CONSTANT PRESSURE AND CONSTANT VOLUME DIRECT SHEAR TESTS ON REINFORCED SAND

JIN-YING QIUⁱ⁾, FUMIO TATSUOKAⁱⁱ⁾ and TARO UCHIMURAⁱⁱⁱ⁾

ABSTRACT

In order to investigate the deformation and strength characteristics of reinforced sand, a series of relatively large-scale direct shear tests was performed. Phosphor bronze strips and sheets with different configurations were used as reinforcement to study the effects of the shape and dimensions of reinforcement. Constant pressure tests and constant volume tests were conducted with different forms of reinforcement and different values of stiffness, surface area, spatial dispersion and surface friction. Test results show that for a given amount of reinforcement material, the reinforcing efficiency is higher with reinforcement having a larger surface area, a rougher surface texture and a larger degree of dispersion. The dilatancy at the residual state depends on the reinforcement property and geometry, and it is larger in more effectively reinforced sand with a larger shear zone. More dilative characteristics of reinforced soil results in larger shear strength under constant volume conditions at large strains.

Key words: (constant-pressure test), (constant-volume test), dilatancy, direct shear test, reinforced soil, shear band, shear strength (IGC: D6)

INTRODUCTION

Soil reinforcement is one effective method to increase the strength of soil, and it has been widely used in practice. Different types of reinforcement are used, in form (grid, bar, strip, sheet, etc.), in material stiffness (high such as metal or low such as polymeric fabrics), in texture (rough or smooth surface) and others. Different types of reinforcement may exhibit different reinforcement mechanisms. For stiff bars placed in stiff soil, for example, the bending stiffness may contribute to strength increase at large strains. For grids, the large pull-out strength may be the most important feature. Though the effects of soil reinforcement are well recognized, the basic mechanism, in particular that related to the dilatancy characteristics of reinforced soil, is not well understood.

The direct shear test is one of the basic testing methods to study the failure mechanism of soil, though it is often criticized for non-uniform stress states within the specimen and development of a forced shear plane. This kind of test is suitable, however, to investigate the interaction of shear band (or shear zone) with reinforcement. A number of experimental investigations on the behavior of reinforced sand by means of direct shear tests have been reported (Gray and Ohashi, 1983; Jewell and Worth, 1987; Shewbridge and Sitar, 1989; Palmeira and Milligan, 1988; Bauer and Zhao, 1993, among others). Some

main findings from these studies are as follows:

- (a) The shear strength envelope in terms of average normal and average shear stresses for a given specimen has a bilinear form, showing the existence of a threshold or critical normal stress, which is predicted as a break point between slippage of reinforcement with regard to the surrounding soil and rupture of reinforcement without slippage when being sheared. The value of critical normal stress and the corresponding shear strength increase depend on reinforcement property, but shear strength envelopes are parallel to each other for normal stresses above the critical normal stress.
- (b) The maximum reinforcing efficiency is obtained when the direction of reinforcement coincides with the direction of the potential maximum tensile strain in unreinforced soil (Jewell and Worth, 1987). Bauer and Zhao (1993) considered the effects of changes in the reinforcement orientation inside a shear zone in their modeling.
- (c) The shear displacements at peak shear stress and at residual stress in reinforced sand are larger than those with unreinforced sand.
- (d) A larger amount of volume expansion occurs in the post-peak regime in reinforced sand (Palmeira and Milligan, 1988; Gray and Ohashi, 1983). The width of the shear zone of reinforced sand is not distinct

ⁱ⁾ Lecturer, Wuhan University of Hydraulic and Electric Engineering (Formerly Visiting Researcher at University of Tokyo).

ⁱⁱ⁾ Professor, Faculty of Engineering, University of Tokyo.

ⁱⁱⁱ⁾ Research Assistant, ditto.

Manuscript was received for review on December 18, 1998.

Written discussions on this paper should be submitted before March 1, 2001 to the Japanese Geotechnical Society, Sugayama Bldg. 4F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101-0063, Japan. Upon request the closing date may be extended one month.

Fig. 1. Schematic diagram of prestressed reinforced soil structure

(Shewbridge and Sitar, 1989).

The first object of the present study is therefore to investigate the effects of the stiffness, surface area, surface texture and spatial dispersion of reinforcement on the shear strength and volume characteristics of reinforced sand. This is essential to understand the load-transfer mechanism of reinforcement-sand composites. To this end, phosphor bronze trips and sheets having various values of width, thickness and roughness were used as the model reinforcement. This material was selected, as it has a well-defined linear elastic property, which makes the analysis of test results simpler than it would be under otherwise the same conditions. Its use also makes it possible to reliably evaluate tensile force in reinforcement by strain measurements when necessary. The second objective is to understand the failure mechanism of reinforced soil under constant volume conditions, or under conditions where the volume change is highly restricted, as with submerged reinforced soil structures subjected to seismic loads, or reinforced soil structures that are prestressed as shown in Fig. 1 (Tatsuoka et al., 1997; Uchimura et al., 1998). In the latter case, in particular, the lateral and vertical expansion of the structure is restricted by, respectively, laterally placed reinforcement layers and vertical prestressing system (vertical tie rods with top and bottom reaction blocks).

Herein described are a direct shear apparatus developed for the present study, test results from a comprehensive series of constant pressure and constant volume direct shear tests on unreinforced and reinforced air-dried Toyoura sand and some analysis of test results.

