CHAPTER 21
SOIL IMPROVEMENT

21.1 INTRODUCTION

General practice is to use shallow foundations for the foundations of buildings and other such structures, if the soil close to the ground surface possesses sufficient bearing capacity. However, where the top soil is either loose or soft, the load from the superstructure has to be transferred to deeper firm strata. In such cases, pile or pier foundations are the obvious choice.

There is also a third method which may in some cases prove more economical than deep foundations or where the alternate method may become inevitable due to certain site and other environmental conditions. This third method comes under the heading foundation soil improvement. In the case of earth dams, there is no other alternative than compacting the remolded soil in layers to the required density and moisture content. The soil for the dam will be excavated at the adjoining areas and transported to the site. There are many methods by which the soil at the site can be improved. Soil improvement is frequently termed soil stabilization, which in its broadest sense is alteration of any property of a soil to improve its engineering performance. Soil improvement

1. Increases shear strength
2. Reduces permeability, and
3. Reduces compressibility

The methods of soil improvement considered in this chapter are

1. Mechanical compaction
2. Dynamic compaction
3. Vibroflotation
4. Preloading
5. Sand and stone columns
6. Use of admixtures
7. Injection of suitable grouts
8. Use of geotextiles

21.2 MECHANICAL COMPACTION

Mechanical compaction is the least expensive of the methods and is applicable in both cohesionless and cohesive soils. The procedure is to remove first the weak soil up to the depth required, and refill or replace the same in layers with compaction. If the soil excavated is cohesionless or a sand-silt clay mixture, the same can be replaced suitably in layers and compacted. If the soil excavated is a fine sand, silt or soft clay, it is not advisable to refill the same as these materials, even under compaction, may not give sufficient bearing capacity for the foundations. Sometimes it might be necessary to transport good soil to the site from a long distance. The cost of such a project has to be studied carefully before undertaking the same.

The compaction equipment to be used on a project depends upon the size of the project and the availability of the compacting equipment. In projects where excavation and replacement are confined to a narrow site, only tampers or surface vibrators may be used. On the other hand, if the whole area of the project is to be excavated and replaced in layers with compaction, suitable roller types of heavy equipment can be used. Cohesionless soils can be compacted by using vibratory rollers and cohesive soils by sheepsfoot rollers.

The control of field compaction is very important in order to obtain the desired soil properties. Compaction of a soil is measured in terms of the dry unit weight of the soil. The dry unit weight, $\gamma_d$, may be expressed as

$$\gamma_d = \frac{\gamma_t}{1 + w}$$

(21.1)

where,

$\gamma_t$ = total unit weight
$w$ = moisture content

Factors Affecting Compaction

The factors affecting compaction are

1. The moisture content
2. The compactive effort

The compactive effort is defined as the amount of energy imparted to the soil. With a soil of given moisture content, increasing the amount of compaction results in closer packing of soil particles and increased dry unit weight. For a particular compactive effort, there is only one moisture content which gives the maximum dry unit weight. The moisture content that gives the maximum dry unit weight is called the optimum moisture content. If the compactive effort is increased, the maximum dry unit weight also increases, but the optimum moisture content decreases. If all the desired qualities of the material are to be achieved in the field, suitable procedures should be adopted to compact the earthfill. The compactive effort to the soil is imparted by mechanical rollers or any other compacting device. Whether the soil in the field has attained the required maximum dry unit weight can be determined by carrying out appropriate laboratory tests on the soil. The following tests are normally carried out in a laboratory.

1. Standard Proctor test (ASTM Designation D-698), and
2. Modified Proctor test (ASTM Designation D-1557)
21.3 LABORATORY TESTS ON COMPACTION

Standard Proctor Compaction Test

Proctor (1933) developed this test in connection with the construction of earth fill dams in California. The standard size of the apparatus used for the test is given in Fig 21.1. Table 21.1 gives the standard specifications for conducting the test (ASTM designation D-698). Three alternative procedures are provided based on the soil material used for the test.

**Test Procedure**

A soil at a selected water content is placed in layers into a mold of given dimensions (Table 21.1 and Fig. 21.1), with each layer compacted by 25 or 56 blows of a 5.5 lb (2.5 kg) hammer dropped from a height of 12 in (305 mm), subjecting the soil to a total compactive effort of about 12,375 ft-lb/ft$^3$ (600 kN-m/m$^3$). The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of water contents to establish a relationship between the dry unit weight and the water content of the soil. This data, when plotted, represents a curvilinear relationship known as the compaction curve or moisture-density curve. The values of water content and standard maximum dry unit weight are determined from the compaction curve as shown in Fig. 21.2.

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**Table 21.1 Specification for standard Proctor compaction test**

<table>
<thead>
<tr>
<th>Item</th>
<th>Procedure</th>
<th>Procedure</th>
<th>Procedure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Item</td>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>1. Diameter of mold</td>
<td>4 in. (101.6 mm)</td>
<td>4 in. (101.6 mm)</td>
<td>6 in. (152.4 mm)</td>
</tr>
<tr>
<td>2. Height of mold</td>
<td>4.584 in. (116.43 mm)</td>
<td>4.584 in. (116.43 mm)</td>
<td>4.584 in. (116.43 mm)</td>
</tr>
<tr>
<td>3. Volume of mold</td>
<td>0.0333 ft$^3$ (944 cm$^3$)</td>
<td>0.0333 ft$^3$ (944 cm$^3$)</td>
<td>0.075 ft$^3$ (2124 cm$^3$)</td>
</tr>
<tr>
<td>4. Weight of hammer</td>
<td>5.5 lb (2.5 kg)</td>
<td>5.5 lb (2.5 kg)</td>
<td>5.5 lb (2.5 kg)</td>
</tr>
<tr>
<td>5. Height of drop</td>
<td>12.0 in. (304.8 mm)</td>
<td>12.0 in. (304.8 mm)</td>
<td>12.0 in. (304.8 mm)</td>
</tr>
<tr>
<td>6. No. of layers</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>7. Blows per layer</td>
<td>25</td>
<td>25</td>
<td>56</td>
</tr>
<tr>
<td>8. Energy of compaction</td>
<td>12,375 ft-lb/ft$^3$ (600 kN-m/m$^3$)</td>
<td>12,375 ft-lb/ft$^3$ (600 kN-m/m$^3$)</td>
<td>12,375 ft-lb/ft$^3$ (600 kN-m/m$^3$)</td>
</tr>
<tr>
<td>9. Soil material</td>
<td>Passing No. 4 sieve (4.75 mm). May be used if 20% or less retained on No. 4 sieve</td>
<td>Passing No. 4 sieve (4.75 mm). Shall be used if 20% or more retained on No. 4 sieve and 20% or less retained on 3/8 in (9.5 mm) sieve</td>
<td>Passing No. 4 sieve (4.75 mm). Shall be used if 20% or more retained on 3/8 in. (9.5 mm) sieve and less than 30% retained on 3/4 in. (19 mm) sieve</td>
</tr>
</tbody>
</table>
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Modified Proctor Compaction Test (ASTM Designation: D1557)

This test method covers laboratory compaction procedures used to determine the relationship between water content and dry unit weight of soils (compaction curve) compacted in a 4 in. or 6 in. diameter mold with a 10 lb (5 kg) hammer dropped from a height of 18 in. (457 mm) producing a compactive effort of 56,250 ft-lb/ft³ (2,700 kN·m/m³). As in the case of the standard test, the code provides three alternative procedures based on the soil material tested. The details of the procedures are given in Table 21.2.

