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Ground Improvement in Deep Waters Using Dynamic Replacement

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ABSTRACT

Dynamic Replacement is a ground improvement technique used for treating soft compressible cohesive soils. It has been used in numerous land projects and a number of offshore works with seabed as deep as 15 m below sea level. Recently, works of similar nature was carried out in Southeast Asia with the intention of exploring the possibility of treating soils in deeper waters. In this case, the seabed was 30 m below sea level, and to the knowledge of the authors, is a world record as the deepest Offshore Dynamic Replacement or Dynamic Compaction works. The pressuremeter test was used to verify the results and to estimate the soil parameters.

KEY WORDS: dynamic; replacement; ground; soil; improvement; marine; pressuremeter

INTRODUCTION

Dynamic Replacement (DR) is a ground improvement technique developed by Louis Menard in 1975 for the treatment of soft cohesive soils. As shown in Fig. 1, in this technique a heavy pounder is systematically dropped a number of times onto specific points in order to drive granular material into soft compressible cohesive soils and to compact the driven material sufficiently to meet the project's design criteria.

Dynamic replacement is a very cost effective, efficient and rapid method of treating soft soils and has been used in numerous land projects including the 2.6 million square meter mega soil improvement project of King Abdulla University of Technology in Saudi Arabia (Chu et al., 2009).

Dynamic replacement or its counterpart ground improvement technique for granular soils, dynamic compaction, have previously been used for the treatment of soft or loose marine soils in offshore projects such as Brest Naval Port in France (Menard, 1974;Boulard, 1974; Renault and Tourneur, 1974; Gambin 1982), Pointe Noire in Gabon (Menard 1978), Uddevalla Shipyard Wharf (Techniques Louis Menard, 1975; Gambin 1982), Kuwait Naval Port (Gambin, 1982; Chu et al., 2009), Sfax Fishing Quay in Tunisia (Menard, 1981; Gambin 1982), and Lagos Dry Dock in Nigeria (Gambin, 1982; Gambin and Bolle, 1983) with seabed as deep as 15 m below seawater level.

The first offshore dynamic compaction project was carried out by Menard in 1973 as part of the construction of Brest Naval Port's dry dock. In this project a specially designed 11 ton pounder was used to compact 3 m of loose alluvium on the seabed.

In Kuwait Naval Base a 32 ton pounder was used to compact a 5 m thick layer of silty sand and a 1.5 to 2 m thick rock fill blanket at the depth of 10 m below seawater level to mitigate the risk of liquefaction of a breakwater foundation due to swell action.

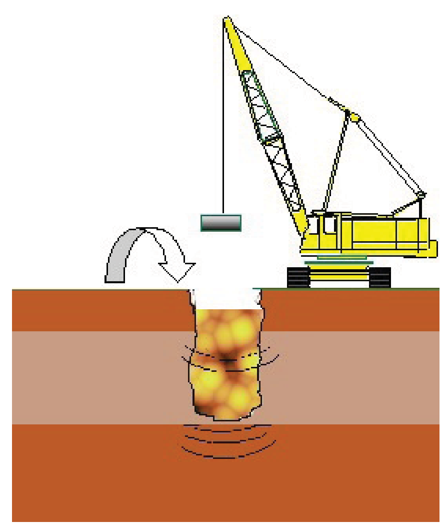

Fig. 1: The process of dynamic replacement

In Sfax Fishing Quay a 17 ton pounder was used to compact 3 to 4 m of loose silty sand to provide bearing for the gravity type quay walls.

In Lagos, a 40 ton pounder was used to compact 10 m of silty sand. The seabed was 15 m deep.

Gambin (1982) predicted that the combination of engineering knowledge and equipment capable of lifting heavy pounders will make it feasible to improve much deeper seabed and to help the oil industry in open sea. Regretfully, this prediction, although achievable, went unnoticed for a much longer time than Gambin had anticipated.

Although a water depth of 15 m will suffice the needs of many ports and other marine and offshore facilities, the developments in Panamax ships, Post-Panamax ships and super tankers and the needs of the oil industry has introduced the requirement of ground treatment at water depths in excess of 20 m. Consequently, the need has arisen to treat seabed with water depth almost double of what had previously been carried out by dynamic compaction and dynamic replacement.

