IN SITU LIQUEFACTION TESTING USING SEQUENTIAL DETONATION OF EXPLOSIVES

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ABSTRACT

The use of the sequential detonation of explosives is described to evaluate the seismic liquefaction potential of soil *in situ*. The method is based on the use of a number of blast pulses to generate shearing strains within a test volume of soil, having similar strain magnitudes and number of strain cycles as a design level earthquake. Residual pore pressure generation is generally dominated by the action of shearing stresses. For compressible soils, large changes in dynamic mean stress which occur close to the blast point also play a role. A method of calculating the influence of cyclic shear strains and changes in dynamic mean stress on residual pore pressure build-up is described. The test method normally involves installation of downhole instrumentation including pore pressure transducers and accelerometers to evaluate the ground response during explosive detonations. Alternatively, nonlinear blast analysis may be carried out to estimate downhole cyclic strains and stresses, calibrated to measured surface response. The test method is a direct form of downhole dynamic testing with particular application to evaluating seismic liquefaction potential of problematic gravelly or silty soils for which indirect penetration test methods may not apply or be subject to considerable interpretation.

Introduction

The evaluation of the potential for triggering soil liquefaction during seismic shaking is most often evaluated using correlations between various types of penetration tests (SPT or CPT) and cyclic resistance using field observations from past earthquakes where surface expression of liquefaction has occurred (Idriss and Boulanger, 2006; Moss, 2003). These methods are based on a more extensive data base for cleaner sand deposits although liquefaction triggering curves (cyclic resistance ratio versus stress normalized SPT or cone tip resistance value) for silty sands with low plasticity (plasticity index PI < 7) have been proposed for various fines contents and are currently under review by various researchers. Difficulties exist in development of all of these penetration-based methods in assessing representative values of penetration resistance, fines content and cyclic shearing stress in which liquefaction is considered to have occurred based on the field data, so that engineering judgment is required in assessing these factors. As a result of these potential uncertainties, Moss (2003) has used probabilistic methods to arrive at

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liquefaction triggering curves (based on CPT data) for various probabilities of occurrence. At this stage of development in the above procedures, no accounting is made for measured pore pressure response during cone penetration which provides an indication of the soil's contractive or dilative response. The latter can provide useful insights as to whether a soil will regain shear strength after cyclic pore pressure generation and thus will limit post-cyclic ground displacements.

The penetration resistance based evaluation methods are less well developed for the following soil types: Type (A) - low plasticity (PI < 7) silty sand and sandy silt soils having a fines content greater than about 35 percent; Type (B) - Plastic silts and clays with PI > 7, assumed to be clay-like in behavior; Type (C) - coarse grained gravelly soils. For Type (A) soils, typical procedures involve converting a penetration resistance value to an equivalent clean sand value. These equivalent clean sand corrections are often large and subject to ongoing evaluation. For Type (A) and (B) soils, it is desirable to obtain high quality soil samples using piston tube sampling or other methods so that cyclic laboratory testing can be carried out to assess seismic liquefaction resistance. In situ testing is also carried out to measure the static undrained strength ($S_{u,st}$) and then static strengths adjusted (based on laboratory testing or prior experience with similar PI soils) to account for cyclic loading effects. Potential problems exist with degree of sample disturbance that might affect the laboratory test results.

In the case of Type (C) soils, CPT or SPT measurements cannot be reliably made due to particle size effects and therefore Becker Density Tests (BDT's) with energy measurements are often used to assess an equivalent SPT resistance in sand (Sy, 2001). The assumption is then made that the gravelly soil deposit is sand-like in behavior for purposes of liquefaction triggering assessment. The latter assumption would be expected to be more reasonable where a large percentage of sand exists in the deposit and the coarse grained particles are separated by the sand matrix. Uncertainties exist in the influence of particle size and Becker casing friction on the equivalent SPT value derived from the BDT, and whether liquefaction triggering curves developed for sands apply to sand and gravel subsoils.

