Explosive Compaction of Granular Soils and In Situ Liquefaction Testing Using Sequential Detonation of Explosives

W. BLAIR GOHL*, TODD MARTIN and RON J. ELLIOTT*****

*****_Explosive Compaction Inc., 14613 64th Avenue,_
Surrey, British Columbia, Canada, V3S 1X6
BGohl@explosivecompaction.com

****** _AMEC Earth and Environmental Ltd., 2224 Douglas Road,_
Burnaby, British Columbia, Canada, V5C 5A9
todd.martin@amec.com

*** Pacific Blasting and Demolition Ltd., 3183 Norland Avenue,
Burnaby, British Columbia, Canada , V5B 3A9
ron@pacificblasting.com

Abstract

This paper documents the successful application of explosive compaction technology for the densification of granular soils using case history data from a number of sites. The data include densification projects in urban areas, for dam foundations and for offshore structures. The paper provides general background information on the theory of explosive compaction of soils, design considerations, and the densification results achieved.

An offshoot technology using the sequential detonation of explosives is also described to evaluate in situ the seismic liquefaction potential of soil. This 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 (or cyclic shear stress ratios) as a design level earthquake.

Keywords: Explosive compaction; blast densification, liquefaction testing.

Introduction

The sequential detonation of below ground explosives placed in cased boreholes to cause compaction of predominantly granular soils has been used in civil and mining engineering for over 70 years. A history of use of the method has been described by Narin van Court and Mitchell (1994) documenting the application of explosive compaction (EC) by Russian, Dutch, American and Canadian engineers for earth dams (including tailings dams), bridges, buildings, port facilities and offshore oil structures. EC can also be used to cause volumetric compaction of mine tailings to increase tailings pond capacities.

EC is attractive, as explosives are an inexpensive source of readily transported energy and allow densification with substantial savings over alternative methods where depths of densification in excess of about 10m are involved. Readily available drill rigs or vibratory casing drivers are required to install the blast casings, minimizing mobilization costs and allowing work in limited headroom conditions. Compaction can be carried out at depths beyond the reach of conventional ground treatment equipment and, through appropriate selection of drilling methodology, can penetrate coarse grained soils. Compaction can also be carried out below near surface cohesive soil layers (not amenable to vibratory densification) and does not require their removal, unlike other methods such as rapid impact or dynamic compaction.

Most EC has been driven by concerns over static or seismically induced soil liquefaction, and has been generally carried out on loose soils below the water table and to depths of nearly 50 m. However, densification of unsaturated granular soils is also possible. For many projects, where shallow and deep densification is necessary, a combination of deep EC with shallower methods of compaction (e.g. rapid impact or dynamic compaction) provides a cost effective solution.

Methodology

EC involves placing charges at several depths in a borehole, with each charge separated by gravel stemming, over the depth to be densified. An array of multiply decked boreholes is installed and then the charges are fired sequentially in a pattern designed to optimize compaction and to minimize offsite vibrations. With each array of charges detonated, the energy released causes liquefaction of the soil close to the blast point and causes cyclic straining of the soil. This cyclic strain process increases residual pore water pressures and, provided strain amplitudes and numbers of cycles of straining are sufficient, the soil mass liquefies. Liquefaction of the soil, followed by time-dependent dissipation of excess water pressure, causes re-consolidation of the soil mass and release of large volumes of water to the ground surface.

Once an area of ground has been shot and pore pressure have largely dissipated, repeated applications ("passes") of shaking caused by controlled blast sequences causes additional liquefaction and consolidation settlement, depending on soil density and stiffness. Following the first pass of blasting, subsequent arrays of blast holes are detonated to re-compact the locally loosened zones of soil in close proximity to a previous charge detonation. Thus, the pattern of blasting employed is critical in order to promote homogeneity of the densification.

