About the damage of the penstock of Xekaman 3 hydropower plant due to sliding and the remedial measures for power generation

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Abstract

The Xekaman 3 hydropower plant is located on the territory of Lao PDR and has a 6008m long headrace tunnel, a 1110m long penstock a, 520m gross head and installed capacity 250 MW. The headrace tunnel and the penstock area consist of sandstone, siltstone and microdiorite. The penstock lies on the site of an ancient stabilized landslide, but due to the construction of the cut slope upstream of the power house, the ancient landslide reactivated, creating a rupture of the penstock and an inclination of vertical shaft N2, so the electrical generation has been stoped. At present, the measures for stabilizing the landslide are being carried out such as removing the weight of soil from the head of landslide, dewatering the cut slope and constructing the concrete shearpile wall, but it is difficult to stop completely the displacement of landslide. In order to early generate electricity, the shorterm alternative of remedial measures are recommended with phylosophy "living together with the landslide. They are measure placing the compensator at horizontal penstock and measure placing the telescopic at vertical shaft N2 which permit the penstock operating safely while the landslide is moving with rate of 5mm/ month. Monitoring the landslide displacement is caried out strictly. At present the power house is generating with one operational unit.

1. Introduction

The Xekaman 3 project is located on the territory of Lao PDR having a 101.5m high concrete face rockfill dam. The energy line consists of the 6008m headrace tunnel with an internal diameter of 4m and a 43.3m high surge tank with an internal diameter of 12m. The tunnel from the surge tank to the power house is called the penstock with a concrete lining and a steel lining. After the surge tank is the 182m long horizontal penstock, then a 171.83m high vertical shaft , connected to another 710.9m long horizontal penstock with an internal diameter of 3.2m. After that there is an inclined 65.75m long open penstock connected to a vertical shaft 91.86m high with an internal diameter of 3,1m. Then the horizontal penstock in 205,5m with internal diameter of 3.1m connected to the power house with an installed capacity of 250 MW, gross head of 520m and having average annual power production of 982,000,000 KWh.

At the cut slope upstream of the power house, the landslide has happened during the rainy season, destroying the penstock so the electrical generation has been stopped. First the short term alternative of remedial measure was recommended to early generate power, because the owner needs the money from electricity sales to pay the bank interest and to repair the structure, while continuing to research the design of a long term alternative to ensure the safe operation of power plant.

Fig.1: The Xekaman 3 hydro power plant

Fig.2: The interval of open penstock (detail)

2. The geological characters of the area of the penstock and the power house

2.1. At the West and the South of the power house and the Xekaman river channel is the sedimentary rock of the Long Dai formation ($0_3 – S_1$ ld) consisting of sandstone, quartzite bedded, tuf sandstone, tuf siltstone, and is strongly altered due to the penetration of several dykes of quartz and mirodiorite. From the surge tank area to the beginning interval of penstock is composed of the sedimentary rock of Song Bung information (T_{1-2} sb_2), gravellite at the bottom and intercalated sandstone and siltstone at the upper part, the rocks are bedded and rather hard. At the end part of the penstock to the power house area there are greenish grey, slightly grey, small grained and massive microdiorite. In addition, there are some dykes of blackish grey, hard of gabbro-diabase.

The collovial- deluvial deposits is widely distributed in this area and limited within the border of the ancient landslide. These deposits consist of soil with fragments and blocks of sandstone, siltstone and magmatic rocks in different sizes from tens centimeters to tens meters.

In this area meet some faults of IV grade in strike of NW – SE and dip of 70-80^0.

On the basis data of boreholes and engineering geological map, this area can be divided into 3 blocks:

- Block 1 at the surge tank area is steep slope of the mountain, the soil layer of the block of complete weathered rock (CW) and highly weathered rock (HW) are rather thin, the bedrocks are rather hard sandstone and siltstone.
- Block 2 mainly is the body of the ancient landslide, consisting of sandstone, siltstone and microdiorite. The surface of this block is covered by the colluvial- deluvial deposits, the thickness of these deposits plus the zone CW and zone HW are large, somewhere reaching to 200m such as at borehole BS6. The penstock is mainly located on this block.
- Block 3 at the powerhouse area, outside of the ancient landslide, consisting of sandstone, siltstone and microdiorite, with the thickness of zone CW and HW averaging 40m.

