Evaluation of leakage in a partially unlined pressure tunnel at Casecnan

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The 160 MW Casecnan multipurpose project in the Philippines has recently been completed. The US$ 500 million BOPT project involved major underground works, including an essentially unlined 26 km-long pressure tunnel. A model developed to estimate water leakage and the measured behaviour during filling is discussed here.

The Casecnan project in the Philippines is an irrigation and hydropower scheme in the northern part of Luzon island, almost 150 km north of Manila. The 160 MW BOPT project will not only increase power generation on the island by 400 GWh/year, but will also contribute to the irrigation of 31,000 ha of agricultural land.

The plant includes two intake weirs diverting flow from the Taan and Casecnan rivers into a 24 km-long power tunnel, feeding two units in an underground powerhouse. A 2 km-long tailrace tunnel releases up to a total nominal flow of 80 m³/s into the existing Pantabangan reservoir.

Less favourable geological conditions than expected, combined with an already defined tunnel alignment, resulted in unusually challenging engineering difficulties, which became evident as the construction works progressed.

One of the key issues is related to the leakage of the essentially unlined power tunnel, where the overburden is significantly below the internal head on long sections.

Geological and hydrogeological data collected during the excavation progress were used to calibrate a numerical model to evaluate the water leakage of the most critical tunnel sections for various operating conditions.

1. Main project features

As can be seen in Fig. 1, the Taan and Casecnan rivers flowing northwards are intercepted just upstream of their confluence at an elevation of approximately 435 m and diverted southwards into the existing Pantabangan reservoir. To provide some daily peaking capacity for the scheme, two gravity dams impound the Taan and Casecnan daily storage basins.

The Taan dam is 12 m high, with a crest length of 90 m, and the Casecnan dam is 30 m high with a crest length of 120 m. Both structures are founded on competent and relatively impervious andesites and agglomerates, and are equipped with relatively large gated spillways. The photograph above shows a downstream view of the completed and impounded Casecnan basin. The reservoirs are interconnected with the 2.5 km long Taan-Casecnan tunnel, which has a maximum flow capacity of 40 m³/s.

Five underground desilting basins, approximately 35 m long and with a 63 m² hydraulic section, are located on the left bank of the Casecnan dam. After passing these basins, the flows enter the transbasin tunnel subdivided into an upper almost horizontal drive and a lower inclined drive.

The upper and the lower drives are separated at the Abuyo river, and the transbasin tunnel runs underneath, only a few metres below the riverbed.

From a geological point of view, the 16 km-long upper drive is located in generally competent andesite and agglomerate formations. Hydrothermal weathering and some fault zones caused local difficulties for the tunnel excavation, carried out with two 6.5 m-diameter TBMs, proceeding from the Casecnan weir downwards and from the Abuyo river upwards, respectively.

The 7.5 km-long lower drive from the Abuyo river to the powerhouse is essentially located in granodiorite
and andesite rocks of highly variable properties. The 42 m² section was excavated by the drill-and-blast method, with an intermediate adit at the location of the surge shaft. The lower drive descends, with a longitudinal slope of between 2 and 5 per cent, directly to the powerhouse elevation (see Fig. 2) reaching a maximum internal hydrostatic pressure of 270 m for a nominal flow capacity of 80 m³/s.

Both the upper and the lower drives are essentially unlined tunnels. A 270 m-long steel lining was placed to underpass the Abujo river, and a 180 m-long tunnel section was provided with a steel lining upstream of the powerhouse. Concrete lining was placed for a total length of approximately 3.2 km, of which more than 1.4 km was in the tailrace tunnel, as shown in Fig. 2.

The internal hydrostatic pressure in the lower drive exceeds the overburden for a total length of about 2.3 km in the area between the surge shaft and the powerhouse, including the connection adit from the transbasin tunnel to the surge shaft. In view of the high permeability of the granodiorites in this area and the lack of any reliable information on the natural water table, limiting tunnel leakage was one of the main factors affecting the lining design of the lower drive.

