Developments in Geotechnical Engineering

Sanjay Kumar Shukla

Fundamentals of Fibre-Reinforced Soil Engineering

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Sanjay Kumar Shukla
Discipline of Civil and Environmental Engineering
School of Engineering, Edith Cowan University
Perth, WA, Australia

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Preface

In the current construction practice, reinforcing the soil is an effective and reliable ground improvement technique for increasing the strength and stability of the soil in various applications, including retaining structures, embankments, foundations, slopes and pavements. The concept of reinforcing the soil with natural materials was originated in ancient times; however, the galvanized steel strips, having a high tensile modulus, are the earliest modern form of soil reinforcement developed in 1966 in France. Later, the use of polymeric products, called the *geosynthetics*, started as the soil reinforcement along with several other applications of specific geosynthetics to achieve different functions as separation, filtration, drainage, fluid barrier and protection. In the noncritical structures, natural products, called the *geonaturals*, are also used as the soil reinforcement. Unlike the metal strips, in general, the geosynthetic and geonatural reinforcements have a much lower tensile modulus. Geosynthetic reinforcements (woven geotextiles, geogrids, some geocomposites, etc.) as well as geonatural reinforcements (bamboo, geocoir, geojute, etc.) are generally used in the form of flexible sheets/mats/meshes. The subject of geosynthetics and geonaturals and their applications are called the *geosynthetic engineering*, and there are some textbooks and reference books available on this subject.

Reinforcing the soil with flexible, discrete fibres is not a new technique in civil/geotechnical engineering. However, as the fibre inclusions bring several technical, economic and environmental benefits, in recent years, a great deal of interest has been created worldwide on the potential applications of fibres within the soils and other similar materials, such as coal ashes and mine tailings. Fibres are generally available in large amounts in natural and waste forms. In many countries, waste fibres (plastic waste fibres, old tyre fibres, etc.) have been creating disposal and environmental problems. Utilization of these fibres in constructions can solve the disposal problems in a cost-effective and environmentally friendly manner.

Over the past 30–35 years, the laboratory and field research studies have shown that the use of natural, synthetic and waste fibres as a tension-resisting element and/or an admixture causes significant modification and improvement in the engineering properties (strength, stiffness, permeability, compressibility, etc.) of soils and other similar materials. The soil reinforced randomly with short, discrete fibres is basically a composite material and is called the *randomly distributed fibre-reinforced soil*, or simply the *fibre-reinforced soil*. The studies indicate that a fibre-reinforced soil exhibits greater extensibility and a smaller loss of post-peak strength; that is, compared to soil alone, the fibre-reinforced soil is more ductile. Soils, especially cohesionless soils, can also be reinforced by the continuous fibres/yarns. In this reinforcing system, a single monofilament is spun or injected in a random pattern simultaneously with the deposition of soil in a specific application.

This book presents the fundamentals of the fibre-reinforced soils within five chapters as an engineering subject, called the *fibre-reinforced soil engineering*. No complete book is currently available on this subject. The book is primarily designed and developed as a textbook as well as a student-centred learning resource for a one-semester course for senior undergraduate and postgraduate students as a part of a geotechnical/civil engineering programme. This course may be offered to students as an elective in the universities/institutes/colleges. The material in all the chapters of this book is presented clearly in simple and plain English and includes the optimum amount of text, illustrations, tables, examples and questions for practice. Each chapter includes many useful references, quoted in text and listed at the end of the chapter, for further study. As the practical solution to an engineering problem often requires the application of engineering judgement and experience, which can be acquired by regular professional practice and self-study, an attempt has been made to provide the practical experience, including the field application guidelines and some case studies. The chapter summary presented at the end of each chapter may help the readers in getting some key learning points easily. Through this textbook, the readers can learn the subject without any major assistance, and some readers can learn the subject even by self-reading only. Apart from students, researchers and teachers, this textbook will be a valuable learning resource for the practising engineers dealing with utilization of fibres in constructions and infrastructure developments worldwide.

For a better learning of the concept of fibre-reinforced soils, it is important to have an understanding of the basic soil properties and core principles of soil mechanics, as presented in Chap. 1, along with the basic description of soil reinforcement. Chapter 2 provides the basic details of fibre-reinforced soils, focusing on fibres and their types, and phase concept along with a brief introduction to the soil reinforced with continuous fibres and multioriented inclusions. Chapter 3 deals with the engineering behaviour of fibre-reinforced soils as reported by various researchers based on their experimental investigations and analyses of test results. Chapter 4 focuses on presenting the reinforcing mechanisms, the models of fibre-reinforced soils and findings of some numerical studies. Chapter 5 covers the details of field applications of fibre-reinforced soils, emphasizing on analysis and design concepts, and field application experience and guidelines. The key research developments have been included as required throughout the book.

I would like to thank Swati Meherishi, senior publishing editor, Aparajita Singh, RaagaiPriya ChandraSekaran, Rajeswari Sathiamoorthy and other staff at Springer

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I wish to extend sincere appreciation to my wife, Sharmila, for her encouragement and support throughout the preparation of the manuscript. I would also like to thank my daughter, Sakshi, and my son, Sarthak, for their patience during my work on this textbook at home.

Finally, I welcome suggestions from the readers and the users of this textbook for improving its content in future editions.

Perth, 2017 Sanjay Kumar Shukla

Contents

About the Author

Dr Sanjay Kumar Shukla is the founding editor-in-chief of *International Journal of Geosynthetics and Ground Engineering* (Springer International Publishing). He is an associate professor and programme leader of civil and environmental engineering at the School of Engineering, Edith Cowan University, Perth, Australia. He is also an adjunct professor at the School of Civil and Chemical Engineering, VIT University, Vellore, India, and a Distinguished Professor of Civil Engineering at the Institute of Engineering and Technology, Chitkara University, Himachal Pradesh, India. He graduated in 1988 with a first-class degree with distinction in civil engineering from BIT Sindri (Ranchi University, Ranchi), India, and earned his MTech in civil engineering (engineering geology) in 1992 and PhD in civil engineering (geotechnical engineering) in 1995 from the Indian Institute of Technology Kanpur, India. He has over 20 years of teaching, research and consultancy experience in the field of Civil (Geotechnical) Engineering. He is the author of ICE textbooks titled *Core Principles of Soil Mechanics* and *Core Concepts of Geotechnical Engineering*, and has authored/co-authored/edited other 7 books and 12 book chapters. He is also the author/co-author of more than 160 research papers and technical articles, including over 100 refereed journal publications. He is a fellow of Engineers Australia, Institution of Engineers (India) and Indian Geotechnical Society and a member of the American Society of Civil Engineers (ASCE), International Geosynthetics Society and Indian Roads Congress. He serves on the editorial boards of the *International Journal of Geotechnical Engineering*, *Ground Improvement*, *Geotechnical Research*, *Indian Geotechnical Journal*, *Cogent Engineering* and *Advances in Civil Engineering*, and he is the scientific editor of the *Journal of Mountain Science*.

Chapter 1
Basic Description of Soil and Soil Reinforcement

1.1 Introduction

For a better learning of the fundamentals of fibre-reinforced soil engineering through this book, as presented in later chapters, basic characteristics of soil, core principles of soil mechanics and an introduction to the soil reinforcement are presented briefly in this chapter. For more details about the soil and its mechanics, the readers can refer to the textbook titled *Core Principles of Soil Mechanics* by Shukla (2014). This chapter also provides the list of commonly used tests, which are generally conducted for investigating the engineering behaviour of fibre-reinforced soils and developing the models to predict their behaviour. At the end, the subject of fibres and their applications, called the 'fibre-reinforced soil engineering', is defined with its importance.

1.2 Soil and Its Phases

Soil comprises all the materials in the surface layer of the Earth's crust that are loose enough to be moved by a spade or shovel. According to Karl Terzaghi, soil is a natural aggregate of mineral particles that can be separated by such gentle means as agitation in water (Terzaghi et al. 1996).

Soil is a particulate and multiphase system consisting of, in general, three phases, namely, solid, liquid and gas (Fig. 1.1). The space in a soil mass occupied by liquid and/or gas is known as the void. A dry soil has air only in the void, while the void volume of a fully saturated soil is occupied by water only. There are several phase relationships and interrelationships; the most common ones are given below, where V and W refer to volume and weight, respectively, and subscripts a, w, s and v denote air, water, solid and void, respectively:

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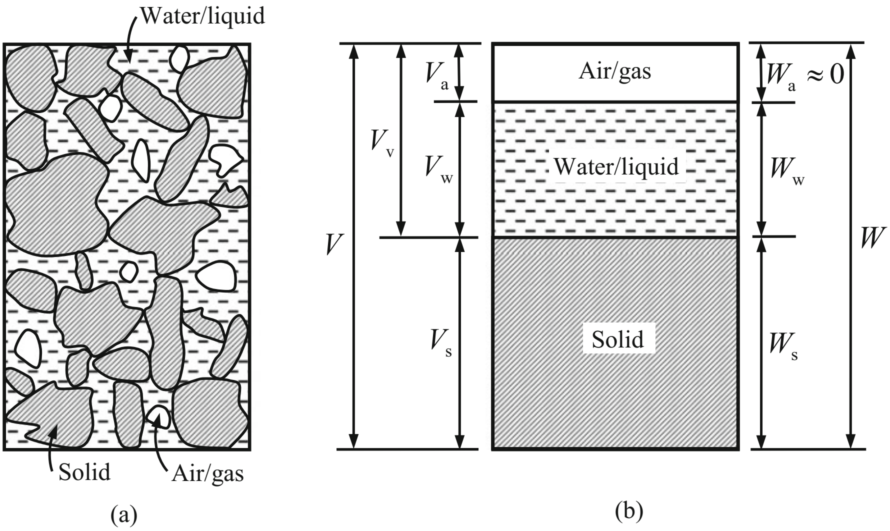

Fig. 1.1 An element of soil mass: (**a**) phases in the natural state, (**b**) phases separated for mathematical analysis

$$n = \frac{V_v}{V} \tag{1.1}$$

$$e = \frac{V_v}{V_s} \tag{1.2}$$

$$S = \frac{V_w}{V_v} \tag{1.3}$$

$$w = \frac{W_w}{W_s} \tag{1.4}$$

$$\gamma = \frac{W}{V} \tag{1.5}$$

$$\gamma_d = \frac{W_s}{V} \tag{1.6}$$

$$\gamma_s = \frac{W_s}{V_s} \tag{1.7}$$

$$\gamma_{sat} = \frac{W_{sat}}{V} \tag{1.8}$$

$$\gamma' = \gamma_{sat} - \gamma_w \tag{1.9}$$

$$G = \frac{\gamma_s}{\gamma_w} \tag{1.10}$$

$$n = \frac{e}{1+e} \tag{1.11}$$

$$Se = wG \tag{1.12}$$

$$\gamma = \left(\frac{G+Se}{1+e}\right)\gamma_{\mathrm{w}} = \left(\frac{1+w}{1+e}\right)G\gamma_{\mathrm{w}} \tag{1.13}$$

$$\gamma_d = \frac{G\gamma_{\mathrm{w}}}{1+e} = \frac{G\gamma_{\mathrm{w}}}{1+\frac{wG}{S}} = \frac{\gamma}{1+w} \tag{1.14}$$

where n is the porosity of soil, e is the void ratio of soil, S is the degree of saturation of soil, w is the water content of soil, γ is the total/bulk/wet/moist unit weight (or simply unit weight) of soil, γ_{d} is the dry unit weight of soil, γ_{s} is the unit weight of soil solids, γ_{sat} is the saturated unit weight of soil, γ_{w} is the unit weight of water (=9.81 kN/m^3), γ' is the submerged/buoyant unit weight of soil, G is the specific gravity of soil solids (i.e. soil particles or mineral matter) and W_{sat} is the saturated weight of soil. The values of n, S and w are normally expressed as a percentage, while the value of e is expressed as a decimal.

Note that mass M and weight W, having SI units as kilogramme (kg) and newton (N), respectively, of a soil element are related as

$$W = Mg \tag{1.15}$$

where g is the acceleration due to gravity, which is taken as 9.81 m/s^2 or sometimes approximately as 10 m/s^2 for simplicity. In most geotechnical engineering applications at various locations of the Earth, weight is preferred.

The total density (or simply density) ρ of a soil element is defined as

$$\rho = \frac{M}{V} \tag{1.16}$$

where V is the volume of the soil element.

The unit weight γ and density ρ of a soil element are related as

$$\gamma = \rho g \tag{1.17}$$

For each of the unit weights (dry unit weight γ_{d}, unit weight of solids γ_{s}, saturated unit weight γ_{sat}, submerged unit weight γ'), you may define a corresponding density (dry density ρ_{d}, density of solids ρ_{s}, saturated density ρ_{s} or submerged density ρ') of the soil element as the total density is defined in Eq. (1.16). You may also write relationships for other unit weights and densities as done in Eq. (1.17), for example, $\gamma_{\mathrm{d}} = \rho_{\mathrm{d}}g$.

Example 1.1

A cubical soil sample of 200-mm edge length is collected from the base level of a proposed shallow foundation. Its mass is found to be 16.1 kg. Determine the total unit weight of the soil.

Solution
From Eq. (1.15), the weight of soil sample,

$$W = Mg = (16.1)(9.81) = 157.9 \text{ N}$$

Since the soil sample is cubical, its volume,

$$V = (0.2)^3 = 0.008 \text{ m}^3$$

From Eq. (1.5), the total unit weight,

$$\gamma = \frac{157.9}{0.008} = 19737.5 \text{ N/m}^3 \approx \mathbf{19.74 \text{ kN/m}^3}$$

Example 1.2
An in situ soil has a total unit weight of 18.75 kN/m^3 and a water content of 28%. Determine the following:

(a) Void ratio
(b) Degree of saturation

Assume that the specific gravity of soil solids is 2.68.

Solution

(a) From Eq. (1.13), the total unit weight,

$$\gamma = \left(\frac{1+w}{1+e}\right) G\gamma_w$$

or, the void ratio,

$$e = (1+w)\left(\frac{G\gamma_w}{\gamma}\right) - 1 = (1+0.28)\left(\frac{2.68 \times 9.81}{18.75}\right) - 1 = \mathbf{0.79}$$

(b) From Eq. (1.12), the degree of saturation,

$$S = \frac{wG}{e} = \frac{(0.28)(2.68)}{0.79} = \mathbf{0.95} \text{ or } \mathbf{95\%}$$

1.3 Soil Properties and Core Principles of Soil Mechanics

Properties of soils are highly variable from one site to another, and even at a single site, they can vary with locations. The air content of a soil has little engineering significance, while the water content influences the engineering properties of soil significantly. The design and stability of geotechnical structures are significantly governed by the properties of soil, mainly permeability, compressibility and shear strength. The subject dealing with the physical properties of soil and the behaviour of soil masses in connection with their practical applications is known as the soil mechanics. Most properties of soils are listed below:

1. *Index/basic properties*: total unit weight (γ), void ratio (e), specific gravity of soil solids (G), water content (w), degree of saturation (S), particle-size distribution, consistency limits [liquid limit (w_L), plastic limit (w_P), shrinkage limit (w_S)] for cohesive soils and relative density (D_r) for cohesionless soils
2. *Compaction characteristics*: maximum dry unit weight (γ_{dmax}) and optimum water content (w_{opt})
3. *Permeability*: coefficient of permeability or hydraulic conductivity (k)
4. *Compressibility*: compression index (C_c), recompression index (C_r), swelling index (C_s), coefficient of volume change (m_v), coefficient of consolidation (c_v) and secondary compression index (C_α)
5. *Shear strength*: total stress-strength parameters as cohesion intercept (c) and angle of shearing resistance (ϕ) and effective stress-strength parameters as effective cohesion intercept (c') and effective angle of shearing resistance (ϕ')
6. *Stiffness*: elastic constants, such as Young's modulus of elasticity (E), shear modulus of elasticity (G), bulk modulus of elasticity (K) and Poisson's ratio (μ)

The core concepts of soil mechanics are briefly described below (Shukla 2014):

- The term ground refers to soil, rock and/or fill in place prior to the execution of the construction projects. Based on the method of formation, soils are classified as residual soils, sedimentary soils, organic soils and fills (or man-made soils).
- The smallest particle size which can be seen with naked eye is typically 0.075 mm. Clay (< 0.002 mm), silt (0.002–0.075 mm), sand (0.075–4.75 mm), gravel (4.75–80 mm), cobble (80–300 mm) and boulder (>300 mm) are the names of particle sizes, and they are also often used to describe soils. The dividing lines between the size limits are arbitrary and vary with different classification systems. Silt-size and clay-size particles are collectively called the fines. A fine-grained soil has fines 50% or more (by dry weight) of soil, while the coarse-grained soil has fines less than 50% (by dry weight) of soil.
- The classification of a soil requires the particle-size distribution based on sieve analysis and consistency limits (liquid and plastic limits). If a soil is described as a silty clay, then the clay content is greater than the silt content in the soil.
- The soil is called a well-graded soil if the distribution of the particle sizes extends over a large range. The soil consisting of particles of almost one size

is called the uniformly graded soil. If a soil has an excess of certain particle sizes and a deficiency of other sizes, then the soil is called the poorly graded soil.

- Soil particles and water are almost incompressible, so any volume change in a saturated soil is equal to the volume of water that drains out of or into the soil.
- For a dry soil, the degree of saturation, $S = 0\%$, and for a fully saturated soil, $S = 100\%$. For a partially saturated soil, S lies between 0 and 100%. The natural water content w of soils can exceed 100% although it is well under 100% for most soils.
- Typical values of saturated unit weight of some common soils are as follows: 19–24 kN/m^3 for sands and gravels, 14–21 kN/m^3 for silts and clays and 10–11 kN/m^3 for peats.
- In the absence of measured values, it is a common practice to assume $G = 2.65$ for sand and $G = 2.70$ for clay.
- The difference, $w_L - w_P$, is called the plasticity index (I_P), which is used in strength correlations and for estimating some compressibility parameters.
- If the natural water content (w) of a soil is close to its liquid limit (w_L), the soil is normally consolidated, while for a medium to heavily overconsolidated soil, w is close to its plastic limit w_P (Bowles 1996).
- The consistency of a cohesionless/granular soil is generally described in terms of relative density defined as

$$D_r = \left(\frac{e_{max} - e}{e_{max} - e_{min}}\right) \times 100\% \tag{1.18}$$

or

$$D_r = \left(\frac{\gamma_d - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}}\right)\left(\frac{\gamma_{d\,max}}{\gamma_d}\right) \times 100\% \tag{1.19}$$

where e is the in situ (or in-place) void ratio of soil; e_{min} is the minimum void ratio, i.e. void ratio in the densest possible state of soil; e_{max} is the maximum void ratio, i.e. void ratio in the loosest possible state of soil; γ_d is the in situ (or in-place) dry unit weight of soil; $\gamma_{d\,min}$ is the minimum dry unit weight, i.e. dry unit weight in the loosest possible state of soil; and $\gamma_{d\,max}$ is the maximum dry unit weight, i.e. dry unit weight in the densest possible state of soil.

- The consistency of the coarse-grained soil is described as very loose ($0\% < D_r < 15\%$), loose ($15\% < D_r < 35\%$), medium ($35\% < D_r < 65\%$), dense ($65\% < D_r < 85\%$) and very dense ($85\% < D_r < 100\%$).

- The total vertical stress at a depth z from the ground surface can be computed as

$$\sigma_v = \gamma z \tag{1.20}$$

where γ is the total unit weight of the soil.

- Within a saturated soil mass, the effective vertical stress σ'_{v} at any depth is equal to the total vertical stress σ_{v} minus the pore water pressure u at that depth. Thus,

$$\sigma'_{\mathrm{v}} = \sigma_{\mathrm{v}} - u \tag{1.21}$$

  Equation (1.21) states Terzaghi's effective stress principle.

- For water table below the ground surface, a rise in the water table causes a reduction in the effective stress at any point within the soil. For water table above the ground surface, a fluctuation in the water table does not alter the effective stress at any point within the soil.
- The permeability of a soil indicates its ability to conduct fluid, and it depends on the characteristics of both the soil and the permeant. The flow through a soil mass may be estimated using Darcy's law, which is stated as

$$v = ki \tag{1.22}$$

where v is the flow or discharge velocity of water through an element of the soil mass, $i = \Delta h/L$ is the hydraulic gradient causing the flow, Δh being the hydraulic head causing the flow, L is the length of soil element in the direction of the flow, and k is the coefficient of permeability or hydraulic conductivity of soil. Equation (1.22) holds good for most soils, especially finer than coarse sands, in which the flow can be laminar. The permeability of sands ranges from 10^{-2} to 10^{-5} m/s while that of clays is equal to or less than 10^{-9} m/s.

- As water can only flow through the voids in the cross section of the soil mass, the average effective velocity of flow, called the seepage velocity v_{s}, is greater than the discharge velocity v. The value of v_{s} can be determined by using the following relationship:

$$v_{\mathrm{s}} = \frac{v}{n} = \frac{ki}{n} \tag{1.23}$$

  where n is the porosity of soil.

- Above the water table, the pore water pressure is negative due to surface tension, and so there is suction, which can be as high as 6 kPa in fine sand and 600 kPa in clay.
- When water flows through a soil, it exerts force, called the seepage force, on the soil particles through friction drag. In an isotropic soil, the seepage force always acts in the direction of flow. If h is the loss in total head through friction drag over length L of the soil and A is the area of cross-section through which water flows, then the seepage force is

$$J = \gamma_{\mathrm{w}} hA = i\gamma_{\mathrm{w}}(AL) = i\gamma_{\mathrm{w}} V \tag{1.24}$$

  where γ_{w} is the unit weight of water and V is the total volume of the soil mass.

- The seepage force per unit volume of soil, called the seepage pressure, is

$$j = \frac{J}{V} = i\gamma_{\mathrm{w}} \tag{1.25}$$

- As an upward flow of water through a soil mass causes a decrease in effective stress, there is a possibility of zero effective stress within the soil mass at a certain hydraulic gradient, known as the critical hydraulic gradient i_{c} , which can be calculated as

$$i_{\mathrm{c}} = \frac{G - 1}{1 + e} \tag{1.26}$$

where G is the specific gravity of soil solids and e is the void ratio of soil.

- Under the influence of seepage, which can take place in the downstream of Earth dams and other water-retaining structures in the upward direction, the foundation soil particles can continuously move in the seepage direction, if the hydraulic gradient exceeds the critical hydraulic gradient. This phenomenon of soil movement is known as the *soil piping*. *Piping resistance* of soil acts in the direction opposite to seepage force and is equal to the seepage force at which soil particles start moving due to the seepage of water.
- The factor of safety against piping, FS_{piping}, is defined as

$$FS_{\mathrm{piping}} = \frac{i_{\mathrm{c}}}{i_{\mathrm{e}}} \tag{1.27}$$

where i_{e} is the maximum exit hydraulic gradient, that is, the maximum hydraulic gradient near the discharge boundary surface. For no piping, $FS_{\mathrm{piping}} > 1$.

- Compaction refers to the volume reduction of an unsaturated soil mass due to expulsion of air from its voids/pores caused by the external compressive load/stress application. Compaction differs from the consolidation, which is a time-dependent process of volume reduction of mainly a saturated soil mass due to expulsion of water from its voids/pores.
- Within a soil mass, the ratio of effective horizontal stress (σ'_{h}) to effective vertical stress (σ'_{v}), called the lateral stress ratio (K), is typically in the range of 0.2–0.5.
- The compressibility of a soil is a function of soil type/composition, effective stress and stress history. The compression index C_{c} is used to determine the magnitude of primary consolidation settlement of a normally consolidated soil. The coefficient of consolidation c_{v} is used to determine the rate of settlement during the primary consolidation.
- The ratio of maximum past effective vertical stress σ'_{vmax} to the present effective vertical stress (σ'_{v0}) is called the *overconsolidation ratio* (OCR), which is used to express the degree of overconsolidation (or stress history) of soil. Thus,

$$OCR = \frac{\sigma'_{\mathrm{vmax}}}{\sigma'_{\mathrm{v}0}} \tag{1.28}$$

For normally consolidated soils, $OCR = 1$, and for overconsolidated soils, $OCR > 1$.

- The maximum internal resistance per unit area of a soil to applied shear stress is called its shear strength. A soil is rarely required to resist tension, and generally fails in shear even though the applied load is compressive. Therefore, the analysis of strength of a soil is basically a problem of shear strength. The shear strength τ_{f} appears as the shear stress on the failure plane within the soil mass at failure, and its SI unit is N/m^2 or pascal (Pa). It is commonly expressed as the Mohr-Coulomb failure criterion, which is stated as

$$\tau_{\mathrm{f}} = c' + \sigma'_{\mathrm{f}} \tan \phi' \tag{1.29}$$

where σ'_{f} is the effective normal stress on the failure plane at failure, c' is called the effective cohesion (also known as the effective cohesion intercept) and ϕ' is called the effective angle of internal friction (also known as the effective angle of shearing resistance, or simply effective friction angle).

- Since a coarse-grained soil has almost no cohesion, its shear strength depends mainly on the internal friction between the particles. Such soils are called cohesionless, granular, frictional or free-draining soils. A fine-grained soil consists of a significant amount of silt-size and clay-size particles, and therefore its shear strength depends on cohesion as well as internal friction between the particles. Such soils are called cohesive or cohesive-frictional soils depending on the relative significance of cohesion and internal friction between the particles.
- The triaxial compression test is the most versatile test for studying strength and stiffness (stress-strain relationship) properties of a soil. A triaxial test has the following three types: consolidated-drained (CD), consolidated-undrained (CU) and unconsolidated-undrained (UU) tests, depending on the loading conditions simulated as per the field requirements. The stress-strain behaviour of loose sand is similar to that of normally consolidated clay, whereas dense sand behaves similar to overconsolidated clay. A clay subjected to undrained/quick loading or unloading with a constant volume behaves as a purely cohesive material which has angle of shearing resistance equal to zero, and therefore the shear strength, called the undrained shear strength, is

$$\tau_{\mathrm{f}} = c_{\mathrm{u}} = \frac{q_{\mathrm{u}}}{2} \tag{1.30}$$

where c_{u} is the undrained cohesion and q_{u} is the unconfined compressive strength of soil.

- The shear strength of a cohesive soil increases with time from $\tau_f = c_u$ under undrained loading to $\tau_f = c' + \sigma'_f \tan\phi'$ under drained loading as the pore water escapes from the voids, resulting in a decrease in pore water pressure. There is a possibility of a decrease in shear strength of soil as a result of unloading by excavation, an increase in pore water pressure caused by changes in groundwater condition or in seepage pressure, or softening of fissures/cracks present in stiff clays. Shear strength is generally computed for the most critical condition which usually exists immediately upon load application or immediately after construction as an undrained loading (Teng 1962).
- A granular soil is generally an excellent foundation material and the best embankment and backfill material, while a clayey soil is a very poor material for foundation, embankment and backfill. Clay is nearly watertight because of its low permeability, and therefore, it is the best soil material for construction of impervious layers/liners/barriers for ponds and landfills.
- A cohesive soil generally loses a portion of its shear strength upon remoulding/disturbance. If the unconfined compression test is conducted on both the undisturbed and remoulded specimens of the same cohesive soil at an unaltered water content, the effect of remoulding/disturbance, that is, the amount of strength loss, may be expressed in terms of *sensitivity* S_t, defined as

$$S_t = \frac{q_{u(\text{undisturbed})}}{q_{u(\text{remoulded})}} = \frac{c_{u(\text{undisturbed})}}{c_{u(\text{remoulded})}} \quad (1.31)$$

Based on the sensitivity, clays are classified as insensitive ($S_t = 1$), low sensitive ($S_t = 1 - 2$), medium sensitive ($S_t = 2 - 4$), highly sensitive ($S_t = 4 - 8$), extra sensitive ($S_t = 8 - 16$) or quick ($S_t > 16$).

- Swelling and shrinkage characteristics of highly plastic clays due to the presence of mainly montmorillonite mineral cause damages to foundations, pavements and other structures.
- An organic soil has a spongy structure, low shear strength, high compressibility, acidity and injurious characteristics to construction materials, and therefore, it is not a suitable foundation material. It undergoes creep (continued compressions at constant effective stress), which contributes significantly to long-term settlement.
- An unsaturated soil gains a part of its strength from the suction of capillary water within the voids, but this strength is lost when the soil becomes saturated, thus resulting in collapse of soil structure with a significant deformation/settlement.
- A retaining wall is usually constructed to support a soil, rock or any other soil-like material (coal ash, mine tailing, etc.) that cannot remain stable with a vertical or sloping face. The stability of a retaining wall or other similar structure (abutment, basement wall, sheet pile wall, etc.) depends on the amount of lateral earth pressure from the soil, called the backfill, supported by the wall.
- The stability of a soil slope is significantly governed by the shear strength of the soil.

- The term 'foundation' refers to the load-carrying structural member of an engineering system (e.g. building, bridges, road, runway, dam, tower, pipeline or machine) constructed on or below the ground surface as well as the soil/rock mass that finally supports the loads from the engineering system. The load per unit area at the base level of the structural part of the foundation (often called the footing) that causes the shear failure to occur in the soil is termed the *ultimate bearing capacity* or simply the *bearing capacity* of the foundation. The bearing capacity of a foundation greatly depends on the shear strength of the foundation soil in addition to other factors such as shape and size of the foundation, depth of foundation, water table location and load inclination to the vertical.

Example 1.3
A soil deposit has a void ratio of 0.85. If the void ratio is reduced to 0.60 by compaction, determine the percentage volume reduction due to compaction.

Solution
Within the soil deposit, consider a soil element with V as its total volume, V_v as the void volume and V_s as the solid volume. Given, initial void ratio, $e = V_v/V_s = 0.85$; and void ratio after compaction, $e_1 = V_{v1}/V_s = 0.60$, V_{v1} being the void volume after compaction.

If p is the percentage volume reduction due to compaction, then

$$p = \left(\frac{V_v - V_{v1}}{V}\right) \times 100\%$$

Considering the basic definitions of phase relationships and substituting the values,

$$p = \left(\frac{V_v - V_{v1}}{V}\right) \times 100\% = \left(\frac{V_v - V_{v1}}{V_v + V_s}\right) \times 100\%$$
$$= \left(\frac{V_v/V_s - V_{v1}/V_s}{V_v/V_s + 1}\right) \times 100\% = \left(\frac{e - e_1}{e + 1}\right) \times 100\%$$

or

$$p = \left(\frac{0.85 - 0.60}{0.85 + 1}\right) \times 100\% = \mathbf{13.5\%}$$

Example 1.4
If the unconfined compressive strength of saturated clay is 40 kPa, what will be its undrained shear strength? Can you describe the consistency of this soil?

Solution
Given, unconfined compressive strength, $q_u = 40$ kPa. From Eq. (1.30), the undrained shear strength is

$$c_u = \frac{q_u}{2} = \frac{40}{2} = \mathbf{20\ kPa}$$

As $12.5\ \text{kPa} < c_u < 25\ \text{kPa}$, the soil is **soft**.

Note: The following are the descriptive terms for consistency of cohesive soils:

Very soft ($c_u < 12.5$ kPa), soft ($12.5\ \text{kPa} < c_u < 25$ kPa), firm ($25\ \text{kPa} < c_u < 50$ kPa), stiff ($50\ \text{kPa} < c_u < 100$ kPa), very stiff ($100\ \text{kPa} < c_u < 200$ kPa) and hard ($c_u > 200$ kPa).

1.4 Soil Reinforcement

Soil supports the structural foundations, and it is used as a construction material in various civil/geotechnical engineering projects. Soil and other similar materials, such as fly ash, mine tailings, etc., are strong in compression but weak in tension. In the current construction practice in civil and environmental engineering as well as in some mining, agricultural and aquacultural engineering projects, various types and forms of reinforcement materials are introduced to improve the engineering properties (e.g. strength, stiffness, permeability, compressibility, etc.) of soil and other similar materials, thereby resulting in relatively a new material, called the *reinforced soil*. Thus a reinforced soil is a composite material in which tension-resisting elements are embedded in a soil mass, which is weak in tension.

The concept of soil reinforcement is not new as soils have been reinforced since prehistoric times. In nature, even some insects, birds and animals use soil reinforcement in suitable forms for creating different structures as they need to meet their requirements. However, the modern form of soil reinforcement was first developed by Vidal (1966, 1969) in the form of steel/metal strips, and the reinforced soil was patented in the name of *Reinforced Earth* (Schlosser and Long 1974). The metal strips are available with smooth or ribbed surfaces. To avoid the corrosion problem, the steel strips are often galvanized suitably to maintain their designed life within the soil mass.

In most applications, soil reinforcement is often used nowadays in the form of continuous geosynthetic or geonatural reinforcement inclusions (e.g. sheets, nets, meshes, grids, strips or bars) within the soil mass in a definite pattern, resulting in the *systematically reinforced soil*. For more details on such systematically reinforced soils, called the *geosynthetic-reinforced soils* (ply soils by McGown and Andrawes 1977), which are studied under the subject of *geosynthetic engineering*, the readers can refer to the books by Shukla (2002, 2006, 2012, 2016) and Koerner (2012). Note that the geosynthetic or geonatural reinforcements as well as the metal strips are normally oriented in a preferred direction; thus, the reinforcement orientation is generally one dimensional and is installed sequentially in alternating layers as per the design requirements of the specific application. Figure 1.2 illustrates an example of the systematically reinforced soil. The geotextile-reinforced backfill helps reduce the lateral earth pressure significantly, thereby

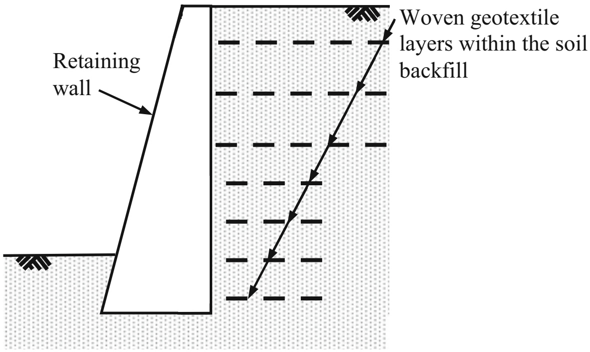

Fig. 1.2 A retaining wall supporting a systematically reinforced soil backfill

decreasing the thickness of the wall. With the geosynthetic-reinforced soil backfill, there is a possibility to completely avoid the construction of a thick wall by providing a thin skin face to mainly prevent the surface soil erosion.

With respect to stiffness (modulus of elasticity), steel and fibreglass reinforcements in the form of strips and bars having a high modulus of elasticity are often categorized as *ideally inextensible reinforcements*, while the geosynthetic and geonatural reinforcements, including plant roots, having relatively low modulus of elasticity, are considered *ideally extensible reinforcements*. The distinction between the two categories of soil reinforcement, ideally inextensible reinforcements and ideally extensible reinforcements, was explained by McGown et al. (1978) in terms of a ratio of reinforcement tensile (longitudinal) modulus of elasticity ($E_{\text{Reinforcement}}$) to average soil modulus of elasticity (E_{Soil}) as follows:

for ideally inextensible reinforcements,

$$\frac{E_{\text{Reinforcement}}}{E_{\text{Soil}}} > 3000 \tag{1.32}$$

and for ideally extensible reinforcements,

$$\frac{E_{\text{Reinforcement}}}{E_{\text{Soil}}} < 3000. \tag{1.33}$$

The difference between the influences of inextensible and extensible reinforcements is significant in terms of the stress-strain behaviour of the reinforced soil system, as noted in Fig. 1.3 (McGown et al. 1978). The soil reinforced with extensible reinforcement, termed *ply soil* by McGown and Andrawes (1977), has greater extensibility (ductility) and smaller loss of post-peak strength compared to soil alone or soil reinforced with inextensible reinforcement, termed *Reinforced Earth* by Vidal (1966, 1969). The inextensible reinforcements generally have rupture strains smaller than the maximum tensile strains in the unreinforced soil, thereby resulting in catastrophic failures, while the extensible reinforcements may have rupture strains larger than the maximum tensile strains in the unreinforced soil, thus they seldom rupture.

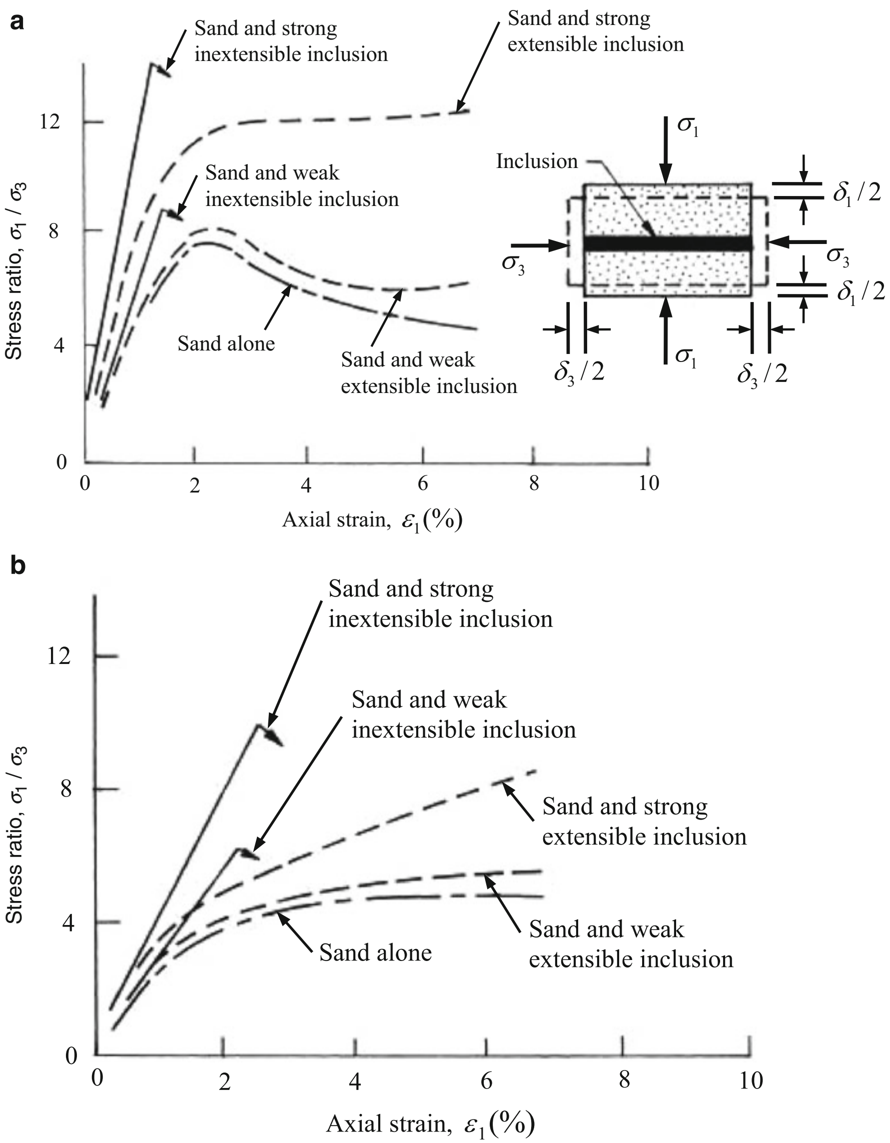

Fig. 1.3 Postulated behaviour of a unit cell in plane strain conditions with and without inclusions: (**a**) dense sand with inclusions, (**b**) loose sand with inclusions (Adapted from McGown et al. 1978)

In spite of some differences in the behaviour of inextensible and extensible reinforcements, a similarity between them exists in that both significantly strengthen the soil and inhibit the development of internal and boundary deformations of the soil mass by developing tensile stresses in the reinforcement. Thus, both forms of reinforcements are tensile strain inclusions.

Note that it is not always necessary to reinforce the soil with a reinforcement having a very high modulus as compared to the modulus of soil alone as some structures may be allowed to strain significantly without any functional failure.

In the past few decades (30–35 years), experimental and mathematical studies have been conducted to investigate the behaviour of soil reinforced randomly with different types of discrete, flexible, synthetic and natural fibres, called the *randomly distributed/oriented fibre-reinforced soils* (RDFRS), or simply the *fibre-reinforced soils* (FRS), with an intention of improving the soil strength and other engineering characteristics (Shukla et al. 2009; Shukla 2016). Within the fibre-reinforced soil, the fibre arrangement is basically two dimensional or three dimensional, depending on their placement or mixing. In field applications, the one-dimensional arrangement of fibres in any specific direction within the soil mass is a difficult task, and so no effort is made to consider this one-dimensional arrangement. In general, the reinforced soil is obtained by mixing the soil and fibres in such a way that the fibres are introduced into the soil mass randomly and homogeneously. The quality of mixing is very important to avoid any planes of weakness, such as those parallel to the oriented reinforcement, or even very small portions with insufficient fibre content. Figure 1.4 illustrates an example of the randomly distributed fibre-reinforced soil. The fibre-reinforced backfill also helps reduce the lateral earth pressure significantly, thereby decreasing the thickness of the wall, as the systematically reinforced soil backfill works.

The concept of reinforcing soil with fibres, especially the natural ones, originated in the ancient times in some applications. Civilizations in Mesopotamia added straws to mud bricks to provide integrity to a weak matrix by arresting the growth of cracks (Majundar 1975; Hoover et al. 1982). For example, the use of natural fibres in composite construction can be seen even today in the rural areas of some countries, such as India. In fact, the scientific interest in the fibre reinforcement of soils began in the 1970s with an attempt to estimate the influence of plant and tree roots on the stability of earth slopes (Waldron 1977). Subsequently, the randomly distributed fibre-reinforced soils have attracted increasing attention in geotechnical engineering for their field applications.

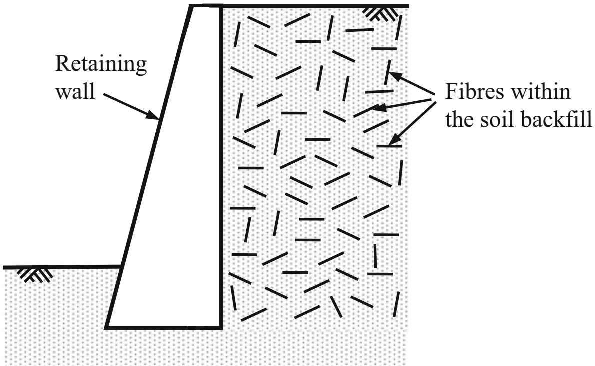

Fig. 1.4 A retaining wall supporting a randomly distributed fibre-reinforced soil backfill

Note that synthetic and natural fibres, including the plant roots, behave as extensible reinforcements as they have relatively a lower modulus of elasticity. Chapter 2 presents the basic description of fibre-reinforced soils, and their engineering behaviour, as studied by the researchers worldwide, is described in Chap. 3. You will notice that several laboratory tests and some field tests have been used to investigate the behaviour of fibre-reinforced soils. Some of these tests are listed below:

- Direct shear test
- Static and cyclic triaxial compression tests
- Unconfined compression test
- Compaction test
- Hydraulic conductivity test
- Consolidation test
- California bearing ratio (*CBR*) test
- Plate load test
- Brazilian indirect/splitting tensile strength test
- Durability test
- Ring shear test
- Torsional shear test
- Bender element test
- Resonant column test
- Flexural strength test
- Fibre pullout test

All these tests, which have originally been developed for assessing the characteristics of unreinforced/plain soils, are also conducted in accordance with relevant standards (e.g. ASTM standards), as applicable in the specific study/project. Direct shear, static triaxial compression and unconfined compression tests have been widely used to investigate the strength behaviour of fibre-reinforced soils, considering variations in several parameters, including the fibre characteristics, as discussed in Chap. 2. *CBR* and plate load tests are conducted to obtain the results for direct applications in the design of pavement layers and shallow foundations. Brazilian indirect/splitting tensile test is an indirect method of applying tension through splitting. The stresses developed during a splitting tensile test are analysed using the theory of elasticity. The original ring shear apparatus was developed by Bishop et al. (1971). The effect of fibre reinforcement at very large strains (higher than those possible in triaxial compression tests) is generally evaluated by ring shear tests. Bender elements test, introduced by Shirley and Hampton (1978), is commonly conducted for determining the elastic shear modulus of soil at very small strains. Bender element system is often set up in the triaxial compression test, but it can also be set up in other laboratory tests. The fibre-soil interface friction angle can be determined by fibre pullout test in a modified shear box apparatus.

Based on the observations in the experimental studies, attempts have been made by the researchers to present the models to predict their behaviour of fibre-reinforced soil, as described in Chap. 4.

In the current construction practice, discrete, flexible, synthetic and natural fibres have several field applications, as described in Chap. 5, in conjunction with soils and other similar materials, such as coal ashes, mine wastes, etc.

1.5 Fibre-Reinforced Soil Engineering

The subject of fibres and their applications in conjunction with soils and other similar materials, such as coal ashes, mine wastes, etc., is known as the 'fibre-reinforced soil engineering', which can be defined as follows:

> Fibre-reinforced soil engineering deals with the application of scientific principles and methods to the acquisition, interpretation and use of knowledge of fibre products for the solution to the problems in geotechnical, transportation, geoenvironmental and hydraulic engineering.

As the fibres are used in conjunction with soils and other similar materials, they are often called the *geofibres*. Throughout this book, the term fibres always refer to the geofibres.

This book presents the fundamentals of fibre-reinforced soil engineering, focusing on use of fibres in different areas of civil engineering as this subject has been defined. The soil can also be reinforced randomly with continuous fibres or multioriented (three dimensional, 3D) inclusions. Some details for these reinforced soils are also provided in Chap. 2. Note that some wastes, such as the municipal solid wastes, may be physically similar to randomly distributed fibre-reinforced soils, and, therefore, the concepts of fibre-reinforced soil engineering, as presented in this book, are equally applicable to such wastes to manage their disposal and utilization in engineering practice. Also note that characteristics of fibre-reinforced soils and their applications as described in this book may also be applicable to some mining, agricultural and aquacultural projects, which are very similar to some civil engineering projects.

Note that fibre reinforcement is not only alternative ground improvement technique, but it may also make several construction projects cost-effective as well as environmentally friendly. As fibres can be obtained from many waste products, such as old and used tyres, used plastic materials, etc., their utilization can greatly help solve waste disposal problems; otherwise, a significant volume of the landfills can be occupied by these wastes. Thus the fibre-reinforced soil engineering has become a subject of special importance in civil engineering and other related fields.

Chapter Summary

1. Soil is a particulate and multiphase system consisting of, in general, three phases, namely, solid, liquid and gas. There are several phase relationships and interrelationships.
2. The classification of a soil requires the particle-size distribution based on sieve analysis and consistency limits (liquid and plastic limits).

3. Soil particles and water are almost incompressible, so any volume change in a saturated soil is equal to the volume of water that drains out of or into the soil.
4. The design and stability of geotechnical structures are significantly governed by the properties of soil, mainly permeability, compressibility and shear strength.
5. A granular soil is generally an excellent foundation material and the best embankment and backfill material, while a clayey soil is a very poor material for foundation, embankment and backfill. Clay is nearly watertight because of its low permeability, and therefore, it is the best soil material for construction of impervious layers/liners/barriers for ponds and landfills.
6. The reinforced soil is a composite material that consists of soil mass (or other similar material) and some form of reinforcement material (inextensible such as steel strips or extensible such as geosynthetics or fibres), which provides an improvement in the engineering characteristics (e.g. strength, stiffness, permeability, compressibility, etc.) of soil.
7. The reinforcement can be included into a soil mass systematically or randomly, resulting in systematically reinforced soil (e.g. geosynthetic-reinforced soil) and randomly distributed fibre-reinforced soil, respectively.
8. The inextensible reinforcements generally have rupture strains smaller than the maximum tensile strains in the unreinforced soil, thereby resulting in catastrophic failures, while the extensible reinforcements may have rupture strains larger than the maximum tensile strains in the unreinforced soil, thus they seldom rupture.
9. The behaviour of fibre-reinforced soils is generally studied by conducting the tests developed for unreinforced soils with minor changes in some cases.
10. The subject of fibres and their applications in conjunction with soils and other similar materials, such as coal ashes, mine wastes, etc., is known as the 'fibre-reinforced soil engineering', which provides cost-effective and environmentally friendly solutions in a sustainable manner for several construction projects in civil engineering and some other related areas.

Questions for Practice

(Select the most appropriate answer to the multiple-choice questions from Q1.1 to Q1.5.)

1.1. A soil has a water content of 320%. This statement
   (a) Can be correct
   (b) Is incorrect
   (c) Has no physical meaning
   (d) Often holds good at most construction sites

1.2. Within a silty sandy soil,
   (a) Silt content is higher than sand content
   (b) Sand content is higher than silt content

(c) Sand content is equal to silt content
(d) Clay content is always zero

1.3. Within a medium dense sandy soil mass, saturated by a rise of water table above the ground surface, the effective stress at a depth of 5 m below the ground surface can be approximately
(a) 0
(b) 25 kPa
(c) 50 kPa
(d) 100 kPa

1.4. Which of the following is an extensible reinforcement?
(a) Steel
(b) Woven geotextile
(c) Polyester fibre
(d) Both (b) and (c)

1.5. Which of the following soils when loaded fails at relatively large strain?
(a) Dense sand reinforced with strong inextensible reinforcement
(b) Dense sand reinforced with strong and extensible reinforcement
(c) Loose sand reinforced with strong inextensible reinforcement
(d) Dense sand with weak inextensible reinforcement

1.6. Differentiate between void ratio and porosity of a soil. How can you determine the void ratio of a soil?
1.7. What are the index properties of soils? What is their importance in field applications?
1.8. What is relative density? Is it applicable to all types of soil? Can it be greater than unity? Explain briefly.
1.9. What is the effect of surcharge loading on the effective stress?
1.10. How does consolidation differ from compaction?
1.11. Name three most important properties of soil.
1.12. Discuss the terms used to describe consistency of cohesive and cohesionless soils.
1.13. What is the importance of critical hydraulic gradient? Give a field example of its application.
1.14. How does a CD triaxial compression test differ from a UU triaxial compression test? Which one is a quick test?
1.15. For a partially saturated soil deposit at a project site, water content, $w = 12\%$, void ratio, $e = 0.55$, and specific gravity of soil solids, $G = 2.66$. Determine the weight of water required to fully saturate 10 m^3 of soil.
1.16. The plastic and liquid limits of a soil are 14 and 29, respectively. What is the plasticity index of the soil? Can a soil have no plasticity index?
1.17. What is the theoretical maximum dry unit of a soil whose solid particles have a specific gravity of 2.67?

1.18. Differentiate between a systematically reinforced soil and a randomly distributed fibre-reinforced soil. Explain with the help of some examples of these reinforced soils.
1.19. What is basis for classifying the reinforcements as inextensible or extensible?
1.20. Do Reinforced Earth and ply soil refer to the same reinforced soil?
1.21. List the tests, which are used to study the behaviour of fibre-reinforced soils? What problems do you expect while testing such soils by conventional tests, which have been originally developed for soils without reinforcement?
1.22. How can concepts of fibre-reinforced soil engineering benefit the community?
1.23. Visit a local construction site where fibres are being used. Observe the challenges being faced during their introduction into soils.

Answers to Selected Questions

1.1 (a)
1.2 (b)
1.3 (c)
1.4 (d)
1.5 (b)
1.15 14.7 kN
1.16 15, Yes
1.17 26.19 kN/m^3

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Chapter 2
Basic Description of Fibre-Reinforced Soil

2.1 Introduction

Fibre reinforcement in soils can be observed in nature. In our day-to-day life, you may notice that the roots of vegetation (natural fibres) stabilize the near-surface soil that has low shear strength, mainly because of low effective stress, on both level and sloping grounds. Figure 2.1 shows how the fibres of different sizes (smaller than 1 mm to larger than 70 mm) as the roots of a tree strengthen the foundation soil for its long-term stability. The presence of plant roots is a natural means of incorporating randomly distributed fibre inclusions within the soil mass. The root fibres improve the strength of soil and the stability of soil foundations and slopes. With learning from the root reinforcements, the fibre reinforcement concept has also become significant in engineering construction practice. In fact, the soil strengthening/stabilizing/reinforcing effects of the natural fibres as the roots of vegetation may be replicated artificially by including different types of natural and synthetic fibres or as fibres from waste materials such as used plastic materials and old tyres (which pose challenging environmental and disposal problems) within the soil mass. In construction works, fibres are generally mixed randomly with soil, resulting in *randomly distributed fibre-reinforced soil* (RDFRS), which is often simply called the *fibre-reinforced soil* (FRS), as explained in Sect. 1.4 of Chap. 1. In some cases, materials like cement, lime, fly ash or bituminous products are also added to the soil along with fibres for achieving additional improvement in the engineering properties of soil. Note that fibres are a form of structural reinforcement, and their main role is working as a tension member to improve the strength characteristics of soil in addition to their roles in influencing other properties of soil (e.g. permeability, compressibility, etc.).

This chapter presents the basic description of fibres and fibre-reinforced soils, focusing on types of fibres and their characteristics, and phase concept of fibre-reinforced soil mass. Brief details of the soil reinforced with continuous fibres and multioriented inclusions are also provided.

