REMEDIAL MEASURES FOR SAMANALAWEWA LEAKAGE AND STABILITY OF THE RIGHT ABUTMENT BASED ON GEOLOGICAL AND HYDROGEOLOGICAL DATA

by

Vernon F. Pereira

1.0 Introduction

Samanalawewa is located about 160 Km South-East of Colombo, about 10 Km East of the town of Balangoda. The project consist of a zoned rockfill dam with a vertical centre clay core on the Walawe ganga (just downstream of the confluence with Belihuloya) with a crest length of 530 m and a height of 100 m above river bed level, a gated spillway, a waterway system, 4.5 m diameter, 5.4 Km long tunnel with an intake structure about 6 Km upstream of the dam on the Walawe Ganga, a surface power station with an installed capacity of 120 mw (2 x 60 mw) and a tailrace canal connected to the Katupath Oya.

The main project objective was to meet the growing demand of electrical energy for Sri Lanka with an installed capacity of 120 MW. The Samanalawewa hydropower project is designed to produce a firm 420 GWH of electricity annually and some secondary energy. A secondary objective would be the increase of the irrigation capacity of Udawalawe. Successful commencement of impounding of Samanalawewa was expected in January 1991.

The Samanalawewa Dam & Reservoir Project encountered severe leakage problems even during the first filling. The reservoir was therefore not raised to Full Supply Level and the Project is due for major remedial works. Detailed studies were made during construction in order to ascertain particularly the water tightness of the immediate right abutment and saddles for cut-off works. These studies mainly consisted of 4800 m of drilling investigations to depths of the order of 200 m with piezometers installed. The piezometer monitoring within a few months revealed a flat groundwater table at river level responding to river fluctuations. Constructing a blanket on the invert and on the steep slopes of the reservoir in the suspected ingress area was considered at the initial stages for cut-off design. Subsequently, however a 105 m deep extensive grout curtain from an 1315 m long adit driven a little above river level was decided to be provided as a positive cut-off measure. Grouting from this gallery towards the top in a selective manner to elevations varying from E*l.* 420 m to FSL at E*l.* 460 m was also done. The main curtain below the valley, with 53,600 m drilling/13,450 tons of cement grouted with the maximum take of 15.5 tons/linear metre, proved ineffective and no hydraulic gradient could be created across it. The first trial impounding in June 1991 disclosed the previous condition for the rightbank with leaks occurring when the reservoir had barely risen to E*l.* 398 m i.e. about 30 m above the river bed against the intended height of 90 meters to E*l.* 460 m.

The second trial impounding commenced in March 1992. Potentiometric levels rose in the right bank ridge to the reservoir level within about one days time lag as earlier. Leaks slowly increased, progressively small earthslips occurred in the weathered mantle on downstream of dam along right bank. Water came into the dam cavity increasing the volume in the dam seepage measuring chamber. Also about 3 Km far away in Killekandura area on the leftbank downstream of the dam a leakage of about 40 *l*/s occurred when the reservoir was around E*l.* 424 m.

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This was noticed in June 1992 for the first time. The sudden burst 320 m downstream of the dam on the right bank around El. 400 m bringing out much more than 7 cumecs occurred in October 1992 when the reservoir was at El. 438.99 m; at the same time all the piezometers dropped drastically by almost about 25m down to El. 415 approx. The discharge decreased to 2 m^3/s, then the GWL rose again by 6-7 m presumably due to partial obstruction of the natural leakage privileged path around the hydraulic control section. It is observed that with the drainage which stays around 2 cumecs that the difference between reservoir water level and ground water level stays around 10 m for reservoir level variation from El. 426 m to El. 433 m so far. The point to be taken note of here is the fact that the long term average annual inflow at Samanalawewa dam is only 19 cumecs. Another factor which has to be considered is to what degree the natural unplanned but privileged path developed during the burst can be relied upon as a dependable measure to keep the piezometric pressure lower than the water level in the reservoir.

Internationally reputed experts were consulted 1) to assess the stability of Samanalawewa dam, as built and under the present hydro-geological condition 2) to assess the stability of the right bank of Samanalawewa Reservoir under present and short term future conditions before remedial measures are complete and 3) to review the available proposals for conceptual design of remedial measures for the Samanalawewa dam and reservoir and to suggest the most appropriate proposal.

Concluding the panel stated that the dam is safe for full reservoir operation and that work is required to reduce leakage for economic and not safety reasons. The dam and the right abutment rock just downstream of the dam is safe against high abutment piezometric levels and drainage is not necessary. The unusual geological conditions make improvement of the curtain unpractical. The most appropriate measure with the probability of significant reduction in leakage is blanketing.

From October 1992 to date (March 1995) the reservoir has been kept around El. 430 m (not letting it fill to El. 460 m, design TWL) in a controlled manner for partial generation of power. Geophysical investigation, seismic reflection survey on Samanalawewa reservoir to provide information on the ingress area/location of likely ingress areas in the reservoir have been conducted. Monitoring RWL, GWL and leakage are carried out on regular basis. Groundwater chemical analysis to identify leakage paths are also carried out since July 1992 on a monthly basis.

Whilst respecting the conclusions and suggestions of the Panel, it is the author's view that opinions on the stability of the immediate right abutment of the dam, on the nature and extent of the very pervious aquifer on which the reservoir is perched on and thereby on the success for wet blanketing are matters of concern.

An alternative set of remedial measures suggested are as follows : Internal drainage in multiple tiers to reduce pore pressures for stability of the hill which shoulders the dam and a grout curtain sub parallel to the internal drainage system to reduce reservoir leakage to desired levels are suggested. In the unlikely event of the leakage being still beyond acceptable economic limits pumping back to the reservoir against a head of 100 m to enable generation of power with the available head of 325 m is proposed.

2.0 The Weathered Right Abutment

The weathered right abutment, under drainage in the right abutment and saddles have been addressed in particular by the different investigating and reviewing parties since 1958. Several proposals have been made to take care of the problems identified in the right abutment.

2.1 Treatment of the Right Abutment, Initial Proposal

As a result of the 1986/87 additional geotechnical investigations, the original dam axis in the Russian proposal was partly diverted, curving it downstream on the right abutment. The bed rock on the right wing of the dam axis (Russian) proved to be intensively weathered to the level of El. 420 m, some 40 m lower than the contemplated level of the dam crest. Investigations at approximately 30 m downstream of the original dam axis (Russian) on the right abutment proved that the depth of intensive weathering was not so deep, the bottom of the weathering at elevation 445 m against elevation 420 m (see Fig. 1).

The unresolved uncertainties regarding water tightness of the right abutment were expected to be confronted by driving four grouting/investigation adits at different elevations into the right abutment along the dam axis (see Fig. 1).

The adits were placed between El. 463.4 m and El. 384.6 m invert levels spaced at an average of 26.25 m in the same plane. Adit 'A' with invert level at El. 463.40 m was designed to be driven 100 m and adit 'D' with invert level at El. 384.6 m was designed to be driven 309 m. A system of jet flush and grouting was introduced to be done between these adits in the soft zone, consisting clayey silt, erodible material to provide a positive cut-off.

