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 ($p_f = 0.5\%$), $\gamma_{dmax} = 11.06$ 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

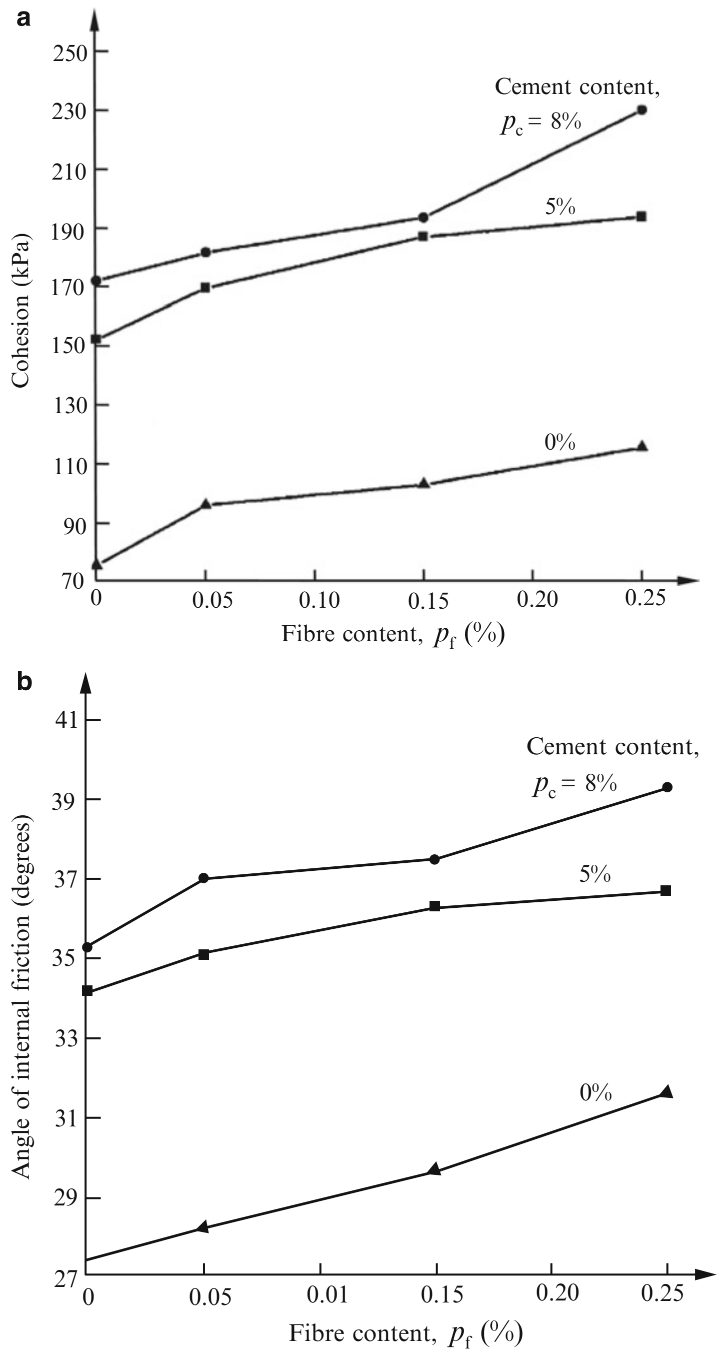

Fig. 3.1 Variation of shear strength parameters with fibre content p_f for uncemented and cemented clayey soils after 28 days of curing: (**a**) cohesion c; (**b**) angle of internal friction ϕ (Adapted from Tang et al. 2007b)

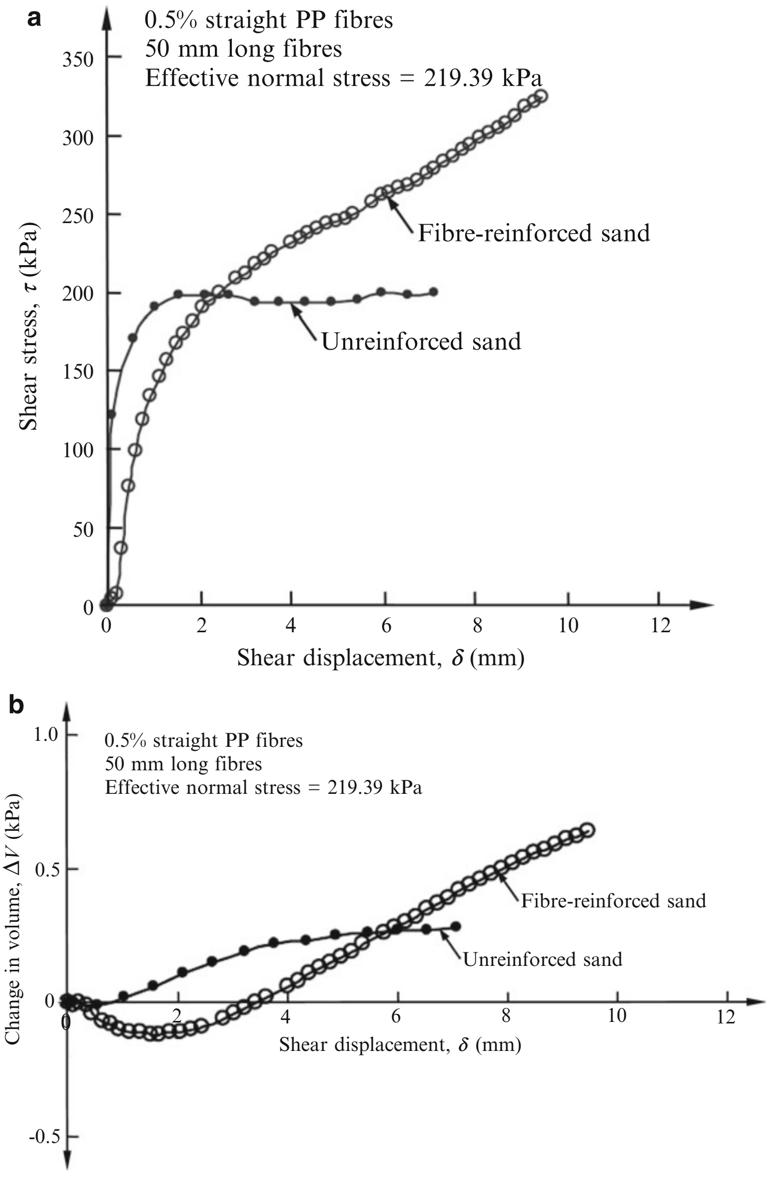

Fig. 3.2 Direct shear test results for fibre-reinforced sand: (**a**) shear stress τ versus shear displacement δ; (**b**) change in volume of the specimen ΔV versus shear displacement δ (Adapted from Falorca and Pinto 2011)

