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Counterface soil type and loading condition effects on granular/cohesive soil – Geofoam interface shear behavior

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Abstract

Soil – geofoam interfaces have been studied through an extensive experimental program by performing multiple series of interface shear tests using two different granular soils (i.e. beach sand and construction material sand) and one cohesive soil (i.e. bentonite clay) as well as a soil mixture containing 75% sand and 25% clay by dry weight at distinct loading conditions (i.e. normal stresses (σ): 25, 100, 250; low, moderate, high loading conditions, respectively). Using the shear stress versus horizontal displacement curves obtained, some important engineering design parameters including peak shear stress, residual shear stress, interface sensitivity (i.e., peak/residual ratio) and displacement required to reach peak stress have been determined and the variations in those interface mechanical properties as a function of loading condition and counterface soil type have been investigated. It was shown that the peak as well as residual shear stresses increased with an increase in normal stress for all the interface systems tested. Further, the granular soil (sand) interfaces demonstrated relatively larger frictional strengths (both peak and residual) as compared to both the cohesive soil (clay) interface and the sand/clay admixture soil interface. Additionally, the higher the angularity of granular soil particles became, the larger the interface shear strengths (peak and residual), when sheared against geofoams, developed in light of experimental results attained as a result of interface shear tests on different material combinations. For comparison, the detected peak strength at average for the construction material sand, the beach sand, and the sand/clay admixture soil interfaces as compared to the bentonite clay interface were improved 59.8%, 43.4%, and 20.3%, respectively. Additionally, the detected residual strength at average for the construction material sand, the beach sand, and the sand/clay admixture soil interfaces as compared to the bentonite clay interface were improved 53.9%, 28.6%, and 15.4%, respectively.

1. Introduction

The primary function of the geofoams consists of providing; i) lightweight fill for construction on soft ground (i.e., embankment), ii) relatively stiff base for subgrade installation below a highway (i.e., roadway, runway foundation), bridge approach (i.e., abutment backfill), and iii) slope stabilization for retaining structures. In those applications, the geofoams are in direct contact with soils and this interaction results in development of an interface where likelihood of a failure to initiate is higher. For this reason, the frictional resistance and the type of shear response mobilizing at these soil – geofoam interfaces control the stability of composite system, and hence, govern the integrity of the infrastructure.

The sole and/or the mutual mechanical behavior (i.e., tensile, compressive, and shear) of soil (i.e., granular and cohesive) and/or geosynthetics (e.g., geofoam and geomembrane) in infrastructural applications have always attracted the attention of researchers. In this regard, the laboratory and/or the in-situ tests are performed to further evaluate their stand-alone/independent or integrated responses against the induced stresses and strains. To this end, the behavior of expanded polystyrene (EPS) geofoam under triaxial loading conditions was examined by Padade and Mandal [1]. Besides, the behavior of EPS geofoam in true triaxial compression tests was studied by Leo et al. [2]. Moreover, the behavior of EPS geofoam in stress-controlled cyclic uniaxial tests was reported by Trandafir et al. [3]. Additionally, the interface shear strength of EPS–concrete elements of various configurations was investigated by Özer and Akay [4] that shearing resistance between the EPS and precast concrete was found to be higher than that of the flat EPS–EPS interface owing to the improvement of the interface shear behavior as a result of interlocking and the modification of shear mechanism taking place at the interface of EPS–precast concrete from purely frictional to frictional-adhesional. Furthermore, the shear strength properties of compacted high plasticity clay soils based on different laboratory tests were examined by Yilmaz and Türköz [5]. A number of twenty high plasticity soil samples were used and the geotechnical identification tests including direct shear as well as unconfined compression tests were performed such that the shear strength properties of the samples prepared by compression in their compaction characteristics were determined to further evaluate effective shear strength and total shear stress parameters. Further, the geomechanical properties of fiber reinforced cohesive soils were investigated by Ertuğrul and Canogullari [6]. It has been observed that the strength of soil was improved with increasing density of the fibers up to a certain level based on the percentage of added fiber. The performed statistical analysis revealed that the improvement of strength in soil is a function of fiber density, LL/PL ratio (LL: liquid limit; PL: plastic limit). Moreover, the effect of silica fume on the undrained strength parameters of dispersive soils was studied by Öztürk and Türköz [7]. As such, the dispersibility behavior of the soil samples prepared was determined by the crumb test in adding silica fume to improve the soil properties for which the soil specimens mixed with silica fume at different proportions ranging from 0% up to 30% by 5% increments in mass proportion were compacted by using the standard proctor test so that the dispersibility potential of the prepared specimens was evaluated through the crumb tests. Additionally, the strength properties of soil samples were assessed by conducting unconsolidated undrained (UU) triaxial tests. As a result, it was observed that the dispersibility feature of the soil samples was treated and the strength properties were improved accordingly depending on the silica fume content.

Furthermore, geofoam, produced from expanded polystyrene (EPS), has been employed over 45 years in geotechnical applications for the enormous extent of diverse infrastructural projects requires lightweight fill material including highways, railways, airport runways, embankments, bridge abutments, earth retaining structures (i.e., retaining walls), slope stabilization, structural foundation fills, building fills, plaza decks, stadium seating, utility protection and compressible. Among the most versatile lightweight materials available in the industry, geofoam – having predictable material behavior – is an engineered product unlike the other lightweight fill materials that can be variable in composition. Additionally, geofoam is an ultra-lightweight such that it only weighs about 1% weight of soil or rock. Further, the physical properties of geofoam will not degrade (i.e., non-biodegradable) assuring long term performance and durability in engineered geotechnical applications. In those various infrastructural applications aforementioned, the geofoams utilized are in direct contact and interaction with the surrounding soils as those kinds of systems are in composite nature composed of synthetic polymeric materials (i.e., geofoam) and natural soils (i.e., sand, silt, clay mixtures).

The physical, index and mechanical properties as well as material characteristics of synthetics can be controlled and predetermined as opposed to that of natural soils. Moreover, an interaction zone, called interface, between two dissimilar materials will develop such that the mechanical behavior in terms of strength, durability (load-bearing capacity and long-term resistance, respectively) will be governed by the shearing mechanisms and the mobilized frictional properties along the contact surface. To this end, soil – geofoam interface shear behavior plays a critical role in controlling stability, steadiness, and long-lasting performance of those diverse geotechnical applications.

2. Geofoams: Lightweight Fill Applications

In lightweight fill applications, the geofoams are employed principally as specified for providing opportunity; (i) in replacing heavy fill materials to reduce settlement, (ii) safely support super-structural (i.e. overlying structure) loading without over-stressing the underlying soils and to reduce differential movement, (iii) in site development on weak and poor load-bearing foundation soils by reducing the load on underlying compressible soils to minimize deformation, (iv) reduction of lateral pressure on vertical walls as well as improves slope stability by means of stabilization, (v) reduction of induced stress or load on underground structures and/or utilities (i.e. underlying structures and services), (vi) cost-effective solutions where conventional construction methods have failed or demonstrated sub-standard performance. As such, the typical engineering approaches in design and the traditional techniques in construction practices aim to develop an operational mechanism to work for resisting the forces of nature. On the other hand, the design methodology and the implementation of the project using geofoams goal to mobilize a functional execution in order to work with the forces of nature in resolving structural fill challenges such that the geofoams utilized in the infrastructural applications reduce the forces acting on a structure (i.e., retaining walls) or foundation soil (i.e. embankments) instead of reinforcing them to withstand the forces that would exist without geofoam inclusion and/or substitution [8 – 10].

Although, the literature especially on the mechanical behavior (i.e., material response under compression) of geofoams employed infrastructural applications appears in the 1970s, an organized technical knowledge can be found in Horvath [11] where the generic term “geofoam” was proposed to describe rigid-plastic-foams utilized in geotechnical applications. In this regard, this polymeric synthetic material (geofoam) is now recognized as a category of geosynthetics. Seismic response (i.e., dynamic behavior) was first studied extensively by Bathurst and Alfaro [12], and additionally, the results of a finite element method (FEM) study on the seismic response attracted attention presented by Pelekis et al. [13] pointing out that a substantial decrease in seismic earth pressures acting on a retaining structure (i.e., cantilever type retaining wall) protected by a layer of geofoam placed at the wall – backfill interface. Since then, a remarkable amount of research work has been published concerning with mechanical behavior of geofoams including static and dynamic loading conditions such as Zarnani and Bathurst [14]. On the other hand, a combined response analysis of soil – geofoam composite systems have not received sufficient interest in the studies, and thus, a comprehensive and detailed examination in regards to geofoam – surrounding soil contact behavior and resulting interaction mechanisms is required. However, soil – geofoam composite systems, subjected to the action of both static and dynamic loads in the aforementioned geotechnical projects, require rigorous analysis (proper and neat) as well as organized (elaborate and comprehensive) modeling to accomplish proper engineering design of the relevant technical works prior to applying in the field. To this end, in order to achieve this engineering process, the mechanical behavior of geofoam, in particular, the interaction mechanism at the contact surface (i.e., interface shear behavior) with counterface soils needs to be known extensively and evaluated precisely so as that the frictional response could exactly be predicted. Within this scope, a laboratory testing program has been conducted to examine soil – geofoam interface shear behavior under the influence of different loading conditions as well as the effect of counterface soil type as being course or fine grained at the contact surface (i.e., interface).

3. Laboratory Testing Program

The laboratory testing program consists of four series of shear tests on four different composite interface systems comprised of geofoam counterfaced against distinct soils. The physical properties as well as characteristics of the tested material will be described in this section along with some detailed information provided on testing device and experimental procedures, and additionally, further explanation given on the test matrix.

