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A field case of rock-bolt deformations in pullout tests

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ABSTRACT: The performance of any reinforcement support depends on the efficiency of the stress transfer mechanism. Extensive research was conducted over several decades to understand the complex interaction mechanism of load transfer along the reinforcement in soil as well as rock media. This paper presents a lucid comparative study of rock bolt deformations in pullout tests with a simplified theoretical prediction for geotechnical evaluations. Brief literature reviews on the topic are also summarised. Some selected field experiment results of typical rock bolt pullout tests of the Lam Ta Khong Pumped Storage project, Thailand are illustrated. The underground geology of the project site consists of soft quaternary deposit of clay/mudstone as well as sedimentary formation of sand/siltstones. The general patterns of deformation responses in rock-bolt pullout tests were observed to be linear.

1 INTRODUCTION

The Lam Ta Khong project area is about 180 kilometre north-east of Bangkok, Thailand. Major geology of the area consists of Jurassic sedimentary rocks of the Khorat (Phu-Kradung) formation and some quaternary deposits. Most of the underground openings are in sedimentary rocks of sandstone, sand/siltstone, siltstone and some openings are partially in quaternary deposits of clay/mudstone also. Deformation performances of the tunnel were observed by regular deformation monitoring programs. Routine quality control tests for the performance of the support reinforcements were conducted. Some aspects on geotechnical observations and deformation monitoring about the project may be obtained from Gurung and Iwao (1998). Brief technical notes on the deformation analysis of rock bolt in pullout test related to the project tunnels are presented.

Rock bolting has become a common measure for providing ground support, particularly in underground excavations. Performance of the support depends on the degree of load transfer mechanism of the reinforcement system. The inherent cohesion and friction in soils, and the strength of rocks in tension, compression, and shear are limited. If the applied loads induce greater stresses than those limits, then failure will result. The basic aim in any support design is to use the inherent strength of the geomaterials. Rock bolt reinforcements are applied to strengthen any relative weakness of earth materials in response to tension forces.

2 LITERATURE REVIEW

Extensive researches both experimental and theoretical were conducted over several decades to understand the reinforcement interaction mechanism in rock and soil media. Photoelastic model studies of rock-bolt action as reported by Lang (1961), Handa (1964) and Knill et al. (1966) reveal stress distribution patterns around the bolt. For an ideal rock mass within its elastic limit, the well-known Kirsch solution provides the variation of tangential and radial stresses around circular openings. Muskhelishvili (1953) presented mathematical solutions for various other shapes. Bray (1967) proposed analytical relations for the plastic behaviour around tunnels. Farmer (1975) has compared theoretical shear-stress distributions, with computed distributions from tests on instrumented bolts. A pullout formula for resin grouted anchorage systems in hard rock was presented as

$$T_p = 0.05\pi d_o L S_c \tag{1}$$

where is T_p the pullout force required (de-bonding), S_c is the rock compressive strength, d_o is the bolt hole diameter, and L is the bolt length.

The pullout resistances of weaker rocks depend mainly on the mobilised shear component. In a pre-failure state, Knill et al. (1966) postulated a linear increase in tension as the fractured rock mass dilates, provided no slip occurred. The induced tensions along a rock-bolt are reported to be time dependent and the applied tension at the free end is fully transferred to the media (Nitzsch and Haas, 1976). In any discontinuum analysis of underground structures, the patterns of stress distribution are important. For a simple model of a rock-bolt pullout test, the stress-displacement response may be safely assumed to be linear. Garga and Wang (1993) had numerically modelled rock-bolts in a jointed rock mass as a series of 1-D elements of block-spring models interacting through nodal points. A simple simulation of a rock-bolt model was based upon fixed two ends of the springs on the rock bolts. The relative displacements result in an axial force T in the bolt as

$$T = K_b \Delta L \tag{2}$$

The stiffness of the rock-bolt is computed as

$$K_b = \frac{E_r A_r}{L} \tag{3}$$

where E_r, A_r, L are modulus of elasticity, cross sectional area and length of the rock-bolt material, respectively.

