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Ground Improvement Against Liquefaction with Dynamic Compaction and Utilization of Jet Grouting Against Settlements - A Case Study

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ABSTRACT: A factory structure was planned on deep alluvial deposits within a seismically active region. Several ground improvement techniques were evaluated during the planning stage to mitigate the existing liquefaction risk. Dynamic compaction was proposed as an alternative to high-modulus soilcrete columns (e.g., deep soil mixing), with the intent of creating a dense crust layer and reducing liquefaction potential. Pre- and post-treatment field investigations were conducted to validate the design, and back-analyses were performed where necessary. The influence of liquefaction on the structure was assessed using the LPIish method, based on field test data obtained before and after ground improvement. Dynamic compaction was implemented over an area of approximately 100,000 m², targeting improvement within the upper 10 meters of the soil profile. Trial zones were established to determine suitable dynamic compaction parameters, and field loading tests, CPTs, and SPTs were performed within these areas. Design assumptions were compared against the results of the field tests. The majority of the factory's foundation system consists of continuous footings. To control settlements, jet-grouting was applied beneath these footings, and verification tests (including core sampling, unconfined compressive strength testing, and diameter measurements) were conducted to compare design parameters with field performance. In locations subjected to higher structural loads, jet-grouting was also applied beneath the slab to reduce differential settlements, and load tests were carried out on selected jet-grout columns. In sections with raft foundations and excavation depths reaching up to 6 meters, jet-grout columns combined with steel tubes were utilized as the shoring system.

KEYWORDS: Liquefaction, Dynamic Compaction, Jet Grouting, Ground Improvement

1 INTRODUCTION

The construction of engineering structures in seismic zones with high liquefaction potential requires careful evaluation of ground improvement techniques in terms of both technical performance and constructability. In this study, a factory planned on a liquefiable site in northwest Türkiye was assessed using several soil improvement options during the planning phase. Dynamic compaction (DC) was ultimately identified as the most reliable, rapid, and cost-effective method for mitigating liquefaction risk when compared to high-modulus soil-crete techniques such as deep soil mixing.

The DC program aimed to create a dense crust within the upper 10 meters of the soil profile over an area of approximately 100,000 m². To evaluate the effectiveness of the improvement, pre- and post-treatment field tests (including CPT, SPT, and plate load tests) were performed. In addition, the liquefaction potential index was calculated using the LPI_{ish} approach proposed by Maurer et al. (2015), which builds on the method of Ishihara (1985). These analyses were used to assess changes in liquefaction potential before and after treatment.

The factory foundation system predominantly consists of continuous footings, beneath which jet-grouting was proposed to reduce settlements. For heavily loaded slab zones, additional jet-grouting treatment was incorporated. Jet-grout columns combined with steel tubes were also utilized to support deep excavations reaching depths of up to 6 meters.

This paper presents an evaluation of the dynamic compaction and jet-grouting techniques employed in the project, supported by field testing, back-analyses, and performance assessments, with the objective of providing practical insights into the optimization of ground improvement in liquefiable soils.

2 SOIL PROFILE

The soil profile predominantly consists of silty sand layers. Although gravel and clay layers are also encountered in certain locations within the site, these layers are not continuous across the entire project area. Therefore, they were not considered in the soil model used for the dynamic compaction design.

The idealized soil profile adopted in the analysis is presented in Table 1 and Figure 1.

Table 1. Soil Profile.

Layer	Thickness (m)	γ (kN/m³)	E (MPa)	c' (kPa)	Ø' (°)
Silty Sand	*3*	*17*	*7*	0	31
Silty Sand	*15*	*17*	*8*	0	32
Sandy Gravel with Interbedded Clay	*5*	*19*	*9*	5	31
Clayey Silty Sand	*Deeper Levels*	*17*	*10*	0	35

