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Evaluation of Interface Shear Strength Properties of Geogrid-Reinforced Construction and Demolition Materials using a Modified Large Scale Direct Shear Testing Apparatus

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Abstract

The interface shear strength properties of geogrid-reinforced Construction and Demolition (C&D) aggregates were determined using a modified large scale direct shear test (DST) apparatus. Comparisons were made between the results of the various C&D aggregates reinforced with biaxial and triaxial geogrids as well as with the unreinforced aggregates by means of the modified and conventional DST methods. The modified DST method employed, sought to increase interlocking between the C&D aggregates with the geogrids and thus ascertains the true interface shear strength properties of the recycled demolition aggregates. Biaxial and triaxial geogrids were used as the geogrid-reinforcement materials. The C&D aggregates tested with the DST were Recycled Concrete Aggregate (RCA), Crushed Brick (CB) and Reclaimed Asphalt Pavement (RAP). The modified DST results indicated that the interface shear strength properties of the geogrid-reinforced C&D aggregates were higher than that of the conventional test method as well as the respective unreinforced materials. Geogrid-reinforced RCA was found to have the highest interface peak and residual shear strength property of the C&D materials. RAP was found to have the smallest interface shear strength properties of the C&D aggregates. The higher stiffness triaxial geogrid attained higher interface shear strength properties than that of the lower stiffness biaxial geogrid. The modified device also showed some increased measured interface coefficients compared to a conventional DST. The geogrid-reinforced recycled C&D aggregates was found to meet the peak and residual shear strength requirements for typical construction aggregates used in civil engineering applications.

Keywords: recycled materials; waste; demolition; geogrids; interface shear strength; direct shear test.

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Introduction

The interface shear strength of geosynthetic-reinforced structures can be determined with the usage of the DST apparatus (Liu et al. 2009a, 2009b; Palmeira and Antunes, 2010; Zekkos et al. 2010). In recent years, the large scale DST apparatus has been increasingly used to determine the interface shear strength of geosynthetic-reinforced structures with various soils, aggregates (Liu et al. 2009a, 2009b; Kazimierowicz, 2007; Araujo et al. 2009; Rowe and Taechakumthorn, 2011; Palmeira et al. 2010) and other materials such as municipal solid waste (Zekkos et al. 2010).

Geogrids are used as a reinforcement material in various geotechnical engineering applications such as roads (Palmeira and Antunes, 2010) and railway embankments (Arulrajah et al. 2009, Arulrajah et al. 2013a). The drained internal friction angle (ϕ') and cohesion (c') of geogrid interfaces with soils or aggregates are the key input parameters for the design of earth structures reinforced with geogrids. As geogrids have longitudinal and transverse ribs, the interaction mechanisms between geogrids with soils or aggregates, under direct shear mode, provides frictional resistance between the soil and the surface of the geogrids as well as internal shear resistance of the soil and passive resistance of the transverse ribs (Liu et al. 2009a, 2009b; Alfaro et al. 1995; Tatlisoz et al. 1998). The apertures of geogrids furthermore provide significant passive resistance on geogrid-soil interfaces (Bergado et al. 1993).

Interface shear strength properties of soil mass reinforced with geogrid materials has been reported by various researchers to be lower than that of the unreinforced control materials in direct shear tests by the conventional method, which has been attributed to the lack of

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interlocking between the geogrids and the soil/aggregates (Liu et al. 2009a, 2009b; Lee and Manjunath, 2000; Abu-Farsakh et al. 2007; Ling et al. 2008; McCartney et al. 2009).

A modified testing method has been employed in this research to determine the interface shear strength properties of C&D aggregates and to compare the results with the conventional test method. The relative displacement between recycled C&D materials and geogrid to be mobilized has not been ascertained to date, hence the need for this research to ascertain the peak and residual shear strengths of the recycled C&D materials by means of the DST. The peak shear strength represents the best case scenario of full mobilization of friction between the C&D aggregates and the geogrids, whereas the residual shear strength represents the worst case situation for example after failure and hence both peak and residual shear strength properties are relevant.

Recycling of C&D waste materials into sustainable civil engineering applications is of global importance, as we seek new ways to conserve our natural resources as well as reduce reusable waste materials from being landfilled (Aatheesan et al. 2010; Hoyos et al. 2011; Arulrajah et al. 2013b; Rahman et al. 2013). C&D aggregates have recently been found to be viable alternative materials in civil engineering applications such as pavements, footpaths and other road construction applications. This includes C&D aggregates such as RCA (Gabr and Cameron, 2012; Azam and Cameron, 2012; Poon and Chan, 2006a, Poon and Chan 2006b; Arulrajah et al. 2012a; Arulrajah et al. 2013c), CB (Aatheesan et al. 2010; Arulrajah et al. 2011; Arulrajah 2012b), RAP (Taha et al. 2002; Hoyos et al. 2011; Puppala et al. 2011; Arulrajah et al. 2013d; Arulrajah et al. 2013e), crushed glass (Ali et al. 2011; Arulrajah et al. 2013f; Disfani et al. 2011; Disfani et al. 2012; Imteaz et al. 2012) and waste excavation rock (Arulrajah et al. 2012c). However, the properties of these alternative C&D aggregates are not

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fully understood and hence their usage in civil engineering applications is still limited. Research and evaluation of the geotechnical engineering properties of these C&D aggregates, such as their usage in reinforcement with geogrids as in this study, is therefore required to understand the behaviour of these alternative materials when reinforced with geogrids.

