OFFSHORE GROUND IMPROVEMENT RECORDS

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ABSTRACT

Numerous ground improvement technologies have been in use for many years on land based projects with various applications. These techniques have provided alternatives that are frequently more affordable and require shorter construction periods than deep foundations. Implementation of these methods in the sea and marine environments is more challenging as specialised equipment are usually either only appropriate for land based projects or have low efficiency and production capability at sea. However, requirement of seabed treatment and improving the characteristics of marine foundations has necessitated the introduction of soil improvement technologies to offshore projects. Some of the ground improvement techniques that have especially evolved to satisfy the requirements of offshore and seabed ground improvement are dynamic compaction, vibro compaction, dynamic replacement, and stone columns. The first two techniques are used for the treatment of granular seabed while the latter two technologies are most appropriate for improving silty and clayey marine foundations. In this paper initially marine and offshore ground improvement techniques with a focus of the mentioned above methods will be discussed. Two case studies of ground improvement for the treatment of soft clays in record water depths will also be introduced. In the first case offshore dynamic replacement was carried out in Southeast Asia at a location where seabed was approximately 30 m below sea level. In the second project stone columns were installed beneath the quay wall and breakwater of the first and second phases of Port of Patras (Greece). The sea depth was up to approximately 40 m and the columns were as long as 20 m.

1. INTRODUCTION

Ground improvement, as we know it by its modern definition, began to take the form of a branch of geotechnical engineering in the mid-20th century, and was finally realised as the 17th technical committee of ISSMGE many years ago (Varaksin and Hamidi, 2012). While it may not be immediately apparent, ground improvement methods have made considerable advances since today's commonly practiced techniques first began to develop and evolve in the first half of the 20th century; however most techniques have gone through changes, mostly due to new ideas, advances and innovations in equipment and technological capabilities and the emergence of newer technologies has provided the geotechnical engineer with additional tools for optimising foundation design and treatment of particular soils.

It can be observed that the notion of improving the ground for engineering purposes initially developed implicitly to resolve subaerial issues as foundation problems were and are most often encountered on land due to the fact that the percentage of marine foundations is much less than overland foundations. However, the 20th century was witness to a number of marine and onshore geotechnical failures such as the 1916 collapse of Gothenburg Harbour's Stigberg Quay in Sweden (Massarsch and Fellenius, 2012) and the 1979 failure of Nice Harbour in France (Dan et al, 2007). Hence, it was inevitable that sooner or later attention would be drawn towards modifying or adjusting ground improvement techniques for application to subaqueous near shore and offshore projects.

1.1. DYNAMIC COMPACTION AND DYNAMIC REPLACEMENT

Louis Menard invented and promoted Dynamic Compaction (DC) as early as 1969 but it was not until 29 May 1970 that he officially patented his invention in France. The technique was later also patented in many other countries, including Australia in 1981 (Hamidi et al., 2009a).

The concept of this technique is improving the mechanical properties of the soil by transmitting high energy impacts to loose soils that initially have low bearing capacity and high compressibility potentials. The impact creates body and surface waves that propagate in the soil medium. In non-saturated soils the waves displace the soil grains and re-arrange them in a denser configuration. In saturated ground the soil is liquefied and the grains re-arrange in a more compact state. In both cases the decrease of voids and increase in inner granular contact will directly lead to improved soil properties. Impact energy is delivered by dropping a heavy weight or pounder from a significant height. The pounder weight is most often in the range of 8 to 25 tons although lighter or heavier weights are occasionally used. Drop heights are usually in the range of 10 to 20 m although other drop heights may sometimes be used. Hamidi et al. (2011) have described the advances of dynamic compaction.

Dynamic Replacement (DR) is a ground improvement technique that was also developed by Louis Menard in 1975 for the treatment of soft cohesive soils. As shown in Figure 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.

