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REPORT

OF

THE

ONE-MAN

PRESIDENTIAL COMMISSION OF INQUIRY

APPOINTED

TO

INVESTIGATE

THE LEAKAGE OF WATER

FROM THE

SAMANALAWEWA RESERVOIR

COMMISSIONER : J F A SOZA
27 OCTOBER 1993

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2. On 03 November 1992 Mr D W R M Weerakoon Senior Deputy Director of Irrigation was appointed Secretary to the Commission.

3. The original date fixed for furnishing my report was 27 January 1993 but by three subsequent extensions the returnable date of my warrant was extended to 27 October 1993.

4. In the warrant issued to me there was a difference between the Sinhala and English versions which I brought to the notice of His Excellency by my letter of 31 December 1992. The expression " පීඩන ප්‍රමාණය යනා කරුණු " in paragraph (අ) of the Sinhala version appears as "water tightness" in paragraph (a) of the English version. The expression "causes" in paragraph (c) of the English version has been omitted in the Sinhala version. Section 5 of the Commissions of Inquiry Act (Cap. 393) empowering alterations did not appear to permit amendments correcting discrepancies of this type. In law if there are discrepancies of this type the Sinhala version must prevail. Hence it is the Sinhala version that I would adopt as the basis of my Inquiry. As the Sinhala version in paragraph (ආ) reads " පීඩන ප්‍රමාණය යන කරුණු " meaning literally "matters like degree of water pressure", I consider this term of reference wide enough to include "watertightness" in addition to "degree of water pressure". Further paragraph (ඇ) of the terms of reference permits me to address my attention to the question of " කාන්දුවීම්වල බලපෑම් හා ඊට අදාළ ගැටළු " i.e "water leakages its effects and problems relating thereto" and this would include the question of "causes" as well as "watertightness". In these circumstances I do not think these differences in the Sinhala and English version of the terms of reference would bar me from addressing myself to the questions of "water leakages" and their "causes" and the question of "watertightness".

5. After the Samanalawewa dam works on the Walawe Ganga were completed a trial impounding was commenced in June 1991. On 11 June 1991 when the impounding was in progress and the reservoir was at EL 399.6 m a new spring appeared 300 m downstream of the dam toe on the right bank of the Walawe river accompanied by a small overburden slip. By 11 July 1991 the discharge from this spring was 21 l/s. Thereafter installation of the grout curtain on the right bank was completed; a second trial impounding was initiated in June 1992. When this impounding was in progress at

approximately 13 00 hours on 22 October 1992 there was suddenly a massive burst discharging a torrent of muddy water close to the site where the first spring appeared. The burst was at approximately EL 400 m on the right bank of the Walawe Ganga about 300 m downstream of the dam when the water level in the reservoir had been raised to EL 438.99 m. The blow-out triggered a landslide on the right bank of the river and this aggravated the torrent of muddy water disgorging itself from the hillside. About midnight of the same day the outflow steadied to 7 cumecs but at its peak the torrent of muddy water flooding out of the hillside was not only much in excess of 7 m^3/sec but also washed away about 20 to 40,000 m^3 of earth. The access road above the landslide showed gaping tension cracks necessitating closure of the road in the interests of the safety of the public. By the next day 23 October the water gushing out from the burst was running clear at 3 m^3/sec. The water now cascading from this rather ominous looking portal (which is about 2m wide and one metre high) in the hillside runs clear at 2 m^3/sec but emits an offensive smell. The panic in the minds of the public as evinced by the Kaltota witnesses who testified before me has yet to be allayed.

6. The appointment of the Commission followed upon public agitation over this spectacular blow-out downstream of the right ridge of the Samanalawewa dam which occurred on 22 October 1992 raising serious concerns about the watertightness of the reservoir, the stability of the banks and the safety of the dam itself.

Chapter II

PRELIMINARY ACTION

7. Notices were published in Sinhala, Tamil and English in the newspapers Dinamina, Thinakaran and Daily News of 19 November 1992 inviting members of the public and organizations and institutions desirous of making representations and/or giving oral evidence to send in their representations and/or indicate their willingness to give oral evidence within two weeks from the day of publication of the notice (later extended to 16 December 1992) by registered post. Copies of these newspaper publications appear in Appendix 2 to this report.

8. Several members of the public, organizations and institutions responded to the invitation referred to above. A list of their names appears in Appendix 3 to this report.

9. On my request for the services of Counsel to assist in leading the evidence the Attorney-General very kindly assigned Mr Samith de Silva Senior State Counsel to the Commission. The services of Mr R L de S Munasinghe Civil Engineer/Geologist were secured to advise State Counsel on technical matters. Mr Sumith A Liyanage was placed in charge of administration with Mr P P Anura as Chief Clerk.

10. Although my warrant was issued on 27 October 1992 it took time to set up the infra structure. Advertisements had to be published giving time for representations and in view of the highly technical and scientific nature of the matters adumbrated in the terms of reference, mobilization time was involved before the public sittings could start.

Chapter III

PROCEDURAL METHODS FOLLOWED

11. Mr Samith de Silva presented the evidence of the witnesses assisted by Mr R L de S Munasinghe. All the evidence was recorded in Colombo except for 16 witnesses whose evidence was recorded at Kaltota on 07 August 1993 and one witness whose evidence was recorded at Killakandurakumbura in the village of Kumbalgama on 08 August 1993. A list of witnesses who gave evidence before me appears in Appendix 4 of this report.

12. I made two site inspections of the dam site and appurtenant works over two week-ends accompanied on both occasions by Mr Samith de Silva and Mr D W R M Weerakoon. On the second visit Mr R L de S Munasinghe and Mr Sumith A Liyanage also accompanied me.

13. At the beginning of the proceedings I made it clear that this was not a censorial inquiry but only a fact finding inquiry. Although one of the terms of reference mentions negligence, in the absence of definite allegations no finding would be made, in any event, on culpability. When I deal with the particular term in the reference I will discuss the question of negligence more fully.

14. All who had indicated their willingness to give oral evidence were called as witnesses except that where the party offering evidence was an organization or institution, only representatives were called. In addition persons who were closely connected with the Samanalawewa Project or were otherwise conversant with it whether as individuals or as representing institutions or organizations and were available, were called as witnesses. As part of the evidence, documents were marked which were in the possession of witnesses or which were directed to be produced. A list of the documents marked in evidence appears in Appendix 5 of this report.

15. The procedure followed was for Counsel assisting the Commission to present the evidence, viva voce and documentary, through witnesses. No allegations were framed and the inquiry proceeded purely as a fact-finding probe. My task was to receive, collate and analyse facts in order to make my findings on them.

Chapter IV

TOPOGRAPHICAL FEATURES OF THE WALAWE BASIN

16. Sri Lanka is divided into the Coastal Regions and the Central Highlands. The Central Highlands are in the southern central part of the island and are characterized by two terrace or peneplain areas probably the result of two block uplifts. The lower terrace area at about elevation 520m rises from the lower plains at about elevation 120 m. The upper terrace area has a general elevation of about 1200 m from which the higher mountain peaks rise. Here the topography is generally rugged and abrupt with juvenile drainage patterns. The main hill - mass of central Sri Lanka which constitutes the Central Highland Region has for its southern limit a great escarpment which is one of the most imposing topographic features in Sri Lanka. This escarpment is popularly referred to as the Balangoda escarpment. In most places, the northern watershed lies along the crest of this escarpment. The small rolling plateau known as Horton Plains is the only area north of the escarpment crest that is drained to the Walawe Ganga system. The break-in-slope along the base of the escarpment is the southern boundary of the region and lies entirely within the wet zone.

17. The designation "Upland Platform" has been given to the rugged deeply dissected plateau lying between the base of the main, south- facing Balangoda escarpment of the Central Highlands and the north edge of the lowland plain. The west part of the contact between the plateau and the plain is marked by an escarpment about 200 to 300 m high. This feature is described as the Kaltota scarp. The most prominent structural feature in this area is a very broad syncline which plunges west-northwest. The Kaltota scarp has been developed in beds which close the eastern end of this syncline. Immediately east of the Kaltota scarp a northeast-southwest foliation trend parallel to the scarp, persists. To the West of the Upland Platform there lie the Balangoda Platform and the Petiyagala Ridge - see map of natural Regions of the Walawe Ganga Basin - Appendix 6.

18. The Balangoda escarpment of the Central Highlands rises some 1200 to 1400 m above the level of the Upland Platform Region. The Upland

Platform Region is sub-divided into a northern Upland Intermediate Zone and a southern Upland Dry Zone. The component structures of the Samanalawewa Project are located on the northwestern edge of the middle peneplain of the northern Upland Intermediate Zone. This peneplain is confined on the north by the Balangoda escarpment and composed of metamorphic rock of the pre-Cambrian crystalline complex. The peneplain terminates in the south and south-west in the vast valley extending to the southern coast of the island.

19. The main stream of the Walawe Ganga rises in the southern part of the Central Highlands in the mountains west of Balangoda at elevation 2300 m. For the uppermost 50 km of its course, the river flows in the deep gorge to the South-East. At Watawala it turns more or less at a right angle to the North-East and at the confluence with the Belihul Oya it turns abruptly again South-East and emerging from the hills at Uggalkaltota it turns sharply southwards and traverses the lowland plain for a distance of 75 km before discharging into the Indian Ocean at Ambalantota on the southern coast of Sri Lanka. One of the tributaries of the Walawe Ganga is the Belihul Oya which has its source in Horton Plains and flows down the escarpment face on the south of the Central Highlands in deep V- shaped fault - depressions and eventually joins the Walawe Ganga.

20. The headwaters of the Walawe Ganga and the Belihul Oya rise in the Wet Zone. The Belihul Oya flows from an area which has copious rainfall especially during the inter-monsoon and north-east monsoon periods. The main stream of the Walawe Ganga and the tributaries entering it from the west originate in an area of heavy rainfall mostly during the inter-monsoon and south-west monsoon periods. The Walawe Ganga has a perennial flow thoughout its length although the flow fluctuates greatly from season to season and from year to year. Belihul Oya is olso a perennial stream. The river flow of the Walawe Ganga features a flood season and a low water season the former accounting for 85% of the river flow. The yearly river flow records at the Samanalawewa dam site from 1921 to 1976 have been studied. The maximum recorded river flow at the dam site was 2270 m^3/sec in May 1940 when there was a disastrous flood and the minimum discharge was 2 m^3/sec in September 1976. The drainage area rises to an elevation of 2300m.

21. The Walawe Ganga is the third largest river in Sri Lanka and has a good tributary system. Some note of a few of these tributaries is appropriate. Upstream of its confluence with the Belihul Oya, the Walawe Ganga is fed by another tributary the Mulgama Oya. Downstream of the Samanalawewa dam close to its left toe the Matihakka Ara a tributary flows in and a little further downstream is the tributary Kalunaide Arawa Ara flowing in from the right. As the Walawe Ganga emerges into the valley beyond Uggalkaltota two tributaries which join it are the Weli Oya which flows in from the left and the Diyawini Oya on the right.

22. The controversial dam has been built at Samanala so named it is said, from its butterfly shaped configuration, some 250 m downstream of the confluence betweeen the Belihul Oya and Walawe Ganga. The dam is situated in a gorge of the Walawe Ganga immediately upstream of its confluence with the Mathihakka Ara and about 9 km upstream of Uggalkaltota. The headworks of the project are at an elevation of about 400 m. The catchment area serving the dam is about 342 km^2. The diversion tunnel leads water southward from the reservoir to a power house at the base of the Kaltota scarp near and upstream of the confluence between the Katupath Oya with the Diyawini Oya a tributary of the Walawe Ganga. The Diyawini Oya and its tributary Katupath Oya flow in general direction eastwards more or less parallel to the Walawe Ganga. Where the Walawe Ganga and Katupath Oya come closest to each other, the distance is 6 km and here the natural water level in the Walawe Ganga exceeds that of the Katupath Oya by about 300m. The scheme uses the 300 m difference in elevation between the Walawe Ganga and the Katupath Oya where these two rivers are only 6 km apart.

23. The Uda Walawe reservoir operating since 1968 with a full storage capacity of 270 million m^3 is located in the downstream reach about 44 km away along the river.

Chapter V

SOME OBSERVATIONS ON THE GEOLOGICAL & HYDROGEOLOGICAL CONDITIONS

REGIONAL GEOLOGY

24. The Samanalawewa Project structures are located within the so called middle peneplain which in the north is confined by the central highlands'main escarpment (the world's end escarpment) and in the south and southeast terminates in the Kaltota-Hapugala escarpment and the vast flat land extending to the south coast of the island. The main escarpment is generally referred as the Balangoda escarpment and the other escarpment as the Kaltota scarp. The hilly surface of the middle peneplain, which rises above the coastal valley as a bench about 300m to 500m, high is dissected by river valleys and numerous ravines. Usually the hills have steep slopes and form ridges extending along main tectonic structures. Hill tops are 150m to 200m above gorge bottoms.

25. While the basin drops from the Central Highlands in a general southerly direction, the feeder streams of the main river have east southeasterly flow directions in the western parts and south southwesterly flow directions in the eastern parts. A significant feature is that while the foothills continue southwards at comparatively high elevations on the western part of the Walawe basin, very flat plains having an elevation of 120m to 150m are found in the eastern part of the basin. Two prominent directions are found in the Kaltota - Hapugala escarpment. These are south to southwest adjoining the Walawe and the east to west along the Katupath Oya.

26. With respect to the project area the main structural element of the first order is the Balangoda syncline. The syncline is characterized by the northwest strike and the asymmetric inclination of the synclinal limbs. The dip of the southwest limb is 35° to 40° and that of the northeast limb is 25° to 30°, the fold bends gently dipping 15° to 20° to the northwest direction. The synclinal limbs are complicated by low order folds and faults. Dislocations or a break in continuity are found in excavations, mappings, seismic surveys, airphotographs, satellite images and drill core chartings. The axis of the syncline plunges west northwest. It runs somewhat parallel

to the upper reaches of the Walawe Ganga and passes through the north of Balangoda. The repetitive east southeast or west northwest directional trend of streams is due to the structural trend of the region. Several stream lines are found to lie in the southwest direction which indicate that strike slip faults or major joints occur in this direction.

27. The Samanalawewa project area lies wholly within precambrian high grade metamorphic rocks of the Highland Series. The rock types consist of the following:

(a) Charnockite, charnokitic gneiss, migmatized biotite charnockite;
(b) Garnet biotite gneiss, granulite, khondalite migmatized garnet biotite gneiss;
(c) Quartzite, biotite gneiss, garnetiferous gneiss;
(d) Marbles, dolomitic marbles, calcsilicate felds, calc granulite, impure crystalline limestone and other "limestone";
(e) Pegmatites and quartz veins - which belong to the Kaltota Formation.

The rock group in the project area has been ascribed to the Kaltota Formation. The Formation is divided into two groups, the Upper and the Lower. The dam and reservoir are located in the core of the synform of in the upper Kaltota group. The intake structure, the tunnel and the power station are located in the lower Kaltota Formation.

GENERAL GEOLOGY OF THE DAM & SADDLES

28. At the dam site these rocks consist of the following units (from the left bank in the north to the saddle area of the right bank in the south):

1. Lower Charnockite layer (CHA) - This unit consists predominantly of charnockite with intercalated of marble and granulitic gneiss.
2. Lower Granulite (GRA 1) - Massive to faintly gneissic granulite, thickness 10 to 25m.
3. Calcareous Bed (CAL) - This unit is composed of marble and charnockite with layer of granulite. Quite frequently the marble beds are contorted

and in places the marble is replaced by a charnockite breccia with carbonatic matrix or by a so called "porous rock" which can be interpreted as the infilling of ancient karstic caverns. In the dam site area this unit occurs at the right bank and is marked by the presence of some important cavities.

4. Upper Granulite (GRA 2) - Granulitic gneiss with relatively frequent hornblende-biotite bands and with carbonate layers. This bed is sometimes contorted.
5. Upper Granulite (GRA 3) - Granulitic gneiss with a few hornblende-biotite bands but with no carbonate interlayers.
6. Upper Granulite (GRA 4)- Granulitic gneiss with frequent hornblende-biotite bands and with marble beds.
7. Charnockite (CHA 2) - Predominantly charnockite alternating with carbonatic rocks.
8. Granulitic Gneisses (GRA 5).

These lithological units are roughly parallel to each other and dip around 30° to the SW. Apart from 2 faults there are some discontinuities which are expressed in the topography as lineaments and which are exposed in some artificial outcrops. The major set of discontinuities is subvertical and strikes SWNE more or less parallel to fault No 1. Wherever these continuities have been observed, they were clearly tensional fractures, frequently open. In addition to this fault No 2 occurs in the saddle 2 area, is subvertical and strikes ESE. The rock in the area to the north of fault No 1 is considerably less fractured than the rock south of it. The project geologists have denominated the more compact zone "Area A" and the more fractured one "Area B".

29. The weathering penetrates to a depth of 60 to 80 meters with some variations attributable to the extent and degree of fracturing and the type of the rock. Mechanical and chemical breakdown of the rock mass can be a sequel to hydrothermal action.

HYDROGEOLOGICAL CONDITIONS

30. Owing to their high degree of metamorphism, all rock types present in the project area are inherently impermeable. However, secondary

processes can lead to an opening up of the rock mass: The marble beds are susceptible to dissolution by surface and underground waters which carry carbon dioxide and this leads to the formation of karstic cavities which can eventually form very extensive cave systems. Caverns several kilometers in length have been encountered. The far less soluble silicate rocks (charnockite, granulite) can be made permeable by intense fracturing associated with faults or by the opening of tensional discontinuities. The more intense fracturing or the presence of tensional cracks facilitate the penetration of the karstic processes into the carbonate layers.

31. In the specific case of the Samanalawewa dam, the Area A, which includes the dam site is little affected by tensional jointing. The rock mass is only a little fractured, the permeability is limited and the karst phenomena are limited to the surface or to very negligible depths. In contrast, Area B is cut by faults and tension joints. The rock mass is intensely fractured and the karstic dissolution penetrates for hundreds of metres. Overall permeability is high.

32. The behaviour of the ground water levels observable in the piezometers of the two areas reflects these different conditions. The levels measured in the piezometers of Area A are relatively high, are unrelated to each other and show little short-period variations. The ones in Area B in contrast are at a lower, uniform level for all piezometers and respond very rapidly to variations in the river levels. After cessation of the river flood, the maximum peaks of the groundwater levels recede rapidly followed by a gradual further descent. However, none of the piezometers in Area B has been seen to drop below elevation 379m, with the exception of borehole GW 17 which has its lowest reading at elevation 378m. From the piezometer readings, groundwater contour maps have been drawn up and these clearly show a flat depression for Area B at about elevation 380m. These same contours confirm the direction of the various "leakage routes" postulated by the project geologists.

33. An attempt has been made to mark the groundwater of Area B with a tracer but no reappearance of this marker has been observed within the wider dam site area. A number of drill holes indicate the presence of two deeper permeable horizons at elevations 300m and 220m respectively. The

piezometric levels in these zones correspond to the ones in Area B.

34. A comparison between atmospheric pressure and groundwater has been done and shows a minor interdependence. The variations of the barometric pressure produce changes of ground water level which are only a fraction of the variation in the river level.

35. Attempts to estimate the possible leakage quantities through the right bank have been made but these may not yield reliable results as they are based on assumptions of a homogeneous porous media, which is in contrast to the highly inhomogeneous karstic conditions which prevail in the "entry" area.

36. The dissimilar behaviour of the water tables in Area A and B indicates the existence of two separate aquifers.

37. The observable immediate response in Area B of all piezometers to increase of the river water level is only possible if high-capacity feeder channels ("express routes") penetrate the area. Karstic cavities along the carbonatic beds within the lithological units GRA 4 and CHA 2 can easily provide the necessary access to all parts of Area B.

38. The rapid reduction of the main peak of the groundwater levels after a flood may be due to a return of the water along the same entry channels, until the groundwater within the aquifer has been reduced to about river level. The further, much slower decrease of the levels in the piezometers indicates that there are no open leakage channels towards the downstream area. This decrease may be due to a seepage through the lithological units GRA 3, GRA 2, CAL 2, GRA 1 and CHA 1 towards the river downstream of the dam. However in this area an impervious sill must exist at an elevation of about 379/378m which prevents the aquifer of area B from descending below elevation 379m (or 378m for GW 17).

39. This area B would form a partially confined aquifer with an open upper entry and a much less permeable bottom exit. The behaviour of the piezometer in the 300m and 220m aquifers indicates that they are confined: They rise with the river level but redescend much more slowly

than the Area B piezometers.

40. The more intensive fracturing of the rockmass to the south of fault 1 (in Area B) has allowed a deep penetration of the karstic dissolution along the layers of carbonate rocks. This, together with the opening of joints in the silicate rocks has created the space for a highly permeable aquifer which communicates directly with the water level in the river. Drainage of the aquifer towards NE (downstream) occurs through a semi-impervious barrier with an impermeable sill at elevation 378m. The two deeper zones of high permeability (at elevations 300m and 220m) appear to be confined aquifers, fed from Area B through the vertical joint system.

41. The remedial work programme has to put first priority on the grouting and obturation of the karstic openings. As these are confined to the layers of carbonate rocks, these latter ones have to be individuated in the adits and grouting has to be directed towards their projections to the grout curtain. Second prority should be given to the grouting of the open joints. As these joints appear to be subvertical, the grout holes will have to be inclined. The inclination and the direction of the groutholes will have to be decided on the basis of the results of the structural survey in the adit. The grouting adit at elevation 390m is 70m below reservoir level. It must be protected against heavy inflows of water and its downstream portal will have to be structured in such a way that at the worst a heavy outflow does not affect the toe of the dam.

42. The rock mass above the adit is to a large degree affected by weathering. In general the weathered rock is less permeable and in places slopewash or eluvial deposits form a blanket of fine grained material which also has a reduced permeability. This weathered mantle will be a good sealing membrane against water leakage. However, nothing is known about the presence of ancient karst systems at levels higher than elevation 380m, which may be hidden by a blanket of soil or slopewash. At impounding, such karst systems could be flooded and would collapse, leading to severe reservoir losses, reactivated erosion and slope instability. Here, too, the most critical areas are the beds of carbonatic rocks.

43. More information was needed about the detailed distribution of carbonate layers and about the detailed structure of the right bank for an assessment of the grouting requirements. The mapping of the grouting adit would answer most of the questions but the extent to which the karst has developed will be shown only by the grout hole drilling

44. Many witnesses spoke of the geological and hydrological conditions of the Project area. Their evidence and the report of Dr. A Baumer Consultant Engineering Geologist, which I have closely followed in this chapter were indeed very helpful and are thankfully acknowledged.

Chapter VI

PREVIOUS OFFICIAL STUDIES

45. This subject is a specific item in the terms of reference.

IRRIGATION DEPARTMENT STUDIES

46. The initial investigation for the Samanalawewa Project was carried out by the Irrigation Department. The Irrigation Department investigation was conducted mainly by the drilling of boreholes for preliminary study. A preliminary report was published in May 1957 followed by a fuller report in September 1957. The Department collaborated in subsequent studies prior to the study by Snowy Mountains Engineering Corporation in 1972.

TUDOR INGINEERING CORPORATION STUDY

47. The next study was by Tudor Engineering Company of Washington DC in response to a request by the Sri Lanka Government. The Tudor Engineering Company issued a reconnaissance report in March 1958 entitled "Hydroelectric and Irrigation Projects - Seven Virgins and Samanalaya". The report advised further engineering surveys to establish feasibility of the Samanalawewa Project.

HYDROTECHNIC CORPORATION, NEW YORK & ENGINEERING CONSULTANTS INC. STUDY

48. As a result of the Tudor Engineering Company report, the Irrigation Department embarked on detailed site surveys and drilling programs throughout the project area. In February 1959 the International Cooperative Administration engaged Hydrotechnic Corporation, New York and Engineering Consultants, Inc., (ECI) Denver, to prepare a feasibility report on the project. From March to December 1959, an American team of engineers and specialists from Hydrotechnic and ECI assisted the Irrigation Department in collecting and analyzing the necessary field data. These investigations were presented in a report entitled "Report on Samanala Wewa Irrigation and Hydroelectric Project - Ceylon", May 1960.

THE PHOTOGRAPHIC SURVEY CORPORATION LTD STUDY

49. Almost alongside with the 1960 report came the report of July 1960 by a team of Canadian consultants from The Photographic Survey Corporation Ltd, Toronto under the Colombo Plan under the heading "A Report on a Reconnaissance Survey of the Resources of the Walawe Ganga Basin." In this latter report eight sites for the Samanalawewa reservoir were discussed but owing to the weight of geological considerations the present site, a modest distance downstream of the confluence of Belihuloya and Walawe Ganga, was favoured. The Canadian study assisted by the Surveyor-General of Ceylon began in 1957. Investigations of the Samanalawewa dam site had already been initiated by the Irrigation Department in the same year. The Irrigation Department team drilled 20 bore holes in 1958.- 1959 and 20 more bore holes in 1964. In addition an observation adit was bored 20m into the right abutment with an open excavation 15m long in front of it. But more work remained to be done.

ENGINEERING CONSULTANTS INC. STUDY

50. In July 1964 the task of preparing a detailed technical report on the Samanalawewa project was entrusted to Engineering Consultants Inc. (ECI), of Denver, Colorado. The exploratory work done by the Irrigation Department was made use of by ECI. The continued assistance of this Department was made available to ECI. In this report four different layouts were considered but the favoured location of the dam was 900 feet (277m) downstream of the confluence of Walawe Ganga and Belihul Oya. Roughly it was the same as the present dam site. The project was considered to be essentially multipurpose providing both power and irrigation benefits.

51. The summary of the ECI drill log records on the geological conditions of the recommended dam site is very pertinent:

"In summarizing the drill log records it can be stated that moderate water losses occurred in the drill holes of the left abutment to depths of 60 feet (18 metres) between ground elevations 1400 ft (426 m) and 1535 ft (467 m) (crest of dam). Negligible water losses occurred at lower elevations on the left abutment below ground elevation 1400 ft (426 m) as well as across the river bed and up the right abutment to elevation 1300 ft (400 m). Holes drilled in the right abutment above elevation 1400 ft (426 m) showed moderate to heavy water losses. Water losses were quite prevalent throughout the right abutment between elevation 1400 (426m) and 1450 (442m). In reviewing the results of exploration drilling done in 1958, it was noted that underground cavities or vugs may exist in the right abutment. This was revealed by the drill rod drops noted on the drill log for Hole No 6N. These rod drops were cause (sic) considerable concern. In order to evaluate the nature and extent of such cavities, additional drilling was undertaken and the exploratory adit was excavated.

"Field investigations, coupled with additional drilling, has (sic) shown that limestone lenses occur throughout the Samanala dam area and are the source of local vugs. The limestone lenses are very irregular and seldom extend for great distances; many of them being continuous for only a distance of 40 feet (12 m). The limestone is a well crystallized calcite, usually containing an abundance of graphite. The crystalline nature of the calcite, together with the graphite inclusions, suggests that the limestone has slowly crystallized at a temperature very near the melting point of the calcite. The limestone shows considerable evidence of solution in those areas where ground water has been able to penetrate the lenses by descent along fractures. As the ground water in the area is very acidic, the calcite is slowly dissolved over a prolonged period. Due to the irregularity and discontinuity of the limestone lenses, widespread solution has not occurred... Based on the investigations carried out, it is believed that the foundation of the dam can be rendered relatively impervious by grouting methods. It is anticipated that the cavities in the right abutment which are local in nature, can be effectively closed while grouting the cut off curtain; however the grout take may be above the average and the depth of the curtain may have to be extended beyond normal requirements. Many of the drill hole logs show a great amount of 'very highly weathered rock.' Field examinations of test pits and the exploratory adit indicate that, although the depth of weathering is generally quite deep, some of the 'very highly weathered rock' represents a somewhat weaker rock than that preserved in drill cores, but certainly not a completely weathered zone".

52. The ECI study began in July 1964 and their report was

presented in May 1966. The river bed level is around 368 m. The ECI report recorded that up to 1300 feet (369 m) the water losses are negligible on the right bank but heavy water losses were recorded between 426m to 442m. The ECI had done drilling along the proposed dam axis, in the neighbourhood of it and on the right bank. The ECI had done 40 bore holes in the dam area, 4 bore holes in Saddle 1 (610m from the dam) and 6 bore holes in Saddle 2 (1220m from the dam). They had investigated these saddles for an ungated spillway of the reservoir. The maximum depth of the drilling in saddle 1 was 39 metres and the total depth drilled was 135 m. They went to a depth of 72 m in saddle 2 and a total cumulative depth of 369 m. Yet they did not find sound rock. The ECI found a considerable amount of cavities attributable to surface water in circulation in fractured zones. The ECI bore holes did not go deep enough to get to the limestone bands. They had not ascertained how deeply cavitated the limestone band was and how deeply connected it was with the reservoir. The ECI had done permeability tests on the ground surface in the weathered mantle within the dam area. The tests were to see how pervious the top weathered mantle was and the type of soil. Topographically a dam located in the present dam site area was the best option. The ECI proposal was for a dam on the Walawe with a connecting tunnel to a second reservoir created by a dam on Diyawini Oya. A deep shaft from the Diyawini dam would take the water to a power station sited on the ground surface just by the side of the hill.

THE SNOWY MOUNTAINS ENGINEERING CORPORATION STUDY

53. The next study was in 1972 by Snowy Mountains Engineering Corporation (SMEC) in conjunction with the Engineering Studies Organization, Mahaweli Board, Sri Lanka. The geological findings accord with those of the earlier reports. The SMEC report issued in August 1973 states that the Samanalawewa Project lies in the Southern foot-hills of the Central Highlands which rise to elevation above 550 m in the vicinity of the Project area. While these foothills continue southwards at comparatively high elevations for many miles on the western part of the Walawe basin, very flat plains having elevations of 120 m to 150 m are found in the eastern part of this basin. All major features of the Samanala Project were concentrated in the Upland Platform region with ESE directional streams. The same ESE directions were maintained at the Diyawini where ECI proposed a dam and also near the

Power House site on the left bank of the Katupath Oya. SMEC confirm that the most prominent structural feature in this area is a syncline with its axis plunging WNW. Although no direct evidence of faulting had been observed in the field, air photographs indicated signs of faulting or jointing in the south-west directon, with minor changes of direction to SSE and ENE. Since the rock types were all metamorphic and belonged to the pre-Cambrian age, field evidence was not readily obtainable for proving any faulting. However, core drilling in some of the stream valleys following strike directions indicated highly weathered conditions at depths of 60m to 90m while fresh rocks had been penetrated at higher elevations. Further in certain exceptional cases gouge material or breccia had been found. In a few drill holes water had commenced flowing out of the holes after penetration over 60m. It was possible that what were shown as faults in air photographs were only joints. On the rock types SMEC findings are that they are metamorphosed sediments and were occasional bodies of charnockite. Pegmatite veins were found occasionally. Quite competent gneisses and related rocks were seen in the Walawe river bed. The dolomite limestones were important for consideration because of their inherent weakness in having karsts and solution cavities. The bands were 12m to 13m in thickness or smaller and were found outcropping in the dam site and needed proper treatment by cut-offs and grouting. Though rock was seen in the Walawe Ganga itself, the valleys and hill slopes showed signs of shallow to deep weathering. The hill slopes contained remnants of very deep weathering. Core drilling had shown that the left bank area was satisfactory for the construction of an earth, rockfill or concrete dam. But the right area had certain problems arising from the karstic nature of the metamorphic limestones present. The saddles on the right bank were narrow and one of these had a steep downstream slope. The saddles were generally at topographic elevations 15m to 46m above the reservoir. The major part of the drilling had been done during 1959/60 and 1965/1966. SMEC did three boreholes on the dam site of which the aggregate was 184m and the deepest was 77m. One borehole was done on Saddle 4 to a depth of 62 m 2286 m from the dam. The SMEC work was for ascertaining weaknesses in the limestone and feldspathic gneisses in the dam area and saddles. The dam site was bounded on the west by the confluence of Belihul Oya with the Walawe Ganga and on the east by the point of entry of the Methihakka Ara.