The concept of the subject of this paper was to explore the possibility of performing dynamic replacement at water depths almost double the previous works and to verify the achievements and estimate the soil parameters using the Menard pressuremeter test (PMT).

DEEP WATER DYNAMIC REPLACEMENT TRIAL

Recently, a full scale trial of dynamic replacement was carried out in an area of 22.5 m by 22.5 m in Southeast Asia with the objective of treating soft marine deposits at the depth of 30 m below seawater level. To the knowledge of the authors, this is a world record in offshore dynamic replacement.

The Geotechnical and Geo-environmental Conditions

The original seabed of the site was composed of very stiff clay with shear strength of about 250 kPa and SPT blow counts exceeding 50. However dredging works and reducing the seabed level to about -30 m CD (Chart Datum) disturbed the upper 1 to 1.5 m of the superficial clay layer and post dredging geotechnical tests indicated that the clay's shear strength had dropped within one to two weeks to about one third of its original value; i.e. to about 80 kPa. Further testing at later stages by the Pressuremeter test suggested that the shear strength had even further reduced from this value to only 16 kPa.

Further removing the softened clay and replacing it with more suitable granular soil did not appear effective because it was expected that this would lead to the disturbance of deeper clay layers and the problem would persist.

Being in the open sea, the tide in the location of the site was expected to be within the range of 5 m per day.

The Proposed Solution: Offshore Dynamic Replacement

Due to the nature of the soft soil and its thickness, dynamic replacement was envisaged as a possible treatment solution. Based on previous experiences, it was anticipated that if proper equipment; i.e. a large enough stable barge, a special crane with a sufficiently powerful winch system for lifting a heavy pounder and resisting tidal action, and a special pounder for transmitting sufficient energy at seabed level were available, it would then be possible to drive granular material into the soft clay and improve its properties.

Unlike land based dynamic replacement where suitable material can be pushed into the crater by a loader either directly from the transition layer or from dumped truck loads, in offshore dynamic replacement this possibility does not exist, and material can only be punched in from the transition layer.

In addition to distributing the load by arching (Hamidi et al., in review), the transition layer in offshore dynamic replacement also prohibits the contamination of seawater by the flow and dispersion of suspended clay particles caused by the pounder impact.

In the proposed dynamic replacement methodology it was assumed that a granite rock fill blanket would be placed over the soft clay layer. The blanket material was chosen in such a way that 30% of the stone diameters were from 150 to 200 mm and the remaining 70% were from 200 to 300 m.

DiMaggio (1987) has proposed the application of equivalent strength approach using homogenized soil properties in lieu of the granular column and soft soil properties:

$$\gamma_{eq} = a_c \gamma_c + (1- a_c)\gamma_s \tag{1}$$

$$\gamma_{eq} = a_c c_c + (1- a_c)c_s \tag{2}$$

$$E_{eq} = a_c E_c + (1- a_c)E_s \tag{3}$$

$$\tan\varphi_{eq} = a_c \tan\varphi_c + (1- a_c) \tan\varphi_s \tag{4}$$

γ_{eq}= equivalent density
c_{eq}= equivalent cohesion
E_{eq}= equivalent modulus of deformation
φ_{eq} = equivalent internal friction angle
γ_c= DR column density
γ_s= subsoil (soft soil) density
c_c= DR column cohesion
c_s= subsoil (soft soil) cohesion
E_c= DR column modulus of deformation
E_s= subsoil (soft soil) modulus of deformation
φ_c= DR column internal friction angle
φ_s= subsoil (soft soil) internal friction angle
a_c= area replacement ratio, equal to the ratio of the area of a DR column, Ac, to the area of the DR unit cell (or the plan area of ground per DR column), A, shown as the shaded area in Fig 2.

