APPLICATION OF DYNAMIC COMPACTION IN RECLAIMED ROADS

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ABSTRACT

Pavement layers are systematically constructed as engineered fills with specified properties and criteria; however these well built layers may be underlain by loose saturated subgrades that, if not treated, may be subject to undesirable and damaging deformations. This may be especially true for roads that are constructed on reclaimed land.

Dynamic Compaction is a ground improvement technique that can and has been effectively utilised for treating thick loose layers of saturated *in situ* or reclaimed granular soils. In this paper, the application of Dynamic Compaction for improving loose sub-grades will be discussed using three case studies. The case studies have been specifically selected in a manner to demonstrate the applicability of this technique to hydraulic fills and truck dumped fills, to very large projects such as the 900,000 m^2 Abu Dhabi Corniche, to moderately large projects such as Marjan Island Main Road corridor and to relatively small sized projects such as the 10,000 m^2 approach roads of Reem Island Causeway. The projects can be in undeveloped locations or in urban areas.

1 RECLAIMED SUBGRADES

Inevitably, every reclamation project, regardless of its function and purpose, will have roads. The roads may have single, double, triple or more carriage ways in each direction, and will be designed to provide access to the required points in the reclaimed land.

Based on the road function and traffic expectation road pavements are designed according to engineering standards and base and sub-base layers are required to meet pre-determined minimum densities. However, the reclaimed land will not necessarily provide a subgrade that satisfies the road specifications without the implementation of certain measures.

Reclamation can either be land based and by dump trucks tipping fill into the sea or by hydraulic placement from the sea. Sladen and Hewitt (1989), Lee *et al*. (1999), Lee *et al. (*2000), Lee (2001) and Na *et al.* (2005) have studied the effects of placement methods on the geotechnical behavior of sand fills. The density of sand that is dumped by trucks and then pushed into the sea by a bulldozer is usually low, with relative density of about 20%. Exceptions can be thin layers which have been compacted by the traffic of earthmoving equipment. Hydraulic placement can be subaqueous by hoppers or bottom dump barges. When possible sand is discharged by means of a big door located on the bottom of the hull, but when the water is shallow alternative methods; i.e. pipeline discharge or subaerial rainbow discharge will be used. In pipeline discharge a low velocity water-sand slurry is pumped; however in the rainbow method the dredger sprays a high velocity water-sand mixture onto the reclamation. These processes are schematically shown in Figure 1.

The variation in fill densities achievable by hydraulic placement is large and closely related to the placement method. Hopper placed sand is denser than pipeline placed sand. Sand deposited by hydraulic filling below water level generally has a low to medium relative density of about 20% to 60% due to the loose packing from self-weight sedimentation of sand particles under water. The zone with the least strength could be expected to be just beneath water level if fill is placed by subaqueous discharge through hydraulic pumping. Sand placed above water table by hydraulic filling tends to have a higher relative density in the range of 60% to 80% because of dense packing from downward seepage and reduction in void ratio as a result of sliding and rolling of the sand particles mixture.

Hopper or bottom dumping achieves a higher density than pipeline discharging for a number of reasons. Firstly, the sand mass stored in a hopper has a higher bulk density than the sand slurry that is discharged from a pipe. Also, dumping a large quantity of sand from a hopper in a short period will result in the sand mass falling as a slug rather than as individual particles. Furthermore, the simultaneous opening of all bottom doors prohibits the entrapment of fresh water into the slug that would reduce the fall velocity and expand the slug size. The fall energy of the slug is likely to be dissipated in compaction of berm through impact and shearing. The loosest possible state would likely be achieved if the pipe discharge was placed near the water surface in such a way to allow maximum fresh water entrapment. In such a case the slurry becomes a clod with falling velocity being close to the falling velocity of individual grains. Each particle will basically come to rest in the position that it makes contact with the previously placed fill. Impact may result in some pushing around of the grains, but the impact velocities and forces can be expected to be small. Subaerial rainbow dredging can be expected to yield similar results to pipe discharging.

