DESIGN OF REINFORCED CONCRETE WATERWAYS

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ABSTRACT: This paper presents a study of the required steps for the design of reinforced concrete waterways. To achieve this purpose, there are two study-cases, with *horseshoe* and circular cross-section shapes, inserted in variable geotechnical conditions. Construction methods used for the construction of RC galleries are presented, taking into account the typical conditions of this type of structures and the associated details of underground constructions. A linear elastic analysis is performed in order to determine the inner-forces installed in the structure, with a finite element model in *FE* software and also with theoretical results of *Beggs* method, so they can be compared and the *FEM* model validated. At the same time, the rock- structure interaction is taken into account by evaluating its contribution to the overall behavior of the structure. The design of the study-cases is performed by security checks, according to the *Eurocodes*, associated to the most relevant ultimate limit states as well as the serviceability limit states according to the environmental and service conditions. It is also performed a nonlinear material analysis, with the *ICONC* software, in order to evaluate the failure mode and load bearing capacity of the structure, depending on the inclusion or not of stirrups. Finally it is performed an analysis of the transition between cross-sections with different shapes, circular and rectangular, where the most important aspects related with the situation are analyzed, with a *3d* linear elastic model. The transition is modeled with shell elements.

KEYWORDS: RC waterways, Rock-structure interaction, Linear analysis, Structural design, Non-linear analysis

1 INTRODUCTION

Tunnel liners are underground structures that provide the transport of water from a reservoir to a refund area. Usually they are made either with steel (penstock), or with reinforced concrete (Figure 1).

Generally, these structures are built as part of dams, in order to carry the stored water from the artificial lake until the power plant turbines; thus, one of the fundamental characteristic of waterways is the fact that the adduction water pressure represents an important load in the design.

The main goal of this task is to evaluate the required steps for the analysis and design of RC liners. As these underground structures are commonly inserted in rock means, it is also important to consider the interaction between the structure and the rock, which resists against inner loads. This consideration could provide more economic reinforcement ratios.

Figure 1 – Reinforced concrete liner (interior view) (EDP, 2011)

2 CONSTRUCTION METHODS

One of the most important steps of an underground tunnel project is its method of execution; essentially it has to take into account: (i) the longitudinal extension and (ii) the geotechnical characteristics of the construction area. A correct choice increases the economic viability of the project and the security during the excavation phase.

For galleries inserted in rock means, the most common execution methods are: (i) the *New Austrian tunneling method* (*NATM)*, (ii) *Drilling and blasting* (traditional methods) and *Tunnel boring machine* (*TBM)* (current method). The construction is finalized when the final equilibrium state is achieved, after the final support is installed.

Figure 2 – *NATM* excavation stages (Slides da disciplina de Obras Subterrâneas, 2010)

2.1 FINAL SUPPORT

The final support, for *TBM*, in most cases, is assembled through pre-casted concrete staves.

For traditional methods of construction, generally, the final concrete support is casted *in situ*. Before it, the reinforcement is installed according to the project specifications.

Usually, the concrete casting is performed in two or more stages: the first in the bottom of the gallery, so it serves as a support area for the casting of the remaining section at a later stage (Figure 3).

Figure 3 – Concrete casting in site (bottom) (EDP, 2011)

3 DESIGN OF THE CROSS-SECTION

In order to illustrate the design process, the geometry of two cross-sections is considered. This, the surrounding rock characteristics and the actions considered are based on a real project.

The geometry of the cross sections analyzed throughout this task is of: *horseshoe* and *circular* shapes (Figure 4).

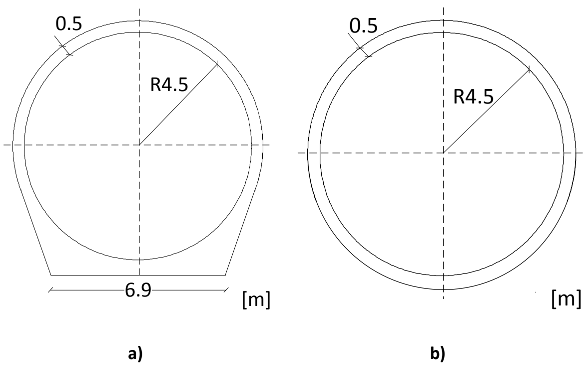


Figure 4 – Cross-section's geometry: a) *horseshoe* b) *circular*

For the analysis of the chosen cross-sections there are considered three different *Geotechnical Areas (GA)*, characterized by their *Young's* moduli (E_{rock}) (Table 1).

