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FIELD AND LABORATORY INVESTIGATIONS INTO THE BEHAVIOR OF SILTY SANDS THAT LEADS TO LIQUEFACTION TRIGGERING

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Abstract

Since the 1980s, the instrumented Wildlife Liquefaction Array (WLA) in Imperial Valley, California, has been the focus of ongoing seismic and soil-liquefaction research. In this paper, the results from five separate soil liquefaction studies are examined and the data are combined to develop a more comprehensive analysis regarding the behavior of liquefiable soils at the WLA site. The experiment data are derived from three types of field tests (CPT, crosshole seismic, and full-scale, in-situ shaking tests) and two types of laboratory tests (resonant column and cyclic triaxial). The work presented herein is part of an effort to confirm and advance in-situ liquefaction testing using large, hydraulically-operated shakers by examining the results of three previous experiments and comparing them with the results from laboratory testing and data-fitted models. The authors believe that the triggering of excess pore pressure generation leading to soil liquefaction is a strain-controlled phenomenon and present results in further support of this theory. Key findings specific to the liquefiable soil at the WLA site include: (1) field identification of the threshold strain for pore pressure generation (γ_t^{pp}), (2) field data for development of a pore pressure generation model, and (3) development of a modified hyperbolic shear modulus reduction relationship.

Keywords: soil liquefaction; shear strain; pore pressure generation; in-situ shaking tests; T-Rex

1. Introduction

1.1 Background of the Wildlife Liquefaction Array (WLA)

The area where the Wildlife Liquefaction Array (WLA) is located has represented a field research facility for seismic and soil liquefaction studies since the early 1980s [1, 2]. The WLA site is situated along the Alamo River approximately 13 km north of Brawley, California. One area of the site was originally instrumented by the United States Geological Survey (USGS) in 1982 [1, 2]. More recently, a second area was instrumented in 2004 by the National Science Foundation (NSF) as part of the Network for Earthquake Engineering Simulation (NEES) Program. Both areas are identified in Fig.1. In general, the instrumentation has included surface and borehole arrays of accelerometers and pore pressure transducers [1, 2].

The WLA site is located in Imperial Valley, California, on the southernmost terminus of the San Andreas Fault system. This location provides ideal research conditions for seismic and soil liquefaction studies because the area is subject to regular seismic activity as well as repeated soil liquefaction triggering. The most recent case of earthquake-induced soil liquefaction in Imperial Valley was observed on April 4, 2010, from the El Mayor-Cucapah earthquake (M7.2). In this event, soil liquefaction was triggered primarily in sandy river deposits and caused damage along roads, canals, and levees [3].

1.2 Geology and Soil Profile Associated with the WLA Site

The WLA site is located in the Salton Sink formation that includes Imperial Valley and the Salton Sea, an area that is approximately 136 km in length, with a maximum width of about 48 km. The ground surface of the Salton

Sink is approximately 70 m below sea level and is part of the same depression as the Gulf of California that lies to the south. Much of the sediment found in the Salton Sink has been continuously deposited over the last 20 million years in a process that began during the late Miocene epoch. As a result of the deposition, alluvial sands and silts as well as lacustrine silts and clays are abundantly found in this region [4].

As a part of the overall program of instrumenting the original site by the USGS in 1982, an extensive soil characterization study of the top 26.5 m was undertaken by Bennett et al. (1984) [1] using penetration testing and soil sampling. In the 26.5-m thick soil profile, the zone most likely to trigger liquefaction during an earthquake was identified in the top 7 m. The generalized soil profile in the upper 12 m is summarized in Table 1. The depth of the water table is relatively stable at approximately 1 m below the ground surface due to the existence of the relatively constant elevation of the water in the Alamo River. The liquefiable layer of interest in this profile is Layer #2. The other two layers are non-liquefiable. Layer #2 is, however, quite variable, with the soil type ranging from sandy silt to silty sand to fine sand.

1.3 Field and Laboratory Testing Associated with Soil-Liquefaction Research at the WLA Site

A large number of field and laboratory tests have been performed on the soils at the WLA site since the time that the initial instrumentation was installed in 1982. A full discussion of these results and conclusions are beyond the scope of this paper. The focus of this liquefaction study includes results from: (1) field testing that involved cone penetrometer testing (CPT), crosshole seismic, and in-situ shaking tests, and (2) laboratory testing that involved dynamic torsional resonant column (RC) testing of intact specimens and strain-controlled cyclic triaxial (CTx) testing of reconstituted and intact specimens. The data and results from these tests have previously been published in reports and dissertations but they have never been compiled into a single analysis as done herein.

Laboratory testing of the liquefiable soil from Layer #2 was performed in the 1980s using RC testing by Haag and Stokoe (1985) [5] at The University of Texas at Austin (UT Austin) and strain-controlled, cyclic triaxial testing by Vucetic and Dobry (1986) [6] at Rensselaer Polytechnic Institute (RPI). These laboratory results provide soil-specific behavior of the liquefiable soils regarding: (1) the reduction in shear modulus with increasing shear strain (RC testing) and (2) the coupled relationship between shear strain and the generation of excess pore pressure (strain-controlled, cyclic triaxial testing). Haag and Stokoe (1985) [5] performed six resonant column tests on intact sand specimens that were recovered from Layer #2. Vucetic and Dobry (1986) [6] performed 13 strain-controlled,