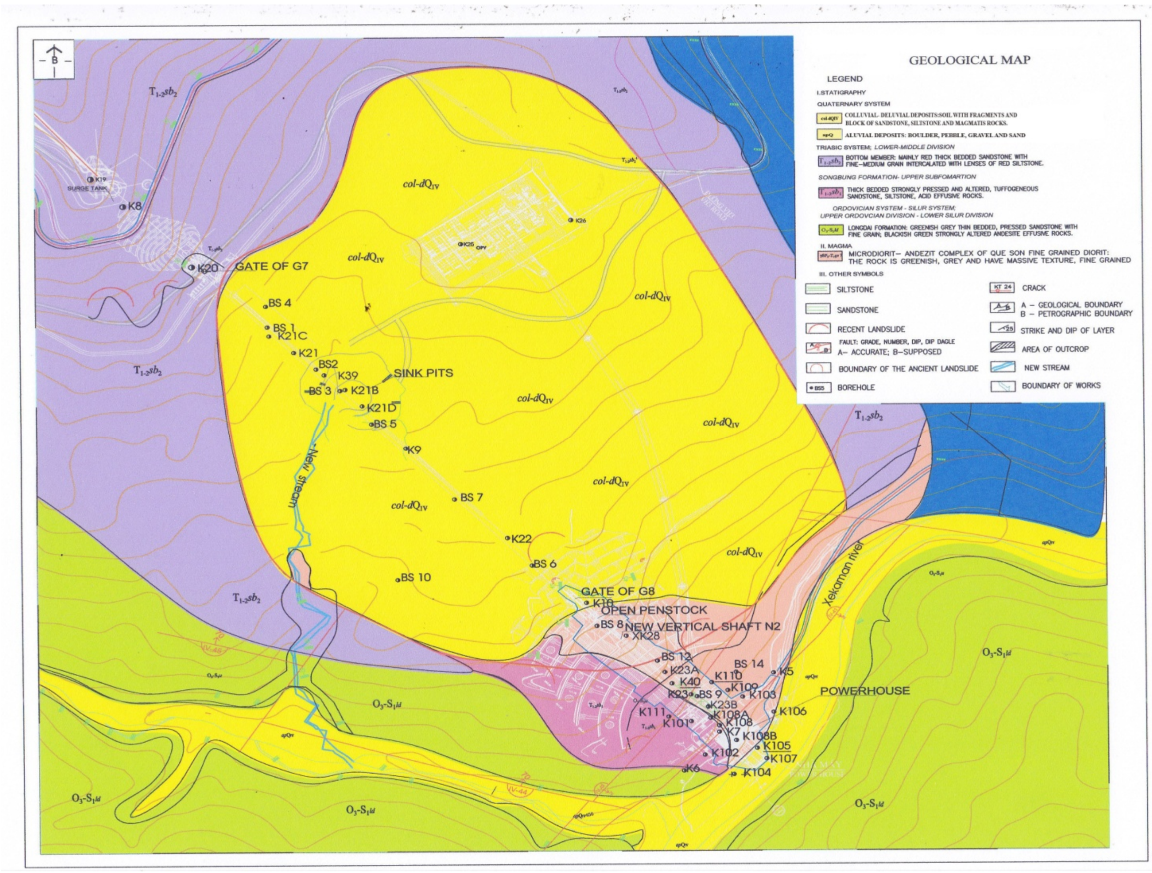


Fig.3 Geological map of the Xekaman 3 hydropower plant

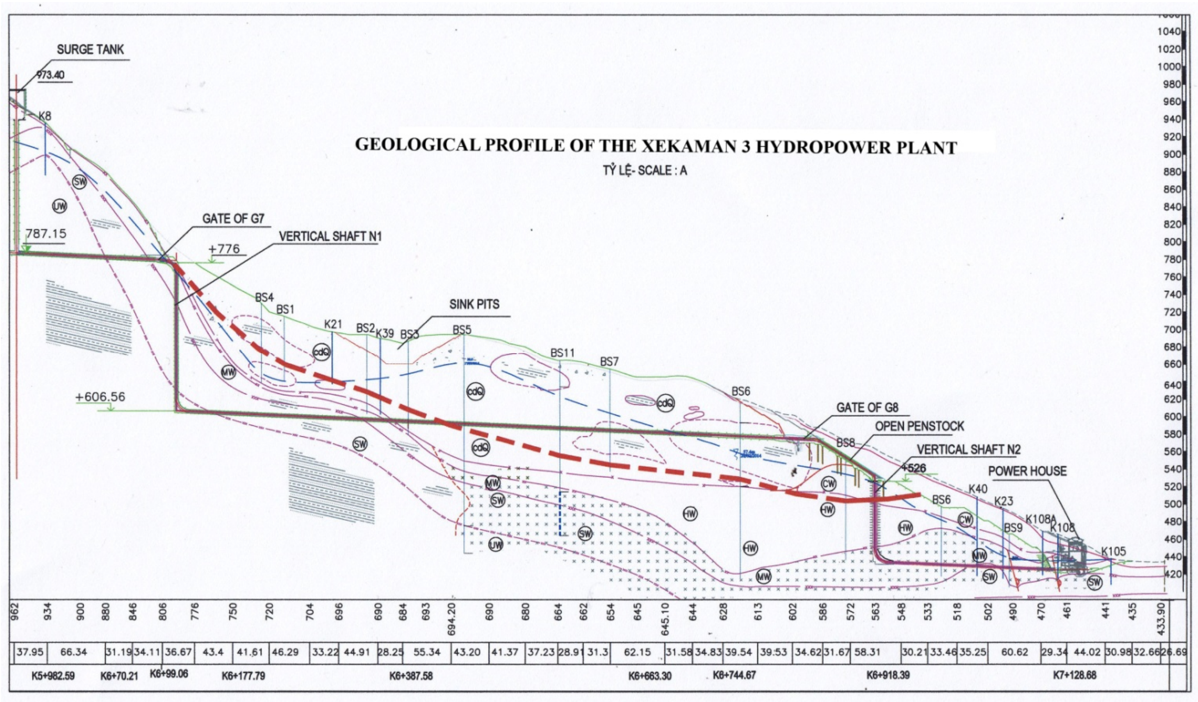


Fig. 4 Geological profile of the Xekaman 3 hydropower project

2.2 During the process of excavation of tunnel and shafts, the geological describe and calculating the value of RMR and Q are carried out, the quality of rockmass are classified as very good (class I), good (classII), fair (class III), poor (Class IV) and very poor (classV).

Table 1. The following table represents the quality of rock mass at the penstock

Tunnel face	*Chainage*	The length of Tunnel section, (m)	*Zone of weathered rock*	*RMR value/ and Q value*	*Rock class*	*Remark*
G8	6+118 ÷ 6+320	136	Fr - SW		II	
	6+320 ÷ 6+369	49	MW		IV	
	6+369 ÷ 6+384	15	HW		V	
	6+384 ÷ 6+440	56	Col -dQ			
	6+440 ÷ 6+702	262	HW		V	
	6+702 ÷ 6+815,7	113,7	MW		IV	
G9	6+828 ÷ 7+118,9	290,9	MW		IV	From bottom of shaft N2 to power house

Table 2. The table of quality of rockmass at the vertical shafts N1 and N2 as follows

Zone of weathered rock	**Rock class**	**Vertical shaft N1**	**Vertical shaft N2**
		Elevation, from- to (m)	**Elevation, from - to (m)**
HW	V	-	562 - 474
MW	IV	777 – 760,5	474 - 442
SW	III	760,5 - 720	-
Fr	II	720 - 597	-

