

BEHAVIOR OF PRESSURE TUNNELS AND GUIDELINES FOR LINER DESIGN

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ABSTRACT: Numerous cases of pressure-tunnel failures have occurred in recently commissioned projects. The cost of the remedial measures, particularly the lost revenue during the time taken for the implementation of these measures, is significant. This paper presents a framework to identify and evaluate the variables that control tunnel behavior and thus determine the key decisions made in the design of a pressure tunnel. An approach is also given for incorporating the hydraulic and mechanical liner-ground interaction in the evaluation of the water tightness of various types of liners under operating conditions as well as for evaluating the capability of these liners to control the pore-water pressure in the surrounding rock mass. This approach also permits an evaluation of potential benefits derived from consolidation grouting of the rock mass surrounding the tunnel. Finally, specific liner design recommendations and guidelines are given to accommodate a wide combination of rock mass characteristics, topography, ground-water levels, and operating conditions.

INTRODUCTION

Several recently commissioned pressure tunnels have shown an unsatisfactory performance during first filling or shortly afterward. In all cases, failures have occurred because either a fundamental mode of failure was not recognized at the design stage or because the design was carried out by extrapolation from existing precedent without a proper understanding of the variables controlling the behavior of the liner and/or surrounding media. In most cases, the cost of remedial measures, including the loss of revenue, exceeded the estimated cost of a more conservative initial design. Furthermore, in several cases initial remedial measures implemented after failure were inadequate, requiring additional repair efforts and further delays.

In the writer's experience, most of the difficulties resulted from the lack of a valid analytical framework permitting designers to identify key variables controlling tunnel behavior, as well as the sensitivity of the tunnel behavior to improvements in one or several of these key variables.

This paper presents a framework for evaluation of the behavior of unlined, as well as lined, tunnels, taking into account the hydraulic and mechanical interaction of the liner and surrounding media. In the analysis, both the liner and the surrounding fractured rock mass have been idealized as isotropic and homogeneous media and therefore in many cases the actual flow regime and the absolute value of pressure and leakage may differ from those estimated from this analysis. However, the relative significance of the variables involved and the sensitivity of the rock-liner system to changes in these variables can be adequately evaluated with this model. Thus, the framework proposed in this paper provides for a systematic review of the technical basis supporting the main decisions made in pressure tunnel design. These decisions include the location of the tunnel with respect to the sur-

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rounding topography and ground-water level, the need for liner and/or rock treatment at various sections along the alignment, and the design of plugs at construction adits. The paper also provides design guidelines to accommodate a wide combination of rock-mass characteristics, topography, ground-water level, and operating conditions.

Additional studies using the discrete finite-element method (DFEM), which can account for the discontinuous nature of fractured rock masses and fully consider all hydromechanical coupling in the system, are currently underway to adjust for the nonhomogeneous and nonisotropic conditions that can be encountered in the fractured rock mass around a tunnel.

POTENTIAL MODES OF FAILURE

The most common modes of failures observed in pressurized tunnels have been previously described by Hendron et al. (1987) and can be summarized as follows.

Excessive Leakage

Excessive leakage can develop in areas of low in-situ stress, where hydraulic fracturing/or hydrojacking of the surrounding rock mass can result in intolerably large flows out of the tunnel. Excessive leakage can also develop in areas of adequate in-situ stress but high hydraulic gradient, which include locations like the vicinity of underground openings, the intersections between the pressure tunnel and permanent access adits, or areas of the tunnel adjacent to deep valleys or gullies.

Environmental considerations can also limit the amount of tolerable leakage to relatively small amounts. In addition, leakage out of tunnels excavated in materials susceptible to erosion and/or dissolution can result in the removal of liner support, resulting in the collapse of the opening.

Excessive Pore-Water Pressures

Excessive pore-water pressures in the rock mass surrounding pressurized tunnels have triggered extensive slides and/or significant movements in nearby slopes, resulting in temporary shutdowns of power plants and the need for extensive remedial measures. In cases where parallel pressure conduits are operated independently, large pore-water pressures induced around a pressurized conduit have caused buckling of the liner in the adjacent unloaded tunnels.

Failure of Linings

A common mode of failure is the unsatisfactory performance of a liner, which may be subjected to loading conditions either not considered in the design or much more severe than those initially considered. The buckling of a steel liner under high external pressures is a typical example of this occurrence. Failure of the liner can also occur because the actual structural behavior of the liner is different than the structural behavior considered in the design. For example a poorly executed contact grouting program will deprive the liner of the beneficial constraint provided by the surrounding rock.

Collapse of Openings

Collapse and closure of long (up to 1 km) shotcrete-lined sections of pressure tunnels have occurred due to loss of strength of the materials

around the opening as they take on water from the tunnel. Typical examples of this condition are openings excavated in shales that can be supported initially by a light system of bolts and shotcrete. However, once the tunnel is filled and the pressurized water contacts the materials around the opening, a substantial reduction of shear strength occurs as the materials swell, resulting in the collapse of the tunnel.

VARIABLES EVALUATED IN DESIGN AND LAYOUT OF PRESSURE TUNNELS

An evaluation of the potential development of the first three modes of failure requires an estimate of the leakage out of the tunnel and the pore-water pressures induced in the surrounding rock mass. This paper presents an approach to evaluate these two parameters for unlined and lined tunnels. The main variables involved in determining the magnitude of the leakage and the pore-water pressure in the surrounding media have been previously described by Hendron et al. (1987) and can be summarized as: (1) Topography and ground-water elevation along the tunnel alignment; (2) the susceptibility of the rock materials to dissolution, deterioration, and/or erosion; (3) the deformability of the rock mass, and (4) the permeability of the rock mass. Special efforts should also be made to identify the location and more pertinent characteristics of faults, shear zones, and weathered or fractured zones intersecting or within the immediate vicinity of the alignment. Because a substantial amount of this information is obtained as the rock materials are exposed in the tunnel walls, specifications for pressure tunnel liners should be flexible enough to benefit from the data collected during excavation.

EVALUATION OF LEAKAGE AND PORE-WATER PRESSURE DEVELOPMENT OUTSIDE PRESSURE TUNNELS

An analytical approach is developed to estimate the amount of leakage and the pore-water pressure distribution in the rock mass around the pressurized tunnel assuming that the rock mass behaves as an isotropic, homogeneous permeable medium. An evaluation is first made of an unlined tunnel, where the main controlling variables are the location of the ground-water level, the internal tunnel pressure and the rock-mass permeability. In a subsequent section an evaluation is made of the beneficial effects of a liner and/or rock-mass treatment around the excavation.

Evaluation of Hydraulic Conditions around Unlined Pressure Tunnels

Unlined pressure tunnels can be successfully used in various geological environments (Deere 1983) provided that: (1) The materials around the opening are self-supporting rock not susceptible to dissolution, erosion, deterioration, or substantial reduction in strength; (2) the in-situ stress along the alignment is adequate to preclude hydrofracturing or hydrojacking of the surrounding rock mass; (3) the permeability of the rock mass around the opening is low; and (4) localized zones of fair- to poor-quality rock in the unlined sections are treated with rock bolts, liner, or dental concrete.

For a homogeneous, isotropic mass, the magnitude of the leakage out of the tunnel as well as the pore-water pressure distribution in the rock mass around the excavation can be approximated using the image well method proposed by Harr (1962) to evaluate the flow between two wells, a source

and a sink, of equal strength. The pressurized tunnel can be considered as a source located at a distance L from the center of a mirror-image sink as shown in Fig. 1. For most cases, the distance L , between the center of the wells, will be determined by the location of the ground-water level above the tunnel. The tunnel (source) and the mirror-image sink can be considered to be equidistant from the existing ground-water level prior to excavation. Thus the distance, L , can be approximated as twice the depth of the tunnel below the ground-water level, h_o .