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Fig. 2.1 Foundation soil reinforced randomly with natural root fibres (visible in an excavated trench closer to the tree) supporting a tree

2.2 Fibres

A *fibre* is a unit of matter characterized by flexibility, fineness and a high ratio of length to thickness (or diameter) (Fig. 2.1). In this book, the fibre has been considered a general term that refers to all filaments, yarns, staples, bristles/hairs, buffings, chips, crumbs and other similar highly flexible entities. A filament is an untwisted individual fibre and can be crimped or uncrimped. A yarn refers to a bundle or series of filaments twisted to produce a single fibre in which the individual filaments cannot be separated. Crimping of filaments helps prevent filament separation when the yarn is made. A staple is a cut length of fibre, measured and expressed in millimetres.

The ratio of length L to thickness (or equivalent diameter) D of the fibre is called the *aspect ratio* a_r. Thus

$$a_r = \frac{L}{D} \tag{2.1}$$

Fibres are obtained from natural, synthetic and waste (nonhazardous type) materials, and therefore, they may be categorized into the following three types:

- Natural fibres (Fig. 2.3)
- Synthetic fibres (Fig. 2.4)
- Waste fibres (Fig. 2.5)

In addition to coir and jute fibres, as shown in Fig. 2.3, there are several other natural fibres, such as wood chips, bamboo fibres, sisal fibres, palm leaves, grasses, banana fibres, corn stalks, oat and flax straws, manila fibres, cotton fibres, etc. They are available locally in different places worldwide, and can be used as the geofibres. Human and animal hairs are also available as natural fibres. Most natural fibres

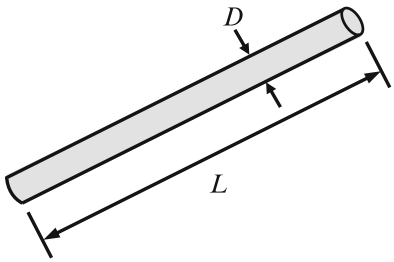

Fig. 2.2 Geometrical dimensions of a typical fibre

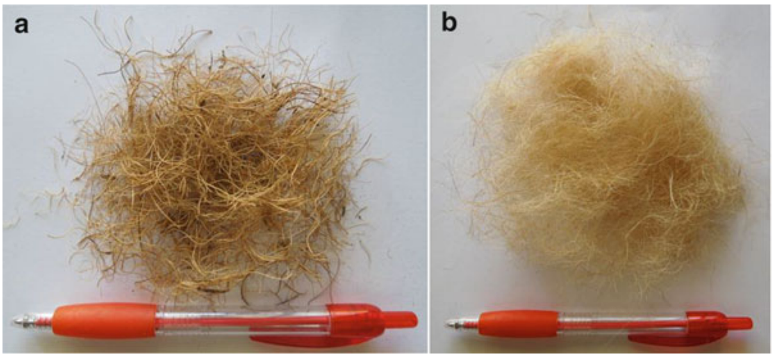

Fig. 2.3 Natural fibres: (**a**) coir fibres, (**b**) jute fibres

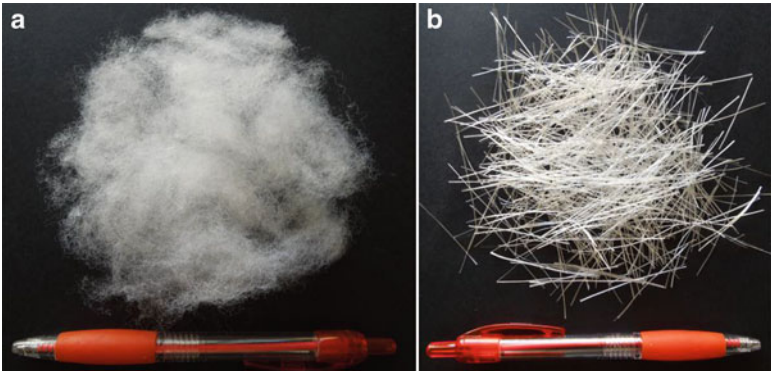

Fig. 2.4 Synthetic fibres: (**a**) polypropylene (PP) fibres, (**b**) glass fibres

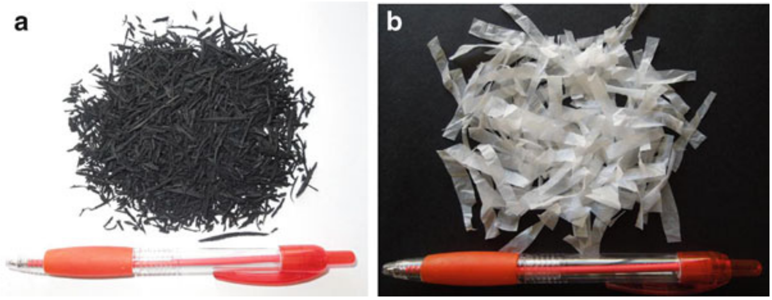

Fig. 2.5 Waste fibres: (**a**) old/used tyre fibres, (**b**) waste/used plastic fibres

originate from plant and vegetation, animal and mineral sources. Bamboo grids and mats are also used as soil reinforcement to create systematically reinforced soils as the geosynthetic reinforcements are used.

Of most natural fibres, coir has the greatest tensile strength and retains this property even in wet conditions. It is a biodegradable organic fibre material containing 40% lignin and 54% cellulose (Rao and Balan 2000). The high content of lignin, which is a complex hydrocarbon polymer, makes the coir fibres degrading slowly, and so they are useful in different applications, as discussed in Chap. 5. The coir fibres may play their roles for a long period (1–2 years) when included within the soil mass, even in saline environment. Bamboo fibres are very strong in tension but have low modulus of elasticity and high water absorption. The best quality of bamboo fibres is that they are seldom eaten by pests or infected by pathogens. Certain fibres, such as sisal fibres, have very high initial tensile strength and are strong as the equivalent polyester fibres.

Table 2.1 provides the values of some specific properties of coir, jute and bamboo fibres, determined by Biswas et al. (2013), using a mini-tensile/compression testing machine, but these values contradict the observations reported in the fibre-reinforced

Table 2.1 Physical/mechanical properties of some fibres

Fibre types	Tensile strength (MPa)	Young's modulus of elasticity (MPa)	Strain at failure (%)
Coir (brown)	165–222	3.79	41.0
Coir (white)	185–237	3.97	38.7
Jute	331–414	28.43	2.6
Bamboo	615–862	35.45	4.1

After Biswas et al. (2013)

soil literature. This demonstrates that for any project work, the tabulated values of fibres may not be the realistic values for their direct use in design of fibre-reinforced soil structures. The properties of fibres, therefore, should be determined by conducting the tests on the fibres being used in the project, as the fibres can vary significantly in their properties even for the fibres coming from the same natural plant.

Natural fibres have affordable cost, strength, environmentally friendly characteristics (e.g. contributing to greener Earth by reducing the greenhouse gas emissions in construction) and bulk availability, but they have some practical drawbacks such as reproducibility and biodegradability. In addition, the fibre geometry varies significantly, thus the design procedure with application of natural fibres may require a special attention. Except coir fibre, most natural fibres have a poor resistance to alkaline environment.

Natural fibres exhibit progressive loss of strength and other characteristics when included within the soil mass. The rate of loss of property varies with type of fibres. The problem of biodegradability of geonaturals can be overcome by suitable treatment methods, such as alkali and other chemical treatments, enzyme treatment, UV grafting with monomers, physical and chemical coatings using synthetic polymers or resins, antimicrobial finishing, etc., with some additional cost. Thus, the natural fibres can be used as an alternative low-cost reinforcing materials or admixtures for improving the engineering behaviour of weak soils or other similar materials in some field applications, such as construction of pavements for village and forest areas, or at least in the short-term applications, such as erosion control, where strength durability of fibres is not an issue. The use of natural fibres for erosion control is a sustainable and environmentally friendly application.

Figure 2.4 shows only two types of synthetic fibres (PP fibres and glass fibres), but there are several others, such as polyester (PET) fibres, polyethylene (PE) fibres, nylon/polyamide (PA) fibres, carbon fibres, steel/metal fibres, etc. There are several environmental factors that affect the durability of polymers. Ultraviolet component of solar radiation, heat and oxygen and humidity are the factors above the ground that may lead to degradation. Below the ground, the main factors affecting the durability of polymers are soil particle size and angularity, acidity/alkalinity, heavy metal ions, presence of oxygen, water content, organic content and temperature. The resistance of commonly used polymers to some environmental factors is

Table 2.2 A comparison of the resistance of some polymers

Influencing factors	Resistance of polymers			
	PP	PET	PE	PA
Ultraviolet light (unstabilized)	Medium	High	Low	Medium
Ultraviolet light (stabilized)	High	High	High	Medium
Alkalis	High	Low	High	High
Acids	High	Low	High	Low
Salts	High	High	High	High
Detergents	High	High	High	High
Heat, dry (up to 100 °C)	Medium	High	Low	Medium
Steam (up to 100 °C)	Low	Low	Low	Medium
Hydrolysis (reaction with water)	High	High	High	High
Micro-organisms	High	High	High	Medium
Creep	Low	High	Low	Medium

Adapted from John (1987) and Shukla (2002)

Table 2.3 Typical properties of polymers

Polymers	Specific gravity	Melting temperature (°C)	Tensile strength at 20 °C (MPa)	Modulus of elasticity (GPa)	Strain at break (%)
PP	0.90–0.91	160–165	400–600	1.3–1.8	10–40
PET	1.22–1.38	260	800–1200	12–18	8–15
PE	0.91–0.96	100–135	80–600	0.2–1.4	10–80
PVC	1.38–1.55	160	20–50	2.7–3	50–150
PA	1.05–1.15	220–250	700–900	3–4	15–30

After Shukla (2016)

compared in Table 2.2. The basic properties of these polymers are given in Table 2.3. It must be emphasized that the involved reactions are usually slow and can be retarded even more by the use of suitable additives. When the polymers are subjected to a higher temperature, they lose their weight. What remains above 500 °C is probably carbon black and ash (Fig. 2.6).

The PP fibres have been widely used in experimental investigations of fibre-reinforced soils. The primary attraction is that of low cost. It is easy to mix PP fibres with soil, and they have relatively a high melting point, which makes it possible to place the fibre-reinforced soil in the oven and conduct the water content determination tests. Also, the PP is a hydrophobic and chemically inert material which does not absorb or react with the soil moisture or leachate. The fibres are produced in fibrillated bundles. If the fibres are added to the soil during mixing cycle, the mixing action opens the bundles and separates them into multifilament fibres (Miller and Rifai 2004).

Synthetic fibres have the following two advantages over natural fibres (Krenchel 1973; Hoover et al. 1982):

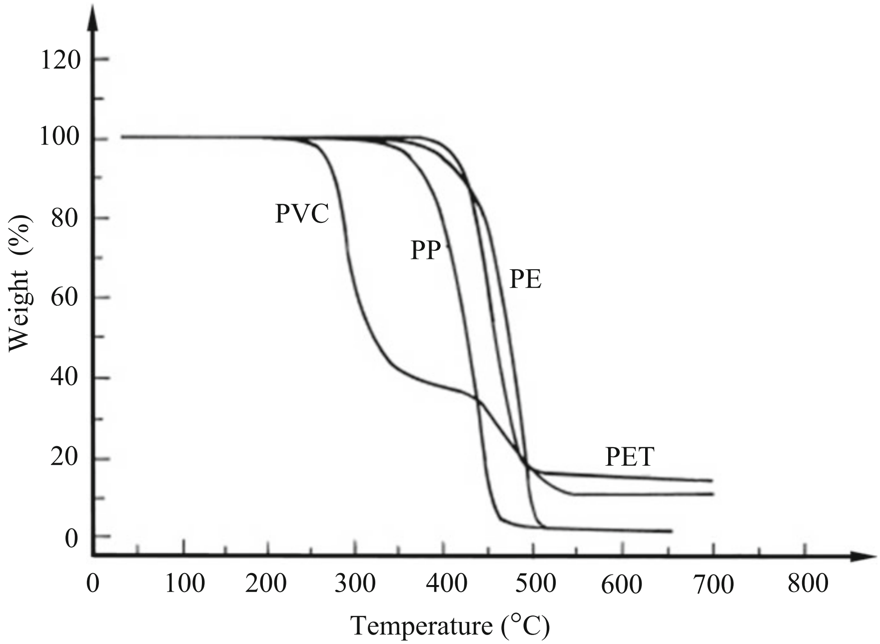

Fig. 2.6 Effect of temperature on some geosynthetic polymers (After Thomas and Verschoor 1988)

1. Synthetic fibres can be produced according to desired specifications. For example, geometry of fibres can be controlled and shape of fibres and surface conditions can be altered in order to enhance the frictional properties of fibres.
2. Most synthetic fibres do not biodegrade when subjected to variable environments of moisture, heat, cold or sunlight.

Old/used tyres and waste plastic materials are available in large quantities worldwide; they may be utilized in construction projects in various forms, especially in the granular form, chips or fibres; otherwise they may occupy a large volume of the landfills when disposed of. The type of the tyre chips and fibres depends primarily on the design of the shredder (i.e. machine for cutting). The average specific gravity of tyre chips/fibres typically ranges from 1.13 to 1.36 (average value of 1.22) depending on the metal content. The tyre chips/fibres without metal have a narrow range of specific gravity around 1.15. The unit weight of pure tyre chip fills typically ranges from 3 to 6 kN/m^3 (Edil and Bosscher 1994). Tyre buffings, also called the rubber fibres, are a by-product of the tyre retread process. They have an elongated fibrous shape with variable length of even dust size and high strength and extensibility, and so they can be used as soil reinforcement. Utilization of waste fibres, including some waste natural fibres (e.g. human and animal hairs) as soil reinforcement, can avoid the environmental and disposal problems. Additionally the use of waste fibre reinforcement helps in sustainable development of various infrastructures developed on weak and unsuitable soils.

In the area of fibre composites, fibres are also classified based on their length as (Agarwal and Lawrence 1980; Hoover et al. 1982) short fibres and continuous fibres. The fibres smaller than 76.2 mm (3 in.) are usually called the short fibres. The fibres longer than 76.2 mm (3 in.) when included in matrix material (e.g. soil) extend throughout the material mass, and so they are called the long/continuous fibres. The mechanics of stress transfer differs in composites reinforced with short and continuous fibres. For short fibre-reinforced composite, the applied stresses are first transferred to the matrix material, then to the fibres through the fibre ends and the surfaces of the fibres near the fibre ends. For continuous fibre-reinforced composite, the applied stresses are transferred to the fibres and matrix (soil) at the same time. Details of the soil mass reinforced with almost infinite length of fibres are briefly presented in Sect. 2.2. This book has mainly focused on presenting the fundamentals of soil reinforced with short, discrete, flexible fibres, as they have been studied and their applications have been reported, without strictly following the upper length limit for short fibres as classified in the area of fibre composites.

Basic properties of fibres used for reinforcing the soils in several laboratory and field studies of fibre-reinforced soils are given in Table 2.4.

The following points regarding fibres are worth mentioning:

1. A long continuous roll of a single filament, groups of filaments or yarns is called the tow.
2. Fibres can be either straight or crimped (texturized). Crimping is one of the texturing procedures for fibres.
3. In the fibre industry, the fibres are also described in terms of linear mass density (kg/m), which is generally expressed in denier (grams per 9 km of the fibre) or tex (grams per 1 km of the fibre). Thus, 1 denier = 1 g/9 km, 1 tex = 1 g/ km, and 1 tex = 9 denier.
4. Denier is an indirect measure of fibre diameter. For example, if 9 km of PET filaments weighs 120 g, it is classed as a 120-denier filament.
5. It is possible to convert denier to more conventional diametric measure by relating denier to specific gravity through the volumetric relation for a circular cylinder. As an example, a 75-denier filament would have a diameter corresponding to a fine-textured human hair, while a 2500-/250-denier yarn would correspond in size to a packing twine. A 2500/250 yarn of fibre denotes a fibre with a 2500 total denier measure but composed of 250 individual filaments, each of which is 10 denier (Hoover et al. 1982).
6. The fibre properties, such as tensile strength, tensile modulus, elongation/strain at break, tenacity, etc., are based on the denier of the fibre.
7. Tenacity is a measure of the tensile strength of a fibre expressed in terms of grams/denier. A 100-denier filament that breaks under a 250-g load is rated at 2.5 g/denier. Elongation at break refers to strain characteristic of the fibre, i.e. a measure of longitudinal deformation that occurs prior to rupture, and expressed as a percentage (Hoover et al. 1982).

Table 2.4 Properties of fibres

Characteristics of fibres	Kaniraj and Havanagi (2001)	Consoli et al. (2009)	Jha et al. (2015)	Park (2009)	Spritzer et al. (2015)	Lovisa et al. (2010)	Khattak and Alrashidi (2006)	Babu et al. (2008)	Spritzer et al. (2015)	Sarbaz et al. (2014)
Type	PET fibres	PP fibres	PE fibres	PVA fibres	Nylon fibres	Glass fibres	PC fibres	Coir fibres	Jute fibres	Date palm fibres
Specific gravity, G_f	1.3	0.91	0.99	1.3	1.15	1.7	1.5	1.07	1.47	0.92
Average length, L (mm)	20	24	12	12	6–18	10–15	3	15	7–9	295
Equivalent diameter, D (mm)	0.075	0.023	0.035	0.1	0.003–0.01	0.02	0.015	0.25	0.005–0.025	0.42
Tensile strength σ_f (MPa)	80–170	120	600	1078	300	300	500	102	331–414	123
Strain at break/failure (%)		80								5.10
Tensile modulus, E_f (GPa)	1.45–2.5	3					50	2		2.47

Note: PET polyester, PP polypropylene, PE polyethylene, *PVA* polyvinyl alcohol, *PC* processed cellulose (derived from processing of wood)

8. Steel fibres are prone to rust and acids. Glass fibres are expensive.
9. Natural fibres lose their strength when subjected to alternate wetting and drying environment. They are environmentally friendly construction materials.
10. What part of the plant the fibres come from, the age of the plant, and how the fibres are isolated are some of the factors which affect the performance of natural fibres in a natural fibre-reinforced soil (Rowell et al. 2000).

Example 2.1
Determine the aspect ratio of the PP fibres if their average length and diameter are 20 mm and 0.05 mm, respectively.

Solution
Given: $L = 20$ mm and $D = 0.05$ mm
From Eq. (2.1), the aspect ratio,

$$a_r = \frac{L}{D} = \frac{20}{0.05} = \mathbf{400}$$

2.3 Phases in a Fibre-Reinforced Soil Mass

Fibres can be considered similar to soil solid particles. Hence, like an unreinforced soil mass, the fibre-reinforced soil mass may be represented by a three-phase system as shown in Fig. 2.7, with fibre and soil solids represented separately. The concept of phase relationships, being adopted widely in soil mechanics (Shukla 2014) and geotechnical engineering (Shukla 2015) for unreinforced soils, can be utilized for developing the phase relationships for fibre-reinforced soils. Some phase relationships for the fibre-reinforced soils are defined below:

Void ratio of the soil mass only,

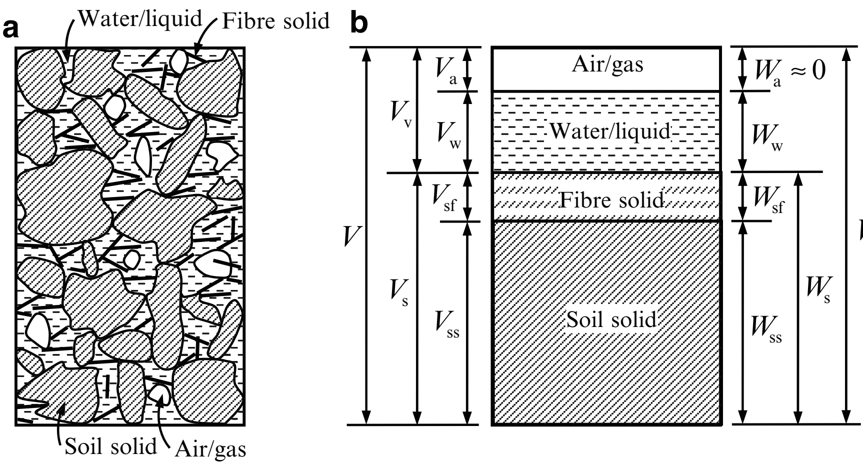

Fig. 2.7 An element of fibre-reinforced soil mass: (**a**) phases in the natural state, (**b**) phases separated for mathematical analysis

$$e_{\mathrm{s}} = \frac{V_{\mathrm{vs}}}{V_{\mathrm{ss}}} \tag{2.2}$$

Void ratio of the fibre mass only,

$$e_{\mathrm{f}} = \frac{V_{\mathrm{vf}}}{V_{\mathrm{sf}}} \tag{2.3}$$

Void ratio of the fibre-reinforced soil mass,

$$e_{\mathrm{R}} = \frac{V_{\mathrm{v}}}{V_{\mathrm{s}}} \tag{2.4}$$

Volume ratio of the fibre-soil solids,

$$V_{\mathrm{r}} = \frac{V_{\mathrm{sf}}}{V_{\mathrm{ss}}} \tag{2.5}$$

where V_{vs} is the volume of soil voids, V_{ss} is the volume of soil solids, V_{vf} is the volume of fibre voids, V_{sf} is the volume of fibre solids, $V_{\mathrm{v}}(=V_{\mathrm{vs}} + V_{\mathrm{vf}} = V_{\mathrm{a}} + V_{\mathrm{w}})$ is the volume of voids of fibre-reinforced soil and $V_{\mathrm{s}}(=V_{\mathrm{ss}} + V_{\mathrm{sf}})$ is the volume of solids of fibre-reinforced soil.

Note that in Fig. 2.7, the subscripts a, w, v and f refer to air, water, void and fibre, respectively. In symbols with dual subscripts, the subscript s when appears first refers to 'solid' but it refers to 'soil mass' when appears as the second subscript.

The following terms are often used to describe the behaviour of fibre-reinforced soils:

Fibre content (a.k.a concentration of fibres or weight ratio or weight fraction or gravimetric fibre content),

$$p_{\mathrm{f}} = \frac{W_{\mathrm{sf}}}{W_{\mathrm{ss}}} \tag{2.6}$$

Fibre area ratio,

$$A_{\mathrm{r}} = \frac{A_{\mathrm{f}}}{A} \tag{2.7}$$

Volumetric fibre content,

$$p_{\mathrm{vf}} = \frac{V_{\mathrm{sf}}}{V} \tag{2.8}$$

where W_{ss} is the weight of soil solids (i.e. dry weight of soil, W_{ds}), W_{sf} is the weight of fibre solids (i.e. dry weight of fibres, W_{df}), A_{f} is the total cross-sectional area of fibres in a plane (e.g. shear/failure plane) of area A within the reinforced soil mass and V is the total volume of the fibre-reinforced soil element.

The following points are worth mentioning:

1. In the denominator of Eq. (2.6), the weight of soil solids is used instead of the weight of fibre-reinforced soil solids for maintaining the consistency with soil mechanics practice as the water content is defined. This practice is often followed in stabilization of soil with admixtures such as cement, lime and fly ash. For example, the cement content p_c in cement-stabilized soil is defined as

$$p_c = \frac{W_{sc}}{W_{ss}} \tag{2.9}$$

   where W_{sc} is the weight of cement solids (i.e. dry weight of cement, W_{dc}).
2. In the case of cement-stabilized fibre-reinforced soil, the fibre content may be defined as

$$p_f = \frac{W_{sf}}{W_{ss} + W_{sc}} \tag{2.10}$$

3. The values of p_f, p_{vf}, A_r and p_c are normally expressed as a percentage (%).
4. If the fibres are oriented perpendicular to the failure plane in any specific application, $A_r = p_{vf}$.
5. It is easier to measure p_f than p_{vf}. Hence, for practical purposes, such as preparation of test specimens of fibre-reinforced soil and its field applications, p_f is conveniently used. The use of p_{vf} is often done in modelling of fibre-reinforced soils based on theory of mixtures because the mechanical properties of the composite constituents (soil and fibres), especially when the unit weights of the constituents differ greatly, are governed significantly by the volumetric parameters, and they are not necessarily related to the unit weights (Michalowski and Zhao 1996).

From Eq. (2.8),

$$p_{vf} = \frac{V_{sf}}{V} = \frac{V - (V_{ss} + V_v)}{V} = 1 - \frac{V_{ss} + V_v}{V} = 1 - p_{vs} \tag{2.11}$$

where p_{vs} is the volumetric soil content in fibre-reinforced soil. Thus

$$p_{vs} = \frac{V_{ss} + V_v}{V} \tag{2.12}$$

and

$$p_{vs} + p_{vf} = 1 \tag{2.13}$$

The parameters p_{vs} and p_{vf} are also known as volumetric concentration factors for the soil matrix and fibres, respectively. These factors are used in the development of constitutive models for the fibre-reinforced soil (Ibraim et al. 2010). Note that in the definition of p_{vs}, the volume of voids V_{v}, excluding the part occupied by the fibres, is considered to be the part of the volume of soil matrix.

Using a mixture rule, the effective stress σ'_{R} within the fibre-reinforced soil can be divided into soil matrix and fibre matrix components as (Ibraim et al. 2010):

$$\sigma'_{\mathrm{R}} = p_{\mathrm{vs}}\sigma' + p_{\mathrm{vf}}\sigma'_{\mathrm{f}} \tag{2.14}$$

where σ' is the effective stress of the soil matrix and σ'_{f} is the effective stress of the fibre matrix. For an unreinforced soil, $p_{\mathrm{vf}} = 0$, $p_{\mathrm{vs}} = 1$ and $\sigma'_{\mathrm{R}} = \sigma'$, which represents the effective stress for soil only in a conventional way as per the Terzaghi's effective stress concept.

One can develop useful phase interrelationships in view of their practical applications as we do in soil mechanics. Several researchers have considered the phase concept and used specific phase relationships and interrelationships while analysing the behaviour of fibre-reinforced soils as required in their investigations. For example, Maher and Gray (1990) considered fibres to be a part of solid fraction of fibre-reinforced soil with different specific gravity from that of soil solids in their study as considered in Fig. 2.7. Shukla et al. (2015) presented the following phase interrelationships:

$$e_{\mathrm{R}} = \frac{e_{\mathrm{s}} + V_{\mathrm{r}}e_{\mathrm{f}}}{1 + V_{\mathrm{r}}} \tag{2.15}$$

$$V_{\mathrm{r}} = p_{\mathrm{f}}\frac{G}{G_{\mathrm{f}}} \tag{2.16}$$

$$e_{\mathrm{R}} = \frac{G_{\mathrm{R}}\gamma_{\mathrm{w}}}{\gamma_{\mathrm{dR}}} - 1 \tag{2.17}$$

$$G_{\mathrm{R}} = (1 + p_{\mathrm{f}})\left(\frac{1}{G} + \frac{p_{\mathrm{f}}}{G_{\mathrm{f}}}\right)^{-1} \tag{2.18}$$

where γ_{w} is the unit weight of water, G is the specific gravity of soil solids, G_{f} is the specific gravity of fibre solids, G_{R} is the specific gravity of fibre-reinforced soil solids and γ_{dR} is the dry unit weight of fibre-reinforced soil.

Referring to Fig. 2.7, the expression for γ_{dR} is given as

$$\gamma_{\mathrm{dR}} = \frac{W_{\mathrm{sf}} + W_{\mathrm{ss}}}{V} = \frac{W_s}{V} \tag{2.19}$$

From Eqs. (2.6), (2.8) and (2.19),

$$p_{\mathrm{vf}} = \frac{p_{\mathrm{f}}\gamma_{\mathrm{dR}}}{(1 + p_{\mathrm{f}})G_{\mathrm{f}}\gamma_{\mathrm{w}}} \tag{2.20}$$

or

$$p_{\mathrm{f}} = \frac{G_{\mathrm{f}}\gamma_{\mathrm{w}}p_{\mathrm{vf}}}{\gamma_{\mathrm{dR}} - G_{\mathrm{f}}\gamma_{\mathrm{w}}p_{\mathrm{vf}}} = \frac{1}{\frac{\gamma_{\mathrm{dR}}}{G_{\mathrm{f}}\gamma_{\mathrm{w}}p_{\mathrm{vf}}} - 1} \tag{2.21}$$

Using Eqs. (2.17) and (2.18), Eq. (2.20) becomes

$$p_{\mathrm{f}} = \frac{(1 + e_{\mathrm{R}})G_{\mathrm{f}}p_{\mathrm{vf}}}{G[1 - (1 + e_{\mathrm{R}})p_{\mathrm{vf}}]} \tag{2.22}$$

Example 2.2
Derive the expressions given in Eq. (2.15) and (2.16).

Solution
Derivation of Equation (2.15):

$$e_{\mathrm{R}} = \frac{V_{\mathrm{v}}}{V_{\mathrm{s}}}$$

(using Eq. (2.4))

$$= \frac{V_{\mathrm{vs}} + V_{\mathrm{vf}}}{V_{\mathrm{ss}} + V_{sf}}$$

(Void volume occupied by air and/or water may be considered to be contributed by soil solids and fibre solids separately.)

$$= \frac{\frac{V_{\mathrm{vs}}}{V_{\mathrm{ss}}} + \frac{V_{\mathrm{sf}}}{V_{\mathrm{ss}}} \times \frac{V_{\mathrm{vf}}}{V_{\mathrm{sf}}}}{1 + \frac{V_{\mathrm{sf}}}{V_{\mathrm{ss}}}}$$

(dividing numerator and denominator by V_{ss})

$$= \frac{e_{\mathrm{s}} + V_{\mathrm{r}}e_{\mathrm{f}}}{1 + V_{\mathrm{r}}}$$

(using Eqs. (2.2), (2.3) and (2.5))

Derivation of Equation (2.16):

$$V_{\mathrm{r}} = \frac{V_{\mathrm{sf}}}{V_{\mathrm{ss}}}$$

(using Eq. (2.5))

$$= \frac{(W_{\mathrm{sf}}/\gamma_{\mathrm{w}}G_{\mathrm{f}})}{(W_{\mathrm{ss}}/\gamma_{\mathrm{w}}G)}$$

$$\left(\because G = \frac{W_{\mathrm{ss}}/V_{\mathrm{ss}}}{\gamma_{\mathrm{w}}} \text{ and } G_{\mathrm{f}} = \frac{W_{\mathrm{sf}}/V_{\mathrm{sf}}}{\gamma_{\mathrm{w}}}\right)$$

$$= \frac{W_{\mathrm{sf}}G}{W_{\mathrm{ss}}G_{\mathrm{f}}}$$

$$= p_f\frac{G}{G_{\mathrm{f}}}$$

(using Eq. (2.6))

Example 2.3
If a clayey soil is reinforced with 1.25% of PET fibres, determine the specific gravity of the fibre-reinforced soil. Assume the specific gravity values for soil solids and fibre solids are 2.70 and 1.3, respectively.

Solution
Given: $p_{\mathrm{f}} = 1.25\%$, $G = 2.70$, and $G_{\mathrm{f}} = 1.3$
From Eq. (2.18), the specific gravity of fibre-reinforced soil is

$$G_{\mathrm{R}} = (1 + p_{\mathrm{f}})\left(\frac{1}{G} + \frac{p_{\mathrm{f}}}{G_{\mathrm{f}}}\right)^{-1} = (1 + 0.0125)\left(\frac{1}{2.70} + \frac{0.0125}{1.3}\right)^{-1} = 2.66$$

Note that the phase relationships and interrelationships, as presented here, are fairly of general nature and can be utilized in several applications of fibre-reinforced soils in a way similar to that traditionally used in soil mechanics and geotechnical engineering for the analysis of unreinforced soils. Based on the laboratory experiments, Shukla et al. (2015) presented the variation of void ratio e_{R} of the fibre-reinforced sand with logarithm of $(100V_{\mathrm{r}} + 1)$, as shown in Fig. 2.8, with the following empirical expression:

$$e_{\mathrm{R}} = a\ln(100V_{\mathrm{r}} + 1) + b \tag{2.23}$$

where $a = 0.0333$ is the slope of the linear relationship and represents the type of fibre used (i.e. virgin homopolymer polypropylene) and $b = 0.4913$ is the value of e_{R} at $V_{\mathrm{r}} = 0$ and represents the type of sand used (i.e. poorly graded silica sand). For other types of fibre and/or sand, a and b can be obtained from the data obtained from a minimum of four compaction tests on the fibre-reinforced soil at hand from which the measured values of V_{r} and the corresponding e_{R} can be calculated and used to determine a and b, then the equation similar to Eq. (2.23) can be used for future prediction of the void ratio of the specific fibre-reinforced soil.

The coefficient of determination R^2 of the correlation in Eq. (2.23) is 0.9935. Note that R^2 is an index of the reliability of the relationship. A regression equation

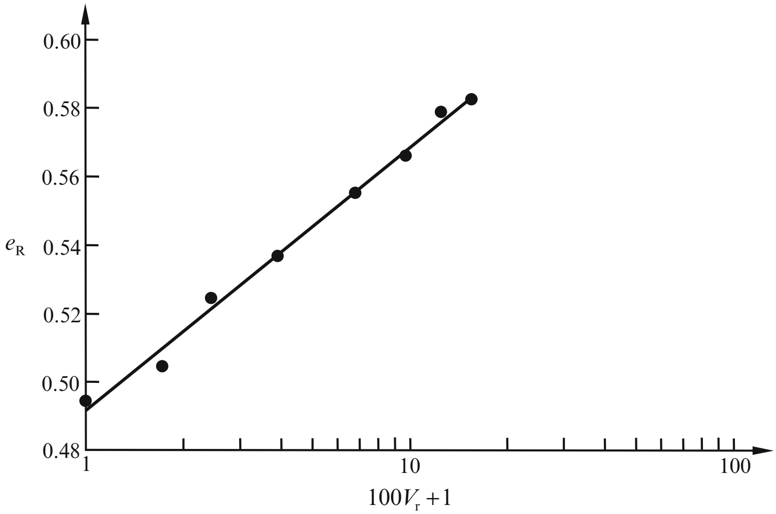

Fig. 2.8 Variation of void ratio e_R of the fibre-reinforced sand with logarithm of $(100\,V_r + 1)$ (Adapted from Shukla et al. 2015)

that lies close to all the observation points gives a higher value of R^2. The maximum value of R^2 is unity. A suitable numerical software package, such as MATLAB, is generally used for developing the statistical relationships between two or more variables. Such relationships are called the statistical or regression models. A few more regression models are presented in Chap. 4.

It is expected that the researchers may carry out experimental studies with several types of fibre and soil and also with cemented fibre-reinforced soils to compare their results with the phase relationships as presented here and also develop similar new expressions as required for their use by practicing engineers.

2.4 Soil Reinforced with Continuous Fibres and Multioriented Inclusions

The soil can be reinforced randomly with continuous fibres or multioriented (3-dimensional, 3D) inclusions. Some studies are available for such reinforced soils.

The sand reinforced with continuous synthetic (PET, PP, etc.) fibres/threads was originally conceived by Laboratoire Central des Ponts et Chaussees (LCPC) and registered as *Texsol* (Leflaive 1982). To obtain the soil reinforced with continuous fibres, a number of threads are pneumatically or hydraulically projected on sand in a movement at the extremity of a conveyor belt or at the vent of a pipe used to build a hydraulic fill at the construction site. A special machine was designed to produce up to 20 m^3/h of Texsol with the use of about 100 km/m^3 of thread, projected simultaneously from 50 bobbins at the speed of 10 m/s (Leflaive et al. 1983; Prisco and Nova 1993).

In the thread-reinforced soil, the sand particles and threads are closely connected because of friction between them, and hence the threads react in tension to any extensional deformation caused by loading, thus giving an artificial confinement to sand. As a consequence, both strength and stiffness of sand in the presence of threads increase (Prisco and Nova 1993). The fibre content typically varies between 0.1 and 0.2%. The triaxial compression tests on thread-reinforced sand shows that significant cohesions of the order of 200 kPa for 0.2% fibre content by weight of sand may result. The angle of internal friction of reinforced sand is, however, essentially the same as that of unreinforced sand. Note that the way in which such a material is put in place gives the reinforced soil an ordered structure, so that the measured cohesion and friction angle vary with the orientation of the bedding plane with respect to the principal axes of stress (Leflaive and Liausu 1986; Prisco and Nova 1993). Failure normally occurs at strains larger than those of unreinforced sand. Thus, the thread-reinforced soil is more ductile than sand and can be considered essentially as a cemented soil. To achieve the full strength of thread-reinforced soil, significant strains and consequently significant deformations may take place; this may not be a desirable characteristic for some applications, such as the use of thread-reinforced soil to support structural loads as a foundation bed. The design of structures constructed with thread-reinforced soils can be based on limit equilibrium methods when the strains/deformations are limited. Numerical and constitutive models have also been presented for the analysis of such reinforced soils (Villard and Jouve 1989; Prisco and Nova 1993).

In the past, thread-reinforced sandy soils have been used for constructing earth retaining walls, particularly on soft compressible soils, with facing slope angles as high as 75°, filter or drain resistant to erosion, embankments with steep slopes, foundation beds over compressible soils and explosion-resistant facilities in civil and military installations for storage of explosives and liquefied gas. Thread-reinforced soils are also used for slope stabilization works (Leflaive 1985; Khay et al. 1990; Ishizaki et al. 1992). The thread-reinforced soils provide a suitable environment for vegetation to grow on the reinforced soil structures, and they are also effective to resist dynamic loads.

The soil can be strengthened by inclusion of multioriented/3D geosynthetic elements (termed jacks) within or upon a compacted or naturally deposited granular soil (Lawton and Fox 1992; Lawton et al. 1993). These elements can be oriented randomly within a compacted soil mass by mixing them with soil prior to compaction; placed in layers on lifts of loose soil, compacted soil or naturally deposited soil layer; or placed in a layer on the surface of a naturally deposited soil or compacted fill and pressed into the soil by compaction. 3D elements can also be used as lightweight, highly porous artificial soil.

A large number of 3D elements of different sizes and shapes are available, and they can also be designed to meet specific requirements of the project. Lawton et al. (1993) used five types of 3D inclusions in their research. All 3D elements consisted of six stems extending radially from a central hub, with enlarged heads on four of the stems, but they differed in material and manufacturing process. Both layered and random oriented inclusions are effective in strengthening and/or stiffening the granular soil under certain conditions. Increases in deviator stress at failure as large as above 100% can be determined in triaxial tests. Additional details about behaviour of soils with 3D inclusions can be found in the works of Zhang et al. (2006) and Harikumar et al. (2016).

Chapter Summary

1. A *fibre* is a unit of matter characterized by flexibility, fineness and a high ratio of length to thickness (or diameter). In general, the fibre is a general term that refers to all filaments, yarns, staples, bristles/hairs, buffings, chips, crumbs and other similar highly flexible entities.
2. Fibres available for use in construction can be classified as natural fibres (e.g. coir fibres, jute fibres, etc.), synthetic fibres (e.g. polypropylene fibres, polyester fibres, etc.) and waste fibres (old/used tyre fibres, used plastic fibres, etc.).
3. The PP fibres have been widely used in experimental investigations of fibre-reinforced soils. The primary attraction is that of low cost.
4. In the fibre industry, the fibres are also described in terms of linear mass density (kg/m), which is generally expressed in denier (grams per 9 km of the fibre) or tex (grams per 1 km of the fibre).
5. An element of fibre-reinforced soil can be represented as a three-phase system consisting of solid, liquid and gas phases.
6. Fibre aspect ratio, fibre content, fibre area ratio and volumetric fibre content are commonly used parameters in the fibre-reinforced soil engineering.
7. There are several phase relationships and interrelationships for their use in analysis of fibre-reinforced soils.
8. The volume ratio of fibre-soil solid is uniquely related to the void ratio of the fibre-reinforced soil.
9. Texsol is a continuous thread-reinforced soil mass, generally created at the construction site, as a homogeneous construction material by mixing sand and threads in a highly specialized manner for construction of retaining walls and some other structures.
10. Soil can be improved by 3D inclusions of different shapes and sizes as per the requirement of the specific project.

Questions for Practice

(Select the most appropriate answer to the multiple-choice questions from Q 2.1 to Q 2.5.)

2.1 The specific gravity of glass fibres is
   (a) 0.91
   (b) 0.99
   (c) 1.38
   (d) 1.7

2.2 A 90-denier filament has a weight of
   (a) 1 g/km
   (b) 9 g/km
   (c) 10 g/km
   (d) 90 g/km

2.3 The ratio of length of a fibre to its average diameter is called
   (a) Fibre content
   (b) Fibre aspect ratio
   (c) Fibre area ratio
   (d) Volumetric fibre content

2.4 Which one of the following can be measured easily?
   (a) Volumetric fibre content and aspect ratio
   (b) Fibre content and volumetric fibre content
   (c) Fibre content and aspect ratio
   (d) Fibre content, volumetric fibre content and aspect ratio

2.5 In soil reinforced with continuous fibres, the fibre content typically varies between
   (a) 0.1% and 0.2%
   (b) 0.5% and 1%
   (c) 1% and 2%
   (d) 2% and 4%

2.6 Classify the available fibres on the basis of their sources.
2.7 What are the favourable and unfavourable characteristics of natural fibres?
2.8 Polypropylene (PP) fibres have been used widely for reinforcing soil in research works. Can you explain the reasons?
2.9 Compare the resistances of PP and PET fibres against the following: UV light, acids, alkalis and heat.
2.10 Compare the specific gravity values of different fibres.
2.11 Define the following: fibre content, aspect ratio and area ratio.
2.12 How is cement content in cement-stabilized soils defined? What are the two different ways of defining fibre content in cement-stabilized soils?
2.13 Develop a relationship between volumetric soil content and volumetric fibre content.
2.14 Write an expression for the effective stress within a fibre-reinforced soil mass.
2.15 Derive the following expressions in which the symbols have their usual meanings:

$$\text{(a)} \quad e_{\mathrm{R}} = \frac{G_{\mathrm{R}}\gamma_{\mathrm{w}}}{\gamma_{\mathrm{dR}}} - 1$$

$$\text{(b)} \quad G_{\mathrm{R}} = (1 + p_{\mathrm{f}})\left(\frac{1}{G} + \frac{p_{\mathrm{f}}}{G_{\mathrm{f}}}\right)^{-1}$$

2.16 If a sandy soil is reinforced with 1% of PP fibres, determine the specific gravity of the fibre-reinforced soil. Assume the specific gravity values for soil solids and fibre solids are 2.65 and 0.91, respectively.

2.17 For a fibre-reinforced soil, the following details are available:

Fibre content, $p_f = 0.75\%$
Specific gravity of soil solids, $G = 2.66$
Specific gravity of fibre solids, $G_f = 0.99$
Determine the volume ratio of fibre-soil solids, V_r.

2.18 For a PP fibre-reinforced poorly graded sand, compacted at its maximum dry unit weight, the volume ratio of fibre-soil solids, $V_r = 5\%$. Estimate the void ratio of fibre-reinforced sand.

2.19 What is Texsol? List its different applications. Do you expect any difficulty in construction?

2.20 Collect the information about different multidirectional inclusions, which are available in your local area, for soil improvement purpose. How do these inclusions differ from short, discrete fibres?

Answers to Selected Questions

2.1 (d)
2.2 (c)
2.3 (b)
2.4 (c)
2.5 (a)
2.16 2.60
2.17 2%
2.18 0.545

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Chapter 3
Engineering Behaviour of Fibre-Reinforced Soil

3.1 Introduction

In the early days of soil reinforcement practice, most experimental and mathematical studies considered reinforcing the soil with continuous metal and geosynthetic reinforcement elements, such as strips, bars, discs and meshes, in a definite pattern (e.g. horizontal, vertical and inclined orientations) for investigating the behaviour of systematically reinforced soils. McGown et al. (1985) investigated the strengthening of a granular soil using randomly distributed polymeric mesh elements and observed the improvement in strength of the soil at all strain levels, even at very small strains. The action of randomly distributed polymeric mesh elements is to interlock particles and groups of particles together to form a unitary, coherent matrix. Because of the limited practical scope of random distribution of continuous metal and geosynthetic reinforcement elements, these elements have not been used widely for reinforcing the soils randomly, and hence the studies of such reinforced soil systems have not been given much attention.

Over the past 30–35 years, the engineering behaviour of randomly distributed, discrete, flexible, fibre-reinforced soils has been studied in significant details by many researchers worldwide. Most studies are based on the laboratory and small-scale tests, such as direct shear tests, triaxial compression tests, unconfined compression tests, compaction tests, California bearing ratio (*CBR*) tests, plate load tests, etc. This chapter presents the engineering behaviour of randomly distributed, discrete, flexible, fibre-reinforced soils as investigated through these tests by focusing on the key factors that govern the behaviour and test observations and their critical analysis in view of their field applications.

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3.2 Factors Affecting the Engineering Behaviour

Improvement in the engineering properties (strength, stiffness/modulus, permeability, etc.) of soil by inclusion of discrete flexible fibres within the soil mass depends on several factors relating soil characteristics; fibre characteristics; fibre concentration; distribution and orientation; type of admixtures, if any; mixing and compaction methods; and test/field conditions, as listed below:

- Soil characteristics: types (cohesionless/cohesive/cohesive-frictional); particle shape and size; gradation, generally expressed in terms of mean particle size (D_{50}) and coefficient of uniformity (C_u); unit weight of soil solids; total unit weight; water content; and shear strength
- Fibre characteristics: fibre types and materials (natural/synthetic/waste); shapes (monofilament, fibrillated, tape, mesh, etc.); fibre diameter, fibre length and aspect ratio (length to diameter ratio); specific surface; specific gravity of fibre solids; tensile strength, longitudinal stiffness/modulus of elasticity, linear strain at failure; fibre texture (straight or crimped); surface roughness and skin friction with soil; water absorption; melting point; and durability (resistance to biological and chemical degradation)
- Fibre concentration (fibre content, fibre area ratio), distribution and orientation
- Types of admixtures, such as lime, cement, fly ash, etc., if any
- Method of mixing of fibres with soil
- Type and amount of compaction in tests and fields, as applicable
- Test and/or field conditions and variables: confining/normal stress level, rate of loading, rate of strain, etc.

During laboratory and/or field investigations, as many of these factors/parameters as possible should be varied in a scientific manner to ascertain the influence of fibres on the engineering behaviour of soil and other similar materials when reinforced with fibres. The complete investigation is also required to validate the theoretical models of fibre-reinforced soils as presented in Chap. 4.

The engineering behaviour of fibre-reinforced soil, especially the strength characteristics, is significantly governed by interfacial shear resistance of the fibre-soil interface, which is affected by the following four primary parameters (Tang et al. 2007a; Gelder and Fowmes 2016): friction, bonding force, matrix suction and interface morphologies. These primary parameters are controlled by the following four indirect factors: water content, size effect, soil dry unit weight and cement inclusions. As a result of load application on the fibre-reinforced soil, the soil particles at the soil-fibre interface may have a tendency of rotation, and the fibre in tension may undergo plastic deformation at the sharp corners/edges of the particles. In the presence of lime or cement in soil, the rotation of particles at the soil-fibre interface is restricted by increased bonding force due to cementation. The hydrated product developed on the surface of the fibre increases the roughness and penetrates the surface. Addition of lime in clay with fibres causes flocculation and results in flocculated silt-size particles, which contact the fibre surface and help in cementing the flocculated structure to the fibre surface, resisting rotation of particles at the interface and hence increasing the interfacial shear resistance.

3.3 Shear Strength

Shear strength of fibre-reinforced soils has been studied by carrying out both direct shear tests and triaxial compression tests, mostly in accordance with standards applicable to unreinforced soils. The strength behaviour of fibre-reinforced soil is presented here as observed in these tests separately.

3.3.1 Observations in Direct Shear Tests

Based on the direct shear tests, the improvement in shear strength of soil, as a result of inclusion of fibre reinforcement, can be expressed in terms of the *shear strength improvement factor* I_s, defined as

$$I_s = \frac{\Delta\tau}{\tau_{fU}} = \frac{\tau_R - \tau_{fU}}{\tau_{fU}} = \frac{\tau_R}{\tau_{fU}} - 1 \tag{3.1}$$

where τ_{fU} is the shear strength of the unreinforced soil and τ_R is the shear stress on the fibre-reinforced soil specimen corresponding to the shear displacement $\delta(=\delta_{fU})$ of the unreinforced soil specimen at failure. The factor I_s basically presents the relative gain in shear strength of soil due to inclusion of fibres and is normally expressed as a percentage.

Note that in Eq. (3.1), τ_R can be the shear stress on the reinforced soil specimen at any selected/permissible shear displacement as required for the safety of the specific structure under consideration. For $\tau_R = \tau_{fR}$ (shear strength of reinforced soil) at $\delta = \delta_{fR}$ (shear displacement of reinforced soil specimen at failure), Eq. (3.1) reduces to the *shear strength improvement factor at failure* I_{sf}, given as

$$I_{sf} = \frac{\Delta\tau_f}{\tau_{fU}} = \frac{\tau_{fR} - \tau_{fU}}{\tau_{fU}} = \frac{\tau_{fR}}{\tau_{fU}} - 1 \tag{3.2}$$

Gray and Ohashi (1983) carried out the direct shear tests on a dry, clean, beach sand (relative density of 20% and 100%) reinforced with natural and synthetic fibres and metal (copper) wires (1 mm–2 mm thick, 20–250 mm long). The fibres were always placed in a regular pattern at approximately equal spacings from each other and from the sides of the shear box in either a perpendicular orientation to the shear plane or at some other predetermined orientation. The experimental observations were compared with theoretical predictions based on a force-equilibrium model of fibre-reinforced sand, as described in Sec. 4.3.2. The test results indicate the following:

1. Relatively low-modulus, fibre reinforcements (natural and synthetic fibres) behave as 'ideally extensible' inclusions; they do not rupture during shear. Their main role is to increase the peak shear strength and to limit the magnitude of post-peak reduction in shear resistance in a dense sand.
2. Shear strength increases of sand are directly proportional to fibre area ratios, at least up to 1.7%.
3. Shear strength increases as a result of fibre reinforcement are approximately the same for loose and dense sands. However, larger strains are required to reach the peak shear resistance in the loose case.
4. Shear strength envelopes for fibre-reinforced sand clearly show the existence of a threshold confining stress below which the fibres tend to slip or pullout. The strength envelops also indicate that the fibres do not affect the angle of internal friction of the sand. Thus the fibre inclusion in sand plays their role mainly to increase the cohesion intercept for the sand.
5. The fibres oriented either perpendicular to or randomly around the shear plane yield approximately the same shear strength increase.
6. Shear strength increases are greatest for the fibre initial orientations of 60° with respect to the shear surface. This orientation coincides with the direction of maximum principal tensile strain in a direct shear test.
7. Increasing the length of fibre reinforcements increases the shear strength of the fibre-reinforced sand, but only up to a point beyond which any further increase in fibre length has no effect.

Kaniraj and Havanagi (2001) conducted direct shear tests at a deformation rate of 0.25 mm/min on specimens (60 mm × 60 mm × 29 mm) of Rajghat fly ash, Delhi silt, mixture of 50% Rajghat fly ash and 50% Delhi silt and mixture of 50% Rajghat fly ash and 50% Yamuna sand with and without random distribution of PET fibre reinforcement. The results show that whereas the unreinforced fly ash-soil specimens reach their failure shear stress at displacements of 1–2 mm, the corresponding displacements in fibre-reinforced specimens are generally more than 4 mm and even exceed 10 mm at higher normal stresses. This is the evidence of the fibre reinforcement imparting ductility to the unreinforced fly ash-soil specimens. The compacted specimens show the usual tendency of dilation. However, the vertical displacement is significantly higher in the fibre-reinforced specimens than in the unreinforced specimens. Table 3.1 provides the total stress-strength parameters (cohesion intercept c and angle of shearing resistance ϕ) obtained from direct shear tests on unreinforced and fibre-reinforced soils.

In Table 3.1, you may notice that the trend in the change of c and ϕ due to fibre inclusions is not very consistent. However, the fibre inclusions generally tend to increase the shear strength.

The test results obtained from the direct shear tests conducted by Yetimoglu and Salbas (2003) using a standard laboratory shear box (60 mm by 60 mm in plan and 25 mm in depth) indicate that the peak shear strength and initial stiffness of the clean,

Table 3.1 Direct shear test results of fly ash, silt and soil mixtures with fibre content (defined as the ratio of weight of fibre solids to sum of weights of soil solids and fly ash solids), $p_f = 1\%$, for the reinforced cases (After Kaniraj and Havanagi 2001)

	Unreinforced		Fibre reinforced	
Soil type	c (kPa)	ϕ (degrees)	c (kPa)	ϕ (degrees)
Fly ash	19.6	37.5	3.88	45.5
Silt	15.7	29.5	21.3	38.4
Mixture of 50% fly ash and 50% silt	14.7	36	32.5	35.1
Mixture of 50% of fly ash and 50% sand	9.8	33.0	17.4	39.4

Note: For fly ash reinforced with fibres ($p_f = 0.5\%$), $\gamma_{dmax} = 11.06\ \text{kN/m}^3$ and $w_{opt} = 33.1\%$

oven-dried, uniform river sand having particles of fine to medium size (0.075–2 mm) at a relative density of 70% are not affected significantly by the PP fibre (length = 20 mm; diameter = 0.05 mm) reinforcements varying from 0.10 to 1%. The horizontal displacements at failure are also found comparable for reinforced and unreinforced sands under the same vertical normal stress. However, the fibre reinforcements reduce the soil brittleness providing a smaller loss of post-peak strength. Thus, there appears to be an increase in residual shear strength angle of the sand by adding fibre reinforcements.