Figure 21.1  Proctor compaction apparatus: (a) diagrammatic sketch, and (b) photograph of mold, and (c) automatic soil compactor (Courtesy: Soiltest)
Table 21.2 Specification for modified Proctor compaction test

<table>
<thead>
<tr>
<th>Item</th>
<th>Procedure</th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Mold diameter</td>
<td>4 in. (101.6 mm)</td>
<td>4 in. (101.6 mm)</td>
<td>6 in. (101.6 mm)</td>
<td></td>
</tr>
<tr>
<td>2. Volume of mold</td>
<td>0.0333 ft³ (944 cm³)</td>
<td>0.0333 ft³ (944 cm³)</td>
<td>0.075 ft³ (2124 cm³)</td>
<td></td>
</tr>
<tr>
<td>3. Weight of hammer</td>
<td>10 lb (4.54 kg)</td>
<td>10 lb (4.54 kg)</td>
<td>10 lb (4.54 kg)</td>
<td></td>
</tr>
<tr>
<td>4. Height of drop</td>
<td>18 in. (457.2 mm)</td>
<td>18 in. (457.2 mm)</td>
<td>18 in. (457.2 mm)</td>
<td></td>
</tr>
<tr>
<td>5. No. of layers</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>7. Energy of compaction</td>
<td>56,250 ft-lb/ft³ (2700 kN-m/m³)</td>
<td>56,250 ft-lb/ft³ (2700 kN-m/m³)</td>
<td>56,250 ft-lb/ft³ (2700 kN-m/m³)</td>
<td></td>
</tr>
<tr>
<td>8. Soil material</td>
<td>May be used if 20% or less retained on No. 4 sieve.</td>
<td>Shall be used if 20% or more retained on No. 4 sieve and 20% or less retained on the 1/8 in. sieve.</td>
<td>Shall be used if more-than 20% retained on 3/8 in. sieve and less-than 30% retained on the 3/4 in. sieve (19 mm)</td>
<td></td>
</tr>
</tbody>
</table>

Test Procedure

A soil at a selected water content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 or 56 blows of a 10 lb (4.54 kg) hammer dropped from a height of 18 in. (457 mm) subjecting the soil to a total compactive effort of about 56,250 ft-lb/ft³ (2700 kN-m/m³). The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of water contents to establish a relationship between the dry unit weight and the water content for the soil. This data, when plotted, represents a curvilinear relationship known as the compaction curve or moisture-dry unit weight curve. The value of the optimum water content and maximum dry unit weight are determined from the compaction curve as shown in Fig. 21.2.

Determination of Zero Air Voids Line

Referring to Fig. 21.3, we have

Degree of saturation,
\[ S = \frac{V_w}{V_v} \]

Water content,
\[ w = \frac{W_w}{W_s} \]

Dry weight of solids,
\[ W_s = V_s G_s, \quad \gamma_w = G_s \gamma_w \text{ since } V_s = 1 \]

\[ V_w = \frac{W_w}{\gamma_w} = \frac{w G_s \gamma_w}{\gamma_w} = w G_s \]

Therefore
\[ S = \frac{w G_s}{V_v} \]  
(21.2)
In Eq. (21.3), since $G_s$ and $\gamma_w$, remain constant for a particular soil, the dry unit weight is a function of water content for any assumed degree of saturation. If $S = 1$, the soil is fully saturated (zero air voids). A curve giving the relationship between $\gamma_d$ and $w$ may be drawn by making use of Eq. (20.3) for $S = 1$. Curves may be drawn for different degrees of saturation such as 95, 90, 80 etc.
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percents. Fig. 21.2 gives typical curves for different degrees of saturation along with moisture-dry unit weight curves obtained by different compactive efforts.

**Example 21.1**

A proctor compaction test was conducted on a soil sample, and the following observations were made:

<table>
<thead>
<tr>
<th>Water content, percent</th>
<th>7.7</th>
<th>11.5</th>
<th>14.6</th>
<th>17.5</th>
<th>19.7</th>
<th>21.2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass of wet soil, g</td>
<td>1739</td>
<td>1919</td>
<td>2081</td>
<td>2033</td>
<td>1986</td>
<td>1948</td>
</tr>
</tbody>
</table>

If the volume of the mold used was 950 cm$^3$ and the specific gravity of soils grains was 2.65, make necessary calculations and draw, (i) compaction curve and (ii) 80% and 100% saturation lines.

**Solution**

From the known mass of the wet soil sample and volume of the mold, wet density or wet unit weight is obtained by the equations,

$$
\frac{M}{V} = \frac{\text{Mass of wet sample in gm}}{950 \text{ cm}^3}
$$

or

$$
\gamma_f = \frac{\text{g/cm}^3}{(\text{kN/m}^3)} = 9.81 \times \rho_f (\text{g/cm}^3)
$$

Then from the wet density and corresponding moisture content, the dry density or dry unit weight is obtained from,

$$
\rho_d = \frac{\rho_f}{1 + w} \quad \text{or} \quad \gamma_d = \frac{\gamma_f}{1 + w}
$$

Thus for each observation, the wet density and then the dry density are calculated and tabulated as follows:

<table>
<thead>
<tr>
<th>Water content, percent</th>
<th>7.7</th>
<th>11.5</th>
<th>14.6</th>
<th>17.5</th>
<th>19.7</th>
<th>21.2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass of wet sample, g</td>
<td>1739</td>
<td>1919</td>
<td>2081</td>
<td>2033</td>
<td>1986</td>
<td>1948</td>
</tr>
<tr>
<td>Wet density, g/cm$^3$</td>
<td>1.83</td>
<td>2.02</td>
<td>2.19</td>
<td>2.14</td>
<td>2.09</td>
<td>2.05</td>
</tr>
<tr>
<td>Dry density, g/cm$^3$</td>
<td>1.70</td>
<td>1.81</td>
<td>1.91</td>
<td>1.82</td>
<td>1.75</td>
<td>1.69</td>
</tr>
<tr>
<td>Dry unit weight kN/m$^3$</td>
<td>16.7</td>
<td>17.8</td>
<td>18.7</td>
<td>17.9</td>
<td>17.2</td>
<td>16.6</td>
</tr>
</tbody>
</table>

Hence the compaction curve, which is a plot between the dry unit weight and moisture content can be plotted as shown in Fig. Ex. 21.1. The curve gives,

Maximum dry unit weight, $MDD = 18.7 \text{ kN/m}^3$

Optimum moisture content, $OMC = 14.7 \text{ percent}$

For drawing saturation lines, make use of Eq. (21.3), viz.,

$$
\gamma_d = \frac{G_s \gamma_w}{1 + wG_s}
$$

where, $G_s = 2.65$, given, $S =$ degree of saturation 80% and 100%, $w =$ water content, may be assumed as 8%, 12%, 16%, 20% and 24%.
Hence for each value of saturation and water content, find \( y_d \) and tabulate:

<table>
<thead>
<tr>
<th>Water content, percentage</th>
<th>8</th>
<th>12</th>
<th>16</th>
<th>20</th>
<th>24</th>
</tr>
</thead>
<tbody>
<tr>
<td>( y_d ) kN/m(^3) for ( S = 100% )</td>
<td>21.45</td>
<td>19.73</td>
<td>18.26</td>
<td>17.0</td>
<td>15.69</td>
</tr>
<tr>
<td>( y_d ) kN/m(^3) for ( S = 80% )</td>
<td>20.55</td>
<td>18.61</td>
<td>17.00</td>
<td>15.64</td>
<td>14.49</td>
</tr>
</tbody>
</table>

With these calculations, saturation lines for 100% and 80% are plotted, as shown in the Fig. Ex. 21.1.

Also the saturation, corresponding to \( MDD = 18.7 \text{ kN/m}^3 \) and \( OMC = 14.7\% \) can be calculated as,

\[
18.7 = y_d = \frac{G_s y_w}{1 + \frac{0.147 G_s}{S}} = \frac{2.65 \times 9.81}{1 + \frac{0.147 \times 2.65}{S}}
\]

which gives \( S = 99.7\% \)

**Example 21.2**

A small cylinder having volume of 600 cm\(^3\) is pressed into a recently compacted fill of embankment filling the cylinder. The mass of the soil in the cylinder is 1100 g. The dry mass of the soil is 910 g. Determine the void ratio and the saturation of the soil. Take the specific gravity of the soil grains as 2.7.

