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TREATMENT OF A HYDRAULICALLY RECLAIMED PORT PROJECT BY DYNAMIC COMPACTION

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Abstract

Dynamic Compaction (DC) has recently been used to improve 175,000 m^2 of hydraulically reclaimed land as part of the expansion project of Port of Ras Laffan in Qatar. The material used for reclamation was calcareous sand and gravel. The treatment thickness was variable and up to a maximum of 16 m.
Acceptance criterion for the project was originally based on minimum relative density values that had to be verified by CPT, but later alternative criteria based on performance were introduced and implemented. Pressuremeter Tests (PMT) were also carried out for verification purposes.
Due to the required depth of treatment, in this project in addition to the commonly used pounders that are typically used in heavy dynamic compaction, the innovative MARS technology was also used to drop a 35 ton pounder in free fall and to then to grab it automatically.
Post ground improvement test results and calculations have been able to demonstrate the technical, financial, construction and contractual benefits and advantages of implementing a performance based acceptance criteria instead of the relative density based criterion. Thus, this paper recommends the implementation of performance acceptance criteria that have been developed and realized by considering design criteria.

Keywords: ground improvement, soil improvement, dynamic compaction, hydraulic fill

1. Introduction

Ras Laffan is located on the southern coast of the Persian Gulf, is about 70 km north of Qatar's capital city, Doha, and houses the onshore facilities of the world's largest gas field. Nakilat Ship Repair Yard is part of Port of Ras Laffan's expansion program, and has recently been reclaimed by a hydraulic fill.

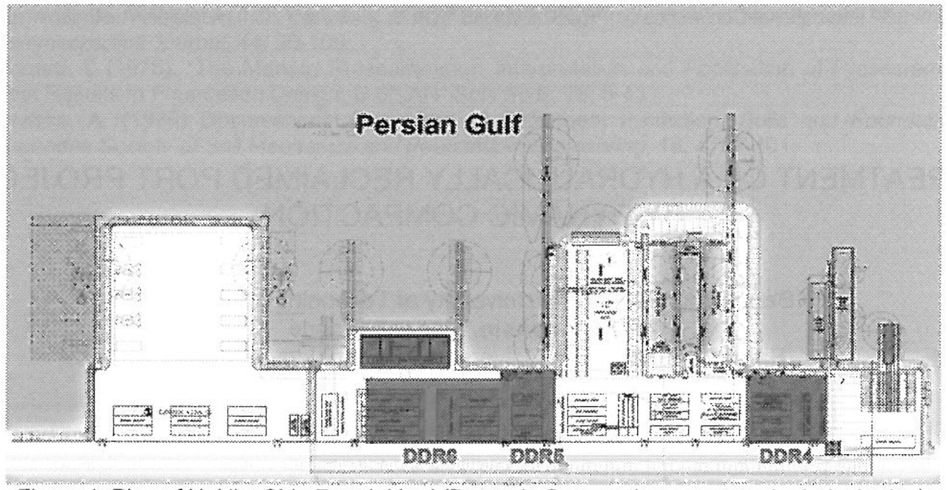


Figure 1. Plan of Nakilat Ship Repair Yard (Dynamic Compaction areas are in dark shades)

1.1. Initial Ground Conditions

Seabed level at the location of the project was variable from -9.1 m to -13.2 m CD (chart datum). Design (final platform) level was specified to be at +3.5 m CD. However as the engineers' past experiences suggested that hydraulic fills would generally be reclaimed in a loose state, it was anticipated that ground improvement would be required. Ground subsidence to do treatment was estimated to be about 0.6 to 0.8 m; hence the working platform was reclaimed to a variable level of about +4.1 to +4.3 m CD.

Reclamation was carried out using the carbonate sand and gravel that was dredged for deepening the port. The fill's grain size was generally less than 75 mm, but stones as large as 500 mm in diameter were also present. The maximum fines content of the fill was mostly less than 10% on the upper elevations, but the fill became considerably siltier at depth with occasional lenses of silt with thicknesses ranging from 0.2 to 0.4 m.

