Water Science and Engineering, 2009, 2(3): 87-95
doi:10.3882/j.issn.1674-2370.2009.03.009

http://kkb.hhu.edu.cn
e-mail: wse@hhu.edu.cn

Tunnel design considering stress release effect

Van-hung DAO*[1, 2]

1. College of Water Conservancy and Hydropower Engineering, Hohai University, Nanjing 210098, P. R. China
2. Faculty of Hydraulic Construction, Water Resources University, Hanoi, Vietnam

Abstract: In tunnel design, the determination of installation time and the stiffness of supporting structures is very important to the tunnel stability. This study used the convergence-confinement method to determine the stress and displacement of the tunnel while considering the counter-pressure curve of the ground base, the stress release effect, and the interaction between the tunnel lining and the rock surrounding the tunnel chamber. The results allowed for the determination of the installation time, distribution and strength of supporting structures. This method was applied to the intake tunnel in the Ban Ve Hydroelectric Power Plant, in Nghe An Province, Vietnam. The results show that when a suitable displacement u_0 ranging from 0.086 5 m to 0.091 9 m occurrs, we can install supporting structures that satisfy the stability and economical requirements.

Key words: *tunnel; supporting structures; stability; counter-pressure curve; stress release effect*

1 Introduction

Rock in the natural environment, especially in deep layers, is influenced by the upper stratum and its gravity load. Stresses developing within the rock mass due to these impacts are very complicated and difficult to define. During tunnel excavation, an amount of rock normally serving to receive pressure from the weight of the rock on the tunnel roof is removed, and tension stresses, which sometimes reach rather high values, are generated within the rock mass surrounding the tunnel. The transition from a tri-axial compression stress state to a bi-axial stress state due to the stress release around the circumference of the excavated chamber results in the deformation of rock surrounding the excavation boundary. During the tunnel construction process, the supporting structures needs to be installed for the purpose of maintaining or improving the load-bearing capacity of rock masses in order to maximize supporting capacity and to create favorable development of the stress field within the rock mass.

Fenner (1938) carried out research on the interaction between the upper stratum and the hydraulic structure, and found out the specific curve of the foundation and the solution for a problem in an elastic-plastic medium. Pacher (1963) carried out the same study and obtained the same solution. When the design of the tunnel considers the interaction between the upper stratum and the hydraulic structure, the result is suitable for actual structures and the New

*Corresponding author (e-mail: *daohung_dhtl@yahoo.com*, *daohung@hhu.edu.cn*)
Received Mar. 12, 2009; accepted Jul. 10, 2009

Austrian Tunnelling Method (NATM). Besides, in tunnel design, the interaction between the tunnel lining and the rock surrounding the tunnel chamber, as well as the counter-pressure curve of the ground base, are usually considered (Panet and Guenot 1982; Panet 1995). The convergence-confinement method is considered to be effective in designing the tunnel. In Vietnam, Nguyen (2007) researched the influence of changing underground water pressure on the load, which affects the tunnel lining. Vu and Do (2007) applied the convergence-confinement method in designing the tunnel with the assumptive displacement u_0 for the calculation.

According to Fenner (1938) and Pacher (1963), if a rigid supporting structure ② (Fig. 1) is installed early, it will have more load-bearing capacity, since the deformation of the rock mass surrounding the excavated chamber is not large enough to reach equilibrium. Beyond point C of the p_i curve (Fig. 1), the rock properties become non-linear (plastic). When the supporting structure ① are installed after a certain displacement has occurred (point A), the system reaches equilibrium with the smaller load on the tunnel lining. After the σ_r curve reaches its minimum value (marked B in Fig.1), the loosening begins and the pressure on the tunnel lining increases very quickly. If the supporting structures are installed at the moment of permissible deformation, pressure on the supporting structures reaches its minimum value without resulting in the instability of the tunnel, as shown in Fig. 1.

