Vibro-Probe Technique Evaluation in Soil Improvement against Liquefaction

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SUMMARY:
Densification of saturated loose sands and silty soils is essentially a process involving controlled liquefaction as recent case histories show that vibro compaction techniques may be effectively used to densify silty sands to improve factor of safety against liquefaction.
This paper is a result of the experimental studies performed on a liquefaction susceptible site located in north of Iran (with marine deposits) before and after using vibro-Probe method. This method involves sequence of processes stating with insertion of vibratory probe with rotating eccentric mass (FHWA 2001) into ground. Once the design depth is reached, the probe is withdrawn in lifts.
Obtained results in this study can evaluate reliability of Vibro-Probe technique in improving of soil resistant properties (e.g. densification) against liquefaction using analysis based on correlation between corrected standard penetration test number (N'60) and liquefaction probability (PL).

Keywords: Soil improvement, Vibro compaction techniques, Liquefaction.

1. INTRODUCTION

Soil liquefaction describes a phenomenon whereby a saturated soil substantially loses strength and stiffness in response to an applied stress, usually earthquake shaking or other sudden change in stress condition, causing it to behave like a liquid. In soil mechanics the term "liquefied" was first used by Hazen in reference to the 1918 failure of the Calaveras Dam in California. (Hazen, 1920)

The phenomenon is most often observed in saturated, loose to medium dense (low density or uncompacted), sandy soils with fairly uniform grain size distributions, covering the silty sandy range.
The most critical soil is fine sandy grained with some silt content. Fig. 1.1 shows the bandwidth susceptible to liquefaction. (Keller, 2012)

Methods to mitigate the effects of soil liquefaction have been devised by earthquake engineers and include various soil compaction techniques such as vibro compaction (compaction of the soil by depth vibrators), dynamic compaction, and vibro stone columns. The use of vibratory techniques to improve soils at depth has been practiced for more than 70 years and is well-established in North America for a range of applications. (Laynegeo, 2012) It has been extending all over the world. These methods result in the densification of soil and enable buildings to withstand soil liquefaction.

The principle of sand compaction (in Vibroflotation) process consists of a flotation of the soil particles as a result of vibration, which then allows for a rearrangement of the particles into a denser state. (Moretrench, 2012)

The effects of soil compaction may be summarized as: (Moretrench, 2012)
- The sand and gravel particles rearrange into a denser state.
- The ratio of horizontal to vertical effective stress is increased significantly.
- The permeability of the soil is reduced 2 to 10 fold, depending on many factors.

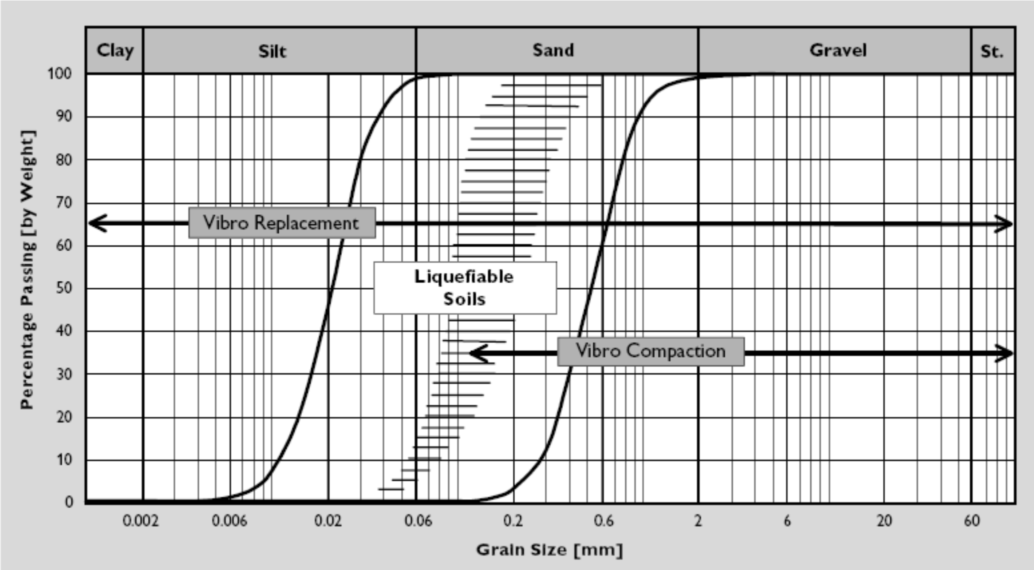


Figure 1.1. Liquefiable soils and application ranges of the deep vibratory compaction techniques. (Keller, 2012)

- The friction angle typically increases by up to 8 degrees.
- Enforced settlements of the compacted soil mass are in the range of 2 per cent to 15 per cent, typically 5 per cent.
- The stiffness modulus can be increased 2 to 4 fold.
- Better bearing capacity can be achieved as a result of higher shear strength parameters.

