EVALUATION OF MANY LOAD TESTS OF PASSIVE ROCK BOLTS IN THE CZECH REPUBLIC

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Abstract

Within the research project "FR-TI4/329 Research and development - creating an application system for the design and analysis of soil and rock anchors including the development of monitoring elements", an extensive stage of field load tests of rock bolts was carried out. The tests were conducted at 14 locations with varied rock composition. Before the initial tests, a loading stand was designed and constructed. A total of 201 pieces of tensile tests of bolts having lengths from 0.5 up to 2.5 m, a diameter of 22-32 mm, were performed. These were fully threaded rods, self-drilling rods, and fiberglass rods. The bolts were clamped into the cement and resin. The loading tests were always performed until material failure of bolts or shear stress failure at the interface cement-rock. At each location, basic geotechnical survey was carried out in the form of core drilling in a length of 3.0 metres with the assessment of the rock mass in situ, and laboratory testing of rock mechanics. Upon the completion of testing protocols, rock mass properties analysis was performed focusing on the evaluation of shear friction at the grouting-rock interface.

Key words: loading test, rock bolt, drilling, rock mass, friction, failure, core drilling

1 INTRODUCTION

As part of the research project – rock anchors, bolts specifically, the application focuses on one of the design parameters for the assessment of the elements – friction between the rock bolt and rock mass. The function of the steel elements (bolts) is clear – the stabilization of the geological environment in terms of preventing movements and the imbalance of forces. The essence of such stabilization consists in both reinforcing steel elements that take the tension and part of shear stress, partly constricting (squeezing) the rock pretension bolts anchored to mobilize much needed friction on the surfaces of existing discontinuities. Changes to the stability of the rock mass are caused either by stress changes due to construction activities (underground structures, cut, lop off a load structures) or geological factors. The driving force is primarily gravity loads, mainly the self-weight of the rock.

1.1 Monitored parameters

The design and evaluation of rock bolts require the determination of several parameters. The main one is the resistance against pulling the root portion of the elements. If the root portion is along the entire length of the bolt, it is mainly the shear strength in the plane intersected the discontinuity, and after a certain deformation, we can speak of a tensile effect of the bolt [4, 5, 6].

Own resistance against the loading test, or if you want, the ability to take over the rock anchoring force is given by the rock strength in shear or tension, as well as the length and diameter of the root anchorage. The tensile strength is particularly used for healthy little disturbed rocks, while the shear strength for rocks with a greater frequency of discontinuities [1]. This is related to the evaluation of quality of rock – *RQD* index. This was used as the direction-dependent parameter for the evaluation of the total of 10 locations of 12. Its determination is amply described in widely available literature [11]. Its primary advantage consists in the independence of rock type, which was used within the research project.

The size of skin friction at the interface 1 steel-root and 2 root-rock is another parameter important for the design and assessment. It is clear that not every violation occurs in contact with the rock. It is therefore important to assess both states and based on them adapt the dimensions of anchoring. Within the research, the whole interpretation of tensile tests focused on the area 2 root-rock namely because 100% of failures were observed just here and also because skin friction values, proven for decades, can be considered for the area 1 [12].

1.2 Loading stand

For tensile tests the ENERPAC 600 hydraulic system was used (Fig.1), which had been calibrated by the Czech Metrological Institute. The hydraulic system consists of a hollow cylinder RCH 603, hydraulic pump, pressure gauge indicating the load, thermoplastic hydraulic hose, precision calliper, and a magnetic stand. The device is capable of exerting maximum tensile force of 600 kN.

Prior to the tensile test, a bolt preloaded on the strength of 0.5 kN was tested. The zero reading (displacement) was then deducted. Further, loading was carried out according to loading levels (total 5-7 degrees) to achieve maximum strength or failure of steel – activation of skin friction between grout and rock, in parallel with using the monitoring gauge to find out whether there is a pressure drop – tensile strength or the development of deformation of the bolt. The time delay between each load stage was twenty minutes or until the stabilization of shifts. The shift values in individual load levels were subtracted from the digital slide gauge reading. The results were recorded in the test protocols for further processing.