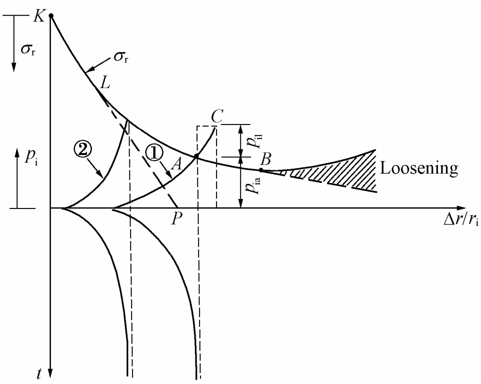

Fig. 1 Interactive curve between rock and lining according to Fenner (1938) and Pacher (1963)
(p_i is the supporting pressure, σ_r is the radial stress, Δr is the radial deformation, r_i is the tunnel radius, and p_{ia} and p_{il} are the support resistances of outer and inner arches, respectively)

This study presents the convergence-confinement method of determining the stress and displacement of the tunnel while considering the counter-pressure curve of the ground base, the stress release effect, and the interaction between the tunnel lining and the rock surrounding the tunnel chamber.

2 Models of interaction between ground base and lining

2.1 Stress computation considering ground reaction curve

In the case that the initial stresses are hydrostatic stresses (coefficient of lateral pressure equals unity), the stress distribution surrounding the excavated chamber has a radius of r_{i}, as shown in Fig. 1 (Hoek and Brown 1980). The assumption here is that the radius of plastic region r_{e} depends on the magnitude of the initial stress field p_0, the supporting pressure p_{i}, and the characteristics of the rock material.

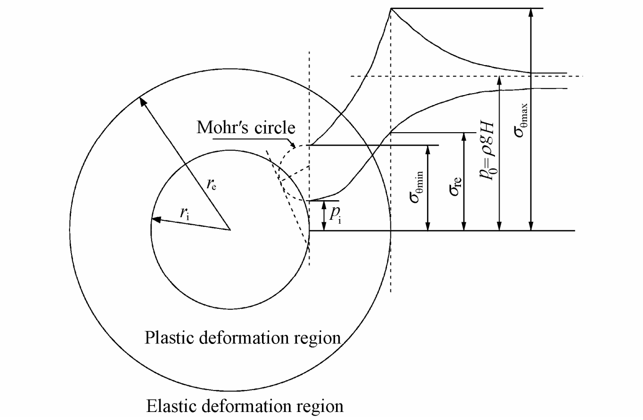

Fig. 2 Elastic-plastic model and stress field surrounding tunnel

The stress at the boundary of the plastic deformation region is

$$\sigma_{\mathrm{re}} = \left(\rho gH + C\cot\varphi\right)\left(\frac{r_{\mathrm{e}}}{r_{\mathrm{i}}}\right)^{\frac{2\sin\varphi}{1-\sin\varphi}} - C\cot\varphi \tag{1}$$

where ρ is the density of the rock, g is the acceleration of gravity. H is the excavation depth, C is the apparent cohesion of the rock mass, and φ is the angle of internal friction of the rock. The stresses within the elastic deformation region ($r \ge r_{\mathrm{e}}$) are

$$\begin{cases} \sigma_{\mathrm{r}} = \rho gH\left(1-\dfrac{r_{\mathrm{e}}^2}{r^2}\right) + \sigma_{\mathrm{re}}\dfrac{r_{\mathrm{e}}^2}{r^2} \\ \sigma_{\theta} = \rho gH\left(1+\dfrac{r_{\mathrm{e}}^2}{r^2}\right) - \sigma_{\mathrm{re}}\dfrac{r_{\mathrm{e}}^2}{r^2} \end{cases} \tag{2}$$

where σ_{r} is the radial stress, σ_{θ} is the shear stress, and r is the radius of the considered region. The stresses within the plastic deformation region ($r_{\mathrm{i}} \le r \le r_{\mathrm{e}}$) are

$$\begin{cases} \sigma_{\mathrm{r}} = \left(p_{\mathrm{i}} + C\cot\varphi\right)\left(\dfrac{r}{r_{\mathrm{i}}}\right)^{\alpha} - C\cot\varphi \\ \sigma_{\theta} = k\left(p_{\mathrm{i}} + C\cot\varphi\right)\left(\dfrac{r}{r_{\mathrm{i}}}\right)^{\alpha} - C\cot\varphi \end{cases} \tag{3}$$

where $k = \dfrac{1+\sin\varphi}{1-\sin\varphi}$, and $\alpha = k-1$.

