A CASE HISTORY OF THE EXCAVATION OF SAMANALAWEWA POWER TUNNEL

by

A K D N Atukorale

Abstract

The Ceylon Electricity Board initiated the Samanalawewa Hydro- Electric Project to harness the waters in the upper reaches of Walawe Ganga by generating 120 MW of electrical energy. This project consists of the construction of a 94 m high rock-fill dam across Walawe Ganga, 5.4 Km long power tunnel and a 2*60 MW power station. This paper discusses the procedure adopted for the excavation of the power tunnel and the practical difficulties encountered while the excavation progressed.

1.0 Introduction

The Samanalawewa Hydro-Electric Project is situated about 160 Km from Colombo and about 10 Km east of the town of Balangoda (Annexure 1).

In the region of the project the Walawe Ganga and its tributary Katupath Oya flow in parallel in an easterly direction. At the point where the Walawe Ganga and Katupath Oya come closest to each other at a distance of 6 Km, the natural water level in the Walawe Ganga exceeds that of Katupath Oya by about 300m. The object of this project is to utilize this head difference for generation of electricity at a power station constructed close to the Katupath Oya.

The main part of the water conveyance system consists of a 5.4 Km long concrete lined tunnel having 4.5 m internal diameter and a downward tunnel gradient of 1%. The intake of the tunnel is situated 6 Km upstream of the dam at Watawala and the outfall in the middle of the Mahawalatenna - Kaltota escarpment.

The Design Engineers appointed for the entire Samanalawewa Hydro- electric Project were Sir Alexender Gibb and partners and EPD Consultants Ltd. both of United Kingdom. The Supervising Engineer was the Joint Venture Samanalawewa consisting of Nepon Koie Ltd, of Japan and Electrowatt International of Switzerland assisted by Central Engineering Consultancy Bureau of Sri Lanka. The contract for construction of Power Tunnel was awarded to the Balfour Beatty Construction International of United Kingdom.

2.0 Investigations

Detailed appraisal of anticipated geological conditions along the tunnel route have been made in a number of reports namely:

1966 - Technical Report by Engineering Consultants Incorporated
1973 - Technical Report by Snowy Mountains Engineering Corporation
1978 - Technical Report by Technopromexport
1984 - Technical Report by Balfour Beatty, GEC Energy Systems Ltd, Sir Alexender Gibb & Partners and EPD Consultant Ltd.

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He graduated from the University of Ceylon, Peradeniya in 1977 with BSc(Eng)Hons, and obtained the Diploma in Hydraulic Engineering (Delft) Specialising in Computational Hydraulics, in 1987.

He obtained Membership of The Institution of Civil Engineers (London) in 1981 and Membership of The Institution of Engineers (Sri Lanka) in 1982.

Since 1978 he has been attached to Central Engineering Consultancy Bureau and was seconded to Joint Venture Samanalawewa as a Senior Engineer is Waterways Section of the Samanalawewa Hydro-Electric Project from 1987 to 1991. Presently working as the Chief Resident Engineer for the construction of the Quay at Naval Base at Trincomalee.

The contents of these reports have been used to assess the anticipated geological conditions for the design of the tunnel route. The anticipated geological conditions were derived by extrapolating the geological data presented by Technopromexport (TPE) report along strike to the design line. The present tunnel alignment is approximately 300m east of the alignment shown in the TPE report at its maximum deviation at the surge chamber. (See Annexure 2).

According to the TPE report about 70 percent of the tunnelling would be in good rock with high strength. In the zones of discontinuities and in heavily marbalized limestones the ground water breakthrough into the tunnel was expected upto 100 litres per second and would not exceed 600 litres per second from the entire length of tunnel.

In addition to the above investigations seismic testing was carried out along the tunnel in selected weak zones in order to establish a suitable value for rock modulus to be used in design of tunnel lining reinforcements.

2.0 Geology

The tunnel trace leads across the southern limb of a large synform. Hence all the rock layers cross cut the tunnel with a moderate dip ranging from 30 to 60^{o} towards the upstream. All the rocks were subjected to similar tectonic conditions with localised variations. The foliation joints were prominent as the tunnel crossed the layering.

The main rock types encountered in the tunnel excavation were

1. Granulite and garnet biotite gneiss
2. Charnokitic gneiss
3. Quartzite
4. Calcic gneiss and crystalline limestone

Major discontinuities were almost normal to the tunnel trace and subvertical, hence the tunnel cross cutting area was minimum. A secondary set of subvertical joints occurred along the tunnel and were more discontinuous in nature. The tunnel trace crossed several major fault zones in the downstream half of the tunnel. Some of the fault zones were associated with calcic gneiss which had been affected by solution, to produce completely weathered material and/or solution features. These solution features were filled with water, and as a result sudden water inflows were encountered when the excavation hit solution features. In the worst case the initial flow exceeded 350 1/sec and came to an established flow of 260 1/sec. after a period of a few weeks.

The faults encountered in the upstream half of the tunnel were of deeply weathered zones which consisted of partially to completely weathered rock fragments associated with clayey material. The width of these zones varied from 10m to 100m at the tunnel level. Since the material in these zones was relatively impervious not as water bearing as those of the downstream half.

3.0 Setting out surveys

Setting out points for the tunnel excavation were established from primary control beacons, established in the project area using the Survey Department Trig. Stations. At the beginning of the excavation the tunnel face was marked at the portals from the setting out points established along the tunnel centre line on the surface. A theodolite and a level instrument were used to establish the tunnel grade and the center line. As the tunnel drive progressed, a laser beam with three targets was used to mark the face. The targets used consisted of steel plates with one 6mm diameter hole. The laser equipment and targets were mounted on brackets fixed to the tunnel wall. The laser beam and the holes in the targets were aligned by a theodolite with Distomat to give the line and grade of a reference point in the tunnel face.

