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Use of Controlled Detonation of Explosives for Liquefaction Testing

W. Blair Gohl
Pacific Geodynamics Inc.
Vancouver, B.C. - CANADA

J.A. Howie
University of British Columbia
Vancouver, B.C. - CANADA

C.E. Rea
Pacific Geodynamics Inc.
Vancouver, B.C. - CANADA

ABSTRACT

A field liquefaction test was conducted in Delta, B.C., Canada. The target layer was a loose sandy silt between the 10 and 12 m depth. The test layer was instrumented with two triaxial accelerometers, dynamic and static pore pressure transducers, and Sondex tubes to measure vertical ground strain. An array of boreholes was drilled around the instrument cluster and charged with explosives. Delays were introduced to the detonation sequence in order to generate multiple blast pulses. The cyclic loading from the blasting generated a series of shear and compressive strain pulses. We consider that shear strain dominates residual pore pressure rise and that shear strain amplitudes can be induced by blasting similar to those caused by an earthquake. In this way, the susceptibility of the ground to pore pressure generation caused by cyclic shear straining and post-liquefaction deformations of the ground are tested. Details of methods used to estimate shear strains induced by the blasting process are described.

INTRODUCTION

The need for ground improvement in cohesionless soils is often dictated by seismic design requirements relating to control of soil liquefaction. This is important is seismic design for a wide range of civil engineering structures where the need for ground improvement is dependent on a careful assessment of *in situ* liquefaction potential generated by cyclic (seismic) loading. The assessment has traditionally been carried out using correlations between various types of soil penetration testing and seismic liquefaction resistance, supplemented by cyclic laboratory testing on reconstituted cohesionless soil samples (Seed , 1979; Harder and Seed, 1986). The penetration test-based methods have been developed primarily for sands and silty sands and their application to highly gravelly soils or low plastic silts and clays is uncertain.

For low plastic silts and clays which are deemed to have a high risk of liquefaction based on application of the "Chinese criterion" (Wang, 1979), cyclic laboratory testing is often used to confirm liquefaction susceptibility. For important projects, ground freezing followed by coring of frozen soil samples to minimize soil disturbance effects may be used to recover soil samples for cyclic laboratory testing. Other possible approaches in assessing liquefaction potential involve conversion to an "equivalent clean sand" Standard Penetration Test value which involves large corrections to penetration number. The liquefaction triggering correlations developed for sands and silty sands may be inapplicable for these low plastic silts and clays.

In gravelly soils, there is a limited data base relating liquefaction susceptibility to penetration resistance and for this reason one is forced to convert a blow count measured in a gravelly soil using, for example, a Becker Density Test or large diameter penetration test (LDPT), to an equivalent clean sand $N_{1,60}$ value. One then assumes that the liquefaction triggering curves developed for sands apply to gravels based on an equivalent penetration number. This approach may also be unreliable.

The results of cyclic laboratory tests need to be critically reviewed to account for sample disturbance, soil fabric and soil ageing. There is also a growing recognition of the role of stratigraphic effects on the generation and redistribution of excess pore water pressures during and after shaking (Dobry *et al*, 1995). This complicates laboratory idealization of field drainage conditions. Vaid and Sivathayalan (2000) have shown that the occurrence of volume change (drainage) during or following cyclic loading can transform a dilative sand to a contractive strain softening behaviour. Thus, the typical laboratory idealization of undrained soil response during cyclic loading may lead to an unconservative assessment of liquefaction potential for the actual field case.

For the above reasons, techniques have been developed based on the controlled detonation of explosives 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. The advantages of such a test are:

- The method can be applied to all soil types, especially problematic soils such as sands and gravels, or low plastic silts and clays where current liquefaction evaluation methods are subject to considerable interpretation and uncertainty.

- The liquefaction resistance of the ground is evaluated *in situ* under its existing confining stress state with no soil disturbance effects.

- There is no necessity to carry out soil sampling for purposes of cyclic laboratory testing. Furthermore, the mass behaviour of the ground is tested rather than just an elemental volume considered in a laboratory test.

- There is no need to idealize the drainage conditions of the cyclic loading as being purely undrained. The amount of drainage that occurs during testing will be dictated by the permeability characteristics of the subsoils, the areal extent of pore pressure build-up and pore pressure gradients, and the rapidity of cyclic loading.

- One can measure the consequences of pore pressure build-up and soil softening in terms of vertical or lateral deformation potential.

- The results of the field test can be back-analysed to obtain dynamic soil properties for use in modelling other cyclic loading conditions.

GENERAL BACKGROUND

The basic requirements of the test are a source of "down hole" vibrational energy, such as explosive charges or various kinds of vibratory probes. The authors have used explosive charges since these are readily transportable, easily installed in drilled boreholes, and generate ground velocity and displacement amplitudes over a relatively large volume of soil that are similar to those caused by an earthquake.

The basic principles of the blasting test method and field observations indicate:

- The explosive detonations cause dynamic cavity expansion and shock front propagation causing relatively large amplitude shear straining of the soil mass whose amplitudes decay with distance from the blast point.

