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RESEARCH ARTICLE

FRICTION ANGLE OF POLISHED SURFACES OF SANDSTONE AND CONGLOMERATE FROM THE SEMANGGOL FORMATION, BERIS DAM, KEDAH DARUL AMAN

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ABSTRACT

The Beris Dam is founded on a sequence of thick bedded to massive conglomerate and gritstone with some sandstone and mudstone, mapped as the Semanggol Formation of Triassic age. Portable shear box tests on polished surfaces of a sandstone, and a conglomerate, core yield friction angles of 18.0^o and 20.8^o, respectively. These friction angles are comparable with residual friction angles of between 17.5^o and 19.0^o determined in field and laboratory tests on sheared mudstone surfaces of the Semanggol Formation at the Muda Dam. Apparent cohesion values determined in the portable shear box tests result from the restricted normal displacement test conditions and should not be considered in shear strength calculations.

KEYWORDS

Friction angle, polished sandstone and conglomerate

1. INTRODUCTION

The portable shear box (or field shear strength apparatus) was developed by Dr. E. Hoek and designed to evaluate the shear strength of a discontinuity plane in the field (Aufmuth, 1973). The apparatus can accommodate plaster of Paris mounted core samples of up to 102 mm size, or block samples with a face area of up to 120 mm x 150 mm. The box operates in the same manner as a laboratory shear box and comprises two halves; the lower half having a hydraulic ram for shear, and the upper half a hydraulic ram for normal loading (Hoek and Bray, 1977). Although the portable shear box is widely used for routine testing it does have a number of disadvantages when precise control over testing conditions is required. The use of manually-operated jacks for instance, can make it difficult to control shear displacement and to maintain a constant normal stress throughout the test (Priest, 1993).

The type of shear test that can be carried out with the portable shear box is up to the individual user as several shearing modes are available (Aufmuth, 1973). The apparatus can be used to determine the initial peak strength using standard procedure, or it may used to obtain the residual shear strength by either manually returning the shear box to its initial position and continuing shearing in the same direction, or by using a second hydraulic shearing jack at the opposite end from the first and shearing back and forth (Aufmuth, 1973). There is, however, some uncertainty as to which of these modes is more representative of the shearing process in the rock mass as the detrital material is disturbed when the specimen is manually lifted back to the original position, whilst the direction of rolling of the particles is reversed. Hoek and Bray (1977) have thus preferred the single jack system as they feel that the direction of shearing under load should be kept constant.

One of the problems associated with practical use of the portable shear box is that the device for applying the normal load is a stiff one which restrains dilation and leads to an increase in the normal stress (Sher Bacha *et al.*, 2014). A shear test conducted under restricted normal displacement conditions will thus generally yield a considerably higher shear strength than one conducted under constant normal stress because normal joints are dilatant (Goodman, 1974). Dilation during shear can be measured by monitoring the normal load changes during shearing.

The limited specimen size that can be tested in the portable shear box also means that it is very difficult to test joints with surface roughness that are representative of *in situ* conditions in the rock mass. It has thus been suggested that the portable shear box assembly be limited to measurement of the basic friction angle (φ_b) which can be done by testing rock surfaces prepared by means of a clean, smooth diamond saw cut (Hoek and Bray, 1977). It has also been pointed out that it is possible to test field samples and then determine the basic friction angle (Φ_b) by subtracting the average roughness angle (i), measured on the specimen surface before testing, from the measured angle ($\varphi_b + i$) as determined in the test (Hoek and Bray, 1977).

In Malaysia, there is limited published data on the shear strength of discontinuities in bedrock; the first report stating that the shear strength of mudstone of the Triassic Semanggol Formation at the Muda Dam in Kedah State is close to its residual value with good correspondence in the residual friction angle (φ_r) of 17.5^o to 19^o determined by field and laboratory tests (James, 1969). It has also been noted that field and laboratory shear tests on undisturbed and remoulded mudstone from the Muda Dam in Kedah State indicated a residual angle of shearing resistance of about 18^o, whilst peak shear strengths were more variable (Clarke et al., 1970).

The use of the Phi-10 Roctest shear box assembly has been described, though no details were provided on the 15 samples said to have been tested (Mohd. For Mohd. Amin et al., 2000). The use of the of the Phi-10 Roctest shear box assembly to test 15 rock cores from a cut slope at the

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APMC Quarry in Rawang yielded a peak shear strength between 0.6 and 2.3 MPa, and a residual shear strength between 0.3 and 1.6 MPa (For Mohd. Amin et al., 2000).