DIRECT SHEAR APPARATUS

A number of problems with the conventional type direct shear device, which may seriously affect test results, include: (a) rotation of the shear box, (b) unknown effects of the opening between the top and bottom halves of the shear box, and (c) effects of side-wall friction. Some features of different types of direct shear device were discussed by Shibuya et al. (1997). The top half box that sustains the applied lateral shear load tends

to rotate as the applied shear load develops large moment with respect to the shear plane. If the compensating moment is not applied to the top shear box (and/or the top loading platen placed on the top of specimen), it (and/or the loading platen) will rotate with a non-uniform stress distribution along the shear plane, making the actual strength and deformation characteristics of the specimen difficult to evaluate. The problem associated with the side-wall friction can be essentially alleviated if a proper method is used to measure the total shear and normal loads acting on the shear plane (as in the present study).

A medium size shear box apparatus was designed in the present study with efforts to avoid the above mentioned problems. The inside specimen size is nominally 30 cm × 30 cm × 30 cm. Figure 2 shows its schematic diagram. It has the following features:

- (a) Two pairs of air cylinders (four in total) are used to apply the vertical load and compensating moment. Each pair is controlled independently by means of a computer servo system to keep the top loading platen and the upper shear box level by compensating for the moment of applied lateral shear load and, at the same time, maintain the total vertical load at a desired value.
- (b) Sponge tape is glued to the upper periphery of the lower box to prevent sand from the opening during shearing. As sponge compresses and swells easily, it exhibits very small vertical and shear forces if not extremely pressed. As shown in the figure inset in Fig. 2, a thin but stiff steel channel member is placed over the passive side sponge tape to sustain some shear load on the passive vertical plane for the thickness of the opening, while preventing damage to the sponge. In this way, the inside volume of the shear box does not increase because of shear displacement. The volume change occurring in the specimen is then due solely to the dilatation or contraction of sand.
- (c) Vertical load is applied on the whole top area of the upper shear box through a massive steel loading platen, not only on the top of the inside soil specimen. Despite the above side-wall friction that may develop, because the upper shear box can move simultaneously with the specimen in the vertical direction, the total vertical load measured (plus the weights of upper shear box including sand and loading platen) is essentially the same as the value applied on the shear plane of the specimen.
- (d) On the bottom of the lower shear box, nine load cells are placed in three parts to measure distribution of normal and shear stresses working in the shear direction, despite the fact that these stresses are largely affected by the side-wall friction.
- (e) In addition to constant pressure tests, constant volume tests can also be performed using the computer servo loading system. The air pressures of the two pairs of vertical air cylinders are controlled separately so that the vertical displacement of the specimen is kept to zero. In tests on highly dilative

Fig. 3. Results from typical unreinforced (U) and reinforced (R) sand tests

not using a set of a sponge piece and a steel channel member at the passive side of opening, with an initial opening of 6 mm, are shown in Fig. 4. The shear stress τ_{vh} continuously dropped associated with vertical compression of the specimen during “the residual state”. This was due

to an undesirable increase in the specimen volume by the amount of “shear displacement $\times$ specimen width $\times$ opening”.

TEST PROGRAM

Test Sand and Reinforcements

Air-dried Toyoura sand was used in all the tests. It has a mean grain diameter (D_{50}) of 0.17 mm and a coefficient of uniformity (C_u) of 1.70. The minimum and maximum void ratios are 0.597 and 0.977, respectively. Sand specimens were prepared by the air-pluviation method using a multi-layer mesh hopper. The specimen density was computed based on the total weight of sand after each test. 30 cm-long phosphor-bronze strips (or sheets) of different forms were used as reinforcement. Considering many difficulties in preparing homogenous specimens with inclined reinforcement, the reinforcement was installed vertically for all the reinforced sand tests. Only dense specimens were reinforced in the present study.

Test Series

Three series of reinforced sand tests were performed to investigate the effects of the shape and dimensions of reinforcement on the stress and deformation characteristics of reinforced sand (see Table 1 and Fig. 5)

- (a) **RA series:** Two families were prepared; (a) RA3-1 through RA3-5, and (b) RA1-1 and RE5. The reinforcements in each family had the same total surface area (or covering ratio); but with different thicknesses. Thus the reinforcements had different values of longitudinal and bending stiffness. Family (a) had a covering ratio that was larger by three times than that of family (b).
- (b) **RE series:** The reinforcements had the same total

