In-situ shear strength testing of conglomerate formation for a large RCC dam (Koysha, Ethiopia)

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Abstract. Koysha Dam, located in the Southern Nations, Nationalities and Peoples' Region of Ethiopia, is the fourth plant of the Gibe-Omo cascade comprising Gilgel Gibe (IP=200 MW), Gibe II (IP=420 MW) and Gibe III (1'870 MW) all in operation. The plant includes a 200 m high RCC gravity dam, a gated spillway on the left bank and an open-air powerhouse housing 6 Francis turbines fed by 2 steel penstocks crossing the dam body. The right abutment of the dam is partly founded on conglomerate, composed by a variable proportion of cobble and gravel sized basalt and rhyolite-trachyte sub-rounded elements surrounded by a weak matrix of fine sand and silt particles. The geotechnical characterization of such a complex material resulted in a very challenging task due to the substantial impracticality of collecting high quality, undisturbed and representative samples. Therefore, the mechanical behaviour of the conglomerate has been assessed by means of in-situ large scale shear tests carried out into a 70 m long inspection tunnel. The test results show that in the range of selected confinement pressures (0.3 to 1.0 MPa) the conglomerate exhibits a softening behaviour. Peak friction angle and cohesion are coherent with data and empirical models presented in the literature which indicate a strong correlation between volumetric block proportion and mechanical properties of the Block-In-Matrix (BIM) rock.

1. Introduction

Koysha dam is located on the Omo river, 130 km downstream of Gibe III dam in the Southern Nations, Nationalities and Peoples' Region of Ethiopia. The project includes a 200 m high RCC gravity dam, a gated spillway on the left bank and an open-air powerhouse housing 6 Francis turbines fed by 2 steel penstocks crossing the dam body. The owner of the project is the Ethiopian Electric Power Company (EEPC) whereas the main contractor is WeBuild SpA (Italy). Koysha is the 4th plant on the Omo river, downstream of the operating Gilgel Gibe, Gibe II and Gibe III. With its 1620 MW of installed power and 6260 GWh of annual energy production, Koysha is one of the most important projects in the Ethiopian Government's commitment to meet the country's present and future power requirements.

The project presents many geotechnical challenges, one of the most important being the presence of heterogeneous Conglomerate outcropping on the right dam abutment. The Conglomerate is a melange formation composed by unweathered to moderately weathered gravel and pebble clasts into a silty and sandy matrix. From a geotechnical point of view, the Conglomerate represents a typical block-in-matrix rock (bimrock) [1], [2] and [3]. These geological mixtures exhibit a complex behaviour with considerable spatial, lithological and mechanical variability and common fundamental engineering problems related to their geotechnical characterization [4] and [5].

2. Geological and geo-structural framework

At broad scale, the project area is located in a physiographical region known as the "western plateau". This region was affected by Cenozoic volcanism triggered by a mantle superplume uplift whose decompression melting generated massive quantities of basaltic magma in the lithosphere and resulted in the formation of a flood basalt province.

The site area consists almost entirely of volcanic rocks composed by Andesitic and Basalt Andesitic flows (A-Formation) that form the lower part of the stratigraphic sequence. This unit belongs to a pre-rift phase and is constituted by a continuous layered sequence of sub-aerial lava flows some meters thick, dipping towards West at low angle. According to its textural characteristics the A-Formation is divided into a brecciated (B1) and massive member (B2). Both members formed mainly during the quenching of the lava flows. No unconformities in the stratigraphic succession were observed except for decomposed to highly weathered horizons formed after some break during the lava emplacement.

The A-Formation is capped by the Conglomerate Formation (C) on the right bank and by the Columnar basalt Formation (Bc) on the left bank, which constitutes the top of the plateau on both the banks. The plateaux were part of a single tabular structure subsequently dissected by the river erosion that followed the upheaval of the Ethiopian region.

The geo-structural setting of the project area is characterized by a series of tectonic features whose direction is consistent with the one of the main faults at regional scale (20-30°N) that are related to the development of the East African Rift System (EARS) creating a horst-graben tectonic style with system of basins and ranges bounded by faults with a general tilting to the W.

2.1. Conglomerate formation

The right abutment of the dam is characterized by three main lithological units (Figure 1):

- Upper Basalt formation (Bc) at the top of the right abutment.
- Conglomerate formation (C) which overlies unconformably the Andesite unit (A) and is capped by the columnar basalt (Bc).
- Andesite formation (A) at the bottom of right abutment.

