Advanced models for stress evaluation and safety assessment in steel-lined pressure tunnels

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An accurate estimation of the stress in a steel liner is required for the design of steel-lined pressure shafts, both with regard to safety assessment of ageing steel-lined waterways of hydropower plants built in the 20th century, and for new projects which may be subject to harsher operational conditions. This paper proposes some new considerations for enhancing the standard calculation model, developed by the authors in the scope of practical engineering needs, by introducing more complexity.

Steel-lined pressure tunnels and shafts (SLPT&S) are stretches of the pressurized waterways connecting reservoirs to hydropower plants [Schleiss, 2012[1]]. The cross-section (see Fig. 1) outlines a multi-layer structure commonly composed of a steel liner, backfill concrete, a near-field rock zone (generally loosened, considered as radially cracked) and a sound far-field rock zone. A steel liner during the erection process is shown in Photo (a). The initial annular gap between the backfill concrete and the steel liner is generally grouted, to ensure maximum participation of the concrete-rock system in withstanding internal pressure.

Safety factors for the design of penstocks and SLPT&S are recommended in recognized technical literature such as the CECT recommendations [1980[2]] or the ASCE Manual [2012[3]]. Stresses in welded plated steel structures, such as exposed penstocks, can be evaluated with different levels of detail through dedicated design standards [EN 13445-3, 2014[4]; ASCE, 2012[3]; ASME, 2007[5]]. However, computational models for steel-lined pressure shafts considering the multilayer structure often involves conservative simplified assumptions, such as axisymmetrical behaviour with the lowest elastic modulus measured in situ [Pachoud and Schleiss, 2016[6]].

The presence of an annular gap between the steel liner and the backfill concrete has a significant influence on the ability of the concrete-rock system to withstand part of the internal pressure. Several authors proposed assumptions (see, for example, a synthesis proposed by Hachem and Schleiss [2009[7]]), often considering a single value or formula accounting for all sources of annular gap, namely thermal shrinking and non-recoverable deformations of the concrete-rock system. Furthermore, the influence of grouting in terms of mechanical behaviour under internal pressure has been considered for concrete-lined pressure tunnels [Schleiss, 1986[8]] but not yet explicitly for steel liners.

This paper first presents a selected literature review of the standard axisymmetric model used to compute stresses and displacements in SLPT&S, as well as recent developments taking into account anisotropic rock behaviour. Second, a simple approach to consider a global 'creeping' factor for considering plastic deformation of the concrete-rock system is proposed. In this approach, the plastic deformation (creep) is considered to occur only under the static component of the internal water pressure. The grouting external pressure is then included in the standard set of equations. Finally, an example application of the model for a pressure test is presented.

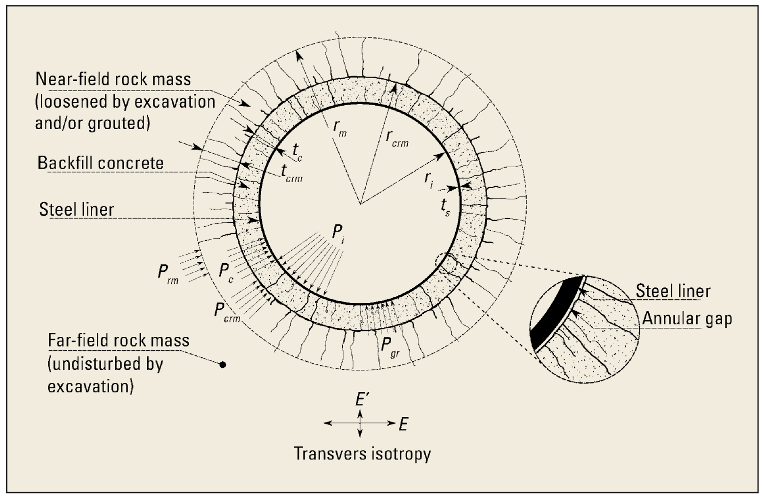
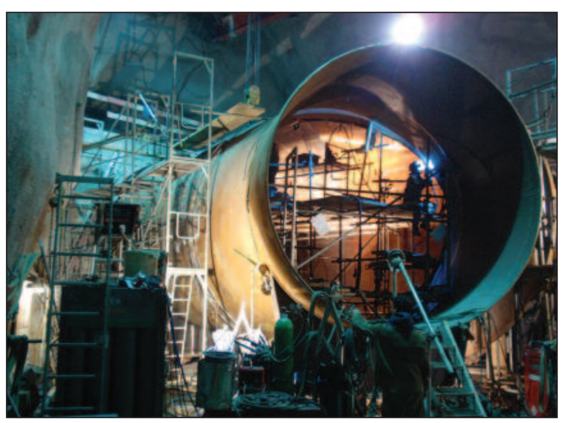