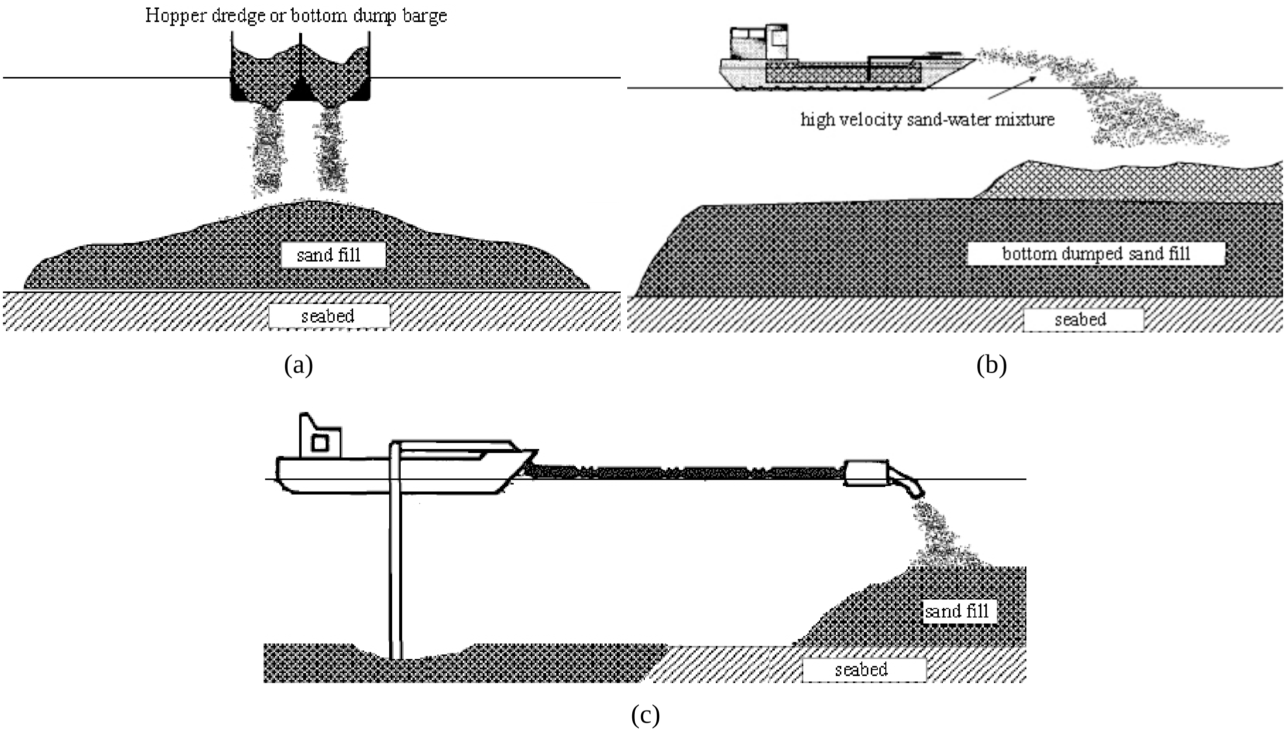


Figure 1: (a) subaqueous discharge by hopper or bottom dump barge (b) subaerial rainbow discharge (c) pipeline discharge

Once the process of reclamation is understood, it will not be difficult to be able to foresee that reclaimed sand fills will most probably be loose and subject to settlement, excessive deformation, and other geotechnical problems. Hence, project developers and designers must be aware that they may require specific geotechnical solutions such as ground improvement in reclaimed projects.

2 DESIGN AND ACCEPTANCE CRITERIA

It is not difficult to anticipate that road engineers who are accustomed to using soil density as acceptance criterion in their design may be tempted to continue the same practice by specifying a minimum dry density for the soil. However, measuring soil density below groundwater level is very difficult, time consuming and expensive. At the same time, density is generally not a foundation design parameter and in reality road engineers only use it because they hardly have any better tool to quantify the quality of a thin engineered fill layer that is approximately 0.3 m thick

Yet, as the thickness of loose soil in reclaimed ground is much more, geotechnical engineers have more meaningful ways of specifying design and acceptance criteria that are more compatible with the design objective. Ground improvement acceptance criteria can be envisaged to be in three main forms based on quality of work, minimum test values or directly on design criteria (Hamidi *et al*., 2011a).

