Paper No. 512

TARBELA DAM PROJECT SEEPAGE PROBLEMS AND REMEDIES

BY

Dr. Izhar Ul Haq

&

Jawed Ahmed

Paper No.512

TARBELA DAM PROJECT

SEEPAGE PROBLEM & REMEDIES

By

Dr. IZHAR UL HAQ*

&

JAWED AHMAD**

SYNOPSIS

Tarbela Dam Project is founded on alluvium more than 700 ft. deep. Although the sandy gravels and cobbles are quite dense in place but they contain open work (skip graded) layers. A positive cut off to the bed rock with grout curtain, cut off trench or slurry trench was not feasible. It was for the first time that 480 ft. high dam extending about 9000 ft. across the main river valley was designed with an upstream impervious blanket in extention of the core of the dam to control seepage and exit gradients. An impervious blanket generally extending 7500 ft. upstream of centreline of dam was laid over the river alluvium. The contact of the blanket at the rim was meticulously designed and carried out.

During the summer of 1974, the reservoir was filled for the first time to 80% of its maximum height when a mishap occured in Tunnel 2, requiring emptying of the reservoir. Many wells in the downstream area flowed more than 1 ft^3/Sec. when reservoir reached EL 1460 ft. On emptying the reservoir, the exposed upstream blanket revealed 362 sinkholes 1 to 40 ft. (0.3 to 12 m) in diameter as well as 140 cracks in the blanket. The sinkholes and cracks were capped with additional blanket material and the blanket thickened in its mid-section during the low-flow season September 1974 - May 1975. In order to facilitate drainage with minimum head loss additional wells in the existing well line and an additional line of wells downstream of existing line was installed. After refilling the reservoir, the development of additional sinkholes in the blanket was monitored by side-scan sonar and ORE profiling. Bottom dump barges were mobilized so that any additional sinkholes that might develop could be treated. During the 1975 filling, 350 new sink holes formed in main blanket. Thereafter the development of sinkholes decreased substantially and by 1978 had practically ceased. Two sinkholes developed on the upstram slope of the main embankment, one

*Chief Engineer DMO, WAPDA.
**Director (Geotechnical) DMO, WAPDA.

each in 1976 and 1984. Four sinkholes devloped on the upstream slope of Auxiliary Dam No.1, three in 1977 and one in 1986.

Seepage problem was faced in both the left and right abutments of main dam. Additional grouting curtains were provided from additional/existing adits. Additional drainage was provided from the drainage galleries and adits. The drains which passed fines were grouted and replaced with adjacent drains equipped with filter.

The paper describes the seepage problems, remedies and the behaviour of the dam. The performance of the main dam, Auxiliary dams and the abutments has been satisfactory. Vigilent monitoring and surveillance has been the key factor. Whenever any abnormality occurred precautionary and remedial measures were taken immediately.

INTRODUCTION

Tarbela Dam one of the biggest earthfill dam is built on a very deep and difficult foundation of alluvium. The dam is located on the Indus River at about 70 km northwest of Rawalpindi. The project provides 9.4 million acre ft (11.5 km^3) of stored water each year for irrigation which started in 1976. The first 4 units of 175 megawatt each on tunnel No.1 went on line in 1977. Six units of 175 megawatt each were added on tunnel No.2 during 1982-85. Four units of 432 megawatt each are being installed on tunnel No.3 and would be operative in October 1991. The layout of the project is shown in Fig-1. It consists of a Main Embankment Dam and Upstream Blanket: Auxiliary Embankment Dam 1 and Auxiliary Embankment Dam 2; a gated Service Spillway of 615,000 ft^3/sec capacity and an Auxiliary Spillway of 795,000 ft^3/sec capacity; four tunnels on the right abutment and one on the left abutment. Releases from the tunnels and the spillway return to the river downstream of the dam for withdrawal for irrigation. Salient features of the project are given in Table-1.

After the initial filling of the reservoir in 1974, the project experienced several problems. On emptying of reservoir due to mishap in Tunnel No.2, the upstream blanket was exposed which revealed 362 sinkholes 1 to 40 ft in diameter as well as 140 cracks in the blanket. The sinkholes and cracks were capped with additional blanket and the blanket thickened and extended. After refilling the reservoir, the development of additional sinkholes in the blanket was monitored by side scan sonar & ORE profiling. Bottom dump barges were mobilized so that any additional sinkholes that might develop could be treated. The paper describes the seepage problems, the implications of open-work in the foundations, the design of the blanket, the formation of sinkholes and presents analysis of the extensive piezo-metric and seepage data collected through its operation of 14 years. Piezo-metric pressures and seepage quantities have indicated satisfactory behaviour

under the similar tail water conditions. The under seepage quantity which was 177 cusecs in 1977 reduced to 101 cusecs in 1980 and it further reduced to 6 cusecs in year 1987

GEOLOGY

Tarbela Dam is located in the Hazara Hills, which are part of the mountain group Himalayas. The Hazara Hills are composed of crystalline and metamorphic rocks with non-fossiliferous sedimentary deposits and gabroic intrusions, all ranging in age from Precambrian to Permian. The present Geologic structure is the result of extensive folding, shearing and faulting associated with regional crystal deformation arising from the northward subduction of the Indian Sub-continental Plate below the Eurasian Plate.

There are three distinct geological formations at the Tarbela dam site: the Salkhala Formation, forming the right bank; the Hazara Formation forming the bedrock base of the Indus Valley; and the Kingriali Formation, forming the left bank. The general orientation of bedding indicates that the banks of the river are the limbs of an anticline, the axis of which has been eroded by the Indus River. The geological mismatch between the right and left banks is considered to be the result of displacement along a nearly vertical fault running along the right side of the valley. The right and left banks are separated by the 6,000 ft. (1.8 km) wide flood plain of the Indus River. The river flows past the site in a braided stream pattern on alluvial deposits at about El.1110(338 m), See Fig: 2.

Right Bank Geology

The right bank (including the MED right abutment, Tunnels 1,2,3 and 4 and the power house foundation) is formed by the Salkhala Formation, which is characterized by an extreme variety of rock and intense tectonism. It includes an euxinic (deposited in an anaerobic environment) calcareous member which overlies a chloritic schist member. An overthrust has cut the previously folded strata of the formation into wedges and heaped them into a pile. The resulting structure consists of a group of overturned and faulted folds, oriented generally in an upstream-downstream direction.

The chloritic schist member of the Salkhala constitutes the major part of the abutment east of Gate Shaft 1. It also forms the downstream parts of the tunnels, and the foundations of the stilling basins and the powerhouse. In the latter areas the chloritic schist has been juxtaposed by faulting and folding with carbonaceous schist, limestone and gypsum belonging normally to the calcareous member. The more competent parts of the folded and faulted areas contain a series of open joints with a general orientation of N 20-40^{O}E. These open joints have accepted large volumes of grout; they possibly form a preferred seepage path, designated as Waterway 1, through the chloritic schist member.

The lower and upper parts of the limestone series have been identified as preferred upstream-downstream seepage paths and have been designated respectively as Waterway 2 and Waterway 3. Waterway 2 dips into the abutment, passing between Tunnels 1 & 2 in the gate shaft area. Waterway 3 outcrops in the reservoir at and to the west of the tunnel intake area. It contains a series of Karstic caverns below the reservoir slope west of the intakes. Waterways 2 and 3 carry most of the seepage passing through the abutment and are drained by an extensive multi-level system of adits. Piezometric levels indicate that the two waterways are connected below the diabase intrusion. As the seepage passes through the abutment, it dissolves the gypsum which is disseminated throughout the calcareous member. Fig. 3 shows a geologic cross-section of the right abutment along the grout curtain.

Alluvial Deposits and MED Foundation.

From extensive site investigations made before and after the start of construction, it was determined that the bedrock surface below the Indus flood plain is several hundred feet deep and forms ridges and valleys which run parallel with the river. A Section through the alluvium under the MED is shown on Fig:2. The depth of bedrock varies across the river and exceeds 700 ft (213 m) at the deepest point. The buried ridges occasionally rise to the surface, forming bedrock islands in the flood plain. The irregular rock bottom most likely was formed by erosion, but the steep, nearly vertical sides of rock ridges in the foundation area suggest that fault movements have contributed to the configuration of the present bedrock surface.

The depth of alluvium is greatest in the dam foundation area and becomes shallower upstream; the width of alluvial foundation is narrowest at the dam and widens in the upstream direction. The rock of the right bank forms a bench, approximately 2000 ft (610 m) wide, which is covered with shallow alluvial deposits. On the river side of the bench the rock drops off sharply and forms a nearly vertical escarpment, the depth of which varies between 700 ft(213 m) at the main dam site and 400 ft (120 m) near the upstream edge of the impervious blanket. This escarpment represents a portion of the Darband Fault, a major strike-slip feature trending parallel to near the right bank, extends approximately 7500 ft (2300 m) upstream of the main embankment base line, while the left side channel reaches about 4000 ft (1200 m). The average depth of open work zones is less in the vicinity of Cofferdam C than at the dam, being about 100 ft (30 m). Openwork zones were actually exposed near the diversion channel entrance at 20 to 30 ft (6 to 9 m) depths.

Left Bank Geology

The left bank (icluding the MED left abutment, and the Service Spillway, Tunnel 5, Auxiliary Dam2 (AD2), Auxiliary Spillway and Auxiliary Dam1 (AD1) foundations) is formed by the Kingriali Formation. The Kingriali Formation consists of dolomitic limestone with phyllite, quartzite and sills of basic igneous rock. Thick beds of quartzite and phyllite, the latter with minor intercalations of limestone, outcrop at the base of the MED left abutment and in the approach channel of the Service Spillway. The formation continues eastward with massive dolomitic limestone and phyllite, visible at the Service Spillway upper chute areas and the upstream parts of Tunnel 5 and AD-2.

Continuing eastward, the left abutment of AD-1 consists of a competent and massive unit of small and medium gently folded quartzite beds. The contact between the dolomitic and quartzite units is marked by a brecciated shear zone with weathered phyllite oriented parallel to the bedding planes. The Service Spillway is benefitted by a favourably oriented geologic structure which prevents excessive underseepage. The vertically dipping phyllite beds in the lower part of the Kingriali Formation cut across the approach channel and act as a natural curtain.