Fig. 1. Cross-section of a steel-lined pressure shaft in anisotropic rock. Grouting pressure p_{gr} is also represented.

Multi-layer model in anisotropic rock for steel-lined pressure shafts

Denoting p_s the pressure taken by the steel liner, and p_c the pressure withstood by the backfill concrete-rock system (at the backfill concrete interface, see Fig. 1), the internal water pressure p_i can be written as follows:



(a) Welding of a steel liner heading towards the underground powerhouse before backfilling with concrete (courtesy of A.J. Schleiss).

$$p_i = p_s + p_c \quad \ldots(1)$$

The compatibility conditions on the displacements at the boundaries of each layer yield a closed-form solution for p_c, as presented next [Pachoud and Schleiss, 2016[6]; Pachoud *et al.*, 2017[9]]:

$$p_c = \frac{\frac{1+v_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2v_s)\, p_i\, r_i^2 + p_i\, r_i^2] - \Delta r_0}{\frac{1+v_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2v_s)(r_i+t_s)^2 + r_i^2] + r_c\frac{1}{E_{eq}}} \quad \ldots(2)$$

where the term E_{eq} is expressed as follows:

$$\frac{1}{E_{eq}} = \frac{1-v_c^2}{E_c}\ln\left(\frac{r_{crm}}{r_c}\right) + \frac{1-v_{crm}^2}{E_{crm}}\ln\left(\frac{r_{rm}}{r_{crm}}\right) + \frac{1+v}{E'} \quad \ldots(3)$$

and:

- r_i = internal radius of the steel liner;
- t_s = steel liner thickness;
- r_c = internal radius of the backfill concrete layer;
- r_{crm} = radius at the interface between backfill concrete and the near-field (cracked) rock;
- r_{rm} = radius at the interface between the near- and the far-field rock;
- Δr_0 = initial gap between the steel liner and the backfill concrete;
- E_s = elastic modulus for steel;
- v_s = Poisson's ratio for steel;
- E_c = elastic modulus for concrete;
- v_c = Poisson's ratio for concrete;
- E_{crm} = elastic modulus for the near-field (cracked) rock;
- v_{crm} = Poisson's ratio for near-field (cracked) rock;
- E' = elastic modulus for far-field rock; and,
- v = Poisson's ratio for the far-field rock.

Common assumptions for the initial gap Δr_0 were synthesized by Hachem and Schleiss [2009[7]].

The pressure p_{rm} transmitted at the sound far-field rock (see Fig. 1) can be estimated as:

$$p_{rm} = p_c \frac{r_c}{r_{rm}} \quad \ldots(4)$$

In a recent research project, Pachoud and Schleiss [2016[6]] have systematically studied the influence of anisotropic rock behaviour on SLPT&S using the finite element method, and particularly the case of transverse isotropy. Transversely isotropic rocks are characterized by five independent constants, denoted E, E', ν, ν', and G' [Amadei *et al.*, 1987[10]]. The prime superscript refers to the direction perpendicular to the plane of isotropy (in general the direction of lowest stiffness). For example, in a foliated rock, the plane of isotropy would correspond to the main foliation. To estimate the maximum stress in the steel lining, Pachoud and Schleiss [2016[6]] proposed to include empirical correction factors into the analytical solution in isotropic rock as follows:

$$\frac{1}{E_{eq,aniso}} = \frac{1-v_c^2}{E_c}\ln\left(\frac{r_{crm}}{r_c}\right) + \frac{1-v_{crm}^2}{E_{crm}} \times \ln\left(\frac{r_{rm}}{r_{crm}}\right) + \left[\left(\frac{E}{E'}\right)^{-0.65}\left(\frac{G}{G'}\right)^{0.50}\left(\frac{1+v}{1+v'}\right)^{-0.56}\right]\frac{1+v}{E'} \quad \ldots(5)$$

The cross-shear modulus G' can be estimated considering the following empirical relation as proposed by Saint-Venant:

$$G' = \frac{E'}{1+(E'/E)+2v'} \quad \ldots(6)$$

Although Eq. (6) is widely used in literature, as most published data support its validity, one should keep in mind that there are still major exceptions [Gonzaga *et al.*, 2008[11]; Hakala *et al.*, 2007[12]].