2.1 ACCEPTANCE CRITERIA BASED ON QUALITY OF WORK

Sometimes the responsibility of the contractor is limited to providing the working team, equipment, material and execution of the works according to the specifications and drawings that have been prepared by others. In this type of project, acceptance criteria is generally non-technical and based on performing the works correctly. Testing is generally specified, but the contractor has no responsibility to meet predefined results as long as the works are performed correctly. If results are not satisfactory the contractor may be required to perform additional work for which he will be fully paid. This type of specifications can be useful in small projects and in the absence of specialist ground improvement and geotechnical contractors. However, its use in large size projects and in the presence of specialist contractors is questionable.

2.2 ACCEPTANCE CRITERIA BASED ON MINIMUM TEST VALUES

Sometimes acceptance criteria are based on minimum test values or correlations to test values. At times, the specified testing method is practically infeasible. For example, it may be specified to carry out *in situ* density tests for a thick reclaimed fill. As discussed this will be very difficult, expensive and time consuming without actually measuring the foundation design parameters. Sometimes specifications stipulate an impractical testing method, but attempt to resolve the problem by correlating the specified test to a practical testing method. For example the specifications may stipulate a percentage of *in situ* density and then correlate density to CPT cone resistance. However, this is quite meaningless as CPT results can be interpreted on their own and without the need of introducing density into the testing methodology.

Although sometimes erroneously specified in projects, implementation of relative density is also a poor choice and there is overwhelming evidence that this parameter must not be used as acceptance criteria of ground improvement projects. Hamidi *et al*. (2011d, accepted, in review) have reviewed the problems associated with the concept of relative density and relative density correlations. In extreme cases, application of relative density can be as accurate as making a wild guess.

The above discussions have been recognised by many geotechnical engineers, and thus a trend has been realised whereas minimum values for practical, efficient and economical field tests such as SPT, CPT or PMT (Menard Pressuremter Test) have been stipulated as acceptance criteria. This, itself, is a positive step forward as it recognises that establishing acceptance criteria based on direct measurement of parameters is more rational and beneficial than making purposeless correlations; however it is not enough. What lies behind these types of acceptance criteria are calculations that have been carried out by geotechnical engineers to ensure certain design requirements such as bearing capacity, total and differential settlements, liquefaction mitigation or long term consolidation have been satisfied. However, the condition in which all test values of the soil layers just reach the minimum value is only one of countless possibilities that may satisfy the design criteria, and statistically speaking, the least probable of them all. Furthermore, in techniques with inclusions such as dynamic replacement, stone columns, jet grouting, deep soil mixing and controlled modulus columns, where loads are distributed between the in-situ soil and the inclusions by arching (Hamidi *et al*., 2009b) the minimum value concept becomes blurred as the in-situ soil parameters improve negligibly compared to the much higher inclusion parameters.

2.3 ACCEPTANCE CRITERIA BASED ON DESIGN CRITERIA

There is no better way of making sure that a certain aspect of design has been fulfilled than directly verifying that specific criterion itself; hence it would be very rational to assume that optimised results can be achieved when acceptance is based on design criteria.

Acceptance criteria for Nakilat Ship Repair Yard that was undertaken as part of Port of Ras Laffan expansion in Qatar was initially based on a relative density correlation with a calcarenite correction factor. Alternative specifications based on design criteria were later proposed by a ground improvement specialist contractor who was awarded the project. Calculations were able to demonstrate that test results that would not have satisfied the relative density requirements in total were able to provide more bearing capacity and lesser settlements than the relative density specification (Hamidi *et al.*, 2010b).

Similarly, acceptance criteria for Madina A’Zarqa (Blue City) in Oman was initially based on minimum CPT values, but was later modified to include PMT and analyses of bearing capacity and settlements. In this project, it was also observed that it was possible to meet design criteria without strictly complying to the specified minimum test values (Hamidi ., 2012c).