Blast Design

An excellent summary of empirically based, blast design procedures has been provided by Narin van Court and Mitchell (1994). In addition to these empirically based methods, dynamic cavity expansion theory has been applied by the writers for use in practical EC design using a finite element model developed by Wu (1996). The model assumes that a charge detonation may be idealized by assuming a blast pressure-time input applied normal to the surface of a spherical cavity. The charge weight per delay is assumed to be proportional to the size of the spherical cavity with larger charge weights resulting in a larger cavity size and a broader effect of the charge detonation. Nonlinear shear stress-shear strain response of the soil and additional strain rate dependent viscous damping is simulated in the Wu model. For a given soil layering profile, defined by dynamic undrained shearing strengths, small strain shear stiffnesses, and compression and shear wave velocities, the program outputs dynamic shearing strains, ground accelerations and velocities, plastic volume change potential and residual pore water pressures for both single and multiple charge detonations. Parameters used in the Wu model are calibrated based on initial estimates of the relative densities of the granular soils and analysis of single and multiple-hole test blasts at a site in question.

Sand behavior observed in cyclic laboratory testing has shown that residual pore pressure build-up increases with increasing shear strain amplitude and number of strain cycles (Dobry et al, 1982). The Dobry data for various sand relative densities and confining pressures can be used to estimate the shear strain amplitude and number of strain cycles required, induced by a series of charge detonations, to cause sand liquefaction at a location of interest. The Wu model then allows one to select the charge weights per delay, blast hole spacings and number of charges to be detonated sequentially necessary to achieve target shear strain levels and liquefaction within an array of blast holes.

In summary, application of cavity expansion theory indicates: (a) the accumulated shear strains induced by sequential blasting as well as large mean stress changes near a blast hole govern the amount of residual pore pressure and cumulative settlement achieved; (b) multiple cycles of blasting will be more effective than single cycles; (c) the zone of influence of a given charge detonation increases as the size of the cavity increases (larger charge weight per delay), but the radius of the zone of disturbance caused by a given charge will depend on the mean confining stress and the strength and stiffness of the soil surrounding it; and (d) it is necessary to increase the charge weight as the depth increases. Dimensional analysis suggests that charge weight should increase proportionally to the square root of depth (Gohl et al, 2000).

Vibration Control

It is desirable to maximize the charge weight per delay in order to employ the broadest blast hole spacing but achieve acceptably large radii of liquefaction and zones of settlement around a blast hole. However, the charge weights per delay must be limited to keep offsite vibrations to below acceptable limits. Also, when blasting is carried out on or adjacent to slopes, blast patterns are adjusted to restrict the zone of residual pore water pressure build-up and minimize the risk of slope instability.

Design of the appropriate charge delays is carried out using the following process: (a) Ground vibration time histories are determined at a particular location of concern remote from the blast point due to a single charge. This is best done using field measurements but can also be carried out theoretically; (b) The frequency range and amplitudes of potentially damaging vibrations are selected based on structural vibration theory, and (c) The effects of sequential charge detonations from a decked array of blast holes are assessed by a simple linear combination of the single charge wave trains in which time delays between decks and between adjacent blast holes are varied. Optimum blast delays are then determined to minimize the peak particle velocity or the vibration energy content in the frequency range of interest.

Explosive Product Selection

The explosive types used in previous work have varied, and are selected based on safety, suitability for underground work and being loaded under water, resistance to "dead pressing" and/or sympathetic detonation, handling convenience, and producing negligible groundwater quality effects. Most recent projects have involved the use of pre-packaged cylindrical cartridges of blasting emulsion that may be loaded in PVC casings of up to 100mm diameter. Preference is given to explosives which have a lower velocity of detonation which tends to promote heave and shearing of the soil mass and also reduces peak dynamic blast pressures in close proximity to a charge which can be damaging to instruments (pore pressure transducers or accelerometers) located near a charge detonation.

Selection of explosive detonators is made considering the accuracy of firing times and their high shock resistance. While electric and non-electric, long period detonators have been used for many projects, their accuracy is typically limited to about 10% of their rated delay times. Where more accurate control of firing times is required in blasting carried out close to vibration-sensitive structures, preference is given to use of precision electronic detonators which are computer controlled and have firing time accuracies of 1 to 2 msec.

Instrumentation

Instrumentation on blasting projects typically includes: (a) surface geophones to measure vibration response at critical locations ; (b) pore pressure transducers to measure residual pore pressures generated by the blasting; (c) hydrophones installed in water-filled casings near blast zones used to identify charge detonations; (d) Sondex tubes to measure settlements with depth in a soil profile following blasting; (e) ground surface settlement measurements; and (f) inclinometers where blasting is carried out near slopes to measure slope movements (if any). In some applications (Elliott et al, 2009), where additional confirmation of explosive detonations has been required, electronic coaxial cables have been installed down blast holes and have been used to measure firing times of explosive decks using high speed data acquisition systems. High speed filming of the firing of non-electric delays can also be employed to monitor charge detonations.