Experimental Procedure

A large scale DST apparatus measuring 305 mm in length x 305 mm in width x 204 mm in depth was used in the experimental works. The testing apparatus has two boxes, a fixed upper box and a moveable lower box; each 100 mm in depth. The large scale DST apparatus was undertaken by the conventional test method as well as compared with a modified method with the use of a geosynthetic-clamping steel frame of 7 mm thickness attached to the top of the lower shear box. Testing of geogrids with the modified shear box arrangement would induce a shear plane 7 mm above the geogrid placement level. Fig. 1(a) presents a schematic diagram of the large scale DST apparatus when used by the modified testing method. The steel frame and geogrid were fixed to the lower shear box using several screws and a rough surface plate is shown in Fig. 1(b). The steel frame used just fitted into the shear box and had a provision to fix geogrid in the back and front sides of the steel frame. The authors' hypothesis is that a stiffened zone is present below the conventional DST shear plane leading to higher peak and residual shear stresses. The works of Konietzky et al. (2004) and McDowell et al. (2006) is in line with this hypothesis. A 7 mm thick steel frame was selected as the aggregate size used for local road pavement subbase applications is typically less than 14 mm. The concept was to induce a shearing plane at the midpoint of the aggregates and to achieve gridlock interaction. The geogrid was placed 7 mm below the shear plane to ensure the maximum size of particles interlock with the geogrid as well as being equally distributed in the upper and lower boxes. A steel plate was also used to prevent slippage of the geogrid during the shear tests. The geogrid fixed at the shear plane in the conventional DST apparatus moves with the lower shear box on application of shearing load and develops sagging and slipping tendencies. Hence, to prevent sagging and slipping, the geogrid in the modified method was placed 7 mm below the shearing plane where sufficient geogrid interaction is achieved without sagging or slipping. At this location, the provision of a smooth interface is avoided and significant interlock is realised thereby better representing the true field conditions.

Fig. 2(a) and Fig. 2(b) show the interaction mechanism between aggregates and geogrid on application of shear and normal load for the conventional and modified DST, respectively. The mechanism shown in Fig. 2(b) shows the additional shear resistance and friction that develops by using the modified DST method due to geogrid placed below the critical shear plane.

The large scale DST was undertaken on the reinforced C&D aggregates with Biaxial (Biaxial) geogrids with square apertures and Triaxial (Triaxial) geogrids with triangular apertures. Control tests were also undertaken on unreinforced C&D aggregates to study the effect of the geogrid-reinforcement. Commercially available biaxial geogrids (Biaxial) with an ultimate strength of 20 kN/m and triaxial geogrids (Triaxial) with a slightly higher ultimate strength of 32 kN/m were used in the tests. Table 1 summarises the physical and geotechnical properties of the geogrids. The C&D aggregates tested were RCA, CB and RAP obtained from a recycling site in Melbourne, Australia. The C&D aggregates had a maximum particle size of 19 mm.

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Physical tests undertaken on the C&D aggregates, included modified compaction test, particle density, particle size distribution and water absorption. Modified compaction tests were conducted according to ASTM D1557 (2009) to determine the maximum dry density and optimum moisture content of the C&D aggregates. A cylindrical mould having an internal diameter of 152.4 mm was used in the modified compaction tests.

The particle size distribution tests of the samples were conducted by sieve analysis according to ASTM D422-63 (2007). CBR tests were carried out according to ASTM D1883 (2007) on specimens subjected to modified Proctor compaction effort at the optimum water content and soaked for 4 days to simulate the worst-case scenario.

Organic content tests were performed in accordance with ASTM D2974 (2007). The loss by ignition method was used to determine the organic content of the samples. Particle density and water absorption tests of coarse aggregate (retained on 4.75 mm sieve) and fine aggregate (passing through 4.75 mm sieve) were carried out according to AS1141.5 (SAA 2000a) and AS1141.6.1 (SAA 2000b), respectively.

For the DST, oven dried C&D samples were mixed with water at optimum moisture content and kept in a cool place for approximate 12 hours in a closed container to ensure that water was mixed uniformly with the samples. Initially the lower and upper boxes were clamped when preparing samples for the tests. Lubricating oil was used on the platform of the shear box to enable easy movement. The samples were compacted in the shear box in three layers by using a vibratory compactor at 98% of maximum dry density.

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The large DST was conducted for the geogrid-reinforced and unreinforced C&D aggregates at normal stresses of 30 kPa, 60 kPa and 120 kPa. The horizontal displacements, vertical displacements and shear stresses were recorded with LVDTs and load cells which were computer controlled with a specialized software program. When the consolidation stage for the test was completed, the connection between the lower and upper boxes was released, which provided an approximate 2 mm gap between the upper and lower boxes for friction minimization. The tests were conducted as per ASTM D5321 (2008). The tests were terminated once the horizontal shear displacement reached approximately 75 mm. The peak and residual shear strength of the unreinforced and reinforced C&D aggregates from the DST were obtained from the shear stress and horizontal displacement output graphs.

Results and Discussion

The physical properties of RCA, CB and RAP aggregates obtained from the laboratory tests are summarized in Table 2. The physical properties were tested from three replicate samples for each test. Three samples of each C&D materials was tested to maintain consistency of the results, the ranges and mean values of which are presented in Table 2. The variability of the test results was approximately 5-10% and shows that there is little variation as the tests were performed under same laboratory conditions. The small variation in the test results satisfies the specific requirements for each test. The particle-size distribution results for RCA, CB and RAP undertaken before and after compaction with modified compaction effort, is shown in Fig. 3. It is observed that there is only minimal breakdown of the C&D aggregates noted after compaction as compared to before compaction. The particle size distribution curves for the C&D aggregates were consistent with the requirements of typical aggregates in civil engineering applications (Aatheesan et al. 2010; Arulrajah 2011) which indicates the

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materials as suitable aggregates for civil engineering applications such as pavement subbases/bases and footpaths.

The particle densities of coarse aggregates (retained on 4.75 mm sieve) are generally higher than that of fine aggregates (passing 4.75 mm sieve) except for CB. The water absorptions of coarse aggregates were less than the fine aggregates except for CB. The CBR tests results for RCA and CB were found to be within values of 80-120 normally specified for pavement base and subbase aggregates (Arulrajah 2013b). The organic content of the C&D aggregates was low. The modified compaction results for the RCA, CB and RAP were found to be typical of that specified of construction aggregates.

The DST were conducted at three different normal stresses for each conventional and modified test. The samples were compacted with modified compaction efforts and a constant shear displacement rate of 0.025 mm/min was maintained throughout the shearing stage. The DST results were obtained from computer controlled program. The large scale DST results were analysed to determine the effect of interface shear strength of the geogrid-reinforced C&D aggregates by the conventional and modified method.

