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SOIL-STRUCTURE INTERFACE BEHAVIOR OF CEMENTED-PASTE BACKFILL MATERIAL MIXED WITH MINING WASTE

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ABSTRACT: The use of mining wastes as a component of cemented paste backfill provides an environmentally acceptable method of waste disposal at a lesser cost as the method does not require tailing dams for storing the large volume of wastes. This study determines the applicability of cemented-paste backfill materials mixed with aggregate quarry wastes as ground support to concrete structures. Aggregate quarry waste with varying fine contents was used as a substitute for sand in cemented-paste backfill and the mixture was referred to as cemented-paste tailing backfill (CPTB). Its micro fabric structure was determined through SEM-EDX tests. Test results showed that CPTB with 20% and 40% fine contents has acceptable values of strength properties in terms of its unconfined compressive strength and interface friction angle. The unconfined compressive strength in relation to its curing period is in the range of 120 kPa to 150 kPa which can be described as having stiff consistency. The stress-strain interface behavior between CPTB and concrete structure was evaluated through a direct shear test using strain rates that simulate the rapid and slow rates of loading. In both conditions, the stress-strain behavior exhibits strain softening. The average interface friction angle is 38^o which can be associated with its dense condition. The modified hyperbolic model was applied to evaluate the soil-structure interface behavior of CPTB. Hyperbolic parameters were formulated to predict the interface shear stress - shear strain behavior of the CPTB when in contact with concretes structure at any value of shear strain and normal effective stress.

Keywords: Cemented-paste Backfill, Unconfined Compressive Strength, Interface Friction Angle, Modified Hyperbolic Model

1. INTRODUCTION

Cemented paste backfill is a cementitious composite normally made of coarse and fine aggregates mixed with a hydraulic binder which is typically Portland cement. It is a means of ground stabilization used for strengthening and solidification of the underground foundation of a building which is done at early parts of construction activities. It also provides stable platform and ground support to the structure sitting on it. In the mining industry, the extraction of valuable ore creates very large voids which need to be backfilled. The backfill material makes use of waste rocks or tailings mixed with a binder, usually cement, to form a cemented-paste backfill. The use of cemented paste backfill is an increasingly important component of underground mining operations and is becoming a standard practice for use in many cut-and-fill mines around the world [1]. This is an effective means of tailing disposal because it negates the need for constructing large tailing dams at the surface [2].

In the Philippines, sustainable solution to manage wastes being produced by mining industries is essential. Moreover, an effective open-pit mine rehabilitation method is strongly required. Backfilling, which requires a large volume of soil mass, is one of the major activities of open-pit mine rehabilitation to bring back the mined area into beneficial use. The use of mining wastes specifically wastes from the aggregate quarry, as a component of cemented paste backfill provides an environmentally acceptable method of waste disposal and rehabilitation process at a lesser cost. Waste from the aggregate quarry is proven to be stable when used as embankment material [3]. It is also a feasible component in concrete mix as a substitute for fine aggregates [4]. However, its strength properties when used as a component of cemented-paste backfill have yet to be determined. Its interface behavior when in contact with concrete structure has to be evaluated.

Understanding the soil-structure interface behavior is an important tool for use in analyzing, designing, and monitoring geotechnical structures. Several constitutive modeling has been used to obtain an accurate solution to many soil-structure interaction problems. The hyperbolic interface model developed by Clough and Duncan [5] presented a systematic approach to model the behavior of the retaining wall-to-soil interfaces in the primary loading stage. An extended hyperbolic model for interfaces was developed that can capture important aspects of interface response under the type of loading expected to occur in a wall-backfill interface at different stages of construction and operation of a lock wall [6]. In this study, the modified hyperbolic model which was formulated to characterize the stress-strain behavior of tailings determined from direct shear tests [3] was applied to evaluate the soil-structure interface behavior of cemented-paste tailing backfill.

