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STATE-OF-THE-ART PAPER

The state of practice of in situ tests for design, quality control and quality assurance of ground improvement works

Serge Varaksin[1] · Babak Hamidi[2]

Abstract
In the state-of-the-art report that was published on ground improvement processes at the 17th ISSMGE conference, ground improvement was defined in five categories. This paper has focused on the ground improvement techniques that either mechanically stabilize the soil or incorporate admixtures or inclusions and the most common in situ geotechnical tests that are used during the geotechnical investigation, quality control and quality assurance phases of these techniques. In addition to the suitability and feasibility of the technique itself, the level of success of any ground improvement program is also related to the applicability and suitability of the criteria that is to be satisfied and the testing campaign that is to be undertaken to verify the works. Experience of the authors indicates that the optimal approach is when acceptance is based on the project's actual geotechnical requirements rather than on minimum test results. At the same time, ground improvement design parameters can only be properly determined when the ground conditions are correctly comprehended, which is possible through meaningful geotechnical investigation. Similarly, applied treatment can only be confidently verified when testing is able to well relate to acceptance criteria. Hence, tests that are able to predict the acceptance criteria without reliance on experimental correlations and published work from other sites will result in the best engineering practice and confidence in results.

Keywords Testing · Quality control · Ground improvement

Introduction

In the state-of-the-art report [27] that was published on ground improvement processes at the 17th International Conference on Soil Mechanics and Geotechnical Engineering in 2009, ground improvement was defined in five categories of which the first three included non-structural techniques, namely (A) ground improvement without admixtures in non-cohesive soils or fill materials, (B) ground improvement without admixtures in cohesive soils, (C) ground improvement with admixtures or inclusions, (D) ground improvement with grouting type admixtures and (E) earth reinforcement. The techniques of these categories have been summarized and briefly described in Table 1.

The focus of this paper is on the state-of-practice of ground treatment techniques and associated in situ geotechnical tests, but it does not intend to provide advice on the choice of technology and methodology. The improvement methods that will be discussed will either be by improving the soil mass' properties or by introducing local inclusions or admixtures into it. Ground improvement and the associated tests for techniques that improve the ground's behavior by stabilizing the entirety of the soil mass by various grouting or mixing methods are not within the scope of this paper.

Geotechnical field testing

Ground improvement design parameters can only be properly determined when the ground conditions are correctly comprehended, which is possible through meaningful geotechnical investigation. Similarly, applied treatment can only be confidently verified when testing is able to well relate to acceptance criteria. Hence, tests that can predict the acceptance criteria without reliance on experimental correlations and published work from other sites will result in the best engineering practice and confidence in results.

✉ Serge Varaksin
s.varaksin@apageo.com

1 Apageo, ISSMGE TC-211, Paris, France

2 Menard Oceania, ISSMGE TC-211, Sydney, Australia

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Table 1 Non-structural ground improvement techniques without or with non-grout inclusions [27]

Category	Method	Principle
A. Ground improvement without admixtures in non-cohesive soils or fill materials	A1. Dynamic compaction	Densification of granular soil by dropping a heavy weight from air onto ground
	A2. Vibrocompaction	Densification of granular soil using a vibratory probe inserted into ground
	A3. Explosive compaction	Shock waves and vibrations are generated by blasting to cause granular soil ground to settle through liquefaction or compaction
	A4. Electric pulse compaction	Densification of granular soil using the shock waves and energy generated by electric pulse under ultra-high voltage
	A5. Surface compaction (including rapid impact compaction)	Compaction of fill or ground at the surface or shallow depth using a variety of compaction machines
B. Ground improvement without admixtures in cohesive soils	B1. Replacement/displacement (including load reduction using lightweight materials)	Remove bad soil by excavation or displacement and replace it by good soil or rocks. Some lightweight materials may be used as backfill to reduce the load or earth pressure
	B2. Preloading using fill (including the use of vertical drains)	Fill is applied and removed to pre-consolidate compressible soil so that its compressibility will be much reduced when future loads are applied
	B3. Preloading using vacuum (including combined fill and vacuum)	Vacuum pressure of up to 90 kPa is used to pre-consolidate compressible soil so that its compressibility will be much reduced when future loads are applied
	B4. Dynamic consolidation with enhanced drainage (including the use of vacuum)	Similar to dynamic compaction except vertical or horizontal drains (or together with vacuum) are used to dissipate pore pressures generated in soil during compaction
	B5. Electro-osmosis or electro-kinetic consolidation	DC current causes water in soil or solutions to flow from anodes to cathodes which are installed in soil
	B6. Thermal stabilization using heating or freezing	Change the physical or mechanical properties of soil permanently or temporarily by heating or freezing the soil
	B7. Hydro-blasting compaction	Collapsible soil (loess) is compacted by a combined wetting and deep explosion action along a borehole

Table 1 (continued)

Category	Method	Principle
C. Ground improvement with admixtures or inclusions	C1. Vibro replacement or stone columns	Hole jetted into soft, fine-grained soil and back filled with densely compacted gravel or sand to form columns
	C2. Dynamic replacement	Aggregates are driven into soil by high energy dynamic impact to form columns. The backfill can be either sand, gravel, stones or demolition debris
	C3. Sand compaction piles	Sand is fed into ground through a casing pipe and compacted by either vibration, dynamic impact, or static excitation to form columns
	C4. Geotextile confined columns	Sand is fed into a closed bottom geotextile lined cylindrical hole to form a column
	C5. Rigid inclusions	Use of piles, rigid or semi-rigid bodies or columns which are either premade or formed in situ to strengthen soft ground
	C6. Geosynthetic reinforced column or pile supported embankment	Use of piles, rigid or semi-rigid columns/inclusions and geosynthetic girds to enhance the stability and reduce the settlement of embankments
	C7. Microbial methods	Use of microbial materials to modify soil to increase its strength or reduce its permeability
	C8 Other methods	Unconventional methods, such as formation of sand piles using blasting and the use of bamboo, timber and other natural products
D. Ground improvement with grouting type admixtures	D1. Particulate grouting	Grout granular soil or cavities or fissures in soil or rock by injecting cement or other particulate grouts to either increase the strength or reduce the permeability of soil or ground
	D2. Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate to either increase the strength or reduce the permeability of soil or ground
	D3. Mixing methods (including premixing or deep mixing)	Treat the weak soil by mixing it with cement, lime, or other binders in situ using a mixing machine or before placement
	D4. Jet grouting	High speed jets at depth erode the soil and inject grout to form columns or panels
	D5. Compaction grouting	Very stiff, mortar-like grout is injected into discrete soil zones and remains in a homogenous mass so as to densify loose soil or lift settled ground
	D6. Compensation grouting	Medium to high viscosity particulate suspensions is injected into the ground between a subsurface excavation and a structure in order to negate or reduce settlement of the structure due to ongoing excavation

Table 1 (continued)

Category	Method	Principle
E. Earth reinforcement	E1. Geosynthetics or mechanically stabilized earth (MSE)	Use of the tensile strength of various steel or geosynthetic materials to enhance the shear strength of soil and stability of roads, foundations, embankments, slopes, or retaining walls
	E2. Ground anchors or soil nails	Use of the tensile strength of embedded nails or anchors to enhance the stability of slopes or retaining walls
	E3. Biological methods using vegetation	Use of the roots of vegetation for stability of slopes

Not only must the ground improvement technique that is to be implemented be feasible and suitable and acceptance criteria be appropriately defined to ensure the satisfaction of the design requirements, but testing and interpretation of test results must also be able to demonstrate that acceptance has been achieved. Not every test is suited for every instance. As obvious as this may seem, the authors have encountered numerous projects where the stipulated testing is either extremely difficult, if not impossible to perform, provides irrelevant or low-value information or is highly dependent on correlations.

For example, the plate load test is a very valuable test that is able to assess soil bearing capacity and deformation modulus, but the depth of influence is only approximately twice the plate diameter. Therefore, this test will not be able to provide any information for deeper layers of ground. While some engineers totally miss this point, others who are aware of the depth limitation specify that the test be performed in pits. However, this approach is only practical to very limited depths, beyond which the cost of excavation and providing a safe soil retaining system around the pit wall becomes significant. The presence of groundwater at testing level will even further complicate the performance of the test. In such cases, it would simply be wiser and better to develop an alternative testing program that is also able to assess the required ground parameters or to enlarge the loading area from a small-diameter plate to a zone load test [24].

There are many commonly used correlations between these tests and soil properties. The authors note that even though application of these correlations have their merits and can be of great assistance to the geotechnical practitioner, there are numerous occasions where, out of desperation and lack of data, correlations for specific types of soils are over-extended to other types of soil, which ultimately results in erroneous predictions of soil properties. Based on years of personal experience and research, the authors specifically warn against application of relative density in general [44] and its correlations with geotechnical testing methods [45] as acceptance criteria for ground improvement. Relative density is not a soil property, it is a formulation that was developed to define the looseness and denseness of sand or sand-gravel soils in a meaningful way with the assumption that important soil properties would well correlate quite well by this means. Confusion in the use of relative density began as soon as engineers began utilizing it as a soil parameter, and further problems arose when it began appearing as an acceptance criterion.

