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THE DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA

MINISTRY OF POWER AND ENERGY
CEYLON ELECTRICITY BOARD

SAMANALAWEWA HYDROELECTRIC PROJECT

THIRD PARTY REVIEW PANEL

29 November-3 December, 1992
Colombo

REPORT SMW-1

G. POST

P. LONDE

Paris, February 1992 3

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MAIN CONCLUSIONS AND RECOMMENDATIONS

1. The grout curtain has no effect on the seepage control in the right bank ridge, as shown by the horizontal ground water surface indicated by the numerous piezometers, located both downstream and upstream of the curtain. This is due to the fact that the curtain is not connected to a reasonably watertight formation, particularly in depth. The extension of the grout curtain so as to make it efficient would be very costly, time-consuming and of doubtful result. It is recommended that no additional work be carried out on the curtain.

2. The main dam safety, as it can be assessed from the results of the ample instrumentation installed both in the embankment and in its foundations, is not jeopardized by the high water table which developed in the right bank after the impounding of the reservoir in June 1992. The pressure release resulting from the water burst of 22 October, 1992, has a global beneficial effect, the present conditions being more favourable than those which prevailed last summer. These conditions can be tolerated without risks for some time. However, for the dam safety over the long term it is recommended that the seepage flow in the dam foundations be better controlled, by additional drainage, **and means to render the right bank ridge more watertight.**

3. The stability of the reservoir right bank under present and short-term future conditions is reasonable. However, the sudden opening of new springs, more or less similar to the water burst of October, cannot be ruled out, owing to the high water table existing in the whole ridge. Such springs could punch the overburden materials covering some of the gullies slopes, particularly in the areas of the southern saddles. But it is considered that such possible events would not threaten the safety of the scheme.

4. Remedial measures are required to ensure the long term stability of the scheme and to reduce the amount of leakage from the full operational level of the reservoir to an acceptable value. Our best guess of the full reservoir leakage in the absence of remedial watertight treatment is between 10 and 20 m^3/s. After a detailed review of the conceptual design prepared by the Consultants, we generally support their findings. The most appropriate remedial measures, both technically and economically, consist of additional drainage of the

downstream side of the ridge and placement of an impervious blanket on the upstream side where the water ingress zones are located. Some recommendations on these remedial measures are made in the following sections.

5. A drainage gallery should be driven along the right bank abutment of the main dam, extending 300 m to 500 m downstream, so as to "organize" and control the natural (but inadequate) drainage created by the October water burst. 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.

6. A blanket, made of properly graded fine materials, has to be placed on the right bank slope and on the bottom of the reservoir. Placement in water (wet blanketing) is the preferred procedure, for its advantages regarding the programme of the works (particularly no power shut-down) and for its probable higher efficiency as compared with a blanket placed in the dry (spontaneous penetration into small voids). Special investigations are required to design the most efficient procedures and to determine the extent of the area to be lined. We agree with the investigation programme proposed by the Consultants.

7. It will be particularly valuable to know the flow conditions for the low reservoir levels which should be obtained soon, down to El. 424. In addition it is recommended to test the effect of a new rise, up to El. 430, so as to check whether the underground water paths are stable or otherwise, (accurate seepage measurements at least once a day or for every change in the Reservoir Water level not exceeding 0.50 m).

8. The main and most vital reservoir remedial works to be implemented for the benefit of the project is to reduce the leakage to an acceptable value (normally not more than a few percent of the average annual flow), depending on the flow, if any, which should be released downstream of the dam.

   For the stability of the right bank ridge, a drainage network, in particular in the vicinity of the right dam abutment, is essential to secure adequate safety on a long term basis, as a second line of defense in case of deterioration of the sealing works provided upstream of the ridge.

   Watertightness and drainage measures cannot be considered separately and should be associated in the same remedial works design.

SAMANALAWEWA Hydroelectric Project

THIRD PARTY REVIEW PANEL

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1. INTRODUCTION

Following the decision made by the Cabinet Sub-Committee, Mr. G. Post convened as a member of the Third Party Review Panel to visit the Samanalawewa Site on 29 and 30 November 1992 and to hold discussions in Colombo with the interested parties from 1 to 3 December 1992 (see program of G. Post's visit in annex 4). The terms of Reference of the Panel are attached in annex 1.

After a meeting on December 3, 1992, with Prime Minister Hon. D.B. Wijetunga, some immediate decisions were notified in a short note by N.A.J. Perera, Chairman of CEB, which is appended in annex 3.

The list of participants is given in annex 5 and the different documents handed over to Georges Post are listed in annex 6.

Because Mr. P. Londe was not available to visit the site, it was agreed that G. Post will submit the documents to P. Londe and discuss the problems with him in order to present their common recommendation in this report as agreed by Mr. N.A.J. Perera Chairman of the Ceylon Electricity Board in his letter PDS/C/leak/55 dated 26th November 1992.

Due to schedule difficulties, the Main Conclusions and Recommendations of this report were sent in priority to the chairman of C.E.B. by Mr. G. Post with letter GP/MLB - 12100 dated 21st December 1992.

The author wishes to thank the Ceylan Electricity Board the Joint Venture Samanalawewi, Sir Alexander Gibb and Partners, the Central Consultancy Bureau and all persons present during the site visit and Colombo's discussions for their explanations and comments which greatly facilitated the Panel's work.

2. HISTORICAL BACKGROUND

2.1. Pre-tender studies

The Samanalawewa hydroelectric project has been studied for many years, as early as 1958. Up to 1987 six consulting firms were involved in these studies and investigations. The total number of drill holes carried out during this time was 135 for the dam site (total length 7 360 m) and 16 holes in the right bank ridge (total length 1 230 m).

The right bank is formed by a long ridge with a number of saddles (5) with the fifth one about 2 km south of the dam abutment (see figure 1).

At this time it was considered that additional investigations were required along the right bank ridge in the saddles areas but all parties thought it unlikely that these saddles would provide high leakage greater than a few hundred litres per second (see Tables 1 and 2 Annex 6 reference A.3).

The site belongs to the Sri Lanka Precambrian complex and it consists of an open synform plunging towards the NW. The stratigraphy as shown in the following simplified table consists of metamorphosed and crystalline rocks interfoliated with discontinuous limestone "lenses".

One layer (GRA3) of granulitic gneisses was considered as an impervious boundary in which a grout curtain could be tightened.

2.2. Additional investigation during construction (after August 1988)

The right bank was known to be deeply weathered and affected by an inclined foliation fault which produced underdrainage of the ground water table that was normally around El. 400 to El. 375.

For this reason the end (about 125 m) of this lowest grouting gallery D (El. 388 approx.) in the right bank was abandoned and deviated towards the West (Adit D_b). Also the aerial photographs and false colour infra red spot satellite imagery permitted the visualization of lineaments and faults, difficult to detect on the site because of the vegetation and residual soil overburden cover. These faults trending principally NW.SE together with shear foliation faults are breaching the so-called impervious layer GRA3 (see figures 3 and 4 which are self-explanatory). Additional holes were drilled to investigate this stratigraphy and the permeability, and were equipped with piezometer tips located at different levels (Holes GW1 to GW19, MS1 to MS4 and

D1 to D5, i.e. a total length of 4800 m of drilling with a maximum depth of 230 m). A few dissolution cavities were found during the excavation of the dam foundation (12 several m^3 in volume) to be added to the largest one observed by the Russian team (Technoprom-export) in 1987, 300 m upstream of the dam axis on the right bank (2 to 7 m high, 7 to 19 m wide and 40 m deep). Also small cavities (maybe due to hydrothermal effects) were encountered in the piezometer holes (25 for the GW series of holes totalling about 4000 m of drilling) which are empty or filled with clay, mica or dolomitic products.

Therefore it was decided to drive a grouting lined gallery (1300 m long and 3.50 m in diameter) to investigate (with invert Elevation at 396 for chainage 0 and 390 for chainage 1315) the rock condition and permeability and the Faults F1, F2 and F3 are the main ones (see figure 3). The adit was driven above the underground water table. The fault zones were found between CH700 and 820 for fault F1, between CH330 and 400 for fault 2 and between CH120 and 145 for fault 3. These areas were zones of high grout takes. Fault F1 appeared as one on the major route for leakage, the calcareous component of unit CHA.2 (charnockite with marblelised limestone) being dissolved or porous.

Summary of Site Stratigraphy

UNIT	DESCRIPTION
CHA 2	Predominantly Charnockite with Limestones
GRA 4	Interfoliated Granulitic and Charnockitic gneisses with thin Limestones
GRA 3	Granulitic Gneisses ("Impervious Membrane")
GRA 2	Interfoliated Granulitic and Charnockitic limestone
CAL	Interfoliated Charnockitic Gneisses and Limestone
GRA 1	Granetiferous Granulite (Marker Band)
CHA 1	Predominantly Charnockite with Limestones
GRA 0	Interfoliated Granulitic and Charnockitic Gneisses

It was possible to divide the right bank ridge into two different areas :

Area A where the calcareous component of limestones and charnockite is more or less intact with low permeability with major solution only along the boundary of the calcareous discontinuous unit along the inclined fault and close to the original ground surface,

Area B where the calcareous component of limestones and charnockites has been removed by solution (possibly hydrothermal) replaced partially or fully by secondary dolomitic cement. The joints and foliations are open by tensional effect, (as seen with T.V. Camera) 5 to 20 mm with spacing of 1 to 5 m and small cavities about 0.50 m deep. The permeability is significantly greater.

The boundary between areas A and B is still approximate, although confirmed by grouting results (see figure 3 and 4). The southern limit of area B beyond Chainage 0 of the grouting adit has not yet been defined but the piezometer GW18, 1100 m south of point C (chainage 0) reacts exactly as the other piezometers following the river level fluctuations.

2.3. Ground Water Levels and River Levels

As an example, the contour lines of the ground water level (GWL) as of May 1990 is shown in figure 5.

In area A, the GWL is stable around 400 (except along the inclined fault where underdrainage occurs) and is not influenced by the river's fluctuation.

In area B, the GWL is flat (378.5 to 379.5) and the piezometers react to the river level with a short time lag, not exceeding 24 hours, regardless of the elevation of the piezometer tips at El. 380 or 300, above or below unit GRA3, upstream or downstream. There is no hydraulic lag between the different piezometers. This shows that GRA3 is not impervious and breached at least by F1.

The slightly higher GWT in GW6 may show the proximity of a recharge area : F2 may provide connections to the thalweg of Kalunaide Area downstream of saddle 2 (see figure 1).

It should be noted that the low water table in GW 14,15 and 16 is similar to what was observed by the Russian teams in holes RS4 to RS7.

The high grout take between CH 0 and 260 is associated with the rock layer CHA 2. The calcareous limestone has been dissolved out probably by hydrothermal action and the rock is porous and provides one of the principal leakage routes through the ridge.

2.4. Grouting Results

The grouting programme was very tight in order to start the reservoir impounding not later than March 1992. The grouting curtain which could not act as a positive cut-off stricto-sensus was designed to cross the privileged irregularly developed solution features connected by open tensional discontinuities and limited to a practical depth of 100-120 m where grouting is easiest (GRA3) and most effective. But it was anticipated that the curtain will be "hanging" at least in five sections : CH80-130, CH310-360, CH710-750, and CH900-950, i.e. a total length of about 200 m.

Laterally it has been terminated after the Southern leakage path (zone of Fault F3) but it has not been proved that this extension is sufficient to avoid by-passing the curtain beyond the southern end of adit F.

The conventional split spacing method has been used with cement-bentonite grout generally with the following component : (0.7 W/1.0 C/0.02 B). Sometimes in case of very high take, sodium silicate was added to limit the penetration. The grouting pressure was 2 MPa for all 10 m stages except for the first one behind the adit lining where it was limited to 1 MPa.

A total of 52 000 metres has been grouted with 13 200 tons of dry cement, i.e. an average of 254 kg/m, which is high and much higher than anticipated. One hole H/756/P/D took 15.5 t/l.m. in the stage from 25-35 m depth.

Only very few spots have been grouted above the adit up to El. 424 and rarely to El. 460 (see figure 6), taking into account the geological data collected in the adit (areas with open fractures, fault and shear zones) but it is not sure that ungrouted windows have not been left. Certain sections between CH 1200 and CH 1315 have not been grouted, being considered impervious. Check holes or observation wells equipped with a slotted plastic pipe should be drilled to control this extrapolation. Grouting has reduced the void ratio of the curtain by up to 95 %.

The zones of higher grout takes have been treated with cored check holes and water tested (5 m stages) but only 25 check holes have been carried out for a 1 300 m long curtain (1 555 m of check holes with 311 water tested stages).

In most areas more than 80 % of the stages were terminated with less than 100 kg cement per linear meter, but only 60 % of the stages that were water tested gave less than 10 lugeon units (about 10^{-6} m/s).

The complete closure of the curtain was achieved only when the final hole spacing was 2.00 m (quaternary holes). The wider openings have been sealed but it has not been proved that the smaller fractures (< 2 mm) have been sealed, and if they are closely spaced, they may give high Lugeon values. The curtain length from chainage 0 to chainage 1188 which has been treated with :

sextary	holes	(0.50 m spacing)	represent	2 %
quinary	holes	(1.00 m spacing)	represent	17 %
quaternary	holes	(2.00 m spacing)	represent	34 %
tertiary	holes	(4.00 m spacing)	represent	27 %
secondary	holes	(8,00 m spacing)	represent	25 %

Seven core holes have been cored to a depth of 180 m below the adit (El. 220 approximately), showing that fractured rock and solution paths or cavities still exist at these depths, even below 50 m of intact rock (hole H/568/T/D). From the primary holes it seems that the curtain is "hanging" in at least sections CH 0-40, CH 110-180, CH 430-500, CH 620-870, which is more than anticipated from the investigations (total "hanging" length of 430 m compared to less than 200 m estimated before grouting).

2.5. Trial Impounding (June 91 - March 92)

A trial impounding was carried out from June 1991 to the beginning of 1992 (maximum water level attained 402 with fluctuations around 396). The Ground Water Table was very flat, with generally less than a difference of 1.00 in head between the piezometers and with no development of an hydraulic gradient through the curtain. The ridge is very pervious with an acquifer partially confined.

On 11 June 1991 (with RWL 399.6 and GWL 394.5) a new spring appeared 300 m downstream of the dam toe along the right bank with a small overburden slip. On 11 July 1991, the discharge (M_3 + M_4 weir) was 21 l/s.

The dam toe monitoring weir gave 6 l/s (rainfall effect) with no relation with the reservoir water level. The horizontal hole D3 at El. 389.5 (50 m) from the end of the abandoned southern part of the grouting gallery gave 12 l/s on 10 October 1991 (with GWL at 398 and RWL at 399.50). It was not equipped with a plastic perforated pipe and therefore plugged to avoid regressive erosion (ϕ = 66 mm, velocity 3.50 m/s) of the weathered rock.

From the lack of change in groundwater response (16 deep piezometers) it cannot be concluded that during next reservoir impounding, the leakage will stay below an acceptable value as far as the economical aspect of the project is concerned.

3. RESERVOIR IMPOUNDING AND MONITORING

The reservoir impounding was started in June 1992. Just before, the ground water level was very flat and between 378-380. The reservoir level was raised according to figure 2 to attain the maximum elevation 439.51 on 14 October 1992 (at 24 h) and stay at/or above 434.50 until 16 October 1992 (at 24 h). The spring at M_3 increased until the end of July but stayed constant at 77.5 l/s from the beginning of August until 22 October 1992, though the R.W.L. increased by 8 m (431 to 439). The water of this spring became muddy during the night of 21 October 1992 and all the piezometers followed the R.W.L. very closely, with one day time lag (see figure 2), until 22 October 1992 at 13:00 h, when a sudden burst, blow out, occurred, washing out 25 000 m^3 of an old slide and uncovering an open conduit just below a fault in the limestone, which gave a discharge of about 7 m^3/s. At the same time all piezometers (at least in area B) dropped drastically by almost 25 m down to El 414-415. The discharge decreased to 3 m^3/s, then the G.W.L. rose by 6 or 7 m (some rock or debris fell down in the seepage path and reduced the section controlling the flow) to continue to decrease following the reservoir drawdown and giving a discharge of about 1.8 m^3/s for the G.W.L. at 419 and the RWL at 428.50 approximately (see figure 2). A section along a possible leakage route (close to F1) with the GWL and RWL at different dates is shown on figures 6 bis and 7. It is interesting to see in figure 8 that the ground water response from two piezometers (GW) close to the fault zone F1, on the dam axis and GW18 close to saddle 5, far from the south end of the grout curtain at a distance of 2 450 m, follow each other without time lag and with less than 0.50 m of difference, GW1 = 420.12 on 31.10.1992, GW18 = 420.58 same date (see figure 8 taken from Monitoring Report of October 1992 Annex 6 - document C).