3.1. Geofoam characteristics and soil physical properties

The geofoam utilized in the entire experimental program is a polymeric material produced from expanded polystyrene (EPS). As such, the EPS geofoam is foam plastic, that is, thermoplastic closed-cell polymer based building and infrastructure material manufactured from hard foam blocks of polystrol granules. Further, geofoams, being glassy foam based polymeric materials, exhibit visco-elastic behavior when subjected to load application. Therefore, the compressive stress at 1%, 2%, 5%, and 10% strains (i.e., deformations) is measured and reported instead of compressive strength. To this end, the physical and mechanical properties of the EPS geofoam utilized throughout the entire laboratory experimental program are listed in Table 1. This type of geofoam possessing relatively high density and intact material characteristics was intentionally selected owing to the fact that this is a generally preferred geosynthetic material for the design of infrastructural projects as well as for the construction of relevant geotechnical applications containing geofoam and surrounding soil interfaces which involves in the interaction of granular and/or cohesive soil versus geofoam over the entire extent of contact area between those counterface materials that is exactly being the scope of this research study. As per polymeric material characteristics, the geofoam used in the entire testing program possesses relatively larger bearing capacity properties such that the resultant amount of compressive deformation mobilized due to compressive forces/stresses is generally marginal. This shows that the material inherent ability in terms of the generated compressive resistance against loading for the geofoam is very considerable, remarkable and significant compared to the other infrastructural construction materials such as natural soil.

Table 1. Physical and mechanical properties of EPS Geofoam used in testing program.

Physical and Mechanical Properties	Density (kg/m^3)	Compressive Resistance (kPa), min. at 2% deformation	Compressive Resistance (kPa), min. at 5% deformation	Compressive Resistance (kPa), min. at 10% deformation	Flexural Strength (kPa), min
Geofoam	20	60	80	100	150

Furthermore, although the EPS geofoams possess relatively high strength as compared to most of the construction materials, it has very low density attained by expanding and compressing various closed air (gas) filled cells.

Moreover, two types of poorly graded granular soil were selected to be used as a specimen for the interface shear tests in the experimental program. The physical and index properties (Table 2) of sand specimens are similar to each other whereas the construction material sand composed of grains (i.e., particles) possessing angular features in comparison to that of beach sand possessing subrounded features. This was purposefully intended to examine the influence of sand particle shape and geometry on the developed interface shear behavior and on the mobilized frictional strength characteristics such as engineering design parameters including peak shear stress, residual shear stress, interface sensitivity and displacement to peak. The test methods followed to determine those required physical and index properties of granular soils used in the experimental program are listed in Table 2 for clarification. Further, both beach sand and construction material sand soils are classified as SP (poorly graded sand) according to Unified Soil Classification System (USCS). Additionally, the bentonite clay used in the experimental program is classified as high plasticity clay (CH) based on USCS. The construction material sand (75%)/bentonite (25%) clay admixture soil can be classified as poorly graded sand with clay (SP-SC) per USCS, accordingly. Furthermore, the mean grain size (D_{50}) and the effective particle size (D_{10}) for the tested bentonite clay are 79 μm and 46 μm, respectively.

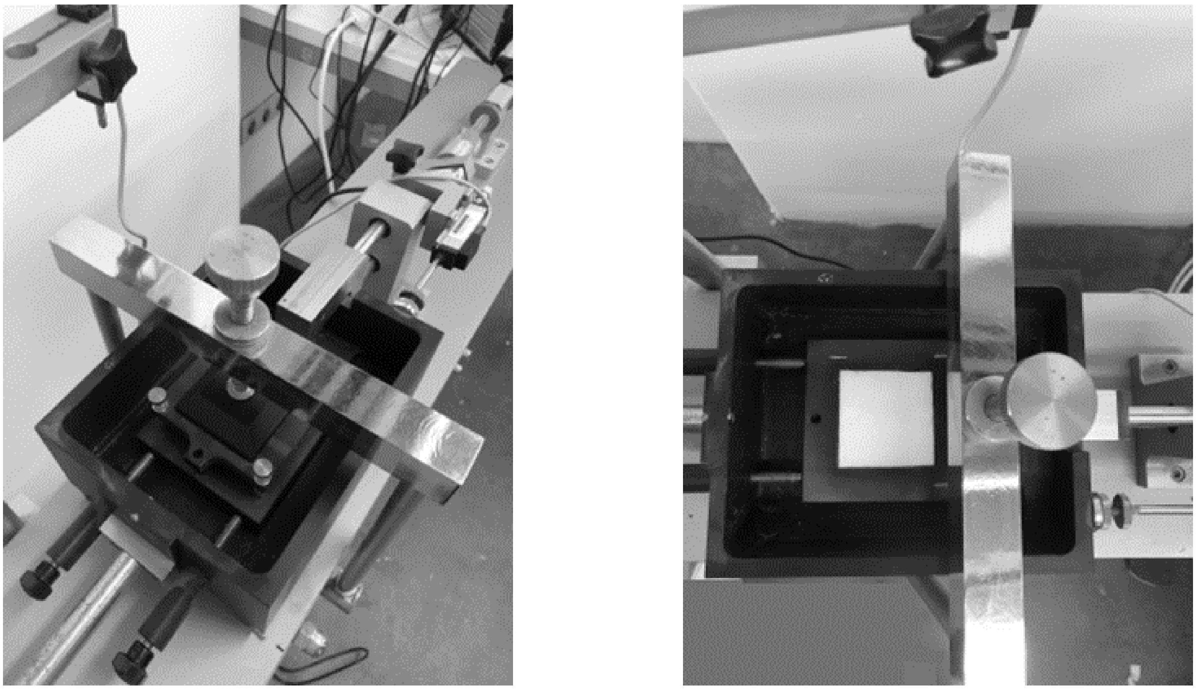
Table 2. Physical and index properties of granular soils used in testing program.

Sand Properties	D_{50} (mm)	C_u[a]	C_c[b]	G_s[c]	e_{max}[d]	e_{min}[e]	D_r[d,e]
Beach Sand	0.34	2.30	0.91	2.67	0.73	0.57	69.8% ± 1.8%
Construction Material Sand	0.39	2.37	0.96	2.68	0.82	0.68	70.1% ± 1.7%

Note: [a]$C_u = D_{60}/D_{10}$; [b]$C_c = D_{30}^2/(D_{10}*D_{60})$; [c]ASTM D854 [15]; [d]ASTM D4253 [16]; [e]ASTM D4254 [17]

3.2. Testing device and experimental procedures: Specimen preparation

A strain-controlled interface device (Figure 1) has been used to perform shear tests in which frictional response and engineering strength as well as deformation properties of different soil versus geofoam interfaces have been measured and detected so that the frictional characteristics of contact surfaces for composite layered systems between geofoams and distinct soil types including granular, cohesive and mixed soils have been investigated. The experimental program was conducted under dry conditions. The tests were performed at a constant strain rate (i.e., constant lateral speed) such that the rate of lateral displacement was set as 0.1 mm/min and it was allowed to up to a total horizontal displacement of 10 mm. This shearing speed (i.e., loading speed) was selected intentionally to observe the essential and accurate interface shear response and frictional resistance behavior both in cohesive soil and granular soil interface systems. The shear force and both lateral as well as vertical displacements were measured by employing a load cell and linear displacement transducers (LVDTs), respectively. The measurement data was logged and recorded into a computer through a controller program enabling analog-digital communication in between the sensors and the computer.



(a) (b)

Figure 1. Testing device and shear box.

The geofoam specimens were cut and prepared by utilizing a hot wire equipment and placed at the bottom half of the shear box. Further, the soil specimens, placed at the top half, were prepared in dry form by applying air pluviation technique to attain the same relative density for the specimens for achieving same denseness or concentration in the box. Thereafter, the shear box was closed by a cap on which a LVDT is located to measure vertical displacement during the test progress. Afterwards, the normal load was applied and the test process was initiated by facilitating horizontal displacement at a constant lateral speed.

3.3. Test matrix

The laboratory experimental program consists of 12 interface shear tests at various loading conditions ranging from 25 kPa up to 250 kPa to observe the shear behavior at low, medium and high stress levels, respectively (Table 3).

Four different composite systems including two different granular materials such as beach sand and construction material sand as well as one cohesive soil (bentonite clay) and a soil mixture composed of 75% sand and 25% clay were formed (constituted) to investigate the influence of soil type – counterfaced with geofoam – on the developed interface response and frictional characteristics including strength and deformation properties. The laboratory program test matrix provided in Table 3 presents the details and the extent of the experimental study.

Table 3. Laboratory program test matrix.

Materials Tested	Beach Sand	Construction Material Sand	Clay (Bentonite)	75% Construction Material Sand and 25% Clay (Bentonite)
Geofoam	Beach Sand vs. Geofoam (σ = 25 kPa)	Construction Mat. Sand vs. Geofoam (σ = 25 kPa)	Clay (Bentonite) vs. Geofoam (σ = 25 kPa)	75% Cons.Mat. Sand 25% Bentonite Clay vs. Geofoam (σ = 25 kPa)
Geofoam	Beach Sand vs. Geofoam (σ = 100 kPa)	Construction Mat. Sand vs. Geofoam (σ = 100 kPa)	Clay (Bentonite) vs. Geofoam (σ = 100 kPa)	75% Cons.Mat. Sand 25% Bentonite Clay vs. Geofoam (σ = 100 kPa)
Geofoam	Beach Sand vs. Geofoam (σ = 250 kPa)	Construction Mat. Sand vs. Geofoam (σ = 250 kPa)	Clay (Bentonite) vs. Geofoam (σ = 250 kPa)	75% Cons.Mat. Sand 25% Bentonite Clay vs. Geofoam (σ = 250 kPa)

4. Experimental Investigations

The experimental findings will be presented in two subsections for which in the foremost one, the shear stress versus horizontal displacement curves obtained for different interface systems at distinct loading conditions will be demonstrated to expand the understanding on the characteristics of the observed interface behavior, and additionally, in the latter one, the frictional properties regarding to some important engineering design parameters including peak shear stress, residual shear stress, interface sensitivity (i.e. peak/residual ratio) and displacement required to reach peak stress determined as a result of the measured interface shear response curves will be presented. Further, the change in the values of those interface mechanical properties as a function of loading condition and counterface soil type will be shown.

4.1. Interface test results: Shear response

The shear responses of different interface systems including the counterface materials such as bentonite clay, bentonite clay/construction material sand admixture, beach sand, and construction material sand at various normal loading conditions are demonstrated in Figures 2, 3, and 4 for the normal stress levels of 25 kPa, 100 kPa, and 250 kPa, respectively.