The choice of the geotechnical test to be performed for ground improvement should be considered on a case by case basis, and local practices do not necessarily always provide a suitable soil investigation method for ground improvement; especially in the case of soft soils where many in situ tests are unable to confidently provide consolidation analysis parameters. However, in reality, local practice and availability are usually determining as testing can only be done with what is available and the results would only be of value if competent operators undertake it.

There are numerous in situ geotechnical tests that can be carried out and, if relevant, should yield results that can be used to demonstrate that ground improvement works have been carried out to the satisfaction of the design requirements. Testing should be carried out to depths that enable the geotechnical engineer to input sufficient data into the calculation and analysis processes to capture ground behavior with sufficient accuracy. In a similar manner, the frequency of testing should be enough to be able to explain ground behavior with sufficient accuracy throughout the treatment zone. However, the testing regime should not turn into a critical path activity that governs the project's schedule. In the end, testing is not the purpose, it is the means to confirm that ground improvement works have been undertaken correctly. Briaud [21] suggests that the amount of testing should consider the required confidence to meet a reliability target and the volume of soil that is mobilized during testing.

Popular tests that are globally utilized are the Standard Penetration Test (SPT), the Cone Penetration Test (CPT) and the Pressuremeter Test (PMT), but other types of tests such as the Dilatometer Test (DMT) are also commonly used in certain regions and countries. The Vane Shear Test (VST) is also a handy tool for assessing the shear strength of saturated fine-grained soils. The history and application of these tests are briefly reviewed.

Standard penetration test (SPT)

In the Standard Penetration Test (SPT), a split-barrel sampler is driven into the ground to obtain a representative disturbed soil sample for identification purposes, and to measure the soil's resistance to the sampler's penetration. The weight and drop height of the driving hammer should, respectively, be 623 N and 0.76 ± 0.03 m. At each interval the hammer is driven for three consecutive 150 mm increments and the number of blows is counted for each interval. The SPT blow-count number, N, is the summation of the number of blows of the last two drive increments [8]. The SPT setup is shown in Fig. 1.

Rogers [82] has documented the historical backgrounds of the SPT based on the publications of Fletcher [34, 70] and others. In 1902, Gow began making exploratory borings using a sampler that was driven into the ground by repeated blows of a hammer. Twenty years later, Gow's company became a subsidiary of Raymond Concrete Pile Co. (RCPC) and continued to use the pipe sampler.

The split-spoon soil sampler, which was manufactured in a variety of sizes, was introduced by Sprague and Henwood, Inc. in the mid-1920s. The 2-inch (5 cm) split-spoon sampler

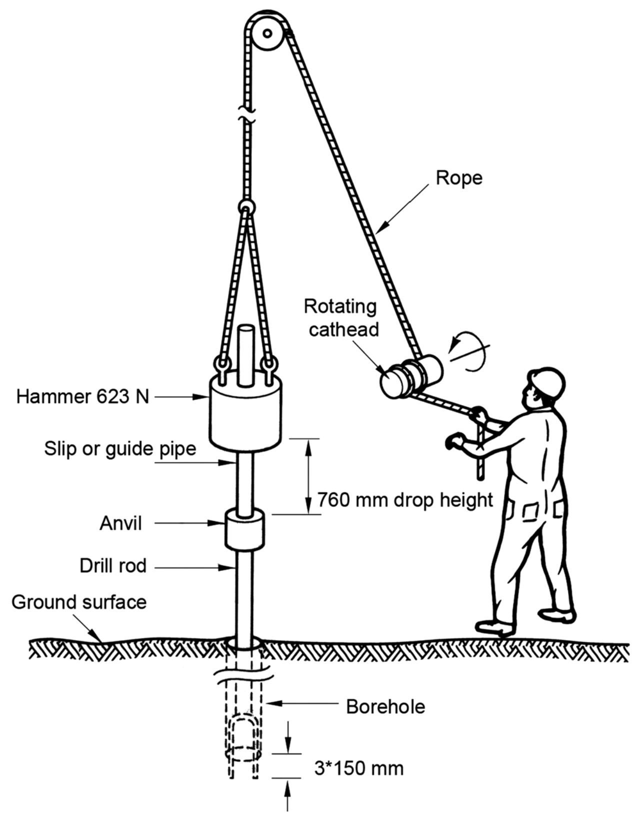


Fig. 1 The SPT procedure, modified from Kovacs and Salomone [53]

was introduced in 1927 by three Gow engineers, i.e., Hart, Mohr and Fletcher. Mohr measured the numerical values for driving the sampler 12 inches (30.5 cm) using an average driving weight of 140 lbs (63.4 kg) that was dropped from an average height of 30 inches (76.2 cm). This sampler recovered 3.5-cm-diameter samples.

The SPT gradually established its position among the geotechnical engineers. Terzaghi liked the Raymond Sampler because Mohr had collected more than 30 years of data. Along with Casagrande, he promoted the split-spoon sampling procedure until ASCE's Committee on Sampling and Testing of the Soil Mechanics and Foundations Division was formed in 1938 [82]. Terzaghi also observed that N could be correlated with many properties of the soil, including bearing capacity and named the sampler procedure the "Standard Penetration Test" in 1947. Soon after, [96] published the first SPT correlations. Many other researchers also followed suite and correlated N with various properties, parameters and behavior of soils, including [84, 85] who proposed the highly popular procedure for evaluating liquefaction potential using the SPT, which was further advanced by [101].

ASTM standardized the SPT when it published the first issue of D-1586 in 1958. This standard is still active, with its latest revision published in 2011 [8, 9]. Other institutions have also standardized the SPT procedure. The method of International Standard Organization [50] has also been adopted by British Standards (BS) and European Standard (EN).

Research commencing in the 1980s indicates that the SPT blow count requires energy correction for effective overburden pressure, hammer energy, borehole diameter, rod length, and sampler type. ASTM [9] has a standardized procedure for normalizing penetration resistance of sands.

The SPT can also be considered as the basis of many other dynamic penetration tests such as the dynamic cone penetrometer (DCP), the Perth penetrometer, the Italian Meardi AGI, and the German light and heavy ramming probes, i.e., the LRS (*Leichte Rammsonde*) and SRS (*Schwere Rammsonde*).

The SPT is a very valuable tool that not only assesses ground resistance to standardized blows but also provides a disturbed sample; however, the SPT also has its drawbacks. Samples extracted from the SPT sampler (Fig. 2) can be used for moisture content determination, for identification and classification purposes and for laboratory tests that are appropriate for soil obtained from a sampler that will produce large shear strain in the sample. However, the sample quality is generally not suitable for advanced laboratory testing for engineering properties because the process of driving the sampler will cause disturbance of the soil and change its engineering properties [8].

Although the SPT is well suited to granular soils, the test results are commonly misinterpreted when the sampler encounters rocks that are slightly larger than its sleeve diameter or clasts larger than approximately 3.5 cm. As shown in Fig. 3, in these cases, very high blow counts can be recorded, and a floater within the colluvium can easily be misinterpreted to be very hard ground or bedrock. That is why experienced SPT operators usually take their borings 3 m into the supposedly rock layer to be sure of the interpretation by the drilling resistance rather than solely relying on the SPT blow counts [82].

One of SPT's disadvantages is that it reports the accumulative blows per 0.3 m of drive. Therefore, the record is only representative of the average tested thickness and the influence depth beneath the sampler and can miss identifying thin soft layers solely based on the blow counts. Similarly,

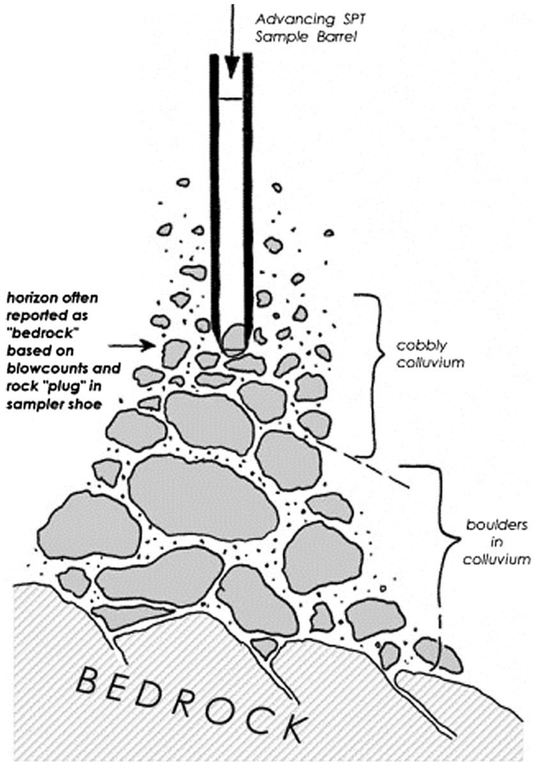


Fig. 3 Driving the SPT sampler into rock floaters in colluvium can be misinterpreted as encountering bedrock [82]

Fig. 2 The SPT split-spoon sampler

the testing procedure does not capture the resistance of the soils in between the testing intervals.

