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ORIGINAL RESEARCH PAPER

Investigation of explosive compaction (EC) for liquefaction mitigation using CPT records

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Abstract Explosive compaction (EC) or Blast densification (BD) has been realized as an efficient technique for soil improvement and mitigation of the liquefaction potential in loose saturated sands. Due to providing continuous and precise records, Piezocone (CPTu) is the most applicable in situ test in geotechnical practice for evaluation of liquefaction potential. In this research a data bank including eight case histories in different locations has been compiled for investigation of EC effects on mitigation of loose sands instability. The sites geomaterials are in the category of fine to medium sand, silty sand and mixture of sand and gravel with relative density between 30 and 60 % and thickness of 5–40 m. Four CPT-based criteria have been used including cyclic stress ratio approach, cone tip resistance (q_c) variations before and after modification, Q_{tn} and q_{c1N}, and soil behavior classification charts. Analyses have shown that due to EC the state of soil changes from loose to dense, the contractive behavior of sands changes to dilative, and the liquefaction potential diminishes. Also, by using soil behavior classification charts pre and post explosion, it can be observed that improved soils are not in the liquefiable zone, anymore. This improvement has a significant effect on layers where located in deeper zones, whereas in surface layers in some cases, liquefaction phenomenon has been observed. Moreover, by blasting in two stages between first and phases for boreholes, liquefaction potential decreases significantly.

Keywords Deep soil improvement · Explosive compaction (EC) · Loose deposits · Liquefaction · CPT records

1 Introduction

Explosive compaction (EC) includes the use energy from buried explosive materials in order to increase density of loose saturated sand deposits. This technique for compaction has been used successfully for a wide range of non-cohesive soils including alluvial

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deposits, hydraulically deposited fills usually in thermal power plants or mining waste sites, and saturated layers with fine granular sediments (Hausmann 1990; Narin and Mitchell 1994; Gohl et al. 2000). Also, EC is proposed as one of the common methods for densification of loose soils existing under the foundation of dams and sea beds for building ports and artificial islands. The most effective parameters in the performance of EC are: geotechnical conditions of surrounding soils, transformed energy from explosive material to soil, charge weight per borehole, explosion network plan, and distribution pattern of charge along borehole (Shakeran et al. 2012).

In the 1980's, the ability of EC as a deep improvement technique for densification of alluvial deposits with thickness of 15 m was observed in Jebba dam project (Solymar 1984). The effectiveness of this method was confirmed in densification of hydraulic deposits in Capcode Channel in Texas and Almond Dam in New York (Narin 1994; Gandhi et al. 1999).

In the 1990's and 2000's this method was used for improvement of bridges abutment backfill, dam foundations, reducing the size and increasing the capacity of waste pools and also mitigation of liquefaction potential (Hachey et al. 1994). Another successful use of this method can be observed in the compaction of alluvial deposits in order to increase resistance against liquefaction of soils in Seymour Falls Dam Project in Canada (Murray et al. 2006).

The most appropriate method for monitoring the performance of explosive improvement is using records of CPTu and CPT before and after EC (Robertson and Wride 1998). Cone penetration test, CPT, is used for investigation of EC effectiveness in loose saturated soils due to providing accurate and continuous records, reduction of operator influence on test results, optimized costs, and measurement of excess pore pressure, cone tip, and sleeve resistances in very close depth intervals (Campanella et al. 1986; Robertson 2012).

In this research, the results of CPT have been employed for assessment of liquefaction potential in improved deposits by EC method. Accordingly, CPT records are used as a comparison geotechnical tool. Investigations have been made via four criteria including: Cyclic Stress Ratio (CSR) versus cone tip resistance, q_c variations pre and post explosion, Q_{tn} and q_{c1N}, and a couple of commonly used soil behavior classification charts.

2 Mechanism of explosive compaction

The first explanation of EC was given by Ivanov (1967). Compaction of granular soils due to explosion happens because of transformed impaction and stress shocks of explosion to soils, tendency to liquefaction, and ground subsidence. Generally, EC is execution of boreholes in square or triangle forms within the definite depths and distances, charging of boreholes at one level or multiple levels using explosive materials, and blasting in one or multiple phases with time sequences in the plan and profile. The effective parameters in the design of EC are: explosive material weight per borehole, explosion network plan, distribution pattern of explosive material in height, arrangement of each borehole explosion, and timing of each phase explosion.

Due to explosion, pore pressure builds up and liquefaction occurs. Due to liquefaction large settlement in modified soil can be observed. Figure 1 illustrates wave propagation and the effect of explosion on ground subsidence, soil liquefaction, and sand boil.

The explosive materials that are used in EC release energy in two different stages, the energy of explosion shock and the energy of gas expansion. After explosion, the energy

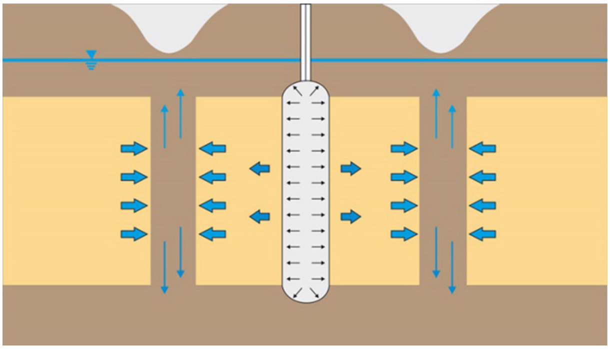


Fig. 1 Wave propagation caused by explosion shock, soil liquefaction, and settlement

between explosion source and supposed point of surrounding soil will be depreciated. The depreciation rate of energy is related to various factors including the distance of target point from explosion center and charge weight. The function which describes the energy depreciation is expressed as R_h/W, in which R_h represents the distance of target point from explosion center and W is explosive material weight (Hausmann 1990).