The radial displacement of the tunnel is

$$u_{\mathrm{r}} = \frac{r_{\mathrm{i}}}{2G}\chi \tag{4}$$

where G is the shear modulus of the rock mass, and

$$\chi = (2v-1)(\rho g H + C\cot\varphi) + (1-v)\frac{k^2-1}{k+k_{\mathrm{p}}}(p_{\mathrm{i}} + C\cot\varphi)\left(\frac{r_{\mathrm{e}}}{r_{\mathrm{i}}}\right)^{\alpha}\left(\frac{r_{\mathrm{e}}}{r}\right)^{k_{\mathrm{p}}+1} + (1-v)\frac{kk_{\mathrm{p}}+1}{k+k_{\mathrm{p}}-v}(p_{\mathrm{i}} + C\cot\varphi)\left(\frac{r}{r_{\mathrm{i}}}\right)^{k_{\mathrm{p}}-1} \tag{5}$$

where v is the Poisson's ratio for the rock mass, $k_{\mathrm{p}} = \dfrac{1+\sin\psi}{1-\sin\psi}$, and ψ is the angle of volumetric expansion of the rock mass in a disintegrating state.

2.2 Stress release coefficient and radial displacement of tunnel boundary along tunnel axis

Under the influence of the heading face and the non-excavated rock, the maximum rock radial displacement u_{rmax} without consolidation can only be reached at a certain distance from the heading face (the result from experimental measurements is usually 1.53 × $2r_{\mathrm{i}}$). The relation between $u_{\mathrm{r}}/u_{\mathrm{rmax}}$ and the distance x from the heading face of a tunnel with a radius of r_{i} was established as follows by two researchers, on the basis of field measurement data: from elastic models of the problem represented in Fig. 3, Panet (1995) suggested the following relationship between $u_{\mathrm{r}}/u_{\mathrm{rmax}}$ and distance x from the face:

$$\lambda_{\mathrm{d}} = \frac{u_{\mathrm{r}}}{u_{\mathrm{rmax}}} = 0.25 + 0.75\left[1 - \left(\frac{0.75}{0.75 + \dfrac{x}{r_{\mathrm{i}}}}\right)^2\right] \tag{6}$$

where λ_{d} is the stress release coefficient. This relationship (6), which applies to positive values of x (i.e., behind of the face), is plotted in Fig. 4.

Chern et al. (1998) presented measured values of convergence in the vicinity of the face for a tunnel in the Mingtam Power Cavern Project. The measured data are plotted as dots in Fig. 4. Based on this data, Hoek (1999) suggested the following empirical best-fit relationship between $u_{\mathrm{r}}/u_{\mathrm{rmax}}$ and distance x from the face:

$$\lambda_{\mathrm{d}} = \frac{u_{\mathrm{r}}}{u_{\mathrm{rmax}}} = 1 + \exp\left(\frac{-x}{1.10 r_{\mathrm{i}}}\right)^{-1.7} \tag{7}$$