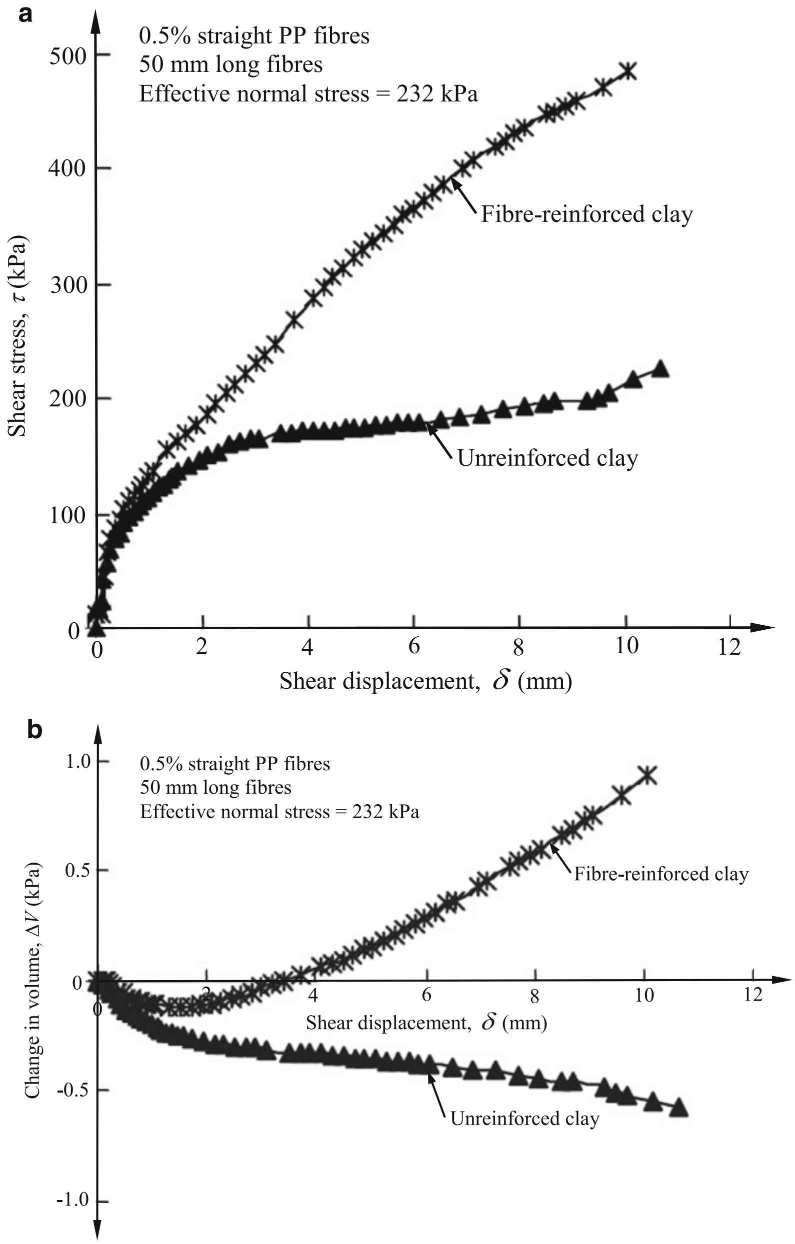

Fig. 3.3 Direct shear test results for fibre-reinforced clay: (**a**) shear stress τ versus shear displacement δ; (**b**) change in volume of the specimen ΔV versus shear displacement δ (Adapted from Falorca and Pinto 2011)

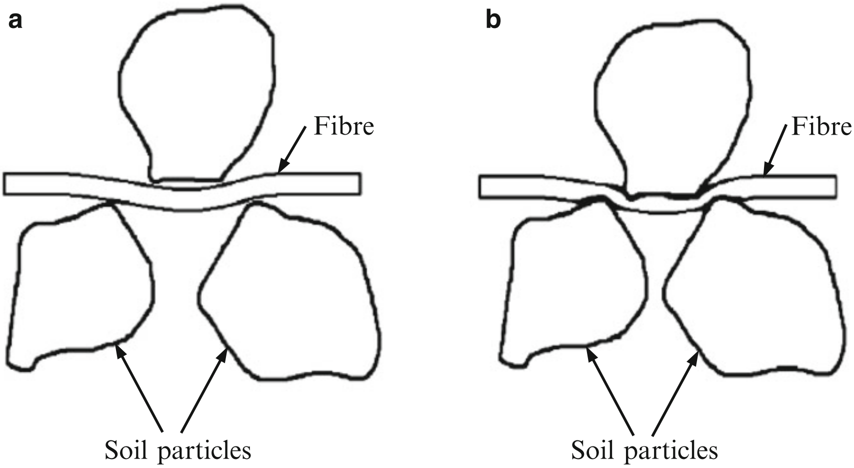

Fig. 3.4 Interaction mechanisms between fibre and soil particles: (**a**) soil particles being prevented by fibre from packing tightly; (**b**) soil particles causing fibre stretch and imprints on the fibre, thus allowing adhesion to develop (Adapted from Falorca and Pinto 2011)

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

$$I_{\mathrm{d}} = \frac{\Delta(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_{\mathrm{fU}}} = \frac{(\sigma_1 - \sigma_3)_{\mathrm{R}} - (\sigma_1 - \sigma_3)_{\mathrm{fU}}}{(\sigma_1 - \sigma_3)_{\mathrm{fU}}} = \frac{(\sigma_1 - \sigma_3)_{\mathrm{R}}}{(\sigma_1 - \sigma_3)_{\mathrm{fU}}} - 1 \qquad (3.3)$$

where $(\sigma_1 - \sigma_3)_{\mathrm{fU}}$ is the failure deviator stress of the unreinforced soil specimen, and $(\sigma_1 - \sigma_3)_{\mathrm{R}}$ is the deviator stress of the fibre-reinforced soil specimen corresponding to the axial strain $\varepsilon(=\varepsilon_{\mathrm{fU}})$ 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), $(\sigma_1 - \sigma_3)_{\mathrm{R}}$ 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 $(\sigma_1 - \sigma_3)_{\mathrm{R}} = (\sigma_1 - \sigma_3)_{\mathrm{fR}}$ at $\varepsilon = \varepsilon_{\mathrm{fR}}$ (axial strain of reinforced soil specimen at failure), Eq. (3.3) reduces to the *deviator stress improvement factor at failure* I_{df}, given as

$$I_{\mathrm{df}} = \frac{\Delta(\sigma_1 - \sigma_3)_{\mathrm{f}}}{(\sigma_1 - \sigma_3)_{\mathrm{fU}}} = \frac{(\sigma_1 - \sigma_3)_{\mathrm{fR}} - (\sigma_1 - \sigma_3)_{\mathrm{fU}}}{(\sigma_1 - \sigma_3)_{\mathrm{fU}}} = \frac{(\sigma_1 - \sigma_3)_{\mathrm{fR}}}{(\sigma_1 - \sigma_3)_{\mathrm{fU}}} - 1 \qquad (3.4)$$