In the explosive compaction, building up the excess pore pressure is the most important part in densification process (Dowding and Hryciw 1986). As mentioned, excess pore pressure is induced in saturated sand due to explosion shock, gas pressure, and volume changes. In the explosion, three stages of pore pressure changes can be observed as presented in Fig. 2 by pressure ratio (Ru) (Charlie et al. 1985). These stages include: Peak pore pressure that is induced because of explosion shock, residual pore pressure, and depreciation of induced pore pressure.

Usually, residual pore pressure is less than peak pore pressure or dynamic pore pressure. This pore pressure has long stability duration and can remain from some minutes to a few days regarding the soil permeability and fabric (Narsilio et al. 2009). While the effective stress becomes zero due to building up of pore pressure, liquefaction may occur.

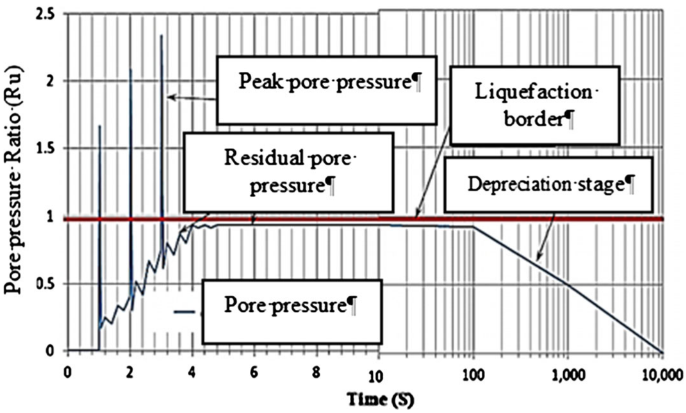


Fig. 2 Three stages of pore pressure changes due to explosion

3 CPT-based evaluation of liquefaction phenomena

Loose saturated sand deposits have tendency to densification and reduction of volume due to vibration of earthquake or any other dynamic loading. Researches indicated that volume change due to liquefaction and settlement of loose sand is approximately 3–5 % of soil thickness (Ishihara and Yoshimine 1992). If the earthquake duration is too short comparing to the duration of soil drainage, rapid drainage and volume reduction are not possible; therefore, pore pressure will build up. Liquefaction is defined as the transformation of granular materials from solid to liquefied state as a consequence of increased pore pressure and reduced effective stress due to seismic shaking.

This paper is concerned with CPT-based evaluation of soil liquefaction potential. Accordingly, four approaches are used for evaluation of liquefaction potential as follows:

3.1 Cyclic stress ratio (CSR) versus q_c chart

An analytical approach for liquefaction potential evaluation is calculating a factor of safety that can be achieved by Eq. 1

$$FS_L = \frac{CRR}{CSR} \tag{1}$$

where CSR is cyclic stress ratio induced in soil by earthquake and CRR is Cyclic Resistance Ratio. CSR is calculated by using Seed's method (1971) as proposed in Eq. 2

$$CSR = \left(\frac{\tau_{ave}}{\sigma'_{v0}}\right) = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) r_d \tag{2}$$

where 0.65 is weighing factor introduced by Seed and Idriss (1971) to calculate the number of uniform stress cycles required to produce the same pore water pressure increase as an irregular earthquake, $\sigma'_{v0}, \sigma_{v0}$ are the effective vertical overburden and total vertical overburden stresses respectively, $a_{\max}$ is the Peak Horizontal Ground Acceleration (PGA), and r_d is a stress reduction coefficient recommended by NCEER in 1997 (Youd et al. 2001).

There are several methods to evaluate the $CRR_{7.5}$ from CPT data. In this paper Seed's method has been used. This approach is based on the SPT records and $CRR_{7.5}$ liquefaction curves versus corrected SPT blow counts, which have been converted to $CRR_{7.5}$ liquefaction curves versus corrected CPT tip resistance (Seed and De Alba 1986; Eslami et al.2014). The drived formula is as following:

$$CRR^{7.5} = \begin{cases} 0.833\left[\frac{q_{c1f}}{1000}\right] + 0.05 & for\ q_{c1} < 50 \\ 93\left[\frac{q_{c1f}}{1000}\right]^3 + 0.08 & for\ 50 \le q_{c1} \le 50 \end{cases}$$

$$CRR = CRR^{7.5} \times MSF \tag{3}$$

where MSF is Magnitude Scaling Factor and q_{c1f} is measured corrected cone tip resistance for fines content. The $CRR_{7.5}$ liquefaction assesment for CPT records, as for the SPT based curves, for clean sand. Therefore, cone tip resistance values of soil containing fines have to be increased to take into account the higher liquefaction resistance as presented in Eq. 4.

$$q_{c1f} = q_{c1} + \Delta q_{c1} \tag{4}$$

where Δq_{c1} is fine content correction and q_{c1} is the corrected for overburden pressure by Eq. 5

$$q_{c1} = C_q.q_c \tag{5}$$

where q_c is the measured tip resistance in MPa and C_q is given by Eq. 6

$$C_q = \frac{1.8}{0.8\left(\frac{\sigma'_{v0}}{P_a}\right)} \tag{6}$$

where σ'_{v0} is the effective vertical overburden stress in KPa, and P_a is reference pressure equal to one atmosphere i.e. 100 kPa.

3.2 Determination of liquefaction potential index (LPI)

Liquefaction potential index (LPI) is a single-valued parameter to evaluate regional liquefaction potential. LPI at a site is computed by integrating the factors of safety (FS) along the soil depth up to 20 m depth. A weighting function is added to give more weight to the layers closer to the ground surface. The level of liquefaction severity with respect to LPI as per Iwasaki et al. (1982) and Luna and Frost is given in Table 1.