The flow net that develops under steady-state seepage between the two wells is also shown in Fig. 1. The lower part of this flow net shows the flow lines between the tunnel and the existing ground-water level. As indicated in Fig. 1, these flow lines intersect the equipotential line established by the

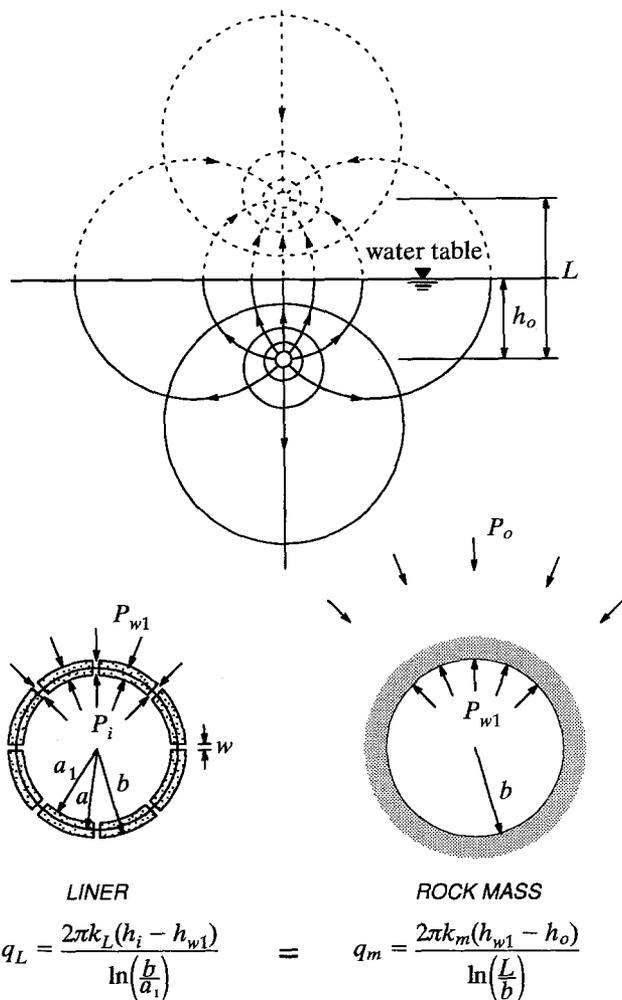


FIG. 1. Hydraulic Model for Estimation of Rate of Exfiltration and Superposition of Flow through Liner to Flow into Rock Mass

existing ground-water level at an angle of 90°. Although this flow model strictly applies to a horizontal water table, it can be used for tunnels excavated under an inclined slope, assuming that the water table is parallel to the slope, provided that the tunnel is located at a distance of at least 20 radii from the valley walls. If a drainage gallery is drilled parallel to the tunnel, the value of L can be adjusted and made equal to the center to center distance between the pressurized tunnel and the gallery, but the net driving head used to compute the inflows into the gallery is $(h_i - h_0)/2$, where h_i is the hydraulic head inside the tunnel.

Pore-Water Pressure Distribution

The pore-water pressure distribution in the rock mass surrounding the tunnel can be evaluated from the equation defining the equipotential lines in the flow net of Fig. 1, as proposed by Harr (1962). Based on this equation, the excess pore-water pressure induced by the internal tunnel pressure at any point around the opening was estimated by Fernández and Alvarez (1994) as

$$P_w(r, \theta) = \Delta P_w \frac{\ln \left(1 + \frac{L^2}{r^2} - 2 \frac{L}{r} \cos \theta \right)}{\ln \left(1 + \frac{L^2}{b^2} - 2 \frac{L}{b} \cos \theta \right)} \quad (1)$$

where $\Delta P_w = \gamma_w(h_i - h_0)$ = net driving pressure in excess of hydrostatic; γ_w = unit weight of water; h_i = hydraulic head inside the tunnel; h_0 = depth of the tunnel below the ground-water level; r = distance from the center of the tunnel to the point where the pore-water pressure is to be estimated; b = excavated radius of the tunnel; L can be estimated as twice the depth of the tunnel below the ground-water level unless a drainage gallery is excavated parallel to the tunnel; and θ = clockwise angle between a vertical line passing through the center of the tunnel and the radius to the point of interest.

Along the springline, (1) simplifies to

$$P_w(r) = \Delta P_w \frac{\ln \left(1 + \frac{L^2}{r^2} \right)}{\ln \left(1 + \frac{L^2}{b^2} \right)} \quad (2)$$

The total water pressure along the springline of the tunnel corresponds to the summation of the excess pore-water pressure described previously, plus the initial water pressure, $P_0 = \gamma_w h_0$, generated by the original ground-water level.

The rate of decay of the excess pore-water pressures away from the tunnel walls depends mainly on the magnitude of L/b . For relatively small values of L/b , the excess pore-water pressure decays very rapidly and becomes almost negligible within a relatively short distance away from the tunnel walls. In this case, the hydraulic gradient in the vicinity of the walls is high. On the other hand, for large values of L/b , the excess pore-water pressure decays at a much slower rate and the hydraulic gradient in the vicinity of the tunnel walls is small. In most power tunnels the initial location of the

groundwater level will not be significantly changed by the leakage out of the tunnel and thus the magnitude of L/b will remain constant during operations.

As a point of reference the distance r_{50} , from the center of the tunnel to the point where 50% of the excess pore-water pressure has been dissipated can be approximated by making $P_w = 0.5 \Delta P_w$ in (2) and solving for the radius, which results in:

$$r_{50} = \sqrt{Lb} \quad (3)$$

Rate of Flow Out of Tunnel

The estimated rate of exfiltration per unit length of tunnel based on the approach proposed by Harr (1962) can be approximated as

$$q_m = k_m i A \quad (4a)$$

where q_m = flow rate into the rock mass per unit length of tunnel; k_m = permeability of the rock mass; i = hydraulic gradient across a cylindrical element of thickness, dr , coaxial with the tunnel; and A = perimeter area of the cylindrical surface. Since the flow rate is the same across similar sections located at different radial distances from the center of the tunnel, i and A can be conveniently evaluated at the walls of the tunnel.

The pressure gradient along the springline, dP_w/dr , corresponding to the flow regime of Fig. 1, from an opening of radius b , has been defined by Fernández and Alvarez (1994) as

$$\frac{dP_w}{dr} = \frac{-\frac{1}{r} \Delta P_{w1} \frac{2L^2}{r^2}}{\left(1 + \frac{L^2}{r^2}\right) \ln \left(1 + \frac{L^2}{b^2}\right)} \quad (4b)$$

and the hydraulic gradient is related to the pressure gradient as follows:

$$i = \frac{1}{\gamma_w} \frac{dP_w}{dr} \quad (4c)$$

At the excavated rock wall, $r = b$, the area is $A = 2\pi b$, and assuming that $L/b \gg 1$, the hydraulic gradient is

$$i = -\frac{1}{\gamma_w} \frac{\Delta P_{w1}}{b \ln \frac{L}{b}} \quad (4d)$$

where $\Delta P_{w1} = P_{w1} - P_0 = \gamma_w \Delta h_{w1}$ = pressure loss across the rock mass, and $\Delta h_{w1} = (h_{w1} - h_0)$; and h_{w1} = hydraulic head at the rock-liner boundary, as shown in Fig. 2.