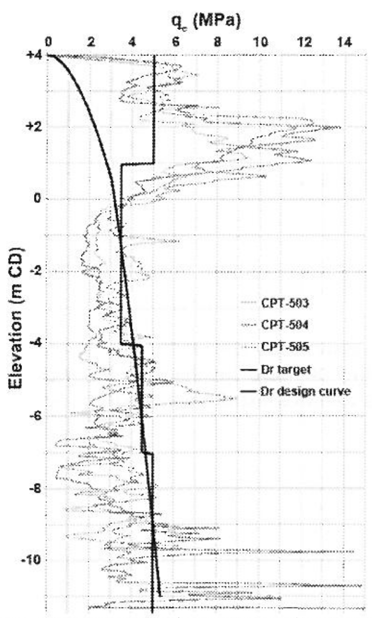


Figure 2. Comparison of CPT cone resistance before ground improvement with target relative density

CPT tests were carried out as part of the geotechnical investigation. In areas DDR4 (57,064 m^2), DDR5 (35,643 m^2) and DDR6 (82,962 m^2) of the dry dockyards (see Figure 1) the soil in the upper 3 to 5 m was medium to very dense with q_c ranging from as low as 5 to more than 20 MPa. Then the soil became loose to medium dense with cone resistance fluctuating between 1 to 7 MPa. Dense seabed was encountered at depths of 13 to 17 m, and CPT friction ratio was understood to be generally well below 1%. Three typical cone resistance logs are shown in Figure 2.

1.2. Design and Acceptance Criteria

While it was understood that less sensitive areas of the project would require lesser ground treatment, areas DDR4, DDR5 and DDR 6 were deemed to be sensitive and the ground was required to possess a relative density, D_r, of 60% based on Baldi et al. [1]:

$$D_r = \frac{1}{2.41} ln\left[\frac{q_c}{157(\sigma'_{vo})^2}\right] \tag{1}$$

q_c= CPT cone resistance (kPa)

σ'_{vo}= effective vertical stress (kPa).

According to Al Hamoud [2], Almeida et al [3] and Nutt [4] the penetration resistance of calcareous sands is lower than quartz sands with a similar grain size distribution. In this project the correction factor to be applied to the cone resistance for calcareous sand was stipulated to be 1.94.

For the purpose of calculations it was specified that the saturated density, γ_{sat}, and unsaturated density, γ_{unsat}, were respectively 18.7 and 15.2 kN/m^3. Average groundwater level was assumed to be at +0.5 m CD.

Figure 2 also shows the 60% relative density target curve and the relative density design curve which takes the soil's better characteristics in the upper layers into account and assumes a number of line segments in lieu of the actual curve. It can be observed that the relative density requirement was not met, and it was reconfirmed that ground improvement would be needed. At the same time it was noted that an alternative acceptance criteria based on design criteria could satisfy the project requirements more affordably due to the mass behaviour of the ground. Bearing capacity and settlements are the accumulative behaviour of the individual particles of the soil, not the independent response of each soil particle. Although meeting a minimum requirement for every soil particle *may* (but also as will be seen later in this paper may not) signify that the ground as a whole will meet an envisaged requirement, the other way around is definitely not true; i.e. if each soil particle does not meet a certain minimum value it does by no means signify that the ground as a whole will not satisfy a certain anticipated requirement.

The alternative criteria specified that for an isolated footing subject to a 4000 kN load the allowable bearing capacity should be at least 200 kPa, and the maximum settlement would be less than 50 mm. If the ground was able to meet these design criteria, then technical requirements would have been totally satisfied without going into the unnecessary discussion of relative density and minimum testing values.