The liquefaction potential index (LPI) proposed by Iwasaki et al. (1978, 1982) is expressed as follows:

$$LPI = \int_0^{20} F(z).\omega(z)\,dz \tag{7}$$

where z is depth of the midpoint of the soil layer (0–20 m) and dz is differential increment of depth. The weighting factor, $\omega(z)$, and the severity factor, F(z), are calculated as per the following expressions:

$$F(z) = 1 - FS \quad for\ FS < 1 \tag{8}$$

$$F(z) = 0 \quad for\ FS \geq 1$$

$$\omega(z) = 10 - 0.5za \quad for\ z < 20$$

$$\omega(z) = 0 \quad for\ z \geq 20$$

3.3 q_c variations pre and post modification

The cone penetrometer test is economical, supplies continuous records with depth and allows a variety of sensors to be incorporated with the penetrometer. The numerical values produced by the cone test have been used as input to geotechnical formula, usually of empirical nature to determine capacity and settlement of shallow and deep foundations, and for soil profiling. CPT also has become the most common field investigation method for compaction projects and gently replacing the standard penetration test (SPT) which previously was the dominate in situ testing method for this purpose.

Table 1 The level of liquefaction severity

LPI	Iwasaki et al. (1982)	Luna and Frost
LPI = 0	Very low	Little to none
0 < LPI < 5	Low	Minor
5 < LPI < 15	High	Moderate
15 < LPI	Very high	Major

Many researchers have proposed different classification for soil behavior based on q_c value. In these approaches the state of soil (loose, medium, and dense) can be determine by q_c measurement. In Table 2 one of these approaches for soil classification is presented (Moseley and Kirsch 2004). Soils that are in loose and very loose sands category i.e. with q_c less than 5 MPa, have tendency to liquefaction in saturated state and must be modified.

In this research, CPT records before and after blasting are used for evaluation of EC performance. For this purpose, the penetration records before and after blasting were scanned and digitized.

3.4 Q_{tn} and q_{c1N} criteria

The dilation behavior of sand is affected by grain size, density, and confining pressure. It is expected that the factors affecting dilation behavior also affect measured cone tip resistance, q_c. The border between dilation and compression behavior based on CPT results is determined by Q_{tn} according to Eq. 9

$$Q_{tn} = \frac{Q_t}{(\sigma'_V)^{0.65}} \quad (9)$$

where Q_{tn} is normal cone tip resistance, σ'_v is effective vertical stress (bar), and Q_t is corrected cone tip resistance because of pore pressure.

Sladen and Hewitt (1988) suggested that for sands with Q_{tn} less than 70, the soil will show compression behavior and for soil with Q_{tn} more than 70, the soil has dilation behavior.

Campanella and Kokan (1993) by studying the RCPTu results in different cases expressed that sands with Q_{tn} more than 55 under cyclic loading will show compression behavior and have tendency to liquefaction.

Robertson and Wride (1998) proposed corrected cone tip resistance as Eq. 10

Table 2 Soil classification for sands based on q_c (Moseley and Kirsch 2004)

State of soil	q_c (kg/cm^2)	Relative density (D_r) %
Very loose	<5	<15
Loose	50–100	15–35
Medium	100–150	35–65
Dense	150–200	65–85
Very dense	>200	85–100

$$q_{c1N} = \left(\frac{q_c}{P_{a2}}\right) CQ = \frac{q_{c1}}{P_{a2}} \tag{10}$$

where q_{c1N} is corrected cone tip resistance, q_c is measured cone tip resistance, CQ is correction factor which equals to Pa/σ'_v and Pa is atmospheric pressure.

Robertson and Wride (1998) proposed q_{c1N} equal to 75 as the border between loose and dense sands. As they suggested sands with q_{c1N} more than 75 are dense and are not prone to liquefaction. For these sands under cyclic loading, effective stress will not become zero and volume changes will be less than loose sands.

3.5 Soil behavior classification charts and liquefaction zone

Cone penetration test, CPT, and Piezocone, CPTu, allow for the soil type to be determined from the measured values of cone resistance (q_t), sleeve friction (f_s), and mobilized pore pressure in cone tip (u_2). Classification of soils behavior based on CPT and CPTu results has been studied by many researchers and different charts are proposed for soil behavior classification from cone penetration test results. The most commonly used methods for soil classification based on CPTu results are Douglas and Olsen (1981), Campanella et al. (1986), Robertson (1990), Jefferies and Davies (1991), Eslami and Fellenius (2004), and Robertson (1986) methods. In all of the above mentioned charts a liquefiable or collapsible zone is determined based on case history records. The soils in this zone have tendency to liquefaction and large settlement due to liquefaction. In this research these charts are used for a better assessment of soil behavior and liquefaction potential, by using CPT records before and after explosive compaction.

4 Case history records

In order to evaluate the performance of EC as one of the deep improvement methods, a database of practical EC cases was collected and investigated with complete information including geotechnical conditions of improved deposits, design of EC and results of monitoring pre and post execution of explosion. In all of the case histories surface settlement caused by EC is measured and the CPT data is reported before and after execution of explosion.

The improved geomaterials often are in the category of sand with small to medium size, hydraulic and alluvial deposits, silty sands or mixture of sand and gravel and fine percentage was about 5–35 %. Maximum underground water table level was 3 m.

The design of explosive compactions in sites is in the form of square arrangement nets or in triangular form which were performed in one or multiple phases (maximum to 4 phases). The distance of explosion boreholes according to the sites limitation is variable between 3 and 6 m. In this section, sites are introduced and summary information of them is presented in Table 3. As mentioned, cone tip resistance increases after explosion. But this phenomenon does not happen suddenly; therefore, in this research last record of CPT has been used as cone tip resistance after explosion. The last records of CPT in cases were measured 35 to 180 days after explosion in different sites.