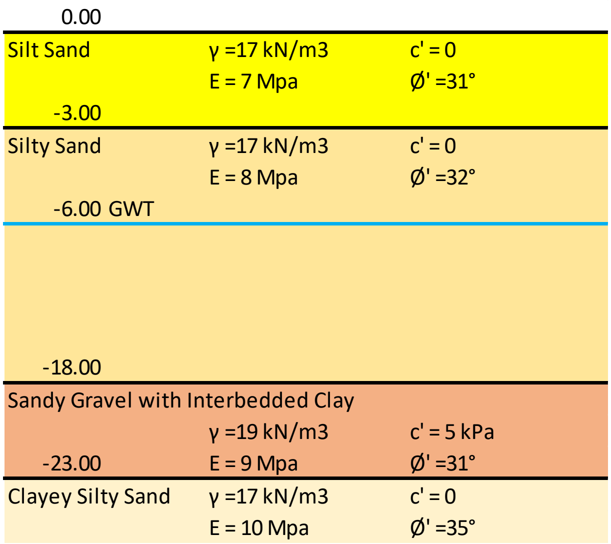

Figure 1. Idealized Soil Profile

3 DYNAMIC COMPACTION

Among various ground improvement techniques, dynamic compaction is widely used to enhance the engineering and mechanical properties of loose soils or uncontrolled fills, especially at significant depths, with the aim of reducing total and differential settlements and eliminate the liquefaction risk.

This ground improvement method primarily requires two main components: a heavy-duty crane and weights. Most of these cranes are specially designed to drop specific weights from varying heights (determined by project-specific criteria) allowing the necessary compaction energy to be transmitted into the soil through one or more drops (FHWA, 1995).

According to the FHWA Dynamic Compaction manual (Lukas, 1986), the recommended unit energy to be applied is 200–250 kJ/m³ for coarse-grained soils, and 250–350 kJ/m³ for semi-permeable fine-grained soils.

An improvement depth of 10 meters was targeted. According to Figure 2. of the FHWA Dynamic Compaction manual (Lukas, 1986), for a target SPT N-value of 30, the required applied energy was estimated to be approximately 3000 kJ/m².

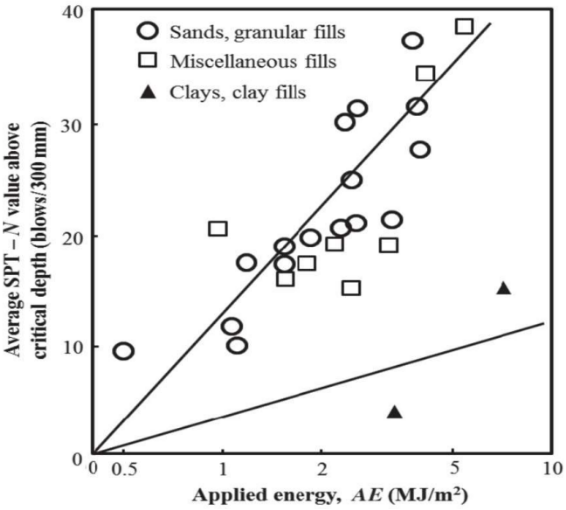

Figure 2. Average SPT N value after improvement (after Lukas,1995)

As part of the preliminary design for the targeted ground improvement, the following dynamic compaction parameters were considered: a drop height of 23 meters, a weight of 17 tons, 2 passes, and a grid spacing of 6 meters (Figure 3), resulting in an estimated 16 drops per point.

$$AE = N * W * H * P / sh^2 \quad (1)$$

The applied energy (AE) required to achieve a 10-meter improvement depth is calculated using the parameters of the dynamic compaction process, including the spacing between impact points (sh), the number of drops per point (N), the number of passes (P), the tamper weight (W), and the drop height (H). Calibration of the preliminary dynamic compaction design was carried out in trial areas (Figure 4). Following the compaction process, in-situ field tests were conducted to verify that the improved soil meets the target geotechnical parameters.

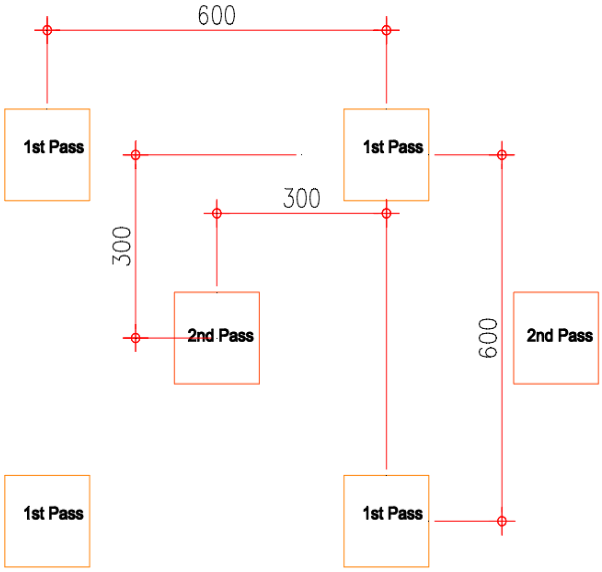

Figure 3. Dynamic Compaction Pass Pattern

Dynamic compaction is carried out using a tamper attached to a crane, which is repeatedly dropped from a certain height onto the same point to densify the soil (Figure 5). As a result of these drops, craters are formed due to ground settlement (Figure 6).