3. The physico- mechanical properties of the soil and rock and results of the calculation and design of the cut slope upstream of the power house

Table 3. On the basis of laboratory test and in situ tests, the design value of the physico-mechanical properties of the soil and rock mass as follows

Physico –mechnical properties of rockmass	Zone CW	Zone HW	Zone MW	Zone SW	Zone Fr	Fault in V grade
Natural density ,t/m^3	1,8	2,1	2,45			
Saturated density,t/m^3	1,9	2,2	2,47	2,7	2,75	
gree): natural saturated	22 -	30 -	- 35	- 37	- 41	- 31
C (MPa) : natural saturated	0,03	0,05	- 0,11	- 0,3	- 0,4	- 0,1
Modulus of deformation, MPa	20	70	1000	3000	4000	
Poisson ratio	0,35	0,32	0,30	0,27	0,25	
Allowable bearing capacity, MPa	-	1,5	5	15	20	

Using the value of physico- mechanical properties in table 5, the design consultant has calculated the sliding stability of the slope from elevation above 600m to the berm of 450.8m of 3 cross sections around the penstock. The program Geostudio of Canada was used to calculate the stability of slope and gave following results:

Table 4. Result calculate the stability of slope by Geostudio software

Cross section	**Case**	**Factors of safety of different calculated method**				**Allowable factor of safety**
		Ordinary	**Bishop**	**Jambu**	**Mor. price**	
1- 1	1	1,758	1,785	1,731	1,782	1,25
	2	1,658	1,711	1,669	1,708	1,25
2 – 2	1	1,608	1,649	1,588	1,646	1,25
	2	1,397	1,442	1,385	1,440	1,25

3 - 3	1	1,431	1,516	1,471	1,513	1,25
	2	1,341	1,364	1,328	1,362	1,25

Remark: Case 1 is soil and rock in natural state, Case 2 is in saturated state.

Table 5. The angle of the cross-sections of the entire cut slope from elevation over 600m to 450,8m as follows

N Cross section	**Elevation, m From - to**	**The height of cut slope**	**The angle of entire cut slope**
1 - 1	605 – 476	129	1/ 2.55 or $21^0 40$
2 - 2	623 – 450,8	172,2	1/ 2.43 or $22^0 34$
3 - 3	619 – 450,8	168,2	1/ 1.28 or $23^0 68$

Although the cut slope is rather high (maximum 172.2m) but it is rather gentle. While each slope between 2 berms is 1/1,5 ($33^0.40$), there are several wide berms so the angles of the entire slope (from elevation of over 600m down to 450.8m) change from 21^0 to 24^0, similar to the natural slope of the hill. The results of the stable calculation of 3 cross sections show that from the elevation over 600m down to 450,8m have the factor of safety of cut slope changing from 1,328 to 1,669, higher than allowable safety coefficient of 1,25. While the surface of the natural slope mainly consists of soil of colluvial-deluvial, eluvial deposits and zone CW, the surface of cut slope at upstream of power plant is mainly composed of strongly weathered rock (HW) having higher shear strength (see table 4).

The calculation of the cut slope shows that it will be stable for a long time. Due to subjective thinking, the owner did not construct the rainy water drainage system and the protection layer of slope on time , so they suffered severe sequence.

4. The lanslide of the cut slope at the penstock

After the penstock was damaged due to the landslide, an additional investigation was carried out to estimate the size and features of the landslide and to recommend the remedial measures. The data of additional engineering geology investigatios showed that the penstock lay on the ancient landslide area.

4.1 The ancient landslide area

The ancient landslide has rather large dimensions being 750m long and 720m wide. From face G7 up, the slope of mountain is rather steep, with bed rock exposed on ground surface or lying under the thin soil cover. But at the lower area, from borehole BS4 through face G8 to borehole BS12, the ground surface is rather gentle and is covered by a thick layer of colluvial- deluvial deposits, they are landslide body composed of soil and rock block, collapsed from the upper mountain slope.