The Vibro Compaction technique is most suitable for medium to coarse-grained sand with a silt content of less than 12% passing sieve size of 0.074mm (No. 200) and clay content of less than 2 per cent passing sieve size of 0.005mm (See attached grain size curve, Figure 2). (Keller, 2012)

The classic vibro compaction method (Vibro probe) equipments contains of a hydraulic vibrator that is consists of a torpedo shaped horizontally vibrating probe, typically 2 to 4 m (7-11 ft) in length, which vibrates horizontally at frequencies of up to 3000 cpm and with amplitudes of 10 to 23 mm (1/2 to 1 in) that more commonly called a vibroflot. The vibrator is attached to a follow up pipe and hose length which can be varied according to the desired depth of improvement. (FHWA 2001)

The classic sequence of performance processes can be illustrated in three steps as following: (See also Fig. 1.2.) (Laynegeo 2012)

- Penetration: The Vibroprobe penetrates by vibration and aid of compressed air and water to the required depth.
- Compaction: The Vibroprobe is retracted from the maximum depth in 0.5m (1.5ft) intervals. The in-situ sand or gravel is flowing towards the Vibroprobe.
- Backfill: The compaction is achieved either with backfill from the top or with in-situ soil only.

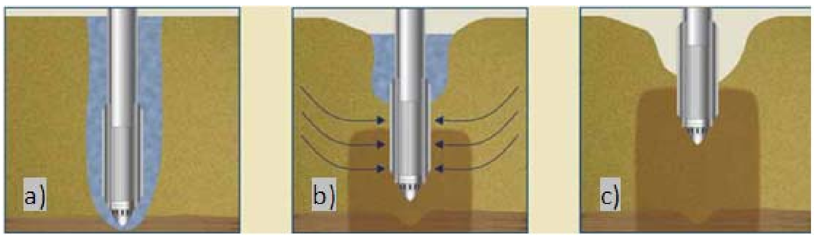


Figure 1.2. The sequence processes of classic Vibro probe:
a) Penetration. b) Compaction. c) Backfill. (Moretrench, 2012)

Typical depths for vibroprobe method range from 3 to 15m (10 to 50 ft), but it can be as shallow as a one meter (3 ft) and as deep as 36 m (120 ft). (FHWA, 2001) This paper is a result of the experimental studies performed on a liquefaction susceptible site (with marine deposits) which want to evaluate Vibro-Probe method effects on ground properties before and after using the improvement technique.

2. THE SITE SITUATION

2.1. Site Location

We performed this exploration in a site located in northern area of Iran (36°36'39.59"N 52°121'00"E). The distance to Elburz Mountains is about 15km (9.3mi) and to Caspian Sea is about 0.7km (0.44mi) in the nearest way. (Fig. 2.1)

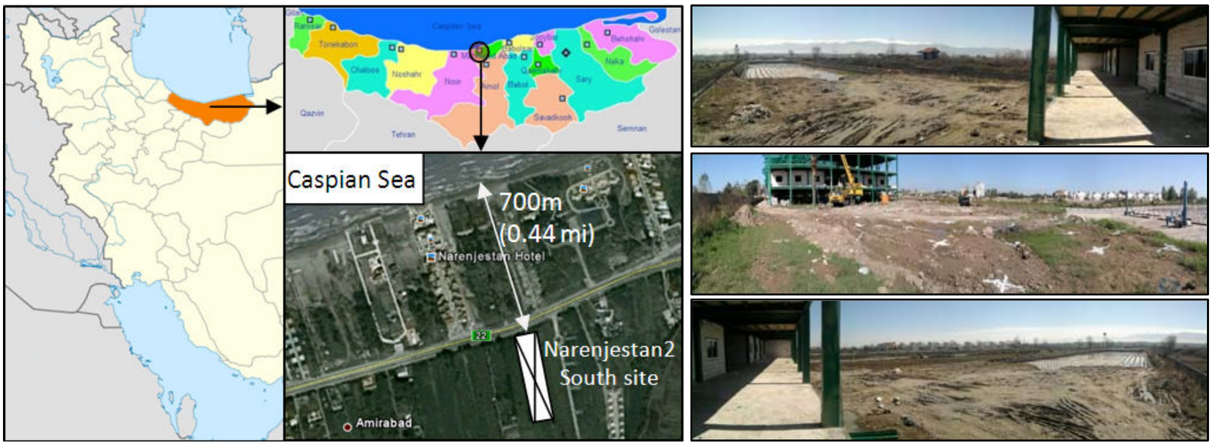


Figure 2.1. The site location and some pictures (Panorama180) of its limits and neighbourhood.