Let us consider a general case in an underground opening. Suemasa et al. (1997) present theoretical formulations based on the elasto-plastic theory for deformation pattern of soil around a pile. A similar situation in Figure 1 depicts stresses around the rock-bolt in an underground site.

The well-known Kirsch solution may be used to compute the variations of stresses. The basic constitutive laws for underground rock-bolted section may be summarised as

$$\begin{bmatrix} \Delta\sigma_r \\ \Delta\sigma_\theta \end{bmatrix} = \frac{E_s}{(1+\nu_s)(1-2\nu_s)} \begin{bmatrix} 1-\nu_s & \nu_s \\ \nu_s & 1-\nu_s \end{bmatrix} \begin{bmatrix} \Delta\varepsilon_r \\ \Delta\varepsilon_\theta \end{bmatrix} \tag{4}$$

where $\Delta\sigma_r$ and $\Delta\sigma_\theta$ are incremental radial stress and tangential stress and $\Delta\varepsilon_r$ and $\Delta\varepsilon_\theta$ are incremental radial strain and tangential strain, respectively. The elasticity modulus and Poisson ratio are denoted as E_s and ν_s respectively.

The governing equation of stress distribution for bolted section (Goodman, 1989) may be expressed as

$$\frac{d\Delta\sigma_r}{dr} + \frac{\Delta\sigma_r - \Delta\sigma_\theta}{r} = 0 \tag{5}$$

where r = distance from opening centre in bolted section.

The rock-bolt supported section assumes the total radial stress by rock and bolts as

$$\sigma_r = \Delta\sigma_r + \Delta\sigma_{rb} \tag{6}$$

where $\Delta\sigma_r$ and $\Delta\sigma_{rb}$ refer to radial stress shared by rock and bolts. The differential displacement of rock-bolt model can be expressed as

$$\frac{d^2 u_b}{d\eta^2} - \alpha^2 u_b = -\alpha^2 \Delta u_h \tag{7}$$

where $\eta = r - a$, $\Delta u_h = u^*_{eq} - u^*_{in}$ = difference of displacement of grout-rock interface at equilibrium ($_{eq}$) and installation ($_{in}$) states.

The solution of the non-homogeneous differential equation for given boundary condition can be solved and the axial bolt displacement may be expressed in general form as

$$u_b = \sum Af(e^{-\alpha\xi}) \tag{8}$$

3 INTERFACE PULL-OUT EXPRESSION

The modulus of elasticity, diameter and length of the rock-bolt are E_r, d_r and L respectively. Generally, the tensile strength T_y of a rock-bolt is significantly higher compared to the pullout stress so that rock-bolt breakage is not possible. The applied pullout force, T_0, mobilises interface bond stresses, τ, along the length of the reinforcement.

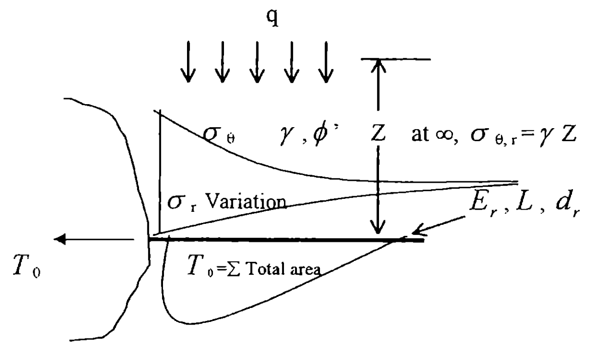

Figure 1. Underground stresses around rock-bolt.

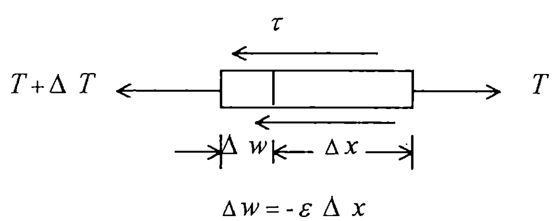

Figure 2. Free body diagram for bolt element.

