# TECHNO-ECONOMIC ASSESSMENT STUDY FOR ROGUN HYDROELECTRIC CONSTRUCTION PROJECT

## PHASE II: PROJECT DEFINITION OPTIONS

## Volume 2: Basic Data

## Chapter 3: Geotechnics

**March 2014**

Report No. P.002378 RP 45 rev. C

| C | 31/03/2014 | Final Version | C. Vibert | L. Corti | N. Sans |
|---|---|---|---|---|---|
| B | 12/06/2013 | First revision | C. Vibert | A. Lara | N.Sans |
| A | 11/04/2013 | First Emission | C. Vibert | R. Albert | N.Sans |
| **Revision** | **Date** | **Subject of revision** | **Drafted** | **Checked** | **Approved** |

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# 1 ABREVIATIONS AND REFERENCES

## 1.1 List of abbreviations used in the text

GIN is the grouting intensity number as defined by Lombardi for dam curtain grouting execution.

GSI: Geological strength index, as defined by Hoek and other authors

HPI: Hydroproject Institute, Moscow, Russia

HPT: Hydroproject Tashkent, HPI Branch in Tashkent, Uzbekistan (formerly part of the Soviet Union)

ISRM is the International Society for Rock Mechanics, which has issued a large number of recommendations and suggested methods for rock testing.

RQD means “Rock Quality Designation” according to Deere, 1964, i.e. within one drill turn, the percentage of the cumulated length of cores longer than 100 mm to the total length of the turn.

RMR: Rock Mass Rating, as defined by Bieniawski in 1989

CSGNÈO: Service Center for Geodynamical Research in Energy Industry (a branch of HPI)

## 1.2 Terminology

Work was performed in English translation of the original documents, therefore many cross-checks with the original documents in Russian had to be made.

Wherever references including Cyrillic letters are listed, the Roman alphabet equivalent was used. As an example, the Russian abbreviation “ЦСГНЭО” is transliterated as CSGNÈO. The only exception to this rule is for reference numbers, especially boreholes and galleries, where the corresponding Cyrillic letter has been transliterated by the Roman letter of the same rank in the Roman alphabet to reflect the original sequence. Therefore, and as an example, boreholes “1004б” and “1004в” have been referred to as boreholes 1004b and 1004c.

“Aleurolite” is the Russian word used in many translation, but can be considered equivalent to “siltstone”. In order to avoid any confusion, it has been preferred to use siltstone as a correct translation, this last term being by far the most employed internationally. Nevertheless, the word “aleurolite” may remain in some figures or tables.

”Argillite”, similarly, has been as much as possible replaced by its “claystone” equivalent.

Dipping attitude of geological layers are conventionally noted using the direction of dipping (using three digits from 000 to 360 degree), and the dip angle with respect to horizontal (two digits). Hence, 130/70 means a plane dippig 70 degrees with respect to horizontal towards a direction making an angle of 130 degrees clockwise from North.

Hydraulic conductivity means the large-scale rock mass permeability. In Russian documents the common unit for hydraulic conductivity is meter per day, i.e. approximately $1.16 \times 10^{-5}$ meter per second, while intakes from water tests are usually counted in meter per minute (and per meter length of borehole).

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# 2 GENERAL DESCRIPTION OF THE NATURAL CONDITIONS ON THE DAM SITE

## 2.1 Location within regional geological frame

The Rogun dam site is located in the central mountainous region of Tajikistan, where the dominant trends are high mountain ranges and intense folding of the sedimentary series under high tectonic stresses. The area is tectonically active, with major active faults at the very proximity of the site (regional Gissar-Kokshal Fault and Illiak-Vakhsh Fault, then the comparatively minor Ionakhsh and Gulizindan faults).

Detailed description of the geological context is made in Phase II Report - Volume 2 – Chapter 2 - Geology.

Briefly speaking, the two Gissaro-Kokshal and Illiak-Vakhsh Faults, inferred following the course of the river upstream of the site, constitutes the main contact between the northern Tian Shan block, essentially composed of intrusive rocks in this area, and the so-called Tajik Depression, here mainly composed of intensively folded sedimentary series dating back to Jurassic.

These sedimentary series are affected by overthrusts towards north-west, created under the high compressive tectonic stresses. The two major faults near to the dam site, namely Ionakhsh Faults and Gulizindan Faults, pertain to this system of overthrusts.

Additionally, antithetic faults have been generated by the upwards movement of the blocks along the thrust faults. The main fault pertaining to this family is the one known as Fault 35, overthrusting towards south-east, and cutting the dam foundation.

The seismic activity of the faults has been demonstrated in the past, when geodetic measurements demonstrated that slow creep movements of about 1 to 2 mm/year average were occurring along Ionakhsh, and Gulizindan faults, as detailed in Ref. [1] (§ 2.2.3.) and subsequently issued documents. Movement was also evidenced along Fault 35, at a lower rate (see Phase II Report - Volume 2 – Chapter 2 - Geology, § 3.3.2).

Farther north, and just upstream of the dam site, the 1978 Design Report mentions that movement is also most likely to occur along Fault 367 (Ref.[1], § 2.6.4).

Throughout the site, other faults are present, presenting a similar attitude as either Ionakhsh Fault or Fault 35. One of the main conspicuous on site is Fault 70, of similar attitude than Fault 35, which is reported to have been monitored in the investigation gallery 1030 of the left bank. Monitoring (accuracy of 0.1 mm) stopped after two years with reportedly no movement evidenced along this fault (Ref. [4], § 1.1.1.2).

Consequently it was assumed in the Original Project by HPT that no deformation movement was taking place within the block delimited by Ionakhsh Fault on the upstream side and Fault 35 on the downstream side. Consequently, this block has been selected to shelter the main structures of the power scheme.

Therefore, and as a result of the tectonic kinematics, the dam site itself can roughly be divided into three parts:

- The sector located north of Ionakhsh Fault, where thrust movement along this major tectonic feature generated an asymmetrical syncline (Kirbich syncline)
- The thrust block limited to the north by the Ionakhsh Fault and Fault 35 to the south, where the main structures of the hydropower plant are to be located; bedding of the rocks has a similar attitude as Ionakhsh Fault, dipping steeply towards SSE,
- The area located downstream of Fault 35

This configuration is illustrated by Figure 2.1. (Gulizindan Fault, being at some 1 km distance from the dam site, is not represented on the figure).

Attitude of Ionakhsh Fault is 130/70-80 (then towards downstream), while Fault 35 has a 330-340/45 average attitude towards upstream. The overall attitude of the bedding of the rock averages 130/70.

Generally, on-site observations in surface corroborates the evidence of tectonic activity, since comparatively to many other sites, fractures within the rock massif appear as fresh, clear cuts, often with some aperture in surface, so that it gives an impression of (geologically) recent movements of the rock masses. We will come back on this topic later, which is deemed very important.

## 2.2 Nature of rocks of the dam foundation

With regard to the nature of the rocks of the foundation, the sedimentary series of the dam site are essentially made of an alternance of less resistant claystones and siltstones, against more resistant sandstones, with diversely represented gypsum. Younger formations such as Upper Cretaceous and Paleogene additionally present strata of limestone, shales or chalk.

Sandstones are reported to be fine-grained (mostly rounded grains of quartz for 20 to 45%, sometime up to 60%, the rest mainly feldspar), with carbonaceous cement.

Salt rock is present within the Ionakhsh Fault (see paragraph 2.3) and in the Gulizindan Fault, as well as in diapirs more or less aligned along the main Illiak-Vakhsh Fault. The salt probably acts as décollement surfaces for the thrust faults.

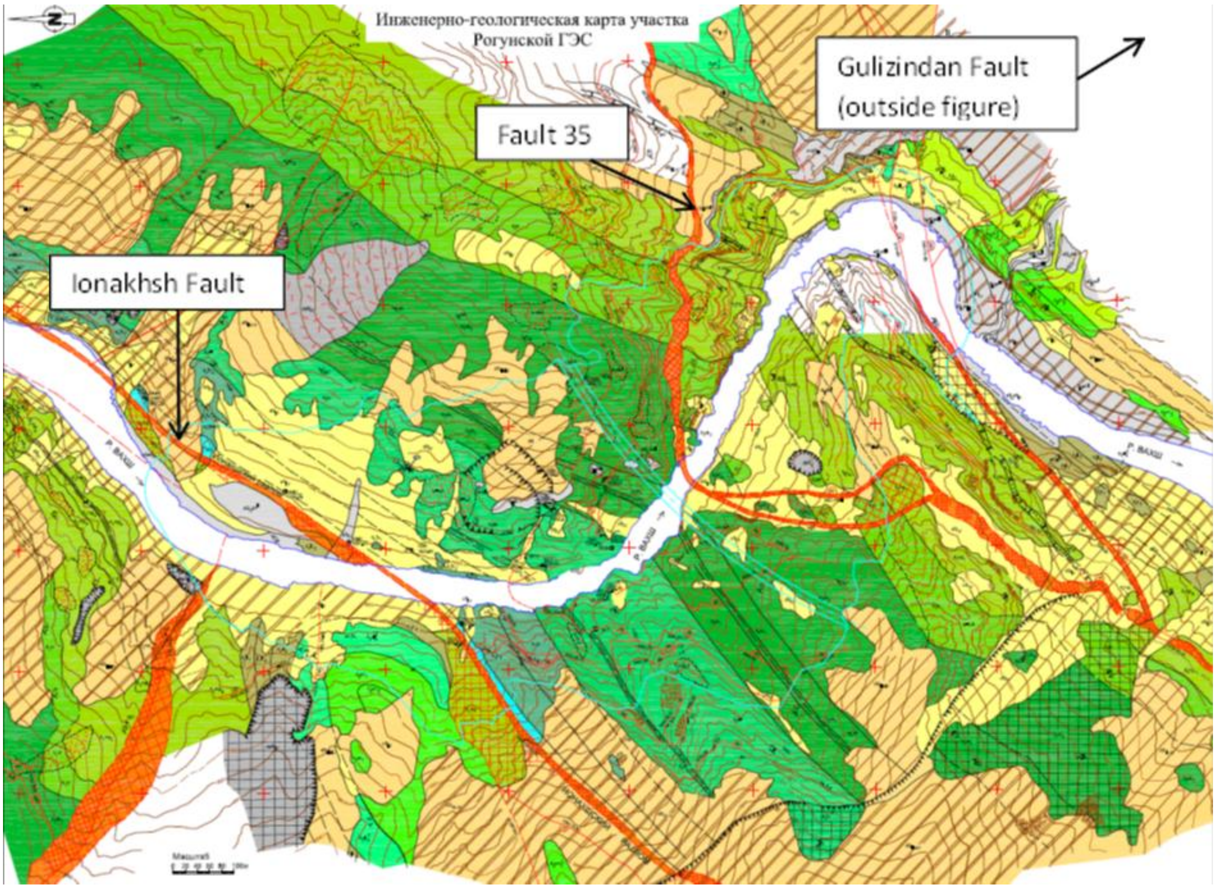



**Figure 2.1: Identification of major tectonic features on the dam site (Gulizindan Fault is located further in south-east direction and outside of the frame of the figure, it is subparallel and of similar attitude as Ionakhsh Fault)**

The bedding of the rock foundation within the gorge of the dam site dips steeply towards south-east, with a general attitude of 130/70, roughly parallel to Ionakhsh Fault.

| Geological age | | Formation | Symbol | Thickness of geological layer (m) | Thickness on dam site (m) (4) | % of content of the different kinds of rocks (4): Sandstones | Siltstones (aleurolites) | Mudstones (argillites) | Limestone | Gypsum | Lithology and description of rocks |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Cretaceous | Maastrichtian | | $K_2m$ | 80 - 110 | – | – | – | – | – | – | Grey, pink-grey, massive LIMESTONE with rare interlayers (0,5-10 m) of SANDSTONE. Limestone in the basis are slightly argillaceous, grayish-green and fossiliferous. |
| | Campanian | | $K_2cp$ | 50 - 55 | – | – | – | – | – | – | Green, grayish-green mudstone, slightly sandy, with interlayers (0,5 m) of marls; grayish-green, weakly sandy limestone with fossil debris; grey sandstone. |
| | Santonian | | $K_2st$ | 53 | – | – | – | – | – | – | Bedded, greenish-grey and pink, bedded gypsum; grey, yellowish-grey, brown, reddish, greenish-grey, purple mudstone, siltstone and sandstone. |
| | Coniacian | | $K_2cn$ | 65 - 80 | – | – | – | – | – | – | Greenish-yellow micaceous sandstone with interlayers of grey, sandy limestone and yellowish argillaceous limestone. |
| | Turonian | Upper Turonian | $K_2t_2$ | 150-160 | 231 | | | 50 | 49 | 1 | Upper suite: thin alternant grey, and dark grey shales and yellowish-grey, clayey limestone and marls. At the base of this suite, up to 20 m thick fossiliferous limestone layer. Lower suite: grey fossiliferous limestone, greenish-grey, marly shales with interlayers of fossiliferous marls. Gypsum mentioned in source (2) |
| | | Lower Turonian | $K_2t_1$ | 90 | 82 | | | 95 | 5 | | Dark-grey and greenish-grey shales with rare interlayers of grey fossiliferous marls. |
| | Cenomanian | Upper Cenomanian | $K_2cm_2$ | 33 - 60 | 61 | 5 | | 18 | 74 | 3 | Alternant grey, fossiliferous clayey limestone with dark grey and grey shales, micaceous sandstone, siltstone and shales with gypsum lenses. |
| | | Lower Cenomanian | $K2cm_1$ | 45 - 50 | 45 | 15 | | 30 | 55 | | (1): Mudstone, sandstone, limestone, gypsum; (2):Grey, pinkish-grey cryptocrystalline compact limestone, greyish-green, locally brownish, medium-grained sandstone. (3): alternant dark grey and gray shales, with gypsum lenses, grey argillaceous limestone. |
| | Albian | Upper Albian | $K_1al$ | 50-65 | 50 | 20 | | 45[(b)] | 1 | 34 | Mostly alternant brownish-reddish claystone and gypsum, with grey sandstone. |
| | | Upper Lyatoban | $K_1lt_3$ | | 17 [(a)] | | 26 | 39 | 16 | 19 | Frequent interlayers of white and pink saccharoidal gypsum with greenish-grey and brownish claystone, siltstone and sandstone. Rare interlayers of light-grey limestone. |
| | | Middle Lyatoban | $K_1lt_2$ | 30 - 130 | 21 | 59,1 | 23,4 | 14,1 | | 3,4 | Dark-brown, fine-medium grained, micaceous sandstone with interlayers of greenish-grey siltstone (0,1-0,2 m). |
| | | Lower Lyatoban | $K_1lt_1$ | 35 - 115 | 38 | 24 | 15 | 23,2 | 33,3 | 4,5 | Thin, dark-grey, greenish-grey shales with thin layers (0,3-0,5 m) of light-grey cryptocrystalline limestone, dolomite, marls (0,6 m) and white saccharoidal gypsum (0.1 to 0.4 m). In the medium part of the layer - |
| | | Mingbatman (5) | $K_1mg_5$ | 130 - 375 | 51 | 86 | 13,5 | 0,5 | | | Dark-brown to light-grey, fine-grained sandstone with some interbeds of siltstone (0.5 to 3 m) |
| | | Mingbatman (4) | $K_1mg_4$ | | 74 | 64 | 25 | 11 | | | Alternance of dark-brown, fine-grained sandstone, with siltstone and claystone layers of variable thickness); 8 m-thick layer of brown sandy siltstone with some claystone in the lowest part |
| | | Mingbatman (3) | $K_1mg_3$ | | 73 | 87,5 | 10 | 2,5 | | | Pinkish-brown to reddish-brown sandstone with thin interbeds of siltstone and claystone; some interbeds of breccia (0.15 to 0.8 m) |
| | | Mingbatman (2) | $K_1mg_2$ | | 29 | 45 | 12 | 43 | | | Irregular alternance of light-brown sandstone, siltstone and claystone |
| | | Mingbatman (1) | $K_1mg_1$ | | 97 | 80,5 [(c)] | 17 | 1,5 | | | Reddish-brown to dark brown, fine to medium-grained sandstone with thin interlayers of siltstone and claystone (up to 0.5 m) |
| | | Karakuz | $K_1kr$ | 85 - 175 | 101 | 69 | 12 | 18,9 | | 0,1 | Reddish-brown sandstone with interlayers of grey and brownish mudstone; continuous gypsum layer of average 0.25m thickness close to the lower contact |
| | Hauterivian - Aptian | Upper Obigarm | $K_1ob_2$ | 110 - 240 | 229 | 96,1 | 2,7 | 1,2 | | | Sandstones, mainly brownish-reddish and purple with brownish-grey and light-grey interlayers; subordinate thin mudstone interlayers. |
| | | Lower Obigarm | $K_1ob_1$ | 80 - 115 | 93 | 0,1[(d)] | 85[(d)] | 14,6[(d)] | | | Brown, dark-grey, green mudstones, siltstones; white gypsum interlayers, up to 30 cm thick in the middle part; upper part includes light-grey sandstone interlayer. |
| | Valanginian - Hauterivian | Kyzyltash | $K_1kz$ | 165 - 205 | 192 | 83,3 | 6,6 | 10,1 | | | Brownish-red, fine-medium grained, micaceous sandstone, subordinate interlayers of siltstone and mudstone. |
| | | Upper Yavan | $K_1jv_2$ | 40 - 100 | 54 | 13 | 41 | 46 | | | Reddish-brown and brown, foliated mudstone, siltstone. Source (2) mentions malachite (Cu) in two interlayers of greenish-grey, bluish-grey mudstone. |
| | | Lower Yavan | $K_1jv_1$ | 30 -285 | 48 | | 55 | 45 | | | In the upper part, reddish-brown, brown and purple fine-grained sandstone, siltstone and mudstone. Lower part: dark-brown and reddish-brown shales and siltstone, some gypsum lenses 0.1 - 0.3 m thick. |
| Jurassic | | Gaurdak | $J_3gr$ | 400 | 22 | | | 100 | | | Reddish-brown mudstone with gypsum, overlying salt rock (halite) |

*Sources mentioned if the description is different: (1) FS 1978; (2) Stratigraphic column 1962-63; (3) Geological report (draft) 2012, Barki Tojik*

*(4) 1174-03-78, Sheet 1, FS 1978*

*(a) Estimated from the drawing (no mention)*

*(b)From Table 2.4 of FS 1978 (total not found to be 100% in drawing 1174-78-03, sheet 1)*

*(c) Total not found to be 100%; probably 81.5%?*

*(d) From Table 2.4 of FS 1978 ( total not found to be 100% in drawing 1174-78-03, sheet 1)*

**Table 2.1: Stratigraphy of sedimentary rocks of the dam site, with description of rock characteristics and approximate distribution of rock types (synthetic table from different sources)**

The sequence of geological strata and formations, from Jurassic to Upper Cretaceous, as well as the distribution in the nature of the rocks, is presented in Table 2.1.

A special mention shall be made of gypsum, which is found scattered in many of the various geological formations of the site. As emphasised in the Phase 0 report, dedicated to the analysis of the salt rock dissolution processes, gypsum is present near to the salt rock as a

product of the hydratation of the anhydrite impurities contained in the salt rock after dissolution of the latter. It is especially found in the Gaurdak claystones adjacent to Ionakhsh Fault, but gypsum can be found as layers of some 50 to 200 mm thickness within Lyatoban Formation, but especially within Upper Albian Formation, where thickness of gypsum layers reaches 6 m.

Distribution of the rock masses is as follows:

- Geological formations from Jurassic (Gaurdak) to Mingbatman make the foundation of the projected dam, on the south-east side of Ionakhsh Fault,
- Mingbatman Formation and younger one are present on the north-eastern side of Ionakhsh Fault, as well as further in the south-east, beyond the dam site, and within the "disturbed zone" (see paragraph 2.4.3).

Their geotechnical characteristics are described in paragraph 6.5 and 8.2 below.

## 2.3 Salt rock of Ionakhsh Fault

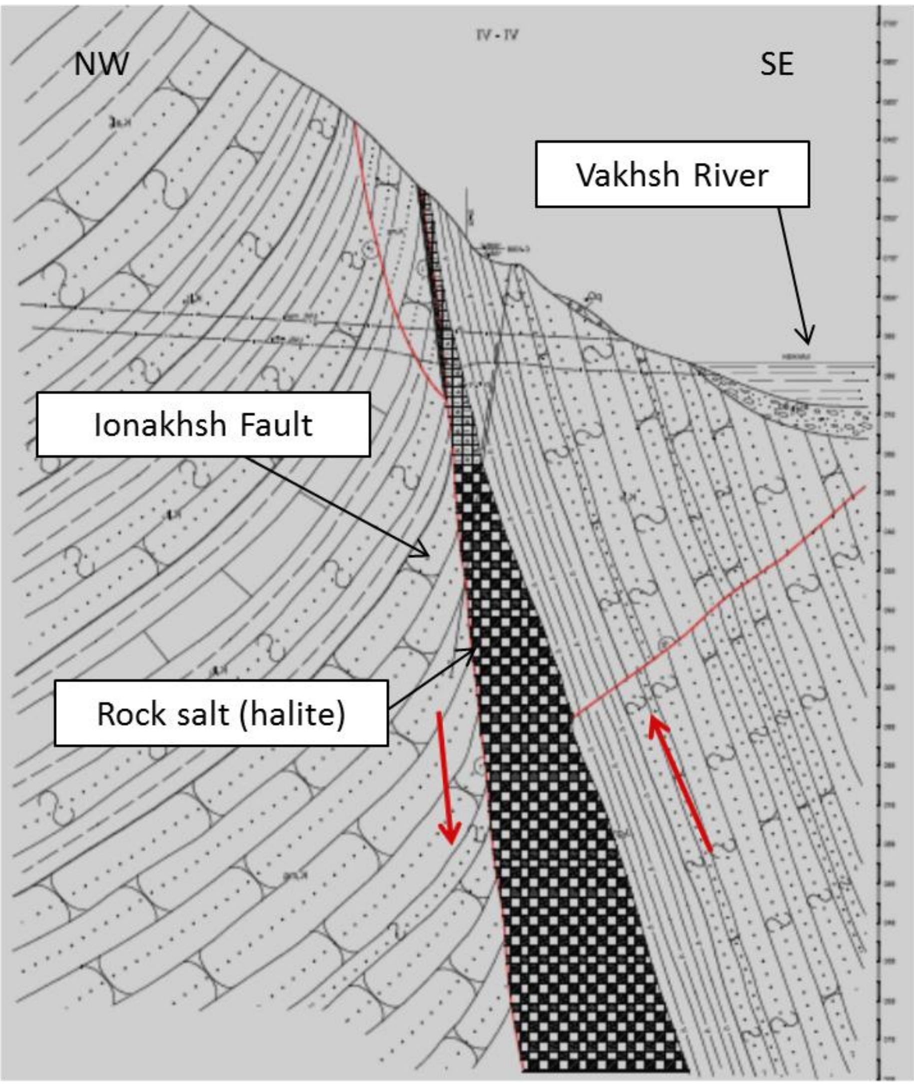
As briefly mentioned above, salt rock pertaining to the Jurassic Gaurdak Formation is present along the two main thrust faults (Ionakhsh and Gulizindan), as well as diapirs along the Illiak-Vakhsh Fault, upstream of the dam site.

Given that Ionakhsh Fault is to be located below the Stage 1 dam (elevation 1110 m a.s.l), and therefore under the upstream shell of the main dam, many investigations have been dedicated to the knowledge of the geometry of the salt wedge,

Geotechnical issues related to the presence of the salt wedge within the Ionakhsh Fault and the assessment and the impact of a possible dissolution are the object of the Phase 0 Report, and will not be dealt in detail here.

Nevertheless, it is worth to mention here the main results of the different investigations to understand the geological and geotechnical frame of the site. According to investigations for the 1978 Original Project (Ref. [1], § 2.6.6.), the geometry of the salt wedge within Ionakhsh Fault is the one of a V-shaped wedge bottom-up, opening in depth, due to the different inclinations of the two rims of the wedge, namely dipping towards upstream of 82 to 85 degree for the lying rim (Ionakhsh Fault thrust), and of 70 to 75 degree for the upper rim. On the basis of the available results, increase of thickness of the salt would be in average 15 m every 100 m. The thickness of the top extremity of the wedge is about 1.5 or 2 m, up to 12 m, tending to increase from the left bank to the right bank (Ref.[1], § 2.6.6 and relevant drawings).

Elevation of the top of the salt wedge has been first recognised as up to 970 masl on the left bank (borehole 1019, where its thickness is reduced to less than 2 m), and was reportedly locally unveiled by the excavation for the diversion tunnels (Ref. [16], § 2.1.1.). Elevation of the top of the salt rock in the valley bottom is estimated to 950 masl, hence about 32 m below the lowest water level in the river. Within the right bank, elevation of the top of the salt wedge varies between 956 masl and reaches 964 masl (borehole 2013c) at the deepest location of the investigated section inside the bank (Ref.[1], § 2.4, and corresponding drawings; note that in the same document, § 2.6.6, figures are different, notably due to an apparent confusion between left and




right banks).

**Figure 2.2: Typical cross-section showing the shape of the salt wedge (black and white squares) within Ionakhsh Fault, right bank of the Vakhsh river, upstream of the dam site; thickness of the salt wedge increases towards the right bank**

Figure 2.2 shows the corresponding typical cross-section of the salt wedge within Ionakhsh Fault. Within the right bank, investigation stopped at the end of the investigation gallery 1001, some 500 m from the river bed along the fault. The gallery did not reach the limit of the “disturbed zone” (see paragraph 2.4.3).

The rock salt appeared as a strong rock, without fissures and watertight, where halite (salt mineral) makes 76 to 92% of the whole rock (Ref.[1], § 2.2.2), and 79.3% in average (same reference, § 2.5.2, and Ref. [16], § 2.1.6.2.). The remaining part of the rock is mostly made of anhydrite (estimated to an average 50% of components other than halite), and pieces of embedding rock. The estimated specific weight of the salt rock is estimated to 21.8 kN/m³, with a porosity of 0.027 (Ref.[1], § 2.5.2). Ref.[26] (§ 1.3.1.) considers different values for the average rock salt composition, where halite makes only 60.5% of the rock salt, the remaining part being constituted of anhydrite 25% and 14.5% of insoluble residues, on the basis of tests made by the Scientific Research Institute of HPI in 1985 (document not made available to the Consortium).

Disking observed during core drilling demonstrated that high compression stresses were acting on the fault, and in such condition, studies by HPT estimated that a steady rising of the salt wedge occurs and balances the leaching process by underground waters. Equilibrium between the two phenomena (salt rising and leaching process) is the basic assumption made for the calibration of the dissolution models (see Ref. [26] and Phase 0 Report). By analogy with the rate of rising of tectonic lenses of Fault 35 and Ionakhsh Fault, HPT estimates the ascending movement of salt as “close to 2-3 mm/year” (Ref.[1], § 2.2.3). However, this rising rate has been revised to some 2 cm per year in studies from 2005 on (see for instance Ref. [16]).

Indeed, above the top of the salt wedge, a distressed volume of rock is present, probably due to constant leaching of the salt. Even fall of tools were experimented during drilling of investigation holes (borehole 1004b; Ref. [1], § 2.4) in that distressed space, commonly named “caprock” in project documents, which dimensions are close to those of the width of the top of the salt wedge (about 8 to 10 m in width in the right bank) and 12 to 14 m in height according to Ref. [1], § 2.6.6 (7 to 8 m in height only according to Ref.[23]).

Hydraulic conductivity and porosity at large scale of the “caprock” could be assessed by means of interpretation of a pumping test performed by end 2012 on the left bank of the Vakhsh River. A hydraulic conductivity of some $1.2.10^{-4}$ m/sec and a porosity of 0.13 were found (see Phase 0 Report ).

Evidence of precipitation of secondary gypsum – by hydration of anhydrite - in the space where salt has been leached appears from cores of the different investigation boreholes, and the rock overlying the dissolution zone appears as brownish-red, sandy and silty mixture with inclusions of gypsum, anhydrite and siltstone (Ref. [1], § 2.4). The proportion of secondary gypsum is said to be about 6% and up to 19% above the groundwater level, and decreases to 2 to 3% below it (Ref. [1], § 2.6.6.).

As emphasized above, the impact of potential salt leaching within the Ionakhsh Fault on the Works are the aim of the Phase 0 Report.

Same phenomena related to the presence of rock salt are likely to occur within the Gulizindan Fault, where the same wedge-shape salt rock has been detected. This fault joins the Illiak- Vakhsh Fault some 4 km upstream of the dam site, on the left bank of Vakhsh River. Downstream of the dam, it intersects the Obi-Shur river, at elevation 1,060 (Ref. [1], § 4.2). Potential leakage from the reservoir through this fault is dealt farther in this report (see paragraph 13.2 and Geological Report, § 6.5.).

## 2.4 Geomorphological features of the dam site

### *2.4.1 General aspect*

At the dam site, the Vakhsh River makes a sharp bend from a NE-SW direction parallel to the regional lithology, then turns to NW-SE around the downstream toe of the forecasted stage 1 dam, where it flows perpendicular to the bedding of the embedding rock formations. It finally turns back to the NE-SW direction at the downstream toe of the main Rogun dam.

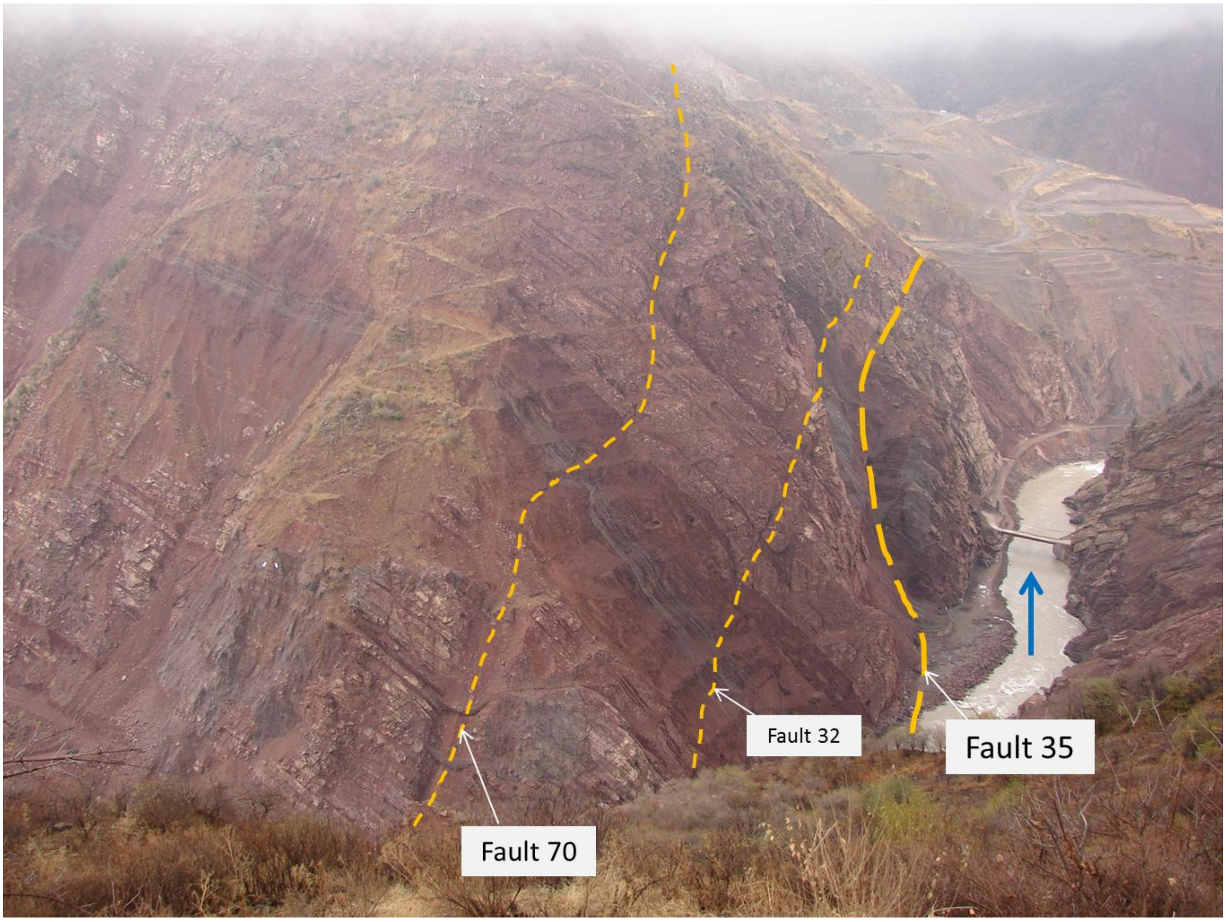



**Figure 2.3: View of the gorge of Vakhsh River on the dam site, towards downstream; Fault 35 is highlighted in the left bank, as well as approximate traces of Fault 70 and the minor Fault 32 (named as per original drawing 1174-03-78, Sheet 1 of Ref.[1]), both of them of similar attitude; the dam axis is near to the intersection of Fault 35 and the river bed**

The gorge at the dam site is V-shaped, with steep flanks of inclination from 40 to 60 degree with locally steep cliffs along the river stream, especially in the sandstones formations. Figure 2.3 gives an good idea of the topographic features of the site.

### *2.4.2 Geodynamical processes*

With regard to geodynamics phenomena, rockfalls, collapses and landslides are intensively developed on and around the dam site.

Rockfalls are common, especially when raining, due to the steepness of slopes and differential erosion between the interlayered siltstones and sandstones.

Gravitational sagging is also a potential cause of instability of rock masses on the dam site.

Main slope instabilities occur on the left bank, apparently potential structural instabilities along fractures of the massif with similar attitude as Fault 35, dipping towards North-West, (300-40/10-50) which are commonly continuous and infilled with clay (see Figure 2.3 and farther paragraph 6.6.1.2 and 8.3.2). Such a landslide has been identified just above the axis of the stage 1 dam, in the shoulder zone of the main dam, between elevation 1060 and 1180, more or less between the dam core and Ionakhsh Fault, with an approximate width of about 130 m and extending along the slope for about 160 m. (Ref.[24], 2.3.3.2.1.). Location of this landslide is shown on Figure 2.4.

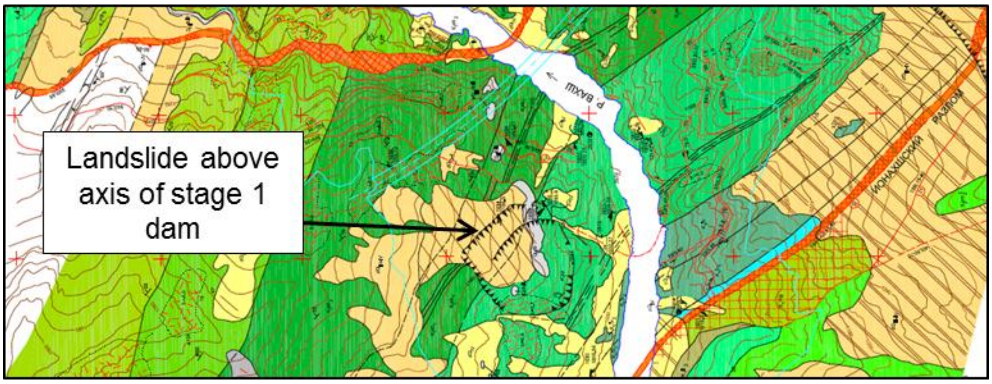



**Figure 2.4: Identification of the landslide on the left bank**

Another creeping mass is located more upstream, just above the entrance of the diversion tunnels, and is most probably responsible for damages registered there.

A description of main landslides and slope instabilities on and around the dam site, as well as in the reservoir, is also available in Phase II Report - Volume 2 – Chapter 2 - Geology.

### *2.4.3 "Disturbed zone" of right bank*

The right bank of the Vakhsh River is characterized by a peculiar morphologic feature, whereas it presents, on the top, a large and relatively flat plateau at elevation 1700-1750. This very particular feature of the site is visible on Figure 2.5.

Formation of such a feature was explained in the 1978 Original Project, by the past occurrence of massive landslides, in various stages. However, and although the density of investigations was high on the dam site itself, this structure has been comparatively very scarcely investigated (it is to recognise that detailed investigation of the subsurface structure of such a large mass would have requested quite a lot of time, with technics probably not as efficient as today).

Nevertheless, in the 2009 HPI design for completion of Rogun HPP (Ref. [27], § 1.7.1.), the level of knowledge of this zone has been considered as insufficient, due to the absence of boreholes and other adequate investigations. Hence, complementary investigations were dealt necessary, and investigations in two steps are recommended in the document (surface investigations, completed by an investigation gallery 1001b extending from the existing gallery 1001a. towards the interior of the right bank, to recognise the geological and geotechnical conditions of this zone below the level of the future reservoir).

Therefore, it was of paramount importance to assess the effect of impounding the reservoir on this zone, which is here called "disturbed zone". Especially because the topographical characteristics of this zone also recall that of karstic areas, i.e. suggest that some dissolution phenomena has been or is going on there. Therefore, it is essential to check if rising of the pore pressure in the massif when impounding the reservoir may lead to reactivation of ancient landslides, lead to further erosion of soluble rocks within this bank.

The majority of the complementary investigations ordered by the Consortium therefore dealt with the investigation of this part of the right bank, which structure is detailed in Phase II Report - Volume 2 – Chapter 2 - Geology.

The prior interpretation of huge ancient landslides for generation of this zone being discarded, there appears to be no risk of a huge landslide of involving the whole "disturbed zone".

The present report deals more precisely with the geotechnical features of the "disturbed zone" with regard to the dam and reservoir in paragraph 12.2.

It is however to mention the presence of large potentially unstable masses in the front part of this structure, at the foot of which large amount of debris produced by the scouring of the slope accumulate. According to Ref.[25] (§ 1), the river bed of the Vakhsh River was moved off the right bank of 70 to 90 m between 1978 and 2005 due to this reason.

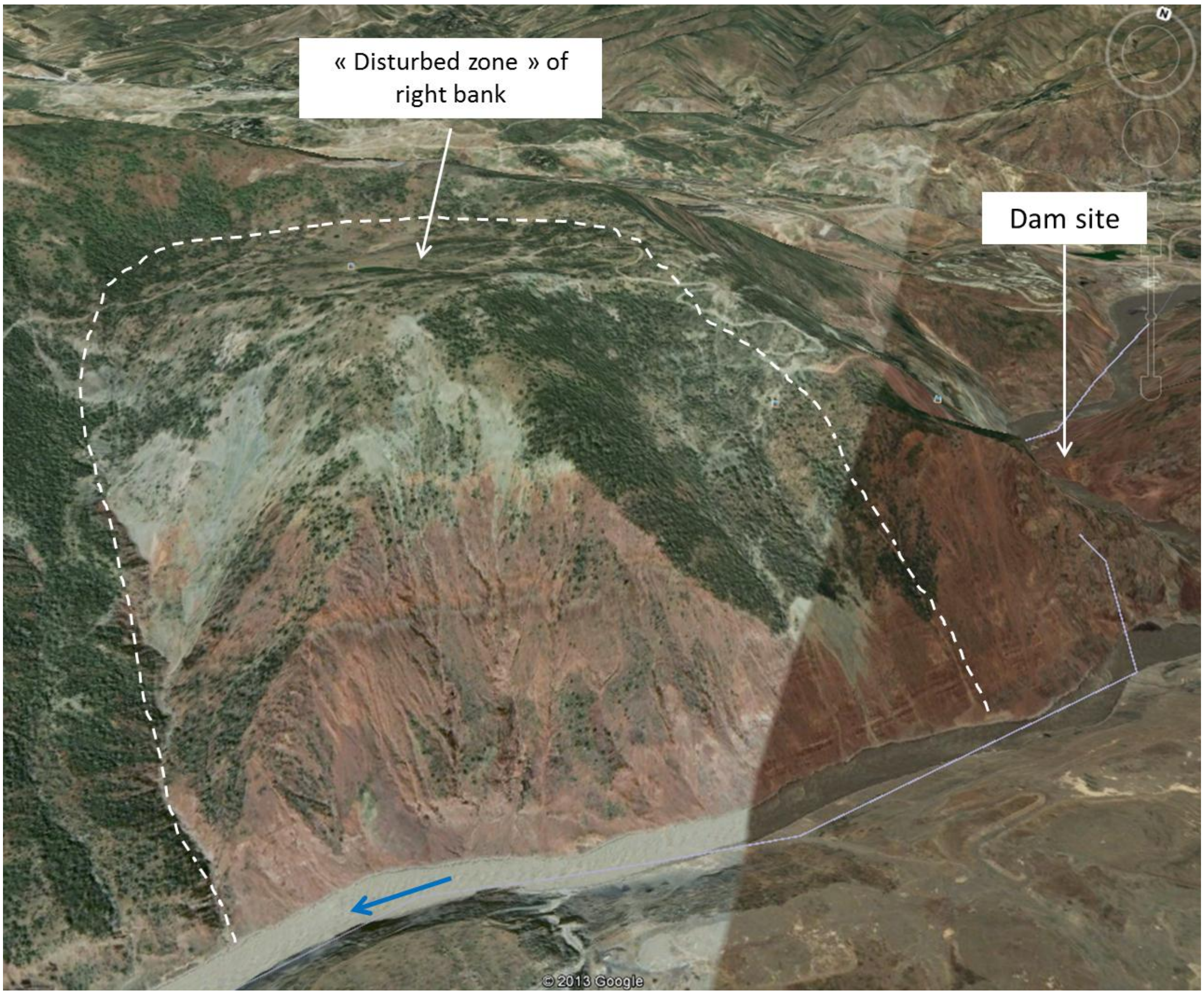



**Figure 2.5: 3D view of the dam site (river gorge on the right hand of the figure), and the "disturbed zone" of the right bank with its approximate limits (from Google freeware)**

## 2.5 Mudflows

Due to the high tectonic activity and rapid rising of the relief along the tectonic structures, and also sometime due to the presence of soluble rocks (gypsum, salt), slope instabilities are common in the catchment areas of the diverse tributaries of the Vakhsh River.

Especially, mudflows flowing down the Obi-Shur river, which joins with the Vakhsh River on left bank immediately downstream of the dam site, have already proved to present a subsequent risk for the power plant. As stated in Ref. [9], § 1.4, on 16 June 1993, mudflows from the Obi-Shur River - occurring subsequently to the breach of the upstream cofferdam in the night of 8 to 9 May 1993 – temporarily dammed the Vakhsh River, resulting in the flooding of the major underground structures, machine and transformer cavern (the work had been already suspended during the previous year).

Risks related to mudflows are specifically dealt in paragraph 12.3.

However, it is worth to mention here that a specific dam of 70 m height is under construction in the course of the Obi-Shur River, dedicated to retain the coarser part of the debris carried by the mudflows which regularly affect this river.

# 3 LAYOUT OF MAIN WORKS WITH RESPECT TO GEOLOGICAL FORMATIONS, ACCORDING TO HPT/HPI PROJECT

## 3.1 Layout of works

Different layouts have been worked out, but location of the dam foundation and of the underground structures being now excavated or partly excavated is likely not to be radically modified. An overall look of the underground structures and their geological environment is presented in Figure 3.1 (bolded black lines on this figure point out the geological contacts at the elevation of the power house in the left bank).

The geological environment and status of the main works is briefly described in the following paragraphs.

## 3.2 Dam foundation

Three stages of construction can be distinguished:

- Construction of the cofferdams and river closure,
- Construction of the first stage dam, with crest elevation at 1110 masl and normal water level at 1100 masl,
- Construction of the second stage or final dam, with crest elevation at 1300 masl and normal water level at 1290 masl.

The upstream cofferdam and the first stage dam are to be founded above the Ionakhsh Fault, at the upstream end of the gorge of the Vakhsh River.

Foundation of the final dam will include the whole of the gorge (North-South direction), its core and filters being founded mainly on the Lower Obigarm siltstones of lower hydraulic conductivity, and upstream of Fault 35 which outcrops in the downstream part of the dam body, within the downstream shell.

Excavations had been performed for the dam core foundation before 1993.

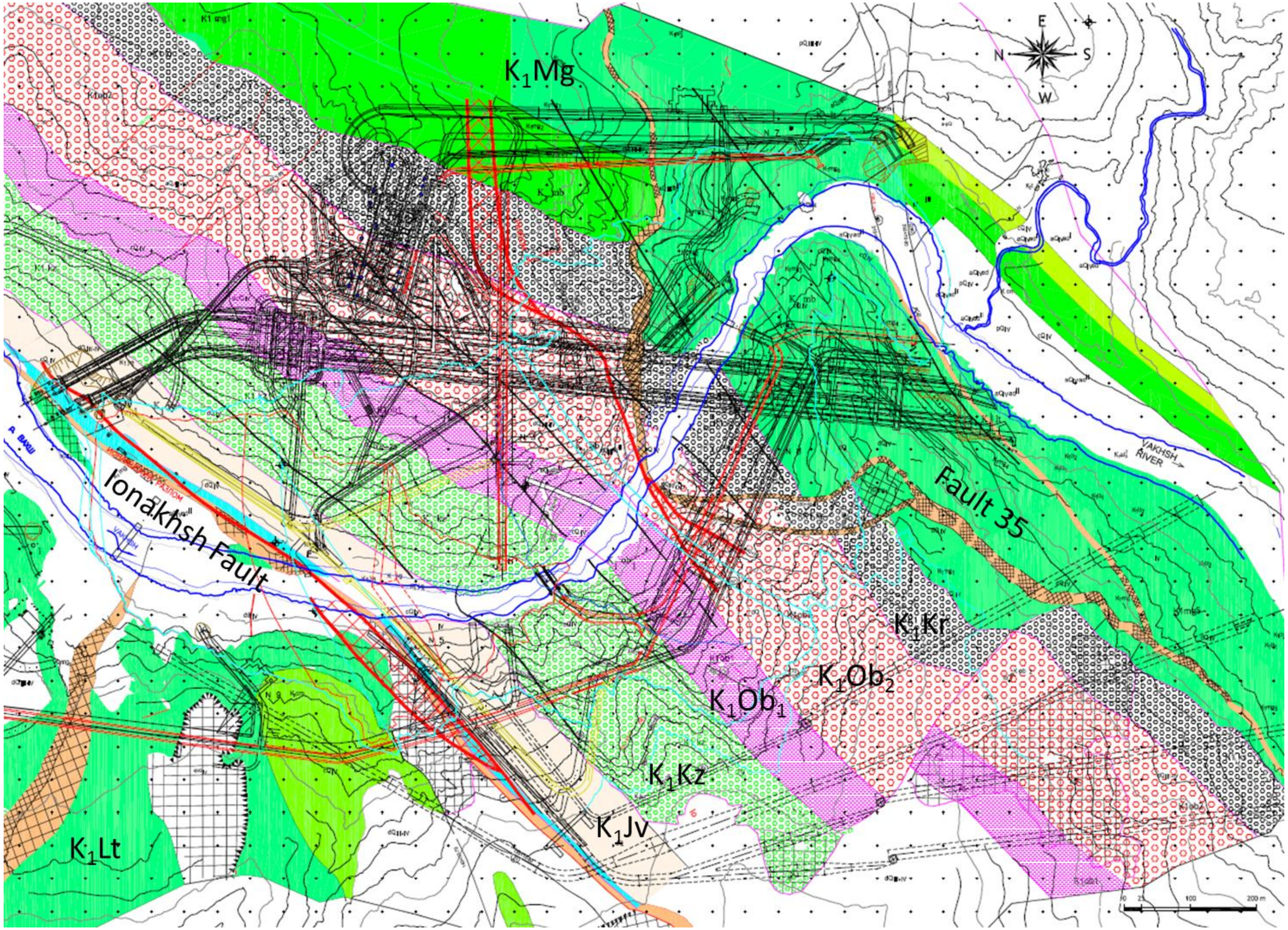



**Figure 3.1: Location of main works with respect to geological structures; note that bolded black and red lines are the projection of the geological contacts and faults respectively on an horizontal plane crossing underground power house (2012 layout)**

## 3.3 River diversion works during construction

The two main diversion tunnels are to cross the whole geological structures, from their inlet near to Ionakhsh Fault to the Mingbatman Formations downstream. They cross Fault 35, and the river just downstream.

