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The Effect of the Quantity and Degree of saturation of Fine grained Portion of Granular Soils in DC Operations

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Abstract

DC (Dynamic Compaction) can be regarded theoretically similar to earthquakes, with regard to liquefaction and cyclic mobility trigger, and also blast, with regard to inducing a high amount of energy in a very small fraction of the time. But although there are some similarities, it is none of the above that makes the prediction and explanation of the phenomena difficult. It was found that three main factors as 1) quantity of fine grained portion and atterberg limits 2) the physico-chemical formation of the fine grained portion 3) degree of saturation governs the efficiency and predominant mechanism of the DC operations. A case history is discussed briefly and then some conclusions are presented. It was found the in granular soil with high percentage of fine grained portion based on the Atterberg limit and the water content of the soil two distinct outcomes may occur, as where before the borderline the DC gave satisfactory results, the results for the same soil with exactly identical tamping pattern were unacceptable and the tamping energy was completely dissipated and the soil column compaction in the exploitation zone was far below the required specifications.

Keywords: Dynamic compaction, clay, Ground Improvement, SPT, Atterberg limits

1. INTRODUCTION

The soil treatment method of dynamic compaction (DC) involves releasing a heavy tamper from a predetermined height to impact the underneath soil mat. Compared with other reclamation techniques, DC has great advantages in reinforcement depth, efficiency, and overall cost [1-3]. Most of the investigations can be categorized on the following subjects: the degree and the depth of improvement of DC [4, 5], the factors affecting the overall efficiency of DC [6, 7], and the dynamic properties of the compacted soil [8-10]. The DC process can be summarized as a soil foundation suffering a series of responses with the impact load of the tamper. The three key elements in studying the DC process are: (1) the time-domain characteristics of the input affecting the soil changes; (2) the regular pattern of the soil response with the impact load; and (3) the method for evaluating the reinforcement level. [10]. This article deals with the second and the third part of this category.

[11] reported that for a given tamping energy per blow, an increase in momentum was found to improve the crater depth, whereas for a fixed tamping momentum, increasing energy had comparatively less effect on the crater depth. Neither the same tamping energy nor the same momentum can produce the same crater depth because of different combinations of hammer weight and drop height. [11] stated that the DC efficiency is affected by various factors including the hammer weight and drop height while other important factors, such as particle-size distribution, water content, and soil type, that also make a difference

Regarding the exploitation of land from the sea, soils from pre-determined borrow areas are poured in the sea and after achieving the required level based on the drawing from MSL (mean sea level) it is compacted based on the DC patterns. The focus of the most published article regarding DC compaction in land exploitation from the sea is mainly on migrating liquefaction [12], in situ tests [13] or using construction waste as the fill material [14] and less emphasis is placed on the different sedimentary-soil type that might be used in the operation. This article deals with a case history where as a borrow material i.e. GC (granular with high clay content) was used in a land exploitation from the sea and described the obstacles and solutions faced regarding the soil from the borrow area in that project.

2. DC for Sedimentary soil in land exploitation from the sea

Based on the publication No.FHWA-SA-95-037 Circular No.1 [15] the soil under DC categorized in three groups: Zone 1 that is the most favorable soil deposits, Zone 2 intermediate soil deposits which special care should be regarding them and Zone 3 that is unfavorable soil for DC. As it can be seen in figure 1, the main difference between these groups is the hydraulic conductivity where in zone 1 is greater than 10^{-3} while in zone 3 it is less than $10^{-8.}$ Zone 2 hydraulic conductivity lies between 10^{-5} and 10^{-8} that is considered as a semi- pervious. The most determinative factor in the suitability of a soil type to be improved by dynamic compaction when groundwater table is near the ground surface, is its ability for dissipation of excess pore pressure[16]. This is because when granular materials extend below the water table, a high proportion of dynamic impulses is transferred to the pore water. If applying impacts continues before allowing excess pore pressures to dissipate, pore pressure eventually rises to a level which induces local liquefaction. In these conditions, compaction effects of further drops of heavy tampers decrease dramatically, rendering the technique to become un-economic. So it is important to predict the level of the induced pore pressures during heavy tamping, especially in semi pervious soils [16]. As it can be seen in figure one, the sand is placed in zone 1 while silts and clay soils are the least favorable soils for DC.

