# CHAPTER 20

# *Compaction*

## 20.1 GENERAL

*Compaction* refers to the densification of shallow soil layers by rollers. These rollers may be static cylindrical rollers, smooth or with protrusions; vibratory cylindrical rollers; or impact noncylindrical rollers. *Conventional compaction* refers to use of noninstrumented rollers, whereas *intelligent compaction* refers to use of instrumented rollers with feedback loops. *Dynamic compaction*, also discussed in this chapter, refers to dropping large weights from a given height onto the ground surface; this process creates a crater and compacts that material under the crater.

Compaction is required in many instances; examples include for the base layer of pavements, for embankment fills, for retaining wall backfills, for fill around pipes, and for landfills. Depending on the soil type and the size of the project, different compactors are used (Figure 20.1). For example, hand tampers (also called *jumping jacks*) are used in small areas around pipes, rollers are used for roadway compaction, and drop-weight compactors are used for dynamic compaction of large areas at larger depth.

The rollers are typically 50 to 150 kN in weight; the drums are 1 to 2 m in diameter and 2 to 3 m wide. The frequency of vibration for vibrating rollers is from 30 to 70 Hz. For dynamic compaction, the drop weight commonly varies from 50 to 250 kN and the drop height from 5 to 25 m. The depth over which the soil is compacted is up to 1 m for rollers and up to 10 m for dynamic compaction.

The compaction process typically takes the following steps:

1. Perform laboratory tests on the material to be compacted (Proctor test, for example) and establish the value of the soil property to be reached in the field work. These properties are most commonly the dry density and the water content. The modulus of deformation can also be used.
2. Write the field specifications, including the target dry density or target modulus within a chosen range of water content.
3. In the field, use compacting equipment to compact the soil in 0.15 m lifts after it is brought to the chosen water content.
4. In the field also carry out field tests to verify that the target values listed in the specifications have been reached.

## 20.2 COMPACTION LABORATORY TESTS

Laboratory tests are used to establish the characteristics of the soil to be compacted, to establish the target values to be achieved in the field, and to write the specifications for field work. The compaction process and the compaction curve associated with laboratory tests are described in detail in section 9.3; this section gives a brief summary.

The compaction curve links the dry density or a soil modulus to the water content. The dry density vs. water content curve is relatively flat, as the dry density is not very sensitive to the water content. Within the range of dry density variation, the curve has a bell shape (Figure 20.2). The reason for this is that at point A in Figure 20.2, the soil is relatively dry and it is difficult for a given compaction energy to bring the particles closer together. At point B the water content is such that water tension exists between the particles and hinders the effectiveness of the compaction process. At point C, the water tension loses its effect and the primary role of the water becomes to lubricate the contacts between particles, thereby allowing the given compaction effort to reach a low void ratio and a high dry density. At point D, the soil is nearing saturation and the added water simply increases the volume of the voids, which negates the benefit of the compaction. The maximum dry density $\gamma_{dmax}$ and the optimum water content $w_{opt}$ are two important parameters obtained from the curve (Figure 20.2). This curve is obtained in the laboratory with the Standard Proctor or Modified Proctor Compaction Test (Figure 20.3). These tests are described in detail in section 9.3.

The lines of equal degree of saturation can be presented on the same diagram as the dry density vs. water content curve (Figure 20.4). The equation for the saturation lines is:

$$\gamma_d = \frac{SG_s}{S + G_s w}\gamma_w \tag{20.1}$$

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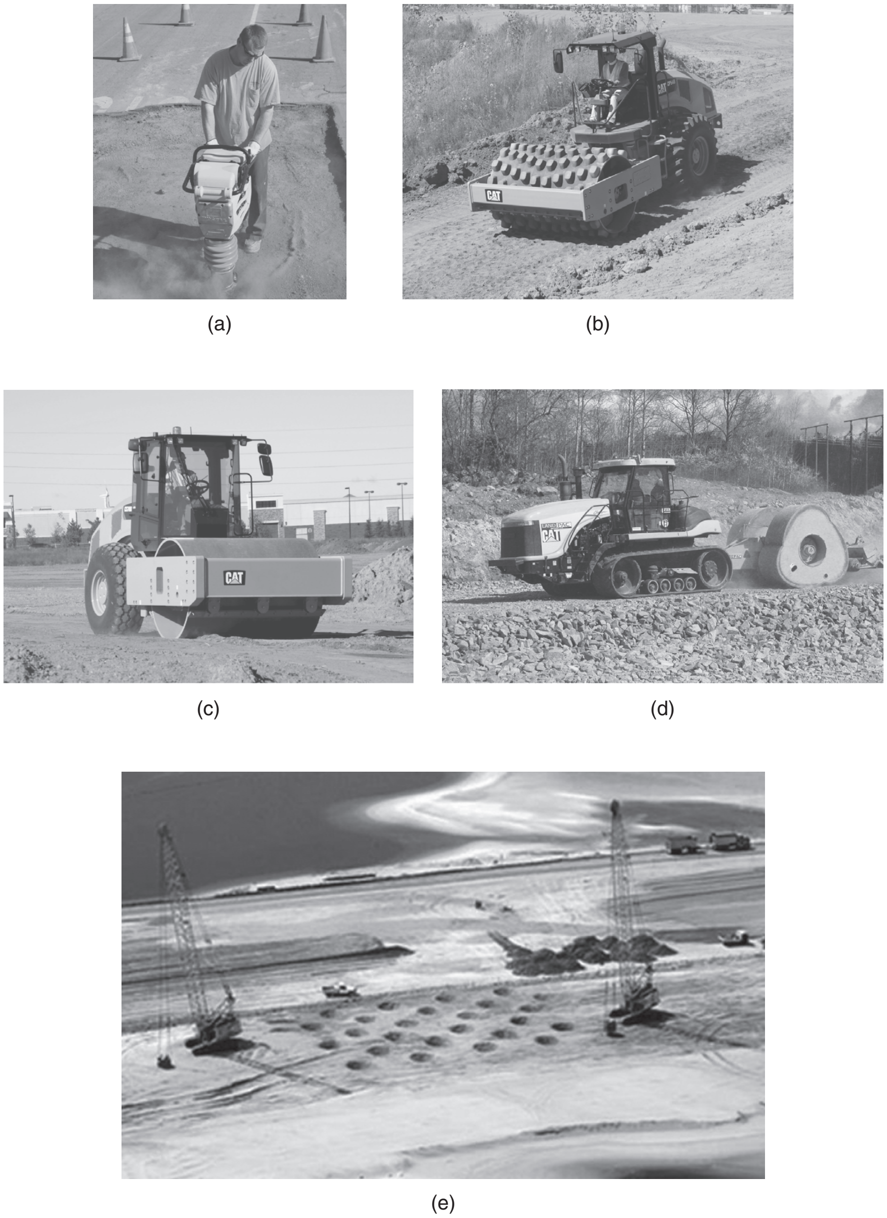



**Figure 20.1** Compaction equipment: (a) Hand tamper. (b) Sheep-foot roller. (c) Smooth cylindrical roller. (d) Impact noncylindrical roller. (e) Drop-weight compactor. (a: Courtesy of Multiquip. b, c: Images courtesy of Caterpillar. d: Courtesy of LANDPAC. e: Courtesy of Serge Varaksin).

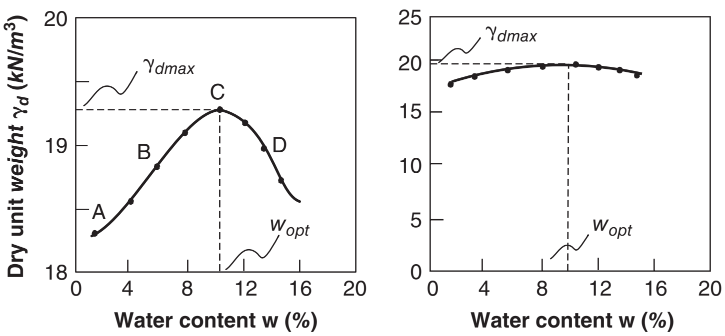



**Figure 20.2** Compaction curve: dry density.

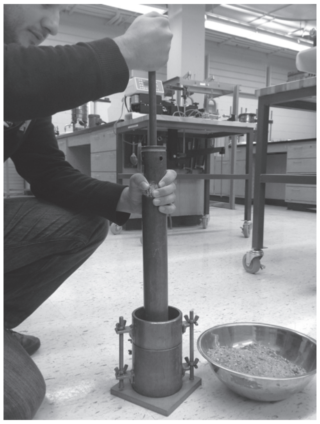


**Figure 20.3** Proctor compaction laboratory test.

where $\gamma_d$ is the dry density, S is the degree of saturation, $G_s$ is the specific gravity of the solids, w is the water content, and $\gamma_w$ is the unit weight of water. The derivation of this equation is shown in section 9.3.