The custom loading stand (Fig.1) was constructed in the shape of tripod. The arms are made of rolled profiles of type HEA. These are connected to the upper and lower gusset plates and each of them is an element used for increasing the stiffness associated with high-strength and welded to the central tube. At the ends, the arms are fitted with the threaded rods as a locking device relative to the axis of the bolt. The weight of the device is put over the temporary upper anchorage decoupling. The reason why this device was designed consists in the transfer of compressive forces into solid rock away from the borehole and to a minimum distance of 15 times of the diameter of the bolt. In this way, the core to clamp around the bolt is not affected.

Fig. 1 Loading stand with hydraulic system

1.3 Laboratory of rock mechanics

After determining the index *RQD*, from the sample boxes appropriate fragments of the drill core were excluded, from which regular rock bodies – rollers were cut with a diamond saw. With regard to the necessary amount of samples – 5 pcs in the set, the height of the rollers 75 mm was required for the diameter of 45 mm – i.e. a slenderness ratio of about 1.67. Other samples – irregular fragments of the drill core are selected directly from the sample, according to the needs of the appropriate fragments.

Determining the index *RQD* by Hendron in [11] was performed in a conventional manner – by measuring the length of the core fragments stored in containers and by the calculation according to the standard equation:

$$RQD = \frac{L_{10}}{0{,}01L}\,[\%] \tag{1}$$

where L_{10} is the length of all drill core fragments ≥ 10 cm in a given section of the borehole, *L* is the length of the relevant part of the borehole; in this case always the length of 1 meter is considered.

The determination of bulk density of rock material was carried out by exact measurements of prepared rock samples – cut rolls along with the calculation of volume and weighing. The determination of scleroscope hardness of rock material was carried out using the apparatus Shore - D type (The Shore Instrument & Mfg. Co. NY) adapted methodology and correlation to the tensile uniaxial pressure $\sigma_{c,ss}$ according to [9] - see (Fig. 2).

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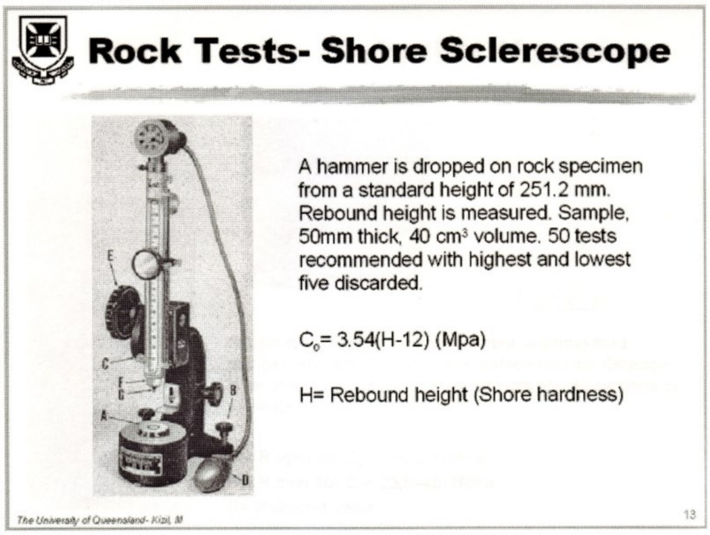

Fig. 2 The test rocks using Shore scleroscope - type D

The determination of scleroscope hardness of rock material was performed using the Schmidt hammer, type L (Proceq) using the standard methodology for regular rock glands (rollers) and irregular samples (fragments of drill core). The regular rock bodies were applied in those cases of an excess of such samples (> 5), irregular in other cases. The routine rock bodies (rollers) were tested in the axial direction of the drill core and perpendicularly thereto, irregular ones only transversely to the axis of the drill core. The correlation to the uniaxial compression strength σ_c was performed by means of the graphic transfer using the Bieniawsky diagram (Fig.3) [2, 3 et all.].

The uniaxial compression strength σ_c was carried out using a standard procedure when the test bodies (rollers) of known parameters were clamped to the clamping jaws of the test press (Controls) and loaded at a rate of about 0.3 MPa/sec until failure. The custom uniaxial compressive strength was then calculated by using the maximum achieved by the loading force and the initial cross-sectional area of the sample.

