Under pressure in the Philippines

Unfavourable geological conditions on the Casecnan Multipurpose Project, in the Philippines, led to the abandonment of TBM excavation in favour of drill and blast. Roger Bremen, head of hydraulic structures, and Fabio Tognola, junior engineer, for Lombardi Engineering Ltd, Switzerland, describe the implications of this decision.

The 160MW Casecnan Multipurpose Project in the Philippines became fully operational in 2002. The US$550M project involved extensive underground works, including an essentially unlined 24km long pressure tunnel. For the tunnelling works the Italian contracting JV of Pizzarotti and CMC used both drill and blast and TBM excavation methods. This enables comparisons of the two methods in terms hydraulic efficiency of the headrace tunnel.

The Casecnan Project is an irrigation and hydroelectric power generation scheme, located almost 150km north of Manila in the northern part of Luzon Island. The ambitious 160MW build operate own transfer (BOOT) project has not only increased the power generation on the island of 400GWh/year, but equally contributes to the irrigation of 31,000ha of agricultural land.

The plant includes two dams, diverting the flows of the Taan and Casecnan rivers into a 24km long power tunnel feeding the two generating units in the underground powerhouse. A 2km long tailrace tunnel releases a total nominal flow of up to 80m³/s into the existing Pantabangan Reservoir (Figure 1).

Less favourable geological conditions than expected, combined with an already defined tunnel alignment resulted in uncommonly challenging engineering difficulties, which revealed themselves during the progress of the construction works.

The use of drill and blast excavation for longer tunnel sections, instead of the planned TBM excavation, required various project modifications to meet the construction schedules and the hydraulic requirements of the waterways. In order to meet the specified maximum head losses of the plant exactly, measurement of the effective tunnel section and surface roughness, together with an accurate hydraulic analysis, had to be undertaken in order to achieve project optimisation.

Main project features

The northward flowing Taan and Casecnan rivers are intercepted shortly upstream of their confluence, at an elevation of approximately 435m asl, and diverted southward into the existing Pantabangan Reservoir. To provide storage for the daily peak water capacity, two gravity dams impound the Taan and Casecnan storage basins. These reservoirs are interconnected with the 2.5km long Taan-Casecnan tunnel, which has a maximum flow capacity of 40m³/s.

Five underground desilting basins of approximately 35m in length and 63m² unit hydraulic section are located on the left bank of the Casecnan dam. After passing through these basins, the flows enter the transbasin tunnel (TBT), subdivided into an upper nearly horizontal drive and a lower inclined drive.

The upper and lower drives are separated at the Abuyo river crossing, underpassed by the transbasin tunnel only a few meters below the riverbed. The 7.5km long lower drive descends from the Abuyo river, with a longitudinal slope between 2% and 5%, directly to the powerhouse; reaching a maximum internal hydrostatic pressure of 270m for a nominal flow capacity of 80m³/s. The 160MW underground powerhouse releases the flows into a 2km long tailrace tunnel supplying the Pantabangan Reservoir.

From the contractual point of view, only relatively general requirements were included in the clauses of the engineering, procurement contract (EPC) for the tunnelling works. Basically the contractor was
asked to design and build the TBT according to the following requirements:

- Flow capacity up to 80m³/s with the global head losses not exceeding 0.0055 x Q², where Q is the discharge in m³/s
- Limitation of leakages from the tunnel during normal operation and of infiltrations during dewatering without indication of maximum values
- Limitation of possible rock falls and adequate design of rock traps, taking into account an inspection interval of five years
- 50 years of operational life under acceptable conditions

In particular, no specific indications were given on the need, extent and type of lining to be provided. The contract signed by the Contractors’ JV was based on a feasibility study assuming overall favourable rock conditions, despite the very limited site investigation programme. The general plant layout consisting of an essentially unlined high head headrace tunnel, with only a very short steel lined section immediately upstream of the powerhouse, indicated that the designer had no doubts about the favourable rock properties.