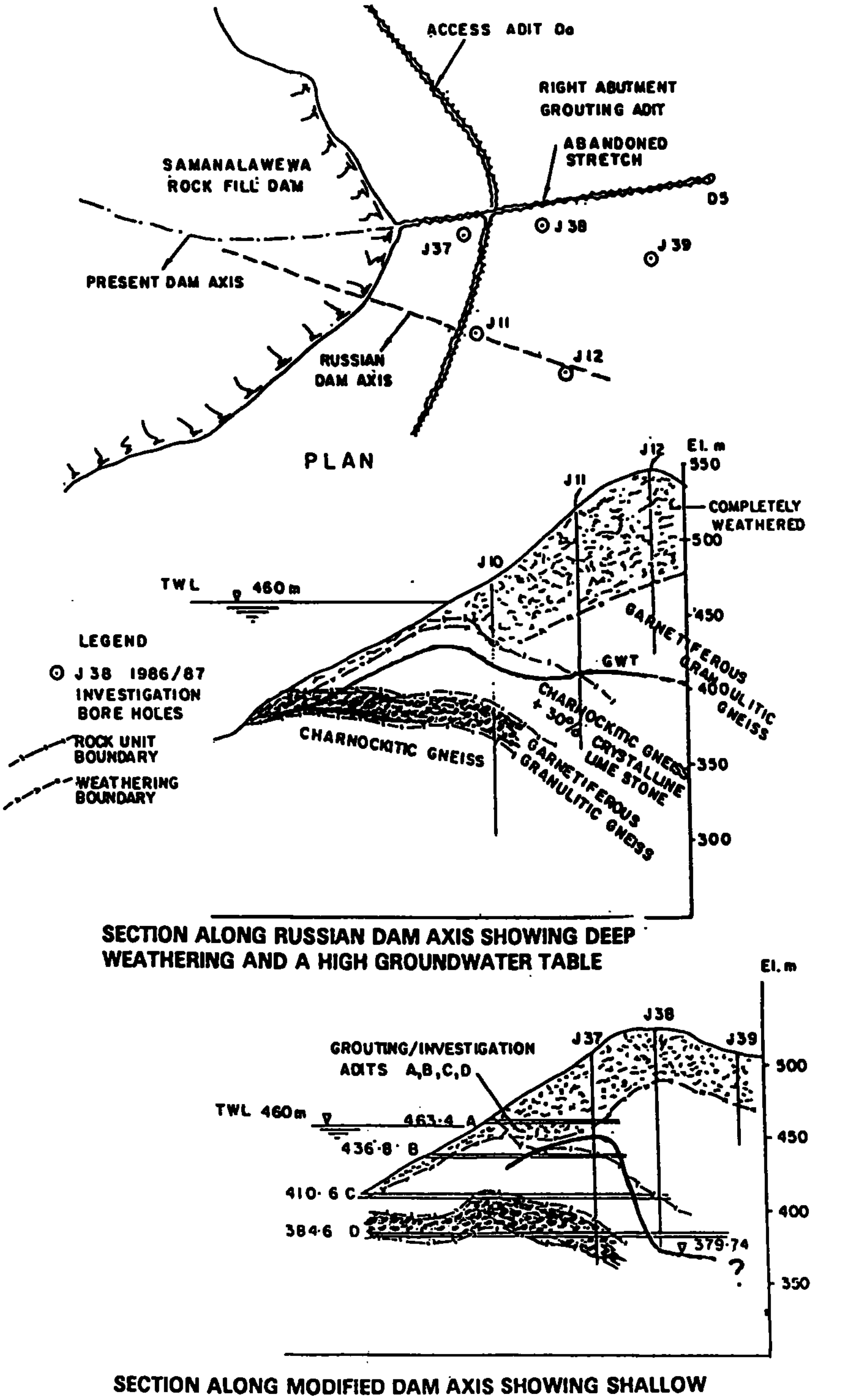


Figure 1 - The Treatment of the Right Abutment Initial Proposal

2.2 Initial Right Abutment Drainage Proposal

Twenty metres downstream of the four grouting adits at around El. 411.63 m a single drainage adit, 3.5 m in diameter 182 m long, running parallel to the grout curtain was designed at the onset of the project (and later canceled). The drainage curtain designed incorporated 55 mm diameter perforated PVC pipes at 5 m to 10 m intervals taken to El. 455 m and downwards to El. 395 m.

3.0 Under-Drainage

During construction, the excavation of the lower grouting adit 'D' revealed a fault zone and in adit 'C' a cave at El. 410 m in the limestone containing rock unit. Exploratory holes were then drilled from the lowest adit 'D'. These holes indicated another 50 m of fractured rock. Piezometers installed showed a groundwater level at El. 379 m. Permeabilities measured in these holes gave results of 5 - 80 lugeons. The design criteria used from the dam did not permit the grout curtain to terminate in the ground investigated by the 'D' series boreholes (see Fig. 2).

With the objective of assessing the geometry to tie the right abutment cut-off, deep boreholes were then done from the surface. These 200 m and more deep boreholes revealed continuing fractured rock and high permeabilities. It was only during construction in 1989, that piezometers installed in these boreholes revealed that there was a flat groundwater table at El. 380 m on the right abutment/rightbank and that it was responding to river fluctuations (see Fig. 2).

The features identified were indicative of potential seepage paths at depth, within the right bank as suspected before tender. It was therefore clear as earlier proposed that further study and investigation works were required for the dam right abutment cut-off works and in the saddle areas for any cutoff design (see Fig. 3).

During construction, the right abutment and all the five saddle areas were investigated by core drilling and permeability testing. Piezometers were installed in all 30 cored holes. The maximum depth drilled was 230 m. The total cored drilling was 4806 m. The deepest elevation reached was to El. 208 m cored from the grouting adit 'D' at around El. 388 m (see Fig. 2 & 4). The hydrogeological study with the aid of piezometers on the rightbank proved very effective, revealing two distinct groundwater tables. One was observed to be a relatively stable, high ground water table at around El. 400 m - 410 m localized on the right abutment hill. The other was observed to be a flat ground-water table at around El. 379 m extending to the 4th saddle approximately 2.5 Km from the dam on the rightbank where GW 18 was done (see Fig. 5).

4.0 Dam Right Abutment Cutoff

The right abutment cutoff works designed to be done from the four adits A, B, C and D were abandoned beyond chainage 601 m abandoning 126 m of tunnelling in adit D the lower most. The top most adit A and the bottom most adit D was diverted almost through 90° upstream to tie up the dam grout curtain to an area where there was a high groundwater table at El. 400 m - 410 m and good rock at depth (see Fig. 5).

4.1 Soft Zone leakage rout 1

The soft cavitated layer between the limestone containing rock unit and the garnetiferous granulitic gneiss rock unit proved to be thinning out towards the upstream (see Fig. 6). Diverted adit A(b) was driven 100 m and adit D(b) was driven 300 m. One of the leakage paths designated as rout I was intersected by these adits (see Fig. 4).

Adit A(b) encountered highly weathered to completely weathered rock. Adit D(b) encountered 1m to 2m wide slickensided sub-vertical faults striking (NNE-SSW) around chainages 90m, 110m and 223m.

A completely weathered shear zone the incline fault sub-parallel to the foliation was encountered between chainages 150m and 163m in adit D(b). Moderately to slightly weathered sheared rock, resembling a biotite schist was encountered between chainages 163m and 213m which was wet during excavation.

4.2 Dam Grouting

The total drilling for grouting from adits A(b) and D(b) was 15,649 m. Altogether 1254 tons of dry cement was pumped into this area. The average grout take was 80 Kg/m (see Fig. 6).

The total drilling for grouting along the dam axis between chainage 0 to 612 m was 15,540 m. The total consumption of dry cement was 71 tons. The average take was 4.6 Kg/m. The primary holes were spaced at 4 m. The cutoff was set at 3 lugeons.

However, it should be noted that between chainages 396 m to 612 m from right bank cutoff adits that most of the drilling and grouting was done in the dam. A total of 8018 m of drilling was done and 61.3 tons of cement was pumped into the soft, cavitated upper reaches of this area. Special grouting techniques such as jet grouting and high pressure grouting was done in the soft zone in the cutoff of the right abutment (see Fig. 6).

* PIEZOMETERS INSTALLED IN 1989 DURING CONSTRUCTION REVEALED THAT THERE WAS A FLAT GROUND WATER TABLE AROUND EL. 380 m WHICH RESPONDED TO RIVER FLUCTUATIONS IN THE RIGHT ABUTMENT AND RIGHT BANK.

* 51% OF THE DRILLING AND 86% OF THE CEMENT FOR GROUTING OF THE DAM GROUT CURTAIN WAS DONE BETWEEN CHAINAGES 396 m AND 612 m.