Densification Results Achieved

Standard Penetration Testing (SPT), Becker Penetration Testing (BPT) or electronic Cone Penetration Testing (CPT) is commonly used to assess the improvement in soil density obtained following EC. For sand and silt/sand sites, use of CPT is considered to provide the most reliable and reproducible results. However, it is well known that CPT tip resistance (q_c) will be reduced by increasing fines content of the soil due to its effect on frictional and compressibility characteristics of the soil and this must be considered in evaluation of the effectiveness of densification.

It has also been observed for vibratory densification processes that there is a considerable time effect on the values of penetration resistance. In some cases, penetration resistance reduces after blasting even though substantial settlements have indicated that densification has been achieved, and then resistance continues to climb weeks following blasting (Gohl et al, 1996). Blasting causes destructuring of soils and erases the influence of soil ageing which was shown by Skempton (1986) to have considerable effect on penetration resistance. Prior to blasting, the penetration resistance would include the effects of ageing. Immediately after blasting, these effects will have been destroyed and q_c values measured after the blast could be lower than pre-blast values even though settlement measurements indicate volumetric compaction has been achieved. We consider that the main reason for these time effects relates to increases in soil stiffness and lateral stress under sustained confining stress due to soil particle creep. Other possible causes relate to the physical-chemical bonding at particle contact points.

A summary of EC case histories with which the writers have been involved documenting densification results achieved are presented in Table 1. Our experience indicates that post-blast settlements are greater than settlements to be expected following earthquake-induced liquefaction (Gohl et al, 2000) due to the strong shaking and liquefaction induced by the blasting. This indicates the benefits of EC in "pre-conditioning" the soil to the effects of earthquakes and minimizing post-seismic settlement. Relative densities (D_r) in clean granular soils are typically increased to values in the range of 65 to 75% corresponding to stress normalized, average q_{c1} values in the range of 100 to 140 bars for fines contents of up to 10%. Correlations between relative density and q_{c1} will depend on sand gradation and soil compressibility and need to be assessed on a site-specific basis. The effect of fines content (estimated from CPT data using NCEER, 1997) on q_{c1} for a silt/sand tailings from the Guindon Dam site is shown in Figure 1 which indicates the typical reductions in q_{c1} to be expected with increasing fines content due to the influence of fines on soil friction angle, compressibility and shear modulus. This trend is to be expected for all methods of vibratory densification. For silty sands, gravelly sands or sands having high compressibility for which correlations between q_{c1} and seismic liquefaction susceptibility are not well established, site-specific testing is often required. A case history of this approach is discussed subsequently.

Table 1. Summary of Case Histories.

Project	Maximum EC Depth	Soil Type	Charge Density (gm/m^3)	Avg. Vert. Strain (%)	Avg. q_{c1} (bars)*	Avg. Final D_r**
Molikpaq I	21	Dredged clean sand	36	6.4	140 (120-175)	75
Molikpaq II	41	Dredged clean sand	52	8.0	120 (80-150)***	70
Trans-X	15	Alluvial sand/silt	37	3.7	100 (80-120)	67
Quebec SM-3	20	Clean alluvial sand	38	6.2	110 (95-135)	65
Coldwater Ck.	40	Volcanic debris	42	3.6	130 (105-140)	65
Kitimat Hosp.	12	Alluvial sand/silt	48	3.5	125 (100-135)	75
Guindon Dam (Area 1)	17	Sand/silt tailings	79	10.0	120 (90-150)	75
Seymour Dam	27	Sand, gravel, cobbles	125	3-15	n/a	n/a

n/a = not available (site owner has not released penetration resistance data – blast design carried out by others)
* in cleaner sand layers averaged over all layers. Figures in parentheses show typical range in data.
** estimated as $D_r = \sqrt{(N_{1,60}/40)}$ for un-aged clean sands based on data from Skempton (1986) and $N_{1,60} = q_{c1}/\beta$ where β has site-specific values, typically ranging between 5 and 7
***lower range of values at perimeter of sand core of offshore oil platform due to blast wave reflections from hull of structure.