The measured shear stress increases with an increase in normal stress regardless of the material combinations at the interface. As such, the shear stress versus displacement envelope (i.e., curve), showing load versus deformation behavior, enlarges with an increase in normal stress by locating at upper space of shear stress versus displacement plots for all the interface systems comprised of either granular or cohesive soils or soil mixtures. Furthermore, additional important particular detail observed in the developed shear stress versus displacement curves is that the displacement softening behavior is exhibited for all the interface systems tested such that a reduction in shear stress is displayed indicating strength reduction with continued displacement develops at geofoam and soil contact surfaces. This is attributed to the shearing mechanism mobilizing when a continuum material (geofoam) is counterfaced with particulate material (soil).

The largest shear stress versus displacement curves is observed at the interface of construction material sand compared to that of the interface of bentonite clay at which the smallest shear stress – displacement curves are shown regardless of the magnitude of loading applied onto the interface. In-between, the interface of beach sand demonstrates higher shear stress versus displacement curves (envelopes) compared to that of the soil mixture including 75% construction material sand and 25% bentonite clay by dry weight. This indicates that although the content of cohesive soil is less than the content of construction material sand by one-third, the clay initiates to predominate the interface behavior at the contact surface resulting in a reduction in frictional strength of the construction materials sand even below of the beach sand in the light of the diminishing (i.e. decrement) displayed in the curves of soil mixture in comparison to the curves of pure construction material sand due to addition of bentonite clay. Further, gradually progressive curves are observed for all the interface systems tested in such a way that, instead of sharp peak transformation, smoother transition develops at peak stages for soil – geofoam composite layered systems.

Further, although the physical and index properties of sand specimens were similar to each other, the construction material sand composed of grains (i.e. particles) possessing angular features in comparison to that of beach sand possessing subrounded features. This was purposefully intended to examine the influence of sand particle shape and geometry on the developed interface shear behavior and on the mobilized frictional strength characteristics such as engineering design parameters including peak shear stress, residual shear stress, interface sensitivity and displacement to peak. As shown in Figures 2-4, regardless of loading condition either low, medium, or high, the construction material sand specimens were able indent and plough through counterface geofoam surface owing to greater interlocking features of particles which resulted in obtaining relatively higher frictional resistances (i.e., larger shear strengths) at the interface during shearing displacement.

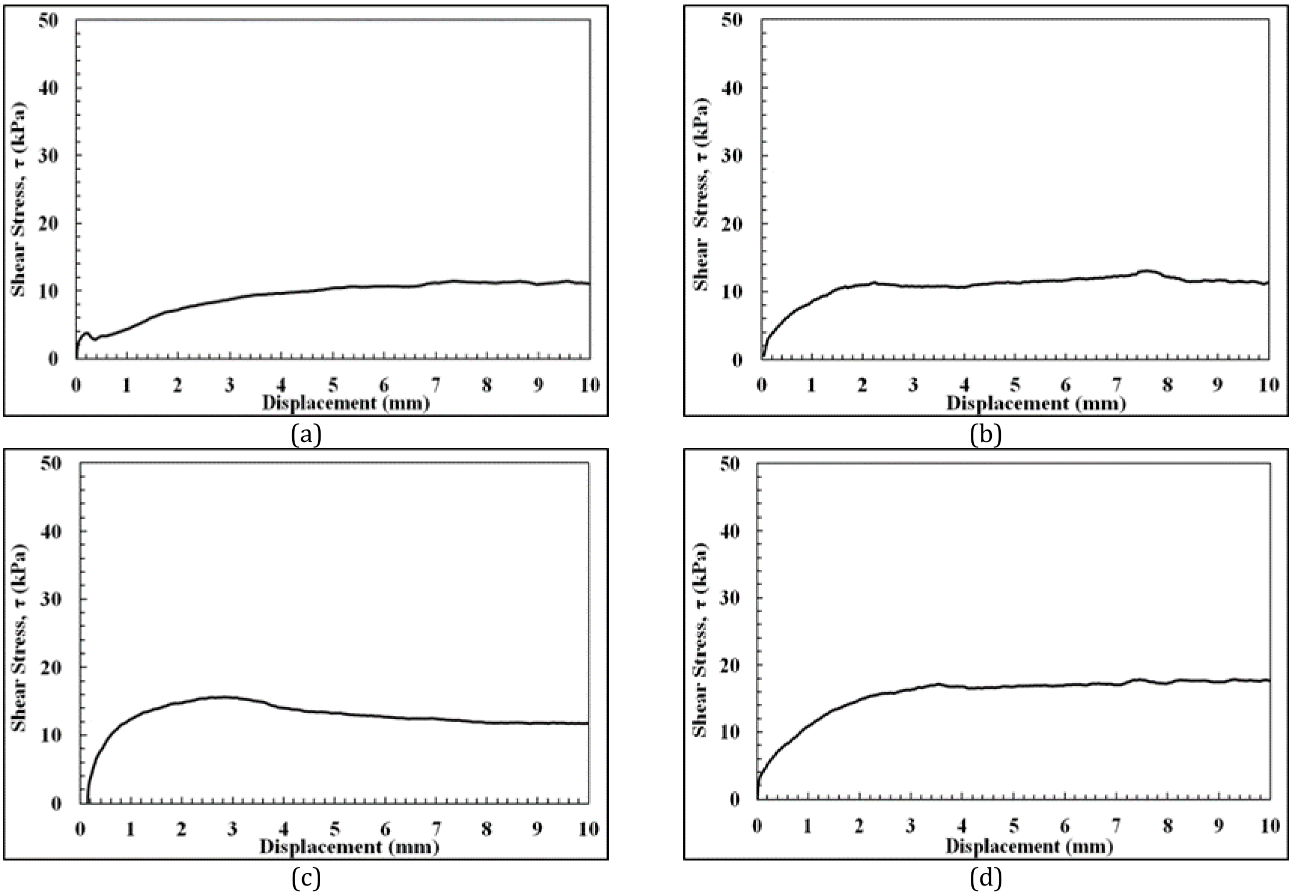


Figure 2. Shear response of different interface systems at 25 kPa normal stress level: **(a)** Bentonite clay; **(b)** Bentonite clay/construction material sand admixture; **(c)** Beach sand; **(d)** Construction material sand.

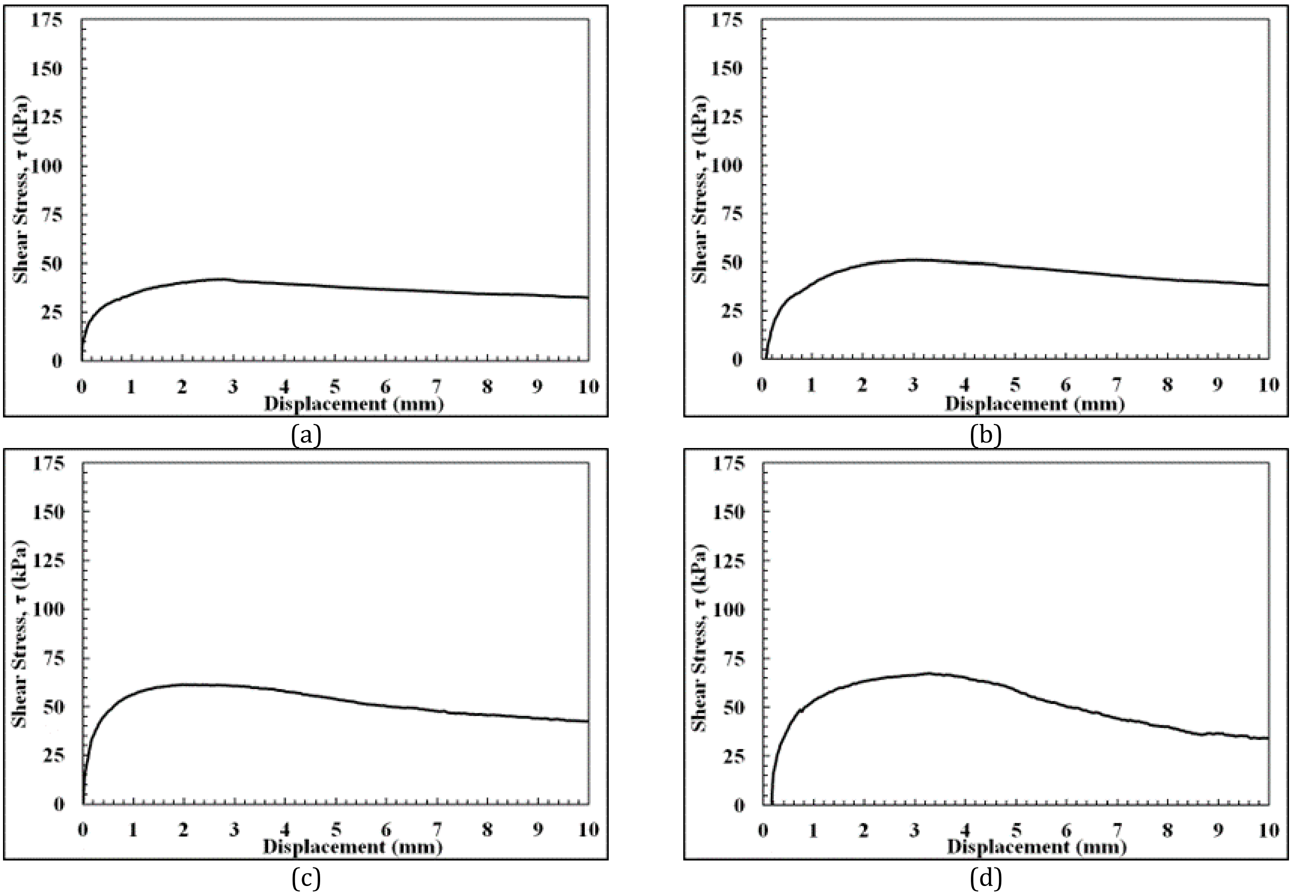


Figure 3. Shear response of different interface systems at 100 kPa normal stress level: **(a)** Bentonite clay; **(b)** Bentonite clay/construction material sand admixture; **(c)** Beach sand; **(d)** Construction material sand.