* DAM GROUTING, TOTAL DRILLING 15,540m, CEMENT TAKE TONS 70·9

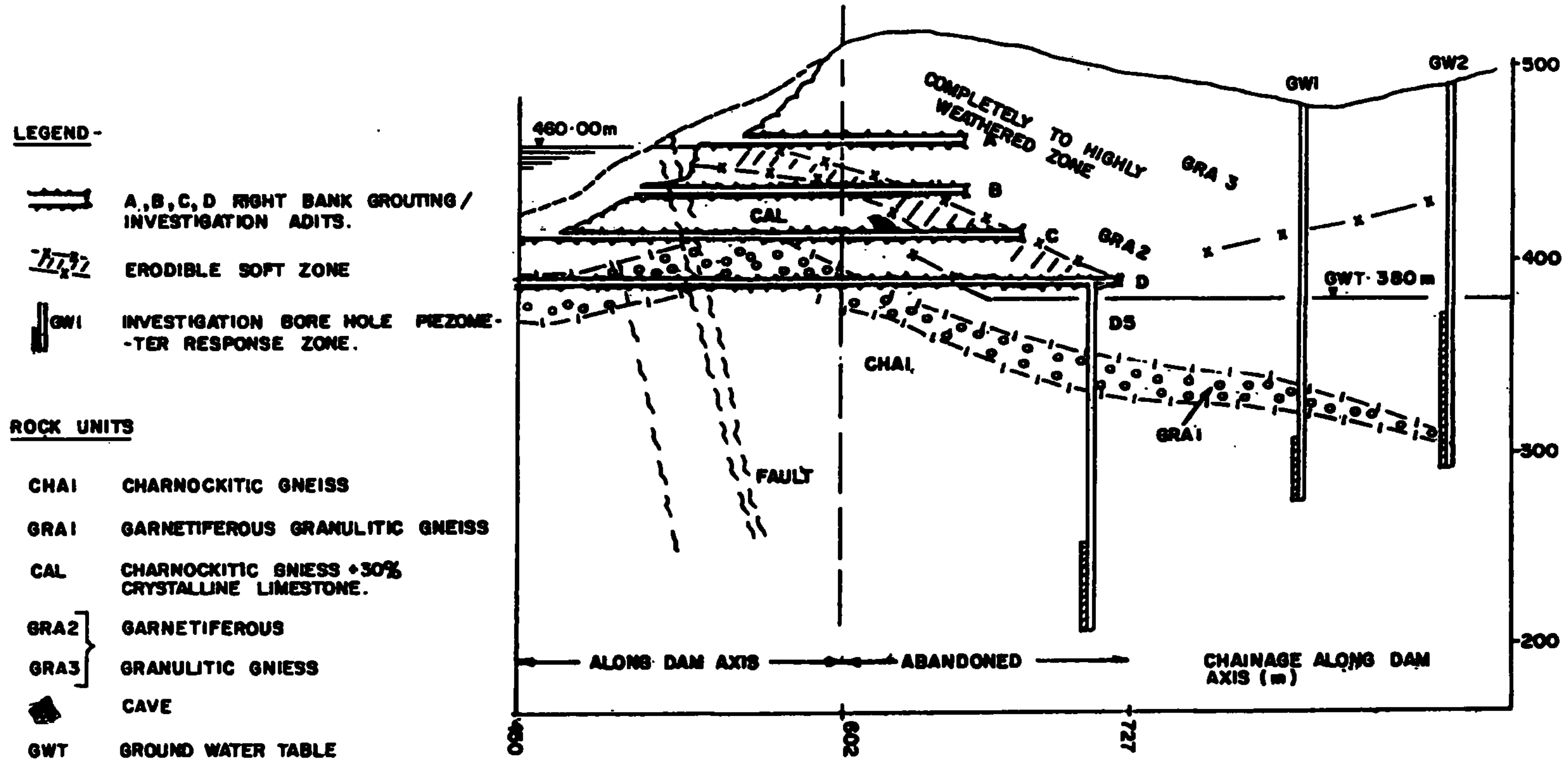


Figure 2 - The Deeply Weathered/Pervious Right Abutment

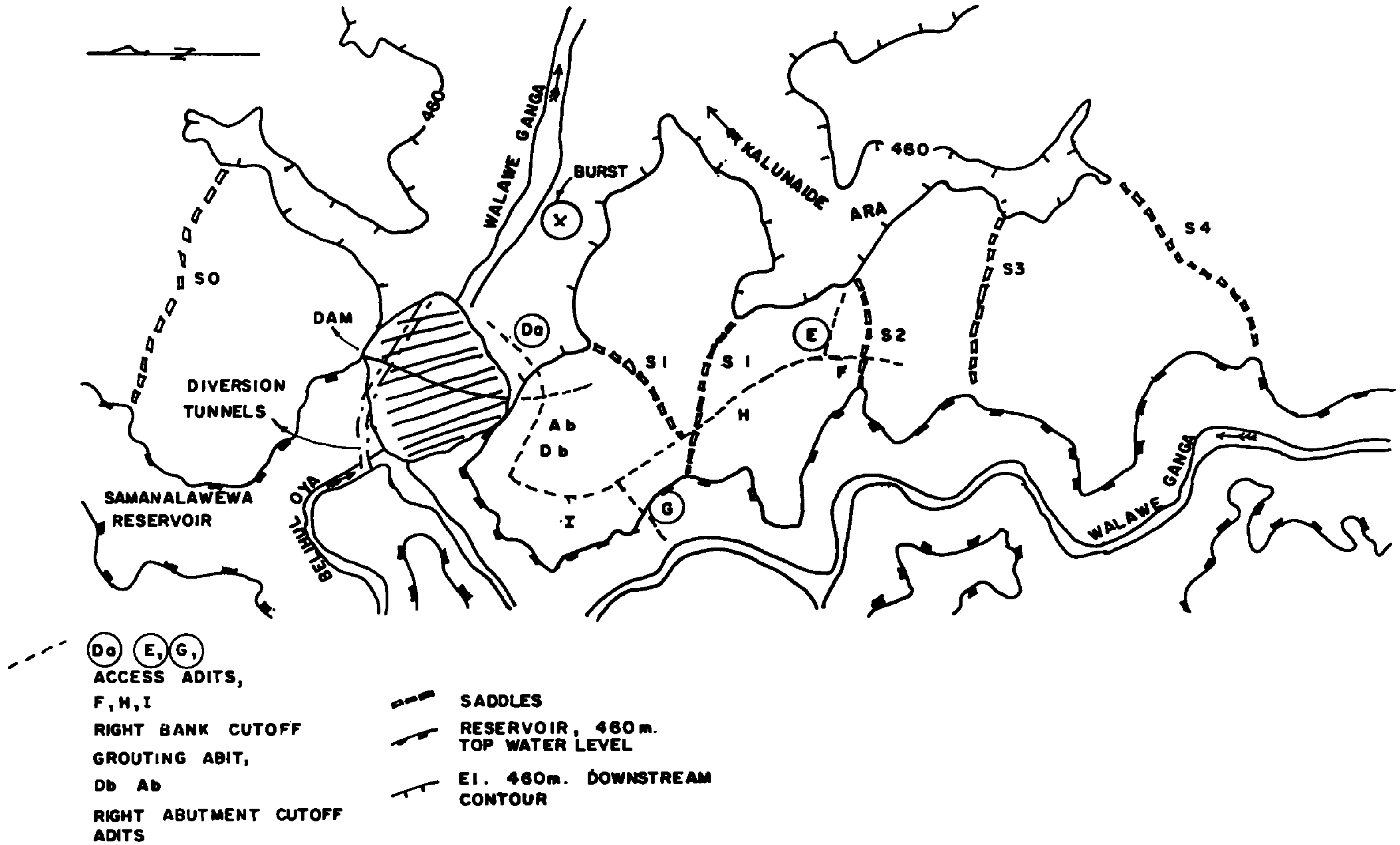


Figure 3 - The Five Saddles

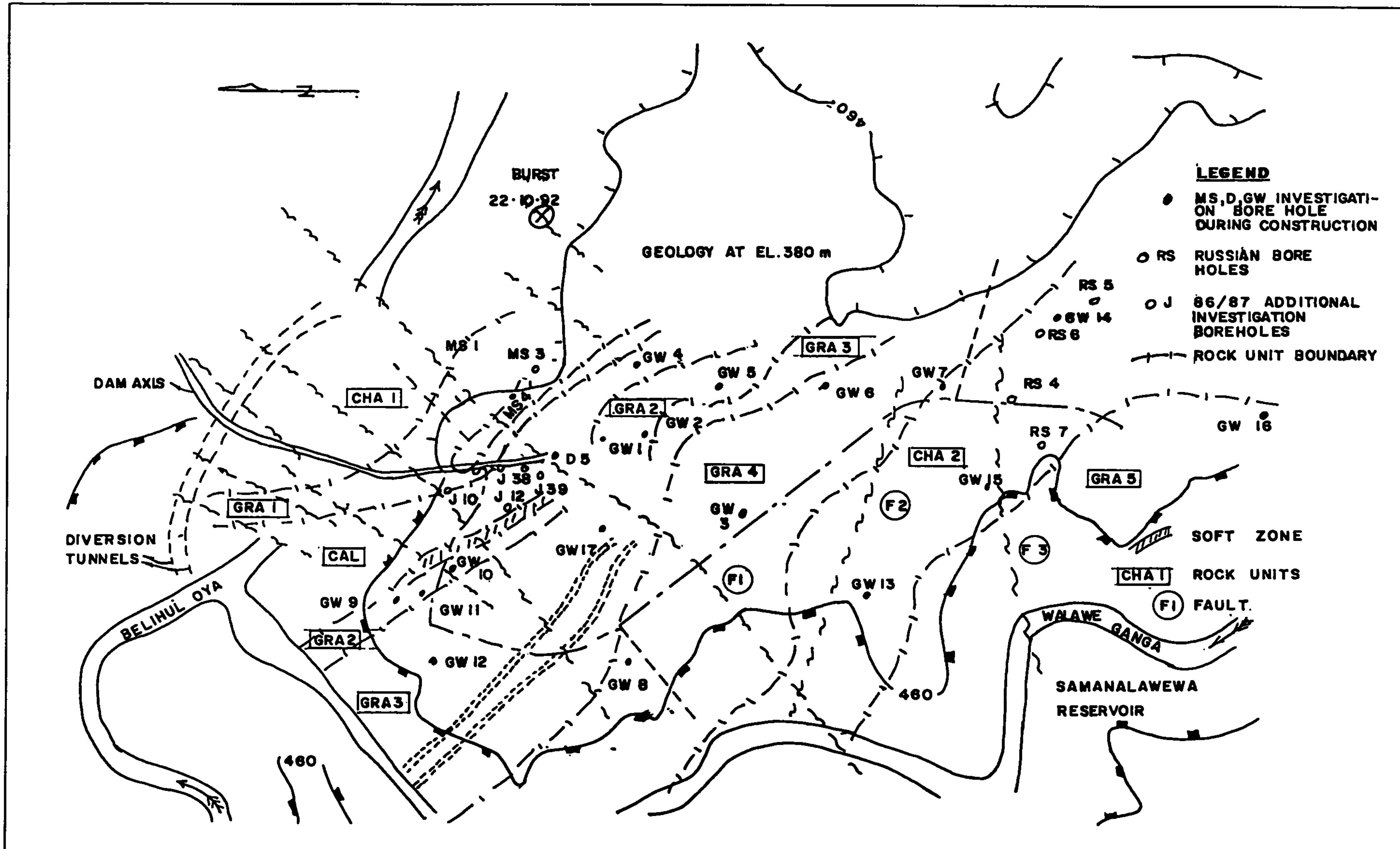


Figure 4 - Drilling Investigations during Construction

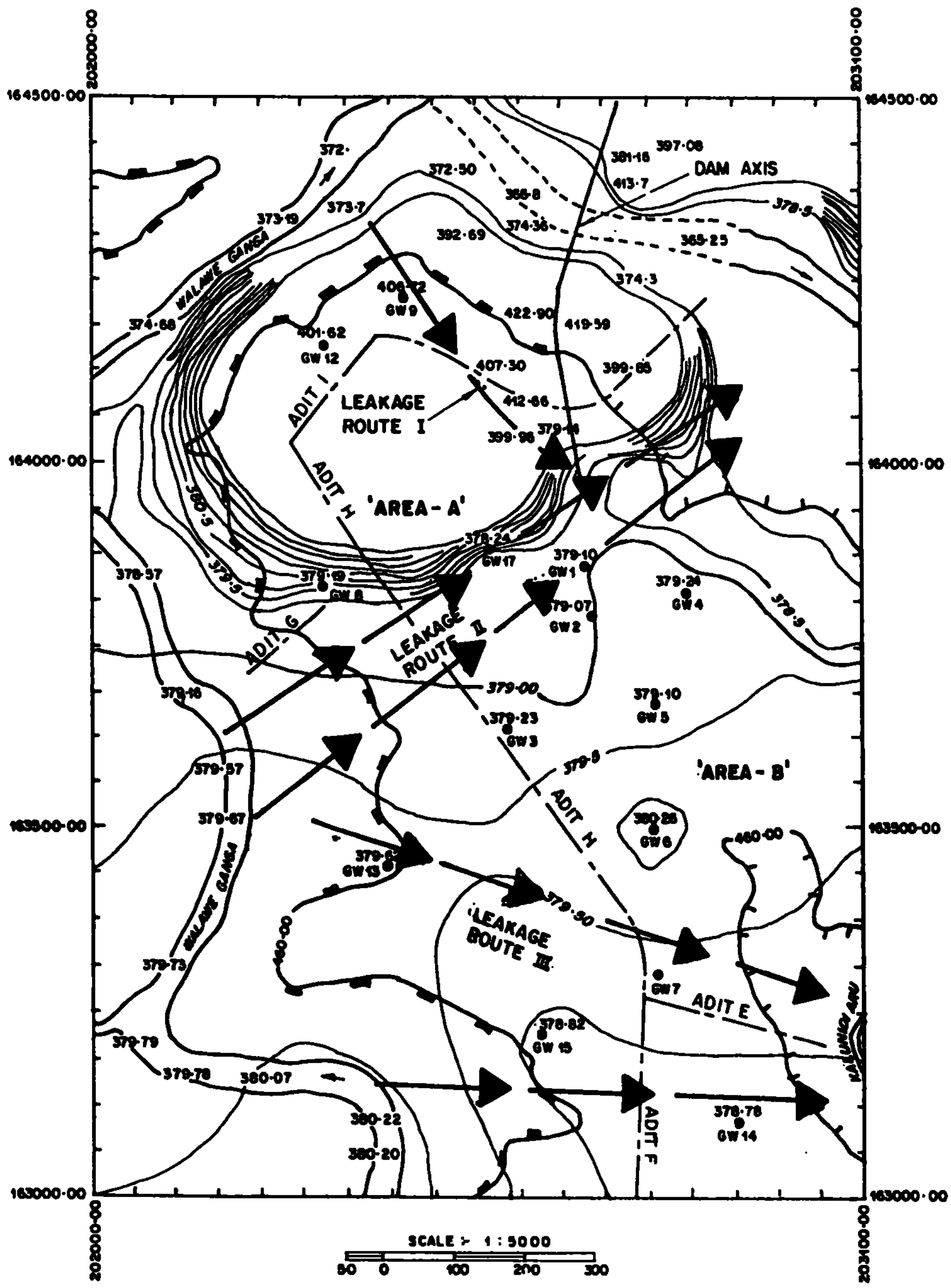