2.2. Geotechnical Aspects

The required area for the construction is about 2000 m^2 (2392yd^2) of land lied on marine alluvial deposits. We performed a set of geotechnical investigations involving in situ tests (e.g. SPT) and laboratory ones (e.g. essential physical tests for classification, direct shear, etc.) The schematic stratified layers are shown in Fig. 2.2.

Considering UCS classification (ASTM 2487), the medium to coarse-grained sand with a silt content of less than 12% passing sieve size of 0.074mm (No. 200) is found as the dominate type of the subsurface layers, except a thin zone in depth 3 m (10ft) and 14m (46ft) contains more cohesive soils including silt. This fact can be confirmed in Fig 2.3.

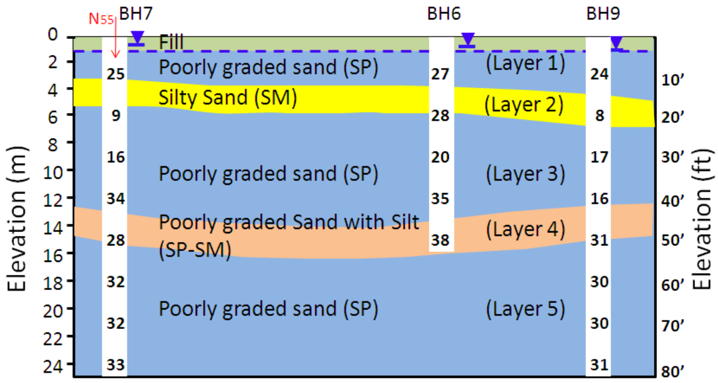


Figure 2.2. The schematic stratified layers data based on borings data.

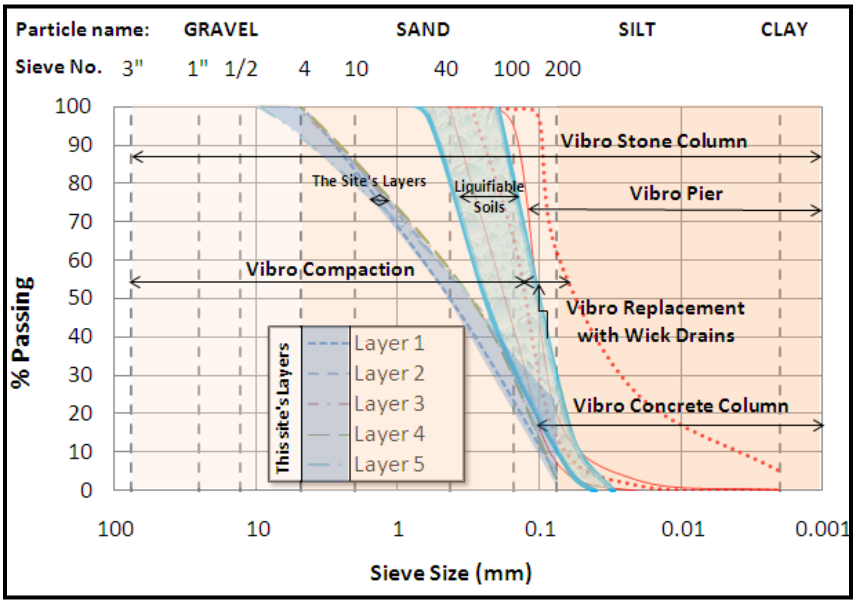


Figure 2.3. The average sieve grading curves of all sub-layers in the site is available for comparison the feasibility of various deep methods; moreover the liquefiable soils zone is presented (Keller, 2012).