The lower part of the diversion tunnels, including where crossing Fault 35, are to serve as free-flow tailrace tunnels.

These tunnels had been excavated before 1993, but suffered rock collapses, especially at their intersection with Fault 35.

## 3.4 Intake area and headrace tunnel

The permanent water intakes are to be located within the rocks of the Yavan Formation, on the left bank, at an approximate elevation of 1170 masl. It is to convey water to the power house via a tunnel crossing mainly through Kyzyltshash sandstones and Lower Obigarm siltstones before reaching the power house.

## 3.5 Power house and transformer cavern

The power house – the more generally used term “power house” has been preferred to “machine hall” – is a large cavern to shelter 6 units. Dimensions of the excavations as per original project, according to which the already existing portions were excavated, are as follows (source Ref. [29]):

- Total length of 219.7 m, including 62.7 m long erection bay and technical building,
- Height of 54.3 m in current part (maximum height 68.7 m), 36.95 m in erection bay and technical building section; more precisely, elevation of top of the vault is 1,001.2, with excavations down to 946.9 in current parts and 932.5 at the base of the spiral cases; excavation is down to elevation 964.25 for erection bay and technical building,
- Width of 20.8 m.

The transformer cavern is located at 42.7 m distance from the power house (rock pillar width), in the downstream direction, its approximate dimensions being 199.6 length, 18.8 m width and approximately 44 m height.

As can be seen on Figure 3.1 from the projection of the geological contacts, the caverns are to be mainly located within the hard sandstones of Upper Obigarm, except for a part of the power house, near to the its western extremity (units No.5 and No.6), which is located within the Lower Obigarm siltstones.

Both caverns have been oriented in HPI Project in a favourable direction with regard to the bedding.

The upper part of the two caverns has been partially excavated before 1993, but they were flooded the same year, until pumping out of the water in 2005.

### 3.6 Reservoir

Upstream of the dam site, the reservoir will extend some 70 km along the two main regional faults, namely Illiak-Vakhsh Fault and Gissaro-Kokshal Fault.

Details of the geological conditions in the reservoir are given in Phase II Report - Volume 2 – Chapter 2 - Geology.

## 4 SUMMARY OF GEOTECHNICAL INVESTIGATIONS PERFORMED

### 4.1 Before initiation of construction or during construction

The 1978 Technical Project of Rogun dam (Ref.[1]) presents (Table 2.1 of the said reference) the nature and amount of investigations performed for establishing the dam design, according to which construction was initiated.

Some additional investigations have apparently been performed after edition of this Technical Report, the final amount being, according to Ref. [18], the following:

- Geological field surveys at various scales,
- Core drilling for a total of 8,960 m, partially with permeability testing (557 No. water injection tests before 1978),
- Excavation of investigation galleries (about 4,000 m cumulated length), boring chambers and shafts,
- Performance of seismic profiles in investigation galleries (3,400 m before 1978) and in boreholes (480 m before 1978),
- Seismic and ultrasonic tomography,

Field and laboratory tests, including in-situ stress measurements, were also performed.

Figure 4.1 presents a map of the net of investigation galleries, boreholes, shafts and trenches performed on the dam site. It shall however be noted that, this figure being a translation of drawing 1079-03-183 Add, reproduced from Ref.[19], some errors have been spotted with respect with the original map (especially, boreholes in the downstream part of the river bed are actually located on the banks, and mentioned elevations of these boreholes is erroneous).

Apart from the stress measurements, which are specifically analysed in paragraph 0, other geotechnical in-situ tests were the following:

- deformability measurements of the rock massif by means of flexible borehole dilatometer were performed in gallery 1030 (investigation gallery to the caverns),
- measurements of the deformation of a round-shaped chamber near to Gallery 1030, loaded by radial jacks.

A number of additional investigations were achieved at the initiation and during progress of the construction, as more information was collected, especially from the excavation of the underground works. However, apparently only part of this information could be made available to the Consortium (some documents it is referred to could not be found, such as the report by Sredazgidropoyekt, “Working as-built documentation – Comparative estimation of engineering and geological conditions”, Tashkent, 1989).

## 4.2 After stopping the construction in 1993

Some minor investigations were made in 2005, especially seismic investigation for the right bank.

Further knowledge of the site was given by then by the re-initiation of the construction, from 2008 onwards.

Additional investigations, the reason of which will be explained shortly farther, were carried out in the frame of the present Feasibility Study, mainly focused on investigation of the plateau of the right bank (“disturbed zone”) and amounted in:

- 5 boreholes with Lugeon tests at relevant depths, and some additional rock mechanics tests,
- 3 seismic profiles,
- Microgravimetric survey,
- Installation of 19 piezometers,
- Springs: discharge measurements and chemical analyses of the water,
- 1 pumping test within dissolution zone of Ionakhsh Fault,

Geological and geotechnical field survey were also performed, mainly in surface on the right bank, but also on the dam site, especially by visiting the accessible former investigation galleries and the existing underground works.

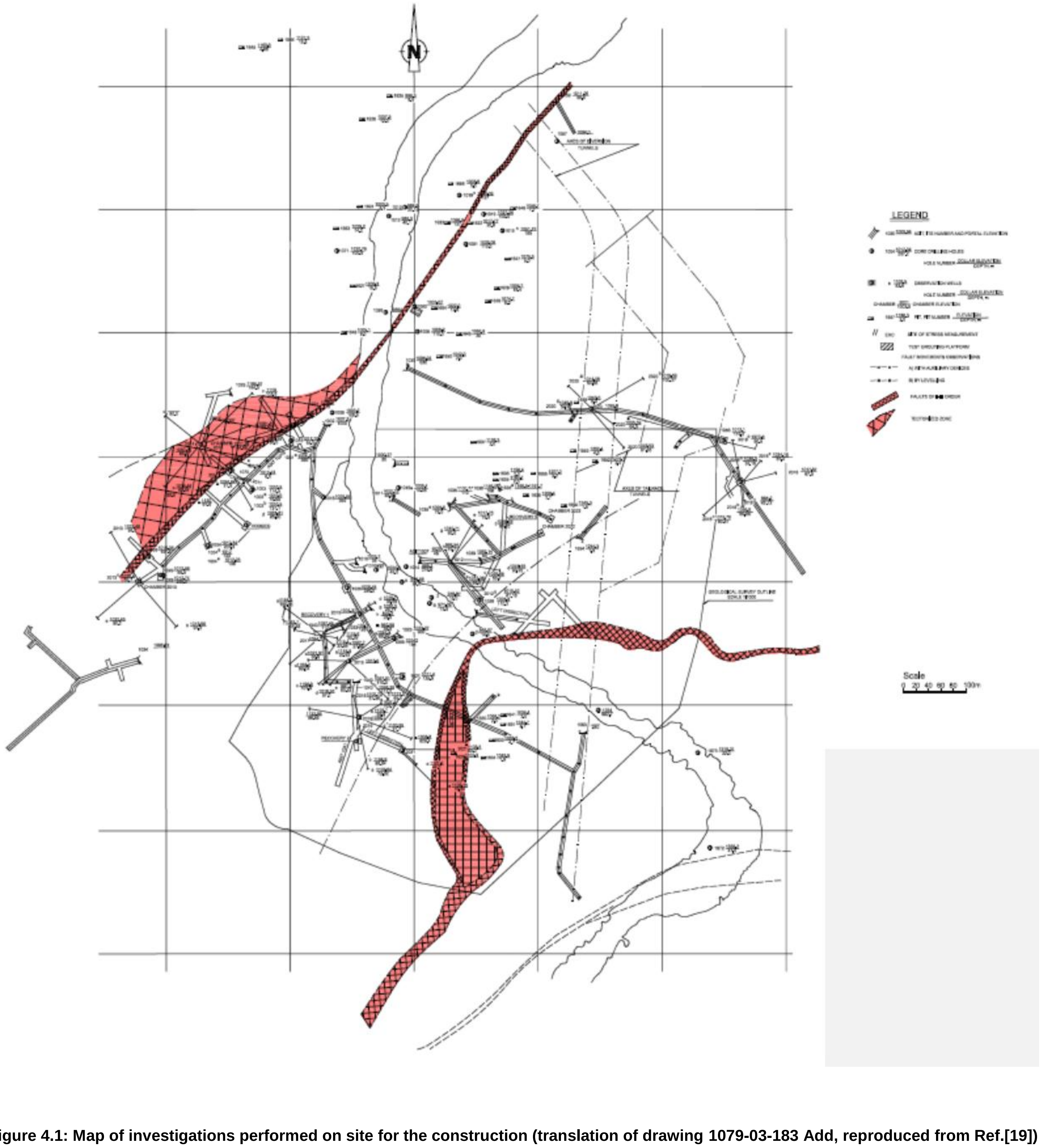



**Figure 4.1: Map of investigations performed on site for the construction (translation of drawing 1079-03-183 Add, reproduced from Ref.[19])**

For more detail on these additional investigations, see Phase II Report - Volume 2 – Chapter 2 - Geology and paragraph 8.1.1 and 9.1 here below.

# 5 ANALYSES OF IN-SITU STRESSES

As one could expect with the presence of the overthrusting faults (Ionakhsh and Gulizindan), the site is submitted to high compression stresses.

The high compressive stresses prevailing within the rock massif were evidenced by disking within boreholes and observations in galleries below 100 m depth under the ground surface in geotechnical zone IV (undisturbed rock mass, without any weathering or distressing effect).

The 1978 Design Report states that measurements of in-situ stress field was carried out by unloading method (no further precision) within this zone IV. Results show that in-situ stresses were significantly higher than the weight of the overlying rock (which is in the range of 7.5 to 8.5 MPa at the considered location).

The maximum values measured were 26 MPa in average, directed more or less parallel to the bedding joints and Ionakhsh Fault (direction towards north-east, “with angle of 5-8 degrees”, Ref.[1], § 1.2.3), and 24 MPa in the vertical direction (Ref.[1], § 2.2.3 and drawing 1174-03-F12).

It therefore concludes to a high compressive state of stress in the dam foundation, coherent with the slow, but regular creeping observed along the main regional thrust faults (Ionakhsh, Gulizindan).

Drawing 1174-03-F13 of Ref.[1] shows the location of three stress measurements within the investigation gallery 1030 (left bank, leading to the caverns), which crosses three of the main geological formations making the dam foundation (Kyzyltash – sandstones with some siltstones alternances, Lower Obigarm – siltstones, and Upper Obigarm – mostly sandstones).

Table 5.1 shows a synthesis of the results obtained in the gallery 1030, as per Ref.[1], drawing 1174-03-F13. Results show that:

- Measured vertical stresses are much over the weight of the vertical overburden in the locations of the measurements (up to almost five times in Lower Obigarm siltstones at PK 280),
- Maximum horizontal stresses parallel to the bedding were measured higher than those perpendicular to the bedding.

Drawing 1174-03-F13 of Ref.[1] shows the location of three stress measurements within the investigation gallery 1030 (left bank, leading to the caverns), which crosses three of the main geological formations making the dam foundation (Kyzyltash – sandstones with some siltstones alternances, Lower Obigarm – siltstones, and Upper Obigarm – mostly sandstones).

| Location / Geological formation | Vertical overburden / Corresponding estimated vertical stress | Measured vertical stress | Measured horizontal stress, along bedding | Measured horizontal stress, perpendicular to bedding |
|---|---|---|---|---|
| Gallery 1030 | m / MPa | MPa | MPa | MPa |
| Around PK 280, Lower Obigarm, (near contact with Kyzyltash)? | 250 / 7 | 33 | 36 | 33 |
| Around PK 470, Upper Obigarm, between Fault 70 and Fault 35 | 370 / 10 | 22 | 34 | 25 |
| Around PK 565, Upper Obigarm, some 30 m upstream of Fault 35 | 340 / 9 | 23 | ? | 26 |

**Table 5.1: Results of measurement of natural stresses in investigation gallery 1030, with vertical stress values as estimated from the weight of the overburden in second column, for comparison (data from Ref.[1], drawing 1174-03-F13)**

Table 5.1 shows a synthesis of the results obtained in the gallery 1030, as per Ref.[1], drawing 1174-03-F13. Results show that:

- Measured vertical stresses are much over the weight of the vertical overburden in the locations of the measurements (up to almost five times in Lower Obigarm siltstones at PK 280),
- Maximum horizontal stresses parallel to the bedding were measured higher than those perpendicular to the bedding.

On the contrary to what may be expected from the presence of the thrust faults (Ionakhsh, Gulizindan) the main horizontal stress would therefore be oriented towards north-east, roughly perpendicular to the planes of the faults.

From a synthesis of stress measurements, by “stress release technique”, geophysics and correlations, it is concluded that, below 100 m depth, the stress field may be assumed as quasi-homogeneous, with a vertical stress of about 12 to 14 MPa, and horizontal stresses ranging between 11 to 19 MPa. The vector of maximal stress is therefore mentioned to be dipping 10 to 15 degrees towards SSE, with an amplitude of 19 to 20 MPa (Ref. [3] and Ref. [4], § 1.1.5). No mention of the stresses up to 30 MPa like in Table 5.1 is made at this stage, neither as in the subsequent documentation we could review.

This does however not change the fact that the maximum horizontal stress was measured more or less parallel to the bedding. Considering the regional tectonics, this would confirm that the dam site is, from the point of view of the tectonics, a singular point, with the Illiak-Vakhsh Fault experiencing a sudden and sharp bend precisely here.

The reason for these complications in the stress field is not clear, and HPI noted that rapid variations in the stress field could occur, even within the same borehole (Ref. [24], § 2.3.3.). The evaporitic tectonics evidenced within the “disturbed zone” of the right bank (see Phase II Report - Volume 2 – Chapter 2 - Geology) may also be a partial explanation to this situation. Complementary stress measurements would be of interest in this regard, especially in the right bank, where no such measurements are reported to have been made.

For conditions of excavation of the power house and transformer cavern caverns, investigations performed have shown that the state of stress tends to be uniform in depth. Subsequently retained values in this location are a maximum magnitude of in-situ stress of 19 to 20 MPa, trending almost parallel to the bedding, with magnitudes from 15 to 18 MPa for the horizontal stress, and about 12 MPa for the vertical stress (Ref. [9], § 1.5, Ref. [10] and [11]).

# 6 GEOTECHNICAL CHARACTERISTICS OF ROCK MASSES AND RELEVANT DESIGN OPTIONS AS PER ORIGINAL PROJECT

## 6.1 Foreword

In this chapter, we will describe the approach made by HPT for establishing the Original Design of 1978, which prevailed for initiation of the construction. Included in this chapter are also data and results given by those of the subsequent documents issued from 1978 until the interruption of the construction in 1993 which have been made available to the Consortium.

No particular assessment will be made in this chapter, since the main analysis and comments about the different investigations performed by the different entities will be made further on, on the basis of the observations made by the Consortium (see paragraph 8).

The reason for the choice of this approach (rather than direct comparison between the different studies for each aspect) is that the 1978 Design Report and subsequent documents produced during the period of construction are the basis of all different studies carried afterwards, which most of the time used the data mentioned in those documents of the Original Project. This is largely justified by the fact that almost all investigations performed on site were made during this period.

Therefore, recalling first the basic data from those documents is dealt as the most suitable for good understanding of the analysis which follows.

Comments will be limited in this chapter to factual aspects of the analysis.

## 6.2 Main principles for definition of rock mass characteristics

HPT performed, for establishing the 1978 Project Report (Ref.[1]) a large number of tests on the different formations in the aim of assessing their hydraulic and mechanical properties. Other information was provided during the course of preparation and initiation of construction. Very few information is given about the methodology of these tests and their conditions of realisation, but they most probably all refer to Soviet Standards applicable by that time.

These tests have shown that the hydraulic and mechanical properties of the rocks highly depend on the type of rock, the degree of weathering and distressing, and correlatively; the degree of fracturation, (width, density and eventually infilling of the cracks).

The rock foundation is composed of sandstone for one part, and siltstone or claystone for the other part, from Jurassic Gaurdak Formation up to Mingbatman Formation. Apart from their difference in geomechanical characteristics, the main difference between siltstones (and claystones) and the sandstones is their notably different sensitivity to weathering. Differential erosion between siltstone levels and sandstone levels is easily noticed on dam site, with the more competent sandstone beds left protruding between the sifter siltstone levels. Figure 6.1 illustrates this difference.

HPT explains in Ref. [1] (§ 2.5.3.1) that, because of these inherent different manifestations of weathering and distressing between sandstones and siltstones, the latter having their strength sharply reduced towards surface, on the contrary to sandstones, another criterion than strength of the intact rock or opening of the fissures had to be adopted for characterising the amount of weathering and distressing of the rock masses.

In this respect, the results of water test in boreholes have been found by HPT to be the best indicator for the definition of geotechnical zones, i.e. zones where the rock masses can be assumed of similar geomechanical characteristics.

**Figure 6.1: Effect of differential erosion of mudstone and sandstones layers on the dam site (here cut by Fault 70, right bank of the river)**

## 6.3 Geotechnical zoning of rock masses

The concept of geotechnical zoning according to weathering and distressing is common in the projects designed according to Soviet, and then Russian standards.

As stated here above, the intake from water tests in boreholes was selected for the definition of four geotechnical zones for the site, with decreasing intake with depth. These four zones are the following:

- **Zone I of strong weathering and distressing**: this zone can be found in the near-surface zones of the rock masses and is characterized by strong weathering and distressing; sandstones are extensively fissured, whereas strength and shear properties of siltstones and claystones are considerably reduced,
- **Zone II of weathering and distressing**: in this zone, the cracks observed in the sandstones and siltstones have smaller openings than the one observed in zone I, whereas the rock strength does not practically change,
- **Zone III of distressed rock:** in this zone, located more deeper, the weathering effect is considered negligible, but effect of distressing is still noticeable,

- **Zone IV of intact rock (without weathering nor distressing effect):** in this zone, the rock mass is considered to be free of distressing effect from the slopes; the cracks are closed in siltstones and have a minimal width in the sandstones.; a sub-zone IVa is even distinguished where the rock massif is overstressed by the high in-situ stresses (see paragraph 6.5.1).

To be complete, one shall add the decompression zone located just above the top of the salt wedge located within the Ionakhsh Fault. commonly referred to as “caprock” in project documents, as already presented in 2.3 here above.

## 6.4 Justification of the geotechnical zoning as per Original Project

### *6.4.1 Procedure for water tests in boreholes*

According to Rogun HPP (Ref.[32]), the procedure for watertests aims at determining the specific intake discharge q of the rock mass in a test interval of a borehole, in litres per minute and unit length of borehole, under a pressure of 1 m water. Assumption is made that the intake discharge increases proportionally with the pressure within the test section, that is to say that q (in litre/min/m²) is given by the following relation:

$$q = \frac{Q}{Hl}$$

where Q (litre/min) is the measured discharge through a test section of length l (in meter), under the pressure H (in meter of water column).

The assumption on linearity of the relation discharge versus pressure is an approximation, since formally, changes in the flow regime through the fissures as pressure increases shall occur.

In Russian technical documentation, the specific absorption is therefore often given in litre/min, a discharge of 1 l/mn under 1 m water head being theoretically equal to 100 Lugeon unit.

Like the Lugeon test as defined originally, the specific intake of the rock mass is different from a true permeability test. One Lugeon unit is usually taken as equivalent to a hydraulic conductivity – or permeability -of $10^{-7}$ m/sec.

Similarly, the hydraulic conductivity K (in metre per day) of the rock mass is deduced from the specific water intake q by the following relationship:

$$K_{m/d} = 3q$$

with q in litre/min. Therefore, the hydraulic conductivity, measured in meter per second is:

$$K \approx 1.16.10^{-5} K_{m/d} \approx 3.47.10^{-5} q$$

Another coefficient than 3 may be used in some conditions, as was tentatively done for the calibration of the hydrogeological model (see Ref.[33], Table 2.1.5.).

### *6.4.2 Geotechnical zoning according to investigation results*

According to the Design Report of 1978 (Ref.[1], § 2.1), a total number of up to 557 water injection tests were performed in 92 boreholes, usually with 10 m length of tested interval, and more seldom, 5 m, under variable pressures, and up to 2 MPa. Farther in the same document, (Ref.[1]) § 2.3.2.1), a total of 964 water tests of every kind is mentioned, apparently for a larger area around the site.

It is worth to note that, in parallel, geophysical investigations in galleries and in boreholes provided, for the different rock masses, measurements of the in-situ velocity of longitudinal waves $V_P$.

Since elevation had to be taken into account in the assessment of the extension of each of the four geotechnical zones, the analysis was carried out dividing the rock foundation into four layers, namely,

- Below elevation 1010, that is to say, no more than 50 m above river bed, approximately,
- Between elevation 1010 and 1090,
- Between elevation 1100 and 1170,
- Between elevation 1170 and 1290.

In Figure 6.2 and Figure 6.3, the variations of the average values of specific water intake and longitudinal wave velocities obtained by the different tests are plotted in a same graph (values are those from Ref.[1], drawings 1174-03-F18 to 23).

Figure 6.2 shows that in the sandstones,

- Both longitudinal wave velocity and water intake have reached their extreme values (respectively some 4 km/s and less than 10 Lugeon units) between some 60 to 70 m distance from the surface,
- The thickness of zone I and zone II in the sandstones vary with elevation from a total thickness of about 35 m below elevation 1010, to about 60 to 65 m above elevation 1100.

In siltstones, Figure 6.3 shows that:

- Both longitudinal wave velocity and water intake have reached their extreme values (respectively 3 to 3.5 km/s and less than some 2 Lugeon units) at about 40 to 50 m distance from the surface,
- The thickness of zone I and zone II in the siltstones vary with elevation from a total thickness of about 25 m below elevation 1010, to some 40 m above elevation 1100.

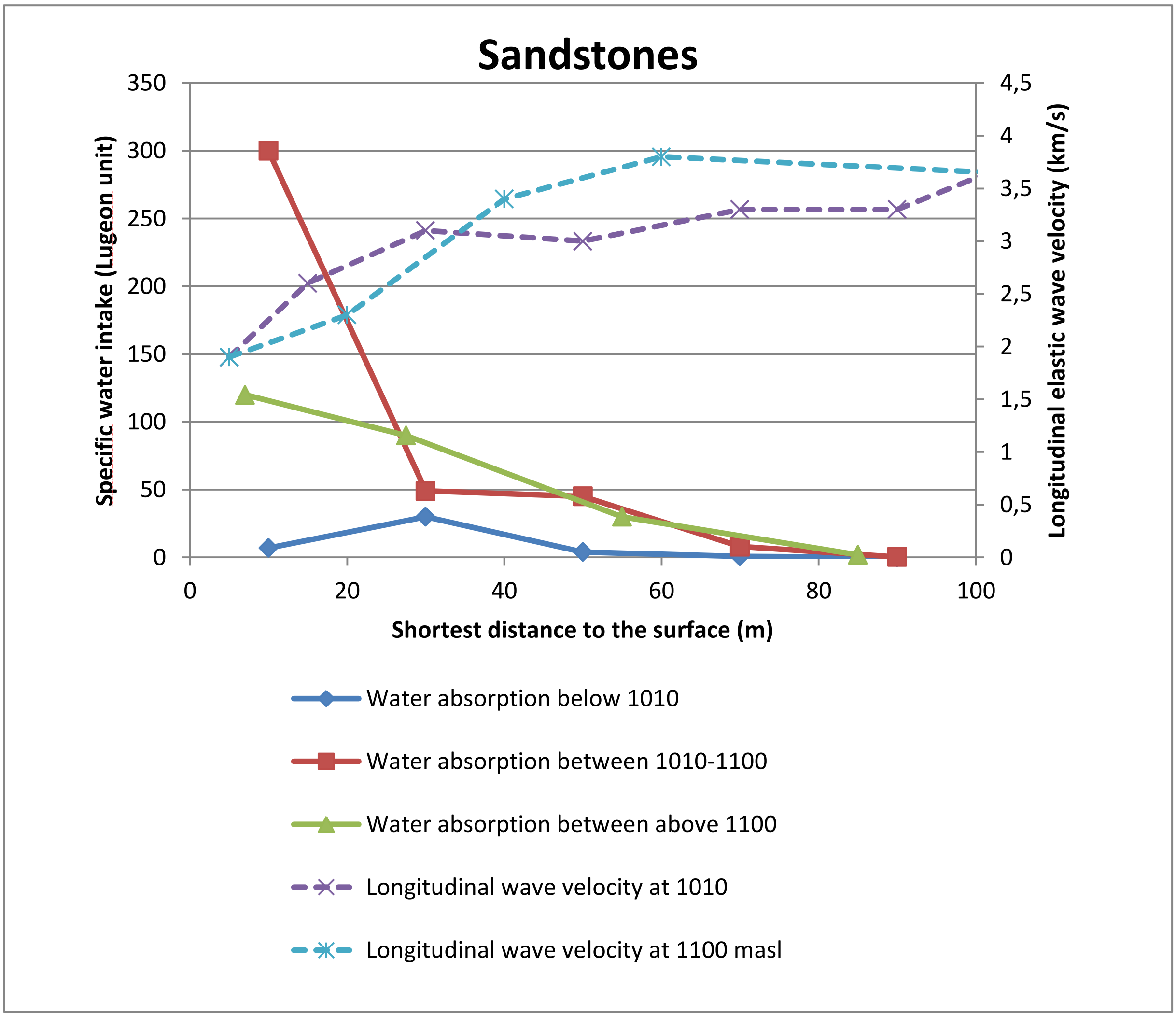



**Figure 6.2: Variation of the hydraulic and mechanical properties of the sandstones with the shortest distance to the surface (from values given in Ref.[1], drawings 1174-03-F18 to 23)**

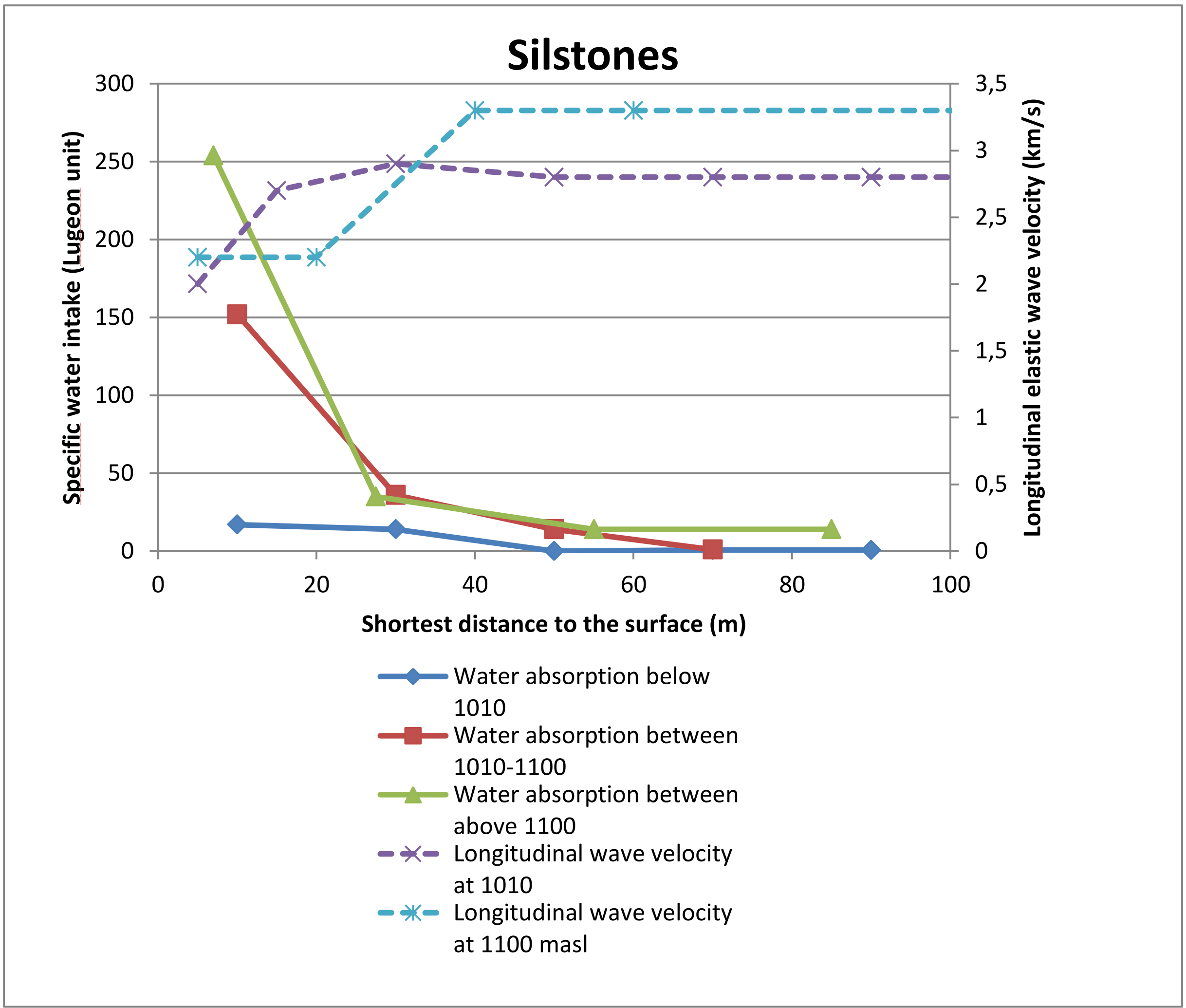



**Figure 6.3: Variation of the hydraulic and mechanical properties of the siltstones with the shortest distance to the surface (from values given in Ref.[1], drawings 1174-03-F18 to 23)**

These general trends in properties variations of sandstone are consistent with the expected larger hydraulic conductivity of sandstones with respect to siltstones. In a similar way, it is consistent with better geomechanical properties of the sandstones.

The difference in the thickness of the weathered / distressed zones is coherent with the fact that, due to the steepness of the valley walls, weathered and distressed zones have been eroded in the lower part.

The Design Report refers to drawing 1174-03-78 sheet 2, which provides the characteristic values of water intake and longitudinal wave velocity recommended for the different geotechnical zones, as well as the extreme values recorded. The corresponding part of this drawing is translated and reproduced in Table 6.1.

**Sandstones**

| Interval of elevations | Geotechnical zone (*) | Shortest distance between surface and lower limit of the zone (m) | Water absorption q (100 l/mn roughly Lugeon unit) | | | | | | | | | | Velocity of longitudinal waves Vp (km/s) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Number of tests | Characteristic values | | | | Highest values | | | | | Number of tests | Characteristic values | | | | Lowest values | | | | |
| | | | | From | To | Average | Standard deviation | Percentage of the tests | From | To | Average | Standard deviation ?? | | From | To | Average | Standard deviation | Percentage of the tests | From | To | Average | Standard deviation ?? |
| Below elevation 1010 | I | 0-10 | | | | 100 | | | | | | | 14 | 1,2 | 4 | 2,1 | 0,9 | | | | | |
| | II | 35 | 8 | 3,6 | 50 | 23 | 17 | | | | | | 47 | 1,8 | 4,8 | 3,1 | 0,8 | 9,6 | 0,9 | 1,6 | 1,4 | 0,3 |
| | II / 35 | | | | | | | | | | | | 7 | 0,6 | 2,5 | 1,7 | 0,5 | | | | | |
| | III | 60 | 5 | 0,8 | 9 | 4 | 3 | | | | | | 26 | 1,7 | 4,4 | 3,1 | 0,8 | 3,7 | | | 1,4 | |
| | III / IF | 50 | 12 | 0 | 1,3 | 0,4 | 5 | | | | | | | | | | | | | | | |
| | IV | 150 | 9 | 0 | 2 | 0,2 | 0,5 | | | | | | 14 | 2,7 | 4,8 | 3,6 | 0,6 | 12,5 | 2,1 | 2,2 | 2,2 | |
| | IVa | >100 | | 0 | 1,8 | 0,2 | 0,5 | 7 | 4 | 4 | 4 | 0 | 37 | 3,2 | 6,1 | 4,1 | 0,8 | 75 | 2,6 | 2,8 | 2,7 | |
| | IV / IF | >150 [1] | 18 | 0 | 0,7 | 0,2 | 0,2 | 40 | 1 | 5 | 2 | 1 | 8 | 2,2 | 3,8 | 3,3 | 0,6 | | | | | |
| | IVa / 35 | 500 | 9 | 0 | 0 | 0 | | | | | | | 3 | 3,6 | 3,6 | 3,6 | 0 | | | | | |
| Between elevations 1010 and 1100 | I | 13 | 3 | 60 | 480 | 250 | 210 | | | | | | 6 | 1 | 2,5 | 1,6 | 0,6 | | | | | |
| | I / 35 | | 4 | 11,5 | 47 | 24 | 15 | | | | | | | | | | | | | | | |
| | II | 45 | 21 | 1 | 80 | 30 | 25 | 8,6 | 120 | 160 | 140 | | 8 | 2 | 3,3 | 2,65 | 0,6 | 11 | | | 1,5 | |
| | II / 35 | | | 0 | 19,2 | 6 | 7 | | | | | | | | | | | | | | | |
| | III | 65 | 5 | 0,3 | 5 | 2 | 2 | | | | | | 6 | 2,6 | 4,8 | 3,6 | 0,8 | | | | | |
| | III / 35 | | | | | 1 | | | | | | | | | | | | | | | | |
| | IV | ≈ 180 | 7 | 0 | 1 | 0,3 | 0,4 | | | | | | | | | 3,8 | | | | | | |
| | IV / 35 | | | 0 | 1,5 | 0,3 | 0,5 | | | | | | | | | | | | | | | |
| Between elevations 1090 and 1170 | I | 30 | 61 | 0,6 | 500 | 120 | 110 | | | | | | | | | 2 | | | | | | |
| | II | 65 | 72 | 0 | 350 | 48 | 65 | 4 | 710 | 3080 | 1700 | 1230 | | | | 3 | | | | | | |
| | III | 100 | 8 | 0 | 16 | 9 | 17 | 11,1 | | | 25 | | | | | 3,5 | | | | | | |
| | IV | ≈ 190 | | | | 0,2 | | | | | | | | | | 3,8 | | | | | | |

**Siltstones**

| Interval of elevations | Geotechnical zone (*) | Shortest distance between surface and lower limit of the zone (m) | Water absorption q (100 l/mn roughly Lugeon unit) | | | | | | | | | | Velocity of longitudinal waves (km/s) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Number of tests | Characteristic values | | | | Highest values | | | | | Number of tests | Characteristic values | | | | Lower values | | | | |
| | | | | From | To | Average | Standard deviation | Percentage of the tests | From | To | Average | Standard deviation ?? | | From | To | Average | Standard deviation | Percentage of the tests | From | To | Average | Standard deviation ?? |
| Below elevation 1010 | I | 0 - 8 | | | | 100 | | | | | | | | 1,2 | 3,6 | 2,1 | 1 | | | | | |
| | II | 25 | | 0,9 | 65 | 12 | 20 | | | | | | 23 | 1,6 | 4 | 2,7 | 0,8 | 8 | 1,2 | 1,4 | 1,3 | |
| | III | 40 | 8 | 0 | 4 | 1 | 2 | 20 [5] | 70 | 80 | 75 | | 16 | 1,9 | 4,9 | 3,2 | 0,8 | 5,8 | | | 1,2 | |
| | IV | 15 | 21 | 0 | 2 | 0,1 | 0,4 | | | | | | | | | 3,8 | [2] | | | | | |
| | IVa | >150 [4] | 16 | 0 | 0,6 | 0,2 | 0,2 | [3] | | | | | 7 | 3 | 5,5 | 4,3 | 0,9 | [3] | | | | |
| Between elevations 1010 and 1100 | I | 10 | 8 | 20 | 380 | 150 | 120 | | | | | | 13 | 1 | 3,6 | 2,2 | 0,8 | | | | | |
| | II | 40 | 44 | 0 | 80 | 11 | 19 | 6,3 | 110 | 170 | 140 | 30 | 18 | 2,2 | 3,2 | 2,6 | 0,4 | 20 | 1,2 | 1,5 | 1,4 | |
| | III | 60 | 17 | 0 | 5,5 | 1 | 7,7 | | | | | | | | | 3,3 | | | | | | |
| | IV | ≈ 180 | 4 | | | 0,1 | 0 | | | | | | | | | 3,6 | | | | | | |
| Between elevations 1090 and 1170 | I | 20 | 7 | 50 | 460 | 210 | 150 | | | | | | | | | 2 | | | | | | |
| | II | 60 | 17 | 0 | 90 | 13 | 23 | 5,5 | | | 220 | | | | | 2,6 | | | | | | |
| | III | 80 | | | | 2 | | | | | | | | | | 3,6 | | | | | | |
| | IV | ≈ 190 | | | | 0,2 | | | | | | | | | | 4 | | | | | | |

*Notes*

**After drawing 1174-03-78, Sheet 2**

*(1) 150-220m depth measured normal to surface*
*(2) Gaurdak claystones: Vp=3.2km/s with standard deviation of 0,6km/s*
*(3) Values of q and Vp in zone IVa are measured down 380m depth*
*(4) The area of increase of in-situ stresses extends beween 150-220m depth (measured perpendiculary to the surface)*
*(5) It seems that the number of tests should be 2 rather than 20*
*(*)" / IF" after the zone number means that measures are made within the tectonic lens of Ionakhsh Fault*
*Similarly, "/ 35" after the zone number means that measures are made within the tectonic lens of Fault 35*

Table 6.1: Values of water intake and longitudinal wave velocity for the different geological zones, for sandstones and siltstones (after drawing 1174-03-78 Sheet 2)

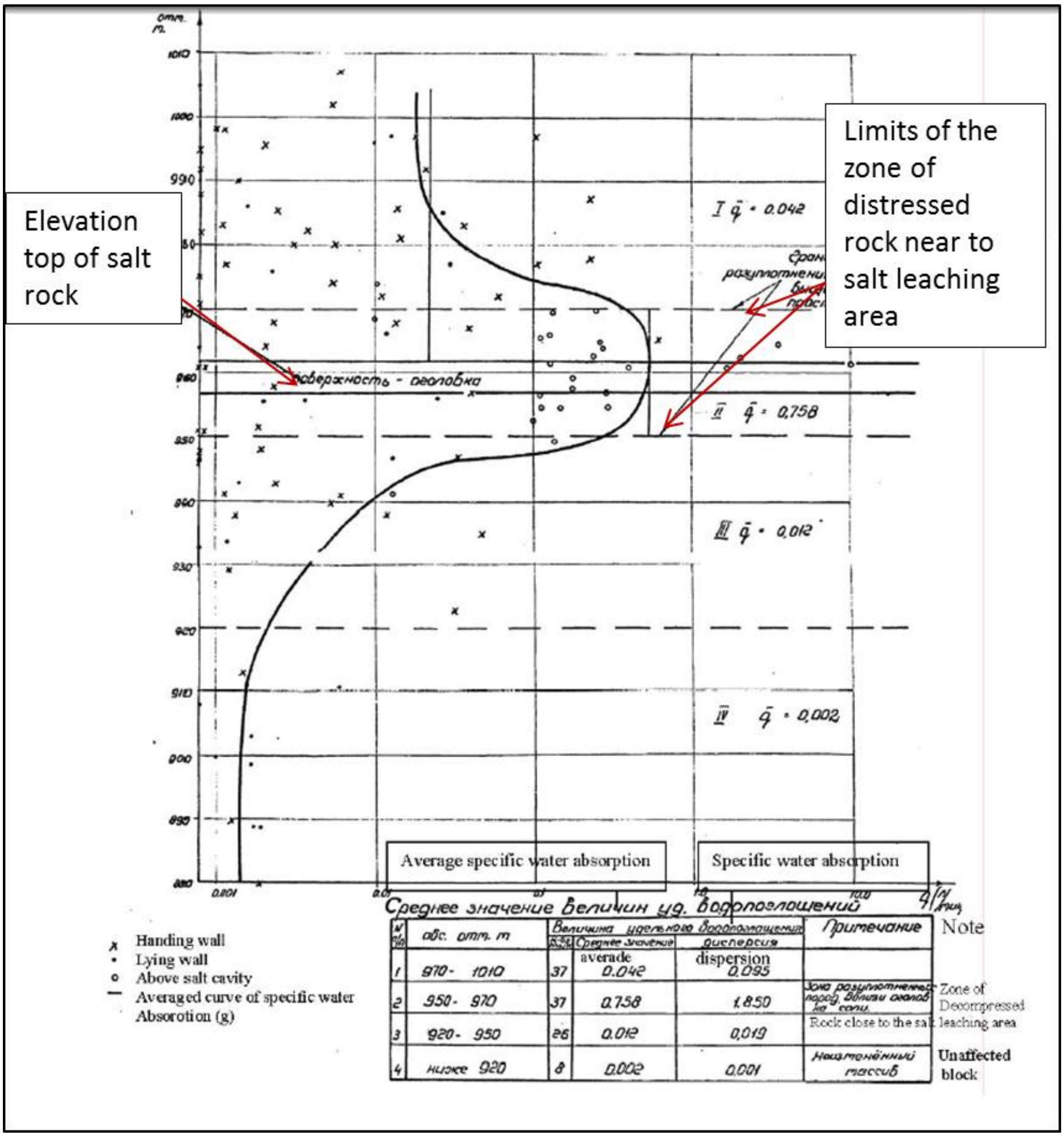



**Figure 6.4: Specific water intake in the salt head area; logarithm of water intake values x-axis, elevation in y-axis (reproduced from Ref.[1])**

Note that values of water intake may be more scattered than represented in Table 6.1. The Original Design Report mentions that low values of water intake do not exclude some widely open fracture, and mentions that in holes of the 2018 group, located in investigation gallery 1030, just downstream of the transformer cavern, several high intake values were encountered - up to 22 l/min, i.e. some 2,200 Lugeon units in borehole 2018g, although located in Zone IV, below elevation 1,010 (see Ref.[1], § 2.5.3.2).

The rock mass of the salt layer is a particular zone in which water intake, as well as the elastic and strength properties are of similar order of magnitude as the properties of the rock of zone I. Figure 6.4 shows the distribution of the water tests results near to the salt wedge of

Ionakhsh Fault, where a notable increase in water intake can be noticed (0.76 to 10.0 l. /min, hence 76 to 1000 Lugeon unit according to Ref.[1], § 2.5.3.2).

The rock formations near to the top of the salt wedge are therefore submitted to particular conditions due to salt leaching, and have their own characteristics and properties. One shall refer to the Phase 0 Report, specific for the analysis of the salt rock, for more details.

## 6.5 Geomechanical characteristics of the rock masses as per original design

### *6.5.1 Design values of the 1978 Project and period of construction*

The 1978 Design Report (Ref.[1]) mentions 160 numbers of samples taken, but the details of the whole investigation programme for assessment of rock properties is not given in the document.

Obviously, after initiation of the construction and as the Works were progressing, and as further geological data were made available, other complementary tests and investigations were performed. The detail is not available, but the conclusions of this extensive work are summarized in the Project documents.

This observation is supported by the fact that geotechnical characteristics of the rock masses for the different geotechnical zones, and especially taking into account the influence of the various stages of distressing and weathering (zones I to IV as defined in paragraph 6.3) are presented in the 1978 Design Project in the table of drawing 1174-03-78 Sheet 3.

However, another table, this time presented as dedicated to the underground works is presented in the drawing 1079-03-180 DP Sheet 4, dated 1993 (Ref. [5]). This drawing is probably an excerpt of a report by HPT, which could not be identified.

Table 6.2 presents a synthesis of the geomechanical parameters for intact rock samples, as per given in drawing 1174-03-78 Sheet 3, with International Unit System.

We therefore regrouped in same tables, for easy comparison, the information given by both drawing 1174-03-78 Sheet 3 and Ref. [5]. Table 6.3, Table 6.4, Table 6.5 and Table 6.6 show the data for zone IV, zone III, zone II and zone I respectively.

These four tables present general properties extracted from drawing 1174-03-78 Sheet 3, which are essentially permeability (we converted the original litre/minute unit to Lugeon unit), velocity of longitudinal waves and deformation properties of the rock mass. The rock mass coefficient f is the Protodyakonov strength factor, used for lining calculations of underground works, according to Ref. [16], § 2.1.7.).

On the right hand of the tables, geomechanical properties assessed for underground works (rock in natural conditions) as per Ref.[25], with essentially wave velocities, deformation properties and shear characteristics of the rock.

The only noticeable difference between 1978 data and those of 1993 for underground works can be spotted in Table 6.3 for the Lower Obigarm formation, made at 98% of siltstone and claystone, where the velocity of longitudinal waves for general properties where assessed to 3,800 m/s in 1978, and 4,500 m/s in 1993 for underground works, most probably by taking into account further measurements made in depth around the underground works (e.g. Ref. [12]).

This is actually probably due to the fact that measurement of longitudinal waves in gallery 1030 evidenced a zone of globally higher values of longitudinal wave velocities in depth. This led to the definition of a geotechnical zone IVa “overstressed rock mass” more or less corresponding to the Lower Obigarm siltstones. The profile of longitudinal wave velocities and stress measurements performed in this gallery shows that most of the measured values range between 3.5 to more than 5.0 km/sec, especially within Lower Obigarm. This effectively tends to demonstrate that those siltstones are under high compressive strength, and according to HPT, justifies the definition of the geotechnical zone IVa (see Table 5.1).