- Triaxial ground displacements and strains are induced at the shock front caused by stress propagation away from the blast point, with the shear strains considered to be primarily responsible for residual pore pressure build-up in the soil.

- The blast causes high frequency acceleration of the near field soil mass, much higher than that of real earthquakes, but with ground velocity and displacement amplitudes similar to those caused by strong earthquake shaking.

- The dominant shear strain amplitudes within 100 metres or so of a blast hole propagate at the P-wave velocity of the medium with smaller amplitude shear strain pulses propagating at the slower S-wave velocity.

- Shear strain pulses have durations of about 10 msec (100 Hz frequency) and therefore the blasting induces relatively high strain rates in the soil mass.

- The explosive charge weights and distances of the test volume from the blast points (for the test configuration described in the present paper) create peak hydrodynamic pressure pulses of up to 5 Mpa, necessitating the use of robust accelerometers and pore pressure transducers that can withstand these blast pulses.

Strains generated by unequal principal stress changes in a soil element during blasting are responsible for residual pore pressure generation. 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.

GENERAL TEST SET-UP AND DATA PROCESSING

Nonlinear blast analysis using a spherically symmetric blast model discussed by Wu (1995, 1996) and experience with the blasting method on various sites, is used to select charge weights (typically in the range of 2 to 6 kg per charge) to obtain shear strain amplitudes which cover the range of strain amplitudes anticipated for a design earthquake at a site within a test volume of soil. These strain amplitudes may be estimated using one or two dimensional site response analyses commonly used in geotechnical earthquake engineering. The number of blast (shear strain) pulses is chosen to be in the range of 10 to 15 to mimic the number of effective cycles of shaking of a large design earthquake (M7 to M7.5). Charge sizes may be limited to minimize offsite vibration effects but this is not a serious constraint since the test volume of soil can be located relatively close to the blast holes, allowing lower charge weights to be used to achieve the same strain level.

In practice, a circular array of blast holes containing 1 to 2 decks of explosives are detonated sequentially using long period delays to cause long duration, cyclic straining. The blast array is chosen to "hit" a test volume of soil from different directions. The test volume of soil is instrumented with high-g triaxial accelerometers, a "high speed" dynamic pore pressure transducer to measure peak dynamic pressure pulses, and a "slow speed" pore pressure transducer to measure residual pore water

pressures during the blast and after cessation of blasting.

It is important to couple the triaxial accelerometers and pore pressure transducers to the ground and to avoid the influence of drilling rods on instrumentation response. This was achieved by mounting the accelerometers and pore pressure transducers into a specially designed cone tip, pushing the cone into the ground to the desired depth using a drill rod string, and then removing the drill rods from the ground so that only a flexible, low mass electrical cable emanated from the cone tip up to the ground surface.

A high speed data acquisition system having a sampling rate of 20,000 samples per second per channel was used to acquire the accelerometer and dynamic pore pressure data.

Other instrumentation included a Sondex tube to measure vertical strains in the test layer of interest due to soil consolidation following pore pressure dissipation, and surface geophones to measure ground surface velocities at different distances from the centre of the blast area.

The accelerometer data is processed (from suitable integration of the measured high speed accelerometer data) to give peak acceleration, velocity and ground displacement in all 3 coordinate directions for each blast pulse. The ground displacements measured in 3 coordinate directions at two different locations across the test volume are used to calculate average differential displacements and strains in the test volume, and from these compute maximum shear strain.

SITE CONDITIONS AT LOCATION OF FIELD TRIAL

A farmer's field located south of Vancouver, B.C. in an area of Holocene estuarine deposits along Boundary Bay was selected for the liquefaction field trial. Prior to blasting, electronic cone penetration testing (CPT), mud rotary drilling to obtain soil samples, and downhole seismic testing to measure shear wave velocity profiles were carried out within the zone of testing. The drilling and CPT data indicated interlayered silt, sand and clayey silt (estuarine) deposits down to at least the 15 m depth. The water table was at or near the ground surface.

The soil layer that was selected for the *in situ* liquefaction test was located between the 10 and 12 m depth and had an average shear wave velocity of 160 m/sec. The layer consisted mostly of low plastic sandy silt to silty sand having a liquid limit of 27%, a plastic limit of 22% and a natural water content in the range of 38 to 40%. The soil classification is ML. Fines contents (percentage passing the U.S. no. 200 sieve size) were found to be in the range of 40 to 70%. Using the Chinese criterion this material would be deemed susceptible to liquefaction.

Processing of the CPT data between the 10 and 12 m depth has indicated corrected Standard Penetration $N_{1,60}$ values in the range of 5 to 10. These relatively low $N_{1,60}$ values also imply that the material is susceptible to liquefaction using liquefaction triggering curves presented by NCEER (1997) for silty sands having fines contents of at least 35%. Using the NCEER triggering curves and assuming a moderate earthquake at the Boundary Bay site, the test layer is predicted to liquefy at a peak ground surface acceleration of 0.13 g or greater. Current seismic design requirements in the Vancouver Lower Mainland consider that a design M7 earthquake could produce peak ground surface accelerations of up to 0.3 g. Calculations indicate that this would be expected to produce average shear strains at the 10 to 12 m depth of 0.4% to 2% assuming equivalent shear moduli 0.1 to 0.5 times G_{max} . Thus, the field trial was designed to produce average shear strains per blast pulse in the above range.

