SHORT COMMUNICATION

Shear behaviour of subgrade soil with reference to varying initial shear stress and plasticity index

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Abstract
The influence of stress anisotropy (K) (i.e. the ratio between effective horizontal and vertical stresses) on the shear behaviour of soils has received significant attention in past studies, but how its influence depends on different values of the plasticity index (PI) has not been properly quantified. In this study, the results of a series of undrained triaxial tests on anisotropically consolidated soil at different values of K are reported, and together with past experimental data, the interactive roles of K and PI on the shear behaviour of soil are rigorously interpreted. The findings indicate that the peak shear strength increases with higher brittleness, whereas the peak excess pore pressure diminishes when the value of K decreases. Moreover, increasing the value of PI up to 35 tends to increase the peak shear strength, but beyond that the influence of PI seems marginal. Based on the findings of this study, empirical equations incorporating PI and K to estimate the undrained shear strength are proposed with acceptable accuracy.

List of symbols

σ_3'	Effective horizontal stress
σ_1'	Effective vertical stress
EPWP	Excess pore water pressure
K	Ratio between effective horizontal and vertical stresses = (σ_3'/σ_1')
K_0	Coefficient of earth pressure at rest
LB	Lower boundary
p'	Mean effective stress = $(\sigma_1' + 2\sigma_3')/3$
p_o'	Effective mean stress before shearing
p_c'	Pre-consolidation stress
q	Deviator stress = $(\sigma_1' - \sigma_3')$
q_{max}	Maximum shear strength
q_{res}	Residual shear strength
UB	Upper boundary
ε_a	Axial strain
Δq	Difference between the current and initial (i.e. before shearing) deviator stress
Δq_{LB}	Difference between the maximum deviator stress and the lower boundary

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1 Introduction

The geological evolution itself upon the sedimentation and consolidation of a soil results in an inherent in situ anisotropy, while under applied external stresses, the resulting strains and rearrangement of the soil particles also induce anisotropy in relation to stress, permeability and compressibility [8]. The stress anisotropy can have a significant effect on the undrained shear behaviour of soils, and this is the reason why considerable efforts have been made in the past to better understand its role [19, 29, 30, 38]. For example, Zdravković et al. [39] showed that failing to consider the effect of stress anisotropy on embankment stability analysis leads to an underestimation of horizontal

displacements. Moreover, Oda and Koishikawa [28] found that if the influence of stress anisotropy were ignored, then the bearing capacity of a shallow foundation would be overestimated, leading to a possible unsafe design. The influence of stress anisotropy is very important for understanding and interpreting the behaviour of transport infrastructure built on subgrade soft soil, apart from general slopes and foundation stability analysis as well as excavations, including tunnelling situations. Stress anisotropy also becomes a vital factor for post-compaction response of various soils including land reclamation projects. In relation to shallow railway subgrade, any reduction in confining pressure (i.e. decreased value of K) may exacerbate the lateral strains in the transverse direction (parallel to the sleepers), unless side restraints are installed in the track shoulders. In this respect, the stress anisotropy that is most appropriate in real-life design of a rail track should consider an optimum balance between the required subgrade shear strength and the acceptable lateral movement [16].

Previous studies have shown some contradicting results regarding the effect of anisotropy on the undrained shear strength. For instance, Mayne [24] indicated that the anisotropic shear strength of a clayey soil would be less and could be estimated as approximately 87% of the isotropic shear strength. In contrast, Hyodo et al. [13] found that the anisotropic shear strength of a normally consolidated clay can increase up to 77% when K decreases from 1 to 0.33 (i.e. initial deviator stress increases); however, the material becomes increasingly more brittle during its post-peak response. Similarly, other studies have shown that certain soil specimens, such as sandy clay and kaolin clay, experiencing a smaller value of K can achieve a higher shear strength requiring prudent attention in design [17, 34]. A reduction of K from unity (isotropy) can be sustained by either (i) decreasing the effective confining pressure or (ii) increasing the effective vertical stress. Depending on the method of changing K and the corresponding maximum deviatoric stress imposed on the test specimen, the implications on the undrained shear strength can be distinctly different in relation to the mode of imminent yielding/failure (i.e. shear banding, barrelling or a combination of both). Although this behaviour has been reported in past studies, for instance, the local bounding surfaces defining the effect of stress anisotropy in the 2D p'-q stress plan [17], the influence of initial stress anisotropy on the resulting plastic shear strain (magnitude and rate) and the generation of excess pore water pressure (EPWP) for different types of fine-grained soils has still been the subject of investigation in 3D stress space.