In addition to the geotechnical parameters, an average value of the depth of the lower limit of each of the geotechnical zones (actually shortest distance to the surface), depending on the elevation, is presented in the tables related to zone I, II and III, thereby providing a geometrical limit to the extension of the weathering and distressing inside the slopes.

| Geological Age | | | | | | | | | Uniaxial compressive strength | | | Uniaxial Tensile Strength | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Period | Section | Stage | Formation | | Thickness | Lithology Type of rock | Contents | Density | Dry | Saturated | Softening coefficient | | Intake from water tests | Static modulus of deformation $E_d$ |
| | | | | Symbol | [m] | | [%] | [kN/m³] | [MPa] | [MPa] | | [MPa] | [%] | [GPa] |
| Jurassic | Upper | | Gaudark | J3Gr | 21.7 | Siltstone | 100 | 24.4 | 16 | 6.2 | 0.6 | 0 | 4.2 | 13 |
| | | | | | 400 | Salt rock | --- | 22.1 | 31 | 23 | 0.75 | 2 | 0.6 | 13 |
| Cretaceous | Lower | Valanginian | Lower Yavan | K1Jv1 | 47.5 | Siltstone | 100 | 27 | 101 | 37 | 0.31 | --- | 1.1 | 21 |
| | | | Upper Yavan | K1Jv2 | 54 | Sandstone | 13 | 26.7 | 100 | 86 | 0.77 | 7.5 | 0.9 | 33 |
| | | | | | | Siltstone | 87 | 27.3 | 68 | 42 | 0.57 | --- | 1.1 | 22 |
| | | | Kyzyltash | K1Kz | 198.6 | Sandstone | 87.3 | 26.2 | 126 | 102 | 0.8 | 10.6 | 0.7 | 37 |
| | | | | | | Siltstone | 12.7 | 26.2 | 118 | 117 | 0.9 | --- | 0.65 | 31 |
| | | Hauterivian | Lower Obigarm | K1Ob1 | 93.5 | Siltstone | 99.5 | 27.1 | 59 | 57 | 0.66 | 5.5 | 0.8 | 26 |
| | | | | | | Gypsum | 0.5 | --- | --- | --- | --- | --- | --- | --- |
| | | | Upper Obigarm | K1Ob2 | 235.4 | Sandstone | 94.6 | 26 | 120 | 111 | 0.96 | 10 | 1.2 | 39 |
| | | | | | | Siltstone | 5.4 | 26.5 | 108 | 97 | 0.9 | 6 | 0.7 | 29 |
| | | Albian | Karakuz | K1Kr | 102.6 | Sandstone | 71.4 | 26 | 106 | 66 | 0.66 | 10 | 0.9 | 28 |
| | | | | | | Siltstone | 28 | 26 | 72 | 53 | 0.73 | --- | 1.1 | 25 |
| | | | | | | Gypsum | 0.6 | --- | --- | --- | --- | --- | --- | --- |
| | | | Mingbatman | K1Mg1~5 | 350.5 | Sandstone | 77.9 | 25.8 | 121 | 83 | 0.67 | 7.5 | 1.3 | 34 |
| | | | | | | Siltstone | 22.1 | 26.2 | 110 | 82 | 0.75 | --- | 0.8 | 30 |
| Tectonic lenses of Fault 35 and Ionakhsh | | | | | | | | 26.2 | 20~80 | 10~70 | 0.3~0.7 | 2.0~5.0 | 0.8~4.50 | 10~30 |

**Table 6.2: Summary of main geotechnical characteristics of intact rock samples from the main geological formations of the dam site (reproduced from Ref.[1], excerpt of drawing 1174-03-78 Sheet 3)**

| Zone IV- Intact rock mass | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | General properties | | | | | | | Geomechanical properties of rock for underground works | | | | | | | | | | | | | | |
| Geological Age | | | | Joint Characteristics | | | | Geotechnical parameters | | | Velocity of elastic waves | | | | Geotechnical properties | | | Shear parameters | | | | | | | |
| Period | Formation | | Lithology Type of rock | Opening n | Spacing b | Long. wave velocity Vp | Permeability q | Deform. modulus Ed | Ko | f Rock mass coefficient | Vp Natural moisture content | Vp Saturated rock | Vs Natural moisture content | Vs Saturated rock | Elasticity modulus Ee | Deform. modulus Ed | Poisson's Ratio v | Displacement through rock mass | | Displacement along fissures | | Shear parameters of joint infilling | | Shear parameters of rock surfaces along faults | |
| | | Symbol | | [%] | [cm] | [km/s] | [Lugeon unit] | [GPa] | [MPa/m] | | [km/s] | [km/s] | [km/s] | [km/s] | Gpa | Gpa | | φ [°] | C [Mpa] | φ [°] | C [Mpa] | φ [°] | C [Mpa] | φ [°] | C [Mpa] |
| Jurassic | Gaudark | J3Gr | Siltstone with gypsum | 4,5 | 8,7 | 3,2 | 0,2 | 3 | 2000 | 3 | 3,2 | 4,2 | 1,75 | 1,75 | 21 | 3 | 0,28 | 56 | 0,5 | 40 | 0,2 | 26 | 0,02 | 28 | 0,02 |
| | | | Salt rock | 0 | --- | 3,6 | 0 | 10 | 2000 | 6 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Cretaceous | Lower Javan | K1Jv1 | Siltstone | 0,3 | 16 | 3,2 | 0,1 | 4 | 3000 | 3 | 3,3 | 4,2 | 1,8 | 1,8 | 23 | 4 | 0,28 | 63 | 1,5 | 45 | 0,3 | 26 | 0,02 | 29 | 0,03 |
| | Upper Javan | K1Jv2 | Siltstone Sandstone | 0,3 | 16 | 3,4 | 0,1 | 5 | 4000 | 4 | | | | | | | | | | | | | | | |
| | Kyzyltash | K1Kz | Sandstone Siltstone | 0,5 | 25 | 3,7 | 0,2 | 8 | 6000 | 8 | 3,7 | 4,5 | 1,95 | 1,95 | 27 | 8 | 0,31 | 67 | 2 | 45 | 0,3 | 26 | 0,02 | 33 | 0,04 |
| | Lower Obigarm | K1Ob1 | Siltstone 98% | 0,4 | 18 | 3,8 | 0,1 | 5,5 | 5500 | 5 | 4,5 | 4,75 | 2,25 | 2,25 | 36 | 5,5 | 0,33 | 63 | 1,5 | 45 | 0,3 | 26 | 0,02 | 29 | 0,03 |
| | Upper Obigarm | K1Ob2 | Sandstone 96% | 0,5 | 25 | 3,6 | 0,3 | 8,5 | 7000 | 8 | 3,8 | 4,5 | 2 | 2 | 30 | 9 | 0,3 | 67 | 2 | 45 | 0,3 | 26 | 0,02 | 32-35 | 0,03 |
| | Karakuz | K1Kr | Sandstone Siltstone | 0,4 | 18 | 3,6 | 0,2 | 7 | 6000 | 6 | 3,6 | 4,4 | 1,9 | 1,9 | 24 | 7 | 0,32 | 63 | 2 | 45 | 0,3 | 26 | 0,02 | 30 | 0,03 |
| | Mingbatman | K1Mg1~5 | Sandstone Siltstone | 0,5 | 20 | 3,6 | 0,3 | 8 | 6000 | 7 | 3,8 | 4,6 | 1,98 | 1,98 | 30 | 8 | 0,32 | 67 | 2 | 45 | 0,3 | 26 | 0,02 | 32 | 0,02 |
| Rock properties of tectonic lenses of Fault 35 and Ionakhsh Fault | | | | 3,2 | 8 | 3,3 | 0,2 | 2 | 1500 | 1.5~2 | 2,6 | 3,8 | 1,53 | 1,53 | 15 | 2 | 0,23 | 56 | 0,5 | 35 | 0,1 | 26 | 0,02 | 30 | 0,03 |

***Notes***

*See Table 2.1for detailed lithological composition and thicknesses*

*Reproduced from drawing 1174-03-78, Sheet 3 for general properties, and from 1079-03-180 DP, Sheet 4 for properties for underground works*

**Table 6.3: Summary of geomechanical characteristics of intact rock masses (zone IV), as per Ref.[1] (drawing 1174-03-78 Sheet 2) for general properties and Ref. [5] for remaining data**

## Zone III- Distressed rock mass

| | | | | General properties | | | | | | | | | | | Geomechanical properties of rock for underground works | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Geological Age | | | | | | | | Joint Characteristics | | | | Geotechnical parameters | | | Velocity of elastic waves | | | | Geotechnical properties | | | Shear parameters | | | | | | | |
| Period | Formation | | Lithology Type of rock | Location of lower limit of weathering zone (perpendicular to rock surface) | | | | Opening n | Spacing b | Vp | Permeability q | Deform. modulus Ed | Ko | f Rock mass coefficient | Vp Natural moisture content | Vp Saturated rock | Vs Natural moisture content | Vs Saturated rock | Elasticity modulus Ee | Deform. modulus Ed | Poisson's Ratio v | Displacement through rock mass | | Displacement along fissures | | Shear parameters of joint filling | | Shear parameters of rock surfaces along faults | |
| | | Symbol | | Below el. 1010 | el. 1010-1090 | el. 1090-1170 | el. 1170-1290 | [%] | [cm] | [km/s] | [Lugeon unit] | [GPa] | [MPa/m] | | [km/s] | [km/s] | [km/s] | [km/s] | Gpa | Gpa | | φ [°] | C [Mpa] | φ [°] | C [Mpa] | φ [°] | C [Mpa] | φ [°] | C [Mpa] |
| **Jurassic** | Gaudark | **J3Gr** | Siltstone with gypsum | 30 | 50 | 60 | 80 | 4,7 | 8,5 | 2,5 | 1 | 2,5 | 2000 | 3 | | | | | | | | | | | | | | | |
| | | | Salt rock | | | | | | | | | | | | | | | | | | | | | | | | | | |
| **Cretaceous** | Lower Javan | **K1Jv1** | Siltstone | 40 | 60 | 70 | 100 | 0,3 | 15 | 3 | 1 | 3,7 | 3000 | 3 | 3,1 | 4,1 | 1,7 | 1,7 | 21 | 3 | 0,28 | 63 | 1 | 45 | 0,2 | 26 | 0,02 | 29 | 0,03 |
| | Upper Javan | **K1Jv2** | Siltstone Sandstone | 40 | 60 | 70 | 100 | 0,4 | 15 | 3,2 | 2 | 4,8 | 4000 | 4 | | | | | | | | | | | | | | | |
| | Kyzyltash | **K1Kz** | Sandstone Siltstone | 60 | 65 | 100 | 130 | 0,4 | 20 | 3,3 | 6 | 7,5 | 6000 | 8 | 3,3 | 4,2 | 1,8 | 1,8 | 20 | 5,7 | 0,28 | 67 | 0,5 | 45 | 0,2 | 26 | 0,02 | 33 | 0,04 |
| | Lower Obigarm | **K1Ob1** | Siltstone 98% | 40 | 60 | 80 | 100 | 0,4 | 16 | 3,2 | 2 | 5 | 4000 | 5 | | | | | | | | | | | | | | | |
| | Upper Obigarm | **K1Ob2** | Sandstone 96% | 60 | 65 | 100 | 140 | 0,4 | 18 | 3,3 | 8 | 8 | 6000 | 8 | 3,3 | 4,1 | 1,8 | 1,8 | 23,5 | 6 | 0,29 | 67 | 1,5 | 45 | 0,3 | 26 | 0,02 | 32 - 35 | 0,03 |
| | Karakuz | **K1Kr** | Sandstone Siltstone | 50 | 60 | 90 | 120 | 0,4 | 16 | 3,2 | 4 | 6,5 | 5800 | 6 | 3,2 | 4,2 | 1,8 | 1,8 | 20,5 | 3,5 | 0,31 | 63 | 1,5 | 45 | 0,3 | 26 | 0,02 | 30 | 0,03 |
| | Mingbatman | **K1Mg1~5** | Sandstone Siltstone | 60 | 65 | 100 | 130 | 0,4 | 20 | 3,3 | 6 | 7,8 | 6000 | 7 | 3,3 | 4,2 | 1,8 | 1,8 | 20 | 6 | 0,28 | 67 | 1,5 | 45 | 0,3 | 26 | 0,02 | 32 | 0,02 |
| Rock properties of tectonic lenses of Fault 35 and Ionakhsh Fault | | | | 30 | 50 | 60 | 80 | 35 | 7 | 2,5 | 3 | 2 | 1500 | 1.5~2.0 | | | | | | | | | | | | | | | |

***Notes***

*See Table 2.1for detailed lithological composition and thicknesses*

*Reproduced from drawing 1174-03-78, Sheet 3 for general properties, and from 1079-03-180 DP, Sheet 4 for properties for underground works*

**Table 6.4: Summary of estimated depth and geomechanical characteristics of distressed rock masses (zone III), as per Ref.[1] (drawing 1174-03-78 Sheet 2) for general properties and Ref. [5] for remaining data**

## Zone II- Weathered and distressed rock mass

| | | | | **General properties** | | | | | | | | | | | **Geomechanical properties of rock for underground works** | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Geological Age | | | | | | | | Joint Characteristics | | | | Geotechnical parameters | | | Velocity of elastic waves | | | | Geotechnical properties | | | Shear parameters | | | | | | | |
| Period | Formation | | Lithology Type of rock | Location of lower limit of weathering zone (perpendicular to rock surface) | | | | Opening n | Spacing b | Vp | Permeability q | Deform. modulus Ed | Ko | f Rock mass coeffcient | Vp Natural moisture content | Vp Saturated rock | Vs Natural moisture content | Vs Saturated rock | Elasticity modulus Ee | Deform. modulus Ed | Poisson's Ratio v | Displacement through rock mass | | Displacement along fissures | | Shear parameters of joint filling | | Shear parameters of rock surfaces along faults | |
| | | Symbol | | Below el. 1010 | el. 1010-1090 | el. 1090-1170 | el. 1170-1290 | [%] | [cm] | [km/s] | [Lugeon unit] | [GPa] | [MPa/m] | | [km/s] | [km/s] | [km/s] | [km/s] | Gpa | Gpa | | φ [°] | C [Mpa] | φ [°] | C [Mpa] | φ [°] | C [Mpa] | φ [°] | C [Mpa] |
| **Jurassic** | Gaudark | **J3gr** | Siltstone with gypsum | 20 | 30 | 55 | 70 | 5 | 8 | 1,8 | 5 | 1,5 | 1200 | 2 | | | | | | | | | | | | | | | |
| | | | Salt rock | | | | | | | | | | | | | | | | | | | | | | | | | | |
| **Cretaceous** | Lower Javan | **K1jv1** | Siltstone | 20 | 30 | 55 | 70 | 0,3 | 14 | 2,4 | 15 | 3 | 2300 | 4~5 | 2,4 | 2,9 | 1,45 | 1,45 | 12 | 2,8 | 0,22 | 63 | 0,5 | 45 | 0,2 | 26 | 0,02 | 29 | 0,03 |
| | Upper Javan | **Kjv2** | Siltstone Sandstone | 20 | 30 | 55 | 70 | 0,4 | 14 | 2,4 | 15 | 3 | 2300 | 4~5 | | | | | | | | | | | | | | | |
| | Kyzyltash | **K1kz** | Sandstone Siltstone | 30 | 45 | 65 | 75 | 0,5 | 18 | 2,9 | 30 | 4,5 | 3500 | 5 | 2,9 | 4 | 1,64 | 1,64 | 17,5 | 4 | 0,27 | 63 | 1,5 | 45 | 0,3 | 26 | 0,02 | 33 | 0,04 |
| | Lower Obigarm | **K1ob1** | Siltstone 98% | 25 | 40 | 60 | 70 | 0,4 | 15 | 2,6 | 12 | 3,2 | 2500 | 4 | | | | | | | | | | | | | | | |
| | Upper Obigarm | **K1ob2** | Sandstone 96% | 35 | 45 | 65 | 80 | 0,5 | 20 | 2,9 | 30 | 4,5 | 3500 | 4~5 | 3,1 | 4,1 | 1,7 | 1,7 | 20 | 5 | 0,28 | 53 | 1,5 | 45 | 0,3 | 26 | 0,02 | 32 - 35 | 0,03 |
| | Karakuz | **K1kr** | Sandstone Siltstone | 35 | 45 | 65 | 80 | 0,5 | 16 | 2,8 | 20 | 4 | 2700 | 4~5 | 2,8 | 4 | 1,7 | 1,7 | 17 | 4 | 0,24 | 56 | 1,5 | 40 | 0,3 | 26 | 0,02 | 30 | 0,03 |
| | Mingbatman | **K1mg1~5** | Sandstone Siltstone | 35 | 45 | 65 | 80 | 0,5 | 20 | 2,9 | 30 | 4,5 | 3500 | 4~5 | 2,9 | 4 | 1,64 | 1,64 | 1,75 | 4 | 0,27 | 63 | 1,5 | 45 | 0,3 | 26 | 0,02 | 32 | 0,02 |
| Rock properties of tectonic lenses of Fault 35 and Ionakhsh Fault | | | | 15 | 20 | 30 | 40 | 3,5 | 7 | 1,8 | 6 | 1,5 | 1200 | 1.0~1.5 | | | | | | | | | | | | | | | |

***Notes***

*See Table 2.1for detailed lithological composition and thicknesses*
*Reproduced from drawing  1174-03-78, Sheet 3 for general properties, and from 1079-03-180 DP, Sheet 4 for properties for underground works*

**Table 6.5: Summary of estimated depth and geomechanical characteristics of weathered and distressed rock masses (zone II), as per Ref.[1] (drawing 1174-03-78 Sheet 2) for general properties and Ref. [5] for remaining data**

## Zone I- Strongly weathered and distressed rock mass

| | | | | General properties | | | | | | | | | | | Geomechanical properties of rock for underground works | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Geological Age | | | | | | | | Joint Characteristics | | | | Geotechnical parameters | | | Velocity of elastic waves | | | | Geotechnical properties | | | Shear parameters | | | | | | | |
| Period | Formation | | Lithology Type of rock | Location of lower limit of weathering zone (perpendicular to rock surface) | | | | Opening n | Spacing b | Vp | Permeability q | Deform. modulus Ed | Ko | f Rock mass coeffcient | Vp Natural moisture content | Vp Saturated rock | Vs Natural moisture content | Vs Saturated rock | Elasticity modulus Ee | Deform. modulus Ed | Poisson's Ratio v | Displacement through rock mass | | Displacement along fissures | | Shear parameters of joint filling | | Shear parameters of rock surfaces along faults | |
| | | Symbol | | Below el. 1010 | el. 1010-1090 | el. 1090-1170 | el. 1170-1290 | [%] | [cm] | [km/s] | [Lugeon unit] | [GPa] | [MPa/m] | | [km/s] | [km/s] | [km/s] | [km/s] | Gpa | Gpa | | φ [°] | C [Mpa] | φ [°] | C [Mpa] | φ [°] | C [Mpa] | φ [°] | C [Mpa] |
| **Jurassic** | Gaudark | **J3gr** | Siltstone with gypsum | 5 | 10 | 15 | 20 | 5 | 7 | 1,7 | 50 | 1,5 | 1200 | 1 | | | | | | | | | | | | | | | |
| | | | Salt rock | | | | | | | | | | | | | | | | | | | | | | | | | | |
| **Cretaceous** | Lower Javan | **K1jv1** | Siltstone | 5 | 10 | 15 | 20 | 1 | 10 | 1,8 | 180 | 1,5 | 1200 | 3 | | | | | | | | | | | | | | | |
| | Upper Javan | **Kjv2** | Siltstone Sandstone | 5 | 10 | 15 | 20 | 1 | 10 | 1,8 | 180 | 1,5 | 1200 | 3 | | | | | | | | | | | | | | | |
| | Kyzyltash | **K1kz** | Sandstone Siltstone | 7 | 15 | 25 | 35 | 2 | 18 | 2 | 220 | 2 | 1500 | 2~3 | 2,2 | 3,6 | 1,37 | 1,37 | 11 | 2,2 | 0,22 | 56 | 0,5 | 40 | 0,2 | 26 | 0,02 | 33 | 0,04 |
| | Lower Obigarm | **K1ob1** | Siltstone 98% | 6 | 10 | 20 | 30 | 0,9 | 14 | 2,1 | 180 | 1,5 | 1200 | 3 | | | | | | | | | | | | | | | |
| | Upper Obigarm | **K1ob2** | Sandstone 96% | 8 | 13 | 30 | 40 | 2,2 | 18 | 2 | 220 | 2,5 | 1900 | 2~3 | 2,2 | 3,3 | 1,37 | 1,37 | 11 | 2,4 | 0,18 | 56 | 0,5 | 40 | 0,2 | 26 | 0,02 | 32 - 35 | 0,03 |
| | Karakuz | **K1kr** | Sandstone Siltstone | 7 | 15 | 25 | 35 | 1 | 14 | 1,9 | 200 | 2 | 1500 | 2~3 | 1,9 | 3,1 | 1,25 | 1,25 | 7,8 | 1,7 | 0,2 | 56 | 0,5 | 40 | 0,2 | 26 | 0,02 | 30 | 0,03 |
| | Mingbatman | **K1mg1~5** | Sandstone Siltstone | 7 | 15 | 25 | 35 | 1,8 | 15 | 2 | 220 | 2,5 | 1900 | 2~3 | 2,2 | 3,6 | 1,37 | 1,37 | 11 | 2,2 | 0,22 | 56 | 0,5 | 40 | 0,2 | 26 | 0,02 | 32 | 0,02 |
| Rock properties of tectonic lenses of Fault 35 and Ionakhsh Fault | | | | 5 | 8 | 12 | 15 | 4,5 | 5 | 1,6 | 20 | 1,2 | 1000 | 0,5 | | | | | | | | | | | | | | | |

***Notes***

*See Table 2.1for detailed lithological composition and thicknesses*
*Reproduced from drawing 1174-03-78, Sheet 3 for general properties, and from 1079-03-180 DP, Sheet 4 for properties for underground works*

**Table 6.6: Summary of estimated depth and geomechanical characteristics of strongly weathered and distressed rock masses (zone I), as per Ref.[1] (drawing 1174-03-78 Sheet 2) for general properties and Ref [5] for remaining data**

### *6.5.2 Commentaries of HPT about geotechnical properties*

HPT describes in Ref.[1], § 2.5.2, the geomechanical features of the different rocks. The main conclusions are listed here below.

The strongest rocks are the sandstones of Kyzyltash and Upper Obigarm formations, with average uniaxial compressive strengths in saturated state ranging between 102 and 110 MPa, but obtained values can reach up to 160 MPa. For these sandstones, described as fine-grained with strong carbonaceous cement, triaxial tests gave values of cohesion of 24 to 26 MPa and friction angles from 55 to 59 degrees, while the static deformation modulus of the intact rock, measured on samples, were reported in the range of 37 to 39 GPa, and modulus of elasticity in the range of 20.5 to 44 GPa (Ref.[1], § 2.5.2).

The sandstones of the other formations (Yavan, Karakuz, Mingbatman) were found to have similar strength on dry samples, but notably lower in saturated state (66 to 83 MPa in saturated state). The modulus of deformation of those sandstones is reported to be 23.8 to 33 GPa.

With regard to siltstones and indurated claystones, the siltstones of the Kyzyltash Formation were found to provide rather high values, since they are assigned as design value up to 117 MPa, and the softening coefficient (ratio of dry to saturated compression strength) found to be only 0.9, values of nearly same order of magnitude than the one found for sandstones. Value of deformation modulus for the siltstones of the Kyzyltash Formation (intact rock) is reported to be 31 GPa.

Siltstones and indurated claystones constitute more than 99% of the rock mass of the Lower Obigarm Formation. They are of intermediate strength, with design value of the uniaxial compressive strength of saturated samples of 57 MPa, and softening coefficient of 0.66. The design value of deformation modulus of intact rock is 26 GPa.

Siltstones of the Lower Yavan Formation provides the lowest uniaxial compressive strength (37 MPa in saturated state), with lowest softening coefficient, found to be 0.31 (see Table 6.2). Values of deformation modulus measured on rock samples are 21 to 22 GPa.

The design report mentions that siltstones and claystones, with argillo-carbonaceous cement, may rapidly loose strength when extracted from depth and brought to the surface, and turn in clay and gravel in some hours, as for the one of Gaurdak Formation, which measured compressive strength on samples in saturated state is 6.2 MPa as design value, softening coefficient being 0.6.

With regard to the tectonic lenses of Fault 35 and Ionakhsh Fault, results are quite inhomogeneous, with uniaxial compressive strengths in saturated state from 10 to 86 MPa, and softening coefficients ranging between 0.3 and 0.8.

The salt rock of Gaurdak Formation, identified within Ionakhsh Fault, could never been observed in-situ, but only on cores. Its composition is described in 2.3 and discussed in the Phase 0 Report.

The volumetric weight of salt rock was found to range between 19.7 and 23.3 kN/m³, giving an average value of 21.8 kN/m³. Its uniaxial compressive strength under salt-saturated water is reported to be 23 MPa in average, for 11.4 GPa and 26 GPa for deformation modulus and elasticity modulus respectively. Aspect of the rock is massive, without cracks, and it can be considered as watertight.

These characteristics are discussed farther in paragraph 8.4.

## 6.6 Structural geology and joint characteristics

### *6.6.1 Description of the main faults of the dam site*

#### *6.6.1.1 Ionakhsh and Gulizindan Fault*

The general tectonic frame of the dam site has been briefly presented in paragraph 2.1 (more details are available in Phase II Report - Volume 2 – Chapter 2 - Geology) with the main Ionakhsh Fault and Fault 35, proved by geodetic measurements to have a creeping movement of the order of 0.5 to 2 mm/year, with apparently extrusion of the tectonic lenses associated with them.

Numerous other faults cross the dam site in an attitude similar to Fault 35, but with largely variable dip towards NNE.

In the Design report of 1978, the Ionakhsh Fault is presented as a thrust fault, with, on the south-eastern side, the general monocline of the Jurassic-Cretaceous series overthrusting the rocks of the Kirbich syncline, a steep dissymmetric rock fold within Upper-Cretaceous formations, which axis plunges towards north-east.

The SE limb of the fault is rather well-traceable, with a 1 to 1.5 m thick seam of brown-reddish clay containing small inclusions of claystones (1 to 2 mm in size).

It is in contact with a tectonic lens, which has been recognised by the investigation gallery 1001a and connected branches; the thickness of this lens seems variable, and vary between 80 m (vault of gallery 1001a) to 28 m, and seems to decrease towards the interior of the right bank. On the left bank, the thickness of the tectonic lens does not exceed some meters. It is composed by crushed rocks of Upper Cretaceous, dragged by the movement of Ionakhsh Fault; which dip angle aligns with the one of the fault. The tectonic lens is presented as crushed at interval of 100 to 150 mm with amplitude of displacement from some centimetres to some metres.

Contact of the tectonic lens with the Kirbich syncline rocks is described as very irregular, sometime with inverted bedding. Thickness of this contact varies depending upon nature of the rock, from around 1 cm in claystones to some 30 cm in limestone.

The Gulizindan Fault is not described in detail, since outside the strict frame of the dam site, but it is very likely to have similar features.

#### *6.6.1.2 Other faults cutting the monocline downstream of Ionakhsh Fault*

According to the 1978 Design Report (Ref.[1], § 2.2.3), other major faults cutting the monocline, although with various extension, amplitudes of displacement and thickness of crushed zones, almost all have the same kinematics, origin and period of generation.

The majority of them are antithetic faults generated by the movements of Ionakhsh and Gulizindan faults. Their attitude is in the range of 310-350/20-50.

Fault 35 is the biggest of these faults and the most conspicuous on the dam site, and was investigated by galleries and boreholes. With an attitude of 330-340/45, it has been traced down to 360 m depth below the river, approaching the salt wedge of Ionakhsh Fault.

It is presented with complex characteristics, made, on the dam site, of two seams of different amplitudes of displacement limiting a tectonic lens of up to 60-70 m thickness. According to Ref.[1], § 2.2.3, the main seam in right bank is the south-eastern one, with the major displacement (80% of the total estimated 120 m on this fault), while it is reversed in the left bank, where the main seam is the north-western one.

The seams of the Fault 35 are described as infilled with no more than 15 to 20 cm clay, while within the tectonic lens between the two seams, the rock is highly crushed by a great number of fractures.

Apart from Fault 35, faults pertaining to this group of similar attitude in direction and dip angle are reported to have an off-set limited to no more than 1 m, with amplitude of dislocation reducing when moving away from Ionakhsh Fault. Their extent is higher than 200 m, and spacing between them 40 to 60 m (Ref.[1], § 2.6.1.). Only Fault 70 has major amplitude of displacement of 15 m.

All these fractures are reported infilled with clay and breccia, which thickness however does not exceed some centimetres.

Of more peculiar attitude are Fault 28 and Fault 367. Fault 28 (310/80) is different as it is a subvertical thrust fault in the downstream part of the dam site, with reported offset of 100 m. Fault 367 has a similar attitude as Fault 28, but it is located upstream of Ionakhsh Fault.

### *6.6.2 Discontinuity sets*

The 1978 Design Report (Ref.[1]) distinguishes three groups of discontinuities, namely lithogenetic (bedding), tectonic and exogenous (cracking due to weathering of the rock).

Discontinuities of the first group, namely bedding joints are continuous over the whole rock massif, with spacing ranging between 5 to 10 cm (siltstones, claystones) and up to 3-5 m (in sandstones). Average thickness is said to be 1 mm, with silty infilling.

Analysis of the tectonic discontinuities led to the definition of 4 joint sets, sets 1, 2 and 3 formed by folding of the geological formations, and set 4, the most recent, related to the development of the faults. They are presented in Table 6.7.

Characteristics of these discontinuities are given to be:

- Length: 0.5 to 7 m, 3 m in average,
- Spacing: some tens of centimetres,
- Aperture: average in galleries 0.6 mm,
- As hollow fissures with rough surfaces.

The design values for spacing and opening of the joints in each geotechnical zones have been given in drawing 1174-03-78, Sheet 3 of Ref.[1], and are reproduced in Table 6.3, Table 6.4, Table 6.5 and Table 6.6

| Joint set number | Dip azimuth / Dip angle |
|---|---|
| 1 (parallel to bedding joints) | 130 / 60-75 |
| 2 | 20-50 / 10-25 |
| 3 | 230-240 / 30-50 |
| 4 (similar to Fault 35) | 310-340 / 10-40 |

**Table 6.7: Orientation of the joint sets according to 1978 Design Report (Ref.[1])**

## 6.7 Hydrogeological characteristics of the dam foundation

### *6.7.1 General description of regional hydrogeology*

The 1978 Design Report distinguishes four types of groundwater flows, which are:

- water circulating within the Quaternary deposits of the river valley,
- water circulating within the fissures of the weathered and distressed zones of the rock massif,

- water circulating through fissures of the thrust faults zone,
- water circulating through karsts within saltiferous or gypsiferous rocks.

Mention shall also be made of the Obigarm thermal waters, of 40 to 50°C temperature, rising from the regional Illiak-Vakhsh and Gissaro-Kokshal Faults. The piezometric surface of these waters is located, according to Ref.[1], "not higher than 20-40 m above the normal operation level of the projected Rogun dam after full completion (full supply level at elevation 1290).

Except flow within the Quaternary deposits, in the area of the dam foundation, HPT states that most of the water circulates through the fissures of the rock, the Vakhsh River being the main drainage system.

From the yearly 750 to 900 mm precipitations, only 15% is assumed to infiltrate in the rock, the major part of the rain waters running down the slopes.

The slope of the underground water level within the left bank is estimated to 4 to 6 %, and permanent springs are scarce, except for two springs of 5 to 10 litre/sec discharge in the right bank, just upstream of Ionakhsh Fault, but at higher elevation than the supposed water table.

Mention is made of some recharge of the aquifers by the river itself during high-waters period, which is however dealt as insignificant.

On the dam site, the underground waters contain salt or sulphate in various concentrations according to their location, especially with respect to Ionakhsh Fault and its salt wedge (see paragraph 2.3).

A salt body is mentioned in the catchment area of the Passimurakho stream, which flows roughly in an East-West direction in the left bank, and joins the Vakhsh River just upstream of the dam site. Saline waters have been detected coming from this area, which is located more or less along the assumed trace of the Illiak-Vakhsh Fault.

A permanent spring of 100-150 litre/sec discharge is mentioned located on the left bank, in Obi-Shur River, left bank tributary of the Vakhsh River, just downstream of the dam site. However, this spring surges from the core of a big synclinal, located some 2.5 km upstream of intersection of Obi-Shur Valley by the Gulizindan Fault, i.e. rather far from the dam site.

### *6.7.2 Hydrogeological features of the dam site*

According to Ref.[1] (§ 2.3.2), the hydrogeological assessment of the dam site was made on the basis of investigations started in 1968, with a monitoring net of 2 to 17 boreholes, where water levels, chemistry of waters have been monitored.

Two types of behaviour were evidenced in the boreholes:

- holes where the water level is correlated with the variation of water level of the Vakhsh River, namely around elevation 983-985 during low water period (from March to mid-June), and elevation 990-991 maximum during high water period (mid-June to March of the following year),

- holes where groundwater levels remain more constant, with amplitudes of 2.8 to 4.2 m, although the maximum also occurs during the months of June and July; maximum levels may differ every year by 1 to 3 m, and this maximum does not appear every year.

Figure 6.5 shows the location of the main piezometers, with piezometers of the first group highlighted in blue colour, and the others in red colour, and the supposed position of groundwater isohypses.

It can be seen from this figure that, as expected, most of the piezometers of the first group are located very near to the river, but also along the Ionakhsh Fault, which presents particular hydrogeological conditions (see paragraph 6.7.3).

With respect to the slope of the groundwater level, it is around 4% in the left bank and 6% in the right bank.

However and as it can be guessed from Figure 6.5, piezometric data are not sufficient in number to establish reliable cross-sections of the piezometric level around the dam axis. Isohypses of Figure 6.5 obviously follow the topographic contours on the basis of the average slope of the groundwater level deduced from the mentioned observation wells.

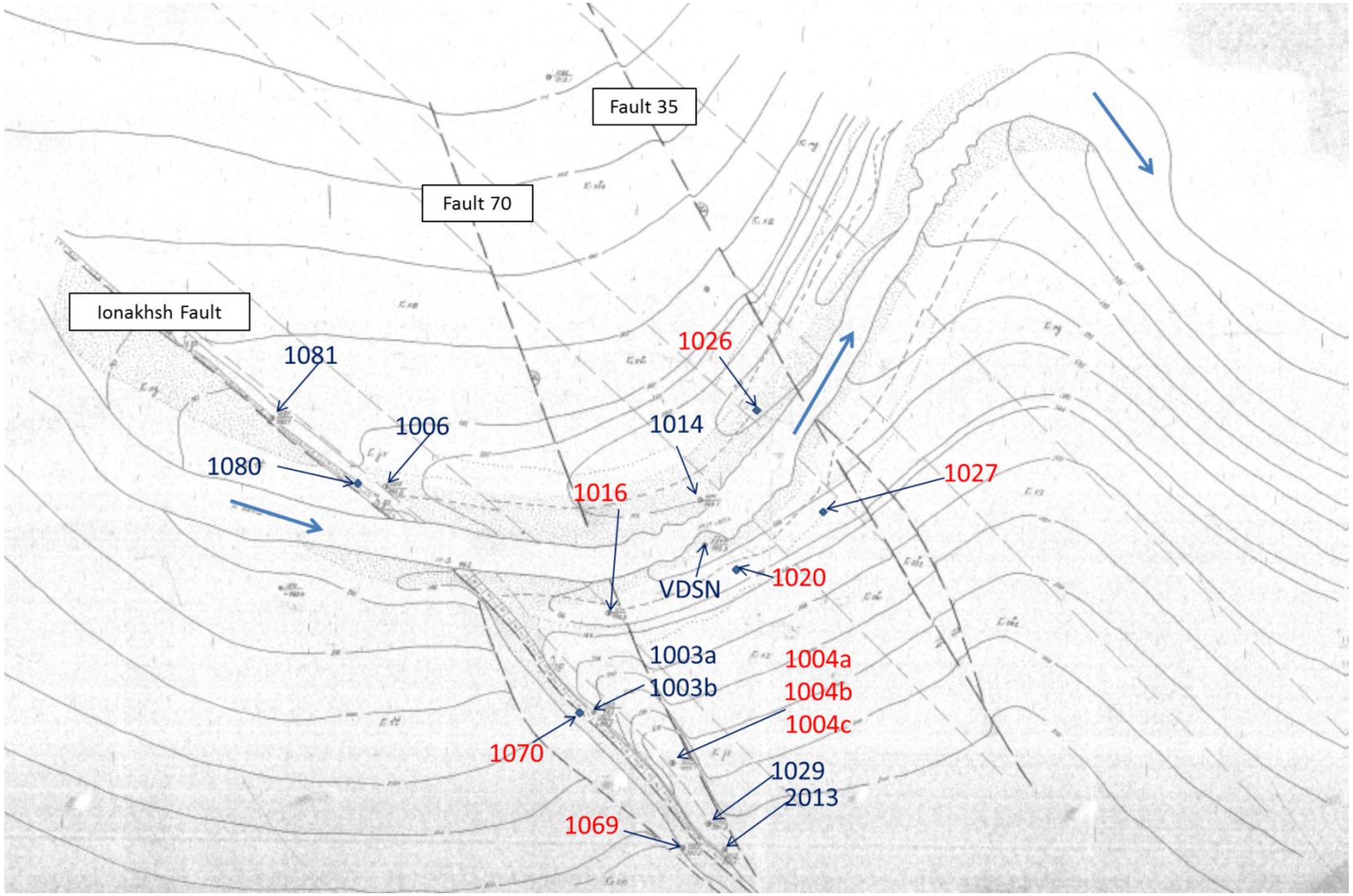



**Figure 6.5: Map of main piezometers used for assessment of hydrogeological conditions of the dam site; those mentioned in blue are reported to follow variations of the river level (after drawing 1174-03-76 of Ref.[1]**

The assumed hydraulic conductivities of the rock foundation is deduced from the water tests, which results are presented in Table 6.1. On the basis of these results, HPT assumes an average hydraulic conductivity of the undisturbed rock (zone IV as per definition of paragraph 6.3) of around $0.3 \times 10^{-7}$ m/sec (0.003 m/day). The hydraulic conductivities are rising rapidly when nearing the ground surface and the distressed, weathered zones of the rock massif.

### *6.7.3 Hydrogeological regime along Ionakhsh Fault*

Existence of a distressed zone left by dissolution of the salt above the top of the salt wedge ("caprock" of Russian documents) has already been mentioned (se paragraph 2.3), as well as the greater hydraulic conductivities with respect to the embedding rocks (see paragraph 6.4.2).

It can be seen from Figure 6.5 and Figure 6.6 that boreholes 1069 and 1070 are actually entirely located within the Mingbatman Formation, upstream of the fault. The other boreholes are mainly drilled within the Gaurdak claystones overlying the salt rock on the downstream side of the fault, and the salt rock itself.

Borehole 1004b (left side of Figure 6.6) is seen crossing the rock volume just above the top of the salt wedge. It experienced fall of tools, and the water levels in this boreholes as well as in the neighbouring boreholes 1004 and 1004c is reported to have dropped down from "up to 6-9 m" as the hole reached the space above the top of the salt wedge. Levels only partially restored during the following days (Ref.[1], § 2.3.2.1). No water test results are reported to have been made in borehole 1004b within this zone.

On the right side of Figure 6.6, it can be seen that a water test made in borehole 1029, very close to this distressed space of rock above the salt wedge, gave a value of up to 10 litre/min (around 1000 Lugeon unit).

This emphasises the particular behaviour of the distressed zone above the top of the salt wedge, which behaves, with respect to the embedding formation as a draining feature of larger hydraulic conductivity. This drainage effect of the Ionakhsh Fault's "caprock" zone is evidenced in the isohypses of Figure 6.5, and it is presumed that during floods, water from the river can feed this zone.

Because of the presence of the salt wedge, brines and salty waters circulate in the embedding rock masses of Ionakhsh Fault. Also, precipitation of gypsum occurs as a result of the hydratation of the anydrite contained within the rock salt, once the latter has been dissolved.

The specific problems linked to the dissolubility of the salt (transport, diffusion) are the object of the Phase 0 Report.

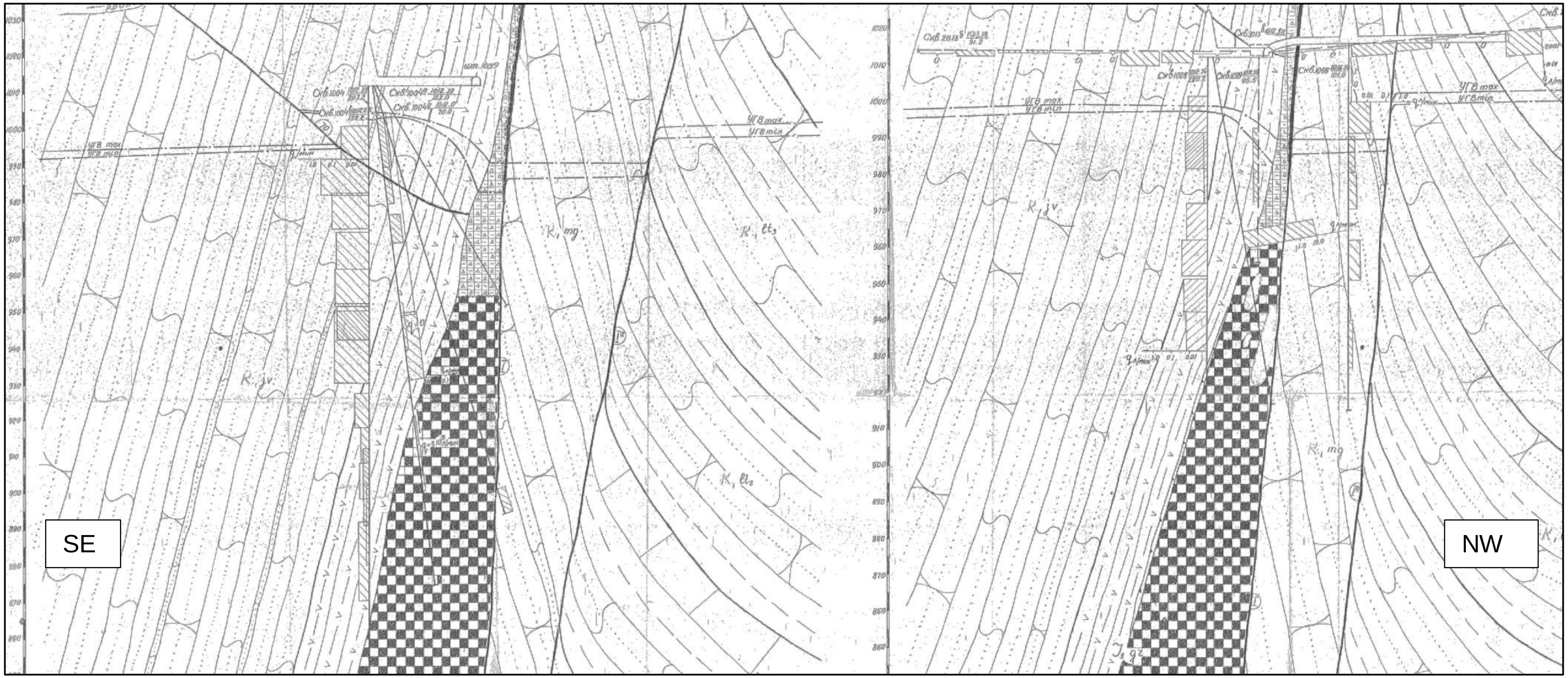



**Figure 6.6: Transversal cross-section of Ionakhsh Fault, along boreholes of 1004 group on the left side, along borehole 1069 and 1029 on the right side; water intake during water tests is plotted along the borehole lines (drawing 1174-3-41 Sheet 6, Ref.[1])**

Presence of waters with high concentration of halite (salt) and sulphates is therefore to be taken into account in the materials to be used for the Civil Works.

### *6.7.4 Impact of other fault zones on the hydrogeology of the site*

The 1978 Design Report also provides very valuable information about the impact of the other faults of the site on the hydrogeological regime.

During drilling of boreholes through Fault 35, and also Fault 70, located just upstream, between Ionakhsh fault and Fault 35 (see location on Figure 6.5), evidence of discontinuity in the ground water regime appeared.

When boreholes 1016 and 1080 reached the Fault 70, the groundwater level rose of 1.5 to 5 m. Similarly, when boreholes 1014, 1020 and 1027 – which head is located between elevation 1010-1013 - crossed Fault 35 in depth, they became artesian, with a discharge of hardly 0.5 litre/min.

Based on these observations, and given the clay infilling noticed in Fault 35, this fault is assumed to constitute a watertight screen within the dam foundation. It is also considered in the Original Project as the border between saline waters upstream and sulphate sodium water with some pressure below.

The discontinuity in water pressure between the two walls of Fault 70 is less marked.

It is actually worth to note that both faults are of similar attitude as joint set 4, as defined in Table 6.7, and that the 1978 Design Report states that similar persistent discontinuities (attitude 320-340/20-40) are present every 40 to 60 m distance within the dam foundation. Their length exceeds 200 m (crossing the foundation of the dam core), and their walls are infilled with clay gouges of 1 to 3 cm thickness. Thickness of the crushed zone is reported to not exceed 1 to 2 m (Ref.[1], § 2.6.1).

## 6.8 Conclusions of the 1978 Design regarding the different components of the Project

### *6.8.1 With regard to the disturbed zone of the right bank*

The 1978 Design Report assumes that the “disturbed zone” of the right bank is the result of large landslides occurred in the past, and involving up to 900 million cubic metres, with lowest elevation of failure surfaces down to elevation 1,200.

It is stated that there is no doubt about actual stability of these large-scale old landslides when impounding the reservoir, but that smaller landslides of some hundreds of thousand cubic metres may be triggered in the slope as a consequence of impounding of the reservoir, near to the tailrace tunnels outlet (downstream right bank).

To avoid the occurrence of such landslides, and in order to maintain the groundwater levels in this bank near to their present levels, two drainage galleries with a cumulative length of 2,134 m, fitted with long drainholes were forecasted, near to this area (Ref.[1], § 2.6.7.).

### 6.8.2 With regard to potentially unstable masses

The 1978 Design Report especially points out the zone of the upper left bank, located just above the entrance of the deviation tunnels (already briefly described in paragraph 0), which is there described of a loose mass of Kyzyltash rocks, of some 60,000 to 80,000 m3 in volume, extending between elevations 1,050 and 1,150 approximately.

The head scarp has a height of 3 to 4 m, and open fractures of 0.5 to 2 m width, with depth up to 8 m are present there.

As this rock mass is stated as not stable with regard to construction and operation conditions of the plant, the report recommends this mass to be removed before construction of the upstream cofferdam (Ref. [1], § 2.6.1. and § 2.6.5.).

Generally, this problem of warranting stability of rock masses along the joints of the No.4 family (see Table 6.7) in left bank of the Vakhsh River is a topic to be dealt with further on (see paragraph 12.1). The report points out that gravitational sagging plays also a role in such instabilities (Ref. [1], § 2.4).

Other potentially unstable masses are listed. It is also made mention of a mass of about 50,000 to 60,000 m3 of Mingbatman sandstones which sled along a discontinuity plane on the alluvium, at the location of the outlet portals of the tailrace tunnels.

### 6.8.3 With regard to the reservoir

With the normal water level at elevation 1,290, the reservoir will reach a volume of some 13.3 km³, extending over an area of 170 km² until 70 km upstream of the dam site.

The Designers point out that the reservoir will extend over most of its length along the Illiak-Vakhsh Fault, a seism-generating active fault (as well as it will cover part of the Gissaro-Kokshal neighbouring fault). Therefore, they acknowledge that forecasting the related modifications of the seismic background of the region is difficult.

With regard to potential slope instabilities which may occur after impounding, and even if Quaternary landslides of several millions m3 in volume have been noticed, the banks of the reservoir are assumed to remain stable (Ref. [1], § 4.1). This issue of potential landslides along the reservoir banks is dealt with more detail in Phase II Report - Volume 2 – Chapter 2 - Geology.

