

Application of the dynamic replacement method in geotechnical soil improvement

Tennison Freire de Souza Junior^{1#} , Ana Paula Vianna Mohseni² 

Review Article

Keywords

Dynamic replacement
Soil improvement
Civil construction
Reinforcement
Bearing capacity

Abstract

Soils with low bearing capacity and/or high compressibility are widely encountered in both onshore environments — such as urban, industrial, and transportation infrastructure areas — and offshore settings in Brazil and worldwide. This widespread occurrence requires geotechnical interventions aimed at stabilizing and improving soil parameters and properties. Various ground improvement techniques are available, ranging from reinforcement and consolidation methods to compaction and stabilization approaches. Among them, the Dynamic Replacement (DR) method stands out as an effective solution for soft soils, organic clays, and similar deposits, through the partial replacement of the soil to be improved, using principles derived from dynamic compaction but applied to soil reinforcement. Currently, in Brazil, there is a relative scarcity of information available in the literature, as well as limited practical application in geotechnical works. Therefore, the present paper aims to provide a literature review addressing the fundamental concepts of Dynamic Replacement, as well as the equipment involved, monitoring and efficiency assessment procedures, existing numerical models, and representative case studies of the technique's applications in Brazil and internationally.

1. Introduction

In geotechnical engineering, it is common to encounter a wide diversity of soil types, whose specific characteristics guide the selection of the appropriate stabilization technique to meet the requirements of civil construction. Numerous variables must be considered in both technical and economic analyses when defining the type of foundation, including soil genesis, mechanical properties, thickness, and structural characteristics, among others.

Historically, areas with soil exhibiting low bearing capacity and/or highly deformable characteristics were often deemed unsuitable for construction. However, advances in understanding soil behavior, significantly influenced by developments in laboratory and in situ testing, have enabled substantial progress in ground improvement, making construction feasible even on sites with challenging geological or geotechnical conditions. Several ground improvement techniques exist, which can be broadly classified into: Grouting methods, consolidation techniques, reinforcement with artificial and natural elements, compaction techniques, chemical stabilization, Thermal techniques, and others.

Among the commercially recognized geotechnical methods, Dynamic Replacement (DR) stands out for its low

cost, speed, reuse of materials, and the absence of cementitious materials. According to Hamidi et al. (2010a), the method was patented by Louis Menard in 1975. Similar to dynamic compaction, DR enhances soil strength parameters through the impact of a free-falling hammer from significant heights, differing primarily in the use of replacement material for reinforcement. The resulting composite ground exhibits increased bearing capacity and shear resistance, reduced and accelerated settlements, and improved performance against slope instability and liquefaction (Han, 2015).

Given the scarcity of Brazilian literature on the subject, this article presents a comprehensive literature review on Dynamic Replacement. The objective is to synthesize the state of the art of the technique, focusing on its fundamental concepts, equipment, monitoring methods, numerical modeling, and practical applications in engineering projects.

2. Dynamic replacement – concepts, specifications, and executive procedure

Dynamic replacement (DR) is a ground improvement technique in which large-diameter granular columns, known as dynamic replacement columns (DRCs), are formed by repeatedly dropping a heavy tamper onto soft ground (Figure 1).

[#]Corresponding author. E-mail address: tennisonufpr@outlook.com

¹Universidade Federal do Paraná, Departamento de Engenharia Civil, Curitiba, PR, Brasil.

²University of Florida, Herbert Wertheim College of Engineering, Florida, FL, USA.

Submitted on December 26, 2025; Final Acceptance on July 7, 2026; Discussion open until May 31, 2027.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2027.020625>



This is an Open Access article distributed under the terms of the Creative Commons Attribution license (<https://creativecommons.org/licenses/by/4.0/>), which permits unrestricted use, distribution, and reproduction in any medium, provided the original work is properly cited.

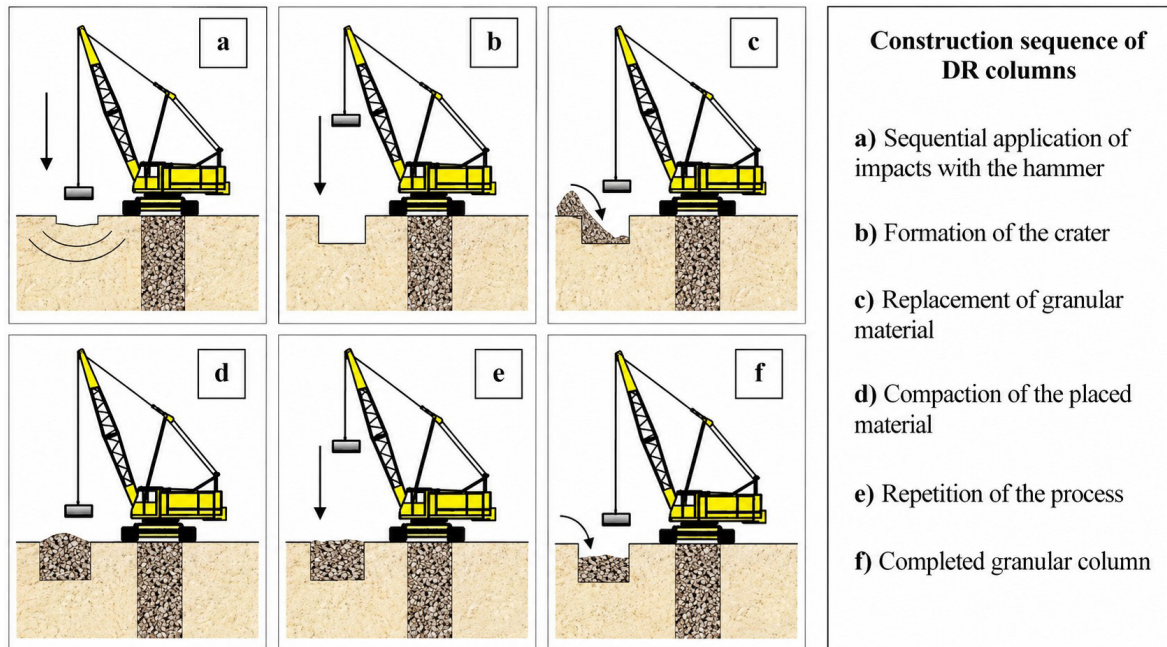


Figure 1. Illustration of the dynamic replacement procedure in soils: (a) sequential application of hammer impacts; (b) crater formation; (c) placement of granular material; (d) compaction of the placed material; (e) repetition of the process; and (f) completed granular column.

The tamper penetrates the soil and creates a crater that is filled with granular material (e.g., sand, gravel, or demolition debris) and compacted by successive impacts. The method can involve the dynamic compaction of granular backfill through successive hammer drops weighing between 150 and 250 kN from heights of 10 to 25 m, resulting in large-diameter reinforced soil columns typically ranging from 1.6 to 3.0 m in diameter and extending to depths between 4 and 8 m (Lupieżowicz & Kanty, 2015; Han, 2015; Tarawneh et al., 2017; Menard, 2023).

Although Dynamic Replacement (DR) employs equipment similar to that used in dynamic compaction, the applied energy is primarily intended to introduce and compact granular material within the ground rather than to densify the in-situ soil (Tarawneh et al., 2019). Recommended construction practices include the use of working platforms with thicknesses ranging from 0.6 to 1.2 m (Menard, 2023) or, alternatively, a 1.0 m thick compacted granular stress transition layer that replaces the working platform and ensures uniform load distribution over the DR columns (Souza Junior et al., 2023). When the presence of a rigid surface layer limits improvement at greater depths, pre-excavation may be adopted to facilitate granular material insertion and promote the dissipation of excess pore pressures (Hamidi et al., 2010a).

Replacement ratios of up to 25% are commonly achieved, with individual columns capable of supporting loads of approximately 1,500 kN, and the technique is widely applied in the reinforcement of embankments, particularly in highway construction projects in countries such as Poland. Regarding backfill materials, Han (1998) reported the use of sand,

gravel, and coarse aggregates. Other authors have noted that columns can be constructed using various granular materials with nominal grain sizes up to 400 mm, fines content below 15%, and free from organic matter (Tarawneh et al., 2017, 2019). Recommended materials include crushed concrete, gravel, sand, and dredged materials. Sękowski et al. (2018) obtained satisfactory results using a 1:1 mixture of fine gravel, coarse sand, and rubble.

The lengths and shapes of DR (Dynamic Replacement) columns are highly variable; four typical shapes can be distinguished in the field: cylindrical, truncated cone, inverted truncated cone, and barrel (symmetrical or asymmetrical) (Kwiecień et al., 2025). During the execution of the DR column, local soil softening occurs due to structural disturbance, pore pressure generation, and remolding of fine-grained soil; however, over time, there is a progressive gain in stiffness and strength associated with pore pressure dissipation and reconsolidation (Sękowski et al., 2018).

In general, the dynamic replacement process can be regarded as an extension of conventional dynamic compaction. According to Stinnette et al. (1997), it represents an enhanced approach for stabilizing organic deposits by partially substituting the native soft soil with compacted sand columns. The tamper is systematically dropped multiple times at predefined locations to drive granular material into compressible cohesive soils, forming the so-called *tamper phase* (Hamidi et al., 2010a; Tarawneh et al., 2017; Tarawneh et al., 2019). Following this stage, an *ironing phase* is carried out, in which a flat tamper levels the ground surface, and finally, surface compaction is performed using a vibratory roller (Menard, 2023).

Laboratory model tests indicate that the depth of the initial crater, the volume of backfill material and its grading strongly control the final shape, support conditions, and efficiency of dynamic replacement columns. Shallow initial craters partially backfilled favor the formation of end-bearing dynamic replacement columns with increasing diameter with depth, whereas deeper and fully filled craters tend to produce floating or irregular columns with lower bearing efficiency (Kwiecień et al., 2025).

In the field test, specifically CPTu tests, showed that the soil immediately adjacent to the column initially exhibited a temporary reduction in strength during column formation as a function of excess positive pore pressure. However, several hours after installation, an increase in stiffness and shear strength was observed. These findings suggest that the structural changes induced in the soil surrounding a DR column are time-dependent and lead to progressive improvement in the mechanical properties of the treated ground (Kwiecień et al., 2025).

