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TREATMENT OF THICK SATURATED LOOSE SUB-GRADES USING DYNAMIC COMPACTION

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Abstract

Road engineers are very keen to properly compact base and sub-base layers to meet the design requirements; however these well built engineered layers are sometimes constructed on loose saturated subgrades. Consequently, the pavement may undergo undesirable deformations and settlements for a number of reasons.
Dynamic Compaction is a ground improvement technique that can and has been effectively utilized for treating the loose layers of in-situ or reclaimed granular soils. In this paper, the application of Dynamic Compaction for improving loose sub-grades will be discussed using two case studies. The case studies have been selected to demonstrate that the technique is equally applicable to hydraulic fills and truck dumped fills, very large projects such as the 900,000 m^2 Abu Dhabi Corniche (Beach Road) to relatively small 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.
The need for special attention in the development of technical specifications that can optimize or jeopardize the ground improvement programme is highlighted, and recommendations are proposed to develop optimized criteria.

Keywords: ground improvement, soil improvement, dynamic compaction, saturated fill, subgrade

1. Introduction

It is general practice in road construction to compact the pavement layers using vibratory rollers to a defined compaction that will satisfy the design. Commonly, the amount of compaction is measured by comparing the dry density of the soil with the maximum Proctor or modified Proctor dry density.

Occasionally, there are roads that are to be constructed on grounds that have been reclaimed from the sea either by hydraulically placed fill or dumping soil using tipper trucks. In any case, these fills are most often very loose to loose and frequently with a dense upper crust above groundwater level.

Figure 1. Failure of a road constructed on loose soil, 2004 Niigata-ken Chuetsu Earthquake [2]

In most cases, one way or the other, roads constructed on thick saturated loose subgrades will experience some kind of problem during the road's life time. Although thick layers of well compacted base and sub-base or the implementation of geo-grids may alleviate differential settlements by redistributing traffic loads, these measures are not effective for mitigating subsidence caused by traffic vibration, liquefaction caused by earthquake, creep or self bearing settlements due to the re-arrangement of soil particles under its own weight, slope stability of the edges or bearing capacity issues of utilities. Ground subsidence can be most evident in the form of unpleasant bumps at the interface of the approach road and the bridge abutment [1]. There are also many well documented cases of road failures due to liquefaction. Figure 1 shows a road that was partially built on a loose back fill. Although the road could have performed well during the 6.8 Richter 23 October 2004 Niigata-ken Chuetsu Earthquake, the section found on the loose backfill has totally failed [2].

It can be readily observed that classical compaction by rollers is infeasible for improving the strength parameters of thick saturated loose subgrades and classical testing by measuring the in-situ soil density is also equally impractical. Vibratory roller compactors are usually used to effectively densify soil layers with a maximum thickness of 0.3 m. Impact rollers are able to improve the mechanical properties of thicker layers of soil and are used for effectively compacting fills with 1 m thicknesses or more. However this thickness is still not sufficient for treating thick saturated loose soil layers. Dewatering the site, excavating the loose soil and replacing it with engineered fill is feasible but very impractical due to the extraordinary amount of time and finances that have to be allocated. Likewise, extracting undisturbed samples for accurate determination of the in-situ dry density is not impossible, but is quite impractical and not worth the large amount of time, effort and cost that are required. Occasionally, it is proposed to perform another test and to correlate the results to in-situ dry density, but one would wonder why such a correlation would be needed at all, and why not simply adopt an appropriate and direct verification method?

At this point it may be the right time to pose another question, why are we using in-situ density for verification in the first place? The answer is quite obvious, because more sophisticated methods such as CPT (cone penetration test) or PMT (pressuremeter tes) that are able to directly or indirectly measure the required parameters are not applicable in the very thin (0.3 m) layer that the roller compactor is able to treat. In fact, when treatment thickness is thin, we are very limited in the testing method options and will not be able to adopt a testing program that would normally be the preference of geotechnical engineers.

Once it has been accepted that in-situ dry density measurement is not the proper method of testing and verifying the ground conditions of thick saturated loose soils and it has been acknowledged that in any case and at best density can only be correlated to actual design parameters and requirements, it would seem rational to adopt a verification method that is capable of directly assessing the design basis.

