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The Treatment of a Loose Submerged Subgrade Using Dynamic Compaction

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Abstract: It is not unforeseeable to have pavements that have lost functionality and drivability due to excessive total, differential or creep settlements; liquefaction; or local shear failures of the subgrade layer. If the subgrade material does not have the necessary strength, it may be mandatory to carry out a ground treatment program to improve the ground conditions and to allow the safe construction of the subsequent layers. The causeway of Abu Dhabi to Reem Island has been constructed by reclaiming the approach road on the two sides of the bridge from the sea. The 8-m thick reclamation was performed by dumping sand into the sea. Geotechnical tests indicated that the reclaimed material did not meet the design requirements for constructing the bridge's approach roads and the foundations of the mechanically stabilized earth (MSE) walls. Dynamic Compaction was carried out to improve the ground conditions of the reclamation. Upon completion of soil improvement, pressuremeter tests were performed to verify the results. The test results demonstrated that the design and acceptance criteria were achieved.

Key words*: Dynamic Compaction; Pressuremeter; Soil improvement.*

Introduction

Reem Island, previously called Abu Shaoum, is a small island that is located about 0.4 km north of Abu Dhabi, United Arab Emirates. The island was basically vacant until 2005 when it was decided to turn it into a modern and luxurious suburb as part of the general development plan of the nation's capital city.

One of the first requirements of the new development was the construction of a causeway to link the island to the rest of the city. According to the design, the causeway was to be composed of approach roads on the two sides and a bridge structure in the center. The approaches on each side were to be approximately 150 m in length. As can be seen in Fig. 1, the reclamation was anticipated to be about 135 m long on Abu Dhabi's side and 50 m long on Reem Island's side.

The approach road was to be constructed on the coastal grounds and on reclamation. The road level was anticipated to be from +2.0 m RL (reduced level = mean sea level, MSL) to +7.00 m RL at bridge level. The maximum elevation difference between the low and high points of the approach road was 5 m, and the road slope was 3.25%.

The approach road and bridge were designed to have four lanes in each direction. The width of the approach road leading to the bridge was 28 m. An additional lane was envisaged on each side for drivers wishing to turn back without entering the bridge. In order to limit the total width of the road to 38 m, the stability of the two sides of the bridge's access road was to be provided by an MSE wall.

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Ground Conditions and Fill Description

The longitudinal profile of the project (Abu Dhabi side) is shown in Fig. 2. The natural ground levels (NGL) in Abu Dhabi and Reem Island were respectively at about -0.5 m and +1.0 m RL but rapidly dropped to about -7.00 m RL and -5.50 m on the sides of the bridge. Groundwater level in the boreholes varied from +0.7 to -0.7 m RL.

Although NGL in the marine boreholes differed, as summarized in Table 1, the in-situ ground profile was generally the same within the project's area. The upper 0.8 to 1.5 m of soil was soft sandy, silty clay. This layer was followed by a very loose to very dense sandy layer with a variable thickness of nil to 2 m and with less than 20% fines. This latter layer overlaid bedrock. The bottom elevation of the loose sandy layer was from about -6.0 to -8.0 m RL.

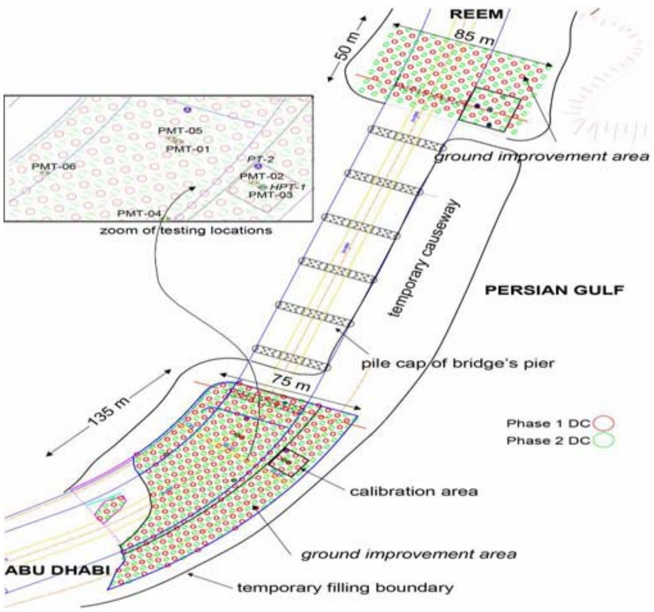

Fig. 1. Site Plan.