54. SMEC found about 4 to 5 dry streams in each bank spaced at about 90 m to 120 m and notably the streams on one bank lay directly opposite the streams on the other. About 3 or 4 small springs were noted in the upstream zone of the right bank but the discharges were negligible. The bedrock in the river bed consisted of basic biotite gneisses and charnockite gneisses. The upper part of both banks contained garnetiferous granulitic gneisses which were often feldspathic and schistose in character. Some quartzitic bands and pegmatites were also noted in the drill cores. Dolomitic limestones and calcareous gneisses were found interbedded with basic and charnockitic gneisses in thin bands. On the left bank, several outcrops of limestone were seen. On the right bank only one area of outcrop of limestone was seen. The presence of limestones was however revealed from a study of the drill cores in different cross sections. Drilling investigations had shown complete loss of water, lack of core, easy rotation of drill rods by hand and cavities.

55. Samanalawewa would be confined within the narrow valleys of the Belihul Oya and Walawe Ganga. Compared to the Belihul Oya, the Ganga valley slopes were more steep. In several sections of the Ganga between the dam and the pressure tunnel inlet, the slopes approximate to 1 on 1. The reservoir would impound 77m to 92m feet of water within these slopes and though slides have occurred in the region at different times, no evidence of major slides had been seen in the proposed reservoir area.

. Some narrow saddles occurred in the peripheral areas, on both left and right banks of the reservoir. These saddles were generally at topographic elevations 15m to 46m feet above the reservoir. The saddles on the right bank were narrow and one of these had steep downstream slopes but the saddles on the left bank were wider at reservoir levels.

EC1 drilling investigations had been done in the saddles 615m and 1230m from the right abutment of the dam with a view to use of these saddles as possible spillways. However core recovery was unsatisfactory even at depths of 62m. Complete loss of drilling water had also occurred in some of the drill holes at depths of 21.5m, 28.3m and 31m. In the saddle 2300m from the right bank of the dam, a drill hole S1 had been done from ground elevation 500 EL to 436 EL. This hole had been cased at 17.4m (482 EL) and the core recovery was unsatisfactory throughout. Complete loss of drilling water had occurred at 59m (440 EL).

Arrangements had been made to make a continuing study of water table fluctuations in the drill holes discussed above. It was felt that the saddles would not affect the feasibility of the proposed reservoir. The results of these studies would enable decisions to be made regarding any grouting work which might become necessary to avoid leakage through the saddles.

The grouting programme would have to be extensive in certain sections of the right bank and possibly time consuming. Water Pressure Tests done in the investigation drill holes indicated the zones that would require concentrations of grout holes. Additional holes were required where loss of grout was considerable. SMEC advised that better core recoveries could be obtained by using triple tube core barrels in the weathered rocks where double tube core drilling had not given adequate core recoveries. Continued studies of seasonal groundwater table fluctuations at the dam site and reservoir periphery saddles were recommended. The dry weather flow of the river occurred through gullies formed between the outcrops and large boulders. SMEC treated the project as a single purpose power project but gave some consideration to irrigation benefits too. The layout as envisaged in the SMEC report differed from what had been recommended earlier.

56. The main reservoir and dam were similar in concept to the previous recommendations. The dam height was however reduced and a chute spillway was incorporated at the same site as the dam. The tunnel dropped relatively steeply from the intake to become a 3.8m diameter, high pressure, concrete lined tunnel for most of its length but towards the downstream end it was steel lined and the diameter was reduced to 3.00m; and at this point it was about turbine elevation and eliminated the surface penstock. The Diyawini reservoir was retained and connected to the tunnel by an inclined surge shaft. The steel tunnel liner became a concrete encased steel penstock with a bifurcation to two 1.8m diameter distributor pipes supplying two 60MW turbines in a surface powerhouse. The tailrace channel discharged to the Katupath Oya and eventually fed the existing Uda Walawe reservoir. The streamflow series used in the reservoir operation studies were derived from estimated basin rainfalls over the Samanala and Diyawini basins for the 38

year period 1934-1971. Details were given of reservoir operation studies carried out for the final recommended layout. The SMEC report was very comprehensive and a good feasibility study.

THE RUSSIAN TECHNO-PROM-EXPORT STUDY

57. In September 1975 the Ministry of Irrigation, Power and Highways commissioned the Hydroproject Institute, Ministry of Power and Electrification USSR "Technopromexport" to prepare a detailed Project Report for the Samanalawewa Project on the Walawe Ganga. The Central Engineering Consultancy Bureau (CECB) collaborated with the Russians and were responsible for a report on "Climate, Hydrology & Water Management" issued as Volume 1 of the Detailed Technical Report issued in 1978.

58. The climate in this area is recognised as sub-equatorial. Wet equatorial air conditions exist during both northeast and southwest monsoonal seasons. The mean monthly air temperature fluctuated between 26C° and 28C°. Air humidity was present throughout the entire year. The long-term annual rainfall was 2920 mm for the Samanalawewa catchment area. The maximum rainfall occurred in March-May and October-December and the minimum in July-September. The winds of southwest direction (April-October) and northeast direction (December-March) prevailed at the site. Maximum design velocity of wind was equal to 27 m/sec for all directions. The evaporation value from the water surface of the reservoir for a mean wet year was 1073 mm. In examining the hydrology, measurements of the run-off and streamflow were studied. Streamflow gauging had been conducted 460m downstream of the proposed dam site and measurements were available from January 1959 to September 1967 and from January 1973 to 1974. Values for the run-off for the period 1934 to 1971 were obtained by using an improved method of computation. The CECB took into consideration the importance of the project on the national economy, the vulnerability of the high zoned earth and rock-fill dam if overtopped and the safety of the Uda Walawe complex downstream and determined that the design should be able to cope with a maximum probable flood (MPF). Since a 10,000 year flood approaches the maximum probable flood, the 10,000 year flood was considered the design flood. Such a flood would have a peak of 3600 m^3/sec. The largest rainfall increment was calculated as likely to occur 12 hours after the commencement of the

storm. Flood hydrographs at the Samanalawewa dam site were plotted using the available data to work out a unit hydrograph. The hydrograph of the 10,000, 1000 and 100 year flood were plotted.

59. The downstream Uda Walawe reservoir was constructed for re-regulation of the river flow to irrigate 24.0 thousand hectares. The Samanalawewa reservoir did not carry any special flow regulation for irrigational needs. According to the CECB study, the construction of Samanalawewa would significantly improve the regulation of the Uda Walawe reservoir and thereby the irrigation under the Uda Walawe Project. Owing to this improvement, it would be possible to extend the cultivation under the Uda Walawe reservoir by an additional 32,500 hectares. From the Samanalawewa reservoir the irrigation needs of 8000 hectares could be met. The average annual addition of firm energy to the national power grid would be increased by 630 million kilowatt hours.

60. The Russians in their detailed project report considered the geological structure under four heads :

(i) Stratigraphy and Lithology;

(ii) Tectonics;

(iii) Hydrogeological conditions;

(iv) Physical and geological Features.

61. Under head (i) the Russian project report identified the Samanalawewa Project structure as being located within the middle peneplain which in the north was confined by the Central Highland main escarpment and in the South and South-West by the vast valley extending to the Southern coast of the island. This middle peneplain was composed of metamorphic rock of the pre-Cambrian crystalline complex. The complex of rocks was primarily of sedimentary origin and had undergone well defined metamorphism. This complex of rocks can be referred to as the Kaltota Formation which in this condition comprises two members: the upper member and lower member. The upper member consists of rocks composing the synclinal fold in the central part of the peneplain and exposed in the beds of the Walawe Ganga and its tributary the Belihul Oya. The lower member consists of rocks located along the periphery of the big syncline in the south and north-east parts of the

peneplain and sometimes exposed in the south escarpment and in thalwegs of rivers and springs. The rock benches in the Kaltota Formation in accordance with the ~~pre~~dominance of lithologic types were -

(a) benches of migmatized and biotite charnockites and charnockitic gneiss intercalated with garnetiferous biotite gneiss and limestone bands;
(b) benches of migmatized, garnetiferous biotite gneiss intercalated with charnockitic gneiss bands and limestone bands;
(c) benches of quartzite alternating with biotite and garnetiferous gneiss;
(d) benches of marbleized limestones, dolomites and calcite gneiss.

62. The rocks of both members were similar except that marbleized limestones were more predominant in the lower member. The limestone bands ran thick and thin. Erosion and weathering in the upland prevailed over accumulative processes. In the weathered crust three zones were distinguishable:

(a) zone of rocks completely or considerably disturbed by weathering:
(b) zone of clumpy eluvium;
(c) zone of slightly weathered heavily fissured rocks.

The thickness of weathered zones (a) and (b) was 30-40 m and the thickness of the slightly weathered rock in zone (c) reached 100 m near heavy tectonic displacements. Beyond the tectonic zones the thickness varied from 2-3 m to 40-50 m. On some of the highly eroded sections the crust of weathering was totally or partially absent. Talus deposits were widespread and formed mainly owing to erosion and redeposits of weathered rock materials. They almost continuously covered the bedrocks. The thickness of the talus mantle reached 4-5 m. Only the steep slopes and ravine thalwegs were without talus deposits. The Walawe Ganga and its tributary beds had practically no alluvium deposits owing to the steep gradients.

63. Under the head of tectonics, the Russian report picks the main structural element as the broad syncline described as the Balangoda syncline where the project structures are located. It was characterized by the North-West strike and a little asymmetric inclination of synclinal limbs - the dip angle of the South-West limb being 35-40° and that of the North-East limb 25-30°. The fold bend trended in a gentle dip 15-20° in the North-West direction. The synclinal limbs were complicated by low order

folds and dislocations with a break in continuity. Along the dislocations there were almost vertical displacements and in rare cases - shifts. Among the most probable displacements confirmed by drilling were the fault of the sublatitudinal strike on the left bank of the Diyawini Oya, at the dam site where the bench of marbleized limestones was down faulted and the transversal fault of the submeridianal strike causing the sharp turn of the Diyawini Oya at the Diyawini dam near the saddle on the escarpment (Kaltota scarp). Tectonic fractures with openings of 2-10 cm were widespread in this area.

64. The Samanalawewa Project area could be identified as a zone of low seismic activity.

65. On the subject of the hydrogeological conditions attention was drawn to the fact that the rocks occurring in the Project area can be subdivided into two complexes so far as water permeability is concerned: "porous" rocks and fissured rocks. The eluvium of the crust of weathering, talus mantle and alluvium deposits were referable to the first complex. The second complex comprised metamorphic Pre-Cambrian bed rocks. From here the water regime is best described in the words of the Russian report:

66. "The pore and fracture waters are hydraulically connected, have similar chemical composition and form a single water-bearing horizon with impermeable layer formed of preserved slightly fissured metamorphic rocks. The ground water penetrates into the massif to the depth of 200-300m in the zones of intensive fissuring along displacement and separate layers and benches of highly fissured quartzites. In such cases the ground water is of stratal-fissure pressure nature. The artesian flow at 32 l/min yield took place from the depth of 74.7 m from quartzites in CT-6 borehole drilled at Watawala village.

"The ground water is replenished by rainfall. In the rainy period the ground water stage rises, but this rising does not last long because of high permeability of cover deposits where the water is drained by rivers and deep ravines. Sometimes ground water escapes on steep slopes in the form of springs with low yield. The yield of big springs on the right-bank slope of the Walawe Ganga River valley reaches 0.8-1.0 l/sec. The major part of springs is of seasonal character and in dry period they dry out.

"The permeability tests showed that the preserved metamorphic rocks below the zone of weathering are practically impermeable. The specific water absorption for these zones changes from zero to a hundredth fraction of a litre per minute. The quartzites with dense system of microfissures which makes then highly permeable and watery, are the exception. The permeability coefficients of quartzites are 2-3 m/day.

"The maximum water permeability and high water content are observed on the stretches of intensive fissuring in the zones of displacement. The drilling water was completely absorbed. The consumption being 70-80 l/min, when bore-holes were drilled on such stretches. In the zone of weathering of bed rocks the ground water is circulating in the dense system of fissures. The direction of ground water flow is predetermined by the relief and the roof gradient in reference to weakly permeable rocks. In low parts and in the zones of deep weathering fissured rocks are water logged. On the upland sections and in slopes the water content of the zone depends on the number and intensity of rains. The springs with low yield located in the zone of weathered rocks dry out in the dry period.

"The water permeability of weathered fissured bed rocks varies within wide range depending on the degree of karstification, fissuring and filler nature. The specific water absorption varies here from 0.01 to 50.0 l/min.

"Eluvial formations of the crust of weathering are rather heterogeneous. They are very porous and water permeable. According to the data on pouring into holes and bore-holes the permeability coefficients vary from 0.01 to 16.0 m/day depending on the density and texture of soil. The eluvium of the crust of weathering is watered in low parts. Numerous wells drilled in eluvium are used for drinking water supply for the local population. The high water content in the eluvium deposits is closely connected with the rainfall. In the period of heavy stormy rains the ground water level rises and drops rapidly.

"Talus deposits are usually without water. They are watered only in the period of rains. The permeability of talus deposits is characterized by the permeability coefficients from 0.17 to 2.55 m/day".

67. Karst, bottom and lateral erosion, weathering and rock gravity displacements on slopes were characteristics of the physical and geological features of the project area. The Russians felt that "the most important feature" in the evaluation of the engineering and geological conditions of the project area was karst developed owing to the wet tropical climate. Their analysis is worth repetition:

"The karsted rocks are Pre-Cambrian marbleized limestones and widely spread dolomites. Karst processes are genetically associated with rock fissuring and drainage in river valleys, deep ravines and zones of tectonic disturbances.

" Bore-holes and excavations made in the area of hydraulic structures of the Project discovered that karst is usually located in the river valley slopes in the zone of bed rock weathering. The local base level of erosion is the border of the karst process to the depth. At a greater depth the karst process developed in the zones of tectonic disturbances drained by the remote erosion cuts. The dimensions of solution cavities depend on the thickness of karsted rocks. The solution cavities the volume of which is several thousand cubic metres were found in the thick benches of marbleized limestone in the left slopes of the Diyawini River and in the right slope of the Walawe Ganga not far from the Belihul Oya River falling.

" In separate layers of marbleized limestones interbedded with karst free rocks, there are small solution cavities located along bedding fissures. Solution cavities are of various forms: from solution channels extending along fissures to isometric karst caves. Empty solution cavities as well as solution cavities filled completely or partially with sandy-clayey and fragmental material are encountered in the area under study.

" Karst was not found in the zones of preserved impermeable rocks in marbleized limestones and dolomites".

68. There was erosion in the river bed and springs and on steep slopes. In the process of bed rock weathering, chemical weathering was predominant. So also the tectonic jointing of rock contributed to the intensity and depth of the weathering process. The petrographic composition

of the bedrocks had a bearing on the rate of weathering. Gneisses, garnetiferous gneisses and granulites were liable to weathering but charnockites composing the positive topographic forms were less so. The gravity displacements appeared in the form of landslides on the steep slopes of the upland and gorge slopes. These landslides usually affected the surface talus and eluvial formations to the depths of not more 2-3 m. The occurrence of big landslides had not been observed. However big dislodgements rolling down from the escarpment could pose dangers to the structures located at its foot.

69. The Russian report also carries a detailed analysis of the Engineering and Geological conditions of the dam site, the dam and the other structures of the project.

70. The dam site was in a monoclinal V- shaped gorge with somewhat asymmetric slopes in the Walawe Ganga valley. In steepness the right-bank slope was 28-32° and the left bank slope was 23 to 26°. The slopes on both banks were cut by shallow ravines which carried water only during heavy rains. The slopes were covered with jungles but in some places were sparsely wooded. The dam site was composed of pre-Cambrian metamorphic rock of the upper member of the Kaltota Formation. The following four benches were identified going from top to bottom and in mapping the short nomenclatures indicated against them have later been given:

(i) The 160-170 m thick upper granulitic bench represented by interbedding granulites, garnetiferous and biotite gneisses - GRA 4 & GRA 3 on the basis of the stratigraphy;

(ii) the 40-70 m thick upper carbon - bearing bench represented by interbedding biotite and garnetiferous gneisses,charnockites and mamorized limestones -CAL and GRA 2 on the basis of stratigraphy;

(iii) the 20-25 m thick lower granulitic bench represented by granulites - GRA 1 ;

(iv) the more than 150 m thick lower-carbon - bearing bench interbedded by marmorized limestones, charnockites, biotite and garnetiferous gneisses - CHA

All lithological varieties of the bedrock of the dam site in preserved state were slightly jointed, hard and stable. A crust of weathering was widely developed on the metamorphic rock and increased in thickness as it went up the slope. It was practically absent in the river channel. Eluvium was thickest on the right bank slope where it reached 24-25 m. On the left bank eluvium did not exceed 12 m. Talus deposits varying in thickness from a fraction of a metre to 3-4 m represented by heavy loam containing fragments of weathered rock up to 10% were observed on the surface. Alluvial river channel deposits 1.5-2.0 m thick were found as accumulations of boulders and pebbles in small depressions of the original river bed. The dam site was located on the north-eastern slope of the Balangoda syncline where complicated folding of lower orders occurred. On the left-bank slope the rock was characterised by a south-western dip at an angle of 30-35° and on the right bank at an angle of 35-45°. In the river channel part the angles of the dip were more gentle (5-10°). At the foot of the right-bank slope a north-eastern dip of the beds and a small fold of lower order with its axis going along the right-bank slope nearly parallel to the river were observed. There was also to be seen a number of small steeply dipping tectonic dislocations with almost vertical displacement of rock up to 1.2-2.0 m. The thickness of the slackened rock within the zones of tectonic dislocations did not generally exceed 3-5 m but rarely the thickness was 10 m. Ground water in this area was confined to the upper eluvial and slightly weathered jointed zone. Groundwater circulating on the preserved practically impermeable rock was drained by the river. Availability of water in the rock in the zone of weathering depended on rainfall. Tests showed that permeability in the preserved bedrock, including limestones within the zone of karsted rock was zero or nearly zero. High permeability however was observed in the zone of weakened rock along the tectonic dislocations. Lowering of the ground water stage in the downstream directions on the right bank was attributable to the draining effect of the zone of tectonic dislocations of the north-eastern strike. Marmorized limestones of the upper and lower carbon-bearing benches within the zone of weathering were karsted - the more so on the right bank slope. Solution cavities were hollow in the right bank slope or sometimes filled with sand and clay material. A large solution cavity was found on the right bank of the Walawe Ganga in the upstream pool.

71. The dam itself was planned to be rock and earthfill. The

height was to be 97 m, length along the crest 500 m and width at the bottom in the river channel 465 m. At the dam foundation, the gorge slopes on the surface were composed of talus and eluvial deposits of a total thickness of 15 m. As these deposits were of low shearing strength and high permeability they had to be removed. Block eluvium and slightly jointed bedrock would serve as dam foundation in the abutments. In the channel portions, the dam foundation would be in slightly jointed hard rock. As in the upper part of the right bank slope the thickness of highly weathered eluvial rock was 24-25 m, the design provided for the slope to be made gentle with partial removal of eluvium so as to provide against the possibility of breaking and sliding on the upstream slope. A grout curtain would be put down to impermeable rock to eliminate seepage in the dam foundation and wash out of filler from the joints and solution cavities caused by piping.

72. The Russian report on the reservoir itself deserves to be reproduced in full :

"The seasonal storage reservoir of 210 million m^3 capacity at the Normal HWL of 460 m will extend along the Walawe Ganga canyon for 11 km and along the Belihul Oya canyon for 5.5 km.

" In the widest portion its width does not exceed 1.0-1.2 km. The river valleys within the reservoir are V- shaped gorges with steep (20-35°) slopes cut by numerous ravines. Slopes are covered by jungles and sometimes with sparse growth of trees.

" Areas of cultivated land are located along the Belihul Oya River. There are no large villages or structures in the zone of flooding.

"The reservoir slopes are composed of metamorphic rocks of the Pre-Cambrian Kaltot Formation upper member ...... represented by interbedding benches of charnockites, charnockite, biotite and garnetiferous gneisses with subordinate layers of granulites, khondalites and marbleized limestones. In river channel bed rocks are often exposed, except for the areas having small gradient where there are thin alluvial deposits.

" On the gorge slopes on metamorphic rocks, there is a crust of weathering the thickness of which increases upward along the slope. It reaches the maximum thickness (up to 25 m) on the right-bank slope which joins the dam. Upstream, in the reservoir area at the Normal HWL the thickness of the zone of weathering decreases to 5-10 m. The talus cover to 4 m thick is developed everywhere at the surface.

"The gorge slopes are sufficiently stable and gravity displacemnts were not observed, but after the creation of the reservoir the highly weathered rock and talus deposits will be water logged and small landslides can develop on steep slopes. The reservoir banks will not be considerably transformed.

"The ground water in the reservoir slopes is drained by the river which is confirmed by the presence of springs discharging on the slopes on both banks of the Walawe Ganga and Belihul Oya Rivers.

"Taking into consideration the relief, geological structure and hydrogeological conditions it is believed that the seepage losses from the Samanalawewa reservoir will take place only in the dam foundation, past its wings and in the right slope in saddle No 1, 1300 m from the dam. By tentative calculations the seepage losses in the dam foundation, and past its wings without cut off measures will be 0.15 m^3/sec. The grout curtain in the dam foundation will decrease considerably the seepage losses.

"At the saddle No 1 where bed rocks are deeply weathered the seepage can take place from the reservoir to the ravine at the downstream side. The calculations showed that at this reach 1.1 km long the water losses can amount to 50 l/sec. The considerable length of seepage paths and correspondingly small exit gradients allow one to think that the piping is not expected".

73. The conclusions of the Russian investigation are given as follows:

"(1). The area of the Project under design is composed of metamorphic Pre-Cambrian rocks - various kinds of gneisses, granulites, charnockites, quartzites, marbleized limestones and dolomites. The carbonaceous varieties of rocks are affected by karst. The karst is confined to the zone of weathered bed rocks, the karst base level is the recent river water edge.

The rocks are displaced and dissected by numerous steep dislocations of various degree of shift.

(2). The crust of weathering, the thickness of which varies within a wide range and sometimes exceeds 100 m, is developed on the said territory. Three zones can be distinguished in the crust of weathering depending on the degree of their preservation. The upper eluvium zone up to 25-30 m thick is characterized by relatively low parameters: ....

(3). The underground water is fresh, forming the single horizon confined to the weathered zone of bed rocks. The preserved bed rocks are practically impervious and without water. The ground water has slight carbon dioxide leaching and general acidity action on the concrete.

(4). There are no considerable gravity displacements on the slopes, after the creation of reservoirs falls - landslides may take place on steep slopes.

(5). The design provides for the removal of the eluvial rocks from the Samanalawewa dam foundation, as they are characterized by the low shear strength and the high permeability coefficient (from 3 to 15 m/day). The underlying preserved and slightly weathered bed rocks and block eluvium can be a safe foundation for the adopted type of dam. The grout curtain in the dam foundation cutting off the karst-affected upper carbonaceous bench to the river elevations will eliminate the concentrated seepage to the downstream pool and minimize the seepage losses.

(6). It is planned to remove the filler of solution cavities in marbleized limestones from the foundation of the Diyawini dam. The grout curtain in the bench of karsted limestones will be extended to its foot enveloping the zone of revealed dislocation.

(7). The pressure tunnel will mainly pass in hard sufficiently rigid metamorphic rocks and will cross some dislocations with zones of crushed and milonitized rocks up to 30m thick requiring the strengthening of the tunnel arch lining. In such places the concentrated water inflow with discharge of 50-100 l/sec is possible. The total water inflow along the whole route of the tunnel is expected to be not more than 0.6 m^3/sec.

(8). The engineering and geological conditions of intake structures, power house and tailrace raise no doubts.

(9). The seepage losses in the banks of the Samanalawewa and Diyawini reservoirs are inconsiderable.

(10). The local building materials are available for construction of the dam core and shells. The materials for filters and concrete aggregate are obtained by the crushing of stone as there are no sandy pebble rocks at site.

(11). As far as the engineering and geological conditions are concerned the construction of the Samanalawewa complex is possible at the considered site".

74. The layout adopted by the Russians with whom the Central Engineering Consultancy Bureau (CECB) collaborated, incorporated various field work and topographical and geological surveys and developed further the design and construction details. They did over 7000m of core drilling together with driving of adits, test pitting, exploratory trenches, pumping tests, seismic profiling and laboratory tests. Four bore holes were drilled in saddle 2. Leaving the main reservoir and dam unchanged the Russians opted for a crest level raised by 2m to give added dead storage in the reservoir. They reverted the headrace tunnel to the low pressure configuration but Diyawini was retained as an upper expansion tank on the surge shaft. A surface penstock of 2.8m diameter constructed of reinforced concrete with a steel liner was indicated. The surface powerhouse and tailrace channel discharging to the Katupath Oya were retained. A cost-benefit analysis was also included. Extensive work had been carried out on the dam site. Fifty-five bore holes totalling 3212m were done at the dam site. The maximum depth drilled was 200m. The drillings included pumping in and some pumping out by way of permeability tests. Adit driving on the right and left banks

estimated that the seepage losses past the wings without cut off measures would be 150 l/s. Limestone was identified in the dam area going thick and thin. Perhaps I should also add that a major solution cavern of dimensions 37x6x10 m^3 = 2220m^3 was taken note of by the Russians. This was upstream of the confluence on the right bank of the Walawe. The cavern was well known even to the villagers and its origins and history are steeped in ancient legend and colourful homespun myth.

75. The Hydroproject Institute made the following recommendations:

"(1) Finalize the topographic survey scale 1:500 at the Samanalawewa dam site, determine a number of approach triangulation points at the shaft site, inlet and outlet portals of the pressure tunnel and interconnect all major features of the project by levelling of the 3 order.

(2) Finalize elevations of the penstock trace basing on the findings of additional field geological investigations.

(3) Carry out experimental excavation of decayed quartzites in the vicinity of the pressure tunnel inlet portal with a view of evaluating its suitability for the dam transition zones and concrete aggregates. Positive results of this experiment could reduce the quantity of sand and crushed stone to be manufactured from hard rock.

(4) In execution of earth and rock moving operations on the cofferdam of the Samanalawewa dam, experiments should be conducted to evaluate geotechnical properties of rock fill and eluvial sandy loam to be used in the cofferdam blanket. It might be expected that for instance, shear properties of the rock fill material determined by experiments would be higher than those adopted in the design. Basing on supplemental calculations it will permit steepness of the dam slopes to be increased and the volume of embankment to be cut down. These experiments can be also used to perfect the procedure of work execution in filling the impervious features of the dam.

(5) Analyze stability of the existing slopes in the cuts (roads, foundation pits etc.) in the project area that were made in the upper eluvium-

bearing bed rock subjected to atmospheric exposure of various duration. It is also recommended to analyze intensity of natural vegetative cover growth at the slopes.

The design bearing capacity of the eluvial material permits the steeper slopes to be adopted for the foundation pits and dry excavations than those accepted in the design. But to ensure stability of the slopes subjected to the atmospheric exposure, the slopes are flattened out as compared to the calculation results.

The above analysis may enable additional recommendations to be worked out on increase in the steepness of both temporary and permanent (above water) slopes, which results in reduction of soft soil excavations.

(6) Analyze the intensity and extent of bank transformation at the existing reservoirs located in similar conditions (geology, wave action, shore wood cover etc.). These data will permit additional assessment to be made of dead storage sedimentation by bank washouts and if required, to work out the necessary recommendations.

(7) When the reservoir is filled up to the normal head water level, the inflow in excess of the discharge capacity of the hydroelectric station (45 m^3/s), will be spilled by passing the turbines. It is recommended to calculate daily intervals (for the period supported by the records of mean daily discharges) to determine the quantity of waste water spill, finalize the design firm and long-term average power generation. It seems that provisions can be made in the operating rules of the hydroelectric station for filling the reservoir somewhat below the normal head water level to have a spare storage to accumulate the short-period flood inflows and then use them for power generation.

(8) In critically dry conditions beyond the design firm energy yield, the deficiency will reach 60% of the required power and generation.

It seems expedient to work out the rule curve for operation of the reservoir calling for timely limitation of the energy yield (by 20-25%) in case of a deep (but not full) draw down of the reservoir.

Under such operating conditions, the number of deficiency periods will increase but necessity of limiting electric power users during deficiency periods will be eliminated.

(9) Power regulations of the Walawe Ganga flow with the aid of the Samanalawewa reservoir can change inflow to the Uda Walawe reservoir and releases from this reservoir for irrigation purposes.

(10) Reject the spare section (bay) of the overflow spillway at the types and constructions of the outlet structures adopted for the Samanalawewa dam in the design.

(11) Finalize the geotechnical properties of rock in the course of pressure tunnel excavation, and basing on the secured results, finalize constructions of the tunnel lining, with the calculations covering in more detail all elements including timbering (temporary lining).

(12) Establish a network of piezometric holes to check seepage flow from the reservoir through a depression in the right shore.

The results of these observations help to decide on necessity of cut off actions in this depression.

(13) At the beginning of grouting operations on the major features of the project, pilot grout curtain sections should be set up to finalize quantities of the forthcoming grouting basing on the special programme of experimental work. It is felt that the full quantities taken in the project report with 20% reserve, may be somewhat reduced.

(14) Study additionally the problem of probability of the design construction period discharge adopted for the Samanalawewa dam bearing in mind that regulatory documents in force in the USSR allow the diversion structures in similar conditions to be designed for a flow of 10% probability."

76. In their recommendation No 12 the Russians recommended a network of piezometers to be installed to check seepage flow from the reservoir through a depression in the right shore.

77. Prof. Karl Evert in his work on "Rock Grouting with Emphasis on Dam Sites" published in 1985 points out that the investigation programme of the Hydrogeological Scope should begin with a study of the following complexities:

"The basic question will be whether there is another valley in the neighbourhood of the reservoir area. It could drain off water from the reservoir if there were any permeable rock zones in between functioning as a hydraulic connection if the original groundwater were below the intended reservoir level....This complexity cannot always be cleared by geological mapping alone, which is particularly true of the position of the groundwater table. Whenever the groundwater seems to fluctuate below the reservoir level, piezometers should be installed". Prof. Evert further explains (p 405) that "the formation of water carrying openings along joints and other types of discontinuities is a long-lasting process. Once a certain network of paths has developed the direction of the groundwater flow cannot change easily and it still maintains its influence even under the conditions of an impounded reservoir". Hence the importance of the original hydrogeological regimes and the advisability to adapt the treatment of the underground to this situation. Investments can be considerably economized if this relationship is analysed and taken into account beforehand. Having reasoned so Prof. Evert concludes: (p 405) "This calls for the early installation of a sufficient number of piezometers already in the phase of the preliminary investigation. It is highly advisable to measure the groundwater table and its precipitation - dependent fluctuations over a longer period including at least one dry and one rainy season. It is also important to provide for a good graphical representation of the readings, maps of groundwater contours and piezometer hydrographs should be plotted. The interpretation of maps and hydrographs permits conclusions concerning:

- average permeability of the rock;
- extension of an eventual grout curtain;
- sections which can possibly be excepted from the treatment;
- existence of only one or more groundwater regimes also including sections with perched water;
- natural reaction of the groundwater to the precipitation (particularly important to interpret groundwater behaviour during impoundment)".

Hydrogeological investigations with their due presentation were of primary importance to make a reliable judgement of permeability, scope of sealing measures and treatment. The situation which would predicate the need for piezometric monitoring existed at Samanalawewa exactly as adumbrated by Prof. Evert. There were two valleys in the neighbourhood of the reservoir area with permeable rock zones, solution cavities, karst and limestone bands functioning as a hydraulic connection which could drain off the water of the reservoir. These were the Mathihakka Ara valley and the Kalunaide Ara valley. Here was a textbook case which cried aloud for investigation by piezometric monitoring even at the stage of feasibility studies. Sadly enough none of the world renowned engineers and engineering geologists who did successive studies did an adequate piezometric monitoring survey of the right bank which should have figured high among the priorities because of the bearing the results may have had on the Project concept itself.