In compaction control, the dry density can be replaced by the soil modulus as a governing parameter for specifications and quality control. The advantage of using the modulus is that the modulus is directly involved in the design calculations, whereas the dry density is not. The drawback is that the modulus depends on many factors (see section 14.2) and is not a single parameter for a given soil, whereas the dry density is. In the case of the modulus, the curve has the same bell shape as the dry density vs. water content curve, but is much more sensitive to the water content, especially on the wet side of the optimum water content (Figure 20.5). At low water contents, the modulus is influenced by the water tension that develops in the soil, whereas the dry density is not. As a result, the modulus curve can go back up at low water contents. The laboratory test to obtain the modulus vs. water content curve is the BCD test (Figure 20.6 and section 9.4), which is performed on the Proctor test sample. This is convenient because a dry density curve and a modulus curve can be obtained at the same time.

## 20.3 COMPACTION FIELD TESTS

The specifications indicate that the compacted soil must reach a dry density equal to a percentage of the maximum dry density measured in the laboratory (typically 95 to 100%) within a range of water content around the optimum water content. The specifications may also indicate that the compacted soil must reach a soil modulus equal to a percentage of the maximum soil modulus measured in the laboratory (typically 75% or so) within a range of water content around the optimum water content. Table 20.1 shows some possible target modulus values for pavement applications. Field tests are used to verify that the compaction work has been done according to specifications.

The field tests are divided into classic tests and new tests. The classic tests have been used for a long time and are relatively slow (15 to 30 minutes per test). They include the sand cone test for dry density, the rubber balloon test for dry density, and the nuclear density gage for dry density and water content. The new tests take only a few minutes to perform. They include the lightweight deflectometer, the BCD, and the field oven (Figure 20.7). All these tests are described in detail in section 7.11.

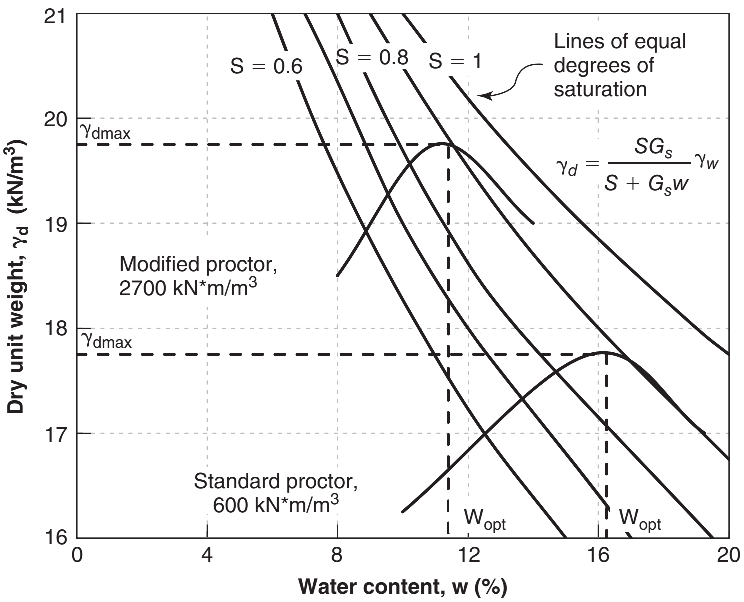



**Figure 20.4** Compaction curves and saturation lines.

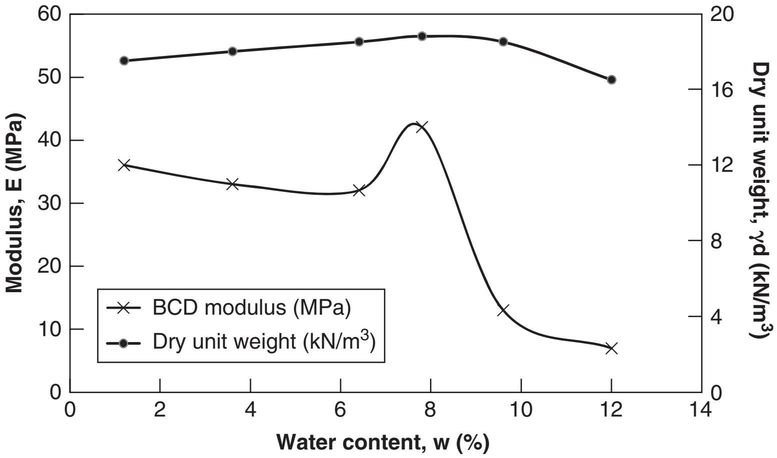



**Figure 20.5** Compaction curve: modulus.

**Figure 20.6** BCD-Proctor laboratory compaction test for modulus determination.

**Table 20.1 Modulus Target Values for Pavements**

| Soil Layer | Plate Test Reload Modulus (MPa) | Lightweight Deflectometer Modulus (MPa) | BCD Reload Modulus (MPa) |
|---|---|---|---|
| Base course | 100–150 | 100–150 | 55–85 |
| Subgrade soil | 45–80 | 45–80 | 25–45 |

## 20.4 COMPACTION AND SOIL TYPE

Different soils react differently to different compaction equipment. Coarse-grained soils are most effectively compacted through vibration combined with pressure. Pressure alone increases the effective stress and therefore the friction between particles, thereby preventing their sliding into a more compact position. Vibration breaks the friction bonds and lets the particles settle into a tighter arrangement. Fine-grained soils are most effectively compacted through kneading and pressure. Vibration may simply increase the water stress if the soil is saturated. Also, coarse-grained soils tend to reach optimum compaction at water contents lower than fine-grained soils. However, coarse-grained soils tend to reach maximum dry densities that are higher than those of fine-grained soils (Figure 20.8). Table 20.2 shows a rating of applications for various pieces of compaction equipment.

## 20.5 INTELLIGENT ROLLER COMPACTION

*Continuous control compaction* (CCC) refers to compaction with rollers that are instrumented, make measurements on the fly, and give an image of the complete compacted area

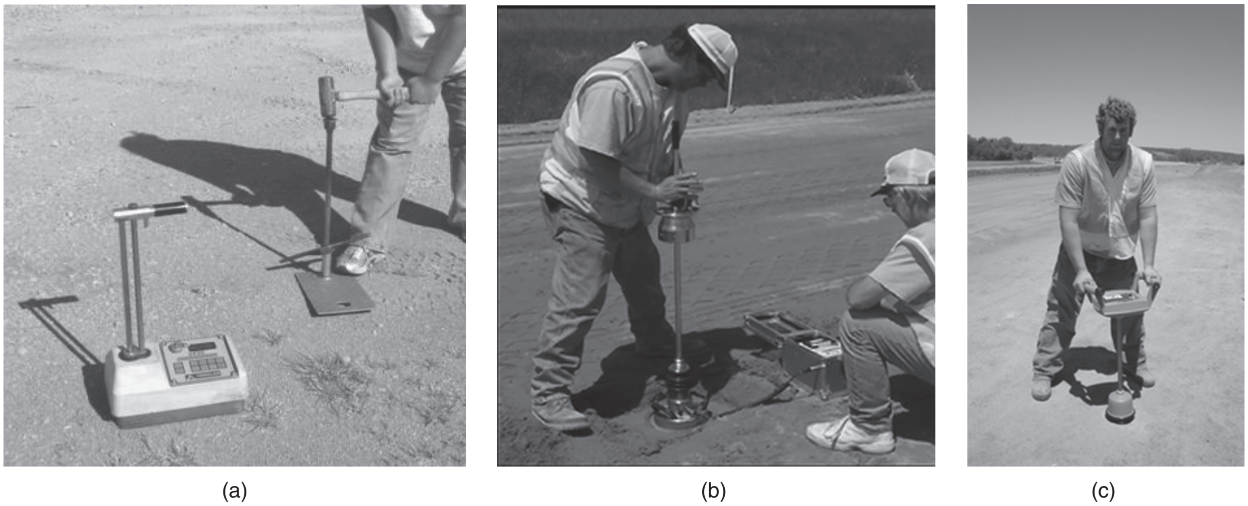



**Figure 20.7** Compaction control tests in the field: (a) Nuclear gage. (b) Lightweight deflectometer. (c) BCD. (a: Photo by Lindsey D. Fields, Envirotech Engineering & Consulting, Inc. b: Courtesy of Minnesota Department of Transportation.)