Figure 4. Dynamic Compaction - Site Photo

When the crater depth exceeds the tamper height by more than 30 cm, the craters are filled with gravel and the compaction process continues. After the compaction of specific areas is completed, ironing passes are performed, marking the completion of the dynamic compaction process (Figure 7).

Figure 5. Crane Equipped with a Tamper Attachment

Figure 6. Crater Formed During Dynamic Compaction

Figure 7. Ironing After Gravel Filling

4 CALIBRATION OF DYNAMIC COMPACTION

Trial calibration studies were conducted in four separate zones across the site. Each calibration zone consisted of 16 dynamic compaction points. The drop height was kept constant, while the number of blows was varied between points to determine the optimal compaction energy. The number of blows applied in each calibration zone is summarized in Table 2.

Table 2. Dynamic Compaction Calibration Table

Trial Zone	Drop Height (m)	Number of Blows
1	23	10
2	23	15
3	23	8
4	23	14

For the determined average SPT-N values, the target elastic modulus of the improved ground was estimated as 22,500 kN/m², based on well-established empirical correlations in the literature. To verify whether this level of improvement had been achieved, area load tests were performed within the calibration zones. Using Plaxis 3D, a settlement criterion of 10 mm was defined corresponding to the target modulus. The field test results demonstrated that the measured settlements remained below 10 mm in all calibration zones, confirming that the required improvement had been successfully attained. The settlement values recorded during the field loading tests (Figure 8) are summarized in Table 3.

Table 3. Dynamic Compaction Calibration Results

Trial Zone	Number of Blows	Measured Settlement (mm)
1	10	6.93
2	15	5.54
3	8	7.11
4	14	5.44

Figure 8. Area Loading Test

Considering the large construction area and the variability of the soil profile, the use of 14 drops was identified as the most optimized and practical solution.

Additionally, CPT and SPT tests were performed in the calibration areas both before and after improvement. For the liquefiable soil layers, it was observed that post-treatment SPT and CPT values increased to levels sufficient to eliminate liquefaction susceptibility. In these saturated sand layers, SPT-N values increased to approximately 30, while cone resistance (qc) values exceeded 9 MPa.

Following shallow excavations for foundation construction, the compacted fill layers formed by the dynamic compaction process were exposed, as shown in Figure 9.

Figure 9. Densified Layers Observed in Excavation After Dynamic Compaction

5 ASSESMENT OF LIQUEFACTION RISK

A total of 36 boreholes were conducted on site, and SPT-based liquefaction assessment was performed using the methodology proposed by Idriss and Boulanger (2010). Based on the computed factor of safety (FS) values, the Ishihara-inspired Liquefaction Potential Index (LPI$_{ish}$), developed by Maurer et al. (2015), was evaluated by taking into account the presence of a crust layer.

The Liquefaction Potential Index (LPI) assumes that each liquefiable soil layer contributes to the potential for surface damage. The shallower and/or thicker these layers are, the greater their contribution to surface manifestations of liquefaction, compared to deeper layers. It is generally assumed that liquefied layers below 20 meters do not contribute significantly to surface expressions of liquefaction.

$$LPI_{ish} = \int_{H_1}^{20\,m} F_{LPI_{ish}}(FS)\,\frac{25.56}{z}\,dz \qquad (2)$$

$$F_{LPI_{ish}}(FS) = 1 - FS \text{ Eğer } FS \leq 1 \text{ ve } H_1.m(FS) \leq 3 \qquad (3)$$

$$F_{LPI_{ish}}(FS) = 0 \text{ If } FS > 1 \text{ ve } H_1.m(FS) > 3 \qquad (4)$$

$$m(FS) = exp\left\{\frac{5}{25.56.(1-FS)}\right\} - 1 \text{ If } FS \leq 0.95 \qquad (5)$$

$$m(FS) = 100 \text{ If } FS > 0.95 \qquad (6)$$

The optimal LPI$_{ish}$ thresholds corresponding to varying severities of surficial liquefaction manifestations depend on the liquefaction triggering procedure used to calculate the factor of safety (FS) and the soil profile characteristics. Within the LPI$_{ish}$ framework applied in this study, the following interpretations are used:

- LPI$_{ish}$ < 5: Predicts none to minor surficial liquefaction manifestations.
- LPI$_{ish}$ > 15: Predicts severe surficial liquefaction manifestations.