This study determines the applicability of cemented-paste backfill materials mixed with aggregate quarry wastes as ground support to concrete structures like footings or retaining walls. The mixture is called cemented-paste tailing backfill (CPTB). Strength properties such as unconfined compressive strength at various curing time and interface friction angle were determined. Hyperbolic parameters were formulated to predict the interface shear stress – shear strain behavior of the CPTB when in contact with concretes structure.

2. MATERIALS AND TEST METHODS

The material composition of cemented-paste tailing backfill (CPTB) is waste from the aggregate quarry (WAQ) as a sand substitute with Portland cement as a hydraulic binder. WAQ was collected from quarry site in Ternate, Cavite. These quarry wastes are residues of mountain rocks which went through crushing processes to produce fine aggregates. The residues, considered as solid wastes, are produced during the washing of crushed rocks in the siltation pond through the natural process of sedimentation. WAQ has a specific gravity of 2.57 and classified as fine-grained soil with no plasticity [4].

The aggregate quarry wastes in the CPTB mixture had varying fine contents ranging from 20%, 40% and 60% by weight. This is done to investigate the effect of fineness of grains on the strength properties of CPTB. The fine content for this experimentation is defined as percent grain particles passing the #200 sieves. Cement consists of 5.5 percent by dry mass of WAQ which is equivalent to 3.8 weight percent of the entire mixture. The water-cement ratio of 7:1 was used in the mixture. Table 1 shows a typical mix proportion of CPTB.

To determine the strength properties of CPTB, unconfined compression strength (UCS) test in accordance with ASTM D2166 was performed. The cylindrical sample has 63.5 mm diameter with 158.8 mm height. The samples were tested after the 7^{th}, 14^{th}, 28^{th} and 42^{nd}-day curing period to determine the effect of curing time on its strength development. The 42^{nd}-day curing time was intended to observe the long-term strength of CPTB.

Direct shear test in accordance with ASTM D3080 was performed to determine the interface friction angle and to describe the soil-structure interface behavior of CPTB. The mixture that produced the maximum unconfined compressive strength was used for the direct shear test. The experimental set-up simulates a concrete structure resting on CPTB material. The lower part of the shear box contains the CPTB mixture while the upper part of the shear box contains the concrete mortar mix to represent a rigid concrete structure. The schematic of the experimental set-up is shown in Fig. 1. Samples were subjected to normal stresses of 13.625 kPa, 20.4375 kPa, and 27.25 kPa. The direct shear test was done using the fast strain rate of 1.25mm/min. to simulate the rapid loading condition and slow strain rate of 0.12mm/min to simulate the long-term loading condition. From the plot of shear stress vs. shear strain behavior obtained from the direct shear test, hyperbolic parameters using the modified hyperbolic model technique were determined to predict the soil-structure interface behavior of CPTB at any value of shear strain and normal effective stress.

Table 1 Typical mix proportion of CPTB for each cylindrical sample for UCS test

% of Fine Contents of WAQ	Cement (g)	Water (g)	WAQ (g): Passing #200 Sieve	WAQ (g): Passing #4 sieve, retained on #200 Sieve
20%	15.9	111.5	58.3	233.4
40%	15.9	111.5	116.7	175.0
60%	15.9	111.5	175.0	116.7

