Chang et al. (2011) suggest the following quality control and assurance procedure:

- Ninety percent or more of the individual construction area shall meet the optimum pass requirement and 70% of the target ICMV determined from the test sections.
- Rework and reevaluation are necessary for the construction area not meeting the ICMV criterion prior to continuing with the production in that area.

3.5 DEEP DYNAMIC COMPACTION

3.5.1 Introduction

Basic Concept Deep dynamic compaction is to repeatedly drop a weight ("tamper") freely from a height onto the ground surface in a pattern to compact problematic geomaterial to a deep depth as shown in Figure 3.16. Repeated impacts reduce voids, densify the geomaterial, and induce ground movement. A tamper typically has a weight of 5–40 tons and drops from a height of 10–40 m. Different from shallow compaction, deep dynamic compaction can compact problematic geomaterial down to a depth of 10 m. The concept of dynamic compaction can be traced back to Roman times. The modern technology has been credited to the French engineer, Louis Menard since 1960s. The article published by Menard and Broise in 1975 provided theoretical bases for dynamic consolidation of fine-grained soils by heavy tamping.

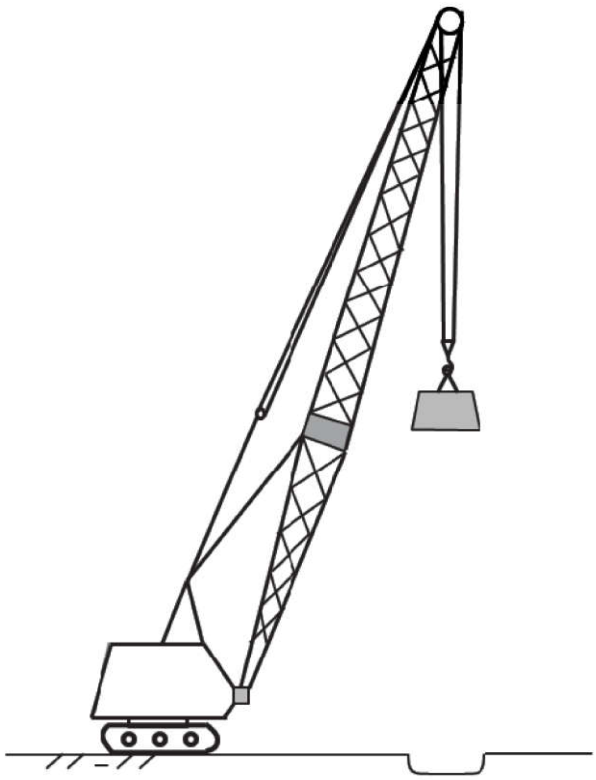


Figure 3.16 Dynamic compaction.

Suitability Deep dynamic compaction is suitable for the following conditions:

- Loose and partially saturated fills
- Saturated free-drained soils
- Silts with plasticity index less than 8
- Clayey soil with a low degree of saturation (moisture content lower than plastic limit)

Deep dynamic compaction is generally not recommended for clayey soil with high plasticity index (greater than 8) and high degree of saturation. However, this method has been used to improve clayey soils in some countries (Han, 1998; Liang and Xu, 2011). Drainage and/or dewatering are often required to reduce excess pore water pressure in clayey soil generated by deep dynamic compaction. A certain waiting period is necessary for the dissipation of excess pore water pressure. High groundwater table (within 2 m of the starting level) minimizes the effectiveness of dynamic compaction. Under such a condition, dewatering may be necessary. Deep dynamic compaction is more economic when the area of a site is larger than 5000 m^2. Due to the size of a crane for deep dynamic compaction operation, certain clearance is necessary. Table 3.8 lists adverse situations for dynamic compaction.

Applications Deep dynamic compaction has been used to improve problematic geomaterials by increasing bearing capacity, reducing settlement, minimizing collapsible potential, and mitigating liquefaction for commercial and residential buildings, storage tanks, highways and railways, airports, and harbors.

Advantages and Limitations Deep dynamic compaction can improve a large area of geomaterials in a relatively short time at low cost. It is very effective to densify loose and partially saturated fill with less than 15% fines. This method often can detect weak or loose areas during operation so that they can be treated properly, such as overexcavation and replacement. Dynamic compaction can change a heterogeneous geomaterial to a more uniform, denser, and stronger material. The major equipment needed for this method is a crane and a tamper, which are readily available from many contractors.

Deep dynamic compaction is generally less to not effective to improve saturated clayey soils. Special measures have to be taken for this method to be reasonably effective for

Table 3.8 Adverse Situations for Dynamic Compaction

Adverse Situation	Possible Difficulty
Soft clays (undrained shear strength less than 30 kPa)	Insufficient resistance to transmit tamper impulse
High groundwater level	Need to dewater and to consider possible effects of subsequent recovery in water level
Vibration effects (may be worse if groundwater level is high)	Distance from closest structure to be of the order of 30 m or more
Clay surface	May be inadequate for heavy cranes and unsuitable for imprint backfilling
Clay fills	May be subject to collapse settlement if inundated later
Flying debris	Precautions for site and public safety
Voided ground or Karst features below treated ground	Treatment may not reach the voided zone or may make it less stable
Biologically degrading material	Compaction may create anaerobic conditions and regenerate or change the seat of the biological degradation

Source: Mitchell and Jardine (2002).

these soils, such as providing drainage and/or dewatering and having a long waiting period for dissipation of excess pore water pressure. Impact by deep dynamic compaction induces noise, vibration, and lateral movement, which may cause problems to nearby buildings, substructures, and utility lines. This method often requires instrumentations to monitor vibration, noise level, and ground movement. When it is used in saturated clayey soils, piezometers are needed to monitor generation and dissipation of excess pore water pressure. The tamping work may cause flying debris, which poses danger to workers onsite. The mobilization cost may be high when large crane and tamper are used.

3.5.2 Principles

Dynamic Densification When dynamic compaction is used on unsaturated granular geomaterial, the impact by a heavy tamper immediately displaces particles to a denser state, compresses or expels air out of voids, and reduces the volume of voids. Under such a condition, typically there is ground depression without any ground heave. A hard plug is formed under the tamper (Moseley and Kirsch, 2004).