Although the sieve grading ranges being out of the liquefiable soil range, according to Fig 2.3, but we performed some analysis to evaluate the liquefaction probability in this site. We checked liquefaction probability using the NovoTech^{TM} software (i.e. NovoLiq^{TM}). A simplified procedure was originally developed by Seed and Idriss (1971) using blow counts from the Standard Penetration Test (SPT) correlated with a parameter representing the seismic loading on the soil, called the Cyclic Stress Ratio (CSR). This parameter is compared to Cyclic Resistance Ratio (CRR) of the soil [CRR is estimated from SPT counts] and if it exceeds CRR, the soil is likely to be liquefied. This software uses following simple formula (NovoTech, 2011) to calculate Safety Factor against liquefaction phenomena:

$$Safety\ Factor = \frac{CRR_{1ave}}{CSR} \times MSF \times K_\sigma \times K_\alpha \tag{2.1}$$

Where:
CRR_{1ave}: calculated Cyclic Resistance Ratio (the average of all selected methods at a desired depth);
MSF: the Magnitude Scaling Factor which is defined in each method individually.
K_σ: the overburden stress correction factor (only applied to Vancouver task force report (2007), NCEER (1996), Cetin et al. (2004) and Idriss & Boulanger (2004) according to its own manual);
K_α: the ground slope correction (is assumed to be 1 in NovoLiq).
The Cyclic Stress Ratio (CSR) is calculated by Seed and Idriss (1971) formula:

$$CSR_{7.5} = 0.65\left(\frac{\sigma_v}{\sigma'_v}\right)\left(\frac{a_{max}}{g}\right).r_d \tag{2.2}$$

Where:
$CSR_{7.5}$: The Cyclic Stress Ratio with reference to earthquake magnitude of 7.5;
σ_v: Total overburden pressure at the depth considered;
σ'_v: Effective overburden pressure at the same depth;
a_{max}: Maximum horizontal acceleration at the ground surface;
g: Acceleration due to earth gravity;
r_d: Stress reduction factor which is defined in each method individually.

The following method is implemented in NovoLiq for estimating the probability of soil liquefaction which is recommended in NCEER workshop report. Indeed, the software uses two formulas for the estimation:

- Youd and Noble, 2001: they used a logistic analysis to analyze case history data from sites where effects of liquefaction were or were not reported following past earthquakes. This analysis yielded the following probabilistic equation:

$$Logit\ (PL) = \ln\ (PL/\ (1\text{-}PL)) = -7.633 + 2.256M - 0.258N1_{(60)cs} + 3.095\ln\ (CRR) \qquad (2.3)$$

  Where M is earthquake magnitude, PL is the probability that liquefaction occurred, 1-PL is the probability that liquefaction did not occur, and $N1_{(60)cs}$ is the corrected blow count, including the correction for fines content. Equation for defining the Youd and Noble MSF are listed below:

  Probability, PL < 20% MSF = 103.81 / M4.53 for M < 7
  Probability, PL < 32% MSF = 103.81 / M4.33 for M < 7
  Probability, PL < 50% MSF = 104.21/ M4.81 for M < 7.75

- Cetin et al., 2004: recently there have been technical discussions about the accuracy of this method. Therefore we just have noted it. However, in the results we can see the differences of two methods.

We represent the results of primary studies (using Kokusho et al. (1983), NCEER workshop (1997) and Boulanger Idriss (2004) for calculating CRR_{1ave} and setting earthquake magnitude to 7.5 and a_{max} equal 0.34) which are available in Fig. 2.4.