Also, silt and clay resistance to driving is different when they are dry or moist. If these materials become moister later, they may not show the stiffness that was interpreted from the SPT [82].

A lesser known problem of the SPT is the influence of strata thickness and changes in stiffness. As shown in Fig. 4, when the sample barrel approaches (approximately 5 sampler diameters) an appreciably stiffer layer, the penetration resistance will increase, even though the sampled material remains constant throughout the softer medium. This can lead to overestimation of strength solely based on blow-count values [82].

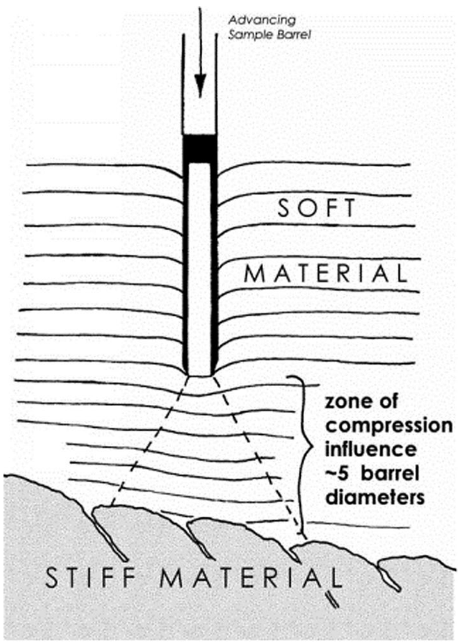


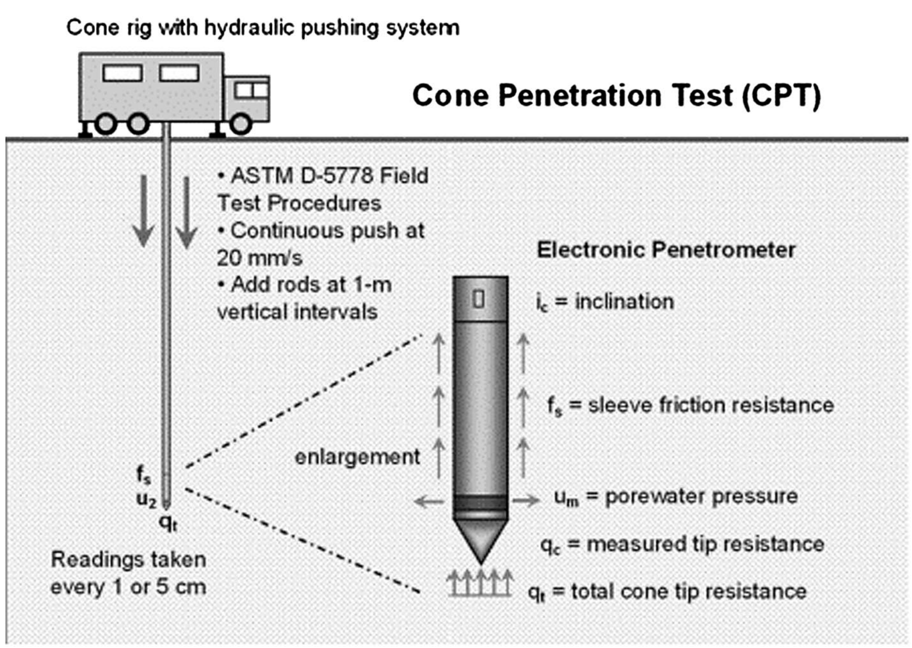
Fig. 4 The soil as an infinite number of springs [82]

Many researchers have proposed correlations between SPT blow counts and bearing capacity. One of the earliest and popular relationships was published by Terzaghi and Peck [97], but an accumulation of field observations has shown that those curves are too conservative. Meyerhof [67, 68] has also published correlations between allowable bearing capacity of cohesionless soils and a settlement of 25 mm that yield results that are similar to those of Terzaghi and Peck. Bowles [23] has adjusted Meyerhof's correlations to achieve 50% more bearing capacity.

Cone penetration test (CPT)

The Cone Penetration Test (CPT) measures the point resistance during penetration of a conical shaped penetrometer as it steadily advances into a subsurface soil and the frictional resistance of a cylindrical sleeve located below the conical point as it also steadily advances through subsurface soils [10, 13]. Figure 5 shows a CPT setup. While mechanical CPTs are still in use today, many devices are electronic. Some devices are also equipped with pore pressure instrumentation, thus the CPTu. The electronic piezocone penetrometer is an electronic cone penetrometer equipped with a low volume fluid chamber, porous element, and pressure transducer for determination of porewater pressure at the porous element soil interface measured simultaneously

Fig. 5 The CPT procedure [72]



with end bearing and frictional components of penetration resistance.

The CPT, previously also well known as the Dutch cone penetrometer, was introduced in the early 1930s. Meigh [63] cites Barensten [16] for introducing the CPT in 1934 in a form that is recognizable today. Lunne et al. [58] who also traced the device back to Barensten state that it was made in 1932.

While the CPT does not provide any soil samples nor does it classify the soil based on its grain size and plasticity, work initiated by Begemann [19] and advanced by Sanglerat [86], Sanglerat et al. [87], Douglas and Olsen [32], and Robertson et al. [79, 80] shows that the CPT can predict the soil behavior type (SBT) using the CPT cone resistance, q_c, and friction ratio, f_s (also the corrected or normalized versions of them). Robertson [80] notes that soil classification criteria based on grain size distribution and plasticity often relate reasonably well to in situ soil behavior; however, there are cases when differences arise. For example, a soil with 60% sand and 40% fines may be classified as silty sand or clayey sand using the Unified Soil Classification System [14], but its SBT could be identified as clayey silt or silty clay if the fines have high contents of highly plastic clay.

The CPT is commonly used in geotechnical investigations as it is highly productive, repeatable and supported by numerous correlations for determining various soil properties. While liquefaction evaluation originally commenced with SPT, Youd et al. [101] have also proposed the application of CPT.

The CPT provides almost continuous data as it advances into the ground, which mitigates the risk of missed out layers of ground; however, this test is also bound by its limits. Rogers [82] has made some notes of caution when using the CPT. The cone tip generates a passive failure of the ground in front of the tip. As shown in Fig. 6, the instrumented tip senses soil resistance about 5–10 cone diameters ahead and behind the tip. This means that the tip resistance reported as undrained shear strength is an average value, taken over the zone within 18–36 cm of the electric cone (with 10 cm^2 projection area). The minimum layer thickness to ensure full tip and skin friction response is somewhere between 36 and 71 cm. If the tip penetrates low strength layers that are less than this thickness, then the tip resistance reported on the CPT log may be much higher than reality. In summary, thin layers are sensed by the CPT but not fully sensed in that the values of tip resistance and skin friction may be artificially high.

Rogers [82] notes that another problem with the CPT is that cone soundings through desiccated clay will often be interpreted as sand or silt mixtures because of the recorded sleeve friction.

While experienced operators can probably alleviate any concerns, but personal experience of the authors over the

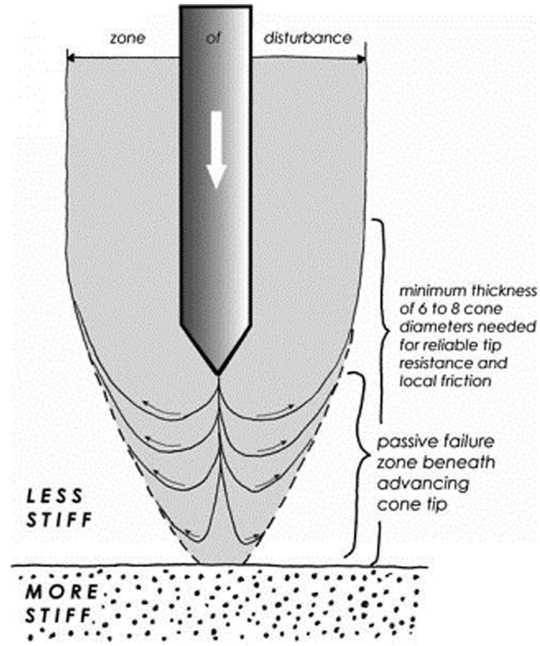


Fig. 6 The tip resistance recorded by the instrument is an average across the tip influence zone [82]

years has made them have some reservations about the determination of soil properties such as the coefficient of consolidation from CPT dissipation tests. It appears that test results are frequently on the optimistic side, possibly due to the saturation issues of the device.

Schmertmann [89] has proposed correlations between q_c and the bearing capacity factors (N_q, N_γ) of Terzaghi's bearing capacity equation [95]. Obviously, other CPT correlations with internal friction angle or cohesion can also be used in the equation.

Schmertmann [88] has also proposed a commonly used method for calculating settlement in sand using correlations between q_c and the soil's Young modulus, but it is the author's experience that the proposed correlation is conservative and overestimates settlements. Lee and Salgado [55] have summarized the correlations of Schmertmann et al. [91] and Robertson and Campanella [81] for young normally consolidated silica sand, aged consolidated silica sand and overconsolidated silica sand that are more compatible with the authors' observations.