3. Technical feasibility of the dynamic replacement method

According to Taube & Swift (2021), builders historically handled low-strength soils by adding fill or removing/replacing the material. Although percussive techniques like dynamic compaction were known earlier, soil improvement gained broad acceptance as a viable solution only in the last half-century. As a reinforcement technique, Dynamic Replacement (DR) may be applied to a wide range of soils, including stiff clays, silts, sandy silts, soft clays, and organic soils. Furthermore, DR is observed to be effective within a conventional treatment depth of approximately 3 m (100 kN).

Due to its versatility, DR is widely applied to foundations of buildings, tanks, embankments, bridge abutments, and infrastructure such as pipelines and wind turbines, particularly in fine-grained soils, organic soils, and anthropogenic deposits (Lupieżowicz & Kanty, 2015).

Terashi & Juran (2000) note that the main disadvantage of Dynamic Replacement (DR) is that the improved soil may still experience up to 40% of the estimated settlement of the untreated ground. This occurs because the surrounding soil between the columns, although benefited by lateral compression, is not treated entirely. Field experimental studies conducted on foundations over soft soils demonstrate that, during dynamic replacement, excess pore water pressure dissipates rapidly, allowing the continuity of the construction process (Li et al., 2025).

A crucial aspect of the technique is its sustainable orientation (Souza Junior et al., 2023; Menard, 2023). The backfill material may consist of natural granular soil or, promisingly, demolition waste. This option promotes the circular economy, valorizing debris and reducing pressure on landfills. Unlike methods that use cement or lime, DR

improves the soil through interparticle friction, avoiding energy-intensive binders. This feature aligns the method with low-carbon construction practices, integrating technical efficiency with sustainability goals.

The combined application of dynamic replacement (DR) with complementary ground improvement techniques has been shown to be a feasible and effective solution for soft soils. Ishan (2014) provides a comprehensive assessment of DR integrated with prefabricated vertical drains (PVDs) for the treatment of soft, highly compressible clay deposits. In a similar context, the use of impact-based methods, such as dynamic compaction combined with vertical drains, has been previously investigated by Wang et al. (2001) and Tarawneh & Matraji (2014), who reported accelerated consolidation of the sedimentary deposits studied.

Within this integrated approach, DR acts not only as a load-transfer and reinforcement mechanism through the installation of large-diameter granular columns but also as a means of increasing stress levels and promoting faster consolidation in cohesive soils. The incorporation of PVDs plays a crucial role by significantly shortening drainage paths, thereby accelerating pore-water pressure dissipation and reducing the time required to achieve primary consolidation—one of the main limitations of DR when applied alone in very low-permeability clays.

From a design perspective, the DR–PVD system leads to substantial improvements in undrained shear strength (S_u), with reported increases on the order of 65–85% compared to untreated conditions. These strength gains are directly reflected in higher bearing capacity and markedly reduced foundation demands, as evidenced by reductions in required pile lengths of approximately 80–85% following ground improvement. Settlement analyses conducted using both analytical approaches (Priebe's method) and finite element modeling exhibited consistent trends, with differences of around 15%, underscoring the importance of employing multiple evaluation methods to achieve realistic and reliable performance predictions.

4. Failure mechanisms in dynamic replacement

Han (2015) consolidated the failure modes of granular columns (such as stone columns or compaction piles) into four possible scenarios (Figure 2) when subjected to compressive loading, which are analyzed using the calculation methods presented in Section 6. The Figure 2a illustrates the top crushing effect, which occurs when the granular material of the column (crushed stone, gravel, compacted sand, etc.) cannot withstand the applied vertical compressive stresses, consequently resulting in a volumetric contraction of the column. Next, Figure 2b reproduces the phenomenon of general shear failure typical of shallow foundations, which commonly occur in dense or stiff soils. In this type of failure,

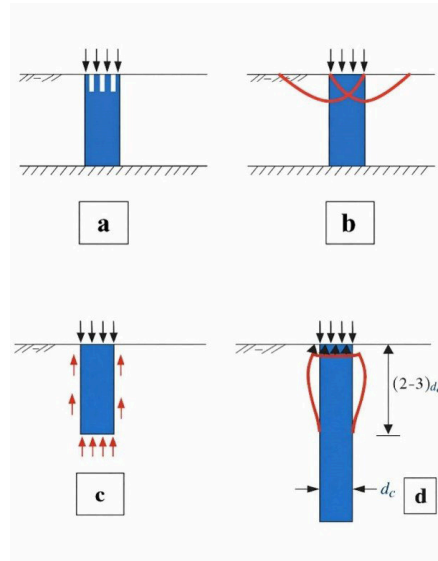


Figure 2. Modes of failure of a granular column embedded in soil under vertical loading: (a) Crushing; (b) general shear failure at the column head; (c) slip failure; (d) bulging failure. (Han, 2015).

a shear rupture surface develops after the material reaches its maximum shear strength and then propagates toward the ground surface.

The punching phenomenon (Figure 2c) occurs when the column exhibits relatively high stiffness, such that the underlying soil does not provide sufficient shear resistance to support the applied stresses, and consequently, the column penetrates into the soil. This type of failure represents a rigid mechanism like that observed in concrete or steel piles. However, this failure mode is generally considered inappropriate in the context of granular columns, as it neglects the potential for horizontal displacements that naturally occur during the application of vertical loading at the column head (Madhav & Vitkar, 1978).

Finally, the lateral expansion failure (Bulging) shown in Figure 2d is quite common in situations where the surrounding soil does not provide sufficient confining pressure to maintain the column's geometry. The surrounding soil offers limited lateral confinement, and the column "bulges" in this zone until the passive resistance of the soil is exceeded. This expansion generally occurs at a depth of 2 to 3 column diameters below the top of the column ($2-3d_c$).

Hughes & Withers (1974) studied granular columns (12.5 mm to 38 mm diameter) and found that vertical and radial deformation patterns were essentially the same regardless of column length. Their work showed that a single column's ultimate load capacity is primarily governed by the maximum lateral resistance of the soil around the bulging zone. They correlated the failure mechanism with the column's Length-to-Diameter ratio (L_c/d_c), if $L_c/d_c < 4$, the column fails before reaching the bulging stage. In soils with constant undrained shear strength (S_u) vertical stress decreases to zero at a critical depth of approximately $4.1 d_c$.

If the column is too short (base stress exceeds $9.0 S_u$) failure occurs before bulging development. Otherwise, the columns mobilize their ultimate resistance by transferring stress to the base as length decreases.

4.1 Influence depth – empirical formulations

The literature presents several other approaches for estimating improved depth, based on the findings and experimental studies of various researchers. Table 1 summarizes a compilation of studies that propose empirical relationships for estimating the depth of improvement that may occur beneath the impact point.

Mayne et al. (1984) compiled data from multiple sites worldwide across different soil types and concluded that the empirical coefficient η for improvement typically ranges between 0.3 and 0.8. Rollins & Rogers (1994) recommended $\eta = 0.4$ for collapsible soils and $\eta = 0.4-0.5$ for dry sandy silts. Leonards et al. (1980) suggested $\eta = 0.5$ for granular soils. However, Lukas (1980) concluded that $\eta = 0.65-0.80$ was more appropriate for the eight cases he studied. At some sites, η values may reach as high as 1.0. In Brazilian experience, Giese (2019) and Souza Junior et al. (2023) determined through SPT tests that the value of η for the frictional cohesive soil of Passo Fundo, Brazil, was 0.5.

4.2 Shear strength estimation methodology for DR columns

The shear strength of the soil/stone column composite is based on the premise that the column material undergoes shearing from the outset, while the surrounding soil reacts elastically. Furthermore, it is assumed that the soil around

Table 1. Summary of empirical equations developed by various authors in the literature for estimating influence depth.

Authors	Equations
Menard & Broise (1975)	$WH > D^2$
Mitchell (1981)	$D = \eta \sqrt{WH}$
Mayne et al. (1984)	$\eta = \text{empirical coefficient}$
Varaksin (1981)	$D = c \cdot \alpha \sqrt{WH}$
	$c = \text{Equipment efficiency factor - 1 (Free fall) or 0.9 (cables)}$
	$\alpha = \text{Soil efficiency factor (0.5 or 0.7 depending on whether the soil was heterogeneous or granular)}$
Charles et al. (1981)	$D = 0.4 \sqrt{\frac{(E_d \cdot d)}{(A_p \cdot S_u)}}$
	$E_d/A_p = \text{Total energy applied in the area}$
	$d = \text{Hammer diameter}$
	$S_u = \text{Undrained shear strenght}$
Smits & De Quelerij (1989)	$DI = \frac{W}{A_p \cdot \rho_{\max}} \left[-1 + \sqrt{\frac{\sigma_v + \alpha_{pr} V_0^2}{\sigma_v}} \right]$
	$\alpha_{pr} = \text{density ratio}$
	$W = \text{Hammer mass}$
	$A_p = \text{Hammer contact area}$
	$\rho_{\max} = \text{Maximum dry density}$
	$\sigma_v = \text{Total Vertical stress}$
	$V_0 = \text{impact velocity}$
Luongo (1992)	$D = K_1 + K_2 (WH)$
	$K_1 = \text{Depth factor}$
	$K_2 = \text{Energy fator}$
	In terms of average values, the variable parameter is the deposit permeability, for which K_1 ranges from 2.6 to 7.9 and K_2 ranges from 0.009 to 0.016.
Rollins & Kim (1994)	$D = 0.586 \sqrt{WH} - 0.009(WH)$
Vuola & Hartikainen (1999)	$D = f_c \cdot f_m \cdot f_b \cdot \sqrt{WH}$
	$f_c = \text{Impact fator (0.8 to 0.9)}$
	$f_m = \text{Soil factor (0.65 to 0.75 in unsaturated sands)}$
	$f_b = \text{Dimensionless factor (1.5 to 2.0)}$
Fei et al. (2002)	$D = \sqrt{w^{\frac{2}{3}} \cdot H \cdot N \left[d_{cr} \cdot \rho_d (1-\omega) \right]^{-1}}$
	$N = \text{Blows number}$
	$d_{cr} = \text{Crater diameter}$
	$\omega = \text{Moisture content}$
	$\rho_d = \text{Dry unit weight}$
Ghassemi et al. (2009)	$D = 2(WHR)^{0.25}$
	$R = \text{Radius of influence}$
Hajjalilue-Bonab and Rezaei (2009)	$D = 3.1D_c + 1.75d$
	$D_c = \text{Crater Depth}$
	$d = \text{Hammer diameter}$

Legend: W = hammer mass; H = hammer drop height; DI = net impact depth (excluding crater depth); Developed based on rapid impactcompaction.

