19th Australasian Tunnelling Conference – Acosta-Martínez, Stewart and Hudson-Smith (Eds.)
© 2025 Australian Tunnelling Society, ISBN 978-0-85825-002-4

Design considerations and criteria for underground structures – structural verifications of waterway concrete linings in pumped storage power plants

F. Cippà & M. Cadenelli
Lombardi SA Ingegneri Consulenti, Giubiasco, Switzerland

1 INTRODUCTION

Final linings of hydraulic pressure tunnels and shafts are designed following several different practices. As this can lead to significant differences in the results, the question of the most suitable methods for analysis and design is of utmost importance and is approached in this article for some typical hydraulic underground works.

This is even more crucial for pumped storage powerplants (PSP) as rapid start/stop manoeuvres and fast transitions between generation and pumping modes create more frequent and generally higher peaks of hydraulic dynamic pressures than traditional hydropower plants.

2 HYDRAULIC VS. NON-HYDRAULIC UNDERGROUND WORKS AND GEOTECHNICAL VS. STRUCTURAL MODELS

Hydraulic pressure tunnels and shafts mainly differ from other underground facilities by the water contained by the lining during normal operation. This persistent internal pressure acting on the lining during most operating conditions is a missing load case for many other underground structures such as road and railway tunnels. For these dry structures, the final lining design is normally controlled by load combinations with external geotechnical actions covering a vast variety of occurrences, from moderate wedge pressures to high pressures given, for example, by squeezing or swelling rock masses. The advances in numerical analysis have allowed increasingly comprehensive modelling of most geotechnical load cases, including the time-dependent rock mass behaviour typically associated with tunnel and shaft excavation. By means of stress fields, thrust forces, bending moments, deformations, and other results, they provide a sufficient output for the design of most non-hydraulic underground structures. As the geotechnical actions are determinant for the lining design of these structures, the analysis assumptions are normally aimed at maximizing their effects, seeking for the most unfavourable lining demand. However, in hydraulic tunnels and shafts, the effects of external geotechnical actions are often opposite to those of internal water pressure.

Therefore, designing a hydraulic tunnel or shaft like a non-hydraulic structure by adding the internal water pressure as an additional step of a sequential, time-dependent geotechnical analysis is often misleading, given that the internal water pressure is likely to release the effects of the geotechnical actions, possibly not showing the hazards related to internal pressure load cases.

Instead, the designer of any hydraulic tunnel or shaft is advised to carefully question whether the external geotechnical actions can be reliably accounted for when designing the final lining under internal pressure conditions. Higher mechanical properties of the rock mass than conservatively assumed for the geotechnical design, more favourable characteristics of the rock mass discontinuities, as well as variations of the time-dependent rock mass behaviour, are typical issues able to significantly reduce the effects of geotechnical actions on the lining. In fact, in many cases, the geotechnical actions are finally deemed so unpredictable that the internal pressures are dealt with in a separate design where geotechnical actions are fully neglected or are applied only locally to induce some unfavourable additional bending in the lining. Therefore, the further considerations in this article refer to analyses performed on structural models rather than geotechnical models.

3 LINING PERMEABILITY

The structural behaviour and the design of hydraulic tunnel and shaft linings depend on their permeability, where three options are distinguished. In case of the first option, measures are deliberately taken to establish a high lining permeability (drains through the lining, unsealed construction joints, etc.), thus speaking of a drained lining. The second option does not include watertight layers to assure the lining impermeability, but permeability is lowered as far as possible (low-permeability lining). The third option is impervious, for example by the use of a continuous steel lining or of an embedded or exposed waterproofing membrane.

An impervious lining is typically needed when the ratio between rock cover and internal pressure is below the necessary minimum for safety against hydrofracturing as well as in case of water-sensible rock masses, as indicated by Deere-Lombardi (1988).

