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Research Article

Gravel impact compaction piers as a method of soil improvement

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Abstract: The purpose of this study was to evaluate the performance of gravel impact compaction piers system (GICPs) in improving a 3.5m thick loose silty sand in a multilayer coastal soil system located in Bushehr, Iran. The liquefiable sandy soil layer was layered on clay layers with moderate to very stiff consistency and below the engineering embankment layer with a thickness of 1.8 m. Implementation of gravel impact compaction piers is a new generation of aggregate piers. Gravel impact compaction piers were used to improve the liquefiable soil layers and to increase the bearing capacity and reduce subgrade settlement to withstand surface infrastructures. The process of making gravel impact compaction piers in a triangular or square-patterned grid was created using three types of special mandrels and feeding and compacting the gravels in the cavity in several stages without removing the soil from the cavities. The experience gained in this case study showed that artificial liquefiable was created immediately after the construction of these piers in a limited area and the soil became unstable. After about 11-14 days, the soil stabilized rapidly. The results of the standard penetration test in the matrix soil around the piers showed that the amount of (N1)60 in compacted soils was in the range of 21-30 and on average 15 times the amount of (1-3) in the initial soil. Also, the relative density of the initial soil was increased from 25% to 75% after soil improvement. Accordingly, by relying merely on the compaction properties of the piers and without relying on other primary soil remediation factors, such as piers drainage and soil texture change, the safety factor of the improved soil is 1.7-1.95 times the minimum required according to the two risk levels in the design.

Keywords: Liquefaction, gravel impact compaction piers, soil improvement, soil compaction, multi-layer soil.

1. Introduction

Over the last three decades in Iran, with progress in urbanization and the development of industrial, tourist and economic areas, especially in coastal areas, pristine coastal lands have been used at an increasing rate. Most of these sites are close to cities and have the potential for investment in industry and tourism, despite various geotechnical challenges, including; liquefaction risk, high settlement, lack of bearing capacity and modulus of subgrade reaction, and various environmental challenges, such as tidal currents, high groundwater levels, the presence of corrosive chlorine and sulfate ions, flood currents, and other aggressive and threatening factors, have received special attention from most employers.

On this basis, many experiences have been gained in the various processes of land improvement and preparation. This research specifically presents the experiences gained in the soil improvement process in part of the coastal lands of Persian Gulf Science and Technology Park in Bushehr Port, Iran. In 2013, the first phase of improving the soil of the park was commissioned. Previously based on the park's initial layout plan, 2.8 hectares of the entire complex was covered by a 1.8 m thick engineering embankment.

As long as the soil is pristine and intact, most soil improvement methods can be used. However, if consciously or unconsciously before improvement of the loose underlying soil, engineering embankment consisting of a relatively thick rock fill layer and the multi layered mixed soil is implemented on it, it is not easily possible to improve the lower loose layer and, given restrictions on location, the potential of using the most common and effective soil improvement techniques will face geotechnical challenges. In coastal areas, preservation of drainage structure of rock fill and rubble layers of engineering embankment in coastal areas, use of soil improvement methods based on natural materials for the presence of aggressive ions in coastal environments, lack of need for heavy machinery for soil improvement is one of the main criteria in selecting soil improvement methods. Also, if the construction of short buildings and the use of low-cost improvement methods are considered by the employer, in the case of a relatively medium-thick, loose-grained, liquefiable layer located on clay layers with a relatively soft and medium consistency, the use of soil improvement methods based on the construction of column elements without any change in compaction or texture in the peripheral soil block is not suitable and effective. The failure mechanism of short column elements in a loose soil environment is punching mode and, under normal conditions, these elements cannot withstand the desired load.

According to these criteria, most soil improvement methods which are based on the construction of short column elements consisting of cement and steel elements, including common and popular methods of mixing soil with cement, high-pressure injection of cement slurry and micro-pile were removed from the selection circle. The cement slurry injection method also failed due to the high seawater flow and the openness of the boundary between the rock fill layer and the loose granular subgrade in the pilot phase. Finally, after conducting the necessary research, the use of gravel piers was prioritized.

Although the prerequisite for using different methods of aggregate piers construction is drilling cavity in engineering embankment and crossing hard and stone substrates, however, for various technical and operational reasons some known methods cannot be used. Vibrating stone columns are one of the most common soil improvement options that was removed from the selection circle for several reasons including; (a) the need to place stone columns on a stiff or hard layer, (b) the loose and saturated sand layer under the engineering embankment load and the hole instability created by the probe as it progresses, (c) filling the voids between the rubbles and the rock fills with silty sand aggregates and destroying the drainage property of engineering embankment, and (d) Not changing the relative density of the soil around the piers. According to American and Indian standards, it is not appropriate to use the aggregate piers method in cases where the bottom of columns cannot be mounted on a firm layer (Barksdale & Bachus, 1983), and (IS 15284 - part 1, 2003). Shields et al. (2004) proposed a method of making rammed aggregate piers in sandy soils.

In granular soils, casing pipes are often used to maintain the cavity wall stability during construction. This method was excluded from the selection circle, due to excavation and removing of soil from inside the casing pipe, a high volume of gravel consumed during ramming, water and silty sand rising from the bottom of the casing pipe while removing the casing pipe, the ineffectiveness of compaction the contaminated gravel mass, and the increase of cost and runtime (Shields et al., 2004). Gravel compaction piers are similar to sandy piers with drainage performance at the time of the earthquake, with shortening of the drainage path and prevention of overpressure reduces the potential for liquefaction.

These aggregate piers can also reduce the potential for soil liquefaction by compressing the soil around the piers or eliminating it in general (Kitazume, 2005). Hatanaka et al. (2008) performed experiments in situ and laboratory before and after soil improvement, in order to directly evaluate and validate the effects of soil improvement by sand compaction piers on density, deformation, static strength characteristics, and sandy soil liquefaction, it was concluded that the compaction sand piers increased the N-number of standard penetration tests in the range of 5 to 20 and the dry density and relative density of sandy soil increased dramatically. Also with increasing density, the drained shear strength of sandy soil was increased. The increase in the internal friction angle was between 3 to 5 degrees. As the relative density increased, the liquefaction resistance after sand densification had suddenly increased more than 3 times before that.

