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Investigation of Soil-Concrete Interface Behaviour by Simple Shear Apparatus

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SUMMARY This paper presents the results of a laboratory investigation into soil-concrete interface behaviour for application to the finite element analysis of retaining walls. The interface tests were carried out through the modification of the NGI simple shear apparatus. Three different soils and two concrete surfaces of different texture were examined. Some interface tests were also undertaken with a filter fabric. The experimental results show that for a particular soil-concrete interface, the shear stress-interface deformation plots for various normal loads can be characterised by a single curve through a normalisation procedure. The interface shear strength is dependent upon the concrete surface texture and the clay content of the soil. The results also show that large shear strains in the soil are necessary to fully mobilize the interface skin friction. The effects of interface dilation are found to be negligible.

1 INTRODUCTION

In the investigation of various geomechanics problems relating to piles, retaining walls and reinforced earth, it is important to take into consideration the interaction between the soil and the structure. A knowledge of the strength and stress-deformation behaviour of the interfaces is required if the analysis of such problems using the finite element technique is to prove meaningful.

Following the comprehensive work of Potyondy (1961) using the direct shear apparatus to study the soil-structure interaction behaviour of various soil types and construction materials, similar studies have been carried out by Clough and Duncan (1971), Kulhawy and Peterson (1979) and Acar et al. (1982) to obtain interface properties for the application of the finite element method to soil-structure interaction problems.

However, there are some shortcomings associated with the direct shear test. They include:- (i) Progressive shear failure at the ends and sides of the soil occur as a result of the non-uniformity of shear strains and shear stresses, (ii) the apparatus is unable to measure independently the shear strain of the soil and the slip along the interface, (iii) the shear box tends to tilt at light loads, (iv) tests cannot be conducted under constant volume conditions with determination of any change in effective normal stress, (v) the area of the contact surface is continually varying, and (vi) the volume change of the soil specimen may be inhibited by the vertical side friction of the shear box for tests conducted under constant normal stress conditions.

Some attempts have been made to overcome these limitations. Brumund and Leonards (1973) developed a test device in which a circular rod is inserted coaxially into a cylinder of soil surrounded by a light membrane, and the shearing stress is applied axially to the rod. Yoshimi and Kishida (1982) used a ring torsion apparatus to evaluate the skin friction characteristics between sand and a steel surface and studied the soil deformation with the aid of radiographic observations. However, in comparison with the direct shear apparatus, these laboratory techniques have the drawback of involving complex methods of sample preparation as well as being difficult to operate.

Therefore as part of a comprehensive project to investigate retaining wall problems using the finite element method, the authors have modified the NGI Direct Simple Shear Device (Bjerrum and Landva, 1966) to examine the soil-structure interface behaviour behind retaining walls. This device has the advantage of overcoming the limitations mentioned previously as well as being able to reproduce approximately the shear mechanism generally found behind actual retaining walls. In addition, the laboratory technique and the degree of effort required for sample preparation is not much more complex than that required for the direct shear apparatus.

As the simple shear device does not provide complementary shear stresses on the vertical sides of the specimen, the soil experiences a non-uniform shear stress on the top and bottom faces. However, finite element analyses carried out by Lucks et al. (1972) and the present authors show that approximately 70% of the sample is found to have a remarkably uniform stress distribution and the average shear stress is within 2% of the expected pure shear value. Hence, for practical purposes it is justifiable to interpret the tests as under pure shear conditions.

2 TEST PROCEDURE

2.1 Test Apparatus

A schematic representation of the modified NGI device is shown in Figure 1. The cylindrical soil specimen is constrained laterally by the wire reinforced rubber membrane. The specimen top platen is connected to a frame which keeps it parallel to the base during shearing, but allows it to move vertically. Slippage between the soil specimen and the top platen is minimised by using a uniformly rough grooved soil platen for sand materials, and a platen with attached pins which can be inserted into the specimen, in the case of clay materials. The height of the sample can be varied from 10mm to 40mm.