4.2 The recent landslides

At present, the ancient landslide has reactivated and destroyed the penstock. After filling with water to prepare for electrical generation, at the chainage of km 6+384.56m where the penstock lies within the col-dQ deposits has been ruptured, the concrete lining and the steel lining has seperated about 7cm. The water from the penstock has penetrated through the top layer of 70m thick of soil and rock blocks due to hydraulic fracturing and sprayed to ground surface with discharge of about 30 m^3/s, forming a large sinkhole and a new stream.

Fig. 5. The first damage of penstock

Fig.6. The second damage of penstock

Fig.7. The sink pit

Fig.8. The new stream has been formed

At that time, the cause of these defects was attributed to the poor quality of the welding and concrete lining, so the remedial measure are rewelded and add additional steel for the penstock and add cement grouting around the penstock and filled the sinkhole. The power house has operated again, but during the rainy season a defect occurred at the chainage of km 6+ 396m within the colluvial- deluval deposits, far from previous breaking location, at welding point of a length of one steel tube. The concrete lining and the steel lining has been ruptured and has separated a distance of 4cm, causing a large discharge of water on the ground surface. At the separation location, it has been observed that the penstock has moved very slowly downstream, the distance growing from 4cm to 6cm. According the results of calculations, the col-dQ deposits (from the chainage km 6+384 to km 4+440m) has a low modulus of deformation (21 MPa), so during the operation process of penstock under high pressure is subjected to a longitudinal stress $\sigma_Z = 350.8$ MPa, while the allowable stress of a steel tube is only 429 MPa. Surrounding the outer interval of the penstock is sandstone and siltstone of zone MW having a modulus of deformation of 1000MPa, so the longitudinal stress of penstock is much smaller. Therefore when the landslide happened, the penstock was pulled, increasing the longitudinal stress so that the penstock was ruptured within the area of the col-dQ deposits and at the weakest location such (the welding point).

Observing the surface of the slope found several cracks, at the vincinity of where borehole BS6 meets a 50m long crack and the ground surface slides down 70cm along this crack. At the foot of the landslide there occurred several shear cracks, their strikes are perpendicular to the sliding direction and to the penstock. According to the monitoring data, from July 2012 to January 2014 the control points on the open penstock have moved 15-17cm towards the power house. The location of the landslide surface is only estimated, may be the plastic clay layer at the borehole BS6 is 95-105m deep, at BS5 95-145m deep, and at LKS1 33-75m deep. But it is determined at shaft N2, from its depth of 28m to upper the reinforced concrete shaft was inclined but almost vertical to lower. The second location is at the foot of the landlide at borehole BS12 where the hard rock lies near the ground surface and has no cracks.

Apart from that, some active small landslides meet on the right and on the left of the power house. This cut slope lies on the ancient landslide body which has being stable for thousands years and was covered by dense forest. But now the ancient landslide reactivate due to following causes:

- The surface of the cut slope mainly are soil and weak rock of zone HW and colluvium- deluvium deposits, however it was not protected in time from erosion. The forest has disappeared, but during 3 rainy seasons the concrete plate system was not constructed yet on the surface of cut slope. When the excavation of the slope was completed, several locations are zone HW, but now became zone CW.
- The trench drains were not completed, so a lot of rain water on the hill flows directly onto the cut slope during rainy season. Previously the slope surface was almost dry, but due to the damage of penstock, the water sprayed with large discharge to the ground surface increasing the level of underground water in the cut slope. Only on the right of the penstock at berm of 526m, the water spring appeared both during rainy and dry season. According to the data of the boreholes, the level of ground water in the slope lies at the depth of 28-60m.