Various researchers, including Salahi et al. (2015), Okamura et al. (2006), Harada and Ohbayashi (2017), Yoshimi et al. (1989), Okamura et al. (2003), Hatanaka (2008), and Farias et al. (2005) have evaluated the performance and mechanism of stone columns as well as sand compact piles to reduce the risk of liquefaction and improvement of soils prone to liquefaction. Based on the research literature and the limitations of existing methods in stone columns and sand compaction piers as soil improvement methods, to develop a method and technology that meets the criteria specified for soil improvement, a new method to reduce constraints was given priority. On this basis, the new method and technology as gravel impact compaction piers system (GICPs) were evaluated in this research.

2. Materials and methods

2.1. *Site introduction and identification of the soil*

The place of case study and experiments is located in Bushehr Port, on the northwestern margin of the Persian Gulf and southwestern of Iran. The site of the Persian Gulf Science and Technology Park with an area of about 7.3 hectares is located on the northeast coast of Bushehr Port. The test site covers an area of over 4000 square meters as part of the 2.8-hectare site on which the engineering embankment was performed. According to the site plan of the park collection, 2-story office buildings are located on the site.

In order to identify the physical, mechanical and consolidation characteristics of the soil layers, and to identify soil problematic potentials, also to provide the necessary parameters for designing the foundation, the safe bearing capacity, and foundation settlement according to various foundations, in a location with the approximate dimension of 167 × 167 m and an area of approximately 2.8-hectares under full coverage of the engineering embankment, a total of 13 boreholes of varying length were excavated by rotary drilling method and after obtaining undisturbed specimens, disturbed and complete specimens, necessary laboratory and field tests were performed. Grading, hydrometer, Atterberg limits, moisture content, density, and specific gravity were laboratory tests, and standard penetration tests were the most commonly used field tests before soil improvement. All laboratory and field experiments were performed according to different ASTM standards.

The results of drilled boreholes indicate that the studied site was mainly implemented up to a depth of 40 m from the top of embankment, including the engineering embankment (3 sub-base layers with a relative compaction of 98- 100% and a thickness of 0.45 m, mix layer (0.25 m thick) and rubble and rock fill layers with diameters of 7.5 to 65 cm (1.10 m thick) with a total thickness of 1.8 m, a very loose granular soil as SM layer (brown silty sand) with an average thickness of 3.5 meters, the CL layer with soft to medium consistency (gray or yellow-green clay with low plastic property) with an average thickness of 11.7 meters, the CL layer with relatively hard consistency (brown and gray clay with low plastic property) with a thickness of 23 meters. In the study area, the technical specifications of the silty sand layer with an average thickness of 3.5 m as a liquefiable layer are presented in Tables (1) to (6). Table (1) shows the values of the parameters D10, D30, D50, D60, and F (percent passing the No. 200 sieve), and Table (2) shows the range of values the percentage of the constituent elements obtained from the grading curves at different depths of the sand layer.

Table 1. Soil classification results.

Depth (m)	D10 (mm)	D30 (mm)	D50 (mm)	D60 (mm)	F (%)	LL (%)	PI (%)	UCS
0.00-3.50	0.003-0.006	0.04-0.08	0.09-0.14	0.10-0.18	14-33	-	-	SM

Table 2. Particle weighted distribution.

Clay (%)	Silt (%)	Sand (%)	Gravel (%)
3-7	14-27	67-86	-

The range of variations in the values of physical properties of sandy soil including moisture content (ω), saturation density (γ sat), dry density (γd), void ratio (e) and soil mechanical properties including effective internal friction angle (Φ') and effective adhesion (c') of soil are presented in Table (3). Direct shear test (DST) was used to determine soil mechanical parameters. During direct shear tests, the specimens were remolded with great accuracy.

Table 3. Physical and mechanical properties of natural soil.

Depth (m)	ω (%)	γ_{sat} (kN/m^3)	γ_d (kN/m^3)	Gs (-)	e (-)	Φ (deg)	C (kPa)
0.00-3.50	32-33	18.48	14	2.67	0.87	27-28	0.01-0.02

Table (4) shows the results of calculating the relative density of the sandy soil layer.

Table 4. Relative density results.

Very Loose and Very Dense Soil				Natural Soil		`Dr (%)
e_{min} (-)	$\gamma_{d\text{-}max}$ (kN/m^3)	e_{max} (-)	$\gamma_{d\text{-}min}$ (kN/m^3)	e (-)	γ_d (kN/m^3)	
0.6	16.5	0.96	13.5	0.87	14	25

Table (5) comparatively shows the measured laboratory results showing moisture content, dry density, saturation density and void ratio for both natural and remolded soils.

Table 5. Comparison of physical properties between natural and remoulded soils.

Soil Type	ω (%)	γ_d (kN/m^3)	γ_{sat} (kN/m^3)	e (-)
Natural	32	14	18.48	0.87
Remoulded	31	14	18.34	0.83

Table (6) shows the initial and modified results of the standard penetration test (SPT) at different boreholes and within the sandy layer, together with the various parameters calculated based on the correlation formulas on the SPT test results. As can be seen, the results of the dry density and soil-based internal friction angle on the laboratory tests and the SPT test are relatively well adapted.

Table 6. Standard penetration test results.

Depth (m)	N	N_{60}	$(N_1)_{60}$	Φ (deg)	γ_d (kN/m^3)
0.00-3.50	1-2	1-2	2-3	18-22	14

2.2. Technology and the process for construction of gravel impact compaction piers

The assumptions for the design and implementation of gravel impact compaction piers, as one of the methods of soil improvement, are based on the following principles and criteria (US9915051, 2018):

- No need to dig for pier installation in a weak layer using a water jet, compressed air or mechanical auger and no need to remove the soil inside the cavities outside the workshop.
- Creating cavities within the weak layer through a casing pipe with a closed conical end and the possibility of opening and closing its middle axis.
- Construction of a gravel impact compaction pier by pouring the gravel into the casing pipe and removing the casing pipe.
- Increasing the pier diameter through repeated drive of the casing pipe with a closed conical end and a long tamper device, thereby significantly increasing the radial compaction and density of the matrix soil.
- Reinforcing the loose and saturated sandy soil layer against liquefaction is created by increasing the radial compaction created by the construction of gravel impact compaction piers in a triangular or square grid pattern.
- Possibility to run under submerged conditions similar to coastal land below sea level.
- Possibility to implement piers on urban land.
- If there is any hard and rock fill layer above the weak soil layer, dig and cross it.
- Fast implementation at a low cost.

