

Effect of a Non-Woven Geotextile Layer on the Permeability and Unconfined Compressive Strength of a Soft High-Plasticity Road Subgrade: A Laboratory Case Study

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Abstract

Soft, fine-grained road subgrades are both weak and slow to drain. This laboratory case study examined whether a single non-woven geotextile layer changes the permeability and the unconfined compressive strength of a soft subgrade from the Gresik–Lamongan national road (STA 27+250 to STA 32+550, East Java). The soil is a high-plasticity fine-grained soil plotting below the A-line (LL = 68.4%, PI = 24.0%, MH/OH) with a void ratio of 1.49. Remoulded specimens from one bulk sample were tested with and without a geotextile disc at mid-height. In falling-head tests on five technical replicates per condition, the mean permeability increased by 24.6% (ratio of means 1.25, 95% CI 1.21–1.29). An analysis of the head-decay records indicates that the recorded times were most likely cumulative; this changes the absolute k values by about 45% but leaves the treatment ratio almost unchanged (1.25–1.28). In one unconfined compression test per condition, the largest recorded stress increased from 0.379 to 0.489 kg/cm² (+29.0%) and the secant modulus E_{50} by 34%, with a nearly constant stress ratio of 1.28 between the two curves. Because the strength result rests on a single specimen per condition, it is indicative and must be confirmed by replicated, saturated tests. The results suggest that a non-woven geotextile gives a modest but repeatable improvement in the drainage and strength of this soil, and that its main value in pavements lies in drainage, separation and confinement rather than as a substitute for chemical stabilisation.

Keywords: Non-woven geotextile, Soft subgrade, Unconfined compressive strength, Falling-head permeability, Soil–geotextile interface, Pavement foundation

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INTRODUCTION

Many national road sections on the low-lying north coast of East Java are built on soft, fine-grained subgrades, and every pavement ultimately depends on the subgrade beneath it. The subgrade is the natural soil that receives the traffic load after it has been spread by the pavement layers above. When the subgrade is weak or moisture-sensitive, the pavement fails early, even if the upper layers were built well. A pavement-defect study on expansive subgrades showed that weak, swelling subgrades are a major cause of road defects, and that raising the subgrade CBR while lowering its swell reduced rutting, fatigue and permanent deformation of the pavement [1]. A recent review of expansive pavement subgrades reached the same conclusion and listed many stabilisation routes that are now used to make such soils usable [2]. Very wet, fine-grained materials such as dredged sludge can also be turned into road subgrade, but only after treatment raises their strength and lowers their water content [3]. Similar soft, wet, fine-grained soils that are both weak and slow to drain are common in the coastal lowlands of East Java, where clayey subgrades of the Tuban–Babat–Lamongan–Gresik national road have been treated by grouting [4] and soft clay on the Tuban ring road required a geotextile separator [5], and the Indonesian pavement design manual requires such subgrades to be treated, for example with a geotextile separator and a capping layer, before the pavement is placed [6].

These two weaknesses are not separate. The stiffness of a subgrade depends strongly on its moisture state: pavement analyses that include suction show that a wetter, lower-suction subgrade has a lower resilient modulus and gives a weaker pavement response [7]. Under repeated traffic and wet–dry cycles, silty clay subgrades also lose strength progressively [8]. The reason lies in pore pressure. When a fine-grained subgrade is loaded cyclically, excess pore-water pressure accumulates faster at higher stress levels and dissipates only during rest periods [9]. Because the soil skeleton carries only the effective stress, this pore pressure lowers the shear strength. How fast pore water can leave depends on permeability, which in fine-grained soils is controlled by their physical properties; data-driven models now estimate the saturated permeability of such soils from their index properties [10], and simpler linear expressions have long been used for the same purpose [11]. Compaction energy also changes the permeability of compacted clayey silt [12]. In short, a soft subgrade is a coupled problem of strength and drainage, and a good remedy should address both.

Fine-grained soils with a high liquid limit hold a lot of water and change volume when their water content changes. When such a soil is loaded repeatedly by traffic, it deforms plastically, ruts form, and fine particles can pump up into the granular layer above [13]. Expansive and soft subgrades therefore drive pavement defects unless their strength is raised and their swelling is controlled [1], [2]. Two practical consequences follow. The pavement must spread load over a larger area to keep the subgrade stress low, and the subgrade must be kept from becoming wetter over time [14]. Both consequences link strength and drainage.

Engineers have three broad families of remedies. Chemical stabilisers such as lime and cement build new bonds between particles [1], [15]. Granular, fibre and waste additives change the structure of the soil [2], [16]. Geosynthetics, and especially geotextiles, add a separate engineered layer that can separate, filter, drain and reinforce [17]. Non-woven geotextiles are made of randomly arranged polymer

fibres bonded by needle-punching or heat. Their open, felt-like structure lets water pass both across and along the sheet while holding back soil particles, although their drainage capacity falls when they are compressed [18]. In pavements, a geotextile placed between subgrade and subbase reduces the migration of subgrade fines into the subbase [13], and subsurface drainage remains one of the main tools for controlling subgrade moisture [14]. Laboratory tests on unpaved road models over soft clay showed that woven coir geotextiles improve the load response of the road [19], and lime-treated coir geotextile mats reduced the swelling pressure of an expansive subgrade by 52.19% in a single layer and 81.89% in a double layer [20]. Finite-element studies of embankments on soft soil showed that geotextiles raise the safety factor [21], although the extra gain becomes small beyond about three reinforcement layers [22]. Earlier laboratory work on reinforced clay showed that inclusions able to drain, such as non-woven geotextiles or thin sand layers, increase the strength of saturated clay mainly by allowing excess pore pressure to dissipate at the inclusion [23], [24], [25].

Yet the published evidence leaves three gaps. First, most studies measure either a mechanical gain or a hydraulic gain, but rarely both on the same soil, so the coupling between them is assumed rather than shown. Second, results on soil–geosynthetic interfaces seem to conflict: rough interfaces can be stronger than the soil itself, while smooth or wet interfaces can be weaker [26], suction raises the strength of a soil–geotextile interface [27], and repeated wetting, drying, freezing and thawing can change the interface peak strength by tens of percent [28]. Third, many reports state that an improvement is “significant” without a formal test, without checking model assumptions, and without asking whether the laboratory data contain influential points. The present study addresses the first gap directly by testing the same soil for both properties and the third by reporting all statistical checks explicitly; the second gap is not tested here, because no interface shear tests were made, and is addressed only through the published interface data in the discussion.

This paper addresses these gaps with primary laboratory data from a soft subgrade on the Gresik–Lamongan national road in East Java. The study has four aims: (i) to measure the hydraulic and mechanical response of the same soil with and without a non-woven geotextile; (ii) to test these gains with a full set of statistical checks, including regression diagnostics and outlier screening; (iii) to explain the size of the gains with simple, well-established relations from soil mechanics and geosynthetics; and (iv) to place the results within the wider body of local and international evidence. The original contribution of the paper is a paired hydraulic and mechanical data set for one Indonesian soft subgrade, analysed with explicit statistical checks. Because all specimens come from one bulk sample, the results are presented as a laboratory case study that supports, but does not prove, the mechanisms discussed, and they are not intended as design values.

METHOD

1. Governing Relations

a. Seepage, effective stress and falling-head permeability

Water flow through soil is described by Darcy’s law, which states that the discharge velocity is proportional to the hydraulic gradient [11], [29]:

$$v = k \cdot i \quad (1)$$

where v is the discharge velocity, k is the coefficient of permeability and i is the hydraulic gradient (head loss per unit length of flow). Darcy's law is the basis of permeability testing of fine-grained soils [29]. For a soft subgrade, the practical meaning is simple. A soil with a low k needs a high gradient, and therefore a high pore-pressure difference, to move the same amount of water.

For fine-grained soil, k is measured in the laboratory with a falling-head arrangement because the flow is too slow for a constant-head test [29]. Water in a thin standpipe of area a drains through a specimen of length L and area A , and the head falls from h_1 to h_2 in time Δt :

$$k = 2.303 \cdot (a \cdot L) / (A \cdot \Delta t) \cdot \log_{10}(h_1/h_2) \quad (2)$$

Equation (2) assumes that k is constant during the test, that the specimen is saturated, and that the flow obeys Darcy's law [29]. When a thin geotextile layer of thickness t_g and permeability k_g lies across the flow path inside a soil specimen of total length L , the two materials act in series, and the equivalent permeability normal to the layers is [30]:

$$k_{eq} = L / [(L - t_g)/k_s + t_g/k_g] \quad (3)$$

where k_s is the soil permeability. Equation (3) will be used in part 4a of the Result and Discussion to check whether the measured gain can be explained by the geotextile acting as a simple series element. Pore pressure matters because the soil skeleton carries only the effective stress [9]:

$$\sigma' = \sigma - u \quad (4)$$

in which σ is the total stress and u is the pore-water pressure. Any measure that lets excess pore pressure dissipate faster, such as a drainage path, raises σ' and therefore raises the frictional strength that the soil can mobilise; conversely, the accumulation of excess pore pressure under cyclic loading is what weakens a saturated subgrade [9].

b. Unconfined strength and undrained shear strength

The unconfined compression test loads a cylindrical specimen axially with no lateral support until it fails [31]. The axial stress is calculated on a cross-section that grows as the specimen shortens, and the undrained shear strength of a saturated clay is taken as half of the unconfined compressive strength [15], [31]:

$$q_u = P_{max} / A', \quad A' = A_0 / (1 - \epsilon), \quad c_u = q_u / 2 \quad (5)$$

where P_{max} is the peak axial load, A_0 is the initial area, ϵ is the axial strain and c_u is the undrained shear strength. The value of q_u is widely used to judge the effect of soil improvement, because it is quick to measure and directly linked to bearing capacity [15], [32]. On the usual consistency scale, a q_u of 25–50 kPa marks a soft clay and 50–100 kPa a medium clay.

c. Soil-geotextile interface and bearing mechanisms

A geotextile can only reinforce soil if shear stress can pass across the interface. This is described by an interface friction angle δ , which is compared with the soil friction angle ϕ through the interface ratio [17], [33]:

$$R_{int} = \tan \delta / \tan \phi \quad (6)$$

A ratio near or above 1.0 means that the interface is as strong as the soil itself, so slip will occur within the soil rather than along the sheet. This tangent form is the usual definition; the simpler ratio δ/ϕ differs from it by only a few percent for the small angles of soft soils [17]. Interface friction was not measured in this study, so Eq. (6) is used only qualitatively in part 4b of the Result and Discussion. Interface behaviour is controlled by the zone of soil a few grain diameters thick next to the sheet. In this shear band, fibres of a non-woven geotextile can interlock with soil grains, while water can be drawn into the sheet, so the local water content, suction and strength may differ from those of the bulk soil [17], [27].