Fig. 4, presents the shear stress and vertical displacement versus horizontal displacement curves for unreinforced and reinforced RCA. Shear stress is observed to increase for the RCA, RCA+Biaxial and RCA+Triaxial with an increase in the normal stress. The shear stress is observed to reach a peak shear stress and then levels off to a residual shear stress under large strain. The peak interface shear stress of reinforced RCA+Biaxial and RCA+Triaxial is found to give results that are higher than the peak shear stress of unreinforced RCA. The shear stress of RCA+Triaxial is found to be higher than that of RCA+Biaxial, which is

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expected, as the stiffness of the triaxial geogrid is higher than that of the biaxial geogrid. In Fig. 4, positive vertical displacement indicates contractive behaviour while negative vertical displacement indicates dilative behaviour. The results indicate that an initial vertical contraction takes place until the sample cannot compress which is followed by dilation. The samples are observed to behave like dense materials which similarly show increasing higher compression with an increase in normal stress. Similar behaviour has been reported (Disfani et al. 2011) for DST on dense recycled glass. An increase in normal stress is observed to increase the tendency of compression. The contraction effect at the beginning of the test is observed to become more noticeable as the normal stress levels are increased.

Fig. 5 presents the shear stress and vertical displacement versus horizontal displacement curves for unreinforced and reinforced CB with geogrids by the modified DST method. Fig. 6 presents the shear stress and vertical displacement versus horizontal displacement curves for unreinforced and reinforced RAP by the modified DST method. Similar trends are apparent in these figures for the modified DST methods, with similar higher shear strengths observed for the geogrid-reinforced aggregates as compared to the respective unreinforced aggregates and similar higher shear stresses recorded for the Triaxial as compared to the Biaxial geogrids.

The shear strength parameters of cohesion (c') and internal friction angle (ϕ) of the C&D materials were obtained from Mohr Coulomb failure envelope line. Fig. 7 presents the Mohr Coulomb failure envelope lines from peak shear stress for the conventional and modified DST. Fig. 8 presents the residual shear stresses for the conventional and modified DST. The peak and residual shear stresses parameters are summarised in Table 3. Table 3 also presents the comparison results of peak and residual shear strengths for the geogrid-reinforced C&D

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aggregates by using the conventional and modified interface shear strength testing methods as well as the unreinforced C&D aggregates. RCA has shown high apparent cohesion values for the reinforced and the unreinforced DST because RCA has gained strength from further hydration by absorbing water from the mix. Touahamia et al. (2002) and Piratheepan et al. (2013) have reported similar high apparent cohesion value for RCA for the same reason. The crushed clay particles from the CB during compaction has mixed with water and formed a binding paste, leading to a high apparent cohesion value (Piratheepan et al. 2013). The apparent cohesion value of RAP is not as high as that of RCA and CB, however, the small apparent cohesion in RAP can be attributed to the presence of bitumen which will bind during compaction (Piratheepan et al. 2013). It is apparent from these figures and table that the interface shear strength and the interaction between the geogrid-reinforced C&D aggregates is higher than that of the unreinforced C&D aggregates and for the Triaxial as compared to the Biaxial geogrid.

The interface or interaction between geosynthetics-soil reinforcement behaviour can be expressed as the coefficient of soil-reinforcement friction (Liu et al. 2009b; Tatlisoz et al. 1998; Bergado et al. 1993; Lee and Manjunath 2000). The interface shear strength coefficient is obtained from the following equation.

$$\alpha = \frac{\tau_{reinforced}}{\tau_{unreinforced}} \qquad \text{Eq. (1)}$$

Where α is the interface shear strength coefficient, $\tau_{\text{reinforced}}$ and $\tau_{\text{unreinforced}}$ are the shear strength values obtained from reinforced and unreinforced DST, respectively. Table 4 presents the interface shear strength coefficient values for the geogrid reinforced C&D aggregates by using conventional and modified interface shear strength testing methods. From Table 4, it is apparent that the interface coefficient between the geogrid-reinforced C&D aggregates improved in the modified test setup as compared to the conventional method

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as well as the respective unreinforced materials. It is also apparent that the interface coefficient is higher for the triaxial geogrids as compared to biaxial geogrids, which is consistent with the findings of the peak and residual interface shear strengths.

Granular soils, such as dense sands and gravels, specified in geotechnical engineering applications typically have peak friction values of 40 to 48 degrees and residual values of 32 to 36 degrees (Sivakugan and Das, 2010). Based on the modified DST results, the geogrid-reinforced and unreinforced C&D aggregates would meet the shear strength requirements for usage as a construction material in civil engineering applications.

The interface shear strength properties of the geogrid-reinforced C&D aggregates were found to be consistently higher than that of the respective unreinforced C&D material in the modified DST set-up. The interface peak shear strength values of the C&D aggregates were noted to be higher than that of the respective residual values, which is as expected. The interface shear strength properties of RCA is observed to be higher than that of CB while RAP is noted to have the smallest interface shear strength properties of the C&D aggregates. The higher strength Triaxial geogrids was found to attain higher interface shear strength properties than that of the lower strength Biaxial geogrids.

The interlocking mechanism is an important parameter for the performance of any geogrid-reinforced pavement subbase materials. Coarse particles placed in the geogrid's apertures lock them in place while applying lateral and vertical forces. This importance of the interlocking mechanism between geogrid and aggregates has been discussed by Jewell et al. (1984). Results obtained from DST with rectangular and triangular shaped apertures, indicates that triangular shape geogrid provided significant stiffness due to them having

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constraints ribs. The interlock between geogrid and aggregate depends on factors such as aggregate particle size, aperture size and the mechanical properties of the geogrid (Tutumluer et al. 2012).

In this study laboratory tests were undertaken to determine the usage of geogrid-reinforced aggregates for subbase applications. Factors that affect their usage include geogrid types, size and shape as well as the aggregates shape, texture, angularity, gradation, moisture content and density. The relative shear stress and displacement between soil and geogrid mobilized during the interface DST also depends on the aggregate quality, reinforcement quality and reinforcement length.

The traditional shearing plane in the conventional DST is set at the weak point at the geogrid placement layer and this is not the reality in the field. In this research, the critical shearing plane has been induced to occur at a distance below the traditional shearing plane. In this research, the assumption has been made that the shearing plane should be induced at half the aggregate size of the geogrid. This is a reasonable assumption for these sizes of recycled aggregates. Mobilization of C&D materials to peak strength can be achieved with small strain, as can be seen in the DST test results.

The peak and residual shear strengths as determined by the DST are not essential for the study of pavement subbase responses which tends to be more inclined towards dynamic tests, such as the repeat load triaxial. However, the authors have undertaken this research to study the fundamental behaviour of the interaction between geogrids and C&D materials as these are relatively new alternative materials for which there is still little fundamental understanding of their properties.