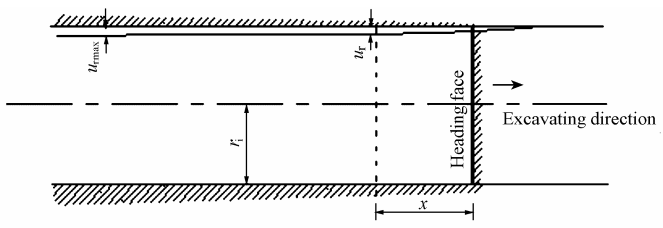

Fig. 3 Profile of radial displacements u_r for an unsupported tunnel

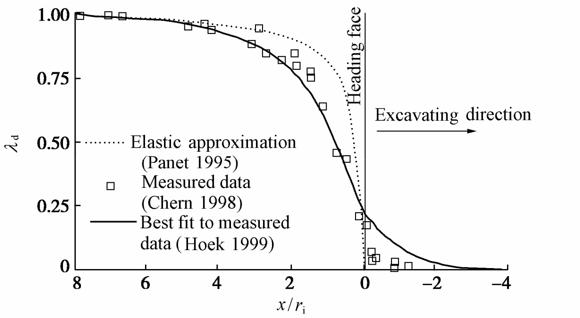

Fig. 4 λ_d profiles derived from elastic models (Panet 1995), measurements in tunnel (Chern et al. 1998), and best fit to measurements (Hoek 1999)

2.3 Characteristic curve of supporting structure

The characteristic curve shows the working capacity of the supporting structures (concrete, gunite, rock anchor or form steel). It is based on the linear relation between supporting pressure p_i and radial displacement u_r, and it is applied to a supporting section for a unit length along the tunnel axis.

Assuming the stiffness of supporting structures to be K_s, the elastic section of the support characteristic curve can be calculated using the following formula:

$$P_s = K_s u_r \tag{8}$$

The stiffness of concrete or gunite structures is

$$K_s = \frac{E_c}{1+\nu_c} \frac{r_i^2 - (r_i - t_c)^2}{(1-2\nu_c) r_i^2 + (r_i - t_c)^2} \tag{9}$$

where E_c is the elastic modulus of gunite (concrete), ν_c is the Poisson coefficient of gunite (concrete), and t_c is the lining thickness.

The stiffness of a steel support structure is calculated with the following formula:

$$\frac{1}{K_s} = \frac{S r_i}{E_s A_s} + \frac{S r_i^3}{E_s I_s} \frac{\theta(\theta + \sin\theta\cos\theta)}{2\sin^2\theta} + \frac{2S\theta t_B}{E_B W^2} \tag{10}$$

where S is the distance between the supports along the tunnel axis (m), θ is the half of the angle between the tamping bars (°), W is the width of the tamping blocks (m), A_s is the cross-sectional area of the section (m^2), I_s is the moment of inertia of the section (m^4), E_s is the Young's modulus for the steel (MPa), t_B is the thickness of the block (m), and E_B is the Young's modulus for the block material (MPa).

The stiffness of a supporting structure using a mechanical anchor or chemical bonding anchor with a length of l_b and a diameter of d_b can be calculated as follows:

$$\frac{1}{K_s} = \frac{S_l S_c}{r_i} \left(\frac{4l}{\pi d_b^2 E_b} + Q \right) \tag{11}$$

where S_c is the distance between the anchors along the tunnel circumference, S_l is the distance between the anchors along the tunnel axis, Q is the anchor pulling force, E_b is the elastic modulus of anchor materials, and l is the free length of the bolt or cable.

When composite supporting structures are used, the components of the composite supporting structures are all assumed to be installed at the same time, and the stiffness of the composite supporting structures is assumed to be the sum of the stiffness of each of the structure's components:

$$K_s = K_{s1} + K_{s2} \tag{12}$$

where K_{s1} is the stiffness of the first supporting structure, and K_{s2} is the stiffness of the second supporting structure.

Therefore, the characteristic curve of the supporting structure is specified by the following equation:

$$u_p = u_0 + \frac{P_i r_i}{K_s} \tag{13}$$

where u_p is the displacement component of supporting structures and compressed rock, and u_0 is the initial displacement component of the tunnel before the lining is installed (defined by means of the stress release effect).

3 Example study

3.1 Description of example and design parameters

A survey of the intake tunnel of the Ban Ve Hydroelectric Power Plant (Nghe An Province, Vietnam) was carried out. The material parameters of the tunnel are shown in Table 1.

