Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

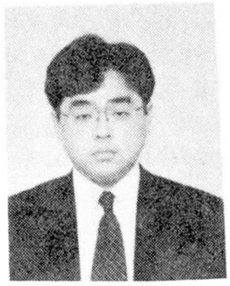
Band: 80 (1999)

Artikel: Evaluation of cracked soft rock on in-situ test results

Autor: Shimamine, Tetsuo / Watanabe, Akiyuki / Sonoda, Kosei

DOI: https://doi.org/10.5169/seals-60738

Evaluation of Cracked Soft Rock on In-Situ Test Results



Tetsuo Shimamine
Civil Engineer
Deputy Manager of
construction supervision sect
East Japan Railway Company
Joshin-etsu Construction
office Takasaki, Japan

Akiyuki Watanabe
Civil Engineer
Assistant Manager of
construction supervision sect
East Japan Railway Company
Joshin-etsu Construction
office Takasaki, Japan

Kosei Sonoda
Civil Engineer
Assistant Manager of
construction supervision sect
East Japan Railway Company
Joshin-etsu Construction
office Takasaki, Japan

Summary

The base rock expected to be used for the foundation of a railway arch bridge(180m span) currently in the planning stage is heavily-cracked auto-brecciated andesite (rock grade about C_M - C_L~Rock classification of dam foundation in Japan). There was a need to accurately evaluate the bearing capacity, since a load amounting to as much as about 2.1kN (normal condition) would be applied to the base rock at each bridge pier. Since being an arch bridge, there was a desire to restrain support point displacement of the foundation to the utmost, and since it was also necessary to take into consideration the reduction in bearing capacity due to the effect of slopes due to the fact that the foundation would be constructed on slopes facing the river. We thus conducted block shear tests as a way of improving the accuracy of the shear constant. We evaluated bearing capacity of the base rock and shear constants (c, ϕ), analyzed the test results as well as the results of prior tests on the same type of base rock and determined the material values to be used in design. And we explained various methods for evaluating the bearing capacity, shear constants of cracked soft rock.

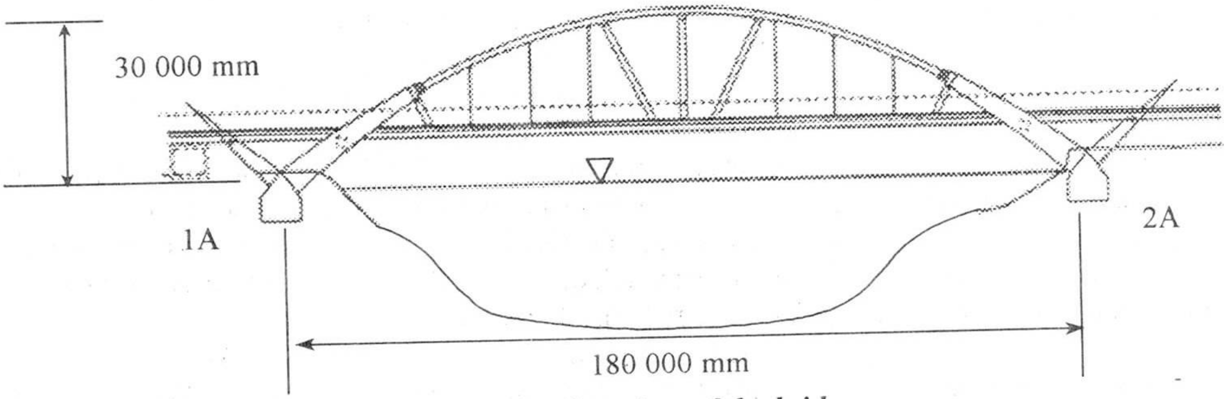


Fig.1 Outline view of the bridge

1. Estimations of the bedrock conditions based on geological surveys

We will next give an explanation of the strata composition in the vicinity of the foundation. Side 2A is characterized by surface deposits (fill, old surface soil) with an N-value of about 5 to a depth of GL-4m, and terraced deposits (sand and gravel mixed with cobblestone) at a depth of GL-7-9m with soft stone tuff-breccia with heavily-cracked auto-brecciated andesite intrusions) at depths greater than GL-9m (refer to Fig.1 and Table 1).