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:

$$DSR = \frac{(\sigma_1 - \sigma_3)_{\mathrm{R}}}{(\sigma_1 - \sigma_3)_{\mathrm{fU}}} \qquad (3.5)$$

$$DSR_{\mathrm{f}} = \frac{(\sigma_1 - \sigma_3)_{\mathrm{fR}}}{(\sigma_1 - \sigma_3)_{\mathrm{fU}}} \qquad (3.6)$$

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):

$$I_{\mathrm{b}} = \frac{(\sigma_1 - \sigma_3)_{\mathrm{peak}} - (\sigma_1 - \sigma_3)_{\mathrm{ultimate}}}{(\sigma_1 - \sigma_3)_{\mathrm{ultimate}}} = \frac{(\sigma_1 - \sigma_3)_{\mathrm{peak}}}{(\sigma_1 - \sigma_3)_{\mathrm{ultimate}}} - 1 \qquad (3.7)$$

where $(\sigma_1 - \sigma_3)_{\mathrm{peak}}$ and $(\sigma_1 - \sigma_3)_{\mathrm{ultimate}}$ 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 $\sigma_{3\text{crit}}$).

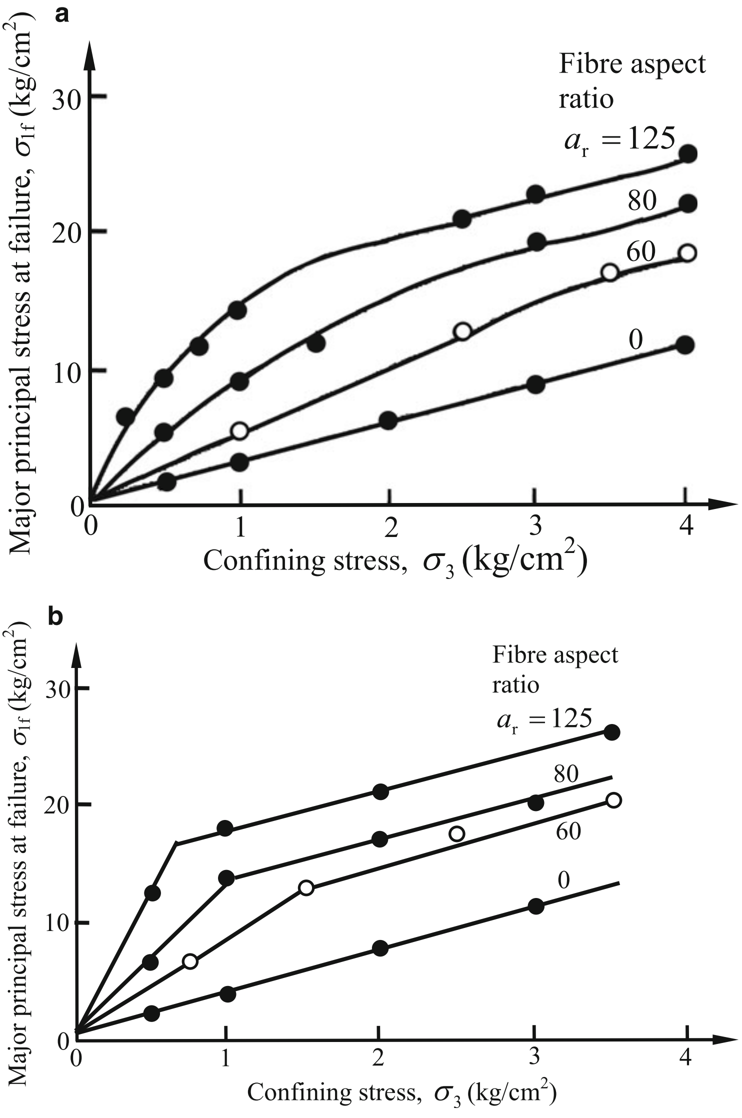

Fig. 3.5 Principal stress envelopes from triaxial compression tests on reinforced sands: (**a**) Muskegon dune sand; (**b**) mortar sand (Adapted from Maher and Gray 1990). (Note: Glass fibre content = 3%; 1 kg/cm^2 = 98.07 kPa)

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.

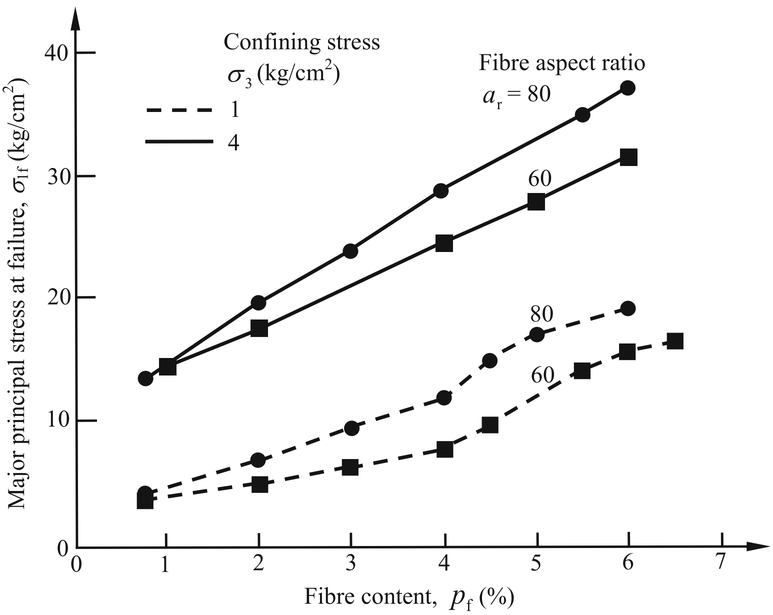

Fig. 3.6 Effect of glass fibre content and aspect ratio on strength increase in Muskegon sand at *low* and *high* confining stresses (Adapted from Maher and Gray 1990). (Note: 1 kg/cm^2 = 98.07 kPa)