Table 1 – *Geotechnical areas* elastic moduli

Rock type	E_{rock} (GPa)
Bad (GA3)	1,5
Medium (GA2)	5,0
Good (GA1)	20,0

The acting loads (Table 2) considered are: (i) the dead weight of the structure (D), (ii) the surrounding rock pressure, which is divided in two areas: top of gallery (TR) and lateral pressures (LR), whose value is half of the top (Esteulle, Colombet, & Bouvard-Lecoanet, 1992) and also (iii) the hydrostatic pressures: exterior (W_{ext}), interior (W_{int}) (adduction water) and interior due to dynamic effects (*Water hammer*). The exterior type occurs due to rock fragmentation under the groundwater level; its value is conservatively considered the same of the interior (under static effects).

The hydrostatic pressures values are determined in function of the level difference between the center of the galleries and the water project level considered.

Table 2 – Acting loads

Load type	Value (kN/m^2)
TR	135
LR	67,5
W_{ext}	774
W_{int}	774
W_{dyn}	1125

The actions disposals are shown in Figure 5, for both cross-sections shapes. The load correspondent to hydrostatic pressure, as it is referred to the center of the conduit, is applied uniformly along the cross-section.

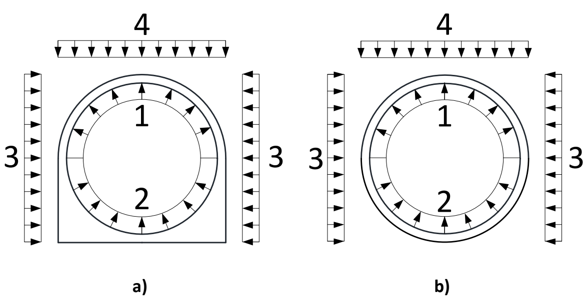


Figure 5 – a) *horseshoe* b) *circular*; Loads applied: 1) W_{int}; 2) W_{ext}; 3) LR; 4) TR

For structural design there are considered several limit states. Load combos for ultimate limit states and serviceability limit states are shown in Table 3 and Table 4, respectively.

Table 3 – ULS load combos (conditioning in gray)

Combo	D	LR	TR	W_{int}	W_{ext}
ULS 1	1,35	1,50	-	-	-
ULS 2	1,35	-	1,50	-	-
ULS 3	1,50	-	-	-	-
ULS 4	1,35	1,50	1,50	-	-
ULS 5	1,35	1,50	-	-	1,50
ULS 6	1,35	-	1,50	-	1,50
ULS 7	1,35	1,50	1,50	-	1,50
ULS 8	-	-	-	1,50	-
ULS 9	1,35	-	-	1,50	-

Table 4 – SLS load combos (conditioning in gray)

Combo	D	LR	TR	W_{int}	W_{ext}
SLS 1	1,0	1,0	-	-	-
SLS 2	1,0	-	1,0	-	-
SLS 3	1,0	-	-	-	-
SLS 4	1,0	1,0	1,0	-	-
SLS 5	1,0	1,0	-	-	1,0
SLS 6	1,0	-	1,0	-	1,0
SLS 7	1,0	1,0	1,0	-	1,0
SLS 8	-	-	-	1,0	-
SLS 9	1,0	-	-	1,0	-

4 ELASTIC ANALYSIS WITH *BEGGS* METHOD

In order to perform the structural design, it is of major importance the knowledge of the internal forces originated by the acting loads. The *Beggs* method (Phillips & Allen, 1968) provides a set of results, for the internal forces of cross-sections, whose values are tabulated in function of the load value and liner slenderness, for various load conditions and cross-section shapes.

The main contribution of these theoretical results is to understand, at a first look, the behavior of the studied galleries; when subjected to the considered load combos and how inner forces vary according to the geotechnical mean. On the other hand, it also serves as base for the *FEM* modulations, as it is possible to compare the results obtained and validate the model.

The *horseshoe* gallery available in *Beggs* tables is not completely equal to the case-study. However, it is considered that it represents a good base for comparison with the *FEM* modulation.