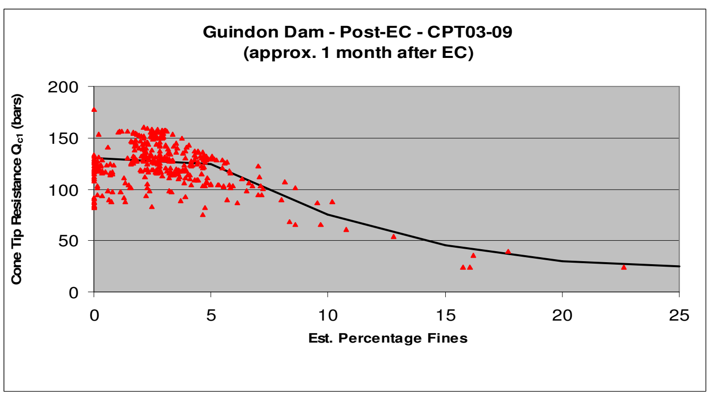

Figure 1: Influence of estimated fines content on stress normalized, corrected cone tip resistance q_{c1} following EC in sand and silty sand tailings (Guindon Dam – Area 1).

Liquefaction Testing Using Sequential Detonation of Explosives

Planning for eventual closure of the Kemess Mine tailings dam in Northern British Columbia led to concerns about the potential for elevated pore water pressures following a design earthquake event in the upstream tailings in the near vicinity of the south abutment of the centerline-raised tailings dam. Due to the presence of fractured, hydraulically-conductive bedrock in the south abutment, high post-earthquake pore pressures within the impounded tailings was a concern. Extensive soil sampling and cone penetration testing was carried out in the tailings. This indicated that the tailings comprised silty sands with variable fines contents, typically less than 25 percent, and depths of tailings of up to 55m. CPT data (2006 series) indicated relatively low tip resistances (stress normalized q_{c1} values averaging 50 bars), suggesting relatively low seismic liquefaction resistance and the potential for significant pore pressure generation during the design earthquake (M6, peak ground acceleration on rock of 0.19g). However, the CPT data also indicated strong dilative response (negative dynamic pore pressures) of the tailings during cone penetration and that the tailings were moderately compressible. These factors could lead to underestimates of the seismic liquefaction triggering potential based on traditional evaluation procedures using CPT data (NCEER, 1997).

A field test was devised involving the sequential detonation of explosives within a single blast hole containing 6 decks of explosives to test the cyclic pore pressure generation characteristics of the tailings below the 25m depth. Two single hole tests were carried out at a distance of about 90m upstream of the crest of the dam. Maximum charge weights per delay were selected to keep vibration and strain levels within the core of the dam to below acceptable limits. Based on initial interpretation of the 2006 CPT data and previous experience with blasting in looser tailings, the charge weights per delay selected were expected to induce liquefaction within about 15m of a blast hole and lower levels of excess pore pressure at greater distances. Pore pressure transducers were installed at distances as close as 12m to a charge detonation. Ground surface vibrations were also monitored using triaxial geophones at various distances from a blast hole.

The measured attenuation of ground surface vibrations was later used to calibrate a nonlinear, axisymmetric finite element model to compute blast-induced strains and stresses with distance from a blast hole. A total stress model of 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. The analysis indicated cyclic shear stress ratios (maximum shear stresses divided by vertical effective stress) in the range of 0.05 to 0.21 (average about 0.12) and cumulative shear strains of about 1.2% from the 6 charge detonation, encompassing the ranges expected under design levels of seismic shaking.

The cyclic pore pressure response at a distance of 12m from the second test blast S-2 and at a depth of 34.5m is shown in Figure 2. The gradual increase in residual pore pressure with each of the 6 charge detonations is shown. The maximum pore pressure increase at the piezometer 5B location occurred following the fourth charge detonation, which was located closest to the piezometer where maximum incremental shear strains from the blast pulse developed. An excess pore pressure of 3.3 lbf./sq. in. (22.7 kPa) relative to the initial pre-blast pore pressure occurred at the end of the 6 blast pulses. This corresponds to an excess pore pressure ratio (= excess pore pressure divided by initial vertical effective stress prior to blasting) of 0.05.

