Arch structure foundations in rock

Fondation d'une construction en arc dans la roche

Gründung der bogenkonstruktion im felsen

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ABSTRACT: The solution used for the design of foundations for a wild animal crossing over a deep trench during the construction of a highway in Croatia is presented. The foundation design of the crossing arch structure had to be changed after the excavation works revealed that the foundation rock, consisting of upper Triassic dolomite, was greatly heterogeneous. Several limit equilibrium and stress-strain analyses were carried out after the rock classification resulted in four rock and soil categories. Three different foundation types, satisfying the requirement of uniform rock deformations, were finally chosen. Horizontal displacements measured at the arch abutment show very good agreement with predicted values.

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1 INTRODUCTION

A crossing for wild animals was constructed over a 20 m deep trench along the Zagreb-Rijeka highway in Croatia. The crossing is half a prefabricated arch structure, 100 m wide, with a span of 30 m. The foundations were constructed in the upper Triassic dolomite.

It was first decided to construct a three-joint prefabricated arch with 1.5 m wide elements, and small foundations adequate for the good quality rock mass. After the cut-off was made, it was determined that there were significant variations in the quality of the rock. It was thus, decided to design a two-joint arch with cross-enforcement to pass over weaker zones. The two prefabricated elements were fixed at the crown.

2 ANALYSIS OF FOUNDATION DESIGN

2.1 *Geological structure of the region*

The rock mass consists of the upper Triassic late diagenetic dolomites of different texture types. Wide clay beds exist in the fault zones. The uniaxial compressive strength of the basic rock is 50 MPa. There are three basic groups of discontinuities. The interbedding has the position of 30-35/20-40°, and the layer thickness varies between 10 and 60 cm. The layer surfaces are deflected and rough, and they often have a limonite coating. The walls of the interlayer cracks, as well as the layering itself, do not show any signs of weathering. The aperture is generally 0.1 - 1.0 mm wide. The weathered layers, which are up to 3 m thick, can only be found in the surface region, where the interlayer cracks have been widened by the carstification process, and filled with clay. The cracks parallel to the cleavage of the axial plane generally have the position of 165-190/70-80°, and the cracks perpendicular to the structural "b" axis the position of 255-270/75-85°.

The geotechnical classification of the rock mass was conducted (Bieniawski 1978, 1979) based on data from geological mapping, according to methods suggested by ISRM (1978). The whole region was divided into four sections with the values of the Geological Strength Index (Hoek, 1994), GSI > 65, GSI = 54, GSI = 44, and GSI < 44 respectively. The shear strength was determined for each section according to the empirical strength criterion (Hoek & Brown 1978, 1988), and the deformability was determined by the measured deformation modulus (Serafim & Pereira 1983). Based on the geotechnical classification, a local categorization of the rock mass was made in order to decide on the foundation type.

Parameters used in geotechnical analyses are presented in Table 1, where RMR is the Bieniawski Rock Mass Rating, m_i, m_{dist} and s_{dist} are Hoek-Brown strength parameters (index *i* denoting the intact rock, and *dist* disturbed rock), σ_c is the uniaxial compressive strength, E is the deformation modulus, and c and φ are Mohr-Coulomb strength parameters.

A rock mass with a value of RMR = 23 is a clay with rock fragments. The Standard Penetration Test in this material gave the average number of blows $N = 50$.

In order to perform the stress-strain analysis, the empirical Hoek & Brown (1978, 1888) rock strength criterion had to be adapted for the use of Mohr-Coulomb strength parameters c and φ. The nonlinear empirical strength criterion was used in the range of mobilized stresses in the rock mass under the foundations to determine the equivalent strength parameters c and φ, as shown in Figure 1 for RMR = 44 and RMR = 23.