78. The Russians bored four drill holes on Saddle 2 but the core recoveries are now not available. No mention is made of suspected karst and limestone in the saddles. Recommendation No 12 made by the Russians is however significient. A network of piezometric holes in the depression on the right shore is recommended. There was disagreement among the witnesses as to the identity of the particular depression. One view was that the reference is to the depression shown in the groundwater contour map of the right toe of the dam and the other view was that the reference is to the geomorphological depression in the saddle on the right abutment downstream of the dam. Though this cannot be said of most of the other recommendations, recommendation No 12 is clearly a reference to work to be done in the future for designing the cut-off works if necessary. The Russians had already installed a network of piezometers in the depression at the right toe of the dam in the area shown in the groundwater contour map. This depression therefore cannot be the one referred to in the Russian recommendations. I accept the view that the depression referred to is the geomorphological depression which is really a continuation of the depression shown in and around the right toe of the dam where a network of piezometers had already been installed by the Russians. The network of piezometers was a sine qua non to determine the design of the cut off work necessary in this depression.

79. The Russians logged the results of the four drill holes totalling

549 m in Saddle 2 on the right bank. The deepest borehole RS1 was drilled to a 150 m depth. A falling ground water table was recorded and core loss around 150 m (373.40 EL) was noted. No piezometers however were installed using the four bore holes in Saddle 2. Nor were piezometers installed in the geomorphological depression to which they adverted in their recommendation No 12. This was to be done at the next stage of design development. However the Russians were well aware that the bedrocks in Saddle 1 were deeply weathered and seepage could take place from the reservoir at the downstream side. They had calculations which showed that at this reach 1.1 km long the water losses could amount to 50 l/sec. They did not expect piping because of the considerable length of seepage paths and correspondingly small exit gradients. Their final conclusion was that so far as the engineering and geological conditions were concerned the construction of the Samanalawewa complex was possible at the considered site. And the project was to build a seasonal storage reservoir with a 210 million m^3 capacity at the Normal HWL 460m extending along the Walawe Ganga canyon for 11 km and along the Belihul Oya canyon for 5.5 km and of a width not exceeding 1.0-1.2 km at the widest. The Samanalawewa dam itself was planned to be a rock-and-earthfill dam 97m in height with a crest length of 500m and 465m in width at the bottom of the river channel.

80. I have dealt with the work of Hydroproject of Russia done in collaboration with the Central Engineering Consulancy Bureau (CECB) in fair detail and quoted in extenso from the Russian reports because the Russian presentation was in reality more than a mere feasibility study. Their report in contract parlance could be called a Tender design with a BOQ. It is crystal clear that their concept of the Project was to construct a reservoir of 210 million m^3 capacity at the normal HWL of 460m and that so far as the engineering and geological conditions were concerned the construction of the Samanalawewa complex was possible at the considered site. Theirs was not a run-of-the-river Project. Mr Vernon Pereira in his evidence at page 229 assessing the Russian work said " 90% and more is almost final with the data they have, but there are other sections and investigations to be done". The following question was put to him:

"Q. It is your position that this design report contemplates another stage of design development after certain explanations or collections of data ?"

His answer was as follows:

"Yes, in the preparation of the detailed project report some proposals came out which could not be implemented at the present stage of design because they necessitated carrying out additional field investigations and studies, not included originally in the responsibilities of the parties. Considering these proposals allows for a more complete assessment of the benefits that are expected from the project construction, which may also result in reduction of work in this field on separate structures and elements. Hence the cutting down of costs of the project. The proposed partial alteration will not entail changes of the engineering solutions and economic conditions and, therefore, they will not affect the dates set for construction activities. The proposals can be implemented in the course of working 'final design development'. This is what they said ". After stating that before construction, data should be extracted on which to base the cut off work, he added "Well, you do the cut-off work; during construction you invariably open up many places and you come to know more about the geology and then you decide the cut-off as well. You may have piezometers installed during construction." In other words what Mr Pereira said was that the piezometric monitoring should begin before construction and be continued during and after construction. You could start with a cut-off design before construction but the final design will depend on what you encounter during construction. This seems logical and appropriate.

81. If Hydroproject of Russia had been commissioned to execute the Samanalawewa Project they would have installed the nest of piezometers in the depression on the right shore as recommended by them at the next stage of design development. The results of the piezometer readings would undoubtedly have indicated the need for any additional cut-off works.

CECB, would then, as they do now, no doubt have advised internal drainage on the right abutment with, if required, a grout curtain to cut-off the weak zone. With this easy-to-execute and comparatively inexpensive option available it would be unrealistic to expect that they would have embarked on a new or modified concept for the Project having already reached a 90% stage of completion of investigations. After the Russian presentation the Project was 'go' and CECB too went along with it.

THE NIPPON-KOEI RECONNAISSANCE REPORT, 1982

82. This report was compiled by Engineers from Nippon-Koei after a brief investigation of the Samanalawewa Project beginning December 1981. This report drew lavishly from the Russian presentation and also from the 1981 CEB Expansion Planning Study. The latter study was issued by the Ceylon Electricity Board. It incorporated descriptions of the development of a computer model of hydroelectric and thermal power installations on the system (the Macro Model), potentially an important tool both for planning and operations. Nippon-Koei made no special field investigations of their own and their report made no significant contribution to the accumulated mass of factual data of the previous studies. A cost-benefit analysis was included but the methodology and basic assumptions adopted were not clearly formulated. The Nippon-Koei formulation of the cost-benefit analysis differed from those incorporated in previous studies. Though no new information was brought in, the report concluded that the Project was viable and opened up an avenue for future Japanese involvement.

THE BALFOUR BEATTY REPORT

83. In February 1983 a team formed from the leading Engineers, Suppliers and Contractors at that time engaged on the Mahaweli Ganga Hydroelectric Project were invited to make a study of the Hydroelectric Power Project at Samanalawewa. The team was headed by Balfour Beatty Limited and the other members were from GEC Energy Systems Limited, Sir Alexander Gibb & Partners and EPD Consultants Limited. Their terms of reference were -

(a) to review the work carried out under previous studies;
(b) to establish the optimum design of the project;
(c) to analyse the project and prepare cost estimates and forecasts for total capital requirements and expenditures;
(d) to carry out detailed programming for the most suitable and economic development of the scheme;

84. The Balfour Beatty team submitted its report in April 1984. The report carried an Engineering Review and Recommendations arranged in seven chapters under the following heads:

i. Introduction

ii. The Power System

iii. Climate and Hydrology

iv. Geology

v. Project Selection & Optimisation

vi. Description of the Recommended Development

vii Cost Estimate and Programme

It is not necessary that I should embark on a discussion of all these chapters in a report like mine but some of the chapters deserve some attention in view of the proximity in time and the influence of the review on the final decision to commence work on the Project.

85. Chapter 1 refers inter alia, to the location of the project site, the main features of the recommended project and the previous studies. The noteworthy aspects of the ground traversed in this chapter have been already sufficiently covered by me. Chapter 2 deals with the Power system about which no serious problems have arisen. No questions about the soundness of the work executed under this head have been raised before me.

86. Chapter 3 deals with Climate and Hydrology. A detailed study done by CECB for the Russian presentation has already been referred to by me. No marked differences on the questions of temperature and humidity, wind and rainfall from CECB's findings have been recorded by the Balfour Beatty team. A comparative summary of the data sources and the results of the previous streamflow studies have been set out in tabulated form. The Balfour Beatty group adopted updated data for their formulations. New records used by them were gauge measurements on the Walawe Ganga about 0.5 km downstream of the proposed dam site from January 1959 to September 1967 and from January 1973 to February 1980. Measurements were generally taken at hourly intervals from 7.00 a.m. to 5.00 or 6.00 p.m. A rating curve relating gauge height to streamflow was prepared from current meter measurements of the streamflow at low and medium discharges. On runoff no serious shortcomings in the CECB analysis were found. The 100 year, 1000 year and 10,000 year flood hydrographs were derived for the Russian report and these were considered acceptable. The Balfour Beatty review points out that guidelines on design floods are given in "Floods and Reservoir Safety" published by the British Institution of Civil Engineers. The recommendation

in these guidelines is that where a breach of a dam will endanger lives in a downstream community and where overtopping of the dam is not tolerable the dam should resist the Probable Maximum Flood (PMF). However, where a breach will only endanger lives not in a community and will result in extensive damage, the dam should be designed to control the 10,000 year flood. On the basis of these considerations Balfour Beatty decided to adopt the view taken in the Russian study that the Samanalawewa dam and spillway could be designed to withstand the 10,000 year flood. The 100 year flood was adopted as the diversion design flood.

The flood hydrographs show the following statistics:
100 Year Flood - Peak 2000 m^3/s
1000 Year Flood - Peak 2709 m^3/s
10000 Year Flood - Peak 3600 m^3/s

87. On the Geology, the review agrees that Samanalawewa project area lies wholly within an area of Precambrian high grade metamorphic rocks consisting of granulites, gneisses, quartzites and crystalline limestones (marbles) of the Kaltota Formation. The rocks were folded and the project area lay wholly within the major fold referred to as the Balangoda syncline. The Kaltota Formation has two members: The Upper Member and the Lower Member. The Upper Member forms the core of the syncline upon which the Samanala Wewa dam and reservoir are located. Gneisses are the dominant rock type with subsidiary quartzites, crystalline limestones and granulites. The quartzites are more abundant than the limestones. The Lower Member forms the limbs of the syncline upon which the Diyawini dam, reservoir and the power station are located. Gneisses are again the dominant lithology with subsidiary quartzites, crystalline limestones and granulites. The limestones are more abundant than the quartzites. The project area has been affected by deep tropical weathering associated with the development of residual soils. Transported soils are generally thin and are confined to talus deposits and river sands.

88. The major fold structure in the project area is the slightly asymmetrical Balangoda syncline. The axis of the fold plunges towards the northwest at 15°-20°, with the southwest and northeast limbs dipping at 35°-40° and 25°-30°, respectively, towards the centre of the syncline. The

overall fold structure is complicated by the presence of minor folds and small faults.

89. Three major joints sets were present:

(a) Dominant set dipping at 50° towards the north;

(b) Two secondary sub-vertical sets striking at 020° and 335°. Joint spacings varied from wide (200-600mm) to very narrow (6mm). Joint roughness varied from rough to slickensided.

90. The presence of a number of major faults was observed from aerial photographs and from geological mapping and drilling. The faults have a wide variety of orientations and appear to be characterized by the presence of fault zones up to several tens of metres wide containing crushed and altered rock.

91. Four main rock types occur within the Kaltota Formation: gneisses, granulites, quartzites and crystalline limestone which are coarse to medium grained pure, pure to very micaceous, marbles. On the weathered limestones the Balfour Beatty comments are significant:

"The weathered limestones generally contain solution hollows and cavities ranging from 0.5 to 5.0m in diameter..... Significant variations occur in the project areas as to the depth to which the solution features occur. In general the depth of the weathering profile is only 5-10m. However, in areas adjacent to faults and where the foliation of the rocks dips into the hillside, the weathering and solution effects may penetrate to much greater depths, up to about 100m. A characteristic of the weathering profile is the rapid variation which can occur in its thickness with variations form 5-10m to more than 30m thickness occurring in lateral distances of only 10-20m."

92. Tropical weathering has affected the rocks throughout the project area and produced a deep weathering profile characterised by gradational change upwards from fresh rock through to weathered rock and ultimately to residual soils at the surface. In general, the weathering profiles are thickest (30-100m) towards the crests of slopes and thinnest (0-10m) in the valley bottoms.

93. Superficial materials within the project area consist of residual sandy and clayey soils. However alluvial sands and gravels occur within the main river channels and thin layers of talus occur at the toes of the larger slopes.

94. The hydrogeology of the project area received considerable attention from the Balfour Beatty team. The groundwater table was generally within about 10m of the ground surface throughout the project area. But in areas where fault zones or weathered limestone horizons gave rise to deep permeable zones, the water table was drawn down by underdrainage of the rock mass. Weathering had a major influence on the permeability of the rock mass. The fresh to slightly weathered rock generally can allow permeability (less than 1×10^{-7} cm/s) except in isolated zones of highly fractured rock or in interbedded weathered limestone horizons which could have a high permeability (1×10^{-3} cm/s). In moderately to highly weathered rock the permeability is controlled by the fracture spacing and is typically less than 1×10^{-4} cm/s although, again there are frequent, more permeable horizons with a mass permeability of approximately 1×10^{-3} cm/s. In the completely to highly weathered rock, the mass permeability is generally high (greater than 1×10^{-3} cm/s. Rock is clasified as impermeable/impervious or permeable /pervious. The Lugeon test is the standard criterion. In Lugeon's formula where one litre of water per minute is absorbed by the rock by a one-metre-borehole under a test pressure of 10 kg/cm^2 it constitutes one Lugeon unit. According to Lugeon a rock absorbing less than one Lugeon unit is considered to be reasonably watertight. One Lugeön unit is approximately equivalent to $K=10^{-5}$ cm/s (Darcy's law) - see "Rock Mechanics and Engineering" by Charles Jaeger 2nd Ed. 1979 p 333. K is the coefficient of permeability or the hydraulic conductivity. Permeability is a measure of the velocity of fluid flow through a porous or fissured sample under the hydraulic head operating within the sample. The sample is housed in a permeameter. Permeability values express the ease with which water will pass through a rock and convey information about the degree of interconnection between pores or fissures. Permeability tests can be used in assessing the susceptibility of rock to weathering. Decreased rates of water flow during testing can be due to the formation of alteration products which, on swelling, block the pores and fissures. Although porosity and permeability are related, the relationship can be quite complicated. The pores need to be of a sufficient size and to be interconnected for flow to occur.

The inferences on permeability made by Balfour Beatty are supportable under the Lugeon test.

95. The Balfour Beatty group also found the dam site selected by the Russians quite acceptable. On the physiography of the dam site Balfour Beatty decided that the dam will be located on the Walawe Ganga immediately downstream of its confluence with the Belihul Oya. The location of the dam is constrained at its upstream end by this confluence and at its downstream end by the confluence of the Mathihakka Ara on the left bank and a minor tributary on the right. The valley begins to widen downstream of these two tributaries. At the dam site the river bed is approximately 60m wide, with bedrock outcrop over most of its length.

The left bank of the Walawe Ganga immediately upstream of the proposed dam rises at a general slope angle of about 26° with only occasional outcrops in stream valleys. On the right bank the general slope rises at 29°-33°. Both banks are covered with thick vegetation near the river thinning up to grassland at crest. Within the dam site area, there are on each bank several ephemeral streams perpendicular to the Walawe Ganga and spaced at about 80m intervals. Many of these streams occur in pairs one on each bank flowing opposite each other. This feature suggests they are controlled by joints or small faults. On the right bank there are a number of small groundwater springs which have a low discharge throughout the year.

96. The Samanalawewa dam site is located on the northern limb of the Balangoda syncline and is underlain by gneisses, granulites, quartzites and crystalline limestones of the Pre-cambrian Kaltota Formation.

97. Drilling had shown that discontinuities, both joints and foliation are generally rough. Discontinuity spacings, as determined from borehole cores are in the range of 1.10-3.0 m.

98. Gneisses are the dominant rock type at the dam site, with frequent bands of interbedded quartzite. On the left bank the quartzites

occur as bands generally 5-15m thick, whilst on the right bank they tend to reach a thickness up to about 40m. Crystalline limestones are less abundant but tend to affect the pattern of weathering in the rock mass. The limestones occur in two forms:

(a) as thin lenses (less than 1m thick) distributed throughout the rock mass, and,

(b) as thicker bands and lenses (4-5m thick).

They are more prevalent on the right bank than on the left. Within the zone of weathering the limestone bands are affected by solution. On the right bank, where the foliation dips into the hillside, the weathering effects have penetrated very deeply with the result that partially infilled solution cavities in limestone bands had been encountered at depths up to 100m. Granulites occurred as thin bands (less than 1m thick) interbedded with the other rock types and tended to weather to greater depths than gneiss.

99. The Balfour Beatty team addressed itself to the groundwater condition and rock mass permeability. In situ permeability tests were carried out in some boreholes in each of the site investigation phases.

A high proportion of these tests showed very limited flow at maximum test pressures. Where a loss of drilling water occurred there were often no test results recorded or insufficient pressures were obtained. These high permeability zones were (through not invariably) generally associated with logged patches of highly fractured rock or limestone with solution features.

100. Water level readings taken during drilling operations along with a limited number of piezometers in which groundwater elevations were monitored over a period of time, suggested that a different groundwater regime is present on each bank of the dam site. On the left bank, groundwater circulation appeared to be confined to the open fissures and porous material within the weathering profile. This made the phreatic surface roughly parallel to and 10-15m below the ground surface with a gentle hydraulic gradient towards the river. On the right bank underdrainage of the hillside appeared to be occurring. Loss of drilling water at depth in a number of

boreholes, the falling water levels recorded as drilling progressed and anomalously low piezometer readings indicated this. The underdrainage was thought to be due primarily to the presence of weathered limestone layers which provided a high permeability path to the river, both upstream and downstream of the dam site. Faulting was also a factor causing underdrainage of the right bank.

101. The foliation dipped steeply into the hillside on the right bank and weathering effects had penetrated deeply along bands of limestone and granulite which were more susceptible to weathering. On the left bank the foliation was sub-parallel to the ground surface.

102. The reservoir which would be created behind the Samanalawewa dam would be "U" shaped. Each limb would be about 8 km long and the base of the "U" 5 km long. The reservoir would be located within the core of the Balangoda syncline. The significance of the reservoir being located in the core of the major syncline was twofold:

(a) As virtually all the beds along the outer face of the reservoir dipped towards the reservoir, it would aid watertightness since leakage must cross all individual beds;

(b) The reservoir was surrounded by land significantly higher than top water level for considerable distances from the reservoir margins - approximately 3 km.

The only exceptions to these two generalizations occurred near the dam site in three specific areas:

Saddle 1 - on the right bank, 1200m from the dam (Saddle 2 in the evidence before me)

Saddle 2 - on the right bank, 600m from the dam (Saddle 1 in the evidence before me)

Saddle 3 - on the left bank, 700m from the dam

Perhaps I should add here that there was also Saddle 4 on the right bank 2100m from the dam. All the saddles on the right bank were on the south of the dam while the one on the left bank was on the northeast of the dam.

103. Ground lower than reservoir top water level occurred at each of these sites on the downstream side of a saddle within 400-600m of the reservoir margin. The geological structure was such that potential seepage paths existed along the strike of the foliation. Saddle 1 (Saddle 2 as numbered before me) was investigated during the period of 1975-1978. Four boreholes indicated that zones of highly to completely weathered rock and residual soil extended to depths of 120m below proposed reservoir top water level. Losses of drilling water occurred overnight and there were zones where pressures during permeability testing could not be attained. Simplified calculations suggested that seepage losses of between 20 and 100 l/s could occur through this saddle under full reservoir conditions. No information regarding saddles 2 (saddle 1 as numbered before me) and 3 was available but in view of the condition found at saddle 1 (saddle 2) and at the main dam site, karstic limestone bands or faults could provide seepage paths out of the reservoir area at these locations also. No investigation data were available for saddle 1 on the right bank (I adopt the numbering used in the evidence before me) and saddle 3 on the left bank but their geological condition being similar to those of saddle 2 seepage loss must be expected at these locations also following impounding.

104. Balfour Beatty concluded that further work at all three saddles would be necessary "to provide a sufficiently reliable estimate of seepage and to provide data for the design of any cutoff works". They considered that "the three saddles are the only areas through which significant reservoir seepage was likely to occur. Elsewhere the low rock mass permeability, long seepage paths and the probability of a high groundwater table were expected to limit seepage to negligible amounts".

105. There were no studies on reservoir margin instability available. But in view of the location of the reservoir in the core of a major syncline and the consequent occurrence of foliation planes dipping in towards the reservoir, further study of the stability of the reservoir margins was necessary.

106. In the next chapter 5 project selection and optimization are discussed but these subjects need not detain us as the problems that have surfaced during my inquiry do not impinge on those areas.

107. Chapter 6 deals with the description of the recommended development. Certain matters discussed in this chaper are worth looking into.

108. The embankment dam located at the confluence of the Walawe Ganga and Belihul Oya would be 102m high with a crest length of 470m. The dam would be a zoned rockfill embankment with a clay core of reduced width. On the right flank, the foliations dipped steeply into the slope and bands of weathering penetrated up to 100m below the surface. These tended to be associated with limestone rich layers. Hence, the depth to competent rock would be deeper on the right bank than on the left bank. Several large trench excavations were expected to be carried out to remove the upper parts of the deeply weathered bands. Grouting galleries would be a necessary feature for an effective seepage cut-off by a grout curtain. The spillway was to be designed to pass the 10000 year flood of 3600 m^3/s with all gates fully open and a maximum flood reservoir elevation of 464.70m. It was anticipated that the peak discharge through the spillway would increase by over 30% to 4700 m^3/s before the reservoir rose to the level of the crest of the dam.

109. The Balfour Beatty concept of the Samanalawewa Scheme does not differ from that identified in the Hydroproject/CECB study. The sites of the main dam, intake structure and powerstation remain the same and the report proposes a straight alignment for the tunnel between the intake structure and powerstation.

110. Balfour Beatty proposed twin diversion tunnels, located on the left bank in preference to the single tunnel on the right bank proposed by the Russians. The low level outlet was also located in the left bank. The spillway was located on the left abutment and was to be equipped with three radial counterweighted gates similar to those installed on Victoria arch dam.

111. The intake was located in a deep trench and incorporated an anti-vortex structure. The low pressure tunnel between the intake structure and the surge chamber was 4840m long, horse-shoe section of 2.75m radius and with a longitudinal slope of 1:150. The tunnel would be generally unlined with rock support being afforded by shotcrete and rockbolts as necessary. The

invert however would be concreted throughout.

112. Balfour Beatty proposed an underground design for the waterways between surgechamber and powerhouse, which is a basic change from the surface solution selected by the Hydroproject/CECB report, A vertical shaft 252m deep and with an optimal stage 1 diameter of 4.5m (or 6m diameter to allow for later doubling of installed capacity) would lead into a horizontal high-pressure tunnel leading to a short section of surface penstock adjacent to the powerhouse. The initial 483m of the high pressure tunnel would be concrete-lined and the remaining 350 steel-lined.

113. The surface powerstation was located on the left bank of Katupath Oya at the foot of the Kaltota scarp and is equipped with two 60 MW Francis turbine units discharging to a 580m tailrace canal. Provision was made for an extension to the power station for the installation of an additional two units of 60 MW each.

114. The second stage of development was to include the construction of a dam across Diyawini Oya slightly downstream of the surgechamber, conversion of the surgechamber to an intake, construction of a second high pressure tunnel and extension of the powerstation. This dam site was located on the Diyawini Oya about 2km from where this Oya plunges down the Kaltota-Hapugala escarpment towards the Katupath Oya.

115. The report used hydrologic data from the CECB report published as part of the Russian presentation in August 1978.

116. Reservoir operation studies were made to select the best out of three options and to determine the optimum values of the design parameters of the scheme. A computer model was used for these calculations which allows simulation of the operation of the Samanalawewa Reservoir over a long period in such a way that firm energy production is maximised. The assumed generation pattern was such that each month the same quantity of energy is produced. The firm energy was defined as that quantity of energy which can be produced throughout the operation period of 53 years from 1921-1973 with a reliability of 98% assessed on a monthly time interval. The results of the simulations indicated that, without Diyawini dam, the

recommended alternative (Alternative III) gave a firm energy production of 431 GWH per year and a secondary energy production of 38.5 GWH per year, with the maximum operating level of Samanalawewa Reservoir optimised at EL. 464m.

117. Though a cost estimate was given for the project, a project evaluation was not made. The construction programme was estimated at 72 months inclusive of 12 months for preliminary works.

118. Chapter 7 deals with cost estimates and the programme, which are subjects outside my terms of reference.

119. The Balfour Beatty group came to terms whith the problems in the right bank and saddles. In reviewing the hydrogeology they realised that fault zones, fractures, cavities and weathered limestone horizons gave rise to deep permeable zones which drew the water table down by underdrainage of the rock mass and further investigation was needed.

THE 1984 ELECTRO WATT STUDY

120. In November 1984 Electro Watt Engineering Services Ltd (EW1) of Zurich, Switzerland tendered a Review Report of the Balfour Beatty presentation.

121. On the Balfour Beatty Proposal, EW1 agreed that the overall concept proposed by Balfour Beatty for the first stage was in line with previous studies and was satisfactory. Yet the degree of detail of the report did not reach that of a tender design. To raise the Balfour Beatty submission to the level of a full tender design, on the basis of which a contract could be drawn up further detailed design work and refinement of the optimization studies would be required. In view of the intensive data available from previous studies, EW1 did not share the opinion of Balfour Beatty that the start of their tender design must await the results of further site investigations.

122. EW1 agreed that a rockfill embankment with impermeable clay core would be the best type of dam for the site. The overall dimensions

and precise location indicated by Balfour Beatty could also be supported. EWI was in full agreement with the proposal of Balfour Beatty to construct twin diversion tunnels through the left abutment of the dam, rather than a single tunnel on the opposite bank as selected in the Russian study. In their final design Balfour Beatty would have to give their attention to the questions of the protective excavation of the steep and potentially unstable slopes which the Russians identified on the right abutment, thickness of the impermeable core of the dam and the main spillway on the left bank. The 4800m long pressure tunnel between Samanalawewa reservoir and the surge chamber would have to be fully lined. Changes in the high-pressure hydraulics system between surge chamber and powerhouse were also recommended.

123. EWI agreed that Balfour Beatty had correctly appraised the geology of the dam site and had correctly distinguished the very different geotechnical characteristics of the right and left dam abutments, the heavy overburden, the deep weathering on the right bank and the bedding planes dipping towards the valley of the left abutment. The left abutment was considered more stable than the right abutment where thicker bands were undercut by the soft weathering products of karstified marble.

124. EWI observe that the geotechnical interpretation of the geological features at the damsite was based on drillholes and adits. Geophysical studies like seismic soundings had also helped. On the basis of this material it was possible to prepare a map which indicated the thickness of the overburden by means of contour lines of the rock foundation, with increasing thickness of the overburden from the river bottom to the higher levels of the valley flanks, particularly on the right abutment where at and above dam crest level the overburden was 20 to 30m thick. Whatever this rock surface was, Balfour Beatty were right to state that additional exploratory work was needed to determine the precise excavation depth required for the dam. It was correct to say that, while slightly weathered to fresh rock with high core recovery and high seismic velocities (>3500 m/s) and moderately to slightly weathered rock with some core recovery and moderate to high seismic velocites (2000-3500 m/s) but still requiring blasting, must be excavated from the foundations of concrete structures and the dam core, highly to completely weathered rock and residual soils with low core recovery and low seismic velocities (<2000 m/s) should be completely removed

from the shell foundation.

125. The permeability of the bedrock had never been quantified. Yet, the karstified marble exhibited high to very high permeablity and would need grouting. The general fracturing and loosening in bedding planes and near surface joints were moderate. The general competence of the rock as foundation for an embankment structure could be assumed.

126. Balfour Beatty had chosen the left dam abutment for the major structures on geotechnical grounds. Sound rock is found at much shallower depth than on the right abutment. Karstified marble was not expected at river bed level where it could hamper driving of the diversion tunnel. The somewhat delicate superficial zone at the tunnel inlet and outlets, where the rock is split up into big blocks which are often detached and slightly dislocated would require detailed study and suitable stabilization treatment. The tunnel portal zones each about 100m long would require rock support during excavation. Balfour Beatty had taken into account the predominant fracture system which crosses the valley in preparing their design. The tunnels would be excavated through moderately fractured but sound rock. The spillway was founded on sound and competent rock over its entire length.

127. A morphological depression on the extended left abutment, beyond the spillway intake, gave low seismic velocities (approx. 1000 m/s) in the bedrock. EW1 felt this should be investigated and if further studies confirm that this is a weak zone then treatment would have to be considered to prevent leakage and erosion towards the downstream valley.

128. The Russians had prepared a geological map of the reservoir which showed the general structure of the area concerned and the main tectonic discontinuities though they did not furnish details of the watertightness of the reservoir and the stability of the slopes. The general watertightness of the reservoir was never questioned and the gradients towards other depressions were favourable with the exceptions of three saddle zones, two of them on the right bank and one on the left bank. EW1 considered further investigations of these three locations necessary to ascertain the stability and permeability of these potentially weak sections of the reservoir rim.

Balfour Beatty too had felt these investigations essential. EW1 fully supported Balfour Beatty's proposal to make a survey of reservoir slope stability by a field survey and interpretation of aerial photographs.

129. EW1 fully endorsed Balfour Beatty's plans to construct a zoned rockfill dam. The structure would comprise upstream and downstream shells of rockfill and a vertical clay core in the dam axis. An upstream cofferdam would be incorporated into the main dam, and a toe load provided downstream. Full account had rightly been taken of the morphological and geological/geotechnical characteristics of the site. Balfour Beatty's proposed dam design was similar to the zoned rockfill embankment selected for implementation by the Russians. The design was good and appropriate. Balfour Beatty did not furnish stability analyses but such calculations had been carried out by the Russians with satisfactory results.

130. EW1 discuss the problems which would be involved in excavation operations for the dam. Appropriate technical specifications would have to be prepared at the time of tender design. Balfour Beatty had not provided for excavation beyond the boundaries of the dam foundation. It would be necessary to reprofile the steep valley flanks adjacent to the dam, particularly upstream of the right abutment in the interests of stability and to avert possible landslides. Providing excavation beyond the dam boundaries must be carried out as envisaged by the Russians.

131. In view of the comparatively weak foundation and particularly because of the deep local weathering both the Russians and Balfour Beatty provide for a grout curtain to prevent seepage under the dam core. In addition Balfour Beatty will perform consolidation grouting of the core contact area as a whole. These two steps were indispensable and EW1 advise that the designs must be made and carried out with utmost care and diligence.

132. Drainage downstream of the grout curtain was a vital requirement. Balfour Beatty had made provision for this and when carried out, seepage will be intercepted and uplift reduced. In this way any major leakage under the dam site shell can be prevented.

133. The location of the spillway on the left abutment of the Samanalawewa dam is well selected. The concept of a structure with gated frontal weir, inclined chute and final flip bucket accorded with established practice and the design presented by the Russians and was acceptable.

134. In designing the spillway to cope with floods, Balfour Beatty have followed internationally laid down criteria. Their design leads to a maximum flood reservoir elevation of 464.9m, which will accommodate 3600 m^3/s evacuation of flood water, and was sound.

135. EW1 have commented and made suggestions on the Balfour Beatty presentation relating to the High and Low Pressure Hydraulic Systems, the Samanalawewa Power Station and the cost estimates. These have not been identified as problem areas in the Project as presently executed and need not detain us.

136. Balfour Beatty had indicated in a letter to EW1 that they were prepared to await the report of their additional studies to make a final evaluation.

BALFOUR BEATTY CONSTRUCTION LTD - PROPOSAL (MARCH 1985)

137. In March 1985, Balfour Beatty submitted what may be called an addendum to their proposal. This contains their proposals for the working arrangements, the contract and technical proposals. Most of the technical comments and matters discussed are repetitions of what they had set out in the 1984 Technical Report. The only important changes to the original design were on:

(a) Lining of the low pressure tunnel;
(b) Additional excavation on the dam as recommended by EW1 ;
(c) Slight lowering of the level of the turbines and generators.