A pore pressure generation model developed by the senior 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 theoretically the build-up of residual pore pressure during the 6 charge detonation. The results of the computations are shown in Figure 3. The model requires as input the total mean stress and shear strain time histories induced by blasting, which were computed from the FE model described previously, as well as one dimensional compressibility and shear strain-residual pore pressure generation parameters deduced from laboratory testing discussed subsequently. Background to the key features of the model is described by Fragaszy and Voss (1986) and Finn (1979).

Examination of Figure 4 indicates that incremental changes in total mean stress during a load-unload cycle (incorporating hysteretic soil skeleton volume change) can be responsible for a small amount of residual pore pressure build-up during blasting, but that the majority of residual pore pressure results from shear strain increments during each blast pulse. The latter is similar to the mechanism normally 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.

Following the first and second test blast, no evidence of soil liquefaction or significant ground surface settlement was observed. Pore water pressures were observed to gradually climb over time at the piezometer location. This was considered due to dilation of the tailings generated by the strong shearing caused by the blasting, resulting in pore water flows towards the blast holes and piezometers from surrounding areas. This behavior is consistent with the strong dilative behavior observed during cone penetration testing. Peak residual pore pressures occurring several hours after blasting divided by pre-blast vertical effective stresses are plotted in Figure 4 versus scaled charge weight and distance $R/W^{0.33}$. Here R is the hypocentral distance between a piezometer and the mid-depth of all the charges in the blast hole and W is the average charge weight per delay (22 kgf.). The Kemess data indicate much lower residual pore pressure generation compared to other data obtained from blasting in loose to medium dense silt/sand tailings, and suggests that the Kemess tailings are denser than interpreted from the 2006 CPT data if considering tip resistance in isolation from negative dynamic pore pressure response, as is typical practice for piezocone-based evaluation of seismic liquefaction potential. One dimensional compression testing of the tailings samples also indicated moderately compressible sand, which would explain in some part the dichotomy between the pore pressure response of the tailings (to piezocone penetration and the test blasts) and the tip resistance.

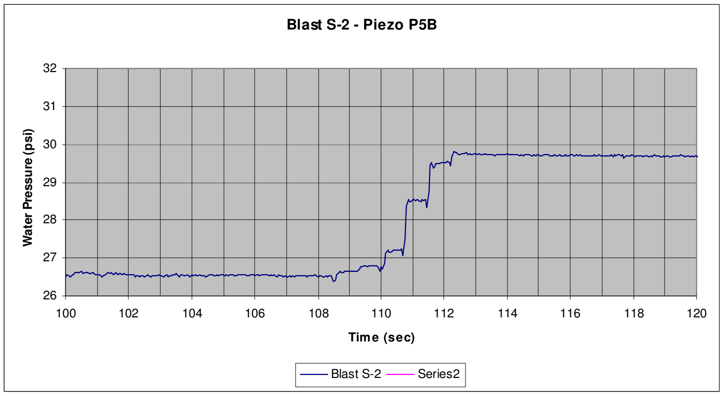

Figure 2: Residual pore pressures (lbf/sq.in. units) versus time during 6 charge detonation sequence during test blast S-2.

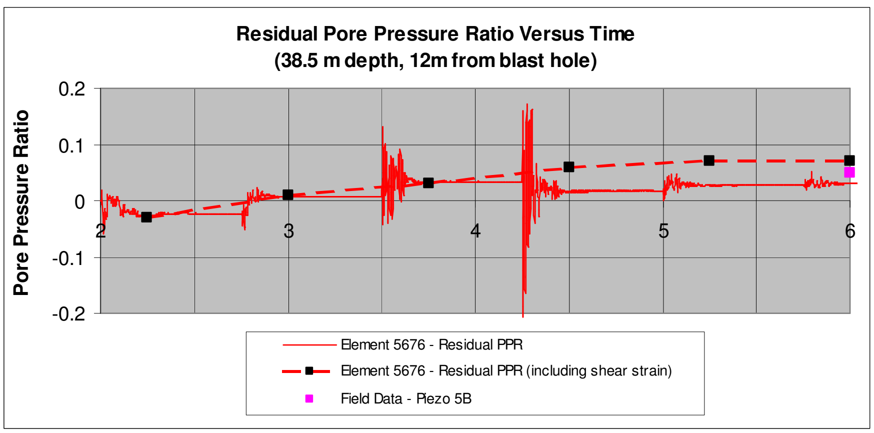

Figure 3: Computed residual pore pressure ratios versus time from 6-charge detonation. The solid lines are PPR's resulting from total mean stress changes and the solid data points are the residual PPR's including shear strain increments from each blast pulse.