Table 1. Strength and deformation parameters used in the analysis for upper Triassic dolomites with different degrees of cracking and weathering

RMR	m_i	m_{dist}	s_{dist}	σ_c MPa	E GPa	c MPa	φ degr
65	7	0.5746	0.002928	50	30.0	0.73	34.4
54	7	0.2619	0.000468	50	8.0	0.51	27.0
44	7	0.1282	0.000088	50	0.2	0.31	22.2
23	7	0.0286	0.000003	50	0.05	0.05	23.5

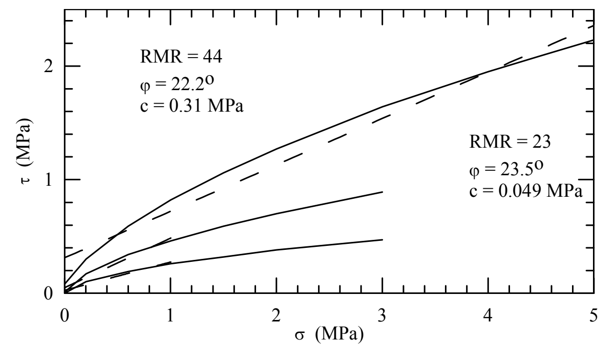

Figure 1. Hoek-Brown strength criterion for RMR = 44 and RMR = 23 and parameters from Table 1 (full line), and equivalent Mohr-Coulomb criterion (dotted line).

2.2 *Bearing capacity of rock*

The allowable load was determined in two ways, by the limit equilibrium analysis, and the stress-strain analysis. Three types of foundations were designed with the purpose to level the deformations of the rock mass.

The limit equilibrium analysis was performed for the assumed model of failure, for which the rock mass first fails under the foundation, and then the failure zone progresses laterally, which results in the radial rupture of the original rock mass around the foundation (Ladany 1972). It is assumed that the rock mass failure will cause the shear strength reduction for one category. The described analysis resulted in a diagram of allowable load dependence on the quality of rock mass, for a rock mass with RMR ranging from 40 to 65 (Fig. 2).

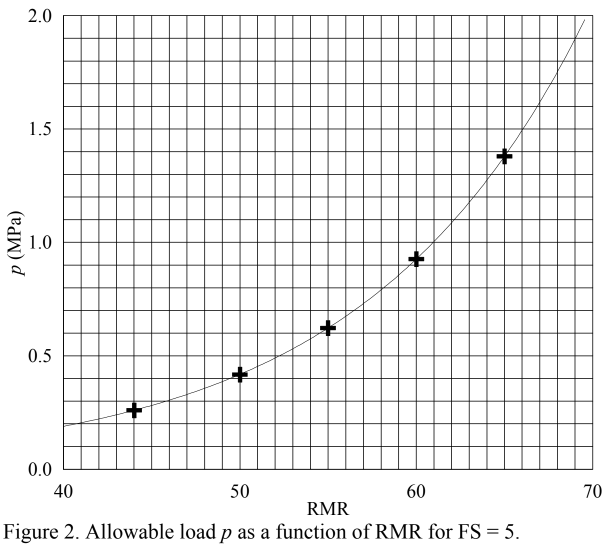

Figure 2. Allowable load p as a function of RMR for FS = 5.

The stress-strain analysis was performed in order to assess the arch foundation displacements, taking into consideration the stiffness and strength of the rock or soil mass for complex boundary conditions encountered in such cases, for which there are no simple expressions that can be used. The conducted stress-strain analysis also takes into account the strength characteristics of the foundation rock, along with elastic characteristics. Thus, the calculation of deformations included the determination of the rock bearing capacity, and it was not necessary to perform a separate bearing capacity analysis, which is otherwise required in classical geotechnical engineering.

It was assumed that the foundation rock is an isotropic material with linear-elastic properties up to the failure, and the Mohr-Coulomb failure criterion was adopted. Plastic deformations develop when the rock strength is reached, which prevents stresses from taking values outside the elastic region limited by the failure criterion. In the adopted model, deviatoric plastic deformations can only develop, so that the model does not exhibit dilatation. The calculations were carried out by the computer program SIGMA/W Ver. 3 developed by Geoslope from Alberta, Canada. Four analyses covering possible combinations of foundation shapes and rock properties were performed.

The static equilibrium calculation of the arch gave the magnitude of the maximum force in the arch abutment (support) with the vertical component $V = 1238$ kN/m′ and the horizontal component $H = 1109$ kN/m′. The four analyses with different foundation shapes and rock properties are as follows:

1. a small foundation block separated from the concrete foundation base (Fig. 3) on rock having RMR = 44;
2. a small foundation block closely connected to the foundation base (Fig. 4) on rock having RMR = 23 (clay with rock fragments);
3. a large foundation block (Fig. 5) on rock having RMR = 23 (clay with rock fragments);
4. a large foundation block on clay having conservatively chosen parameters: $c = 25$ KPa, $\varphi = 20^o$, $E = 0.025$ GPa.