Table 1. Continued...

Authors	Equations
Mohammed et al. (2010)	$D = \eta_{RI} \sqrt{WHN}$
η_{RI} = Empirical coefficient of rapid impact compaction	
N = Blows number	

Legend: W = hammer mass; H = hammer drop height; DI = net impact depth (excluding crater depth); Developed based on rapid impact compaction.

the column has already expanded during installation to such an extent that its initial resistance corresponds to the hydrostatic state (Priebe, 1995).

The stress distribution in stone column-reinforced ground is primarily governed by the stress concentration factor (n_r) and the area replacement ratio (a_r). The parameter n_r , defined as the ratio between the stress acting on the column and that acting on the surrounding soil, represents the degree of load transfer to the columns, whereas a_r , defined as the ratio between the column area and the unit cell area, quantifies the proportion of soil replaced by granular material. (Barksdale & Bachus, 1983; Han, 2015; Tarawneh et al., 2019). Typical n_r values range from 1.0 to 5.0 for granular columns and may exceed 10.0 for concrete columns or geosynthetic-reinforced platforms (Han, 2015), and regarding a_r , most of the applications of stone columns give an average value between 15% and 35%.

Thus, the relations in Equations 1, 2, 3 and 4 are used to calculate the equivalent stiffness between the DR columns and the surrounding soil:

$$A_c = \frac{\pi d_c^2}{4} \quad (1)$$

$$a_r = \frac{A_c}{A} \quad (2)$$

$$n_r = \frac{E_c}{E_s} \quad (3)$$

$$m = \frac{a_r n_r}{1 + a_r (1 - n_r)} \quad (4)$$

Where A_c is the column area, A is the total area, E_c is the Elastic modulus of column, E_s is the Elastic modulus of soil, m is the Equivalent stiffness factor.

Madhav & Nagpure (1996) proposed a calculation model for the reinforcement and stabilization of embankments constructed on granular columns. The evaluation of the equivalent strength of the soil mass, improved through dynamic replacement, can be carried out using either rock columns or sandy soils. As shown in Figure 3, the proposed model considers a system in which an embankment of height H_e is placed over soft soil reinforced with granular columns

of length L_c and diameter d_c , spaced at distance s and filled with granular material, at a certain distance D_s from a rigid stratum. In this model, the improvement of the soil deposit results from the partial absorption of vertical stresses by the granular columns (see Figure 3), thereby enabling a reduction of stresses in the surrounding soil.

Based on Figure 3, the stress concentration effect on the column is expressed by Han (2015) using Equations 5 and 6:

$$\Delta \sigma_s = \mu \Delta \sigma_z \quad (5)$$

$$\mu = \frac{1}{1 + (n_r - 1)a_r} \quad (6)$$

Assuming that the deformations of the column and the soil are one-dimensional and identical, it is appropriate to adopt the concept of equivalence of soil strength and deformability parameters in order to model the effect of DR columns on the soil. DiMaggio (1987) proposed the development of equivalence formulas for strength parameters (c , ϕ , and γ) and deformation parameters (E) in the context of rock column installation, such that the soil after improvement could be represented by equivalent parameters, as expressed in Equations 7, 8, 9 and 10 or 11. However, when the load distribution rate increases, DiMaggio (2009 *apud* Hamidi et al., 2010b) developed a drained-strength approach, as described in Equation 6 and applied in studies of vibro-replacement with rock columns in the Malaysian Peninsula by Arulrajah et al. (2009).

These formulations provide a framework for quantifying the post-treatment mechanical behavior of soils improved with dynamic or rock columns and serve as the basis for the subsequent analytical and numerical evaluations presented in this study.

$$\gamma_{eq} = a_r \gamma_c + (1 - a_r) \gamma_s \quad (7)$$

$$c_{eq} = a_r c_c + (1 - a_r) c_s \quad (8)$$

$$E_{eq} = a_r E_c + (1 - a_r) E_s \quad (9)$$

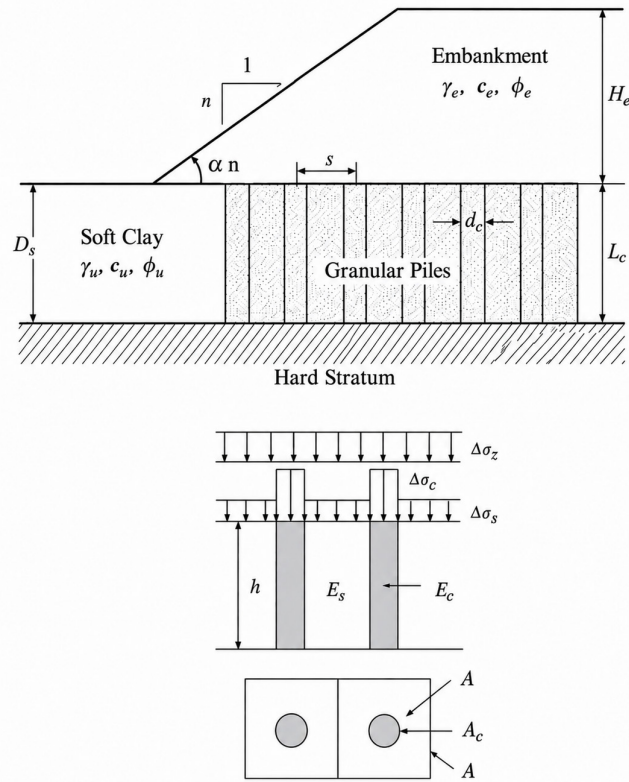


Figure 3. Applicable context of the analytical dynamic replacement model. Madhav & Nagpure (1996) and Han (2015).

$$\tan \varphi_{eq} = a_r \tan \varphi_c + (1 - a_r) \tan \varphi_s \quad (10)$$

$$\tan \varphi_{eq} = m' \tan \varphi_c + (1 - m') \tan \varphi_s \quad (11)$$

Where γ_{eq} is equal to equivalent density, c_{eq} is equal to equivalent cohesion, E_{eq} is equal to equivalent Young modulus, φ_{eq} is equal to equivalent internal friction angle, γ_c is equal to density of the dynamic replacement (DR) column, γ_s is equal to density of the subsoil (soft soil); c_c is cohesion of DR columns, c_s is equal cohesion of the soil, E_c is equal to the elastic modulus of the column, E_s is the Elastic modulus of soil φ_c is equal to the internal friction angle of the DR column, φ_s is equal internal friction angle of the subsoil (soft soil), and m' : load distribution ratio.

Based on the Mohr–Coulomb failure criterion, considering the individual failure of the granular column material and the surrounding low-bearing-capacity soil (e.g., soft clay), the stresses induced by the embankment on the soil are calculated using Equations 12 and 13. Equation 14, as illustrated in Figure 3, describes the additional stress in the soil surrounding the column, whereas Equation 15 describes the stress increment on the granular column generated by the embankment, accounting for the stress relief in the surrounding soil based on Equation 16. It is noteworthy that

Equation 17 represents the equivalent cohesion of the soil improved with the granular column.

$$\sigma_v = \sigma_h \left[(N_s - N_{gc}) a_r + N_{gc} \right] + 2 \left[c_s a_r \sqrt{N_s} + c_c (1 - a_r) \sqrt{N_{gc}} \right] \quad (12)$$

$$\sigma_v = \sigma_h N_{eq} + 2 c_{eq} \sqrt{N_{eq}} \quad (13)$$

Where:

$$N_{eq} = a_r N_s + (1 - a_r) N_{gc} \quad (14)$$

And:

$$N_s = \tan^2 \left(45 + \frac{\varphi_s}{2} \right) \quad (15)$$

$$N_{gc} = \tan^2 \left(45 + \frac{\varphi_c}{2} \right) \quad (16)$$

$$c_{eq} = \frac{c_s a_r \sqrt{N_s} + c_c (1 - a_r) \sqrt{N_{gc}}}{\sqrt{a_r N_s + (1 - a_r) N_{gc}}} \quad (17)$$

In situations where the granular fill material has a zero cohesive intercept ($c_u = 0$) and the low-bearing-capacity soil has zero internal friction ($\varphi_u = 0$), the equivalent parameters of the treated soil (c_{eq} and φ_{eq}) are described by Equations (18) and (19):

$$c_{eq} = (1 - a_r) c_u \sqrt{K} \quad (18)$$

$$\varphi_{eq} = \frac{\pi}{2} - 2 \tan^{-1} \sqrt{K} \quad (19)$$

Where K is calculated according to Equation 20:

$$K = \frac{1}{\sqrt{N_{eq}}} = \frac{1 - \sin \varphi_c}{1 + (2\alpha_r - 1) \sin \varphi_c} \quad (20)$$

Madhav & Vitkar (1978) developed a solution for the ultimate bearing capacity at the base of a granular column (plane strain) based on the general shear failure mechanism, using the upper-bound theorem and kinematic considerations, as can be seen in Figure 4. This solution, although originally developed for granular columns installed by excavation and backfilling, can be used to represent the estimation of the mechanical resistance of DR columns. The work of Madhav and Vitkar (1978) evaluates the interaction between the granular column and the surrounding soil without establishing an equivalent effect.