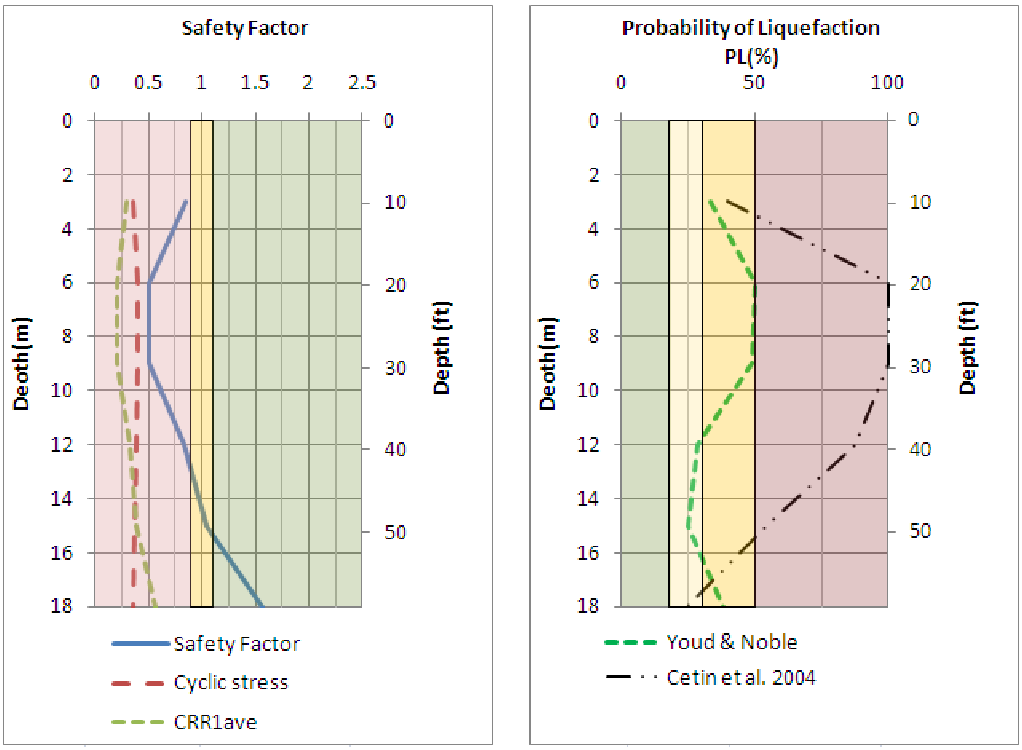


Figure 2.4. The results of the liquefaction analysis using NovoLiq software for the site, before improvement.

3. SOLUTION

The performed researches indicated the soil of the site had a Liquefaction potential, especially down to 12m (39.4ft), therefore the owner asked to find a suitable soil improvement solution. At first sight, it

Figure 3.1. Craters with diameters of 3 to 4.5 m (10 to 15 feet) can form around vibrator if backfill is not added. a) Before penetration. b) After penetration. (Keller 2012)

was confirmed that a deep densification technique could be more effective than the reinforcement methods (e.g. micropile, jet grouting, injection, etc.). (See the hatched area in Fig. 2.3.) Furthermore, the soil combination was not so poor and fine to need a replacement technique (e.g. Vibro replacement, Vibro stone, etc.). (Fig. 2.3)

The vibro probe technique sounded to be adequate solution, but the professional equipments were not available during the economic prohibitions. The backfill requires to prevent craters (see Fig. 3.1) was the other reason that limited the technique. Indeed, only the vibration seemed to be enough in the saturated field, so we recommended a solution as described as following steps:

- A pile of HEB steel profile (e.g. HEB360 with 12 m (39.4 ft) in length) was lifting up, vibrating with a vibratory hammer (e.g. 90 kW with 2000cpm with amplitudes of 10 to 25mm (0.4 to 1in)) and driving into ground just like a driven pile operation. (Fig. 3.2a, b)

- A triangular style was selected for the network of points which had to be driven. (Fig. 3.2c) Spacing of the probe compaction points depends upon soil type, density requirements, and probe/vibrator characteristics. The typical spacing range is reported from 1.5 to 4 m (5 to 12 ft). (Brown 1977, Welsh 1987, Wightman 1991). In this project a 3.5m (11.5ft) span for gridding was assessed to be satisfied. (See Fig. 3.2c)

- After vibrating of the pile in the desired depth, the pipe was withdrawn and after outing of the ground, another point was attacked one by one in turn. (Fig. 3.3)