Sites No. 1 and 2: Narin (1997)
Bordeaux, France (P6A and P8A)

Table 3 Summary of case records

No	Site	References	Soil layer	Fine (%)	Layer thickness (m)	GWT (m)	Phase of delay	Well arrangement	Goal of improvement
1	Bordeaux (P6A)	Narin (1997)	Sand (SP)	–	10	1.5	1	T	I&L
2	Bordeaux (P8A)	Narin (1997)	Sand (SP)	–	10	1.5	1	T	I&L
3	Quebec SM-3Dam	Gohl et al. (2000)	Fine sand	–	2	3	2	S	L&I
4	South Carolina	Narsilio et al. (2009)	Fine sand	4	5.5	1	4	S	L&I
5	Jebba Dam (zone I)	Solymar, (1984), Solymar et al. (1984)	Coarse sand- gravel	0	15	2	3	S	L
6	Jebba Dam (test)	Solymar (1984), Solymar et al. (1984)	Coarse sand- gravel	0	15	2	3	S	L
7	St. Petersburg I (without delay)	Minaev (1993)	Fine and coarse sand	–	7–7.5	1.2	4	S	I
8	St. Petersburg II (with delay)	Minaev (1993)	Fine and coarse sand	–	7–7.5	1.2	4	S	I

S, square arrangement; T, triangular arrangement; I, increasing bearing capacity and settlement control; L, liquefaction control

In Bordeaux port, five EC tests were performed. In this site hydraulically deposits in SP category were compacted by EC technique. Soil thickness was about 10 m and ground water table was 1.5 m below the surface level. There is no record about settlement in this site after compaction but CPT results before and after explosions are reported.

Site No. 3: Gohl et al. (2000)
Quebec SM-3Dam, Canada

The improved deposit beneath this rock fill dam includes clean sand with thickness of 20 m and density of 45 %. Design of explosion includes using 38 gr/m^3 Emulsion explosive materials in sand deposits. After explosion, the settlement was about 6.2 % of layer thickness and the density was 75 %. The goal of modification in this site was increasing bearing capacity and decreasing settlement and liquefaction potential of soil.

Site No. 4: Narsilio et al. (2009)
South Carolina, USA

The layer of sand deposits in this site was in the depth of 7.5–13 m, with relative density between 20 and 30 % and fine percentage about 4 % and was in the fully saturated state. Figure 3 illustrates the soil condition and settlement measurement of this site. The explosion was performed and designed in 4 phases with checkered arrangement in 8 months. The surface settlement of site after first phase was about 160 mm and in the next three stages were about 120, 120, and 90 mm respectively. The total settlement of surface was 490 mm (9 % of thickness of soil layer). Figure 4 presents explosion pattern distribution in South Carolina site.

Sites No. 5 and 6: Solymar et al. (1984)
Jebba Dam, Nigeria

Jebba rock fill dam was structured on the Niger River with the height of 42 m, upon the alluvial deposits with the thickness about 70 m including clean sand, fine grain soils and granular soils. The EC at this site was performed in three phases with the explosive boreholes in checkered form.

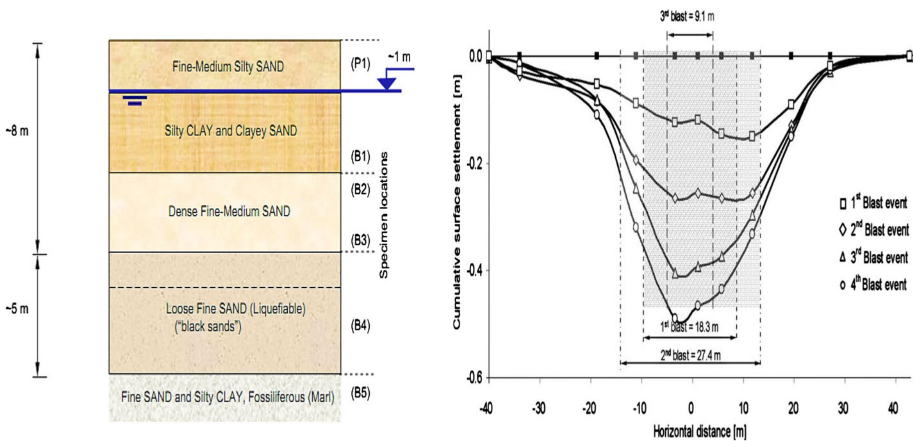
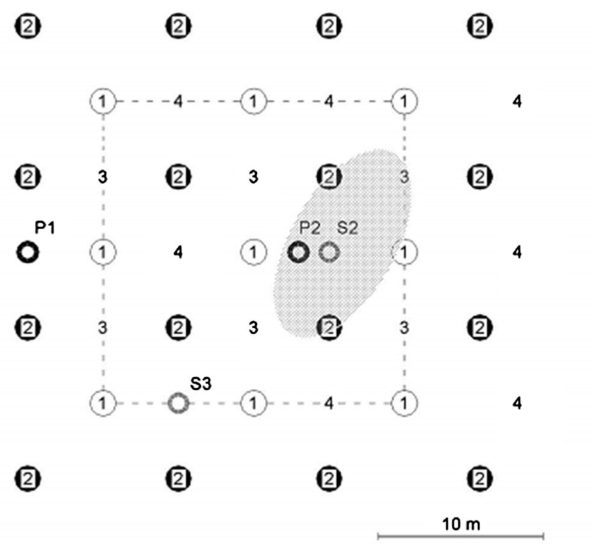


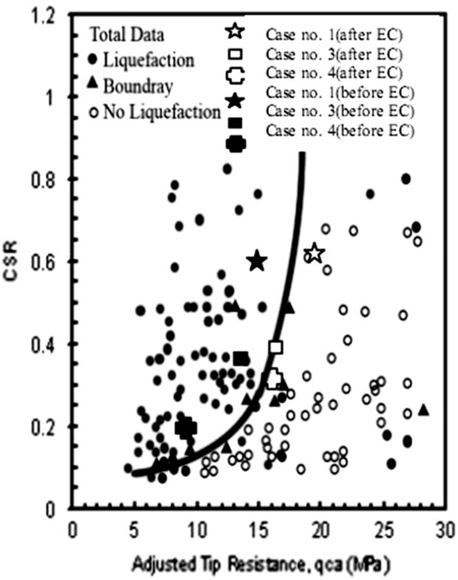
Fig. 3 Soil condition and settlement measurements in South Carolina (site No. 4)