Figures 10 and 11 illustrate the pre- and post-improvement LPI$_{ish}$ analysis results for all 36 boreholes.

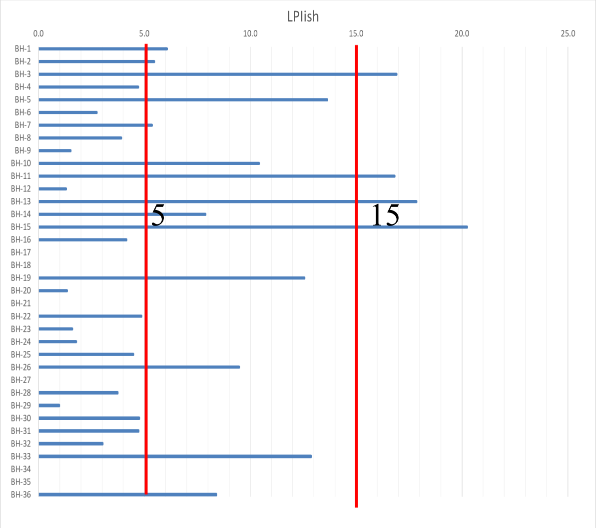

Figure 10. Pre-improvement LPI$_{ish}$ analyses

Prior to ground improvement, LPI$_{ish}$ analyses indicated that moderate to severe surficial liquefaction manifestations were expected in certain areas (Figure 10). To mitigate the occurrence of such surface manifestations, the dynamic compaction method was selected as the preferred ground improvement technique.

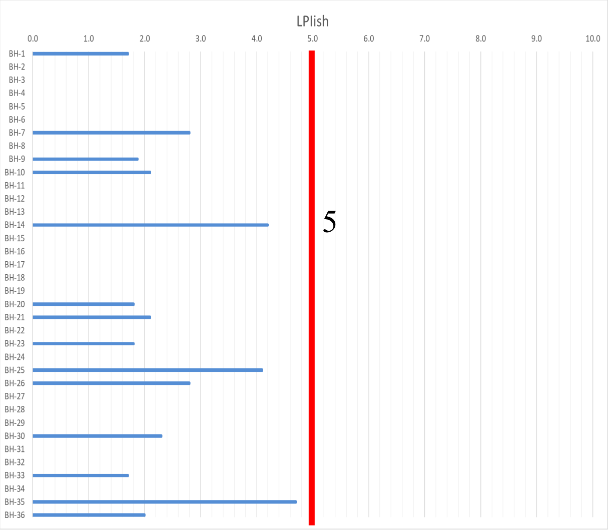

Figure 11. LPI_{ish} analyses after post-improvement

Following dynamic compaction, the formation of a 10-meter-thick crust layer resulted in LPI_{ish} values falling below 5 across all boreholes. Consequently, none to minor surficial liquefaction manifestations are predicted (Figure 11).

6 GROUND IMPROVEMENT BENEATH STRIP FOUNDATIONS

In order to both limit settlements and eliminate liquefaction risk beneath the continuous footings, jet grouting was applied down to a depth of 20 meters from the surface (Figure 12). Target diameter was determined as 800 mm. The width of the strip foundation varies between 3 to 5 meters. To prevent damage to the jet grout columns, dynamic compaction was carried out first to improve the ground, followed by jet grouting.

Trial columns were initially constructed on site. During the first set of trials, sufficient excavation was performed around the columns for diameter verification, and it was observed that the target diameter was not achieved. This was attributed to increased SPT values in the upper layers following dynamic compaction, which resulted in higher ground strength and subsequently greater energy requirements for achieving the target jet grout diameter.

Following this, revised jet grouting parameters were established, and additional trial columns were produced. The new trials successfully achieved the target diameter, and full-scale production commenced accordingly (Figure 13).