4.1 ACTING LOADS IN *BEGGS* METHOD

The loads available are all self-equilibrated and applied with uniform disposal, as it was shown before. However, *Beggs* models are free of restraints; so for vertical loads the foundation is assumed as a load, whose distribution is influenced by the modulus of elasticity of the foundation material. As the foundation modulus increases, the foundation load distribution approaches a concentration at the outside corners of the gallery, and as it decreases the load approaches a uniform distribution. However, the load correspondent to foundation is applied uniformly because it maximizes the internal forces.

4.2 ROCK-STRUCTURE INTERACTION

The design of reinforced concrete waterway is commonly controlled by the opening of radial cracks due to high tensile forces installed in the concrete liner, because of the inner water pressure. Thus, to provide economical reinforcement ratios and intelligent design of such elements, the rock-structure interaction must be considered along with the non-linear behavior of the liner by considering it cracked (Vaz Rodrigues, 2011).

This situation is considered, through an *indirect method* which divides the total internal pressure (P_{int}) *in rock* (P_{rock}) *and liner* (P_{ring}) *parcels* (equations 1, 2, and 3), according to (Vaz Rodrigues, 2011). This division is based on the system's interface compatibility.

The gallery is considered to be surrounded by a fissured rock layer which in turn is surrounded by bedrock (Moore, 1989). The rock is fissured because it is considered that the gallery is excavated using the *Drilling and blasting* method. For equations (4) and (5): ε_{cm} represents the concrete's contribution between cracks and σ_s the steel stress. The remaining variables are: the system radial deformation ($\frac{\Delta R}{R}$); rock Poisson ratio (ν); bedrock elastic modulus (E_r); inner radius (R); section height (h); reinforcement ratio (ρ); steel elastic modulus (E_s).

$$\boldsymbol{P_{int} = P_{rock} + P_{ring}} \tag{1}$$

$$\boldsymbol{P_{rock} = \frac{E_r}{1 + \nu + 2,772}\,\frac{\Delta R}{R}} \tag{2}$$

$$\boldsymbol{P_{ring} = E_s\,\rho\,\frac{h}{R}\left(\frac{\Delta R}{R} + \varepsilon_{cm}\right)} \tag{3}$$

$$\boldsymbol{\varepsilon_{cm} = \frac{k_t\,\frac{f_{ct,eff}}{\rho_{p,eff}}\,(1 + \alpha_e\,\rho_{p,eff})}{E_s} \leq 0,4\,\frac{\sigma_s}{E_s}} \tag{4}$$

$$\boldsymbol{\sigma_s = E_s\left(\frac{\Delta R}{R} + \varepsilon_{cm}\right)} \tag{5}$$

Design example (*circular* shape)

In this example it is shown graphically the internal pressure division in two parcels (*rock and structure*), through an interaction abacus (Figure 7), for the *circular* shape inserted in *GA3*. The analyzed cross-section has symmetric reinforcement disposal (Figure 6).

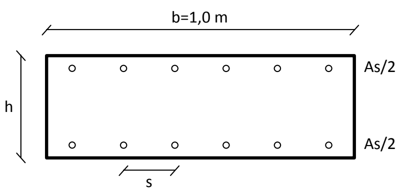


Figure 6 – Cross-section analyzed

The value of internal water pressure is 774 kN/m^2. This value should be multiplied by its partial safe coefficient in ULS (1.5), i.e. 1161 kN/m^2. The interaction comes in function of surrounding rock elastic modulus (*E*) and *ρh/R*. For the *circular* gallery the section height is 0.5 meters and the inner radius 4,5 meters. The rock elastic modulus is 1,5 GPa (*GA3*). Other relevant info for this example could be found in Table 5.

Table 5 – Design example data

Data	Value
Reinforcement disposal	Ø25//0.125
A_s (m^2/m)	0.0078
ρ	0.0156
$\rho h/R$	1,733x10^{-3}

It is possible to show graphically that the values obtained for the pressure absorbed by the *rock* is 554 kN/m^2 and the actual pressure for the gallery's design is 606 kN/m^2, instead of 1161 kN/m^2.