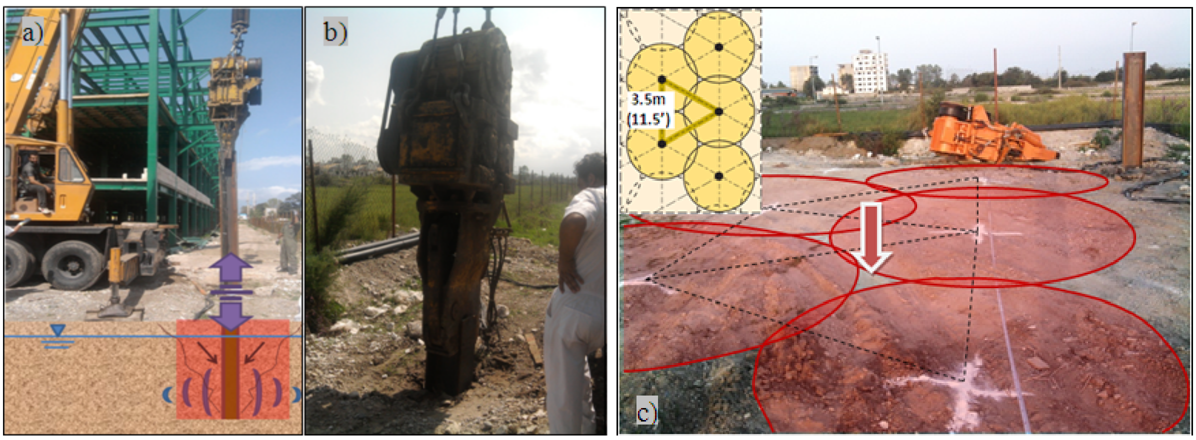


Figure 3.2. Some pictures of the suggested solution performance: a) A vibratory hammer is holding and vibrating the HEB360. b) The vibratory hammer and HEB360 in lower level of work. c) Triangular arrangement of destination points (with grid spacing 3.5m (11.5'). The arrow is showing the suspected place for SPT assessment after the improvement operation.

Figure 3.3. After vibrating a pile in the desired depth, the pipe was withdrawn and after outing of the ground, other points were attacked one by one in turn.

Figure 3.4. a) The plan view of just driven HEB360 in soil. b) The taper shaped end of the HEB360 pile.

Making more convenient for penetrating into cohesive layers, a taper shaped end was accepted for the pile (see Fig. 3.4). But the operator was bound for keeping the pile in the desired depth for designed time which was checked by the supervisor engineer, frequently.

4. RESULTS

Total number of all attacked points is N = 216 in this project (see Fig. 4.1) so considering a penetration area equal A_p = 0.13m^2 (1.4ft^2) for each point, a penetration area ratio can be defined as Eqn. 4.1.

$$\% A_{rp} = \frac{\sum A_p}{A_{site}} \times 100 \tag{4.1}$$

Where area of site is A_{site} = 2000m^2 (2392yd^2) as it mentioned above, thus A_{rp} is equal to 1.4% for this site. A_{rp} can be considered as similar as the stone column literature concepts.

We planned a set of geotechnical exploration after the above-mentioned soil improvement operation to evaluate soil parameters changes. In Fig. 3.2c, the arrow is showing suspected place for SPT assessment after improvement operation.

Several authors discuss the reported phenomena of the continuing stiffening and the strength gain up to several months after the densification. Use of test results immediately following vibro-compaction will give conservative results. (Lukas 1997) Hence we performed the tests in two month later by boring and performing various in situ and laboratory tests.

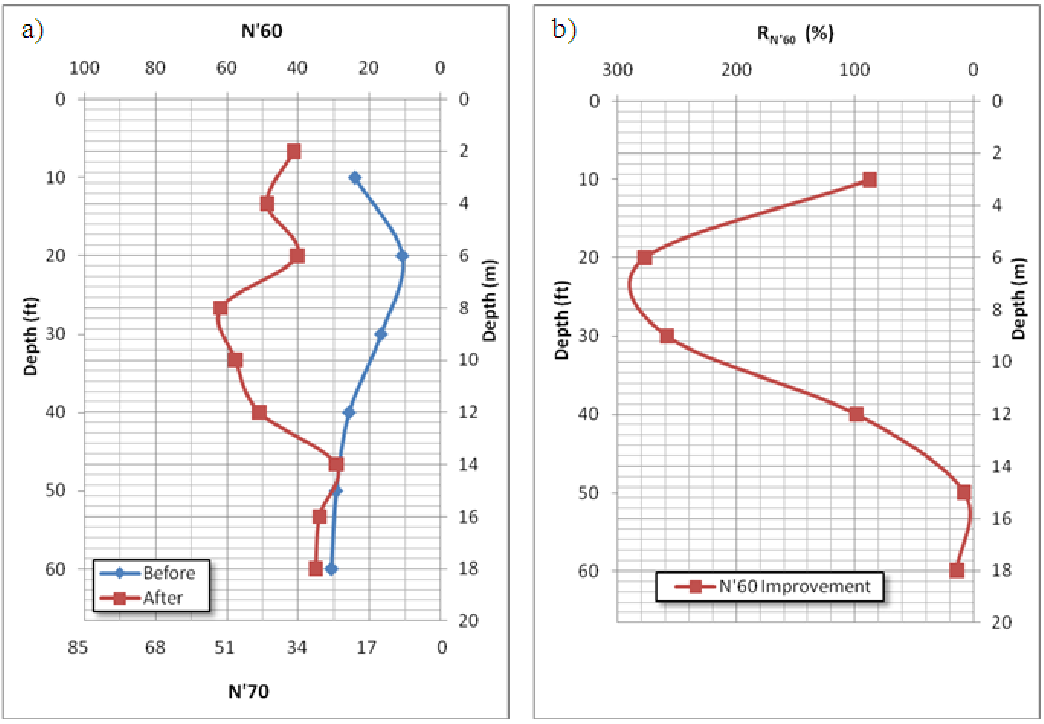


Figure 4.1. a) The comparison of corrected blow counts (N'$_{60\ \&\ 70}$) before and after the improvement operation against depth based on SPT results, b) the ratio of corrected blow count ($R_{N'60}$) changes after improvement against depth using Eqn. 4.2.