**Site geology and design concepts**

Both the upper and lower drives were designed as essentially unlined tunnels. A 270m long steel lining was placed to underpass the Abuyo river and a 180m long tunnel section was provided with a steel lining upstream of the powerhouse. Concrete lining was placed on an approximate total length of 3.2km, of which more than 1.4km was in the tailrace tunnel.

The internal hydrostatic pressure in the lower drive exceeds the overburden on a total length of approximately 2.5km in the section between the surge shaft and the powerhouse, as well as the connection adit from the TBT to the surge shaft. Considering the locally high permeability of the granodiorites in this area and the lack of any reliable information on the natural water table, the limitation of leakages from the tunnel was one of the factors affecting the lining design of the lower drive.

From the geological point of view, the 16km long upper drive is essentially located in generally competent to fair intrusive and volcanic rocks (mainly andesite and agglomerate formations) dipping with an angle of 30°-50° to the northeast. Local hydrothermal weathering, some fault zones, dykes and some spalling, due to stress release on the tunnel sections by the highest overburden, caused local difficulties to the tunnel excavation, carried out with two open face 6.5m diameter TBMs. A Robbins machine proceeding downstream from the Caseclan weir and a Wirth machine upstream from the Abuyo river. The average excavation rate of the TBMs was about 15m-20m/day, with some peaks up to 30m/day where favourable rock conditions were encountered. A dismantling chamber was excavated in a geologically favourable zone once both TBMs had excavated their respective sections of the upper drive. Dismantling of the TBMs as well as completion of the lining between Abuyo crossing and the dismantling chamber required more time than initially planned.

The 7.5km long lower drive from the Abuyo river to the powerhouse is mostly located in granodiorite and andesite rocks of highly variable properties. Locally extensive hydrothermal weathering and several extensive faults caused the TBM, advancing from the tailrace tunnel upward, to be definitively stopped in the powerhouse area. In order to avoid further delays due to geologic difficulties, the contractor decided to abandon the TBM excavation of the lower drive and proceed only by drill and blast. The excavation was thus carried out upstream from the powerhouse, from an intermediate access at the location of the surge shaft (Adit 2) and from the Abuyo crossing downstream (Adit 5). The overall average daily progress of the drill and blast excavation was about 6m-8m/day, with peaks over 20m/day in favourable geological conditions.

A concrete lining has been provided in the powerhouse and the surge shaft area, where the lack of overburden in comparison to the internal head combined with the high permeability of the granodiorites would have caused excessive leakages. However, since the lining would not be able to resist the high internal pressures without a certain amount of cracking, the reduction of the permeability in the vicinity of the tunnel was considered necessary. The latter was achieved by

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**Opposite page:** Fig 1 - Location and general layout of the Caseclan project: 1) Caseclan reservoir; 2) Taan reservoir; 3) Taan-Caseclan tunnel; 4) Desilting basins; 5) Transbasin tunnel - upper drive; 6) Abuyo river crossing; 7) Transbasin tunnel - lower drive; 8) Surge shaft; 9) Underground powerhouse; 10) Tailrace tunnel; 11) Tailrace open channel

Below: Fig 2 - Longitudinal cross section of the Caseclan project
intensive grouting of the rock mass in the tunnel vicinity using the Grouting Intensity Number (GIN) procedure. Depth, amount and location of the cement grouting as well as permeability requirements of the grouted rock had to be quantified in order to optimise the works, both in terms of cost and schedule.

Full or partial shotcrete linings were applied for stability purposes in those stretches where leakages were not expected due to the high overburden, or where the expected seepage rate did not represent any major concern. The lined surfaces have been equipped with drainage holes, so that the internal/external loads on the structures remain within an acceptable range. The 200mm thick concrete invert of the lower drive was cast on a drainage layer and provided with drainage holes.