The expressions for internal energy were derived following Chen (1969), accounting for cohesion mobilized along multiple failure surface boundaries and the work performed by the weight of the soil, surcharge, and external

load. The ultimate bearing capacity q_{ult} is described by Equation 21 with reference to Figure 4a and b is based on Equations 22 and 23.

$$q_{ult} = C_s N_c + D_f \gamma_s N_q + \frac{1}{2} \gamma_s B N_\gamma \quad (21)$$

Where:

$$N_c = \frac{C_t}{C_s} N_{ct} + N_{cs} \quad (22)$$

$$N_\gamma = \frac{\gamma_c}{\gamma_s} N_{\gamma t} + N_{\gamma s} \quad (23)$$

Where q_{ult} is the ultimate bearing capacity at the base of the column; C_s is the cohesion of the surrounding soil; γ_s is the natural unit weight of the surrounding soil; B is the width of the load applied on the granular column (Figure 4); D_f is the distance from the ground surface to the top of the granular column (see Figure 4), C_t is the cohesion of the column fill material; γ_c is the natural unit weight of the granular column fill material; N_{ct} and N_{cs} are the bearing capacity factors for trench and soil cohesion, respectively; $N_{\gamma t}$ and $N_{\gamma s}$ are the bearing capacity factors for trench and soil unit weight, respectively. Madhav and Vitkar (1978) explained that bearing capacity factors were calculated based on upper bound theorem.

Depending on the column arrangement, Madhav & Nagpure (1996) suggest that a_r should be calculated using Equation 24 and see in Figure 5. In relation to the geometry of the DR column arrangement, when S_f equals S_2 , the pattern

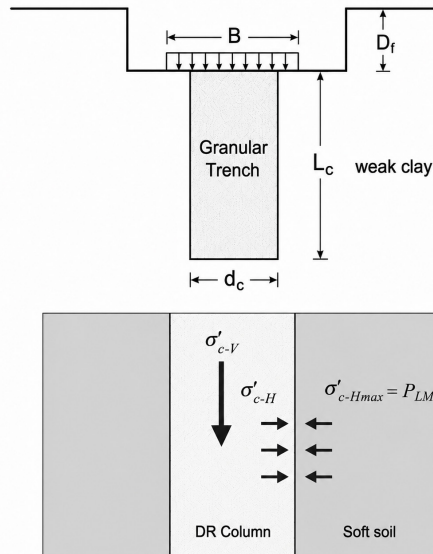


Figure 4. Analytical model of bearing capacity of granular columns generated by dynamic replacement. Adapted to Hamidi (2014).

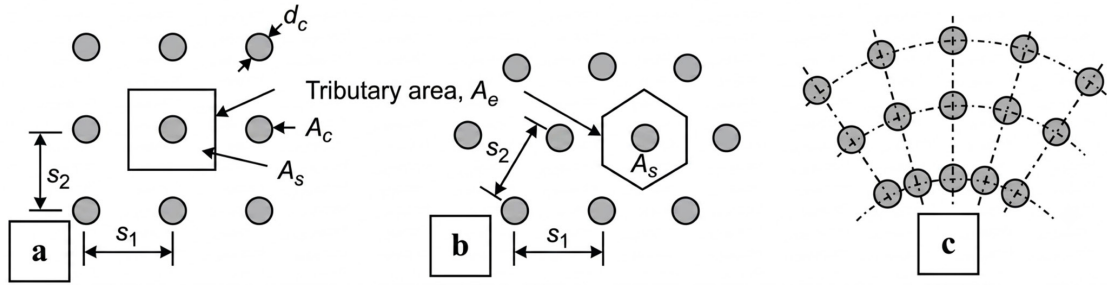


Figure 5. Spatial arrangement of DR columns in the soil: (a) Square; (b) Triangular, and (c) Radial. Adapted from Han (2015).

is square; however, when there is discontinuity between rows, an equilateral triangular pattern is formed. Rectangular and triangular patterns are the most used in foundations, while the radial pattern is more suitable for circular or annular foundations, such as tanks (Han, 2015).

$$a_r = c_1 \left(\frac{d_c}{s} \right)^2 \quad (24)$$

Where c_1 equals 1.023 (triangular pattern) or 1.27 (square pattern), based on Madhav & Nagpure (1996), but Han (2015) suggests 0.907 (equilateral triangular pattern) or 0.785 (square pattern); d_c is the diameter of the granular column, and s is column spacing. It is important to note that there is no universal specific value of c_1 for radial arrangement, since it depends on the radial spacing, the number of columns per ring, and the variable radius.

Hughes & Withers (1974) observed that clay within a cylindrical zone of approximately 2.5 times the column diameter (d_c) was significantly influenced by the applied loading. Based on this, they suggested a spacing of about $2.5 d_c$ between adjacent granular columns for independent performance.

Madhav & Vitkar (1978) developed design charts that relate the bearing capacity factors N_c , N_q , and N_γ to the ratio between the granular column width and the applied load width (B/D). In these charts, failure is characterized by a generalized top rupture mechanism, which incorporates the shear strength of both the column material and the surrounding soil.

Although its use is somewhat limited for estimating the bearing behavior of soils improved through the dynamic replacement method—considering that the application of the dynamic compaction principle can produce significant densification within the bearing layer at the toe of the granular column—it would still be reasonable to employ the approaches proposed by Madhav & Vitkar (1978) and Madhav & Nagpure (1996), depending on the column depth and the stiffness of the material used.

Barksdale & Bachus (1983) proposed that the bearing capacity of an isolated column, or a stone column within a

group, can be expressed in terms of the maximum stress applied to the column, as given by Equation 25

$$q_{ult} = S_u N_c \quad (25)$$

Where q_{ult} is ultimate bearing capacity; S_u is undrained shear strength; N_c is bearing capacity factor of the stone column ($18 \leq N_c \leq 22$) by Barksdale & Bachus (1983).

Hamidi (2014) established a relationship between surrounding soil confinement and the expansion pressure of the granular column. This draws on Hughes & Withers (1974), who conceptualized column bulging as a cylindrical expansion, analogous to the pressuremeter test (PMT) described by Vesic (1972; 1977). Consequently, the horizontal stress within the dynamic replacement (DR) column is limited by the surrounding soil's earth pressure. To maintain stability, the PMT limit pressure (P_{LM}) must exceed the horizontal stress exerted by the DR column (Equation 27; Figure 4b).

$$K_a \sigma'_{c-V} \leq P_{LM} \quad (26)$$

$$q_{ult} = \sigma'_{c-V} \leq \frac{1 + \sin \phi}{1 - \sin \phi} P_{LM} \quad (27)$$

Hamidi (2014) reported, based on practical experience, that it is advisable to apply a safety factor of 2 to the lateral stress, such that $\sigma'_{c-V} \leq 2P_{LM}$ condition is satisfied.

Tarawneh et al. (2017) proposed a methodology to estimate the settlement of clayey soils improved with the Dynamic Replacement (DR) technique, based on the Schmertmann method (1970 and 1978), and validated this proposal using a Finite Element Model (FEM) in the ABAQUS software, employing the Drucker-Prager constitutive model with CPTu data and Hardening cap soil. When comparing the plate load test results (Reference) with the results of the Schmertmann (1970) and Schmertmann et al. (1978) and the numerical modeling in FEM, an overestimation of 2.16 mm to 2.86 mm of the design stress was observed. Equations

28 to 30 express the proposed model for calculating the DR column strength.

$$E_{eq} = 5q_{ceq} \quad (28)$$

$$R_r = N_p \frac{\pi r^2}{S^2} \quad (29)$$

$$q_{eq} = R_r q_p + (1 - R_r) q_{in} \quad (30)$$

Where R_r is DR replacement ratio, N_p is number of pillars in the unit cell ($N=2$ recommended), q_{ceq} is equivalent tip resistance, E_{eq} is Equivalent soil modulus of elasticity and q_{in} is tip resistance of the soil between columns.

4.3 Equipment used in dynamic replacement

Although dynamic replacement is a ground improvement technique and dynamic compaction represents a soil compaction method, the equipment used in the execution of both procedures is essentially the same. In onshore projects, the use of tracked vehicles, commonly referred to as heavy-duty cranes, is predominant, as shown in Figure 6. According to Hamidi (2014), dynamic compaction equipment has evolved in terms of efficiency, cost, and

logistics, leading him to develop and manufacture his own platforms.

The 700-ton platform, illustrated in Figure 6a, can lift hammers weighing 25 tons. For filling the crater produced during the process, a backhoe is employed (Figure 6b). In Brazil, some companies have been investing in Rapid Impact Compaction (RIC) equipment due to its low cost, easy mobility, and safety. Unlike deep dynamic compaction equipment, these systems consist of crawler-mounted machines equipped with hydraulic hammers, typically weighing approximately 9 tons, which operate through controlled drops ranging from 0.3 m to 1.2 m onto a patented circular steel foot with a diameter of about 1.5 m.

In offshore soil improvement projects, barges equipped with lifting and release systems, also referred to as heavy-duty cranes, are used (see Figure 7) with typical dimensions of 15×50 m (Hamidi, 2014). Hammers used in offshore zones are specifically designed to minimize water resistance, as this resistance reduces the efficiency of the impacted energy and, consequently, affects soil improvement. To mitigate these effects, the hammers are made heavier and feature openings in their cross-section to reduce surface tension and hydraulic thrust (see Figure 8).

In geometric terms, compaction hammers exhibit various shapes—circular, hexagonal, square, and others (Figure 8). According to Hamidi (2014), they can be manufactured



Figure 6. Ground improvement procedure using dynamic replacement in soils: (a) Site with Dr columns; (b) Crawler bulldozer filling the columns. Zetas (2015).



Figure 7. Offshore dynamic replacement. Adapted to Hamidi et al. (2010b).