For roads, the best-known link between subgrade strength and geotextile reinforcement is the bearing-capacity approach first proposed by Giroud and Noiray [34], later extended by Giroud and Han [35], and used in later design and analysis of geotextile-reinforced unpaved roads [36], [37]. The allowable pressure on the subgrade is:

$$p = N_c \cdot c_u, \quad N_c = \pi \text{ (unreinforced)}, \quad N_c = \pi + 2 \text{ (geotextile-reinforced)} \quad (7)$$

Without reinforcement, only the elastic limit of the subgrade can be used ($N_c = \pi \approx 3.14$). With a geotextile, the sheet restrains lateral movement and confines the soil, so the full plastic bearing capacity can be mobilised ($N_c = \pi + 2 \approx 5.14$) [36], [37]. Equation (7) is therefore a way to translate a laboratory change in c_u , together with a change in mechanism, into a change in subgrade capacity.

2. Site, Sampling and Geotextile

Soil was taken from the subgrade of the Gresik–Lamongan national road between STA 27+250 (in front of Duduksampeyan Market) and STA 32+550 (Jalan Deket alley), Gresik Regency, East Java (Figure 1). This segment was chosen because its subgrade is known to drain poorly and it carries heavy, regional traffic. The soil was dug from the exposed subgrade with hoes and spades, packed into steel tubes, and sealed with plastic at both ends to keep its moisture during transport to the Soil Mechanics Laboratory of University of Widya Gama Malang. The sampling therefore produced disturbed (remoulded) material, which was later formed into test specimens. Because one bulk sample was used for all tests, the results characterise this sample and not the variability of the subgrade along the 5.3 km segment.

Specimen preparation. The bulk soil was compacted statically into the moulds to a target wet unit weight of 1.58 g/cm^3 (the value in Table 1) at the natural water content of 48.1%. Treated specimens were formed in two halves with the geotextile disc between them.

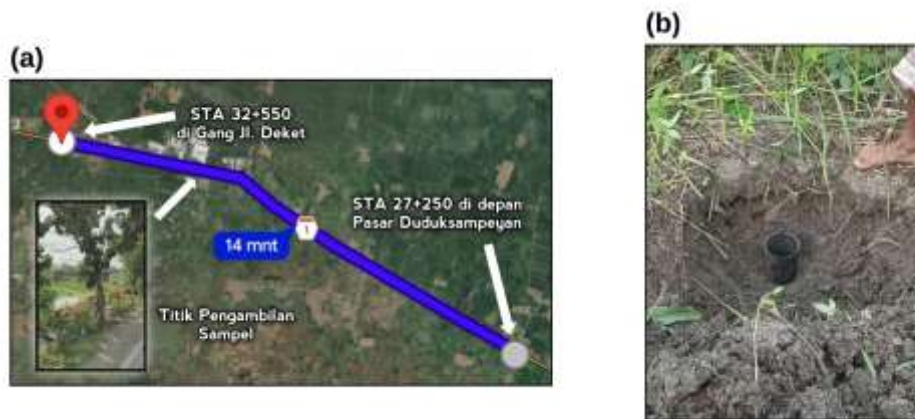


Figure 1. Study site and sampling: (a) Gresik–Lamongan national road segment STA 27+250 to STA 32+550 with the sampling point; (b) steel sampling tube in the exposed subgrade (author documentation).

A commercial non-woven polymer geotextile was used as the treatment (Figure 2b). A woven geotextile is shown for comparison (Figure 2a): its regular weave gives high tensile stiffness but a smoother surface and, being non-wicking, it can even act as a barrier to downward moisture movement in silty soils [38], while the random fibres of the non-woven sheet give a rough, compressible, highly permeable layer [18]. The permeability of the geotextile sheet was 0.25×10^{-3} cm/s (2.5×10^{-3} mm/s), about three orders of magnitude higher than that of the soil. It is lower than the cross-plane permeability typical of new needle-punched non-woven geotextiles (of the order of 10^{-2} to 10^0 cm/s), so the conclusions that depend on it (Eq. 3 and Figure 12b) are conservative. In every treated specimen, one geotextile disc was placed at mid-height, normal to the specimen axis. The specimen diameter was 60 mm for permeability and 37 mm for compression.

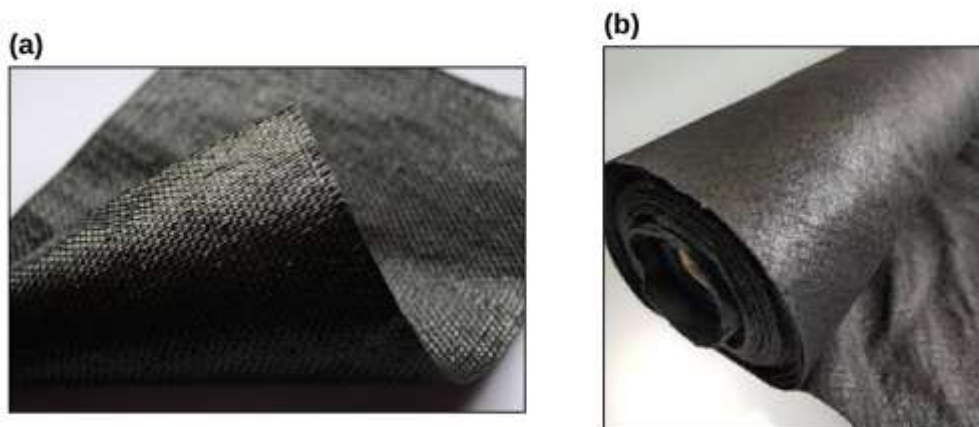


Figure 2. Geotextiles: (a) woven geotextile, shown for comparison; (b) non-woven geotextile roll used as the treatment (author documentation).

3. Laboratory Testing Programme

a. Index and phase-state tests

Natural water content, specific gravity (pycnometer), wet unit weight and Atterberg limits (Casagrande cup and thread-rolling) were measured with standard

laboratory procedures (Figure 3). Atterberg limits were determined with the multipoint Casagrande method (ASTM D4318 [39]). The soil was classified with the Casagrande plasticity chart, using the A-line $PI = 0.73(LL - 20)$ [40]. Grain-size analysis (sieve and hydrometer) and the oven-dried liquid limit or loss-on-ignition test, which are needed under ASTM D2487 [41] to confirm a fine-grained classification and to separate MH from OH, were not available; the soil is therefore reported as MH/OH. The void ratio e , porosity n , degree of saturation S_r and dry unit weight γ_d were derived from w , G_s and γ with standard phase relations.

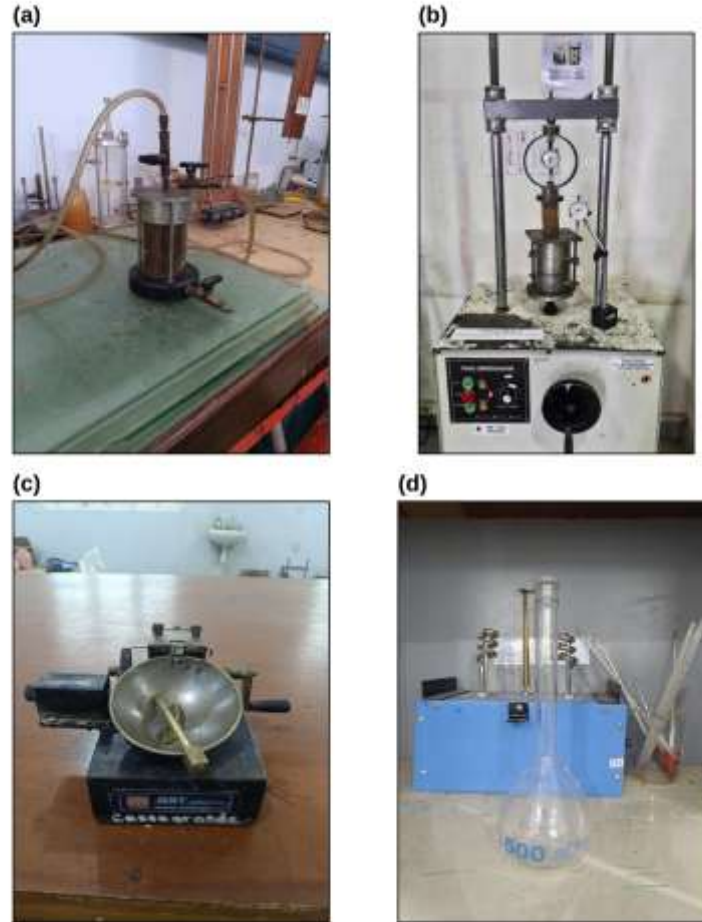


Figure 3. Laboratory apparatus: (a) falling-head permeameter; (b) unconfined compression machine during testing; (c) Casagrande liquid-limit device; (d) pycnometer for specific gravity (author documentation).