**Solution**

Wet density of soil

\[
\rho_i = \frac{1100}{600} = 1.83 \text{ g/cm}^3 \text{ or } y_i = 17.99 \text{ kN/m}^3
\]

Water content,

\[
w = \frac{1100 - 910}{910} = \frac{190}{910} = 0.209 = 20.9\%
\]
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Dry unit weight, \( \gamma_d = \frac{\gamma_f}{1 + \omega} = \frac{17.99}{1 + 0.209} = 14.88 \text{ kN/m}^3 \)

From, \( \gamma_d = \frac{G_s \gamma_w}{1 + e} \) we have \( e = \frac{G_s \gamma_w}{\gamma_d} - 1 \)

Substituting and simplifying

\[
e = \frac{0.209 \times 2.7 \times 100}{14.88} - 1 = 0.78
\]

From, \( Se = wG_s \), or \( S = \frac{wG_s}{e} = \frac{0.209 \times 2.7 \times 100}{0.78} = 72.35\% \)

21.4 EFFECT OF COMPACTATION ON ENGINEERING BEHAVIOR

Effect of Moisture Content on Dry Density

The moisture content affects the behavior of the soil. When the moisture content is low, the soil is stiff and difficult to compress. Thus, low unit weight and high air contents are obtained (Fig. 21.2). As the moisture content increases, the water acts as a lubricant, causing the soil to soften and become more workable. This results in a denser mass, higher unit weights and lower air contents under compaction. The water and air combination tend to keep the particles apart with further compaction, and prevent any appreciable decrease in the air content of the total voids, however, continue to increase with moisture content and hence the dry unit weight of the soil falls.

To the right of the peak of the dry unit weight-moisture content curve (Fig. 21.2), lies the saturation line. The theoretical curve relating dry density with moisture content with no air voids is approached but never reached since it is not possible to expel by compaction all the air entrapped in the voids of the soil.

Effect of Compactive Effort on Dry Unit Weight

For all types of soil with all methods of compaction, increasing the amounts of compaction, that is, the energy applied per unit weight of soil, results in an increase in the maximum dry unit weight and a corresponding decrease in the optimum moisture content as can be seen in Fig. 21.4.

Shear Strength of Compacted Soil

The shear strength of a soil increases with the amount of compaction applied. The more the soil is compacted, the greater is the value of cohesion and the angle of shearing resistance. Comparing the shearing strength with the moisture content for a given degree of compaction, it is found that the greatest shear strength is attained at a moisture content lower than the optimum moisture content for maximum dry unit weight. Fig. 21.5 shows the relationship between shear strength and moisture-dry unit weight curves for a sandy clay soil. It might be inferred from this that it would be an advantage to carry out compaction at the lower value of the moisture content. Experiments, however, have indicated that soils compacted in this way tend to take up moisture and become saturated with a consequent loss of strength.

Effect of Compaction on Structure

Fig. 21.6 illustrates the effects of compaction on clay structure (Lambe, 1958a). Structure (or fabric) is the term used to describe the arrangement of soil particles and the electric forces between adjacent particles.
Figure 21.4 Dynamic compaction curves for a silty clay (from Turnbull, 1950)

Note: 6 in. diameter mold used for all tests

Figure 21.5 Relationship between compaction and shear strength curves

Figure 21.6 Effects of compaction on structure (from Lambe, 1958a)
The effects of compaction conditions on soil structure, and thus on the engineering behavior of the soil, vary considerably with soil type and the actual conditions under which the behavior is determined.

At low water content, $w_A$, in Fig. 21.6, the repulsive forces between particles are smaller than the attractive forces, and as such the particles flocculate in a disorderly array. As the water content increases beyond $w_A$, the repulsion between particles increases, permitting the particles to disperse, making particles arrange themselves in an orderly way. Beyond $w_B$, the degree of particle parallelism increases, but the density decreases. Increasing the compactive effort at any given water content increases the orientation of particles and therefore gives a higher density as indicated in Fig. 21.6.

**Effect of Compaction on Permeability**

Fig. 21.7 depicts the effect of compaction on the permeability of a soil. The figure shows the typical marked decrease in permeability that accompanies an increase in molding water content on the dry side of the optimum water content. A minimum permeability occurs at water contents slightly above optimum moisture content (Lambe, 1958a), after which a slight increase in permeability occurs. Increasing the compactive effort decreases the permeability of the soil.

![Figure 21.7 Compaction-permeability tests on Siburua clay (from Lambe, 1962)](image)
Effect of Compaction on Compressibility

Figure 21.8 illustrates the difference in compaction characteristics between two saturated clay samples at the same density, one compacted on the dry side of optimum and one compacted on the wet side (Lambe, 1958b). At low stresses the sample compacted on the wet side is more compressible than the one compacted on the dry side. However, at high applied stresses the sample compacted on the dry side is more compressible than the sample compacted on the wet side.

21.5 FIELD COMPACTION AND CONTROL

The necessary compaction of subgrades of roads, earth fills, and embankments may be obtained by mechanical means. The equipment that are normally used for compaction consists of

1. Smooth wheel rollers
2. Rubber tired rollers
3. Sheepsfoot rollers
4. Vibratory rollers

Laboratory tests on the soil to be used for construction in the field indicate the maximum dry density that can be reached and the corresponding optimum moisture content under specified methods of compaction. The field compaction method should be so adjusted as to translate
laboratory condition into practice as far as possible. The two important factors that are necessary to achieve the objectives in the field are

1. The adjustment of the natural moisture content in the soil to the value at which the field compaction is most effective.
2. The provision of compacting equipment suitable for the work at the site.

The equipment used for compaction are briefly described below:

**Smooth Wheel Roller**

There are two types of smooth wheel rollers. One type has two large wheels, one in the rear and a similar single drum in the front. This type is generally used for compacting base courses. The equipment weighs from 50 to 125 kN (Fig. 21.9). The other type is the tandem roller normally used for compacting paving mixtures. This roller has large single drums in the front and rear and the weights of the rollers range from 10 to 200 kN.
Rubber Tired Roller
The maximum weight of this roller may reach 2000 kN. The smaller rollers usually have 9 to 11 tires on two axles with the tires spaced so that a complete coverage is obtained with each pass. The tire loads of the smaller roller are in the range of 7.5 kN and the tire pressures in the order of 200 kN/m². The larger rollers have tire loads ranging from 100 to 500 kN per tire, and tire pressures range from 400 to 1000 kN/m².

Sheepsfoot Roller
Sheepsfoot rollers are available in drum widths ranging from 120 to 180 cm and in drum diameters ranging from 90 to 180 cm. Projections like a sheepsfoot are fixed on the drums. The lengths of these projections range from 17.5 cm to 23 cm. The contact area of the tamping foot ranges from 35 to 56 sq. cm. The loaded weight per drum ranges from about 30 kN for the smaller sizes to 130 kN for the larger sizes (Fig. 21.10).