3 DYNAMIC COMPACTION CASE HISTORIES IN RECLAIMED ROADS

Dynamic compaction is a ground improvement technique in which the mechanical properties of the soil are improved 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, decrease the voids and increase intergranular contact that will directly lead to improved soil properties. The 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 (Hamidi *et al*., 2009a).

Figure 2: Pipeline hydraulic filling in Abu Dhabi Corniche

3.1 ABU DHABI NEW CORNICHE

Abu Dhabi New Corniche (Beach Road) is a 6 km long reclamation with an area of 900,000 m^2 that has been hydraulically reclaimed from the Persian Gulf (Hamidi *et al*., 2010a, 2012b). The reclamation began at the face of the original beach road and on average extended 160 m into the sea. The maximum width of the reclamation was 300 m and the maximum reclamation thickness of about 12 m at the sea facing is retained by sheet piles. In addition to the leisure areas, pedestrian pathways and bicycle lanes on each side of the road, the road itself is composed of 4 lanes in each direction that are separated in the centre by a variable median.

The project was reclaimed by placing hydraulic fill using pipes (see Figure 2). The geotechnical investigation that was performed after reclamation reported that the seabed was composed of medium dense fine-grained sand followed by a dense layer of sand, shells and ultimately the limestone bedrock.

Project specifications stipulated that reclaimed material had to contain less than 10% fines. Based on this expectation, acceptance criteria required that relative density be at least 80% with an SPT blow count, *N*, correlation. However when the ground was reclaimed, testing indicated that the fill was loose to very loose with *N* in the range of 1 to 10; hence ground improvement was stipulated and the works were awarded to a specialist contractor who had proposed dynamic compaction.

Further testing during the works revealed that the hydraulic fill had segregated and a soft silty layer at least 0.5 m thick covered the seabed.

This project is an example of one of many problems associated with relative density. Noting that relative density is applicable to soils with less than 15% fines (ASTM, 2006a, 2006b) the project's specifications suddenly became void. The authors have observed that sometimes an equivalent relative density notion is developed, but that would be even more meaningless than relative density itself. It has to be stressed and realised that the concept of relative density is faulty (Hamidi *et al.*, accepted) and there are so many buts and ifs in the correlations (Hamidi **et al**., in review) that leave no room for salvaging relative density and putting it into use.

(a) (b)

Figure 3: Dynamic compaction in Abu Dhabi Corniche with 25 ton pounders

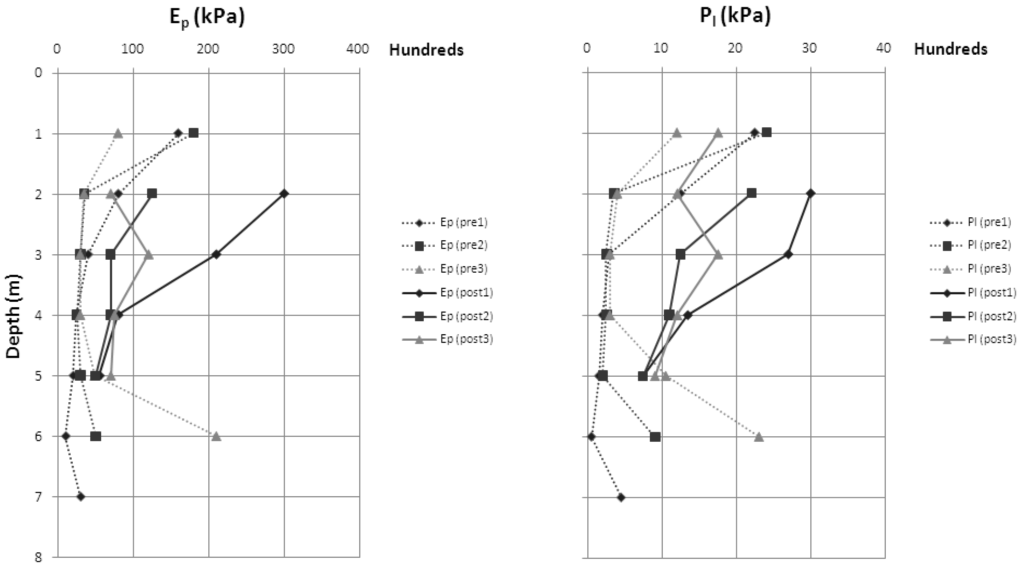
Figure 4: Dynamic compaction works in the vicinity of residential and office buildings