b. Falling-head permeability

Five specimens were tested for each condition (natural soil and soil with geotextile) in a falling-head permeameter (Figure 3a). The specimen had a diameter of 60 mm ($A = 2827.4 \text{ mm}^2$) and a length of 32 mm, and the standpipe had a diameter of 1.5 mm ($a = 1.767 \text{ mm}^2$), so that $a \cdot L/A = 0.020 \text{ mm}$. For each specimen the head was read three times, as it fell from 98 to 75 mm, from 75 to 50 mm and from 50 to 25 mm, and k was calculated with Eq. (2) for each reading. The specimen value of k is the mean of its three readings, which is the statistical unit used for testing. Because the three readings of one specimen are not independent, treating them as fifteen separate observations would inflate the sample size; this pseudo-replication was avoided. The permeameter was a rigid-wall mould, and the specimens were not

back-pressure saturated in a flexible-wall cell, as recommended for fine-grained soils by ASTM D5084 [42]. Capillary rise in the 1.5 mm standpipe (up to about 20 mm for a clean glass tube) is of the same order as the lower heads and affects the absolute value of k ; because both conditions used the same heads, standpipe and specimen geometry, it does not affect the ratio of k values (part 2a of the Result and Discussion).

c. Unconfined compression

Unconfined compression tests followed ASTM D2166 [43] on cylindrical specimens 37 mm in diameter and 81 mm high ($A_0 = 10.75 \text{ cm}^2$, height-to-diameter ratio 2.19), using a strain-controlled machine of the type commonly used for soft clays (Figure 3b) [31]. Load-ring and dial readings were converted to axial stress with the area correction of Eq. (5). One stress-strain curve was obtained for each condition. Loading stopped at an axial strain of 8.64% (natural) and 8.40% (geotextile); the largest recorded stress was taken as q_u , and $c_u = q_u/2$. Only one specimen per condition was tested; this is the main limitation of the strength results and is discussed in part 4f of the Result and Discussion.

4. Statistical Modelling, Validation and Outlier Screening

Permeability. The two groups of specimen means were compared with Student's two-sample t-test (equal variances), with Welch's t-test as a check, and with the exact Mann-Whitney U test, which does not assume normality. Normality was checked with the Shapiro-Wilk test, which is among the most powerful tests for small samples [44], and equal variance with Levene's test. The effect size was expressed as Hedges' g , and the 95% confidence interval of the mean difference was obtained both from the t-distribution and from 20,000 bootstrap resamples. With five specimens per group the bootstrap distribution is coarse and tends to understate the interval, so the t-based interval is treated as primary. The ratio of the two means, with a 95% confidence interval from the log-ratio method, is reported as the engineering effect measure. Because all specimens came from one bulk sample, the tests describe the repeatability of the difference for this sample, not the variability of the subgrade along the road.

Stress-strain curves. Each curve was fitted with a linear model and a second-order polynomial in strain. A pooled model was then fitted to both curves: $\sigma_d = \beta_0 + \beta_1 \varepsilon_c + \beta_2 \varepsilon_c^2 + \beta_3 G + \beta_4 G \cdot \varepsilon_c$, where ε_c is the centred strain and G is 0 for natural soil and 1 for geotextile. The terms β_3 and β_4 measure the shift and the change in slope caused by the geotextile. Each model was judged with R^2 , adjusted R^2 , RMSE, MAE, the overall F-test (ANOVA) and the p-values of each coefficient. Residuals were checked for normality (Shapiro-Wilk [44]) and constant variance (the studentized Breusch-Pagan test [45]); collinearity was checked with the variance inflation factor (VIF); and influence was checked with Cook's distance, leverage and externally studentized residuals. Because successive readings of one curve are measured on the same specimen, they are serially correlated and are not independent replicates. The regression statistics are therefore used only to describe and compare the shape of the two curves; they are not used as inferential evidence of a geotextile effect on q_u .

Outliers. Possible outliers were screened with three rules: $|z| > 2$ (or > 3 for strong outliers), the $1.5 \times \text{IQR}$ fences, and Cook's distance above $4/n$ (attention) or above 1 (influential). No point was removed automatically. For the permeability groups ($n = 5$) the $|z| > 2$ rule cannot flag any point, because the largest possible $|z|$

in a sample of five is $(n - 1)/\sqrt{n} = 1.79$; for these data only the IQR fences are informative. Each flagged point was first classified as a recording error, a physically plausible point, or a structural point caused by the model form, and only then was a decision made.

RESULT AND DISCUSSION

1. Index Properties and Phase State of the Subgrade

Table 1 and Figure 4 summarise the index properties. The natural water content is 48.14%, the specific gravity is 2.66, and the wet unit weight is 1.58 g/cm³. The liquid limit is 68.44% and the plastic limit is 44.44%, so the plasticity index is 24.0%. On the plasticity chart, the A-line at LL = 68.44% lies at PI = 35.4%, so the soil plots well below it (see also Figure 12c). The soil is therefore a high-plasticity silt or organic soil (MH/OH), not a high-plasticity clay (CH) [40]. This distinction is important because MH/OH soils often combine a high plastic limit with a loose, open fabric.

Table 1. Index and phase properties of the studied subgrade (primary data).

Property	Unit	Value	Remark
Natural water content, w	%	48.14	Mean of three specimens
Specific gravity, G_s	–	2.66	Pycnometer
Wet unit weight, γ	g/cm ³	1.58	Ring sample
Liquid limit, LL	%	68.44	Casagrande cup
Plastic limit, PL	%	44.44	Thread rolling
Plasticity index, PI	%	24.00	PI = LL – PL
A-line PI at the same LL	%	35.4	PI = 0.73(LL – 20) [40]
Group on plasticity chart	–	MH/OH	Below the A-line [40]
Liquidity index, LI / consistency index, CI	–	0.15 / 0.85	From w, LL and PL
Void ratio, e / porosity, n	–	1.49 / 0.60	Phase relations
Degree of saturation, S_r	%	85.7	Phase relations
Dry unit weight, γ_d	g/cm ³	1.07	$\gamma/(1 + w)$
Untreated q_u	kg/cm ²	0.379	Unconfined compression

Note: phase values were derived from w, G_s and γ of this study.

The phase relations explain why the soil is soft even though its liquidity index is low (LI = 0.15). The void ratio is 1.49 and the porosity is 0.60, so 60% of the soil volume is pore space (Figure 4b). Because the plastic limit is high (44.44%), even a water content of 48% places the soil close to its plastic limit, yet the open fabric gives little particle-to-particle contact. The apparent conflict between a ‘stiff’ consistency index (CI = 0.85) and a ‘soft’ measured strength is therefore resolved by fabric and void ratio, not by water content alone. This agrees with the view that plasticity alone does not define clay-like behaviour and that the position of a soil relative to the A-line carries information about its mineralogy and fabric [40]. The degree of saturation is about 86%, which means that some air remains in the pores. This affects the permeability measurements, as discussed in part 4f of the Result and Discussion.

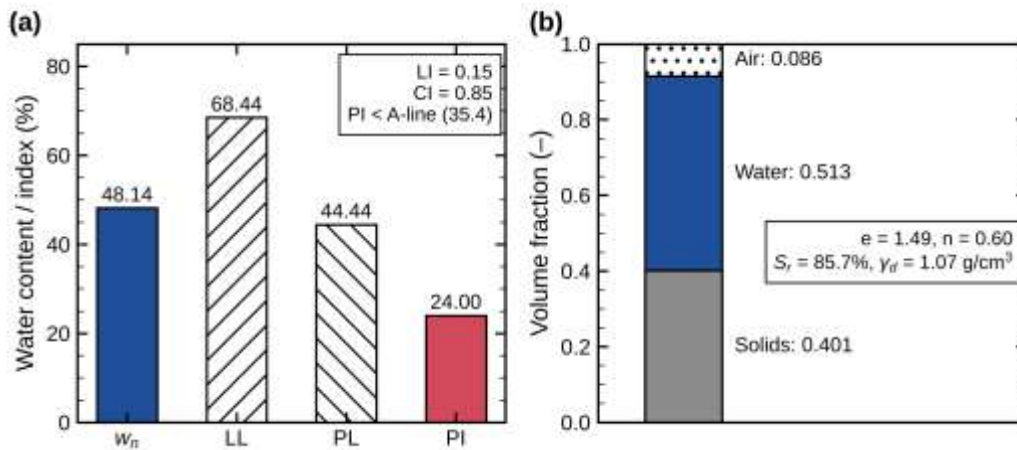


Figure 4. Index and phase state of the subgrade (primary data): (a) natural water content and Atterberg limits with liquidity index (LI) and consistency index (CI); (b) three-phase volume fractions derived from w , G_s and γ .

Compared with other soft subgrade soils reported in the literature, the Gresik-Lamongan soil is weaker than Pekan soft clay after lime treatment, which reached 116 kPa at its optimum lime content of 4% [15], and it shares with black cotton subgrades the need for treatment before use [46]. Its high liquid limit (68.4%) is typical of the moisture-sensitive subgrades that cause pavement defects [1].

2. Permeability Response

a. Gradient dependence and time-recording check

Table 2 lists the permeability of each specimen. The natural soil gave a mean k of $1.425 \times 10^{-6} \text{ mm/s}$ ($1.43 \times 10^{-7} \text{ cm/s}$), a very low value typical of fine-grained soils [10], [11]. With the geotextile, the mean rose to $1.776 \times 10^{-6} \text{ mm/s}$. The scatter between specimens was very small (coefficient of variation 1.9% and 2.3%), which shows that specimen preparation was repeatable.

Table 2. Falling-head permeability of specimens (primary data), outlier screening and hypothesis tests.