Portable shear box tests on joint planes in meta-sedimentary rocks from the Simpang Pulai to Cameron Highlands Highway yielded friction angles of between 23.7^o and 30.0^o, and apparent cohesions of between 76 and 783 kPa for peak and residual shear strengths (Nkpadobi *et a*l., 2015). In Singapore, a basic friction angle (φ_b) of 32^o was reported from portable shear box tests on rough sawn sandstone blocks of the Triassic Jurong Formation exposed at Bukit Batok New Town (Pitts, 1988).

In this paper are presented the results of portable shear box tests carried out to determine the friction angle along polished surfaces of sandstone and conglomerate from the Triassic Semanggol Formation outcropping at the Beris Dam in Kedah State.

2. Geological Setting of Samples

The concrete-faced rockfill Beris Dam is located in the narrow valley of Sungai Beris, some 1.6 km upstream of its confluence with Sungai Muda in Sik District in Kedah State (Figure 1). The dam, which is 40 m high and about 155 m long at its crest, was completed in 2004 and used to regulate flows in the Sungai Muda drainage basin to augment water available for irrigation as well as domestic and industrial water supply and other uses (DID, 2015). The dam has a catchment area of 166 km^2; the reservoir at normal pool level covering an area of 13.7 km^2 and at maximum pool level inundating an area of 16.1 km^2 (Tajul and Ismail, 2003).

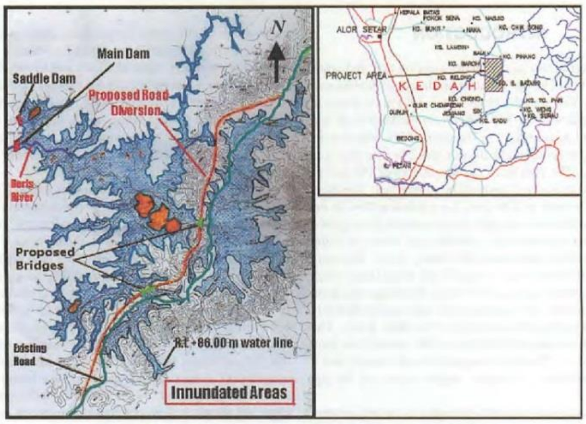

Figure 1: Location of the Beris Dam in Kedah Darul Aman. [From Figure 1 (Jamaluddin and Yusoff, 2003)].

The Beris Dam and its spillway are founded on a sequence of massive to thick bedded conglomerate and gritstone with some sandstone and mudstone that is mapped as the Semanggol Formation of Triassic age (Teoh, 1992). Conglomerate predominates at the right abutment and underneath the dam, whilst at the left abutment and spillway, the conglomerate is inter-bedded with gritstone and coarse sandstone (Tajul and Ismail, 2003). The conglomerate, gritstone and sandstone then exposed at the foundation and abutments as well as at the spillway, were reported to be slightly to moderately weathered (Grades II - III) (Tajul and Ismail, 2003).

The matrix-supported, polymict conglomerate contains gravel to pebble-sized clasts of black to dark slate and mudstone, cert, quartz and other rock fragments (possibly volcano-clastics, sandstone and quartzite), whilst the matrix comprises coarse sandy to gritty materials of quartz, feldspar and rock fragments (Tajul and Ismail, 2003). The rocks were reported to be generally hard, compact and well indurated; requiring several blows of the geological hammer to collect samples (Tajul and Ismail, 2003).

The gritstones are transitional between the conglomerate and sandstone and composed of fine gravel to coarse sand grains of quartz, quartzite, sandstone, chert and mudstone as well as other rock fragments. They are grey, hard, compact and occur as inter-beds in the conglomerate and sandstone. The sandstone is generally a light grey, fine to coarse-grained, hard, compact and well indurated rock. In places, the thick sandstone beds contain shale/mudstone partings (Tajul and Ismail, 2003). The bedding planes are often not clearly defined due to the thick to massive bedding. At the right abutment, however, bedding planes strike about west to west-southwest with dips of 15^o to 30^o towards north. At the left abutment, the bedding strikes about east-west with dips of 45^o to 52^o towards south. The rocks are intensely faulted and jointed with a total of 5 to 6 major joint sets having been identified (Tajul and Ismail, 2003). Several boreholes were drilled during Site Investigation works for the Beris Dam and associated structures and some broken cores were provided to the author for study and determination of their physical properties.