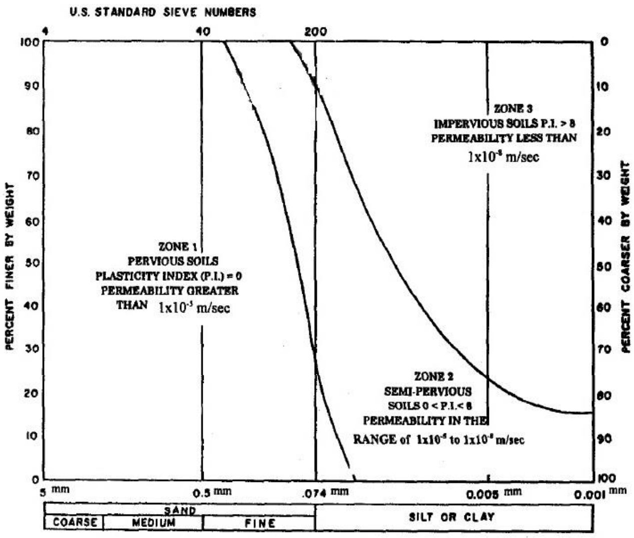


Figure 1: grouping of soils for DC [15]

In the real world the soils acquired mostly from the borrow area are sedimentary soils i.e. a mixture of coarse-grained and fine-grained portions. Such soils have some specifications of fine-grained and coarse-grained at the same time that can't be divided from engineering perspective. While a fine portion might only be a small fraction of the soil structure, mostly 5 to 25 percent, the dominant behavior of the sedimentary soil is defined by it. Based on the quantity and genetics and degree of saturation of fine grained portion of granular soils the designation and efficiency of the DC Operations might be changed completely.

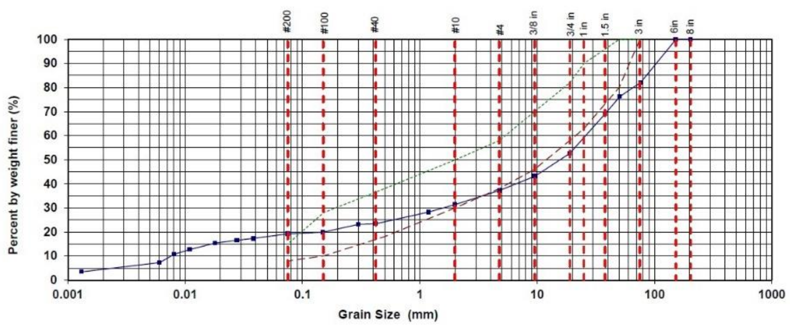
By heavy tamping, compression waves, shear waves and surface waves are introduced to the soil mass. Compression waves are principally responsible for the rupture of the in-place skeleton structure [17] and shear waves cause shear displacement and rearrangement of the soil particles [18]. Body waves affect the soil mass in different ways. Part of the energy may cause favorable effects in the densification process. On the other hand, a great deal of energy attenuates via displacement of soil particles (without volume reduction), elastic deformations, noise, and heat [16]. In dry/moist granular soils, physical displacement of the particles and consequently the collapse of soil skeleton is the main compaction mechanism. To a lesser extent, low frequency excitation after each impact also contributes in the compaction process by reduction of void ratio and increase of relative density [16]. While only modest levels of stress impulse might be expected to cause collapse of hitherto loose zones in dry soil, saturation implies that such cavities are filled with pore water and collapse would be inhibited by resistance of the water to compression and a stress pulse would at most destroy some arching structures near a proportion of potential compaction zones and initiate some small amount of long-term consolidation under ambient ground stress. However, real saturated soils may have a few percent of total void space occupied by occluded air or gas. In this condition, the stress wave intensity would be greatly reduced because of the low bulk modulus of the pore water, but the impact stresses would still ensure the compaction of the loose structure due to the reduction of gas volume in the voids[19]. Ground response to DC in cohesive soils is different and of course more complex. Menard and Broise [35] introduced the following four factors as the main causes for densification of fine-grained saturated soil layers: high compressibility of water due to the presence of air microbubbles, increase of permeability of the soil mass due to progressive fissures around the impact points, gradual liquefaction under repeated impacts, and thixotropic recovery [20].