Substituting (4d) into (4a) the magnitude of the rate of flow into the rock mass is

$$q_m = \frac{2\pi k_m \Delta h_{w1}}{\ln \frac{L}{b}} \quad (4e)$$

For an unlined tunnel $h_{w1} = h_i$ and $q_m = q_0$; therefore

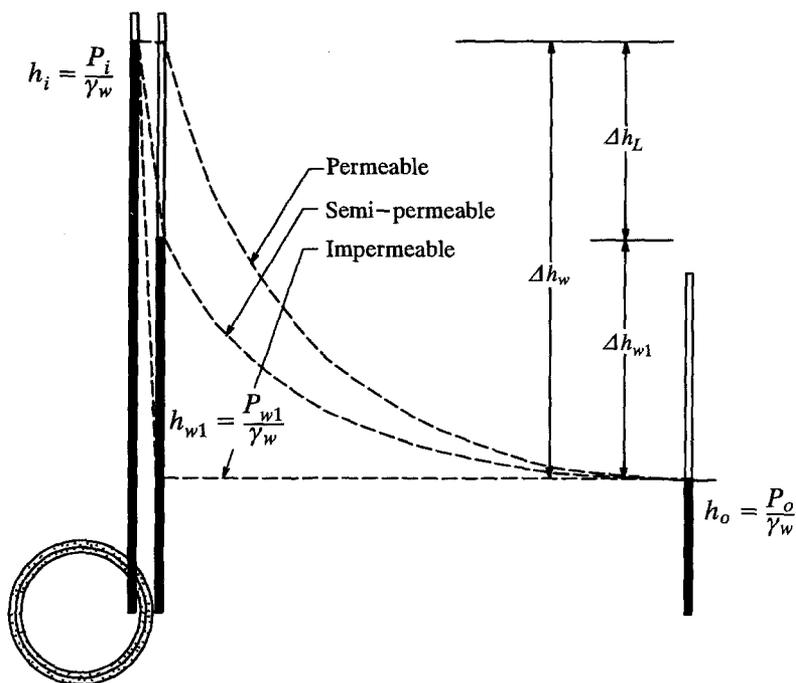


FIG. 2. Head Losses Across Liner and Surrounding Medium

$$q_0 = \frac{2\pi k_m (h_i - h_o)}{\ln \frac{L}{b}} \quad (4f)$$

Eqs. (4e) and (4f) are similar to those proposed by Bouvard and Pinto (1969) and by Schleiss (1986), but do not depend on an undefined reach of flow at which the effects of seepage become negligible. In (4e) and (4f) the reach of flow is given by the location of the water table.

The magnitude of the dimensionless term in the denominator ranges from a value of 3 for L/b ratios in excess of 20 to a value of 1 for L/b ratio of the order of 3.

Eq. (4) can also be used to estimate the permeability of a rock mass, k_m , during excavation by establishing the initial location of the groundwater level and measuring the infiltration rate, q_0 , per unit length of tunnel as the tunnel is advanced.

Structural Behavior of Lined Pressure Tunnels

In a lined tunnel the rate of leakage and the pore-water pressure distribution in the surrounding rock mass are determined not only by the permeability of the rock mass surrounding the tunnel, k_m , but also by the permeability, k_L , of the liner.

Thus, the hydraulic and mechanical interaction between the liner and the surrounding rock mass has to be taken into account to estimate the leakage as well as the loss of hydraulic head across the liner. Once these two parameters are determined, the hydraulic conditions in the surrounding rock

mass can be estimated as those of an unlined tunnel with a slightly larger radius, that includes the thickness of the liner, and a reduced internal hydraulic pressure that takes into account the head losses across the liner.

If a liner cracks such that its permeability increases by several orders of magnitude and the permeability of the materials around the opening is low, the magnitude of the hydraulic pressure at the liner-rock boundary will be close to the internal tunnel pressure; and the rate of flow will be determined by the permeability of the surrounding medium. On the other hand, if a relatively impermeable liner (steel, uncracked concrete) is placed against a permeable medium, the hydraulic pressure immediately outside the liner will be determined by the ground-water level in the area; and the rate of flow out of the liner will be negligible.

An analysis of the interaction between the liner and the surrounding medium needs to be carried out to evaluate the rate of leakage and the hydraulic pressure, P_{w1} , that builds up at the liner-rock contact. The analysis is based on the superposition of the behavior of the liner and the excavated opening in the surrounding rock as shown in Fig. 1.

As the pressure is applied inside the tunnel, the liner and the surrounding ground tend to expand inducing an outward displacement of the tunnel walls. During the initial loading, the radial displacements of the liner and the rock are equal at the liner-rock boundary, and the magnitude of the displacement is determined by the mechanical interaction of the liner and the surrounding rock. Under these conditions part of the net tunnel pressure, $\Delta P_w = (P_i - P_0)$, is transferred to the surrounding rock mass and part is absorbed by the liner. Thus the outward displacement of the liner is a function not only of thickness and elastic properties of the liner materials, but also of the stiffness of the surrounding rock mass. The magnitude of the outward displacement of the tunnel walls determines the tensile strain level in the liner. In a concrete liner, the magnitude of the tensile strain, in turn, controls the degree of cracking and thus the permeability of the liner.

Simultaneously with the outward expansion of the liner, the net tunnel pressure also generates a hydraulic gradient which tends to drive the water through the liner and into the surrounding medium. The continuity of flow requires that the rate of leakage out of the liner be equal to the rate of flow into the surrounding rock mass. This condition is achieved once the hydraulic pressure outside the liner, P_{w1} , reaches an "equilibrium" value, which is determined by the relative permeability of the liner with respect to the permeability of the surrounding rock mass.

Once the equilibrium of flow is reached, the hydraulic pressure, P_{w1} , at the liner-rock contact applies a uniform, all-around pressure along the outside perimeter of the liner as well as against the excavated rock walls. If the magnitude of the increase in hydraulic pressure, $\Delta P_{w1} = P_{w1} - P_0 = \gamma_w(h_{w1} - h_0)$, is lower than the pressure transferred to the rock during the initial loading, the liner and surrounding rock mass will remain in intimate contact and the tensile strain and thus the permeability of the liner will remain unchanged after the flow-equilibrium condition is established.

However, if the increase in hydraulic pressure, ΔP_{w1} , outside the liner exceeds the pressure initially transferred to the rock, a gap will develop across the liner-rock boundary, because the hydraulic pressure will push the liner away from the excavated rock walls. The pressure on the outside perimeter of the liner will reduce the initial tensile strain induced before the gap developed, reducing the permeability of the liner and decreasing

the initial rate of leakage. The magnitude of the hydraulic pressure outside the liner, P_{w1} , would then subsequently adjust to satisfy continuity of the new flow regime. Thus, these considerations clearly indicate that the hydraulic (continuity of flow) and mechanical (compatibility of displacements) interaction of the liner and the surrounding rock mass are mutually dependent.

An analysis is presented here to approximate the hydraulic and mechanical liner-ground interaction separately and then combine the effect of the two interactions to approximate the rate of leakage and to estimate the induced hydraulic pressure outside typical concrete liners.

Hydraulic Liner-Ground Interaction

The continuity of flow at the liner-rock boundary can be established, as shown next. The rate of flow across the liner can be estimated, as indicated by Goodman (1980), and for most cases is given by

$$q_L = \frac{2\pi k_L \Delta h_L}{\ln \frac{b}{a_1}} \quad (5)$$

where q_L = flow rate across the liner per unit length of tunnel; k_L = permeability of the liner; Δh_L = hydraulic head loss across the liner; and b and a_1 = outside and inside radii of the liner, respectively.

The head loss across the liner, Δh_L , can be expressed as $\Delta h_L = h_i - h_{w1}$, where h_i is the hydraulic head inside the tunnel and h_{w1} is the hydraulic head at the rock-liner boundary as shown in Fig. 2.

The rate of flow into the rock mass can be estimated using (4e); therefore the continuity of flow at the liner-rock boundary can be established by making the flows given by (4e) and (5) equal. In addition, the hydraulic interaction of the two systems requires that the net hydraulic head driving the water out of the tunnel, $\Delta h_w = h_i - h_0$, be equal to the sum of the hydraulic head loss across the liner, Δh_L , plus the hydraulic head loss, Δh_{w1} , within the rock mass as

$$\Delta h_w = \Delta h_L + \Delta h_{w1} \quad (6)$$

This condition indicates that for a given value of h_i and h_0 the continuity of flow is achieved at a unique value of the hydraulic head, h_{w1} , at the liner-rock boundary. Combining (4e), and (5), and taking into account the head loss considerations in (6), the normalized loss of hydraulic head across the liner can be obtained as

$$\frac{\Delta h_L}{\Delta h_w} = \frac{1}{1 + C \frac{k_L}{k_m}} \quad (7)$$

where

$$C = \frac{\ln \frac{L}{b}}{\ln \frac{b}{a_1}} \quad (8)$$

The values of C generally range from 10 to 50 for most pressure tunnels.