Fig. 4. Results from a preliminary test showing inadequate treatment of opening

Fig. 5. Horizontal cross-section of reinforcements

Table 1. Summary of test conditions for direct shear tests

Test type	Test code	Phosphor bronze reinforcements (Texture; thickness $\times$ width $\times$ number)	Stiffness (EA)	Covering ratio (CR)	Initial void ratio (e_i)	Comments
Constant pressure RA series	URP5	Unreinforced	Non	Non	0.6169	Stress-strain relationship of this test is presented.
	RA1-1	Smooth; 0.1 mm $\times$ 60 mm $\times$ 1	1 (unit)	0.2	0.6140	In total, 28 tests were performed.
	RA3-1	Smooth; 0.1 mm $\times$ 60 mm $\times$ 3	3	0.6	0.6183	
	RA3-2	Smooth; 0.2 mm $\times$ 60 mm $\times$ 3	6	0.6	0.6262	
	RA3-3	Smooth; 0.3 mm $\times$ 60 mm $\times$ 3	9	0.6	0.6287	
	RA3-5	Smooth; 0.5 mm $\times$ 60 mm $\times$ 3	15	0.6	0.6207	
	RAPR1-1	Sand-glued; 0.1 mm $\times$ 60 mm $\times$ 1	1	0.2	0.6197	
	RAPR3-1	Sand-glued; 0.1 mm $\times$ 60 mm $\times$ 3	3	0.6	0.6186	
	RAPR3-2	Sand-glued; 0.2 mm $\times$ 60 mm $\times$ 3	6	0.6	0.6251	
	RAPR3-3	Sand-glued; 0.3 mm $\times$ 60 mm $\times$ 3	9	0.6	0.6265	
	RAPR3-5	Sand-glued; 0.5 mm $\times$ 60 mm $\times$ 3	15	0.6	0.6240	
Constant pressure RE series	RE1	Smooth; 0.1 mm $\times$ 300 mm $\times$ 1	5	1.0	0.6228	
	RE2	Smooth; 0.2 mm $\times$ 150 mm $\times$ 1	5	0.5	0.6281	
	RE3	Smooth; 0.3 mm $\times$ 100 mm $\times$ 1	5	0.33	0.6228	
	RE4	Smooth; 0.4 mm $\times$ 75 mm $\times$ 1	5	0.25	0.6310	
	RE5	Smooth; 0.5 mm $\times$ 60 mm $\times$ 1	5	0.2	0.6217	
	REPR1	Sand-glued; 0.1 mm $\times$ 300 mm $\times$ 1	5	1.0	0.6244	
	REPR2	Sand-glued; 0.2 mm $\times$ 150 mm $\times$ 1	5	0.5	0.6189	
	REPR3	Sand-glued; 0.3 mm $\times$ 100 mm $\times$ 1	5	0.33	0.6263	
	REPR4	Sand-glued; 0.4 mm $\times$ 75 mm $\times$ 1	5	0.25	0.6249	
	REPR5	Sand-glued; 0.5 mm $\times$ 60 mm $\times$ 1	5	0.2	0.6221	
Constant pressure RD series	RD1	Smooth; 0.2 mm $\times$ 180 mm $\times$ 1	6	0.6	0.6205	In total, 7 tests were performed.
	RD3	Smooth; 0.2 mm $\times$ 60 mm $\times$ 3	6	0.6	0.6262	
	RD4	Smooth; 0.2 mm $\times$ 45 mm $\times$ 4	6	0.6	0.6234	
	RD30	Smooth; 0.2 mm $\times$ 6 mm $\times$ 30	6	0.6	0.6188	
	RDPR1	Sand-glued; 0.2 mm $\times$ 180 mm $\times$ 1	6	0.6	0.6223	
	RDPR3	Sand-glued; 0.2 mm $\times$ 60 mm $\times$ 3	6	0.6	0.6251	
	RDPR30	Sand-glued; 0.2 mm $\times$ 6 mm $\times$ 30	6	0.6	0.6212	
Constant volume	URV4	Unreinforced	Non	Non	0.6215	
	RAV3-5	Smooth; 0.5 mm $\times$ 60 mm $\times$ 3	15	0.6	0.6338	
	RAPRV3-5	Sand-glued; 0.5 mm $\times$ 60 mm $\times$ 3	15	0.6	0.6323	
	REPRV3	Sand-glued; 0.3 mm $\times$ 100 mm $\times$ 1	5	0.33	0.6228	
	REPRV5	Sand-glued; 0.5 mm $\times$ 60 mm $\times$ 1	5	0.2	0.6314	
	RDPRV1	Sand-glued; 0.2 mm $\times$ 180 mm $\times$ 1	6	0.6	0.6237	
	RDPRV30	Sand-glued; 0.2 mm $\times$ 6 mm $\times$ 30	6	0.6	0.6348	

cross-section area (or longitudinal stiffness); but with different thicknesses. Thus the reinforcements had different values of surface area and covering ratio.

- (c) **RD series:** The reinforcements had the same total cross-sectional area, thickness and surface area, but each reinforcement of the RD series was split into different numbers of strips.

For all the three series, constant pressure tests of reinforced sand were performed with either a smooth or perfectly rough (sand-glued) surface of reinforcement. The constant volume tests were performed only on a limited number of specimens. For constant pressure tests on reinforced sand specimens, the average vertical pressure σ_v was 50 kPa. The initial vertical stress for the constant volume tests was selected as a lower value of 20 kPa, considering a high vertical stress increase during shearing and the limited load capacity of the loading. Table 1 is a summary of all the reinforced sand tests. In addition, a

series of unreinforced sand tests with various void ratios ($e_i=0.61\sim0.86$) and normal stresses ($\sigma_v=50\sim200$ kPa) were performed as the reference for the reinforced sand ones. In Table 1, only two unreinforced sand tests with nearly the same void ratio as those of the reinforced sand tests under constant pressure and constant volume conditions are listed.

TEST RESULTS

Shear Strength and Volume Change of Unreinforced Sand

Figure 6 shows the relationships among the shear stress, the vertical displacement and the shear displacement from the unreinforced sand tests with different initial void ratios at $\sigma_v=50$ kPa. Different peak strengths and volume change characteristics with nearly the same residual strength were obtained. In Fig. 7, the peak stress ratios $(\tau_{vh}/\sigma_v)_p$ are plotted against the initial void ratio e_i .