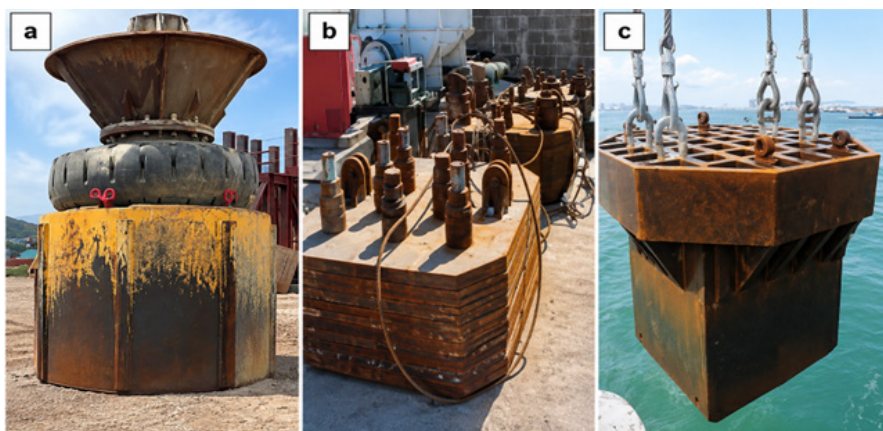


Figure 8. Hammers used in dynamic replacement: (a) onshore steel hammer for free-fall; (b) onshore steel hammer with cable system (after Hamidi, 2014); (c) offshore steel hammer. Adapted from Hamidi et al. (2010b).

from any material. However, they are typically made of reinforced concrete encased in a steel box or steel plates fastened together with large steel bolts. From a practical standpoint, however, concrete hammers have a relatively short service life, whereas steel plate hammers can be used frequently with minimal maintenance.

It is also noteworthy that dynamic replacement hammers feature smaller bases to increase soil shearing and penetration. These hammers can achieve soil improvement either through

free fall or via cable systems; however, cable-operated systems are less efficient due to energy losses during hammer release.

4.4 Performance monitoring and geotechnical investigations

Monitoring and evaluation methods for Dynamic Replacement (DR) are like those used in Dynamic Compaction. The Menard Pressuremeter Test (PMT) is one of the most

widely cited tests for assessing post-improvement shear strength, reported in several case studies (e.g., Hamidi et al., 2010a, 2011; Hamidi, 2014).

Piezocene tests (CPTu) and Load Tests are also used for pre- and post-evaluation (Tarawneh et al., 2019, 2017; Sękowski et al., 2018), alongside SPT and PMT. A common field criterion is evaluating blows versus hammer penetration (refusal), indicating adequate material densification (Mayne et al., 1984; Lukas, 1995; Hamidi, 2014). Monitoring lateral soil displacements using inclinometers is crucial to prevent damage to public utilities (Lukas, 1995; Giese, 2019; Souza Junior et al., 2023).

The generation of DR granular columns produces vibratory waves (Lupiczowiec & Kanty, 2015). Strong hammer impacts generate frequencies around 10 Hz, which can correspond to the resonant frequency of buildings. Hammer acceleration is also critical, as it translates into wave propagation velocities that must be limited to prevent structural damage. Other geotechnical methods applied include Spectral Analysis of Surface Waves (SASW) or the Continuous Surface Wave (CSW) technique, which is effective for deeper improvements (Kopf et al., 2010).

It is further observed that the maximum soil acceleration decreases with increasing distance from the compaction point, being more pronounced in the vertical and radial directions, while energy propagation occurs predominantly through low-frequency waves (Souza Junior et al., 2023; Li et al., 2025). Regarding vibration limits, dynamic compaction typically produces frequencies between 6 Hz and 10 Hz (Lukas, 1995). The U.S. Bureau of Mines recommended Peak Particle Velocities (PPV) below 13 mm/s for older structures and 19 mm/s for modern ones. Other thresholds

are 40 mm/s (structural damage) and 2.5 mm/s (human discomfort) (Slocombe, 2013).

5. Case studies

There are numerous projects worldwide that have employed dynamic replacement, including in the United Arab Emirates, Australia, the United States, and other regions. Three successful cases of dynamic replacement in the United Arab Emirates are highlighted below, as reported by Hamidi et al. (2010a) and Hamidi et al. (2010b). Additional cases can be found in Hamidi (2014).

5.1 Jaber Al-Ahmad City Project – Kuwait city

Tarawneh et al. (2017) described the infrastructure development project in Jaber Al-Ahmad City, located on the coastal plain of Kuwait Bay, approximately 25 km west of Kuwait City. Both dynamic compaction and dynamic replacement were employed in soils with layers of medium-density silty sands and sandy silts, silty clays, and highly plastic clays up to 5 m depth; and medium- to high-density silty sands, and medium- to highly plastic silty clays and clayey silts between 5 and 8 m depth. A 15-ton hammer was used, with an average of 8 blows per phase (ranging from 6 to 12 blows), and 2 to 4 blows during the ironing phase (see Figure 9).

SPT, CPT, and PMT tests were conducted at identical locations before and after ground improvement, covering both areas directly influenced by the granular columns and the inter-column zones treated by dynamic replacement (DR) and dynamic compaction. All tests were performed 72 h after

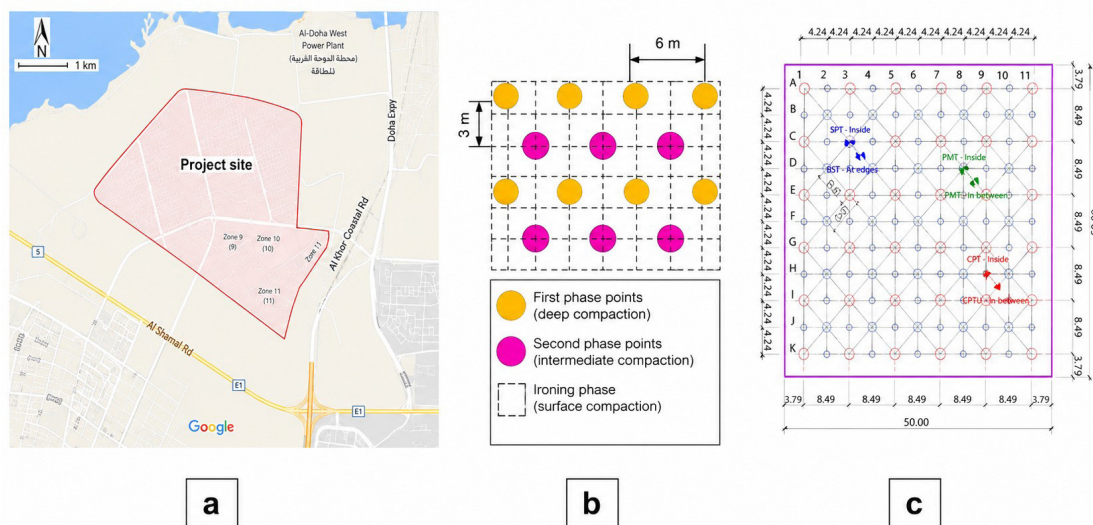


Figure 9. Jaber Al-Ahmad City Project: (a) Project location plan; (b) Compaction grid layout; (c) Ground improvement plan. Adapted from Tarawneh et al. (2017).

treatment to ensure pore pressure dissipation. Comparative results are presented in Figure 10.

SPT results show that the average N_{SPT} increased from 8 blows in the pre-improvement condition to 25 blows within the granular columns and 21 blows between columns, corresponding to increases of approximately 213% and 163%, respectively. Significant improvement was achieved within the granular columns down to a depth of approximately -2.0 m. Between columns, similar gains were observed except within the upper 1.5 m, where the presence of fine-grained Sabkha soils limited the improvement.

Consistently, CPT measurements indicate that the average cone resistance (q_c) increased from 5.41 MPa before treatment to 19.34 MPa within the granular columns and 11.76 MPa between columns after DR, representing increases of approximately 257% and 117%, respectively. Pressuremeter (PMT) results indicate that the average limit pressure increased from 0.89 MPa in pre-treatment condition to 2.37 MPa within the granular columns and 1.99 MPa between columns after deep compaction, representing increases of approximately 166% and 124%, respectively. Despite these significant improvements, the absolute values remain relatively low due to the presence of fine-grained Sabkha soils, which exhibit

limited compaction potential. Moreover, within the upper 1.5 m of the soil profile, reduced stiffness and strength were observed at the column tip compared to the original soil, attributed to the application of high compaction energy and pre-existing damage in the near-surface stiff layer before ground improvement.

Consistent trends were reported by Ismael et al. (2023) in a large-scale residential development in Jaber Al-Ahmad City, where dynamic replacement effectively transformed very loose sandy layers into medium-dense to dense states. Substantial increases in CPT cone resistance, SPT values, and PMT limit pressures were documented, resulting in allowable bearing pressures of 1.79–2.6 MPa, exceeding project requirements, with minimum foundation pressures of 300 kPa readily achieved.

The dynamic replacement can significantly enhance soil strength and stiffness, reduce compressibility, and provide reliable support for shallow foundations. The statistical analysis of the pressuremeter test results indicates that dynamic replacement led to an average increase of approximately 17% in the limit pressure within the columns (PL_{pillar}) compared to the surrounding soil (PL_{soil}). The average pressure limit within the columns (PL_{pillar}) was 2.50 MPa, with a standard deviation of 0.38 MPa, whereas the adjacent soil exhibited