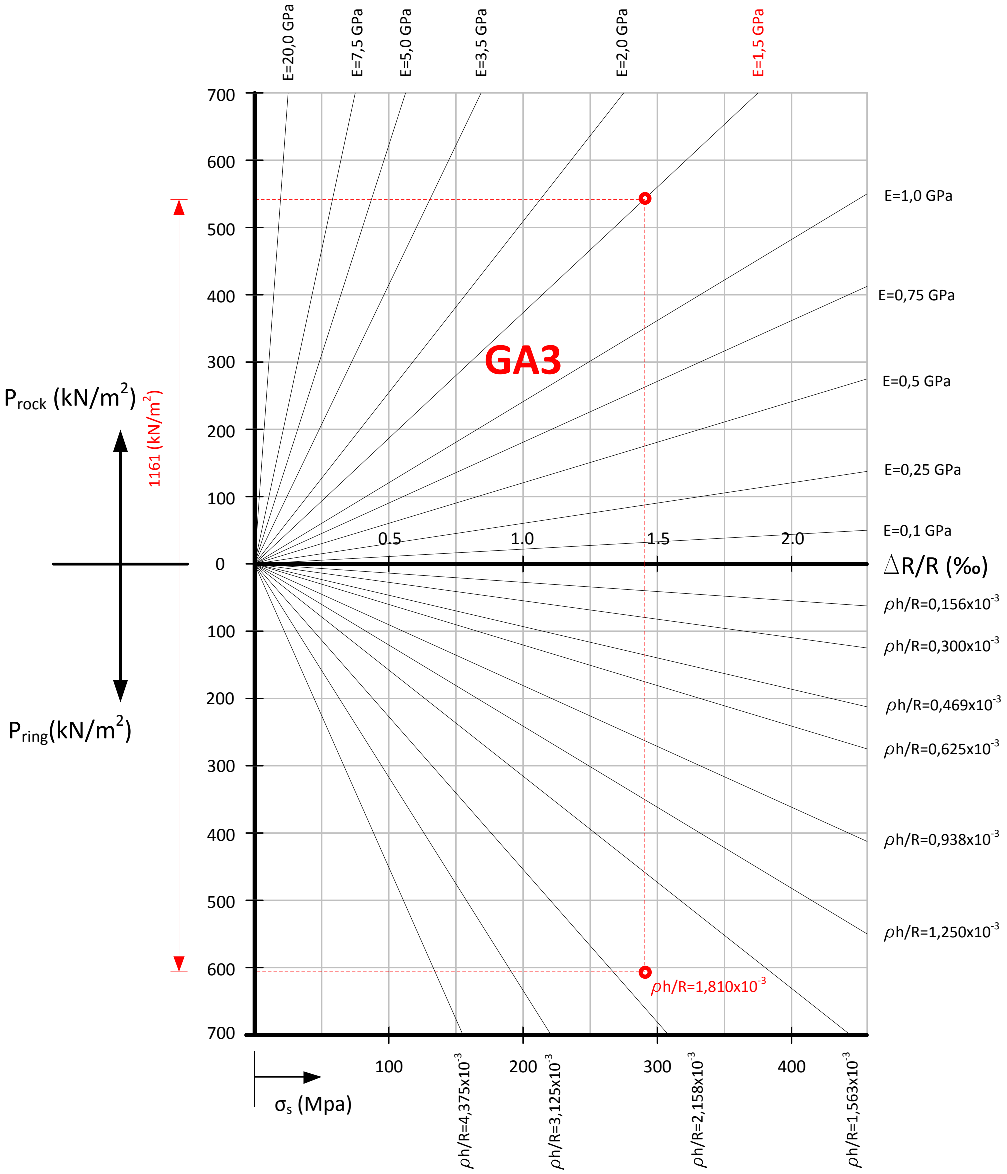


Figure 7 – Rock-structure interaction abacus (design example) (Vaz Rodrigues, 2011)

4.3 INTERNAL FORCES

In the analysis through *Beggs* method it is observed that the internal forces, mainly axial forces, increase significantly as weaker the *Geotechnical Area* is; this happens only for the *ULS 8* and *ULS 9* load combos, since the model used is free of restraints and the rock stiffness is only taken into account through the inner actions. At this level the *horseshoe* and *circular* shapes shows the same behavior.

As an example there are shown the axial forces diagrams, according to the *Geotechnical Area*; in Figure 8 and Figure 9, for *horseshoe* and *circular* shapes, respectively (N<0: tension).