The hydraulic head outside the liner, h_{w1} , is obtained from (7) as

$$h_{w1} = h_i - \frac{\Delta h_w}{1 + C \frac{k_L}{k_m}} \quad (9)$$

A graphical representation of the normalized loss of the hydraulic head across the liner as a function of the ratio, b/a_1 , (outer over inner liner radius) for different values of the permeability ratio, k_L/k_m , is shown in Fig. 3.

In common engineering practice the thickness of the tunnel liners is such that the ratio b/a_1 , usually ranges between 1.1 and 1.2. Thus, the relationships in Fig. 3 indicate that for relatively impermeable liners, with k_L/k_m approximately equal to 1/80 to 1/100, the hydraulic head loss across the liner can be about 80–90% of the net hydraulic head. For semipermeable liners with a permeability close to 1/20 to 1/10 times the permeability of the rock mass, the hydraulic head loss across the liner can be approximately 50% of the net hydraulic head. For liners with permeabilities similar to those of the surrounding rock mass the loss of head across the liner probably will not exceed 5% of the net hydraulic head. The reduction of the hydraulic head across the liner has a significant impact on cases where potential slope instabilities may be triggered by an increase in the porewater pressure within the rock mass around the tunnel as well as in cases where the potential exists for hydraulic fracturing or hydrojacking to develop along existing rock-mass discontinuities.

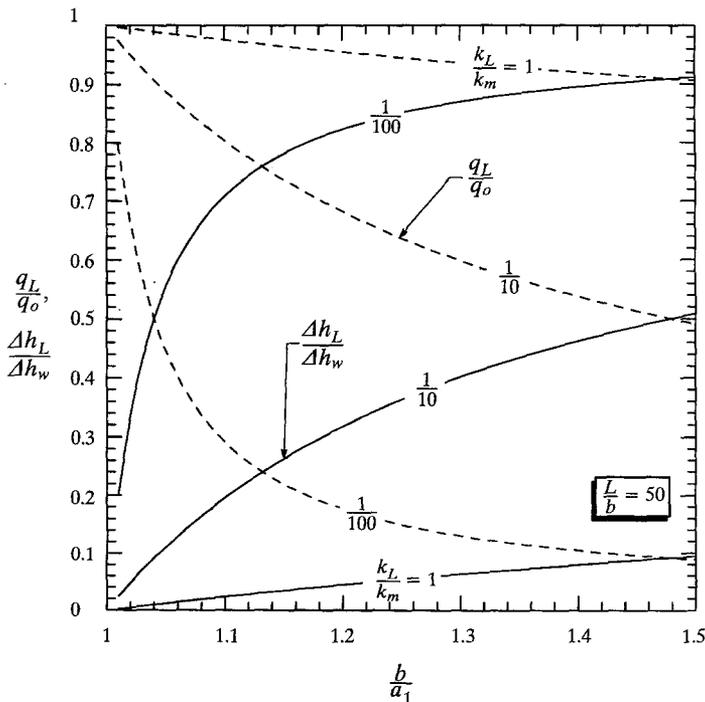


FIG. 3. Hydraulic Head Loss Across Liner and Ratio of Flow Rates of Lined over Unlined Tunnel versus Liner Thickness for Different Rock-Mass Permeabilities

The beneficial effect of any given liner in reducing the rate of flow out of the tunnel can be evaluated based on the ratio of the flow out of the lined tunnel over the flow that would occur if the tunnel is left unlined. Based on (4f), (5), and (7), this ratio can be estimated as

$$\frac{q_L}{q_0} = 1 - \frac{1}{1 + C \frac{k_L}{k_m}} \quad (10)$$

A graphical representation of the ratio of flow rates q_L/q_0 as a function of the thickness of the liner (in terms of the b/a_1 ratio) for different values of the permeability ratio, k_L/k_m , is also shown in Fig. 3. In most tunnels, the use of relatively "impermeable" liners with permeabilities of about 1/100 times the permeability of the surrounding rock mass can reduce the rate of leakage to 10% of the rate of leakage that would occur if no liner is installed. "Semipermeable" liners with permeabilities of about 1/20 of the permeability of the surrounding rock mass can reduce the rate of leakage to 50–60% of the rate of leakage that would occur if the tunnel is left unlined.

Mechanical Liner-Ground Interaction

During the initial loading, before the steady-state seepage regime is established, it can be safely assumed that an intimate contact exists between the liner and the surrounding rock mass and thus the pressure-induced radial displacements in both media are equal at the liner-rock boundary. A thorough contact grouting program should be implemented to ensure proper contact before pressurization. Under these conditions, the resulting circumferential strain, $\varepsilon_{\theta L}$, in the liner, assuming an elastic behavior in both the liner and the surrounding medium, can be established as

$$\varepsilon_{\theta L} = \frac{\Delta P_w (1 + \nu_m)}{M E_m} \quad (11a)$$

$$M = 1 + \frac{E_L t_L}{E_m a} (1 + \nu_m) \quad (11b)$$

where E_m = "average" rock modulus; ν_m = Poisson's ratio of the rock mass; E_L = modulus of elasticity of the liner; t_L = effective thickness of the liner; and a = radius to the center of the liner.

In a plain concrete liner, with no longitudinal cracks, E_L is the modulus of concrete materials and $t_L = t$, the average thickness of the liner. If longitudinal cracking of the plain concrete occurs, E_L and t_L become zero. In cracked, reinforced concrete liners, $E_L = E_s$, the modulus of elasticity of the reinforcement; and t_L is equal to t_s , where t_s is the equivalent thickness of steel, corresponding to the amount of reinforcement in the liner and can be estimated as $t_s = \rho t$, where ρ is equal to A_s/A_c , and t is the thickness of the liner. This latter case neglects the beneficial effect of tension stiffening of the longitudinally cracked reinforced concrete liner and is therefore conservative.

In most tunnels the rock mass in the immediate vicinity of the walls is loosened during the excavation process. The average rock modulus in (11) is the combined modulus of the loosened zone as well as the undisturbed rock mass behind, and can be estimated as proposed by Jaeger (1972). An

alternative method to determine the circumferential strain is given by Hendron et al. (1987). Both methods yield similar strain values.

The share of the internal pressure initially transferred to the surrounding rock mass, P_r , can be estimated as

$$P_r = \frac{\Delta P_w}{M} \quad (12)$$

Prior to the development of the steady-state flow regime, the mechanical pressure, P_r , across the liner-rock boundary can be considered to be an effective stress. As leakage takes place, the additional hydrostatic pressure, $\Delta P_{w1} = P_{w1} - P_0$, that builds up at the liner-rock contact reduces the effective stress, but contact across the liner-rock boundary will be preserved as long as the value of ΔP_{w1} is smaller than P_r . Under these conditions, the share of the internal tunnel pressure absorbed by the liner can be estimated as $(\Delta P_w - P_r)$. However, if the leakage-induced hydrostatic pressure ($P_{w1} - P_0$) at the liner-rock boundary exceeds the value of P_r in (12), the liner will separate from the surrounding rock walls. The share of the internal tunnel pressure absorbed by the liner after the gap is formed is equal to $\Delta P_L = P_i - P_{w1} = \gamma_w(h_i - h_{w1})$. Based on (9) and (12), it can be shown that the gap develops for the permeability ratios, k_L/k_m , equal or higher than the critical value

$$\left(\frac{k_L}{k_m}\right)_{\text{crit}} = \frac{1}{(M - 1)C} \quad (13)$$

Coupled Hydraulic and Mechanical Interactions of Concrete Liners and Surrounding Ground

The permeability of concrete liners depends on the quality of the concrete mix but is mainly controlled by the magnitude of the tensile circumferential strain, $\varepsilon_{\theta L}$, induced by the internal tunnel pressure. The circumferential strain, $\varepsilon_{\theta L}$, in turn is controlled by the hydraulic and mechanical interaction between the liner and the surrounding rock. A detailed description of the various steps involved to estimate liner permeability is given next. In addition, two nomograms have been developed to facilitate concrete liner design in pressure tunnels.