Also the flow from a **left bank spring** (which appeared on 25 September 92) at Killekandura Ara has been influenced by the reservoir raising and has significantly been reduced since the burst of 22 October 1992, even though this spring is at an aerial distance of about 3 km from the dam.

All these facts confirm the great extent and permeability of this semi-confined aquifer.

A tentative oversimplified analysis of this high leakage was carried out assuming no new "burst", for instance above the spoil bank in the Kalunaide Ara from saddles 2 and 3 (see Annex 2). It appears that without any additional treatment of the watertightness of the ridge and depending on the efficiency of the drainage adits and holes, the complete reservoir filling may produce an unacceptable flow of 10 to 20 m^3/s.

Therefore, if the safety of the dam is not endangered by this high leakage, it is true that the economic value of the project will depend completely on the success of the watertightness treatment of the ridge.

Special attention should be paid to the evolution of piezometers in the immediate vicinity of the Dam Right abutment which can be by-passed by the leakage route I bis (see figure 6 bis) using shear joints and tensional fracture joining Fault system F_1. It should be noted that certain of the piezometers (see document C annex fig. J-10 and J-11) placed upstream and downstream of the grout curtain Db have been affected by the sudden "burst" of 22 October 92. These piezometers are not very deep, only 10 m about below the grouting adit invert level, i.e. El. 380 approx.:

- **Upstream piezometer**

SP59 raising to 411.5 then sudden decrease to 408.40 on 23 October 92.

SP61 constant level 426 from 1 July to 31 October.

SP63 raised to 439.50 with RWL then decreased rapidly from 23 October to reach 432.30 on 31 October.

SP65 raised to 438.40 with RWL then decreased rapidly from 23 October to reach 430.40 on 31 October.

SP67 raised to 440 with RWL then decreased rapidly from 23 October to reach 435 on 31 October.

- **Downstream piezometer**

SP60 raised to 406.80 up to 13 Sept. then stayed constant.

SP62 constant level 399.5 from 1 July to 31 October.

SP64 raised to 436.20 with RWL then decreased rapidly from 23 October to reach 430.3 on 31 October.

SP66 raised to 425.80 on 9 August and then stayed constant.

SP68 raised with RWL to reach 427.20 on 20 September and stayed constant until 23 October to decrease to 424.30 on 31 October.

One can see that 4 upstream piezometers and 2 downstream ones have reacted clearly to the sudden drop of the GWL and therefore the limit between area A, non-affected, and area B, affected by solution and tectonic features, is not as well defined as one might assume.

On the other hand, it should be noted (see document C, annex 6, fig. J-9) the piezometers at the dead end of adit D (abandoned on 125 m to be replaced by D_b) immediately followed the drop of the Ground Water Level after the "blow-out" leakage on 22 October:

- SP55 tip around 360 dropped 22 m from 439.50 to 417.40
- SP56 tip around 360 dropped 22 m from 439.60 to 417.60
- SP58 tip around 285 dropped 21 m from 438.20 to 417.20

which confirms they are in area B.

Also the piezometers of the series MS should be analyzed (see document C Annex B fig. B-3 and Table C, 5 of 6 and 6 of 6). The drop of piezometer MSI started on 16 October at El. 433.92 (RWL 439.51) to reach 414.41 on 28 October (RWL at 435.32). It should be noted that this open piezometer was artesian (top 414.82) until 25 October 92. For the piezometer MS2, the drop started on 22 October at 412.11 (RWL 439.01) and reached 405.47 on 29 October (RWL 434.55), with an abrupt drop on 22 October from 426 to 412.

The piezometer MS3 follows very well the lower part of the evolution of the ground water level decreasing from 434.51 on 9 October (RWL 437.43) to 414.60 on 28 October (RWL 435.32) and 420.01 on 31 October (RWL 433.33), but a larger part of the curve (see Fig. B-3) is missing. These data again could prove the proximity of the area B. In October 1992, piezometer levels along the axis of the access adit Da stayed high (B_1 = 438.27 to 433 El., B2 about 443 to 442 El, B3 = 441.5 to 439.83 El.) close to the dam axis and decreased significantly when approaching the portal of adit Da (B4 = 410.74 to 414.09 El., B5 = 413.92 to 404.19). From these data it is possible to assume that the boundary between area A and B passes between MSI and the adit Da then between MS4 and B3 and continues to cross the abandoned adit D in the shear or fault zone close to its dead end. It should be recalled that 8 upward drain holes have been drilled from Da (DRI to DR8, 20 to 35 m long, downstream of B5).

Another fact which should be considered is the piezometric level measured in the solution cavity 4 discovered downstream of the core excavation on the right side of the valley) which shows an evolution of its level, rising from 406 to 413-410 and then decreasing to 403, but all fluctuations were occurring before 22 October with no true response to the RWL fluctuations (maybe affected by the opening of the drain holes in access adit Da).

On the contrary, the weir M-1 measuring the spring below the dell close to the dam toe (100 m downstream) seems to respond to the reservoir level, increasing from 6 l/s on 2 October and then decreasing to 3 l/s on 29 October (6 l/s on 31 October due to rain effect).

To conclude, it can be said that there are many concurring facts which permit one to consider that there is a leakage path around the right abutment connected to the aquifer and closer to the dam than it was assumed. This assumption should be kept in mind in the design of the remedial measures to the high leakage of October 1992, which should take into account the maximum reservoir operating level (460), i.e. more than 20 m above the level attained during the October 1992 impounding.

4. STABILITY OF DAM AS-BUILT AND UNDER PRESENT HYDROGEOLOGICAL CONDITIONS

After a careful study of the dam and reservoir monitoring Report of October 1992, it can be concluded that the behavior of the dam is completely satisfactory.

The foundation piezometers of the dam did not react to the Right Bank GWL and the piezometric levels in the valley, downstream of the core, correspond to approximately 37 % of the reservoir head. The seepages are very low (less than 1 l/s as measured in the seepage collecting chamber at the toe of the dam). The deformations of the dam are normal.

However, it should be noted that a dye test in borehole B-6 with a cavity at 28.4 to 29.5 m in depth (located on the access adit Da carried out on 16 July 92 showed some effect in the measuring chamber, 2 or 3 days later. This hole gave high seepage down to the access adit and drainage holes DR4 to DR6 also contributed to this seepage (5 l/s on 07.07.92 with RWL at 425.3 and 26 l/s on 06.08.92 with RWL at 431.50).

A connection might exist with the cavity 4 downstream of the core shown by the variation of the piezometric head in this cavity (see annex 6 document A-3 figure). Also piezometers MSI and MS3 followed the drop of GWL on 22 October 92.

Therefore there are several indications in favor of a protection of the downstream dam shell by a drainage curtain drilled from the access adit downward and upward. At least a few of these drain holes should be drilled as soon as possible and equipped with plastic slotted pipes to avoid erosion of weathered material, in case of high flood. Some downward drain holes could be drilled also as monitoring instruments in the invert of the access conduit to the dam's longitudinal grouting gallery.

Under these conditions, it is most probable that the behaviour of the dam itself will be satisfactory, for a reservoir impounding, carried out with the necessary observations and monitoring steps, up to the maximum operating level.

With the drainage curtain AD from a gallery at Elevation 400-415 (see figure 9), the dam and its vicinity are fully protected against an unforeseen high water table.

5. STABILITY OF RIGHT BANK UNDER PRESENT AND SHORT TERM CONDITIONS BEFORE REMEDIAL MEASURES ARE COMPLETE

With the present drawdown capacity and starting from the reservoir level at El. 414 (minimum operating level for the power plant), the reservoir level cannot be controlled above El 440 (maximum reservoir level attained in October 92 : 439.50) for a flood in excess of the 100 year flood.

The hydrographs show very different durations for the flood if one considers the Russian (TPE) hydrographs or the Electrowatt (EWI) ones.

Starting from El 414 the levels attained using the low level outlet and the 2 turbines are the following:

Flood period of occurrence	TPE Hydrographs(72 h)	EWI Hydrographs (24 h)
25 years	444.3	431.50
50 years	450.8	435.80
100 years	454.5	439.20

As the Russian hydrographs are probably more realistic (to be checked), it can be seen that even for the 25 year flood the Reservoir Level may exceed Elevation 440.

Above the spillway sill Elevation (446.70) the floods may be controlled provided that the discharge released in the Walawe Ganga River does not exceed 500 or 600 m^3/s above which there will be too many damages in the valley. Therefore it can be considered that during a few years, until the remedial measures are completed, the chances of the reservoir level exceeding 450-451 are quite low and it is very unlikely that high damages occurred downstream or through the ridge. Nevertheless bursting similar to the one which occurred in October 92, could happen in the upper part of the Kalunaid Ara thalweg, above the spoil bank, between levels 400 and 440 through saddle 2, along a leakage conduit in the area of fault 3. This may involve the slide of overburden soils and the washing out of the spoil bank but without any damage for the project, except unfortunately the increase of leakage loss above an acceptable economical value (less than 2 m^3/s). This risk might be detected by a fan of 3 or 4 drain holes with slotted pipes drilled from access adit E at Elevation 400 approximately.

Also it should be checked that besides the ridge, there is no other possible important leakage path, because it seems that the watertightness study of the reservoir itself is limited in the documents listed in annex 6 to the following affirmation (document A.3. Part 1 § 1.5. page 1-4):

"Away from the ridge saddles no other watertightness problems were anticipated because of the regional syncline which forms the reservoir margins elsewhere". Is this fact sufficient to conclude so firmly? For instance, the headrace tunnel to the power station has encountered several shear features or faults belonging to the F1 family and water bearing (more than 100 l/s in certain areas). The water drainage through the seepages occurring during the headrace tunnel excavation dried out the ground in this area.

As the southern escarpment (toe elevation at about 160) is only at 5 000 m (average hydraulic gradient about 6%) from the intake area (southern end of the reservoir) one could imagine privileged leakage paths (tensional fractures and solution channels through impure discontinuous limestone that is more or less karstified). Such an assumption has probably been ruled out for good reasons but it should be confirmed and documented.

6. PROPOSED REMEDIAL MEASURES FOLLOWING THE RESERVOIR SEEPAGE INCIDENT OF OCTOBER 1992

6.1. Main problems

6.1.1. The incident of October 1992 has confirmed what was already prognosticated by the trial impounding, i.e. "excessive leakage might happen in the course of raising the reservoir water level up to the maximum Reservoir Water Level".

In fact, the seepage occurred for the RWL at 439 and it may be assume that for RWL at 460, this seepage may greatly exceed 10 m^3/s, (see Annex 2), the more that new leakage path may develop, especially in Kalunaid Ara thalweg (where springs S4 and S1 increased in October 1992 from 83 l/s to 98 l/s on 22 October 1992), for higher head.

It is obvious that with an average inter-annual flow of the Walawe Ganga of 19 m^3/s, the economy of the project cannot support a loss of water, and therfore of energy, above a few percent (maximum acceptable under 460 R.W.L. should not exceed 2 m^3/s, to be confirmed).

Therefore the main problem is by all means to reduce the seepage.

6.1.2. The second problem which is closely connected to the first one is to secure the safety of the dam by eliminating the risk of leakage by-passing the right abutment for high R.W.L. and producing too much seepage below the downstream shell of the dam which could not be passed through the Dam Seepage Measuring Chamber at the toe of the dam.

6.2. Drainage means

6.2.1. The current natural drainage provided by the development of the October leakage has reduced the Ground Water Level pressure very efficiently and one should take advantage of it to improve its stability at its exit by upgrading the outlet portal by excavation and stabilization of the slope above and around it.

6.2.2. In addition to this main drainage system it is recommended to build a second drainage system as a second line of defense in case of a blockage of the natural leakage conduits.

Two alternatives have been considered (see figure 9):

- the first one AD is located just around the dam abutment with its invert starting from El. 415 approx. at point D to reach El. 400 at the portal A. At the same time the dead end (on about 100 m long) of the abandoned grouting gallery D could be used as a drainage adit. The same applies to the access adit Da to protect the immediate proximity of the dam,

- the second one BD extends from the end of adit D at a higher Elevation (about 415) to the access adit E at El. 400 approx. This alternative is longer than the first one and it does not seem necessary to protect saddles 2 and 3, if the seepages are substantially reduced by the remedial measures, because the seepages in the Kulaneida ara are far enough from the dam toe.

6.2.3. All the drain holes should be equipped with special PVC slotted pipe (see annex 7) to avoid erosion of the badly weathered rock (3" pipes with 0.5 to 1.0 mm slots). If the finest materials are not stopped by these slots, a second pipe with thinner slots (0.2 to 0.3 mm see figure 10) may be placed inside the first one. This has been applied with success recently (Vieux Pré Dam in France).

It is important to recall that the drain holes should be drilled through a blow-out preventer (stuffing-box) if there is significant back pressure around the gallery, which should not be the case in the dry season with a drainage gallery invert set between El. 415 and 400. As the aquifer is very pervious, these drainage measures will produce excessive seepages if at the same time the watertightness of the ridge is not significantly reduced, a difficult operation which is a "must" for the economy of the project.

6.3. Sealing of the Right Bank Ridge

There are three possible alternatives to improve the watertightness of the ridge :

6.3.1. The first one is to try to improve the Right Bank grout curtain. However, the impounding up to level 439.5 has shown that at the present time it will be very difficult, lengthy and expensive to improve the curtain which is inefficient because it is "hanging" over a long length (500 m or more), and the depth and lateral extent of this cut-off curtain has not yet been determined with sufficient accuracy. But, in any case the curtain should most probably be deepened "from 100 m to or beyond a depth of 180 m". Additional adits at El. 310-315 would be preferable to decrease the length of holes to be drilled but it will have to be excavated in poor saturated rocks under at least 100 m head, using shafts as accesses.

Therefore this alternative is not very attractive and it will require extensive additional investigations to fix the boundaries of the curtain.

However, it should be noted that at Khao Laem Dam (see Right Bank Deep Grout Curtain : Cut-Off Treatment Method in Karstic Limestone at Khao Laem Dam (Thailand) by S.Watakeekul and A.J. Coles. R2Q58 15th ICOLD vol. 1 Lausanne 1985). The Right Bank grout curtain, which is 250 m deep and 200 000 m^2 in area, has been drilled from only one gallery 3 m diameter and 800 m long, using down hole pneumatic hammers down to 150 m then rotary drilling with tricone roller bits for the lower 100 m.

But one can conclude, with J.V.C. and the Designer, that in terms of time and cost, this grout curtain alternative appears to be unreasonable.

6.3.2. The second method is to provide an upstream impervious blanket, upstream of the ridge, in the Walawe Gaga bottom and probably up to a certain level on the valley sides. For that it is necessary to locate the preferred section of river water ingress because from saddle 5 to the saddle 1, the length of the river is about 2.5 km and it is hoped that only certain areas will have to be treated.

Two methods of placement may be used.

6.3.2.1. The wet method by dumping underwater a well graded clayey sandy gravel from special barges, as was done to repair the Tarbela dam impervious blanket (see Foundation Design - Tarbela Dam by John Lowe III - Fourth Nabor Carillo Lecture. Mexican Soc. of Soil Mechanics 1978). The blanketing on the banks will have to be placed in the dry, after the required trimming of the slopes has been done, and should be protected by rip-rap placed on a transition layer to stabilize the blanket during rapid drawdown of the reservoir.

The wet placement may have the advantage of showing muddy water downstream as a kind of crude dye test. This method cannot work efficiently if there are large sink holes, or "swallow" holes too big to permit a filtering and sealing effect of the material. In this case these holes should be plugged before, filling them with sandy gravel or transition material or tremy concrete, if they have been detected by divers or by lowering the RWL to show up eddies or vertices at the entrance of these holes.

One of the main advantages of this method is the possibility to avoid drawing down the reservoir completely and diverting the river. It also permits the production of energy if the wet method is used from the reservoir level above El. 424.

6.3.2.2. The dry placement requires the drawdown of the reservoir and diversion of the river which in any case will require the re-opening of diversion tunnel No. 2 (see drwg 82870 RII - 18, 19, 20 and 21 document A-4 Annex 6) which is a difficult operation: demolition of 3.50 m of concrete infill placed in the slots over 8 concrete (1.50 m wide) bulkhead beams (each 25 t in weight), demolition of the central mass concrete plug (760 m^3) and of the downstream one (about 100 m^3).

Blasting should be limited in order to limit the velocity particles to 20 or 30 mm/s which should be controlled and it might be better to use concrete breakers and/or Cardox carbon dioxide tubes emitting huge heaving gaseous forces or Demex demolition swelling agents which can be used under water for vibration-free cracking of rock or concrete.

