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Evaluation of Compaction Effects on Granular Backfill Using CPT

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Abstract: This paper examines the cone penetration testing (CPT) results prior to and after two passes of deep dynamic compaction (DDC) to investigate compaction effects on the loose saturated granular backfill between steel sheet pile walls and peat bogs. The site characterization results show that the backfill was significantly densified after each pass of compaction. Two factors, (1) the existence of soft soil layers, and (2) severe liquefaction after DDC, greatly reduced compaction effects. The depth to which significant densification took place ranged from 6.5 to 9.5 m after the first pass of DDC, and 7.5 to 11 m after two passes. Finally, this paper recommends parameters for the evaluation of densification depth based on the comparison of the results from this study with those obtained from the literature.

INTRODUCTION

The U.S. Route 44 relocation project starts from the existing Route 44 at Carver, MA to the existing Route 3 near Plymouth, MA. The relocated roadway will be a four-lane divided highway with a typical median width of 18.3 m instead of the current two-lane road. Parts of Route 44 relocation project at Carver, MA traveled through cranberry bog areas with deep peat deposits. At some stations, peat deposits were up to 10.5 m deep. Water table levels were near the existing ground surface. Figure 1 presents typical peat bog construction at this site. Because of its poor engineering properties and environmental concerns, all the peat in the proposed roadway sides were required to be excavated and then backfilled with granular soils, using sheet pile walls as retaining structures. The sheet pile wall was located about 18.3 m off the proposed highway centerline. As the site was not dewatered during this process, it was suspected that most backfill was in a loose saturated state, susceptible
to liquefaction during an earthquake. Two passes of deep dynamic compaction (DDC) were performed to transform the heterogeneous backfill soils into a more uniform layer, which would consequently mitigate potential liquefaction hazards, increase soil strength, and decrease compressibility and settlement.

Using cone penetration testing (CPT) and seismic cone penetration testing (SCPT) prior to and after DDC, compaction effects on the backfill were evaluated (the CPT and SCPT results around stations 156+00 to 160+00 had been reported by Hajduk et al. in 2004). In order to verify whether all the loose, saturated backfill is densified, the influence depth for each pass of DDC is determined. In the meantime, observation on CPT data shows that two factors, (1) existence of soft soil layers, and (2) severe liquefaction after DDC, will reduce compaction effects. Finally, a parameter, $n$, is recommended for the evaluation of densification depth. The authors believe that the parameter, $n$, can be used in other DDC design and construction.

**SOIL CONDITIONS**

The backfill depths at the project site range between 2.5 and 10.5 m. Based on the grain size analysis results (refer to Figure 2), the backfill mainly consists of fine to coarse sand, with trace amount of gravel and silt. It is considered liquefiable soil according to the procedures developed by Tsuchida (1970). Triaxial tests were performed on representative backfill samples, which were compacted in the laboratory to similar relative density and moisture content as the in-situ backfill before and after DDC, respectively. The test results indicate its effective friction angle, $\phi'$, was around $32^\circ$ at a unit weight of $18 \text{ kN/m}^3$ before DDC, while $36^\circ$ at a unit
weight of 19.5 kN/m³ after DDC. The backfill was underlain by medium dense to dense sand.

**Figure 2. Grain-size distribution of the backfill (from Tan 2005)**

**DEEP DYNAMIC COMPACTION (DDC)**

As a widely used ground improvement method, DDC was “rediscovered” by Mr. Louis Menard in French in 1965, who transformed the crude tamping method into a rational compaction procedure. This method is carried out essentially by repeatedly dropping a very heavy tamper onto the soil surface from a relatively great height. DDC can be used for both unsaturated and saturated granular soils. The densification process in unsaturated soils is the process of expelling air out from voids, thereby decreasing void volume and making soil denser. The mechanism of densification of saturated soils is based on dynamic consolidation (Ye et al., 1992). The energy produced by DDC in soils destroys the soil structure and causes the water trapped between soil particles to be expelled out. As a result, cracks are formed, allowing rapid dissipation of pore pressure generated during DDC. Then, the soil consolidates, and soil particles are forced into a denser state.