Specimen / statistic	Natural k ($\times 10^{-6} \text{ mm/s}$)	z-score	Geotextile k ($\times 10^{-6} \text{ mm/s}$)	z-score
Specimen 1	1.446	0.77	1.803	0.63
Specimen 2	1.453	1.04	1.745	-0.75
Specimen 3	1.418	-0.28	1.823	1.11
Specimen 4	1.385	-1.50	1.789	0.31
Specimen 5	1.425	-0.03	1.722	-1.31
Mean $\pm$ SD	1.425 ± 0.027		1.776 ± 0.042	
Coefficient of variation (%)	1.88		2.34	
Shapiro-Wilk p [44]	0.679		0.685	
Points outside $1.5 \times \text{IQR}$ fences	0		0	
Mean k if recorded times are cumulative	2.06		2.63	
Levene test (equal variance)	$p = 0.39$			
Student t-test, $df = 8$	$t = -15.87, p = 2.5 \times 10^{-7}$			

Welch t-test	$p = 1.2 \times 10^{-6}$
Mann–Whitney U (exact)	$U = 0, p = 0.0079$
Mean difference Δk (95% CI)	0.351 (0.300–0.402)
Bootstrap 95% CI of Δk	0.312–0.389
Hedges g / relative gain	9.07 / +24.6%

Note: each specimen value is the mean of three readings computed with Eq. (2); $a \cdot L/A = 0.020$ mm. Test statistics refer to specimen means ($n = 5$ per group).

Figure 5a reveals a feature that is hidden by the mean values. Within each specimen, the calculated k falls by about half from the first to the third reading, from 2.13 to 1.05×10^{-6} mm/s for natural soil and from 2.53 to 1.34×10^{-6} mm/s with geotextile, as the mean hydraulic gradient falls from 2.70 to 1.17 . For a soil obeying Eq. (1), k should not depend on the gradient. There are three possible explanations. The first is physical: laboratory permeability measurements on fine-grained soils are sensitive to gradient, saturation and test procedure [29], and partly saturated pores ($S_r \approx 86\%$) can trap air bubbles that block flow as the driving head falls. The second is procedural: the stopwatch times may have been recorded as cumulative times from the start of the test rather than as the duration of each interval. The third is capillary rise in the thin standpipe: a constant capillary head of up to about 20 mm, if not subtracted from the measured heads, lowers the calculated k most strongly in the last interval, where the heads are smallest.

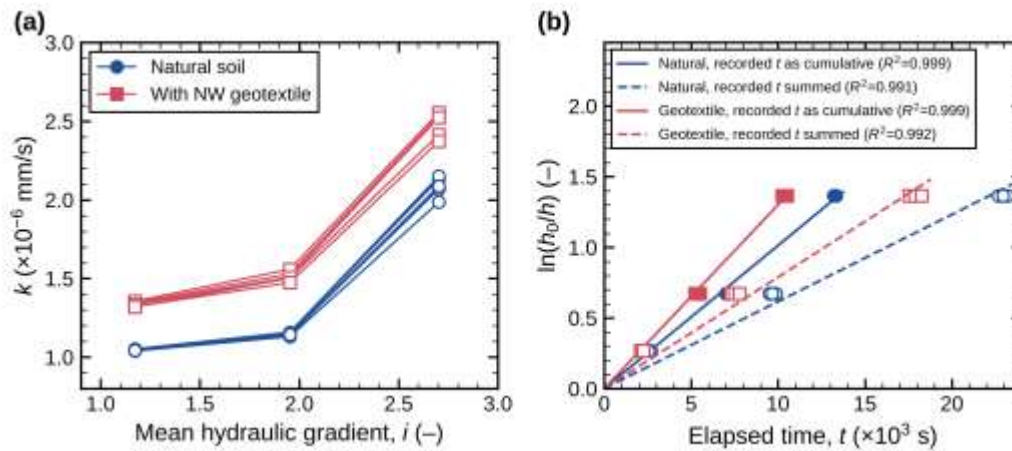


Figure 5. Falling-head diagnostics (primary data): (a) permeability of each reading against the mean hydraulic gradient for five specimens per condition (filled symbols: specimen 1); (b) head-decay linearity, $\ln(h_0/h)$ against elapsed time, under the two possible time-recording interpretations.

Figure 5b tests the second explanation. If k is constant, $\ln(h_0/h)$ must grow linearly with elapsed time. When the recorded times are treated as cumulative, the data are almost perfectly linear ($R^2 = 0.999$ for both conditions). When the recorded times are added together, as the interval interpretation requires, the line bends and the fit is weaker ($R^2 = 0.991$ – 0.992). This is supporting evidence that the times were cumulative. A more direct check is given by the ratio of the interval values. If k is constant and the recorded times are cumulative, the interval-based calculation must

give $k_3/k_1 = \ln(50/25)/\ln(98/25) = 0.51$ (and $k_2/k_1 = 0.60$). The measured ratios are $1.05/2.13 = 0.49$ for natural soil and $1.34/2.53 = 0.53$ with geotextile, in close agreement. The cumulative reading is therefore adopted as the most likely interpretation, and the corresponding values are given in Table 2 beside the originally calculated values. A capillary effect cannot be excluded and would act in the same direction. Under that reading, the specimen permeability becomes 2.06×10^{-6} mm/s for natural soil and 2.63×10^{-6} mm/s with geotextile, about 45% higher than the reported means. The key point for this study is that the treatment ratio is almost unchanged: 1.246 under the reported calculation and 1.278 under the cumulative reading. Because a , L , A , h_1 and h_2 are identical for both conditions, the ratio of k values is simply the inverse ratio of the drainage times, and is therefore robust to this uncertainty. The absolute k values should not be used for design without new tests with recorded interval times.

b. Hypothesis testing of the drainage gain

The specimen means passed the Shapiro–Wilk test in both groups ($p = 0.68$ and 0.69), and Levene’s test found no difference in variance ($p = 0.39$), so the assumptions of Student’s t -test are met. The test gave $t(8) = -15.87$ and $p = 2.5 \times 10^{-7}$ (Table 2). Welch’s test ($p = 1.2 \times 10^{-6}$) and the exact Mann–Whitney test ($U = 0$, $p = 0.0079$, the smallest p -value possible with five specimens per group) agree. The one-way ANOVA gives $F(1, 8) = 252.0$, which is the square of the t -statistic. The mean gain is 0.351×10^{-6} mm/s, with a 95% confidence interval of 0.300 – 0.402 (t -based) and 0.312 – 0.389 (bootstrap), as shown in Figure 6.

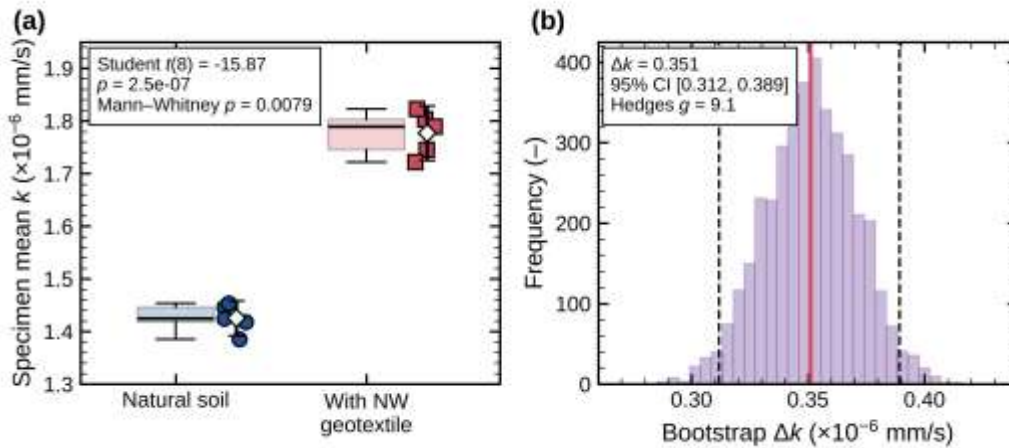


Figure 6. Treatment effect on permeability (primary data): (a) box-and-strip plot of specimen means with the mean and 95% confidence interval; (b) bootstrap distribution (20,000 resamples) of the mean difference Δk with its 95% interval.

The effect size is very large (Hedges’ $g = 9.1$), but this needs careful reading. The large value comes mostly from the very small scatter between specimens prepared from one bulk sample, not from a large physical change. In engineering terms, the geotextile raised k by about one quarter. That is a statistically clear, highly repeatable, but modest change. Expressed as a ratio of means, the gain is 1.25 (95% CI 1.21–1.29), which is the most useful effect measure for engineering use. Because all specimens were prepared from one bulk sample, this interval describes the repeatability of the measured difference for this sample, not the variability of the

subgrade along the road. It does not move the soil into a different drainage class (Figure 12b).

3. Stress–Strain Response

a. Proportional strengthening

Figure 7a shows the two stress–strain curves and Table 3 summarises the strength and stiffness values. With the geotextile, the stress was higher at every strain. The largest recorded stress rose from 0.379 to 0.489 kg/cm² (37.2 to 48.0 kPa), a gain of 29.0%, and c_u rose from 0.19 to 0.245 kg/cm² (18.6 to 24.0 kPa). Because the natural specimen was loaded to a slightly larger strain, a fairer comparison is made at the same strain of 8.40%, where the natural stress is 0.374 kg/cm² and the gain is 30.7%. The secant modulus at half of q_u (E_{50}) rose from 912 to 1221 kPa (+33.9%), and the initial modulus rose from 5.2 to 6.4 MPa.

Table 3. Strength and stiffness parameters from unconfined compression tests (primary data).

