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SOIL IMPROVEMENT OF A VERY THICK AND LARGE FILL BY DYNAMIC COMPACTION

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Abstract

Al Quoa'a is a remote and isolated desert township in the United Arab Emirates. Desert dune sands were levelled without compaction to create a flat construction platform. The first phase of the project was constructed without implementation of specific foundations solutions. Consequently, most of the structures that were built on the fill areas suffered from very severe damages and deep cracks. The second phase of the project has also been constructed on a levelled platform with a non-engineered fill area of about 1.13 million m^2 and a maximum fill thickness of 28 metres.

Ground improvement was carried out based on a design and build concept. Design and acceptance criteria were specifically tailored to satisfy the project needs and Dynamic Compaction was used to treat the loose dry sands of the very thick and large fill for bearing capacity, total, differential and creep settlements within a contractual duration of 10 months using 6 specially modified rigs. The treatment was optimized by implementing a combination of different pounder weights and impact energies. For the first time ever, the innovative and patented MARS (Menard Automatic Release System) was used to efficiently drop a 35 ton pounder in free fall. Menard Pressuremeter Tests were used to verify that acceptance criteria had been fully satisfied.

Keywords: ground improvement, soil improvement, dynamic compaction, pressuremeter test

1. Introduction

Al Quoa'a is a remote and isolated Emarati desert township that is located about 100 km from Al Ain and on the border of the United Arab Emirates and Oman. The first phase of this new town was constructed by cutting and filling the dune sands and levelling the ground for creating the town's platform. Although the buildings were only two stories high, most that were built on the fill areas suffered from severe damages and substantial amounts of deep cracks, and it became evident to the developers that specific measures had to be implemented during later phases of the project to avoid more damage.

Figure 1. Backfilling and construction of the working platform by dumping

The second phase of Al Quoa'a, consisting of 450 two floor villas and the associated infrastructure was planned to be constructed on a site with a rectangular shape of 2.3 km by 1.65 km and an area of more than 3.8 million m^2. This phase was also anticipated to be constructed on a relatively flat platform. Thus, as shown in Figure 1, in early 2003 the dune hills were once again cut and dumped into the lower level areas.

The fill areas were eventually measured to be covering an area of 1,135,000 m^2. More detailed assessments revealed that 44% of the fill material was placed at depths less than 6 m, 35% were from 6 to 12 m deep, 13% were from 12 to 16 m deep and the remaining 8% were located at depths deeper than 16 m from the finished working platform (±0.0 m RL). The maximum depth of the fill was 28 m.

CPT test results indicated that while the ground condition in the cut areas was satisfactory enough to support the structures and infrastructures, as could have been predicted the dumped dune sands in the fill areas were in a rather loose state and except for the upper two meters, the cone resistance was constantly in the low range of 2 to 4 MPa. The higher values of the soil strength (q_c in the range of 10 to 15 MPa) in the upper crust was contributed to the effects of the earthworks equipment. Likewise, Pressuremeter Tests (PMT) tests that were later carried out as part of the post ground improvement programme indicated that the limit pressure, P_l, was in the range of 0.1 to 0.5 MPa.

As shown in Figure 2, the sieve analyses of the dune sand showed that the soil was poorly graded fine clean sand.

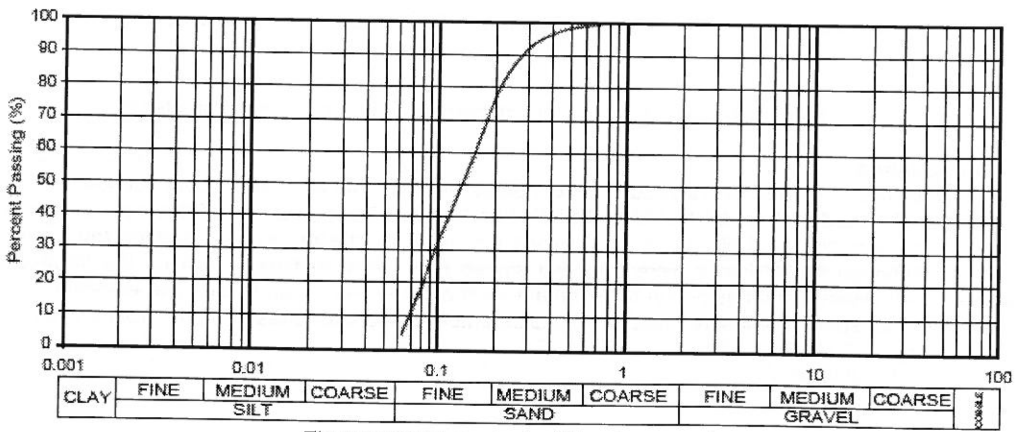


Figure 2.Sieve analysis of the dune sand

Groundwater was not observed or recorded in any of the tests that were carried down to a maximum depth of about 30 m.