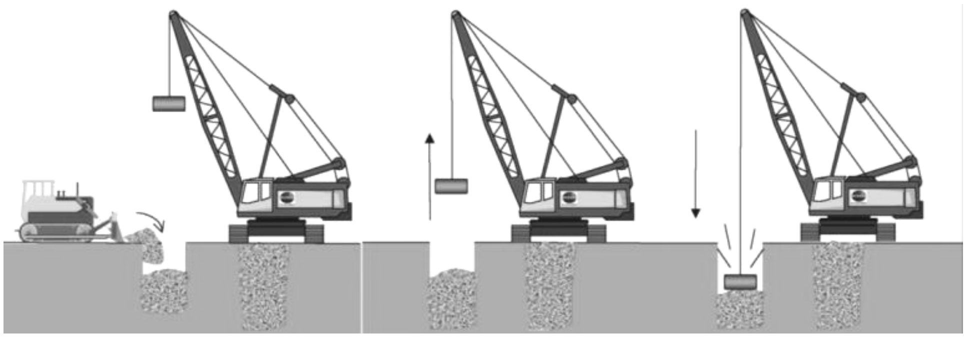


Figure 1: The process of dynamic replacement

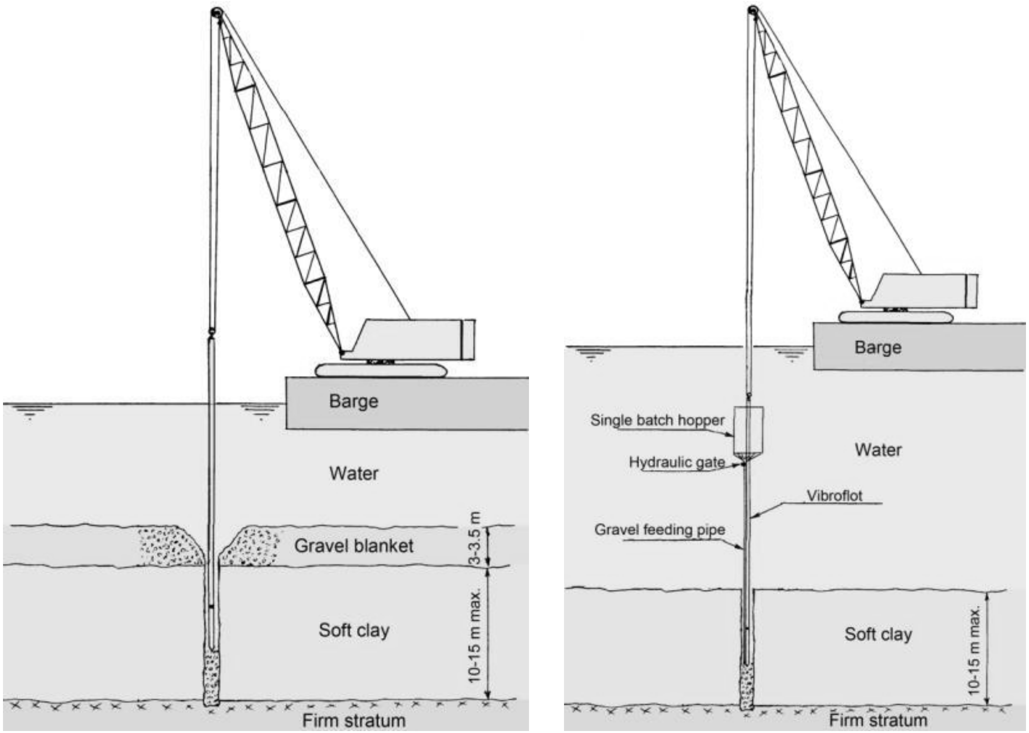
1.2. VIBRO COMPACTION AND VIBRO REPLACEMENT (STONE COLUMNS)

Vibro compaction, also known as vibroflotation, is a deep ground compaction technique that was developed in 1934 with the invention of the first vibroflot by Degen and Steuermann (Better Ground website) in Germany. This technique is best suitable for the treatment of soils with limited amounts of fines. Mitchell (1981) proposes that the best desirable soils for vibro compaction are when the ground's fines content is limited to 18%. As also deemed more appropriate by the authors through their personal experience, Woodward (2005) proposes that best results can be achieved when fines content is less than 10%.

The vibroflot, sometimes also referred to as a vibroprobe or vibrating poker, is a hollow steel tube containing an eccentric weight mounted on a vertical axis in the lower part so as to give a horizontal vibration. The vibroflot itself is connected to extension tubes that are supported by a rig, usually a crane. The vibroflot is either flushed down to the required depth in the soil using water jets or vibrated dry with air jets. When the vibroprobe reaches the required depth, during withdrawal, material is added from the ground surface, and the vibroflot is moved in an up and down motion at certain intervals. The horizontal vibrations form a compacted cylinder of soil with a depression at the surface due to the reduction of void ratio in the ground. Depending on the vibroflot power, the zone of improved soil extends from 1.5 m to more than 4 m from the vibrator.

