THE SHEAR BEHAVIOR OBTAINED FROM THE DIRECT SHEAR AND PULLOUT TESTS FOR DIFFERENT POOR GRADED SOIL-GEOSYNTHETIC SYSTEMS

Chiwan Wayne Hsieh [1], Gee Ham Chen [2], and Jeng-Han Wu [3]

ABSTRACT

This paper discussed the difference between the test results obtained from ASTM D5321, the direction shear test, and ASTM D6706, the pullout test for poor graded soil/geosynthetic systems. If the soil particles could be able to penetrate through the geosynthetic openings and to develop the passive bearing capacity resistance, the pullout resistance would not increase in proportion to the increase of soil normal pressure. The pullout interaction coefficient, C_i, would decrease as increasing the normal pressure. Otherwise, the efficiency on friction for the direct shear test can be correlated to the interaction coefficient for pullout test for the most test conditions. The interaction coefficient is about 50% to 65% of the friction efficient for the direct shear test. Pullout interaction coefficients ranged from 0.182 to 1.251 for the test conditions. For direct shear test, if the soil particles are smaller than the geosynthetic openings, this could cause the shearing surface occurred above the soil/geotextile interface, and the frictional behavior would quite similar to that of the test soil itself. If not, the soil particles were expected to turn around and slide along the geosynthetic surface, and low friction efficiency at ultimate strain was commonly obtained. These friction efficiencies varied from 0.36 to 0.98 for the test conditions.

Key words: Direct shear test, pullout test, geosynthetic, geotextile, geogrid, interaction coefficient.

1. INTRODUCTION

Geosynthetics (geotextiles and geogrids) are often called upon to provide anchorage for many applications within the reinforcement function. Such anchorage usually has the geosynthetic sandwiched between soils on either side. The resistance can be modeled in the laboratory using a pullout test. The pullout resistance of the geosynthetic is obviously dependent on the normal force applied to the soil, which mobilized shear forces on both surfaces of the geosynthetic. The ASTM D-6706 (2001) standard pullout test method is intended as a performance test to provide the design parameters, which can be used in the design of geosynthetic-reinforcement retaining walls, slopes, and embankments, or in other applications where resistance of a geosynthetic to pullout under simulated field condition is important. The pullout resistance is a function of soil gradation, plasticity, as-placed dry unit weight, moisture content, length and surface characteristics of the geosynthetic and other test parameters.

The interface friction angle and adhesion between a geosynthetic and soil are the primary and most contentious variables used in geosynthetic reinforcement structure stability design and analysis. Direct shear tests are performed to provide the design engineer with the friction angle and adhesion coefficient for the various interfaces within the design. The direct shear test is also used as a form of quality control to ensure product compliance to the values used in the design. The ASTM D-5321 (2002) standard direct shear test method is commonly used for determining the Bond Coefficient between soil and geosynthetic or geosynthetic and other geosynthetic systems.

Test results by Collios *et al*. (1980) show a relationship of pullout test results to shear test results with some notable exceptions. For pullout testing, if the soil particles are smaller than the geotextile openings, efficiencies are high; if not, they can be low. In all cases, however, pullout test resistances are less than the sum of the direct shear test resistances. This is due to the fact that the geosynthetic is taut in the pullout test and exhibits large deformation. This, in turn, causes the soil particles to reorient themselves into a reduced shear strength mode at the soil-to-geotextile interfaces, resulting in lower pullout resistance. The stress state mobilized in this test is a very complex one requiring additional research. However, granular soils and geogrids are more commonly used in mechanical stabilized earth structures. Interesting comparison between steel grids, steel plate, polymer geogrids, and polymer geonets are reported by Ingold (1983). This behavior comes about by virtue of the large apertures in the geogrid allowing for soil strike-through from one side of the geogrid to the other. Obviously, the soil particles must be sufficiently small to allow for full penetration. Sarsby (1985) has proposed the optimum transfer of shear stress occurs when the minimum width of geogrid aperture is greater than 3.5 time of D_{50}. Moreover, this recommendation is based upon the test results obtained from direct shear tests.

ASTM D5321 and ASTM D6706 standard test methods were adopted by ASTM on 1992 and 2001, respectively. The dimensions of typical direct shear test specimen are 300 mm by 300 mm. However, the dimensions of the typical large pullout box were 0.91 m wide, 1.9 m long, and 1.1 m depth (Koerner 1998). The dimensions of the test specimen were 300 mm wide

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and 750 to 1000 mm long (Retzlaff and Recker 2003; Ochiai *et al.* 1992). The pullout test specimen were modified to a minimum dimensions of 300 mm by 600 mm in ASTM D6706 test standard in 2001, which is double size of direct shear test specimen in ASTM D5321 test method. Therefore, to compare the test results obtained from the tests using the standard dimension specimens would limit the size effect. Therefore, the objective of this paper is to discuss the difference in shear resistance behavior at the soil-geosynthetic interface for uniform graded granular soils and different types of geosynthetics.

2. DIRECT SHEAR TEST PRINCIPLES AND TEST STANDARD

The direct shear test is used to measure the shear strength of soils. This test was modified to evaluate the shear strength behavior when shearing geosynthetics against soils. During tests using geomembranes, the soil is forced to slide along a geomembrane under a constant rate of displacement, while a constant load is applied normal to the plane of relative movement. The maximum shear stress at large displacement is obtained and the test is conducted at different normal confining pressures. Shear stress/displacement curves for specimens tested under different normal pressures, as well as the Mohr-Coulomb envelope (1994). The cohesion or adhesion, and friction angles of the soils and soil/geosynthetic system can be determined. Using the obtained data, the friction efficiency (E_φ) and the cohesion efficiency (E_c) can be calculated based upon the following equations:

$$E_\varphi = (\tan \delta)/(\tan \varphi) \tag{1}$$

$$E_c = c_a/c \tag{2}$$

where φ = soil internal friction angle,
δ = friction angle of geomembrane against soil,
c = soil cohesion,
c_a = adhesion.

In 1991, the American Society for Testing and Materials (ASTM) adopted the currently used D-5321 standard test method for "Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method". The standard test method suggested that both square and rectangular shear boxes could be used. These boxes should have a minimum dimension that is greater than 300 mm. A normal stress load device should be capable of applying and maintaining a constant uniform normal stress on the specimen for the duration of the test. The shear force load device should be capable of applying a shear force to the specimen at a constant rate of displacement (strain controlled) in a direction parallel to the direction of travel of the soil container. The shear force normally is applied at a rate of 1.0 mm/min. The rate of displacement should be controlled to an accuracy of ±10% of the displacement rate.

3. PULLOUT TEST PRINCIPLES AND TEST STANDARD

The anchorage strength or pullout resistance is a result of three separate mechanisms. One is the shear strength along the top and bottom of the longitudinal ribs of the geogrid. The second is the shear strength contribution along top and bottom of the transverse ribs. The third mechanism is the passive resistance against the front of the transverse ribs. In the last mechanism the soil goes into a passive state and resists pullout by means of bearing capacity. It has been analytically shown that this bearing capacity can be a major contributor to the overall anchorage strength of geogrids (1989). An interaction coefficient C_i can be determined based upon the following equation. However, the value of C_i is function of soil type and test parameter specific.

$$T = 2\ C_i\ L_e\ \sigma'_n\ \tan \varphi' \tag{3}$$

where
T = anchorage capacity per unit width (kN/m),
C_i = interaction coefficient,
L_e = Length of geogrid embedment (m),
σ'_n = effective normal stress in the geogrid (kN/m^2), and
φ' = effective soil friction angle (deg.).