Table 1 Physical and mechanical parameters of tunnel

r_i (m)	ρ (kg/m^3)	Rock elastic modulus E (MPa)	H (m)	C (MPa)	v	φ (°)
1.7	2 600	1 291	280.8	5.3	0.27	46.88

The applied supporting structure was a combination of Gunite M300 with a thickness of 10 cm and steel anchors with diameters of 20 mm and lengths of 2 m. Anchor spacing along the tunnel circumference and along the tunnel axis was 1.5 m. The Matlab programming language was used for the computation.

3.2 Calculation results and analysis

The stress value of the ground base $p_0 = 7.3008$ MPa. Figs. 5 and 6 show the stresses within the plastic and elastic regions, respectively. It can be seen from Fig. 5 that the maximum plastic region radius $r_{emax} = 1.1451$ m. Therefore, the stress at the elasto-plastic boundary $\sigma_{re} = 4.0195$ MPa. This is the maximum pressure value that the supporting structure is able to bear. The maximum displacement $u_{rmax} = 0.1118$ m, which corresponds to $p_i = 0$ (without support). Fig. 7 shows the stress release coefficient of the tunnel boundary without support along the tunnel axis. The interactive curves between the ground base and supporting structures at different initial displacements are shown in Fig. 8.

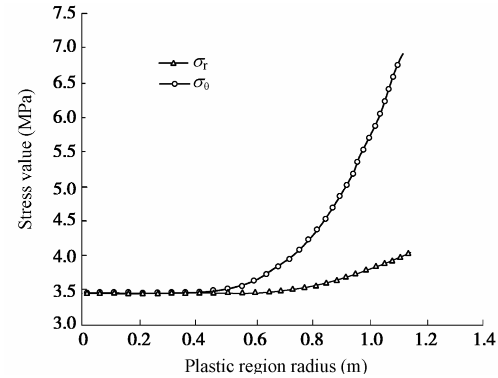

Fig. 5 Rock stress within plastic region

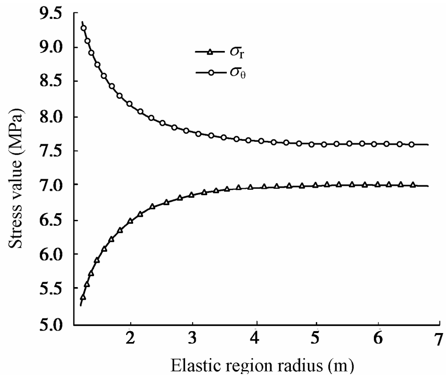

Fig. 6 Rock stress within elastic region

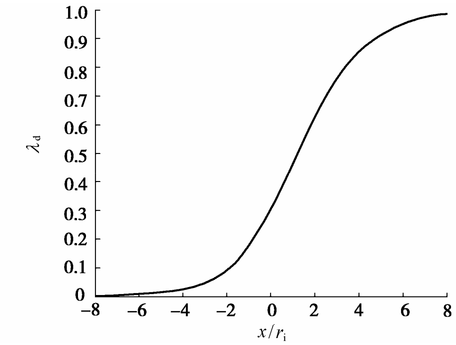

Fig. 7 Stress release coefficient of tunnel boundary without support along tunnel axis

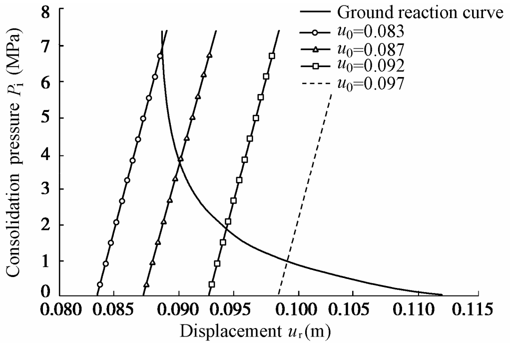

Fig. 8 Interactive curve between ground base and supporting structures for different initial displacements

Based on the Fenner-Pacher theory and *The Vietnamese Construction Design Standard for Underground Works* (Ministry of Construction 2003), we compared the pressure on the supporting structures with the maximum pressure value that the supporting structures are able to bear (which is equal to the stress at the elasto-plastic boundary σ_{re}) to analyze the above results. It can be concluded that:

Supporting structures installed when the initial displacement $u_0 = 0.083$ m result in the following: There is immediate consolidation after the tunnel excavation. Pressure on supporting structures $P_i = 6.902$ MPa > $\sigma_{re} = 4.019\ 5$ MPa. With unfavorable operation of supporting structures, the rock continues deforming after the support is in place. This results in local instability.