When fines content is high the vibroflot is used in an alternative process called vibro replacement or stone columns. In this method, crushed stones are fed into columnar cavities and compacted using the vibroflot to form semi rigid columns. The common construction methods for stone columns include the wet top feed and the dry bottom feed methods. The major difference between these two processes is the stone feeding system whereas in the top feed method stone is fed to the column from the ground surface while in the bottom feed method stone is fed to the tip of the vibroflot through a pipe.

Execution of top feed and bottom feed stone columns is more challenging when works are to be performed offshore and in marine conditions. In the marine wet top feed method, a 3 to 3.5 m thick gravel blanket is initially placed on the seabed. This blanket will feed the stone columns. In this process the equipment weight will be less than what will be required by the dry bottom feed method for the same treatment depth which is an advantage, but maintaining the annular space around the vibroflot is more challenging than land based work due to the absence of water head difference in between the hole and the surrounding ground. Also, the maximum stone column lengths that can be constructed using the gravel blanket are in the order of 10 to 15 m as longer columns may be starved out of stone in the top metres of the columns. Further advantages and numerous drawbacks of this construction method have been described by Debats and Degen (2001). The blanket wet top feed method is shown in Figure 2(a).



(a) (b)

Figure 2: Marine stone columns (a) blanket wet top feed and (b) single batch wet bottom feed methods (Debats and Degen, 2001)

In the single batch wet bottom feed stone is fed to the tip of the vibroflot via a feeding pipe with a large hopper at its head. The hopper has a capacity in excess of the expected stone consumption for one column, and is equipped with a hydraulically operated gate that controls the discharge of the stones into the feeder pipe. The advantages and drawbacks of this method are also described by Debats and Degen (2001). The single batch wet bottom feed method is shown in Figure 2(b).

2. MARINE GROUND IMPROVEMENT

The first applications of marine ground improvement can be traced back to the 1970s. Menard carried out the first offshore dynamic compaction project in 1973 as part of the construction of Brest Naval Port's dry dock in France. In this project a specially designed 11 ton pounder was used to compact 3 m of loose alluvium on the seabed (Menard, 1974; Boulard, 1974; Renault and Tourneur, 1974; Gambin, 1982). 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 (Gambin, 1982; Chu et al., 2009). Other dynamic compaction or dynamic replacement projects with seabed as deep as 15 m below seawater level included Pointe Noire in Gabon (Menard, 1978), Uddevalla Shipyard Wharf (Gambin, 1982), Kuwait Naval Port, Sfax Fishing Quay in Tunisia (Menard, 1981; Gambin, 1982), and Lagos Dry Dock in Nigeria (Gambin, 1982; Gambin and Bolle, 1983). More recently a deeper marine dynamic replacement has been reported by Hamidi et al (2010) and Yee and Varaksin (2012).

The first marine ground improvement project in Australia has been carried out as part of the expansion of Port Botany in Sydney. In this project 800,000 m^3 of sand was compacted using the marine vibro compaction technique to support the precast counterfort retaining walls (Berthier et al., 2009). Other published references to projects using the vibroflot techniques include Port of Patras (Debats and Degen, 2001, Loukakis and Yegian, 2004), Port of Monaco (Debats and Londez, 2003), Bay Area Rapid Transit (BART) in San Francisco Bay, USA (Wu et al., 2003), and the Golden Ears Bridge in Vancouver, Canada (Naesgaard, 2008). Other unpublished work include Aktio-Preveza Crossing (stone columns) in Greece, Bali Wharf (stone columns) in Indonesia, Cuenca de Plata Terminal (vibro compaction) in Montevideo, Dung Quat Refinery (vibro compaction) in Vietnam, Dunkirk Port and Dunkirk LNG Terminal (stone columns) in France, National City Marine Terminal (stone columns) in the USA, North Lantau Expressway (vibro compaction) in Hong Kong, Pasir

Panjang Terminal Phases 3 and 4 (vibro compaction) in Singapore, and Richards Bay Berth 306 (stone columns) in South Africa.

An advance in construction methods and equipment has enabled ground improvement to be carried out in more challenging conditions and depths. The focus of this paper will be the case history of two world records for treating seabed using the dynamic replacement and stone columns techniques.

2.1. MARINE DYNAMIC REPLACEMENT FOR A CONTAINER TERMINAL IN SOUTHEAST ASIA

Recently, dynamic replacement was carried out in Southeast Asia to treat soft marine deposits more than 30 m below seawater level for the construction of a wharf using caisson seawalls (Hamidi et al, 2010; Yee and Varaksin, 2012).