Table 1 Results of geological surveys of the bedrock (2A)

Type of rock		Andesite
Wet density γ_t (g/cm³)		2.20
Water absorption Q (%)		12.5
Triaxial compression test (CU)	cohesion strength c' (N/cm²)	1.00
	Angle of internal friction(°)	43.3
Unconfined compression strength (N/cm²)		5.62
Ultrasonic velocity V_{p0} (km/s)		2.54
Elastic wave velocity V_p (km/s)		1.80

The bedrock of side 2A is rock grade C_M-C_L and has R.Q.D (Rock Quality Designation; percentage of the total hole length made up of cores with a length of 10cm or more acquired by boring. And we must estimate immediately after retrieval) of 10-25.

It is moreover classified as "extremely bad" on Deere's bedrock quality scale (refer to Table 2). In addition, the number of cracks per unit volume 'Jv' estimated from the RQD is 27-32. The rock is therefore fragile in quality and is conjectured to be composed of small blocks.

Table 2 Bedrock quality table according to RQD (Deere)

RQD indication	explanatory expression
0 - 25	extremely bad
25 – 50	bad
50 – 75	generally good
75 – 90	good
90 - 100	extremely good

2. Evaluation of various evaluation methods for bedrock bearing capacity and shear constant

2. 1. Method using past examples of design based on rock type

According to data from nationwide surveys of material values used in the design of long bridges with soft rock as the bedrock, it is conjectured that cohesion strength c and angle of internal friction ϕ of the andesite at side 2A, which would be indicated with in the range in Table 3. These should probably be used strictly as reference values, however, since there are no classifications of the conditions of cracking, rock grade or other elements.

Table 3 Examples of c and ϕ employed in design by rock quality

Classification	Type of rock	c (N/cm²)	ϕ(°)
Pyroclastic material	andesite	0.5	40

2.2. Method according to the Japan Roadway Public Corporation ; Method using the cracking index

There is a method that uses the cracking index that quantitatively indicates the effect of bedrock cracking on strength constants. The cracking index is shown in formula (1).

$$Cr = 1 - [V_p/V_{p0}]^2 \quad \ldots\ldots(1)$$

Where Cr: Cracking index
V_p: Natural ground horizontal elastic wave velocity
V_{p0}: Specimen ultrasonic wave velocity

In addition, Figs. 2 & 3 indicate data compiled by the Japan Roadway Public Corporation relating to the relationship between the cracking index and the reduction index of cohesion strength and the angle of internal friction. This data included the test data of the Railway Technical Research Institute of the National Railway. It is possible to estimate the values of c and ϕ by means of a curve that takes the lower limits of variation indicated in Fig. 2 & 3. In other words:

$$c = k_c \times c_0 \quad \ldots\ldots(2) \qquad \phi = k_\phi \times \phi_0 \quad \ldots\ldots(3)$$

where k_c: reduction index of cohesion strength; k_ϕ: reduction index of the angle of internal friction; c_0: specimen cohesion strength; ϕ_0: specimen angle of internal friction

Material values for the bedrock of side 2A determined from the above are shown in Table 4. According to this method, it is necessary to carry out a reduction of about 80% for cohesion strength and about 40% for the angle of internal friction relative to the results of triaxial compression tests. However, it is possible that shear constants c and ϕ are underestimated with this method, since there are various reasons, such as the fact that basically the minimum values are used for the reduction index and the impossibility of differentiating quantitatively between open and closed cracks because of the use of elastic waves.