The backfill at Rt. 44 was impacted by two passes of a 14.4-ton tamper with a base area of 1.8 m², dropped from a height of 18.3 m. Each tamping point was repeatedly impacted nine times. The time interval between the two passes was six days. Figure 3 presents a typical DDC square grid-pattern with a spacing of 4.6 m and the locations of CPT soundings used in this study. The second pass of DDC was conducted at the center spacing of the first pass. The distance between tamping locations and sheet pile wall was no less than 4.6 m to avoid unexpected large sheeting deflection during DDC. Following the completion of each pass of DDC, all the craters caused by DDC were filled.
A series of cone penetration tests (CPT) were carried out inside the compaction treated area and outside the compaction treated area to evaluate compaction effects. The initial evaluation test (EI) was conducted prior to the start of DDC. The second test (EV1) was conducted following the completion of the first pass. The subsequent third test (EV2) was conducted following the completion of the second pass. In this paper, only the CPT results inside the compaction treated area are presented and discussed. The CPT soundings in this study were located at the center spacing of the first and second pass of tamping points, and spaced 30.5 m apart along the central line of the constructed roadway (refer to Figure 3).

**CONE PENETRATION TEST (CPT)**

Figures 4(a), 4(b), and 4(c) present typical CPT results achieved before and after the DDC at stations 140+00, 142+00, and 144+00, respectively. The backfill mainly consisted of clean sands and gravelly sands. However, at station 142+00, layers of soft soils were identified at a depth of 9.5 to 11 m. The soft soils were the residual peat which had not been replaced. This was also verified by the standard penetration test (SPT) performed by the Massachusetts Highway Department (MHD). Except for topsoil near the ground surface, the cone tip resistance, $q_t$, and side friction, $f_s$, of the backfill were significantly improved after each pass of DDC. For most of the backfill, the $q_t$ values were improved up to 7 times after the first pass, and up to 12 times after two passes. The water table levels did not show significant changes because of the cut-holes in the steel sheeting, which were designed to allow the water table level on
both sides of sheeting to be at the same level to keep sheeting structure in balance. The distribution of friction ratio, \( R_f \), versus depth was more uniform after DDC. At station 140+00, following the completion of the first pass of DDC, the total vertical stress, \( \sigma_v \), along depth had some increase, while the second pass did not cause apparent changes. At station 142+00, there was small increase in the total vertical stress after the first pass. Similar to station 140+00, the second pass at station 142+00 did not induce changes. At station 144+00, the total vertical stress along the depth had significant reduction after the first pass, while it returned to its original state after the second pass. This reduction of total vertical stress after DDC could be attributed to the severe liquefaction induced by the pore pressure, which had not dissipated completely after DDC. If the maximum pore pressure generated during DDC cannot create cracks in the soil by expelling the water trapped between soil particles, pore pressure cannot be drained quickly after DDC and remains at a high level. As a result, soil particles will lose contact with each other (liquefaction). The CPT results also indicate that the compaction did not induce significant densification effects on the in-situ medium to dense sand underneath the backfill.

**Figure 4(a). Typical CPT site-characterization results at station 140+00**

Figure 5 shows the comparison of tip resistance, \( q_t \), at stations 140+00, 142+00 and 144+00 before and after DDC. In order to present clear comparisons, the measured \( q_t \) values at the depths of 3.5 to 6 m are discussed in greater detail. Before the DDC, the measured \( q_t \) values were close at all three stations. Following the completion of the first pass, the \( q_t \) values at both stations 142+00 and 144+00 were still the same, but about 3 to 4 MPa less than the values achieved at station 140+00. This could be attributed to the existence of residual peat at station 142+00 and the severe liquefaction at station 144+00. After the second pass, the \( q_t \) values at station 144+00 were around 3 to 6 MPa less than those at station 140+00, but 1 to 2 MPa larger than those of station 142+00. One possible explanation is that the residual peat at station 142+00 acts as an impermeable boundary at the bottom of backfill. Compared to the other two stations (double-drainage), the pore pressure induced by DDC in the
backfill at station 142+00 (single-drainage) could not dissipate quickly after compaction, which thus reduced densification effects. Another possible explanation is that the residual peat could not provide strong confinement to its overlying backfill during DDC. This suggests that prior to DDC, soft soils existing at relatively shallow depths should be replaced. The comparison results also indicate that if backfill liquefied severely after DDC, an additional pass of compaction should be applied.

