon determination of the site-specific attenuation characteristics. Fig. 9 shows the attenuation of the peak vector sum of particle velocity (PPV) for the three orthogonal velocity components (vertical, longitudinal, and transverse) with scaled distance for BE4 and BE5. Scaled distance is defined as the ratio of hypocentral distance between the points of detonation and observation and the square root of the mass of the explosive charge (Wiss 1981). The peak particle velocities were consistently generated by the fourth blast, which consisted of an equivalent of 1.82 kg of TNT at a depth of 8.8 m. The attenuation characteristics followed both the general form suggested by Wiss (1981) as well as that reported by Gohl et al. (2001) for liquefaction experiments in alluvial silty sands. No significant differences between the attenuation characteristics of PPV and individual motion components were noted. Considering the shared attenuation characteristics, the near-field ground surface motions can be estimated reliably using measured near-field velocity time histories and scaled in accordance with the site-specific attenuation relationship. For example, the PPV estimated above the treated zone for BE5 using the site-specific attenuation curve in Fig. 9 is 0.44 m/s, representing a scale factor of approximately 4 for the nearest measured velocity time history, observed at an epicentral distance of 24 m and

![Fig. 7. Example comparison of excess pore pressure response measured for the unimproved control zone to that of improved ground (the transition to a dilative response at approximately 3.7 s for the PPT in the treated Zone 3) (maximum heave of 25 mm), but this response likely represents an outcome of the blast program, and would not occur during an earthquake if supporting embankment, bridge abutment, or structural loads.](image)

![Table 1. Comparison of Peak Residual $r_u$ across the Control and Treated Zones (Corresponding to 12 s)](image)

<table>
<thead>
<tr>
<th>Nominal PPT depth (m)</th>
<th>Excess pore pressure ratio, $r_u$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control zone</td>
<td>Zone 1</td>
</tr>
<tr>
<td>4.6</td>
<td>73</td>
</tr>
<tr>
<td>6.1</td>
<td>82</td>
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<tr>
<td>7.6</td>
<td>93</td>
</tr>
<tr>
<td>9.1</td>
<td>104</td>
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</table>

*No PPT at this location.*

![Fig. 8. Postblasting ground surface settlement (mm) measured at the treated zones](image)
Table 2. Comparison of Pile Head Settlement of Adjacent Piles with and without Embedment into the Dense Sand Layer Following Dissipation of Excess Pore Pressure

<table>
<thead>
<tr>
<th>Pile #</th>
<th>Embedment (m)</th>
<th>Settlement (mm)</th>
<th>Comparable pile #</th>
<th>Pile #</th>
<th>Embedment (m)</th>
<th>Settlement (mm)</th>
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<td>10.0</td>
<td>73</td>
<td>4-4</td>
<td>12.9</td>
<td>12</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 9. Attenuation of peak particle velocity with scaled distance measured at the test site for BE4 and BE5, and comparison with those measured by Gohli et al. (2001) in alluvial silty sands

Assessment of the Cyclic Stress Reduction Design Approach

Shear Strain Compatible Design Approach

Baez (1995) proposed a design approach for use with stone column (or aggregate pier) reinforcement with or without the effect of densification, whereby a portion of the earthquake-induced cyclic shear stresses within potentially liquefiable soils could be assumed to be diverted to the stiffer reinforcing elements. This design approach required the assumption of shear strain compatibility (SSC) between the reinforcement and the surrounding soil, implying that the element would not exhibit flexure during strong ground motion. The designer estimates the cyclic stress ratio using the current formulation of the simplified method (Seed and Idriss 1971), then sizes the stone column spacing or the area replacement ratio, $A_p$, until the shear stress reduction factor is low enough to reduce the cyclic stress ratio, CSR, to an acceptable level (e.g., 50 to 60%; Baez 1995). In this approach the CSR of the unimproved ground, $CSR_u$, at each depth of interest is multiplied by the shear stress reduction factor, $K_G$, given by

$$ K_G = \frac{1}{G_r[A_p + \frac{1}{G_r}(1 - A_p)]} \tag{1} $$

where $G_r = G_{sc}/G_s$ is the shear modulus ratio; $G_{sc}$ = shear modulus of stone column; and $G_s$ = shear modulus of the soil during shaking (i.e., accounting for modulus reduction). Stone columns are generally characterized with $G_r$ between 2 and 7 (Baez and Martin 1993), as a function of the gradation of the stone column and the stiffness of the surrounding soil.