In any case the capacity of diversion tunnel 2, once reopened, is limited to 300 m^3/s under R.W.L. 380 and 600 m^3/s under R.W.L. 403. (for the 25 year flood peak equal to 900 m^3/s - see document A-1 annex 6), and 800 m^3/s under RWL at 430 for the 100 year flood of 2100 m^3/s.

Also the closure dome (ϕ 3.20 m) upstream of the Low Level Outlet (on Diversion Tunnel 1) should be re-opened after re-erection by divers of the hoisting gantry on the platform at 382 El.

Using the opened closure dome, then the low level outlet may discharge 30-35 m^3/s under the RWL at El. 395 and 18-20 m^3/s at El. 380. This shows that the protection of the dry blanket work will be delicate and the work carried out by successive stages risks to be inundated depending on the time required for the construction of this blanket, i.e. depending upon its length and height on the flanks which cannot be fixed right now.

7. ADDITIONAL INVESTIGATION REQUIRED FOR RESERVOIR REMEDIATION

7.1. Location of Water ingress preferred Paths

The location of the largest water ingress entrance holes is one of the most important points. Unfortunately it is always a very difficult job when using divers after dyes (rhodamine WT, Amino G acid, Salt) test and/or radioisotopes (Sodium Iodure) tests, (see "Recherche et localisation de fuites sur retenues naturelles ou artificielles par techniques de traceurs", J. Molinari C.E.A. La Houille Blanche n° 3/4 1976).

It is recommended to draw down the reservoir to the lowest level (400 to 415) giving still significant seepage in order to detect possible eddies or vertices if the water ingress is concentrated in some areas upstream of the ridge (faults). However, the direct identification by visual inspection of reservoir water ingress is "likely to prove problematic" as written by the Designers and therefore the blanket construction may have to be built in several stages to plug any sink hole which may develop after new impounding until equilibrium is reached, as was the case at Tarbela Dam (Pakistan).

Geophysical investigations by seismic reflection across the reservoir areas to try to locate the zone of probable water ingress as recommended by the Consulting Engineers should also be tried with the help of a specialized firm well experienced in this type of problem. After these geophysical investigations are carried out and taking into consideration their results, a few holes should be drilled on the upstream side of the ridge to carry out dye/salt or mud tests.

7.2. Core Borings

The additional core borings as recommended (see fig. 9) are approved to try :

- to assess the stability of the downstream slopes and to provide piezometers on the downstream side of the ridge,
- to determine in a more accurate way the boundary between area A and area B, especially on the downstream side of the right dam abutment,
- to get more geological data to choose the best layout of the additional drainage adits.

7.3. Test drainage holes

The dead section of adit D (about 125 m) is proposed to be used as a secondary drainage adit. Some drainage holes should be drilled and equipped with slotted pipe (ϕ 3") to measure their discharge in relation with the G.W.L.

Also one or two downward drainage holes could be drilled in the access adit Da and in the access conduit to the dam longitudinal grouting conduit. In any case, the access adit Da already equipped with a few upward drains should be used as a secondary drainage adit with some downward drainholes.

7.4. Blanket Materiel

Investigations for a borrow pit to provide the material for building the blanket have to be carried out soon, in order to try some "wet blanketing tests" by dumping adequate material in the most promising section of the river bed around El. 380, according to the groundwater responses (between coordinates 163 250 N and 163 750 N).

8. NEXT IMPOUNDING AND MONITORING

8.1. Impounding

To the extent possible, the next impounding should be limited to 440, once the natural drainage system produced during the leakage of 22 October 1992 has been enhanced by stabilizing the existing portal. It should be noted that an auxiliary outlet could be built below the spillway (ϕ = 4.50 m to 5.00 m) with its exit at El. 412 in the axis of the spillway chute and its upstream portal at El. 424, just in front of the spillway weir (length of the tunnel about 100 m fed by a canal with its invert sloping from 425 to 424). Such work should be done with special precaution to avoid excessive vibration. Its discharge should be 200 to 250 m^3/s under Reservoir Elevation 446.7 (spillway sill) in order to provide better control of the Reservoir Level (except with the occurrence of high floods).

8.2. Monitoring

Of course all the recommendations for reservoir and dam monitoring already specified for the first impounding are still valid with the addition of some new piezometers, installed in the additional investigation holes, and new drainage holes. A few deep piezometers might be useful south of GW18 on the Right Bank and one on the Left Bank close to the new spring.

8.3. Optimization

The optimization of the reservoir operation taking into account the residual water loss in relation to the RWL, once the reservoir remediation has been carried out, should be done to maximize the projects benefits.

CEYLON ELECTRICITY BOARD
SAMANALAWEWA HYDROELECTRIC PROJECT

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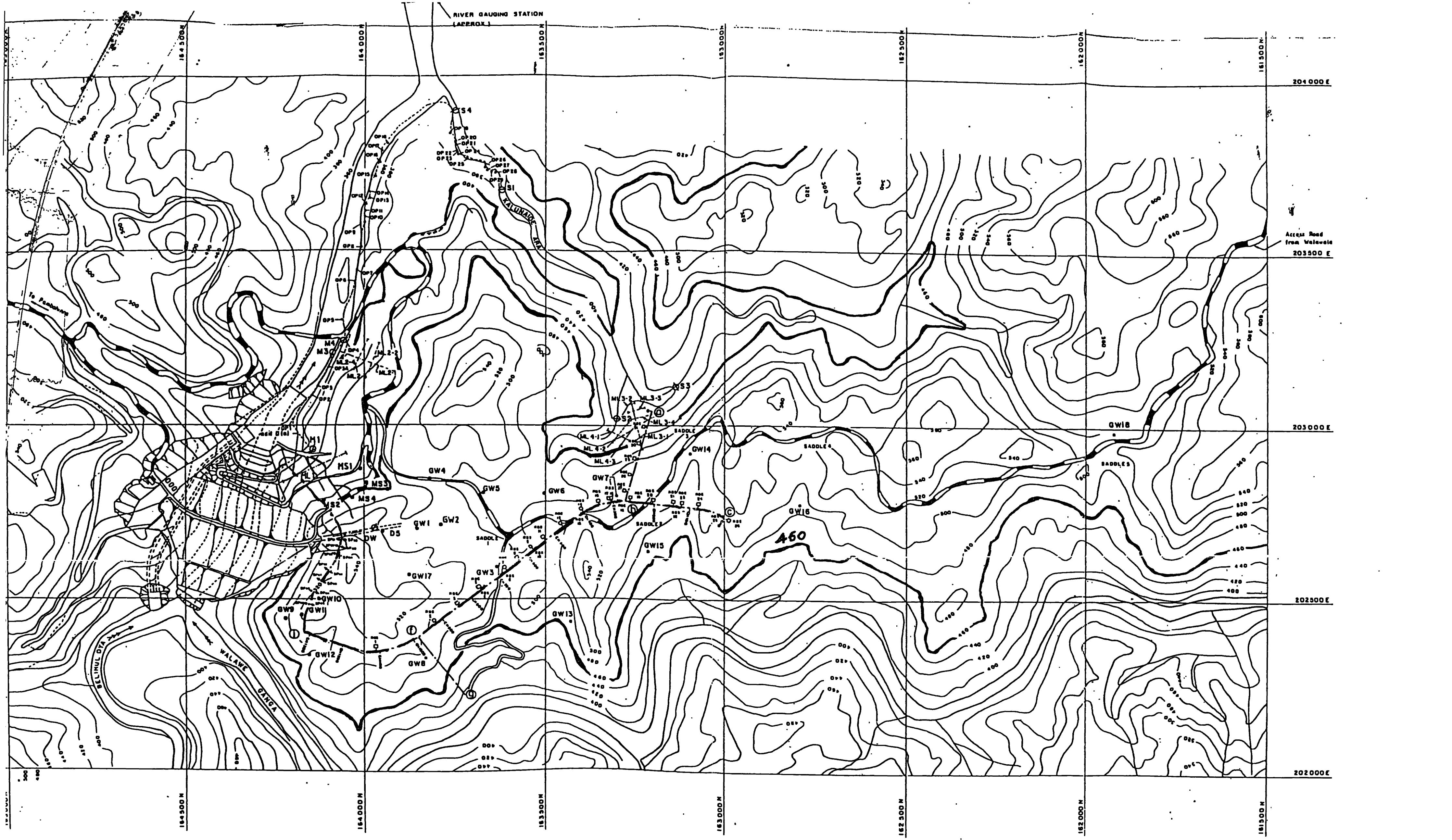
February

F I G U R E S



LEAKAGE MONITORING PATH

SLOPE STABILITY BASE LINES

• GW1 Groundwater Monitoring

RIGHT BANK
RIDGE
1/10000

කෞතුකාගාරය
12 AUG 2003
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CEYLON ELECTRICITY BOARD
SAMANALAWEWA HYDRO-ELECTRIC PROJECT

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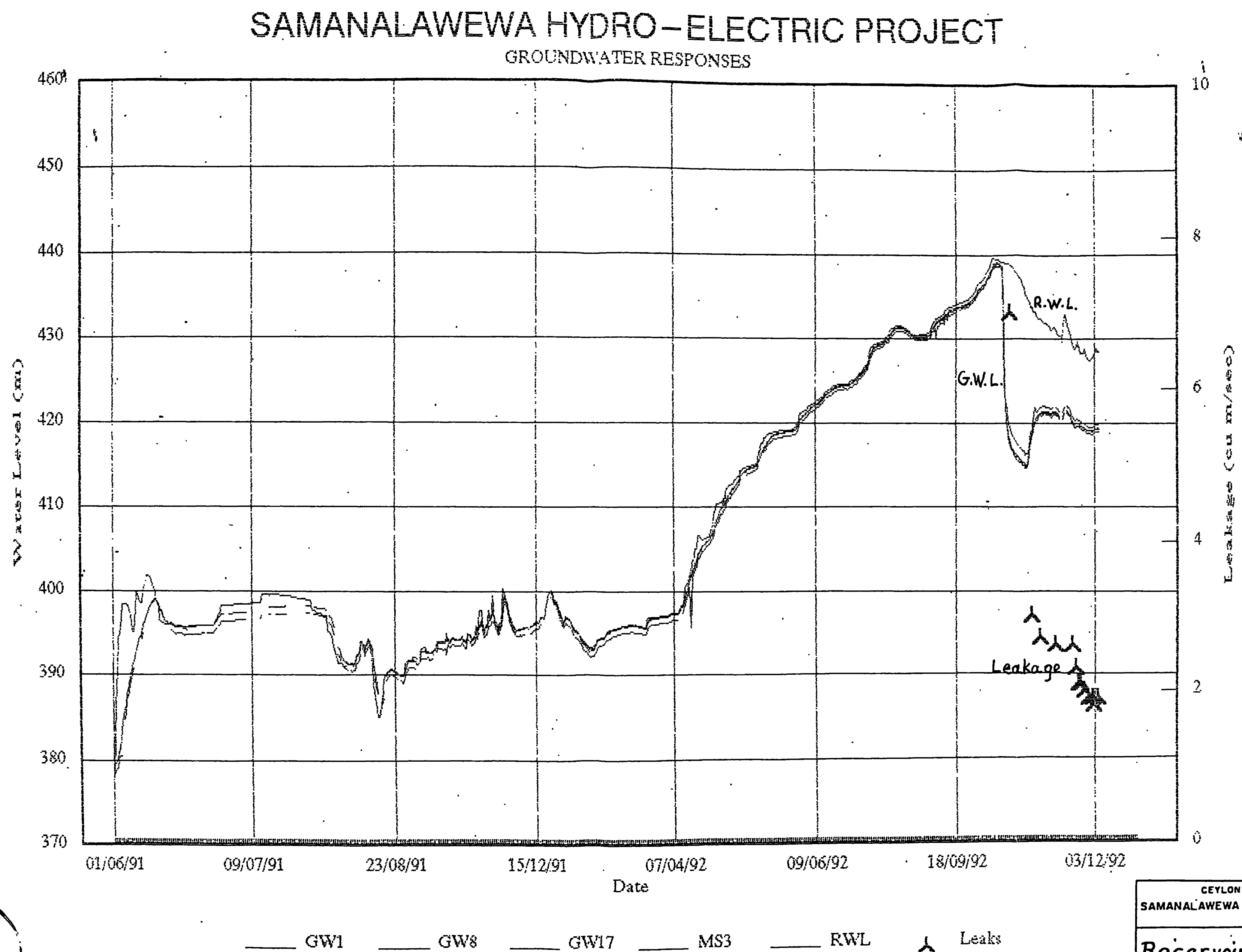
30 December 92	Figure 1



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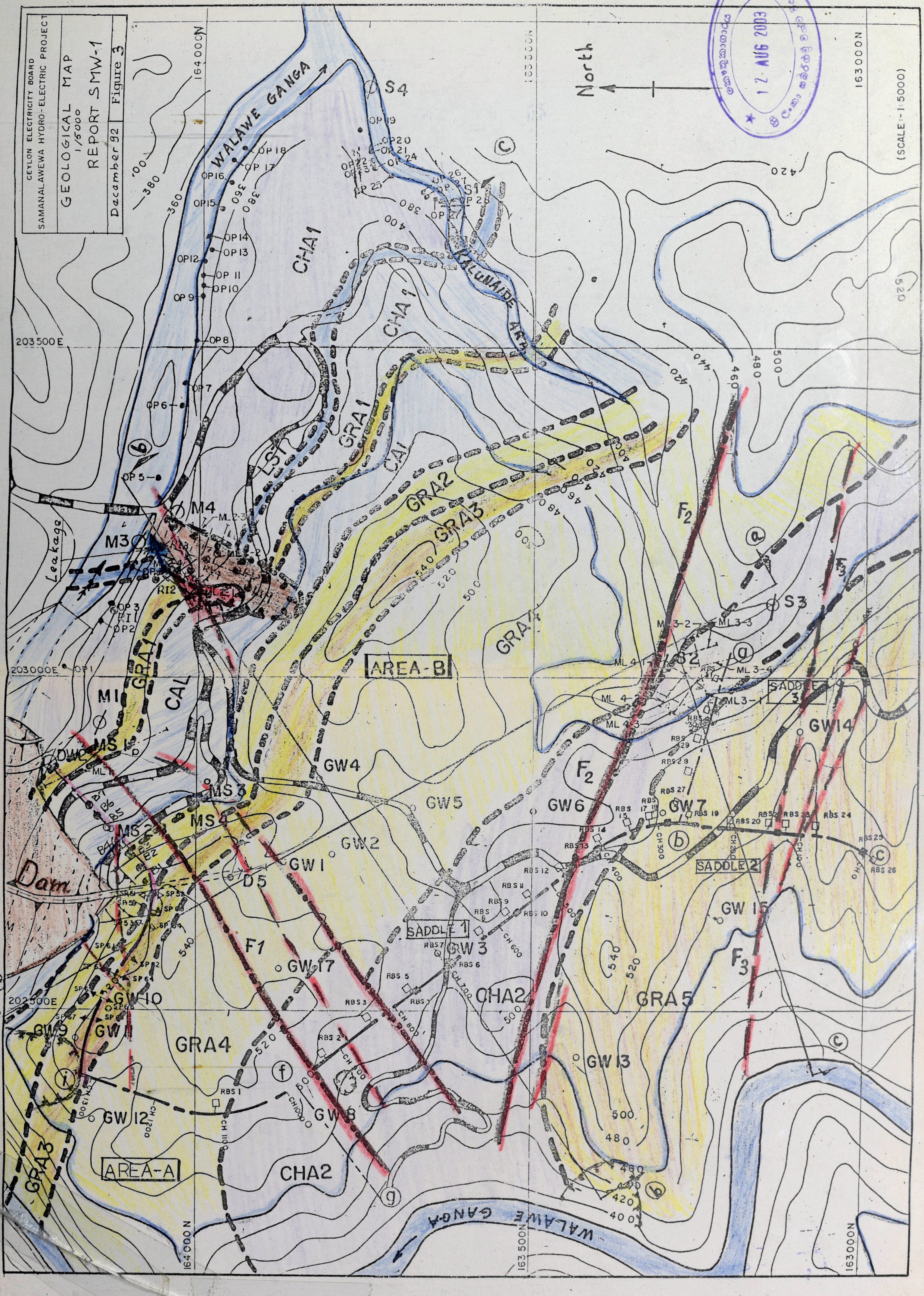
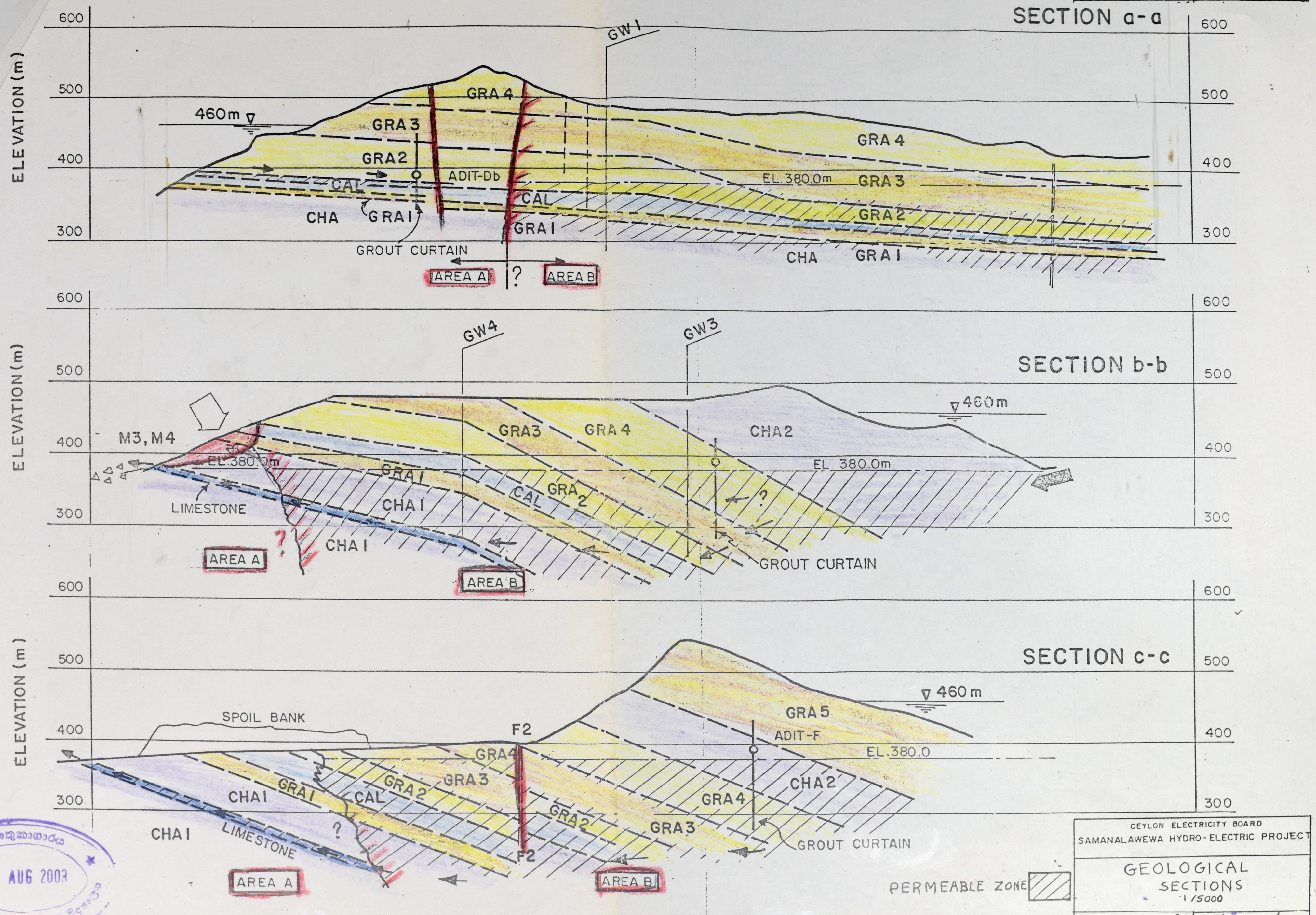
Reservoir Impounding
LEAKAGES and G.W.L.