Figure 5 - Groundwater Contours, Leakage Routes and Rightbank Cutoff

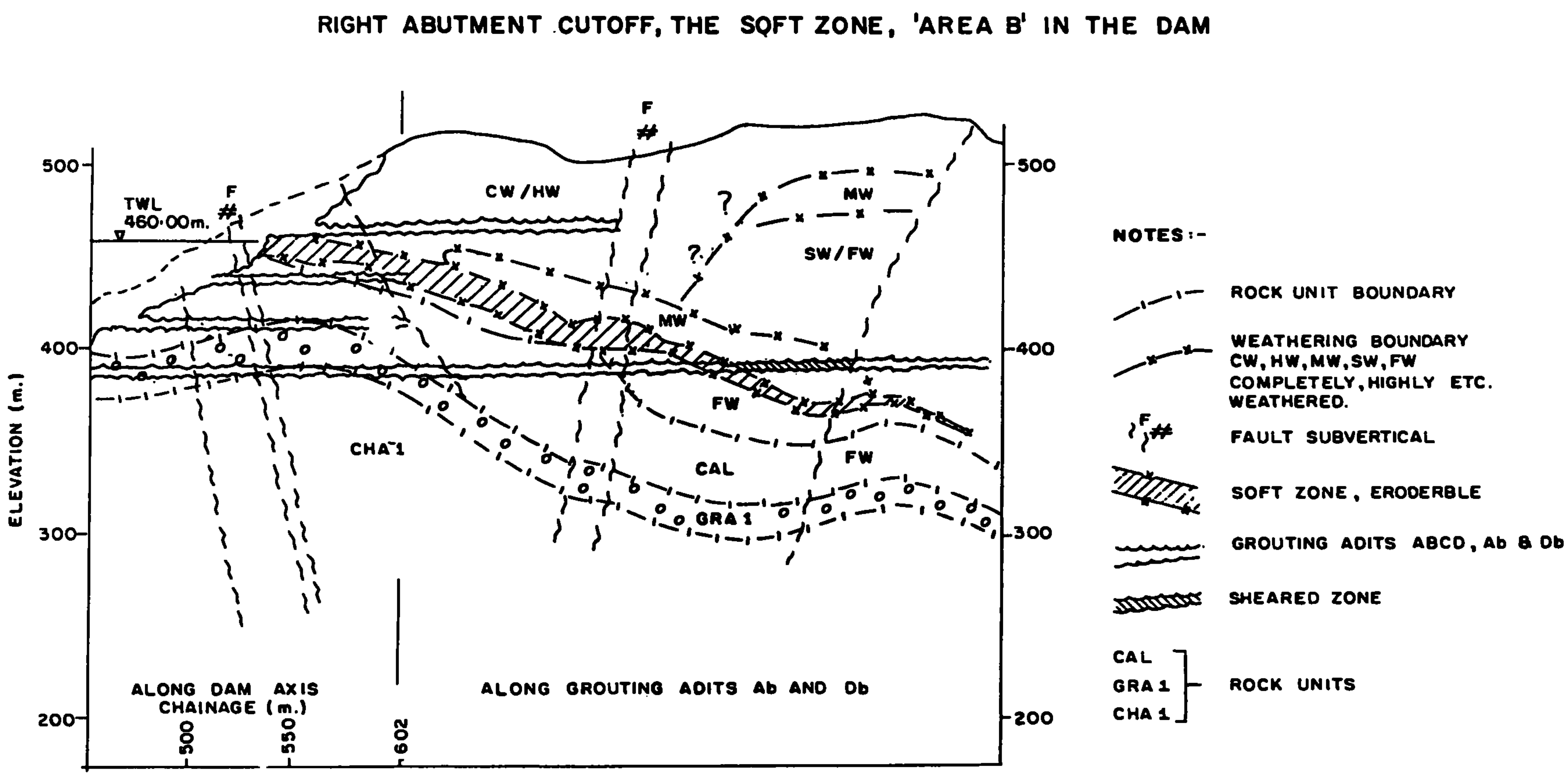


Figure 6 - Geological Section Along Dam Axis and Right Abutment Grouting Adits Ab and Db

5.0 Design of the Rightbank Cutoff

Three major leakage routes were envisaged by the designer at the time of the rightbank cutoff design (see Fig. 5).