Originally one laser beam per face was used at the spring level on the right hand side with the beam centered at the inner edge of the finished tunnel (inner face of concrete lining). As the work progressed specially for erection of steel ribs a second laser was installed on the centre line at crown level to give the inner face of the concrete lining. The distance from the face to the laser was limited to 150m as beyond this distance the scattering of the beam would cause too much inaccuracy. The laser instrument and the targets were shifted forward as the face advanced to stay within this limit. The tunnel face center was marked by measuring the offsets from the laser spots appearing on the face. Having marked the face centre, the face profile was marked using a tape or a marking pole. Once the profile was marked the shot holes were marked by the shift bosses according to the drilling pattern. Due to the presence of underbreaks at several occasions in the initial stages the radius of the outer pitch circle was increased by 100 mm for perimeter holes. This enlarged profile was helpful to the Contractor to accommodate the ground supports in the weak zones.

Setting out of bends was done by the theodolite and level instrument at each face advance. When a sufficient length of straight tunnel was excavated beyond the bend (more than 50m) a laser beam was re-established to mark the tunnel face.

4.0 Excavation

4.1 Method of excavation

The excavation of the 5.4 Km long 5.2 m diameter horse-shoe shaped tunnel was done by conventional drill and blast method with two faces from the adits near to the Intake and Outfall. Full face excavation was done in fresh and moderately weathered rock with hand held pneumatic rock drills with pusher legs on a three deck platform. An average of 12 - 15 nos. rock drills were used at the face having 4 - 5 nos. in each deck. Whenever the rock was highly to completely weathered and full face excavation was difficult then top heading and bench method was adopted. The full size top heading with its floor at spring level was maintained with a bench of maximum 5.0m long. Usually the heading was blasted with light charges and the bench was excavated with pick hammers and the mucker. The completely weathered areas were excavated using a "Hymac" backactor which was mounted on a Plymouth 25T locomotive engine and was suitably converted for tunnel use. After introducing this machine the face and roof collapses were reduced and the progress was increased because the time duration between face opening and application of temporary supports could be minimised.

The drilling pattern used for full face blasting was generally 87 nos. of 37 mm dia 2.4 m deep holes with one 75 mm uncharged relief hole at the centre. The pneumatic rock drills were operated with compressed air supplied through the service pipes from the static compressors installed outside the tunnel.

The explosives used was Dynamit Nobel 65% and 80% for easer and lifter holes and Profilex was used for perimeter holes to control overbreak. Dynamit Nobel was delivered in 25mm dia and 400 mm long sticks and Profilex was supplied in 600 mm long tubes with connecting sockets. The total explosives used for full face 2.4 m long drill holes were 110 Kg of which 8.5 Kg was Profiles. The quantity of explosives and number of drill holes were varied slightly, depending on the type of rock and the availability of discontinuities in the face. The powder factor ranged from between 1.23 to 2.88 Kg/m^3 with an average of 2.16 Kg/m^3 over the entire tunnel length.

"NONEL" (Non Electric) detonators were used to detonate the round with different delays. The NONEL detonator consists of a delay detonator and a plastic tube which is fully sealed and contains a small amount of explosive to transmit the shock wave. These detonators have the advantage of not being affected by stray induced electrical currents. Once the face has been charged, rock bolting can be carried out to ensure completion prior to the firing to save time.

The first 400 m of both tunnel drives were driven prior to the construction of site access roads, which made the installation of the rail equipment in the tunnel impossible. During this period mucking was done with pneumatic tyred front end loader "Cat 966". Once the rail equipment was installed, the mucking was done with electrically operated "Godman" convey muckers. Muck cars having capacity of 10 m^3 were filled by the convey mucker and those were toed by 15T and 25T diesel locomotives and 10T battery locomotives. A 75 m long California switch with two tracks was used to change the much cars while filling. The buckets of much cars were of the side tipping type and the much tipped outside the tunnel was handled by pneumatic tyred lorries to the spoil tips.

For tunnel ventilation "Kaffmann" fans with a diameter of 1400 mm set in pairs blew fresh air to the face through 1700 mm dia. flexible canvas ducts. A booster fan was installed at Ch. 1500 approx. as the excavation of the downstream drive was foreseen up to Ch 3000 approx.

The main service lines into the tunnel were

1. 150 mm dia. Victolic pipes for compressed air
2. 100 mm dia. Victolic pipes for service water
3. 3.3 Kv power cables

The compressed air supply was provided from 4 nos. 750 cfm Atlas Copco static compressors installed outside the tunnel for each drive. Water supply was from the storage tanks which were placed well above the tunnel portals to obtain required pressure and the storage tanks were fed from the Walawe river or from the Diyawini stream. When the Diyawini went dry, after the tunnel hit water the storage tank was fed from the water running from the tunnel.

The power supply to the tunnel was tapped from the National Grid but standby diesel generators (Wyes Power 500 Kva) were installed outside the tunnel. The main supply into the tunnel was through 3.3 Kv cables and step-down transformers (300 Kva) were installed inside the tunnel at 600 m intervals approximately. These transformers provided the power supply for tunnel illumination (110 V) and three phase supply for the dewatering pumps, ventilation fans etc. A transformer mounted on a mobile gantry was used to provide power supply to the convey mucker and the illumination of the drilling gantry.

4.2 Excavation progress

Although the tunnel excavation commenced prior to the schedule (Annexure 2) the progress was hindered due to weak zones and water inflows encountered and more time was spent on support measures. At the end of the excavation a delay of 10 months from the anticipated breakthrough date was observed with the downstream drive extending further 700 m due to the slow progress achieved in the upstream drive.