$$a_c = \frac{A_c}{A} \tag{5}$$

Eq. 4 was based on the original understandings of the composite column-soil behavior and with the thought that that for shear loading undrained soil conditions were appropriate. However, as additional projects and knowledge progressed the understanding, the importance of the load distribution ratio greatly increased and a move toward a drained strength approach was developed (DiMaggio, 2009). For long term strength Eq. 6 (Terrasol, 2005; Arulraja et al. 2009) is used:

$$\tan\varphi_{eq} = m \tan\varphi_c + (1- m) \tan\varphi_s \tag{6}$$

where

m = load distribution ratio, and equal to

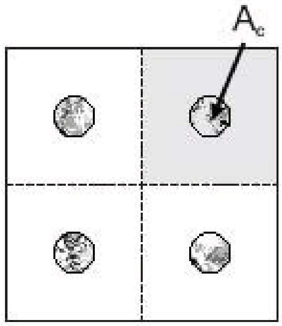

Fig. 2: Dynamic replacement unit cell

$$m = \frac{a_c n}{1 + a_c(n-1)} \tag{7}$$

and

$$n = \frac{E_c}{E_s} \tag{8}$$

For the purpose of the trial, the dynamic replacement grid was specified to be 4.5 m by 4.5 m. Consequently, there were a total number of 25 columns in the trial. Upon construction of the DR columns, an ironing phase was also perceived to compact the rock fill blanket.

Verification of the initial ground conditions and the improvement results was to be done by using the pressuremeter test. The DR impact points and testing locations are shown in Fig. 3.

The Challenge: Execution of Dynamic Replacement Works

Prior to commencement of dynamic replacement works and as per the design a 1.8 m rock fill blanket was placed over the soft seabed.

As shown in Fig. 4 and based on Menard's past experiences and projects a specially designed offshore pounder with a base size of 1.7 m by 1.7 m and weighing 38.5 tons was fabricated. The special shape of this pounder allowed minimum water resistance during the pounder's drop and penetration in the seawater.

Due to the relatively heavy and out of norm weight of the pounder a heavy duty crane with sufficient stability and winch capacity was required to lift the pounder. For this purpose an adapted HS895 Liebherr crawler crane was used.

As shown in Fig. 5, a minimum barge size of 15x50 m^2 was assumed to be sufficient for supporting the crane, pounder and other equipment and executing the works.

Each dynamic replacement print location was subject to 30 blows. Furthermore, 3 to 6 blows were applied as the ironing phase over the entire treatment area.

Previous experiences by the working team suggested that water resistance will nullify the effect of significantly high drops. Hence, the drop height during the trial was set at 5m.

Measurement records of the crane's winch speed during the works indicate that the maximum drop speeds were in the range of 430 m/min. This speed is the equivalent to a free fall with a drop height of 2.6 m and verifies the original assumption that excessive drops would not have increased the impacts' kinematic energy.

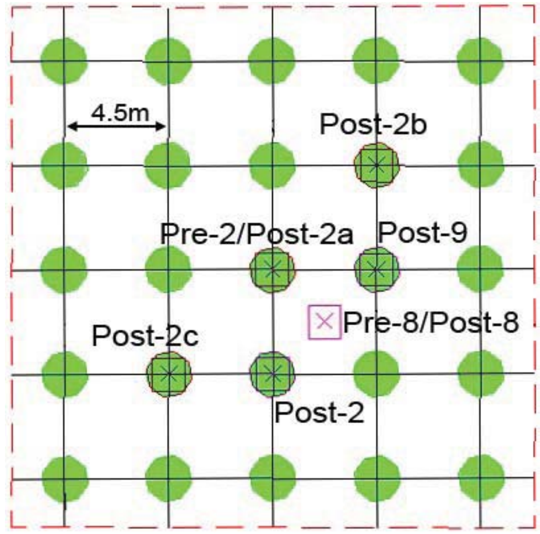

Fig. 3: Dynamic Replacement and Pressuremeter test locations


Fig. 4: Specially designed and fabricated offshore DR pounder (Chu et al., 2009)

Verification of Dynamic Replacement Works

Due to the importance and uniqueness of the trial and the technical influences that it was expected to have on future procedures of deep water dynamic replacement and ground improvement works, in addition to the pressuremeter tests that were carried out as the primary verification procedure, additional measurements and observations were also made to realize the maximum amount of information.

As shown in Fig. 6 the penetration of the pounder into the ground was measured for every blow. It can be observed that while the pounder penetrated the ground at a more pronounced rate during the first four blows, the penetration rate then rapidly decreased to the point where it appears that no penetration was practically observed after the 15th blow.

During the first 15 blows, the amount of pounder penetration was variable from 1.1 to 1.7 m. Comparing these figure with the thickness of the soft soil prior to dynamic replacement, it can be interpreted that the pounder impact was able to effectively drive the granular material of the blanket to the end of the soft soil layer with the first 4 to 12 blows and then to further compact the granular rock fill. It can also be observed that the maximum penetration values per print are sometimes more than the assumed soft soil layer's thickness. This indicates that either the DR columns have penetrated into the stiffer clay or that the actual soft layer's thickness was more than originally anticipated at some locations.

