Hydraulic design of the diaphragm's orifice at the entrance of the surge shaft of FMHL pumped-storage power plant

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ABSTRACT: The surge tank located at the upstream head of penstocks and shafts of hydropower plants is an effective measure to protect the headrace tunnels against high pressure fluctuations. It reflects the transient pressure waves that are generated by the variation of the flow discharge inside the powerhouse. The up- and down-surges inside the tank, as well as, the percentage of the transmitted pressure waves towards the headrace tunnel are significantly affected by the local hydraulic head losses situated at the entrance of the tank. The circular orifice diaphragm generates such a local head loss by contracting the flow inside the connection tunnel between the surge tank and the headrace tunnel. The diameter of the orifice is an important parameter to adjust in order to fulfill the hydraulic design criteria of the projects.

The construction works to double the actual 240 MW capacity of the existent pumped-storage power plant FMHL (Forces Motrices Hongrin-Léman SA) in Switzerland are now ongoing. This plant belongs to the shareholders: Romande Energie SA, Alpiq SA, Group E and the city of Lausanne. The works include the construction of a new powerhouse and a surge shaft of 7.2 m of diameter connected to the headrace tunnel by mean of a tunnel 28.5 m long with 2.2 m of internal diameter. The numerical calculations of transient flows carried out on the waterway system have shown that additional losses are needed at the entrance of the new surge shaft to keep the transient pressures inside the latter above the minimum value of 10 m W.C. The preliminary value of the interior diameter of a thin steel diaphragm has been determined by using analytical formulations. This diameter has been then optimized by series of physical tests carried out on a model at the geometrical scale factor of 1/18.2. The model covers a headrace tunnel reach of 152 m length (prototype scale), the connection tunnel and the surge shaft. It was operated respecting the Froude similarity within acceptable Reynolds limits for turbulent flow. Four splits of flow directions at the T-junction between the headrace and the connection tunnels were modeled. For each split, different discharge distributions between the three branches were tested with a main discharge that varies from 1 m³/s to 57 m³/s for generation mode and from 1 m³/s to 43 m³/s for pumping mode.

The first part of this paper presents a short description of the pumped-storage power plant. In the second part, the numerical model with its main results and the objectives of the additional hydraulic losses at the entrance of the surge shaft are presented. The third part provides the preliminary design of the diaphragm's diameter and the fourth part presents the results of the diameter optimization using the physical model tests. The conclusions are given in part five.

KEY WORDS: transient pressure, orifice diaphragm, local head loss, pumped-storage plant, surge tank.

1 INTRODUCTION

The FMHL pumped-storage power plant is situated in the Canton of Vaud, in the south-west part of Switzerland (Figure 1). It has been in operation since 1971 with a total installed capacity of 240 MW generated by 4 units inside an underground powerhouse. The scheme connects the 52 Mio m³ Hongrin Reservoir at el. 1255.00 m a.s.l. (the upper reservoir) to the natural 89'000 Mio. m³ Léman Lake at el. 372 m a.s.l. (the lower reservoir).

In order to enhance the flexibility of the scheme in generating peak energy, the owner has decided to double its installed capacity by constructing a new underground powerhouse located to the north of the existing one (Figure 1). This powerhouse will be equipped with two ternary units including Pelton turbine, motor-generator and multi-stage pumps of 240 MW of total capacity. The existing 8 km-long headrace tunnel, the 1.4 km-long pressure shaft and the water intake/outlet structures have all enough capacity to transfer the new generation and pumping discharges of 57 m³/s and 43 m³/s, respectively. Nevertheless, the transient calculations of the upgraded scheme have shown that the volume of the exiting surge tank will be deficient regarding the water mass oscillation. Therefore, a new surge shaft of 152 m length and 7.2 m of internal diameter will be constructed at the upstream end of the pressure shaft to the south-east of the existing one. It will be connected to the headrace tunnel by mean of 28.5 m-long tunnel with 2.2 m of internal diameter. The plan view and the longitudinal profile of the surge shaft and the connection tunnel are shown on Figure 2.