If an impervious lining is not necessary, a choice is needed between a drained and a low-permeability lining. The drained lining is typically more cost-effective because of the reduced structural capacity compared to the low-permeability lining. In fact, the primary objective of drains lies in balancing the pressures in the tunnel and in the rock mass to reduce the lining demand imposed by hydraulic loads. However, the execution of drained tunnels and shafts is subject to a number of conditions and limitations, listed below, requiring a proper evaluation before the drained lining solution is adopted:

- The rock mass permeability should be controlled by a system of widespread, interconnected and differently oriented discontinuities, highly likely to be intercepted by drains.
- The rock mass, including the fill material in faults and cracks, should not be prone to be washed out by water or, at least, it should be possible to control the risk by filters and similar provisions in the drains.
- Drained linings are mostly used in waterway sections not affected by frequent and intense water pressure variations. In this way, on the one hand, excessive stress on the drains is avoided (a particularly important aspect in modern highly flexible pumped storage schemes); on the other hand, drains subject to severe water hammer can hardly be so effective as to prevent transient hydraulic loads on the lining, thus not attaining the goal of a thin and cost-effective lining.
- To prevent excessive water losses from the waterways, the groundwater pressure in the rock mass surrounding a drained lining should be higher than the internal pressure.
- The effects of a drained tunnel or shaft on the local hydrogeological conditions should be evaluated. Drained works in permeable formations can cause groundwater levels to decline and springs to dry up.
- Surface settlements induced by the construction of underground works should be assessed, considering that settlements tend to be greater if the works are drained.

4 HYDRAULIC LOADS ACTING ON THE LINING

In the case of the third option (impervious lining), the internal water pressure and the external water pressure (groundwater pressure) are independent actions and the relieving effect of an external water pressure in the internal pressure design shall only be considered after careful evaluation. In many cases, despite a possible location below the groundwater table, the external pressure is neglected, either due to technical reasons (for example, in low permeability rock masses, the rock mass can be locally dry even below the groundwater table) or if the reference design codes require to do so.

In the case of the first and second option (drained lining and low-permeability lining) the internal water pressure and the water pressure on the outer lining surface are interacting through the drains or through lining seepage given, for example, by primary concrete permeability or, more likely, by cracks in the lining concrete. Internal and external pressures on the lining are thus no longer independent. While the internal water pressure is still given by the hydraulic operating conditions of the waterways (static head, head losses, transient effects), the water pressure on the outer lining surface depends on a series of parameters:

- In the case of the first option (drained lining), the pressure on the outer lining surface is normally assumed to be equal to the permanent component of the internal water pressure.
- In the case of the second option (low-permeability lining), the pressure on the outer lining surface depends on the relative permeability of the lining and of the rock mass, as well as on the groundwater level in the rock mass.

In both cases the variable or dynamic component of the internal water pressure is normally assumed not to affect the water pressure on the outer lining surface, as it is unlikely that the system has enough time to react to such a short-term load variation. However, extreme loading conditions may require novel constructive solutions where the drains are designed to provide such an intense effect as to compensate, at least to some extent, also the variable component of the internal water pressure, with the inclined shaft of the Snowy 2.0 PSP in New South Wales being an outstanding example.

So, the permanent component of water pressures – the main actions in most hydraulic tunnels and shafts – can be seen as "conventional" mechanical loads only in the case of impervious linings, whereas in all other cases the water load magnitude on the lining results from a seepage flow through the lining and the surrounding rock and depends on the permeability of the system components, as indicated by Schleiss (1986). For the variable or dynamic component of water pressures (water hammer or surge events in the waterways, for example), the inertia necessary for the seepage flow to develop must be additionally considered, meaning that these actions are typically modelled as mechanical loads, especially when they are affected by sudden variations.

A summary of hydraulic loads acting on tunnel linings is given in Table 1, including also the load case of tunnel dewatering for inspection and maintenance. For this last case, the effect of drains is significant as they allow the rock mass to release water, decreasing the external pressure on the lining.