Fig. 6. Stress and vertical displacement curves from unreinforced sand tests ($\sigma_v = 50$ kPa)

Fig. 7. Peak stress ratio versus void ratio relationship from unreinforced sand tests

A regressed equation of the exponential decay type was obtained. The peak stress ratio appears to approach to the residual value, which is the average residual stress ratio of 0.651 for $\sigma_v = 50$ kPa, as e_i approaches a very loose

Fig. 8. Vertical displacement versus void ratio relationship from unreinforced tests

Fig. 9. Shear strength and vertical displacement curves from unreinforced sand tests

state above 0.90. The regressed relationship shown in Fig. 7 is used to obtain the peak stress ratio of unreinforced sand to be compared with that of the corresponding reinforced sand test for the same void ratio.

In order to investigate the volume change characteristics, the vertical displacement d at a shear displacement

Fig. 10. Dependency of shear strength and vertical displacement on σ_v from unreinforced tests

of 50 mm was chosen as the ultimate value (Fig. 8). A linear relationship was fitted to the data points.

Figure 9 shows the relationships between τ_{vh}/σ_v , d and s for a similar initial void ratio (average $e_i \approx 0.627$) from unreinforced sand tests at $\sigma_v = 50 \sim 200$ kPa. It may be seen that within the range of σ_v examined, these relationships are not very sensitive to σ_v . To better see the above-mentioned feature, the relationships between $(\tau_{vh}/\sigma_v)_p$ and σ_v , and between d at $s = 50$ mm and σ_v for $e_i \approx 0.627$ were obtained (Fig. 10). It may be seen that both the peak stress ratio $(\tau_{vh}/\sigma_v)_p$ and the dilatation d at $s = 50$ mm slightly decrease with the increase in σ_v , as indicated by fitted broken curves. The value of $\phi_p = \tan^{-1}(\tau_{vh}/\sigma_v)_p$ averaged for the range of σ_v examined is 39.3° (for average $e_i \approx 0.627$). Considering the relatively small pressure-dependency of these two quantities, the average properties, as represented by the solid linear relationship in Fig. 10(a), will be referred to later when analyzing the results from constant volume tests on reinforced specimens.

Constant Pressure Tests of Reinforced Sand

RA Series:

Figure 11 shows the results from an unreinforced sand test and reinforced sand tests using reinforcement members with a smooth surface (RA series) for similar e_i values corresponding to those listed in Table 1. It may be

Fig. 11. Results from constant pressure tests (series RA, smooth)

seen that the pre-peak behavior and the peak strength are very similar among these tests, showing very small effects of reinforcement. At the residual states, slightly larger shear strength and dilatation are seen with thicker reinforcement members.

Similar results from reinforced sand tests with sand-glued surface (RAPR series) are shown in Fig. 12. It may be seen that by reinforcing, the overall stress-strain behavior has changed at shear displacements s exceeding the value corresponding to the peak state of the unreinforced sand. With the reinforced sand, as the peak strength is attained at s values larger than that with the unreinforced sand, larger peak and residual strengths and larger ultimate dilatation are obtained. However, the effects of reinforcement thickness are not noticeable. It may also be seen that in the post-peak regime, the effects of the rough reinforcement on the shear stress decrease as s increases, approaching to those of the smooth one. Despite the above, the dilatation rate at large s (say, at $s = 50$ mm) is noticeably larger with the rough reinforcement than with the smooth reinforcement.

RE Series:

The results from RE series tests using reinforcements with smooth and rough surfaces are shown in Figs. 13 and 14, respectively. It may be seen that for the same total cross-sectional area (i.e., the same longitudinal stiffness) of reinforcement, larger effects on the strength and dilatancy characteristics are obtained for a larger contact area of reinforcement with sand. The effects are larger with the rough reinforcements. It can be seen in Fig. 14

Fig. 12. Results from constant pressure tests (series RA, rough)

Fig. 13. Results from constant pressure tests (series RE, smooth)

Fig. 14. Results from constant pressure tests (series RE, rough)

that the largest dilatancy at a shear displacement of 50 mm is observed in the test with the largest contact area of reinforcement with sand (test REPR1), which is more than two times that of the unreinforced sand. In comparison, the effects are much smaller with the smooth reinforcement. As with the RA series, the effects of reinforcing on the volume change characteristics of reinforced sand are small for both smooth and rough reinforcements at shear displacements before the peak strength is attained in the unreinforced sand.

RD Series:

To investigate the effects of the degree of spatial dispersion of reinforcement, this series of tests was performed using reinforcements having the same total cross-section area and the same total surface area of the planes normal to the direction of shearing, but having different numbers of strips (or sheets). The results from the tests using smooth and rough reinforcements are shown in Figs. 15 and 16, respectively. The two extreme cases were tests RD1 and RDPR1 using a single sheet of 180 mm wide and tests RD30 and RDPR30 using 30 strips with a width of 6 mm. It is interesting to see that the effects of reinforcing on the strength characteristics become larger as the reinforcement is arranged in a spatially more dispersed way. Again in this series of tests, the effects of this factor are much more noticeable with rough reinforcement. It was suggested by Jewell (1980) that the tensile deformation in a volume of sand adjacent to a reinforcement member is restrained, as schematically shown in Fig. 17. Under otherwise the same conditions of reinforcement,

Fig. 15. Results from constant pressure tests (series RD, smooth)

Fig. 16. Results from constant pressure tests (series RD, rough)

the total volume of sand in which the development of strain is restrained by tensile reinforcement increases with the increase in the degree of spatial dispersion of reinforcement (i.e., with the increase in the number of strips with a smaller width), resulting in an increase in the strength of reinforced sand. It is noted that the dilatancy characteristics of reinforced sand are not largely affected by the degree of spatial dispersion of reinforcement.