138. Balfour Beatty had proposed arrangements for execution of the works on their design. They proposed that the Samanalawewa Scheme be released by the following companies:

(a) Anglo-French Civil works by Balfour Beatty in joint venture with Spie Batignolles;

(b) Anglo-French Equipment contract by Consortium of GEC Energy Systems and CGEE Alsthom;

(c) Design Engineer Sir Alexander Gibb and Partners, in association with Engineering & Power Development Consultants.

The contractor or contractors responsible for the work covered by the Japanese package had yet to be nominated.

139. The offer was summarised under the following heads:

(a) Detailed construction programme in particular for the diversion works, main dam and low-pressure tunnel;

(b) Revised "Relationship Chart", reflecting the distinct responsibilities of the Supervision Engineer and Design Engineer;

(c) Modified draft "Coordinating Agreement".

REVIEW BY EW1 MARCH 1985 PROPSALS OF BELFOUR BEATTY

140. In May 1985 Electrowatt Engineering Services Ltd, Zurich, Switzerland (EW1) submitted their review of the Proposal submitted in March 1985 by Balfour Beatty Construction Ltd.

141. EW1's comment on the March 1985 proposal of Balfour Beatty was that although it was generally comprehensive, it was inadequate on a number of important technical and contractual matters. These matters related inter alia to the proposed design of the civil works and in particular equipment, and to the contracts which would be entered into for the realisation of the project. In the Appendix to their report EW1 outlined the information and data that had to be furnished. The only noteworthy changes to the original design related to the lining of the original tunnel, additional excavations on the dam abutments and a slight lowering of the level of the turbines and generators.

142. Balfour Beatty had submitted a Bill of Quantities but had not supplied details of the composition of the most important unit prices.

The bills were separated between the two proposed contractual package as follows:

Anglo-French Package (embankment dam, drilling and grouting, instrumentation, grouting and tunnel, river diversion works, spillway).

Japanese Package (intake structure, low pressure tunnel, surge chamber and pressure shaft, high pressure tunnel, penstock, powerhouse and tailrace, as well as hydro-mechanical equipment).

143. EW1 was of the view that the technical design proposed by Balfour Beatty for the civil works in 1984 had been repeated in their March 1985 proposals with only a few changes. No account had been taken of the latest hydrological data either for the definition of reservoir storage level or the design of the spillway and diversion works. EW1 remained of opinion that "the level of detail of Balfour Beatty design is still insufficient to form the basis for the conclusion of a normal contract".

SAMANALAWEWA POWER PROJECT REPORT BY CECB APRIL 1985

144. At the request of the Ceylon Electricity Board the CECB compiled a Technical Report for the following purposes:

(a) to summarise the work carried out under previous studies;

(b) to establish the project parameters and features on the basis of previous studies and reviews;

(c) to update the project cost, project benefit and project evaluation.

145. To meet Sri Lanka's energy requirements, the CECB's planning indicated that a further generating station should be commissioned in 1990. Following the completion of the hydroelectric stations currently under construction, and proposed to be constructed, the most suitable and economically beneficial hydroelectric project to satisfy this requirement is the Samanalawewa Power Project.

146. The main features of the recommended project were:

"(a) Samanalawewa Dam located on the Walawe Ganga River immediately downstream of the confluence with the Belihul Oya and immediately upstream of the confluence with the Matihakke Ara. The dam is of the rockfill type with a clay core having a crest level of 467.5m, 105m above assumed river bed level, with a crest length of 480.0m. The dam structures include an overflow chute spillway with three radial gates on the left bank and a low level outlet/irrigation discharge outlet, also on the left bank. The reservoir can be used for seasonal control of the flow in Walawe Ganga and for a limited amount of overyear storage.

(b) A conduit system consisting of a 5150m long, 4.2m diameter, lined, horseshoe-shape low-pressure tunnel; a 75m deep, 14m diameter concrete-lined surge chamber; a 204m long, 3.8m diameter steel-lined, tunnel penstock, a bifurcation, valve house, 533m long 3.3m and 3.2m diameter steel penstock and a bifurcation.

(c) A surface power station located on the left bank of the Katupath Oya. The powerhouse is equipped with two 60 MW Francis turbine units discharging into a 580m long tailrace canal. Energy will be transmitted from a 132kv switchyard over a double circuit 17 km overhead line to the existing substation close to Balangoda"

147. A second stage development would include construction of a dam across the Diyawini Oya, incorporation of an intake to the surge chamber, installation of a second surface penstock and extension of the power station with a further installed capacity of 120 MW.

148. In its new review, CECB discuss climate and hydrology much on the same lines as in their earlier report published as part of the Russian Hydroproject Institute presentation. One chapter is devoted to the geological aspects of the project area. The lithology discussed follows the pattern of the earlier reports and does not require more than a passing mention. What has been said about the structure will bear repetition:

"The Project area lies within a major fold structure known as the Balangoda syncline. The Balangoda syncline is asymmetric with its axis plunging towards the NW at 15° to 20°. The dip angle of the SW limb is 35° to 40° and that of the NE limb is 25° to 30°. The rocks folded into the Balangoda syncline are cut by a number of discontinuity sets. The majority of the discontinuities are steep and dip at 70° to 90° North with strikes 20° and 335°. Another east-west striking discontinuity set dips at 45° to 50° towards north. It is estimated that the thickness of crushed zones along individual faults does not exceed 10-15m. Some of the crushed zones along the faults are healed by pegmatites. Along other faults weathered zones are developed. These faults daylight on the river bed and on the face of Kaltota escarpment."

149. On the important question of limestone CECB noted that crystalline limestone bands occurring in the gneisses and granulites had undergone karst weathering: "The base of karstification is the local base level of erosion (i.e. level of the Walawe river). At a greater depth the karst process has developed in the zones of tectonic disturbances drained by remote erosion paths. The dimensions of the solution cavities formed by karstification depend on the thickness of the limestone bands. Cavities with a volume of several thousand cubic metres are found in thick crystalline limestone bands. The form of these cavities vary from more or less equidimensional caverns to long narrow solution channels extending along discontinuities. Some of the caverns are filled completely or partially with sandy clay or fragments of rock."

150. Ground water encountered in this area is associated with two rock/soil complexes:

(i) Weathered mantle, talus mantle and alluvium deposits occurring over almost the entire area as a blanket over fresh bed rock;

(ii) Metamorphic bed rocks, particularly karstified zones and deeply weathered zones along faults.

The ground waters of both complexes are hydraulically interconnected, forming a single ground water horizon resting on fresh intact or slightly fissured rock basement. Along deeply weathered fault zones ground water may penetrate as deep as 200-300m. Groundwater in the Karst limestones and fault zones is of a confined nature and artesian flows have been observed. The direction of groundwater movement is generally controlled by the ground relief but within the second complex the movement of ground water is affected by the structure of the fault zones and karst features.

151. The tropical weathering of the metamorphic rock has resulted in formation of a crust of weathering, the thickness of which is negligible at the river bed and increases towards the upper elevation of the banks. Three zones could be distinguished in the crust of weathering:

Zone 1 : Residual soil, completely weathered rock, highly weathered rock ;

Zone 2 : Moderately weathered rock (Blocky mass. Blocks are fresh, Joints are weathered and volume of weathered material 10 to 15% of total massif volume) ;

Zone 3 : Slightly weathered rock (Fresh rock. Joints are stained or slightly weathered).

Thickness of weathered Zones 1 and 2 reached 30-40m. Thickness of slightly weathered Zone 3 did not exceed 2-3m to 30-40m in areas free of joints. Within faults, Zone 3 extended to depths of up to 100m or more.

152. In the dam foundation, the surfaces of the gorge slopes were composed of talus and eluvial deposits having a maximum total thickness about 15m. These deposits were characterised by low shear strength and high permeability. The river channel section was composed of slightly jointed hard rock. In the upper part of the river bank the thickness of the highly weathered rock reached 24-25m. Weathering had penetrated to a greater depth on the right bank where the strata dip into the hillside. This deep weathering is common along the bands of limestone and granulites. The limestones within the zone of weathering are karsted. The limestones of the upper carbonate bearing bench on the right bank are karstified to a greater extent. Hollow solution cavities as well as partially infilled solution cavities are present in this zone.

153. For purposes of geotechnical design of the foundation excavation beneath the various zones of the embankment dam and its appurtenant structures, a rock mass quality classification into three grades, as follows, has been established (in the Balfour Beatty study).

Grade A : Slightly weathered to fresh rock with high core recovery and high seismic velocities (>3500 m/s).

Grade B : Moderately to slightly weathered rock with good core recovery and moderate to high seismic velocities (2000-3500 m/s).

Grade C : Highly to completely weathered rock and residual soils with core recovery generaly less than 25% and often zero and seismic velocities <2000 m/s.

This rock mass quality classification was applied to the ground conditions at the dam site to provide a basis for describing the engineering geological conditions at the site. It could be applied elsewhere too.

154. On the left bank the foliation was sub-parallel to the valley slope which was covered with a mantle of Grade C rock varying in thickness from zero at the river bed to 10-15m at the top of the slope. Grade B rock occurred below the mantle of Grade C as a layer (5-20m thick), and also as isolated lenses at depth within Grade A rock. The boundaries between different grades were generally gradational and irregular.

On the right bank, the foliation dipped steeply into the hillside and weathering had penetrated deeply along bands of limestone and granulites. The thickness of Grades B & C was therefore much greater on the right bank than on the left bank.

The mantle of Grade C rock on surface varied in thickness from zero at river bed to about 50m at the top of the slope. Bands of Grade C material extended into the rock mass parallel to the foliations to depths up to 100m below ground level. Such weathered bands were up to 5m thick and had resulted in a very irregular contact with the underlying Grade B rock. The Grade B occurred as a continuous layer beneath the Grade C rock and had deeply penetrating bands parallel to the foliation. Many of these bands extended into the hillside to levels well below river level and would thus be penetrated by any tunnel driven through the right bank.

Beneath the lower slopes, the fresh to slightly weathered Grade A rock was found in isolated areas at depths of about 20m but such areas tended to be separated by penetrating bands of Grades B and C rock. Beneath the higher slopes the weathering typically penetrated to depths of more than 70m, frequently below river level. Grade C material was generally absent in the river bed area and depths to Grade A rock formation were small, being less than 10m.

155. On ground water condition and rock mass permeability, CECB observe that the permeability test results showed that the preserved bed rock was practically impermeable or had a very low permeability. The high permeability was characteristic for the crust of weathering, highly fractured, slightly weathered bedrock and the zone of slackened rock along tectonic dislocations.

On the left bank the ground water table occurred parallel to and 10-15m below the ground surface. From the fact that the crust of weathering also was limited by a similar profile, it could be deduced that the groundwater flow was confined to the permeable weathered zone. On the bank, away from the river channel, the depth to the ground water table was as large as 50m. With the presence of foliation dip towards the hillside, and high permeable layers of rock, it was more likely that underdrainage was taking place. The occurrence of low ground water levels and the observed drill water losses during core drilling, may possibly be due to this underdrainage which was taking place.

156. The "U" (or is it a "V" ?) shaped Samanalawewa Reservoir extends along the Walawe Ganga for about 11 km and along the Belihul Oya about 5.5 km. The reservoir is located in the core of the Balangoda syncline. The reservoir slopes are composed of interbedded benches of Gneiss, Granulites, Limestones and Quartzites. The thickness of the crust of weathering increases upward along the slope and is about 5-10m at the Normal HWL of the reservoir. A talus layer up to 4m thick is present at the surface.

Since, at the outer rim of the reservoir, the rock strata dip towards the reservoir, any seepage path had to cross the bedding giving an added advantage with respect to reservoir watertightness. The gorge slopes were believed to be sufficiently stable except for small landslides confined within the weathered zone of the steep slopes. Seepage losses were likely to occur only in the dam foundation and two right bank saddles located at 600m and 1200m from the dam and the left bank saddle located at 700m from the dam. The right bank saddle at 1200m from the dam had been investigated during the period 1975 - 1978 but the other two saddles had not been investigated.

157. Grouting, both curtain and consolidation, with provision of adequate drainage in the dam foundation is considered a necessary feature of the design of Samanalawewa dam which is founded on comparatively weaker and weathered rock. The right bank which is more weathered, as compared to the left bank, will require extensive grouting. For this purpose, drainage and grouting galleries will have to be provided below the entire length of the core. Another gallery connected with this will be provided under the right bank abutment at EL 400.00 to provide a grout curtain. Yet another grouting adit of short length will be provided on the right bank at the dam crest level. A drainage curtain will be drilled immediately downstream of the grout curtain to intercept rising seepage before it can emerge below the downstream shell of the dam. The main advantage of the use of galleries is the facility to detect and respond to unforeseen seepage patterns. Additional grouting or drainage work can be accomplished as required even after impounding has commenced. Among other benefits of a foundation gallery are the programming flexibility of being able to form the grout curtain beneath the core with consequent option to use higher grouting pressures, and improved access and instrumentation.

158. CECB made a comprehensive review and observed that the Samanalawewa Project will be operated essentially as a power project connected to the CEB system. However, since it is located upstream of Udawalawe reservoir, it will also result in better regulation of the Uda Walawe reservoir. The construction programme will have three main activities:

1. Design which, it is assumed, will start prior to the award of the contract and continue during project execution ;
2. Mobilization - a period of six months is usual;
3. Construction and installation.

159. The investment cost of the project would match the proposed construction programme and commissioning of the project by 1990.

160. As a result of the optimisation studies, the recommended development consists of:

i. a 105m high rockfill dam across Walawe Ganga with crest level EL 467.5 and a chute spillway on the left bank,

ii. a power intake from the right bank of the Walawe Ganga about 5.5 km upstream of the dam,

iii. a 4.2m diameter concrete lined power tunnel 5150m long connecting to a surgetank 14m in diameter,

iv. steel penstock liner, from the surge tank to the valve house and a steel penstock line laid along the escarpment from valve house to the power house,

v. power house located on left bank of Katupath Oya with an installation of 120 (2x60) MW,

vi. Tailrace works.

A second stage development is also envisaged at a later date in which the waters of Diyawini Oya would be diverted to the surgetank of stage I with the high pressure system of stage I duplicated and two more units of 60 MW added to the Samanalawewa power house to meet peak power requirements around 2000 A.D.

CHAPTER VII

TERM OF REFERENCE (b)

161. Under this term of reference (b) I am required to report on whether the current location and design of the Samanalawewa dam and reservoir and the appurtenant works thereto were based on adequate investigations and studies, including geological and hydrological studies.

LOCATION OF THE DAM AND RESERVOIR

162. The object of building a dam is to achieve storage of water. Hence the concern must be to have a watertight reservoir. This involves ensuring that the dam and the banks are sufficiently watertight and that leakages and seepages do not exceed tolerable limits. In addition the dam and reservoir must last. Its durability cannot be limited. I will consider first the location of the dam.

163. The Samanalawewa dam has been located about 250m downstream of the confluence of the Walawe Ganga and Belihul Oya. The CECB report states that this is the most suitable site both from the viewpoint of entrapment of water from both the Walawe Ganga and Belihul Oya as well as from the point of view of topography and geology. The dam and appurtenances are so disposed that they are all accommodated within the short section of river between the confluence of Walawe Ganga and Belihul Oya on the left bank on the downstream. The dam is located on a site at the northwestern edge of the Balangoda synform on the middle peneplain of Sri Lanka between the Coastal Plain and Central Highlands. The foliation on the left bank dips generally 25° - 35° towards the river, while on the right bank it dips 30°-60° into the hillside. These foliation angles result in an asymmetric valley with the left bank sloping at 30°-35° and the right bank sloping at 35°-45°. In the river channel, the rock has a foliation dip of 5°-10°. The variation of the foliation dip is due to a folding of lower order. The most prominent joint set in the dam site area has a dip of 50° towards north. Two sub-vertical joint sets striking at 020° and 335° are also common. In addition to the fold trend, a number of fault systems have also been identified. The river valley within the dam site is relatively narrow with

moderately steep banks. Several shallow ravines and gullies which carry water only during the rainy seasons occur in pairs and perpendicular to the river and opposite each other. The lineation of each of these ravines and gullies appears to extend from one bank to the opposite bank and indicates joints or faults in the underlying rock. The right abutment of the dam site rises up to a peak elevation of 545m and then descends to a low ridge which, extending southward, forms the right bank or eastern border of the reservoir. There are parts of saddle topography at these locations on this right ridge within a distance of 2.1 km from the dam site numbered 1,2 and 4. Saddle 1 is 600m from the dam, saddle 2 is 1220m from the dam and saddle 4 is 2100m. In addition, there is saddle 3 on the left ridge 700m from the dam. All these saddles are higher (by 20m to 60m) than the planned high water level (460m) of the reservoir. The narrowest saddle is No 2 with a thickness of about 300m at the high water level of the reservoir.

164. The bedrock of the dam site consists of metamorphic rocks of the pre-Cambrian complex which are covered by quaternary deposits on the surface comprising surface soil, talus deposits and river deposits. The bedrock consists of complicated interbedding of various gneisses and calcareous rocks which can be classified as granulitic gneiss bed, calcareous bed, granulite bed and charnockite gneiss bed. The metamorphic rock at the dam site is composed of gneisses, granulites, quartzites and crystalline limestone of the Kaltota formation. The dominant lithological varieties of the bedrock at the dam site are biotite gneiss and charnockitic gneiss. Limestones occur in the form of layers and lenses of various thickness (0.1-0.2m to 5-7m) inter-bedded with the two predominant types of rock. The limestone layers in the right bank are thicker than those in the left bank. Inter-bedded quartzite banks are also encountered.

165. As for the geological structure the strata or foliation shows a monoclinic bedding at large with a general strike of NW-SE dips ranging from 20° to 40°. Local undulations and wide variations of the strikes and dips are observable. Three major joint systems could be identified at the river bed outcrops; while the angle of dip is 10° to 20° on the river bed it increases even up to 50° in the higher part of the right bank.

166. There were two sets of trend for faults in this area: one with

a north to south trend and the other parallel to the strike of rock beds. No serious faults are seen at many places in the upper parts of the right abutment. A couple of medium class strike fault fractures 2 to 3m wide intersect the dam axis on the right bank at elevations 465m and 480m. The bedrock is deterioiated intensively to a great depth along these faults. The faults provide routes for infiltration of weathering and hydrothermal alteration. Though these fractured zones are located marginally off the dam foundation they are extremely close to the right wing of the dam and form a deep weathered zone in the bedrock adjacent to the dam foundation on the right abutment.

167. The agencies of weathering of the bedrock at the dam site are air and water from the ground surface and hydrothermal influence from underground. The hydrothermal influence not only weakens the rock by altering the component minerals but also makes weathering by air and water easier. There was deep weathering at the right wing of the dam. Outside this, the weathering was thinner at the bottom of the valley and thicker at the higher parts of the slopes. Weathering was exceptionally deep under the right bank ridge off the right wing of the dam especially in the granulite gneiss bed.

168. Water premeability should also be a matter of prime concern as the success of the dam and reservoir would depend on watertightness. The hydrogeology is important. Groundwater in the dam site area could be associated with rock/soil conditions such as the weathered mantle, talus mantle and alluvium deposits and the metamorphic bed rocks intercalating limestone bands. Groundwater movement is associated also with karstified zones and deeply weathered zones.

169. In general the rock zones deeper than 15m on an average indicate Lugeon values less than 5 on the left abutment and less than 10 on the right abutment. Even in the zone of deep weathering off the right wing of the dam, bedrock with less than 10 Lugeon units is reached at the depth of 40m at most except for a few sections with ~~cavities~~ of limestone. At the bottom of the valley the seepage potentiality is less than 10 Lugeon units for the entire bedrock under the river and zero for most parts.

170. The requisite excavation of weathered material to sound rock was feasible from the engineering angle and could be economically accomplished. All weaknesses could be treated by grout action. Further the fact that the Katupath Oya ran 300m lower than the Walawe Ganga when only six kilometres separated them was a weighty consideration as the power house could be sited on the banks of the Katupath Oya. The prospects of further development were good with a dam on the Diyawini Oya. The works could regulate the water supply to the Uda Walawe reservoir and hence the irrigational facilities of that reservoir. An added matter of importance was the availability of construction material in the vicinity of the proposed Samanalawewa dam. Therefore it could be said that the best available location has been selected for construction of the dam and reservoir.

171. The proposed dam site was satisfactory from the geological point of view. Site investigations had been carried out for nearly 30 years. Core drilling and exploratory adits, geological mapping, in situ permeability tests, laboratory testing of foundation materials and borrow materials, rock sample tests and surveys had been carried out and the weight of opinion of the world renowned engineers who did these tests favoured the present location of the dam. I agree that the location of the dam at its present site was the best choice.

172. I will now consider the selection of the type of dam. The type of dam was determined after considering the following types of dam:

(i) Concrete dam;
(ii) Dam with upstream impermeable membrane;
(iii) Zoned-earthfill dam;
(iv) Rockfill dam with central clay core.

The merits and demerits of each of these types of dam were considered before the selection of the rockfill with central clay core dam for Samanalawewa. Topographical and geological considerations as well as the easy availability of construction materials tilted the balance in favour of the rockfill dam with central clay core. The dam section would comprise an upstream and downstream shell of rock fill with a vertical clay core at the centre with interlayers of transition filter zones. The upstream coffer-

-dam would eventually be incorporated in the main dam. It would have its crest at 413m MSL. The coffer-dam will be made up of rockfill shell material with upstream clay blanket protected on the upstream by rock excavation spoil.

On the basis of optimization studies it was determined that a retention level of reservoir at 464.0m MSL would give optimum benefits. Based on wave height calculations a free board of 3.5m was considered adequate. Accordingly the top of the crest of the dam was fixed at 467.5m MSL. This gave a height of about 105m above existing river bed level for the dam.

173. On the question of the location of the dam I will refer presently to a slight change of the dam axis at its right wing that became necessary during investigations. My conclusions on the appropriateness of the location of the dam site and the selection of the type of dam apply to the changed dam axis also.

174. By way of investigation Nippon Koei and EW1 assisted by CECB felt the following items had to be studied regarding the detailed design of the dam and the appurtenant structures:

"(1) Excavation to the proper foundation;
(2) Stability of slopes, both natural and formed by cut work;
(3) Watertightness of foundation;
(4) Leakage through limestone beds and/or weathered portions;
(5) Method of treatment;
(6) Embankment materials and concrete aggregates.

"The core portion of the rockfill dam is to be placed on a firm foundation which is watertight or can be improved watertight. The Lugeon value of the foundation rock mass is to be carefully measured along the dam axis. The Lugeon profile along the dam axis is also useful to design the curtain grouting work.

"The firm foundation is to be ~~detected~~ by core drilling with water pressure tests. Rock materials are to be recovered carefully by using elastic tube samplers contained in a core barrel, especially for weathered and loose material portions, to know precisely the geological condition.

"In addition, the excavation to the firm foundation and to the cut slope above the crest at the right abutment is to be carefully studied, as a deep weathered zone exists here. A test adit is to be driven at the crest elevation on the right bank for the visual observation of the subsurface weathered rock, by which the excavation line for the foundation and the stable inclination angle of the cut slope will be determined.

"The drilling investigation with water pressure test along the dam axis and the test adit at the crest elevation on the right bank proposed here are urgently required for the tender design of the dam. The high priority is to be given to the said works at the initial stage of the additional investigation, accordingly.

"The foundation of the other portion of the dam and the appurtenant structures such as the spillway is to be inspected by drilling investigation with water pressure test to a depth properly to fit the structure.

"As for the limestone bearing bed on the right bank, deep drillings with water pressure tests are to be performed to the level of EL300 metres in order to know distribution of the limestone and caves therein. The piezometric head in the limestone, separate from the perched water, is to be measured by using a partial water pressure measuring device, in several boreholes such as drilling Nos 10, 12, 19 and 26 which are placed to obtain a hydraulic gradient at the right bank.

"Standard penetration tests would be carried out at 1.5m intervals in depth of the vertical drillings for the overburden and intensively weathered rock zone, for evaluation of their mechanical characteristics.

"Four saddle portions, one at the left bank and three on the right bank ridge, were to be inspected by drilling with water pressure test to a depth at which the ridge has a reasonable width of sound rock mass. The narrowest portion is also inspected by an inclined borehole in combination with a vertical borehole, if a fractured zone occurs here. Those boreholes, except an inclined one, are to be cased with a perforated PVC pipe in order to monitor water table fluctuation before and after impounding."

175. On the dam site itself comprehensive investigations were done: field geological mapping of the dam site and quarry site, geophysical exploration by the method of seismic refraction prospecting for 3520m of total traverse length, core drilling as will be described hereafter, and a 100m long exploratory adit driven at the contemplated level of the dam crest, on the right bank of the dam site.

176. Core drilling on the dam site was done extensively. At first 26 boreholes (J1 to J26) were planned for the dam site and river bed, four for the quarry (J31 to J34) and four more for the saddles (J27 to J30). The drilling involved was 2380m. The boreholes drilled on the original straight line axis of the dam as proposed by the Russians showed deep weathering near the right abutment. This prompted a borehole J35 to be drilled to a depth of 60m in October/November 1986 on a tentative alternative axis veering downstream from the original axis. Thus there were in all 35 boreholes. These 35 boreholes involving 2464.5m of drilling were done. Four boreholes J31 to J34 totalling 300m of drilling were done on the quarry site. All the 31 remaining boreholes involving 2164.5m of drilling were on the dam site. Of these 31 boreholes, 10 boreholes (J1 to J4, J13, J14 and J20 to J23) were on the left bank of the dam site. Seventeen boreholes (J8 to J12, J16 to J19, J24 to J26 and J35 to J39) were on the right bank of the dam site and four boreholes (J5 to J7 and J15) were on the dam site river bed. Of these boreholes J1 to J12 were on the original dam as planned by the Russians, J13 to J29 were on the shell area of the foundation and J20 to J26 were on the foundation. The borehole J6 of a depth of 100m was inclined 45° from the horizontal to the right bank. All the other boreholes were at 90°. The borehole J12 on the right abutment was drilled in December 1986/January 1987. It was planned for a depth of 250m but it was jammed at 145.35m. Borehole J38 also on the right abutment planned for 250m depth and drilled from elevation 525.34m, was jammed at 168.20m. Borehole J39 again on the right abutment drilled from elevation 515.39m was jammed at 70.50m. The drilling of the boreholes was conducted with special care by using triple-tube core barrels with polythene foil tube inside. Some of the deeper holes were drilled by the wireline system. The minimum diameter of drilling was 66 milimetres for the triple-tube method and 75.31 milimetres for the wireline system.

177. The core recoveries of the drilling revealed deeply weathered rock conditions. They revealed that the thickness of weathering to the grade that is not acceptable for foundation of the impervious earth core zone of rockfill dam, was 15m to 20m. The bedrock on the right wing of the dam site was intensively weathered to depths more than 50m or even 100m or to the level of elevation 420m, some 40m lower than the contemplated level of the dam crest. This deep intensive weathering was seemingly due to local hydrothermal alteration along faults and located marginally off the foundation of the dam structure. The same conditions operated to develop solution cavities in limestone. Some cavities with soft clay, one of which was nearly five metres in size, were found in limestones in the sections of 45.0-49.5m of the hole J10, 110.2-112.4m of the hole J11, and 16.2-17.4m and 18.0-19.0m of the hole J36.

178. When checking water permeability, or potentiality for seepage, (of the bedrock by means of water pressure test, or Lugeon test) in the boreholes, the test was made in the descending-stage by steps of 5 metres, as a rule. Some variations in the length of test sections were caused by local softness of bedrock where a packer could not be installed effectively. Water was injected into each test section sealed by the packer, controlling the flow rate and the pressure in the ground surface. Each test was performed under several different pressures. On zones near the ground surface, with less overburden, the peak pressure was controlled low. The water-take, or injection rate, was observed for 10 minutes under each step of the pressure.

Rapid water losses were experienced several times during drilling, when some cavities were encountered but the general trend of the seepage conditions in the bedrock in many sections were low permeability and less than one Lugeon unit. Even in the weathered rock zones, the permeability was low in general. High permeabilities were observed only in zones very close to the ground surface.

In view of probable change in the groundwater head depending upon the depth owing to the existence of limestone cavities, the localised groundwater pressure was observed for every 5m sections of the boreholes J17, J19, J36 and J38 all on the right bank of the site. The observation was made by electrically measuring water pressure in sections of boreholes isolated by setting double packers.

179. An exploratory adit with 100m of length was driven at the level of contemplated dam crest on the right bank of the dam site. The adit was situated mostly in the granulitic gneiss zone which was intensively weathered in many places. The granulitic gneiss intercalated frequently layers of biotite-gneiss which were several centimetres to more than a metre thick and often selectively softened almost into aggregates of disintegrated particles. Minor shearings and slickensides were common and a four to five metre thick fault fracture was encountered between 60m and 65m in distance from the adit portal. The fractured zone was composed of soft clayey material partly the product of hydrothermal alteration, and fragmental blocks of hard rock. It was observed that the direction of foliation changes from NNW-SSE to NNE-SSW in the exploratory adit.

180. The sections with limestone cavities did not always take as much water as expected. This could be because of impervious clayey material filling the cavity. There were on the other hand a few cases of water losses or rapid drawing down of drilling water at some sections in boreholes where no large openings but only open cracks were found. However apart from limestone cavities it is not unusual for open cracks to provide substantial leakage paths or cause drawing-down of water in boreholes in the course of drilling.

181. The successful outcome of the core drilling J35 on the right abutment induced a turning of the right wing of the dam axis about 20° downstream in its position above E 420m on the right bank. This was to reduce the amount of weathered material to be removed. In seeking a change by way of 580m of additional drilling at the dam site it was sought to eliminate for the time being 230m for two drillings at saddle 2. By this process the additional drilling would be limited to 350m.

182. However sound rock had to be found to tie the grout curtain that would have to be constructed to protect the dam and its right abutment. Unlike with the dam not much design work appears to have gone into the curtain grouting on the right abutment.

183. It was realised that the knowledge at that time of the openings in the limestones was inadequate. In the stage of construction, every cavity within an extent substantially affecting the dam function had to be picked up by ~~rows of~~ closely arranged drill holes for curtain grouting, and grouted completely along the dam axis. The curtain grouting had to be planned to cover all zones where any harmful cavities may possibly exist. The calcareous bed had to be covered completely by the grout curtain at least for its extent above the level of the river bed; also where conditions for solution of limestone were always present with circulation of groundwater with carbon dioxide from the atmosphere. The clay in the cavity had to be washed out well before grouting.

184. Some comment regarding the failure to do the boreholes on the saddles at the stage of investigation would be apposite. The Russians made no mention in their 14 recommendations of any work to be done on the saddles.

A Japanese expert in geology had done a surface mapping of the banks and reported the absence of limestone and no problems. There were small springs on the hilltops of the banks above the dam crest level and the Russians too made no mention of saddles in their 14 recommendations or karst or limestone in the logs of the boreholes they bored on Saddle 2. As the saddles were regarded as not posing any urgent problems priority was given to the dam site investigations. Budgetary constraints and the time factor necessitated this. Hence the investigations planned for the saddles were deferred to be done during construction time owing to budgetary constraints and the time factor. Use was made of the provision for drilling the 4 boreholes planned to be done on the saddles for drilling to be done at the dam site.

185. Although the proposal (of 14 December 1986) was only to eliminate 230m of drilling (planned for Saddle 2 and involving J28 and J29) in the New Work Schedule that was submitted all four boreholes J27, J28, J29 and J30 were eliminated. In the result no bore holes were to be drilled on the saddles. When drilling was concluded what had been achieved was less by about 84m from what had been planned. In his evidence Mr K Wada Chief Engineer of Nippon Koei Co Ltd's team of Engineers said that budgetary constraints, time schedules and the fact that as the saddles did not appear

to pose urgent problems they postponed the drilling projected for J27, J28, J29 and J30 involving a total drilling of 450m to be carried out during the currency of the construction works. The tally of boreholes for the dam site and river bed was increased to 31 while the four boreholes planned for the quarry remained. The dam axis was inclined 20° downstream. This was a good move because it cut costs by reducing the quantum of weathered material that had to be excavated to get to sound rock for the foundation of the dam.