The lower than anticipated pore water pressures generated from the blasting prompted additional high quality tube sampling (including Standard Penetration Testing) and cyclic laboratory testing of the tailings in 2008. The tube samples were observed from gamma ray scans to be of good quality and indicated relative densities of the tailings prior to reconsolidation of approximately 75%, indicative of dense tailings. Constant volume, cyclic simple shear tests on the samples at cyclic shear stress levels representative of design earthquake shaking (6 cycles at an average cyclic shear stress ratio of 0.10) indicated a low excess pore pressure ratio of 0.15 at the end of cycling, entirely consistent with the blast test results. There were discrepancies between the Vs measurements of the lab samples and in situ measurements, raising questions about the representativeness of the tube samples and whether densification could have occurred during sampling. Additional seismic CPT work is planned to investigate this issue. Regardless of the results from the cyclic laboratory testing and whether this was carried out on truly undisturbed samples, the balance of evidence from the blasting tests and the more traditional approach of evaluating seismic liquefaction susceptibility from SPT or CPT-based measurements indicates very small potential for pore pressure build-up.

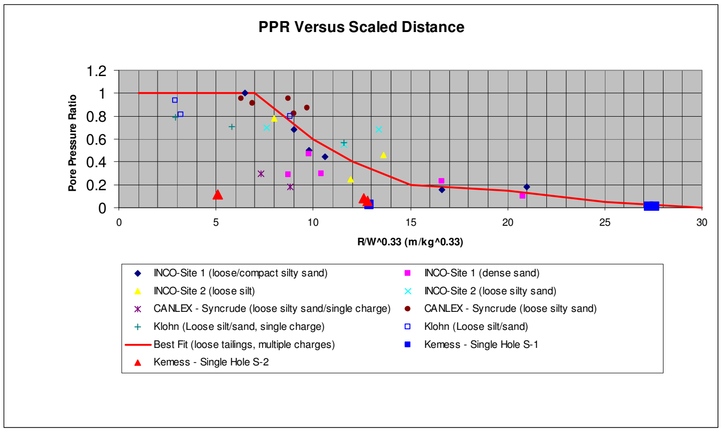

Figure 4: Plot of pore pressure ratio versus scaled distance ($R/W^{0.33}$) from Kemess blast trials and comparison with blast data from other tailings sites.

Subsequent SPT testing of the tailings in 2008 (distinct from the earlier 2006 CPT data) indicated energy corrected SPT $N_{1,60}$ values in the range of 13 to 23 with an average value of 16 (or average, equivalent clean sand $N_{1,60}$ values of 21). These SPT values indicate that dilative tailings response should be expected. Subsequent analysis of seismic site response under design levels of earthquake shaking, in combination with the latest 2008 series of SPT data, indicates acceptably high factors of safety against liquefaction triggering of about 2. As a result of the pore pressure generation behavior deduced from the blast tests, the cyclic laboratory testing and the latest series of SPT testing, the risk of significant excess pore pressure build-up during earthquake shaking in the tailings and associated fines migration was deemed to be acceptably small. Planning for tailings dam closure is currently underway.

Conclusions

Our experience with EC indicates it is safe, effective and predictable, producing volume changes in excess of those to be expected for strong earthquake shaking and post-blast penetration resistances similar to those to be expected for other vibratory methods of densification. Interpretation of post-EC penetration resistances must be made with due attention to the effects of time, fines content and soil compressibility. Current numerical simulators are adequate for blast design, although test blasts remain very desirable at any site before production blasting.

The use of the sequential detonation of explosives represents a convenient down hole energy source for use in creating cyclic shearing strains and stresses representative of a design earthquake at a site and testing the residual pore pressure generation potential of the subsoils. The method 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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