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Ground vibration from blast-induced liquefaction testing in Christchurch, New Zealand

K. Kato[1], H. B. Mason[2], and S. A. Ashford[3]

ABSTRACT

Blast-induced liquefaction testing was conducted in Christchurch, New Zealand during 2013-2014 to investigate the effectiveness of soil reinforcement techniques for liquefaction mitigation. During the test series, ground vibrations were measured in both reinforced and natural soils. Explosives, buried at elevations from -2.4m to -9.7 m with charge weights of 0.55 kg to 2.8 kg, were detonated with a delay of 105 ms. This paper summarizes peak particle velocity (PPV) measurements from the blast-induced liquefaction testing with different reinforced soils and compares the PPV values to the other sites to rate the effectiveness of reinforcement on the ground vibrations. The results showed that a maximum PPV of 0.29 m/s was recorded in the natural soil and a maximum PPV of 0.26 m/s was recorded in reinforced soils. Comparisons of results from the Christchurch, New Zealand site and previous blast-induced liquefaction testing sites showed that the measured PPV values for all tests fell within the range of naturally deposited soils regardless of type of soil reinforcement technique employed, the charge properties, and the soil profiles.

Introduction

Controlling ground vibrations resulting from buried explosives is important for protecting engineered structures during blasting. Peak particle velocity (PPV) is commonly used as a threshold of ground velocity, because allowable extensional stress of engineering materials under vibration is estimated using the rate of displacement (e.g., Athanasopoulos and Pelekis 2000). Empirical relationships between charge weight and PPV have been proposed based on in-situ blast-induced liquefaction testing (e.g., Narin van Court and Mitchell 1994). However, the empirical relationships are developed using PPV values measured in the free-field, and accordingly, the empirical relationships are only valid for naturally deposited soils and not improved soil.

Blast-induced liquefaction testing was conducted in Christchurch, New Zealand during 2013-2014 to investigate the effectiveness of soil improvement techniques for liquefaction mitigation (Earthquake Commission 2014). The first and third authors participated in the field testing. Ground vibrations were measured in reinforced soils during the testing. This paper summarizes PPVs measured during testing at the Site 4 in Avondale, Christchurch. Site 4 contained different ground improvement techniques. PPV results from Site 4 are compared to free-field sites to understand the effect of reinforcement on the ground vibrations.

[1]Ph. D Candidate, Oregon State Univ, Civil and Constr. Eng., Corvallis, OR, USA, katoke@onid.oregonstate.edu
[2]Assistant Professor, Oregon State Univ, Civil and Constr. Eng., Corvallis, OR, USA, ben.mason@oregonstate.edu
[3]Dean, Oregon State Univ, College of Eng., Corvallis, OR USA, scott.ashford@oregonstate.edu

Experimental setup

The ground improvement program was designed to strengthen residential land in Canterbury, New Zealand, and controlled blasting test was performed to simulate earthquake shaking (Earthquake Commission 2014). Examples of the blast test layout are shown in Figure 1. For the pre-production phase (Blast Locations No. 1-3), the blast casings were placed along a 4.8 m diameter circle with 1.1 kg and 2.7 kg charge weights at elevations of -2.6 m and -7.0 m, respectively. For the production phase (Blast Locations No. 5 and 7), the blast casings were placed along 4.8 m and 7.5 m diameter circles with 0.55 kg, 2.4 kg, and 2.8 kg charge weights at elevations of -2.4 m, -6.0 m, and -9.7 m, respectively. All explosives were ignited with 105 ms time delays. The explosives at lower elevations were detonated first and, in sequence, the explosives at the middle and upper elevations were ignited. Concrete blocks, with approximately 1 m^3 volume, were placed on the center of each circle to apply stress on the reinforced soils. Three-dimensional geophones were installed at elevations between 0 m to -1.0 m in the center of the circles to measure ground vibrations. Soil reinforcement techniques were deployed inside of the circles to investigate the effect of reinforcement on ground settlement and pore water pressure response. At Site 4, the employed soil reinforcement techniques were: rammed aggregate piers (RAP), driven timber piles (DTP), continuous flight auger piles (CFA), low mobility grout (LMG), resin injection (RES), gravel raft (GR), soil cement raft (SCR), horizontal beam double row (HBD), and rapid impact compaction (RIC). The RAPs were constructed to final elevations of -2.5 m, -1.5 m, and -0.5m. The DTPs were driven to an elevation of -2.5 m. Grouting (LMG and RES) was injected at an elevation of -2.5 m. The rafts (GR and SCR) were constructed at an elevation of -0.8 m. The HDB was embedded to elevations of 0 m to -0.3 m. More details about the experimental setup are given in the Earthquake Commission (2014) report.

