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# Application of Dynamic Replacement Soil Improvement Method to an Early Site Preparation Works Project in Saudi Arabia

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## Abstract

Early site preparation works were required for a big capital project site located close to the Arabian Gulf Coast in the Eastern Province of Saudi Arabia. The ground conditions at the delineated area at site comprise 5 to 6 m thick of very loose Silty Sand interbedded with Silt & Soft Clay, overlying medium dense to very dense silty Sand. The scope of the early site works within the delineated area (approx. 1.1 million sq.m) on site was limited to soil improvement by applying the Dynamic Replacement (DR) soil improvement due to the very high fines content of upper soils and the unsuitability of utilizing conventional mechanical compaction, e.g. Dynamic Compaction (DC) & Rapid Dynamic Compaction (RDC). The DR method was utilized to reduce the compressibility, improve the density and increase the bearing capacity of the underlying soils prior to the application of approx. 2 m thick fill across the site to reach desired final grade level. Pre and Post CPTs were undertaken at the site for initial calibration trial and post quality assurance of the main DR works. The results of the post quality tests show significant improvement of the soil, and the acceptance criteria of 100 kPa for a footing of 3 m by 3 m set for the project were met.

## Keywords

Dynamic Replacement (DR), Dynamic Compaction (DC), Impact Compaction, Soil Improvement, Site Preparation

## 1. Introduction

Typical capital grass root oil and gas infrastructure projects in the Kingdom of Saudi Arabia (KSA) span across large land expanse requiring an early site preparation work with fill and cuts to achieve final grade level. To limit the amount of earthworks re-work in loose or soft soils formation after the top facility contractor is awarded, limited soil improvement is included in the early site preparation work scope prior to site hand-over to top facility contractor. This is to ensure that the soil bearing capacity and settlement limits are able to accommodate structures with light to medium loadings before handing over the site to main top facility contractor. Soil improvement is a crucial aspect of geotechnical engineering, particularly in regions where weak, compressible soils pose challenges for infrastructure development. Dynamic Replacement (DR) is a soil improvement technique that combines aspects of Dynamic Compaction (DC) and Stone Column installation making it effective for stabilizing weak, cohesive and organic soils. The most common ground improvement technique for densifying shallow subsurface soils across the eastern province of KSA is the Dynamic Compaction (DC) or Rapid Dynamic Compaction (RDC). The application of this methods is due to the predominant nature of the drift geology across this area which is mainly Dune Sand, underlain by flat lying sedimentary rocks of continental origin, known collectively as the Hofuf Formation. However, there are many parchments of saline flats (Sabkha) spread across this area that cannot be improved by mechanical compaction means and will therefore require either excavation and replacement of material for shallow deposits or inclusion type soil improvement methods such as stone columns or rigid inclusions for deeper depth extent. In this case study, the Dynamic Replacement (DR) method of ground improvement which is a modification of the conventional DC method was used for the improvement of the shallow subsurface fine-grained and Sabkha soils encountered at the site due to the speed and schedule benefit in deploying the DR method in comparison to other inclusion type of soil improvement.

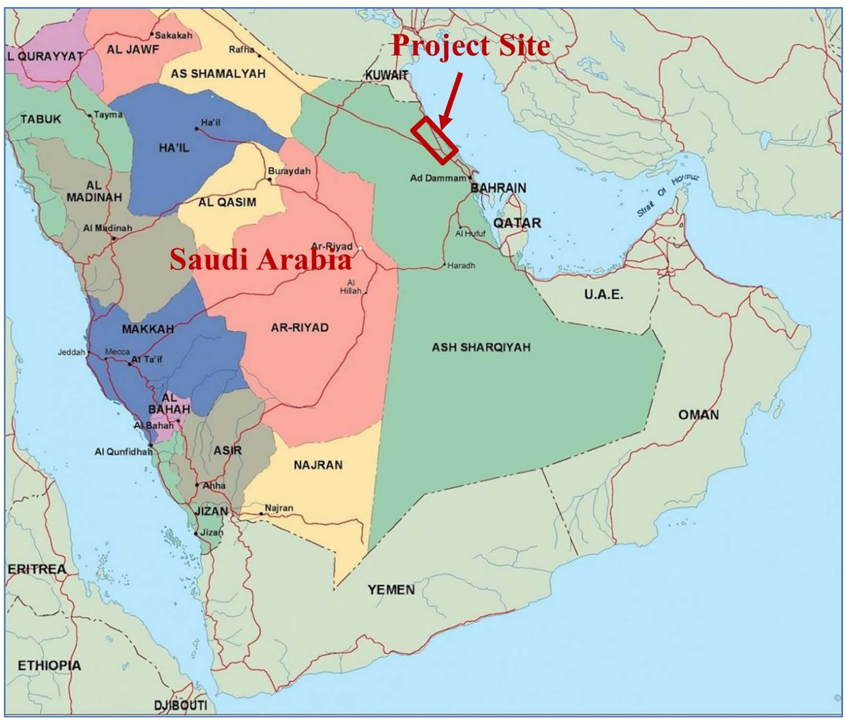



**Figure 1.** Project site location (Adapted from [22]).

The study site considered in this paper is a big capital project site located close to the Arabian Gulf Coast in the Eastern Province of Saudi Arabia. The total area of the site is approx. 3,310,000 $m^2$ with estimated total volume of earthworks required to achieve final grade level set at approx. 9,280,000 $m^3$ (8,980,000 $m^3$ of fill and 300,000 $m^3$ of cut). The ground conditions at some delineated areas at site comprise 5 to 6 m thick of very loose Silty Sand interbedded with Silt & Soft Clay, overlying medium dense to very dense silty Sand. These areas were treated with DR soil improvement method prior to fill and immediately after cut, due to very high fines content of upper soils and the unsuitability of utilizing conventional DC or RDC mechanical compaction method.

The objective of this case study is to detail the general field application of the DR technique, analyze its effectiveness and post quality assurance strategy in meeting project acceptance criteria for an early site preparation project (**Figure 1**).

## 2. Overview of Dynamic Replacement (DR) Soil Improvement Technique

DR is a ground improvement technique that combines dynamic compaction with the use of granular inclusions to enhance the bearing capacity and settlement characteristics of weak soils such as soft clay, silt, and peat. This method is particularly effective for improving loose or organic soils, where traditional dynamic compaction alone may not be feasible [1]. This method involves the systematic dropping of heavy tamping weights from considerable heights to drive coarse granular materials into weak soils, forming stiff, high-capacity columns thereby creating craters, which are in turn backfilled with granular material. Successive tamping cycles densify the granular columns and the surrounding soft soil, improving load-bearing capacity and reducing compressibility [2].