Liner Permeability

Field infiltration measurements in a 9.7-km-long (6-mi-long) and 3.3-m-diameter (10-ft-diameter) concrete lined tunnel under no internal pressure resulted in an average liner permeability of about 1.7×10^{-6} cm/s (5.7×10^{-8} ft/sec). Circular shrinkage cracks, spaced 3.0–4.6 m (10–15 ft) along the axis were present in this liner. Because the permeability of the rock mass surrounding the tunnel was a few orders of magnitude larger than the concrete liner it can be safely assumed that the measured infiltrations reflect the permeability of the liner. Since the opening of the annular cracks is not likely to increase significantly during pressurization, the permeability value measured in this tunnel can be considered to be typical of concrete liners with shrinkage cracks and tensile circumferential strains lower than 1.5×10^{-4} where pressure-induced longitudinal cracking of the liner has not developed. For concrete liners, where the pressure-induced tensile strain, $\varepsilon_{\theta L}$, exceeds 1.5×10^{-4} , longitudinal cracking of the liner will take place and in most cases the permeability of the liner k_L will no longer be controlled

by annular shrinkage cracks but will be determined by the width of the longitudinal cracks.

Assuming a laminar flow regime concentrated along longitudinal cracks of aperture (w), evenly distributed around the perimeter of the liner, the equivalent permeability of the cracked concrete liner can be estimated using the cubic law for viscous flow between parallel plates (Stagg and Zienkiewicz 1968) as follows:

$$k_L = \frac{\gamma_w}{12\mu} \frac{w^3}{S} \quad (14a)$$

where γ_w = unit weight of water; w = average crack width; S = average spacing between cracks; and μ = dynamic viscosity of the water [equal to 10^{-3} Pa·s (2.1×10^{-5} psf·sec)] at 20°C.

The average crack width, w , that develops in the liner can be expressed as

$$w = \varepsilon_{\theta L} S \quad (14b)$$

Thus, the permeability of the liner can be simplified to

$$k_L = \alpha \varepsilon_{\theta L}^3 \quad (14c)$$

where

$$\alpha = \frac{\gamma_w}{12\mu} S^2 \quad (14d)$$

In unreinforced concrete liners, only two cracks typically develop parallel to the plane of minimum in situ stress, and thus the crack spacing can be estimated as $S = \pi a_1$; where a_1 is the inside radius of the tunnel. The average spacing, S , between cracks in reinforced-concrete members acting under tension was evaluated by Rizkalla and Hwang (1984) and their expression simplified by Hendron et al. (1987) as follows:

$$S = \frac{d}{10\rho} \quad (15)$$

where d = diameter of the reinforcing bar; and ρ = ratio of the area of steel to the area of concrete.

A graphical representation of the various steps involved in the design of a pressure-tunnel concrete liner, taking into account the hydraulic and mechanical ground-liner interaction is given in the nomograms of Figs. 4 and 5.

1. The initial tensile strain in the liner, $\varepsilon_{\theta L}$, is estimated from (11a), assuming compatibility of displacements at the rock-liner boundary. The calculated tensile strain value, $\varepsilon_{\theta L}$, is entered in the horizontal axis of quadrant 1 in nomogram 1 to determine the initial permeability of the liner, k_L , which is given in the vertical axis. The amount and distribution of reinforcement is taken into account by using the appropriate d/ρ ratio curve. The values of k_L in the nomogram were estimated using (14c) if the magnitude of $\varepsilon_{\theta L}$ is larger than 1.5×10^{-4} and longitudinal cracking of the liner takes place. For strains, $\varepsilon_{\theta L}$, equal or lower than 1.5×10^{-4} a liner permeability of 10^{-6} cm/s was assigned because longitudinal cracking is not likely to take place.

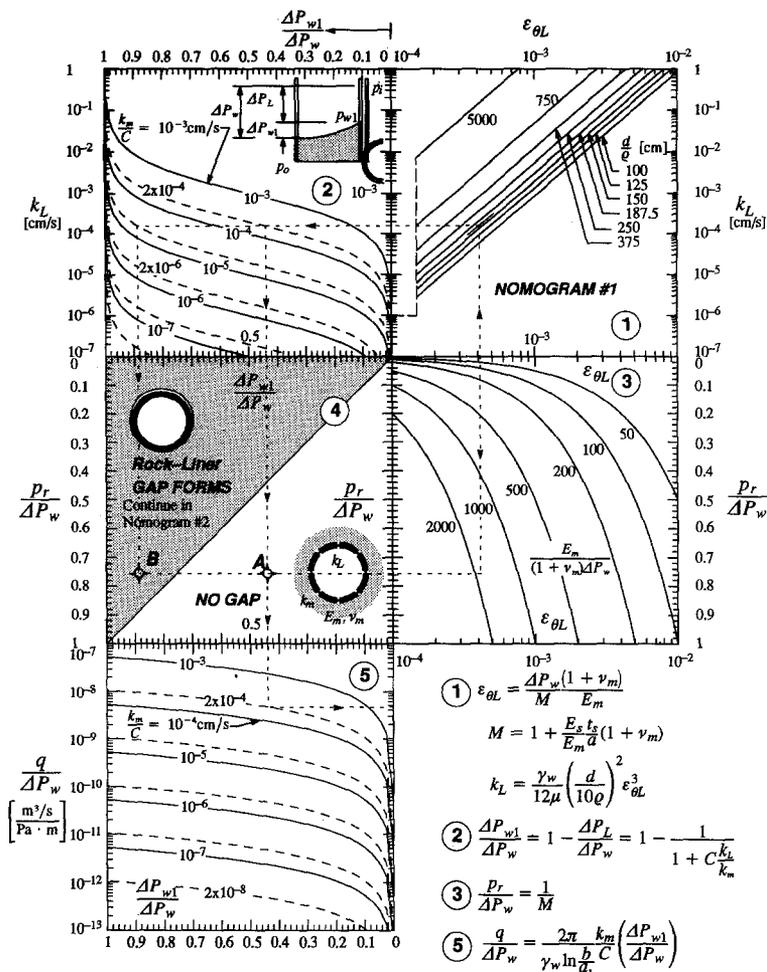


FIG. 4. Nomogram 1 to Estimate Leakage and Pore Pressure Outside Pressurized Tunnel Liners

2. Once the liner permeability, k_L , is established, this value is entered in the vertical axis of quadrant 2 to determine the magnitude of the additional hydraulic pressure, $\Delta P_{w1} = P_{w1} - P_0 = \gamma_w (h_{w1} - h_0)$, developed at the liner-rock boundary under steady-state flow, for different permeability values, k_m , of the surrounding medium. Normalized values of $\Delta P_{w1} / \Delta P_w$, where $\Delta P_w = P_i - P_0$, are given in the horizontal axis of quadrant 2. Then ΔP_{w1} values were estimated from (9).

3. The normalized rock pressure, $P_r / \Delta P_w$, transferred to the surrounding medium during initial loading is estimated entering the initial strain value, $\epsilon_{\theta L}$, in the horizontal axis of quadrant 3 and choosing the curve corresponding to the appropriate modulus of the surrounding mass. It was estimated from (12).