MAIN EMBANKMENT DAM

Foundation Condition

Major part of the dam approximately 6000 ft (1830 m) from the left bank is founded on a deep pervious alluvium and the remainder of the dam, adjacent to the right bank about 2000 ft(610 m) is founded on a bedrock bench. Investigations of the alluvial soils indicated that they were composed predominently of boulder-gravels choked with fine to medium sand. A part of the foundation area of the dam and blanket where islands existed was covered with upto 15 ft(5 m) of loose, fine uniform sand, silty sand, and silt. Borings showed occasional sand and silt strata interspersed throughout the deeper alluvium. Downhole permeability tests and mud loss tests performed during drilling indicated the existence of areas of high permeability, attributable to open work zones in the alluvium. Laboratory studies showed that the gradation of the alluvial materials were conducive to the migration of sand within the boulder-gravel skeleton.

Impervious Core.

Typical cross sections of the dam on alluvial foundation and rock foundation are shown on Figures 4-A and 4-B respectively. The core (zone B1) was specified to be a well-graded mixture of angular gravel-sand and silt, and was designed to be self-healing for settlement cracks should they occur. The core is sloping towards upstream thereby reducing the quantities of sound

materials required for the upstream shell and minimizing the impervious material necessary for the extension connecting the core to the blanket (zone B2). To take advantage of the high shear strength of the free-draining shell material (zone A2), the slope of the core was steepened above El.1440 ft(439 m) from 1V:1.8H to 1.5V:1H. This caused the crest and the entire rockfill cap to be shifted upstream, resulting in substantial saving of material along the entire downstream portion of the embankment.

The sloping configuration of the core enhanced its stability under static and dynamic loading conditions. The comparatively shallow depth of the rock fill shell above the sloping core reduced the load on the core to a lower stress range, within which the developed strength envelope showed a maximum gain in strength with increasing normal load. The smaller confining pressures also reduced the possibility of a loss of strength from the development of pore pressures caused by sudden compression or by material degradation during an earthquake. The relative thinness of the core permits the rapid disssipation of any such pore pressures, which would otherwise diminish the effective fill strength. The hydraulic gradient across the core was limited to no more than 4.0 at any point.

SEEPAGE CONTROL

Control of seepage water from the reservoir through and under the embankment dam was necessary for three reasons:

(i) To conserve water for power generation during periods of low river flow;

(ii) To preserve the stability of the dam by preventing saturation and toe erosion of the downstream slope;

(iii) To prevent migration of material from the embankments foundations, and abutments at points where seepage flows emerge.

Seepage Through the Dam

Seepage through the dam structure is limited by the impervious core, which was designed to extend from a level 10 ft(3.1 m) above the maximum design reservoir pool down to either a positive contact with bedrock or with the upstream impervious blanket. The thickness of sloping core varied from 15 ft (4.6 m) at the top, to 125 ft (38 m) at the base, which was wider than the generally accepted minimum criterion for core thickness of one-fourth of the maximum head. The core section was thickened at the base where it connected to the upstream blanket because the connection point would be most susceptible to cracking from differential settlements and lateral

deformations produced by variations in loads arising from fluctuating reservoir levels.

The core material was surrounded upstream and downstream by the free-draining zones A3 and T respectively. Seepage downstream under normal reservoir operating conditions, with an effectively stationary pool level, would drain through the T-zone composed of a blend of well-graded boulder-gravel and sand, See Fig:4. The quantity of seepage flow expected to occur through the impervious core was estimated from a flow net, in which the horizontal and vertical permeabilities of the compacted core were assumed to be equal and 10^{-5} cm/sec. The maximum seepage flow expected through the impervious core, under full reservoir head, was estimated to be 0.04 cu ft/min per foot of dam (3.7 Litres per minute per meter of dam), or 5.3 cfs (150 L/sec) for the entire length of the dam.

Seepage Through the Alluvial Foundation

For control of underseepage beneath the Main Dam, a long upstream impervious blanket was chosen in preference to a deep cement and bentonite grout curtain for several reasons: the cost of implementing an effective grout curtain would be greater than constructing the necessary length of upstream blanket; inspection of the blanket would be possible during construction to ensure completeness and compliance with the specifications, whereas a grout curtain might have undetected gaps; the blanket would become more effective over time as sediments accumulated and thickened on the blanket, whereas a grout curtain would tend to deteriorate by erosion under high seepage gradients; and finally a blanket would be less affected by earthquakes than would a grout curtain, an important consideration at the seismically active Tarbela site.

The effectiveness of the long upstream impervious blanket design in controlling underseepage relied upon increasing the length of the underseepage path to an extent that the seepage gradients would become very low. Lowering the seepage gradients would reduce the flow velocity and decrease the quantity of underseepage in direct proportion. The importance of forcing the under-seepage to pass beneath the full length of the blanket required that an efficient seal be achieved between the blanket and the bedrock along the valley walls.

Blanket

Details of the design of the impervious blanket are as follows. The length of the blanket 7,500 ft (2286 m) was set to give an overall seepage gradient in the foundation not steeper than 1/15. The length of seepage path considered in this computation was the distance from the upstream end of the blanket to the line of drainage wells; the head was the difference in elevation

between normal maximum reservoir level, El.1550 ft, and minimum tailwater, El.1100 ft. At the sides of the valley, the blanket overlapped the rock abutments by amounts sufficient to make the creep paths at least equal to the differential head acting along the blanket contact. Where space for overlapping was restricted, a grout curtain was installed to add length to the creep path.

Where the blanket crossed a river channel, the banks of the channel were shaped as necessary to achieve a slope flatter than 1 vertical on 4 horizontal. Surface deposits of fine sand were removed to expose the typical cobble gravel. The entire foundation was inspected to ensure that all alluvium was fully choked before impervious blanket material was placed upon it. Where the blanket was underneath the upstream shell of the embankment, the surface of alluvium was compacted with 4 passes of 10-ton vibratory roller. For a 1200 ft (365 m) length of the deepeset part of the main river channel, complete dewatering of the bottom of the channel was not feasible and well graded fill was placed in a maximum depth of water of 15 ft (4.6 m). After the surface of this fill was above water level, it was compacted with 4 passes of the 10-ton vibratory roller before placement of the impervious blanket.Fig-5 shows a typical cross-section of the blanket.

Underseepage: The quantity of underseepage expected beneath the embankment dam and blanket was estimated by the use of a flow net in which the simplified four-layer foundation profile was reduced to an idealized homogeneous deposit 300 ft (91 m) thick and composed of three materials. The ratio of horizontal to vertical permeability was assumed to be 10 for each zone. The estimated underseepage flow was 145 cfs (4100 L/sec) beneath the MED.

A graph of total measured seepage vs. reservoir level is shown on Figure 6. During the first reservoir filling the maximum total seepage which occured before collapse of Tunnel-2 in 1974 was 225 ft^3/sec (6.4 m^3/s) at reservoir level 1460. The wells contributed 149 ft^3/sec (4.2 m^3/s) of the above total seepage flow. The wells on the right side, where the main open work is located showed the greatest flow.

At full reservoir level the underseepage measured for the years 1977 to 1987 is shown on Fig. 7. The figure shows that underseepage has generally reduced from 177 cfs in 1977 to about 6 cusecs in 1987.

Condition of Upstream Impervious Blanket after Emptying Reservoir: As the reservoir was emptied in September 1974, the blanket was exposed after dewatering. Inspection of the blanket revealed a total of 362 sinkholes plus about 140 cracks as well as several compression ridges. In many instances, the sinkholes were located along cracks. Most of the sinkholes had diameters in the range of 1 to 15 ft (0.3 to 4.5 m) and depths in the range of 4 to 6 ft. (1.2 to 1.8 m). The largest had a diameter of 40 ft. (12.2 m) and the deepest a depth of 13 ft. (4.0 m). In no instance did the open sinkhole extend into the alluvium. Generally, serveral inches of openwork type alluvium was found immediately underneath the blanket.

Hypothesis of Sinkhole Development: The hypothesis of sinkhole development is as follows: Cracks open in the tension zones at the crests of humps or ridges created by the unavoidably uneven settlement which the blanket experienced under reservoir loading. Appreciable seepage occurs through these cracks, and at places where the sand choking the cobble gravel may not be dense, the seeping water causes downward migration of the sand in the voids,to form a more dense condition in the lower part of the cobble gravel.

With the development of a thin layer of openwork immediately underneath the impervious blanket, the finer fraction at the bottom of the blanket and along the sides of the lower part of the crack falls into voids in the openwork and is carried along the openwork by the water flowing into the openwork from the crack. It was expected that the blanket would be perfectly self-filtering, but experience indicated that this has not been the case.

Repair of Blanket: Repair of the sinkholes consisted of filling them with filter material, then constructing a mound of additional blanket material over them. A typical treatment is shown in Figure-8. The blanket thickness varied from 42 ft (12.8 m) at the toe of the Main Embankment Dam to 5 ft (1.5 m) at Cofferdam C. Where the blanket was less than 15 ft (4.6 m) thick, downstream of Cofferdam C, its thickness was increased to 15 ft. Most of the material for the mounds and thickening came from the regular borrow areas for blanket material.

Installation of Additional Drainage Wells: Certain wells flowed more than 1 ft^3/sec (28 L/s) of water when the reservoir reached El.1460 (445 m). To discharge this amount of water, several feet of head is required just to develop the necessary velocity of flow in the well. For the well drainage to occur freely, with minimum head loss, additional wells were installed along the existing line of wells and an additional line of wells was installed downstream of the existing line, Fig.9 shows the location of the wells.

Development of Additional Sinkholes: After refilling the reservoir, the development of additional sinkholes were monitored by the side scan sonar and ORE profiling. The survey indicated 350 additional Sinkholes developed in the main blanket during 1975. Treatment was carried out by dumping the material from bottom dump barges. In subsequent years the development of sinkholes reduced considerably and no sinkhole was developed in 1987 due to improvement caused by natural sedimentation. Fig. 10 shows histogram of sinkholes with years.

AUXILIARY DAM 1

Foundation Treatment

With bedrock exposed on both abutments and covered generally by 100 to 200 ft

(30-60 m) of alluvium across the valley floor, a positive cutoff to bedrock was considered to be the most feasible solution. Accordingly, a trench was excavated through the alluvium for the cutoff using the standard earth-moving equipment. Dewatering in advance of the trench excavation was implemented without difficulty.