All tests were carried out in the rock's "instantaneous" state – i.e. in the moisture corresponding to the laboratory environment [7, 8].

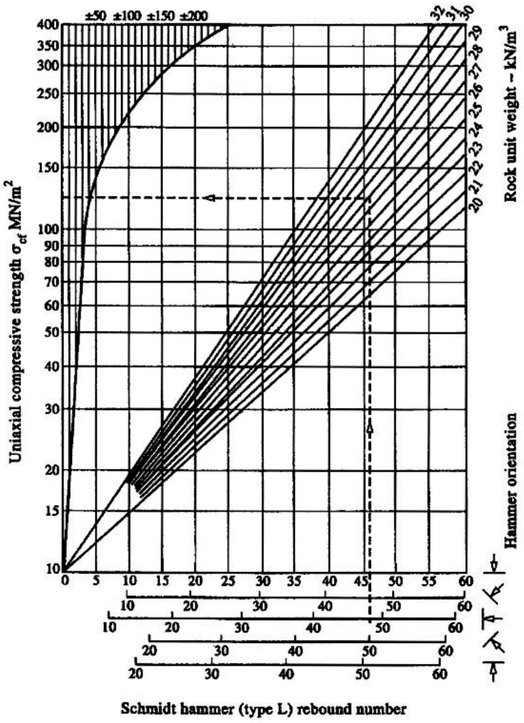

Fig. 3 The test rocks using Schmidt scleroscope - type L

The determination of uniaxial tensile strength σ_t was carried by correlation from the strength in uniaxial compression σ_c, (Fig.4) according to Kim and Lade, or according to Horák [10].

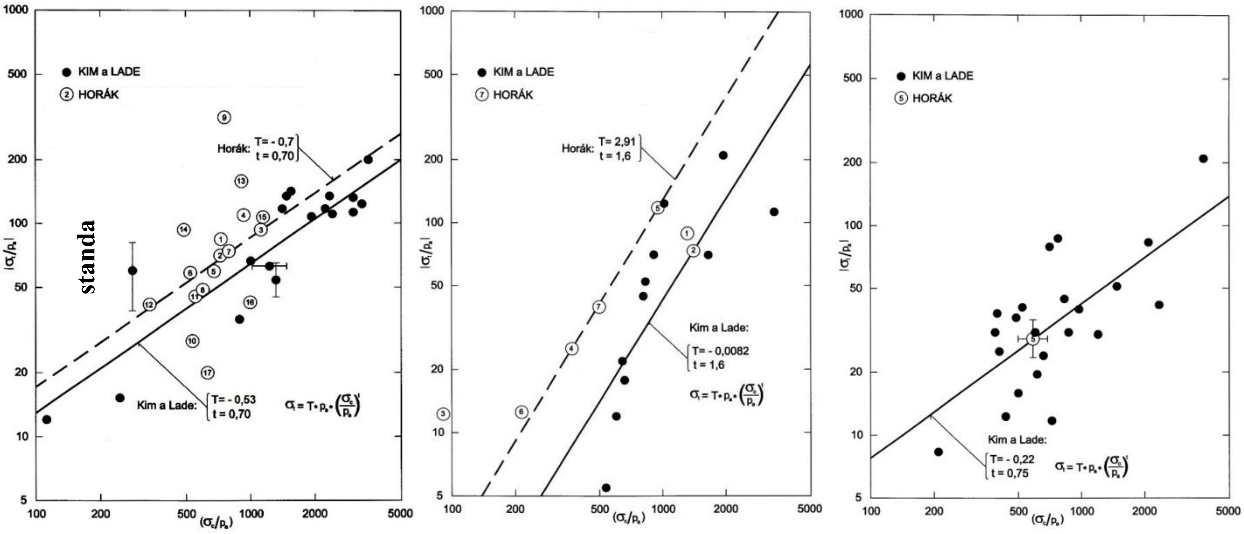

Fig. 4 Diagrams of correlation dependencies between uniaxial tensile strength σt and strength in uniaxial compression σc of Kim Lade and by Horák (from left: igneous – metamorphic–sedimentary rocks)

2 RESULTS OF LABORATORY TESTS

Laboratory tests were arranged in tables (Tab.1 etc.). For further orientation, the locations are identified only by number (Tab.2 etc.).