This equation could be modified to handle cohesive soils, usually granular soils are selected for backfill materials and if not, the omission of a cohesion term leads to a conservative design. Currently, the pullout test has been adopted by ASTM as test method D6706. The box should be square or rectangular within minimum dimensions 610 mm long by 460 mm wide by 305 mm deep. The pullout system must be able to apply the pullout force at a rate of 1 mm/min ±10%. The pullout resistance versus displacement and maximum pullout resistance versus normal stress curves are commonly plotted for analysis.

4. TEST PROGRAM

This study incorporated ASTM D5321, Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method, and ASTM D6706, Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil, were used in the study. The test materials, equipments, and test conditions are discussed in the following paragraphs.

4.1 Geosynthetic Reinforcements

One high-strength polypropylene geotextile woven by silt film fibers, and one type uni-axial geogrid, were used in the study. The thicknesses and widths of the silt-film fibers in machine direction and cross machine direction are 0.21 mm, 2.17 mm, and 0.31 mm and 2.48 mm, respectively. The polyester geogrid was produced from white 4000-diner PET multifilament fibers and coated with PVC resin. The polyester yarns were all manufactured in Taiwan. The average carboxyl end group (CEG) value and number average molecular weight of the PET yarns were 23.2 meq/kg and 31,003, respectively. The test uni-axial geogrid was plane woven with typical aperture of 14 mm by 27 mm. All of these geosynthetic samples are manufactured by different local companies. The engineering properties of these geosynthetics are summarized in Table 1 and Table 2.

4.2 Test Soils

In order to evaluate the effect of geosynthetic reinforcement, the selection criterion for the test soil included the variation of

Table 1 Properties of polypropylene silt film geotextile

Item		Test method	Results
Thickness (mm)		ASTM D5199	1.253
Mass per unit area (g/cm^2)		ASTM D5261	396.90
Apparent opening size (mm)		ASTM D4751	0.179
Grab tensile strength (kN)	M.D.	ASTM D4632	3.274
	C.M.D.		2.444
Tearing strength (kN)	M.D.	ASTM D4533	1.480
	C.M.D.		1.085
Width tensile strength (kN/m)	M.D.	ASTM D4595	81.60
	C.M.D.		57.63
Elongation at break (%)	M.D.	ASTM D4595	13.68
	C.M.D.		8.86

Table 2 Properties of PVC coated polyester geogrid

Item		Test method	Results
Wide width tensile strength (kN/m)	M.D.	ASTM D6637, Method B	156.66
	C.M.D.		34.6
Elongation at break (%)	M.D.	ASTM D6637, Method B	9.59
	C.M.D.		12.75
Aperture (mm)	M.D.	Calipers	27.24
	C.M.D.		14.33
Junction strength efficiency (%)	M.D	ASTM D6637, Method A & GRI-GG2	12.37

maximum and minimum dry density of the test soil must be limited within 1%. Three types of poorly graded granular soils, a rounded white quartz sand, a brownie gray rounded river bed medium gravel, and a large crashed gravel, were selected in the study. Test soils were compacted to a minimum relative density of 60% for the direct shear and pullout tests. The D_{50} and coefficient uniformity (C_u) of the test soils were 0.64 mm, 6.9 mm, 9.5 mm, and 1.46, 1.43, 1.85, respectively. The engineering properties of the soils are summarized in the Table 3.

4.3 Test Equipment

A large-scale direct shear/pullout test machine was employed in this study. The large-scale test machine was fabricated according to the ASTM D-5321 and ASTM D6706 standard test methods with some modifications. A server controlled hydraulic vertical load system and an air bag compression loading mechanism were used for direct shear and pullout test, respectively. DC motor was used to drive the shear/pullout devices. A computerized data acquisition system was setup to collect the displacement and shear stress data.

5. TEST RESULTS AND DISCUSSIONS

Generally, a series of direct shear tests and pullout tests according to ASTM D5321 and D6706 standard test methods respectively were performed in the study. Three different normal stresses were applied to the test specimens, the applied normal stress were 49.05 kN/m^2, 98.10 kN/m^2, and 147.15 kN/m^2. Three types of poor graded granular soils, a rounded white quartz sand, a brownie

Table 3 Properties of test poorly graded soils

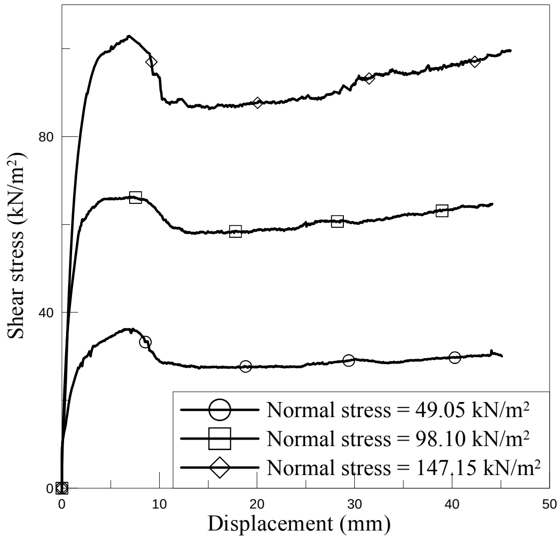
Item		Test method	Quartz sand	Riverbed gravel	Crushed stone
Soil classification		ASTM D2487	SP	GP	GP
D_{50} (mm)		ASTM D422	0.64	6.9	9.5
Coefficient of uniformity C_u		ASTM D422	1.46	1.43	1.85
$\gamma_{d(max)}$ (kN/m^3)		ASTM D4254	15.94	17.14	15.47
$\gamma_{d(min)}$ (kN/m^3)		ASTM D4253	13.70	1.640	14.07
γ_d @D_r = 60% (kN/m^3)		————	14.96	16.71	14.88
Angle of Friction @D_r = 60%	Peak	ASTM D5321	37.52	42.21	58.41
	Ultimate		36.81	38.40	53.00

gray rounded river bed medium gravel, and a large crashed gravel, were selected in the study. The soils were compacted to 60% relative dry density for testing. The pullout and shear resistance data were all converted to SI stress unit (kN/m^2) for analysis.

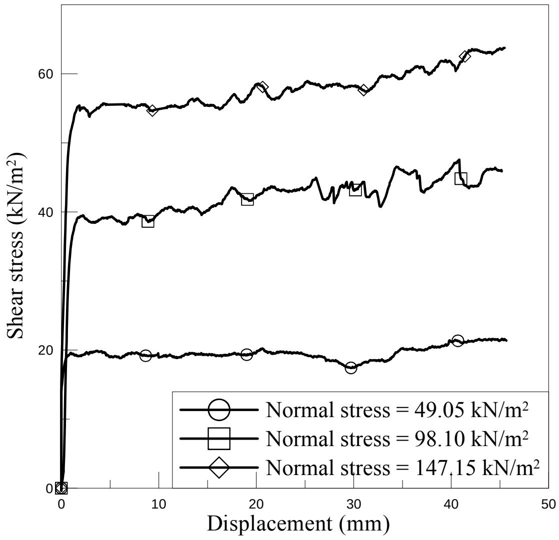
5.1 Direct Shear Test Results

Figure 1 showed the typical direct shear test results for the test geotextile and test soils at 60% relative dry density condition. Because the width of geotextile silt film fiber was larger than the quartz sand, the quartz sand could be interlocked into the geotextile surface. This would cause the shearing surface occurred above the soil/geotextile interface, and the frictional behavior would quite similar to that of the quartz sand itself. The peak and ultimate shear resistance occurred at the shear displacement of 7 ~ 8 mm and 12 ~ 18 mm, respectively. The friction angle and efficiency (E_φ) at peak and ultimate displacement are 35.4°, 29.3°, and 0.92, 0.73, respectively. Because the surface of the riverbed gravel was relatively smooth and is much larger than the width of geotextile silt film fiber, the gravels were expected to turn around and slide along the geotextile surface during direct shear test, the peak shear strength was not very clear and occurred at very low shear displacement (2 ~ 3 mm) condition. However, the sharp surface of the crushed stone would not allow the stone to smoothly move along the geotextile interface, no peak shear resistance was observed for the direct shear tests of the geotextile and crushed stone, low friction efficiency at ultimate displacement of 0.36 was observed.