Dynamic Consolidation The theory of dynamic consolidation was proposed by Menard and Broise (1975) to explain why saturated fine-grained soil can also be improved by repeatedly dropping a heave tamper. They attributed dynamic consolidation to four main mechanisms: (1) compressibility of saturated soil, (2) liquefaction, (3) change of permeability, and (4) thixotropic recovery. Repeated impacts do not necessarily always liquefy fine-grained soils, instead, they generate excess pore water pressure, which can accumulate under repeated loading. The accumulated excess pore water pressure starts to dissipate once tamping stops. Therefore, it is more appropriate to refer this mechanism to the generation and dissipation of excess pore water pressure.

1. Compressibility of Saturated Soil It is common knowledge that saturated fine-grained soil is incompressible and cannot have volume change under immediate loading (i.e., an undrained condition). Menard and Broise (1975) attributed the immediate volume change of saturated fine-grained soil to the existence of microbubbles in most quaternary soils ranging from 1 to 4%.

2. Generation and Dissipation of Excess Pore Water Pressure As mentioned above, dynamic compaction induces excess pore water pressure during the operation. A waiting period is necessary to dissipate the excess pore water pressure. The dissipation of excess pore water pressure is a consolidation process, which can induce settlement and compress the soil. Due to the low permeability of fine-grained soils, prefabricated vertical drains are often installed to accelerate the dissipation. Alternatively, vacuum dewatering can be applied through preinstalled vertical vacuum pipes and horizontal drainage pipes to lower groundwater table and reduce excess pore water pressure (Liang and Xu, 2011). Liang and Xu (2011) indicated that this method is suitable for fine-grained soil with permeability greater than 5×10^{-9} m/s.

3. Change of Permeability Under high-energy tamping, vertical fissures are generated around the impact points. These vertical fissures significantly increase the permeability of the fine-grained soil, which also accelerates the dissipation of excess pore water pressure and consolidation.

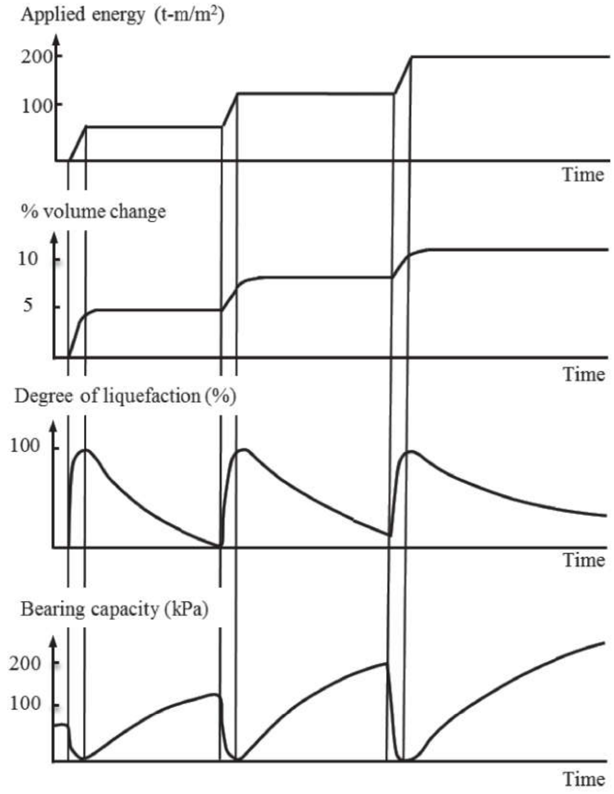


Figure 3.17 Variations of volume, excess pore water pressure, and soil strength during and after the tamping process (after Menerd and Broise, 1975).

4. Thixotropic Recovery Due to the disturbance of fine-grained soil caused by tamping, it degrades and reduces its strength. This strength regains with time due to the thixotropic recovery. This is also the reason why fine-grained soils should be evaluated at least 30 days after tamping.

The changes of volume, excess pore water pressure, and soil strength during and after tamping are illustrated in Figure 3.17.

Dynamic Replacement When a clayey soil is too soft and has too low permeability, it is not effective to be densified or consolidated during and after tamping. Instead of improving the soil, the soil can be displaced by tamping and replaced by stones or coarse aggregates. The process of dynamic replacement involves tamping, backfilling, and continued tamping until stone columns are formed, as shown in Figure 3.18. The design of dynamic replacement is similar to that for stone columns installed by a vibro-probe or casing method to be discussed in Chapter 5. Therefore, this method will not be discussed further in this chapter.

3.5.3 Design Considerations

Site Investigation Before the design of deep dynamic compaction, a geotechnical investigation is required to evaluate the site conditions, which include:

- Geomaterial profiles including geomaterial type, particle size, fine content, degree of saturation, and Atterberg limits
- Relative density of cohesionless geomaterial
- Groundwater level
- Possible voids
- Possible presence of hard lenses within the depth of improvement
- Possible sensitive soil

Influence Factors The design of deep dynamic compaction should consider the following influence factors:

- Geomaterial type
- Depth and area of improvement
- Tamper geometry and weight
- Drop height and energy
- Pattern and spacing of drops
- Depth of crater
- Number of drops and passes
- Degree of improvement
- Induced settlement
- Environmental impact (vibration, noise, and lateral ground movement)

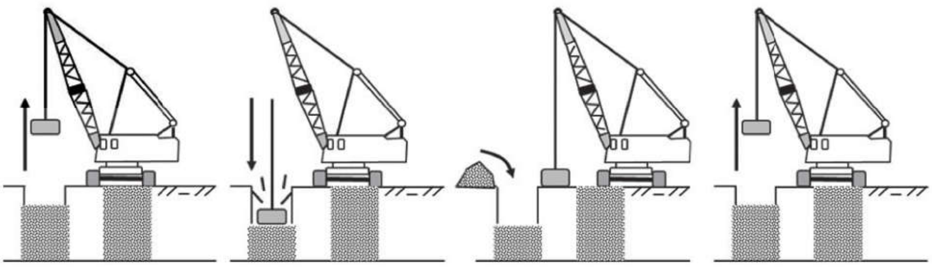


Figure 3.18 Dynamic replacement (after Yee and Ooi, 2010).