Tab. 1 Results of the determination of RQD index per common meter drill core

N. of locations	Locations	Petrographic rock type	Index RQD
1	Dolní Kounice	Granodiorite type Tetčice	0÷1 m 22% 1÷2 m 64% 2÷3 m 37%
2	Ústí nad Labem – Mariánská skála	Trachyte	0÷1 m 0 1÷2 m 20% 2÷3 m 99%
3	Velké Opatovice	Sandy marlite	0÷1 m 60% 1÷2 m 81% 2÷3 m 87%
4	Hrob	Two-mica paragneiss	0÷1 m 11% 1÷2 m 0 2÷3 m 21%
5	Čertovy Schody	Micrite limestone	0÷1 m 56% 1÷2 m 75% 2÷3 m 73,5%
6	Dolní Žleb	Quartzy sandstone	0÷1 m 88% 1÷2 m 66% 2÷3 m 65%
7	Vlastějovice	Orthogneiss, scarn	0÷1 m 36% 1÷2 m 24% 2÷3 m 11%
8	Hanušovice	Amphibolite	0÷1 m 36% 1÷2 m 57% 2÷3 m 24%
9	Vilémov	Phyllite until quartzite	0÷1 m 74,5% 1÷2 m 54% 2÷3 m 27,5%
10	Železný Brod	Two-mica phyllite	0÷1 m 0 1÷2 m 0 2÷3 m 10%
11	Vrané nad Vltavou	Tuffite	0÷1 m 58% 1÷2 m 97% 2÷3 m 93%
12	Štěchovice	Slate	0÷1 m 44% 1÷2 m 81% 2÷3 m 80%

Tab. 2 Summary of results

N. of locations	Ø scleroscope hardness Shore	Ø uniaxial strenght σ_c [MPa]	Bulk density ρ [kg.m^{-3}]	Ø scleroscope hardness Schmidt	Correlated uniaxial tensile strenght σ_t (by Horák) [MPa]
1	67,7	74	2 618	84	7,2
2	66,8	65	2 423	86	6,5
3	37,3	55	2 152	52	1,4
4	39,6	29*)	2 519*)	28	1,7*)
5	46,3	51	2 669	55	1,3
6	35,0	31	2 016	32	0,9
7	68,4	66	2 579	96	6,1
8	58,0	62	2 869	121	5,6
9	69,4	50	2 628	48	3,9
10	34,4	6,1**)	2 535**)	29	0,1**)
11	74,6	86	2 627	87	2,0
12	65,2	29	2 690	72	0,9

Note.: *) 2 samples **) 1 sample

As standard 5 series (resp. 4) of regular rock bodies were tested. The samples No. 4 and No. 10 failed to comply with the frequency – the results should therefore be viewed as tentative. The correlated uniaxial tensile strength in metamorphic rocks (sample numbers 4, 7-10), in some cases, compared with the uniaxial compressive strength, appears to be disproportionately high.

3 EVALUATION OF LOADING TEST RESULTS

The research project was carried out using extensive testing – loading tests of different types of rock bolts, having various lengths and diameters, with different grouting (cement, resin) and various types of rocks. A comprehensive overview of the tests are tabulated and expressed in the appendix to the methodology – working diagrams (Fig.5). The diagrams also contain test reports with waveform loading tests, including their evaluation according to relevant sites. Due large amounts of data, it is possible to request the author to provide the data in a separate appendix.

Only 68 of 201 pieces subjected to loading tests were used for further evaluation, i.e. about 34%. The interpretation of the results was not focused on deformation parameters, but tracked the development of progressive loading vs. displacement. From the tests, the limit of occurrence of fully mobilized skin friction was then evaluated, in particular before reaching the yield strength of steel (in the case of fully threaded rods "C" and injectable rods "I") and the breaking of the thread (in the case of composite rods "R" and injectable composite rods "RI").