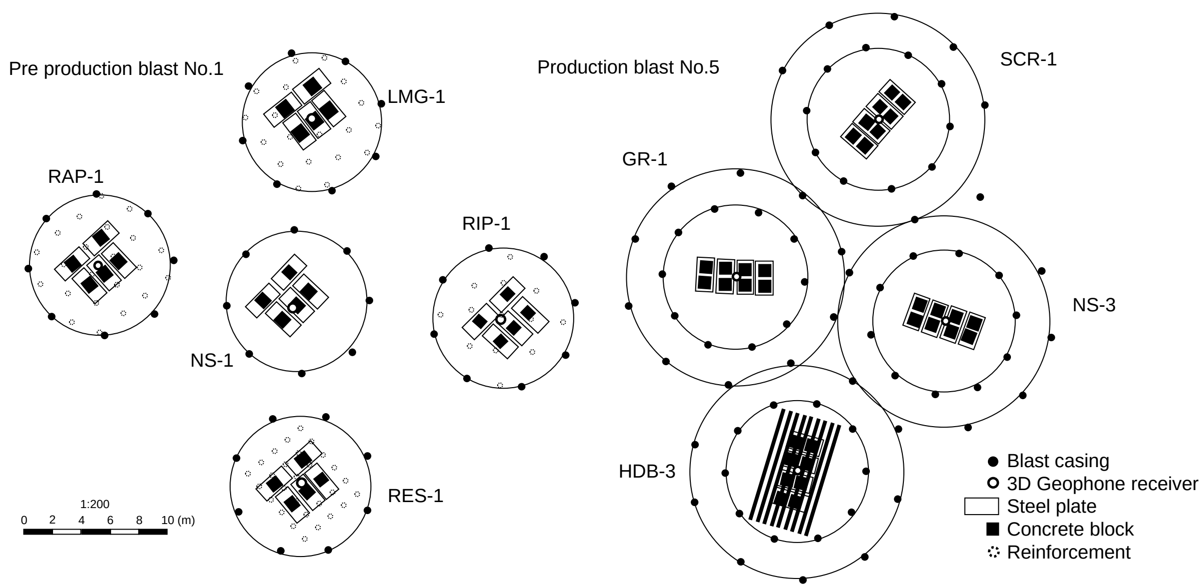

Figure 1. Example of the blast test layout employed at the site 4 in Avondale (adapted from Earthquake Commission 2014)

Soil profiles at Site 4

At Site 4, the soil is generally characterized by medium gravels with low plasticity organics (ground surface to an elevation of 0.3 to 0.5 m), followed by loose silty sand (to elevations between 0.0 and -1.0 m), followed by fine to medium sands and some silty sands (to an elevation of approximately -4.5 m). Cone penetration testing (CPT) tip resistance, P-wave velocities, and S-wave velocities for natural and reinforced soils are shown in Figures 2 and 3. The CPT tip resistances were measured 28 days after ground improvement was performed. The P-wave and S-wave velocities were taken from crosshole testing measured using the earthquake simulator “T-rex” (Earthquake Commission 2014, Stokoe et al. 2014). The ground water table (GWT) was located at an elevation of 0.3 m during the testing.