3. Methodology

Textural features of a 7.2 cm long sandstone core from borehole QR 3 (38.41-38.71 m depth), and a 9.0 cm long conglomerate core from borehole QR 3 (13.42-13.68 m depth) were first described before thin sections were prepared of representative samples. Densities, unit weights and apparent porosities of the representative samples were then determined using the saturation and bouyancy technique of ISRM, whilst the specific gravity of the constituent mineral grains was determined with a pycnometer (ISRM, 1979; GBRRL, 1959). The two core samples were then sawn into two halves (longitudinally) using a thin diamond blade. The flat surfaces of the cores were then ground on a lathe with a set rotation of 160 revolutions per minute, using first 6 µ diamond paste for about 10 minutes and then with 0.3 µ alumina powder for about 5 minutes. The ground surfaces were finally polished by hand on glass plates using 0.05 µ alumina powder for about 5 minutes.

The two halves of the cores were then mounted in plaster of Paris in molds of specific shape and size before being allowed to harden (set). After hardening, the mounted samples were carefully placed in the upper and lower halves of the portable shear box to ensure that the polished surfaces were in proper contact. The hydraulic jack for applying the shear load was then attached to the upper half of the shear box, before the hydraulic jack for applying the vertical load was set to a specified value. The hydraulic pressure in the jack for applying the shear load was then gradually increased by hand and the horizontal (or shear displacement) of the upper half of the core relative to the lower half monitored. Readings of the gauges showing the horizontal displacement and the hydraulic pressure in the jack for the shear load were recorded at regular intervals till a total displacement of 12 mm was reached. The upper half of the shear box was then lifted and replaced at the original starting position before a new vertical load applied and the process of shearing repeated. This process of manually resetting the upper half of the shear box to the original starting position was to ensure that the direction of shearing remained the same in all tests. It is to be noted that vertical displacements were not monitored during the tests as appropriate gauges were not available.

4. Results

4.1 Petrography of investigated rock materials

In thin section, the moderately to poorly sorted conglomerate is seen to consist of 1.0 to 6.0 mm sized clasts of mainly chert, quartz and rock fragments in a finer grained (0.1-0.5 mm size) matrix of similar composition. The clasts are of subangular to angular shapes with the rock fragments including quartz-mica schist, siltstone and sandstone. Both mono-crystalline and poly-crystalline quartz clasts are present with some grains being well rounded. In thin-section, the well sorted sandstone is seen to consist of sub-angular to angular grains of mainly quartz, chert and rock fragments (siltstone and quartz-mica schist). The grains are 0.13 to 1.5 mm in size with a mean size of 0.25 mm. Some grains are well rounded, while opaque minerals are sometimes seen.

4.2 Physical properties of investigated rock materials

The conglomerate, gritstone and sandstone exposed at the foundation and abutments of the Beris Dam as well as Spillway have been reported to be slightly to moderately weathered (Tajul and Ismail, 2003). The polymict conglomerates were said to be generally hard, compact and well indurated, requiring several blows of the geological hammer for collection of samples. The sandstone was said to be generally a light grey, fine to coarse-grained, hard, compact and well indurated rock (Tajul and Ismail, 2003). Laboratory determined densities and unit weights of the tested sandstone and conglomerate cores show them to be dense, compact and well indurated rock materials (Table 1). Differences in densities and unit weights, furthermore, show the sandstone with apparent porosities of 3.8% to be more compact and dense than the conglomerate which has apparent porosities of about 10%.

Table 1: Physical properties of sandstone and conglomerate cores

Sample	Bulk Density (kg/m^3)	Dry Density (kg/m^3)	Apparent Porosity (%)	Bulk Unit Weight (kN/m^3)	Dry Unit Weight (kN/m^3)	Specific Gravity Particles
Sandstone 1	**2,634**	**2,597**	**3.7**	**25.83**	**25.46**	**2.64**
Sandstone 2	**2,615**	**2,576**	**3.9**	**25.64**	**25.26**	**2.65**
Conglomerate 1	**2,525**	**2,423**	**10.1**	**24.76**	**23.77**	**2,67**
Conglomerate 2	**2,531**	**2,434**	**9.7**	**24.82**	**23.87**	**2.67**
Conglomerate 3	**2,534**	**2,437**	**9.7**	**24.85**	**23.90**	**2.67**

4.3 Plots of horizontal displacement versus shear stress

Plots of horizontal (or shear) displacement versus shear stress for different normal stresses of the polished sandstone core are shown in Figure 2, and those for the polished conglomerate core in Figure 3. It is to be noted that the stress values are based on uncorrected areas (i.e. original area in contact), even though it is recognized that areal changes occur during the horizontal displacements. The main reason for use of uncorrected areas is that the normal and shear stresses to be considered for determination of the shear strength parameters (using the Mohr-Coulomb's equation) are those of very limited horizontal displacements (1-3 mm).