BLAST DESIGN AND INSTRUMENTATION LAYOUT

The blast array is shown in Figure 1. It surrounds a central instrumentation cluster where the *in situ* ground response was monitored.

The explosive type used in the blasting trials was Apex Ultra 60 which is an emulsion-based explosive product having a rated velocity of detonation of 5000 m/sec and a bulk density of 1.24 gm/cu.cm. It is rated to have 106% of the efficiency of TNT evaluated on a weight strength basis. The explosive is packaged in cylindrical cartridges which are lowered down to the desired depth.

Two 6 kg charges were placed within blast holes located at a 12 m horizontal distance from the centre of the test area. Two 2 kg charges were placed within blast holes located within 6 m of the centre of the test area. Gravel stemming was used between each charge to ensure a minimum of 2 m separation between adjacent charges. This separation was designed to minimize the occurrence of sympathetic detonation of adjacent charges. The charges within each blast hole were centred at the 8 and 12 m depths, respectively. The top charge in each blast hole was detonated first, followed by the bottom charge in the same blast hole.

The field trial was detonated in a sequence that allowed for one second intervals between individual boreholes, and about 400 millisecond intervals between the decks in each hole. The time delays achieved, however, varied from this 400 millisecond value due to inaccuracies in the electrical blasting caps used. The total length of time between beginning and end of the field trial was about 8 seconds with 16 decks of explosives being detonated. A total of 15 seconds of high speed data acquisition was acquired during the trial.

To check that all instrumentation was functioning properly before the 16 charge blast, a single 8 kg charge detonation was carried out 5 hours before the main blast series. The blast hole was located at a horizontal distance of 12 m from the centre of the test zone. The single charge data demonstrated that all

instrumentation was functioning properly before the main blast.

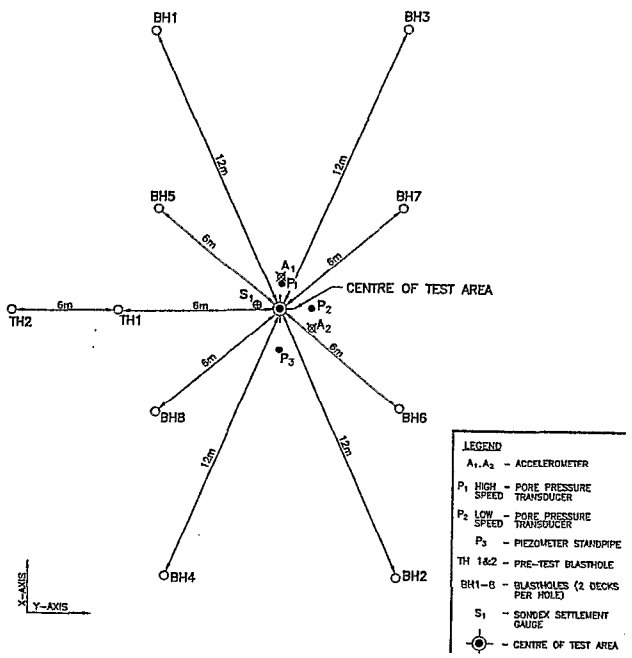

Fig. 1 Blast hole and instrumentation layout.

MEASURED INSTRUMENT RESPONSE

Ground Surface Vibration

Measured ground surface vibrations, expressed as the peak vector sum of particle velocities measured in 3 perpendicular coordinate directions (longitudinal, transverse and vertical), are plotted versus scaled charge weight and distance (R/√W) in Figure 2. Here R is the hypocentral distance between the centre of the blast area and the observation point where $R = X^2 + d^2$. X is the horizontal distance between the measurement point and the centre of the blast area and d is the average depth of the blast which has been set equal to 10 m. W is the maximum charge mass detonated per shot, which equals 6 kg. The Boundary Bay data are also compared against similar data obtained during blasting at other alluvial sites. The Boundary Bay surface vibration data are consistent with that obtained from other field tests and shows peak particle velocities of 0.25 m/sec in the middle of the field trial (scaled distance of 6.4), decreasing to 0.10 m/sec at a horizontal distance of 30 m from the centre of the test array. The ground surface velocity achieved within the central area of the field trial is reasonably representative of what would be expected during a major earthquake in the Vancouver Lower Mainland, according to the 1995 National Building Code of Canada.

Peak horizontal and vertical ground surface accelerations of 4.0 g and 20.5 g, respectively, were recorded within the centre of the field trial. Ground motions in the centre of the field trial are dominated by high frequency components. Due to this high frequency content, ground acceleration levels within the field test are considerably higher than what is considered in earthquake design in the Vancouver Lower Mainland. However, double integration of the high frequency accelerometer data (discussed subsequently) has yielded displacements that are up to 50 mm per blast pulse, in line with what would be expected during strong earthquake shaking at a site in the Vancouver Lower Mainland.