The analyzed region was modeled by 588 quadrilateral finite elements with four nodes. As it is important for non-linear analyses to define a proper initial stress state, the first phase consisted of the determination of self-weight initial stress condition in the rock mass. The highway excavation works were simulated in the second phase. The arch design load was applied in three equal increments in the third calculation phase, in the form of horizontal and vertical concentrated forces at the arch support. The uniform load by earthfill, acting directly on the foundation rock, was also applied in the third phase in three increments.

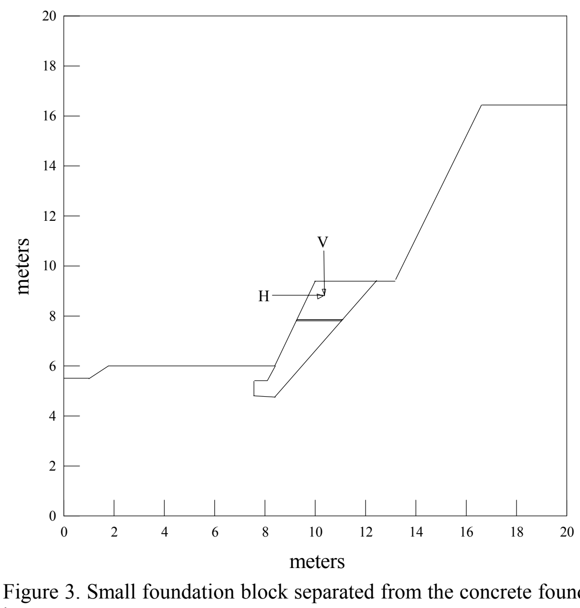

Figure 3. Small foundation block separated from the concrete foundation base.

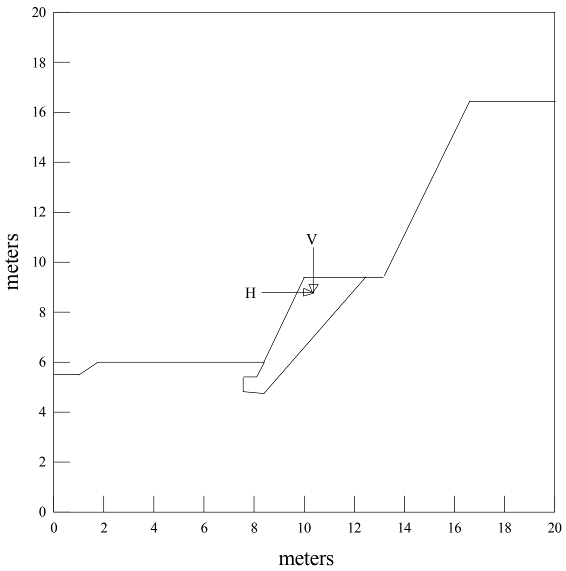

Figure 4. Small foundation block closely connected to the foundation base.

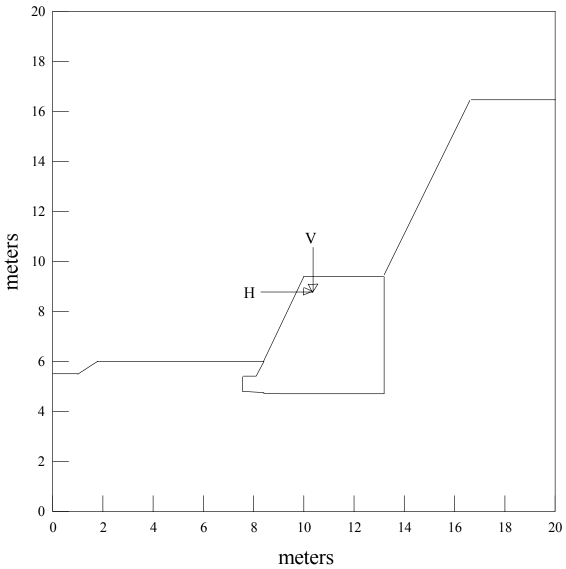

Figure 5. Large foundation block.