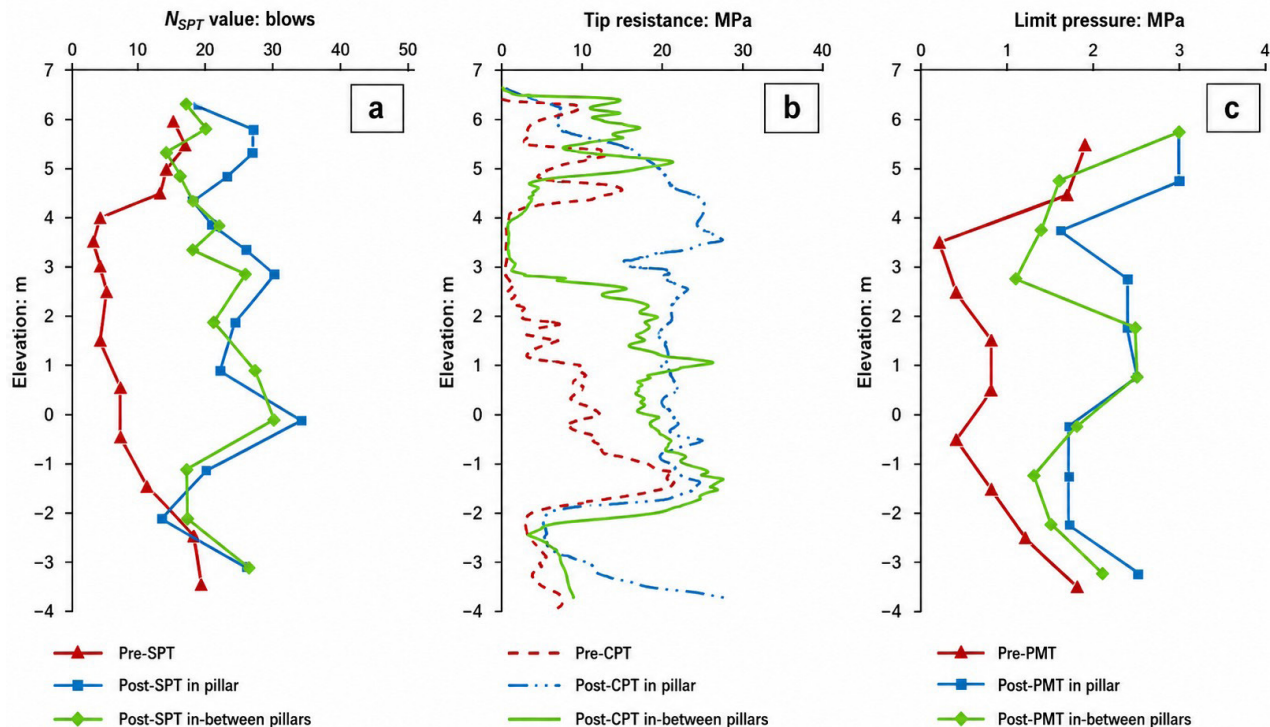


Figure 10. Comparative plots of soils at Jaber Al-Ahmad City: pre-improvement, post-improvement within DR granular columns, and post-improvement between DR columns, based on field tests: (a) SPT; (b) CPT; (c) PMT. Adapted to Tarawneh et al. (2017).

an average value of 2.14 MPa, with a standard deviation of 0.28 MPa, representing an average increase of approximately 17% within the columns. The equivalent limit pressure of the treated ground ($PL_{equivalent}$) showed an average value of 2.08 MPa and a standard deviation of 0.18 MPa, indicating low dispersion and a high degree of homogeneity of the improved ground.

5.2 Foundation reinforcement project – Abu Dhabi Ritz Carlton Hotel, Abu Dhabi

Hamidi et al. (2010a) described the application of dynamic replacement (DR) for the construction of base trenches to support reinforced retaining walls at the Abu Dhabi Ritz Carlton Hotel in Abu Dhabi (United Arab Emirates). The adopted solution involved mechanized excavation of the trench, dynamic replacement using a granular fill composed of sand and demolished concrete, followed by the construction of the reinforced retaining wall on the improved soil.

The objective was to increase the factor of safety of the embankment–retaining wall–soil system, as illustrated in Figure 11. In this specific project, the natural soil conditions were unfavorable, consisting of saturated silty clays extending to depths of up to 5 m, with standard penetration test (SPT) values ranging from 1 to 3 blows.

Comparing the pre-treatment and post-treatment stages using PMT, an improvement of 100% in the limit pressure was observed (from 110 kPa to 220 kPa) between the DR granular columns. Within the granular columns themselves, typical minimum limit pressures reached 700 kPa. The maximum improvement within the columns exceeded

900%, representing an increase of 5.15 times greater than the improvement observed between the columns.

5.3 Offshore marine deposit improvement – southeast Asia

In Southeast Asia, a large-scale dynamic replacement test was conducted over an area of 22.5 m × 22.5 m to treat soft marine deposits to a depth of 30 m below sea level. The original in-situ material consisted of very stiff clay, with a shear strength of approximately 250 kPa and N_{SPT} values exceeding 50. However, dredging operations and lowering of the water level to –30 m disturbed the upper 1–1.5 m of the clay surface layer. Subsequent pressiometer tests indicated that the shear strength had decreased to 16 kPa. Given the low strength of the disturbed material, the adopted solution was the removal of the softened clay and its replacement with a more suitable granular soil via dynamic replacement (Hamidi et al., 2010b).

The DR grid was specified at 4.5 m × 4.5 m, resulting in a total of 25 columns across the improvement plan. After the construction of the DR columns, an “Ironing” phase was performed to densify the granular fill layer (Hamidi et al., 2010b).

During the first 15 hammer blows, penetration varied between 1.1 and 1.7 m. The hammer impact effectively conveyed the granular fill to the bottom of the soft soil layer within the first 4 to 12 blows, with the remaining blows used to further compact the granular rock fill (Hamidi et al., 2010b). Following treatment, although the rock fill was directed into the soft clay, Menard pressuremeter tests indicated increases of 118% in the

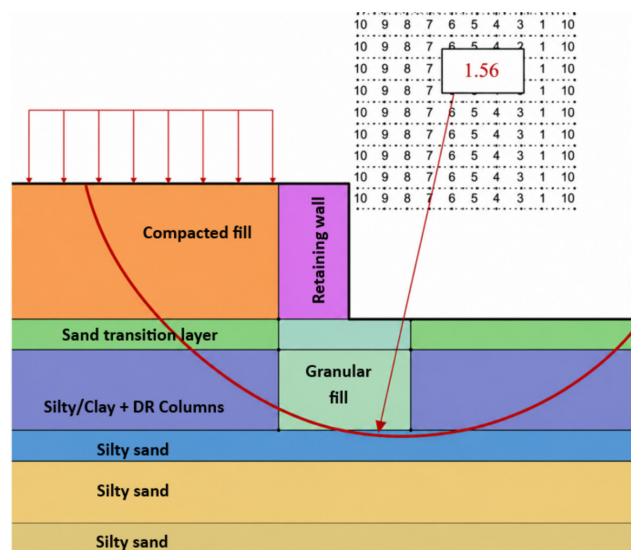


Figure 11. Limit equilibrium simulation of gravity wall improved with Column DR at the Abu Dhabi Ritz Carlton Hotel (United Arab Emirates). Adapted from Hamidi (2014).

pressiometric modulus E_p and 132% in the limit pressure PI . The average post-improvement values of E_p and PI were 8.05 MPa and 1.40 MPa, respectively, resulting in an average ratio E_p/PI of 5.75 within the DR columns. The maximum PI recorded during testing exceeded 2.2 MPa.

Hamidi et al. (2010b) also cited other offshore projects employing the dynamic replacement method: at the Sfax fishing quay in Tunisia, a 170 kN compaction hammer was used to densify a 3–4 m thick layer of loose silty sand as a foundation for retaining walls; in Lagos, Nigeria, dynamic replacement was applied to compact a 10 m thick layer of silty sand located 15 m below sea level.

5.4 Dynamic Replacement applied in lateric soil – Rio Grande do Sul/Brazil

Souza Junior et al. (2023) published an experimental field study evaluating the load-bearing capacity and stiffness response of a lateritic clayey soil from Passo Fundo, Rio Grande do Sul, with and without the installation of Dynamic Replacement (DR) columns incorporating soil–cement layers (10%) at the top of the column, with thicknesses of 150 mm and 500 mm, as can be seen in Figure 12. Plate load tests confirmed a significant increase in both stiffness and bearing capacity as a function of the soil–cement layer thickness, reaching values on the order of 45–60 kPa for the 150 mm layer and 140–180 kN for the 500 mm layer, the latter exhibiting the highest stiffness levels observed in the study. Since the

subsoil conditions were relatively homogeneous within the investigated domain, column length did not significantly influence performance. Conversely, dynamic replacement with underlying granular material contributed substantially to the improvement in system stiffness.

5.5 Dynamic replacement for the Coal Export

Terminal 3 (CET3) stockyard on Kooragang Island, Australia

Chua et al. (2008) presented the geotechnical design considerations and analytical work performed for the development of one of the largest Dynamic Replacement (DR) projects in the Southern Hemisphere for its scale and technical challenge in stabilizing land. The project involves the stabilization of massive stockpiles of a 30 Mt/year coal stockyard area at the Coal Export Terminal 3 (CET3) in Kooragang Island, NSW, Australia. The coal stockyard of CET3 covers an area approximately 1.2 km long by 300 m wide. The site is underlain by soft compressible soil strata of variable thickness. The stockyard supports 21 m high coal stockpiles and the coal stacker-reclaimer machinery loads.

Dynamic replacement has been demonstrated to be an effective ground-improvement solution for heavily loaded coal stockyard infrastructure constructed over highly variable, soft, and compressible soils. The method was adopted in preference to preloading because of programme constraints and limited availability of dredged fill, with treatment targeted to the upper weak strata to depths of approximately 6 m. Field trials and