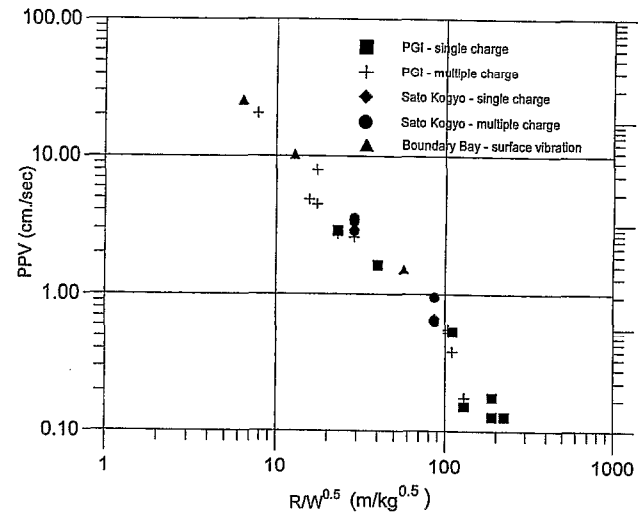

Fig. 2 Peak particle velocities at ground surface versus scaled distance R/√W.

Downhole Pore Pressure Response

High speed pore water pressure data from the multiple hole blasts are shown in Figure 3 which shows both the peak hydrodynamic pore pressure resulting from arrival of the blast-induced shock front as well as the residual pore water pressure resulting from distortion of the soil mass, following passage of the shock front. It is evident from the hydrodynamic pressure traces shown that some blast pulses resulted in very small pressure amplitudes and that there were only 11 well defined blast pulses. The individual boreholes were charged with different boxes of explosives and it is surmised that the quality of the explosive contained in one box was poor, resulting in low order detonation in two blast holes (blast holes 2 and 4).

The ratchetting up of residual pore pressure resulting from the multiple strain pulses is shown in Figure 3. Post-blast re-calibration of the high speed piezometer P1, following removal of the probe from the ground, indicated that damage to the sensor had occurred; therefore the amplitude of pore pressure changes is considered unreliable. The data are presented to show the trend of the gradual increase in pore pressure that developed during the blasting sequence.

Residual pore pressure ratio (excess pore pressure divided by initial effective overburden stress) measured by slow speed piezometer P2 after 11 significant blast pulses is shown in Figure 4. The measured pore pressures indicate a pore pressure ratio (PPR) of 0.475 was achieved as a result of the multiple hole detonation. The single charge detonation in advance of the main test resulted in a PPR of 0.15 so that the total PPR after all blasts was 0.625. Complete soil liquefaction was not observed. Pore pressures then gradually decreased over time corresponding to pore pressure migration away from the zone of instrumentation. Post-blast recalibration of piezometer P2 confirmed that the field test measurements were accurate.

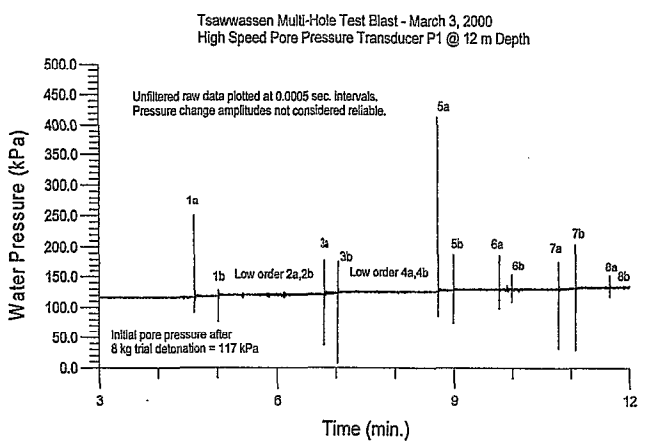

Fig. 3 High speed piezometer P1 response during 16 charge test blast.

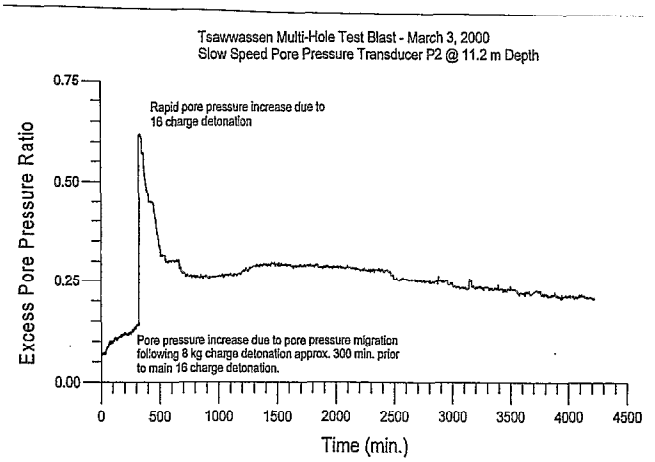

Fig. 4 Slow speed piezometer P2 response during 16 charge test blast.