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

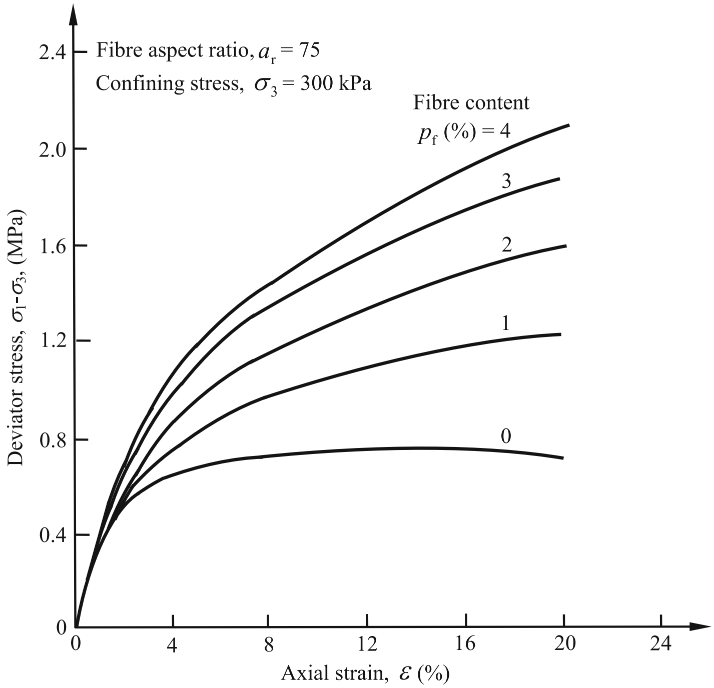

Fig. 3.7 Stress-strain behaviour of plastic fibre-reinforced fine sand (Adapted from Ranjan et al. 1994, 1996)

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

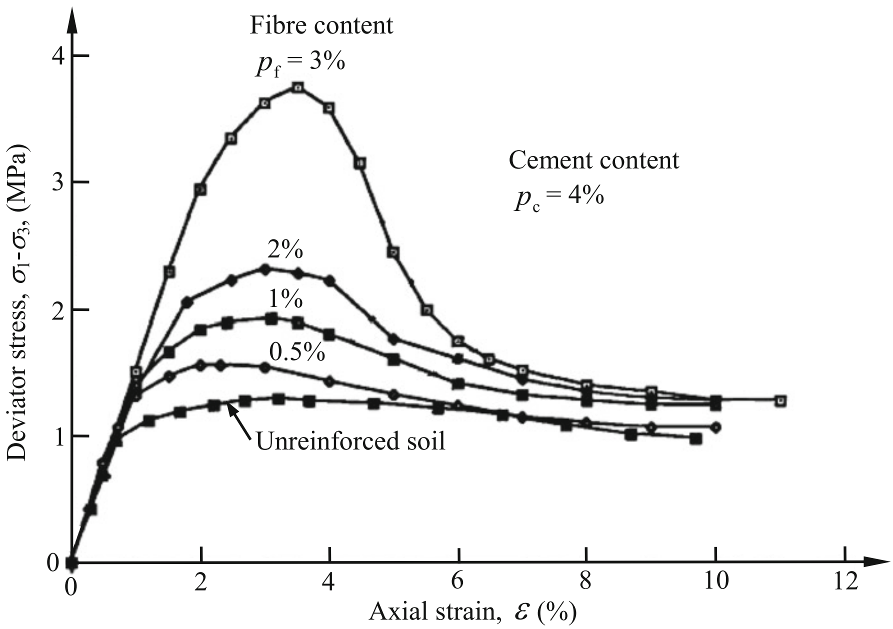

Fig. 3.8 Effect of glass fibre inclusion on stress-strain behaviour of cement-stabilized standard Ottawa sand after 28 days of curing (Adapted from Maher and Ho 1993)

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:

$$E_i = kp_a\left(\frac{\sigma_3}{p_a}\right)^n \tag{3.8}$$

   where k is the intercept at $\sigma_3/p_a = 1.0$ and n is the slope of the line of variation of E_i/p_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% ($p_{vf} = 1.25\%$, $a_r = 40$) 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 $\log I_{df}$ 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.

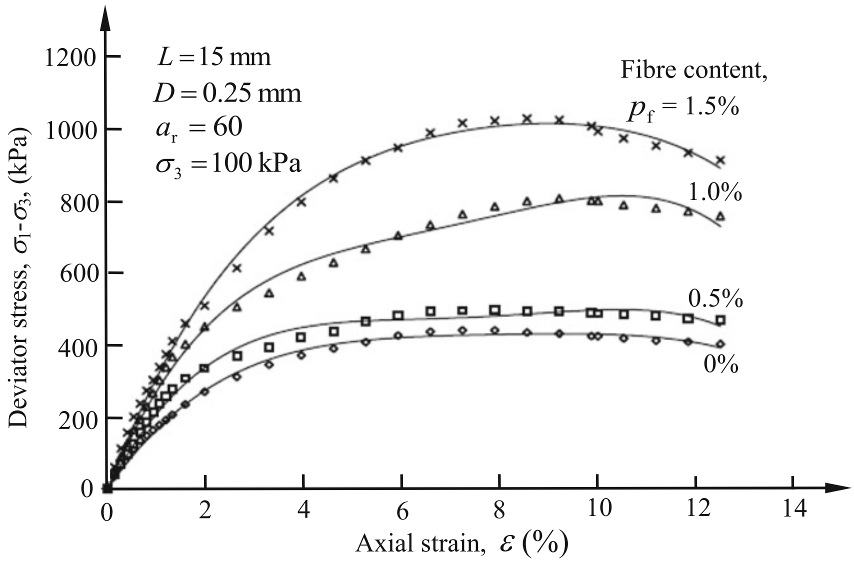

Fig. 3.9 Stress versus strain curves for coir fibre-reinforced sand (Adapted from Babu et al. 2008a)

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

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.

