Theun Hinboun Expansion Project (Lao PDR): overview of the general design of the main waterways system

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Introduction

The first hydropower plant of the Theun Hinboun Power Company (THPC) is located in both Bolikhambay and Khammouane Provinces in Lao PDR with commercial operation started during the year 1998. The company is owned by Electricité du Laos (EdL) who represents the major shareholder (60%). The remaining shares are evenly distributed between the Norwegian Statkraft SF and GMS Lao Company Limited. This first run-of-river project generates 1’100 GWh/year from an installed capacity of 220 MW, 95% of which is purchased by Electricity Generating Authority of Thailand (EGAT). After a decade of operation, THPC decided to expand the project to a new total capacity of 500 MW by regulating the water volume of the Nam Gnouang River, which is one of the tributaries to the existing project. A total of 440 MW will then be sold to EGAT and the remaining 60 MW will be reserved exclusively to Electricité du Lao (EdL). The latter additional power supply will provide Lao PDR relatively important power and energy deemed necessary for industrial growth and development of the country.
In 2008, the Italian construction company CMC di Ravenna won the design-build contract of the new expansion project and assigned Lombardi Eng. Ltd. to become its designer for the main waterways system consisting principally of a headrace tunnel, a penstock and a surge tunnel. Lombardi was equally charged to follow the construction of key structures on site and provide daily support to the contractor requirements.

In this paper a general overview of this scheme is presented and some relevant design aspects are explained.

1. General layout of the main waterways system

The expansion project is mainly composed of two major schemes: around 20 km upstream of the existing weir a flow regulating RCC dam (70 m high) improves the seasonal power generation by storing wet season runoff and releasing water during the dry season, and a new water intake downstream located at the existing weir to be used for power generation at a new 220 MW capacity powerhouse equipped with a single Francis unit. The RCC dam works also included a 60 MW power station. The waterways system between the water intake and the 220 MW powerhouse, is composed of a 5.5 km long headrace tunnel with a 110 m³/s capacity and 6.90 m internal diameter, a 920 m steel penstock mainly encased with reinforced concrete and buried in trenches with a diameter varying from 5.00 m to 5.80 m, and of which 235 m are composed of an underground steel lined section. The waterways system comprises also a 950 m surge tunnel for mass oscillation about 28 m² cross section that might also be used for future inspection works. A 30 m deep TBM dismantling shaft is foreseen around 200 m downstream of the water intake, almost at the end of the underground section of the headrace tunnel, to dismantle the TBM thus allowing a full independence of the excavation works with regards to the flood season. The upstream reach of the headrace tunnel between the dismantling shaft and the water intake is composed of an underground drill and blast tunnel and an open trench section excavated in the river bed, and partially steel lined.

Finally, it must be mentioned that the choice to excavate the headrace tunnel by TBM was mainly based on environmental requirements. For this mechanically excavated tunnel, no adits will then be needed, and the construction of access roads in the middle of a dense forest is no longer necessary.

Fig. 1. Theun Hinboun existing weir: 60% overflow of the annual available water volume.
2. Contractual and technical constraints

2.1 Contractual constraints: hydraulic efficiency

The main contractual condition for the design of the above mentioned works is based on system efficiency criteria, such as the total head loss of the headrace system calculated from the head pond level to the gauge located immediately upstream of the converging cone of the turbine inlet valve. The total head loss to be guaranteed should not exceed 7.20 m with the TH3 turbine-generator unit operating at the maximum discharge of 110 m$^3$/s. This performance related contractual value of head loss represents almost 3% of the available total head.

To insure the respect of the required system performance, a systematic sensibility analysis was initially carried out in order to select a cost-optimized headrace internal section taking into account considerations such as the minimum required lining thickness, TBM characteristics, geology, segmental lining roughness values, etc. The same analysis was carried out for the penstock section. With respect to the steel cost, a sound variation of the diameter is applied in order to maintain a cost efficient design while respecting head loss requirements (Table 1).