As most previous studies have mainly focused on limited soil types under specific anisotropic ratios within a limited range of plasticity index (PI), there is still sufficient scope to study how PI can influence the anisotropic shear behaviour. For example, Nakase and Kamei [25] revealed that the undrained shear strength of soils increased with the plasticity index (PI), but their findings were restrained to PI < 30 and $K \cong 0.42$. Mayne [24] investigated the anisotropic shear strength of various clayey soils but did not quantify the role of PI on the anisotropic shear behaviour. More recently, Won [35] compared the stress anisotropy of different clayey soils under triaxial extension and compression; however, a specific relationship between shear strength and PI of soils was not identified. In view of the above, a specific empirically based guide that enables the shear strength of soils to be estimated for a given set of PI and K can be most beneficial in practice.

The purpose of this study is to understand how the coupled effect of stress anisotropy and soil properties (represented by the Plasticity Index, PI) can influence the shear behaviour of normally consolidated soils so that more reliable and cost-effective designs can be accomplished in practice. For typical Australian's coastal conditions in low-lying regions of Eastern NSW where soft clayey soils are mostly normally consolidated to lightly over-consolidated (OCR < 1.4), $K < 1$ is measured for subgrade soils in situ [26, 27]. To this end, a clayey soil (PI = 20) was subjected to a series of undrained shear tests considering values of K ranging from 0.45 to 0.94, encompassing typical K values for normally consolidated soils in the state of NSW, Australia, which approximately follows Jaky's relationship for at-rest earth pressure coefficient ($K_0 = 1 - \sin \varphi$). The effects of varying K on the stress paths and the corresponding EPWP were analysed. Then, in comparison with various soils from past studies with different values of PI, some empirical equations are proposed to estimate the undrained shear strength capturing the combined role of K and PI.

Several past studies [7, 33, 36] have shown that the over-consolidation ratio can influence the anisotropic shear behaviour of soils. For example, the larger the OCR, the greater the shear strength as K decreases [7]. Indraratna et al. [14] showed that ignoring the over-consolidation ratio would lead to an overestimation of the yield stress and the corresponding undrained shear strength. Besides that, very dense sands and highly compacted soils, including boulder clays and highly over-consolidated glacial tills, can exhibit inherent anisotropic stress ratios significantly exceeding unity (e.g. Long and Menkiti [23], Potts et al. [31]). However, this paper is limited to the effect of anisotropy on normally consolidated soils of $K < 1$, which is applicable for most Australian coastal (estuarine) soils.

2 Experimental programme

2.1 Soil properties and preparation

A natural foundation soil (saturated subgrade known for undrained instability) beneath a low-lying rail track near south of Sydney was augured (i.e. 0.5–1.0 m depth). The soil had a liquid limit, LL = 38 and a plastic limit, PL = 18, thus PI = 20, with a specific gravity of 2.71 [2, 4]. Conventional sieve analysis [1] and laser diffraction method for particles finer than 0.075 mm indicated this soil as a clayey sand of low-moderate plasticity (i.e. SC; [3]) with sand, silt and clay contents of 58%, 15% and 27%, respectively.

In this study, slurry consolidation was chosen to prepare normally consolidated soil samples while simulating a natural subgrade foundation with respect to recent site investigations along the South Coast of NSW, Australia [26]. The collected soil was dried and carefully mixed with distilled and de-aired water at a moisture content of 1.2 times the LL. The slurry was then transferred into consolidation cells of 50 mm in diameter and 180 mm in height. The consolidation tests were carried out in incremental loading stages up to a vertical stress of 50 kPa in conformity with the in situ stress range for shallow subgrades beneath railways [15]. The specimens were extruded and trimmed to the standard size (50 mm × 100 mm) before being subjected to triaxial shearing.

2.2 Triaxial shear tests

The tests were conducted in a stress-path triaxial automated system. The cell and back pressures were applied by hydraulic pressure controllers with a range of 2 MPa, while the axial force was applied by a submersible load cell with 8 kN capacity. The axial displacement was externally measured at the top of the specimen using a linear variable differential transformer (LVDT) with ± 100 mm range. The pore pressures transducers had a pressure range of 1 MPa and were located at the top and bottom of the specimen, so the average value was used for the analysis. All the measuring components had an accuracy of 0.10–0.15% within the full range output as per the manufacture's specifications.