During testing, additional P-wave and S-wave testing was performed on the reinforced and natural soils. The following nomenclature is used in Figures 2 through 4: NS-1, RPA-1, LMG-1, and RES-1 were obtained at Blast Location No. 1; NS-2, RPA-3, HDB-1, and HDB-2 were obtained at Blast Location No. 2; RAP-2, CFA-1, DTP-1, and DTP-2 were obtained at Blast Location No. 3; and NS-3, GR-1, SCR-1, and HDB-3 were obtained at Blast Locations No. 5 and No. 7. Note that NS are the natural soil tests. Figure 3 shows that the P-wave velocities varied with reinforcement type near ground subsurface and reached approximately 1,500 m/s below an elevation of -2.0 m in all cases. Figure 4 shows that measured S-wave velocities ranged from 100 to 200 m/s at elevations between 1.5 and -4.5 m. Furthermore, S-wave velocities slightly increased with depth except HDB. The P-wave and S-wave velocities clearly indicate that the soil properties for the HDB tests were significantly different from the others. Notably, the HDBs were installed in horizontal directions to reduce ground settlement and accordingly disturbed the surrounding soils during construction.

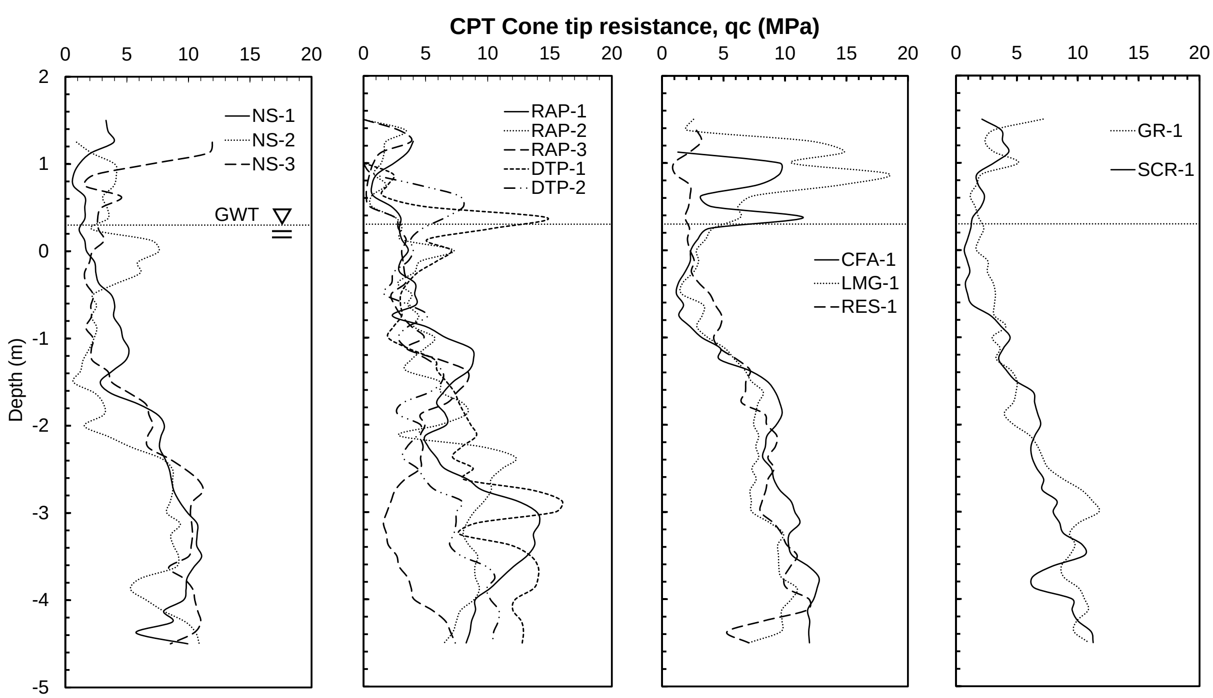

Figure 2. CPT tip resistance of post improvement soils at the site 4 in Avondale (adapted from