Leakage route 1, the leakage path through 'area A' which is along the inclined thrust fault running sub parallel to the strike of the rock, the soft zone.

Leakage route II running along the NNE-SSW, strike slip fault F1, which cuts across the strike of the rocks, which also more or less marks the boundary of 'area A' and 'area B'.

Leakage route III through Saddles 1 and 2 running along the strike of charnockitic rocks and the EW strike slip faults F2 and F3. The faults are sub parallel to the strike of the rocks in this area. (see Figs 5 and 7).

In 'area A', the mineral calcite in limestone is present. Caverns and cavities in this area are broadly attributed to the work of surface water and air in circulation. The groundwater table in this area is high.

In 'area B', the calcite in the limestone has been removed by corrosion and alteration possibly by hydrothermal action. Corrosive features alterations and secondary deposits are prevalent in this area. The groundwater table is flat at E*l*. 380 m, at almost river level in this area (see Fig. 5).

The potential leakage paths were considered to be along strike, principally through karstic rock units and fault zones. It was also the view of the Designer that high permeability sub horizontal zones existed around levels E*l*. 380 m, El. 300 m and around E*l*. 225 m as inferred from the drilling and water pressure testing data. The grout curtain was invariably designed also to cutoff the two upper sub horizontal pervious zones as well.

Without any cutoff measures the Design Engineer estimated the leakage would be in the range of 1 to 6 cumecs. The Design Engineer considered the ground very permeable similar in nature to fine gravel or course sand. The bulk permeability of the privilege path was estimated to be 1×10^{3} cm/sec.

As stated earlier, leakage rout I was intersected by adits A(b) and D(b). Finally the right bank cutoff works was constructed from a continuous adit system with invert level close to El. 390 m, starting for Saddle 2 aligned across leakage route III, across leakage route II and linking to the dam right abutment adit D(b) leakage route I area right abutment cutoff (see Fig. 5).

The adit across leakage route II was located with the intention of meeting the central part of a comparatively tight rock unit (garnetiferous granulitic gneiss, GRA3) at the level around El. 300 m, the identified sub-horizontal pervious zone, to make grouting effectively cutting off this zone.

However, the grout curtain even at design stage was well known to be hanging. It was designed to reduce the quantity of water running downstream through the very pervious zones.

The right bank grouting adit 3.5 m in diameter is concrete lined, is 1315 m in length up to the right abutment cutoff adit D(b). The continuous adit system is self draining through the dam adits D(b) and the access adit D(a) located just downstream of the dam.

The grout curtain was limited to a depth of 105 m below the grouting adit at E*l*. 390 m irrespective of the grout take at the final stage. Grouting upwards from the gallery was in a selective manner to E*l*. 420 m in general and to E*l*. 460 m rarely. A total of 53,200 m had been grouted with 13640 tons of dry cement, an average of 256 Kg/m (600 liters/metre). After the grouting operation no gradient across the grout curtain was observed. The only conclusion was that the grouting operation plugged major holes to reduce leakage.

6.0 Reservoir Lining, Earlier Proposal

Lining part of the reservoir had been studied earlier before the rightbank cutoff was put into practice but had not been implemented for the following reasons. It was not possible to specifically target the areas for treatment by blanketing because of the lack of a well defined hydraulic gradient in the rightbank which made it impossible to locate the potential reservoir water ingress areas. There was also no evidence that leakage would be confined to the river. Consequently, the area requiring treatment included the steep reservoir slopes where construction would be very difficult. In addition to enable the construction of the blanket to proceed in dry condition, extensive river diversions would have been necessary. Initial estimates indicated that to cover the entire potential area of reservoir water ingress, with an impermeable blanket some five million cubic metres of material might have been required.

7.0 First Trial Impounding

The first 'Trial Impounding' was on the 2nd June 1991. The reservoir level recorded the maximum level E*l*. 402.40 m on the 14th June.

7.1 A Spring

The striking observation apart from the quick harmonious response of the potentiometric levels in the piezometers was the development of the new spring just 320 m downstream of the dam. The spring was noticed on 11th June 1991. The leakage measured 5 liters per second when the reservoir recorded E*l*. 398 m and the average potentiometric level in the boreholes recorded E*l*. 393 m.

7.2 The Groundwater Table

A hissing noise with air emanating was recorded in the main power tunnel intake area during impounding. A similar phenomenon was also observed in some of the grout holes in the cutoff adit during impounding. The groundwater table in RT-1A a borehole 140 m from the intake gate shaft along the tunnel trace had read E*l*. 380m before construction. It was therefore suspected that the flat groundwater table on the rightbank was as far as the intake.

7.3 Impounding Abandoned

Further, impounding the reservoir was abandoned considering it dangerous with the incomplete grout curtain on the right abutment and the right bank and also with the incomplete concrete lining of the 290 m long access adit E driven in completely to highly weathered rock (see Fig. 4).

8.0 Second Trial Impounding

The second trial impounding commenced on the 5th March 1992 after the right abutment and rightbank cutoff curtains were done to design.

8.1 Deeper Grouting

There was a proposal to grout 4000m of 150m deep holes down to E*l*. 250 m in the fault F1, leakage route II area. However, time constraints, tech[illegible]l considerations and budgetary constraints taken into account prevented the exercise.

8.2 Followup Grouting

Follow up provisions were allocated preparing for the event of seepage losses being found to be unacceptably high. Tentatively 5000m of drilling with 2000 tons cement was roughly set aside for this purpose.

8.3 Tolerable Leakage

Broad guide lines of leakage monitoring was set by the Engineer accounting from downstream requirement. Upto 500 *l*/s of leakage at E*l*. 460 m reservoir top water level was to be acceptable leakage. Between 500 *l*/s to 2000 *l*/s of leakage at E*l*. 460 m was to be the range of marginally acceptable leakage. Beyond 2000 *l*/s at E*l*. 460 m was unacceptable leakage.

8.4 Pore Pressures

Events after the second trial impounding were as dramatic as for the first. All surface piezometers in 'area B' of the rightbank downstream and upstream of the grout curtain, all deep piezometers under the grout curtain and piezometers in the abandoned adit, responded reading reservoir levels within about a day's time lag. Some piezometers in 'area A' in the dam right abutment cutoff were also noted to be responding to the reservoir water level.

Wet patches to leaky patches continued to rise in elevation with the reservoir along weathered limestone bands existing on the rightbank just downstream of the dam.

In May 1992 as the reservoir level moved upto E*l*. 415 m and above, water from borehole MS1 flowed out adding to the knowledge of how pervious the ground was. This borehole is located 130 m from the edge of the dam (or 110m downstream of the access adit D(a) in the dell where fault F1 cuts across. The collar of borehole MS1 is at E*l*. 415 m. The first 20 m ie. to E*l*. 395 m in this borehole is completely to highly weathered.

8.5 Water in the Dam Body

In early June 1992, the piezometer located in cavity No. 4 near the clay core in the dam body at E*l*. 401.8m, 23.31 m downstream of the dam axis, across chainage 449.90m, started to respond with rising reservoir levels. So did the dam seepage measuring chambers volume start to increase before arresting. The maximum piezometer level reached in cavity No. 4 was to E*l*. 412 m and the maximum seepage measured was 13 *l*/s in the dam seepage measuring chamber. Water was suspected to come to the dam body over and across the downstream access adit D(a). Seven B series boreholes were cored from the surface along the trace of the access adit D(a). A 2m cavity was encountered in borehole B6. When B6 was drained into the access adit D(a) it was observed that seepage to the chamber reduced. Later a dye test too proved that the cavity in B6 was directly connected to cavity No. 4 in the dam body (see Fig. 11).