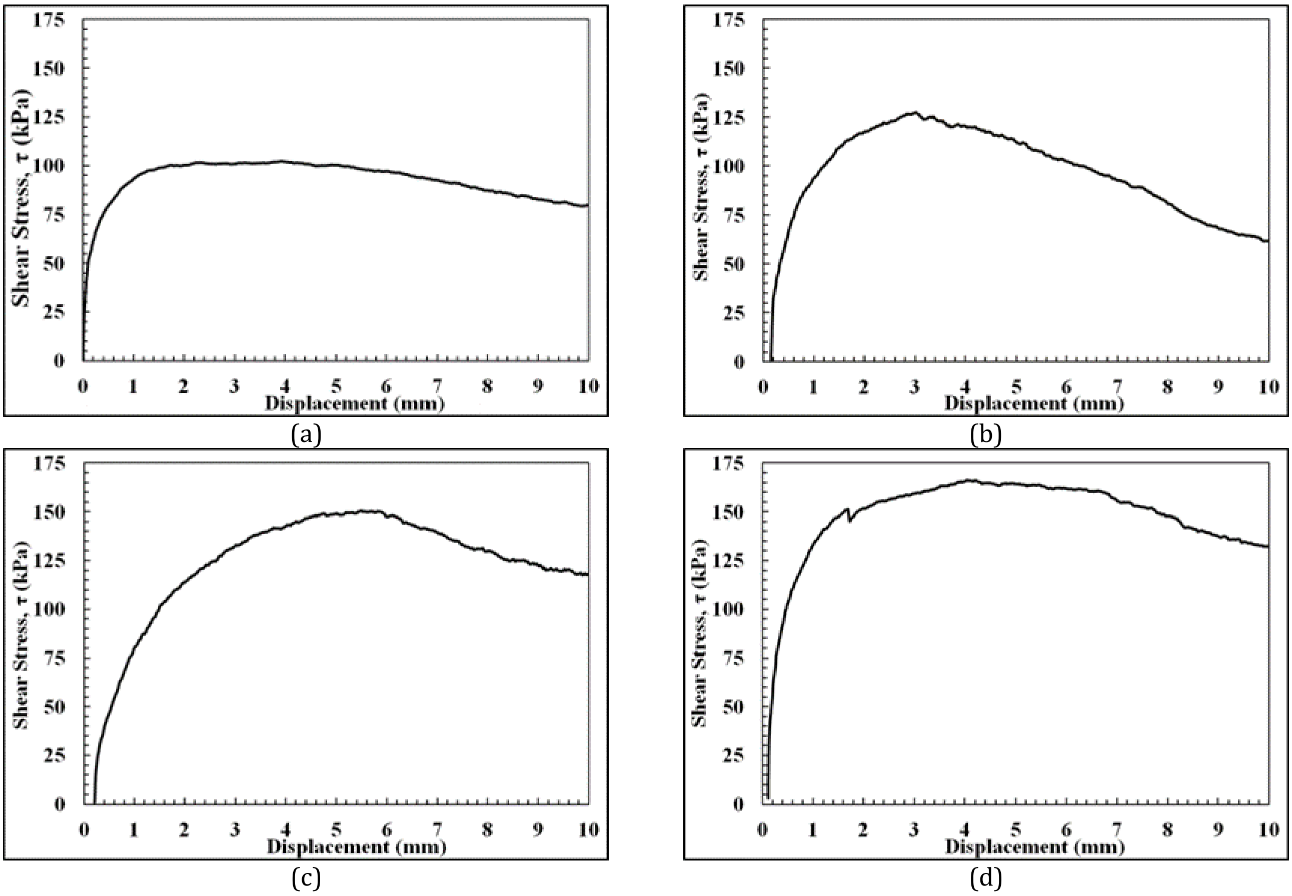
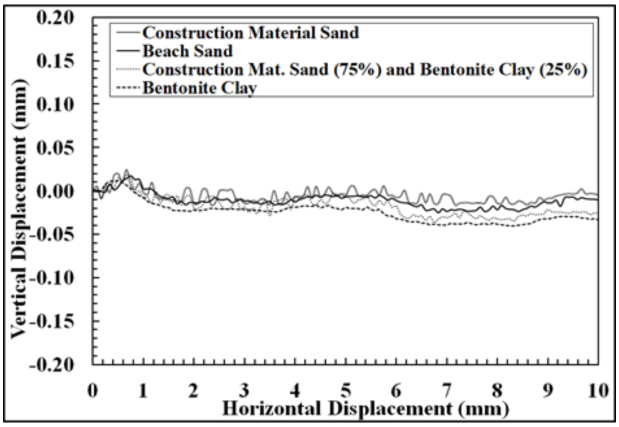
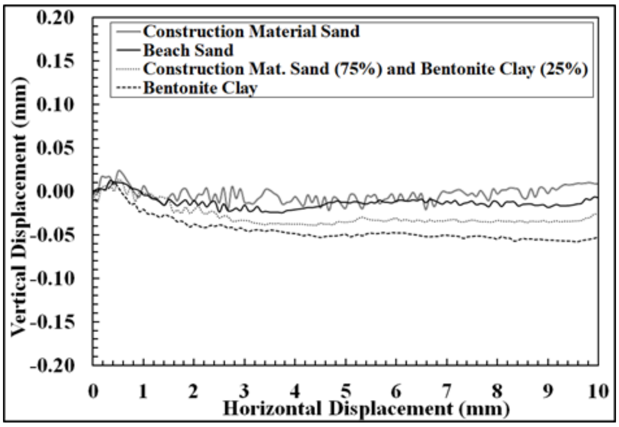


Figure 4. Shear response of different interface systems at 250 kPa normal stress level: **(a)** Bentonite clay; **(b)** Bentonite Clay/Construction material sand admixture; **(c)** Beach sand; **(d)** Construction material sand.

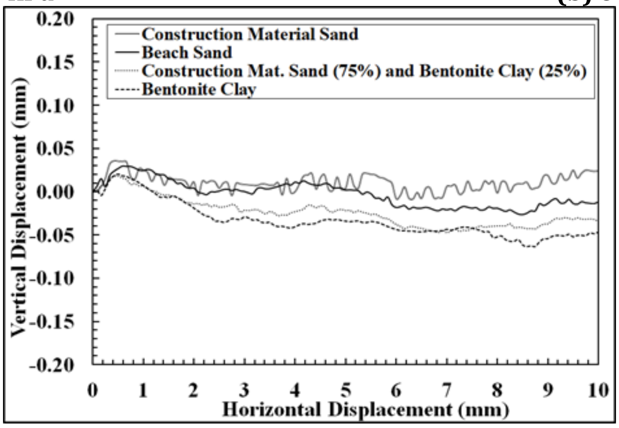
Moreover, the variation in vertical displacement during shearing for bentonite clay, bentonite clay/construction material sand admixture, beach sand, construction material sand interfaces are presented in Figures 5a, 5b, and 5c for various loading conditions (i.e., low, medium, high) including 25 kPa, 100 kPa, and 250 kPa, respectively. The surface topography of the man-made surface (i.e., a geosynthetic surface) was quantitatively linked to the stress and volume change behavior of the interface systems for the development of insightful interface behavior models in which the two general end-member conditions for the behavior of interfaces between soil and a man-made construction material (i.e., polymeric geosynthetics) were earlier defined in the literature [18-20]. The first condition was when the size of soil grains was large with respect to the asperity height and spacing on the construction material surface (e.g., a sand grain contacting a smooth geosynthetic surface). Experimental results for dense sand specimens against smooth geomembranes from Dove and Frost [20] indicated that soil volume changes were small and the soil above the interface did not participate in the shear process. These interface systems were referred to as nondilative as exhibited in the testing program by the interface system of bentonite clay as well as bentonite clay and construction material sand admixture soil. Further, it was showed that peak strength of nondilative interface systems was controlled by particle contact conditions which, in turn, are principally controlled by material hardness and surface roughness of the bodies at the scale of the contact. Moreover, the second end member condition was defined as the situation in which there occurs significant volume changes and interface strength can reach frictional resistance levels as great as, or greater than the soil internal shear strength. These interface systems were referred to as dilative as exhibited in the testing program by the interface system of construction material sand as well as beach sand. Additionally, it is evident that it is possible to observe the occurrence of an infinite number of states between the nondilative and dilative conditions at particulate – continua contact surfaces. In light of the discussion provided herein, the variation in vertical displacement at the interface plotted against horizontal displacement for normal stress levels of 25, 100 and 250 kPa (Figures 5a, 5b and 5c, respectively) shows that the cohesive soil and its admixture soil – continua (i.e. geofoam) interface displayed relative contraction as shearing progresses due to the mobilization of increased frictional shear strength taking place at the interface at all loading conditions including low, medium, high normal stress levels of 25 kPa, 100 kPa, and 250 kPa, respectively. On the other hand, the variation in vertical displacement at the interface plotted against horizontal displacement for normal stress levels of 25, 100 and 250 kPa (Figures 5a, 5b and 5c, respectively) shows that the construction material sand and the beach sand– continua (i.e., geofoam) interface displayed relative dilation as shearing progresses due to larger resistance of the counterface material against the sand particles during the course of shearing in the tests with a greater contribution of the plowing effect mobilized mechanism at the interface during shear such that sand grains in the interface were able to penetrate deeper into the surface of the geofoam. The sand specimen experienced a relative volumetric expansion to overcome the greater shear resistance of the counterface geofoam due to occurrence of deeper plowing of the grains during shear displacement. In addition, a relatively larger volumetric dilation compared to that at the beach sand interface system was exhibited at the construction material sand interface system as a result of the angular features of the particles of construction material sand leading to greater penetration/embedment of grains into and plow through counterface geofoam surface.