Downhole Ground Vibration Response

Typical downhole triaxial acceleration response is shown plotted in Figure 5. The acceleration response was measured at the location of accelerometer A1 during detonation of the upper 2 kg charge in blast hole 5 (denoted blast pulse 5A). The accelerometer was located at a hypocentral distance of 5.8 m from the centre of the charge and recorded peak accelerations of over 2000 g's. The high frequency nature of the accelerations is evident. The horizontal x and y components of acceleration are nearly equal in peak amplitude as would be expected since the charge was located at a 45° angle off the x-axis of the accelerometer. There was also a large vertical z acceleration component indicating that the charge detonation caused vertical (shearing) motions of the ground.

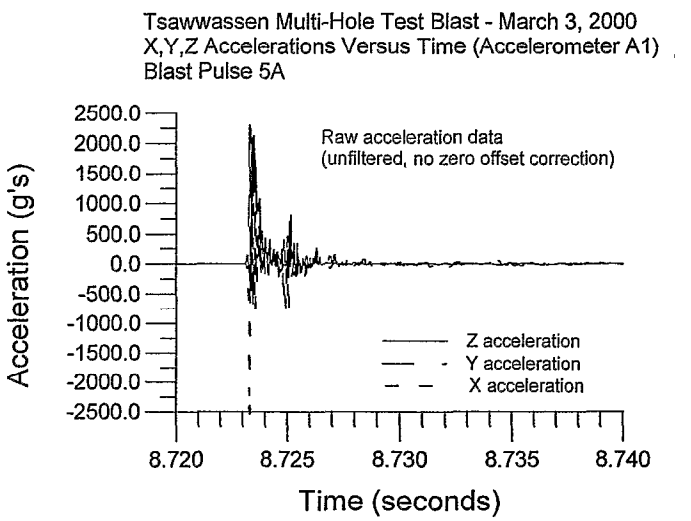

Fig. 5 Triaxial accelerations measured by accelerometer A1 during blast pulse 5A.

These large amplitude ground vibrations, which cause both compressional and shearing strains in the ground and contain a multiplicity of frequencies, travel at the P-wave speed of the medium. This is a "near field" effect (i.e. an effect observed in close proximity to a shear disturbance in the ground) as pointed out by Aki and Richards (1980). Lower amplitude ground waves which cause predominantly shearing motion arrive later and travel at the S-wave velocity of the medium. These secondary wave arrivals are less important in generating significant strains near to a blast point.

We have integrated the acceleration traces recorded in the x, y and z directions for a particular blast pulse over 20 msec time windows to compute particle velocities and displacements in each direction. We have used a very simple baseline correction procedure prior to integrating the acceleration - time traces which involved subtracting off the mean acceleration over the time window selected from the raw acceleration values. This baseline process resulted in component velocities which were approximately equal to zero at the end of the time window.

The vector sum of particle velocities and particle displacements was then computed from the individual velocity and

displacement components for each time step during a blast pulse. A typical variation of the vector sum of particle velocity versus time is shown in Figure 6 based on data recorded from accelerometer A1 during blast pulse 5A. A single velocity pulse, ramping up to a peak value and then decreasing to zero at the end of the pulse is shown in the figure. The vector sum of particle displacement versus time is shown in Figure 7, which shows a gradual ramp up of particle displacement and permanent displacement after passage of the blast pulse.

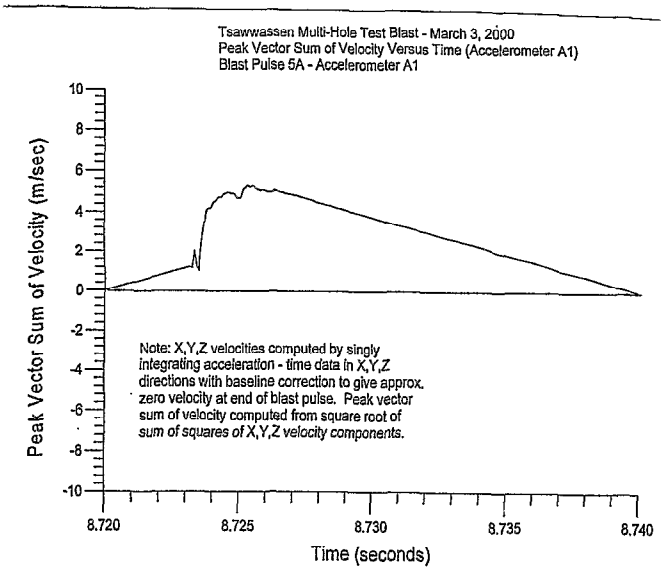

Fig. 6 Peak vector sum of particle velocity versus time at accelerometer A1 during blast pulse 5A.