4. The normalized value of the additional hydraulic pressure, $\Delta P_{w1} / \Delta P_w$,

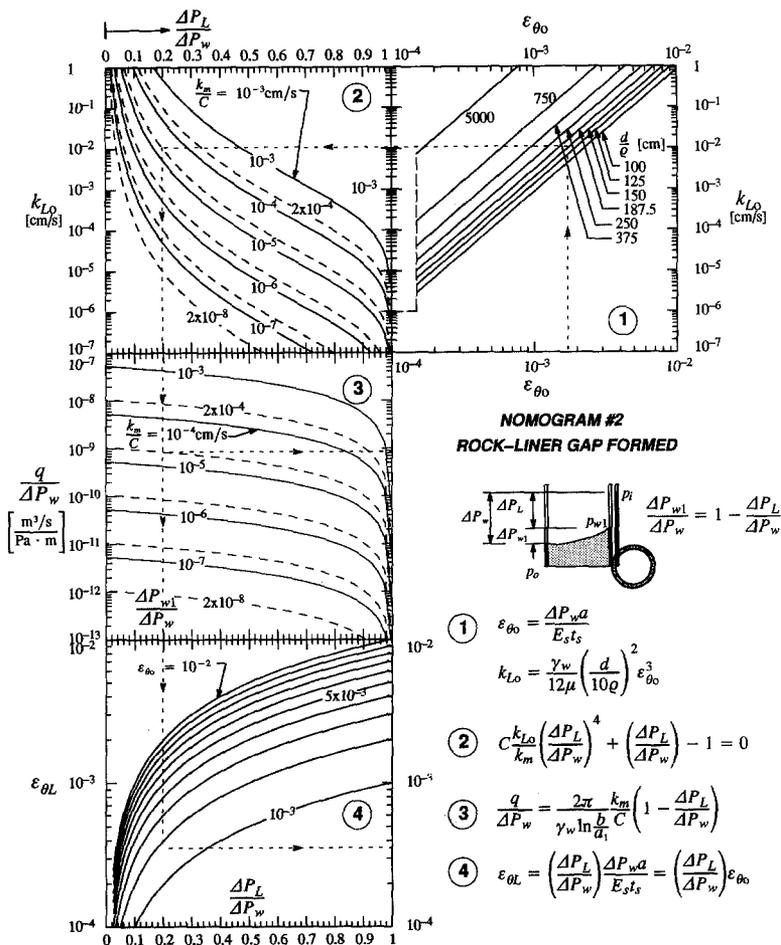


FIG. 5. Nomogram 2 to Estimate Leakage and Pore Pressures Outside Liners with Gaps at Rock-Liner Boundary

induced after steady-state flow develops is then compared in quadrant 4 with the normalized rock pressure, $P_r/\Delta P_w$, initially transferred to the surrounding medium. The liner will remain in intimate contact with the surrounding medium if the normalized rock pressure, $P_r/\Delta P_w$, is larger than the normalized hydraulic pressure $\Delta P_{w1}/\Delta P_w$, which corresponds to all points plotting below the 45° line in quadrant 4. In this case, the steady-state hydraulic pressure at the liner-rock boundary is P_{w1} , from (9), and the effective radial stress at the liner-rock boundary will be equal to $P_r - \Delta P_{w1}$. The ratio of $\Delta P_{w1}/\Delta P_w$ is an index that can be used to evaluate the beneficial effects of the liner in reducing the potential for hydrofracturing or hydro-jacking of the surrounding rock mass. The amount of leakage, per unit length of tunnel, can be estimated from (6d) for different permeability values of the surrounding medium and can be obtained from chart 5 in nomogram

1. For the no-gap case, the initial liner permeability, k_L , remains constant during operations.

5. On the other hand, if the magnitude of the additional hydraulic pressure, ΔP_{w1} , exceeds the magnitude of the rock pressure, P_r , initially transferred to the surrounding medium, then a gap develops across the liner-rock boundary. This case corresponds to all points plotting above the 45° line in quadrant 4 in Fig. 4. The radial displacements of the rock and the liner cease to be compatible across the rock-liner boundary once the gap develops, and the circumferential strain in the liner, $\varepsilon_{\theta L}$, decreases after the formation of the gap. If the concrete liner had not initially cracked during pressurization, the reduction in the liner's circumferential strain level, $\varepsilon_{\theta L}$, does not change the permeability of the liner, k_L , and thus, the estimated hydraulic pressure, P_{w1} , is the actual pressure in the gap across the liner-rock boundary. However, if the concrete liner had cracked during initial pressurization the reduction in the liner's circumferential strain, $\varepsilon_{\theta L}$, will reduce the width of the concrete cracks and thus it will also reduce the permeability, k_L , of the liner. The magnitude of the steady state hydraulic pressure outside the liner, P_{w1} , would in turn change to satisfy continuity of flow under the reduced permeability of the concrete liner.

6. The steady-state value of the hydraulic pressure outside the liner, P_{w1} , once the gap develops, can be obtained by combining (9) with the equation corresponding to the reduced value of the circumferential strain, $\varepsilon_{\theta L}$, once the gap develops. The circumferential strain, $\varepsilon_{\theta L}$, after the gap can be represented as

$$\varepsilon_{\theta L} = \frac{\Delta P_L a}{E_s t_s} \tag{16}$$

The combination of (9) and (16) results in the following relationship:

$$C \frac{k_{L0}}{k_m} \left(\frac{\Delta P_L}{\Delta P_w} \right)^4 + \frac{\Delta P_L}{\Delta P_w} - 1 = 0 \tag{17}$$

The term, k_{L0} , represents an upper boundary of the permeability of the liner, for a fully pressurized tunnel with only the ground-water level pressure, P_0 , acting outside the perimeter of the liner, and can be estimated as

$$k_{L0} = \alpha \left(\frac{\Delta P_w a}{E_s t_s} \right)^3 \tag{18}$$

where α was introduced in (14d); and all other terms have been previously defined.

The solution to (17) is given in the nomogram of Fig. 5. In this nomogram, an initial strain, $\varepsilon_{\theta 0}$, corresponding to the fully pressurized liner with an outside ground-water pressure, P_0 , is estimated and entered in the horizontal axis of quadrant 1. This initial strain, $\varepsilon_{\theta 0}$, is equal to the term within parentheses in (18). The corresponding liner permeability can be read in the vertical axis of the same quadrant, choosing the appropriate curve for the corresponding level of reinforcement d/ρ .

Once the magnitude of k_{L0} is obtained, the normalized hydraulic pressure, $\Delta P_{w1}/\Delta P_w$, generated by the steady-state flow after gap development, can

be read in the horizontal axis of quadrant 2. The normalized hydraulic pressure, $\Delta P_{w1}/\Delta P_w$, is in turn entered in the horizontal axis of quadrant 3 to determine the rate of leakage out of the tunnel for various permeabilities of the surrounding media.

The final strain in the liner, $\varepsilon_{\theta L}$, after steady-state flow conditions are reached is given by (16) and is obtained in the lower part of the nomogram of Fig. 5. Because the magnitude of the radial displacements induced at the tunnel walls by mechanical or net hydraulic pressure of equal magnitudes are the same (Fernández and Alvarez 1994), the circumferential strain in the excavated rock walls, $\varepsilon_{\theta r}$, can be calculated as

$$\varepsilon_{\theta r} = \Delta P_{w1} \frac{(1 + \nu_m)}{E_m} \quad (19)$$

and the width of the gap between the liner and the surrounding rock is approximated as the difference between the radial displacements as

$$\delta_g = b\varepsilon_{\theta r} - a\varepsilon_{\theta L} \quad (20)$$

The proposed design approach is illustrated in the nomograms of Figs. 4 and 5 with the example of a power tunnel excavated in a rock mass with two shear zones of large permeability ranging from 6.1×10^{-3} cm/s (2×10^{-4} ft/sec) (case A) to 6.1×10^{-4} cm/s (2×10^{-5} ft/sec) (case B). An average Young's modulus of 2,070 MPa (300,000 psi) and a Poisson's ratio, $\nu_r = 0.3$ were assumed for both shear zones. The tunnel has an excavated radius of 2.7 m (106 in.) and is to be lined with a 0.3-m-thick (12 in.-thick) concrete liner, reinforced with rings of No. 11 steel bars spaced 0.15 m (6 in.) along the tunnel axis. The internal tunnel pressure, P_i , is equal to 1.378 MPa (200 psi) and the ground-water level is located 52 m (170 ft) above the springline of the tunnel.