2. Design concepts of the transbasin tunnel

As far as the transbasin tunnel was concerned, only relatively general requirements were included in the clauses of the EPC contract. The contractor was asked to design and build this tunnel according to the following requirements:

- Flow capacity of up to 80 m³/s with the global head losses not exceeding 0.0055 × Q², where Q is the discharge.
- Limitation of tunnel leakage during normal operation and of infiltrations during dewatering, without the indication of maximum values.
- Limitation of possible rock falls and adequate design of rock traps based on five year inspection intervals.
- 50 years of operational life under acceptable conditions.

For a 26 km-long essentially unlined tunnel passing various geological formations and excavated with different methods, it is obviously not an easy task to identify the optimum design from the contractor's point of view. In particular, several factors such as the deterioration rate, sediment transport and tunnel leakage/infiltration are difficult to quantify, since they mostly depend on the geological and hydrogeological properties of the massif.

Since the excavation of the tunnel was already in progress when the authors became involved in the project, the only design parameter to be considered was the type and distribution of the tunnel linings. However, it should be mentioned that the geological properties of the massif, especially in the powerhouse area were significantly less favourable than expected, thus significantly affecting the original design concepts.

Fig. 3 shows typical sections of the upper and lower drives. A concrete lining has been provided in the powerhouse and the surge shaft area, where the insufficient overburden in comparison with the internal head, combined with the high permeability of the granodiorites would have caused excessive leakage. However, since the lining would not be able to resist the high internal pressures without a certain amount of cracking, a reduction in the permeability in the vicinity of the tunnel was considered necessary. This was achieved by grouting according to the GIN procedure. The depth, amount and location of the cement grout

![Fig. 2. Longitudinal profile of the Caecekam multipurpose project.](image)

3. Hydraulic model to simulate leakage and infiltrations

3.1 Purpose of the analysis and general assumptions

The analysis has been extended to the 1.1 km-long transbasin tunnel stretch upstream of the steel lining near the powerhouse, where the greater amount of leakage was expected for the reasons mentioned previously. The main purpose of the simulation was to provide the basis for a comparison of different lining options, to allow for an optimisation of the final lining design. The following results were thus required:

- evaluation of the leakage to be expected during normal operation, related to the concern of excessive
infiltration into the powerhouse cavern:
- evaluation of the amount and tendency of infiltra-
tions to be expected at dewatering of the tunnel, relat-
ed to the difficulties that these may imply during
inspection and/or maintenance of the tunnel.

Basically, the numerical model developed considers
that a relationship exists between the water infiltration
observed during the tunnel excavation and the infra-
filtrations/leakage to be expected during the succes-
seive operation phases of the tunnel. Therefore, it can be
assumed that, if a model is developed to reproduce the
infiltrations during construction, then the latter can be
in principle be used to investigate the hydraulic behav-
ior of the tunnel during the following operational
conditions, that is, during dewatering or filling and
during normal operation. Obviously, the final lining of
the tunnel (impervious steel liner, concrete lining,
installation of check valves and so on) has to be duly
taken into account.

3.2 Definition of the hydraulic model

The proposed hydraulic model assimilates the tunnel
stretch under study to a number of vertical shafts, each
of them representing a tunnel section (of any length)
with specific geological and/or hydrogeological prop-
erties encountered during excavation. The features of
one of these 'equivalent' shafts is shown in Fig. 4. The
parameters of the model then have to be selected in
such a way that the non-stationary infiltrations mea-
sured in each tunnel section during excavation can be
simulated by the emptying of the respective shaft. The
total infiltrations in the stretch considered in the analy-
sis are thus given by the sum of the flow rates dis-
charging from each shaft.

For the Cosegna project, the assumption of inde-
pendent shafts was justified since the nearly vertical
rock fractures in the faulted zones crossing the tunnel
axis resulted in concentrated and well defined infiltra-
tion areas. These highly permeable zones did not seem
to be connected to one another.