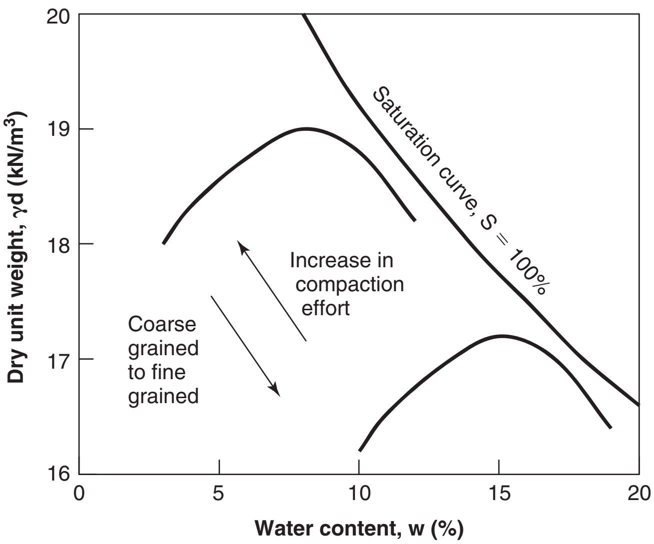
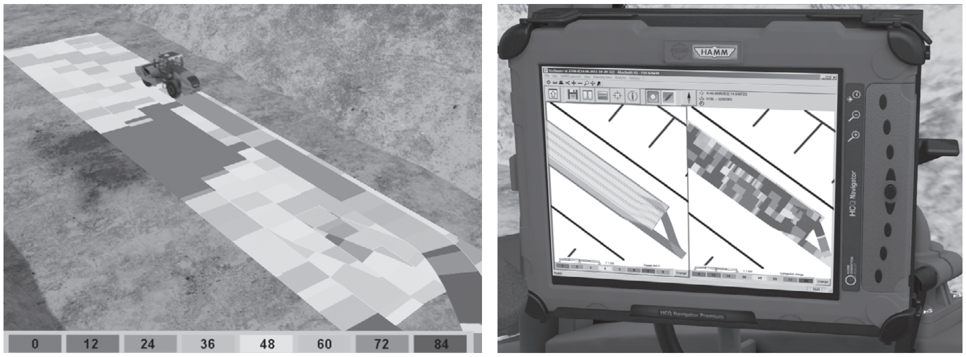



**Figure 20.8** Influence of soil type and compaction effort on dry density curve.

**Table 20.2 Applicability of Compaction Equipment for Various Soils**

| Soil Type | Static Sheep Foot Roller | Vibrating Cylindrical Roller | Impact Noncylindrical Roller | Dynamic Compaction |
|---|---|---|---|---|
| Gravel | Poor | Good | Good | Good |
| Sand | Poor | Very good | Good | Good |
| Silt | Good | Poor | Poor | Medium |
| Clay | Very good | Poor | Medium | Poor if saturated, good if unsaturated. |
| Domestic waste | Good | Poor | Good | Very good |

with the values of the soil parameter measured (Figure 20.9). *Intelligent compaction* (IC) refers to CCC with the added feature that the roller is able to change its settings nearly instantaneously when it comes to a soft spot and to optimize the compaction process (Figure 20.10) while keeping track of the global position through GPS (Figure 20.11). In CCC and IC, the soil parameter most often measured is a soil modulus E.




(a) (b)

**Figure 20.9** Continuous coverage in CCC and IC: (a) Continuous mapping of soil stiffness. (b) Screen display. (Courtesy of HAMM AG.)

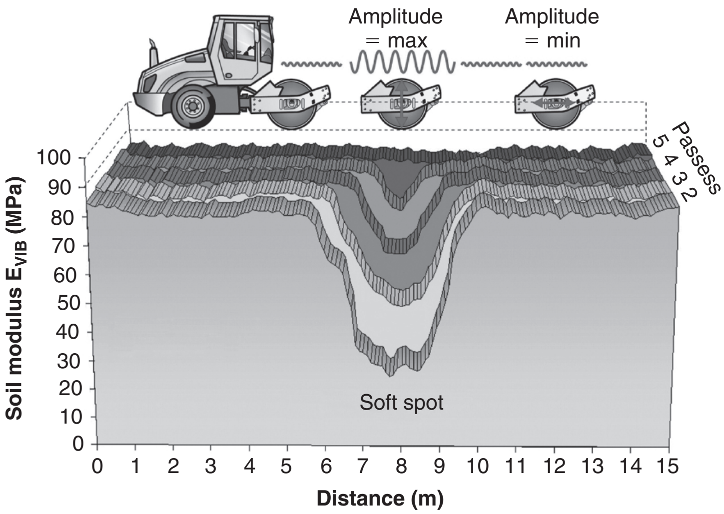



**Figure 20.10** Intelligent compaction roller adjustment settings. (Courtesy of BOMAG.)

(a) (b)

**Figure 20.11** Intelligent rollers and readout equipment: *(a)* Roller. *(b)* Readout and control. (a: Courtesy of BOMAG; b: Courtesy of Ammann.)

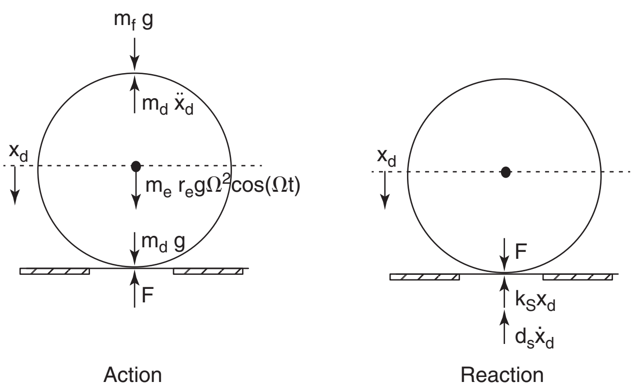



**Figure 20.12** Forces acting on the drum of a vibrating roller (Anderegg 1997).

### 20.5.1 Soil Modulus from Vibratory Rollers

In the case of vibratory rollers, the modulus $E$ is obtained from measurements of the acceleration of the roller (Anderegg 1997). A single degree of freedom model is used to represent the roller-soil interaction (Figure 20.12). Vertical equilibrium of the drum gives:

$$F = -m_d \ddot{x}_d + m_e r_e \omega^2 \cos(\omega t) + (m_f + m_d) g \tag{20.2}$$

where $F$ is the vertical force at the bottom of the drum, $m_d$ is the mass of the drum, $\ddot{x}_d$ is the linear vertical acceleration of the drum, $m_e$ is the eccentric mass creating the vibration, $r_e$ is the radial distance at which $m_e$ is attached, $\omega$ is the circular frequency of the rotating shaft, $t$ is the elapsed time, $m_f$ is the mass of the frame, and g is the acceleration due to gravity. All quantities on the right side of Eq. 20.2 come from the roller specifications and are known except for $\ddot{x}_d$, which is measured with an accelerometer on the drum axis. The vibration frequency of most rollers ranges from 30 to 70 Hz, their weight from 50 to 150 kN, their diameters from 1 to 2 meters, and their width from 2 to 3 meters.

The soil resistance $F$ can also be obtained as follows if the soil is represented by a spring-and-dashpot model:

$$F = k_S x_d + c_S \dot{x}_d \tag{20.3}$$

where $k_s$ is the soil spring stiffness, $x_d$ is the vertical displacement of the drum, $c_s$ is the soil damping coefficient, and $\dot{x}_d$ is the vertical velocity of the soil boundary. In Eq. 20.3, $k_s$ is the parameter to be solved for, $x_d$ and $\dot{x}_d$ are obtained by integration of the acceleration signal, and $c_s$ is typically assumed to be about 20% of critical damping. Numbers in the range of 50 to 100 MN/m for $k_s$ and 150 to 250 kN s/m for $c_s$ have been measured (Van Susante and Mooney 2008). An example of contact force vs. time and contact force vs. displacement is shown in Figure 20.13. Equations 20.2 and 20.3 are combined and the soil stiffness $k_s$ can be obtained from the combined equation:

$$k_S = \frac{m_e r_e \omega^2 \cos(\omega t) + (m_f + m_d) g - m_d \ddot{x}_d - c_S \dot{x}_d}{x_d} \tag{20.4}$$

As explained in section 14.6, the soil stiffness $k_s$ is not an independent soil parameter because it depends on the size of the loaded area. The soil modulus $E$, in contrast, is an independent soil parameter; therefore, it is desirable to know how to obtain $E$ from $k_s$. This problem was solved by Hertz in 1895 and further developed by Lundberg (1939):

$$k_s = \frac{F}{x_d} = \frac{\pi L E}{2(1-\nu^2)\left(2.14 + \frac{1}{2} Ln\left[\frac{\pi L^3 E}{16\left(1-\nu^2\right)(m_f + m_d) R g}\right]\right)} \tag{20.5}$$

where $k_s$ is the soil stiffness, $F$ is the force applied, $x_d$ is the settlement of the drum, $L$ is the drum width, $\upsilon$ is Poisson's ratio, $Ln$ is the natural logarithm, $m_f$ and $m_d$ are the masses contributed by the frame and the drum of the