Figure 2


CEYLON ELECTRICITY BOARD
SAMANALAWEWA HYDRO-ELECTRIC PROJECT
GEOLOGICAL MAP
1/5000
REPORT SMW-1
December 92
Figure 3
North
(SCALE:- 1:5000)
12 AUG 2003
WALAWE GANGA
KALUNAIDE ARA
AREA-A
AREA-B
SADDLE 1
SADDLE 2
SADDLE 3
Dam
Leakage
CHA1
CHA2
GRA1
GRA2
GRA3
GRA4
GRA5
CAL
LST
F1
F2
F3
GW1
GW2
GW3
GW4
GW5
GW6
GW7
GW8
GW9
GW10
GW11
GW12
GW13
GW14
GW15
GW17
S1
S3
S4
M1
M3
M4
MS1
MS2
MS3
D5
164000N
163500N
163000N
203500E
203000E
202500E

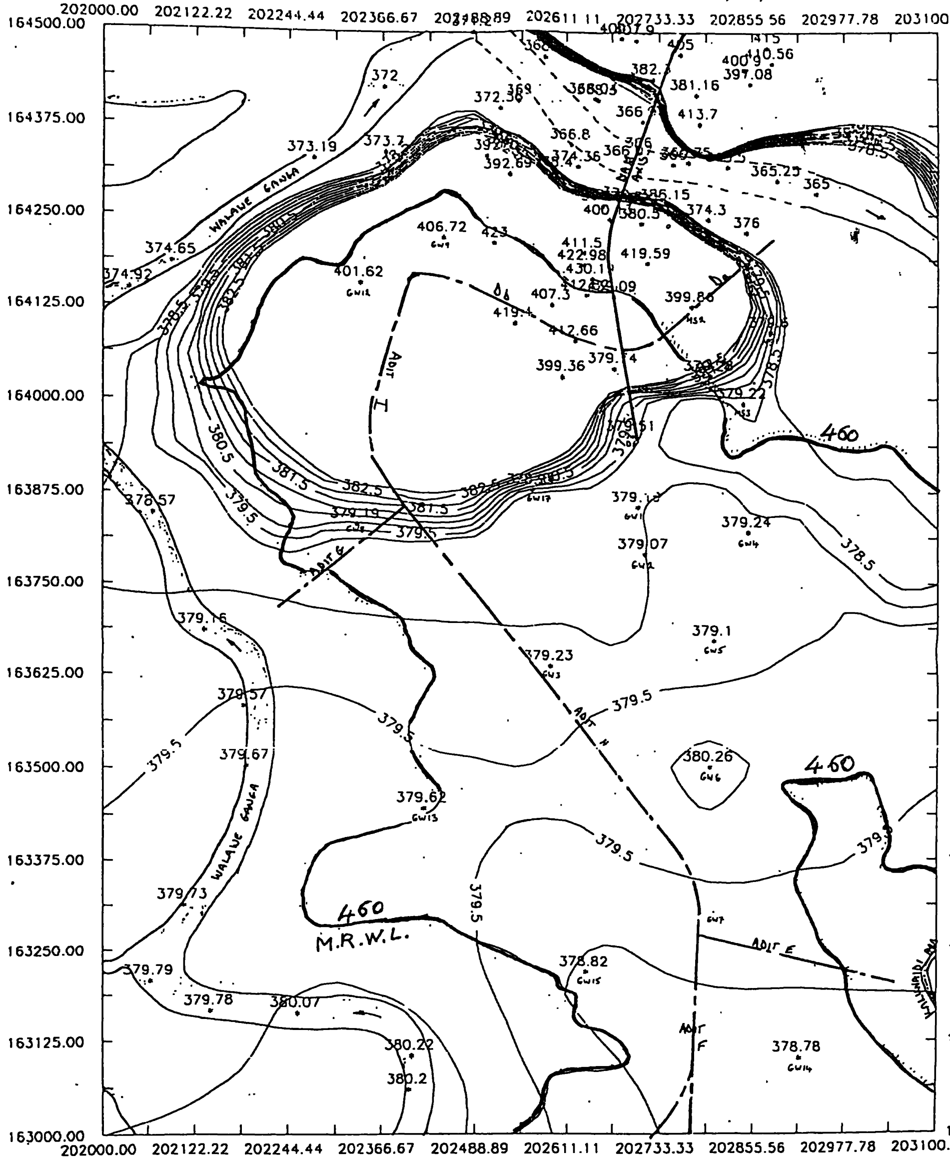
GEOLOGICAL SECTIONS

REPORT SMW.1



CEYLON ELECTRICITY BOARD SAMANALAWEWA HYDRO-ELECTRIC PROJECT
GEOLOGICAL SECTIONS 1/5000
December 92 — Figure 4

SAMANALAWEWA DAM: GROUNDWATER 29/5/90



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SAMANALAWEWA HYDRO-ELECTRIC PROJECT

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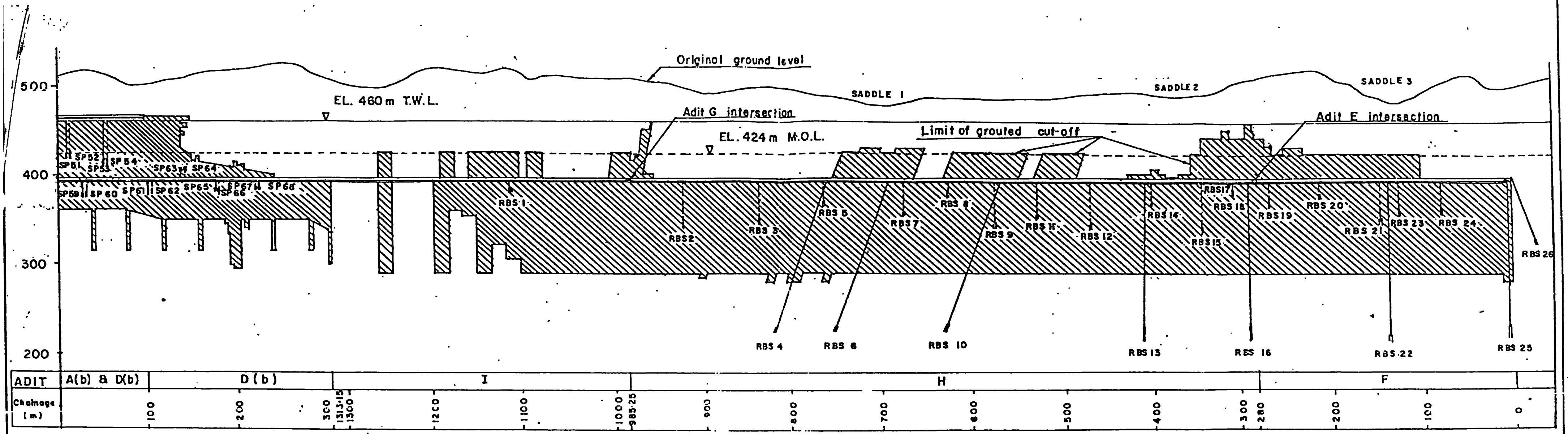
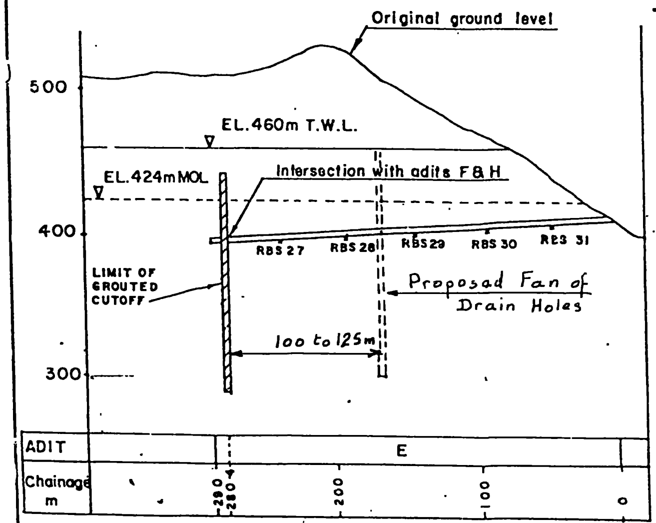
30 December 92 | Figure 5

Right Bank Cut-Off: Groundwater Contour Plan
May 1990

FIGURE 6



LONGITUDINAL SECTION ALONG RIGHT BANK GROUTING ADITS



LONGITUDINAL SECTION ALONG ACCESS ADIT E

KEY :–

STANDPIPE PIEZOMETERS UPSTREAM OF WITHIN THE GROUTED CUT OFF ARE SHOWN THUS

STANDPIPE PIEZOMETERS DOWNSTREAM OF THE GROUTED CUT OFF ARE SHOWN THUS

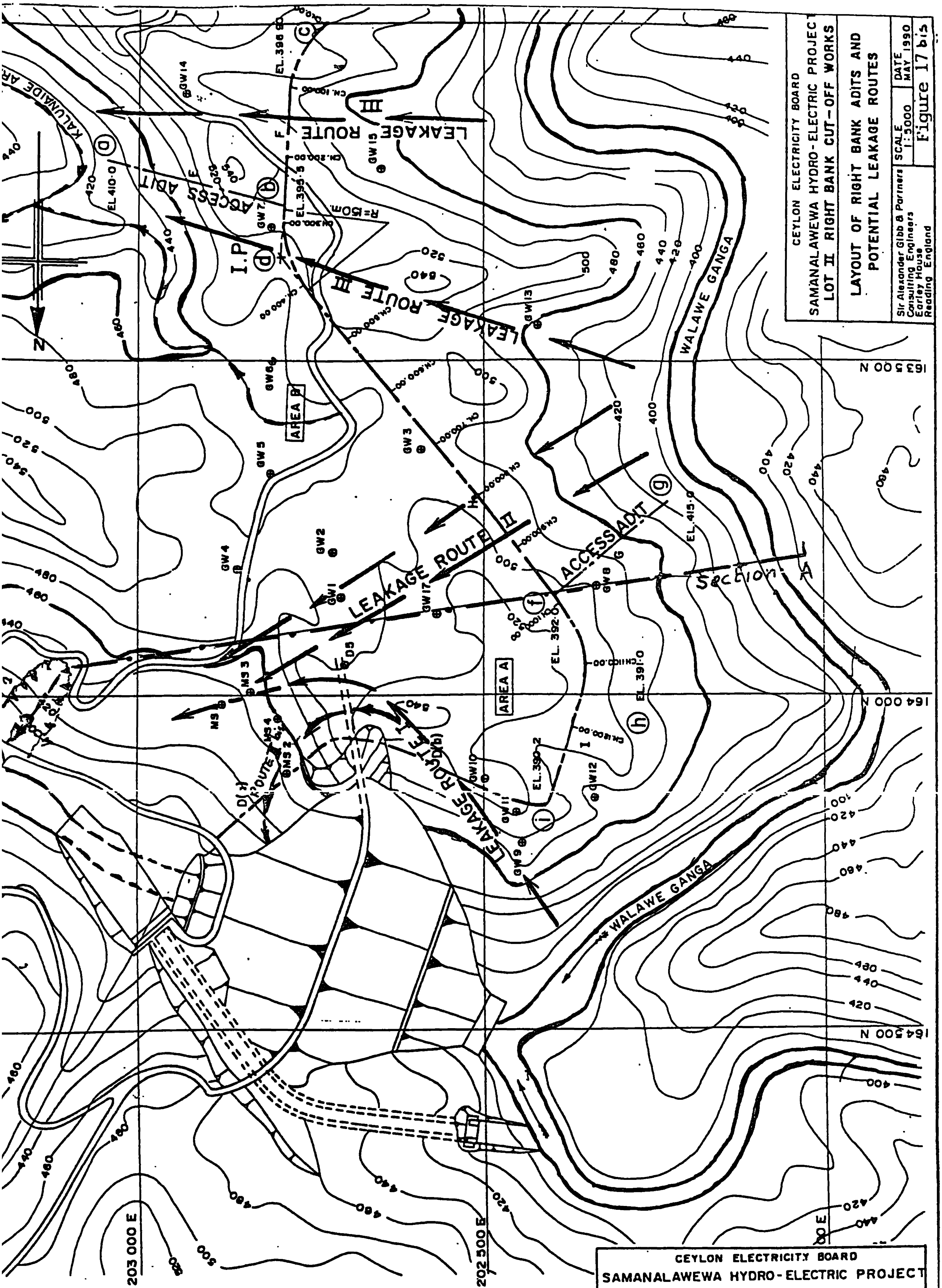
THE AREA OF THE GROUTED CUT OFF IS SHOWN THUS .