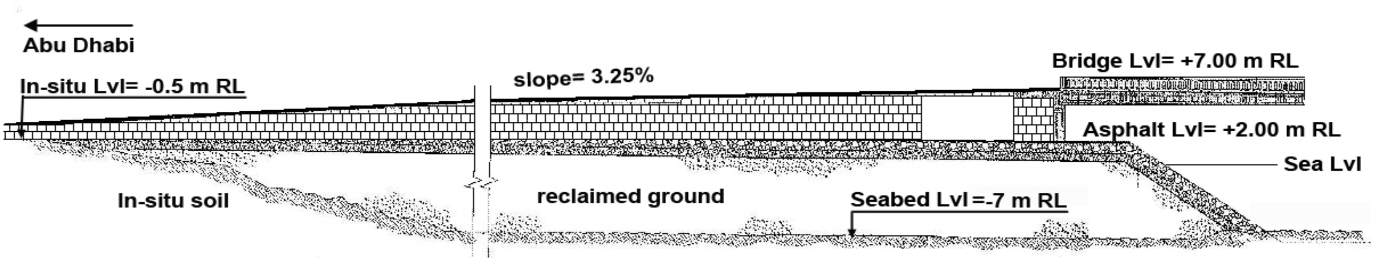

Fig. 2. Longitudinal Profile of the Approach Road.

Table 1. Ground Profile of Reclamation Area Before Ground Improvement.

Description	Thickness (m)	N_{SPT}	fines content	PMT P_l (kPa)	Comment
Subgrade (Reclaimed by Dumping)	Up to 9	-	< 20%	250 to 400	PMT Values after Reclamation and before Ground Improvement
Marine Mud (Sandy Silty Clay)	0.8 to 1.5	0-2	50 to 80%	-	Removed before Ground Improvement
Loose in-situ Sand	0 to 2	4 to 30	< 20%	500 to 700	
Bedrock	-	-	-	-	Encountered at Elevation -6 to -8 m RL

The Geotechnical Concern

Although the marine mud thickness was at most 1.5 m, it was understood that the consolidation of this layer during the life time of the project could cause excessive settlements. Since the soil did not contain any contaminants, was also not potentially acidic, and did not require any treatment, it was deemed that the most appropriate method for dealing with this problematic layer was to simply remove it by dredging the seabed prior to filling and reclamation.

Likewise, due to the poor ground conditions and the marine environment in which the bridge piers had to be constructed in, the foundations of the piers were designed as drilled piles.

Reclamation was anticipated to be done by dump filling sandy material into the sea, and soil layers above the sea level were to be compacted using vibratory rollers.

The engineers of the project who were also involved in the construction of the much larger but somewhat similar Abu Dhabi Corniche [1] were well aware that dumping sandy material into the sea would result in a very loose to medium dense fill. Although it was expected that the granular materials that comprised the fill would consolidate in a relatively short period under the embankment loads, it was also recognized that the submerged subgrade could pose a number of problems such as insufficient bearing capacity, excessive differential settlements of the MSE walls, and excessive total and differential settlements under vibratory traffic loads. These problems could be most evident in the form of unpleasant bumps at the interface of the approach road and the bridge abutment [2].

Based on the previous concerns, ground improvement of the submerged fill was envisaged to be carried out in the form of a design and construct (D & C) contract.

Treatment of the Submerged Subgrade Fill

Among the D & C ground improvement offers, the contract for soil treatment was awarded to a specialist contractor who had proposed the application of optimized design criteria, implementation of Dynamic Compaction [3-4] for the treatment of the submerged dumped fill, and an objective testing method based on Pressuremeter Testing (PMT).