Based on the probable geological conditions, the placement of a weak soil layer in conventional ground layers in high level or submerged groundwater conditions can be classified into one of the following modes that are unique to each of the dispositions, processes, and technologies to build gravel impact compaction piers;

A. Weak and uniformly soil with limited thickness.
B. Weak to moderate soil as non-uniformly and limited thickness condition.
C. Weak soil layer with limited thickness located beneath a strong low-thickness layer.
D. Weak soil layer with limited thickness located beneath the engineering embankment consisting of one or more strong rock fill layers with low thickness.

In this study, due to the presence of engineering embankment with a thickness of about 1.8 m on a loose and liquefiable silty sand layer with an average thickness of 3.5 m, the general construction method of gravel impact compaction pier is based on 4 main steps; (step 1) Drilling and crossing the engineering embankment layer on the surface, (step 2) hammering and driving the shuttle system into the weak soil layer, opening and removing the middle axis and pouring the gravel material into the pipe and then removing the casing pipe and repeat this step if necessary, (step 3) hammering the long tamper device and feeding the gravel to the bottom of the pier frequently, and (step 4) implementation of an extensive gravel layer on the ground and ramming the gravel layer especially piers location with a flat tamper device.

The steps of gravel impact compaction pier implementation in a weak soil layer below the engineering embankment consist of one or more compact engineering layers with low thickness and they are presented in both schematic and operational form in Fig. (1).

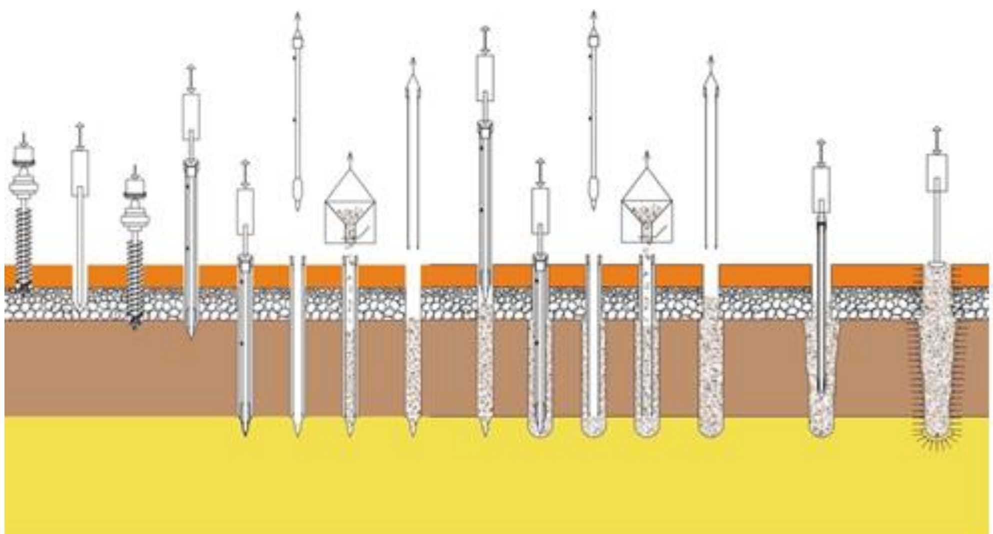

Figure 1. The steps of gravel impact compaction pier implementation in the presence of a liquefiable sandy soil layer located below the engineering embankment (US9915051, 2018).

Typically, mechanical auger with rock bit is used for drilling cavities in hard soil layers or loose rocks. In rocky embankment due to the presence of rock fragments with a diameter of 7.5-65cm, crossing the rocky layer was not possible only by means of mechanical auger equipped with rock bits even with strong gearboxes (because rock bits get stuck between the rocks) or lead to time-consuming and cause early damages to drill equipment (gearbox and junction breakage, and loss of rock bits). In order to solve this problem, an impact steel shaft with splitter end under impact loads and a mechanical auger equipped with rock bits were used.

In fact, by crushing and moving larger rock fragments by a long steel shaft mounted on a hydraulic hammer, it was possible to continue drilling by a mechanical auger equipped with rock bits until the cavity in the upper layer was completed. In practice, by dropping rock fragments from the cavity wall to the bottom, hammering the conical end of the initial shuttle into the weak layer became difficult, leading to an increase in the duration of the shuttle hammering and serious damage to the shuttle. To address this problem, the end of the new shuttle was designed and implemented in a combination of splitting wedge and cone shape. In this method, after drilling cavities in the upper layers, hammering the shuttle into the weak layer twice and primary and secondary gravel feeding, a long tamper device was used to increase the compaction of gravel inside the pier and matrix soil.

By repeated ramming and feeding of gravel materials by long tamper device into the cavity, three main objectives were provided including pier diameter development, gravel material compaction and increase of pier stiffness modulus and re-radial compaction of the soil around the gravel pier. The frequency of ramming was determined by a special tamper device and feeding of gravel materials inside the piers depending on the soil conditions of the site and pilot experiments at each site. In the final step, the gravel material was poured into the upper cavity as layered and rammed by a special tamper device with a flat beveled disk using dynamic impacts. In practice, the volume of gravel material used in these piers was well controlled. Due to the ramming of gravel materials inside the cavity, the bearing capacity of these piers is not very dependent on the bottom of it.

2.3. Components of gravel impact compaction piers

Selecting the type of material for the creating of compaction piers according to various criteria including; (a) rapid drainage of the matrix soil, (b) durability against physical disintegration due to continuous wetting and drying in the vicinity of sea-water, (c) high hardness against dynamic impacts of tamper devices, (d) high density and internal friction angle, (e) having coarse-grained texture to minimize the force required to remove the shuttle and tamper device from inside the gravel cavity was selected. On this basis, gravel aggregates produced by different mines were examined and standardized tests were performed following ASTM standard.