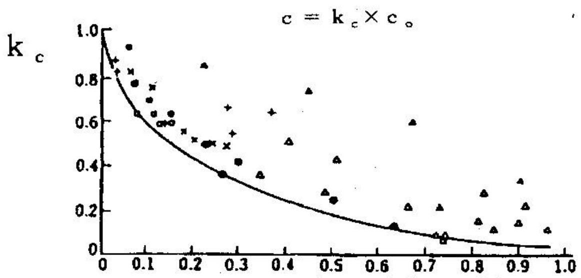


Fig.2 Relationship of Cr and k_c

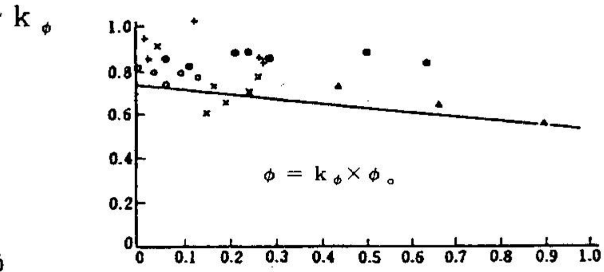


Fig. 3 Relationship of Cr and k_ϕ

Table 4 Shear Constant estimated by using the cracking index

Cohesive strength c			
Triaxial compression test c_0	Cracking coefficient c_r	Correction factor k_c	Corrected value c
1.0 N/cm²	0.50	0.2	0.2 N/cm²
Angle of internal friction ϕ			
Triaxial compression test ϕ_0	Cracking coefficient c_r	Correction factor k_ϕ	Corrected value ϕ
43.3°	0.50	0.6	26°

2.3. Method of the Honshu-Shikoku Connecting Bridge Public Corporation

In addition to rock classification and triaxial compression tests, the Honshu-Shikoku Connecting Bridge Public Corporation also conducted numerous other tests. These were including RQD, unconfined compression strength and longitudinal elastic wave velocity tests as well as in-situ plate bearing tests, boring hole horizontal loading tests and block (or rock) shear tests and determined the various correlations between them. As a method for estimating shear constants c and ϕ, Guidelines for the Determination of Weathered Granite Bearing Characteristics (draft) were set up. Thereby, we were able to estimation at three levels of precision using (1) actual measurements of c and ϕ as the result of block (or rock) shear tests in in-situ or similar base rock as the main estimates, (2) estimates consisting of measurements in boring holes and also making use of correlations to prior tests and (3) estimations carrying out rock classification by macroscopic observation and using the results of correlations to prior tests.

It was thought that using a method such as this would make it possible to grasp the material values of the base rock with a considerable degree of precision if various surveys and tests of similar base rock were conducted and correlations were determined. From the standpoint of economy, however, it would be difficult to conduct the same surveys and tests on ordinary bridges as on a massive structure such as the Honshu-Shikoku Connecting Bridge. In addition, since the guidelines are limited to weathered granite, they could not be applied intact as a method for the evaluation of the base rock in the present case.

3. Evaluation of shear constant by block shear tests

The bedrock of side 2A is heavily-cracked intrusive rock and, in terms of quality, was judged to be "extremely bad." We therefore decided to focus first on the bedrock and conduct block shear tests in order to promote an improvement in the precision of the shear constant.

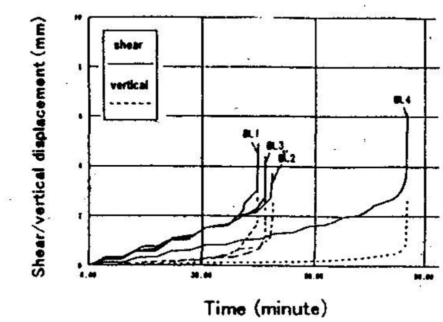


Fig. 7 Relationship between time and shear/vertical displacement (average)