Application of correlations developed for silica sands to calcareous sands can lead to incorrect predictions of ground behavior. Almeida et al. [3] carried out calibration chamber tests on the calcareous Quiou sand and conclude that for the same relative density, cone resistance in the calcareous sand was up to half the value of q_c measured in silica Ticino sand. Al Hamoud and Wehr [2] cite unpublished work of Gudehus and Cudmani who performed calibration chamber tests on Dubai's calcareous sand and Karlruhe's quartz sand and state that a shell correction factor of 1.5 for depths greater than 8 m, 1.6 for depths of 4–8 m, and 1.7 for depths less than 4 m must be applied to Dubai sand.

Pressuremeter test (PMT)

Menard developed the pressuremeter as the dissertation of his bachelor's degree in civil engineering and filed for a patent for it in 1954. He later improved his invention and carried out the first tests with the new probe while studying for a master's degree under the supervision of Peck and filed for a second patent in 1959 [39].

The PMT is an in situ stress–strain test that is performed on the borehole wall using a laterally expanding cylindrical probe that allows the evaluation of both the deformation and failure properties of the ground in a single test by measuring the pressuremeter modulus, creep pressure and limit pressure.

Contrary to SPT and CPT that are advanced into the ground by hammering or pushing, the PMT is performed in a borehole. Hence, the test can be done in almost any kind of ground, from soft soils to rock and it is the authors' experience that test results are very reliable when undrained shear strength is higher than 20 kPa. The general layout of the PMT is schematically shown in Fig. 7.

Bearing theories originating from Prandtl [74], further developed by Terzaghi [95] and advanced by others are generally based on Mohr–Coulomb failure criteria. Prandtl developed his formulation based on his study of a long hard object punching into softer material and assumed that (1) the material was homogeneous, isotropic, and softer than the puncher, (2) the material was weightless and possessed only friction and cohesion, (3) the problem was two dimensional, (4) the base of the puncher was smooth, (5) the material behaved as a rigid body, (6) the volume change was null, and (7) the deformation was plastic.

Menard's approach is different from those of Prandtl and Terzaghi. Using Terzaghi's approach, bearing capacity can be calculated as a function of footing width, embedment depth, soil density, soil internal friction angle, and soil cohesion. Menard's method relies on neither the internal friction angle nor the cohesion. Instead, (ultimate) bearing capacity, q_u, is a function of total overburden pressure at the periphery of the foundation level after construction, q_o, a bearing factor, k, total at rest horizontal earth pressure at the test level, P_o, and the soil's limit pressure, P_{LM}. The concepts of the methods of Terzaghi and Menard are schematically compared in Fig. 8.

Menard also introduced a new approach for calculation of settlements using the concept of volumetric compression and shear (deviatoric) deformation [64, 66]. The first settlement component is caused by the spherical element of the stress tensor. The increase in bulk pressure causes a reduction in volume of the material in relation to the modulus of volumetric compression. At the same time, the latter part of settlement is caused by the deviatoric

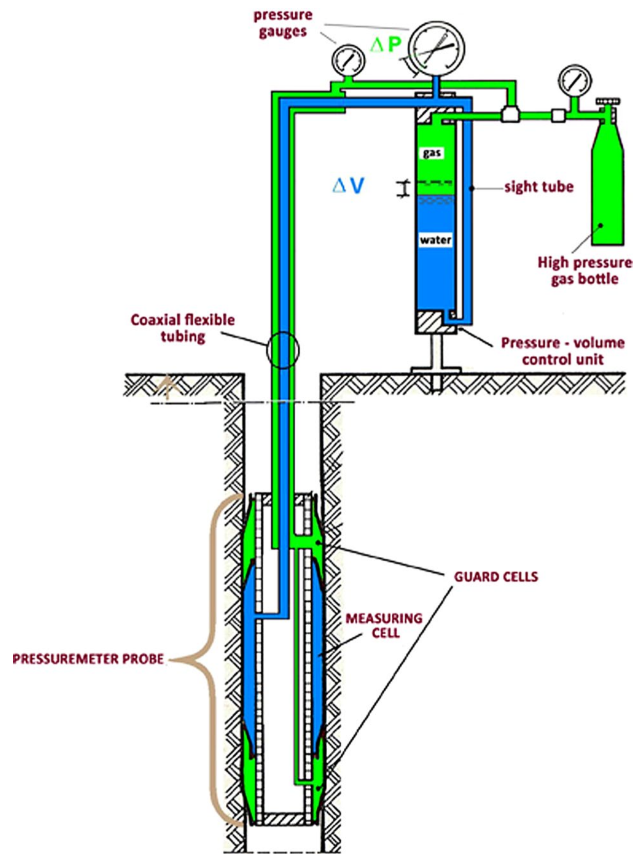


Fig. 7 The general layout of the Menard Pressuremeter Test (Courtesy of Apageo)

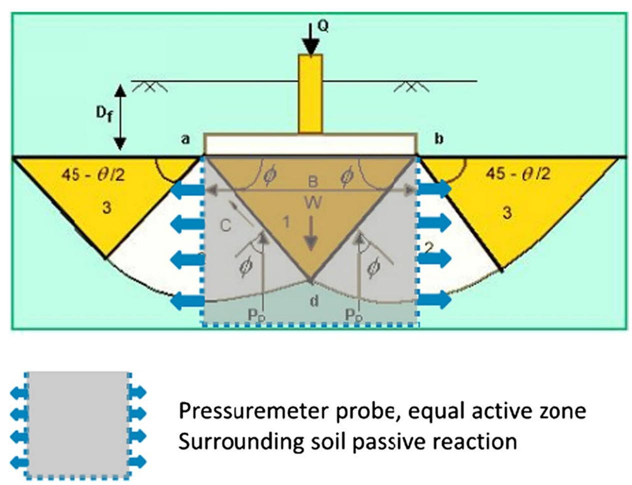


Fig. 8 Schematic modeling of shallow foundation failure by Prandtl–Terzaghi and Menard methods [98]

elements of the stress tensor, and displacements occur without variation in volume of the material.

The spherical and deviatoric components of the stress tensor are very different at depth. The first component has a maximum value right under the base of the footing; however, the latter component has a maximum value at a depth that is equal to half of the footing's width. Shear deformation is dominant under footings, shafts and piles, but volumetric compression predominates under rafts and embankments.

Contrary to many other geotechnical testing tools, settlement of soil under self-weight can be assessed using the pressuremeter [64, 99].

Like the SPT and CPT, while retaining its conceptual design and performance, the PMT has also undergone advances in precision, automerization and drilling methods [5, 6]. The ASTM standard for PMT is D-4719 [7].

While the PMT is an extremely valuable tool that enables the geotechnical engineer to measure both shear failure and the (Menard) modulus of deformation in the widest range of ground conditions, similar to the CPT, it does not extract any samples. Hence, efforts have been made to develop a graphical method to describe the ground's behavior type rather than its classification based on grain size and plasticity [18].

Menard [64] has proposed the interpretation and calculation of bearing capacity and settlement using the pressuremeter. His methodology still remains the basis of practically all standards and codes of practice.

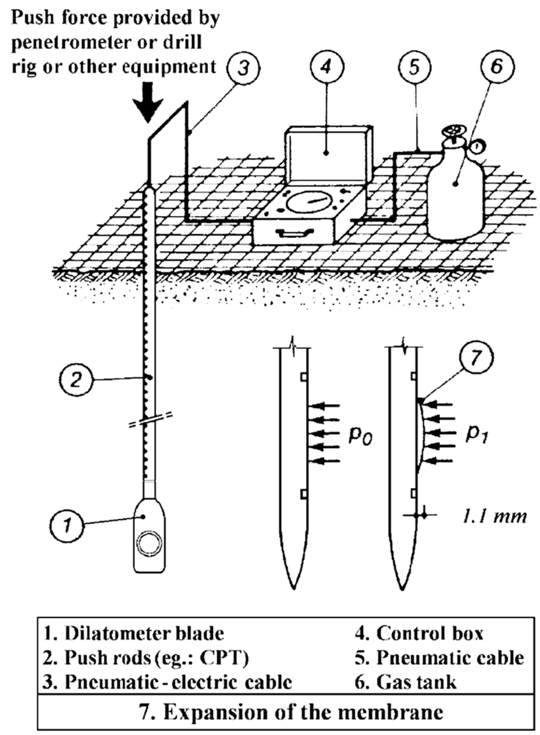


Fig. 9 General layout of the Dilatometer Test [61]

Flat dilatometer test (DMT)

The Flat Dilatometer Test (DMT) was developed by Marchetti in Italy. The first publication on this device dates back to 1975 [60]. The intent of the test was primarily to investigate the values of soil modulus for laterally loaded driven piles, where horizontal movements are preceded by penetration, but was then expanded to other deformability and settlement problems.