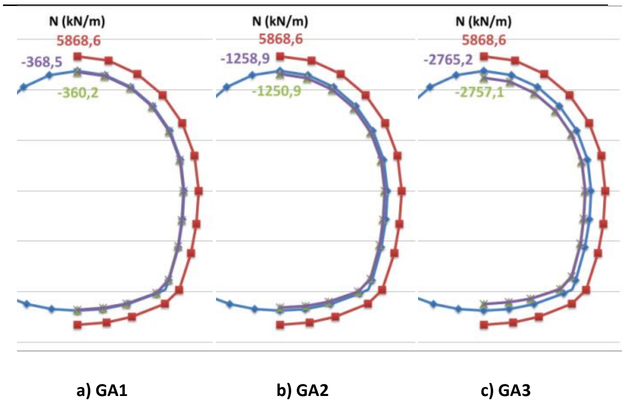
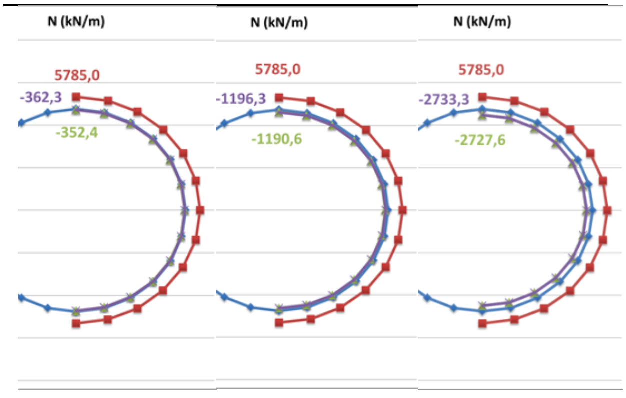


Figure 8 – *Horseshoe*: Axial forces (red: *ULS 7*; green : *ULS 8*; purple: *ULS 9*)



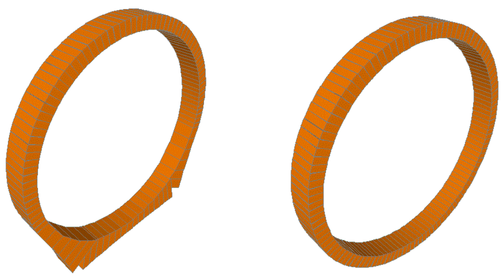
a) GA1 b) GA2 c) GA3

Figure 9 – *Circular*: Axial forces (red: *ULS 7*; green *: ULS 8*; purple: *ULS 9*)

5 Finite element analysis (FEM)

After the analysis with *Beggs* theoretical results, a *2d* finite element approach is performed. Now it is possible to model the structures with the case-study's geometry, namely the *horseshoe* gallery.

The modeling is performed with the *FE* software *SAP2000* (CSI, Computers and structures Inc., 2008); the galleries are modulated as linear *frames*, with respect to its centroid, with one meter depth (in the longitudinal direction) and appropriate mesh discretization (Figure 10); the criteria used for this, is that the internal forces shouldn't vary more than 5%.



a) b)

Figure 10 – Extruded view of FEM models: a) *horseshoe*; b) *circular*

5.1 Acting loads on FE analysis

The loads are applied in a similar way as in *Beggs* analysis, all uniform, with the exception of the foundation load that is already taken into account with *line-springs* (see *5.2*).

5.2 Boundary conditions

The way that boundary conditions are applied to the model depends on how the presence of surroundings is considered; by using a *FEM* software it is possible to consider the rock-structure interaction in different ways: (i) modeling the rock in finite elements, (ii) using springs with stiffness proportional to the rock elastic modulus (*hyperstatic reactions method*) or (iii) indirectly, by dividing the internal load in two parcels (rock and structure) (see 4.2).

Since it is a linear elastic analysis, it is possible to consider the rock mass in the most convenient way for each type of action; it is decided that for inner actions the *indirect method* may be used. It divides the applied load (P_{int}) in two parcels (equation 6), as it was described before.

$$P_{int} = P_{rock} + P_{ring} \qquad (6)$$

For the reaming actions the simulation of the rock is considered with the *hyperstatic reactions method*. This method consist in the application of non-linear springs that only work when compressed (Figure 11); whose stiffness is given by the equation (7) (Esteulle, Colombet, & Bouvard-Lecoanet, 1992).