A consortium called "GIHLEM" composed of three international engineering companies Stucky Ltd, EDF and Emch+Berger have been mandated to carry out the final design and the tender documents and to assist the owner during the constructions works.
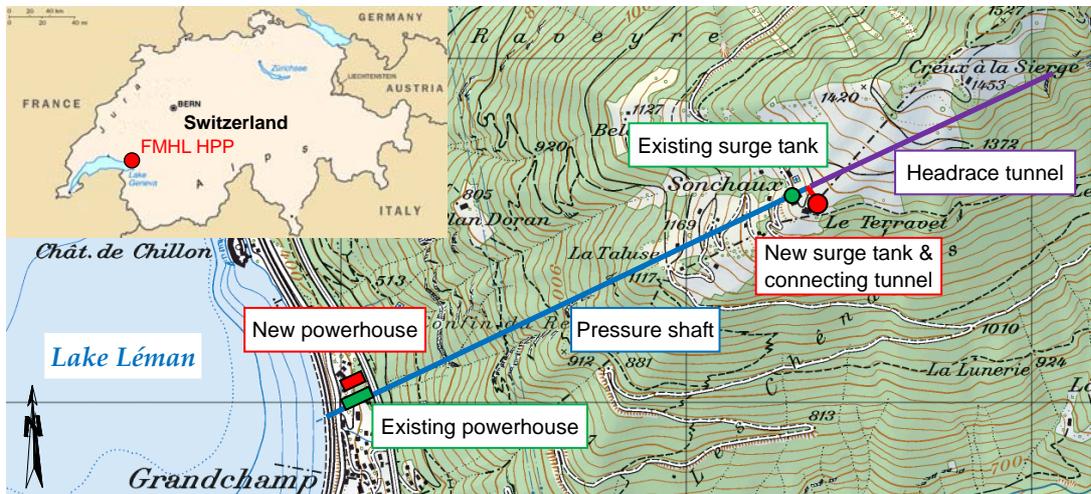
Figure 1 Left, General location of the FMHL pumped-storage plant; right, Plan view of the existing scheme and the locations of the new powerhouse, surge tank and connecting tunnel
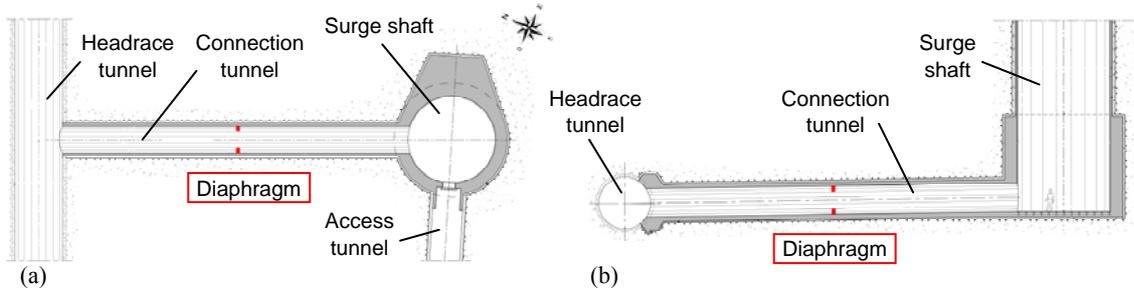
Figure 2 The new surge shaft and the connection and headrace tunnels; (a) Plan view; (b) Longitudinal section. The diaphragm inside the connection tunnel is also shown

2 NUMERICAL MODEL FOR TRANSIENT CALCULATION

The increase of capacity from 240 MW to 420 MW of an existing hydroelectric power plant is a challenging task regarding the hydraulic transient phenomena induced by both generating and pumping modes of operation. For the FMHL project, this increase of capacity was made possible by means of the construction of a new vertical surge shaft located at the upstream side of the pressure shaft. The upper ends of the surge shaft and the existing inclined surge tank will be connected together by a shallow underground gallery as illustrated in Figure 3. The transient behavior of the complex surge scheme was achieved through the collaboration of GIHLEM consortium and Power Vision Engineering Sàrl. An iterative process of transient simulation in pumping and generating modes in order to converge to the most cost effective solution has been carried out. This process has considered the construction methodologies and time schedule constraints, the geological formation and the operation modes. Once the general layout of the new surge tank was defined, its design has been optimized considering a diaphragm in the connection gallery between the bottom of the new surge shaft and the headrace tunnel as shown in Figure 2.
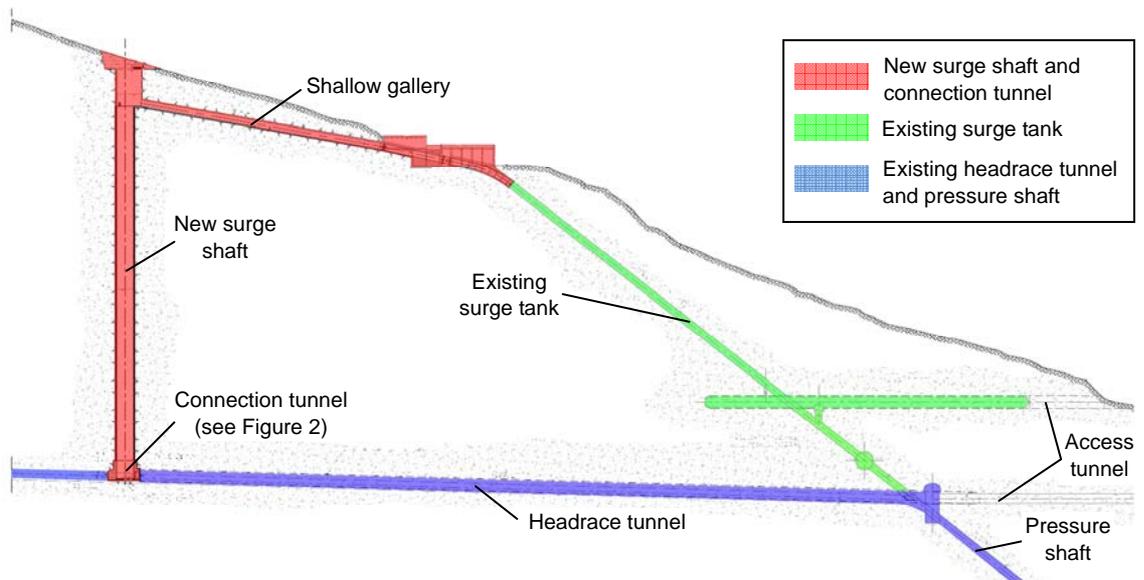
Figure 3 The new vertical and existing inclined surge shafts connected by a shallow gallery at the top. The connection tunnel between the new surge shaft and the headrace tunnel is also shown