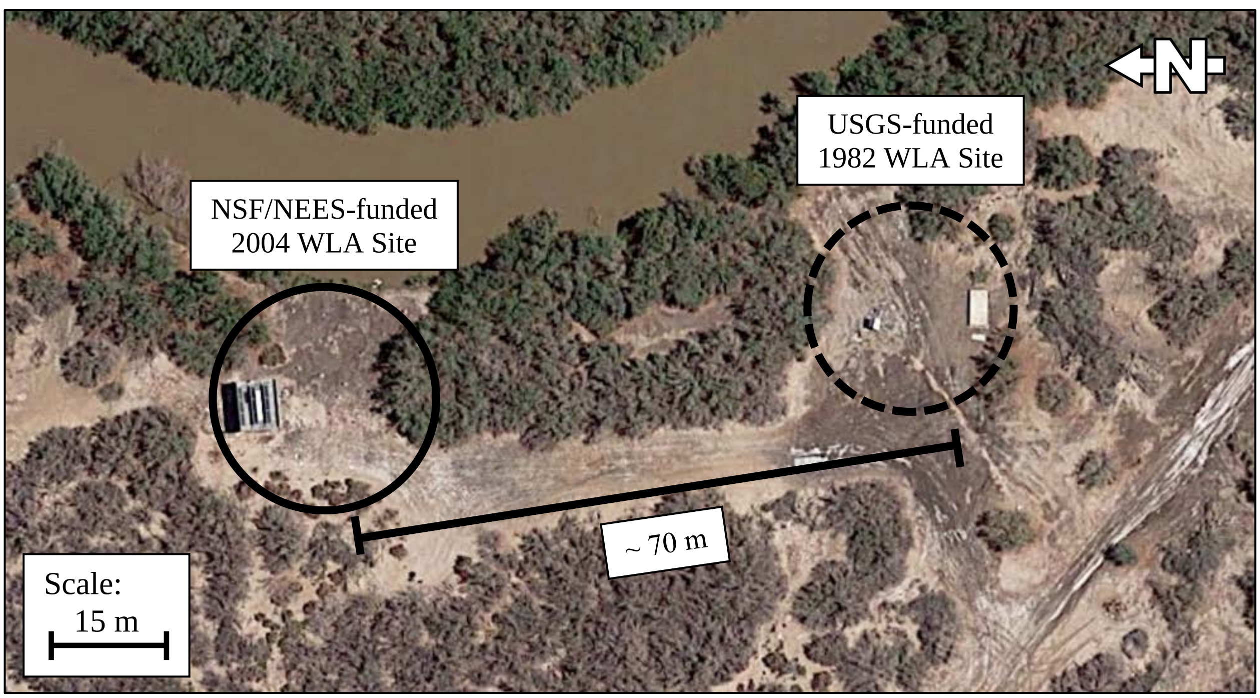


Fig.1 – Aerial view of the Wildlife Liquefaction Research Area showing the locations of instrumented arrays for both the 1982 site [1, 2] and the 2004 site [2]

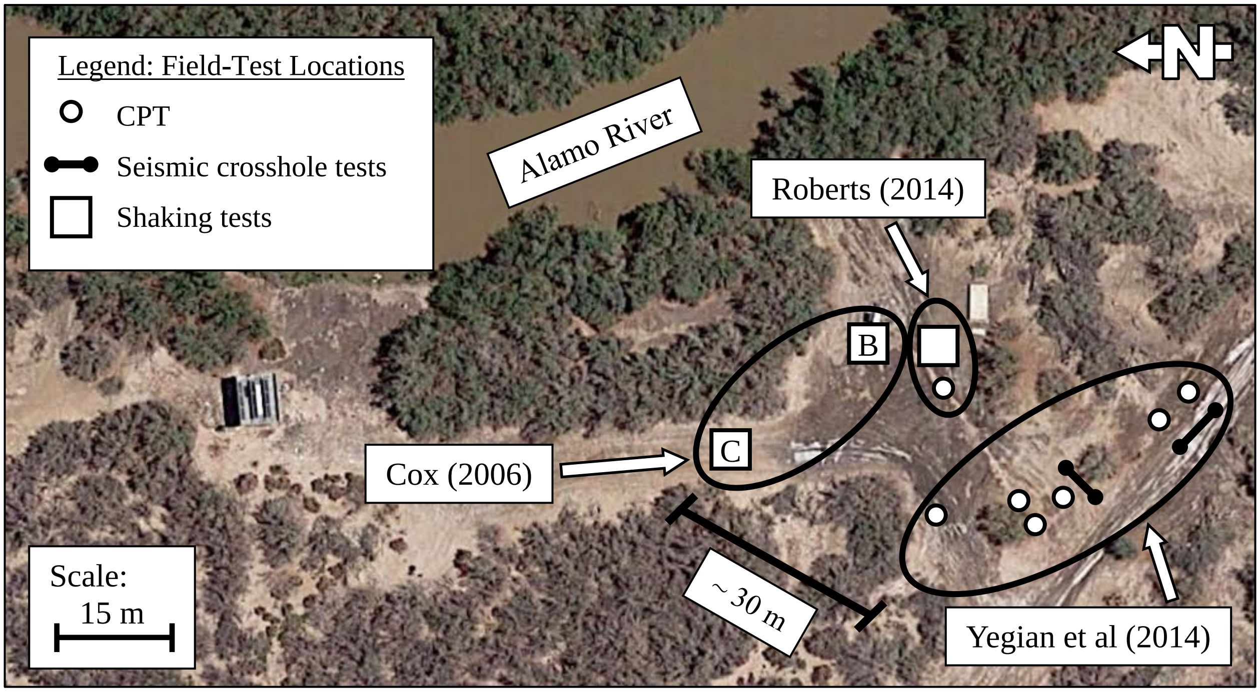


Fig.2 – Approximate locations of recent field tests including CPT, crosshole seismic, and field shaking tests performed by researchers from UT Austin for soil liquefaction research studies

Table 1 – Generalized soil layering at the WLA Site (modified from Bennet et al. (1984) [1])

Layer No.	Thickness	Soil Description
1	2.5 m	Sandy silt, silt, and clayey silt
2	4.3 m	Very loose to loose sandy silt and loose to medium-dense silty sand to very fine sand
3	5.2 m	Medium to stiff clayey silt and silty clay

cyclic triaxial tests on sand specimens retrieved from the liquefiable WLA layer, 11 of which were on reconstituted specimens. The results from three of the RC tests and the 13 CTx tests are used in this study.

More recently, the development of a direct, large-scale, in-situ shaking test method using a large vibroseis named T-Rex made it possible to observe the fundamental relationship between shear strain and the generation of excess pore pressure in the natural, in-situ conditions. This testing method was originally developed by Cox (2006) [7] and Cox et al.. (2009) [8] with initial tests performed at the WLA site in the mid-2000s. The results of two, in-situ shaking tests and two crosshole seismic tests from the original work of Cox (2006) [7] are included herein. The results from two follow-up NEES experiments are also included as follows: (1) a 2012 experiment with one CPT, one crosshole seismic, and one in-situ shaking test by Roberts (2014) [9] and (2) a 2014 experiment with six CPT and two crosshole seismic tests that were conducted for a project by Yegian et al. (2014) [10]. The location of each of these CPT, crosshole seismic, and in-situ shaking tests are shown in Fig.2.

2. Laboratory Testing of Intact and Reconstituted Specimens

In the 1980s, laboratory testing provided the only means of evaluating the nonlinear behavior of soil in a systematic and controlled manner. Dynamic RC testing and strain-controlled cyclic triaxial testing were performed during that time and provided the best insight into the moderate- to large-strain behavior of liquefiable sands at the WLA site. These insights are still relevant today and complement the large-scale, field shaking tests discussed herein.

Table 2 – Soil properties of intact specimens from Layer #2 that were testing using the RC method at the estimated, in-situ, mean effective stress (modified from Haag and Stokoe (1985) [5]

Specimen	Depth (m)	Fines Content (% < 75 μm)	Clay Content (% < 5 μm)	Liquid / Plastic Limit (%)	Soil Type (USCS)	Water Content (%)	Void Ratio, e	Degree of Saturation (%)	γ_{sat} (kN/m^3)	σ_o' (kPa)	G_{max} [1] (MPa)	V_S [1] (m/s)
W12A1	2.7	95	22	33 / 26	ML	34	0.92	99	18.4	27.3	14.2	86
W11A1	2.7	50	11	23 / NP	SM	24	0.81	78	18.9	27.3	30.8	127
W3A4	3.3	57	15	23 / 21	ML	29	0.8	97	19.0	30.8	26.7	118
W3B1[2]	4.5	12	2	22 / NP	SP-SM	12	0.74	44	19.3	37.9	41.2	146
W12B2[2]	4.5	23	10	23 / NP	SM	21	0.75	77	19.2	38.5	31.4	128
W3B3[2]	4.8	38	18	25 / NP	SM	28	0.84	90	18.7	40.1	27.3	119

Notes: 1. Values calculated at the estimated, in-situ mean effective stresses.
2. Only specimens used in this study.