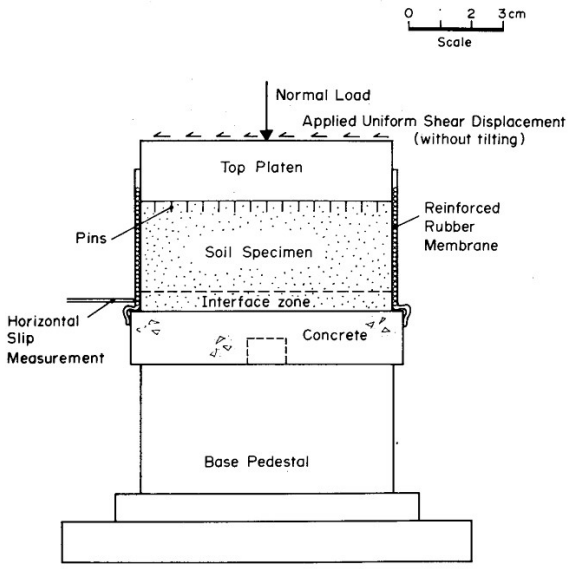


Figure 1. Schematic side view of the interface simple shear apparatus

A dial gauge with a minimum graduation of 0.001mm was used to measure the relative movement between the soil and the concrete. A rectangular bar was fastened to the sides of the testing rig, and eight holes were drilled which enable a depth micrometer with a minimum graduation of 0.001 inch to pass through and measure the movement at various points along the height of the specimen. Separate dial gauges were also used to measure the change in height of the soil specimen during shear to determine the volume change of the soil specimen, and to measure the horizontal translation at the top of the soil specimen.

Saturated interface tests were carried out by fastening a specially designed perspex cylinder to the base pedestal. A small hole was drilled into the side of the cylinder and fitted over with a thin rubber membrane. A slender horizontal pointer was attached to both sides of this membrane. One end of the pointer was kept in contact with the soil sample and a dial gauge was connected to the other end. This enabled the interface horizontal deformation to be measured.

2.2 Soil Samples

Three different materials, representative of the soils commonly found behind retaining walls, were tested. They were

(i) Frankston sand, a well graded sand,

(ii) moist clayey sand, consisting of 88% Frankston sand and 12% Monash clay (w=8%), and

(iii) sandy clay, consisting of 50% Frankston sand and 50% Monash clay (as compacted w=11%), tested flooded.

2.3 Sample Preparation

The sample preparation was similar to conventional simple shear testing, with the major difference being the simulation of conditions similar to those in the field. The Frankston sand specimens were prepared by the vibration technique to obtain a controlled medium dense sand. Both the clayey sand and sandy clay specimens were compacted to standard compaction test conditions in a specially designed square compaction mould into which the circular

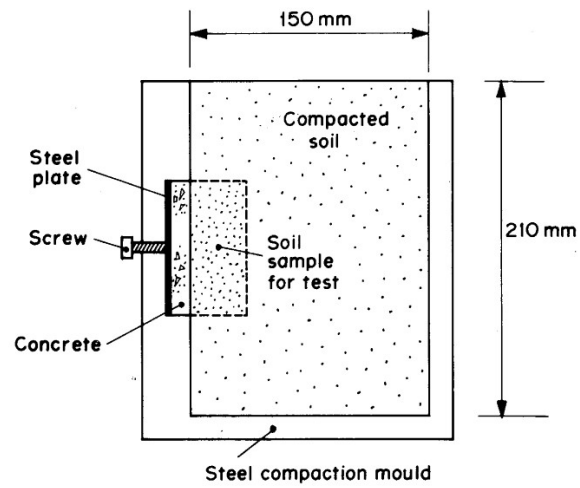


Figure 2 Side view of the compaction mould

concrete specimen was placed. This is illustrated in Figure 2.