The preceding discussions have focused on fine-grained soils (i.e., silts and clays) for which the fines content (percent passing the No. 200 sieve) is greater than 50% by definition, but the same findings may be extended to soils with slightly lower fines contents in certain cases. The key issue is whether or not the fines fraction constitutes the stress-carrying matrix or skeleton for the soil mass, with the larger sand-sized (or larger) particles essentially floating (isolated from each other) within the matrix. For many soils, it is likely that the fines fraction forms the load carrying matrix when the fines fraction exceeds roughly 35%, but the transition may occur at higher or lower fines contents in any specific soil depending on factors such as the soil's full gradational characteristics, mineralogical composition, particle shapes, and depositional environment or fabric. For projects where this transition point is of critical importance, it would be prudent to perform an appropriate program of in situ and laboratory testing to evaluate the soil's behavioral characteristics prior to extending these criteria to fine contents less than 50% [20]. Since for the DC compaction there are reports stating that demolition of the grains was observed [21-23]

Tests and engineering judgment are the sole tools for the determination of the suitability of such soils for DC operations. Unfortunately, in common practice the selection of borrowed material for land exploitation is mainly based on the availability of the material. Since in some regions the availability of good borrowed material is limited, there might be cases where soils that based on their hydraulic conductivity lie in zone 2 or even zone 3 might be selected for DC operations. In such cases intensive tests and monitoring should be done for assurance of the pre-defined quality. In the case of application of borrowed soils with low conductivity less than 10^{-5} the physico-chemical specification of the fine grained part is the main determinative factor that is mostly neglected in the projects. Atterberg limits is one of the tools that gives unique precious information regarding the fine-portion part of the sedimentary-soil. As it can be seen in figure 1, a range is specified regarding this matter, that is for zones the PI should be from 2 between 0 to 8 while in soils in zone 3 it is greater than 8. Regarding sedimentary -soil including a high amount of fine grained percentage the saturation should be below the plastic limit. If the saturation reaches the plastic limit, regardless of the energy applied, the tamping energy would be completely dissipated and even much higher tamping energy compared to the preliminary pattern doesn't improve the treatment operation anymore.

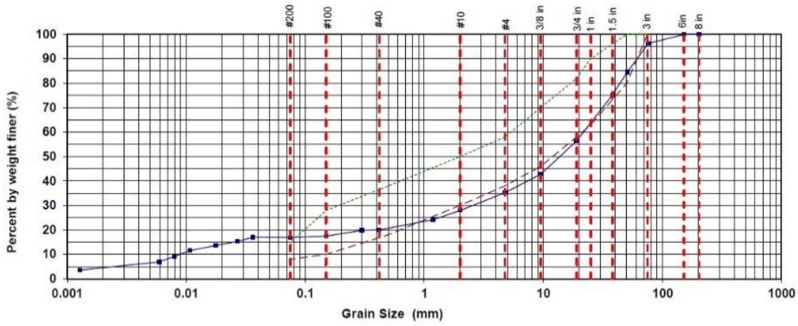
3. QUANTITY OF THE FINE GRAINED PORTION AND ATTERBERG LIMITS

Figure 2 illustrates the sieve analysis of the soil taken from the borrow site. It is clear that the percentage of the fine grained portion is very high as 19.2 and 16.9 percent. Since there is a high percentage of fine grained in the soil, the plasticity index (PI) is also as high as 15 and 11.5 respectively. In the first glance the soil seems to be well-graded but it is very misleading that would be discussed in the following sections.
The preliminary technical specification had restricted the percentage of the fine grained portion to 5 percent and PI to 8 percent. Based on unified system the favorable soil with this specification would be well graded gravel with little (less than 5%) or no fines or GW or Poorly-graded gravels i.e. gravel-sand mixtures with little (less than 5%) or no fines that surely is not the case here.