Thus, in addition to the non-functional criteria of relative density and the SPT correlation, PMTs were carried out for the design of sheet piles and further verification of the work.

The works were carried out with up to 7 rigs working two shifts per day. Peak ground improvement production rate was 200,000 m^2 per month.

The energy per unit area of treatment, pounder weight, drop height and number of phases were varied based on the treatment thicknesses. Pounder weights were from 12.5 to 25 tons and maximum drop height was 20 m. In areas with less than 6 m thickness, two phases of deep treatment using 12.5 and 16 ton pounders were utilised. In deeper areas three phases of deep compaction using 25 ton pounders were carried out. Sometimes due to the build up of pore pressure, the phases were divided into sub phases.

Figures 3(a) and 3(b) show a 25 ton pounder in the thicker reclaimed areas. Also noticeable in Figure 3(a) is that the works are being performed before reclamation was completed. As shown in Figure 3(b), dynamic compaction was carried out to the reclamation edge.



(a) (b)

Figure 5: Menard Modulus and limit pressure before and after Dynamic Compaction in New Corniche

As can be seen in Figure 4, dynamic compaction was performed in close proximity to hundreds of buildings. Although waves generated by this technique can damage structures if the structures are too close to the works, it is possible to estimate vibration parameters (Hamidi *et al*., 2011c), and, if necessary, to implement measures, such as installing isolation barriers (Hamidi *et al*., 2012), to reduce vibration. Human comfort zone and tolerance (Reiher and Meister, 1931 cited in Wiss and Parmelee, 1974) is much more sensitive than structures (Siskind *et al*., 1980); hence night shifts were scheduled in a manner in which works would be carried out at in the furthest areas from the residential buildings.

PMT parameters E_p, Menard Modulus, and P_l, limit pressure, before and after dynamic compaction without performing ironing (light pounding phase) are shown in Figure 5. It can be observed that before ground improvement the upper crust of the reclamation was quite dense due to construction equipment traffic, but the soil rapidly became loose and even subject to creep (Menard, 1975). However, after soil improvement P_l increased at its most from about 600 to 1000% at the depth of approximately 3 m. It should be noted that in this project ironing was not necessary for meeting the specifications and thus not carried out.

3.2 MARJAN ISLAND MAIN ROAD CORRIDOR

Marjan Island, translating to Coral Island, is the first manmade group of islands of its kind that has ever been reclaimed from the Persian Gulf in the emirate of Ras Al Khaimah, UAE (Hamidi *et al.*, 2012a). This project is located approximately 27 km southwest of the city of Ras Al Khaimah and 54 km northwest of the city of Dubai. The group of islands are composed of a peninsula followed by four coral shaped islands that are connected together via bridges.

Unlike most manmade islands in the UAE where land is reclaimed from the sea by hydraulic filling, trucks were used to cart and dump sand more than 3 km into the Persian Gulf, on average to elevation +4 m ACD (Admiralty Chart Datum). Average groundwater was at +2 m ACD.

The preliminary geotechnical investigation on the main road corridor passing through the Peninsula, Island 1 and Island 2 consisted of 37 SPT boreholes drilled down to a maximum depth of 16 m. These tests indicated the presence of a fill with heterogeneous strength. On average, in the top 7 m the fill was composed of very loose to medium dense sand, occasionally interbedded with boulders at different depths. Fines content was variable from 13 to 30% and SPT blow counts (*N*) were generally low; sometimes as low as 4 and rarely more than 50. The second layer of soil, extending down to -12 m ACD, was composed of medium dense to very dense silty sand with occasional interbedded pockets of sandy silt. Fines content was generally from 5 to 30% and *N* was from 10 to more than 50.