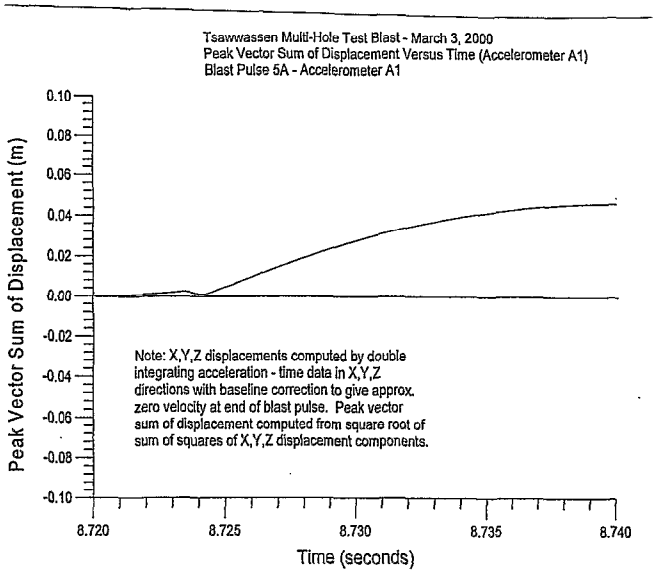

Fig. 7 Peak vector sum of particle displacement versus time at accelerometer A1 during blast pulse 5A.

The maximum particle velocity (PPV) and particle displacement (PPD) computed for a particular blast pulse was then determined from the above integration process. Computed PPD's versus scaled distance SD (= $R/W^{0.33}$) are shown in Figure 8 which shows the scatter in particle displacements for each blast pulse at each accelerometer location. Computed peak particle displacements ranged between 0.4 and 46 mm.

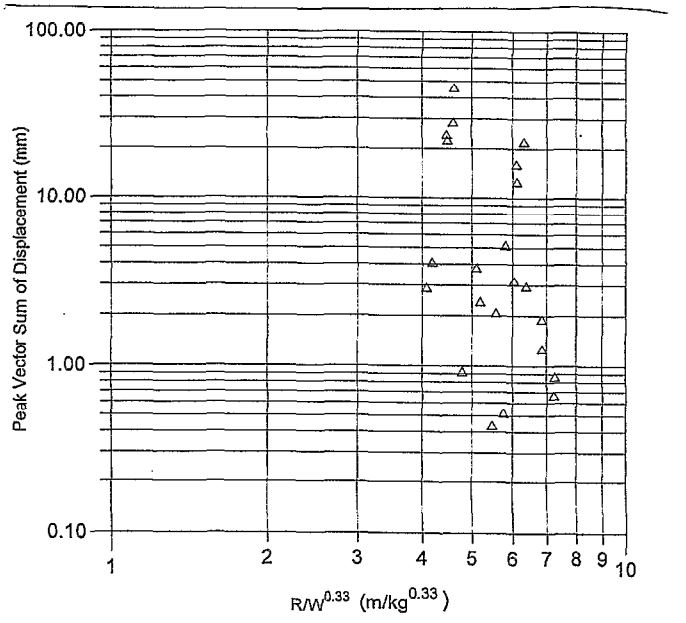

Fig. 8 Peak vector sume of particle displacement versus scaled distance $R/W^{0.33}$ at accelerometers A1 and A2 for all blast pulses.

The PPD's show considerable scatter over SD values in the range of 4 to 7. This scatter is possibly due to the following factors:

- differences in soil characteristics (stiffnesses, strengths, P-wave velocities) and soil layering between the blast source and accelerometer, causing wave scattering (this could be broadly termed travel path effects)
- progressive softening of the ground due to residual pore presssure build-up with each detonation, causing progressive changes in soil stiffness and damping
- differences in the explosive energy efficiency for each charge detonated
- inability of the type of accelerometers used to accurately record high frequency acceleration components much higher than about 3 kHz

LABORATORY CYCLIC SIMPLE SHEAR TEST DATA

It is of interest to compare the previous field measurements of residual pore pressure generation with that measured in the laboratory during cyclic simple shear tests on low plastic silts.

Figure 9 shows residual pore pressure generation response versus number of shear stress cycles measured in a cyclic simple shear test carried out on a sample of low plastic silt. The silt was obtained from a site in the Vancouver Lower Mainland and had a plasticity index of 2.5% and a water content greater than its liquid limit. These properties are similar to the Boundary Bay silt. Cyclic shear stress amplitudes equal to 18% of the vertical consolidation stress (= 125 kPa) were applied.

The cyclic shearing caused peak to peak shear strain amplitudes equal to 1 to 1.5% during the first 10 to 15 cycles of shearing in the laboratory test. As discussed subsequently, these shear strain amplitudes are within the range inferred from the field blasting trial. The cyclic shearing in the laboratory test resulted in a pore pressure rise equal to 55 kPa after 10 cycles of shearing. This corresponds to a pore pressure ratio of 55/125 = 0.44 which may be compared with a PPR of 0.475 measured during the field blasting trial for 11 significant blast pulses. The laboratory test data is in close agreement with the field test results, which lends credibility to the field procedures.