17 AUG 2003

SAMANALAWEWA PROJECT

GROUT CURTAIN
LOCATION OF STANDPIPE PIEZOMETERS IN RIGHT BANK ADITS

December 92 | Report SMW.1



POTENTIAL LEAKAGE PATHS

SECTION AA FIG7

(THROUGH 22 OCTOBER 1992 LEAKAGE)

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SAMANALAWEWA HYDRO-ELECTRIC PROJECT

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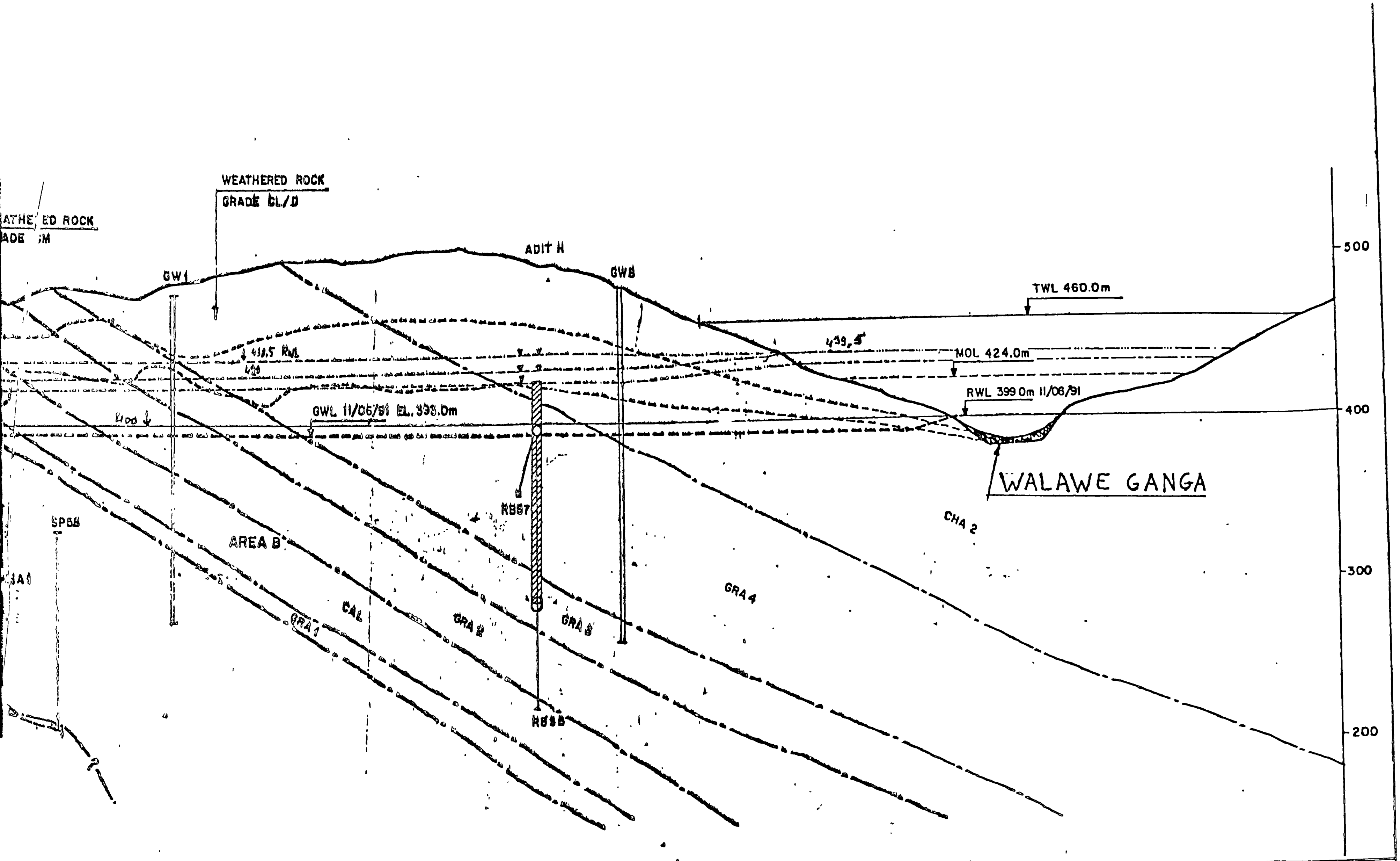
30 December 92 | Figure 6 bis



SAMANALAWEWA PROJECT	
E-W RIGHT BANK SECTION ALONG POSSIBLE LEAKAGE ROUTE	
December 92	Report SMW-1
G. Post - P. Londe	FIGURE 7

CEYLON ELECTRICITY BOARD
SAMANALAWEWA HYDRO-ELECTRIC PROJECT

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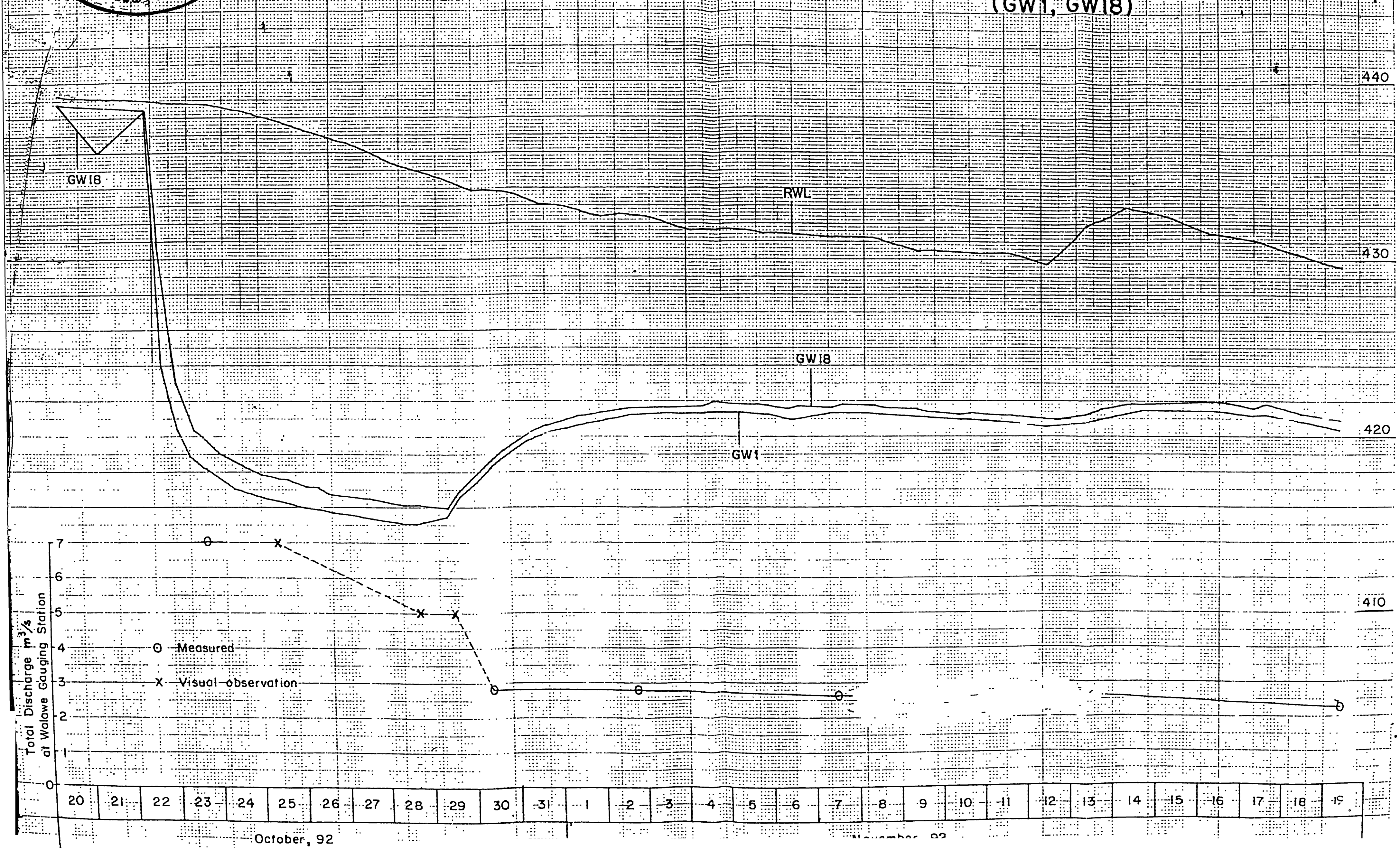
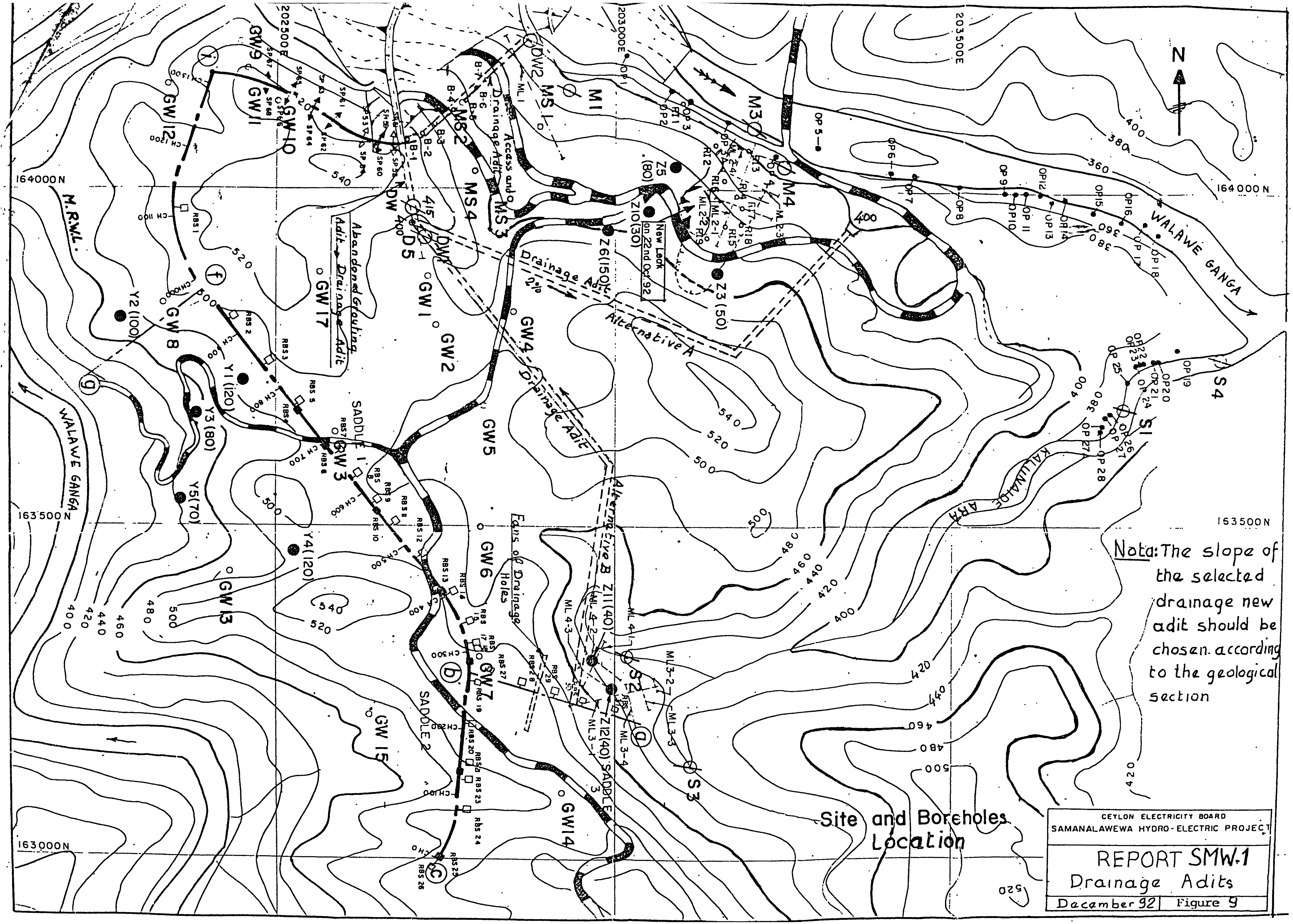
30 December 92 | Figure 8

Reservoir Water Level - Ground Water Level - Leakage Volume (GW1, GW18)





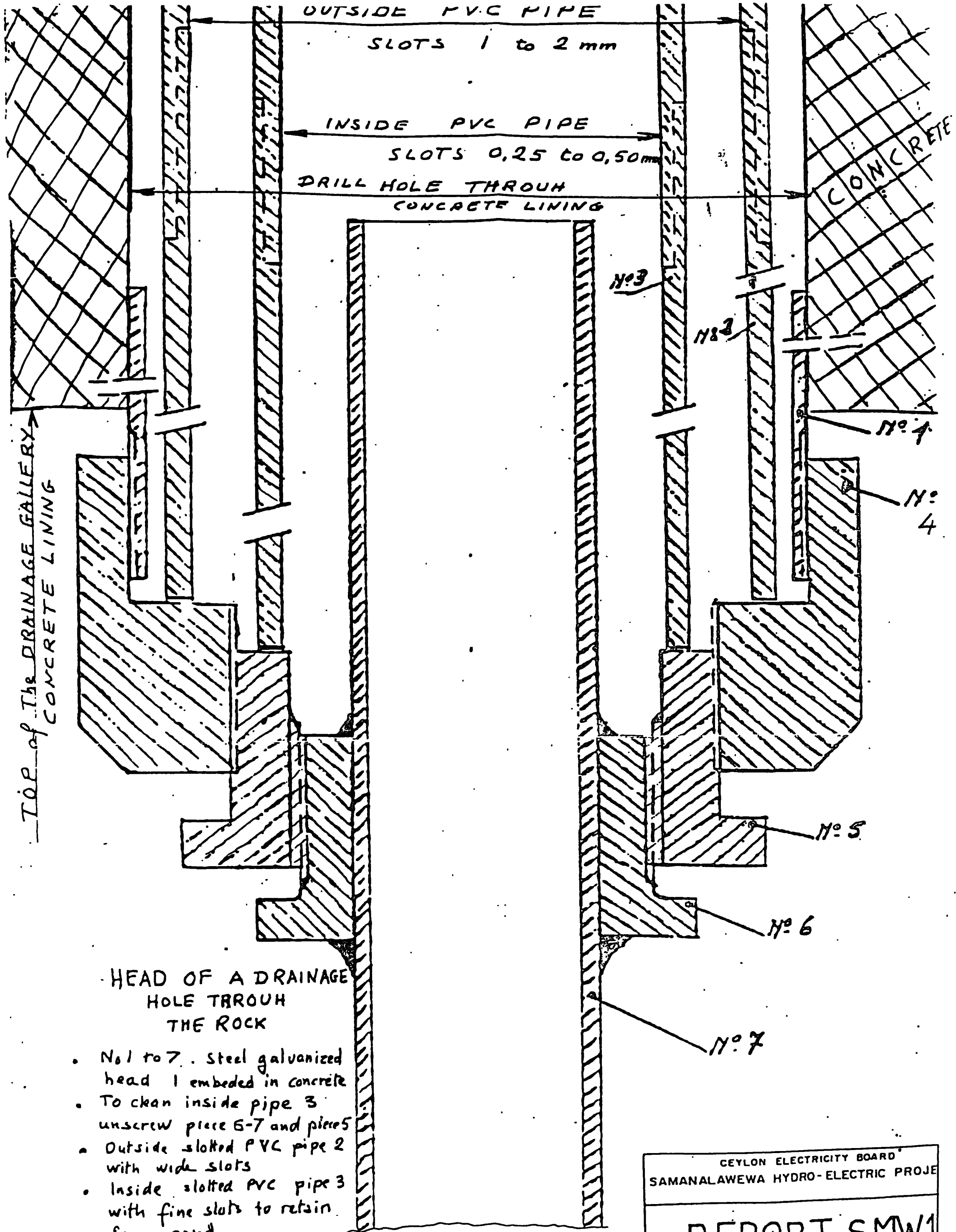
CEYLON ELECTRICITY BOARD
SAMANALAWEWA HYDRO-ELECTRIC PROJECT

REPORT SMW.1

Drainage Adits

December 92 | Figure 9

Site and Boreholes Location



HEAD OF A DRAINAGE
HOLE THROUH
THE ROCK

- No1 to 7. Steel galvanized head 1 embeded in concrete
- To clean inside pipe 3 unscrew piece 6-7 and piece 5
- Outside slotted PVC pipe 2 with wide slots
- Inside slotted PVC pipe 3 with fine slots to retain fine sand

CEYLON ELECTRICITY BOARD SAMANALAWEWA HYDRO-ELECTRIC PROJE
REPORT.SMW.1
30 December 92 | Figure 10

CEYLON ELECTRICITY BOARD

SAMANALAWEWA HYDROELECTRIC PROJECT

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ANNEXURES

1. Terms of reference for the Review Panel
2. Right Bank Ridge seepages : Groundwater Behaviour and Leakages Analysis
3. Cabinet Sub-Committee Main Recommendations on Samanalaweva Right Bank landslide and leakage
4. Itinerary for Mr. G. Post's Site Visit
5. List of participants
6. Documents handed to G. Post
7. P.V.C. Drainage slotted pipes (List of Manufacturers)

SAMANALAWEWA HYDROELECTRIC PROJECT

THIRD PARTY REVIEW PANEL

Report SMW-1

ANNEXE 1

TERMS OF REFERENCE FOR THE REVIEW PANEL

1. To assess the stability of Samanalawewa Dam, as built and under the presently (29 November - 3 December 1992) hydrogeological conditions.