The materials from the cutoff trench were used in the Auxiliary Dam 1 embankment section. To enable construction of the dam to proceed during construction of the cutoff, the cutoff trench was located just upstream of the embankment section. The cutoff was connected to the core of the dam with a short section of impervious blanket mostly underneath the upstream shell of the dam. The side slopes of the cutoff trench were 1V:1.4H and the base width was equal to the differential head across the cutoff. Typical X-section of the dam is shown on Fig.11.

The impervious material forming the cutoff is the same as that of the core of the dam; placed along the bottom and downstream face of the trench, it was 40 to 50 ft. (12 to 15 m) thick on the downstream face and 20 to 50 ft (6 to 15 m) thick on the bottom. Spoil material was placed on top of the impervious material to a slope of 1V:4H. Seepage underneath and through the cutoff was collected by the 100 to 200 ft(30 to 61 m) of alluvium downstream of the cutoff. Because of the relatively high permeability of the angular boulder-gravel strata in this alluvium and the horizontal stratification, seepage collected did not emerge at ground surface until reaching the plunge pool of the Auxiliary Spillway, about 700 ft (213 m) away from the downstream toe of the dam at its closest point.

The blanket and cutoff were designed to be tied continuously to bedrock along the valley walls and at the bottom of the cutoff trench excavation. Where rock and overburden conditions permitted, the contact between the blanket and bedrock was equal in width to the differential head across the contact. For Auxiliary Dam 1 the maximum head loss was 400 ft. (122 m), which is the difference between the maximum pool and water level in the Auxiliary Spillway plunge pool. The width of the cutoff was reduced to a minimum of 0.8H at the abutments where curtain grouting was carried out.

The contact between the core and the non-overflow monoliths of the Auxiliary Spillway at the right bank had a minimum length equal to twice the head acting at each point. The contact was secured by enveloping the concrete monolith in the core zone, providing contact on both faces of the monolith; the core section's near-vertical position facilitated this contact.

Drainage

The underseepage cutoff intercepted the major subsurface flows through the alluvial foundations. Grout curtains were established in the bedrock behind the blanket-bedrock

contacts at both abutments to seal the bedrock near the surface in zones of potential weathering or open joints. A drainage adit in the bedrock of the right abutment downstream of the core-bedrock contact collected any seepage passing the grout curtain and discharged by gravity into the Dal Darra downstream of the dam. Groundwater levels in the alluvium downstream of the cutoff treatment, reflecting seepage through the core, blanket and bedrock, were controlled by the invert elevation of the spillway outlet channel, El.1160 (354 m).

Embankment Dam

An embankment type dam was selected because of the deep alluvium and the presence of a wide fault zone in the underlying bedrock. The locating of the cutoff trench through the alluvium upstream of the dam required an impervious blanket underneath the upstream shell to connect the core and cutoff. As the construction materials available for Auxiliary Dam 1 were substantially the same as those for the Main Embankment Dam, the same cross section was used for both.

A three-line grout curtain in the limestone on the right side of the valley was installed. Some of the holes of the middle line in the limestone were drilled as deep as El.1000 ft (305 m) or to a maximum depth of 160 ft(49 m), to investigate any open rock adjoining the Dal Darra fault.

AUXILIARY DAM 2

Auxiliary Dam 2 closes a saddle to the right of and adjacent to the Auxiliary Spillway. The dam is located directly on bedrock with its left abutment enveloping the non-overflow monolith of the Auxiliary Spillway. It has a maximum height of 220 ft (67 m) and a crest length of about 860 ft (262 m).

Sub-surface Conditions

The bedrock at the site comprises fine-grained, brecciated dolomitic limestones and interbedded phyllites, which are most talcosic, poorly cemented and highly fractured. Bedding planes strike approximately parallel to the dam axis and dip downstream at an angle of about 55^{o}. A minor fault cuts through the foundation perpendicular to the axis of the dam.

Embankment.

The cross section of the Auxiliary Dam 2, shown on Figure 12 has a steeply inclined impervious core seated on bedrock and surrounded by free-draining rockfill shells upstream and downstream. A transition zone between the core and shell prevents migration of fine material.

Foundation Treatment

Underseepage at Auxiliary Dam 2 is controlled by an impervious contact between the core and the bedrock, and the sealing of joints and fractured zones near the bedrock surface. The required length of the impervious core-bedrock contact was equal to the total difference in head acting across the contact. To achieve this contact length, a short impervious blanket had to be extended downstream from the core, as in the case of the part of the MED founded on the right side rock bench. A continuous grout curtain, extending along the core contact consists of 3 lines of holes. The outer two lines are 50 ft (15 m) deep; the centre line generally extends to a depth of 100 ft.(30.5 m) and connects with the grout curtain of the Auxiliary Spillway. The impervious core envelops the non-overflow monoliths of the Auxiliary Spillway at the left abutment, with a minimum contact length equal to twice the total difference in head between the concrete and the impervious core.

RIM TREATMENT

Rim treatment is work undertaken to seal and preserve the rim of a reservoir against excessive and uncontrolled seepage erosion or other unstable conditions. Rim treatments constructed at Tarbela have included the installation of extensive grout and drainage curtains, upstream blankets of silt over alluvial foundations and an impervious covering of silt and gunite for the Right Bank Intake Nullah.

Work Prior to Impoundment

Left Bank: The left bank rim treatment began with a grout curtain in the left abutment of the MED, installed from the surface and from grouting adits LGA-1, LGA-2 and LGA-3 to control seepage behind the wraparound of the MED impervious core as shown in Fig. 13 A & 13 B. The single-line curtain extended from the ground surface to El.1040 ft (317 m) and inclined slightly upstream to meet a second grout curtain installed under the axis of the permanent access road. A drainage curtain was installed 200 ft. (61 m) behind this grout curtain in adits LDA-1 and LDA-2 with single, slotted PVC pipes spaced at 20 ft. (6.1 m) centres. This curtain was designed (1) to

collect any water escaping through the LGA-1 and LGA-2 grout curtains, and (2) to control uplift pressures.

In 1971 the MED blanket contact grout curtain was extended some 1,500 ft. (457 m) upstream from the portal of adit LGA-2 with 100 ft (30.5 m) grout holes spaced at 10-ft. (3.05 m) centres drilled from the surface. This curtain was installed to protect the blanket contact by increasing the seepage path through alluvium immediately under the blanket's edge. The second grout curtain, installed under the axis of the permanent access road, linked the left abutment curtain with grout curtains for the Service Spillway headworks. This second curtain was installed from the ground surface to El.1340 ft (408.4 m).

Service Spillway rim treatment was conventional with a single-line grout curtain and a downstream drainage curtain. As the rock was found more weathered than originally anticipated, an additional grout curtain was therefore installed, approximately 100 ft (30.5 m) upstream of the original curtain in the approach channel, with both curtains tying into the permanent access road grout curtain. There was no continuation of the left bank rim treatment to the left (east) of the Service Spillway.

Left bank rim treatment at Auxiliary Dam 2 (AD-2) with a grout curtain installed between 40 to 100 ft (12.2 to 30.5 m) into the AD-2 foundation rock. To the west this curtain began high in the abutment; to the east it tied into grout curtains for the headworks of the Auxiliary Spillway. During excavation of the AD-2 foundation, deposits of silt and a silty sand in two rock troughs oriented in an upstream-downstream direction were uncovered. As they were thought to be associated with possible infilled faults in the foundation, an adit 200 ft (60 m) below foundation level was excavated to investigate the extent and nature of these deposits. The rock encountedred proved to be sound and relatively impermeable. Drains were consequently not installed prior to the first reservoir impoundment, but the adit was concrete-lined.

Like the Service Spillway, the tender design of the Auxiliary Spillway rim treatment was conventional with a Single line grout curtain and extensive pressure relief from drainage galleries. A review of subsurface conditions in 1971 after the foundation had been excavated, revealed the rock to be considerably more weathered than originally anticipated. The design was accordingly revised to provide an upstream impervious blanket and an additional triple-line grout curtain at the upstream edge of the blanket in addition to the previously specified gallery grout and drainage curtains.

The Auxiliary Spillway protection slab extended 225 ft.(69.6 m) upstream of the ogee weir and comprised a 5 ft. (1.5 m) thick impervious blanket of B-1 material protected against erosion by a 1-ft (0.3 m) thick articulated concrete slab. The thickness of the blanket was increased near the concrete structure by filling in the foundation excavation to allow the construction of a

reliable tie. The blanket was also extended laterally to tie into the cores of the two auxiliary dams located on either side of the spillway ogee weir along with the single-line curtain from the spillway gallery.

To ensure the stability of the Auxiliary Spillway ogee and chute structures a drainage curtain was installed around them from the surface and from adits wrapping around the entire chute structure. Due to the poorer quality of rock at the Auxiliary Spillway site this drainage curtain was more extensive than the Service Spillway Curtain. The Auxiliary Spillway drainage curtain at the headworks extended from a gallery in the ogee weir down to El.1130 ft (344 m) through drainage adits at El.1330 ft (405 m) and El.1227 ft (374 m), see Fig. 14.

Two grout curtains were constructed from the abutments of AD-1 extending under the upstream edge of the core and blanket contacts with the right abutment curtain tying into the Auxiliary Spillway headwork's triple-line grout curtain. Grout hole depths varied from 50 ft (15.2 m) at the top of the abutment to 100 ft. (30.5 m) at the bottom with holes spaced at 10 ft (3.05 m) centres. The left abutment AD-1 blanket contact curtain was 915 ft (279 m) long and at the right abutment 2,750 ft (838 m) long beginning at the Auxiliary Spillway. These curtains at the edge of blanket and core contacts served to increase the length of seepage through alluvium and rock immediately under the blanket and core contact where gradients were expected to be highest.

Right Bank. Four power and irrigation tunnels and their associated control structures pass through the rock of the right abutment. A main grout curtain and several drainage curtains were initially constructed prior to completion of the MED to both control seepage in the area of the tunnels and to ensure stability of high cut slopes at the tunnel outlets. The original grout curtain for the right bank made use of consolidation grouting for the tunnel gate shafts to serve as an abutment curtain. This abutment curtain was extended with grouting from adits RGA-5 and RGA-6 to link up with a grout curtain under the MED core/rock contact extending upstream under the MED blanket/alluvium contact. Thus a continuous grout curtain was formed from the gate shafts into the reservoir. The blanket and core contact grout curtain was 5,700 ft (1,737 m) long and ran behind right limb of Cofferdam C and across the diversion channel. This curtain was constructed with 100 ft (30.5 m) deep grout holes spaced 10 ft (3.05 m) apart under the MED blanket/alluvium and MED core/rock contacts to increase the seepage path at these contacts.