Figure 4(b). Typical CPT site-characterization results at station 142+00

Figure 4(c). Typical CPT site-characterization results at station 144+00
Influence Depth Evaluation

A criterion for ground improvement evaluation based on CPT tip resistance \( q_t \) values has been widely used. Dove et al. (2000) developed the ground improvement index \( I_d \), which is calculated in the following way:

\[
I_d = \frac{q_{t,\text{after}}}{q_{t,\text{before}}} - 1
\]

where, \( q_{t,\text{before}} \) is tip resistance before ground improvement, and \( q_{t,\text{after}} \) is tip resistance after densification at the same location. The increase of \( q_t \) values \((I_d>0.0)\) indicates densification effects. Figure 6 shows typical calculated \( I_d \) at three investigated stations. Based on the investigation results at twelve stations, the depth of densification, \( D \), at the site was around 6.5 to 9.5 m after the first pass of DDC, and around 7.5 to 11 m after two passes.

Figure 7 presents the relationship between square root of energy per drop and the depth to which significant densification took place. Menard and Broise (1975) suggested that the depth of densification, \( D \), could be calculated by \( D = \sqrt{W \cdot h} \), in which \( W \) is weight of tamper in tons and \( h \) is drop height in m. Many researchers lately pointed out that the equation proposed by Menard and Broise (1975) overestimates effective densification depth substantially and suggested that it should be adjusted by multiplying a factor, \( n \), to account for other factors that affect the depth of densification besides the mass of the tamper and the drop height, namely:

\[
D = n \sqrt{W \cdot h}
\]

where, \( n \) is empirical coefficient less than 1.0. Leonards et al. (1980) suggested that \( n \) was about 0.5. Mayne et al. (1984) pointed out that \( n \) was around 0.3 to 0.8. Gambin
(1987) suggested that \( n \) was around 0.5 to 1.0. Ye et al. (1992) recommended that for cohesionless soils, \( n \) could be 0.5; for cohesive soils, it could be 0.35 to 0.5. For the granular soils at this study, the determined \( n \) was around 0.35 to 0.55 with an average value of 0.45 following completion of the first pass of DDC, while 0.45 to 0.65 with an average value of 0.55 after two passes. At the same tamping point, two passes of compaction resulted in greater densification depth than that induced by only one pass. Comparing the results from this study with those available from the literature, the developed \( n \) values reasonably reflect the available experience.

**CONCLUSIONS**

Based on evaluation of the CPT results prior to and after DDC, the following conclusions were obtained:

1. DDC was very effective for densifying loose saturated granular soils at the site studied. After two passes of DDC, all backfill had been improved by densification.
2. CPT is an effective tool to characterize densification effects of DDC. After compaction, the tip resistance, \( q_t \), and side friction, \( f_s \), of backfill increased significantly. The \( q_t \) values could be increased up to 7 times after the first pass of DDC and up to 12 times after two passes. The distribution of friction ratio, \( R_f \), with depth was more uniform after each pass of DDC. The water table level did not show significant changes because of the cut-holes in the sheet pile walls.
3. To achieve desirable compaction effects on granular soils, the identified soft soil layers at relatively shallow depths should be completely replaced prior to DDC. Otherwise, the existence of soft soils may reduce densification effects.
(4) If saturated granular soils are found to liquefy severely after the first pass of DDC, an additional pass of compaction should be applied.

(5) The depth to which significant densification took place ranged from 6.5 to 9.5 m after the first pass of DDC, and 7.5 to 11 m after two passes.

(6) The empirical coefficient, $n$, equal to 0.45 is recommended for evaluating densification depth in granular soils impacted by one pass of DDC, and 0.55 for those impacted by two passes.

**ACKNOWLEDGEMENT**

The authors would like to express appreciation to the following people and organizations for providing related information: Dr. E. L. Hajduk of WPC, Inc.; Dr. N. Hourani and Mr. P. Connors of Massachusetts Highway Department (MHD); P. A. Landers, Inc., Massachusetts; TerraSystems, Virginia; Geotechnical Testing and Research, Inc. (GTR), Massachusetts, and WPC, Inc.

**REFERENCES**