Design Criteria

As part of an optimized design procedure, three criteria that directly addressed the geotechnical concerns of the project were stipulated:

1. Safe bearing capacity under the approach road: 120 kPa with a safety factor of 3.
2. Total settlement of the fill in the approach road with a uniform loading of 20 kPa: 30 mm
3. Differential settlement of the fill in the approach road with a uniform loading of 20 kPa: 1:500

It can be noted that each of these are specifically targeting one of the geotechnical concerns that was mentioned. The maximum required bearing capacity of the fill would be at the location where the approach road reaches the bridge elevation at +7.00 m RL. That will be realized by constructing 5 m of embankment in between the MSE walls. Quite conservatively assuming that the unit weight of the engineered fill is 20 kN/m^3 and adding an additional 20 kN/m^2 for traffic loads, the required bearing capacity will be 120 kPa. At the same time, the pavement of the road was designed to be able to sustain a maximum total settlement with a condition that differential settlements did not exceed 1:500.

It has come to the attention of the authors that unfortunately in some projects sufficient attention is not designated to the project's criteria, and what is stipulated as a specification is not what is really required. Unfortunately, in some projects, the specifications can even be irrelevant. For example, in this project, it could have been naively possible to have systematically specified that settlements were to be limited to the desired figures under the same load intensity as the allowable bearing capacity, i.e., for 120 kPa of uniform loading. However, these criteria would not have gained the project anything but extra costs and time delays. While a bearing capacity of 120 kPa is a genuine requirement that ensures the safe behavior of the elevated sections of the approach road, the

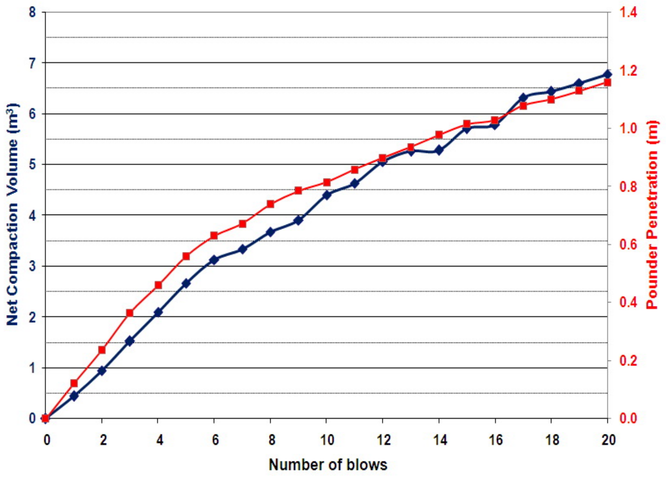

Fig. 3. HPT-2, Pounder Penetration and Compaction Volume During Phase 1 of Dynamic Compaction.

embankment static settlements due to raising the approach road to the final project levels would have happened during construction of the engineered fill and would not be practically measurable to be criteria. It is quite unreasonable and meaningless to assume that after completion the road will be subject to a uniform load of 120 kPa, approximately equivalent to a 12-m high oil tank or 10-story building spread out over the project's area. On the other hand, total and differential settlements that are caused by extreme traffic loads (of 20 kPa) may affect the functionality and drivability of the road and are a realistic concern that should be dealt with.

Hence, it is stressed that stipulating specifications and project criteria based on requirements is a superior and an optimized approach that allow works to be carried out in the shortest possible duration and with the most affordable cost while satisfying engineers' and the project's requirements without the introduction of any technical drawbacks or sacrifices.

The Description of Dynamic Compaction

Dynamic Compaction was chosen as the ground improvement method as it was the experience of the construction team that in equal conditions, it is more affordable and faster to execute than alternative ground improvement techniques. The technique is also efficiently applicable to a wide range of soils, from silty sands and collapsible soils to large diameter boulders [5-6]. Research also suggests that this technology is relatively environmentally friendly and produces less carbon emissions than alternative technologies [7].

The basic principle of Dynamic Compaction is the transmission of high energy impacts to loose soils that initially have low bearing capacity and high compressibility. The impact energy is delivered by dropping a heavy weight from a significant height [5].