Vibratory Roller
The weights of vibratory rollers range from 120 to 300 kN. In some units vibration is produced by weights placed eccentrically on a rotating shaft in such a manner that the forces produced by the rotating weights are essentially in a vertical direction. Vibratory rollers are effective for compacting granular soils (Fig. 21.11).
Selection of Equipment for Compaction in the Field

The choice of a roller for a given job depends on the type of soil to be compacted and percentage of compaction to be obtained. The types of rollers that are recommended for the soils normally met are:

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Type of roller recommended</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesive soil</td>
<td>Sheepsfoot roller, or Rubber tired roller</td>
</tr>
<tr>
<td>Cohesionless soils</td>
<td>Rubber-tired roller or Vibratory roller.</td>
</tr>
</tbody>
</table>

Method of Compaction

The first approach to the problem of compaction is to select suitable equipment. If the compaction is required for an earth dam, the number of passes of the roller required to compact the given soil to the required density at the optimum moisture content has to be determined by conducting a field trial test as follows:

The soil is well mixed with water which would give the optimum water content as determined in the laboratory. It is then spread out in a layer. The thickness of the layer normally varies from 15 to 22.5 cm. The number of passes required to obtain the specified density has to be found by determining the density of the compacted material after every definite number of passes. The density may be checked for different thickness in the layer. The suitable thickness of the layer and the number of passes required to obtain the required density will have to be determined.
In cohesive soils, densities of the order of 95 percent of standard Proctor can be obtained with practically any of the rollers and tampers; however, vibrators are not effective in cohesive soils. Where high densities are required in cohesive soils in the order of 95 percent of modified Proctor, rubber tired rollers with tire loads in the order of 100 kN and tire pressure in the order of 600 kN/m² are effective.

In cohesionless sands and gravels, vibrating type equipment is effective in producing densities up to 100 percent of modified Proctor. Where densities are needed in excess of 100 percent of modified Proctor such as for base courses for heavy duty air fields and highways, rubber tired rollers with tire loads of 130 kN and above and tire pressure of 1000 kN/m² can be used to produce densities up to 103 to 104 percent of modified Proctor.

Field Control of Compaction

Methods of Control of Density

The compaction of soil in the field must be such as to obtain the desired unit weight at the optimum moisture content. The field engineer has therefore to make periodic checks to see whether the compaction is giving desired results. The procedure of checking involves:

1. Measurement of the dry unit weight, and

There are many methods for determining the dry unit weight and/or moisture content of the soil in-situ. The important methods are:

1. Sand cone method,
2. Rubber balloon method,
3. Nuclear method, and
4. Proctor needle method.

Sand Cone Method (ASTM Designation D-1556)

The sand for the sand cone method consists of a sand pouring jar shown in Fig. 21.12. The jar contains uniformly graded clean and dry sand. A hole about 10 cm in diameter is made in the soil to be tested up to the depth required. The weight of soil removed from the hole is determined and its water content is also determined. Sand is run into the hole from the jar by opening the valve above the cone until the hole and the cone below the valve is completely filled. The valve is closed. The jar is calibrated to give the weight of the sand that just fills the hole, that is, the difference in weight of the jar before and after filling the hole after allowing for the weight of sand contained in the cone is the weight of sand poured into the hole.

Let

$$W_s = \text{weight of dry sand poured into the hole}$$

$$G_s = \text{specific gravity of sand particles}$$

$$W = \text{weight of soil taken out of the hole}$$

$$w = \text{water content of the soil}$$

Volume of sand in the hole = volume of soil taken out of the hole

that is, $$V = \frac{W_s}{G_s \gamma_w} \quad (21.4a)$$
The dry unit weight of soil, \( \gamma_d = \frac{\gamma_i}{1 + w} \)

**Rubber Balloon Method (ASTM Designation: D 2167)**

The volume of an excavated hole in a given soil is determined using a liquid-filled calibrated cylinder for filling a thin rubber membrane. This membrane is displaced to fill the hole. The in-place unit weight is determined by dividing the wet mass of the soil removed by the volume of the hole. The water (moisture) content and the in-place unit weight are used to calculate the in-place dry unit weight. The volume is read directly on the graduated cylinder. Fig. 21.13 shows the equipment.
Figure 21.13 Rubber balloon density apparatus, (a) diagrammatic sketch, and (b) a photograph

Nuclear Method

The modern instrument for rapid and precise field measurement of moisture content and unit weight is the Nuclear density/Moisture meter. The measurements made by the meter are non-destructive and require no physical or chemical processing of the material being tested. The instrument may be used either in drilled holes or on the surface of the ground. The main advantage of this equipment is that a single operator can obtain an immediate and accurate determination of the in-situ dry density and moisture content.

Proctor Needle Method

The Proctor needle method is one of the methods developed for rapid determination of moisture contents of soils in-situ. It consists of a needle attached to a spring loaded plunger, the stem of which is calibrated to read the penetration resistance of the needle in lbs/in² or kg/cm². The needle is supplied with a series of bearing points so that a wide range of penetration resistances can be measured. The bearing areas that are normally provided are 0.05, 0.1, 0.25, 0.50 and 1.0 sq. in. The apparatus is shown in Fig. 21.14. A Proctor penetrometer set is shown in Fig. 21.15 (ASTMD–1558).
Laboratory Penetration Resistance Curve

A suitable needle point is selected for a soil to be compacted. If the soil is cohesive, a needle with a larger bearing area is selected. For cohesionless soils, a needle with a smaller bearing area will be sufficient. The soil sample is compacted in the mold.

![Proctor needle](image1)

![Proctor penetrometer set](image2)

**Figure 21.14** Proctor needle

**Figure 21.15** Proctor penetrometer set (Courtesy: Soiltest)

**Figure 21.16** Field method of determining water content by Proctor needle method
The penetrometer with a known bearing area of the tip is forced with a gradual uniform push at a rate of about 1.25 cm per sec to a depth of 7.5 cm into the soil. The penetration resistance in kg/cm² is read off the calibrated shaft of the penetrometer. The water content of the soil and the corresponding dry density are also determined. The procedure is repeated for the same soil compacted at different moisture contents. Curves giving the moisture-density and penetration resistance-moisture content relationship are plotted as shown in Fig. 21.16.

To determine the moisture content in the field, a sample of the wet soil is compacted into the mold under the same conditions as used in the laboratory for obtaining the penetration resistance curve. The Proctor needle is forced into the soil and its resistance is determined. The moisture content is read from the laboratory calibration curve.

This method is quite rapid, and is sufficiently accurate for fine-grained cohesive soils. However, the presence of gravel or small stones in the soil makes the reading on the Proctor needle less reliable. It is not very accurate in cohesionless sands.

**Example 21.3**

The following observations were recorded when a sand cone test was conducted for finding the unit weight of a natural soil:

Total density of sand used in the test = 1.4 g/cm³

Mass of the soil excavated from hole = 950 g.

Mass of the sand filling the hole = 700 g.

Water content of the natural soil = 15 percent.

Specific gravity of the soil grains = 2.7

Calculate: (i) the wet unit weight, (ii) the dry unit weight, (iii) the void ratio, and (iv) the degree of saturation.

**Solution**

Volume of the hole

\[ V_p = \frac{700}{1.4} = 500 \text{ cm}^3 \]

Wet density of natural soil,

\[ \rho_w = \frac{950}{500} = 19 \text{ g/cm}^3 \text{ or } \gamma_w = 18.64 \text{ kN/m}^3 \]

Dry density

\[ \rho_d = \frac{\rho_w}{1 + w} = \frac{19}{1 + 0.15} = 1.65 \text{ g/cm}^3 \]

\[ \rho_d = \frac{G_s \rho_w}{1 + e} \text{ or } 165 + 1.65e = 2.7 \]

Therefore

\[ e = \frac{2.7 - 1.65}{1.65} = 0.64 \]

And

\[ S = \frac{wG_s}{e} = \frac{0.15 \times 2.7 \times 100}{0.64} = 63\% \]
Example 21.4
Old records of a soil compacted in the past gave compaction water content of 15% and saturation 85%. What might be the dry density of the soil?