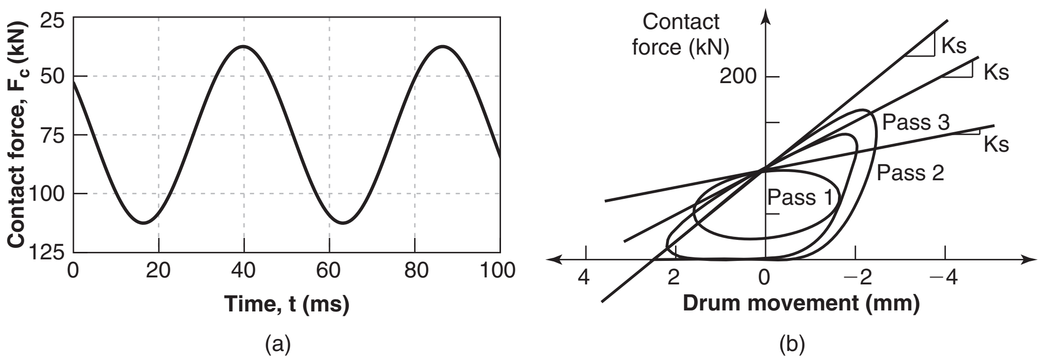



**Figure 20.13** Contact force under the vibrating roller. *(a)* Force-time (Van Susante and Mooney 2008). *(b)* Force-displacement (Floss and Kloubert 2000)

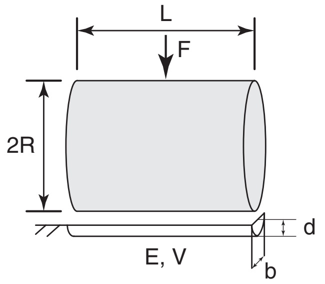



**Figure 20.14** Drum on elastic soil problem. (After Lundberg 1939.)

roller respectively, $R$ is the radius of the drum, and $g$ is the acceleration due to gravity. Lundberg also gave the width $b$ of the contact area between the drum and the soil:

$$b = \sqrt{\frac{16}{\pi}\frac{R(1-\upsilon^2)}{E}\frac{F}{L}} \tag{20.6}$$

As can be seen and as could be anticipated, the width b is inversely proportional to the soil modulus $E$. The roller soil contact area has a length $L$ and a width $b$. Because the ratio $L/b$ is very large, the loading is similar to a strip footing. Under static conditions, the depth of influence under a strip footing is $4b$. Thus, under the first pass, the width $b$ is large because the soil is not very stiff and, as a result, the depth of influence is larger. Under subsequent passes, the soil stiffens and $b$ decreases, and so does the depth of influence (Figure 20.14). The width $b$ varies commonly between 200 mm on soft soils to 20 mm on very stiff soils.

### 20.5.2 Roller Measurements as Compaction Indices

The *machine drive power* (MDP) (White et al. 2005) is a roller index that can be used to evaluate the degree of compaction generated by any roller. The principle is that if the soil is soft, it will take more power for the roller to roll forward and compact the soil; if the soil is stiff, it will take less power for the roller to roll forward. The difference, as illustrated in Figure 20.15, is that the roller on soft soil has to overcome a lot more soil deformation energy than the roller on stiff soil. It is similar to the difference you feel when you run on loose sand compared to running on pavement; it is a lot more difficult (and takes more energy) to run on loose sand than on pavement.

The MDP is calculated as follows:

$$\text{MDP} = P_g - Wv\left[\sin\alpha + \frac{a}{g}\right] - (mv + b) \tag{20.7}$$

where MDP is the machine drive power (kJ/s), $P_g$ is the gross power needed to move the machine (kJ/s), W is the roller weight (kN), $v$ is the roller velocity (m/s), $\alpha$ is the slope angle (roller pitch from a sensor), a is the machine acceleration (m/s$^2$), $g$ is the acceleration due to gravity (m/s$^2$), and m and b are the machine internal loss coefficient specific to a particular machine (kJ/m and kJ/s respectively). The second and third terms in Eq. 20.7 represent the machine power associated with a sloping grade and the internal machine loss respectively. The MDP represents only the machine power associated with the soil properties (White and Thompson 2008) and decreases as the soil becomes more compact.

For vibrating rollers, the compaction meter value (CMV) can be used. Some of the early work on continuous compaction control demonstrated that various indices incorporating drum acceleration amplitude and the amplitude of its harmonics could be linked to the stiffness of the underlying soil. Based on this early research, the CMV was proposed (Thurner and Sandström 1980). The CMV is a dimensionless compaction parameter that depends on roller dimensions (drum diameter and weight), roller operation parameters (frequency, amplitude, speed), soil mechanical properties (strength and stiffness), and soil stratigraphy. It is determined using the roller acceleration signal and calculated as:

$$CMV = C\frac{a_{2\Omega}}{a_{\Omega}} \tag{20.8}$$

where C is a constant (300), $a_{2\Omega}$ is the acceleration of the first harmonic component of the vibration, $a_{\Omega}$ is the acceleration of the fundamental component of the vibration, and $\Omega$ is the vibrating frequency of the roller.

If the soil is soft, the roller stays in contact with the soil and the roller and the soil move together; therefore the signal is sinusoidal and there is no other frequency content in the signal except for $\Omega$, and CMV is thus zero. As the soil

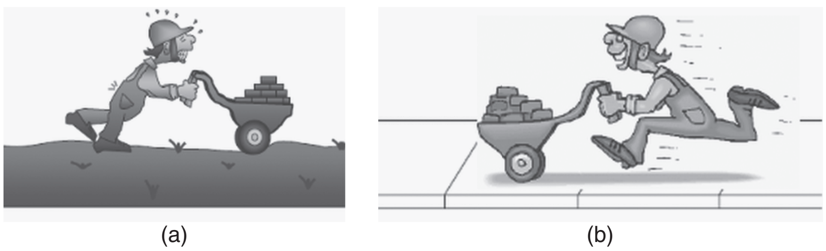



**Figure 20.15** Principle of machine drive power: *(a)* Soft soil = hard to push. *(b)* Hard flat soil = easy to push.

becomes stiffer, the roller starts to jump and knock; this increases the frequency content of the signal, which becomes more complicated than just a sinusoidal signal, and the value of $a_{2\Omega}$ increases. A Fourier transform analysis of the time domain signal gives the frequency content and therefore the value of $a_{2\Omega}$. The CMV at a given point indicates an average value over an area with a width equal to the width of the drum and a length equal to the distance the roller travels in a set time period (0.5 seconds, for example).

The soil modulus E, or the MDP, or the CMV can be used to evaluate the degree of compaction achieved by the roller and a degree-of-compaction map of the area covered by the roller can be generated and located according to the GPS (Figure 20.9).

## 20.6 IMPACT ROLLER COMPACTION

Traditionally, compaction rollers have been cylindrical and have used their static weight, kneading action, or vibratory force to achieve the specific soil stiffness and soil strength. However, traditional rollers may have an energy capacity that is too low compared to the need. This might be the case for breaking the interparticle bonds of collapsible sands, for example. Impact compaction rollers were developed to alleviate this type of problem. They have noncircular drums (Figure 20.16) that rotate and fall to impact the ground surface. Such rollers tend to provide deeper compaction because the impact generates a wave that propagates at depth. Figure 20.17 demonstrates this point. It shows a freeze-frame picture of a numerical simulation movie describing the stress field in the soil as the roller passes over that spot (Kim 2010). The simulation compares the case of a cylindrical roller with one of a triangular impact roller. These types of simulations were used to generate the depth chart of Figure 20.18, which indicates that the depth of influence decreases as the soil becomes stiffer and as the roller becomes closer to a cylindrical roller. Here the depth of influence is defined as the depth at which the stress becomes equal to one-tenth of the stress under the roller at the ground surface. In that sense, impact rollers are more efficient; however, the biggest drawback is that they do not provide evenly compacted surfaces (Figure 20.16).