In limit equilibrium, consider a small differential element of length, Δx, in the rock-bolt, (Fig. 2). The equilibrium of horizontal forces is satisfied by

$$(T+\Delta T)-T+\tau(\Delta x)\pi d_r = 0 \tag{9}$$

where T and $(T+\Delta T)$ are the pullout forces in the reinforcement on the left and right ends, τ - the mobilised bond resistance, and Δx - the element of reinforcement. The elongation, Δw, is related to the strain, ε, as

$$\Delta w = -\varepsilon \Delta x \tag{10}$$

while the strain, ε, is related to the tensile force, T, as

$$\varepsilon = 4T / E_r \pi d_r^2 \tag{11}$$

Hence, Eq. (9) can be expressed as

$$\frac{dT}{dx} + \tau . \pi d_r = 0 \tag{12}$$

Noting that for the x - axis positive to the right, $\varepsilon = -\frac{dw}{dx}$, all these are combined to give

$$E_r \; d_r \; \frac{d^2 w}{dx^2} - 4\tau = 0 \tag{13}$$

Equation (13) is a basic equation for a pull out test of a rock bolt. It is somewhat similar to the general governing expression (7) for the bolted section for $\Delta u_h = 0$ value. Idealising the interface bond stresses, τ, as $\tau = k \; w$, to be governed by the Winkler subgrade response, k, say a simple slope parameter for stiffness. The above expression (13) can be solved easily in a similar manner of axially loaded pile, and to obtain a closed form solution. Solving for the boundary conditions, one may easily get the deformation at the pull end $x = 0$, as

$$T_o = -\{\frac{\lambda E_r \pi d_r^{\,2}(1-e^{2\lambda L})}{4(1+e^{2\lambda L})}\} w \tag{14}$$

4 PULL-OUT TEST COMPARISIONS

The project site of soft quaternary deposits and sedimentary formations is selected. Major underground structures are the Access tunnel to the Powerhouse, Powerhouse, Drainage tunnel in Upper Pond, Penstock tunnels, Headrace, Power Cable and Tailrace tunnels. A standard support pattern based on the rock mass classification was adopted. Rock mass classifications are applied to solve support design problems. Fundamental geological factors for rock mass classification are the state of weathering, hardness and joints spacing, which were recorded, in digital expressions. Input details on degree of weathering, hardness, rock quality designation (RQD), rock mass rating (RMR), joint details (ISRM, 1978) like strike/dip, aperture, infilling, roughness, state and persistence, inflow and support type were used to classify the rock mass geology at the project site. Geotechnical observations, regular mapping, descriptions, photographing and rock mass classification (EPDC, 1996) were executed. The project correlation for rock mass types, hardness and RMR values were also made. Probable cohesion and friction of rock mass may be tentatively estimated (Hoek and Brown, 1980) using the Geomechanics classification.

In the case of the rock-bolt pullout tests, usually, linear trends of load-deformation patterns are observed, except in minor cases of slip failures. The standard material quality of steel bolts, resins or

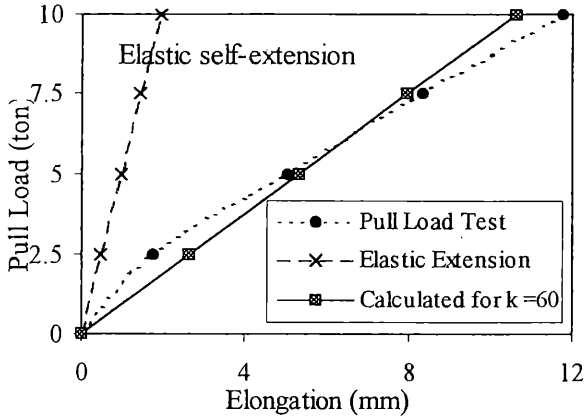

Figure 3. Rock-bolt pullout deformations at sta. 0+74 m in top gallery.