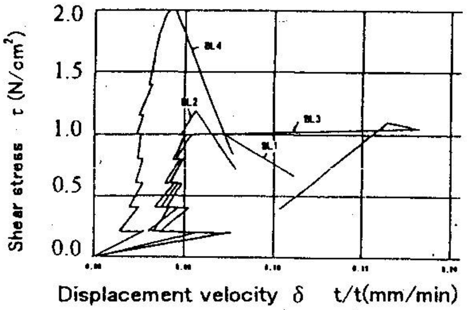


Fig. 8 Relationship between shear stress and displacement velocity

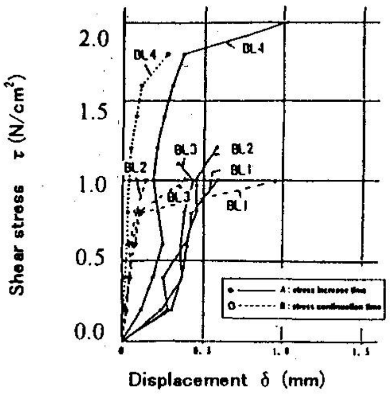


Fig. 9 Relationship between shear stress and the amount of shear displacement

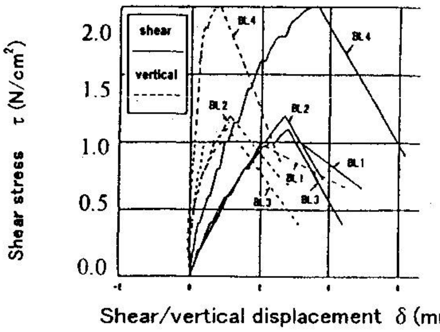


Fig. 10 Relationship between shear stress and shear/vertical displacement (average)

Table 5 Shear strength by base rock origin

(τ_o : N/cm², ϕ : ° f : tan ϕ)

Rock type by origin		C_H			C_M			C_L		
		τ_o	ϕ	f	τ_o	ϕ	f	τ_o	ϕ	f
Paleozoic and Mesozoic deposits	Average	2.0	50	1.2	1.9	45	1.0	0.7	45	1.0
	ceiling	3.1	57	1.5	2.9	53	1.3	1.3	51	1.2
	lower limit	1.2	44	1.0	1.0	39	0.8	0.5	33	0.7
Regional metamorphic rock	Average	2.4	50	1.2	1.4	45	1.0	0.6	45	1.0
	ceiling	3.9	55	1.4	2.5	53	1.3	0.8	49	1.2
	lower limit	1.1	50	1.2	0.8	41	0.9	0.4	40	0.8
Igneous rock, abyssal rock	Average	4.4	51	1.2	3.0	47	1.1	2.2	45	1.0
	Ceiling	8.2	52	1.3	6.8	50	1.2	4.0	45	1.0
	lower limit	2.0	50	1.2	1.4	45	1.0	0.8	45	1,0
igneous rock volcanic rock	Average	2.7	47	1.1	1.9	45	1.0	0.7	40	0.8
	ceiling	3.5	53	1.3	2.6	50	1.2	1.1	44	1.0
	lower limit	1.5	45	1.0	1.3	44	1.0	0.3	35	0.7
volcanic eposits (Tertiary and later)	Average	3.5	51	1.2	2.9	51	1.2	–	–	–
	ceiling	4.8	55	1.5	2.8	55	1.4	–	–	–
	lower limit	2.2	50	1.2	1.1	50	1.2			

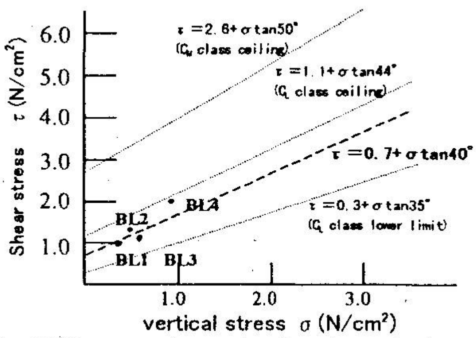


Fig. 11 Test results and volcanic rock shear strength properties

The following is a summary of the results of the tests that we conducted.

(1) Base rock conditions of the test surface and failure surface

Observation of the conditions of the shear plane indicated that the failure of blocks BL1-3 was deeper than in block BL4 and there was also a tendency in all of the blocks to form a slope inclining generally toward the front. In regard to the shear plane conditions, blocks BL1-3 sheared in the direction of the joint of the breccia contained in the auto-brecciated andesite. However, block BL4 sheared with a number of cracks appearing in the base rock itself in the direction of the shear and in the perpendicular direction as shear loading was applied.