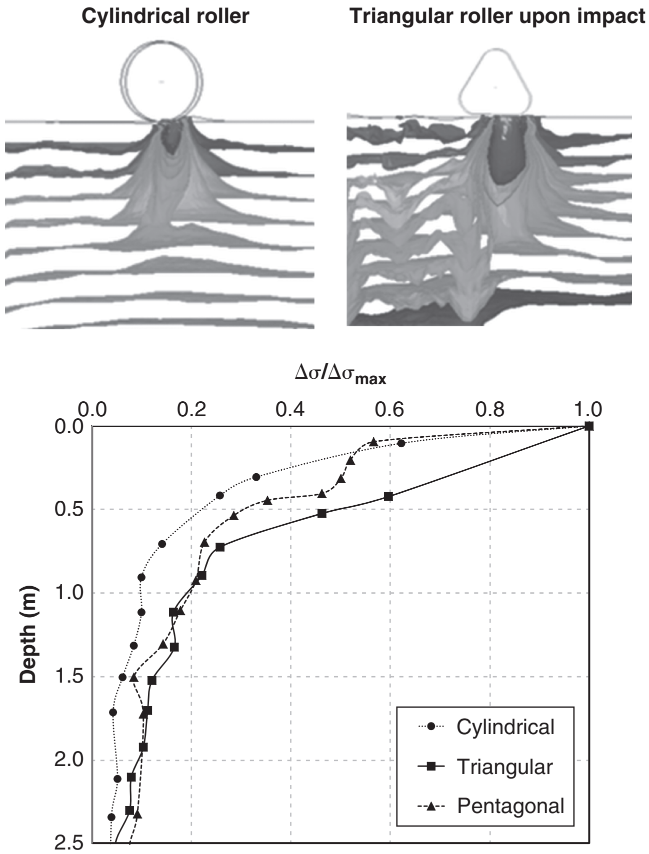
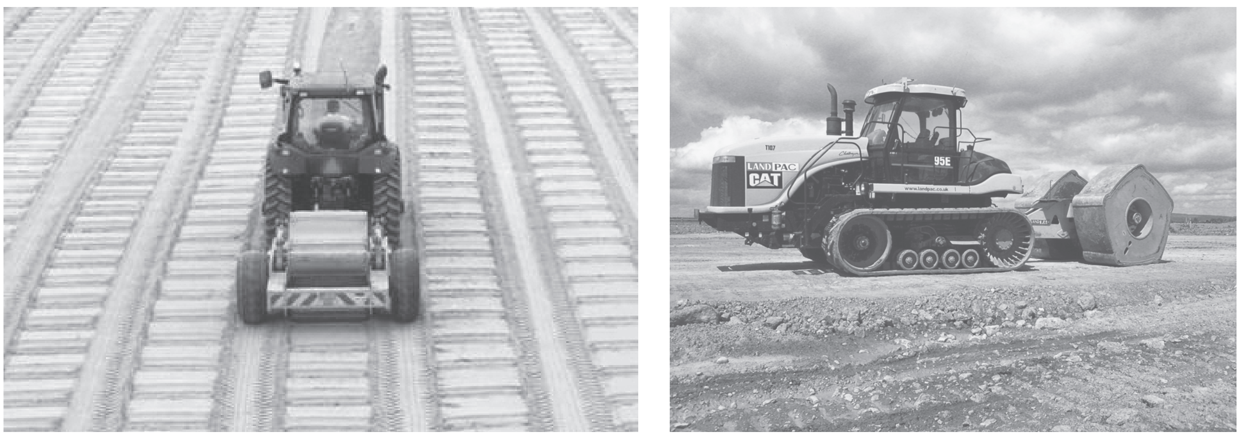



**Figure 20.17** Stress field under rollers (Kim 2010).




(a) (b)

**Figure 20.16** Impact rollers. (a: Courtesy of LAND-PAC; b: Courtesy of Brooms)

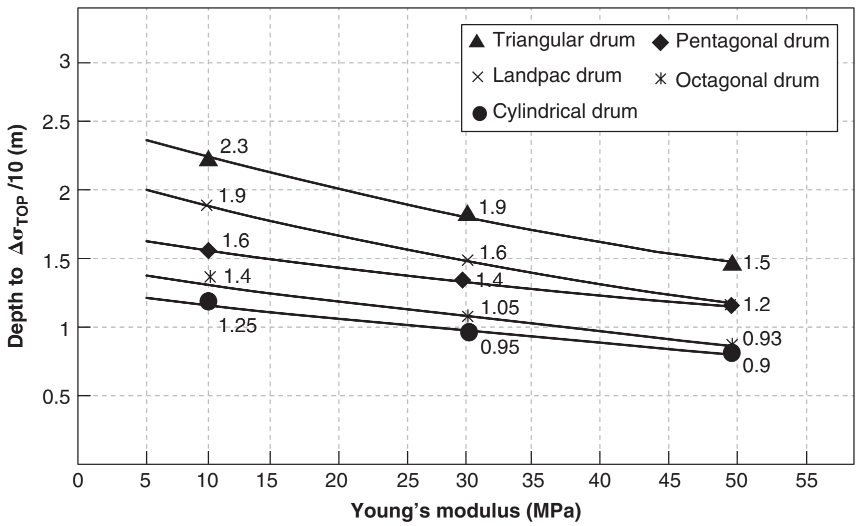



**Figure 20.18** Depth of influence of rollers (Kim 2010).

The following comments summarize the situation with cylindrical and impact rollers:

1. The width of the contact area between the drum and the soil controls the depth of compaction. The softer the soil is, the deeper the roller sinks in the soil, the wider the contact area is, and the deeper the compaction is. Therefore, the depth of compaction depends on the stiffness of the soil. Hence, the depth of compaction decreases with the number of passes.

2. The surface pressure controls the degree of compaction. This pressure is higher for impact rollers than for cylindrical rollers due to the dynamic effect. However, the distribution of the pressure is much more uneven for impact rollers than for cylindrical rollers.

3. The depth of compaction is larger for impact rollers because they impart higher stresses that increase the penetration of the roller drum into the soil, thereby increasing both the width and depth of influence. The increased depth of influence is also due to wave propagation during the impact. These waves can propagate much deeper than the typical depth of influence for static loading.

4. If time and equipment allow it, it makes sense to compact first with an impact roller and use several passes to minimize the extent of the areas between impacts. Then, finish by using a cylindrical roller to provide a more evenly compacted surface and optimize the compaction of the shallow layers.

5. The process described in item 4 combines the benefits of both types of rollers: compaction of the deep layers (0.5 to 1.5 m) with the impact roller followed by evening out of the compaction of the shallow layers (0 to 0.5 m) with the cylindrical roller without disturbing the deep layers.

## 20.7 DYNAMIC OR DROP-WEIGHT COMPACTION

Dynamic compaction is often credited to Louis Menard (1975). It consists of lifting a heavy weight of mass $M$ and dropping it from a preset height $H$ so as to pound the soil and compact it in the process (Figure 20.19). The pounding is repeated at the same spot for a number of drops (say, 6 times) and thus creates a crater; then the crane moves to another

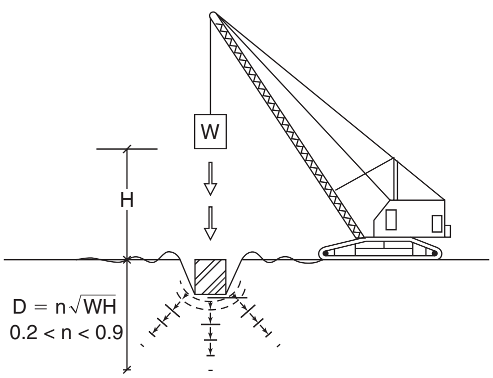



**Figure 20.19** Dynamic compaction. (b: Courtesy of Menard Bachy Pty Ltd.)

location and repeats the process in a grid pattern. The spacing between impact points is about 2 times the diameter of the tamper. The crater should not be any deeper than 1.5 to 2 times the height of the tamper, to avoid collapse of the walls of the crater and associated difficulties in pulling the tamper out of the crater. The craters are typically backfilled with coarse-grained soil. After completing the first grid, the crane does a second pass by dropping the weight on the intermediate spots to complete the surface treatment. The drop weights commonly weigh 50 to 300 kN and drop from heights of up to 30 m, reaching velocities of 10 to 20 m/s at impact. During the final pass, called *ironing*, a flatter weight is dropped to smooth out the bumps.

Upon each drop, the energy generated by the impact propagates to the deeper layers by compression and shear wave propagation. Thus, the effectiveness of this compaction process depends on the dynamic response characteristics of the soil being compacted. Trials are usually run ahead of time to evaluate the potential results, but dynamic compaction works best for unsaturated coarse-grained soils and is not applicable to saturated fine-grained soils. The maximum depth $D$ that can be compacted by dynamic compaction is influenced by many factors, including the soil properties, the groundwater level, the number of drops at each location, and the amount of time elapsed between the grids. The following equation is recommended by Lukas (1995):

$$D = n\sqrt{MH} \tag{20.9}$$

where $n$ is a site factor less than 1 (Table 20.3), M is the mass of the tamper in tonnes (1000 kg), and H is the average drop height in meters.