8.6 Leftbank Leakage

About 3Km far away in Killekandura ara on the leftbank downstream of the dam a leakage of about 40 *l*/s occurred when the reservoir was around E*l*. 424

m. The phenomenon was observed in June 1992 for the first time. Farmers of the area had observed a wet soggy patch developing in the paddy field during the dry spell of the year. How water gets into this area is still not well understood.

9.0 Burst

The reservoir level was raised continuously and was on its way to top water level El. 460m irrespective of the pore pressure development of the right abutment.

On the 22.10.92 at approximately 13.00 hours a spectacular water burst occurred which turned a 77 *l*/s spring into a river of muddy water which swelled upto about 10m^3/s. The maximum elevation attained by the reservoir was El. 439.51m.

The burst developed at around El. 400m in the upper reaches of the same gully where the first leak developed in June 1991 which is located on the right bank of the Walawe Ganga 320m downstream of the dam. The reservoir water level was at El. 438.99m when the burst occurred.

9.1 Sudden Reduction

With the burst all piezometers in area B dropped drastically to El. 419-420m from El. 438.43 within 24 hours. The discharge decreased to 5m^3/s by the 28th and 29th of October while the groundwater level went down to El. 415 - 416m. From the 29th October to the 31st October it was observed that when the discharge reduce from 5m^3/s to 3m^3/s that the groundwater level rose by 6 to 7m. This sudden rise in the groundwater table is attributed to a blockage of the leakage path caused by fallen rock or debris reducing the section controlling the flow. Such blockage of the leakage path could happen again increasing the rightbank pore water pressures.

9.2 Extensive Pervious Aquifer

It is interesting to note that groundwater responses of piezometers as far as 2.5 Km apart such as GW1 in fault 1, saddle 1 and GW18 in saddle 4 far from the south end of the grout curtain, closely reads the same groundwater levels with response to the reservoir (see Fig. 7 & 8).

9.3 'Area B' is in 'Area A'

It is also interesting to note that in 'area A', throughout the burst incident, groundwater levels in the immediate right abutment of the dam, 4 upstream piezometers and 2 downstream piezometers in cutoff adit D(b) have been affected. Two piezometers on the trace of the access adit D(a) just downstream of the dam in Area A also have been affected. These observations and other geological inferences confirm the vulnerability of the dam right abutment as the deeply weathered, corroded 'area B' inter fingers into the dam body (see Fig. 4 & 6).

9.4 Accelerated Weathering and Solution

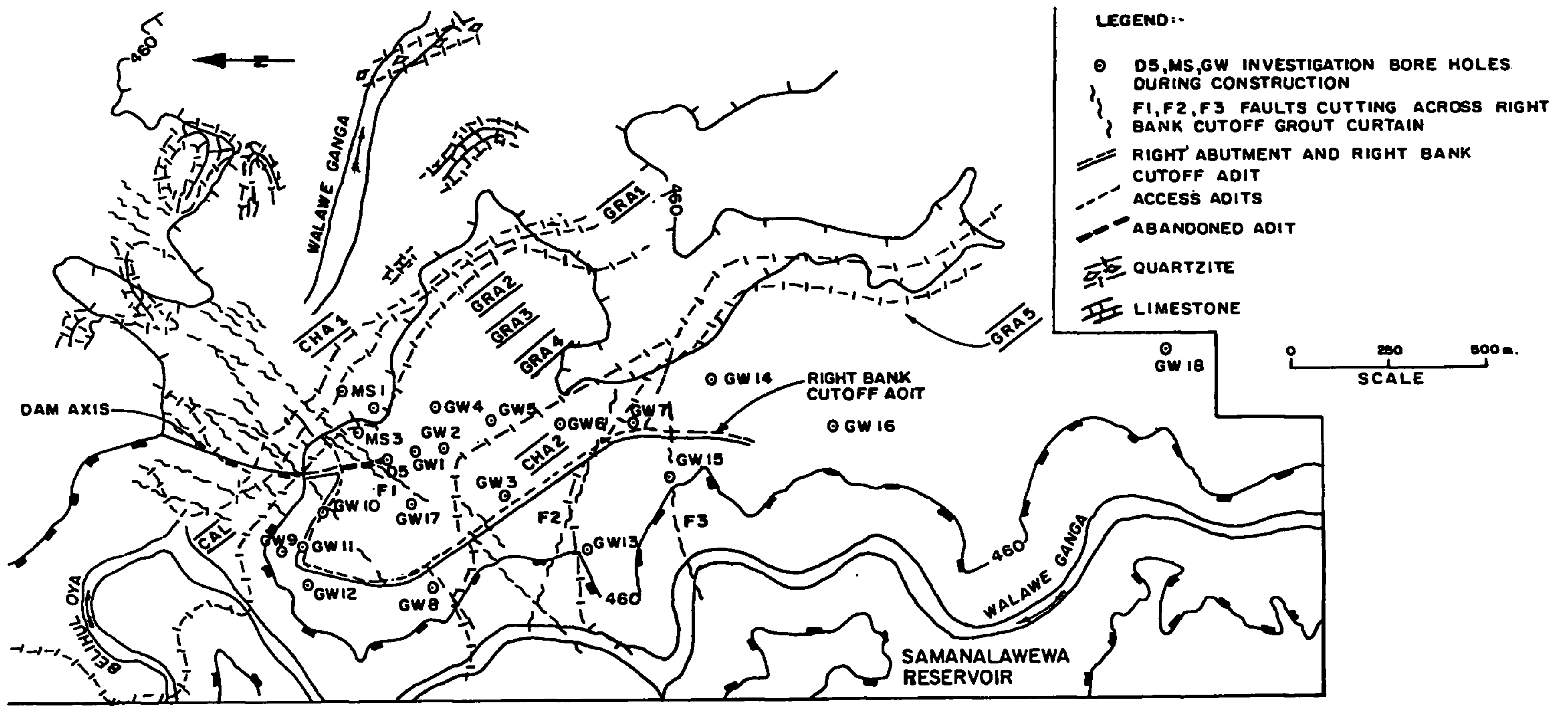
After the water burst in October 1992 the reservoir has been strictly controlled and maintained just over the minimum operating level of El. 424m to generate power. The leak has stayed around 2m^3/s and the difference between reservoir water level and groundwater level has stayed in the order of 10m. The right abutment and the whole right bank ridge of the dam at present stays saturated to El. 415 m approximately. As a result on the rightbank a tremendous amount of solution and accelerated weathering is taking place below El. 415m which is saturated.

10.0 Problem, Right Abutment

The dam is safe for full reservoir storage independent of the right ridge leakage conditions was a conclusion made by the Third Party Review Panel. However, the author wishes to point out that concern is with the stability of the immediate right abutment hill on which the final 20m of the dam abuts. the cavitated, erodible band encountered here on the right abutment is the problem. Most of the drilling and grouting on the right abutment was done to seal off this soft zone. Leakage route I was identified across this band. Piezometers in this zone responded to the reservoir burst confirming 'area B' to exist within 'area A' as well and thereby within the dam itself (see Fig. 6). This band which is situated on top of the limestone bearing rock unit and under the deeply weathered garnetiferous granulitic gneiss rock unit runs towards the major F1 fault zone which is deeply weathered to El. 390m in the dell and already saturated to El. 415m.

10.1 Alternate Proposal for Safe Drainage with Second Line of Defence

Geomorphological, geological and hydrogeological features in the F1 faulted area suggest that, the most vulnerable and the most dangerous area susceptible to failure and erosion is within the immediate rightbank of the dam. The Third Party Review Panel in their first report submitted in December 1992 stated in one of their main conclusions and recommendations that a drainage gallery should be driven along the right bank abutment of the main dam extending 300m to 500m downstream so as to organise and control the natural but inadequate drainage created by the October water burst. It said that this gallery should be equipped with a large number of high capacity drain holes and with a monitoring system consisting mainly of flow gauges and piezometers (see Fig. 9).



ROCK UNITS DIPPING INTO THE RIGHT BANK

CHA 1, CHA 2 - CHARNOCKITIC GNEISS + INTERCALATIONS OF GARNET BIOTITE GNEISS, IMPURE CRYSTALINE LIMESTONE
CAL - CHARNOCKITIC GNEISS + 30% IMPURE CRYSTRLLINE LIMESTONE
GRA 1, GRA 3, GRA 5 - GARNETIFEROUS GRANULITIC GNEISS
GRA 2, GRA 4 - GARNET BIOTITE GNEISS

Figure 7 - Geology of the Dam and Saddles

The likelihood and possibility of blowouts and severe landslide and erosion of the erodible seam which would adversely effect the stability of the dam itself in the extreme right abutment area is foreseen. Considering the serious risk, a foolproof second defence line is proposed for the stability of the right abutment.