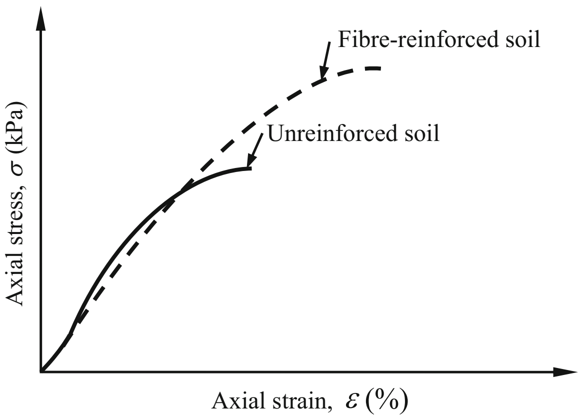

Fig. 3.10 Typical unconfined compression stress-strain curves for unreinforced and fibre-reinforced soils (Adapted from Freitag 1986).

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), $p_f = 1\%$, 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:

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

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).

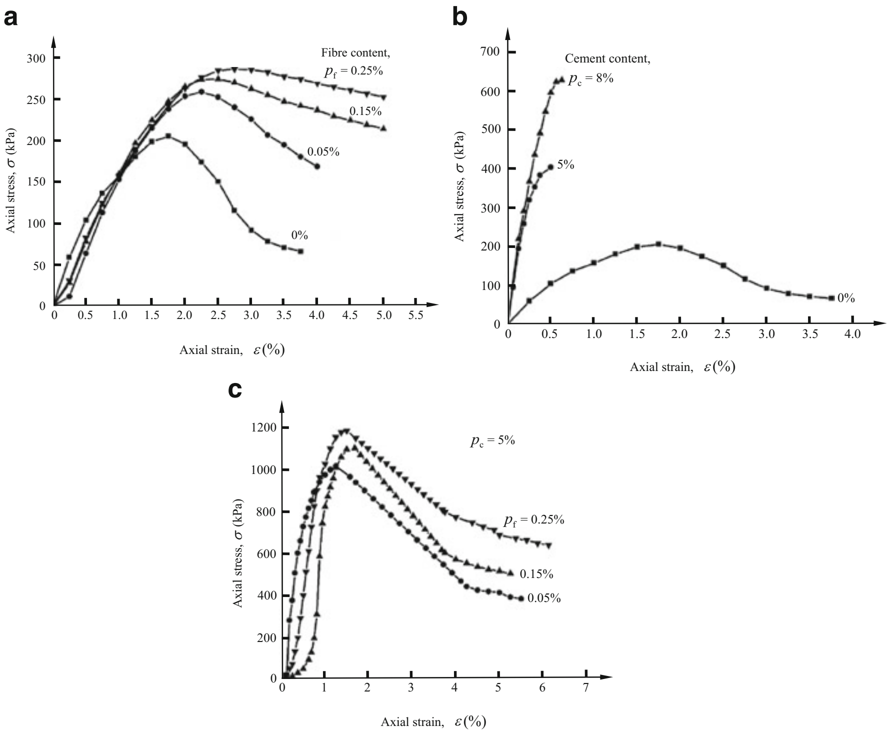

Fig. 3.11 Stress-strain curves obtained from unconfined compression tests: (**a**) fibre-reinforced uncemented clayey soil; (**b**) cement-stabilized clayey soil after 28 days of curing; (**c**) fibre-reinforced cemented clayey soil with cement content $p_c = 5\%$ after 28 days of curing (Adapted from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

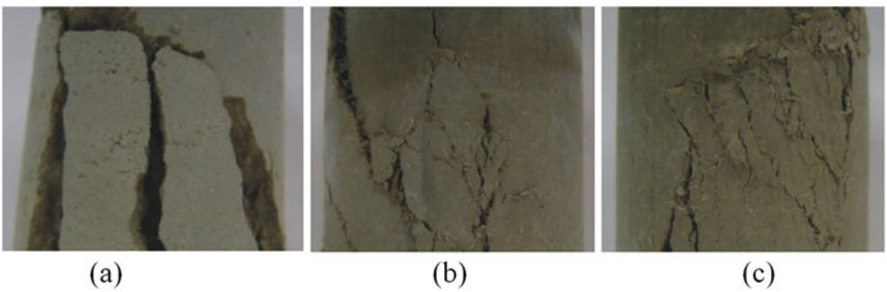

Fig. 3.12 Effect of fibres on the failure pattern of cement-stabilized clayey soil in unconfined compression tests with cement content, $p_c = 8\%$: (**a**) fibre content, $p_f = 0\%$; (**b**) $p_f = 0.05\%$; (**c**) $p_f = 0.25\%$ (After Tang et al. 2007b)

(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

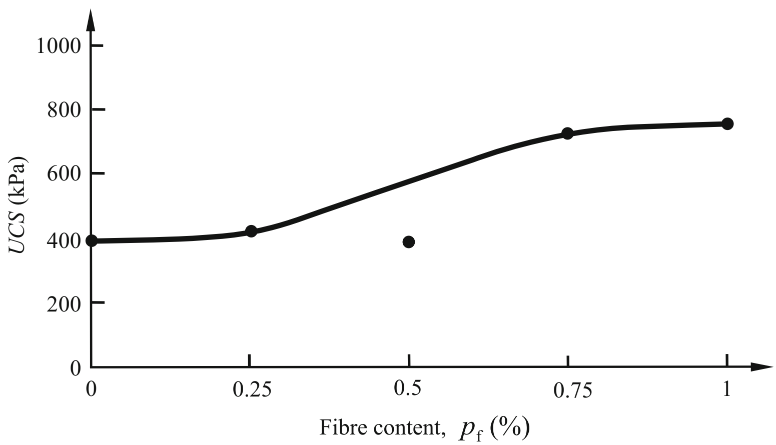

Fig. 3.13 Effect of fibre content on unconfined compressive strength of fine-grained soil (Adapted from Zaimoglu and Yetimoglu 2012)

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), $p_f = 1\%$, 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 ($p_f = 0.5\%$), $\gamma_{dmax} = 11.06$ kN/m^3 and $w_{opt} = 33.1\%$

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).