2.1 Model description

Figure 4 presents the numerical model setup of FMHL project using the simulation software SIMSEN (Nicolet, 2007), developed by the Ecole Polytechnique Fédérale de Lausanne. This model includes:
-    The upper Hongrin and the lower Léman Lake reservoirs,
-    the 8000 m-long headrace tunnel,
-    the new vertical surge shaft and the modified existing surge tank,
-    the 1400 m-long pressure shaft,
-    the existing Veytaux 1 power house including 4 horizontal ternary units of 60 MW each, with Pelton turbine, multi-stage pump with a feeding pumps and a synchronous motor-generator,
-    the new Veytaux 2 power house, including 2 vertical ternary units of 120 MW each, with Pelton turbine, multi-stage pump, and a synchronous motor-generator.

The total installed capacity will be equal to 480 MW, but only 420 MW will be used for generating and pumping operation with 60 MW as reserve. The maximum gross head is 887 m WC.
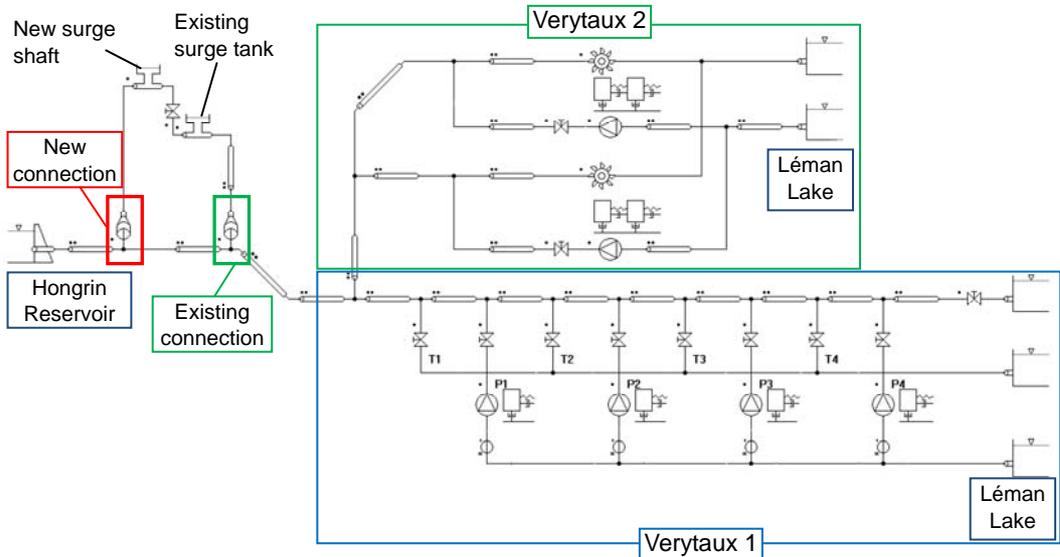
Figure 4 FMHL numerical scheme modeled by SIMSEN software. This scheme includes the two power houses "Veytaux 1" and "Veytaux 2", the waterways and the new and existing surge shafts