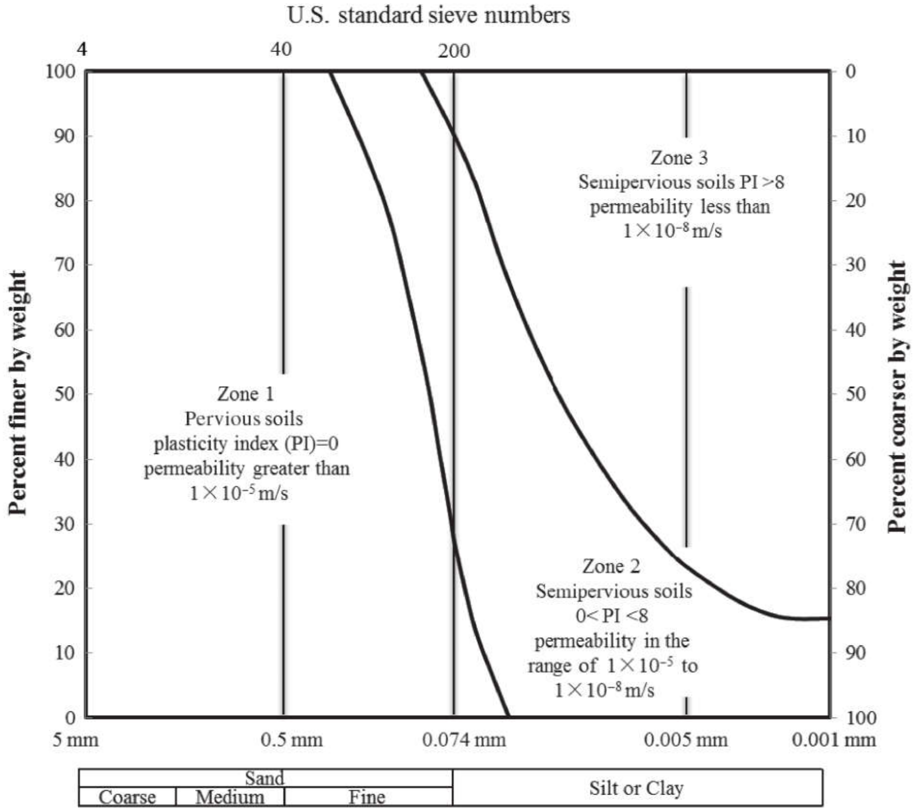


Figure 3.19 Soil types for dynamic compaction (Lukas, 1995).

- Presence of soft layer
- Presence of hard layer
- High groundwater table
- Elapsed time
- Pilot trial

Soil Type Lukas (1995) defined three types of soil that are suitable for dynamic compaction: (1) previous soil deposits—granular soil, (2) semipervious deposit—primary silts with plasticity index less than 8, and (3) semipervious deposit—primary clayey soil with plasticity index greater than 8. The gradations of these soils are presented in Figure 3.19.

Depth and Area of Improvement Depth of improvement depends on project requirements for desired performance. For example, a loose and saturated sand layer, susceptible to liquefaction, should be improved to the depth below which no liquefaction will occur. An empirical formula developed based on field data is available to estimate the depth of improvement as follows:

$$D_i = n_c \sqrt{W_t H_d} \tag{3.16}$$

where D_i = depth of improvement (m)
W_t = weight of tamper (ton)
H_d = height of drop (m)
n_c = constant, depending on soil type, degree of saturation, and speed of drop

It should be pointed out that the above formula is units dependent. The specific units as noted in the definitions should be used. Table 3.9 provides the recommended n_c values.

Field data show that the depths of improvement for granular soils are mostly up to 10 m while those for cohesive soils and clay fills are limited to 5 m.

The area of the improvement should be that beyond the area of loading with a distance equal to the depth of improvement on each side.

Tamper Geometry and Weight Most tampers are made of steel or steel shell infilled with sand or concrete and have a circular or square base with an area of 3–6 m^2 or larger. Tampers with smaller base areas (3–4 m^2) are commonly used for granular soils while those with large base areas (larger than 6 m^2) are used for cohesive soils. The weight of a tamper typically ranges from 5 to 40 tons.

Table 3.9 Recommended n_c Value

Soil Type[a]	Degree of Saturation	n_c
Pervious soil deposits—granular soils	High	0.5
	Low	0.5–0.6
Semipervious deposits—primary silts with PI < 8	High	0.35–0.4
	Low	0.4–0.5
Semipervious deposits—primary clayey soils with PI > 8	High	Not recommended
	Low (w < PL)	0.35–0.4

[a]PI = plasticity index, w = moisture content, and PL = plastic limit. For $W_t H_d$ = 1–3 MJ/m^2 and a tamper drop using a single cable.
Souce: Lukas (1995).

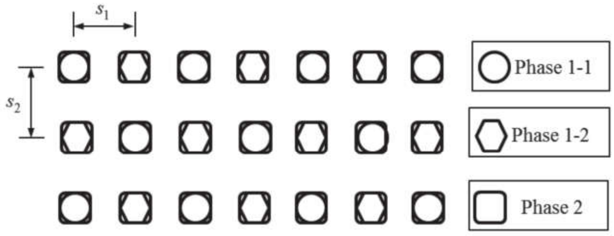


Figure 3.20 Layout of drop points.

Drop Height and Energy The height of tamper drop is typically 10–40 m. Based on Mayne et al. (1984), the energy per drop in practice mostly ranges from 800 to 8000 kN·m. Mayne et al. (1984) also provided a chart of relationship between weight of tamper and drop height based on field data. This relationship can be approximately expressed as follows:

$$H_d = (W_t H_d)^{0.54} \tag{3.17}$$

where $W_t H_d$ = energy per drop of tamper (ton-m), which is determined from Equation (3.16) based on the required depth of improvement.

The calculated drop height may be adjusted based on the available tamper owned by the contractor.

Pattern and Spacing of Drops Square and triangular patterns of drops are commonly used. Often both patterns are used on the same job to accommodate different passes of compaction in the same phase. Figure 3.20 shows a typical layout of drop points in two primary phases (phase 1 and phase 2). Phase 1 has two passes (also called as phase 1-1 and phase 1-2 in Figure 3.20). Compaction often starts with drop points at larger spacing (e.g., phase 1-1 and phase 1-2), which are to densify deeper soil layers. Drop points at smaller spacing (e.g., phase 2) are to densify shallower soil layers. The secondary phase of compaction uses a lower energy tamper to cover the whole site. This compaction technique is also called ironing compaction. The depth of densification by this dynamic compaction sequence and the change of geomaterial property are illustrated in Figure 3.21. During phase 1-1, only the deeper geomaterial is densified. Phase 1-2 further densifies the deeper geomaterial. Phase 2 densifies the geomaterial within the intermediate depth. During the two phases of compaction, surface deposit is often loosened to the depth of crater penetration due to low overburden stresses. Ironing phase with lower energy is to densify the loosened deposit. Lukas (1995) indicated that the maximum improvement usually occurs between $D_i/3$ to $D_i/2$ (D_i is the maximum depth of improvement). Spacing of drop points (s_1 or s_2) is commonly selected to be 1.5–2.5 times the diameter or width of a tamper (s_1 and s_2 are often equal to create uniform compaction).