Table 1: Hydraulic load acting on the lining as a function of lining permeability and type of hydraulic load.

<table>
<tr><td colspan="3" rowspan="3"></td><td colspan="4">Lining permeability</td></tr>
<tr><td>Option 3</td><td>Option 2</td><td colspan="2">Option 1</td></tr>
<tr><td>Impervious lining</td><td>Low permeability lining</td><td>Conventionally drained lining</td><td>Intensely drained lining</td></tr>
<tr><td rowspan="3">Hydraulic loads</td><td rowspan="2">Load cases with resulting internal pressure</td><td>Permanent loads</td><td>Entire permanent load</td><td>Partial permanent pressure given by seepage model</td><td>Almost no load</td><td>No load</td></tr>
<tr><td>Variable loads</td><td>Total variable load</td><td>Total variable load</td><td>Total variable load</td><td>Partial variable load given by hydraulic model</td></tr>
<tr><td colspan="2">Dewatering (load case with resulting external pressure)</td><td>Entire external pressure</td><td>Partial external pressure given by seepage model</td><td colspan="2">Highly reduced external pressure given by seepage model or minimum loading according to good practices</td></tr>
</table>

5 LINING STIFFNESS

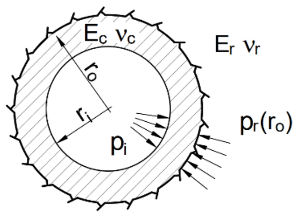
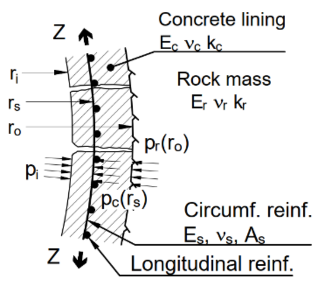
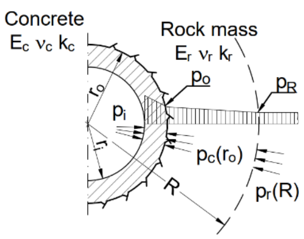
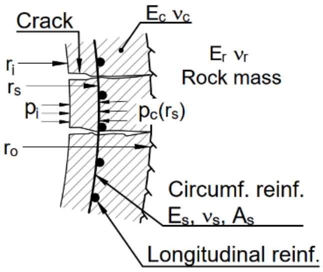
Once the internal water pressure and the pressure on the outer lining surface are known, the system comprising the lining and the surrounding rock mass can be calculated. This structural analysis yields the necessary values for the lining design, with the analysis results depending upon the relative stiffness of the lining and of the rock mass. It is noted that the lining is typically subject to circumferential tension and the stiffness is therefore affected by cracking. Furthermore, the analysis should consider the effect of the outer pressure on the rock mass and a contact non-linearity at the lining-ground interface, given that a structural interaction of both elements is normally only admitted over effective compressive stresses, whereas tensile stresses can't be taken into account as the tensile strength at the interface is neglected for design purposes.

6 MODELLING OPTIONS FOR NON-LINEARITIES

It has been shown in the two previous sections that the effects of the water pressure acting on the lining of hydraulic tunnels and shafts depend on the relative permeability of lining and rock mass and on their relative stiffness. Material non-linearity affects the lining stiffness in tension, whereas a contact non-linearity should be introduced at the lining-ground interface. The non-linear behaviour of the system is enhanced by the interrelationships between some of the material characteristics: the permeability of a reinforced concrete lining, for example, strongly depends on its stiffness, given that open cracks cause a significant permeability increase.

Depending on the extent to which these points are considered, a set of possible modelling options is obtained as shown in Table 2 for low-permeability linings.