(a) σ = 25 kPa



(b) σ = 100 kPa



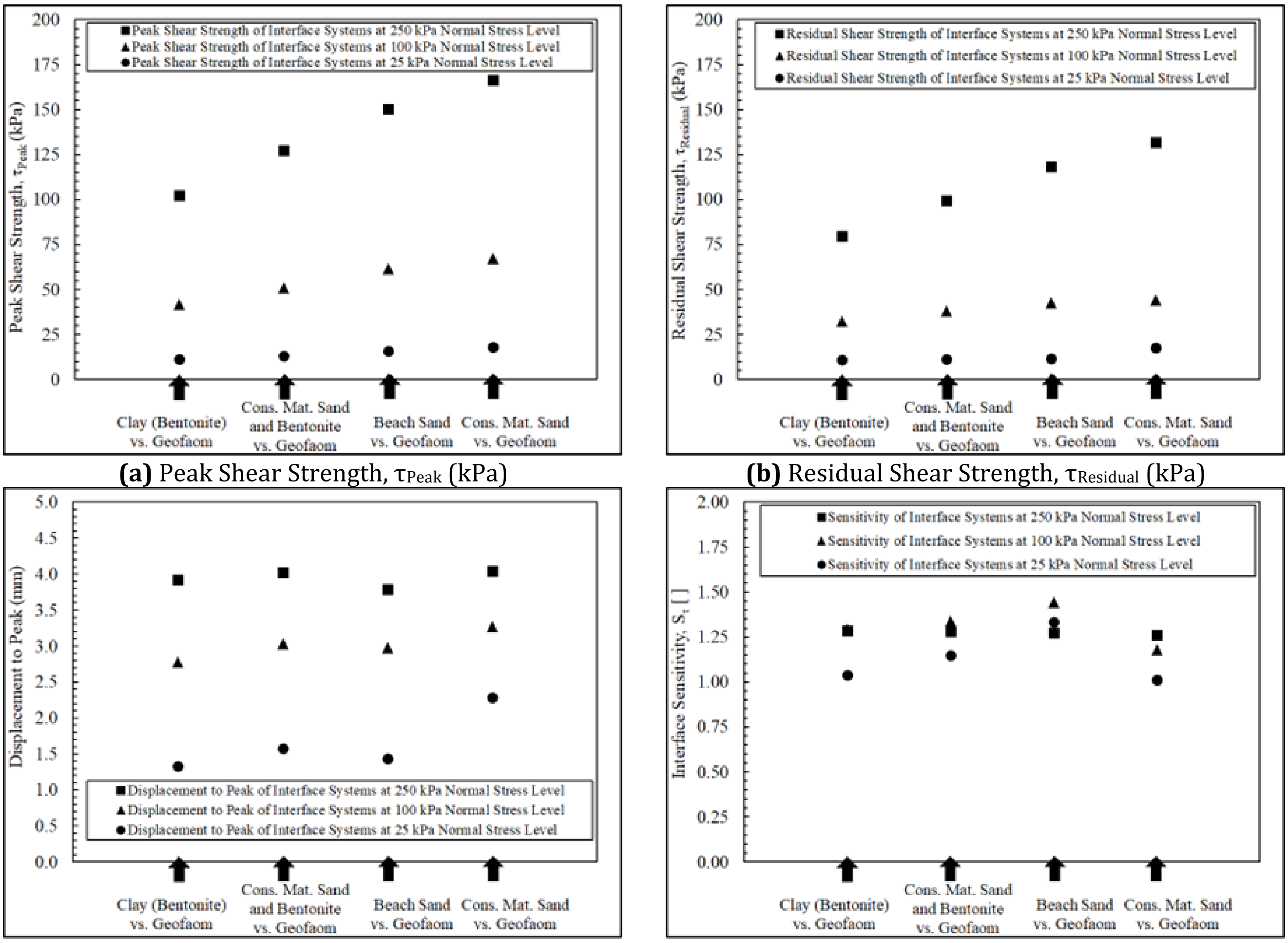
(c) σ = 250 kPa

Figure 5. The variation in vertical displacement during horizontal displacement mobilized in interface shear tests at different normal loading conditions: **(a)** 25 kPa; **(b)** 100 kPa; **(c)** 250 kPa.

4.2. Frictional properties: Engineering design parameters

The characterization of shear response of geofoam – soil interfaces involve in determination of the frictional properties. These interface mechanical properties regard to some important engineering design parameters including peak shear stress, residual shear stress, interface sensitivity (i.e., peak/residual ratio) and displacement required to reach peak stress investigated as a result of the measured shear response curves. Figure 6 shows the frictional strength and deformation properties of the different soil versus geofoam interface systems tested in the entire experimental program. At all normal stress levels including 25 kPa, 100 kPa and 250 kPa, the greatest peak shear strength (τ_{Peak}) was obtained in the interface system of construction material sand followed by beach sand, soil mixture (75% sand and 25% clay) and bentonite clay in a decreasing order of the measured magnitude of τ_{Peak} at the interface. Similarly, the construction material sand interface system exhibited the largest residual shear strength ($\tau_{Residual}$) mobilized that is followed by a sequential decrease in the measured values for the magnitudes of $\tau_{Residual}$ attained in the interface system of beach sand, soil mixture (75% sand and 25% clay) and bentonite clay. Therefore, among the interface systems tested, the lowest values of both τ_{Peak} and $\tau_{Residual}$ were observed in the cohesive soil (bentonite clay) interface at all loading conditions; low, moderate and high stress levels. Consequently, the granular material interfaces displayed larger frictional resistance when counterfaced with geofoams as compared to that of cohesive soil interfaces not only for the peak state but also for the residual state of the interface in the circumstances when subjected to shear displacement or lateral deformation under the action of mechanical loading.

Moreover, comparing different interface systems, the trend for τ_{Peak} and $\tau_{Residual}$ were similar such that the difference in the measured values for different counterface materials (granular, cohesive soil or mixture) were largest at 250 kPa normal stress level, whereas were lowest at 25 kPa normal stress (Figures 6a and 6b). In order to assess load versus deformation characteristics of interfaces for composite layered systems, an engineering parameter being displacement required to reach peak shear strength was also determined for the different interface systems tested. The highest values were observed in the interface of construction material sand regardless of the magnitude of loading followed by the interface system being its mixture with bentonite clay (75% construction material sand and 25% bentonite clay). For the interfaces of pure bentonite clay as well as pure beach sand, the almost similar values were attained with an exception of high normal stress level of 250 kPa where the bentonite clay interface displayed slightly (marginally) greater values of displacement to peak (Figure 6c).



(a) Peak Shear Strength, τ_{Peak} (kPa)
(b) Residual Shear Strength, $\tau_{Residual}$ (kPa)
(c) Displacement to Peak (mm)
(d) Interface Sensitivity, S_τ []

Figure 6. Frictional strength and deformation properties of different soil vs. Geofoam interface systems.

Furthermore, the interface sensitivity that is the ratio of peak shear strength with respect to residual shear strength ($S_\tau = \tau_{Peak}/\tau_{Residual}$) was computed for all interface systems. This is a very important interface mechanical property, gauging deterioration of frictional resistance, and thus, allowing to quantify the amount and magnitude of strength reduction at the interface with the continued displacement in order to evaluate the degree of strain softening behavior mobilized at the contact surface of geofoam – soil composite layered systems. The more or less analogous sensitivity values were attained at 250 kPa high normal stress level for all the interface systems tested. On the other hand, at low and moderate stress levels of 25 kPa and 100 kPa, respectively, the highest sensitivity was observed in the interface system of beach sand whereas the lowest sensitivity was depicted in that of construction material sand. A trivial (very minor) difference in the sensitivities for the interface system of bentonite and soil mixture (sand and clay) was observed at 25 kPa and 100 kPa loading conditions as well (Figure 6d).

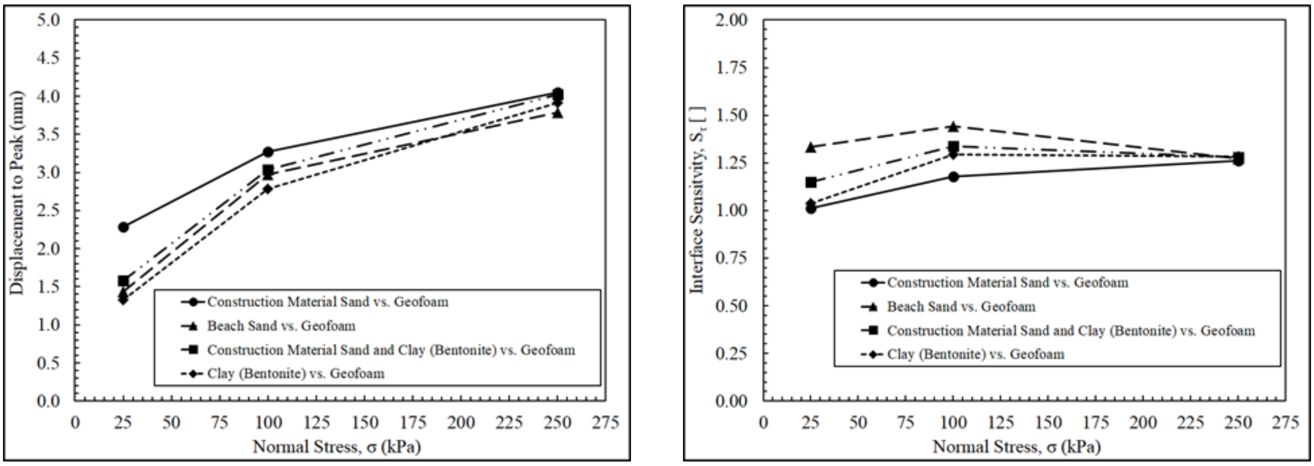
5. Further Analysis on Test Results

A further comparative analysis on test results was performed to extend understanding on the general interface behavior and shear response. To this end, the displacement required to reach peak interface strength being an important and decisive frictional property for the distinct interface systems tested was determined and the variation of which with increasing normal stress (σ) was plotted and presented in Figure 7a.

Similar behaviors were observed for the tested different systems in consideration such that the displacements necessitated arriving to peak strength conditions increased at a relatively higher rate from low loading conditions (σ = 25 kPa) until medium loading conditions (σ = 100 kPa), and thereafter, the rate of this increment became lower and the increase in displacement to peak maintained up to the largest loading conditions (σ = 250 kPa). This shows that, regardless of counterface soil type (sand and/or clay), the peak strength conditions could be obtained at greater shearing displacements at soil – geofoam interfaces.