Blast Event	Date	# of charges	Individual charge	Detonation delay
○ 1st	11/21/03	9	19 kg	100 ms between rows
● 2nd	12/23/03	16 (+1*)	34 kg	50 ms between rows
◇ 3rd	06/04/04	6	11 kg	10 min between rows
✱ 4th	08/05/04	7	11 kg	10 min between rows
⊙ P Piezometer	○ S Subsurface settlement	- - - Initial blasting area		

Fig. 4 Explosion boreholes pattern of South Carolina (site No. 4)

Fig. 5 CSR versus q_c value and liquefiable zone (Seed and De Alba 1986)



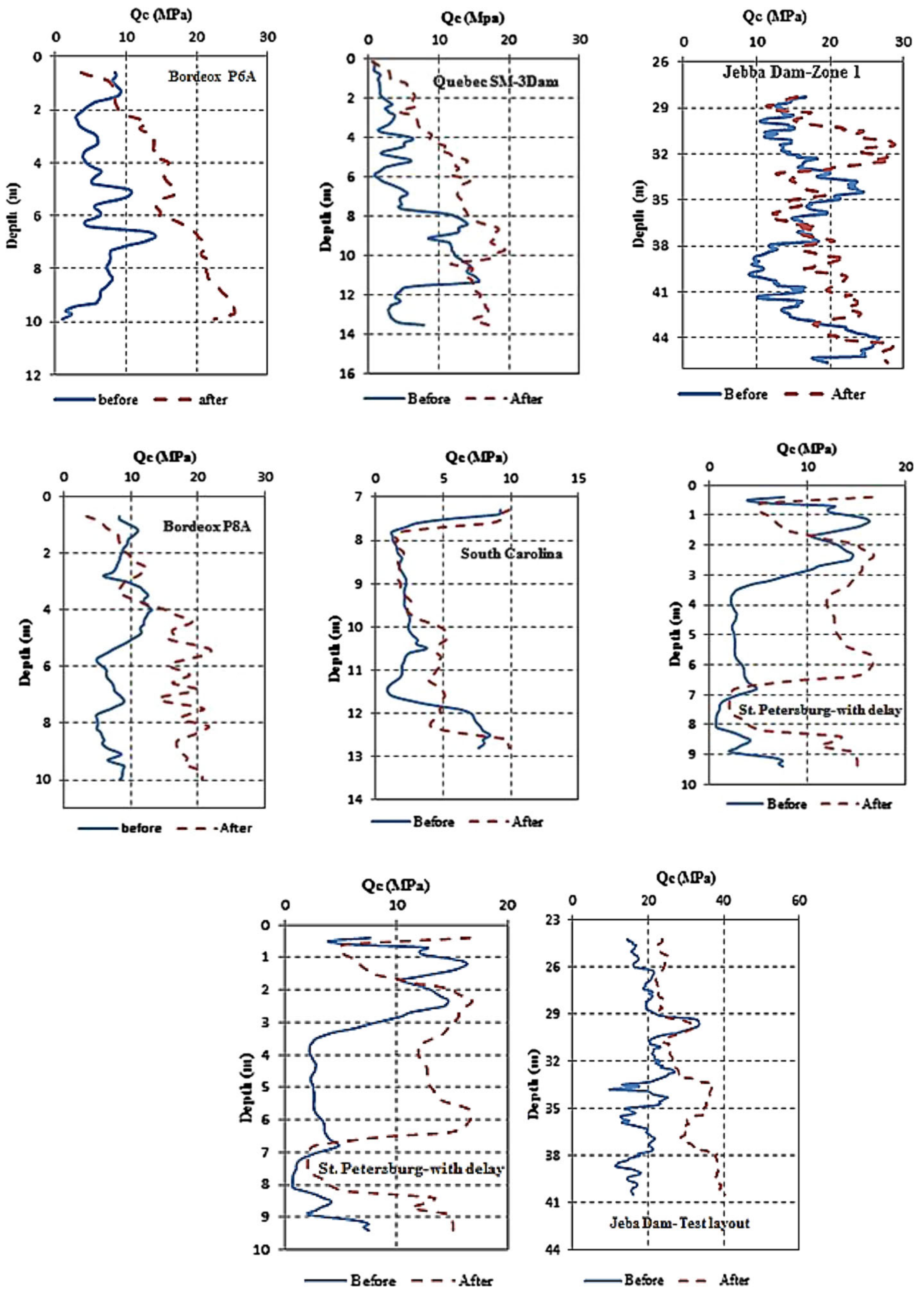


Fig. 6 Comparison of q_c measurements before and after explosion for case histories