2. The Ground Improvement Solution: Dynamic Compaction

Dynamic Compaction was chosen as the most preferable ground improvement technique because:

1. It is able to efficiently treat the widest range of soils, from silty sands with up to about 30% fines and collapsible soils to large diameter boulders [5]. An alternative technique such as Vibrocompaction loses effectiveness at much lower fines content [6].
2. It is affordable. Lukas [5] has carried out a cost comparison between different ground improvement solutions. His research shows that Dynamic Compaction is the most affordable ground improvement technique.

The basic principle of Dynamic Compaction [7-8] is the transmission of high energy impacts to loose soils that initially have low bearing capacity and high compressibility potentials in order to significantly improve their characteristics with depth. This impact energy is delivered by dropping a heavy weight from a significant height [9].

The project's schedule stipulated that mobilization, ground improvement and testing to be completed according to the below milestones:

- DDR4: 154 days after issuance of notice to proceed.
- DDR5: 63 days after issuance of notice to proceed.
- DDR6: 91 days after issuance of notice to proceed.

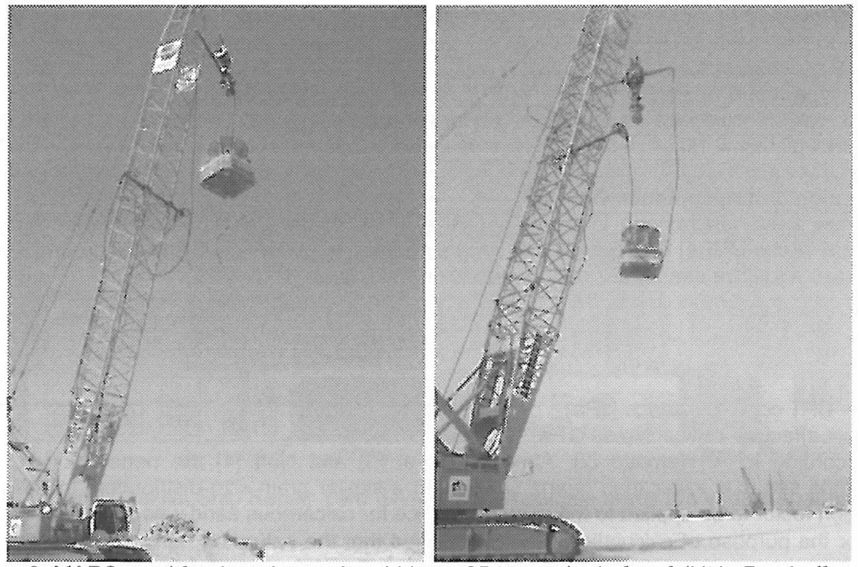


Figure 3. MARS used for dropping and grabbing a 35 t pounder in free fall it in Ras Laffan

Two specially equipped cranes with very heavy lift capacities were mobilized to treat the 175,000 m^2 in accordance to the defined schedule.

Depth of improvement in Dynamic Compaction is a function of the pounder weight and drop height. Menard and Broise [10] 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 and based on further site experiences Mayne and Jones [11] introduced a coefficient, c, to the original equation; thus:

$$D = c\sqrt{WH} \qquad (2)$$

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

Based on the fill thickness and the phase of Dynamic Compaction, soil improvement was carried out in areas DDR4, DDR5 and DDR6 using a combination of 15, 25, 28 and 35 ton pounders. The 35 ton pounder was dropped in free fall from 25 m and without engagement to the winch and cabling system using the innovative and patented MARS (Menard Automatic Release System) technology. This device was used for the first time in 2004 during the ground improvement works of a very large and thick loose fill with a maximum thickness of 28 m in Al Quoa'a, UAE [9,12]. Figure 3 shows the dropping of a 35 ton pounder using MARS.