<table>
<thead>
<tr>
<th>Scheme</th>
<th>Calculated head loss [m]</th>
<th>% of total head loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Headrace ($k_s=1.50$ mm)</td>
<td>5.42</td>
<td>76.99</td>
</tr>
<tr>
<td>Penstock ($k_s=0.05$ mm)</td>
<td>1.13</td>
<td>16.05</td>
</tr>
<tr>
<td>Singular head loss</td>
<td>0.49</td>
<td>6.96</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>7.04</strong></td>
<td><strong>100%</strong></td>
</tr>
</tbody>
</table>

Table 1: Head loss distribution (calculated) in the waterways system.

2.2. Waterways geology and configuration

The first relevant technical constraint concerns principally the headrace tunnel. This constraint is associated with the geology and topographical configuration of the tunnel. Being excavated in the vicinity of the existing headrace tunnel, the local geology of the new headrace tunnel is relatively well known based on the mapping carried out during the drill and blast excavation of the old tunnel. After a thorough initial analysis of the existing data, it appeared that the geological layer in the area is formed by a sequence of mudstone, siltstone and sandstone pertaining to the Nam Xot Formation K1nx. The layer is normal to the tunnel axis and dips at a lower angle. Most of the rock mass is classified as fair to good quality. Some limited sections of the headrace are expected to cross faults where weak and extremely weathered rock might be found. Keeping in mind the prevalence of this soft rock normally subjected to swelling during excavation and eventual wedge breaking, two main initial considerations where decided: the TBM should be equipped with a single shield to allow a safe erection of segmental lining and the cutterhead should be equipped with overcutters allowing the machine to excavate 100 mm beyond the nominal tunnel diameter.

The upstream section of the headrace tunnel ends with a portal situated directly at Nahm Theun River. Although a cofferdam was foreseen, the risk of flooding the entire headrace tunnel, in case the TBM excavation is completed up
to the portal, appeared to be excessive knowing that flood seasons in Laos could be extreme. Moreover, the rock overburden of the last 100 m section of the tunnel appeared to be quite low to resist the internal water pressure. Thus it was decided to construct a shaft some 100 m before reaching the upstream portal to dismantle and remove the TBM before reaching the end of the tunnel. The last section is realized by a controlled drill and blast excavation method.

The initial geological investigation made for the penstock indicated the presence of clay foundation along almost 50% of its open-trench section. This major constraint determined the design of the penstock encasement taking into account that the full consolidation of the foundation will only be reached years after the completion of the construction works.

3. Single shield TBM for the headrace tunnel

3.1 General concept

The alignment of the new headrace tunnel ran almost parallel to the existing one. It is almost 5.5 km long with a constant slope of 1.14%. It was excavated ascending from the downstream TBM assembling yard. The first 75 m section of the headrace tunnel used as a TBM launching tube was excavated by drill and blast method. This launching length was calculated as a function of the TBM requirements and the needed place for its back-up system. Additionally, a space was reserved in this tube for the storage of the continuous conveyor belt inside the tunnel used for mucking. Only the precast invert segment was placed in this section after building a concrete slab. This allowed the access of the TBM and the future train traffic during the excavation works.

The main section of the headrace tunnel as illustrated in Figure 5 is around 5.2 km long. It is excavated using a single shield TBM with precast segmental lining. The nominal excavation diameter is 7.65 m (with new cutters), while the internal hydraulic diameter is 6.90 m. Beyond the TBM dismantling shaft at the upstream portal of the headrace tunnel, two different sections were considered. The underground section, excavated by drill and blast, is lined with reinforced concrete, which is designed to absorb almost the entire internal water pressure due to insufficient rock overburden. The other section, connecting the headrace with the water intake, is built in the riverbed in open trench excavation. It is mainly constructed with reinforced concrete while the section with bends are steel lined.
3.2 Segmental lining

The universal segment lining system of the headrace tunnel consists of 6 precast concrete segments 28 cm thick: 1 invert (trapezoid), two identical segments on the right (parallelogram), two identical segments on the left (parallelogram) and one keystone at the crown (trapezoid). All rings are identical, each 1.6 m long. No curves were envisaged and the misalignments are compensated on site by wedging the ring prior to tightening the bolts.

