Structural design of a circular drop shaft with baffles

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Abstract

Within the scope of Plano Geral de Drenagens de Lisboa, Lisbon's City Council launched several projects for the control of water flow during storms, including intercepting wells in specific locations of the city.

In this dissertation, it is intended to design, based on data provided, an alternative solution for the drop shaft located on Avenida Almirante Reis.

Based on the geometric definition of the well structure, the construction method, and actions to which the structure is subjected were defined.

After the implementation of a finite element model, the results of the structural analysis were used to perform the safety checks of the structure according to the Ultimate Limit States and Service Limit States.

Finally, at the level of development of a preliminary study, the structural drawings including the geometric layout of the structure and rebar reinforcement were presented.

Keywords: Drop shaft, reinforced concrete, structural design, Lisbon's drainage, finite element model.

1 Introduction

1.1 Background

Due to the increase of the soil occupation and climate changes, more precisely the increase of the sea level and extreme precipitation phenomenon the city of Lisbon is subjected to an increasing risk of flooding.

One of the main objectives *of Plano Geral de Drenagem de Lisboa* (PGDL) [1] is to define short- and medium-term intervention plans that meet current and future challenges of drainage in the city, focusing on the protection of people and foods, considering economic, social and environmental sustainability.

According to the PGDL there are 3 great watersheds that are the base of the municipality's drainage system: The Alcântara system, the Beirolas system and the Chelas system.

According to the PGDL [1] the adjacent basins that drain to the municipality of Odivelas and Loures, and Lisbon's waterfront should also be considered. The combined system has a total area of 10 239 ha and is represented in Figure 1.

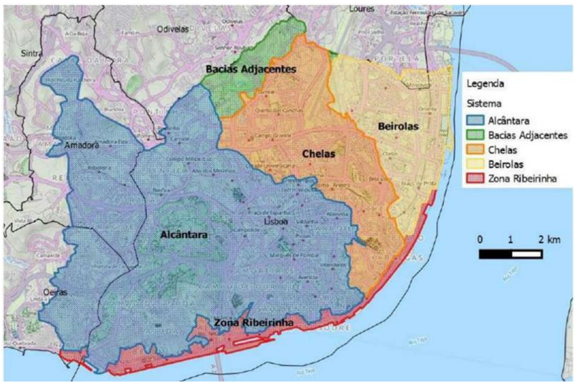

Figure 1: Lisbon's drainage systems

The objectives presented in the PGDL should be concretized in 2 phases.

The first phase has the objective of: updating all the available information, the revision of the previous plan, identification of the main drainage problems of the city and definition of the priority interventions.

The second phase gives continuity to the previous, not only verifying the relevance and feasibility of the previous phase, but also looking for alternative solutions that are more viable. The second phase also focuses on identifying and detailing the interventions that are more urgent in short-medium term.

For the 3 main drainage systems of the city the PGDL presents 3 different broad solutions that are based on the principles of increasing the system's capacity, flow deviation or the creation of water storage reservoirs, that can decrease the peak flow rate.

For the Alcântara system, it appears that solution C, associated with flow deviation, is technically the most favorable solution in controlling flood risk, allowing an adequate response to flows associated with 20-year return periods, with minimum surface interventions and, consequently, reduced social impacts.

The mentioned solution foresees the construction of a flow diversion tunnel between Monsanto-Santa Apolónia (TMSA) with a length of 5 km in which 3 intersection shafts are inserted, as shown in Figure 2.

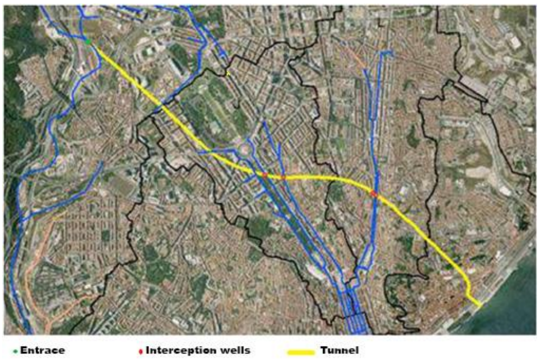

Figure 2: Tunnel and interception shafts location

According to the PGDL [1], for a return period of 100 years, a flow of about 130 m^3/s is diverted from Caneiro de Alcântara to the TMSA, which otherwise would go to Alcântara's downtown, where the flooding problems due to the Caneiro's discharge capacity are largely conditioned by the tide variations. In addition to diverting the flow of Caneiro de Alcântara, the tunnel allows diverting the flow that would otherwise go to Pombaline downtown, through the 3 interception shafts, located on *Avenida da Liberdade, Rua de Santa Marta* and *Avenida Almirante Reis.*