186. The four boreholes drilled by the Russians on Saddle 2 showed highly to moderately weathered granulites and biotite gneisses extending to a depth of 120m below proposed reservoir level and complete water losses at 79-80m below retention level. They drilled borehole RS1 to 200m. Rough calculations indicated an estimated loss of 50 l/s at saddle 2. Although the core recoveries are not available the logs show core losses, water losses, cavities and other indicia of underdrainage. No investigation data are available for saddles 1 and 4 on the right bank and saddle 3 on the left bank but their geological conditions must also be expected to be the same. It was generally recognised that further exploratory work on the saddles was necessary. It was known that the groundwater tables were irregular and lower than the reservoir at 460m. The hydraulic gradient and the problematic limestone had to studied. When J12, J38 and J39 were jammed the hydrogeological problem arising from that fact was not looked into.

187. If boreholes additional to those provided for the dam site area were needed, they should have been drilled as an extra item of work for which additional payment and extended time could have been obtained. Such incidents are common in contracts. To eliminate the boreholes involving a modest 450m planned for the saddles in order to use the funds so saved for the additional boring on the dam site even with the client's consent was a costly error. There is a difference between doing the investigations before construction and after. If the investigations were done before, modifications could have been effected to the design and may be if the client became aware of his commitment he may have opted for a less ambitious dam. The Russians recommended a nest of piezometers to be installed in a depression in the right shore at the next stage of design development. I have already made my observations on the location of this depression. Let us for the

sake of argument concede that the location at which the piezometers were recommended to be installed were not the saddles. The Russians had drilled four boreholes RS1 to RS4 in Saddle 2. RS1 was to a depth of 200m. Although the core recoveries were not available, the logs were and they spoke volumes. The story in the logs is of core losses, losses of water, cavities revealing an enigmatic subsurface. If that was so of Saddle 2, Saddle 1 too would have posed the same problems, both saddles being in Area B. To say that the Russian omission to use the word 'saddle' in their recommendations or logs is hardly an excuse for minimising the problems in the saddles which everybody knew or should have known to have existed. EC1 had explored the saddles 1 and 2 to see if an ungated spillway could be built there but abandoned the idea on realising the unpromising nature of the saddles. The Russian logs gave a red alert that there were problems at least in saddle 2. To rely on geophysical mapping and the existence of springs on the hills (which could have indicated perched water tables) looks naive. The cost of drilling a modest 450m could not have been a deterrent. Had the investigations been done before the contract was entered into, tender designs could have been prepared and the successful contractor would have brought down equipment which could meet the needs of the construction works. As it happened when the cut off grout curtain was done on the right bank the equipment needed for the work was not at hand - at least for deeper grouting.

The BOQ mentioned only a 30m deep grout curtain. Hence the contractor was not likely to bring in equipment for a far deeper grout curtain. This affected very adversely the execution of the cut-off works at the right bank. The cut-off grout on the right bank curtain was a hanging curtain - hanging at chainages not anticipated and in more than double the estimated length. Although it is claimed that it has cut off some 'privileged paths', I find myself unable to agree that it is to any reasonable degree effective. On the whole this two billion rupee cut-off grout curtain must be written off as a failure. It had to be constructed under constraints of time, soaring costs and inadequate equipment. Even if it meant extending time schedules, the planned investigations should have been done before construction. After all the investigations had spanned a period of nearly 30 years. A few months delay in the investigation would have ensured a less costly design, more efficient arrangements for the work and importation of adequate equipment. As it happened the cut-off curtain had to be cut according to the equipment on site. The whole episode demonstrates the wisdom of a reasonable investigation before embarking on a massive construction. It is no doubt possible that investigations could and have to be done during construction and that certain risks can and have to be taken. But taking risks without doing any investigations at all on the vital banks of the reservoir is, in my opinion, an unwise gamble. Sri Lanka was building a reservoir not only a dam. The banks of the reservoir are of equal importance. Before I part with this question I wish to made one more observation. On 8 August 1993 I visited a small tract of paddy fields called Killakandura in the village of Kumbalagama. A co-owner gave evidence at the spot and pointed out several new springs on the field of which one spring on the south of the field was a large one. This field is downstream of the Samanalawewa and when the reservoir was impounded these springs had appeared and the spring on the south of the field washed a sizeable section of paddy plants. Even at the time of my visit the springs were active. This field is at the bottom of a ravine and about two kilometers away, as the crow flies, from the left bank of Samanalawewa. There is a saddle on this left bank and this field among many others downstream responded to the impounding. The left bank has not received much attention all these years. Is there underdrainage from the left bank of the Samanalawewa reservoir? It may not be safe to ignore the left bank. In fact one "tolerable leak" is even today to be seen close to the toe of the left bank.

188. So far as the appurtenant works go they have been carried out after adequate investigations and studies. These are briefly described below:

The overflow chute spillway was to be founded on competent rock composed mainly of biotite gneiss and garnetiferous biotite gneiss. Small zones of charnockite, granulite and limestones would also be encountered. The diversion tunnels had to be driven through similar rocks and in satisfactory tunnelling conditions.

The low pressure tunnel located at a depth of 70 - 250m below the ground surface would be driven in pre-Cambrian metamorphic rocks comprising gneisses, granulites, quartzites and crystalline limestone. The strata dip at 30°-60° towards the north-east striking almost perpendicular to the tunnelling conditions, with little or no water inflows. A number of fault zones cross the tunnel route. The total length of the tunnel affected by these fault zones would be about 160m. The tunnel lengths affected by individual faults might vary between 10m and 30m. Water inflows up to 100 l/sec would be encountered in these fault zones and other water bearing zones such as highly jointed quartzite and heavily jointed limestone.

The intake area of the low pressure tunnel was composed of interbedded biotite, garnetiferous, quartzitic and charnockitic gneisses. Here the overburden consisted of sandy-clayey soil and weathered rock debris of thickness varying from 3-13m. This zone is underlain by a 13-14m thick layer of jointed, slightly weathered rock. Towards upstream and downstream of this site, the thickness of the overburden increases to 30m.

The surge shaft was excavated in slightly jointed fresh gneisses and granulites.

The penstock is located not on the route proposed by the Russians but on the steep slope of escarpment mainly composed of biotite, garnetiferous and charnokitic gneisses. This was a good change. In the top portion of the slope, the bed rocks are overlain by a 3-5m thick talus-eluvial layer. Preserved bed rocks are exposed at the bottom portion of the slope.

At the power station site the ground gently slopes from the foot of the escarpment to the Katupath Oya. The site is underlain by gneisses and granulites. The bed rock has a foliation dip of 35°-45° towards the north. The area is overlain by shallow alluvial deposits and weathered rocks. Outcrops of gneissic rock are found on the site.

189. The reasons for not investigating the right bank are not tenable. In my view both banks should have been investigated before construction started. The investigation works on the saddles proposed by Gibb in 1984, EW1 in August 1985 and Nippon Koei in April 1986 are now built into the Lot II contract. Thorough ~~in~~vestigations with respect to watertightness were proposed, intended to be done but unfortunately not done. Saddle treatment is included in the Lot 2 contract but not defined. In the meanwhile the Lot 1 contract had been already entered into on 5 February '87 and the works had started. In this situation the Lot 2 contract was in effect

foreclosed. My finding is that adequate investigations aimed at ensuring watertightness of the two banks, essential components of the Samanalawewa reservoir, had not been done. The current location of the dam and reservoir and the design of the Samanalawewa dam have been based on adequate geological and hydrological studies and investigations. No investigations have been done on the Saddles of the two banks and adequate construction designs to ensure the watertightness of the reservoir could not be drawn up.

Chapter VIII

TERM OF REFERENCE (c)

190. Under this term of reference I am required to report on the present condition of the said dam and reservoir, including the causes which led to the reported leakage the effects and implications thereof and in particular whether there has been any negligence in the design and construction of the dam and reservoir.

PRESENT CONDITION OF THE DAM AND RESERVOIR

191. Before I deal with this subject perhaps it will be useful to outline the contractual arrangements. For tender design Balfour Beatty made a presentation. This was reviewed by EW1. The proposal for an Anglo-French combine to implement the Project did not work out as the French Partners decided not to join and Balfour Beatty too dropped out of the combine. Eventually the following arrangements were worked out: Joint Venture Samanala (JVS) comprising Nippon Koei and EW1 to be consultants and Supervisors. Alexander Gibb & Partners were appointed Design Engineers. CECB who originally were to be in the role of Co-Consultants were appointed to assist JVS with staff. The construction contractors were to be Kumagai-Hazama-Kajima Joint Venture (KHK) comprising Kumagai Gumi Co Ltd, Hazama Gumi Ltd and Kajima Corporation all of Japan. The decision to start the project was taken on 01.02.1986. The other relevant dates were:

1. Civil works - Lot 1 : Diversion Tunnels etc.

Contract awarded	-	05.02.1987
Commencement	-	01.03.1987

Lot 2 : Dam & Structures

Contract date	-	26.10.1987
Engs.'commence order		25.11.1987

Funding Sources for Lot 1 and 2 :OECF & CEB

(Overseas Economic Co-operation Fund)

Lot 3 : Waterway & Power House
Contract Date - 24.12.1986
Work commenced on - 23.01.1987

Funding Sources - UK Outright Grant, Commonwealth Devlp. Corporation (CDC), Lloyds Bank & CEB

2. E/M works - Lot 4 : Contract Date - 14.01.1988
Contractor - Mitsui & Co Ltd (Japan)

Funding Sources - OECF & CEB

3. Electrical Works- Lot 5 : Contract Date - 21.11.1986
Effective Date - 24.12.1986
Contractors - GEC Turbine Generator Ltd.

Funding Sources - UK Grant CDC, Lloyds Bank & CEB

4. Mechanical Work - Lot 6 : Contract Date - 10.12.1987
Contractors - Sumitomo-Mitsubishi-Kurimoto-Marushima Joint Venture(Japan)

Funding Sources - OECB & CEB

5. Supervision Contract Lot 6 : Effective Date - 08.01.1987
Contractors - Joint Venture Samanalawewa (Nippon Koei Co (Japan) Electrowatt Engineering Services (Switzerland).

Funding Sources - OECF & CEB

6. Design Engineering Contract Lot 6 : Contract Date - 21.11.1986
Effective Date - 24.12.1986
Contractors - Design Contract - Sir Alexander Gibb & Partners UK with Engineering & Power Devlp. Consultants Ltd

Funding Sources - UK Grant, CDC, Lloyds Bank, CEB

7. Local Contracts - Lot 7 : Local contractors

8. K V Line etc. - Lot 8 : Contract Date - Aug. 1988
Contractors - Cogelex Alsthom (France)
Consultant - Eubank Preece UK & Inter.G (France)

Project milestones of the dam and cut-off works

Decision to start	01.02.86
Award of contract Lot 1	05.02.87
Commencement of contract Lot 1	01.03.87
Award of contract Lot 2	26.10.87
Commencement of contract Lot 2	25.11.87
Diversion tunnels, Lot 1 completed	25.04.88
River diversion	17.05.88
Main cofferdam construction commenced	20.07.88
Main cofferdam construction completed	19.10.88
Spillway excavation completed	.01.89
Rockfill dam embankment commenced	22.02.89
Main dam foundation excavation completed	.03.89
Main dam core filling commenced	07.06.89
Core filling topping off	06.11.90
Right bank grouting adits excavation completed	.03.89
Concrete works grouting gallery commenced	04.89
Concrete works grouting gallery completed	02.90
Concrete lining of adits commenced	10.88
Concrete lining of adits completed	06.89

Right bank cutt-off works, adits access, grout curtain adit :

Adit	Excavation commenced	Completed
Da	27.07.89	14.10.89
Db	16.10.89	03.02.90
Ab	01.12.89	04.04.90
E	22.12.89	05.05.90
G	20.01.90	05.06.90
F(bc)	17.06.90	09.90
H(bf)	17.06.90	01.11.90
H(fi)	27.06.90	01.11.90
I(fi)	27.06.90	01.11.90

192. It has been agreed on all sides that the dam has been well constructed and bears a neat finish. But there is danger lurking in the immediate right abutment. There is a calcareous (CAL) zone, as well as heterogenous rock and highly weathered areas. A grout curtain reaching impervious rock which is not at very great depth here, could be a good safety measure.

193. As for the reservoir I would say the right bank is in bad shape and the left bank needs watching. The Lot 2 contract was entered into on 26.10.87 and work commenced on 25.11.87. It is not within my terms of reference to examine in any detail the contracts entered into among the several parties except generally insofar as is relevant to my inquiry. On the contracts entered into Sir Alexander Gibb and Partners of England were the Design Engineers, JVS (Joint Venture Samanalawewa) comprising Nippon Koei Co Ltd of Japan and Electrowatt Engineering Services Ltd (EW1) of Switzerland were the Consultants and Supervising Engineers while CECB had to assist with professional and technical staff. The clients were the Ceylon Electricity Board (CEB) and the contractors were Kumagai - Hazama-Kajima of Japan (KHK).

194. The following action was proposed during construction to resolve right bank problems which were anticipated due to the presence of deep weathering, cavities with soft material and low groundwater tables:

(a) Additional drilling work with a view to establishing the limits for the grout curtain.

(b) Further investigations for the saddle area, principally on the right bank.

The uncertainties regarding watertightness of the right abutment were also expected to be dealt with by driving four grouting/investigation adits at different elevations into the rightbank along the dam axis.

Details of Right Bank grouting/investigation adits along dam axis were as follows:

Adit Invert El. (m)	Design portal Chainage along dam axis (m)	Excava.chainage(m) End	Start	Excav. depth (m)	Gradient	Date Start	Finish
A 463.40	550.50	652.60	550.50	102.10	1:100	03.09.88	04.10.88
B 436.63	522.03	652.60	518.47	134.13	1:100	28.11.88	27.01.89
C 410.63	468.16	681.85	430.50	251.35	1:100	17.05.88	22.04.89
D 384.63	415.82	727.66	383.30	344.36	1:100	March 88	26.07.89

Excavated Diameter - 4.10 m
Finished Diameter - 3.50 m
River Bed Level at Dam Axis - 363.5 m
Total Length of Adits excavated - 831.94 m

The Adits were placed between El 463.40 and El 384.63 invert levels and spaced at an average of 26.25 m. The excavated diameter was 4.10 m and the finished diameter was 3.50 m. Adit A was driven to 102.10m. This adit was placed almost at dam crest level 463.50m and was the shortest.

The excavation in Adit D which was the lowest revealed -

(i) A cavity on the invert between chainages 645m and 649m.

(ii) A fault zone slickensided highly to completely weathered between chainages 650m to 685m.

(iii) A fault, tectonic breccia completely weathered between chainages 685m to 720m.

The rock mass from 400m to 645m was very good. The impure crystalline limestone was found slightly weathered to fresh.

Adit C the next lowest when excavated revealed -

(i) A cavern in the calcareous formation, where 40 m^3 of concrete was pumped, encountered at chainage 602m.

(ii) From chainage 602m to 655m a completely weathered micaceous schist and completely weathered tectonic breccia were encountered. The rock mass from chainage 655m to 682m was completely weathered.

These features emphasized the need for further investigations in the search for the limits of the grout curtain. Adits D and C were then further excavated to chainages 720m to 727.4m and from chainages 650m to 682m respectively.

Four exploratory boreholes D1, D2, D3 and D4 were drilled from the extremity of Adit D before access was lost to the adit with the commencement of the dam foundation excavation. All these boreholes were drilled 50m in length. D3 was drilled parallel to the tunnel axis from the dead end face at chainage 727.6m. D4 was drilled at chainage 726.4m, 62 degrees downwards from the horizontal in the direction of the tunnel axis towards the face. Boreholes D1 and D2 were drilled at angles 37 degrees and 30 degrees to the horizontal on either side perpendicular to the tunnel axis at chainage 721.1m. These boreholes indicated fractured rock very pervious in nature. Piezometers installed in the three inclined boreholes read a flat groundwater level of 379.0 m. Later borehole D5 was done from Adit D up to 180m to reach the lowest depth investigated.

Further drilling investigations were done from the surface of the right bank saddles and downstream of the dam on the right bank. In all 27 boreholes were done the deepest being drilled up to 230m and the shallowest drilled up to 50m. A total of 4806.15m of core drilling was done on the right bank with permeability testing below elevation 460m.

The most striking features of these boreholes were the low groundwater table around elevation 380m, deep weathering, high permeabilities recorded at depth and a secondary cement associated with small cavities. It was decided to deal with these adverse features with a cutoff grout curtain.

One borehole GW19 was done to a depth of 200m in the left bank. No serious problems were encountered. Piezometers were installed in all the boreholes except D1 to D5 and EW14. I annex Appendix 7 a summary of the Additional Investigation Drilling.

194. On the rather dismal results of core boring and piezometer readings JVS and Gibb decided on a grout curtain. The grouting gallery was to be positioned between 396m to 390m. The chainage to be grouted was 1315m. The curtain was to run through saddles 1, 2 and 3 and saddle sections of areas A and B. At its depth it was expected the curtain would reach GRA3 but a portion totalling 190m was expected to hang. GRA3 though the most impervious of the rock layer material was fractured at the faults. There were many dislocations. Gibb supported 68,000m of grouting going to a depth of 150m. Mr Vernon Pereira, Geologist of CECB seconded for service under JVS suggested a deviation of the grout curtain between two points but this was summarily rejected and he was reminded that the contractual rights were vested in JVS. One of the main problems was JVS had no equipment to handle grouting in excess of 100m depth. The grouting resulted in nearly 430m hanging sections. The particulars were as follows:

Chainages expected be hanging		Chainages of curtain hanging factually	
80 - 130m -	50m	0 - 40m -	40m
310 - 360m -	50m	110 - 180m -	70m
710 - 750m -	40m	430 - 500m -	70m
900 - 950m -	50m	620 - 870m -	250m
Anticipated hanging	**190m**	Factual hanging	**430m**

The revelations emerging from the grout curtain operation would have been certainly baffling. The grout curtain as decided upon followed a tortuous course taking off from where the dam right abutment curtain ended. A grouting gallery at a lower level may have helped to reach the more impervious areas of GRA3 which dipped away unco-operatively into the hillside. Seven boreholes were driven to a depth of 180m up to EL220 from the grouting gallery but what was revealed was a situation of fractured rock, solution paths, cavities underneath the grout curtain and a gloomy prospect that drainage under and on the sides of the curtain may result. A widespread underground aquifer whose geometry was unknown added to the gloomy picture.

The grout curtain may it be noted was projected as a continuation of the grout curtain installed as a cut-off, curtain to protect the dam. I will describe the grouting operations in the words of Mr Post and Mr Londe:

"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 sensu was designed to cross the privileged irregularly developed solution features connected by open tensional discontinuities and limited to a practical depth of 100-120m 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 200m.

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 10m 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/1.m in the stage from 25-3m depth.

Only very few spots have been grouted above the adit up to El 424 and rarely to El 460 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 (5m stages) but only 25 check holes have been carried out for a 1,300m long curtain (1, 555m 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 2.00m (quaternary holes). The wider openings have been sealed but it has not been proved that the smaller fractures (2mm) have been sealed, and if they are closely spaced, they may give high Lugeon values. The curtain length from chainage 0 to chainage 1180 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 180m below the adit (El 220 approximately), showing that fractured rock and solution paths or cavities still exist at these depths, even below 50m 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 430m compared to less than 200m estimated before grouting)."

The 1315m grout is not tied to impervious rock at the extreme end and it is not tightened in an impervious boundary. Nearly one third of it is hanging. The curtain may have closed some of the 'privileged paths', the wider openings and cavities lying across its area but even so not effectively. It is clear it has no effect on the seepage control in the right bank ridge. At the second impounding the surface piezometers and piezometers in the right bank cut-off adit, rose quickly responding to reservoir water levels. We must also content with the ubiquity of the aquifer presence in the dam site area and even beyond. The aquifer is very pervious and must be inferred to contain preferential leakage paths within its mass. The burst of 22 October 1992 and the reaction of the piezometers and the reduction of a spring close to Killakandura confirm the great extent and permeability of the aquifers through tensional fractures found at great depth.

The reservoir is functioning today as a run-of-the-river scheme. Project consultants and the review panel have indicated that -

"(i) there is no possibility of the dam, or the right bank failing and therefore there is no risk of a catastrophic disaster causing damage to people and property downstream of the reservoir.

(ii) if the groundater level in the right bank ridge exceeds that at which the water burst occurred on 22 October 1992 at some time in the future then similar bursts in other areas are likely.

(iii) Additional water bursts, which being alarming when they occur, will not compromise the safety of the dam and reservoir, however they are likely to result in increased losses and this would not acceptable on economic grounds."

The situation as adumbrated calls for urgent remedial action. JVS and Gibb in reporting this pronouncement have rightly added

"It is therefore imperative that measures are taken to ensure that the groundwater level in the right bank is maintained well below the level at which the water burst occurred (EL 438m) so that the risk of further instability and hence increased leakage is minimized. This must be achieved without increasing the current water losses from the reservoir and additional drainage measures are therefore not a long term option."

According to JVS and Gibb groundwater control could be achieved by adopting one of two approaches:

(1) Operate the project as run-of-the-river scheme with an additional spillway - a second spillway to ensure that reservoir levels remain below EL438 during floods. The cost of this second spillway will be very high.

(ii) Create a positive cut-off on the right margin of the reservoir by either extending the grout curtain "agreed by all parties to be technically impractical" or constructing an upstream blanket over the areas of reservoir water ingress to control leakage, and hence groundwater levels, to less than the downstream requirements at Full Supply Level. The blanket could be constructed either in the dry, using conventional techniques, or in the wet by tipping or dumping. I will discuss the remedies in the next chapter.

196. The first trial impounding was begun on 02 June 1991 and continued till early 1992. The grout curtain across the saddles of the right bank was still in the process of being installed. When the water level reached 402m with fluctuations around 396m and the river water level was 399.6m and the GWL was 394.5m a spring appeared 300m downstream of the dam toe along the right bank with a small overburden slip. This happened on 11 June 1991. By 11 July 1991 the discharge was 21 l/s and by 10 October 1991 the outflow had reduced to 12 l/s ~~when~~ the groundwater level (GWL) was 398m and the river water level (RWL) was 399.50m.

The grout curtain was completed in early 1992 and a second trial impovnding was begun in June 1992. The GWL was 378/380m. Before that Dr

A N S Kulasinghe head of CECB advised opening of the piezometers on 12 May 1992. The JVS ignored this request and instead shut up borehole D3 in the abandoned adit D which was discharging water at about 20 l/s. This act no doubt contributed to the build up of water in the right bank and more importantly to the push up of the pore pressure. This shutting up certainly would have aggravated the increase of pore pressure in the hillside.

197. And then came the spectacular burst of 22 October 1992 when the water level at the reservoir was 439.51m. At 13 00 hours on this day there was this sudden massive blow-out about 300m downstream in the same area where the earlier spring had appeared on the right bank. I have already described the blow-out. With the blow-out all piezometers dropped. On October 22/23 the piezometer levels dropped to 420m and later by 29 October the level was 416. Today the discharge is running at a steady 2 m^3/s. The grout curtain is said to have blocked "the privileged paths" but that was not effective to stop the blow-out. The path taken through the hillside by the water discharging at the portal of this blow out is not identified but one must not discount the posibility that boulders that would have been held up at the initial stages by the pressure of the flow may drop in the way of the water causing it to take a diverted route. If another burst occurs then the exit portal could be elsewhere on the right bank. It is quite likely that several leakage water paths are converging at some point inside the hill and emerging from the portal on the hillside. There is reason to assume that the leakage path around the right abutment is connected to the aquifer and closer to the dam than at first thought. In addition to the natural drainage to ensure the safety of the dam a second drainage system as a second line of defence in case of a blockage of the natural leakage conduits would be in the best interests of the safety of the dam.

The banks of the reservoir have yet to be rendered safe. Apart from leaks, landslides cannot be ruled out as the stability of the hillsides would undoubtedly be impaired by these occurrences. The right dam toe too could be in jeopardy should Mother Nature, to use an expression which some of the witnesses used to describe the vagaries of natural phenomenona, stage something spectacular.

198. I will now turn to the question whether there has been any

negligence in the location, design and construction of the dam and reservoir. The concept of negligence has been succinctly explained by the well known jurist John Fleming as "conduct falling below the standard demanded for the protection of others against reasonable risk of harm. This standard of conduct is ordinarily measured by what the reasonable man of ordinary prudence would do in the circumstances. It is impossible to formulate precise rules of conduct for all conceivable situations. In order to ensure a high degree of individualization in the handling of negligence cases, the law has adopted an abstract formula, that of the reasonable man, and has left to the jury, or to a judge in their stead, the task of concretising and applying that standard in individual cases."

199. The general standard of conduct, expected by the law is a complement of the legal concept of 'duty' as contemplated in the answers to the questions 'was a duty of care owed to the party complaining ? If so what precisely was required to discharge this obligation ? The reasonable man of ordinary prudence is the central figure in the formula. In order to objectify the law's abstactions like 'care', 'reasonableness', 'foreseeability', 'the man of ordinary prudence' a concrete model of the standard required has been devised. As the embodiment of all the qualities expected of the good citizen, not exactly a model of perfection, what would a reasonable man of ordinary prudence do in the circumstances of the case? the reasonable man of ordinary prudence is described sometimes in colourful expressions. In England he is the man in the Clapham omnibus or the Bondi tram. In America he is the man who takes the magazines at home, and in the evening pushes the lawn mower in his shirt sleeves - see Hall v Brooklands Club [1933] 1 K.B 205 at 224. In Sri Lanka we apply the standard of the reasonable prudent man. In special cases where a standard of skill and experience is expected from persons of a profession the test applied would be the standard of the average member of the profession. In the case of a Consultant Engineer the question asked would be 'what would a Consultant Engineer of ordinary prudence have done in the particular situation ?' An error of judgment would not be classed as negligence. In the specific case before me, the question would be 'what would a Consultant of ordinary skill and competence have done ?' Conduct inviting criticism or rigid attitudes in regard to contrary opinions do not per se fall within the pale of negligence. In the location of the dam and the reservoir and

in designing the dam sufficient care had been excercised. The mistakes that were made in regard to the reservoir though open to severe criticism cannot yet be regarded as negligence. Re location of the reservoir there can be no question of negligence. The same view applies to the design and construction of the dam. The next question is - has there been negligence in the design and construction of the reservoir ? There have been mistakes, and costly ones at that, which have crystallized in hindsight. Where designs and construction are concerned, where differences of engineering opinion are involved and contrary views have been expressed, what has turned out in execution to be a mistake will not be called negligence. In a contract, such a mistake could fall into the class of breach of contract. In the Samanalawewa Project there have been contrary views expressed, omissions and costly errors but none of them culpable as negligence.

Chapter IX

REMEDIAL MEASURES

200. I will now turn to the remedial measures to arrest the situation and other measures necessary to prevent occurrences of a similar nature in the future. This is item(d) in my terms of reference.

The reservoir today is being operated as a run-of-the-river scheme. With appropriate remedial measures the Project as planned could be realised. In discussing remedies, it would be useful to outline some features of the reservoir:

(i) Before impounding, the groundwater level recorded in the right bank was almost flat at about EL/380 m and it fluctuated in response to changing river levels. These responses were most likely to have been due to the ingress and egress of water from the river and therefore the ingress paths would be around EL/380 m in the river channel. The river bed is at EL/380 m.

(ii) Three major faults, F-1, F-2 and F-3, intersect in an area of the reservoir between 700 m to 1700 m upstream of the dam. A fault seen in the area of the water-burst also trends into this section. These faults are probably linked to the leak paths.

(iii) Chemical analysis of water samples taken for water quality assessments have shown that groundwater in areas of intact rock and slow groundwater movement (Area A) have a high sulphate ion (SO4) content. Reservoir water typically has a low sulphate ion content. Therefore groundwater with a low sulphate ion content is likely to be connected to the reservoir by privileged paths along which there is a relatively rapid flow of water. Groundwater of this type has been recorded at the place where the water burst occurred and at other places on Kalunaide Ara. All these points are apparently associated with the faulting that intersects in the reservoir in the area between 700 m to 1700 m upstream of the dam.

201. When constructing the right bank grout curtain, high grout takes were recorded at the bottom of the grout curtain in areas intersecting the faults F-1, F-2 and F-3. A number of piezometers were installed 180 m below the bottom of the grout curtain in these zones and the results recorded since impounding began indicates that high permeability zones exist, ungrouted, below the grout curtain.

From these facts it is possible to infer that the main zone of the water ingress probably exists on the river bed between 700 to 1700 m upstream of the dam. However it is possible that other zones of ingress do exist, but their effects are masked by the size and efficiency of the main zone.

202. I have referred earlier to the divisions of the right bank ridge into area A and area B. In area A the ground water is stable around 400(except along the inclined fault where underdrainage occurs) and is not influenced by the river's fluctuation. But in area B the groundwater level is flat and the piezometers respond to the river level within a short time. There is no hydraulic lag between the different piezometers upstream and downstream.

The main dam is not in immediate jeopardy but the idea of the reservoir is lost if its waters are drained downstream. We require a water-tight reservoir but as it is doubtful that this is attainable our interest should be to control the leaks and see that we have no more than tolerable leaks.

203. Hence remedial measures are urgently needed. Nature has provided us with the answer with a natural drainage channel which has reduced the groundwater level pressure very efficiently. We should take advantage of it and improve its stability at its exit by upgrading the outlet portal by excavation and stabilization of the slope above and round it. This should be the main drainage system. To meet the eventuality of a blockage of the natural leakage conduits, a second internal drainage system should be installed. The dam safety over the long term can be ensured by having an effective control on the seepage paths by additional drainage. The most appropriate remedial measures, both technically and economically, lie in the direction of additional drainage of the right bank ridge.

Internal drainage regulated in multiple tiers and controlled so as to reduce the pore pressures in stages and reduce the flow velocities in the fissures, will stabilize the hillside in a very economical way. Internal drainage will keep the right bank ridge from becoming saturated and bursting its banks.

204. A grout-curtain sub-parallel to the flow of the river and along the strike of the rock will bar the influx of reservoir leakage into the critical zones. The grout curtain will insulate the internal drainage system and should be provided in two stages:

(i) From a grouting adit at approximately EL 450 m up to a bottom gallery at EL 380 m.

(ii) From the bottom gallery at 380 m down to varying depths as required.

This alignment of the grout curtain will make use of the delineation of the sound granulitic band GRA1 and will be tied to a sound rock formation at the downstream end.

Two alternative drainage systems have also been suggested by Mr. G. Post and P. Londe:

"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.

"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....

"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 Kaluneida Ara are far enough from the dam toe.

"All the drain holes should be equipped with special PVC slotted pipe 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)may be placed inside the first one."

If, still, more water is being lost than we could afford to lose within economic limits, then pumping back the water to the reservoir can be resorted to. An observational approach monitoring every stage is very necessary. It is without doubt a time consuming process.

205. Sealing the leaks with a wet blanket would be the best method if we know where the points of ingress are. But we dont. Further the best techniques would be necessary. Sri Lanka would be the first to try this method. Although wet blankets have been tried in various countries, on no occasion has it been tried on a reservoir whose banks are steep. The flow velocities of the Walawe Ganga are slower than those of the Belihul Oya. Clay dumping may be efficient on a flat bottomed reservoir with flatter hill banks than the steep banks which flank the Walawe Ganga on its right.

Mr. G.R. Post and Mr. P. Londe two well known Engineering geologists in their report of February 1993 said -

"For the dam safety over the long term it is recommended that the seepage flow in the dam foundation be better controlled by additional drainage, and means to render the right bank ridge more watertight"

This was followed by the panel of celebrated experts comprising Messrs J.B. Cooke, P. Londe, A.H. Merritt and G.R. Post issuing Report No 2.