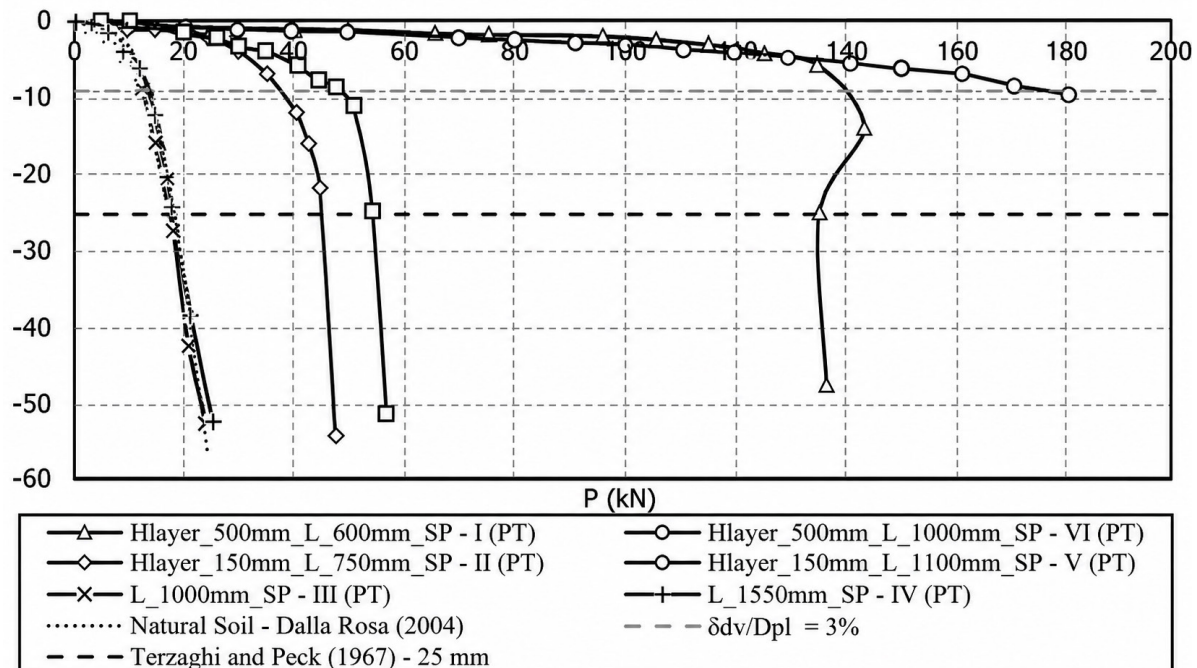


Figure 12. Comparative study of dynamic replacement columns. Adapted to Souza Junior et al. (2023).

verification testing showed that the constructed aggregate columns achieved substantial diameters (on the order of 2.5–3.3 m) and mobilised high shear strength and stiffness, with interpreted friction angles of roughly 36°–40°. To learn more about the study, it is possible to consult Chua et al. (2008) and Hamidi (2014).

6. Conclusions

Dynamic Replacement (DR) is a cost-effective and technically efficient ground improvement technique capable of significantly enhancing the mechanical behavior of low-strength soils, such as soft clays, at considerable depths and over large areas. In addition to its technical advantages, DR presents relevant sustainability benefits, including the reuse of construction and demolition waste and the elimination of cementitious materials in reinforcement columns.

Despite its proven effectiveness, the application of DR in Brazil remains limited. Nevertheless, the technique shows strong potential in tropical soils (lateritic soils) due to their chemical stability associated with natural porosity and their tendency to achieve high stiffness and shear strength when compacted, as well as in sedimentary deposits, particularly in large-scale infrastructure projects such as ports, airports, highways, industrial facilities, and building complexes. The literature still lacks comprehensive studies addressing the influence of granular fill properties on the efficiency of DR, especially regarding soil deformability and shear strength, indicating the need for further experimental investigations.

At present, design practices are predominantly based on empirical formulations, while numerical and analytical modeling approaches remain scarce. Additional well-documented case studies and monitoring programs are essential to support model calibration and improve the prediction of ground response. Furthermore, systematic monitoring is critical to evaluate the performance of granular columns, assess vibration effects on adjacent structures, and ensure adequate shear strength and deformation behavior of the improved soil under service loading conditions.

Acknowledgements

The authors would like to thank CAPES for funding the research through the PPGE (Civil Engineering Postgraduate Program) at UFRGS (Federal University of Rio Grande do Sul) and UFPR (Federal University of Paraná). We also extend our gratitude to the anonymous reviewers for their insightful comments and suggestions, which significantly improved the content of this article.

Declaration of interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Authors' contribution

Tennison Freire de Souza Junior: conceptualization, methodology, and writing - original draft preparation. Ana Paula Vianna Mohseni: investigation, data curation, reviewing, and validation.

Data availability

The datasets generated and analyzed during the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

The authors declare that no generative artificial intelligence (AI) tools were used in the conception, analysis, interpretation of data, writing, or revision of this manuscript. All content was produced solely by the authors based on their own expertise and original work.

List of symbols and abbreviations

c	Equipment efficiency factor
c_l	Area replacement constant
c_c	Cohesion of the column material
c_{eq}	Equivalent Cohesion
c_s	Cohesion of the soil
c_u	Undrained cohesion
d	Hammer diameter
d_c	Column diameter
d_{cr}	Crater diameter
f_b	Geometric factor
f_c	Impact factor
f_m	Soil factor
k_a	active earth pressure
m	Equivalent stiffness factor
m'	Load redistribution factor
q_c	CPT tip resistance
q_{eq}	Equivalent resistance
q_{in}	Resistance of soil between columns
q_{ult}	Ultimate bearing capacity
s	Column spacing
A	Total Area
A_c	Column area
A_p	Hammer contact area
B	Width of the distributed load
CV	Coefficient of variation
D	Depth of influence
D_c	Crater depth
D_f	Column base depth
DI	Net impact depth
D_s	Soil thickness
E	Young's modulus

E_c	Column Young modulus	φ_s	Friction angle of the soil
E_{eq}	Equivalent Young modulus	ω	Moisture content
E_p	Pressuremeter modulus	ABAQUS	Finite Element Analysis Software
E_s	Young Soil modulus	CPT	Cone Penetration Test
H	Drop height	CPTu	Piezcone Penetration Test
H_e	Embankment height	CSW	Continuous Surface Wave
K	Resistance parameter	DC	Dynamic Compaction
K_1	Depth factor	DMT	Dilatometer Test
K_2	Energy factor	DR	Dynamic Replacement
L_c	Column length	DRC	Dynamic Replacement Column
M	Oedometer modulus	FEM	Finite Element Method
N	Number of blows	GRCS	Geosynthetic-Reinforced
N_c	Bearing capacity factor (cohesion)		Column-Supported Embankment
N_{ct}	Bearing capacity factor of trench (cohesion)	PMT	Pressuremeter Test (Menard Pressuremeter Test)
N_{cs}	Bearing capacity factor of soil (cohesion)		
N_p	Number of pillars in the unit cell	PPV	Peak particle velocity
N_q	Bearing capacity factor (surcharge)	PVD	Prefabricated Vertical Drain
N_γ	Bearing capacity factor (unit weight)	SASW	Spectral Analysis of Surface Waves
$N_{\gamma t}$	Bearing capacity factor of trench (unit weight)	SPT	Standard Penetration Test
N_{ys}	Bearing capacity factor of soil (unit weight)		
N_{gc}	Grain column factor		
N_s	Soil resistance factor		
PL	Limit pressure		
PLM	Menard limit pressure		
$PL_{equivalent}$	Equivalent Limit Pressure		
PL_{pillar}	Limit Pressure between pillars		
PL_{soil}	Limit Pressure of soil		
R	Radius of influence		
R_r	DR replacement ratio		
S_u	Undrained shear strength		
V_o	impact velocity		
W	Tamper weight		
α	Soil efficiency factor		
α_{pr}	Density ratio		
α_r	Stress concentration ratio		
γ	Unit weight		
γ_c	Unit weight of the column material		
γ_s	Unit weight of the soil		
$\Delta_{\sigma s}$	Stress increment in the soil		
$\Delta_{\sigma z}$	Total vertical stress increment		
μ	Influence stress coefficient		
η	Empirical coefficient		
η_{RI}	Empirical coefficient of rapid impact compaction		
η_r	Stress coefficient ratio		
ρ_d	Dry density		
ρ_{max}	Maximum dry density		
σ_{cv}	Confining stress of the column		
$\sigma'_{c max}$	Maximum confining effective stress		
σ_v	Total vertical stress		
σ'_v	Effective Horizontal confining stress		
σ'_{c-h}	Effective Vertical confining stress		
σ'_{c-v}	Effective Vertical confining stress		
φ_{eq}	Friction angle equivalent		
φ_c	Friction angle of the column material		

References

- Arulrajah, A., Abdullah, A., Bo, M.W., & Bouazza, A. (2009). Ground improvement techniques for railway embankments. *Ground Improvement*, 162(1), 3-14. <https://doi.org/10.1680/grim.2009.162.1.3>.
- Barksdale, R.D., & Bachus, R.C. (1983). *Design and construction of stone columns* (FHWA/RD-83/026, Vol. 1, 194 p.). Federal Highway Administration.
- Charles, J.A., Burford, D., & Watts, K.S. (1981) Field studies of the effectiveness of dynamic consolidation. In *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering* (Vol. 3, pp. 617-622). Stockholm, Sweden.
- Chen, W.F. (1969). Soil mechanics and theorems of limit analysis. *Journal of the Soil Mechanics and Foundations Division*, 95(2), 493-518. <https://doi.org/10.1061/JSEFAQ.0001262>.
- Chua, C.C.H., Lai, M., Hoffmann, G., & Hawkins, B. (2008). Ground improvement using dynamic replacement for NCIG CET3 coal stockyard. *Australian Geomechanics*, 43(3), 63-74.
- DiMaggio, J.A. (1987). *Stone columns for highway construction* (Demonstration Project, No. 46, Report No. FHWA-DP-46-1). Federal Highway Administration.
- Fei, X.Z., Wang, Z., Zhou, Z.B., Gao, L.Y., & He, L.H. (2002). Model test of improvement depth of dynamic compaction. *ESE. Journal of Sichuan University*, 34(4), 56-59.
- Ghassemi, A., Pak, A., Shahir, H. (2009). Validity of Menard relation in dynamic compaction operations. *Proceedings of the Institution of Civil Engineers*, 162, 37-45.
- Giese, D.N. (2019). *Compactação dinâmica em solo coesivo/friccional* [Dissertação de mestrado]. Programa de Pós-Graduação em Engenharia Civil, Universidade Federal do Rio Grande do Sul. (In Portuguese)