The low strength of the fill indicated that the ground would not be able to safely support the footing loads of the villas. It was expected and calculated that the facilities of this phase of the project would suffer from more settlement, cracks and other damages if no specific remedy was anticipated for the loose fill.

It was understood by the project's designers that in addition to settlements originating from structural loads, the very young fill could have also been subject to settlements under its own weight, or to subsidence due to vibration or washing and densification of material.

Piling was deemed as an unfavourable solution for a number of reasons. Firstly, the solution was only feasible for supporting the structural loads, and was not applicable for preventing the other settlements mentioned earlier. Furthermore and although calculations were not made at that phase, the piles had to be longer than the fill thickness to avoid the settlements of the piles themselves in the creeping soil. This would have incurred unjustifiable costs and construction time.

Ground improvement was deemed as a possible solution for the treatment of the very loose fill.

2. The Ground Improvement Solution

Among the number of solutions that were submitted to the consultant and project manager by specialist ground improvement contractors, the concepts of the proposal that was of most interest, technically, financially and with the shortest construction schedule, was based on a design and construct (D&C) methodology using Dynamic Compaction with a contract period of 10 months.

2.1. Developing the Solution

In this proposal it was acknowledged that in sufficiently optimized ground improvement projects the design criteria must be well understood and defined. Experience of the specialist contractor suggested that it is more effective, more affordable, more rational and faster to treat specific problems with acceptance criteria directly verifying that the design parameters have been satisfied rather than developing generalized requirements.

The approved design criteria of the project are summarized in Table 1. In this project the concerns were related to bearing capacity and settlement of the structures. There was also the settlements associated with creep or self bearing of the soil. At the same time, the villa and non-villa areas were sufficiently large enough to be able to address their requirements separately.

In villa areas, single footing loads were known to be less than 100 tons, so it was possible to stipulate an allowable bearing capacity (200 kPa) and thus a maximum footing size (2.25 x 2.25 m^2). Total settlements were limited to 25 mm, and angular distortions were stipulated to be less than a very stringent value of 1/1250 due to the special psychological concerns and the previous disappointing results of the first phase of the project. It was also possible to determine the stress bulb influence depth and to specify self bearing treatment for further depths.

In non-villa areas a nominal bearing capacity of 100 kPa was stipulated for light and unanticipated loads.

As bearing capacity, load induced and creep settlements can all be calculated using the Menard Pressuremeter Test [1], this testing method was chosen for verification of results.

P_l is a suitable characteristic for evaluating soil creep, and a value of 0.6 MPa is deemed sufficient for eliminating creep in sands [1-2]. The ultimate bearing capacity can be also calculated using Menard's equation [1]:

$$q_l - q_o = k(P_l - P_o) \tag{1}$$

q_l= 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.

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

Design Criteria	Villa Areas	Non-Villa Areas
Bearing capacity	200 kPa at -0.75m RL	100 kPa at -0.75m RL
Total settlement	25 mm	25 mm
angular distortion	1/1250	1/1250
Creep settlement	To be eliminated	To be eliminated

Table 1. Design criteria

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:

$$P_l = \sqrt[3]{P_{l1}P_{l2}P_{l3}} \quad (2)$$

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

2R is equal to the width of the foundation.

It is not necessary to specify any values for P_l and PMT results in conjunction with Equations 1 and 2 can be directly used to verify that bearing capacity has been achieved; however in this project it was agreed to implement a minimum P_l value conservatively and based on a safety factor of 3.

Similarly, total settlements can be calculated that footings with a half width more than 0.3 m, [1]:

$$s = \frac{1.33}{3E} + pR_o\left(\lambda_2 \frac{R}{R_o}\right)^{\alpha} + \frac{\alpha}{4.5E} p\lambda_3 R \quad (3)$$

s= settlement

p= mean contact stress on a rigid footing

R_o= a reference length equal to 0.30m

λ_2, λ_3= shape coefficients

E= Menard modulus of deformation

α= rheological factor

Once again, it is possible to use E values from the PMT to calculate settlements using Equation 3, but it was agreed to conservatively specify a minimum value for the modulus. The acceptance criteria are summarized in Tables 2 and 3.