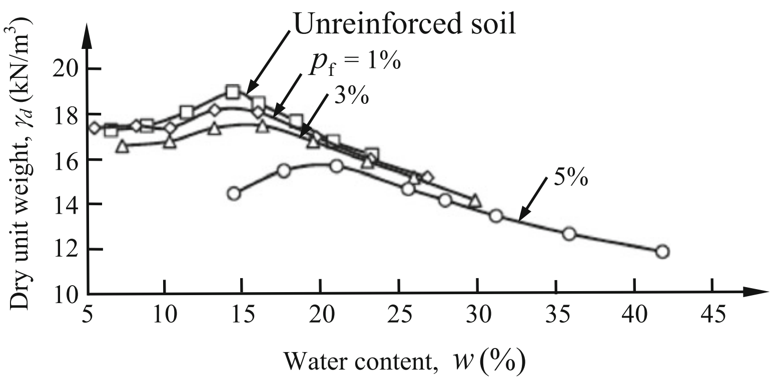

Fig. 3.15 Effect of fibre content on compaction curves for carpet waste fibre-reinforced clay soil (a combination of 90% natural clay (CL as per unified soil classification system) from the Northwest region of UK and 10% sodium activated bentonite) (Adapted from Mirzababaei et al. 2013)

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

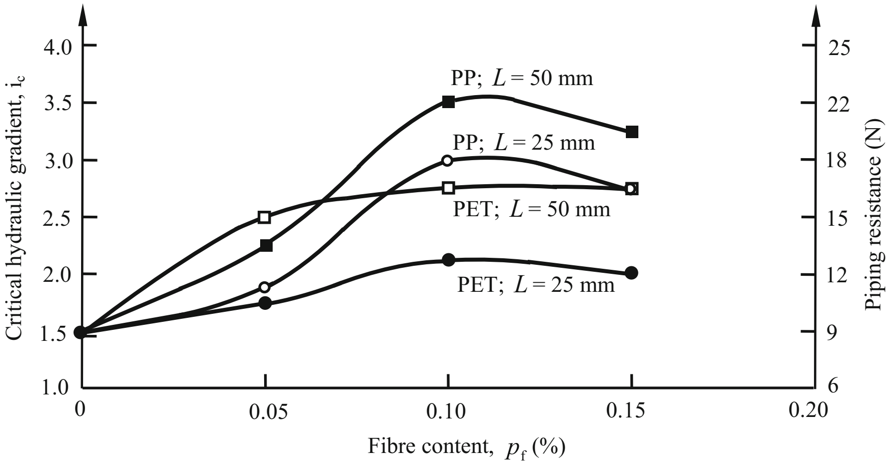

Fig. 3.16 Variation of critical hydraulic gradient and piping resistance of soil with fibre content (Adapted from Das and Viswanadham 2010)

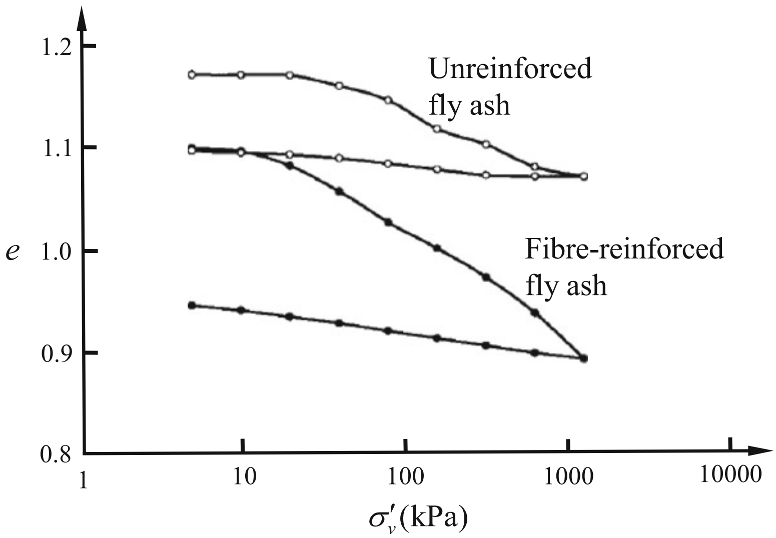

Fig. 3.17 $e - \log_{10}\sigma'_v$ curves for unreinforced and fibre-reinforced fly ash specimens (Adapted from Kaniraj and Havanagi 2001)

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).

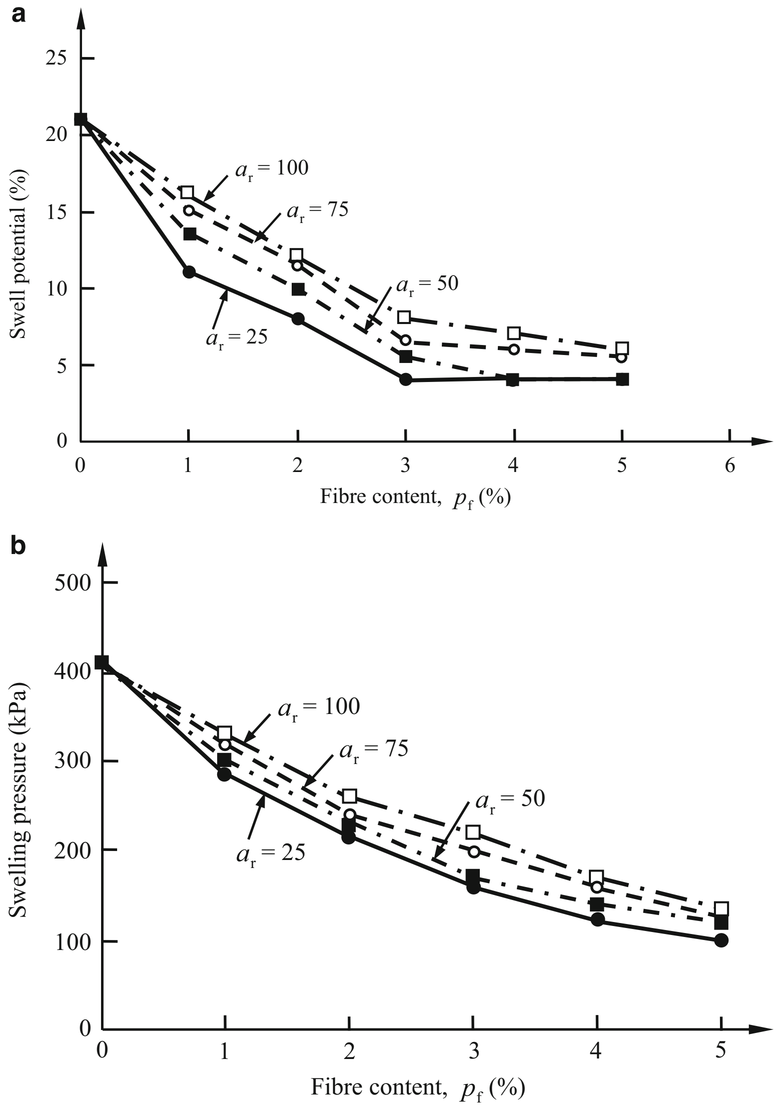

Fig. 3.18 Effect of nylon fibres on: (**a**) swell potential; (**b**) swelling pressure of clayey soil (classified as CH with liquid limit of 78% and plasticity index of 43%) (Adapted from Al-Akhras et al. 2008)

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).