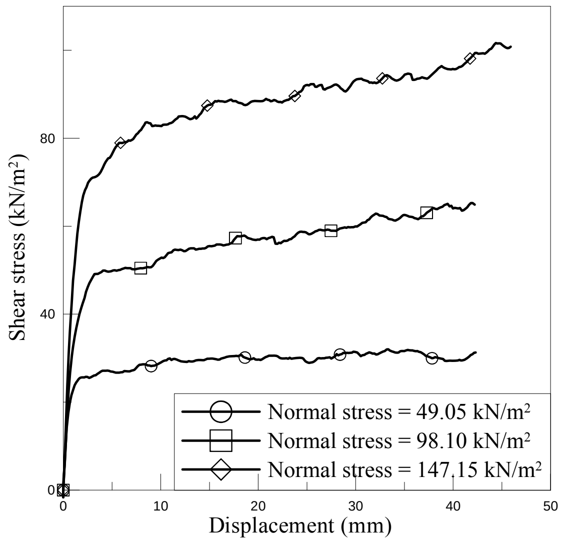
The direct shear test results for the test geogrid and the test soils at 60% relative dry density were showed in Fig. 2. The approximate thickness of the longitudinal and transverse ribs, and the junction were 1.24 mm, 1.12 mm, and 1.61 mm, respectively. The test soils could completely or partially penetrate through the geogrid ribs. Because the interlock phenomena, the shearing surface would occur beyond the geogrid. The test soils and geogrid shear behavior would similar to that for the test soils. In general, the shear strength at peak and ultimate displacement would slight less than that of the test soil itself. Tables 4 and 5 summarized the friction angles and efficiencies of the test geosynthetics and test soils, respectively. In general, the geotextile consists of relatively smooth surface, and the geogrid consists of apertures which allow certain size soil particles be able to penetrate through and to



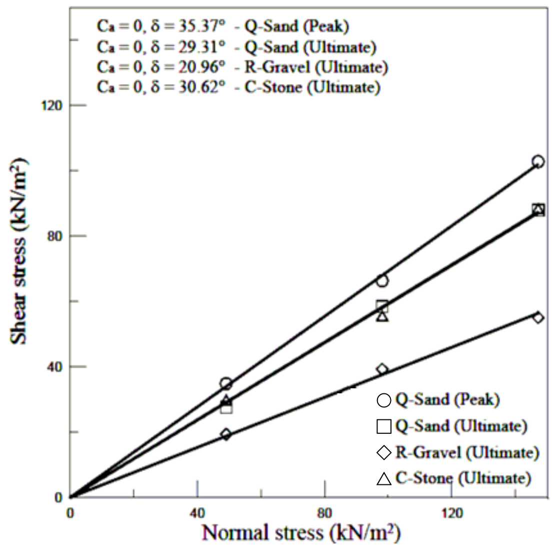
(a) Geotextile and quartz sand (Q-Sand)



(b) Geotextile and riverbed gravel (R-Gravel)



(c) Geotextile and crushed stone (C-Stone)



(d) Direct shear test failure envelope

Fig. 1 Direct shear test results of the test geotextile and the test soils (Q-Sand, R-Gravel, and C-Stone)

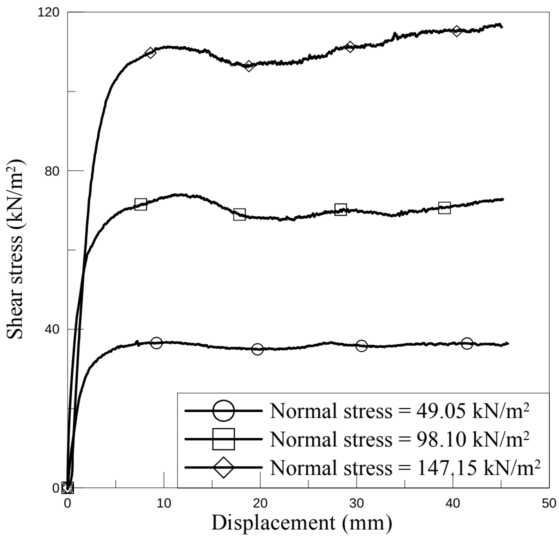
Table 4 Friction angle of the direct shear test for the test soils or soil/geosynthetic systems

Test soil	Type	Internal friction angle of the soil (°)	Interface friction angle for the test geotextile and test soil (°)	Interface friction angle for the test geogrid and test soil (°)
Quartz sand	Peak	37.52	35.37	37.00
	Ultimate	36.81	29.31	35.60
Riverbed gravel	Peak	42.21	N.A.	N.A.
	Ultimate	38.40	20.96	38.69
Crushed stone	Peak	58.41	N.A.	N.A.
	Ultimate	55.00	30.62	43.49

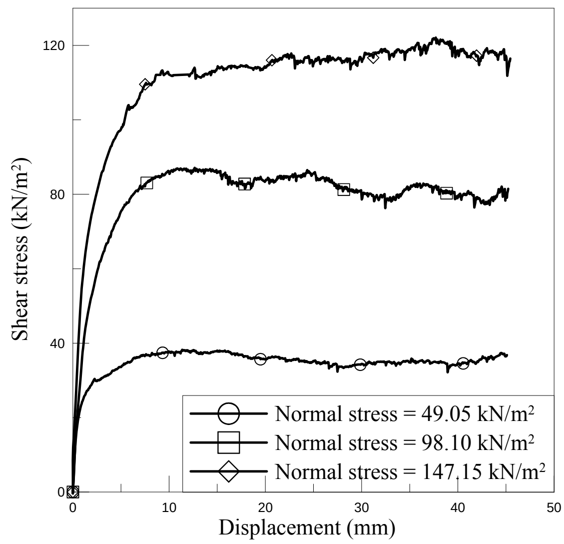
Remark: test soils were compacted to 60% relative dry density for testing

Table 5 Summary of direct shear test friction efficiency and pullout test interaction coefficient