For the above reasons, techniques have been developed based on the controlled detonation of below ground explosives using relatively low charge weights per blast delay to generate long duration, cyclic shaking of the ground and thereby test the *in situ* liquefaction potential of the ground. The basic principle of the test is to induce multiple shear strain cycles and observe pore pressure build-up versus number and amplitude of strain cycles. Detonation of a cylindrical charge produces dynamic cavity expansion and shock front propagation. A large measure of shearing strain (and shearing stress) is then introduced into the ground, in addition to normal strain (and mean stress) components. The effects of the latter can also produce residual pore pressure build-up but, as will be discussed later, the shearing strain component dominates residual pore pressure build-up at reasonable distances from a charge detonation. This is analogous to the approach advocated by Dobry *et al.* (1982) who related pore pressure build-up to cyclic shear strain amplitude and number of strain cycles based on laboratory testing of clean sand. Provided one can simulate about the same shear strain levels during the blasting test as are anticipated from a design earthquake for a particular site, one can achieve a downhole simulation of the effects of earthquake shaking on residual pore pressure generation in a mass of soil.

Previous Work

Previous applications of the method within an instrumented test volume of soil located south of Vancouver, B.C. in the Fraser River Delta (denoted the Boundary Bay test site) are described by Gohl *et al.* (2001). The instrumentation consisted of two triaxial accelerometers, two pore pressure transducers, and Sondex tubes to measure ground settlements versus depth following pore pressure dissipation. The two accelerometers were offset in (x,y,z) space within the test volume and the acceleration data integrated to produce velocities and displacements versus time for each explosive charge detonated. The differential displacements were then used to calculate six components of strain (three normal strains, three shearing strains) from small strain solid mechanics theory. These strain components were then used to compute the maximum shear strain at a particular instant in time during a blast pulse.

The computed shear strains for an 11 charge detonation at the Boundary Bay test site are shown plotted in Figure 1 and ranged between 0.3 and 7% within the test volume. These are representative of very strong earthquake shaking. The accuracy of the shear strain calculation strongly depends on the accuracy of the input acceleration data and so the use of accelerometers having a high acceleration range and having accurate high frequency response is necessary. A detailed comparison of permanent displacements and ground strains computed using the present accelerometer method and using, for example, blast resistant inclinometers has not been carried out at this stage. The cyclic straining produced a residual pore pressure ratio (PPR) of 0.475 (excess pore pressure divided by vertical effective stress). The laboratory test data are in close agreement with the field test result. Stress controlled, cyclic simple shear test data on a sample of normally consolidated, low plasticity (PI < 5) silt having similar properties as the soil at the Boundary Bay blast site indicated a PPR of 0.44 after 11 constant amplitude shear stress cycles. The cycling produced peak to peak shear strain amplitudes of 1 to 1.5% after 11 cycles and are within the strain range inferred from the field blast tests. This suggests that the field method used to estimate cyclic ground strain is reasonable and gives pore pressure – shear strain results consistent with the available laboratory test data.

Data from single and two charge detonations are also shown in Figure 1. The single charge detonations were carried out at a different site in the Vancouver Lower Mainland (denoted the Annacis Island site), where a clean sand deposit was subjected to blast loading. Maximum shear strains from the single and two-charge detonations were calculated based on nonlinear blast modeling (Gohl, 2000) calibrated to measured downhole accelerations. The data in Figure 1 show a trend of increasing PPR with shear strain level and number of cycles of shaking. The field blast data also indicate that significant pore pressure build-up does not occur for shear strain amplitudes less than about 0.01%, in good agreement with data reported by Dobry.

The development of in situ liquefaction testing has also been proposed using a surface vibrator by Rathje *et al.* (2005). They have proposed using an array of accelerometers to measure horizontal and vertical accelerations at the four corners of a rectangular area. Double integration of the acceleration time histories is carried out to compute ground displacements. Assuming a linear variation of horizontal and vertical displacement between the four measurement locations and using finite element theory, strains at the mid-point of the array are computed. Data have been presented relating shear strain, number of cycles and residual pore pressure development within reconstituted saturated sand samples. The procedure is analogous to the approach outlined in the earlier blasting studies except that a steady state vibration source is used, strain amplitudes are generally smaller than those induced by blasting and limited by the strength of the vibrator, and only near surface soils may be tested.