The shape of the Conglomerate Formation is that of a flat top surface dipping 4-17° toward 216-222 N, with a morphologic low located 600 m downstream of the dam axis.

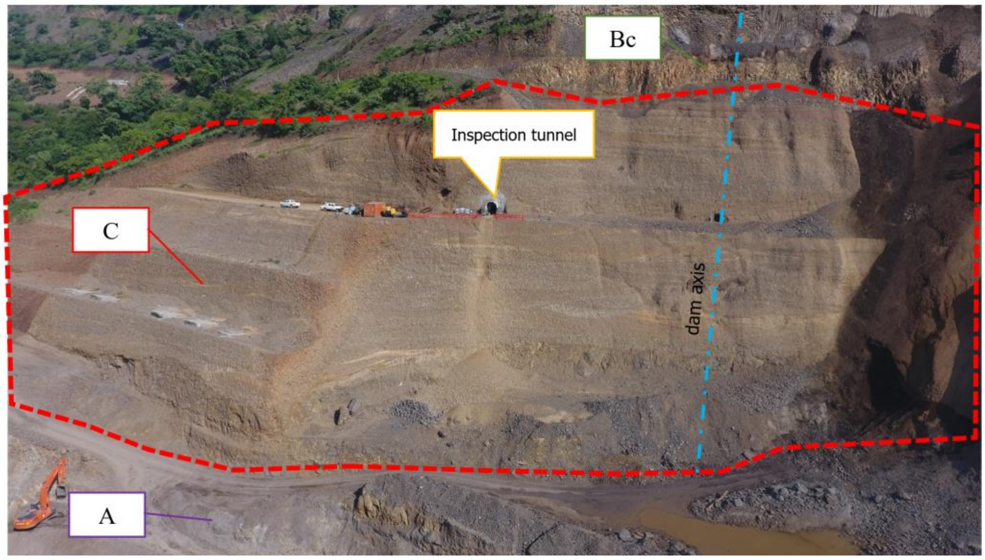

Figure 1. Geology of the right dam abutment.

The Conglomerate formation is composed by two members:

- CONGLOMERATE (Co) (92 % of C-Formation) – The Conglomerate member (Co) covers almost the totality of the C-Formation. It is composed by unweathered to moderately weathered gravel and pebble clast supported conglomerate mainly formed by basalt and rhyolite-trachyte rounded elements. In the boreholes, it occurs as a cemented, strong to moderately strong rock. The matrix is predominantly composed by sand and silt in varying proportions. Locally the conglomerate is interbedded by decimetric plastic clayey levels.
- SILTSTONE AND SANDSTONE (Cs) (8 % of C-Formation) – These two rock types appear as a brown and moderately strong rock. Rock cores have generally a low RQD value (< 45%). Cs levels are placed at the top of the conglomerate formation or as intercalations.

The typical sedimentary succession observed on site is illustrated in Figure 2. The Cs lenses outline the following main characteristics:

- A lateral downlap geometry in the foreset portion of the fluvial bar.
- A paleocurrent indicating a roughly right to left direction truncated by subsequent channel in a higher fluvial energy period.
- An average thickness generally less than 50 cm (locally up to 1 m).

The above characteristics indicate the absence of an upstream to downstream continuity of the Cs levels, due to the braided depositional geometry at time.