Earthquake Commission 2014)

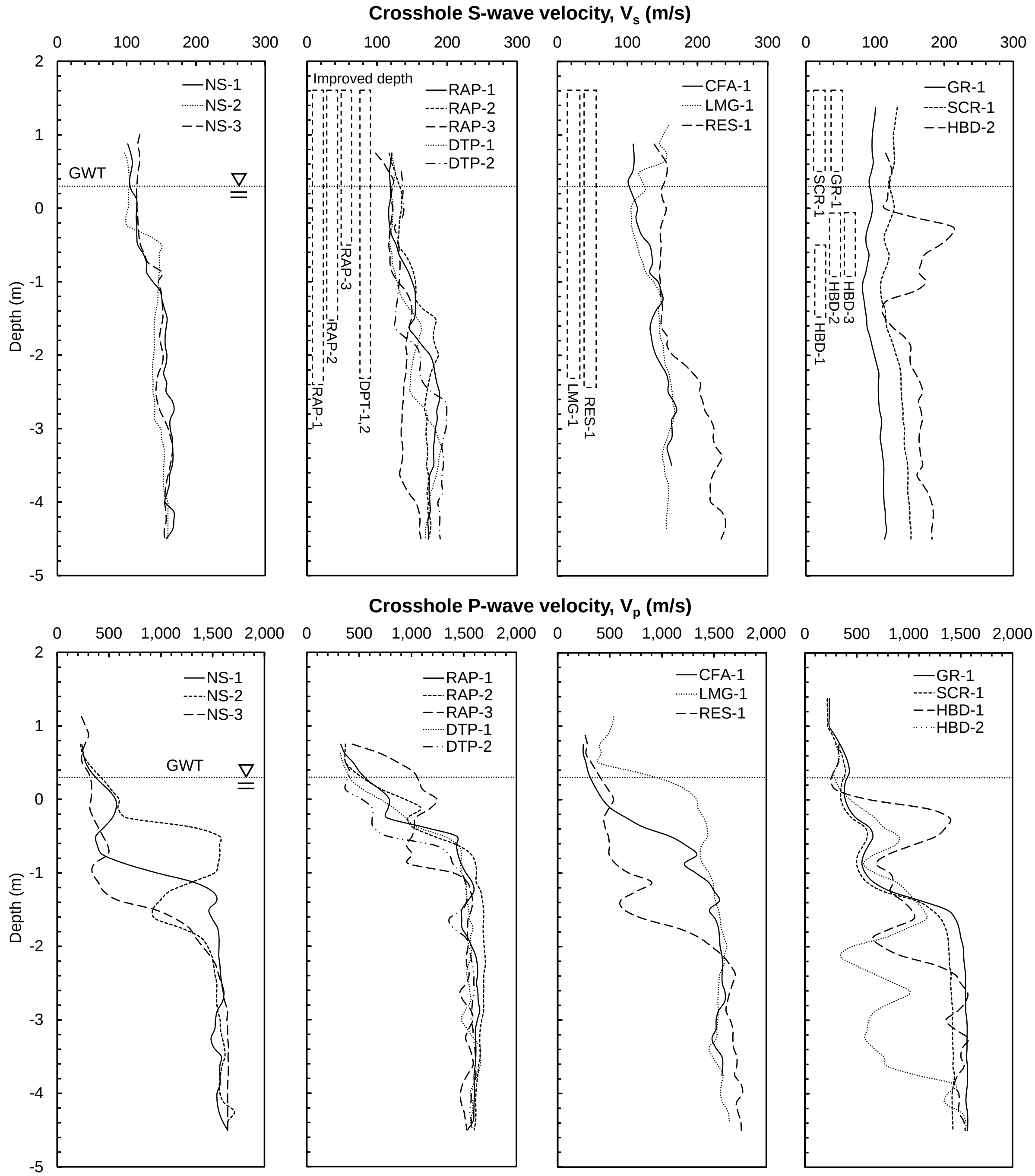

Figure 3. P-wave and S-wave velocity of post improvement soils at site 4 in Avondale (adapted from Earthquake Commission 2014)

