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CONTENTS

Chapter 25

The role of ground improvement

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Ground improvement can take many forms to cause temporary or permanent change, usually with some specific engineering purpose in mind. Common with all geotechnical design, specific engineering purposes (e.g. stable foundation) must be defined and the ground's current state in the zone of influence fully established. Analysis of the problem in light of this information usually yields a range of solutions, some of which might be altering the ground's properties. Techniques are available to strengthen or stiffen soft or weak ground, either by physically altering its structure or by changing the ground's chemical properties. Similarly, groundwater flow regimes can be altered to achieve greater stability or to stop flow of contaminants. Any analysis of ground treatment must therefore encompass the engineering purpose(s), the ground's current state and its potential to be altered, the proposed means of its alteration, and any side effects of treatment. Physical ground improvement includes application of static, vibrationary or dynamic loading; water can be drained using gravity, increased stresses or an electrical gradient; and chemicals can be introduced as solids or liquids. Hybrids of these processes and lateral-thinking approaches (e.g. ground freezing) complete a range of alternatives to 'structural' solutions and often provide more sustainable outcomes.

25.1 Introduction

Ground improvement covers any method by which the ground, whether natural or disturbed in some way by anthropogenic processes, has its performance for any specific (geotechnical) purpose enhanced. Typically the goal is strengthening and/or stiffening of soft or weak soil or fill, either temporarily or permanently, although other specific improvements (e.g. reduction or increase of hydraulic conductivity) might bring about the desired change or be the goal of the exercise. Equally the goal might be to make the properties worse (e.g. slurries introduced at the face of tunnelling machines to facilitate excavation and transportation of soil), while some techniques have unwanted side effects (e.g. vertical stone columns provide a route for venting gases), so the term 'improvement' should be treated with caution. Nevertheless it can be generally stated that a ground improvement process has some associated desired physical or chemical consequence, such as:

- settlements of overlying or embedded structures being reduced, speeded up, or rendered unimportant;
- the risk of failure being removed or reduced;
- the need for excessive or unnecessary use of natural resources (import of virgin material to a site) and/or waste generation (poor ground disposed of in landfill) being obviated;
- the movement of chemicals being enhanced (for extraction) or inhibited (for containment);
- the excavation of contaminated ground for treatment, thereby exposing it to the environment, being avoided.

Ground improvement for physical purposes is generally an alternative to structural solutions which either bypass the soils concerned (e.g. deep foundations) or act in spite of the soils concerned (e.g. rigid raft foundations). Ground improvement for chemical purposes might be used instead of permanent physical solutions (e.g. barriers to stop chemical contaminant migration), novel semi-permanent processes (e.g. permeable reactive barriers; see Boshoff and Bone, 2005) or costly temporary operations (such as pumping for groundwater lowering to generate an inward hydraulic gradient). Phear and Harris (2008) track developments in this subject.

There are very many reasons for undertaking some form of ground improvement. As with all engineering undertakings, the costs and benefits of the alternative engineering solutions must be examined, and this should be done using the broad three pillar (environmental, social and economic) or four pillar (adding natural resources, see Braithwaite, 2007) sustainability models (Fenner *et al.*, 2006). This is a most important aspect of an engineer's role due to the wider impact of geotechnical processes (Jowitt, 2004) – in the category of resource use falls energy (embodied and directly used) and hence CO_2 emissions, water, and similar concerns. The sustainability arguments are highly germane to ground improvement; if what is in place can be made adequate by some means, then the environmental and social (and resource) benefits are usually very substantial indeed, while economic benefits are also often favourable. The civil engineering industry has now moved to a state of global awareness that demands it goes beyond the traditional 'cost, quality and time' approach to engineering design. Ground improvement alternatives to 'structural' solutions represent an excellent example of how these considerations play out.

The aim of this chapter is to introduce the fundamental principles of ground improvement techniques, the detailed manifestations of which can be seen routinely in academic or professional journals. The scope of this chapter is limited to hydraulic, physical and chemical means of improvement. Soil reinforcement via the introduction of tensile elements in the form of grids, strips, bars, fibres or similar is covered elsewhere (see Chapter 86 *Soil reinforcement construction*). Uncommon techniques, such as the thermal treatment of clays and the use of bitumen, fly ash or other 'non-standard' additives to improve soil properties, lie outside the scope of this manual. For a more complete review of ground treatment techniques, refer to Mitchell and Jardine (2002). Hereafter the term 'soil' will be used to represent undisturbed natural, disturbed natural and artificial ground (such as fills).

25.2 Understanding the ground

Common with all geotechnical design, the specific geotechnical engineering purposes for which the site is to be used must first be defined. These purposes in turn will define what is expected of the ground. So, for example, if the site in question is to house a multi-story building, the expectation of the ground is to provide a stable foundation, whereas if a river is to be protected from contamination migrating from an adjacent industrial site, the expectation of the ground is to provide a barrier to groundwater flow. In light of these expectations, the current state of the ground in the appropriate zone of influence (the influences of the stresses imposed by the building in the case of the foundation, or the potential routes through which the contamination might flow to the river) must then be established. Moreover, the future state of the ground for the design life of the expectation must equally be anticipated.

A useful tool for this analysis is presented by the 'Burland Triangle' (**Figure 25.1**), which provides a framework in which to explore whether the expectation can be met by the ground as it exists and is likely to continue to exist on the site in question. A site investigation will be conducted to establish the ground profile and provide clues to its geological history, and this provides the first pointer to the future likely state of the ground; for example, whether exposure might lead to further weathering of the natural soil and a change in its properties. If the building is to be founded on a clay soil at a certain depth below the ground surface, for example, then the net stress increase (maximum likely stress imposed by the building minus the stress relief due to excavation of soil above the foundation depth) must be determined, although of course it could be a stress reduction were it a lightweight structure founded at depth. The make up of the soil (see Chapter 14 *Soils as particulate materials*) will usually be established by the site investigation, while the properties of the soil (strength and deformation behaviour, see Chapter 17 *Strength and deformation behaviour of soils*) will likewise be established via laboratory testing; for a clay soil the deformation may take several years before it is complete.