Table 2: Summary of main modelling options for reinforced concrete linings of tunnels and shafts, adapted from Schleiss (1986). r_i, r_o = inner and outer lining radius; r_s = steel reinforcement radius; R = radius of the rock mass portion affected by the tunnel; E_c, $E_{c,uncracked}$, E_r, E_{lining}, E_s = modulus of concrete, uncracked concrete, rock mass, lining, reinforcement steel; ν_c, ν_r, ν_s = Poisson's ratio of concrete, ; k_c, k_r = permeability of concrete, rock mass; p_i = internal water pressure; p_o = water pressure acting at the lining-rock mass interface; p_s = water pressure acting at $r = r_s$; p_R = water pressure acting at r = R; A_s = area of circumferential steel reinforcement.

<table>
<tr><td colspan="2"></td><td colspan="3">Lining stiffness</td></tr>
<tr><td colspan="2"></td><td>Uncracked concrete</td><td>Preset value for cracked concrete</td><td>Lining stiffness adjusted based on expected structural behaviour</td></tr>
<tr><td rowspan="2">Model for water pressure distribution</td><td>Impervious pipe model</td><td>Model A.1

</td><td>Model A.2
The same as Case A.1, but with reduced concrete modulus (for example E_{lining} = 1/3 $E_{c\ uncraked}$)</td><td>Model A.3

</td></tr>
<tr><td>Pervious pipe model</td><td>Model B.1

</td><td>Model B.2
The same as Case B.1, but with reduced concrete modulus (for example E_{lining} = 1/3 $E_{c\ uncraked}$)</td><td>Model B.3

</td></tr>
</table>

The first main variable is the model used for the water pressure distribution inside the system. In the upper row of the table (Models A.1 to A.3) the resulting water load, i.e. the internal pressure minus the external pressure, is modelled as a mechanical load applied to the internal lining surface.

Concrete and rock experience this load through changes in their total stresses, while it is assumed that no changes occur in the pore pressures (except for some pore over- and under-pressures).

In the lower row (Models B.1 to B.3), on the contrary, the water loads are the result of a seepage flow through the system, with effects on both the total and the effective stresses.

The expected structural behaviour is:

- Models B for permanent (static) water loads
- Models A or B for dynamic or transient water loads, depending on the application rate:
  - Models A for impulsive, short-term loads
  - Models B or intermediate between A and B for slowly applied loads.

The second main variable is the assumed lining stiffness. In the left column (Models A.1 and B.1), the uncracked lining stiffness is used. In the right column (Models A.3 and B.3), the stiffness results from an in-depth calculation based on the lining loadings, providing the most accurate description of the lining behaviour. However, it has to be kept in mind that this calculation depends on several parameters like the concrete tensile strength and the number of pre-existing cracks (including cracks due to shrinkage and thermal effects), which are subject to a natural variability, thus limiting the reliability of the results. A practical way to overcome these limits and make calculations more straightforward is given in the mid columns (Models A.2 and B.2), consisting in the use a preset value for the lining modulus smaller the uncracked modulus but conservatively greater than that resulting from Models A.3 and B.3. A sound value can often be assumed equal to $E_{lining} = 1/3\ E_{c,uncracked}$.

Please note that all models shown in Table 2 can be used to design low-permeability linings, which are likely to be cracked to some extent. In other words, it should not be understood that Models A.1 and B.1 are intended for uncracked linings, whereas the other models are associated to cracked and consequently more permeable linings.

At this point, three aspects appear to be of interest:

- How much do non-linearities and model selection practically affect the system behaviour and the structural analysis results?
- How can such a complex behaviour be reasonably modelled for design purposes, i.e. putting aside the scientific aim of a precise behaviour description and instead focusing on a safe and efficient design?
- How much freedom is allowed by Australian and international design codes for such methods of analysis and design?

These aspects are treated in the following sections.