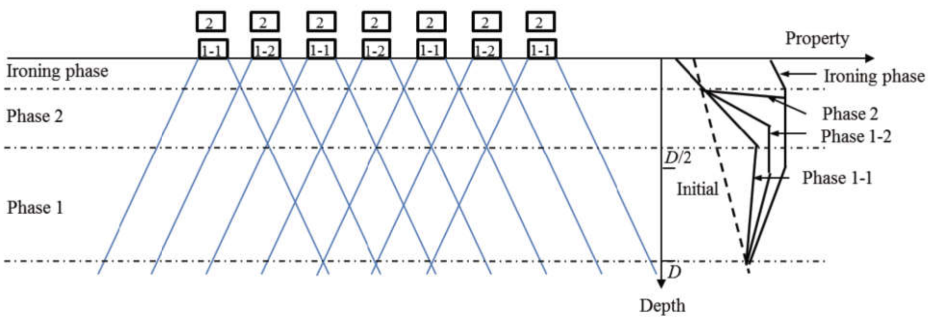


Figure 3.21 Depth of densification and property change (modified from Woodward, 2005).

Depth of Crater A crater is formed under each tamper drop and its depth increases with the number of drops. High-energy compaction can induce a crater of 1.0–1.5 m deep. The crater depth should be limited to the height of a tamper plus 0.3 m to ensure the safety and ease of compaction operation. When the crater depth gets too deep, the compaction operation should be divided into two or multiple phases. Rollins and Kim (2010) proposed empirical formulas to estimate crater depth, d_{cd}, in soils with a low degree of saturation after dynamic compaction:

For a rough estimate

$$d_{cd} = 0.028 N_d^{0.55} \sqrt{W_t H_d} \tag{3.18}$$

For a more accurate estimate

$$\log d_{cd} = -1.42 + 0.553 \log N_d + 0.213 \log H_d + 0.873 \log W_t - 0.435 \log\left(\frac{s_d}{d_t}\right) - 0.118 \log p \tag{3.19}$$

where H_d = drop height (m)
W_t = tamper weight (tons)
N_d = number of drops
s_d = drop spacing (m)
d_t = tamper width or diameter (m)
p = contact pressure in (t /m^2)

Dynamic compaction on soil with a high degree of saturation would result in deeper crater depth.

Number of Drops and Passes The number of drops and passes can be estimated based on applied energy on a site. Applied energy (AE) at each drop point location can be calculated as follows:

$$\text{AE} = \frac{N_d W_t H_d}{A_e} \tag{3.20}$$

where N_d = number of drops by one pass at each drop location (typically 5–10 drops)
W_t = weight of tamper
H_d = drop height
A_e = influence (equivalent) area of each impact point ($A_e = s^2$ for a square pattern or 0.867 s^2 for an equilateral triangular pattern)
s = drop spacing

Total applied energy is the sum of the energy applied during high-energy passes plus ironing pass. Unit applied energy (UAE) is defined based on the depth of improvement as follows:

$$\text{UAE} = \frac{\text{AE}_{\text{total}}}{D_i} = \frac{\text{AE}_{\text{HEP}} N_p + \text{AE}_{\text{IP}}}{D_i} \tag{3.21}$$

where AE_{HEP} = applied energy by a high-energy pass
AE_{IP} = applied energy by an ironing pass
N_p = number of passes
D_i = depth of improvement

Lukas (1995) provided the guidelines for required UAE based on soil type as shown in Table 3.10.

Ironing pass is mainly used to compact loosened soil within the depth of craters. The required applied energy for ironing compaction is estimated as follows:

$$\text{AE}_{\text{IP}} = \text{UAE} \cdot d_{cd} \tag{3.22}$$

where d_{cd} is the depth of the crater. The number of drops for ironing pass can be determined using Equation (3.20) if the weight and drop height of the tamper and the area of the tamper (i.e. the influence area of each impact point) are known.

From the required UAE in Table 3.10 and the AE_{IP}, in Equation (3.22) the total applied energy required for high-energy compaction can be calculated using Equation (3.21) and the number of drops can be calculated using Equation (3.20) if one pass is assumed. When the number of drops at one location in a single pass is too large (greater than 10 passes) or the crater depth is too deep, the operation of compaction should be divided into two or multiple passes.

The number of drops can be determined through trial tamping work onsite. The Chinese *Ground Improvement Technical Code* (China Academy of Building Research, 2000) sets the following criteria to determine the number of drops from trial tamping work:

- The average vertical displacement induced by the last two drops is not greater than 50 mm. When high drop energy is used, it should not be greater than 100 mm.
- No large heave occurs around the crater.
- The crater should not be so deep that lifting of the tamper becomes difficult.

Table 3.10 Required Unit Applied Energy[a]

Soil Type	Unit Applied Energy (kJ/m^3)	% Standard Proctor Energy
Pervious coarse-grained soil	200–250	33–41
Semi-impervious fine-grained soil	250–350	41–60
Landfill	600–1100	100–180

[a]Standard Proctor energy equals 600 kJ/m^3.
Source: Lukas (1995).

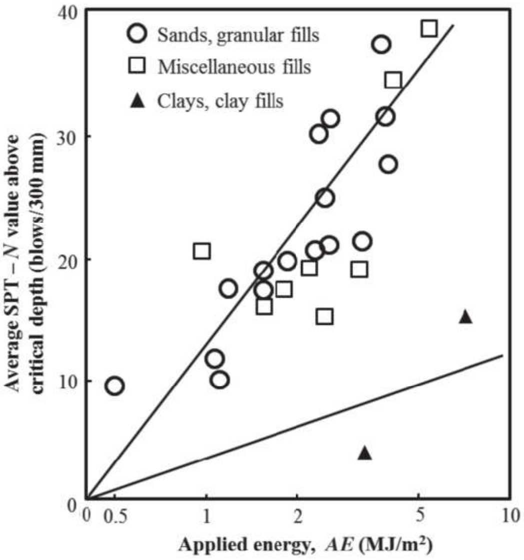


Figure 3.22 Average SPT N value after improvement (after Lukas, 1995).