206. Attempts to identify the points of ingress have failed. When the blow-out took place all the peizometers dropped drastically almost 25 m down to El. 415 approximately. When the outflow at the burst reduced to $2m^3$/sec then the groundwater level rose by 6 to 7 m. A spring close to Killekandura Ara about 3 km downstream on the left bank significantly responded to the burst. All this demonstrates the great extent and permeability of the aquifers through tensional fractures found at great depth. The aquifers are widespread and extensive and straddle either flank of the reservoir and Walawe Ganga on the downstream side of the dam. These aquifers provide preferential leakage paths. The traction force to suck in the dumped material to the points of ingress may not be sufficient. The blanket may not adhere to the slope of the hill where the points of ingress are on the sides.

207. The four member expert panel recommend a wet blanket but two of their members had earlier suggested additional drainage:

"Additional drainage and means to render the right bank ridge more watertight". The additional drainage was meant to supplement and if necessary to be an alternative to the nature-made internal drainage emitting from the portal of the burst just in case it gets blocked. The "means to render the right bank ridge more watertight" is no doubt a reference to the wet blanket recommended by them along with Dr. J.B. Cooke and Dr. A.H. Merritt. It is noteworthy that Mr. Post and Mr. Londe in Report No 1 recommended internal drainage and the means of rendering the right ridge watertight as concurrent remedies.

208. Wet blanketing has been done at the Tarbela dam in Pakistan. But there the reservoir was flatter and the sink holes were identified and the treatment was for the sink holes on the bed of the river. Here in Samanalawewa we are hoping to blanket a 1000 metre stretch of the steep hillside on the right flank of the Walawe with no identification of the points of ingress. There are no precedents for this type of blanketing the steep slopes of a hill side. If it is effective it would create history and I am not averse to that. Questions have been raised on whether the wet blanketing could be successfully accomplished. Questions like the transferred velocity of the river flow in relation to the silt load and the application of Stokes' law have been raised. But I would allow for efficiency in the operation. The four member expert panel(J.B. Cooke, P. Londe, A.H. Merritt, G.R. Post) in their report No. 2 of February 1993 state as follows:

"The wet method can be described as dumping impervious material underwater. The water level would be the level which enables powerplant operation during placement. Above that elevation, dry placement is appropriate to thicken the existing thickness of the saprolite blanket, were it considered necessary. Little dry placement is anticipated to be required.

"The mechanism of sealing by the wet method for the open jointed hard rock, where it is not covered by an adequate thickness of saprolite, is that leakage into the rock mass will draw the fine material into fissures in the rock mass until it is sealed. Strength of underwater placed material is not needed. The important consideration is to use fine material such as the saprolite. Some sand sizes are, of course, desirable. The important material is minus #200 mesh. Occasional rocks, but not a load of rocky material, are acceptable.

"There is much precedent for sealing by underwater placement by bottom dump barge, by end dumping from trucks, and by dredging. For Samanalawewa creating a bench along the abutment seems to be the appropriate method. This will take more volume, but would be substantially the lowest unit cost method and the method that could be more quickly mobilised for.

"During the wet season before the blanket has been in place, additional drainage of a provisional nature should be provided. The existing access gallery D(a) is the appropriate location for these drains. All drains should be fitted with a closure valve.

"The drainage adit suggested in Report No 1 is no longer considered necessary, because new local slides in the overburden can be accepted."

(Report No. 1 was issued also in February 1993 by two members of the same panel — ~~Mr.~~ G. Post and P. Londe).

Even so the blanketing carries a shot-in-the-dark factor because we have not identified the points of ingress and a thousand metre stretch of steep hillside will involve a staggering cost and it will not end there. Despite identification of the sink holes at Tarbela, blanketing is going on perennially. The expenditure will be massive and keep snowballing as in the observational approach the blanketing will not be a once-and-for-all job. Blanketing will have to be carried on to seal the leakages as they are detected. For the above reason in my rating blanketing in these conditions should not be a remedy unless the points of ingress have been identified.

209. My recommendation is that internal drainage with the grouting suggested should be tried first while at the same time maintaining the internal drainage that nature gave us. In addition we can look to Mother Nature handling the blanketing for us at no cost. The sediment that the Walawe brings along can do it over a much longer time with no cost. My recommendation is that an internal drainage system be used in addition to the existing natural drainage. This should be done with the grouting I have earlier recommended. If the reservoir loses more water than we could afford to lose, arrangements should be made for pumping back the water collected downstream. If all this fails my choice would be to run the reservoir as a run-of-the-river project in preference to trying the costly wet blanket with unidentified points of ingress. If the points of ingress are identified then sealing them off with a wet blanket could be the remedy.

Some mention of the low level diversion tunnel is necessary. There were two tunnels. One of them has been plugged and is out of use. The other has had its outlet reduced so that it can pass only 70 m^3/sec. We have only the spillway and this reduced tunnel to cope with a maximum possible flood which may overtop the dam. The evacuation facilities should be therefore enlarged. The second tunnel should be reopened and harnessed for activation in a massive flood.

Chapter X

GENERAL REPRESENTATIONS

Here I will briefly refer to certain matters raised in evidence before me not strictly within my terms of reference but deserving mention.

210. I will mention the complaints of the farmers first. They complained that their buildings were affected by the explosions during quarrying. Their fields were affected specially during the impounding and so were their drinking water wells. They are very sceptical about the success of the scheme and apprehend landslides any floods. We understood that compensation was paid for the damage suffered but the villagers remain dissatisfied. They would have preferred if if the scheme did not take off and they were left free to do their gemming, as of old, in the reservoir bed. Only if the Project is seen to a complete success will the fears of the villagers be allayed.

Another Group that testified before me were men who had worked on the Project. They alleged corruption and dishonesty in the use of materials and equipment but no proofs at all are available on these matters.

211. The Environmental Foundation Limited made representations before me. Apart from asserting that the Project has had an adverse impact on the environment they also contended that an Environmental Impact Assessment report had not been obtained. Although the National Environmental Act No 47 of 1980 as amended by Act No 56 of 1988 was in operation, no regulations had been framed under section 23 Z. The regulations were published only in 1993 in Gazette Extraordinary No 772/22 of 24 June 1993. At the times relevant to the contracts relating to the Samanalawewa Project, there were no regulations in operation. Hence the operation of the Act itself was stymied. In regard to the environment itself no proofs of any sort were placed me. The legal officer of the Environmental Foundation Ltd who gave evidence before me could not speak of the engineering problems or the connected scientific questions with any degree of competence. The Foundation has no Engineer in its ranks and any engineering questions, I

understood, are handled on an ad hoc basis. No engineer or other expert had been commissioned by their Foundation to report on environmental questions of the Samanalawewa Project.

212. I would like to refer to some suggestions made by Mr. Gamini Samarasinghe who is not an Engineer but an innovative scientist with a world-wide reputation. He put forward a sealing device which he said could be used on Samanalawewa but in the obsence of any definite engineering analysis, I am unable to make any pronouncement on this. May be his method of sealing leakages may one day be developed for practical use.

Mr. Samarasinghe also said that if in the quarrying the extraction of metal was carried out in such a way that a natural bowl shaped hollow was left, it could have been used as a water tank where the pure waters of the Belihul Oya in its upper reaches could be impounded and distributed at least in the Balangoda District. It was a suggestion worth looking into but came far too late and, in any event, is outside the terms of reference.

Chapter XI

SUMMARY

213. Sri Lanka is divided into the Coastal Regions and Central Highlands. The Central Highlands are in the central part of the island. The Central Highland has for its southern limit a great escarpment called the Balangoda escarpment. The designation "Upland Platform" has been given to the rugged deeply dissected plateau lying between the escarpment and the lowland plain. The west part of the contact between the plateau and the plain is marked by a small escarpment. The Central Highlands rise some 1200m to 1400m above the level of the Upland Platform Region.

214. The Walawe is the third largest river in Sri Lanka and takes its source west of Balangoda in the Central Highlands Region. The river flows generally eastwards for about 50 km of its course in the deep gorge to the South-East and emerging from the hills at Uggalkaltota it turns sharply southwards and flows along the plain till it discharges into the sea at Ambalantota. The Belihul Oya takes its source from Horton Plains in the Central Highlands and runs in general direction South-East till it flows into the Walawe Ganga about 9 km west of Uggalkaltota. The Samanalawewa dam has been built about 250m downstream of the confluence between Walawe Ganga and the Belihul Oya. Both these rivers receive a heavy rainfall from both the South-West and the North-East monsoons and during the inter-monsoonal periods. The headworks of the Samanalawewa Project are at an elevation of 400m. The Diyawini Oya and its tributary the Katupath Oya flow generally eastwards and parallel to the Walawe Ganga. Where the Walawe Ganga is about 6m away from the Katupath Oya the difference in levels is about 300m and this difference in levels has been used to locate the power house on the banks of the Katupath Oya. Between the Balangoda escarpment, also called the World's End escarpment, and the Kaltota scarp is the Balangoda syncline. The rocks in this area belong to the Kaltota Formation.

215. The Samanalawewa reservoir has on its right bank three saddles numbered 1,2 and 4 which are 600m, 1220m and 2100m respectively from the dam. On the left bank of the reservoir there is saddle No 3 which is 700m

from the dam. The dam and its saddles are situated on the NNE limit of the regional Balangoda syncline and the power tunnel and power house are located on the SSW limit of the syncline. The power house is located at the foot of the Kaltota scarp on the banks of the Katupath Oya.

216. The Walawe Ganga at the dam site flows in a strike valley and the saddles are more or less across the strike of the rock. Local folding sub-parallel to the strike of the rock is present on the right bank of the dam. The dam is located on a site at the northwestern edge of the Balangoda synform on the middle peneplain of Sri Lanka between the Coastal Plain and Central Highlands. The foliation on the left bank dips generally 25°-35° while the foliation on the right bank dips 30°-60° into the hillside. These foliation angles result in an asymmetric vallpey with the left bank sloping at 30°-35° and the right bank sloping at 35°-45°. In the river channel the rock has a foliation dip of 5°-10° at the dam area. The predominant fault direction is NE to SW. East to West faults are also present. The faults are on the right bank saddles.

217. The weathering in the right bank is markedly deep. The entire area is criss-crossed with open joints, faults, fractures and solution cavities. Impure crystalline limestone dissolved by circulating groundwater and hydrothermal action has resulted in extensive subsurface waterways. An underground aquifer whose geometry is unknown is widespread and extends to the saddle area.

218. A series of studies of the Samanalawewa Project were made by different Institutions. They are:

(a) Irrigation Department Study in 1957 and 1958;

(b) Tudor Engineering Corporation Study entitled "Hydroelectric and Irrigation Project - Seven Virgins and Samanalaya" published in March 1958 (Washington);

(c) Samanalawewa Project Technical Report - May 1966 by the New York Engineering Consultants Inc. (ECI) of Denver;

(d) Samanalawewa Project for Development of Hydropower - Technical Report August 1973 by the Snowy Mountains Engineering Corporation (SMEC) of Australia in collaboration with the Mahaweli Development Borad;

(e) Samanalawewa Project - Detailed Project Report 1978 by Hydroproject Institute of Russia in collaboration with the Central Engineering Consultancy Bureau;

(f) Nippon-Koei Reconnaissance Report 1982 (Japan);

(g) Samanalawewa Hydro-Electric Project Technical Report - April 1984 Balfour Beatty Ltd, GEC Energy Systems Ltd, Sir Alexander Gibb & Partners, EPD Consultants Limited all of UK;

(h) Samanalawewa Hydro-Electric Scheme Review Report - Nov.1984/Jan. 1985 Electrowatt Engineering Services Ltd of Switzerland (EW1);

(i) Samanalawewa Power Project - Technical Report April 1985 by Central Engineering Consultancy Bureau (CECB).

As many as nine studies were made over a period of nearly 30 years. There were, it seemed, never ending studies reminiscent of the famous legal analogy of the mythical case of Jarndyce v Jarndyce described by Charles Dickens in "Bleak House". Studies were started and went on and started again and went on and after nearly thirty years when the construction got off the ground, the investigations were short by 450m of vital core drilling on the enigmatic saddles on the right bank of the Samanalawewa.

219. The Samanalawewa Project region is composed of pre-Cambrian high grade metamorphic rocks. In the Project area the rock complex is ascribed to the Kaltota Formation. The rocks belong to the following broadly defined units: Lower Charnoctite/Impure crystalline limestone, upper granulite charnockite and granulitic gneiss. Impure crystalline limestone occurs in thin layers of about 10cm thickness and in bands going thick and thin, the maximum thickness being in the range of 7m to 10m. In a particular area the calcite in the impure crystalline limestone was absent owing to alteration by hydrothermal action. In the dam foundation and in its neighbourhood caverns were encountered at depth of 0.1m to 0.5m during excavation. These caverns were caused by hydrothermal action.

220. Negotiations ended with contracts for the construction of the dam and its appurtenant structures as follows : Alexander Gibb and Partners of UK as Design Engineers, Joint Venture Samanalawewa (JVS) comprising Nippon Koei Company Ltd, Japan and Electrowatt Engineering Services (EW1) of Switzerland as Consultants and Supervisors, Kumagai-Hazama-Kajima Joint

Venture (KHK) as contractors and the Central Engineering Consultancy Bureau (CECB) to assist with staff. The decision to start was taken on 01.02.1986. Lot 1 contract to construct the tunnels and relevant works was awarded to KHK and became effective on 13.02.1987. The work was completed on 25.04.1987. Lot 2 contract to construct the dam and appurtenant structures was awarded on 26.10.1987 and the Engineer's commence order was issued on 25.11.1987 after approval was obtained from the Overseas Economic Co-orporation Fund of Japan (OECF) who were the funding agency for the Lot 1 and Lot II contracts. The contract to construct the waterway and power house was awarded to Balfour Beatty Corporation International Ltd of UK on 24.12.1986 and work was commenced on 23.01.1987. The construction of the diversion tunnels on Lot 1 contract was completed on 25.04.1988.

Under Lot II contract the river diversion was done on 17.05.1988. The main coffer-dam construction was commenced on 20.07.1988 and completed on 19.10.1988. The spillway excavation was completed in January 1989. The work on the rockfill dam embankment was commenced on 22.02.1989. The main dam foundation excavation was completed in March 1989. The main dam core filling commenced on 07.06.1989 and the core filling was topped off on 06.11.1990. The dam is 105m high with a crest level of 467.5m.

221. The Stratigraphy of the rocks in the area has been classified as follows:

1. Lower Charnockite Layer (CHA) : consisting predominantly of charnockite intercalated with impure crystalline limestone and granulitic gneiss.
2. Lower Granulite (GRA 1) : massive to faintly gneissic granulite 10 to 25m thick.
3. Calcareous Bed (CAL) : Impure crystalline limestone and charnockite with layer of granulite - marked by cavities and karstic caverns.
4. Upper Granulite (GRA 2) : Granulitic gneiss with hornblende - biotite bands and carbonate layers.
5. Upper Granulite (GRA 3) : Granulitic gneiss with a few hornblende biotite bands but no carbonate interlayers.
6. Upper Granulite (GRA 4) : Granulitic gneiss with frequent hornblende - biotite bands and impure crystalline limestone

beds.

7. Charnockite (CHA 2) : Predominantly charnockite alternating with carbonatic rocks;
8. Granulitic Gneisses (GRA 5).

Three faults were identified in the saddles on the right bank. Apart the faults discontinuities which are tensional fractures were identified in the topography as lineaments. Faults 1 and 2 are associated with saddles 1 and 2 and fault 3 with saddle 2. The rock in the area to the north of fault No 1 is less fractured than the rock to the south of it. The more compact zone is described as area A and the more fractured zone as area B.

222. For the investigations originally 26 boreholes (J1 to J26) for the dam site; 4 boreholes (J27 to J30) for the saddles (J27 on saddle 1, J28 and J29 on saddle 2, J30 on saddle 4 with a total drilling of 450m) and 4 boreholes (J31 to J34) for the quarry were planned. When J12 was drilled on the dam axis it was jammed. An additional borehole J35 was drilled slightly downstream of the dam axis. It was decided then to incline the dam axis by 20° on the right abutment end and downstream of the dam, instead of going on with the straight line dam axis originally planned. This slight deviation was useful because it would involve excavation of less weathered material. In order to investigate the new route four more boreholes (J36 to J39) were drilled. But J38 and J39 also were jammed. A grout curtain was installed off the right wing end of the dam and, being not well planned, it was taken along a meandering course and eventually however tied to good rock. So the right wing of the dam was secured. To meet the drilling requirements in cost and time the four boreholes J27 to J30 planned to be drilled on the saddles were eliminated, it was said temporarily, to be done during construction. So the parties went to contract without any of the four boreholes in the saddles being drilled. In the meantime it was found that adit D driven in the right bank to a distance of 100m could not be taken any further.

223. The dam itself however was well constructed and had a neat finish. The spillway and two diversion tunnels all on the left wing of the dam were also constructed. During construction four boreholes D1 to D4 were drilled from adit D and 18 boreholes EW1 to EW18 were drilled in the

saddle 1 and 2 areas and EW19 on saddle 3 in the left bank. The drilling showed karstified zones, impure crystalline bands, solution cavities, fractures and fissures - all indicia of underground drainage confirmed by piezometer readings. In area A the groundwater level was stable around 400 (except along the inclined fault where underdrainage occurs) and was not influenced by the river level fluctuations. In area B the groundwater level was flat (378.5 to 379.5) and the piezometers reacted to the river level within a short time lag.

224. To ensure watertightness of the reservoir a grout curtain 1315m long and 105m deep passing through the saddles and taking off from where the grout curtain of the dam ended, was installed. It was expected to position the bottom of this grout curtain on GRA3 which was rated as impervious rock. But GRA3 was dipped down the hillside and fractured at the faults and also at discontinuities caused by tectonic dislocations. The grout curtain even as planned was expected to hang at several chainages to a length of 190m (between chainages 90-130, 310-360, 710-750 and 900-950); but upon construction the grout curtain was found to hang in several sections to a total length of 430m (between chainages 0-40, 110-180, 430-500, 620-870). From the grouting gallery seven boreholes were drilled to a depth of 180m at EL 220m. The results showed that fractured rock, solution paths or cavities still exist at such depths. A total of 52,000 metres was grouted with 13,200 tons of dry cement making an average of 25 kg/m which was much higher than anticipated. It is not certain that ungrouted windows have not been left.

A trial impounding was carried out from June 1991 to the beginning of 1992. The maximum level attained was 402 with fluctuations around 396. The groundwater table was flat with generally less than a difference of 1.00 in head between the piezometers and with no development of any hydraulic gradients through the curtain. The right ridge was very pervious with an aquifer partially confined. On 11 June 1991 with river water level at 399.6 and groundwater level at 394.5 a new spring appeared 300m downstream of the dam toe along the right bank with a small overburden slip. On 11 July 1991 the discharge was 21 l/s. From the absence of change in groundwater response it was evident that during the next reservoir impounding the leakage would stay below acceptable value so far as the economical aspect of the project is concerned.

225. The second reservoir impounding was started in June 1992. Just before, the groundwater level was very flat and between 378-380. The reservoir level was raised in stages to attain the maximum elevation 439.51 on 14 October 1992. The elevation was 434.50 on 16 October 1992. The water of the spring which appeared at the first trial impounding became muddy during the night of 21 October 1992 and all the piezometers followed the river water level closely with one day time lag until 22 October 1992 at 1300 hours when a sudden massive blow-out occurred on the right bank near the spring referred to. The blow out triggered a landslide. Between 20,000 to 40,000 m^3 of earth was washed down. This was earth and debris from on old landslide. The access road showed gaping tension cracks and had to be closed in the interests of the safety of the public. The torrent of muddy water cascading from the burst settled to about 7 m^3/sec. Today the portal at the burst disgorges water about 2 m^3/sec and the water runs clear. The groundwater level is 419 and the river water level is 428.50. The flow from a spring at Killekandura Ara which appeared on 25 September 1992 on the left bank during the second impounding has now reduced since the blow-out of 22 October 1992. This spring is at an aerial distance of about 3 km downstream from the reservoir.

226. The behaviour of the dam is satisfactory and there is no immediate danger to the dam itself. The current location of the dam and reservoir was determined after adequate investigations and studies. The rockfill clay core dam was selected after weighing the merits and demerits of several types of dams. The appurtenant works were located after careful consideration preceded by adequate investigations and studies. So far as the reservoir itself is concerned though the dam is well designed and constructed no real attention was given to the banks particularly the right bank. Despite the existence of logs of boreholes drilled on saddle 2 by the Russians which revealed core losses, water losses, cavities and information from which underdrainage should have been inferred, and the knowledge that there were faults on saddles 1 and 2 the omission to do the 4 boreholes involving only 450m of drilling planned for the saddles was far from satisfactory. In fact there was no investigation of both banks prior to construction. The view that the saddles were not a problem was not a

sound one. JVS particularly did not appear to be alive to the complexities that lay in the subsurface of the saddles.

227. The present condition of the dam is satisfactory but the right bank investigations done during construction showed very great problems and every indication that there would be leakages. The 1315m grout curtain installed at a cost of Rs 2 billion has proved to be on the whole ineffective and has to be written off as a failure despite the fact that some of the privileged paths are cut off. The grout curtain has not fulfilled the objects with which it was installed at such great cost. There is underground drainage. The piezometers indicate that the groundwater level downstream responds to the river water level. The water in the reservoir is being drained downstream. Despite omissions and blunders however there was nothing to justify an inference of negligence.

228. Nature has shown us a very good remedy. From the natural drainage taking place it is clear that internal drainage is the answer. The natural drainage at the portal of the blow-out of October 1992 should be maintained and as a second line of remedial action an additional system of internal drainage introduced. The most appropriate remedial measure both technically and economically should be additional drainage downstream of the right ridge to prevent the pore pressures from building up in the right flank hillside. This will entail construction of a system of drainage adits with interconnecting sub-vertical drain holes. A suitable grout curtain in the right abutment near the right wing of the dam to cut off leakage through the CAL zone, heterognous rock layers and weathered areas and tied to impervious rock which lies at no great depth in this area would be a good safety measure. If despite these measures more water is being lost than it would be economical to lose in the interests of power generation, the water drained downstream could be pumped back. If still the dam function is a failure, it could be operated as a run-of-the-river project as safety of the public and their possessions come first. Further the second diversion tunnel should be reopened as a second low level outlet in addition to the first low level outlet which has been reduced and can emit water at only 70 m^3/sec. This is a necessary safety measure against a maximum possible flood overtopping the dam.

229. On the question of sealing with a wet blanket, my recommendation is that it should not be done without identifying the points of ingress. It must be remembered that the clay blanket procedure is continuous. It would have to be repeated each time leakage routes develop. Such leakage routes can develop even on the left bank of the Belihul Oya. Blanketing a whole area of 1000m or more without identifying the points of ingress carries a shotin-the-dark factor not to mention a continuous swell in the costs.

230. I annex as Appendix 8 a statement of the Principal features of the Samanalawewa Project including the Dam and its appurtenant structures, as Appendix 9 a Geological Map showing the Stratigraphy, Faults and Saddles and as Appendix 10 a map showing the Drainage Adits and the grout curtain on the right bank.

Chapter XII

ACKNOWLEDGEMENTS

231. I wish to express my appreciation of the assistance and co-operation which I have received in the preparation of this report. I am thankful to the Chairman and Officials of the Ceylon Electricity Board for providing me with the documents I needed and facilities for visiting the Samanalawewa dam site. I am also thankful to Messrs Alexander Gibb and Partners, Joint Venture Samanalawewa and the CECB for furnishing me with necessary documents and arranging for their representatives to give evidence before me at short notice. My thanks go out also to my Secretary Mr D W R M Weerakoon, Senior Deputy Director of Irrigation for the interest he took in helping me to hold the inquiry. I also thank Mr Samith de Silva, Senior State Counsel and Mr R L De S Munasinghe, Engineer/Geologist for the tireless work they did in helping me on with the recording of evidence.

The work of the members of the staff headed by Mr Sumith A Liyanage and Mr P P Anura, Chief Clerk was very satisfactory and they gave of their very best to meet the exacting demands I made of them. I very much appreciate the efficient work done by the two Stenographers Mr M H N Cassim and Mr T P Fernando and earlier Mr Ronald Peiris. They had to work long hours to cope with the tasks of transcribing proceedings recorded over 90 long sittings. I have also to thank Miss V Karunananda who measured up well to the arduous task of typing my report.

Finally there are the witnesses who gave evidence before me and produced documents. They attended sittings often at short notice and at very great inconvenience to themselves. I thank them all. I have given due consideration to all the evidence placed before me though I have not referred to witnesses by name except very occasionally.

J F A SOZÁ,
COMMISSIONER.
27 October 1993.

APPENDICES

1. Appendix 1 - Gazette No 738/11 of 28.10.1992 notifying appointment of Commission
2. Appendix 2 - Newspaper advertisement in Sinhala, Tamil & English inviting representations
3. Appendix 3 - List of representations/memoranda in response to advertisement
4. Appendix 4 - List of witnesses who gave evidence
5. Appendix 5 - List of Documents/Productions marked in evidence
6. Appendix 6 - Map showing Natural Regions of Walawe Ganga Basin
7. Appendix 7 - Summary of Additional Investigation Drilling
8. Appendix 8 - Schedule of Principal Features of Samanalawewa Project including Dam and Appurtenant Structures
9. Appendix 9 - Geological Map showing stratigraphy, faults & saddles
10. Appendix 10 - Map showing Drainage Adits & Grout Curtain

APPENDIX - 1

GAZETTE NO.738/11 OF 28.10.1992

NOTIFYING APPOINTMENT OF COMMISSION

ශ්‍රී ලංකා ප්‍රජාතාන්ත්‍රික සමාජවාදී ජනරජයේ ගැසට් පත්‍රය

අති විශෙෂ

අංක 738/11 - 1992 ඔක්තෝබර් 28 වැනි බදාදා - 1992.10.28

(ආණ්ඩුවේ බලය පිට ප්‍රසිද්ධ කරන ලදී.)

I වැනි කොටස : (I) වැනි ඡෙදය - සාමාන්‍ය

ජනාධිපතිතුමාණන් විසින් කරන ලද ප්‍රකාශන ආදිය

ජනා. කා. අංකය : පීපීඒ/6/එන්/177/92.

එල්. ඩී.- 65/44(iv).

ශ්‍රී ලංකා ප්‍රජාතාන්ත්‍රික සමාජවාදී ජනරජයේ ජනාධිපති රණසිංහ ප්‍රේමදාස මැතිතුමන් විසිනි

මුද්‍රාව

ආර්. ප්‍රේමදාස.

ජෝසප් ෆ්‍රැන්සිස් ඇන්තනී සෝසා මැතිතුමා වෙත,

ශුභාශිංසන පෙරටුවයි.

රත්නපුර පරිපාලන දිස්ත්‍රික්කයේ පිහිටි සමනල වැව ජලාශයේ දකුණු හා වම් ඉවුරුවල ජලය කාන්දුවීම් සිදුවී තිබෙන බැව් වාර්තා කර ඇති හෙයින්ද;

මෙහි මින් මතු සඳහන් කරනු ලබන කාර්ය සඳහා පරීක්ෂණ කොමිෂන් සභාවක් පත් කිරීම අවශ්‍ය බැව් මට පෙනී යන හෙයින්, ජනාධිපති රණසිංහ ප්‍රේමදාස වන මම ඔබගේ විචක්ෂණභාවය, සාමාර්ථ්‍යය හා විශ්වාසවන්තභාවය කෙරෙහි මහත් වූ භක්තිය හා විශ්වාසය තබමින් (393 වන අධිකාරය වූ) පරීක්ෂණ කොමිෂන් සභා පනතේ 2 වන වගන්තියේ විධිවිධාන ප්‍රකාර, එකී

ජෝසප් ෆ්‍රැන්සිස් ඇන්තනී සෝසා වන ඔබ, පහත දැක්වෙන කරුණු පිළිබඳව පරීක්ෂා කර වාර්තා කිරීම සඳහා, මගේ කොමසාරිස්වරයා වශයෙන් මෙයින් පත් කරමි:-

(අ) සමනල වැව ව්‍යාපෘති සම්බන්ධයෙන් මීට පූර්වයෙන් කරන ලද තිල අධ්‍යයන පිළිබඳව එකී වේල්ලට සහ ජලාශයට අදාළ භූ විද්‍යාත්මක අංශ හා එහි පිහිටීම සහ සැලැස්ම හා එම ජලාශයේ ජල පීඩන ප්‍රමාණය යන කරුණු ගැන විශේෂ සැලකිල්ලක් ඇතිව පරීක්ෂා කර වාර්තා කිරීම;

(ආ) එකී වේල්ලෙහි සහ ජලාශයෙහි වර්තමාන පිහිටීම සහ සැලැස්ම හා ඊට අනුබද්ධ වැඩ පිළිබඳ භූ විද්‍යාත්මක හා ජල විද්‍යාත්මක අධ්‍යයනයන් ඇතුළුව, ප්‍රමාණවත් විමර්ශන හා අධ්‍යයනයන් මත පදනම් විද යන වග;

(ඇ) සිදුවී ඇතැයි වාර්තා වී ඇති ජල කාන්දුවීම් ඇතුළුව එකී වේල්ලෙහි සහ ජලාශයේ වර්තමාන තත්ත්වය, එම කාන්දුවීම්වල බලපෑම හා ඊට අදාළ ගැටළු සහ විශේෂයෙන්ම එකී වේල්ල හා ජලාශයේ පිහිටීම, සැලැස්ම හා ඉදිකිරීම්වලදී යම් නොසැලකිල්ලක් සිදුවී තිබේද යන වග;

(ඈ) මෙම තත්ත්වය වැළැක්වීමට ගතයුතු පිළියම් ක්‍රියාමාර්ග හා මෙබඳු තත්ත්වයන් මතුවට ඇතිවීම වැළැක්වීමට අවශ්‍ය ධනාත් ක්‍රියාමාර්ග;

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තවද, ඉහත කී කාරණා සම්බන්ධයෙන් අවශ්‍යයයි ඔබට පෙනී යන සියලු පරීක්ෂණ පැවැත්වීමට සහ වෙනත් සියලු විමර්ශන කිරීමට එකී කොමසාරිස්වරයා වන ඔබ වෙත මෙයින් අධිකාරය හා බලය පවරන අතර, ඔබ සොයා දැනගත්තා කරුණු හා ඔබේ නිර්දේශ දැක්වෙන වාර්තාවක් ඔබගේ අත්සන යටතේ මෙහි දිනයේ සිට මාස තුනක් ඇතුළත මා වෙත ඉදිරිපත් කරන ලෙස මම මෙයින් නියම කරමි;

තවද, ඉහත සඳහන් කරුණුවලට අදාළ පරීක්ෂණයේ යම් කොටසක්, ඔබගේ අභිමතය අනුව නිශ්චය කරන පරිදි ප්‍රසිද්ධියේ නොපැවැත්විය යුතු යයි මම මෙයින් විධාන කරමි;

තවද, ඔබගේ පරීක්ෂණවල හා විමර්ශනවල කාර්ය සඳහා සහාය දෙන ලෙස හෝ තොරතුරු සපයන ලෙස ඔබ විසින් ඉල්ලා සිටිනු ලබන සියලු රජයේ නිලධරයන් හා වෙනත් තැනැත්තන් විසින් යථා පරිදි දෙනු ලැබිය හැකි සහ සපයනු ලැබිය හැකි සියලු සහාය ඒ සම්බන්ධයෙන් දිය යුතු බවටත්, සියලු තොරතුරු සැපයිය යුතු බවටත්, මම මෙයින් සියලු රජයේ නිලධරයන්ට සහ අනිකුත් තැනැත්තන්ට නියම කොට විධාන කරමි;

තවද, ඉහත කී පරීක්ෂණ කොමිෂන් සභා පනතේ 14 වන වගන්තියේ විධිවිධාන මෙම කොමිෂන් සභාවට අදාළ විය යුතු යයි මම මෙයින් ප්‍රකාශ කරමි.