The invert is provided with a platform (approx. 3 m wide) and a central ditch with a maximum capacity of 150 l/s in order to evacuate groundwater infiltrations and TBM exhaust water. Two railways were foreseen at both sides of the ditch for backfilling and segment transportation.

The clearance at the crown and the invert is 140 mm and 50 mm, respectively. This results with a lining axis 45 mm below the excavation axis. This configuration allows an easy pea-gravel filling of the annular void from the upper segments while reducing the weight of the invert segment that would then be equipped with smaller socles. To allow the backfilling and grouting of the tunnel lining, each segment is equipped with a perimetral compressive gasket (width 33 mm) installed at the intrados. The positioning of the gasket grant a better penetrability of the grout along the entire joint between each segment, thus improving the transmission of forces between rings and segments. During segment erection, steel bolts were used to connect the segments within the same ring. This feature enhanced the stability of the rings and prevented non-desired displacements between two consecutive segments during erection while not yet fully backfilled and grouted. Connectors were used between subsequent rings in order to connect newly installed segments with the previously erected ring. Segments were lifted using the hydraulic erector.
of the TBM, gripping them through the lifting insert situated at the centre of segments. Guiding rods were inserted in the longitudinal joint between the segments. The gap between the extrados of the ring and the excavation profile were filled later by pea gravel and concrete-based grout mix to form an intimate contact between the tunnel lining and the ground, hence limiting settlements and ensuring an even load distribution on the tunnel ring.

4. Surge tunnel for mass oscillation

4.1 Multipurpose tunnel

The surge tunnel is a hydraulic structure designed principally for mass oscillation similar to the functioning of a pressure shaft during water hammer. In order to optimize construction works the same structure is designed as well as an access tunnel for future inspection and repair works. Therefore, the slope, width, and clearance of the tunnel respect the minimum required standards. The main geometrical characteristics of the tunnel are thus chosen based on a sound compliance between both, hydraulic and accessibility requirements. The surge tunnel is 950 m long, with a constant slope of 13%. The section is D-shape with a width of 5.8 m and maximum height of 5.3 m. It is excavated manually (drill and blast) and lined with fiber-reinforced shotcrete. The design of the support measures were mainly based on the Q-Barton classification. The lower section of the surge tunnel intersects perpendicularly with the headrace tunnel where a rock trap is foreseen in order to prevent eventual detached rock or concrete fragments from reaching the turbine.

4.2 Hydraulic design

The hydraulic analysis of the surge tunnel helped with the definition of the typical section, slopes and the maximum portal elevation. This surge facility is designed for continuous and rapid start-up and shutdown conditions. The main transient design conditions such as required by the owner are summarized in the following.

Considering a maximum operating upstream water level at weir of el. 403.643 m asl and maximum flow of 110 m$^3$/s:
- Emergency shutdown in 10 seconds; and
- Start-up of turbine in 10 seconds followed by emergency shutdown in 10 seconds.

Moreover, for a minimum water level at weir of el. 393.643 m asl the following conditions should be adopted:
- Start-up of turbine in 10 seconds; and
- Load rejection in 10 seconds and restart at maximum backflow from speed no-load.

Additional conditions related to safety factors for a transient design are also given. They should be respected for all afore-mentioned conditions:
- The minimum freeboard of the Surge Tunnel shall not be less than 2.5 m;
- The minimum water level in the Surge Tunnel shall not be below the crown of the Headrace Tunnel;
- The maximum overpressure on the TH3 casing shall not exceed 30% of the gross head; and
- The TH3 hydraulic circuit shall be designed to ensure that a 1% step decrease in power will provide a damping factor for surge oscillations of less that 33%.

Another practical condition required by the contractor is that the surge upstream portal should be situated close to the existing portal where an access road is already present. The maximum elevation at this portal is around 436 m asl.