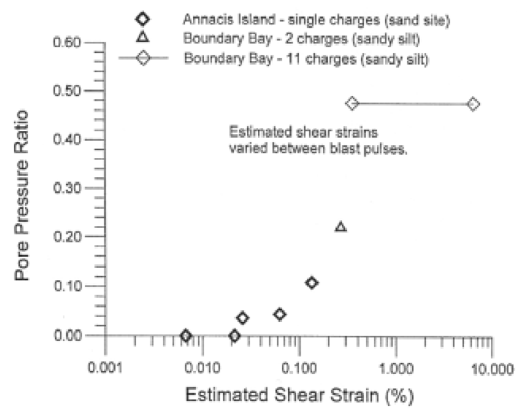

Figure 1: PPR versus shear strain calculated from previous blasting tests on Annacis Island and at the Boundary Bay site (Gohl *et al.*, 2001).

It is generally assumed in earthquake geotechnics that shearing strain is predominantly responsible for residual pore pressure build-up and analytic methods have been developed relating shear strain amplitude for a given load-unload cycle to plastic volume change potential. From the cumulative potential volume change after several cycles of shearing strain, residual pore pressure is computed (Martin, Finn and Seed, 1975). However, during blasting, significant changes in total mean stress occur close to a blast point in addition to large shearing stress (shear strain) changes where the changes in shearing stress are limited by the dynamic undrained shear strength of the soil. Work by Fragaszy and Voss (1986) using high pressure triaxial compression loading has indicated that large changes in total mean stress, involving initial stress increases followed by unloading, can produce residual pore pressure generation, particularly for more compressible sands. This is due to the hysteretic response of sand which indicates higher volumetric compressibility during loading compared to unloading. They have proposed that changes in sand skeleton compressibility during passage of a blast pulse through sand, resulting in large changes in dynamic mean stress, can result in small changes in Skempton's pore pressure parameter B and development of residual pore pressures. Fragaszy and Voss suggest that during blast loading, the two combined effects of shearing strain and changes in total mean stress are likely to contribute to residual pore pressure generation with the mean stress effect likely being more significant closer to a charge detonation.

One-dimensional shock tube tests carried out by Veyera and Charlie (1990) indicated that sand liquefaction can be produced for uniaxial compressive strains in close proximity to a charge detonation due to total mean stress changes. It is important to point out that the test method produces a rather artificial boundary condition for sand response and in the field a blast does not produce only uniaxial compressive strains. Rather, a three-dimensional strain field is produced, generating a large amount of shearing strain, which is also responsible for residual pore pressure generation. Thus, relating residual pore pressure generation to compressive strains induced by blasting (typically estimated from measured peak particle velocities and the compressive P-wave velocity of the medium) is misleading and belies the fact that the two mechanisms discussed above contribute to pore pressure generation. Finn (1978), in his survey of blast-induced soil liquefaction effects, has proposed a mechanism for residual pore pressure generation resulting from the two combined effects of shearing strain (using the Martin-Finn-Seed model cited above) and changes in total mean stress from passage of the blast pulse. It is this approach that was used in the analysis of a blast liquefaction test, discussed subsequently.

Single Hole Blasting Tests – Kemess Mine

Details of single hole blasting tests carried out upstream of an operating tailings dam at Kemess Mine in northern British Columbia are described in a companion paper (Witte *et al.,* 2010). Analysis of pore pressures generated by these *in situ* cyclic loading tests are discussed in the present paper and are compared with liquefaction triggering assessments made using CPT-based approaches. The primary objective of the single hole blasts was to confirm the dynamic response of the tailings in order to facilitate optimization of the design of a larger scale (multiple hole) blasting trial that had as its objective the generation of significant excess pore pressures in the tailings to hydraulically “stress” the abutment in a manner similar to the design earthquake.