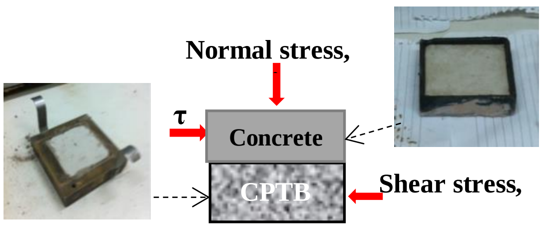


Fig. 1 Schematic of experimental set-up for direct shear test

3. RESULTS AND DISCUSSION

3.1 Unconfined Compressive Strength, q_u

Figure 2 shows the unconfined compressive strength development of CPTB with varying fine contents; the average values are tabulated in Table 2. For CPTB with 20% and 40% fines, the samples exhibited greater q_u at early curing days; however, there is an appreciable decrease in strength beyond the 7th-day curing period followed by an almost constant value of q_u as curing age increases. The observed higher value of q_u at 7th-day curing can be attributed to the pozzolanic reaction between the cement binder and WAQ. This can be explained by the SEM results as shown in Figs. 3 and 4. The microstructure of samples at 7th day curing time showed a more flocculated state with minimum void spaces. This results in a decrease in the total pore volume and a concurrent increase in strength. However, the pozzolanic reaction did not work well when samples are in almost dry condition. At 42nd curing day, the micrograph showed a honeycomb structure with intra-assemblage voids depicting an increase in porosity. The final amount of porosity after the hydration of cement paste depends strongly on the initial water-cement ratio of the paste [7]. The higher water-cement ratio is associated with increased porosity and a corresponding decrease in compressive strength. The almost constant value of q_u ranging from 120 KPa to 140 KPa for CPTB with 20% fines and 140 KPa to 150 KPa for CPTB with 40% fines can be considered as the stable compressive strength condition of CPTB that depicts its long-term behavior. The compressive strength is comparable to clay with a stiff consistency. The mode of failure exhibited by the samples is diagonal shear as seen in Figures 5a and 5b. Meanwhile, CPTB with 60% fines showed an unpredictable trend in its strength development with respect to curing period. There is a weaker bond between cement paste and WAQ when the mixture contains more fines. This is further attested by its mode of failure classified as a failure by axial splitting (Fig. 5c). It is expected that samples which failed by axial splitting give a lower value of q_u than those that failed by diagonal shear [8].

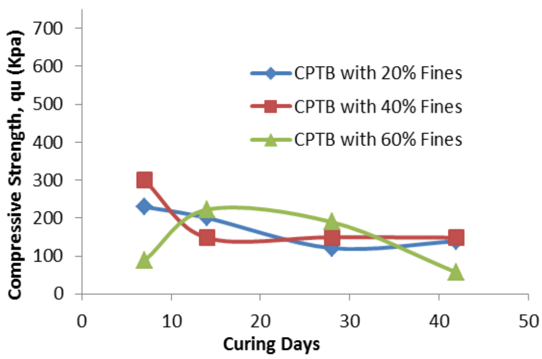
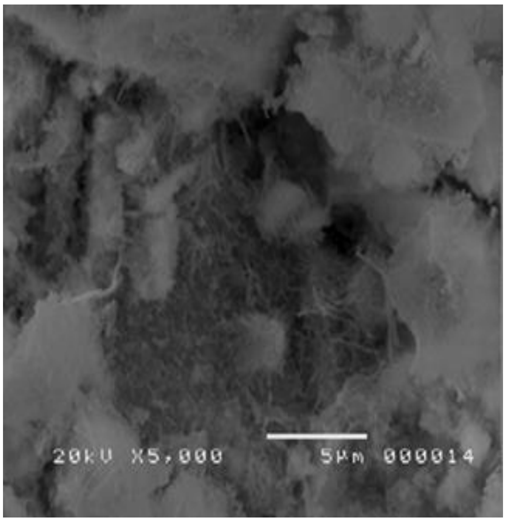


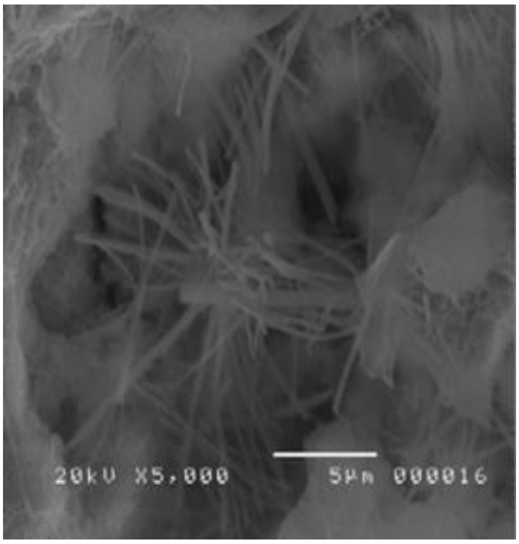
Fig. 2 Unconfined compressive strength of CPTB