The Gulizindan Fault, of similar origin and characteristics as Ionakhsh Fault, and containing a salt wedge, which maximum elevation is assessed to be around elevation 1,110-1,120, is identified as a possible leakage way for water from the reservoir, especially through the distressed space above the salt (as per Ionakhsh Fault, hydraulic conductivity of this space is estimated to $5.10^{-5}$-$10^{-4}$ m/s). The Designers however estimate that potential circulation of water within the fault will be more controlled by the fact that water would have to cross first low-permeability layers of embedding rocks. Nevertheless, they insist that this point is to be checked by adequate geological surveying.

In response to a concern about the impact of reservoir impounding on the thermal waters exploited at Obigarm resort, the 1978 Design Report gives conclusions of work carried out on this matter by other state institutes. It is stated that hydrochemical and temperature conditions of the thermal waters will not change significantly, with about 1 g/l of salinity increase (Ref. [1], § 4.3.).

### *6.8.4 Dam foundation treatment*

The 1978 Design Report forecast the removal from the dam foundation of the distressed materials, apparently until reaching an adequate permeability for the core foundation (siltstones of Lower Obigarm).

Especially, Quaternary deposits (alluvium) are to be completely removed from the core foundation (their thickness in this area is reported to be 10 to 12 m).

The corresponding quantities forecasted to be excavated from the dam foundation (open-air excavations) was assessed to 1,6 million m3 of rock and 130 thousand cubic metre of other materials (earth, soil, etc.). Excavations for dam galleries within the foundation were forecasted to account for about 215,000 cubic metres (Ref.[1], drawing No.1174-10-303, Sheet 1).

As represented on Figure 6.7, the intended foundation treatments for the dam were as follows;

- a grouting curtain, which length could apparently be adjusted according to results of “exploratory” grouting holes,
- a drainage curtain,
- consolidation grouting, apparently made from the galleries of the dam foundation.

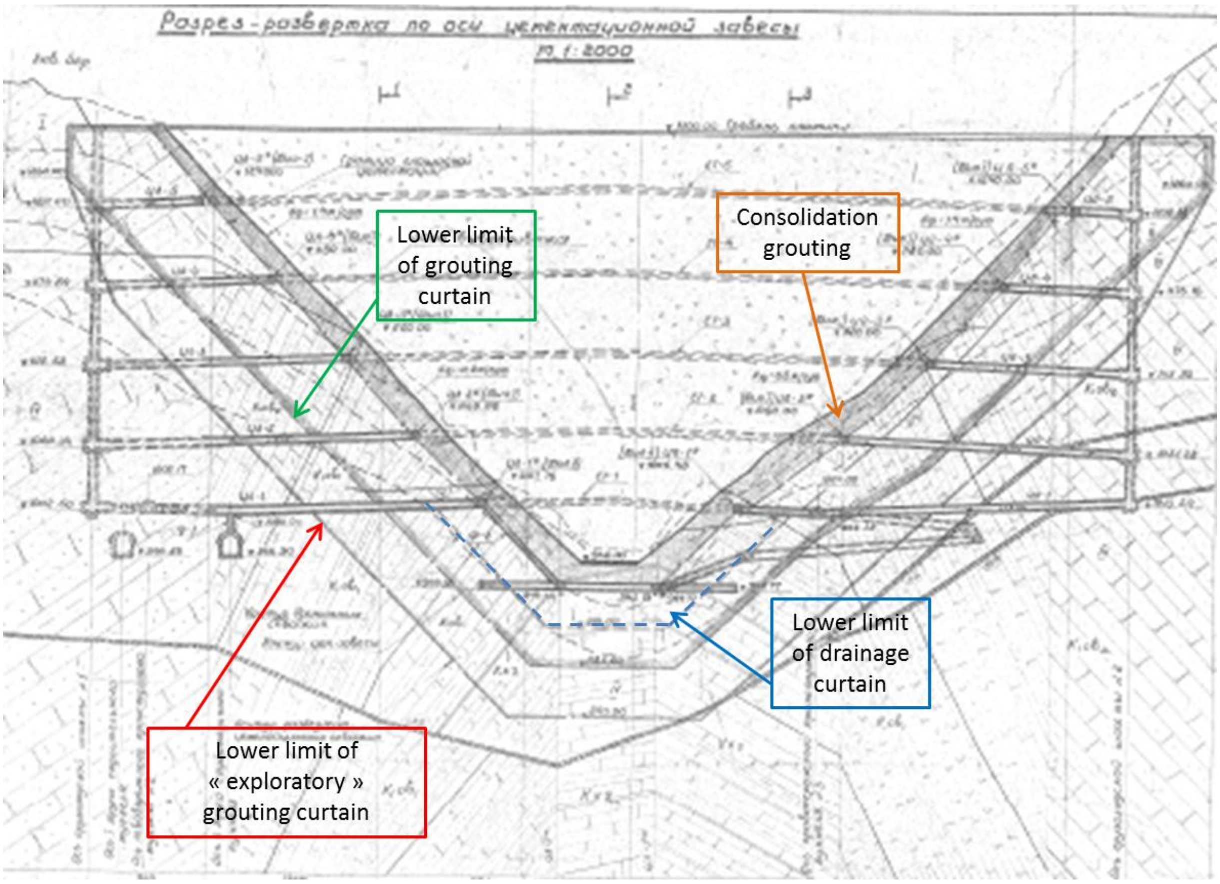



**Figure 6.7: Dam foundation treatment, original project of 1978, vertical cross-section along the dam axis (after Ref.[1], drawing No.1174-10-303, Sheet 1)**

### *6.8.5 Underground works and cavern excavations*

The 1978 Design Report (Ref.[1], § 2.6.2.) briefly recalls the specific rock mechanics tests performed at the location of the caverns, power house and transformer cavern.

The design values retained for the rocks of Lower and Upper Obigarm for excavation of the caverns (intact rock) were the following:

- Siltstones of Lower Obigarm: compressive strength of 70 MPa, tensile strength 5 MPa, cohesion 18 MPa and angle of friction of 53 degrees,
- Sandstones of Upper Obigarm: compressive strength of 130 to 150 MPa, tensile strength 8.5 MPa, cohesion 24 MPa and angle of friction of 59 degrees.

From tests realised through the pressurisation of a 2 m diameter chamber in gallery 1030, deformability of the rock mass of siltstone of Lower Obigarm could be determined; deformation modulus of 8,000 MPa and reaction coefficient $K_0$ of 5,500 MPa/m³.

For sandstones of Upper Obigarm, deformation modulus was assessed as most likely around 9,000 to 12,000 MPa, and reaction coefficient around 7,000 MPa/m.

From the investigations performed on discontinuities, characteristic values could be assessed, and are presented in Table 6.8.

| Discontinuity system number and nature | Orientation | | Distance between fissures (m) | Fissure length (m) | Shear strength along fissures surface | |
|---|---|---|---|---|---|---|
| | Dip azimuth (°/N) | Dip (°) | | | Friction angle (°) | Cohesion (MPa) |
| 1: Bedding | 130 | 68 | 0.7 | 5 | 28 | 0.01 |
| 2: Tectonic | 32 | 28 | 0.35 | 1.2 | 35 | 0.5 |
| 3: Tectonic | 220 | 54 | 0.3 | 2.1 | 35 | 0.5 |
| 4: Tectonic | 340 | 40 | 0.25 | 1.5 | 35 | 0.5 |

**Table 6.8: Characteristics of main fractures within rock masses around power house and transformer cavern (from Ref.[1], Table 2.6.2.; note that shear strength characteristics were determined for a range of normal stress of 0.5 to 1 MPa**

HPT estimates that, given the very low average specific water intake of the rock formations at this depth, i.e. roughly $0.2 \times 10^{-7}$ m/s, the total volume of water to be drained should be insignificant, and not exceed 100 litre/sec for the two caverns.

HPT recalls that, given the sulphate concentrations of the underground waters, up to 10 or 20 g/l, they shall be considered as aggressive to normal Portland cement.

### *6.8.6 Protection against mudflows from Obi-Shur River*

From the early stages of the design, the problem of regular occurrence of debris flows of large volume has been identified in the Obi-Shur River, a left-hand tributary of the Vakhsh River, located immediately downstream of the dam site. A large amount of debris, up to 65 million m3, has been identified as potential source of material for he mudflows in the Obi-Shur river valley. The maximum amount of debris to be discharged by one mudflow is assessed by the 1978 Design Report as being some 3 milllion m3 (the 1972 mudflow brought 600,000 m3 of material).

To ensure that mudflow may not trouble operation of the scheme, the 1978 Design had forecasted the construction of a 105 m high dam and an associated 600 m-long diversion tunnel, some 2.5 km from the Vakhsh River on the Obi-Shur River. At this location, the rocks pertain to the

Mingbatman Formation, and bedding dips towards upstream. A construction by blasting of the river shores was considered. Another tunnel was considered at the confluence with the Vakhsh River.

# 7 COMPLEMENTARY APPRAISALS MADE AFTER THE ORIGINAL PROJECT

## 7.1 Summary of studies performed after stopping the construction

After the construction according to the Original Project of 1978 was stopped in 1993-1994, a number of studies have been made, with the objective to reinitiate construction of a power plant from the existing facilities. With regard to geotechnics, three main studies can be distinguished.

- studies by HPI Moscow in 2000, on behalf of Barki Tojik, for completion of the Stage 1 dam,
- in 2004, HPI Tashkent, from Uzbekistan, performs a comprehensive survey of the installations at Rogun,
- studies from the 2005-2006 period, especially these ordered by the Russian Rusal Company, with the objective to define the best economical alternative according to its interest; Lahmeyer International was contracted by Rusal to produce a “Bankable Feasibilty Study”,
- the design project by HPI Moscow, dated 2009-2010 for achievement of the construction of Rogun power plant, according to the previous requirements of 1978.

The 2000 study radically differs from the 1978 Original Project in the fact that it clearly states that its Stage 1 dam shall not lie on Ionakhsh Fault, since this in not in compliance with new standards about possible seismic displacements along an active fault (see Ref. [6], § 1). Therefore, issue of salt dissolution is not dealt with in this study, and a concrete dam between Ionakhsh fault and Fault 35 is contemplated.

Similarly, the objective of studies ordered by the Rusal Company were aimed, as stated above, at finding the best economic alternative for this company, therefore contemplating a number of alternatives different from the Original 1978 Project.

However, almost all basic information about geology, geotechnics and hydrogeology – except for the particular topic of the deformations of the power house excavation – which is used in the following reports rely on the previous results from the investigation carried out before establishment of the 1978 Design Report and during the construction period (only part of this latter information being available to the Consortium). The noticeable complementary investigations are mainly focused on the power house area and properties of the embedding rocks, after subsequent convergence and distressing occurred during construction and the following years.

## 7.2 2000 Feasibility study for Stage 1

As stated in paragraph 7.1, the objective of this study was completion of the Stage 1 dam, and actually contemplates different alternatives of dam, all being located downstream of the Ionakhsh Fault.

With regard to geotechnical issues, the corresponding report (Ref. [6]) does not bring more information than the previously presented. Moreover, it is mostly focused on the dam alternatives, with few pages dedicated to geotechnics. The presentation of the geomechanical characteristics of the rocks is somehow summarised, differentiation being made, not according to the age of geological formation like in the 1978 Report, but according to the nature of the rock (sandstone, siltstone or tectonic lenses). Table 7.1 presents those characteristics. Some differences with the 1978 data can be spotted, with especially the presented specific water intakes, somewhat higher that the average one considered before and listed in Table 6.1. Compression strength of the sandstone is also taken as 110 MPa in sound rock, a reasonable value for most of the rock formations, except for Karakuz and Mingbatman, where, according to Table 6.3, the compression strength of the sandstone are of 66 and 83 MPa respectively.

This study reports the unexpected rock conditions in the area of the underground caverns, but the figures listed in the report appear not matching with the one from the subsequent reports. Convergence of the power house walls is here reported of 240 to 340 mm in the Upper Obigarm sandstone, stabilised 3 years after excavation, while in the Lower Obigarm siltstone, it is reported to have reached 600 to 660 mm by May 1999 (from which 160 to 180 mm were recorded during the excavation).

Those figures are well above all other figures mentioned in the subsequent reports, where convergence of the power house in siltstone does not exceed 500 mm by end 1999. No explication could be found to these figures.

## 7.3 Report on actual conditions of the construction site by HPI Tashkent, 2004

This report is the result of investigations carried out on behalf of Barki Tojik to assess the actual conditions of the site facilities. The report lists the different works until construction stopped in 1993 and the flooding of most of the underground works as a consequence of the collapse of the diversion tunnels (No.1 collapsed in 1991 and No.2 in 1993, according to Ref. [24], § 2.3.3.1) and the breach of the upstream cofferdam, followed by Obi-Shur mudflows.

| Engineering and geological zone | Blocks of the dominant lithotype | Depth of lower border of the zone as to the surface normal: Actual elevation | | | | Specimen Properties | | | Mass Parameters | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Below 1010 | 1010-1090 | 1090-1170 | Above 1170 | Density, tons/m$^3$ | Strength, Rc, MPa | Blocks, b, cm | Specific water intake, q, l/min | Deformation module, E, $10^3$ MPa | Specific impact resilience, $K_0$, MPa/cm | Design rock-hardness ratio |
| I | Sandstones: K1k2, K1ob2 K1mg | 7 | 15 | 25 | 40 | 2.60 | 90 | 15 | 2.20 | 2.0 – 2.5 | 15-20 | 2.0-3.0 |
| | Aleurites: K1jv K1ob1 K1k2 | 5 | 10 | 15 | 20 | 2.65 | 30-40 | 18 | 2.00 | 1.5-2.0 | 12-15 | 3.0 |
| | Tectonic lenses | 5 | 8 | 12 | 15 | 2.60 | <10 | 5 | 0.20 | 1.2 | 10 | 0.5 |
| II | Sandstones | 25 | 30 | 55 | 70 | 2.62 | 110 | 20 | 0.30 | 4.5 | 35 | 4.0-5.0 |
| | Aleurites | 20 | 45 | 65 | 80 | 2.70 | 60 | 15 | 0.20 | 3.5-4.0 | 23-27 | 4.0-5.0 |
| | Tectonic lenses | 15 | 20 | 30 | 40 | 2.62 | 10-70 | 7 | 0.06 | 1.5 | 12 | 1.0-1.5 |
| III | Sandstones | 50 | 65 | 100 | 140 | 2.62 | 110 | 20 | 0.05 | 7.5-8.0 | 60 | 7.0-8.0 |
| | Aleurites | 40 | 60 | 80 | 100 | 2.70 | 60 | 15 | 0.03 | 4.0-6.5 | 30-55 | 4.0-6.0 |
| | Tectonic lenses | 30 | 50 | 60 | 80 | 2.62 | 10-70 | 7 | 0.03 | 2.0 | 15 | 1.5-2.0 |
| IV | Sandstones | - | - | - | - | 2.62 | 110 | 20 | 0.02 | 8.0-8.5 | 60-70 | 7.0-8.0 |
| | Aleurites | - | - | - | - | 2.70 | 60 | 15 | 0.00 | 4.0-7.0 | 40-60 | 4.0-6.0 |
| | Tectonic lenses | - | - | - | - | 2.62 | 10-70 | 7 | 0.00 | 2.0 | 15 | 1.5-2.0 |
| III, IV | Rock salt | 25 | - | - | - | 2.20 | 23 | massive | 0.00 | 11.5 | | |

**Table 7.1: Geotechnical characteristics of dam foundation according to Ref. [6]**

Very valuable information is provided by this report about excavation of the power house. It states that modelling made before initiation of power house excavation forecasted (supposed for the current section, in sandstone) a convergence of the walls amounting to 240 mm at crane beam level, and 380 mm at mid-height. Modelling of excavation within siltstone forecasted a stabilisation of the convergences 10 years after excavation, forecasting total convergences of 610 mm at crane-beam level, and 635 mm at mid-height and adequate support measures were designed.

For detailed unfolding of the construction, one will refer to reports dealing with underground works. It can be summarised that fractures were observed opening, some anchors breached and excessive displacement occur, especially in the siltstone area or nearby (unit 5 and 6). This situation could partly be explained by some defaults in execution of the works, whereby corrective measures were decided by end 1990. The power house excavation has not been completed before construction stopped in 1993 around elevation 965, and as a consequence of the 1993 events, it is still reported flooded up to an approximate elevation of 985, near to the base of the vault (Ref. [9], § 2).

Convergences were continuously measured, despite losses of some measuring points, showing that movements were still going on. Additionally, an earthquake in January 2002 triggered some relative displacements of the walls. The report recommends more investigations to be performed for stability analysis.

With regard to the amount of convergence in the power house, the report states:

- in the section located in sandstone (Upper Obigarm), convergence by the end of 1991 (completion of 4th bench of excavation) was 85% of the total convergence recorded by end 2002,
- in the section located in siltstones (Lower Obigarm), convergence by the end of 1991 (completion of 4th bench of excavation) was only 47 to 66% of the total convergence recorded by end 2002.

No visible deformations are said to be seen in the transformer cavern, completely excavated in sandstone, although some convergence is occurring there as well.

With regard to geotechnical conditions, it is worth to mention that, according to this report, difficulties were encountered for excavation of diversion tunnel No.1 at its intersection with Fault 35, where the lower bench had remained unexcavated. A rock collapse of approximately 25,000 m3 was noticed at this location during a 1992 inspection, and later, it is stated that permanent inrush of rock from Fault 35 prevented repair of concrete lining there.

The diversion tunnel No.2 is affected by a diagonal crack of 100 to 120 mm opening at the inlet portal, as a consequence of which the portal, almost separated from the tunnel, is dipping towards the river, while the portal abutment wall collapsed in the river (Ref. [9]).

Farther inside, at 97 m distance from the portal, a caving of approximately 2,500 m3 was observed, in the area where the tunnel crosses Fault 70.

Another interesting information related to the Lower Obigarm siltstones characteristics is that excavation for the dam core had been developed in the form of 5 m high berms. Due to the rapid alteration of the Lower Obigarm siltstones when exposed to weathering agents, the excavations should have been protected by shotcrete. However, according to the report (Ref. [9]), this was not performed since actual erosion rate was judged insignificant. Nowadays, this shape given to the excavation is hardly to be seen on site.

The report also describes the works performed for implementation of mitigation measures to avoid salt leaching inside the Ionakhsh Fault, especially driving of grouting galleries and operation of the saline curtain.

It concludes that critical conditions in the power house had to be dealt with as soon as possible, as well as repair of diversion tunnels.

## 7.4 Studies of 2005-2006 period

### *7.4.1 General context*

A number of reports and documents were produced in 2005 and 2006, in the objective of resuming the construction. Most of these documents were actually ordered by the Russian Rusal Company, as said before in paragraph 7.1. With regard to geotechnical conditions, and for those who do not read Russian, special mention shall be made to the “Bankable Feasibility Report” produced by Lahmeyer International, from Germany, on behalf of Rusal, since it provides a summary of the information available at that time, as well as some valuable update of those conditions, and was written in English language.

We will list in this chapter the additional information provided by these 2005-2006 studies for each of the different topics.

### *7.4.2 Geomechanical characteristics of rock and rock masses*

#### *7.4.2.1 Content of additional investigations and studies*

With regard to geomechanical characteristics of rock masses, and after recording the unexpectedly high convergences in the power house excavation, some additional investigations were carried out for a better assessment. The investigations mainly focused on the Lower Obigarm siltstone and Upper Obigarm sandstone and the deformation modulus of the rock masses.

Meanwhile, computer back-analysis were performed, reproducing the excavation sequence of the power house, parameters being adjusted such as to fit with the observed movements of the cavern walls. Once adequate sets of parameters selected, resuming of excavation was simulated.

Lahmeyer (Ref. [18]) additionally carried out an approach of geomechanical characteristics through the rock mass classification as per GSI (Geological Strength Index).

An interesting fact is mentioned in Ref. [12] (§ 1.2.), among others, stating that an earthquake of magnitude 6 on the Richter scale occurred in 2002, after which an additional movement of 5-6 mm was recorded in the siltstone section of the power house. In the sandstone section, reportedly no additional movement was recorded.

The same report relates the dewatering of the power house, in May-June 2005, at a rate of 0.5 to 1 m/day (Ref. [12], § 3.1.2.). Measurements of power house convergence below crane beam level, in the part previously flooded since 1993, resumed just after dewatering operations.

*7.4.2.2 Peculiar features of rock masses*

First of all, considering the Lower Obigarm siltstone, most of the reports emphasise on their behaviour at excavation. It is reported that 8 to 12 hours after opening underground excavations in these siltstones, they become covered with small cracks, developing within one day a weathered zone of 0.3-0.5 m thickness, where rock strength is substantially reduced. Covering the excavation walls with a shotcrete protection layer reveals effective (Ref. [12], § 1.1.).

Lahmeyer (Ref. [18], § 3.7.1.) reports that samples taken from the core excavations, immersed in water, exhibited either cracking or disintegration within one day, or even dispersive effect. Additionally, an analysis by X-ray diffraction of samples of the Obigarm siltstone showed that phyllo-silicate minerals made more than 60% of the rock material, while quartz content was just above 20%. Therefore, strain softening and creep are expected to occur, with residual characteristics of the rock substantially reduced with respect to the peak one. The high values of convergences observed in the siltstone-section of the power house excavation (2.8 to 3% over the width of the cavern) demonstrate it.

The Lahmeyer report also emphasises that there is a risk of suffusion, i.e. internal erosion by migration of fines under the water pressure gradient, in the Obigarm siltstones, which may affect their long-term hydraulic conductivity. Nevertheless, the effect of the dissolution of gypsum, which accounts for 0.5% of the rock, concentrated along joints, is dealt to have a greater potential impact on hydraulic conductivity (Ref. [18], § 3.7.1.).

Additional information is the mention of tests performed by V. Kubetsky on the siltstones, made in Ref. [14]. According to this, creeping would appear when "ratio $\tau/\sigma_{mean}$ >=0.75 under triaxial conditions, 0.3-0.4 MPa of shear stress under simple shear" (criteria taken into account for consideration of a 330 m high concrete dam).

According to Lahmeyer report (Ref. [18]), the potential long-term dissolution of gypsum shall also be accounted for in the more recent formations, and especially within the Lyatoban Formation, where layers of gypsum are present (see Table 2.1).

*7.4.2.3 Results of additional on-site investigations*

The Geodynamic Research Center (GRC) performed geophysical investigations in the first half of 2005 on site, mainly consisting in geophysical surveys with application of seismic and ultrasonic methods, to check the conditions of the rock massif around the existing underground works (Ref. [12]).

Data presented for geomechanical properties of the rock are summarised in a table which appears to be almost same as the one of the 2000-Feasibility Study, reproduced in Table 7.1.

With regard to the specific properties of rock around the power house excavation, two tables are presented in this report, which reportedly summarise results of investigations carried out after initiation of construction (around 1989, according to Ref. [16], which present similar tables).

Table 7.2 presents results of uniaxial and triaxial tests performed on intact rock samples. The right part of the table is addressing the rock mass, and assessment is not different from the 1978 values, Zone IV (see Table 6.3).

| Rock types | Properties in the sample | | | | | | | Properties in the massif | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Density t/m³ | Uniaxial compressive resistance, MPa | | Tensile resistance, MPa | Resisitance under σn=15MPa | | Deformation module, MPa | Deformation module, MPa | Permeability, l/min | Calculated strength coeffiaient |
| | | uniaxial | triaxial, $\sigma_2=\sigma_3=$ 15 MPa | | C MPa | φ degree | | | | |
| Siltstones $K_1ob_1$ | 2.65 | 70 | 110 | 5.0 | 18 | 53 | 28000 | 5500 | ≤0.001 | 5-6 |
| Sandstones $K_1ob_2$ | 2.65 | 130 | 220 | 8.5 | 24 | 59 | 34000 | 9000 | 0.003 | 7-8 |

**Table 7.2: Properties of embedding rock masses of the power house complex; summary of complementary results of 1989 (after Ref. [12], Table 1.3.)**

| Genetic type | # of the system | Orientation elements | | Distance between the cracks, m | The length of the cracks, m | Shearing resistance along crack plate | |
|---|---|---|---|---|---|---|---|
| | | dip azimuth | hade | | | C MPa | φ degree |
| Strata | 1 | 130 | 65 | 0.70 | >10 | 0.03 | 29 |
| Tectonic | 2 | 32 | 28 | 0.30 | 2.5 | 0.02 | 32 |
| Tectonic | 3 | 220 | 45 | 0.20 | 4.0 | 0.02 | 32 |
| Tectonic | 4 | 340 | 50 | 0.30 | 4.0 | 0.02 | 32 |

**Table 7.3: Characteristics and estimated shear strength of discontinuities within rock masses of the power house excavation (after Ref. [12], Table 1.4.)**

Table 7.3 presents the estimated shear strength parameters, which are not far from those of 1978, but distributed according to the different sets of discontinuities.

The report presents the investigations made, which have consisted essentially in seismic tomography and ultrasonic logging of boreholes drilled at several locations from the underground works, and especially around the power house excavation. Since the process of distressing of rock around the excavations had been investigated from the initiation of excavation around 1989, by same means of seismic and ultrasonic logging , measurements performed in 2005 were as much as possible done at same locations, especially around the power house.

To summarise, overall results tend to show that the extent of the distressed zones around the excavations did not really change since 1992, but that rock properties within the distressed zones had substantially deteriorated, with measured velocities significantly lower. A marked distressing is mentioned for the pillar between the power house and the 12 m wide assembly room located on its upstream side, reportedly due to absence of timely installed support (Ref. [12], § 3.2.2.).

By construction time, it could be observed, using the same means of investigation, that after some one to three years stand-by, the wave propagation velocities had increased slightly with regard to values recorded just after excavation, thereby demonstrating some efficiency of the support. One can conclude from the 2005 results that distressing has been going on between 1992 and 2005 within the damaged zone, the support being likely not sufficient for long-term stabilisation.

A summary of the compared results for longitudinal wave velocity between the construction period and the 2005 measurements is presented in Table 7.4, for the different rock formations investigated. The report concludes that, except for the rock around the power house excavation, no major changes were noticed.

From March to May 2006, complementary investigations were carried out by the same Geodynamic Research Centre.

Six boreholes were drilled for pressuremeter tests, seismic tomography and ultrasonic logging, located within Kyzyltash sandstones and Lower Obigarm siltstones (by that time, the objective fixed by the Rusal Company was to assess the suitability of the foundation for an arch dam, of probably lower height than the main dam). Deformation moduli measured by means of pressuremeter were on average 3,500 MPa in Lower Obigarm siltstone, and 4,600 MPa for Kyzyltash sandstones. Different correlations were used for comparison between moduli from pressuremeters, and estimations by ultra-sound velocities and seismic wave velocities, taking into account the scale effect.

The report concludes that differences between moduli from pressuremeters originates from the scale effect and the fact that a distressed zone is formed around the boreholes. Therefore, moduli obtained from seismic wave velocity analysis are considered as more reliable for the undisturbed, stressed rock mass. Seismic velocities are quite similar to the one of Table 7.4 for the Kyzyltash Formation, but higher values were recorded in Lower Obigarm (4.14 km/sec average).

| Geological units | Predominant rock types | Geotechnical zone | Location | Longitudinal wave velocity (km/sec) | |
|---|---|---|---|---|---|
| | | | | 1978-1992 data | 2005 data |
| Lower Lyatoban ($K_1lt_1$) | Low strength, grey siltstones, , with layers of sandstone, dolomite or gypsum | II | Pillar between cable tunnels 1 and 2, studied from the gallery 1007 | 2.6 – 2.8 | 2.6 – 3.4<br><u>3.0</u> |
| Lower Mingbatman ($K_1mg_1$) | Sandstones with layers of siltstone | III - IV | Pillar between transportation tunnels T-3 and T-37 studied from the galleries 1002 and 1030 | 3.5 – 3.9 | 3.1 – 4.5<br><u>3.8</u> |
| Upper Obigarm ($K_1ob_2$) | Massive sandstones | III - IV | Pillar between the machine room and the transformer room<br>Pillar between the transportation tunnels T-3 and T-37 at the break # 35 site, studied from the gallery 1030 | 3.6 – 4.5 | 3.3 – 4.3<br><u>3.8</u> |
| Lower Obigarm ($K_1ob_1$) | Sitlstones, with layers of sandstone | IV | Pillar between the machine room and the transformer room<br>Pillar between the machine room and the assembly room | 3.5 – 4.5 | 3.1 – 4.5<br><u>3.5</u><br><u>3.8 (*)</u> |
| Kyzyltash ($K_1kz$) | Thick sandstones with layers of siltstone | IV | Pillar between the repair, main and wrecking gate rooms of water-discharge tunnels<br>Pillar between the transportation tunnel T-8 and the construction tunnel No. 2 | 3.6 – 4.2 | 3.4 – 4.2<br><u>3.8</u> |
| Gaurdak ($J_3gr$) | Rock salt, gypsum, red claystones, red gypsum clays | II | pillar between the transportation tunnel T-3 and the cementation gallery CSh-2 | 2.7 ± 0.6 | 2.4 – 3.4<br><u>2.9</u> |

**<u>Note:</u>** the" underlined value under 2005 data is the average velocity value
(*) Average value for saturated massif

**Table 7.4: Comparison of longitudinal wave speed within rocks masses around underground works outside excavation influence zone measured during construction and in 2005 (from Ref. [12], Tables 2.3 and 4.1)**

The deduced deformation moduli are given as 8,000 MPa for Lower Obigarm siltstone, and 9,000 MPa for Kyzyltash sandstones (Ref. [20], § 3.3.3.). The value for Lower Obigarm is therefore substantially higher than the 5,500 MPa from the previous studies. Reliability of the different correlations used for interpretation may therefore be questioned.

*7.4.2.4 Computer back-analysis of the power house excavation*

Apparently, and from the curves presented in the different reports, all back-analysis performed considered the convergences as measured at crane beam level (lower part of the cavern was flooded). The movements as measured are reportedly underestimated, because some time gap prevailed between excavation and actual implementation of the measurements (see Ref. [11], between others).

Hydrospetsproyekt performed in 2005 a complete analysis of the excavation of the power house complex, i.e. power house, transformer cavern and assembly chamber (Ref. [13]). The back-analysis has been modelling the different steps of construction already achieved, using both MARC software and Roclab / Phase2 software.

Work was carried out considering two sections, one in the siltstone area of the power house, the other one in the sandstone area. From analysis of all available data, in particular measurements of compression waves, the following values were selected as input parameters of the model. It is précised that, due to the scale effect, actual values of the deformation moduli may be lower.

| Rock | Calculated strength in the sample, MPa | | Calculated shear strength | | Deformation modulus in the mass, MPa | | Poisson ratio, ν |
|---|---|---|---|---|---|---|---|
| | Rc | Rp | tg φ | C, МПа | Uninvaded mass | Decompaction zone | |
| Sandstone | 100 | 10 | 1,2 | 3,0 | 9000 | 6000 | 0,22 |
| Siltstone | 70 | 7 | 1,0 | 2,0 | 5500 | 4000 | 0,30 |

**Table 7.5: Input parameters for MARC model (after Ref. [13])**

MARC model, with elastoplastic, parabolic Mohr-Coulomb failure criterion cut in tension was used in the modelling. However, in order to fit to the convergences, it was found that the properties to be introduced were as presented in Table 7.6.

With the level of natural stress (14 MPa vertical and 18 MPa horizontal), it was found that resuming excavation as per design would lead to instability (lower in-situ stresses required, 11.6 MPa vertical and 12 MPa horizontal).

The Roclab/ Phase2 analysis was made taking into account convergences until 1999 and a Mohr-Coulomb criterion used, with input parameters as per Table 7.7.

| No. | Properties | Sandstones | Siltstone | Fractured zone |
|---|---|---|---|---|
| 1. | Rock mass deformation modulus, MPa | 6000 | 3700 | 2000 |
| 2. | Elasticity modulus, MPa | 8300 | 5300 | 3000 |
| 3. | Poisson ration | 0.22 | 0.30 | 0.23 |
| 4. | Standard compressive strength, MPa | 125 | 87.5 | 35 |
| 5. | Standard tension strength, MPa | 12.5 | 8.75 | 3.5 |
| 6. | Standard tangent of internal friction angle | 1.50 | 1.25 | |
| 7. | Standard adhesion | 3.75 | 2.50 | |

**Table 7.6: Geomechanical parameters to be adopted to fit observed convergences (after Ref. [13], units probably same as for Table 7.5)**

| Parameter | Sandstone rock mass | | Relief zone | |
|---|---|---|---|---|
| | Elastic zone | Plastic zone | Elastic zone | Plastic zone |
| Strain modulus, MPa | 9000 | | 6000 | |
| Poisson's ratio, ν | 0.22 | | 0.3 | |
| Dilatation angle, *i* | | 15° | | 5° |
| Rp, MPa | 0.6 | | 0,4 | |
| φ, deg. | 50 | 47 | 45 | 42 |
| C, MPa | 3 | 2 | 2 | 1.5 |

| Parameter | Siltstone rock mass | | Relief zone | |
|---|---|---|---|---|
| | Elastic zone | Plastic zone | Elastic zone | Plastic zone |
| Strain modulus, MPa | 5500 | | 4000 | |
| Poisson's ratio, ν | 0.3 | | 0.33 | |
| Dilatation angle, *i* | | 5° | | 0° |
| Rp, MPa | 0,4 | | 0.25 | |
| φ, deg. | 45 | 42 | 40 | 37 |
| C, MPa | 2 | 1.5 | 1.5 | 1 |

**Table 7.7: Input parameters for Phase2 model**

For comparison between actual displacements and calculated, it was taken into account that the measured displacements are lower than the actual one, since monitoring does not start instantaneously after excavation, as also emphasised in Ref. [11].

Conclusion of Phase2 analysis evidences extension of plastic yield zones around the caverns and within the pillars (safety coefficients very close to 1):

- between power house and the assembly chamber (entirely located in siltstones)
- between the power house and the transformer cavern (in sandstone).

Overall conclusion is that completion according to the design is risky, and may require much time and expenses.

Finally, the results of the different studies carried out are well summarised in Ref. [22], which exposes the results of geomechanical characteristics of the sandstone of Upper Obigarm and siltstone of Lower Obigarm, where the power house is located.

Notably, a 3D-analysis was performed with Z-soil software, which provides, according to Ref. [22], still a better fit with the actually observed conditions and convergences. Corresponding geomechanical characteristics are given in Table 7.8.

Table 7. Strength and deformation properties of rock mass in machine hall area (results of 3-D analysis).

| | Sandstone | | Aleurolite | |
|---|---|---|---|---|
| Parameters | Unaltered mass | Weakened (stress-relieved) zone | Unaltered mass | Weakened (stress-relieved) zone |
| $\varphi$, dgr | 42 | 38 | 36 | 32,5 |
| Cohesion $C$, MPa | 1,64 | 1,1 | 0,75 | 0,5 |
| Modulus of deformation $E$, MPa | 7500 | 5000 | 4000 | 2670 |
| Poisson's ratio, $\nu$ | 0,22 | 0,30 | 0,30 | 0,33 |

**Table 7.8: Parameters which fits at best with observed convergences of the power house excavation, according to Ref. [22]**

Additionally, a summary of the different deformation modulus values obtained from the various methods is presented in Table 7.9.

Table 8. Summary of moduli of deformation derived by different methods.

| Method | | Values of modulus $E$ in MPa | | | |
|---|---|---|---|---|---|
| | | Sandstone | | Aleurolite | |
| | | Unaltered rock mass | Weakened zone | Unaltered rock mass | Weakened zone |
| Rating by Hoek-Brown classification | | 15.000 | | 6000 | |
| Geophysical methods Seismic prospecting | | $\frac{11000^{*)}}{8400}$ | $\frac{8300^{*)}}{6250}$ | $\frac{8100^{*)}}{6100}$ | $\frac{5800^{*)}}{4300}$ |
| Pressure measurement | | | 4600 | | 3500 |
| Computational methods | 2-D model | 9000 | 6000 | 5500 | 4000 |
| | 3-D model | 7500 | 5000 | 4000 | 2670 |

*)upper figure indicates value of $E_{II}$, lower figure indicates – modulus $E_{\Sigma}$ .

**Table 7.9: Deformation moduli as deduced per different methods (from Ref. [22])**

Therefore, already in 2005, resuming the power house excavation as per Original Project was assessed as risky.

*7.4.2.5 Geomechanical characteristics as per rock mass classification (Lahmeyer, 2005)*

The assessment of the GSI (Geological Strength Index) was used by Lahmeyer to assess the characteristics of the rock masses. We recall here below briefly basics of the method.

The GSI may be estimated by use of charts published in previous papers by Hoek and others (see especially Ref. [18], [21] and [30]) or by means of estimating ratings describing the different rock mass properties, such as compressive strength of the matrix, RQD, spacing and conditions of discontinuities actually on the basis of the estimation of RMR (Rock Mass Rating) as per defined by Bieniawski in Ref [2]. Together with the definition of the compressive strength of the rock mass and parameter $m_i$ (which may be linked to the ratio of tensile strength to compressive strength of the rock matrix), it provides a tentative failure criterion for jointed rock masses.

Charts proposed for estimation of the GSI of rock masses are reproduced here below in Table 7.10 (general for all rock masses) and Table 7.11, adaptation made for heterogeneous rock masses.

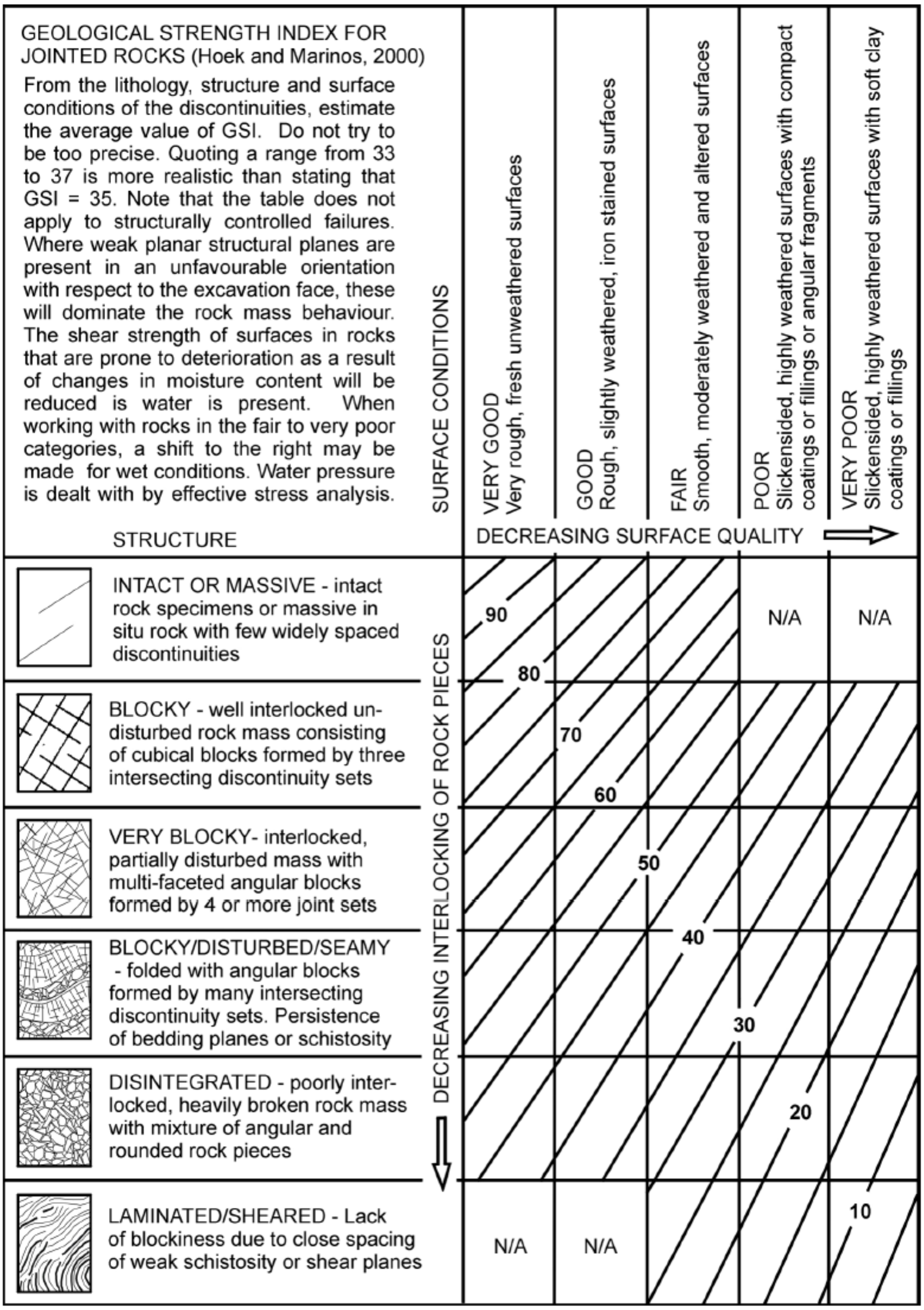



**Table 7.10: Characterization of blocky rock masses on the basis of interlocking and joint conditions (reproduced from Ref [21])**

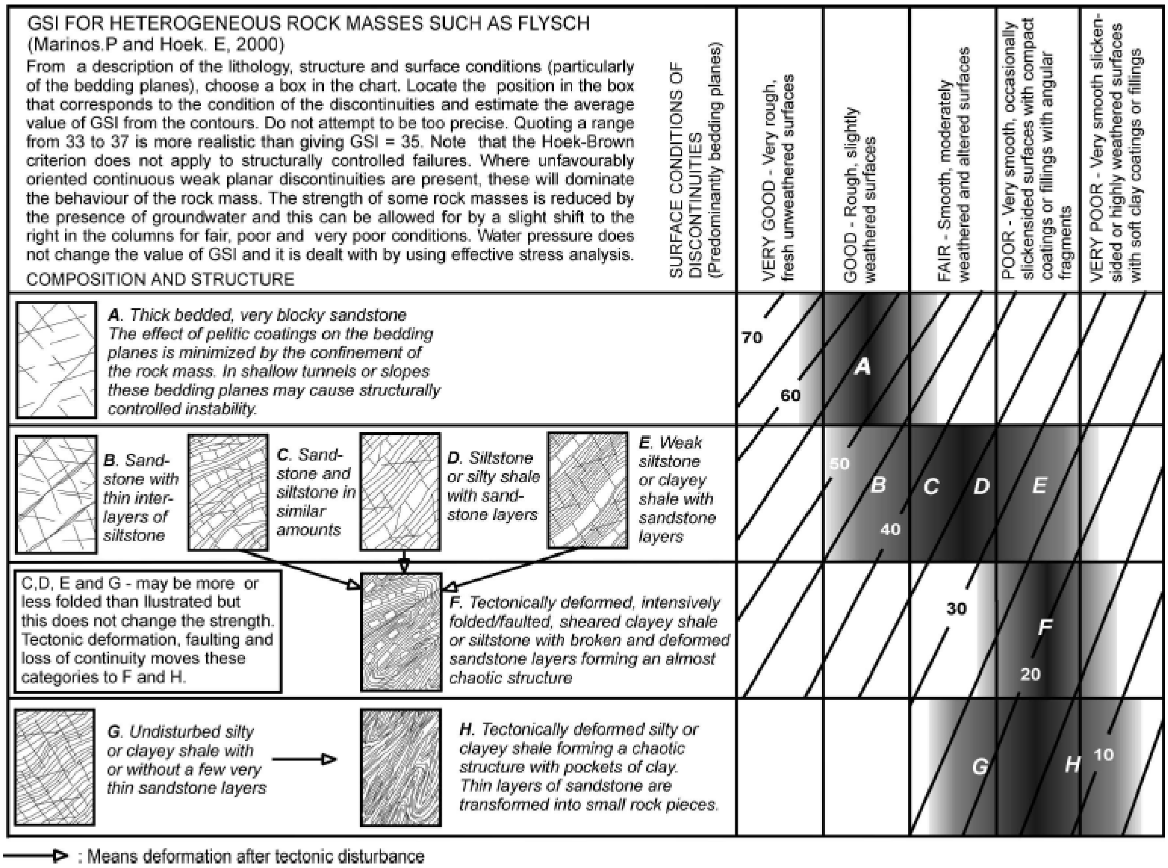



**Table 7.11: GSI estimates for heterogeneous rock masses such as flysch (reproduced from Ref. [21])**

Lahmeyer used the table for heterogeneous rock masses (Table 7.11) to assess the GSI of the rock formations of the Rogun dam site. The assessed structure of the rock was assessed between types A and B, with surface conditions of discontinuities from poor to good (see Table 7.11), with results as follows:

- Massive sandstones (Upper Obigarm) and fresh sandstones (Kyzyltash, Karakuz) with GSI values ranging from 45 to 55,
- Siltstones with lower values, which may drop to 30 in case of weathered, distressed siltstones, as well as if overstressed.

Lahmeyer however estimates that about one third of the siltstone of the Lower Obigarm Formation is actually more argillaceous, and characteristics should be assessed separately from the rest of the formation. The originally assigned value of compressive strength of 57 MPa (see Table 6.2) is estimated to be actually lower, near to 50 MPa (Ref. [18], § 3.8.).

| Parameter | Upper Obigarm sandstone | Lower Obigarm siltstone | Source |
|---|---|---|---|
| GSI | 60 | 50 | Estimated according to geophysical investigation of 2005 |
| | | 35 | According to Ref. [21] |
| $m_i$ | 15 | 7 | According to Ref. [21] |
| Uniaxial compressive strength (MPa) | 100 | 50 | Original Design |
| Peak cohesion $c_{peak}$ of rock mass (MPa) | 1.8 – 2.4 | 0.8 – 1.2 | According to Ref. [21] assuming 375 m overburden and disturbance factor D=0 |
| | 1.5 | 1.2 | Lahmeyer estimation |
| Residual cohesion $c_{res}$ of rock mass (MPa) | - | 0.53 – 0.88 | Using GSI=35 |
| | 1.2 | 0.88 | With disturbance factor D=0.6 |
| Peak friction angle $\varphi_{peak}$ of rock mass (degree) | 44 - 50 | 28 - 36 | According to Ref. [21] assuming 375 m overburden and disturbance factor D=0 |
| | 45 | 36 | Lahmeyer estimation |
| Residual friction angle $\varphi_{res}$ of rock mass (degree) | | 21- 31 | Using GSI=35 |
| | 40 | 30 | Estimated close to disturbance factor D=0.6 |
| Deformation modulus E (GPa) | 14 | 7 | According to GSI |
| | | 3 | Using GSI=35 |
| | 8 | 5 | Previous reports |
| | 6 | 4.3 | Back-analysis of power house deformations |
| Poisson coefficient | 0.3 | 0.33 | Back-analysis of power house deformations |
| Horizontal component of in-situ stress field $\sigma_h$ (MPa) | 18 | 18 | According to Ref. [11] |
| Vertical component of in-situ stress field $\sigma_v$ (MPa) | 12 | 12 | According to Ref. [11] |

**Table 7.12: Summary of rock mass parameters for siltstones and sandstones of the power house, using different estimations; the values retained as representative for the long-term behaviour of the two rock formations are coloured in grey (compiled from Ref. [18])**

Lahmeyer used different estimates of input parameters, mainly from the Original Design of 1978, but also from estimation from GSI, and completed a back-analysis of the power house excavation in a finite element analysis. Since it was difficult to assign values for the residual characteristics of the yielded rock, the disturbance factor D was used for adjustment, in order to find back the measured deformations. Therefore, the disturbance factor D shall not be interpreted here as per Hoek’s definition of blast damage, but a parameter for stress reduction after yielding of the rock. The long-term values of geomechanical parameters to be assigned to sandstones and siltstones of the Obigarm Formation were deduced; summary of the parameters is presented in Table 7.12, with recommended long-term parameters highlighted in grey colour.