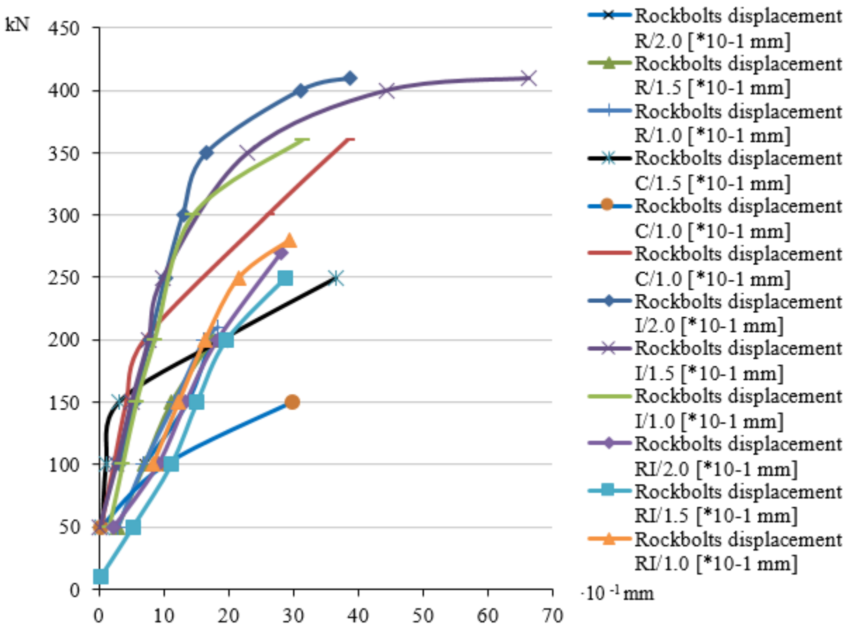

Fig. 5 Example of one of many working diagrams of loading vs. displacement for different rock bolts (type/length)

Based on the interpretation, the data set (Fig.6) was obtained, representing all the variables (type of rock, cement, length and type of bolt) as a relationship of skin friction $R_{\tau b}$ [MPa] and the average value of index RQD_{prum} [-] (weighted average along the embedment length).

For further processing, the data was divided by two variables – the type of cement and the technology of cement, i.e. 4 combinations – each with a different frequency of use = number of locations = number of rocks. The variants "C – s" (cement - sealing), "C – g" (cement - low pressure grouting), "R - g" (resin - low pressure grouting) and "R – s" (resin - sealing) were recognized.

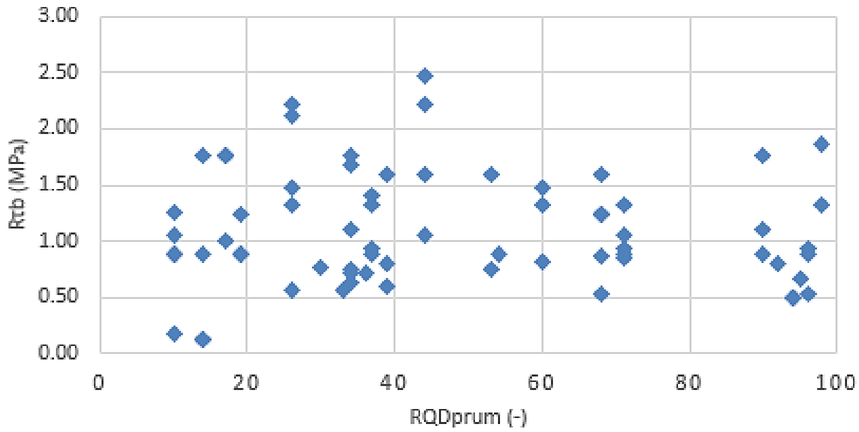

Fig. 6 Data set of competent skin friction and rock quality index without any dependence