Degree of Improvement The degree of improvement depends on geomaterial type, fine content, groundwater table, applied energy, drop layout, and time. Figures 3.22, 3.23, and 3.24 show the average SPT N values, CPT tip resistance, and pressuremeter (PML) limit pressure above the improvement depth. Table 3.11 provides upper bound test values after dynamic compaction. These figures and table can be used as target values for dynamic compaction preliminary design. The actual degree of improvement should be evaluated by in situ testing after compaction.

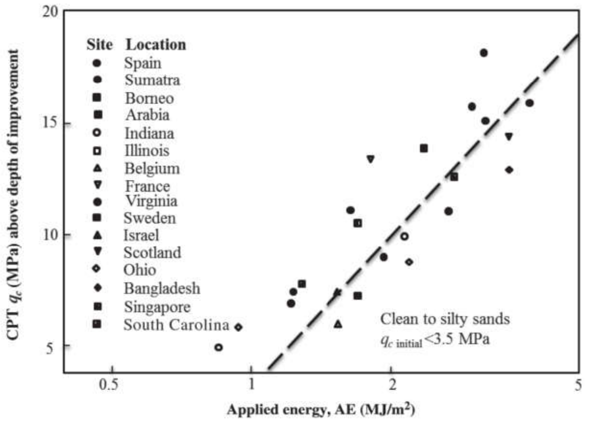


Figure 3.23 Average CPT q_c value after improvement (Lukas, 1995).

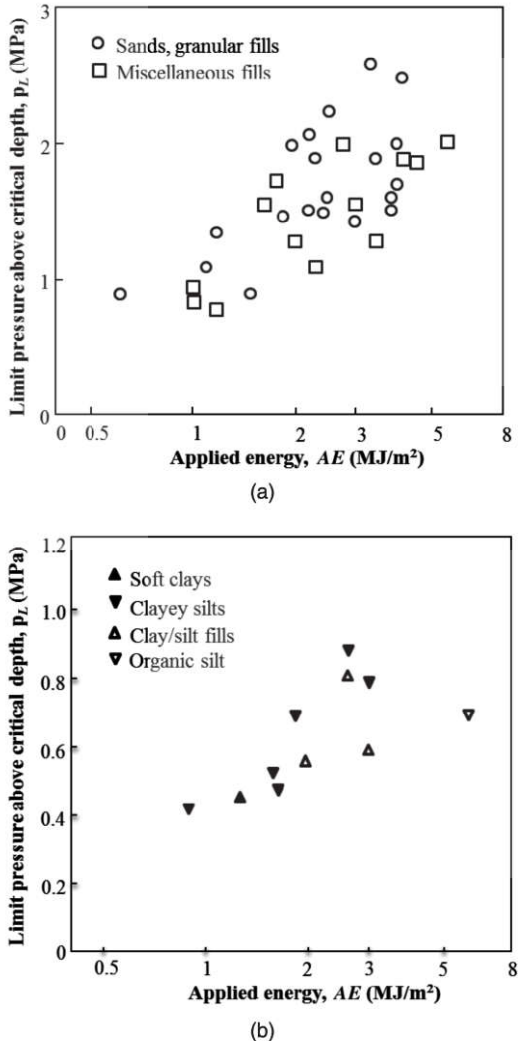


Figure 3.24 Average PMT p_L value after improvement (Lukas, 1995).

Induced Settlement After each pass of dynamic compaction, construction equipment, most commonly bulldozers, is used to level the ground surface. Ground settlement (also called subsidence) is measured based on the current ground elevation as compared with the initial elevation. In unsaturated soil, the settlement occurs immediately after compaction. In saturated soil, however, the settlement increases gradually with time after the initial compression under each compaction. Most of the settlement results from filling large craters induced by tampers. The approximate induced settlement as percent of improvement depth is provided in Table 3.12.

The induced settlement can also be estimated based on the crater depth (see the design example in Section 3.5.5).

Table 3.11 Upper Bound Test Values after Dynamic Compaction

	SPT N (blows/0.3 m)	CPT q_c (MPa)	PMT p_L (MPa)
Previous coarse-grained soil:			
Sands and gravels	40–50	19–29	1.9–2.4
Semipervious soil:			
Sands and gravels	40–50	19–29	1.9–2.4
Silts and clayey silts	25–35	10–13	1.0–1.4
Partially saturated impervious deposits			
Clay fill and mine spoil	30–40	NA	1.4–1.9
Landfills	20–40	NA	0.5–1.0

Source: Lukas (1995).

Table 3.12 Approximate Induced Settlement as Percent of Improvement Depth

Soil Type	Percent of Depth
Natural clays	1–3
Clay fills	3–5
Natural sands	3–10
Granular fills	5–15
Uncontrolled fills	5–20

Source: Modified from Moseley and Kirsch (2004).

Environmental Impact It is expected that applying high-energy impact on ground induces environmental impact, mostly vibration, noise, and lateral ground movement. This fact has to be considered in the selection of a suitable ground improvement technique.

Field measurements show that particle velocity depends on the scaled energy factor and the geomaterial density as shown in Figure 3.25. The scaled energy factor is defined in terms of the applied energy by a single drop and the distance from the point of impact to the point of interest. An increase of the scaled energy factor increases the particle velocity. A loose soil or fill typically generates lower particle velocity. Lukas (1995) indicated that the frequency of ground vibrations induced by dynamic compaction ranges from 6 to 10 Hz. Mayne et al. (1984) provided the following formula to estimate the upper limit of peak particle velocity (PPV) in terms of applied single-drop energy and distance to the drop point:

$$\mathrm{PPV} = 70\left(\frac{\sqrt{W_t H_d}}{x_{dp}}\right)^{1.4} \tag{3.23}$$

where PPV = peak particle velocity (mm/s)
W_t = tamper weight (ton)
H_d = drop height (m)
x_{dp} = distance to the drop point (m)