Tang et al. (2007b) conducted a series of direct shear test on clayey soil cylindrical specimens (diameter = 61.8 mm, length = 20 mm) with inclusion of different percentages of PP fibres (12 mm long) and ordinary Portland cement at vertical normal stresses of 50, 100, 200 and 300 kPa. All the test specimens were compacted at their respective maximum dry unit weight and optimum water content. The specimens treated with cement were wrapped with plastic membrane in the curing box for 7, 14 and 28 days. Figure 3.1 shows the variation of shear strength parameters (cohesion c and angle of internal friction ϕ) with fibre content. It is observed that the values of c and ϕ increase with increasing fibre content. For the same fibre content, the cement inclusion significantly enhances the shear strength parameters.

In foundation and several other applications, the soil is rarely in a dry condition. Hence, Lovisa et al. (2010) conducted the direct shear tests to investigate the effects of water content on the shear strength behaviour of a poorly graded sand (classified as SP) reinforced with 0.25% randomly distributed glass fibres. The test results suggest that the peak friction angle of the fibre-reinforced sand in moist condition is approximately 3° less than that in dry condition for a relative density greater than 50%. At the peak state, the fibre inclusions introduce an apparent cohesion intercept of 5.3 kPa to the soil in the dry state, which remains almost unchanged by an increase in water content. The ultimate-state shear strength parameters are independent of water content and relative density. While the cohesion intercept of unreinforced sand at ultimate state is zero, the inclusion of fibres causes an apparent cohesion of 2.6 kPa.

Falorca and Pinto (2011) studied the shear strength behaviour of poorly graded sand (SP) and clayey soil of low plasticity (CL) reinforced with short, randomly distributed PP fibres by conducting direct shear tests (60-mm square box). The test results show that the fibres increase the shear strength and significantly modify shear stress-displacement behaviour of the soil, as presented in Figs. 3.2 and 3.3. Shear strength increases with increase in fibre content and fibre length. The increase in shear strength

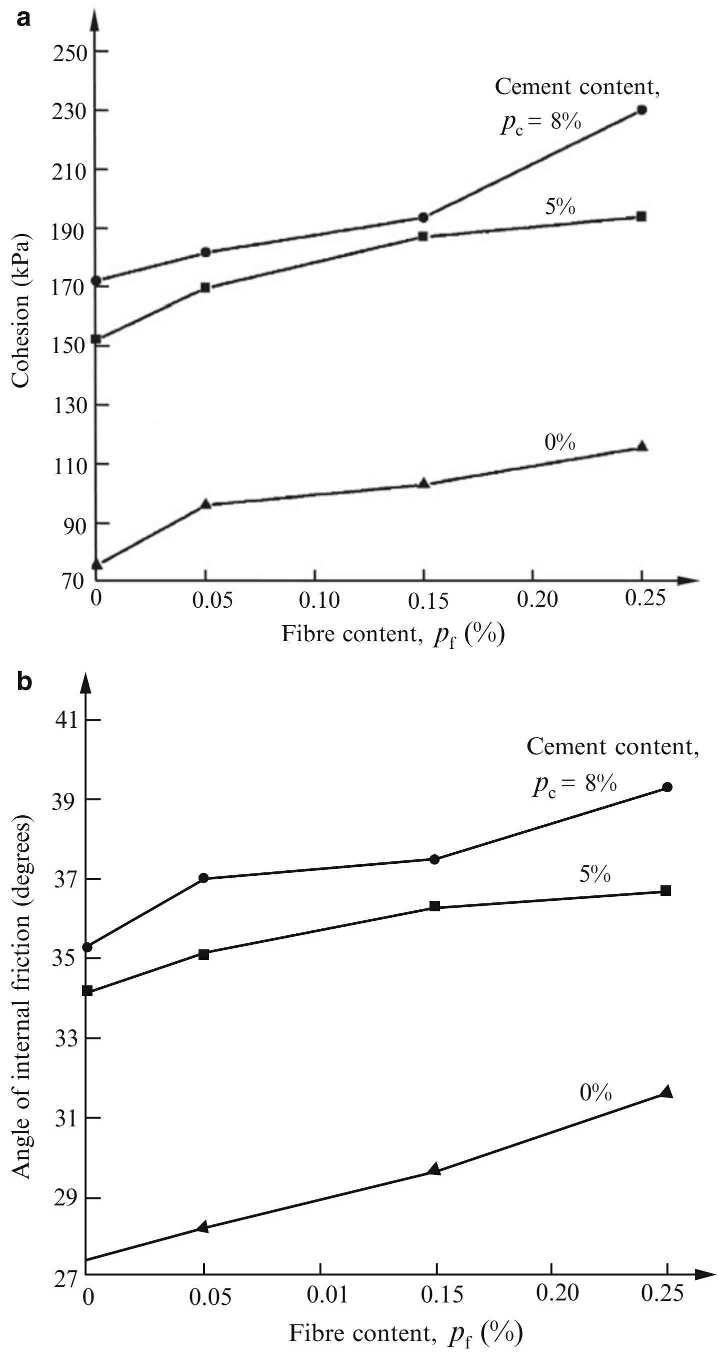

Fig. 3.1 Variation of shear strength parameters with fibre content p_f for uncemented and cemented clayey soils after 28 days of curing: (**a**) cohesion c; (**b**) angle of internal friction ϕ (Adapted from Tang et al. 2007b)

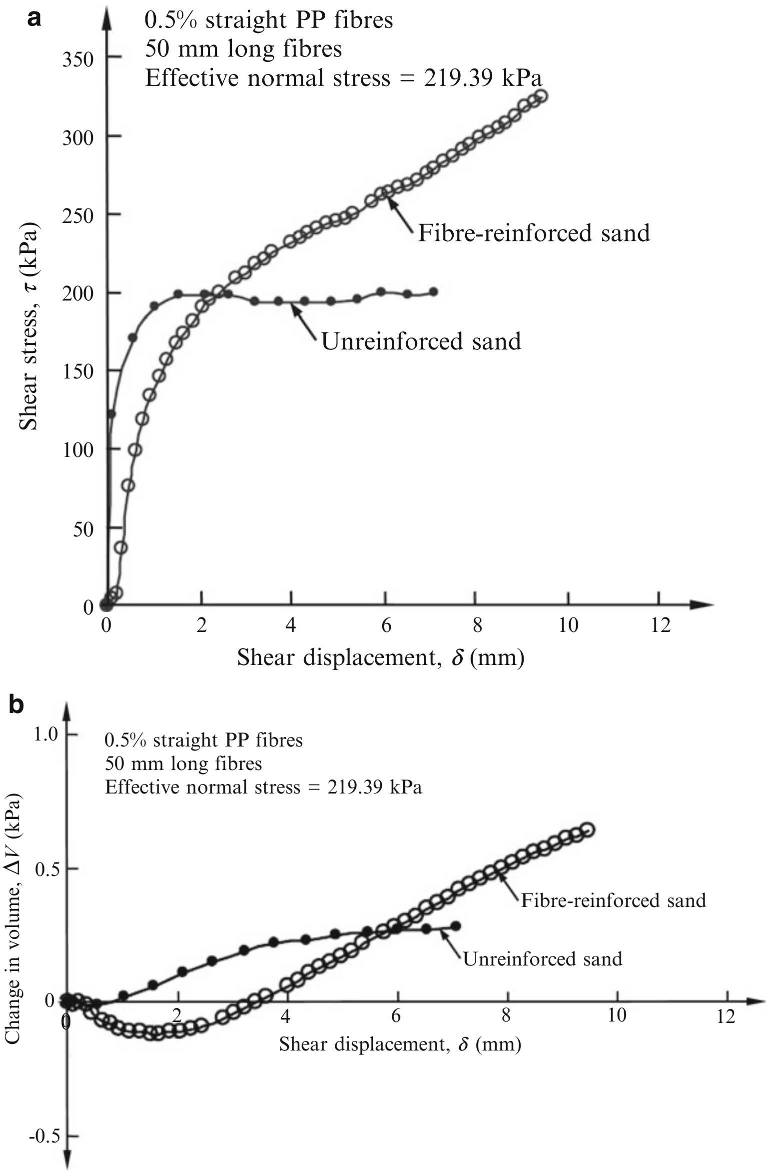

Fig. 3.2 Direct shear test results for fibre-reinforced sand: (**a**) shear stress τ versus shear displacement δ; (**b**) change in volume of the specimen ΔV versus shear displacement δ (Adapted from Falorca and Pinto 2011)

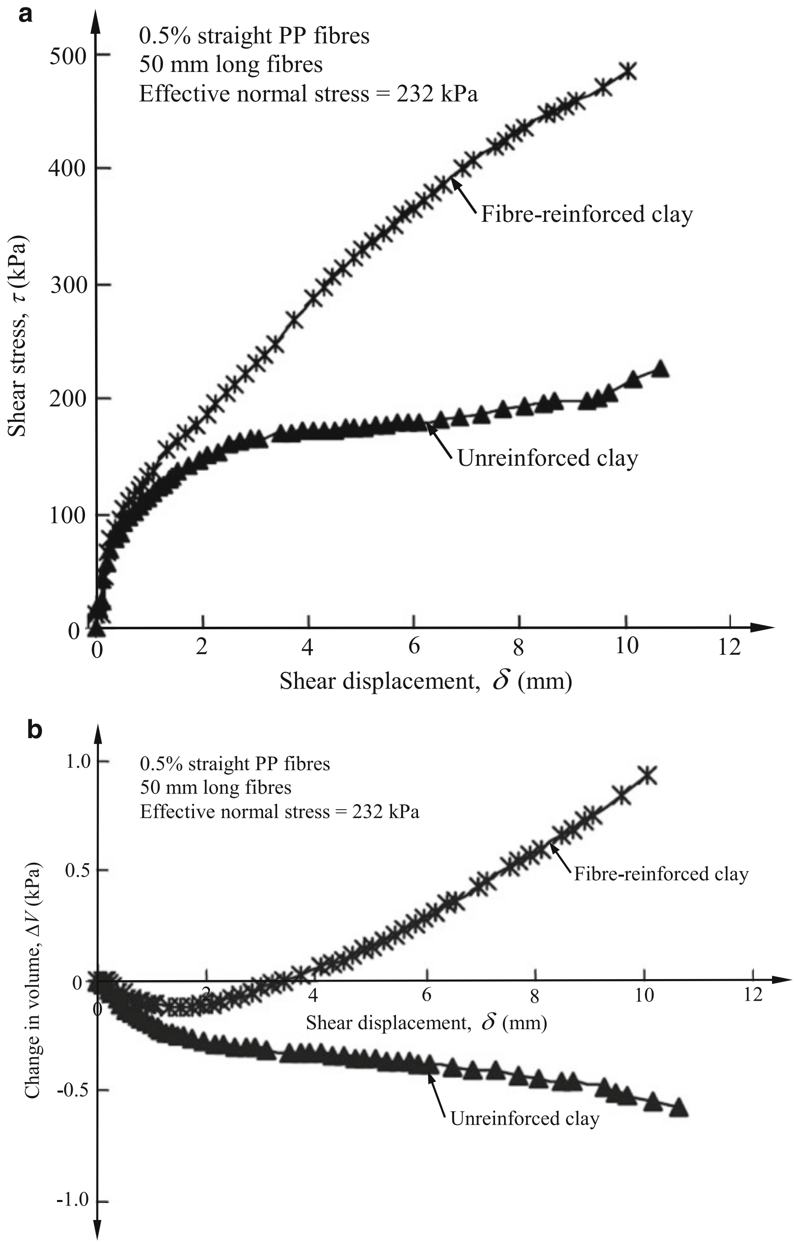

Fig. 3.3 Direct shear test results for fibre-reinforced clay: (**a**) shear stress τ versus shear displacement δ; (**b**) change in volume of the specimen ΔV versus shear displacement δ (Adapted from Falorca and Pinto 2011)

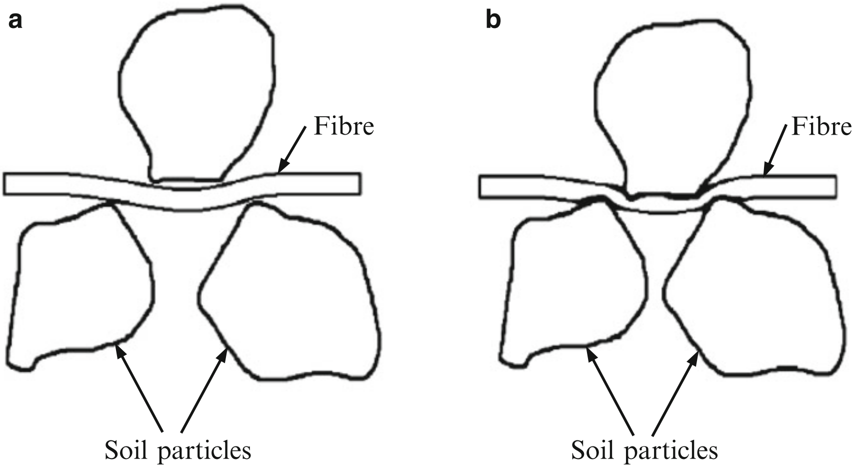

Fig. 3.4 Interaction mechanisms between fibre and soil particles: (**a**) soil particles being prevented by fibre from packing tightly; (**b**) soil particles causing fibre stretch and imprints on the fibre, thus allowing adhesion to develop (Adapted from Falorca and Pinto 2011)

is found to be more important for low normal stresses. No appreciable advantage is achieved by using the crimped (texturized) fibres as far as the shear strength is concerned. There is an increase in both apparent cohesion and the angle of shearing resistance of soils due to inclusion of fibres. The initial stiffness of the reinforced sand decreases with an increase in fibre content, whereas for reinforced clay, there is no significant change.

Falorca and Pinto (2011) also studied how the fibres interact with soil particles using both an optical microscope and a scanning electron microscope. Their study shows that the fibres do not rupture during shear, but they are stretched, and some damage may occur (Fig. 3.4). The damage is essentially fibre indentation, with hardly any cutting. It is believed that the damage mostly occurs during compaction as a result of high impact energy. The fibres lose their straightness and end up with many bends (angularities). Michalowski and Zhao (1996) have also suggested that the fibre kinking may occur during the deformation process. Although the fibres suffer from some permanent localised damage, they are still suitable for soil reinforcement, as the soil particles are held by the indentations in the fibres. The contact area between soil particles and fibres pressed against each other is proportional to the applied load until the fibres undergo plastic deformation (i.e. surface imprints). These imprints allow adhesion to take place within the reinforced soil mass. Note that the fibre prevents the soil particles from packing tightly until fibre stretch and imprinting occur, thus increasing the capacity to hold the particles and therefore allowing adhesion to develop. This mechanism also explains why fibre-reinforced soils have a lower unit weight than unreinforced soils.

3.3.2 Observations in Triaxial Tests

Based on the triaxial compression tests, the improvement in shear strength of soil as a result of inclusion of fibre reinforcement can be expressed in terms of the *deviator stress improvement factor* I_d, defined as

$$I_d = \frac{\Delta(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_{fU}} = \frac{(\sigma_1 - \sigma_3)_R - (\sigma_1 - \sigma_3)_{fU}}{(\sigma_1 - \sigma_3)_{fU}} = \frac{(\sigma_1 - \sigma_3)_R}{(\sigma_1 - \sigma_3)_{fU}} - 1 \quad (3.3)$$

where $(\sigma_1 - \sigma_3)_{fU}$ is the failure deviator stress of the unreinforced soil specimen, and $(\sigma_1 - \sigma_3)_R$ is the deviator stress of the fibre-reinforced soil specimen corresponding to the axial strain $\varepsilon(=\varepsilon_{fU})$ of the unreinforced soil specimen at failure. The factor I_d basically presents a relative gain in deviator stress of soil specimen due to inclusion of fibres and is normally expressed as a percentage.

Note that in Eq. (3.3), $(\sigma_1 - \sigma_3)_R$ can be the deviator stress of the reinforced soil specimen at any selected/permissible axial strain as required for the safety of the specific structure under consideration. For $(\sigma_1 - \sigma_3)_R = (\sigma_1 - \sigma_3)_{fR}$ at $\varepsilon = \varepsilon_{fR}$ (axial strain of reinforced soil specimen at failure), Eq. (3.3) reduces to the *deviator stress improvement factor at failure* I_{df}, given as

$$I_{df} = \frac{\Delta(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{fU}} = \frac{(\sigma_1 - \sigma_3)_{fR} - (\sigma_1 - \sigma_3)_{fU}}{(\sigma_1 - \sigma_3)_{fU}} = \frac{(\sigma_1 - \sigma_3)_{fR}}{(\sigma_1 - \sigma_3)_{fU}} - 1 \quad (3.4)$$

The improvement in shear strength of soil as a result of inclusion of fibre reinforcement can also be expressed in terms of *deviator stress ratio* (*DSR*) or *deviator stress ratio at failure* (DSR_f), as defined below:

$$DSR = \frac{(\sigma_1 - \sigma_3)_R}{(\sigma_1 - \sigma_3)_{fU}} \quad (3.5)$$

$$DSR_f = \frac{(\sigma_1 - \sigma_3)_{fR}}{(\sigma_1 - \sigma_3)_{fU}} \quad (3.6)$$

The *brittleness index*, as defined below, is also used to describe the behaviour of fibre-reinforced soil in triaxial compression (Maher and Ho 1993; Consoli et al. 1998a; Consoli et al. 2002):

$$I_b = \frac{(\sigma_1 - \sigma_3)_{peak} - (\sigma_1 - \sigma_3)_{ultimate}}{(\sigma_1 - \sigma_3)_{ultimate}} = \frac{(\sigma_1 - \sigma_3)_{peak}}{(\sigma_1 - \sigma_3)_{ultimate}} - 1 \quad (3.7)$$

where $(\sigma_1 - \sigma_3)_{peak}$ and $(\sigma_1 - \sigma_3)_{ultimate}$ are peak and ultimate deviator stresses, respectively. A lower value of I_b indicates that there is limited loss of post-peak strength in the triaxial compression, and the fibre-reinforced soil behaves as a ductile material.

The strain energy absorption capacity, which represents the ductility behaviour of the material, is often defined as the area under the stress-strain curve up to 10% strain. A higher strain energy absorption capacity indicates that the material is more ductile.

The factors/ratios/indices as defined here have been used directly or indirectly by the researchers to describe the effects of fibre inclusions on the soil characteristics.

Hoare (1979) presented the results of compaction and triaxial compression tests on dry, angular crushed sandy gravel mixed with polymeric fibres and small strips cut from a geotextile. The results show that the reinforcement offers resistance to densification and has beneficial effects on both strength and ductility, except when the increased amount of reinforcement results in reduced density, in which case the strength may even decrease.

The triaxial compression investigation by Gray and Al-Refeai (1986) indicates that the inclusion of natural and synthetic glass fibres in sand increases its strength (expressed as the major principal stress at failure) and modifies the stress-deformation behaviour with increased stiffness. The increase in strength with fibre content varies linearly up to a fibre content of 2% by weight and thereafter approaches an asymptotic upper limit. The rate of increase is roughly proportional to the fibre aspect ratio. At the same aspect ratio, the confining pressure and the weight fraction, roughness (not stiffness) of fibres tends to be more effective in increasing the strength. Fibre-reinforced specimens fail along a classic planar shear plane, whereas the sand reinforced with fabric (geotextile) layers fails by bulging between the fabric layers. The existence of a critical confining pressure is common to both the fibre-reinforced sand and the fabric-reinforced sand.

Maher (1988) performed laboratory triaxial compression tests on sand reinforced with discrete, randomly distributed glass fibres. The results presented by Maher (1988) and Maher and Gray (1990) indicate the following:

1. Sands reinforced with randomly distributed fibres exhibit either curved-linear or bilinear principal stress envelopes. Uniform, rounded sand exhibits the former behaviour (Fig. 3.5a), whereas the well-graded or angular sand tends to exhibit the latter (Fig. 3.5b). The break in the bilinear curve or transition from a curved to a linear envelope occurs at the threshold confining stress, which is referred to as the *critical confining stress*$\sigma_{3\text{crit}}$.
2. Failure surfaces in a triaxial compression test on randomly distributed fibre-reinforced sand are planar and oriented in the same manner as predicted by the Mohr-Coulomb failure criterion, viz. at the angle of obliquity, or $(45^\circ + \phi/2)$, where ϕ is the angle of internal friction of sand. This finding suggests an isotropic reinforcing action with no development of preferred planes of weakness or strength.
3. An increase in fibre aspect ratio results in a lower $\sigma_{3\text{crit}}$ and higher fibre contribution to shear strength. Fibres with very low modulus (e.g. rubber) contribute little to increased strength in spite of superior pullout resistance (low $\sigma_{3\text{crit}}$).

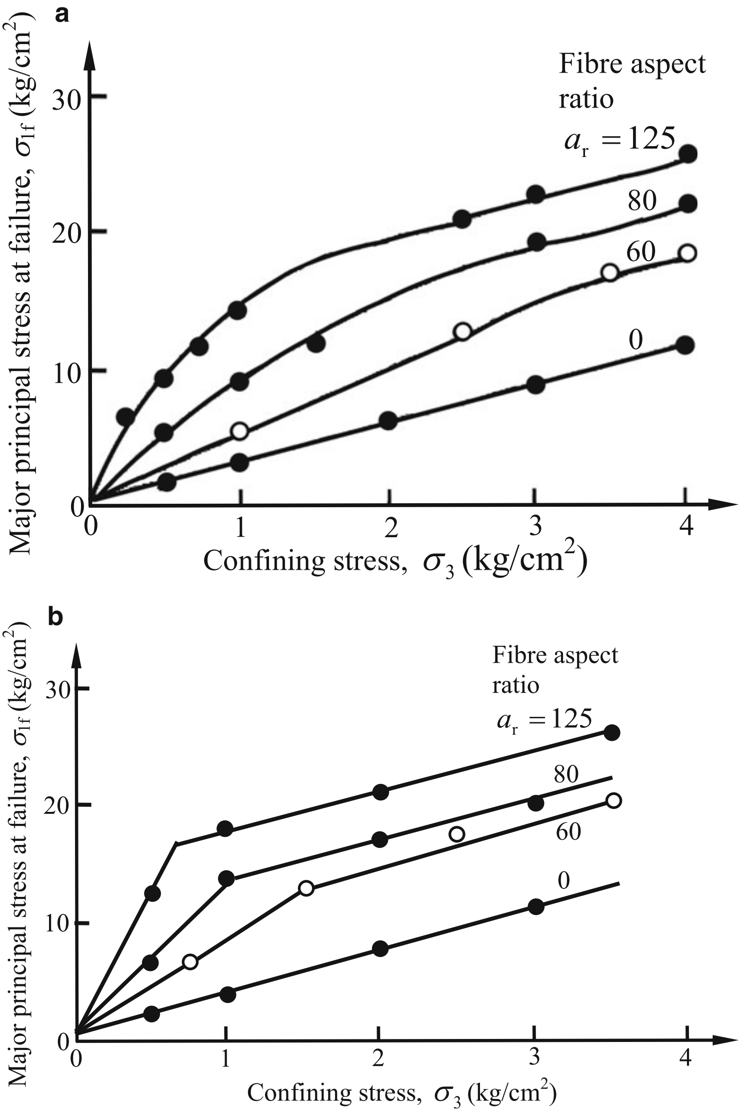

Fig. 3.5 Principal stress envelopes from triaxial compression tests on reinforced sands: (**a**) Muskegon dune sand; (**b**) mortar sand (Adapted from Maher and Gray 1990). (Note: Glass fibre content = 3%; 1 kg/cm^2 = 98.07 kPa)

4. Shear strength increases approximately linearly with increasing fibre content and then approaches an asymptotic upper limit governed mainly by confining stress and fibre aspect ratio (Fig. 3.6).
5. A better gradation or an increase in coefficient of uniformity of soil results in a lower σ_{3crit} and higher fibre contribution to strength, with all other factors being constant.

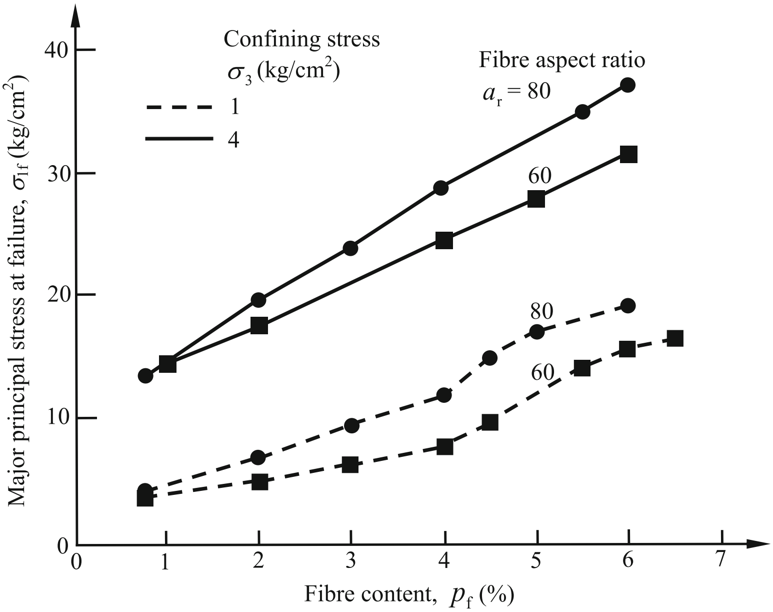

Fig. 3.6 Effect of glass fibre content and aspect ratio on strength increase in Muskegon sand at *low* and *high* confining stresses (Adapted from Maher and Gray 1990). (Note: 1 kg/cm^2 = 98.07 kPa)

6. An increase in soil particle size D_{50} has no effect on σ_{3crit}. However, it reduces the fibre contribution to strength, with all other factors being constant.
7. An increase in particle sphericity results in a higher σ_{3crit} and lower fibre contribution to strength, with all other factors being constant.

Al-Refeai (1991) conducted a series of triaxial tests to investigate the load-deformation behaviour of fine and medium sands reinforced randomly with discrete PP and glass fibres. The results show that shorter fibres ($\leq$25 mm) require a great confining stress to prevent bond failure regardless of size and shape of sand particles. Short fibres decrease the stiffness of medium sand. Soil-inclusion friction interaction depends mainly on the extensibility of the inclusion rather than the mechanical properties of sand. Fine sand with subrounded particles shows a more favourable response to fibre reinforcement than medium sand with subangular particles. The percentage increase in principal stress and secant modulus from the inclusion of glass fibres is directly proportional to fibre length for a constant fibre content. There is an optimum length (75 mm) of fibre for maximizing the strength and stiffness of fibre-reinforced fine and medium sands.

Ranjan et al. (1994, 1996) carried out a series of triaxial compression tests to study the stress-strain behaviour of plastic-fibre-reinforced poorly graded fine sand. The inclusion of plastic fibres (aspect ratio = 60–120, diameter = 0.3–0.5 mm, specific gravity = 0.92, tensile strength = 30 MPa, tensile modulus = 2 GPa, skin friction angle = 21°) causes an increase in peak shear strength and reduction in the loss of post-peak strength of sand. Figure 3.7 shows the stress-strain plot of the

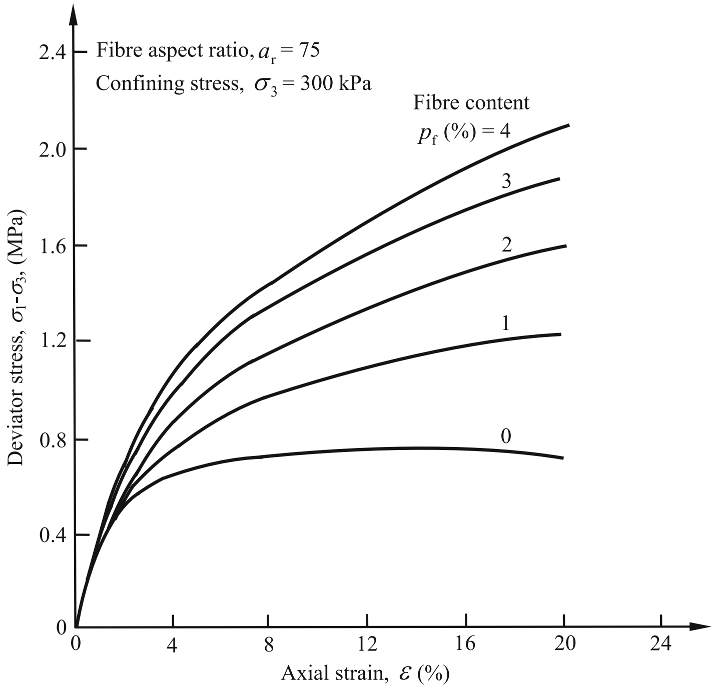

Fig. 3.7 Stress-strain behaviour of plastic fibre-reinforced fine sand (Adapted from Ranjan et al. 1994, 1996)

reinforced soil. Thus, the residual strength of fibre-reinforced sand is higher as compared to that of unreinforced sand. The principal stress envelopes for fibre-reinforced sand are bilinear having a break at a confining stress, called the critical confining stress, below which the fibres tend to slip or pullout. An increase in fibre aspect ratio results in a lower critical confining stress and higher contribution to shear strength, as also reported earlier by Maher and Gray (1990). Shear strength increases approximately linearly with increasing fibre content up to 2% by weight, beyond which the gain in strength is not appreciable.

Maher and Ho (1993) presented the effect of randomly distributed glass fibre reinforcement on the response of cemented standard Ottawa sand under both static and cyclic loads by conducting triaxial static and cyclic compression tests on 28-days cured specimens at a constant void ratio of 0.62. The test results, as presented in Fig. 3.8, indicate that the addition of fibres significantly increases the peak compressive strength of cemented sands. The study also shows the following:

1. The increase in peak compressive strength is more pronounced at higher fibre contents.
2. The contribution of fibres to increased strength is more for longer fibre lengths and lower confining stresses. Apparently at higher confining stresses, the soil is

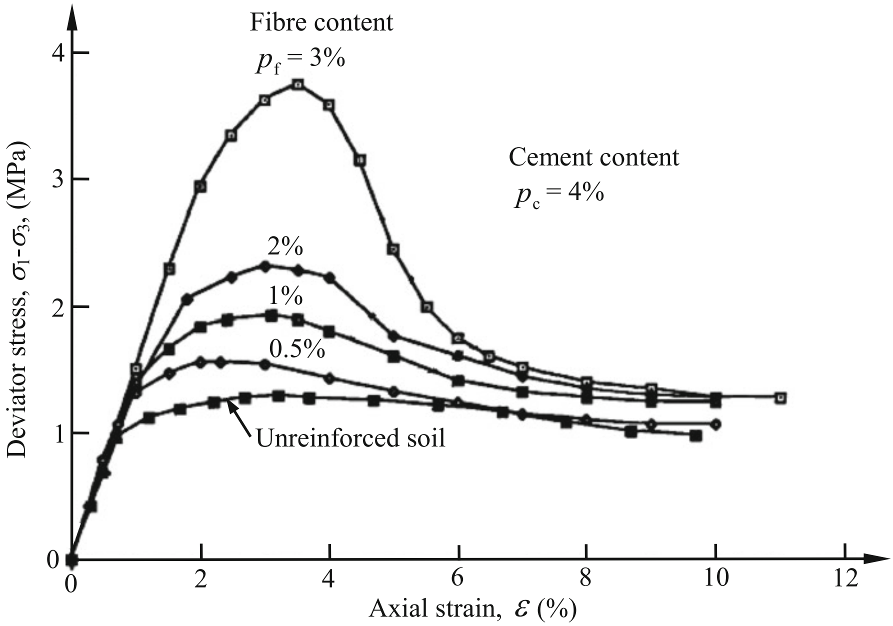

Fig. 3.8 Effect of glass fibre inclusion on stress-strain behaviour of cement-stabilized standard Ottawa sand after 28 days of curing (Adapted from Maher and Ho 1993)

already stiff and the cement or fibre's contribution to strength is not effective as observed under low confining stress.

3. The brittleness index increases with increasing fibre content and length, but decreases with increasing confining stress.
4. The addition of fibres significantly increases the energy absorption capacity of cement-stabilized sand. An increase in energy absorption is more pronounced at higher fibre contents, longer fibre lengths and lower confining stresses.
5. In the triaxial compression test, no significant volume change takes place during the cell pressure application, while during the shearing stage, a strong volume expansion is observed under low confining pressure, which diminishes with increasing confining pressure.
6. In comparison to cement-stabilized sand, where the addition of cement does not affect the angle of internal friction and only increases the cohesion intercept (Clough et al. 1981), the fibre inclusion increases both the angle of internal friction and the cohesion intercept of cement-stabilized sand as the peak shear strength parameters obtained in the tests indicate (see Table 3.2). Increasing the fibre length (or aspect ratio) results in increased angle of internal friction angle and cohesion intercept of cement-stabilized sand. The ultimate shear strength parameters are relatively consistent. With the addition of 3% fibres to cemented sand, the ultimate angle of internal friction angle increases from 32° to 39°, and ultimate cohesion intercept increases from 20 to 250 kPa.

Table 3.2 Peak shear strength parameters (After Maher and Ho 1993)

Cement-stabilized sand without and with fibres (admixture % by weight)	Angle of internal friction (degrees)	Cohesion intercept (kPa)
Sand + 4% cement	37	103
Sand + 4% cement + 0.5% fibres	41	104
Sand + 4% cement + 1% fibres	43	152
Sand + 4% cement + 2% fibres	45	206
Sand + 4% cement + 3% fibres	49	363

7. The initial tangent modulus E_i for cement-stabilized fibre-reinforced sand increases approximately linearly with increasing confining stress σ_3 on log-log scales when both are normalized to atmospheric pressure p_a. Similar to cement-stabilized sands (Clough et al. 1981), the following relationship holds good:

$$E_\mathrm{i} = kp_\mathrm{a}\left(\frac{\sigma_3}{p_\mathrm{a}}\right)^n \tag{3.8}$$

   where k is the intercept at $\sigma_3/p_\mathrm{a} = 1.0$ and n is the slope of the line of variation of $E_\mathrm{i}/p_\mathrm{a}$ versus σ_3/p_a on log-log scale. As the fibre content increases from 0.5 to 3%, the value of k increases from 700 to 1150, while the value of n decreases from 0.41 to 0.24, respectively.

8. The addition of fibres significantly increases the cyclic strength of cement-stabilized sand. The number of cycles and the magnitude of strain required to cause failure in cement-stabilized sand increases significantly as a result of fibre inclusion.

Michalowski and Zhao (1996) conducted a series of triaxial tests on coarse, poorly graded sand reinforced isotropically with galvanized or stainless steel (specific gravity = 7.85) and polyamide monofilament (specific gravity = 1.28). Compaction of the specimens was characterized by the void ratio as the relative density is not an appropriate parameter for characterizing the fibre-reinforced specimens, because the minimum and maximum void ratios of the reinforced soil are very much dependent on the fibre characteristics. In all tests, the initial void ratio of prepared specimens was 0.66, corresponding to a relative density of 70% of unreinforced sand. They reported the stress-strain behaviour similar to that shown in Fig. 3.7. The results show that the addition of steel fibres to sand leads to an increase in the peak deviator/shear stress of about 20% ($p_\mathrm{vf} = 1.25\%$, $a_\mathrm{r} = 40$) for specimens tested under a confining pressure of 100–600 kPa; this relative increase is larger at a very low confining pressure (50 kPa). The specimens exhibit a typical compaction effect at small axial strain and dilation at larger strains. The presence of fibres inhibits the dilation effect to a certain degree. The increase in the content of steel fibres leads to a clear increase in the peak deviator/shear stress and also leads to an increase in the stiffness of the reinforced soil prior to reaching the failure. An increase in the aspect ratio of the fibres contributes to a significant increase in the peak deviator/shear stress. Inclusion of polyamide fibres leads to similar effects with some deviation. Polyamide fibres produce a significant increase in the peak deviator/shear stress for larger confining pressures, but the effect is associated with a considerable loss of stiffness prior to failure and a substantial increase of strain to failure. At a confining pressure of 100 kPa, however, no increase of the peak deviator/shear with respect to the sand alone is recorded.

Kaniraj and Havanagi (2001) conducted unconsolidated undrained (UU) triaxial compression tests on cylindrical specimens (diameter = 37.7 mm, length = 73.5 mm) of Rajghat fly ash, Delhi silt, mixture of 50% Rajghat fly ash and 50% Delhi silt and mixture of 50% Rajghat fly ash and 50% Yamuna sand with and without random distribution of PET fibre reinforcement, compacted at respective maximum dry unit weight-optimum water content states. Confining stresses in the range of 24.5–392.3 kPa were used. The tests results suggest the following:

1. In unreinforced specimens, the deviator strain attains a peak and thereafter remains constant. The strain to attain the peak deviator stress increases with an increase in confining stress.
2. In fibre-reinforced specimens, no peak deviator stress is reached even at 15% axial strain. This may be a manifestation of the ductile behaviour induced by the fibre inclusions.
3. Table 3.3 provides the values of total stress-strength parameters c_{uu} and ϕ_{uu}. The data in this table show that there is a significant increase in both c_{uu} and ϕ_{uu} due to fibre inclusions.
4. The failure envelope plotted as the variation of the major principal stress at failure (15% axial strain) σ_{1f} with confining stress σ_3 is bilinear for the fibre-reinforced fly ash-soil specimens. The initial steeper portion is characterized by the pullout failure of the fibres and the second linear portion by the rupture of the fibres, as reported by Maher and Gray (1990).
5. The value of I_{df} increases as $(\sigma_1 - \sigma_3)_{fU}$ increases, as per the following correlation:

Table 3.3 Shear strength parameters, based on UU triaxial compression tests, of fly ash and soil mixtures with fibre content (defined as the ratio of weight of fibre solids to sum of weights of soil solids and fly ash solids), $p_f = 1\%$, for the reinforced cases (After Kaniraj and Havanagi 2001)

	Unreinforced		Fibre reinforced	
Soil type	c_{uu} (kPa)	ϕ_{uu} (degrees)	c_{uu} (kPa)	ϕ_{uu} (degrees)
Fly ash	43.2[a]	30.2[a]	128.6[a]	36.1[a]
Mixture of 50% fly ash and 50% silt	25.8	30.2	93.3[b]	40.0[b]
Mixture of 50% of fly ash and 50% sand	16.5	30.4	160.0[b]	32.9[b]

[a]For $\sigma_3 > 50$ kPa
[b]For $\sigma_3 > 25$ kPa

$$I_{df} = 16809\left[(\sigma_1 - \sigma_3)_{fU}\right]^{-0.8059} \tag{3.9}$$

The standard error in logI_{df} and the coefficient of determination R^2 of the correlation are 0.073 and 0.885, respectively.

6. Even though the fibre inclusions increase the deviator stress at large strain, they do not necessarily increase the stiffness at small strain. The values of secant modulus E_s calculated at $(\sigma_1 - \sigma_3)_{fR}/2$ and $(\sigma_1 - \sigma_3)_{fU}/2$ show that the fibre inclusions decrease E_s of fly ash-soil specimens, but this trend is not consistent at all confining stresses in pure fly ash.

Consoli et al. (2002) carried out the drained triaxial compression tests on a uniform fine sand (SP) reinforced with PET fibres, with and without rapid-hardening Portland cement. The results show that with the inclusion of fibres and increase in fibre length, the cohesive intercept of sand does not change, whereas the friction angle increases. The fibre reinforcement does not affect the initial stiffness or ductility of the uncemented sand. As expected, the addition of cement to the sand significantly increases the stiffness and peak strength and changes the soil behaviour from ductile to a noticeable brittle one. The cement content increases both the peak friction angle and the cohesive intercept. The initial stiffness of the cemented sand is not affected by fibre inclusion, because it is basically a function of cementation. The efficiency of the fibre reinforcement when applied to cemented sand is found to be dependent on the fibre length. The greatest improvements in triaxial strength, ductility and energy absorption capacity (defined as the amount of energy required to induce deformation and is equal to area below stress-strain curve) are observed for the longer (e.g. 36 mm) fibres.

Babu et al. (2008a) carried out standard triaxial compression tests on coir fibre-reinforced sand for several fibre contents (0, 0.5, 1 and 1.5% by dry weight of sand). The total unit weight of soil specimens in all tests was kept at a constant value of 14.8 kN/m^3. The results as presented in Fig. 3.9 show that the stress-strain response of soil is improved considerably by the addition of coir fibres.

Babu and Vasudevan (2008a) also reported the following characteristics of coir fibre-reinforced soil based on the standard triaxial compression test results:

1. The deviator stress at failure can increase up to 3.5 times over unreinforced/plain soil by fibre inclusion.
2. Maximum increase of stress is observed when the fibre length is between 15 and 25 mm, that is, when the length is 40–60% of the least lateral dimension of the triaxial test specimen.
3. Stiffness of soil increases considerably due to fibre inclusion; hence immediate settlement of soil can be reduced by incorporating fibres in the soil.
4. Energy absorption capacity of fibre-reinforced soil increases as the fibre content increases, and hence toughness of soil can be increased with fibre inclusion.

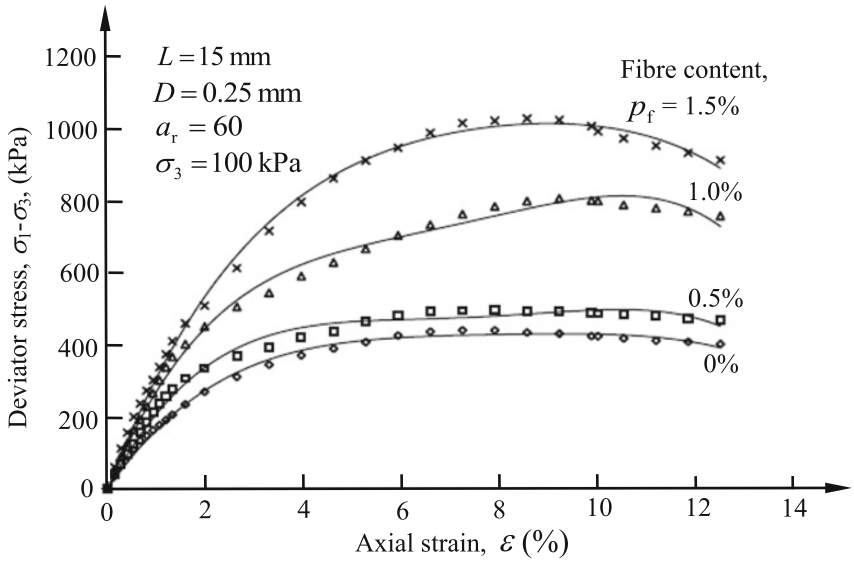

Fig. 3.9 Stress versus strain curves for coir fibre-reinforced sand (Adapted from Babu et al. 2008a)

Babu et al. (2008b) examined the use of coir fibres as reinforcement to improve the engineering properties of black cotton soil (expansive soil) by conducting standard undrained triaxial compression tests. The deviator stress at failure increases with increase in the fibre content as well as with increase in the diameter of the fibres. The maximum increase of major principal stress at failure is found to be 1.30 times over the unreinforced soil. The coir fibres help reduce the swell potential of black cotton soil.

Kumar and Singh (2008) conducted triaxial compression tests to study the behaviour of fly ash reinforced with PP fibres. The results, as presented in Table 3.4, show that the deviator stress at failure increases with an increase in fibre content at a particular cell pressure; but the increments are more significant up to 0.3% fibre content, and the gain in deviator stress is reduced for high fibre contents. The deviator stress also increases with an increase in aspect ratio, but the gain in deviator stress is lower for the aspect ratio more than 100. Based on the cyclic triaxial tests, it is observed that the resilient modulus of fly ash increases due to inclusion of fibres in fly ash.

In triaxial compression, a considerable increase of shear strength is contributed by the presence of fibres, while in extension, the benefit of fibres is very limited. This behaviour confirms that the tamping technique in the moist condition generates preferential near-horizontal orientation of fibres, that is, the anisotropic distribution of fibre orientation (Diambra et al. 2010).

The PP crimped fibres in loose sandy soil reduce the potential for the occurrence of liquefaction in both compression and extension triaxial loadings and convert a

Table 3.4 Variation of deviator stress at failure with aspect ratio, cell pressure and fibre content for fibre-reinforced fly ash (After Kumar and Singh 2008)

Fibre aspect ratio	Cell pressure (kPa)	Deviator stress (kPa) at failure with fibre content (%)					
		0.0	0.1	0.2	0.3	0.4	0.5
80	40	162	212	262	300	320	338
	70	184	267	334	382	405	422
	100	210	92	362	426	453	471
	140	242	320	405	462	498	543
100	40	162	259	320	368	386	398
	70	184	287	359	405	426	450
	100	210	315	392	443	469	487
	140	242	332	424	486	520	563
120	40	162	262	322	367	393	407
	70	184	294	368	409	438	456
	100	210	320	398	451	476	493
	140	242	338	427	490	527	568

strain-softening response into a strain-hardening response. When the full liquefaction of reinforced specimens is induced by strain reversal, the lateral spreading of soil seems to be prevented (Ibraim et al. 2010).

Hamidi and Hooresfand (2013) conducted conventional triaxial compression tests to determine the effect of cement and PP fibres on the strength behaviour of clean and uniform beach sand. The test results show that addition of PP fibres to the cemented soil causes the following:

1. Increase in peak and residual shear strengths as a result of increase in angle of internal friction angle and cohesion intercept. The effect of fibre content on shear strength is greater at higher relative densities of sand.
2. Decrease in initial stiffness and brittleness indexI_b.
3. Increase in energy absorption capacity. This indicates that the fibres change the brittle behaviour to ductile behaviour; the change is more for greater relative density under higher confinement.

Mixing of synthetic fibres (PP fibres and geogrid waste fibres) increases the strength of rice husk ash (RHA), but there exists an optimum fibre content of 1.25% at which the reinforcement benefits are maximum. The stress-strain behaviour of RHA improves considerably due to an increase in fibre content. The secant modulus of reinforced RHA increases with an increase in fibre content up to 1.25%, and thereafter it decreases. The shear strength parameters (cohesion intercept and angle of internal friction) also increase with an increase in fibre content up to 1.25%, and thereafter they decrease (Jha et al. 2015).

Example 3.1
Consider the following values of the deviator stress at failure:

For unreinforced soil, $(\sigma_1 - \sigma_3)_{\mathrm{fU}} = 162\ \ \mathrm{kPa}$
For fibre-reinforced soil, $(\sigma_1 - \sigma_3)_{\mathrm{fR}} = 338\ \ \mathrm{kPa}$

Determine the deviator stress improvement factor at failure. What does this indicate?

Solution
From Eq. (3.4), the deviator stress improvement factor is

$$I_{\mathrm{df}} = \frac{(\sigma_1 - \sigma_3)_{\mathrm{fR}}}{(\sigma_1 - \sigma_3)_{\mathrm{fU}}} - 1 = \frac{338}{162} - 1 \approx \mathbf{1.09} \ \text{ or } \ \mathbf{109\%}$$

This value of I_{df} shows that inclusion of fibres into the soil causes 109% increase in the deviator stress at failure.

3.4 Unconfined Compressive Strength

The improvement in unconfined compressive strength (*UCS*) of soil as a result of inclusion of fibre reinforcement can be expressed in terms of the *unconfined compressive strength improvement factor* I_{UCS}, defined as

$$I_{UCS} = \frac{q_{\mathrm{UR}} - q_{\mathrm{UU}}}{q_{\mathrm{UU}}} = \frac{q_{\mathrm{UR}}}{q_{\mathrm{UU}}} - 1 \tag{3.10}$$

where q_{UR} is the *UCS* of fibre-reinforced soil and q_{UU} is the *UCS* of unreinforced soil. Basically, the factor I_{UCS} presents the relative gain in *UCS* of soil due to inclusion of fibres and is normally expressed as a percentage. Note that $I_{UCS} = I_{\mathrm{df}}$ for $\sigma_3 = 0$ in Eq. (3.4).

Note that the unconfined compressive strength improvement factor can be defined for the same selected/permissible axial strains for reinforced and unreinforced soils as the shear strength improvement factors have been defined earlier. Hence, the unconfined compressive strength improvement factor can also be expressed as:

$$I_{UCS} = \frac{q_{\mathrm{R}} - q_{\mathrm{UU}}}{q_{\mathrm{UU}}} = \frac{q_{\mathrm{R}}}{q_{\mathrm{UU}}} - 1 \tag{3.11}$$

where q_{R} is the unconfined compressive stress on the fibre-reinforced soil specimen corresponding to the axial strain $\varepsilon (= \varepsilon_{\mathrm{fU}})$ of the unreinforced soil specimen at failure. The factor I_{UCS} basically presents the relative gain in shear strength of soil due to inclusion of fibres and is normally expressed as a percentage.

Freitag (1986) investigated the influence of synthetic fibre inclusions (diameter = 0.1–0.2 mm, length = 20 mm) on the *UCS* of a compacted fine-grained soil (lean sandy clay, CL). The test specimens for the unconfined compression tests were compacted over a sufficiently wide range of water content to define the compaction curve. The soil was mixed with water, and then the desired amount of fibres was added and mixed into the soil. Fibre content was 1% by volume. The unconfined compression tests were performed immediately after compaction. The compacted soil layers were thin relative to the length of the fibres, so in the final compacted state, the fibres were not truly randomly oriented in the specimens. The results indicate the following:

1. *UCS* of the reinforced soil compacted near and wet of optimum is greater than for unreinforced soil at the same water content. For specimens compacted well on the dry side of optimum, there does not seem to be any benefit from the presence of the fibres.
2. The type of fibre used does not seem to have a significant effect on strength.
3. The maximum *UCS* occurs somewhat dry of optimum and is not greatly different for unreinforced and reinforced soils.
4. At higher water contents, the strength of the unreinforced soil decreases more rapidly than that of the reinforced soil. An examination of the stress-strain curves (Fig. 3.10) for specimens at the same water content shows that usually the reinforced soil fails at a greater strain than the unreinforced soil. This is most often the case for wet-side soil.
5. The stress-strain relations (Fig. 3.10) are similar at very low strains, but the reinforced soil is able to hold together for more deformation and therefore higher stress at rupture. Thus, the reinforced soil is also able to absorb more energy and hence is ductile.

The laboratory investigation carried out by Santoni et al. (2001) indicates that an inclusion of randomly oriented discrete PP fibres (synthetic monofilament, fibrillated, tape and mesh fibres) significantly improves the *UCS* of sands. An optimum fibre length of 51 mm (2 in.) was identified for the reinforcement of sand specimens. A maximum performance is achieved at the fibre content between 0.6 and 1% by dry weight. The specimen performance is enhanced in both wet and dry of optimum conditions. The inclusion of up to 8% of silt does not affect the performance of the fibre reinforcement.

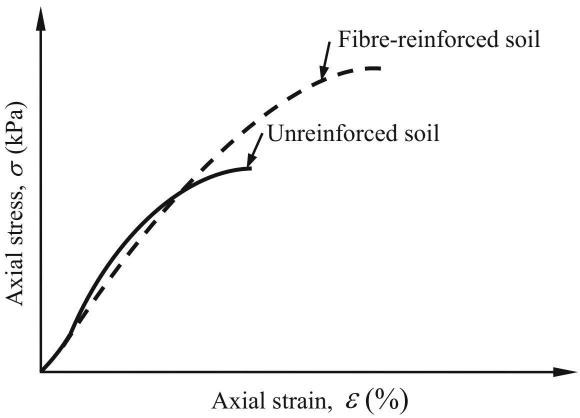

Fig. 3.10 Typical unconfined compression stress-strain curves for unreinforced and fibre-reinforced soils (Adapted from Freitag 1986).

Table 3.5 Average *UCS* of fly ash, silt and soil mixtures with fibre content (defined as the ratio of weight of fibre solids to sum of weights of soil solids and fly ash solids), $p_{\mathrm{f}} = 1\%$, for the reinforced cases (After Kaniraj and Havanagi 2001)

Soil type	q_{UU}(kPa)	q_{UR}(kPa)	I_{UCS}(%) (Eq. (3.10))
Fly ash	65.7	157.9	140
Silt	36.3	411.9	1035
Mixture of 50% fly ash and 50% silt	47.1	304.0	545
Mixture of 50% of fly ash and 50% sand	33.3	436.4	1211

Note: *UCS* values for fibre-reinforced specimens are the values of axial stress at 15% axial strain.

Kaniraj and Havanagi (2001) conducted unconfined compression tests on cylindrical specimens (diameter = 37.7 mm, length = 73.5 mm) of Rajghat fly ash, Delhi silt, mixture of 50% Rajghat fly ash and 50% Delhi silt and mixture of 50% Rajghat fly ash and 50% Yamuna sand with and without random distribution of PET fibre reinforcement, compacted at respective maximum dry unit weight-optimum water content states. The axial stress-axial strain behaviour was markedly affected by the fibre inclusions. In unreinforced specimens, a distinct failure stress was reached at an axial strain of about 2%, whereas the fibre-reinforced soil specimens exhibited more ductile behaviour, and no distinct reduction in axial stress was evident even at 15% axial strain. Table 3.5 presents the *UCS* of reinforced and unreinforced specimens. You may notice that the unreinforced fly ash has a higher *UCS* than silt and fly ash-soil mixtures. However, the inclusion of fibres improves the strength of silt and fly ash-soil mixtures so much that their *UCS* even surpasses that of the fibre-reinforced fly ash. The results in Table 3.5 show that I_{UCS} decreases as q_{UU} increases. They developed the following correlation between I_{UCS} and q_{UU} with standard error in $\log I_{UCS}$ and coefficient of deamination R^2 of the correlation to be 0.126 and 0.989, respectively:

$$I_{UCS} = 9038.3e^{-0.0619q_{\mathrm{UU}}} \tag{3.12}$$

Equation (3.2) is applicable for specific soils and fibre content as used in the experimental investigation by Kaniraj and Havanagi (2001).