The pretty large section of the drill and blast tunnel stretch, which permitted the transit of normal vehicles, allowed for rapid completion of the final lining, except in those stretches where full concrete lining and intensive grouting works were provided.

Although a direct comparison between TBM and drill and blast excavated sections in terms of construction rate, considering both the excavation and installation of the lining, can not be provided (due principally to the different rock conditions) the experience at the Upper TBT showed that in such a long and quite small tunnel the final lining should be installed preferably during the excavation itself, for example using precast elements.

**Prediction and measurement of the hydraulic head losses**

A challenging aspect of a hydroelectric project involving large tunnelling works regards the optimisation of the tunnel section in terms of head losses. In particular the calibration of the surface roughness coefficients associated with drill and blast excavation presents some uncertainties. In fact it often occurs that the hydraulic performance of the prototype in terms of head losses is better than predicted by the conventional Strickler model. In the case of the Casecna project, an overestimate of the head losses would have resulted in unnecessary additional work for the contractor, to smooth out the tunnel overbreaks, whereas an underestimate would have caused contractual penalties to be applied.

For the lower drive, the increased surface irregularity of the drill and blast profile compared to the TBM excavation obviously resulted in the need to increase the tunnel cross section, from 33m$^2$ to 42m$^2$.

The detailed hydraulic analysis involved, as a first step, the survey of the effective tunnel profiles, taking into account locally significant overbreaks, increasing the available flow section. An average waterway cross section was thus obtained and the hydraulic radius was calculated assuming the profile overbreaks as appearing after the final shotcrete layer has been applied.

Once the geometrical parameters were defined, the roughness coefficient $k$, according to the Darcy-Weisbach relation had to be calculated for various tunnel sections. For the drill and blast excavated sections (unlined), the assumed roughness coefficients varied between 100mm and 150mm, depending on the size and number of overbreaks. For the TBM excavated upper drive, a roughness coefficient of 10mm was assumed for shotcreted sections, whereas 15mm has been taken for the completely unlined profiles. In the concrete lined sections, the roughness coefficient was assumed to be 1mm.

In addition to the friction losses distributed over 24.057m of tunnel, the losses due to local changes in profile and cross section had to be considered, including in particular at flow contractions and expansions, junctions, etc.

Finally based on the previous analysis, for a nominal flow of 80m$^3$/s, the total friction losses have been estimated to be 30.6m, of which 17.8m were in the 15kn long upper drive, 9.6m in the 7kn long lower drive and 3.14m for other tunnel stretches (drainage basins at the intake, tailrace tunnel etc.). For the same discharge the losses due to changes in profile etc. are 2.9m, resulting in total estimated head losses of 33.5m.

After the plant was put into operation, the head losses were measured for various discharges, providing performance tests with only one generating unit as well as with both units in operation.
These tests confirmed a precise correspondence between the measured head losses and the hydraulic computation. A certain underestimation of the actual head losses for plant operation using only one unit occurred, being generally higher than for the two units in operation (up to 1 m difference for a 40 m$^3$/s discharge). This discrepancy is basically due to variations of the losses due to changes in profile in the powerhouse area (that is in the manifolds, turbines and draft tubes) under different operational conditions.

**Final remarks**

The head losses computation for the Casacma Trasbasin tunnel was a particularly challenging task, due to the very different tunnel conditions after excavation. While for the TBM profiles of the Upper TBT the actual shape and the surface roughness coefficient might be easily evaluated, the definition of the geometrical parameters and the hydraulic roughness of the drill and blast Lower TBT sections presented more uncertainties, requiring accurate evaluations, mainly based on field measurements.

The hydraulic model developed and the assumed parameters have shown reliable and accurate for the prediction of the head losses under operational conditions.

This hydraulic computation allowed a design optimisation of the final lining in the Lower TBT, where the contractor could renounce adopting specific measures in order to reduce the friction losses, such as smoothing out the major tunnel overbreaks after excavation, in view of the fact that the contractual requirements were fulfilled.