Figure 12. Jet Grout Application

Figure 13. Diameter Measurements of Jet Grout Columns

Following the jet grouting works, full-length core samples were collected from the jet grout columns, and additional core samples were taken from the column heads. The unconfined compressive strength (UCS) tests were conducted on the samples. The UCS values were over the limit value (1.5 MPa). A sample core box is presented in Figure 14.

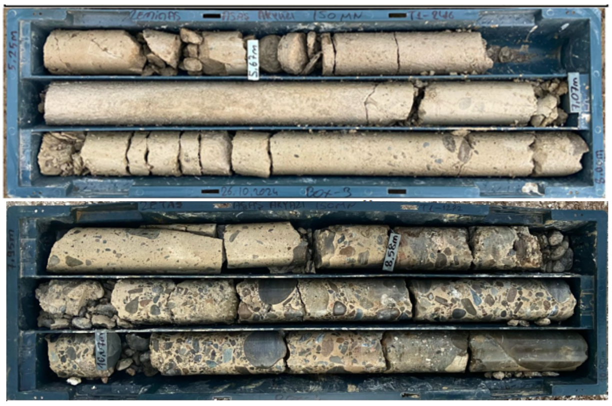

Figure 14. The Core Boxes Collected from Jet Grout Columns

Load tests were conducted on project jet grouts at 1.5 times the working load. The measured settlements are presented in Table 5.

Table 4. Measured Settlements in Jet grout Column Loading Tests

	Settlements (mm)					
	T2	T3	T4	T5	T7	T8
Working Load	*4.22*	3.64	2.00	2.67	2.15	1.33
1.5xWorking Load	*5.05*	7.76	4.15	4.75	2.77	4.08

One of the load settlement curves is given as an example in Figure 15.

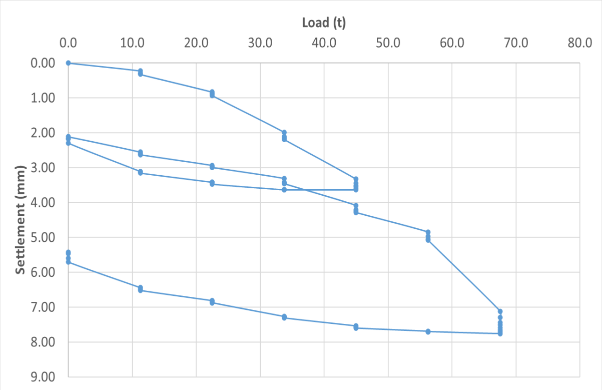

Figure 15. Example Load–Settlement Curve of Jet Grout Column Load Test

To assess the formation quality of the jet grout columns, non-destructive pile integrity tests were performed.

All on-site applications have been successfully completed and have met the requirements of all quality control tests. Figure 16 presents the foundation construction and the erection of the factory building.

Figure 16. Completion of Foundation Construction and Construction of The Factory Building Following the Soil Improvement

7 SHORING APPLICATION FOR RAFT FOUNDATION

The foundation is located approximately 6 meters below the ground surface, and the groundwater level is relatively shallow. Due to these conditions, a temporary shoring system was deemed necessary to ensure the stability of the excavation. A shoring system was proposed with jet grout columns and steel tubes installed inside predrilled and grouted boreholes (Figure 17). This application provided an impermeable solution and effectively minimized groundwater inflow into the excavation area, thereby enhancing the overall safety.

Figure 17. Shoring System with Jet Grout Columns and Steel Tubes

8 CONCLUSIONS

Given the large footprint of the factory project, the feasibility of various ground improvement techniques was assessed during the planning stage. The evaluation of the soil conditions indicated that the site was well-suited for dynamic compaction. However, due to the limited depth of influence achievable with this method, dynamic compaction was supplemented with jet grout columns beneath the foundations to effectively control settlements.

To mitigate surface manifestations of liquefaction, it was aimed to form a 10-meter-thick non-liquefiable crust layer across the entire site through dynamic compaction. Accordingly, dynamic compaction was applied throughout the project area, while jet grouting was confined to zones directly beneath structural foundations where additional settlement control was required.

This combined ground improvement strategy significantly reduced overall construction time and yielded substantial cost savings. The project clearly highlights the role of value engineering through the optimization of both schedule and cost while maintaining the required geotechnical performance.

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