2.1 Resonant Column Testing of Intact Specimens

The RC testing was performed on six intact specimens in the Soil Dynamics Laboratory at UT Austin by Haag and Stokoe (1985) [5]. The RC testing was used to investigate the dynamic properties of the sandy material over a shear strain range of 0.001 to 0.1 %. The RC testing method that was used involved a fixed-free configuration, with the top of the specimen free to move. The top of the specimen is excited in torsional motion and a dynamic response curve is determined from which shear wave velocity (V_S), shear modulus (G), and shear strain (γ) are determined. Material damping is also determined but is not a part of this discussion.

The six specimens that were recovered from Layer #2 come from depths ranging from 2.7 to 4.5 m. The three shallowest specimens (W12A1, W11A1, and W3A4) were excluded from this study because the fines contents are considerably higher than the specimens tested by Vucetic and Dobry (1986) [6]. In addition, two of these specimens contain plastic fines. Each of the other three specimens were tested nonlinearly at two or more of the following effective confining pressures (σ_o'): 27.5 kPa, 55 kPa, 110 kPa, and 220 kPa. These specimen were classified using the Unified Soil Classification System (USCS) [11]. The specimens classify as SP-SM, SM, and ML materials and have fines contents ranging from 12 to 38 %. Based on RC testing, V_S of these materials ranges from 119 to 146 m/s and the shear modulus at small strains, G_{max}, ranges from 27.3 to 41.2 MPa at the estimated in-situ, mean effective stress. Other soil properties of these three specimens are summarized in Table 2, including the water content, void ratio, degree of saturation, and saturated unit weight.

The normalized shear modulus reduction curve (G/G_{max}-log γ) that was developed from RC testing at σ_o' = 55 kPa can be modeled with a modified hyperbolic equation using a process described by Darendeli (2001) [12]. The equation for the modified hyperbolic relationship is defined by:

$$\frac{G}{G_{max}} = \frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^a} = \frac{1}{1+\left(\frac{\gamma}{0.07\ \%}\right)^{0.74}} \tag{1}$$

where "a" is a curvature coefficient, γ_r equals γ at G/G_{max} = 0.5, and γ values are expressed as %.

The important results from the RC testing include: (1) a fitted, modified hyperbolic model for the normalized shear modulus reduction curve (Eq. 1), (2) laboratory V_S values at the estimated, mean in-situ effective stress ranging from 119 to 146 m/s, and (3) laboratory G_{max} values at the estimated, mean effective in-situ stress ranging from 27.3 to 41.2 MPa which, when combined with Eq. 1, permit a range in G-log γ curves to be developed.

2.2 Strain-Controlled Cyclic Triaxial Testing of Intact and Reconstituted Specimens

A total of 13 strain-controlled cyclic triaxial tests were performed on both intact and reconstituted specimens by Vucetic and Dobry (1986) [6], although the majority of the specimens were reconstituted (11 of 13 specimens). The soil samples were retrieved from the liquefiable layer at depths ranging from 2.7 to 4.5 m. Sieve analyses of this sandy material from Layer #2 revealed a fines content (defined as percent passing the No. 200 sieve) ranging from approximately 24 to 37 %. Also, with the 13 specimens, it was determined that the fines content decreases with increasing depth.

The strain-controlled cyclic triaxial tests were performed at an isotropic consolidation stress of 96 kPa on fully saturated specimens. The cycling frequency was 0.2 Hz, and the total number of cycles ranged from 30 to 247. The cyclic axial strain amplitude, ε_{cy}, varied between 0.02 % and 1.35 % and the measured excess pore pressure ratio ranged from 2 to 97 %. The corresponding shear strain, γ, was estimated from the cyclic axial strain using the relationship $\gamma = 1.5\ \varepsilon_{cy}$; hence, Poisson's ratio, ν, = 0.5. From these test data, the Dobry model for predicting the excess pore pressure ratio, r_u, as functions of cyclic shear strain and number of cycles of loading was fitted using an interactive computer graphics program. The following equation for r_u under one-directional loading at σ_o' of approximately 1 atm was determined to be [6]:

$$r_u = \frac{1.04 \times N \times 2.6 \times (\gamma - 0.02)^{1.7}}{1 + N \times 2.6 \times (\gamma - 0.02)^{1.7}} \times 100\% \tag{2}$$

where r_u is the predicted excess pore pressure ratio, N is the number of loading cycles, and γ is the cyclic shear strain. This model for the WLA liquefiable soil in Layer #2 identifies the threshold strain at which excess pore pressure begins to be generated, γ_t, at 0.02 %. In Section 4, the writers use the symbol γ_t^{pp} to represent this threshold.

The important results of this work include: (1) identification of γ_t (also γ_t^{pp}) at 0.02 % for σ_o' ~ 1 atm, (2) a fitted model of pore pressure generation as a function of shear strain and number of cycles that is specific to the WLA liquefiable sands, and (3) a prediction that soil liquefaction will be triggered at γ ~ 1.0 % after 50 cycles of loading.