The effect of tensile reinforcing on the strength of reinforced sand was thoroughly investigated by Jewell (1980) and Jewell and Wroth (1987). That is, referring to Fig. 18, tensile force T develops in the reinforcement as a result of restraining potential tensile strains in the surrounding sand in the direction of reinforcement. The component $T \cdot \sin \theta$ directly resists the shear force, where θ is the angle between the reinforcement direction and the normal of the potential shear plane (or shear zone) with the sign convention indicated in the figure. The normal (vertical) load acting on the potential shear zone increases by the amount of $T \cdot \cos \theta$, resulting in an increase in the shear resistance by the amount of $T \cdot \cos \theta \cdot \tan \phi$.

Tensile force activated in vertically placed tensile reinforcement decreases the vertical pressure in the lower sand zone beneath the bottom end of reinforcement. The vertical load, $P_{central}$, measured with bottom load cells 2 in the central area of the lower boundary (see Fig. 2), is a good indication of the tensile force developed in the reinforcement. The minimum value of $P_{central}$ was attained around the shear displacement where the peak shear strength was exhibited. The ratio of $P_{central}$ to the average

Fig. 17. Schematic diagram showing restraint of tensile deformation in a volume of sand adjacent to a reinforcement member

Fig. 18. Schematic diagram showing the mechanism of tensile reinforcing

of the vertical loads measured by the three parts of bottom load cells $P_{average}$ was obtained for each test. For unreinforced specimens, the original ratio before shearing was about 1.0. For reinforced specimens, however, the

Fig. 19. $P_{central}/P_{average}$ versus SR relationship

original value was larger than 1.0, depending on the reinforcement property. Figure 19 shows the relationship between the strength ratio SR and the minimum value of the ratio ($P_{central}/P_{average}$). The stress ratio SR is defined as the ratio of the peak shear strength $(\tau_{vh})_{peak}$ of reinforced sand to that of the corresponding unreinforced sand for the same initial void ratio of sand, referring to Fig. 7 (note that the peak strength of reinforced sand and unreinforced sand are mobilized at different shear deformations). A rather unique relationship for all the test results that may be seen from Fig. 19 indicates the relevance of the mechanism of tensile reinforcement as illustrated in Fig. 18.

The test results shown in the above indicate that without rupture of reinforcement (as in the case of the present study), the reinforced sand becomes stronger:

- as the total contact area between the sand and reinforcement increases;
- as the roughness of the reinforcement surface increases; and
- as the degree of spatial dispersion of the reinforcement increases.

The above consideration of the mechanism of tensile reinforcing indicates that the tensile strains in the sand are more effectively restrained by factors a), b) and c) above. Thus, the volume increase associated with shearing of reinforced sand should become smaller with the increase in the strength caused by tensile reinforcing. To obtain a better insight into this issue, the shear stress $(\tau_{vh})_{RP}$ of reinforced sand at the shear displacement where the corresponding unreinforced sand exhibited the peak shear strength " $(\tau_{vh})_{UP}$ " was obtained. In Fig. 20, the strength ratio $(\tau_{vh})_{RP}/(\tau_{vh})_{UP}$ for otherwise the same conditions is plotted against the vertical displacement d_{UP} of the rein-

Fig. 20. Relationship between strength ratio $(\tau_{vh})_{RP}/(\tau_{vh})_{UP}$ and vertical displacement d_{UP}

forced sand when the unreinforced sand exhibited the peak shear strength. The results for rough reinforcement shown in this figure support the above consideration. The average relationship for rough reinforcement, denoted by a broken curve, was obtained so as to pass the point representing the average value, $d_{UP} = -0.75$ mm, at $(\tau_{vh})_{RP}/(\tau_{vh})_{UP} = 1.0$ for unreinforced sand and the data points for rough reinforcement, denoted by solid squares. On the other hand, the result for smooth reinforcement scatters too largely to obtain a general trend; we can only note that the reinforced specimens generally exhibited smaller dilatation than the unreinforced ones.

It should be noted, however, that the opposite is true at the residual state; i.e., as shown in the preceding sections, in the post-peak regime after the peak strength is attained, more effectively reinforced sand that shows larger peak and residual strengths becomes more dilative (this point will be analyzed in more detail in the later section). This behavior can be explained as follows. The thickness of a single shear band is about 10~20 times as large as the mean diameter of particle D_{50} (Yoshida and Tatsuoka, 1997). Perhaps, a shear zone that is formed in the middle of a specimen in direct shearing consists of multiple shear bands. The formation and pattern of the shear zone are influenced by the degree of restraint that the apparatus imposes on the sand and will depend on the relative dimension of the apparatus and the typical sand particles (Scarpelli and Wood, 1982; Airey et al., 1985). So in a direct shear test, the number of shear bands may increase with the increase in the ratio of the shear box size to the grain size due to a larger freedom for developing multiple shear bands. For this reason, the shear zone thickness may change greatly from tens to hundreds of times the mean diameter of sand (D_{50}) as the ra-

tio of the shear box size to D_{50} increases (Palmeira and Milligan, 1989). Such a complex shear zone formation is enhanced in effectively reinforced sand, as a shear band cannot freely pass through reinforcement due to its rigidity. In such a case, a thicker shear zone consisting of a greater number of shear bands develops as the tensile reinforcing becomes more effective. Detailed observations of shear zone in the direct shear tests performed in the present study will be reported elsewhere by the authors. It is therefore anticipated that with sand that is effectively tensile-reinforced, showing more dilative behavior under drained conditions, the shear strength would be more enhanced under constant volume conditions.