Kaniraj and Havanagi (2001) also studied the effect of addition of 3% cement on the unconfined strength behaviour of fly ash-silty soil mixtures reinforced with 1% fibre. The results show that the *UCS* of a fly ash-soil mixture increases due to addition of cement and fibres. Depending on the type of mixture and curing period, the increase in *UCS* caused by the combined action of cement and fibres is either more than or nearly equal to the sum of the increases caused by them individually.

For an intermediate cement content of 5%, an increase in fibre content from 0.1 to 0.9% causes average increases in *UCS* of sand by about 40%. The initial stiffness of the cemented sand was not affected by fibre inclusion, since it is basically a function of cementation. The positive effect of fibre length is generally not detected in the unconfined compression tests, clearly indicating the major influence of confining pressure and the necessity of carrying out triaxial compression tests to fully observe the fibre-reinforced soil behaviour (Consoli et al. 2002).

Tang et al. (2007b) conducted a series of unconfined compression test on clayey soil cylindrical specimens (diameter = 39.1 mm, length = 80 mm) with inclusion of different contents of PP fibres (12 mm long) and ordinary Portland cement. All the test specimens were compacted at their respective maximum dry unit weight and optimum water content. The specimens treated with cement were wrapped with plastic membrane in the curing box for 7, 14 and 28 days, and they were submerged under water for 24 h at the last day of each curing period. Figures 3.11a, 3.11b and 3.11c show the stress-strain curves for fibre-reinforced uncemented soil, cemented soil and fibre-reinforced cemented soil with 5% cement after 28 days of curing. Note that in Fig. 3.11, the axial stress is basically the unconfined compressive stress. The peak/maximum value of axial stress is the *UCS*. The following are worth mentioning:

1. Fibre inclusion with 0.05% fibre content enhances the unconfined compressive/peak strength of uncemented soil, but the contribution of further increase of fibre content to unconfined compressive strength decreases significantly (Fig. 3.11a).

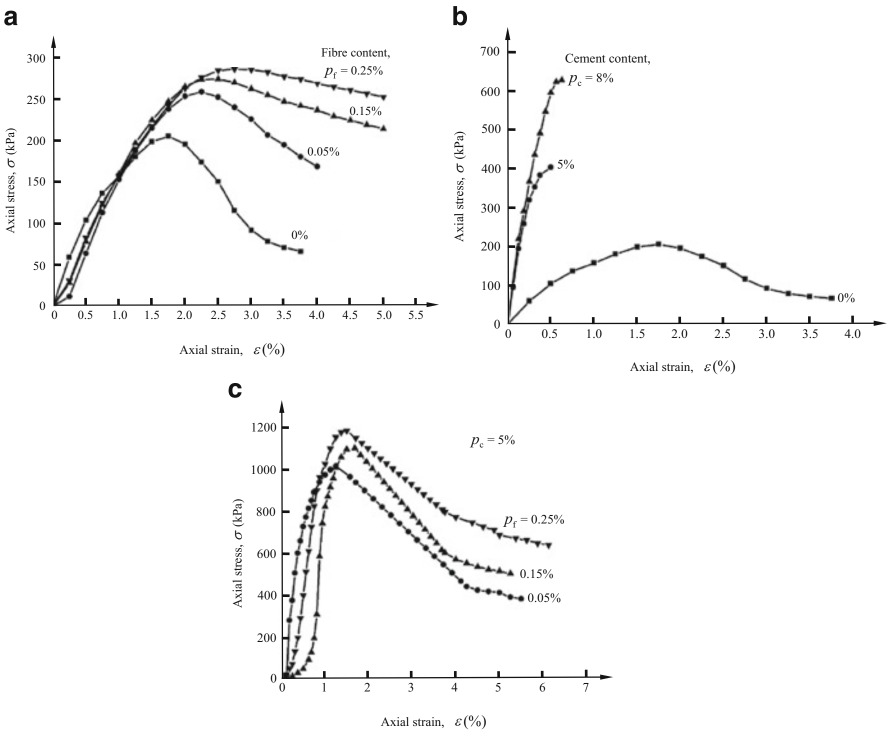

Fig. 3.11 Stress-strain curves obtained from unconfined compression tests: (**a**) fibre-reinforced uncemented clayey soil; (**b**) cement-stabilized clayey soil after 28 days of curing; (**c**) fibre-reinforced cemented clayey soil with cement content $p_c = 5\%$ after 28 days of curing (Adapted from Tang et al. 2007b)

2. The fibre-reinforced uncemented soil exhibits more ductile behaviour and smaller loss of post-peak strength than uncemented soil. The reduction in loss of post-peak strength is more pronounced for higher fibre content (Fig. 3.11a).
3. The initial stiffness of uncemented soil is not affected significantly by the addition of fibres (Fig. 3.11a).
4. The unconfined compressive strength increases dramatically with an increase in cement content, and the cemented soil exhibits a marked stiffness and brittleness, resulting in a sudden drop of post-peak strength to zero. Its failure strain is 0.5–0.75% (Fig. 3.11b), which is much smaller than that of uncemented soil and fibre-reinforced cemented soil.
5. The inclusion of fibres within the cemented soil reduces the brittleness with a gradual reduction of post-peak strength. The failure strain increases and ranges from 1.25 to 1.7% (Fig. 3.11c).

Note that the failure mechanism of cemented soil is triggered by the formation of noticeable wide and long tension cracks. With the presence of fibres in cemented soil, the tension cracks become gradually narrower and shorter with increasing fibre content (Fig. 3.12). In fact, the fibres added to cemented soil serve as bridges and impede the opening and development of cracks, thereby changing the brittle behaviour to ductile behaviour.

Kumar and Singh (2008) reported the improvement in unconfined compressive strength of fly ash with random inclusion of PP fibres. At an aspect ratio of 100, the unconfined compressive strength of fly ash increased from 128 to 259 kPa with increment in fibre content from 0 to 0.5%. The results show that the variation of unconfined compressive strength with fibre content is linear, while with aspect ratio, the variation is nonlinear. The optimum fibre length and aspect ratio were found as 30 mm and 100, respectively.

Park (2009) examined the effect of fibre concentration and distribution on the *UCS* of fibre-reinforced cemented sand. A series of unconfined compression tests were performed on artificially cemented sand specimens with layered polyvinyl alcohol (PVA) fibre reinforcement. The fibres were randomly reinforced at a predetermined layer among five compacted layers in the cylindrical soil specimens

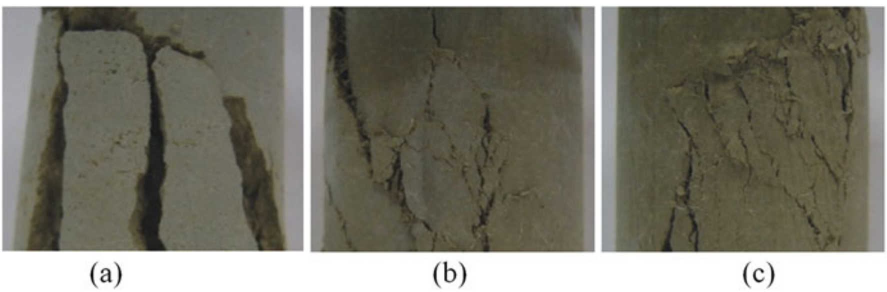

Fig. 3.12 Effect of fibres on the failure pattern of cement-stabilized clayey soil in unconfined compression tests with cement content, $p_c = 8\%$: (**a**) fibre content, $p_f = 0\%$; (**b**) $p_f = 0.05\%$; (**c**) $p_f = 0.25\%$ (After Tang et al. 2007b)

(length = 140 mm; diameter = 70 mm). The results show that the *UCS* of the reinforced soil increases gradually as the number of fibre inclusion layers increases. A fibre-reinforced specimen, where fibres were evenly distributed throughout the five layers, was twice as strong as an unreinforced cemented specimen. Using the same fibre content of 1% (as per Eq. (2.6)) to reinforce two different specimens, a specimen with five fibre inclusion layers was 1.5 times stronger than a specimen with one fibre inclusion layer at the middle of the specimen. Note that the cement content as per Eq. (2.9) was 4% in the study.

Zaimoglu and Yetimoglu (2012) investigated the effects of randomly distributed PP fibre reinforcement (length = 12 mm; diameter = 0.05 mm) on the *UCS* of a fine-grained soil (MH, high plasticity soil) by conducting a series of unconfined compression tests. The test results show that the *UCS* of soil tends to increase with increasing fibre content as shown in Fig. 3.13. However, the rate of increase in *UCS* is not significant for a fibre content greater than 0.75%. Compared to the unreinforced soil specimen, the *UCS* of the reinforced soil specimen at 0.75% fibre content increases by approximately 85% (i.e. from 392 to 727 kPa). The increase in *UCS* might be due to the bridging effect of fibre which can efficiently impede the further development of failure planes and deformations of the soil, as Tang et al. (2007b) also discussed in their study. The shape of the soil specimens after the tests indicates the bulging failure mode along with presence of the bridging effect of fibres (Fig. 3.14).

Mirzababaei et al. (2013) studied the effect of inclusion of carpet waste fibres on the *UCS* of clay soils. The test results suggest the following:

1. The *UCS* of fibre-reinforced clay soil is highly dependent on dry unit weight, water content and fibre content. At a constant dry unit weight and water content, an increase in the fibre content results in a significant increase in the *UCS* value. However, unreinforced and fibre-reinforced clay soil specimens prepared at their

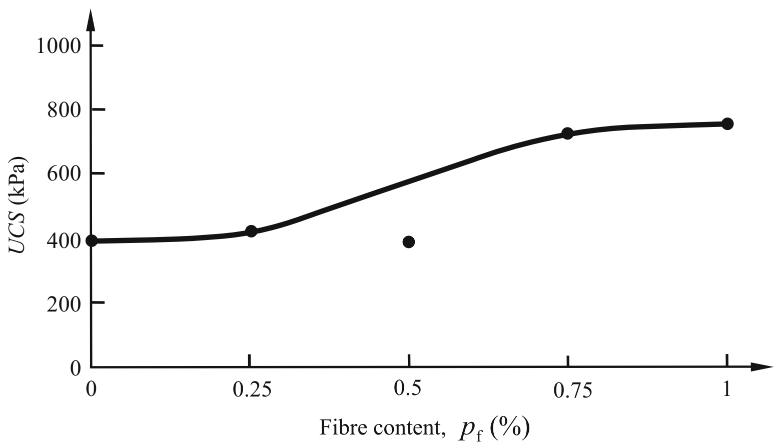

Fig. 3.13 Effect of fibre content on unconfined compressive strength of fine-grained soil (Adapted from Zaimoglu and Yetimoglu 2012)

Fig. 3.14 Fibre-reinforced fine-grained soil specimen after unconfined compression test – bulging failure with bridging effect of fibres (After Zaimoglu and Yetimodlu 2012)

respective maximum dry unit weight and optimum water content show a reduction in the *UCS* value with an increase in fibre content.
2. An increase in the dry unit weight of reinforced clay specimens prepared at a constant water content and fibre content results in a significant increase in *UCS*.
3. An increase in water content of reinforced specimens at the same dry unit weight and fibre content results in a reduction in the *UCS*.
4. A combined increase in dry unit weight and water content at a constant fibre content results in an increase in the *UCS*.
5. Unreinforced soil specimens show the brittle behaviour and fail at a very small axial strain (less than 1%), whereas reinforced specimens at 5% fibre content fail at a relatively large axial strain (15% or more) with strain hardening and ductile behaviour.
6. Failure patterns of unreinforced soil specimens are evident as nearly vertical shear planes. With an increase in fibre content, particularly at 5% fibre content, the failure pattern is gradually transformed to plastic bulging with networks of tiny cracks without an apparent shear failure plane.

The following points regarding the strength aspects of fibre-reinforced soils are worth mentioning:

1. The unconfined compression test data indicate that 19.05 mm (0.75 in. might be a minimum length of PP fibres to achieve some form of strength enhancement of sandy soil. In general, increases in *UCS* on the order of 2–2.5 times that of unreinforced soil can be realized at a PP fibre content of 0.1%. Comparable strength increases are not gained for the glass fibre-reinforced soil until the fibre content of 0.7% is realized. This high fibre content is difficult to handle in the laboratory preparation of specimens, and this fibre content may be equally unworkable in the field. No consistent trends are observed regarding vertical strain moduli in the unconfined compression test. The modulus value varies from positive to reduction. Fibres significantly increase the modulus of toughness as determined from the unconfined compression test, serving as an indicator of how fibres influence the ductility of the soil (Hoover et al. 1982).

2. When tested in saturated condition in cyclic triaxial compression tests, in comparison to sand reinforced with PP fabric, fine steel wire mesh and nylon netting, the sand reinforced with randomly distributed fine PP fibres exhibits a relatively higher increase in resistance to liquefaction (Uzdavines 1987; Noorany and Uzdavines 1989).
3. The mechanism of failure of cylindrical soil specimens in unconfined compression tests may differ from that in triaxial compression tests. The fibres are likely to get pulled out in the unconfined compression tests, whereas in the triaxial compression tests, they may tend to attain their tensile yield stress at higher confining stresses as suggested by Maher and Gray (1990). Probably due to this difference in failure mechanisms, the values of I_{df} in triaxial compression tests are not as high as the values of I_{UCS} in unconfined compression tests (Kaniraj and Havanagi 2001).
4. The addition of fibres causes the soil behaviour to diverge from the frictional characteristics, so that the shear strength is no longer related to the volume change. The effect of fibre inclusion on dilation and volume change is pronounced at higher load and strain levels, probably due to the inhibiting action of the fibres (Maher and Gray 1990; Heineck et al. 2005).
5. Inclusion of fibres increases the peak *UCS* and ductility of kaolinite clay, with the increase being more pronounced at lower water contents of the fibre-reinforced clay. Increase in fibre length reduces the contribution of fibres to peak *UCS*, while increasing the contribution to energy absorption or ductility (Maher and Ho 1994).
6. In the direct shear tests, the shear strength of dry sand reinforced with shredded waste tyre fibres (length = 6–150 mm) is affected significantly by normal stress, fibre content and unit weight of sand. Adding tyre fibres increases the shear strength of dry sand with an apparent friction angle as large as 67°. Only the shredded tyre fibres have friction angle of 30°, while the friction angle of sand varies from 25° to 34° for loose condition (unit weight = 15.5 kN/m^3) and dense condition (unit weight = 17.7 kN/m^3), respectively (Foose et al. 1996).
7. The triaxial compression test data reported by Consoli et al. (1998a) show that the glass fibre reinforcement inclusion in cemented silty sand (SM) increases both peak and residual strengths, decreases stiffness and changes the brittle behaviour to more ductile as indicated by a significant reduction in the brittleness index I_b. The peak strength increase is more effective for uncemented soil, whereas the increase in residual strength is more effective when fibres are added to soil containing cement. The peak friction angle of uncemented silty sand increases from 35° to 46° due to inclusion of glass fibres, while the peak cohesion intercept is just slightly affected by glass fibre inclusion, being a function basically of cementation.
8. The test results of drained triaxial compression tests conducted by Michalowski and Cermak (2002) indicate that the contribution of the monofilament polyamide and galvanized steel fibres to the strength of sand is the largest when they are placed in the direction of largest extension of the fibre-reinforced soil (horizontal in the test). Vertical fibres in triaxial testing are subjected to

compression, and they have an adverse effect on the initial stiffness of the reinforced soil and do not contribute to the strength. Specimens with a random distribution of fibres exhibit a smaller increase in strength than those with horizontal fibres, because a portion of randomly distributed fibres is subjected to compression.

9. The engineering characteristics of fly ash and PET fibre-reinforced fly ash are similar in drained and undrained triaxial compression tests. The deviator stress-axial strain curves in the unconsolidated undrained and drained tests are amenable for hyperbolic analysis (Kaniraj and Gayathri 2003).
10. The triaxial compression test data obtained by Consoli et al. (2003a) show that the friction angle of low plasticity silty clayey sand is barely affected by inclusion of PP fibres as its value increases from 30° to 31°, while the cohesion intercept increases from 23 to 127 kPa. The fibre-reinforced triaxial soil specimens are observed to fail as a uniform bulging.
11. The results obtained from the unconfined compression tests and the triaxial compression tests on fibre-reinforced soils should be used carefully in analysis and design of fibre-reinforced soil structures as these tests have some limitations. A general limitation is the length L of fibres with respect to size (diameter d) of the test specimen. The researchers have used different values of d/L ratios varying from 0.17 to 10.2 in their studies. Ang and Loehr (2003) performed a series of laboratory unconfined compression tests on specimens of a compacted silty clay to evaluate how the size of specimens used for strength testing of fibre-reinforced soil affects the measured strength and stress-strain properties. Their study indicates that there is a significant effect of specimen size both in terms of the magnitude of the measured strengths as well as in terms of the variability of the measured strengths. The effects of specimen size are found to be most important for specimens compacted at water contents dry of optimum water content. For fibre lengths of less than 50 mm, probably use of specimens with diameters of 70 mm or greater (i.e. $d/L > 1.4$) will produce strengths that are reasonably representative of the true mass strengths of fibre-reinforced soils.
12. The inclusion of stiff fibres, such as glass fibres (also, PET fibres), increases the peak friction angle of both cemented and uncemented sand, as well as slightly reduces the peak cohesive intercept and the brittleness of the cemented sand. Relatively extensible fibres, such as PP fibres, exert a more pronounced effect on the mode of failure (from brittle to ductile) and on the ultimate behaviour of cemented sand. An increase in ultimate deviator stress is observed to be directly proportional to the fibre length. Inclusion of glass fibres does not affect the ultimate strength of the reinforced soil. The initial stiffness for both cemented and uncemented soils is slightly affected by inclusion of PET and glass fibres, but it dramatically decreases by inclusion of PP fibres, probably as a result of loss of continuity of the cementitious links caused by the introduction of relatively extensible fibres, which will work just under high deformations. The increase in energy absorption capacity of soil is more if longer fibres are included (Consoli et al. 2004).

13. Based on the direct shear tests, the cohesion intercept of black cotton soil increases, and the angle of internal friction decreases with addition of 2% of glass fibres. However, on further increasing fibre content, the cohesion decreases, and the angle of internal friction increases (Gosavi et al. 2004).
14. The direct shear test results show that as the PP fibre content in poorly graded sand increases, the percentage increase in peak shear stress increases up to 0.2% fibre content. Beyond 0.2% fibre content, the rate of percentage increase in the peak shear stress decreases. As the normal stress increases, the increase in peak shear stress of fibre-reinforced sand decreases. The angle of internal friction of the fibre-reinforced sand is more than that of the unreinforced sand, but an increase in the fibre content does not have a significant effect on the angle of internal friction angle of sand. The increase in the length of the fibre also does not affect the angle of internal friction. At 70% relative density, the angles of internal friction of unreinforced and reinforced sands are comparable. The percentage increase in strength of reinforced sand is more at 50% relative density compared to 70% relative density. At higher relative density (70%), the increase in fibre content does not have much effect on the strength of fibre-reinforced sand (Gupta et al. 2006).
15. Rubber fibres (length = 4–10 mm; diameter = 0.3–1.5 mm) constitute discontinuities in the soil phase. This can have a negative influence on the compactability and strength of the reinforced clay. In general, the presence of rubber fibres does not cause significant change in the drained and undrained strength of the clay (Ozkul and Baykal 2006).
16. Latha and Murthy (2007) investigated the effects of reinforcement form on the mechanical behaviour of a dry river sand (SP) at 70% relative density by conducting a series of triaxial compression tests on unreinforced and reinforced sand with three different reinforcement forms, namely, planar/horizontal reinforcements (PP geotextile and PET geogrid) of circular shape (diameter = 38 mm), discrete strip PET fibres (length = 11 mm; width = 2 mm) and cylindrical geocells made from geotextile or geogrid by stitching with PET threads. The test results indicate that the discrete fibre form of reinforcement is inferior compared to the planar or cellular forms. This may be because of reduction in overall confinement effect due to small-size fibre elements. Among the three forms of reinforcement, the geocell is the most effective in improving the strength. The stress-strain curves for geocells at all confining pressures are found to be almost flat after peak is reached unlike in case of other two forms where the post-peak strength loss is observed. PET geocells are highly efficient in improving the strength of sand compared to PP geotextile cells.
17. Use of coir and PP fibres increases the *UCS* value of silty sand and silty sand-fly ash mixes. The optimum amount of fibres is found to be 0.75% of coir fibres and 1% of PP fibres. The resilient strain is less in soil reinforced with coir fibres than with PP fibres, indicating that coir fibres can help in delaying the failure of subgrade in pavement systems. This is probably due to higher interface friction angle of coir fibres with soil (Chauhan et al. 2008).

18. The unconfined compressive strength reaches its maximum value at a PP fibre (7 mm long)/fat clay (CH) ratio of 1.0% and poorly graded sand (SP)/fat clay (CH) ratio of 7.5%. There is a reasonably strong linear correlation ($R^2 \approx 1.0$) between *UCS* and average initial tangent modulus values. As the sand content increases from 5 to 10%, the average initial tangent modulus increases and reaches its maximum value at 7.5% sand content (Yilmaz 2009).
19. The direct shear test results show that the shear strength improvement of sand induced by inclusion of tyre buffings (length = 8–50 mm; thickness = 2–5 mm) is sensitive to the amount of applied normal stress. The shear strength of sand increases with increasing content of tyre buffings up to a maximum value for buffings content in the vicinity of 20%. Internal friction angle of sand increases from 34° to as large as 45°. Increasing the aspect ratio of tyre buffings from 2 to 12.5 leads to an increase of the overall shear strength of sand. This is probably due to an increased contact area with the soil particles (Edincliler and Ayhan 2010).
20. The direct shear tests conducted by Zaimoglu and Yetimoglu (2012) on randomly distributed PP fibre-reinforced fine-grained soil (MH, high plasticity soil) show that the value of cohesion intercept of soil increases with increasing fibre content up to a value of around 0.75%. However, the angle of shearing resistance does not change significantly with fibre content. These observations might be attributed to the fact that the randomly distributed fibres act as a spatial three-dimensional network to interlock soil particles to form a unitary coherent matrix and restrict any displacements, thus resulting in increased cohesion, but the individual fibre inclusions might not have discernible effect on the microstructure of soil, thus not affecting the angle of shearing resistance of soil (Tang et al. 2007b; Zaimoglu and Yetimoglu 2012).
21. The silica fume (a waste product obtained as a by-product of producing silicon metal or ferrosilicon alloys), scrap tyre rubber fibres (length ranging from 5 to 10 mm; thickness ranging from 0.25 to 0.5 mm; width ranging from 0.25 to 1.25 mm) and silica fume-scrap tyre rubber fibre mixtures increase the *UCS* of clayey soil. The maximum value of *UCS* is obtained by addition of 20% silica fume and 2% fibre. The direct shear test results indicate that the maximum cohesion and internal friction angle values are also obtained by addition of 20% silica fume and 2% fibre (Kalkan 2013).
22. The cyclic triaxial compression test results show that the presence of monofilament PP fibres (length ranging from 6 to 18 mm) has a significant effect in reducing the liquefaction susceptibility of poorly graded beach sand (classified as SP).The number of load cycles causing liquefaction increases with an increase in fibre content and fibre length. Maximum improvement in liquefaction resistance is found to be 280% for a sand sample reinforced with 1% fibre, 18 mm long, at relative density of 40% and confining pressure of 50 kPa. Confining pressure has a considerable effect in increasing the resistance to liquefaction of sand. The liquefaction resistance of sand increases with an increase in relative density. The effect of reinforcement in medium sand

samples is found to be more significant than that of loose samples (Noorzad and Amini 2014).

23. The UU triaxial compression test results indicate that the inclusion of PET fibres (length = 50 mm; diameter = 0.015 mm) influences the internal friction angle and the cohesion intercept of the silty soil (classified as MH). Greater fibre contents result in a lower internal friction angle and greater cohesion intercept. Changes in the friction angle could be attributed to a reduction in the number of contact points between the soil particles due to the presence of fibres and the greater cohesion intercept due to increase in apparent cohesion of the soil particles and fibres (Botero et al. 2015).
24. In the UU triaxial compression test, addition of PP fibres to clay increases both peak and post-peak deviator stresses and increases the ductility of soil. The deviator stress at failure decreases with increasing water content, and the influence of fibres diminishes at higher water content due to the higher water content facilitating pullout failure at the fibre-soil interface. The addition of lime alters not only the soil properties (significant increase in shear strength) but also the soil-fibre interface behaviour. The hydrated lime developed on the surface of the fibres increases the surface roughness of the fibre, increasing the pullout resistance and thereby increasing the mobilized tension within the fibres. Moreover, a reduction of free water within the soil structure by addition of lime also increases the fibre-soil friction resistance (Gelder and Fowmes 2016).

Example 3.2

Polyester fibres are available in the following two forms: (i) $L = 12$ mm, $D = 0.075$ mm; and (ii) $L = 15$ mm, $D = 0.1$ mm, where L and D are length and diameter of fibres, respectively. If a sandy soil is to be reinforced, which form of the fibres should be recommended for strength improvement? Justify your answer.

Solution

From Eq. (2.1), the aspect ratio of fibres is

$$a_\mathrm{r} = \frac{L}{D}$$

For case (i),

$$a_\mathrm{r} = \frac{L}{D} = \frac{12}{0.075} = 160$$

For case (ii),

$$a_\mathrm{r} = \frac{L}{D} = \frac{15}{0.1} = 150$$

In general, the strength of fibre-reinforced sandy soil increases with increasing aspect ratio of fibres, so the fibres having length, $L = 12$ mm, and diameter, $D = 0.075$ mm, should be recommended for achieving a higher strength.

3.5 Compaction Behaviour

The studies have been carried out to investigate the compaction behaviour of fibre-reinforced soils by using the compaction tests in accordance with available standards on unreinforced soil compaction.

Kaniraj and Havanagi (2001) studied the compaction characteristics of Rajghat fly ash, Delhi silt, mixture of 50% Rajghat fly ash and 50% Delhi silt and mixture of 50% Rajghat fly ash and 50% Yamuna sand with and without random distribution of PET fibre reinforcement. The values of maximum dry unit weight (γ_{dmax}) and optimum water content (w_{opt}) as determined from standard Proctor compaction tests are given in Table 3.6.

In Table 3.6, you may notice that except in fly ash, in silt and other fly ash-soil mixtures, the fibres have no significant effect on γ_{dmax} and w_{opt}. In the case of fly ash, as the fibre content increases, the value of γ_{dmax} of fly ash increases, whereas the value of w_{opt} of fly ash decreases.

When the cohesive-frictional soil is reinforced with sisal (natural) fibres, the fibre inclusion reduces the dry unit weight of soil due to low specific gravity (0.962) of sisal fibres. The increase in fibre content (from 0.25 to 1%) and fibre length (from 10 to 25 mm) also reduces the dry unit weight of the soil; the variation is linear for both cases. The dry unit weight of reinforced soil ranges from 16.98 to 17.75 kN/m^3 . The initial inclusion of fibres in soil causes an increase in optimum water content, but a further increase in both fibre content and length reduces the optimum water content. The optimum water content of reinforced soil ranges from 16 to 19.2% (Prabakar and Sridhar 2002).

Table 3.6 Compaction test results of fly ash, silt and soil mixtures with fibre content (defined as the ratio of weight of fibre solids to sum of weights of soil solids and fly ash solids), $p_{\mathrm{f}} = 1\%$, for the reinforced cases (After Kaniraj and Havanagi 2001)

	Maximum dry unit weight, γ_{dmax} (kN/m^3)		Optimum water content, w_{opt} (%)	
Soil type	Unreinforced	Reinforced	Unreinforced	Reinforced
Fly ash	10.52	11.27	36.5	31.7
Silt	17.66	17.60	14.0	14.5
Mixture of 50% fly ash and 50% silt	13.54	13.90	22.6	23.0
Mixture of 50% of fly ash and 50% sand	13.64	13.60	22.6	23.5

Note: For fly ash reinforced with fibres ($p_{\mathrm{f}} = 0.5\%$), $\gamma_{\mathrm{dmax}} = 11.06$ kN/m^3 and $w_{\mathrm{opt}} = 33.1\%$

The standard Proctor tests conducted by Gosavi et al. (2004) indicate that the optimum water content of black cotton soil increases, and its maximum dry unit weight decreases by inclusion of glass fibres up to 2% fibre content. However, the trends are reversed on further increase of fibre content.

The compaction test results reported by Miller and Rifai (2004) for PP fibre-reinforced medium plasticity soil indicate that for the standard compactive effort, the optimum water content varies within approximately 4.6% of the value for the unreinforced soil, while the maximum dry unit weight increases by about 1%, reaching a peak at a fibre content of 0.8%. The trends for modified compaction effort are similar. A comparison between the results of the two compaction efforts shows that the dry unit weight was maximized at fibre contents of 0.8% and 0.6% for standard and modified efforts, respectively. The variations in the maximum dry unit weight and optimum water content are less than 5%. Therefore, the changes in compaction behaviour of the soil due to fibre inclusion are considered insignificant.

Kumar and Singh (2008) observed that the addition of 0.5% of PP fibres resulted in a decrease in optimum water content and maximum dry unit weight of fly ash by 6.8% and 2.8%, respectively.

For a given compactive effort, the maximum dry unit weight of PP crimped fibre-reinforced sand decreases with increasing fibre content, whereas the optimum water content (around 10%) is independent of the amount of fibres (0–0.5%). More compaction energy may be necessary to produce specimens with higher fibre contents at a given dry unit weight (Ibraim et al. 2010).

Mirzababaei et al. (2013) studied the compaction behaviour of carpet waste fibre-reinforced clay soil. As seen in Fig. 3.15, increasing the fibre content leads to a reduction in the maximum dry unit weight and an increase in optimum water content. A decrease in maximum dry unit weight is primarily attributed to the lower specific gravity (1.14) of fibres (100% nylon) compared to the specific gravity of soil solids (2.68).

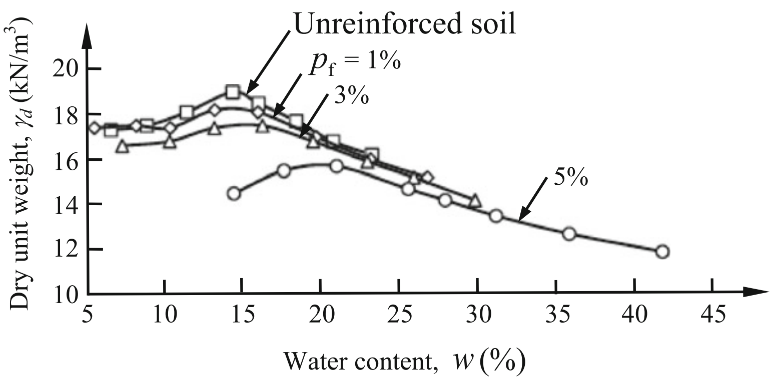

Fig. 3.15 Effect of fibre content on compaction curves for carpet waste fibre-reinforced clay soil (a combination of 90% natural clay (CL as per unified soil classification system) from the Northwest region of UK and 10% sodium activated bentonite) (Adapted from Mirzababaei et al. 2013)

For a uniform mixing of fibre and clay, the natural moisture content of clay may be increased. The moisture content that facilitates easy and uniform mixing is called the *optimum mixing moisture content* (OMMC). For the clay and PP fibres used in the study by Gelder and Fowmes (2016), the OMMC was found to be 26%, although the natural moisture content of the clay was about 16%.

PP fibre inclusions within the clay resist the compactive effort, forming an interlocked structure. As a result, the optimum water content of clay increases, and its maximum dry unit weight decreases with increasing fibre content from 0 to 0.75%, without any significant change in the compaction behaviour in terms of the shape of the compaction curve (Gelder and Fowmes 2016).

The following points regarding the compaction behaviour of fibre-reinforced soils are worth mentioning:

1. Incorporation of fibres within the soil tends to decrease the maximum dry unit weight and increase the optimum water content of the fibre-reinforced soil, mainly due to increased voids caused by fibre separation of the soil particles. The increased compactive effort, from standard to modified compaction, results in increased strength of fibre-reinforced soil and somewhat improved interfacial fibre-soil bond (Hoover et al. 1982).
2. Both the porosity and amount and type of compaction affect the response of a fibre-reinforced soil. The greater the fibre content, the greater compactive effort requires maintaining a given porosity. On the other hand, an increase in compactive effort also results in greater fibre entanglement and distortion, which also affect the response of a fibre-reinforced soil (Gray and Al-Refeai 1986).
3. The presence of randomly oriented PP fibres in cohesionless soil creates a higher resistance to compaction than multioriented inclusions (Lawton and Fox 1992).
4. Inclusion of 10% of tyre buffings (length = 4–10 mm; diameter = 0.3–1.5 mm) reduces the unit weight of the clay. The compaction process causes a preferential alignment of fibres in directions parallel to the compaction plane. This preferential orientation is favourable for resisting the desiccation cracking and tensile stresses that may exit on slopes. The efficiency of compaction does not change at standard compaction energy but decreases when modified compaction energy is used. This causes a decrease in the slope of the line of optimums. Hence the unit weight of the reinforced clay is less sensitive to the compaction water content compared with clay alone. This may be advantageous in the field, as water content is difficult to control (Ozkul and Baykal 2006).
5. The inclusion of PP fibres in Brickies sand (classified as SP) causes a significant reduction in the maximum dry unit weight with a minor increase in optimum water content (Shukla et al. 2015).
6. The addition of lime to clay along with PP fibres reduces the maximum dry unit weight dramatically to 16.48 kN/m^3 and increases the optimum water content to 21%. The percentage change in the maximum dry unit weight and optimum water content is −15.58% and +75%, respectively, compared with the values for clay with no additives. The changes with respect to fibre-only reinforced soil are

−8.5% and +27%. In addition, the inclusion of lime affects the compaction behaviour, as indicated by flattening of the compaction curve for soil with lime (Gelder and Fowmes 2016).

7. As different fibres absorb varying amount of water during the compaction test, the compaction test parameters may differ accordingly. In general, the absorption of water by fibres increases the optimum water content of soil.
8. The trend in the change of compaction test parameters due to fibre inclusions as reported by the researchers is not very consistent. For example, Fletcher and Humphries (1991) and Nataraj and McManis (1997) found the optimum water content and the maximum dry unit weight of soil to slightly decrease and increase, respectively. This contradicts the observations by Mirzababaei et al. (2013) and Gelder and Fowmes (2016). Thus, there is still a need of further research on investigating the effect of various types of fibres and different fibre contents on compaction characteristics of soils and other similar materials such as fly ashes.

3.6 Permeability and Compressibility

Based on some limited tests, the author has experienced that it is difficult to get the consistent results for the permeability (i.e. hydraulic conductivity) behaviour of fibre-reinforced soils with varying fibre and soil parameters. In general, for a given fibre content, the hydraulic conductivity increases with increasing length of the fibres. The effect of fibre content on the permeability depends greatly on the density of the reinforced soil as well as the confinement pressure; thus both increasing and deceasing trends can be seen.

The investigation carried out by Maher and Ho (1994) shows that with an increase in the fibre content, the hydraulic conductivity of normally consolidated fibre-reinforced kaolinite clay (with water content close to the compacted optimum value of 25%) increases for the fibre types (PP fibres, glass fibres and softwood pulp fibres) and lengths (0.55–25.4 mm) used. In practical applications, however, the fibre content should be such that increasing volume stability is achieved without exceeding the allowable limit on hydraulic conductivity.

Embankment dams and other water-retaining structures are often prone to seepage erosion in the form of piping. If fibres are added to soil in making these structures, their presence can affect the piping behaviour of soil. Babu and Vasudevan (2008b) carried out a number of experiments for determining seepage velocity and piping resistance of sand, red soil and mixture of sand and red soil (50% sand + 50% red soil) mixed randomly with coir fibres (length = 40–60 mm; average diameter = 0.025 mm). The fibre contents varied from 0.25 to 1.5%. The tests results indicate the following:

1. Inclusion of coir fibres in soil reduces the lifting of individual soil particles when water flows in the upward direction through the soil mass and increases the

critical hydraulic gradient (i.e. the gradient at which piping failure occurs). Thus, the piping failure due to lifting of soil particles is found to occur in fibre-reinforced soil at high hydraulic gradients, whereas the unreinforced soil fails at comparatively low hydraulic gradients. This is clearly observed for fibre-reinforced red soil and also for fibre-reinforced sand-red soil mixture.

2. An increase in fibre content causes a decrease in seepage velocity and an increase in piping resistance of all three types of soil. The least value of seepage velocity is observed for fibre length of 50 mm. If the length of fibre is low, say 25 mm, the fibre content has no effect on discharge, and similarly, if the fibres are longer than 75 mm, seepage is not reduced substantially.

To investigate the effect of PP and PET fibres on the piping behaviour of silty sand, Das and Viswanadham (2010) carried out the laboratory experiments by developing a one-dimensional piping test apparatus, which simulates the upward seepage through a soil with and without fibres. The test considered the fibre contents of 0.05, 0.1 and 0.15% with two different fibre lengths of 25 and 50 mm. The test results suggest the following:

1. The inclusion of PP and PET fibres reduces seepage velocity and improves the piping resistance of silty sand for fibre content of 0.1% and fibre length of 50 mm. Of the two fibre types, PP fibres are found to be more effective in improving the piping behaviour of soil. For PET fibres, because of a higher specific gravity, the number of fibres reduces for a given fibre content.
2. The seepage velocity increases almost linearly with hydraulic gradient i (see Sec. 1.3 for definition) during the initial stage, indicating a laminar flow until the critical hydraulic gradient i_c is reached. Beyond i_c, the seepage velocity is observed to increase catastrophically.
3. The permeability of silty sand decreases as a result of fibre inclusion. This observed behaviour could be caused by a reduction in the porosity of soil by the fibres blocking flow channels. This is essentially due to the fact that the fibres contribute to the volume of soil solids, leading to reduction in the void ratio.
4. The fibres not only restrict the flow of water but also held the soil particles and resist their movement under an increasing hydraulic gradient.
5. Figure 3.16 shows the variation of critical hydraulic gradient and piping resistance against the fibre content. It is noticed that, irrespective of fibre length or fibre type, the piping resistance increases with an increase in fibre content. Hence the fibres can be mixed into soil for making embankment dams and other water-retaining structures more resistant to piping erosion.
6. A higher fibre content, say greater than 0.15% for PP and PET fibres, may cause accumulation of a cluster of fibres in one location, resulting in more flow of water and reduction in piping resistance.

Kaniraj and Havanagi (2001) conducted the one-dimensional consolidation tests on Rajghat fly ash, mixture of 50% Rajghat fly ash and 50% Delhi silt and mixture of 50% Rajghat fly ash and 50% Yamuna sand with and without random distribution of PET fibre reinforcement. The specimens were compacted statically at their

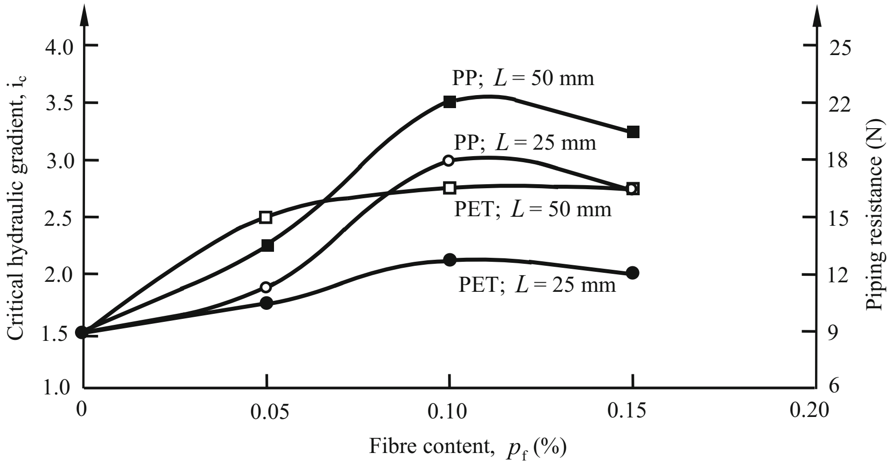

Fig. 3.16 Variation of critical hydraulic gradient and piping resistance of soil with fibre content (Adapted from Das and Viswanadham 2010)

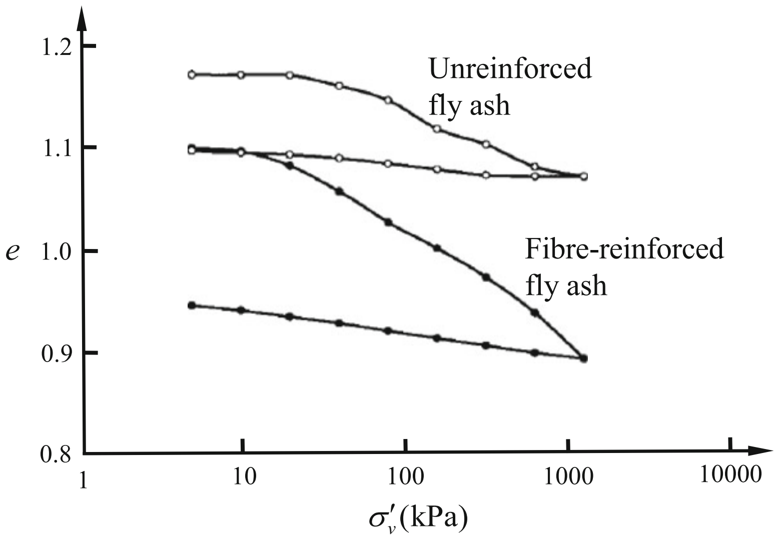

Fig. 3.17 $e - \log_{10}\sigma'_v$ curves for unreinforced and fibre-reinforced fly ash specimens (Adapted from Kaniraj and Havanagi 2001)

maximum dry unit weight-optimum water content state. Figure 3.17 shows the relationship between the void ratio e and the logarithm of the effective vertical stress $\log_{10}\sigma'_v$ for unreinforced and fibre-reinforced fly ash specimens. Table 3.7 presents the test values of compression index C_c, recompression index C_r and coefficient of consolidation c_v. The test results show that the fibre inclusions

Table 3.7 One-dimensional consolidation test results for fly ash and soil mixtures with fibre content (defined as the ratio of weight of fibre solids to sum of weights of soil solids and fly ash solids), $p_f = 1\%$, for the reinforced cases (After Kaniraj and Havanagi 2001)

	Unreinforced			Fibre reinforced		
Soil type	C_c	C_r	c_v (m^2/year)	C_c	C_r	c_v (m^2/year)
Fly ash	0.072	0.017	238–299	0.123	0.022	248–347
Mixture of 50% fly ash and 50% silt	0.123	0.021	254–290	0.148	0.022	224–288
Mixture of 50% of fly ash and 50% sand	0.093	0.016	291–315	0.164	0.025	275–378

increase the value of C_c and cause accelerated consolidation to some extent, especially of reinforced fly ash, as indicated by the higher values of c_v.

Expansive soils/clays change volume when subjected to variation in water content, and any such change causes damages to foundations of structures, such as buildings, bridges and pavements. In recent years, the researchers have tried to study the effect of fibre inclusion on swelling behaviour of such soils in terms of swell potential and swelling pressure. The free swell method is generally adopted to evaluate both swell potential and swelling pressure. During the swell test in a one-dimensional oedometer cell, the soil specimen is allowed to swell freely under a seating load. The maximum swell that the specimen achieves is called the *swell potential* and is presented as a percentage of the initial thickness H of the soil specimen. Load increments are then added to bring the soil specimen to its initial void ratio. The pressure that brings the soil specimen to its initial void ratio is defined as the *swelling pressure* of the soil.

The effect of fibre reinforcement on swelling behaviour of soil can be expressed as the swelling potential ratio (*SPR*), which is defined as a ratio of the swelling potential of the reinforced soil specimen $(\Delta H/H)_R$ to the swelling potential of the unreinforced soil specimen $(\Delta H/H)_U$, where ΔH is the change in the thickness of the soil specimen. Thus,

$$SPR = \frac{\left(\frac{\Delta H}{H}\right)_R}{\left(\frac{\Delta H}{H}\right)_U} \tag{3.13}$$

Al-Akhras et al. (2008) investigated the effect of two types of fibres (synthetic nylon and natural palmyra) on the swelling properties of three types of clayey soils (classified as CH, CH and CL) by conducting swell tests in a one-dimensional oedometer cell. The test results indicate the following:

1. Clayey soils mixed with fibres show significantly lower swelling pressure and swell potential in comparison with the same clayey soils without fibres (Fig. 3.18).

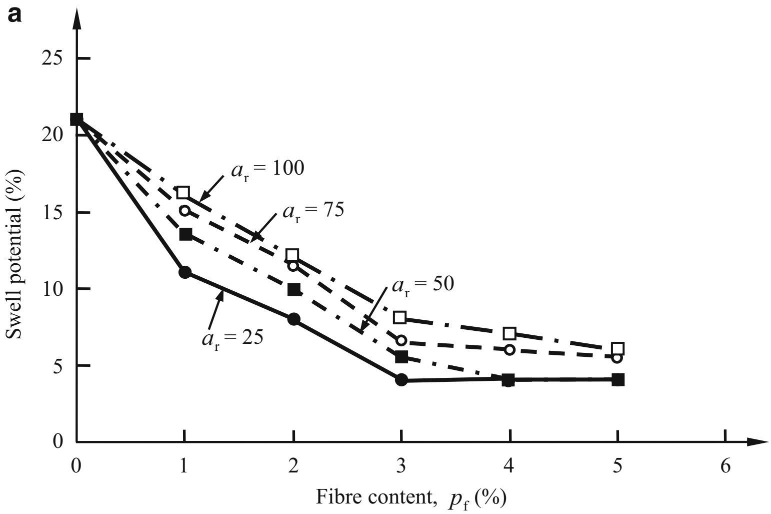

Fig. 3.18 Effect of nylon fibres on: (**a**) swell potential; (**b**) swelling pressure of clayey soil (classified as CH with liquid limit of 78% and plasticity index of 43%) (Adapted from Al-Akhras et al. 2008)

2. Clayey soils mixed with palmyra fibres show substantially lower swelling pressure and swell potential in comparison with clayey soils mixed with nylon fibres at the same fibre content.
3. Clayey soils mixed with fibres with aspect ratio of 25 show lower swelling pressure in comparison with clayey soils mixed with fibres with aspect ratio of 100.

Table 3.8 Compression index of unreinforced and coir fibre-reinforced black cotton soils (After Babu et al. 2008b)

Fibre content, p_f (%)	Compression index, C_c
0	0.51
0.5	0.43
1	0.40
1.5	0.33

Note: For coir fibres, length, $L = 15$ mm, and diameter, $D = 0.25$ mm

4. The impact of the inclusion of fibres in clayey soils increases with increasing clay fraction of the clayey soil.

Babu et al. (2008b) carried out consolidation and swell tests on coir fibre-reinforced black cotton soil, which is an expansive soil. The test results indicate that swelling is reduced by about 40% when the fibre content is increased from 0.5 to 1.5%, and the compression index (see Table 3.8) decreases by about 35% for a fibre content of 1.5%. Thus the coir fibres are suited for controlling the swelling of black cotton soil, and hence the excessive settlement of structures built on such compacted soil deposits can be reduced considerably.

Viswanadham et al. (2009a, b) performed swell-consolidation tests in a conventional oedometer (diameter = 75 mm; thickness = 25 mm) on expansive soil (classified as CH) reinforced with PP fibres using varying fibre contents (0.25–0.5%) and aspect ratios (15, 30 and 45). The test results indicate that an inclusion of PP fibres reduces the heave of the soil. Heave is reduced more at lower aspect ratios than at higher aspect ratios. Swelling decreases with increasing fibre content for all aspect ratios. Reduction in swelling is rapid up to aspect ratio of 15 at all fibre contents. When the aspect ratio is low, even small fibre content is found effective. Swelling pressure decreases as a result of fibre inclusion. Reduction in swelling can be attributed to replacement of swelling soil by fibres and resistance offered by fibres to swelling through clay-fibre contact. When swelling of the soil occurs, the flexible fibres in the soil are stretched, and hence the tension in fibres resists the further swelling. Resistance offered by the fibres to swelling depends upon the soil-fibre contact area. When the long fibres are mixed randomly, they tend to twist or fold. This reflects in the form of loss of effective soil-fibre contact area for restraining swelling.

The following points regarding permeability and compressibility of fibre-reinforced soils are worth mentioning:

1. Use of the fibres decreases freeze-thaw volumetric change on the order of 40% as compared with the unreinforced soil. When the fibre-reinforced soil is modified with a low cement content, freeze-thaw volumetric expansion is eliminated, indicating an extremely stable reinforced soil (Hoover et al. 1982).
2. The experimental study conducted by Miller and Rifai (2004) indicates that the PP fibre inclusion increases the desiccation crack reduction significantly due to increased tensile strength of the medium plasticity soil (classified as CL) due to

fibre inclusion. The optimum fibre content for achieving the maximum crack reduction and maximum dry unit weight, while maintaining the acceptable hydraulic conductivity, is between 0.4 and 0.5%.

3. The initial void ratio is more for unreinforced fine sand than that for fine sand reinforced with plastic waste chips/fibres (polymer = PET, specific gravity = 1.33, length = 6 mm, width = 0.2 mm) because the plastic fibres occupy the voids within the soil solids. The compressibility of soil reduces as the fibre content increases; this is indicated by a decrease in the slope of void ratio versus logarithm of effective stress curve with an increase in fibre content. As the fibre content in the sand is increased from 0.5 to 1%, the permeability of sand decreases by two times that of unreinforced sand (Manjari et al. 2011).
4. The permeability test using the falling-head permeameter shows that increasing fibre content of waste tyre yarn fibres initially increases the hydraulic conductivity of fine-grained soil, and when it crosses 0.6% limit, it decreases (Saghari et al. 2015).

3.7 California Bearing Ratio

The bearing capacity ratio (*CBR*) of fibre-reinforced soil has been studied by several researchers by conducting California bearing ratio (*CBR*) test. For the analysis and design of fibre-reinforced soil bases/subbases/subgrades of the pavements, the effect of fibres on the *CBR* value is generally represented by the *load ratio*, which is a nondimensional parameter, defined as

$$LR = \frac{Q_{\mathrm{R}}}{Q_{\mathrm{pU}}} \tag{3.14}$$

where Q_{pU} is the peak piston load on unreinforced soil and Q_{R} is the piston load on fibre-reinforced soil corresponding to the piston penetration $\rho(=\rho_{\mathrm{pU}})$ on unreinforced soil at the peak piston load Q_{pU}.

Note that in Eq. (3.14), Q_{R} can be the piston load on the reinforced soil at any selected/permissible penetration of the piston as required for the safety of the pavement structure. For $Q_{\mathrm{R}}=Q_{\mathrm{pR}}$ (peak piston load on reinforced soil) at $\rho=\rho_{\mathrm{pR}}$ (peak piston penetration), Eq. (3.14) reduces to *peak load ratio* (PLR) as

$$\mathrm{PLR} = \frac{Q_{\mathrm{pR}}}{Q_{\mathrm{pU}}} \tag{3.15}$$

The effect of fibres on the *CBR* value can also be represented by the *CBR improvement factor* I_{CBR}, or *normalized CBR value CBRR*, defined as

$$I_{CBR} = \frac{CBR_R - CBR_U}{CBR_U} = \frac{CBR_R}{CBR_U} - 1 \tag{3.16}$$

$$CBRR = \frac{CBR_R}{CBR_U} \tag{3.17}$$

where CBR_U is the *CBR* value of the unreinforced soil, and CBR_R is the *CBR* value of the reinforced soil.

The *CBR* tests carried out by Lindh and Eriksson (1990) show that the wet sand reinforced with 48-mm long plastic fibres does not offer a *CBR* value significantly higher than the sand alone at a deformation of 2.54 mm. However, the sand specimens containing fibres show a continuously rising curve in the stress-deformation diagram, unlike sand without fibres, which shows clear fractures (Fig. 3.19).

Tingle et al. (2002) have reported that use of fibres improves the *CBR* of the sand from 6 to 34% over the unstabilized sand shoulder at similar depths. The improvement in load-bearing capacity of the fibre-stabilized sand can be attributed to the confinement of the sand particles by the discrete fibres. When the fibres are mixed into the sand, the fibres develop friction at interaction points with the particles that resist rearrangement of the particles under loading. Thus, the primary stabilization mechanism is the mechanical confinement of the sand. They have also reported that 'springy or sponge-like' behaviour is a fundamental property of the fibre-reinforced soil.

The *CBR* of black cotton soil increases by about 46% and 55% for aspect ratios of 250 and 500, respectively, as a result of inclusion of 2% glass fibre. An increased *CBR* makes the pavement construction economical by reducing the thickness of pavement layers (Gosavi et al. 2004).