2. To assess the stability of the right bank of Samanalawewa Reservoir under present and short term future conditions before remedial measures are complete.

3. To review available proposals for conceptual design of remedial measures for the Samanalawewa Dam and Reservoir and to suggest on the most appropriate proposal.

4. The Review Panel Report and recommendations should be completed and issued to Ceylon Electricity Board by 25th December, 1992.

SAMANALAWEWA HYDROELECTRIC PROJECT

THIRD PARTY REVIEW PANEL

Report SMW-1

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SAMANALAWEWA HYDROELECTRIC PROJECT
THIRD PARTY REVIEW PANEL
Report SMW-1

ANNEXE 2

RIGHT BANK RIDGE SEEPAGES

1. GROUNDWATER BEHAVIOUR AND LEAKAGE ANALYSIS

1.1. General

The fundamental finding obtained from the piezometric measurements is that the groundwater level across the right bank ridge is almost **horizontal**, whatever the reservoir level. Before the start of impounding it was steady at upstream river level, during impounding it closely followed the reservoir level and after the water burst of 22 October, 1992 it dropped, first with the pressure relief due to the heavy leakage, then, still horizontally, with the slow lowering of the reservoir level. The piezometric surface is amazingly horizontal over the whole surface equipped with piezometers, with differences less than one meter, possibly within the accuracy of the reference levels of the individual piezometers. This behaviour indicates a very **high global permeability.**

Another interesting finding is the very short time lag between any change in upstream head and its effect on the groundwater level: This indicates that, although the rock mass is very permeable its **volume of voids is moderate.**

Finally it is obvious that **the grout curtain have no effect on the groundwater**, the downstream heads being equal to the upstream heads. This is most probably due to the fact that the curtain is "hanging" instead of being connected to an impervious formation, both underneath and at its south end, and the impervious layers such as the granulitic gneiss GRA3 are punctured by several NE-SW tensional actonis features and E-W bedding (foliation) shear faults along the strike.

1.2. Summary of Events

A brief summary of the significant events regarding the groundwater behaviour during the construction period until date, is presented for the sake of completeness of the discussion.

1.2.1. Before Start of Impounding

The groundwater level was at El. 380 approximately over the whole area of the ridge. This was the natural level of a broad watertable, reaching a spring far downstream on the left bank. The river level was as El. 380-385 varying rapidly with small flash-floods.

1.2.2. During impounding

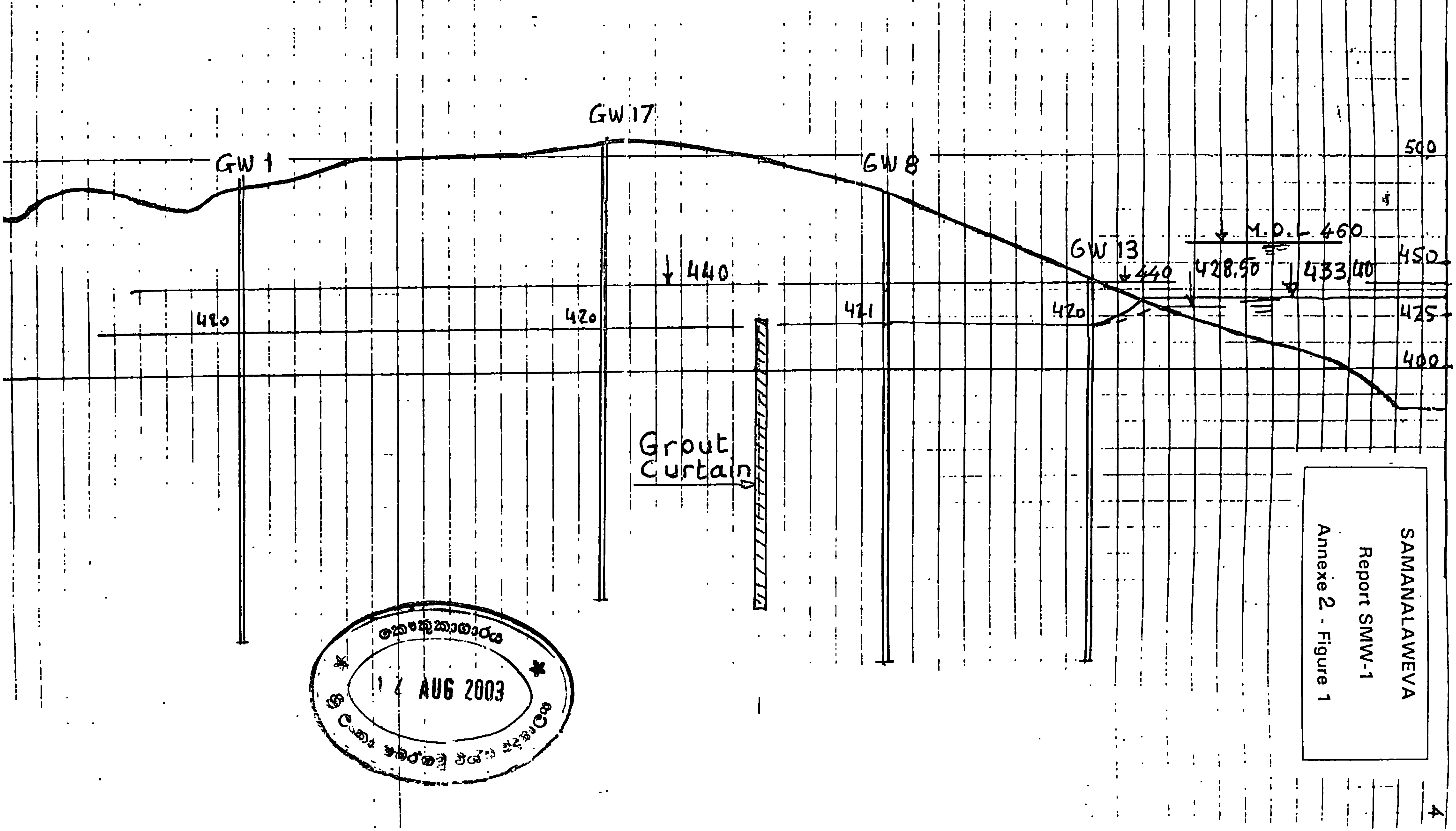
The reservoir raised, starting on 1st June 1991, from El. 371.00 to a maximum of El. 439.51, reached on 15 October, 1992. On 21 October the reservoir level was at El. 439.01. During this period the groundwater raised similarly, in a horizontal fashion, from El. 380 to El. 439 approximately, all piezometers following the reservoir level, including piezometers on the downstream side of the ridge. This condition meant that no significant seepage took place from the reservoir, the ridge being watertight in its downstream zone (Figure 1).

1.2.3. After the Water Burst

The sudden opening, on 22 October 1992, of a natural exit area for the water trapped inside the ridge, and the discharge of approximately 7 m^3/s, resulted in an immediate water pressure release. The drop in the piezometric levels was spectacular and it affected all piezometers simultaneously. Within a few hours the water table dropped by 20 m, followed by a further drop of about 5 m while the reservoir level was lowered. Shortly afterwards there was a slight "rebound" of the water table, which rose back, still in all piezometers, by some 6 m. This passage through a minimum level followed by a return to a higher level is obviously due to a change in the downstream control section.

At the time of the visit (29/11/92), with the reservoir drawdown to El. 427.11, the water table in the ridge was at El. 420 (see Figure 1) and the leakage discharge was of 2 m^3/s. A few days later, on 3 December, the water table was close to El. 419.

SIMPLIFIED SECTION ACROSS THE RIDGE



SAMANALAWEVA

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Annexe 2 - Figure 1

1.3. Model proposed for Explaining the Observed Behaviour

1.3.1. Principle

In order to get a clear understanding of the peculiar behaviour of the groundwater it is proposed to consider three distinct zones in the ridge :

i) the ingress zone at the reservoir contact,

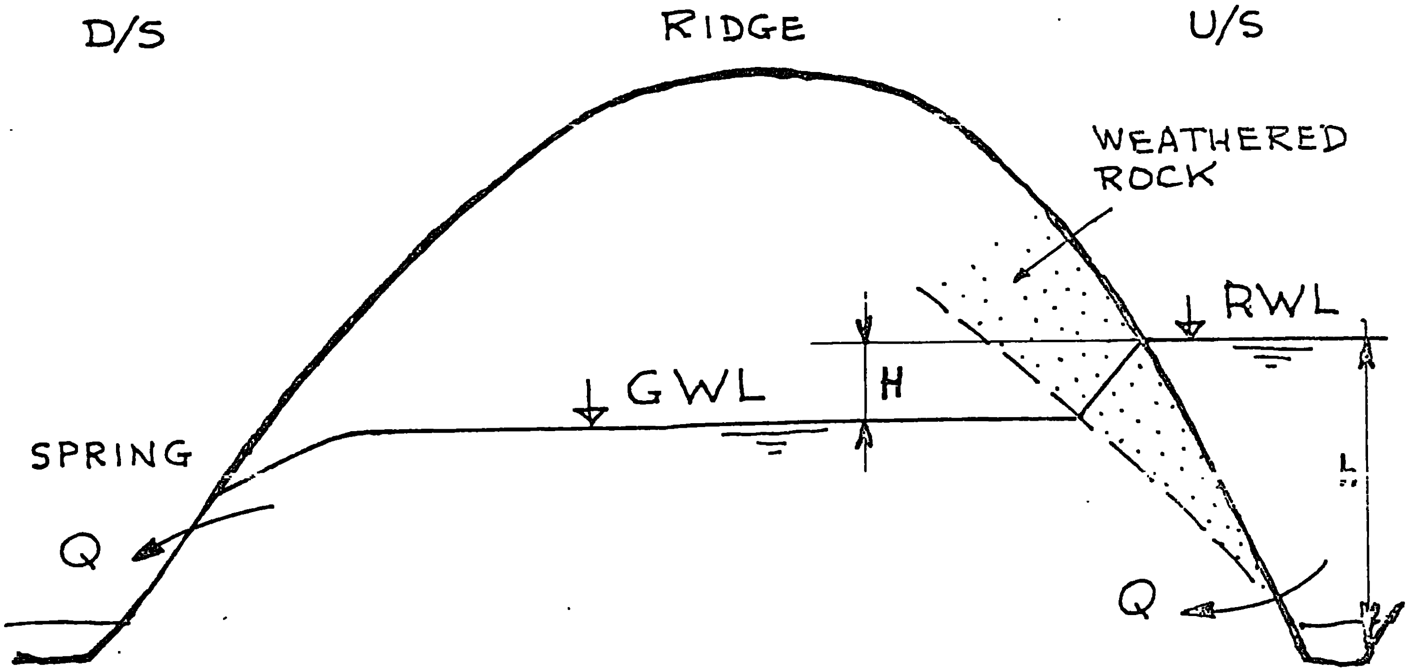
ii) the main groundwater table, always horizontal, and,

iii) the exit zone at the location of the downstream spring (Fig. 2).

The principle of this simplified model is to consider that at any **time the discharge of the water through the ingress area, determined by the level differential between reservoir and watertable, is equal to the downstream spring discharge.** The flow regime Q = f(H) must therefore be consistent with the discharge Q observed downstream, and the head differential H between reservoir level (RWL) and groundwater level (GWL).

Let us consider the approximate values of Q, RWL, GWL measured at three dates after 22 October, 1992, and shown in Figure 3 :

	A 31.10.92	B 09.11.92	C 03.12.92
Q(m^3/s)	3.0	2.5	1.8
RWL(m)	433.5	431.0	427.5
GWL(m)	420.5	420.0	419.5

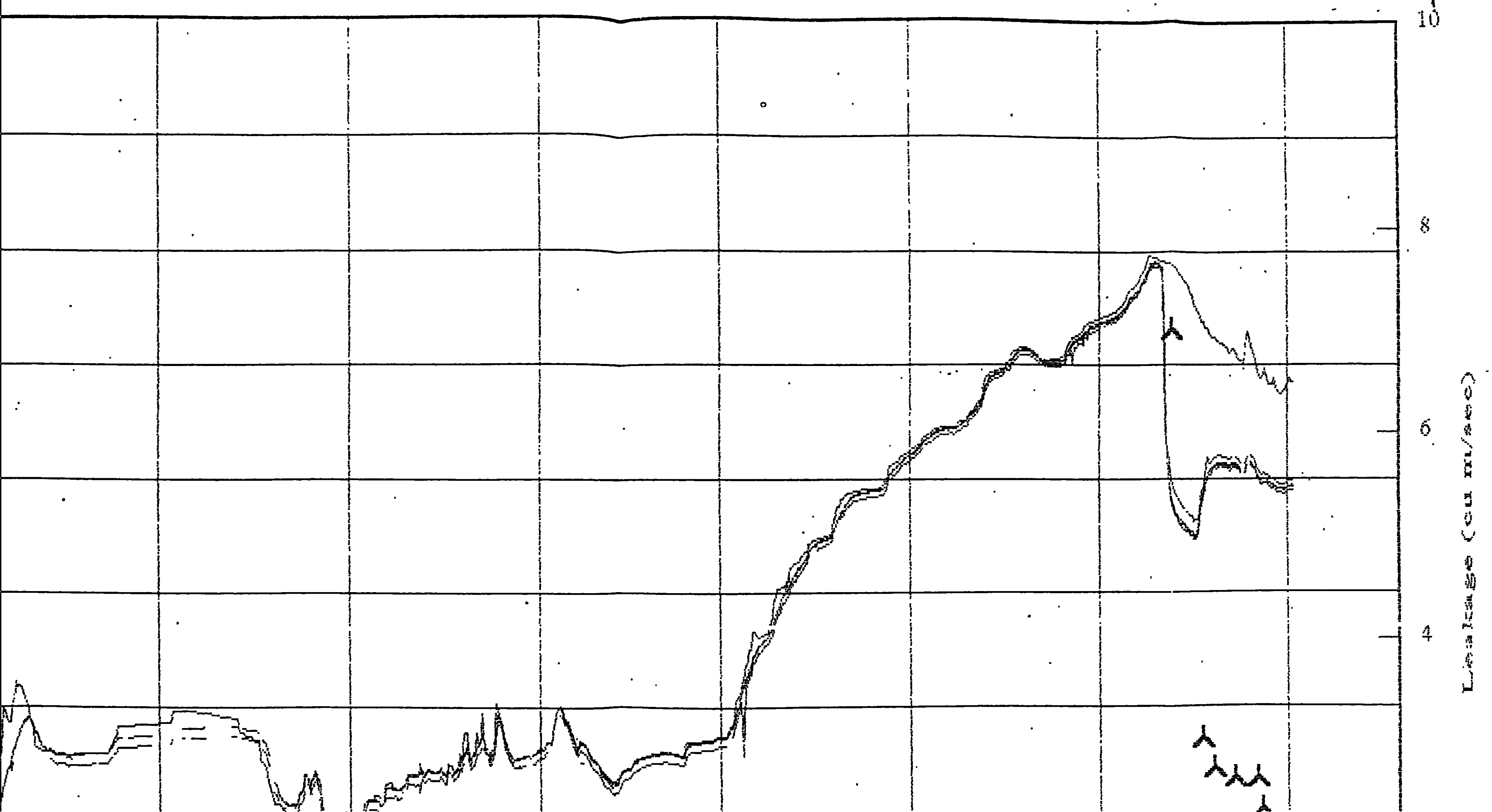


SAMANALAWEVA

Report SMW-1
Annexe 2 - Figure 2
RIGHT BANK LEAKAGE

SAMANALAWEWA HYDRO-ELECTRIC PROJECT

GROUNDWATER RESPONSES



The flow regime through the overburden layer forming the ingress zone is assumed to follow the discharge law :

$$Q = M\,H^n \qquad (1)$$

H is the head differential H = RWL - GWL,

M is a factor incorporating the effective area of percolation S, the gradient of percolation i and the permeability k,

n is a constant depending upon the type of flow regime.

1.3.2. First Approximation

In a first approximation it is assumed that the area S through which the ingress of water takes place **does not increase when the reservoir level is raised.** A number of attempts have been carried out to adjust the above parameters to the actual data. It is interesting to note that any value of $n > 1$ led to inconsistencies, such that the formulae obtained for the lower reservoir levels (on 09.11.92 and 03.12.92) could not accommodate the flow conditions observed at higher levels. Even in assuming that area S varied with the elevation, they implied that S would have to be smaller for higher levels than for lower levels, which would be absurd

The best fit was obtained with $n = 1$, yielding $M_A = 0.232$, $M_B = 0.227$ and $M_C = 0.225$, that is $M = 0.23$ approximately.

$$Q = 0.23\,H \qquad (2)$$

The value $n = 1$, which would strictly apply to a Darcy's regime, is an indication that the percolation is controlled by a **fine pervious soil** rather than by wide open cracks in rock.