In the fresh rock a maximum weekly progress of 72.4m was achieved with an average weekly progress of 48.52 m. The table below gives the average progress rates achieved for different rock support classes over the entire tunnel length.

PROGRESS IN DIFFERENT ROCK SUPPORT CLASSES

ROCK SUPPORT CLASS	I	II	III	IV	V	VI	VII	S
AVERAGE WEEKLY PROGRESS (m)	48.52	35.66	30.38	16.00	9.41	7.80	13.67	8.95

5.0 Ground supports

The support system used during the excavation of the tunnel were based on one or more of the following elements.

1. Rock bolts
2. Sprayed concrete with or without wire mesh
3. Steel ribs and laggings

Eight rock support systems were developed as a combination of the above elements depending on the ground conditions encountered during excavation (Annexure 3).

The ground support was constructed immediately following the excavation and the New Austrian Tunneling Method (NATM) was used where appropriate, consisting mainly of a shotcrete lining of varying thickness reinforced by several methods such as 200*200 mm square mesh and lattice arches, and continuous deformation measurements.

Additionally, provisions were made for the use of other methods when appropriate. In the tender five rock support classes were detailed by the Design Engineer. Eight classes had eventually to be adopted to accommodate for the difficult geological conditions encountered during excavation. The type of rock support to be used was decided jointly by the Section Engineer of the Supervising Engineer and the Tunnel Agent of the Contractor. In practice no difficulties were encountered in this process.

5.1 Rock bolts

The rock bolts used were of resin or cement grout type with varying length from 2.4 to 5.0 m. Resin bolts which were mostly 2.4 m long were used mainly in hard rock where the end anchorage was possible to secure the rock against block failure. Cement grouted bolts were mainly used in weathered ground to form a ground arch around the tunnel with shotcrete lining. 4.0 to 5.0 m long grouted bolts were used in the side walls below the spring level in highly to completely weathered ground to avoid possible wall collapses. In addition "Swellex" bolts, consisting of steel tubes, inserted in the predrilled holes and subsequently "blown" up with pressurised water thereby giving friction adhesion along entire length of the bolts, were specially ordered for use in water bearing zones where resin or cement grouted bolts were less effective.

5.2 Shotcrete

In the Samanalawewa tunnel nearly 60% of the tunnel length has been provided with shotcrete lining with varying thicknesses. The dry shotcrete method was selected by the Contractor for convenience. Transport of shotcrete material into the tunnel was in "Stabilator" trixer plants with two separate bins for storing of aggregate mix and cement. The discharge of aggregates and cement to the shotcrete machine is through helical screws. The aggregate screw rotates at a constant speed whereas the speed of the cement screw can be controlled to attain the required aggregate/cement mix proportions. Calibration (adjustment of cement screw) of trixers was done on a weekly basis during weekends to avoid disruption to the face work.

An accelerator was added for shotcrete to achieve quick initial strength to avoid the collapse of unset shotcrete layers due to water pressure developed behind the layer and to reduce the waiting time for the next blast. Initially the Contractor used "Cormix GA2" which is supplied in power form and was added at the shotcrete machine. As experience was gained, it was observed that the initial strength reached with "Cormix GA2" was not even sufficient to avoid collapsing of the shotcrete under its own weight, when a thicker layer was applied. This forced the Contractor to wait before being able to apply the next layer of shotcrete. The same difficulty arose in areas with heavy seepage.

To overcome this situation the Contractor made trials and used another accelerator "Attack M1" a Japanese product, which resulted in higher initial strength. This gain in initial strength however was to the detriment of ultimate strength (28 day strength) of shotcrete which reduced considerably with the increase of accelerator dosage. A series of trials was carried out to establish a suitable percentage of accelerator to optimum between initial and ultimate strengths. But due to the non- availability of a sophisticated dispensing system accelerator dosage could not be controlled to the required level of accuracy. Therefore the 28 day strength of 20 N/mm^2 achieved from the insitu shotcrete was accepted, and the thickness of the shotcrete layer was increased to compensate for this reduced ultimate strength.

5.3 Steel ribs

Two types of steel ribs were used as part of the ground support for Samanalawewa tunnel. The heavy steel ribs were fabricated from 152 x 152 mm x 37 Kg/m U.C and the lighter ribs were fabricated from 114m x 114 m x 26.78 Kg/m joists. The heavy ribs were mainly used in the collapsed zones and areas where heavy water flows were encountered with completely weathered rock. These ribs consist of 4 pieces namely two crown parts and two legs. In the case of heading and bench excavation the two crown parts were installed during the top heading excavation and the legs were installed as the bench advanced. Steel or concrete laggings were placed between ribs and the gap behind the laggings was filled with bags containing dry shotcrete mix. In some instances dry shotcrete was blown into the void behind laggings.

The light weight ribs were mainly associated with the shotcrete lining. The method of installation was the same as the heavy ribs but the space between the ribs was shotcreted with reinforcing mesh and rock bolts where required, to form the ground arch.

Spile bars were driven ahead of the face when there was danger of a roof until the first layer of shotcrete was applied. Triangular lattice arches or crown collars with rock bolts were used to support the rear end of spile bars. These lattice arches consisted of four arched pieces with a triangular cross section fabricated from 16 or 25 mm diameter high yield steel. Drain holes were provided in the shotcrete lining where the ground was damp and seepage was observed to avoid pore pressure developing behind the lining.