Lukas [3] has carried out a cost comparison between different ground improvement solutions. His research shows that when applicable, Dynamic Compaction is the most affordable ground improvement technique. As a comparison, the treatment of a unit volume of soil by Dynamic Compaction costs 42% to 70% that of Vibro Compaction or 7 to 15% of excavation and replacement.

Dynamic Compaction was chosen as the ground improvement method as it was also the experience of the specialist contractor 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 [4-5]. Research also suggests that this technology is relatively environmentally friendly and produces less carbon emissions than alternative technologies [6]. Furthermore, Dynamic Compaction does not require water or any material, and uses minimum amount of equipment. These advantages can be very worthy in isolated sites where water, and material are difficult to provide and distances from equipment repair facilities can be considerable.

2.2. Dynamic Compaction and its Application to the project

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 [4].

Criteria for Villa Areas	Safe bearing	Self bearing
Thickness where parameters prevail	-0.75 to-5.50 m RL	From -5.50 m RL
P_l (kPa)	750	600
E_p	4.8 MPa	4 MPa

Table 2: Acceptance criteria for villa areas

Criteria for Non-Villa Areas	Safe bearing	Self bearing
Thickness where parameters prevail	From ±0.00 m RL	From ±0.00 m RL
P_l	0.6 MPa	0.6 MPa
E_p	4 MPa	4 MPa

Table 3: Acceptance criteria for non-villa areas

The depth of improvement is a function of the pounder weight and drop height. Menard and Broise [9] 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 [10] introduced a coefficient, c, to the original equation; thus:

$$D = c\sqrt{WH} \quad (4)$$

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

In all, a total of 6 rigs were used for performing the ground improvement works. Typically and depending on the required depth of improvement, a pounder weighing up to 15 tons is dropped by heavy duty cranes from up to 20 m. More impact energy can be applied by using heavier pounders and higher drop heights; however special cranes or specially designed equipment will be needed.

It can be verified that utilizing heavy duty cranes could have provided sufficient lift capacity to treat about half of the project; however the remaining areas were deeper than what could have been treated by such equipment and specialized rigs were required to perform Heavy Dynamic Compaction. Thus, the specialist contractor's locally available special Liebherr cranes that had been used for treating up to 12 m of saturated soils in Abu Dhabi's New Corniche [11] and a specially designed Menard 700 t.m rig were mobilized to drop 25 ton pounders (Figure 3). Although these special plants were able to provide sufficient amount of impact energy to treat about 90% of the site, the energy was never-the-less not sufficient for treating the deepest areas.

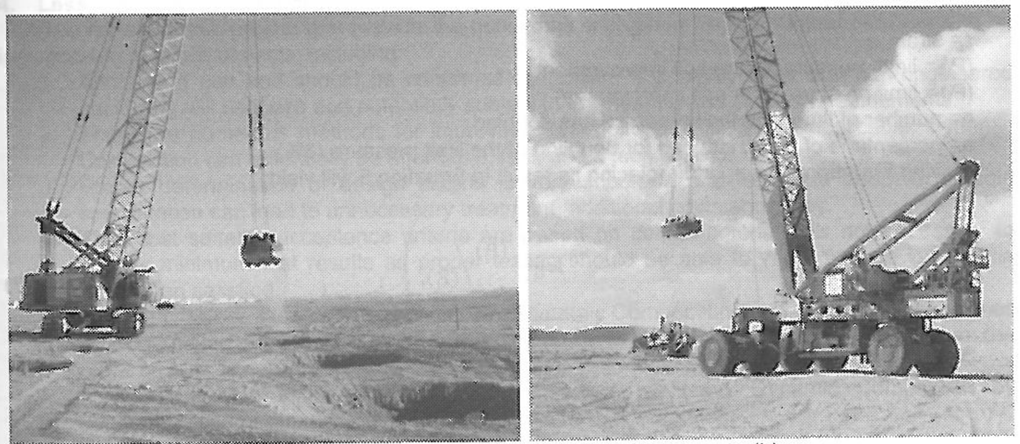
Consequently, the specialist contractor developed a patented technology called MARS (Menard Automatic Release System) that was used to drop a 35 ton pounder in free fall without any connections to the winch and cable system. For the first time ever, the innovative concepts of MARS allowed it to drop and grab the pounder automatically and without the assistance of labour (see Figure 4). The same technology was later used for treating a deep hydraulic fill [12] and pre-collapsing karst cavities [13].