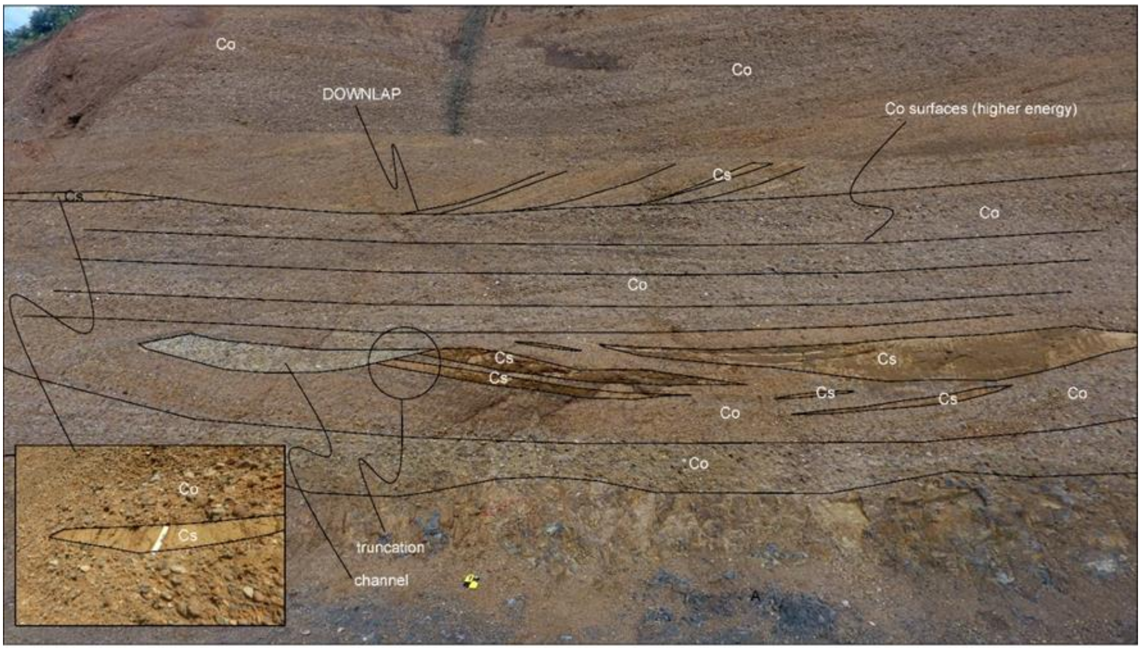

Figure 2. Depositional geometries in the C unit with sandy bars and subsequent cutting channels.

The granulometric curves of Conglomerate samples are reported in Figure 3 whereas some index properties derived from laboratory testing on remoulded samples are shown in Table 1. The Co formation can be classified according to the USCS classification as a Poorly Graded Gravel (GP) with a minor content of material passing through sieve 0.075 mm (5% on average), generally with low plasticity. Samples taken from the inspection gallery where the in-situ shear tests have been carried out have an average saturation degree (*Sr*) of 90% and a void index (*e*) of 0.23. The shear strength parameters of the matrix have been estimated by means of both standard consolidated, undrained and large scale (d_{max}= 50 mm) triaxial tests executed on reconstituted samples. The tests identified a friction angle (ϕ') of 29° and a cohesion (c') of 200 kPa.

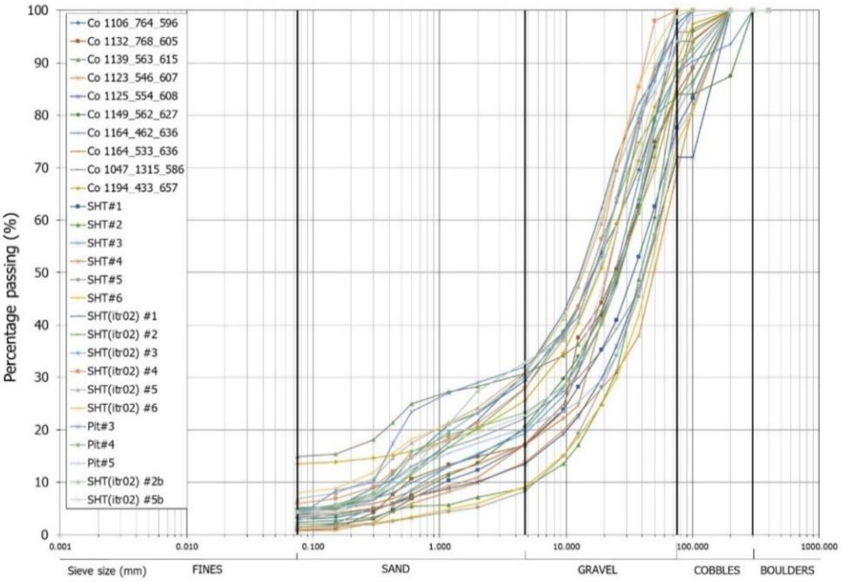

Figure 3. Granulometric curves of Conglomerate samples from sieve analysis.

Table 1. Geotechnical index properties of the conglomerate and the matrix.