2.2 Diaphragm head loss optimization

The head losses between the surge tank and the headrace tunnel play an important role in the transient behavior of a pumped-storage power plant (Stucky, 1958; Steyrer, 1999; Richter *et al*. 2012; Alligne, et *al*. 2012). For the FMHL project, transient analyses have been carried out for normal, exceptional and accidental load cases for both generating and pumping modes. Also, the special load case of hydraulic short-circuits operation, possible with the 2 new units, has been considered. The transient analysis revealed that the 420 MW pumping mode operation at low water level inside the Hongrin reservoir leads to the most challenging conditions regarding the minimum pressure in the headrace tunnel. Therefore, the optimization of the diaphragm head losses was performed on the two following load cases:

-    <u>load case 1</u>: emergency shutdown of 5 units of 420 MW with the water level inside the Hongrin reservoir at el. 1210 m a.s.l.,
-    <u>load case 2</u>: emergency shutdown of 5 units at 420 MW with the water level inside the Hongrin reservoir at el. 1210 m a.s.l. with one of the spherical valves of the two new units (unit 5, 120 MW) is blocked.

The minimum admissible water pressures inside the headrace tunnel were considered as follows:

-    for load case 1 (normal load case), the minimum admissible pressure value was set to 10 m WC above atmospheric pressure,
-    for load case 2 (accidental load case), the minimum admissible pressure value was set to 0 m WC above the atmospheric pressure.

An error margin due to numerical uncertainties was added to the minimum pressure values obtained from the transient calculations before they are compared to the above minimum pressure limits.

Figure 5 presents the results of the numerical simulation of the load case 1. The transient behavior of the pump of unit 5 of "Veytaux 2" (Figure 5a), the penstock transient discharge and pressure (Figure 5b) and the water level oscillations inside the surge tanks with the head at their connections with the headrace tunnel (Figure 5c) are also depicted. The results show that the pump of Vetyaux 2 experiences reverse rotation and discharge only few seconds after shutdown. This reverse flow leads to very low pressure inside the pressure shaft and to low water level inside the surge tanks. After the emergency shutdown, the water level inside the existing surge tank, Hc_1, drops quickly relative to the one inside the new surge tank, Hc_2. This is due to the difference between the horizontal hydraulic surfaces of the existing and new surge tanks. The former is about 4 times smaller than the latter. The combination of both surge tanks leads to a differential surge behavior with fast water level response inside the existing surge tank which mitigates the low and high pressure values inside the headrace tunnel. The slower response of the new surge shaft prevents extreme low and high water levels to occur inside the differential surge system.

The simulations of load cases 1 and 2 were performed for several diaphragm head losses coefficients in order to find the optimal value with respect to:

- the minimum water level in the two surge tanks,
- the minimum pressure value inside the headrace tunnel,
- the minimum pressure in the diaphragm.

These simulations showed that for the FMHL project, the criterion of the minimum pressure inside the headrace tunnel is the most critical for design. The variations of this pressure (with and without the error margin) relative to the head loss coefficient of the new connection tunnel, Kd, for load case 1 are shown in Figure 6. This coefficient includes the friction head loss and the local head losses generated by the T-junction, the diaphragm and the inlet/outlet orifice at the surge shaft. It relates the total head losses, ΔH, to the flow discharge inside the connection tunnel, Q, according to Eq. (1) given below:

$$\Delta H = \frac{Kd}{2 \cdot g \cdot A_{ref}^{2}} \cdot |Q| \cdot Q \tag{1}$$

in which A_{ref} is the reference cross-section area of the connection tunnel and g is the gravitational acceleration.

The critical design load case is case 1, and the minimum pressure values in the most critical point along the headrace tunnel are induced by Kd values between 2.1 and 4.2. The optimal value of $Kd = 3.3$ was then selected as target value for the diaphragm design. The load case 2 generates lower pressure values inside the headrace tunnel relative to case 1. Nevertheless, the load case 1 is more restrictive than case 2 because the minimum admissible pressure value of the latter case is 0 m WC. The optimal coefficient was then considered in the transient analysis performed by the pump manufacturer and electromechanical suppliers, VOITH Hydro Holding GmbH, for more than 32 different load cases. The results have shown that all the design criteria of the project are fulfilled (Lippold and Hellstern, 2012).