වර්ෂ එක් දහස් නවසිය අනූදෙකක් වූ ඔක්තෝබර් මස විසිඅට වන දින වන මෙදින ශ්‍රී ලංකා ප්‍රජාතාන්ත්‍රික සමාජවාදී ජනරජයේ මුද්‍රාව යටතේ කොළඹ දී දෙන ලදී.

උතුමාණන්ගේ අණ පරිදි,

කේ. එච්. ජේ. විජයදාස,
ජනාධිපති ලේකම්.

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ශ්‍රී ලංකා රජයේ මුද්‍රණ දෙපාර්තමේන්තුවේ මුද්‍රණය කරන ලදී.

ශ්‍රී ලංකා ප්‍රජාතාන්ත්‍රික සමාජවාදී ජනරජයේ ගැසට් පත්‍රය

අති විශෙෂ

இலங்கைச் சனநாயக சோசலிசக் குடியரசு வர்த்தமானப் பத்திரிகை

அதி விசேஷமானது

අංක 738/11- 1992 ඔක්තෝබර් 28 වැනි බදාදා - 1992.10.28

738/11ஆம் இலக்கம் - 1992 ஆம் ஆண்டு ஒற்றோபர் மாதம் 28 ஆந் திகதி புதன்கிழமை

(அரசாங்கத்தின் அதிகாரத்துடன் பிரசுரிக்கப்பட்டது)

பகுதி I : தொகுதி (I) — பொது

சனாதிபதியின் பிரகடனங்களும் பிறவும்

ச. அ. இல. : பிபிஏ/6/என்/177/92

சா. வ. 65/44 (1)

இலங்கைச் சனநாயக சோசலிசக் குடியரசின் சனாதிபதி அதியுத்தமராம் ரணசிங்க பிரேமதாச அவர்களினால்

முத்திரை

ஆர். பிரேமதாச.

திருவாளர் ஜோசப் பிரான்சிஸ் அந்தனி குரூஸ அவர்களுக்கு.

வாழ்த்துக்கள் :

இரத்தினபுரி நிர்வாக மாவட்டத்தில் அமைந்துள்ள சமனலவெவ நீர்த்தேக்கத்தின் வலதுபக்க, இடப்பக்க கரைகளில் ஒழுக்குகள் ஏற்பட்டுள்ளன என அறிக்கையிடப்பட்டுள்ளதாதலாலும் :

இதனகத்துப் பின்னர் குறிப்பிடப்படும் நோக்கங்களுக்கென விசாரணை ஆணைக்குழுவொன்றைத் தாபித்தல் அவசியமென எனக்குத் தோன்றுகின்றதாதலாலும் :

ஆகவே, இப்போது, சனாதிபதி பிரேமதாச ஆகிய நான் தங்களது நாணயத்திலும் ஆற்றலிலும் நேர்மையிலும் மிகுந்த நம்பிக்கையும் விசுவாசமும் கொண்டு (393 ஆம் அத்தியாயமான) விசாரணை ஆணைக்குழுக்கள் சட்டத்தின் 2 ஆம் பிரிவின் ஏற்பாடுகளைப் பின்பற்றி பின்வரும் விடயங்களைப்பற்றி விசாரணை செய்து அறிக்கையிடுவதற்கு எனது ஆணையாளராயிருப்பதற்கென சொல்லப்பட்ட ஜோசப் பிரான்சிஸ் அந்தனி குரூஸ ஆகிய தங்களை இத்தால் நியமிக்கின்றேன் :-

(அ) சமனலவெவ கருத்திட்டத்தின் அணைக்கட்டு மற்றும் நீர்த்தேக்கம் தொடர்பில், குறிப்பாக, சொல்லப்பட்ட அணைக்கட்டு மற்றும் நீர்த்தேக்கம் தொடர்பில் புவிசரிதவியல் அம்சங்கள், அவற்றின் அமைவிடம், வடிவமைப்பு, சொல்லப்பட்ட நீர்த்தேக்கத்தின் நீரின் நெருக்கம் என்பன தொடர்பில் முன்னைய அலுவலக முறையான ஆய்வுகள் ;

(ஆ) சொல்லப்பட்ட அணைக்கட்டு மற்றும் நீர்த்தேக்கத்தினதும் தற்போதைய அமைவிடம், வடிவமைப்பு அவற்றிற்கான சார்புடைய வேலைகள் என்பன புவிசரிதவியல் மற்றும் நீர்ப்பாசனவியல் ஆய்வுகள் உட்பட போதிய நுண்ணாய்வுகளினதும் ஆய்வுகளினதும் அடிப்படையில் மேற்கொள்ளப்பட்டனவா

(இ) அறிக்கையிடப்பட்ட ஒழுக்குகளுக்கு ஏதுவான காரணங்கள் உட்பட சொல்லப்பட்ட அணைக்கட்டினதும் நீர்த்தேக்கத்தினதும் தற்போதைய நிலைமை, அதன் பயன்கள், சிக்கல்கள் என்பனவும் குறிப்பாக சொல்லப்பட்ட அணைக்கட்டினதும் நீர்த்தேக்கத்தினதும் அமைவிடம், வடிவமைப்பு, நிர்மாணம் என்பவற்றில் கவனயீனம் இருந்துள்ளதா என்பதும் ;

(ஈ) நிலைமையினைக் கட்டுப்படுத்தும் பரிகார நடவடிக்கைகளும் வருங்காலத்தில் இதனையொத்த இயல்பினவான சம்பவங்களைத் தடைசெய்வதற்கு அவசியமான ஏனைய வழிமுறைகளும்

அத்துடன் மேற்சொல்லப்பட்ட விடயங்களைப் பற்றி அவசியமெனத் தோன்றுகின்றவாறான அத்தகைய விசாரணைகளை நடாத்துவதற்கும், அத்தகைய வேறு எல்லாப் புலனாய்வுகளைச் செய்வதற்கும் இத்தால் அதிகாரமளித்துத் தத்துவமும் வழங்குகின்றேன். அத்துடன் உங்கள் விசாரணைகளின் கண்டுபிடிப்புகளையும் அத்துடன் உங்கள் விதப்புரைகளையும் அதனகத்துள்ள தேதியிலிருந்து மூன்று மாதங்களுக்குள்ளாக, எனக்கு அனுப்பி வைக்குமாறும் உங்களை இத்தால் தேவைப்படுத்துகின்றேன்.

அத்துடன் மேற்கூறப்பட்ட விடயங்கள் தொடர்பிலான தங்களது நற்றுணிபின்மீது தாங்கள் தீர்மானிக்கக்கூடியவாறான அத்தகைய ஏதேனும் விசாரணையின் பகுதி பகிரங்கமாக நடத்தப்படக்கூடாது எனவும் நான் இத்தால் பணிக்கின்றேன்.

அத்துடன் உங்கள் விசாரணைகளினதும் புலனாய்வுகளினதும் நோக்கங்களுக்காக எந்தப் பகிரங்க அலுவலர்களிடமும் வேறெந்த ஆட்களிடமும் நீங்கள் உதவி அல்லது தகவல் கோருகின்றீர்களோ அந்தப் பகிரங்க அலுவலர்களையும் வேறு ஆட்களையும் அதன்சார்பில் முறைப்படி அளிக்கப்படக்கூடியனவும் கொடுத்துதவக் கூடியனவுமான இத்தகைய எல்லா உதவிகளையும் அளிக்குமாறும், அத்தகைய எல்லாத் தகவல்களையும் கொடுத்துதவுமாறும் இத்தால் நான் தேவைப்படுத்திப் பணிக்கின்றேன்.

அத்துடன் மேற்சொல்லப்பட்ட விசாரணை ஆணைக்குழுச் சட்டத்தின் 14 ஆம் பிரிவின் ஏற்பாடுகள் ஆணைக்குழுவுக்கு ஏற்புடையதாதல் வேண்டுமென நான் இத்தால் வெளிப்படுத்துகின்றேன்.

இலங்கைச் சனநாயக சோசலிசக் குடியரசின் இலச்சினையின்கீழ் ஆயிரத்துத் தொளாயிரத்துத் தொண்ணூற்றிரண்டாம் ஆண்டு ஒற்றோபர் மாதம் இருபத்தேழாம் நாளாகிய இன்று அளிக்கப்பட்டது.

அதியுத்தமனாரின் ஆணைப்படி,

கே. எச். ஜே. விஜயதாச,
சனாதிபதியின் செயலாளர்.

இலங்கை அரசாங்க அச்சுத் திணைக்களத்திற் பதிப்பிக்கப்பெற்றது

ශ්‍රී ලංකා ප්‍රජාතාන්ත්‍රික සමාජවාදී ජනරජයේ ගැසට් පත්‍රය

අති විශෙෂ

The Gazette of the Democratic Socialist Republic of Sri Lanka

EXTRAORDINARY

අංක 738/11 - 1992 ඔක්තෝබර් 28 වැනි බදාදා - 1992.10.28

No. 738/11 — WEDNESDAY, OCTOBER 28, 1992

(Published by Authority)

PART I : SECTION (I) — GENERAL

Proclamations, &c., by the President

L. D.—B 65/44 (iv). **P. O. No. PPA/6/N/177/92.**

BY HIS EXCELLENCY RANASINGHE PREMADASA, PRESIDENT OF THE DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA

Seal

R. PREMADASA

To:

Joseph Francis Anthony Soza Esquire.

Greetings :

WHEREAS it has been reported that leakages have occurred on the right and left banks of the Samanalawewa reservoir situated in the administrative district of Ratnapura ;

And whereas it appears to me to be necessary to establish a Commission of Inquiry for the purposes hereinafter mentioned ;

Now therefore, I, Ranasinghe Premadasa, President, reposing great trust and confidence in your prudence, ability and fidelity, do, in pursuance of the provisions of section 2 of the Commissions of Inquiry Act (Chapter 393), by these presents appoint you, the said Joseph Francis Anthony Soza to be my Commissioner to inquire into and report on the following matters :—

(*a*) the previous official studies made in respect of the dam and reservoir of the Samanalawewa Project, with particular reference to the geological aspects relating to, and the location and design of, the said dam and reservoir and the water tightness of the said reservoir ;

(*b*) whether the current location and design of the said dam and reservoir and the appurtenant works thereto, were based on adequate investigations and studies, including geological and hydrological studies ;

(*c*) the present condition of the said dam and reservoir, including the causes which led to the reported leakages the effects and implications thereof, and in particular whether there has been any negligence in the location, design and construction of the said dam and reservoir ;

(*d*) the remedial measures to arrest the situation and other measures necessary to prevent occurrences of a similar nature in the future ;

And I do hereby authorise and empower you, the said Commissioner to hold all such inquiries and make all other investigations into the aforesaid matters as may appear to you to be necessary and require you to transmit to me within three months from the date hereof, a report thereon under your hand, setting out the findings of your inquiries, and your recommendations ;

And I do hereby direct that such part of any inquiry relating to the aforesaid matters as you may in your discretion determine, shall not be held in public ;

And I do hereby require and direct all public officers and other persons to whom you may apply for assistance or information for the purposes of your inquiries and investigations to render all such assistance, and furnish all such information, as may be properly rendered and furnished in that behalf ;

And I do hereby declare that the provisions of section 14 of the aforesaid Commissions of Inquiry Act, shall apply to the Commission ;

Given at Colombo, under the seal of the Democratic Socialist Republic of Sri Lanka, this Twenty-seventh day of October, One Thousand Nine Hundred and Ninety-Two.

By His Excellency's command,

K. H. J. WIJAYADASA,
Secretary to the President.

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APPENDIX - 2

NEWSPAPER ADVERTISEMENT IN
SINHALA, TAMIL, AND ENGLISH
INVITING REPRESENTATIONS

APPENDIX 2

සමනල වැව ජලාශයේ ජලය කාන්දු වීම සම්බන්ධයෙන් පරීක්ෂා කිරීම සඳහා වූ ජනාධිපති පරීක්ෂණ කොමිෂන් සභාව.

ලිඛිත සංදේශ සහ සාක්ෂි කැඳවීම පිළිබඳව දැනුම් දීමයි.

සමනල වැව ජලාශයේ ජලය කාන්දු වීම සම්බන්ධයෙන් පහත සඳහන් කරුණු පරීක්ෂාකර වාර්තා කිරීම සඳහා ශ්‍රීමත් ජනාධිපතිතුමන් විසින් (393 අධිකාරිය වූ) පරීක්ෂණ කොමිෂන් සභා පණත යටතේ කොමිෂන් සභාවක් පත් කර ඇත.

(අ) සමනල වැව ව්‍යාපෘති සම්බන්ධයෙන් මීට පූර්වයෙන් කරන ලද තීල අධ්‍යයන පිළිබඳව එකී වෙල්ලට සහ ජලාශයට අදාළ භූ විද්‍යාත්මක අංශ හා එහි පිහිටීම සහ සැලැස්ම හා එම ජලාශයේ රඳවන ප්‍රමාණය යන කරුණු ගැන විශේෂ සැලකිල්ලක් ඇතිව පරීක්ෂා කර වාර්තා කිරීම;

(ආ) එකී වෙල්ලෙහි සහ ජලාශයෙහි වර්තමාන පිහිටීම සහ සැලැස්ම හා ඊට අනුබද්ධ වැඩ පිළිබඳ භූ විද්‍යාත්මක හා ජල විද්‍යාත්මක අධ්‍යයනයන් ඇතුළුව, ප්‍රමාණවත් විමර්ශන හා අධ්‍යයනයන් මත පදනම් වී ද යන වග.

(ඇ) සිදුවී ඇතැයි වාර්තා වී ඇති ජල කාන්දුවීම් ඇතුළුව එකී වෙල්ලෙහි සහ ජලාශයේ වර්තමාන තත්ත්වය, එම කාන්දුවීම්වල බලපෑම හා ඊට අදාළ ගැටළු සහ විශේෂයෙන් ම එකී වෙල්ල හා ජලාශයේ පිහිටීම, සැලැස්ම හා ඉදි කිරීම්වලදී යම් නොසැලකිල්ලක් සිදුවී තිබේද යන වග.

(ඈ) මෙම තත්ත්වය වැළැක්වීමට ගතයුතු පිළියම් ක්‍රියා මාර්ග හා මෙබඳු තත්ත්වයන් මතුවට ඇතිවීම වැළැක්වීමට අවශ්‍ය වෙනත් ක්‍රියා මාර්ග.

මේ පිළිබඳව කරුණු ලිඛිතව හා/හෝ වාචිකව ඉදිරිපත් කිරීමට කැමැති සියලු අයවලුන් ගෙන්, සංවිධාන හා ආයතනයන්ගෙන් මෙම දැන්වීමෙහි දිනයේ සිට දෙසතියක් තුළ ලැබෙන සේ ලියා පදිංචි තැපැලෙන් එම කරුණු අඩංගු සංදේශ එවන ලෙස හා/හෝ වාචිකව කරුණු ඉදිරිපත් කිරීමට කැමැත්ත පහත සඳහන් ලිපිනයට දන්වා එවන ලෙස මෙයින් කාරුණිකව දන්වනු ලැබේ.

ලේකම්,
සමනල වැව කොමිෂන් සභාව,
බණ්ඩාරනායක ජාත්‍යන්තර සම්මන්ත්‍රණ ශාලාව,
බෞද්ධාලෝක මාවත, කොළඹ 07.

මෙයට,

ලේකම්, සමනල වැව ජලාශයේ ජලය
කාන්දුවීම පරීක්ෂා කිරීම සඳහා වූ
ජනාධිපති කොමිෂන් සභාව.

தினகரன்
1990 நவம்பர் 1992.

சமனலவெவ நீர்த் தேக்கத்தின் நீர் ஒழுக்குப்பற்றி ஆராய்வதற்கு நியமிக்கப்பட்ட சனாதிபதி விசாரணை ஆணைக்குழு

எழுத்து மூல விஞ்ஞாபனங்களையும் சான்றாளர்களையும் அழைத்தல் பற்றிய

அறிவித்தல்

சமனலவெவ நீர்த்தேக்கத்தின் நீர் ஒழுக்குப் பற்றிய பின்வரும் விடயங்களை ஆராய்ந்து அறிக்கையிடுவதற்காக அதிமேதகு சனாதிபதி அவர்களால் (393ஆம் அத்தியாயமான) விசாரணை ஆணைக்குழுக்கள் சட்டத்தின் கீழ் ஆணைக்குழுவொன்று நியமிக்கப்பட்டுள்ளது.

(அ) சமனலவெவ கருத்திட்டத்தின் அணைக்கட்டு மற்றும் நீர்த்தேக்கம் தொடர்பில், குறிப்பாக, சொல்லப்பட்ட அணைக்கட்டு மற்றும் நீர்த்தேக்கம் தொடர்பில் புவிசரிதவியல் அம்சங்கள், அவற்றின் அமைவிடம், வடிவமைப்பு, சொல்லப்பட்ட நீர்த்தேக்கத்தின் நீரின் நெருக்கம் என்பன தொடர்பில் முன்னைய அலுவலக முறையான ஆய்வுகள்;

(ஆ) சொல்லப்பட்ட அணைக்கட்டினதும் நீர்த்தேக்கத்தினதும் தற்போதைய அமைவிடம், வடிவமைப்பு, அவற்றிற்கான சார்புடைய வேலைகள் என்பன புவிசரிதவியல் மற்றும் நீர்ப்பண்பியல் ஆய்வுகள் உட்பட போதிய நுண்ணாய்வுகளினதும் ஆய்வுகளினதும் அடிப்படையில் மேற்கொள்ளப்பட்டனவா;

(இ) அறிக்கையிடப்பட்ட ஒழுக்குகளுக்கு ஏதுவான காரணங்கள் உட்பட சொல்லப்பட்ட அணைக்கட்டினதும் நீர்த்தேக்கத்தினதும் தற்போதைய நிலைமை, அதன் பயன்கள், சிக்கல்கள் என்பனவும் குறிப்பாக சொல்லப்பட்ட அணைக்கட்டினதும் நீர்த்தேக்கத்தினதும் அமைவிடம், வடிவமைப்பு, நிர்மாணம் என்பவற்றில் கவனயீனம் இருந்துள்ளதா என்பதும்;

(ஈ) நிலைமையினைக் கட்டுப்படுத்தும் பரிகார நடவடிக்கைகளும் வருங்காலத்தில் இதனையொத்த இயல்பினவான சம்பவங்களைத் தடை செய்வதற்கு அவசியமான ஏனைய வழிமுறைகளும்.

இதுபற்றி எழுத்துமூலமும் அல்லது / அத்துடன் வாய்மூலமும் தகவல்களைத் தெரிவிப்பதற்கு விரும்பும் பொதுமக்கள், அமைப்பாண்மைகள், நிறுவனங்கள் இந்த அறிவித்தல் பிரசுரிக்கப்பட்ட திகதியிலிருந்து இரண்டு வாரங்களுக்குள் பதிவஞ்சல் மூலம் அந்த விஞ்ஞாபனங்களையும், வாய்மூலம் தகவல்களைத் தெரிவிக்க விரும்புபவர்களின் பெயர்களையும் கீழேயுள்ள முகவரிக்கு அனுப்புமாறு அன்புடன் கேட்கப்படுகின்றனர்.

செயலாளர்,
சமனலவெவ விசாரணை ஆணைக்குழு,
பண்டாரநாயக்க ஞாபகார்த்த சர்வதேச
மாநாட்டு மண்டபம்,
பௌத்தாலோக மாவத்தை,
கொழும்பு—2.

இங்ஙனம்,

செயலாளர்,
சமனலவெவ நீர்த்தேக்கத்தின் நீர் ஒழுக்குப் பற்றி ஆராய்வதற்கான சனாதிபதி விசாரணை ஆணைக்குழு

'Daily News'

19th November, 1992

NOTICE CALLING FOR

MEMORANDA AND REPRESENTATIONS

COMMISSION OF INQUIRY INTO THE LEAKAGE OF WATER FROM SAMANALA WEWA

In pursuance of the provisions of Section 2 of the Commissions of Inquiry Act (Chapter 393), H.E. The President of the Democratic Socialist Republic of Sri Lanka has appointed a Commission of Inquiry to inquire into and report on the following matters

(a) the previous official studies made in respect of the dam and reservoir of the Samanalawewa Project, with particular reference to the geological aspects relating to, and the location and design of, the said dam and reservoir and the water tightness of the said reservoir,

(b) whether the current location and design of the said dam and reservoir and the appurtenant works thereto, were based on adequate investigations and studies, including geological and hydrological studies;

(c) the present condition of the said dam and reservoir, including the causes which led to the reported leakages the effects and implications thereof, and in particular whether there has been any negligence in the location, design and construction of the said dam and reservoir;

(d) the remedial measures to arrest the situation and other measures necessary to prevent occurrences of a similar nature in the future;

Members of the public and organisations and institutions that are desirous of making representations and / or giving oral evidence are hereby requested to send in their representations and / or indicate their willingness to give oral evidence, within two weeks from the day of publication of this notice by registered post addressed to

The Secretary
Samanala Wewa Commission,
Bandaranayaka Memorial International Conference Hall,
Bauddhaloka Mawatha,
Colombo 07.

Secretary
to the Commission of Inquiry into the Leakage of Water from Samanalawewa.

APPENDIX - 3

LIST OF REPRESENTATIONS/MEMORANDA IN RESPONSE TO ADVERTISEMENT

APPENDIX 3

PRESIDENTIAL COMMISSION OF INQUIRY INTO THE LEAKAGE OF WATER FROM SAMANALA WEWA RESERVOIR

ındaranaike Memorial International Conference Hall
ıuddhaloka Mawatha,
ɔlombo 07.

Our Ref: SAM/INQ/1.
Your Ref:

26th October, 1993.

Commissioner,
Samanala Wewa Commission of Inquiry,
B.M.I.C.H.,
Colombo 07.

Dear Sir,

SAMANALA WEWA COMMISSIN OF INQUIRY - LIST OF MEMORANDA

01. Sri Lanka Jathika Govi Sammelanaya,
513/2/1, Elvitigala Mawatha, Colombo 05.

02. Mr. P.G. Joseph, Engineering Consultant,
25/4, Pepiliyana Road, Nugegoda.

03. Mr. G.L.A. Nanayakkara, Law Officer,
Environmental Foundation Ltd.,
No.3, Campbell Terrace, Colombo 10.

04. Vidya Jyothy Dr. A.N.S. Kulasinghe, Chairman,
C.E.C.B., 415, Bauddhaloka Mawatha, Colombo 07.

05. Mr. D.L.O. Mendis,
16/1, George E. de Silva Mawatha, Kandy.

06. Mr. S. Godakumbura, President, Ukgal Kaltota, L/B.,
Left R./D. Society,
183, Medabedda, Kaltota, Balangoda.

07. Mr. S.M. Wijesekera, President, Ukgal Kaltota L/B.,
Farmer Organisation, Medabedda, Kaltota, Balangoda.

08. Mr. A.T.G.A. Wickremasooriya,
Retired Senior Dy. Director, Irrigation,
Civil Engineering Consultant,
118/10, Nawala Rd., Colombo 05.

09. Mr. D.V.A. Senaratne, Engineering Consultant,
136, George R. de Silva Mawatha, Colombo 13.

10. Mr. M.S.M. de Silva, Engineering Consultant,
24/1, Ananda Road, Nugegoda.

11. Mr. K.A.D. Banda,
"Anura Wasa", 15/1, Hokandara South, Hokandara.

12. Rev. Kinchigune Sobitha Thera, Samanalagama, Morahela,
Balangoda.

Yours faithfully,

PRESIDENTIAL COMMISSION OF
INQUIRY INTO THE LEAKAGE OF

APPENDIX - 4

LIST OF WITNESSES

WHO GAVE EVIDENCE

APPENDIX 4

SAMANALA WEWA COMMISSION OF INQUIRY

LIST OF WITNESSES

01. Mr. K.A. Ranaweera, G.M., C.E.B.
02. Mr. E.C. Fernando, Consultant, C.E.B.
03. Mr. N.A.J. Perera, Chairman, C.E.B.
04. Prof. K.K.Y.W. Perera, Former Chairman, C.E.B.
05. Mr. V.F. Perera, Geologist, C.E.C.B.
06. Mr. A.T.G.A. Wickremasooriya
07. Mr. D.L.O. Mendis
08. Mr. C.C.T. Fernando
09. Dr. Gamini Samarasinghe
10. Rev. Kinchigune Sobitha Thero
11. Mr. Y. Saranapala
12. Mr. D.M.J. Bandara
13. Mr. M.A. Weerasinghe
14. Mr. A.G. Premadasa
15. Mr. W. Ranatunge
16. Mr. A. William
17. Rev. Bamarakotuwe Sangaratne Thero
18. Mr. S.M. Premaratne
19. Mr. S.M. Wijesekera
20. Mr. G. Sugathadasa
21. Mr. M.A. Wijesiri
22. Mr. M.A. Appuhamy
23. Mr. K.L. Podimahathmaya
24. Mr. T.M. Dharmaratne
25. Mr. P.O. Squire

26. Dr. H.E. Minor

27. Mr. K. Wada

28. Mr. M.S.M. De Silva

29. Mr. G.G. Jayawardena

30. Mr. D.V.A. Senaratne

31. Mr. H.A.L.S. Yapa

32. Dr. P.A.A. Back

33. Dr. A.N.S. Kulasinghe, Chairman, C.E.C.B.

34. Mr. S. Ganesharajah, Project Director,
Samanalawewa Pr., C.E.B.

35. Mr. K.A. Dingiri Banda

36. Mr. J.C. Gunawardena

37. Mr. H.K.M.K. Jayatillake

38. Mr. G.N. Tandon

APPENDIX - 5

LIST OF DOCUMENTS/PRODUCTIONS

MARKED IN EVIDENCE

APPENDIX 5

SAMANALA WEWA COMMISSION OF INQUIRY - LIST OF PRODUCTIONS

1.	P - 1	:	Agreement - Contract for Consultancy Services for the Samanala Wewa Hydro Electrict Project Nippon Koei Co.Ltd., Electro - Watt Co. Ltd., Joint Venture with Ceylon Electricity Board.
2.	P - 2	:	An Agreement Between JVS & CECB.
3.	P - 3	:	Board Papers of CEB.
4.	P - 4	:	Correspondance of Chairman, CEB with Dr. A.N.S. Kulasinghe.
5.	P - 5	:	Report on the Review of Proposals submitted in 1985 by Balfour Beatty Construction Ltd.- by Electro Watt Engineering Services - May 1985.
6.	P - 6	:	Appendices to the Technical Report of the Feasibility Report on Samanala Wewa Project - by Snowy Mountains Ltd., of Australia.
7.	P - 6(a)	:	Page 6 - Page 15.
8.	P - 6(b)	:	Page 6 - Page 27.
9.	P - 6(c)	:	
10.	P - 7	:	Regional Geology of Samanala Wewa Location.
11.	P - 8	:	Report on Additional Geotech. Investigations for Samanala Wewa by Nippon Koei Engineers - Fig. 1.
12.	P - 8(a)	:	Hydro Geological Section across GW13-GW7 and along Access Adit E - Mr. Vernon F. Pereira.
13.	P - 9	:	Brief Report by Technoprom export.
14.	P - 10	:	Vol. II - Geology & Topography by Technoprom export.
15.	P - 11	:	Geological Drawings, Geological Maps of Samanala Wewa Project Area.
16.	P - 12	:	Rock Grouting with emphasis on Dan Sites by F.K. Ewert (Professor Frederick ~~Karl~~ Ewert) from Federal Republic of Germany - 1985.
17.	P - 13	:	Samanala Wewa HEP - Engineering Review and Recommendation, Technical Report - by Balfour Beatty Ltd., G.E.C. Energy Systems Ltd., Sir Alexander Gibb & Partners, EPD Consultants Ltd., April 1984.

18.	P - 14	:	Samanala Wewa Hydro Electric Scheme - Review Report Vol. - I - Electro Watt Engineering Services Ltd.,- Nov. 1984.
19.	P - 15	:	Planning Report of the Additional Geotechnical Investigation for Design of Samanala Wewa - April 1986 Nippon Koei Co. Ltd.
20.	P - 16	:	Report on Additional Geotechnical Investigations for the Design of Samanala Wewa Dam Foundations - Vol. I July 1986 - by Nippon Koei Co.Ltd, Electro Watt Engineering Co. Ltd, CECB.
21.	P - 17	:	Location Map Saddles Right Bank (* not available)
22.	P - 18	:	
23.	P - 19	:	Bor-holes D.H./R.D.
24.	P - 19(a)	:	" " J Series.
25	P - 20	:	Rock types - Geology/Geological Section across special way.
26	P - 20(a)	:	Ground Water - RB/LB L series.
27.	P - 20(b)	:	Figure 4 (as in P16) in colour.
28.	P - 21	:	Weathering Profile (as in P16) coloured.
29.	P - 22	:	Lugeon values as in P16 fig. 4.
30.	P - 23	:	News artical "Construction Today".
31.	P - 24	:	Letter By Gibb J.R. Westwell to CEB dated 22.11.88.
32.	P - 24(a)	:	Report annexed to P24.
33.	P - 25	:	Draft Tender Document Lot.2.
34.	P - 26	:	Contract Document for Lot.2.
35.	P - 27	:	Bar Chart By V.F. Pereira.
36.	P - 28	:	C.E.C.B. Report - April, 1985.
37.	P - 29(a)	:	Drawing of the Geology of the Dam & Saddle Areas.
38.	P - 29(b)	:	Drawing by J.V.S. Geology along the centre of the Dam.
39.	P - 29( )	:	Coloured Section of P29 (b).
40.	P - 29(c)	:	Drawing indicating Adits and the level of Grout curtain.
41.	P - 29(d)	:	Drawing showing Ground Water Table/River level/Rain fall.
42.	P - 29(e)	:	Gibb drawing showing groun water contour map.

43.	P - 30 :	Letter of 6th November,1988 signed by cole, Chief Design Engineer Gibb.
44.	P - 30(a) :	Samanala Wewa H.E.P Lot.2 Reservoir Water Tightness, Dam Site - Saddles.
45.	P - 30(b) :	G.W. series ground water levels May-1989-August,89.(G.W.1-5)
46.	P - 31 :	Drawing By V.F. Pereira - Different Stratigraphy & Bore Holes in 1989.
47.	P - 32 :	Minutes of 1st Design Meeting.
48.	P - 32(a) :	Design Meeting - 2 (Minutes).
49.	P - 32(b) :	Report on 2nd Design Meeting - JVS Proposal.
50.	P - 32(c) :	3rd. Design Meeting - 24.11.1989.
51.	P - 32(d)&(e)	Drawings annexed to P32 (e).
52.	P - 33 :	First Executive Meeting.
53.	P - 34 :	Report of the Committee appointed by the Chairman CEB on 05.12.1989 re water tightness.
54.	P - 35 :	Notes of the above Committee Meeting 12.01.90.
55.	P - 36 :	Dr. Bauma, report 02.08.1990.
56.	P - 37 :	Results of the 2nd Executive Meeting - 20.11.90.
57.	P - 38 :	Second Executive Meeting R/B cut off discussion material by JVS.
58.	P - 38(a) :	Page 25.
59.	P - 38(b) :	Page 26.
60.	P - 38(c) :	Page 28 Diagram.
61.	P - 38(c1):	Coloured Version of the Diagram.
62.	P - 39 :	Third Executive Meeting - 27/08.04.1991, summary of conclusions written by Gibb.
63.	P - 40 :	CEB Letter 24.07.1991 - PDS/C/Cut off 50 to JVS/GIBB.
64.	P - 41 :	Letter from Mr. Wada to Squire - 27.05.1991.
65.	P - 41(a) :	Report Annexed to P41.
66.	P - 42 :	Ground water response section.
67.	P - 43 :	4th Executive Meeting 20/21st April, 1991.