Ground improvement is an affordable and effective method for improving the strength parameters of thick saturated loose subgrades. As shown in Figure 2, Dynamic Compaction (DC) is a technique that can be used for the treatment of thick saturated loose granular soils [3].

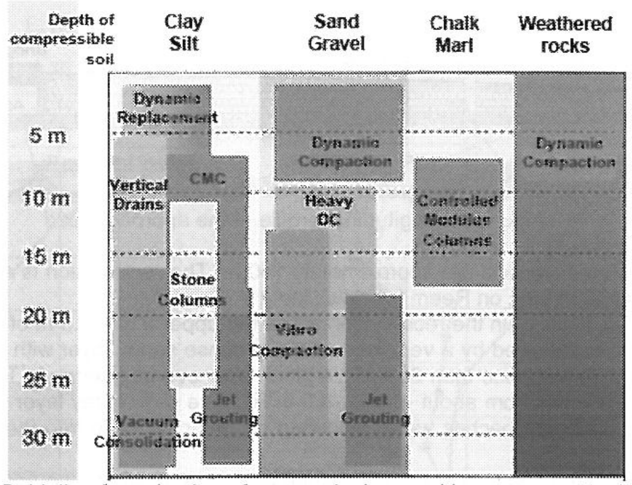

Figure 2. Guideline for selection of economical ground improvement techniques [3]

Dynamic Compaction is a ground improvement technique that was invented by the late French engineer, Louis Menard [4-5], and that can be effectively utilized for treating the loose layers of in-situ or reclaimed granular soils.

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 [6]. 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 soils the soil is liquefied and the grains re-arranged 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.

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. Drop heights are usually in the range of 10 to 20 m.

The application of Dynamic Compaction has been in the attention of highway engineers since the publication of a guideline by the Federal Highway Administration in the United States [7].

2. Case Studies

The objective of this paper is not only to demonstrate the applicability of Dynamic Compaction for the treatment of thick saturated loose fills in road projects, but also to demonstrate the importance and the determinant role of suitable design and acceptance criteria through the presentation of two case studies.

The projects that will be presented range in size from a relatively small project of 10,000 m^2 to a very large project of 900,000 m^2. Project locations were in urban areas or in undeveloped sites, and reclamation works were done either hydraulically by dredgers or by tip dumping trucks.

2.1. Case Study 1: Abu Dhabi – Reem Island Causeway

Reem Island, previously called Abu Shaoum, is a small island that is located about 0.4 km north of Abu Dhabi in the United Arab Emirates, and connected to the capital city by a causeway composed of reclamation on the two sides and a central bridge section in the middle. Each direction of the bridge has four lanes. The width of the approach road leading to the bridge is 28 m. An additional lane is envisaged on each side for drivers wishing to make U-turns 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 is provided by an MSE (mechanically stabilized earth) wall.

The longitudinal profile of the project (Abu Dhabi side) is shown in Figure 3. The approach road is constructed either directly on the coastal grounds or on reclamation. The road level is from +2.0 m RL (reduced level= mean sea level, MSL) to +7.00 m RL at bridge level with a maximum elevation difference between the low and high points of the approach road being 5 m.

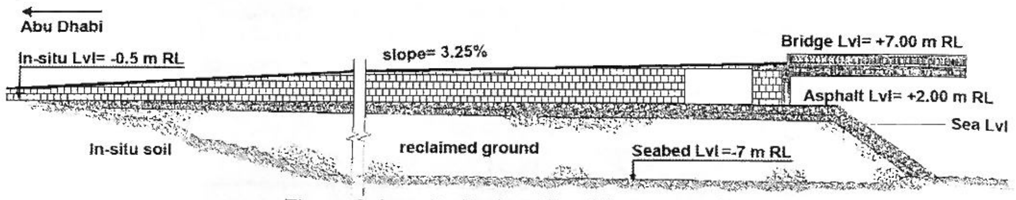

Figure 3: Longitudinal profile of the approach road

The approaches on each side are approximately 150 m. The reclamation is about 135 m long on Abu Dhabi's side and 50 m long on Reem Island's side.