Supporting structures installed when the initial displacement $u_0 = 0.087$ m result in the following: Consolidation occurs at a distance of $x = 1.005 r_i = 1.708\ 5$ m, and the stress release coefficient $\lambda_d = 0.481\ 7$. Pressure on the supporting structure $P_i = 3.781\ 8$ MPa < σ_{re} =

4.0195 MPa. The rock has sufficient deformation, and the tunnel is stable.

Supporting structures installed when the initial displacement $u_0 = 0.093$ m result in the following: Consolidation occurs at a distance of $x = 2.2r_{\rm i} = 3.74$ m, and the stress release coefficient $\lambda_{\rm d} = 0.7327$. Pressure on the supporting structructure $P_{\rm i} = 1.9511$ MPa □ $\sigma_{\rm re} =$ 4.0195 MPa. The rock has major deformation, indicating that the tunnel can be unstable.

Supporting structures installed when the initial displacement $u_0 = 0.097$ m result in the following: Consolidation occurs at a distance of $x = 4r_{\rm i} = 6.8$ m, and the stress release coefficient $\lambda_{\rm d} = 0.868$. Pressure on the supporting structructure $P_{\rm i} = 0.9632$ MPa □ $\sigma_{\rm re} =$ 4.0195 MPa. Rock deformation is too great; there can be rock loosening of the tunnel roof causing the increase of rock pressure. The tunnel is unstable.

Thus, in this case, we can say that at each time, with a certain displacement u_0 ranging from 0.0865 m to 0.0919 m, we can install supporting structures that satisfy the stability and economical requirements.

4 Conclusions

In general, the determination of initial displacement u_0 described in this study is more accurate and detailed than assumptions of the initial displacement value u_0 (Vu and Do 2007). Values of u_0 depend on the stress release effect and, when compared, provide a more complete solution than the solution with curves that exclude the stress release effect (Hoek and Brown 1980; Wiliams 1997).

The survey described above has shown that the convergence-confinement method is an effective design tool for obtaining appropriate supporting time. It is completely different from the traditional tunnel design method, which applies the early consolidation and quickly lining installation rules, considers the supporting structures provisional supporting structures to bear loads of loosened rock, and ignores the load-bearing capacity of rock masses.

However, the problem is limited to the two-dimensional elasto-plastic model, hydro-static inital stress field and circular tunnel cross-section. Therefore, in the case of rock with a non-hydrostatic stress field, or of non-circular tunnel cross-sections, the destructive models such as the non-homogeneous elasto-plastic, visco-elastic, and brittle models, need to be studied so that the convergence-confinement method can be applied more widely in tunnel design.

References

Anagnostou, G., and Kovari, K. 1993. Significant parameters in elastoplastic analysis of underground openings. *Journal of Geotechnical Engineering*, 119(3), 401-419. [doi:10.1061/(ASCE)0733-9410(1993)119:3(401)]

Brady, B. H. G., and Brown, E. T. 2004. *Rock Mechanics for Underground Mining*. Dordrecht: Kluwer Academic Publishers.

Carranza-Torres, C., and Fairhurst, C. 1999. The elasto-plastic response of underground excavations in rock masses that satisfy the Hoek-Brown failure criterion. *International Journal of Rock Mechanics and Mining Sciences*, 36(6), 777-809.