A total of 7 undrained triaxial tests were undertaken on anisotropically consolidated specimens with K varying from 0.45 to 0.94, including $K_0 = 0.50$. Initial geotechnical properties and consolidation parameters are given in Table 1. The test No. 7 is close to the isotropic condition ($K = 0.94$); however, a small deviator stress (i.e. 2 kPa) was applied to ensure that the specimen was docked throughout the test and to ensure that accurate height

variation during saturation, consolidation and shearing could be captured. The specimens were initially subjected to water flushing at an effective confining pressure of 5 kPa until a constant flow rate without any air bubbles was observed. The saturation was carried out by incrementally increasing the back pressure for 24 h up to 500 kPa, where it remained constant for over 12 h until Skempton's coefficient $B > 0.98$ was achieved. During consolidation, the stress paths were controlled to reach the target K ratio in the p'-q plane, where $p' = \left(\sigma'_1 + 2\sigma'_3\right)/3$ and $q = \sigma'_1 - \sigma'_3$. The K_0-consolidation test was undertaken with the aid of radial transducer monitoring to ensure a zero lateral strain while following the normally consolidated test conditions (i.e. σ'_1 = 50 kPa; σ'_3 = 25 kPa). For anisotropic cases, the axial and radial stresses were increased at incremental rates of axial and radial stresses ($\partial\sigma'_1/\partial t \cong 2$ kPa/h; $\partial\sigma'_3/\partial t \cong 1$ kPa/h), until the desired values of $p'_o \cong 34$ kPa (coinciding with pre-consolidation stress, p'_c) and K ratios were attained. Before shearing at a strain rate of 0.01 mm/minute, the test specimens were consolidated under constant stresses for about 24 h.

3 Results and discussion

3.1 Overall anisotropic undrained shear behaviour and critical state

The effective stress paths with K varying from 0.45 to 0.94 and the critical state line (CSL) are shown in Fig. 1a. The slope of the CSL ($M = 1.38$) was obtained independently from a series of isotropically consolidated tests at different confining pressures. The results show that as K decreases, the test specimen exhibits increased brittleness during shearing. Moreover, Fig. 1a shows that the soil specimen reaches a higher peak deviator stress when K is smaller, which agrees with several other soil types [10, 13, 32]. Overall, the anisotropic stress paths seem to reach the same CSL that is determined from the isotropic tests, which indicates the quasi-unique characteristics of CSL despite different levels of stress anisotropy.

The degree of anisotropy affects the contraction and dilation response of the soil. Figure 1b shows that the peak deviator stress (q_{max}) shifts gradually to the right when the initial state of soil becomes closer to the isotropic condition (i.e. $K = 1$). For example, q_{max} is reached quickly at a small shear strain, *i.e.* 0.03% for the highly anisotropic state ($K = 0.45$), but as K increases to 0.94, q_{max} is only achieved at $\varepsilon_a = 1.34\%$. This implies that the early contractive stage where q increases, becomes longer when the initial stress state of the soil specimen is closer to an isotropic condition. Since all the specimens had approximately