According to the original design the soft marine clay at the seabed was to be dredged down to the depth of 30 m below sea level where the shear strength of the stiff clay exceeded 250 kPa. The excavated key was to be then backfilled with sand and compacted using vibro compaction under 3 m of additional overburden sand fill. Next, the surcharge had to be removed, a rubble mound was to be placed over the sand key, and as shown in Figure 3, finally caissons were to be sunk onto the mound.

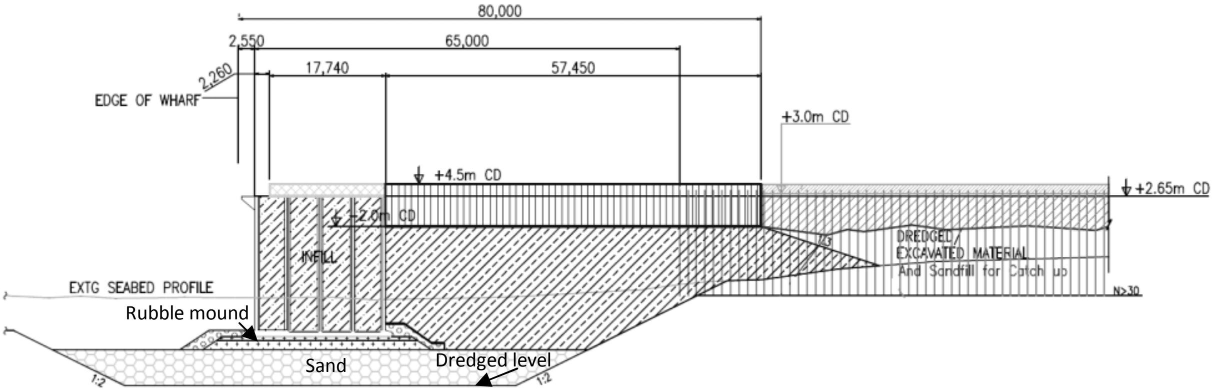


Figure 3: Cross section of container terminal based on original foundation concept

2.1.1. Soil Softening

As SPT blow counts exceeded 50 and the assumed clay shear strength of 250 kPa was achieved at dredge level, works progressed by backfilling sand and compacting the fill using vibro compaction.

While the clay at dredge level was initially very stiff, dredging works and cutting into the clay softened the upper 1 to 1.5 m of the exposed clay surface and post dredging CPT tests performed before the removal of the overburden sand fill indicated that the clay’s shear strength had dropped to about one third of its original value; i.e. to approximately 80 kPa. Further testing at later stages by the pressuremeter test (PMT) suggested that the shear strength had even further reduced at some points to a mere 16 kPa.

2.1.2. The Solution: Offshore Dynamic Replacement

Further dredging of the softened clay and replacing it with more sand fill did not appear to be effective method because it was expected that this would lead to the disturbance of deeper clay layers and the problem would persist.

Due to the nature of the soft soil and its thickness, marine dynamic replacement was envisaged as a possible treatment solution. Based on previous experiences, it was anticipated that if proper equipment; i.e. a large stable barge, a specialised crane with a sufficiently powerful winch system for lifting a heavy pounder and resisting tidal action, and a special pounder for transmitting sufficient impact energy to the seabed 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, in offshore dynamic replacement this is not possible, and material can only be punched in from the transition layer. Hence, a stone blanket was used to feed the DR columns and to provide the transition layer for arching (Hamidi et al, 2009b). This layer also prevented the contamination of seawater by the flow and dispersion of suspended clay particles produced by the pounder’s impacts.

In the proposed dynamic replacement methodology it was assumed that a 1.8 m thick granite rock fill layer 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. The DR rock columns were designed to be 2 m in diameter, in a 4.5 m grid and with a replacement ratio of 15%.

As shown in Figure 4(a), in this project a specially designed grater shaped marine pounder weighing 38.5 tons was used to drive the rock into the columns and to dynamically compact the rock blanket. The pounder's dimensions were 1.7 m by 1.7 m on the DR side and 2.3 m by 2.3 m on the DC side. Figure 4(b) shows the 15x50 m^2 barge that was used for supporting the crane, pounder and other equipment used for executing the ground improvement works.

Previous experiences by the working team suggested that water resistance could greatly reduce the effect of significantly high drops. Hence, the drop height during the trial was set to 5 m above seabed level. 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 equivalent to a free fall with a drop height of 2.6 m (in air) and verifies the original assumption that much of the drops' kinematic energies would have been lost to water resistance.