Table 2 Average unconfined compressive strength of CPTB in KPa

Curing Days	CPTB with 20% Fine	CPTB with 40% Fine	CPTB with 60% Fine
7	229.81	299.26	88.32
14	200.44	148.03	222.15
28	121.02	149.74	189.38
42	139.07	150.48	56.40

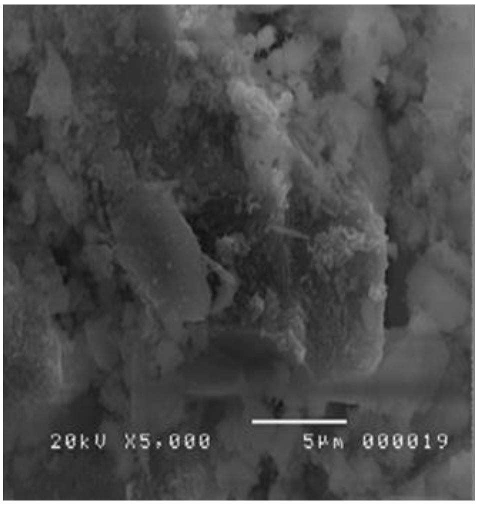


a.) at 7th-day curing

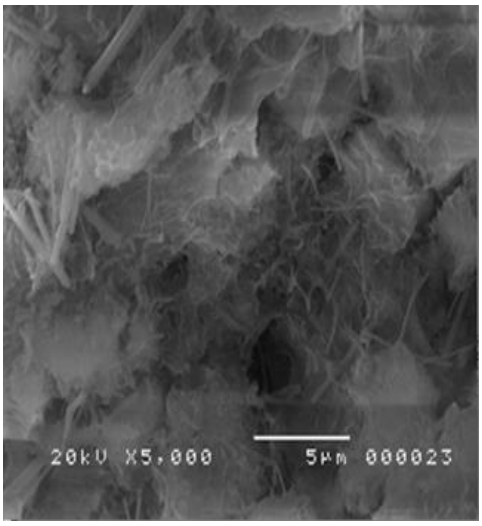


b.) at 42nd-day curing

Fig. 3 Micrographs of CPTB with 20% fines



a.) at 7th-day curing



b.) at 42nd-day curing

Fig. 4 Micrographs of CPTB with 40% fines

a.) 20% fines b.) 40% fines

c.) 60% fines

Fig. 5 Modes of failure of CPTB subjected to compressive test a.) and b.) diagonal shear failure c.) axial split

3.2 Stress-strain Interface Behavior

The stress-strain interface behavior between CPTB and concrete structure was evaluated through a direct shear test using strain rates that simulate the rapid and slow rates of loading. Specimens with 20% and 40% fines were subjected to shear force and normal stress after the 28th-day curing period following the experimental set-up described in Fig. 1. CPTB with 60% fines was no longer included in the experimentation since the compressive strength test did not show acceptable results. The typical stress-strain graphs are shown in Figs. 6 to 9. In both conditions, the stress-strain behavior of CPTB exhibits strain softening. The specimen showed a rapid increase in shear stress reaching a peak value at low shear strains and then decreases with increasing shear strains indicating strain softening until the shear stress at failure is attained. This indicates that specimen failed in a brittle manner. The shear stress at failure is described as the shear stress at which continued shearing occurs without a change in shear stress for a given normal stress.

The stress-strain curve defines the typical response of the dense sample. In this experimentation, the CPTB after 28th days of curing time is comparable to compacted, dense soil. CPTB with 20% and 40% fines, whether subjected to slow or fast strain rates, achieved an almost the same shear stress at failure. The shear stress at failure is greater for specimens subjected to greater normal stress (σ').