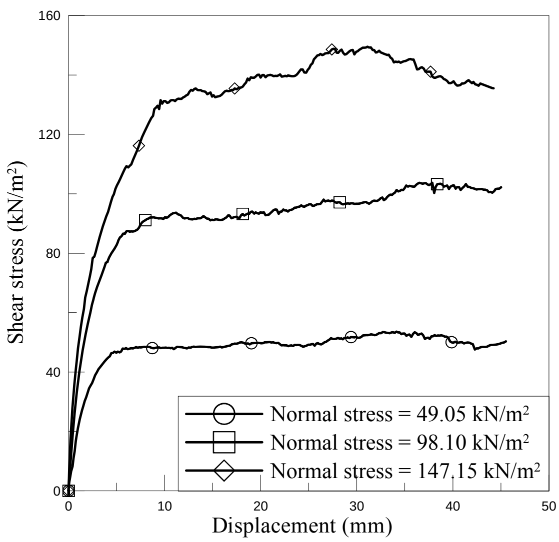
Test soil	Geosynthetic type	Peak friction efficiency	Ultimate friction efficiency	Pullout test interaction coefficient C_i, Normal stress (kN/m^2) 49.05	98.10	147.15
Quartz sand	PP geotextile	0.92	0.73	0.654	0.649	0.489
	PET geogrid	0.98	0.93	0.513	0.660	0.680
Riverbed gravel	PP geotextile	N.A.	0.42	0.182	0.238	0.240
	PET geogrid	N.A.	0.88	1.251	0.788	0.650
Crushed stone	PP geotextile	N.A.	0.36	0.189	0.182	0.186
	PET geogrid	N.A.	0.58	0.691	0.400	0.219



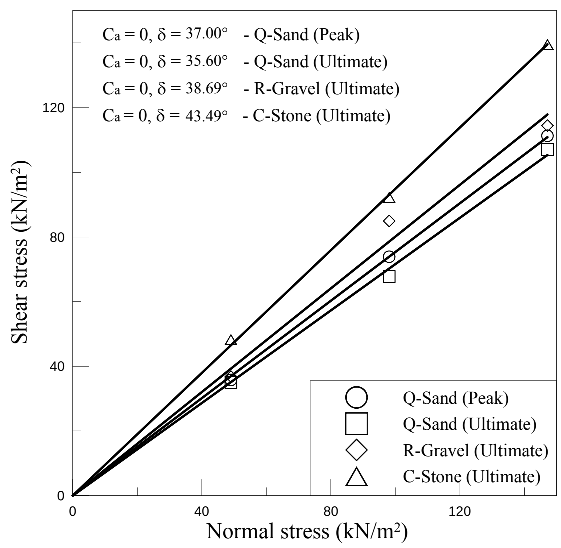
(a) Geogrid and quartz sand (Q-Sand)



(b) Geogrid and riverbed gravel (R-Gravel)



(c) Geogrid and crushed stone (C-Stone)



(d) Direct shear test failure envelope

Fig. 2 Direct shear test results of the test geogrid and the test soils (Q-Sand, R-Gravel, and C-Stone)

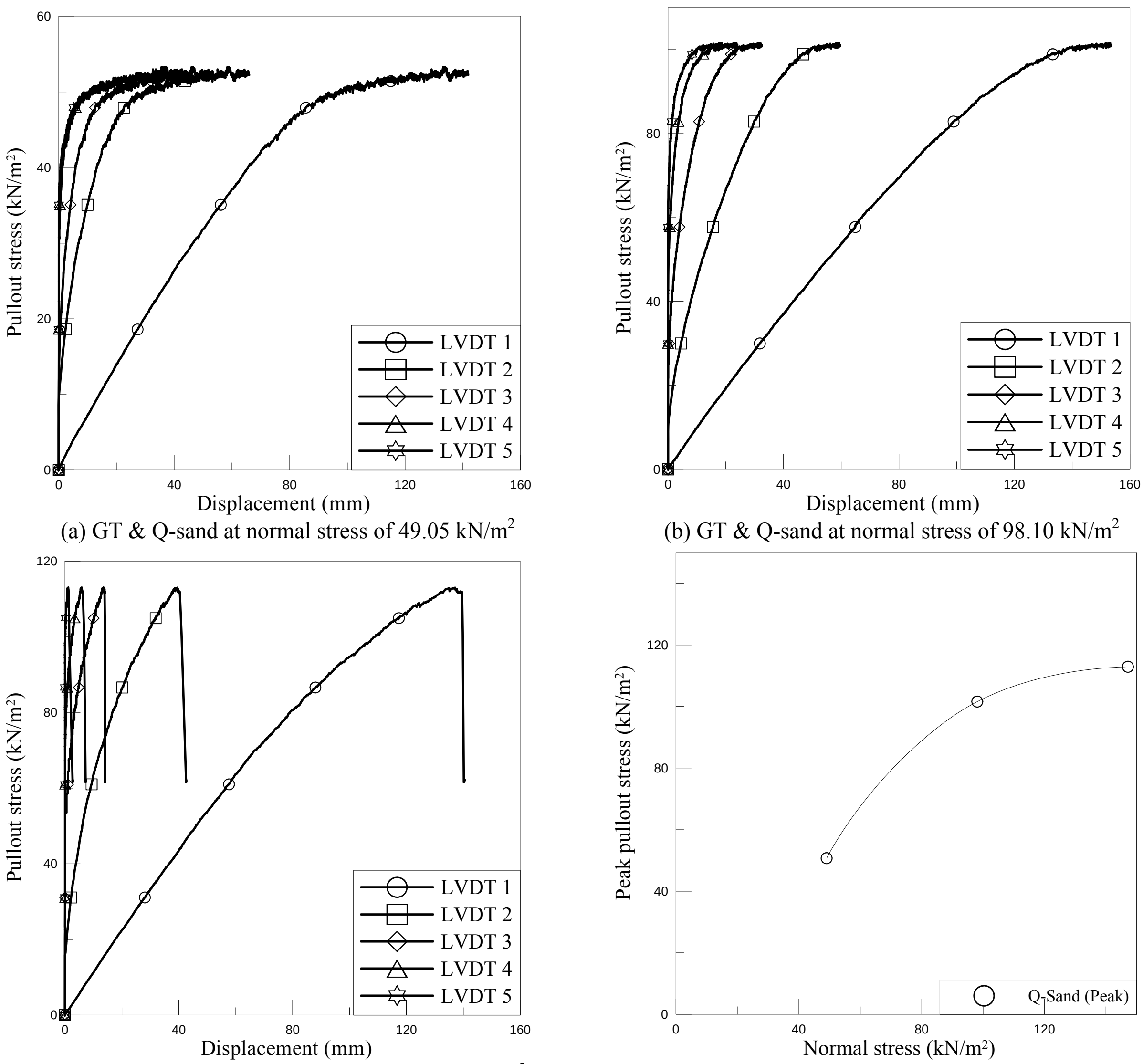
develop some kind of passive resistance. Therefore, if the test soils could allow soil particles penetrating through its opening, the soil/geosynthetic shear behavior would similar to that for the test soil. On the other hand, if interlocking phenomena does not occur, the shearing face is commonly occurred near the soil/geosynthetic interface, and the interface frictional resistance dominates the direct shear test.

5.2 Pullout Test Results

The dimensions of pullout specimen were 300 mm wide and 600 mm long, which was double of that for direct shear test. 150 mm cover soil was used in the pullout tests; however, the thickness of test soils in the direct shear test upper box was only 100 mm. This would induce only 0.5% ~ 1.6% difference in normal compression pressure. This effect was ignored in the discussion.

Five LVDT transducers were used to measure the displacement of the test specimens. LVDT1 was placed in the front of pullout box to measure the displacement of the box during the tests. However, the displacements of the test specimen at different locations were monitored by 4 other LVDT transducers. These LVDT transducers were equally spaced along the specimen. LVDT2 was placed closer to the front face of the box and LVDT5 was located at near the end of the specimen.