To select a good grading, several different granulations of sand, gravel, sand and gravel aggregates were evaluated at different intervals, and the results were successful. However, the use of broken gravel aggregates from the calcareous rocks used in concrete placing with a diameter of 12 to 38 mm and the GP name was a good choice in terms of performance. The dry density of gravel material in dense mode and internal friction angle obtained from the direct shear test with 30 × 30 cm box were in the range of 14-14.5 kN/m3 and 39-41 degree, respectively. The average gravel material drop in the Los Angeles and micro-Deval experiments were in the range of 12-18% (less than the maximum allowable drop of 35%) and 10-16% (less than the maximum allowable drop of 30%), respectively. The results of the granulation, hydrometer, and Atterberg limits tests and the percentages of the constituent elements of the gravel materials are presented in Tables (7) and (8).

Table 7. Coarse aggregate classification results.

D10 (mm)	D30 (mm)	D50 (mm)	D60 (mm)	C_c	C_u	LL (%)	PI (%)	UCS
8-13	16-18	21-24	24-28	08-1.2	2.2-2.8	-	-	GP

Table 8. Coarse aggregate weighted distribution.

Clay (%)	Silt (%)	Sand (%)	Gravel (%)
-	0.1-1	3-6	94-95

2.4. Characteristics and process of constructing gravel impact compaction piers in the test area

2.4.1. Design specifications for gravel impact compaction piers

In principle, by construction of common stone columns, retrofitting and reducing the liquefaction potential of the liquefiable layer are implemented in four ways; (a) replacing between 10 to 40% of the soil surface of the liquefiable layer with hard, non-liquefied grain materials, (b) reducing the amount of shear stresses of the soil block between the columns, (c) creating drainage columns to rapidly reduce excess pore-water pressure during the earthquake, and (d) absorbing a greater percentage of the shear stresses created in the soil layer during the earthquake.

However, with the application of gravel impact compaction piers, retrofitting of the liquefiable sand layer can be accomplished without considering the above four factors, simply by radial compaction of the matrix soil ('Recommendations for the design, calculation, construction and quality control of stone columns under buildings and sensitive structures', 2013).

This is the main distinction between gravel impact compaction piers with common stone columns, because, in common stone columns, the density of the matrix soil does not change. Therefore, the initial design of the diameter and the spacing between the compaction piers is based on increasing the radial compaction of the matrix soil to the necessary extent to retrofit the soil layer against liquefaction. It should be noted, in this study, due to the engineering embankment, it was not possible to swelling the matrix soil and the radial compaction of the matrix soil was done compulsorily.

Since the performance of gravel and sand compaction piles are similar to other aggregate piers methods, including vibration replacement and displacement, then for the design of gravel and sand compaction piles, four methods as A, B, C, and D are used, depending on the geotechnical characteristics of the soil layers. Methods B and C assume that no soil uplift occurs during the installation of sand compact piles (Kitazume, 2005). The design method used in this article is somewhat similar to method B. According to method B, after estimating the void ratios of original soil (ei) and improved soil (et) using standard penetration test results and the target (expected level after soil improvement) for matrix soil and estimation or measurement of emax and emin void ratios of original soil, the replacement area ratio is calculated from Equation (1).

$$Ra=(ei-et)/(1+et) \tag{1}$$

The diameter of the piles determines by calculating Ra and selecting the distance between the compaction sand piles in one of the triangular or square patterns.

According to the design method used in this paper, if the arrangement of the gravel impact compaction piers is an equilateral triangle with a side length (S) and the pier diameter (dp) determination is carried out through a single unit cell with a diameter of De (Fig. 2), in this case, assuming the cylinder boundary and unit cell height (H) remain constant, the volume of soil equivalent to the pier is injected into the volume of the cylindrical soil around the pier.

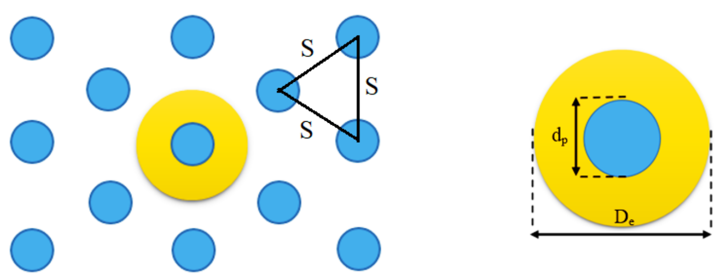

Figure 2. Triangular arrangement and unit cell model.

By equating the dry weight (Wdry1) of the primary unit cell soil before pier installation with dry weight (Wdry2) of the peripheral soil around the pier (matrix soil) in the secondary mode after pier installation, equation 2 was obtained:

$$\gamma_{d_2} = \left(\frac{D_e^2}{D_e^2 - d_p^2}\right)\gamma_{d_1} \tag{2}$$

where, γd1 and γd2 are the primary and secondary soil dry density, respectively. Accordingly, by precisely measuring the primary dry density of the soil, selecting the distance (S) between the compaction piers, and selecting the secondary dry density (target) of the soil matrix, the initial diameter (nominal diameter) of the compaction piers can be calculated. To select the dry unit weight of the secondary soil (after soil improvement), after selecting the (N1)60 target, the relative density value is estimated using the correlation relationships between (N1)60 and Dr. Then, by estimating the Dr value and measuring the values of the emax and emin void ratios, the amount of improved soil void ratio (e) is obtained. Finally, using the specific gravity (Gs), the unit weight of water (γw) and the void ratio of the improved soil (e), the amount of dry density (γd2) of the improved soil is calculated.

In fact, at this stage, different combinations of S (1.6, 1.8, 2, 2.2 and 2.4 m) and dp (0.45 and 0.60 m) are selected and after performing the required liquefaction analysis using different methods, the best option is selected technically and economically. In these analyzes, the equilateral triangle pattern is used to implement the GICPs.

Table (9) shows the results of the calculated unit cell parameters based on the selected compaction piers option (to be sure, using nominal diameter dp rather than extended diameter). Table (10) also presents the measured and calculated (predicted) relative compaction of sandy soils based on the selected scheme.

Table 9. Calculated unit cell parameters.

S (m)	D_p (m)	$D_e = 1.05S$ (m)	$R_a = 0.907\ (d_p/S)^2$ (%)
2	0.60	2.1	8.20
Ra: Replacement Area			

Table 10. Comparison of relative density before and after improvement.