Material	symbol	PERCENT OF BREAKAGE	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	Passing#200	Sand	LL	PL	PI	UNIFIED
SH15.2	—■—	-	0.007	1.647	27.572	19.2%	18%	26.1	11.1	15.0	GC

(a)



Material	symbol	PERCENT OF BREAKAGE	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	Passing#200	Sand	LL	PL	PI	UNIFIED
SH15.1	—■—	-	0.004	2.780	22.699	16.9%	18%	27.0	15.5	11.5	GC

(b)

Figure 2: The sieve analysis of borrow area taken from (a) the mine (b) the site

4. THE ORIGIN OF THE DEPOSITS

Marls or marly geo-materials are present in many parts of Iran, especially in southwest Iran. [24] conduct an extensive field study regarding the geotechnical and geo-engineering characteristics of formations in the south pars zone. In their research it was stated that marly beds mostly belong to the Aghajari and Mishan formations which entail the gray, cream, black, green, dark red and pink types. The first formation is Bakhtiari which is mostly alluvial deposits. Marls can be observed as rock (soft rock) or soil. Marlstone outcrops show a relatively rapid change to soils in the presence of weathering. Marls are classified as weak geo-materials with highly variable engineering behavior which range from soils to rocks. The mechanical behavior of the marls is controlled by the presence of clay minerals and carbonate content where an increase in the carbonate content improves the mechanical property of marls. As indicated by [24] the major clay minerals of the studied marls belong to either the smectite (montmorillonite) or illite groups. The smectite group is classified as a soft phyllosilicate that precipitates from water solution as microscopic crystals. Since the individual crystals of the montmorillonite clay are not tightly bound, water can intervene and cause marly soils to swell. On the other hand, the water absorption activity of the illite group is much lower than that of the smectite group. However, when the clay type in the marls belongs to the smectite group, the clay mineral activity is very important to evaluate. As it can be seen in figure Mishan and Aghajary formations are the dominant groups in the geological map while they are the main source of the marl in the region too. Mishan formation (molasses, carbonate and siliciclastic facies deposited in a carbonate rimmed shelf and of gray marl and marlstone with clay layers, olive green to gray and sometimes red marls), Aghajari formation (fine, medium and coarse grained sediments, usually interpreted as channel deposits and alternating gray to brown calcareous sandstone, gray, dark green and pink to red marl with veins of gypsum, gray marls, green siltstone) with an age attributed to Miocene and Pliocene and alluvial deposits [25]. Figure 3 illustrates the satellite location of Asalouyeh and the earth formation situated there.

As it is shown in figure 4a and 4b, it is clear that the borrow area is the Mishan formation that was explained to consist of carbonate rocks and marl silts and clays. The clay portion is mostly made up of montmorillonite and can be regarded as CL or CH based on the Unified system. The soil is mostly extracted directly from the formation and a small portion from deposits fallen from above formation, which is Aghajari and Bakhtiari formation.