6.0 Convergence Monitoring

The basic principle of the NATM is that the rock masses surrounding the tunnel are considered plastic and that deformations to a certain limit are accepted. This principle results in considerable economy of required ground support, but necessitates constant monitoring of the deformations during the initial period after the excavation, until the rock masses around the tunnel void have ceased shifting under the new load pattern, created by the void and no further supports are required before equilibrium is achieved again. In the Samanalawewa tunnel this principle was applied mainly in strongly weathered areas, where an initial support considered basically of wire mesh reinforced concrete with a minimum of bolts.

To conform the adequacy of such an initial support, the shotcrete shell was regularly observed for the appearance of cracks, which would indicate excessive deformations. As soon as cracks in the shotcrete were detected, convergence measurement stations were set up in the area. Each station consisted of two eye bolts installed approximately 1.0 m below the spring line across the tunnel and a roof bolt. The convergence measurements of the "eye" bolts were made with a tape extensometer and the roof level was measured with an "Engineers" level. The measurements were taken initially on a daily basis, reducing to a weekly and monthly interval. In the places where the deformation increased gradually, and ebbing out could be observed, additional supports were installed which included 5.0 m long cement grouted bolts in the walls and placing of an invert arch in several instances. (The deformation of a typical convergence array is shown in the Annexure 4).

7.0 Dealing with Water

According to the TPE report large water inflows up to 350 1/sec were anticipated in the downstream drive between CH. 600 - 1180. To accommodate such large water inflows the contractor constructed sump niches on the L H S tunnel wall with the sump below the tunnel invert. The first pump sump of this type was excavated at CH 375 in sound rock with a capacity of 50.0 m^3. approx. These pump sumps were provided with two 250 mm dia "Flygt" submersible pumps each having a capacity of 200 1/sec. These pumps were connected to two 250 mm dia pipe lines which extended to the adit junction where the rest of the tunnel is down grade. Altogether four pump sumps were established in the downstream drive approximately 500 m apart. The sumps were provided with float switches to activate the pump when the water level in the sump reached the limiting levels.

As the down stream drive extended beyond the anticipated breakthrough chainage (CH 2370) the fifth sump was established in a natural cavern encountered on the R H S of the tunnel at CH 2519. An unforeseen sudden water inflow was encountered during excavation at CH 3055 with a initial flow of 82 l/sec. As the dewatering pumps available at the face could not cope with this flow, the water started collecting at the face to a depth of 2.0 m and the water reached 200 m backward before the dewatering pumps were installed. After operating a 250 mm dia. pump with the necessary pipe line the water level was reduced and the face work resumed.

No special arrangements were proposed in the upstream drive because the anticipated flows were low and the water draining under gravity. However high water inflows encountered in the upstream drive with a maximum initial flow of 350 l/sec at the face hindered the progress of the excavation. The silt which was carried by this water inundated the rail track and water started flowing over the full width of the invert, causing difficultics to the movements of the locomotives. To overcome this situation water was pumped from the face to the adit junction using two 250 mm dia "Flygt" submersible pumps. In the meantime two side drain channels were constructed along the tunnel to handle a maximum flow of 350 l/sec. As the construction of the drain channels progressed, the over pumping of water was limited from the face to the nearest end of the drain channel.

8.0 Probing ahead of the Tunnel

As the information of ground conditions provided by bore holes is never complete, it is generally prudent to drill forward from the advance face to explore the ground conditions ahead of the face. In the Samanalawewa tunnel probing ahead of the face was included in the specifications in order to avoid major failures and damages during excavation. 64 mm dia drill holes were drilled from the face using a rotary percussion truck drill machine mounted on a rail bogey. The probe drilling was usually done at the end of the last shift before the weekend stoppage to avoid interruption to face advancing. The length of the probe was selected to suit one weeks progress, which of course varied depending on the prevailing ground conditions at the face or the conditions encountered while probing. During probe drilling, the time of penetration for each meter drilling and the colour of flushing water was observed. The samples of flushing materials were also collected for each drill rod and any water inflow or loss monitored. In places where very high flows were encountered the water pressure and approximate flow rate in the probe hole were measured. The rate of drilling indicates the quality of rock ahead or presence of cavities. All these data were used to interpret the type and quality of rock and any irregularities in the ground ahead. Depending on these results the method of excavation was adjusted and precautionary measures were taken to suit the anticipated ground conditions.

9.0 Problems encountered during excavation

A series of roof collapses were encountered in both drives during excavation specially after the blasting of the face. To overcome the progressive collapse of the roof and formation of a chimney above the crown the pile of collapsed material was shotcreted with the exposed part of the face. After the collapse thus was stabilized temporarily, 150 x 50 mm steel channels were driven along the periphery of the crown as forepoling. Then the collapsed material was removed gradually and a suitable support system (usually steel ribs) was installed. In most of these cases the excavation was then proceeded by the top heading and bench method.

Several wall collapses behind the face also occurred a few days after the completion of the support system. In these areas the ground was completely weathered and quite dry during excavation. As the excavation progressed the water from the services started percolating into the ground and the invert started deteriorating due to absorption of water. The movement of heavy equipment such as the mucker and locomotives enhanced further deterioration causing the failure of the shotcrete shell inward. In places where this movement was slow and gradual and could be detected by the convergence measurements, additional supports such as 5.0 m long cement grouted bolts and invert shotcrete was applied.

In some instances where the failure was sudden such as within a few hours after the deformation was observed, and no time was available to provide additional support, vertical and horizontal struts were provided quickly to avoid a major collapse. For this purpose 150 mm dia GI pipes were used, filled with cement grout to stand higher loads. After stabilizing the collapse, the deformed supports were removed and a new support system was installed while removing the temporary struts. In one such occasion the wall had moved 300 mm into the tunnel. After installing a new support system convergence measurements were taken until the readings showed a steady reading for a reasonable time duration. Fissure grouting was done before the removal of the temporary supports to stabilize the deformed ground in places where such ground movements occured.