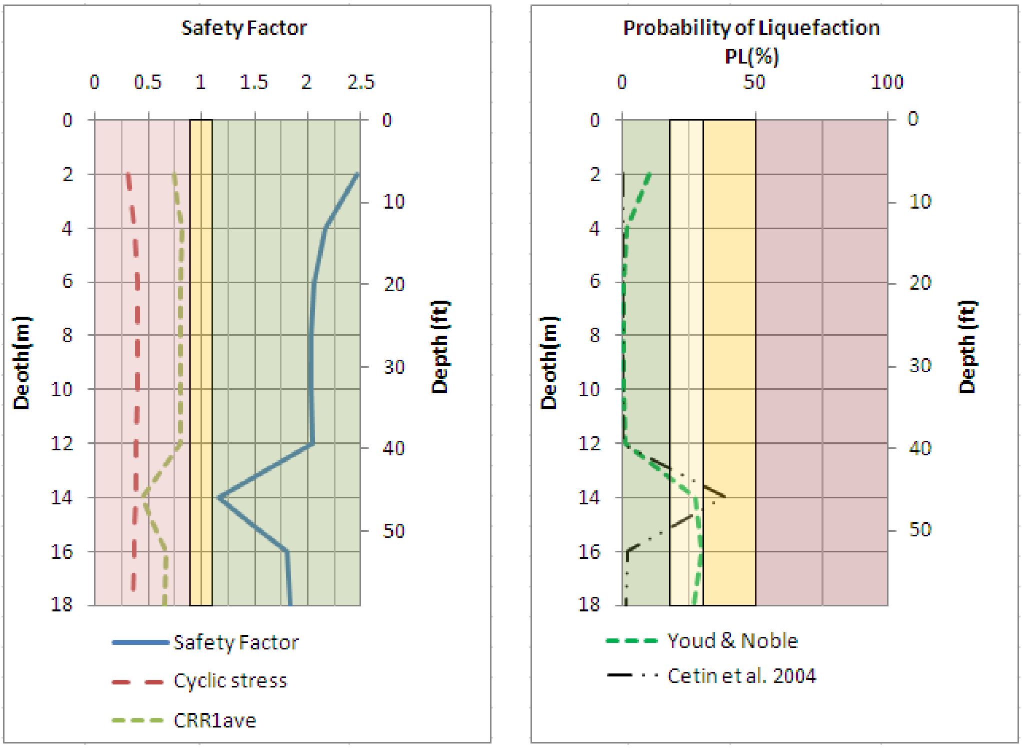


Figure 4.2. The liquefaction analysis results using NovoLiq software for the site, after improvement.

The comparisons of corrected blow counts results of Standard Penetration Test (based on ASTM 1586) (e.g. both N'60 and N'70) against depth are available in Fig. 4.1a.

An improvement ratio ($R_{N'60}$) can be defined as Eqn. 4.2 to show the growth ratio of N'60 (e.g. from before to after improvement which are named $N'60_B$, $N'60_A$ respectively). Therefore the $R_{N'60}$ changes against depth can be shown as Fig. 4.1b.

$$\% R_{N'60} = \frac{N'60_A - N'60_B}{N'60_B} \times 100 \tag{4.2}$$

To evaluate improvement ratio related to Safety Factor, we have defined *RSF* as Eqn. 4.3 definition.