Conglomerate			**Matrix**		
γ (kN/m^3)	**Sr (%)**	**e (-)**	**γ (kN/m^3)**	**w_L**	**I_p**
25	90	0.23	20	45	13

3. In-situ large scale shear tests

3.1.1. Testing procedure

Due to the practical difficulty in collecting undisturbed samples for laboratory testing, 8 in-situ large scale shear tests have been performed to determine the strength parameters of the Conglomerate according to the procedure presented by [6], [7] and [8]. The tests were carried out inside a 70 m long inspection gallery excavated in the Conglomerate Formation below the ground water level as to ensure near-fully saturated conditions of the material. The testing apparatus is reported in Figure 4.

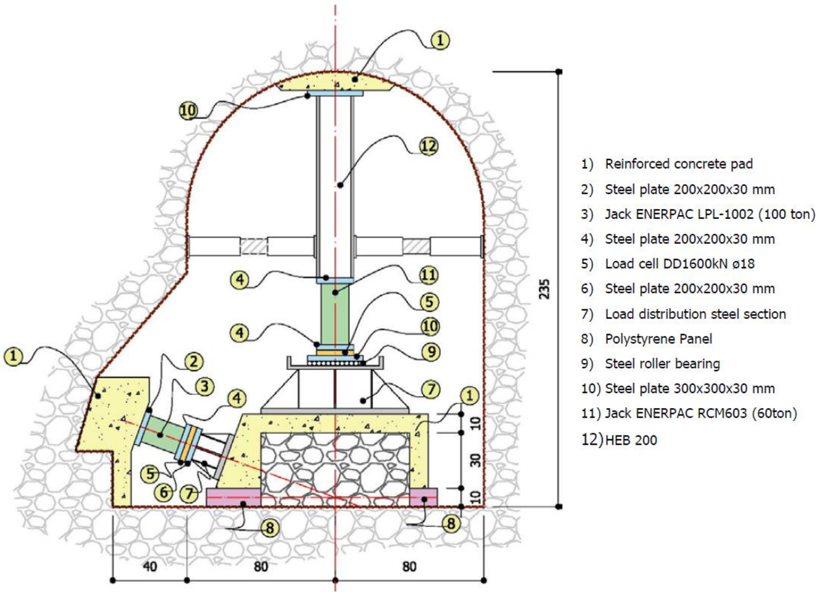

Figure 4. Layout of in-situ large scale shear test apparatus.

The following testing procedure has been adopted:

1. PREPARATION: each block is carefully cut to the required dimensions (800 × 800 × 400 mm) avoiding disturbance and loosening of the material. The block is then encapsulated into a reinforced concrete pad isolated from the ground by means of a polystyrene panel.
2. CONSOLIDATION: the normal load is gradually increased up to the full value determined for the test by means of a vertical hydraulic jack and a steel section helping to distribute uniformly the load to the test block. In this phase, the pore water pressure in the block dissipates under full normal stress before the shear load is applied. The consequent normal displacement is recorded. The consolidation phase is considered completed when the rate of change of normal displacement recorded at each gauge is less than 0.05 mm in 10 minutes. In all tests, the applied normal stress ranges in the interval $\sigma_n = 0.3 \div 1.0$ MPa, selected according to the expected stresses induced by dam load.
3. SHEARING: the shear force is applied continuously at a rate of 0.1÷0.2 mm/min using a second hydraulic jack inclined at an angle of 20° with the horizontal to have the resultant line of applied shear force passing through the centre of the base of the shear plane. Shear force is measured by means of a digital load cell inserted between the hydraulic jack and the load distribution steel section. During testing the normal and lateral displacements are continuously monitored by means of displacement gauges installed on three faces of the test block. Data are plotted as indicated in Figure 5.
4. BLOCK INSPECTION: after the completion of each test, the blocks are turned upside down and cleaned. The profiles of the upper and lower shearing surfaces are surveyed with the Barton's profilometer and by means of photogrammetric techniques. The grading curve of each block and the index properties are determined in the laboratory.

A typical shear stress-shear displacement curve recorded during the in-situ shear tests is reported in Figure 5. The normal displacement measured during testing is also shown. The test blocks show a clear dilatant behaviour with well-identified peak and residual strength. The normal displacement decreases in the first phase of the tests and then increases following the change in volume of the test block. A photograph of the upper portion of the block after testing is reported in Figure 6 along with the measured shearing surface.