The shear stress at failure (τ_f) is plotted against the normal stress (σ') on a graph of τ against σ'. The best fit line joining the τ and σ' can be described by equation $\tau = \sigma'\tan\phi$. This line represents the failure envelope with slope tan ϕ.

The angle ϕ is referred to as the interface friction angle and is tabulated in Table 3. This friction angle describes the frictional resistance of CPTB when in contact with the concrete structure.

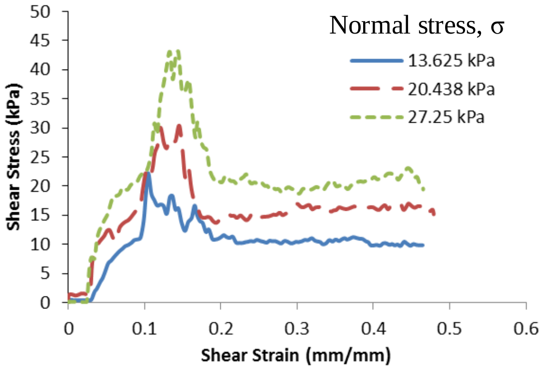


Fig. 6 Shear stress vs. shear strain of CPTB with 20% fines using fast strain rate

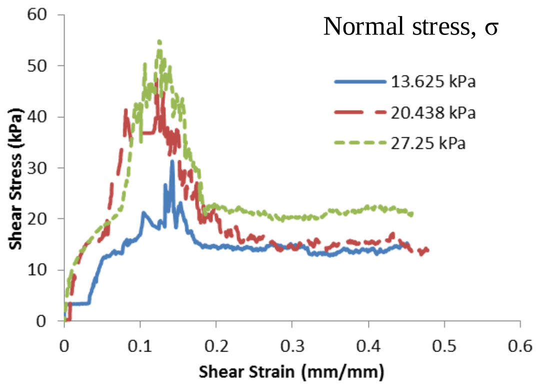


Fig. 7 Shear stress vs. shear strain of CPTB with 20% fines using slow strain rate

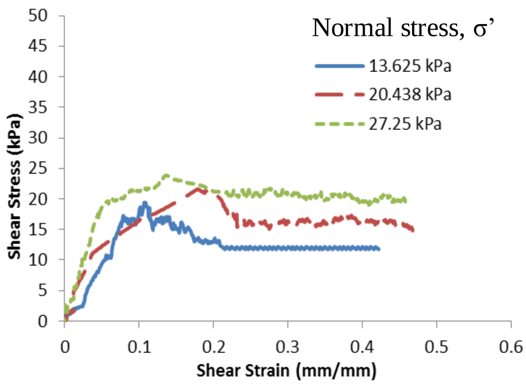


Fig. 8 Shear stress vs. shear strain of CPTB with 40% fines using fast strain rate

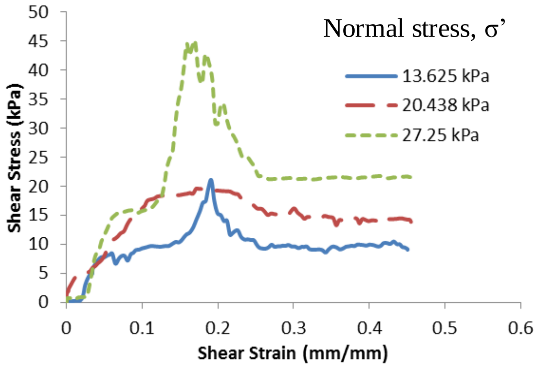


Fig. 9 Shear stress vs. shear strain of CPTB with 40% fines using slow strain rate

Table 3 Interface Friction Angle of CPTB

CPTB with	Interface Friction Angle, ϕ (deg.)
20% Fines - Fast Strain Rate	38.8
20% Fines - Slow Strain Rate	38.5
40% Fines - Fast Strain Rate	37.6
40% Fines - Slow Strain Rate	38.7