2. Consideration of plastic deformation of the concrete-rock system

This model assumes that plastic deformation occurs only under static pressure. The internal water pressure p_i (maximum pseudo-static pressure used for design) will thus be separated into its static and dynamic components as follows:

$$p_i = p_{i,stat} + p_{i,dyn} \quad \ldots(7)$$

Similarly, one can express the circumferential (tangential) stress in the steel liner σ^s_{circ} in two components:

$$\sigma^s_{circ} = \sigma^s_{circ,stat} + \sigma^s_{circ,dyn} \quad \ldots(8)$$

The proposed approach introduces a so-called 'creeping' factor $f_{\%}$ which decreases the concrete-rock participation under static load $p_{c,stat}$. This factor varies from zero (no concrete-rock participation) to unity (100 per cent of the maximum elastic participation). From the thick-walled pipe theory, $\sigma^s_{circ,stat}$ can thus be computed as:

$$\sigma^s_{circ,stat} = \frac{1}{(r_i+t_s)^2 - r_i^2} \times \left[r_i^2 p_{i,stat} - (r_i+t_s)^2 (f_{\%} p_{c,stat}) - - (r_i+t_s) - (r_i+t_s)^2 (f_{\%} p_{c,stat} - p_{i,stat})\right] \quad \ldots(9)$$

where $p_{c,stat}$ is computed with $p_{i,stat}$ as follows:

$$p_{c,stat} = \frac{\frac{1+v_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2v_s)\, p_{i,stat}\, r_i^2 + p_{i,stat}\, r_i^2] - \Delta r_0}{\frac{1+v_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2v_s)(r_i+t_s)^2 + r_i^2] + r_c\frac{1}{E_{eq}}} \quad \ldots(10)$$

In Eq. (10), the initial annular gap Δr_0 now only represents the thermal shrinkage of the steel lining under a temperature variation ΔT. In practice, the 'creeping' factor $f_{\%}$ can be chosen depending on available information, in terms of permanent deformations or decrease of stiffness (see below).

Similarly, the dynamic component of the stress in the steel liner $\sigma^s_{circ,dyn}$ can be computed as:

$$\sigma^s_{circ,dyn} = \frac{1}{(r_i+t_s)^2 - r_i^2}\left[r_i^2 p_{i,dyn} - - (r_i+t_s)^2 p_{c,dyn} - (r_i+t_s)^2 (p_{c,dyn} - p_{i,dyn})\right] \quad \ldots(11)$$

in which $p_{c,dyn}$ is computed considering a residual annular gap $\Delta r_{0,res}$, determined by subtracting the external displacement of the lining under static pressure to the initial gap Δr_0. In case of a negative result (physically not correct), perfect contact between the steel and backfill concrete is assumed for the computation of $p_{c,dyn}$. The radial external displacement of the steel $u^s_{r,stat}$ (r_i+t_s) under static pressure is calculated as:

$$u^s_{r,stat}(r_i + t_s) = \frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}\left[(1-2\nu_s)(p_{i,stat}r_i^2 - f_{\%}p_{c,stat}(r_i+t_s)^2) + (p_{i,stat} - f_{\%}p_{c,stat})r_i^2\right] \quad \text{...(12)}$$

Then, the residual gap (if any) for the computation of $p_{c,dyn}$ can be estimated as:

$$\Delta r_{0,res} = \max\{[\Delta r_0 - u^s_{r,stat}(r_i+t_s)]; 0\} \quad \text{...(13)}$$

The pressure p_{rm} transmitted to the far-field rock is thus written:

$$p_{rm} = (p_{c,stat} + p_{c,dyn})\frac{r_c}{r_{rm}} \quad \text{...(14)}$$

One shall note that if $\Delta r_{0,res}$ is greater than or strictly equal to zero, it would mean that the concrete-rock system is not solicited by the static component of the pressure as a result of the combined effect of the initial gap and plastic deformation (or creeping).