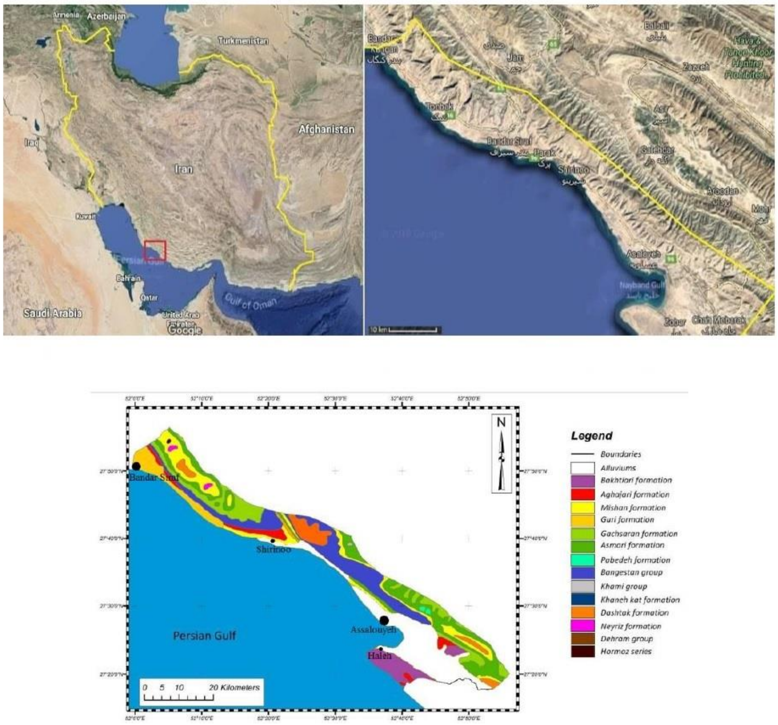


Figure 3: Satellite location of Asalouyeh and the earth formation situated there [24]

5. DEGREE OF SATURATION

The land exploitation project that is discussed here had two parts, as the one in the back of the natural coastline and the part that the exploitation of land has passed the preliminary natural coastline. The degree of saturation in the back part of the coastline had a 1 to 3 meter backfill while the part after the natural coastline had regions with about 10 meters of backfill. It is clear that by crossing the border of the natural coastline the degree of saturation of the backfill had completely changed where in regions with more than 5 meters all the soil column was completely saturated.

(a)

(b)

figure 4: (a) the borrow area (b) the soil embankment before it was taken to the site

6. DISCUSSION

DC can be regarded theoretically similar to the earthquakes, with regard to liquefaction and cyclic mobility trigger, and also blast, with regard to inducing a high amount of energy in a very small fraction of the time. But although there are some similarities, it is none of the above that makes the prediction and explanation of the phenomena difficult. In order to give some explanation of the case history both are discussed briefly and then some conclusions are presented. Regarding the liquefaction, soil vulnerable to liquefaction had an open micro-fabric in which clay aggregations generally gathered at the sand particle contact points, forming low-strength “clay bridges” that were destroyed easily during cyclic or blast loading. On the other hand, the micro-fabric of soil that was resistant to liquefaction appeared to be more compact, with the clay producing a matrix that prevented sand grains from liquefying. In the case of the natural soils, their behavior under dynamic loading is similarly influenced by factors such as clay content, clay mineralogy and plasticity [26]. Dispersive clays are a particular type of soil material in which the clay fraction erodes in the presence of water by a process of deflocculating. This de-flocculation occurs when the inter-particle forces of repulsion exceed those of attraction so that the clay particles go into suspension, and if the water is flowing, as in a crack in an earth embankment, the detached particles are carried away and piping occurs [27].

For the case presented here there were no problems in DC where the borderline of the coastline was not passed. Although the DC operation was done with a very high amount of energy (12 blows of 600 to 800 ton.m for the phase 1 & 2) the soil penetration tests (SPT) were satisfactory as shown in figure 5. But in the areas where the fill height has reached 5 meters or more the problem has aroused. Even in the case of a test pattern with 2 phases of 800 ton.m with 12 impacts there was no significant improvement. Based on the topics that were discussed the following point should be considered: Firstly, since the poured soil (taken from the body of Mishan formation) as a result of high percentage of fine grained marly dispersive clay including montmorillonite (CL or CH) was very sensitive to water, when the degree of saturation was increased, the soil structure was completely changed. Before the coastline border all the soil column has an even degree of saturation less than PL and that is the reason DC operation was possible with a high amount of energy. But after the coastline border where because of sea tide and water the soil column becomes partly saturated, based on the height from the sea bed the LL and PL limits are passed. The regime in the soil column is completely different as in lower parts the clay portion is dissolved or absorbs a high amount of sea-water and becomes swollen. It seems that in the bottom part of the column as a result of the hydraulic gradients of sea tide, a percentage of fine part is washed away. In the mid-height the PL limit had reached a barrier structure is formed that absorbs the tamping energy and as a result no significant compaction was observed from the SPT tests.