11.0 Aquifer Underneath Reservoir

Geomorphological, geological and hydrogeological features of the area indicate that Samanalawewa Reservoir is seated over a very extensive deeply pervious aquifer. Water into the aquifer is not only seeping through the Walawe Ganga but it is also suspected that water is even seeping through the Belihul Oya.

The distinct characters of the quifer is its extensive flat groundwater table and its corroded altered features at depth irrespective of the type of rock. The other features observed were the deposition of a secondary cement and its irregular fracture system with openings in tens of millimetres.

The extensive deeply pervious nature of the aquifer is most of all attributed to the corrosive erosive feature perhaps caused by some thermalfluids emanating from below the earth. Faults joints and cavitated limestone where encountered only adds to the extent of the aquifer bringing it higher than the ubiquitous El. 380m level.

12.0 Wet Blanketing, Reduction of Leakage

Wet blanketing for reduction of leakage is the method, no doubt if the point or points of ingress is known. It is very advantageous because power generation could go on with the sealing operation.

At Samanalawewa a stretch of 1000m (from 700m to 1700m upstream of the Walawe) is said to be identified as the ingress area. Leakage route II is hydrochemically identified to be the main passage bringing water 180m under the grout curtain to the exist, burst area and thereby, the NNE-SSW fault F1 in line cutting across the reservoir in the Walawe is supposed to be the main ingress area. Wet blanketing to the tune of Rs. 2.5 billion spread over a period of 2 years is envisaged. The estimated quantity of material is half a million cubic meters.

Earlier, to blanket this very same area some 5 million cubic metres of material was estimated to be required. It said that there was also no evidence that leakage would be confined to the river, but included the steep reservoir slopes where the sub vertical faults and the limestone bands which cut across the valley. Construction was thought to be difficult on these slopes.

13.0 Alternative Proposal

The alternative proposal primarily addresses the vulnerable immediate rightbank and thereby the right abutment hill of the dam. It does not include the problems of the reservoir watertightness in their totality (see Fig. 10 & 11).

It stresses that there is a need to device measures to reduce the development of pore pressures for stability of the hill which shoulders the dam and to reduce leakage from the reservoir reaching the critical zones and in the process to minimise the total leakage to the extent possible. Internal drainage in multiple tiers are suggested to reduce pore pressures in stages. A grout curtain sub parallel to the flow of the river surrounding the internal drainage system is proposed to reduce the present reservoir leakage.

In the unlikely event of the leakage being still beyond acceptable economic limits pumping back to the reservoir against a head of 100m to enable its utilisation for generation of power with the available head of 325 is also proposed.

Augmentation of the present disproportionately low bottom outlet capacity is suggested to regulate the reservoir when required especially during periods of floods.

14.0 Conclusions

The most vulnerable and the most dangerous area susceptible to failure and erosion exist just downstream on the rightbank of the dam. A failure in this area under the worst case could be catastrophic with the rightbank abutment caught in it. A second defence line in this area is strongly recommended.

Suitable drainage arrangements in the right abutment is proposed with a deep grout curtain surrounding it for leakage reduction and permanent safety reasons not taking any chances.

15.0 Acknowledgement

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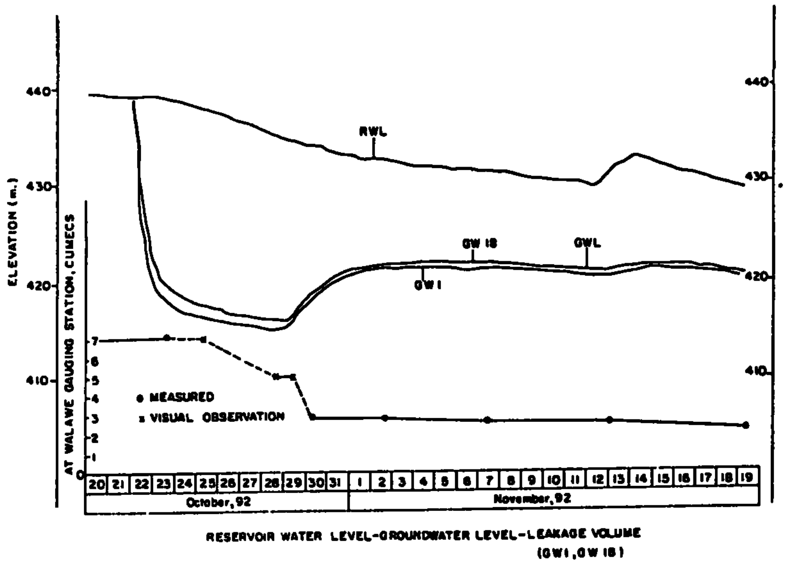