$$k_{rock} = \frac{E_{rock}}{R\,(1+\nu)} \qquad (7)$$

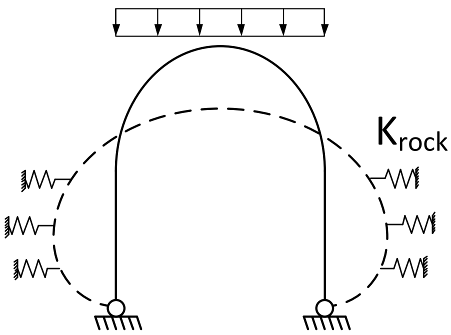


Figure 11 – Non-linear springs (only work when compressed) (Esteulle, Colombet, & Bouvard-Lecoanet, 1992)

When the *hyperstatic reactions method* is used, the line springs itself provides the model's support. When the *indirect method* is used, the structure needs to expand freely, because the presence of the rock is already taken into account directly in the load applied. Thus, special boundary conditions may be used for this situation.

5.3 Internal forces

It is observed that the internal forces, due to inner actions, increase significantly as weaker the *geotechnical zone* is, as it was noticed in *Beggs* analysis; now, it happens also for external loads. In *Beggs* analysis this aspect was not considered as the rock stiffness was not taken into account for external loads. As an example in (Figure 12) and (Figure 13), there are represented the internal forces diagrams for *horseshoe*, inserted in the higher stiffness *geotechnical area* (*GA1*). The diagrams are symmetric about the vertical centerline.

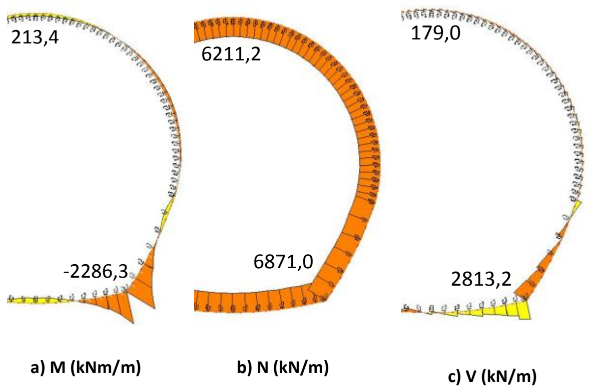


Figure 12 – Internal forces diagrams: *ULS 7*, *GA1* (*SAP2000*)

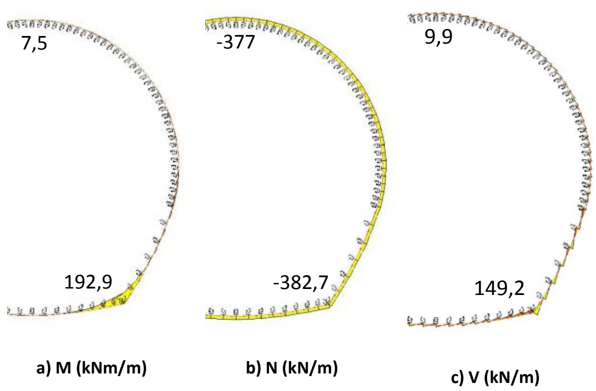


Figure 13 - Internal forces diagrams: *ULS 8*, *GA1* (*SAP2000*)

5.3.1 Comparison with *Beggs* method

By comparing the *circular* shaped gallery results, for the *internal water* and *lateral rock actions*, the relative error observed is approximately 5%. In order to validate the *horseshoe* shaped gallery model, from the study-case, the *Beggs horseshoe* gallery, slightly different from the study-case, was also modulated. For the same actions the error varies from 5-7%, in the constant thickness areas, to 15% in the variable thickness zones.

6 Safety checks (SC)

The structural design of the galleries is associated to safety checks, for each cross-section analyzed, according to the current regulations, the Eurocodes, namely: EN1990 (Bases to structural projects), EN1991-4 (Actions in silos and tanks) and EN1992-1-1 (Design of reinforced concrete structures). Thus, several limit states are evaluated according to the structure's geometry and environment conditions.

It is considered that the actions applied are resisted by the cross-sections; therefore in the longitudinal direction is only designed for long-term effects, namely concrete shrinkage, since the structure is restrained longitudinally.

The materials used are described in Table 6 and the concrete cover used is 5,0 centimeters, according to the environment where the galleries are inserted.