At later phases, 16 PMT were also later carried out as part of a supplementary geotechnical investigation. P_l in these tests also indicated that the ground was sometimes very loose in the upper 7 m, with the lowest P_l being 70 kPa. Although due to truck traffic the upper metre or two of the ground was generally very dense and P_l of more than 1000 kPa was commonly observed, P_l of the deeper layers was commonly less than 600 kPa. Similarly while E_P was generally 4 to 8 MPa at depths of 2 to 7 m, values of less than 1 MPa were also occasionally encountered.

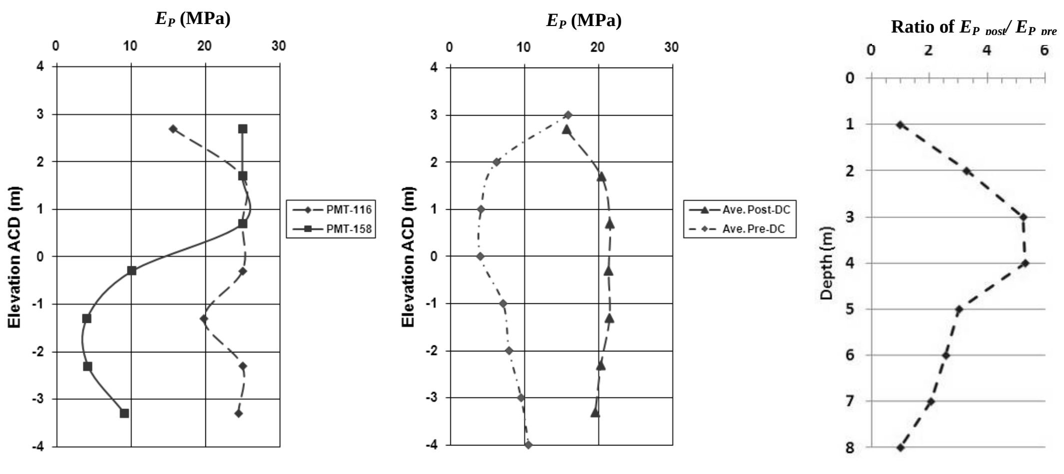
The heterogeneity in soil strength and the presence of loose spots in the proximity of dense spots indicated that it was possible for the ground to undergo large differential settlements due to the self weight of the soil without external loading. Structural and traffic loads further increased the risks of undesirable excessive ground deformations and insufficient bearing capacity. The geotechnical concerns were considered to be of high priority for the main road corridor. Hence, the project management team approached geotechnical specialist contractors to propose solutions for mitigating the geotechnical risks, and consequently a contract was awarded to a specialist ground improvement contractor who had proposed the application of dynamic compaction for the treatment of the 198,000 m^2 main road corridor.

Ground improvement design criteria was specified to be:

1. Maximum total settlement: 25 mm under a uniform load of 20 kPa on the road area
2. Maximum differential settlement: 1:500 between any two points on the road with a distance of 10 m under a uniform load of 20 kPa.

Verification and acceptance of the works was by the PMT and interpretation of test results was specified to be by the Menard (1975) method.

At the beginning of the works a calibration dynamic compaction programme was performed to verify and to optimise the ground improvement parameters (Hamidi *et al.,* 2011e). In this programme a 20 ton pounder was dropped from 20 m. Heave and penetration tests (HPT) were performed to measure the amount of ground compaction per drop and PMT were carried out to verify that acceptance had been achieved.



(a) (b) (c)

Figure 6: (a) Two post dynamic compaction E_P results, (b) Average E_P before and after dynamic compaction, (c) E_P improvement ratio

Dynamic compaction was carried out using 17 and 20 ton pounders using two rigs. Each print location received 5 to 8 blows dropped from 20 m. In the ironing phase either 17 and 20 ton pounders were dropped from 12 m or alternatively a 12 ton pounder was dropped from 17 m.