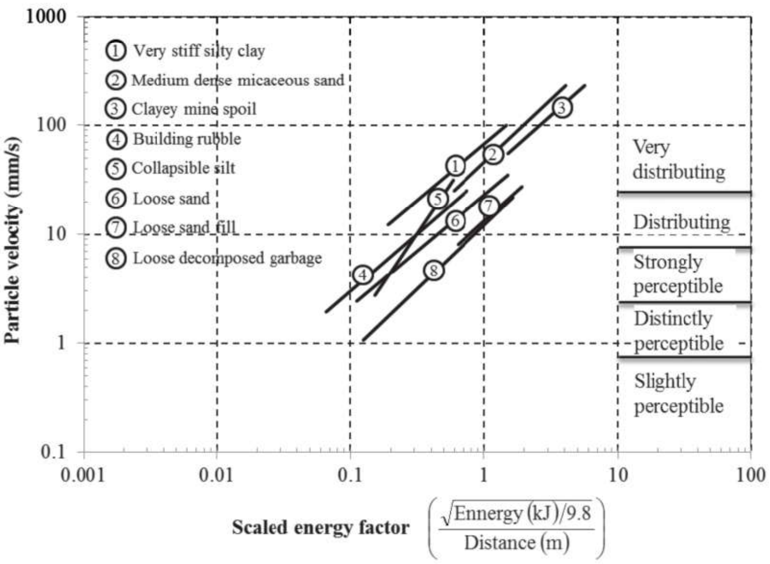


Figure 3.25 Scaled energy factor versus particle velocity (FHWA, 1986).

Table 3.13 Typical Threshold Particle Velocity

Structural Type	Velocity (mm/s)
Commercial, industrial	20–40
Residential	5–15
Sensitive	3–5

Siskind et al. (1980) suggested that threshold particle velocity depends on type of structure and frequency of vibration. Different countries have established different guidelines for threshold particle velocity for vibration-induced damage to buildings. Table 3.13 provides typical values of threshold particle velocity based on the typical vibration frequency generated by deep dynamic compaction. Moseley and Kirsch (2004) suggested that the particle velocity at 2.5 mm/s would annoy occupants of buildings. To reduce particle velocity induced by dynamic compaction, isolation trenches have been commonly used around a construction site to cut off or minimize surface waves (i.e., the Rayleigh waves). Trenches should be 2–3 m deep and at least 1 m wide at the bottom.

Dynamic compaction also induces lateral ground movement, which may damage existing underground utility lines or buried structures. The Federal Highway Administration (FHWA) (1986) documented that lateral ground movement at a magnitude of 19–76 mm was measured at a distance of 6.1 m from the point of impact. Field monitoring of ground vibrations and movement is necessary if there are nearby buildings and/or underground utility lines and structures.

Presence of Soft Layer When a soft layer exists near the ground surface, it may not be able to support the equipment for dynamic compaction operation or absorb applied energy so that limited energy is transmitted to soil at depth. Under such a condition, this soft layer should be excavated or stabilized by a stabilizing layer (typically 0.3–1.2 m thick) to provide a stable working platform for dynamic compaction equipment and to limit crater depth. The most favorable material for the stabilizing layer is a coarse-grained geomaterial, such as gravel, crushed stone, or building rubble. An extra-thick stabilizing layer reduces the depth of improvement below the stabilizing layer; therefore, it should be avoided.

Presence of Hard Layer When a hard layer to a certain thickness (1–2 m) exists near the ground surface, it distributes the applied energy over a wide area so that the energy transmitted to the depth is greatly reduced. As a result, the depth and degree of improvement are reduced. Under such a condition, the hard layer should be removed or loosened. When a hard layer is thin, however, a tamper may penetrate this layer and deliver proper energy to the underlying layer.

High Groundwater Table It is a general requirement for dynamic compaction that the groundwater table should be 2 m below the ground surface. When the groundwater table is within 2 m, dynamic compaction likely encounters some difficulties. Typically, a crater depth ranges from 1.0 to 1.5 m. Dynamic compaction generates excess pore water pressure so that the groundwater rises and enters the craters. The geomaterial and water can be intermixed during compaction. To avoid such a problem, the groundwater table should be lowered by dewatering or additional fill should be added to increase the distance from the ground surface to the groundwater table.

Elapsed Time Dynamic compaction induces excess pore water pressure if the geomaterial is saturated. The excess pore water pressure can accumulate under multiple drops of impact if the geomaterial is not pervious. The accumulated excess pore water pressure reduces geomaterial strength, destabilizes the ground, and minimizes the densification. Under such a condition, the number of drops in each phase or pass should be limited. An elapsed time is needed between two phases or passes to allow the dissipation of the excess pore water pressure. For sandy soil, the dissipation of the excess pore water pressure is rapid and can complete within a few minutes. However, the geomaterial with fine contents may take a few days to weeks to dissipate the excess pore water pressure. To shorten the time for pore water pressure dissipation, prefabricated vertical drains or vacuum dewatering have been used. The details about the function and design of prefabricated vertical drains and vacuum dewatering can be found in Chapters 7 and 6, respectively.

3.5.4 Design Parameters and Procedure

The influence factors discussed above are the design parameters for deep dynamic compaction. The following procedure may be followed for design of deep dynamic compaction:

1. Based on geotechnical profile and potential problem, select the depth of improvement.
2. Based on geomaterial type and degree of saturation, select the n_c value from Table 3.9.
3. Calculate the required energy per blow for the high-energy impact using Equation (3.16) based on the required depth of improvement.
4. Estimate the drop height using Equation (3.17) and then the tamper weight.
5. Based on the applied energy guidelines, the unit applied energy can be selected based on the geomaterial type using Table 3.10.
6. Calculate the required total applied energy using Equation (3.21).
7. Based on the geomaterial type and degree of saturation near the ground surface, the required unit applied energy for the ironing pass can be selected using Table 3.10.

8. Calculate the required applied energy for the ironing pass using Equation (3.22) with an assumed crater depth (typically 1.0–1.5 m).
9. Calculate the required total applied energy for high-energy compaction by subtracting the required applied energy for the ironing pass from the total required applied energy.
10. Based on the tamper diameter, estimate the spacing of drops.
11. Based on the required total applied energy for high-energy compaction and the spacing of drops, calculate the required number of drops (round up to an integer number). If the required number of drops on one location is greater than 10, multiple passes or phases are required.
12. Estimate the crater depth using Equation (3.18) or (3.19).
13. Select target performance values after improvement.
14. Estimate the settlement after improvement based on Table 3.12 or crater depth.