**Table 20.3 Recommended Values of n for Different Soils (Lukas 1995)**

| Soil Type | Degree of Saturation | Recommended $n$ Value |
|---|---|---|
| Pervious soil deposits, granular soils | High | 0.5 |
| | Low | 0.5–0.6 |
| Semipervious soil deposits, primarily silts with plasticity index <8 | High | 0.35–0.4 |
| | Low | 0.4–0.5 |
| Impervious deposits, primarily clayey soils with plasticity index of >8 | High | Not recommended |
| | Low | 0.35–0.40 Soil should be at water content less than the plastic limit |

**Table 20.4 Values of the Equipment Factor C (Chu et al. 2009)**

| Drop Method | Free Drop | Rig Drop | Mechanical Winch | Hydraulic Winch | Double Hydraulic Winch |
|---|---|---|---|---|---|
| Equipment factor C | 1.0 | 0.89 | 0.75 | 0.64 | 0.5 |

For an applied energy of 1 to 3 MJ/m$^2$ and for a temper drop using a single cable with a free spool drum, Eq. 20.9 was modified by Varaksin as follows (Chu et al. 2009):

$$D = C\delta\sqrt{MH} \tag{20.10}$$

where C is an equipment factor given in Table 20.4, and $\delta$ is a soil factor equal to 0.9 for metastable soils, young fills, or very recent hydraulic fills, and equal to 0.4 to 0.6 for sands. Compaction depths of 10 m can be achieved with the heavier tampers (e.g., 20 tonnes dropping 20 m). The improvement ratio f, defined as the ratio of the strength after dynamic compaction over the strength before dynamic compaction, varies with depth and is typically measured by in situ testing (PMT, CPT, SPT). Varaksin proposes the following variation of f with depth below the tamper:

$$f = f_1 + (f_2 - f_1)\left(\frac{z}{D}\right)^2 \tag{20.11}$$

where $f_1$ and $f_2$ are the improvement ratios at the ground surface and at the depth $D$, respectively, and $z$ is the depth at which f is evaluated.

The energy $E$ input in the soil for each drop by dynamic compaction can be presented per unit of surface area compacted ($E_2$ in kJ/m$^2$) or per unit of soil volume compacted ($E_3$ in kJ/m$^3$). The energy per unit surface area compacted $E_2$ is:

$$E_2 = \frac{W \times H \times N \times P}{s^2} \tag{20.12}$$

where $W$ is the weight of the tamper in kN, $H$ is the height of drop in meters, $N$ is the number of drops, $P$ is the number of passes, and s is the grid spacing in meters for the pounding pattern.

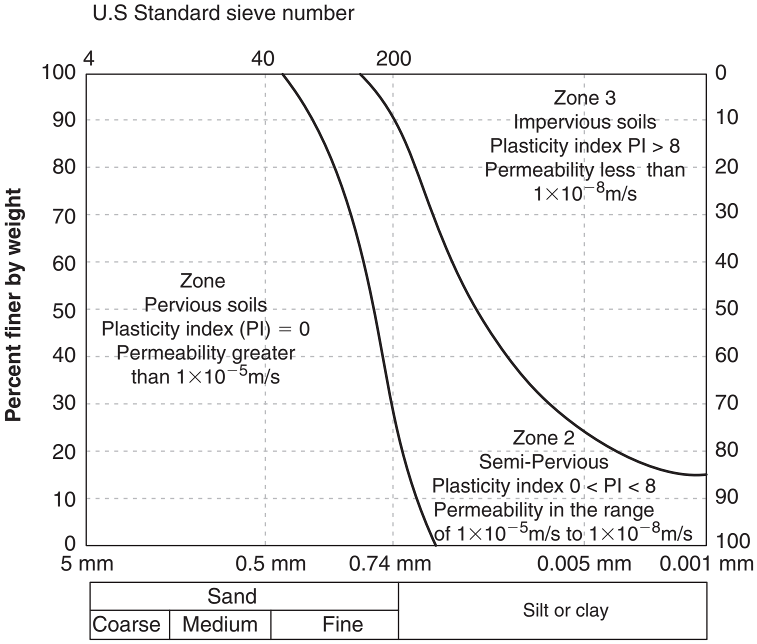
The energy per unit volume of soil compacted $E_3$ is:

$$E_3 = \frac{E_2}{D} \tag{20.13}$$

where $D$ is the depth of soil compacted. Lukas (1995) gives a list of typical energies used for different soil types (Table 20.5).

**Table 20.5 Applied Energy Guidelines for $E_3$ (Lukas 1995)**

| Type of Deposit | Unit Applied Energy (kJ/m$^3$) | Percent Standard Proctor Energy |
|---|---|---|
| Pervious coarse-grained soil, Zone 1 | 200–250 | 33–41 |
| Semipervious fine-grained soils, Zone 2; and clay fills above the water table, Zone 3 | 250–350 | 41–60 |
| Landfills | 600–1100 | 100–180 |




*Note*: Standard Proctor energy equals 600 kJ/m$^3$.

The degree of efficiency of dynamic compaction is measured by comparing the results of soil tests performed before and after the compaction process. The preferred test is the pressuremeter test (Figure 20.20), but the cone penetrometer test and the standard penetration test are also used. The *depth of compaction* is defined here as the depth to which the soil strength has increased compared to the initial state. This increase is not constant with depth, as seen in Figure 20.20. A typical use of dynamic compaction is to dynamically compact the soil deposit so that shallow foundations can be used instead of more expensive pile foundations. The decision is based on comparing the cost of a shallow foundation plus dynamic compaction to the cost of a deep foundation.

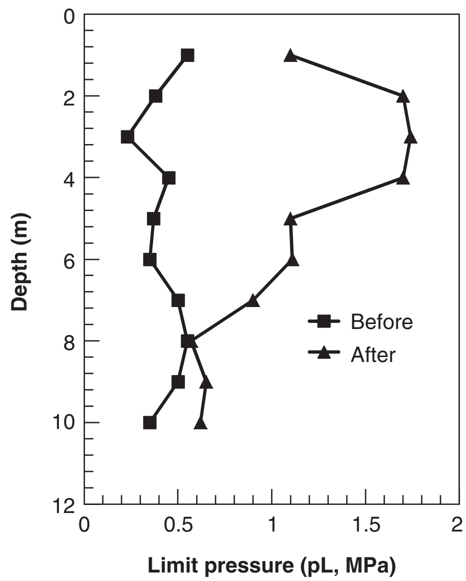



**Figure 20.20** Improvement of soil strength due to dynamic compaction.

Dynamic compaction induces soil vibrations. These vibrations are typically measured in terms of the peak velocity of the soil particles or PPV. The PPV depends on a number of factors, primarily the energy of the impact and the distance from the impact. Mayne (1985) assembled a database

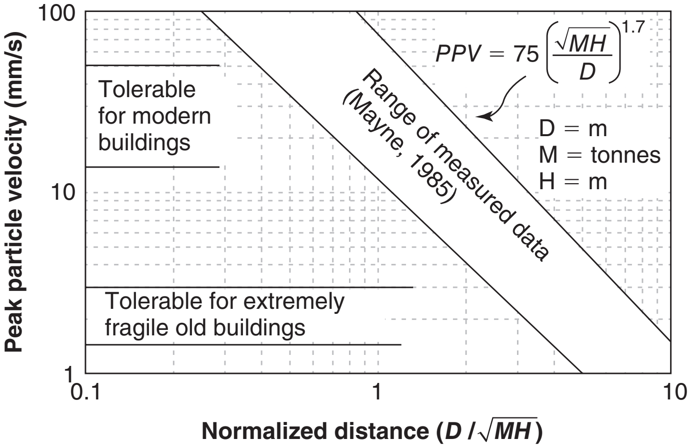



**Figure 20.21** Peak particle velocity due to dynamic compaction. (After Mayne 1985.)

giving the range of values shown in Figure 20.21. The following equation gives an upper bound of the PPV generated by a dynamic compaction impact according to the data in Figure 20.21:

$$PPV = 75\left(\frac{\sqrt{MH}}{d}\right)^{1.7} \tag{20.14}$$

where PPV is the peak particle velocity in mm/s, $M$ is the mass of the tamper in tonnes (1000 kg), $H$ is the drop height in m, and d is the distance from the impact location in m. This value of the PPV can be compared with what is tolerable. Typical values of PPV for damage threshold vary from 1 to 3 mm/s for very old and fragile buildings to 20 to 50 mm/s for modern buildings (Figure 20.21).

## PROBLEMS

20.1 Referring to Figure 20.2, give the maximum dry density and the optimum water content. What is the degree of saturation of the soil at that point if the specific gravity of solids is 2.7?