3. In-Situ Geotechnical Site Investigation Before Shaking Tests with T-Rex

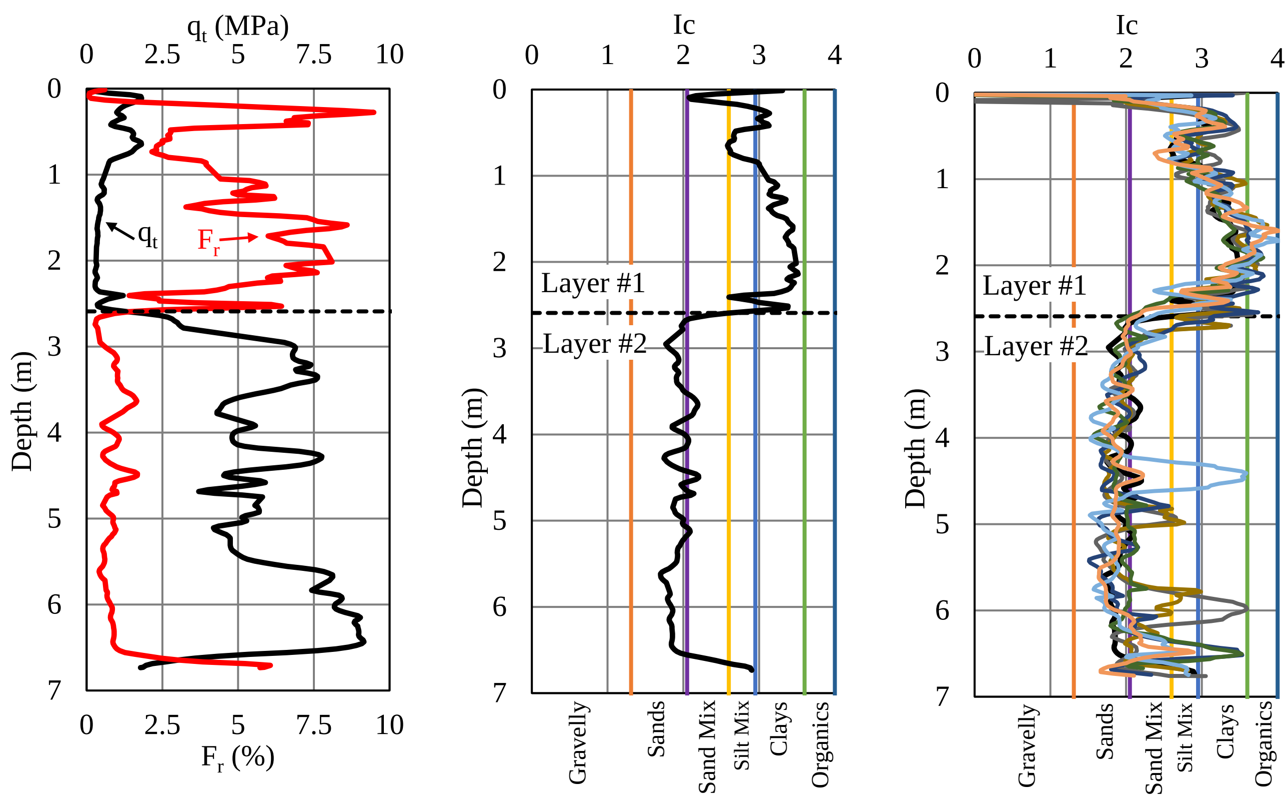
A geotechnical site characterization program before shaking tests with T-Rex was undertaken to investigate the static and dynamic properties of the sandy soil in the liquefiable layer at the WLA site. This site characterization program included seven CPT soundings and three crosshole seismic tests performed between 2004-2014 by researchers from UT Austin [7, 9, 10].

3.1 CPT Testing

One objective of the CPT soundings was to locate the depth of the interface between the non-liquefiable, top layer and the liquefiable layer, Layers #1 and #2, respectively, in Table 1. This layer interface is typically located at a depth of approximately 2.4 m but can vary depending on location around the area. Results from the seven CPT soundings performed prior to shaking tests with T-Rex show the layer interface to range from 2.3 to 2.7 m below the ground surface, with an average depth of 2.6 m. Although these depths are slightly greater than reported in the generalized soil profile (2.4-m depth) in Table 1 from the 1982 WLA site [1], this difference is not unexpected because the locations of the CPT, crosshole seismic, and in-situ shaking tests are slightly further from the Alamo River than many of the original borings and CPTs that were used to develop the generalized soil profile in 1982.

The CPT results from Roberts (2014) [9] are shown in Figs.3a and 3b as an example of typical results for the WLA site where field shaking tests with T-Rex were performed (Fig.2). Profiles for both the corrected total cone resistance (q_t) and the normalized friction ratio (F_r) are presented in Fig.3a. The soil behavior type index (I_c) [13] profile is presented in Fig.3b. The results in Fig.3c show the variability in the soil behavior type index with the one CPT from Roberts (2014) [9] and the six CPTs from Yegian et al. (2014) [10] that were performed by the UT Austin team. Despite the slight variation with depth, the interface between Layers #1 and #2 is easily identified by looking at the I_c profiles, which also indicates that Layer #2 should generally behave like a sand.

The important results from the CPT testing are: (1) identification of the interface between Layers #1 and #2 ranging from about 2.3 to 2.7 m, with an average depth of 2.6 m and (2) knowledge of the uniformity and soil behavior type (sand) for the soil deposit in Layer #2.



(a) Corrected total cone resistance (q_t) and normalized friction ratio (F_r) (Roberts (2014) [9])

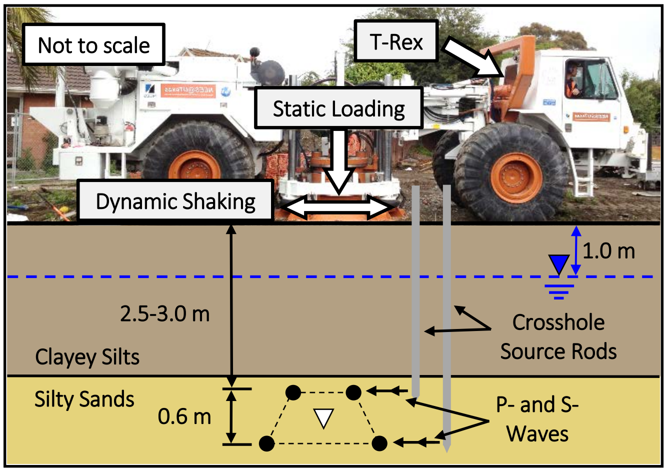
(b) Normalized Soil Behavior Types from One CPT Test (Roberts (2014) [9])

(c) Normalized Soil Behavior Types from Seven CPT Tests (Roberts (2014) [9] and Yegian et al. (2014) [10])

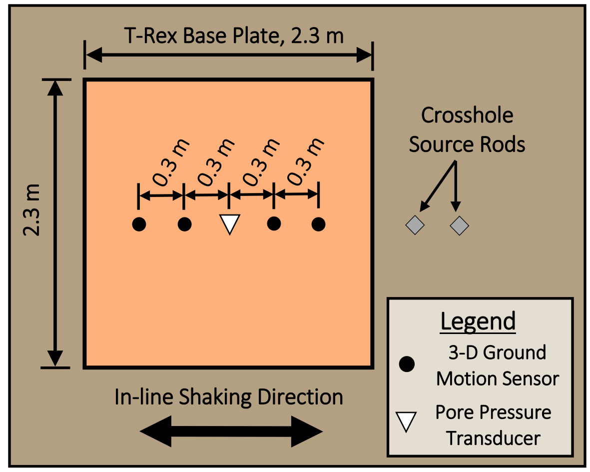
Fig.3 – Results and analysis of recent CPT testing at the WLA site

3.2 Crosshole Seismic Testing

Crosshole seismic testing was performed before all shaking tests by Cox (2006) [7] and Roberts (2014) [9] to determine constrained compression wave velocities (V_P) and shear wave velocities (V_S) and to estimate small-strain dynamic properties of the in-situ soil. The crosshole testing was not used to develop V_P and V_S profiles over the top 4.5 m but was targeted at the zone in the liquefiable layer where the shear strains and excess pore water pressures were measured. Using the sensor array that was installed for the shaking tests, two sets of crosshole tests were performed by Cox (2006) [7] (Cox 2006-B and Cox 2006-C) and one set of crosshole tests was performed by Roberts 2014 [9]. These tests provided V_P and V_S measurements at the top and bottom of the embedded sensor array (depths ranging from 2.5 to 4 m) and were performed by pushing two steel, source rods to the same depths as the already-installed ground motion sensors (accelerometers or geophones) used to monitor at-depth ground motions during shaking. In this manner, a linear crosshole array was created by each source rod and associated two ground-motion sensors as shown in Fig.4. The horizontal distance between the ground motion sensors was 0.6 m at the top of the array and 1.2 m at the bottom of the array. A hand-held hammer, with an attached accelerometer for triggering the waveform analyzer, was used to tap on the top of each steel source rod. The hammer tap created unconstrained compression waves traveling down the source rod. At the bottom of the rod, constrained compression waves (P waves) and shear waves (S waves) were then created, which radiated through the soil from the conical tip at the base of the rod. A waveform spectrum analyzer was used to record the interval travel times of the seismic waves propagating between the pairs of ground motion sensors. (See Fig.4a for location of crosshole source rods relative to the ground motion sensors.) The values of V_P and V_S determined from these crosshole tests are shown in Fig.5.