Further, this is attributed to the larger confinement/restraint of the interface at greater normal stresses (i.e., σ>~100 kPa).

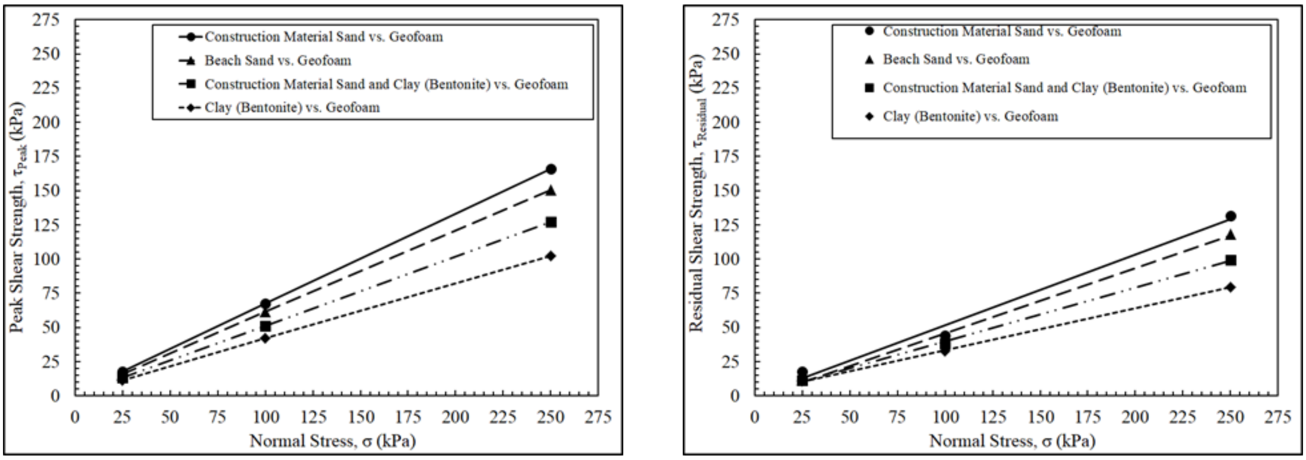
The change in another critical frictional property for assessing shearing response of soil – geofoam interfaces being interface sensitivity (S_τ) with respect to a change in normal stress is shown in Figure 7b. In general, an increase in the detected values of S_τ is displayed for all the interface systems tested from low normal stress level of 25 kPa up to medium normal stress level of 100 kPa, however, beyond which the detected values of S_τ up to high normal stress level of 250 kPa decreased slightly (i.e., trivial reductions are exhibited) for the construction material and clay as well as the beach sand interface systems; maintained approximately constant for the pure clay interface system; and increased marginally for the pure construction material sand interface system. The contrary response observed in the last interface system is attributed to the mechanism of ploughing of angular construction material sand particles through counterface geofoam surface as a result of shearing displacement developed at the interface which results in mobilization of greater residual strengths at continued larger displacements.



(a) Displacement to Peak **(b)** Interface Sensitivity

Figure 7. The variation in Displacement to Peak **(a)** and Interface Sensitivity **(b)** with respect to normal stress.

Based on Mohr-Coulomb criterion, the resulting linear peak strength as well as residual strength failure envelopes from interface shear tests performed on a variety of distinct soil – geofoam systems under different load conditions over a large range of normal stresses from 25 kPa up to 250 kPa are presented in Figures 8a and 8b, respectively. The linear Coulomb-type failure envelopes (Figure 8), generated using least square fit method, were drawn through the data points for a range of normal stresses expected in the field in such a way that those failure envelopes are defined in terms of two interface shear strength parameters which are the interface friction angle (δ) representing its inclination in the shear stress versus normal stress space; and adhesion (α) representing the intercept of the failure envelope with the shear stress axis so that the stability of any slope containing a soil – geofoam interface can be evaluated using these interface shear strength parameters (i.e., frictional properties of interface response).



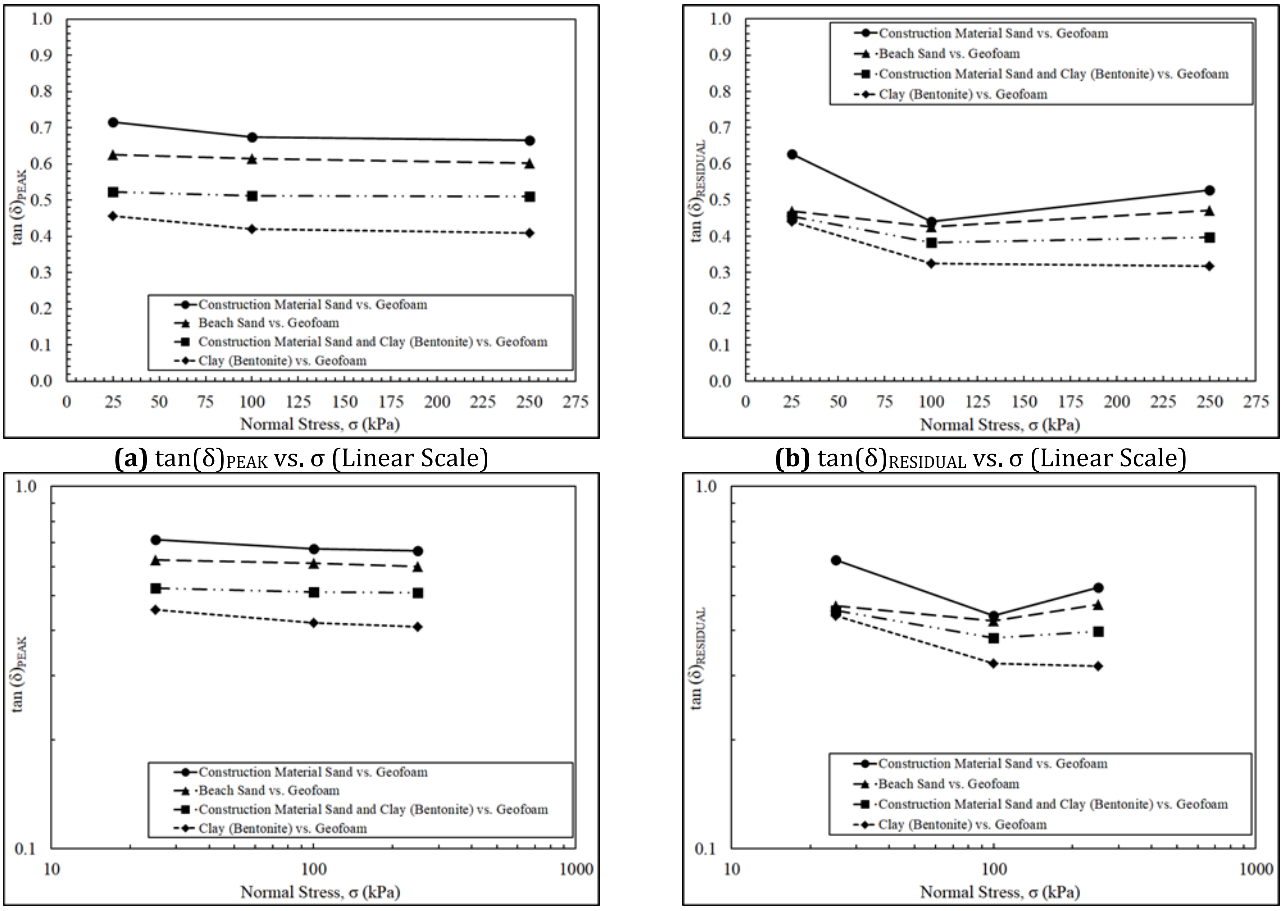
(a) Peak Strength **(b)** Residual Strength

Figure 8. Peak Strength **(a)** and Residual Strength **(b)** Failure Envelopes for different interface systems.

The plots in Figure 8, providing good straight line fits for all the interface systems tested, were developed on a purpose to capture variations in granular soil and/or cohesive soil versus geofoam interface shear behavior as a function of normal stress levels ranging from low loading conditions to very high loading conditions. All the tested soil (granular and/or cohesive) – geofoam composite interface systems clearly exhibited both linear peak and residual strength envelopes over the entire range of normal stresses applied in the laboratory experimental program. Furthermore, it is evident from a comparison of the strength envelopes that the construction material sand system yielded the highest both peak and residual shear strengths, while the bentonite clay system displayed the lowest both peak and residual shear strength over the entire range of normal stresses from 25 kPa up to 250 kPa. The beach sand system exhibited relatively larger both peak and residual shear strengths as compared to that of an admixture soil system including construction material sand and bentonite clay. This is due to the existence of bentonite clay in the admixture leading to reduction of interface frictional resistance sheared against geofoam.

Moreover, among all the interface systems tested, the greatest slope both in peak and residual failure envelopes were obtained in construction material sand interface, while the smallest slope in peak as well as residual envelopes were exhibited by the bentonite clay interface. Furthermore, the slope of both peak and residual failure envelopes for the beach sand interface were larger than that of the admixture soil interface including construction material sand and bentonite clay. Regardless of strength condition; peak or residual, the failure envelopes of construction material sand located at the highest location in shear stress versus normal stress space followed by the order of beach sand, admixture soil, and bentonite clay, respectively.