The induced plastic deformation can be evaluated quantitatively by the two approaches presented below to evaluate the hypothesis of the so-called 'creeping' factor $f_{\%}$ in practice.

2.1 Global radial displacement of the concrete-rock system at the interface with the lining

Re-writing the static participation of the concrete-rock system by introducing a permanent displacement term $\Delta r_{f\%}$ as a result of plastic deformation gives:

$$f_{\%}p_{c,stat} = \frac{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2\nu_s)p_{i,stat}r_i^2 + p_{i,stat}r_i^2] - \Delta r_0 - \Delta r_{f\%}}{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2\nu_s)(r_i+t_s)^2 + r_i^2] + r_c\frac{1}{E_{eq}}} \quad \text{...(15)}$$

In this case, all effects of plastic deformation are equivalent to a permanent, non-recoverable displacement. The total annular gap is represented by thermal shrinkage as well as permanent deformation (see Fig. 3).

From Eq. (15), $\Delta r_{f\%}$ is expressed as:

$$\Delta r_{f\%} + \Delta r_0 = \frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}\left[(1-2\nu_s)(p_{i,stat}r_i^2 - f_{\%}p_{c,stat}(r_i+t_s)^2) + (p_{i,stat} - f_{\%}p_{c,stat})r_i^2\right] - f_{\%}p_{c,stat}r_c\frac{1}{E_{eq}} \quad \text{...(16)}$$

Identifying the first term of the right hand part of Eq. (16) and by conserving only positive values (physically correct), one obtains:

$$\Delta r_{f\%} = \max\{u^s_{r,stat}(r_i+t_s) - f_{\%}p_{c,stat}r_c\frac{1}{E_{eq}} - \Delta r_0; 0\} \quad \text{...(17)}$$

It should be noted that $\Delta r_{f\%}$ is considered (under this nomenclature) in the computation of $\Delta r_{0,res}$ being already accounted for in the computation of $u^s_{r,stat}$ (r_i+t_s).

This formulation allows calibrating (or estimating from engineering judgement) $f_{\%}$ in terms of permanent displacements based on, for example, in-situ testing [Ott, 1964[12]].

2.2 Reduction of the elastic modulus of the concrete-rock system

Similarly, introducing a reduction $E_{f\%}$ of the equivalent elastic modulus of the concrete-rock system, one obtains:

$$f_{\%}p_{c,stat} = \frac{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2\nu_s)p_{i,stat}r_i^2 + p_{i,stat}r_i^2] - \Delta r_0}{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2\nu_s)(r_i+t_s)^2 + r_i^2] + r_c\frac{1}{E_{eq}}\frac{1}{(1-E_{f\%})}} \quad \text{...(18)}$$

It finally yields:

$$E_{f\%} = 1 - \frac{f_{\%}p_{c,stat}r_c}{E_{eq}[u^s_{r,stat}(r_i+t_s) - \Delta r_0]} \quad \text{...(19)}$$

In this approach, creeping (the plastic deformation of the concrete-rock system), is only expressed in terms of a decrease in the global stiffness.

3. Consideration of grouting external pressure

Grouting of the annular gap often performed before filling induces an external pressure on the steel liner denoted p_{gr}, thus introducing a pre-stress in the steel liner (circumferential compression). The allowable pressure of gap grouting is normally very limited because of the risk of buckling of the steel liner. This limited grouting pressure also acts on the concrete-rock system. Grouting of the loosened rock zone as a result of excavation is normally very beneficial in reducing permanent plastic deformation in this zone after first filling of the steel liner.