Table 1 Summary of tests

Test no.	Initial properties		Consolidation stresses			Summary of results		
	ρ_d (g/cm^3)	e_0	σ_1' (kPa)	σ_3' (kPa)	K	q_{max}	q_{res}	ε_a at q_{max} (%)
1	1.65	0.64	54	24	0.45	37.0	24.1	0.03
2	1.63	0.67	50	25	0.50	35.1	25.6	0.04
3	1.63	0.66	49	27	0.55	30.8	22.6	0.06
4	1.64	0.65	47	28	0.60	28.8	23.3	0.07
5	1.64	0.65	43	30	0.70	24.4	20.8	0.27
6	1.64	0.65	40	31	0.78	23.8	21.2	0.44
7	1.65	0.65	36	34	0.94	24.3	22.0	1.34
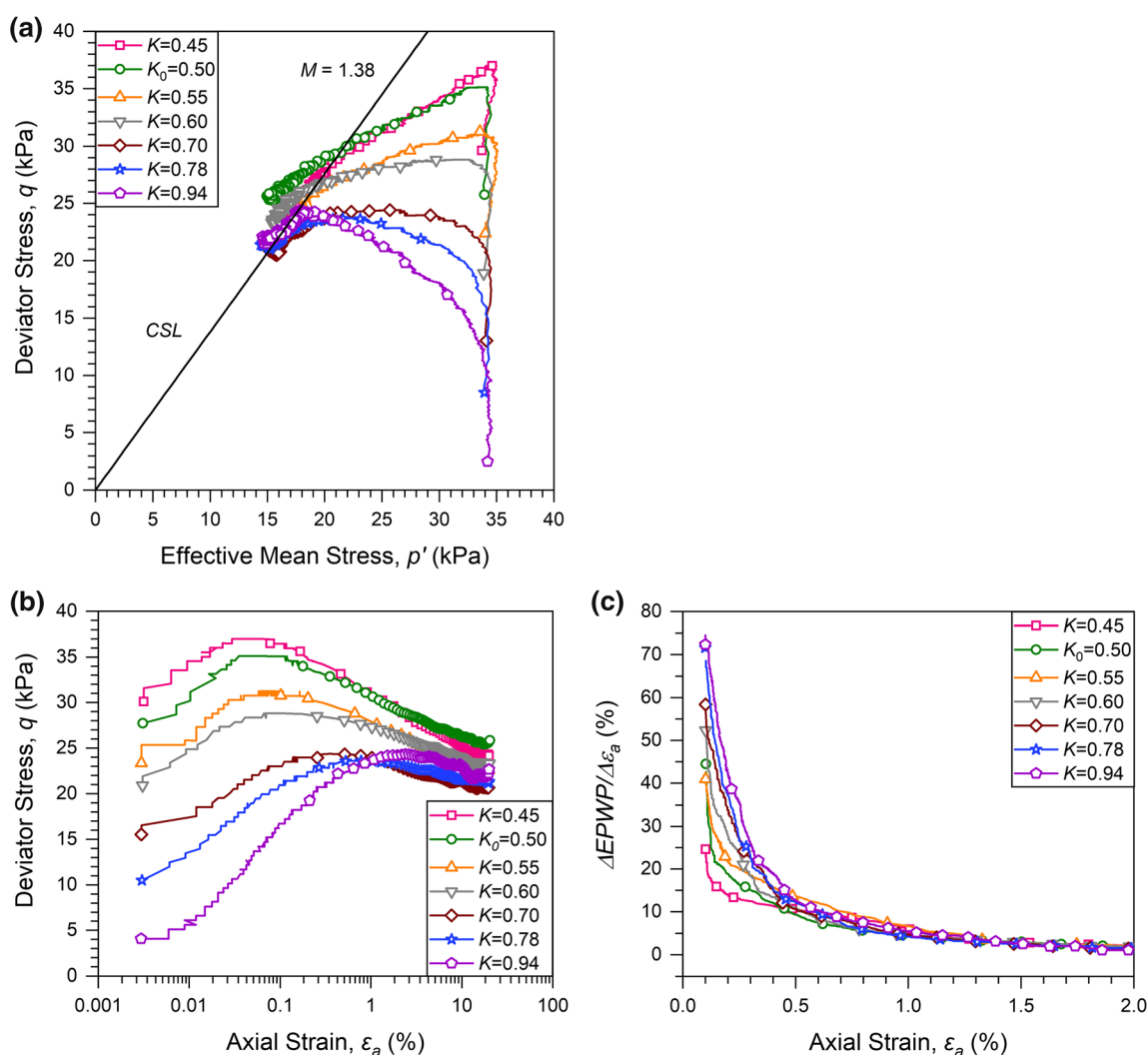
Fig. 1 Shear behaviour at different K **a** effective stress path **b** stress–strain curves **c** incremental rate of EPWP versus axial strain

the same void ratio before shearing ($e = 0.61$; Table 1—after consolidation), this change in soil response might have occurred because the initial shear stress in anisotropic

specimens could have favourably affected the soil fabric to enhance the peak shear strength. In fact, Li and Dafalias [22] explained that the soil fabric under anisotropic
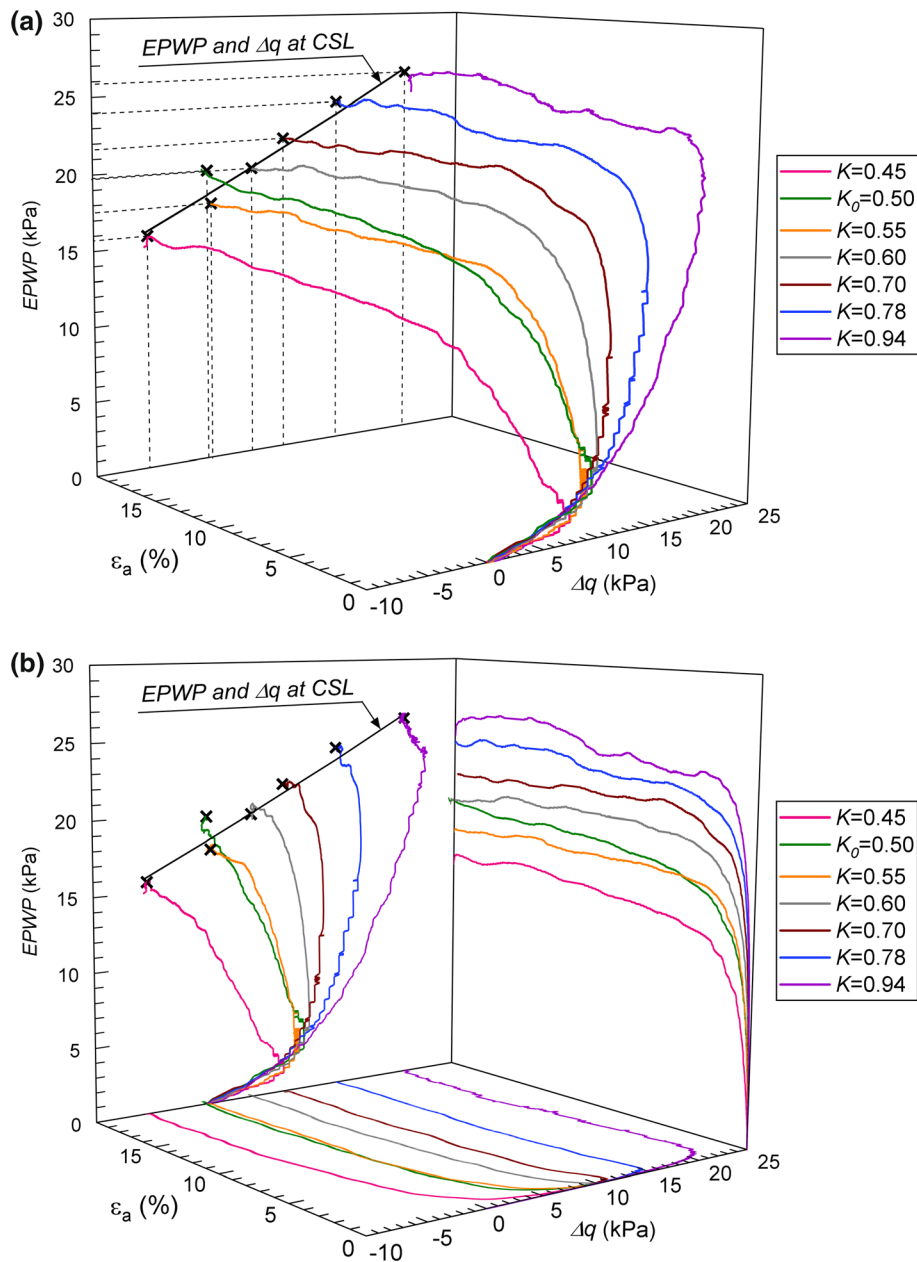
Fig. 2 3D bounding surface of anisotropic undrained shear: **a** 3-dimensional scale **b** projections to individual planes

