Remedial Measures for the Restoration of an Extensive Rock Slope Failure at the Right Abutment of the Thisavros Dam, Nestos River, Greece

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Abstract

An extensive failure occurred on February 10th, 2008, at the right abutment of the Thisavros Dam, over the stilling basin, causing serious safety issues by blocking the operation of the spillways. The Thisavros Dam is one of the biggest earth dams of Europe with a height reaching up to 175m. The complexity of the geological – geotechnical conditions along the slope, the expansion of the failure as well as the economical importance of the hydroelectric power plan, set the study of the stability conditions interesting.

The proposed remedial measures were based mainly on geometry alterations of the slope design and on the application of light support measures. This study proved that by taking into consideration the geological settings and the condition of the rock mass the application of heavy and expensive support measures could be prevented. Also, indicated the importance of the geological setting at the initial design of extensive excavations.

1 Introduction

The Thisavros Dam is one of the biggest earth dams in Europe with a height of about 175m and a mass volume of 11,000,000m$^3$. The capacity of its reservoir is 700,000,000m$^3$ and it has been constructed by the Public Power Corporation (PPC) of Greece, for the annual production of 1,055 GWH. The Thisavros Hydroelectric Power Plant is located along Nestos River, at the northeast of the city of Drama, East Masedonia, Greece (Fig. 1). The construction was completed in 1997.

The numerous geotechnical slope stability problems encountered during the construction of the dam proved that the geological conditions of the construction site were unfavourable (Liakouris, 1995). Two of the main landslides that took place during the construction and led to the partial redesign of the power plan were an 8,000,000m$^3$ failure at the right abutment, along the axis of the dam, as well as a 700,000 m$^3$ failure at the left abutment, downstream to the dam.

On February 10$^{th}$, 2008, more than ten years after the completion of the construction, an extensive failure occurred at the right abutment of the Thisavros Dam, over the stilling basin. The failure blocked the operation of the spillways causing serious safety problems. The right abutment slope has a total length of 800m extending from the axis of the dam with a NE-SW direction and a height reaching up to 170m. The 330,000m$^3$ failure took place at the northern section of the abutment, in a distance of almost 300m away from the base of the dam (Fig. 2). It should be noted here that along the northern section of the slope its direction gradually changes from NE-SW to NNW-SSE and the failure occurred right over the curvature.
For the restoration of the slope a full set of retaining measures as well as geometry alterations have been specified, within the limits of a study carried out for educational purposes. All measures were designed taking into consideration the stability problems of the nearby parts of the slope. Stability analysis were carried out, aiming to examine not only the possibility of wedge failures along the surface of the slope, but also the stability conditions of the weathering mantle, acting more like as a soil material.

The complexity of the geological – geotechnical conditions along the slope, the size of the failure as well as the economical importance of the hydroelectric power plant, constitute an interesting case study.

2 Geological and geotechnical setting

The entire slope of the right abutment is composed of gneisses and granodiorites with intercalations of mica and amphibolic shales. The transition between gneisses and granodiorites is gradual without distinct limits. The mica
and the amphibolic shales occupy the extensive and numerous shear zones intersecting the slope, reducing its mechanical behaviour. The aforementioned formations belong to the lithostratigraphic series of the Sidironeriou unit of the Rhodope Massif (Fig. 3).

Figure 3. Geological section presenting the lithostromatographic condition in Thesaurus dam wider area (IGME, 1973).

The continuity of the rock mass is interrupted by schistosity, joint sets, faults, extensive shear zones and a dense aplitic veins network. Furthermore, the quality of the rock mass is downgraded by the hydrothermal weathering of the intensively fractured formations, forming a thick weathering mantle at the crest of the slope.
The schistosity of the gneisses, in contradiction to that of the shale intercalations, appears to be weak, with a NE dip direction and with an average dip angle of 53° (Fig. 4). The schistosity planes do not contribute to the instability of the slope, as their orientation develops counter to the slope planes.

The aplitic veins consist mainly of quartz minerals, presenting sufficient mechanical characteristics, similar to those of the gneisses.

The faults are developed in three main directions representing the main tectonic phases of the Sidironeriou unit. Those directions are: a) a normal NW-SE faulting, parallel to the schistosity planes, b) a normal NE-SW faulting (Liakouris, 1995) and c) a reverse faulting with a NW dip direction, corresponding to the thrust movement of the Sidironeriou over the Pangeo unit (Mountrakis et al., 1983).

The main joint sets appear to be parallel to the directions of the three aforementioned fault directions. The joints appear very high persistence (>20 m), wide spacing (1-2 m), no separation (< 0.1 mm) and frequent intensive weathering, especially close to the surface. The mean dip/dip direction values of the main joint sets are clearly presented at the Schmidt stereo diagram of Figure 4.

The shear zones develop mainly parallel to the schistosity and secondarily along to the joint sets. They are occupied by weathered and fractured mica and amphibolic shale and their width varies from a few centimetres to some meters. They affect intensively the mechanical behaviour of the rock mass, as they constitute zones of geomaterial with downgraded mechanical characteristics.

The hydrothermal alteration reduces the mechanical behaviour of the rock mass especially along the joints and the schistosity planes. This phenomenon shows great intensity close to the surface, forming a thick weathering mantle that affects the stability of the upper sections of the slope (Fig. 5).