Time	e (-)	γ_d (kN/m^3)	ω (%)	γ_{sat} (kN/m^3)	D_r (%)	$(N_1)_{60}$
Before Improvement (measured)	0.87	14	32	18.48	25	2-3
After Improvement (calculated)	0.72	15.24	27	19.35	67	27

2.4.2. Construction technology of gravel impact compaction piers

According to the invention USPATENT 9915051B2 and per Fig. (1), for the construction of gravel impact compaction piers, four categories of technical equipment have been prepared including; (a) engineering embankment drilling equipment including mechanical auger and steel shaft with splitter end, (b) a casing pipe installation equipment including a shuttle with conical end, (c) devices to pour gravel into the casing pipe and upper cavity containing conical hopper, (d) The gravel compaction devices inside the cavity, including the long and flat tamper devices. All of this equipment can be mounted on a hydraulic excavator. To drill an engineering embankment at specified points, two hydraulic excavator equipped with mechanical auger with rock bits and a steel shaft with splitter end connected to hydraulic hammer were used simultaneously.

The mechanical auger was used to drill and removed the crushed stone material and compacted soil and the steel shaft with splitter end was used to crushing the stone fragments. After creating the upper cavity to 1.8 m depth in the engineering embankment, a shuttle with splitter end and with the ability to open and close the middle axis was used to create the lower cavity in the sand layer without removing the soil. The shuttle hammering into the soil was carried out by a hydraulic excavator equipped with a hydraulic hammer and an edged flat disk rammer mounted on the hammer.

After the shuttle was completely hammered into the soil and the middle axis was removed, gravel was poured into the casing pipe and the casing pipe was removed from the soil. The high looseness of the sandy soil allowed the second stage of shuttle hammering and gravels feeding. On this basis, for gravel piers construction shuttle hammering and gravel feeding were performed twice. Up to this point, the equivalent diameter created for gravel piers was 60 cm. In the next stage, the development of gravel pier diameter was accomplished by repeated long tamper and gravel feeding, using a long tamper with a smaller diameter than the shuttle with long length and a cone-shaped end.

As a result of this process, gravel piers were carrot-shaped with a length of more than 3.5 meters, a bottom diameter of about 60 cm, a top diameter of about 85-90 cm and an average diameter of 75 cm. In the final stage, the upper cavity was filled by gravel materials in layers and compacted by a flat tamper device. The executive challenges created for the technical equipment made in this study led to the creation of the highest efficiency design and manufacturing technical knowledge. During this operation, different hydraulic hammers weighing in the range of 1.2-1.5 tones with 400-650 beats per minute were used.

3. Results

Prior to executing the sand layer improvement project beneath the engineering embankment, to study the performance of the system in soil resistance against liquefaction, in a strip form of test site and adjacent to the engineering embankment with an area of about 400 square meters with about 100 gravel impact compaction piers, a process similar to the desired design was implemented. At run time, the coastal soil bed was completely below 20-40 cm seawater. Immediately after each pier implementation, due to an instantaneous increase in pore-water pressure and a decrease in the effective soil stress due to the shuttle drive dynamically, the matrix soil (around the piers) became unstable and liquefiable.

Subsequently, with the full implementation of the piers, the desired soil range became extremely unstable. The instability was so great that it was not possible to move and with the impact of hydraulic excavator cup on the surface, a wavy movement of the soil layer was observed. Due to the submersion of the desired range, the outflow and discharge of water through the gravel piers were well observed. The daily analysis showed that after 11-14 days, the sandy soil layer was stable and resistant. So the hydraulic excavator could drive easily. After completing the next steps, including the application of an extensive layer of gravel and ballast with a thickness of about 40 cm on the desired area and implementation of a long tamper, after about 7-10 days, the sandy layer reached a higher level of stability and resistance.

Based on the experience gained from the implementation of this pilot project on natural soil bed, the operation of gravel impact compaction piers was carried out from top of engineering embankment. Due to the presence of engineering embankment and the lack of direct access to the natural soil bed, relative density control of the matrix soil was performed through a standard penetration test. Also, to study the extended diameter and the shape of the compaction pier, the slurry cement was injected into the gravel mass inside the pier and excavated a part of the engineering embankment from the natural bed. In order to obtain the maximum relative compaction for the matrix soil due to the construction of compaction piers, confirmatory tests of standard penetration were carried out after the piers were constructed. In the following, the result of the standard penetration tests and liquefaction analyzes of the sandy soil layer before and after soil improvement are presented.

3.1. Standard penetration test results

In this study, the layering and technical properties of the sandy soil in the study area and under the improvement operation which was done by gravel impact compaction piers had a relatively good uniformity. In this regard, to investigate the relative density of the improved soil, several standard penetration tests were carried out at the depth of 1.8–5.3 m at 0.5 m intervals at each of the building blocks. Table (11) shows the results of four standard penetration tests at designated sites after completion of soil improvement operations. As can be seen, the range of N-value variations of standard penetration tests at different depths is very limited. In other words, the average N-values obtained from different experiments can be used to perform the liquefaction analysis at different locations.

Figure (3) shows the comparison of the N-values variation curves of the standard penetration tests performed at different site locations after soil improvement. As can be seen, in the four experiments, the minimum and maximum N-values of the compacted soil were 18 and 27, respectively. In fact, due to the carrot shape of compaction piers and the decrease in pier diameters, with depth increasing, maximum N-value occurs in the upper part and minimum N-value in the lower part of the piers.

Table 11. SPT results on the matrix soils after improvement.