(a) Cross-sectional perspective of T-Rex on the ground surface and embedded array of sensors during shaking



(b) Plan view of the T-Rex baseplate, embedded array of sensors, and approximate locations of crosshole source rods

Fig.4 – Cross-sectional and plan view of T-Rex with embedded array of sensors for shaking tests and approximate locations of source rods used in crosshole seismic testing

Two additional sets of crosshole tests were performed by Yegian et al. (2014) [10]. In these tests, V_P and V_S profiles were determined over the depth range of 0.8 to 5 m, with measurements taken at 20-cm depth intervals. The crosshole testing was performed by pushing two steel rods, approximately 1 m apart, to the same depth. As in the crosshole testing by Cox (2006) [7] and Roberts (2014) [9], a hand-held hammer with an attached accelerometer for triggering was used to tap on the top of the steel source rod to propagate P and S waves through the soil. The steel receiver rod had a 3-D geophone cone sensor screwed onto the end of the rod to record the arrivals of P and S waves.

The V_P measurements were used to identify the depth to 100 % saturation and to verify that the sensors for shaking tests were located in 100 % or nearly 100 % saturated soils. At these test locations, the water table depth is approximately 1 m while the depth to 100 % saturation is approximately 2.8 m below the ground surface as noted in Fig.5a. Complete saturation was easily identified by high-frequency wave arrivals in the time records and a corresponding V_P ranging from about 1,450 to 1,700 m/s [14]. With regard to liquefaction triggering, the soil is considered to behave like a fully saturated sand when the values of V_P are equal to or greater than 750 m/s, which corresponds to a degree of saturation in liquefiable sands greater than 99.7 % [15]. The depth at which $S_r \geq 99.7$ % is about 2.2 m as indicated on Fig.5a.

Measurements of V_S were used to evaluate the shear stiffnesses of the in-situ soil skeleton prior to the shaking tests. In the case of the crosshole seismic tests performed by Cox (2006) [7] and Roberts (2014) [9], the mean, in-situ effective stress at the time of the measurements includes a depth-corrected stress contribution from T-Rex at the ground surface. The small-strain shear modulus, G_{max}, was then calculated from the mass density, ρ, and V_S ($G_{max} = \rho\ V_S^2$). A mass density of 2 kg/m^3 was assumed for the saturated sandy material in Layer #2. Based on crosshole seismic testing, the V_S of the soil subjected to shaking tests with the additional confining pressure from T-Rex (σ_o' = 34 to 39 kPa) ranges from 101 to 141 m/s. The corresponding values of G_{max} for the liquefiable material in Layer #2 under the confining stress of T-Rex range from 27 to 40 MPa. It should be noted that V_S values measured with T-Rex loading the ground surface are increased by about 3 to 6 % compared to the condition without T-Rex at the ground surface.

The important results from crosshole seismic testing are: (1) identification of the depth to 100 % saturation at 2.8 m, (2) calculated shear wave velocities under T-Rex range from 101 to 141 m/s in the zone in the liquefiable Layer #2 at mean, in-situ effective stresses ranging from 34 to 39 kPa, and (3) calculated G_{max} values determined for the V_S values under T-Rex ranging from 20 to 40 MPa.

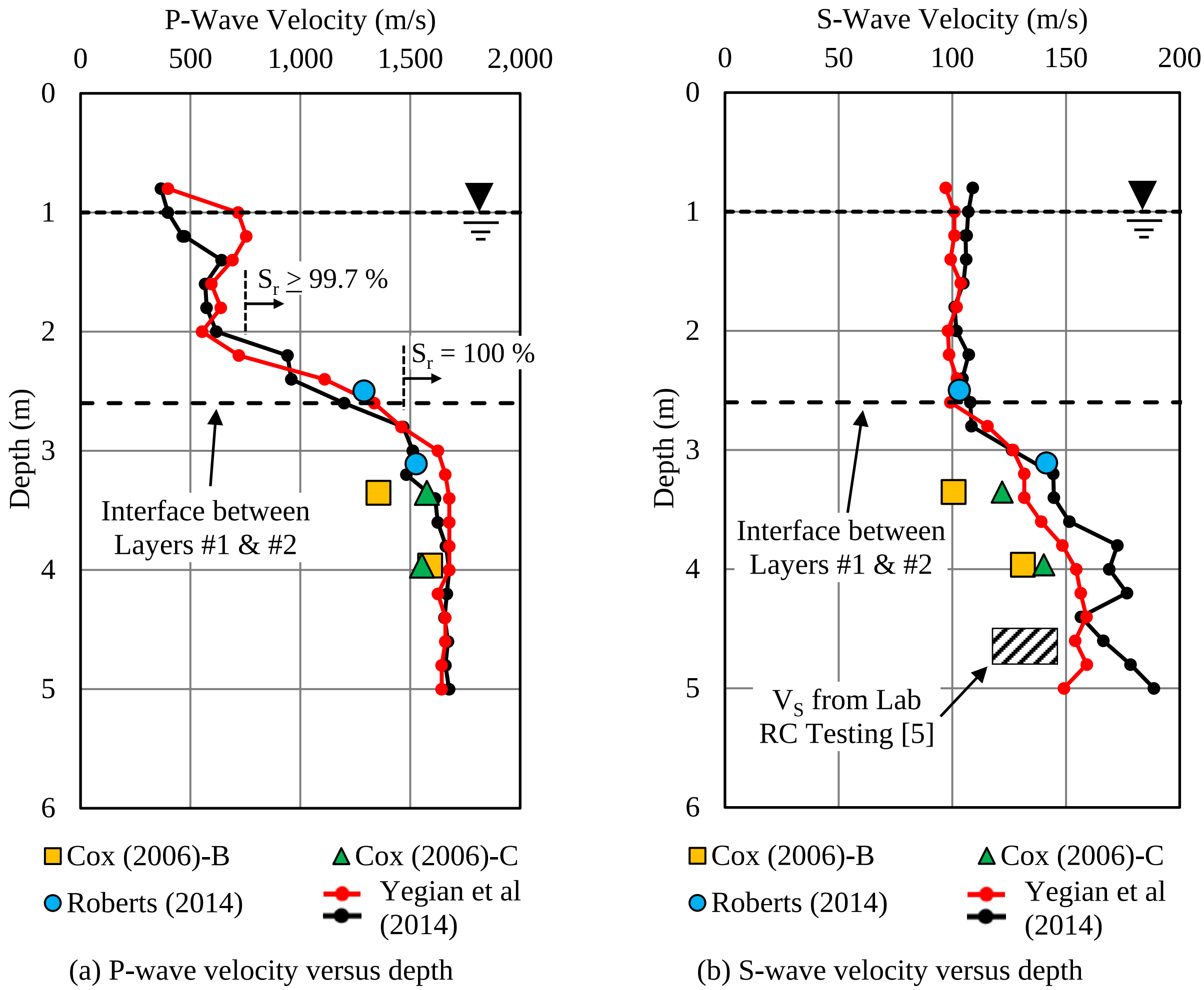


Fig.5 – P- and S-wave velocity profiles from Cox (2006) [7], Roberts (2014) [9], and Yegian et al. (2014) [10]