The depth of improvement is a function of the pounder weight and drop height. Menard and Broise [8] developed an empirical equation in which the depth of improvement, D, was estimated to be equal to the square root of the impact energy, i.e., the product of the pounder weight, *W*, in tons by the drop height, *H*, in meters. Later [9] and based on further site experiences, it was proposed to introduce a coefficient, *c*, to the original equation:

$$D = c\sqrt{WH} \tag{1}$$

c is usually taken to be from as low as 0.5 to as high as 0.9.

Typically and depending on the required depth of improvement, a pounder weighing 10 to 15 tons is dropped by heavy duty cranes from 15 to 20 m. Heavier pounders can be dropped either by using special cranes or by implementing specially designed equipment such as the Menard Giga Machine that holds the world record for lifting the 170-ton pounder during the Dynamic Compaction works of the Nice International Airport [10].

The pounding grid pattern may be pre-defined or optimized during a Dynamic Compaction calibration and later modified due to actual ground conditions and variations.

Execution of Dynamic Compaction in phases with specific grid spacings is more efficient than dropping the pounder contiguously. The initial phase of treatment is carried out in a wide grid with the maximum amount of impact energy or drops per impact point (print). The objective of this phase is to treat the deepest soil layers. The second phase, which intends to treat the intermediate soil layer, may be carried out with less energy or drops. If necessary, the final phase, which is called ironing, will comprise of closely spaced prints with one or two low energy blows per print for improving the upper soil layer.

The pounder impacts will create craters that indicate the amount of soil compaction and reduction in the soil's void ratio. The craters are backfilled before another compaction phase is performed.

Application of Dynamic Compaction

Due to the requirements of the main contractor, ground improvement works were carried out in two separate phases for each end of the bridge. Each phase was executed within two weeks.

Based on the recommendations of the ground improvement specialist contractor, the fines content of the fill material that was used for reclamation was limited to 20% to remain compatible with the in-situ sand and well within the range of Dynamic Compaction treatment. The material was dumped into the sea to an average elevation of +1.35 m RL.

In addition to the road's 38 m width, an extra 5 m was also initially reclaimed on each side of the approach road to ensure that the slopes of the embankment would also receive sufficient compaction. The extra width was later further increased due to requirements of the project that were not related to ground improvement works. After ground improvement works and at later stages of the project, the extra width was removed and replaced with geotextiles and rock armor to protect the reclamation against wave action and erosion.

A 15-ton steel pounder with an area of 2.0 x 2.0 m^2 and a drop height of 20 m was used for Dynamic Compaction. Other Dynamic Compaction parameters, i.e., grid size, number of impacts per print location, and number of phases were optimized and finalized during a calibration program that was carried out at the beginning of the works.

As part of this calibration, two PMT (before and after improvement) and two heave and penetration tests (HPT) were carried out during phase 1 of Dynamic Compaction (see Fig. 1). In HPT, the ground settlement (crater) under the pounder and the

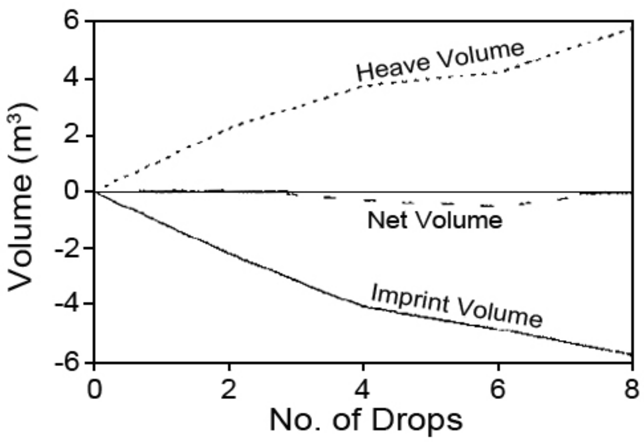

Fig. 4. Compaction and Heave Volumes [11].