Solution
The specific gravity of the soil grains is not known, but as it varies in a small range of 2.6 to 2.7, it can suitably be assumed. An average value of 2.65 is considered here.

\[
\frac{w}{S} = \frac{0.15 \times 2.65}{0.85} = 0.47
\]

Hence, dry density \( \rho_d = \frac{G_s}{1 + e} \rho_w = \frac{2.65}{1 + 0.47} \times 1 = 1.8 \text{ g/cm}^3 \) or dry unit weight = 17.66 \text{ kN/m}^3

Example 21.5
The following data are available in connection with the construction of an embankment:

(a) soil from borrow pit: Natural density = 1.75 \text{ Mg/m}^3, Natural water content = 12%
(b) soil after compaction: density = 2 \text{ Mg/m}^3, water content = 18%.

For every 100 \text{ m}^3 of compacted soil of the embankment, estimate:

(i) the quantity of soil to be excavated from the borrow pit, and
(ii) the amount of water to be added

Note: 1 \text{ g/cm}^3 = 1000 \text{ kg/m}^3 = 10^3 \times 10^3 \text{ g/m}^3 = 1 \text{ Mg/m}^3 where Mg stands for Megagram = 10^6 g.

Solution
The soil is compacted in the embankment with density of 2 \text{ Mg/m}^3 and with 18% water content.

Hence, for 100 \text{ m}^3 of soil

Mass of compacted wet soil = 100 \times 2.0 = 200 \text{ Mg} = 200 \times 10^3 \text{ kg}.

\[
\text{Mass of compacted dry soil} = \frac{200}{1 + w} = \frac{200}{1 + 0.18} = 169.5 \text{ Mg} = 169.5 \times 10^3 \text{ kg}
\]

Mass of wet soil to be excavated = 169.5(1 + w) = 169.5(1 + 0.12) = 189.84 \text{ Mg}

Volume of the wet soil to be excavated = \frac{189.84}{1.75} = 108.48 \text{ m}^3

Now, in the natural state, the moisture present in 169.5 \times 10^3 \text{ kg of dry soil} would be

169.5 \times 10^3 \times 0.12 = 20.34 \times 10^3 \text{ kg}

and the moisture which the soil will possess during compaction is

169.5 \times 10^3 \times 0.18 = 30.51 \times 10^3 \text{ kg}

Hence mass of water to be added for every 100 \text{ m}^3 of compacted soil is

\[(30.51 - 20.34) \times 10^3 = 10.17 \times 10^3 \text{ kg}.\]
Example 21.6

A sample of soil compacted according to the standard Proctor test has a density of 2.06 g/cm³ at 100% compaction and at an optimum water content of 14%. What is the dry unit weight? What is the dry unit weight at zero air-voids? If the voids become filled with water what would be the saturated unit weight? Assume $G_s = 2.67$.

Solution

Refer to Fig. Ex. 21.6. Assume $V$ = total volume = 1 cm³. Since water content is 14% we may write,

$$\frac{M_w}{M_s} = 0.14 \quad \text{or} \quad M_w = 0.14 M_s$$

and since, $M_w + M_s = 2.06$ g,

$$0.14 M_s + M_s = 1.14 M_s = 2.06$$

or

$$M_s = \frac{2.06}{1.14} = 1.807 \text{ g}$$

$$M_w = 0.14 \times 1.807 = 0.253 \text{ g}.$$

By definition, $\rho_d = \frac{M_s}{V} = \frac{1.807}{1} = 1.807 \text{ g/cm}^3$ or $\gamma_d = 1.807 \times 9.81 = 17.73 \text{ kN/m}^3$

The volume of solids (Fig. Ex. 21.6) is

$$V_s = \frac{1.807}{2.67} = 0.68 \text{ g/cm}^3$$

The volume of voids = 1 - 0.68 = 0.32 cm³

The volume of water = 0.253 cm³

The volume of air = 0.320 - 0.253 = 0.067 cm³

If all the air is squeezed out of the samples the dry density at zero air voids would be, by definition,
\[
\rho_d = \frac{1.807}{0.68 + 0.253} = 1.94 \text{ g/cm}^3 \text{ or } \gamma_d = 1.94 \times 9.81 = 19.03 \text{ kN/m}^3
\]

on the other hand, if the air voids also were filled with water,

The mass of water would be \(0.32 \times 1 = 0.32 \text{ g}\)

The saturated density is

\[
\rho_{sat} = \frac{1.807 + 0.32}{1} = 2.13 \text{ g/cm}^3 \text{ or } \gamma_{sat} = 2.13 \times 9.81 = 20.90 \text{ kN/m}^3
\]

### 21.6 COMPACTION FOR DEEPER LAYERS OF SOIL

Three types of dynamic compaction for deeper layers of soil are discussed here. They are:

1. Vibroflotation.
2. Dropping of a heavy weight.

**Vibroflotation**

The vibroflotation technique is used for compacting granular soil only. The vibroflot is a cylindrical tube containing water jets at top and bottom and equipped with a rotating eccentric weight, which develops a horizontal vibratory motion as shown in Fig. 21.17. The vibroflot is sunk into the soil using the lower jets and is then raised in successive small increments, during which the surrounding material is compacted by the vibration process. The enlarged hole around the vibroflot is backfilled with suitable granular material. This method is very effective for increasing the density of a sand deposit for depths up to 30 m. Probe spacings of compaction holes should be on a grid pattern of about 2 m to produce relative densities greater than 70 percent over the entire area. If the sand is coarse, the spacings may be somewhat larger.

In soft cohesive soil and organic soils the vibroflotation technique has been used with gravel as the backfill material. The resulting densified stone column effectively reinforces softer soils and acts as a bearing pile for foundations.

![Figure 21.17 Compaction by using vibroflot (Brown, 1977)](image-url)
Dropping of a Heavy Weight

The repeated dropping of a heavy weight on to the ground surface is one of the simplest of the methods of compacting loose soil.

The method, known as deep dynamic compaction or deep dynamic consolidation may be used to compact cohesionless or cohesive soils. The method uses a crane to lift a concrete or steel block, weighing up to 500 kN and up to heights of 40 to 50 m, from which height it is allowed to fall freely on to the ground surface. The weight leaves a deep pit at the surface. The process is then repeated either at the same location or sequentially over other parts of the area to be compacted. When the required number of repetitions is completed over the entire area, the compaction at depth is completed. The soils near the surface, however, are in a greatly disturbed condition. The top soil may then be levelled and compacted, using normal compacting equipment. The principal claims of this method are:

1. Depth of recompaction can reach up to 10 to 12 m.
2. All soils can be compacted.
3. The method produces equal settlements more quickly than do static (surcharge type) loads.

The depth of recompaction, \( D \), in meters is approximately given by Leonards, et al., (1980) as

\[
D \approx \frac{1}{2} (Wh)^{\frac{1}{4}}
\]

where \( W \) = weight of falling mass in metric tons,

\( h \) = height of drop in meters.

Blasting

Blasting, through the use of buried, time-delayed explosive charges, has been used to densify loose, granular soils. The sands and gravels must be essentially cohesionless with a maximum of 15 percent of their particles passing the No. 200 sieve size and 3 percent passing 0.005 mm size. The moisture condition of the soil is also important for surface tension forces in the partially saturated state limit the effectiveness of the technique. Thus the soil, as well as being granular, must be dry or saturated, which requires sometimes prewetting the site via construction of a dike and reservoir system.

The technique requires careful planning and is used at a remote site. Theoretically, an individual charge densifies the surrounding adjacent soil and soil beneath the blast. It should not lift the soil situated above the blast, however, since the upper soil should provide a surcharge load. The charge should not create a crater in the soil. Charge delays should be timed to explode from the bottom of the layer being densified upward in a uniform manner. The uppermost part of the stratum is always loosened, but this can be surface-compactd by vibratory rollers. Experience indicates that repeated blasts of small charges are more effective than a single large charge for achieving the desired results.