Lahmeyer states that creeping of the Obigarm siltstones is still going on, and that further reduction in the long-term shear parameters of the siltstones is not excluded (Ref. [18], § 4.2.4.1.). It states that, if creeping in sandstones seems to be stabilised, situation within siltstones is critical, and that full excavation of the powerhouse cavern in the Ob1 siltstone is “practically impossible”.

The reason for this is reportedly that the rock pillar between the power house and the transformer cavern is fully plastified, with anchors of the power house “mostly fully overloaded on the downstream side and at least partly on the upstream side”.

Therefore, Lahmeyer suggested to backfill the part of the power house located in siltstone up to the machine floor with concrete, thereby stabilising the cavern in this area, and creating a platform which could be used as erection bay or other purposes (Ref. [17], § 5.2.1.).

With regard to the dam foundation parameters, Lahmeyer proposes to apply to the Lower Obigarm siltstones and the Upper Obigarm sandstone a reduction factor equivalent to the ratio between values deduced from the above power house stability analysis and corresponding Zone IV values of the Original Design. No reduction factor is to be applied to the Kyzyltash Formation. In this way, new geomechanical parameters are obtained for these three rock formations, in each geotechnical zone (as per definition of paragraph 6.3).

*7.4.2.6 Complementary studies of Hydroproject in 2009-2010*

With regard to geomechanical characteristics, HPI states that only results of the additional studies carried out in 2009 are mentioned in their report. This report (Ref.[27], § 1.2.2.) clearly states that studies were resumed on the basis of the former data (1978 and subsequent documents produced during the construction by HPT), but that the table of geomechanical characteristics has not been updated since the 1978 Design. Therefore, HPI insists in its 2009 report that additional investigations would need to be actualised because of some uncertainties about the weathering area location, and their geotechnical parameters.

However, geomechanical characteristics for the dam foundation are given in Table 7.13 from this report, considering only zones I to III, and average values, without distinction between sandstones and siltstones.

For the underground power house, the data which were found to best fit the observed convergences are listed in Table 7.14.

| № | Rock characteristics | Zones | $\rho_н$, ton/m$^3$ | $\varphi^0$ | C, MPa | E, MPa | μ | $K_ф$, m/d |
|---|---|---|---|---|---|---|---|---|
| 1 | Sandstone, aleurolites, mudstone | I | 2.65 | 40 | 0.1 | 2000 | 0.25 | 1.5 |
| | | II | 2.65 | 40 | 1.0 | 4500 | 0.25 | 0.5 |
| | | III | 2.65 | 45 | 2.0 | 7000 | 0.25 | 0.02 |
| 2 | Rock salt | | 2.23 | 68 | 0.6 | 1140 | 0.30 | 0.001 |
| 3 | Crush zone (fault) | | 2.30 | 27 | 0.01 | 1500 | 0.30 | 1.0 |

**Table 7.13: Summary of geomechanical characteristics for the dam foundation (after Ref. [28], § 5.2.2.)**

| Parameter | Siltstone of Lower Obigarm | | Sandstone of Upper Obigarm | |
|---|---|---|---|---|
| | Intact rock | Damaged zone | Intact rock | Damaged zone |
| Deformation modulus (MPa) | 4000 | 2700 | 7500 | 5000 |
| Poisson coefficient | 0,30 | 0,33 | 0,22 | 0,30 |
| Friction angle (degree) | 40 | 36 | 50 | 45 |
| Cohesion, MPa | 1,2 | 0,8 | 2,0 | 1,5 |
| Tensile strength (MPa) | 0,40 | 0,25 | 0,60 | 0,40 |

**Table 7.14: Set of parameters fitting with power house excavation sequence and convergences (after Ref. [28])**

| Referred type of rock | Rock formation | Deformation modulus E (MPa) | | Poisson coefficient ν | | Friction angle (degree) | | Cohesion (MPa) | |
|---|---|---|---|---|---|---|---|---|---|
| | | Intact rock | Damaged zone | Intact rock | Damaged zone | Intact rock | Damaged zone | Intact rock | Damaged zone |
| Siltstone | K1ob1 | 5500 | 1800 | 0,28 | 0,33 | 45 | 37 | 1,2 | 0,5 |
| Sandstone | K1ob2 | 9000 | 3000 | 0,22 | 0,30 | 55 | 42 | 2,0 | 1,0 |
| Sandstone | K1kr | 7000 | | 0,26 | | 50 | | 2,0 | |
| Sandstone | K1kz | 8000 | | 0,24 | | 55 | | 2,0 | |
| Zone of Fault №35 | | 1000 | | 0,35 | | 33/22 | | 0,1/0,02 | |
| Zone of fracture №273 | | 2000 | | 0,32 | | 37/27 | | 0,3/0,05 | |
| Zone of Fault №70 | | 2000 | | 0,32 | | 37/27 | | 0,3/0,05 | |

**Table 7.15: Deformation properties and shear strength of rock masses for underground works (after Ref. [28])**

*7.4.2.7 Golder Associates for World Bank*

In the corresponding report (Ref. [30]), Golder Associates exposes the results of a 2D-back-analysis it performed on the power house, the geomechanical parameters of siltstones and sandstones being estimated from RMR and GSI. According to the report, deformation moduli which best fit the observed convergences would be 7,500 MPa and 5,500 MPa for sandstones and siltstones respectively.

# 8 APPRAISAL OF THE CONSORTIUM ON THE GEOTECHNICAL PROPERTIES OF THE FOUNDATION

## 8.1 On geotechnical zoning

Analysis of drawings 1174-03-F18, F19 and F20 (Ref. [1], §2.5.3.2.) for definition of geotechnical zoning shows that this definition is rather made taking into account the higher range of water intake values within each zone (see principle of definition of geotechnical zoning in paragraph 6.4).

Careful examination of the drawings (drawing 1174-03-F20 should logically represent the tests for elevations above 1090 masl, also not mentioned) show that results are actually, very much scattered. Logically, values for definition of the geotechnical zones (those of Figure 6.2 and Figure 6.3), are rather in the higher range, but it shall not be forgotten that, locally, higher values may be registered.

However, the Consortium globally agrees with the approach used by HPT in the definition of the geotechnical zoning, which is relevant with observations on the site.

Values of the geomechanical parameters within the different zones will be discussed farther.

### *8.1.1 Outline of complementary site geotechnical investigations*

In order to cross-check the enormous amount of information available from the 1978 Design Project (Ref.[1]), completed by further geological and geotechnical investigations performed from 2008 on (Ref.[27]), various site visits were organized, during which assessment of geotechnical conditions using the currently well-used GSI classification was performed (see paragraph 7.4.2.5).

For this purpose, a number of the former investigation adits were rehabilited thanks to Rogun HPP staff. Those adits were systematically visited and rock conditions investigated. Nevertheless, in some zones of adverse geological conditions, rehabilitation could not progress smoothly due to permanent collapses.

This is especially the case for the adit 1002, which runs from upstream, close to Ionakhsh Fault, to downstream of the dam site. The portion near the entrance of this adit, immediately on the downstream side of Ionakhsh Fault was collapsed, and could not be visited. The section of the gallery where crossing Fault 35 could not be visited for similar reasons, nevertheless, the downstream section and adit 1063 could be visited (although no light could have been provided in this downstream portion).

Only 25 m from the entrance of adit 1001, in the Ionakhsh Fault, could be cleared and examined. Safety conditions did not allow to clear further.

Other adits visited include:

- Adit 1030, former investigation adit to the underground caverns,
- Adit 1011, a short adit excavated on the left bank, some 200 m downstream of gallery 1030, with portal at similar elevation, around 1000 masl,
- Adit 1034, which portal is located at elevation 1396 masl on top of the right bank, to investigate the eastern limit of the “landslide” or “disturbed zone”; unfortunately, work was still under progress in this part of the adit in November 2012, so that this zone could not be examined in good conditions.

The visited adits and their location is presented on Figure 8.1.

It is to stress that rehabilitation of the former investigation adits is by no way without utility; apart from allowing direct observation of the rock foundation conditions, these adits are to be cleared and plugged with concrete, in order to preserve the integrity of the foundation and avoid that they open way to leakages from the reservoir.

However, to correctly achieve this task, complete clearing of the adits, including the collapsed parts if to be performed.

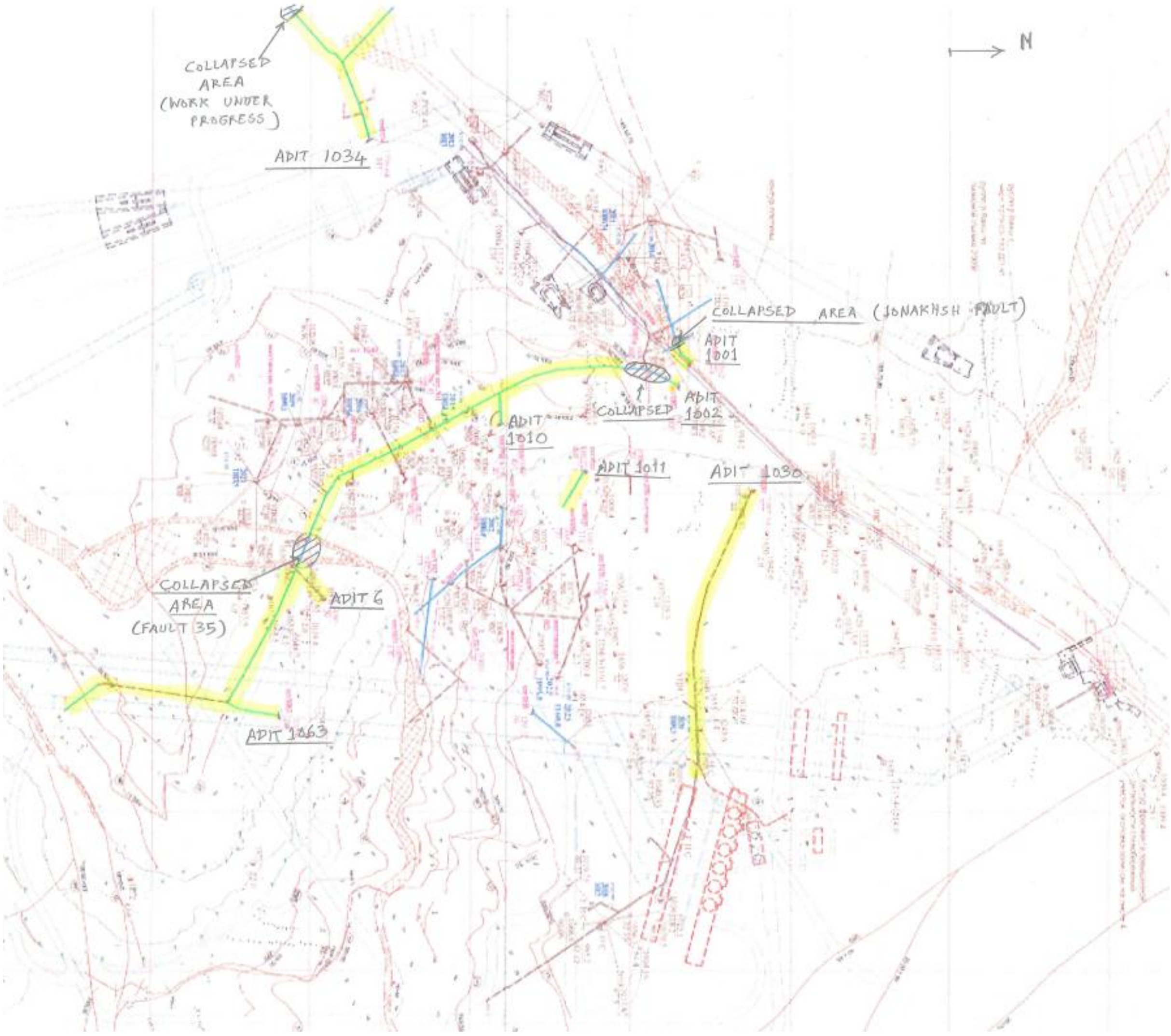



**Figure 8.1: Sketch of the site investigation galleries; the visited portions are underlined in yellow colour**

This information was complemented by the most recent boreholes, for which RQD (Rock Quality Designation) was available, and especially:

- IF1, of 155 m depth, drilled from 1,355 m elevation in right bank, inclined to cross the Ionakhsh Fault, with geotechnical testing of 10 rock samples,
- WRB1, of 110 m depth, drilled from 1,361 m elevation in right bank, vertical, within Kyzyltash Formation, with geotechnical testing of 5 rock samples,
- WRB2/DZ1, of 217 m depth, drilled from 1,537 m elevation in right bank, vertical, within the “disturbed zone”, with geotechnical testing of 8 rock samples,
- DZ2, of 166 m depth, drilled from 1,730 m elevation in right bank, vertical, within the “disturbed zone”.

### *8.1.2 GSI classification and Rocklab deduced parameters*

The GSI approach (Geological Strength Index), introduced by Hoek and other authors, has been used during our different visits to the site for rock formations, in order to get an interesting comparison with values obtained by HPI. The method has been presented above in paragraph 7.4.2.5.

This method shall be used with caution, since it was originally designed for homogeneously jointed rock masses. Secondly, and as the whole of a rock mass properties cannot be reduced to two or three single rates, it is obvious that such a method can only give gives a rough, but generally reasonable order of magnitude of the rock mass properties.

It is therefore obvious that the method works at best for a rock mass "homogeneous in its heterogeneities". It is not to suitable to provide design values without studying the particular features of a rock mass with respect to the works to be constructed. Especially, all possible modes of failures along individual discontinuities are to be considered.

Therefore, detailed studies shall include investigation of the attitudes and characteristics of rock mass discontinuities, as well as the analysis of all possible modes of failure.

We will present in the following the observations made on the geotechnical characteristics of the different rock formations as observed. On this basis, GSI will be estimated and corresponding geotechnical parameters assessed for underground works, dam foundation and slopes, with all reservations as mentioned above.

It is to stress that we did not consider estimation of rock mass deformation modulus by way of GSI classification, since reliability of the corresponding correlation is subject to large variations, but also because a large amount of geophysical prospection has been performed previously.

The obtained parameters will be compared with the previously estimated one, which are the product of extensive investigation work, and in this respect, shall be in neither way disregarded.

Sets of recommended geomechanical parameters for the rock formations of the site will be deduced.

## 8.2 Description of the characteristics of the main formations

### *8.2.1 Alluvial deposits*

Alluvial deposits in the gorge of the dam site consist of boulder and pebble material, with 8 to 12 m thickness. They may include fragments of concrete structures washed out by the river after the burst of the upstream cofferdam of the dam, whose construction had been stopped, in 1993.

Upstream of the dam site, in the reservoir area, the thickness of alluvial deposits is said to reach up to 200 m. The main borrow areas for dam shells are located in this area (borrow areas 15, 15a, Lyabi-Dara).

According to Ref.[1], § 2.5.1, alluvial deposits are mostly made of sand and gravel, together with pebbles and boulders, from which less than 15% is sandstone. The lower part of the alluvium deposits has a high specific weigh, and rocks make about 45% in in weight (10% of the rock pieces have dimension larger than 0.5 m).

According to the same reference, the hydraulic conductivity of the alluvial deposits in their lower part is about 80 m/day, and reduces to 20-30 m/day in the first terrace, down to 12 m/day in the higher terraces which contain some 5% clay particles (1 m/day approximately equals $1.16x10^{-5}$ m/sec).

With regard to shear strength, the alluvial material was assessed with a cohesion of 0.02 to 0.03 MPa and friction angle of 38 to 42 degrees in saturated state. Deformation modulus was estimated to 50 to 80 MPa (Ref. [1], § 2.5.1.).

Downstream of the dam site, alluvial deposits are also more developed than in the gorge, and notably, a large accumulation of debris from the Obi-Shur mudflows is present over the left bank. These mudflow deposits are mainly made of silts and limestone pieces, with very few sandstones. Sandy and clayey material does not represent more than 30-35% in weight of this deposits, and their hydraulic conductivity is estimated at 2 to 5 m/day (same reference as above).

Observations made on site are in agreement with this description.

### *8.2.2 Colluvial and slope wash material, proluvial materials*

Colluvial and slope wash material are generally scarce on site, due to the steepness of the slopes. However, it can be said that where present, there are mostly loose deposits, composed of sandstone and siltstone blocks, silt and clay in various proportions. Hydraulic conductivity is given to be 5 to 100 m/day or more. With regard to shear strength, the colluvial material were assessed with a cohesion of 0.01 MPa and friction angle of 32 to 42 degrees. Deformation modulus was estimated to 15 to 30 MPa (Ref. [1], § 2.5.1.).

A particular mention shall be made of potentially unstable large masses of colluvions, especially on the right bank, downstream of gorge (see Phase II Report - Volume 2 – Chapter 2 - Geology).

The proluvial materials are mainly the debris deposited by the Obi-Shur mud- and debris-flows. Their matrix may be predominantly silty or sandy, with different geomechanical characteristics. Deformation modulus is assessed to some 30 MPa. Shear strength is variable between 0.01 to 0.02 MPa cohesion and 26 to 38 degrees friction angle in saturated state (Ref. [1], § 2.5.1.).

Observations made on site are in agreement with this description.

### *8.2.3 Gaurdak Formation*

The Gaudark Formation pertains to Jurassic, and is made of salt rock in its lower part, the upper part being made of reddish claystones and siltstones.

Salt rock most of the time acts as a décollement layer for the thrusting faults, and its distribution in the dam foundation is limited to Ionakhsh Fault, where a wedge of salt is present, which does not outcrop at the surface (this salt is reported to be also present in the other major thrust faults of the vicinity of the dam site, such as Illiak-Vakhsh Fault and Gulizindan Fault). As already stated in paragraph 2.3 above, one shall refer to Phase 0 Report for more detail about the salt issue.

The claystones and siltstones are visible on site, and were observed in various locations, but especially in the investigations gallery 1034, up the right bank slope, where it intersects Ionakhsh Fault. It can also be seen near to investigation gallery 1001 (only the first 25 m cleared for observation), where rock within Ionakhsh Fault, downstream of the tectonic lens, appears of similar nature.

It appears as reddish claystone to siltstone, of medium hardness, with joint spacing of 10 to 50 mm. Joints are most of the time smooth, and in many locations, joints appeared to have been sealed by fine gypsum (some 1 mm thickness). Figure 8.2 and Figure 8.3 present the Gaurdak claystone as observed in the gallery 1034. From hammer assessment, the overall compressive strength seems not to be over 30 MPa.

No infiltration of water was noticed within the claystone, but observations made outside reveal that these claystones should most likely loose of their mechanical characteristics if distressed and submitted to weathering.

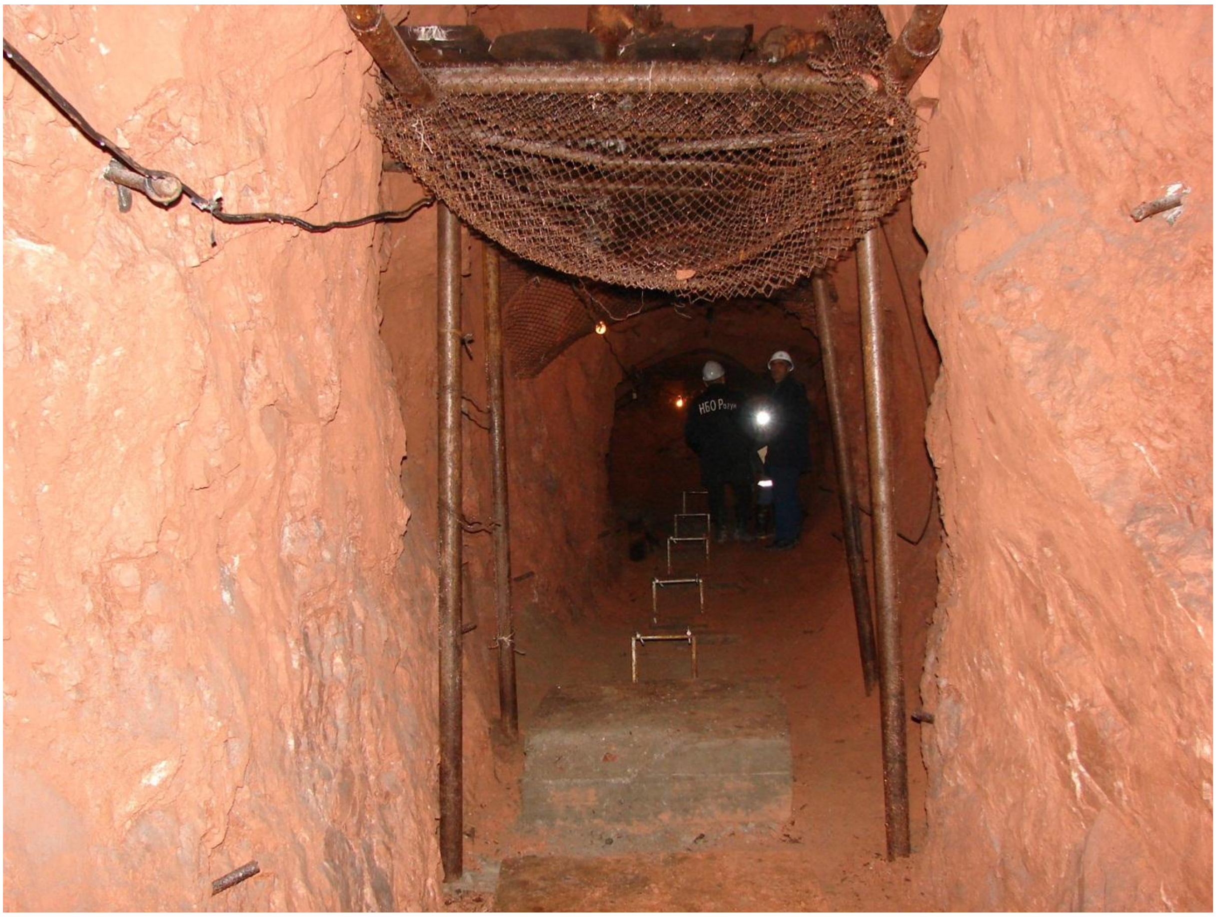


**Figure 8.2: View of instrumentation adit excavated just south of the Ionakhsh Fault, from gallery 1034, within Gaurdak claystone**

Even if assessed within a large range of 25 to 50 (due to some more competent levels, and inversely quite soft layers), the overall GSI for the rock mass is assessed to 35 to 40.

### 8.2.4 Yavan Formation

Rocks of the Yavan Formation were observed in gallery 1034 and in gallery 1002, both on the right bank. They are made principally of reddish-brown siltstones, with interlayers of sandstones. Joints are most of the time persistent, sometime with millimetric, soft clayey infilling. Some slickensided surfaces were observed along bedding joints. The compressive strength of the rock is assessed around 40 to 45 MPa (siltstones), and the GSI estimated within a range 30 to 40.

**Figure 8.3: Close view of Gaurdak claystone and quasi-systematic, fine gypsum encrusting of discontinuities (gallery 1034)**

### *8.2.5 Kyzyltash Formation*

This formation is one of the most important on site, with 200 m thickness. It makes the foundation or embedding rock of a large part of the works on the upstream side. Boreholes from the 2006 campaign (see Ref. [20]) and boreholes WRB1 in 2012 were drilled mainly in this formation. It is mainly constituted of fine-grained, reddish brown micaceous sandstones, with interlayers of reddish siltstone or claystone.

On the left bank, the rocks could be observed in gallery 1030 during our site mission of April 2012, and also in gallery 1011 in November 2012. This was completed in right bank by the visit of gallery 1002 and gallery 1034.

The rock matrix itself is fine-grained, rather hard and competent, but apart from the very continuous bedding joints, at least two other joint sets are present with similar frequency, conferring to the rock a quite fractured aspect. Within these joint sets, the subhorizontal ones are clearly observable, with horizontal slabs in vault galleries.

Some subhorizontal fractures were observed infilled with some 5 to 10 cm clayey material (see Figure 8.4).

When visiting the gallery 1030 in April 2012, water was dripping in many places from the joints, depositing fine layers of clayey material on the walls of the fractures. Even considering that we were in snow-melting period, which would mean that actual permeability is medium, and may be rather high in the direction of the bedding joints. Based on these observations, the assumption of a global permeability of 0.2 LU (some $2x10^{-8}$ m/sec) seems to be underestimated. From observation, a value of some $10^{-6}$ m/sec appears more adequate.

**Figure 8.4: Kyzyltash Formation in investigation gallery 1030; one will note the subhorizontal joint with clay infilling and general dampness of the rock**

Moreover, suffusion phenomenon is to be checked, since water circulations may alter the joints with slow entrainment of joint infilling or joint alteration material. Fine, liquid red clay can be seen slowly taken along the joints of the rock where circulation of water occurs (likely from the works), a phenomenon that can be noticed in the transportation tunnels as well. A good example is presented in Figure 8.5.

From observations made, and using both GSI charts and calculation of RMR values, the corresponding GSI was estimated as ranging from 45 to 55.

**Figure 8.5: View of water circulations within a branch of investigation gallery 1002; red clay is clearly seen being taken along by water along the joints, then depositing on the floor**

### *8.2.6 Lower Obigarm siltstones*

The Lower Obigarm siltstones were observed in gallery 1002, in the right bank. With some 90 m thickness, it makes the foundation of the dam core, and is made almost exclusively of siltstones.

In the gallery 1002, where it was observed, the siltstones appear as a closely –jointed, brown to reddish-brown rock (joint spacing 50 mm average), with most of the time smooth to very smooth walls. Infilling, where present, consist in a very fine clay layer of less than 1 mm thickness. Some more closely jointed sections appear as well, which have been assessed separately, as claystones. They are very close to the siltstones in colour and pattern, but the rock matrix is much softer, and much sensitive to slaking than embedding siltstones (these are probably the claystones Lahmeyer referred to, see paragraph 7.4.2.5).

Water was observed dripping along clayey joints and contacts between siltstones and claystones.

The rapid alteration of the siltstones when distressed and exposed has been emphasized in paragraph 6.5.2 and 7.4.2.2, and the gallery had effectively to be supported with wood structures in some places, dampened with water. In such conditions, the siltstones apparently lose part of their mechanical characteristics. However, from the observations, the siltstones where stressed and confined seem not to be too much affected by this weathering.

**Figure 8.6: Lower Obigarm siltstones in gallery 1002**

Gypsum is reported as making 2% of the siltstones, concentrated along joints, but very few could be observed in the gallery.

The assessed GSI values are in the range of 37 to 43, the lower value corresponding to the more loosely jointed siltstones, close to claystone facies.

From observations related above, it is clear that the siltstone massif, when highly stressed, like it is normally under the stress conditions at Rogun, although fair rock, is adequate for excavations, but strain-softening of the siltstones is to be accounted for. If distressing occurs due to insufficient support pressure (like it is the case in the power house excavations for the siltstone part), the rock mass loses rapidly strength, and even more in presence of water.

### *8.2.7 Upper Obigarm sandstones*

The Upper Obigarm sandstones, as observed both in outcrops and in gallery 1002, appear as massive, hard, fine-grained sandstones. Joint spacing seems to be slightly more than in the other formations (especially compared to Kyzyltash, which contains frequent interlayers of siltstones).

The compression strength of the rock is high (around 100 MPa) and assessed GSI values range from 48 to 67, with an average around 60. It is therefore a rock of quite favourable conditions for underground works.

### *8.2.8 Karakuz and Mingbatman Formations*

The Karakuz and Mingbatman Formations were observed in gallery 1002 and 1063, however, in this downstream part of the gallery, no lightning other than portable torch lamps was available, therefore, observation could not be so detailed as above.

It is worth to note that a continuous gypsum layer of some 200 to 300 mm thickness exists within this formation, very close to the lower contact with the underlying Upper Obigarm sandstone. This layer is remarkably useful to follow the contact on site.

These two formations consist in intercalations of grey to brown sandstones with siltstones of similar colour. Sandstones are hard, but somewhat different from Upper Obigarm and Kyzyltash one. They seem sometime, at least in some areas like the portal of gallery 1053, less cemented, although the overall mechanical strength is good, with compression strength almost similar to Upper Obigarm sandstones.

Siltstones sections are of course of different mechanical characteristics, but appear generally more favourable than siltstones of Lower Obigarm and Yavan Formations.

GSI for sandstones of these formations was assessed around 50.

### *8.2.9 Other rock formations*

The other rock formations could not be observed in the galleries, since Lyatoban is only present in some works of the lower left bank. However, their overall geological description is given in Phase II Report - Volume 2 – Chapter 2 - Geology.

## 8.3 Characteristics of discontinuities of the rock

### *8.3.1 Bedding joints*

The bedding joints in the different formations are generally very persistent and underlined by the interbedding of sandstone and siltstone, which globally makes the great part of the rock masses on site.

One common characteristics, which could be observed in the galleries, is that contact between siltstone and sandstone is often damp or even dripping in some locations, with very fine clay coating. Joints in such conditions can be considered as smooth, with a friction angle assessed around 30 to 35 degrees.

Within sandstones, the bedding joints are of course much more rough, and friction angle probably high, of the order of 45 degrees or more.

### *8.3.2 Other joint families*

As already stressed in paragraph 2.1., the peculiarity of the site is that a large number of cracks appear as especially fresh, sometime exhibiting relative displacements (like for the sub-horizontal joints of the lower left bank cliff, reported in the Geological Report).

Table 6.8 presented the four main directions of jointing on site.

Some complementary information is given in Ref. [27], HPI having achieved a quite detailed and comprehensive study of the fracturation pattern of the rock masses in different sectors, and especially in Lower and Upper Obigarm Formations.

Especially joints of the set 4 have been found to be very scattered, from 280 to 350 in dip azimuth with dip ranging between 25 and 90 degrees. Additionally, HPI (Ref. [27], § 1.3.1.) distinguishes a fifth joint set in Upper Obigarm sandstones, for discontinuities which dip angle is lower than 25 degrees, rather well developed there (near to the power house location).

This fits with the observation made in outcrops, and also in the investigation galleries inspected, where several faults and discontinuities of various dip angle (from sub-horizontal to 45 degrees) where encountered, with dip azimuth similar as Fault 35 and joint set No.4. One of the common features of these discontinuities is that, where very persistent and planar, they intersect all rock formations and are often infilled with plastic clay (up to several centimetres thickness). These observations were already reported in the Original Design Report (see paragraph 6.6.1.2). A typical example of such joints, here rather a fault, is showed in Figure 8.7.

**Figure 8.7: Fault similar in dip azimuth as Fault 35, with some 50 mm infilling of pure plastic clay (investigation gallery 1002, downstream of Fault 35)**

Such joints are of prime importance, since shear strength along them will be quite reduced (25 to 30 degrees friction angle was assessed, with possibly residual characteristics still to be reached). They will have to be taken into account in detail design of all works which may be affected by possible block sliding along this direction.

Figure 8.8 shows one of these main discontinuities (probably Fault 32, as per original drawing 1174-03-78, Sheet 1 of Ref.[1]) cutting both the Lower Obigarm siltstones of the dam core foundation and the Upper Obigarm sandstones.

**Figure 8.8: Fault of similar attitude as Fault 35, dipping towards upstream, cutting the Lower Obigarm siltstone and Upper Obigarm sandstone; in the background; other joints pertaining to the same system are clearly visible (bedding attitude steeply dipping towards upstream)**

The determination of the shear strength along the different categories of joints is a very important data to be used as input in wedge stability calculations. Although general estimations are presented in Table 6.3 and the followings, it is felt that specific shear tests should be carried out to cross-check the data.

Direct shear tests, which may be made in-situ, according to ISRM suggested methods, and laboratory shear tests on intact samples are therefore recommended to be carried out to determine the input data for the detailed studies to follow. Such tests were already recommended in 2005 (see Ref. [18], § 4.2.4.2.) for the power house embedding rocks, but should be extended in our opinion to other discontinuities over the site.

### *8.3.3 Fault 35*

Unfortunately, Fault 35 could not be observed, apart from its upstream and downstream sides, since continuous rockfalls were hampering clearing operations of gallery 1002 at this location. Large amounts of collapsed reddish-brown clay and presence of water was observed.

This fact, combined with the collapse occurred in the diversion tunnels where crossing Fault 35, demonstrates that adverse geological conditions are to be forecasted there, and over the whole tectonic lens between the branches of the fault. Figure 8.9 shows the jointing and overall presence of clay in this area (on the photograph, just upstream the northern branch of the Fault).

**Figure 8.9: View of the rock just upstream the northern branch of Fault 35; fine clay is present along most of the joints (gallery 1002)**

Moreover, since the fault was measured slowly creeping, the problem of maintaining the integrity of the works everywhere intersected by this fault is to be solved.

### *8.3.4 Ionakhsh Fault*

The investigation gallery 1001 allowed the observation of the fault infilling and its contact with the embedding rock in the right bank.

However, the gallery could not be cleared more than 25 m, because of collapses in presence of water, with abundant reddish clay in the fault zone itself. Rock within the fault appears as a compact, laminated rock with gypsum scattered as pieces of thin seams. Gaurdak claystones appear therefore constituting most of the fault breccia, downstream of the tectonic lens of sheared sandstones and siltstones from the upstream geological formations.

The compacity of the breccia is still more evident from the visit of the grouting galleries (for salt leaching mitigation) of the right bank, where it can be observed as well.

No salt rock is visible there, since elevation of the top of the wedge is below the river level.

**Figure 8.10: Breccia of Ionakhsh Fault, in gallery 1001, right bank**

Some precaution is however to be taken when intersecting Ionakhsh Fault, since, as other claystones of the site, they may rapidly lose strength when distressed and in presence of water.

According to Ref. [27], § 1.7, tunnel No.3, encountered, after crossing the tectonic lens on the upstream side, that the fault plane was 1 to 3 m thick, with pieces of rock with argillaceous material. The rock was crushed over a thickness of 5 to 7 m.

## 8.4 Assessment of geomechanical parameters of the rock masses

### *8.4.1 Tentative assessment*

Observations made by the Consortium of the different formations appear in line with the site assessment made already since 1978 by HPT.

A tentative assessment of geomechanical characteristics, and above all shear strength of the rock masses was made for comparison with the previous data.

Table 8.1 presents the rock mass parameters m, s and a for definition of the Hoek & Brown failure criterion (see Ref. [21]) and corresponding shear strength characteristics for dam foundation under maximum height of the dam, for slopes and tunnels (200 and 400 m depth). Note that these values are only indicative, since they result from on-site assessment and recent boreholes.

We took into account in this assessment the data from 2006 boreholes (see Ref. [20]) and results from boreholes IF1 and WRB1 (see Ref. [36] and Ref. [37]), drilled in 2012.

It is to emphasise that shear strength values are based on estimation as per Hoek's approach, with a parabolic failure criterion defined by the parameters m, s and a. If using a Mohr-Coulomb failure criterion, values of cohesion and friction angle depend upon the range of stress. Therefore, different values are listed here for dam foundation, slopes and underground works (200 m and 400 m depth considered).

### *8.4.2 Comparison of results from the different studies*

We attempted to summarise, for the most important geomechanical data of rock masses on site, the results of the different studies performed, in Table 8.2. It is to stress that we considered here the only parameters for Zone IV, especially dedicated to underground works (considering, for back-analysis results of the power house, values within zones remaining elastic and undamaged).

Although some differences exist, due to the methods of assessment, and sometime, the type of rock considered, there appear to be a global coherence between the different results, at least in term of order of magnitude.

The only notable exception is the shear strength of the rock mass, systematically higher in the Original Project from HPT. The way those parameters were defined is not quite clear, but it is for sure quite different that the assessment made by means of GSI estimation, and should explain, in our opinion this difference.

It can also be stressed that, considering the power house excavations and the results of the different back-analysis studies performed, deformation modulus (of undamaged rock) should however be somewhat reduced for the two components of the Obigarm Formation

| | | | | | | ROCKLAB results | | | Dam foundation (8MPa) | | Slopes 300m | | Tunnels 400m depth | | Tunnels 200m depth | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Rock formation | Rock | SigmaCi | GSI | Q | mi | m | s | a | Cohesion | Friction angle | Cohesion | Friction angle | Cohesion | Friction angle | Cohesion | Friction angle |
| | | (MPa) | | | | | | | (MPa) | (degrees) | (MPa) | (degrees) | (MPa) | (degrees) | (MPa) | (degrees) |
| Gaurdak | Claystone | 20 | **36** | **0,167** | 4 | 0,407 | 0,0008 | 0,515 | 0,8 | 17 | 0,6 | 20 | 0,5 | 20 | 0,4 | 25 |
| | Claystone | 20 | **41** | **0,333** | 4 | 0,486 | 0,0014 | 0,511 | 0,8 | 18 | 0,6 | 21 | 0,6 | 22 | 0,4 | 26 |
| Yavan | Claystone | 20 | **26** | **0,037** | 4 | 0,285 | 0,0003 | 0,529 | 0,6 | 14 | 0,4 | 17 | 0,4 | 18 | 0,3 | 22 |
| | Siltstone | 45 | **40** | **0,667** | 7 | 0,821 | 0,0013 | 0,511 | 1,4 | 27 | 1,1 | 30 | 1,0 | 32 | 0,7 | 37 |
| | Siltstone | 40 | **39** | **0,083** | 7 | 0,791 | 0,0011 | 0,512 | 1,3 | 26 | 1,0 | 29 | 0,9 | 30 | 0,6 | 36 |
| Kyzyltash | Sandstone | 55 | **44** | **0,333** | 17 | 2,301 | 0,0020 | 0,509 | 2,0 | 38 | 1,6 | 41 | 1,5 | 42 | 1,0 | 47 |
| | Claystone | 20 | **32** | **0,111** | 7 | 0,617 | 0,0005 | 0,520 | 0,9 | 19 | 0,6 | 22 | 0,6 | 23 | 0,4 | 28 |
| | Sandstone | 80 | **47** | **0,444** | 17 | 2,561 | 0,0028 | 0,507 | 2,4 | 42 | 2,0 | 44 | 1,8 | 46 | 1,2 | 51 |
| | Sandstone, sometime clayey | 50 | **37** | **0,667** | 15 | 1,581 | 0,0009 | 0,514 | 1,7 | 34 | 1,3 | 37 | 1,2 | 38 | 0,8 | 43 |
| | Sandstone, sometime clayey | 50 | **37** | **0,667** | 17 | 1,792 | 0,0009 | 0,514 | 1,8 | 35 | 1,4 | 38 | 1,3 | 39 | 0,8 | 44 |
| | Sandstone, clay films | 50 | **41** | **0,667** | 17 | 2,067 | 0,0014 | 0,511 | 1,9 | 36 | 1,5 | 39 | 1,4 | 40 | 0,9 | 46 |
| | Sandstone | 80 | **54** | **1,333** | 17 | 3,288 | 0,0060 | 0,504 | 2,7 | 44 | 2,3 | 46 | 2,0 | 48 | 1,4 | 53 |
| Lower Obigarm | Claystone | 35 | **37** | **0,188** | 4 | 0,422 | 0,0009 | 0,514 | 1,0 | 20 | 0,7 | 23 | 0,7 | 24 | 0,5 | 29 |
| | Siltstone | 45 | **43** | **0,111** | 7 | 0,914 | 0,0018 | 0,509 | 1,4 | 28 | 1,1 | 31 | 1,0 | 32 | 0,7 | 38 |
| Upper Obigarm | Sandstone | 100 | **61** | **5,333** | 17 | 4,222 | 0,0131 | 0,503 | 3,4 | 48 | 2,9 | 50 | 2,6 | 51 | 1,9 | 56 |
| Karakuz | Sandstone | 80 | **61** | **2,000** | 17 | 4,222 | 0,0131 | 0,503 | 3,0 | 46 | 2,6 | 48 | 2,4 | 50 | 1,7 | 54 |
| Mingbatman 1 | Sandstone | 60 | **49** | **4,667** | 17 | 2,751 | 0,0035 | 0,506 | 2,3 | 40 | 1,9 | 43 | 1,7 | 44 | 1,1 | 49 |
| Mingbatman 4 | Sandstone | 70 | **51** | **3,500** | 17 | 2,954 | 0,0043 | 0,505 | 2,5 | 42 | 2,0 | 44 | 1,8 | 46 | 1,2 | 51 |

**Table 8.1: Assessment of shear strength of rock masses by Rocklab software, assuming a homogeneous fractured rock massif; assessment is made for dam foundation, slopes and tunnels (200 and 400 m depth)**

| Rock Formation | Mode of assessment | Sample strength | | Rock mass characteristics | | | | | Shear strength (peak characteristics if residual are mentioned) | | Shear strength (residual characteristics, where assessed) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Compression strength (saturated) | Tensile strength | Density | Longitudinal wave velocity | Hydraulic conductivity | Deformation modulus | Poisson ratio | Cohesion | Friction angle | Cohesion | Friction angle |
| | | (MPa) | (MPa) | (kN/m3) | (km/sec) | (UL) | (MPa) | | (MPa) | (degree) | (MPa) | (degree) |
| **Gaurdak** | | | | | | | | | | | | |
| Original design, 1978 | | 6,2 | | 24,4 | 3,2 | 0,2 | 3 000 | 0,28 | 0,5 | 56 | | |
| GRC, 2005 | Geophysics | | | | 2,9 | | | | | | | |
| Consortium, 2012 | GSI, Rocklab approach | | | | | | | | 0,5 | 20 - 26 | | |
| **Javan** | | | | | | | | | | | | |
| Original design, 1978 | | 40 | | 27 | 3,3 | 0,1 | 4 000 | 0,28 | 1,5 | 63 | | |
| HPI 2012, Hydrogeological modelling | | | | | | 0,9 | | | | | | |
| Consortium, 2012 | GSI, Rocklab approach | 20 - 45 | | | | | | | 0,8 - 1 | 30 - 35 | | |
| **Kyzyltash** | | | | | | | | | | | | |
| Original design, 1978 | | 102 | | 26,2 | 3,7 | 0,1 | 8 000 | 0,31 | 2 | 67 | | |
| GRC, 2005 | Geophysics | | | | 3,8 | | | | | | | |
| GRC, 2006 | Geophysics | | | | | | 9 000 | | | | | |
| HPI, 2009 | Back-analysis and geophysics | | | | | | 8 000 | 0,24 | 2 | 55 | | |
| Savich, 2007 | | | | | | 0,9 | | | | | | |
| Consortium, 2012, sandstone | GSI, Rocklab approach | 50 - 80 | | | | | | | 1 - 1,4 | 40 - 45 | | |
| **Lower Obigarm siltstones** | | | | | | | | | | | | |
| Original design, 1978 | | 57 | | 27,1 | 3,8 | 0,1 | 5 500 | 0,33 | 1,5 | 63 | | |
| GRC, 2005 | Geophysics | | | 26,5 | 3,5 - 3,8 | 0,1 | 5 500 | | | | | |
| GRC, 2006 | Geophysics | | | | | | 8 000 | | | | | |
| Hisdrospetsproyekt, 2005 | Back-analysis power house, MARC | 87,5 | 8,75 | | | | 3 700 | 0,3 | 2,5 | 51 | | |
| Hisdrospetsproyekt, 2005 | Back-analysis power house, Phase 2 | | | | | | 5 500 | 0,3 | 2 | 45 | 1,5 | 42 |
| Lahmeyer, 2005 | Back analysis power house, no distressed zone | | | | | | 4 300 | | 1,2 | 36 | 0,88 | 30 |
| Savich, 2007 | Back-analysis power house Z-soil | | | | | | 4 000 | 0,3 | 0,75 | 36 | | |
| HPI, 2009 | Back-analysis power house, tensile strength considered | | | | | | 4 000 | 0,3 | 1,2 | 40 | | |
| HPI, 2009 | Back-analysis and geophysics | | | | | | 5 500 | 0,28 | 1,2 | 45 | | |
| HPI 2012, Hydrogeological modelling | | | | | | 0,001 | | | | | | |
| Consortium, 2012 | GSI, Rocklab approach | 45 | | | | | | | 0,5 - 1 | 25 - 35 | | |
| **Upper Obigarm sandstones** | | | | | | | | | | | | |
| Original design, 1978 | | 111 | | 26 | 3,6 | 0,3 | 9 000 | 0,3 | 2 | 67 | | |
| GRC, 2005 | 1989 data? | | | 26,5 | 3,8 | 0,3 | 9 000 | | | | | |
| Hisdrospetsproyekt, 2005 | Back-analysis power house, MARC | 125 | 12,5 | | | | 6 000 | 0,22 | 3,75 | 56 | | |
| Hisdrospetsproyekt, 2005 | Back-analysis power house, Phase 2 | | | | | | 9 000 | 0,22 | 3 | 50 | 2 | 47 |
| Lahmeyer, 2005 | Back analysis power house, no distressed zone | | | | | | 6 000 | | 1,5 | 45 | 1,2 | 40 |
| Savich, 2007 | Back-analysis power house Z-soil | | | | | | 7 500 | 0,22 | 1,64 | 42 | | |
| HPI, 2009 | Back-analysis power house, tensile strength considered | | | | | | 7 500 | 0,22 | 2 | 50 | | |
| HPI, 2009 | Back-analysis and geophysics | | | | | | 9 000 | 0,22 | 2 | 55 | | |
| HPI 2012, Hydrogeological modelling | | | | | | 0,9 | | | | | | |
| Consortium, 2012 | GSI, Rocklab approach | 100 | | | | | | | 2 - 2,5 | 50 - 55 | | |
| **Karakuz** | | | | | | | | | | | | |
| Original design, 1978 | | 53 - 66 | | 26 | 3,6 | 0,2 | 7 000 | 0,32 | 2 | 63 | | |
| HPI, 2009 | Back-analysis and geophysics | | | | | | 7 000 | 0,26 | 2 | 50 | | |
| HPI 2012, Hydrogeological modelling | | | | | | 0,9 | | | | | | |
| Consortium, 2012 | GSI, Rocklab approach | 80 | | | | | | | 1,7 - 2,4 | 50 - 54 | | |
| **Mingbatman** | | | | | | | | | | | | |
| Original design, 1978 | | 82 | | 26 | 3,6 | 0,3 | 8 000 | 0,32 | 2 | 67 | | |
| GRC, 2005 | Geophysics | | | | 3,8 | | | | | | | |
| HPI 2012, Hydrogeological modelling | | | | | | 0,9 | | | | | | |
| Consortium, 2012, sandstone | GSI, Rocklab approach | 70 | | | | | | | 1,2 - 1,8 | 45-50 | | |
| **Lyatoban** | | | | | | | | | | | | |
| Original design, 1978 | | | | | | | | | | | | |
| GRC, 2005 | Geophysics | | | | 3 | | | | | | | |
| **Ionakhsh Fault** | | | | | | | | | | | | |
| Original design, 1978 | | 10 - 70 | | 26 | 3,3 | 0,2 | 2 000 | 0,23 | 0,5 | 56 | | |
| HPI 2012, | | | | | | 0,001 | | | | | | |
| **Fault 35** | | | | | | | | | | | | |
| Original design, 1978 | | 10 - 70 | | 26 | 3,3 | 0,2 | 2 000 | 0,23 | 0,5 | 56 | | |
| HPI, 2009 | Back-analysis and geophysics | | | | | | 1 000 | 0,32 | 0,05 - 0,3 | 27 - 37 | | |
| HPI 2012, Hydrogeological modelling | | | | | | 0,001 | | | | | | |
| **Fault 70** | | | | | | | | | | | | |
| HPI, 2009 | Back-analysis and geophysics | | | | | | 2 000 | 0,32 | 0,05 - 0,3 | 27 - 37 | | |

**Table 8.2: Summary recapitulation of the different assessments performed**

For the detailed studies, and for all rock mechanics issues, it will be necessary to study the potential modes of failures along joints. This is why assessment of shear strength of joints is a very important input data for further studies. Assessment made previously are summarised in Ref. [12], and presented as originating from updating of the Original Project of 1978. Data from the latter on joint infillings and presented data for the other sets are presented in Table 8.3.

| Shear strength on discontinuities | Cohesion | Friction angle |
|---|---|---|
| | (MPa) | (degree) |
| **Stratigraphic** | | |
| 2005, Ref. [12] | 0,03 | 29 |
| **Set 2** | | |
| 2005, Ref. [12] | 0,02 | 32 |
| **Set 3** | | |
| 2005, Ref. [12] | 0,02 | 32 |
| **Set 4** | | |
| 2005, Ref. [12] | 0,02 | 32 |
| **Joint infilling (1978)** | 0,02 | 26 |

**Table 8.3: Previously assessed shear strength properties along joints**

For faults, Ref. [28] (reproduced Table 7.13) states slightly different values, with shear strength estimated by a 0.01 MPa cohesion and 26 degree friction angle, i.e. slightly lower than proposed here. The dam foundation being more distressed than in depth, those values are reasonable as well. However, this stresses the need for further tests to confirm those values.