Some form of modelling will determine whether, in general terms, the ground can fulfil the geotechnical expectation placed on it, taking cognisance of the ground properties, while an understanding of the geotechnical context of the wider area will provide clues as to whether the ground properties are likely to remain stable in the long term. Groundwater flow regimes (see Chapter 16 *Groundwater flow*) are important here, as they

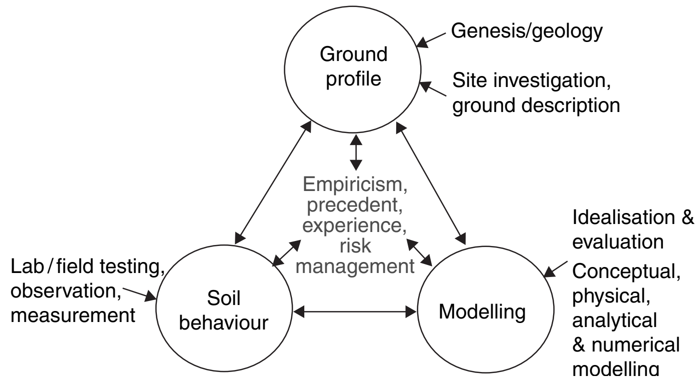

Figure 25.1 The 'Burland Triangle' for analysing the ground and how it might react for a geotechnical engineering purpose (Burland, 1987). Ground improvement processes can be analysed in the same manner

are for the second example of contaminant transport. Similarly, changes to the groundwater profile are important since they will alter the effective stress regimes operating on the site (see Chapter 15 *Groundwater profiles and effective stresses*), which in turn will alter the soil properties. The important point here is that sites are dynamic – the ground properties can change with time, and potential future changes must be accounted for in any geotechnical design, whether it is a conventional 'structural' solution (concrete foundation or cut-off wall) or a ground improvement process that is adopted. Analysis of the problem in the light of the totality of the above information will usually yield a range of solutions, some of which might be to alter the ground's properties. The remainder of this chapter is devoted to outlining the ways in which ground properties can be altered to obviate the need for more elaborate 'structural' solutions, and thereby provide additional weapons in the geotechnical engineer's arsenal.

25.3 Removal of water

25.3.1 Introduction

Water in soil is usually a nuisance (it gets in the way of construction operations) and, more importantly, the strength of a soil is usually very sensitive to both water content and water pressure (see Chapter 15 *Groundwater profiles and effective stresses*). Water is added to soil in certain situations (for example, to aid compaction of a granular soil or to weaken a clay soil to aid excavation) but on the whole, it is a hindrance. The problems posed by water in the context of ground improvement are generally that there is too much of it and/or it cannot escape quickly enough when additional, superimposed stresses are applied to the soil. Moreover, if water flows it can strengthen or weaken the soil according to the direction of flow and the pore water pressure distribution generated by the flow, which can be determined from a flownet (see Chapter 16 *Groundwater flow*). Only saturated soil (soil mineral particles and water) will be considered in terms of the need for water removal, which is the primary focus of the discussion hereafter. Natural groundwater flow is excluded from this discussion, whereas techniques that induce the flow of water out of the soil are central to ground improvement.

From Terzaghi's law of effective stress

$$\sigma' = \sigma - u \quad \text{or more generally} \quad p' = p - u \qquad (25.1)$$

where σ' is the effective normal stress on any given plane in a soil mass, σ is the equivalent total normal stress, u is the pore water pressure, p' is the mean normal effective stress in three dimensions and p is the equivalent mean normal total stress (all measured in kPa). The parameter σ' (or p') is the factor which determines strength of the soil on any given plane (or in three dimensions). If the pore water pressure increases above its original level, σ' (or p') reduces, and *vice versa*.

Given that the pore water pressure increases linearly with depth below the groundwater table in the majority of cases (see **Figure 15.3**, Chapter 15 *Groundwater profiles and effective stresses* and associated text for exceptions to this rule) and water is held by capillary suction such that the suction (or negative pore water pressure) increases with height above the water table until the suction can no longer be maintained (which can be many metres in fine-grained soil having very small pores), one obvious means of ground improvement is to lower the groundwater table by some means of drainage. This principle stood the Romans in good stead when they were creating their roads, or (literally) their highways, in areas with a naturally high water table which adversely influenced the properties of fine-grained soil. Of course the same principle informs our current drainage specification for roads, railways, airport runways and other such traffic-carrying infrastructure. By digging deep ditches on either side of their roads, the Romans depressed the water table, thereby increasing the suction in the soil above the water table, or making u (the pore water pressure) more negative in equation (25.1). This in turn increased σ' (or p') and made the soil stronger (see **Figure 25.2**). Not only this, but by creating a cambered surface and paving it with stone flags, they minimised the possibility of water entering the road pavement structure from above and thereby maintained the suction regime. Drainage is therefore an assured means of ground improvement and is used widely: behind retaining walls, in slope faces and adjacent to dam cores, for example.