21.7 PRELOADING

Preloading is a technique that can successfully be used to densify soft to very soft cohesive soils. Large-scale construction sites composed of weak silts and clays or organic materials (particularly marine deposits), sanitary land fills, and other compressible soils may often be stabilized effectively and economically by preloading. Preloading compresses the soil. Compression takes place when the water in the pores of the soil is removed which amounts to artificial consolidation of soil in the field. In order to remove the water squeezed out of the pores and hasten the period of consolidation, horizontal and vertical drains are required to be provided in the mass. The preload is
generally in the form of an imposed earth fill which must be left in place long enough to induce consolidation. The process of consolidation can be checked by providing suitable settlement plates and piezometers. The greater the surcharge load, shorter the time for consolidation. This is a case of three-dimension consolidation.

Two types of vertical drains considered are

1. Cylindrical sand drains
2. Wick (prefabricated vertical) drains

**Sand Drains**

Vertical and horizontal and drains are normally used for consolidating very soft clay, silt and other compressible materials. The arrangement of sand drains shown in Fig. 21.18 is explained below:

1. It consists of a series of vertical sand drains or piles. Normally medium to coarse sand is used.
2. The diameter of the drains are generally not less than 30 cm and the drains are placed in a square grid pattern at distances of 2 to 3 meters apart. Economy requires a careful study of the effect of spacing the sand drains on the rate of consolidation.
3. Depth of the vertical drains should extend up to the thickness of the compressible stratum.
4. A horizontal blanket of free draining sand should be placed on the top of the stratum and the thickness of this may be up to a meter, and

5. Soil surcharge in the form of an embankment is constructed on top of the sand blanket in stages.

The height of surcharge should be so controlled as to keep the development of pore water pressure in the compressible strata at a low level. Rapid loading may induce high pore water pressures resulting in the failure of the stratum by rupture. The lateral displacement of the soil may shear off the sand drains and block the drainage path.

The application of surcharge squeezes out water in radial directions to the nearest sand drain and also in the vertical direction to the sand blanket. The dashed lines shown in Fig. 21.18(b) are drawn midway between the drains. The planes passing through these lines may be considered as impermeable membranes and all the water within a block has to flow to the drain at the center. The problem of computing the rate of radial drainage can be simplified without appreciable error by assuming that each block can be replaced by a cylinder of radius \( R \) such that

\[
\pi R^2 = L^2
\]

where \( L \) is the side length of the prismatic block.

The relation between the time \( t \) and degree of consolidation \( U_z \% \) is determined by the equation

\[
U_z \% = 100 f(T)
\]

wherein,

\[
T = \frac{C_f}{H^2}
\]

If the bottom of the compressible layer is impermeable, then \( H \) is the full thickness of the layer.

For radial drainage, Rendulic (1935) has shown that the relation between the time \( t \) and the degree of consolidation \( U_r \% \) can be expressed as

\[
U_r \% = 100 f(T)
\]

wherein,

\[
T_r = \frac{C_r r}{4R^2} t
\]

is the time factor. The relation between the degree of consolidation \( U_r \% \) and the time factor \( T_r \) depends on the value of the ratio \( R/r \). The relation between \( T_r \) and \( U_r \% \) for ratios of \( R/r \) equal to 1, 10 and 100 in Fig. 21.19 are expressed by curves \( C_1, C_{10} \) and \( C_{100} \) respectively.

**Installation of Vertical Sand Drains**

The sand drains are installed as follows

1. A casing pipe of the required diameter with the bottom closed with a loose-fit-cone is driven up to the required depth,
2. The cone is slightly separated from the casing by driving a mandrel into the casing, and
3. The sand of the required gradation is poured into the pipe for a short depth and at the same time the pipe is pulled up in steps. As the pipe is pulled up, the sand is forced out of the pipe by applying pressure on to the surface of the sand. The procedure is repeated till the holes is completely filled with sand.
Soil Improvement

The sand drains may also be installed by jetting a hole in the soil or by driving an open casing into the soil, washing the soil out of the casing, and filling the hole with sand afterwards.

Sand drains have been used extensively in many parts of the world for stabilizing soils for port development works and for foundations of structures in reclaimed areas on the sea coasts. It is possible that sand drains may not function satisfactorily if the soil surrounding the well gets remolded. This condition is referred to as smear. Though theories have been developed by considering different thickness of smear and different permeability, it is doubtful whether such theories are of any practical use since it would be very difficult to evaluate the quality of the smear in the field.

Wick (Prefabricated Vertical) Drains

Geocomposites used as drainage media have completely taken over certain geotechnical application areas. Wick drains, usually consisting of plastic fluted or nubbed cores that are surrounded by a geotextile filter, have considerable tensile strength. Wick drains do not require any sand to transmit flow. Most synthetic drains are of a strip shape. The strip drains are generally 100 mm wide and 2 to 6 mm thick. Fig. 21.20 shows typical core shapes of strip drains (Hausmann, 1990).

Wick drains are installed by using a hollow lance. The wick drain is threaded into a hollow lance, which is pushed (or driven) through the soil layer, which collapses around it. At the ground surface the ends of the wick drains (typically at 1 to 2 m spacing) are interconnected by a granular soil drainage layer or geocomposite sheet drain layer. There are a number of commercially available wick drain manufacturers and installation contractors who provide information on the current products, styles, properties, and estimated costs (Koerner, 1999).

With regards to determining wick drain spacings, the initial focal point is on the time for the consolidation of the subsoil to occur. Generally the time for 90% consolidation ($t_{90}$) is desired. In order to estimate the time $t$, it is first necessary to estimate an equivalent sand drain diameter for the wick drain used. The equations suggested by Koerner (1999) are

$$d_{sd} = \sqrt{\frac{d_{w}^2}{n_x}}$$

(21.9a)
Various shapes of cores with nonwoven geotextile filter sleeves

Filter sleeve

Fluted PVC or paper

Core

Figure 21.20 Typical core shapes of strip drains (Hausmann, 1990)

\[ d_e = \sqrt{\frac{4 b t n_s}{\pi}} \]

(21.9b)

where

- \( d_{sd} \) = equivalent sand drain diameter
- \( d_v \) = equivalent void circle diameter
- \( b, t \) = width and thickness of the wick drain
- \( n_s \) = porosity of sand drain

It may be noted here that equivalent sand drain diameters for various commercially available wick drains vary from 30 to 50 mm (Koerner, 1999).

The equation for estimating the time \( t \) for consolidation is (Koerner, 1999)

\[ t = \frac{D^2}{8c_h} \ln \frac{D}{d} - 0.75 \ln \left( \frac{1}{1-U} \right) \]

(21.10)

where

- \( t \) = time for consolidation
- \( c_h \) = coefficient of consolidation of soil for horizontal flow
\[ d = \text{equivalent diameter of strip drain} \]
\[ = \frac{\text{circumference}}{\pi} \]

\[ D = \text{sphere of influence of the strip drain;} \]
a) for a triangular pattern, \( D = 1.05 \times \text{spacing} D \)
b) for a square pattern, \( D = 1.13 \times \text{spacing} \)

\[ D_i = \text{distance between drains in triangular spacing and} \]
\[ D_s = \text{distance for square pattern} \]
\[ U = \text{average degree of consolidation} \]

**Advantages of Using Wick Drains (Koerner, 1999)**

1. The analytic procedure is available and straightforward in its use.
2. Tensile strength is definitely afforded to the soft soil by the installation of the wick drains.
3. There is only nominal resistance to the flow of water if it enters the wick drain.
4. Construction equipment is generally small.
5. Installation is simple, straightforward and economic.

**Example 21.7**

What is the equivalent sand drain diameter of a wick drain measuring 96 mm wide and 2.9 mm thick that is 92% void in its cross section? Use an estimated sand porosity of 0.3 for typical sand in a sand drain.