a)

b)

c)
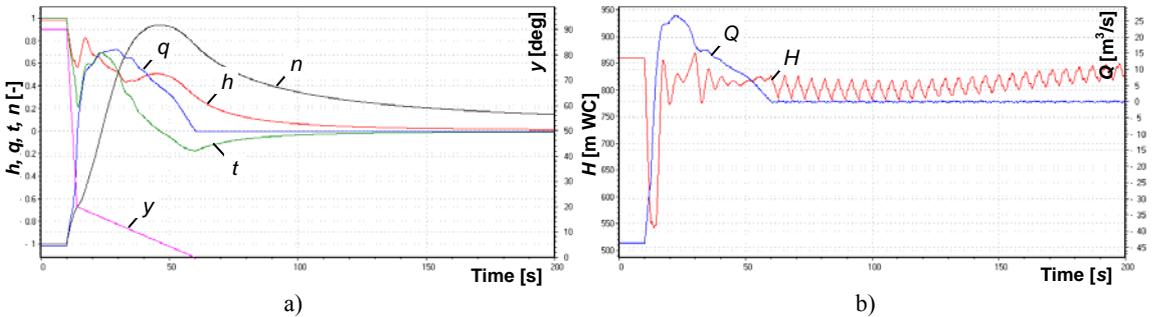
Figure 5 Numerical simulation results of the FMHL hydraulic scheme for the emergency shutdown of 5 units of 420 MW with water level inside the Hongrin reservoir at el. 1210 m a.s.l., a) transient behavior of the pump (unit 5) with h is net head relative to the maximum head, q is the relative discharge, t is the relative mechanical torque, n is the relative rotational speed, and y is the pump spherical valve opening; b) time histories of the pressure, H and discharge, Q at the toe of the pressure shaft; c) time histories of the water levels inside the existing and new surge tanks, Hc_1 and Hc_2, resp., and the head at the existing and new connections with the headrace tunnel, H_1 and H_2, resp.

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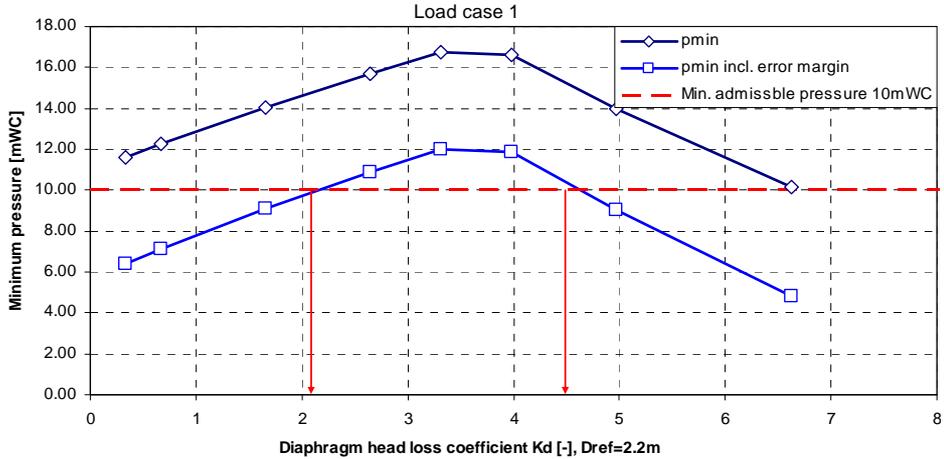
Figure 6 Variation of the minimum pressure at the critical point of the headrace tunnel for load case 1 relative to the total head loss coefficient, *Kd*, due to friction, T-junction, diaphragm and inlet/outlet orifice of the connection tunnel

3 DESIGN OF THE DIAPHRAGM'S DIAMETER

The numerical simulations have shown that the optimal coefficient of the head losses generated along the connection tunnel is equal to 3.3. In order to determine the diaphragm's diameter, the part of the local head losses generated by the diaphragm alone has to be computed. The local head losses along the connection tunnel are the sum of: (i) the linear head loss generated by the friction along the tunnel's wall, (ii) the local head loss at the T-junction, (iii) the local head loss generated by the contraction at the diaphragm, and (iv) the head loss at the inlet/outlet orifice at the new surge shaft. The local head losses relative to the cross-section area of the connection tunnel can be written according to Idel'cik (1960), as follows:

$$Kd = \lambda \cdot \frac{L}{D} + \xi_1 + \xi_2 + \xi_3 \tag{2}$$

in which λ is the coefficient of head loss per linear meter of the tunnel, *L* and *D* are the length and internal diameter of the connection tunnel, respectively, and ξ_1, ξ_2 and ξ_3 are the local head loss coefficients at the inlet/outlet orifice, at the T-junction and at the diaphragm, respectively.

The friction coefficient value for the tunnel's wall expressed in term of the equivalent sand grain roughness, *k*, is taken equal to 2.5 mm. According to Idel'cik (1960), the value of λ which corresponds to this *k* value is equal to 0.025. At the inlet/outlet orifice, the local head loss coefficient ξ_1 is equal to 0.5. This value considers the direction of flow from the surge shaft towards the connection tunnel. At the 90 deg T-junction, the value of the coefficient ξ_2 depends on the ratios between discharges and cross-section areas of the connection and headrace tunnels. For a cross-section ratio of $A_{ref}/A_{headrace} = 0.318$ and a discharge ratio of $Q_{ref}/Q_{headrace} = 0.8$, the coefficient $\xi_2 = 0.821$. Thus, Eq. (2) leads to $\xi_3 = 1.655$ for $Kd = 3.3$. The ξ_3 coefficient is related to the ratio of cross-section areas of the diaphragm and the connection tunnel, as follows:

$$\xi_3 = \left(1 + 0.707\sqrt{1 - A_d/A_{ref}} - A_d/A_{ref}\right)^2 \left(A_{ref}/A_d\right)^2 \tag{3}$$

in which A_d is the interior cross-section area of the diaphragm.