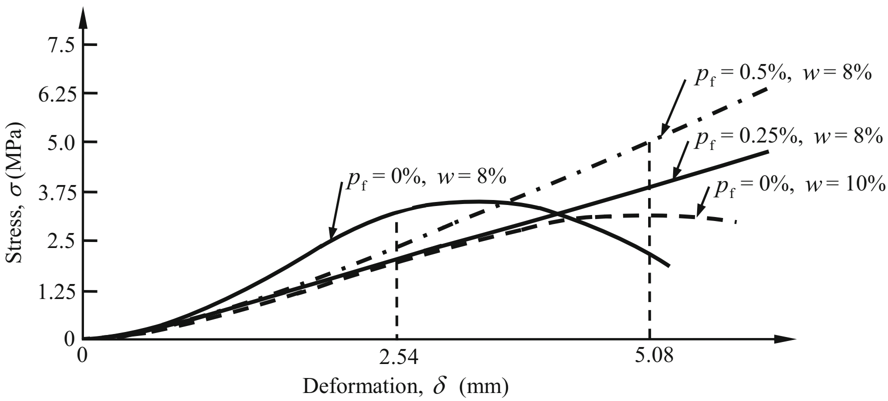

Fig. 3.19 *CBR* test results on wet sand with and without fibres (water content, w = 8–10%) (Adapted from Lindh and Eriksson 1990)

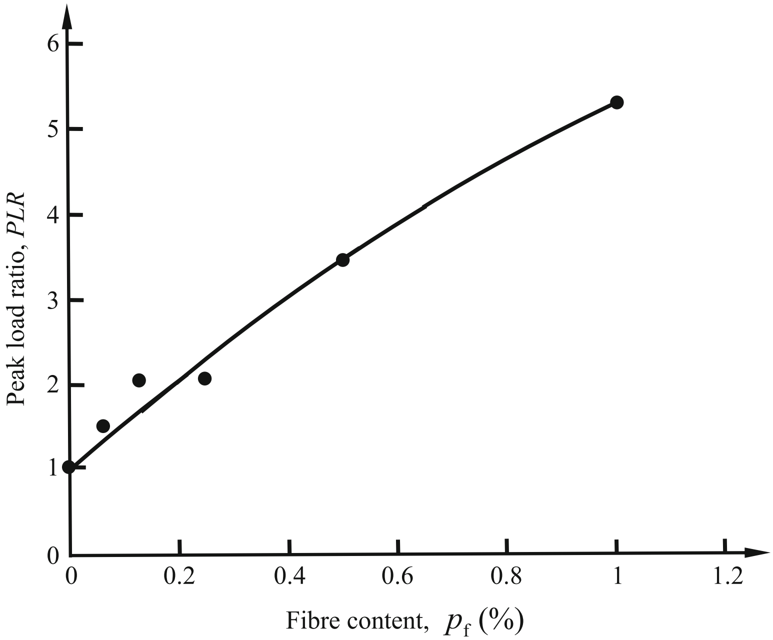

Fig. 3.20 Variation of peak load ratio (PLR) with fibre content (p_f) for fibre-reinforced sand (Adapted from Yetimoglu et al. 2005)

Yetimoglu et al. (2005) performed the laboratory *CBR* tests to investigate the load-penetration behaviour of a clean sand fill reinforced with randomly distributed discrete PP fibres (length =20 mm; diameter = 0.50 mm) overlying a high plasticity inorganic clay with a nonwoven geotextile layer at the sand-clay interface as a separator. Figure 3.20 shows the effect of fibre content on the peak load ratio (PLR) for the fibre-reinforced sand fill. It is noticed that the PLR value increases with an increase in fibre content and becomes approximately five times as high as that of unreinforced sand. The investigation also shows that the initial stiffness (i.e. the slope of the load-penetration curve) is not significantly changed by incorporating the fibres in sand. The penetration values at which the peak loads are mobilized tend to increase with increasing fibre content. It is also observed that increasing fibre content increases the brittleness of the fibre-reinforced sand as indicated by a higher loss of post-peak penetration resistance (strength). Additionally it is observed that the load-penetration behaviour of the sand reinforced randomly with a small amount of fibre inclusion is quite similar to that of the sand reinforced systematically with geotextile layers in a certain pattern.

The experimental study conducted by Kumar and Singh (2008) shows that the soaked and unsoaked *CBR* values of fly ash (classified as silt of low compressibility, ML) reinforced with randomly distributed PP fibres increase with an increase in fibre content at a particular aspect ratio (60, 80, 100 or 120). The increments have been reported to be more up to 0.3% fibre content. Based on the experimental results, a regression relationship between soaked *CBR* and fibre content p_f ranging from 0.1 to 0.5% was presented as

$$CBR = a\ln p_f + b \tag{3.18}$$

where $a = 10.102$, 11.323, 12.037 and 12.187 and $b = 32.394$, 35.855, 39.211 and 40.273 for $a_r = 60$, 80, 100 and 120, respectively. Note that the soaked and unsoaked values of *CBR* of fly ash were 8.5% and 20.2%, respectively.

In general, the *CBR* value of reinforced soils continue to increase with both fibre content and aspect ratio, but mixing soil and fibres is extremely difficult beyond the fibre content of 1.5%. For soils reinforced with PP fibres (length = 15 mm, 25 mm, 30 mm; diameter = 0.3 mm), the experimental study conducted by Chandra et al. (2008) suggests that 1.5% fibre content and an aspect ratio of 100 can be considered optimum values in the case of soils of low compressibility (classified as CL and ML), whereas 1.5% fibre content with an aspect ratio of 84 is found to be optimum for silty sand (classified as SM). The *CBR* values of CL, ML and SM compacted at standard compaction test parameters were found to be 1.16, 1.95 and 6.20%, respectively. These values increased to 4.33, 6.42 and 18.03%, respectively, due to reinforcing the soil at optimum fibre content of 1.5%.

Zaimoglu and Yetimoglu (2012) investigated the effects of randomly distributed PP fibre reinforcement (length = 12 mm; diameter = 0.05 mm) on the soaked *CBR* behaviour of a fine-grained soil (MH, high plasticity soil) by conducting a series of *CBR* tests. Figure 3.21 shows a typical variation of soaked *CBR* with the fibre content. It is observed that the *CBR* value increases significantly with increasing fibre content up to around 0.75% and remains more or less constant thereafter. The maximum increase in *CBR* is around 80% (i.e. from approximately 14% for unreinforced soil to approximately 25% for reinforced soil at a fibre content of 0.75%).

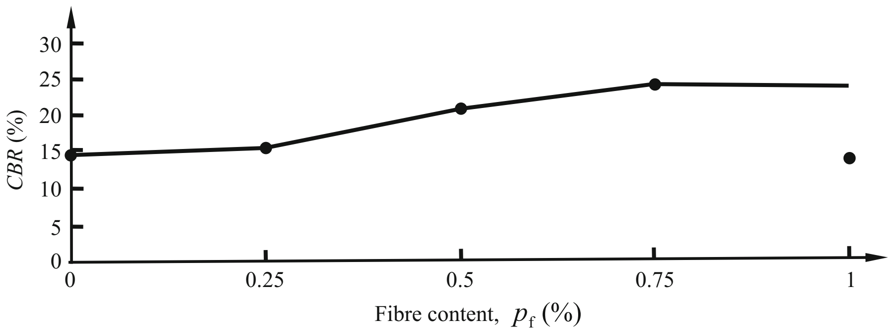

Fig. 3.21 Effect of fibre content on the soaked *CBR* behaviour of fine-grained soil (Adapted from Zaimoglu and Yetimoglu et al. 2012)

Table 3.9 Values of California bearing ratio (*CBR*) for plastic waste fibre-reinforced industrial wastes (After Jha et al. 2014)

Aspect ratio, a_r	Fibre content, p_f (%)	*CBR*(%) Fly ash (specific gravity = 2.20; 49% sand-size particles with 51% fines)	Stone dust (specific gravity = 2.63; 84% sand and 16% fines)	Waste recycled product (specific gravity = 2.87; sand-size particles without fines)
Unreinforced soil		3.11	6.08	41.85
1	0.25	4.38	8.27	49.88
	0.5	9.49	9.73	66.23
	1	11.68	14.11	70.17
	2	13.87	21.41	74.16
	4	19.46	23.36	76.89
2	0.25	7.30	8.56	59.85
	0.5	9.98	17.52	71.00
	1	12.41	21.90	74.99
	2	16.79	24.33	88.08
	4	23.31	27.74	88.56
3	0.25	7.54	9.73	60.24
	0.5	10.70	22.87	82.58
	1	12.65	27.25	88.56
	2	17.27	34.30	96.35
	4	25.06	42.73	99.27

Edincliler and Cagatay (2013) presented experimental results on the improvement of *CBR* performance of sand by addition of granulated rubber (aspect ratio = 1) and fibre shaped buffing rubber (aspect ratios of 4 and 8). Mixtures were prepared at rubber contents of 5, 10, 20, 30 and 40% by weight. The results show that the addition of 30% (by weight) buffing rubber to sand increases the *CBR* value of the reinforced sand, but the addition of granulated rubber decreases it. The use of buffing rubbers with the highest aspect ratio of eight results in an increase in *CBR* from 8 to 16 (100% increase, while the increase in *CBR* is from 8 to almost 12 (44% increase) with inclusion of buffing rubbers having an aspect ratio of 4. Thus use of the longer fibres causes significant benefit. This happens because the longer fibres have a large contact area with the soil particles, resulting in more friction/adhesion.

Addition of HDPE plastic waste strips (length = 12, 24, 36 mm; width = 12 mm; thickness = 0.4 mm) to industrial wastes (fly ash, stone dust and waste recycled product) results in an appreciable increase in the *CBR* value and the subgrade modulus. Table 3.9 provides the *CBR* test values. The subgrade modulus values for fly ash, stone dust and waste recycled product increase from 65 to 525 MPa/m, 127 to 894 MPa/m and 858 to 2078 MPa/m, respectively for 4% strip/fibre content with an aspect ratio of 3. The significant increases in *CBR* value and subgrade modulus indicate that these reinforced industrial wastes can be used in flexible pavement construction, leading to safe and economical disposal of the wastes (Choudhary et al. 2014; Jha et al. 2014).

Sarbaz et al. (2014) conducted the *CBR* tests under dry and submerged conditions on fine sand (SP) reinforced with plain date palm fibres (see Table 2.1 for properties) as well as with bitumen-coated date palm fibres. The test results indicate the following:

1. Adding 0.5–1% fibres enhances the *CBR* strength significantly by up to 56% compared to unreinforced sand. However, this effect gradually diminishes at higher fibre contents. For example, the *CBR* strength at 2% fibre content decreases slightly. Obviously, the presence of fibres in the soil more than what is required for optimum reinforcement can substitute the soil particles with weaker materials, thereby reducing the bearing capacity of the soil.
2. Soil reinforced with longer fibres has higher *CBR* strength than soil reinforced with shorter fibres. This can likely be attributed to the more mobilized frictional resistance around the fibres, and, consequently, higher tensile stresses are developed in the fibres.
3. The *CBR* strength of soil reinforced with bitumen-coated fibres is slightly lower compared with that for soil reinforced with plain fibres. This is because of the reduction of frictional resistance between fibres and soil particles as result of bitumen coating on fibres. With an increase in fibre length, the effect of coating of fibres with bitumen decreases. For example, in soil specimens reinforced with 1% fibre of 40-mm length, coating of fibres with bitumen causes a negligible reduction.
4. Submergence of unreinforced and reinforced soil specimens causes the *CBR* strength to decrease considerably.
5. Dry and wet conditions and semi-saturation condition have no effect on *CBR* strength.

The following points regarding the *CBR* behaviour of fibre-reinforced soils are worth mentioning:

1. The *CBR* value increases nearly linearly to a maximum of six times that of unreinforced soil at fibre content of 0.8% for 1.5-in. long PP fibres. The *CBR* test values indicate that inclusion of fibres is most effective in sandy soils and less effective in the fine-grained soils (Hoover et al. 1982).
2. The addition of PP fibres improves the bearing capacity of the soil by an increase in *CBR* values of as much as 133% (Fletcher and Humphries 1991).
3. The inclusion of fibres increases the *CBR* value of dune sand, and the improvement in the *CBR* can be maintained over a larger penetration range than with unreinforced sand (Al-Refeai and Al-Suhaibani 1998).
4. Addition of waste HDPE plastic strip inclusions (lengths = 12, 24, 36 mm; width = 12 mm; thickness = 0.45 mm) in the stone dust/fly ash overlying saturated clay subgrade results in an appreciable increase in the *CBR* value and the secant modulus for strip content up to 2%. Reinforced stone dust is more effective than

reinforced fly ash overlying saturated clay in improving the behaviour of the system (Dutta and Sarda 2007).

5. The PP plastic bag waste fibre-reinforced stabilized silty soil meets the requirements as subbase and base course materials in terms of its *CBR* values. The *CBR* value of the soil increases up to 3.6 times by mixing lime and rice husk ash. However, the *CBR* value increases considerably up to 8.7 times by adding plastic waste fibres (length = 20–30 mm; width = 2–2.5 mm). The optimum fibre content is found to be in the range of 0.4 to 8% (Muntohar et al. 2013).
6. Improvement in *CBR* value is five times for clay and three times for fly ash with the addition of tire crumbles (dust or powered waste material produced during the process of tire retreading, equivalent to poorly graded sand classified as SP). The *CBR* value of any mix increases with an increase in the content of tire crumbles up to a certain limit of the tire crumbles content, approximately 5%, known as the optimum content, after which further improvement in *CBR* value is not significant. The *CBR* of fly ash-tire crumble mix is relatively greater as compared to the *CBR* of clay-tire crumble mix (Priyadarshee et al. 2015).
7. The *CBR* value of high compressibility clayey soil (CH) increases from 4.70 to 7.75% as a result of inclusion of about 2% human hair fibre (length = 25 mm; diameter = 0.05 mm). The increase in *CBR* may be caused by improved interfacial adhesion between soil particles and hair fibres. Both the undrained shear strength and the *CBR* reduce when the fibre content is more than 2%. It appears that a higher fibre content leads to a reduction in interfacial adhesion between soil particles and fibres in the reinforced soil. Thus 2% fibre content is the optimum quantity to enhance undrained shear strength and *CBR* of clayey soil (Butt et al. 2016).

3.8 Load-Carrying Capacity

Load-bearing capacity and settlement characteristics of fibre-reinforced soil have been studied by several researchers by conducting plate load tests. For the analysis and design of shallow foundations on fibre-reinforced soil beds, the effect of fibres on the load-bearing capacity of the footing resting on reinforced soil is generally represented by the bearing capacity ratio (*BCR*), which is a nondimensional parameter defined as

$$BCR = \frac{q_{\mathrm{R}}}{q_{\mathrm{uU}}} \tag{3.19}$$

where q_{uU} is the ultimate load-bearing capacity of the unreinforced soil and q_{R} is the load-bearing pressure on the reinforced soil corresponding to the settlement $\rho(=\rho_{\mathrm{uU}})$ of the footing resting on unreinforced soil at the ultimate load-bearing capacity q_{uU}.

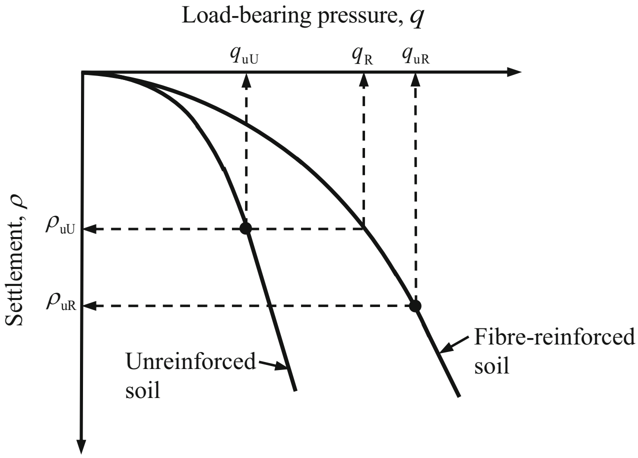

Fig. 3.22 Typical load-settlement curves for unreinforced and fibre-reinforced soils

Figure 3.22 shows the typical load-settlement curves for a soil with and without fibre reinforcement and illustrates how q_{uU} and q_{R} are determined. The procedure for determination of ultimate load-bearing capacity q_{uR} of the reinforced soil and the corresponding settlement ρ_{uR} is also shown in Fig. 3.22.

Note that in Eq. (3.19), q_{R} can be the load-bearing pressure on the reinforced soil at any selected/permissible settlement of the footing as required for the safety of the specific structure. For $q_{\mathrm{R}}=q_{\mathrm{uR}}$ at $\rho=\rho_{\mathrm{uR}}$, Eq. (3.19) reduces to *ultimate bearing capacity ratio* as

$$BCR_{\mathrm{u}} = \frac{q_{\mathrm{uR}}}{q_{\mathrm{uU}}} \tag{3.20}$$

Hence it is important to clarify the definition of *BCR* being used in discussion as the values obtained from Eq. (3.19) and (3.20) can be quite different. In the author's opinion, it is better to use Eq. (3.19), considering the same level of settlement levels in both reinforced and unreinforced cases for describing the benefits of reinforcement in terms of an increase in load-bearing capacity compared to the load-bearing capacity of unreinforced soil.

The improvement in load-bearing capacity of soil as a result of inclusion of fibre reinforcement can also be expressed in terms of the *ultimate bearing capacity improvement factor* I_{UBC}, defined as

$$I_{\mathrm{UBC}} = \frac{\Delta q_{\mathrm{u}}}{q_{\mathrm{uU}}} = \frac{q_{\mathrm{uR}} - q_{\mathrm{uU}}}{q_{\mathrm{uU}}} = \frac{q_{\mathrm{uR}}}{q_{\mathrm{uU}}} - 1 = BCR_{\mathrm{u}} - 1 \tag{3.21}$$

The factor I_{UBC} basically presents a relative gain in load-bearing capacity of soil due to inclusion of fibres and is normally expressed as a percentage.

As explained earlier, the fibre-reinforced soil with or without cementing material exhibits ductile behaviour to some extent, which can be described by deformability index (D_{i}), which is a nondimensional parameter defined as

$$D_{\mathrm{i}} = \frac{\rho_{\mathrm{uR}}}{\rho_{\mathrm{uU}}} \tag{3.22}$$

Note that the concept of *BCR* as defined in Eq. (3.19) was first introduced by Binquet and Lee (1975a, b) in their study of load-bearing capacity of sand bed reinforced with metallic reinforcement layers. The concept of D_{i} has been explained by Nasr (2014).

A series of laboratory model tests on a strip footing resting on the compacted uniform sand bed reinforced by randomly distributed PP fibres (50 mm long) and two different mesh elements (small mesh as 30 mm × 50 mm and big mesh as 50 mm × 100 mm) having the same opening size (10 mm × 10 mm) were conducted by Wasti and Butun (1996). The test results show that the change in the ultimate bearing capacity lies between about +40% and −5%; the higher value is for the big meshes at inclusion contents of 0.1% and 0.15%, and the negative value is for the fibres and the small meshes at the lowest inclusion content of 0.075%. At all inclusion contents, the use of big meshes brings about the greatest improvement compared to others. At an inclusion content of 0.15%, the BCR_{u} value remains the same as for inclusion content of 0.1% for the big meshes and drops somewhat for the small meshes, suggesting the existence of a possible optimum amount of inclusion content for the mesh elements between 0.1% and 0.15%. Unlike the meshes, the BCR_{u} values for the sand with fibres increase linearly as the inclusion content increases. It is reasonable to assume that the BCR_{u} value for the fibres will approach an asymptotic upper limit as observed by Gray and Al-Refeai (1986), Maher and Woods (1990) and Ranjan et al. (1994). It is also observed that the settlement of the footing at failure increases as the inclusion content increases. The increase is also seen to be dependent on the type of reinforcement; the largest one corresponding to the big meshes and the smallest to the fibres.

The plate load tests conducted by Consoli et al. (2003a) using a 300-mm diameter plate show that the addition of PP fibres to the low plasticity silty sandy clayey soil significantly improves the behaviour of soil. A noticeable stiffer response with increasing settlement of the test plate is observed. This happens because of a combined effect of the continuous increase in the strength of the soil at large deformations as observed in the triaxial compression tests.

Consoli et al. (2003b) presented the results of three plate load tests using a circular plate (300-mm diameter; 25.4 mm thick) on a residual homogeneous soil stratum (sandy silty red clay, classified as low plasticity clay) as well as on a layered system formed by two different top layers (300 mm thick) as uniform fine sand-cement and uniform fine sand-Portland cement-PP fibre mixtures overlying a residual soil stratum. The test results, as presented in Table 3.10, show that the addition of fibres to the cemented sand layer over the residual soil stratum keeps the

Table 3.10 Plate load test results for 300-mm circular plate (After Consoli et al. 2003b)

Soil mixtures	Thickness (mm)	Curing period (days)	Load at failure (kN)	Settlement at failure (mm)	Failure mode
None (plate directly on the residual soil) (Consoli et al. 1998b)	None	None	Larger than 40	Larger than 50	Punching
Sand + 7% cement layer	300	28	98	8	Tension fissures initiating on the bottom of sand-cement layer followed by punching
Sand + 7% cement + 0.5% PP fibres	300	28	91	22	Formation of a shear band around the plate border, transferring a higher load to a larger area of the residual soil underlying the cement-stabilized fibre-reinforced soil layer

maximum bearing capacity virtually unchanged but increases settlement at maximum load and improves the ultimate bearing capacity, when compared to the cemented sand layer overlying the residual soil stratum. The fibre reinforcement significantly changes the failure mechanism by preventing the formation of tension cracks, as observed for the cemented sand layer. Instead, the fibres allow the applied load to spread through a larger area at the interface between the fibre-reinforced cemented sand layer and the underlying residual soil stratum as indicated by the formation of a thick shear band all around the plate during the load test (Fig. 3.23).

Gupta et al. (2006) conducted the model footing tests on the PP fibre-reinforced poorly graded sand at 50% relative density and presented the pressure-settlement curves as shown in Fig. 3.24. The tests were conducted on a square footing (150 mm × 150 mm) in a square test tank of size 1 m × 1 m × 0.35 m (deep). It is observed that for a given load-bearing pressure, the settlement of unreinforced sand is more than that of the reinforced sand, and the settlement reduces with an increase in fibre content. The ultimate load-bearing capacity of the unreinforced and reinforced sands determined by the method of plotting the load-bearing pressure versus settlement curve on a log-log graph is given in Table 3.11. It is noted that the ultimate load-bearing capacity of the fibre-reinforced soil increases with an increase in fibre content. The percentage increases in the load-bearing capacity of the fibre-reinforced sand in comparison to unreinforced sand, that is, the value of I_{UBC} is also given in Table 3.11. A plot of I_{UBC} with fibre content is almost linear, although direct shear and triaxial compression tests do not show a linear trend for increase in strength with an increase in fibre content. This suggests that the tests on small specimens may not be a true indicator for prediction of improved strength of soil as a result of fibre inclusions.

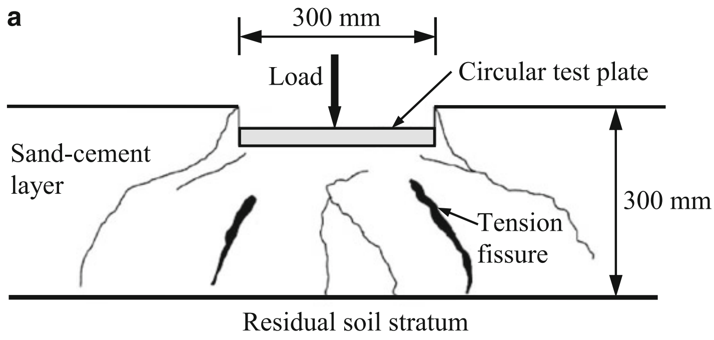

Fig. 3.23 Failure modes for treated soil layer overlying the residual soil stratum: (**a**) sand-cement layer; (**b**) sand-cement-fibre layer (Adapted from Consoli et al. 2003b)

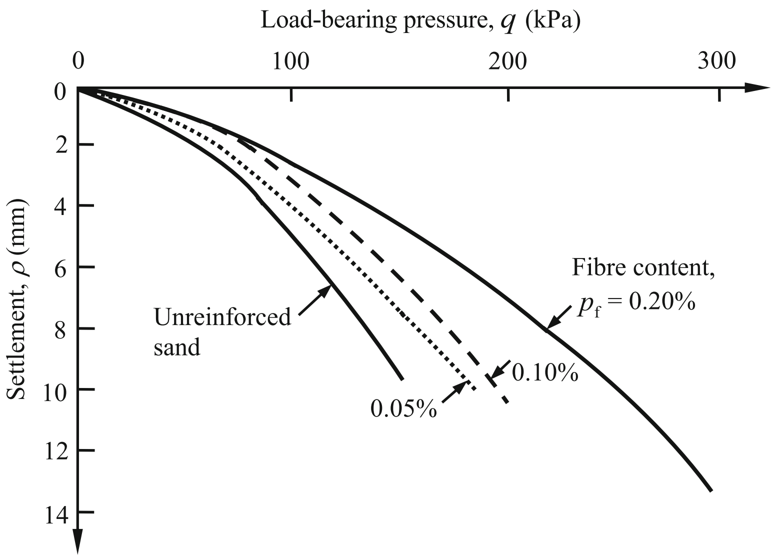

Fig. 3.24 Load-settlement curves for PP fibre-reinforced poorly graded sand at different fibre contents (Adapted from Gupta et al. 2006)

Table 3.11 Effect of increase in fibre content on ultimate load-bearing capacity of poorly graded sand (Adapted from Gupta et al. 2006)

Fibre content (%)	Ultimate load-bearing capacity (kPa)	Increase in load-bearing capacity in comparison to unreinforced sand* as a result of fibre inclusions, that is, I_{UBC} (%)
0	85*	
0.05	100	17.6
0.1	115	35.3
0.2	140	64.7

A series of laboratory model footing tests have been carried out by Hataf and Rahimi (2006) to investigate the load-bearing capacity of sand reinforced randomly with tire shreds of rectangular shape (widths of 20 and 30 mm with aspect ratios of 2, 3, 4 and 5). The shred contents used were 10, 20, 30, 40 and 50% by volume of the dry sand. The results show that addition of tire shreds to sand increases the *BCR* value from 1.17 to 3.9. The maximum *BCR* value is obtained for the shred content of 40% and shred dimensions of 30 mm by 120 mm.

Kumar and Singh (2008) conducted plate load tests on prepared fly ash subbases in test pits with and without fibre reinforcement. It is observed that the modulus of subgrade reaction k_s increases by including the fibres randomly in fly ash and fly ash-soil (poorly graded fine sand (SP)) mixes. The value of k_s at 0.2% fibre content increases by 15.47, 21.4 and 51.2% for fly ash, fly ash + 15% soil and fly ash +25% soil, respectively.

Consoli et al. (2009) carried out the plate load tests on unreinforced uniform fine sand (SP) and this sand reinforced with PP fibres (24 mm long, 0.5% by dry weight), compacted at relative densities (D_r) of 30, 50 and 90%, using a 300-mm diameter circular rigid steel plate. The test results indicate that the load-settlement behaviour of sand is significantly governed by the fibre inclusion, changing the kinematics of failure. The best performance was obtained for the densest ($D_r = 90\%$) fibre-sand mixture, where a significant change in the load-settlement behaviour was observed at very small (almost zero) displacements. However, for the loose to medium dense sand ($D_r = 30\%$ and 50%), significant settlements (50 mm and 30 mm, respectively) were required for the differences in the load-settlement responses to appear. The settlement required for this divergence to occur could best be represented using a logarithmic relationship between settlement and relative density. The overall behaviour seems to support the argument that inclusion of fibres increases strength of sandy soil by a mechanism that involves the partial suppression of dilation and hence produces an increase in effective confining pressure and a consequent increase in shear strength.

Falorca et al. (2011) carried out plate load tests on randomly distributed monofilament PP fibre-reinforced silty sand for small displacements and relatively low load levels using a semi-rigid circular plate, 300 mm in diameter. The fibres were very fine (fibre diameter of the order of 10 μm). Long fibres (75 mm long), that is, fibres with a high aspect ratio, were used, because they were expected to be more effective for soil reinforcement as shown by several past studies. Fibre content was varied from 0 to 0.5%. The reinforced soil was compacted by a 4 t vibratory roller with six passes in lift thicknesses of 200 mm to construct a trial embankment (50 m long, 10 m wide and 0.6 m high), divided into five different sections, each with different reinforcement characteristics. Both monotonic loading and load-unload cycles were adopted. Each load level was applied for at least 10 min before measurements were taken. The modulus of subgrade reaction k_s varied from 170 MPa/m for unreinforced soil to 208 MPa/m for fibre-reinforced soil with corresponding Young's modulus of elasticity from 130 MPa to 115 MPa. The Young's modulus decreases with an increase in fibre content, this reduction being more pronounced for thinner fibres. The test results show that the soil reinforced with the thinner fibres is the more compressive one. Although the reinforced soil is more compressible than unreinforced soil, the reinforced soil shows a considerably large recovery as indicated during repeated loading and unloading. Note that an increase in fibre content affects the physical properties of the compacted soil by essentially increasing the void ratio.

Example 3.3
Consider the following values of ultimate bearing capacity obtained from a plate load test on unreinforced and fibre-reinforced soils:

For unreinforced soil, $q_{uU} = 85\ \text{kPa}$
For fibre-reinforced soil, $q_{uR} = 140\ \text{kPa}$

Determine the ultimate bearing capacity ratio and ultimate bearing capacity improvement factor. What does this factor indicate?

Solution
From Eq. (3.20), the ultimate bearing capacity ratio is

$$BCR_u = \frac{q_{uR}}{q_{uU}} = \frac{140}{85} \approx \mathbf{1.65}$$

From Eq. (3.21), the ultimate bearing capacity improvement factor

$$I_{UBC} = \frac{q_{uR}}{q_{uU}} - 1 = BCR_u - 1 = 1.65 - 1 = \mathbf{0.65}\ \text{or}\ \mathbf{65}\%$$

This value of I_{UBC} indicates that inclusion of fibres causes 65% increase in the ultimate bearing capacity of soil.

3.9 Other Properties

The behaviour of fibre-reinforced soils has also been studied by conducting tests other than those presented in the previous sections. Such tests are Brazilian/indirect/splitting tensile strength tests, ring shear tests, bender element tests, resonant column tests, torsional shear tests, etc. Some key findings of the studies based mainly on these tests as well as some additional characteristics are summarized below:

1. The inclusion of natural and glass fibres has a significant effect on the dynamic response of a uniform dune sand, namely, shear modulus and damping ratio. Both the shear modulus and the damping ratio increase approximately linearly with an increasing amount of fibres to about 4%, then they tend to approach an asymptotic upper limit at approximately 5% of fibre content by weight. At higher fibre content, the reinforcing or stiffening effects are offset by a decrease in composite density and a dilution or loss of interparticle friction between the sand particles themselves. An increase in fibre aspect ratio results in more effective fibre contribution to the dynamic response of sand, that is, shear modulus and damping ratio. An increase in fibre modulus results in increased fibre contribution to shear modulus of fibre-reinforced sand, whereas the effect of fibre modulus on damping ratio is insignificant. The dynamic response of sand reinforced with vertically oriented fibres is very similar to that of randomly distributed fibres (Maher and Woods 1990). Noorany and Uzdavines (1989) and Shewbridge and Sousa (1991) have also reported increase in dynamic shear modulus as well as liquefaction resistance of cohesionless soils as a result of fibre inclusion. Note that the observations made by Shewbridge and Sousa (1991) are based on the cyclic torsional shear strain control tests on large hollow cylindrical sand specimens reinforced uniaxially and biaxially with 3.175-mm (0.125-in.) diameter steel rods.
2. The tensile strength of sand increases significantly as a result of fibre inclusion. Increase in tensile strength is more pronounced for higher fibre contents, longer fibre lengths and higher cement contents, as seen in Fig. 3.25 (Maher and Ho 1993).
3. The tensile strength of kaolinite clay increases significantly as a result of inclusion of fibre reinforcement (PP fibres, glass fibres and softwood pulp fibres). In general, increasing the fibre content increases the tensile strength of fibre-reinforced kaolinite clay, with the increase being more pronounced at lower water contents. This happens because increasing water content reduces the amount of load transfer between the kaolinite particles and the fibres. Increasing fibre length reduces the contribution of fibres to tensile strength. Thus for the same amount of fibres present in the reinforced soil, shorter, and consequently better, dispersed fibres contribute more to tensile strength (Maher and Ho 1994).
4. The toughness of kaolinite clay beam increases as a result of inclusion of fibres (PP fibres, glass fibres and softwood pulp fibres) as noticed in flexural load test using three-point loading method. When a fibre-reinforced kaolinite clay beam is loaded, the fibres act as crack arrestors and thus increase the toughness (Maher and Ho 1994).

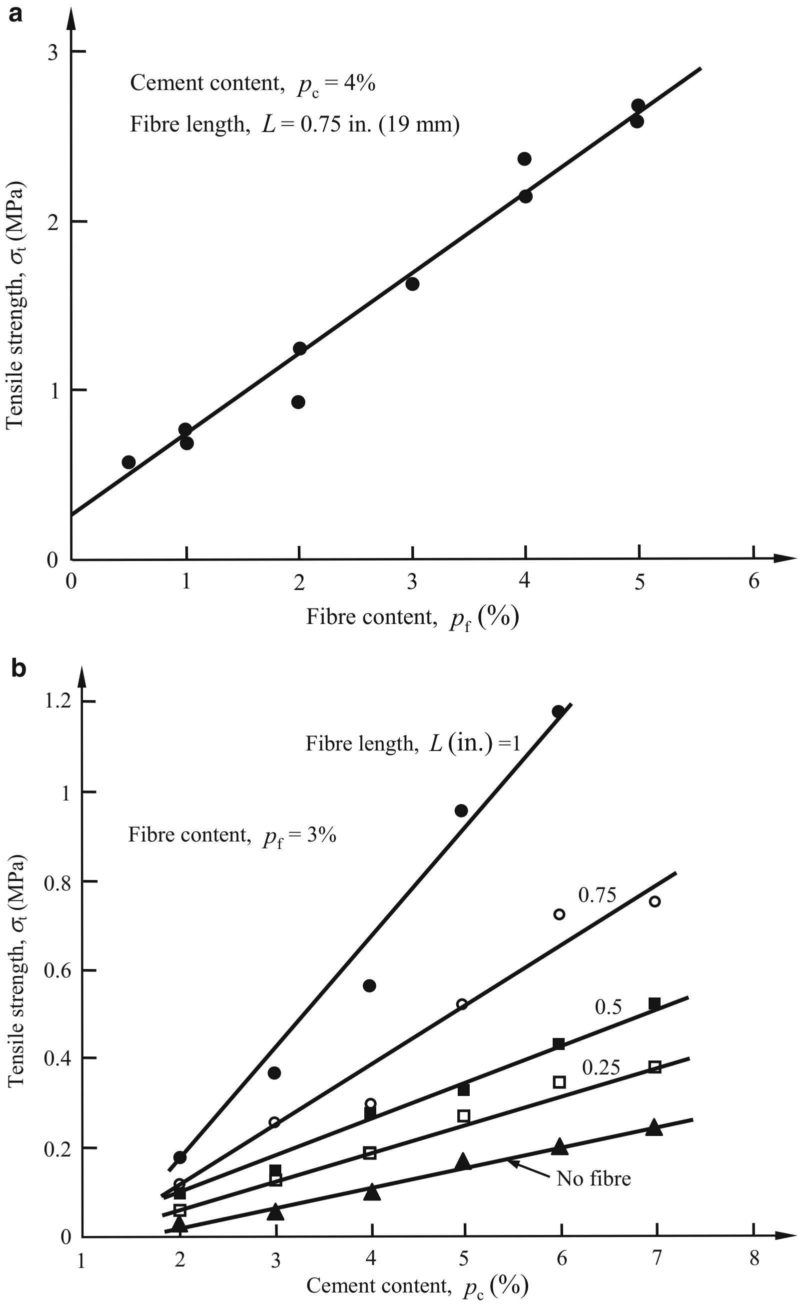

Fig. 3.25 Effect of glass fibre inclusion on the tensile strength of cement-stabilized standard Ottawa sand: (**a**) effect of fibre content; (**b**) effect of fibre length and cement content (Adapted from Maher and Ho 1993)

5. An increase in PET fibre content from 0.1 to 0.9% in uniform fine sand (SP) with an intermediate cement content of 5% causes an average increase in tensile strength of sand by about 78%. The positive effect of fibre length is not detected in the splitting tensile tests, clearly indicating the major influence of confining pressure and the necessity of carrying out triaxial tests to fully show fibre-reinforced soil behaviour (Consoli et al. 2002).
6. At very small shear strains, the bender element test data obtained by Heineck et al. (2005) show that the inclusion of PP fibres in soil does not influence the stiffness at small strains as determined from the stress-strain curves of the triaxial compression tests.
7. The ring shear tests on PP fibre-reinforced soils (Botucatu residual soil and uniform Osorio sand) show that there is a continued increase of shear stress with shear strain, even at the largest strains reached in the ring shear apparatus. In contrast, for the reinforced bottom ash at lower confining stresses, the shear stress reaches an approximately constant value of shear stress. Higher strains, in general, result in greater mobilization of the tensile resistance of the fibres, which in turn results in an enhanced contribution of the fibres to the stability/rigidity of the reinforced soil (Heineck et al. 2005; Consoli et al. 2007).
8. The tensile strength (maximum/peak stress at failure) and toughness related to elastic energy for failure (area under the stress-strain curve till the maximum/peak stress at failure) of cement-stabilized clayey silty soils increases significantly by inclusion of PP and processed cellulose fibres, thus indicating a high resistance to tensile cracking (Khattak and Alrashidi 2006).
9. The beam/flexural tests indicate that the improvement in tensile strain at crack initiation of fine-grained soils (kaolin-sand mixes) is found to depend upon soil type, moulding water content, fibre content and aspect ratio. For soil beams without any fibres, the tensile strain at crack initiation is found to be in the range of 0.83–1.09%. For soil beams reinforced with discrete, randomly distributed PP fibres, it was found to be in the range of 0.97–2.27%. This indicates the significant influence of fibres in enhancing the tensile strength-strain characteristics of moist-compacted soil beams (Viswanadham et al. 2010).
10. The ratio of *UCS* to split tensile strength (*STS*) for PP fibre (7 mm long) – poorly graded sand (PP) – fat clay mixtures varies between 3.2- and 3.5-fold. For 1.0% fibre inclusion, the ratio of *UCS* to *STS* is found to be minimum for 2.5% sand content. On the other hand, for 0.25% fibre inclusion, the ratio of *UCS* to *STS* is observed as the maximum for 7.5 and 10% sand content (Yilmaz 2009).
11. The Brazilian tensile strength test results indicate that the maximum improvement in tensile strength of cement-stabilized fibre-reinforced fly ash occurs by addition of about 1% PP fibre, 60 mm in length, for both soaked and unsoaked conditions (Chore and Vaidya 2015).

Chapter Summary

1. Improvement of engineering properties of soil due to the random inclusion of discrete fibres is a function of a large number of parameters/factors relating soil and fibre characteristics; fibre concentration and orientation; the presence of admixtures, if any; method of mixing; type and amount of compaction; and test and field conditions.
2. Contribution of fibres is more effective after a certain level of shear strain depending on the types of fibre and soil. The inclusion of fibres increases the peak strength (shear, compressive, tensile, etc.) of sandy soil by a mechanism that involves the partial suppression of dilation, resulting in an increase in effective confining stress and a consequent increase in shear strength.
3. Inclusion of fibres in soil reduces the post-peak strength loss, increases the axial strain at failure and changes the stress-strain behaviour from strain softening to strain hardening. Stiffness of soil can decrease at low strains by inclusion of fibres.
4. Inclusion of fibres in a cement-stabilized soil improves its overall engineering behaviour by increasing tensile and compressive strengths, peak angle of internal friction angle, cohesion intercept, ductility (energy absorption capacity), toughness (resistance to impact) and resistance to cyclic loading (fatigue). Fibres do not prevent the formation of cracks, but they control directly the crack propagation and improve the post-cracking properties.
5. *UCS* of fibre-reinforced clay soil is highly dependent on dry unit weight, water content and fibre content. At a constant dry unit weight and water content, an increase in the fibre content results in a significant increase in the *UCS* value. An increase in the dry unit weight of reinforced clay specimens prepared at a constant water content and fibre content results in a significant increase in *UCS*. An increase in water content of reinforced specimens at the same dry unit weight and fibre content results in a reduction in the *UCS*.
6. Failure occurs by frictional slipping of fibres for confining stresses up to a threshold value referred to as the critical confining stress. For stresses greater than the critical confining stress, the failure takes place by the rupture of the fibres, that is, the failure is governed by the tensile strength of the fibres.
7. In general, increasing the fibre content leads to a reduction in the maximum dry unit weight and an increase in optimum water content.
8. Seepage velocity of water through a fibre-reinforced soil mass is governed by fibre content, fibre length and hydraulic gradient. An increase in fibre content causes a decrease in seepage velocity and an increase in piping resistance of soil.
9. Clayey soils mixed with fibres show significantly lower swelling pressure and swell potential in comparison with the same clayey soils without fibres.
10. Inclusion of fibres improves the *CBR* of the soil. The *CBR* value increases significantly with increasing fibre content up to a specific value (say, around 0.75–1%) and remains more or less constant thereafter.
11. Addition of fibres to soil significantly improves its load-carrying capacity. For a given load-bearing pressure, the settlement of unreinforced soil is more than

that of the reinforced soil, and the settlement reduces with an increase in fibre content. The ultimate load-bearing capacity of the fibre-reinforced soil increases with an increase in fibre content up to a specific value.

12. Tensile strength of soil increases significantly as a result of fibre inclusion. For the same amount of fibres present in the reinforced soil, shorter and better dispersed fibres contribute more to the tensile strength.
13. Inclusion of natural and glass fibres has a significant effect on the dynamic response of sand, namely, shear modulus and damping ratio, which increase approximately linearly with an increasing amount of fibres up to a specific value of fibre content.
14. Improvement contributed by the presence of fibres is highly anisotropic because of preferred sub-horizontal fibre orientations in compacted condition.
15. In clayey soils, the use of elevated moisture/water content, called the optimum mixing moisture/water content (OMMC), may be required to produce effective fibre-soil mixing.
16. Though the basic characteristics of fibre-reinforced soils have been studied and reported by several researchers, the findings vary to some extent depending on the variations in the parameters, but some observations also appear to be contradictory. However, the present knowledge of fibre-reinforced soils provides the basic understanding of several factors to be analysed in detail while working for any specific field application of fibre-reinforced soils.

Questions for Practice

(Select the most appropriate answer to the multiple-choice questions from Q 3.1 to Q 3.5.)

3.1. Within the practical ranges of fibre content, in general, the shear strength of fibre-reinforced soil

(a) Increases with increase in the fibre content, but decreases with an increase in the fibre aspect ratio
(b) Decreases with increase in the fibre content, but increases with an increase in the fibre aspect ratio
(c) Increases with increase in both the fibre content and the fibre aspect ratio
(d) Decreases with increase in both the fibre content and the fibre aspect ratio

3.2. Inclusion of PP fibres in sand generally increases its

(a) Strength and stiffness
(b) Strength and ductility
(c) Stiffness and ductility
(d) Strength, stiffness and ductility

3.3. An increase in the dry unit weight of fibre-reinforced clay specimens prepared at a constant water content and fibre content causes the unconfined compressive strength to

(a) Decrease significantly
(b) Decrease slightly
(c) Increase significantly
(d) Increase slightly

3.4. The optimum fibre content for achieving the maximum crack reduction and maximum dry unit weight of clayey soil, while maintaining the acceptable hydraulic conductivity, is typically between

(a) 0.4–0.5%
(b) 0.5–1%
(c) 1–2%
(d) 2–4%

3.5. In general, the *CBR* value of a reinforced soil continue to

(a) Increase with fibre content, but decrease with aspect ratio
(b) Decrease with fibre content, but increase with aspect ratio
(c) Decrease with both fibre content and aspect ratio
(d) Increase with both fibre content and aspect ratio

3.6. List the factors/parameters which govern the engineering behaviour of fibre-reinforced soil.
3.7. Enumerate the fibre characteristics that may affect the engineering behaviour of fibre-reinforced soils.
3.8. Illustrate with neat sketches the effect of increasing fibre content and fibre length on the following properties of fibre-reinforced soil:

(a) Shear strength
(b) Unconfined compressive strength
(c) Permeability
(d) Compressibility
(e) California bearing ratio
(f) Load-bearing capacity
(g) Tensile strength

3.9. Define the following terms:

(a) Shear stress improvement factor at failure
(b) Deviator stress improvement factor at failure
(c) Deviator stress ratio at failure
(d) Unconfined compressive strength improvement factor
(e) Peak load ratio
(f) *CBR* improvement factor
(g) Bearing capacity ratio

3.10. Explain the interaction mechanisms between fibre and soil particles with the help of a neat sketch.

3.11. Using the data presented in Table 3.4, plot the variation of deviator stress at failure with fibre content for fibre aspect ratio of 100, considering cell pressures of 40, 70, 100 and 140 kPa. Discuss the effect of increase of fibre content on deviator stress. Is the variation linear?

3.12. Consider the following values of the deviator stress at failure:

For unreinforced soil, $(\sigma_1 - \sigma_3)_{fU} = 1.65$ MPa
For fibre-reinforced, $(\sigma_1 - \sigma_3)_{fR} = 0.78$ MPa

Determine the deviator stress improvement factor at failure. What does this indicate?

3.13. What is brittleness index? Explain its importance.

3.14. Compare the engineering characteristics of fibre-reinforced sand and cement-stabilized fibre-reinforced sand.

3.15. What is the difference between strength and stiffness of a soil? How does inclusion of fibres affect them?

3.16. Define the critical confining stress, and explain its significance.

3.17. Draw the typical principal stress envelop for the fibre-reinforced sand as obtained from triaxial compression tests. Comment on the shape of the envelope.

3.18. Why do fibre-reinforced soils have a lower unit weight than unreinforced soils? Explain briefly.

3.19. Do you see any significant difference between strength values of soil reinforced with natural and polymeric fibres? Explain briefly.

3.20. Differentiate between the stress-strain curves for unreinforced and fibre-reinforced soils as obtained from unconfined compression tests.

3.21. Discuss the effect of fibres on failure pattern of fibre-reinforced soils when loaded.

3.22. How do the compaction parameters of fibre-reinforced soil differ from those for unreinforced soil?

3.23. Explain the effect of fibre inclusion on the critical hydraulic gradient and piping resistance of soil.

3.24. Can fibre inclusion control the swelling potential of expansive soils? Justify your answer.

3.25. Polypropylene fibres are available in the following two forms: (i) $L = 15$ mm, $D = 0.05$ mm and (ii) $L = 25$ mm, $D = 0.1$ mm, where L and D are length and diameter of fibres, respectively. If a sandy soil is to be reinforced, which form of the fibres should be recommended for strength improvement? Justify your answer.

3.26. What is the effect of fibre inclusion on the *CBR* value of high compressibility clayey soil?

3.27. What is deformability index? Explain its importance.

3.28. Consider the following values of ultimate bearing capacity obtained from a plate load tests on unreinforced and fibre-reinforced soils:

For unreinforced soil, $q_{uU} = 85\ \text{kPa}$
For fibre-reinforced soil, $q_{uR} = 115\ \text{kPa}$

Determine the ultimate bearing capacity ratio and ultimate bearing capacity improvement factor. What does this factor indicate?

3.29. Discuss the failure modes as observed in the plate load tests on cement-stabilized fibre-reinforced sand bed.
3.30. Draw the typical load-settlement curves for unreinforced and fibre-reinforced sands as obtained from plate load tests. Discuss the effect of fibres.
3.31. How does fibre inclusion affect the dynamic response of dune sand?
3.32. What is the effect of fibre inclusion on the tensile strength of cement-stabilized fibre-reinforced sand?
3.33. How can you investigate the stress-strain behaviour of fibre-reinforced soil at large strains? Name the test which may be suitable for this study?

Answers to Selected Questions

3.1 (c)
3.2 (b)
3.3 (c)
3.4 (a)
3.5 (d)
3.12 112%, 112% increase in deviator stress
3.26 length = 15 mm, diameter = 0.05 mm
3.29 1.35, 35%

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Chapter 4
Soil Reinforcing Mechanisms and Models

4.1 Introduction

In Chap. 3, you have learnt that when the fibres are included within a soil mass, they cause changes in the properties (e.g. permeability, compressibility, shear strength, etc.) of soil. The presence of fibres in a soil mass improves its stability, increases its load-carrying capacity and reduces its deformation (i.e. settlement and lateral movement). The effect of fibre inclusions on strength and stiffness behaviours of soil is explained through the role of fibres as a reinforcement element. This chapter presents the basic mechanisms and models of soil reinforcement, focusing on the random distribution of fibres within the soil mass, resulting in *randomly distributed fibre-reinforced soil* (RDFRS), or simply the *fibre-reinforced soil* (FRS), as explained in the previous chapters.

4.2 Basic Soil Reinforcing Mechanisms

If the reinforced soil is considered a homogeneous material, but with anisotropic characteristics, the Mohr-Coulomb failure criterion can be applied to explain the basic mechanism of reinforced soil. Consider a simplified situation, shown in Figs. 4.1a, b, where two cylindrical specimens of a cohesionless soil are subjected to the same triaxial loading with the minor principal stress (confining stress) σ_3 and the major principal stress σ_1. The first soil specimen is not reinforced, but the second one is reinforced with fibres. Figure 4.1c shows a magnified view of the reinforced soil element, as indicated in Fig. 4.1b, with a horizontal fibre. Because of skin friction and/or adhesion between the fibre and the soil, the fibre applies a confining stress σ_R (=$\Delta\sigma_3$, increase in the minor principal stress) on the soil, and in this process, the fibre gets stretched with mobilization of a tensile force T_f as shown

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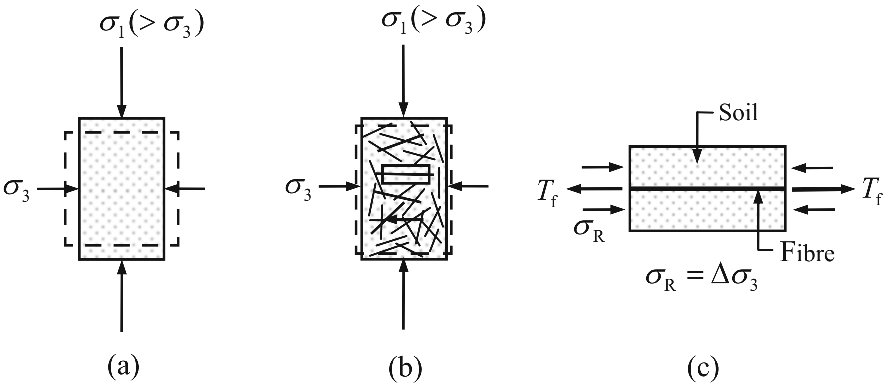


Fig. 4.1 Basic soil reinforcing mechanism: (**a**) unreinforced cylindrical soil specimen; (**b**) fibre-reinforced cylindrical soil specimen; (**c**) magnified view of a reinforced soil element, as indicated in (**b**)

in Fig. 4.1c. Note that the tensile force T_f and hence the confining stress σ_R will vary with orientation of the fibre within the soil mass.

Assume that the Mohr-Coulomb failure criterion has been attained in the unreinforced soil specimen. For this case, the stress state within the soil mass can be represented, in the normal stress (σ) and shear stress (τ) space, by a Mohr circle 'a' as shown in Fig. 4.2, which is tangent to the Mohr-Coulomb failure envelope l_U for the unreinforced soil. If the reinforced soil specimen is subjected to the same stress state, then due to skin friction and/or adhesion bonding between both constituents, the lateral deformation/strain of the specimen reduces. The lateral deformation of the reinforced soil specimen is generally greater than that of the reinforcement but smaller than the lateral deformation of the unreinforced soil specimen. This means that in the case of perfect friction and/or adhesion bonding between reinforcement and soil, the reinforcement is extended, resulting in a mobilized tensile force T_f, and the soil is compressed by additional compressive lateral stress as the reinforcement restraint σ_R ($= \Delta\sigma_3$), introduced into the soil mass in the direction of the reinforcement as shown in Fig. 4.1c. The stress state in soil represented by the Mohr circle 'b' in Fig. 4.2 is no more tangent to the failure envelope l_U, and hence the reinforced soil specimen is able to sustain greater stresses than those in the case of unreinforced soil.