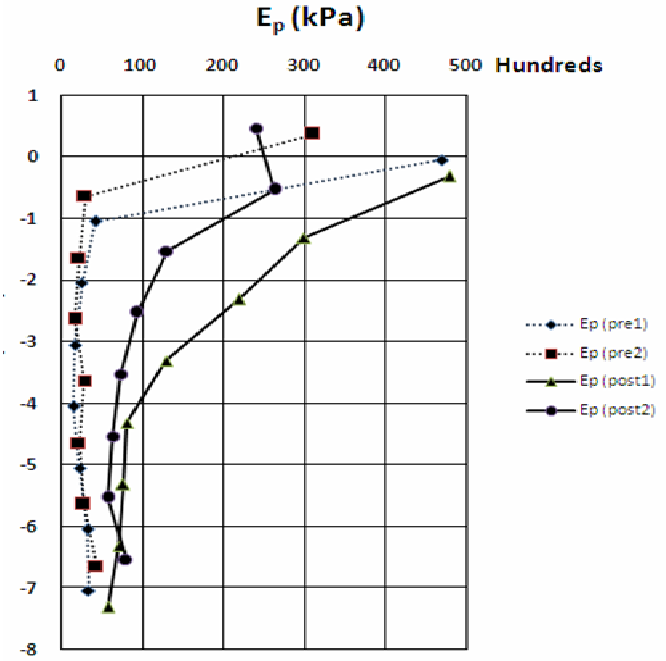

Fig. 5. Menard Modulus Before and After Dynamic Compaction.

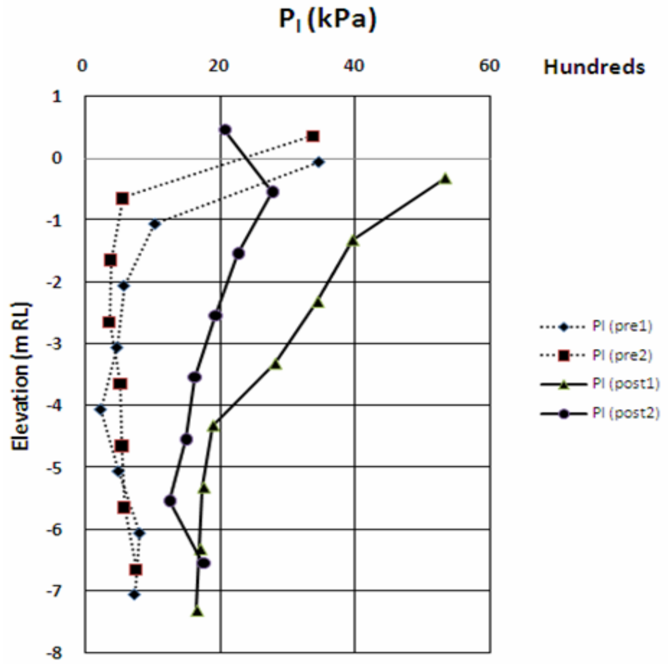

Fig. 6. Pressuremeter Limit Pressure Before and After Dynamic Compaction.

ground subsidence or heave around the crater are measured for the blows. The net compaction volume per blow and possibly the optimized number of blows per print for the combination of a specific pounder weight and drop height can be determined from HPT.

HPT-1 was carried out with 12 blows. HPT-2 was carried out with 20 blows to provide a better understanding of the ground's behavior. Pounder penetration and net compaction volume per blow of HPT-2 are shown in Fig. 3. As can be seen in this figure, the shape of both diagrams is almost identical. The similarity of the shape of the two diagrams is due to the fact that pore water pressure had sufficiently dissipated during the short duration between pounder blows to an amount that allowed an effective and efficient consolidation of the soil. This is not always the case, and there are iso-volumetric circumstances created by the inability of pore pressure to dissipate in time. At one stage, the built up of pressure will result in a liquefied condition in which the pounder impact merely pushes the soil aside with minimal consolidation and the soil undergoes plastic deformations. In such a case, the pounder impact will deepen the crater with almost no volumetric changes as the soil around the impact point will heave. Fig. 4 shows a case encountered during the ground improvement works for a saturated very silty sand/sandy silt section of Dubai Airport Runway where the heave and penetration volumes were almost the same [11].