In order to simulate the situation behind a retaining wall, the concrete specimen was placed vertically as shown and the soil was compacted parallel to the concrete surface. The compaction mould was then rotated so that the concrete surface was horizontal, the adjacent sides of the steel mould were detached and the soil trimmed to a suitable size. The screw and steel plate enabled the concrete and the soil sample to be removed without disturbing the soil-concrete adhesion. The soil specimen was then trimmed to the correct testing size and the membrane fitted onto it by a technique similar to normal simple shear sample preparations.

2.4 Interface details

The same Frankston sand was used as the aggregate in the preparations of the concrete specimens. Two different textures were examined. The first was cast and placed on a perspex plate to obtain a smooth finish. To produce a slightly rough texture in the second, the concrete was cast in the mould and allowed to set without any formwork on the surface. Studies were also carried out with a 7mm thick filter fabric (ICI FILTRAM Type 1B1 - consisting of 2 layers of synthetic fibre separated by a structured plastic mesh), placed in position on the concrete prior to compaction and sandwiched between the soil specimen and the concrete.

2.5 Testing Details

The procedure for the testing was exactly similar to conventional simple shear testing. A uniform deformation rate of 0.066 mm min^{-1} was used. All the tests were undertaken with constant normal stress conditions i.e. the top loading platen was free to move vertically but was restrained to remain horizontal at all times.

3 RESULTS

Typical results of the tests on the Frankston sand and the sandy clay are shown in Figures 3 and 4 respectively. The behaviour for the clayey sand material was found to correspond closely to Frankston sand and graphs are not presented here. Figure 5 shows typical results of tests carried out with the filter fabric interface.

The skin friction-normal stress (τ-σ_n) plots are shown in Figures 6 and 7.