During tunnel construction, consolidation grout holes were drilled from the 4 gate shafts and gate chambers from El.1555 ft. (474 m) to El.1050 ft (320 m). These holes extended 20 to 40 ft. (6.1 - 12.2 m) from the outer face of the shafts in a radial pattern. At the centre line of the gate shafts and chambers a line of intersecting holes were drilled from adjacent shafts to form a continuous curtain. Grouting to the west of the gate shafts was done from the adits in a single line

using spilit-spacing with primary holes located on 10 ft (3.05 m) centres and secondary and tertiary holes split-spaced as required to meet closure criteria.

In December, 1973, it was decided to install a second grout curtain behind the gate shafts from Adits RGA-3 and 4. This curtain was completed prior to the first reservoir filling in 1974. It tied into the curtain established from RGA-6 and RGA-5 on the left, whence it continued into the right abutment along RGA-3 at El.1400 ft (427 m), and along RGA-4 at El.1250 ft (381 m). The vertical limits of the curtain matched the gate shafts curtain extending from El.1550 ft (472 m) to El.1050 ft (320 m) but formed only a partial barrier to seepage flow, as it could not be tied to any impervious beds at the bottom or on the right.

The original adit drainage system built behind the grout curtain was designed primarily to prevent water from accumulating in the rock mass behind the outlet slope. The system was composed of 4 drainage curtains forming a box pattern around the lower portion of the tunnels and a curtain farther downstream to the right of Tunnel 4 as shown in Fig: 15. Adit 8-3R, penetrating 240 ft (73.1m) into the hillside from adit 8-12L, was intended as an instrumentation adit, but was also used for drainage to the right of Tunnel 4.

Following the Stage III diversion in 1973, when the river was diverted through Tunnels 1 and 2 and the reservoir level rose, a sudden increase in seepage flow in the low-level drainage adits of the right abutment was observed; these flows contained a large quantity of dissolved solids. The right abutment grout curtain was consequently extended into the abutment by about 900 ft (270 m) from adits RGA-7 and 8, not to create a tightly grouted zone, but rather to fill large cavities in the rock with grout through a program of both exploratory and curtain grouting. This treatment was decided upon because there was no impervious formation within practical limits to which the grout curtain could be tied. With the tight cut-off impracticable, primary reliance for seepage control in the abutment was delegated to the network of drainage curtains. Additionally, the right abutment drainage system was modified before the 1974 reservoir filling. Adit 8-3R, the low-level adit about 150 ft (46 m) to the right of Tunnel 4, was backfilled with concrete after all the drain holes were grouted full, and a new adit, RDA-21 with widely spaced upward and downward drains, was constructed directly above 8-3R, at about El.1250 ft (380 m).

To protect the outlet cut from excessive water pressures, two new drainage adits, RDA-20 and RDA-22, were constructed about 300 ft (91 m) upstream and parallel to its north face. Adit RDA-20 located to the right of Tunnel 4 at El.1116 ft (340 m), has closely spaced upward and downward drain holes; RDA-22 located over the tunnels at about El.1200 ft (365 m) has similar holes moderately spaced. To protect the outlet cut further, horizontal drain holes were drilled from the permanent access road at El.1160 ft (354 m) into the abutment rock to a depth of 150 ft (45 m).

Modifications After Filling

During the 1974 reservoir filling, large flows containing significant quantities of dissolved solids occurred in the right abutment drainage system, particularly in the deepest part to the right of the tunnels. Consequently, in early 1975 an additional grout adit and curtain, and another drainage adit and curtain, were constructed in that area.

The new grout curtain was installed from the new adit RGA-10, downstream of the main grout curtain, to intercept waterways to the right of the tunnels and to reduce the flow at high reservoir levels to the RDA-20 drainage curtain. This curtain extended down to El.1010 ft (308 m) and tied into impervious beds of chloritic schist on the left of the tunnels but not on the right.

The new drainage curtain was constructed from the new adit RDA-23 to reduce gradients into the upstream most portion of this adit, and to intercept about half of the seepage water passing to the right of the tunnels, thereby reducing the load on the lower level system constructed from RDA-20. Fig. 15 shows the grout and drainage adits in the right bank.

Left Bank During the 1974 initial impoundment, performance of the left abutment grout curtain was not considered satisfactory. Seepage in the drainage adits was excessive and reached approximately 10 cfs (0.28 cms). Small amounts of fines were observed, issuing through several holes in the abutment drainage curtain. During the 1974-75 low reservoir season, measures were taken to correct this situation. The left abutment drainage adits LDA-1 and LDA-2 constituted a system that was too far upstream and provided only a short seepage path from the reservoir through an ineffective grout curtain. It was therefore abandoned and converted to a second grout curtain by grouting the drain holes and then split-spacing additional grout holes between them. The secondary (downstream) drainage adits of the original drainage system,LEA-1 and LEA-2 then became the primary drainage adits; the new secondary drainage adit, LEA-3 without drain holes, was constructed downstream of the "new" primary adits, at about El.1200 ft (365 m) for possible future use.

After completion of grouting from LDA-1 and LDA-2, these former drainage adits were plugged with concrete as their original linings were not adequate to withstand the hydrostatic pressures expected around grouting adits. Similarly, a second internal lining was added to the portions of LAA-1 and LAA-2 between the LGA and LEA galleries. This new lining addition reduced the diameter of those adits to 42 in (1.1 m) (Figs: 13 A and 13 B).

Subsequent Modifications

1975-1979 Performance from 1975 to 1977 was generally considered satisfactory for the entire

Project. Although left abutment seepage from the adits was some five times higher than anticipated, it was well within the capacity of the drainage system. During the 1978 impounding, high increases in seepage were recorded, predominantly from three downdrains and two updrains in Adit LEA-2. At that time the increase in flow as associated with tension cracks which had developed in the upper part of the blanket wraparound.

To reduce left abutment seepage, a number of modifications were made during the 1978/79 construction season, including the grouting of high discharge updrains in LEA-2 and their replacement with sand-filtered drains; grouting of the impervious wrap around to seal cracks; and the installation of additional piezo-meters. After the 1979 impounding, the extra drainage gallery LEA-3 was equipped with 52 new, sand-filtered drains, with 5 additional such drains installed in the most critical areas of Adit LEA-2.

The performance of the Auxiliary Spillway rim treatment was entirely satisfactory from 1975 to 1978. However, during the 1978/79 high reservoir season, discharge of significant amounts of fines from ten drains in the drainage curtain under the headworks structure required that they be plugged. Further exploratory work revealed the existence of sinkholes in highly altered rock immediately upstream of the impervious blanket, and the settlement of the upstream apron slab caused by voids which had developed underneath it. During the 1979/80 low reservoir season, the plugged drains were grouted, replacement drains with sand filters were installed along side them, and remedial grouting done under the apron slab. The sinkholes upstream of the apron were grouted and check grouting done in the grout curtain at the upstream end of the apron to fill any gaps which might have developed in the curtain since 1975.

1980-1983

In 1980, piezometric levels in certain areas of the MED left abutment increased; limited amounts of fines were piped through some drains; several drains discharged excessive flows; and cracks, noticed in the wraparound in 1975, could not be eliminated as a contributing cause of the high seepage. In general performance in 1980 indicated additional deterioration in the left abutment. The problem was discussed at the November 1980 Tarbela Geotechnical Meeting, and remedial treatment proposed consisting of exploratory grouting to investigate the need for additional grouting; the replacement of several drains; and silt dumping from the reservoir at the blanket contact. Eight test grout holes drilled between the left abutment parking area and the Service Spillway indicated the presence of the significant discontinuities within the underlying rocks and original grout curtain. Results from these exploratory holes were used to design appropriate grouting techniques and mixes for later use.

A number of the original left abutment drains were also replaced in 1980/81 with

sand-filtered drains on a 2 for 1 basis after grouting the original drains. Grout takes frequently exceeded the theoretical volume of the existing holes by a considerable volume indicating that the grout had penetrated into rock and filled existing openings. The programme of replacing the drains with sand filtered drains continued upto 1987. A number of holes which discharged less flow were reamed and equipped with sand filters. Fig. 16 shows the left abutment replacement drains installed in 1987.

Barge dumping of silt was done at both high and low reservoir levels along the upstream left abutment blanket/alluvium contact during the 1980/81 season. The limited time available permitted the dumping of only ten loads of material at two locations. Although a much larger amount of material would have been required for any significant and permanent effects, nevertheless, a noticeable improvement in MED blanket performance and left abutment piezometer readings was observed during the 1981 impounding. At the Nov. 1981 Tarbela Geotechnical and Spillway Meeting it was decided to pursue a continued program of grouting and drain installation in both MED abutments and AD-2 during the 1982/83 low reservoir season. In preparation, link adits LGA-4 and LGA-5, were constructed in 1982 in the left abutment.

Additional work carried out in 1982 included the sealing of leakage points in adit LGA-1; filtering of weep holes in the lower drainage adits; additional barge dumping of silt; and the installation of additional measuring weirs in the drainage adits. During the 1982/83 construction season, 389 up-and-down holes totalling 61,532 ft (18,755 m) were drilled and grouted, with 125,399 cu ft (3,549 cu m) of grout from adits LGA-1, LGA-4 and LGA-5 creating 2 new grout curtains, LGA-4 and LGA-5 and augmenting the original curtain from LGA-1. Seepage collected by the left abutment drainage adits decreased from 7.80 cfs in 1982 (0.22 cms) to 4.31 cfs (0.12 cms) at full reservoir upto 1987 following the implementation of works as mentioned above (Fig. 17).

Auxiliary Spillway

During the 1979 impounding 54 additional updrains in the Auxiliary Spillway drainage curtain discharged fines. These were replaced with sand-filtered drains in 1980. Following the 1979 impoundment, 17 additional sinkholes were discovered upstream of the Auxiliary Spillway approach slab, alongwith larger voids under the approach slab itself. In 1980, the sinkholes were filled and grouted, using the previously employed technique of capping and grouting. Voids under the approach slab were also filled with grout and the slab levelled by mud jacking. To further protect and seal the approach channel, the approach slab was extended with concrete over the entire triangular area in front of the original slab and down the upstream slope of the approach channel to El.1430 ft (436 m) with pneumatically placed concrete.