Dynamic Compaction was optimized based on the treatment thickness and acceptance criteria. MARS, Liebherr and 700 t.m rigs were used for the first phase of treatment of the deep fill areas. Heavy duty cranes were used for the treatment of subsequent phases of those areas and the remaining sections.

3. Testing

In all, 250 PMT were carried out at the site. For comparison purposes, 50 tests were carried out before ground improvement, and the remaining 200 tests were performed after Dynamic Compaction to verify the results. Depth of testing was based on the fill thickness and depth of dense in-situ soil.

Figure 5 shows P_l values before and after Dynamic Compaction in four locations. The test values are substantially higher than the minimum required values, indicating the success of implementing Dynamic Compaction even without the need of going into calculations.



(a) (b)

Figure 3. Dynamic Compaction special rigs at work in Al Quoa'a, (a) Liebherr rig, (b) 700 t.m rig

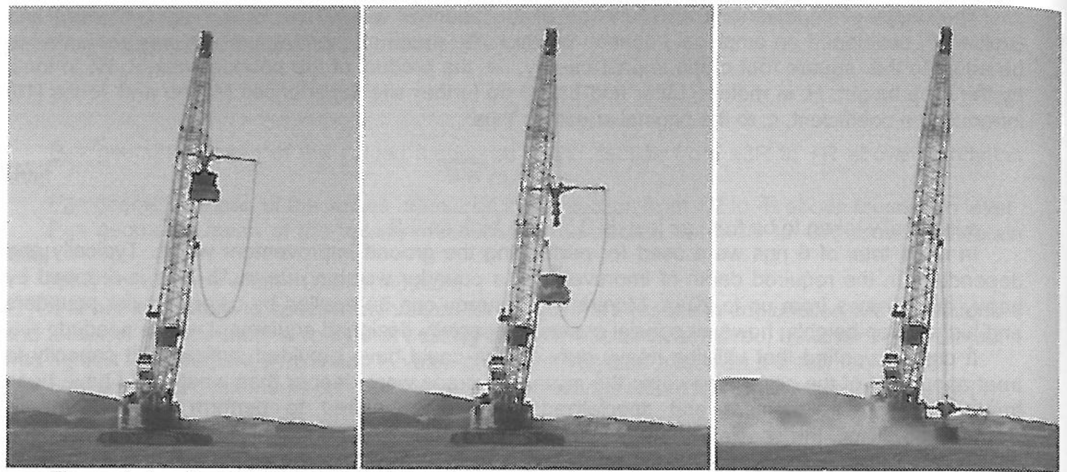


Figure 4. Application of MARS for dropping in free fall and automated lifting a 35 ton pounder

As can be observed all post improvement test results basically follow the variation with depth that Lukas [3] has reported. The P_l values increase to a peak value at about ⅓ to ½ of the influence depth and then deceases to a point where improvement would have become negligible.

Lukas [3,5] also assumes an upper bound post dynamic compaction P_l value of 1.9 to 2.4 MPa for sands and gravels which is in line with the results of the works carried out in this project. However research by the authors and others; e.g. Spaulding and Zanier [14], suggest that higher Pl values is possible.

Lukas [3,5] also suggests an upper bound improvement value of 400% for P_l due to Dynamic Compaction. The results of Figure 5 indicate that depending on the initial P_l value, it is possible to improvement the P_l of very loose dune sands up to 1900%. The post Dynamic Compaction P_l results of this project clearly suggests that, at least for dumped desert (non-saturated) dune sands, Lukas' anticipated upper bound improvement percentage is much lower than what is actually achievable.