Peak particle velocity

For blast testing, the explosive energy is usually correlated with the cube root or square root of the charge weight (Narin van Court and Mitchell 1994). For testing with multiple blasts, the square root of charge weight is commonly used for the correlation, because the cylindrical charge is often employed, and the contribution of explosive energy of multiple blasts is larger than for a single blast. Therefore, for the blast testing at Site 4, the square root of charge weight is used to evaluate the relationship between PPV and charge weight. The particle velocity versus time for both natural and reinforced soils is reported in the Earthquake Commission (2014) report. The PPV of natural and reinforced soils is replotted in Figure 4 using normalized distance. The particle velocity was measured in the vertical, longitudinal, and transverse directions. Within this paper, PPV values were measured in vertical direction. The maximum PPV was approximately 0.26 m/s in the reinforced soil, and the maximum PPV was 0.29 m/s in the natural soils. Regression analysis provided the best fit mathematical model described by $PPV = C\left(R/\sqrt{W}\right)^{-n}$ for all cases, where the coefficient C and the attenuation coefficient n are shown in Figure 4, R is the radial distance from the explosives, and W is charge weight.

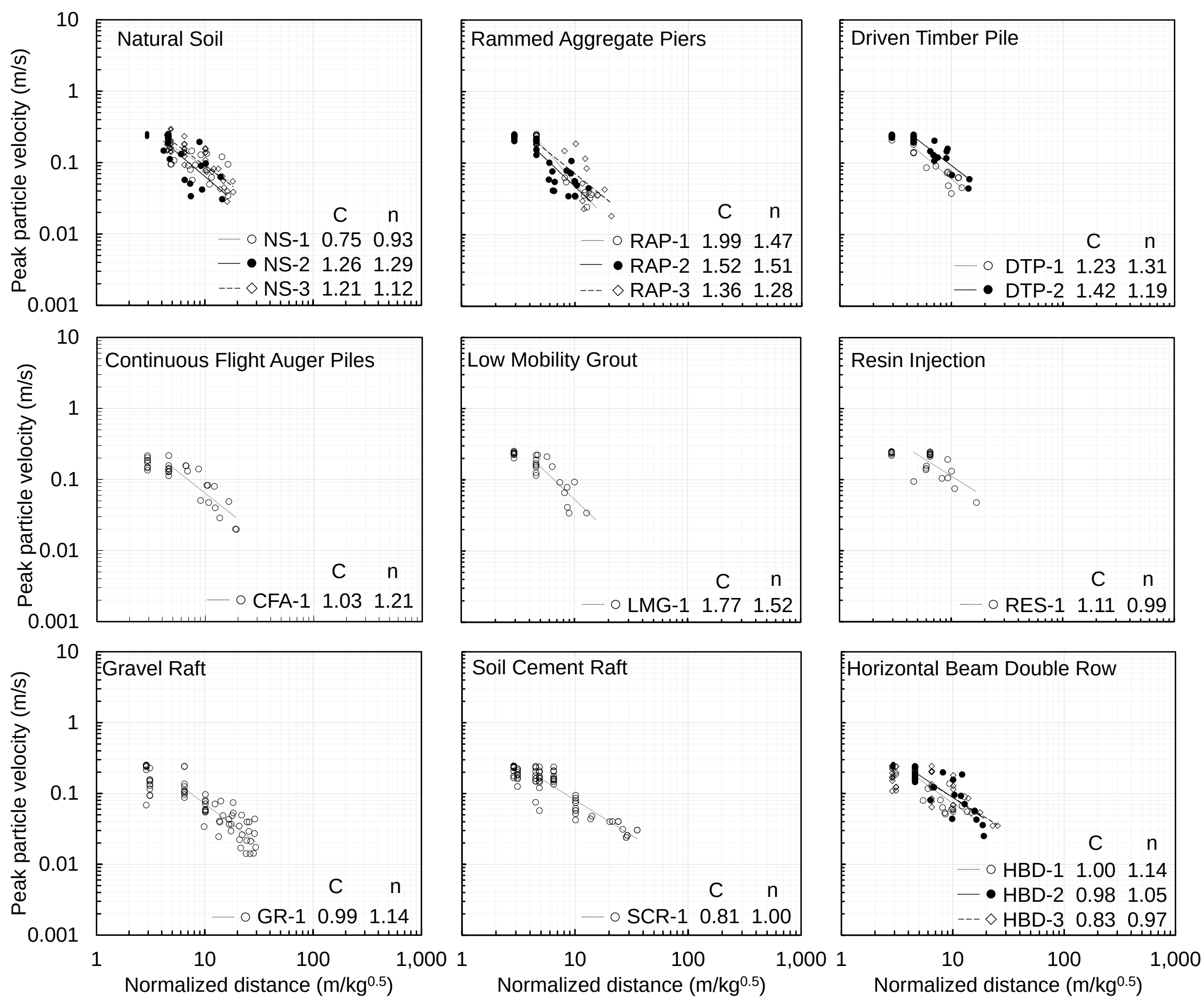

Figure 4. Peak particle velocity in each reinforced soil and natural soils

Comparison of PPV to other sites