Fine-grained soil formation typically involves soil particles transported by water and settling in slow flowing conditions, such as in the base of lakes or the sea. The soil starts off as very wet indeed, but as more soil particles are progressively laid down above it, the soil densifies as water is progressively removed from the soil – it is squeezed out as the vertical (and hence mean normal) stress builds up by a process known as consolidation. The relationship between volume and mean normal effective stress is described by the normal consolidation line (NCL) and is shown using critical state theory terminology in **Figure 25.3**. A natural soil may undergo sequences of erosion and deposition, whereby the effective stress it experiences reduces and increases. One such sequence is shown in **Figure 25.3**, in which the soil is progressively consolidated as p' increases past points A and B until point C is reached. After this, p' reduces (due to erosion) until point D is reached. The degree to which the soil swells back relative to the projection of the critical state line (CSL) is important in determining the future response of the soil to applied stress regimes. The point on the NCL at which the soil at point D (and at any point on the Swellback Line, which is idealised as a straight line here for simplicity; in truth it is curved) has experienced its maximum p', is termed the *preconsolidation pressure*. If the soil at point D is thereafter reloaded, it will recompress and its state will move down the path of the recompression line

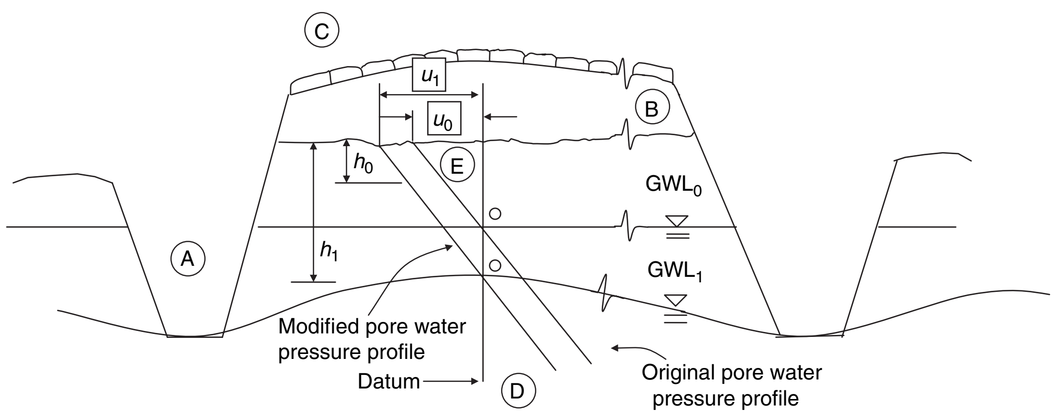

Figure 25.2 The effects on the strength of soil due to groundwater table lowering, as illustrated by a Roman road construction
A – Deep ditches on either side of road lower groundwater level (from GWL_0 to GWL_1)
B – Excavated soil creates highway
C – Stone flags on cambered surface limit surface water ingress
D – Pore water pressure increases linearly with depth below GWL
E – Suction, or negative pore water pressure, increases linearly with height h above GWL such that suction $u_1 > u_0$

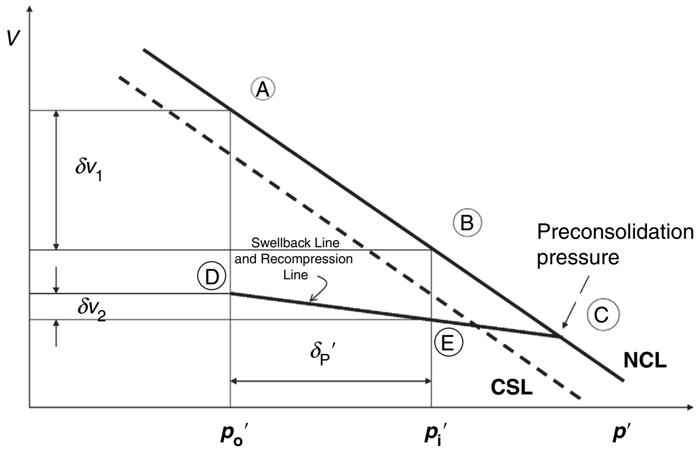

Figure 25.3 Graph of volume against mean normal effective stress

(idealised here as linear and coincident with the Swellback Line) to reach point C. It will then follow the NCL downwards once more.

In practice, if we encounter the soil at point D in equilibrium ($p' = p_0'$), and apply a superimposed stress to it of $\delta p'$, say due to a new building, according to the idealised graph the soil state would move to point E and the amount of volume change would be δv_2 once consolidation due to the imposed stress was complete, i.e. once water had been caused to flow out of the soil and a new equilibrium at $p' = p_1'$ had been reached. If, however, the soil encountered was at point A, also in equilibrium at $p' = p_0'$, and a similar stress of $\delta p'$ was imposed on it due to construction of the same building, then the far greater volume change of δv_1 would result once a new equilibrium at $p' = p_1'$ had been established as the soil state moved to point B; far more water would have been driven out of the soil to achieve this state. For practical purposes, therefore, it is desirable to ensure that the stresses imposed by construction activities remain below the preconsolidation pressure. One form of ground improvement is to create this situation, i.e. induce artificial overconsolidation.

The velocity (v in ms^{-1}) at which water flows through a saturated soil, which is a particulate material with interconnected pore spaces, is governed by the driving force (the hydraulic gradient, i, which is dimensionless) and the resistance to flow (the hydraulic conductivity, k in ms^{-1}) according to Darcy's Law:

$$v = ki. \qquad (25.2)$$

When an increment of total stress (δp) is applied to a soil, this stress is instantaneously carried by the pore water and creates an excess pore water pressure of $\delta u = \delta p$. Water flows from high to low pressure and therefore away from this area of high pressure, thereby progressively reducing δu to zero, causing $\delta p'$ to rise progressively until it increases by δp. The driving force, i.e. the hydraulic gradient, is therefore due to the excess pore water pressure induced by the increment of stress (δp). In effect we have created a seepage problem, for which isochrones could be drawn to illustrate the direction of flow and progression of consolidation within the soil. In fine-grained soil, the rate of flow is small, with coefficients of hydraulic conductivity ranging from ~10^{-5} ms^{-1} for clean silts to ~10^{-11} ms^{-1} for heavy clay soils, while the distance that the water has to flow clearly governs the time required for consolidation to occur.