A 2006 liquefaction triggering assessment was based on stress level normalized cone tip resistance (from 2006 CPT data) and used methods outlined by Moss (2003). This method included fines content corrections inferred from the cone tip resistance and sleeve friction, and was based on cyclic shear stress ratios computed from SHAKE-91 analysis using design earthquake motions for the site. The latter corresponded to an M6 earthquake having a peak firm ground acceleration of 0.19 g. The 2006 analysis indicated factors of safety against liquefaction triggering over the elevation range of interest (elevation 1465m to 1440 m) generally less than 1.5 and that the Kemess tailings could develop significant excess pore pressure build-up under the design seismic event with excess pore pressure ratios of up to 0.5. Peak cyclic shear stress ratios (CSR) in the range of 0.11 to 0.14 (effective CSR values of 0.07 to 0.09 equal to 65% of maximum values) were computed from SHAKE-91 over the elevation range of interest, based on a tailings beach elevation of 1498m at the time of the blast tests. A check of the 2006 analyses was carried out in 2009 using updated ground motions for the M6 (PGA = 0.19 g) event and indicated effective CSR values of 0.06 to 0.07, only slightly lower than the

2006 analysis.

CPT methods are based solely on tip resistance and do not directly account for dynamic pore pressure response, thereby discounting an indicator of volume change response to shear. Tip resistance is non-uniquely related to relative density, being also strongly affected by bulk compressibility and other factors. As such, the CPT-based methodology was judged to be potentially overly conservative. The use of the sequential blasting technique afforded the means of checking the pore pressure generation susceptibility of the tailings.

On July 18, 2007 a series of two single-hole test blasts (blasts S-1 and S-2) were performed using blast holes S1 and S2 discussed in the companion paper. The three installed piezometers (P5B, P7AA and P7B) recorded minor increases in pore pressure response resulting from the two blasts. The pore pressure increases were substantially less than had been expected on the basis of the tip resistance values from the 2006 piezocone soundings and based on previous experience with blasting in looser tailings for similar scaled distances $R/W^{0.33}$ where R = 11m and W is the average charge weight per delay (= 22 kgf.). The maximum excess pore pressure ratio during blast S-2 recorded by piezometer P5B at a distance of 11m from the blast hole indicated an excess pore pressure of 52 kPa or an excess PPR of 0.05. The PPR increased to 0.12 several hours after the blast due to pore pressure redistribution, presumably due to dilation of the soil mass around the blast hole. Peak particle velocities recorded at different distances from the blast holes were also slightly larger than PPV's recorded during blasting in looser tailings where liquefaction had occurred, indicating that the charge weights were sufficiently large so as to induce liquefaction in looser contractant tailings.

Review of the blast data suggested that the tailings were denser than indicated by interpretation of the 2006 CPT tip resistances but consistent with inferences made from interpretation of pore pressures generated during cone penetration. Subsequent energy calibrated SPT measurements made in 2008, combined with piston tube sampling of the tailings, and updated CPT measurements carried out in 2009 around the blast area are discussed in the companion paper. These 2009 measurements indicated slight increases in stress normalized tip resistance of about 25% between 2006 and 2009. Average q_{c1} values of 50 to 60 bars were measured in 2009 over the depth range of interest. The low measured penetration resistance could be a function of the high silt content (i.e. >10%) in the tailings which increases the compressibility of the soil, results in reduced stress-level dependent friction angles compared to cleaner sands, and consequently results in reduced resistance during penetration. The increases in cone tip resistance between 2006 and 2009 were consistent with measured increases in shear wave velocity (about a 13% increase expressed as V_{s1} values) over the same time period. The increased cone tip resistance is likely due to stress densification effects caused by increases in tailings beach elevation of about 10m and possibly soil aging effects. The 2009 data also indicated strong negative pore pressure generation during cone penetration.