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Conclusions

A large-scale DST apparatus was used to determine the interface shear strength properties of geogrid-reinforced RCA, CB and RAP by means of a modified testing method and results compared with the conventional testing method. The modified test method was undertaken with the use of a geosynthetic-clamping steel frame attached to the top of the lower shear box. Testing of geogrids with this modified shear box arrangement would induce a shear plane above the geogrid placement level, thereby increasing the interlocking between the geogrids with the aggregates.

Tests were undertaken on each of the respective C&D aggregates when reinforced with Biaxial and Triaxial geogrids by both the conventional and modified test methods. Tests were also undertaken on unreinforced C&D aggregates for comparisons. The results of the tests were compared between the reinforced with the conventional and a modified test method, as well as with the unreinforced C&D materials.

The interface shear strength properties of the geogrid-reinforced C&D aggregates for the modified test method were found to be consistently higher than that of the conventional test method and the respective unreinforced material. The interface peak shear strength values of the C&D aggregates were higher than that of the respective residual shear strength values. The interface shear strength properties of RCA was consistently higher than that of CB while RAP had the lowest interface shear strength properties of the C&D aggregates for both the conventional and modified test methods.

The higher stiffness triaxial geogrid was found to attain higher interface shear strength properties than that of the lower stiffness biaxial geogrid. The interface coefficient between

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the geogrid-reinforced C&D aggregates improved consistently in the modified test setup as compared to the conventional test methods and for the respective unreinforced materials. The interface coefficient was found to be higher for the triaxial geogrid as compared to the biaxial geogrid, which is consistent with the findings of the peak and residual interface shear strengths. The interface shear strength coefficients were obtained from the peak shear strength of geogrid-reinforced and unreinforced materials. The results show that the interface shear strength coefficient values from the modified DST results showed some increased measured interface coefficients compared to the conventional DST. This is due to extra shear resistance between aggregate and geogrid in the modified test set-up. The interface coefficients depend on the material quality, geogrid quality and applying load. The modified DST apparatus is hence found to be suitable for simulating the true field behavior of pavement subbase materials.

The modified DST method was found to improve the interlocking between the geogrids and the recycled C&D aggregates. The geogrid-reinforced recycled C&D aggregates was found to meet the peak and residual shear strength requirements for typical construction aggregates used in civil engineering applications.

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Table 1: Physical and mechanical properties of the geogrids

Properties	Biaxial	Triaxial
Ultimate Tensile Strength (kN/m)	20	32
Approximate Strain (%)	11	18
Junction Strength (kN/m)	95	135
Unit weight (kg/m^2)	0.22	0.25
Polymer	Polypropylene	Polypropylene
Junction Efficiency (%)	--	90
Aperture size (mm)	39×39	46×46×46

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Table 2. Physical properties of C&D aggregates.

Geotechnical Properties	**RCA**		**CB**		**RAP**	
	Range	Mean	Range	Mean	Range	Mean
Particle density-coarse (kN/m^3)	25.5-27.0	26.50	23.2-24.5	23.6	21.20-24.5	23.0
Particle density-fine (kN/m^3)	24.0-26.5	25.50	23.5-25.8	24.3	22.00-24.0	22.9
Water absorption-coarse (%)	6.5-7.5	6.70	13.0-14.2	13.76	11.50-12.7	12.0
Water absorption-fine (%)	6.5-7.5	7.10	9.5-11.0	10.3	13.05-14.8	13.9
Organic content (%)	1.7-2.1	1.80	1.75-2.1	2.0	3.25-4.7	4.0
Max dry density (kN/m^3)	19.4-21.0	20.4	18.7-21.5	20.0	17.4-20.5	19.0
Optimum moisture content (%)	11.9-13.2	12.5	12.0-13.5	12.7	7.5-9.2	8.3
California Bearing Ratio (%)	166-175	172	127-142	135	31-46	39

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Table 3: Comparison of interface shear strength properties of geogrid-reinforced C&D aggregates by the conventional and modified DST methods.

C&D material	**Geogrid**	**Apparent cohesion (c') (kPa)**				**Interface angle (ϕ')**			
		Peak		**Residual**		**Peak**		**Residual**	
		Conventional	Modified	Conventional	Modified	Conventional	Modified	Conventional	Modified
RCA	-	95	-	80	-	65	-	39	-
CB	-	87	-	30	-	57	-	41	-
RAP	-	15	-	21	-	45	-	38	-
RCA	Biaxial	75	108	25	10	50	69	39	67
CB	Biaxial	67	95	15.5	4.5	45	64	39	68
RAP	Biaxial	6.5	30	12.5	35	40	47	37	23
RCA	Triaxial	83	114	50	12.5	52	71	35	68
CB	Triaxial	80	100	20	12.5	49	66	40	68
RAP	Triaxial	13	39	3.5	36	42	49	40	50

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Table 4: Comparison of interface peak shear strength coefficients of geogrid-reinforced C&D aggregates by the conventional and modified DST methods.

C&D Material	**Geogrid**	**Interface coefficient (α)**						**Mean interface coefficient (μ_α)**	
		Normal stress (kPa)							
		30		**60**		**120**			
		Conventional	Modified	Conventional	Modified	Conventional	Modified	Conventional	Modified
RCA	-	1.00	-	1.00	-	1.00	-	1.00	-
CB	-	1.00	-	1.00	-	1.00	-	1.00	-
RAP	-	1.00	-	1.00	-	1.00	-	1.00	-
RCA	Biaxial	0.66	1.11	0.69	1.24	0.61	1.17	0.65	1.17
CB	Biaxial	0.70	1.11	0.68	1.22	0.64	1.18	0.67	1.17
RAP	Biaxial	0.73	1.36	0.73	1.25	0.80	1.15	0.75	1.25
RCA	Triaxial	0.72	1.19	0.74	1.31	0.65	1.27	0.71	1.26
CB	Triaxial	0.80	1.19	0.83	1.28	0.74	1.27	0.79	1.25
RAP	Triaxial	0.89	1.60	0.87	1.43	0.88	1.28	0.88	1.44