As shown in Fig. 9, the DMT is a stainless-steel blade with a flat circular membrane that is mounted flush on one side [59]. The blade is connected to a control unit on the ground surface by a pneumatic-electric tube running through the insertion rods. The blade is statically advanced into the ground using a CPT rig. The blade advances at 20 cm intervals, the membrane is inflated, and readings are taken of pressures required to lift the center of the member above its support, and move laterally 0.05 mm and then 1.1 mm [11]. The deflation pressure for the membrane to return to the movement of 0.05 mm is also recorded.

The dilatometer modulus can then be calculated as a function of the membrane diameter, the corrected pressure difference, and Poisson's ratio. The DMT can also provide for other soil properties by correlation [22, 59, 90]. Schmertmann [90] suggests that the correlations generally provided reasonable accuracy except in very sensitive clays, weathered clay crusts, and aged or cemented clays.

Two other important dilatometer parameters are the material index and the horizontal stress index [59]. Similar to the CPT and PMT, while the DMT does not provide soil samples, [59] indicated that a relationship exists between soil behavior and the material index of the DMT. ASTM [11] includes a chart that shows such relationship by way of example. The horizontal stress index is the primary index used in the correlation for in situ horizontal stress, over-consolidation ratio, and undrained shear strength in cohesive soils [11].

In advanced form the DMT can be combined with an add-on seismic module for the measurement of shear wave velocity [62].

Robertson who has published CPT-DMT correlations [80] has also compared the two testing methods. He notes that the DMT is simple, robust, repeatable and economical, but is harder to push in very stiff ground compared to the CPT. Also, DMT is carried out every 20 cm, whereas CPT readings are taken every 2–5 cm. The DMT requires a pause in the penetration to perform the test; thus, it produces less data and is slower than the CPT.

Vane shear test (VST)

Osterberg [73] speculates that the vane-borer that we now call the Vane Shear Test (VST) was developed simultaneously in 1928, in Sweden by Olsson and in Germany, as evidenced by a German patent dated 1929. He notes that not much work was done on the vane until 1947, when the Swedish Geotechnical Institute designed several improved vanes and used them on numerous projects.

The VST is applicable to saturated fine-grained soils and provides an indication of in situ undrained shear. In this test a four-bladed vane is inserted into intact soil and rotated to determine the torque required to shear a cylindrical surface with the vane. This torque is converted to a unit shearing resistance of the failure surface by limit equilibrium analysis. Figure 10 shows vane blades and a driving frame that is used for testing in boreholes.

The advantages of the VST are that the test is performed in situ and avoids the problems of stress release and sample disturbance [33]. Also, the test is relatively inexpensive compared to conventional tube sampling and laboratory testing.

However, the VST has its drawbacks and limitations, which based on a number of studies is noted in ASTM [11, 12]. The test is applicable to soils with undrained strengths of less than 200 kPa. Very sensitive soils can be remolded during vane insertion. It is also not applicable to unsaturated soils or to non-plastic silts, sands, gravels, or other high permeability soils. Sand lenses allow total or partial drainage.

Fig. 10 Vane blades and driving frame for testing in boreholes [83]

Unsaturated soils and soils with higher permeability can dilate or collapse in rapid shear and generate negative or positive pore pressures that may or may not dissipate in the shearing process.

The peak undrained shear resistance of the vane test is commonly corrected to determine the undrained shear strength for geotechnical analysis. Since vane shear strength values are almost always higher than field strengths for analyses, they are often checked or compared with other methods of measuring undrained shear strength. CPTs and unconsolidated undrained triaxial compression tests are most often performed for direct comparison to the vane shear strength data [11].

Additionally, the quality of the VST result is dependent on the competence of the personnel performing it, and the suitability of the equipment and facilities used (ASTM).

It is noted that the VST is generally used for determining cohesion and stability; thus, the bearing capacity of the soil and not the settlement.

Ground improvement

It is sometimes possible to predict the need for ground improvement based on experience of similar projects or ground conditions. For example, research [55–57, 71, 92] indicates that ground reclamation from the sea by soil dumping or hydraulic filling will result in the deposition of loose layers of soil. Hence, it would be wise to foresee the need of ground improvement in such projects. However, it is ultimately the combination of the results of the geotechnical investigation and the project requirements that determine whether ground improvement will be required.

Along with geological studies, geotechnical investigations and testing generally commence at the very preliminary stage of a project to provide an initial understanding of the ground conditions. The determination of the most appropriate testing methods, the frequency and distribution of testing and the depth of testing are all very important issues that must be addressed properly when developing the geotechnical investigation plan, but preliminary testing is beyond the scope of this paper and will not be discussed. However, it is noteworthy to state that improper identification of any of the above may result in non-identification of unsuitable ground that could result in great losses in the project.

The geotechnical testing methods, frequencies and depths may be refined, and additional tests undertaken depending on the outcomes of the preliminary testing. Testing may continue throughout the course of the geotechnical construction works to verify and to demonstrate that project requirements and acceptance criteria have been satisfied.

Even if a ground is identified as being loose, soft or highly compressible, per se, this does not necessarily mean that ground improvement is required. The need for ground improvement only becomes meaningful when the existing ground conditions do not satisfy design requirements and additional measures must be undertaken to ensure that the foundation system's behavior will meet the design requirements. Ground improvement must have a purpose and targets to meet.

Design typically requires a minimum bearing capacity and limits on total and differential settlements, but may also require mitigation of liquefaction, increasing lateral soil resistance, stabilizing ground against slope failure, etc.

In addition to the suitability and feasibility of the ground improvement technique itself, the level of success of any ground improvement program is also related to the applicability and suitability of the criteria that is defined to warrant the satisfaction of the design requirements.

Hamidi et al. [42] have studied the various approaches for ground improvement and acceptance criteria specifications. In the first approach, ground improvement specifications are developed in full detail by the party who has prepared the tender documents before the tender and the award of the contract. In such cases, based on the geotechnical advisor's internal design, which is usually not made available, a ground improvement technique is specified, the scope of work to be performed is described in detail and the construction method is outlined. The responsibility of the contractor is usually limited to procurement of the labor and management team, equipment, material and execution of the works as detailed in the scope of works. Acceptance criteria are based on correctly performing the works rather than meeting a technical requirement. Testing is generally specified, but the contractor who has had no technical input is not committed to warrant any technical outcome. Stipulating non-technical acceptance criteria results in success only if the anticipated methodology works perfectly and there are no problems. However, serious issues may arise if the ground improvement works do not result in the desired outcome.

A better approach is for acceptance criteria to be technical requirements rather than reducing them to the quality and quantity of performed work. In such an approach, acceptance criteria are sometimes stipulated in the form of minimum test values. Sometimes specifications stipulate an infeasible testing method but attempt to go around the problems by correlating the test to a practical testing method. However, Hamidi et al. [42, 43] have shown that it is possible to achieve superior results and satisfy design requirements to a greater extent without conforming to minimum test values because the ground behaves as a mass rather than individual layers that have to perform independently.

The most suitable method for specifying acceptance criteria is to base them directly on design criteria as it can optimize treatment energy and confidence in outcome. The purpose of geotechnical testing and its interpretation is to demonstrate that design requirements have been satisfied, and there is no reason to complicate this requirement by introducing additional parameters into the equation. The definition of required bearing capacity and settlement of the project should be complemented by the contractual calculation methods that have been agreed by the specialist contactor and the engineer.

A brief history and methodology of several popular ground improvement techniques from the ground improvement categories are reviewed in this paper. More information can be found in other publications, e.g. [27].

Ground improvement without admixtures in non-cohesive soils or fill materials

Techniques like dynamic compaction, vibrocompaction, explosive compaction, impact rolling, rapid impact compaction and electric pulse compaction are all ground improvement methods that mechanically improve soils with non-cohesive behavior without adding better quality soil or admixtures to the ground. The verification of this category of techniques is essentially governed by the results of in situ testing.

Dynamic compaction

Menard invented and promoted dynamic compaction as early as 1969, but it was not until 1970 that he officially patented his invention in France [40]. In this technique the mechanical properties of the soil are improved by transmitting high energy impacts to loose soils with low bearing capacity and high compressibility. The impact creates body and surface waves that propagate in the soil medium. In non-saturated soils, the waves displace the soil grains and re-arrange them more densely. In saturated soils, the soil is liquefied, and the grains are re-arranged more compactly. In both cases the decrease in voids and increase in inner granular contact will lead to improved soil properties.

Impact energy is delivered by dropping a heavy weight or pounder from a significant height. The pounder weight is usually in the range of 8–25 tons although lighter or heavier pounders are occasionally used. Drop heights are typically in the range of 10–20 m although lower are higher heights may also be used.

The first phase of treatment is carried out at a wide grid with the maximum amount of impact energy per impact point. The objective of this phase is to treat the deepest soil layers. The second phase, which intends to treat the intermediate soil layers may be carried out with less energy, and if necessary, the final phase (ironing) will comprise of closely spaced grid points with one or two low energy blows per print for improving the uppermost soil layer. Figure 11

Fig. 11 Application of dynamic compaction in multiphase grids

shows the application of dynamic compaction in a very large-scale project.