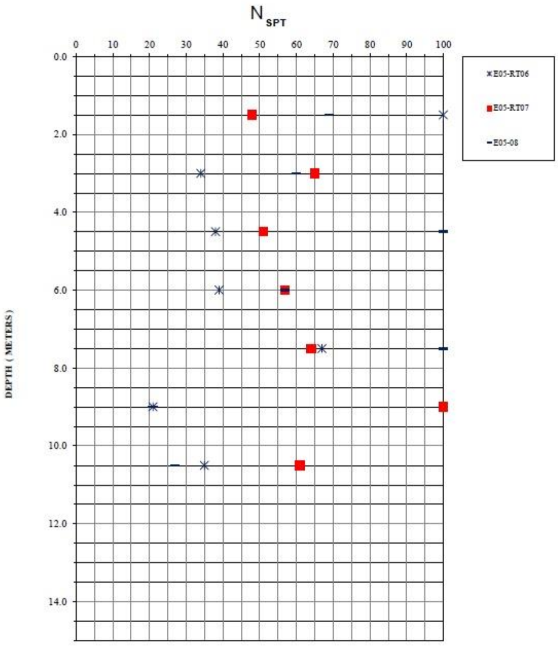


Figure 5: SPT results for the exploitation area before the borderline

7. CONCLUSION

Three factors as 1) quantity of fine-grained portion and Atterberg limits 2) the physio-chemical formation of the fine grained portion 3) degree of saturation governs the efficient and predominant mechanism of the DC operations in granular soils. While the quantity of fine-grained portion and the physio-chemical formation are inherent characteristics of the soil, the degree of saturation is a function of the field condition. For the case presented there were no problems in DC where the borderline of the coastline was not passed. Although the DC operation was done with a very high amount of energy (12 blows of 600 to 800 ton.m for the phase1 & 2) the soil penetration tests (SPT) were satisfactory but in the areas where the fill height has reached 5 meters or more the problem has aroused. Even in the case of a test pattern with 2 phases of 800 ton.m with 12 impacts there was no significant improvement. Based on the topics that were discussed the following point should be considered: Firstly, since the poured soil (taken from the body of Mishan formation) as a result of high percentage of fine grained marly clay including a percentage of montmorillonite (CL or CH) was very sensitive to water, when the degree of saturation was increased, the soil structure was completely changed. Before the coastline border all the soil column has an even degree of saturation less than PL and that is the reason DC operation was possible with a high amount of energy. But after the coastline border where because of sea tide and water the soil column becomes partly saturated, based on the height from the sea bed the LL and PL limits had passed. The regime in the soil column is completely different as in lower parts the clay portion is dissolved or absorbs a high amount of sea-water and becames swollen. It seems that in the bottom part of the column as a result of the hydraulic gradients of sea tide, a percentage of fine part is even washed away. In the mid-height the PL limit had reached and a barrier formed that absorbed the tamping energy and as a result no significant compaction was observed from the SPT tests. For granular soils with high percentage of fine-grained portion based on the Atterberg limit and the water content of the soil two distinct outcomes may occur, as where before the borderline the DC gave satisfactory results, the results for the same soil with exactly identical tamping pattern were unacceptable and the tamping energy was completely dissipated and the soil column compaction in the exploitation zone was far below the required specifications. It seems that in exploitation projects that DC is considered for the improvement of the soil, the fine-grained portion should be restricted to 5 to 10 percent based on the physico-chemical formation of the soil. As shown here even in the case of scarcity of the satisfactory soil, granular soils with high portions of the fine grained should be avoided and other borrow deposits should be considered even if situated in long distances.

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