The final results of mass oscillation analyses are illustrated in Figure 6. They show that the maximum water surge is reached during a start-up followed by a shutdown. However, this predefined maximum surge level as required by the contractor was only achieved after equipping the surge tunnel with a diaphragm (orifice) of 10 m$^2$ section (almost three times smaller than the typical tunnel section) where energy, during mass oscillation is locally dissipated. Finally it must be mentioned that the Thoma criteria regarding the available surge critical area, in order to prevent large oscillations during a small variation of the turbine output, was also checked.
5. Design of the penstock

5.1 The particular geology of the penstock

The penstock has two main typical sections: the underground steel lined section along the first 235 m of the headrace tunnel and the cut-and-cover 685 m section where the penstock is encased with reinforced concrete and covered later by compacted muck material of the headrace tunnel. The diameter of the penstock varies from 5.80 m at the upper section down to 5.0 m close to the powerhouse.

In order to characterise the foundation geology, core drilling and geophysical refraction investigations were carried out during the early stage of the project. They indicated the existence of a alluvial cover layer with a thickness varying between 3 to 20 m depending on the location of the section. This top layer is resting on a slightly to highly weathered rocky substratum composed of mudstone, siltstone, and limestone sequences. Several weak vertical zones have been found during the seismic refraction surveys. An average wave propagation velocity between approx. 2'700 m/s and 3'000 m/s characterized these zones. The middle section of the penstock is characterised by clay deposits. Initial structural investigation of the penstock encasement demonstrated that this section is the most critical and might sustain eventual consolidation settlement. The concept of the reinforced concrete encasement was thus majorly determined by this aspect.
5.2 Structural concept of the penstock

Two main considerations were adopted during the initial conceptual design of the penstock: 1) Thrust/anchor blocks at each curve are passive and no prestressed anchors are to be installed. This avoid future inspections works since all the anchors will be totalled backfilled; 2) The penstock encasement should be capable to sustain small settlement without cracking and should be designed to withhold integrally the earth pressure of the backfill. In order to achieve these requirements, it was decided to pose the penstock cans on a heavily reinforced concrete slab 25 cm thick where no contraction joints are foreseen. This slab will provide an almost uniform load distribution on the foundation and would be capable to slightly deform without breaking. The encasement of the penstock cans are then made by reinforced concrete with 12 m long structurally separated sections. Such feature provide to the encasement the required elasticity where the joints each 12 m are capable of compensating small foundation settlements. The maximum acceptable settlement considered for the design of the encasement is around 20 cm. This value is supposed to be significantly higher than the effective consolidation settlement expected at the alluvium section of the foundation.

6. Overview on the on-going works

The construction works are presently progressing with a great pace and the main schemes are almost finished. The official commissioning is foreseen during the first half of 2012 as scheduled initially. Some salient data with regards to the construction works of the waterways system are summarized in the following:
- The excavation of the headrace tunnel was accomplished in almost 9 months with an average TBM excavation rate of almost 21 m/day (including segment erection).
- Only two major faults were encountered during the excavation of the headrace tunnel as it was foreseen initially. These fault were crossed easily by the TBM and were later treated by consolidation grouting.
- The drill and blast average excavation rate of the surge tunnel was around 3.6 m/day. Beside interconnected cavities found along a small section of the tunnel, no major faults were encountered.
- The erection and encasement of all the penstock cans are accomplished.

7. Conclusions

In the present paper a general overview of the main waterways system of the Theun Hinboun Expansion project is presented. The project aims to increase the capacity of the existing scheme by 280 MW. The main waterways system is composed of three major structures: a TBM excavated headrace tunnel, a surge tunnel for mass oscillation and a cut-and-cover concrete encased penstock. Presently, the construction works are to be accomplished soon and the commissioning of the project is scheduled during the first half of 2012. Thailand will mostly benefit from the produced energy confirming the position of Lao PDR as the main hydropower energy producer in the Southeast Asia.
Fig. 8. TBM breakthrough after 9 months of excavation.

References

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