(a) (b)

Figure 4: (a) Marine DR (bottom side) - DC (top side) pounder (Chu et al., 2009), (b) barge mounted crane used for offshore dynamic replacement (Hamidi et al., 2010)

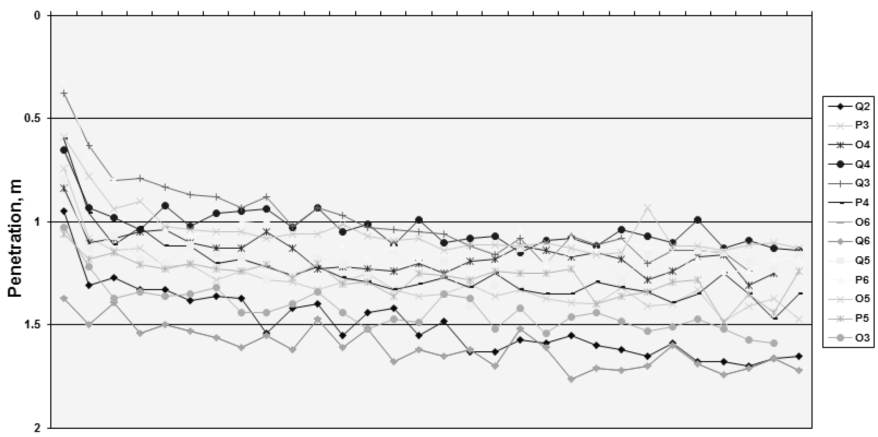


Figure 5: Pounder penetration at several DR print locations

Each dynamic replacement print location was subject to 30 blows. Furthermore, 3 to 6 blows were applied as ironing using the larger end of the pounder. As shown in Fig. 5 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. 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 location 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.

The total ground settlement was measured by echo sounding and the survey showed that the top of the blanket had dropped by 0.38 m as a result of the ground improvement works.

2.1.3. Verification and Results

Divers were used to visually inspect the impact results at seabed level. Based on the larger amounts of crushed rock at the DR column location, it was determined that the columns were 2.4 m in diameter which is equal to the diagonal length of the pounder's base on the DR side. 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. Thus, the actual DR replacement ratio was 22.3% in lieu of the assumed 15%. Target rock friction angle was 45 degrees.

Due to the large water depths and open sea working conditions pressuremeter tests were carried out using 100 mm guide tubes followed by the 60 mm PMT tube. A 63 mm slotted cased Menard pressuemeter was used for the verification. During testing 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. Two pressuremeter tests (Pre-2 and Pre-8) were carried out prior to dynamic replacement and six (Post-2, 2a, 2b, 2c, 8 and 9) were carried out after treatment. The summary of the pre and post treatment tests (pressuremeter modulus, E_p, and limit pressure, P_l) are tabulated in Table 1. 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.

Table 1: Pre-treatment and post 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
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

The comparison of Pre-2 and Post-2a PMT 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%. The average values of E_p and P_l after improvement were respectively 8.05 MPa and 1.40 MPa. The maximum P_l that was recorded during the test exceeded 2.2 MPa. 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)

$$E = \frac{E_p}{\alpha} \tag{1}$$

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

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

$$c = \frac{P*_l}{5.5} \tag{2}$$

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

$$P*_l = P_l - P_o \tag{3}$$

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

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

However, it is the experience of the authors that Eq. 4 underestimates the friction angle in rock fill. The authors note that there is a typing mistake in Equation 13 of Hamidi et al (2010), and the corrected formula as presented in Equation 5 should be used.

$$P*_l = 4 \times 2^{\frac{\varphi - 40}{7}} \tag{5}$$

Based on these values presented in Hamidi et al. (2010), a finite element model can be constructed with the parameters of Table2.

Table 2: Equivalent parameters for finite element model

Layer	elevation below seabed level (m)	E (MPa)	c (kPa)	φ°
rock fill	0 to -1.3	24.1	0	49
composite	-1.3 to -2.8	18.7	12	47

2.2. MARINE STONE COLUMNS FOR PORT OF PATRAS

Patras is Greece's third largest urban area and the regional capital of West Greece. It is located in northern Peloponnese, 215 km west of Athens, and its port is the gateway of the country to Italy and Western Europe.