A deformation modulus of 1,500 MPa is assessed for crushed rock within fault zones.

### *8.4.3 Suggestion of geomechanical parameters for further studies*

#### *8.4.3.1 Rock and rock mass characteristics*

For most of the main parameters, and even if the way they were obtained may not be the same as the one currently used outside of the Russian Federation, geomechanical parameters as previously assessed in the Original Project and subsequent HPI documents appear to fit generally rather well to the site conditions.

It is however recommended, in case of international bidding, to perform a campaign of geotechnical testing according to international standards.

Given the information we have at present, and considering the convergences experienced by the power house excavation, as well as some of the additional investigations, somewhat simplified parameters are proposed in Table 8.4.

It is however to stress that this table applies only for rock masses considered as homogeneous in their heterogeneities. More precisely, the failure mode along a preferential joint set at the scale of the considered work should not be preponderant. Typically, this applies for underground works and the foundation everywhere it can be considered as not substantially distressed by proximity to the surface (some 30 to 40 m below surface, see Figure 6.2 and Figure 6.3).

In comparison with values of HPT (right side of Table 6.3), the deformation modulus of the Lower and Upper Obigarm Formations have been reduced on the basis of the diverse back-analysis of power house convergences. We tried as much as possible to homogenise the other rock formations, for commodity.

One will note that the failure criterion for Kyzyltash, Karakuz and Mingbatman Formations are same; all three formations have in common to be made mostly of sandstones.

With regard to geomechanical characteristics in distressed zones, either the HPT values (Table 6.4, Table 6.5 and Table 6.6) can be used, or reduced characteristics, e.g. using the D factor of Hoek. It is important to keep in mind that strain softening within siltstones and claystones (especially for Lower Obigarm and Gaurdak) is probably substantial, and that residual characteristics of a distressed rock mass around an underground excavation are to be lowered with regard to the peak one presented in Table 8.4.

It is therefore important to state that the presented suggestion of geomechanical parameters is no more applicable to embedding rocks of the power house complex, since they have been distressed, and therefore reached their residual characteristics due to the peculiarity of the situation there (see paragraphs 7.4.2.4 and 7.4.2.5).

*8.4.3.2 Shear strength of discontinuities*

In the absence of other data, shear strength characteristics of Table 8.3 were considered, and the order of magnitude of the values seems us quite relevant. However, in presence of continuous plastic clay infilling along persistent discontinuities, joint infilling characteristics are to be considered.

| Rock Formation | Sample strength | | Rock mass characteristics | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Compression strength (dry) | Compression strength (saturated) | Density | Deformation modulus | Poisson ratio | Considered compression strength | GSI | $m_i$ | m | s | a |
| | (MPa) | (MPa) | (kN/m3) | (MPa) | | (MPa) | | | | | |
| **Gaurdak** | 16 | 6 | 25 | 3 000 | 0,3 | 6 | 35 | 4 | 0,393 | 0,0007 | 0,516 |
| **Javan** | 68 - 101[(1)] | 37 - 86[(1)] | 27 | 4 000 | 0,3 | 50 | 35 | 7 | 0,687 | 0,0007 | 0,516 |
| **Kyzyltash** | 118 - 126 | 80 | 26 | 8 000 | 0,3 | 80 | 50 | 15 | 2,515 | 0,0039 | 0,506 |
| **Lower Obigarm siltstones** | 59 | 57 | 27 | 5 000 | 0,33 | 57 | 40 | 7 | 0,821 | 0,0013 | 0,511 |
| **Upper Obigarm sandstones** | 120 | 100 | 26 | 8 000 | 0,3 | 100 | 60 | 17 | 4,074 | 0,0117 | 0,503 |
| **Karakuz** | 80 - 100 | 53 - 66 | 26 | 7 000 | 0,3 | 60 | 50 | 15 | 2,515 | 0,0039 | 0,506 |
| **Mingbatman** | 110 - 121 | 75 | 26 | 7 000 | 0,3 | 75 | 50 | 15 | 2,515 | 0,0039 | 0,506 |
| **Ionakhsh Fault** | | 10 - 70 | 26 | 2 000 | 0,23 | | | | | | |
| **Fault 35 and others (70,..)** | | 10 - 70 | 26 | 2 000 | 0,3 | | | | | | |

[(1)] Upper values refer to sandstones contained within the Upper Javan formation

**Table 8.4: Suggested geomechanical parameters for rock masses (outside obviously distressed zone)**

For detailed studies, and especially rock wedge or rock slides stability analysis, investigation and determination of shear strength along specific discontinuities will be required. They are recommended to be performed according to International Standards, keeping in mind that availability of such results may be of prime importance for any future international bidding.

### *8.4.4 Geomechanical characteristics for embedding rocks of the power house complex*

The actual situation and evolution of the movements of the power house cavities and neighbouring excavations have evidently led to progressive distressing of the embedding rocks, thereby impacting their original geomechanical properties.

This situation, which may be partly due to lack of timely applied support at the moment of the construction, has been persisting, and movements once apparently stabilised at a steady rate resumed increasing by 2008, when work resumed in the power house.

One will refer to the report dealing with underground works for more details; evolution of the movements is quite preoccupant that is why an independent 2D model is part of the Phase I assessment of existing structures. This model is aimed at verifying the possibility, if any, of improving the wall anchoring system in order to achieve stable conditions for the power house and the whole complex of caverns.

It is to be noted that abandoning the siltstone part of the power house is one of the possible scenarios envisaged by previous designers, as already shown by some of the back-analyses performed previously (see paragraphs 7.4.2.4 and 7.4.2.5). This issue is being discussed in details in the Phase I report including the powerhouse cavern assessment.

## 8.5 Conclusion and recommended complementary investigations for further stages of study

We think that the suggested parameters of Table 6.4, as well as most of the one determined by HPT previously (with the reservation of deformation moduli for Obigarm formations, and some compressive strengths on intact rock, which were found apparently high), are sufficient for the pre-feasibility studies. However, those parameters are no more applicable in the vicinity of the power house complex, where distressing has substantially affected the mechanical properties of the rock masses. Same remark is to be done for other locations where the rock has been significantly distressed, e.g. at the intersection of the construction tunnels with Fault 35, where rock collapses when up to more than 20 m above the tunnel vault.

For the further stage of study of the Project, more detailed investigations of the different parts of the foundation with regard to the projected works, once their definitive layout decided, are in our opinion necessary. This shall include detailed geological investigations, and the Consortium is the opinion that an additional campaign of geotechnical tests should be carried out in order to precise the geomechanical parameters to be considered for the different components of the works.

With regard to the rock foundation itself, basic identification tests and uniaxial compression tests and triaxial tests on rock samples would be welcome to cross-check the 1978 data. A better characterisation of the Lower Obigarm siltstone, especially for creeping behaviour is recommended, and in-situ deformability tests may be carried out for the underground works. The Petite Sismique method (Scarabee method) may also be a good compromise for assessing by simple geophysical method the static deformation modulus of the rock masses (to be done from galleries).

It is however in our opinion, as already stressed above, to better investigate the shear strength of joints, since adequate input data will be needed wherever potential sliding along a joint can occur (slope stability on the upper left bank, for instance). This can be done either by in-situ tests according to ISRM suggested methods on definite locations, but more systematically on samples taken from the field and tested in shear box.

# 9 HYDROGEOLOGY OF THE SITE

## 9.1 Summary of hydrogeological investigations

As already stated, a very large number of water tests was performed during the preliminary investigations, before 1978. During the following years, additional tests were conducted (see paragraph 4.1).

Results of water tests notably allowed defining the geotechnical zoning of the site, as presented in paragraph 6.4.

With regard to investigations of water table levels and their variations, the original groundwater levels map has been generated from rather scarce piezometric data, assuming that groundwater contour lines run more or less parallel to the elevation lines, as illustrated by Figure 6.5. A number of observation wells were located in the vicinity of the salt wedge of Ionakhsh Fault, in order to investigate the local groundwater regime there.

Further investigations followed during the construction, and in 2008, additional observation wells were made available, but concentrated along the Ionakhsh Fault, while some of the previous ones were abandoned during the process of underground excavation works.

Very recently, at the end of 2012, 19 additional observation wells were drilled on the demand of the Consortium throughout the site, within the underground works, to establish the behaviour of the site aquifer. Location of those piezometers is shown on Figure 9.1.

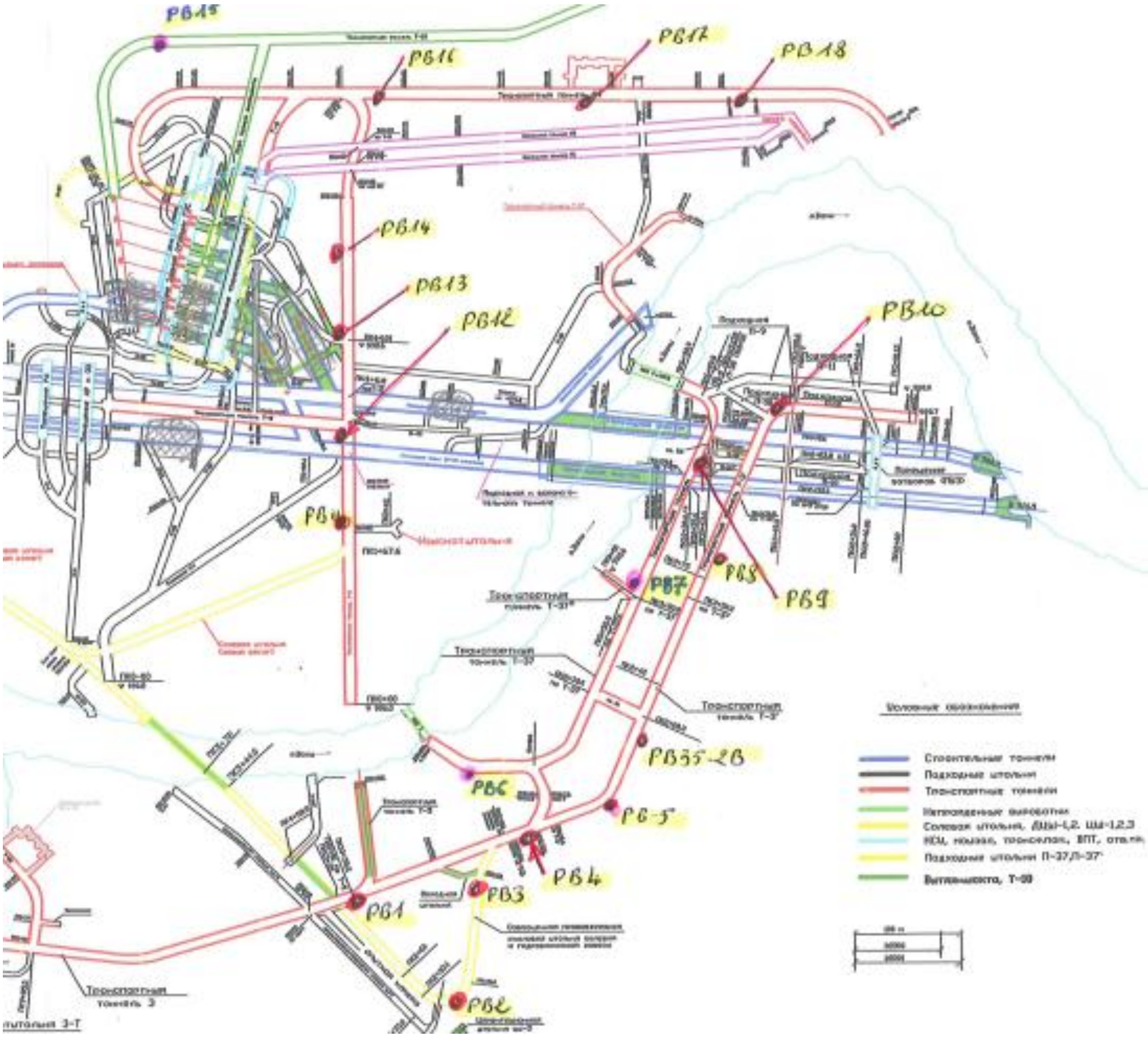



**Figure 9.1: Location of additional piezometers drilled in 2012**

Additionally, all springs from the aquifers of the Project area were identified and characterised, in term of discharge and hydrochemistry as well. Most of these springs were evidently known from the early stage of design, but data about discharge and chemical composition were very scarce). The most outstanding of these springs is located on the right bank, with only some of them permanent (e.g. spring No. 11 at elevation 1142, just north of Ionakhsh Fault), the other temporary.

This spring survey was extended to the “disturbed zone” of the right bank, in order to collect data for an optimal hydrogeological modelling including this zone. Discharge measurements and hydrochemical analyses were performed on the tributaries of the Vakhsh River limiting the “disturbed zone”, i.e. Ararak stream on the western side and Passimurakho stream of the north-eastern side.

Drilling of the complementary boreholes on the right bank also allowed obtaining some more data about this zone. Reliable piezometric data have still to be collected, but according to the drilling reports, water table was found around elevation 1290 at borehole IF1, but borehole WRB1 seems dry. Within the “disturbed zone”, the water table was found during drilling around elevations 1420 and 1655 for boreholes WRB2/DZ1 and DZ2 respectively.

## 9.2 Hydrogeological appraisal of site up to date

### *9.2.1 Hydraulic conductivity of the rock*

The hydrogeological interpretation made by the time of the Original Project is presented in paragraph 6.7.

Comments which can be made over this interpretation deals mainly with the values of hydraulic conductivities adopted for sound rock (i.e. Zone IV), which are low (less than 1 LU). From the 1978 analysis of water tests results on the basis of drawings 1174-03-F18, F19 and F20 (Ref. [1], § 2.5.3.2.), it can be seen that tests made below elevation 1010 are comparatively in smaller number than in higher elevation. The test results below this elevation show a bimodal distribution:

- below or around 1 LU,
- the others randomly distributed at higher values.

Values of drawing 1174-03-78 Sheet 2, reproduced in Table 6.1, give an idea of the dispersion of the values, but this is common in a fractured rock mass. However, observations made in the investigation galleries, within Zones III and IV, with places presenting a general dampness and some water dripping tend to conclude to some underestimation of the considered average hydraulic conductivity of the rock mass.

Ref. [25] (§ 2.2) reports that, with construction, the groundwater level in the left bank, upstream of the watertight Lower Obigarm siltstones, dropped of some 10 to 12 m with excavation of the underground works. It also reports that, on the site of the power house, the groundwater level dropped from elevation 1040 to elevations ranging between 1010 and 1023. No data is however given about duration for this groundwater table depressing.

It also reports that hydraulic conductivities in Kyzyltash sandstones were measured (from one borehole in the left bank, drilled from underground works) substantially higher in Zone IV than assumed in the Original Project. It is suggested that this may be the influence of the underground works excavations.

However, we kept for this feasibility study similar values as the one of the Original Project, but one shall keep in mind that, at least locally, hydraulic conductivities in Zone IV may reach more than the assumed $10^{-7}$ m/sec (although HPI, in Ref. [27], Table 2.1.1. assumed hydraulic conductivities comparatively higher than 1978 values for the sandstones and siltstones of the site in the different geotechnical zones; but reason for this modification is not explained).

Anisotropy of the hydraulic conductivity was not taken into account at this stage, mainly because of the high degree of fracturing of the rock masses in several directions, which reduces this effect with respect with usual, normally stressed stratified rock masses. It should however be taken into account at a further stage.

### *9.2.2 General arrangement of aquifers*

Description of the aquifers on site is made in the Phase 0 Report, paragraph 4.

It is worth to remind that, according to results of discharge measurements performed in Ararak and Passimurakho streams, which show increase of the discharge when going towards downstream, some contribution from the “disturbed zone” should be taken into account.

The aquifers at the dam site and in the vicinity of the Vakhsh River behave distinctly according to the season. During the Vakhsh River high water period, i.e. during summer, as a consequence of glaciers melting, the River level may rise up to 7 m. At that time, the Vakhsh River is recharging the aquifer and the groundwater flow is directed from the river to the banks. with a groundwater gradient of 3% on the left bank and 3.5% on the right bank. During the winter, at low River water period, the groundwater is flowing reverse, from the banks towards the Vakhsh River. This behavior is confirmed by the first measurements made on the piezometers newly drilled from the underground structures.

Nevertheless, the exact configuration of aquifers, hydraulic conductivities and infiltration in the “disturbed zone”, are still to be investigated, since perched aquifers may exist there (see paragraph 9.1). Piezometers fitted in the newly drilled boreholes should provide useful data for this purpose. Nineteen springs were located on the right bank, with some difference in chemical composition depending upon water is flowing from limestone or sandstone.

### *9.2.3 Hydrogeological model of the site*

A hydrogeological modeling of the site was achieved (Ref. [33 and Ref. [34], which results are briefly presented here below. For more details, one will refer to the dedicated HPI reports (Ref. [33] and Ref [34]), respectively for Stage 1 dam and final dam.

Contours of the model are presented in Figure 9.2, for the Stage 1 dam. Boundary conditions are of no-flow along the Gulizindan Fault, assumed watertight, as per investigated in the Original Project. The western boundary follows the watershed limits, and then cuts through the “disturbed zone”, from which no contribution is assumed. The assumed infiltration values are mentioned on the figure (15% of the average annual precipitations).

With regard to the assumed hydraulic conductivities for the different rock formations considered values are listed in Table 9.1., for each of the geotechnical zones.

Tectonic faults are assumed to be watertight, which is in accordance with site observations and the clay or clayey infilling of these faults.

Model calibration was made by comparing piezometers data with modelled water heights, with slight adjustments of hydraulic conductivities; model balance was also checked accordingly.

Assuming complete efficiency of the drainage galleries provided around the underground works (especially power house), water pressure for the final dam according to the model are in the range of 2 to 18 m water head above the crown. Without drainage, it reaches up to some 160 m, as per results of Table 9.2.

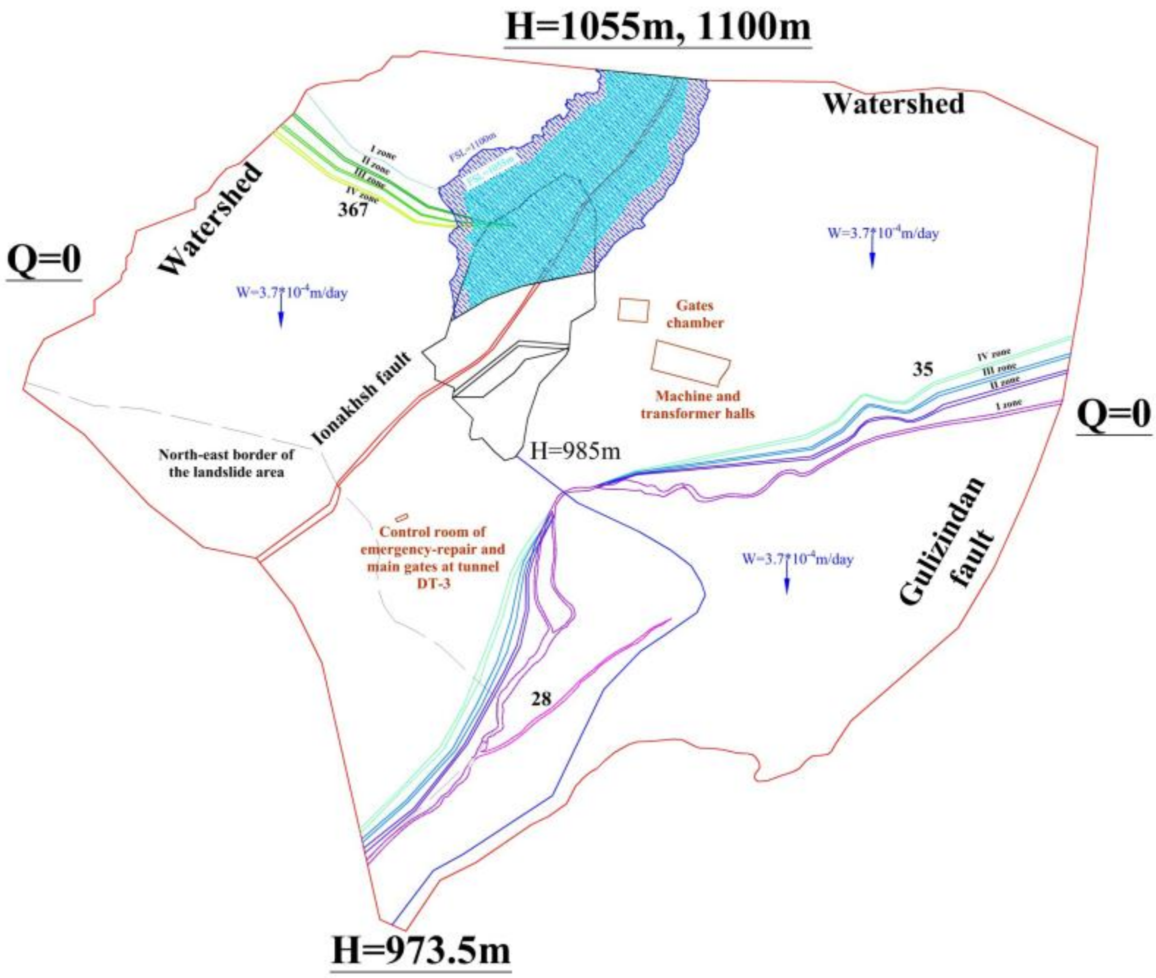



**Figure 9.2: Configuration and boundary conditions of the hydrogeological model (here for Stage 1 dam)**

| Stratigraphic index | Lithological composition of rocks | Seepage coefficient value in model layer, m/day | | | |
|---|---|---|---|---|---|
| | | Numbers of engineering-geological competent zones | | | |
| | | I | II | III | IV |
| | | Numbers of model layers | | | |
| | | (1-2 layer) | (3 layer) | (4 layer) | (5-11 layer) |
| $alQ_{IV}$ | Cobble round-stone, pebble, gravel, sand | 150 | 150 | | |
| $J_3gr$ | Grouting of caprock | 0.0001 | | - | - |
| | Fault filled with salt | - | - | 0.00001 | |
| | Mudstone | 6.0 | 0.3 | 0.06 | 0.009 |
| $K_1jv_{1.2}$ | Siltstone, mudstone with interlayers of sandstone | | | | |
| $K_1kz$, $K_1kr$, $K_1mg$ | Sandstone and siltstone | 6.0 | 0.9 | 0.18 | 0.009 |
| $K_1ob_1$ | Siltstone and mudstone | 6.0 | 0.3 | 0.06 | 0.00001 |
| $K_2t$ | Mudstone, limestone | 6.0 | 0.3 | 0.06 | 0.009 |
| $K_1ob_2$ | Sandstone | 6.0 | 0.9 | 0.18 | 0.009 |
| $K_1lt$, $K_2al$, $K_2cm$ | Sandstone, mudstone, limestone, siltstone with interlayers of gypsum | 6.0 | 3.0 | 0.06 | 0.009 |
| Tectonic faults | Joint with gougl clay | 0.00001 | 0.00001 | 0.00001 | 0.00001 |

**Table 9.1: Assumed hydraulic conductivities for the different rock masses (from Ref. [33]); units in m/day, i.e. some $1.16.10^{-5}$ m/sec.**

| Hydrostatic pressure onto crown of underground structures, ton/m² | | | | Hydrostatic pressure onto floor of underground structures, ton/m² | | | |
|---|---|---|---|---|---|---|---|
| Machine hall | | Transformer hall | | Machine hall | | Transformer hall | |
| Without drainage | With drainage | Without drainage | With drainage | Without drainage | With drainage | Without drainage | With drainage |
| 160-220 | 2-18 | 140-166 | 0-12 | 252-300 | 4-36 | 188-230 | 0-20 |

**Table 9.2: Calculated hydrostatic head over underground structures of the left bank (from Ref. [34])**

| FSL | Seepage flow through, m³/day | | | |
|---|---|---|---|---|
| | Slopes | Foundation | Dam body | $\sum$ |
| 1290 | 234000 | 47500 | 150 | 281650 |

**Table 9.3: Calculated seepage amounts for final dam, FSL 1290 (from Ref. [34])**

These results were taken into account for checking the underground structures, and this issue is discussed in detail in reports dealing with underground works design. It can however be said that, in such conditions, the efficiency of the drainage shall be optimum, and adequate maintenance carried out to maintain it on the long term, during the lifetime of the scheme.

## 9.3 Recommendations for further hydrogeological studies

The current model is a steady state model, not calibrated on the most recent data from the 19 observation wells drilled at the end of 2012, because the groundwater level at these wells were not yet available at the time of the model calibration. As above noticed the right and left bank aquifers behave distinctly during the summer and the winter. Subsequently the model is only semi quantitative. It means that its accuracy is of one order of magnitude for seepage prediction, and within an estimated range of 50% for water pressure distributions. In the frame of detailed design, the Consortium would recommend further investigations about the hydrogeological regime prevailing within the "disturbed zone" of the right bank, since infiltration on the flat area is likely to be higher than assumed on the slopes, acquiring and taking into account all relevant data like piezometric levels, discharges of the tributaries (Ararak and Passimurakho streams), as well as spring discharges.

The model should therefore be extended to include the "disturbed zone" and check that it cannot be of substantial influence on the results.

Investigations for determining the input data of this extended model would also help to better understand the geological structure of this area, since better knowledge of the exact arrangement of the geological formations within it is necessary for the reliability of the results.

Boundary conditions should, as a consequence also be refined to reflect the investigation results.

# 10 VERIFICATION OF ADEQUACY OF THE DAM FOUNDATION

## 10.1 Discontinuity sets of the dam foundation

The discontinuity sets of the rock foundation are not different from the one identified for the whole site and presented by the previous studies, and still updated by HPI in 2009 (paragraphs 6.6 and 8.3).

For checking the adequacy of the foundation, verification of the bearing capacity and rock wedge stability were checked.

Finally, risk linked to possible suffusion within the foundation is to be analysed.

## 10.2 Stability of rock wedges of the abutments

Table 6.7 have presented the joint sets of the foundation, namely the bedding joints and three sets of tectonic fractures.

When trying to identify the possible rock wedge instabilities, it is clear that the three-dimensional effect given by the shape of the foundation is favourable to stability, except for the rock spur located at the downstream right bank. This rock spur will support part of the thrust from the dam when infilled, and stability has been checked for wedges formed by the intersection of joint sets 2 and 4 (stability of rock wedges on the upstream left bank is dealt with specifically in paragraph 12.1).

A photograph of this rock spur, with corresponding joint sets is presented in Figure 10.1.

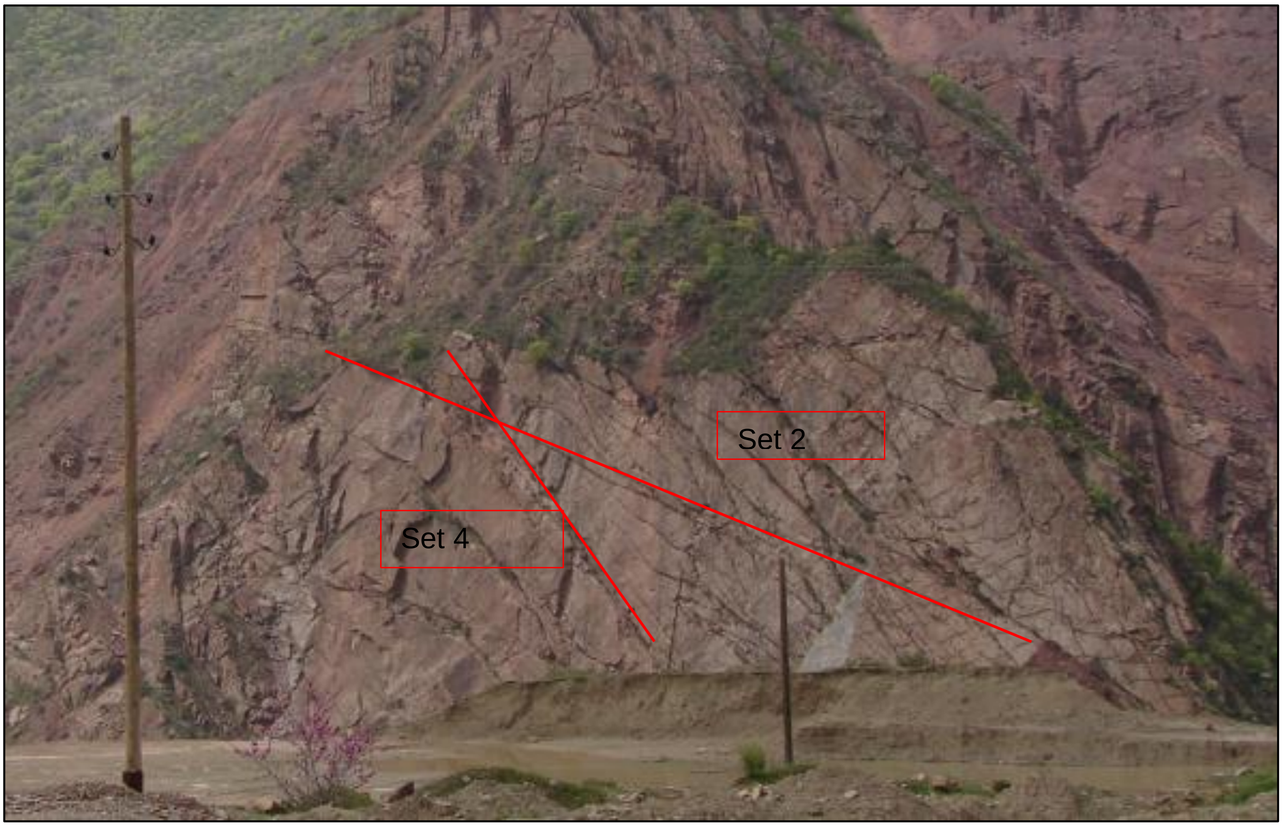



**Figure 10.1: Photograph of the rock spur of downstream right bank, showing joint sets 2 and 4**

On the stereographic projection (Figure 10.2), the wedge stability analysis is carried out in studying the wedge stability for the three sections defined by the different directions of the Vakhsh.

- Section 1: (River direction N-S): On the right bank the wedge stability is controlled by the intersection of sets 3 and 4, with the wedge potentially sliding in a direction perpendicular to the bedding strike. No wedge instability is to feared, but the inclination of the bedding plane is susceptible to lead to toppling failure.
- Section 2 (River direction NW-SE -30): The wedge formed by the intersection of set 2 and set 4, with the prominent shear discontinuities formed by the bedding

planes may lead to wedge instability on the right flank of the gorge; corresponding stability is analysed below.

- Section 3 (River direction NE-SW - 120) the wedge stability is controlled by the intersection of sets 2 and 4, as for section 2. Furthermore, the inclination of the bedding almost parallel to the river direction may lead to toppling failure.

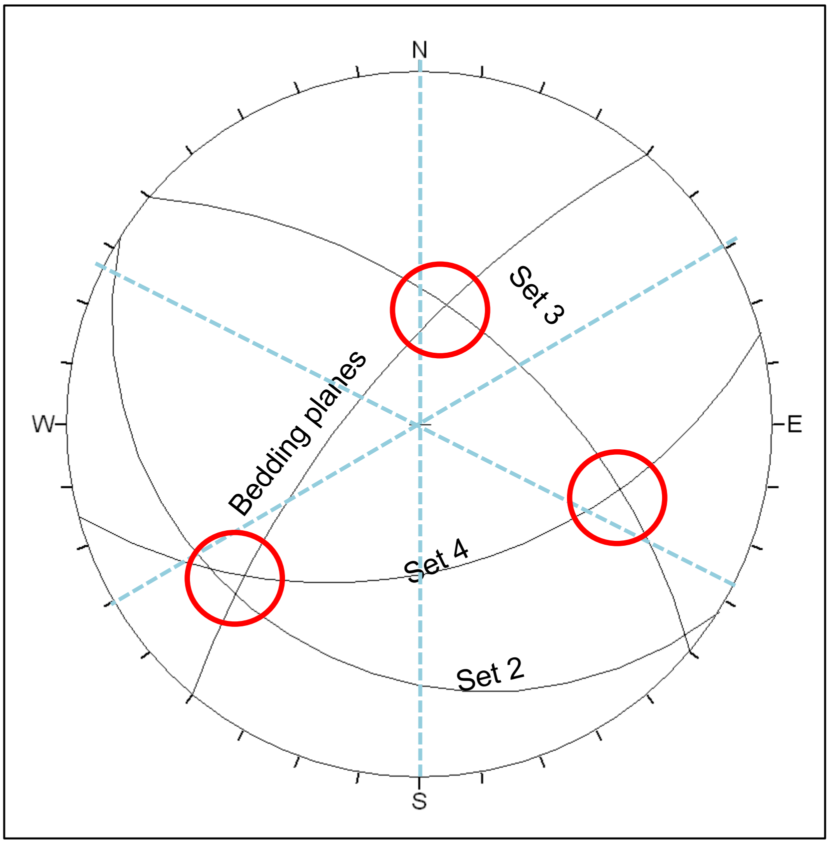



**Figure 10.2: Stereographic projection (upper hemisphere)**

Wedge stability justification was performed for a wedge limited by a bedding planes and joints of set 2, especially for the downstream right bank spur, at the downstream toe of the dam.

The wedge is located within Mingbatman formation ($K_1mg_5$), where the shear strength parameters of the joints are assumed to be as per Table 8.3 (worst parameters).

- Cohesion 0.02 MPa
- Friction angle 26 degree

The wedge stability analysis has been analysed by means of software Swedge. The minimum safety coefficient obtained for this massif is then equal to 1.88. Therefore, no risk of wedge instability is expected to occur.

## 10.3 Excavation of dam foundation

With respect to alluvial deposits in the dam foundation, and since they cannot be observed, final decision to let in place or remove alluvial materials is to be made after river diversion. It will mostly depend upon their degree of compacity (which should be comparable to dam shoulders materials after placement in the dam body) and proportion of fine elements (of the same order of magnitude as within shoulder material as well).

Alluvial material will anyway have to be removed from the foundation of the clay core, to guarantee the watertightness of the dam.

Excavations for the core of the dam achieved before 1993 are still visible (e.g. on Figure 8.8), but weathering of the siltstones and actual aspect demonstrate that renewed excavation is to be performed for the dam core. Considering the sensitivity to weathering of the siltstones, it was already emphasised and commented both in the Original Project and further studies (see paragraphs 6.5.2 and 7.4.2.2). Therefore, it is recommended, during the construction, which is to last several years, to shotcrete the excavations to avoid scouring and weathering. In the bottom, and where possible, leaving over the design level of the foundation a layer of some 1 m thickness, to be removed just before placing the dam body material would be advisable.

With respect to the other parts of the foundation, Zone II in the upper part of the abutments, but Zone III or Zone IV everywhere else shall be reached. This will most probably involve the necessity to remove large potentially unstable blocks in all rock slopes over the work site. Effective support measures shall be applied where scaling cannot be achieved in a satisfactory way.

As a consequence, the amount of excavation and works to secure the work site is important and corresponding measures shall be applied in advance to guarantee the safety of the works during execution, since in actual conditions, rock falls almost systematically occur during rainy episodes.

## 10.4 Bearing capacity of the foundation

GSI can give reliable estimates of the strength and deformation characteristics of the rock masses. The shear strength properties are then deduced from the GSI, the uniaxial compressive strength and on a material constant $m_i$.

Considering that the dam core foundation lays on the siltstones of Zone II (rocks of Zone I will be removed) and that dam shoulder will be placed over siltstones of Zone I, we used the GSI values coupled with the disturbance factor to simulate the weathering and distressing of the foundation (Zone II for the siltstones and Zone I for sandstones). The values we used are those of Table 10.1.

| | Sandstones | Siltstones | Comments |
|---|---|---|---|
| GSI | 45 to 60 | 35 to 50 | **Erreur ! ource du renvoi introuvable.** |
| mi | 17 | 7 | |
| Uniaxial compressive strength of intact rock $\sigma_{ci}$ (MPa) | 100 | 50 | 1978 data |
| Disturbance factor D | 0 | 0 | Assuming smooth blasting |
| Cohesion (MPa) | 2.317 - 3.044 | 1.18 - 1.57 | |
| Friction angle (°) | 44.21 - 48.53 | 27.59 - 32.18 | |
| Tensile strength (MPa) | Between -0.093 and -0.288 | Between -0.053 and -0.165 | |
| Uniaxial compressive strength (MPa) | 4.48 - 10.701 | 1.20 - 3.01 | |

**Table 10.1: Assumed parameters for estimation of bearing capacity of the foundation**

The bearing capacity of the foundation justification was made according to the USACE recommendations. The rock mass of the foundation is assimilated to a jointed rock mass. The mode of failure of the foundation depends upon discontinuity spacing, orientation and conditions of the joints which are dipping between 28 and 69 degrees. Failure of the foundation, if any, would occur along the moderately dipping joints.

Safety coefficients ranging from 30 and more were obtained, thereby demonstrating that no problem of bearing capacity of the foundation is to be feared, as could be expected. The order of magnitude of the safety coefficients obtained largely cover uncertainties about the assumed geotechnical parameters.

## 10.5 Possible consequences of dissolution or suffusion phenomena within the dam foundation

## 10.6 General assessment from past studies and site observations

This issue was especially raised by Lahmeyer in 2005, in view of the rapid disintegration of the Lower Obigarm siltstones when exposed to weathering. The risk of suffusion, or internal erosion by migration of fine particles with water circulations is to be considered. Lahmeyer additionally raised the issue of potential dissolution of the gypsum contained within joints of this rock formation (accounting for 0.5% of the rock, as per Table 2.1).

According to observations made in the underground works, the Consortium considers that a similar phenomenon may occur within the Kyzyltash formation (see paragraph 8.2.5 and Figure 8.5).

## 10.7 Mitigation measures

The grout curtain to be realised, if performed properly should limit and slow down water seepage flows in the foundation to acceptable gradients, in order to avoid excessive fine entrainment towards downstream.

It is also felt that, if dissolution of gypsum is limited in that way within the Lower Obigarm siltstones, amount of fine to migrate from this formation will likely be limited.

With regard to suffusion within Kyzyltash formation, presence of the downstream low-permeability Lower Obigarm siltstones is likely sufficient to limit water gradients to acceptable values, where such phenomena are no more to be feared.

## 10.8 Conclusions about dam foundation treatment

Dam foundation treatment shall therefore address the three following topics.

Excavations for the dam foundation are likely to be substantial, in the core area, in order to reach back down to the unweathered rock foundation, and protecting it adequately. However, a very large amount of work is also needed, in our opinion, in order to reach both objectives of sound rock foundation and safety of the work site. Well before beginning of the works, an extensive and comprehensive campaign of scaling or supporting or rock masses over the work site is to be engaged.

Contact / consolidation grouting is to be performed below the dam core, in order to restore the properties of the rock foundation, which are inevitably affected by blasting operations.

The grouting curtain, which is to stop before reaching Fault 35 (which presents natural watertightness, thanks to its clay infilling) is to be carefully executed, and will require, given the size of the final dam, reaching high pressures, up to 6 or 7 MPa. If GIN (Grout Intensity Number) method is to be adopted, this correspond to high values of the PV (pressure by volume) product.

The extent of the grouting curtain within the right bank is still to be verified in regard of actual geological conditions and hydraulic conductivity of the eastern boundary of the disturbed zone.

Intended design of dam foundation treatment is presented in Vol. 3, Ch. 3, “Design Alternatives”.

# 11 IMPLICATION OF ACTIVE TECTONICS OF THE SITE ON DESIGN AND OPERATION OF HYDRAULIC STRUCTURES

## 11.1 Description of active tectonic movements on project site

### *11.1.1 Creeping of faults*

Monitoring carried out as early as 1968 in some places, evidenced the active tectonic movements going on through the dam site. Ionakhsh Fault, Gulizindan Fault and Fault 35 were all found out to be submitted to slow, but apparently permanent creeping movements. This is the reason why project structures were as much as possible, located within the block limited upstream by the Ionakhsh Fault, and downstream by Fault 35.

Mention of these creeping movements of the faults is made in paragraph 2.1 above, but one will refer to Phase II Report - Volume 2 – Chapter 2 – Geology for more details. To summarise, relative movements of the two walls of the faults were recorded, as well as movements of the tectonic lenses in-between or nearby. Upward vertical movement of the tectonic lenses was found to be systematically higher than the relative displacements of the two walls, suggesting extrusion.

On Ionakhsh Fault, vertical offset of walls was measured at an average of 1.8 mm/year, against 2.8 mm/year vertical uplift for the tectonic lens inside.

On Fault 35, vertical offset of walls was measured at an average of 0.7 mm/year, against 2.3 mm/year vertical uplift for the tectonic lens inside.

The relative vertical displacement along Gulizindan Fault is reported to occur at a rate of 1 mm/year.

Associated to these creeping movements, similar slow angular variations were recorded along the moving faults.

Recording of such creeping movements is not surprising, given the high level of compressive stresses acting within the rock massif, and the plastic behaviour of rock salt, which facilitates such movements.

It is not difficult to understand, from such data, that the movement of the permanently creeping limiting faults cannot remain without effect for the block located in-between, which is to shelter most of the structures of the project. A likely very slow, but certain accommodation to the deformations is without doubt to take place between Ionakhsh Fault and Fault 35. Apart from the reported monitoring of Fault 70 during two years, with reportedly no movement higher than 0.1 mm recorded during this period (see paragraph 2.1), no data is available about such accommodation movements; distribution and rate of movement, location of moving joints, etc. However, site observations and freshness of cracks and faults with offset, which is a peculiarity for this site, fits with the model of permanently deforming rock masses (see paragraph 2.1 and Phase II Report - Volume 2 – Chapter 2 - Geology, paragraph 3.3.3.).

The available data should be complemented by results from the new monitoring system to be installed.

### *11.1.2 Potential co-seismic displacements*

In addition to fault creeping, which allows slow dissipation of part of the tectonic compressive energy, the seismo-tectonic analysis performed in (Sismotectonics Report) showed that two kinds of co-seismic movements could be inferred in case of earthquake, namely:

- a relative co-seismic movement assessed to a maximum of 1 m amplitude along Ionakhsh Fault or Gulizindan Fault,
- a relative co-seismic movement assessed to maximum 0.1 m amplitude along faults or joints located between Ionakhsh and Gulizindan Faults.

Therefore, possible occurrence of such co-seismic movements is to be taken into account in the design of the works.

Along Ionakhsh Fault, a co-seismic displacement of 1 m, and in any case, a cumulated vertical movement of 0.28 m in 100 years.

Along Fault 35 and other faults susceptible to move to accommodate with the co-seismic displacement (or slow creeping), a co-seismic displacement of maximum 0.1 m is assumed, given that anyway, creeping will trigger a vertical movement along Fault 35 of some 0.23 m in 100 years.

## 11.2 Implications for dam and open air structures

As detailed in Vol. 3, Ch. 3, “Design Alternatives”, the amount of creeping movement, as well as co-seismic displacements, should not have any impact on the dam, due to the size and deformability of the dam with respect to the movements. The dam core is located just upstream of Fault 35, and filters are designed to mitigate the consequences of relative displacements during earthquakes.

Impact on open-air structures should remain limited. The high-level outlet open-air spillway, being located on the downstream right bank slope, within the “disturbed zone”, may however be affected by movements of Fault 35 or other discontinuities of similar attitude.

In this slope, part of the “disturbed zone”, active erosion results in permanent scouring, so that if Fault 35 and its movement extend there, it is not visible.

However, all open-air structures have the advantage to be easily observable, so that occurrence of any damage should rapidly be detected by permanent inspection operations, and repairs swiftly engaged.

## 11.3 Implications for underground structures

### *11.3.1 Hydraulic tunnels*

#### *11.3.1.1 General guidelines*

Hydraulic tunnels of the scheme have, for most of them, the drawback of not being easily accessible for inspection, due to the size of the reservoir and corresponding elevated water heads.

Therefore, such tunnels shall be designed to accommodate the tectonic movements actually occurring (permanent creep) or potential (co-seismic movements).

In the particular case of hydraulic tunnels, and especially those with high water velocity like in Rogun, consequences of a relative displacement along a fault can be dramatic, if adequate precautions are not taken from the early excavation. Consequences that may arise from fault movement cutting a hydraulic tunnel are illustrated by cases A, B and C of Figure 11.1.

First of all, a relative displacement will force the water under pressure and high velocity into the ground, where it would have moved such as to oppose the flow direction. Fault material being most of the time either clayey or silty, but rapidly losing mechanical properties in presence of water, erosion and scouring can rapidly develop and result in collapse of the lining. The phenomenon is to repeat farther, and not to stop, and may lead to total obstruction of the tunnel.

Case B and C of Figure 11.1 show consequences over the flow, where cavitation may occur and damage both lining and ground, and subsequent headlosses generated by the discontinuity in the tunnel walls.