The DR process involves two primary mechanisms, which are:

- Displacement and Compaction: The impact energy from the falling weight displaces the weak soil laterally, creating a cavity that is subsequently filled with granular material (e.g., coarse sand, crushed stone or gravel). The repeated compaction of this material forms a dense column.
- Densification of Surrounding Soil: The dynamic energy also densifies the surrounding soil, improving its shear strength and stiffness.

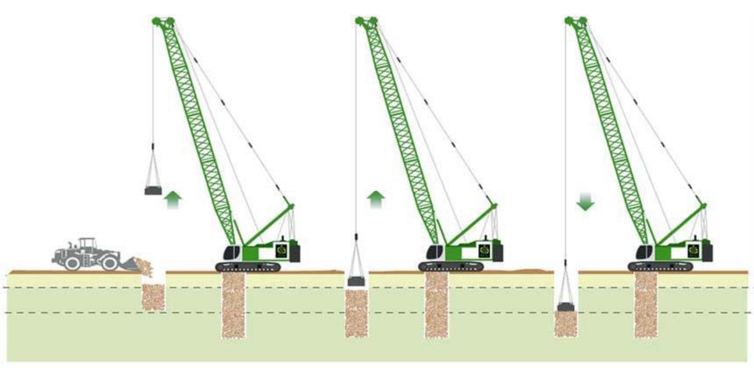


**Figure 2.** Sequence of works for dynamic replacement method [23].

The resulting granular columns act as vertical drains, accelerating consolidation in cohesive soils, and as load-bearing elements, transferring structural loads to deeper, more competent strata (**Figure 2** and **Figure 3**).

**Figure 3.** Post dynamic replacement craters after application.

DR is most suitable for weak cohesive soils with undrained shear strength (Su) between 15 - 50 kPa and organic soils with low bearing capacity [3]. The method is particularly effective for:

- Loose fills and reclaimed land.
- Soft clay and silts with high moisture content.
- Peaty soils with low shear strength.

## Typical Geotechnical Applications of DR

The dynamic replacement method is used in many Civil Engineering projects, particularly in cases where traditional foundation methods are not effective due to weak or unstable soils.

Some common applications include:

1) Foundation Improvement

- Buildings and High-Rises: DRM is commonly used to improve the foundation soil for large-scale construction, particularly in urban areas with soft or saturated soils.
- Infrastructure Projects: For roads, bridges, and railways, DRM is used to stabilize weak soils and provide a stronger foundation for construction.
- Research indicates successful applications in tank farms and warehouse foundations with improved load-bearing capacity [4].

2) Seismic Liquefaction Mitigation

In areas susceptible to liquefaction (where saturated soils lose their strength during earthquakes), DRM can be employed to enhance soil stability. The dynamic compaction of granular materials reduces the risk of soil liquefaction by densifying the soil layers.

3) Marine and Waterfront Projects

In marine construction projects such as docks and ports, dynamic replacement is used to improve the soft, unconsolidated sediments typically found in these areas.

## 3. Past Research and Case Studies on DR Application

Reference [5] analyzed the impact of dynamic replacement on soil stiffness and settlement behavior, providing evidence for its effectiveness in improving the strength of soft soils. Reference [6] also reported improved stability and reduced settlements. Furthermore, [7] investigated the influence of DR on surrounding soils, finding that the technique effectively increased the strength and stiffness of both the treated columns and adjacent soil matrix, leading to improved overall ground performance. Reference [8] in their research compared dynamic replacement with other ground improvement methods (such as stone columns and deep mixing) and highlighted its cost-effectiveness and suitability. **Table 1** below shows the generic comparison between DR/DC and inclusion type methods, while **Table 2** provides a detailed summary, comparison & associated approximate cost of major soil improvement techniques that are typically deployed.

**Table 1.** DR comparison with other common methods [8].

| Method | Bearing Capacity Improvement | Settlement Reduction | Cost | Suitability for Weak soils |
|---|---|---|---|---|
| Dynamic Replacement | High | High | Moderate | Excellent |
| Stone Columns | Moderate | Moderate | High | Good |
| Deep Mixing | Very High | Very High | Very High | Excellent |
| Vibro-Compaction | Low | Low | Moderate | Poor |

Reference [9] explored the feasibility of DR in deep water conditions, conducting a full-scale trial in Southeast Asia at a seabed depth of 30 meters. The study confirmed the potential of DR for offshore ground improvement, expanding its applicability to marine environments. However, this application is yet to be verified in an actual project. In some cases, DR is used in combination with other soil improvement methods, such as vibro compaction or grouting. Reference [10] combined DR with other methods, such as geosynthetic reinforcement, to enhance performance in challenging soil conditions, likewise, [11] investigated the use of recycled materials as granular fill to reduce the environmental impact of DR. Hybrid approach may provide additional benefits particularly in challenging soil conditions.