The plots of horizontal displacement versus shear stress for both cores (Figures 1 and 2) are quite similar with level sections separated by upward or downward sloping sections. The level sections, which indicate displacement under constant shear stress, are prominent in the plots of low normal stress (<1,000 kPa) while those of higher normal stress show more variations. All the plots furthermore, show an increase in shear stress with increasing displacement; an effect that is due to the stiff nature of the device applying the normal stress in the portable shear box (Goodman, 1974; Bacha et al., 2014).

The tests at moderate to large normal stresses (>1,000 kPa) also sometimes displayed "stick-slip" phenomena with the sudden and noisy horizontal displacement of the two halves following an increase in shear stress. This "stick-slip" phenomena is considered to result from over-riding of asperities on both halves, these asperities inter-locking under the moderate to high normal stresses. It is finally to be noted that the plots of horizontal displacement versus shear stress do not all start at the origin for there was a need to apply a minimum hydraulic pressure to the horizontal ram in order to ensure that it did not drop before shearing was started.

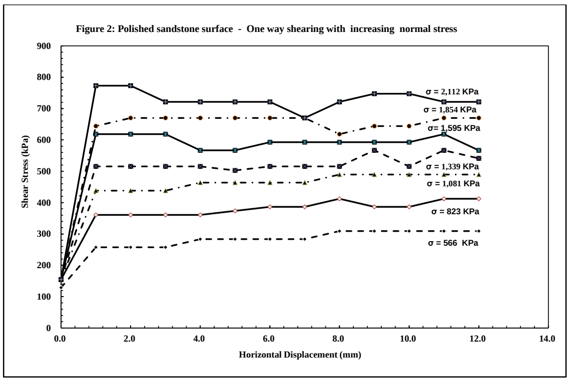

Figure 2: Horizontal displacement versus shear stress for polished sandstone core under different normal stresses

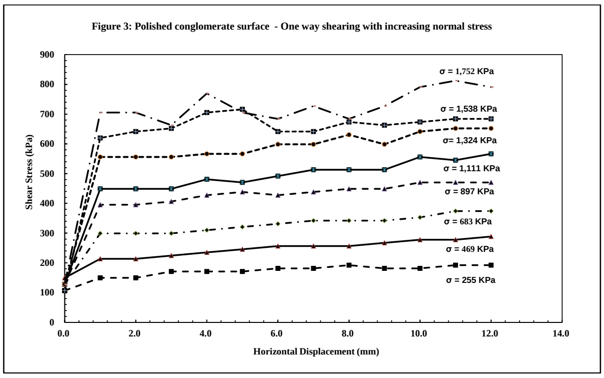

Figure 3: Horizontal displacement versus shear stress for polished conglomerate core under different normal stresses

5. Discussion

Determination of the strength of discontinuities from field or laboratory tests involves application of one or more of the equations that describe the relationship between the shear (τ_f), and normal (σ_n), stresses acting on a discontinuity at the point of failure (Priest, 1993). In view of the present tests involving planar, polished surfaces, the most appropriate relationship (between the shear (τ_f), and normal (σ_n), stresses at the point of failure) would be the Mohr-Coulomb failure criterion which states that (Hoek and Bray, 1977):

$$\tau_f = c + \sigma_n \tan \varphi$$

where c is known as cohesion, and φ is known as the friction angle.

Table 2: Strength parameters of polished sandstone core from plots of normal stress versus shear stress (at 1-3 mm displacement).

Normal Stress σ (kPa)	Shear Stress τ(kPa)	Regression Equation (Linear Relationship)	R^2	Apparent Cohesion (kPa)	Friction Angle (ϕ°)
566	**258**	**y = 0.3247x + 84.3 kPa**	**0.9963**	**84.3**	**18.0**
823	**361**				
1,081	**438**				
1,339	**515**				
1,596	**618**				
1,854	**670**				
2,112	**773**				

Note: Stresses involve uncorrected areas at 1-3 mm displacement.

Table 3: Strength parameters of polished conglomerate core from plots of normal stress versus shear stress (at 1-3 mm displacement).