Therefore, the first estimation of the diaphragm's diameter, determined from Eq. (3) for $\xi_3 = 1.655$, is equal to 1.73 m.

4 PHYSICAL MODELING FOR OPTIMIZING THE DIAPHRAGM'S DIAMETER

The head losses generated by the diaphragm are very sensitive to the change of its internal diameter. Therefore, it was decided to build an experimental set-up for the new surge shaft, the connection tunnel and the T-junction and including short upstream and downstream reaches of the headrace tunnel. The physical model experiments are an efficient tool to quantify the total head losses caused by the interactions between the different elements inside the connection tunnel. It is also fundamental to estimate these losses in both flow directions from and towards the surge shaft. The T-junction acts as junction or bifurcation and the intersection of the tunnel with the surge tank is an inlet or outlet depending on the direction of the flow.

4.1 Experimental set-up and instrumentation

The physical scaled model was built in the Laboratory of Hydraulic Constructions LCH of the Ecole Polytechnique Fédérale de Lausanne EPFL according to Froude similarity which preserves the ratio of inertial to gravitational forces in model and prototype. It has the scale of 1/18.2 (Figure 7).

The tests have been carried out on four different geometrical configurations of the experimental set-up. The configuration (C0) is the basic configuration which has no diaphragm inside the tunnel. The configurations (C1) and (C2) have a diaphragm with an internal diameter of 1.55 m and 1.70 m, respectively. It is located at 12 m from the T-junction in both configurations. In the last configuration (C4), an intermediate diameter of the diaphragm of 1.63 m was tested by following a full test program with different main discharges and flow splitting at the T-junction. The synthesis of these configurations is presented in Table 1.

The measurement equipments used comprise 6 piezoresistive pressure sensors of type "Keller"- series 25. Each pressure sensor was connected to 4 water outputs located symmetrically around the conduit at each measurement section. The mean pressure values obtained from 1000 samples acquired at a frequency of 100 Hz at the 6 measurement sections shown in Figure 8 were collected. The level in the surge shaft was measured visually on a graduated ruler. The data acquisition device is a National Instruments card type USB-6259 series M driven by a LabView routine developed in the laboratory.
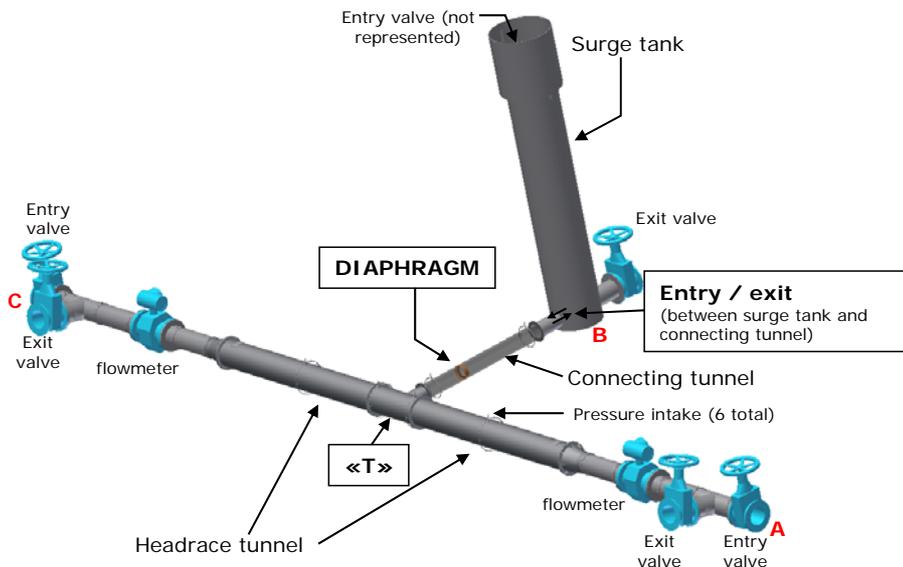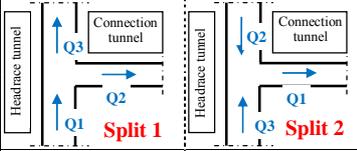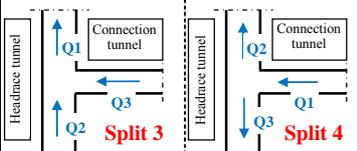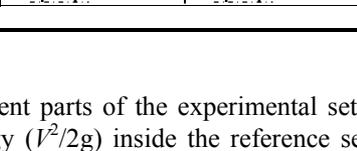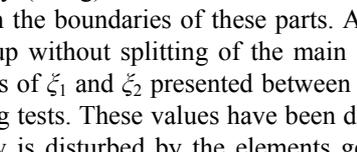
Figure 7 Schematic 3D view of the physical set-up with the diaphragm inside the connection tunnel