It has been known since the construction of the main part of old Patras Harbour and its northern extension that the site was founded on a normally consolidated soft clay layer that was 30 to 38 m thick. The marine structures that were then constructed were built directly on the soft substratum without ground improvement. Views towards this type of construction changed when in late February 1984 a series of moderate earthquakes of magnitude 3.5 to 4.5 occurred in the Patras Gulf. Immediately after these earthquakes settlements in the order of 3 to 4 m were measured on the constructed part of the southern extension of the breakwater. Research by Memos and Protonotarios (1992) indicates that these relatively small earthquakes were sufficient to trigger the failure mechanism of the structure; with the main reason being considerable amplification of the moderate underground seismic motion and further reduction of the already marginal static safety factor due to the presence of the deep soft clay stratum.

Phases 1 and 2 of the new Port of Patras have been constructed approximately 2 km to the south of the old Harbour. Phase 1 includes a 500 m long quay and 900 m of breakwater. Phase 2 of the project includes the extension of both the breakwater and quay wall by approximately 400m. The quay wall has been constructed using precast concrete caissons, each weighing approximately 1400 tons. The breakwater is a composite structure consisting of caissons that rest on approximately 20 to 30 m of rock fill embankment.

The site conditions at the location of the dual phases of the new port are also similar to the old Harbour. Water depth ranges from 10 to 15 m at the quay wall to 30 to 40 m at the breakwater. The soil profile includes very soft sandy silty clay of low to medium plasticity extending 10 to 25 m below seabed along the quay wall front, up to 35 m in the quay wall backfill area, and 5 to 15 m along the breakwater. These layers have been characterised as having very low shear strength and as being highly compressible. Below the soft soils are 5 m of stiff clay followed by up to 70 m of dense sands and gravels and up to 200 m of marl bedrock. The seabed has a unique feature due to the presence of numerous craters with depths of 0.5 to 15 m and diameters of 25 to 180 m. These craters have been created by release of gases that are trapped at the interface of the granular layer and the overlying fine layer. In the breakwater area craters often extend through the top clayey layers and reach the underlying dense sand and gravel layers. (Loukakis and Yegian, 2004).

Active seismicity in the region and the failure of the old Harbour's breakwater stipulated implementation of special measures to ensure that the same would not be repeated in the new facilities; hence ground improvement was incorporated in the scope of works of both phases.

2.2.1. The Solution: Offshore stone columns

The original Phase 1 quay wall ground improvement design was based on the removal of 2 m of the very low strength clay layer and its replacement with sand and gravel. Next, wick drains were to be installed to a depth of 19 m, and the seabed was to be preloaded in two phases. In the first phase, the preloading embankment was to be raised to elevation -14 m RL (reduced level) in the stabilising berm area, and to elevation -11 m RL in the quay wall and backfill areas. After an 8 month waiting period, the preload height was to be lifted to -11 m RL in the stabilising berm area and to ±0 m RL in the quay wall and backfill areas as the second phase of preloading. The second wait period before removal of the preload was designed to be 12 months for reaching 80 to 90% consolidation. The final stage of ground improvement was envisaged to be the installation of 10 m long 0.6 m diameter stone columns within an 80 m wide zone (30 m in the front and 50 m behind the quay wall (Drettas et al, 1997; Loukakis and Yegian, 2004). The 12.8 m high caissons would then be sunk onto their insertion locations.

Additional geotechnical investigation performed by the contractor revealed an extremely irregular seabed crater pattern, particularly in the breakwater area. Furthermore, this investigation also identified several previously unknown thin sand layers within the top clayey soils. These layers appeared to be able to reduce consolidation period and downgraded the effect of the vertical drains, and evaluation of trail embankment monitoring results indicated practically no effect of vertical drain spacing on the consolidation rate.

Tender stone column construction method assumed that works would be performed in the quay wall area either as a land operation prior to the removal of the preloading embankment or as a marine operation after removal of the preload. However, stone column trials demonstrated that vibroflot penetration into the surcharge was extremely difficult due to the composition and degree of surcharge embankment compaction. The trials also showed that a layer of approximately 3 m thickness had to be placed on the seabed prior to construction of the stone columns to generate the necessary overburden pressure required for mitigating bulging near the top of the stone columns.