Consider that the reinforced soil specimen, shown in Fig. 4.1b, is expanding horizontally due to a decrease in applied horizontal stress σ_3 with a constant vertical stress σ_1, and assume that the failure occurs by rupture of the reinforcement, that is, the lateral restraint σ_R is limited to a maximum value σ_{RCmax} depending on the strength of the reinforcement. This state of stress is represented by the Mohr circle 'c' in Fig. 4.2. The strength increase can be characterized by a constant cohesion intercept c_R as an apparent cohesion, introduced due to reinforcement (Schlosser and Vidal 1969). The results obtained from both the triaxial compression tests and the direct shear tests on sand specimens reinforced with tensile inclusions have

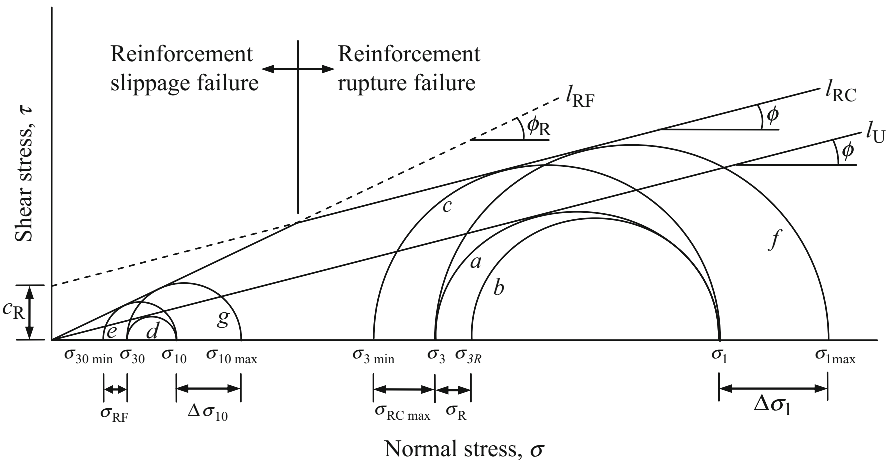


Fig. 4.2 Basic mechanism of fibre-reinforced soil: Mohr circles for reinforced and unreinforced cases

shown that the apparent cohesion of the reinforced soil is a function of the orientation of the inclusions with respect to the direction of the maximum extension in the soil (Long et al. 1972; Schlosser and Long 1974; Jewell 1980; Gray and Al-Refeai 1986). Thus, the strength envelope for a reinforced cohesionless soil for reinforcement rupture condition can be interpreted in terms of the Mohr-Coulomb failure envelope l_{RC} for the homogeneous cohesive soil as shown in Fig. 4.2.

For Mohr circle '*a*', the principal stresses σ_1 and σ_3 are related as

$$\sigma_1 = \sigma_3 \tan^2(45^\circ + \phi/2) \tag{4.1}$$

where ϕ is the angle of shearing resistance (or angle of internal friction) of the unreinforced soil.

For Mohr circle '*c*', representing the stress state of the reinforced soil at failure, the principal stresses σ_1 and σ_{3min} are related as

$$\sigma_1 = \sigma_{3min} \tan^2(45^\circ + \phi/2) + 2c_R \tan(45^\circ + \phi/2) \tag{4.2}$$

Since $\sigma_{3min} = \sigma_3 - \sigma_{RCmax}$, as seen in Fig. 4.2, Eq. (4.2) becomes

$$\sigma_1 = (\sigma_3 - \sigma_{RCmax}) \tan^2(45^\circ + \phi/2) + 2c_R \tan(45^\circ + \phi/2) \tag{4.3}$$

Combining Eqs. (4.1) and (4.3) leads to

$$c_R = \frac{\sigma_{RCmax} \tan(45^\circ + \phi/2)}{2} = \frac{\sigma_{RCmax}\sqrt{K_p}}{2} = \frac{\sigma_{RCmax}}{2\sqrt{K_a}} \tag{4.4}$$

where

$$K_a = \tan^2(45^\circ - \phi/2) \tag{4.5}$$

and

$$K_p = \tan^2(45^\circ + \phi/2) \tag{4.6}$$

Note that K_a and K_p are the Rankine's coefficients of active and passive lateral earth pressures, respectively. Also note that the anisotropic cohesion is produced in the direction of reinforcement orientation, and this concept is based on the laboratory shear strength studies on reinforced soil specimens.

You may now consider that the reinforced soil specimen shown in Fig. 4.1b is expanding horizontally due to a decrease in applied horizontal stress $\sigma_3 = \sigma_{30}$ with a constant vertical stress $\sigma_1 = \sigma_{10}$, as represented by the Mohr circle '*d*', and assume that the failure occurs by a slippage between the reinforcement and the soil, that is, the lateral restraint σ_R is limited to σ_{RF}, which is proportional to σ_{10}. Therefore,

$$\sigma_{RF} = \sigma_{10} F \tag{4.7}$$

where F is a friction factor that depends on the cohesionless soil-reinforcement interface characteristics. This concept is based on the Yang's experimental results (Yang 1972) as presented by Hausmann and Vagneron (1977). The failure state of stress is represented by the Mohr circle '*e*' in Fig. 4.2. The strength increase can be characterized by an increased friction angle ϕ_R. Thus, the strength envelope for a reinforced cohesionless soil for the reinforcement slippage condition can be interpreted in terms of the Mohr-Coulomb failure envelope l_{RF} for the homogeneous cohesionless soil as shown in Fig. 4.2.

For Mohr circle '*d*', the principal stresses σ_{10} and σ_{30} are related as

$$\sigma_{10} = \sigma_{30} \tan^2(45^\circ + \phi/2) \tag{4.8}$$

Substituting Eq. (4.5) into Eq. (4.8) yields

$$\sigma_{10} = \frac{\sigma_{30}}{K_a} \tag{4.9}$$

For Mohr circle '*e*', the principal stresses σ_{10} and $\sigma_{30\min}$ are related as

$$\sigma_{10} = \sigma_{30\min} \tan^2(45^\circ + \phi_R/2) \tag{4.10}$$

Since $\sigma_{30\min} = \sigma_{30} - \sigma_{RF}$, as seen in Fig. 4.2, Eq. (4.10) becomes

$$\sigma_{10} = (\sigma_{30} - \sigma_{RF}) \tan^2(45^\circ + \phi_R/2) \tag{4.11}$$

Substituting Eq. (4.7) into Eq. (4.11) yields

$$\sigma_{10} = (\sigma_{30} - \sigma_{10}F)\tan^2(45^\circ + \phi_R/2) \tag{4.12}$$

Combining Eqs. (3.8) and (3.12) leads to

$$1 = (K_a - F)\left(\frac{1 + \sin\phi_R}{1 - \sin\phi_R}\right)$$

or

$$\sin\phi_R = \frac{1 + F - K_a}{1 - F + K_a} \tag{4.13}$$

You may now consider that the reinforced soil specimen shown in Fig. 4.1b is expanding horizontally due to an increase in applied σ_1 with a constant σ_3, and assume that the failure occurs by rupture of the reinforcement or reinforcement slippage. These failure states of stress are represented by the Mohr circles '*f*' and '*g*' in Fig. 4.2, respectively. Note that the reinforcement increases the compressive strength of the soil by $\Delta\sigma_1$ or $\Delta\sigma_{10}$ depending on the type of failure mode of the reinforced soil as indicated in the figure.

A different concept of the influence of reinforcement on the behaviour of reinforced soil mass was described by Basset and Last (1978). It is suggested that an introduction of reinforcement modifies the dilatancy characteristics of soil with a possible rotation of principal strain directions. This concept is based on the fact that if the dilation of the soil is restricted, the shear strength mobilized will be higher than for the case of no restriction. The presence of reinforcement in soil imposes a condition of restricted dilatancy. It also predetermines the principal incremental strain directions and rotates them relative to the unreinforced case, thereby resulting in a redistribution of stresses.

Note that the behaviour of soil reinforced with extensible reinforcements, such as geosynthetic reinforcements, does not fall entirely within the concepts as described here. For the details of reinforcing mechanisms of geosynthetic-reinforced soils, the readers may refer to the books by Shukla (2002, 2012, 2016) and Shukla and Yin (2006).

Example 4.1

For a reinforced sand, consider the following:

Angle of shearing resistance of unreinforced sand, $\phi = 33^\circ$
Friction factor, $F = 0.1$

Determine the angle of shearing resistance of the reinforced sand.

Solution

From Eq. (4.5), the Rankine's active earth pressure coefficient,

$$K_a = \tan^2(45° - \phi/2) = \tan^2(45° - 33°/2) = 0.295$$

From Eq. (4.13),

$$\sin\phi_R = \frac{1 + F - K_a}{1 - F + K_a} = \frac{1 + 0.1 - 0.295}{1 - 0.1 + 0.295} = 0.674$$

Therefore, angle of shearing resistance of the reinforced sand is

$$\phi_R = \sin^{-1}(0.674) = \mathbf{42.4°}$$

4.3 Basic Models of Fibre-Reinforced Soils

Based on the experimental studies, it has been established that the strength and deformation characteristics of the fibre-reinforced soils are governed by the soil and fibre characteristics as well as by confinement and stress level. The states of stress and strain in fibre-reinforced soils during its deformation and failure are complex. However, for engineering applications, it is possible to explain the mechanism of fibre reinforcements in soils, especially the contribution of fibres to the shear strength increase, and hence the behaviour of fibre-reinforced soils, by mathematical approaches/models. The finite element analyses considering appropriate constitutive relationships can be carried out to investigate the behaviour of fibre-reinforced soils. The commercial software can be used for such analyses without developing the complete codes. In the past, the researchers have attempted to present some simplified models, which are based on different approaches, such as force-equilibrium/mechanistic approach, energy dissipation approach, statistical approach and the approach of superposition of the effects of soil and fibres. Some of these models are presented here.

4.3.1 Waldron Model

The root-reinforced soil mass can be considered a composite material in which the roots of relatively a high tensile strength are embedded in a matrix of lower tensile strength. Waldron (1977) proposed a simple force-equilibrium model to estimate the increase in strength of soil reinforced with non-rigid plant roots, taking into account the tensile force developed in the root reinforcement and considering the same in the Mohr-Coulomb's equation in its modified form as

$$S_R = S + \Delta S = c + \sigma \tan \phi + \Delta S \tag{4.14}$$

where S_R is the shear strength of reinforced soil, S is the shear strength of unreinforced soil, ΔS is the shear strength increase caused by plant root reinforcement, σ is the total normal confining stress applied on the shear/failure plane and c and ϕ are shear strength parameters, namely, the cohesion intercept and angle of shearing resistance, respectively, of the unreinforced soil. This model is based on the observations made when the root-permeated cylindrical soil specimens of 250-mm diameter were brought to zero matric potential and sheared in a large direct shear device. The roots are assumed to be vertical, flexible, elastic, of uniform diameter and extending an equal distance on either side of the horizontal shear plane. Only the partial mobilization of fibre tensile strength is considered depending upon the amount of fibre elongation during the shear. This model does not place any constraint on the distribution or location of the reinforcing fibres.

Note that the plant roots increase the soil shear strength both directly by mechanical reinforcing and indirectly through water removal by transpiration.

4.3.2 Gray and Ohashi (GO) Model

The concept of the Waldron model was extended by Gray and Ohashi (1983) to describe the deformation and the failure mechanism of fibre-reinforced cohesionless soil and to estimate the contribution of fibre reinforcement to increasing the shear strength of soil. The model consists of a long, elastic fibre, extending an equal length over either side of a potential shear plane in sand (Fig. 4.3). The fibre may be oriented initially perpendicular to the shear plane or at some arbitrary angle i with the horizontal. Shearing causes the fibre to distort, thereby mobilizing the tensile resistance in the fibre. The tensile force in the fibre can be divided into components normal and tangential to the shear plane. The normal component increases the confining stress on the failure plane, thereby mobilizing additional shear resistance in the sand, whereas the tangential component directly resists the shear. The fibre is assumed to be thin enough that it offers little if any resistance to shear displacement from the bending stiffness. If many fibres are present, their cross-sectional areas are computed, and the total fibre concentration is expressed in terms of the *fibre area ratio*, A_r, defined by Eq. (2.7) as reproduced below:

$$A_r = \frac{A_f}{A} \tag{4.15}$$

where A_f is the total cross-sectional area of fibres in a plane (e.g. shear/failure plane) within the reinforced soil mass and A is the total area of the plane (e.g. shear/failure plane) within the reinforced soil mass, which includes the soil particles, fibres and voids.

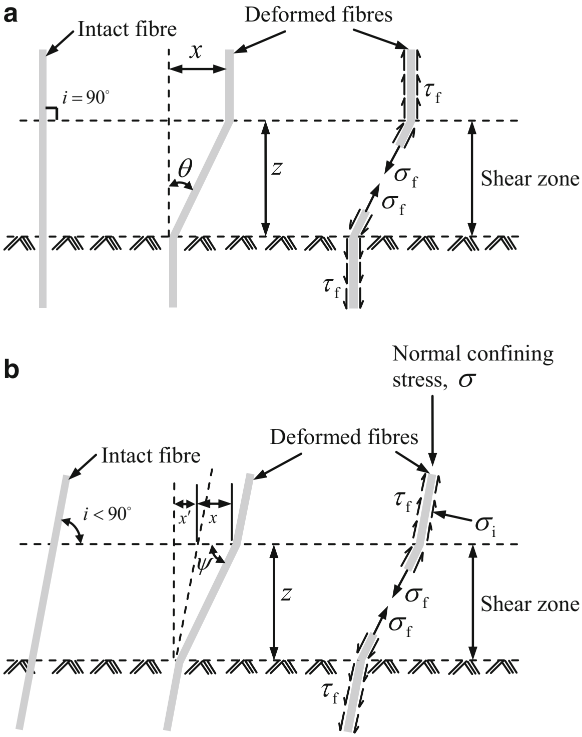


Fig. 4.3 Model for flexible, elastic fibre reinforcement, extending across the shear zone of thickness z: (a) $i = 90°$; (b) $i < 90°$ (Adapted from Gray and Ohashi 1983; Shukla et al. 2009)

The shear strength increase ΔS ($=S_R - S$, S_R and S are shear strengths of reinforced soil and unreinforced soil, respectively) from the fibre reinforcement in sand can be estimated from the following expressions:

$$\Delta S = \sigma_R(\sin\theta + \cos\theta\tan\phi) \tag{4.16}$$

for fibres oriented initially perpendicular to the shear plane (Fig. 4.3a), and

$$\begin{aligned}\Delta S &= \sigma_R\left[\sin\left(90^\circ - \psi\right) + \cos\left(90^\circ - \psi\right)\tan\phi\right] \\ &= \sigma_R(\cos\psi + \sin\psi\tan\phi)\end{aligned} \tag{4.17}$$

for fibres oriented initially at some arbitrary angle i with the horizontal (Fig. 4.3b), with

$$\psi = \tan^{-1}\left(\frac{z}{x + x'}\right) = \tan^{-1}\left(\frac{1}{k + \cot i}\right) \tag{4.18}$$

where σ_R is the mobilized tensile strength of fibres per unit area of fibre-reinforced granular soil, which mainly comes from the fibres; ϕ is the angle of internal friction of unreinforced soil; θ is the angle of shear distortion; x is the horizontal shear displacement; z is the thickness of shear zone; i is the initial orientation angle of the fibre with respect to shear surface, $x' = z \cot i$; and $k(=x/z)$ is the shear distortion ratio.

The mobilized tensile strength of fibres per unit area of soil (σ_R) can be estimated as

$$\sigma_R = A_r \sigma_f = \left(\frac{A_f}{A}\right) \sigma_f \tag{4.19}$$

where σ_f is the maximum tensile stress developed in the fibre at the shear plane, which depends upon a number of parameters and the test variables. The fibres must be long enough and frictional enough to avoid the pullout; conversely, the confining stress must be high enough so that the pullout forces do not exceed the skin friction (i.e. interface shear forces) along the fibre. It is also necessary to assume some sort of tensile stress distribution along the length of the fibre. Two likely or reasonable possibilities are linear and parabolic distributions, with tensile stress a maximum at the shear plane and decreasing to zero at the fibre ends. The resulting tensile stresses at the shear plane for these two distributions are given by the following expressions (Waldron 1977):

$$\sigma_f = \left(\frac{4E_f \tau_f}{D}\right)^{1/2} [z(\sec\theta - 1)]^{1/2} \tag{4.20}$$

for the linear distribution, and

$$\sigma_f = \left(\frac{8E_f \tau_f}{3D}\right)^{1/2} [z(\sec\theta - 1)]^{1/2} \tag{4.21}$$

for the parabolic distribution, where E_f is the modulus or longitudinal stiffness of the fibre, τ_f is the skin friction stress along the fibre and D is the diameter of fibre.

The GO model has been found to predict correctly the influence of various parameters (fibre area ratio, fibre length, fibre modulus, initial fibre orientation and sand relative density), which govern the shear strength of fibre-reinforced soil as observed in the direct shear tests conducted by Gray and Ohashi (1983). For example, in the direct shear tests, the maximum shear strength increase, both theoretically and experimentally, was observed for the fibres placed at an angle of 60° with the shear plane, that is, in the direction of major principal strain.

Note that a variation of Waldron force-equilibrium model was proposed by Jewell and Worth (1987) by placing the stiff reinforcement symmetrically, that is, extended equally about the central horizontal/shear plane in direct shear tests. The force in the reinforcement acting across the shear plane is resolved into components normal and tangential to the shear plane. These two components of the reinforcement force are considered to improve the shearing strength of the soil by directly reducing the shear force acting on the soil and by increasing the available frictional shearing resistance in the soil. The expression for shear strength increase presented by Jewell and Worth (1987) is similar to those proposed by Waldron (1977) and Gray and Ohashi (1983) (Eqs. (4.16) and (4.17)).

4.3.3 Maher and Gray (MG) Model

Based on the observations made in triaxial compression tests and statistical analysis of strength of randomly distributed fibre-reinforced sand, Maher and Gray (1990) proposed a model for predicting its strength behaviour when subjected to static loads. The model considers the following assumptions:

1. The reinforcing fibres have a constant length and diameter, and they do not offer any resistance to bending.
2. The smaller portion of a fibre length on either side of the failure plane is uniformly distributed between zero and half of the fibre length.
3. The fibres have an equal probability of making all possible angles with any arbitrarily chosen fixed axis, that is, the failure surfaces in triaxial compression tests on granular soil are planer and oriented in the same manner as predicted by the Mohr-Coulomb failure criterion.
4. The fibres in the soil mass and, equivalently, their points of intersection with any failure plane are randomly distributed following a Poisson-type distribution.
5. For the sand-fibre composites, the principal stress envelope, that is, the plot of major principal stress at failure (σ_{1f}) versus the minor principal stress (confining stress) (σ_3), is either curved linear or bilinear, with the transition or the break occurring at a threshold confining stress, referred to as the *critical confining stress*, $\sigma_{3\mathrm{crit}}$ (Fig. 4.4). For $\sigma_3 < \sigma_{3\mathrm{crit}}$, the fibres slip during deformation, and for $\sigma_3 > \sigma_{3\mathrm{crit}}$, they stretch or yield. Assuming a Mohr-Coulomb failure criterion, the reinforced soil envelope is parallel to that of unreinforced soil above the critical confining stress.

The increase in shear strength from the fibre reinforcement can be estimated from the following expressions:

$$\Delta S = N_\mathrm{s}\left(\frac{\pi D^2}{4}\right)(2a_\mathrm{r}\sigma_{3\mathrm{av}}\tan\delta)(\sin\theta + \cos\theta\tan\phi)\eta \tag{4.22}$$

for $0 < \sigma_{3\mathrm{av}} < \sigma_{3\mathrm{crit}}$, and

$$\Delta S = N_\mathrm{s}\left(\frac{\pi D^2}{4}\right)(2a_\mathrm{r}\sigma_{3\mathrm{crit}}\tan\delta)(\sin\theta + \cos\theta\tan\phi)\eta \tag{4.23}$$

for $\sigma_{3\mathrm{av}} > \sigma_{3\mathrm{crit}}$, where $\sigma_{3\mathrm{av}}$ is average confining stress in the triaxial chamber, N_s is the average number of fibres intersecting a unit area of the shear plane, a_r is the aspect ratio of fibres, δ is the fibre skin friction angle and η is an empirical

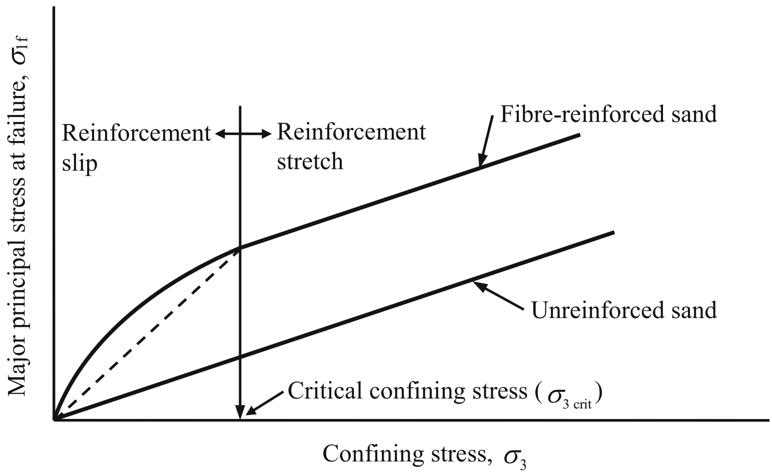


Fig. 4.4 Effect of the fibre inclusion in sand on its principal stress envelope obtained from the triaxial compression tests (Adapted from Maher and Gray 1990; Shukla and Sivakugan 2010)

coefficient depending on the soil characteristics (median/average particle size D_{50}, particle sphericity S_s and coefficient of uniformity C_U) and the fibre parameters (aspect ratio and skin friction). The value of σ_{3crit} can be determined empirically from the experimental measurements, thus depending on the soil characteristics and the fibre properties.

Note that in Fig. 4.4, for $\sigma_3 < \sigma_{3crit}$, the stress envelope for fibre-reinforced sand is either linear or nonlinear depending on the types of sand and reinforcement.

The MG model has predicted the increase in the strength of the fibre-reinforced soil reasonably well when compared to the experimental values. However, the width of shear zone z, which significantly affects the increase in strength (Shewbridge and Sitar 1989, 1990), has not been determined for the reinforced soil, as it is also not determined in the Waldron and GO models. Also, the average expected orientation of fibres is statistically predicted to be perpendicular to the plane of shear failure. It is difficult to determine experimentally the exact orientation of fibres. Note that the model proposed by Jewell and Worth (1987) for stiff reinforcements also does not pay an attention to the formation of shear zone, which significantly affects the increase in shear strength of reinforced soil.

4.3.4 Ranjan, Vasan and Charan (RVC) Model

Ranjan et al. (1996) proposed a model based on the statistical/regression analysis of the data obtained from more than 500 triaxial compression tests on the randomly distributed discrete fibre-reinforced soil. This model quantifies the effect of fibre properties, soil characteristics and confining stress on the shear strength of the reinforced soil. The mathematical expression of the model is as follows:

$$\sigma_{1f} = f(p_f, a_r, f^*, f, \sigma_3) \tag{4.24}$$

where σ_{1f} is the major principal stress at failure of fibre-reinforced soil (i.e. shear strength of fibre-reinforced soil), p_f is the fibre content, a_r is the fibre aspect ratio, f^* is the surface friction coefficient, f is the coefficient of friction and σ_3 is the confining stress. The expressions for f and f^* are given below:

$$f = \frac{c}{\sigma} + \tan\phi \tag{4.25}$$

and

$$f^* = \frac{{c_a}^a}{\sigma} + \tan\phi_i \tag{4.26}$$

where σ is the total normal confining stress applied on the shear/failure plane; c and ϕ are shear strength parameters, namely, the cohesion intercept and angle of shearing resistance, respectively, of the unreinforced soil; c_a is the adhesion intercept; and ϕ_i is the angle of skin friction as determined from fibre pullout tests. A value of 100 kPa for σ may be adopted for calculating f and f^* using Eqs. (4.25) and (4.26), respectively. The regression analysis has considered the following assumptions:

1. The response parameter σ_{1f} is a random quantity with normal distribution law, which is followed by most distributions in nature.
2. The variance of σ_{1f} does not depend on its absolute value. That is, the variances are homogeneous.
3. The values of independent factors (i.e. $p_f, a_r, f^*, f, \sigma_3$, etc.) are not random quantities.

The failure envelope of fibre-reinforced soil is curvilinear with a transition at certain confining stress, called the critical confining stress σ_{3crit} (see Fig. 4.4).

For $\sigma_3 \le \sigma_{3crit}$,

$$\sigma_{1f} = 12.3(p_f)^{0.4}(a_r)^{0.28}(f^*)^{0.27}(f)^{1.1}(\sigma_3)^{0.68} \tag{4.27}$$

For $\sigma_3 \ge \sigma_{3crit}$,

$$\sigma_{1f} = 8.78(p_f)^{0.35}(a_r)^{0.26}(f^*)^{0.06}(f)^{0.84}(\sigma_3)^{0.73} \tag{4.28}$$

Note that the values of coefficient of determination R^2 in Eqs. (4.27) and (4.28) are found to be greater than 0.90, which indicates a good fit of the data used. The value of σ_{3crit} is observed to decrease with an increase in aspect ratio a_r of fibres, as indicated in Fig. 4.5; however, it is relatively unaffected by the amount of fibre

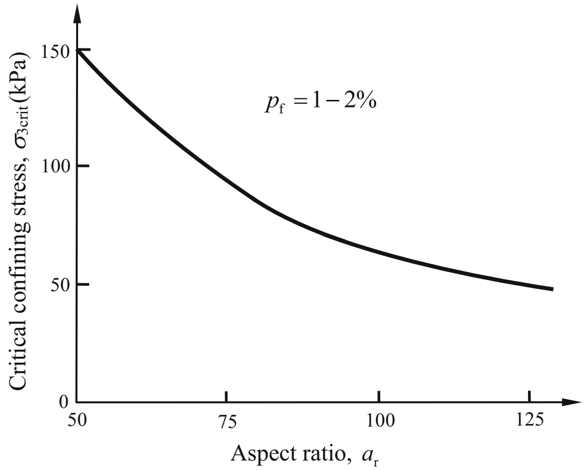


Fig. 4.5 Effect of aspect ratio a_r on critical confining stress σ_{3crit} for plastic fibre-reinforced soil (Adapted from Ranjan et al. 1996)

content p_f as indicated in Fig. 4.5, where the data points of both $p_f = 1\%$ and $p_f = 2\%$ have been represented by the same curve.

4.3.5 Zornberg Model

Zornberg (2002) presented a discrete model for predicting the shear strength of fibre-reinforced soil based on the independent properties of fibres and soil (e.g. fibre content, fibre aspect ratio and shear strength of unreinforced soil). The fibres are assumed to contribute to the shear strength increase by mobilizing tensile stress along the plane of shear (Fig. 4.6). Therefore, the shear strength S_R of fibre-reinforced soil has the following two components: the shear strength S of soil matrix and the fibre-induced distributed tensile force per unit area σ_R. Thus

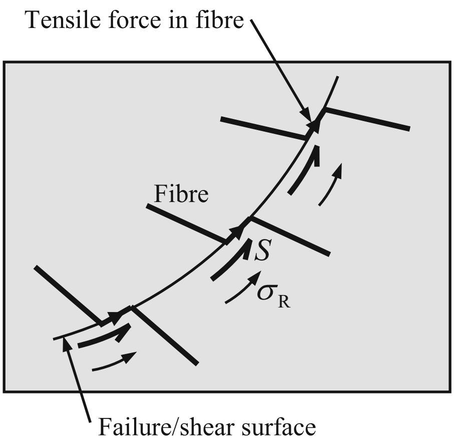
$$S_R = S + \alpha\sigma_R = c + \sigma \tan\phi + \alpha\sigma_R \tag{4.29}$$

where σ is the total normal confining stress applied on the shear/failure plane, c and ϕ are the cohesion intercept and angle of shearing of the unreinforced soil and α is an empirical coefficient that accounts for the orientation of fibres. For the case of randomly oriented fibres, α equals 1. For any preferred orientation, α should be selected suitably between 0 and 1.

Under low confining stresses, when the failure is governed by pullout of the fibres, the fibre-induced distributed tensile force can be estimated as

$$\sigma_{RP} = a_r p_{vf}\left(c_{i,c}c + c_{i,\phi}\sigma \tan\phi\right) \tag{4.30}$$

Fig. 4.6 A fibre-reinforced soil mass with fibres intersecting the failure/shear surface



where a_r is the fibre aspect ratio, p_{vf} is the volumetric fibre content and σ is the average normal stress acting on the fibres. The parameters $c_{i,c}$ and $c_{i,\phi}$ are interaction coefficients defined as

$$c_{i,c} = \frac{a}{c} \tag{4.31}$$

$$c_{i,\phi} = \frac{\tan\delta}{\tan\phi} \tag{4.32}$$

where a is the adhesion between the soil and the fibre and δ is the interface friction angle.

Note that the fibre-induced distributed tension is a function of fibre content p_{vf}, fibre aspect ratio a_r and interaction coefficients $c_{i,c}$ and $c_{i,\phi}$ if the failure is governed by the fibre pullout.

Using Eq. (4.30) with $\sigma_R = \sigma_{RP}$, Eq. (4.29) is expressed to obtain the shear strength of fibre-reinforced soil when the failure is governed by fibre pullout as

$$S_{RP} = c + \sigma\tan\phi + \alpha\sigma_R = c + \sigma\tan\phi + \alpha\left[a_r p_{vf}\left(c_{i,c}c + c_{i,\phi}\sigma\tan\phi\right)\right]$$

or

$$S_{RP} = c_{RP} + (\tan\phi)_{RP}\sigma \tag{4.33}$$

where

$$c_{RP} = (1 + \alpha a_r p_{vf} c_{i,c})c \tag{4.34}$$

and

$$(\tan\phi)_{\mathrm{RP}} = \left(1 + \alpha a_{\mathrm{r}} p_{\mathrm{vf}} c_{\mathrm{i},\phi}\right)\tan\phi \tag{4.35}$$

When failure is governed by the yielding of the fibres, that is, the fibre tensile breakage, the fibre-induced distributed tensile force is a function of fibre content p_{vf} and tensile strength $\sigma_{\mathrm{f,ult}}$ of individual fibres and can be estimated as

$$\sigma_{\mathrm{Rt}} = p_{\mathrm{vf}}\sigma_{\mathrm{f,ult}} \tag{4.36}$$

Using Eq. (4.36) with $\sigma_{\mathrm{R}} = \sigma_{\mathrm{Rt}}$, Eq. (4.29) is expressed to obtain the shear strength of fibre-reinforced soil when the failure is governed by the tensile breakage of the fibres as

$$S_{\mathrm{Rt}} = c_{\mathrm{Rt}} + (\tan\phi)_{\mathrm{Rt}}\sigma \tag{4.37}$$

where

$$c_{\mathrm{Rt}} = c + \alpha p_{\mathrm{vf}}\sigma_{\mathrm{f,ult}} \tag{4.38}$$

and

$$(\tan\phi)_{\mathrm{Rt}} = \tan\phi \tag{4.39}$$

Note that Zornberg (2002) presented the discrete model for the design of fibre-reinforced soil slopes. The fibres as discrete elements are considered to contribute to stability by mobilizing tensile stresses along the shear/failure surface, as shown in Fig. 4.6. At the critical normal stress,

$$\sigma_{\mathrm{RP}} = \sigma_{\mathrm{Rt}} \tag{4.40}$$

Using Eqs. (4.30) and (4.36) with $\sigma = \sigma_{\mathrm{crit}}$, the expression for σ_{crit} is obtained as

$$\sigma_{\mathrm{crit}} = \frac{\sigma_{\mathrm{f,ult}} - a_{\mathrm{r}} c_{\mathrm{i,c}}\, c}{a_{\mathrm{r}} c_{\mathrm{i},\phi}\tan\phi} \tag{4.41}$$

Thus, the critical normal stress σ_{crit} at which the governing mode of fibre failure changes from fibre pullout to fibre breakage is a function of tensile strength of fibres, soil shear strength and aspect ratio, but is independent of the fibre content.

Note that the pullout resistance of a fibre of length L, if required to be quantified, should be estimated over the shortest side of the two portions of the fibre intercepted by the failure surface. The length of the shortest portion of the fibre intercepted by the failure surface varies from zero to $L/2$. Statistically, the average embedment length of randomly distributed fibres, $L_{\mathrm{e,av}}$, can be defined analytically as $L_{\mathrm{e,av}} = L/4$ (Zornberg 2002).

4.3.6 Shukla, Sivakugan and Singh (SSS) Model

Shukla et al. (2010) developed a simple analytical model for predicting the shear strength behaviour of fibre-reinforced granular soils under high confining stresses, where it can be assumed that pullout of fibres does not take place. The derivation of the analytical expression is presented below.

The shear strength of the unreinforced dry granular soil is given as (Lambe and Whitman 1979; Shukla 2014)

$$S = \sigma \tan \phi \tag{4.42}$$

where σ is the total normal confining stress applied on the shear plane and ϕ is the angle of shearing resistance (or the angle of internal friction) of the granular soil.

Most of the experimental studies have shown that if fibres are added to the granular soil, there is an increase in shear strength of the soil. Assuming no pullout under high confining stresses, the increase in shear strength ΔS for the systematically distributed/oriented fibre-reinforced granular soil as shown in Fig. 4.3b can be expressed by Eq. (4.17) (Gray and Ohashi 1983).

Addition of Eqs. (4.17) and (4.42) gives the shear strength S_R of the systematically oriented fibre-reinforced granular soil as

$$S_{\mathrm{R}} = S + \Delta S == \sigma_{\mathrm{R}} \cos \psi + (\sigma + \sigma_{\mathrm{R}} \sin \psi) \tan \phi \tag{4.43}$$

Assuming that the unreinforced granular soil does not carry any tensile stress, the mobilized tensile strength per unit area of reinforced granular soil (σ_{R}) can be estimated using Eq. (4.19).

Note that the fibres are assumed long enough and frictional enough to resist pullout; conversely the confining stress must be high enough to ensure that pullout forces do not exceed skin friction (i.e. interface shear forces) along the fibre. It is also necessary to assume some form of tensile stress distribution along the length of the fibre. Assuming a constant shear stress distribution along the length of the fibre, an expression for the tensile stress σ_{f} developed in the fibre can be derived as

$$\sigma_{\mathrm{f}} \times \left(\frac{\pi}{4} D^2\right) = \left[\left(\frac{L}{2} \times \pi D\right)(\sigma_{\mathrm{i}})\right] \tan \phi_{\mathrm{i}} \sin i$$

or

$$\sigma_{\mathrm{f}} = 2\sigma_{\mathrm{i}} \left(\frac{L}{D}\right) \tan \phi_{\mathrm{i}} \sin i = 2\sigma_{\mathrm{i}} a_{\mathrm{r}} \tan \phi_{\mathrm{i}} \sin i \tag{4.44}$$

where σ_{i} is normal stress on the fibre inclined to the horizontal at an angle i; L and D are the length and the diameter of the fibres, respectively; $a_{\mathrm{r}}(=L/D)$ is the aspect ratio of the fibres; and ϕ_{i} is the fibre-soil interface friction angle. The expression for σ_{i} is given as (Jewell and Wroth 1987)

$$\sigma_{\mathrm{i}} = \left[\frac{1 - \sin\phi \sin(\phi - 2i)}{\cos^2\phi}\right]\sigma \tag{4.45}$$

The term $\sin i$ is included in Eq. (4.44) as an empirical scaling constant to account for the fibre orientation. From Eq. (4.44), for $i = 0^\circ$, $\sigma_f = 0$, which indicates that if fibres are not stretched and are oriented parallel to the shear plane, they will have no tension, because shear mobilization does not take place in the absence of anchoring of the ends of fibres. Here $\sigma_i = \sigma$. For $i = 90^\circ$, $\sin i = 1$, and $\sigma_{\mathrm{f}} \neq 0$. In fact, when fibres are normal to the shear plane, one can expect that the maximum shear mobilization takes place due to full anchoring of the ends of the fibres. Substituting Eq. (4.45) into Eq. (4.44) gives

$$\sigma_{\mathrm{f}} = 2\sigma a_{\mathrm{r}} \tan\phi_{\mathrm{i}} \sin i \left[\frac{1 - \sin\phi \sin(\phi - 2i)}{\cos^2\phi}\right] \tag{4.46}$$

Substituting Eqs. (4.19) and (4.46) into Eq. (4.43) yields

$$S_{\mathrm{R}} = c_{\mathrm{R}} + \sigma_{\mathrm{RS}} \tan\phi \tag{4.47}$$

where

$$c_{\mathrm{R}} = \sigma\left[2A_{\mathrm{r}}a_{\mathrm{r}} \tan\phi_{\mathrm{i}} \sin i \cos\psi \left\{\frac{1 - \sin\phi \sin(\phi - 2i)}{\cos^2\phi}\right\}\right] \tag{4.48}$$

and

$$\sigma_{\mathrm{RS}} = \sigma\left[1 + 2A_{\mathrm{r}}a_{\mathrm{r}} \tan\phi_{\mathrm{i}} \sin i \sin\psi \left\{\frac{1 - \sin\phi \sin(\phi - 2i)}{\cos^2\phi}\right\}\right] \tag{4.49}$$

Comparing Eqs. (4.42) and (4.47), it can be seen that the effect of reinforcing with systematically oriented fibres is to introduce an apparent cohesion c_{R} to the granular soil and to increase the normal confining stress on the shear plane from σ to σ_{RS}, thereby increasing the shear strength of granular soil. The parameters c_{R} and σ_{RS} are the apparent cohesion and improved confining normal stress in the fibre-reinforced granular soil. Note that both c_{R} and σ_{RS} are functions of area ratio A_{r}, aspect ratio a_{r}, skin friction δ, normal confining stress σ and distortion angle ψ.

The shear strength improvement of the granular soil due to inclusion of fibres can be described in terms of a dimensionless ratio, called the *shear strength ratio (SSR)*, as defined below:

$$SSR = \frac{S_{\mathrm{R}}}{S} \tag{4.50}$$

Substituting Eqs. (4.42) and (4.47) into Eq.(4.50) gives

$$SSR = 1 + 2A_r a_r \left[\frac{1 - \sin\phi \sin(\phi - 2i)}{\cos^2\phi}\right]\left(\sin\psi + \frac{\cos\psi}{\tan\phi}\right)\tan\phi_i \sin i \quad (4.51)$$

In a very thin element of the randomly distributed fibre-reinforced soil (Fig. 4.7), if A_{f1}, A_{f2} A_{f3},..., A_{fn} are the areas of cross section of the fibres at a shear plane, A_{soil} is the area of the soil section, including voids at the shear plane, and ΔL is the thickness of the element measured perpendicular to the shear plane, the area ratio $A_r(=A_f/A)$ of the randomly distributed fibre-reinforced soil at the shear plane can be expressed as

$$A_r = \frac{A_f}{A} = \frac{(A_{f1} + A_{f2} + A_{f3} + \ldots.. + A_{fn})}{(A_{f1} + A_{f2} + A_{f3} + \ldots.. + A_{fn}) + A_{soil}} \times \frac{\Delta L}{\Delta L} = \frac{V_{sf}}{V_{sf} + V_{soil}} \quad (4.52)$$

where V_{sf} is the volume of fibre solids and V_{soil} is the volume of soil, including voids, in the element of the fibre-reinforced soil.

Eq. (4.52) can be rearranged as

$$\frac{V_{sf}}{V_{soil}} = \frac{\left(\frac{A_f}{A}\right)}{1 - \left(\frac{A_f}{A}\right)} = \frac{A_r}{1 - A_r} \quad (4.53)$$

The ratio of weight of the fibre solids W_{sf} to the weight of the soil solids W_{ss}, called the fibre content p_f, in an element of reinforced soil (Fig. 4.7), can be expressed as

$$p_f = \frac{W_{sf}}{W_{ss}} = \frac{V_{sf} G_f \gamma_w}{V_{ss} G \gamma_w} = \frac{V_{sf} G_f (1 + e_s)}{V_{soil} G} = \frac{G_f (1 + e_s)}{G_m}\left(\frac{A_r}{1 - A_r}\right) \quad (4.54)$$

or

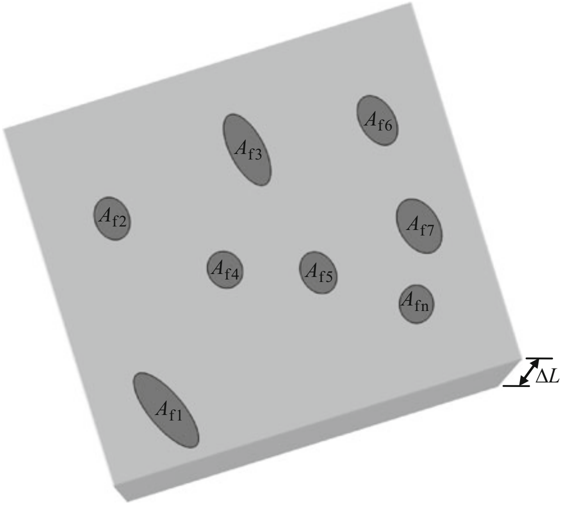
$$A_r = \frac{p_f \left[\frac{G}{G_f(1+e_s)}\right]}{1 + p_f \left[\frac{G}{G_f(1+e_s)}\right]} \quad (4.55)$$

where G_f is the specific gravity of fibre solids, G is specific gravity of soil solids and e_s is the void ratio of soil.

Substituting Eq. (4.55) into Eqs. (4.48), (4.49) and (4.51) provides

$$\frac{c_R}{\sigma} = 2a_r \tan\phi_i \sin i \cos\psi \left[\frac{p_f \left\{\frac{G}{G_f(1+e_s)}\right\}}{1 + p_f \left\{\frac{G}{G_f(1+e_s)}\right\}}\right]\left[\frac{1 - \sin\phi \sin(\phi - 2i)}{\cos^2\phi}\right] \quad (4.56)$$

Fig. 4.7 An element of fibre-reinforced granular soil of thicknessΔL



$$\frac{\sigma_{RS}-\sigma}{\sigma} = 2a_r \tan\phi_i \sin i \sin\psi \left[\frac{p_f\left\{\frac{G}{G_f(1+e_s)}\right\}}{1+p_f\left\{\frac{G}{G_f(1+e_s)}\right\}}\right]\left[\frac{1-\ \sin\phi\sin(\phi-2i)}{\cos^2\phi}\right] \tag{4.57}$$

and

$$\begin{aligned} SSR = &\quad 1+2a_r\tan\phi_i\sin i\left(\sin\psi+\frac{\cos\psi}{\tan\phi}\right)\left[\frac{1-\ \sin\phi\sin(\phi-2i)}{\cos^2\phi}\right] \\ &\times\left[\frac{p_f\left\{\dfrac{G}{G_f(1+e_s)}\right\}}{1+p_f\left\{\dfrac{G}{G_f(1+e_s)}\right\}}\right] \end{aligned} \tag{4.58}$$

Considering a uniform strain distribution, the stress-strain relationship for the fibre can be expressed as

$$\begin{aligned} \sigma_f &= E_f\left[\frac{\sqrt{(x+x')^2+z^2}-\sqrt{x'^2+z^2}}{\sqrt{x'^2+z^2}}\right] = E_f\left(\frac{\sqrt{z^2+z^2\cot^2\psi}}{\sqrt{z^2+z^2\cot^2 i}}-1\right) = \\ &= E_f\left(\frac{\sin i}{\sin\psi}-1\right) \end{aligned} \tag{4.59}$$

where E_f is the Young's modulus of elasticity of fibres in extension. Comparing Eq. (4.46) with Eq. (4.59) yields

$$\sin\psi = \frac{\sin i}{1 + 2a_\mathrm{r}\tan\phi_\mathrm{i}\sin i\left(\frac{\sigma}{E_\mathrm{f}}\right)\left[\frac{1-\sin\phi\sin(\phi-2i)}{\cos^2\phi}\right]} \tag{4.60}$$

and

$$\cos\psi = \frac{\sqrt{\cos^2 i + 4a_\mathrm{r}\tan\phi_\mathrm{i}\sin i\left(\frac{\sigma}{E_\mathrm{f}}\right)\left[\frac{1-\sin\phi\sin(\phi-2i)}{\cos^2\phi}\right]\left[1 + a_\mathrm{r}\tan\phi_\mathrm{i}\sin i\left(\frac{\sigma}{E_\mathrm{f}}\right)\left\{\frac{1-\sin\phi\sin(\phi-2i)}{\cos^2\phi}\right\}\right]}}{1 + 2a_\mathrm{r}\tan\phi_\mathrm{i}\sin i\left(\frac{\sigma}{E_\mathrm{f}}\right)\left[\frac{1-\sin\phi\sin(\phi-2i)}{\cos^2\phi}\right]} \tag{4.61}$$

Substituting values of $\sin\psi$ and $\cos\psi$ from Eqs. (4.60) and (4.61), respectively, into Eq. (4.56), (4.57) and (4.58) gives the following:

$$\frac{c_\mathrm{R}}{\sigma} = 2\beta_1\beta_2\left[\frac{p_\mathrm{f}\left\{\frac{G}{G_\mathrm{f}(1+e_\mathrm{s})}\right\}}{1 + p_\mathrm{f}\left\{\frac{G}{G_\mathrm{f}(1+e_\mathrm{s})}\right\}}\right]\left[\frac{\sqrt{\cos^2 i + 4\beta_1\beta_2\left(\frac{\sigma}{E_\mathrm{f}}\right)\left[1 + \beta_1\beta_2\left(\frac{\sigma}{E_\mathrm{f}}\right)\right]}}{1 + 2\beta_1\beta_2\left(\frac{\sigma}{E_\mathrm{f}}\right)}\right] \tag{4.62}$$

$$\frac{\sigma_{RS} - \sigma}{\sigma} = 2\beta_1\beta_2\left[\frac{p_\mathrm{f}\left\{\frac{G}{G_\mathrm{f}(1+e_\mathrm{s})}\right\}}{1 + p_\mathrm{f}\left\{\frac{G}{G_\mathrm{f}(1+e_\mathrm{s})}\right\}}\right]\left[\frac{\sin i}{1 + 2\beta_1\beta_2\left(\frac{\sigma}{E_\mathrm{f}}\right)}\right] \tag{4.63}$$

and

$$SSR = \frac{S_\mathrm{R}}{S} = 1 + 2\beta_1\beta_2\left[\frac{p_\mathrm{f}\left\{\frac{G}{G_\mathrm{f}(1+e_\mathrm{s})}\right\}}{1 + p_\mathrm{f}\left\{\frac{G}{G_\mathrm{f}(1+e_\mathrm{s})}\right\}}\right] \times \left[\frac{\sin i}{1 + 2\beta_1\beta_2\left(\frac{\sigma}{E_\mathrm{f}}\right)} + \frac{\sqrt{\cos^2 i + 4\beta_1\beta_2\left(\frac{\sigma}{E_\mathrm{f}}\right)\left[1 + \beta_1\beta_2\left(\frac{\sigma}{E_\mathrm{f}}\right)\right]}}{\left[1 + 2\beta_1\beta_2\left(\frac{\sigma}{E_\mathrm{f}}\right)\right]\tan\phi}\right] \tag{4.64}$$

with

$$\beta_1 = a_\mathrm{r}\tan\phi_\mathrm{i}\sin i \tag{4.65a}$$

and

$$\beta_2 = \frac{1 - \sin\phi \sin(\phi - 2i)}{\cos^2\phi} \tag{4.65b}$$

Eq. (4.62) provides an expression for ratio of apparent cohesion to normal confining stress on the shear plane; Eq. (4.63) gives an expression for the ratio of increase in normal confining stress to the normal confining stress; and Eq. (4.64) defines an expression for *SSR*, which is the ratio of shear strength of reinforced soil to that of unreinforced soil.

Note that Eq. (4.64) is quite useful in predicting the variation of *SSR* with fibre content p_f, aspect ratio L/D, ratio of confining stress to modulus of fibres σ/E_f, specific gravity of fibre solids G_f and initial orientation of fibres with respect to shear plane i for any specific sets of parameters in their practical ranges. The effects of p_f, a_r and i on *SSR* for specific sets of parameters are shown in Figs. 4.8, 4.9, and 4.10, respectively.

In Fig. 4.10, you may note that as i increases, *SSR* also increases with its maximum value for $i = 74^\circ$ for the set of parameters taken. A similar trend has also been reported by Gray and Ohashi (1983) in their experimental study. They reported that an initial orientation of 60° with the shear plane is the optimum orientation for maximum increase in shear strength. This direction is approximately the principal tensile strain direction in a dense sand as reported by Jewell (1980).

For the randomly distributed fibre-reinforced soil, i can vary from 0° to 180°. For the purpose of calculating the increase in shear stress caused by inclusion of randomly distributed fibres in a granular soil, $i = 90^\circ$ can be taken for the orientation of fibres in Eq. (4.64) for simplicity in analysis. Though the observations from the analytical model matches with the experimental observations in trends of variation of parameters, it is felt that large-scale tests will be suitable for comparing the observations with the results from this model.

It is worth noting the following:

1. The inclusion of fibres in the granular soil induces cohesion, may be called the apparent cohesion, as well as an increase in the normal stress on the shear failure plane, which are proportional to the fibre content and the aspect ratio, implying that the increase in shear strength is also proportional to the fibre content and the aspect ratio.
2. The increase in shear strength of the granular soil due to presence of fibres is significantly contributed by the apparent cohesion, and the contribution to the shear strength from the increase in normal confining stress is limited.
3. As the initial orientation of fibres with respect to shear plane (i) increases, the *SSR* also increases to a maximum value for a specific value of i depending on the values of other governing parameters.

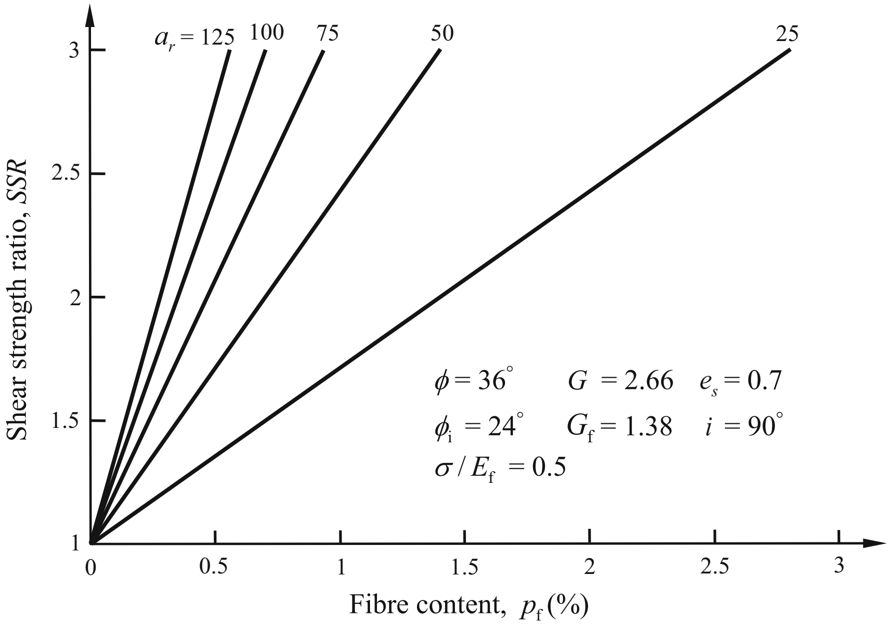


Fig. 4.8 Effect of fibre content p_{f} on the shear strength ratio *SSR* of fibre-reinforced soil (Adapted from Shukla et al. 2010)

Example 4.2
Determine the shear strength ratio (*SSR*) for the following three cases:

(a) Fibre content, $p_{\mathrm{f}} = 0\%$
(b) Soil-fibre interface friction angle, $\delta = 0^{\circ}$
(c) All fibres are oriented parallel to the shear plane, $i = 0^{\circ}$

Explain the physical significance of *SSR* value obtained for each case.