Figure 8 - Reservoir Water Levels Vs. Groundwater Levels after the Burst

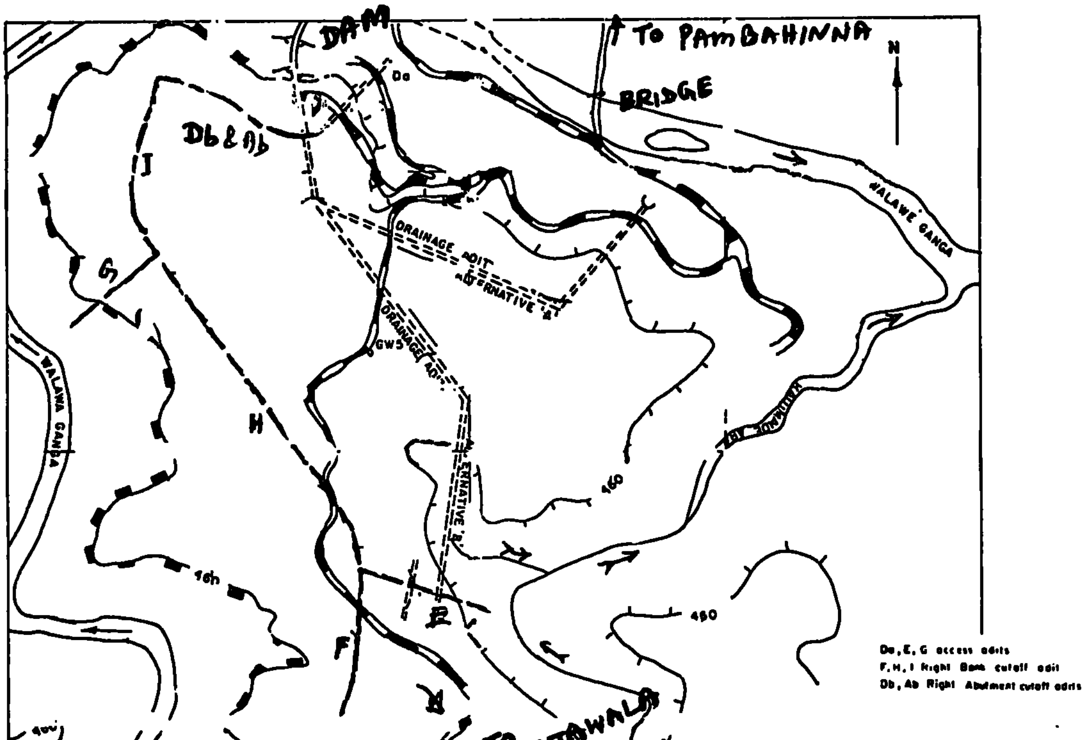

Figure 9 - Drainage Adits Initial Recommendation

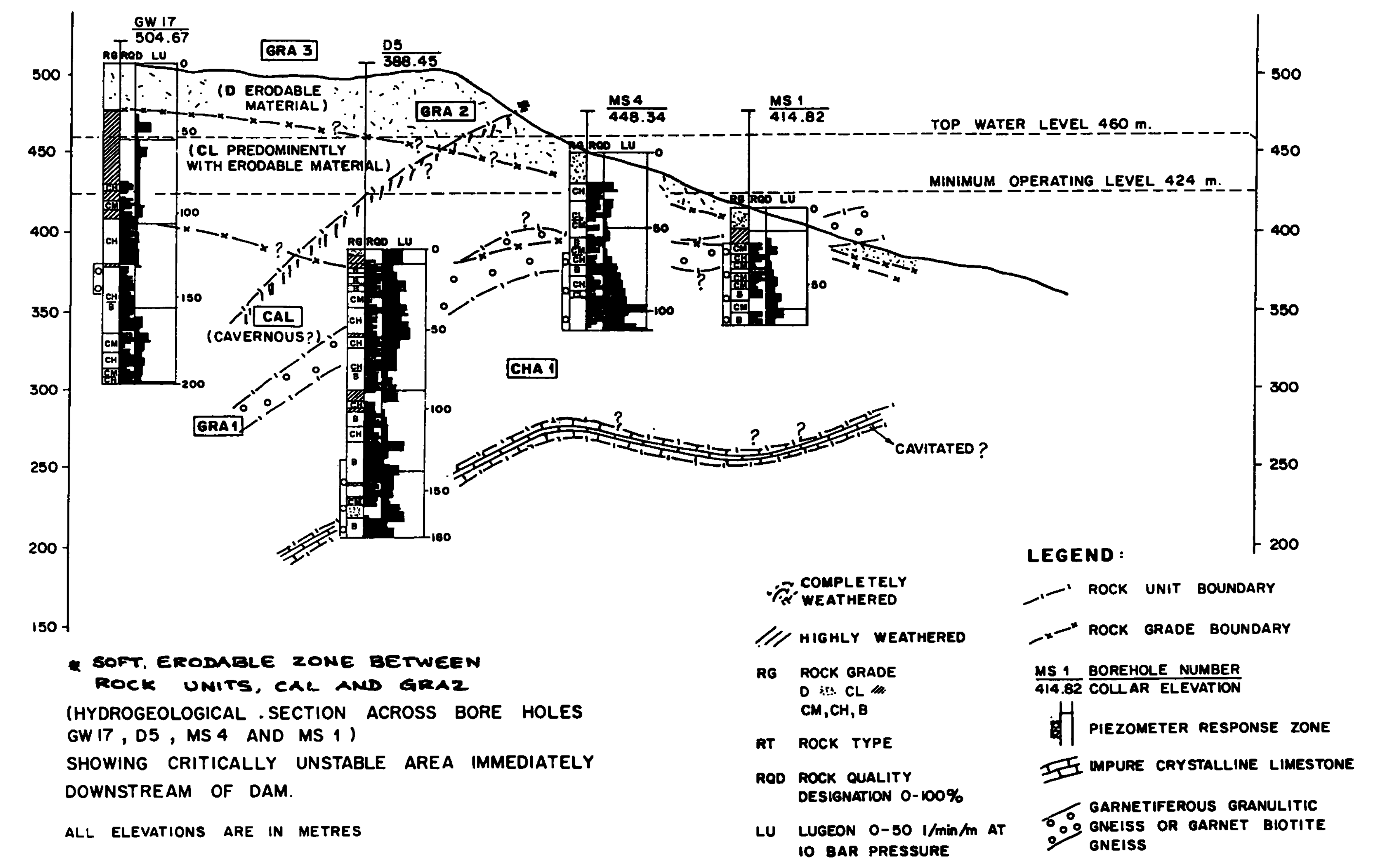

Figure 10 - Hydrogeological Section Down Stream of the Dam

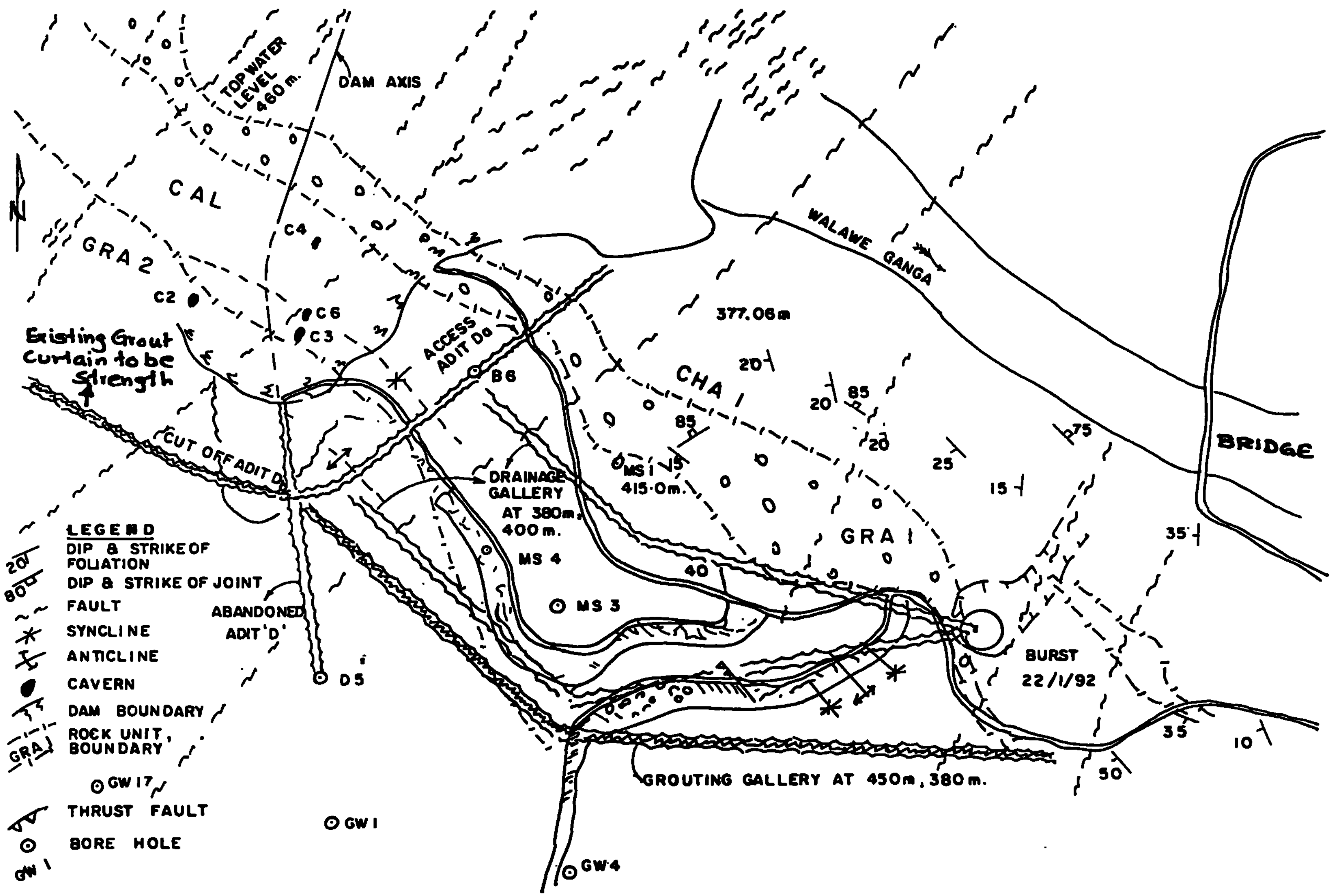

Figure 11 - Conceptual Proposal for Permanent Remedial Works

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Samanalawewa Hydro-Electric Project - Principal Features

1.	**Samanalawewa reservoir**	
	Catchment area	341.7 Km^2
	Annual average runoff	598 * 10^6 m^3
	Flood level	460.7 m
	Normal high water level	460.0 m
	Minimum operating level	424.0
	Live storage above minimum operating level	254 * 10^6 m^3
2.	**Samanalawewa Dam**	
	Type	Rockfill dam with clay core
	Crest length	530 m
	Dam height	100 m
	volume of fill	4.48 * 10^6 m^3
3.	**Spillway**	
	Type	Gated overflow
	Capacity	3600 m^3/s
	Number of gates	3
	Size of gates	12 m wide * 14 m high
	Sill level	446.7 m
4.	**Diversion tunnel/low level outlet**	
	Number of diversion tunnels	2
	Diameter	6.8 m
	Lengths	482.2 m, 501.6 m
	Low level outlet	Jet flow gated, emergency 2m*2m Jet flow gated, irrigation 0.6m dia.
5.	**Power supply tunnel system**	
5.1	**Intake**	
	Type of gate	Fixed wheel type
	Size of gate	4.5m high * 3.5 m wide
5.2	**Low pressure concrete lined tunnel**	
	Diameter	4.5 m
	Length	5423 m
	Intake invert level	416.75 m
5.3	**Surge chamber**	
	Type	Restricted orifice type
	Diameter of riser	6 m
	Diameter of chamber	18 m
	Height of chamber	94 m
5.4	**Penstock**	
	Diameter	3.85 m, 2.85 m
	Length	840 m
5.5	**Valve house**	
	Valve type	Butterfly
	Valve diameter	3.85 m

6. Power Station

Type	Surface
Number of units	2
Type of turbine, elevation	Francis, 109 m
Installed capacity of each unit	60 MW
Speed	320 m
Rated head of turbine	320 m
Rated discharge	42 m^3/s
Generator transformer	2,75 MVA, 3 phase
Average energy	462 GWh
Firm energy	420 GWh

7. Tailrace canal

Type	Open canal
Length	540 m, to katupath Oya