It can be observed that initially the amount of ground (crater) settlement and compaction volume per blow follows a higher rate, but after about 6 blows, the rate decreases, and it can be extrapolated and expected that asymptotes will be reached at about 35 to 40 blows.

It is generally neither necessary nor justifiable to apply the number of blows for reaching the asymptote. It is more preferable to be able to implement the feasible heaviest pounder and highest drop height to make use of the higher rates of improvement with lesser blows. Further blows realize less achievement with the same impact energy.

It should be noted that optimization of Dynamic Compaction is based on both the HPT results and the verification testing that follows. It is possible to optimize the number of blows by reviewing the settlements and volume changes of HPT, but the amount of settlement, even if considerable, does not necessarily imply that design criteria have been satisfied. Design requirements can be confirmed only by proper testing and measuring the parameters that are able to demonstrate that specifications have been satisfied. In this project, PMT was used for this purpose.

As the (maximum) number of blows during the calibration may be more than what will be implemented in the project, comprehension and proper application of the test results does require experience and the ability to relate the volumetric changes of the soil to the trend of changes of the tests. Varaksin *et al.* [12] discussed a method for relating changes in strain to changes in PMT limit pressures. In this method it is considered that for every 3% of strain, the limit pressure will double.

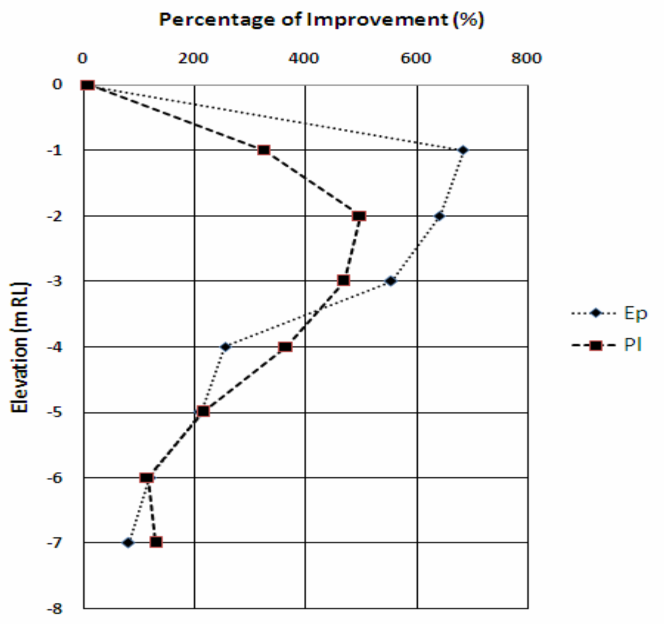

Fig. 7. Comparison of E_p and P_l Values Before and After Dynamic Compaction.

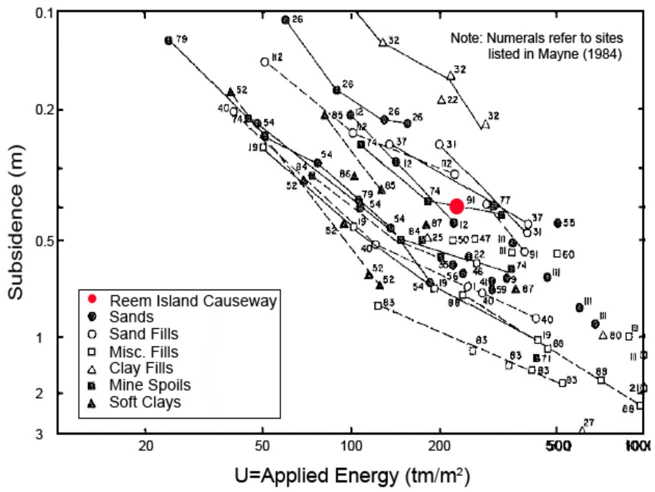

Fig. 8. Comparison of Ground Subsidence Versus Energy with Previous Research [9].