25.3.2 Pre-loading

Returning to the previous example illustrated in **Figure 25.3**, if we encounter a soil at point A and apply temporarily a stress equal to (applying $\delta p = \delta p'$ would take the state to point B) or greater than (say to point C) the stress subsequently to be imposed by the building, and were to allow the stress to remain in place for a sufficient length of time for consolidation of the soil to occur fully, i.e. for the water to flow out and equilibrium to be regained, then removal of the temporary stress and imposition of the new stress due to the building would cause only a small amount of volume change. Unloading (by removal of the temporary stress at points B or C) would cause the soil to swell, i.e. move upwards along parallel Swellback Lines, while the addition of the building would recompress the soil, i.e. move downwards along the relevant Recompression Line (being coincident with the Swellback Line). If this process of stress removal and replacement by the building were to take place quickly, and/or if the soil had a small coefficient of hydraulic conductivity (k), or the flow distance for the water to exit the soil is large, then the heave caused by the unloading and the compression caused by the building could be very small, and for buildings or foundations that are very sensitive to settlement, this process can be used to advantage to create negligible settlements.

This process is effective, especially for soils in which secondary compression (creep) is significant, but it is slow. One way to speed up the process is to increase the temporarily applied stress by increasing the driving force element. However, for thick soil deposits, especially where the direction of flow is upwards only (e.g. the soil overlies an impermeable rock), it can prove impractically slow; the rate of flow is proportional to the square of the drainage path. The temporary load can be earth or rubble fill, although water tanks or vacuum preloading (by pumping from beneath an impervious membrane) have been used. Placement of fill, ensuring continued drainage, is effected in layers as quickly as possible (staged loading is commonly needed to avoid overstressing the soft, weak soil being treated and avoiding failure); likewise, fill is removed as quickly as possible (staged removal is not necessary). The advantages are that this is a simple method that is easy to effect and certain to work, and is suitable for soils that undergo large volume decreases. It can, however, require a long time and much material, which has to be double handled.

25.3.3 Vertical drains

Although it adds to the cost and complicates the construction process somewhat, the time required for consolidation using pre-loading can be greatly reduced by installing vertical drains. The rate of consolidation depends upon the length of the drainage path and the hydraulic conductivity of the soil associated with this path. In general, the water will flow vertically on loading to exit the soil and $k_{vertical}$, which is governed by the lowest coefficient of hydraulic conductivity of the substrata (e.g. interlayered fine sands, silts and clays in alluvium), will determine the rate of flow. By permitting horizontal drainage to vertical drains installed at predetermined intervals, not only is the length of the drainage path greatly reduced, but the rate of flow is governed by $k_{horizontal}$, rather than $k_{vertical}$. In general, the horizontal coefficient of hydraulic conductivity is far greater since it is the higher of the values of the substrata that dominate. For interlayered sands, silts and clays, this can mean a few orders of magnitude. This is why normally consolidated or lightly overconsolidated alluvial soils and stratified soils are most effectively improved using vertical drains. Types of vertical drains include sand-filled boreholes (though horizontal shearing would compromise their performance), sand-filled stockings that are inserted into vertical boreholes, and proprietary band drains consisting of a strong, flexible core (manufactured from polyethylene or similar) containing drainage channels and covered by a filter fabric. A horizontal free draining layer is constructed at the top of the vertical drains prior to application of the pre-load. Connection with an underlying permeable layer (e.g. sand or gravel) is also achieved where possible. Smearing of the borehole sides should be avoided so that horizontal drainage is not compromised (Hird and Moseley, 2000).

25.3.4 Electro-osmosis

When a voltage gradient is induced across a soil, the water in the pore structure of the soil moves from the anodes (often metal rods) to the cathodes (e.g. perforated metal pipes creating a series of filter wells), which are water abstraction points. The electro-osmotic flow of water through soil is expressed by:

$$v_e = k_e \, i_e \tag{25.3}$$

where v_e is the electro-osmotic flow rate, k_e is the coefficient of electro-osmotic permeability (measured in m^2/Vs) and i_e is the potential gradient = V/L (where V is the potential difference and L is distance between the electrodes). This is analogous to Darcy's Law for hydraulic flow (equation (25.2)), yet there is a fundamental difference since k_e is commonly considered to be independent of the size of the pore spaces, while k is very strongly influenced by pore size. The technique is therefore most effective in fine-grained soils, in which k_e is typically 10^{-8} to 10^{-9} m^2/Vs (Mitchell and Soga, 2005, p. 275) whereas k can vary by orders of magnitude down to ~10^{-11} ms^{-1} (Mitchell, 1993, p. 270 provides example comparisons). Electro-osmotic flow is primarily caused by cations in the water surrounding negatively charged clay particle surfaces being transported towards the cathode drawing water (a polar molecule that is attached to the cations) with them. (Very occasionally, positively charged surfaces surrounded by anions will result in flow towards the anode.)

There are many phenomena acting (see Liaki *et al.*, 2008), and some can bring about additional improvements:

- Degradation of a metallic anode and transport of metal ions by electromigration can cause improvement of clay soils via cation exchange on the exchange sites of clay minerals.
- Transport of stabilising agents added as aqueous solutions (electrolytes) at the anode (anolyte) and cathode (catholyte) can result in stabilising reactions.
- Electrokinetic remediation can be used to draw contaminants to the electrodes where they can be removed, hence remediating chemically contaminated sites (Ottosen *et al*., 2008).