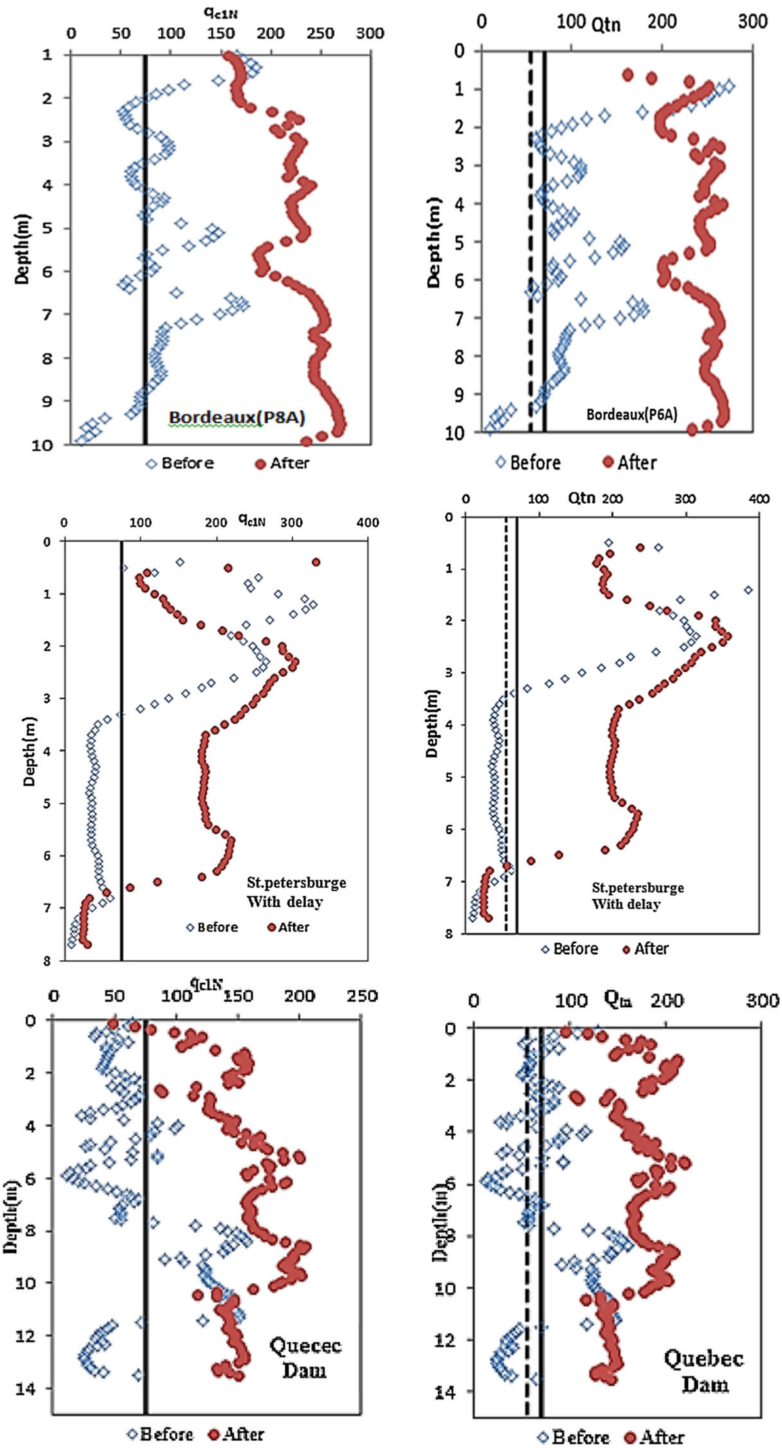


Fig. 7 Variation of Q_{tn} and q_{c1N} before and after explosive compaction

EC performance and efficiency was evaluated by monitoring the surface settlement and comparison of the CPT records before and after the explosion. By the effect of EC in this test, the average final settlement was about 270 mm (reported from phases 1 and 3, 139 and 50 mm, respectively).

Sites No. 7 and 8: Minaev (1993)
St. Petersburg flood-walls (with and without delay of explosion), Russia

To investigate the effect of using delay; site No. 8 between explosions 3–5 min delay was used; between the rows of explosion wells, Minaev (1993) performed two EC tests in St. Petersburg flood-walls system. The EC was designed and performed at site No.7 without delay and with delay at site No. 8.

The explosive compactions at both sites were designed using checkered arrangement and 4 phases were performed. After EC in each site, the settlement of improved layer was measured. By comparison of CPT results before and after explosion, EC performance was evaluated. The average settlement at sites was recorded equal to 210 mm (2.8 % of the improved layer's thickness) and 23 mm (3 % of the improved layer's thickness), respectively.

5 CPT-based interpretation of ground modification by EC

The data from three sites are used for calculation of CSR versus q_c using Seed's diagram (Seed 1986). As presented in Fig. 5, all these sites include soils with high liquefaction potential, but modified soils by explosion are not in the liquefiable zone, anymore. This can underline the effectiveness of explosion compaction technique in diminishing or elimination of liquefaction potential of existed deposits.

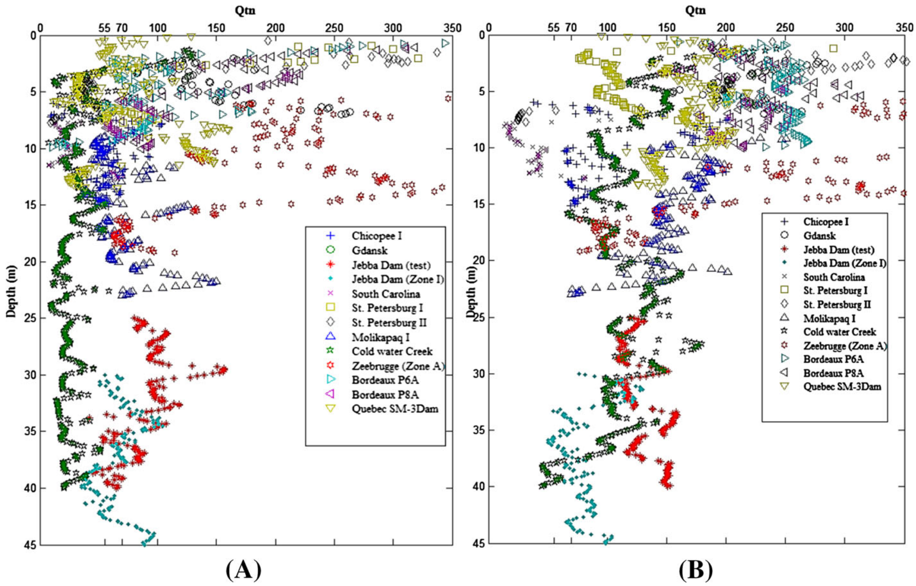


Fig. 8 Evaluation of soil condition for case histories based on Q_{tn} criteria. **a** Before. **b** After explosion