20.2 Use the data points from Figure 20.5 to draw a correlation between dry density and modulus. Find the R square value for that correlation. Discuss whether there should be or should not be a correlation between dry density and modulus.

20.3 Correlate the depth of the imprint that you can make with your thumb as a function of the soil modulus being compacted.

20.4 A vibratory intelligent roller weighs 140 kN; it has a drum diameter of 1.4 m and a drum length of 2.1 m. The eccentric weight generates a moment ($m_e r_e$ in Eq. 20.2) equal to 1.5 kg.m at an angular frequency of 200 rd/s. The drum weighs 30 kN and the added weight from the frame above the drum is 20 kN. The measured peak acceleration of the drum is + or −3g. Assume that the inertia force generated by the vibration of the frame is negligible compared to the one generated by the drum. Draw the acceleration signal, the velocity signal, and the displacement signal at the drum-soil contact point.

20.5 The vibratory roller from problem 4 rests on a soil that has a stiffness $k_s$ to be determined. The damping coefficient of the soil is 200 kN s/m. Calculate the stiffness of the soil $k_s$, the modulus of the soil $E$, and the width $b$ of the contact area.

20.6 A landfill must be compacted by dynamic compaction to improve its bearing capacity. The required depth of compaction is 10 m. Determine the weight of the tamper to be used and the drop height required to achieve the 10 m depth of compaction.

20.7 Regarding the landfill in problem 7, the closest building is located at 100 m from the edge of the compaction zone. Calculate the peak particle velocity that can be expected. Would this be normally tolerable for a recently constructed building?

## Problems and Solutions

### Problem 20.1

Referring to Figure 20.2, give the maximum dry density and the optimum water content. What is the degree of saturation of the soil at that point if the specific gravity of solids is 2.7?

**Solution 20.1**

The maximum dry unit weight and the optimum water content are obtained from the compaction curve. The values from Figure 20.2 are:

- Maximum dry unit weight, $\gamma_{dmax} = 19.3\ \text{kN/m}^3$
- Optimum water content, $w_{opt} = 10\%$
- Degree of saturation, S

Using the equation that links these quantities:

$$\gamma_d = \frac{G_s \gamma_w}{1 + \dfrac{G_s w}{S}}$$

$$S = \frac{G_s w}{\dfrac{G_s \gamma_w}{\gamma_d} - 1} = \frac{2.7 \times 0.10}{\dfrac{2.7 \times 9.81}{19.3} - 1} = 0.725 = 72.5\%$$

**Problem 20.2**

Use the data points from Figure 20.5 to draw a correlation between dry density and modulus. Find the *R* square value for that correlation. Discuss whether there should be or should not be a correlation between dry density and modulus.

**Solution 20.2**

The values of the modulus and dry density are as plotted in Figure 20.1s, and the *R* square value shows that there is not a good correlation between dry unit weight and soil modulus. The modulus depends on many other factors besides the amount of solids per unit volume. Factors such as structure, cementation, and stress history also affect the modulus. It is not surprising that there is no good correlation between dry density and modulus.

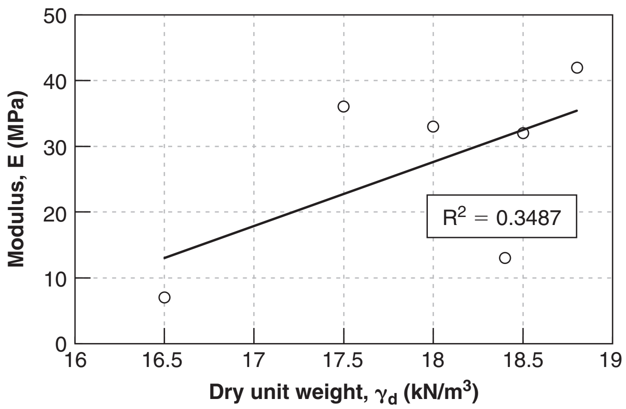



**Figure 20.1s** The correlation between dry unit weight and soil modulus.

**Problem 20.3**

Correlate the depth of the imprint that you can make with your thumb as a function of the soil modulus being compacted.

**Solution 20.3 (Figure 20.2s)**

$\upsilon$ = Poisson's ratio of the soil (assumed to equal 0.35)
S = settlement (m)
E = soil modulus

$$s = \frac{\pi}{4}(1 - \upsilon^2).p.\frac{B}{E}$$

Assuming that you can generate 0.1 kN with your thumb and that the area of contact between your thumb and the surface is 30 by 20 mm, the pressure under your thumb is:

$$p = \frac{F}{A} = \frac{100 \times 10^{-3}}{20 \times 30 \times 10^{-6}} = 166.67(\text{kN/m}^2)$$

The settlement can be calculated as:

$$s(m) = \frac{\pi}{4}(1 - \upsilon^2).p.\frac{B}{E} = \frac{\pi}{4}(1 - 0.35^2) \times 166.67 \times \frac{0.02}{E} = \frac{2.297}{E}$$

or, with E in kPa and s in mm:

$$\text{E (kPa)} = 2300/\text{s (mm)}$$

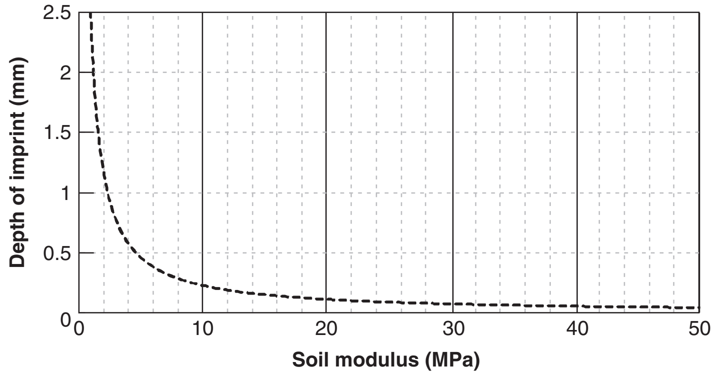



**Figure 20.2s** Depth of finger imprint vs. soil modulus.

**Problem 20.4**

A vibratory intelligent roller weighs 140 kN; it has a drum diameter of 1.4 m and a drum length of 2.1 m. The eccentric weight generates a moment ($m_e r_e$ in Eq. 20.2) equal to 1.5 kg.m at an angular frequency of 200 rd/s. The drum weighs 30 kN and the added weight from the frame above the drum is 20 kN. The measured peak acceleration of the drum is + or –3g. Assume that the inertia force generated by the vibration of the frame is negligible compared to the one generated by the drum. Draw the acceleration signal, the velocity signal, and the displacement signal at the drum-soil contact point.

**Solution 20.4**

$$\ddot{x} = a_{\max} \sin(\omega t) = 30 \sin(200t)$$

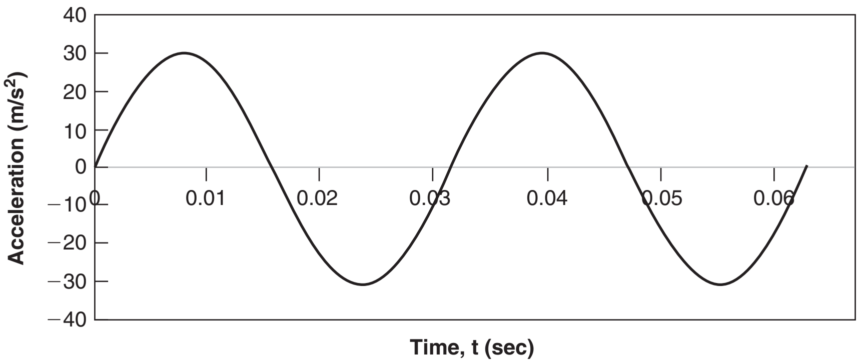



**Figure 20.3s** Acceleration signal.

$$\dot{x} = -\frac{a_{\max}}{\omega} \cos(\omega t) = -0.15 \cos(200t)$$

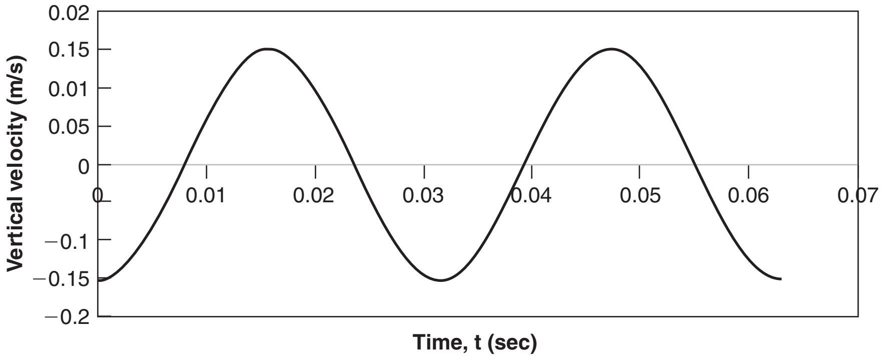



**Figure 20.4s** Velocity signal.

$$x = -\frac{a_{\max}}{\omega^2}\sin(\omega t) = -0.75 \times 10^{-3}\sin(200t)$$