The coefficient of friction ($\tan(\delta)$) versus normal stress plotted on both linear and logarithmic scale as shown in Figures 9a, 9c for peak state and Figures 9b, 9d for residual state, respectively. As evidently seen in Figure 9, the peak coefficient of friction ($\tan(\delta)_{Peak}$) for all the tested granular and/or cohesive soil – geofoam interface systems decreased with normal stress at low normal stress levels up to ~100 kPa that is consistent with Hertzian contact theory [18, 19]. Further, under high normal stress levels beyond ~100 kPa up to ~250 kPa, the peak coefficient of friction ($\tan(\delta)_{Peak}$) became more or less constant and remained in this fashion until the greatest loading condition of 250 kPa. The residual coefficient of friction ($\tan(\delta)_{Residual}$) for all the tested systems decreased with normal stress at low normal stresses up to ~100 kPa. On the other hand, under high normal stresses beyond ~100 kPa up to ~250 kPa, the residual coefficient of friction ($\tan(\delta)_{Residual}$) increased considerably for granular soil (construction sand and beach sand) interface systems, while remained approximately constant for cohesive soil (bentonite clay) and its admixture soil (bentonite clay and sand) interface systems. This increase is evidently observed particularly in the plot generated using logarithmic scales on both axes (Figure 9d). This is attributed to the higher interbedding occurring between the counterfaces at larger normal loads and is considered to be the influence of the ploughing effect that is often detected at granular material versus planar surface interfaces as previously noted by Dove and Frost [20].



(a) $\tan(\delta)_{PEAK}$ vs. σ (Linear Scale)

(b) $\tan(\delta)_{RESIDUAL}$ vs. σ (Linear Scale)

(c) $\tan(\delta)_{PEAK}$ vs. σ (Logarithmic Scale)

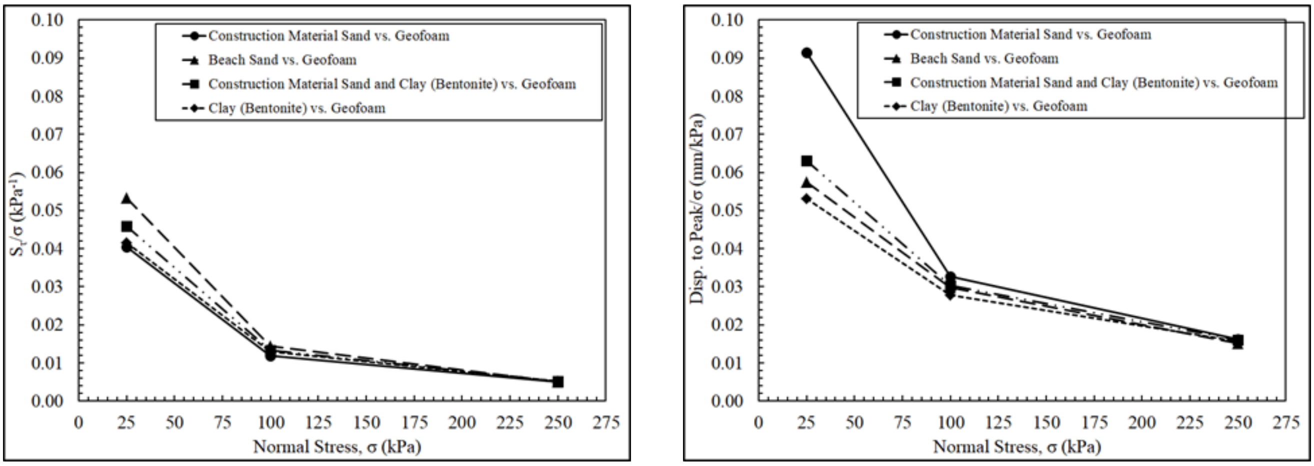
(d) $\tan(\delta)_{RESIDUAL}$ vs. σ (Logarithmic Scale)

Figure 9. The Change in Interface Strength Parameters; Peak as well as Residual Coefficient of Frictions as a function of Normal Stress on Linear Scale **(a)**, **(b)** and on Logarithmic Scale **(c)**, **(d)**.

6. Comparative Discussions on Experimental Findings

The variation in normalized interface engineering design parameters including normalized sensitivity as well as normalized displacement to peak as a function of normal stress are presented in Figures 10a and 10b, respectively.

As earlier described in Section 4.2., the post-peak strength loss exhibited by an interface can be quantified with a ratio called interface sensitivity (S_τ) between peak and residual shear strengths. As such, the differences between the peak stress state relative to the residual stress response of the tested granular and/or cohesive soil versus geofoam interface systems as a function of loading condition can quantitatively be measured and explicitly demonstrated by the interface sensitivity (S_τ) normalized with normal stress (σ) (Figure 10a). For all the tested interface systems, the normalized sensitivity (S_τ/σ) decreased significantly and sharply at a higher rate with increasing normal stress from 25 kPa until 100 kPa beyond which the rate of this decrease reduced, but the slight decrement in the value of the normalized sensitivity (S_τ/σ) maintained up to the largest normal stress level of the testing program (250 kPa). This indicates for granular and/or cohesive soil – geofoam interfaces that a substantial reduction (i.e., about 70% to 75% decrease) was exhibited from low level to medium level loading conditions, while a marginal reduction (i.e., about 20% to 25% decrease) was displayed from medium level up to high level loading conditions in the detected value of S_τ/σ, respectively. Consequently, it is noted that the amount of reduction in frictional strength/resistance of both granular and cohesive soil – geofoam interface systems from peak state to residual state when normalized with the magnitude of normal stress applied during shearing became trivial and minor with increasing normal loading.



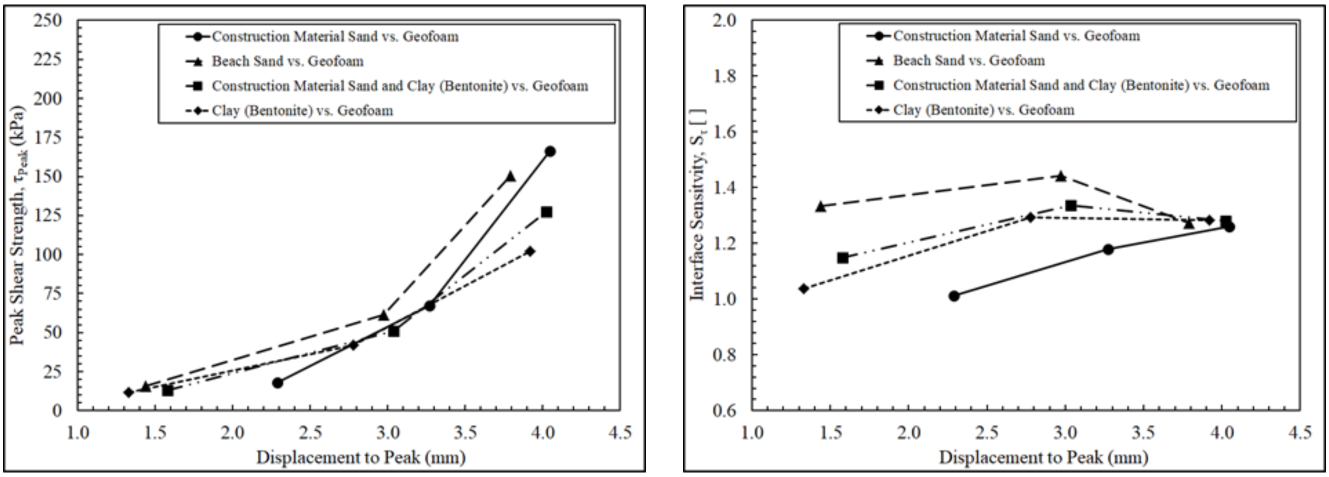
(a) Normalized Sensitivity **(b)** Normalized Displacement to Peak

Figure 10. The variation in normalized interface engineering design parameters; normalized sensitivity (a) as well as normalized displacement to peak (b) as a function of normal stress

The displacement required to reach peak strength state as normalized with the magnitude of normal stress level during shearing process as a function of loading conditions is shown in Figure 10b. As similar to the normalized sensitivity, a sharp decrease with a high rate of reduction (with a very high rate of reduction particularly for construction material sand system) was observed in the detected value of normalized displacement to peak parameter from low loading conditions of 25 kPa until medium loading conditions of 100 kPa beyond which a considerable decrease with a relatively smaller rate of reduction is followed up until large loading conditions of 250 kPa. It is remarkable that the largest amount of reduction in the value of normalized displacement to peak parameter from 25 kPa until 100 kPa was exhibited by the construction material sand interface system (i.e., about 65%) as compared to those of the other interface systems (i.e., ranging about between 40% to 45%) tested. All the systems including granular and/or cohesive soil – geofoam interfaces displayed similarly equivalent subsidiary decreases (i.e., minor) in the values of displacement to peak parameters from 100 kPa up to 250 kPa (i.e., reductions about 30%). This points out and specifies that the necessary displacement required to be mobilized at the interface during shearing when normalized with the magnitude of applied normal load demonstrates a continuous decrease in the measured value as a function of normal stress level. Additionally, the observed behavior in construction sand interface system is different than those of the other systems due to angular features of soil grains (i.e., sand particles).

Furthermore, the variation in two critical interface strength engineering design parameters including peak shear strength and interface sensitivity with respect to displacement to peak are presented in Figures 11a and 11b, respectively. Among all the tested systems, the construction material sand interface evidently showed a distinct behavior with increasing displacement to peak.

The relationship between peak shear strength (τ_{Peak}) and displacement to peak parameter as shown in Figure 11a is a two stage behavior that was observed for all the systems tested along with a continuous increase in the resultant magnitude of peak shear strength with respect to the measured amount of displacement necessitated to arrive/reach to peak frictional resistance state. The increase in the detected resultant value of peak shear strength comprised of small slope increment for the displacements from around 1.5 mm until around 3.0 mm; and large slope increment for the displacements from around 3.0 mm up to 4.0 mm. This shows that relatively larger displacements are necessary for the mobilization of higher peak frictional resistances during shearing to generate peak strength state at the interface of both granular and/or cohesive soil – geofoam interfaces. This is attributed to the characteristics of shearing mechanisms that develops particulate soil – planar continuum geosynthetic interfaces such that the mobilized shear strength results from the relative contributions of the soil particle shearing mechanism including sliding and rolling or ploughing along or into, respectively, the counterface geosynthetic (i.e., geofoam) surface. The sliding and/or ploughing forces combine together to produce total frictional resistance at the interface against shear displacement [21, 22].



(a) τ_{Peak} vs. Displacement to Peak **(b)** S_τ vs. Displacement to Peak

Figure 11. The relationship between τ_{Peak} **(a)** as well as S_τ **(b)** and displacement to peak.