4. Shaking Tests with T-Rex

The coupled behavior between γ and r_u (r_u - log γ) over a large range of strains was evaluated by controlled, staged-loading, horizontal shaking tests using T-Rex at the ground surface and monitoring the soil response at shallow depths with an embedded array of ground-motion and pore-pressure transducers (PPTs) (see Fig.4a). Each shaking test included eight to nine stages of loading at increasingly larger force levels, with any excess pore pressure, r_u, allowed to dissipate fully between loading stages. Besides shaking, T-Rex was also used to install all embedded sensors using the pushing mechanism at the rear of the machine. The relative location of T-Rex and the embedded sensors during the shaking tests are shown in a cross-sectional view in Fig.4a and in a plan view in Fig.4b. The relative locations of the two crosshole source rods used in the pre-shaking crosshole testing discussed in Section 3 are also shown in Figs.4a and b.

In the shaking tests performed by Cox (2006) [7] at test panels Cox (2006)-B and Cox (2006)-C, the embedded array included four, 3-dimensional accelerometers and one PPT at depths ranging from 3.4 to 4.0 m and located entirely within the liquefiable layer. The four, 3-dimensional accelerometers were embedded in the shape of a 4-node trapezoidal array with the PPT located at the center of the array. The shear-strain time records at the center of the array where the PPT was located were estimated using a 4-node, displacement-based shear-strain calculation method presented in Cox et al. (2009) [8]. In these tests, the average peak γ ranged from 0.0009 to 0.0633 %, with measured r_u values up to 19 % after 50 cycles of loading.

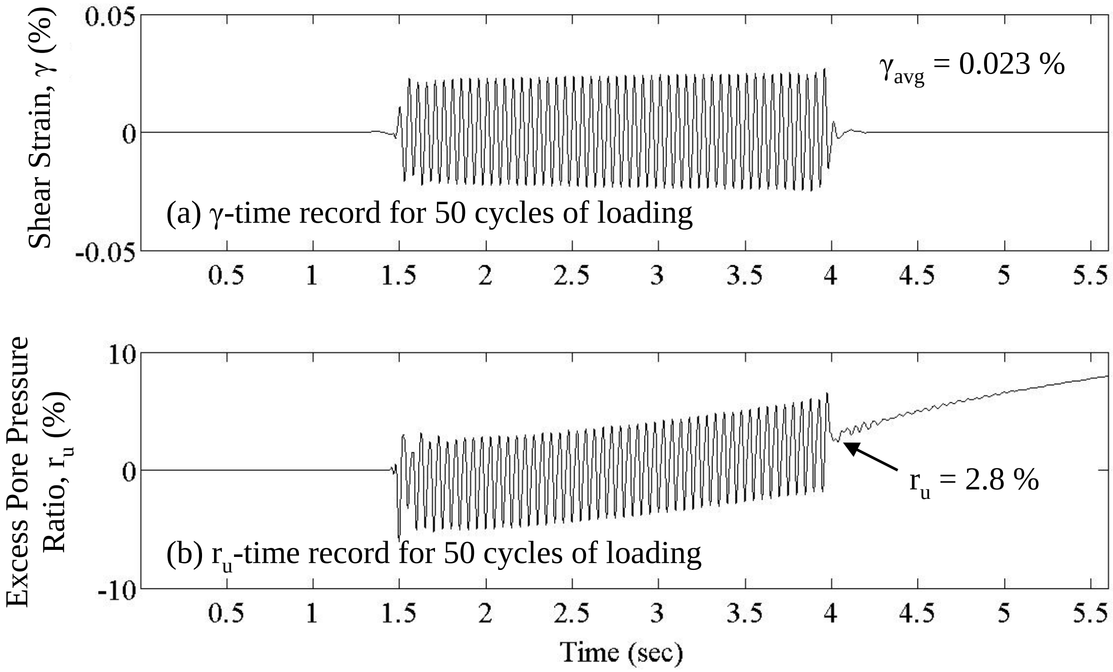


Fig.6 – Time records of γ and r_u from a moderate loading stage in the shaking tests of Roberts (2014) [9]; frequency = 10 Hz and N = 50

The shaking test performed by Roberts (2014) [9] used four, 3D velocity transducers and one PPT at depths ranging from 2.5 to 3.1 m. As in the testing configuration used by Cox (2006) [7], four velocity transducers were embedded in the shape of a 4-node trapezoidal array, with the PPT located at the center. In this case, however, the top, two, 3D velocity transducers were located in the bottom of Layer #1. The PPT and bottom, two, 3D velocity transducers were located in the liquefiable sand layer. This sensor arrangement was selected in order to observe the r_u - log γ relationship as close as possible to the interface between the non-liquefiable and liquefiable layers. Again, γ at all shaking levels was estimated at the center of the 4-node array where the PPT was located using the displacement-based, shear-strain calculation method [8]. The average peak γ values in the staged testing ranged from 0.0006 to 0.0707 %, with r_u values up to 13 % after 50 cycles of loading being measured. Example γ and r_u time records are shown in Figs.6a and 6b, respectively, for one level of staged loading with T-Rex for 50 cycles at a frequency of 10 Hz (from Roberts (2014) [9]). For this loading stage, the average γ is 0.023 % and r_u at the end of 50 cycles is 2.8 %. The combined results of the shaking tests from Cox (2006) [7] and Roberts (2014) [9] are presented in Fig.7, showing r_u versus the average γ after 50 cycles of loading. Also identified on Fig.7 is the range of γ_t^{pp} of 0.015 to 0.020 %, which was determined at an average σ_o' of about 36 kPa.

Important results from the staged, shaking tests include: (1) generation of shearing strains, γ, in the liquefiable sand ranging from 0.0006 to 0.0707 %, (2) identification of the threshold strain at which excess pore pressure begins to be generated, γ_t^{pp}, falling between 0.015 and 0.020 %, (3) determination of values of r_u after 50 cycles of loading ranging from 0 to 19 %, and (4) in-situ measurement of the relationship between shear strain and the generation of excess pore pressure.