Solution
(a) For $p_{\mathrm{f}} = 0\%$, from Eq. (4.64),

$$SSR = \frac{S_{\mathrm{R}}}{S} = \mathbf{1}$$

This result is well expected when there are no fibres present in the soil.
(b) For $\delta = 0^{\circ}$, from Eq. (2.65a), $\beta_1 = 0$, and hence from Eq. (4.64),

$$SSR = \frac{S_{\mathrm{R}}}{S} = \mathbf{1}$$

This suggests that if fibres are not having frictional resistance in contact with soil particles, their inclusion in soil will not be useful. In this situation, there is no

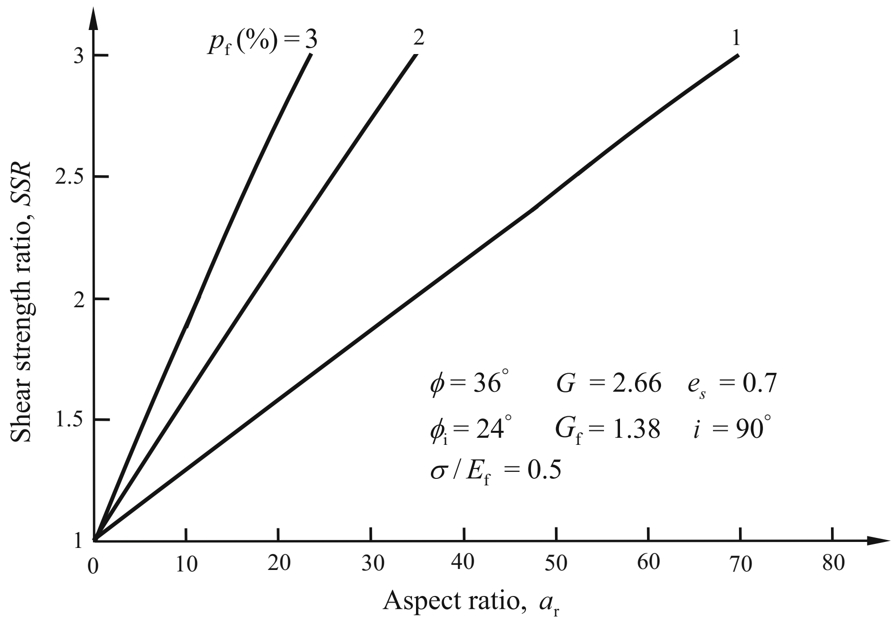


Fig. 4.9 Effect of aspect ratio a_r on the shear strength ratio *SSR* of fibre-reinforced soil (Adapted from Shukla et al. 2010)

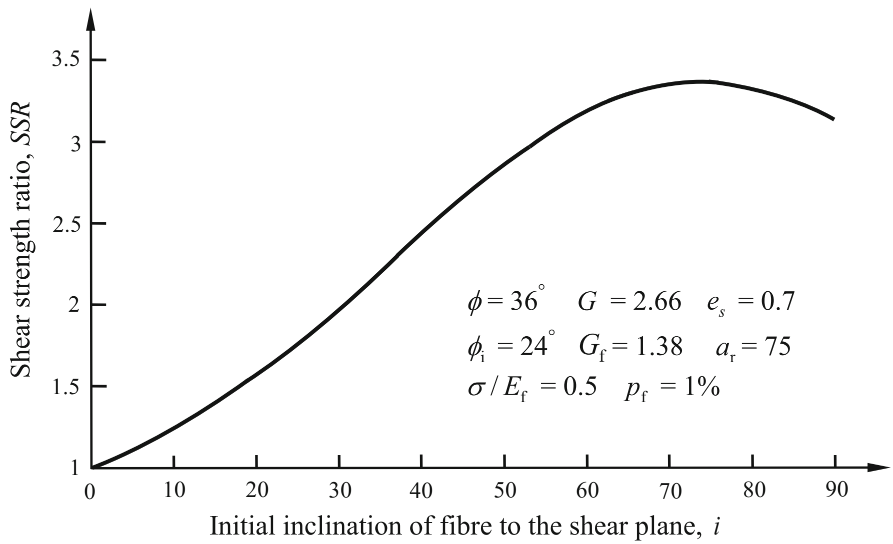


Fig. 4.10 Effect of initial inclination of fibre to the shear plane *i* on the shear strength ratio *SSR* of fibre-reinforced soil (Adapted from Shukla et al. 2010)

shear stress acting along the surface of the fibres, and therefore the fibres remain unstretched.

(c) For $i = 0^\circ$, from Eq. (2.65a), $\beta_1 = 0$, and hence from Eq. (4.64),

$$SSR = \frac{S_{\mathrm{R}}}{S} = \mathbf{1}$$

This suggests that if all the fibres are oriented parallel to the shear plane, there will be no increase in shear strength.

4.4 Other Models and Numerical Studies

The behaviour of fibre-reinforced soils has been investigated numerically by several researchers by developing their own analysis and numerical programmes/codes or by using the commercial finite element/difference software. Some researchers have also attempted to present constitutive models based on certain assumptions for solving boundary-value problems with fibre-reinforced soils as a numerical analysis. The non-uniformity of fibre orientations and anisotropy have also been described in terms of a specific fibre orientation distribution function, resulting in an anisotropic constitutive model (Diambra et al. 2007).

Michalowski and Zhao (1996) proposed a model using the energy-based homogenization (averaging) technique to quantify the effect of fibre inclusion on the shear strength behaviour of granular soil. The model considers that the fibres contribute to the strength of the fibre-reinforced soil only if they are subjected to tension, whereas their influence in the compressive regime is neglected due to possible buckling and kinking. This model is basically a mathematical description of a failure criterion for fibre-reinforced granular soil in terms of in-plane variants q (radius of the Mohr circle representing the state of stress, that is, shear stress, $\left(\sigma_1' - \sigma_3'\right)/2$) and p (mean of the maximum and minimum principal stresses, that is, mean principal stress, $\left(\sigma_1' + \sigma_3'\right)/2$) in a macroscopic/global stress space as stated below:

$$\frac{q}{p_{\mathrm{vf}}\sigma_0} = \frac{p}{p_{\mathrm{vf}}\sigma_0}\sin\phi + \frac{1}{3}N\left[1 - \frac{1}{4a_r p_{\mathrm{vf}}}\frac{\cot\phi_i}{\left(\frac{p}{p_{\mathrm{vf}}\sigma_0}\right)}\right] \tag{4.66}$$

with

$$N = \frac{1}{\pi}\cos\phi + \left(\frac{1}{2} + \frac{\phi}{\pi}\right)\sin\phi \tag{4.67}$$

where p_{vf} is the volumetric fibre content, σ_0 is the yield stress of the fibre material, a_r is the aspect ratio of fibre, ϕ is the angle of internal friction of the granular matrix and ϕ_i is the soil-fibre interface friction angle.

Failure of a single fibre in a deforming fibre-reinforced soil can occur due to fibre slip or tensile rupture. When a fibre fails in the tensile rupture mode, the slip also occurs at both fibre ends up to the distance s, as given below:

$$s = \frac{D}{4}\frac{\sigma_0}{\sigma_\mathrm{n} \tan\phi_i} \tag{4.68}$$

where σ_n is the stress normal to the fibre surface and D is the diameter of the fibre. Note that the slip also occurs at both fibre ends up to the distance s because the tensile strength of the fibre material cannot be mobilized throughout the entire fibre length. A pure slip failure mode may occur if the length of fibres L becomes less than $2s$, or when the aspect ratio

$$a_r < \frac{1}{2}\frac{\sigma_0}{\sigma_\mathrm{n} \tan\phi_i} \tag{4.69}$$

A comparison of the Michalowski and Zhao's model with the experimental results demonstrates the adequacy of the model. When the pure slip occurs, the failure criterion takes the following form:

$$\frac{q}{p_\mathrm{vf}\sigma_0} = \frac{p}{p_\mathrm{vf}\sigma_0}\left(\sin\phi + \frac{1}{3}N p_\mathrm{vf} a_r \tan\phi_i\right) \tag{4.70}$$

When fibres are not present, both Eqs. (4.66) and (4.70) reduce to the standard Mohr-Coulomb failure criterion for granular material as

$$q = p \sin\phi \tag{4.71}$$

Figure 4.11 shows the results from Eqs. (4.66) and (4.70) in the $q - p$ plane with different curves presenting the failure criteria for fibre-reinforced soil with fibres of various aspect ratios. As indicated in Eq. (4.70), the slip mode is described by a linear variation for a constant ϕ, whereas in the tensile rupture mode, the shear strength is not proportional to the mean stress p (Eq. (4.66)). The following points regarding Michalowski and Zhao's model are worth mentioning:

1. Five parameters (volumetric fibre concentration p_vf, fibre aspect ratio a_r, fibre yield stress σ_0, internal friction angle of soil ϕ and soil-fibre interface friction angle ϕ_i) are needed to theoretically predict the failure stress. For a pure fibre slip failure mode, the failure criterion is independent of the fibre yield stress σ_0.
2. The transition from one failure mode to another is continuous and smooth (continuous derivative).
3. The model is applicable for low value of p_vf, say less than 10%, so that interaction between fibres may be neglected. The length L of fibres needs to be at least one order of magnitude larger than the granular soil particle diameter, and the fibre diameter D needs to be at least of the same order as the size of granular soil particle.

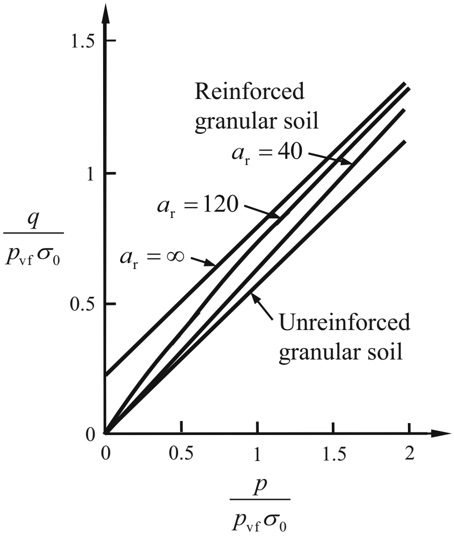


Fig. 4.11 Theoretical failure criterion for fibre-reinforced granular soil (Note: $p_{vf} = 0.02$, $\phi = 35°$ and $\phi_i = 20°$) (Adapted from Michalowski and Zhao 1996)

4. The model is limited to isotropic mixtures, that is, the fibre-reinforced soil with isotropic distribution of fibre orientations.
5. The model indicates that the fibres may be an effective way for soil reinforcement when mixtures with a larger fibre content or larger aspect ratios are used. The stiffness of the soil prior to failure is affected by the addition of fibres, and, for flexible fibres, it drops down with respect to the stiffness of the granular soil.

The contribution of fibres to the strength of fibre-reinforced soils is very much dependent on the distribution and orientation of the fibres within the soil mass. The fibres in the direction of largest extension contribute most to the strength of the reinforced soil, whereas the fibres under compression have an adverse effect on the stiffness, and they do not produce an increase in the strength. Michalowski and Cermak (2002) presented a failure criterion for fibre-reinforced sand with an anisotropic distribution of fibre orientation, considering contribution of a single fibre to the work dissipation during failure of the reinforced sand and integrating this dissipation over all fibres in a reinforced sand element. This failure criterion can be applied directly in methods for solving stability problems for fibre-reinforced sand.

Ding and Hargrove (2006) established a nonlinear stress-strain relationship of the equivalent homogeneous and isotropic material for flexible fibre-reinforced soil, assuming there is no slip at soil-fibre interface and breakage of fibres. The development of the stress-strain relation is based on the consideration of energy in soil and energy in fibres. This constitutive model relates the shear modulus of the fibre-reinforced soil with the fibre content, distribution, geometrical features and soil-fibre interaction.

Babu et al. (2008a) presented the results of a numerical analysis of a cylindrical sand specimen (diameter = 38 mm, height = 76 mm) reinforced with the coir fibres.

They used the finite difference code, Fast Lagrangian Analysis of Continua ($FLAC^{3D}$), for the investigation. The results show that the presence of random fibre reinforcement in sand makes the stress concentration more diffused and restricts the shear band formation. The stress-strain response of the reinforced sand is governed by the pullout resistance of the fibres as the maximum mobilized tensile force (2.1 to 3.6 N) under different confining pressures is generally less than the tensile strength of the fibres (5 N). The mobilized tensile force in a fibre within sand increases with increasing confining pressure. No definite peak in the stress-strain diagram is observed in numerical analysis as the peaks are noticed in experimental stress-strain diagrams.

Based on the standard triaxial compression test data, Babu and Vasudevan (2008a) presented statistical models using regression for predicting the major principal stress at failure σ_{1fR}, cohesion intercept c_R, angle of internal friction ϕ_R and initial stiffness E_{iR} of soil reinforced with coir fibres, as given below:

$$\begin{aligned}\sigma_{1fR}(\text{kPa}) = \; & 159.1 + 3.96\sigma_3 - 0.0083\sigma_3^2 - 2959D \\ & + 4866.5D^2 - 37.01p_f + 17.35p_f^2 + 58.8L - 1.69L^2 \\ & + 2.69\sigma_3 D + 548.61Dp_f + 6.21Lp_f - 0.016L\sigma_3\end{aligned} \tag{4.72}$$

$$c_R(\text{kPa}) = 76.5 + 156.4D - 102.1D^2 + 126.1p_f - 39.3p_f^2 + 20.2Dp_f \tag{4.73}$$

$$\phi_R(\text{degrees}) = 23.1 - 78.55D + 191.1D^2 + 7.03p_f + 2.38p_f^2 - 15.02Dp_f \tag{4.74}$$

$$\begin{aligned}E_{iR}(\text{kPa}) = \; & 8992.2 + 64.94\sigma_3 - 0.14\sigma_3^2 - 94612D \\ & + 186594D^2 - 1744.9p_f + 1167.8p_f^2 + 1765.1L \\ & - 52.1L^2 + 33.3\sigma_3 D + 11707.7Dp_f + 129.77Lp_f \\ & - 0.47L\sigma_3\end{aligned} \tag{4.75}$$

where σ_3 is the confining pressure (kPa), D is the diameter of fibres (mm), L is the length of fibres (mm) and p_f fibre content (%). The value of R^2 for the above-presented four equations are 0.95, 0.98, 0.99 and 0.89, respectively, which are close to unity, and hence these equations can be considered satisfactory for the coir fibre-reinforced soils.

Babu and Vasudevan (2008b) have also presented regression equations for quantifying the seepage velocity and the piping resistance of coir fibre-reinforced soil considering hydraulic gradient, fibre contents and fibre lengths. In another study, Babu et al. (2008b) developed equations similar to Eqs. (4.72), (4.73) and (4.74) for coir fibre-reinforced black cotton soil and also the following expression for its compression index C_{cR} with R^2 of 0.98:

$$C_{cR} = 0.506 - 0.129p_f + 0.01p_f^2 \tag{4.76}$$

Example 4.3
A black cotton soil is reinforced with coir fibres. The fibre content is 0.2%. What is the effect of fibre inclusion on the compression index of soil?

Solution
From Eq. (4.76), the compression index of unreinforced black cotton soil ($p_\mathrm{f} = 0\%$) is

$$C_{\mathrm{cU}} = 0.506 - 0.129 p_\mathrm{f} + 0.01 p_\mathrm{f}^2 = 0.506 - (0.129)(0) + (0.01)(0)^2 = 0.506$$

From Eq. (4.76), the compression index of coir fibre-reinforced black cotton soil ($p_\mathrm{f} = 0.2\%$) is

$$\begin{aligned} C_{\mathrm{cR}} &= 0.506 - 0.129 p_\mathrm{f} + 0.01 p_\mathrm{f}^2 = 0.506 - (0.129)(0.2) + (0.01)(0.2)^2 \\ &= 0.4806 \end{aligned}$$

Thus, the compression index of black cotton soil decreases from 0.506 to 0.4806 due to inclusion of fibres. The percentage decrease is

$$\frac{\Delta C_\mathrm{c}}{C_{\mathrm{cU}}} \times 100 = \frac{C_{\mathrm{cU}} - C_{\mathrm{cR}}}{C_{\mathrm{cU}}} \times 100 = \frac{0.506 - 0.4806}{0.506} \times 100 = \mathbf{5.02\%}$$

Diambra et al. (2010) proposed a modelling approach for coupling the effects of fibres with the stress-strain behaviour of unreinforced soil, based on the basic rule of mixtures, and presented a constitutive model for the fibre-reinforced sand mass, as stated below:

$$\Delta\sigma = \Delta\sigma' + p_{\mathrm{vf}} \Delta\sigma_\mathrm{f} = [M_\mathrm{m}]\Delta\varepsilon + p_{\mathrm{vf}}[M_\mathrm{f}]\Delta\varepsilon \tag{4.77}$$

where $\Delta\sigma$ is the total stress increase, $\Delta\sigma'$ is the stress increase in sand particles, p_{vf} is the volumetric fibre content, $\Delta\sigma_\mathrm{f}$ is the stress increase in fibres, M_m is the stiffness matrix for the sand, M_f is the stiffness matrix for the fibres and $\Delta\varepsilon$ is the strain increase in sand particles and fibres, caused by stress increase $\Delta\sigma$. Equation (4.77) considers that no sliding occurs between sand particles and fibres, and the fibres only act elastically in tension; thus the strains are equal in sand particles and fibres. This assumption is used in developing the stiffness matrix for the fibres. The sand stiffness matrix can be developed using the Mohr-Coulomb model or any other suitable model. In this constitutive model, any distribution of fibre orientations can be accounted for, and importance of considering the fibre orientation relative to the strain conditions can be explained. The model has been calibrated against the results of drained triaxial compression and extension tests, and a good agreement has been observed.

Note that the test results obtained from the direct shear tests conducted by Shewbridge and Sitar (1989) using a large direct shear device show that the shear zones tend to be wider in reinforced soil composites than in soil alone. The width of the shear zones increases with increasing stiffness of the composite due to any combination of increased reinforcement concentration (measured by area ratio), stiffness and reinforcement-soil bond strength. Volume changes associated with the development of the shear zones are not spatially homogeneous. A linear relationship between reinforcement concentration and increased strength is not observed, and thus this observation contradicts the earlier observations. The actual deformation pattern of the reinforcement-soil composite can be described by a smooth asymptotic curve in the following form (Shewbridge and Sitar 1989, 1990):

$$Y = \frac{\delta}{2}\left(1 - e^{-k|X|}\right) \tag{4.78}$$

where X and Y are the axes or coordinate directions perpendicular and parallel to the direction of shear, respectively, with the origin at the centre of the shear zone (Fig. 4.12); δ is the externally applied shear displacement; and k is a deformation decay constant, which is basically a curve-fitting parameter describing the shape of the curve. The value of k can be determined from the maps of deformed soil specimens subjected to a known total shear displacement δ. The constant k provides a measure of the thickness of the shear zone and of the distribution and magnitude of shear strains within it. For the specimens with relatively thin shear zones, which contain large shear strains, the value of k is large. For the specimens with large shear zones containing small shear strains, the value of k is small. The value of k decreases as the fibre area ratio increases, that is, the shear zone width increases with an increase in reinforcement concentration. The value of k also decreases with an increase in reinforcement stiffness and soil-reinforcement interface bond strength. Thus, the shear zone width, as measured by the deformation decay constant k, increases with increasing reinforcement concentration, reinforcement stiffness and soil-reinforcement bond strength.

The deformation pattern, defined by Eq. (4.78), differs significantly from the simple shear deformation pattern assumed in the models, described in Sec. 4.3. In fact, there is still a need of further study based on large-scale field tests to investigate more realistic picture of shear zone development and width of shear zone in the fibre-reinforced soil because the shear zone significantly governs the strength and deformation of the reinforced soil.

Chapter Summary

1. If the failure occurs by rupture of the reinforcement, the strength increase can be characterized by a constant cohesion intercept c_R as an apparent cohesion, introduced due to reinforcement. If the failure occurs by a slippage between the reinforcement and the soil, the strength increase can be characterized by an increased friction angle ϕ_R. In general, a fibre-reinforced soil typically shows the bilinear strength behaviour.
2. If the failure is characterized by pullout of individual fibres, the fibre-induced distributed tension increases linearly with fibre content and fibre aspect ratio.
3. Simplified models of fibre-reinforced soil are based on different approaches, such as force-equilibrium/mechanistic approach, energy dissipation approach, statistical approach and the approach of superposition of the effects of soil and fibres.

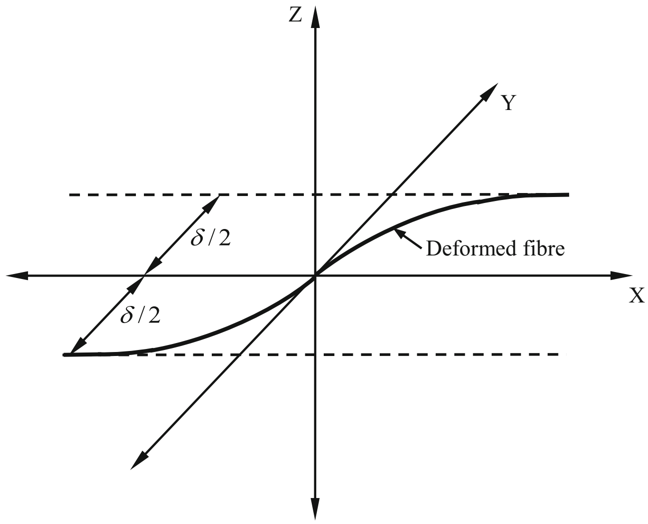


Fig. 4.12 Schematic geometry of deformed reinforcement in the coordinate space (Adapted from Shewbridge and Sitar 1989, 1990)

4. Waldron model and Gray and Ohashi (GO) model have correctly described the characteristics of reinforced soil in direct shear, in which the failure/shear plane is predetermined and the reinforcement is placed at certain angle with the failure plane. Hence these models are applicable only for oriented inclusions.
5. Maher and Gray (MG) model; Shukla, Sivakugan and Singh (SSS) model; and statistical models predict reasonably well the increase in strength of randomly distributed fibre-reinforced soils as these models are based on the triaxial compression tests.
6. The SSS model provides a more generalized analytical expression for estimating the shear strength increase as a result of fibre inclusion within the soil mass.
7. Width of shear zone should be considered for developing the models for more realistic estimation of improvement in the strength of fibre-reinforced soils.
8. Available statistical models provide the empirical expressions based on limited characteristics of soils and fibres, and hence they should not be used as the generalized expressions for estimating the engineering properties of fibre-reinforced soils.
9. Constitutive models can be developed using the basic rule of mixtures as required for any specific application. Equation (4.77) is an example of such a model, which considers that no sliding occurs between sand particles and fibres, and the fibres only act elastically in tension; thus the strains are equal in sand particles and fibres.
10. Behaviour of fibre-reinforced can be investigated numerically considering the appropriate constitutive models. Non-uniformity of fibre orientations and anisotropy can easily be considered in numerical models.

Questions for Practice

(Select the most appropriate answer to the multiple-choice questions from Q 4.1 to Q 4.5.)

4.1. A fibre within a soil mass can apply confining stress by

(a) Skin friction
(b) Adhesion
(c) Both (a) and (b)
(d) Cohesion

4.2. Which of the following conditions may cause the fibres to slip during deformation of the fibre-reinforced soil?

(a) $\sigma_3 < \sigma_{3\text{crit}}$
(b) $\sigma_3 > \sigma_{3\text{crit}}$
(c) $\sigma_3 \geq \sigma_{3\text{crit}}$
(d) None of the above

4.3. The effect of width of shear zone is not considered in

(a) Waldron model
(b) GO model
(c) MG model
(d) all of the above

4.4. Which of the following models is a statistical/regression model?

(a) Waldron model
(b) RVC model
(c) SSS model
(d) None of the above

4.5. Shear strength of the fibre-reinforced soil increases with

(a) Increasing fibre content and decreasing aspect ratio
(b) Decreasing fibre content and increasing aspect ratio
(c) Increasing both fibre content and aspect ratio
(d) Decreasing both fibre content and aspect ratio

4.6. Explain the basic reinforcing mechanism of fibre-reinforced soil.
4.7. What is critical confining stress? Explain its importance. Derive an expression for the critical confining stress.
4.8. Under what conditions, the fibre reinforcement within the soil mass may slip or rupture?
4.9. For a reinforced sand, consider the following:

Angle of shearing resistance of unreinforced sand, $\phi = 30^\circ$
Friction factor, $F = 0.15$
Determine the angle of shearing resistance of the reinforced sand.

4.10. What are the assumptions of Maher and Gray model? Is this model of fibre-reinforced soils based on observations made in triaxial compression tests? Is this model applicable to all types of fibre-reinforced soils? Justify your answer.
4.11. What are the limitations of GO, MO and SSS models?
4.12. Can you use the available statistical models as the generalized models to estimate the engineering properties of soils? Explain briefly.
4.13. List the parameters/factors considered in the SSS model. Derive the analytical expression for the shear strength ratio proposed by the SSS model of the fibre-reinforced soil.
4.14. What is the key advantage of using the Zornberg model?
4.15. Explain the effect of aspect ratio on the critical confining stress.
4.16. Using the SSS model, discuss the effects of the following on the shear strength of fibre-reinforced soil:

(a) Soil-fibre interface friction angle (δ)
(b) Modulus of elasticity of fibres (E_f)
(c) Angle of shearing resistance of soil (ϕ)

4.17. What is physical meaning of having the shear strength ratio (SSR) of unity for a fibre-reinforced soil?
4.18. Derive the following relationship:

$$SSR = 1 + \left(\frac{c_{\mathrm{R}}}{\sigma}\right)\frac{1}{\tan\phi} + \frac{\sigma_{\mathrm{RS}} - \sigma}{\sigma}$$

Show the variation of SSR, c_{R}/σ and $(\sigma_{\mathrm{RS}} - \sigma)/\sigma$ with the fibre content p_{f} for typical values of soil and fibre parameters.

4.19. Explain the model of fibre-reinforced soil developed using the energy-based homogenization technique. What are the key features of this model?
4.20. Explain the constitutive model for the fibre-reinforced soil mass developed using the basic rule of mixtures. Is there any limitation of this model?
4.21. A black cotton soil is reinforced with coir fibres. The fibre content is 0.6%. What is the effect of fibre inclusion on the compression index of soil?
4.22. Shear zones tend to be wider in fibre-reinforced soils than in soil alone. Is it true? Justify your answer.

Answers to Selected Questions

4.1 (c)
4.2 (a)
4.3 (d)
4.4 (b)
4.5 (c)
4.9 43.7°
4.21 14.58% decrease

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Chapter 5
Applications of Fibre-Reinforced Soil

5.1 Introduction

In the previous chapters, you studied about the basic characteristics of soils and fibres, engineering behaviour of fibre-reinforced soils with or without other admixtures/additives (lime, cement, fly ash, etc.) and reinforcing mechanisms and models. In most applications, the discrete fibres are simply added and mixed randomly with soil or other similar materials (coal ashes, mine tailings, etc.), in much the same way as lime, cement or other additives are used to stabilize and improve the soil. However, some difficulties occur at the construction site in getting a uniform mixture of soil and fibres. In the present-day construction practice, the use of fibres is one of the cost-effective and environmentally friendly ground improvement techniques. This chapter describes the possible field applications of fibre-reinforced soils, focusing on presenting specific application/construction guidelines.

5.2 Field Applications

Reinforcing the soil with discrete fibres is a ground improvement technique that has not yet been fully utilized worldwide as this technique can be adopted in the present-day engineering practice. This is probably because of unavailability of standards and codes of practice, especially in developing countries. In comparison with the systematically reinforced soils (i.e. soils with oriented reinforcements such as the geosynthetic-reinforced soils), the randomly distributed/oriented fibre-reinforced soils exhibit some advantages, as listed below:

1. Preparation of randomly distributed fibre-reinforced soils mimics the conventional/traditional soil stabilization techniques, which uses the admixtures, such as lime, cement, fly ash, etc. Hence, the field application or construction

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procedure may be similar to that adopted for conventional soil stabilization techniques.

2. Addition of fibre reinforcement in soil causes a significant improvement in strength and stiffness, although a decrease in stiffness may take place as reported in some studies.
3. Fibre-reinforced soil exhibits greater toughness and ductility and smaller losses of post-peak strength as compared to soil alone.
4. Addition of fibres improves the permeability and compressibility/swelling characteristics of soils.
5. Inclusion of fibres improves the load-carrying capacity of soils.
6. Addition of fibres significantly decreases the liquefaction potential of coarse silts and fine sands.
7. Randomly distributed fibres with even orientations in all directions offer a better isotropy in strength or other properties and limit the potential planes of weakness that can develop parallel to the oriented reinforcement as included in a systematically reinforced soil.
8. Before the failure takes place, because of greater extensibility characteristics of fibre-reinforced soils, one can notice large strains/deformations in the fibre-reinforced soil structures, and hence, suitable corrective measures may be taken easily within the available time.
9. Fibre-reinforced soil can facilitate reinforcement of geoenvironmental barriers where continuous/systematic reinforcements may result in a preferential pathway for contaminant migrations.
10. Traditional planar reinforcements, such as geosynthetic reinforcements, when used in slopes and other such irregular sections, need anchorage and excavation into the slope, and there is a possibility of failures in addition to difficulty in placement. The use of fibre reinforcement in these applications provides a flexible solution.
11. Compared to the geosynthetic reinforcements, the fibre reinforcement can be used in a limited space, especially for the stabilization of failed soil slopes.
12. The use of fibre reinforcement can result in economical solutions because fibres have lower cost, and at some construction sites, they may be available freely as geonaturals or waste materials, such as old tyres and plastics.

In view of several advantages and favourable characteristics, the fibre-reinforced soils have a great potential for their applications in several areas, and they are now recognized as a viable ground improvement technique. The key applications are the following:

- *Geotechnical applications*: backfills behind the retaining structures; stabilization of soils beneath the footings and rafts; stabilization of failed soil slopes; construction of embankments using marginal soils, and over weak soils, such as organic soft soil deposits; lightweight fill materials; admixtures in fine sands and silts to increase the resistance to liquefaction; and strengthening the granular piles and trenches
- *Transportation applications*: pavement subgrades, subbases and bases, especially for low-volume roads; drainage layers for roads, runways, playgrounds, etc.; thermal insulator for limiting the frost penetration; and vibration damping layers beneath railway tracks

- *Hydraulic and geoenvironmental applications*: admixtures in soil to control the hydraulic conductivity; improving the soil resistance against water and wind erosion; stabilization of thin soil veneers, landfill liners and final covers; admixtures for mitigating the formation of shrinkage/desiccation cracks in compacted clays; controlling seepage and preventing the piping erosion in dams and other water-retaining structures (river levees, contour bunds, canal diversion works, check dams, etc.); leachate collection systems; and geotextile tube dewatering applications

In reinforcement applications, the fibres are generally most effective when oriented within the soil mass in the same direction as the tensile strains caused by the applied loads. Hence, for any particular loading condition, the properties of fibre-reinforced soil depend significantly on the orientation of fibres with respect to the loading direction.

In the laboratory testing and the field/practical applications, the distribution of fibres can usually be characterized by a preferred plane of fibre orientation (Michalowski and Cermak 2002; Diambra et al. 2007). The orientation of fibres depends on the following:

- Method of mixing of soil with fibres
- Specimen preparation method for tests
- Method of field placement

The laboratory test specimens are prepared in two stages, namely, mixing and compaction. Water, as required, is added to the dry soil, and then mixed together uniformly, followed by adding the fibres to the wet soil and mixing in order to have the even distribution of fibres. The test specimens in the desired test mould are prepared by one of the following techniques (Michalowski and Cermak 2002; Diambra et al. 2010; Ibraim et al. 2012):

- Tamping technique: the soil-fibre mix is compacted in specific number of layers by tamping with a rammer having a flat base.
- Vibration technique: the soil-fibre mix is densified by vibrating the test mould filled with the soil-fibre mix on a vibration table.

Tamping and vibration techniques for preparing the fibre-reinforced soil specimens in moist conditions lead to a preferred near-horizontal orientation of fibres. Both techniques leave at least 80% of the fibres oriented between $\pm$ 30° of horizontal, and 97% of fibres have an orientation that lies within $\pm$ 45° of the horizontal plane. These techniques generally produce a soil fabric/structure that resembles that of the rolled-compacted construction fills (Diambra et al. 2010; Ibraim et al. 2012). Hence, inclusion of fibres in soil in the field application may not result in isotropic properties of fibre-reinforced soil as generally considered, and hence the use of some simplified isotropic models, as discussed in Chap. 4, may not result in accurate predictions of the benefits attributed to fibres. For cases where the predominant load is perpendicular to the preferred plane of fibre orientation, the isotropic models, in general, under-predict the benefits from the fibres (Michalowski and Cermak 2002).

As there are several factors affecting the engineering characteristics of fibre-reinforced soil (see Sect. 3.2), the selection of fibres for any specific application may not be an easy task. In spite of the fact that a significant amount of research has been carried out worldwide, there is currently no scientific standard or code of practice on fibre-reinforced soils and their applications. Typically, the selection should consider the following (Hoover et al. 1982):

1. Survivability of the fibres within the soil mass with consideration of varying nature of the soil-water system in regard to alkalinity, chemical composition, temperature and environmental variations
2. Range of required mechanical properties of the fibres
3. Availability of fibres in length range required for the specific application
4. Potential inability to properly incorporate fibres into the soil to a random state of orientation
5. Procurement cost of fibres

5.3 Analysis and Design Concepts

In Chap. 3, you learnt that the engineering behaviour of fibre-reinforced soil is governed by a large number of factors/parameters. As described in Chap. 4, attempts have been made to idealize the behaviour of fibre-reinforced soil, and several models are now available to predict the specific engineering behaviour of fibre-reinforced soil. However, it is difficult to suggest sound and rational design methods for specific applications of fibre-reinforced soil as listed in the previous section. Section 5.4 discusses the experience gained in some specific applications and provides several application guidelines. Analysis and design concepts relating the basic characteristics of fibre-reinforced soils for some field applications are presented in this section. The actual design of the structure can be carried out in a conventional way, considering the fibre-reinforced soil as one of the materials used for the construction of a specific structure. For the final analysis and design of a structure being constructed with fibre-reinforced soil, if possible, a suitable field test should be carried out based on the detailed laboratory findings and observations of the behaviour of fibre-reinforced soil.

Design of fibre-reinforced soil structures can be carried out by adopting one of the following two approaches:

1. *Composite approach:* The fibre-reinforced soil structure is analysed in a traditional way, considering the engineering properties of fibre-reinforced soil as a homogeneous material. It is based on the fact that inclusion of fibres contributes to stability due to an increase in shear strength of the homogenized composite reinforced soil mass, although the reinforcing fibres actually work in tension and not in shear. The contribution of the fibres is typically quantified by an equivalent cohesion intercept and angle of internal friction angle of the soil. Several composite models, following different approaches (e.g. mechanical, statistical, energy based, etc.) have been proposed as they are presented in Chap. 4.

2. *Discrete approach:* The fibre-reinforced soil structure is analysed, considering the contributions of soil and fibres separately (Zornberg 2002). Under shearing caused by applied load, the fibre reinforcement contributes to the increase of shear resistance of soil by mobilizing the tensile stresses within the fibres. The analysis requires independent testing of soil and of fibres, but not of fibre-reinforced soil. The results obtained are more realistic. Additionally the fibres can be optimized in terms of their quality, content, aspect ratio, etc. for delivering the cost-effective designs. The fibre-induced distributed tension, σ_R, to be used in this design approach to account for the tensile contribution of the fibres in limit equilibrium analysis is taken as a minimum of σ_{RP} and σ_{Rt} as they are discussed in Chap. 4 for fibre pullout and fibre breakage cases, respectively.

In the current design practice, the fibre-reinforced soil structures are often designed by discrete approach as the most geosynthetic-reinforced soil structures are routinely designed based on working stress design method or limit state design method. Note that the fibres, being flexible, may not remain straight and they may get folded several times randomly within the specimen. It is difficult to consider this situation exactly while analyzing the behaviour of fibre-reinforced soils by discrete approach. Hence, some designers may prefer to design the fibre-reinforced soil by adopting the composite approach, especially when the fibres of relatively higher lengths are used.

In discrete approach, there is a need to properly consider the interfacial interaction of fibres with soil particles through adhesion and/or friction. Studies may be conducted to determine the ratio of adhesion to soil cohesion intercept and also for the ratio of angle of skin friction to angle of internal friction of soil as these factors are determined between soils and other construction materials (Potyondy 1961).

It is important to note that the long fibre-reinforced soils perform well when the application of loading direction and magnitude is known. When the load and its direction are not known, short, discrete, randomly distributed/oriented fibre-reinforced soil may be preferred. The basic principles of reinforcement design are the same for systematically reinforced soil and randomly distributed fibre-reinforced soils, except that suitable reduction factors should be applied in the case of randomly distributed fibre-reinforced soil. In the reinforced soil, only the reinforcements aligned normal to the applied stress, carry any stress. In randomly distributed fibre-reinforced soils, some fibres do not carry any stress at all, and this should be accounted for by *strength reduction factors*, more commonly termed the *efficiency factors* (Hoover et al. 1982).

Achieving a uniform distribution of fibres at the construction site is generally a difficult task. The poor distribution of fibres within the soil mass in the field application may not result in the design property as expected based on the laboratory tests that may have a uniform distribution of fibres in test specimens. If the distribution of fibres is not uniform, the laboratory property value should be adjusted suitably by applying a factor of safety. In the case of strength property, for considering the uncertainty induced by non-uniform fibre distribution, the design strength of fibre-reinforced soil should be reduced to a value lower than the laboratory strength.

Michalowski (2008) presented the limit analysis of anisotropic fibre-reinfroced soil, and considering the ellipsoidal distribution of fibres, defined a term as *distribution ratio* p_r varying practically between 0 and 1. If the fibre-reinforced soil behaves as an isotropic material, p_r equals 1. The kinematic approach of limit analysis was used to present the values of active earth pressure coefficient K_a and the bearing capacity factor N_γ as given in Tables 5.1 and 5.2, respectively. The total active earth pressure P_a against a rough vertical wall from the fibre-reinforced cohesionless soil backfill (Fig. 5.1) and load-bearing capacity (average stress)

Table 5.1 Active earth pressure coefficient $K_a$for a rough vertical wall with fibre-reinforced cohesionless soil backfill

ϕ (degrees)	δ (degrees)	$a_r p_{vf} \tan \phi_i$	p_r	K_a
30	15	0	–	0.301
		0.2	1.0	0.271
			0.5	0.260
			0.2	0.245
		0.4	1.0	0.242
			0.5	0.221
			0.2	0.193
		0.6	1.0	0.215
			0.5	0.184
			0.2	0.145
35	15	0	–	0.248
		0.2	1.0	0.218
			0.5	0.207
			0.2	0.192
		0.4	1.0	0.189
			0.5	0.168
			0.2	0.141
		0.6	1.0	0.162
			0.5	0.131
			0.2	0.094
40	15	0	–	0.201
		0.2	1.0	0.171
			0.5	0.160
			0.2	0.146
		0.4	1.0	0.142
			0.5	0.121
			0.2	0.096
		0.6	1.0	0.115
			0.5	0.085
			0.2	0.048

After Michalowski (2008)

Note: ϕ is the angle of internal friction of soil, δ is the soil-wall interface friction angle, ϕ_i is the fibre-soil interface friction angle, a_r is the aspect ratio of fibres. and p_{vf} is the volumetric fibre content

Table 5.2 Bearing capacity factor N_γ for fibre-reinforced cohesionless foundation soil

ϕ(degrees)	$a_r p_{vf} \tan \phi_i$	p_r	N_γ
30	0	–	21.394
	0.2	1.0	33.239
		0.5	35.775
		0.2	39.598
	0.4	1.0	53.301
		0.5	62.636
		0.2	79.380
35	0	–	48.681
	0.2	1.0	84.305
		0.5	92.280
		0.2	104.612
	0.4	1.0	155.559
		0.5	191.827
		0.2	263.931
40	0	–	118.826
	0.2	1.0	241.893
		0.5	272.732
		0.2	321.365
	0.4	1.0	561.436
		0.5	755.590
		0.2	1207.296

After Michalowski (2008)

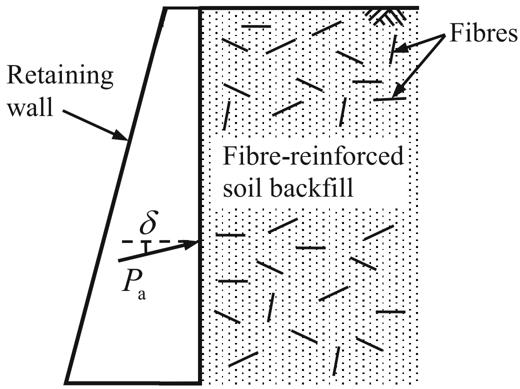

Fig. 5.1 A retaining wall with a vertical back face supporting a fibre-reinforced cohesionless soil backfill

q_a of a strip footing resting over fibre-reinforced cohesionless foundation soil (Fig. 5.2) can be determined using the following expressions along with the values of K_a and N_γ from Tables 5.1 and 5.2, respectively:

$$P_a = \frac{1}{2} K_a \gamma H^2 \tag{5.1}$$

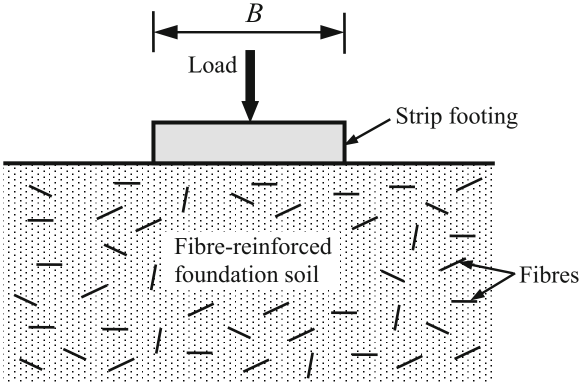

Fig. 5.2 A strip footing resting on a fibre-reinforced cohesionless foundation soil

where γ is the total unit weight of fibre-reinforced backfill and H is the height of the retaining wall.

$$q_a = \frac{1}{2}\gamma B N_\gamma \tag{5.2}$$

where γ the total unit weight of fibre-reinforced foundation soil and B is the width of the strip footing.

Note that P_a is inclined at an angle δ to the normal to the vertical back of the wall because the analysis has considered a rough vertical wall.

The following points are worth mentioning:

1. Addition of fibres to the soil backfill reduces the active earth pressure coefficient K_a and hence the total lateral earth pressure P_a on the wall.
2. The value of P_a decreases with an increase of the concentration of fibres in the backfill, but it is also affected by the distribution of fibre orientation. The near-horizontal-preferred orientations contribute significantly to the reduction of P_a.
3. As the internal friction of sand is increased with the addition of fibres, the bearing capacity factor N_γ also increases. The anisotropic distribution of fibre orientation contributes further to this increase; that is, the distribution of fibres with the horizontal preferred plane benefits the bearing capacity more than the isotropic distribution.

Example 5.1

For a fibre-reinforced sand backfill supported by an 8-m high retaining wall with a vertical back face, consider the following:

Angle of internal friction of sand, $\phi = 30^\circ$
Total unit weight of fibre-reinforced sand, $\gamma = 15.61\ \text{kN/m}^3$
Fibre-reinforced soil-wall interface friction angle, $\delta = 15^\circ$

Fibre-soil interface friction angle, $\phi_i = 20^\circ$
Fibre aspect ratio, $a_r = 75$
Volumetric fibre content, $p_{vf} = 1\%$

Determine the total active earth pressure from the fibre-reinforced sand backfill on the retaining wall, assuming the fibre-reinforced sand behaves as an isotropic material.

Solution
As the fibre-reinforced sand behaves as an isotropic material, the distribution ratio,

$$p_r = 1$$

Using the given values,

$$a_r p_{vf} \tan\phi_i = (75)(0.01)(\tan 20^\circ) = 0.273$$

From Table 5.1, the active earth pressure coefficient,

$$K_a = 0.271 - \left(\frac{0.271 - 0.242}{0.4 - 0.2}\right)(0.273 - 0.2) = 0.260$$

From Eq. (5.1), the total active earth pressure from the fibre-reinforced sand backfill,

$$P_a = \frac{1}{2}K_a\gamma H^2 = \left(\frac{1}{2}\right)(0.260)(15.61)(8)^2 = \mathbf{129.9\ kN/m}$$

Example 5.2
For a fibre-reinforced foundation sand bed supported by a 1-m wide surface strip footing, consider the following:

Angle of internal friction of sand, $\phi = 30^\circ$
Total unit weight of fibre-reinforced sand, $\gamma = 15.61\ \text{kN/m}^3$
Fibre-reinforced soil-wall interface friction angle, $\delta = 15^\circ$
Fibre-soil interface friction angle, $\phi_i = 20^\circ$
Fibre aspect ratio, $a_r = 75$
Volumetric fibre content, $p_{vf} = 1\%$

Determine load-bearing capacity of the surface strip footing resting over the fibre-reinforced sand bed, assuming the fibre-reinforced sand behaves as an isotropic material.

Solution
As the fibre-reinforced sand behaves as an isotropic material, the distribution ratio,

$$p_{\mathrm{r}} = 1$$

Using the given values,

$$a_{\mathrm{r}} p_{\mathrm{vf}} \tan \phi_i = (75)(0.01)(\tan 20^\circ) = 0.273$$

From Table 5.2, the bearing capacity factor,

$$N_\gamma = 33.239 + \left(\frac{53.301 - 33.239}{0.4 - 0.2}\right)(0.273 - 0.2) = 40.56$$

From Eq. (5.2), load-bearing capacity of the surface strip footing resting over the fibre-reinforced sand bed,

$$q_{\mathrm{a}} = \frac{1}{2}\gamma B N_\gamma = \left(\frac{1}{2}\right)(15.61)(1)(40.56) = \mathbf{316.6\ \ kPa}$$

The overall stability of a shallow foundation constructed over the weak foundation soil can be significantly improved by placing a compacted cement-stabilized fibre-reinforced soil layer of a suitable thickness (say $0.3B$, where B being the width of the footing) over the weak foundation soil as shown in Fig. 5.3. The thickness may be decided based on the plate load test. The ultimate load-bearing capacity of this layered soil system can be estimated using the analytical methods proposed by Vesic (1975) and Mayerhof and Hanna (1978). The later considers a punching failure along the footing perimeter. Consoli et al. (2003) have reported that Vesic's method significantly overestimates the bearing capacity of the footing resting on the layered system, while Meyerhof and Hanna's method underestimates it. A research is required to develop an appropriate bearing capacity equation for determining the bearing capacity of footings resting on cement-stabilized fibre-reinforced soil layer over the weak foundation soil.

Note that for the field situation shown in Fig. 5.3, if required, a geosynthetic reinforcement layer (woven geotextile or geogrid layer) can be installed at the interface of the cement-stabilized fibre-reinforced soil layer and the weak foundation soil for having additional reinforcement benefits. At several construction sites, the fibre-reinforced foundation soil with or without geosynthetic reinforcement may be technically feasible for supporting the structural loads, and additionally they may have the following advantages:

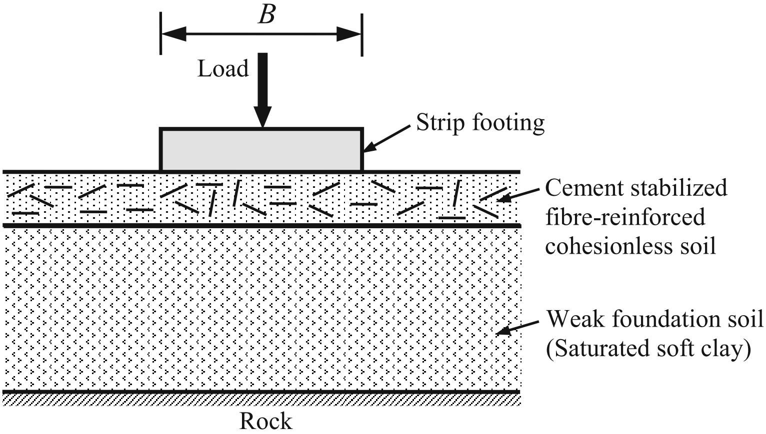

Fig. 5.3 A strip footing resting on a cement-stabilized fibre-reinforced cohesionless soil layer over a weak foundation soil stratum

1. The use of fibres from the waste materials, such as old/used tyres and used plastic materials, in large quantities, whose presence causes environmental problems and their safe disposal in engineered landfills costs significantly
2. Reduced foundation cost compared to the cost of deep foundations, such as piles that carry the loads and transfer them to the rock bed or firm stratum underlying the weak foundation soil

Young's modulus is often the dominant parameter for the design of shallow foundations. Although the fibre-reinforced soil is more compressible than unreinforced soil, it still complies with the stiffness requirements for several applications (50–120 MPa), namely, shallow foundations, subgrade, capping or subbase layers. This compliance means that the fibre-reinforced soil is a suitable material for construction of these structures. In fact, a reinforced soil exhibits a suitable bearing capacity and trafficking under the heavy construction machines. Driving passes of the motor scraper in the close proximity to the borders of fibre-reinforced embankment confirm the good quality of the reinforced soil as a construction material (Falorca et al. 2011).

Design of fibre-reinforced granular pavement layers requires evaluation of fibre-reinforced granular materials by the trafficability test, simulating the conditions of repetitive traffic loading and adverse environment. Details of the test and findings in terms of variation of average rut depths with number of load cycles are presented by Hoover et al. (1982) for some fibre-reinforced soils. The findings show that the inclusion of fibres improves the vertical load stability and prevents lateral shear and/or displacement for a greater number of load cycles when compared to the behaviour of unreinforced soil. Further improvement of stability, compressive characteristics, ductility and control of cracking through brittle failure can be obtained through addition of cement or lime, mainly due to improved soil-fibre interfacial bonding. The crimped PP fibres are found to be most effective in improving the engineering characteristics of soil. The scanning electron microscopy shows that the straight PP fibres exhibit severe surficial damage after compaction and testing, and the glass fibres appear undamaged.

Benefits of reinforcing the pavement layers (subgrade, subbase course and/or base course) with fibres (or other reinforcement types) are generally expressed in terms of an extension of service life of the pavement and/or a reduction in the thickness of pavement layer. An extension of service life of the pavement is typically expressed in terms of the *traffic benefit ratio* (*TBR*), which is defined as follows (Perkins and Edens 2002):

$$TBR = \frac{N_{\mathrm{R}}}{N_{\mathrm{U}}} \tag{5.3}$$

where N is the number of traffic loads/passes required for producing a pavement surface deformation (i.e. rutting) up to the allowable rut depth for the pavement and the symbols U and R denote unreinforced and reinforced pavement sections, respectively. Thus, the *TBR* basically indicates the additional traffic loads/passes that can be applied to the pavement with fibre-reinforced layer, with all other pavement materials and geometry being equal.

The benefit of reinforcing in reduction in the thickness of pavement layer (subbase course, base course and/or surface course) is typically expressed in terms of *layer reduction ratio* (*LRR*), which is defined as follows:

$$LRR = \frac{D_{\mathrm{U}} - D_{\mathrm{R}}}{D_{\mathrm{U}}} \tag{5.4}$$

where D_{U} and D_{R} are the pavement layer thickness of the unreinforced and fibre-reinforced pavement sections, respectively, for equivalent service life. If the layer is base course, the layer reduction ratio (*LRR*) may be better called *base course reduction ratio* (*BRR*) as considered by Perkins et al. (2002).

The pavement can also be designed for any intermediate thickness to reduce the thicknesses of the pavement layers and/or to gain additional benefits in terms of extension of the service life of the pavement. For example, by reinforcing the subgrade soils, the thickness of the subbase course can be reduced as required. If a reduction in the thickness of the subbase course is not required, then with fibre inclusions into subgrade soils, the benefits can be achieved in terms of *TBR*. Thus the actual advantage of fibre inclusions depends upon the option exercised by the pavement designer (Chandra et al. 2008).

For the design of a pavement structure, repeated dynamic load test may be conducted in a model test tank as Kumar and Singh (2008) presented their work using a steel tank of size 0.6 m × 0.6 m × 0.6 m and creating a pavement structure with subgrade, subbase and base courses. Wet Mix Macadam (WMM) was used as the base course. For the rural roads, the traffic load was considered the medium load (41 kN). The number of load cycles applied on the top of the base course was limited to 10,000 only. Table 5.3 shows the deformation of the top surface of the

Table 5.3 Rut depth in a model section after 10,000 cycles in each case

Combination of subbases	Rut depth (mm)	Percentage decrease in rut depth
Fly ash only	6.07	
Fly ash + 0.2% fibres	4.29	29
Fly ash + 0.3% fibres	3.25	46
Fly ash + 25% soil	4.44	27
Fly ash + 25% soil + 0.2% fibres	3.20	47

base course for different combinations of subbases of fly ash (silt of low compressibility, ML) and local soil (poorly graded fine sand, SP) with PP fibres. Note that the rut depth is an indicator of the life of the pavement. A reduction in the rut depth due to loading shows more expected life. Fly ash alone has 6.07-mm rut depth, but in the case of fly ash with 25% soil and 0.2% fibres, rut depth is nearly half of the rut depth in the case of fly ash only.

Jha et al. (2014) analysed the benefit of reinforcing the industrial waste materials (fly ash, stone dust and waste recycled product) with HDPE plastic waste strips in terms of *LRR* for their applications as the subbase course material in the construction of flexible pavements. Their analysis shows that with inclusion of plastic waste strips into industrial wastes, the thickness of subbase course can be reduced up to 50%, depending on the service life and site requirements. The reduced thickness of the pavement layers results in lower total cost of the pavement and lower construction time, thus consuming reduced quantities of natural soil and materials for construction and hence providing an environmentally friendly solutions in a sustainable manner.

Permeability is the most important design parameter for water-retaining structures. The effect of fibre inclusions in soil on its permeability has been presented in Sect. 3.6. In general, inclusion of fibres in soil decreases its coefficient of permeability (a.k.a. hydraulic conductivity), and hence this benefit can be used in reducing the seepage through the body of water-retaining soil structures. Babu and Vasudevan (2008) have explained the benefits of mixing coir fibres into red soil used for the construction of a temporary check dam over an impermeable bed (Fig. 5.4). The seepage analysis shows that an increase in fibre content decreases the discharge per unit length of the dam (Fig. 5.5). Thus the coir fibres are effective in controlling the seepage through the body of check dams, which are often constructed for the water conservation purposes in many countries.