The constant value of $M = S.i.k$ (= 0.23) is consistent with a **constant area of ingress** S, associated with given, values for i and k. For $i = 1$ and $k = 10^{-4}$ m/s the effective area of ingress would be $S = 2\,300$ m^2, entirely located below El. 427.5, and possibly much lower. With $i = 1$ and $k = 10^{-3}$ m/s, or $i = 10$ and $k = 10^{-4}$ m/s, this area would be reduced to $S = 230$ m^2.

1.3.3. Second Approximation

It is now assumed that there is an increase of S with the reservoir height, according to the law $S = c.h^m$. In this equation c and m are two constants and h is the total height, i.e. h = RWL-380. The three sets of data considered in paragraph 1.3.1. allows the determination of the three constants n, m and N of Equation (3).

$$Q = N\,h^m\,H^n \tag{3}$$

A good approximation of the solution is n = 0.77, m = 1.15 and N = 0.0042 yielding

$$Q = 0.0042\,h^{1.15}H^{0.77} \tag{4}$$

As opposed to the previous case the concept of Darcy's permeability is no longer valid and it is not possible to assess the area of the ingress zone. However, the exponents of Equation (4) indicate that most of this area is at the lower levels. The equation can be used for attempting to extrapolate the present conditions to the future full reservoir, as will be shown in the forthcoming section.

1.4. Extrapolations

The above exercise is based on a **limited amount of data**, particularly as regards the range of heads. In that respect it is vital that, as recommended in this report, the reservoir level be further lowered and raised again. Then the flow laws proposed here could be confirmed or otherwise. Pending such a check, Equations (2) and (4) can be used for a tentative extrapolation of the leakage for a full reservoir, assuming that no remedial measures are implemented and that no further degradation develop. This assumption implies that **no other privileged water path will develop**, either by internal erosion or through a downstream blow-out, when the reservoir level is raised above El. 440.

Assuming a constant GWL, that is a full capacity for the downstream spring to discharge a flow similar to the flow of 33 October with no increase in GWL, the head value is H = 460 - 420 = 40 m and Equation (2) yields :

$$Q = 9\ m^3/s$$

Whatever the actual conditions in the ingress area ($S = 2\,300\ m^2$ with $k = 10^{-4}$ m/s, or $S = 230\ m^2$ with $k = 10^{-3}$ m/s, or any other combination depending on k), it is certain that some permeability decrease could be achieved by the **remedial blanket**, resulting in a lower discharge. But it might prove difficult to go down as low as 2 m^3/s in one season, several successive treatments being possibly required.

On the other hand, the **lowering of the water table in the right bank is a must for the safety of the scheme in the long run.** Then, depending on the degree of success of the **remedial drainage**, and for Full Supply Level (El. 460), GWL will go down to El. 415 or better to El. 400. In the latter case Equation (2) would give, with no effect of the upstream blanket.

$$Q = 13.5 \text{ m}^3/\text{s}$$

Equation (4), used with similar hypotheses, gives $Q = 11.5$ m^3/s and $Q = 15.5$ m^3/s respectively, giving slightly less favourable results than the first approximation.

1.5. Study of the Downstream Spring

A similar analysis has been tried for modelling the downstream spring discharge. Unfortunately **the true control level of the spring is not known.** It is obvious that it cannot be El. 400 where the water daylights, because variations from $Q = 3.0$ m^3/s to $Q = 1.8$ m^3/s are not consistent with apparent head variations of $H = 20.5$ m to $H = 19.5$ m respectively (420.5 - 400 = 20.5 m and 419.5 - 400 = 19.5 m). One has to assume that the real exit level, which controls the discharge, **is inside the hill and well above El. 400.** Taking the law expressed by Equation (5) :

$$Q = LH^n \tag{5}$$

one obtains $H = 3.5$ m and $H = 2.5$ m for $n = 1.5$ i.e. a control level at El. 417
$H = 2.5$ m and $H = 1.5$ m for $n = 1.0$ i.e. a control level at El. 418
$H = 1.5$ m and $H = 0.5$ m for $n = 0.5$ i.e. a control level at El. 419

Note that, in the present conditions and in the worst case ($n = 0.5$), the release of $Q = 9$ m^3/s would impose a water table at El. 433.

Should these results be confirmed, they may have an impact on the design of the remedial drainage works.

Note that, whatever the flow type, the minimum orifice area is of the order of 1 m^2, but probably significantly more owing to the high rugosity of its walls. As a matter of fact the orifice most likely consists in a number of small conduits, with a total area much in excess of 1 m^2.

It is obvious that, in order to lower the ground water by drainage, it will be necessary to open an exit section below the present control level, (which seems to be located at present in the vicinity of El. 418), and probably not much above the downstream river level. The natural GWL before impounding was between El. 380 and El. 385, whereas it is around El. 419 at present. After drawing down to El. 424 it might go down to about El. 415. Therefore, the drainage adit will have to be executed above this level, to avoid too many difficulties with underground water.

1.6. Further Measurements

It is vital for a better understanding of the seepage conditions that further measurements of sets of RWL, GWL and Q be analysed. It will be particularly valuable to know the flow conditions for the low reservoir levels which should be obtained soon, down to El. 424. In addition it is recommended to test the effect to a new rise, say up to El. 430, so as to check whether the underground water paths are stable or otherwise.

SAMANALAWEWA HYDROELECTRIC PROJECT

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ANNEXE 3

JAYAWARDENAPURA PARLIAMENTARY SUB-COMMITTEE MEETING-3 December 1992

CABINET SUB-COMMITTEE MAIN RECOMMENDATIONS ON SAMANALAWEWA RIGHT BANK LANDSLIDE AND LEAKAGE

The following decisions were made with the Hon. Prime Minister as Chairman :

1. To operate the turbine with the water level maintained between 430 and 424.
2. To gradually bring down the water level from 428 to 424.
3. During this time the ground water table and leakage downstream will be monitored. Then raise the reservoir level from 424 to 430 and monitor downstream water table and leakages.
4. Mr. G. Post will submit a report before X'Mas after studying the available data and consulting Mr. P. Londe in Paris.
5. As remedial measures to ease the ground water pressure as recommended by Mr. G. Post, drill holes will be provided at Adit D first, as drainage channel, with filtering device to prevent sediment from passing through such drainage pipes.
6. In the absence of other Consultants viz : Mr. Hoek, Mr. Londe and Mr. Barry Cook, Dr. Back said that he would try to contact them and if possible to arrange for a joint visit to Sri Lanka. Mr. Post also agreed to make the joint visit in March or April if it is practically possible.
7. Remedial measures already planned by JVS and Gibbs will be continued.

SAMANALAWEWA HYDROELECTRIC PROJECT

THIRD PARTY REVIEW PANEL

Report SMW-1

ANNEXE 4

ITINERARY FOR Mr. G. POST's SITE VISIT

Sunday	- 29.11.1992	- Arrival at Colombo early morning and then proceed to site. Inspection of site - 2.00 p.m. visit of landslide and leakage, visit of grouting adit and access adits.
Monday	- 30.11.1992	- Continue the inspection on site, inspection of core boxes of drill holes GW1, GW2, GW8, GW11, GW12, GW13, GW17, GW18 MS1, MS2, MS3, MS4, B1, B3, B6 H568 (tertiary hole)
Tuesday	- 01.12.1992	- Move to Colombo. Study of collected information on site.
Wednesday	- 02.12.1992	- 9.00 a.m. - 11.00 a.m. Meeting with Dr. Kulasinghe and others concerned at the Office of the D.G. Mahaweli Authority.
	02.12.1992	- p.m. continue studies.
Thursday	- 03.12.1992	- a.m. continue studies - 03.00 p.m. Meeting with Cabinet Sub Committee at Prime Minister's Office at Jayawardenapura Parliament
Friday	- 04.12.1992	- Departure for Paris.

SAMANALAWEWA HYDROELECTRIC PROJECT

THIRD PARTY REVIEW PANEL

Report SMW-1

ANNEXE 5

LIST OF PARTICIPANTS

A. ATTENDANCE AT THE SITE INSPECTION AND MEETING HELD ON 29.11.1992 ALONG WITH MR. G. POST, MEMBER, PANEL OF EXPERTS AT SAMANALAWEWA DAM SITE

	NAME	*POSITION*	*ORGANISATION*
1.	Mr. M. Yamaguchi	Director Nippon Koei	JVS (Nippon Koei).
2.	Mr. K. Wada	Construction Manager, JVS	JVS (Nippon Koei).
3.	Mr. T. Takahashi	Geologist	JVS (Nippon Koei)
4.	Dr. P.A.A. Back	Director	Sir Alexander Gibb & Partners
5..	Mr. P.O. Squire	Geologist	Sir Alexander Gibb & Partners
6.	Mr. David Chisnall	Geologist-Geotechnical Engineer	Sir Alexander Gibb & Partners
7.	Mr. Vernon Pereira	Senior Geologist	Central Engineering & Construction Bureau
8.	Mr. Sri Kantha	Grouting Engineer	Central Engineering & Construction Bureau
9.	Mr. Chandrasiri	Dam/Monitoring Engineer	Central Engineering & Construction Bureau
10.	Mr. Laxsiri	Reservoir Engineer	C.E.B.
11.	Mr. S. Ganesharajah	Project Director Samanalawewa HEP	C.E.B.
12.	Mr. N.A.J. Pereira	Chairman, CEB	C.E.B.
13.	Mr. G. Post	Member, POE Samanalawewa	Individual Consultant

B. ATTENDANCE AT THE SITE INSPECTION AND MEETING HELD ON 30.11.92 ALONG WITH MR. G. POST, MEMBER, POE, AT THE SAMANALAWEWA DAM SITE OFFICE

	NAME	*POSITION*	*ORGANISATION*
1.	Mr. M. Yamaguchi	Director	JVS (Nippon Koei)
2.	Mr. K. Wada	Construction Manager, JVS	JVS (Nippon Koei).
3.	Mr. T. Takahashi	Geologist	JVS (Nippon Koei)
4.	Dr. P.A.A. Back	Director	Sir Alexander Gibb & Partners
5.	Mr. P.O. Squire	Geologist	Sir Alexander Gibb & Partners
6.	Mr. David Chisnall	Geologist-Geotechnical Engineer	Sir Alexander Gibb & Partners
7.	Mr. Vernon Pereira	Senior Geologist	Central Engineering & Consultancy Bureau
8.	Mr. Sri Kantha	Grouting Engineer	Central Engineering & Consultancy Bureau
9.	Mr. Chandrasiri	Dam/Monitoring Engineer	Central Engineering & Consultancy Bureau
10.	Mr. S. Ganesharajah	Project Director, Samanalawewa HEP	C.E.B.
11.	Mr. G. Post	Member, POE, Samanalawewa	Individual Consultant

C. ATTENDANCE AT THE MEETING HELD AT 9.00 a.m. ON 02.12.1992 WITH DR. A.N.S. KULASINGHE AND MR. POST, MEMBER, PANEL OF EXPERT, AT THE OFFICE OF THE DIRECTOR GENERAL MAHAWELI AUTHORITY

	NAME	*POSITION*	*ORGANISATION*
1.	Mr. K.H.S. Gunatilake	Director General	Mahaweli Authority
2.	Dr. A.N.S. Kulasinghe	Chairman (CEBC)	Central Engineering Consultancy Bureau
3.	Mr. Tandon	Consultant (CEBC) Hydrologist	Central Engineering Consultancy Bureau
4.	Mr. Vernon Pereira	Senior Geologist (CEBC)	Central Engineering Consultancy Bureau
5.	Mr. T.D. Wickramasinghe	Hydrologist (CEBC)	Central Engineering Consultancy Bureau

6.	Mr. S. Ganesharajah	Project Director, Samanalawewa HEP	Ceylon Electricity Board
7.	Mr. Georges Post	Member, Panel of Experts, SHEP	Individual Consultant

D. ATTENDANCE AT THE CABINET SUB COMMITTEE MEETING HELD AT 3.00 A.M. ON 03.12.1992 AT THE PRIME MINISTER'S OFFICE AT JAYAWARDENAPURA PARLIAMENT REGARDING SAMANALAWEWA PROJECT - RIGHT BANK LEAKAGE

Present :

Hon. D.B. Wijetunga	- Prime Minister and Minister of Finance
Hon. K.D.M.C. Bandara	- Minister of Power and Energy
Hon. Ranil Wickremasinghr	- Minister of Industries, Science and Technology
Hon. Gamini Atukorale	- Minister of Lands, Irrigation and Mahaweli Development
Hon. P.Dayaratne	- Minister of Re-Construction, Rehabilitation & Social Welfare
Mr. Ackiel Mohamed	- Secretary, Ministry of Power and Energy
Mr. N.A.J. Perera	- Chairman, CEB
Mr. S. Ganesharajah	- Project Director Samanalawewa HEP
Dr. P.A.A. Back	- Director, Sir Alexander Gibb and Partners
Mr. T. Yoshimatsu	- Director, Nippon Koei & Co.
Mr. K. Wada	- Construction Manager, JVS
Mr. P.O. Squire	- Sir Alexander Gibb & Partners
Mr. T. Takahashi	- Joint Venture Samanalawewa
Mr. G. Post	- Member Panel of Experts

SAMANALAWEWA HYDROELECTRIC PROJECT
THIRD PARTY REVIEW PANEL
Report SMW-1

ANNEXE 6

DOCUMENTS HANDED TO G. POST

A. REPORTS EDITED BEFORE 22 OCTOBER 1992

1. Design Review Report Nippon Koei Co. October 1987.
2. Report on Additional Geotechnical Investigations For Samanalawewa Dam, Part 1 : Dam Foundation - Nippon Koei - Electrowatt - CEBB July 1987.
3. Samanalawewa Project - Reservoir Watertightness and the Right Bank cut-off works. A summary, November 1991 P.O., Squire, S. Takahashi, V.F. Pereira and H.L. Gunaratna.
4. Embankment Dam and Associated Works. Civil Engineering Works Operation and Maintenance Manual. Volume Ii Record Drawings. Sir Alexander Gibb & Partners December 1991
5. Review of Grouting Results, January 1992, JVS and Design Engineer.
6. Supplementary Notes to Reservoir Impounding and Monitoring Plan with letter JKW/POS/OO2 dated 10 February 1992 signed K. Wada & P.O. Squire
7. Calculation sheets showing drawdown capability for various assumptions (low level outlet, spillway, power plant) with letter J 82870C/600 dated 7 december 1992 signed by P. Back.

B. GEOLOGICAL MAP AND SECTIONS WITH PIEZOMETRIC LEVELS

1. Geological map and sections of the Right Bank Ridge scale 1/5 000 (colored)

2. Piezometric levels of Boreholes and MS3 October 1992 — Area B = GW13, GW2, GW3, M51

3. Area B = GW4, GW5, GW7 and GW8

4. Area A = GW9, GW11, GW12 and MS2

5. Area A = B1, B2, B3, B4 and B5

6. Reservoir Water level - Ground water level (GW1 - GW18) - Leakage volume

7. location Map of cross sections along Adit Da, aa, bb, cc 1/500

8. Geological sections with GWL, along Adit Da 1/1000

9. Geological sections with GWL, along section aa (MS4, MS1) 1/1000

10. Geological sections with GWL, along section bb (MS1, Adit Da) 1/1000

11. Geological sections with GWL, along section cc (MS3, MS4, B3) 1/1000

12. Piezometric Head of Right Abutment, on 21 October 1992, downstream of the Grout Curtain. Section along adit Da-Db 1/2000.

13. Piezometric Head (standpipe piezometers) downstream of the dam Grout Curtain on 21 October 1992, Dam Foundation (Fig. B-4), Elevation 1/2000.

C. MONTHLY REPORT ON DAM - RESERVOIR MONITORING

October 1992 supervised by J.V.S. prepared by CECB.

D. REPORTS ON 22 OCTOBER LANDSLIDE INCIDENT

1. Right bank status updated 2 November (one graph RWL, GWL seepage) by K. Wada, S. Nishiota, S. Takahashi - P. Squire, S. Sutton.

2. Proposed investigation works for reservoir remediation, P. Back and T. Yoshimatsu 26.11.92.

3. Review of Dam Embankment safety following reservoir seepage incident of October 1992, 4.11.92.

4. Summary of Current status and Recommendations 9 November 1992, P. Back and T. Yoshimatsu.

5. the landslip and subsequent leakage of 22 October 1992, Draft Report for CEB (to send to ODA via DER) P. Back and T. Yoshimatsu.