The water level in the shaft corresponds to the
groundwater level just above the tunnel. The flow rate
Q, representing the leakage or the infiltrations in the
tunnel, is given by a calibrated orifice at the junction
between the shaft and the tunnel, as a function of the
momentary groundwater level and the internal water
pressure in the tunnel ('head' A). The rate of decrease of
the flow rate Q is a function of the shaft section S
as well as other hydraulic parameters.

The flow rate Q, representing the flow from the
ground into the shaft or vice versa, is a function of
the difference between the original groundwater table
and the momentary water level in the shaft ('head' B).

For the present simulation, it was considered
acceptable to assume a linear relationship between
the variables. The phenomena are thus described by
the following basic equations:

\[ Q = \alpha (z - z_p) \]
\[ Q_k = \beta (z_l - z) \]
\[ \frac{dz}{dt} = \frac{Q_k - Q}{S} \]

3.3 Calibration of the hydraulic model

As mentioned previously, the model has been calibrated
to simulate the infiltrations measured during tunnel
evacuation, which can typically be defined by an initial
flow \( Q_i \), a residual flow \( Q_f \) reached after a certain time
when the draw down is achieved and an initial rate of
decrease \( R \) (expressed, for instance, by a characteristic
time \( T \) so that \( R = Q/T \)).

The 1.1 km-long tunnel stretch to be studied has been
sub-divided into a number of representative sections (in
the present case 21), each one characterized by bound-
dary conditions (elevation \( z \), overburden \( z_o \), original
groundwater level \( z_l \)) as well as by the values \( Q_i, Q_f, z \)
and \( T \) describing the water infiltrations observed during
evacuation.

The calibration of the model then consists of defin-
ing the constants \( \alpha, \beta \) and \( S \) of each shaft, so as to fit
the following conditions:

\[ Q = Q_i \quad \text{at} \ t = 0 \]
\[ \frac{dQ}{dt} = R = \frac{Q_f}{T} \quad \text{at} \ t = \infty \]

Fig. 5 illustrates the simulation of the infiltrations into
the tunnel at the time of construction after calibration of
the model. The diagram shows the total infiltrations as
a function of the excavation progress, as well as the
actual measured flow rate at the downstream end of the
considered transbasin tunnel stretch, confirming that
the model can adequately represent the hydraulic
behavior of the tunnel.
peak infiltration of 280 l/s is attained after nine days, reducing to 100 l/s after about 25 days.

4.2 Simulation of the leakage during filling

The groundwater table at the beginning of the simulation was assumed to be at the elevation resulting at the end of the previously considered case, that is, several days after dewatering of the tunnel.

The piezometric level in the tunnel was then increased from zero to the maximum operational level of 460 m with a constant filling rate of 1 m/h (corresponding to the prescribed maximum filling rate), resulting in a total duration for the filling operation of 10 days.

For the completely unlined alternative, the leakage reaches a maximum rate of 1200 l/s after 10 days from the beginning of the filling, decreasing to a value of 800 l/s after about 20 days. Also, this case, the simulation for a partially lined alternative shows a very significant reduction of the flow rates, if compared with the completely unlined tunnel. The resulting peak leakage reached after 10 days is only 160 l/s, whereas a stable value of 100 l/s is attained after about 20 days.

4.3 Final consideration on the simulations and the hydraulic model

The diagrams confirm that the applied model allows for a realistic simulation of the behaviour of the transbasin tunnel during dewatering and filling, as well as at the normal operational level, giving a significant contribution to the design of the final lining design.

If compared with a conventional approach for the computation of the infiltrations/leakage in a pressure tunnel, the calibration of the model developed for the Casseboun project has the advantage of being totally independent from the geological and hydrogeological properties of the massif, such as for instance the rock permeability and porosity. On the contrary, only parameters are used which can easily be measured on site or even evaluated.