As summarized in Table 1, in the reclamation areas 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 0 to 2 m, and fines content less than 20%. This latter layer overlaid bedrock. The bottom elevation of the loose sandy layer was from about -6.0 to -8.0 m RL. The soft clayey layer was dredged before reclamation and the road approaches were reclaimed by dumping sand into the Persian Gulf using dump trucks.

due to the poor ground conditions and the marine environment where the bridge piers were to be located in, the foundations of the piers were designed and constructed as drilled piles. The loose fill, with a maximum thickness of 9 m, was of concern to the project's engineers as they were not confident that bearing capacity requirements, and settlement limits could be met.

Design and acceptance criteria of the ground improvement works were based directly on the project needs; i.e.:

1. Safe bearing capacity under the approach road: The maximum required bearing capacity of the fill would be at the location where the approach road has reached the bridge elevation at +7.00 m RL. That will be achieved 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. Safety factor was stipulated to be 3.
2. Total Settlement: Although sands settle very rapidly under external loads, young fills may be subject to excessive settlements caused by self weight [8]. Well compacted or dense sand fills with pressuremeter (PMT) net limit pressures above 600 kPa will not be subject to creep[9]. Hence, total settlement was stipulated to be less than 30 mm for the traffic load (that was conservatively assumed to be 20 kPa).
3. Differential settlement: Based on the engineer's experience and the drivability requirements differential settlement of the fill was defined to be 1:500 under a uniform load of 20 kPa.

It can be observed that each criterion has been defined based on a specific problem and independent from other requirements. The authors regretfully note that in some projects the engineer stipulates settlement limitations not under the actual loading conditions but undiscerningly under a load equivalent to the stated bearing capacity. In large slabs, general shear failure and bearing capacity are most likely not critical and settlements will govern. Hence, engineers may systematically stipulate high bearing capacities and then enforce settlement limitations for such a load that does not exist in reality. This is clearly poor engineering and a waste of the project's resources.

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 -6 to -8 m RL

Table 1: Ground profile of reclamation area before ground improvement

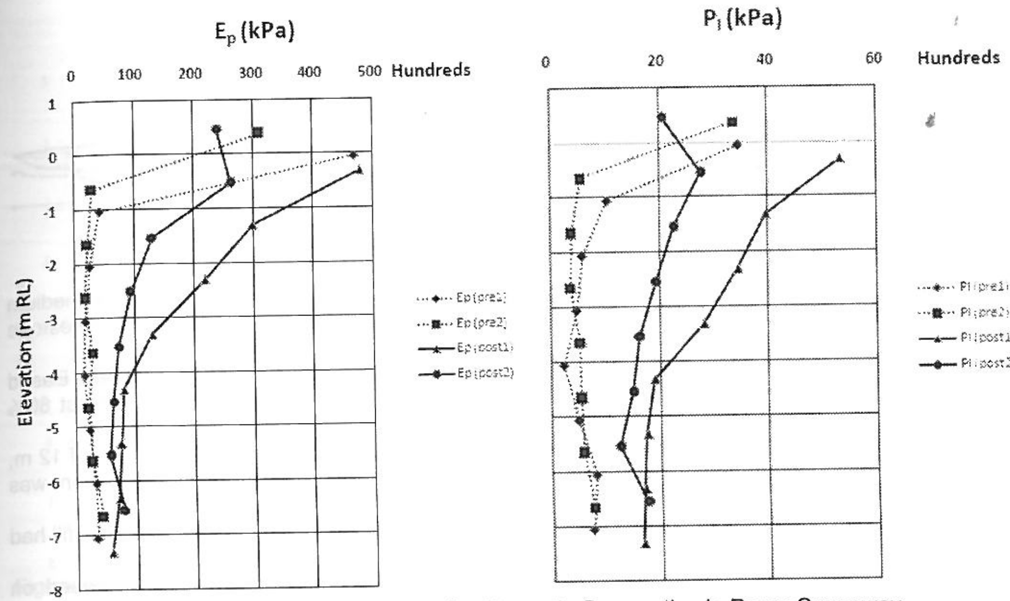

Figure 4: E_p and P_l before and after Dynamic Compaction in Reem Causeway

In this project while the bearing capacity has been set to 120 kPa for an actual requirement, it is well understood that the road will not be subject to such a load that is equivalent to a 10 m high oil tank, but conservatively subject to a much smaller amount equal to 20 kPa.

Due to the requirements of the main contractor, one Dynamic Compaction rig was mobilized twice and works were carried out in two separate phases for each end of the bridge. Each phase was executed during a period of two weeks.