Since this design approach was developed in 1995, it has been applied to a variety of ground improvement techniques. For example, design guidance for transportation infrastructure in the Second Strategic Highway Research Program (SHRP2 2015) point to the use of Eq. (1) for much stiffer reinforcement elements such as deep soil mixing columns, jet grouted columns, drilled displacement piles, continuous flight auger piles, and other ground reinforcement techniques. However, questions regarding the applicability of the SSC assumption have developed as a result of numerical and analytical studies by Olgun and Martin (2008), Gueguin et al. (2013), and Rayamajhi et al. (2014), and centrifuge tests by Rayamajhi et al. (2015). These studies have suggested that flexure of reinforcing elements decreases the magnitude of cyclic shear stresses reduction; however, these findings have not been confirmed at full-scale. The results of the full-scale field trial described in this paper can be used to investigate the appropriateness of the SSC design approach for very stiff elements.
Assessment of Cyclic Stress Reduction and Shear Strain Compatibility

The simplified method for liquefaction triggering (Seed and Idriss 1971) can be rearranged in terms of shear strains, $\gamma_{\text{comp}}$, of any reinforced or composite mass under the assumption of SSC (Baez 1995)

$$\gamma_{\text{comp}} = \frac{0.65 \cdot a_{\text{max}} \cdot \sigma_{\text{vo}} \cdot r_d \cdot K_G}{g \cdot G_{\text{comp}}} \quad (2)$$

where $a_{\text{max}}$ = peak acceleration; $\sigma_{\text{vo}}$ = total vertical stress at a depth of interest; $r_d$ = shear stress reduction coefficient; $g$ = gravitation acceleration; and $G_{\text{comp}}$ = shear modulus of the composite mass. The depth-varying shear stress reduction factor was computed for Zones 1 and 2 using the assumed invariant shear modulus of the timber pile (870 MPa), the depth-varying maximum shear modulus of the improved soil [estimated using the shear wave velocity reported by Stuedlein et al. (2016)], $G_{\text{max}}$, and the depth-varying $A_v$. Owing to modulus reduction during cyclic straining, the shear modulus of the soil used to compute $K_G$ was assumed equal to one-third of $G_{\text{max}}$ at the moment of peak residual excess pore pressure (consistent with the measured excess pore pressures and modulus reduction described subsequently).

An estimate of $a_{\text{max}}$ is required to compute $\gamma_{\text{comp}}$ using Eq. (2) and to illustrate the range in shear strains possible under SSC. Fig. 10 indicates the peak transverse (S wave dominated) ground accelerations, $a_{\text{max},T}$, estimated from the measured site-specific attenuation characteristics and scaled measurements of velocity time histories. Accordingly, this intensity measure was used in Eq. (2) for purposes of comparison to the generated excess pore pressure–estimated shear strain curves described subsequently. Because this magnitude of $a_{\text{max}}$ is generally higher than those associated with typical seismically-induced strong ground motions, hypothetical shear strains are computed for the case of $a_{\text{max},EQ} = 0.40$ g to provide an appropriate context for applicability under earthquake loading (n.b., this assumes $M = 7.5$). Separately, a simplified approach to map measured $r_u$ to a first order estimate of shear strain is illustrated in Fig. S1 as described in Supplemental Appendix S1, and this approach is used to make a direct comparison of estimated in situ shear strains induced in the displacement pile-improved ground to those calculated under SSC. This approach relies on the modulus reduction curves for South Carolina soils by Zhang et al. (2005), the PPT measurements of peak residual $r_u$ presented previously, and interpolation of $r_u$ for locations in between the PPTs.

Fig. 11 presents the $a_{\text{max}}$-varying and depth-varying shear strains computed under SSC along with the range in $r_u$-shear strain curves reported by Dobry et al. (1982) for Monterey 0 sand with