Table 1 Configurations of the experimental set-up and the test program with different main discharge values and directions and different flow splitting

Configurations			
C0 Without diaphragm	**C1** With diaphragm d=1.55 m	**C2** With diaphragm d=1.70 m	**C3** With diaphragm d=1.63 m
Flow direction			**Flow direction and splitting**
A → B	A → B	A → B	
B → A	B → A	B → A	
C → B	C → B	C → B	
B → C	B → C	B → C	
	A → C	A → C	
	C → A	C → A	

The C3 "Flow direction and splitting" sub-table:

Mode	Flow direction		Main discharge Q1 [m³/s]	Flow splitting
Shutdown of turbines	Headrace tunnel — Q3 — Connection tunnel	Headrace tunnel — Q2 — Connection tunnel	57, 49, 43, 33, 24, 15, 6, 1	Q2 ; Q3 1.0·Q1 ; 0.0·Q1 0.8·Q1 ; 0.2·Q1 0.6·Q1 ; 0.4·Q1 0.5·Q1 ; 0.5·Q1 0.4·Q1 ; 0.6·Q1 0.2·Q1 ; 0.8·Q1 0.0·Q1 ; 1.0·Q1
Start up of pumps	Headrace tunnel — Q2 — Q1 **Split 1**	Headrace tunnel — Q1 — Q3 **Split 2**	1, 6, 15, 24, 33, 43	
Start up of turbines	Headrace tunnel — Q1 — Connection tunnel	Headrace tunnel — Q2 — Connection tunnel	1, 6, 15, 24, 33, 43, 49, 57	
Shutdown of pumps	Headrace tunnel — Q3 — Q2 **Split 3**	Headrace tunnel — Q1 — Q3 **Split 4**	1, 6, 15, 24, 33, 43	

4.2 Test results

The local loss coefficients ξ_i of the different parts of the experimental set-up are the slope of the linear curve fit between the flow kinetic energy ($V^2/2g$) inside the reference section of the connection tunnel and the head loss ΔH_i measured between the boundaries of these parts. A summary of the results for the different configurations of the test set-up without splitting of the main discharge of 57 m³/s (at prototype scale) is shown in Table 2. The values of ξ_1 and ξ_2 presented between parentheses have shown high disturbances and standard deviations during tests. These values have been determined from pressure sensors located around sections where the flow is disturbed by the elements generating the local head losses. Therefore, these values have been discarded and the coefficients of the head loss induced by the diaphragm have been computed by subtracting the values of the total coefficient Kd with diaphragm from the value of configuration C0 (values depicted in bold in Table 2). The values of ξ_3 for the diaphragm of d = 1.55 m computed for the flow from and towards the surge shaft are different. This is caused by the interaction between the local head losses generated by the T-junction and the diaphragm with the smallest internal diameter. Additional tests will be carried out to determine the variation of the head losses relative to the distance between the diaphragm and the T-junction. These tests will be done for different flow splitting and different internal diameters of the diaphragm.

The variations of the total loss coefficient as a function of the discharge splitting rate are shown in Figure 8 for the test configuration C3. For splits 2 and 4, the main discharge Q1 flows inside the connection tunnel and, thus, the coefficient is rather constant. This is not the case for splits 1 and 3 where the total loss coefficient increases from 0.1 to 3.4 and 3.8, respectively.

According to Blevins (1984), the head loss coefficient ξ_3 caused by the diaphragm can be written as follows:

$$\xi_3 = \alpha \left(\frac{D}{d} \right)^4 \left(1 - \left(\frac{d}{D} \right)^4 \right) \Big/ C^2 \tag{4}$$

in which α is a parameter function of the ratio of diaphragm to tunnel diameters and C is the discharge coefficient of the diaphragm. By taking C = 0.628 and by fitting Eq. (4) to the values of ξ_3 shown in Table 2, the parameter α can be obtained. It is equal to 0.392 if the flow direction is from the surge shaft and 0.347 if the flow is towards the surge shaft. For ξ_3 = 1.655, the d is equal to 1.72 m (flow from the surge shaft) and 1.69 m (flow towards the surge shaft). These values are very close to 1.73 m

computed by Idel'cik formulation (Eq.(3)).