Normal Stress σ (kPa)	Shear Stress τ(kPa)	Regression Equation (Linear Relationship)	R^2	Apparent Cohesion (kPa)	Friction Angle (ϕ°)
255	**150**	**y = 0.3811x + 43.7 kPa**	**0.9971**	**43.7**	**20.9**
469	**214**				
683	**299**				
897	**395**				
1,111	**449**				
1,324	**556**				
1,538	**641**				
1,752	**705**				

Note: Stresses involve uncorrected areas at 1-3 mm displacement.

Tables 2 and 3 show the values of the normal, and shear, stresses acting on the polished surface of the sandstone, and conglomerate, cores respectively, for limited displacements between 1 and 3 mm. The best fit linear relationships for plots of these normal and shear stresses furthermore, yield the equations shown in Table 2 for the sandstone core ($\tau_f = 84.3 + 0.3247\sigma_n$ kPa) and in Table 3 for the conglomerate core ($\tau_f = 43.7 + 0.3811\sigma_n$ kPa). R-squared (R^2) values (coefficient of determination) show a very good fit between the regression lines and all of the data plots for both polished core (Tables 2 and 3).

Arising from the Mohr-Coulomb failure criterion, the friction angle (φ) for the polished sandstone core is 18.0° (Table 2), whilst that for the polished conglomerate core is 20.8° (Table 3). These values represent friction angles along polished surfaces and are thus directly comparable with the residual friction angle (φr) of between 17.5° and 19.0° determined by field and laboratory tests on sheared mudstone from the Semanggol Formation at the Muda Dam in Kedah (James, 1969). Furthermore, researchers have stated that these mudstones have extensive and continuous shear planes with a residual angle of shearing resistance of about 18°, whilst peak shear strengths were more variable (Clarke et al., 1970).

Published data on basic friction angles (φ_b) for diamond sawn, conglomerate, and sandstone, blocks furthermore, are reported to be 35°, and between 25° and 35°, respectively (Hoek and Bray, 1977). In Singapore, a basic friction angle (φ_b) of 32° has been determined from portable shear box tests on rough sawn, sandstone blocks from the Triassic Jurong Formation (Pitts, 1988). These published values for diamond sawn surfaces are much larger than those of the presently determined angles of 18.0°, and 20.8°, for the polished sandstone, and conglomerate, surfaces respectively.

It is, however, to be noted that polishing of rock surfaces leads to a decrease in the friction angle for there is smoothening of asperities (Hencher and Richards, 2015). For instance, have shown that polishing of Palaeozoic carbonate rocks leads to a considerable reduction in friction angles with highly polished limestone surfaces having friction angles of between 11.8° and 13.0°, though tilt tests yield a basic friction angle of between 21.5° and 41.3° (Hu and Cruden, 1988).

An unexpected result in application of the Mohr-Coulomb failure criterion for determination of the best fit line is the presence of a cohesion intercept. This apparent cohesion value is unexpected for there is no "bonding" or adhesion between the two halves of the polished cores. It is likely that this apparent cohesion is due to the stiff nature of the hydraulic jack applying the normal load which restrains movement in the vertical direction (dilation) (Bacha et al., 2014). A shear test conducted under restricted normal displacement conditions, furthermore, generally yields a considerably higher shear strength than one conducted under constant normal stress (Goodman, 1974).

Lorcher and Rieder have also stated that the apparent cohesion value determined in laboratory direct shear tests (involving surface areas of about 200 cm^2) on bedding planes in layered Jurassic Limestone should be disregarded, as no such cohesion was found in field direct shear tests (involving surface areas of about 3 m^2) (Lorcher and Rieder, 1969). A group researchers have also pointed out that in view of extensive and continuous shear planes in mudstone of the Semanggol Formation at the Muda Dam in Kedah, it is unwise to rely upon an angle of shearing resistance in excess of 18° and to ignore any cohesion in stability calculations (Clarke et al., 1970).

6. Conclusions

It is concluded that portable shear box tests on polished surfaces of a sandstone, and a conglomerate, core yield friction angles of 18.0° and 20.8°, respectively. These friction angles are comparable with residual friction angles of between 17.5° and 19.0° determined in field and laboratory tests on sheared mudstone surfaces of the Semanggol Formation at the Muda Dam. Apparent cohesion values determined in the portable shear box tests result from the restricted normal displacement test conditions and should not be considered in shear strength calculations.

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