3.5.5 Design Example

A 5-m-high highway embankment is to be constructed over a landfill that has a fine-grained soil cover underlain by a soil mixture with the total thickness ranging from 5.0 to 8.2 m. This soil mixture includes primarily silts and clays with construction waste (concrete blocks, brick fragments, etc.). At certain locations, there are voids and loose pockets within the landfill. Standard penetration tests performed prior to ground improvement indicate SPT values ranging from about 5 to 20 with an average of 10. The predicted settlement ranged from 140 to 274 mm. Dynamic compaction is selected to reduce the anticipated total and differential settlements. The required SPT N value after improvement should be at least 20. The surface of the landfill is strong enough to support the dynamic compaction equipment. Leachate inside the landfill was at a relatively shallow depth (approximately 2.5 m from the existing surface). To minimize the generation of excess pore water pressure, multiple pass construction may be needed. The contractor has an 18.2-ton tamper that has the diameter of 1.5 m and the height of 1.5 m.

You are requested to provide a preliminary design for the dynamic compaction project and estimate the settlement after compaction.

Solution

Considering the thickness of the landfill typically ranging from 5.0 to 8.2 m, the depth of improvement is selected as 8.2 m. Based on the composition of the landfill, it can be considered as a semipervious soil deposit. Since the landfill has a high degree of saturation, the n_c value is selected as 0.35. As a result, the required energy per blow can be computed as follows:

$$W_t H_d = \left(\frac{8.2}{0.35}\right)^2 = 550\ t\text{-}m$$

The contractor provided an 18.2-t tamper, therefore, the required drop height is 550 t-m/18.2 t = 30.2 m. Based on Equation (3.17), the estimated drop height is also 30.2 m.

Based on the applied energy guidelines, the unit applied energy for landfills ranges from 600 to 1100 kJ/m^3. The average unit applied energy is 850 kJ/m^3, therefore, the required total applied energy is $\mathrm{AE}_{\mathrm{total}}$ = 850 kJ/m^3 × 8.2 m = 6970 kJ/m^2 = 6.97 MJ/m^2.

Ironing passes are typically used to compact the geomaterial near the surface, which is close to the depth of the craters. Typically, the crater depth ranges from 1.0 to 1.5 m. The geomaterial above the landfill is most likely fine grained. Since the geomaterial near the surface is above the groundwater table, the unit applied energy for the semipervious fine-grained soils of 300 kJ/m^3 may be used for the ironing passes. Therefore, the required total applied energy for ironing passes is $\mathrm{AE}_{\mathrm{IP}}$ = 300 kJ/m^3 × 1.5 m = 450 kJ/m^2 = 0.45 MJ/m^2. The required total applied energy for high-energy compaction is $\mathrm{AE}_{\mathrm{HEP}}\ N_p$ = 6.97 MJ/m^2 – 0.45 MJ/m^2 = 6.52 MJ/m^2.

To allow for pore water pressure dissipation during energy application, multiple passes are needed. Assume two passes are adopted. The required applied energy for each pass is $\mathrm{AE}_{\mathrm{HEP}}$ = 6.52 MJ/m^2/2 = 3.26 MJ/m^2.

Typical drop spacing is 1½–2½ times the tamper diameter. The factor of 2.0 is selected for this site, that is, drop spacing = 2.0 × 1.5 m = 3.0 m (assuming a square pattern). The number of drops at each specific drop point location can be computed by

$$N_d = \frac{\mathrm{AE}_{\mathrm{HEP}} \times (A_e)}{W_t H_d} = \frac{3260\,\mathrm{kJ/m^2} \times (3.0\,\mathrm{m})^2}{18.2\mathrm{t} \times 30.2\,\mathrm{m} \times 10\,\mathrm{m/s^2}} = 5.3$$

Select the number of the drops for each pass at 6. Example Figure 3.2 depicts the layout of tamper drops.

For the number of drops at one location at 6 for each pass, the crater depth can be estimated as follows:

$$d_{cd} = 0.075\sqrt{W_t H_d} = 0.075\sqrt{550} = 1.75\,\mathrm{m}$$

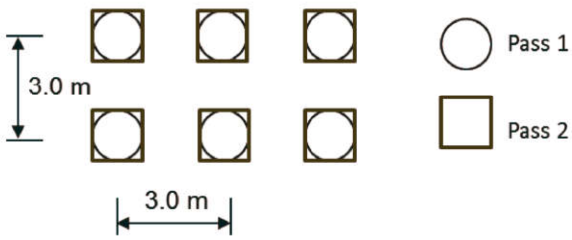
The allowable crater depth for construction is 1.5 + 0.3 = 1.8 m, which is the same as the estimated crater depth expected in the field; therefore, it is OK.

Based on the FHWA guidelines, the upper bound of SPT N value after dynamic compaction ranges from 20 to 40.

The induced settlement for uncontrolled fill ranges from 5 to 20%. If the average percentage (i.e., 13%) is considered, the possible induced settlement is $0.13 \times 8.2\ \text{m} = 1.10\ \text{m}$. However, based on the estimated crater depth, the expected settlement may be estimated as follows (assume the crater diameter is the same as the tamper diameter and no heave). The area ratio of improvement, defined as the area of each crater to the influence area of each tamping point, is:

$$\text{Area ratio of improvement} = \frac{3.14 \times (1.5/2)^2}{3.0^2} = 0.20$$

the induced settlement by two passes of dynamic compaction $= 2 \times 0.20 \times 1.75 = 0.70$ m. If heave is considered, the induced settlement will be smaller.



Example Figure 3.2 Layout of tamper drops.

3.5.6 Construction

Before construction, the equipment used for lifting and dropping a tamper should be selected based on the weight of the tamper. FHWA (1986) provided a guideline for the selection of equipment for different tamper weights as shown in Table 3.14. A conventional crawler crane with a single cable and a free spool is typically sufficient for a tamper weighing up to 220 kN. For heavier tampers, the crawler crane should be reinforced with stronger components.

A typical flow of tamping work is shown in Figure 3.26 and its detailed steps are as follows:

Table 3.14 Required Equipment for Different Tamper Weights

Tamper Weight (kN)	Crawler Crane Capacity (kN)	Cable Size (mm)
50–70	360–440	19–22
70–130	440–890	22–25
130–160	890–1100	25–29
160–220	1300–1600	32–38

Source: Modified from FHWA (1986).