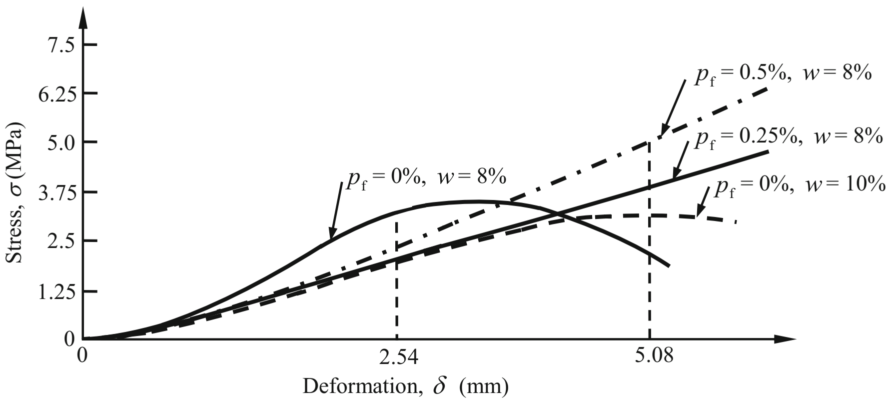

Fig. 3.19 *CBR* test results on wet sand with and without fibres (water content, w = 8–10%) (Adapted from Lindh and Eriksson 1990)

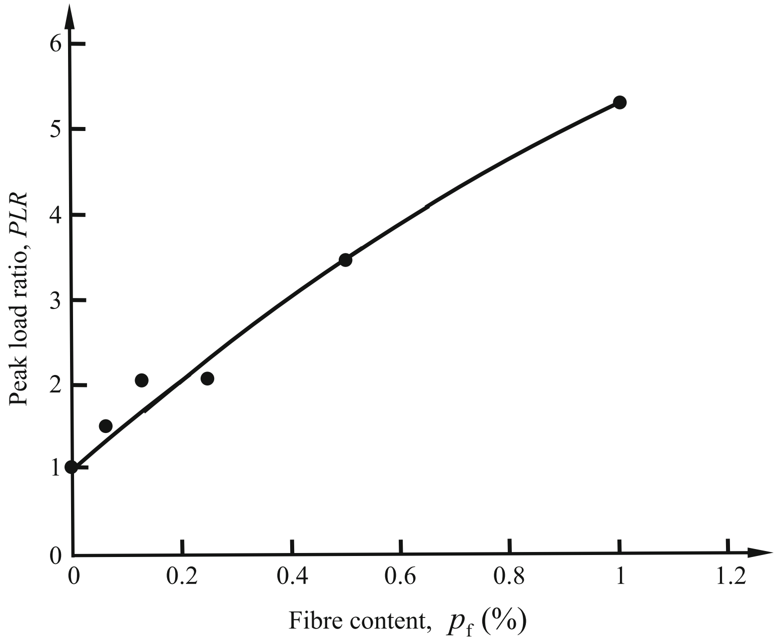

Fig. 3.20 Variation of peak load ratio (PLR) with fibre content (p_f) for fibre-reinforced sand (Adapted from Yetimoglu et al. 2005)

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

$$CBR = a \ln p_f + b \quad (3.18)$$

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%).

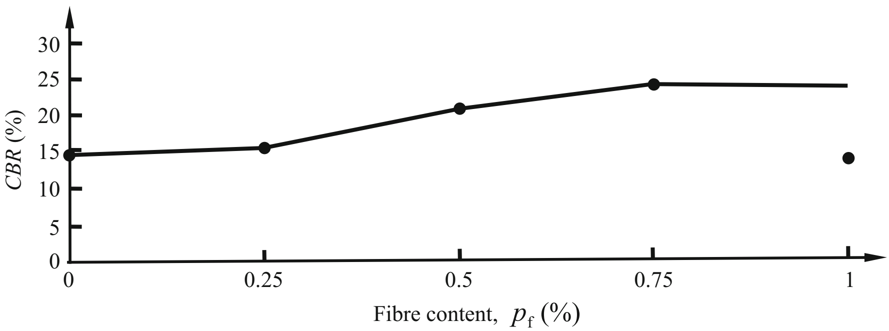

Fig. 3.21 Effect of fibre content on the soaked *CBR* behaviour of fine-grained soil (Adapted from Zaimoglu and Yetimoglu et al. 2012)

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}.

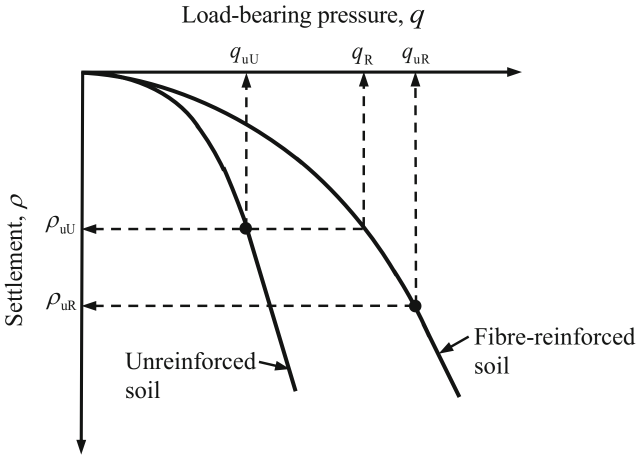

Fig. 3.22 Typical load-settlement curves for unreinforced and fibre-reinforced soils

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 $q_R = q_{uR}$ at $\rho = \rho_{uR}$, Eq. (3.19) reduces to *ultimate bearing capacity ratio* as

$$BCR_u = \frac{q_{uR}}{q_{uU}} \tag{3.20}$$

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

$$I_{UBC} = \frac{\Delta q_u}{q_{uU}} = \frac{q_{uR} - q_{uU}}{q_{uU}} = \frac{q_{uR}}{q_{uU}} - 1 = BCR_u - 1 \tag{3.21}$$

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

$$D_i = \frac{\rho_{uR}}{\rho_{uU}} \tag{3.22}$$

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 $\times$ 150 mm) in a square test tank of size 1 m $\times$ 1 m $\times$ 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.