Depth	N_{SPT1}	N_{SPT2}	N_{SPT3}	N_{SPT4}	$N_{ave(1-4)}$
2.3	25	27	26	27	26
2.8	25	27	27	25	26
3.3	24	26	25	23	25
3.8	24	25	24	24	24
4.3	23	24	22	21	23
4.8	22	22	19	18	20

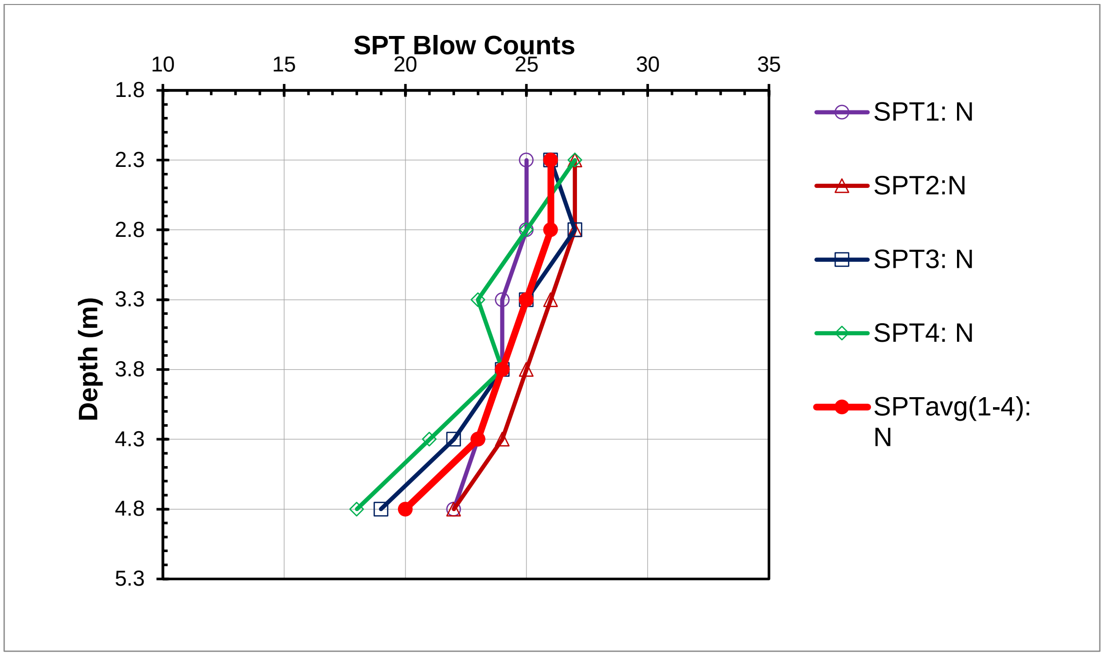

Figure 3. Comparison of N-values variation curves of standard penetration tests after soil improvement.

Table (12) shows the average values of standard penetration tests includes N, N60 and (N1)60 comparatively, before and after soil improvement. Also in Figure (4) the parameters N, N60 and (N1)60 variation curves of standard penetration tests before and after soil improvement are presented comparatively. Based on the results shown in Table (12), the changes of (N1)60 before and after soil improvement are in the range of 1-3 and 21-30, respectively. Also, in the depth range of the sandy soil layer, the amount of (N1)60 in compacted soils is 8–29 times and the average of 15 times as much as in the primary soil (before improvement).

Table 12. Comparison of SPT results on the matrix soils after improvement.

Depth	Natural Soil (before soil improvement)			after soil improvement		
	N	N_{60}	$(N_1)_{60}$	$N_{avg(1-2)}$	N_{60-avg}	$(N_1)_{60-avg}$
2.3	2	2	3	26	20	30
2.8	1	1	1	26	20	29
3.3	2	2	3	25	19	26
3.8	1	1	1	24	19	25
4.3	2	2	3	23	19	24
4.8	2	2	2	20	17	21

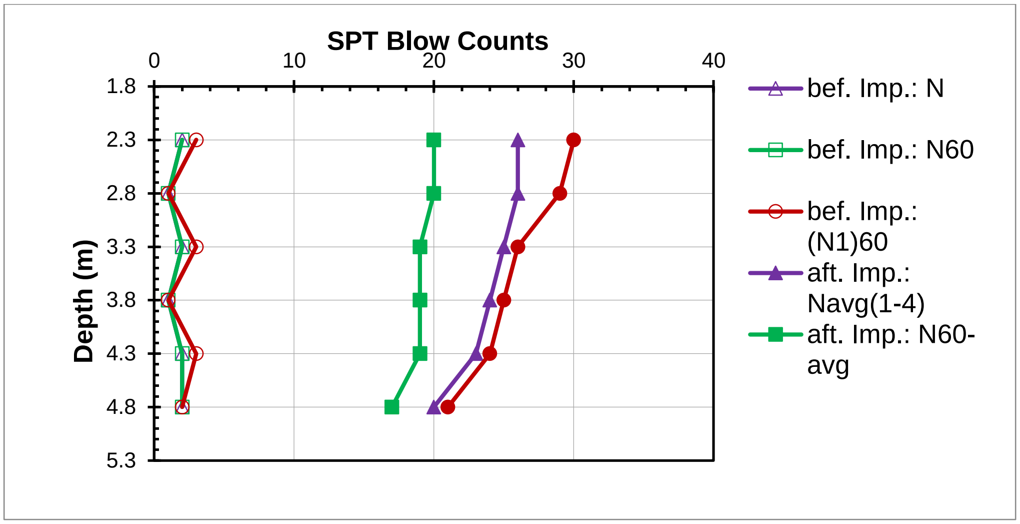

Figure 4. Comparison of N, N60 and (N1)60 variation curves of standard penetration tests before and after soil improvement.

3.2. Results of liquefaction analysis

In principle, various analytical and numerical methods have been developed to analyze the liquefaction of improved soils using methods based on aggregate piers. As a popular method, Priebe in 1995, presented a method of calculating the safety factor against liquefaction for vibrating stone columns based on gravel column drainage characteristics, structural modification of soil texture and thereby increasing soil shear strength and the matrix soil density (H. J. Priebe, 1995 and 1998). In practice, the hassle of making a valid prediction is easily resolved by specifying the corrected amount of soil dry density through speculation. Based on the experience gained from the application of gravel impact compaction piers, it is possible to retrofit the liquefiable layer solely by optimally increasing the radial compaction of the matrix soil.