Constant Volume Tests of Reinforced Sand

A set of reinforced sand specimens were sheared under constant volume conditions with the initial vertical stress σ_v of 20 kPa to evaluate the effects of the dilatancy potential of reinforced sand. The results are shown in Figs. 21, 22 and 23. The drained failure envelop presented in Fig. 10(a) is drawn in Fig. (d) of these figures. Because of high dilatancy, although a very stiff reaction frame was used, noticeable heave of the upper box took place, as seen from Figs. 21(e), 22(e) and 23(e). So the truly constant volume strengths were underestimated in these "constant volume tests". The correction for this undesired expansion of the specimen will be attempted in a later section. The following trends of behavior may be seen from these test results:

- The maximum stress ratio τ_{vh}/σ_v in the reinforced sand is not considerably larger than that in the corresponding unreinforced sand when compared to the case with the corresponding constant pressure tests. This would be due to that the development of tensile force in the reinforcement was restrained by the constant volume conditions.
- On the other hand, the increase in σ_v after the peak state of τ_{vh}/σ_v is mobilized is much larger in the reinforced sand than that in the unreinforced sand, resulting in higher values of τ_{vh} . This is due to more dilative behavior in the post-peak regime with the reinforced sand than with the corresponding unreinforced sand, as seen from Figs. 12, 14 and 16. It is interesting to see that the stress paths of the constant volume reinforced sand do not largely exceed the drained failure envelop of the unreinforced sand. This would be due to that the shear strength increase by the tensile force in the reinforcement is not significant, being masked by the shear strength increase due to the increase in σ_v .
- The reinforced sand that exhibits a larger volume expansion at the residual state generally shows a larger increase in σ_v , resulting in a larger increase in τ_{vh} at large shear deformations; i.e., rough versus smooth reinforcements (Fig. 21); larger versus smaller surface areas of reinforcement (Fig. 22); and spatially more versus less dispersed reinforcement (Fig. 23). The contribution of the last factor is very small and the smallest among the effects of these three factors.

Fig. 21. Results from constant volume tests (influence of surface roughness)

Fig. 22. Results from constant volume tests (influence of surface area)

Fig. 23. Results from constant volume tests (influence of spatial distribution)

Such an enhanced shear resistance of constant volume reinforced sand would help to predict the seismic behavior of submerged reinforced soil and the behavior of the newly developed pre-loaded and pre-stressed reinforced soil structures (Fig. 1), in which the nearly constant volume condition may prevail. It is expected that in such cases, the reinforced soil structures exhibit high strength, behaving in a ductile way as the deformation becomes large.

Shear Strength of Reinforced Sand

The tests with sand-glued strips, which showed noticeable reinforcing effects, are analyzed herein to quantify the reinforcing effects of different factors. Figures 24 and 25 show the effects of total longitudinal stiffness (EA) and covering ratio (CR) on the peak strength of reinforced sand under constant pressure. It can be seen from Fig. 24 that for the same covering ratio (CR) equal to 0.6, the stiffness EA has nearly no reinforcing effects. On the other hand, it may be seen from Fig. 25 that for the same rigidity $EA=5$, the strength ratio SR increases largely with the covering ratio CR. It can also be seen from Figs. 24 and 25 that even for the same CR with similar EA values, the strength ratio SR increases with the number of strips n ; i.e., SR becomes larger as the reinforcement is placed in a more dispersed way. Although the effects of reinforcement stiffness and shape were studied within a limited range in the present study, these test results indicate that unless rupture failure in the reinforcement occurs, the reinforcement stiffness could be a less important factor than the shape of reinforcement in terms of covering ratio and spatial dispersion.

The volume expansion ratio (VER) is defined as the ratio of the vertical displacement of reinforced to unreinforced sand, both at a shear displacement of 50 mm (refer to Fig. 8). Figure 26 shows the relationship between SR and VER. The increase in the shear strength is generally in accordance with the increase in VER. It may be seen, however, that for the same volume expansion ratio VER, the value of SR is larger for a larger number of strips (n); i.e., for larger degrees of dispersion of reinforcement. This means that the increase in the peak strength by reinforcing at relatively small shear displacements and the enhancement of the dilatancy characteristics activated at larger shear displacements by the inclusion of tensile reinforcement are by different mechanisms. That is, the increase in the peak strength by reinforcing is due to the development of tensile force in the reinforcement caused by the development of tensile strains in the direction of reinforcement in sand, which restrains the specimen volume from free expanding (see Fig. 17). On the other hand, more dilatant behaviour that is observed at large shear displacements of the reinforced specimen is due to the development of multiple shear bands. These two mechanisms are linked to each other, as both are different consequences of the interaction between the reinforcement and the sand.

Peng et al. (2000) reported results from a series of plane strain compression tests on an unreinforced speci-

Fig. 24. EA versus SR relationship

Fig. 25. CR versus SR relationship

men of Toyoura sand and those reinforced with horizontal layers of geogrid. The specimens were 57 cm-high $\times$ 24.4 cm $\times$ 21.4 cm in the σ_3 direction, and the tests were performed at a constant confining pressure of 19.6 kPa. Although the peak strength became substantially larger by reinforcing, the ultimate volume expansion was similar between the unreinforced and reinforced speci-

Fig. 26. VER versus SR relationship

mens. At the same time, multiple shear bands were observed within the reinforced specimens, compared with a single shear band in the unreinforced specimen. This apparent contradiction is due to the fact that larger dilatation by shear deformation of the reinforced specimens was suppressed by larger volume contraction due to a larger increase in the pressure level associated with larger peak strength. The results from a FEM analysis showed that the amount of shear deformation-induced volume expansion of the specimen (i.e., dilatancy-induced volume increase) was larger with the reinforced specimens. The direct shear tests under constant pressure are therefore better than such plane strain compression tests to evaluate the effects of reinforcing on the shear deformation-induced volume change characteristics of sand.