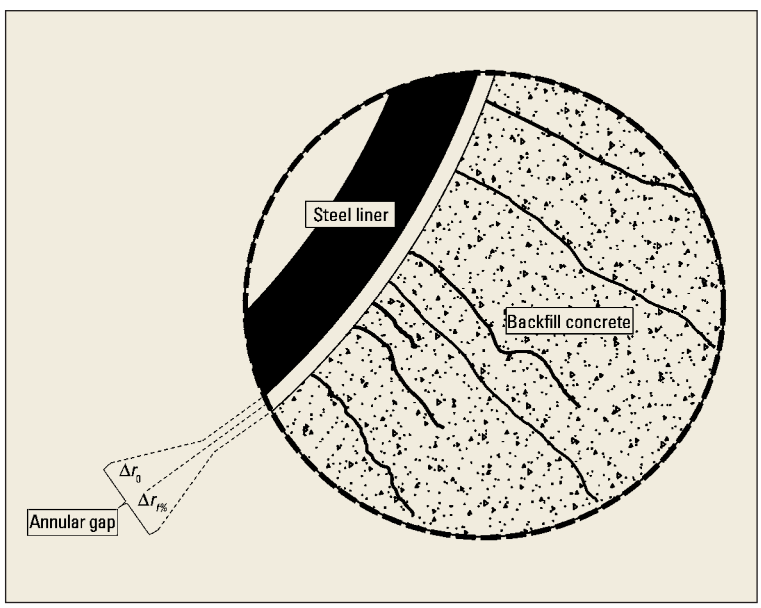


Fig 3. Illustration of the two components Δr_0 (thermal shrinkage at first filling) and $\Delta r_{f\%}$ (accounting for permanent plastic deformations) of the annular gap after first filling.

Therefore, in this model, one can assume that the thermal shrinkage of the steel would first eliminate this pre-stress, prior to generating the annular gap. Similarly to the grouting model, and to account for a loss of efficiency of the grouting through time, a grouting factor $f_{\%,gr}$ is also introduced.

During the initial state, prior to thermal shrinkage at first water filling, the steel liner is subject to a negative radial displacement (shrinkage) because of grouting, expressed as below on the external surface of the lining:

$$u_r^{s,gr}(r_i + t_s) = -\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}\left[(1-2\nu_s) f_{\%,gr} p_{gr} (r_i+t_s)^2 + f_{\%,gr} p_{gr} r_i^2\right]$$

…(20)

As explained above, $p_{c,stat}$ is thus computed as follows:

$$p_{c,stat} = \frac{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}\left[(1-2\nu_s) p_{i,stat} r_i^2 + p_{i,stat} r_i^2\right] - \Delta r_0 - u_r^{s,gr}(r_i+t_s)}{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}\left[(1-2\nu_s)(r_i+t_s)^2 + r_i^2\right] + r_c \frac{1}{E_{eq}}}$$

…(21)

where it can be observed that the initial gap is ‘reduced’ by the pre-stress introduced by grouting. $p_{c,stat}$ can be equal to zero in cases where the annular gap is not covered by the radial displacement induced by the static component of the water pressure. In such a case, the grouting pressure would have also been completely eliminated by the thermal shrinkage.

Considering the plastic deformation (or 'creeping') of the concrete-rock system previously introduced, it follows:

$$\sigma_{circ,stat}^{s} = \frac{1}{(r_i+t_s)^2-r_i^2}\left[r_i^2 p_{i,stat} - (r_i+t_s)^2 (f_{\%} p_{c,stat}) - (r_i+t_s)^2 (f_{\%} p_{c,stat} - p_{i,stat})\right]$$

…(22)

and:

$$\sigma_{circ,dyn}^{s} = \frac{1}{(r_i+t_s)^2-r_i^2}\left[r_i^2 p_{i,dyn} - (r_i+t_s)^2 p_{c,dyn} - (r_i+t_s)^2 (p_{c,dyn} - p_{i,dyn})\right]$$

…(23)

The external radial displacement of the steel liner is then computed as:

$$u_{r,stat}^{s}(r_i + t_s) = \frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}\left[(1-2\nu_s)(p_{i,stat} r_i^2 - f_{\%} p_{c,stat} (r_i+t_s)^2) + (p_{i,stat} - f_{\%} p_{c,stat}) r_i^2\right]$$

…(24)

giving a residual annular gap $\Delta r_{0,res}$ for the computation of $p_{c,dyn}$ determined as:

$$\Delta r_{0,res} = \max\{[\Delta r_0 + u_r^{s,gr}(r_i+t_s) - u_{r,stat}^{s}(r_i+t_s)]; 0\}$$

…(25)

The dynamic part of the pressure withstood by the concrete-rock system is then obtained by:

$$p_{c,dyn} = \frac{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}\left[(1-2\nu_s) p_{i,dyn} r_i^2 + p_{i,dyn} r_i^2\right] - \Delta r_{0,res}}{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}\left[(1-2\nu_s)(r_i+t_s)^2 + r_i^2\right] + r_c \frac{1}{E_{eq}}}$$

…(26)

The pressure p_{rm} transmitted to the far-field rock is thus written as:

$$p_{rm} = (p_{c,stat} + p_{c,dyn}) \frac{r_c}{r_{rm}}$$

…(27)

where the effect of p_{gr} is already included in the computation of $p_{c,stat}$.