Electro-osmosis can be used for temporary dewatering in soils not readily treated by well pumping, or for permanent improvement, especially when accompanied by chemical changes. Moreover, it is fast and can be used in a confined area, though it remains little used in practice. The soil immediately adjacent to the anodes tends to dry out, causing increased resistance and current requirements, but can be countered by periodic short duration reversals of polarity or by an appropriate anolyte solution. Mitchell and Soga (2005) is a particularly helpful reference on the subject of ion movements.

25.3.5 Groundwater lowering

Well pumping is conceptually simple, being more of an accepted construction technique than a geotechnical process, and works best in soils with relatively high coefficients of hydraulic conductivity such as sands and gravels (see Chapter 80 *Groundwater control*).

25.4 Improvement of soils by mechanical means

25.4.1 Introduction

Compaction is widely used to increase the density of soils, and thereby increase their strength and stiffness, notably in road construction where surface compaction technology has reached a considerable level of sophistication. The principal goal is to cause the soil particles to pack together as closely as possible so as to minimise voids and to maximise 'particle interlock' – so that when the soil is sheared, the soil in the sheared region must dilate (i.e. particles must do work against the normal, or confining, force) as well as doing work against the frictional force. The types of rollers available include smooth-wheel dead weight rollers, vibrating rollers and sheepsfoot rollers (see Chapter 75 *Earthworks material specification, compaction and control*). Compaction technology has developed into deep compaction for more general ground improvement based on both vibratory and impact methods. Explosives have also been used to induce deep densification (Hausmann, 1990), but this approach lies outside the scope of the standard approaches and is not covered. A static form of compaction is achieved by pre-loading, which is covered in section 25.3.2.

25.4.2 Dynamic compaction

Compaction of soil by the repeated dropping of weights on the ground has been practiced for centuries with the aim of improving the bearing capacity. The extension of this technique to dynamic compaction, involving heavy weights and large drop heights, was used in the early 1970s to provide a stable foundation for structures of large plan area and for reclaimed land development. The principle of operation is simply based on the application of large vertical stresses to a considerable depth, thereby compressing the soil skeleton directly where the degree of saturation (S_r) is low (Gu and Lee, 2002). Where S_r is not low, high excess pore water pressures are generated; if they are sufficiently high they will cause failure and fissures (or cracks) to be created extending from the surface to significant depths. As the excess pore water pressures progressively dissipate, aided greatly by the fissures providing convenient drainage paths, the bearing capacity of the soil increases such that it exceeds the previous maximum value.

Dynamic compaction is effective in coarse-grained fills (including decomposed domestic waste and industrial fills) and soils, but relatively ineffective in soft clays and peats that absorb the applied energy. The weights, or pounders, typically have a mass of 10–20 tonnes (100–200 kN) and are typically dropped, using a crane, from heights of 10–20 m at wide intervals initially, the spacing being progressively reduced. Repeated application of the pounder causes craters, which are backfilled with coarse fill prior to the next series of load applications. The surface layers require rolling at the end of the process. It is a simple, rapid, relatively inexpensive method of treatment involving large equipment (though it cannot be used in confined spaces) and considerable ground vibration (hence it cannot be used adjacent to sensitive buildings or structures). A depth of compaction of 5–6 m can be easily achieved; a 'rule of thumb' for the treatment depth given by Hausmann (1990) is $0.5\sqrt{(WH)}$, where W is the mass of the weights, and H the height from which they are dropped. The same principle is adopted by *rapid impact compactors*, which were developed to provide a more controlled, rapid and thus readily applied form of the technique (see Serridge and Synac, 2006) and which typically deliver an energy input of around 8 tonne metres at a rate of 40 blows per minute. Guided or leader dynamic compaction rigs are also available.

25.4.3 Vibrocompaction

Vibrocompaction (sometimes termed vibroflotation) aims to cause deep densification, and hence strengthening and stiffening, of a specifically designed volume of ground by means of lateral vibration using a poker lowered by a crane. The poker contains an eccentric weight mounted on a vertical shaft which is rotated by a motor. The poker vibrates laterally thus compacting the ground while sinking to the design depth under its self weight. This is known as the 'dry method' of operation; the 'wet method' involves forcing water through the end of the poker while it is lowered into the ground, thereby producing what is stated to be a 'flushing effect'. The principles that underlie the standard compaction curve (described in section 25.5.2 and **Figure 25.4**) also apply here. A void is thus formed within an area of dense, compacted soil. The poker is withdrawn and a granular fill is dropped into the void and is compacted in 250–500 mm lifts by lowering the poker to refusal. The purpose of the granular fill is to compensate for the reduction in volume due to compaction of the existing soil. Vibrocompaction thus aims to create a uniformly dense, uniformly stiff block of soil onto which to locate some type of foundation. The size of each treated block of ground is designed to accommodate the bulb of pressure beneath the foundation, i.e. the plan area of the treated block is square for a pad foundation (using typically 4, 9, 16 or 25 insertions) and linear for a strip foundation. The poker typically compacts the soil to a radius of 1.0–2.0 m from the insertion point, the volume decrease being generally about 5–10%, and occasionally up to 15% (see Chapter 85 *Embedded walls*).

Loose natural soils and fills ranging from fine sand to coarse gravel are most effectively treated, while coarser material is difficult to penetrate unless it is well graded. A high fines content prevents rapid dissipation of the excess pore water pressures that are generated by vibration in soils with a high degree of saturation. It also serves to dampen the vibration, hence reducing the area of influence of compaction. Cohesionless soils with less than 15% fines are consequently commonly quoted as being suitable; see Slocombe *et al.* (2000). The technique has also been widely applied to fills consisting of brick rubble, ash and general demolition debris, the voids in these materials being broken down by regular insertion of the poker and granular fill forced into the remaining spaces.