condition is oriented towards the loading direction, thus promoting soil dilatancy while shortening the initially contractive response. Furthermore, Fig. 1c shows that the rate of increase in EPWP over the axial strain decreases as K decreases, which indicates that a greater portion of the shear stress is carried by the soil skeleton, resulting in an increased shear strength. It is noteworthy that all test specimens show post-peak barrelling, which is expected since the soil exhibits a dilatancy response regardless of the K value.

3.2 3D bounding surface of anisotropic undrained shear

Previous sections of the paper have shown stress paths (p' and q) and the corresponding EPWP of soil specimens sheared under different degrees of anisotropy. This section further links the soil behaviour in terms of q and EPWP while increasing the shear strain as illustrated in 3D (Fig. 2a, b). Here, the increases in EPWP and deviator stress (Δq) from the initial state before shearing were analysed, and it is confirmed that the lower the value of K,
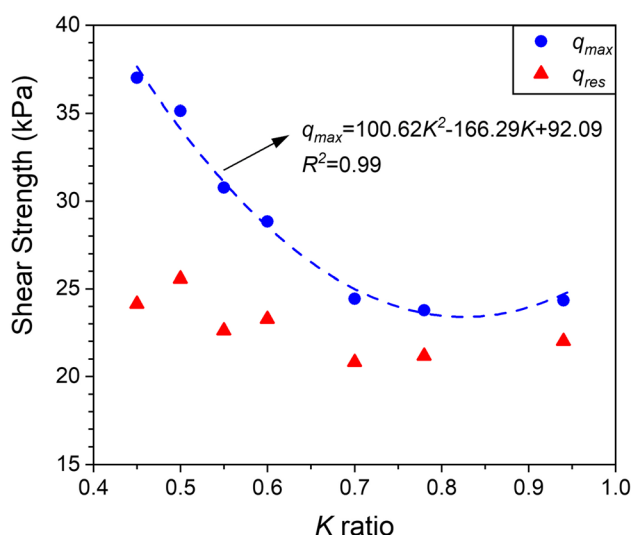
Fig. 3 Peak and residual shear strength at different values of K

the lower the magnitude of EPWP. For example, for $K = 0.45$, only 1.5 kPa of EPWP is developed in relation to the largest increment of deviator stress (Δq), whereas for $K = 0.94$, EPWP is about 15 times greater at the peak Δq. The peak EPWP also increases with rising K; however, they all reach a constant level as axial strain (ε_a) continues to increase, which represents a critical state where EPWP is almost constant despite increasing ε_a. The results indicate a 3D bounding surface where all the stress paths are confined under increasing shear strain despite the different values of K.