However, it is worth noting that the proposed procedure represents a simplified approach to the phenomena. Some improvements of the model would probably offer better simulation results.

Finally, it should be mentioned that the calibration of the model based on the measurements performed at the time of the tunnel excavation could apparently represent a problem for the application of the same approach to an existing structure, for which the data going back to the construction could not be available. However, in this case, it would be possible to proceed to the model calibration on the basis of measurements done during a successive tunnel dewatering.

5. Leakage during and after first filling of the transbasin tunnel

The estimation of the leakage during first tunnel filling and during the subsequent months, when the internal pressure was maintained at the operational level, as well as the water level in the transbasin tunnel are shown in Fig. 7. It should be noted that, because of some delay with the finishing works in the transbasin tunnel, the prescribed filling rate of 1 m/hour could not be always maintained, and the completion of the filling operation took one month instead of the planned 10 days.

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During the filling of the lower drive (that is, up to el. 420) the leakage has been continuously estimated on the basis of the difference between the water introduced in the tunnel (natural infiltrations and additional water) and the measured increase of the water level, whereas during filling of the upper drive this estimate was not possible.

When the water level in the transbasin tunnel reached el. 420, a total leakage of 1200 l/s was measured, whereas a rough estimation made at the end of the filling indicated peak leakage of about 1900 l/s.

From the time when the filling was completed, the total leakage could be estimated more accurately on the basis of the lowering rate of the tunnel water level after closing of the inlet gate at the intakes. In this respect, it must be stressed that the leakage measured according to this procedure actually represents an overall balance between the leakage and the infiltrations in those sections, where the water table lies above the internal water pressure level.

The first measurement was done one day after completion of the first filling, giving a total leakage of 1250 l/s. The estimations were then repeated at regular intervals in the following two and half months, showing a decrease in the leakage down to 35 l/s.

6. Comparison between calculated and measured leakage

Although the hydraulic model for the simulation of the leakage was only applied to a 1.1 km-long stretch in the powerhouse area, while the measured water leakage refers to the whole tunnel, some final considerations on the accuracy of the theoretical computation are possible.

Before the start of tunnel filling, the total infiltrations in the stretches where the tunnel overburden is higher than the operational head (that is, in the upper drive and in the initial sections of the lower drive) were 150 l/s and 100 l/s respectively. Obviously, after tunnel filling, the infiltrations in these stretches should have decreased to some extent, so that a current total infiltration of around 180 to 150 l/s can be assumed.

It can thus be concluded that the leakage in the lower section of the transbasin tunnel (between the surge shaft and the powerhouse) are at present almost 150-200 l/s.

Considering that part of this 150-200 l/s leakage occurs in the surge shaft area not taken into account in the compilation, and that the concrete-lined sections were considered to be totally impervious, this value confirms that the final leakage of 100 l/s, resulting from the numerical simulation is in agreement with the measured rates.

On the other hand, the diagram of the measured leakage shows that a stable value is reached after some 80 days or even more, whereas the simulation for a partially lined tunnel indicated the final rate after only 20 days.

7. Final remarks

The design of the Casecan transbasin tunnel, from the hydraulic point of view, was particularly challenging, combining high internal pressures with high rock permeabilities and locally low overburden. To optimize the lining distribution and define appropriate criteria for the grouting works, a hydraulic model of the lower tunnel section was established. However, since the results of such models depend greatly on the parameters used, particular attention has been paid to recording the tunnel behavior during excavation.

In fact, it was considered more reliable to base the hydraulic model on the observed water infiltrations during the tunnel excavation than to use other hydrogeological parameters. The proposed calibration of the numerical model has been shown to be useful and relatively accurate for the prediction of the leakage under operational conditions.

This approach made it possible to optimize the lining distribution and the grouting works by quantifying for each tunnel section the leakage/infiltrations behaviour.

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