1. Prepare a site by removing large objects (e.g., trees), leveling the ground, dewatering, and filling existing ponds and local depressed area. If the groundwater is within 2 m from the ground surface, it should be lowered by dewatering or additional fill is placed. If the surface soil is too weak to support equipment, a construction platform should be constructed first.
2. If there are nearby existing structures or utility lines, an isolation trench is required to minimize vibration and lateral movement. Trench should be at least 2–3 m deep and 1 m wide at the bottom of the trench.
3. Place stakes at the locations for the centers of all the drop points for each pass and survey the ground elevations.
4. Position the equipment and move the tamper right above the drop point.
5. Survey the top elevation of the tamper on the ground.
6. Lift the tamper to the desired height and then let it drop freely onto the ground. Survey the top elevation while the tamper is still in the crater. Alternatively, measure the dimensions of the crater after removing the tamper. If the tamper is tilted after reaching the ground, level the base of the crater after removing the tamper.
7. Repeat step 6 until the number of drops on one tamping point reaches the target value and other criteria are met. Move to the next tamping point.
8. Repeat steps 4–7 until all the tamping points are complete for the first pass.
9. Use bulldozers to level the ground and measure the ground elevation. The difference between the current elevation and the previous elevation is the induced settlement.
10. After an elapsed time depending on geomaterial and groundwater conditions, repeat steps 3–8 until all the tamping points are complete for the next passes if needed.
11. Apply ironing tamping over the whole compaction area.

3.5.7 Quality Control and Assurance

Before any tamping work, the height of drop and locations of drop points should be verified. During field tamping operation, it is important to have monitoring and close visual observations. Adjustments may be made based on monitoring and observations. For example, if one drop location has a much deeper crater depth than other locations, this is an indication that much weaker geomaterial exists at that location. Special measures may be necessary to improve this area, such as overexcavation and replacement. If additional tamping induces large heave around the crater, this is an indication that further densification is not effective so that the tamping should be suspended or terminated at this location. Common field monitoring includes piezometers in

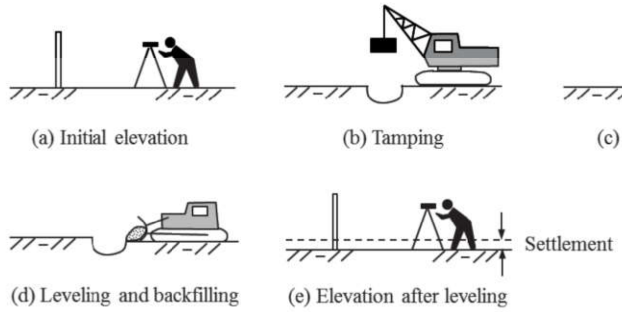


Figure 3.26 Flow of tamping work: (a) initial elevation, (b) tamping, (c) crater depth, (d) leveling and backfilling, and (e) elevation after leveling.

saturated fine-grained geomaterial, inclinometer casings for lateral movement, and accelerometers for ground vibrations.

After the completion of the tamping work, field explorations should be conducted to evaluate or verify the degree and depth of improvement. Depending on geomaterial type and groundwater level, for coarse-grained geomaterial, the field evaluation should be performed at least in 1–2 weeks after the completion of tamping; for fine-grained geomaterial, the evaluation should be performed at least in 3–4 weeks after the completion of tamping. Field explorations include sampling for laboratory tests, SPT, CPT, or PMT. The depth of the test should be below the design depth of improvement. Static plate load tests may be performed in a large project site. Since PMT and plate load tests are more sensitive to the change of soil stiffness than SPT and CPT, they are good methods to be adopted for this purpose.

3.6 RAPID IMPACT COMPACTION

3.6.1 Introduction

Basic Concept Rapid impact compaction is an intermediate compaction method between conventional shallow compaction and deep dynamic compaction. It densifies geomaterial by repeatedly dropping a hydraulic hammer mounted on an excavator at a fast rate as shown in Figure 3.27. The weight of hammer is typically 5–12 tons, which is dropped freely from a height of 1.2 m on a circular steel foot with a diameter of 1.0–1.5 m (the most common one is 1.5 m in diameter). The rapid impact compaction machine can generate 40–60 blows per minute, which is much faster than the deep dynamic compaction machine. There is a monitoring system on the machine to record impact energy and foot penetration. The depth of improvement depends on geomaterial type and properties, groundwater table level, and applied energy. This technology has been adopted since its initial use in the United Kingdom in 1990s (Adam and Paulmichl, 2007). The production rate of each machine is up to 500 m^2 improvement area per day.

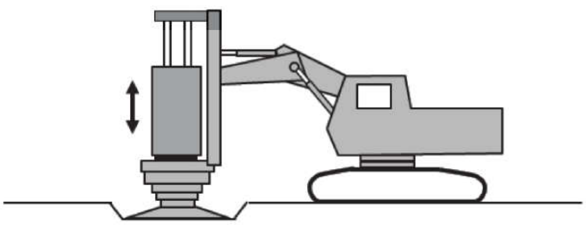


Figure 3.27 Rapid impaction compaction.

Suitability This method is generally suitable for granular geomaterials, including gravel, sands, silts, uncontrolled fills (i.e., a mixture of sand, silt, and clay), and industrial and mine wastes. It has also been successfully used to minimize collapsible potential of loess. This method generally can improve geomaterials up to a depth of 6 m deep (mostly 3–4 m).

Applications Rapid impact compaction has been used for increasing bearing capacity and stiffness of geomaterial to support building foundations, floor slabs, tanks, highways, railways, parking lots, and airport runways, mitigating liquefaction, and reducing waste volume and collapsible potential. It has also been used to compact granular fills in large lifts (up to 3 m).

Advantages and Limitations The operation of rapid impact compaction is fast and under a much controlled manner as compared with deep dynamic compaction. It induces smaller vibrations than deep dynamic compaction due to low impact energy; therefore, it can be operated at closer distances to existing structures. Because the impact foot is always in contact with the ground, it eliminates the risk of generating flying debris. Similar to deep dynamic compaction, it can detect weak areas during the construction. It has better mobility and works in areas with difficult access.

The key limitation for this technology is that the depth of improvement is smaller than that of deep dynamic compaction.