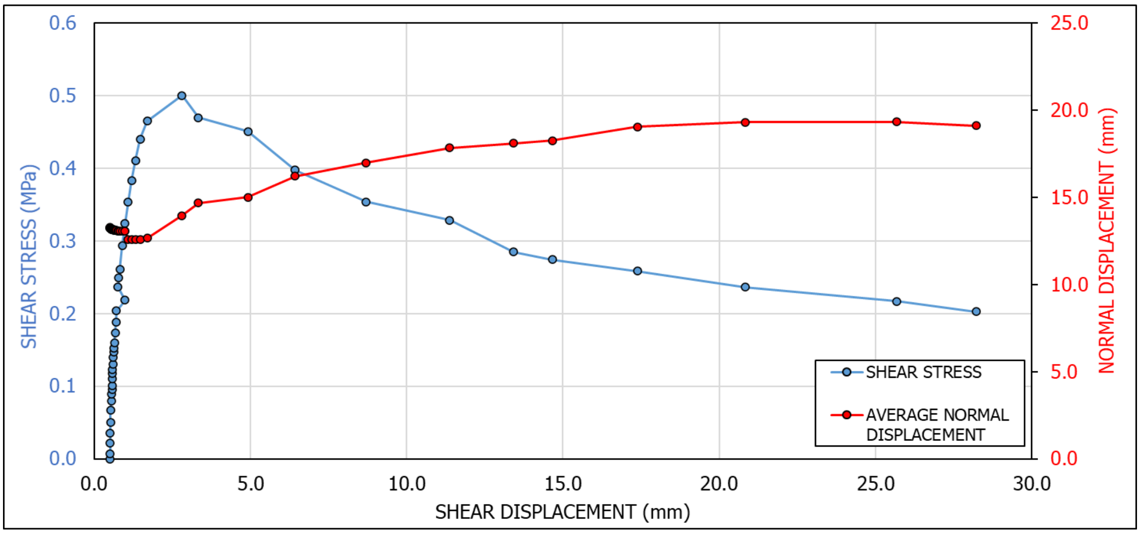

Figure 5. In-situ large scale shear test, typical shear stress vs displacement curves (block 02B).

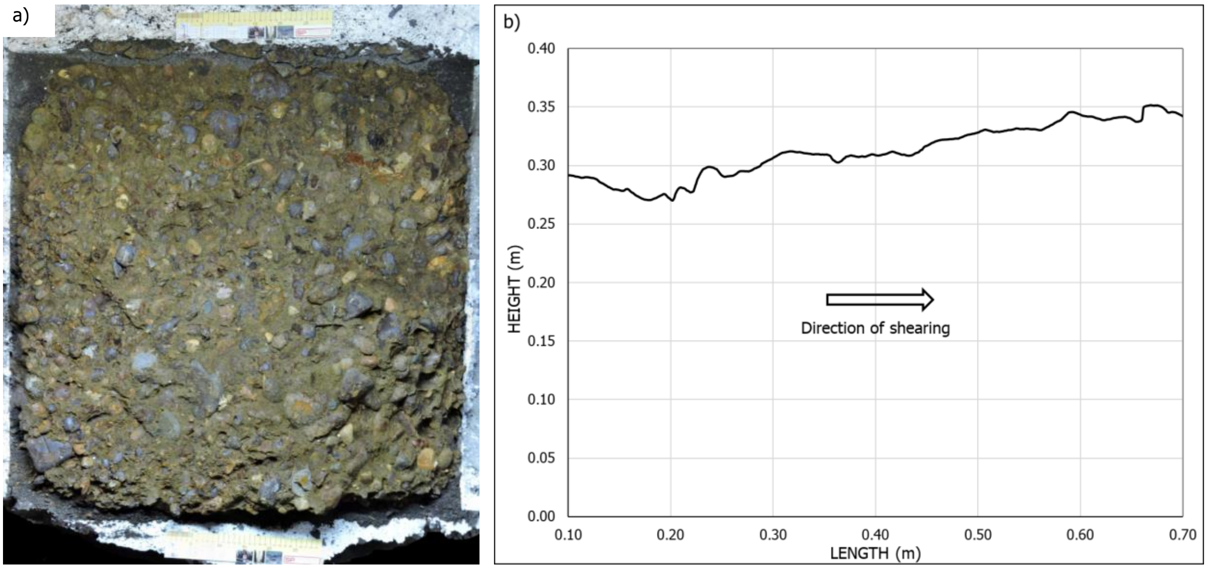

Figure 6. In-situ large scale shear test; a) typical photograph of a tested block (block 02B) and b) corresponding failure surface profile determined with the Barton's profilometer.