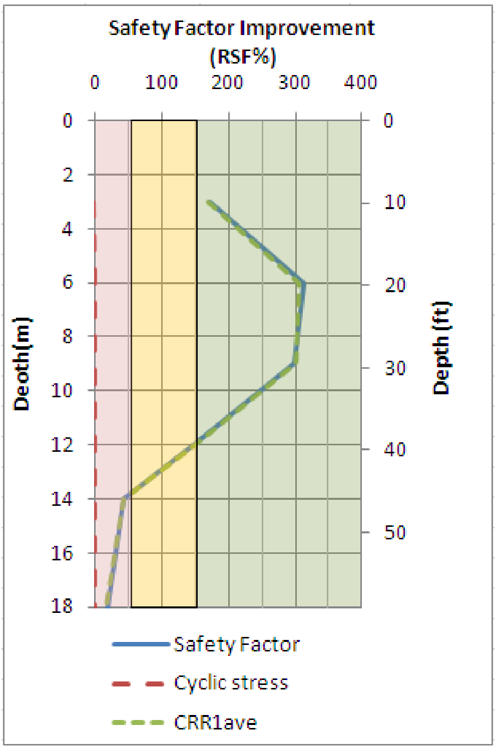
$$\% RSF = \frac{SF_A - SF_B}{SF_B} \times 100 \tag{4.3}$$

Where SF_B and SF_A are Safety Factors against Liquefaction (according to Eqn. 2.1) before and after improvement, respectively. Therefore the RSF changes against depth can be shown as Fig. 4.3a. It is obvious that ground has the most improvement between 6 to 9 m (20 to 30 feet) depth and the minority over 15 m (50 feet).

As another try to show the improvement ratio, we defined a *RPL* factor as Eqn. 4.4:

$$\% RPL = \frac{PL_A - PL_B}{PL_B} \times 100 \tag{4.4}$$

Where PL_B and PL_A are Probability of Liquefaction (according to Eqn. 2.3) before and after improvement, respectively. Both Youd & Noble and Cetin et al. Methods have been applied. Therefore *RPL* changes against depth can be shown as Fig. 4.3b. Negative sign shows the improvement and a smallest value of *RPL* (e.g. -100%) means more improvement action in descending the probability of liquefaction.



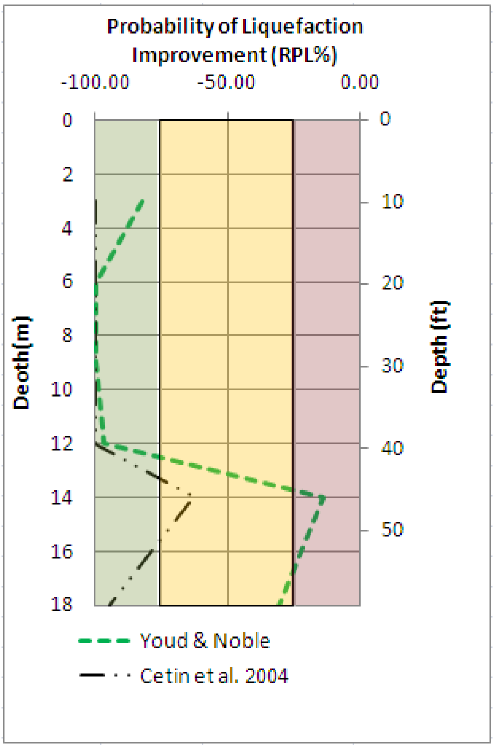


Figure 4.3. The evaluation of improvement ratios against Liquefaction in depth in different ways: a) changes of Safety Factor (*RSF* according to Eqn.4.3), b) changes of probability of liquefaction (*RPL* based on Eqn. 4.4).

5. DISCUSSION AND CONCLUSION

Based on above mentioned results, using A_{rp} = 1.4% and the triangular grading span equal to 3.5m (11.5ft) with a pile length equal to 12m (39.4ft), this method has been adequate for this site. Because:

- None of *SF* values are below 1 and all *PL* values are less than 50% after improvement.
- Considering the N'60 improvement graph ($R_{N'60}$), there is a significant growth (e.g. Max= 280% and Avg=250%) between 6 to 10m (20 to 33ft) depth. It can be seen the same results for *RSF* and *RPL* as Max= 300% and -100% also Avg=275% and -100%, respectively.

Some lacks may be considered as follows:

- Being out of vibrating pile scope, the method presents less improvement ratios in the zones deeper than 12m (39ft) (e.g. Avg= 50%, 60% and 75% for $R_{N'60}$, *RSF* and *RPL*, respectively).
- Besides, the technique shows less improvement effects on more cohesive layers (e.g. around 3m (10ft) and 14m (46ft)) in this project.

However none of above lacks can reject the successes and the reliability of this technique in the site.
To continue this exploration, we recommend determining relations between A_{rp} and the improvement factors (e.g. $R_{N'60}$, *RSF* and *RPL* factors) in other soils and sites.

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