Similarly to above, and in order to evaluate the hypothesis of the ‘creeping’ factor $f_{\%}$ in practice, the induced plastic deformation can be evaluated quantitatively by two approaches presented as follows.

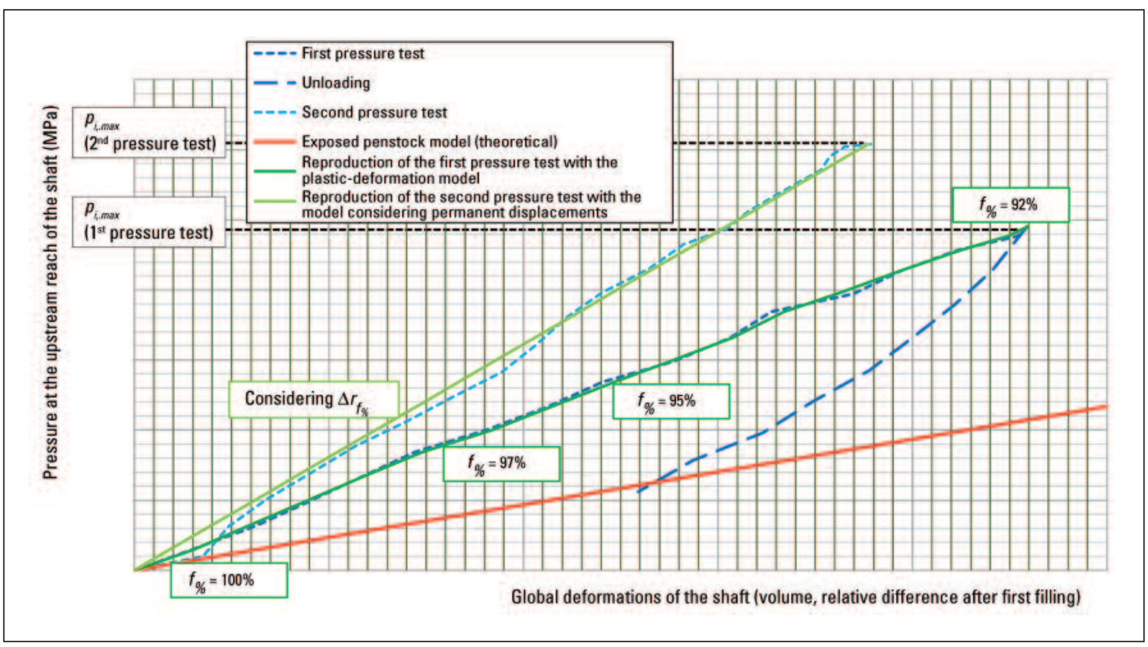


Fig. 4. Conceptual example of calibration of the model considering plastic deformations against pressure tests performed in the middle of the 20th century after the construction of an undisclosed high-head hydropower plant.

3.1 Global radial displacement of the concrete-rock system at the interface with the lining

Introducing a permanent displacement $\Delta r_{f\%}$ as a result of permanent plastic deformation, one obtains:

$$f_{\%}p_{c,stat} = \frac{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2\nu_s)p_{i,stat}r_i^2+p_{i,stat}r_i^2]-\Delta r_0-\Delta r_{f\%}-u_r^{s,gr}(r_i+t_s)}{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2\nu_s)(r_i+t_s)^2+r_i^2]+r_c\frac{1}{E_{eq}}} \quad \ldots(28)$$

where the part of the static component taken by the concrete-rock system is obtained by adding the positive effect of the negative displacement $u_r^{s,gr}$ (r_i+t_s) induced by grouting. The following equivalent permanent displacement is obtained:

$$\Delta r_{f\%} = \max\{u^s_{r,stat}(r_i+t_s) - f_{\%}p_{c,stat}r_c \frac{1}{E_{eq}} - \Delta r_0 - u_r^{s,gr}(r_i+t_s); 0\} \quad \ldots(29)$$