In addition, 10 check grout holes were drilled into the middle line of the Auxiliary Spillway triple-line grout curtain in 1980. Grout takes were generally 30 percent higher than previously recorded for correspondingly located holes in the original curtain, confirming a loss of integrity in the original curtain. Four of the holes were drilled to a depth of 260 ft (79 m) and high-pressure, tube-a-manchette grouting performed successfully to test the possible applicability for future use.

Continued concern over the stability of the Auxiliary Spillway Foundation prompted a decision in 1980 to undertake an extensive program of rim treatment improvements. During the 1980/81 construction season all remaining original drains were replaced on a 2 for 1 basis with redesigned sand-filtered drains and additional piezometers were installed. The articulated joint between the upstream end of the approach slab and headworks was rebuilt alongwith one of the approach slab and several water stops. A grout curtain was installed to a maximum depth of 265 ft (81 m) just downstream of the original triple-line curtain through the approach slab. This new curtain was completed with numerous tertiary grout holes spaced at 5 ft (1.5 m) centres and extended laterally to connect with the cores, blankets and grout curtains in the Auxiliary Dams on either side. Due to implementation of above programme the seepage collected from the drainage adit decreased from 5.77 cusecs in 1980 to 2.25 cusecs in 1987 (Fig. 18).

Auxiliary Dam 2

In 1982,the original unfilled weepholes and short updrains in the drainage adit below AD-2 were provided with filters. This caused a noticeable rise in piezometric level around the adit and increased the flow of the short updrains. At the November 1982, Geotechnical Meeting it

was decided to correct this condition by including the installation of a drainage curtain from this adit as a part of the extensive 1982/83 drilling and grouting programme mentioned above under left abutment grouting.

In 1983, a drainage curtain consisting of 100 ft (30.5 m) downdrains and 60 to 100 ft (18.3 m to 30.5 m) updrains, restricted to provide a minimum rock cover of 40 ft (12.2 m) was established in the drainage adits below AD-2. All holes were drilled at 20 ft (6.1m) centres and fitted with 3 in (76 mm) diameter sand-filtered drains.

Right Bank

The Intake Nullah to the right of the right bank tunnel intakes was suspected as being a major inlet for seepage into the MED right abutment. To test this theory several loads of windblown sand mixed with silt were barge-dumped in the Intake Nullah at high reservoir level in September 1980. This was followed by decreases in flows from the RAA-2, RDA-22 and 8-14L drainage systems, and in the levels of six piezometers in the vicinity of the Gandaf gate shaft. These were in excess of normal drops associated with the drawdown of the reservoir and occurred during the dumping trials. Blanketing of the Intake Nullah as a means of reducing seepage was consequently proposed in 1980 based on the pattern of piezometric levels in the abutment at high reservoir level and the results of the barge dumping trials.

A programe to seal the Intake Nullah with gunite and silt fill was recommended at the November, 1981 Tarbela Geotechnical Meeting and implemented during the 1981/82 construction season.

The general arrangement of treatment carried out to seal permeable surface areas in the Intake Nullah is shown on Fig. 19. Slopes on both sides of the nullah were cleaned and covered with fibre-reinforced gunite. A one-way drainage system consisting of slotted PVC pipe collector drains installed at 20 ft (6.1 m) centres was placed underneath the gunite. Large voids were filled with dental concrete, and zones of rock too poor in quality to hold a gunite layer, were sealed with formed concrete. Along the thalweg of the nullah, silt fill was placed in three areas; behind debris Dam B (Area I), below debris Dam B (Area II), and behind debris Dam A (Area III). A small concrete dam was constructed in sound basic rock to retain the silt placed in Area II. All areas of silt fill were covered with a layer of slush-grouted riprap to provide a durable surface for runoff.

The Intake Nullah treatment extended from El.1380 ft (421 m) upto El.1555 ft(474), m), with the exception of an area behind the debris Dam B on the north side of the nullah, which was cleaned and treated subsequently above El.1534'ft(767 m). Piezometers were installed at several locations to monitor the effect of the treatment during reservoir rise.

During the 1982/83 construction season an extensive program of grouting was completed in grout Adits RGA-3, 4, 5, 6, 11 and 12. In RGA-11 and RGA-12, 158 downholes totalling 23,683 ft. (7,219 m) were drilled and injected with 22,531 cu ft (638 cu m) of grout to improve the foundations of Tunnels 1 and 2, particularly in karstic and gypsiferous deposits which might have developed solution cavities or voids. A total of 258 upholes and downholes were also drilled for 35,680 ft (10,875 m) from Adits RGA-3, 4, 5, and 6 and injected with 10,251 cu ft (290 cu m) of grout to augment and strengthen existing grout curtains and fill possible solution cavities in the right abutment, particularly in areas known to contain seepage paths.

In addition, a test program of chemical grouting was conducted in 1983 at Adit RGA-4 between Stations 1+30 and 2+10, Two panels with multiple lines of closely spaced grout holes were injected with a variety of chemical and bentonite grouts to penetrate the smaller fissures and voids that the previously employed cement grout had been unable to fill.

Fig.20 shows the seepage from the right abutment adits. Seepage after the treatment of intake nullah has decreased substantially and is now approaching stabilization.

INSTRUMENTATION

For monitoring overall performance a large number and variety of measuring instruments were installed in the embankments, their foundations, and in the MED blanket, see Table-2. Among these was the double fluid settlement device, which was successful in providing continuous settlement profiles. In addition to in-situ instrumentation side-scan sonar and related equipment were also employed to monitor the development of sinkholes in the blanket and on the faces of the dam. A large number of vibrating-wire piezometers, some with leads as long as 11,000 ft (3,350 m) were installed and have proven successful, with only a moderate loss of instruments since their initial installation. About 40% of the MED piezometers were installed in the foundation; the remainder were embedded in the core material at selected elevations. Each of these instrumented sections and levels was provided with a mechanical extensometer oriented normal to the MED base line; it extended from the downstream instrument house to a point near to, but short of, the core. Certain sections of the MED (MX and MD) had extensometer systems running parallel to the base line.

In the winter of 1972-73, after the first set of foundation piezometers proved that large and extensive openwork zones existed in the foundation, 23 additional standpipe piezometers were installed to a depth of about 125 ft (38 m) in the blanket foundation area and areas both upstream of the blanket and downstream of the MED. Twenty-six shallow piezometers, 25 to 30 ft (7.6 to 9.1 m) deep, were installed immediately downstream of Cofferdam C, where openwork zones

were also suspected. Many of the standpipe piezometers used in the previous programme were converted to a remote-reading piezometers. The majority of these were the vibrating-wire type, selected because the reading of this type of instrument is unaffected by the length of the cables.

Temperature measurements were made in 50 piezometers and relief wells, to provide data for thermonic studies. After the filling operations began drainage wells downstream of the MED were also used as piezometers.

The observational method proved to be an effective and economical technique for evaluating and assuring the safety of the Tarbela Embankment, particularly during initial reservoir filling. The method required more instrumentations and observation than had previously been considered customary, but provided an assurance that could not have been attained in any other way. Monitoring of the foundation has shown no clear indication of significant internal sand migration.

PIEZOMETRIC PRESSURES (U/S Blanket)

After the treatment of the blanket during 1974-75 and subsequently by barge dumping at the locations of new sink holes, the blanket generally has continued to become tight with natural sedimentation and time consolidation resulting in progressive reduction in piezometric potentials as well as underseepage.

This improvement has not been shared by left side due to feeding from the blanket/rock contact and there is an indication of deterioration. Efforts to improve the left side area by barge dumping during 1980, 81-82 have not proved so successful. Its effect remains for some time and then diminishes. Reservoir opration also plays an important part in the behaviour of blanket area. During the year when the reservoir is not drawndown to minimum, the sediments from the delta do not travel to the blanket area resulting an increase in pressures and seepage compared to previous year. Increase in piezometric potentials and underseepage was noticed during 1983 when reservoir was drawndown to elevation 1341'ft and thereafter impounding started. Maximum improvement in a single year as compared to all the previous years (reduction of 2-5% in potentials) was obtained in 1985 due to reservoir remaining at low level (1300'ft) for about two months and deposition of about 7 to 8 ft. of sediments over the blanket, (Fig: 21). The seepage collection in the downstream ditches also reduced from 32 cusecs in 1984 to only 8 cusecs in 1985 (Fig-7).

Development of sink holes have almost ceased. Two sinkholes developed during 1986 out of which one was treated by barge dumping whereas the other was kept under observation which has been filled up by natural sedimentation as indicated by 1987 Sonar Survey of the blanket area.

CONCLUSIONS

According to the conclusions of the First Periodic Inspection carried out in 1986, 87, Tarbela Dam Project is being operated and maintained in a staisfactory manner.

There still exists the possibility of formation of new sinkhole or redevelopment of an old sinkhole due to some local adjustment phenomena. This requires constant monitoring and in some cases treatment.

The upstream blanket has been successful in controlling the underseepage and the exit gradients. It gradually improved with the silt deposition. The underseepage reduced from 332 cusecs in 1975 to 5.7 cusecs in 1987 at full reservoir level (1550 ft.).

The relief wells are in good working condition, none was discharging at the time of the inspection. The downstream piezometers, adjacent to the relief wells have shown gradual drop in piezometric levels.

Seepage in both the left and right abutments was controlled by additional grouting and drainage from the galleries.

The observational method proved successful.

REFERENCES

1. Completion Report on Design and Construction-Tarbela Dam Project. Volume: I, II, III, IV, V, 1984 by TAMS.
2. Report on First Periodic Inspection of Tarbela Dam 1986-87 Volume: I & II by TAMS.
3. John Lowe III: Foundation Design- Tarbela Dam Fourth Nabor Carrillo Lecture, 1978.
4. Kalman Szalay and Mario Marino: Instrumentation of Tarbela Dam ASCE International Convention, New York, May 1981.
5. Annual Performance Reports of Tarbela Dam Project from 1974 to 1987.

TABLE 1.