After the execution of the tests on Conglomerate, 6 GE-RCC (Grout enriched rolled compacted concrete) blocks have been poured and vibrated at the same location to investigate the shear strength of the contact GE-RCC/Rock. Two different RCC mixes have been tested with different content of cement (190 and 155 kg/m^3).

The GE-RCC/Rock shear tests have been carried out about 60 days after the casting of the GE-RCC blocks, applying the same procedure used for Conglomerate blocks.

3.1.2. Test results

Test results have been elaborated according to the procedure reported in [6] and [7]. Shear τ and normal σ_n stresses are computed as follows:

$$\tau = \frac{P_s}{A} = \frac{P_{sa} \cdot cos\alpha}{A} \tag{1}$$

$$\sigma_n = \frac{P_n}{A} = \frac{P_{na} + P_{sa} \cdot sin\alpha}{A} \tag{2}$$

Where:

P_s	shear force
P_n	normal force
P_{sa}	applied shear force
P_n	applied normal force
α	inclination of the applied shear force
A	area of the shear surface

The linear interpolation of the results of Conglomerate shear block tests (Figure 7) in the range $\sigma_n = 0.3 \div 1.0$ MPa provides the peak and residual strength parameters reported in Table 2. It is highlighted that the results of 2 tests have been discarded from the determination of the peak strength parameters due to the following reasons:

- Irregularities occurred during test execution (tests were interrupted and restarted).
- Odd shape of the failure surface showing a marked downward profile in the direction of shearing.

- Anomalous shear force - normal displacement curve, showing contractant behaviour instead of dilatancy.

Table 2. Peak and residual shear strength parameters derived from in-situ shear testing.

	Peak	*Residual*
c (kPa)	140	0
ϕ (°)	42	35

Regarding the GE-RCC blocks, test results reported in Figure 8 show that the cohesion ranges between 110 and 430 kPa, with the highest values observed for the mix with higher cement content. The friction angle appears less dependent on the cement content and varies between 42° and 43.5°, close to that of the Conglomerate blocks. As already mentioned, these values have been measured 60 days after GE-RCC casting. According to [9], an increase of the GE-RCC/Conglomerate contact shear-strength should be expected over time.

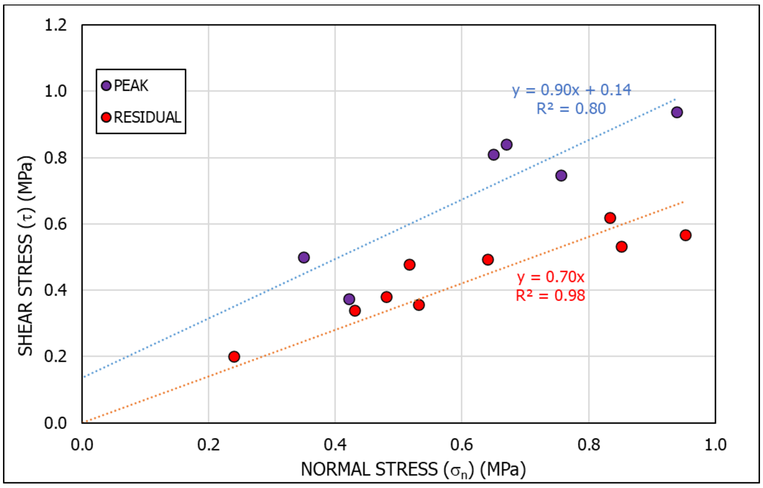

Figure 7. Conglomerate, shear strength vs normal stress results with linear interpolation of the peak and residual values.