Based on the HPT and PMT results, during the ground improvement works, a 15-ton pounder was dropped from a maximum height of 20 m and with a total energy of 2,364 kNm/m^2 (240 tm/m^2) in three phases.

Verification Testing and Findings

Comparisons of the pressuremeter moduli, E_p, and pressuremeter limit pressures, P_l, before and after improvement are shown respectively in Fig. 5 and Fig. 6. It can be well observed that while additional weight of the engineered fill required for reaching final road level.

Post treatment testing demonstrates that the soil parameters have significantly improved throughout the fill and the subgrade layer has become very dense.

Lukas [13] has reported that the upper bound for P_l after Dynamic Compaction in sands and gravels will be from 1.9 to 2.4 MPa. It can be observed that the maximum P_l value achieved in this project has exceeded that expectation in one of the test location. The P_l values are also higher than what Mayne and Jones [9] have reported for a number of sites. However, previous experiences of the authors are in line with the results of the PMT carried out in this project and indicate that it is possible to achieve larger values than the mentioned range. Case studies by other authors, e.g., Spaulding and Zanier [14] also indicate that higher P_l values of up to 4 MPa can be achieved after Dynamic Compaction.

The comparison of E_p and P_l before and after Dynamic Compaction is shown in Fig. 7. It can be observed that in the upper several meters of soil the pressuremeter parameters have increased by 500 to 700%, which is quite significant and more than what is expected (400%) as the common range of improvement by Lukas [6]. It is interesting to note that improvement at depth and at the bottom of the subgrade fill the improvement is still substantial and in the range of 80 to 130%. This massive improvement may have been due to the fact that the very young fill was placed only a short period before ground improvement works and in a very loose state. Varaksin *et al*. [1] have reported improvements of similar magnitude due to Dynamic Compaction works of the hydraulic fill of Abu Dhabi Corniche.

The ultimate bearing capacity of a foundation can be calculated construction equipment traffic had improved the parameters of the upper meter of soil crust, deeper layers were initially very loose and with P_l values less than 600 kPa, even subject to creep or settlement under the self weight of the dumped soil in the remainder of the reclamation thickness [12, 15]. Consequently, it could have been expected that the ground would have settled for several years after construction even without the effects of traffic loads and the using Menard's equation [15] for bearing capacity.

$$q - q_0 = k(P_l - P_0) \tag{2}$$

Where,

q = ultimate bearing capacity;

q_o = overburden pressure at the periphery of the foundation level after construction;

k = bearing factor varying from 0.8 to 9 according to the embedment, the shape of the foundation and the nature of the soil; and

p_o = at rest horizontal earth pressure at the test level (at the time of the test).

When the foundation rests on a layer with variable P_l values with depth, the equivalent limit pressure is defined as the geometric mean of the values:

$$P_l = \sqrt[3]{P_{l1} P_{l2} P_{l3}} \tag{3}$$

Where,

P_{l1} = geometric mean of the values measured in the section from +3R to +R above foundation level,

P_{l2} = geometric mean of the values measured in the section from +R to -R above foundation level,

P_{l3} = geometric mean of the values measured in the section from -3R to -3R above foundation level, and

Fig. 9. Dynamic Compaction Craters.

$2R$ = the width of the foundation.

The bearing capacity can be conservatively calculated by assuming a value of 0.8 for bearing factor (k) and neglecting q_o and P_o in Eq. (2). It can be calculated that the geometric mean of the two post treatment P_l values are respectively 2640 kPa and 1890 kPa, thus with a safety factor of 3 the allowable bearing capacity will be respectively 704 kPa and 505 kPa, which by far exceeds the design criterion.

Similarly, the harmonic mean of the PMT Modulus for the two post treatment tests can be calculated to be respectively 10.7 MPa and 9.3 MPa. Young Modulus can be calculated from the PMT modulus with the rheological factor [15] which is 1/3 for sands. Thus the harmonic Young Modulus for two test locations can be calculated to be respectively 32.1 MPa and 27.9 MPa.