5. Comparison of Field and Laboratory Results

5.1 Comparison of V_S values determined from laboratory and field testing

A comparison of V_S values determined from RC testing (Haag and Stokoe (1985) [5]) and seismic crosshole testing (Cox (2006) [7] and Roberts (2014) [9]) shows similar small-strain stiffnesses across all specimens. The V_S values determined from RC testing range from 110 to 146 m/s over the range of estimated, mean effective in-situ stresses (37.9 to 40.1 kPa). The V_S values determined by from seismic crosshole testing at the depths and locations of the

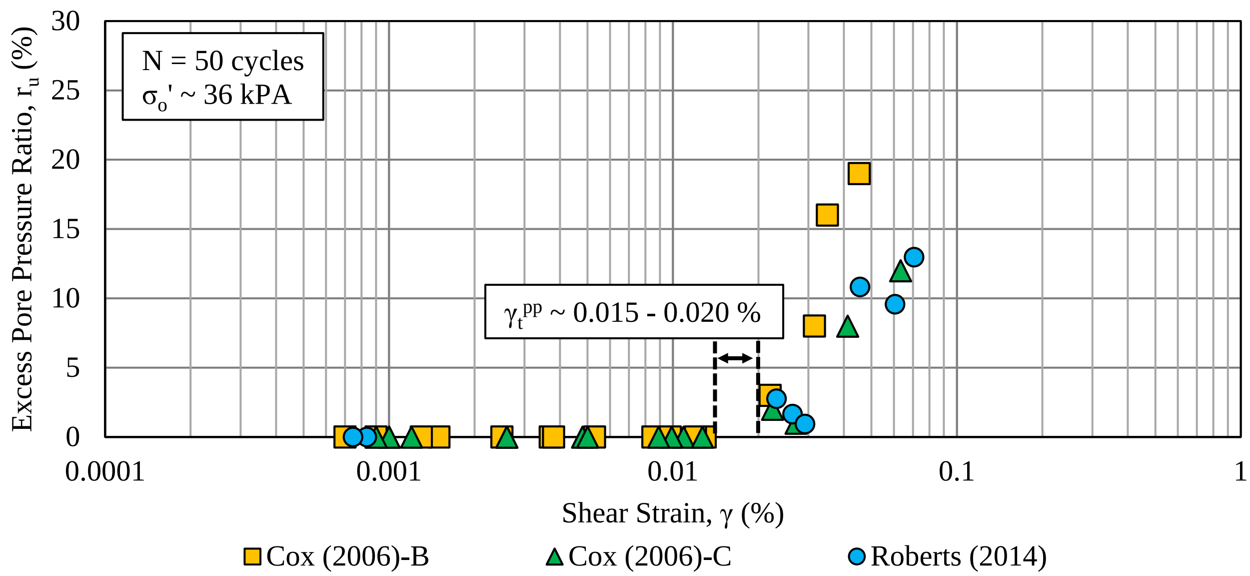


Fig.7 – Results from shaking tests with T-Rex by Cox (2006) [7] and Roberts (2014) [9] showing r_u after 50 cycles of loading versus average cyclic shear strain during each stage

shaking tests range from 101 to 141 m/s over the range of mean effective in-situ stresses including a stress distribution from the weight of T-Rex at the ground surface (33.9 to 38.9 kPa). The corresponding G_{max} values of the specimens range from 27 to 41 MPa for RC testing and from 20 to 40 MPa for seismic crosshole testing. These results show that the initial material stiffnesses of the specimens are similar as shown by the V_S comparison in Fig.5b and further comparisons of the results in the nonlinear strain range are reasonable.

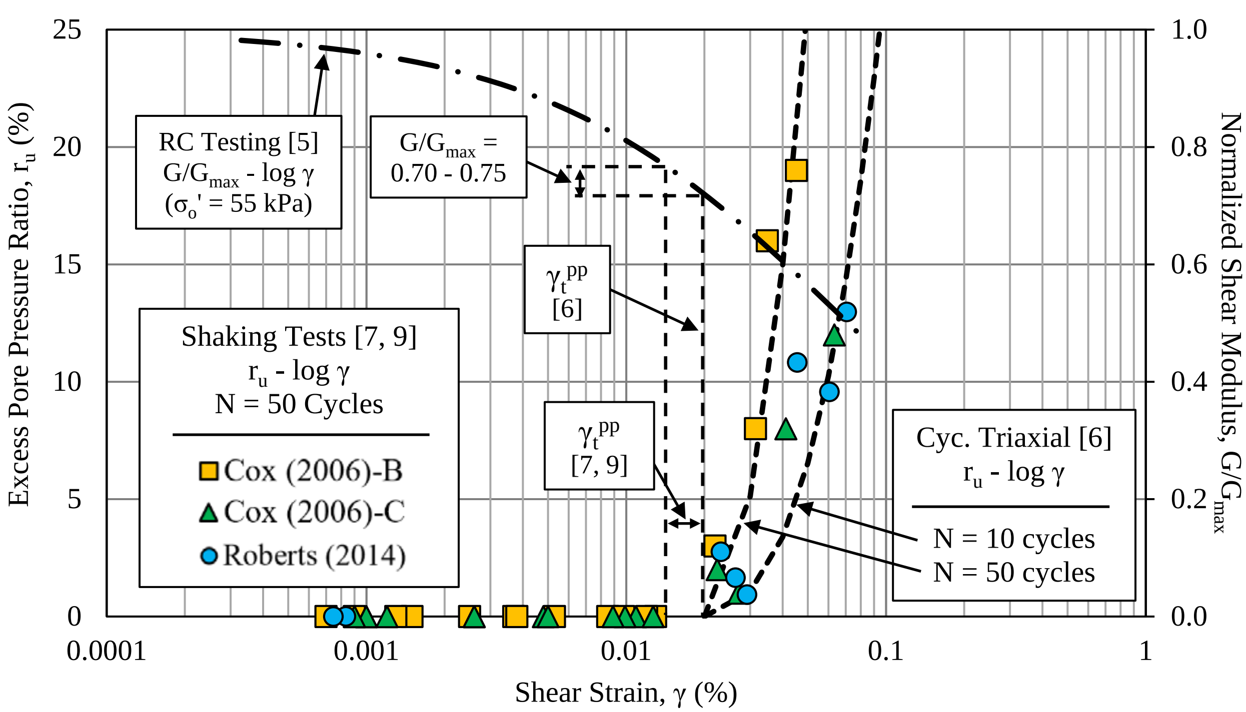


Fig.8 – Comparison of the r_u – log γ and G/G_{max} – log γ relationships for the liquefiable sand at the WLA

5.2 Comparison of field r_u-log γ and laboratory G/G_{max}-log γ relationships

The behavior that leads to liquefaction triggering is best understood in terms of the r_u-log γ relationship at a selected number of loading cycles (N). Laboratory cyclic triaxial testing by Vucetic and Dobry (1986) [10] and large-scale, in-situ shaking tests by Cox (2006) [7] and Roberts (2014) [9] both provide data that clearly define the beginning portion of this relationship for the liquefiable soil from Layer #2 at the WLA site. In Fig.8, the results from the shaking tests are plotted together with the Dobry model for pore pressure generation at 10 and 50 cycles on the primary vertical axis, excess pore pressure ratio, r_u (%). This comparison shows good agreement between: (1) the field measurements and the laboratory-derived model for the r_u-log γ relationship and (2) the location of the threshold strain for pore pressure generation, γ_t^{pp} (also called the threshold strain, γ_t, by Vucetic and Dobry (1986) [6]), which is between 0.015 and 0.020 %.