32 PMTs were also carried out after ground improvement to verify the ground conditions and to confirm that acceptance had been achieved. Of these post soil improvement tests 14 were on the Peninsula, 5 were on Island 1 and 13 were on Island 2.

For comparative purposes post improvement E_P values of two test locations that appeared substantially different are shown in Figure 6(a). Although in reality the distance between these two tests was much more than 10 m, it was of interest to study the total and consequently the differential settlements in between these two locations (with the assumption that they were 10 m apart).

Hamidi *et al*. (2012a) have calculated the settlement of the point with the lesser E_p using the interpretation of Menard (1975) and with the assumption that an area of 100 m by 10.5 m is subjected to a uniformly distributed load of 20 kPa. E_p for all layers deeper than the tested depth was conservatively assumed to be 20 MPa. Settlement was thus calculated to be merely 2.2 mm which is much less than the acceptable differential settlement between two points.

Figure 6(b) shows the average E_p before and after dynamic compaction. It can be seen in Figure 6(c) that maximum improvement in the modulus was achieved at about half the depth of improvement. The average of maximum improvement ratios was 5.31 (431% improvement). Although the authors have observed much higher ratios in some dynamic compaction projects such as Abu Dhabi Corniche, this figure is quite compatible with the indicative upper bound figure of 400% that has been proposed by Lukas (1986) as a guideline.

3.3 REEM ISLAND CAUSEWAY

Reem Island, previously known as Abu Shaoum, is a small island located about 0.4 km north of Abu Dhabi. The island was basically vacant until 2005 when it was decided develop it. One of the first requirements of the new development was the construction of a causeway, a bridge structure in the center and approach roads on its two sides, to link the island to the mainland. The approach road and bridge were designed to have four lanes in each direction. An additional lane was envisaged on each side for drivers turning back without crossing the bridge (Hamidi *et al*., 2011b).

The approaches on each side were to be approximately 150 m long. The reclamation was anticipated to be about 135 m long on Abu Dhabi’s side and 50 m long on the Island’s side. 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 (mechanically stabilized earth) wall. The maximum elevation difference between the lowest and highest points of the approach road was 5 m. A schematic cross section of the project's Abu Dhabi side is shown in Figure 7.

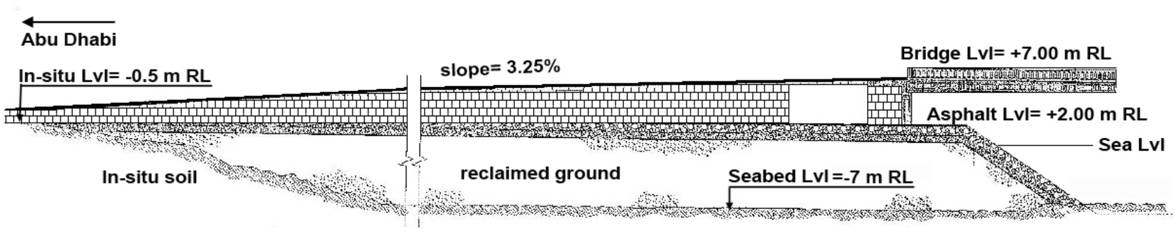


Figure 7: Longitudinal profile of the approach road

Natural ground level in Abu Dhabi and Reem were respectively about -0.5 m and +1.0 m RL, but rapidly dropped to seabed level of about -7 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 seabed level in the marine boreholes differed to an extent, 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.

Although the marine mud thickness was at most 1.5 m, it was understood that the consolidation of this layer during the lifetime of the project could cause excessive settlements. Since the soil did not contain contaminants, was 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.

Reclamation was done by dump trucks tipping sand into the sea. Geotechnical testing indicated that, as expected, the sand was in a loose state with P_l in the range of 250 to 400 kPa; hence a specialist ground improvement specialist was appointed to carry out dynamic compaction with project design requiring allowable bearing to be 120 kPa, total settlement to be limited to 30 mm under a uniform load of 20 kPa and differential settlement not to exceed 1:500 under the same load.

Deep densification was achieved for improving up to 9 m of loose sand by dropping a 15 ton pounder from 20 m.