3.3 Shear strength of soil under varying K

Figure 3 presents the relationships between the peak shear strength (q_{max}) and residual shear strength (q_{res}—equivalent to the shear strength at the critical state), and the magnitude of K. The results show a distinct influence that

varying K has on q_{max}, whereas it does not indicate any significant effect on q_{res}. For example, q_{max} decreases considerably from 37 to 24.4 kPa when K increases from 0.45 to 0.70; however, q_{res} only decreases slightly from 24.1 to 20.8 kPa, given the same change in K. It is important to find that when K exceeds a threshold of approximately 0.70, q_{max} stabilises at around 24 kPa despite K increasing to 0.94. The larger q_{max} at a higher degree of anisotropy can be attributed to the soil fabric rearrangement under the initial shear stress of those anisotropic specimens. When a threshold value of $K > 0.7$ is attained, the assembly of soil particles due to initial shearing can approach an ultimate state where continuation of shearing does not lead to further significant rearrangement of particles. The existence of a critical state of fabric anisotropy has been indicated in past studies based on micromechanical observations and their impact on the macro-scale shear behaviour of soils [21, 22]. Besides that, at $K = 0.70$, q_{max} becomes close to q_{res}, which means that the peak shear strength is close to the ultimate value of shearing resistance (i.e. the critical state) where the observed shear stress becomes almost constant. Compared to past studies [7, 10, 13], the current study is novel in a way that it addresses the behaviour of q at both the peak and residual states. The limited influence of K on the residual strength was expected; this is because at the critical state the soil fabric would have been almost totally rearranged and, therefore, could only have a marginal influence on the shear behaviour (see Fig. 1a).

3.4 Influence of plasticity index

In order to understand how the anisotropic shear behaviour of soil would change with different values of PI, the results from this study were compared with selected past studies

Table 2 Soil properties of previous studies

| Studies	LL (%)	PI (%)	Soil type	Specimen preparation method	p_o' (kPa)
Hyodo et al. [13]	124	73	MH	Slurry consolidation	200
Donaghe and Townsend [9]	79	53	CH	Slurry consolidation	98–147
Ladd [20]	80	38	MH	Undisturbed	203–294
Donaghe and Townsend [9]	57	36	CH	Slurry consolidation	196–294
Khera and Krizek [18]	55	29	CH	Slurry consolidation	98
Bjerrum and Lo [5]	52	28	CH	Undisturbed	180–245
Stipho [32]	52	26	CH	Slurry consolidation	214
Ladd [20]	33	15	CL	Undisturbed	415–588
Gens [10]	25	12	SC	Slurry consolidation	233
Georgiannou [11]	–	–	Sand	Slurry consolidation	300
Hyodo et al. [12]	–	–	Sand	Dry pluviation	100
Yang and Pan [37]	–	–	Sand	Dry deposition	100
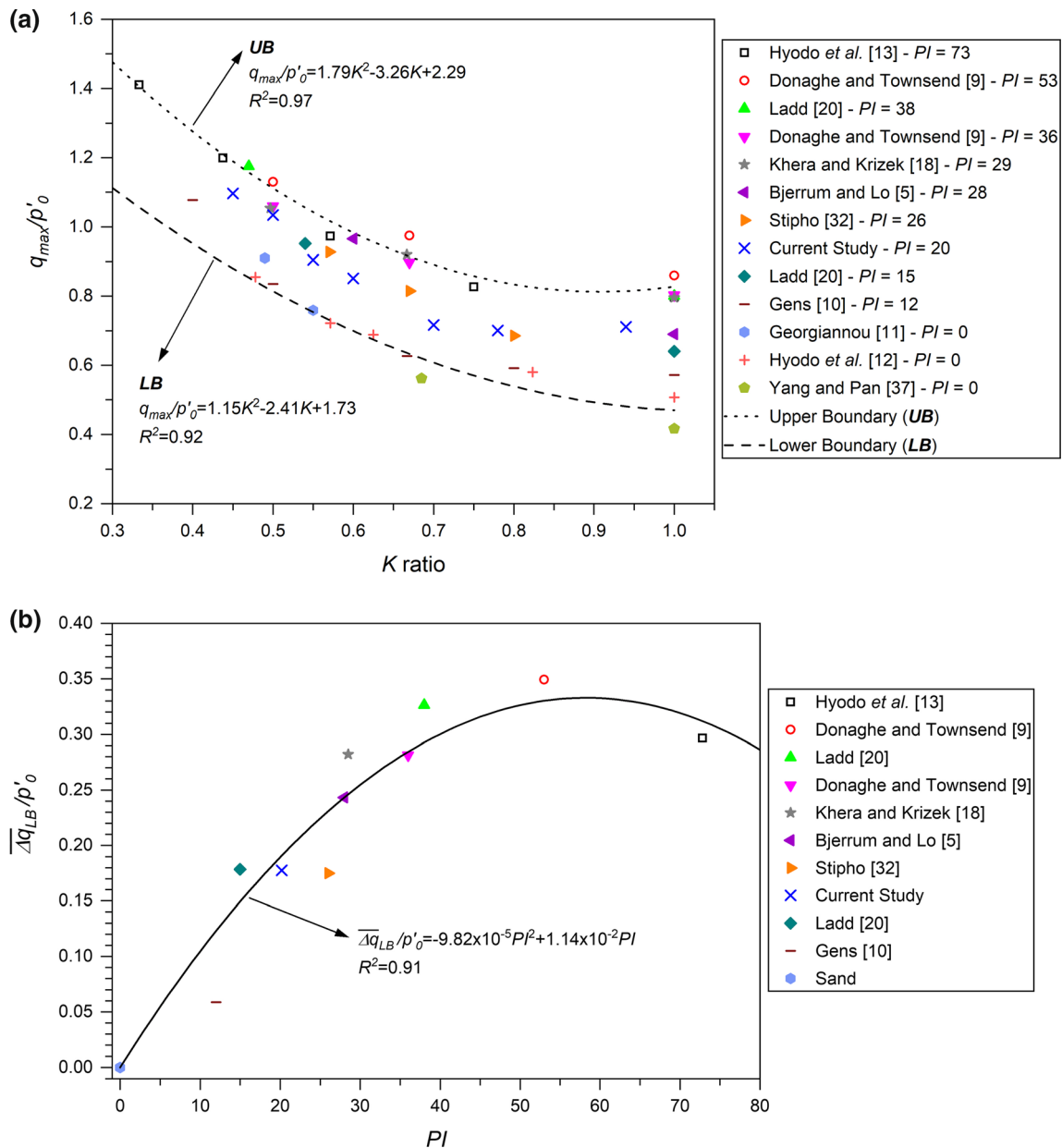
Fig. 4 **a** Boundaries of shear strength based on soil plasticity and anisotropy **b** Influence of plasticity index on shear strength