3.2 Reduction of the elastic modulus of the concrete-rock system

Similarly, introducing a reduction $E_{f\%}$ of the equivalent elastic modulus of the concrete-rock system, obtains the following:

$$f_{\%}p_{c,stat} = \frac{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2\nu_s)p_{i,stat}r_i^2+p_{i,stat}r_i^2]-\Delta r_0-u_r^{s,gr}(r_i+t_s)}{\frac{1+\nu_s}{E_s}\frac{r_i+t_s}{(r_i+t_s)^2-r_i^2}[(1-2\nu_s)(r_i+t_s)^2+r_i^2]+r_c\frac{1}{E_{eq}(1-E_{f\%})}} \quad \ldots(30)$$

giving:

$$E_{f\%} = 1 - \frac{f_{\%}p_{c,stat}r_c}{E_{eq}[u^s_{r,stat}(r_i+t_s)-\Delta r_0-u_r^{s,gr}(r_i+t_s)]} \quad \ldots(31)$$

4. Application example

The proposed model was used to reproduce pressure tests performed in the steel-lined pressure shaft of an undisclosed high-head hydropower plant built in the mid-20th century (see Fig. 4).

A series of two pressure tests were performed, after initial filling of the shaft. During the pressure tests, the water added by pumping was measured (accounting for estimated leakages), allowing for the estimation of the global dilation of the steel liner along the shaft. The first unloading after the first pressure test was also monitored. The first filling is not documented, as well as what happened between the unloading and the second pressure test, which limits the exploitable data.

To reproduce these pressure tests, all the parameters of the model were determined *a priori*, based on available data (archives and more recent investigations), accounting for grouting pressure and rock anisotropy. As shown in Fig. 4, the factor $f_{\%}$ was fitted to the observations of the first pressure test. The permanent deformation was then estimated by Eq. (29), and was considered to reproduce the second pressure test (performed the following day), in which an elastic behaviour was observed, and where it was observed to be a very good fit. Of course, this exercise only validates in this case the global rigidity of the system, and it should be noted that only a global 'creeping' factor $f_{\%}$ was used. However, specific estimations of $f_{\%}$ depending on local conditions could also be considered.

Such results, in this specific example, only validated the relative rigidity of the system and global participation of the concrete-rock system. The results are only exact with respect to the applied overpressure. As the shaft was already filled prior to monitoring the deformations, the absolute value of the annular gap could not be validated. However, the results guarantee that the annular gap along the shaft remains below a threshold value for which the results would no longer fit the tests.

5. Conclusions

The paper proposes some new developments to be incorporated in standard equations for stresses and displacements in steel-lined pressure tunnels and shafts, to account for: (i) plastic, permanent deformation of the concrete-rock system in a quantitative manner (which can also be used to account for 'creeping'); and, (ii) grouting of the annular gap between the steel liner and the backfill concrete.

The proposed approach is particularly useful in practical cases within the scope of safety assessment, where data of ageing steel-lined waterways of hydropower plants built in the 20th century are available. Such plants have often been designed with assumptions yielding initial safety factors, which may no longer comply with modern standards, and/or that may not even be checked conceptually because of a lack of computational solutions. This allows reasearch into the effective safety factors using probabilistic approaches that may exist today.

In the case of new hydropower plants, the proposed reassessment approach can be used to interpret pressure test results and therefore calibrate the model to the measured behaviour of the steel-lined pressure shaft [Hammer *et al.*, 2018[14]; Chène, 2013[15]]. The scope of application is particularly interesting as modern technologies used to measure deformations allow for the capture of the behaviour of the system throughout the first filling.

The developments presented here also constitute a further contribution towards comprehensive safety and reliability assessment of steel-lined pressure tunnels and shafts [Pachoud, 2017[16]]. In the context of ageing high-head hydropower plants, an accurate assessment of the reliability of the existing steel-lined pressure tunnels and shafts is paramount to inform proper rehabilitation decisions. ◊

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