Therefore, hydraulic tunnels shall be designed such as to cope with the consequences of fault movements. Inspection of all hydraulic tunnels is desirable after such a fault movement, and shall be performed as soon as possible to allow for repairs.

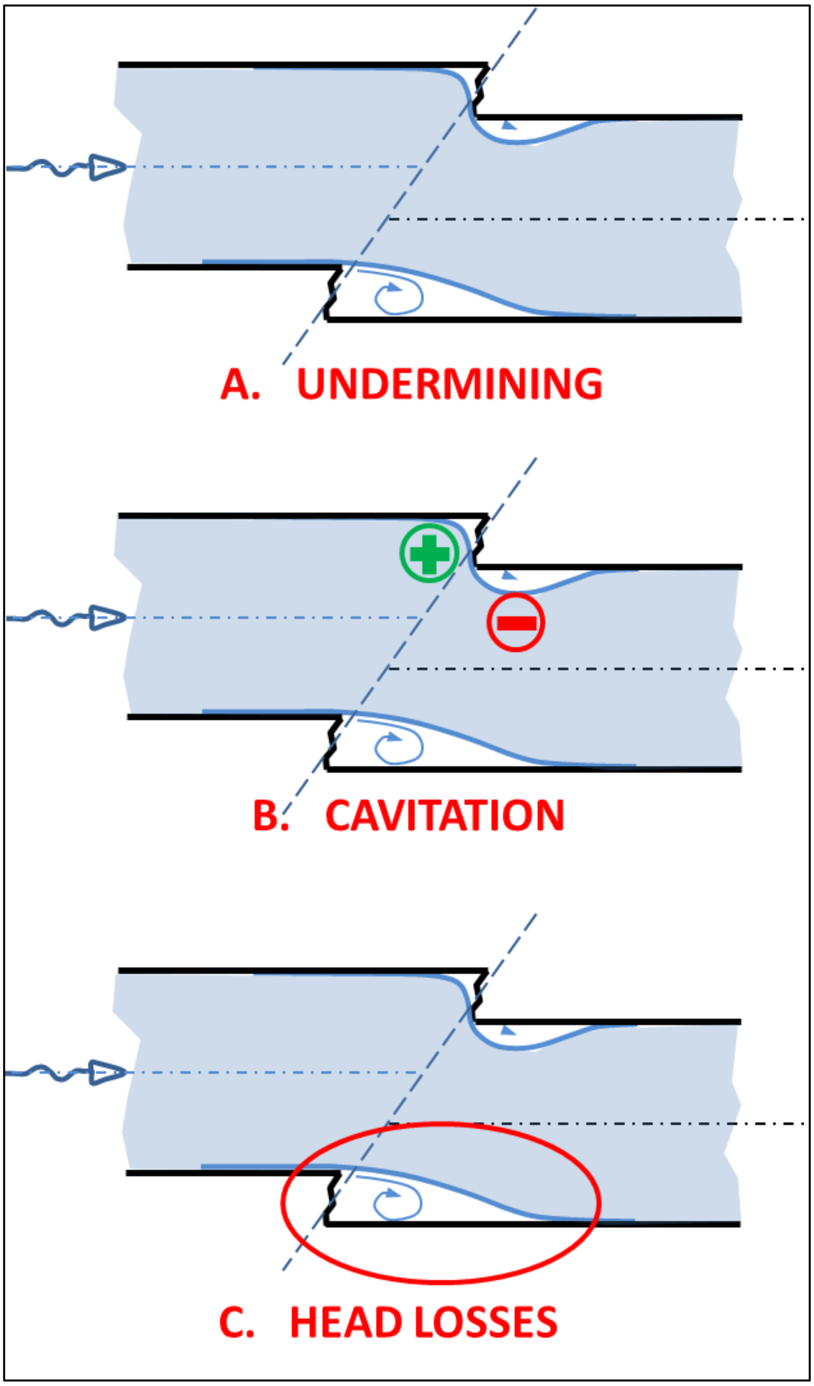



**Figure 11.1: Ranked by order of seriousness, potential damages consecutive to the relative displacement of an hydraulic tunnel by fault movement**

*11.3.1.2 Hydraulic tunnels crossing Fault 35 or similar*

Configuration of the site is such that some of the hydraulic tunnels will have to cross Fault 35 or minor faults of the same S4 family.

Tunnels crossing Fault 35 shall be designed such as to cope with a relative fault movement of 0.1 m (amount of co-seismic movement). But this assumption is made wherever crossing faults which are assumed to accommodate seismic movements. Exact definition of the faults to be treated in this way (except Fault 35) is to be made at a stage of detailed studies, but we assumed that major faults of S4 family (like Fault 70 or Fault 32) located between Fault 35 and the Ionakhsh Fault would be potentially affected by 0.1 m co-seismic movement.

Distribution of faults where tunnels will have to be specially designed to accommodate possible creeping or co-seismic movements with 0.1 m relative displacement is assumed, at this Feasibility Stage, to be every 40 to 60 m in the block limited by Ionakhsh Fault and Fault 35 (see paragraph 6.6.1.2), or even 20 m according to Phase II Report - Volume 2 – Chapter 2 - Geology. The dip angle over horizontal of such faults ranges between 20 to 50 degrees.

In absence of co-seismic movement, the allowance for 0.1 m relative displacement largely covers the cumulated amount of creeping displacements that may occur (maximum assessed 0.23 cm in 100 years on Fault 35). Three arrangements are suggested for hydraulic tunnels where crossing such faults.

The first one is illustrated by the illustrative sketch of Figure 11.2, whereby an overexcavation would be performed when crossing the fault, all along the possible zone of influence of fault movement. This overexcavation, to define, would remain limited to a maximum of 1 m over the diameter of the tunnel (which may reach more than 15 m), in order to absorb the 0.1 m displacement.

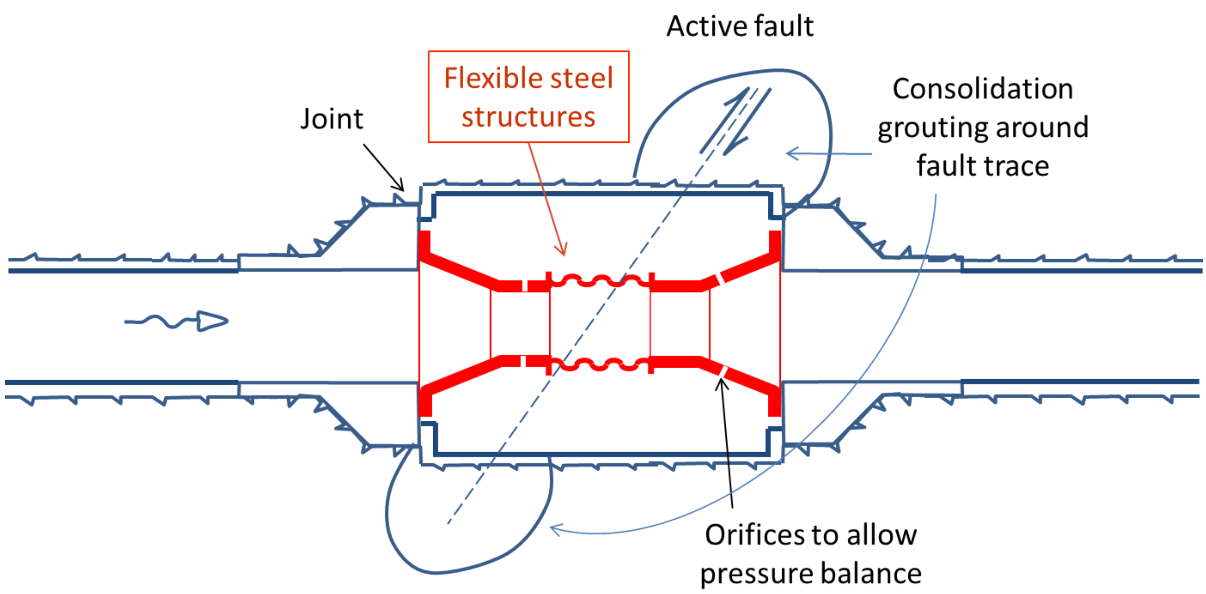



**Figure 11.2: Illustrative sketch of suggested arrangement for minor fault crossing (anticipated relative displacement no more than 0.1 m), first solution**

The section would then be lined with reinforced concrete (eventually with displacement joints, depending upon length), and connected to the current section of the tunnel on both extremities by means of joints. Inside the reinforced concrete excavation, a specially designed steel lining with extensible and compressible joints is to be installed, with lateral holes allowing the water pressure balance on both sides of the flexible steel structure. To reinforce the ground around the fault, an additional measure would be achieving consolidation grouting around its trace.

Detailed design of such an arrangement is of course to be performed at a later stage of studies, but is found to be a suitable alternative in case of fault movement, by keeping the flow of water within a watertight lining accepting some amount of relative displacement.

It is however obvious that such a solution cannot be adopted if the zone to be treated is too long, because self-support of the deformable steel structure in the centre would not be guaranteed.

The second suggested arrangement is a more simple one, less costly, but likely of least effectiveness than the first arrangement of Figure 11.2. The corresponding illustrative sketch is presented in Figure 11.3.

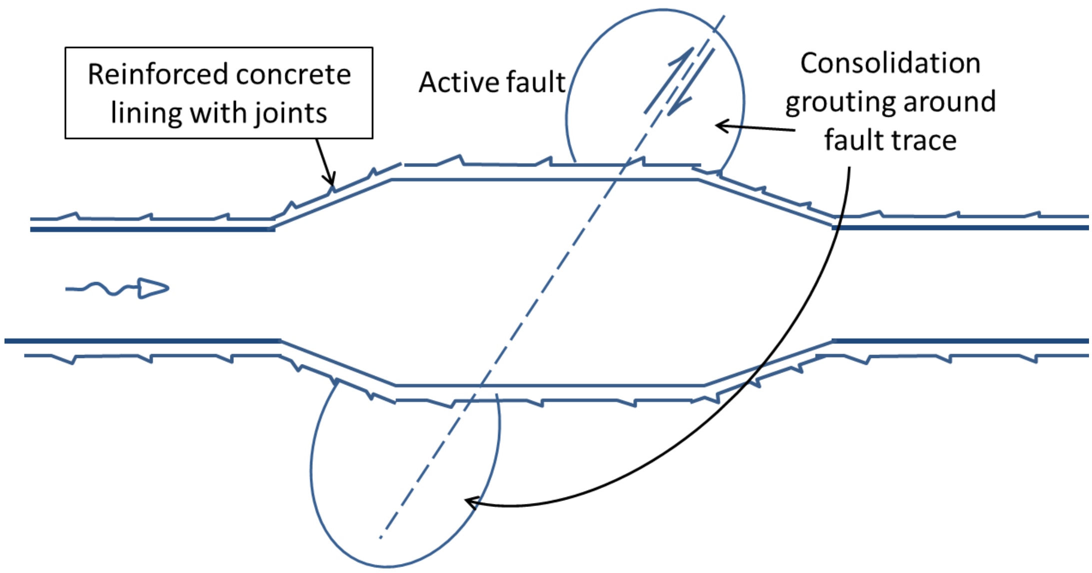



**Figure 11.3: Illustrative sketch of suggested arrangement for minor fault crossing (anticipated relative displacement no more than 0.1 m), second solution**

It consists in removing the steel structure from the first arrangement, keeping only the over excavated section over the whole length of tunnel potentially affected by fault movement, lined with heavily reinforced thick concrete, cut every few meters by joints fitted with extensible waterstops designed to support displacements (shearing, traction or compression) of several centimeters.

The basic principle is that, in case of fault movement, the reinforced concrete lining should only suffer minor damages, without substantial decrease of the support pressure, therefore maintaining the stability of the excavation, and keeping a low permeability to avoid scouring phenomena, which are the most to be feared. If fault rupture occurs over a very short distance (like a cut of saw), the first solution should be preferred, since in that case the reinforced concrete lining may turn locally very damaged just at the fault location.

In view of these considerations, a third arrangement is proposed, which is likely the most efficient, if a reduction in the available section of the tunnel can be accepted between the time of occurrence of the earthquake triggering the co-seismic movement and the time of repair of the tunnel. Since the maximum displacement is 0.1 m, it would simply consist in considerably increasing the thickness of the concrete lining and adequately reinforce it, in order to keep the tunnel lined even after the co-seismic displacement. The reinforcement should be dimensioned as to avoid the fall of large parts of the lining or embedding rock.

Considering the geomechanical characteristics of the rock between Fault 35 and Ionakhsh Fault, no major stability problems are anticipated for over excavation and support, since thickness of fault zones does not exceed some meters.

Nevertheless, in case of Fault 35, the tectonic lens of the right bank, between the two seams of the fault, reaches a thickness of 60 to 70 m. Seams are filled with 15 to 20 cm thick clay, while the rock in between is highly fractured (for more details, see description of Phase II Report - Volume 2 – Chapter 2 - Geology, paragraph 3.3.2).

Therefore, the solution to be selected for this major fault of the site, to be crossed by some hydraulic tunnels (namely diversion tunnels and mid-level outlet 2), is that proposed in the drawings attached to the TEAS and shown in the abstract there below.

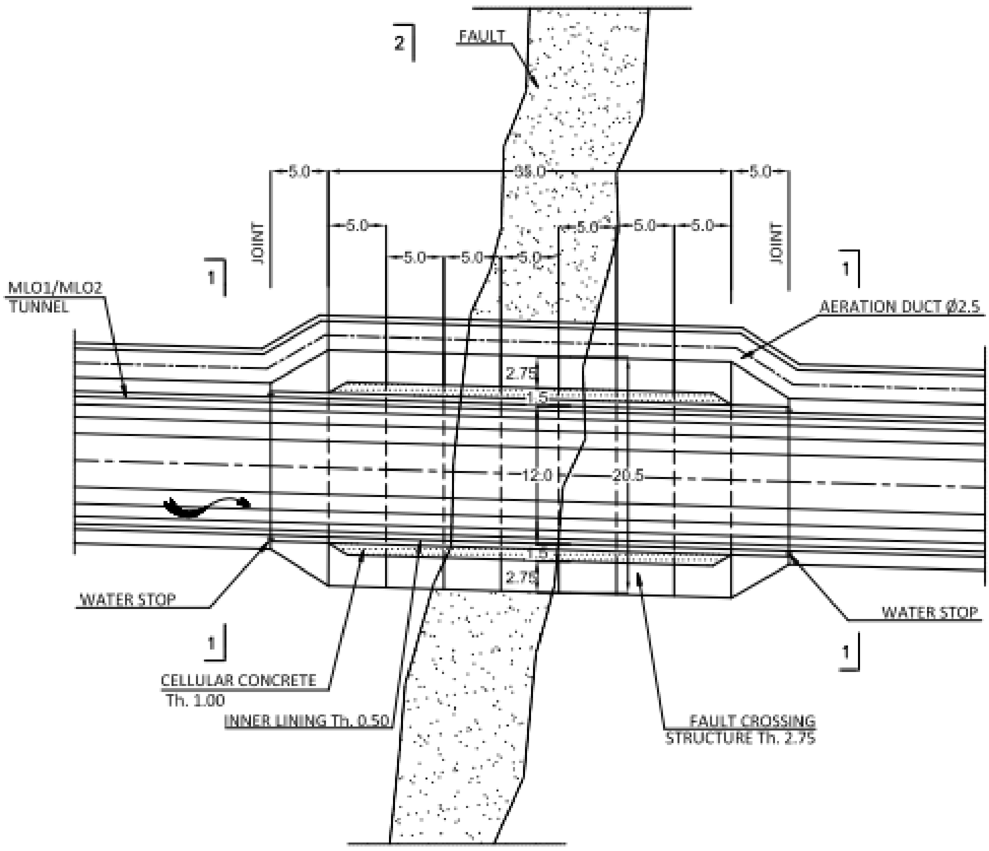



**Figure 11.4: Illustrative sketch of suggested arrangement for major fault crossing**

However, the kinematics of Fault 35 movement would have to be studied with more detail in further stages of study, with the objective of determining more precisely locations of Fault 35 to be fitted with special arrangement.

Considering the conditions observed in the investigation gallery 1002, which could not be cleared due to permanent rock falls where crossing Fault 35, or considering what happened in the diversion tunnels, over excavation will not be easy, and careful excavation and support in divided sections is to be anticipated. This will depend upon the final solution adopted.

*11.3.1.3 Hydraulic tunnels crossing Ionakhsh Fault*

Potential co-seismic movement along Ionakhsh Fault being of maximum 1 metre amplitude, any hydraulic tunnel crossing it should be fitted with adequate arrangement to cope as much as possible with such movement, or at least mitigate the consequences to an acceptable risk level. Actually, hydraulic tunnel crossing or anticipated to cross Ionakhsh Fault is the third diversion tunnel only, which should not be utilised for more than about 10 years.

The thickness of the tectonic lens within Ionakhsh Fault reaching up to 80 m in right bank, it is essential, as for Fault 35, to try as much as possible to locate places where fault rupture is likely to occur.

Considering such a movement, we suggest:

- either to adopt an arrangement similar to Figure 11.3,
- either the adequate reinforcement and increase by some 1.2 m of the tunnel lining as per the third arrangement proposed in paragraph 11.3.1.2., if the condition of temporary loss of available section of the tunnel can be accepted.

In both cases, the arrangement shall be adapted to the Ionakhsh Fault, with a progressive enlargement of the section of the tunnel to decrease the speed of water. The tunnel would be over excavated, increasing its excavated diameter (by at least some 5 meters at least in the first solution, less in the second), and lined with heavily reinforced concrete lining rings of more than 1 m thickness and joints at some 5 to 10 m interval (depending upon length and further studies).

It is expected that in case of co-seismic movement of 1 m amplitude, the reinforcement will somewhat distribute the deformation over a certain length, and that, since thickness of the lining is more than the expected movement, reinforcement will help keep the cavity stable, avoiding extensive erosion and collapse as per case A of Figure 11.1.

An illustrative sketch of this solution is presented in Figure 11.5.

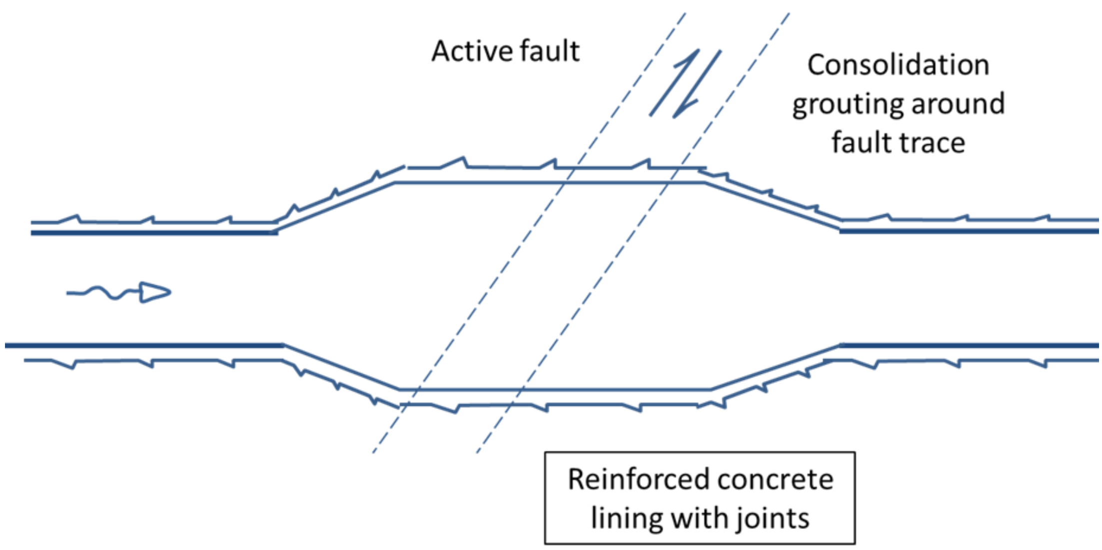



**Figure 11.5: Illustrative sketch of arrangement of hydraulic tunnels where crossing Ionakhsh Fault (dimensions to be defined on the basis of detailed study of fault kinematics)**

However, and given the diameter of the hydraulic tunnels (more than 15 metres), increase in excavated diameter would require the excavation of the size of a cavern. Considering the poor characteristics of the Ionakhsh Fault's tectonic lens, and the presence of the soft Gaurdak claystones over some 25 m thickness on the downstream side, excavation should be of round shape, and carried out with utmost precautions, in divided sections, with immediate application of support and precise monitoring.

Another solution to improve excavation conditions is to cross the fault by means of two or three tunnels, instead of only one, introducing a bifurcate – or trifurcate – upstream and downstream of it.

Detailed study of the kinematics of the Ionakhsh Fault should also help to determine the most probable locations where the movement may take place, and scheme for fault zone treatment detailed on this basis.

*11.3.1.4 Conclusions about fault crossing by hydraulic tunnels*

The different solutions proposed for the lining of hydraulic tunnels where crossing active faults have been presented. In the drawings, and for cost estimate, the solution based on a significant thickening of the lining up to 5.5 m (crossing of Ionakhsh Fault by third level diversion tunnel) has been adopted.

As emphasized above, some more investigations would be required at detailed design studies to try to locate at best the possible location where co-seismic fault movement may occur. Principles for solution are presented, both for Ionakhsh Fault (1 m co-seismic movement), but for other faults, including Fault 35, along which a 0.1 m co-seismic displacement is anticipated.

Extension of the special treatment to other faults is justified by the possible propagation or relay of co-seismic displacements along minor faults, as well as location where creeping is suspected to take place.

Examination of the conditions of the diversion tunnels No.1 and No.2 by HPI in 2009 identified, apart from the creeping movements observed at the portals and disorders linked to Fault 35 activity, other creeping movements having led to extensive cracking of the concrete lining some 70 m and 90 m from the portals for diversion tunnels No.1 and No.2 respectively, interpreted as gravitational creeping along a S4 joint. In the derivation tunnel No.2, a collapse located between the latter joint and Fault 35, within Kyzyltash sandstones, is not explained in another way than high or abnormally high stress levels (Ref. [24], § 2.3.3.1.). But review of inspection documents (Ref. [15], § 5.2.) locates the corresponding 10,000 m$^3$ rock collapse at the intersection of Fault 70 (the generated cavity extends on the western side of the tunnel over some 18 m above the crown).

In any case, should an earthquake trigger co-seismic displacements, all hydraulic tunnels should be as soon as possible inspected for eventual repair operation.

### *11.3.2 Non-hydraulic underground structures*

With regard to access tunnels and ancillary excavations, the expected co-seismic displacement being not anticipated more than 0.1 m, and given that such tunnels are normally easily accessible for observation, the repair of eventual damages should not be a problem.

However, with regard to the caverns, power house, transformer cavern, assembly chamber, gate chambers or other excavation of prime importance for operation of the scheme, possible co-seismic displacements of 0.1 m, as well as some creeping might occur during the lifetime of the scheme. Faults such as Fault 70, which intersects the power house, may relay some seismic displacement.

The impact of such a movement, but also impact of possible creep movements accommodating those occurring along Ionakhsh Fault and Fault 35, shall be taken into account in the design.

Similarly, changes in inclination of the units that may occur as a consequence of the creep movements within the block limited by Ionakhsh Fault and Fault 35 have reportedly been taken into account for in the Original Design (verbal communication of Mr. Kolichko).

At following stages of study of the Project, a detailed monitoring and study of the kinematics of fault movements is recommended to be resumed for identification of location of potential fault movements and subsequent treatment, as already emphasised in Phase II Report - Volume 2 – Chapter 2 - Geology, paragraph 7.2.

## 11.4 Reservoir induced seismicity

The great depth and volume of the reservoir will extend for most part along the upstream located Vakhsh Valley. As emphasised in previous studies and Phase II Report - Volume 2 – Chapter 2 - Geology (paragraph 2.) two regional active faults are running within this valley: the Gissaro-Kokshal Fault and the Illiak-Vakhsh Fault. Reservoir-induced seismicity is therefore very likely to occur during and after impounding.

It is therefore recommended to establish a reliable seismic monitoring system around the Project to determine the background component.

Induced seismicity may lead to adopt adequate operation instructions for reservoir level variations, as this might have been the case for the downstream Nurek reservoir.

# 12 SITE SPECIFIC ISSUES REGARDING GEODYNAMICS

## 12.1 Slope stability along joints of set 4

### *12.1.1 Presentation of the problem*

Faults and continuous joints of the S4 family, with variable dip angle are present in the foundation, as already emphasised in paragraph 8.3.2. According to the Original Project (paragraph 6.6.1.2), confirmed by observations made in the galleries, these are continuous over at least 200 m, with spacing 40 to 60 m.

The last part of this paragraph will be dedicated to the assessment of slope stability of the upstream left bank along joints of set 4, which appear unfavourable. Stability of the slopes where temporary and permanent intake structures will be located shall be checked in this respect. Damages noticed on the intake structures of the diversion tunnels, demonstrate the reality of such creeping movements along joints of the S4 family, which, with reservoir impounding and drawdowns, could eventually slide down in the reservoir.

This risk by gravitational sliding is one of the conclusion of Phase II Report - Volume 2 – Chapter 2 - Geology, where it is detailed.

### *12.1.2 Proposed remedial measures*

The principles for remedial measures are mainly:

- Drainage, everywhere possible (above reservoir level or in the zone of level variations),
- Unloading of the slope by re-profiling, as much as possible,
- Concrete shear keys along the identified joint, for its reinforcement,
- Slope support by means of long anchors or tendons.

An illustrative sketch of those measures is presented in Figure 12.1.

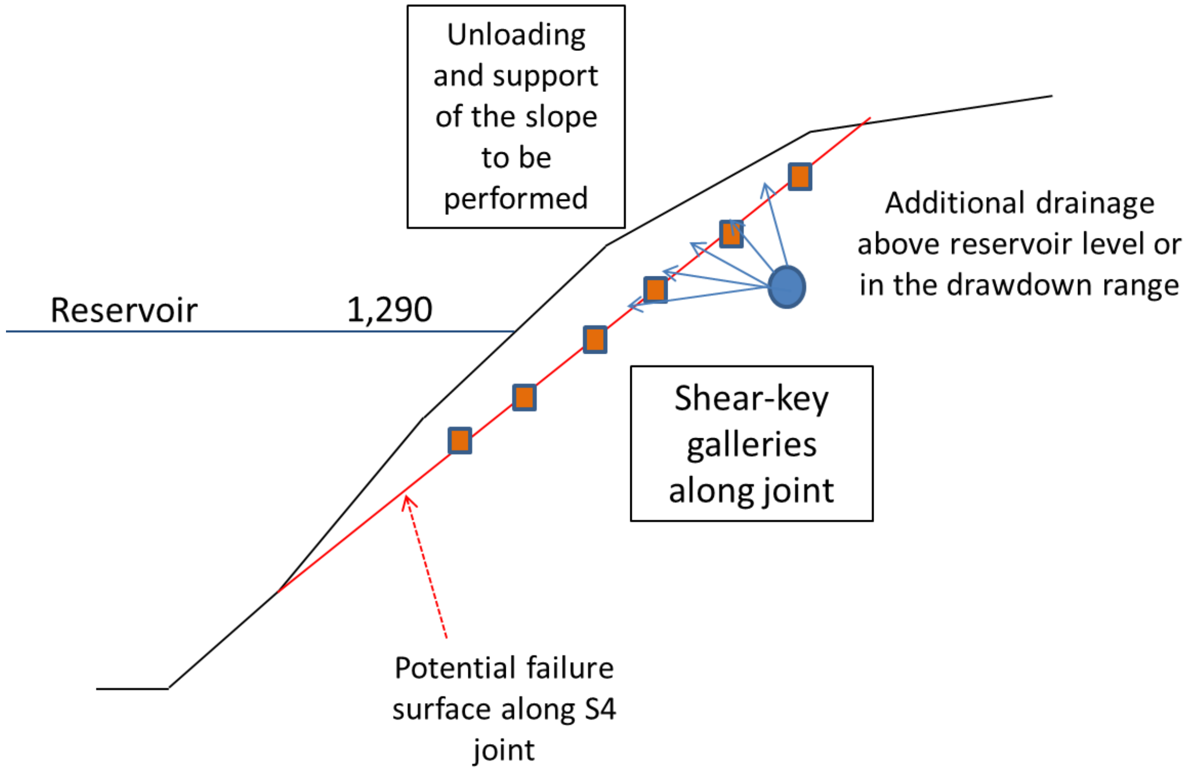



**Figure 12.1: Illustrative sketch of remedial measure for stabilisation of potential rockslides along joints of the S4 family (upstream left bank)**

Shear keys are galleries of about 3 to 4 m height, excavated through the potentially sliding joint, and following its surface over the whole area to be treated. They are backfilled with concrete, in order to be able to impede the initiation of a sliding movement. They have the advantage that they can also be completed before dam impounding for those located below reservoir level.

Shear keys at different levels are generally necessary to guarantee the stability, and such a solution has been considered by Coyne et Bellier in several projects. For instance, it has been adopted for stabilisation of the abutment of the 230 m-high arch dam of Karun IV dam, in Iran.

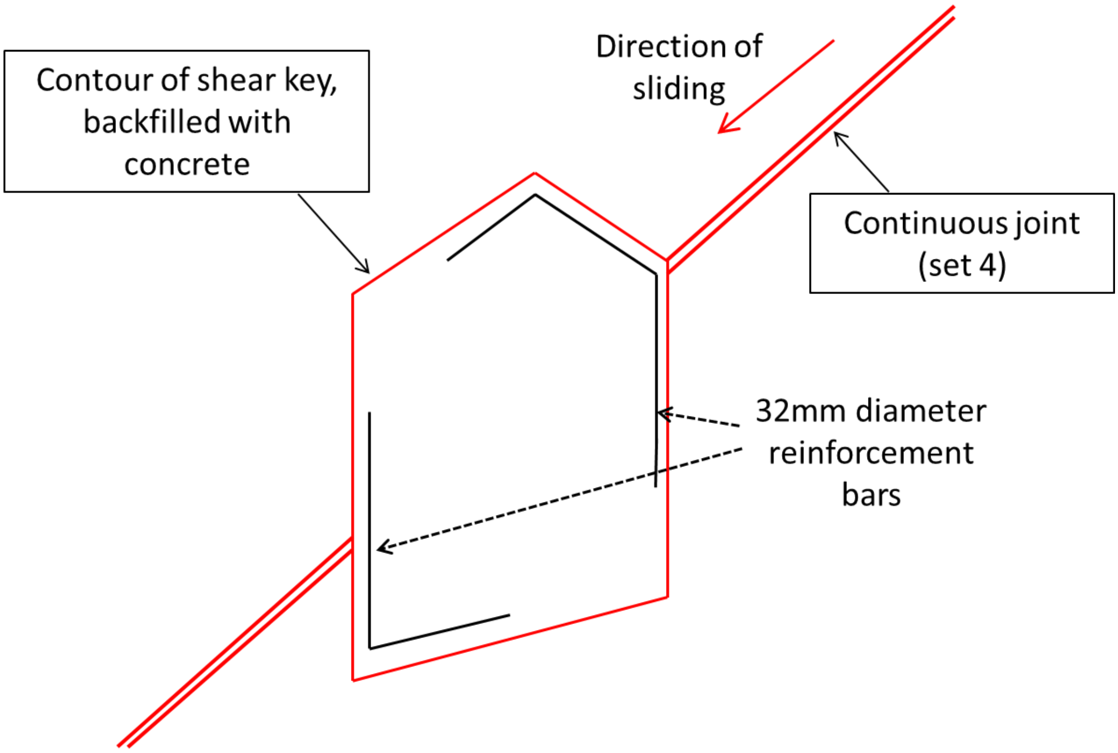



**Figure 12.2: Illustrative sketch of shear key (gallery excavated along the joint and backfilled with reinforced concrete)**

An illustrative sketch of a shear key (cross-section) is showed on Figure 12.2.

Realisation of shear keys of course implies that the discontinuity or fault along which sliding may occur is clearly identified. Therefore, detailed investigations of the potentially unstable masses of the upstream left bank, along joints of the S4 family such Fault 70 should begin sufficiently early for design and realisation of the adequate remedial measures before raising of the water level.

In case the clay infilling is reduced to some centimetres like observed in the galleries of the dam site, such shear keys are likely not necessary. Shear keys are generally used on continuous joints filled with a substantial thickness of soft material.

The remedial measures shall be complemented by permanent monitoring, using high-performance and long duration quality material, to survey the slopes.

## 12.2 Downstream right bank and disturbed zone

### *12.2.1 Potential risks incurred from presence of disturbed zone*

The interpretation of the generation and structure of the disturbed zone by the Consortium has been presented in Phase II Report - Volume 2 – Chapter 2 - Geology. The former hypothesis of huge old landslides presented in the 1978 Original Project, which could have been reactivated by impoundment and filtration through the right bank of the reservoir, is discarded, and explained by tectonic deformations.

As exposed in Phase II Report - Volume 2 – Chapter 2 - Geology, the “disturbed zone” was actually generated by salt tectonics, with, roughly speaking, extrusion of gypsum masses subsequently karstified in surface. This structure is much more favourable to stability and the risk of huge landslide can be discarded.

Bending of the layers from the dam site to the slope over the right bank of the Vakhsh River on the downstream side, is probably sharp with some zones of weaknesses, but the overall integrity of the massif is preserved on this large slope.

However, near to the river, the ground water level has a very mild slope, while at least one permanent spring is visible around elevation 1,100 upstream of the Ionakhsh Fault, suggesting presence of perched aquifers. Presence of the reservoir will fill this space. Meanwhile, the rooting of the gypsum mass in depth, upstream of the Ionakhsh Fault, is not known, nor its arrangement with the other potentially karstic formations.

Therefore, once could fear that the raising of the reservoir level, coming in contact with gypsum, could provoke leaching of the latter, and thus, leakage farther downstream. This issue is however discarded (see details in paragraph 13.2).

As stated in Phase II Report - Volume 2 – Chapter 2 - Geology (paragraph 7.3), local increase of pore pressure cannot be discarded, considering the karstic formations and heterogeneous underground flow which can be anticipated from the interlayering of highly and poorly permeable formations.

That is the reason why the monitoring of the zone is indispensable, and drainage measures to be implemented where this would reveal necessary.

### *12.2.2 Stability of colluvium masses of the front slope of the disturbed zone*

Scouring by rain water and infiltration from the top of this zone has generated an intensive surface erosion of the slopes of the downstream right bank, just above the Vakhsh rRver, probably together with some gravitational sagging. As already emphasised in paragraph 2.4.3, some potentially unstable masses of colluvium remain on this slope, downstream of the dam site, just over the Vakhsh River. The most prominent one is being presently monitored (Illustration No.51 of Phase II Report - Volume 2 – Chapter 2 - Geology), and is shown on Figure 12.3.

Small springs are present at the base of the slope, with discharges ranging between 0.2 to 1.0 litre/sec.

**Figure 12.3: Right bank slope of the Vakhsh River, just downstream of the dam site, in the disturbed zone; the white arrow indicates the head scarp of an apparently active landslide**

The volume of colluvial material susceptible to collapse in the river in case of failure of the whole of the mass shown on Figure 12.3., which head scarp is clearly visible in the topography, is estimated to up to 500,000 m3, depending upon the geometry of the failure surface extent upwards of the failure. In case of sliding of this whole mass, a total volume of material of similar magnitude may further collapse in the river, by way of progressive failure.

Since 500,000 m3 are sufficient to dam the river for some time and result in subsequent elevation of downstream river level, similar consequences as the one of the 1993 mudflows from Obi-Shur are to be feared.

With regard to realisation of underground works or surface structures on this slope, and especially the surface spillway, it is obvious that distressed and weathered ground is to be anticipated over at least some 15 m thickness from the actual surface of the slope, but which can reach up to 40 m, as per values of Table 6.6. This has been taken into account in the design of the proposed surface spillway (see Vol. 3, Ch. 3, “Design Alternatives”).

### *12.2.3 Proposed mitigation works*

The karstic features evidenced by the 2012 investigations, presence of gypsum, even at higher level, and the fact that the actual internal structure of the disturbed zone could not be investigated may let think that impounding of the reservoir could feed water tables within the weathered zone and trigger landslides (as also explained in Phase II Report - Volume 2 – Chapter 2 - Geology, § 7.3.).

In order to secure the potentially unstable mass of colluvium of Figure 12.3, the failure of which cannot be discarded in case of major earthquake even in actual conditions, it is additionally proposed to perform re-profiling of the slope, such as to discharge as much as possible the creeping masses. Since water is the most important triggering factor, two levels of drainage galleries are proposed to be excavated below the assumed failure surface of the landslide, while a drainage trench shall be excavated above the head scarp to limit surface infiltrations in this area. An illustrative sketch of these proposed remedial measures is shown on Figure 12.4.

This project should of course be adapted to real conditions, once more investigations would have been performed, especially concerning the potential failure surface. For this purpose, the installation of borehole inclinometers with sliding captor is recommended on this landslide. High-quality, perennial instruments are required for this purpose.

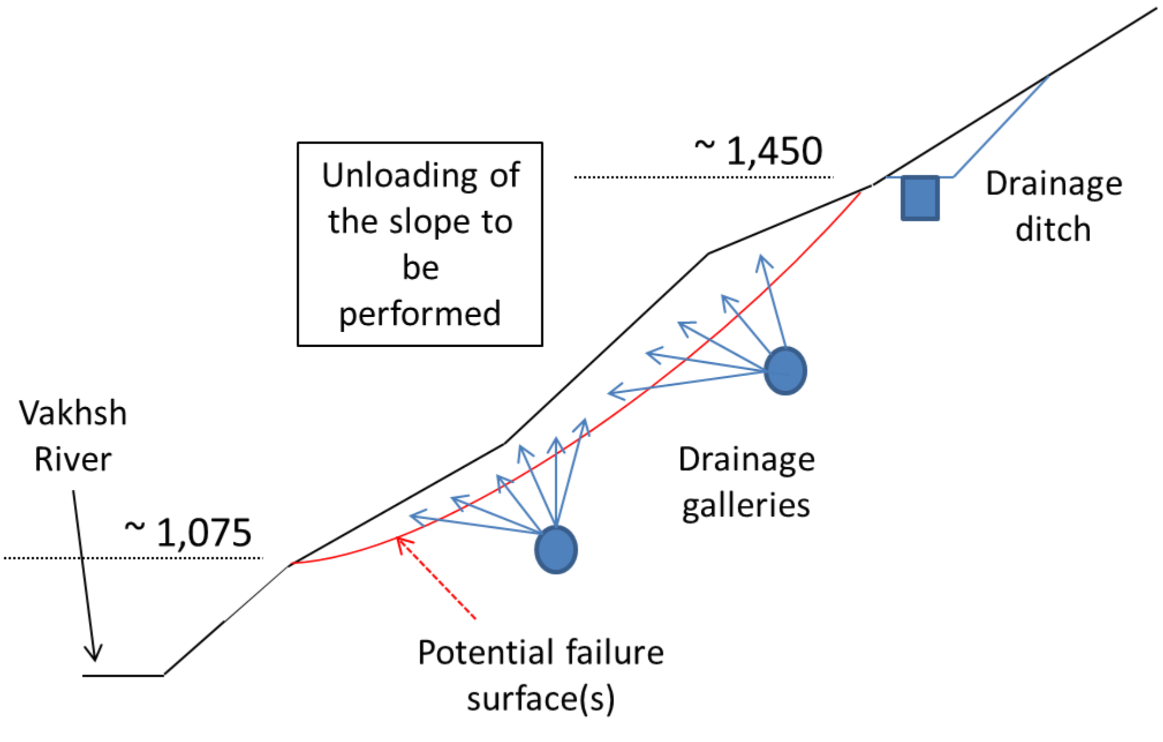



**Figure 12.4: Illustrative sketch of remedial measures for treatment of potential landslide of the front slope of the disturbed zone**

## 12.3 Protection against Obi-Shur mudflows

### *12.3.1 Characteristics of Obi-Shur mudflows*

Mudflows regularly occurring in the Obi-Shur river, left bank tributary of the Vakhsh river, immediately downstream of the dam site have since long been identified by HPI as a threat for the Project, and extensive and comprehensive studies have been performed on the subject.

The extremely active erosion in the Obi-Shur catchment area, which includes steep slopes and landslide-prone areas, creates accumulation of debris in the river bed. Rapidly loaded saturated deposits or failure of small landslide dams easily result in huge mudflows travelling at high speed. The estimated volume of debris currently to be potentially washed down by mudflows in the Obi-Shur valley has been assessed to more than 60,000,000 m3.

According to Ref. [27], § 4, mudflows occur almost every year in the Obi-shur valley, and sometime even several times within a year. Apart from the 1993 mudflow, which dammed the river and resulted in the inundation of the underground works, mudflows are reported to have dammed the river as well in 1969 and 1971, with increase in the water level in the Vakhsh River of 10 to 14 m (see also Ref. [24], § 2.3.2.4.).According to the first reference, the volume of material brought by one single mudflow was maximum in 1983 and 1992, when estimated amount of debris were respectively 3,100,000 and 1,185,000 m3.

### *12.3.2 Prevention of mudflow hazard in Obi-Shur*

In order to avoid the damming of the river by debris from the mudflows, the 2010 HPI Project decided the construction of a 70 m high dam on the Obi-Shur River, for retention of the debris. This dam is scheduled to be elevated once filled.

Figure 12.5 and Figure 12.6 present respectively the plant view and vertical cross-section along the axis of the Obi-Shur retention dam at crest elevation 1125. The dam is a concrete dam designed with 5 m wide apertures, in order to let water and minor blocks go through, retaining the coarser elements like boulders (as –built drawings made be slightly different).

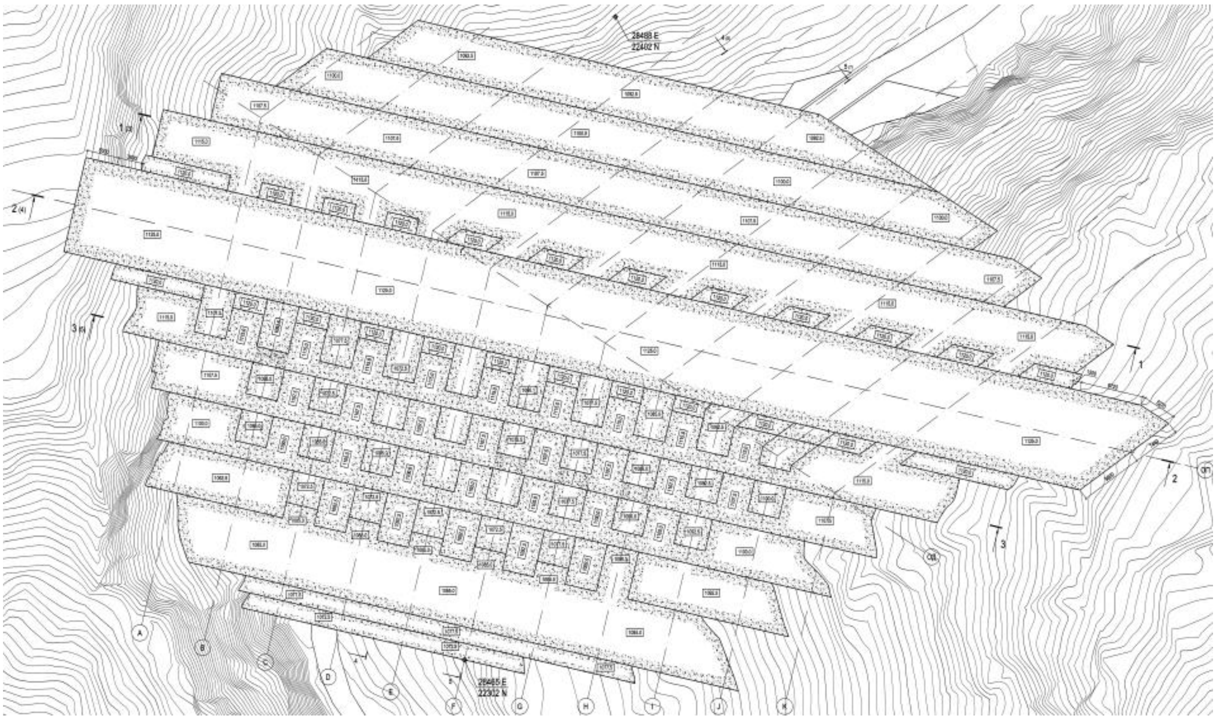


**Figure 12.5: Plant view of Obi-Shur retention dam at crest elevation 1125 (HPI drawing 1900-13-1, Sheet 2, 2012)**

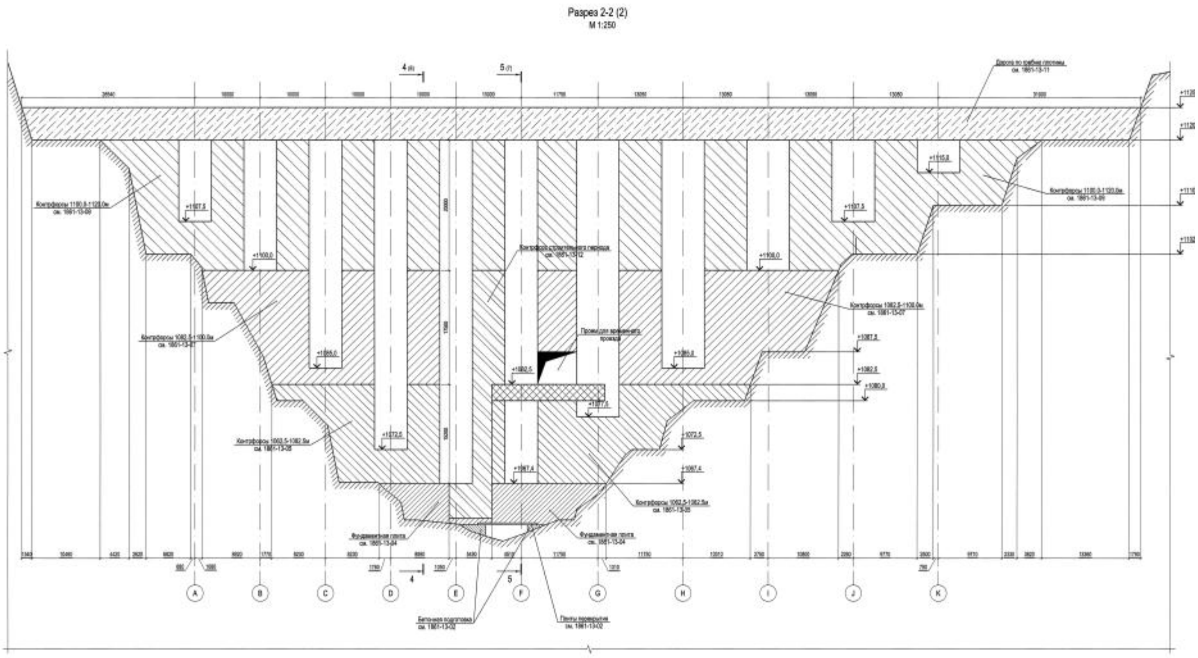



**Figure 12.6: Vertical cross-section along dam axis of Obi-Shur retention dam at crest elevation 1125 (HPI drawing 1900-13-1, sheet 4, 2012)**

In August 2012, the upstream face of the dam appeared to be backfilled over most of the completed part (see Figure 12.7).