(Table 2), where normally consolidated soils of different PI were considered. To eliminate the influence of initial effective mean stress (p'_o), the peak shear strength q_{max} was normalised with p'_o. An interesting influence of soil plasticity on the relationship between the normalised shear strength and the value of K is shown in Fig. 4a. While all studies confirm that the shear strength decreases as K increases, there is a distinct range of shear strength established by two limit boundaries over different values of K and PI. The upper boundary (**UB**) is delineated based on the results of very high plasticity soils (PI = 53 and 73), whereas the lower boundary (**LB**) represents purely

cohesionless materials (sand) where PI is assumed to be near zero. The shear strength of all other soils (0 < PI < 73) lies within these boundaries. Furthermore, it is interesting to note that the relationship between the normalised q_{max} and K at different magnitudes of PI seems to be parallel to each other.

With the aim of eliminating the influence of K to assess the effect of PI on the shear strength, the average of $\Delta q_{LB}/p'_o$ (i.e. $\frac{\overline{\Delta q_{LB}}}{p'_o}$) is plotted against PI in Fig. 4b, where Δq_{LB} is the difference between q_{max} and the lower boundary **LB** across different soils. The results show that for those with PI < 35, the soil plasticity significantly
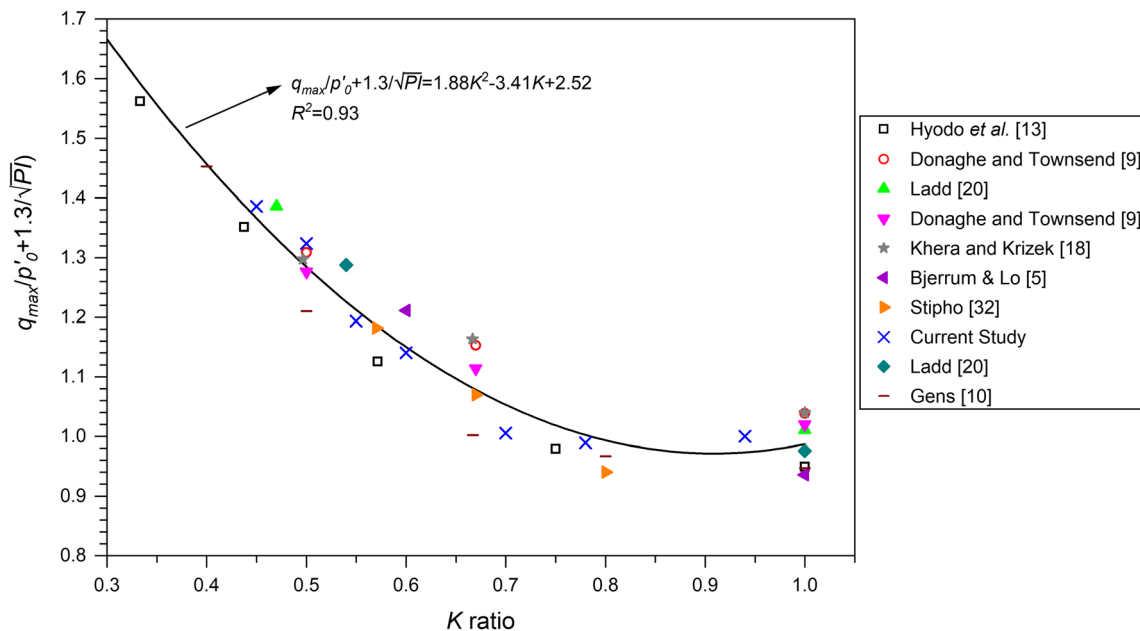
Fig. 5 Empirical relationship to predict shear strength