Figure 3 showed the pullout test results of the geotextile and quartz sand under three different normal compression stresses. Approximate linear elastic pullout behavior was observed until reaching the pullout strength. Pullout phenomenon was observed as the test specimen showing around 20 ~ 30 mm pullout



(a) GT & Q-sand at normal stress of 49.05 kN/m^2

(b) GT & Q-sand at normal stress of 98.10 kN/m^2

(c) GT & Q-sand at normal stress of 147.15 kN/m^2

(d) Normal stress versus peak pullout stress

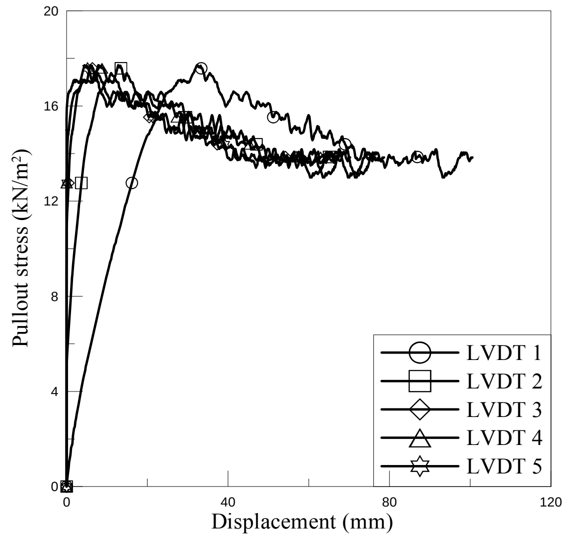
Fig. 3 Pullout test results of the test geotextile and the quartz sand

displacement which was similar to the required displacement for the quartz sand reaching the ultimate strength during the direct shear test. As shown in Fig. 3, the displacement at the front surface of pullout box was about 80 mm more than that for the test specimen within in the test box, this elongation of test specimen was primary occurred around the pullout grip. Since the allowable traveling distance for the test machine is limited, the set up of test specimen for limiting the elongation around grip is an important step to ensure pullout phenomenon can be observed in the pullout test. Because the strength of test specimen was limited, rupture failure was observed for the geotextile specimen at around 110 kN/m^2 pullout stress under 147.15 kN/m^2 compression stress.

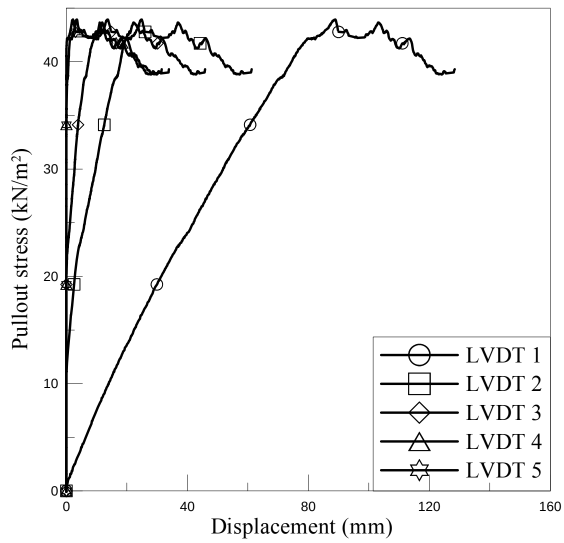
The pullout test results for the geotextile and the riverbed gravel at 49.05 and 98.10 kN/m^2 normal stresses were showed in Fig. 4. Because the riverbed gravel is rounded medium gravel and its surface quite smooth, the test gravel would not be easily interlocked with the test geotextile. The gravels could be rolling and turning around the geotextile surface during pullout test. The associated pullout resistance was less than that for the quartz sand itself. The pullout interaction coefficients ranged from 0.182 to 0.240.

The geotextile and crushed stone pullout test results at 49.05 and 98.10 kN/m^2 normal stress were also showed in Fig. 4. As shown in the figure, no significant interlocking phenomenon was observed at the interface. The test specimen showed progress rupture as reaching the pullout strength. The pullout interaction coefficients varied from 0.182 to 0.189.

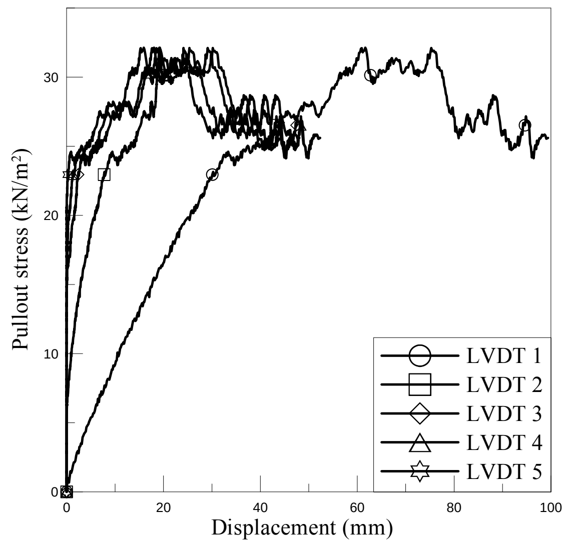
The pullout test results for the test geogrid and the three type test soils were all showed in Fig. 5. Since the tensile modulus of PET fiber is higher than PP silt film fiber, very small amount displacements were observed before pullout occurred for the normal stress of 49.05 kN/m^2 test condition. The pullout resistance curves consisted three linear sections for the normal stress of 98.10 kN/m^2 test condition. The initial section was expected to be related to the friction resistance on the geogrid ribs. The second section could be related to the development of passive resistance around the transverse ribs. The third section should be the test specimen pulling out from the box. The passive resistance consisted only around 35% of the total pullout resistance. The pullout interaction coefficients varied from 0.513 to 0.680 under different normal stresses.



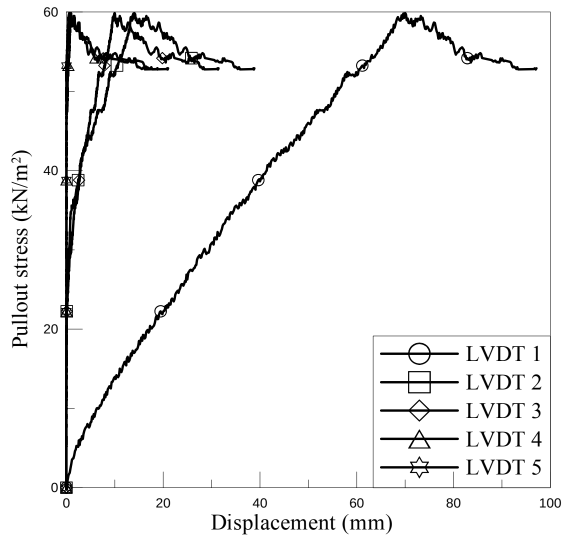
(a) GT & R-Gravel at normal stress of 49.05 kN/m^2



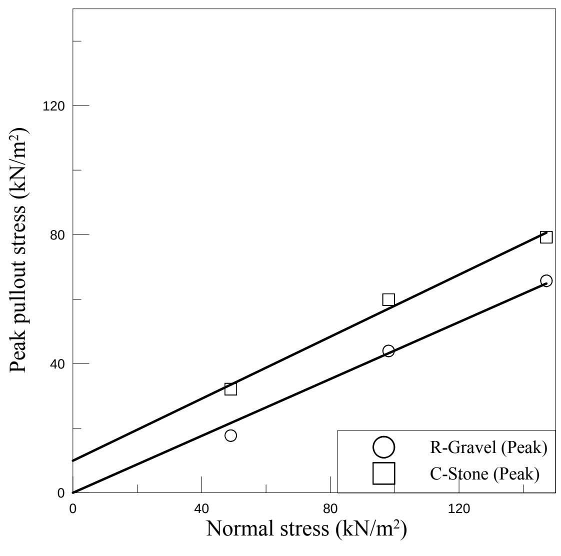
(b) GT & R-Gravel at normal stress of 98.10 kN/m^2