The analytical method presented in this section is based on comparing the relative compaction of soil and conducting liquefaction analysis before and after the application of compaction piers. In fact, two important properties of gravel piers drainage and improved structure of sandy soil mass and accordingly the increase soil shear strength is not considered, conservatively. The results of the initial soil liquefaction analysis (before improvement) at the L1 risk level (M=6.75, amax=0.3, FSmin=1.5) for a 75-years return period is shown in Table (13) and at the L2 risk level (M=6.75, amax=0.3, FSmin=1) for a 475-years return period is shown in Table (14). Accordingly, the values of the safety factor against liquefaction using eight common methods for L1 risk level are in the range of 0.37- 0.40 (less than FS=1.5) and in the risk level of L2 it is in the range of 0.28- 0.30 (Less than FS=1).

Table 13. Calculation of safety factor against liquefaction by different methods at M= 6.75 and a_{max}=0.3 before soil improvement.

Depth(m)	Safety factor Based on M= 6.75, a_{max}=0.30 and R.P. =75 years								Average
	NCEER Workshop (1997)	Boulanger & Idriss (2004)	Vancouver Task Force (2007)	Chinese Code	Seed et al. (1983)	Tokimatsu & Yoshimi (1983)	Shibata (1981)	Kokusho et al. (1983)	Safety Factor
2.3	0.18	0.32	0.18	0.25	0.48	0.47	0.66	0.52	0.38
2.8	0.16	0.29	0.16	0.24	0.47	0.44	0.66	0.5	0.37
3.3	0.18	0.32	0.18	0.26	0.49	0.48	0.67	0.52	0.39
3.8	0.16	0.29	0.16	0.25	0.48	0.45	0.67	0.5	0.37
4.3	0.19	0.32	0.19	0.26	0.49	0.49	0.68	0.53	0.39
4.8	0.19	0.33	0.19	0.27	0.5	0.49	0.68	0.53	0.40

Table 14. Calculation of safety factor against liquefaction by different methods at M= 7.5 and a_{max}=0.3 before soil improvement

Depth(m)	Safety Factor Based on M= 7.50 & a_{max}=0.30 and R.P. =475 years								Average
	NCEER Workshop (1997)	Boulanger & Idriss (2004)	Vancouver Task Force (2007)	Chinese Code	Seed et al. (1983)	Tokimatsu & Yo-shimi (1983)	Shibata (1981)	Kokusho et al. (1983)	Safety Factor
2.3	0.14	0.24	0.14	0.19	0.37	0.36	0.5	0.39	0.29
2.8	0.12	0.22	0.12	0.18	0.36	0.34	0.51	0.38	0.28
3.3	0.14	0.24	0.14	0.19	0.37	0.36	0.51	0.4	0.29
3.8	0.12	0.22	0.12	0.19	0.36	0.34	0.51	0.38	0.28
4.3	0.14	0.25	0.14	0.2	0.37	0.37	0.51	0.4	0.30
4.8	0.14	0.25	0.14	0.2	0.38	0.37	0.52	0.4	0.30

As the same way, the results of liquefaction analysis on improved soil at L1 risk level (M=6.75, amax=0.3, FSmin=1.5) for a 75-years return period are shown in Table (15) and at L2 risk level (M=7.5, amax=0.3, FSmin=1) for a 475-years return period are shown in Table (16). Accordingly, the calculated safety factor against liquefaction using eight common methods for the L1 risk level is ranged from 2.45 to 2.62 (greater than FS=1.5) and at the L2 risk level is in the 1.85-1.98 (more than FS=1).

Table 15. Calculation of safety factor against liquefaction by different methods at M= 6.75 and a_{max}=0.3 before soil improvement

Depth(m)	Safety Factor Based on M= 6.75, a_{max}=0.30 and R.P. =75 years								Average
	NCEER Workshop (1997)	Boulan-ger & Idriss (2004)	Vancou-ver Task Force (2007)	Chinese Code	Seed et al. (1983)	Tokimatsu & Yo-shimi (1983)	Shibata (1981)	Kokusho et al. (1983)	Safety Fac-tor
2.3	2.64	2.91	2.64	2.64	2.64	2.64	2.64	1.87	2.58
2.8	2.66	2.92	2.66	2.66	2.66	2.66	2.66	1.88	2.60
3.3	2.67	2.94	2.67	2.67	2.67	2.67	2.67	1.80	2.60
3.8	2.69	2.93	2.69	2.69	2.69	2.69	2.69	1.72	2.60

4.3	2.71	2.92	2.71	2.71	2.71	2.71	2.71	1.76	2.62
4.8	2.73	2.91	2.73	2.73	2.24	2.50	2.19	1.54	2.45

Table 16. Calculation of safety factor against liquefaction by different methods at M= 7.5 and a_{max}=0.3 before soil improvement

Depth(m)	Safety Factor Based on M= 7.50 & a_{max}=0.30 and R.P. =475 years								Average
	NCEER Workshop (1997)	Boulanger & Idriss (2004)	Vancouver Task Force (2007)	Chinese Code	Seed et al. (1983)	Tokimatsu & Yoshimi (1983)	Shibata (1981)	Kokusho et al. (1983)	Safety Factor
2.3	2.02	2.22	2.02	2.02	2.02	2.02	2.02	1.43	1.97
2.8	2.02	2.23	2.02	2.02	2.02	2.02	2.02	1.43	1.97
3.3	2.03	2.24	2.03	2.03	2.03	2.03	2.03	1.36	1.97
3.8	2.04	2.22	2.04	2.04	2.04	2.04	2.04	1.31	1.97
4.3	2.05	2.21	2.05	2.05	2.05	2.05	2.05	1.33	1.98
4.8	2.06	2.20	2.06	2.06	1.69	1.89	2.66	1.17	1.85

Figure 5 and Table (17) illustrate the values of safety factors against liquefaction comparatively for soil conditions before and after improvement at the two risk levels of L1 and L2, respectively. As can be seen, based merely on the effect of the radial compaction of the compaction piers on the matrix soil, the obtained safety factors are sufficiently higher than the minimum values required at each hazard level.