This work focuses on the structural design of the interception shaft located on *Avenida Almirante Reis.*

2 Structural definition

Due to the location of the interception shaft, the limitation of available space, and the socio-economic impacts, due to an open-air excavation with lane cut in one of the city's main avenues, it is proposed the development off a solution with a drop shaft with baffles

With the information provided was possible to design a more compact structure trying thus to reduce the negative impacts of the intervention.

In Figure 3 shows the 3D model of the interception shaft. The model can be divided in 3 structures: the flow deviation box, responsible for deviating the peak flow from *Alcântara* system into the drop shaft and consequently into the TMSA; the drop shaft, which the main purpose is to reconcile the flow that is being deviated with the TSMA's flow, and the tunnel which connects the drop shaft with the TSMA.

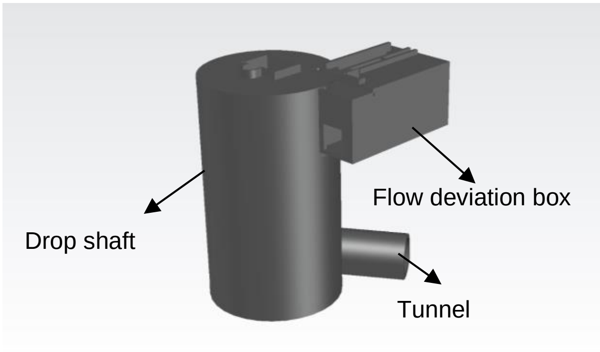

Figure 3: 3D Model of the interception shaft.

The structure of the drop shaft which reaches 19 m underground is generically constituted by an outside cylinder of reinforced concrete, with 0.35 m of thickness, with a foundation slab with 0.8 m and a top slab with 0.30 m thickness. The baffles have a variable thickness, between 0.25 m and 0.40 m.

The flow deviation box, that is connected to the drop shaft, has an area of approximately 10.65x 6.45 m^2 and a maximum height of 6.4 m. The outside walls and top slab have 0.30 m of thickness while the foundation slab and central wall have respectively 0.4 m and 0.5 m of thickness.

The tunnel was designed with a horseshoe section, with 2.8 m of maximum width and 3.0 m of maximum height. The tunnel invert slab has 0.50 m of thickness while the vault has a thickness of 0.30 m.

3 Materials

According to the NP EN 1990 [2] structures should be designed and built in such a way that during their lifetime and within a certain degree of reliability, they are able to withstand all the actions they may be subjected to during the period of construction and use.

Since the drop shaft is a large-scale structure, it should be designed for a lifetime of 100 years, but according to the original project, the flow deviation box is only be designed for a lifetime of 50 years.

Following NP 1992-1-1 [3] and LNEC E464 2005 [4] guidelines and considering that the exposure class of the structure is XC4, the material chosen for the structure were:

- C40/50 concrete with a nominal cover of 50 mm for the drop shaft and tunnel;
- C35/45 concrete with nominal cover of 40 mm for the deviation box;
- A500 NR steel bars.

4 Construction phasing

Due do the location of the interception shaft and its proximity to nearby buildings it's necessary to considerer the existence of a containment structure. One possible solution for the containment structure is the construction of a curtain of piles with a diameter of 0.80 m that should be built 6 meters below the shaft´s foundation slab. Figure 4 shows the containment structure.

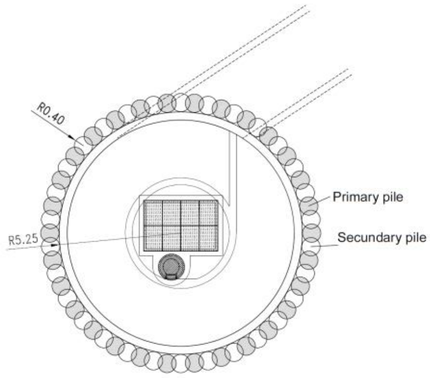

Figure 4: Containment structure representation

5 Project criteria

The interception shaft structure should be designed to verify the safety according to the Ultimate Limit States (ULS) and Service Limit States (SLS).