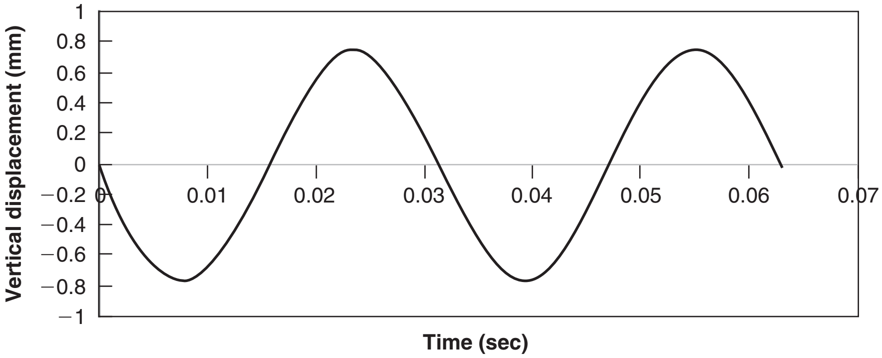



**Figure 20.5s** Vertical displacement signal.

**Problem 20.5**

The vibratory roller from problem 4 rests on a soil that has a stiffness $k_s$ to be determined. The damping coefficient of the soil is 200 kN s/m. Calculate the stiffness of the soil $k_s$, the modulus of the soil $E$, and the width $b$ of the contact area.

**Solution 20.5 (Figures 20.6s, 20.7s)**

$W_{\text{roller}} = 140$ kN
Drum radius, R = 0.7 m
Drum length, L = 2.1 m
$m_e r_e = 1.5$ kg.m
$\omega = 200$ rad/sec
$W_{\text{drum}} = 30$ kN
$c_s = 200$ kN s/m

Poisson's ratio, $\nu = 0.35$ (assumed)
The equations are:

$$\ddot{x}_d = \pm 3g\sin(\omega t)$$

$$\dot{x}_d = \mp\frac{3g}{\omega}\cos(\omega t)$$

$$x_d = \pm\frac{3g}{\omega^2}\sin(\omega t)$$

$$F = -m_d\ddot{x}_d + m_e r_e\omega^2\cos(\omega t) + (m_f + m_d)g$$

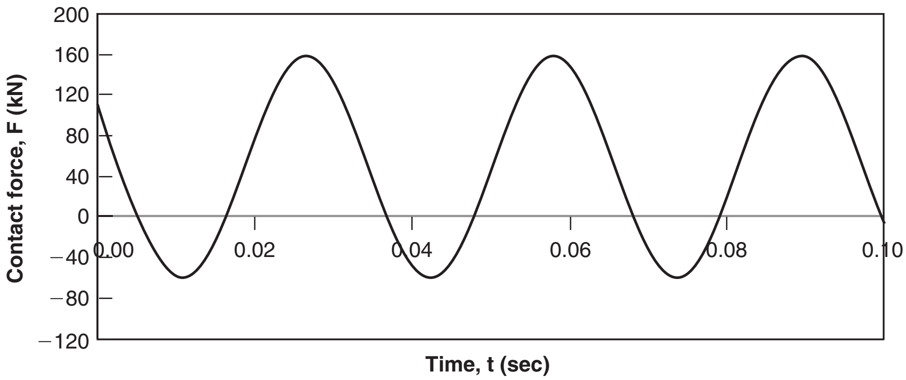



**Figure 20.6s** Contact force versus time.

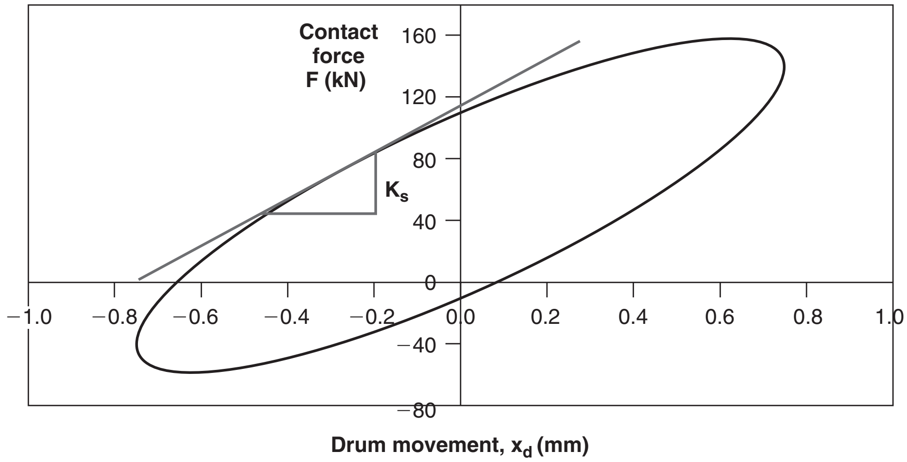



**Figure 20.7s** Contact force versus movement and soil stiffness.

The stiffness can be taken as the slope as shown in Figure 20.7s, or as the slope of the loop:

$$k_s = \frac{160\ \text{kN}}{1.06\ \text{mm}} = 150.9\ \text{kN/mm}$$
$$k_s = 150900\ \text{kN/m}$$

Then, the soil modulus is:

$$k_s = \frac{F_s}{x_d} = \frac{\pi L E}{2(1-\upsilon^2)\left(2.14 + \frac{1}{2}Ln\left[\frac{\pi L^3 E}{16\left(1-\upsilon^2\right)(m_f + m_d)Rg}\right]\right)}$$

$$k_s = \frac{\pi \times 2.1 \times E}{2(1-0.35^2)\left(2.14 + \frac{1}{2}Ln\left[\frac{\pi \times 2.1^3 \times E}{16\left(1-0.35^2\right) \times \left(\frac{20}{g} + \frac{30}{g}\right) 0.7 \times g}\right]\right)}$$

$$k_s = \frac{6.6 \times E}{1.755 \times \left(2.14 + \frac{1}{2}Ln\left[\frac{29.1 \times E}{491.4}\right]\right)}$$

$$40125.7 = \frac{E}{\left(2.14 + \frac{1}{2}Ln\left[\frac{29.1 \times E}{491.4}\right]\right)}$$

and solving for $E$ gives $E = 281$ MPa.

To compute the contact width b between the roller and the soil at the time of the highest force, $F = 160$ kN, we use the following equation:

$$b = \sqrt{\frac{16}{\pi} \frac{R(1-\upsilon)}{E} \frac{F}{L}}$$

$$b = \sqrt{\frac{16}{\pi} \times \frac{0.7(1-0.35)}{280900} \times \frac{160}{2.1}} = 0.025 \text{ m}$$

$$b = 25 \text{ mm}$$

**Problem 20.6**

A landfill must be compacted by dynamic compaction to improve its bearing capacity. The required depth of compaction is 10 m. Determine the weight of the tamper to be used and the drop height required to achieve the 10 m depth of compaction.

**Solution 20.6**

The following equation is used to evaluate the depth of compaction, $D$:

$$D = \alpha\sqrt{MH}$$

From the problem statement, $D = 10$ m. Alpha is typically between 0.3 and 0.8; let's assume 0.5:

$$D/\alpha = 10/0.5 = 20$$

$$20 = \sqrt{MH}$$

$$M = 20 \text{ tonnes}$$

$$H = 20 \text{ m}$$

**Problem 20.7**

Regarding the landfill in problem 6, the closest building is located 100 m from the edge of the compaction zone. Calculate the peak particle velocity that can be expected. Would this be normally tolerable for a recently constructed building?

**Solution 20.7**

The peak velocity of the soil particles (PPV) (in mm/s) caused by the dynamic vibration is calculated as:

$$PPV = 75\left(\frac{\sqrt{MH}}{d}\right)^{1.7}$$

Where $M$ is the mass of the tamper in tonnes, $H$ is the drop height in meters, and $d$ is the distance from the impact zone in meters. From problem 20.6, $M = 20$ tonnes and $H = 20$ m. Then, with $d = 100$ m, the PPV is:

$$PPV = 75\left(\frac{\sqrt{MH}}{d}\right)^{1.7} = 75 \times \left(\frac{20}{100}\right)^{1.7} = 4.86 \text{ mm/s}$$

The damage threshold for modern buildings is set to be $20 \sim 50$ mm/s; therefore, this PPV is tolerable for a recent building located 100 m away from the closest edge of the compaction zone.