(c) GT & C-Stone at normal stress of 49.05 kN/m^2

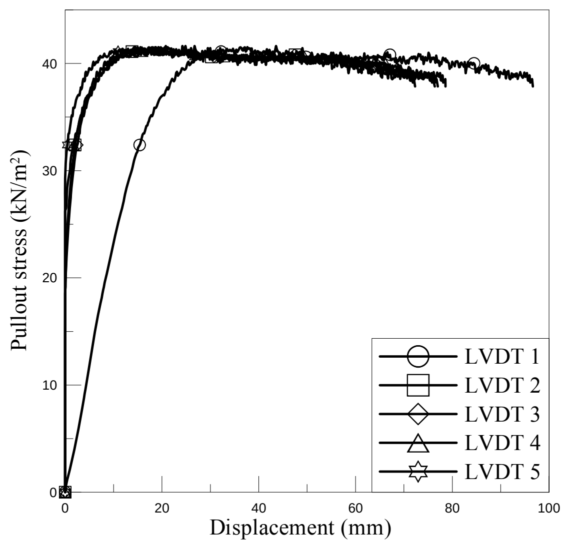


(d) GT & C-Stone at normal stress of 98.10 kN/m^2

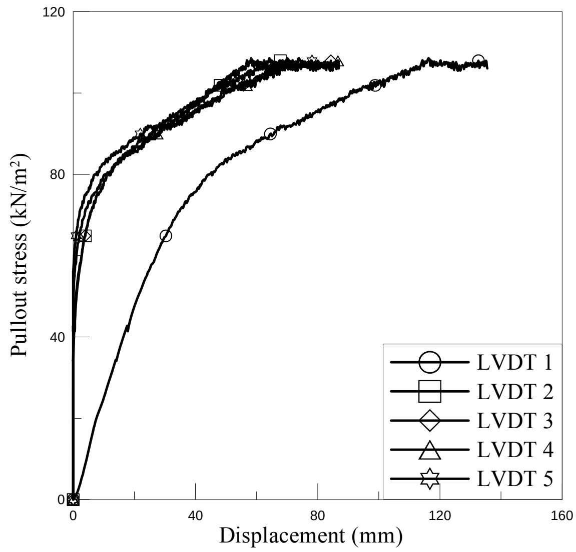


(e) Normal stress versus peak pullout stress

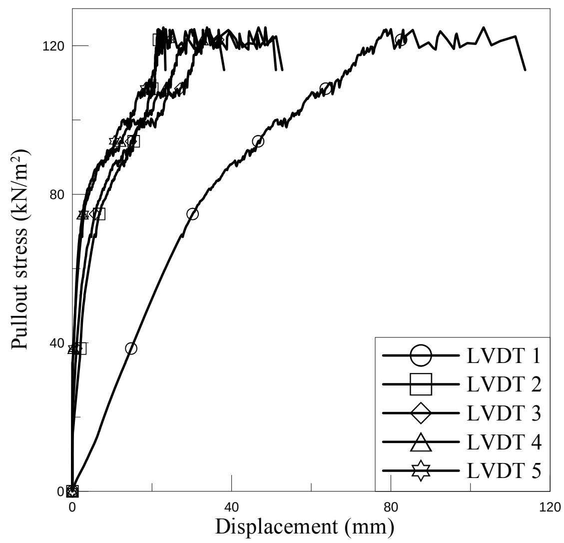
Fig. 4 Pullout test results for the test geotextile and the riverbed gravel or crushed stone



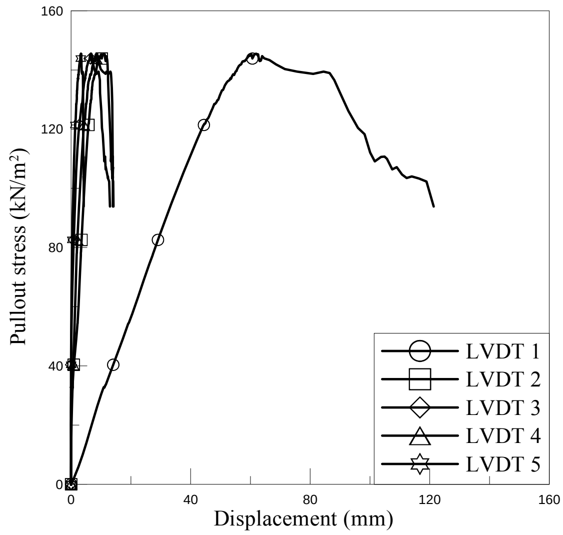
(a) GG & Q-Sand (Normal stress of 49.05 kN/m^2)



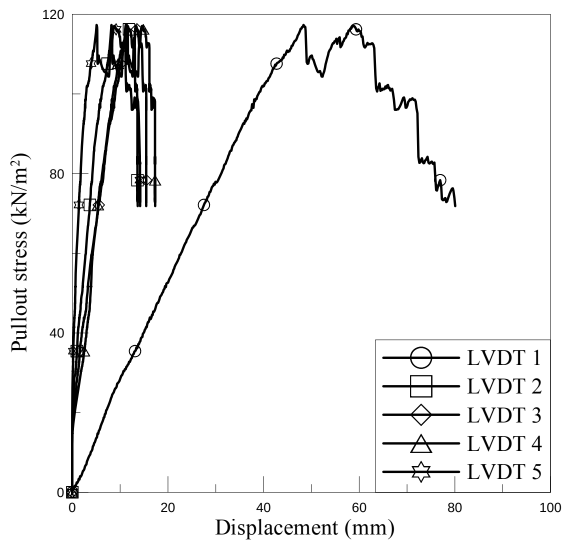
(b) GG & Q-Sand (Normal stress of 98.10 kN/m^2)



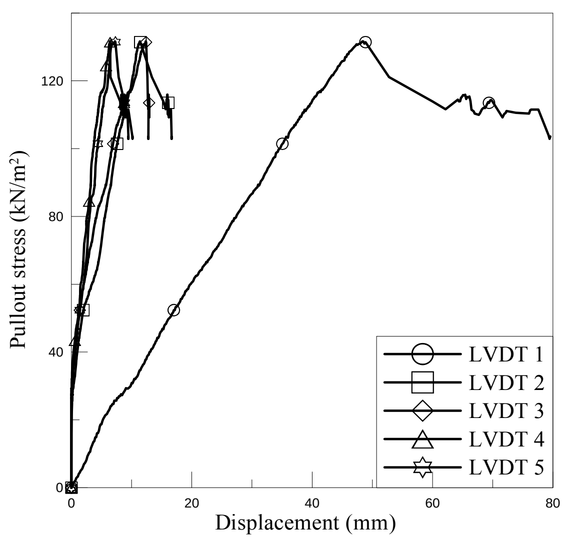
(c) GG & R-Gravel (Normal stress of 49.05 kN/m^2)



(d) GG & R-Gravel (Normal stress of 98.10 kN/m^2)



(e) GG & C-Stone (Normal stress of 49.05 kN/m^2)



(f) GG & C-Stone (Normal stress of 98.10 kN/m^2)

Fig. 5 Pullout test results of the test geogrid and the test soils at normal stresses of 49.05 and 98.10 kN/m^2

Since the riverbed gravel could develop the interlock phenomenon with the test geogrid. It was only required around 60 mm displacements at the front face of the pullout box to develop the passive resistance, however, the corresponding displacements for the LVDT transducers within the box were only about 5 ~ 15 mm. In addition, the pullout resistance would increase and decrease rapidly after the specimen reaching the pullout phenomenon. Even the pullout resistance increased as increasing the normal stress increases, the increased pullout resistance was very little. Therefore, the pullout interaction coefficient decreased as increasing the normal stress. The pullout interaction coefficients and friction angles for the test conditions were 1.251, 0.788, and 0.650, respectively.