25.4.4 Stone columns

The same equipment and construction methods are used as for vibrocompaction, although a single-sized crushed rock is used as the granular fill. The end result is a 0.9–1.2 m diameter column of dense stone within a less competent stratum – typically a saturated weak/compressible fine-grained soil, though stone columns can in principle be installed in any soil type. In this case, the vibration does not necessarily influence the soil between the columns significantly, especially when the soil has a high fines content and a high water content. This will lead to largely undrained shearing displacements since there is no time for the water to flow out of the soil (although any air within the soil voids would of course be compressed). The process of adding stone and compacting it using the poker, however, will inevitably cause stone to be forced sideways into the soil until it is resisted. Three zones of material might be considered once the process is complete: largely undisturbed soil that has undergone some displacement due to undrained shearing, a soil–stone intermediate zone, and the compacted stone column itself. This technique is sometimes known as vibroreplacement or vibrodisplacement.

The stone columns serve to reduce settlement of overlying foundations by acting as bearing piles and to strengthen the soil by virtue of zones of granular material having a much higher shear strength than the soil. It is also suggested that the stone columns act as vertical drains for the rapid dissipation of excess pore water pressures, although this remains a matter of debate associated with the degree to which the displaced soil fills the voids between the compacted stone. The depth of stone columns rarely exceeds 12 m, although lengths approaching 30 m have been recorded. The column fill typically consists of uniform 20–50 mm crushed stone, gravel or slag. Surface disturbance (due to a lack of confinement to resist the compaction energy) means that surface rolling is required after the columns have been formed (see Chapter 85 *Embedded walls*).

25.4.5 Micro-piles or root piles

Micro-piles were originally introduced for underpinning operations. They are now used for slope stability problems, retaining structures and underground construction. Micro-piles are constructed primarily to carry tensile and compressive forces and thereby increase the strength of soil for a particular purpose. They have only a limited degree of flexural (or bending) resistance and are therefore generally constructed at appropriate angles to carry direct forces. They can be used in most types of soil.

To create a micro-pile, a casing is jetted, driven or drilled to the desired depth, reinforcement is inserted into the casing and cement mortar is pumped in to fill the hole while the casing is withdrawn. In the case of root piles, the cement mortar is forced into the holes under higher pressure using a grouting technique so that it penetrates cracks or soft layers around the void. This creates a rough pile surface, thereby gaining maximum skin friction.

25.4.6 Soil nails, ground anchors and soil reinforcement

These techniques do not really lie under the heading of ground improvement and they are mentioned here only for completeness. Their purpose is to give the soil into which they are installed or buried an enhanced strength, either by providing tensile elements, increasing the mean normal effective stress, holding in place some form of structural facing to retain soil, or a mixture of all three. Soil nails are described in Chapters 72 *Slope stabilisation methods*, 73 *Design of soil reinforced slopes and structures*, 74 *Design of soil nails* and 88 *Soil nailing construction*. Ground anchors are described in Chapters 64 *Geotechnical design of retaining walls*, 87 *Rock stabilisation* and 89 *Ground anchors construction*. Soil reinforcement is described in Chapters 62 *Types of retaining walls*, 64 *Geotechnical design of retaining walls*, 72 *Slope stabilisation methods*, 73 *Design of soil reinforced slopes and structures* and 86 *Soil reinforcement construction*.

25.5 Improvement of soils by chemical means

25.5.1 Introduction

The addition of chemicals to clay soils to bring about improvements in strength, stiffness and volume stability (i.e. to inhibit shrinkage and swelling) or for other purposes, is hardly new – the Romans were adept at using the technique. The fact that lime has been used for this purpose in stabilising clays is equally unsurprising since limestone and shale (which is highly compressed clay) are the two primary ingredients of cement; cement factories being located where limestone and shale outcrop together. Other than the use of chemicals to dry very wet soils and make sites workable following heavy rain (quicklime is particularly effective for this), chemical soil stabilisation can be divided into two categories: (i) soil improvement as a result of the combined action of the soil constituents and the stabiliser to create a cemented product (as occurs by mixing lime and clay); and (ii) addition of a stabiliser that binds the soil particles together (as is the case with a sand-cement mix). The traditional manner of introducing the stabiliser is either as a powdered solid or a liquid suspension that is mixed with the soil at the surface, laid and compacted. However, more modern techniques have evolved to mix the stabilisers at depth or to pump stabilising solutions, such as cement grouts, into the ground. There are several applications (see for example Rogers *et al*., 1996) and while all have different requirements, the basic principles remain the same. Problems can be encountered when stabilising soils having a high plasticity index due to difficulties in achieving a thorough mix and ensuring a sufficiently large volume of stabiliser to chemically modify the clay minerals, leading to some concern over long-term softening due to water uptake.

25.5.2 Lime and cement stabilisation

Traditionally, lime has been used to improve the properties of clay soils by surface mixing, wetting where necessary, laying and compaction. Mixing lime with a clay soil alters its physical properties by fundamentally changing its nature as a result of cation exchange – a process termed *modification*. If sufficient lime has been added, and the modified material is compacted and allowed to cure for a significant period of time, the material cements by a process termed *stabilisation* (or *solidification*). The technique is most commonly used as a general ground improvement process, essentially to improve soft, weak clay soils, but it can also be used to treat contaminated soils.

Lime is a general term used to describe either quicklime (calcium oxide, CaO, created by heating hydrated lime to 450°C), hydrated lime (calcium hydroxide, $Ca(OH)_2$, which is formed when CaO comes into contact with water or water vapour) or lime slurry (a mixture of hydrated lime with water). Agricultural lime refers to calcium carbonate ($CaCO_3$), which is unreactive and will form if lime comes into contact with the atmosphere; carbonation must be avoided. In its simplest application, quicklime is used as a construction expediency to dry out sites that are unworkable due to water via a strongly exothermic reaction that turns CaO into $Ca(OH)_2$; dehydration taking place due to the reaction and to steam being released. Lime is simply spread evenly on the surface to effect this improvement, perhaps with some limited surface mixing.