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 a result of the ground improvement works by Dynamic Compaction the fill settled on average about 40 cm.

Comparisons of the pressuremeter moduli, E_p, and pressuremeter limit pressures, P_l, before and after improvement are shown in Figure 4. It can be observed that while before Dynamic Compaction construction equipment traffic had improved the soil parameters of the upper meter of soil crust, deeper layers were very loose and even subject to creep. However, after ground improvement the pressuremeter parameters have 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 using Menard's proposed method [9] 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 substantially below the acceptable value of 30 mm.

2.2. Case Study 2: Abu Dhabi New Corniche (Beach Road)

As shown in Figure 5, Abu Dhabi New Corniche is a 6 km long reclamation with an area of 900,000 m^2 that has been hydraulically reclaimed from the Persian Gulf. 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 supported by sheet piles.

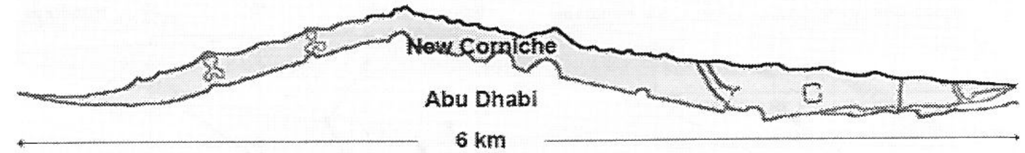

Figure 5: Plan of Abu Dhabi New Corniche

The preliminary geotechnical investigation revealed 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 contain less than 10% fines. Based on this expectation, acceptance criterion required the relative density of the soil to be at least 80% with a SPT blow count correlation as shown in Table 2.

Once the ground was reclaimed, testing indicated that the fill, with a maximum thickness of 12 m, was loose to very loose with SPT blow counts in the range of 1 to 10; hence ground improvement was stipulated and Dynamic Compaction was deemed as the most suitable method.

Further testing during the works revealed that contrary to initial expectations the hydraulic fill had segregated and a layer of at least 0.5 m thick and frequently more was covering the seabed.

It is mentionable that in such a condition, projects whose acceptance criteria are based on relative density may experience a gap of specification, and consequently a contractual dilemma. Relative density testing methods and consequently relative density itself are applicable only to soils with less than 15% fines (dry mass passing 75 microns), provided that they still have cohensionless free draining characteristics [10-11].

At this point it is worth to wonder while ASTM [12] has withdrawn its D2049-69 standard for relative density (replaced by D4253 [10] for the measurement of maximum index density) and distinguished textbook authors such as Bowles [13] go to the point where it is suggested that "relative density test or criterion is almost worthless" then why is relative density still being used by engineers as a measurement of soil strength?

Relative density was originally developed with the notion that it would be an appropriate means to define the looseness and denseness of sand or sand-gravel soils in a meaningful way because it was thought that important soil properties could be correlated quite well by this means [14], and was thought to be useful in liquefaction studies. However, that has even been reviewed now, and the application of relative density in evaluation of liquefaction resistance of soils [15] has been abandoned.

One of the most important shortcomings of relative density is that it involves three density determinations (minimum index density, in-situ density and maximum index density) [14]. Errors or different results obtained can significantly magnify the errors or differences determined for relative density. Holtz [14] notes that in worst conditions, i.e. lowest maximum and minimum values and highest in-place values as compared with the highest maximum and minimum values and lowest in-place values, the range of relative densities can be as great as 46 to 91%. This represents a range from unsuitable density to very satisfactory density.

To overcome this problem, and furthermore as extraction of undisturbed in-situ samples at depth and in submerged conditions is quite impractical, correlations are frequently used [16-19]. However research [20] shows that existing correlations that are developed based on chamber tests of silicate sands can lead to underestimation of relative density if they are applied to sand of similar stress history but with a higher compressibility.

Theoretically, the split spoon sampler of the SPT device should be able to extract 45 cm of sample; however this is not always the case in practice, and in some projects the sample may be just a few centimetres and sometimes the sampler will have a total loss of sample.