**Table 2.** Cost comparison of major soil improvement techniques (Adapted from [24]).

| Soil Improvement Technique/Method | Soil Type | Effective Depth Range | Limitations & Disadvantages | Material Sourcing | Price Assessment – Global Estimate | Production Rate |
|---|---|---|---|---|---|---|
| Removal and replacement | Mainly very soft clay and peat | Up to 30 m | Time consuming, environmental implications | Select or general fill material is required for the replacement | **25 - 120 USD/m$^3$** (price vary per region) | **100 - 500 m$^3$/day** Using a backhoe, large excavator & dump truck and compactor |
| Pre-loading | Mainly soft clay and peat | 30 m to 60 m | Lack of control. Not conventional and risky in sandy/silty material. Large amount of fill material required to apply surcharge load | Fill material is required. For less consolidation time both fill material & Prefabricated Vertical Drain (PVD) are required | **10 - 30 USD/m$^2$** (without PVD) **25 - 80 USD/m$^2$** (with PVD) | **Fill placement**: 200 - 500 m$^3$/day **PVD Installation**: 2,000 - 10,000 m/day **Pre-loading duration**: Without PVDs – 6 - 18 months With PVDs – 3 - 9 months |
| Roller Compaction | Cohensionless | 0 m to 3 m | Mainly for engineered fill. | Select or general fill material as per project specification | **1.5 - 6 USD/m$^2$**. Includes roller and operator & maintenance | **6000 – 15000 m$^2$/day** Depends heavily on roller type, working width, lift thickness & number of passes |
| Rapid Impact Compaction (RIC) | Cohensionless | 2 m to 4 m | Maximum 1.5 m thick layers | Fill required for working platform | **7.5 USD/m$^2$** | **15000 m$^2$/month/rig** 1 rig for 300 m$^2$ per shift with 2 shifts per day and 25 days per month |
| Dynamic Compaction (DC) | Cohensionless | 5 m to 7 m | Not adapted for natural soil with high variability and high fines content | Fill required for working platform | **16 USD/m$^2$**. Approximately 4 USD/m$^2$ may be required for additional material due to subsidence | **25000 m$^2$/month/rig** 1 rig for 500 m$^2$ per shift with 2 shifts per day and 25 days per month |
| Dynamic Replacement (DR) | Cohesive | 0 m to 8 m | 1.5 m above water level. | Fill required for working platform. Granular material for soil plug required. | **20 USD/m$^2$**. Approximately 4 USD/m$^2$ may be required for additional material due to subsidence | |
| Vibro Compaction (VC) | Cohensionless | 10 m to 30 m | Strict criteria on fines content (less than 10%to 15%) | Material required for working platform | **19 USD/m$^2$**. Includes re-compaction of the surface by roller compaction | **12000 m$^2$/month/rig** 1 rig for 240 m$^2$ per shift with 2 shifts per day and 25 days per month |
| Vibro Replacement (VR) | All | 10 m to 20 m | Need for crushed stones or gravel | Crushed stones as reinforcement is required. Volume of stones estimated at 0.6 to 0.7 m$^3$/m$^2$ | **55 USD/m$^2$**. Includes cost and transport of stones at approx. 23 USD/m$^2$ | |
| Soil Mixing | All | Shallow method (<12 m). Deep method (3 m to 50 m) | Need for admixture for general soils, and binders suitable for very soft soils. | Cement to be sourced. Quantity estimated at 1 ton/m$^2$ | **180 USD/m$^2$**. Includes cost of cement at 100 USD/ton and execution cost at 80 USD/m$^2$ | **5000 m$^2$/month/rig** 1 rig for 600 m$^3$ per shift Approx. 1 rig for 100 m$^2$ per shift with 2 shifts per day and 25 days per month |
| Rigid Inclusion /Concrete Columns | All | 5 m to 30 m | Need for concrete and very stiff load transfer platform with sufficient thickness due to high stresses from column | Concrete to be sourced, volume of concrete estimated to be about 0.3 m$^3$/m$^2$ (based on 6 m column length). Volume of stones for LTP estimated at 0.5 to 0.6 m$^3$/m$^2$ | **95 USD/m$^2$**. Includes cost of 0.5 m thick crushed stone LTP | **8000 m$^2$/month/rig** 1 rig for 160 m$^2$ per shift with 2 shifts per day and 25 days per month Time for placement of 0.5 m thick LTP shall be added |

A number of studies have demonstrated the effectiveness of DR in various contexts, some of which are highlighted below:

- Reference [12] reported on the simultaneous use of DR and Rapid Impact Compaction (RIC) to improve granular fill on a 45,000 m$^2$ artificial island near Ras Al-Khaimah in the United Arab Emirates (UAE). Field tests, including cone penetration tests (CPT), indicated significant enhancements in soil properties, achieving the desired improvement within the influence zone of planned structures.
- At the Port of Gothenburg, Sweden, DR was employed to mitigate liquefaction risk in loose, sandy soils, with post-treatment testing showing a substantial increase in soil density and shear strength [13].
- DR was used to improve the bearing capacity of soft clay for runway construction at Embakasi Airport, Kenya, resulting in significant settlement reduction and increased stability [3].
- Reference [14] reported the use of DR to stabilize soft soils beneath highway embankments in China which achieved the required bearing capacity and reduction in post-construction settlement.

DR is a versatile and effective ground improvement technique that has been successfully applied in a wide range of geotechnical projects. Its ability to rapidly improve cohesive (silt and clay) soil properties, combined with its cost-effectiveness, makes it a popular choice for engineers in such soil conditions. However, careful design, execution, and quality control are essential to ensure its success, particularly in complex soil conditions.

# 4. Early Site Preparation Study Site

## 4.1. Site Conditions

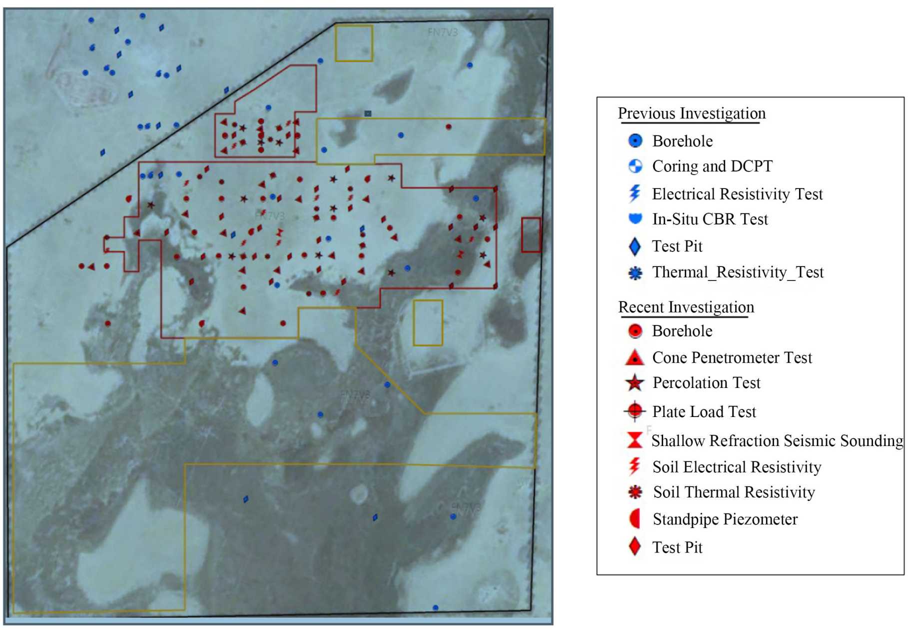



**Figure 4.** Site layout of field tests [24].

Figure 4 shows the investigated site extent with preliminary field test layout. The encountered ground conditions comprise very loose to loose silty to very silty SAND (up to 6 m thick in some areas) underlain by medium dense to very dense silty to very silty SAND till 10.0 m below ground level. Shallow and surface ground water at lower levelled ground were encountered at the site due to considerable variation in ground elevation at the site (from 9 m to 2 m elevation). Groundwater was encountered at depths between 0.5 m and 4 m below ground level. Sabkha (saline flats) were encountered across the site and easily identified by its characteristic grey and brown colour with varying shades depending upon the proportion of the clay, silt and sand present and proximity of the groundwater table below top of the Sabkha surface.

### 4.2. Early Site Works Screening Criteria


Figure 5. Early work screening of a segment on site with pre-CPT boxes [23].

The project requires soil improvement works by using combination of several soil improvement techniques including but not limited to: Surface/Roller Compaction, Dynamic Compaction (DC), Dynamic Replacement (DR) and Rapid Impact Compaction (RIC) techniques. In order to select the appropriate soil improvement design methodology, an extensive pre-CPTs (prior to soil improvement) campaign following a frequency of 1 PCPT per 1,000 $m^2$ was adopted across the site to delineate areas according to applicable methods. This required dividing the site into activity boxes of 1,000 $m^2$ each with each having its unique identity as illustrated in Figure 5 below for different improvement methods. The Pre-CPTs will be required for the following:

- Performing the liquefaction analysis and determine the depth of liquefiable layer (if any).
- Evaluating the Pre-CPTs results, determine the loose layer depth and check the presence of any silty material (SBT Ic > 2.05) within the target depth of improvement.