7 MODEL IMPACT ON STRUCTURAL ANALYSIS RESULTS

To visualize the modelling significance on the analysis results, an example of a vertical shaft is shown. The main geometric characteristics are 25.0 m inner diameter, 0.80 m lining thickness, 2 reinforcement layers d24 spaced 200 mm (A_s = 4,524 mm^2/m). The concrete properties are $E_{c,uncracked}$ = 32,800 MPa, Poisson's ratio $\nu_c = 0.20$, permeability $k_{c,uncracked} = 10^{-9}$ m/s. The rock mass properties are E_r = 7000 MPa, Poisson's ratio $\nu_r = 0.25$, permeability $k_r = 10^{-7}$ m/s. The calculated load case is an increase in the internal water pressure by 60 m, i.e. 600 kN/m^2.

The corresponding increase $\Delta\sigma_s$ in the reinforcement tension stress in shown in Table 3 for the six modelling options previously presented in Table 2. The results greatly vary between the extreme cases represented by Model A.1 and Model B.3. While the value $\Delta\sigma_s$ = 550 MPa for Model A.1 exceeds the tensile strength of conventional reinforcement and calls for a substantial reinforcement increase, the values according to other models are well below and compatible with the declared reinforcement.

Table 3: Summary of main modelling options for reinforced concrete linings of tunnels and shafts.

	Lining stiffness		
	Uncracked concrete	Preset value for cracked concrete	Lining stiffness adjusted based on expected structural behaviour
Impervious pipe model	Model A.1 $\Delta\sigma_s$ = 550 MPa	Model A.2 $\Delta\sigma_s$ = 274 MPa	Model A.3 $\Delta\sigma_s$ = 247 MPa
Pervious pipe model	Model B.1 $\Delta\sigma_s$ = 476 MPa	Model B.2 $\Delta\sigma_s$ = 191 MPa	Model B.3 $\Delta\sigma_s$ = 148 MPa

The example confirms that model selection plays a key role in the structural design of concrete linings for hydraulic tunnels and shafts. In fact, the modelling relevance can be so significant that other typical design inputs like load factors, maximum surge oscillations, and groundwater levels, which are often treated more in depth than modelling assumptions, may seem to become secondary. The designer of hydraulic tunnels and shafts should be aware of how much the design results for concrete linings are affected by modelling and support the corresponding assumptions with experienced judgement to ensure the structure can fulfil all structural and durability requirements during its design working life.

8 REAL STRUCTURAL BEHAVIOUR VS. DESIGN MODEL

The accurate modelling of the system behaviour is decidedly complex, with the most targeted approach likely being the monotonic loading increase proposed by Stucchi (2024). The procedure requires modelling of the progressive lining cracking, updating of the permeability calculation after each loading step as a function of the estimated number of cracks and by considering the corresponding stress redistribution given that, as mentioned above, the tensional state also depends on the permeability of the lining and the rock mass. Beside hydraulic pressure, the analysis should include thermal and concrete shrinkage effects that are likely to produce cracks in the lining even before the first waterway filling.

The natural variability of analysis parameters like the concrete strength and deformation properties, as well as the difficulty in predicting the effectiveness of contact and consolidation grouting and its effects on the system permeability, are examples of issues that even the most sophisticated deterministic model can hardly control. Therefore, for design purposes, the use of simplified models may be preferred, e.g. referring to Model B.2 in Table 2 instead of Model B.3 or paying attention to the assumed preset value of lining stiffness in order to not underestimate the lining demand.

9 DESIGN CODE PRESCRIPTIONS

The Australian standard AS 3600 for concrete structures supports non-linear analysis as the basis for design, requiring that “the calculations shall be undertaken using the mean values of all relevant material properties […], such as concrete strength, initial elastic moduli, and yield stress and yield strength of the steel reinforcement”. Furthermore, “additional analysis shall be considered using other values of material properties to allow for variability” and “checks shall be made to investigate the sensitivity of the results of non-linear stress analysis to variations in input data and modelling parameters”. For concrete linings of hydraulic tunnels and shafts, this typically means that the permeability of the cracked lining, determined with models based on number and width of the cracks and on assumptions for the seepage characteristics through the cracks, should be set conservatively low. The concrete tensile strength should instead be assumed conservatively high (at least equal to the mean tensile strength of the used concrete), as the number of longitudinal cracks induced by the internal water pressure will be less with increasing tensile strength, thus causing the lining to be stiffer.