**Solution**

Total area of wick drain = \( b \times t = 96 \times 2.9 = 279 \text{ mm}^2 \)

Void area of wick drain = \( n_d \times b \times t = 0.92 \times 279 = 257 \text{ mm}^2 \)

The equivalent circle diameter (Eq. 21.9b) is

\[ d_s = \sqrt{\frac{4 \times b \times t}{n_d}} = \sqrt{\frac{4 \times 257}{3.14}} = 18.1 \text{ mm} \]

The equivalent sand drain diameter (Eq. 21.19a) is

\[ d_{sd} = \sqrt{\frac{d_s^2}{n_s}} = \sqrt{\frac{18.1^2}{0.3}} = 33 \text{ mm} \]

**Example 21.8**

Calculate the times required for 50, 70 and 90% consolidation of a saturated clayey silt soil using wick drains at various triangular spacings. The wick drains measure 100 x 4 mm and the soil has a \( c_h = 6.5 \times 10^{-6} \text{ m}^2/\text{min} \).

**Solution**

In the simplified formula the equivalent diameter \( d \) of a strip drain is

\[ d = \frac{\text{circumference}}{\pi} = \frac{100 + 100 + 4 + 4}{3.14} = 66.2 \text{ mm} \]
Using Eq. (21.10)

\[ t = \frac{D^2}{8c_h} \ln \left( \frac{D}{d} \right) - 0.75 \ln \left( \frac{1}{1-U} \right) \]

substituting the known values

\[ t = \frac{D^2}{8(6.5 \times 10^{-6})} \ln \left( \frac{D}{0.0062} \right) - 0.75 \ln \left( \frac{1}{1-U} \right) \]

The times required for the various degrees of consolidation are tabulated below for assumed theoretical spacings of wick drains.

<table>
<thead>
<tr>
<th>Wick drain spacings (D/m)</th>
<th>Time in days for various degrees of consolidation (U)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50%</td>
</tr>
<tr>
<td>2.1</td>
<td>110</td>
</tr>
<tr>
<td>1.8</td>
<td>77</td>
</tr>
<tr>
<td>1.5</td>
<td>49</td>
</tr>
<tr>
<td>1.2</td>
<td>29</td>
</tr>
<tr>
<td>0.9</td>
<td>14</td>
</tr>
<tr>
<td>0.6</td>
<td>4.8</td>
</tr>
<tr>
<td>0.3</td>
<td>0.6</td>
</tr>
</tbody>
</table>

For the triangular pattern, the spacing \( D_t \) is

\[ D_t = \frac{D}{1.05} \]

### 21.8 SAND COMPACTION PILES AND STONE COLUMNS

**Sand Compaction Piles**

Sand compaction piles consists of driving a hollow steel pipe with the bottom closed with a collapsible plate down to the required depth; filling it with sand, and withdrawing the pipe while air pressure is directed against the sand inside it. The bottom plate opens during withdrawal and the sand backfills the voids created earlier during the driving of the pipe. The in-situ soil is densified while the pipe is being withdrawn, and the sand backfill prevents the soil surrounding the compaction pipe from collapsing as the pipe is withdrawn. The maximum limits on the amount of fines that can be present are 15 percent passing the No. 200 sieve (0.075 mm) and 3 percent passing 0.005 mm. The distance between the piles may have to be planned according to the site conditions.

**Stone Columns**

The method described for installing sand compaction piles or the vibroflot described earlier can be used to construct stone columns. The size of the stones used for this purpose range from about 6 to 40 mm. Stone columns have particular application in soft inorganic, cohesive soils and are generally inserted on a volume displacement basis.
The diameter of the pipe used either for the construction of sand drains or sand compaction piles can be increased according to the requirements. Stones are placed in the pipe instead of sand, and the technique of constructing stone columns remains the same as that for sand piles.

Stone columns are placed 1 to 3 m apart over the whole area. There is no theoretical procedure for predicting the combined improvement obtained, so it is usual to assume the foundation loads are carried only by the several stone columns with no contribution from the intermediate ground (Bowles, 1996).

Bowles (1996) gives an approximate formula for the allowable bearing capacity of stone columns as

\[ q_a = \frac{K_p}{F_s} (4c + \sigma'_r) \]  \hspace{1cm} (21.11)

where \( K_p = \tan^2(45° + \phi/2) \),
\( \phi' = \) drained angle of friction of stone,
\( c = \) either drained cohesion (suggested for large areas) or the undrained shear strength \( c_u' \),
\( \sigma'_r = \) effective radial stress as measured by a pressuremeter (but may use \( 2c \) if pressuremeter data are not available),
\( F_s = \) factor of safety, 1.5 to 2.0.

The total allowable load on a stone column of average cross-section area \( A_c \) is

\[ Q_a = q_a A_c \] \hspace{1cm} (21.10)

Stone columns should extend through soft clay to firm strata to control settlements. There is no end bearing in Eq. (21.11) because the principal load carrying mechanism is local perimeter shear.

Settlement is usually the principal concern with stone columns since bearing capacity is usually quite adequate (Bowles, 1996). There is no method currently available to compute settlement on a theoretical basis.

Stone columns are not applicable to thick deposits of peat or highly organic silts or clays (Bowles, 1996). Stone columns can be used in loose sand deposits to increase the density.

### 21.9 SOIL STABILIZATION BY THE USE OF ADMIXTURES

The physical properties of soils can often economically be improved by the use of admixtures. Some of the more widely used admixtures include lime, portland cement and asphalt. The process of soil stabilization first involves mixing with the soil a suitable additive which changes its property and then compacting the admixture suitably. This method is applicable only for soils in shallow foundations or the base courses of roads, airfield pavements, etc.

**Soil-lime Stabilization**

Lime stabilization improves the strength, stiffness and durability of fine grained materials. In addition, lime is sometimes used to improve the properties of the fine grained fraction of granular soils. Lime has been used as a stabilizer for soils in the base courses of pavement systems, under concrete foundations, on embankment slopes and canal linings.

Adding lime to soils produces a maximum density under a higher optimum moisture content than in the untreated soil. Moreover, lime produces a decrease in plasticity index.

Lime stabilization has been extensively used to decrease swelling potential and swelling pressures in clays. Ordinarily the strength of wet clay is improved when a proper amount of lime
is added. The improvement in strength is partly due to the decrease in plastic properties of the clay and partly to the pozzolanic reaction of lime with soil, which produces a cemented material that increases in strength with time. Lime-treated soils, in general, have greater strength and a higher modulus of elasticity than untreated soils.

Recommended percentages of lime for soil stabilization vary from 2 to 10 percent. For coarse soils such as clayey gravels, sandy soils with less than 50 per cent silt-clay fraction, the percent of lime varies from 2 to 5, whereas for soils with more than 50 percent silt-clay fraction, the percent of lime lies between 5 and 10. Lime is also used with fly ash. The fly ash may vary from 10 to 20 per cent, and the percent of lime may lie between 3 and 7.

**Soil-Cement Stabilization**

Soil-cement is the reaction product of an intimate mixture of pulverized soil and measured amounts of portland cement and water, compacted to high density. As the cement hydrates, the mixture becomes a hard, durable structural material. Hardened soil-cement has the capacity to bridge over local weak points in a subgrade. When properly made, it does not soften when exposed to wetting and drying, or freezing and thawing cycles.

Portland cement and soil mixed at the proper moisture content has been used increasingly in recent years to stabilize soils in special situations. Probably the main use has been to build stabilized bases under concrete pavements for highways and airfields. Soil cement mixtures are also used to provide wave protection on earth dams. There are three categories of soil-cement (Mitchell and Freitag, 1959). They are:

1. Normal soil-cement usually contains 5 to 14 percent cement by weight and is used generally for stabilizing low plasticity soils and sandy soils.
2. Plastic soil-cement has enough water to produce a wet consistency similar to mortar. This material is suitable for use as water proof canal linings and for erosion protection on steep slopes where road building equipment may not be used.
3. Cement-modified soil is a mix that generally contains less than 5 percent cement by volume. This forms a less rigid system than either of the other types, but improves the engineering properties of the soil and reduces the ability of the soil to expand by drawing in water.