Table 17. Comparison of safety factor against liquefaction between before and after soil improvement at two level of risk

Depth (m)	Bef. Improvement		Aft. Improvement	
	FS(L_1)	FS(L_2)	FS(L_1)	FS(L_2)
2.3	0.38	0.29	2.58	1.97
2.8	0.37	0.28	2.60	1.97
3.3	0.39	0.29	2.60	1.97
3.8	0.37	0.28	2.60	1.97
4.3	0.39	0.3	2.42	1.98
4.8	0.4	0.3	2.45	1.85

L1: M= 6.75, a_{max}=0.30 and R.P. =75 years
L2: M= 7.50 & a_{max}=0.30 and R.P. =475 years

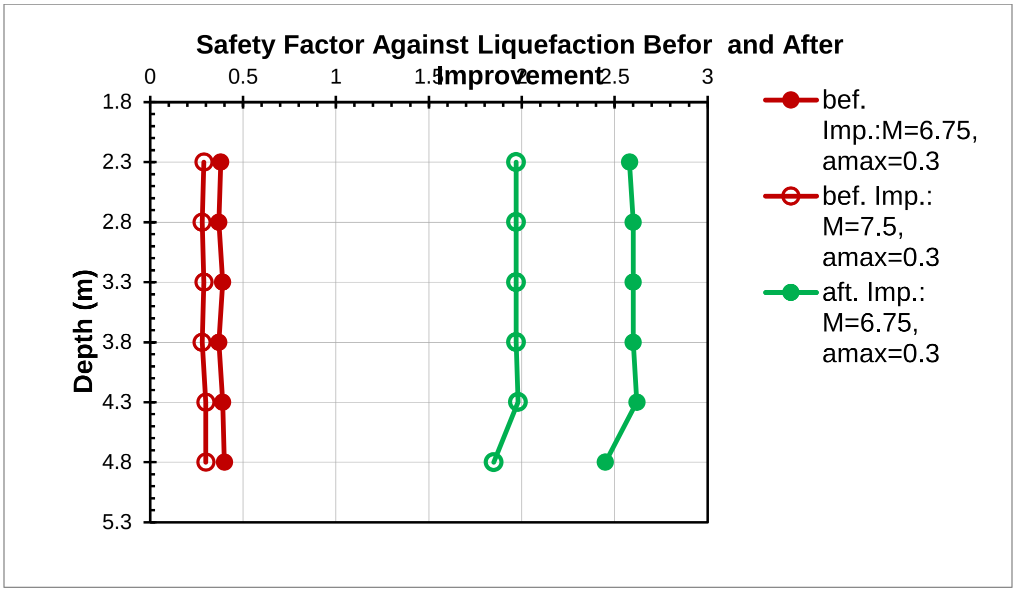

Figure 5. Comparison of the safety factor values against liquefaction for soil conditions before and after improvement at two risk levels of L1 and L2.

3.3. Measurement of density and moisture content of soil

Despite the impossibility of obtaining undisturbed samples from loose silty sand soils and saturation in semi-deep conditions, it is possible to obtain undisturbed samples by Shelby tube sampler in case of high compaction for these types of soils. In this regard, after implementation of gravel impact compaction piers and increasing the radial compaction of the matrix soil, a Shelby tube was prepared and the local moisture content and density of soil were determined.

Based on local measurements, on average, saturation density, dry density, and soil moisture content were 19.1 kN/m3, 15.15 kN/m3 and 26%, respectively. Table (18) shows a comparison between the measured and calculated results for the void ratio, dry density, moisture content, saturation density, and soil relative compaction parameters before and after soil improvement. As can be seen, the values predicted and measured in after improvement conditions show relatively good adaptation.

Table 18. Comparison of calculated and measured soil compaction parameters before and after soil improvement

Time	e (-)	g_d (kN/m^3)	ω (%)	g_{sat} (kN/m^3)	Dr (%)
Before Improvement (measured)	0.87	14	32	18.48	25
After Improvement (calculated)	0.72	15.24	27	19.35	67
After Improvement (measured)	0.69	15.15	26	19.10	74

4. Conclusions

To retrofit against liquefaction for a very loose silty sand layer of a multi-layer soil with 3.5 m thick located under the 1.8 m engineering embankment, a new method as "gravel impact compaction piers system" was used. The main results of this study are as follows:

In order to retrofit the loose and liquefiable granular soil layers beneath the engineering embankment, it is possible to simultaneously and optimally excavate the engineering embankment and pass through the rock fill and rubble layers using a special mechanical auger combination system equipped with rock bits and a steel shaft with splitter end under dynamic impacts mounted on two chain hydraulic excavator.

1. Due to the large-scale of engineering embankment on the loose and liquefiable sandy soil layer and create the necessary overburden to prevent swell of the upper part of matrix soil caused by the compaction piers construction, preliminary calculations for determining the required diameter for high-precision compaction piers can be made from the idea of the unit cell.

2. In the depth range of the sandy soil layer, the (N1)60 amount of the compacted soil was in the range of 21-30 and equal to 8-29 times, and on average 15 times the amount of (1-3) in the primary soil.

3. Without relying on the drainage property of the compaction piers and without considering the created change on the initial soil texture, the calculated safety factor against liquefaction are based solely on the compaction properties of compaction piers, using eight common methods for L1 risk levels is in the range of 2.45-2.62, on average, 1.7 times the minimum required value (FS=1.5) and at (L2) risk level it is in the range of 1.98-1.85 times the minimum required value (FS=1).

4. In the depth range of the sandy soil layer, the calculated safety factor against liquefaction of compacted soil was equal to 8–29 times and on average 15 times the amount in the initial soil (before improvement).

5. By implementing gravel impact compaction piers, through creating repeated radial compaction for the matrix soil in the process of hammering and feeding the gravel, the first time shuttle, hammering and feeding the gravel, the second time, hammering and feeding the gravel repeatedly by the long tamper, and hammering and feeding of gravel is frequently done by a flat tamper device, with increasing pore-water pressure, the desired area is artificially liquefiable and highly unstable, and becomes stable and resistant within 11-14 days.

6. With the application of gravel impact compaction piers, the relative compaction of initial soil increased from 25% to 75% (300%) and the dry density of initial soil increased from 14 to 15.15 kN/m3 compared to the soil without improvement.

7. Gravel impact compaction piers are a self-regulating system for increasing relative soil density. In fact, gravel injection is done through gravel columns created under a specified energy level of the hydraulic hammer installed on the hydraulic excavator and is done as long as it is not possible to inject gravel. Based on this, it creates a uniform stiffness modulus for the subgrade.

8. The soil improvement method using gravel impact compaction piers is a definitive soil improvement method and after the improvement operation, soil compaction can be measured.

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