TARBELA DAM - SALIENT FEATURES

RESERVOIR		
Length	60 miles	97 km.
Area	95 Sq. miles	240 Sq.Km
Gross capacity	11.3 MAF	14,000 Mm^3
Live Storage	9.4 MAF	11,600 Mm^3
Dead Storage	1.9 MAF	2,300 Mm^3
Maximum Elevation	1550 ft.	472.4 m
Minimum Elevation	1300 ft.	396.2 m
Minimum Head for Power Generation	435 ft.	133 m
Minimum Head for Power Generation	185 ft.	56 m
MAIN DAM		
Crest Length	9000 ft	2.75 Km
Crest Width	40 ft.	12 m
Base Width	2000 ft.	610 m
Maximum Height	470 ft.	143 m
Volume	139 Mcy	106 Mm^3
Blanket Volume	30 Mcy	23 Mm^3
Length	6640 ft.	2025 m
AUXILIARY DAM-1		
Crest Length	2340 ft.	713 m
Maximum Height	345 ft.	105 m
Volume including blanket	20 Mcy	15 Mn^3
AUXILIARY DAM-2		
Crest Length	960 ft.	293 m
Maximum Height	220 ft.	67 m
Volume	2.4 Mcy	1.8 Mm^3
SERVICE SPILLWAY		
Gates 7 No.	50' x 61'	(15.2 m x 18.6 m)
Design Discharge	615,00 cfs	17,400 cms
Crest Elevation	1492 ft.	454.8 m
Flow width	350 ft.	107 m

Depth of Plunge Pool	150 ft.	46 m
AUXILIARY SPILLWAY		
Gates 9 No.	50' x 61'	(15.2 m x 18.6 m)
Design Discharge	795,000 ft.	22,500 cms
Crest Elevation	1492.0 ft	434 cms
Flow width	450 ft	137 m
TUNNELS 1-4		
Length	2400-2700 ft.	731-823 m
Inside diameters upstream of gate shafts (concrete lined)	45 ft	13.7 m
Inside diameter downstream of gate shafts (Steel lined) Tunnels 1.2 and 3	43.5'	13.3 m
Tunnel 4	36 ft.	11.0 m
TUNNELS-5		
Length	3,675 ft.	1120 m
Inside diameter, upstream of gate shaft (concrete lined)	45 ft.	13.7 m
Inside diameter downstream of gate shaft (steel lined)	36 ft.	11.0 m
IRRIGATION DISCHARGE CAPACITY EL. 1500		
Tunnel 4	92,000 cfs	2605 cms
Tunnel 5	81,000 cfs	2300 cms
IRRIGATION DISCHARGE CAPACITY EL.1300		
Tunnel 4	63,000 cfs	1784 cms
Tunnel 5	44,000 cfs	1250 cms

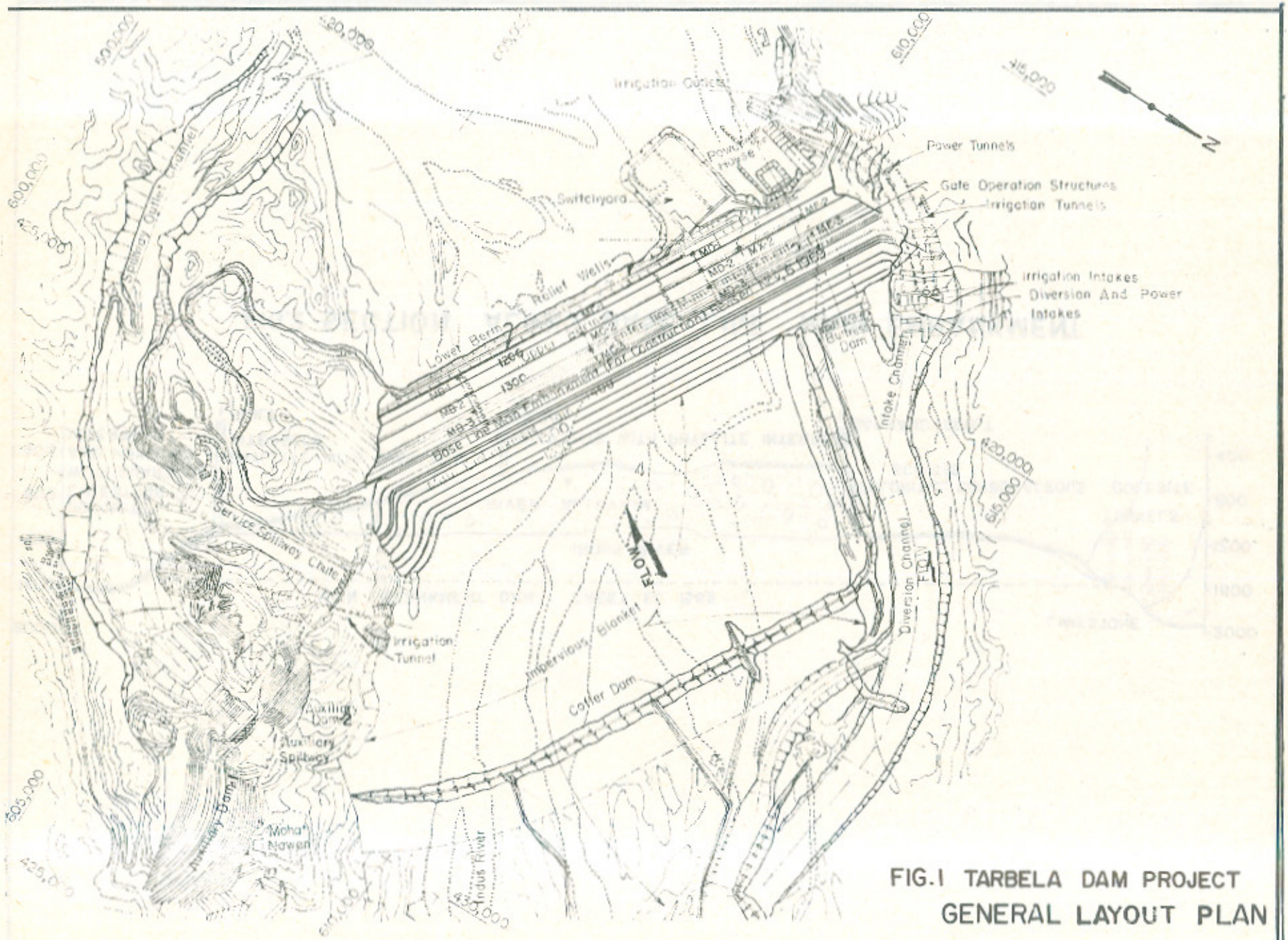


FIG.1 TARBELA DAM PROJECT
GENERAL LAYOUT PLAN

Naeem

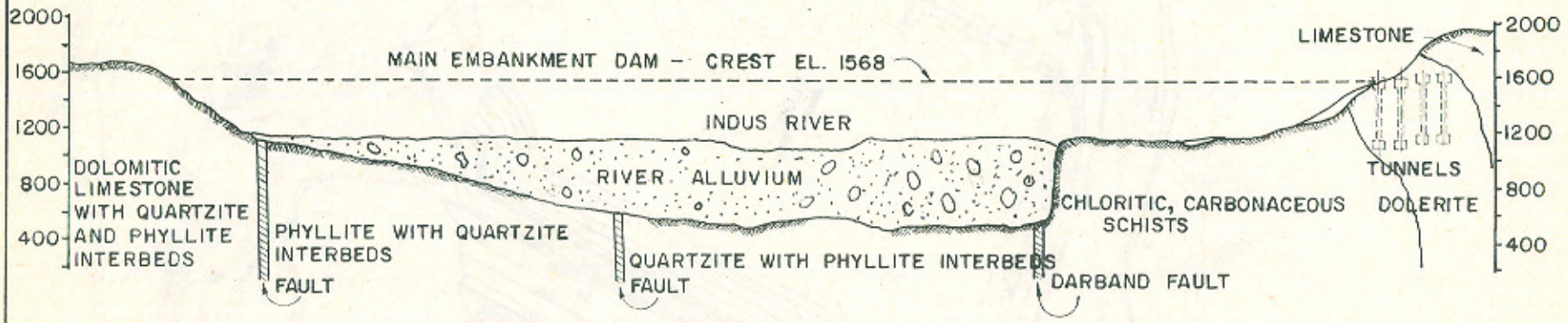


FIG.2 SECTION ALONG BASE LINE MAIN EMBANKMENT

Naeem ...

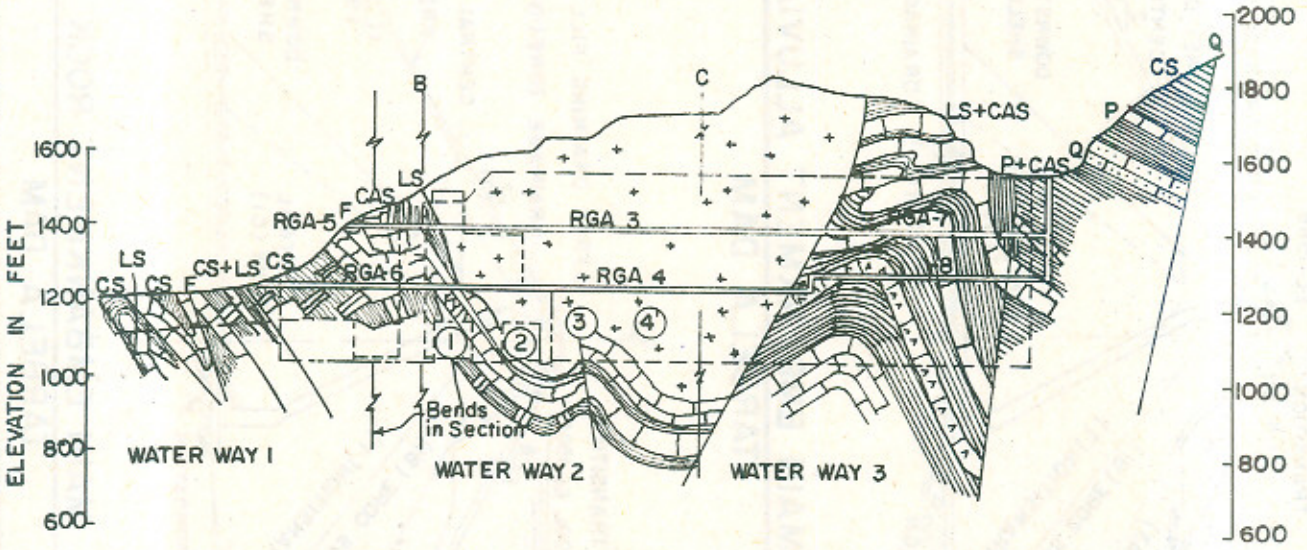


FIG. 3 RIGHT ABUTMENT GEOLOGIC SECTION ALONG THE GROUT CURTAIN

LEGEND

CS — CHLORITIC SCHIST

Q — QUARTZITE

LS — LIMESTONE.