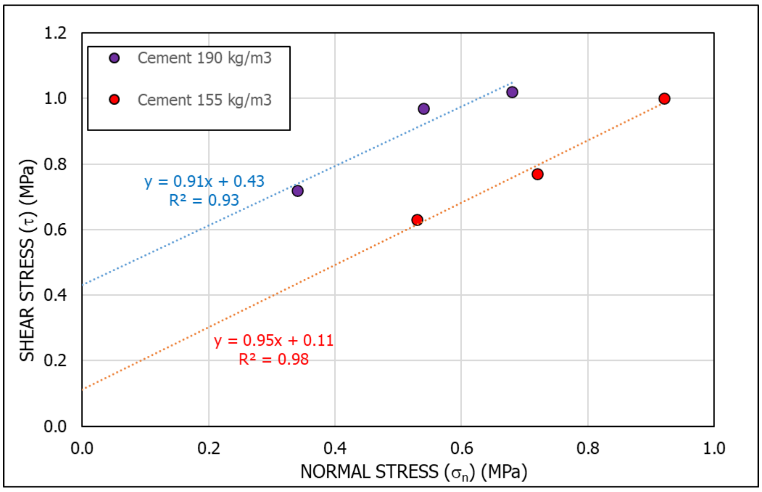

Figure 8. GE-RCC, shear strength vs normal stress values with linear interpolation of test results at different cement contents.

3.1.3. Comparison with literature data

A comparison of test results and literature data has been done to confirm the coherence of the measured shear strength parameters. The Conglomerate peak cohesion and friction angle determined via in-situ testing are in good agreement with the empirical methodology proposed by [10] for the determination of the strength parameters of bimrocks based on the volumetric block proportion (*VBP*) and the strength parameters of the matrix (Figure 9).

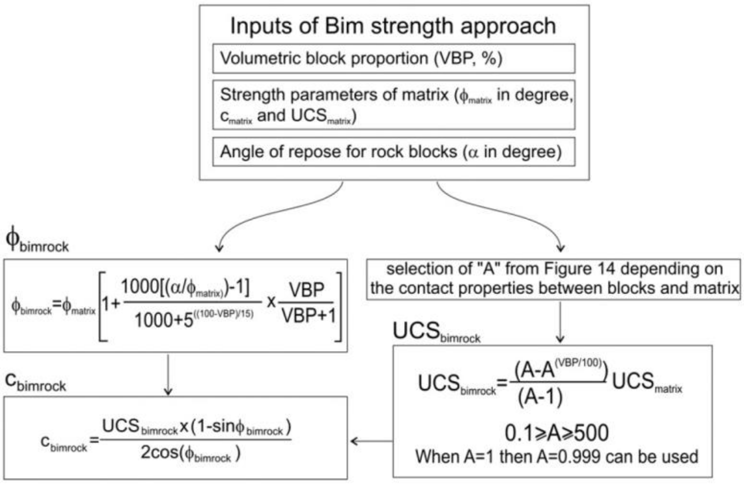

Figure 9. Flow diagram for the prediction of the strength parameters of bimrocks, after [10].

According to [1] and [11], the dimension of the smallest significant block in a bimrock depends on the scale of the problem via the relation $L_b = 0.05\ L_c$ (L_b = dimension of the smallest block, L_c = characteristic dimension). Considering the dimensions of the shear-test Conglomerate samples, this corresponds indicatively to blocks of 30÷50 mm. Therefore, based on the granulometric distributions reported in Figure 3, one can assume an average *VBP* of about 30% ÷ 40%.

In the selected range of *VBP*, the predicted shear strength parameters determined for $\alpha = 25°$ (angle of repose of blocks) and A = 3.0 (model parameter according to [10]) are $\phi_{bimrock} = 37°÷43°$ and $c_{bimrock}$ = 110 kPa ÷ 140 kPa, which are coherent with the in-situ test results. If a smaller *VBP* is considered, a reduction of the predicted friction angle and an increase of the cohesion are observed with respect to the predicted values (Figures 10a and 10b).