3.3 Hyperbolic Parameters

Shear stress vs. shear strain behavior of CPTB as presented in Figs. 6 to 9 was used to determine the hyperbolic parameters using the modified hyperbolic model. The modified hyperbolic model uses the shear stress against shear strain to describe the hyperbolic stress-strain behavior from the direct shear test. The hyperbolic relation between the changes in stress and strains is defined in terms of an initial shear modulus, *Gi* and the shear strength at failure, τ_f. The modified model approximates the stress-strain behavior from direct shear tests by a hyperbolic relation presented in a transformed plot in the form of γ/τ (shear strain/shear stress) versus γ (shear strain). The intercept of this straight line on the γ/τ axis is the reciprocal of initial shear modulus, G_i while the slope of the line is the reciprocal of the asymptotic shear stress, $\tau_{ult.}$ The variation of the initial shear modulus, G_i in response to changes in normal effective pressure can be represented using the power law approach as suggested by Janbu [9]. The parameters K (shear modulus number) and n (shear modulus exponent) describing initial shear modulus *(Gi)* are obtained from a best-fit straight line drawn through data points of the logarithmic diagram showing the values of normalized shear modulus (Gi/Pa) against the values of normalized normal effective stress (σ'/Pa), where the normalizing parameter *Pa* is the atmospheric pressure. The value of shear modulus number *K* is equal to the value of normalized *Gi* given by the best-fit line for a normal effective stress of one atmosphere. The slope of the line is the shear modulus exponent *n*. The failure ratio R_f that relates the asymptotic shear stress with shear stress at failure is the ratio of the shear stress at the failure to the asymptotic shear stress, τ_{ult}. The variation of the angle of internal friction ϕ' with respect to normal stress, σ' is described in terms of hyperbolic parameters ϕo and Δϕ. The parameters are obtained from a best-fit line drawn through data points on the plot of ϕ' vs. logarithm of normalized σ'/Pa. The modified hyperbolic model concepts and procedure for the determination of hyperbolic parameters are discussed in the study of Adajar and Zarco [3]. The summary of hyperbolic parameters is presented in Table 4. Applying these hyperbolic parameters in equations (1) to (3) allows the prediction of the interface shear stress – shear strain behavior of the CPTB when in contact with concretes structure at any value of shear strain and normal effective stress.

$$\tau_f = \sigma' \tan \phi' \quad (1)$$

$$\phi' = \phi_o - \Delta\phi \log\left(\frac{\sigma'}{P_a}\right) \quad (2)$$

$$\tau = \frac{\gamma}{\dfrac{1}{K \cdot P_a \left(\dfrac{\sigma'}{P_a}\right)^n} + R_f\left(\dfrac{\gamma}{\tau_f}\right)} \quad (3)$$

where:

τ = shear stress, KPa
γ = shear strain
σ' = normal effective stress, KPa
Pa = atmospheric pressure = 101.325 KPa

Using the determined hyperbolic parameters, the model's response to the test data was compared with experimental data. The comparison of the test data and the calculated hyperbolic response are shown in Figs. 10 to 12. The modified hyperbolic model provides a good approximation of the interface shear stress – shear strain behavior of the CPTB. The limitation of the model is that it cannot capture the peak shear stress [3], but it can provide a good prediction of shear stress at failure.