- Conclude the target depth of improvement (maximum depth between loose layer and liquefiable layer).
- Define the soil improvement technique's distribution over the area by considering all the following factors:
  - Target Depth of improvement (DOI) and type of material within this DOI.
  - Capability of the soil improvement technique to meet the project's requirements in line with the type of material.

### 4.3. Project Acceptance Criteria

The project acceptance criteria are target bearing Capacity of 100 kPa for a footing of 3 m × 3 m with embedment depth between 1.5 m to 2 m below rough grade level (RGL). Maximum allowable settlement of 25 mm. Also, minimum Factor of Safety (FOS) of 1.0 against liquefaction (M = 5.5, Pga = 0.096 g after amplification for Site Class D).

## 5. Post Quality Test and Analysis Methodology

Post testing by CPTs is required after DR soil improvement works to verify achievement of project criteria where needed. For each activity box, a pair of CPT test was conducted, one test between the DR grids, and the other within the DR print. To assess the relative improvement from the DR technique, soil homogenization concept illustrated in Figure 6 was adopted in calculations after performing the post-CPTs. The concept is based on the replacement ratio (reflecting the percentage of the original area replaced with DR pillars). The replacement ratio is defined as below:

$$\rho = A_p / A$$

where:

- $\rho$ = Replacement ratio;
- $A_p$ = Cross Sectional area of Dynamic Replacement pillar;
- $A$ = Cross-sectional area, *i.e.* in case 5 m × 5 m square grid selected $A$ will be 25 m$^2$.

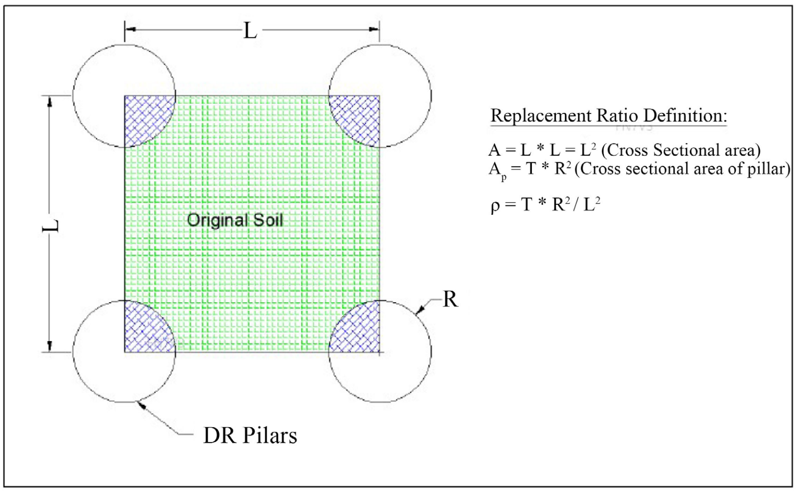



Figure 6. Homogenization concept [23].

The composite deformation modulus and tip resistance (from original soil and DR Pillars CPT tests) are estimated from the equations presented below:

- $E_c = \rho * E_p + (1-\rho) * E_{soil}$
- $(Q_c)_c = \rho * (Q_c)_p + (1-\rho) * (Q_c)_{soil}$
- $(R_f)_c = \rho * (R_f)_p + (1-\rho) * (R_f)_{soil}$

where:

- $E_c$ = Composite Deformation modulus.
- $E_p$ = Deformation modulus within the pillar.
- $E_{soil}$ = Deformation modulus within the soil between the pillars.
- $(Q_c)_c$ = Composite Cone Tip Resistance.
- $(Q_c)_p$ = Cone Resistance within the pillar.
- $(Q_c)_{soil}$ = Cone Resistance within the soil between the pillars.
- $(R_f)_p$ = Friction ratio within the pillar.
- $(R_f)_{soil}$ = Friction ratio within the soil between the pillars.
- $\rho$ = Replacement ratio.

Based on the above, composite soil properties (from the original soil and DR Pillars) were estimated from the CPT tests undertaken at each activity box by using equations presented above. Young Modulus value was calculated by [15]:

$$E_s = \alpha_E \cdot \left(qt_{avg}^{rm} - \sigma_{v0}\right)$$

where:

$\alpha_E$ = is a coefficient depending on soil behavior.

$qt_{avg}^{rm}$ = is the rolling mean average tip resistance at each depth corrected for total overburden pressure.

$\sigma_{v0}$ = is the total overburden pressure.

## 5.1. Bearing Capacity

The net allowable bearing capacity was calculated in accordance with the methodology proposed by [16] and presented in [17] which relies on the direct approach to estimate the bearing capacity based on CPT results for granular soils.

Net allowable bearing capacity is estimated for a pad footing of 3.0 m by 3.0 m embedded 1.5 m 2.0 m below the finished level with the factor of safety of three by the following equation;

$$q_{ult\ net} = SCF \cdot k_{\varnothing} \cdot qt_{avg}^{rm}$$

where:

$k_{\varnothing}$ = coefficient assumed depending on the ratio between width of footing and depth of embedment as well as shape of footing (conservative value of 0.16 will be used in calculations).

$qt_{avg}^{rm}$ = rolling mean average tip resistance at the concerned depth considering shell correction factor (SCF) = 1.0.

$$q_t = q_c + u_2(1-a)$$

where:

$q_c$ = measured tip resistance at the concerned depth.

$u_2$ = water pore pressure measured behind the cone at the concerned depth.

$a$ = net area ratio determined from laboratory calibration with a value of 0.8.

$$q_{all\ net} = \frac{q_{ult}}{FoS}$$

$q_{ult}$ = ultimate bearing capacity.

$FoS$ = factor of safety (3).

## 5.2. Settlement

Static settlement was determined according to [18] approach for granular soils as presented in [17], using footing size of 3.0 m by 3.0 m embedded 1.5 m - 2.0 m below the finished grade level and maximum applied pressure of 100 and 250 kPa. A typical strain influence factor for a foundation width of B is presented in Figure 7 below.