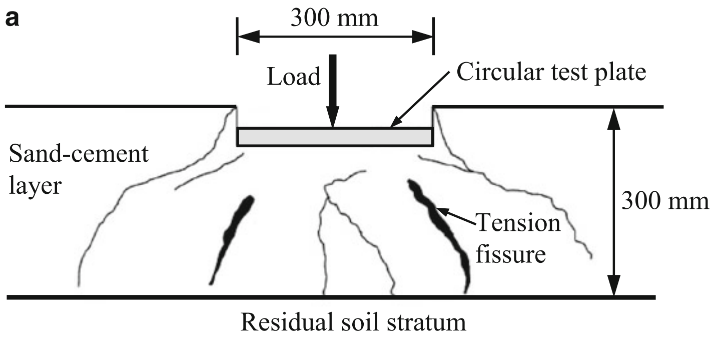

Fig. 3.23 Failure modes for treated soil layer overlying the residual soil stratum: (**a**) sand-cement layer; (**b**) sand-cement-fibre layer (Adapted from Consoli et al. 2003b)

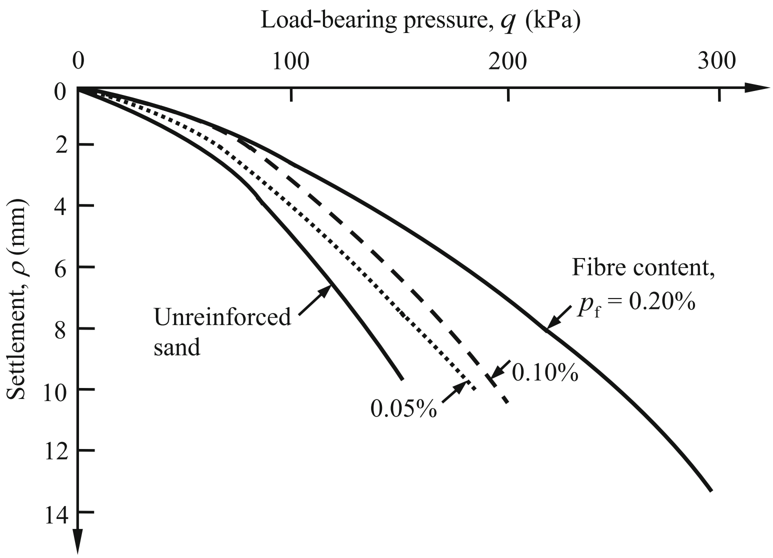

Fig. 3.24 Load-settlement curves for PP fibre-reinforced poorly graded sand at different fibre contents (Adapted from Gupta et al. 2006)

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).

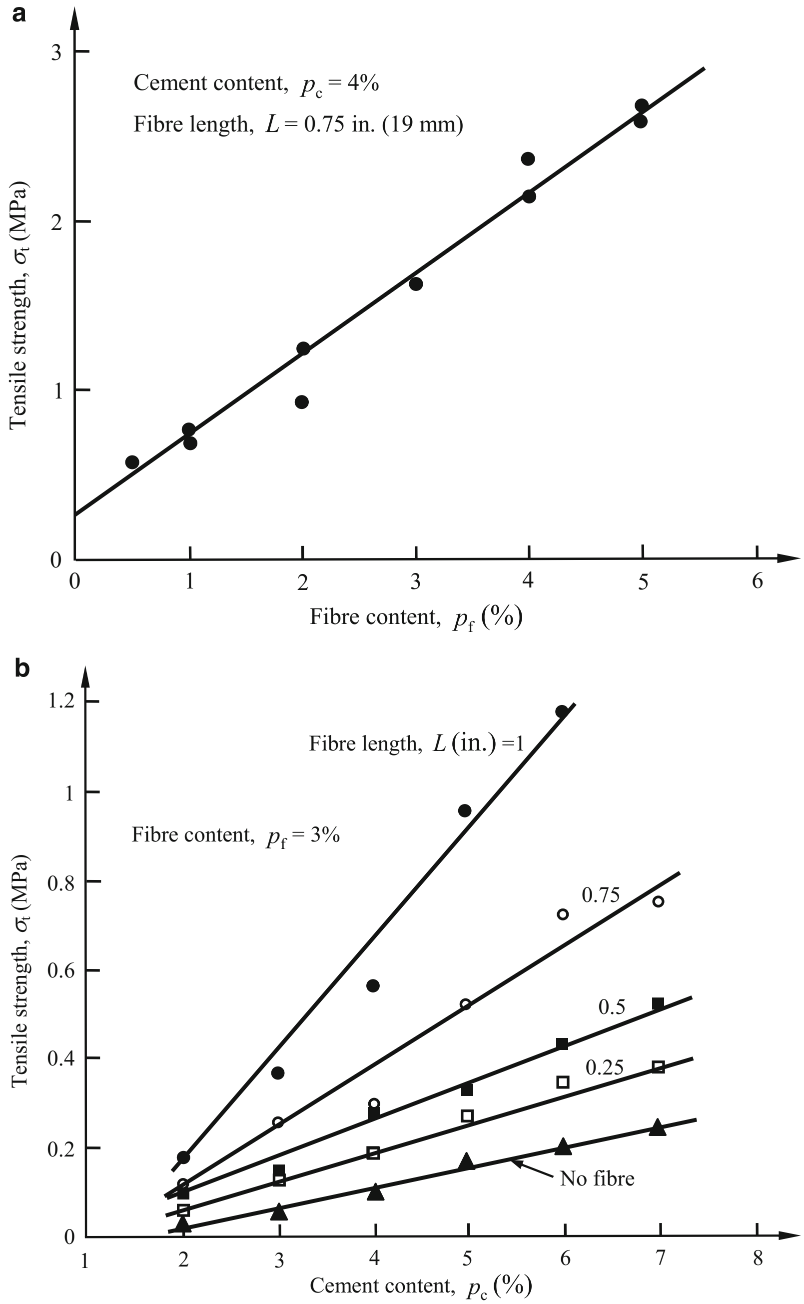

Fig. 3.25 Effect of glass fibre inclusion on the tensile strength of cement-stabilized standard Ottawa sand: (**a**) effect of fibre content; (**b**) effect of fibre length and cement content (Adapted from Maher and Ho 1993)

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

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