Peak particle velocities from blast-induced liquefaction testing have been reported by others at various sites worldwide. For example, Elliot et al. (2009) reported PPV measured at the Seymour Falls Dam in Canada, which is a site characterized primarily by gravelly soils. Ashford et al. (2004) reported the PPV from blast-induced liquefaction testing at Treasure Island, California. Tsujino et al. (2007) and PARI (2003) reported PPV measured at the reclaimed Tokachi Port in Japan. Rollins (2004) and Rollins et al. (2004) documented blast-induced liquefaction testing at British Columbia and Hawaii, respectively. To compare the PPV values measured at Site 4, the PPV values at the aforementioned sites are plotted in Figure 5a. The mean and upper bound PPV regression lines – which have C coefficients of 1.47 and 3.21, respectively and both have n coefficients of 1.33 – are also plotted in Figure 5a. Figure 5b shows the data from Site 4. Clearly, the PPV values measured at Site 4 fall within the range of the past field tests regardless of soil improvement methods, blast properties, and soil profiles. The empirical model developed by Narin van Court and Mitchell (1994) is also plotted in Figure 5; however, the model does not fit well to the measured PPV values.

The detonation interval is shown in Figure 5 for most of the cases. Long et al. (1981) reported that a detonation interval of 100 ms could reduce ground vibrations, and that detonations intervals less than 60 ms were experimentally difficult to separate. Lafosse and Gelormino (1991) showed that a detonation interval of 125 ms could cause smaller ground vibrations than a detonation interval of 55 ms. The detonation interval used in the past field tests and the testing at Site 4 ranges from 105 ms to 500 ms. As shown in Figure 5, the measured PPV values fall within the range of the mean and upper bound regression equations regardless of differences between each case history. Therefore, the use of a detonation interval of 105 ms is justified.