The other major problem mainly in the upstream drive was the high water inflows encountered either during drilling of probe holes or immediately after blasting the face. Large amounts of water seriously hindered the progress of the drive and the maximum flow had amounted to 350 1/sec. measured 100 m behind the face. Performances were considerably reduced and a series of collapses also occurred. The extensive and difficult reinstatement work lasted for several weeks. The inflows of water gradually decreased and reached a stabilized flow of 265 1/sec. after completing the excavation of entire tunnel length measured at the portal.

The downstream drive was relatively dry compared with the water inflows encountered in the upstream drive. On several occasions, krastic solution cavities were encountered of which some of them extended more than 25 m along the foliation, having variable cross sections. Most of these cavities were filled with silt and water and it was understood that the initial very high flows were due to breaching of these water pockets.

10.0 Impact on the Environment

As a result of heavy water inflows into the tunnel which was nearly 300 1/sec. over the upstream drive the ground water table in the vicinity of the southern half of tunnel was drawn down to tunnel level as observed from the water levels monitored in a bore hole drilled close to the tunnel trace. Some of these water sources seem to have formed a network of underground streams along the solution cavities in limestone bands in the area. About four months after the tunnel hit water, some perennial springs situated about 2-4 Km west of tunnel trace dried up. All these springs had hard water indicating that this water is flowing through some calcic bands. The Diyawini Oya which crosses the tunnel trace also dried up during dry weather periods at this time. Another stream called Arambe Ara which is situated at the toe of a quartzite layer also went dry when the tunnel hit the quartzite band and drained water into the tunnel. A bore hole situated 500 m west of tunnel trace which previously flowed under artesian pressure, also went dry during the tunnel excavation.

As a result of the breaching of the ground water table, people living in the area were unable to find water for their domestic requirements and their cultivations which was their main income. In the villages of Imbulamura and Godakumbura the paddy fields, which were fed by the water from the springs, were abandoned. The paddy fields in Handagiriya which were situated at the toe of the escarpment and fed from the above springs were also abandoned. As a result of the drying up of the Diyawini, paddy fields downstream of the Diyawini falls at Dorawela, Kaduruwa, Henegama and Pabbarapota were also abandoned. The total area of paddy fields abandoned due to this depletion of the ground water table in the vicinity of the tunnel was 800 Ha. approximately. Considering these facts the Client (CEB) made arrangements to supply drinking water in bowsers and compensations were paid to those who lost their cultivations due to the nonavailability of water.

It is noteworthy that approximately one month after the completion of the concrete lining in November 1990, the ground water level observed in the above described bore hole, started to rise at the rate close to one metre per week. After the completion of grouting works in the tunnel in February 1991, this rate increased to around two metres per week until begining of June 1991, when the curve started to flatten out, as the ground water level approached original levels of the preconstruction period. By October 1991, the ground water level all but ceased to rise further, and with the old springs and wells bearing water again, it can be assumed that ground water conditions of before the tunnel drive have eventually re-established themselves as predicted.

11.0 Conclusion

The excavation of the 5.4 Km long power tunnel took 33 months to complete. The downstream drive extended 700 m beyond the anticipated breakthrough chainage due to the slow progress of the upstream drive. The support measures during excavation had to be increased by over 340 percent from the original foreseen quantities, resulting in considerable extension of construction time. Several face work suspensions due to collapses and dealing with excess water inflows also contributed to the additional time required for excavation work. Due to the adverse ground conditions encountered in the original tunnel route a revision was made to deviate the tunnel, the resulting bends themselves causing time extensions for negotiating them and for extended tunnel length.

The progress was also hindered by the situation prevailing in the country in 1988 and 89 resulting in more than sixty unproductive days. Apart from the construction difficulties, more than a month per face was also lost due to an industrial dispute originated between the expatriate staff and local work force. With all these hindrances the tunnel excavation was successfully completed on March 22, 1990 only ten months behind the original scheduled completion date.

12.0 Acknowledgement

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Mr. P. Molinari, Deputy Construction Manager, JVS
Mr. P.o. Squares, Senior Engineering Geologists, GIBB