Table 4 Hyperbolic parameters of CPTB

Hyperbolic Parameter	Description	Value
K	Shear modulus number	170.99
n	Shear modulus exponent	0.9809
R_f	Failure ratio	0.9971
$\emptyset_o$	Friction angle parameter	31.021°
$\Delta\emptyset$	Friction angle parameter	5.126°

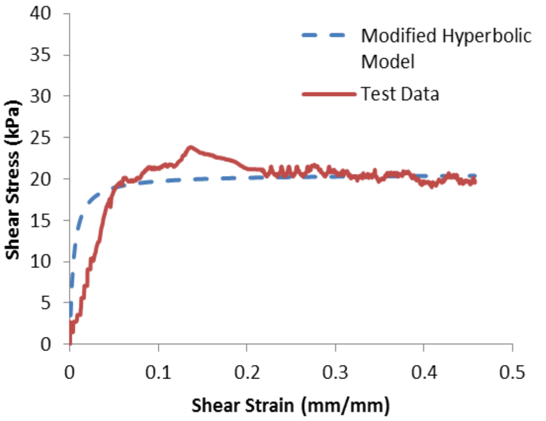


Fig. 10 Comparison of stress-strain curve using modified hyperbolic model and from test data of CPTB with 40% Fines and normal stress of 27.25 KPa

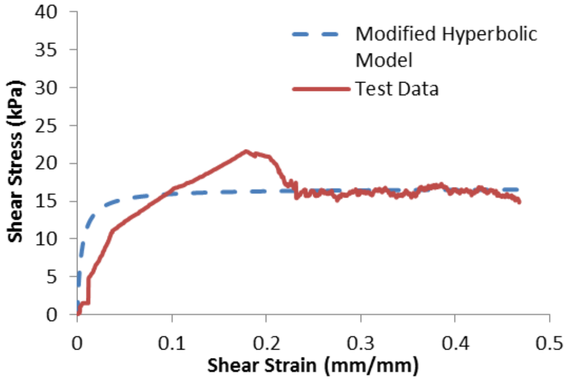


Fig. 11 Comparison of stress-strain curve using modified hyperbolic model and from test data of CPTB with 40% Fines and normal stress of 20.438 KPa

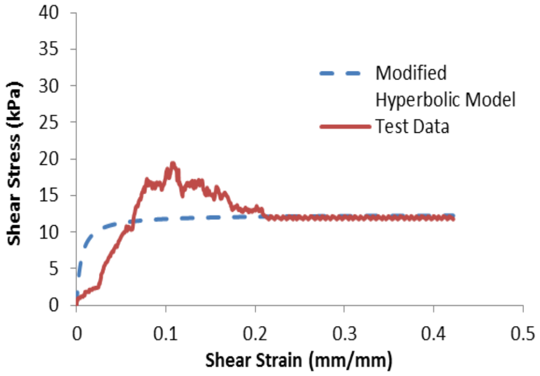


Fig. 12 Comparison of stress-strain curve using modified hyperbolic model and from test data of CPTB with 40% Fines and normal stress of 13.625 KPa

4. CONCLUSION

Strength properties of cemented-paste tailing backfill with waste from aggregate quarry as a sand substitute were determined through unconfined compressive strength test and direct shear test. Its soil-structure interface behavior was evaluated. The following are the conclusions drawn from test results:

Cemented-paste tailing backfill with fine contents up to 40% showed acceptable values of unconfined compressive strength and interface friction angle suitable for ground support to structures. The unconfined compressive strength depicting its long-term behavior is in the range of 120 kPa to 150 kPa comparable to clay with a stiff consistency. The mode of failure exhibited by the samples is diagonal shear. Its microstructure showed an increase in porosity as curing age increases.

The stress-strain interface behavior between CPTB and concrete structure exhibits strain-softening. The average interface friction angle is 38^{o} which can be associated with its dense condition. The modified hyperbolic model was applied and hyperbolic parameters were formulated to predict the interface shear stress-shear strain behavior of the CPTB when in contact with concretes structure at any value of shear strain and normal effective stress. The modified hyperbolic model provides a good approximation of the interface shear stress-shear strain behavior of the CPTB. The model cannot capture the peak shear stress, but it provides a good prediction of shear stress at failure.

The CPTB used in this study can be a good alternative to the conventional cemented-paste backfill when used as a free-standing fill if the design criteria are not requiring a high compressive strength value.

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