influences the shear strength, but when PI > 35, the influence of the plasticity index on anisotropic shear strength becomes marginal. Specifically, the ratio $\frac{\overline{\Delta q_{LB}}}{p'_o}$ increases from zero for cohesionless soils to approximately 0.3 when PI increases to 35, but it only varies in a narrow range, i.e. 0.3 to 0.35 as PI varies from 35 to 73. For soils with PI > 35, the role of cohesion in the overall shear strength becomes more significant than the frictional component. Therefore, the variation in undrained shear strength is less pronounced compared to soils with a lower plasticity index. For example, in a classical study, Bjerrum and Simons [6] have shown that the ratio between the undrained shear strength of clay to the in situ effective overburden stress increases approximately 100% for PI ranging from 10 to 40, while for PI varying from 40 to 70, this ratio only increases around 32%. An empirical relationship between $\frac{\overline{\Delta q_{LB}}}{p'_o}$ and PI is found with an acceptable agreement ($R^2 > 0.91$) as follows:

$$\frac{\overline{\Delta q_{LB}}}{p'_o} = -9.82 \times 10^{-5} \text{PI}^2 + 1.14 \times 10^{-2} \text{PI} \tag{1}$$
$$(\text{For PI} < 73)$$

The following empirical equation is proposed to provide an estimation of shear strength based on PI and K of soils:

$$\frac{q_{max}}{p'_o} = 1.88K^2 - 3.41K + 2.52 + \frac{1.3}{\sqrt{\text{PI}}} \quad (\text{PI} > 0; K < 1) \tag{2}$$

Figure 5 indicates that Eq. (2) provides a reasonably accurate estimation of shear strength compared to the experimental data ($R^2 = 0.93$) regardless of the specimen preparation technique adopted by different studies (Table 2), which offers a practical assessment of shear strength that can be adopted in preliminary design. This proposed simple calculation is beneficial to practising engineers, as it enables one to conveniently estimate the maximum (peak) shear strength of a soil of given plasticity index PI, based on its initial mean effective stress, p'_o, and the anisotropic stress ratio, K. It is of interest to realise that Fig. 5 shows independent data sets from several past studies where the initial void ratios of these soil specimens are distinctly different. However, after normalizing q_{max} with p'_o, it is clear that the role of e_o becomes insignificant. Therefore, the proposed Eq. (2) has significant practical value as it eliminates the influence of e_o when presenting the normalized shear behaviour. Figure 5 also indicates that the threshold value of K, above which q_{max} remains relatively constant, is slightly influenced by PI within a relatively narrow range of K = 0.7–0.8.

4 Conclusion

Considering the influence of plasticity index (PI), this paper presented in further detail the influence of stress anisotropy (K) on the undrained shear behaviour of soil. The following conclusions can be drawn.

- In relation to the stress paths, the findings showed that increasing K from 0.45 to 0.94 caused the peak shear strength to decrease with reduced brittleness.

- The rate that EPWP increased over a unit axial strain seemed to rise significantly (i.e. from 25 to 75 kPa) with increasing K during the initial stage, before reaching the same level of approximately zero when $\varepsilon_a > 1.5\%$.

- A threshold value of $K = 0.70$, above which the maximum (peak) shear strength remained relatively constant, has been identified for the soil investigated in this study. In comparison with previous studies, it has been shown that the threshold K is slightly influenced by PI and may vary within a relatively narrow range of $K = 0.70–0.80$.

- Plasticity index (PI) of soil had considerable effects on the corresponding anisotropic shear behaviour, i.e. the larger the PI, the larger the peak shear strength despite varying K; however, when PI exceeded 35, this relationship became marginal. The relationship between q_{max}/p'_o and K was found to be confined within a certain zone (named **UB** and **LB**) even though PI varied from 0 to 73.

- The proposed empirical equations to estimate soil shear strength based on PI and K showed an acceptable agreement with experimental data, and these relationships can serve as convenient initial guides for practising engineers.

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