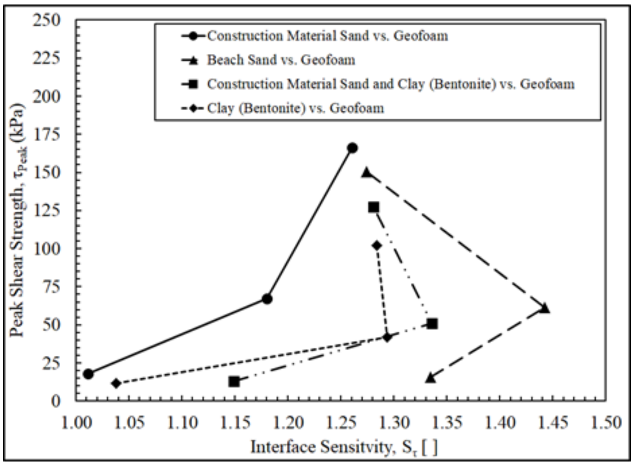
The variation in interface sensitivity (S_τ) with respect to displacement to peak, as presented in Figure 11b, depicts evidently more or less similar behaviors for all the systems tested with an exception of construction material sand interface. For the other interface systems tested, an increase in the resultant detected values of S_τ exhibited starting at the displacements from about 1.5 mm until about 3.0 mm beyond which the increment has ended by displaying a saturation plateau and the trend in the variation of S_τ has become lateral by maintaining approximately constant in the resultant detected values of S_τ. However, for construction material sand interface system, a continued rise in the resultant detected values of S_τ was demonstrated over the entire range of the displacements from about 2.0 mm up to about 4.0 mm. A very minor/trivial decrease in the rate of increment for the resultant values of S_τ was seen at about 3.0 mm displacement being roughly the half way between the departure (2.0 mm) and the arrival (4.0 mm) displacements. The distinct behavior observed in the interface system of construction material sand is due to angular sharp features of soil particles/grains that result in the mobilization of ploughing at the interface [23, 24] whereas the shearing mechanism, in general, is predominated by sliding and/or rolling over the areal extent of the other interface systems tested including beach sand comprised of rounded smooth granular features and bentonite clay consist of cohesive platy-like very fine particles.

The inter-relationship between peak shear strength (τ_{Peak}) as well as residual shear strength ($\tau_{Residual}$) and interface sensitivity (S_τ) is presented in Figures 12a and 12b, respectively. It is clearly seen from the comparison of Figure 12a with Figure 12b that, for the selected interface systems, both the peak shear strength and the residual shear strength displayed similar behaviors as a function of increased sensitivity. This indicates that the test measurements are robust, consistent and precise regardless of the counterface materials existed at the interface during shearing including granular and/or cohesive soil and geosynthetic geofoam.

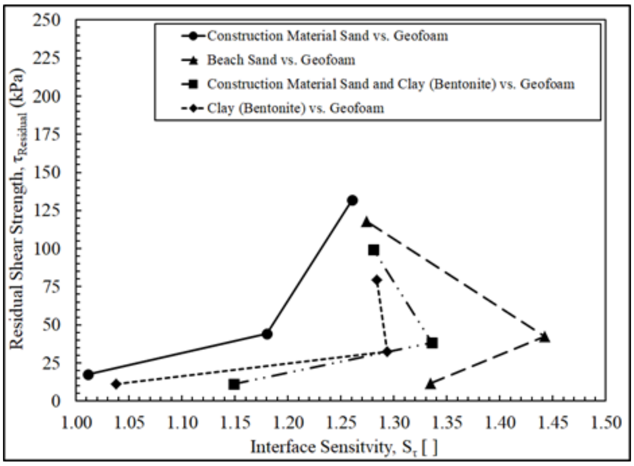
For the construction material sand interface system, the peak as well as residual shear strength increased at a slower rate with increasing sensitivity from $S\tau$=1.00 until $S\tau$=1.20, and subsequently, the rate of this increase has become faster and greater by depicting a sharp rise up to the highest recorded sensitivity of the system ($S\tau$=1.20) in the experimental program.

As opposed to that of construction material sand system, the other interface systems including smooth rounded beach sand, cohesive bentonite clay and its admixture with sand displayed similar trends in behavior. As such, the resultant measured values of both peak shear strength (τ_{Peak}) and residual shear strength ($\tau_{Residual}$) with increasing interface sensitivity (S_τ) showed lower rate increase from $S\tau$=1.35 until $S\tau$=1.45; from $S\tau$=1.05 until $S\tau$=1.30; from $S\tau$=1.15 until $S\tau$=1.35 for beach sand interface, bentonite clay interface and its admixture with sand system, respectively; and thereafter, an inversely proportional behaviors were observed for all three interface systems in consideration up to the largest magnitude of both τ_{Peak} and $\tau_{Residual}$ frictional resistances measured for the corresponding interface systems. This is attributed to the modification of shearing mechanism mobilized over the entire areal extent of the interface due to shear displacement from sliding/rolling to ploughing at greater normal stress conditions particularly above 100 kPa, and as a result of which the induced and recorded greater shear stresses at peak as well as residual state on to the counterface material during shearing displacement [25 –27].

To sum up, in addition to counterface soil type either granular or cohesive and/or their admixtures, the level of loading conditions either low, medium, high will certainly and significantly influence the resultant frictional behavior exhibited at the interface of composite systems (i.e., granular and/or cohesive soil versus geosynthetic geofoam) during the course of shear displacement mobilized and progressed over the overall extent of contact area of counterface materials. Consequently, in design and application of such composite layered systems, the selection of counterface materials and their compatibility play a critical and crucial role as per implementation of engineering activities for the development of infrastructural facilities in geotechnical profession. In this regard, the compatibility and/or interaction of counterface materials is of importance for proper development of strength at the interface that is vital for the durability of infrastructural facility.



(a) τ_{Peak} vs. S_τ



(b) $\tau_{Residual}$ vs. S_τ

Figure 12. The inter-relationship between τ_{Peak} **(a)** as well as $\tau_{Residual}$ **(b)** and S_τ.

7. Conclusion

The frictional resistance and the type of shear response mobilizing at soil – geofoam interfaces control the stability of composite layered systems, and hence, govern the integrity of the infrastructure constructed using geofoams counterfaced with soils. To this end, soil – geofoam interfaces were studied through an extensive experimental program by performing multiple series of interface shear tests using two different granular soils (i.e., beach sand and construction material sand) and one cohesive soil (i.e., bentonite clay) as well as a soil mixture containing 75% sand and 25% clay by dry weight at distinct loading conditions (i.e., normal stresses: 25, 100, 250; low, moderate, high loading conditions, respectively). The following principle key conclusions are drawn based on the attained test results and experimental findings as well as in light of further comparative analysis conducted on experimental investigations:

- Regardless of counterface soil type (sand and/or clay), the peak strength conditions could be obtained at greater shearing displacements at soil – geofoam interfaces. This is attributed to the larger confinement/restraint of the interface at greater normal stresses (i.e., σ>~100 kPa).
- The resultant detected values of interface sensitivity (S_τ) for the construction material sand system displayed a continued increase over the entire range of loading conditions (25 kPa– 250 kPa). This contrary response observed in the construction material sand interface is attributed to the mechanism of ploughing of angular construction material sand particles through counterface geofoam surface as a result of shearing displacement developed at the interface which results in mobilization of greater residual strengths at continued larger displacements.
- All the tested soil (granular and/or cohesive) – geofoam composite interface systems clearly exhibited both linear peak and residual strength envelopes over the entire range of normal stresses applied in the laboratory experimental program.
- Furthermore, the construction material sand system yielded the highest both peak and residual shear strengths, while the bentonite clay system displayed the lowest both peak and residual shear strength over the entire range of normal stresses from 25 kPa up to 250 kPa. The beach sand system exhibited relatively larger both peak and residual shear strengths as compared to that of an admixture soil system including construction material sand and bentonite clay. This is due to the existence of bentonite clay in the admixture leading to reduction of interface frictional resistance sheared against geofoam.
- Moreover, among all the interface systems tested, the greatest slope both in peak and residual failure envelopes were obtained in construction material sand interface, while the smallest slope in peak as well as residual envelopes were exhibited by the bentonite clay interface.
- The peak coefficient of friction ($\tan(\delta)_{Peak}$) for all the tested granular and/or cohesive soil – geofoam interface systems decreased with normal

stress at low normal stress levels up to ~100 kPa that is consistent with Hertzian contact theory. Further, under high normal stress levels beyond ~100 kPa up to ~250 kPa, the peak coefficient of friction ($\tan(\delta)_{Peak}$) became more or less constant and remained in this fashion until the greatest loading condition of 250 kPa.

- The residual coefficient of friction ($\tan(\delta)_{Residual}$) for all the tested systems decreased with normal stress at low normal stresses up to ~100 kPa. On the other hand, under high normal stresses beyond ~100 kPa up to ~250 kPa, the residual coefficient of friction ($\tan(\delta)_{Residual}$) increased considerably for granular soil (construction sand and beach sand) interface systems, while remained approximately constant for cohesive soil (bentonite clay) and its admixture soil (bentonite clay and sand) interface systems. This is attributed to the higher interbedding occurring between the counterfaces at larger normal loads and is considered to be the influence of the ploughing effect that is often detected at granular material versus planar surface interfaces.
- To sum up, in addition to counterface soil type either granular or cohesive and/or their admixtures, the level of loading conditions either low, medium, high will certainly and significantly influence the resultant frictional behavior exhibited at the interface of composite systems (i.e. granular and/or cohesive soil versus geosynthetic geofoam) during the course of shear displacement mobilized and progressed over the overall extent of contact area of counterface materials.
- Consequently, in design and application of such composite layered systems, the selection of counterface materials and their compatibility play a critical and crucial role as per implementation of engineering activities for the development of infrastructural facilities in geotechnical profession. In this regard, the compatibility and/or interaction of counterface materials is of importance for proper development of strength at the interface that is vital for the durability of infrastructural facility.

Conflicts of interest

The authors declare no conflicts of interest.

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