Comparison between Constant-Volume and Constant-Pressure Tests

For the representative five constant volume tests of reinforced sand, the ratios (VPR) of the vertical stress σ_v measured at $s=50$ mm to the corresponding value of unreinforced sand, equal to $\sigma_v=311.6$ kPa, were obtained. In Fig. 27, these values are plotted against the respective volume expansion ratio VER at $s=50$ mm obtained from each corresponding pair of constant pressure tests on reinforced and unreinforced sand performed at $\sigma_v=50$ kPa (open square symbols). It may be seen that under otherwise the same conditions, the reinforced sand that exhibits a larger dilatation under constant pressure conditions shows a larger increase in vertical stress under constant volume conditions. To correct the measured value of σ_v for the specimen expansion in the “nominally

Fig. 27. Constant volume VPR versus constant pressure VER relationship

Fig. 28. Assumption for vertical pressure correction in constant volume test

constant volume tests”, the assumption illustrated in Fig. 28 was used. In this figure, CP means a constant pressure test performed at $(\sigma_v)_c=20$ kPa. As constant pressure tests on reinforced sand at $(\sigma_v)_c=20$ kPa were not performed in the present study, it is assumed that the dilatancy characteristics for $(\sigma_v)_c=20$ kPa are the same as those for $(\sigma_v)_c=50$ kPa under otherwise the same conditions. NCV means a nominally constant volume test as performed in the present study, which started from the initial value $(\sigma_v)_0=20$ kPa. In this test, the value σ_v

Fig. 29. Constant volume SR versus constant pressure SR relationship

becomes $(\sigma_v)_1$ at a certain shear displacement s^* ($s^*=50$ mm will be used in the following). TCV means a truly constant volume test. Assuming a linear relationship between σ_v and d at a given shear displacement s^* in a given test, the value of σ_v at s^* in the TCV test, $(\sigma_v)_2$, is obtained by extending the linear line as shown in Fig. 28. The ratios of these estimated values, $(\sigma_v)_2$, for the five reinforced sand tests to the corresponding value in the unreinforced sand are denoted by cross-marks in Fig. 27. As the estimated values of $(\sigma_v)_2$ were about 1.3 to 1.4 times the value of $(\sigma_v)_1$ for both reinforced and unreinforced sand, their ratios did not change largely by this correction. The assumption used here is an approximation, and more study will be necessary to evaluate the truly constant volume behavior of unreinforced and reinforced sand in direct shear.

Then, for the five reinforced sand tests, the strength ratios SR were obtained, which is defined as the ratio of the shear stress τ_{vh} at a shear displacement $s=50$ mm in a reinforced sand test to that in the corresponding unreinforced test. In Fig. 29, the values of SR from the constant volume tests are plotted against the respective strength ratio SR from the corresponding pair of constant pressure tests on unreinforced and reinforced sand. The open square symbols mean that the plotted SR values for constant volume are measured values, while the cross symbols mean that they are the estimated values. It may be seen that the strength ratio SR is larger under constant volume conditions than that under constant pressure (or drained) conditions, which means that under other otherwise the same conditions, reinforced sand becomes stronger than unreinforced sand to a larger extent under constant volume conditions than under constant pressure (or

drained) conditions.

The ratio of the peak strength of unreinforced sand under constant volume conditions (CV) to that under constant pressure conditions (CP) obtained from the present study is:

$$A = (\tau_{vh})_{peak(U,CV)} / (\tau_{vh})_{peak(U,CP)} = 5.79$$

On the other hand, the ratios of the values of $SR_{(CV)}$ to $SR_{(CP)}$ that are shown in Fig. 29 range from about 1.15 to 1.68. Then the ratio of strength for the reinforced sand is obtained as:

$$(\tau_{vh})_{peak(R,CV)} / (\tau_{vh})_{peak(R,CP)} = SR_{(CV)} / SR_{(CP)} \times A$$

Thus, the ratio is about $(1.15 \sim 1.68) \times A = 6.6 \sim 9.7$. This means that under the test conditions in the present study, the reinforced sand becomes 6.6~9.7 times stronger by changing the test conditions from constant pressure to constant volume.

Finally, the ratio of the peak strength of reinforced sand under constant volume conditions to that of unreinforced sand under constant pressure conditions, $(\tau_{vh})_{peak(R,CV)} / (\tau_{vh})_{peak(U,CP)}$, is obtained as $SR_{(CV)} \times A$. This ratio is 9.7~13.2, which suggests that soil can become very strong by means of tensile-reinforcing with constant volume conditions.

In direct shear tests, gross constant volume conditions for an entire specimen do not mean the constant volume conditions of the shear zone and the remaining unfailing part of the specimen. That is, the shear zone can dilate while the dilatation is absorbed by compression of the remaining part of specimen. Therefore the grossly constant volume behavior of a unreinforced or reinforced specimen is controlled by the ratio of the specimen height to the particle size. As the ratio becomes larger, the constant volume strength gained by dilatant characteristics of sand will decrease. To evaluate the effects of this ratio, a limited number of tests were performed in the present study, as will be reported elsewhere by the authors.