Dynamic Analysis of Blast Tests

The sequential detonation of blast hole S-2, located 11m from piezometer P5B where maximum excess pore pressures were recorded, was modeled numerically using the nonlinear, finite element program LSDYNA (Livermore Software, 2001). LSDYNA has been extensively used for blast modeling by the first author because it has a large strain formulation and has a variety of nonlinear, stress-strain models suitable for dynamic analysis of soils. An axi-symmetric FE model was developed incorporating the surrounding tailings and the adjacent tailings dam located within 90 m of the nearest blast hole. The six charge detonation during blast S-2 was simulated by applying pressure time histories perpendicular to the walls of the borehole cavity over the length of each charge. Charge weights per delay up to 28 kg. (average 22 kg.) were considered with the larger charge weights modeled by using a longer length over which the blast pressure was applied. The blast pressures were assumed to have a rapid rise time and exponential decay with peak amplitudes selected based on the properties of the explosive used and accounting for air gap caused by dewatering of the interior of the blast casing (Henrych, 1979). A total stress model of saturated soil-water response was used, incorporating the undrained shear stress – shear strain response of the tailings and assuming zero total volume change during passage of blast waves. A shear strength equal to 25% of the vertical effective pressure for the tailings was assumed, derived from constant volume, direct simple shear (DSS) tests on piston tube samples of the tailings. Other laboratory tests on the tailings sampled over the 30 to 45m depth range included constant volume, cyclic DSS and one dimensional compressibility tests (with one load-unload cycle). Shear wave velocities were also measured after consolidation of the samples using bender element methods for comparison with downhole seismic measurements. These indicated V_s values at the upper end of seismic CPT measurements carried out in 2009.

The measured attenuation of ground surface vibrations was used to calibrate damping parameters used in the FE model to compute blast-induced strains and stresses with distance from the blast hole. The computed cyclic shear stress ratios (maximum shear stress on the horizontal plane divided by the vertical pre-blast effective pressure) at the 38m depth (mid-depth of the three piezometers monitored during the tests) and at a distance of 11m from the blast hole is shown plotted in Figure 2. The analysis indicated shear stress ratios in the range of 0.05 to 0.20 with an average stress ratio of 0.11. Shearing from the six blast pulses equaled or exceeded that anticipated from the design level earthquake based on the SHAKE-91 analysis. Computed cyclic shear stress – shear strain response on the horizontal plane for the six charge detonation is shown in Figure 3, indicating the strong cyclic shearing action induced by the blasting. Shear strain pulse amplitudes of up to 1.1% were computed (see Figure 4), with the maximum strain pulses occurring from detonation of Charge 4 located closest to the soil element under consideration. Note that the computed shear strain on the horizontal plane was not significantly different than the computed maximum shear strain in the finite element model. Permanent strain offsets at the end of blasting were computed from the analysis. Figure 5 shows the computed changes in total mean stress at the element under consideration with computed peak blast pressures of up to 12 MPa. These dynamic mean stress changes would be expected to have some effect on residual pore pressure generation, discussed subsequently.

The cyclic pore pressure generation resulting from blast S-2 has been discussed in the companion paper, and indicated a residual PPR of 0.05 immediately at the end of blasting, much lower than anticipated assuming loose tailings behavior. A pore pressure generation model developed by the first author, incorporating the effects of total mean stress change during passage of a blast pulse, hysteretic volume change response of the soil skeleton due to mean effective stress changes, and the influence of shear strains on residual pore pressure development, has been used to compute the theoretical build-up of residual pore pressure during the six charge detonation. The results of the computations are shown in Figure 6. The model requires as input the total mean stress and shear strain time histories induced by blasting, which were computed for this case from the FE model. Alternatively, this input can be measured *in situ* using an array of accelerometers and dynamic piezometers. The model also requires one dimensional compressibility and shear strain-residual pore pressure generation parameters which were derived from the available laboratory test data for the tailings sands. Background to key features of this model is described by Fragaszy and Voss (1986) and Finn (1979). It is noteworthy that the one dimensional compressibility data indicated a compressible sand, given its apparently high relative density, and that this may explain the dichotomy between the lower than anticipated cone tip resistances and the negative pore pressure response during cone penetration.