Sheet pile walls have several applications in civil engineering as permanent and temporary structures. A sheet pile wall embedded in a soil is often used to retain water. Babu and Vasudevan (2008) have also explained the benefits of replacement of the soil deposit by coir fibre-reinforced soil on the downstream side of the sheet pile wall (Fig. 5.6). The analysis for soil piping shows that as the fibre content in soil increases, the factor of safety against the piping failure, as determined from Eq. (1.27), also increases (Fig. 5.7). The maximum exit hydraulic gradient may be calculated using the following relationship (Harr 1962):

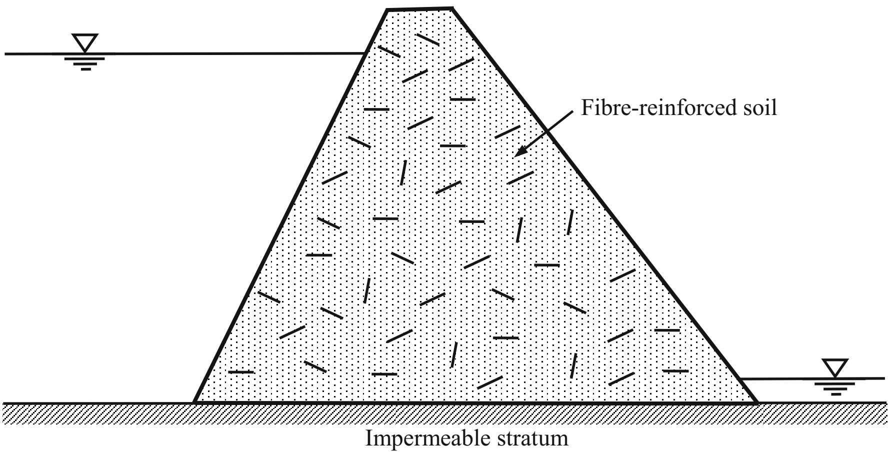

Fig. 5.4 A check dam constructed with coir fibre-reinforced red soil

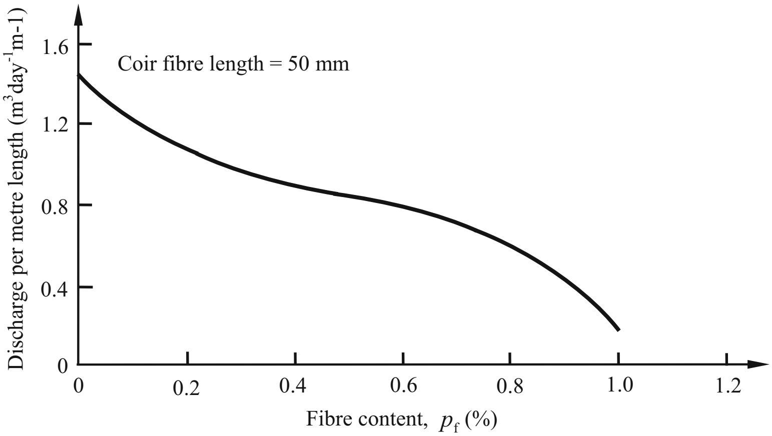

Fig. 5.5 A typical variation of discharge per unit length of the check dam, constructed with coir fibre-reinforced red soil over an impermeable stratum, with fibre content (Adapted from Babu and Vasudevan 2008)
Note: Base width of dam = 10 m; height of dam = 2 m; top width of dam = 2.48 m; side sloping angle = 28°; dry unit weight of soil = 14.3 kN/m^3; moulding water content = 17.8%; and free board = 0. 2 m

$$i_e = \frac{H}{3.14D} \tag{5.5}$$

where H is the total head lost in the flow and D is the depth of penetration of sheet pile wall. Equation (5.5) is based on the assumption that the impermeable layer is available at a shallow depth.

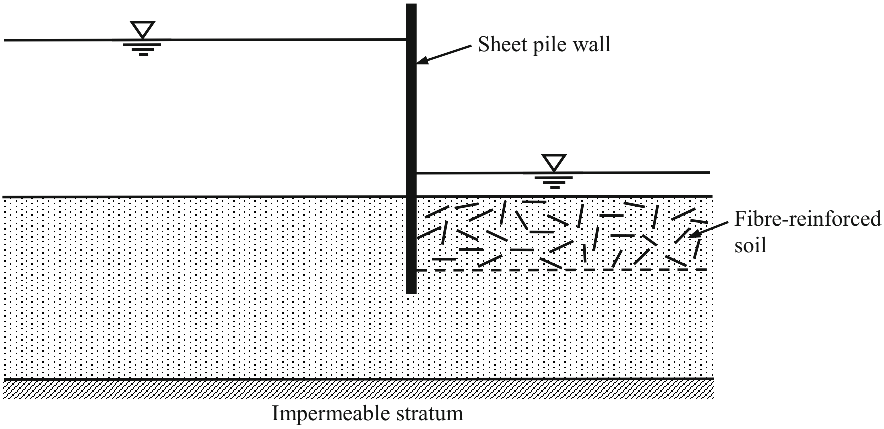

Fig. 5.6 A sheet pile wall with compacted coir fibre-reinforced soil on the downstream side

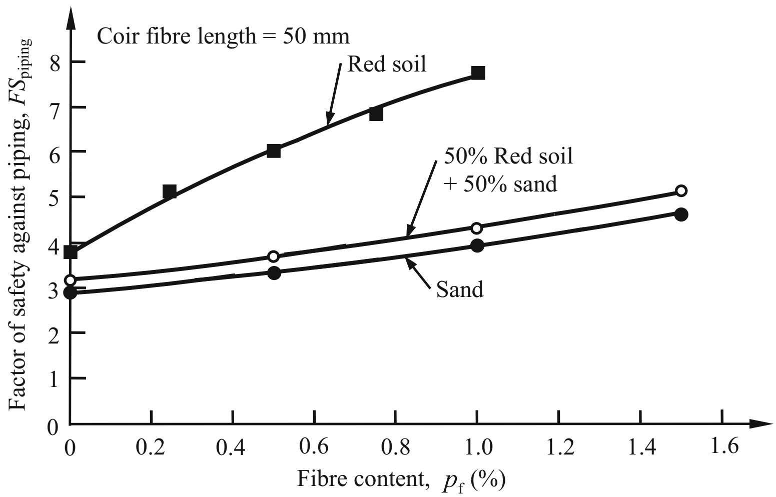

Fig. 5.7 Effect of fibre content on factor of safety against piping failure for the sheet pile with coir fibre-reinforced soil on the downstream side (Adapted from Babu and Vasudevan 2008)
Note: Embedded depth = 2 m; upstream water depth = 2.5 m; downstream water depth = 0.5 m

Desiccation cracking of soil is a problem encountered in many engineering disciplines, including geotechnical and geoenvironmental engineering. Desiccation of landfill clay liners is a major factor affecting landfill performance. Desiccation leads to the development of shrinkage cracks. The cracks provide pathways for moisture migration into the landfill cell and thus increases the generation of waste leachate and ultimately increases the potential for soil and groundwater contamination. For liner design applications, Miller and Rifai (2004) presented the concepts of crack evaluation. The following terms can be used to assess the liner crack potential:

1. Crack intensity ratio (*CIR*): It is defined as the ratio of cracked area (A_C) to the total surface area (A) of the soil, that is,

$$CIR = \frac{A_C}{A} \tag{5.6}$$

   The value of *CIR* is normally expressed as a decimal.

2. Crack Reduction Factor (I_{CR}): It is defined as

$$I_{CR} = \frac{CIR_U - CIR_R}{CIR_U} \tag{5.7}$$

where CIR_U is the crack intensity ratio for unreinforced soil and CIR_R is the crack intensity ratio for fibre-reinforced soil. The value of (I_{CR}) is normally expressed as a percentage.

For landfill applications, Miller and Rifai (2004) studied the effect of PP fibre reinforcement (length varying from 0.5 to 2 in.) on desiccation cracking of medium plasticity soil (classified as CL with liquid limit = 40 and plasticity index = 17) compacted at 2% wet of the optimum water content as a function of fibre content and crack reduction. The relationship between crack reduction and fibre content (Fig. 5.8) shows that increasing the fibre content, from 0.2 to 0.8%, significantly increases the crack reduction, from 12.3 to 88.6%, respectively. The slope of the curve in Fig. 5.8 suggests that increasing the fibre content would have increased the crack reduction further. However, exceeding a fibre content of 0.8% is not practical due to difficulty in fibre-soil mixing to obtain uniform distribution of fibres within the soil. The cracks are observed wider and more intensive in the natural soil specimen than those shown in the fibre-reinforced soil specimen. The cracks in the latter are so small that they are barely visible.

The magnitude of hydraulic conductivity of clayey soil should be one of the primary characteristics used to judge its acceptability for containment structures (i.e. landfill covers and bottom liners). Therefore, it is critical that the effect of fibre inclusion on the hydraulic conductivity of the clayey soil should be evaluated as one of the design steps for covers and liners. Miller and Rifai (2004) conducted hydraulic conductivity test on a medium plasticity soil (classified as CL with liquid limit = 40 and plasticity index = 17) using modified compaction effort and 2% wet of optimum water content. The tests were performed using fibre contents of 0.0, 0.2, 1.0, 1.5 and 2.0%. The test results, as presented in Fig. 5.9, indicate that the hydraulic conductivity of the fibre-reinforced soil is dependent on the fibre content, generally increasing with fibre content increase. The slight decrease of hydraulic conductivity noted around 0.2% fibre content is within the limits of experimental error and should not be used to infer that minor fibre additions improve the

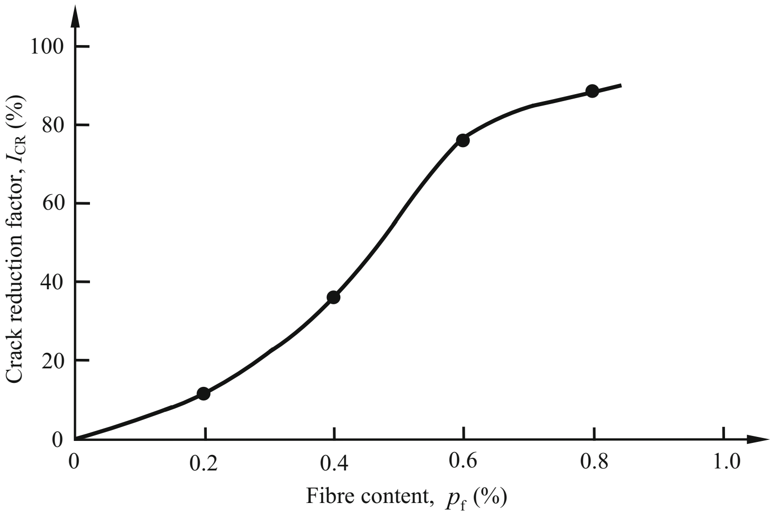

Fig. 5.8 Variation of crack reduction factor for a medium plasticity soil with fibre content (Adapted from Miller and Rifai 2004)

hydraulic conductivity. The increase in hydraulic conductivity is most significant for fibre contents exceeding 1%.

Note that the optimum fibre content that is necessary to achieve the maximum crack reduction, maximum dry unit weight and acceptable hydraulic conductivity within the range of mixing workability is found to be between 0.4 and 0.5% in the experimental study conducted by Miller and Rifai (2004).

5.4 Field Application Experience and Guidelines

Influences of the engineering properties of soil and reinforcement and the scale effects on the properties of the fibre-reinforced soils have not been investigated fully, and hence the actual behaviour of fibre-reinforced soil is not yet well known. The large-scale investigations for all possible applications of fibre-reinforced soil are limited in the literature, and hence they are the subject of further study. Thus, the use of randomly oriented discrete fibres for different applications, as described here, requires investigations at a large scale.

Park and Tan (2005) conducted full-scale tests on a retaining wall with PP fibre-reinforced soil backfill (with 0.2% fibre content), with and without geogrid reinforcement (tensile strength of 50 kN/m in machine direction and 20 kN/m in cross machine direction), in Korea Railway Research Institute, using a large soil box (22 m in length, 5 m in width and 3 m in depth), a loading frame and a reaction plate. The test was fully instrumented with earth pressure cells, displacement gauges, load cells and settlement plates. The numerical model of the wall with

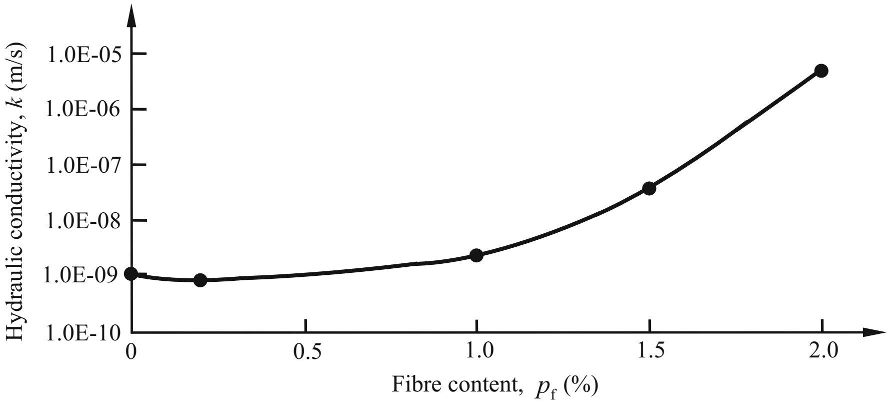

Fig. 5.9 Variation of hydraulic conductivity of medium plasticity soil with PP fibre content (Adapted from Miller and Rifai 2004)

reinforced backfill was also developed using a finite element programme (2D PLAXIS version 7.2). Some of the key observations are given below:

1. Distribution of vertical earth pressure is almost the same for all cases of unreinforced as well as reinforced soil system. When a load is applied on the backfill, the fibre-reinforced soil backfill shows slightly smaller vertical stress increase compared to unreinforced soil.
2. Immediately after construction, and upon loading, the fibre-reinforced soil backfill without geogrid appears to have the smallest horizontal pressure distribution.
3. Wall deflection is smaller if the fibres are included within the backfills. The presence of geogrid reinforcement reduces the wall deflection further to some extent, but very small reduction in deflection cannot justify the cost of using geogrid reinforcement along with fibres.

The fibre-reinforced compacted soil backfill is thus stronger and stiffer than unreinforced compacted soil backfill. Inclusion of geosynthetic reinforcement layers within the fibre-reinforced soil backfill can lead to an economical construction of very high retaining wall, even with a vertical face, especially in built-up areas.

At some sites, shallow foundations and pavements are often constructed along the ground supported by sheet pile walls or other retaining structures, as shown in Fig. 5.10. Nasr (2014) studied experimentally and numerically the potential benefits of reinforcing the poorly graded sand backfills at a relative density of 50% in the active zone behind a model steel sheet pile wall (750 mm long, 499 mm wide and 3.3 mm thick) by using PP fibres (12 mm long, 0.023 mm thick) and cement kiln dust (15.42% SiO_2, 3.92% Al_2O_3, 2.95% Fe_2O_3, 51.23% CaO, etc.). The test involved loading a rigid strip footing (499 mm long, 100 mm wide and 25 mm thick) resting on the sand backfill surface in the active zone adjacent to the sheet pile wall in a steel test tank (1.5 m long, 0.5 m wide and 0.9 m high). The sheet pile

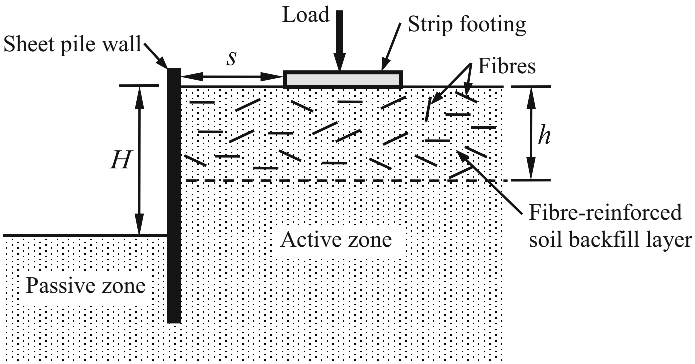

Fig. 5.10 Loaded active zone adjacent to a sheet pile wall (Adapted from Nasr 2014)

wall was embedded 500 mm in the sand bed. The results obtained from the study suggest the following:

1. Addition of PP fibres to the cemented sand increases the ductility as indicated by the increase in deformability index D_i. For lower fibre content ($\leq 0.5\%$), D_i is weakly influenced by the cement kiln dust content.
2. The presence of fibres in the cemented sand backfill behind the sheet pile wall decreases the lateral deflection of the wall significantly. For the cement kiln dust content (as per Eq. (2.9)) of 9%, the maximum lateral deflection decreases by about 51% at a fibre content (as per Eq. (2.10)) of 0.75%.
3. Ultimate bearing capacity of the strip footing resting on the fibre-reinforced cemented sand in the active zone increases with an increase in the thickness of reinforced sand layer. However, for higher fibre content ($\geq 0.75\%$), an increase in ultimate bearing capacity is not significant at a thickness h of fibre-reinforced cemented sand layer greater than $0.4H$($H=$ wall height; see Fig. 5.10).
4. Increasing the distance between the strip footing and the sheet pile wall leads to a significant increase in ultimate bearing capacity of the footing and a decrease in the maximum lateral deflection of the sheet pile wall. However, for higher fibre content ($\geq 0.75\%$), if the distance s between the strip footing and the sheet pile wall is more than $0.8H$, there is no appreciable increase (about 4.8%) in ultimate bearing capacity of the footing.

Sand and sandy gravels are generally not used in the top courses (high level of base course or surface course) of pavements although they are easily available at several construction sites because their properties do not meet the technical requirements of top courses. In order to utilize sand as a construction material on a larger scale in the top courses, sand can be reinforced with short fibres by mixing sand and fibres by the mix-in-plant process. Lindh and Eriksson (1990) provided the details of a pilot experiment performed in 1988 on two pavement test stretches, 20 m and 40 m, by incorporating a 100-mm layer of sand in the pavement mixed with short (48-mm) plastic fibres at 0.25 and 0.5% fibre contents, respectively. Sand, water and plastic fibres were mixed successfully in a concrete mixing plant of the drum

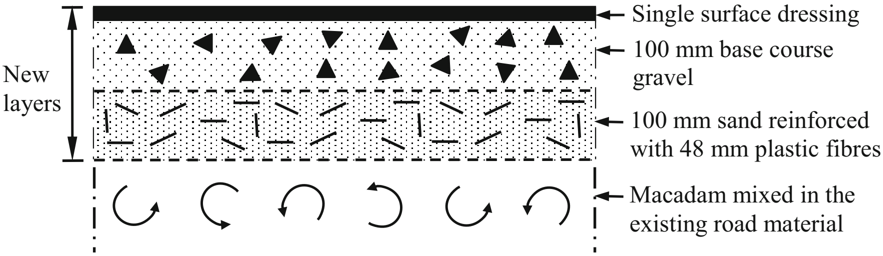

Fig. 5.11 Longitudinal test section of road pavement (lengths of 20 m and 40 m with the use of plastic fibre contents of 0.25% and 0.5%, respectively, in the sand layer) (Adapted from Lindh and Eriksson 1990)

mixer type. The 100-mm reinforced sand layer was applied directly on the old road surface. A 100-mm layer of base course gravel was applied on the sand layer, followed by a surface dressing (see Fig. 5.11). Prior to applying the fibre-reinforced sand, the grading curve of the material in the surface layer of the old road was improved by mixing in macadam. The spreading and rough levelling of the fibre-reinforced sand were performed with a grader. The normal shaping could not be performed on the reinforced sand as it became matted, and adhered to the shaper in lumps. The final shaping of the surface therefore had to be performed with hand tools. After compaction, the fibre-reinforced sand had considerably better stability than the layers of packed sand, and showed no rutting of the road surface during about the first 2 years' use by traffic; thus the test stretches performed well. With inclusion of fibres, the sand absorbs tensile strains and can therefore improve resistance to permanent deformation of sand layer when loaded.

Santoni and Webster (2001) described the laboratory and field tests conducted using a new fibre stabilization technique for sands. Laboratory unconfined compression tests using 51-mm long monofilament PP fibres to stabilize a poorly graded (SP) sand showed an optimum fibre content of 1% (by dry weight). The field test sections were constructed and traffic tested using simulated C-130 aircraft traffic with a 13,608 kg tyre load at 690 kPa tyre pressure and a 4536 kg military cargo truck loaded to a gross weight of 18,870 kg. The test results showed that the sand-fibre stabilization over a sand subgrade supported over 1000 passes of a C-130 tyre load with less than 51 mm of rutting. The top 102 mm of the sand-fibre layer was lightly stabilized with tree resin to provide a wearing surface. Based on limited truck traffic tests, 203-mm thick sand-fibre layer, surfaced with a spray application of tree resin, would support substantial amounts of military truck traffic.

Tingle et al. (2002) reported the details of two field test sections to evaluate the ability of fibre-stabilized sand beds to sustain military trucks. It is observed that the fibrillated fibres provide the best rut resistance, followed by the monofilament, the tape and then the Netlon mesh elements. The 0.8% fibre content recommended by Santoni et al. (2001) showed to provide adequate structural support for the test traffic. Slightly better performance was noted at a fibre content of 1%, but the slight increase in rut resistance did not justify the added cost of the additional fibres. The field tests demonstrated similar performance between 51-mm (2 in.) and 76-mm (3 in.) fibre lengths, but the 76 mm tended to hand up on the mixing equipment. Thus, 51-mm fibre length appears to be more appropriate for field use. In general, the design criteria for unsurfaced roads are based on the development of a 76-mm (3 in.) rut upon completion of the design traffic. The amount of rutting, 63–89 mm (2.3–3.5 in.), exhibited in the field tests indicates that the design thickness of 203 mm (8 in.) was appropriate for the design traffic, except for the case with the Netlon mesh fibres. The test sections demonstrated the need for a surfacing for fibre-stabilized layers to keep the tyre friction from pulling the fibres out of the sand. Road Oyl provided a good wearing course for the applied traffic. Cousins Pine Sap Emulsion and PennzSuppress D also provided adequate resistance to fibre pullout. The hexagonal mat surfacing provided an adequate wearing course, but no additional structural strength was provided by the mat since the sand had already been confined by the fibres. Note that the majority of the permanent deformation or rutting that occurred during the test period consisted of densification of the stabilized sand and supporting sand layers.

Based on the available experience and studies reported in the literature as well as the author's experience, some application guidelines are given below:

Care and Consideration

- There are a large number of factors and variables that affect the engineering behaviour of fibre-reinforced soils. The stress-strain properties of fibre-reinforced soils are functions of fibre content, aspect ratio and skin friction along with the soil and fibre index and strength characteristics and confining pressure. Thus, the design and construction of fibre-reinforced soil structures should properly consider all these variables.
- Because of the major influence of confining stress, the design parameters should be based on the triaxial compression tests, especially on the large-size specimens.
- The method of fibre-reinforced soil placement should be similar to the method used for preparing the fibre-reinforced soil specimens for the tests conducted to obtain the design parameters. This is essential to maintain the fibre orientations the same in both the tests and field applications.
- In general, fibres are most influential when orientated in the same direction as the tensile strains. Therefore, for any particular loading condition, the effectiveness of fibre inclusions depends on their orientation, which in turn depends on the sample mixing and formation procedure. However, an attempt should be made to make the reasonably uniform distribution of the fibres within the soil mass.
- The in situ mixing and compacting may cause preferred near-horizontal orientations; hence, the properties of fibre-reinforced soil are anisotropic to some extent.
- The consequence of an assumed isotropy as generally expected in the soil-fibre mixes may result in an overestimation or underestimation of reinforced soil design parameters, depending on the direction of practical importance.
- Since the limited settlement is generally the design criterion for actual foundations on soil, a comparison of the bearing pressure values at some selected

settlement levels for the reinforced and unreinforced cases should be made for the design purpose.
- Site-specific laboratory and field tests should be conducted to determine the design parameters/variables.
- As several contradictory observations/conclusions have been reported in different studies, they should be used carefully in analysis and design of fibre-reinforced structures.
- As the significant influence of fibre reinforcement on the ultimate strength of fibre-reinforced soil continues to be observed even at very large shear strains (horizontal displacements) in the laboratory study, there is no tendency to lose strength, and the fibre-reinforced soils would therefore be unlikely to suffer from brittle failure in field applications even in cases where the strains localize, as they do in a ring shear apparatus. Thus there is a great potential of PP and other similar fibres as the soil reinforcement (Heineck et al. 2005).
- The use of natural fibres such as coir and jute fibres may make possible constructions, such as embankments and bunds, rural road bases, etc., cost-effective and environmentally friendly, especially in applications for short duration of 2–3 years in order to have short-term stability. Of most natural fibres, coir has the greatest tearing strength and retains this property even in wet conditions.

Mixing, Placement and Compaction

- The simplest method of mixing fibre and soil in a rotating drum mixer does not result in a uniform mixture due to a large difference in specific gravities of fibre and soil. The drum mixing method usually results in the segregation or floating of fibres even when some water is added. In fact, because of the lightweight and low specific gravity of fibres (for some types it can be even less than unity) compared to soil particles, the fibres cannot be uniformly distributed in the mixture during drum rotation.
- The effective method of fibre inclusion can be spraying the fibres over each soil lift during field compaction, especially in pavement bases and subbases. The main advantages of fibre reinforcement in bases and subbases are increase in load-carrying capacity, reduction in rut depth, reduced cost, etc.
- One of the most satisfactory mixing techniques can be provided by blowing fibres into a rotary mixer chamber with a mulch spreader equipped with a flexible hose. Blade mixing provides a reasonable satisfactory random distribution of fibres (Hoover et al. 1982).
- Mixing is a critical factor in the case of discrete, randomly oriented fibre reinforcement. Blade- or paddle-type mixers do not work as they tend to drag and ball up the fibres. Vibratory mixers tend to float the fibres up. A special oscillatory or helical action mixer can be used to avoid these problems; but even this type of mixer has limitations on the maximum fibre content that can be uniformly and randomly distributed in the mix. The degree of randomness in the mixture may be determined by visual inspection (Gray and Al-Refeai 1986).
- Compared to hand mixing, the z-blade mixer produces suitable even mixes of clay and PP fibres within a reasonable time (Gelder and Fowmes 2016).

- Water content of the soil should be lower than the optimum water content corresponding to the level of compaction required to facilitate pulverization of the soil particles.
- In order to easily mix the fibres uniformly, the initial/natural water content of the cohesive soil may be increased to optimum mixing moisture content (OMMC) prior to the introduction of fibres if lime has to be added. By introducing quicklime, excess moisture is removed through the hydration (exothermic reaction) process, thus improving the workability of soil. If lime is not added, the mixture should be allowed for air drying at the site to get the moisture content reduced to the target value (Gelder and Fowmes 2016).
- For preparing a good-quality mixture of fibres and clayey soil, the water content of the soil may be kept near its plastic limit, which may fall on the wet side of the compaction curve (Falorca and Pinto 2011).
- For proper mixing of fibres with soil, water required to obtain the target water content should be added to soil prior to placing the fibres on the soil. This is done to keep the fibres from sticking together during mixing (Tingle et al. 2002).
- Higher fibre content, longer fibres and crimped fibres can make the mixing procedure more difficult.
- Compaction of soil fibres can be done in two stages: one pass of the motor scrapper followed by 4–6 passes of a roller.
- A vibratory rubber-tyred roller may be most suitable for the compaction of fibre-reinforced soils (Falorca et al. 2011).
- More compaction energy may be necessary to produce specimens with higher fibre contents at a given dry unit weight. Thus, the fibre-reinforced soil may provide an increased resistance to compaction, even in field compaction.
- In general, for the embankment construction, the soil-fibre mixture should be produced and placed in a single step only. It is not feasible to spread and level the mixture by maintaining the homogeneity, as some clods of fibres are produced and dragged by the motor scraper. The reinforced layers must therefore be ready for compaction immediately after placement, with no need for levelling procedures. For a better uniform distribution of fibres within the compacted soil mass, the soil and fibres may be spread in a sandwich pattern and then mixed with a rotary tiller, in the following steps (Falorca et al. 2011):

  - The predetermined quantity of fibres is spread uniformly by hand or other suitable means over the surface.
  - The necessary amount of soil is then spread over the fibres.
  - Finally, the rotary tiller is driven along the entire area of the section in a regular pattern at a slow speed, combined with high rotation, in order to have a satisfactorily homogeneous mixture.

These steps can be repeated until a 0.2-m thick layer is completed. The number of sub-layers needed to complete a 0.2-m thick layer is found to depend on the fibre characteristics. This procedure becomes impractical when the number of sub-layers is greater than six. For the highest fibre content and lowest fibre diameter, the maximum number of fibres needs to be mixed with the soil. This is the most difficult situation, which requires the highest number of sub-layers.

- A thin top soil layer of about 50-mm thickness should be placed over the entire surface area of the embankment to protect the synthetic fibres, such as PP fibres from UV degradation. The top soil layer also helps minimize fissures which may be caused by a recovering deformation of the reinforced soil (Falorca et al. 2011).
- In pavement base/subbase construction, the selected type and amount of fibres should be weighed and can be uniformly spread by hand across the surface of the moist base/subbase material. Four to six passes of a self-propelled rotary mixer should be initially used to mix the fibres with base/subbase material. Then the material should be piled and releveled, and four to six additional passes of the rotary mixer should be used to uniformly mix the fibres with the material. The fibre-reinforced material should then be dumped in place as required for compaction by rollers.
- If required, the individual fibres from the yarns can be separated by first, punching a few holes with a paper hole punch near the closed end of a large (say, 125 L) plastic bag. Next, a handful of yarn fibres are placed in the bag. The bag is hand-held closed around an air nozzle and inverted, and air is blown through the fibres. The air separates the fibres from the yarn effectively and promptly. The separated fibres form the fluffy bundles that resemble cotton candy (Santoni and Webster 2001).
- When tyre chips are used, construction activities may be eased by specifying tyre chips less than 75 mm (maximum dimension). For compaction of tyre fibre-reinforced soil, vibratory roller should not be specified. Compaction specifications should not be based on a final unit weight, but the optimum number of passes should be determined based on a test section in the field. Compressibility is the governing parameter in designing structural fills using tyre chips. To achieve minimum compressibility, a minimum soil cover thickness of 1 m over the tyre chips should be specified. The use of a geotextile to separate the cover soil from the porous tyre chip fill is recommended to prevent migration of the soil into the tyre chips pores. Sand-tyre chips mixtures exhibit higher moduli than clay-tyre chips mixtures at the same soil-tyre chips ratio (Edil and Bosscher 1994).

Durability

- Attempts are being made to increase the long-term durability of fibres in a cost-effective way, such as coating of fibres with phenol and bitumen, and probably in the future, several coating methods will be available. However, natural fibres can be used routinely in less critical applications (e.g. pavement bases/subbases) or short-term applications (e.g. erosion control).
- The surface of fibres may be made rough by cementing a layer of suitable materials, such as fine sand particles, to the fibres for achieving a full mobilization of soil angle of internal friction during shearing.

- Natural fibres may be protected from biodegradation by coating with suitable materials. Ahmad et al. (2010) coated oil palm empty fruit bunch (OPEFB) fibres with acrylonitrile butadiene styrene (ABS) thermoplastic for increasing the resistance to biodegradation. A layer of coating on a fibre also increases its diameter and the surface area, resulting in increased interface friction of fibre and soil as well as the fibre tensile strength. Hence, as an additional advantage, the inclusion of coated OPEFB fibres in silty sand increases its shear strength much more compared to uncoated fibres, as reported by Ahmad et al. based on the results obtained from consolidated drained and undrained triaxial compression tests. Sarbaz et al. (2014) used bitumen-coated palm fibres for reinforcing fine sand and studied the effect of bitumen coating on the *CBR* strength of sand.

Application Experience

- Stability of retaining walls under dynamic load conditions may require the use of more stable backfills. For increase in stability of walls by the reduction of lateral earth pressure, short fibres (say, 60-mm long PP fibres) may be included randomly in soil backfills (Park and Tan 2005). The beneficial effect can be more to some extent when short fibres are used in combination with geogrid reinforcement within the backfill.
- Inclusion of tyre chips (20 mm long, 10 mm wide and 10 mm thick; specific gravity = 1.08 and unit weight = 6.45 kN/m^3) up to 30% by weight in the poorly graded sand backfill behind the wall effectively works to reduce the wall displacements and lateral earth pressures against the wall by about 50–60%, thereby resulting in reduced dimensions of the wall (Reddy and Krishna 2015).
- Shallow foundations on fibre-reinforced cohesionless/granular soil deposits should be designed based on the findings of large-scale model footing tests or, if possible, by testing full-scale trial footings; otherwise, a simple test may be devised, for example, in the form of measuring the imprint dimensions or the penetration depth of a falling object (Wasti and Butun 1996).
- Even though the sand-cement layer provides a higher bearing capacity when compared to cement-stabilized fibre-reinforced sand layer, the latter, in terms of post-peak behaviour, leads to a more reliable solution and possibly to a reduction in the design safety factor, because the fibre inclusion reduces dramatically the brittle response of the foundation soil system (Consoli et al. 2003).
- The beneficial effects of fibre reinforcement in soil can be enhanced significantly by stabilizing the fibre-reinforced soil with cement in a suitable quantity, say using cement content of 5–10%.
- The footing or pavement load should be placed suitably away from the retaining structure to get maximum benefits from inclusion of fibres in terms of increase in the load-bearing capacity of the footing.
- The granular pile and trench, which are used for stabilizing the weak clayey foundation soil, can be strengthened by inclusion of fibres in place of reinforcing them by layers of geosynthetic reinforcement (Gray and Al-Refeai 1986).
- With increase in shear strength of soil with inclusion of fibres, a steeper slope may be constructed, resulting in saving of the land area, which may be utilized

suitably. The slope stability analysis of the fibre-reinforced slopes may be carried out using the discrete approach, which considers the fibre-induced distribution of tension parallel to the failure plane along with the soil shear resistance separately.

- Decrease in rut depth, which is an indicator of increase in the life of pavement, takes place as a result of inclusion of fibres in soil and other similar materials such as fly ash.
- Stabilization of sand with discrete fibres is a viable alternative to traditional stabilization techniques for low-volume road applications. Fibre stabilization requires less material (fibre additives) by dry weight than most traditional stabilization techniques. The construction and maintenance techniques have been shown to be practical and successful in terms of maintaining the serviceability of the road. However, the densification of fibre-stabilized materials under repeated traffic loadings may limit the applicability of this technology for use in situations in which settlements and deformations cannot be tolerated (Tingle et al. 2002).
- During trafficking by the heavy construction machinery, in terms of depth of ruts, the fibre-reinforced soil sections are observed to be more stable than the unreinforced sections.
- In case of rural roads, the subbase material should have a minimum soaked *CBR* of 15%. Fly ash reinforced with PP fibres with 0.2% fibre content has been found to have *CBR* of 16.6%; therefore, this is suitable for rural road subbases. Fibre content can be reduced to 0.1% if fly ash is mixed with 25% of poorly graded fine sand (SP) (Kumar and Singh 2008).
- Cement-stabilized soils are often used as pavement base courses, backfills behind retaining walls, embankments and foundation. As the cement content increases, the strength properties of soil improve significantly, but an increase in cement content causes a brittle or sudden failure without a plastic deformation. As the brittle/sudden failure is undesirable in engineering applications, for preventing the brittle failure, fibres may be added to cement-stabilized soil in a suitable quantity with a random distribution.
- Fibre inclusions with an optimum aspect ratio increase the *CBR* of the soil subgrade/subbase/base courses and hence may cause a substantial decrease in design thickness of the pavement layers, thus making the pavement project economical.
- Fibre inclusions mixed with soil subgrade also provide needed tensile strength under traffic loads.
- The fibre reinforcement of roadway soils should be associated with base or subbase courses having adequate surfacing (Hoover et al. 1982).
- Shredded waste tyres can be used as soil subgrade reinforcement, aggregate in leach beds for septic systems, additive to asphalt, substitute for leachate collection stone in landfills, daily cover materials in landfills, lightweight fills, edge drains, etc.
- The *CBR* values decrease significantly as the amount of tyre buffings (gradation between 1.0 mm and 12.5 mm) increases in the subbase gravel (crushed stone, classified as well-graded gravel, GW). Subbase gravel with 3% cement and 5%

tyre buffings give higher *CBR* values than gravel alone. The use of gravel, tyre buffings and cement can reduce the tyre disposal problems in pavement granular courses (Cabalar and Karabash 2015).

- In pavement applications, the stabilization of cohesionless materials, such as sand and gravel, with discrete fibres requires that some form of surfacing should be used as a wearing course to prevent the fibres from being pulled out of the sand/gravel during trafficking if used as the surface layer of the pavement structure, especially in unsurfaced (unpaved) pavements. Friction forces imparted on the surface of the pavement by the vehicle tyres tend to pull the individual fibres from the sand and gravel over time. As the fibres are withdrawn out of sand and gravel, the reinforcement of the stabilized layer is degraded (Tingle et al. 2002). Resin-modified emulsion, biodegradable emulsion composed of tree sap and water-emulsified resin base are some examples of spray-on materials available commercially under different trade names. Different types of ultraviolet ray-resistant plastic mat panels are also available in the market for use as surfacings.
- The cement-stabilized fibre-reinforced soil may be used to construct the top layers of an embankment if it has to support a pavement or any other structural load.
- The monofilament fibres show a great potential for use in rapid stabilization of sandy soils for pavement base/subbase applications. The field demonstration tests should be carried out to test fibre stabilization performance under actual road/air traffic landing. The field tests should also be conducted to test the durability and maintenance requirements for sand-fibre pavement layers (Santoni and Webster 2001).
- The laboratory investigations carried out by Santoni et al. (2001) indicate the optimum conditions for fibre reinforcement to include the following: a dirty sand with 1–4% silt; the use of 51-mm (2 in.) long fibrillated fibres at the smallest available denier; a fibre content of 0.8% by dry weight; fibres and soil mixed at $\pm$ 2% of optimum water content of the of the composite material.
- In general, the addition of cement significantly increases the strength of soil under both static and dynamic loads, contributes to the volume stability, and increases the resistance to liquefaction in fine sandy and coarse silty soils. The strength of cement-stabilized soil can further be increased significantly by addition of fibres.
- For reinforcing the black cotton soils with glass fibres, the maximum fibre content can be limited to 2% by weight to achieve the optimum benefits (Gosavi et al. 2004).
- Since a potential use of fibre-reinforced cohesive soil is in landfill liners and covers, it is important to assess the effect of specific fibre inclusion on hydraulic conductivity of soil.
- For achieving more tensile strength of fibre-reinforced clayey soil, with a given fibre content, shorter and better dispersed fibres should be used (Maher and Ho 1994).

- PP and glass fibre reinforcements can be used in kaolinite clay beams undergoing flexural load to limit the cracks and increase the toughness (Maher and Ho 1994).
- Fibres can be mixed into soil for making embankment dams and other water-retaining structures more resistant to piping erosion, provided the optimum fibre content is selected based on a suitable piping test. A high fibre content, say greater than 0.15% for PP and PET fibres, may be harmful and may actually reduce the piping resistance (Das and Viswanadham 2010).
- A cluster of fibres in one location within the soil, which may take place with high fibre content, can result in increased permeability, reduced piping resistance and reduced strength.
- Dredging of sediments and dewatering of the dredged sediment/slurry through the geotextile tubes is common engineering tasks in several countries. A variation in slurry particle sizes, particularly fine-grained particles, creates issues with soil loss and dewatering rates within the geotextile tubes. The dewatering rate and final filter cake properties have the most importance to dewatering applications. The faster the flocculated sediments settle and dewater, the quicker the next geotextile tube can be filled. The pressure filtration tests show that the dewatering time decreases significantly with inclusion of 0.5% of both nylon and jute fibres, but is not dependent on the fibre length. However, the increase in shear strength of filter cake is dependent on fibre length. Jute fibres do not show a high strength gain as the nylon fibres but increase the dewatering times by an average of 14–22% compared to filter cakes with nylon fibres. One possible explanation for variations in dewatering rates between nylon and jute fibres can be by the fact that jute fibres have a larger diameter and overall surface area, which allows for more soil-fibre interaction, thereby causing an increase in void volume through which water can pass easily and, therefore, expedite the dewatering rate (Spritzer et al. 2015).
- Fibre reinforcement would be an efficient method in limiting or even preventing the occurrence of the lateral movement of the sandy soils due to liquefaction as normally observed for unreinforced sands (Noorzad and Amini 2014).

Quality Control

- Regular visual inspections should be carried out to control the randomness and both the horizontal and vertical uniformity of the fibre distribution. The actual fibre content at different locations should be checked by suitable methods and compared with the design fibre content.
- While constructing the fibre-reinforced soil structure, the fibres should be introduced in each sub-layer by mixing the amount of fibres with soil as per the designed content.
- The compaction control through the dry unit weight measurement is of little interest for fibre-reinforced soils because fibres do not just have a significantly lower specific gravity than that of the soil particles; they are also a minor physical component of the composite material. It is better to measure the material stiffness for compaction control, which can be done by means of

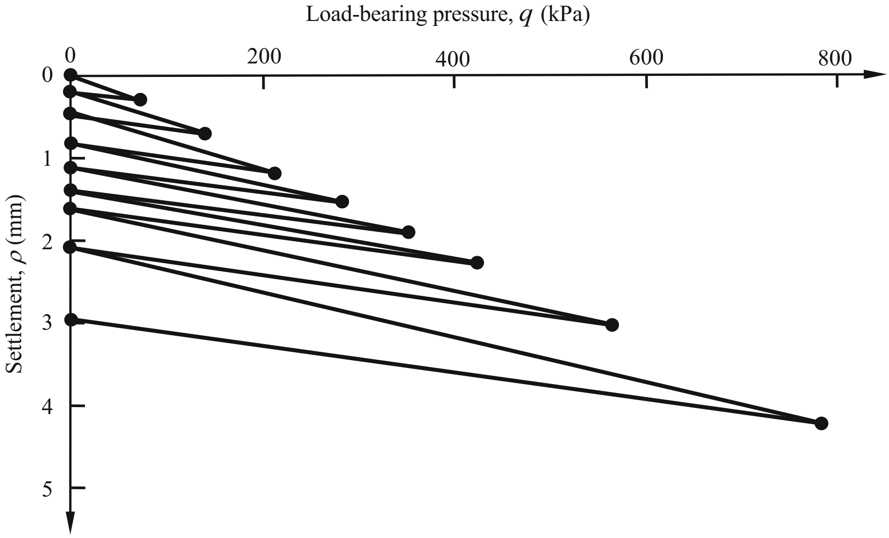

Fig. 5.12 Results of the plate load tests conducted on randomly distributed monofilament PP fibre-reinforced silty sand embankment (50 m long, 10 m wide and 0.6 m high), using a semi-rigid circular plate, 300 mm in diameter, under repeated loading and unloading (Adapted from Falorca et al. 2011)

compaction equipment integrating continuous control compaction (CCC) (Falorca et al. 2011).

- The quality control may be done by conducting the plate load tests, which allow direct characterization of the soil compressibility with the applied pressure, as seen in Fig. 5.12.
- The ratio of porosity n to the volumetric cement content p_{vc}, all expressed as a percentage of the total volume, can be useful in the field control of cement-stabilized fibre-reinforced soil. Based on the unconfined compressive strength tests on specimens submerged for 24 h and achieving an average degree of saturation of 87%, the following relationships have been suggested to estimate the unconfined compressive strength q_u (Consoli et al. 2010):

For cement-stabilized soil,

$$q_u(\mathrm{kPa}) = 2.2 \times 10^7 \left(\frac{n}{p_{vc}^{0.28}}\right)^{-3.08} \quad \left(R^2 = 0.98\right) \tag{5.8}$$

For cement-stabilized fibre-reinforced soil,

$$q_{\mathrm{u}}(\mathrm{kPa}) = 1.0 \times 10^{7}\left(\frac{n}{p_{vc}^{0.28}}\right)^{-2.73} \quad \left(R^2 = 0.95\right) \tag{5.9}$$

Equations (5.8) and (5.9) are relevant for specific materials (nonplastic silty sand, classified as SM, Portland cement of high early strength, monofilament PP fibres with fibre content ranging from 0 to 0.5%, cement content ranging from 0 to 8%) and test conditions, as considered by Consoli et al. (2010) in their experimental study. Similar relationships may be developed for other materials and site conditions for the specific field application. Attempts can also be made to present the generalized relationships. During the quality control, once a poor compaction is identified, it can be readily taken into account in the design, using these equations and adopting corrective measures accordingly, such as the reinforcement of the treated layer or the reduction in the load transmitted.

5.5 Scope of Research

As mentioned earlier, the actual behaviour of fibre-reinforced soils is not yet well known because the current understanding is largely based on small-scale laboratory investigations and very limited large-scale or field studies. Hence, further studies, especially involving field tests and performance evaluation of different applications, are essentially required to better understand the behaviour of fibre-reinforced soils so that the fibre-reinforcements can be routinely utilized based on a more rational analysis and design. In spite of some limitations, reinforcing the soils with discrete flexible fibres can be a cost-effective means of improving their performance in several applications. Additionally the use of fibres from the waste materials can reduce the disposal problem in an economically and environmentally beneficial way. Detailed investigations are expected in the future on several aspects of fibre-reinforced soils, including the following:

- Cost-effective mixing technique
- Optimum size and shape of fibres for different applications
- Durability of fibres in different physical and environmental site conditions
- Drainage and pore water pressure developments within the fibre-reinforced soil
- Effective stress concept for fibre-reinforced soil
- Creep behaviour of fibre-reinforced soil
- Freezing-thawing behaviour of fibre-reinforced soil
- Cyclic loading behaviour of fibre-reinforced soil
- Fibre-clay interface behaviour
- Generalized analysis and design methods for different specific applications of fibre-reinforced soil
- Cost-benefit analysis of different applications of fibre-reinforced soil

It is expected that applications of fibres in mining, agricultural and aquacultural engineering will also be reported significantly in the future, although the applications discussed here are equally applicable for similar projects in these areas.

Note that some wastes, such as the municipal solid wastes, may be physically similar to randomly distributed fibre-reinforced soils, and, therefore, the concepts of fibre-reinforced soil engineering, as presented in this book, are also equally applicable to such wastes to manage their disposal and utilization.

Chapter Summary

1. The technique of reinforcing soils with discrete fibres has several advantages, and hence this has become one of the cost-effective and environmentally friendly ground improvement techniques in the present-day construction practice.
2. There are a large number of applications of fibre-reinforced soils in civil and other related engineering areas. Major applications can be categorized as geotechnical applications, transportation applications, and hydraulic and geoenvironmental applications.
3. Much care is required to obtain a reasonably uniform distribution of fibres within the soil mass, especially when the fibre content is large. Mixing of fibres with soil can be carried out using an oscillatory- or helical-type mixer to avoid fibre segregation, dragging, balling and floating problems, which are associated with commonly used blade-type mixers.
4. Fibre-reinforced soil structures can be designed by composite or discrete approach. The composite approach considers the fibre-reinforced soil a homogeneous material, while the discrete approach uses the contributions of soil and fibres separately in the design.
5. Orientation of fibres must be considered properly in analysis and design of fibre-reinforced soil structures. For field applications, the fibre content and the aspect ratio should be selected based on experimental observations with specific soil and fibres under consideration as these two parameters significantly govern the behaviour of fibre-reinforced soil, and moreover, they can be controlled easily.
6. Addition of fibres to the soil backfill behind a retaining wall increases the stability of the wall with reduced lateral earth pressure and displacement of the wall.
7. Overall stability of a shallow foundation constructed over the weak foundation soil can be significantly improved by placing a compacted cement-stabilized fibre-reinforced soil layer of a suitable thickness (say $0.3B$, where B being the width of the footing) over the weak foundation soil.
8. Benefits of reinforcing the pavement layers with fibres are generally expressed in terms of an extension of service life of the pavement and/or a reduction in the thickness of pavement layer. Traffic benefit ratio (*TBR*) and layer reduction ratio (*LRR*) are two commonly used parameters for design of pavement layers.
9. Fibres can be used effectively in reducing the seepage through the body of water-retaining soil structures, increasing the soil-piping resistance and

controlling the desiccation cracks in compacted clay layers used in liner and cover applications.

10. Some application experience and guidelines are available for successful applications of fibre-reinforced soils. Further studies, especially involving field tests and performance evaluation, are expected to develop the confidence level for different applications.

Questions for Practice

(Select the most appropriate answer to the multiple-choice questions from Q 5.1 to Q 5.5.)

5.1 Tamping and vibration techniques for preparing the fibre-reinforced soil specimens in moist condition lead to

(a) Vertical orientation of fibres
(b) Near-vertical orientation of fibres
(c) Horizontal orientation of fibres
(d) Near-horizontal orientation of fibres

5.2 Fibre inclusion in the pavement base soil

(a) Increases the dry unit weight
(b) Decreases the unconfined compressive strength
(c) Increases the *CBR* value
(d) Both (a) and (c)

5.3 The most suitable roller for compacting fibre-reinforced soils is

(a) Smooth wheel roller
(b) Vibratory rubber-tyred roller
(c) Sheepsfoot roller
(d) Grid roller

5.4 For protecting the synthetic fibres from UV degradation, the entire surface area of fibre-reinforced soil embankment should be covered with a top soil layer having a thickness of about

(a) 50 mm
(b) 100 mm
(c) 150 mm
(d) 200 mm

5.5 The beneficial effects of fibre reinforcement in soil can be enhanced significantly by stabilizing the fibre-reinforced soil with cement in a suitable quantity, say using cement content of

(a) 0.5–1%
(b) 1–5%
(c) 5–10%
(d) 10–20%

5.6 What are the advantages exhibited by the fibre-reinforced soils?

5.7 In the laboratory, prepare a mixture of sand and fibres by any suitable means in dry and wet conditions, and compare your observations about the fibre orientation with those reported by the researchers as presented in this chapter.

5.8 List the potential application areas for fibre reinforcement.

5.9 Describe the techniques for preparing the laboratory test specimens of fibre-reinforced soil.

5.10 What kind of fibre orientations is present in rolled-compacted fibre-reinforced construction fills?

5.11 What considerations are required for the selection of fibres?

5.12 What are the different approaches for design of fibre-reinforced soil structures? Discuss their merits and demerits.

5.13 How can you determine the lateral earth pressure on a retaining wall from fibre-reinforced cohesionless soil backfill?

5.14 For a fibre-reinforced sand backfill supported by a 10-m high retaining wall with a vertical back face, consider the following:

Angle of internal friction of sand, $\phi = 35^\circ$
Total unit weight of fibre-reinforced sand, $\gamma = 16.85\ \mathrm{kN/m^3}$
Fibre-reinforced soil-wall interface friction angle, $\delta = 15^\circ$
Fibre-soil interface friction angle, $\phi_i = 20^\circ$
Fibre aspect ratio, $a_\mathrm{r} = 75$
Volumetric fibre content, $p_\mathrm{vf} = 1.25\%$

Determine the total active earth pressure from the fibre-reinforced sand backfill on the retaining wall, assuming the fibre-reinforced sand behaves as an isotropic material.

5.15 For a fibre-reinforced foundation sand bed supported by a 0.75-m wide surface strip footing, consider the following:

Angle of internal friction of sand, $\phi = 35^\circ$
Total unit weight of fibre-reinforced sand, $\gamma = 16.85\ \mathrm{kN/m^3}$
Fibre-reinforced soil-wall interface friction angle, $\delta = 15^\circ$
Fibre-soil interface friction angle, $\phi_i = 20^\circ$
Fibre aspect ratio, $a_\mathrm{r} = 75$
Volumetric fibre content, $p_\mathrm{vf} = 1.25\%$

Determine load-bearing capacity of the surface strip footing resting over the fibre-reinforced sand bed, assuming the fibre-reinforced sand behaves as an isotropic material.

5.16 How can you control the desiccation cracks in clay liners and covers?

5.17 Define the following terms and explain their practical significance:

(a) Traffic benefit ratio
(b) Layer reduction ratio
(c) Crack intensity ratio
(d) Crack reduction factor

5.18 What considerations are required to design fibre-reinforced pavement layers?
5.19 How can fibres help in using sand in top courses of a pavement?
5.20 Describe a practical method for improving the foundation soil using fibre inclusions.
5.21 What is the effect of fibre inclusions into the soil backfill on the lateral earth pressure against a retaining wall?
5.22 Can the use of fibres reduce the seepage through water-retaining structures? Explain with the help of a neat sketch.
5.23 What is soil piping? What is the effect of fibres on piping resistance of soil?
5.24 Discuss the effect of fibre content on the hydraulic conductivity of medium plasticity soil when it is reinforced with fibres.
5.25 With the help of a neat sketch, discuss the effect of fibre content on crack reduction factor when fibres are included in clay layers?
5.26 Visit a local construction site where the soil is being reinforced with fibres. Based on your observation, write a technical note, and identify the key aspects of the application technique.
5.27 Why should water be added prior to inclusion of fibres into soil for creating the fibre-reinforced soil?
5.28 List some major cares and considerations required for field applications of fibre-reinforced soils.
5.29 What is the most effective method of fibre inclusions in soil? What are the difficulties in mixing fibre and soil in a rotating drum mixer as the cement concrete is prepared?
5.30 What do you mean by optimum mixing moisture content? What is its practical significance and how you can estimate it?
5.31 Describe the procedure for an embankment construction using the fibre-reinforced soil.
5.32 How can you increase the durability of natural fibres for their use as soil reinforcement?
5.33 What are the benefits of inclusion of fibres into dredged sediments while dewatering them through geotextile tubes?
5.34 Discuss the quality control considerations for applications of fibre-reinforced soils.

Answers to Selected Questions

5.1 (d)
5.2 (c)
5.3 (b)
5.4 (a)
5.5 (c)
5.14 166.8 kN/m
5.15 856.4 kPa

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Index

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