CONCLUSIONS

From the results of the constant pressure and constant volume direct shear tests on unreinforced and reinforced sand presented above, the following conclusions can be drawn:

- (1) For reinforcement with a rougher surface, with a larger surface area and with a spatially more dispersed arrangement, reinforced sand exhibits a larger peak strength under constant pressure conditions. In particular, with the limited test conditions in the present study without rupture failure of reinforcement, the effects of the surface area are much more important than those of the rigidity of reinforcement. In addition, even with the same surface area of reinforcement, the reinforcing effect on the peak strength becomes larger as the reinforcement is arranged in a more dispersed way.
- (2) Generally, the shear displacements at the peak strength and the volume increase at the residual

state increase with the increase in the reinforcing effects.

- (3) Reinforced sand shows larger dilatancy than unreinforced sand in the post-peak regime. The dilatancy characteristics increase with the increase in roughness and the area of the reinforcement surface. Within the limited test conditions, the reinforcement rigidity showed no noticeable effects, and the dispersion of reinforcement had very little effect.
- (4) Reinforced sand that exhibits a volume expansion at post-peak large shear displacement that is larger than that of unreinforced sand in a constant pressure test shows an increase in vertical pressure that is larger than that of unreinforced sand in a constant volume test. Consequently, the constant volume shear strength of reinforced sand can become very large, particularly, at larger displacements.

The test results suggest that under otherwise the same conditions, dilative reinforced sand structures can become stronger than unreinforced sand structures to a larger extent under constant volume conditions than under constant pressure (or drained) conditions. This factor should be considered when evaluating the stability at large deformations of dilative reinforced soil structures loaded under restrained or constant volume change conditions.

ACKNOWLEDGMENT

The first author acknowledges the financial support provided by the Ministry of Education, Science and Culture (Monbusho), Japan.

REFERENCES

- 1) Airey, D. W., Budhu, M. and Wood, D. M. (1985): "Some aspect of the behaviour of soils in simple shear," *Developments in Soil Mechanics and Foundation Engineering*—2, Stress-strain Modeling of Soils, Banerjee, P. K. and Butterfield R. (eds.), Elsevier, pp. 185–213.
- 2) Bauer, G. E. and Zhao, Y. (1993): "Evaluation of shear strength and dilatancy behavior of reinforced soil from direct shear tests," *Geosynthetic Soil Reinforcement Testing Procedures*, ASTM STP 1190, pp. 138–151.
- 3) Gray, D. H. and Ohashi, H. (1983): "Mechanics of fiber reinforcement in sand," *J. of Geotech. Engrg., ASCE*, Vol. 109, No. 3, pp. 335–353.
- 4) Jewell, R. A. (1980): "Some effects of reinforcement on the mechanical behaviour of soils," Ph. D. thesis, Cambridge Univ.
- 5) Jewell, R. A. and Wroth, C. P. (1987): "Direct shear tests on reinforced sand," *Géotechnique*, Vol. 37, No. 1, pp. 53–68.
- 6) Palmeira, E. M. and Milligan, G. W. E. (1987): "Scale effects in direct shear tests on sand," *Proc. 12th Int. Conf. on SMFE*, Rio de Janeiro, Vol. 1, pp. 739–742.
- 7) Palmeira, E. M. and Milligan, G. W. E. (1989): "Large scale direct shear tests on reinforced soil," *Soils and Foundations*, Vol. 29, No. 1, pp. 18–30.
- 8) Peng, F. L., Kotake, N., Tatsuoka, F., Hirakawa, D. and Tanaka, T. (2000): "Plane strain compression behaviour of geogrid-reinforced sand and its numerical analysis," *Soils and Foundations*, Vol. 40, No. 3, pp. 35–74.
- 9) Scarpelli, G. and Wood, D. M. (1982): "Experimental observations of shear band patterns in direct shear tests," *IU TAM Symp. on Deformation and Failure of Granular Materials*, Vermeer, P. A. and Luger, H. J. (eds.), Delft, pp. 473–484.
- 10) Shewbridge, S. E. and Sitar, N. (1989): "Deformation characteristics of reinforced sand in direct shear," *J. of Geotech. Engrg., ASCE*, Vol. 115, No. 8, pp. 1134–1147.
- 11) Shibuya, S., Mitachi, T. and Tamate, S. (1997): "Interpretation of direct shear box testing of sands as quasi-simple shear," *Géotechnique*, Vol. 47, No. 4, pp. 769–790.
- 12) Tatsuoka, F., Uchimura, T. and Tateyama, M. (1997): "Preloaded and prestressed reinforced soil," *Soils and Foundations*, Vol. 37, No. 3, pp. 79–94.
- 13) Uchimura, T., Tatsuoka, F., Tateyama, M. and Koga, T. (1998): "Behavior of the first prototype and full-scale models of PLPS geosynthetic-reinforced soil structure," *Boston, Annual Convention, ASCE: Geosynthetics in Foundation Reinforcement and Erosion Control Systems, Proc. of Geo-Congress '98*, ASCE, Boston, pp. 34–48.
- 14) Yoshida, T. and Tatsuoka, F. (1997): "Deformation property of shear band in sand subjected to plane strain compression and its relation to particle characteristics," *Proc. 14th Int. Conf. on SMFE, Hamburg*, Vol. 1, pp. 237–240.