The modified hyperbolic shear modulus reduction curve model developed from RC testing by Haag and Stokoe (1985) [5] is also shown in Fig.8 using the same shear strain axis with a secondary vertical axis of normalized shear modulus, G/G_{max}. By plotting the normalized shear modulus reduction curve using the same strain axis as the r_u-log γ data, it is evident that the γ_t^{pp} from the r_u-log γ relationship corresponds to a G/G_{max} value in the range of approximately 0.70 to 0.75 , indicating that the soil is in the nonlinear range and has lost about 25 to 30 % of its initial stiffness before the shear strain is large enough to generate excess pore pressure; hence, large enough to create volume change.

6. Conclusions

Field and laboratory measurements from five research projects are compiled into a single study focusing on the behavior of silty sands that leads to liquefaction triggering at the WLA site. Laboratory and field tests performed for these projects include: (1) RC testing by Haag and Stokoe (1985) [5], (2) strain-controlled cyclic triaxial testing by Vucetic and Dobry (1986) [6], and (3) CPT testing by Roberts (2014) [9] and Yegian et al. (2014) [10], (4) crosshole seismic testing by Cox (2006) [7] and Roberts (2014) [9], and (5) shaking tests by Cox (2006) [7] and Roberts (2014) [9]. The conclusions from this study are as follows.

1. The liquefiable layer at the WLA site ranges from a depth of about 2.5 to 6.8 m and is composed of very loose to loose sandy silt and loose to medium-dense silty sand to very fine sand [1, 2].
2. The water table fluctuates but is generally about 1 m below the ground surface and the depth to 100 % saturation is approximately 2.8 m below the ground surface [10].
3. The shear modulus reduction curve (G/G_{max} – log γ) determined at σ_o' ~55 kPa for the liquefiable material can be described by the modified hyperbolic equation (Eq.1) [5]:
$$G/G_{max} = 1\Big/\left(1+\left(\frac{\gamma}{\gamma_r}\right)^{a}\right) = 1\Big/\left(1+\left(\frac{\gamma}{0.07\ \%}\right)^{0.74}\right).$$
4. Values of V_S determined from in-situ crosshole seismic testing range from 101 to 141 m/s for the liquefiable soil at mean effective stresses ranging from 34 to 39 kPa. These crosshole tests were performed as part of the in-situ shaking tests. Values of G_{max} calculated from the crosshole seismic results range from 20 to 40 MPa [7, 9].
5. The cyclic shear strains, γ, generated from the shaking tests range from 0.0006 to 0.0707 % and the values of excess pore pressure ratio, r_u, range from 0 to 19 % [7, 9].
6. The pore pressure threshold strain, γ_t^{pp}, for the liquefiable soil at the WLA site ranges from 0.015 to 0.02 % at an average σ_o' ~ 36 kPa [6, 7, 9]. This strain level also corresponds to the strain required to initiate volume change.
7. The values of G/G_{max} at the pore pressure threshold strain, γ_t^{pp}, determined from the in-situ shaking tests are approximately 0.70 to 0.75.

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8. References

[1] Bennett, MJ, McLaughlin, PV, Sarmiento, JS, Youd, TL (1984): Geotechnical Investigation of Liquefaction Sites, Imperial Valley, California. *Open-File Report 84-252*, Department of the Interior, U.S. Geological Survey, Menlo Park, California, USA.

[2] Steidl, JH, Nigbor, RL, Youd, TL (2004): Permanently Instrumented Field Sites in NEES: A Resource for Future Geotechnical Research. *Geotechnical Engineering for Transportation Projects*, p.1637-1644. GeoTrans Conference 27-31 July 2004. Los Angeles, California, USA.

[3] McCrink, TP, Pridmore, CL, Tinsley, JC, Sickler, RR, Brandenberg, SJ, Stewart, JP (2011): Liquefaction and Other Ground Failures in Imperial County, California, from the April 4, 2010, Mayor-Cucapah Earthquake. *U.S. Geological Survey Open-File Report 2011-1071 and California Geological Survey Special Report 220*, 94 p. pamphlet. [http://pubs.usgs.gov/of/2011/1071/] [http://conservation.ca.gov/cgs]. Accessed 26 April 2016.

[4] USGS (1966): Mineral and Water Resources of California. *Special Report to the Committee on Interior and Insular Affairs of the United States Senate*. Washington, D.C., U.S. Government Printing Office.

[5] Haag, ED, Stokoe II, KH (1985): Laboratory Investigation of Static and Dynamic Properties of Sandy Soils Subjected to the 1981 Westmorland Earthquake. *USGS-sponsored Geotechnical Engineering Report GR85-11*. Geotechnical Engineering Center in the Civil Engineering Department at The University of Texas at Austin, Austin, USA.

[6] Vucetic, M, Dobry, R (1986): Pore Pressure Build Up and Liquefaction at Level Sandy Sites During Earthquakes. *Research Report*. Department of Civil Engineering at Rensselaer Polytechnic Institute, Troy, USA.

[7] Cox, BR (2006): Development of a Direct Test Method for Dynamically Assessing the Liquefaction Resistance of Soils In Situ. *Dissertation*. The University of Texas at Austin, Austin, USA.

[8] Cox, BR, Stokoe II, KH, Rathje, EM (2009): An In Situ Test Method for Evaluating the Coupled Pore Pressure Generation and Nonlinear Shear Modulus Behavior of Liquefiable Soils. *Geotechnical Testing Journal*, **32** (1).

[9] Roberts, JN (2014): Direct In-Situ Evaluation of Liquefaction Susceptibility. *Thesis*. The University of Texas at Austin, Austin, USA.

[10] Yegian, MK, Stokoe II, KH, Thevanayagam, S (2014): Induced Partial Saturation (LPS) Through Transport and Reactivity for Liquefaction Mitigation. *Network for Earthquake Engineering Simulation Project 1081, DOI:10.4231*.

[11] ASTM Standard D2487, 1966 (2011): Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System). *ASTM International*. West Conshohocken, Pennsylvania, USA.

[12] Darendeli, MB (2001): Development of a New Family of Normalized Modulus Reduction and Material Damping Curves. *Dissertation*. The University of Texas at Austin, Austin, USA.

[13] Robertson, PK, Wride, CE (1998) Evaluating cyclic liquefaction potential using the cone penetration test. *Canadian Geotechnical Journal*. 35(3): 442-459. DOI:10.1139/cgj-35-3-442.

[14] Valle-Molina, C (2006): Measurements of V_P and V_S in Dry, Unsaturated and Saturated Sand Specimens with Piezoelectric Transducers. *Dissertation*. The University of Texas at Austin, Austin, USA.

[15] van Ballegooy, S., Roberts, J.N., Stokoe, K.H., Cox, B.R., Wentz, F.J., Hwang, S. (2015): Large-Scale Testing of Shallow Ground Improvements using Controlled Staged-Loading with T-Rex. *Procedures. 6th International. Conference on Earthquake Geotechnical Engineering*. 1-4 November 2015. Christchurch, New Zealand.