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Figure 1

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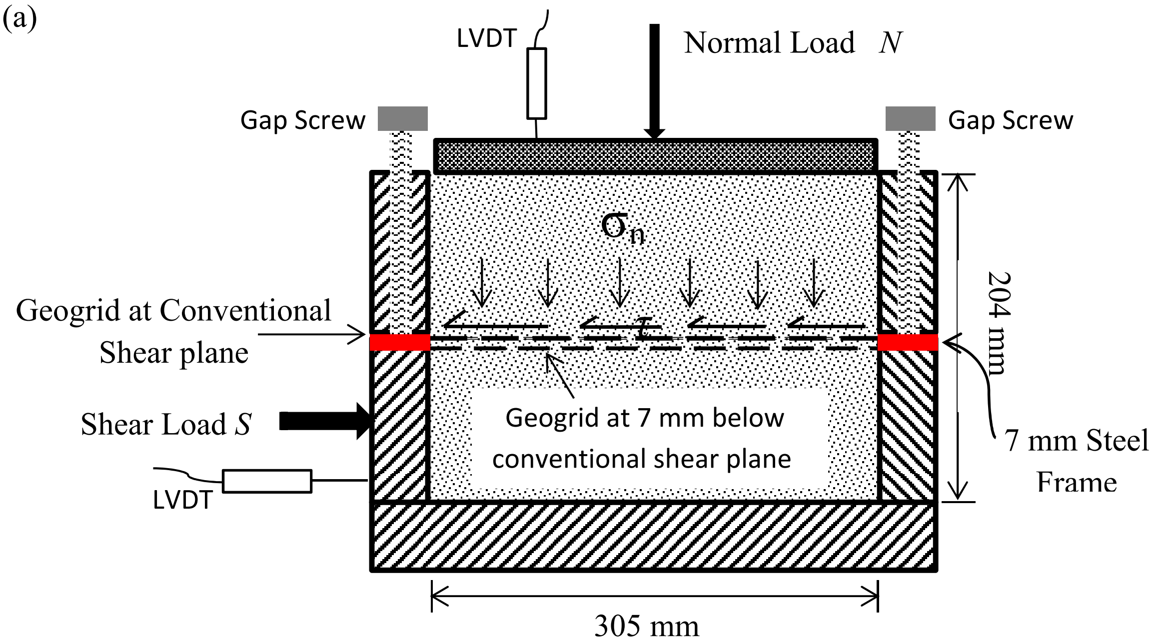
(a)



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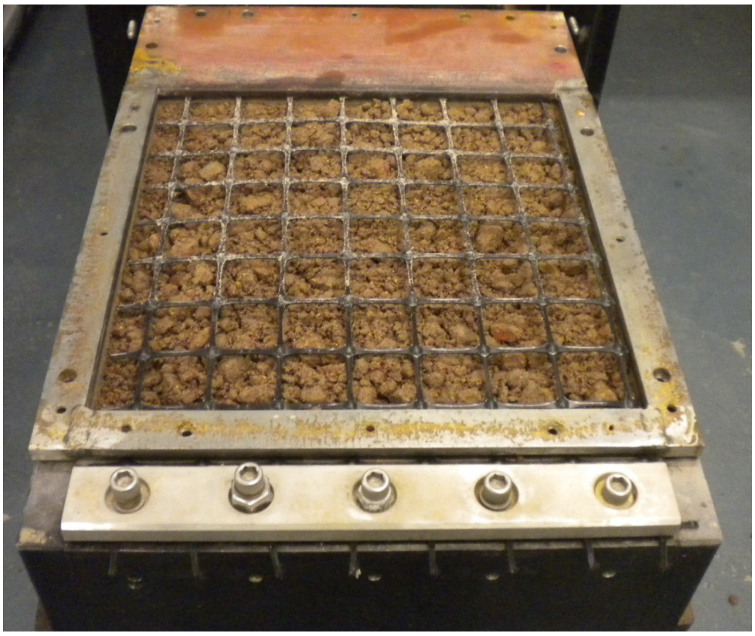

Figure 2

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Fig. 2(a)

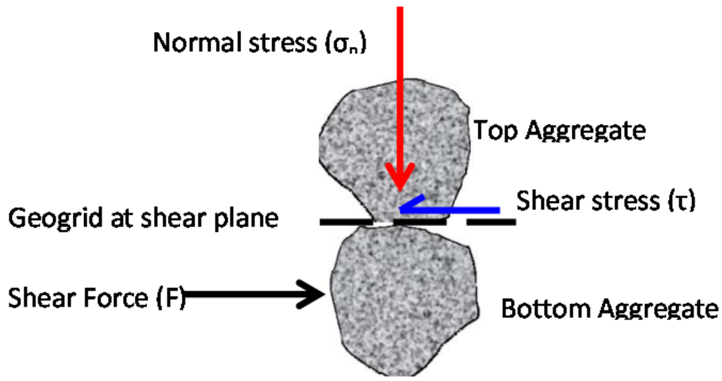

Fig. 2(b)