(2) Shear loading strength properties

* Relationship between time and shear/vertical displacement (average)

Failure took 2-3 times longer with BL4 than with BL1-3.

* Relationship between shear stress and displacement velocity

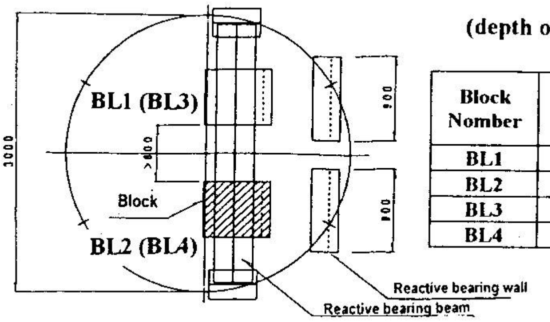
The displacement velocity of BL4 was about 1/2 that of BL1-3 and the progress of the displacement was slow.

* Relationship between shear stress and amount of shear displacement (average.)

3.1. Test apparatus and method

Deep foundation holes with a diameter of 3m were excavated in the vicinity of the foundation on side 2A and the tests were conducted using 2 blocks each at a depth of 8.5m (block numbers BL 1 & 2) and a depth of 10.0m (block numbers BL 3 & 4) from G.L. A plane view of the test conditions is shown in Fig.4 and an inclined view is shown in Fig.5.

The shear plane was 60cm × 60cm in size and, since load strength was conjectured to be in the range of 0.4N/cm^2 under normal loading conditions at the bottom of the foundation, vertical loading was carried out in the stages of 0.1, 0.2, 0.3 and 0.5N/cm^2. The loading velocity was set at 2.5×10^{-2} N/cm^2/min with a stationary period of 5 minutes at each stage. In addition, horizontal shear loading was carried out at a velocity of 5.0×10^{-2} N/cm^2/min with a stationary period of 5 minutes after each 0.2N/cm^2 loading and the load was increased until shear failure occurred. The blocks were set up so that they would shear in the direction of the river.



(depth of 10.0m)

Block Nomber	Vertical loading (N/cm^2)
BL1	0.1
BL2	0.2
BL3	0.3
BL4	0.5

Fig.4 Plane view of the conditions of block shear tests **[depth of 8.5m]**

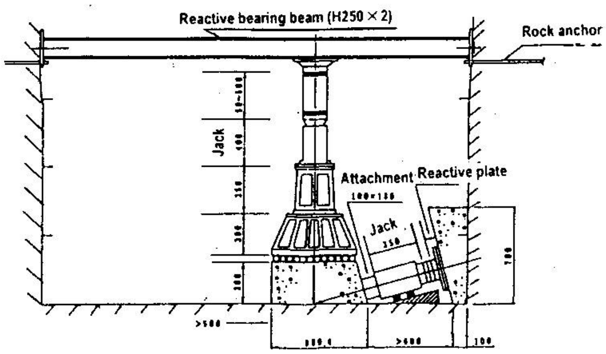


Fig.5 Side view of the conditions of bock shear tests

3.2. Results

The base rock at the bottom of the deep foundation holes was not uniform and there were portions of hard material here and there. It was necessary to carry out the shear plane of each block with the same quality and grade of rock and conduct the tests in base rock that was typical of the rock quality in the deep foundation holes. Therefore, we decided to conduct the tests with C_L grade base rock.

The conditions of failure of the shear plane of the blocks are indicated in Fig.6. The relationship between time and shear/vertical displacement (average) is indicated in Fig.7. The relationship between shear stress and displacement velocity is indicated in Fig.8. The relationship between shear stress and the amount of shear displacement is indicated in Fig.9. And the relationship between shear stress and shear/vertical displacement (average) is indicated in Fig.10.