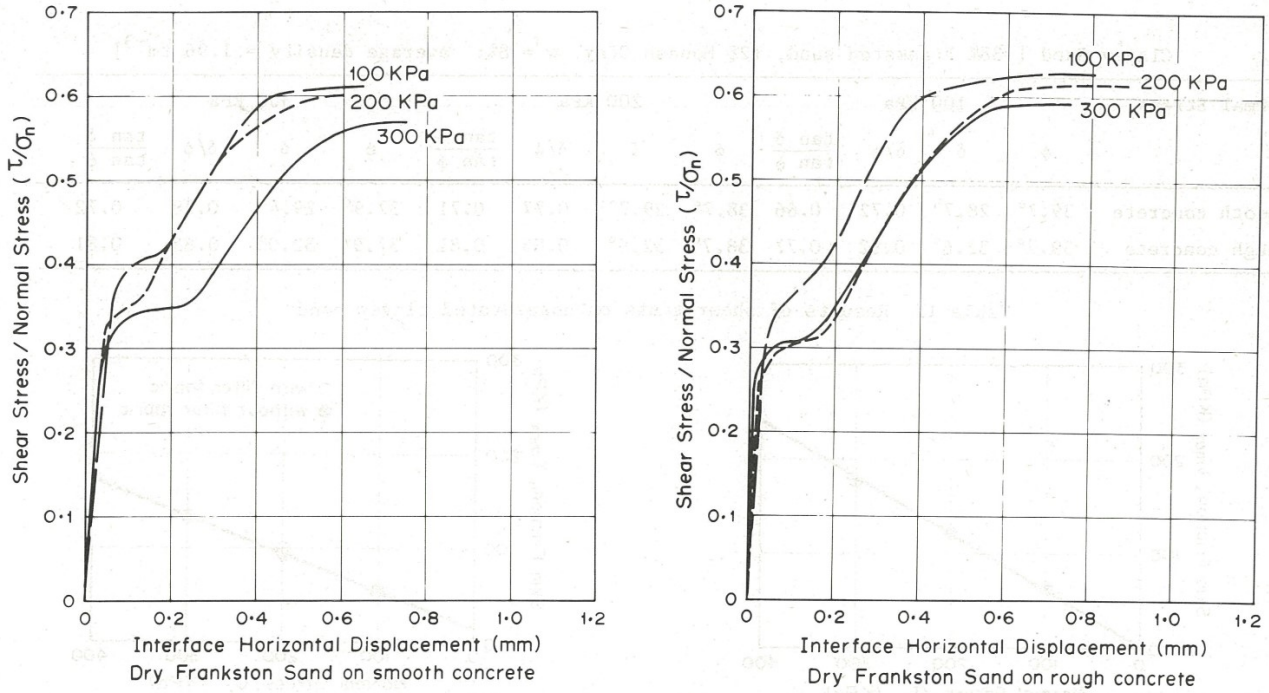


Figure 3 Typical tests on Frankston sand

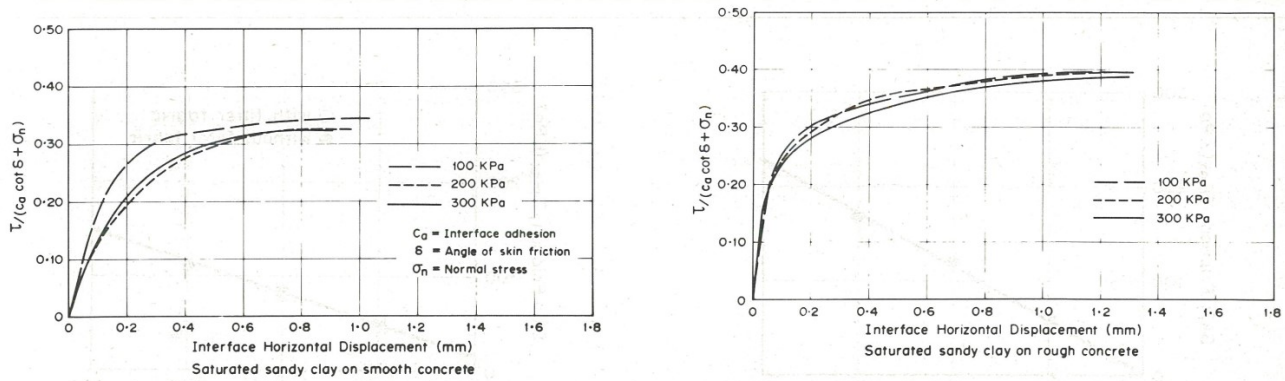


Figure 4 Typical tests on sandy clay

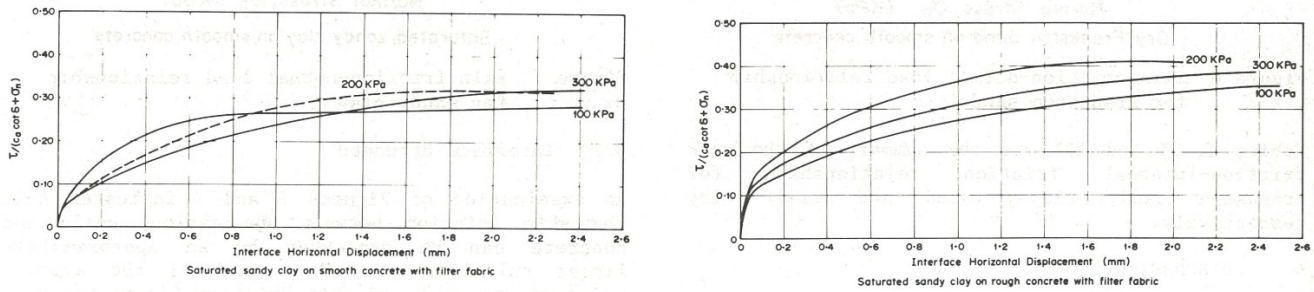


Figure 5 Typical tests on sandy clay with filter fabric

Frankston sand [Average density = 1.76 tm^{-3}]

Normal Stress	100 kPa				200 kPa				300 kPa			
	ϕ	δ	δ/ϕ	$\frac{\tan\delta}{\tan\phi}$	ϕ	δ	δ/ϕ	$\frac{\tan\delta}{\tan\phi}$	ϕ	δ	δ/ϕ	$\frac{\tan\delta}{\tan\phi}$
Smooth concrete	41.0°	31.0°	0.76	0.69	39.5°	30.8°	0.78	0.73	38.0°	29.8°	0.78	0.73
Rough concrete	41.0°	31.7°	0.77	0.71	39.5°	31.4°	0.80	0.74	38.0°	30.7°	0.81	0.76