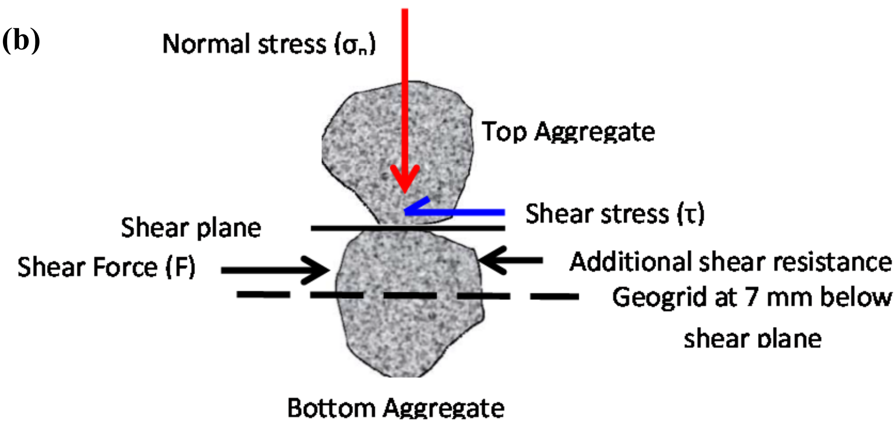

Figure 3

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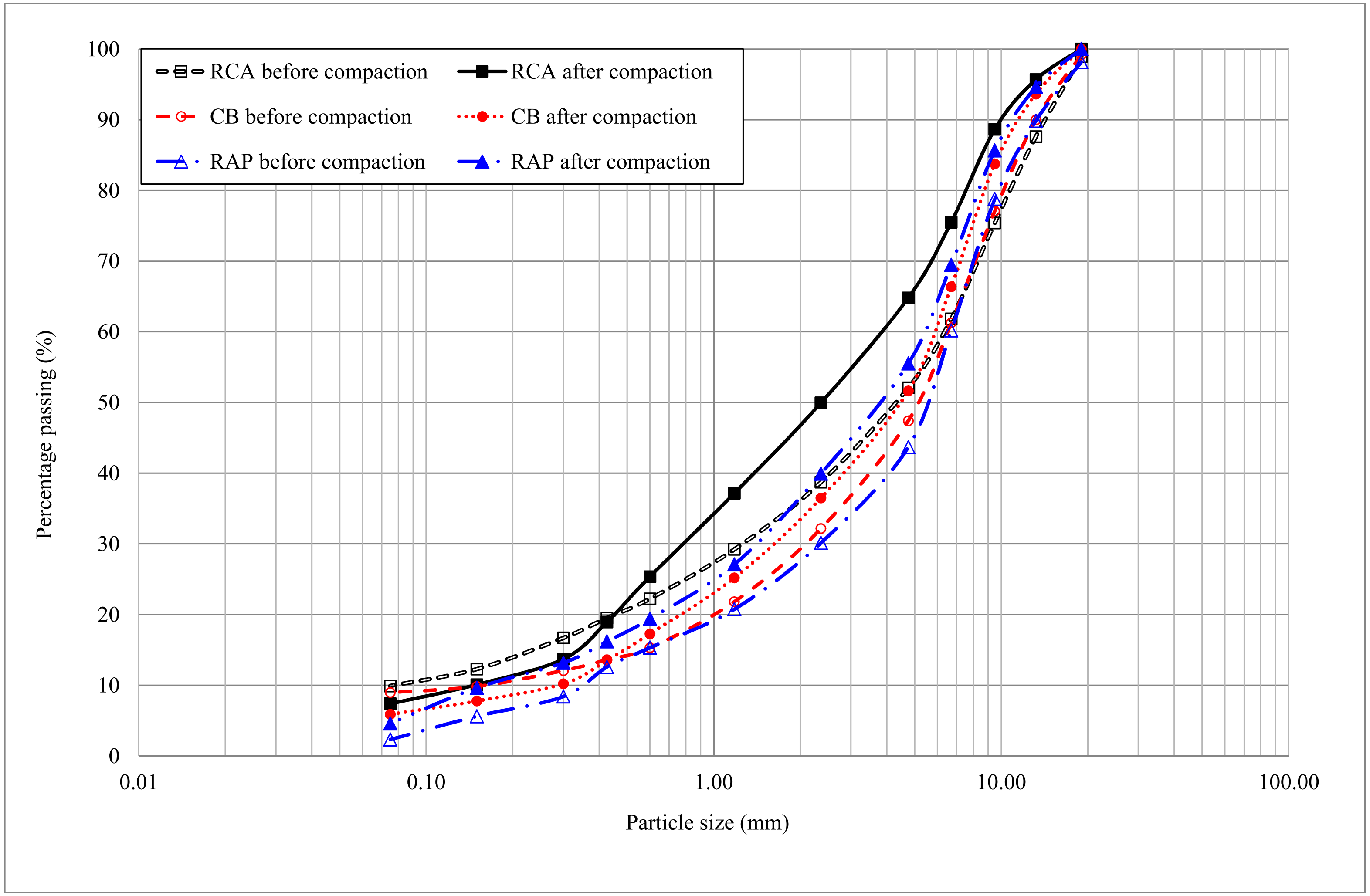

Figure 4

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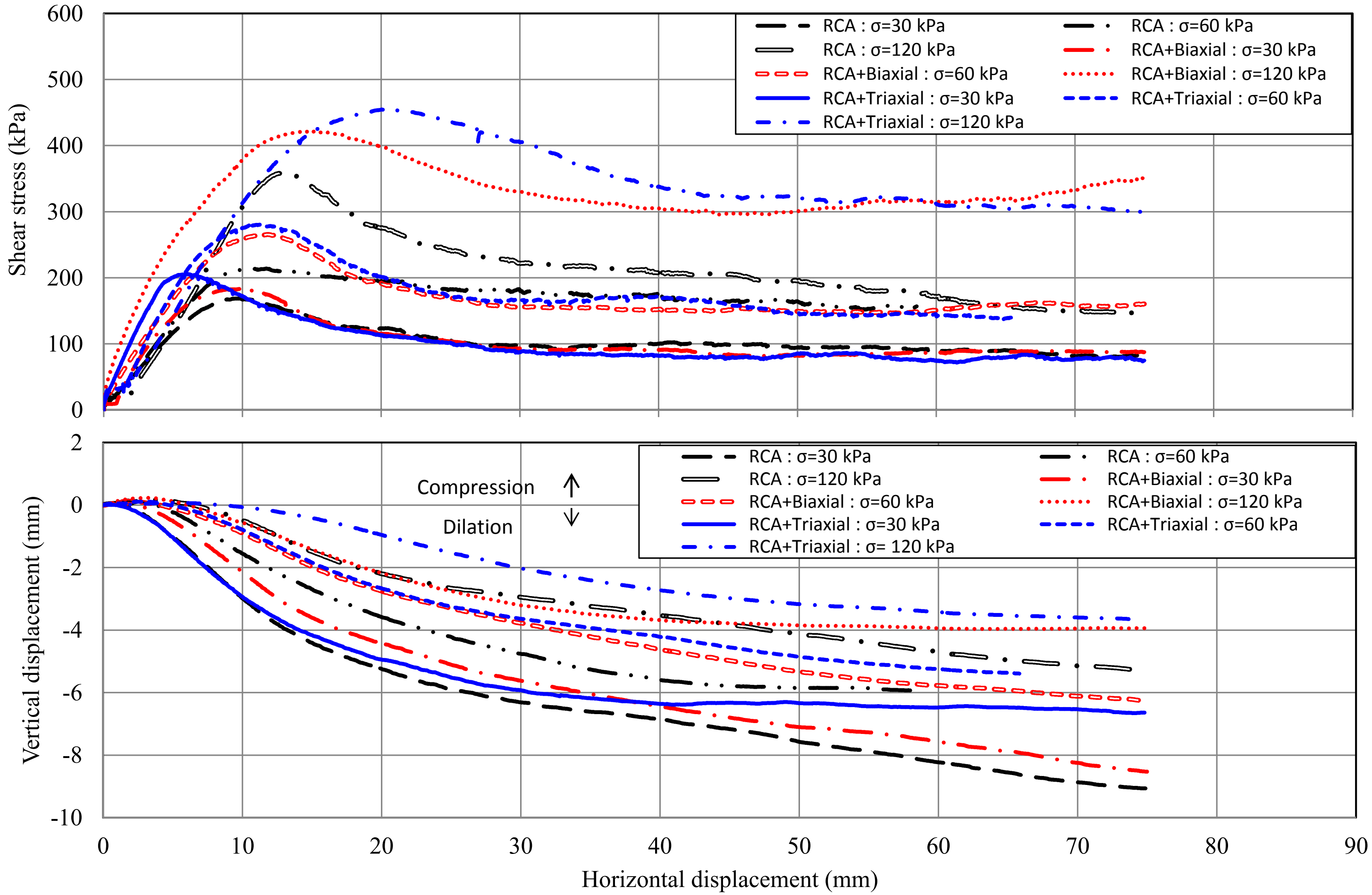