Dynamic compaction depth of improvement is a function of impact energy [65], i.e., pounder weight and drop height; hence, it can be understood that the depth of improvement is practically limited by the lifting capacity. In practice, typical depths of improvement are between 8 and 14 m although deeper improvements have been reported [35, 41].

With consideration that both dynamic compaction and the PMT have been invented by Menard, common practice in situ testing for this technique has been by PMT. This practice is further reinforced with recognition that dynamic compaction can be carried out when the ground consists relatively large cobbles and boulders, which makes the application of other testing methods very difficult, if not impossible.

Vibrocompaction

Vibrocompaction, also known as vibroflotation, is a deep ground compaction technique that was developed in 1934 with the invention of the first vibroflot by Degen and Steuermann [20] in Germany.

The vibroflot, sometimes also referred to as a vibroprobe or vibrating poker, is a hollow steel tube containing an eccentric weight mounted on a vertical axis in the lower part of the tube to give it horizontal vibration. The vibroflot is connected to extension tubes that are supported by a rig, which is usually a crane but can also be an excavator. The vibroflot is either flushed down to the required depth in the soil using water jets or vibrated dry with air jets. When the vibroprobe reaches the required depth, soil (sand) is added from the ground surface during withdrawal, and the vibroflot is moved in an up and down motion at certain intervals. The horizontal vibrations form a compacted cylinder of soil with a depression at the surface due to the reduction of voids. Depending on the vibroflot power, the zone of improved soil can extend radially 1.5 m to more than 4 m from the vibroflot.

Vibrocompaction is usually carried out by attaching one vibroflot to a rig, but production can be increased by connecting a pair of tandem vibroflots to a single rig. A typical single vibroflot arrangement is shown in Fig. 12.

This technique is best suitable for the treatment of soils with limited amounts of fines. Mitchell [69] proposes that vibrocompaction is best suited when fines content is limited to 18%. In line with the authors' personal experience, Woodward [100] proposes that best results can be achieved when fines content is less than 10%.

While piling soil at the vibroflot insertion point can be helpful, vibroflotation is not very successful in compacting the top layer of soil due to lesser confinement and overburden pressures; however, it is typically able to treat much deeper layers of soil than dynamic compaction.

In this technique, vibration propagates and dampens radially away from the vibroflot; therefore, the magnitude of improvement also reduces radially from the insertion point. Consequently, test results will vary depending on the distance from the vibroflot insertion points, such that the maximum test result will be in the insertion points and the least results will be in the centroid of the insertion points. In reality, the ground behaves as a mass and a more representative test should be done in a point in between the insertion points and their centroids.

CPT, SPT and PMT are the most common tests that are utilized for verifying that vibrocompaction works have resulted in the satisfaction of acceptance criteria.

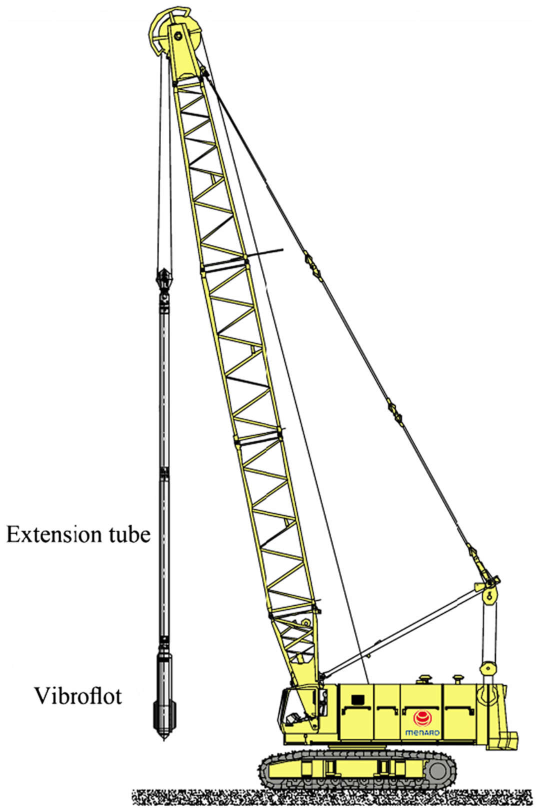


Fig. 12 General arrangement of an equipment for vibrocompaction

Ground improvement without admixtures in cohesive soils

These methods can be from their most simplistic form as preloading of soft soils without vertical drains to their most advanced form as vacuum consolidation.

Vacuum consolidation

The idea of vacuum consolidation was proposed more than 60 years ago [52]; however, practical use of this technology is more recent. Zhu and Miao [102] have reported its use in a large-scale project at Tianjin in 1982. Instead of increasing the effective stress by increasing total stress by surcharging, the technique provides an enhanced method of surcharging with vertical drains by reducing the pore pressure while maintaining a constant total stress [29].

In the Menard system, suitable vertical drains are installed to the required depth, and horizontal drains are placed in longitudinal and latitudinal directions in the sand blanket [28]. An impermeable membrane is used to seal off the ground surface. If required, due to the presence of pervious continuous layers, impermeable isolation trenches or cut-off walls are placed around the treatment area. Vacuum pumps are connected to the discharge module extending from the trenches, and vacuum pressure is applied to the ground. The uniqueness of this system is the dewatering below the membrane, which permanently keeps a gas phase between the membrane and the lowered water level [27].

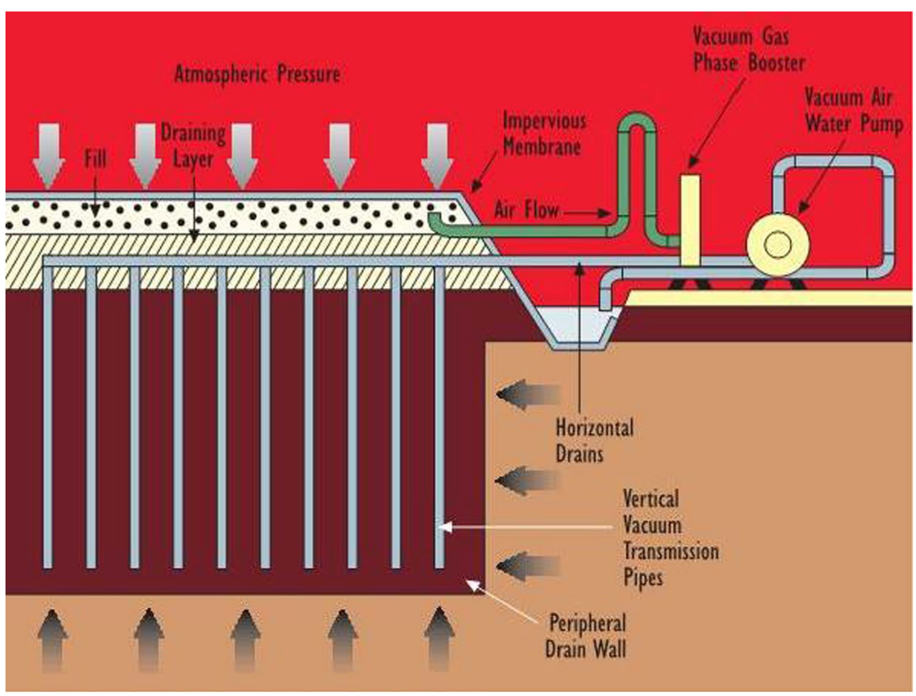


Fig. 13 General arrangement of Menard vacuum consolidation method (courtesy Menard)

Figure 13 illustrates the general arrangement of the Menard vacuum consolidation process.

The advantages of vacuum consolidation compared to surcharging with or without vertical drains are that this method does not require fill material for surcharging, the construction period can be shorter because no stage loading is required, it may be more economical than using fill surcharge and the vacuum creates immediate stability [27]. However, vacuum consolidation generates inward lateral movement that can be an advantage at times and a problem at other times. The magnitude of movement can be balanced by adding a conventional fill surcharge that generates outward lateral displacement.

The outcome of preloading techniques is a direct function of the soil's consolidation and compressibility properties. Deviations from the assumed design values can have dramatic consequences in consolidation time and settlement magnitude. Once preloading has been applied, practical corrective measures are limited to increasing the surcharge or accepting longer consolidation periods. Compensating for settlements greater than predicted may result in accepting reduced project levels, but this approach is much more complex when the platform level will be reduced too close to groundwater level or in the worse scenario fall below groundwater level. Even worse is when the problem is realized after construction of permanent structures and facilities.

Verification of preloading techniques may be done with CPTu or VST, but consolidation process is usually checked by measuring settlement and predicting long-term settlements or consolidation times. Methods proposed by Asaoka [4] and Tan [93] are very popular for predicting settlements, but at least 60% of consolidation must occur for these techniques to provide accurate estimates of the ultimate primary consolidation settlement and the in situ consolidation coefficients [94]. The authors recommend using CPTu results for determining the coefficient of consolidation with caution as in their experience the test results in some projects, possibly due to improper saturation of the cone, were on the high side, which resulted in optimistic values of time.

The authors practice has been more oriented toward the implementation of undisturbed samples (by the Osterberg piston sampler) and laboratory testing for design purposes.