Table I Results of shear tests on dry Frankston sand

Clayey Sand [88% Frankston sand, 12% Monash Clay, w = 8%; average density = 1.96 tm^{-3}]

Normal Stress	100 kPa				200 kPa				300 kPa			
	ϕ	δ	δ/ϕ	$\frac{\tan\delta}{\tan\phi}$	ϕ	δ	δ/ϕ	$\frac{\tan\delta}{\tan\phi}$	ϕ	δ	δ/ϕ	$\frac{\tan\delta}{\tan\phi}$
Smooth concrete	39.7°	28.7°	0.72	0.66	38.7°	29.7°	0.77	0.71	37.9°	29.4°	0.78	0.72
Rough concrete	39.7°	32.6°	0.82	0.77	38.7°	32.8°	0.85	0.81	37.9°	32.0°	0.85	0.81

Table II Results of shear tests on unsaturated clayey sand

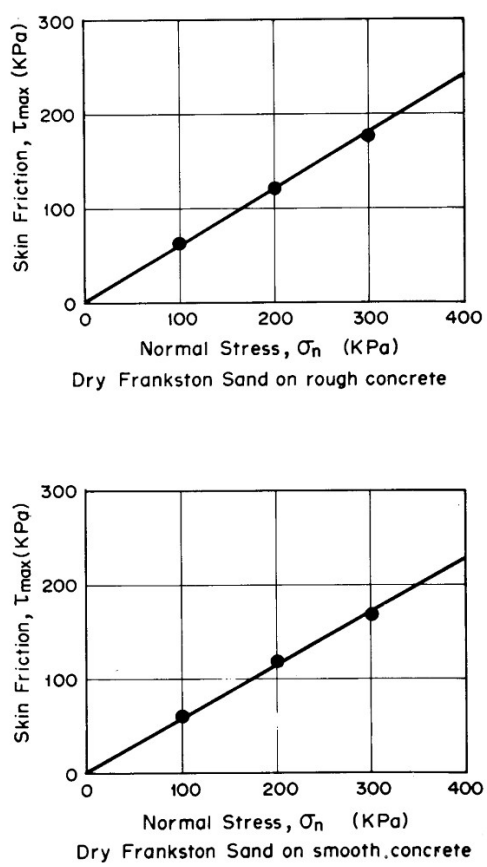


Figure 6 Skin friction-normal load relationship for Frankston sand

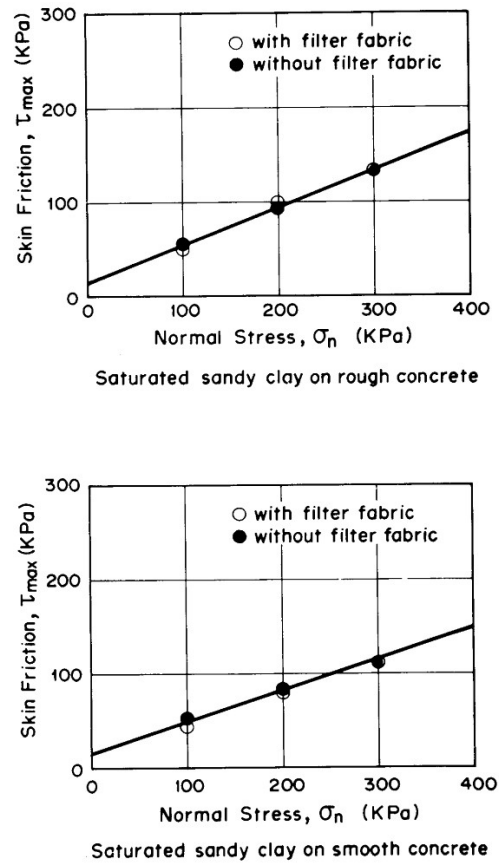


Figure 7 Skin friction-normal load relationship for sandy clay

Tables I, II and III are the summary of the peak friction-internal friction relationships for Frankston sand, clayey sand and sandy clay respectively.