Figure 4. Schmidt stereo diagram presenting the main joint sets and the schistosity planes, in relation to the slope planes.
3 Failure mechanism of the landslide

The intense fracturing, the extensive shear zones and the hydrothermal alteration of the rock mass as well as the thick weathering mantle and the steep geometry have been constituted the preparatory causal factors of the failure (WP/WLI, 1994).

The landslide appears to be a domino of successive wedge failures. As presented in the Schmidt stereo diagram of Figure 6 the main joint sets intersecting the slope phase, form unstable wedges with an axis dip angle of 50°, greater than the joint plane friction angle. Apart from the wedges failures, plane failures can also occur depending on the variation of the slope’s orientation.

The existence of a wide shear zone intersecting the slope in the middle of the failed section definitely contributed to the manifestation of the landslide. The crushing of shear zones’ shale materials by the overlay gneiss rock mass weight contributed to the triggering of the failure. Some of the activated failure planes as well as a section of the 2m wide shear zone are clearly presented at the pictures of Figure 7.

The failure led to the formation of an extensive cone of debris at the base of the slope. As presented in Figures 2 and 7 the 330,000m³ cone has blocked the exit of the stilling basin setting the spillways inactive. The landslide clearly affected the upper sections of the slope. The extension of the failure at the base of the slope cannot be detected because of the debris materials. Nevertheless, it is certain that the magnitude and the volume of the failure have also caused extensive damages to the lower benches and access roads.

The extensive weather mantle at the crest of the slope acted as an external load. The weathered materials react as soil materials manifesting transitional failures (Varnes, 1978). These movements are able to displace the loads at the crest of the slope and to contribute as a triggering factor at an extended failure.

Note that all aforementioned preparatory causal factors manifest at the entire slope. So the possibility of facing a proportional failure at the nearby sections of the slope is strong. This fact has been taken into consideration during the design of the remedial measures.
Figure 6. Schmidt stereo diagram indicating the unstable wedges. The plains intersection is located into the "zone of potential instability" (grey area) proving the instability of the wedges (Hocking, 1976; Hoek & Bray, 1981; Wyllie & Mah, 2004). Furthermore, plain failures can occur in slope sections with favorable direction along the $J_1$ and $J_2$ planes.

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<thead>
<tr>
<th>ID</th>
<th>Dip / Direction</th>
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<tr>
<td>$J_1$ (Joint Set 1)</td>
<td>55 / 260</td>
</tr>
<tr>
<td>$J_2$ (Joint Set 2)</td>
<td>50 / 326</td>
</tr>
<tr>
<td>Sch (schistosity)</td>
<td>48 / 053</td>
</tr>
<tr>
<td>SL1 (Slope 1)</td>
<td>70 / 305</td>
</tr>
<tr>
<td>SL2 (Slope 2)</td>
<td>70 / 265</td>
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Figure 7. A) View of the failure, pointing failure planes exposed at the left side of the slope, B) Close view of the joint – failure plane belonging to the $2^{nd}$ joint set (yellow arrows polygon) and the 2m wide shear zone (green shaded area) and C) view of the debris cone.
4 Remedial measures

For the restoration of the slope a combination of remedial measures, as well as geometry alterations have been specified. All measures were designed taking into consideration the stability problems of the entire slope. As already mentioned, the crest of the slope is occupied by an extensive weathering mantle, so the possibility for removing those top layers was examined. Also, the stability problems of the existing service roads set them inaccessible from heavy vehicle, excluding them from the configuration of the new design of the slope.

The main measure proposed for the restoration of the failed section of the slope was the total redesign of its geometry. The proposed geometry is clearly presented in Figure 8 and is sufficiently gentle with a mean inclination of 50° (1.2/1). The new inclination is similar to the dip angle of the mean wedge axis, so the number of the wedges rising on the surface of the slope reduces effectively. Considering that the failure has already caused extensive over excavations and that the mean inclination of the slope prior to the failure was 60°, the proposed earthworks are not enormously extensive and they were estimated to 700,000m³, including the 330,000m³ of the failure.

Figure 8. The proposed geometry for the restoration of the slope. The section included in the green polygon concerns the weathering mantle and its final geometry should be redesigned for the entire slope. The final geometry of the crest is going to finalise the extension and the height of the connection slopes (polygon with net pattern).
Considering the stability problems of the existing service roads a new independent access road, approaching the site from the top of the slope, was designed. Consequently, by starting the earthworks from the upper sections of the slope, the construction of the successive hairpins of the road can be simultaneously constructed. The new access road network is completely independent, considering the old service roads.

For the prevention of rock pieces’ breaking out, the installation of a steel wire mesh was proposed. The high tensile wire mesh must be anchored with 1m long rock nails, in a 3m X 3m arrangement, in order to be pulled as tightly as possible over the slope surface. Considering the wide spacing of the joints the 1m long rock nails are sufficient. Along the access roads, boundary steel ropes should be built in the wire mesh and, 3m long, lateral grouted anchors should be installed, with 2-3m spacing. Along the base of the slope, considering its height, a 2000 kJ rockfall protection barrier can be precautionary installed for the protection of the main access road, as indicated in Figure 8.

Furthermore, this case study indicates the importance of the geological parameters, which should be considered during the design and the construction of such extensive excavations. Practically a geologically compatible design reduces the cost of the support measures and increases the life of the constructions.

5 References