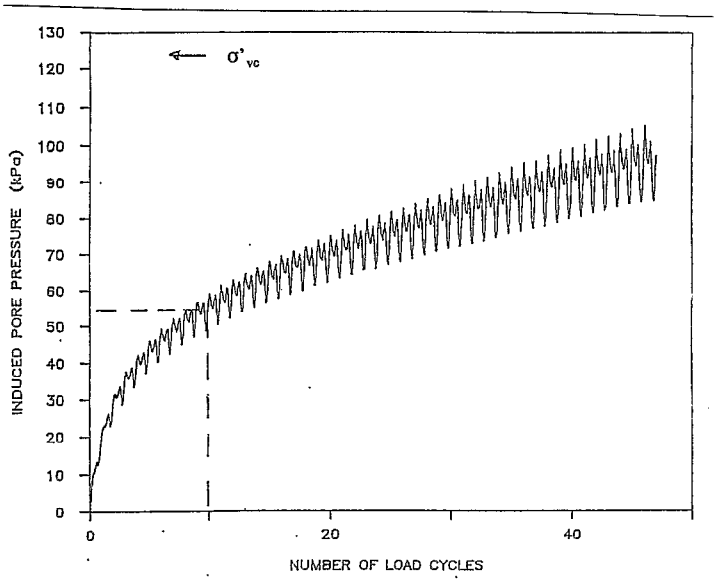

Fig. 9 Residual pore pressure build-up versus number of constant shear stress cycles from cyclic simple shear test data on low plastic silt.

COMPUTED AVERAGE SHEAR STRAINS IN THE TEST VOLUME

The range in computed peak displacements at each accelerometer location indicates that the multiple charge detonations have produced a range of shear strains in the test volume of soil. We have estimated the 6 components of strain (3 normal strains, 3 shear strains) from small strain solid mechanics theory based on the differential displacements in the x,y and z directions over the test volume. We then used the strain components to compute the maximum shear strain at a particular instant in time during a blast pulse.

It should be noted that the above strains represent average strains over the soil test volume whose centre is defined at the mid-point between the two accelerometers.

Using the above approach for each blast pulse, we have computed average maximum shear strains at the mid-point location between the two accelerometers. The shear strain has then been plotted versus SD in Figure 10 where the hypocentral distance used in calculation of SD corresponds to the distance between the mid-point of the accelerometers and the charge location.

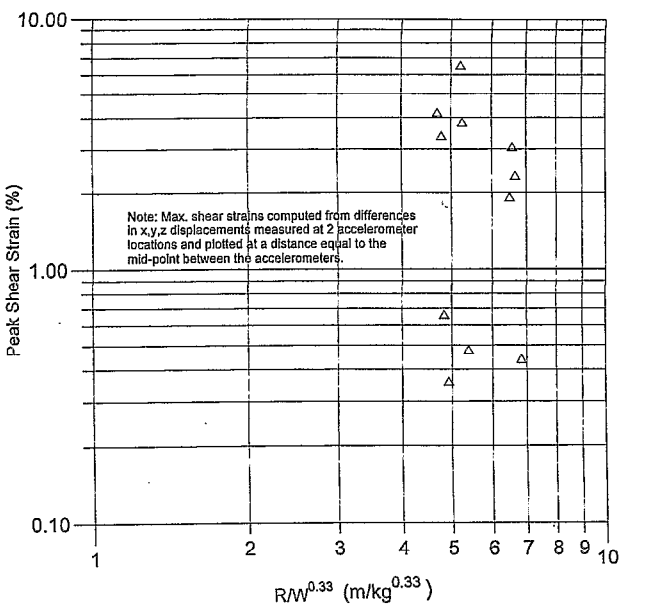

Fig. 10 Maximum average shear strain in the soil test volume versus scaled distance $R/W^{0.33}$ computed for each blast pulse from the accelerometer data.

The average maximum shear strains have been computed to be in the range of 0.36% to 6.5% for SD values in the range of 4.5 to 7. These are rather high shear strains and considered to be representative of strain levels that an extremely strong earthquake affecting the site might produce.

CYCLIC SHEAR STRAIN VERSUS RESIDUAL PORE PRESSURE RATIO

Measured PPR's for both single, two charge and multiple charge detonations obtained at the Boundary Bay (silt) site and another clean sand site located on Annacis Island, B.C. (Gohl, 1998) are shown plotted versus inferred shear strain induced by each blast pulse in Figure 11. The single and two charge data, and methods of data processing and analysis have been reported by Gohl (1999a,b). In the latter case, nonlinear blast analysis based on both spherically symmetric and 3-D finite element models have been used to estimate strains induced by the blasting process within the zone of interest. The blast analysis is used to do a

"signal match" of the measured downhole accelerations at the centre of the test volume of soil. The blast pulse is simulated using an applied blast pressure at the borehole cavity. The analysis incorporates a nonlinear soil stress-strain model with cyclic hysteresis. Knowledge of the soil stratigraphy at a test site and each soil layer's dynamic undrained strength and stiffness (small strain shear modulus G_{max} measured using geophysical methods) is important for this blast modelling. Once the signal match is considered satisfactory, dynamic stresses and strains are computed in the test volume and provide another estimate of peak shear strain induced by a particular blast pulse.