Parameter	Natural soil	With NW geotextile	Change
q_u , largest recorded stress (kg/cm ²)	0.379	0.489	+29.0%
q_u (kPa)	37.2	48.0	+29.0%
$c_u = q_u/2$ (kPa)	18.6	24.0	+29.0%
Axial strain at end of test (%)	8.64	8.40	–
σ_d at equal strain $\varepsilon = 8.40\%$ (kg/cm ²)	0.374	0.489	+30.7%
Secant modulus E_{50} (kPa)	912	1221	+33.9%
Strain at 0.5 q_u , ε_{50} (%)	2.04	1.96	–3.6%
Initial modulus at $\varepsilon = 0.25\%$ (MPa)	5.20	6.43	+23.7%
Linear slope $d\sigma_d/d\varepsilon$ (kg/cm ² per %)	0.0299	0.0391	+30.8%
Strength ratio R_σ (mean $\pm$ SD), trend p	–	1.276 $\pm$ 0.040, p = 0.54	–
Consistency class by q_u	Soft	Soft (near medium)	–
Allowable pressure $p = N_c \cdot c_u$, Eq. (7) [36] (kPa)	58.5 ($N_c = \pi$)	123.5 ($N_c = \pi + 2$)	$\times 2.11$

Note: specimens 37 mm $\times$ 81 mm; area-corrected stress by Eq. (5) [31]. Neither curve reached a peak, so q_u is a lower bound.

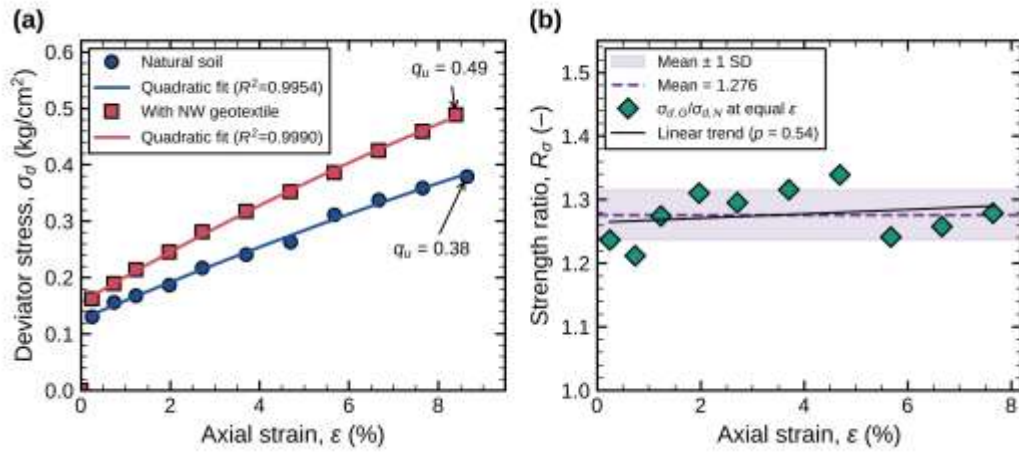


Figure 7. Unconfined compression response (primary data): (a) deviator stress against axial strain with quadratic fits (origin excluded from the fits); (b) strength ratio $R_\sigma = \sigma_{d,G}/\sigma_{d,N}$ at equal strain with its mean, ± 1 SD band and linear trend.

Figure 7b shows a further feature of the two curves. The ratio of geotextile stress to natural stress at the same strain stays almost constant over the whole test, with a mean of 1.276 and a standard deviation of only 0.040. A linear trend of this ratio against strain is not significant (slope = 0.0034 per %, $p = 0.54$). In other words, the geotextile multiplies the stress response by a nearly fixed factor from the first reading to the last. Such proportional strengthening is what one would expect if the treatment changes a material property of the whole specimen, such as its effective stress state or its stiffness, rather than adding a separate tensile force that grows only after large strains. Tensile membrane action in a geotextile needs large deformations to develop [17], [37], and at strains below 1% it would be negligible. The constant ratio from 0.25% strain upward is therefore consistent with a drainage and confinement effect at the interface. With one specimen per condition, however, two other explanations cannot be excluded: lateral restraint of the specimen by friction at the disc, which acts from small strains because the lateral strain grows with the axial strain, and small differences in density or water content between the two specimens, which would also scale the whole curve.

On the consistency scale, both soils remain in the soft class (25–50 kPa), but the treated soil moves close to the boundary of the medium class. Neither curve reached a clear peak before the test ended. Since ASTM D2166 takes q_u as the peak stress or the stress at 15% strain when there is no peak, both measured values are lower bounds of the true q_u .

b. Regression validation and outlier classification

Table 4 lists the fit and validation statistics. When the origin point ($\epsilon = 0$, $\sigma = 0$) is included, the fits are poor ($R^2 = 0.90$ – 0.93), the residuals fail the normality test ($p < 0.001$ for the linear models), and the origin has an externally studentized residual of 16 to 31. This is not a bad measurement: the first loading step includes seating of the platens and an initial stiff response, so the origin does not follow the same law as the rest of the curve. It was classified as a structural point and was kept in Figure 7a but left out of the regression.

Table 4. Regression models of the stress–strain response: goodness of fit, assumption checks and influence diagnostics.

Statistic	N-lin (origin)	G-lin (origin)	N-lin	N-quad	G-lin	G-quad	Pooled
n	12	12	11	11	11	11	22
R ²	0.896	0.902	0.9942	0.9954	0.9968	0.9990	0.9978
Adjusted R ²	0.886	0.892	0.9936	0.9943	0.9965	0.9987	0.9973
RMSE (kg/cm ²)	0.0338	0.0423	0.0063	0.0056	0.0060	0.0034	0.0048
MAE (kg/cm ²)	0.0227	0.0283	0.0051	0.0048	0.0048	0.0029	0.0038
ANOVA F	86.3	92.0	1551	871	2808	3965	1927
p (overall F)	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
p of ε^2 term	–	–	–	0.186	–	0.003	0.004
Shapiro–Wilk p	0.001	0.001	0.856	0.674	0.969	0.891	0.249
Breusch–Pagan p	0.142	0.134	0.094	0.228	0.138	0.997	0.338
VIF max (raw / centred)	–	–	–	15.8 / 1.09	–	16.5 / 1.07	2.07
Max Cook's D	1.37	1.40	0.84	0.85	0.80	1.30	0.27
Max leverage h	0.34	0.32	0.35	0.63	0.33	0.58	0.50
Max studentized r*	16.6	21.8	2.06	2.56	2.82	1.95	3.17

Note: N = natural soil; G = with geotextile; lin = linear; quad = quadratic; “origin” = fit includes the point $\varepsilon = 0, \sigma = 0$; Pooled = $\sigma_d = \beta_0 + \beta_1\varepsilon_c + \beta_2\varepsilon_c^2 + \beta_3G + \beta_4G \cdot \varepsilon_c$. Shapiro–Wilk test [44]; studentized Breusch–Pagan test [45]. Pooled coefficients: $\beta_3 = 0.0706 \pm 0.0023$ ($p = 2.7 \times 10^{-16}$), $\beta_4 = 0.0091 \pm 0.0008$ ($p = 4.8 \times 10^{-9}$).

Without the origin, all models fit very well. For natural soil, the quadratic term is not significant ($p = 0.19$), so a straight line is adequate ($R^2 = 0.9942$, slope 0.0299 kg/cm² per % strain). For the treated soil, the quadratic term is significant ($p = 0.003$), which shows a slight concave shape, and the linear slope is 0.0391 kg/cm² per %, 31% steeper than the natural soil. The raw VIF of the strain and strain-squared terms is 15.8–16.5, which signals collinearity; centring the strain cuts it to 1.07–1.09 without changing the fit.

The pooled model explains 99.8% of the variance (adjusted $R^2 = 0.997$, RMSE = 0.0048 kg/cm², MAE = 0.0038 kg/cm², $F = 1927$, $p < 10^{-15}$). The geotextile shift $\beta_3 = 0.0706 \pm 0.0023$ kg/cm² ($p = 2.7 \times 10^{-16}$) and the slope change $\beta_4 = 0.0091 \pm 0.0008$ kg/cm² per % ($p = 4.8 \times 10^{-9}$) are both significant, and the partial F-test for the two geotextile terms gives $F(2, 17) = 524.9$ ($p = 5.3 \times 10^{-16}$). The residuals are normal (Shapiro–Wilk $p = 0.25$) and show constant variance (Breusch–Pagan $p = 0.34$), and the largest VIF is only 2.07 (Figure 8a, b).

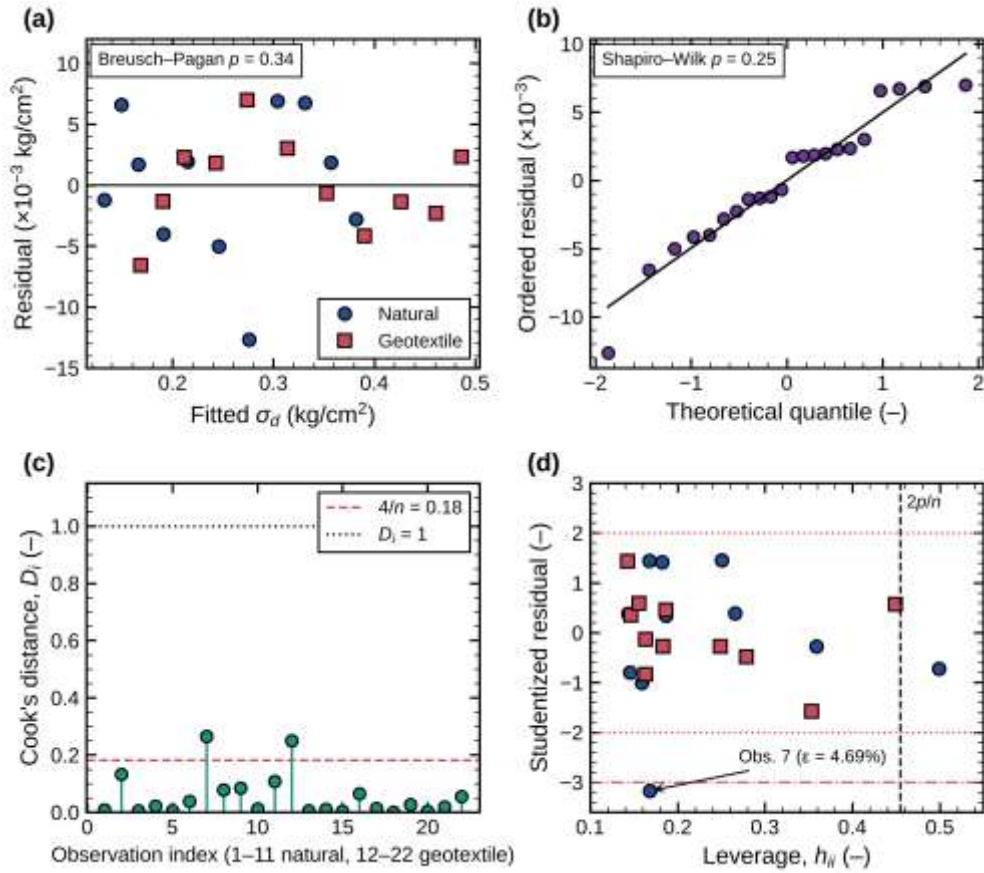


Figure 8. Validation of the pooled stress–strain model: (a) residuals against fitted values; (b) normal Q–Q plot of residuals; (c) Cook’s distance with the $4/n$ and $D = 1$ thresholds; (d) leverage against externally studentized residuals.