Figure 5

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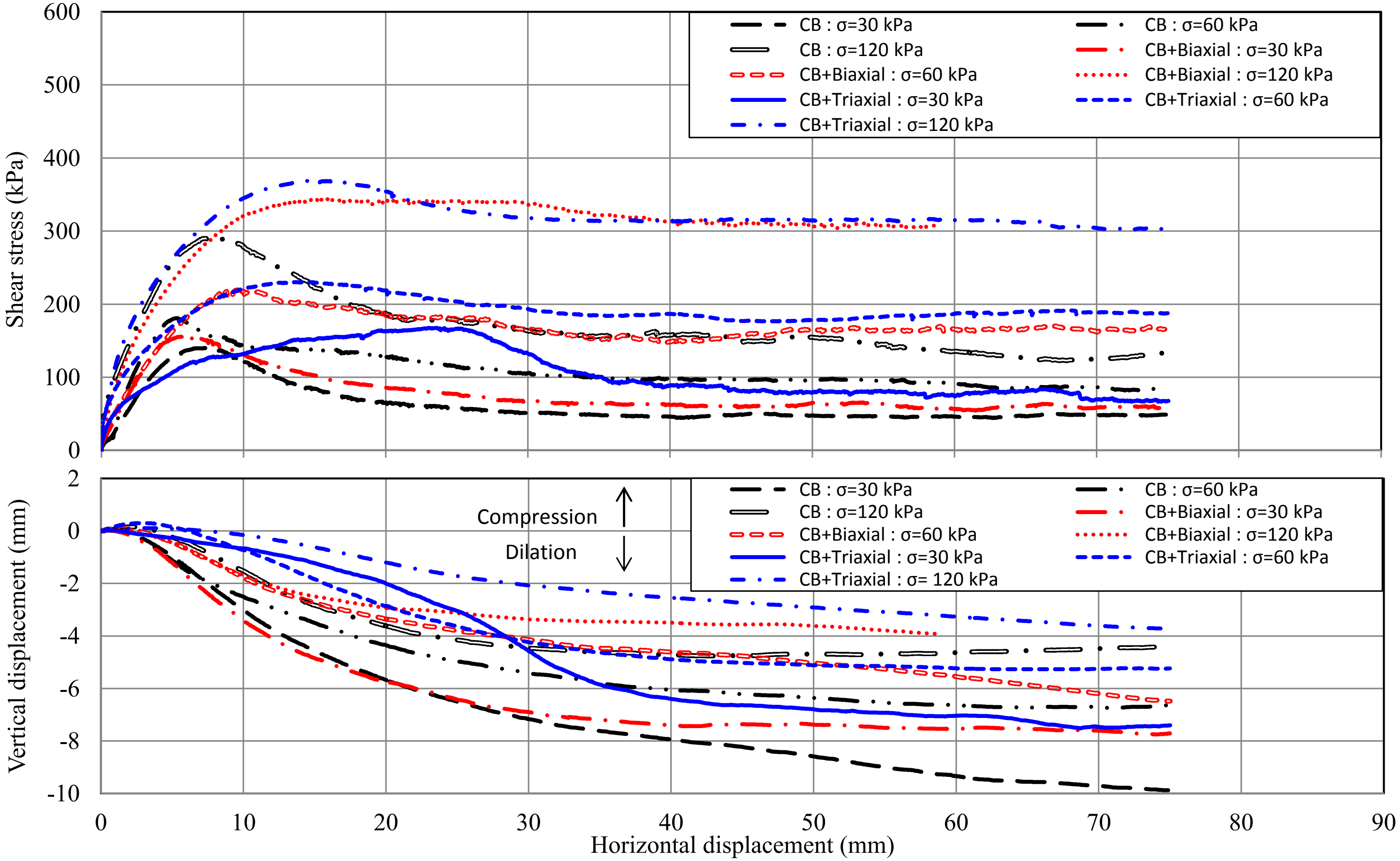