While at first glance, it appears that application of piezometers can be useful for estimating the amount of consolidation for soils with vertical drains, the location of the piezometers will never be in the center of drains due to verticality of the drains and piezometer boring. Literature has also reported delayed access porewater pressure dissipation despite comparatively large amounts of settlement in very soft clays. Chu et al. [26] attribute these phenomena to sedimentation and self-weight consolidation stage prior to consolidation under additional fill. During this stage, the slurry-like very soft clay transforms from a liquid-like to a solid state in which water dissipates, but the soil particles do not have sufficient contacts to allow the soil skeleton to take up external load. The compression index of the very soft clay in the low stress level could be significantly high, which suggests that a large settlement could be induced by only a small effective stress gain. The Mandel–Cryer effect and non-uniform consolidation of soil around the vertical drain can also be other reasons accounting for the lack of porewater dissipation. Chu et al. note that during consolidation, the soil element near the drain consolidated faster and took a larger share of the vertical load. As a result, local radial strain developed and extra porewater pressure was induced. The induced pressure neutralized the pore pressure dissipation or even elevated the pore pressure in the soil further away from the drain.

As reported by Choa et al. [25], the authors have also observed delayed dissipation of excess porewater pressure in the marine clays of Changi Airport second runway project. Hence, it can be concluded that monitoring of settlement is more reliable than that of excess porewater pressure. It is also the experience of the authors that installation of deep settlement gauges can be a successful method for measuring deformations of each layer to reach the design void ratio.

Ground improvement with admixtures or inclusions, without stabilizing the soil mass

Techniques like dynamic replacement, vibro replacement (stone columns), controlled modulus columns and jet grouted columns are well-known techniques in which ground behavior is improved by introducing a grid of columns in the soil that have superior properties. The inclusions can be from natural materials such as sand and stone to manufactured products such as recycled concrete, concrete or grout that is in situ mixed with soil.

Analysis and design of a ground improvement system with inclusions is more challenging than when there are no inclusions. Many researchers have proposed calculation procedures, but introduction of finite element and finite difference techniques in the form of computerized commercial software has been very useful tools and has greatly assisted the analyses procedures.

Computer applications require the user to provide the soil's geotechnical properties, which are usually the modulus of deformation, cohesion, internal friction angle, dilation angle and permeability (for consolidation problems), none of which is directly measured by in situ geotechnical tests. Hence, the entire process of analysis will have to depend on correlation.

To yield reliable values for the input properties, the correlation must be reliable and meaningful, and its implementation must be justifiable. Clearly a correlation developed from highly scattered data is less reliable compared to when there is a close match between the data points and the proposed best fit curve. Also, correlations are frequently highly dependent on ground conditions, and extending the application of correlations that have been developed for one specific ground condition to other ground condition, if at all, should be done with extreme caution as inapplicable assumptions may have major impacts on the results of the analyses.

Dynamic replacement

Dynamic Replacement is a ground improvement technique that was also developed by Menard in 1975 for the treatment of soft cohesive soils. Similar to dynamic compaction, in this technique a heavy pounder is systematically dropped a number of times onto specific points to drive granular material into soft compressible cohesive soils and to sufficiently compact the driven material to meet the project's requirements. The process of dynamic replacement is schematically shown in Fig. 14.

The distribution of loads will be by arching through a granular engineered fill, which is called the load transfer platform. Several researchers such as Hewlett and Randolph [47] and Kempfert et al. [51] have proposed calculation procedures to calculate the load distribution and inclusion efficacy.

PMT has proven to be an applicable and useful tool in dynamic replacement as it is able to determine the shear strength and modulus of both the surrounding soil and the highly compacted crushed rock that creates the columns. In landfill, the zone load test may prove to be a useful tool for calibrating the PMT rheological factor.

Analysis of dynamic compaction can be done using both analytical and numerical methods.

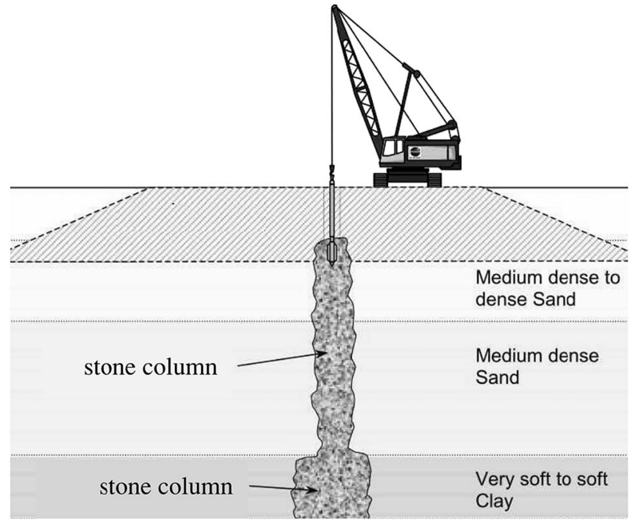


Fig. 15 The process of vibro replacement or stone columns [31]

Stone columns

Vibrocompaction is inefficient or ineffective when fines content is high, and ground improvement must be performed by the vibroflot in an alternative process called vibro replacement or stone columns. As shown in Fig. 15, in this method, crushed stones are fed into the columnar cavity that is formed in the soil and compacted using the vibroflot to make a semi-rigid inclusion. The common construction methods for stone columns include the wet top feed and the dry bottom feed methods. The major difference between these two processes is the stone feeding mechanisms, whereas in the first method stone is introduced into the column from the ground surface by pushing and gravity action while in the latter method stone is fed to the tip of the vibroflot via a pipe (Figs. 16, 17).

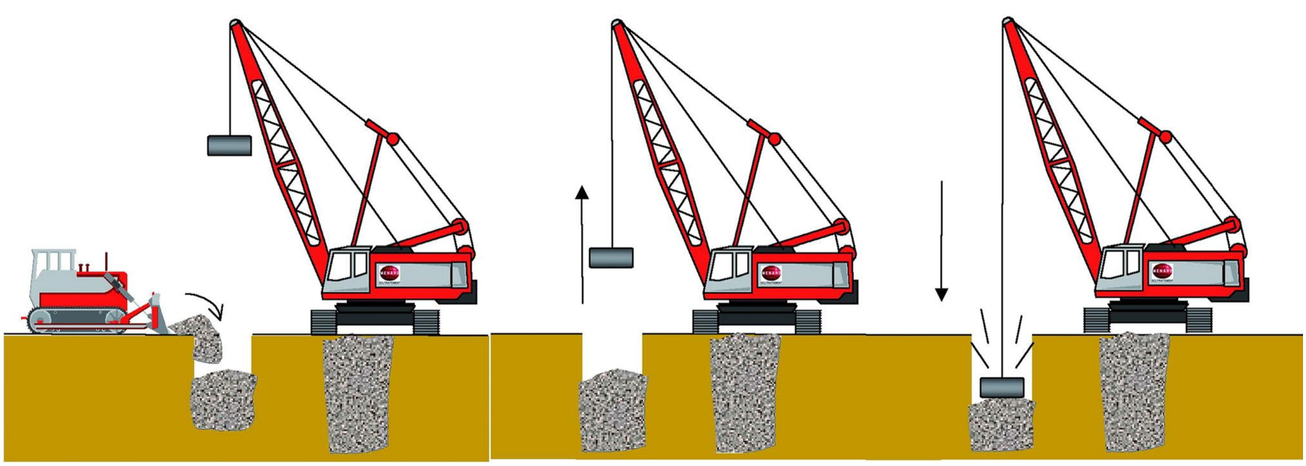


Fig. 14 The process of dynamic replacement [46]

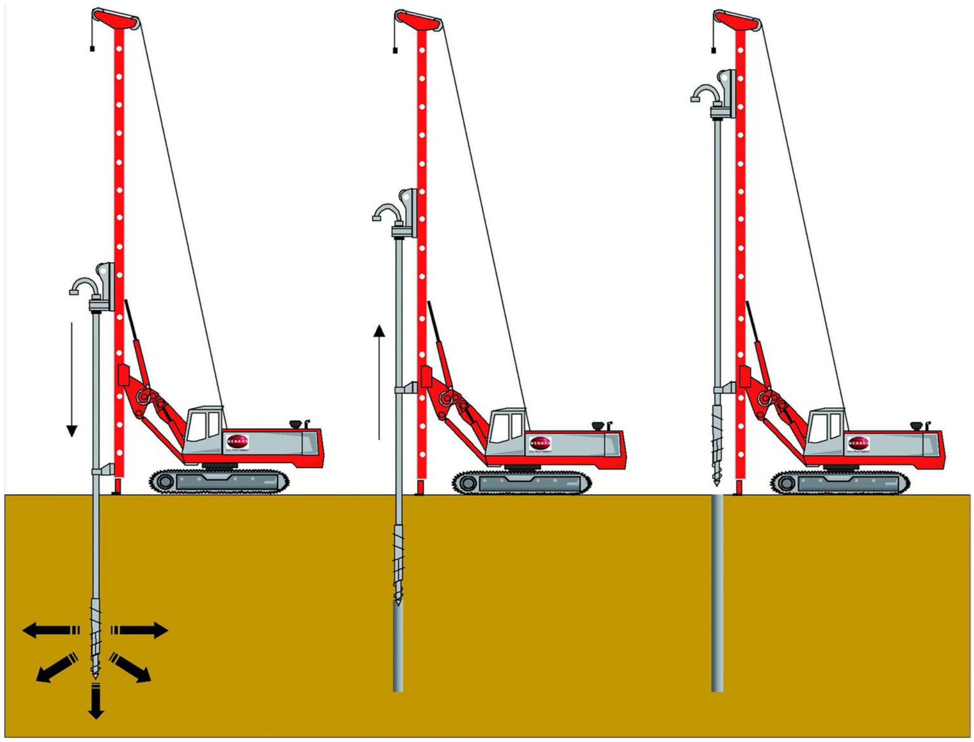


Fig. 16 The procedure of CMC (courtesy Menard)