Examination of Figure 6 indicates that incremental changes in total mean stress during a load-unload cycle (incorporating hysteretic soil skeleton volume change) can be responsible at the location under consideration for a small amount of residual pore pressure build-up during blasting (shown by the solid red lines in the figure), but that the majority of residual pore pressure results from shear strain increments during each blast pulse (indicated by the solid black squares in the figure). The latter is similar to the mechanism considered to be dominant during earthquake shaking. As such, it is the writers' opinion that use of sequential blasting can be used to provide a direct, *in situ* evaluation of residual pore pressures to be expected during strong earthquake shaking where the blast test is designed to encompass the shear strains and shear stresses expected from design levels of seismic shaking. Because of the potential influence of total mean stress changes during blasting at closer distances to a blast hole, the test method will provide a somewhat conservative evaluation of residual pore pressure build-up resulting from shear strains alone. The influence of mean stress change for load-unload cycles is expected to be more significant for compressible sands. *In situ* liquefaction testing using the sequential blasting method should therefore be designed to reduce the potential impacts of mean stress change, where possible.

Additional Testing

Given the contrary results about liquefaction susceptibility indicated from the 2006 CPT data and the 2007 blasting tests, cyclic laboratory testing of tailings tube samples obtained in 2008 was carried out. This indicated relatively small pore pressure generation potential (pore pressure ratio of about 0.08) after six cycles (representative of an M6 earthquake) at an effective CSR value of 0.09 derived from the 2006 SHAKE-91 results. This was in close agreement with the blast test results, although the possibility of sample disturbance cannot be discounted given that the lab V_s measurements were at the upper end of field measurements carried out in 2009 as shown in the companion paper. It is also possible that stress densification occurring between the time of the blast tests in 2007 and soil tube sampling in 2008 has resulted in small differences in soil properties between 2007 and 2008. Difficulties in obtaining truly undisturbed soil specimens for cyclic laboratory testing also point to the need to developing a reliable *in situ* liquefaction testing method. Additional CPT testing carried out in 2009 combined with an update of SHAKE-91 analysis confirmed the high cyclic liquefaction resistance of the tailings with factors of safety against liquefaction computed to be in excess of 2.0. The differences between the 2006 and 2009 CPT-based analyses stem from re-evaluation of seismic demand on the tailings under the design M6 event and the roughly 25% increase in CPT q_{c1} value between 2006 and 2009 due to rise in tailings beach elevation (stress level effects). However, the blast test results in 2007 should be compared against the 2006 CPT evaluation of liquefaction susceptibility for similar tailings beach elevations. This comparison suggests that use of CPT tip resistances for seismic liquefaction evaluation with no consideration of the effects of tailings compressibility on tip resistance and dilative pore pressure response can lead to overly pessimistic predictions of seismic liquefaction susceptibility.

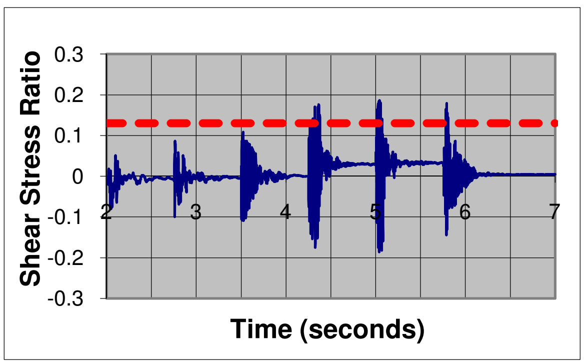

Figure 2: Computed cyclic shear stress ratios on the horizontal plane versus time. The dashed line represents the average CSR.

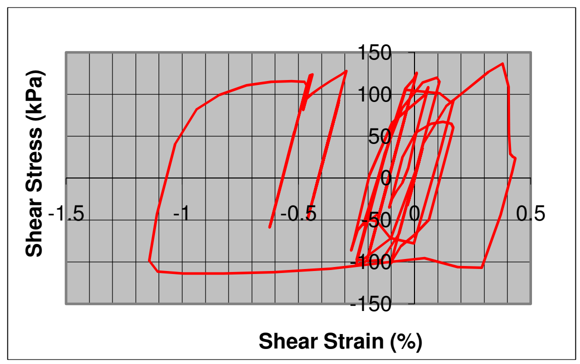

Figure 3: Computed shear stress versus shear strain on the horizontal plane.