13.0 References

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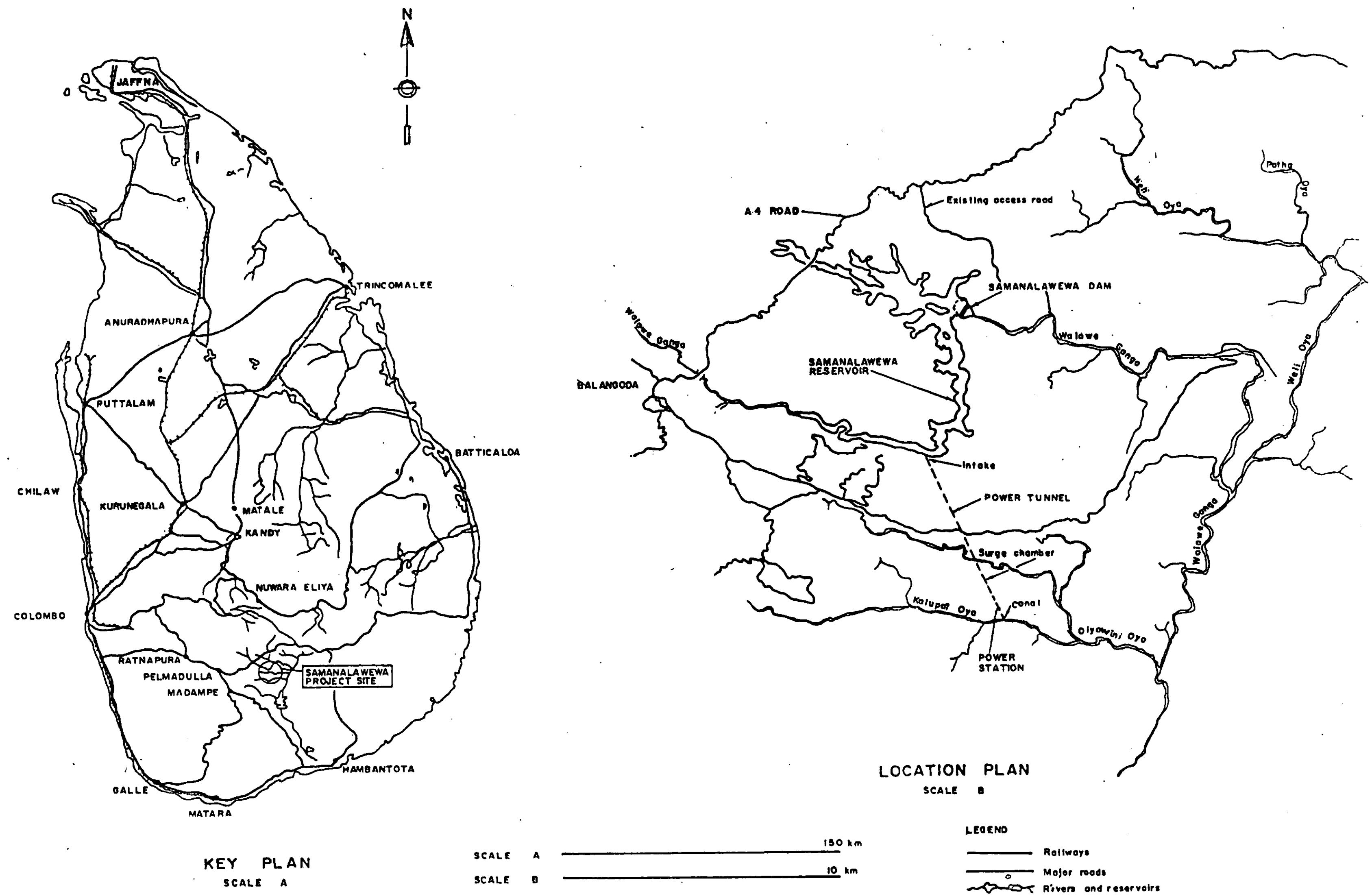
Balfour Beatty Ltd., GEC Energy System Ltd., Sir Alexender Gibb & Partners and EPD Consultants Ltd. (1984) - Unpublished property of CEB.

ANNEXURE 1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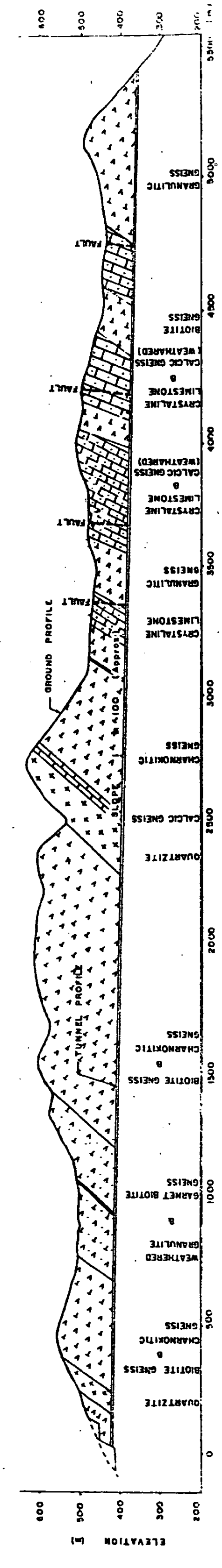
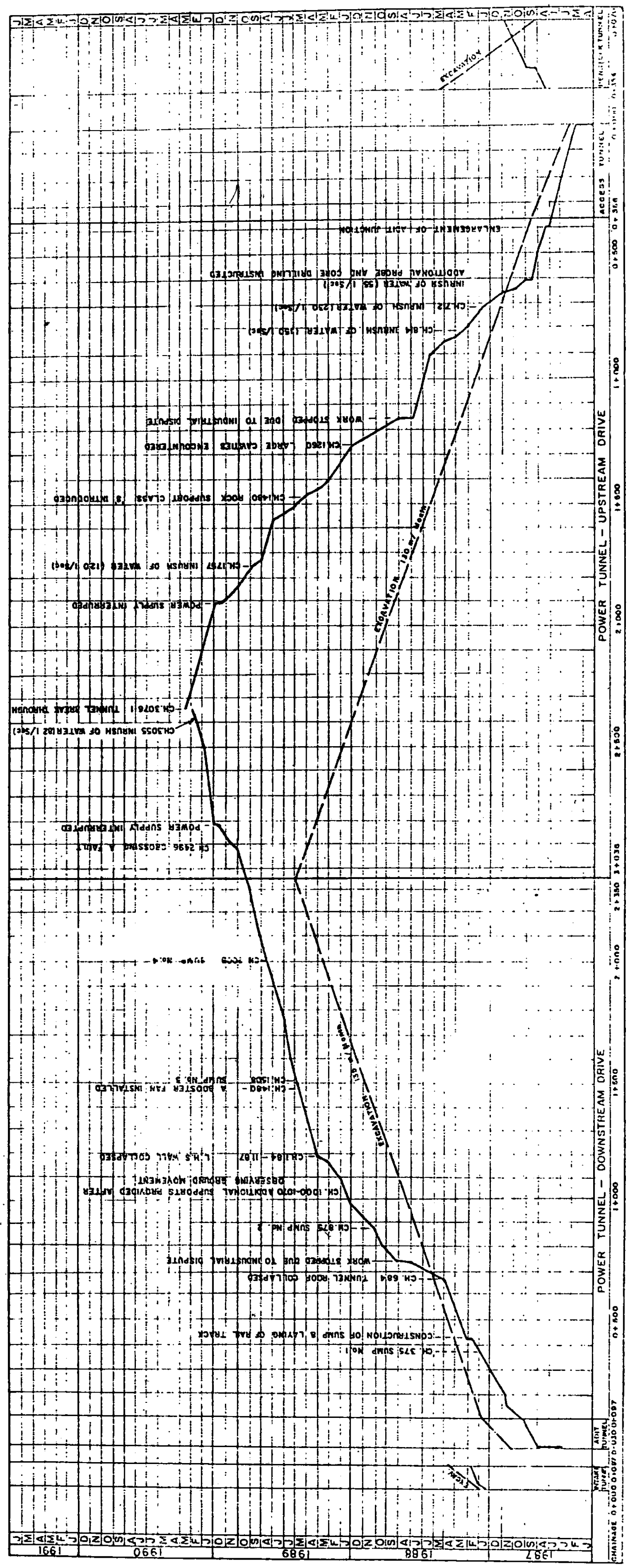
JAFFNA
TRINCOMALEE
ANURADHAPURA
PUTTALAM
BATTICALOA
CHILAW
KURUNEGALA
MATALE
KANDY
NUWARA ELIYA
COLOMBO
RATNAPURA
PELMADULLA
MADAMPE
SAMANALAWEWA PROJECT SITE
HAMBANTOTA
GALLE
MATARA
KEY PLAN
SCALE A
SCALE A 150 km
SCALE B 10 km
A-4 ROAD
Existing access road
Weli Oya
Potha Oya
SAMANALAWEWA DAM
Walawe Ganga
SAMANALAWEWA RESERVOIR
BALANGODA
Intake
POWER TUNNEL
Surge chamber
Canal
Kalupat Oya
Diyawini Oya
POWER STATION
LOCATION PLAN
SCALE B
LEGEND
Railways
Major roads
Rivers and reservoirs