The pullout behavior for the geogrid and crushed stone condition was quite similar to that for riverbed gravel. The test results were also showed in the Fig. 5. The pullout interaction coefficient also decreased as the normal stress increased. The pullout interaction coefficients for the test conditions were 0.691, 0.400, and 0.219, respectively. However, the required displacement to mobilize the pullout phenomenon decreased with increasing normal stress.

5.3 Comparison Between the Direct Shear and Pullout Test Results

The comparison between the test results obtained from the direct shear tests of test soils, direct shear tests of the test geosynthetics and the test soils, and the pullout tests of the geosynthetics embedded in the test soils was discussed in this section. Figure 6 showed the test results for the test geotextile and test soils under 49.05 and 98.10 kN/m^2 normal stresses only. The test results related to the normal stress of 149.15 kN/m^2 were omitted due to limited spaced. The direct shear test of the quartz sand showed slightly higher peak shear strength than that for the direct shear test of geotextile and quartz sand. However, the difference in ultimate shear strength for these two conditions was relatively higher than that for the peak strength. This implied more dilation of the quartz sand occurred during the direct shear test of the geotextile and quartz sand. The required displacement for the development of ultimate shear strength was only about 20 ~ 30 mm. This figure also indicated that it required more than 100 mm displacement to reach the pullout mechanism in the pullout test. The required pullout displacement increased with the increasing of normal stress. This implied that the dilation phenomenon was more significant in the pullout test. The unit surface shear resistance for the pullout test definitely would be less than that for the direct shear test.

It was quite interesting to know that the direct shear resistance for the riverbed gravel and the geotextile was much lower than that for direct shear strength of riverbed gravel itself as shown in the Fig. 6. It was implied that the riverbed gravels could sliding on the geotextile surface during the direct shear tests. The required pullout displacement increased as increasing the normal stress, and the pullout resistance was lower that the both direct shear tests.

Figure 6 also showed the comparison test results for the crushed stone and the geotextile. No significant peak strength was observed for the direct shear tests under both 49.1 and 98.1 kN/m^2 normal stress conditions. The direct shear tests of crushed stone showed the highest shear strength. Even the shear strengths for the direct shear or pullout test for the crushed stone and geotextile were similar to each other under both normal stress conditions, however, the required the displacement to reach the ultimate strength was quite difference to each other which implied the failure mechanism was quite different to each other.

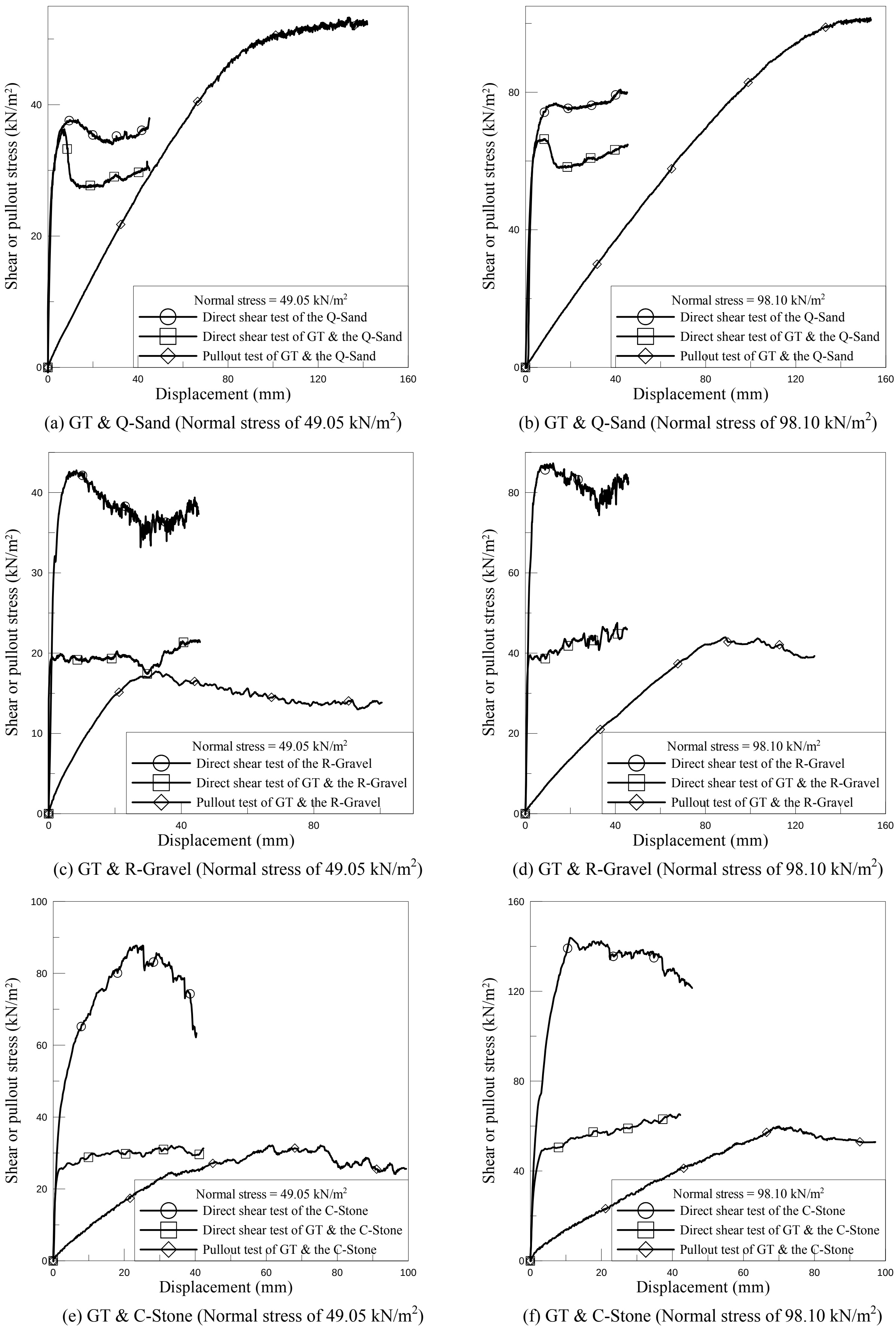
Figure 7 showed the direct shear tests and pullout test results for the geogrid and the quartz sand. The shear stress versus displacement curves for the both direct shear tests under 49.1 and 98.1 kN/m^2 test conditions were almost identical to each other. The required displacement to reach peak shear strength was about 10 mm. The difference between the peak and ultimate shear strength was only about 3 ~ 5 kN/m^2. It implied that the shearing plane was not near the soil/geotextile interface and should be occurred above the geogrid and soil interface. The required displacement to reach pullout condition was about 25 ~ 40 mm. The pullout resistance gradually increased after the initial pullout developed for 98.1 kN/m^2 normal stress condition. It is expected that the passive resistance was gradually developed and the soil dilation was progress occurred during the pullout test. Due to the dilation of the quartz sand, the pullout interaction coefficient was less than the friction efficient. Similar test results also show for the pullout test of the riverbed gravel and the geogrid.