In the first analysis it was found that a small foundation block separated from the concrete foundation base gave satisfactory results for RMR = 44 (Fig. 6), so it is expected to provide satisfactory results for all values of RMR > 44. In the second analysis, a small foundation block closely connected to the foundation base gave satisfactory results for design loads only for a rock mass with a value of RMR > 23 (Fig. 7).

The displacement diagrams from the four analyses provided the values of horizontal (x) and vertical (y) displacements, as well as failure loads T_f (loads which cause failure of the foundation rock or soil). The corresponding factors of safety were calculated for the design load in the arch support, as shown in Table 2.

It can be seen that the displacements and the factors of safety are all satisfactory. The fourth analysis (softer clay as foundation soil) gives larger displacements than the ones that can be expected in-situ, because the carried stress-strain analysis was conservative in assuming that the foundation clay was both underneath and sideways of the large foundation block, whereas in reality it is limited to a much smaller region. In this analysis, which is only hypothetical, the allowable deformations of the structure are exceeded. Analyses No. 2 and 3 are more realistic in terms of clay properties.

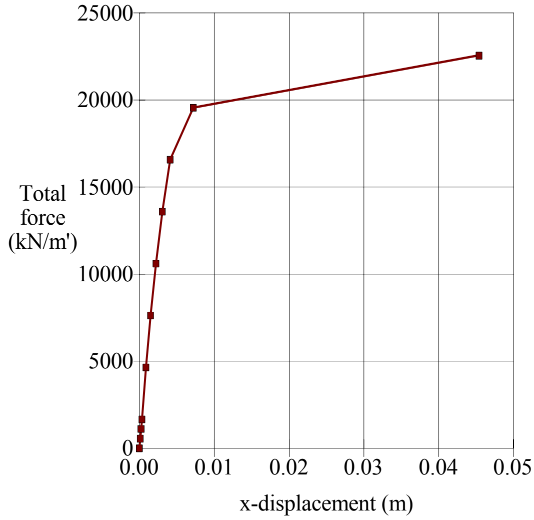

Figure 6. Analysis No. 1: relationship between total force in the arch support and horizontal support displacement.

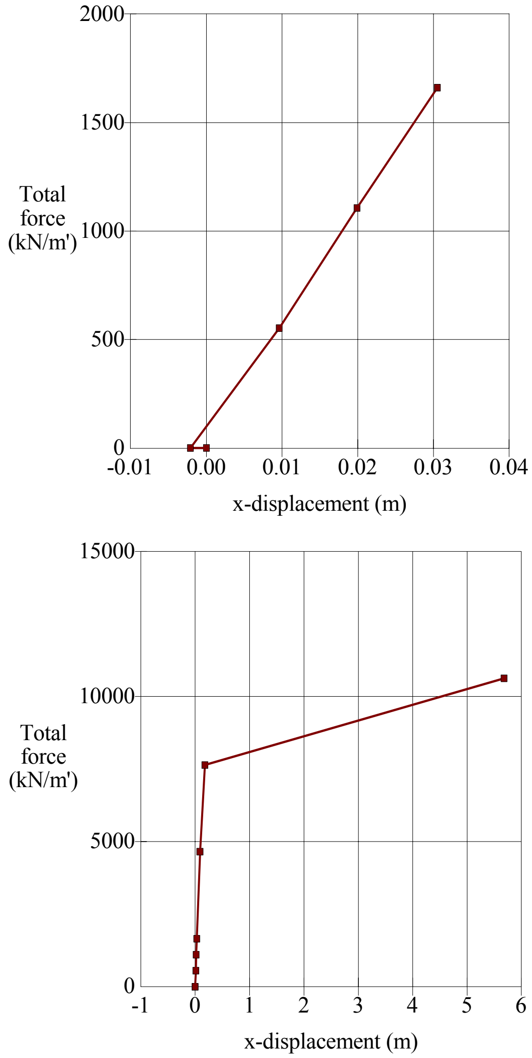

Figure 7. Analysis No. 2: relationship between total force in the arch support and horizontal support displacement (the lower diagram is the enlarged portion of the upper one, up to the value of the design load).

Table 2. Arch support displacements for the design load $T = (H^2 + V^2)^{1/2} = 1662$ kN/m′, rock failure load T_f, and factor of safety FS = T_f / T.