- Hajjalilue-Bonab, M., & Rezaei, A.H. (2009). Physical modelling of low-energy dynamic compaction. *International Journal of Physical Modelling in Geotechnics*, 9(3), 1-14. <https://doi.org/10.1680/ijpimg.2009.090302>.
- Hamidi, B. (2014). *Distinguished ground improvement projects by dynamic compaction or dynamic replacement* [Thesis doctoral]. Department of Civil Engineering, Curtin University.
- Hamidi, B., Nikraz, H., & Varaskin, S. (2010a). Dynamic replacement constructing embankments and walls of soft soil. In *Proceedings of the 3rd International Conference on Problematic Soils* (pp. 105-111). Adelaide, Australia.
- Hamidi, B., Nikraz, H., & Yee, K., Varaskin & Wong, L.T. (2010b). Ground improvement in deep waters using dynamic replacement. In *Proceedings of the 20th International Offshore and Polar Engineering Conference* (pp. 848-853). Beijing, China.
- Hamidi, B., Nikraz, H., & Varaskin, S. (2011). Application of dynamic replacement in a steel pipe factory. In *Proceedings of the International Conference on Advances in Geotechnical Engineering* (pp. 867-871). Adelaide, Australia.
- Han, J. (1998). Ground modification by a combination of dynamic compaction, consolidation, and replacement. In *Proceedings of the International Conference on Case Histories in Geotechnical Engineering* (pp. 341-346). New York, USA.
- Han, J. (2015). *Principles and practice of ground improvement*. Hoboken, NJ: John Wiley & Sons.
- Hughes, J.M., & Withers, N.J. (1974). Reinforcing of soft cohesive soils with stone columns. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, 47(3), 42-49. [https://doi.org/10.1016/0148-9062\(74\)90643-3](https://doi.org/10.1016/0148-9062(74)90643-3).
- Ishan, E.S.M. (2014) Effects of consolidation settlement by dynamic replacement and prefabricated vertical drain on pile length design. *Technical Report*, 1-17. <https://doi.org/10.13140/RG.2.2.35781.55525>.
- Ismael, N.F., Ismael, D., & Otaibi, N. A. (2023). Ground improvement in loose sandy soils through dynamic replacement. *Geotechnical Engineering Journal of the SEAGS & AGSSEA*, 54(4), 1-7.
- Kopf, F., Paulmitch, I., & Adam, D. (2010). Modelling and simulation of heavy tamping dynamic response of the ground. In *Proceedings of the 14th Danube-European Conference on Geotechnical Engineering* (pp. 1-10). Bratislava.
- Kwieceń, S., Ihnato, V.S., & Kowalska, M. (2025). Influence of selected technological parameters on dynamic replacement column formation – results of laboratory model tests. *Archives of Civil Engineering*, 71(4), 497-515. <https://doi.org/10.24425/ace.2025.156184>.
- Leonards, G.A., Holtz, R.D., & Cutter, W.A. (1980). Dynamic compaction of granular soils. *Journal of the Geotechnical Engineering Division*, 106(1), 35-44. <https://doi.org/10.1061/AJGEB6.0000914>.
- Li, S., Liu, Z., Zhang, X., Qin, G., & Nie, R. (2025). Field experimental study on dynamic response of soft foundation during dynamic compaction replacement construction. *Advances in Civil Engineering*, 2025(1), 8032417. <https://doi.org/10.1155/adce/8032417>.
- Lukas, R.G. (1980). Densification of loose deposits by pounding. *Journal of the Geotechnical Engineering Division*, 106(4), 435-446. <https://doi.org/10.1061/AJGEB6.0000947>.
- Lukas, R.G. (1995). *Dynamic compaction* (Geotechnical Engineering Circular, No. 1, Rep. No. FHWA/SA-95/037). U.S. Federal Highway Administration.
- Luongo, V. (1992). Dynamic compaction: predicting depth of improvement. *Geotechnical Special Publication*, 2(30), 927-939.
- Lupiezowicz, M., & Kanty, P. (2015). Analysis of dynamic replacement column construction process on neighbouring engineering structures. *Archives of Civil Engineering*, 61(3), 3-18. <https://doi.org/10.1515/ace-2015-0021>.
- Madhav, M.R., & Nagpure, D.D. (1996). Design of granular piles for embankments on soft ground. In *Proceedings of the 12th SE Asian Geotechnical Conference* (pp. 285-290). Kuala Lumpur.
- Madhav, M.R., & Vitkar, P.P. (1978). Strip footings on weak clay stabilized with a granular trench or pile. *Canadian Geotechnical Journal*, 15(4), 605-609. <https://doi.org/10.1139/t78-066>.
- Mayne, P.W., Jones Junior, J.S., & Dumas, J.C. (1984). Ground response to dynamic compaction. *Journal of Geotechnical Engineering*, 110(6), 757-774. [https://doi.org/10.1061/\(ASCE\)0733-9410\(1984\)110:6\(757\)](https://doi.org/10.1061/(ASCE)0733-9410(1984)110:6(757)).
- Menard, L. (2023). *Dynamic replacement*. Retrieved in May 30, 2023, from <https://www.menard-group.com/soil-expert-portfolio/dynamic-replacement/>
- Menard, L., & Broise, Y. (1975). Theoretical and practical aspect of dynamic consolidation. *Geotechnique*, 25(1), 3-18. <https://doi.org/10.1680/geot.1975.25.1.3>.
- Mitchell, J.K. (1981). Soil improvement: state-of-the-art report. In *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering* (pp. 509-565). Estocolmo, Suécia.
- Mohammed, M.M., Hashim, R., & Salman, A.F. (2010). Effective improvement depth for ground treated with rapid impact compaction. *Scientific Research and Essays*, 5(18), 2686-2693.
- Priebe, H.J. (1995). The design of vibro replacement. *Ground Engineering*, 28(10), 31-37.
- Rollins, K.M., & Rogers, G.W. (1994). Mitigation measures for small structures on collapsible alluvial soils. *Journal of Geotechnical Engineering*, 120(9), 1533-1553. [https://doi.org/10.1061/\(ASCE\)0733-9410\(1994\)120:9\(1533\)](https://doi.org/10.1061/(ASCE)0733-9410(1994)120:9(1533)).
- Rollins, K.M., & Kim, J.H. (1994). US experience with dynamic compaction of collapsible soils. In American

- Society of Civil Engineers (Ed.), *In-situ deep soil improvement* (ASCE Geotechnical Special Publication, No. 45, pp. 26-43). Atlanta.
- Schmertmann, J.H. (1970). Static cone to compute static settlement over sand. *Journal of the Soil Mechanics and Foundations Division*, 96(3), 1011-1043. <https://doi.org/10.1061/JSFEAQ.0001418>.
- Schmertmann, J.H., Brown, P.R., & Hartman, J.P. (1978). Improved strain influence factor diagrams. *Journal of Geotechnical Engineering*, 104(8), 1131-1135.
- Sękowski, J., Kwiecień, S., & Kanty, P. (2018). The influence of dynamic replacement method on the adjacent soil. *International Journal of Civil Engineering*, 16(10), 1515-1522. <https://doi.org/10.1007/s40999-017-0231-6>.
- Slocombe, B.C. (2013). Dynamic compaction. In K. Kirsch & A. Bell. *Ground improvement* (3rd ed.). Glasgow, Germany: Chapman & Hall.
- Smits, M.T., & de Quelerij, L. (1989). The effect of dynamic compaction on dry granular soils. In *Proceedings of the 12th International Conference on Soil Mechanics and Foundation Engineering* (pp. 1415-1418). Rio de Janeiro, Brazil. A.A. Balkema.
- Souza Junior, T.F., Heineck, K.S., Ruver, C.A., Mohseni, A.P.V., & da Lopes, L.S. (2023). Application of dynamic replacement to lateritic clay. *Geotechnical and Geological Engineering*, 41(8), 4845-4864. <https://doi.org/10.1007/s10706-023-02531-4>.
- Stinnette, P., Gunaratne, M., Mullins, G., & Thilakasiri, S. (1997). A quality control programme for performance evaluation of dynamic replacement of organic soil deposits. *Geotechnical and Geological Engineering*, 15(4), 283-302.
- Tarawneh, B., & Matraji, M. (2014). Ground improvement using rapid impact compaction: case study in Dubai. *Građevinar*, 66(11), 1007-1014.
- Tarawneh, B., Bodour, W., Shatnawi, A., & Ajmi, K.A. (2019). Field evaluation and behavior of the soil improved using dynamic replacement. *Case Studies in Construction Materials*, 10, 1-11. <https://doi.org/10.1016/j.cscm.2018.e00214>.
- Tarawneh, B., Sbitne, A., & Hakam, Y. (2017). Lessons learned from 11 million m² of dynamic compaction and replacement. *Proceedings of the Institution of Civil Engineers - Ground Improvement*, 170(4), 208-217. <https://doi.org/10.1680/jgrim.17.00025>.
- Taube, M.G. & Swift, S.S. (2021). Ground improvement. *CSE Engineer Magazine*, 14-16.
- Terashi, M., & Juran, I. (2000). Ground improvement – State of the art. In *International Conference on Geotechnical and Geological Engineering* (pp. 461-519). Melbourne, Australia.
- Varaksin, S. (1981). Recent development in soil improvement techniques and their practical applications. *Sols Soils*, 38-39, 7-32.
- Vesic, A.S. (1972). Expansion of cavities in an infinite soil mass. *Journal of the Soil Mechanics and Foundations Division*, 98(3), 265-290. <https://doi.org/10.1061/JSFEAQ.0001740>.
- Vesic, A.S. (1977). *Design of Pile Foundations* (Synthesis of Highway Practice, No. 42, 68 p.). National Cooperative Highway Research Program, Transportation Research Board, National Research Council.
- Vuola, P., & Hartikainen, J. (1999). Aspects for dynamic compaction of saturated sand. In F.B.J. Barends, J. Lindenberg, H.J. Luger, L.D. Queleri, & A. Verruijt (Eds.), *Proceedings of the 12th European Conference on Soil Mechanics and Geotechnical Engineering* (pp. 1603-1606). Amsterdam, Netherlands. Rotterdam: A.A. Balkema.
- Wang, R., Zhu, C., Wang, J., & Yang, M.Z. (2001). Improvement of soft clays by dynamic compaction with PVDs. In American Society of Civil Engineers (ASCE) (Org.), *GeoDenver 2000* (pp. 248-257). American Society of Civil Engineers (ASCE).
- Zetas. (2015). *Dynamic replacement*. Zetas Zemin Teknolojisi A.Ş. Retrieved in April 21, 2015, from <https://www.zetas.com.tr/en/offer-portfolio/dynamic-replacement/>