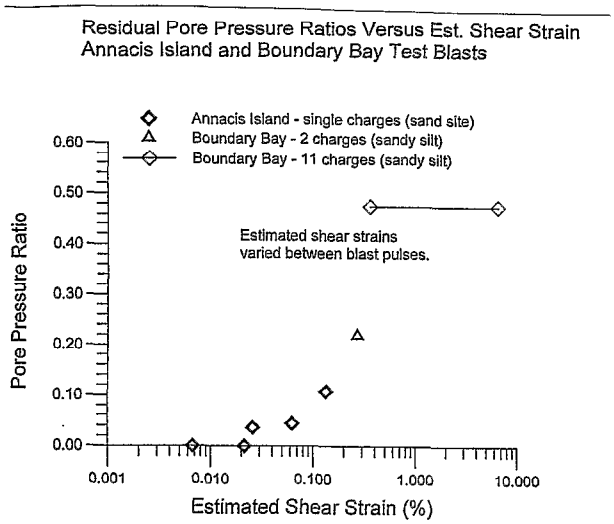

Fig. 11 Pore pressure ratio versus estimated maximum shear strain from single, 2-charge and multiple charge detonations for sand and sandy silt sites.

Figure 11 indicates that the Boundary Bay multiple charge test blast induced variable amplitude shear strain pulses with shear strains in the range of 0.3 to 7%. This is representative of the effects of strong earthquake shaking and covers the range likely for a design earthquake at the Boundary Bay site. Estimated shear strains from the earlier single and two charge tests indicate strain levels in the range of 0.007 to 0.27%. The trend of increasing PPR with shear strain level and number of cycles of shaking is seen. This is analogous to laboratory test data reported by Dobry *et al* (1982) for clean sands which show that PPR increases with shear strain amplitude and number of strain cycles. The field blast data also indicate that significant pore pressure build-up does not occur for shear strain amplitudes less than about 0.02%, in good agreement with data reported by Dobry.

POST-CYCLIC SETTLEMENTS

Following dissipation of excess pore water pressures caused by the test blasts, settlement of the ground occurs. The Sondex settlement gage recorded a total settlement **within** the target layer of 21.3 mm which represents 1.1% of the thickness of the layer. This settlement occurred as a result of the multiple blast field trial and was completed within 7 days following the trial. The settlement of the base of the Sondex gage, seated in the underlying dense sand at the 15 m depth, was 12 mm over the entire period of testing.

The surface settlement was indicated as 68 mm over the entire testing period, indicating that the blasting caused volumetric strain of materials over a broad depth range and not just within the target layer between the 10 and 12 m depth.

The target silt layer has corrected Standard Penetration Test resistances $N_{1,60}$ of 5 to 10 based on available CPT data for the site. Correcting for fines content, the "equivalent clean sand" $N_{1,60}$ for the layer is estimated to be in the range of 10 to 17. According to data presented by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992), complete liquefaction of clean sands (i.e. achieving a PPR of 1.0) having corrected Standard Penetration Test resistances ($N_{1,60}$'s) of 10 to 17 would cause vertical strains in the range of 2.0 to 3.5%. Where only partial pore pressure build-up occurs (PPR < 1),as indicated from the field test measurements, the settlement potential is considerably reduced. Ishihara and Yoshimine suggest that for factors of safety against liquefaction of 1.61 corresponding to a PPR of 0.625 (including the effects of the single and multiple hole test blasts), the post-earthquake vertical strain potential within clean sands having the above corrected $N_{1,60}$ values would be 0.2% or less. Due to the high silt content and compressibility of the target layer, larger strain potentials would be expected as were in fact observed.

The advantage of the *in situ* liquefaction test is that vertical strain potentials are measured corresponding to both the amount of pore pressure achieved (full or partial liquefaction), and the complex variability in soil compressibility and permeability within a native soil deposit.

SUMMARY AND CONCLUSIONS

A field test method is proposed to evaluate the potential for cyclic pore pressure generation using the controlled detonation of explosives. Small charges are detonated sequentially which generate a number of shear strain pulses in an instrumented test volume of soil. Pore pressures, accelerations and soil deformations are measured within the test volume. The data are used to infer the relationship between shear strain amplitude, number of strain cycles and residual pore pressure generation, as well as post-cyclic soil deformations following pore pressure dissipation.

The method is considered will supplement existing penetration test-based methods of seismic liquefaction evaluation, and cyclic laboratory tests. The development of a downhole *in situ* liquefaction test is considered highly advantageous in evaluating the liquefaction potential of problematic soils such as low plastic silts, and sand and gravel deposits. Use of the method at a clean

sand site would be beneficial to compare the field test predictions against liquefaction evaluation procedures based on the use of more traditional penetration testing.

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