Table 2 Kd and ξ_i values for the different configurations and flow directions inside the test set-up and without discharge splitting (Q1=57 m³/s)

Flow from the surge shaft							Total head
Configurations	Kd	ξ_1	ξ_2	$\xi_{3,\,d=1.55\,m}$	$\xi_{3,\,d=1.70\,m}$	$\xi_{3,\,d=1.63\,m}$	loss [m WC]
C0	1.5	0.6	0.9	-	-	-	17.2
C1	4.6	(0.6)	(0.6)	**3.1** (3.4)	-	-	52.7
C2	3.2	(0.7)	(0.8)	-	**1.7** (1.7)	-	36.7
C3	3.8	(0.6)	(0.7)	-	-	**2.3** (2.5)	43.5
Flow towards the surge shaft							
C0	1.2	0.4	0.7	-	-	-	13.8
C1	3.7	0.4	(0.6)	**2.5** (2.7)	-	-	42.4
C2	2.9	0.5	(0.7)	-	**1.7** (1.7)	-	33.2
C3	3.4	0.6	(0.5)	-	-	**2.2** (2.3)	39.0

i) Values between parentheses are measured by pressure sensors located inside a hydraulic disturbed zone.
ii) Values in bold of ξ_3 are determined by subtracting the value of Kd with diaphragm from the value of configuration C0.
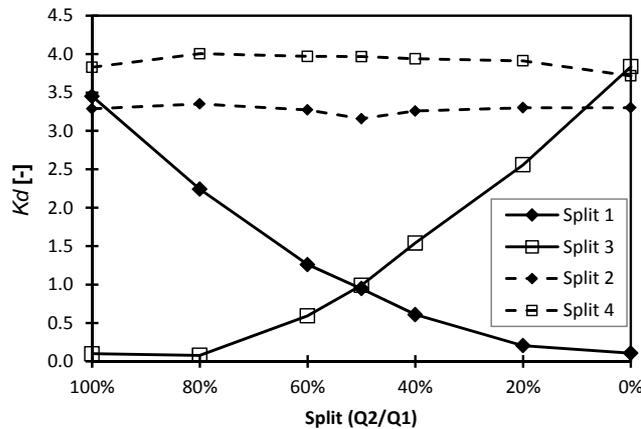
Figure 8 Variations of the total loss coefficient Kd as a function of the discharge splitting rate (Q2/Q1) for the test configuration C3

5 CONSTRUCTION OF THE DIAPHRAGM

The diaphragm was fabricated and erected by Alstom SA who is in charge of all the steelworks of the FMHL project. The pumping and generation activities have been interrupted during winter 2013 (from January to May) for the rehabilitation of the intake valves located at Hongrin dam. During this period, the connection tunnels for the new surge shaft and for the new powerhouse have been excavated and the steel lining including the diaphragm of 1.63 m of internal diameter has been erected (Figure 9).

Figure 9 Welding of the diaphragm inside the connection tunnel

6 CONCLUSIONS

The new hydraulic transient phenomena induced by the future generating and pumping modes of the modified FMHL pumped-storage plant require the construction of a new vertical surge shaft located at the upstream end of the pressure shaft. In order to guarantee acceptable minimum water pressure inside the headrace tunnel, the numerical calculations of the entire pumped-storage scheme have shown that a diaphragm in the connection tunnel between the bottom of the new surge shaft and the headrace tunnel is needed. The optimal coefficient of the total head losses caused by friction, diaphragm, inlet/outlet, and T-junction relative to the cross-section area of the connection tunnel is equal to 3.3. The preliminary hydraulic design of the diaphragm has led to an internal diameter of 1.73 m. The design has then been optimized by series of physical tests carried out on an experimental set-up according to Froude similarity at the geometrical scale of 1/18.2. It covers the new surge shaft, the connection tunnel, the T-junction and short upstream and downstream reaches of the headrace tunnel. The tests have been carried out on four different geometrical configurations with one without diaphragm and three others with diaphragm of internal diameter of 1.55, 1.70 and 1.63 m. The configuration with the latter diameter was tested for different main discharges and flow splitting at the T-junction. For this diameter, the experimental head loss coefficient Kd is equal to 3.8 and 3.4 for the flow direction from and towards the surge shaft, respectively. The internal diameter of 1.63 m for the diaphragm has been finally retained. It has a higher loss coefficient compared to the optimal value of 3.3 determined by the numerical analysis and is 0.1 m lower than the value determined from mathematical models.

7 ACKNOWLEDGMENTS

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