As indicated in Fig. 4, in the more permeable shear zone (case A) the liner remains in contact with the surrounding media after a steady-state flow is established, and the pressure losses across the liner, ΔP_L , correspond to around 56% of the net hydraulic pressure, ΔP_w . The normalized rate of leakage for case A can then be directly obtained from the lower plot in the nomogram of Fig. 4, and is estimated as 4.7×10^{-9} m³/s/(Pa·m), which corresponds to 4.1×10^{-3} m³/s/m (20 gpm/ft). This rate of leakage is equal to around 44% of the flow without a liner. On the other hand in the less permeable shear zone (case B), a gap develops at the liner-rock boundary, and the use of the nomogram 2 in Fig. 5 is required. As indicated in the nomogram, Fig. 5, in the less permeable zone (case B) the pressure losses across the liner correspond to only 20% of the net hydraulic pressure, ΔP_w . The normalized rate of leakage can be directly obtained from quadrant 3 in Fig. 5 and are equal to 8.6×10^{-10} (m³/s/Pa·m), which corresponds to 7.5×10^{-4} m³/s/m (3.6 gpm/ft). This flow is equal to 80% of the flow without a liner. In addition the final strain in the liner is 80% of the strain in case A.

As indicated in the nomograms of Figs. 4 and 5, the hydraulic losses across a reinforced concrete liner are large for tunnels excavated in relatively permeable media. The reduction of the hydraulic pressure outside the liner reduces the potential for hydrofracturing and/or hydrojacking of the surrounding media, and decreases the rate of leakage out of the tunnel.

As the permeability of the surrounding mass decreases, the beneficial effect of a reinforced-concrete liner in reducing the rate of leakage becomes less significant and eventually in a relatively tight rock mass the increase in

hydraulic pressure, ΔP_{w1} , behind a concrete liner is large enough to generate a gap at the liner-rock boundary, and reduce the permeability of the liner to values similar to those of the surrounding rock mass.

However, reinforcement provides an additional, beneficial effect (not previously emphasized in the literature) when the hydraulic pressure, ΔP_{w1} , behind the liner is just large enough to start opening up existing discontinuities within the rock mass (hydrojacking). Under these conditions, the permeability of the rock mass, k_m , tends to increase very rapidly by several orders of magnitude; however the presence of reinforcement precludes an uncontrolled increase in the permeability of the concrete liner. Thus, the value of the permeability ratio k_L/k_m is maintained low, resulting in significant head losses across the liner, which in turn limits the propagation of the hydrojacking phenomenon.

Alternatively, as the permeability of the rock mass increases with respect to that of the liner, the exfiltration rate is largely controlled by the permeability of the liner, which acts as a relatively impermeable membrane with respect to the surrounding media. Thus, the presence of reinforcement also limits the flow rate that exfiltrates into the rock mass around the opening, limiting the propagation of the hydrojacking phenomena. It is important to point out that this beneficial effect of reinforcement is especially significant in long tunnel sections excavated in areas where the in-situ stresses are marginal. A properly designed reinforced concrete liner can maintain the hydraulic pressure outside the liner comfortably below the minimum in-situ stress of the surrounding rock mass.

The nomograms also indicate the desirability to minimize loosening of the rock materials around the tunnels during excavation, to prevent large reductions in the stiffness of the surrounding medium. If substantial loosening occurs, the use of reinforcement in the liner or consolidation grouting in the crown should be considered.

Leakage estimates given in the nomograms of Figs. 4 and 5 correspond to a steady-state flow that develops sometime after the initial filling. Field measurements indicate that the initial leakage, Q_i , during filling can be significantly larger than the final leakage that develops under steady-state flow. An upper bound estimate of the initial leakage, Q_i , can be obtained following the same procedure described previously but assuming the external water pressure, $P_0 = 0$, and using a value of L corresponding to the final groundwater elevation, h_0 , which will be reached under steady-state flow.

DESIGN GUIDELINES FOR PRESSURE TUNNELS

A summarized approach to develop a preliminary design of pressurized conduits can be outlined as follows.

Location of Ground-Water Level

Once a tunnel alignment is chosen, the ground-water level should be estimated along the profile and compared with the hydraulic head generated by the internal tunnel pressure. At those sections where the ground-water level is higher, water will tend to infiltrate into the tunnel increasing the flow in the pressurized conduit. Linear requirements under those conditions mainly depend on the type and quality of the excavated materials as well as on the hydraulic performance of the tunnel. Plain concrete liners are commonly used to line tunnels in materials susceptible to erosion or deterioration and are also used to improve the hydraulic characteristics of tunnels

excavated in any type of rock. A slow rate of loading is highly recommended to allow a simultaneous recharge of the ground-water level, and thus minimize the development of longitudinal cracks in the liner. Shrinkage reinforcement is recommended in liners installed in tunnels excavated in rocks susceptible to dissolution where the recommendation of a slow rate of loading is especially emphasized. Rock bolts, mesh, and shotcrete can be used to support hard, fractured rock materials where isolated rock blocks can become unstable.

Estimate of Hydrojacking and/or Hydrofracturing Potential

At those locations where the groundwater table is located below the hydraulic head inside the tunnel, leakage can occur out of the pressurized conduit. The zones of potential leakage can be divided in two main categories: (1) Those sections where hydrojacking or hydrofracturing of the rock mass can occur; and (2) those sections where the in-situ stress in the rock mass is high enough to preclude these phenomena. Sections of potential hydrojacking or hydrofracturing of the rock mass can be conservatively established using available empirical criteria (Broch 1984; Fernández and Alvarez 1994). After excavation, hydraulic fracturing tests can be carried out at strategically selected locations to confirm the boundaries of potential hydrofracturing zones chosen on empirical rules. Once these boundaries are established, the rock treatment, and/or type of liners to be used in each section can be treated separately.

Zones of High Hydrojacking and/or Hydrofracturing Potential

A special liner is required for those locations where the internal tunnel pressure is close or exceeds the in-situ stress in the rock mass around the excavation. According to most of the existing criteria a steel liner must be installed at all locations where the minimum in-situ stress is below 1.3 times the internal tunnel pressure under normal operation (Benson 1986; Brekke and Ripley 1986). These criteria have shown to be adequate in the vicinity of most free surfaces where the rock is fairly massive and stress relief is not pronounced. For tunnels excavated in fractured rock masses near the edge of deep valleys it is also suggested to provide ample lateral rock cover (Fernández and Alvarez 1994) and to place a reinforced concrete liner immediately upstream of the steel liner until the in-situ stress in the rock mass becomes equal to 1.4–1.5 times the internal tunnel pressure (Deere 1983).

On the other hand, in flat areas of marginal in-situ stresses where some amount of leakage can be tolerated, and slope instabilities are not likely to develop a heavily reinforced-concrete liner can be considered as an alternative to long steel liners. The reinforcement will preclude an uncontrolled increase in the permeability of the concrete liner, maintaining the permeability ratio k_L/k_m at a relatively low level, which generates significant head losses across the liner. Large head losses across the liner will result in reduced pore-water pressures in the rock mass surrounding the tunnel, thus minimizing the potential for development of the hydrofracturing or hydrojacking phenomena.

Zones of Adequate In-Situ Stress

At those locations where hydrojacking or hydrofracturing is not likely to occur, the need for a liner, and the type of liner to be used is mainly

determined by the permeability of the rock mass, the amount of leakage that can be tolerated, and the magnitude of the ground pore-water pressures that can be safely induced without generating instabilities in the surrounding media. Potential deterioration, erosion, or collapse of the excavated materials are also taken into account at this point. If a liner is required to control leakage or to minimize excess porewater pressures outside the tunnel, the efficiency of the liner is determined by the ratio of its permeability with respect to the permeability of the surrounding media. Because the permeability of concrete liners depends to a large degree on the magnitude of its circumferential strain, $\epsilon_{\theta L}$, after pressurization, the level of this strain, $\epsilon_{\theta L}$, should be used as an index to determine the type of liner to be used.

Permeable Media

For those cases where the rock mass is relatively permeable (rock mass permeability in excess of 1×10^{-5} cm/s (3.3×10^{-7} ft/sec), the desirable type of liner can be chosen based on the circumferential strain level generated by the internal tunnel pressure. A series of design guidelines are given next.