Figure 6

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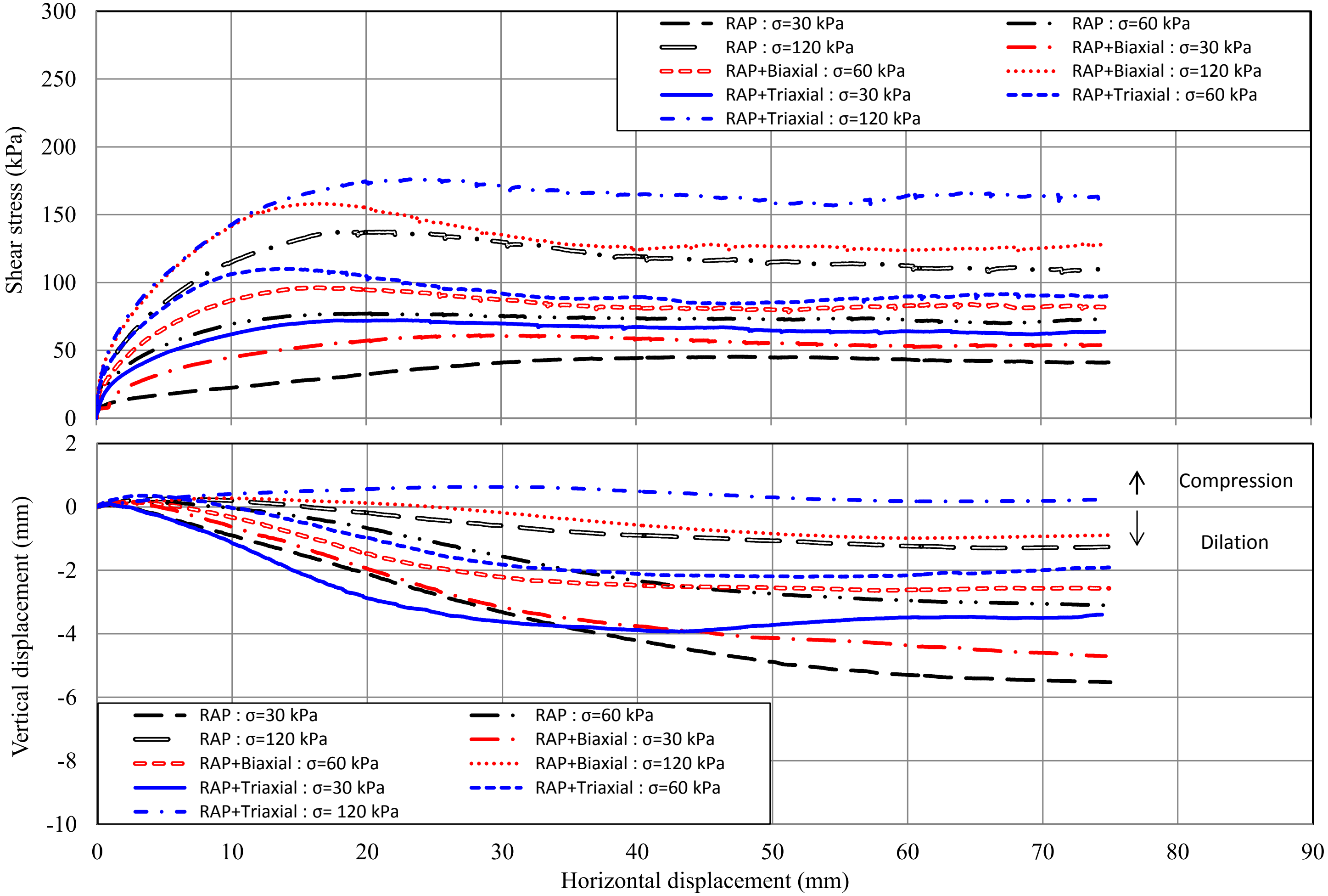

Figure 7

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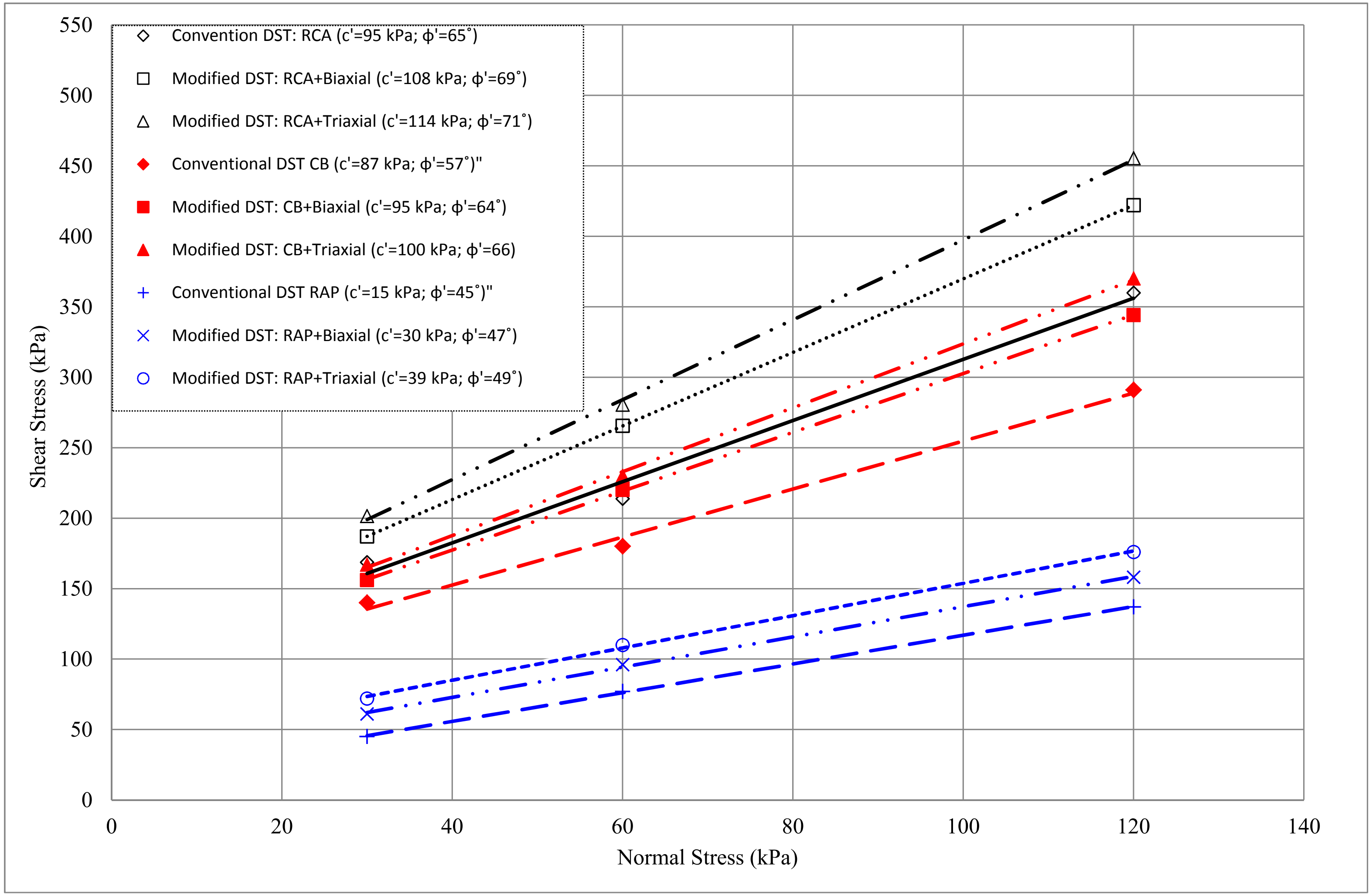

Figure 8

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