Table 6 – Material's class

Material	Class	Norm
Steel	S500NR	-
Concrete	C25/30 - D_{max} 32 S3	NP EN206-1 (EC2)

6.1 Ultimate limit states

In ULS it is evaluated: (i) the *combined bending and axial force*, (ii) the *shear resistance*, with and without transverse reinforcement and also (iii) the *interaction between bond and deviation forces in spalling failures of arch-shaped members* (Figure 14). Cover spalling failures, in arch-shaped members, are in general originated by the combination of two phenomena. The first one is the transverse tensile stresses due to the deviation forces of a curved reinforcement. The second one is the tensile splitting stresses originated by the bond of deformed reinforcement (Muttoni, Fernandez, & Plumey, 2010). In order to verify this situation, equation (8) must be checked, where: (i) d_b is the radial is the reinforcement bar diameter, (ii) k is an experimental coefficient that takes into account the bar deformation, (iii) b_{eff} is the effective width around the bar and (iv) R is the internal radius of the gallery.

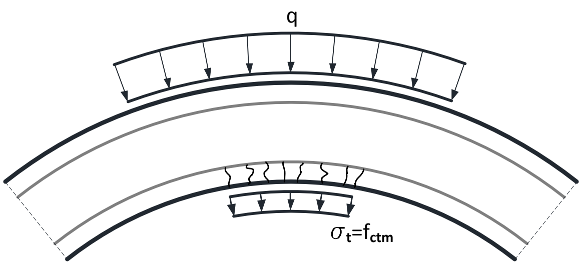


Figure 14 – Spalling failure in arched-shaped member, adapted from (Leonhardt & Monnig, 1979)

$$k\, f_{ctd}\, b_{eff} \geq \frac{f_{yd}\, \pi/4\, d_b^2}{R} \qquad (8)$$

6.2 SERVICEABILITY LIMIT STATES

In SLS it is evaluated the crack opening limitation: (i) along the cross-section and also (ii) in the longitudinal direction. Cracking limitation is very important for the structure's durability, since the galleries are inserted in a chemically aggressive environment and in "permanent" contact with water.

7 NON-LINEAR ANALYSIS

When the loads applied to a concrete structure reach a certain value, the linear elastic behavior is no longer valid; this situation happens due to two distinct reasons: (i) In general, for high loads, deformations are large, which leads to significant changes in the structure's geometry and consequent introduction of second order effects; (ii) the second reason is related to the occurrence of non-linearity in the constitutive relations of the materials. The concrete has low tensile strength and when cracking takes place the presence of reinforcement becomes relevant and plays a key role in terms of strength and stiffness of the piece. In this task, only non-linear material analysis is performed.

In the SC's, many times stirrups are not required, for the load combinations previously considered. In this chapter the *horseshoe* shaped gallery is evaluated under two concentrated loads (Q), applied laterally, to find out if Q_{plast} can be attained (Figure 15). This two models with and without stirrups are tested with typical radial reinforcement (obtained in *Safety Checks* chapter) (Table 7).

Table 7 – Reinforcement amounts

Reinforcement type	Amount
Radial	Ø16 // 20.0 (*GA1*)
Stirrups	Ø12 // 30.0

The model used (Figure 15) is performed with shell elements, with three nodes, through *ICONC* software (EPFL , 2012), with a *2d* analysis. The concrete tensile stress is neglected as its tensile strength is very low when compared with the reinforcement steel.

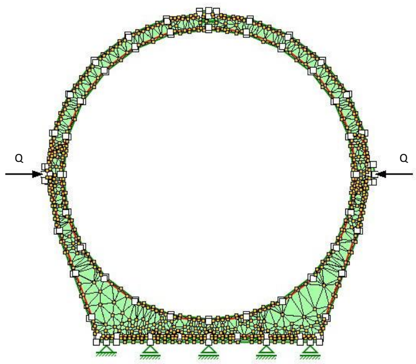


Figure 15 – *Horseshoe* gallery modulation (*ICONC)*

The boundary conditions consist in supports along the base and on top, in order to simulate the presence of rock.

7.1 RESULTS

With the purpose of understanding the structure's non-linear behavior, (i) the displacement ($\triangle$) and (ii) the steel stress (σ_s) are evaluated, at *point A*, which is over the interior radial reinforcement and in the load direction

(Figure 16). These parameters are evaluated as a function of the applied load (*Q*).