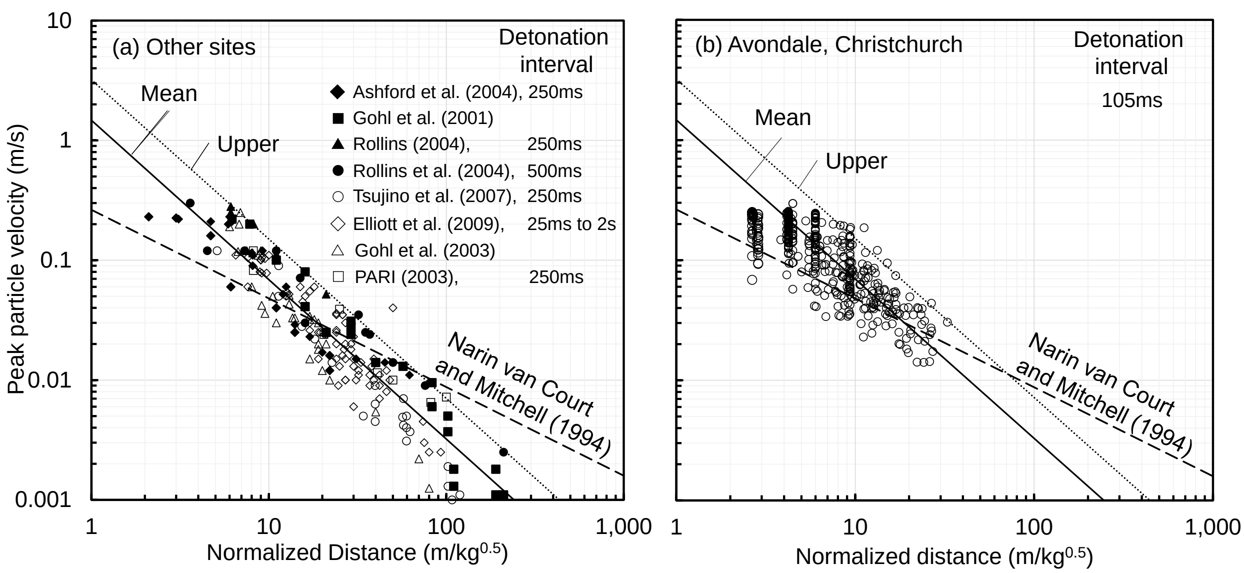

Figure 5. Comparison of peak particle velocity for (a) past blast-induced liquefaction testing, and (b) Site 4, The detonation interval is given along with the Mean and Upper Bound regression lines as well as the Narin van Court and Mitchell (1994) model.

Dowding and Duplaine (2004) indicated that blast-induced ground motions change with distance because body waves were attenuated and surface waves became more critical. Dowding and Duplaine (2004) showed that surface waves existed 60 m away from explosives. Tsujino et al.

(2007) also showed that blast-induced ground motions changed at 100m away from the explosives, because the surface waves became explicit at that distance. All PPVs reported in this paper were measured within 25 m of the explosives; therefore, the effect of surface waves on the reported PPV values is likely negligible. In addition, Dowding and Duplaine (2004) showed that saturated clay exhibits higher PPV values than granular or rock materials. The soils at Site 4, as well as the soils at the other test sites, are largely granular, which explains why the average PPV trend from the past field tests (Figure 5) matches well with PPV values measured at the Site 4.

Conclusions

Blast-induced liquefaction testing was conducted in Christchurch, New Zealand to investigate the effectiveness of soil improvement techniques for liquefaction mitigation. At Site 4 in Avondale, Christchurch, the ground and subsurface vibrations were measured in both reinforced and natural soils during detonation sequences. Herein, the peak particle velocity is plotted with normalized distance to correlate explosive energy contribution on ground motions. The following overarching conclusions are developed:

1) A maximum PPV of 0.29 m/s was measured in the natural soil during blasting test, and a maximum PPV of 0.26 m/s was measured in all the reinforced soils. Accordingly, ground improvement techniques reduced the peak particle velocity.

2) The measured PPV values in the Site 4 test series fell within the range of past field blast-induced liquefaction tests even though the soil profiles, reinforced methods, and blasting test properties were different. A reason for this observation is that all the test sites were characterized by granular soils.

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