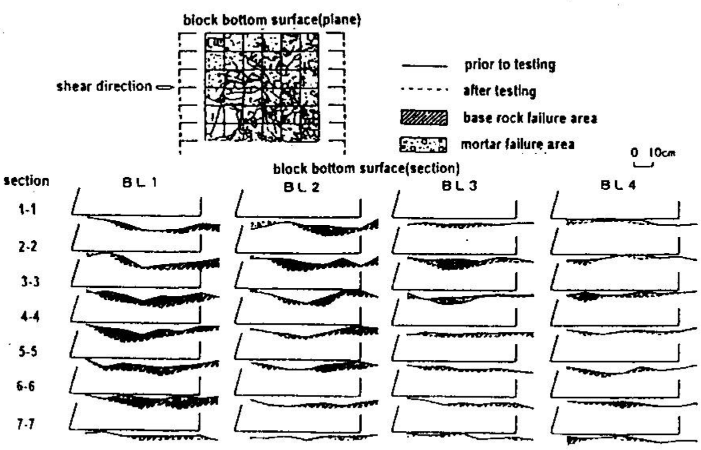


Fig.6 Conditions of failure of the shear plane of the block

The amount of shear displacement was about 1/2 that of BL1-3 for both stress increase time and continuation time. This would indicate that the base rock of BL4 is hard and that fractures are closed.

The above points out differences between BL4 and BL1-3 in dynamic properties and we feel that it is necessary to treat them differently in the compilation of test results. n addition, based on our observations of the base rock, we conjectured that cracks in the base rock were closed and that there was also little polarity in the cracking.

3.3. Material evaluation

Table 5 shows values for general shear strength by type of rock obtained from in-situ shear tests conducted at a dam site. The andesite in these tests is classified as volcanic or igneous rock and we propose that the shear strength t_0(= c) of rock grade C_L is 0.3-1.1N/cm^2 and that the angle of internal friction is 35-45°.

In addition, if we plot the values obtained in the tests in the shear strength properties of the volcanic rock shown in Fig.11, we could see that the test values for BL1-3 would be average values for C_L grade rock and that the test values of BL4 would be at the upper limit for C_L grade rock. Based on this, we can state that appropriate values were obtained in the tests corresponding to the rock type and grade.

If we therefore consider the data for BL1-3 to be typical of the shear strength of the andesite in the tests and use the average angle of internal friction (ϕ = 40°) for the igneous and volcanic rock of Table 5, it would be possible to formulate the following relational expression:

$\tau = 0.7 + \sigma \tan 40°$ Where, τ: shear strength (N/cm^2), σ: normal stress (N/cm^2)

However, taking into consideration the risk of creep deformation due to sustained loading, residual displacement in the wake of massive earthquakes and other deterioration as well as the scale effect, cohesive strength c was reduced to about 1/3 as an engineering judgment.

Based on these, we decided to use the following as the shear constant for the bedrock on side 2A in the design of this bridge:

Cohesive strength: $c = 0.2$N/cm^2 Angle of internal friction: ϕ = 40°

4. Conclusion

It is thought necessary to take the following into account when applying these to base rock material evaluation and design.

(1) Although there is a variety of methods for evaluating base rock bearing capacity and shear constant, there are no established methods. Therefore, instead of adhering to one single method, it is necessary to conduct wide-ranging examinations of rock type, rock grade, cracking direction, converted N-value, design examples and other elements.

(2) When using the method for evaluating the shear constant of base rock by means of the cracking index, shear constants c and ϕ may possibly be underestimated.

(3) In-situ tests are effective in the evaluation of base rock shear constant and, especially in the case of base rock with little prior experience, block (or rock) shear tests, which can grasp the shear constant to a certain degree by mass, are effective. However, since costs will increase as more tests are conducted, it is desirable to conduct evaluations while referring to shear strength by base rock origin for which data from prior in-situ shear tests is available.

Postscript

The above is an account of our implementation of block shear tests to an unusual base rock as a bridge foundation and the application of the results to design. There are no established methods for the evaluation of base rock bearing capacity and we feel that it will be necessary hereafter to accumulate in-situ test data and conduct research into bearing capacity theory with the addition of the influence of cracking.