Fig. 5: Implementation of a heavy duty crane on a 15x50 m^2 barge

The total ground settlement was measured by echo sounding and the survey showed that the top of the blanket had dropped by 38 cm as a result of the ground improvement works.

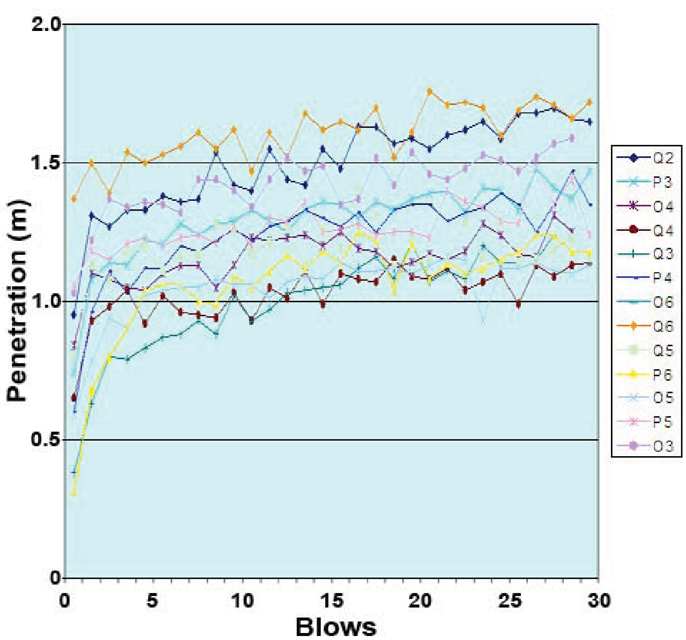

Fig. 6: Pounder penetration at each DR print

Divers were also sent to visually scrutinize the impact results. Based on the larger amount of crushed rock at the DR column location, they were able to confirm that the column diameters were 2.4 m. This figure is equal to the diagonal length of the pounder's base. It can be interpreted that the larger DR columns' diameter as compared to the pounder's base may have been formed by a combination of soft soil being pushed away laterally due to the high horizontal stresses exceeding the soil's strength at impact location and possible rotations of the pounder during the impacts.

Due to the large water depth and open sea working conditions pressuremeter tests were carried out by using 100 mm guide tubes followed by the 60 mm PMT tube. A 63 mm slotted casing Menard type pressuemeter was used for the verification according to ASTM (2007).

During the PMT, visual observation on the return of drilling fluid was recorded. When there was no return of drilling fluid, it indicated that the test was carried out in the free-draining rock material whereas testing in impervious clay was indicated by the return of the drilling fluid.

As shown in Fig. 3, during the process of the trial two pressuremeter tests (Pre-2 and Pre-8) were carried out prior to dynamic replacement and six were carried out after treatment (Post-2, Post-2a, Post-2b, Post-2c, Post-8 and Post-9). A summary of the pre and post treatment tests (pressuremeter modulus, E_p, and limit pressure, P_l) are tabulated in Table 1 and Table 2.

Table 1: Pre-treatment PMT results

Test No.	Depth (m)	E_p (MPa)	P_l (MPa)	Comment
Pre-2	-29.1	1.63	0.34	rock fill
	-29.9	0.17	0.09	clay
Pre-8	-28.7	3.75	0.63	rock fill
	-29.9	11.34	1.44	clay

Table 2: Post treatment PMT results

Test No.	Depth (m)	E_p (MPa)	P_l (MPa)	Comment
Post-2a	-29.2	3.56	0.79	rock fill
	-30.0	6.34	1.17	rock fill
Post-2b	-29.1	22.22	2.82	rock fill
Post-2c	-29.1	6.86	1.32	rock fill
	-29.9	2.64	0.78	rock fill
	-30.7	7.98	1.40	rock fill
Post-2	-29.3	7.04	0.99	rock fill
	-30.2	7.34	1.63	rock fill
Post-9	-29.0	9.13	1.36	rock fill
	-29.8	7.37	1.78	rock fill

Table 3: Cyclic post treatment PMT results

Test	Depth (m)	E_R (MPa)	E_A (MPa)
Post-2	-29.3	15.61	13.96
	-30.2	9.22	6.96
Post-9	-29.0	13.51	9.82
	-29.8	12.10	9.13

It was observed that Post-8 registered a non-yielding curve with a high value of P_l, probably due to a localized closer matrix of rock pieces in the vicinity of the slotted casing and as such was deemed as non-representative and excluded.