Figure 8c and d show the influence checks. Two points exceed the attention level of Cook’s distance ($4/n = 0.18$): the natural-soil reading at 4.69% strain ($D = 0.27$, studentized residual -3.17) and the first geotextile reading ($D = 0.25$). Neither exceeds $D = 1$. The 4.69% reading lies just before a visible step in the load–dial record, from 0.263 to 0.311 kg/cm², which is typical of a stick–slip response in a soft specimen. It was classified as physically plausible and kept; removing it changes β_3 by only 2.4%, and removing the first geotextile reading changes β_3 by 3.5%, so neither point drives the conclusion. The last reading of each curve has high leverage ($h = 0.45$ – 0.50) because it lies at the end of the strain range, which is a design feature of the test, not an error. The permeability data contain no outliers by any rule: the largest $|z|$ is 1.50 for specimen means and 1.49 for single readings, and no value lies outside the IQR fences (Table 2). The recommendation is to keep all points and to extend future tests to 15% strain, which would reduce the leverage of the end points. The residuals of the single-curve fits are serially correlated by construction, so the p-values in Table 4 describe the fit of the curve shape and not the reliability of the geotextile effect.

4. Mechanisms and Engineering Implications

a. Why a thin sheet raises equivalent permeability: a series-flow check

Equation (3) gives a simple test of the drainage mechanism. Because the geotextile is about a thousand times more permeable than the soil, the term t_g/k_g is negligible, and Eq. (3) reduces to $k_{eq} \approx k_s \cdot L/(L - t_g)$. For the measured ratio of 1.246 and $L = 32$ mm, the geotextile would have to replace about 6.3 mm of soil. Needle-punched non-woven geotextiles are usually only a few millimetres thick, and they compress under load, which also lowers their own drainage capacity [18]. Unless the sheet were about 6 mm thick, which is unusual for this type of product, simple series flow can explain only part of the measured gain. The rest must come from changes at the interface. Three mechanisms are possible. First, the sheet creates a highly permeable boundary in the middle of the specimen, so the drainage path for pore water and trapped air is halved; this helps the partly saturated soil to saturate and so raises its apparent k . Second, during specimen preparation the soil next to the sheet is less dense than the bulk soil, so a thin, more open zone forms at the interface. Third, any gap between the sheet edge and the mould wall can act as a side-wall leak. If side-wall leakage contributed, part of the measured gain would be a test artefact rather than a property of the treated soil, and future tests should use a flexible-wall permeameter to exclude this path. Laboratory work on geosynthetic-reinforced soil has shown that the inclusion of geosynthetic layers can change the equivalent permeability of the soil mass and that this effect depends on the layer arrangement and the flow direction [30]. The present data cannot separate the three mechanisms, but they show that the geotextile works mainly by changing the soil next to it, not by adding its own permeability in series.

This reading may also help to explain the strength gain, although strength and permeability were measured on specimens of different geometry, so the two percentages cannot be compared directly. Pore pressure in a fine-grained soil dissipates in a time that grows with the square of the drainage path. Halving the drainage path therefore cuts the dissipation time by about four, even if k itself rises by only a quarter. A faster dissipation of the pore pressure raises the effective stress in Eq. (4) during sample preparation and loading, and this effect is felt through the whole specimen. This is consistent with the constant stress ratio in Figure 7b. However, the compression specimens were not consolidated under an applied stress and were sheared relatively quickly, so the amount of pore-pressure dissipation during preparation and loading is unknown. Without pore-pressure measurement this mechanism remains a hypothesis, to be tested with consolidated-undrained triaxial tests.

b. Interface friction in wet and dry states: reconciling the literature

Figure 9 shows the direct-shear results reported by He et al. for a soil-composite geotextile interface subjected to wet-dry-freeze-thaw (WDFT) cycles [28]. After a single cycle at negative temperature, the average interface peak strength rose by 39%, 56.9% and 65.3% at -2 °C, -6 °C and -10 °C (Figure 9a). With further cycles the gain declined, and after ten cycles it was only 6.6%, 19.3% and 29.2% (Figure 9b). The interface cohesion and friction angle followed the same trend, and the interface cohesion was the parameter most sensitive to temperature and cycling [28]. Freeze-thaw does not occur in East Java; these results are used here only as evidence that interface strength depends on the moisture history of the soil, which

in the tropics is driven by wet–dry cycles. The lesson for a tropical subgrade is that the soil–geotextile interface is not a fixed property: it changes with the moisture history of the soil next to the sheet.

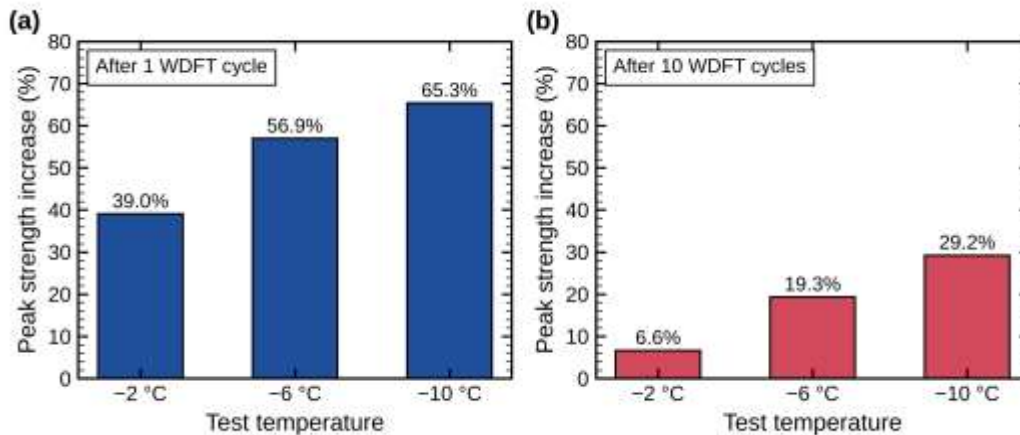


Figure 9. Soil–composite geotextile interface under wet–dry–freeze–thaw (WDFT) cycles reported by He et al. [28]: average increase in interface peak strength at –2, –6 and –10 °C (a) after one cycle and (b) after ten cycles (redrawn in a uniform OriginPro-style format from the values reported in [28], without modifying the data).

Other interface studies point the same way, although they seem at first to disagree. For silt against a cement-mortar surface, a rough interface was stronger than the silt itself while a smooth interface was weaker, and the interface strength fell markedly as the silt approached saturation [26]. For unsaturated soil–geotextile interfaces, higher suction gave higher interface strength [27], and the soil–woven geotextile interface could be modelled from direct-shear results only when its non-linear, roughness-controlled behaviour was taken into account [33]. The apparent conflict, interfaces that are sometimes stronger and sometimes weaker than the soil, can be explained by two controls: surface roughness and the water state of the thin soil layer at the interface [17]. A rough, felt-like non-woven sheet in a drier soil can match or exceed the soil strength ($\delta/\phi \geq 1.0$), but a wet, low-suction fine-grained soil forms a weak film at the interface. For the Gresik–Lamongan soil, which is a wet MH/OH material, the safer design assumption is therefore an interface ratio below 1.0, and the beneficial role of the non-woven sheet should be credited mainly to drainage and confinement, not to interface friction.

c. From laboratory strength to rut resistance and embankment stability

Laboratory gains matter only if they carry over to the pavement. Figure 10 shows the results of Ghasemi Nezhad et al. for a clay subgrade reinforced with natural (basalt, bagasse) and synthetic (polyester) fibres [16]. With 2% of 7.5 mm fibres, the CBR reached 19.17% with polyester, 15.43% with basalt and 13.16% with bagasse (Figure 10a), and the indirect tensile strength reached 48.57, 60.7 and 47.48 kPa (Figure 10b). Polyester, the same family of polymer used in many non-woven geotextiles, raised cohesion by about 70%, while basalt fibres raised the friction angle by about 100% [16]. These results show that polymer inclusions in clay mainly add cohesion and tensile capacity, which is consistent with the proportional strengthening seen in our specimens.