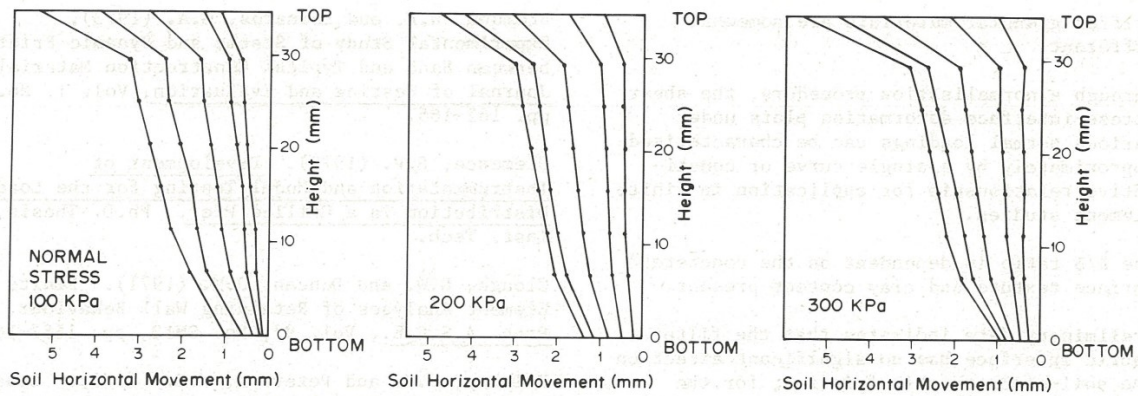
4 DISCUSSION

4.1 Shear Stress-Deformation

Figures 3 and 4 show that the normalisation of the shear stress-interface deformation plots for various normal loadings results in almost identical curves. Consequently, the stress-interface deformation behaviour for each soil-concrete interface can be characterised by a single relationship for utilisation in finite element studies. The results also demonstrate that the cohesionless and clayey sand soils exhibit a very stiff initial response followed by a small amount (approximately 0.1mm) of sliding. It is unlikely that this slip phenomenon will occur in actual field situations as the height of the retaining wall involved will be considerable.

4.2 Interface Strength

An examination of Figures 6 and 7 indicates that the skin friction between the various soils and concrete can be expressed by an approximately linear relationship. From Table I the average value of the angle of skin friction (δ) to internal friction angle (ϕ) for smooth concrete is 0.77. This is in good agreement with the value of δ/ϕ=0.76 and δ/ϕ=0.80 for dry dense sand reported by Potyondy (1961) and Clemence (1973) respectively. The δ and ϕ values generally decreased slightly as the load was increased, with the ϕ value being more sensitive to the load than δ. A comparison of Tables I and II illustrates that the addition of a small amount of clay to the sand is only significant in the case of the rough concrete material. Table III further exemplifies the fact that increasing the composition of the clay in the mixture decreases the internal friction of the soil and results in the lowering of interface shear strength.



Clayey sand on rough concrete

Figure 8 Typical results of soil profile measurements

Sandy clay [50% Frankston sand, 50% Monash clay, w = 11%; density = 2.15 t/m^{-3}]

	c or c_a kPa	δ or φ	c_a/c	δ/φ	$\frac{\tan\delta}{\tan\phi}$
Soil	16.2	28.0°	-	-	-
Smooth concrete	14.0	18.3°	0.86	0.65	0.62
Rough concrete	15.0	21.0°	0.93	0.75	0.72
Smooth concrete and filter fabric	14.0	18.3°	0.86	0.65	0.62
Rough concrete and filter fabric	15.0	21.0°	0.93	0.75	0.72

Table III Results of shear tests on saturated sandy clay

The ratios of the adhesion to cohesion (c_a/c) and δ to φ obtained from Table 3 show a somewhat distinct variation from Potyondy's reported data (c_a/c=0.42, δ/φ=0.84, for smooth concrete; c_a/c=0.80, δ/φ=0.95 for very rough concrete) for a cohesive granular soil (50% sand, 50% clay). This can be attributed to the compaction of the sandy clay against the concrete resulting in a higher adhesion strength.