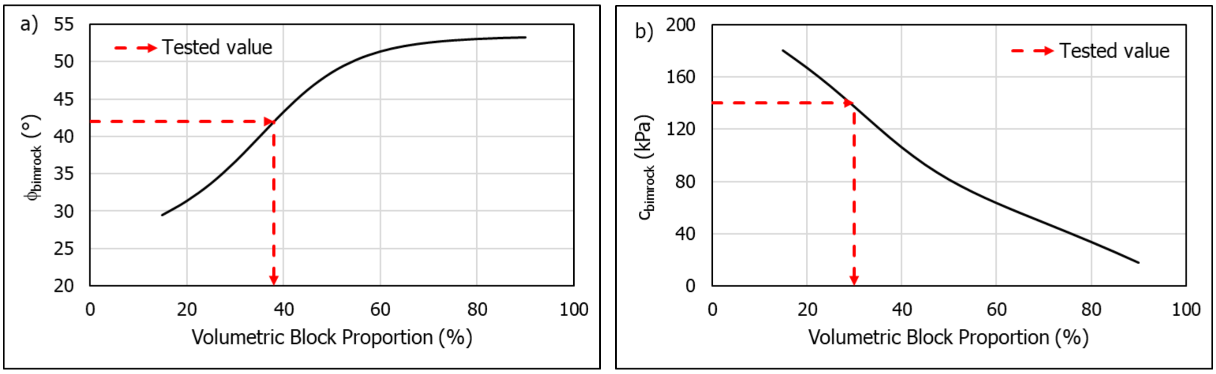

Figure 10. a) Variation of the predicted friction angle as a function of *VBP*; b) variation of the predicted cohesion. After [10].

4. Conclusions

The geotechnical characterization of the Conglomerate formation at Koysha Dam site and the determination of the strength parameters to be used for the design resulted in a very challenging task, due to the heterogeneous nature of the material and the practical impossibility of collecting undisturbed samples. From a geotechnical point of view, the conglomerate can be defined as a bimrock composed of hard blocks in a weaker matrix. In this study the determination of the strength parameters of the Conglomerate by means of in-situ large scale direct shear tests has been presented. The tests have been performed by means of a full-scale testing apparatus capable of testing Conglomerate blocks of dimensions 800 × 800 × 400 mm. Interpretation of the test results identifies a peak friction angle and a cohesion of the conglomerate of 42° and 140 kPa, respectively. These values are in good agreement with the prediction model proposed by [10] to estimate the overall strength of the bimrock for a Volumetric Block Proportion (*VBP*) ranging between 30% and 40%, corresponding to blocks larger than 40÷50 mm according to the sieve analyses.

References

[1] Medley E 1994. The engineering charcaterization of melanges and similar block_in_matrix rocks (bimrocks). *Dept of Civil Engineering, University of California, Berkley, California, PhD diss*

[2] Medley E and Lindquist E S 1995. The engineering significance of the scale-independence of some Franciscan melanges in California, USA. *Proceeding of the 35th US Rock Mechanics Symposium, Reno, 907-914, Rotterdam Balkema*

[3] Medley E 2008. Engineering the Geological Chaos of Franciscan and Other Bimrocks. *Proceedings of ARMA 2008 - American Rock Mechanics Association, 42nd Rock Mechanics Symposium, San Francisco, June 28-July 5 2008.*

[4] Napoli M L, Barbero M, Scavia C 2019. Slope stability in heterogeneous rock masses with a block-in-matrix fabric. *14th International Congress on Rock Mechanics and Rock Engineering, ISRM 2019; Foz do Iguaçu; Brazil; 13-18 September 2019*

[5] Napoli M L, Barbero M, Scavia C 2021. Tunneling in heterogeneous rock masses with a block-in-matrix fabric. *International Journal of Rock Mechanics and Mining Sciences, Vol. 138, February 2021*

[6] ISRM 1974. Suggested Method for in situ determination of Direct Shear Strength. *The Complete ISRM Suggested Method for Rock Characterization, testing and Monitoring*

[7] ASTM D4554-02 2002. Standard Test Method for In Situ Determination of Direct Shear Strength of Rock Discontinuities. *ASTM International, West Conshohocken, PA*

[8] Coli N, Berry P, Boldini D, Bruno R 2009. In situ large size non-conventional shear tests for mechanical characterization of a BIMROCK in the Santa Barbara open pit mine (Italy). *43rd US Rock Mechanics Symposium – Ashville*

[9] Nawy G E 2008. Concrete Construction Handbook, *Taylor and Francis*

[10] Kalender A, Sonmez H, Medley E, Tunusluoglu C, Kasapoglu K E 2014. An approach to predicting the overall strengths of unwelded bimrocks and bimsoils. *Eng. Geology 183, 65-79*

[11] Tsiambaos G 2010. Engineering Geological Behaviour of Heterogeneous and Chaotic Rock Masses. *Bullettin of the Geological Society of Greece, Vol. 43.*