Figure 7 also summarized the test results for the geogrid and crushed stone for the two test conditions with two normal stresses. Since only the interlock phenomenon significantly developed for the pullout test at the crushed stone and geogrid interface, the passive resistance was developed and pullout resistance was relatively higher than the shear resistance for both direct shear tests. The maximum pullout resistance was slightly different under different normal stress. However, the difference in maximum pullout resistance and shear resistance was not significant as increasing the normal stress. It implied that the shear resistance for the crushed stone and geogrid was proportionally related to the normal stress and the pullout resistance was almost no relationship with the normal stress.

6. SUMMARY AND CONCLUSIONS

Generally, a series of direct shear tests and pullout tests according to ASTM D5321 and D6706 standard test methods respectively were performed in the study. The conclusions of the study were summarized as followed:

1. The failure mechanism of direct shear test and pullout test are quite different. One sliding surface within soil or at soil/geosynthetic interface is commonly observed in direct shear test. Two sliding surfaces above and below test soil or at soil/geosynthetic interface are existed in pullout test.
2. Geosynthetics (geotextiles and geogrids) are often provided anchorage for reinforced earth structures, this anchorage resistance can be modeled in the laboratory using a pullout test. The pullout resistance is a function of soil gradation, plasticity, as-placed dry unit weight, moisture content, length and surface characteristics of the geosynthetic and other test parameters.
3. The interface friction angle and adhesion between a geosynthetic and soil are also important variables for geosynthetic reinforcement structure design and analysis.



(a) GT & Q-Sand (Normal stress of 49.05 kN/m^2)

(b) GT & Q-Sand (Normal stress of 98.10 kN/m^2)

(c) GT & R-Gravel (Normal stress of 49.05 kN/m^2)

(d) GT & R-Gravel (Normal stress of 98.10 kN/m^2)

(e) GT & C-Stone (Normal stress of 49.05 kN/m^2)

(f) GT & C-Stone (Normal stress of 98.10 kN/m^2)

Fig. 6 Comparison between the results of the direct shear tests and pullout tests for the test geotextile and test soils

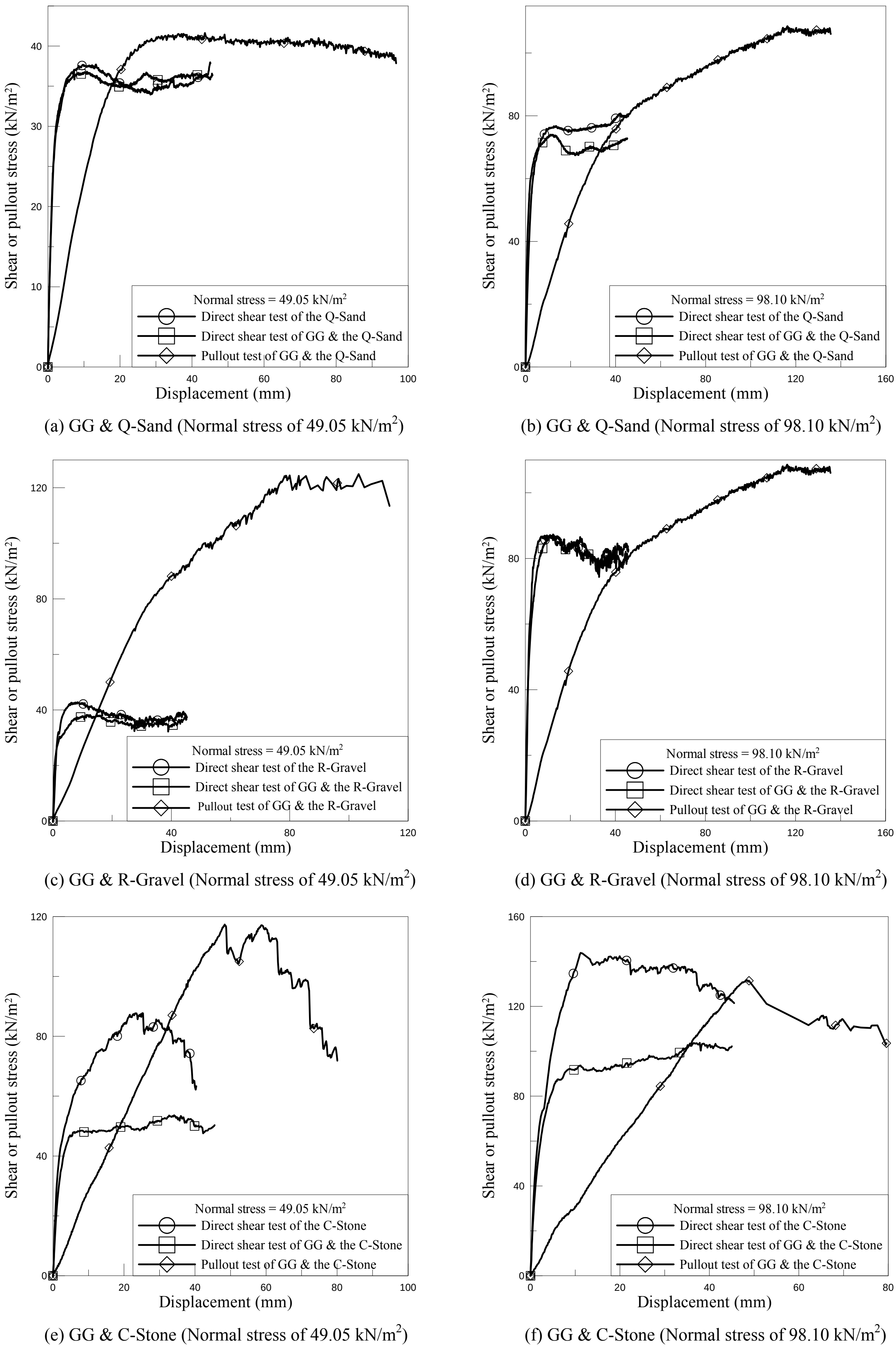


Fig. 7 Comparison between the results of the direct shear tests and pullout tests for the test geogrid and test soils

4. For pullout test, if the soil particles are smaller than the geosynthetic openings, which allow for soil strike-through from one side of the geogrid to the other. The interaction coefficients are high, and the coefficient decreased as increasing the normal stress. If not, they can be low and the interaction coefficient is about 50% to 65% of the friction efficient for the direct shear test. In general, pullout test resistances are less than the sum of the direct shear test resistances. This is due to the fact that the geosynthetic exhibits large deformation and causes the soil particles to reorient themselves into a reduced shear strength mode at the soil-to-geotextile interfaces, resulting in lower pullout resistance.
5. For direct shear test, if the soil particles are smaller than the geosynthetic openings, which allowing the soil particles be interlocked with geosynthetic. This could cause the shearing surface occurred above the soil/geotextile interface, and the frictional behavior would quite similar to that of the test soil itself. If not, the soil particles were expected to turn around and slide along the geosynthetic surface, no significant peak shear strength could be observed, and low friction efficiency at ultimate strain was commonly obtained. The ultimate friction angle for the geotextile and the riverbed gravel was only 20.96^{o}.

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