No specific rules are included in AS 3600 for partial coefficients to be used for design based on non-linear analysis, meaning that the same partial coefficients shall be used as for conventional linear analysis.

The recently issued second generation Eurocodes (EN 1990:2023 with the basis of structural and geotechnical design, and EN 1992-1-1:2023 for the design of concrete structures) include similar rules, calling for sensitivity analysis with respect to significant material characteristics like concrete strength. With regards to the partial coefficients, the Eurocodes confirm those for actions as used for linear analysis, while the option of using different safety formats to determine the design value of the structural resistance is introduced. Similar safety formats are included in the FIB Model Code 2010, discussed in technical literature and deemed generally capable of yielding a less conservative design.

10 CONCLUSIONS

The evaluation of the hydraulic boundary conditions, lining and rock mass permeabilities, and lining stiffness, as well as the consequent adoption of a proper structural resistance model of the linings, require careful attention by the designer when performing the structural verifications of the concrete linings.

Overestimating the actual forces acting on the linings could lead to unnecessarily high costs or even render the project unfeasible. On the other hand, uncertainty of the hydraulic boundary conditions (and effective lining permeability) and of the linings stiffness, if underestimated, may result in underestimating the loads in the linings, and consequently result in an unsafe design.

The model selection for water pressure distribution and stiffness plays a key role in the structural design of concrete linings for hydraulic tunnels and shafts. Other typical design inputs, like load amplification factors, maximum/minimum surge oscillations, and groundwater design levels, which are often treated more in depth than modelling assumptions, may become secondary. The designer of hydraulic tunnels and shafts should be aware to what degree the concrete lining design results are affected by modelling and support the corresponding assumptions with experienced judgement to make sure that the structure can fulfil all structural and durability requirements during its design working life.

For design purpose, the expected structural behaviour of low-permeability linings is:

- Models B (pervious pipe model) for permanent (static) water loads; e.g. operation steady conditions
- Models A (impervious pipe model) or B for dynamic or transient water loads, depending on the application rate:
  - Models A for impulsive, short-term loads, e.g. water hammers and surge
  - Models B or intermediate between A and B for slowly applied loads, e.g. during dewatering operations

The use of simplified models may be preferred, e.g. referring to Model B.2 in Table 2 instead of Model B.3, while paying attention to the assumed preset value of lining stiffness. If the less conservative Model B.3 is used, parameters subjected to a natural variability should be selected cautiously in order not to underestimate the stresses in the lining.

11 REFERENCES

AS 3600:2018 *Concrete structures*

Deere D.U., Lombardi G. 1988, Lining of pressure tunnels and hydrofracturing potential, in *De Mello Volume*, Edgar Blücher Ltd

EN 1990:2023 Eurocode – *Basis of structural and geotechnical design*

EN 1992-1-1:2023 Eurocode 2 – *Design of concrete structures – Part 1-1: General rules and rules for buildings, bridges and civil engineering structures*

International Federation for Structural Concrete (FIB), *Model Code 2010*, 2012

Schleiss A. 1986, Bemessung von Druckstollen – Teil II: Einfluss der Sickerströmung in Betonauskleidung und Fels*, Mechanisch-hydraulische Wechselwirkungen, Bemessungskriterien*, VAW-Mitteilung 86, ETHZ

Stucchi R. 2024, Hydro-mechanical behaviour of reinforced concrete linings in hydraulic tunnels: steady state, in *Proceedings of the Institution of Civil Engineers – Engineering and Computational Mechanics*. Emerald. https://doi.org/10.1680/jencm.24.00020