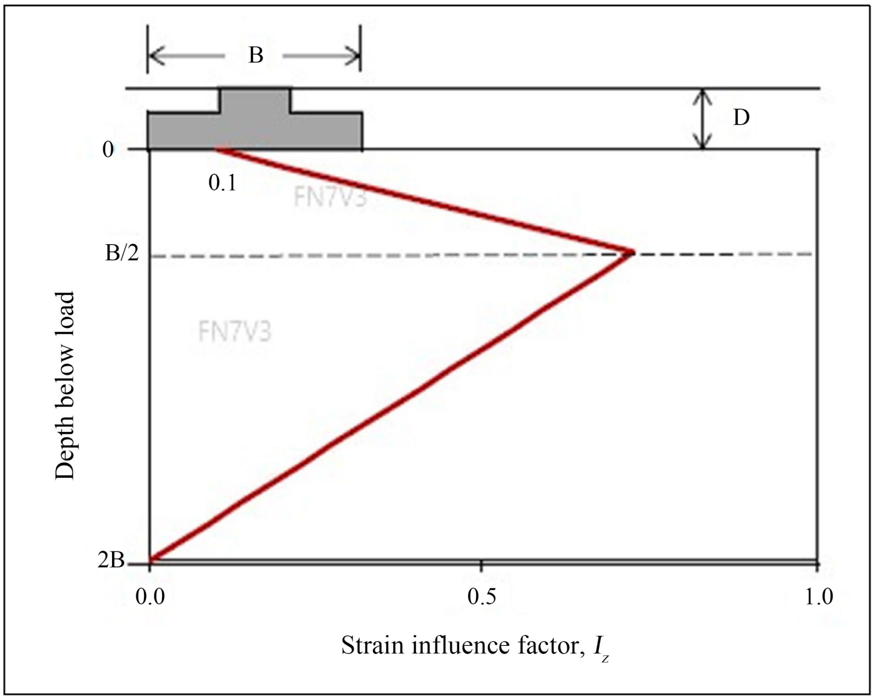



Figure 7. Strain influence factors from Schmertmann [18].

The static settlement will be calculated by the following equation:

$$S_{static,total} = C_1 C_2 \left(q - \sigma'_{v0}\right) \int_0^z \frac{I_z}{C_3 E_s} \mathrm{d}z$$

where:

$C_1$ = depth correction factor.

$$C_1 = 1 - 0.5\left(\frac{\sigma'_{v0}}{q - \sigma'_{v0}}\right)$$

$\sigma'_{v0}$ = effective vertical stress at foundation level.

$q$ = foundation load pressure.

$C_2$ = creep correction factor.

$$C_2 = 1 + 0.2\log\left(\frac{t}{0.1}\right)$$

$t$ = time in years.

$C_3$ = correction factor for shape of footing.

$I_z$ = vertical strain influence factor. Maximum vertical strain influence factor is defined within the depth of influence of the applied foundation load.

$$I_{z,\max} = 0.5 + 0.1\sqrt{\frac{q - \sigma'_{v0}}{\sigma'_{vz\max}}}$$

$\sigma'_{vz\max}$ = effective vertical stress at level z = B/2.

### 5.3. Liquefaction

Liquefaction analysis was based on the methodology described in "Liquefaction Resistance of Soils: Summary Report from 1996 NCEER & 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils" by [19].

The analysis is based on the calculation of Cyclic Resistance Ratio (CRR) which expresses the soil strength and Cyclic Strength Ratio (CSR) which expresses the induced seismic load. These values are used to calculate the factor of safety against liquefaction. The soil behavior type index *Ic* is used to assess the liquefaction potential of certain soil/sub-layer in the soil profile. The factor of safety against liquefaction (FS) defined as follows:

$$\text{FS} = \frac{\text{MSF} \cdot \text{CRR}_{7.5}}{\text{CSR}}$$

Magnitude Scaling Factor (MSF) to be taken into account based on the method recommended by the NCEER [20]. The Cyclic Stress Ratio (CSR) is the seismic demand of the soil layer and is calculated by normalizing the above equivalent shear stress with the initial effective overburden pressure:

$$\text{CSR} = 0.65 \cdot \alpha_{\max} \cdot \frac{\sigma_{v0}}{\sigma'_{v0}} \cdot r_d$$

where:

$\alpha_{\max}$ = maximum ground acceleration at the surface.

$\sigma_{v0}$ = total vertical stress.

$\sigma'_{v0}$ = effective vertical stress.

$r_d$ = depth reduction factor.

Depth reduction factor ( $r_d$ ) is calculated as recommended by NCEER:

$$r_d = \frac{1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}}{1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2}$$

where:

$z$ = depth.

The Cyclic Resistance Ratio for a magnitude of 7.5 can be related to the normalized and corrected CPT cone tip resistance. The modified flow-chart procedure proposed by [21] was used, and it is based on the recommendations and method proposed in the NCEER. According to the flow-chart procedure proposed by [21], soils with $Ic$ greater or equal to 2.6 are considered as non-liquefiable. However, based on industry experience soils with $Ic$ greater than 2.4 are non-compactible and non-liquefiable. In this case, a transition zone corresponding to 25 cm above and 25 cm below such soils were not considered in the assessment of compaction.

## 6. DR Site Trial/Calibration Works and Result

The DR trial & calibration was performed on site to confirm several parameters such as optimum grid pattern size, range of compaction energy (height of drop, number of blows) and number of passes required. Two DR grid spacings to identify the optimum DR grid for the project. The trial grids are 5.00 m × 5.00 m, and 6.00 m × 6.00 m, as illustrated in Figure 8 below.

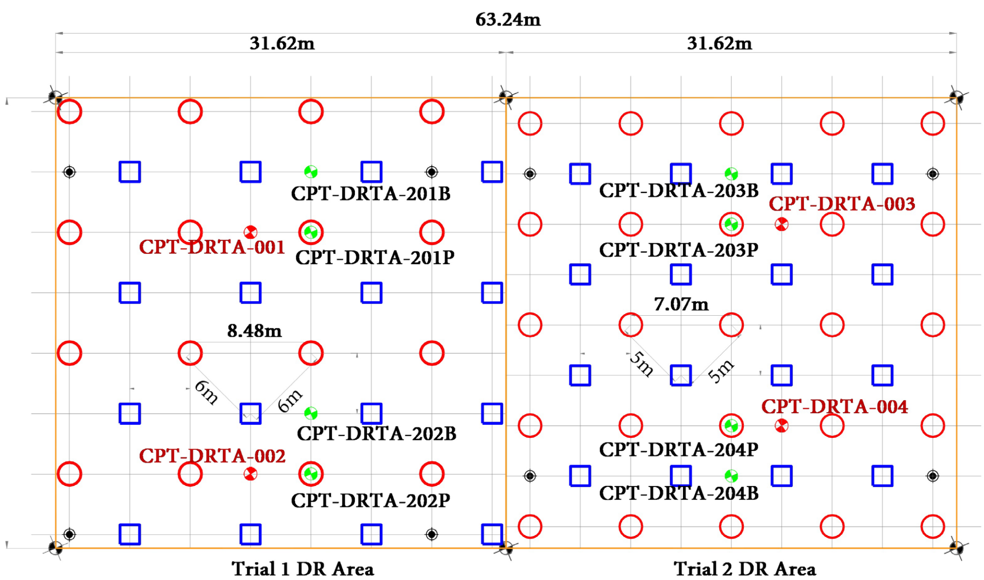



Figure 8. Grid details for DR trials [23].