Due to the project's programme, one dynamic compaction rig was mobilised twice and soil improvement on each side of the bridge was carried out in two separate periods. Each time the works was performed within a period of two weeks.

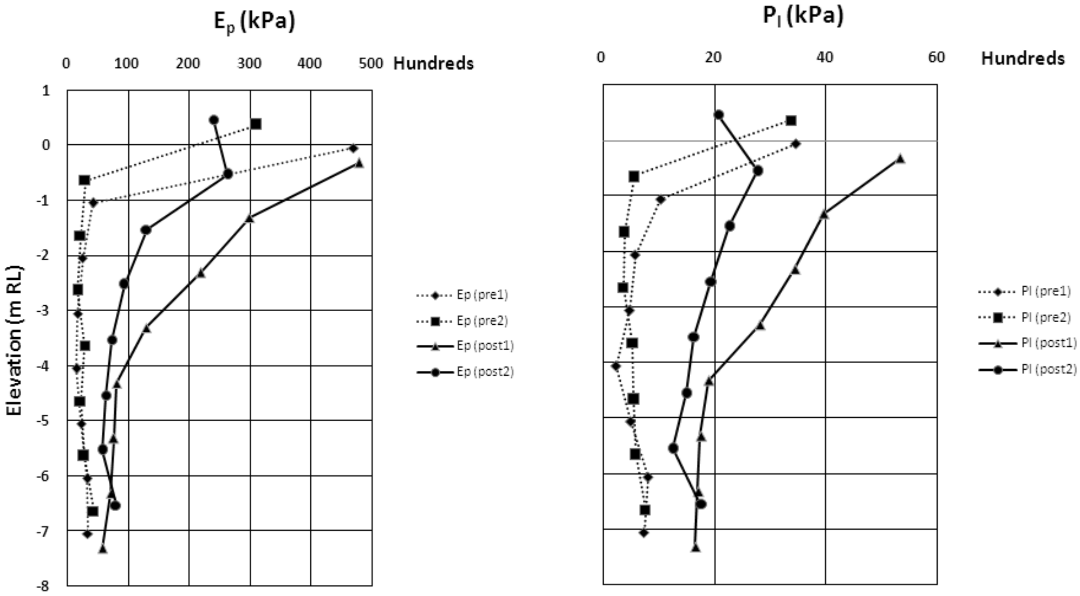


Figure 8: E_p and P_l before and after dynamic compaction in Reem Causeway

Comparisons of E_p, and P_l before and after dynamic compaction are shown in Figure 8. It can be observed that, as expected, while the construction equipment traffic had compacted the sand and improved its geotechnical parameters in the upper meter of ground before dynamic compaction, deeper layers were originally very loose and even subject to creep (Menard, 1975). However, after ground improvement the pressuremeter parameters increased significantly to more than 500% of the initial values. Even at depth, the improvement is still considerable and in the range of 80 to 130%. This massive improvement may have been due to the fact that the very loose young fill was placed only a short period before ground improvement works.

Calculation of the allowed bearing capacity (Menard, 1975) shows that even with conservative calculations the allowed bearing capacity will be in the range of 510 to 750 kPa which is much more than the required value of 120 kPa. Conservative settlement calculations with the assumption that the 20 kPa uniform load's stress reduction in the fill is negligible will yield a settlement of about 5.8 mm which is also substantially below the acceptable value of 30 mm.

4 CONCLUSION

Reclamation of land, either by dump truck tipping or by hydraulic placement of sand, will result in loose saturated ground that can be subject to various geotechnical problems. Consequently, construction of roads on untreated sandfill reclamations can lead to faulty roads with geotechnical risks. However, saturated sandfill subgrades can be improved using dynamic compaction.

Dynamic compaction was successfully applied for the treatment of the subgrades of three projects in the UAE. The size of these projects ranged from about 10,000 m^2 to approximately 900,000 m^2. Reclamation techniques included truck dump tipping and hydraulic filling using pipelines. Pressuremeter tests were used to verify that acceptance criteria had been satisfied.

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