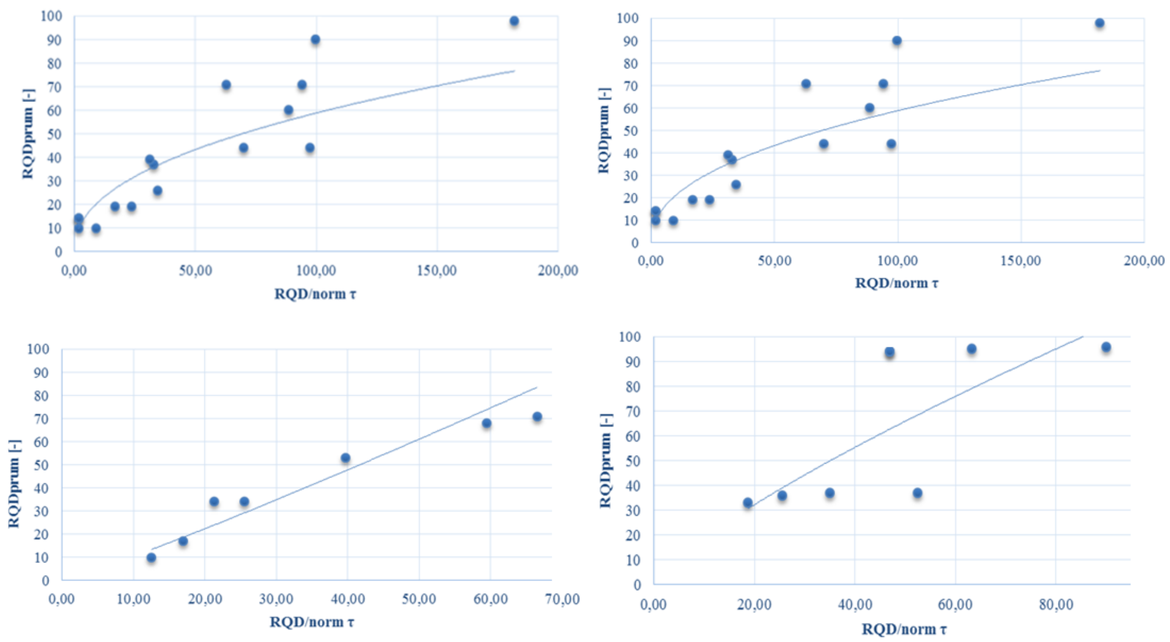

Fig. 7 Data for variants: C-s (left up), C-g (right up), R-g (left down), R-s (right down)

Based on the numerical solution, the final equation was determined, describing the behaviour of skin friction Rτb [MPa] in relation to the average rock quality index:

$$R\tau_b = \tau_{cal} \times k_1 \times RQD_{prum}{}^{k_2 - 1} \tag{2}$$

where the coefficients k_1 [-] and k_2 [-] (Tab.3) reflect the impact of the type of cement and the technology of its implementation; RQD_{prum} [-] describes the quality of the rock along the expected anchor length, and τ_{cal} [MPa], the calibration (the default value) of skin friction, is determined numerically according to the median of data set of the obtained skin friction and index RQD_{prum}.

Tab. 3 Parameters

Variant	τ_{cal} [MPa]	k_1 [-]	k_2 [-]
C-s	*0.98*	*0.35*	*1.32*
C-g	*1.01*	*1.18*	*0.91*
R-g	*2.66*	*0.23*	*1.28*
R-s	*1.01*	*0.01*	*2.26*

Whereas it is the cross-section of varied rock spectrum, it is possible to calibrate the function of the data itself even for a single rock type. The correlation can be saved, because the index *RQD* is an petrographic independent parameter.

4 CONCLUSION

The application methodology, as a result of the research project, is an outcome of the efforts to automate entering and obtaining results, in particular in the form of software applications. This is, in conjunction with graphical and tabular data intended for free download on the project website. The application system itself is not running in the regime proposal/report, but needs further implementation into already established design methods, or geotechnical software. A major goal of this research project consists in its meaningful use in building practice. The application is designed in 5 easy steps – entering values, possible calibration (iterative process), and the receipt of income – the appropriate skin friction. Point 5 is optional. The user manual has been replaced by a direct form of hidden comments to individual items; the numeric input is checked by defined limits. The value of obtained skin friction is thus available for the next design and assessment processes of the fastener, which finds its application in design practice (integration into existing procedures) and serves as a stimulus for further research activities as regard to e.g. the relationship between shear or tensile strength of rock and the optimal setting anchoring length.

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