Table 3. DR trial CPT tests layout.

| DR Grid Spacing | TEST 1 | | | TEST 2 | | |
|---|---|---|---|---|---|---|
| | Pre-CPT | Post CPT at Grid Center | Post CPT at DR Pillar | Pre-CPT | Post CPT at Grid Center | Post CPT at DR Pillar |
| 6.00 m × 6.00 m | CPT-DRTA-001 | CPT-DRTA-201-B | CPT-DRTA-201-P | CPT-DRTA-002 | CPT-DRTA-202-B | CPT-DRTA-202-P |
| 5.00 m × 5.00 m | CPT-DRTA-003 | CPT-DRTA-203-B | CPT-DRTA-203-P | CPT-DRTA-004 | CPT-DRTA-204-B | CPT-DRTA-204-P |

Table 3 presented below highlights the pre and post CPT tests undertaken at the trial area.

The production parameters were: One to three Passes for Phase 1 and Phase 2.

- Height of drop utilized for trial is 20 m (maximum) depending on the soil conditions.
- Number of blows for each pass will vary between 10 to 12.
- Weight of pounder between 16 to 25 tons.
- Replacement ratio of at least 15.51% is applicable

The processed pre and post CPT results, including the equivalent/composite tip resistance estimated from equations presented in **Section 5**, is provided in Figure 9 and Figure 10 below. In addition, bearing capacity, settlement and liquefaction analysis for the trial calibration works is also presented in Figure 11 and Figure 12, for Test 1 of Trial 02 (5.0 m × 5.0 m Grid). The results presented in the Figures are based on the equations provided in Sections 5.1 - 5.3.

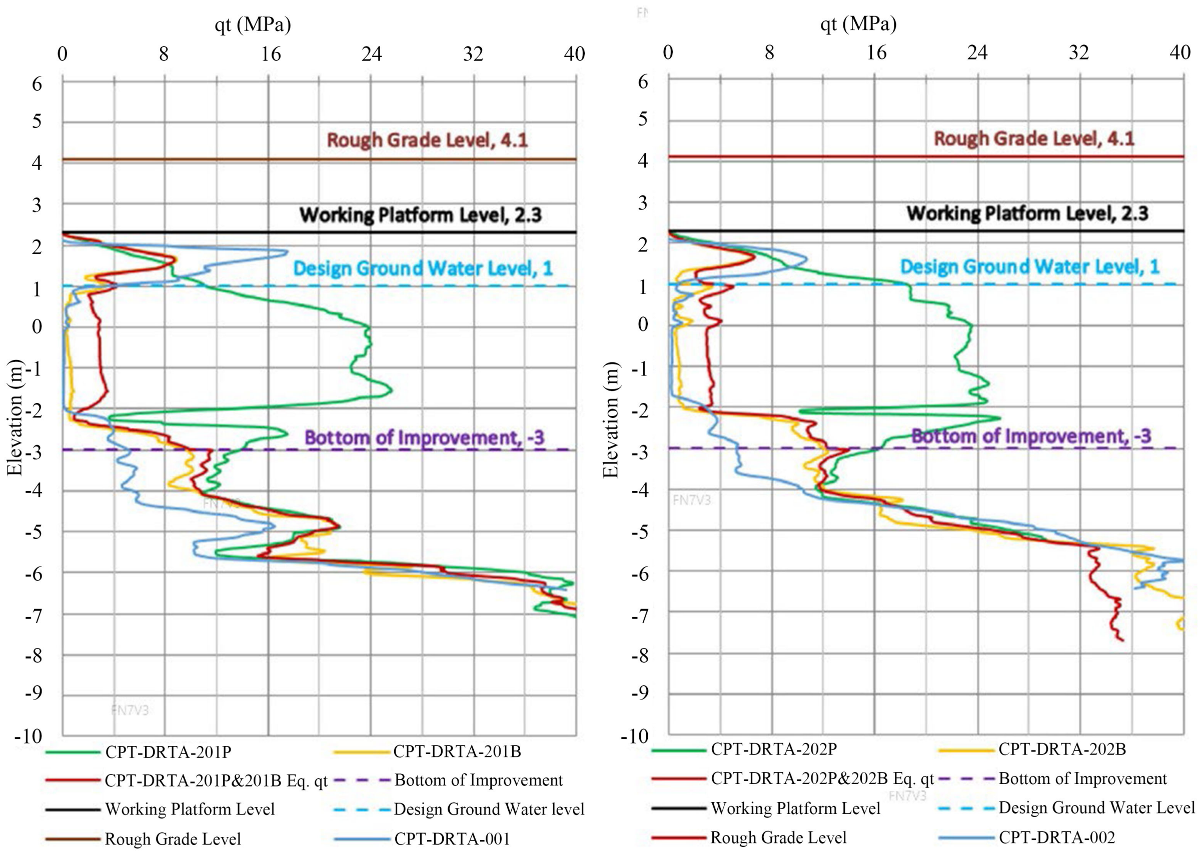



Figure 9. Processed pre & post CPT results for trial 01 (6.0 m × 6.0 m grid) [23].

Summary of the trial result is presented below in Table 4.

The bearing capacity, settlement and liquefaction assessments undertaken with processed post CPT results for both 5.0 m × 5.0 m and 6.0 m × 6.0 m grids were adequate and met project acceptance criteria. From Table 4 below, the bearing capacity and settlement obtained for the 5.0 m × 5.0 m grid is approx. 5% (slight disadvantage in bearing capacity magnitude) and 6.25% (slight advantage in settlement magnitude) less in magnitude respectively, in comparison to the 6.0 m × 6.0 m grid.