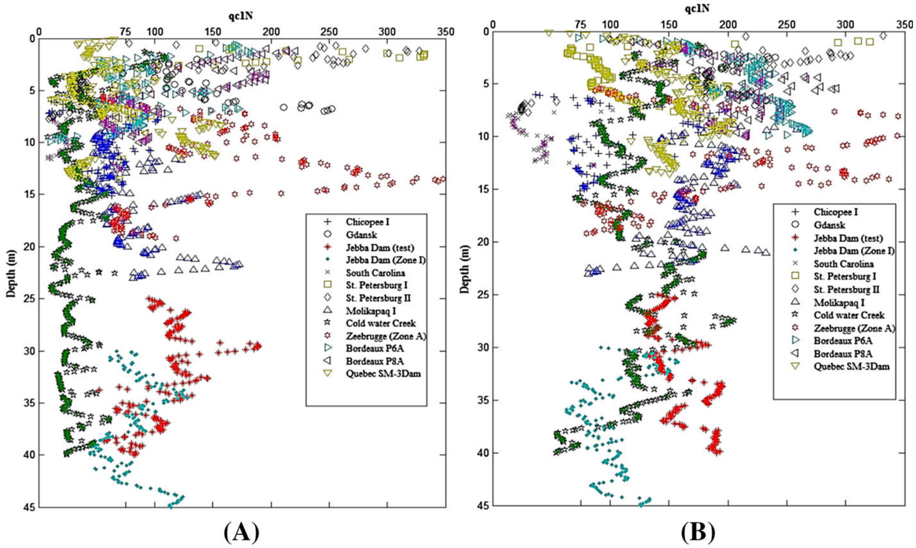


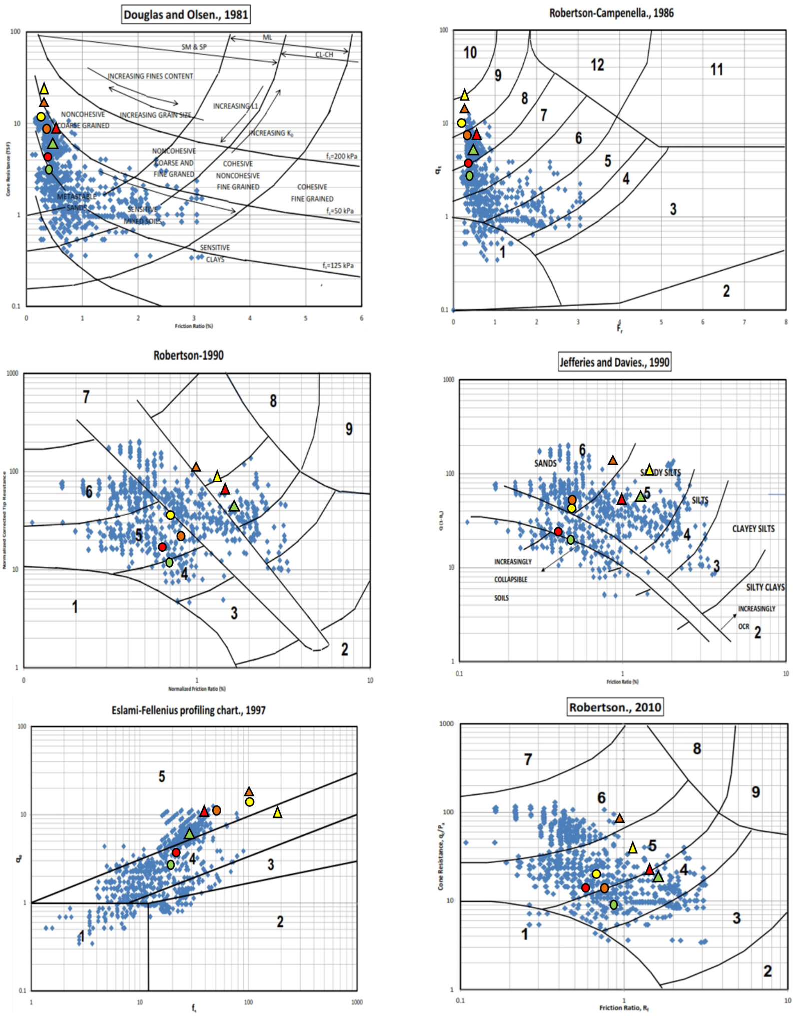
Fig. 9 Evaluation of soil condition for case histories based on q_{c1N} criteria. **a** Before explosion. **b** After explosion

Table 4 Liquefaction potential assessment for case histories using Q_{tn} and q_{c1N} criteria

Case	After explosion		Before explosion	
	q_{c1N} (avg.)	Q_{tn} (avg.)	q_{c1N} (avg.)	Q_{tn} (avg.)
Bordeaux P6A	218.5	241	94	110.9
Bordeaux P8A	203.5	221	121.5	136.8
QuebecSM-3Dam	152.67	166.11	72.4	78
South Carolina	39.1	37.9	29.9	28.9
Jebba Dam (1)	159.6	130.3	109.8	90.1
Jebba Dam (2)	101.8	81.6	83.5	66.9
St. Petersburg I (without delay)	110.7	130.4	79.9	118.8
St. Petersburg II (with delay)	176.1	216.7	109.9	145.7

For evaluation of EC effects on soil properties and investigation of blast densification (BD) performance, q_c average values have been calculated according to CPT records pre and post blasting. Figure 6 presents CPT records before and after explosion compaction for all eight case histories. As illustrated in this figure, densification and decreasing liquefaction potential of soils for surface layers (2–3 m) do not happen in some cases and q_c values changed marginally. But in deeper layers, CPT records after explosion show higher values than before explosion and this means that soil in these zones improved very well. According to this figure, EC has been able to increase the average of initial q_c in most of the cases.

For case histories Q_{tn} and q_{c1N} prior and following of explosion were measured and compared in order to evaluate the effect of explosion on soil behavior changes and decreasing liquefaction potential. Figure 7 presents amount of above mentioned factors for



Jebba Dam(Zone I) after explosion	Jebba Dam(Zone I) before explosion
Jebba Dam(Test) after explosion	Jebba Dam(Test) before explosion
St. Petersburg I (without delay) after explosion	St. Petersburg I (without delay) before explosion
St. Petersburg II (with delay) after explosion	St. Petersburg II (with delay) before explosion

Fig. 10 CPT-based classification charts for soil behaviour assessment before and after explosion

some cases. Results of comparison indicate promising variations due to blast densification in targeted depth. For evaluation of Q_{tn} and q_{c1N} parameters, all of the records are presented in Figs. 8 and 9, respectively. These figures present better illustration about effect of explosion compaction on soil behavior and liquefaction potential control.