This demonstrates the reality of the problem. In order to minimise the volume of materials retained by the dam, the structures should be able to allow easy passage of water together with the finer part of the material carried out by the mudflow. Only blocks and boulders of metric dimensions should ideally be retained, by internal drainage effect.

However, and if no removal of debris from behind the dam is carried out timely, the occurrence of successive mudflows may also, after sometime, lead to plugging of voids and loss of efficiency of the work, impeding drainage through the structure. Then, the whole of the materials carried down by the mudflows will remain behind the structure.

In view of Figure 12.7, it seems that this is already the present situation, with debris from the mudflows that occurred recently stored behind the dam like in a reservoir, including gravels. It seems that rapidly, the 5 m wide openings of the dam were blocked, so that drainage effect on the mudflows disappeared. The shape of the openings, most of them with an angle, also probably did not facilitate evacuation with water of the smaller pieces of rock.

**Figure 12.7: Obi-Shur retention dam in August 2012; mudflows already almost backfilled the river behind the completed part of the dam**

Adequate design of such structures is difficult to achieve, and most of the time, efficiency of the debris flow retaining structures depends closely about the particular conditions of the site and flow characteristics (which may even be different between two consecutive events).

In case of Obi-Shur, and considering the actual situation, the amount of material carried down by the river is exceptionally big. Additional studies are recommended on the basis of investigation of the efficiency of the present dam during the most recent mudflow events. Surelevation of the dam, or construction of another dam upstream is required, in order to avoid that the present one is overflown by future mudflows.

One may consider that this surelevation of the retention dam, by allowing the backfilling of the valley, will create a flat area where mudflows are to slow down, But in case of rainy episodes, if the surface of the already deposited debris is saturated, they may as well be remobilised by further flows.

Moreover, and especially if mudflows are generated by breaching of landslides temporarily damming the Obi-Shur river upstream, even re-profiling the bed of the river with several dams would be of little help.

Suggestions that can be done to deal with the problem are the following:

- Modelling of the dynamics of mudflows, based on return of experience from the present dam; this should especially help to assess the real slowdown effect that progressive backfilling of the valley may have on future mudflows,
- Hydraulic modelling in order to test new types structures, to allow a durable drainage effect when mudflows encounter the structure,
- Installation of adequate monitoring devices to detect as soon as possible formation or occurrence of mudflows,
- Envisage how Rogun dam and its power-generating structures could be adapted to cope with a temporary surelevation of the downstream level, either by building protective structures, together with operation instructions.

It is indeed preferable to stop production temporarily, by closing all gates and accesses, rather than to take the risk to inundate of the power-generating structures. Therefore, we think the two last options should be further studied, considering the high frequency of occurrence of mudflows.

# 13 OTHER ISSUES

## 13.1 Potential landslides or mudflows in the reservoir area

Mudflow risks are presented in Phase II Report - Volume 2 – Chapter 2 - Geology, § 7.4, and an inventory of mudslide-prone water courses around the dam site is presented in Ref. [24], § 2.3.2.4.

Mudflows from this water courses present a risk during the construction. It should also be checked that mudflows from the Passimurakho stream do not present a danger for the dam at the different stages.

Apart from the risk of rock sliding along joints and faults of the S4 family (such as Fault 70), a number of potential landslides of large volume has been spotted in the reservoir area.

Stability of each of those potential landslides, which movement could be triggered by impounding, variations of the reservoir level or seisms, is to be assessed on a case by case basis. It is to avoid that failure of a landslide in the reservoir cannot result in a wave which could overtop the dam, at its different stages of construction.

A potential triggering effect of landslides and mudflows may also be the result of rock salt bodies dissolution once the reservoir impounded and when submitted to reservoir level fluctuations. Rock salt is reported to be present within the major regional faults, like the Gissaro-Kokshal Fault, running along the foothills of the left bank slopes. Salt diapirs are also reported, like the one present in Passimurakho stream gully, the small right bank tributary of the Vakhsh River just upstream of the dam site.

## 13.2 Leakage from reservoir

The actual interpretation of the right bank geology does not let to fear substantial leakage from the reservoir to the disturbed zone. As already emphasised in Phase 0 report, the Ionakhsh Fault infilling of breccia and the Gaurdak claystones just downstream form a watertight barrier, such as it delimits two aquifers within the right bank of the dam site.

Upstream of the Ionakhsh Fault, and according to the investigations performed in 2012, karstified structures in gypsum seem to be limited to higher elevations than the reservoir. Considering the hydrogeological conditions of the right bank upstream of Ionakhsh Fault, with a ground water elevation near to that of the river, and the presence of the permanent, karstic-like spring surging there from the Kirbich syncline, presence of a perched aquifer within this syncline is inferred. This means that impounding of the reservoir is to saturate the part of the right bank located between the actual groundwater level and the bottom of the perched aquifer. One could think that possible extension in depth of the gypsum layers, if karstified, may open way to leakage towards downstream. However, considering the watertightness of Ionakhsh Fault, the only way out for a leakage to occur downstream would be the Ararak stream. Elevation of Ionakhsh Fault there (close to the reservoir level), and the distance of percolation between the dam site (more than 4 km) makes it improbable.

Downstream of the Ionakhsh Fault, and considering the low hydraulic conductivity of the rock formations, no substantial leakage is expected, as demonstrated by the results of the hydrogeological model (see paragraph 9.2.3.).

The other potential way for leakage is the Gulizindan Fault; which separates from Illiak-Vakhsh Fault on the upstream left bank of the reservoir until reaching the Obi-Shur valley, just downstream of the dam.

According to past investigations, seismic wave velocities were measured up to 5,000 m/sec. in the fault, which was intersected by an investigation gallery. But salt is reported to infill Gulizindan Fault at lower elevations, just as for Ionakhsh Fault. Therefore, some leakage through the potentially more permeable space just over the salt cap might be possible, should water from the reservoir reach it.

Piezometric monitoring is recommended at the downstream end of the Gulizindan Fault, and a grouting gallery is recommended to be excavated for observation and immediate performance of grouting operations is revealed necessary.

## 13.3 Impact of potential salt dissolution within Ionakhsh Fault

According to Report P002378, and provided the mitigation measures are achieved, salt dissolution could not occur over more than 25 m depth with regard to the present elevation of the top of the salt wedge of Ionakhsh Fault.

It has been verified that, in case such dissolution occur, it will not impact the watertight dam core (see Annexure 1 of the same report). Therefore, and provided the dissolution does not exceed the 25 m, it should not present any risk for the dam.

# 14 GEOTECHNICAL CONDITIONS RELATED TO MODIFICATIONS PROPOSED BY THE TEAS CONSULTANT

## 14.1 Gate chambers of third diversion tunnel

### *14.1.1 Geometry of the gates chambers*

The third level diversion tunnel is to be fitted with two gates chambers. The first one will be located upstream of the point of intersection of the tunnel with Ionakhsh Fault and is called maintenance/emergency gate chamber, to allow inspection of the sector of Ionakhsh Fault. Its geometry is presented on Figure 14.1. The excavated height of the gate chamber is 21.4 m, while the width varies from 21.4 to 30.7 m at the gates (4 No.) along the 160 m long structure. A 18.4 m high gate operation chamber is located above. Elevation of the invert is around 1032.

The second one is located downstream of this intersection point and is called sector and emergency gates chamber. Its geometry and characteristics presented on Figure 14.2. Dimensions of this chamber are quite similar to that above described, except that the total length is 201 m. Elevation of the invert is around 1028.

### 14.1.2 Maintenance/emergency gates chamber

#### 14.1.2.1 Geological conditions

The gates chamber is to be located within geotechnical zone IV (unweathered and undistressed rock) within rocks of the Upper Albian and Lower Cenomanian (the upper term of Lyatoban suite and some tectonically disturbed Mingbatman may be encountered at the upstream and downstream ends respectively). Those rocks are mainly intercalations of claystone, siltstone and sandstone with frequent interlayers of gypsum and limestone within the Cenomanian term.

Albian is most likely to be encountered in the lower part of the gates chamber, while the gates operation chamber is to be located in the harder Cenomanian suite.

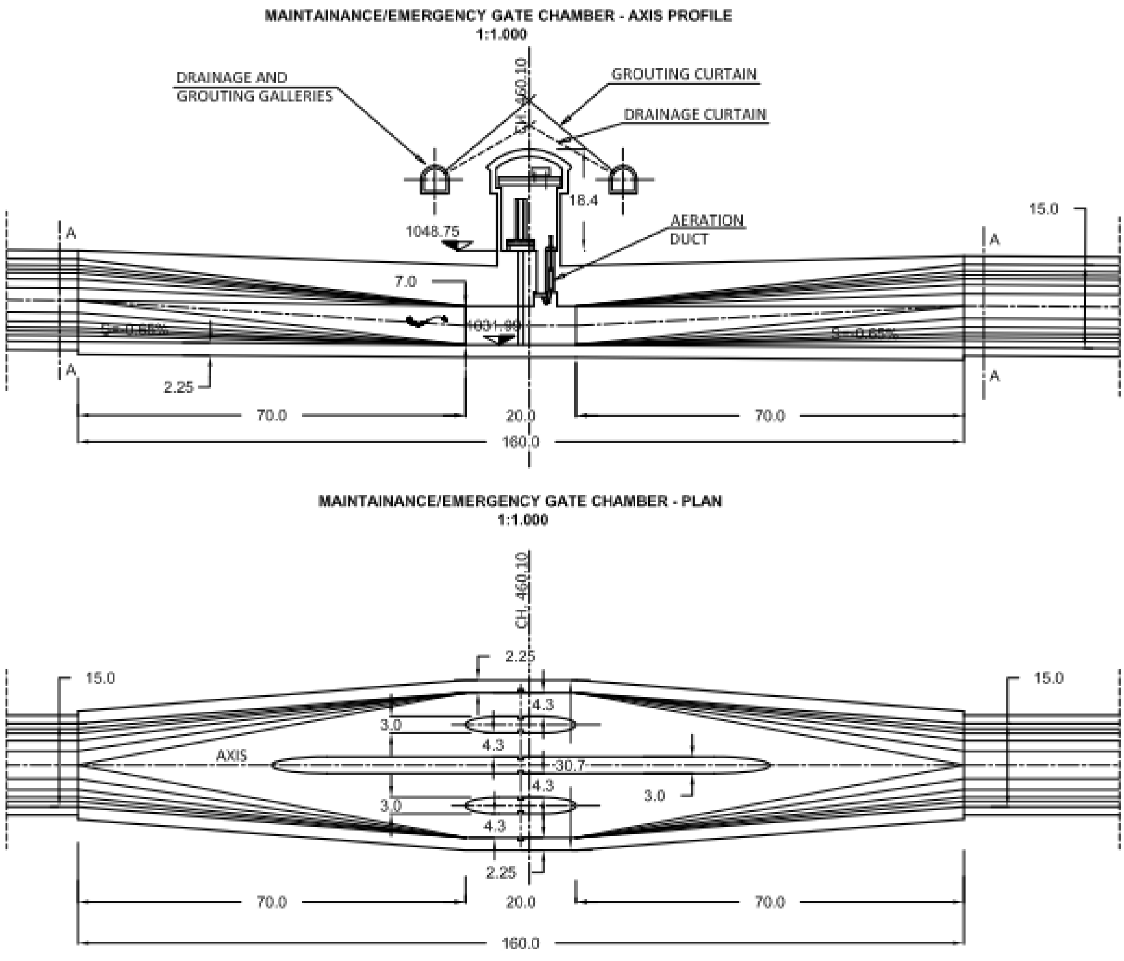



**Figure 14.1: Profile and cross-section of maintenance/emergency gates chamber of third level diversion tunnel**

The geotechnical characteristics of those rocks do not appear explicitly in the Original Project, The limestone of the Cenomanian are hard, in compact beds, but Albian as the whole may be assumed with geotechnical conditions similar to siltstones, except that account shall be taken of the presence of the gypsum beds.

*14.1.2.2 Excavation and support*

Excavation will probably have to be made in divided sections, given the large width of the chamber around the gates, with a support of anchors and reinforced shotcrete. Construction of the concrete structures for the gates will help support the excavation at final stage.

In case the gates operation chamber is excavated afterwards, from the access gallery, fibre-glass anchors are to be used in the vault.

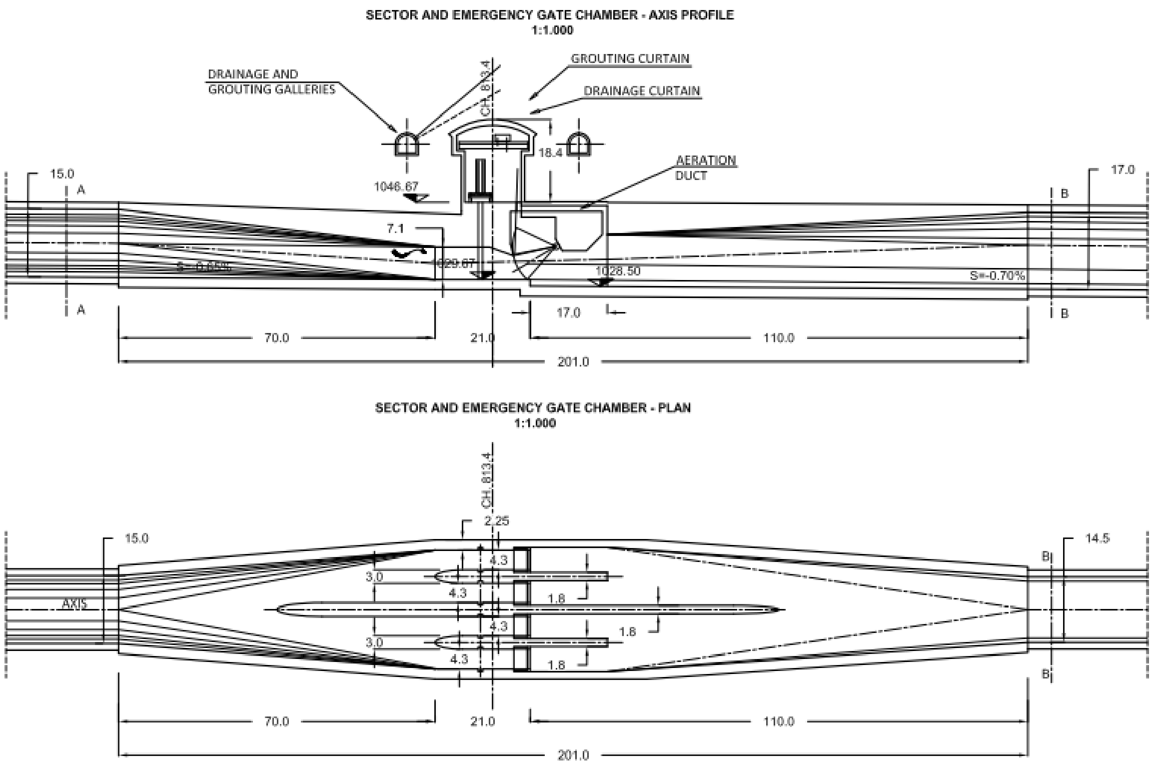



**Figure 14.2: Profile and cross-section of sector and emergency gates chamber of third level diversion tunnel**

Excavation and support of the gate chamber is therefore feasible, but the sequence of construction has to be thought over and modeled to find the best alternative. Excavation shall proceed carefully, with regular check of monitoring results to check stabilization of the cavity.

With regard to the presence of gypsum, the final concrete lining will likely to be adequately reinforced, in order to distribute cracks and limit as much as possible leakages in the ground.

### *14.1.3 Sector and emergency gates chamber*

*14.1.3.1 Geological conditions*

The sector and emergency gates chamber is to be located within geotechnical zone IV (unweathered and undistressed rock). It will most probably cross intersect, from upstream to downstream, the Gaurdak claystones, the Lower and Upper Javan and finally the Kyzyltash sandstones.

Faults of the S4 family, of similar attitude than Fault 35, are to be encountered, persistent and with some centimetres of clay infilling.

*14.1.3.2 Excavation and support*

Excavation will probably have to be made in divided sections, at least for the greatest cavities within Gaurdal and Lower Javan, with support of anchors and reinforced shotcrete. Construction of the concrete structures for the gates will help support the excavation at final stage.

Like for the maintenance/emergency gate chamber (see paragraph 14.1.2.2), excavation and support of the gate chamber is judged feasible, but the sequence of construction has to be thought over and modeled to find the best alternative. Excavation shall proceed carefully, with regular check of monitoring results to check stabilization of the cavity. Local reinforcement of the support may be needed when crossing faults of similar attitude than Fault 35.

## 14.2 Gates chambers of proposed mid-level outlets

### *14.2.1 Gates chambers of mid-level outlet 1*

*14.2.1.1 Maintenance gates chamber*

Dimensions and characteristics of this maintenance gates chamber are presented in Figure 14.3. It is 19.5 m wide, and the only notable additional excavation with regard to the current section of the tunnel is the gates operation chamber located above (21.3 m height).

The chamber is located at elevation 1082, within sandstones of the Kyzyltash Formation, and its excavation and support should not present more problems than for the main tunnel.

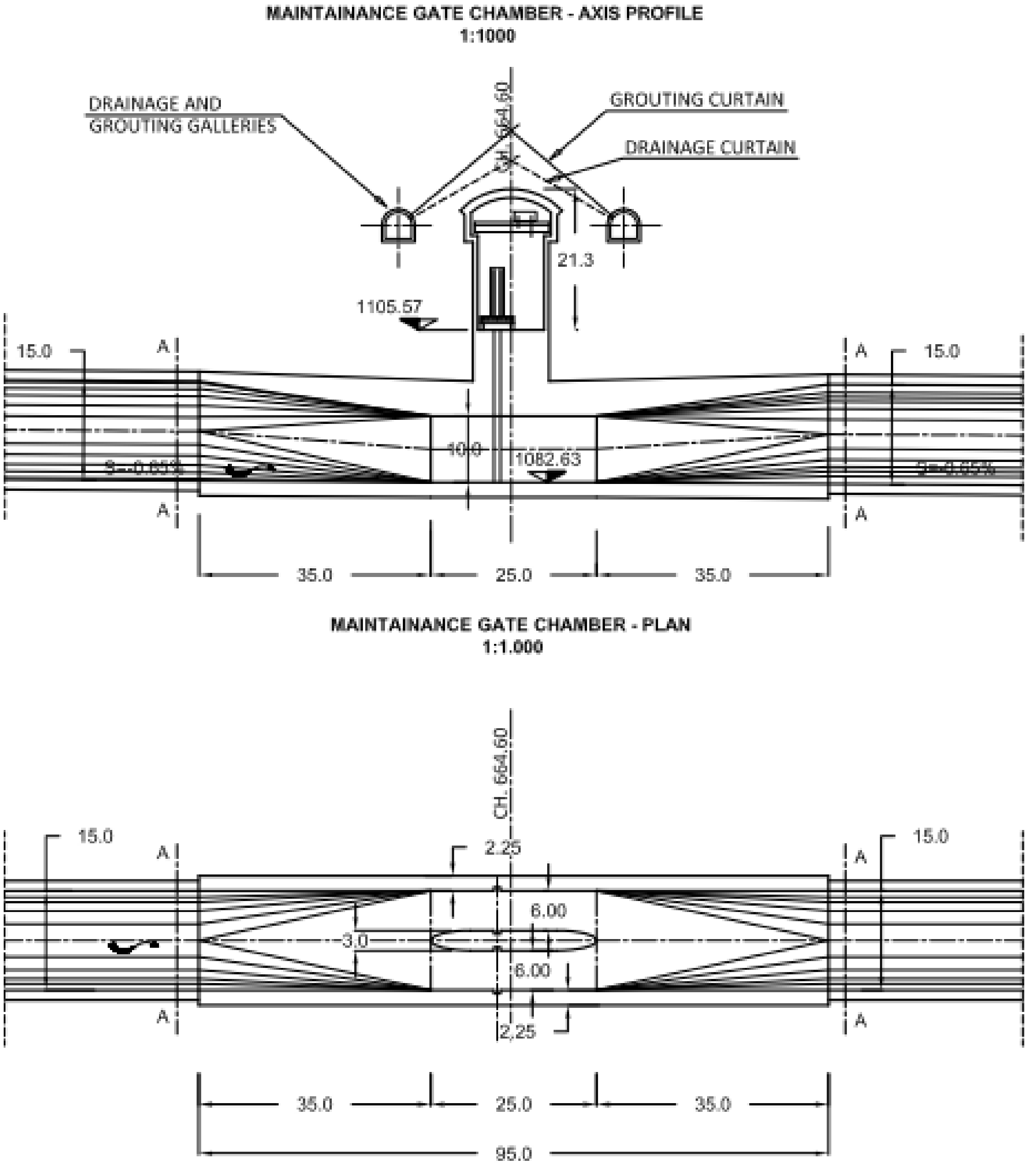



Figure 14.3: Dimensions and characteristics of the maintenance gates chamber of mid-level outlet 1

*14.2.1.3 Sector and emergency gates chambers*

The sector and emergency gates chambers of the midlevel outlet 1 has dimensions and characteristics similar to the sector and emergency chamber of the third level diversion tunnel (see Figure 14.2), except that the total length is reduced to 118 m. Invert elevation is around 1080.

It is likely to be excavated, from upstream to downstream, in the hard sandstones of the Upper Obigarm, the Karakuz formation and likely the Mingbatman Formation at the lower end and at the location of the vortex.

No major problems would be expected for excavation of this chamber, but it is actually located in the immediate vicinity or within the limit with the “disturbed zone”. Gallery 1034 showed that, even if there was not discontinuity in the bending, it is sharply bent over in this location, and cut by some faults with zones of crushed rock, subvertical, and trending more or less parallel to the limit of this “disturbed zone”.

Therefore, excavation of these gates chambers shall be anticipated with care, since fractured rock may be encountered.

This drawback is however manageable if the studies, modeling, and subsequently excavation and monitoring are made following state-of-the-art rules.

### *14.2.2 Gates chambers of mid-level outlet 2*

*14.2.2.1 Maintenance gates chamber*

Dimensions and characteristics of this maintenance gate chamber are quite similar to those of the maintenance gate chamber of mid-level outlet 1 presented in Figure 14.3.

The chamber is located at elevation 1137, within sandstones of the Kyzyltash Formation, for the upstream part, and siltstones of the Lower Obigarm downstream.

Excavation shall proceed with care, after detailed study, modeling, and with monitoring, in divided sections, because of the presence of the siltstones, and the unfavourable attitude of the bedding joints.

*14.2.2.2 Sector and emergency gates chamber*

The sector and emergency gates chamber of the midlevel outlet 2 has dimensions and characteristics similar to the sector and emergency chamber of the third level derivation tunnel (see Figure 14.2), except that the total length is reduced to 180 m. Invert elevation is around 1134.

It is likely to be excavated mostly in hard sandstones of the Upper Obigarm. In the upstream part, the Lower Obigarm siltstones and zones of crushed rock linked to the limit of the “disturbed zone” may be encountered. The rock may be slightly more distressed, because of the proximity to the assumed lower limit of geotechnical Zone III.

Provided that excavation and monitoring is made adequately, such situation is however manageable.

## 14.3 Rock conditions and design of surface spillway excavations

### *14.3.1 Description and location of the surface spillway*

In order to be able to have a permanent and visual control of the spillway structures, three parallel surface spillways has been designed, which are to be located in the right bank, downstream of Ionakhsh Fault and apart from Fault 35, in order to avoid as much as possible to be affected by creeping tectonic deformations.

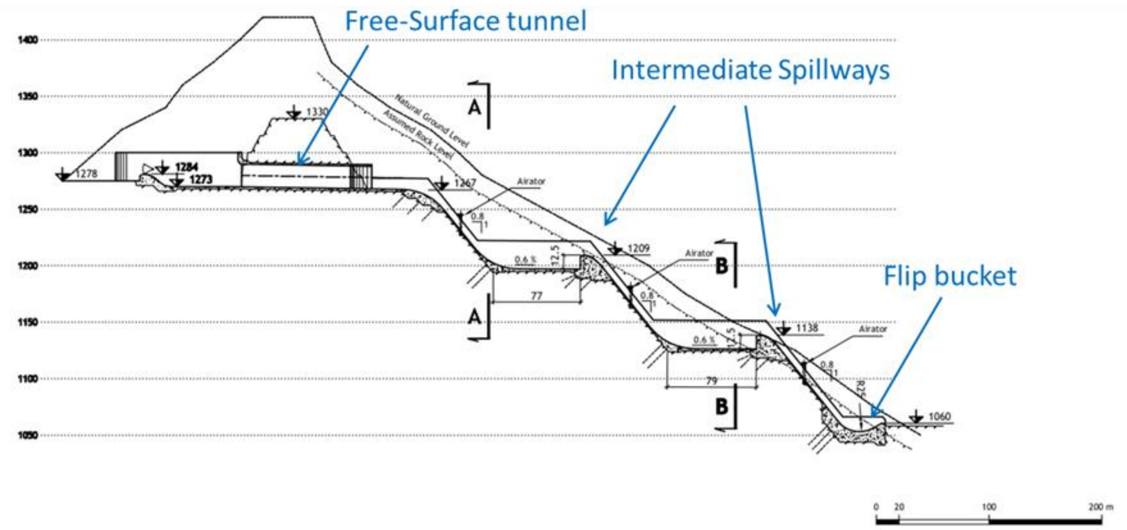



**Figure 14.4: Cross-section along one of the surface spillways**

Figure 14.4 presents the typical cross-section along one of the three channels of the surface spillway structures. Given the exceptional height of the dam, it is necessary to dissipate the energy of the spilling water before reaching the Vakhsh River, whereby two intermediary dissipation basins have been designed on the slope.

It can also be seen that all three channels composing the surface spillway cross the rock spur on the right side in tunnels, designed to be free-flow tunnels. Location within the topography of the right bank of the surface spillway structure can better be assessed from the plan view of Figure 14.5.

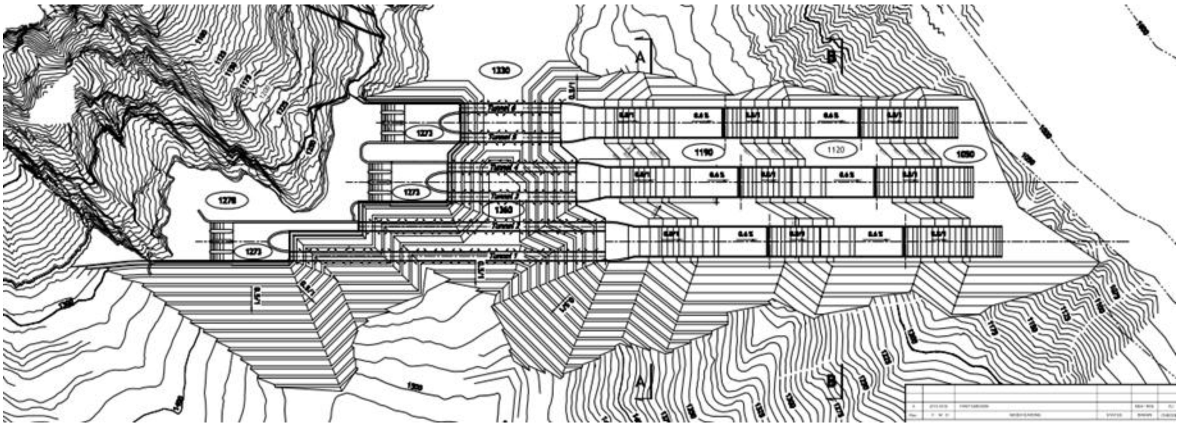


**Figure 14.5: Plan view of the surface spillway structure and excavations**

### *14.3.2 Geotechnical conditions of the surface spillway structure*

The trace of the surface spillway structure, originating from the right side of the final dam crest, goes below the rock spur of the right bank and follows its course towards the downstream right bank.

As a consequence, and as can be seen from Figure 14.6, all three channels of the spillways are to cross the eastern limit of the "disturbed zone" and be partially founded on its slope. The layout has been adjusted as a minimum of the colluvium resting on the slope is affected.

The overall geological structure, apart from surface screes, is rather favourable, since the axis of the spillway structure makes an angle of roughly 40 to 45 degrees with the (horizontal) direction of the bedding and joint set 3 (see Figure 10.2). Joint sets 2 and 4 are dipping favourably. This however does not imply that unfavourable joints are not be encountered, depending on the exact geometry of the excavations, requiring adaptation of the support.

Therefore, the main difficulties to be expected in the construction of the spillway are expected to be more related to the rock mass quality and its density of fracturing rather than from unfavourable joint sets.

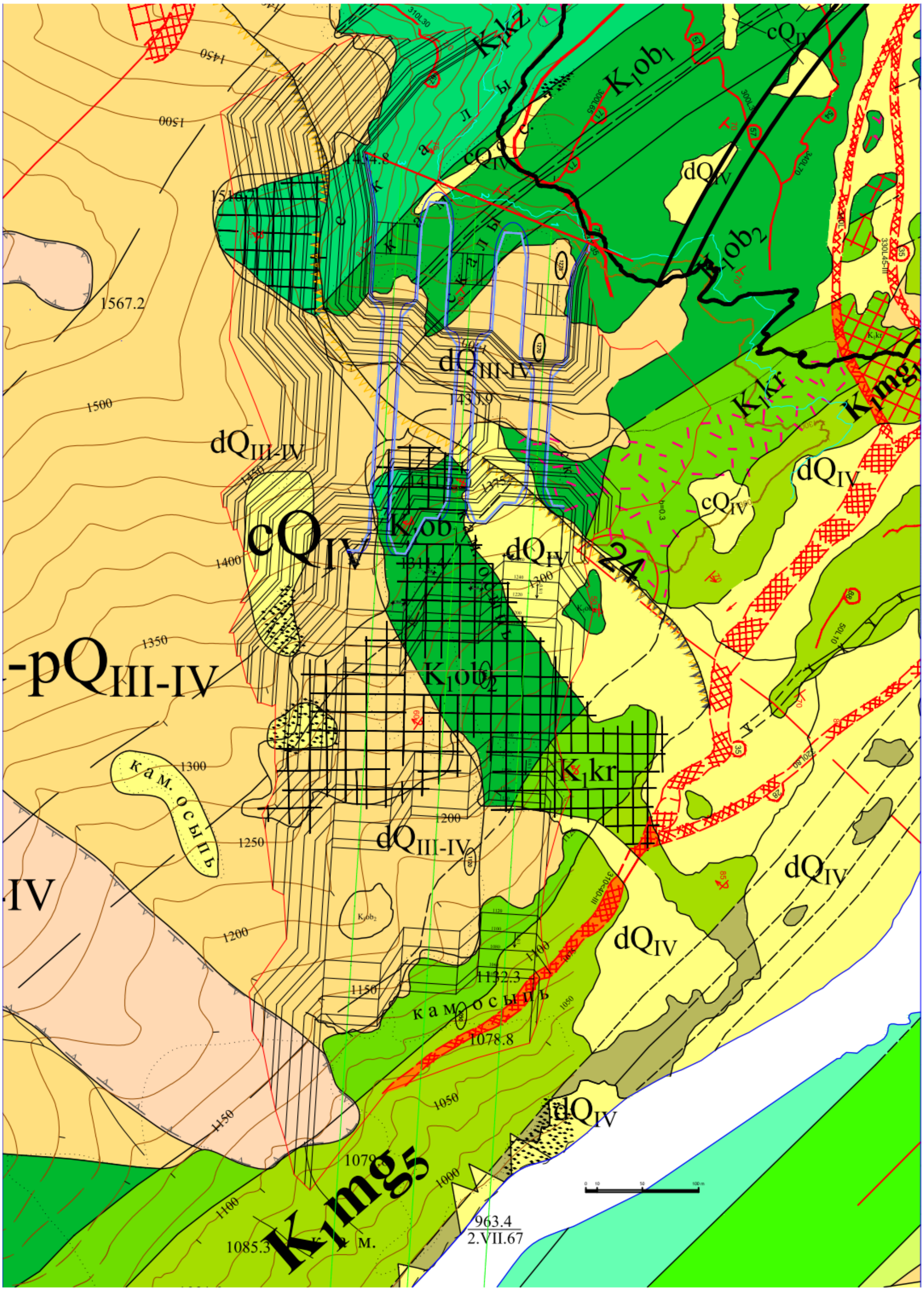



**Figure 14.6: Plan view of the surface spillway in its geological environment**

In this respect, two aspects deserve consideration:

- The rock quality and density of fracturing along the eastern limit of the disturbed zone, where bedding is sharply bent over within a short distance; observation of adit 1034 showed some consequent fractured zones were present when nearing this limit,

- The presence of the Lower Obigarm siltstone and claystone layer, which is, as described above prone to weathering once distressed and exposed to meteorical agents.

The eastern limit of the "disturbed zone", where crushed rock is locally expected within faults, runs through the excavations of the upper part of the spillway structure, and crosses the three tunnels.

The Lower Obigarm layer, following the dam crest, is to be encountered in a large part of the right side excavations of the spillway structure, due to the inversion of its bedding at entering the "disturbed zone".

### *14.3.3 Design criteria for realization of the spillway structure*

#### *14.3.3.1 Surface excavations*

Considering the above, the design criteria for determination of the rock support necessary for surface excavations depend upon:

- Location within the limit of the "disturbed zone" or within the Lower Obigarm siltstones,

- Presence of unfavourable dip of layers or discontinuities.

In order to limit the amount of excavations, slopes have been designed at 0.5 horizontal to 1 vertical, except for the slops required for the efficiency of the stilling basins, which are of 0.8 horizontal to 1 vertical. The corresponding distribution of slope inclinations is shown on Figure 14.7.

The support necessary for slope stabilization was deduced in order to secure such slope inclinations, taking into account that excavations shall be permanent during the life of the scheme, and that their height reaches locally more than 200 m.

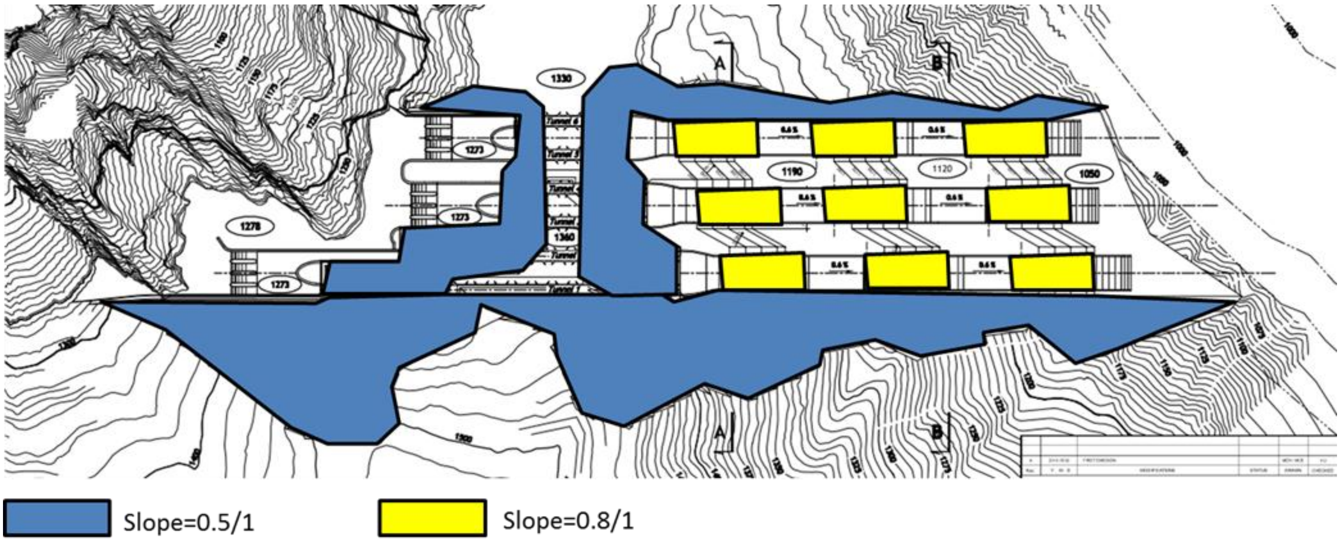



**Figure 14.7: Definitions of slopes of surface excavations of the surface spillway structure**

Two types of support were defined, depending upon the location, as previously defined. This was done for each of the two slope inclinations considered, the corresponding support definition is presented in Table 14.1. The length of the anchors has been provisionally taken as 12 m. The shotcrete is to be reinforced by wire mesh.

| | **Slope 0.8/1** | | **Slope 0.5/1** | |
|---|---|---|---|---|
| | **Type I** | **Type II** | **Type I** | **Type II** |
| **Anchorage Diameter [mm]** | 25 | 25 | 32 | 25 |
| **Anchorage mesh** | 2.5m*2.5m | 2m*2m | 2.5m*2.5m | 2m*2m |
| **Shotcrete thickness (m]** | 0.20 | 0.15 | 0.1 | 0.20 |

**Table 14.1: Support definition for surface excavations of the spillway**

The assumed distribution of the two types of support within the surface excavation of the spillway structure is shown on Figure 14.8, taking into account two criteria of location and unfavourable dip.

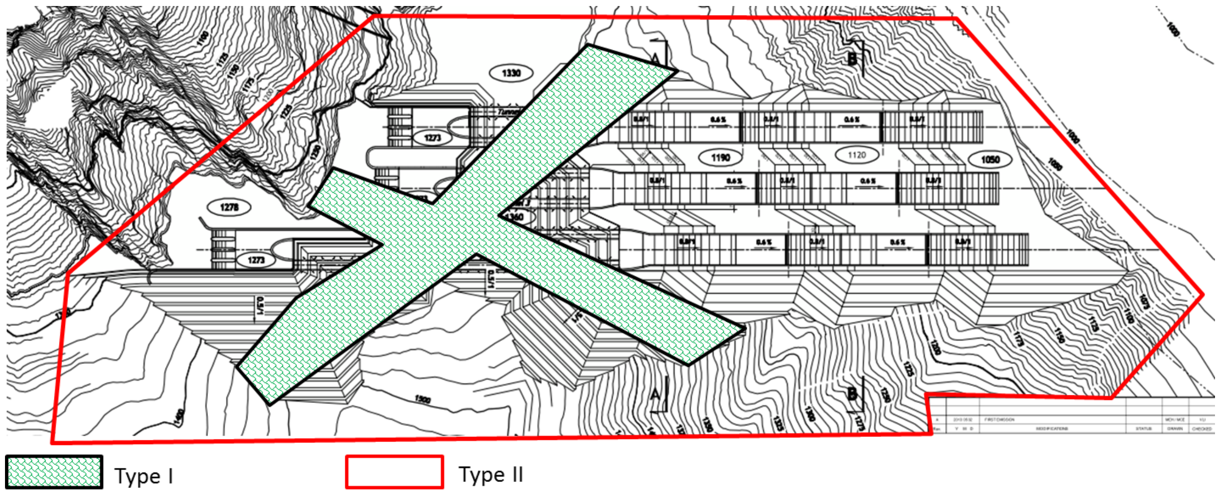



**Figure 14.8: Distribution of support types within the surface excavations**

Slopes left blank between the channels on Figure 14.7 are actually the steepest and a specific support has been designed for those intermediary slopes between spillway channels. It consists in 25 m long anchors of 32 mm diameter, at a mesh of 1.5 m, and tendons of 40 m length (one for 4 anchors), with some 200 mm thick shotcrete reinforced by wire mesh.

Additional support will of course be provided to warranty the stability of the slabs of the spillway channels and stilling basins.

*14.3.3.2 Tunnel excavations*

Considering that the tunnels are to be excavated mostly within the Lower Obigarm siltstone, and below low overburden, in distressed zone, support is likely to be important. A support with steel ribs and wire mesh-reinforced shotcrete is contemplated, over almost half of the cumulative length. In sections with less fractured and only few distressed, the support will be reduced to anchors and reinforced wire mesh.

A final lining of reinforced concrete is contemplated over the whole length of the tunnels, with contact and cavity grouting to be performed for a quality bond with the embedding rock.

## 15 CONCLUSIONS AND RECOMMENDATIONS

Main conclusions and recommendations consecutive to the assessment of geotechnical conditions of the dam site made in this report are listed below. Generally speaking, geotechnical conditions of Rogun dam site, although presenting some particularly acute difficulties, should not compromise the feasibility of the Project, under the conditions that adequate measures are effectively taken, especially with regard to fault creeping and stability of slopes, with selection of high-quality, perennial monitoring instruments, suitable with the extreme weather conditions of the site. The major issues are recalled here below.

## 15.1 Movements associated with fault creeping and possible co-seismic displacements

Monitoring carried out as early as 1968 and until 1992-1993 evidenced the slow creeping movements of Ionakhsh Fault, Gulizindan Fault and Fault 35. The tectonic fracturing pattern and the freshness of cracks observed on site are indicative, in the Consortium's opinion, of readjustment movements occurring even within the block limited by Ionakhsh Fault and Fault 35. No long term measurements being available between the two faults, the actual kinematics of movement is not known (it may be negligible with respect to the lifetime of the scheme, but remains to be demonstrated). A special design is to be applied for underground structures wherever they cross Ionakhsh Fault, but also Fault 35 and main faults of main attitudes (such as Fault 70), along which co-seismic displacements have been estimated as possible. These are in our opinion the minimal mitigation measures to be taken for safe operation of the scheme, and ideally it should be made possible to inspect all underground structures immediately after occurrence of an earthquake suspected to have generated co-seismic displacements on the site.

Associated to these creeping movements, similar slow angular variations were recorded along the moving faults, which may result in changes of inclination of the turbine axis; adequate design measures are to be taken with regard to this aspect as well.

Preventive measures for hydraulic tunnels crossing creeping faults exist, and although it shall be studied with more detail, provided these measures are taken, the problem can be dealt with. Moreover, solutions proposed by the Consortium in terms of layout of the underground works and the surface spillway allow to avoid as much as possible the crossing of creeping faults.

## 15.2 Detailed investigation of the “disturbed zone” of downstream right bank

Investigations performed in 2012 on the demand of the Consortium allowed discarding the possibility of re-activation of huge landslides within the “disturbed zone” (the total volume of materials involved in old landslides being previously estimated to more than 500 million m3). A close geodetic monitoring as scheduled is to be performed, especially at the time of impounding and during operation, in order to check its behaviour, and add drainage means where necessary. However, the following issues remain to address.

Unstable zones of colluvium are present on the front slope of the disturbed zone, and landslides of up to several hundred thousands of m3 may slide to the river with formation of a temporary dam (even in present conditions, e.g. in case of major earthquake). In order to avoid occurrence of such event, it is recommended to remove from the slope, as much as possible, the concerned colluvial materials and to re-profile the slope. Adequate drainage galleries are deemed necessary for stabilization of the remaining materials, but design of these measures necessitates a good knowledge of the geological and hydrogeological conditions in this area, and especially location of the failure surface(s). It is therefore recommended to install inclinometers to locate it (them) and characterize the type of movement of this mass.

This task can be achieved without major problem, since it consists in a classical stability analysis of rock masses and design of associated mitigation works (unloading, drainage and support). It does not impact the feasibility of the Project.

Incorporation of the “disturbed zone” into the hydrogeological model is recommended, to fine-tune actual results. This also requires the knowledge of the exact arrangement of geological formations of the “disturbed zone” below reservoir level.

Similarly, some more information about natural groundwater levels in this area are recommended for the detailed design stage.

### 15.3 Stability of upstream left bank slopes

Creeping of rock masses along joints of set 4 (of similar attitude as Fault 35) has been identified and deemed to be the cause for damages to the portals of the construction tunnels. Since those rock masses will be located just upstream of the dam, dedicated geological and geotechnical investigations are to be carried out in this area to design the reinforcement measures to be taken. Especially, location of faults, thickness, geometry and persistence is to be investigated to define those measures. Monitoring of the slopes, considering the importance of the issue, is a must.

Once again, this is a problem of rock slope stability, for which the remedial measures are known and shall be implemented, but which do not impact the feasibility of the project if adequately dealt with.

### 15.4 Additional geotechnical investigations

The general results of geotechnical investigations performed for Rogun dam are globally well assessed, and only some minor modifications have been recommended, which are actually already known (experience from power house convergences). Test methods which were employed to obtain these results are therefore globally adequate. Cross-checking with others methods have proved results to be adequate to conclude on the feasibility of the Project.

Nevertheless, a campaign of geotechnical tests on rock materials, comprising laboratory tests and in-situ tests is recommended to be performed.

The first reason for this is that, as stressed by HPI in 2009, practically all tests were made for the Original Project of 1978 or sometime after, during construction period, and some actualization is needed. The second reason is that design of new types of works proposed by the Consortium, such as the surface spillway will in any case necessitate performance of new tests.

Especially, actual shear strength of the different types of discontinuities needs to be better characterized, and will be required for project of stabilization of the potential landslides. Investigations of formerly collapsed zones in the construction tunnels are also to be performed for their safe design (collapses have left voids up to 25 m or even more above the vault of the tunnel where crossing Fault 35, and HPI mentions a niche created in the left bank just above this location).

Although results provided by testing performed according to Soviet and then, Russian GOST standards have proved to be quite relevant, few people are now familiar with them outside of the countries of the former Soviet Union. In view of international bidding, it is hence highly recommended that this additional campaign of geotechnical testing is performed according to internationally recognized standards familiar to Contractors, like ISRM suggested methods, ASTM, or others.

Specific geotechnical investigations to be carried out for the salt wedge of the Ionakhsh Fault are dealt with in Report P002378-R38. Investigations for constructions materials are dealt in a separate report.

## 15.5 Excavations of the dam foundations

As emphasized in the report, and given the very active geodynamical context of the site, it is absolutely necessary, prior to beginning with dam foundation excavations, and then dam construction, to secure the whole site by adequate scaling of unstable blocks and/or reinforcement of the slopes. With regard to the site conditions, this is a huge task to perform, that we recommend starting as soon as possible. Securing the site should then avoid deadly rock falls and allow smooth unfolding of the dam construction.

## 15.6 Backfilling of investigations galleries and transport tunnels before starting impounding

Before starting impounding, it is necessary to achieve complete clearing of the former investigation galleries, and especially those going from upstream to downstream, and to carefully backfill them with concrete. Transport tunnels should receive similar treatment by that time, and concrete plugs with local curtain grouting may be required in some locations.

## 15.7 Mitigation measures for Obi-Shur mudflows

Although the Obi-Shur dam is not directly part of the project layout, while few data are available about the on-going mitigation works against mudflows, we suggest in the report that, considering the size and frequency of the mudflow events, real-time monitoring should be installed, in order that in case of large mudflow occurrence, all precautions can be taken (closure of accesses, valves, etc.) to cope with a temporary increase of the downstream level, even if operation is to be stopped temporarily.

## 15.8 In–situ stresses measurements

In a similar way as geotechnical investigations, and considering the progress made since the 1980's in terms of in-situ stress measurements, assessment of in-situ stress field around the underground excavations by means of hydraulic fracturing may be performed if deemed necessary for underground works.

## 15.9 Necessity of employing sulphate-resistant construction materials

Considering the presence of the salt wedge of Ionakhsh Fault, and presence of gypsum in various proportions within the dam foundation, all construction materials, and primarily cement, shall be sulphate-resistant.