Table 6. The ground water level in the boreholes are represented in following table

Number of observation borehole	Elevation of ground water level in boreholes, m			
	Measurement during dry season		Measurement during rainyseason	
	April 11-2014	July 25-2014	September 4-2014	October 11-2014
BS4	652,8	653,1	658,4	658,5
BS7	593,5	594,2	595,3	597,1
BS9	436,5	437,3	439,6	439,7

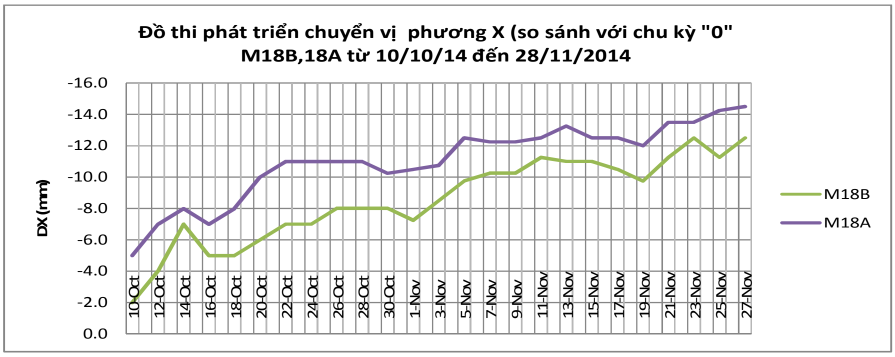
As above, it is found that the fluctuation of ground water level during rainy season and dry season is only from 3m to 6m.

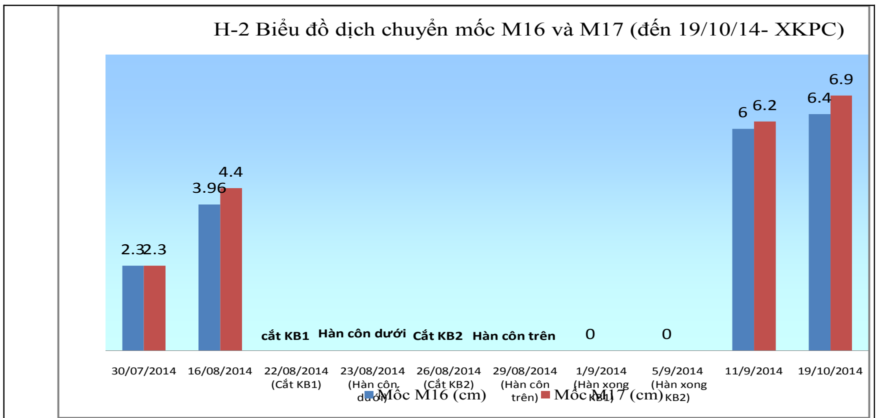
The power house is located outside the ancient landslide. Its foundation consists of microdiorite, sandstone and siltstone of zone MW, having the shear strength in a saturated state of $\varphi = 35^0$, C= 0,11 MPa, bearing capacity of 5 MPa, and moreover its foundation is embeded into the bedrock 23-28,8m, so it is almost stable against sliding.

6. Monitoring the displacement of the landslide

At the cut slope, from face G8 to the power house there are 27 control points, the monitoring results as follows:

– Three points M16, M17 and M18 on the open penstock, from December 2013 to March 3-2014 (the end of rainy season to beginning of dry season), they had displacements of 22mm, 23mm and 27mm, averaging 10mm/month. During the dry season from March 3 to July 30-2014 they had a smaller displacement, being 3-5mm, averaging 1-2mm/month. During the rainy season from July 30 to September 11-2014, they had a displacement of 35mm to 39mm, averaging 19mm/month. From September 11 to October 11 (the end of rainy season), they had an average displacement 10mm.
– The landslide displacements gradually increase from the sink hole to the vertical shaft N2 and then quickly decrease towards the powerhouse. This proves that behind the vertical shaft N2 is a passive block, against the movement of the landslide.- The inclinometer shows that the direction of landslide is towards the powerhouse.





Biểu đồ chuyển dịch ngang tổng thể M16, M17 đến 19/10/14 (XKPC)

Fig.9 The graphic of horizontal displacement of the control points M18, M18a, M16,M17

7. The shorterm alternative of the remedial measures to the early recovery of the operation of the power house

This alternative consists of the measures for stabilizing the landslide and the engineering measures for "living together with the landslide".