The cement requirement depends on the gradation of the soil. A well graded soil containing gravel, coarse sand and fine sand with or without small amounts of silt or clay will require 5 percent or less cement by weight. Poorly graded sands with minimal amount of silt will require about 9 percent by weight. The remaining sandy soils will generally require 7 percent. Non-plastic or moderately plastic silty soils generally require about 10 percent, and plastic clay soils require 13 percent or more.

**Bituminous Soil Stabilization**

Bituminous materials such as asphalts, tars, and pitches are used in various consistencies to improve the engineering properties of soils. Mixed with cohesive soils, bituminous materials improve the bearing capacity and soil strength at low moisture content. The purpose of incorporating bitumen into such soils is to water proof them as a means to maintain a low moisture content. Bituminous materials added to sand act as a cementing agent and produces a stronger, more coherent mass. The amount of bitumen added varies from 4 to 7 percent for cohesive materials and 4 to 10 percent for sandy materials. The primary use of bituminous materials is in road construction where it may be the primary ingredient for the surface course or be used in the subsurface and base courses for stabilizing soils.
21.10 SOIL STABILIZATION BY INJECTION OF SUITABLE GROUTS

Grouting is a process whereby fluid like materials, either in suspension, or solution form, are injected into the subsurface soil or rock.

The purpose of injecting a grout may be any one or more of the following:

1. To decrease permeability.
2. To increase shear strength.
3. To decrease compressibility.

Suspension-type grouts include soil, cement, lime, asphalt emulsion, etc., while the solution type grouts include a wide variety of chemicals. Grouting proves especially effective in the following cases:

1. When the foundation has to be constructed below the ground water table. The deeper the foundation, the longer the time needed for construction, and therefore, the more benefit gained from grouting as compared with dewatering.
2. When there is difficult access to the foundation level. This is very often the case in city work, in tunnel shafts, sewers, and subway construction.
3. When the geometric dimensions of the foundation are complicated and involves many boundaries and contact zones.
4. When the adjacent structures require that the soil of the foundation strata should not be excavated (extension of existing foundations into deeper layers).

Grouting has been extensively used primarily to control ground water flow under earth and masonry dams, where rock grouting is used. Since the process fills soil voids with some type of stabilizing material grouting is also used to increase soil strength and prevent excessive settlement.

Many different materials have been injected into soils to produce changes in the engineering properties of the soil. In one method a casing is driven and injection is made under pressure to the soil at the bottom of the hole as the casing is withdrawn. In another method, a grouting hole is drilled and at each level in which injection is desired, the drill is withdrawn and a collar is placed at the top of the area to be grouted and grout is forced into the soil under pressure. Another method is to perforate the casing in the area to be grouted and leave the casing permanently in the soil.

Penetration grouting may involve portland cement or fine grained soils such as bentonite or other materials of a particulate nature. These materials penetrate only a short distance through most soils and are primarily useful in very coarse sands or gravels. Viscous fluids, such as a solution of sodium silicate, may be used to penetrate fine grained soils. Some of these solutions form gels that restrict permeability and improve compressibility and strength properties.

Displacement grouting usually consists of using a grout like portland cement and sand mixture which when forced into the soil displaces and compacts the surrounding material about a central core of grout. Injection of lime is sometimes used to produce lenses in the soil that will block the flow of water and reduce compressibility and expansion properties of the soil. The lenses are produced by hydraulic fracturing of the soil.

The injection and grouting methods are generally expensive compared with other stabilization techniques and are primarily used under special situations as mentioned earlier. For a detailed study on injections, readers may refer to Caron et al., (1975).

21.11 PROBLEMS

21.1 Differentiate: (i) Compaction and consolidation, and (ii) Standard Proctor and modified Proctor tests.
21.2 Draw an ideal 'compaction curve' and discuss the effect of moisture on the dry unit weight of soil.

21.3 Explain: (i) the unit, in which the compaction is measured, (ii) 95 percent of Proctor density, (iii) zero air-voids line, and (iv) effect of compaction on the shear strength of soil.

21.4 What are the types of rollers used for compacting different types of soils in the field? How do you decide the compactive effort required for compacting the soil to a desired density in the field?

21.5 What are the methods adopted for measuring the density of the compacted soil? Briefly describe the one which will suit all types of soils.

21.6 A soil having a specific gravity of solids \( G_s = 2.75 \), is subjected to Proctor compaction test in a mold of volume \( V = 945 \text{ cm}^3 \). The observations recorded are as follows:

<table>
<thead>
<tr>
<th>Observation number</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass of wet sample, g</td>
<td>1389</td>
<td>1767</td>
<td>1824</td>
<td>1784</td>
<td>1701</td>
</tr>
<tr>
<td>Water content, percentage</td>
<td>7.5</td>
<td>12.1</td>
<td>17.5</td>
<td>21.0</td>
<td>25.1</td>
</tr>
</tbody>
</table>

What are the values of maximum dry unit weight and the optimum moisture content? Draw 100% saturation line.

21.7 A field density test was conducted by sand cone method. The observation data are given below:

(a) Mass of jar with cone and sand (before use) = 4950 g, (b) mass of jar with cone and sand (after use) = 2280 g, (c) mass of soil from the hole = 2925 g, (d) dry density of sand = 1.48 g/cm³, (e) water content of the wet soil = 12%. Determine the dry unit weight of compacted soil.

21.8 If a clayey sample is saturated at a water content of 30%, what is its density? Assume a value for specific gravity of solids.

21.9 A soil in a borrow pit is at a dry density of 1.7 Mg/m³ with a water content of 12%. If a soil mass of 2000 cubic meter volume is excavated from the pit and compacted in an embankment with a porosity of 0.32, calculate the volume of the embankment which can be constructed out of this material. Assume \( G_s = 2.70 \).

21.10 In a Proctor compaction test, for one observation, the mass of the wet sample is missing. The oven dry mass of this sample was 1800 g. The volume of the mold used was 950 cm³. If the saturation of this sample was 80 percent, determine (i) the moisture content, and (ii) the total unit weight of the sample. Assume \( G_s = 2.70 \).

21.11 A field-compacted sample of a sandy loam was found to have a wet density of 2.176 Mg/m³ at a water content of 10%. The maximum dry density of the soil obtained in a standard Proctor test was 2.0 Mg/m³. Assume \( G_s = 2.65 \). Compute \( \rho_d \), \( S \), \( n \) and the percent of compaction of the field sample.

21.12 A proposed earth embankment is required to be compacted to 95% of standard Proctor dry density. Tests on the material to be used for the embankment give \( \rho_{\text{max}} = 1.984 \text{ Mg/m}^3 \) at an optimum water content of 12%. The borrow pit material in its natural condition has a void ratio of 0.60. If \( G_s = 2.65 \), what is the minimum volume of the borrow required to make 1 cu.m of acceptable compacted fill?

21.13 The following data were obtained from a field density test on a compacted fill of sandy clay. Laboratory moisture density tests on the fill material indicated a maximum dry
density of 1.92 Mg/m$^3$ at an optimum water content of 11%. What was the percent compaction of the fill? Was the fill water content above or below optimum.

Mass of the moist soil removed from the test hole = 1038 g
Mass of the soil after oven drying = 914 g
Volume of the test hole = 478.55 cm$^3$

21.14 A field density test performed by sand-cone method gave the following data.

Mass of the soil removed + pan = 1590 g
Mass of the pan = 125 g
Volume of the test hole = 750 cm$^3$

Water content information
Mass of the wet soil + pan = 404.9 g
Mass of the dry soil + pan = 365.9 g
Mass of the pan = 122.0 g

Compute: $\rho_d$, $\gamma_d$, and the water content of the soil. Assume $G_s = 2.67$