Analysis No.	x-displacement* cm	y-displacement* cm	Failure load kN/m′	FS
1	0.035	0.028	19500	11.7
2	3.2	3.7	7700	4.6
3	2.7	0.8	11000	6.6
4	11.0	3.5	4700	2.8

*Positive *x*-displacement is to the right, and positive *y*-displacement is downward

3 SUGGESTED ARCH FOUNDATIONS

The foundations were chosen based on the described analysis, and taking into account the detailed partition of the rock mass along both sides of the structure, as well as the structure segments. It was suggested to use three foundation types: a small foundation block with the allowable stress of 0.8 MPa, a horizontally extended foundation block with the allowable stress of 0.6 MPa, and a foundation block closely connected to the foundation base with the allowable stress of 0.4 MPa. Some foundation modifications were made to the numerical model, and it was estimated that horizontal displacements would not exceed 1 cm for all foundation types.

The second and third analyses have shown that there is no substantial difference between displacements under a large foundation block and a small block closely connected to the foundation base. Therefore, a foundation similar in shape to the one used in the second analysis was suggested for the region of clay with rock fragments, and concrete was used for filling underneath and behind the foundation block (Fig. 8).

It is important to emphasize that the boundaries of different categories of rock mass do not also represent the boundaries of the corresponding foundation types. Taking into account all influence factors, the crossing was subdivided into four segments, two 20 m, and two 30 m wide. These segments were founded according to the dominant rock mass characteristics.

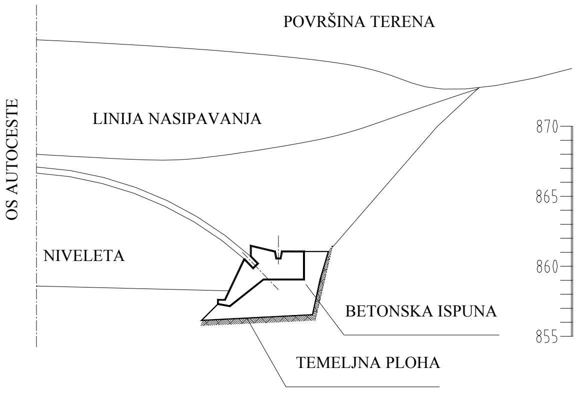

Figure 8. Foundation in the region of clay with rock fragments.

The stress-strain analyses gave the deformation criterion for the foundation rock or soil, so that the convergence and absolute displacements were monitored in situ. The convergence was monitored at four locations, one in the mid-section of each structural segment. These measurements showed uniform displacements along the whole structure (Fig. 9). It can be seen that maximum displacement values of 4 to 7 mm are in very good agreement with estimated horizontal displacements.

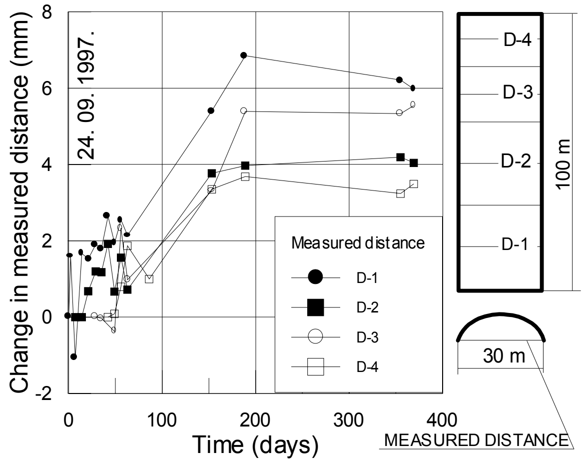

Figure 9. Monitored convergence

4 CONCLUSION

The case history presented in the paper shows the necessity of reconsidering the foundation design once the real nature of rock or soil is exposed, as the cut-off is made. The design reevaluation, described in the paper, was based on rock classification, determination of strength and deformation parameters, and numerical analysis. It resulted in a new crossing structure, subdivided into four segments, and three foundation types, as opposed to the original simple design. It is shown that the required uniformity of the arch structure foundation displacements is assured by the suggested design.

In-situ displacements were monitored at the arch abutment. Measured values correspond very well to predicted values.

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