E. THE REMEDIAL MEASURES REQUIRED FOR SAMANALAWEWA RESERVOIR

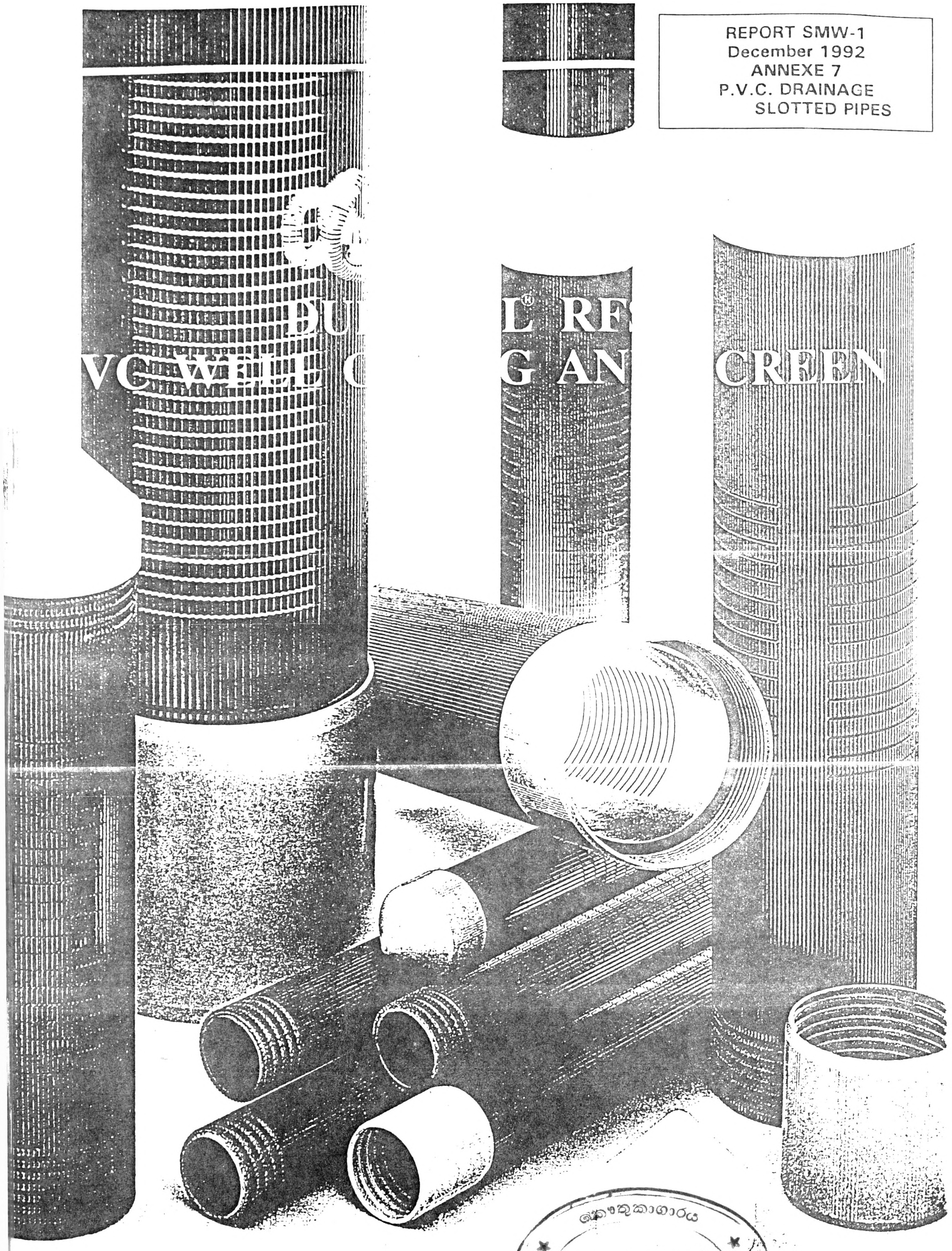
A summary of Proposed principles. Draft November 1992. GIBB-JVS.

F. AERIAL PHOTOGRAPHS (1/20 000) N° 56, 57, 58, 59

G. REPORT ON PRELIMINARY TERRAMETER (RESISTIVITY SURVEY) INVESTIGATIONS CONDUCTED ON THE RIGHT BANK OF SAMANALAWEWA DAM

D.V.A. Senaratne and S. de S. Wijesundara, 2 November 1992.

REPORT SMW-1
December 1992
ANNEXE 7
P.V.C. DRAINAGE
SLOTTED PIPES



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Il PVC rigido usato per la realizzazione di tubi e filtri «DURVINIL RFS» resiste senza subire alterazioni a tutti i tipi di acque sotterranee, all'acqua marina ed a soluzioni acide o alcaline diluite.

I filtri ed i tubi ciechi «DURVINIL RFS» possono subire ripetuti trattamenti con le sostanze chimiche comunemente usate per la disinfezione e la rigenerazione dei pozzi.

I tubi sono provvisti esternamente di rigature longitudinali allo scopo di impedire alla sabbia ed al ghiaietto di depositarsi sulla superficie del tubo in prossimità delle fessure.

È stato dimostrato che il tubo realizzato con rigature esterne longitudinali raddoppia la permeabilità rispetto ad un filtro liscio dello stesso diametro, pertanto è possibile ridurre il diametro della perforazione ottenendo un notevole risparmio economico.

Le rigature esterne contribuiscono inoltre alla stabilità del filtro stesso ed offrono una maggiore resistenza alla compressione esterna ed al carico assiale della sezione tubolare portante.

I tubi e i filtri «DURVINIL RFS» vengono forniti generalmente in barre lunghe 3 o 6 metri, filettate maschio-maschio alle 2 estremità con un filetto trapezoidale passo 4 filetti per pollice.

Ogni barra è fornita completa di un manicotto di giunzione in A.B.S., filettato

Su richiesta si possono fornire anche tubi ad alta resilienza per impiego in condizioni particolari.

Nei diametri fino a 4" sono inoltre disponibili manicotti filettati con passo Withworth per permettere modifiche ed adattamenti dei tubi in cantiere.

«DURVINIL RFS»

Rigid PVC used for the fabrication of pipes and screens «DURVINIL RFS» resists all types of underground water, to sea water and to acidic or alkaline solutions.

Well casings and screens made of «DURVINIL RFS» can be subjected to repeated tratements with chemical substances usually used for disinfecting and regenerating wells.

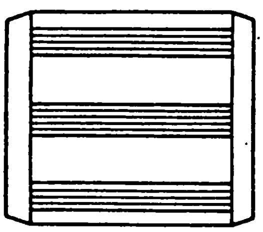
Pipes are supplied with external longitudinal ribs with the purpose of alleviating the deposition of sand and gravel on the pipe surface near the slots. It has been shown that pipes having external longitudinal ribs double permeability in comparision with a smooth screen of the same diameter, so it is possible to reduce the well hole obtaining substantial savings.

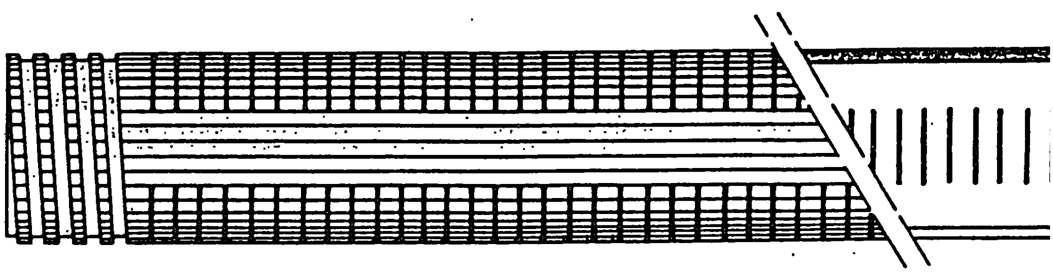
The external ribs contribute to the structural stability of the screen and they offer a higher resistance external radial compression and axial loads on the tubular section.

Pipes and screens made of «DURVINIL RFS» are generally furnished in lenghts of 3 or 6 m threaded at both ends with a trapezoidal thread pitch of 4 threads per inch. Each length is supplied complete with a threaded sleeve in A.B.S.

Upon request it is possible to supply pipes at high resilience to be used in particular conditions.

For pipes of up to 4 inch. diameter Withworth threaded, sleeves are available to achieve compatibility or interchangebility with existing pipe work.

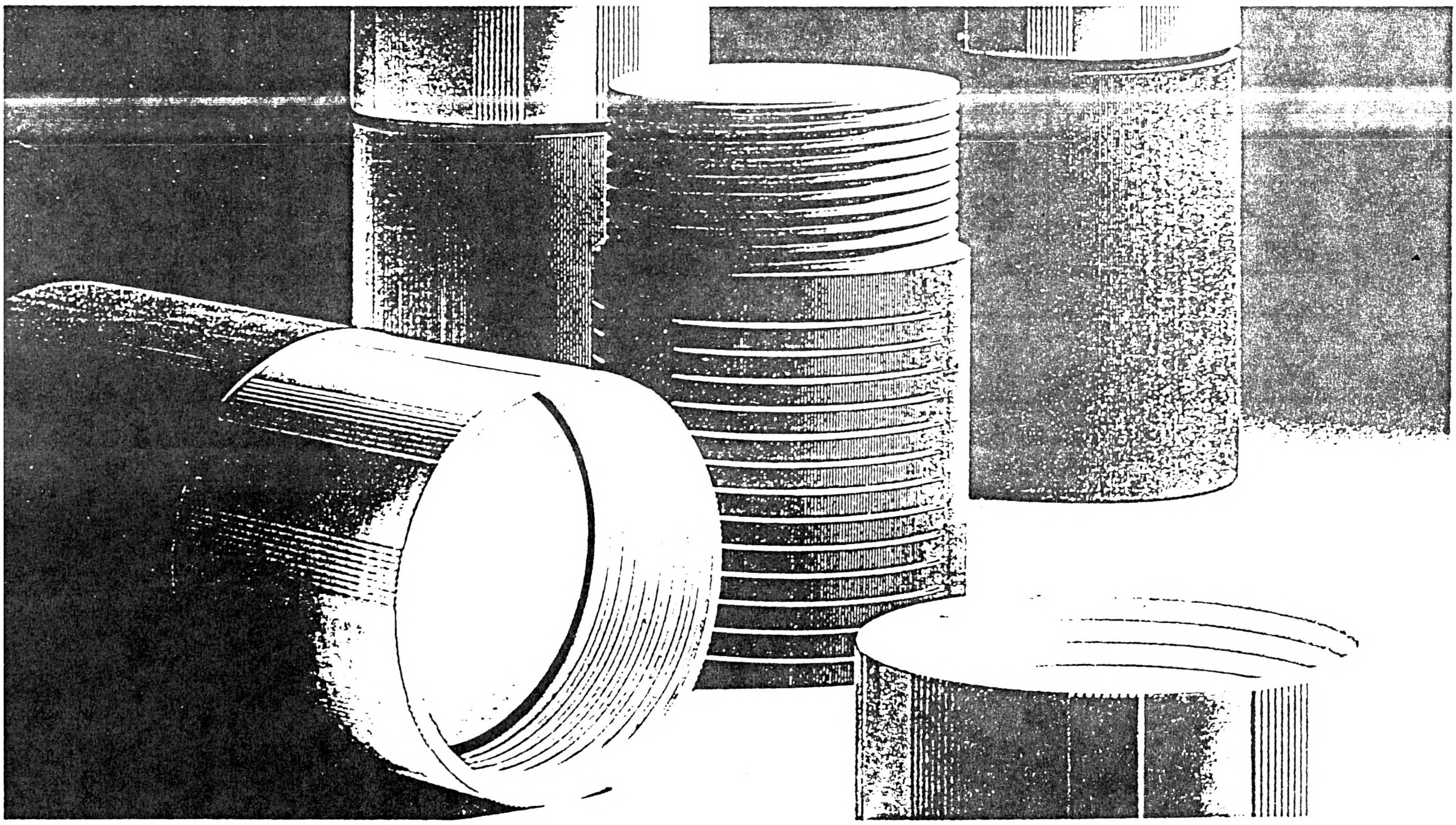




PROPRIETÀ FISICHE DEL DURVINIL ® RFS

MATERIAL SPECIFICATIONS DURVINIL® RFS

PROPERTY	UNIT	TEST METHOD				DURVINIL RFS
		ASTM	DIN	ISO	UNI	
Specific gravity	g/ml	D 792	53479	R 1183	4294	1,48
Thermal linear expansion	cm/cm°C	D 696	53328		6061/67	$5 \cdot 10^{-5}$
Softening point Vicat (5 kg. vas. oil)	°C	D 1525	53460	R 306	5642/65	82
Deflection temperature	°C	D 648	53461	R 75	5641/65	72
Tensile strength (5 mm/min) Yeld strenght Tensile strength Elogation at yeld	 kg/cm² kg/cm² %	D 638	53455	R 527	5819-66	 455 470 4
Elastic Modulus	kg/cm²	D 790	 53457	R 178	7219-73	32.000 33.500
Impact strenght A - Izod a+23°C a+ 0°C a—10°C B - Charpy a+23°C a+ 0°C	 kg. cm/cm kg. cm/cm²	 D 256-A	 53453	 R 180 R 179	 6323-68 6062-67	 7 6 5 4,5 4
Hardness A - Rockwell, L scale B - Shore D		 D 785 D 2240	 50103 53505	 R 868	 CT 218 4916-74	 90 81



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MISURE STANDARD

Standardisized Range:

DIAMETRO NOMINALE NOMINAL DIAMETER	In	1½"	2"	2½"	3"	4"	4½"	5"	6"	8"	10"
DIAMETRO INTERNO INSIDE DIAMETER	mm	40	52	66	79	105	116	129	152	205	257
DIAMETRO ESTERNO OD OVER PIPE	mm	50	62	77	90	118	130	145	170	230	285
SPESSORE WALL THICKNESS	mm	5	5	5,5	5,5	6,5	7	8	9	12,5	14
DIAMETRO MANICOTTO MAX. MM. OVER CONNECTIONS	mm	54	67	85	97	127	140	155	181	243	295

SPESSORI DIAMETRI DIVERSI A RICHIESTA - OTHER THICKNESS IS AND DIAMETERS ON REQUEST

SLOT SIZE			0,25	0,5	1	1,5	2
DN	n	Σ ā	≈ f				
1½"	3	84	3,5%	6,6%	6,6%	9,5%	—
2"	3	108	3,5%	6,6%	6,6%	9,5%	—
2½"	3	138	3,5%	6,6%	6,6%	9,5%	—
3"	3	165	—	6,6%	6,6%	9,5%	12%
4"	3	220	—	6,6%	6,6%	9,5%	12%
4½"	3	242	—	6,6%	6,6%	9,5%	12%
5"	6	270	—	—	6,6%	9,5%	12%
6"	6	318	—	—	6,6%	9,5%	12%
8"	6	425	—	—	6,6%	9,5%	12%
10"	5	596	—	—	—	9,5	12%
PASSO FESSURE SLOT PITCH		mm	4,75	5	10	10,5	11

≈ f = AREA LIBERA DI PASSAGGIO IN PERCENTUALE

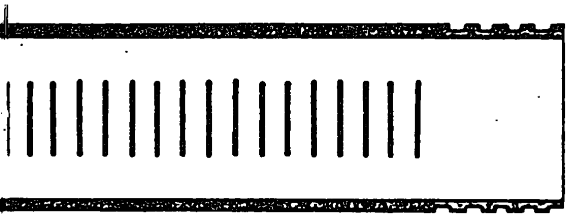
Σ ā = SOMMATORIA DELLE LYNGHEZZE DELLE FESSURE SULLA SEZIONE TRASVERSALE INTERNA DELLA CIRCONFERENZA

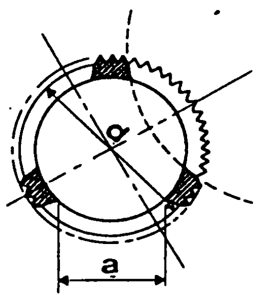
n = NUMERO DELLE FESSURE SULLA SEZIONE TRASVERSALE

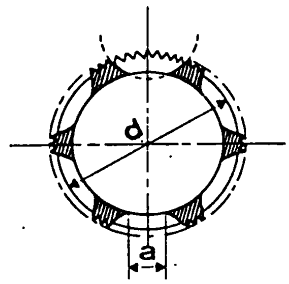
≈ f = OPEN FREE AREA IN %

Σ ā = SUMMATION OF SLOT LENGHTS OVER THE INTERNAL CIRCUMFERENCE OF THE CROSS-SECTION

n = NUMBER OF SLOT ON THE CROSS-SECTION







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