ANNEXURE 2

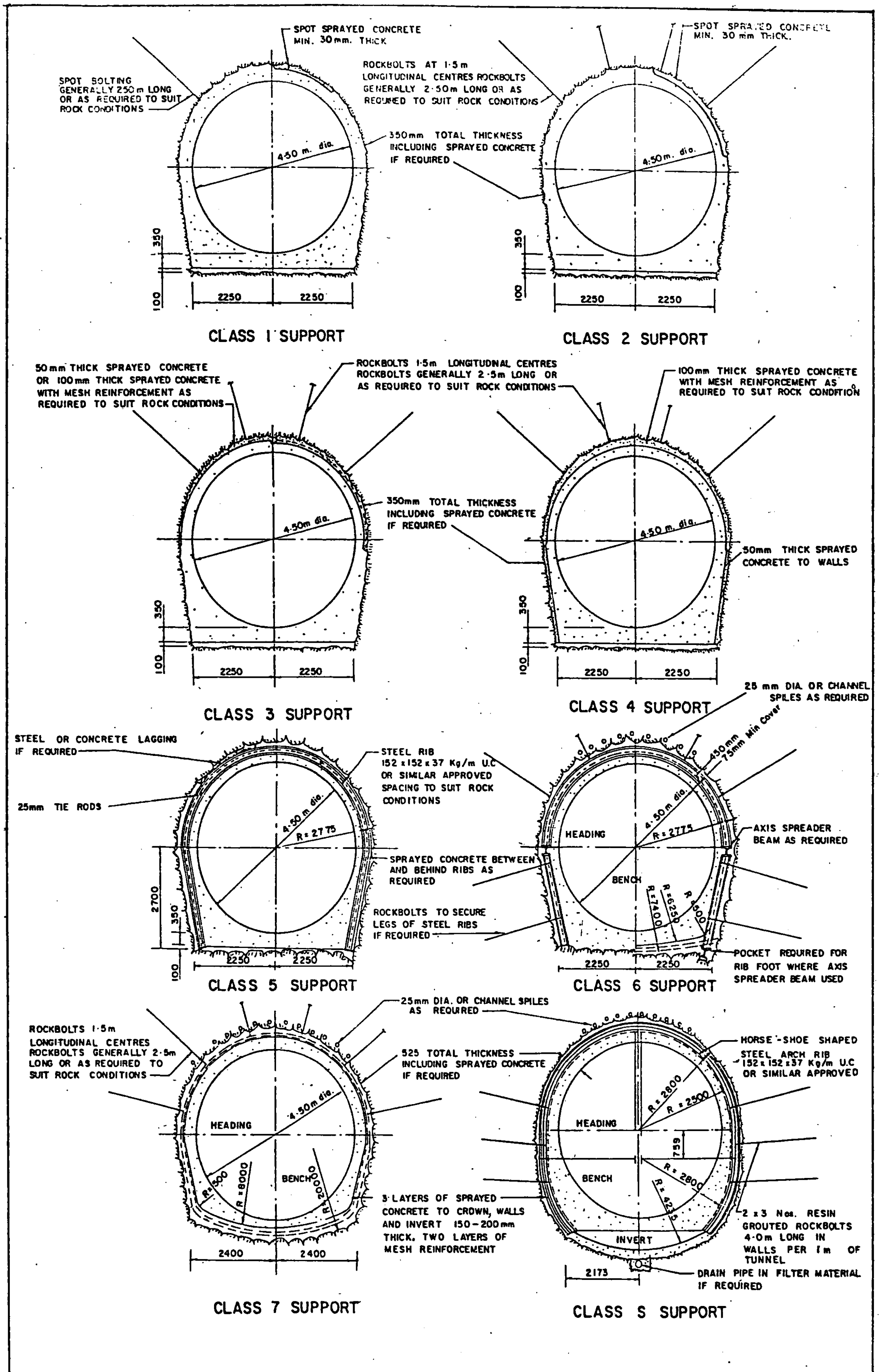


LONGITUDINAL SECTION ALONG TUNNEL TRACE



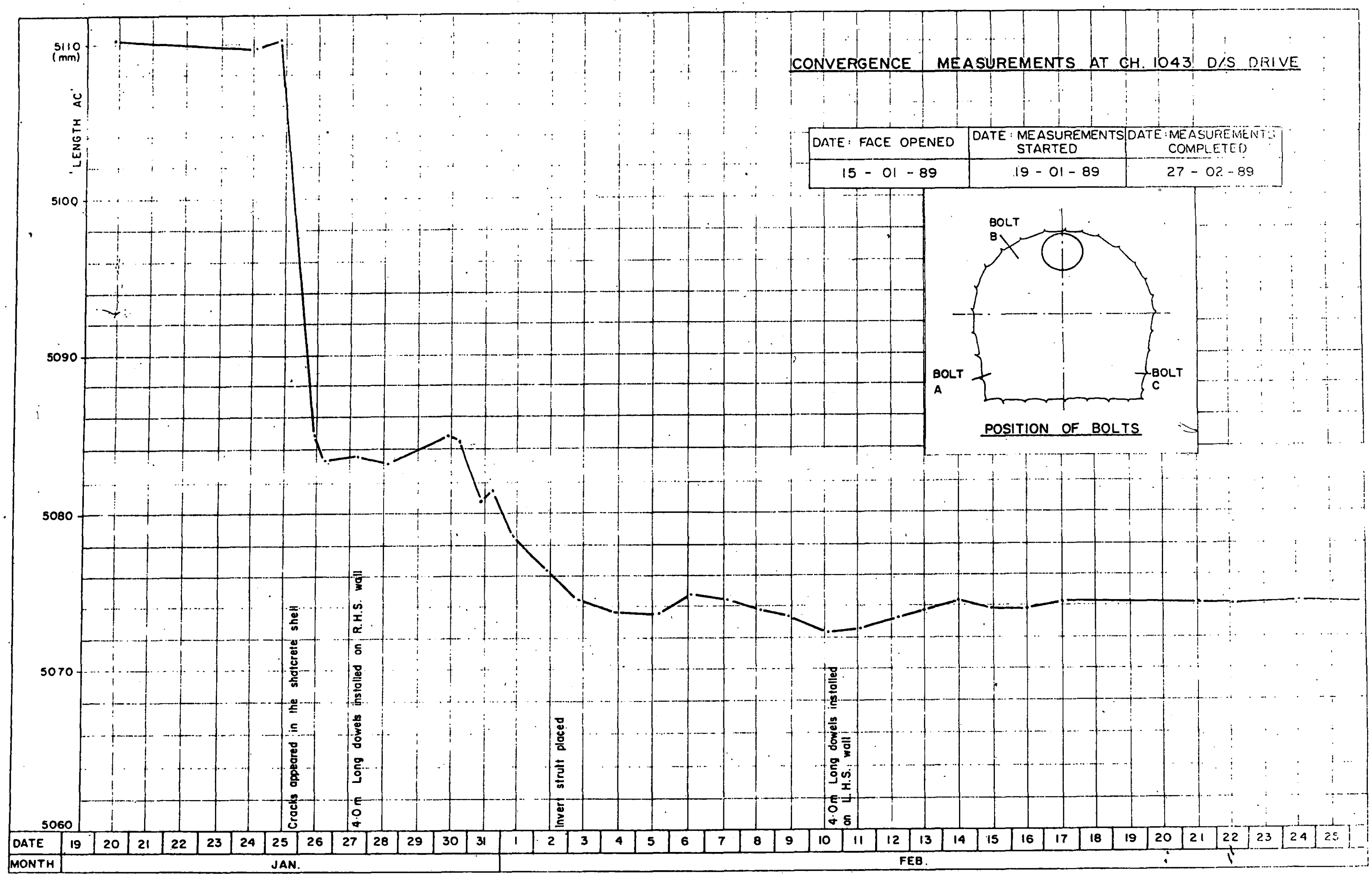
PROGRESS OF TUNNEL EXCAVATION

ANNEXURE 3



CLASIFICATION OF ROCK SUPPORT CLASSES

ANNEXURE 4


CONVERGENCE MEASUREMENTS AT CH. 1043 D/S DRIVE
DATE : FACE OPENED
15 - 01 - 89
DATE : MEASUREMENTS STARTED
19 - 01 - 89
DATE : MEASUREMENTS COMPLETED
27 - 02 - 89
BOLT B
BOLT A
BOLT C
POSITION OF BOLTS
5110 (mm)
'LENGTH AC'
5100
5090
5080
5070
5060
Cracks appeared in the shotcrete shell
4·0 m Long dowels installed on R.H.S. wall
Invert strut placed
4·0m Long dowels installed on L.H.S. wall
DATE
19 20 21 22 23 24 25 26 27 28 29 30 31 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25
MONTH
JAN.
FEB.