Based on Varaksin et al. [2], every time the limit pressure doubles there will be 3% of strain. Hence, it can be said that:

$$\varepsilon = na \tag{5}$$

$$\frac{(P_l)_j}{(P_l)_i} = 2^n \tag{6}$$

ε= strain
$(P_l)_i$= limit pressure before soil improvement
$(P_l)_j$= limit pressure after soil improvement
n= number of times the limit pressure has doubled
a= percentage of strain induced for doubling of the limit pressure (3%)
Solving Equation 6 for *n*, and replacing its result in Equation 5 will yield:

$$\varepsilon = \frac{\log\left(\frac{(P_l)_j}{(P_l)_i}\right)}{\log 2} a \tag{7}$$

Settlement can be calculated to be:

$$s = \sum_{k=1,m} h_k \varepsilon_k = \frac{a}{\log 2} \sum_{k=1,m} h_k \log\left(\frac{(P_l)_j}{(P_l)_i}\right)_k \tag{8}$$

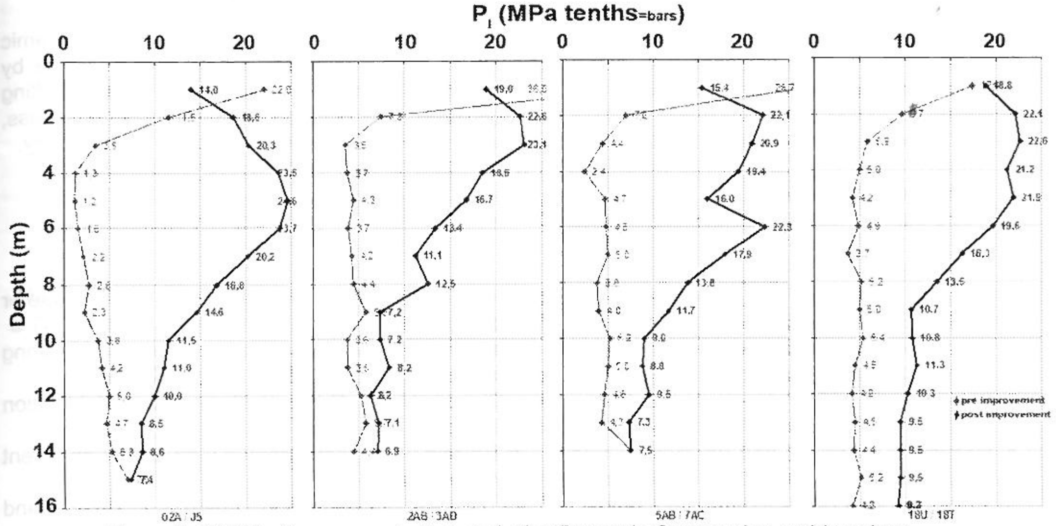


Figure 5. PMT limit pressure before and after Dynamic Compaction at 4 locations

m= number of pressuremeter tests in the borehole within the improvement zone (i.e. the depth where P_l has increased), and h_k is the testing interval. If $(P_l)_j/(P_l)_i$ is denoted by r (r ≥ 1), then the settlement can be calculated to be:

$$s = \sum_{k=1,m} h_k \frac{log(r)_k}{log\,2} a \tag{9}$$

Replacing the values of *a* (0.03) and log 2 in Equation 9 will yield

$$s = 0.1 \sum_{k=1,m} h_k \log(r)_k \tag{10}$$

For example, it can be observed that the maximum Dynamic Compaction induced strain in the first set of curves of Figure 5 (PMT-02A and PMT-J5) is 13.1% at the depth of 5 m. Total settlement can be calculated to be about 89 cm which is in line with site measurements.

4. 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. Although it can and should be confirmed by testing, it is highly likely that non-engineered backfilling will be loose and potentially subject to low bearing and excessive settlements.
2. There are numerous methods for improving non-engineered fill. When applicable, Dynamic Compaction can potentially be affordable, fast, and a technique of interest.
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.
4. The most suitable acceptance criteria are based on design criteria. It is not necessary to specify minimum test results as proper testing should be able to verify that design criteria have been satisfied.
5. In large projects, it is preferable to mobilize Dynamic Compaction rigs with different capacities and to optimize treatment by providing different levels of impact energy based on the requirements of each zone.
6. It is possible to improve the depth of influence in Heavy Dynamic Compaction by implementation of the free falling and automatic MARS technology.
7. It is possible to improve the P_l values of dune sands by up to 1900%; however the peak amount of improvement decreases with depth. Of course, the amount of improvement will be a function of impact energy.

5. Conclusion

The present paper demonstrates the ability to improve very thick and large fills using Dynamic Compaction in remote locations with harsh conditions. The achievement has become possible by implementing optimizing design criteria, allocating sufficient number of rigs with the required lifting capacity, providing optimized levels of impact energy based on the design criteria and fill thickness, and ultimately performing tests that are able to measure the design and acceptance criteria properly.

6. Acknowledgement

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

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