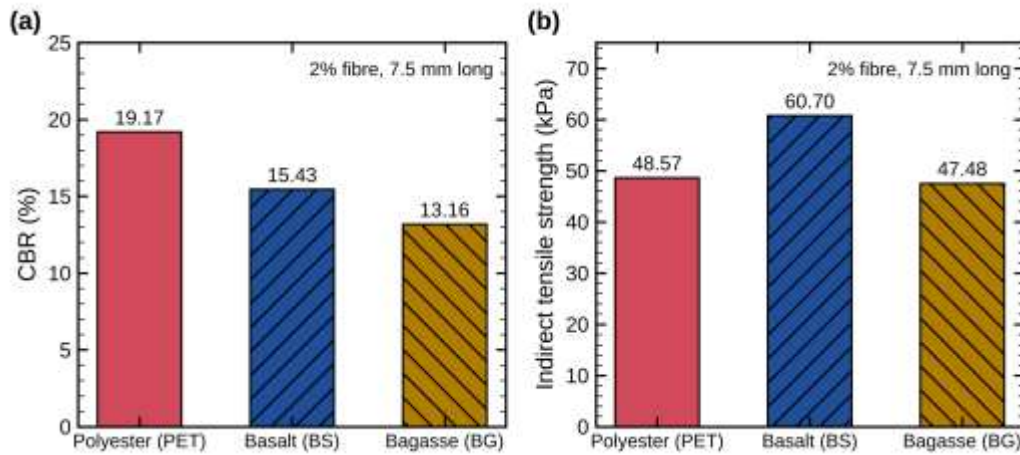


Figure 10. Clay subgrade reinforced with natural and synthetic fibres (2% content, 7.5 mm length) reported by Ghasemi Nezhad et al. [16]: (a) California bearing ratio and (b) indirect tensile strength for polyester, basalt and bagasse fibres (redrawn in a uniform OriginPro-style format from the values reported in [16], without modifying the data).

Full-scale field tests on unpaved roads over soft subgrade showed that geosynthetics were most effective when the base layer was thin and the subgrade was very weak [47], and laboratory models of unpaved roads on soft clay showed clear benefits from woven coir geotextiles placed at the base–subgrade interface [19]. Large-scale tests further showed that the benefit of reinforcement persists after surface maintenance of the road [37]. Triaxial tests on reinforced clay showed that the strength gain depends on drainage at the reinforcement; thin sand layers around the reinforcement improved the response because they let pore pressure escape and raised the interface friction [23]. That finding fits our own results closely. A non-woven geotextile in a clay plays the same role as the sand layer: it offers a drainage boundary next to the reinforcement. Lime and fibre added together also raised both strength and ductility of clay, because cementation increases soil–fibre interaction at the interface [48].

Geotextiles also control the volume change of problematic subgrades. Figure 11 shows the results of Tiwari and Satyam for an expansive soil subgrade with lime-treated coir geotextile mats [20]. One mat reduced the upward swelling pressure by 52.19% and two mats by 81.89% (Figure 11a), so the normalised swelling pressure fell from 1.00 to 0.48 and then to 0.18 (Figure 11b); shear strength and bearing capacity also improved [20]. At the scale of embankments, finite-element analyses of geotextile-reinforced road embankments on soft soil found that the required geotextile tensile strength is governed by the target safety factor [21], and a 72-case parametric study on soft coastal clay found that the safety-factor gain becomes small beyond three reinforcement layers, with the FEM results agreeing with limit-equilibrium analysis within 1.0–6.8% [22]. This again points to the same idea: in soft soil, the main value of a geotextile is to change how the soil mobilises its strength, not to add very large tensile forces.

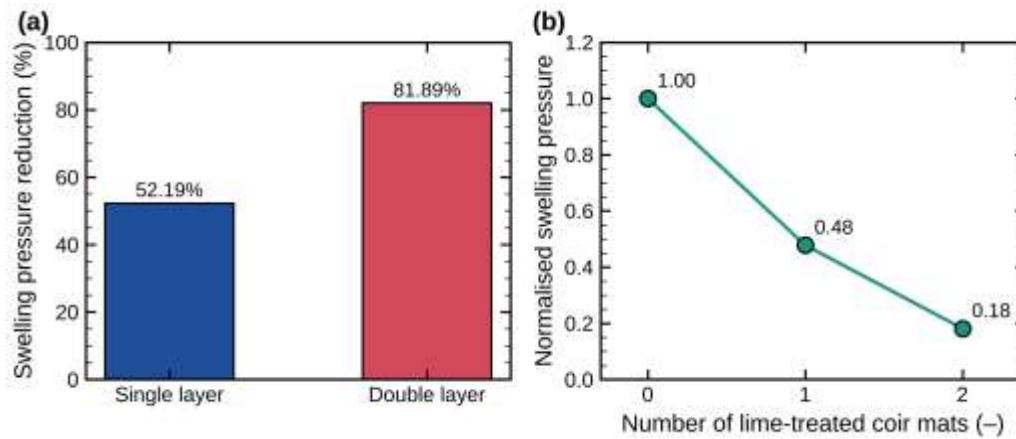


Figure 11. Expansive soil subgrade with lime-treated coir geotextile mats reported by Tiwari and Satyam [20]: (a) reduction of the upward swelling pressure for a single and a double layer; (b) the same results expressed as normalised swelling pressure against the number of mats (redrawn in a uniform OriginPro-style format from the values reported in [20], without modifying the data).

d. Geotextile versus chemical and granular stabilisation

Figure 12a places the geotextile gain beside other treatments that were reported with the unconfined compression test. Biopolymer treatment of kaolin raised the unconfined strength by up to 5.23 times with xanthan gum and 8.53 times with guar gum [32]. Lime raised Pekan soft clay to 116 kPa at its optimum content of 4% [15], and sawdust ash combined with lime gave the best reduction of plasticity and the best CBR of a black cotton subgrade at 16% ash and 4% lime [46]. Cement and lime binders raised CBR and reduced swell of expansive subgrades [1], and a calcium-carbide-residue binder brought a very wet sludge to 215.4 kPa with its water content cut to 32% [3]. By comparison, the geotextile gain of 1.29 times is the smallest. These values come from different soils, test conditions and curing times and are therefore not directly comparable; they indicate orders of magnitude only.

This comparison should not be read as a ranking of value. Chemical and biopolymer methods change the soil itself, need mixing, curing and quality control, and lime can make the soil brittle unless fibres are added [48]. A geotextile changes the boundary conditions of the soil: it drains, separates and confines [13], [14]. Its benefit is immediate, does not depend on curing, and, as part 4c shows, is most powerful when it changes the bearing mechanism of the whole pavement rather than the strength of a small specimen. The two approaches are therefore complementary. A geotextile is most attractive where the subgrade must stay drained and separated from the base, while chemical stabilisation is needed where the subgrade itself must become much stronger.

e. Integrated evidence map and design-space surface

Figure 12 brings the evidence together. Figure 12b shows the permeability of the Gresik-Lamongan soil under both time-recording interpretations, with and without geotextile, against the permeability of the geotextile sheet itself. The sheet is about a thousand times more permeable than the soil, which confirms that it can act as a drain only if it is connected to an outlet; a sheet that does not wick or drain can even hold water above it [38], and subsurface drainage design must therefore treat

the outlet as part of the system [14]. Figure 12c places the soil on the plasticity chart [40], below the A-line, which explains its tendency to drain poorly and lose strength when wet. Figure 12d collects the size of the effects reported for interface and inclusion treatments [16], [20], [28] beside the present results, and shows that the drainage and strength gains of a single non-woven sheet are modest but of the same order as many inclusion-type treatments.

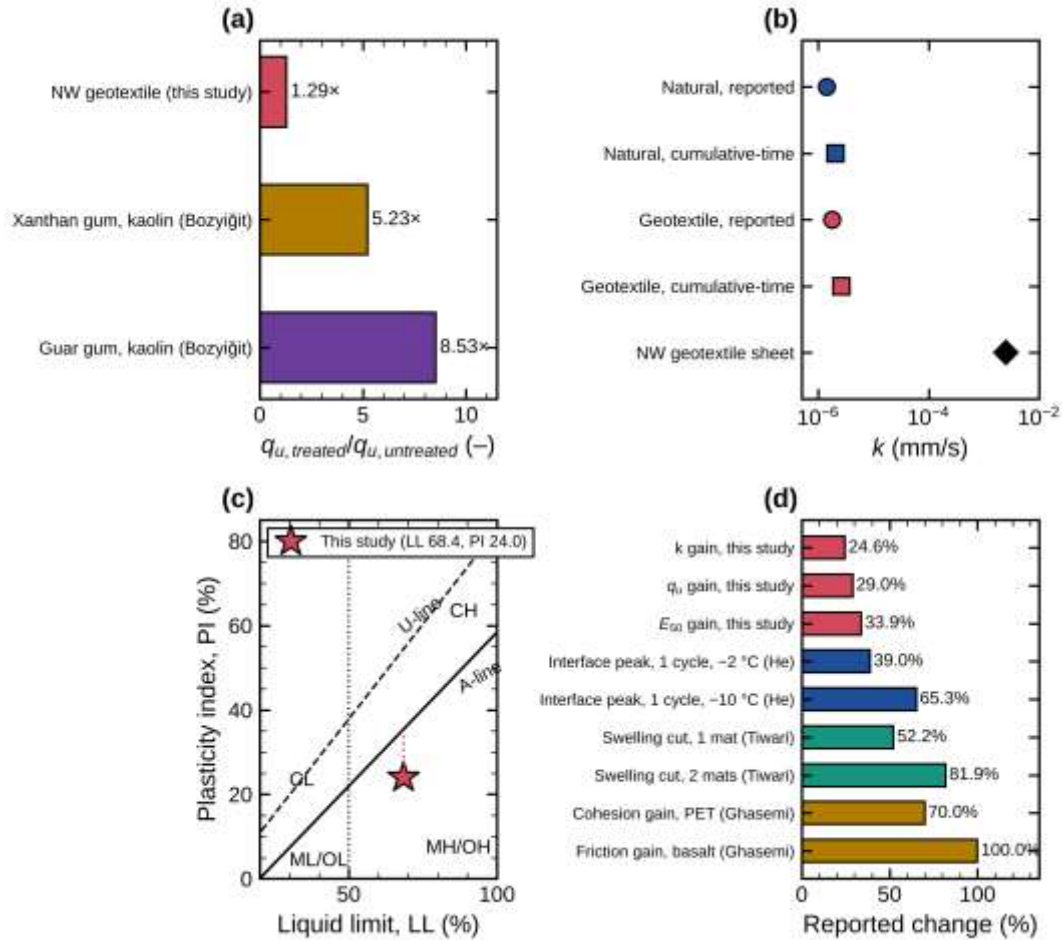


Figure 12. Integrated evidence map: (a) unconfined strength-gain ratio of the present geotextile and of biopolymer-treated kaolin [32]; (b) permeability of the present soil under both time-recording interpretations and of the geotextile sheet; (c) the present soil on the plasticity chart with the A- and U-lines [40]; (d) size of the effects reported for the present treatment and for interface and inclusion treatments [16], [20], [28] (literature values plotted without modification).