The ULS verifications for section strength were performed for the Fundamental Combination of loads and Seismic Combination of loads.

Regarding the SLS cracking and deformation will be checked, based on the Quasi-permanent combination of loads.

5.1 Load definition

To accurately design the structure, all of the loads that act upon the structure need to be identified.

The permanent loads considered along this work were: weight of the structure, soil loads, hydrostatic load and remaining permanent loads.

The variable loads considered were: road traffic load, train load, over soil live load, hydrostatic loading due to the peak flow (internal and external) and seismic action.

5.1.1 Structure self-weight

For the weight of the structure, a reinforced concrete density of 25 kN/m³ was considered.

5.1.2 Soil load

For the soil loads, the following soil proprieties were considered: saturated density of 21 kN/m³, modulus of elasticity (E') of 100 MPa and a soil angle of friction (ϕ') of 35º.

For the calculation of the soil horizontal impulses, the following equation was used:

$$I_h = \frac{1}{2} k\gamma h^2 \quad \textbf{(1)}$$

Where k is the coefficient of earth pressure, γ is the soil density and h is the soil height.

For the design of the tunnel, the soil load was calculated based on the height of decompressed soil over the tunnel, and was estimated using the empirical method of K. Terzahhi described in [5], with the following expression:

$$\boldsymbol{H_p} = \boldsymbol{K}(\boldsymbol{B} + \boldsymbol{H_t}) \quad \textbf{(2)}$$

Where H_p is the height of decompressed soil, K a factor depended of soil type, B and H_t are the tunnel width and height. The visual representation of these parameters is shown in Figure 5.

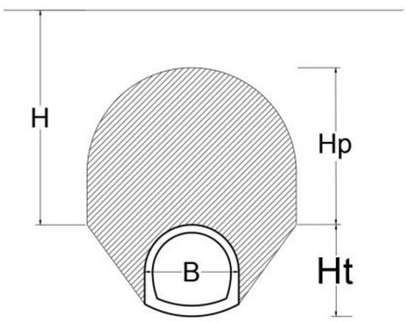

Figure 5: Representation of decompressed soil height calculation parameters.

5.1.3 Remaining permanent loads

The remaining permanent loads considered were the weight of soil and road pavement over the structure, both using a density of 20 kN/m³.

The weight of coating concrete inside of the derivation box was also considered.

5.1.4 Hydrostatic pressure

Since the shaft is an underground structure, the hydrostatic pressure load was considered. Since the groundwater level was not known beforehand, it was conservatively considered at the top of soil.

5.1.5 Road traffic load

The road traffic load was considered using the NP EN 1991-2 [6] guidelines.

Both the Load model 1 and Load model 2 were considered during the design of the structure.

5.1.6 Train load

Since [6] specifically states that the train loads that are described within the document don't apply for electric trams, the electric tram load described in article 50º of *Regulamento de Segurança e Ações* were used instead.

5.1.7 Over soil live load

An additional load of 10 kN/m² over the soil top was considered.

5.1.8 Hydrostatic load due to peak flow

Due to the presence of water inside the drop shaft during peak flow, the actions of a column of water with 4.5 m of height were considered over the steps and walls.

5.1.9 Seismic action

Seismic action was defined according with the NP EN 1998-1 [7]. In Table 1 the parameters used to define the seismic spectre are shown.

Table 1 -Parameters used for defining seismic specter.

	Type 1 earthquake	Type 2 earthquake
Seismic Zone	1.3	2.3
Soil Type	B	B
$a_gR\ [m/s^2]$	1.50	1.70
Importance class	II	II
γ_I	1.00	1.00
$a_g\ [m/s^2]$	1.50	1.70
S_{max}	1.35	1.35
S	1.29	1.27
q	1.5	0.10
$T_B\ [s]$	0.10	0.10
$T_C\ [s]$	0.60	0.25
$T_D\ [s]$	2.00	2.00
β	0.20	0.20

As the shaft is an underground structure the norm NP EN 1998-5 [8] was used to define the analysis method for the seismic action.