LS+GY — LIMESTONE AND GYPSUM

CAS + P — CARBONACEOUS SCHIST AND BLACK PHYLLITE

B — BASIC IGNEOUS ROCK

F — FAULT

MANSOOR.

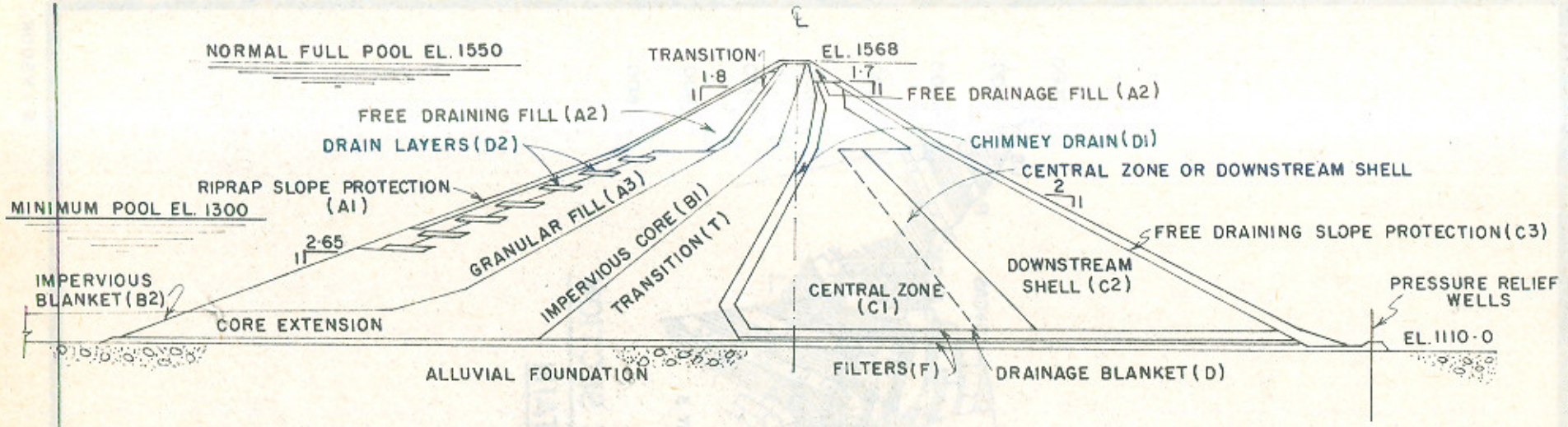


FIG. 4A TYPICAL SECTION MAIN EMBANKMENT ALLUVIAL FOUNDATION
TARBELA DAM

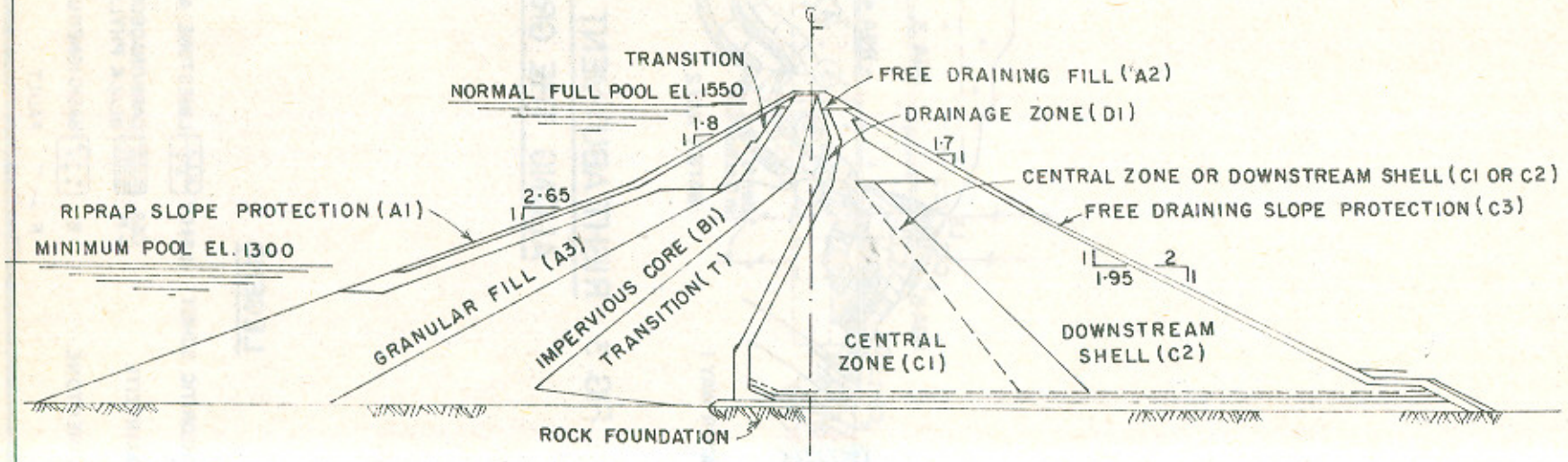


FIG. 4B TYPICAL SECTION MAIN EMBANKMENT ROCK FOUNDATION
TARBELA DAM

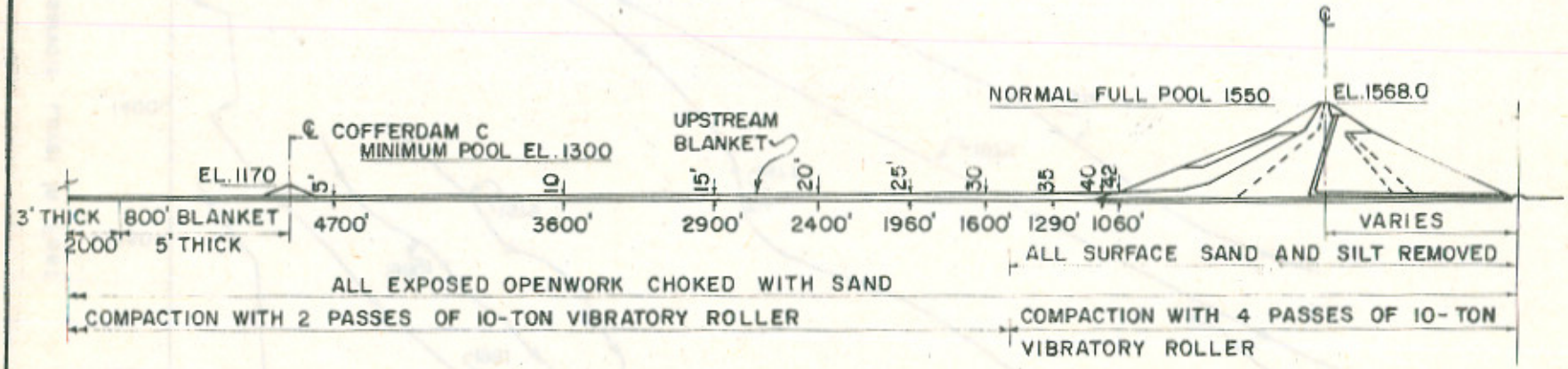
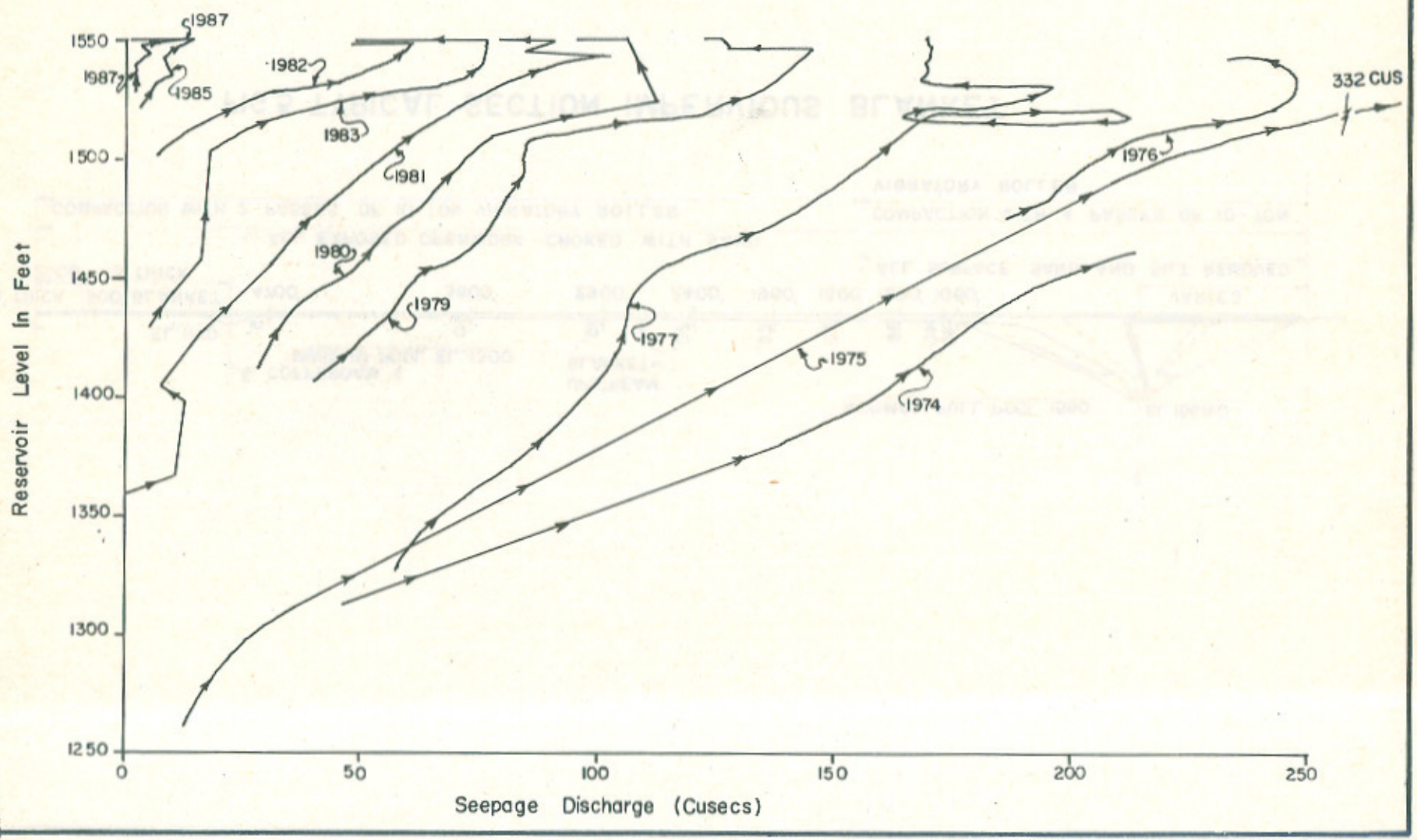