Lime modification occurs as a result of cation exchange, the calcium ions exchanging with the cations on the clay mineral sites (usually monovalent sodium or potassium ions), which causes:

- a reduction in the thickness of the adsorbed water layer, and hence a reduced susceptibility of the clay to water;
- flocculation of the clay particles, due to greater electrical attraction (as the water layer has thinned);
- an increased internal angle of friction, i.e. greater shear strength;
- a textural change from a plastic clay to a friable material that appears granular in nature;
- a reduced plasticity as a result primarily of the plastic limit (PL) rising considerably.

To bring about this improvement the lime is spread evenly on the surface, mixed thoroughly with the clay to the required depth using a rotavator and allowed to mellow (or cure) for 24–72 hours. This timespan will ensure that all quicklime is fully hydrated (it hydrates via an expansive reaction and this must not be allowed to happen post-compaction), and that the cation exchange reactions are complete – the surface of the mixed material having been lightly rolled to seal it from rain and to inhibit carbonation. Lime modification can be achieved with the addition of a modest quantity of lime that is determined using the initial consumption of lime (ICL) test, which establishes the quantity of lime required to fully satisfy the cation exchange reactions. The ICL test is based on measuring pH changes in the soil, the ICL value being the lime addition that causes the pH to rise to 12.4 (or failing that, 12.3). This result can be confirmed by Atterberg limit tests of clay mixed with lime at different quantities.

To bring about the lime stabilisation reactions, sufficient additional lime is needed to raise the pH to a high level (>12). Also, once the cation exchange demand of the clay has been met, additional calcium ions are needed to react with constituents of the clay minerals (which are dissolved in the high pH environment) to achieve stabilisation via pozzolanic reactions (Boardman *et al.*, 2001). It is important that, as the reactions are taking place, there is a close proximity of the clay flocs so that the reaction products can bind the particles together in a coherent cemented mass. This process takes place in an environment that is free from chemical or organic contaminants that adversely affect the reaction processes. The reactions can be superficially stated as:

$$Ca^{2+} + OH^- + Al_2O_3 + H_2O \rightarrow \text{calcium aluminate hydrate gel}$$

$$Ca^{2+} + OH^- + SiO_2 + H_2O \rightarrow \text{calcium silicate hydrate gel}$$

and are the result of alumina (Al_2O_3) and silica (SiO_2) dissolving from the clay mineral under conditions of high pH. The two gels crystallise over time to yield strong, brittle bonds. This causes a considerable increase in strength and results in a material that is subject to brittle fracture (i.e. is non-plastic).

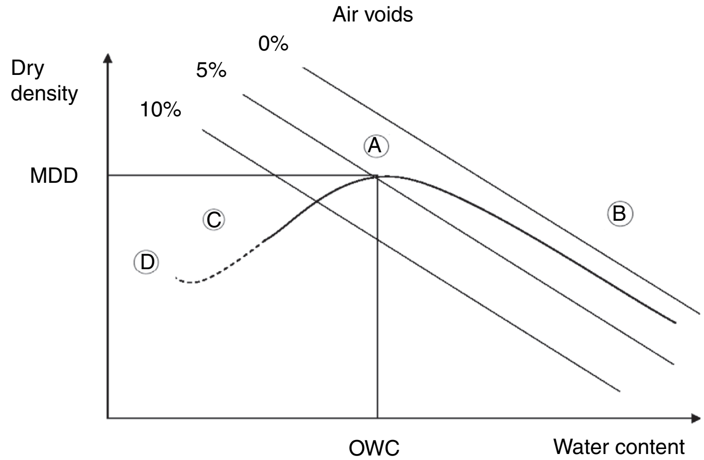

Figure 25.4 A standard compaction curve for fine-grained soil. Compaction seeks to maximise density and minimise air voids; for this application both of these compaction objectives are important
A – The maximum dry density (MDD) is achieved at the optimum water content (OWC) at which point the soil has minimum resistance to shear, hence densification
B – Additional water inhibits densification; undrained shearing occurs at progressively greater water content (hence lower dry densities)
C – As soil dries below OWC, suctions progressively increase thereby inhibiting shear under compaction stresses and the dry density falls
D – Where there is insufficient water, suction is lost and the dry density increases

The construction process is similar to that of modification, except that the mellowing process should not be as long (24 hours should be ample, and even this length of time has been considered to be detrimental to the long-term performance of the lime stabilised clay). The material should then be remixed, adding water if necessary to bring the water content up to the right level (between 0 and 2% above the optimum water content so that density is maximised while air voids are minimised, see **Figure 25.4**). It can then be placed and compacted in layers of appropriate thickness for the compaction energy applied, to ensure a density close to the maximum dry density throughout the full depth of the layer. The material should then be rolled with a smooth-wheeled roller if necessary to seal the surface while ensuring that there is a slight fall to avoid water ponding. Finally, it needs to be left to cure, free from frost and traffic until the reactions have advanced. The sulphate content must be sufficiently low (some specify <0.5%, others <1.0%) to avoid the creation of ettringite which forms via a highly expansive reaction, destroying the compacted state and compromising long-term strength. Organic material in the clay can be detrimental because it tends to be acidic, and hence lowers the pH, but more importantly calcium reacts preferentially with the organic material and thus less is available for the stabilisation reactions. The most assured method of design is to carry out laboratory trials of compacted mixes with different lime contents above the ICL value and allow them to cure for different periods before being tested for strength.