Once again, the settlement can be conservatively calculated with the assumption that the 20 kPa uniform load's stress reduction in the fill is negligible. It can be readily calculated with Hooke's Law that the settlement will be respectively 5 mm and 6 mm.

The above figures show that while the settlements are in reality much less than the design criterion, had the settlement requirements been inappropriately tied to the required bearing capacity (which is 6 times more than the actual design load), then the settlements calculated from the second post treatment test would have been more than acceptance. This comparison well demonstrates why the authors have continuously stressed on the importance of developing proper design criteria.

As a result of the ground improvement works by Dynamic Compaction, the fill settled on average about 40 cm. Should this amount of Dynamic Compaction-induced ground subsidence be plotted against the applied energy of 240 tm/m^2 and compared with previous research [9], it can be seen (in Fig. 8) that compared to other sand fills, the amount of subsidence in this project falls approximately in the middle of other sites for the same amount of energy. Thus, it can be anticipated that the P_l values of this site are not extraordinary either, and indeed it may be possible to have higher P_l values than what Lukas conceives.

Also, as shown in Fig. 9, crater depths were about 80 cm.

Lessons to be Considered

The review of this project can provide the pavement engineer with a number of lessons to be incorporated in future projects, including:

1. Experience of previous projects suggests that it would be realistic to assume that reclaimed fills will be in a loose state and will require some kind of ground treatment. Although the soil above groundwater level may be dense due to construction equipment traffic or other reasons, it is highly likely that there will be loose layers of soil below groundwater level.
2. There are numerous methods for improving subgrades; it is the responsibility of the engineer to identify the best feasible solutions and to implement the one or ones that can provide the most benefits to the project. In this project, the thin layer of clayey soil was simply removed by dredging. However, that method was not applicable to the fill, and Dynamic Compaction was used to improve a relatively thick submerged subgrade with fines content up to 20%.
3. Proper determination of design criteria is very important, and failure to adopt a suitable specification can lead to unnecessary treatment, additional costs, and delay. Indeed, improper design criteria can result in the deferral of a project, and can swing an economical treatment method into an uneconomical and even unfeasible solution. Design and acceptance criteria must be developed on a project by project basis with the intention of addressing specific risks and requirements.
4. It is possible to optimize the ground improvement parameters by performing an objective calibration, which includes the HPT and PMT tests. The appropriate number of blows per print should be chosen based on the net compaction volume and the number of blows that will satisfy acceptance criteria. Improper pounding of the ground with excessive soil heaving will not provide any benefits and in such cases compaction energy will be simply wasted without productivity.
5. The most preferable means for verification testing is to implement a method that is capable of measuring the design criteria as directly as possible. Testing methods that are based on correlation and indirect measurements increase the risk of insufficient or excessive treatment without providing any benefits to the project.
6. It is possible to achieve P_l values greater than 2.4 MPa after Dynamic Compaction in saturated sands; however, the peak value decreases with depth.
7. Noting that the amount of improvement is a function of impact energy, it is possible to improve the P_l value of saturated sands by 500%. However the peak amount of improvement decreases with depth. The maximum amount of improvement appears to be in the upper half of the depth of improvement.

Conclusion

The experience and the ability of the project's team to foresee that the dumped fill will be in a loose state and to plan remedial measures in advance enabled the project to be planned realistically and to be executed without compromising the work's program by the introduction of geotechnical surprises.

The proper stipulation of specifications benefited the project by allowing the engineers to introduce an affordable and fast to execute ground improvement method. The application of Dynamic Compaction for the treatment of a submerged subgrade has proven to be very successful, and pressuremeter tests were able to demonstrate that the design criteria were readily satisfied. Improvement of pressuremeter parameters in the upper several meters of soil was 500 to 700%. The parameters' improvements were still substantial and in the range of 80 to 130% at depth.

Abu Dhabi – Reem Island Causeway is completed and currently open to traffic. As anticipated and expected, due to the ground improvement works, the dumped fill subgrade is performing as per the project requirements, and no settlements or indications of cracking due to poor foundation or bumping has been reported.

Acknowledgement

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