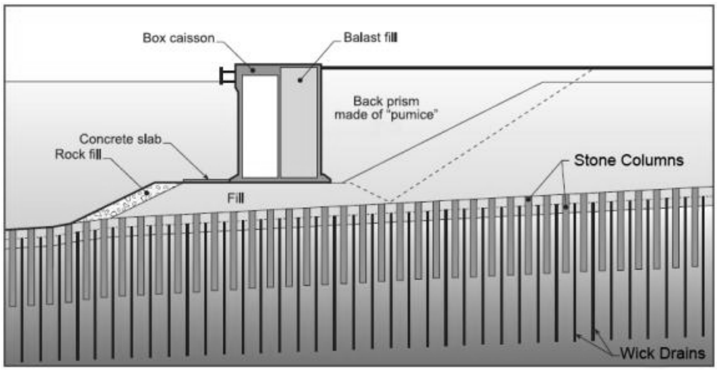
Thus, in the modified construction sequence stone columns were installed after the placement of the first stage of preloading. This modification significantly improved the ground's stability during the second stage of preloading.

Similarly in the breakwater area, the original design anticipated that the upper 2 m of the very soft seabed would be removed and replaced with sand and gravel. Next, 12 m long wick drains were to be installed and two layers of geotextiles were to cover the entire footprint of the breakwater to increase resistance against slope failure. In this area the rubble mound was designed in three stages with waiting periods in between them. Stabilising berms were to be constructed on both sides of the main rock fill embankment. The first stage included lifting the rock fill and berms to -30 m RL. After 1 to 2 months the second stage would commence, and construction would be elevated to level -24 m RL. The final stage was to begin after another 9 months of waiting period. In this stage the rock fill would be lifted to -11 m RL, and the caissons would then be sunk onto their locations (Platis et al, 1997; Loukakis and Yegian, 2004).

Construction as per the above methodology was very difficult with consideration of the seabed's crater field and water depths of approximately 40 m in some craters of the breakwater area; hence the contractor proposed an alternative construction method in which a 3 m thick gravel blanket would be placed on the seabed followed by installation of stone columns penetrating 5 to 17 m into the clayey soils and reaching the underlying granular layer. This process not only reduced construction time by eliminating a total of 20 months of waiting period but also reduced the stabilising berms on the two sides of the main rock fill embankment due to the better ground properties and increased resistance against slope failure.

During Phase 1 of the Port construction wick drain and stone columns were installed in water depths of up to 38 m. A total of about 28,000 wick drains were installed down to the depth of 19 m below seabed level at the quay wall area, and approximately 23,000 stone columns, up to 20 m long, were installed almost equally in the quay wall and breakwater areas using the single batch bottom feed technique shown in Figure 2(b). Stone columns in the quay wall area were 0.86 m in diameter and installed in a 2.85 m square grid which equates to a replacement ratio (Hamidi et al, 2009b) of 7%. In the breakwater area the stone column diameter was 1 m and the installation square grid was 2.7 m which equates to a replacement ratio of 11%. Typical cross sections of the quay wall and breakwater ground improvement schemes are shown in Figure 6.

(a)



(b)