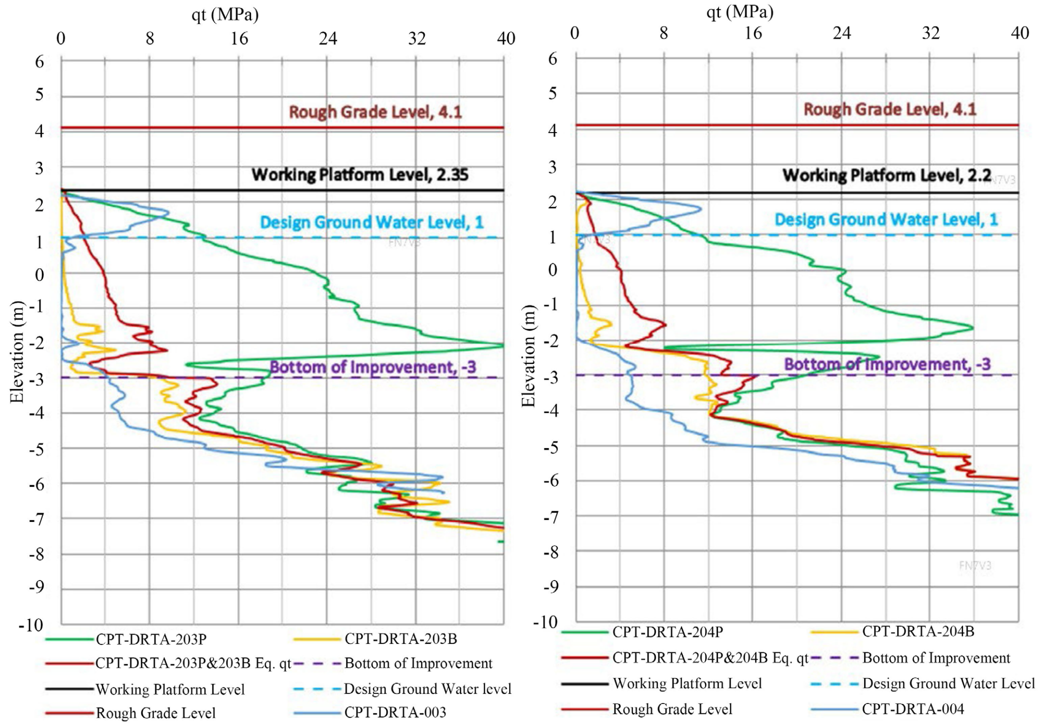
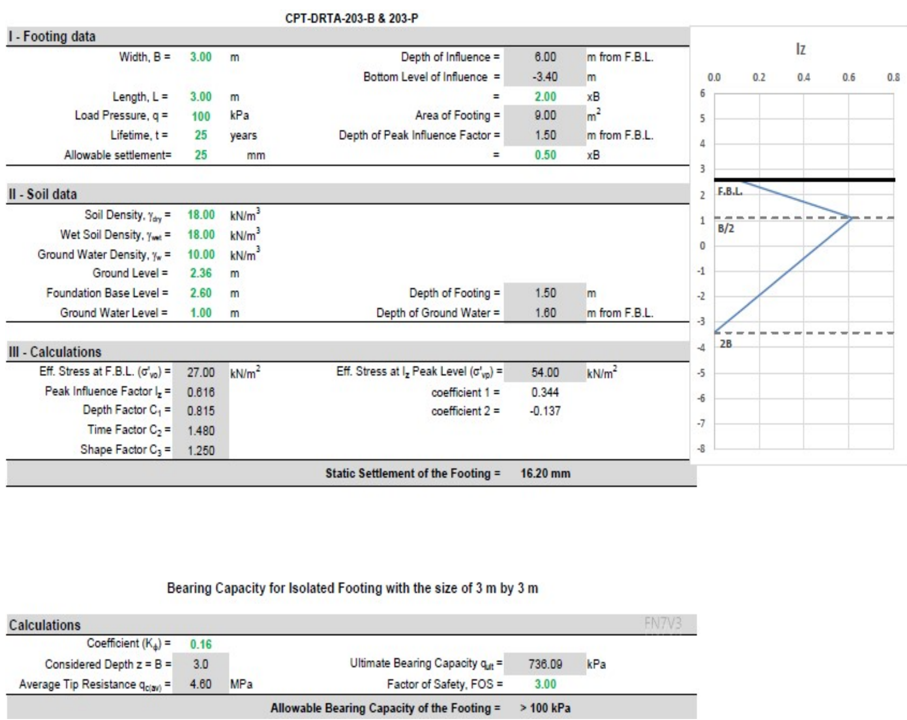



Figure 10. Processed pre & post CPT results for trial 02 (5.0 m × 5.0 m Grid) [23].



CPT-DRTA-203-B & 203-P

| I - Footing data | | | | | |
|---|---|---|---|---|---|
| Width, B = | 3.00 | m | Depth of Influence = | 6.00 | m from F.B.L. |
| | | | Bottom Level of Influence = | -3.40 | m |
| Length, L = | 3.00 | m | = | 2.00 | xB |
| Load Pressure, q = | 100 | kPa | Area of Footing = | 9.00 | $m^2$ |
| Lifetime, t = | 25 | years | Depth of Peak Influence Factor = | 1.50 | m from F.B.L. |
| Allowable settlement= | 25 | mm | = | 0.50 | xB |
| **II - Soil data** | | | | | |
| Soil Density, $\gamma_{dry}$ = | 18.00 | $kN/m^3$ | | | |
| Wet Soil Density, $\gamma_{wet}$ = | 18.00 | $kN/m^3$ | | | |
| Ground Water Density, $\gamma_w$ = | 10.00 | $kN/m^3$ | | | |
| Ground Level = | 2.36 | m | | | |
| Foundation Base Level = | 2.60 | m | Depth of Footing = | 1.50 | m |
| Ground Water Level = | 1.00 | m | Depth of Ground Water = | 1.60 | m from F.B.L. |
| **III - Calculations** | | | | | |
| Eff. Stress at F.B.L. ($\sigma'_{vo}$) = | 27.00 | $kN/m^2$ | Eff. Stress at $I_z$ Peak Level ($\sigma'_{vp}$) = | 54.00 | $kN/m^2$ |
| Peak Influence Factor $I_z$ = | 0.616 | | coefficient 1 = | 0.344 | |
| Depth Factor $C_1$ = | 0.815 | | coefficient 2 = | -0.137 | |
| Time Factor $C_2$ = | 1.480 | | | | |
| Shape Factor $C_3$ = | 1.250 | | | | |
| | | | Static Settlement of the Footing = | 16.20 mm | |



Bearing Capacity for Isolated Footing with the size of 3 m by 3 m

| Calculations | | | | | |
|---|---|---|---|---|---|
| Coefficient ($K_\phi$) = | 0.16 | | | | |
| Considered Depth z = B = | 3.0 | | Ultimate Bearing Capacity $q_{ult}$ = | 736.09 | kPa |
| Average Tip Resistance $q_{c(av)}$ = | 4.60 | MPa | Factor of Safety, FOS = | 3.00 | |
| | | | Allowable Bearing Capacity of the Footing = | > 100 kPa | |

Figure 11. Bearing capacity & static settlements for isolated footing of 3 m by 3 m size [23].