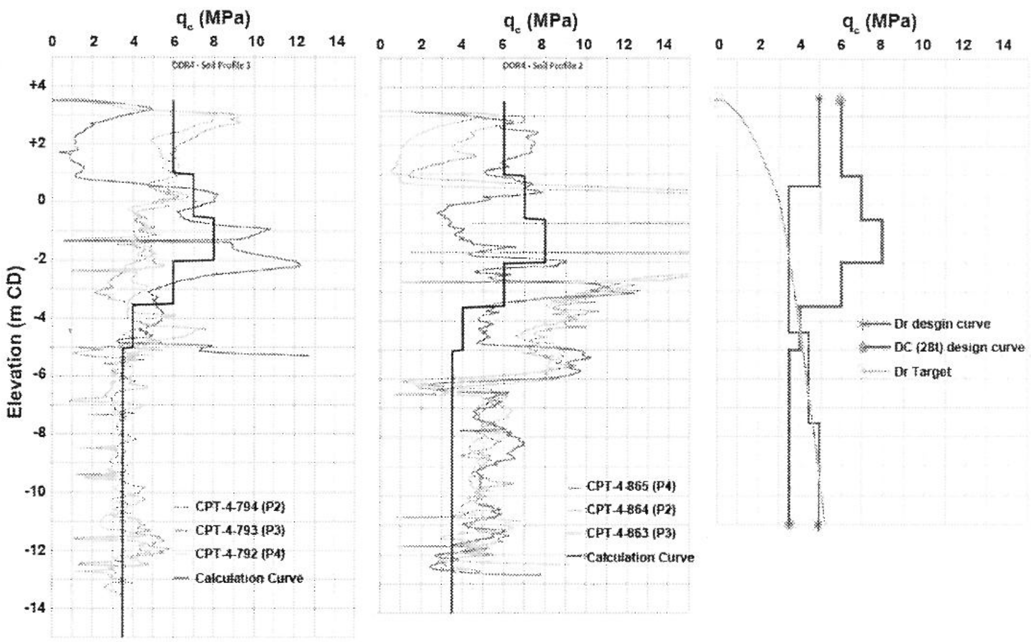
3. Testing

In order to verify that the project requirements had been satisfied one CPT was carried out per every 600 m^2 of improved ground. A number of PMT (pressuremter) tests were also carried out for comparative purposes. This provided an opportunity to perform a number of CPT and PMT tests in the same locations and to correlate the results for carbonate sands [13].

Two commonly encountered post ground improvement (using a 28 t pounder) soil profiles in area DDR4 and four commonly encountered post ground treatment soil profiles in area DDR6 are respectively shown in Figure 4 and Figure 5. The soil composition of these profiles are summarized in Table 1. It can be well seen that while the fines content of the upper layers in all areas and the entire fill thickness in some areas is composed of clean sand as originally anticipated, the extensive amount of post treatment tests were able to reveal that at depth in other areas the soil was substantially siltier than originally expected. In can be seen that in these areas there are a number of very silty sand or silt layers interbedded in the more granular soils. Fines content is some layers are considerably higher and even up to 100%. Similarly, the CPT friction ratio in some locations has been measured to be up to 7%.

Soil Profile	Levels (m CD)		Layer information		Friction Ratio	Fines Content
	Ground		Description	bottom level (m CD)		
DDR4- Profile 1	+3.5 to +4	-13.5	clean sand	-13.5	0.2%	0 to 5%
DDR4- Profile 2	+3	-13	clean sand	-4.5 to -7	0.2%	0 to 5%
			silty sand	-7 to -9.5	1 to 3%	15 to 50%
			clean sand	-13	0.2%	0 to 5%
DDR6- Profile 1	+2.5	-11	clean sand	-6 to -9	0.2%	0 to 5%
			silty sand	-11	0.2 to 0.5%	5 to 10%
DDR6- Profile 2	+3	-10.5	clean sand	-8.5 to -9.5	0.2%	0 to 5%
			silty sand	10.4	0.2 to 0.5%	5 to 10%
			silt (0.5 m thick): interbedded in silty sand layer		4 to 5%	50 to 70%
DDR6- Profile 3	+2.5 to +3	-10.5	clean sand	-5 to -6.5	0.2%	0 to 5%
			silty sand	-10.5	0.5 to 1%	20 to 30%
			silty sand/ silt: three bands, each 0.2 to 0.4 m thick		1.2 to 5%	30 to 70%
DDR6- Profile 4	+1.5 to +3.2	-10.5	clean sand	-3.5 to -8	0.2%	0 to 5%
			silty sand	-10.5	0.5 to 1%	20 to 30%
			silty sand/ silt: four bands, each 0.2 to 0.6 m thick		4 to 7%	70 to 100%