![Fig. 10. Comparison of ground motion characteristics to generation of excess pore pressure for treated Zone 2: (a) comparison of normalized Arias intensity to excess pore pressure time history; (b) comparison of scaled longitudinal ground motion with normalized Arias intensity; (c) comparison of scaled transverse ground motion with normalized Arias intensity; insets show the first and last acceleration pulse with measured excess pore pressure](image-url)
isotropic consolidation pressures ranging from 25 to 100 kPa and $D_j$ ranging from 45 to 80% for comparison. Fig. 11 shows that the depth-varying shear strains estimated under SSC can vary over several orders of magnitude depending on the magnitude of shaking intensity and area replacement ratios considered. For example, the range in shear strains computed under SSC for Zone 1, with a pile head spacing of 5$D_j$ and for $a_{\text{max},T}$ is consistent with those inferred from the excess pore pressure measurements (Appendix S1). Conversely, the excess pore pressures associated with strains computed under SSC are significantly lower than those measured for piles spaced at 3$D_j$ (i.e., Zone 2, with $A_r = 8.7\%$ at the ground surface and 3.6% at the bearing layer). The range in $r_u$ and corresponding shear strain reported by Dobry et al. (1982) shows that the measured excess pore pressures cannot be associated with the magnitude of strains under SSC for Zone 2 with pile head spacing of 3$D_j$. Rather, the estimated shear strain associated with the measured $r_u$, also shown in Fig. 11, is consistent with the data reported by Dobry et al. (1982), NRC (1985), and blast-induced liquefaction experiments of unreinforced ground reported by Gohl et al. (2001). Conversely, if one considers the lower intensity of shaking more commonly associated with earthquake-induced ground motions (i.e., $a_{\text{max},EQ} = 0.40 g$ and $M = 7.5$), one would conclude that no excess pore pressures would have been generated in Zone 2.

The observations regarding SSC can be explained by the flexure that the timber piles likely exhibited during the blast-induced ground motions, which limits diversion of shear stresses to the stiffer pile element (Rayamajhi et al. 2014). Fresh cracking around the soil-pile interface was noted in the field following blasting, and this pointed to possible cyclic gaping between the soil and the pile during the blast-induced ground motions, which would have limited a composite or strain-compatible response. Based on numerical and analytical models (Olgun and Martin 2008; Gueguin et al. 2013; Rayamajhi et al. 2014), centrifuge model tests (Rayamajhi et al. 2015), and the field tests described in this paper, the use of the shear strain compatibility assumption and corresponding design approach for relatively stiff reinforcement liquefaction mitigation ground improvement may not be appropriate.

**Summary and Conclusions**

Full-scale, controlled blasting field tests on driven displacement pile-improved ground were conducted to study the response of densified and reinforced ground to blast-induced excess pore pressures. This paper described controlled blasting of unimproved ground using sufficient explosive charges to induce liquefaction and the resulting postliquefaction settlements measured to provide a baseline for comparison against the improved ground. Excess pore pressures generated in the improved ground were observed to be smaller than that in the unimproved ground, and resulted in settlements that were generally one-sixth to one-third of that measured in the unimproved ground. Piles tipped into a dense bearing layer settled significantly less than those in the surrounding soil. Importantly, measured excess pore pressures pointed to a change in response from contractive to dilative during execution of the cyclic detonation pattern, indicating that the improved ground mobilized significant strength during blasting, representative of soils densified by the installation of the displacement piling.

In order to help compare this work to and place it within the context of earthquake-induced ground motions, a site-specific attenuation relationship was generated from measured ground velocity time histories. The attenuation characteristics were similar to those reported for other liquefiable sites and suggested that ground motions could be confidently estimated above the center of the...
blasted areas. Peak ground accelerations thus estimated ranged from 1.1 to 1.8 g for transverse and longitudinal components, with mean periods of about 8 Hz, consistent with previously reported characteristics of blast-induced ground motions. The corresponding Arias intensity suggested that the blast program produced similar intensity as a moment magnitude 7.0 earthquake within a 10 km fault-rupture distance.

An assessment of the commonly-used shear strain compatibility assumption for use with reinforcement-based ground improvement methods was conducted by inferring shear strains from excess pore pressure measurements. Shear strains computed assuming a composite, compatible reinforced mass were inconsistent with those estimated in the improved ground, as well as inconsistent with the measured excess pore pressures for the case of pile head spacing at three diameters (with a range in area replacement ratio of 8.7–3.6%). Moreover, the estimated shear strain-excess pore pressure response of the improved soils at the test site compared favorably to the shear strain–excess pore pressure response of sandy soils reported in the literature. These comparisons, as well as those recently reported and based on numerical and centrifuge studies, suggest that the assumption of shear strain–compatible deformation for use with reinforcement-based liquefaction mitigation ground improvement design methodologies may not be appropriate.

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Supplemental Data

Fig. S1 and other material are available online in the ASCE Library (www.ascelibrary.org).

References