68.	P - 43(a)	:	Joint Statement R/B Cut off.
69.	P - 43(b)	:	Statement by GIBB.
70.	P - 43(c)	:	Summary of Conclusions by JVS.
71.	P - 44	:	Explanation Sheet for updated Grouting plan.
72.	P - 45	:	Drawing - Leakage after the 1st and 2nd Impounding.
73.	P - 46	:	Geology & Configuration of Grout Curtain to be.
74.	P - 46(a)	:	- do - (Coloured)
75.	P - 47	:	Document dated 05.08.1991 by JVS to GIBB.
76.	P - 48	:	Letter to O.E.C.F.
77.	P - 49	:	R/B cut off - Primary Grout Takes Ch 0-1320 mt.
78.	P - 50(a)	:	P50L (1), (2), (3), P50(P)1-3. Correspondance with Dr. Kulasinghe (In one file).
79.	P - 51	:	New Civil Engineer - 23.01.1992.
80.	P - 52	:	Sketch showing & incidents after second impounding.
81.	P - 53	:	Report & Drawings by CECB.
82.	P - 54	:	Drawing.
83.	P -54(a)	:	)
84.	P - 54(b)	:	) Drawings.
85.	P - 54(c)	:	)
86.	P - 55	:	Springs on the L/B.
87.	P - 55(a)&(b)		2 Sketches.
88.	P - 56	:	Fax Message - Mr. Wada to Westwell.
89.	P - 57	:	A set of instructions from JVS to KHK "Follow up works Additional Investigations along Adit D a".
90.	P - 58	:	Letter from Mr. Wada to Mr. Squire - 07.07.1992.
91.	P - 59	:	Letter to Mr. Ganesharajah of CEB - Nippon Koei on 13.07.1992.
92.	P - 60	:	Russian Dr. A. Vargas reply to Art. in New Civil Engineer.
93.	P - 61	:	Fax Message - 21.07.1992 - to By.. from Mr. Squire (Reply to P60).
94.	P - 62	:	13.07.1992 dated letter from Wada to CEB.
95.	P - 63	:	Report by Electro Watt - August 1985 on Additional Studies.
96.	P - 63(a)	:	Geological Map in Report SMW 1 December 1992.

97.	P - 64	:	Samanala Wewa Hydro Electric Project 3rd Party Review Panel Report 29th November to 3rd December 1992. SMW 1 report February 1993 by Post and Londe.
98.	P - 65	:	Grout Take Maps - Primary Grouts.
99.	P - 65(a)	:	" " " & Geology by Vernon Pereira.
100.	P - 66	:	Drainage Curtain - Investigations along Adt D a.
101.	P - 66(a)	:	)
102.	P - 66(b)	:	) Covering letter to P66 and anextures (b) & (c).
103.	P - 66(c)	:	)
104.	P - 67	:	Grout Holes in Adit Ab & Db.
105.	P - 68	:	Adits Ab & Db Showing grout take and pezo meters.
106.	P - 69	:	Fig 9 in P64.
107.	P - 70	:	JVS Drawing of Grouting Adit Da & Db.
108.	P - 71	:	Letter from J.R. Westwell to Project Director SHEP dated October 1992.
109.	P - 72	:	Letter from J.R. Westwell to Project Director dated 09.10.1993.
110.	P - 72(a)	:	Report annexed to P72.
111.	P - 73	:	Comments on the Report (SMW - 2) by C.E.C.B.
112.	P - 74	:	Memorandom from V.F. Pereira Mr. K. Wada dated 04.04.90.
113.	P - 74(a)	:	Drawing annexed to P74.
114.	P - 75	:	Gibb Contract.

Productions:

115.	AW1	:	Letter from Mr. A.T.G.A. Wickremasooriya dated 14.12.92 to Samanala Wewa Commission of Inquiry.
116.	AW1(a)	:	Paper/presented by Mr. A.T.G.A. Wickremasooriya to the O.P.A. on 11.12.92.
117.	AW1(b)	:	Article in the Island on 12.11.92.
118.	AW1(c)	:	Article in the Island on 19.11.92.
119.	AW2 :	:	Letter of 12.02.93.

120.	AW3	:	Drawing.
121.	AW4	:	Island - Article 18th March, 1993.
122.	AW5	:	Drawing - Walaweganga Upper Basin.
123.	D.L.O.M1	:	Memo submitted to the Commission by Witness D.L.O. Mendis.
124.	G.S.1	:	Extract from Island - 18th July, 1988.
125.	G.S.2	:	Belihul-Oya Multipurpose Project - Page 464-466 of Sprit of Enterprise.
126.	G.S.3	:	Letter to the Commission by Witness G. Samarasinghe dated 05.08.1993.
127.	G.S.4	:	Article in Sprit of Enterprise - 1987.
128.	G.S.4(a)	:	Excerpt from Sprit of Enterprise - 1987.
129.	K.S.1	:	Letter of Rev. Kinchigune Sobitha dated 31.01.93 to the President.
130.	W.R.1	:	Feasibility report by Hydrotechnic Corp. Inc. Denver.
131.	Gibb-1	:	Gibb Report 9th November, 1989.
132.	Gibb-2	:	Sketch by Witness - P.O. Squire.
133.	Gibb-3	:	Reservoir Water Tightness Report-& R.B. Cut-off works report November 1991.
134.	Gibb-4	:	Drilling Programme of G.W.16.
135.	Gibb-5	:	Letter from Gibb to Director SHEP dated 02.12.1992.
136.	Gibb-5A	:	Report annexed to Gibb-5.
137.	Gibb-6	:	Gibb ground water contour April 1989.
138.	Gibb-7	:	Recommendation for further investigation and initial instrumentation - 17.03.1989 by Gibb.
139.	Gibb-8	:	Notes on Technical Meeting of 02.06.88 - J.V.S.C Gibb.
140.	Gibb-9	:	Sketch describing unconfined/confined acqui
141.	Gibb-10	:	Geology along centre lieue of Dam & R.B.
142.	Gibb-11	:	Geological Section along Dam and Adit Db.
143.	Gibb-12	:	Borehole details.
144.	Gibb-13	:	Site proposed boreholes & seismic refraction Survey location plan.
145.	Gibb-14	:	Summary interpretation.

146.	Gibb-15	:	Letter to Secretary P.&.E. dated 21.12.91 by Director SHEP (with annexed report).
147.	Gibb-16	:	Letter of 11.11.86 to D.G.M. Sam. C.E.B. by K. Wada (with annexed report).
148.	EWI1	:	Additional Optimisation Studies Main Report by EW1 (Aug.1985).
149.	EWI2	:	Lot2 - Civil Works - Tender documents (outer cover).
150.	NK1	:	Aug. 83 Samanala Wewa - Reconnaissance Report.
151.	NK2	:	Minutes of 11th April, 1986 - O.E.C.F. discussion.
152.	NK3	:	Technical proposals of consultancy services by Nippon Koei Electrowatt.
153.	NK4	:	Proposals for additional core drilling by N.K. dated 14.12.86.
154.	NK5	:	Letter from K. Wada along with the Geological Report of Mr. Nishiyoda - 12.11.86.
155.	NK5A	:	Report of Mr. Nishiyoka.
156.	M.S.M.1	:	Professional record of witness - M.S.M. De Silva.
157.	D.V.A.1	:	Report from D.V.A. Senaratne.
158.	DVA(1A)	:	Letter dated 14.12.1992.
159.	ANS1	:	Memo - Some cases of Sealing Material placed under water - 02.02.93.
160.	ANS2(a)	:	- do -
161.	ANS2(b)	:	Large Dam salvation abandoned construction today - May 1990.
162.	ANS2(c)	:	EI Chocon repairs hang in the balance.
163.	ANS3	:	Samanala Wewa H.P. - Review Report of the Expert Panel.
164.	ANS4	:	Gibb Fax Message to Project Director S.H.E.P. dated 09.10.92.
165.	ANS4(A)	:	- do - dated 08.10.92
166.	ANS4(b)	:	Fax Message from Gibb to JVS dated 28.07.92.
167.	ANS5	:	File containing communication with Chairman CEB.
168.	ANS6	:	Letter to Minister of Lands, Irrigation and Mahaweli Development dated 16.11.92 - by Dr. A.N.S. Kulasinghe.

169.	ANS6	:	S.H.E.P. Summary of current status & Recommendations on 09.11.92.
170.	ANS6B	:	S.H.E.P. Observations on the above summary.
171.	ANS7	:	Review of flood studies carried out by Techno Promexport & EW 1.
172.	ANS8	:	Recommendation for Remedial measures to be taken for Samanala Wewa Reservoir - March 1993 by J.V.S & Gibb.
173.	SG1	:	Letter of 28.03.88. From Secretary Power and Energy to G.M. C.E.B.
174.	SG2	:	Minutes of the Steering Committee Meeting held on 02.08.85.
175.	SG3	:	Report of the Technical Evaluation Committee.
176.	SG4	:	Cabinet Memo of 20th November '79.
177.	SG5	:	Cabinet Memo of 9th June 1983.
178.	SG6	:	C.F.C & Review Report.
179.	SG7	:	Remedial Measures for the Leakage along R/B hill presented to the review Panel on 18th February 93 by C.E.B.
180.	SG8	:	Letter of 04.12.86 by the Director External Resources.
181.	SG9	:	Letter of 10th August 1990 by the Deputy Director External Resources.
182.	SG10	:	Letter of 29.04.88 by the Advisor for Director External Resources.
183.	SG11	:	Balfour Beatty Letter dated 01.03.85 to Chairman Samanala Wewa Hydro Electric Project Steering Committee.
184.	SG12	:	C.E.B. Memo to Technical Evaluation Committee. dated 18th Aug '93.
185.	SG13	:	Gibb Letter of 6th July '93 to J.V.S.
186.	SG14	:	Recommendation for Remedial Measures for Samanala Wewa Reservoir, March '93.
187.	SG15	:	Report - S.H.E.P. Reservoir Remedial Measures Seismic Reflectory Survey.
188.	SG16	:	J.V.S. Water Quality Analysis Report 17.05.93.
189.	SG17	:	C.E.B. Letter of 9th August '93 to O.C.E.F.
190.	SG18	:	S.H.E.P. Terms of reference in respect of an E.I.A re Clay dumping.

APPENDIX - 6

MAP SHOWING NATURAL REGIONS OF

WALAWE GANGA BASIN

APPENDIX 6 FIGURE

WALAWE GANGA BASIN

NATURAL REGIONS

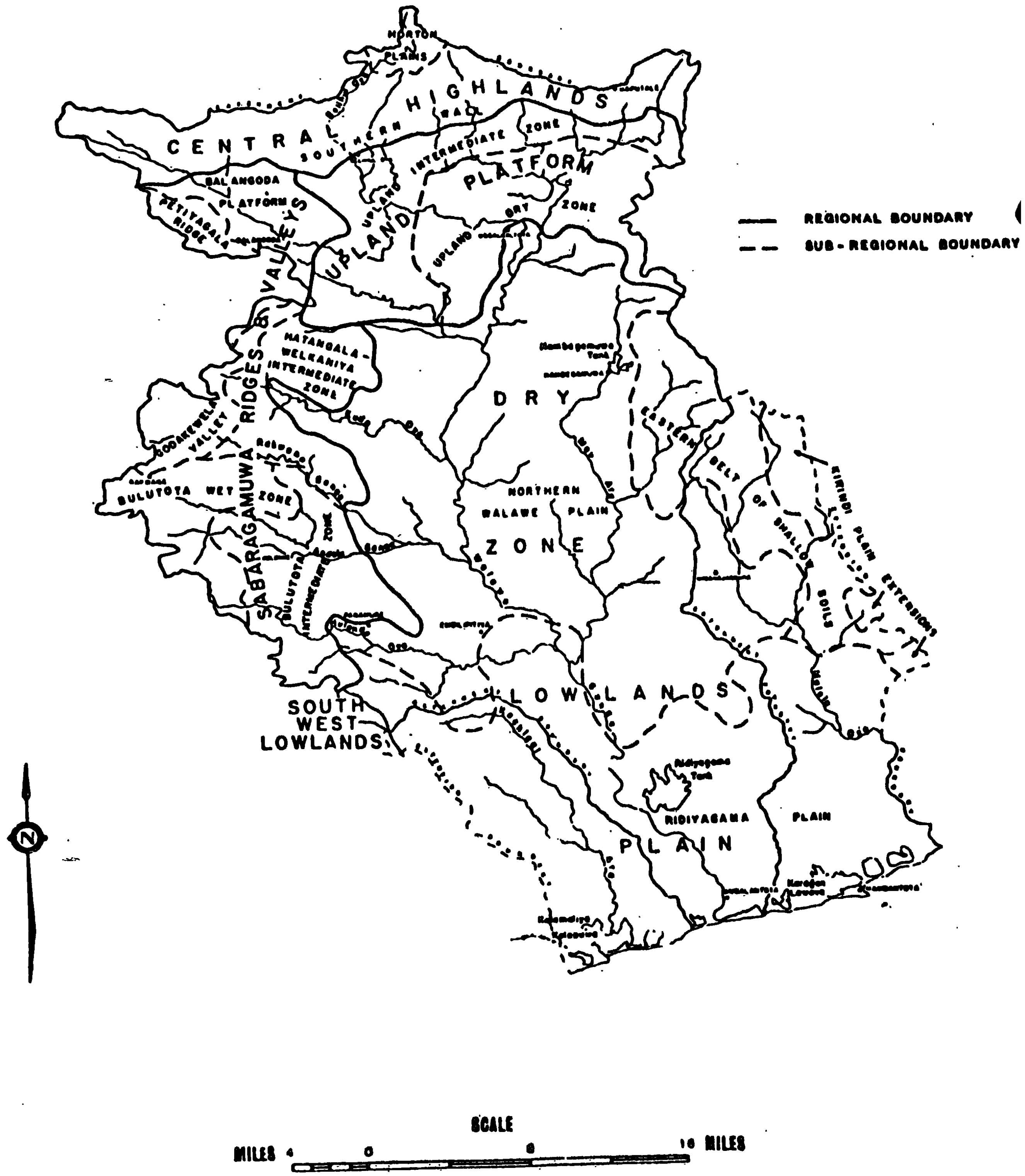


TABLE 5 SUMMARY OF PROGRESS

BOREHOLE NO.	SURVEY INFORMATION Eastings (m)	Northings (m)	Collar Elevation (m)	Inclination (deg.)	Drilled Depth (m)	No. of S.P.T Tests	REMARKS	Top of Section (m)	Bottom of Tip (m)	PIEZOMETER DATA Elevation of Tip Top (m)	Bottom (m)	Piezometer Collar El (m)	REMARKS
D 1	202888.50	163853.10	388.75	40	50.00	8	Ended in faulted CAL			388.75	388.75		
D 2	202899.30	163853.30	388.90	30	50.00	10	Ended in faulted GRA1			388.90	388.90		
D 3	202899.20	163847.20	389.92	1	50.00	10	Ended in faulted CAL			389.92	389.92		
D 4	202899.00	163848.80	388.47	61	50.00	9	Ended in faulted GRA1			388.47	388.47		
D 5	202898.42	163850.85	388.44	90	180.00	36	Ended in GRA0	133.00	180.00	255.44	208.44	389.62	Piezometric Section CHA1
WS 1	202879.79	164083.64	414.82	90	75.00	11	Ended in CHA1	36.00	75.00	378.82	339.82	415.32	Piezometric Section CHA1
WS 2	202774.80	164125.16	442.83	90	90.00	14	Ended in CHA1	45.00	90.00	397.83	352.83	443.29	Piezometric Section GRA1,CHA1
WS 3	202842.92	163994.24	433.99	90	80.00	13	Ended in faulted CHA1	56.00	80.00	377.99	353.99	434.19	Piezometric Section CHA1
WS 4	202800.07	164030.52	448.34	90	110.00	20	Ended in CHA1	63.00	110.00	385.34	338.34	449.01	Piezometric Section GRA1,CHA1
GW 1	202704.51	163854.63	480.07	90	201.65	38	Ended in cavity in CHA1	173.00	201.65	307.07	278.42	481.19	Piezometric Section CHA1
GW 2	202714.22	163789.60	497.78	90	200.00	32	Ended in CHA1	120.40	165.40	377.38	332.38	498.75	Piezo' Sec' GRA2,CAL,GRA1,CHA1. Sand 165-200
GW 3	202588.30	163635.14	478.78	90	230.00	41	Ended in CAL	175.00	230.00	303.78	248.78	479.52	Piezometric Section GRA3,GRA2,CAL
GW 4	202850.11	163820.03	477.00	90	200.00	38	Ended in CHA1	97.35	200.00	379.65	277.00	477.41	Piezometric Section CAL,GRA1,CHA1
GW 5	202805.00	163670.21	472.50	90	225.00	36	Ended in CHA1	105.00	225.00	367.50	247.50	472.60	Piezometric Section GRA2,CAL,GRA1,CHA1
GW 6	202800.00	163439.75	471.50	90	220.00	39	Ended in CHA1	145.00	150.00	326.50	321.50	472.03	Piezo' Section GRA2,CAL,GRA1. Sand 150-220
GW 7	202799.86	163299.73	504.70	90	205.00	36	Ended in GRA3?	110.00	138.00	394.70	366.70	505.16	Piezo' Section CHA2,GRA4. Grout 138-205
GW 8	202332.86	163831.04	497.96	90	220.00	34	Ended in GRA3?	160.00	200.00	337.96	297.96	498.29	Piezometric Section GRA4. Grout 200-220
GW 9	202446.07	164239.48	497.66	90	200.00	28	Ended in CHA1	125.00	150.00	372.66	347.66	498.17	Piezo' Sec' CAL,GRA1,CHA1. Sand 150-200
GW 10	202501.38	164128.22	534.27	90	200.00	26	Ended in CAL	160.00	175.00	374.27	359.27	534.74	Piezo' Section GRA2,CAL. Sand 175-220
GW 11	202454.97	164176.77	524.99	90	180.00	23	Ended in CAL	150.00	165.00	374.99	359.99	525.47	Piezo' Section GRA2,CAL. Sand 165-180
GW 12	202337.05	164158.54	482.17	90	200.00	29	Ended in GRA1	110.00	150.00	372.17	332.17	482.17	Piezo' Section GRA2,CAL,GRA1. Sand 150-200
GW 13	202421.71	163444.53	462.26	90	200.00	31	Ended in GRA4	172.50	176.00	289.76	286.26	462.52	Piezometric Section GRA4. Sand 176-200
GW 14	202919.13	163100.38	507.90	90	132.50	16	Abandoned in Cavity, GRA4						None Installed, Rods Abandoned in Hole.
GW 14A	202918.37	163101.27	507.90	90	200.00	14	Ended in GRA4	110.00	150.00	397.90	357.90	508.49	Piezometric Section CHA2. Grout 150-200
GW 15	202838.94	163218.19	467.47	90	200.00	32	Ended in GRA4	160.00	200.00	307.47	267.47	467.89	Piezometric Section CHA2, GRA4
GW 15A	202838.43	163218.23	467.46	90	57.00	0	Ended in GRA5	47.00	57.00	420.46	410.46	467.91	Piezometric Section GRA5
GW 16	202776.13	162793.80	479.83	90	200.00	37	Ended in CHA2	110.00	167.00	369.83	312.83	480.88	Piezometric Section top of CHA2.
GW 17	202564.84	163879.08	504.70	90	200.00	32	Ended in GRA2	125.00	145.00	379.70	359.70	505.24	Piezo' Section GRA4, GRA3. Grout 145-200
GW 18	202983.42	161926.22	495.26	90	200.00	33	Ended in GRA5?	122.30	177.00	372.96	318.26	496.20	Piezometric Section GRA5? Sand 177-200
GW 19	203028.55	165121.38	503.69	90	200.00	21	Ended in CHA1	114.00	144.00	389.69	359.69	504.11	Piezometric Section CHA1. Grout 144-200
TOTALS			30		4806.15	745			3901.05			25	

APPENDIX - 7

SUMMARY OF

ADDITIONAL INVESTIGATION DRILLING

APPENDIX - 8

SCHEDULE OF PRINCIPAL FEATURES OF SAMANALAWEWA PROJECT INCLUDING DAM AND APPURTENANT STRUCTURES

APPENDIX 8

Principal Features of the Samanalawewa Project

(1) Samanalawewa Reservoir
- Catchment area : 341.7km²
- Annual average runoff : 598 million m³
- Flood level : EL. 460.7m
- Normal high water level : EL. 460.0m
- Minimum operation level : EL. 424.0m
- Live storage above minimum operation level : 254 million m³

(2) Samanalawewa Dam
- Type : Rockfill dam with clay core
- Crest length : 530m
- Dam height : 100m
- Volume of fill : 4.5 million m³

(3) Spillway
- Type : Gated overflow
- Capacity : 3600m³/s
- Number of gates : 3
- Size of gates : 11m wide x 14m high
- Sill level : EL. 446.7m

(4) Diversion Tunnels/Low Level Outlet
- Number of Diversion tunnels : 2
- Diameter : 6.8m
- Length, tunnel No. 1 : 520m
- Tunnel No. 2 : 545m
- Diversion capacity : 1,500m³/s (design flood)
- Low level outlet : Jet flow gate (Emergency) 1.4m x 2.0m; Jet flow gate (Irrigation) 0.6m dia.

(5) Power Supply Tunnel System

(a) Intake
- Type of gate : Fixed wheel type
- Size of gate : 4.5m high x 3.5m wide

(b) Low pressure concrete lined tunnel
- Diameter : 4.5m
- Length : 5,150m
- Intake invert level : EL. 416k.75m

(c) Surge chamber
- Type : Restricted orifice typ
- Diameter of riser : 6.0m
- Diameter of chamber : 18.0m
- Height of chamber : 94m

(d) Penstock
- Diameter : 4.5m to 2.85m
- Length : 840m (approx)

(e) Valve house
- Valve type : Butterfly
- Valve diameter : 4.5m

(6) Power Station
- Type : Surface
- Number of units : 2
- Type of turbines : Francis
- Installed capacity of each unit : 60Mw
- Speed : 500rpm
- Rated head of turbines : 320m
- Rated discharge : 42.0m³/s
- Generators
  - Type : Conventional 3 bearing vertical
  - Cooling : Closed air/water cooled
  - Rating : 70.6MVA 60MW at .85 pf.
- Generator Transformer : 2 x 71MVA, 3 phase, 132/10.5KV
- Auxiliary Transformer : 2 x 800KVA, 10.5KV/400V Emergency
- Diesel Generator : 280KW, .8 p.f.
- Average Energy : 402GWH per annum
- Firm energy : 430GWH per annum

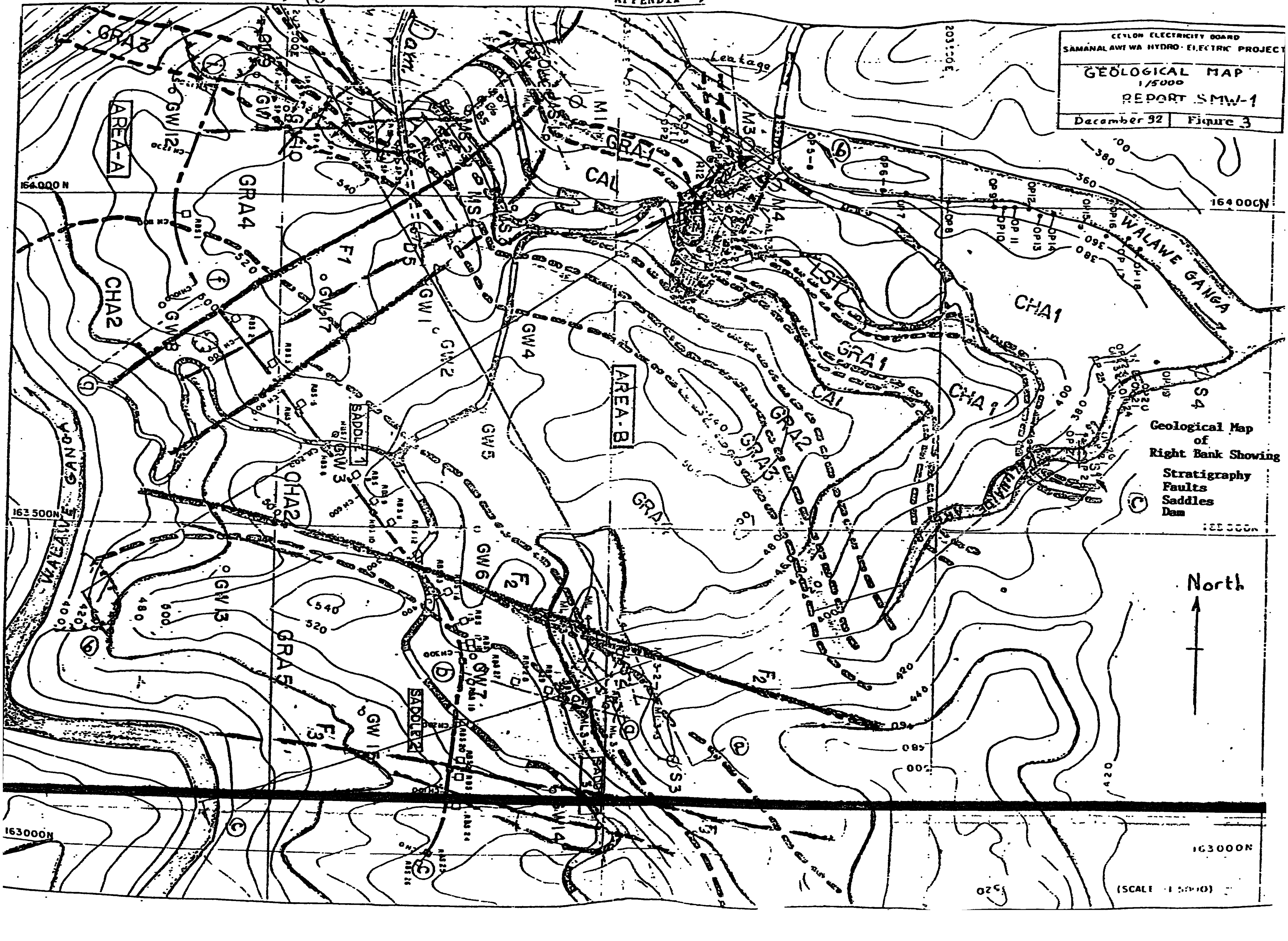
(7) Tailrace Canal
- Type : Open canal
- Length : 585m (from power station to the Katupat Oya)

(8) Power Transmission
- Type : 132KV, double circuit of Zebra Conductors
- Length : 17KM to Balangoda
- Switch Yard - 132KV Bay :
  - 4 feeder circuits
  - 1 bus coupler
  - 2*2 Generator Transformer Circuit

APPENDIX - 9

GEOLOGICAL MAP SHOWING
STRATIGRAPHY, FAULTS, AND SADDLES

APPENDIX 9

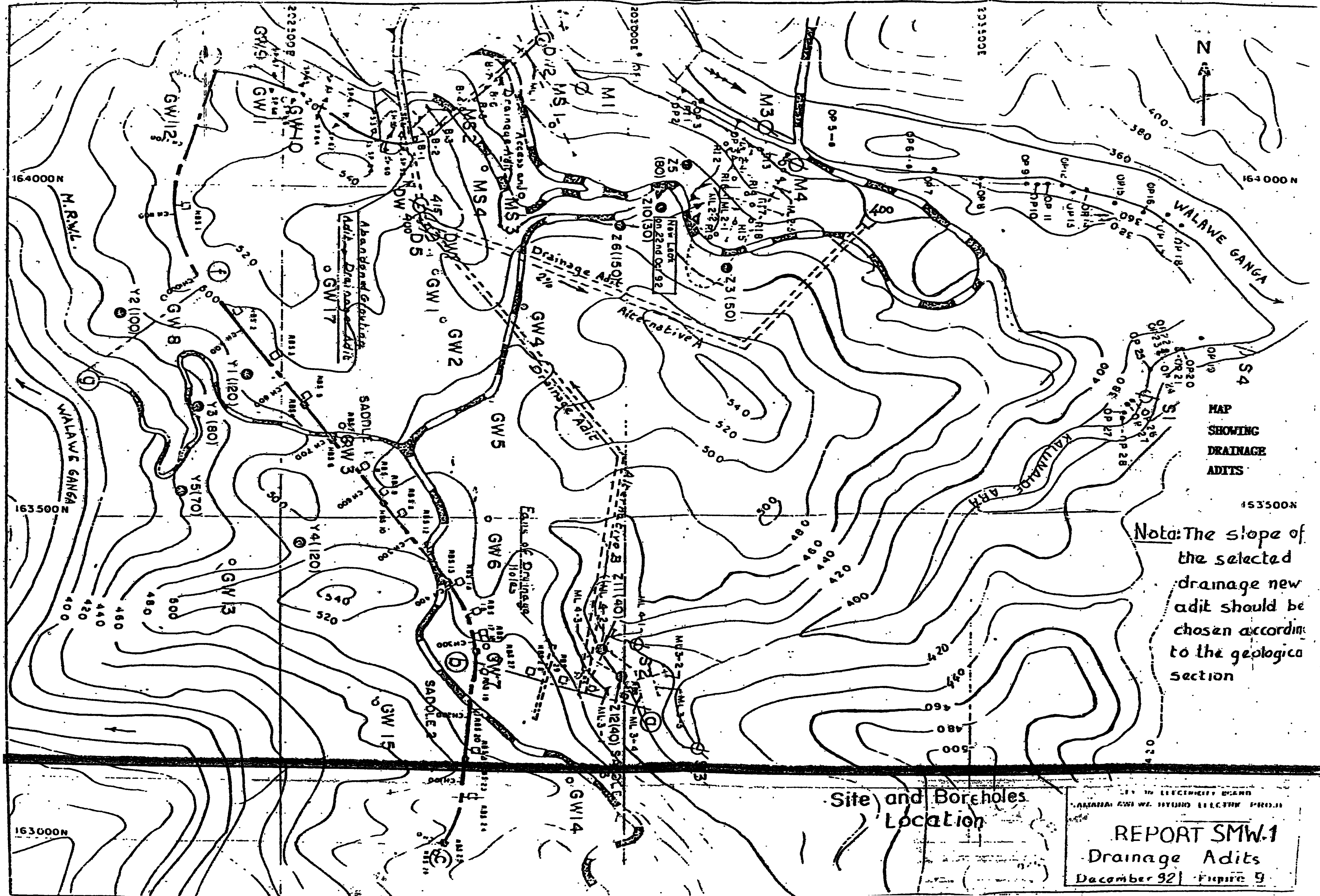

CEYLON ELECTRICITY BOARD
SAMANALAWEWA HYDRO-ELECTRIC PROJECT
GEOLOGICAL MAP
1/5000
REPORT SMW-1
December 92
Figure 3
Geological Map
of
Right Bank Showing
Stratigraphy
Faults
Saddles
Dam
North
WALAWE GANGA
AREA-A
AREA-B
SADDLE 1
SADDLE 2
CHA1
CHA2
GRA1
GRA2
GRA3
GRA4
GRA5
CA1
LST
F1
F2
F3
GW1
GW2
GW3
GW4
GW5
GW6
GW7
GW10
GW12
GW13
GW14
GW17
M1
M3
M4
MS
D5
S3
S4
Dam
Leakage
164000N
163500N
163000N
(SCALE 1:5000)

APPENDIX — 10

MAP SHOWING

DRAINAGE ADITS AND GROUT CURTAIN

APPENDIX 10



MAP SHOWING DRAINAGE ADITS
N
WALAWE GANGA
KALUNAIDE ARA
M.R.W.L.
Drainage Adit
Alternative A
Alternative B
Access and Drainage Adit
Abandoned Grouting Adit - Drainage Adit
Fans of Drainage Holes
New Leak on 22nd Oct 92
SADDLE 1
SADDLE 2
Note: The slope of the selected drainage new adit should be chosen according to the geologica section
Site and Boreholes Location
REPORT SMW.1
Drainage Adits
December 92 | Figure 9

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