Table 5. Evidence matrix of soft-subgrade improvement used in the discussion.

Source	Soil / system	Method	Key metric	Reported result
This study	MH/OH road subgrade	One NW geotextile layer (lab)	k ; q_u	+24.6%; +29.0% ($R_\sigma = 1.28$)
[20]	Expansive subgrade	Lime-treated coir geotextile mats	Swelling pressure	-52.19% (one mat); -81.89% (two mats)
[16]	Clay subgrade	PET, basalt, bagasse fibres (2%, 7.5 mm)	CBR; ITS	19.17/15.43/13.16%; 48.57/60.7/47.48 kPa
[28]	Soil-composite geotextile interface	Direct shear, WDFT cycles	Peak strength	+39–65.3% (1 cycle); +6.6–29.2% (10 cycles)

[26]	Silt–mortar interface	Direct shear, water content, roughness	Interface strength	Falls markedly towards saturation
[22]	Embankment on soft coastal clay	72 FEM runs + RSM	Safety factor	Small gain beyond 3 layers; FEM–LEM 1.0–6.8%
[32]	Kaolin	Xanthan and guar gum	q_u	$\times 5.23$ (xanthan); $\times 8.53$ (guar)
[15]	Pekan soft clay	Hydrated lime	q_u	116 kPa at 4% lime
[3]	Dredged sludge for subgrade	CCR-based binder	q_u ; w	215.4 kPa; w down to 32%
[38]	Silty sands	Wicking vs non-wicking geotextile	Moisture reduction	Effective up to 15% fines; non-wicking sheet blocks flow
[36], [37]	Unpaved road on soft subgrade	Bearing-capacity approach	N_c	π unreinforced; $\pi + 2$ geotextile

Note: NW = non-woven; PET = polyester; ITS = indirect tensile strength; WDFT = wet–dry–freeze–thaw; FEM = finite element method; LEM = limit equilibrium method; RSM = response surface method; CCR = calcium carbide residue.

Table 5 summarises the evidence used in this discussion. Figure 13 then combines the strength result with Eq. (7). Each point on the surface is an allowable subgrade pressure $p = N_c \cdot c_u$. The natural soil, with $c_u = 18.6$ kPa and no reinforcement ($N_c = \pi$), gives $p = 58.5$ kPa. The treated soil, with $c_u = 24.0$ kPa and a geotextile mechanism ($N_c = \pi + 2$), gives $p = 123.5$ kPa, which is 2.11 times higher. Of this factor, only the 1.29-fold strength gain was measured in this study; the 1.64-fold factor from the change of N_c is an assumption of the design method that applies to any geotextile placed at the base–subgrade interface and was not tested here. The surface makes the two parts of this gain easy to see. Moving along the c_u axis alone (the laboratory strength gain) gives 1.29 times; moving along the N_c axis alone (the change of mechanism) gives 1.64 times; together they multiply. This surface is an illustrative design space, not a field prediction: it assumes that the laboratory strength gain will develop in the field through the drainage mechanism and that the geotextile is placed at the base–subgrade interface where it can act as assumed in the bearing-capacity approach [36], [37].

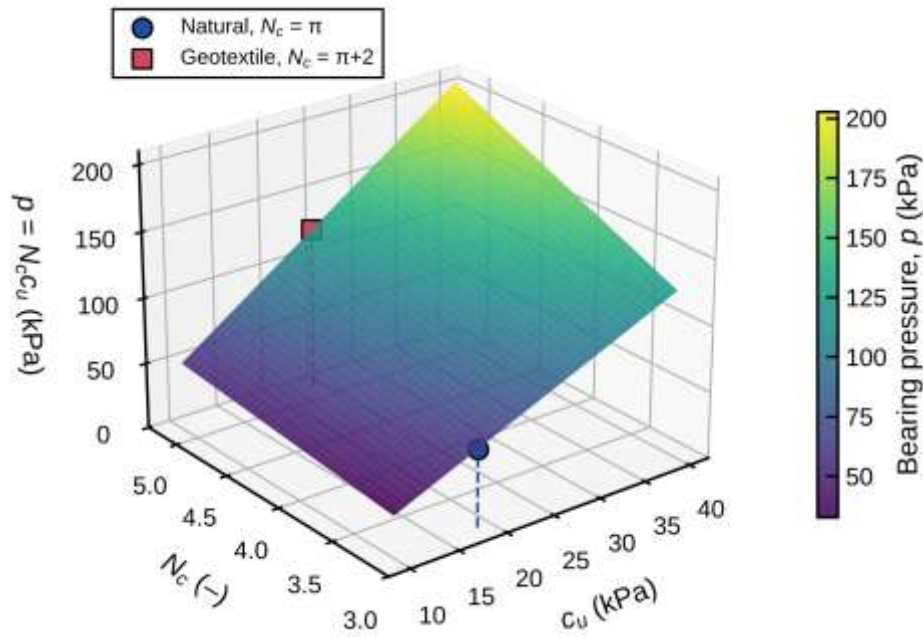


Figure 13. Three-dimensional design-space surface of the allowable subgrade pressure $p = N_c \cdot c_u$ from Eq. (7) [36], [37], with the natural soil ($N_c = \pi$, $c_u = 18.6$ kPa) and the geotextile-treated soil ($N_c = \pi + 2$, $c_u = 24.0$ kPa) of this study.

f. Methodological limitations

Several limitations must be kept in mind. (i) All specimens came from one location and one bulk sample, so the five permeability specimens are technical replicates; the results describe this soil, not the whole road. (ii) Only one unconfined compression curve was available for each condition, so no inferential test could be made on q_u itself; the statistical tests of the stress–strain response treat the readings within one curve as observations and therefore describe the curve shape, not the variability between specimens. (iii) Both compression tests stopped before a clear peak and before 15% strain, so q_u is a lower bound. (iv) The specimens were remoulded from disturbed soil and were not back-pressure saturated ($S_r \approx 86\%$), so the absolute values of k and q_u may differ from those of the intact subgrade. (v) The time-recording question raised in part 2a of the Result and Discussion affects the absolute value of k by about 45%, although the treatment ratio is robust. (vi) Placing the geotextile at mid-height of a laboratory specimen is not the same as placing it at the base–subgrade interface in the field; the field mechanism includes separation and membrane action that a small specimen cannot reproduce. (vii) The literature comparisons use values reported by other authors for different soils, test types and boundary conditions, so they indicate orders of magnitude rather than directly comparable design values. (viii) The 1.5 mm standpipe and the low heads make the absolute k sensitive to capillary rise and to the time-recording interpretation. (ix) The regression statistics of the stress–strain curves are descriptive only, because the readings of one curve are not independent. (x) Grain-size, organic-content, compaction and CBR results are not reported, which limits the classification and the direct use of the results in pavement design.

CONCLUSION

Within the limits of a single bulk sample and one compression test per condition, the following conclusions can be drawn:

1. The Gresik–Lamongan subgrade is a high-plasticity silt or organic soil (MH/OH) with a very open fabric ($e = 1.49$). Its softness comes mainly from this fabric rather than from a high liquidity index, so judging such soils by water content or consistency index alone will overestimate their strength.
2. A single non-woven geotextile layer raised the falling-head permeability by about one quarter (24.6%), and this gain is statistically clear by parametric and non-parametric tests. The soil nevertheless stays in the same very-low drainage class, so the sheet must be connected to an outlet in the field if it is to act as a real drain.
3. The ratio of permeability with and without geotextile is robust to the way the falling-head times were recorded (1.25–1.28), while the absolute k values are not (they may be about 45% higher). For comparative studies, the drainage-time ratio is a more reliable result than the absolute k .
4. The geotextile raised the unconfined strength by 29% (31% at equal strain) and the secant stiffness by 34%. The stress ratio between treated and natural soil stayed constant at 1.28 across all strains, which is consistent with a proportional strengthening of the whole specimen; this strength result is indicative until confirmed by replicated tests.
5. A series-flow check shows that the geotextile's own permeability can explain only a small part of the measured gain. The improvement comes mainly from the soil next to the sheet: a shorter drainage path, better saturation and a more open interface zone.
6. Combined with the Giroud–Noiray bearing relation, the modest laboratory strength gain (1.29 times) and the change of bearing mechanism (1.64 times) would multiply to about a 2.1-fold rise in allowable subgrade pressure; only the first factor was measured, so this value is an illustrative calculation, not a test result. The geotextile is therefore most valuable as a mechanism-changing layer at the base–subgrade interface, and it complements rather than replaces chemical or biopolymer stabilisation, which gives much larger strength gains (about 5–9 times in reported cases) at higher cost and with curing time.

Recommendation. Future work should run instrumented, saturated tests with several specimens per condition (falling-head tests with recorded interval times, and consolidated-undrained triaxial tests with pore-pressure measurement up to 15% strain), varying the geotextile position and thickness, and then confirm the predicted 2.1-fold gain with a field or large-scale wheel-tracking trial on this road, so that the drainage-driven strengthening mechanism can be measured directly.

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