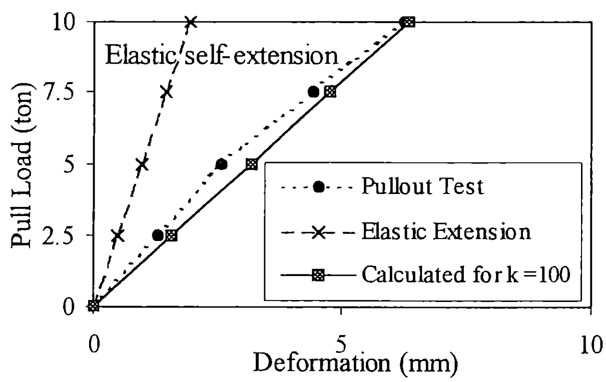

Figure 4. Rock-bolt pullout deformations at sat. 0+100 m in upper access tunnel to penstock.

cement grouts were observed to be good. Except for a few minor cases on poor workmanship, the geological defects such as low adhesion and poor bond may be the reason assumed for pullout failures in some soft ground in the Drainage tunnel, Upper Access tunnel to Penstock and Upper Horizontal tunnel to Penstock. In a few worse cases in the clay/mudstone, the ultimate capacity of bond stress seems to be limited to 50 to 70 kN only. The specified safe load for a satisfactory rock bolt was 100 kN at least. The rock-bolts of steel material of 25 mm diameter and 2 m to 3 m lengths were installed in most tunnels. Pullout loads were applied in increments and the deformations were recorded.

Low geological ratings on adhesion, friction and hardness state (geological scale) and or uncertain workmanship hindered the actual evaluation of the ultimate pull load capacity in soft wet ground. The contractor was even advised to drill just appropriate tight holes to avoid any possible losses of resins or grouts in geological voids. The test results, theoretical calculation and elastic self-extension are plotted for various tunnels. Linear responses are prominent in most of the cases. Figures 3 and 4 illustrate the deformation performance of rock bolts in pull load tests. The modulus of subgrade reaction k can be defined as the ratio of stress to deformation and it has units the same as unit weight. Average value of 70-170 kN/m^2/mm on estimate of k worked satisfactorily. It reflects the subgrade stiffness (load per deformation) of the ground and can be related to shear and elasticity modulus like in similar case of pile foundation. The pull test stiffness, k may be related to the shear and elasticity modulus of adjacent ground as in the case of pile foundation (Scott, 1981) as

$$k \cong \frac{E_s}{4(1-v_s^{\ 2})d_r} \tag{15}$$

Near failure deformations in some weak soft ground (ST-3, refer EPDC 1996) indicated a non-linear response, where the stiffness k appears to degrade, indicating on going deterioration at higher pull out loads. Except in a few non-linear failures most of the deformation results of rock bolt pull tests are explainable by the above formulation.

Some non-linearity in failure cases shows the basic differences in the shear formulation. Probable errors may be on the part of distribution of the shear stress along the rock-bolt length as the actual stress distribution along the rock-bolt is complex. It depends upon many factors say bar type and bonded length, relative pull out load level, boundary, face rigidity and end effects, modulus of elasticity, deformation and shear modulus of ground, geological discontinuities, installation method, shrinkage, creep and time lags etc. Mizuno and Watanabe (1963) showed variations in bond stress distributions (non-linear) for different types of reinforcement bars embedded in concrete. Typical ranges of modulus, E_s for different types of ground are as follows.

Table 1 Elasticity modulus for soils.

Soil type	E_s (Mpa)
Very soft clay	2-15
Soft clay	5-25
Medium clay	15-50
Hard clay	50-100
Silt	2-20
Shale	150-5000

5 CONCLUSIONS

Rock-bolt pullout test performed in incremental steps with deformation measurements helped geotechnical evaluation of the ground. The subgrade modulus from the pullout load-deformation analysis may be used to estimate the values of elasticity and shear modulus of the tested bolt interface with ground. Typical field pullout tests conducted in soft ground of quaternary deposit related to the Lam Ta Khong project were illustrated. The presented theoretical predictions and test results showed comparable agreement. The patterns of deformation responses in rock-bolt pullout tests were observed to be linear in general.

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