4.3 Filter Fabric

As shown in Figures 5,7 and Table III, the addition of the filter fabric between the soil and the concrete does not appear to have any appreciable effects on the shear strength of the soil, in spite of the less stiff shear stress-interface deformation response. However, further tests with other soil materials will have to be made before any definite conclusions can be drawn.

4.4 Soil Deformation

The typical behaviour of the soil sample under various loadings as it is forced to move horizontally at the top is shown in Figure 8. The shear strain is relatively uniform throughout except near the ends. The soil profile indicates that the interface shear zone is less than 6.0mm Although very little movement (1.0mm) at the interface is sufficient to fully mobilise the skin friction, a comparison with typical shear stress-sample shear strain curves in Figure 9 shows that the shear strain of the whole sample ranges from 7% for clayey sand to 28% for sandy clay. As only about 0.2% horizontal translation (Terzaghi, 1934) of the retaining wall is sufficient for the backfill soil to reach the active pressure state, it is unlikely that interface shear strength will be

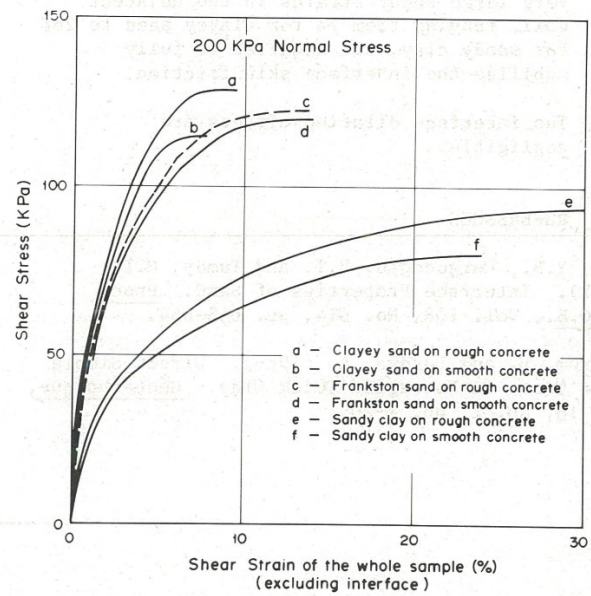


Figure 9 Typical shear stress vs. shear strain behaviour for soil alone

fully mobilised in either actual field situations or in finite element analysis problems.

In the comprehensive set of tests carried out on the dense soil samples it was found that the dilation effects were negligible (change in vertical height/original height ≈ 1.0%), and can therefore be ignored in any finite element formulation.

5 CONCLUSIONS

The modified simple shear apparatus provides a useful means for studying the strength and stress-deformation behaviour of various soil-structure interfaces, from which the relevant material parameters can be derived for implementation into finite element analyses involving soil-structure interaction.

On the basis of the results presented herein the following conclusions can be drawn:

(i) The interface shear strength results are generally consistent with results reported in the literature, even though the stress-interface deformation responses for the

stiffer granular materials are somewhat different.

(ii) Through a normalisation procedure, the shear stress-interface deformation plots under various normal loadings can be characterised approximately by a single curve or constitutive relationship for application in finite element studies.

(iii) The δ/ϕ ratio is dependent on the concrete surface texture and clay content present.

(iv) Preliminary data indicates that the filter fabric interface has no significant effect on the soil-concrete skin friction, for the sandy clay material.

(v) The skin friction-normal stress relationship is approximately linear.

(vi) Very large shear strains in the adjacent soil, ranging from 7% for clayey sand to 28% for sandy clay, are required to fully mobilise the interface skin friction.

(vii) The interface dilation effects are negligible.

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