1. If the magnitude of the strain induced in the liner is lower than 1.5×10^{-4} an unreinforced concrete ring with a thorough contact grouting program could provide a relatively "impermeable" barrier with an average permeability in the range of 10^{-7} – 10^{-8} cm/s (3.3×10^{-9} – 3.3×10^{-10} ft/sec). Under most conditions the amount of leakage likely to develop out of this liner is low enough to be acceptable. For those cases where the permeability of the surrounding medium is high, a consolidation grouting program can be considered in order to lower the permeability of a ring of rock around the liner and reduce tunnel leakage. If the quality of the water in the tunnel is such that contact with the surrounding ground-water level must be prevented, the inner liner surface can be treated to make it impermeable.

2. If the magnitude of the tensile, circumferential strain in the liner, $\epsilon_{\theta L}$, exceeds 1.5×10^{-4} , longitudinal cracks will develop in the concrete increasing the permeability of the liner. If the liner is unreinforced its permeability will increase by several orders of magnitude, and the leakage and porewater pressures behind the liner will be similar to those of an unlined tunnel. If the unlined conditions are not acceptable an alternative design must be considered. Steel bar reinforcement can be used to control the width and spacing of longitudinal cracks maintaining the permeability of the liner low enough to behave as an effective flow barrier. Alternatively a consolidation grouting program can be implemented to reduce the permeability of the rock mass around the tunnel. The alternative to be chosen depends mainly on the strain level likely to develop in the liner and the permeability of the surrounding rock mass. If the circumferential strain induced in the liner is lower than 4.0×10^{-4} and the rock-mass permeability is larger than 10^{-5} cm/s (3×10^{-7} ft/sec) the use of a plain concrete liner with a consolidation grouting program can be considered. The length of the grout holes should be equal or slightly longer than the radius of the tunnel; and the injection pressures should be equal or slightly larger than the internal tunnel pressure. The permeability, k_1 , of a properly cement-grouted rock mass can be considered to be of the order of 10^{-6} cm/s (3×10^{-8} ft/sec).

The beneficial effects of the consolidation grouting program can be evaluated by using the permeability ratio, k_1/k_m , of the grouted over the untreated rock mass to determine the drop in hydraulic head across the ring of grouted rock. Consolidation grouting can also be helpful to minimize potential erosion of loose soil materials present within rock-mass discontinuities.

3. If the strain level exceeds 4×10^{-4} , large tension cracks will develop across the unreinforced liner and extend into the adjacent, grouted rock mass increasing substantially the permeability of the liner-grouted rock system. A reinforced concrete liner can be used to reduce the strain level within the grouted zone around the liner, maintaining its low permeability. The amount of reinforcement installed should be sufficient to maintain the strain in the liner below 6×10^{-4} to preclude the propagation of tension cracks across the grouted rock mass and to maintain the permeability of the liner compatible with the grouted rock mass. The initial filling of the tunnel should be allowed to proceed slowly enough to allow the recharge of the surrounding ground-water level and thus reduce the "net" difference between the internal and external water pressures. Under these conditions, the efficiency of the combined reinforced-concrete liner and consolidation grouting can be evaluated from the relationships plotted in Fig. 3 assuming that the low permeability of the grouted rock ring has been preserved. If further reductions of leakage and/or induced pore-water pressures are required, or if the strain level even with a large percentage of reinforcement exceeds 6×10^{-4} the installation of a plastic membrane embedded within the reinforced-concrete ring can be considered. This option is feasible as long as the circumferential strain in the liner remains of the order of 8×10^{-4} or less. At larger strain levels the stress in the reinforcing bars approaches the recommended long-term design value of 1/2 of the yield stress of the steel (*Manual of ACI* 1990).

4. If the estimated circumferential strain in a well-reinforced concrete liner exceeds the 8×10^{-4} value, a thin steel membrane, embedded within the concrete liner, can be considered. Design criteria requires the circumferential strain in the steel to be maintained below 1×10^{-3} . If the internal pressure in the tunnel is fairly large and/or the stiffness of the materials around the opening is relatively low, the thickness of the steel membrane required might become large enough to preclude buckling under the external load. At this point a regular steel liner can be designed.

Impermeable Media

If the rock-mass permeability is relatively low (less than 1×10^{-5} cm/s), the installation of a plain or reinforced-concrete liner, with or without a consolidation grouting program, will not be very effective in reducing leakage out of the tunnel or the magnitude of the induced pore-water pressure in the surrounding media. However, rock masses with these hydraulic characteristics are fairly tight and massive, resulting in small rates of leakage. If the rock is nonerrodible, and is not susceptible to solution or deterioration the excavation can be left unlined if the rock is massive and sound. However, a plain concrete liner is recommended where the rock is jointed and fractured or if the hydraulic performance of the tunnel requires it. A slow rate of loading is highly recommended to minimize the "net" hydraulic pressure driving the water out of the tunnel. Furthermore, nominal reinforcement is suggested if the rock is errodible or susceptible to deterioration and the

circumferential strain, $\epsilon_{\theta L}$, induced in the concrete liner exceeds a value of 6×10^{-4} . If the rock is susceptible to solution, reinforcement is suggested if the circumferential strain, $\epsilon_{\theta L}$, in the liner exceeds a value of 3×10^{-4} . If design considerations are such that only a reduced amount of leakage can be tolerated, or if the magnitude of the pore-water pressures needs to be controlled, an impermeable seal embedded within the concrete liner can be considered as a design alternative. This seal can consist of a plastic membrane or a thin steel laminae depending on the strain level likely to be induced in the liner.

Study of Precedent

The preliminary liner design obtained from the aforementioned guidelines should be validated by study of pertinent precedent where "key" variables are similar.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

- A_c = gross area of concrete;
 A_s = area of steel reinforcement;
 a = radius to center of liner;
 a_1 = internal radius of tunnel;
 b = excavated radius of tunnel;
 C = dimensionless flow parameter;
 d = nominal diameter of reinforcement bars;
 E_L = modulus of elasticity of liner;
 E_m = average modulus of elasticity of rock mass;
 E_s = modulus of elasticity of steel reinforcement;
 h_i = internal head of water;
 h_0 = external hydrostatic head;
 h_{w1} = water head at rock-liner interface;
 k_L = permeability of liner;
 k_{L0} = fictitious liner permeability used to compute head losses when gap develops;
 k_m = permeability of rock mass;
 $L = 2h_0$ = distance between center of the tunnel and imaginary mirror-image sink;
 M = inverse of the proportion of net driving pressure shared by rock mass;
 n = dimensionless ratio;
 P_i = internal tunnel-water pressure;
 P_0 = external hydrostatic-water pressure;
 P_r = mechanical stress transferred to rock walls by liner;
 P_w = excess pore-water pressure;
 P_{w1} = total pore-water pressure at the rock-liner interface;
 Q_i = initial rate of exfiltration;
 q_L = flow rate across liner;
 q_m = flow rate into rock mass;
 q_0 = flow rate from unlined tunnel;
 r = radius to point under consideration;
 r_{50} = radius to point where 50% of excess water pressure has been dissipated;
 S = average spacing between longitudinal tension cracks in liner;
 t = average thickness of liner;
 t_L = equivalent thickness of liner;
 t_s = equivalent thickness of steel;
 w = average aperture of longitudinal tension cracks in liner;
 α = liner permeability parameter that includes fluid properties and crack spacing;
- $\Delta h_L = h_i - h_{w1}$ = head loss across liner;
 $\Delta h_w = h_i - h_0$ = net driving head;
 $\Delta h_{w1} = h_{w1} - h_0$ = excess head at rock-liner interface;
 $\Delta P_L = P_i - P_{w1}$ = pressure drop across liner;

- $\Delta P_w = P_i - P_0$ = net-driving water pressure;
 $\Delta P_{w1} = P_{w1} - P_0$ = pressure drop across rock mass;
 δ_g = width of the rock-liner gap;
 $\varepsilon_{\theta L}$ = circumferential strain in liner;
 $\varepsilon_{\theta 0}$ = fictitious circumferential strain in liner used in nomogram 2;
 $\varepsilon_{\theta r}$ = circumferential strain at rock walls;
 γ_w = unit weight of water;
 μ = dynamic viscosity of water;
 ν_m = Poisson's ratio of rock mass;
 θ = angle measured clockwise from crown of tunnel; and
 ρ = reinforcement ratio.