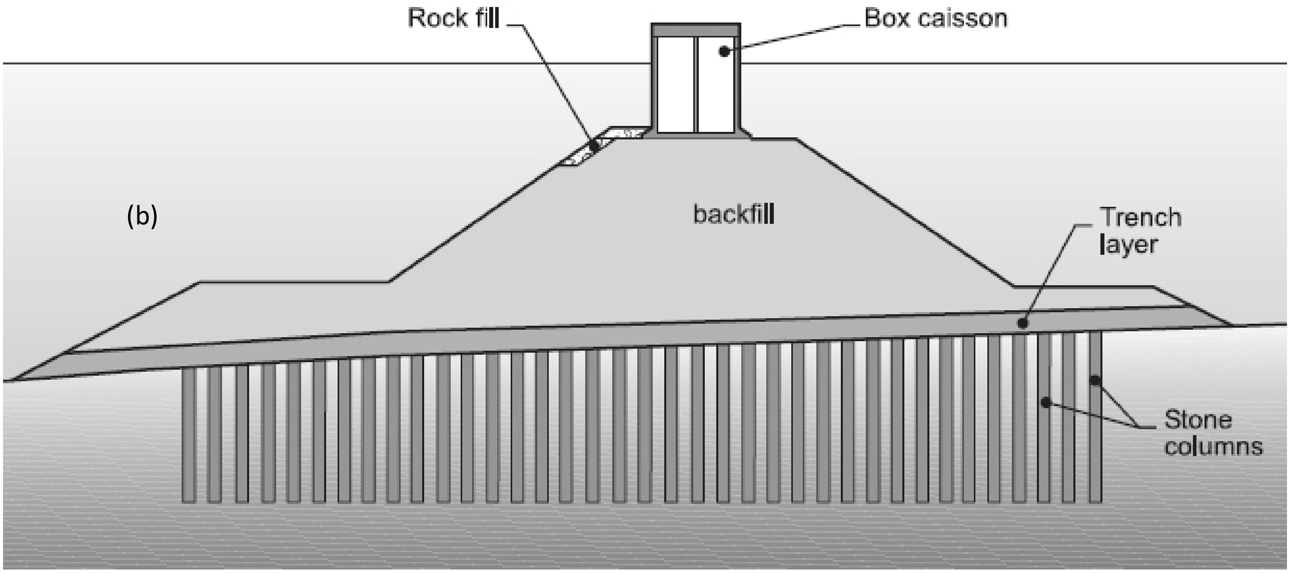


Figure 6: Typical cross sections of (a) the quay wall and (b) the breakwater

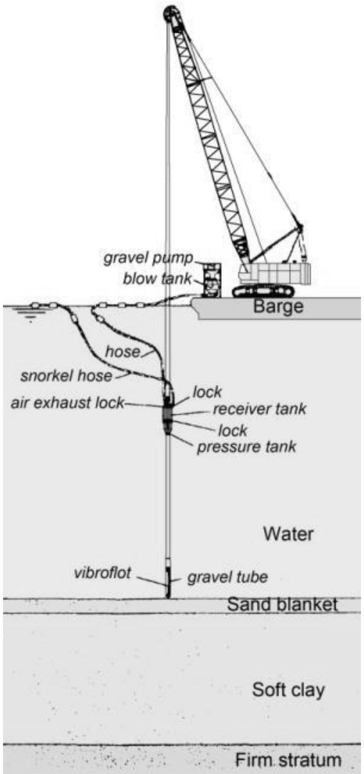
Similarly, in Phase 2 of the project, a combination of wick drains and surcharging followed by installation of stone columns was performed in the quay wall area. In this area, a total of 3,073 columns, 1 m in diameter, with an average length of 10 m, an average grid size of square 3.3 m, and with a total length of 30,730 m were installed.

In the breakwater area initially a 2 m thick sand blanket was placed on the seabed. Then stone columns were installed to depths of 50 m below seawater level. During this process a total number of 4,830 stone columns, 1 m in diameter, with an average length of 16 m, an average square grid size of 2.7 m, and a total length of 77,280 m were installed (Debats and Degen, 2001).

2.2.2. Innovation in Offshore Stone Column Technology

Lessons learned from the disadvantages and drawbacks of the then existing offshore stone column construction technologies, especially the inability to accurately measure the volume of stone used in each stone column, (Debats and

Degen, 2001) resulted in an innovative and patented bottom feed stone column technology using a double lock and gravel pump technology that was first used in the Phase 2 of Port of Patras. In this construction method that is shown in Figure 7(a) the marine double lock gravel pump has a snorkel hose that is attached to the receiver tank at the air exhaust lock. The snorkel hose and locks are operated in such a way that, regardless of water depth, there is always atmospheric pressure in the receiver tank when the gravel is being pumped into the hoses. By this means an air compressor can pneumatically move the gravel from the blow tank to the receiver tank. Since one of the locks is always closed at any one time, the high pressure is sufficient to surmount the water and soil pressures in the gravel tube at the tip of the vibroflot. Theoretically, using this technology, is should be possible to reach water depths of in excess of 200 m before the hoses fail (Debats and Degen, 2001).



(a)

(b)

Figure 7: Typical cross sections of (a) the quay wall and (b) the breakwater (Debats and Degen, 2001)

As shown in Figure 7(b) the marine double lock gravel pump dry bottom feed system used in Phase 2 of Port of Patras had a total length of 24 m which means that the system was fully submerged below sea level at all times during the installation process.

2.2.3. Quality Control

Quality control for the project was provided by logging the installation data including the start and end times, the total installation time, the penetration time, stone column diameter, the stone volume per metre, total volume, and seabed level for each column installation. Also, time and depth based graphs were prepared for each stone column with information regarding the volume of stone consumed, the diameter of the column at each level and the treatment amperage at each level.

3. CONCLUSION

Recently, offshore ground improvement technologies have had major advancements and it is now possible to treat soft or loose soils at great depth. Dynamic replacement has been used to treat soft clays at the depth of 30 m below sea level and stone columns have been installed at depth of more than 50 m. To the knowledge of the authors, both of these figures are world records for the techniques, and it is expected that ground improvement can now be more effectively implemented in deep waters based on the experience and know-how that has been gained through these projects.

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