According to this norm, there are 2 simplified methods accepted for analysing the seismic action in underground structures: the *Mononobe-Okabe* method and the soil impulse for rigid structures method.

The soil impulse for rigid structures method is applied for rigid structures supported in rock or piles, where it is more appropriate to consider the resting state of the soil.

The document Stability Analysis of Concrete Structures [9] also states that the rigid structures method is more conservative that the *Mononobe-Okabe* and is more appropriate for non-yielding backfills.

Since the shaft is underground, the probability of an active state of the soil occurring is very low, consequently the rigid structures method was used.

In [8] the expression that allows to calculate the dynamic forces (ΔP_d) due to the increase in earth pressure is given as:

$$\Delta P_d = \alpha S \gamma H^2 \quad \textbf{(3)}$$

Where α is the ratio between the acceleration of the soil top and the gravitational acceleration, S is the soil coefficient shown in Table 1, γ is the soil density and H the soil height.

In the method, the kinetic F_k forces due to the acceleration should also be considered using the following expression:

$$F_k = m \times a_g \quad \textbf{(4)}$$

Where m is the mass of the element, and a_g is the acceleration at the soil top.

5.2 Ultimate limit states

The ULS are defined in [2] as the collapse or structural ruin of an element. According to the norm, the design load(E_d) should be inferior to the design strength of the element (R_d).

According to [2] the value of (E_d) should be defined using 3 different load combinations: Fundamental combination of loads, Accidental load combination and Seismic combination.

5.3 Service limit states

According with [2] the SLS are the conditions beyond which the requirements for the utilization of the structure stop being met.

For this project, the SLS that were checked where the crack width and deformation.

5.3.1 Crack opening

According with [3] the maximum crack width in any element (w_k) should be inferior to the maximum width (w_{max}).

The national annex of [2], states that for structures of XC4 class, w_{max} value is 0.3 mm.

Since the structure that is being designed is a hydraulic structure, the maximum crack limit should also be verified according with

EN 1992-3 [10]. Considering that the structure has a tightness class of 1, water leakage should be limited, and the maximum width for cracks that are expected to cross the entire concrete sections (w_{k1}) is 0.20 mm.

5.3.2 Deformation

According with [3], the structure deformation should be limited so its functionality is not affected. The maximum deflection in a beam, slab or console should be limited to a value of L_{spam}/250 for the quasi-permanent load combination. Nevertheless, a more current limit value of L_{spam}/400 was used for the structural design.

In order to calculate de deflection of an element in the long term(δ_∞) the following expression was used:

$$\delta_\infty = (1+\varphi)\delta_{t,0} \qquad \textbf{(5)}$$

Where φ is the creep coefficient, a value of 2.5 was used, and $\delta_{t,0}$ is de instantaneous deflection of the element.

5.4 Loss of equilibrium Limit State

According with NP EN 1997-1 [11] underground structures subjected to the groundwater level should verify the safety against hydraulic uplift (UPL). To verify the safety, the following equation should be true:

$$\boldsymbol{V_{dst;d}} \leq \boldsymbol{G_{stb;d}} + \boldsymbol{R_d} \qquad \textbf{(6)}$$

Where $\boldsymbol{V_{dst;d}}$ is the sum of the destabilizing loads, $\boldsymbol{G_{stb;d}}$ is the sum of the stabilizing loads and R_d is an additional resistance to the global uplift.

Since the structure is mostly hollow, and the groundwater level was very conservatively considered at the surface, the equation (6) could not me meet without considering the additional resistance.

In order to verify the equation, the lateral friction resistance of the piles (R_s), that can be included in the R_d, was estimated using the method described by Bourne-Webb, P in [12] using the following expressions:

$$R_s = \pi D L q_s \qquad \textbf{(7)}$$

$$q_s = k_s \sigma'_{v,avg}\ tg\phi' \qquad \textbf{(8)}$$

Where D is the diameter of the piles, L is their length, q_s is the piles resistance tension, k_s is the soil coefficient, $\sigma'_{v,avg}$ the average horizontal tension in the pile and ϕ' the friction angle of the soil. Considering the curtain of piles shown in Figure 4 the structural behaviour of the containment system was matched to a single pile with diameter of 12.1 m and length of 24.4 m.

In Table 2, the values calculated for each variable of (6) are presented.