As illustrated in Fig. 8 soil behavior changes after improvement and the amounts of Q_{tn} are higher than 70 for most of the cases. Also, improved soils are not in the zone of liquefiable soils due to explosive compaction.

Table 5 Computation of LPI before and after EC

LPI	Site1	Site2	Site3	Site4	Site5	Site6	Site7	Site8
Before EC	8.21	6.48	6.33	7.81	5.42	8.45	8.36	7.89
After EC	1.31	3.21	3.87	2.01	4.93	1.78	2.21	3.63

Based on the information presented in Fig. 9, for the improved soil layers the amount of q_{c1N} are higher than 75. Therefore, after EC soil behavior changes from compression to dilation. Summary information of q_{c1N} and Q_{tn} values for cases in data bank are presented in Table 4.

In this research soil classification charts are used for a better assessment of soil behavior and liquefaction potential. By using CPT records before and after EC soil classification are presented in Fig. 10.

According to Douglas and Olsen (1981) and Campanella et al. (1986) charts after explosion, soil classification zone changes to an upper zone and this means that soil resistances increase and liquefaction potential decreases. All of the cases before densification are in the category of sensitive and non-cohesive soils, however, after EC soils are not in the metastable zones, anymore.

According to Robertson (1990) method for soil classification, before explosion soils belong to zones 4–6 which include loose sands and after explosion shift to zones 7 and 8 which contain dense and very dense sands. This results show that modified soils are excluded from liquefiable soils category. Also in Robertson (1986) method soils after explosion compaction are not in the category of liquefiable soils.

As presented in Jefferies and Davies (1991) diagram, modified soils are in silty sands and dense sands category (zones 5 and 6) and these soils have liquefaction potential less than loose sands. Before modification soils were collapsible and unstable, but after EC instability of soils decreases significantly.

Also as shown in Eslami and Fellenius (2004) chart, modified soils after explosion are not within the region of collapsible and unstable soils based on q_E and f_s values.

6 Discussions

The appropriate selection of number of explosion phases and executing the explosion in more than one phase and arranging the next phase boreholes between the boreholes of previous phase can cause a relatively appropriate compaction. The effectiveness of this technique was observed in Bordeaux site (sites No. 1 and 2). In South Carolina site (site No. 4), after three years settlement occurred, but cone tip resistance did not increase significantly. This phenomenon can be related to complex behavior of sands and should be considered in EC assessment and evaluation of settlement and cone resistance should be used simultaneously.

Sequence of blasting with using delay can increase densification and decrease liquefaction potential. Minaev (1993) presents two case histories in Russia (sites No. 7 and 8). In the first case (site No. 7) all explosion occurred simultaneously but in second case (site No. 8) between explosions 3–5 min delay was used. In the first case cone tip resistance increased 30 %. But in the second case cone tip resistance increased 80 %. Moreover, dividing the deposits thickness to the several sub-layers and setting the explosive charges within the several levels will produce a uniform level of compaction comparing to the placement of all designed explosive materials in one borehole in a specific depth. It is observed that by distribution of the explosive charge throughout the target layer, a uniform densification occurs. Also, by performing delay between the various levels of a borehole every charge will compress the sub-layer associated to it and suitable compaction and less liquefaction potential will be observed.

Moreover for evaluating site mitigate to liquefaction LPI method has been used. LPI calculation for all 6 sites is presented as Table 5. As shown in Table 5, calculated LPI ranged high liquefaction potential that after EC it becomes minor liquefaction suspect.

7 Conclusions

Liquefaction is one of the most important geotechnical hazards that usually occurs in loose saturated sands. Explosive Compaction (EC) or Blast Densification (BD) can increase internal stability and decrease liquefaction potential significantly. One of the most practical methods for investigation and assessment EC efficiency is CPT and CPTu records. In this research a couple of CPT-based approaches are used for evaluation of liquefaction phenomena including: analytical approach focusing on CSR, q_c variations pre and post modification, Q_{tn} and q_{c1N} criteria, and soil behavior classification charts including Douglas and Olsen (1981), Campanella et al. (1986), Robertson (1990), Jefferies and Davies (1991), Eslami and Fellenius (2004), and Robertson (1986) methods.

To evaluate the liquefaction potential of granular deposits improved by EC technics, a database has been investigated including eight case histories of practical EC from five countries. The material of improved deposits was in the class of saturated and loose to medium sand, hydraulic and alluvial deposits, silty sand and gravel with fine percentage between 1 % - 35 % and relative density of 30–60 %.

Using CSR versus q_c chart suggests that after explosion, soils are not in the liquefiable zones anymore and generally shifted to stable situations. This emphasizes effectiveness of EC for diminishing liquefaction potential of soils. Moreover, Analyses indicate that after explosive compaction, q_c values increased substantially. This increment is more considerable in deeper layers and this means that soil in these zones improved very well. Also, LPI approach proposed by Iwasaki et al. (1982) includes weighted depth and factor of safety against liquefaction. It has been addressed using this index that higher susceptibility of liquefaction can be reduced by EC.

Results of investigation show that EC can change soil behavior from compression ($Q_{tn} < 70$) to dilation ($Q_{tn} > 70$) conditions. Also, this phenomena can change the state of soil from loose ($q_{c1N} < 75$) to dense ($q_{c1N} > 75$) condition. In most of the sites after explosion the amount of q_{c1N} was greater than 100. Accordingly, it is derived that the soil condition can be changed from compressive to dilative behavior as a result increasing to internal stability.

Comparison between CPT records before and after explosion and using soil classification charts show that due to explosive compaction, soils are not in the liquefiable zones anymore and generally transformed to dense or over consolidated situations. This can pronounce effectiveness of EC for elimination of liquefaction potential.

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