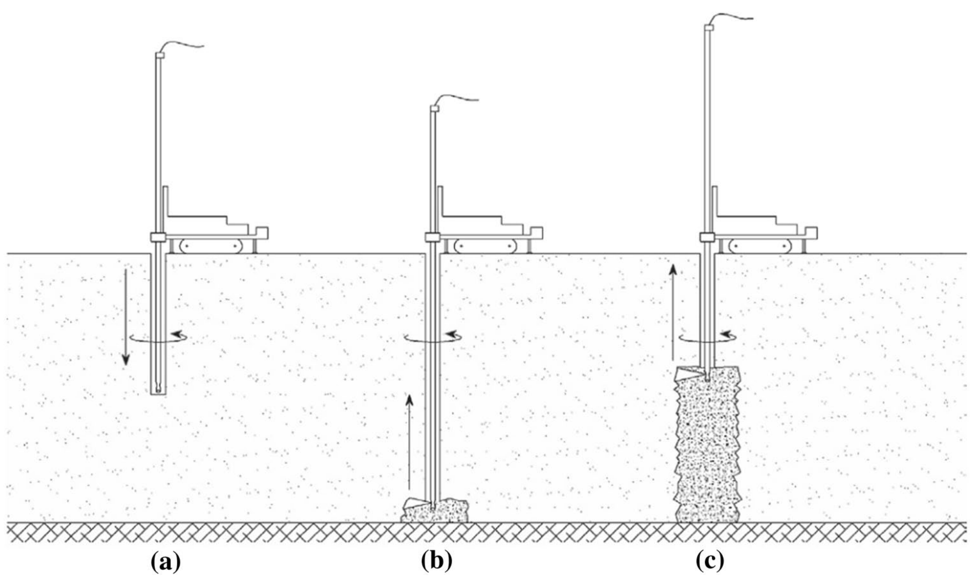


Fig. 17 Typical jet grouting procedure: **a** drilling, **b**, **c** jet grouted column formation [30]

Stone columns can be installed to greater depths than what is achievable by dynamic replacement; however, there are practical limitations on the depth of improvement due on a number of parameters such as the resisting friction between the vibroflot and the soil.

Furthermore, with consideration that there is no internal cohesion in the aggregate that forms the stone column, the actual compaction and stability of the stone will be a dependent of the soil's properties. Stone columns will bulge and fail if the vertical loading of the columns generate lateral loads that exceed the soil's limit pressure [17].

Stone columns are frequently utilized for liquefaction mitigation. Baez and Martin [15] proposed an analysis and design method that assumes that shear strains for both loose soil and stiff stone columns are compatible and conclude that stresses are concentrated in the stone column proportional to a ratio of shear moduli between stone column and soil. This assumption is valid in short columns; however, adopting it for long columns will result in an overestimation of the stone columns' share of stresses [1, 36, 37, 78].

Both dynamic replacement and stone columns are constructed by installing well compacted crushed stone columns into the ground; however, the diameter of the latter technique is much smaller than the first. While the testing method that is adopted for verifying the properties of the columns in both techniques must be able to penetrate the material, additional care must be exercised for the latter to ensure that the testing will not deviate to outside the column. The PMT has proven to be a powerful instrument for testing semi-rigid inclusions. Some engineers also find the load zone test as a convenient testing method.

Analytical and numerical methods are widely used for analysis and design of stone columns. Barksdale and Bachus [17] have reviewed a number of commonly used empirical procedures, including methods proposed by Priebe and Greenwood [38]. Priebe's method was later revised [75, 76].

Controlled modulus columns

Controlled Modulus Columns (CMC) technology was developed by Menard in 1994 [27]. As shown in Fig. 16, these rigid inclusions are installed into loose or soft ground using a specially designed auger that laterally displaces the soil with practically no spoil or vibration to form a cylindrical cavity. The auger is screwed into the soil to the required depth, and concrete is pumped through the hollow stem auger during its extraction to create concrete column that typically has a diameter of 250–450 mm. Similar to dynamic compaction and stone columns, the loads are transferred to the ground via the load transfer platform.

Unlike stone columns, where stability relies on the horizontal retainment of the soil or deep soil mixing where column strength is dependent on the in situ soil's properties, CMCs neither rely on the soil for lateral stability nor are their strengths affected by the surrounding soil. The columns' moduli of deformation are also the same as concrete. Consequently, this technology can reduce settlements more efficiently and effectively than other techniques with inclusions.

As there is no in situ incremental compaction procedure as in stone columns nor an in situ mixing process as in deep soil mixing, the installation rate of CMCs is much higher than other semi-rigid or rigid inclusion ground improvement techniques. This advantage reduces construction time and cost, which when feasible, makes this technology very favorable.

It is not possible nor required to perform any of the geotechnical tests that have been described in this paper on CMCs because none can penetrate the concrete. At the same time, there is no need to carry such tests and concrete quality control tests will suffice and provide the required verifications. However, implementation of static load tests on a CMC will provide valuable information that can be used for verification of the works.

As with other ground improvement techniques, analytical and numerical analysis methods have significantly developed for this technology. The result of a national project that was undertaken in France on rigid inclusions [49] has become a main reference publication. The English translation of this publication is also available for non-French-speaking engineers. Innovative modeling techniques have also evolved, which incorporate empirical adjustments of the PMT parameters to fit reality [77].

Jet grouting

Jet grouting was invented by the Japanese in the 1970s [48], and as shown in Fig. 17 is a method that is performed by drilling down with a small-diameter rod, injecting high pressure fluids while rotating and withdrawing the rod to erode and mix the soil with cement grout to form a rigid cementitious column. Jet grouting installation methods include single, double and triple fluid injection systems. In the single fluid method neat cement grout that is injected through a small nozzle at high pressure is used to erode and mix the soil. In the double fluid method, the cement grout is aided by a concentric cone of compressed air, which shrouds the grout injection. The air reduces the friction loss and allows the cement grout to travel further from the injection point and to produce larger column diameters. In the triple fluid method, water and air erode the soil and cause partial substitution of the finer soil particles. Cement grout is injected independently through a nozzle located beneath the air or water nozzle.

Design of column diameter is a complex specialty that involves many parameters including soil type, soil strength, nozzle size, lift rate, rotation speed, injection pressure and grout mix. Consequently, jet grouting columns are almost always designed by specialist jet grouting contractors. However, Croce et al. [30] have proposed a method that they have published in their book.

Jet grouting can be used for numerous purposes, including ground improvement in the form of columns or as blocks, retaining ground, or creating a water barrier. When used as rigid inclusion, the same analysis concepts can be utilized. Jet grouted columns are composed of a mixture of in situ soil and cement grout; therefore, testing of the quality of the columns is also in accordance with grouting tests.

Conclusions

Every justifiable ground improvement project commences with a geotechnical investigation and design requirements that have to be satisfied by implementing a suitable treatment technique. The method of choice will depend on many factors, but will be highly dependent on the ground conditions, such as type and depth of soil that requires treatment. Similarly, the criteria that have to be satisfied can govern the method of choice. The most suitable method for defining acceptance criteria is performance based with post-treatment quality control geotechnical testing that is able to directly link testing to the design criteria. Testing must be purposeful and aimed at demonstrating that acceptance criteria have been satisfied and can include more than one method in large-scale projects.

All tests have their advantages and disadvantages. The SPT provides disturbed samples that allow visual identification of soil type that can be very important in the selection of the ground improvement technique. Liquefaction analysis was also originally developed based on this technique, which has resulted in a very large databank of information; however, the test is not applicable or does not result in reliable results in some ground conditions. The CPT provides the soil stratigraphy and behavior type, which are also very important for selecting the ground improvement technique. However, the basis of calculating settlement and bearing capacity by using SPT or CPT are empirical and based on correlations that should be used with extreme caution, especially if ground conditions vary from the basis of the correlations. Bearing capacity and settlement parameters are the direct outputs of the PMT, which greatly improve the reliability of the pseudo-analytical bearing and settlement calculation processes. PMT can also be performed in the widest range of ground conditions; however, the quality of drilled holes will depend on the driller and his expertise and this testing system is yet to provide soil stratigraphy.

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