7.1 The measures for stabilizing landslide

These measures include:

- The weight of soil was removed from the head of the landslide at face 8 to reduce the driving forcet, with the excavation volume being about tens thousands cubic meter.
- It is preparing to construct the concrete shear pile walls on the berms to prevent the displacement of landslide.
- Dewatering the cut slope will be applied. Several horizontal drains will be required on the right of penstock, due to the spring on the surface at berm of 526m.

The construction of these remedial measures do not obstruct the electrical generation of the power house. The monitoring the displacement of landslide is being carried out strictly to appraise the effective of the preventative treatment. These treatments diminish the rate of the landslide's displacement, but it is difficult to stop it.

7.2 The engineering measures

- In the interval from the zone of poor quality rock (zone HW and collovial- deluvial deposits) to the face G8, the steel tube has been placed into the 3,1m diameter horizontal penstock. This 2m diameter and 30mm thick steel tube has 2 compensators, so it can operates safely when the landslide is moving.
- The 3,1m diameter, concrete lining and steel lining vertical shaft, from the depth of 28m up to ground surface, has been inclined 15cm causing large shear stress nearly to the limited state. For the shear stress become 0, the telescopic (a 2m diameter and 30mm thick steel tube) has been placed into the vertical shaft at the middle section, where the shear stress is largest, so that the penstock can work safely while the the rate of sliding placement being 5mm/month.

Fig.10. The compensator at the horizontal penstock

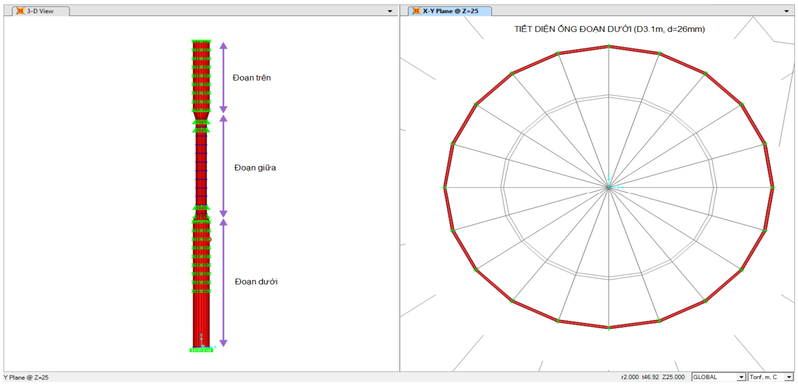


Fig. 11. The telescopic (a 2m diameter, 30mm thick and 26m long steel tube) has been placed in the middle section of penstock

8. Conclusion

The penstock of Xekaman 3 is located on an ancient landslide which was stable, but after the excavation of the cut slope upstream of the power house has been completed, the ancient landslide reactivated, destroying the penstock, so the electricity generation has to stopped, causing economic loss for the owner. At present, some alternatives of remedial measures are designing to ensure the stable operation of the power house for a long time, such as the penstock would lie under the landslide surface or the penstock would be almost open...but they would require a lot of time and of suspension of electrical generation and their costs are expensive. In order to facilitate rapid generation of electricity, a shorterm alternative of remedial measure has been recommended, consisting of the measures for stabilizing the landslide and the engineering measures.These measures diminish the rate of displacement but it is difficult to stop completely the displacement of the landslide.

At present the remedial work of the penstock of the shorterm alternative has been completed during 45 days and the repair cost was 17 billions VND (or 0,84 million USD). The power house is operating safely, with an average monthly generation of 92 millions KWh, netting about 92 billions VND (or 4.3 million USD).

It is necessary to prompt an out dated lesson, but often being violated in Viet Nam, this is the protective measures of the cut slope were not constructed on time, causing serious consequences, particular during the rainy season.

Acknowledgements

The authors used the geological investigation, design, completion and monitoring materials of Viet Lao power JSC, Song Da consultant JSC for this paper, we would like to convey our thanks to them.

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