Depth (m)	N_{SPT}
0- 2	15
2- 5	18
5- 8	20
8-11	22

Table 2: Acceptance criteria based on correlation to 80% relative density

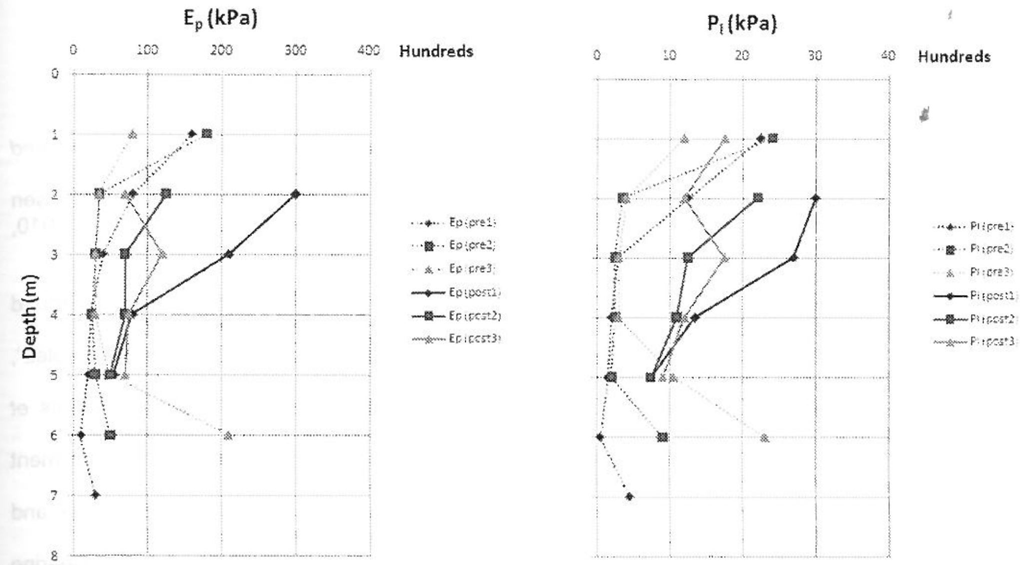

Figure 6: Menard Modulus and limit pressure before and after Dynamic Compaction in New Corniche

Thus, in addition to criteria of relative density and SPT, in the project pressuremeter testing was also carried out for the design of sheet piles and further verification of the work.

The works were carried out with a maximum number of 7 DC rigs in two working shifts with a production rate of 200,000 m^2 of improved ground per month.

The energy per unit area of treatment, pounder weight, drop height and number of phases were varied based on the treatment thickness and confirmed by a calibration programme. Pounder weights varied from 12.5 to 25 tons and a maximum drop height of 20 m was implemented. In areas with less than 6 m thickness, two phases of deep treatment using 12.5 and 16 ton pounders were utilized. In deeper areas four phases of Dynamic Compaction comprising of three phases using 25 ton pounders and the fourth phase using 15 ton pounders were carried out. Sometimes due to the buildup of pore pressure the first three phases had to be performed in sub phases. In such a case enough time was permitted for the excessive pore pressure to dissipate to tolerable values.

For comparative purposes with Reem Island Causeway, the results of the PMT before and after Dynamic Compaction are shown in Figure 6. It can be observed that here, similar to the Reem project, before ground improvement the upper crust of the reclamation was quite dense due to traffic of construction equipment, but the soil rapidly became very loose and subject to even creep. However, after soil improvement the P_l in the boreholes increases at its most from about 600 to 1000% at the depth of around 3 m. It should be noted that in this project ironing or light pounding of the upper crust was not necessary for meeting the specifications and thus not carried out, and improvement of the upper crust could have been even better if that phase was performed.

3. Conclusion

The study of the soil profile in two reclamation projects indicates that while the upper crust of the soil may be very dense due to the construction equipment traffic, the lower layers of fill (subgrade) may be very loose, subject to a number of geotechnical concerns and a hazard to the functionality of roads during the service period.

The development of appropriate design and acceptance criteria for ground improvement works is essential, critical and unsuitable or improper specifications can cause both technical and contractual problems during the course of the works. It is recommended that specifications be based on the actual concerns and written in a manner that facilitates direct assessment and evaluation of the requirements.

Dynamic Compaction can successfully be implemented for treating thick saturated loose subgrades. The amount of improvement will depend on the amount of impact energy that will be. It is possible to increase the PMT limit pressure in certain layers generally to more than 500%; however treatment effects reduce with depth.

4. Acknowledgement

The authors wish to express their gratitude to Menard for providing the paper's data.

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