Table 1 : Soil Profiles in areas DDR4 and DDR6



(a) (b) (c)

Figure 4

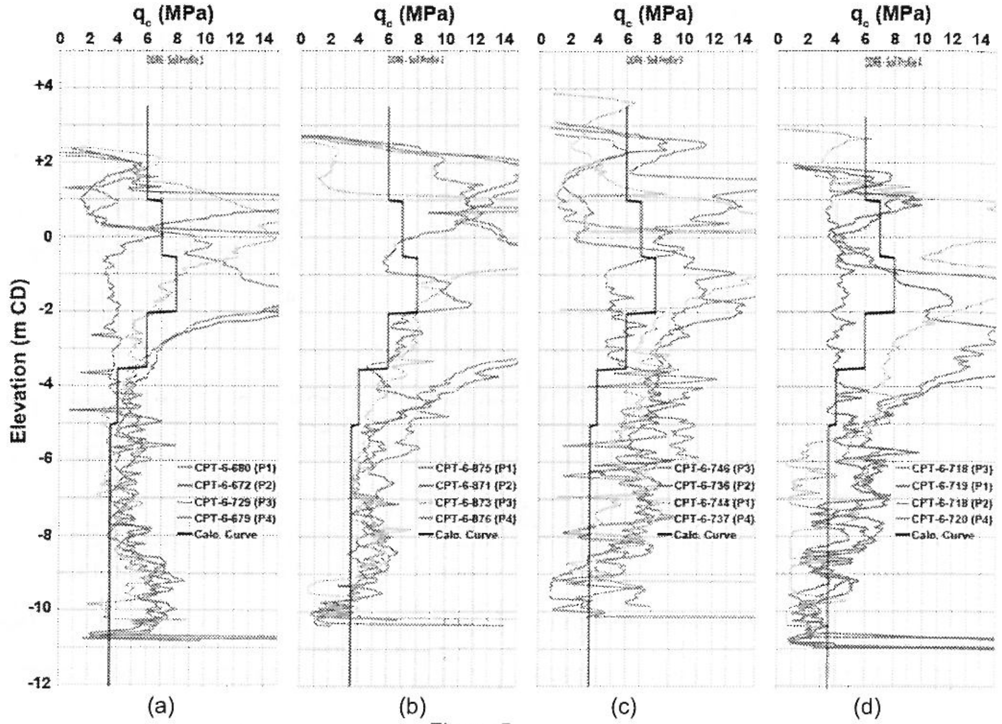


Figure 5

For design purpose, a worst case scenario of the q_c curve has been chosen from DDR4 Profile 1 (the dark line in Figure 4a), and simplified into a number of segments. The same line has been copied onto Figure 4b and Figure 5 for comparison purposes. It can be well noted that the actual ground improvement is considerably better in the other profiles.

4. Verification of Results and Comparison of Criteria

A review of the test results can immediately highlight a very important technical and contractual ambiguity: 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 [14-15]. As the fines content of many locations exceeds the maximum permissible fines content, had the alternative performance based criteria not been introduced, the project would have suddenly become without a criterion in those locations. Simply suggesting to enforce the relative density formula of Baldi et al. with the proposed correction factor is even more meaningless, and one should question how an experimental curve that was developed for the clean sands of Ticino and Hokksund can be expanded to include soils fines exceeding 50%.