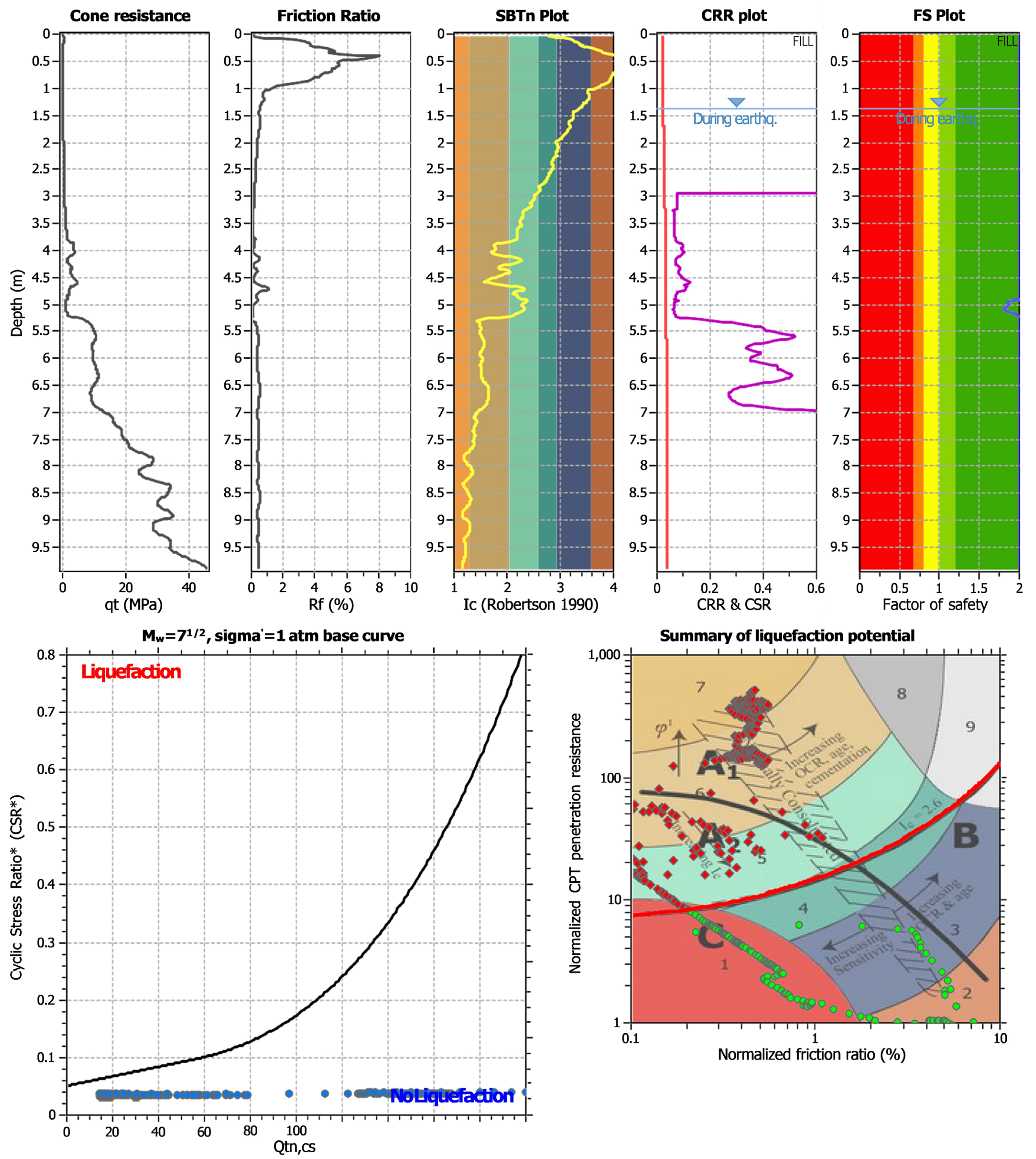



Figure 12. Input parameters and results of liquefaction potential for post CPT-DRTA-203B at center of two adjacent DR columns for test 1 of trial area 02 (5.0 m × 5.0 m Grid). analysis from CLiq v.3.3.3.2 - CPT liquefaction assessment software [23].

The weighted average of the equivalent post CPT tip resistance from CPT at DR print and between DR pillars from platform level to bottom of DR improvement (approx. 5 m to 5.5 m depth) is 5 MPa and 4 MPa for the 5.0 m × 5.0 m and 6.0 m × 6.0 m grids respectively. The reason why the results show a slightly higher bearing capacity magnitude for the 6.0 m × 6.0 m grid is the higher equivalent average CPT tip resistance of 4 MPa within the upper 1.5 m depth in comparison to the 2

MPa estimated for 5.0 m × 5.0 m. Based on the overall trial results, the 5.0 m × 5.0 m grids was conservatively adopted for the DR main production works.

**Table 4.** Settlement and bearing capacity results of post CPTs for 100 kPa (with 1 m thick Load Transfer Platform consideration).

| DR Grid Spacing | Test Reference | Rough Grade Level (RGL) (m SAVD | Footing Elevation (m SAVD) | Required Bearing Capacity (kPa) | Allowable Settlement (mm) | Post Improvement Bearing Capacity (kPa) | Post Improvement Settlement (mm) |
|---|---|---|---|---|---|---|---|
| 6 m × 6 m | CPT-DRTA-201B & CPT-DRTA-201P | +4.1 | +2.6 | 100 | 25 | 270 | 17 |
| | CPT-DRTA-202B & CPT-DRTA-202P | | | | | 260 | 16 |
| 5 m × 5 m | CPT-DRTA-203B & CPT-DRTA-203P | | | | | 245 | 16 |
| | CPT-DRTA-204B & CPT-DRTA-204P | | | | | 245 | 16 |

## 7. Conclusions

Both DR trials achieved the technical requirements of the project. The DR pillars in Trial-02 with final grid spacing of 5.0 m × 5.0 m have shown a slightly better weighted average of the equivalent post CPT tip resistance than the DR pillars with final grid spacing of 6.0 m × 6.0 m. Therefore, 5.0 m × 5.0 m grids have been conservatively adopted for the DR main production works. Consequently, and based on the results obtained from DR Trials work, the following production parameters were used for actual production works at site:

- Final grid spacing of 5.0 m × 5.0 m;
- Two to three Passes for Phase 1 and Phase 2 prints;
- Prints in Phase 1 and Phase 2 were subjected to 5 to 12 blows with drop height of 6 m to approx. 12 m with a pounder having minimum weight of 16 tons;
- Replacement ratio of at least 15.51% is applicable;
- Backfilling the craters created by Phase 1 and 2 prints via an Ironing pass with high energy impact compactor (HEIC).

A contingency plan was also put in place by the early works contractor in case a localized extensive thickness of very loose Silty Sand, Silt & Soft Clay is encountered beyond the design depth of 6 m. The contingency measure will include an additional Phase 3 prints which will involve the construction of an extra DR print between the adopted trial grids, with an approximate increase in replacement ratio to 31.03%.

## Conflicts of Interest

The authors declare no conflicts of interest regarding the publication of this paper.

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