Carranza-Torres, C., and Fairhurst, C. 2000. Application of the convergence-confinement method of tunnel design to rock masses that satisfy the Hoek-Brown failure criterion. *Tunneling and Underground Space Technology*, 15(2), 187-213. [doi:10.1016/S0886-7798(00)00046-8]

Carranza-Torres, C., Alonso, E., Alejano, L. R., Varas, F., and Fdez-Manin, G. 2002. Elasto-Plastic analysis of deep tunnels in brittle rock using a scaled form of the Mohr-Coulomb failure criterion. Hammah, R., Bawden, W., Curran, J., and Telesnicki, M., eds., *Proceedings of the 5th North American Rock Mechanics Symposium and the 17th Tunnelling Association of Canada Conference (NARMS-TAC 2002): Mining and Tunnelling Innovation and Opportunity*, 283-293. Toronto: University of Toronto.

Carranza-Torres, C. 2003. Dimensionless graphical representation of the exact elasto-plastic solution of a circular tunnel in a Mohr-Coulomb material subject to uniform far-field stressess. *Rock Mechanics and Rock Engineering*, 36(3), 237-253. [doi:10.1007/s00603-002-0048-7Rock]

Chern, J. C., Shiao, F. Y, and Yu, C. W. 1998. An empirical safety criterion for tunnel construction. *Proceedings of Regional Symposium on Sedimentary Rock Engineering*, 222-227. Taipei, China.

Fairhurst, C., and Carranza-Torres, C. 2002. Closing the circle: Some comments on design procedures for tunnel supports in rock. Labuz, J. F., and Bentler, J. G., eds., *Proceedings of the University of Minnesota 50th Annual Geotechnical Conference*, 21-84. Minneapolis: University of Minnesota Press.

Fenner, R. 1938. Untersuchungen zur erkenntnis des gebirgsdruckes. *Glückauf*, 32, 681-695.

Goodman, R. E. 1980. *Introduction to Rock Mechanics*. New York: John Wiley and Sons.

Hoek, E., and Brown, E. T. 1980. *Underground Excavations in Rock*. London: Institute of Mining and Metallurgy.

Hoek, E., Kaiser, P. K., and Bawden, W. F. 1995. *Support of Underground Exactions in Hard Rock*. Rotterdam: Balkema.

Hoek, E. 1999. Support for very weak rock associated with faults and shear zones. *Proceedings of International Symposium on Rock Support and Reinforcement Practice in Mining*. Kalgoorlie.

Ministry of Construction. 2003. *The Vietnamese Construction Design Standard for Underground Works*. Hanoi: Construction Publisher. (in Vietnamese)

Nguyen, P. D. 2007. Influence of the change in underground water pressure to the load effecting on the tunnel lining. *Journal of Road and Bridge*, 1(2), 11-18. (in Vietnamese)

Nguyen, Q. H., and Nguyen, T. H. 2004. *Design of the Underground Traffic Structure*. Hanoi: Traffic Publisher. (in Vietnamese)

Pacher, F. 1963. Deformationsmessungen in Versuchsstollen als Mittel zur Enforschung des Gebirgsverhaltens und zur Bemessung des Ausbaues. *Proceedings of the 14th Symposium of the Austrian Regional Group of the ISRM*, 149-161. Salzburg.

Panet, M., and Guenot, A. 1982. Analysis of convergence behind the face of a tunnel. *Proceedings of the International Conference on Tunnelling*, 197-204. Brighton: The Institution of Mining and Metallurgy

Panet, M. 1995. *Le Calcul des Tunnels par la Méthode Convergence-Confinement*. Paris: Presses de l'École Nationale des Ponts et Chaussées. (in French)

Tran, B. V., and Bui, T. C. 2006. The method of convergence-confinement in vault designing. *Journal of Road and Bridge*, 26(2), 126-139. (in Vietnamese)

Tran, T. G., and Ta, T. D. 2003. *The Design of Underground Structures*. Hanoi: Construction Publisher. (in Vietnamese)

Vu, T. T. G., and Do, N. T. 2007. Applying the convergence-confinement method in designing the tunnel. *Proceeding of the International Workshop Institute of Transport Science and Technology*, 24-35. Danang: Danang University. (in Vietnamese)

Wiliams, O. 1997. *Engineering and Design-Tunnels and Shafts in Rock*. Washington: U. S. Army Corps of Engineers.