Without the alternative criteria in place, at such a stage the engineers would most probably had been forced to try to develop and to agree on an alternative design criteria as a remedy in any case. The alternative design criteria would most probably have had to be based on the actual project requirements to ensure that the project would function properly during its life time. Needless to say any variation during the process of the works will most probably have additional costs, and it is clear that introducing the alternative design criteria at an earlier stage has been a very thoughtful approach.

The above should clearly demonstrate that implementing relative density as a criterion may have serious impacts on a project. Bowles [16] goes to the point where he suggests that "relative density test or criterion is almost worthless".

Irrespective of all the serious drawbacks and ambiguities that the application of relative density can have on a project, it is still possible to draw the relative density curve (knowing that it does not apply when fines content exceeds 15%) and to compare it with the worst case scenario design curve. Figure 4c compares the q_c design curve, developed in Figure 4a, with the target relative density curve and relative density curve, developed in Figure 2.

It can be observed that while the worst case scenario design curve has higher values in the upper soil layers, the relative density curve has greater values at depth.

Profile	settlement (mm)
DDR4- Profile 1	35.52
DDR4- Profile 2	30.67
DDR4- *Dr* design curve	50.16
DDR6- Profile 1	28.33
DDR6- Profile 2	33.66
DDR6- Profile 3	33.66
DDR6- Profile 4	35.06
DDR6- *Dr* design curve	50.16

Table 2: Settlements of the *Dr* design curve and the soil profiles subject to a load of 4,000 kN

Using the equation proposed by Bowles [17] for estimating the ultimate bearing capacity, q_{ult}, for a square footing on a cohesionless soil

$$q_{ult} = 4800 - 0.9(300 - q_c)^2 \quad \text{(kPa)} \tag{3}$$

Jullienne [18] has calculated the allowable bearing capacity for each of the q_c curves with a factor of safety of 3. The allowable bearing capacity for the relative density design curve and worse scenario design curve have both been calculated to be much more than 200 kPa and respectively 400 and 414 kPa.

Likewise, using the method proposed by Schmertmann [19-20], Jullienne has also calculated the settlements for the relative density design curve and each of the soil profiles that have been shown in Figure 4 and Figure 5. In his calculation Jullienne has assumed that the square footing is subject to 4,000 kN and with a pressure of 200 kPa (4.5mx4.5m footing). The result of the calculations is summarized in Table 2. It can be seen that it is in fact the relative density design curve that will result in the maximum amount of settlement. The settlement of that curve is about 40 to 80% more than what the ground will actually settle under the design load.

The above calculations clearly show that although the q_c results after Dynamic Compaction at depth may have been less than the relative density curve, yet all soil profiles have performed better than the relative density specification. This is clearly another reason to avoid relative density based specification and to utilize performance based criteria.

5. Lessons to be Remembered

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

1. Young hydraulic fills are most likely going to be in a loose state and potentially subject to low bearing and excessive settlements.
2. Dynamic Compaction is an affordable and high production ground improvement technique that can be used for treating loose saturated sands..
3. Proper determination of design criteria is very important and failure to adopt a suitable specification can redefine a project.
4. Acceptance criteria should be based on design criteria and actual project requirements. Specifying a minimum test value based on relative density not only complicates the works and may lead to contractual and technical ambiguities due to the presence of layers with more than 15% fines content, but also does not necessarily provide a more stringent criterion. Indeed it can be demonstrated that adopting such a criterion may unjustly fail results that perform better than the specification itself.

6. Conclusion

The present paper has demonstrated the ability of Dynamic Compaction to treat loose hydraulic fills as deep as 16 m. The technique has been successful even when fines content was more than what was originally expected.

More importantly, this paper has shown that once it has become known that the in-situ soil conditions will not satisfy the project requirements, proper realization of the project's specifications are crucial and failure to adopt performance based criteria founded on actual project needs and stipulation of other criterion such as relative density can seriously jeopardize the project's outcome. Specifications based on relative density may falsely fail results that are otherwise better in performance (more bearing capacity and lesser settlements).

7. **Acknowledgement**

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