Cement stabilisation can be used in any soil (there is no requirement for the alumina and/or silica from the clay mineral). The principles behind the technique and the processes of design and construction are similar. A larger quantity of cement than lime is usually required to provide sufficient binding agent to create a matrix sufficient to encompass the soil particles. Cement contains a significant proportion of quicklime and, like lime stabilisation, the use of cement causes a large rise in pH. This effect can be reduced if a cement replacement material, such as ground granulated blast furnace slag (GGBFS), is used.

25.5.3 Lime, lime-cement and cement columns

The modern techniques of lime column construction have been developed in Sweden and Japan, although there is now a far greater use of cement only and lime-cement mixes rather than lime only. This technique, commonly referred to as 'deep mixing', has generally only been carried out in soft/weak soils due to the difficulty of operating the mixing plant in stiff/strong soils. The principles listed above remain valid for the deeper process, notably the need for sufficient water for the reactions to take place, the mixing to be achieved thoroughly and the resulting material to be adequately densified. The mixing takes place with a purpose-designed tool that is augered into the ground; the densification occurs by counter-rotation of this tool such that there is downward pressure on the mixed material as the tool is withdrawn. The correct water content can be achieved by the inclusion of quicklime in a powder mix (nowadays consisting primarily of cement to guarantee the reaction products) to dry out wet soils, or, by using a cement slurry in cases where additional water is needed (see Chapters 84 *Ground improvement*).

25.5.4 Lime piles

An alternative stabilisation technique, commonly restricted to clay slope stabilisation, is to create lime piles, which are columns of compacted quicklime. The quicklime hydrates via the exothermic hydration reaction causing water content reduction and immediately reducing the pore water pressure and creating suctions; these have been found to arrest slope movement (Rogers *et al*., 2000a). The lime hydration releases calcium and hydroxide ions into the ground and they migrate from the quicklime pile into the surrounding soil, which is thus stabilised, to a distance that is dependent on the water content of the soil and the reactivity of the clay (Rogers and Glendinning, 1996). This distance is usually a few tens of millimetres, although clay cracking due to the rapid dehydration causes fissures that radiate from the pile and these provide conduits along which the lime migrates and stabilises the clay further away from the pile. The process can be enhanced by the inclusion of different additives, GGBFS being particularly effective (Rogers *et al*., 2000b).

25.5.5 Lime slurry pressure injection

Lime slurry pressure injection is another alternative that borders on the use of cement grout, but remains distinct since the aim of the process is to cause chemical changes in the clay soils into which the slurry is injected. A lime-water mix is injected at different depths into the clay on a grid pattern with the aim of creating an approximately uniform distribution of slurry. Migration of ions into the soil adjacent to the slurry seams results in the classical lime stabilisation reactions occurring. The technique has been used for clay embankment stabilisation in the USA, but has been little used in the UK and requires further research before its efficacy can be proved.

25.5.6 Grouting

Grouting has been practised for roughly 100 years and can be effected in many ways. The aim is generally to strengthen the soil by introducing a cementitious grout and/or by causing densification as a result of introducing the grout, or to prevent the flow of water by filling the voids in the soil. Essentially a fluid is injected into the soil and hardens at some later stage. The 'split spacing' method of group injection is generally used, whereby pre-determined quantities of grout are injected through holes spaced at two or three times the final spacing, the hole spacing gradually being reduced. The voids become progressively smaller and the ground progressively tighter. Grouting is covered in Chapters 84 *Ground improvement*, and 90 *Geotechnical grouting and soil mixing*, and so will be covered only briefly here.

25.5.6.1 Permeation grouting

Permeation grouting refers to the process in which no volume change or soil structure change occurs as the grout permeates the soil via the pore spaces. Once the grout is in position it hardens with time to strengthen the soil. Particulate grouts can be used where the soil is relatively coarse grained. Chemical grouts can penetrate smaller voids (medium silts or coarser), but are unsuitable for soils having more than 15% fines. Electrochemical injection can be used in silts and silty clays.

25.5.6.2 Displacement grouting

Displacement grouting refers to the process by which high viscosity grouts are pumped into the soil under high pressures with the aim of causing lateral or radial compaction of the soil. The grout usually consists of mixtures of cement, soil and/or clay, and water. Either spherical bulbs of grout are formed at the end of the grout pipe or columns of grout are formed by withdrawing the pipe. The process was first developed in the 1940s in France, but was of limited use until more recently developed pumping capabilities enabled the method to be more effective. The technique is applicable to soils which are readily compactable.

25.5.6.3 Jet grouting

Jet grouting is a variation that was introduced in Japan in the 1970s, the technique being based on the use of high speed water jets. The native soil is either mixed with a stabiliser (grout) *in situ* or, in the case of poor soils, the soil can be removed and grout columns formed in their place.

25.5.7 Artificial ground freezing

Artificial ground freezing is a construction expedient only. It is a temporary measure used to stabilise an area or zone of saturated soil while a particular construction operation is performed. It is a potentially dangerous construction process (significant precautions are necessary) and an expensive last resort, but it is a useful technique in certain cases.

The water within a soil mass is frozen by installing closely spaced freezing pipes through which a continuous supply of coolant is passed. The frozen soil has a greatly increased bearing capacity, a far higher strength and a permeability approaching zero. Excavation (for tunnels, underpinning, etc.) can proceed in safety once the ground is frozen.

Freezing causes a volume increase that is dependent on the water content of the soil, though a significant heave of the soil can be expected and this can cause problems similar to those of expansive clays if precautions are not taken. Conversely, adverse settlements occur during thaw and these too can create unwanted side effects. Sands and gravels are little affected because they are free draining, whereas movement can be large in silts and clays.

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.