Table 2 – UPL verification

$\boldsymbol{V_{dst;d}}$ (kN)	$\boldsymbol{G_{stb;d}}$ (kN)	$\boldsymbol{R_d}$ (kN)
26898.4	25114.5	35573.8

6 Finite element model

Since the interception shaft has a complex structure, a finite element program (*SAP2000*) was used in order to analyse the structural behaviour of the shaft. Figure 6 and Figure 7, show the finite element models used.

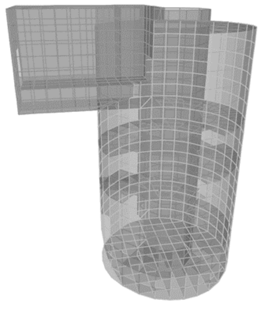

Figure 6: Finite element model of the drop shaft and deviation box.

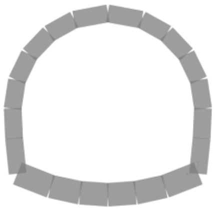

Figure 7: Finite element model of horseshoe tunnel

As both models should be continually supported in the soil, it is necessary to currently modulate the soil behaviour. For this, Winkler's method was used, allowing to simulate the elastic behaviour of the soil using coils of linear behaviour, thus considering that the reaction of the soil is the product between the reaction module of the soil (k_s) and the deformation that it is subjected to. The reaction module of the soil was determined using Vesic method (1961) using the following expression:

$$k_s = \frac{0.65E_s}{B(1-\upsilon_s{}^2)} \times \sqrt[12]{\frac{E_sB^4}{E_fI_f}} \qquad \textbf{(9)}$$

Where E_s and E_f are the elasticity modules of the soil and the foundation, υ_s is the Poisson coefficient, I_f is the moment of inertia of the foundation and B is the foundation width.

Using the aforementioned method, a reaction module of 55 000 kN/m/m² and 63 000 kN/m/m² was determined, respectively for the shaft and tunnel foundation.

Conservatively, on the shaft´s outer walls and the deviation box foundation slab a value of 35 000 kN/m/m² was used instead.

7 Structural design

7.1 Ultimate Limit States

In order to design the structure for the ULS, the following method was used.

An envelope of all fundamental and seismic combinations was created, in order to easily determine the maximum stresses acting upon the structural elements.

The maximum and minimum bending moments were determined and the normal stresses for the same section were determined.

If the section was compressed, a simple bending moment verification was done, in order to calculate the required rebar for that section.

If the section was subjected to tension stresses, a complex bending verification was done for the pair of M-N stresses, using the tables presented in [13].

The shear resistance of the elements without shear rebar ($V_{Rd,c}$) was calculated with an expression presented in [3]. If the shear resistance of the element was inferior to the shear stress, shear rebar area ($\frac{A_{s,w}}{s}$) would be calculated using the expressions shown in the norm [3].

7.2 Service Limit States

In order to design the structure for the SLS, the following method was used.

An envelope of all quasi-permanent combinations was created, in order to easily determine the maximum stresses acting upon the structural elements and also do determine the maximum deflections.

In order to verify the crack width, it was checked, in the first place, if the bending moment acting upon the element, for que quasi-permanent load combination, was superior to the cracking moment (M_{cr}) for the section. M_{cr} can be calculated using the following expression

$$M_{cr} = f_{ctm} \times w \qquad \textbf{(10)}$$

Where f_{ctm} is the mean tensile strength of the concrete and w module of bending resistance of the section.

For rectangular cross sections the value of w is calculated using the expression:

$$w = \frac{bh^2}{6} \qquad \textbf{(11)}$$

Where *b* is the section width and h the section height.

If the bending moment was inferior to M_{cr} no further verification was needed, since it means that the most stressed fibre has a tensile stress inferior to f_{ctm} and thus the section doesn't crack.

If the moment was superior, the neutral line position, rebar and concrete stresses were calculated using the tables presented in [13].

With the rebar tension stress the crack width was calculated using the method described in the norm NP EN 1992-1[3].

The element deformation was directly obtained from the model and the deflection was verified using the expression (5).

8 Conclusion

The performed safety verifications for the circular drop shafts with baffles, showed that the proposed solution is well adapted. It is expected that reduced impacts arise, as well as a smaller implantation surface for this type of structures, due to its compact design that is well adapted to urban sites.

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