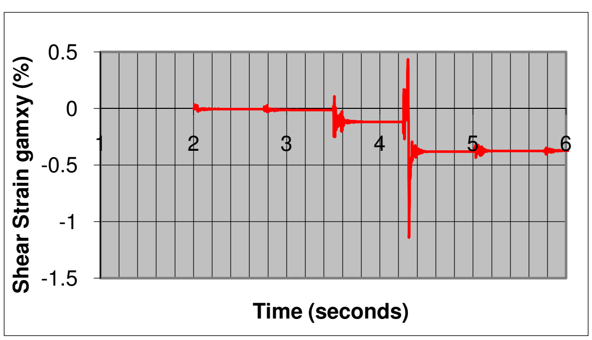

Figure 4: Computed shear strain on the horizontal plane versus time.

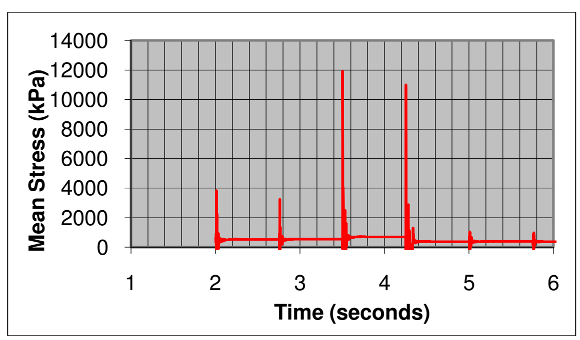

Figure 5: Computed mean stress versus time.

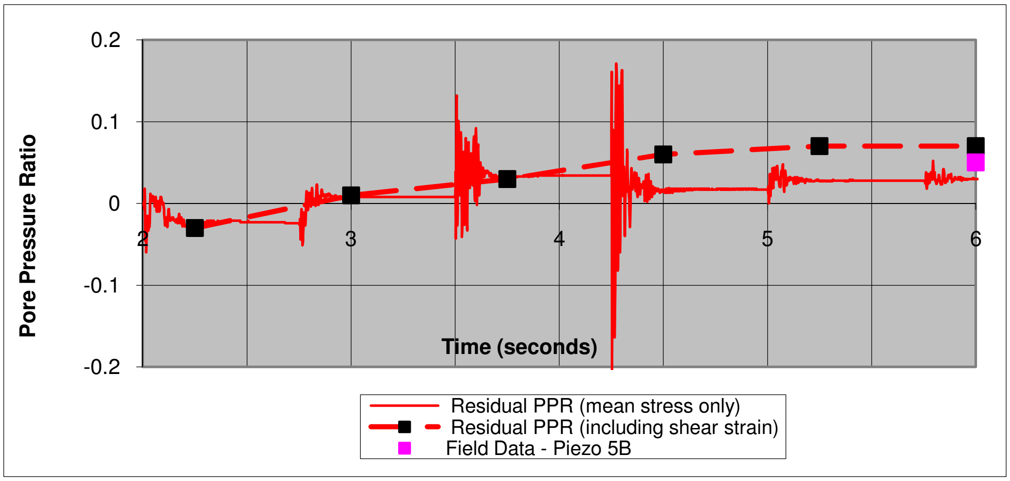

Figure 6: Computed pore pressure versus time, showing influence of total mean stress changes and shear strains during six charge detonation.

Summary

The use of the sequential detonation of explosives represents a convenient downhole energy source for use in creating cyclic shearing strains and stress representative of a design earthquake at a site and testing the residual pore pressure generation potential of subsoils. In the case of the Kemess Mine tailings, the method provided additional information supporting the conclusion that the tailings were denser than anticipated based purely on consideration of cone tip resistance and that the subsoils were resistant to cyclic pore pressure generation under design levels of seismic shaking. The use of sequential blasting is considered particularly useful for testing *in situ* silty and gravelly soils for which current field and laboratory-based methods of evaluating seismic liquefaction potential are problematic.

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