In addition to the above mentioned tests and as shown in Table 3, at the location of Post-2 and Post-9 cyclic PMT was carried out as well. E_R denotes the PMT reload modulus.

It can be observed that the P_l value of the soft clay layer was measured to be less than 0.1 MPa which indicates the very low strength of the material.

The comparison of Pre-2 and Post-2a that were done in the almost same location indicates that while the rock fill has been driven into the soft clay, its E_p and P_l values have also increased respectively by 118% and 132% to more than double their original values.

The average values of E_p and P_l after improvement were respectively 8.05 MPa and 1.40 MPa which yields an average ratio of 5.75 for E_p/P_l in the DR columns. The maximum P_l that was recorded during the test exceeded 2.2 MPa.

It can be readily calculated that for a 4.5 m by 4.5 m grid with DR columns with a diameter of 2.4 m, the area replacement ratio, A_c is 22.3%. It can also be calculated that the harmonic mean of E_p in the rock fill after improvement is equal to 6.03 MPa. The Young modulus of the clay and rock fill can also be calculated from (Menard, 1975)

$$E = \frac{E_p}{\alpha} \tag{9}$$

α= rheological factor, ¼ for rock fill and ½ for altered clay.

Hence, E_s= 0.34 MPa and E_c= 24.12 MPa and consequently n= 70.9 (Eq. 7) and m= 95% (Eq. 6).

The shear strength parameters can also be estimated from the pressuremeter test. According to Baguelin et al. (1978), Menard (1970) proposes

$$c = \frac{P^*_l}{5.5} \tag{10}$$

P_l^*= net limit pressure and can be calculated from

$$P^*_l = P_l - P_o \tag{11}$$

P_o= at rest horizontal earth pressure at the test level at the time of the test. Briaud et al. (1986) note that P_o can be obtained from the beginning of the pre boring PMT curve (starting point of the pressure at pseudo-elastic phase of the straight line portion of the pressure-volume curve) provided that sufficient number of data points are collected.

Baguelin et al. also state that Menard (1970) proposes that for sands

$$P^*_l = 2.5 \times 2^{\frac{\varphi - 24}{4}} \tag{12}$$

However, it is the experience of the authors that Eq. 12 under estimates the friction angle in rock fill and it would be more appropriate to use

$$P^*_l = 2.5 \times 2^{\frac{\varphi - 40}{7}} \tag{13}$$

c_c and φ_s can be assumed to be zero. From Eq. 10, c_s= 16 kPa. c_{eq} can be calculated from Eq. 3 to be equal to 12 kPa. The harmonic mean of φ_c is 49°. From Eq. 6, φ_{eq} can be calculated to be 47°. Based on these values, a finite element model can be constructed with the parameters of Table 4.

Table 4: Equivalent parameters for finite element model

Layer	elevation (m)	E (MPa)	c (kPa)	φ^o
rock fill	0 to -1.3	24.1	0	49
composite	-1.3 to -2.8	18.7	12	47

CONCLUSION

The offshore dynamic replacement trial has demonstrated that it is possible to perform this technique and to perform verification by PMT at the depth of 30 m and even deeper. Test results can be used for constructing suitable models for required analyses.

In addition to the knowledge of how to perform DR, other parameters that should be taken into consideration are:

- Barge size: The barge must be large enough to safely support the personnel and equipment. The barge size will be influenced by the location of the project and the sea conditions.
- Pounder: offshore pounders must be designed to minimize water resistance. As water resistance reduces impact energy, marine pounders are generally heavier than land pounders. This is to compensate the impact energy losses.
- Rig size: The rig should be sufficiently large to provide the required stability and winch capacity.
- Supply of material: Suitable material (rock fill) must be placed on the seabed prior to commencement of works.
- Grid size and number of blows: These parameters appear not to be very different from the parameters that would be used for land based DR.

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