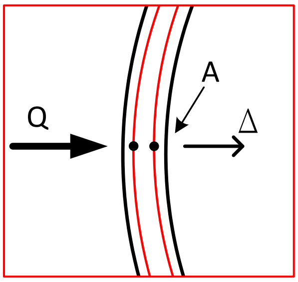


Figure 16 – Point A

After the analysis it is noticed that the load-bearing capacity, for the gallery without stirrups, is approximately 40% less than the gallery with it and it is also associated to a brittle failure mode. On the other hand, the gallery with stirrups features a "ductile" behavior as it is displayed in Figure 17, exploring the structure's resistant capacity in a better way.

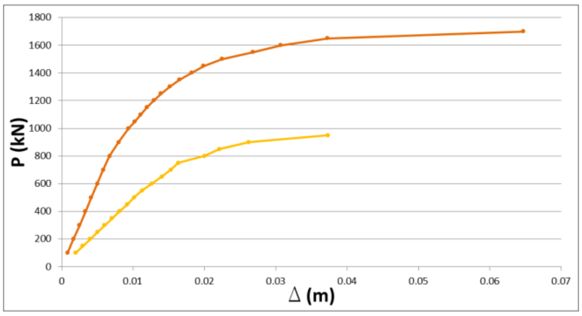


Figure 17 – Load-displacement plot (Orange: stirrups; Yellow: no stirrups)

8 Geometry of transition analysis

Generally, along with the longitudinal development of waterways, it is common to observe changes in the shape. In this task, it is analyzed the geometry of transition between a *rectangular* cross-section and a *circular* one, with the same geometry of the *circular* shape previously studied (Figure 18).

The main goal is to perform a *tridimensional* analysis of the galleries and to understand if the structural design changes a lot when compared with the constant shape waterways.

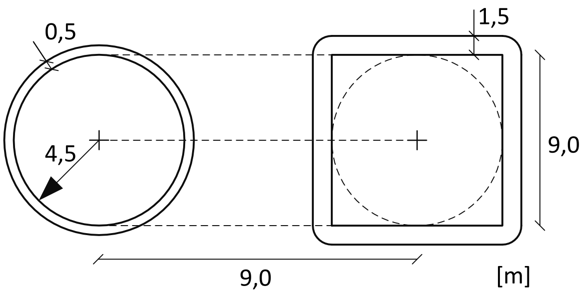


Figure 18 – Geometry of transition analyzed

In order to analyze the transition, a *3d* elastic analysis is performed. The structure is modulated with *shell* elements, with four nodes each, and the software used is *SAP2000* (Figure 19). The boundary conditions are based on the *hyperstatic reactions method* (see 5.2), as area springs, whose stiffness is considered to be uniform, taking as radius (*R*) the square side. This is a conservative hypothesis as it minimizes the spring's stiffness and increases the internal forces in the structure, namely for internal actions.

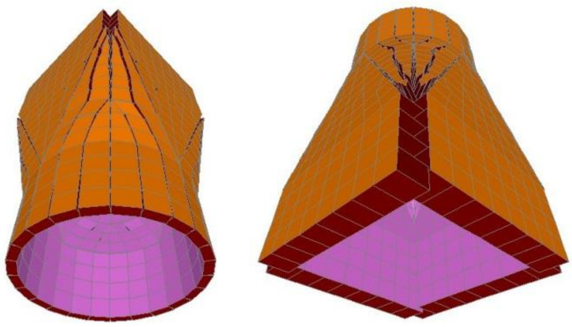


Figure 19 – Extruded view of the transition modulation (*SAP2000*)

After the analysis, the structural design is performed, with appropriate safety checks (as in the *SC's* chapter). Since it is not possible to use the *indirect method* of rock-structure interaction, as the transition hasn't a *circular* shape, the results from this pure elastic analysis may require more reinforcement amounts than the previous SC's, for the constant shape cross-sections.

9 Conclusions

This thesis presents the required steps for the analysis and design of underground concrete structures, by studying its details and presenting reinforcement drawing. It is also based on a real study-case, making this subject less abstract.

Along this task, several aspects were covered as: (i) the most relevant constructive processes, (ii) the elastic analysis, with *FEM* and theoretical results (*Beggs*), (iii) the rock-structure interaction, (iv) the structural design, according to the current standards and also (v) the non-linear analysis, that allowed to analyze the structural behavior of RC waterways through a more realistic perspective.

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