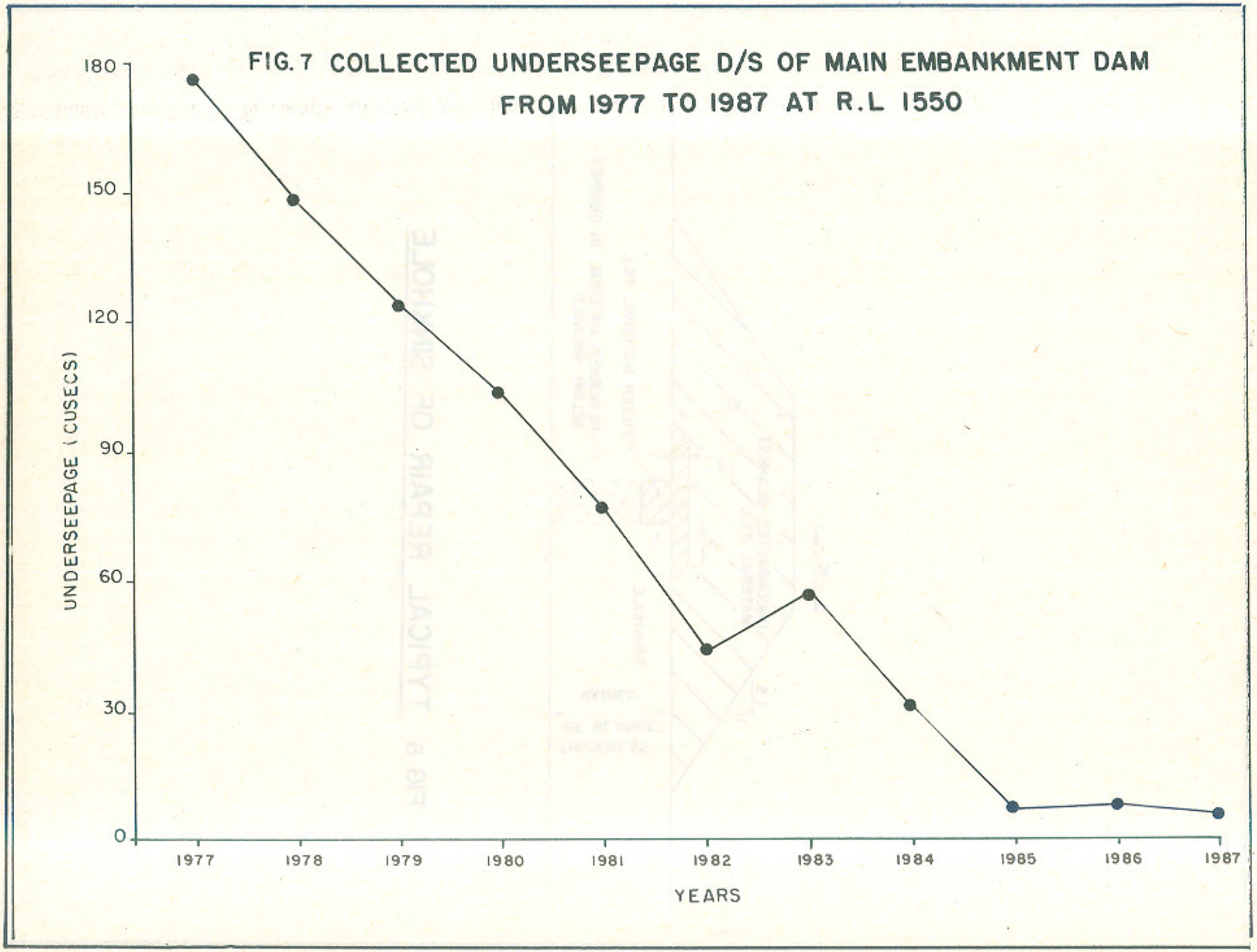
FIG.5 TYPICAL SECTION IMPERVIOUS BLANKET

Naeem

FIG.6 RESERVOIR LEVEL V/S TOTAL SEEPAGE



S.MANSOOR


FIG. 7 COLLECTED UNDERSEEPAGE D/S OF MAIN EMBANKMENT DAM FROM 1977 TO 1987 AT R.L 1550
UNDERSEEPAGE (CUSECS)
180
150
120
90
60
30
0
1977
1978
1979
1980
1981
1982
1983
1984
1985
1986
1987
YEARS

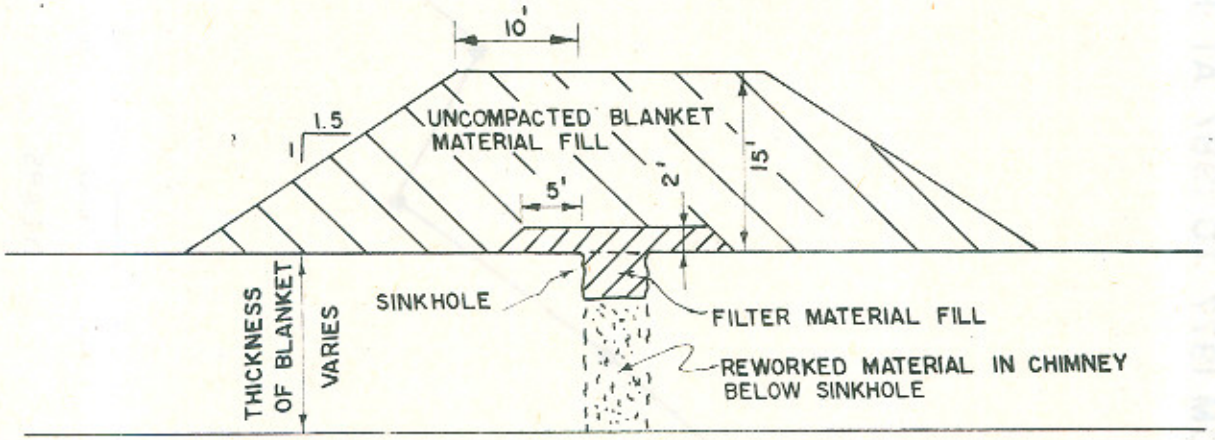


FIG. 8 TYPICAL REPAIR OF SINKHOLE

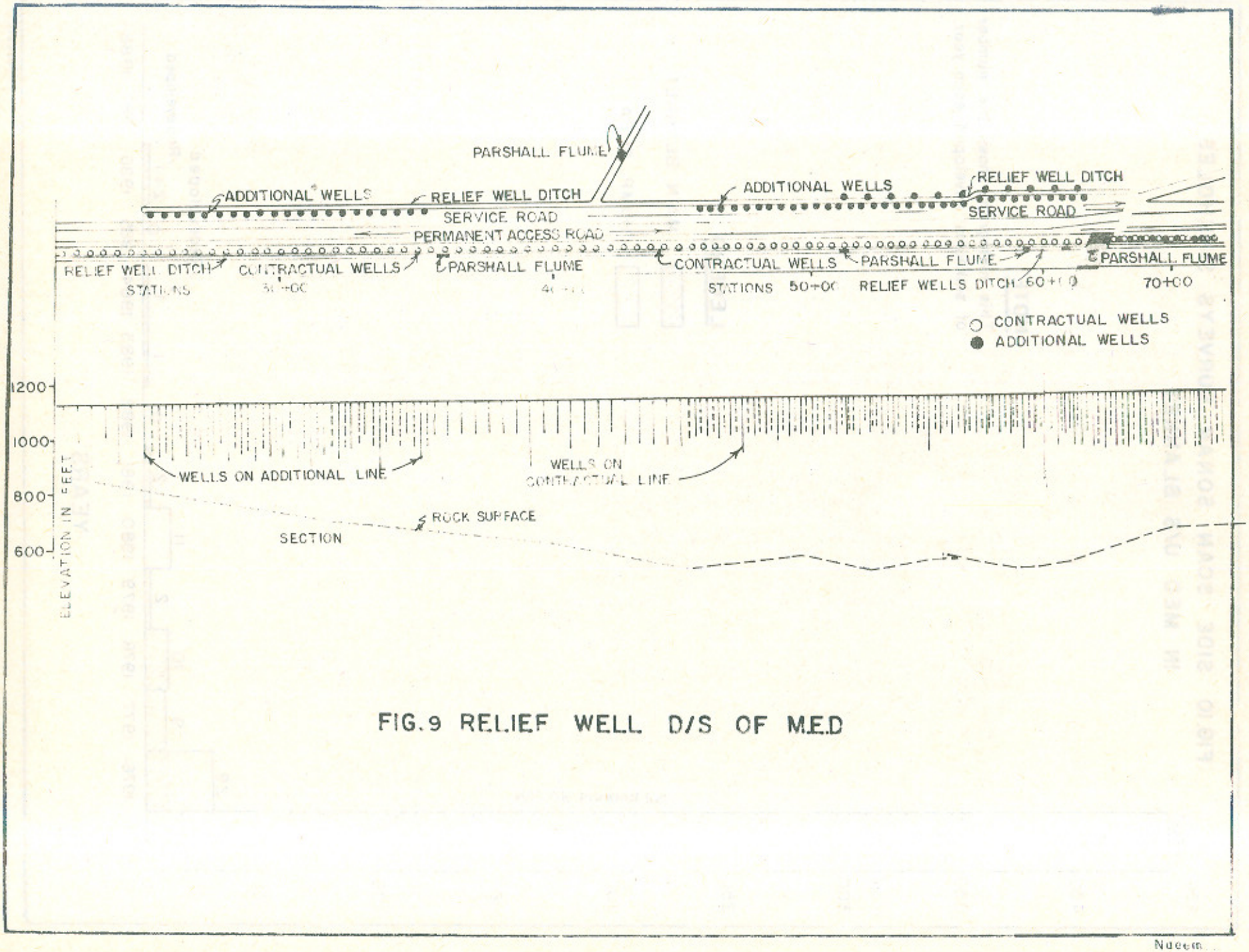
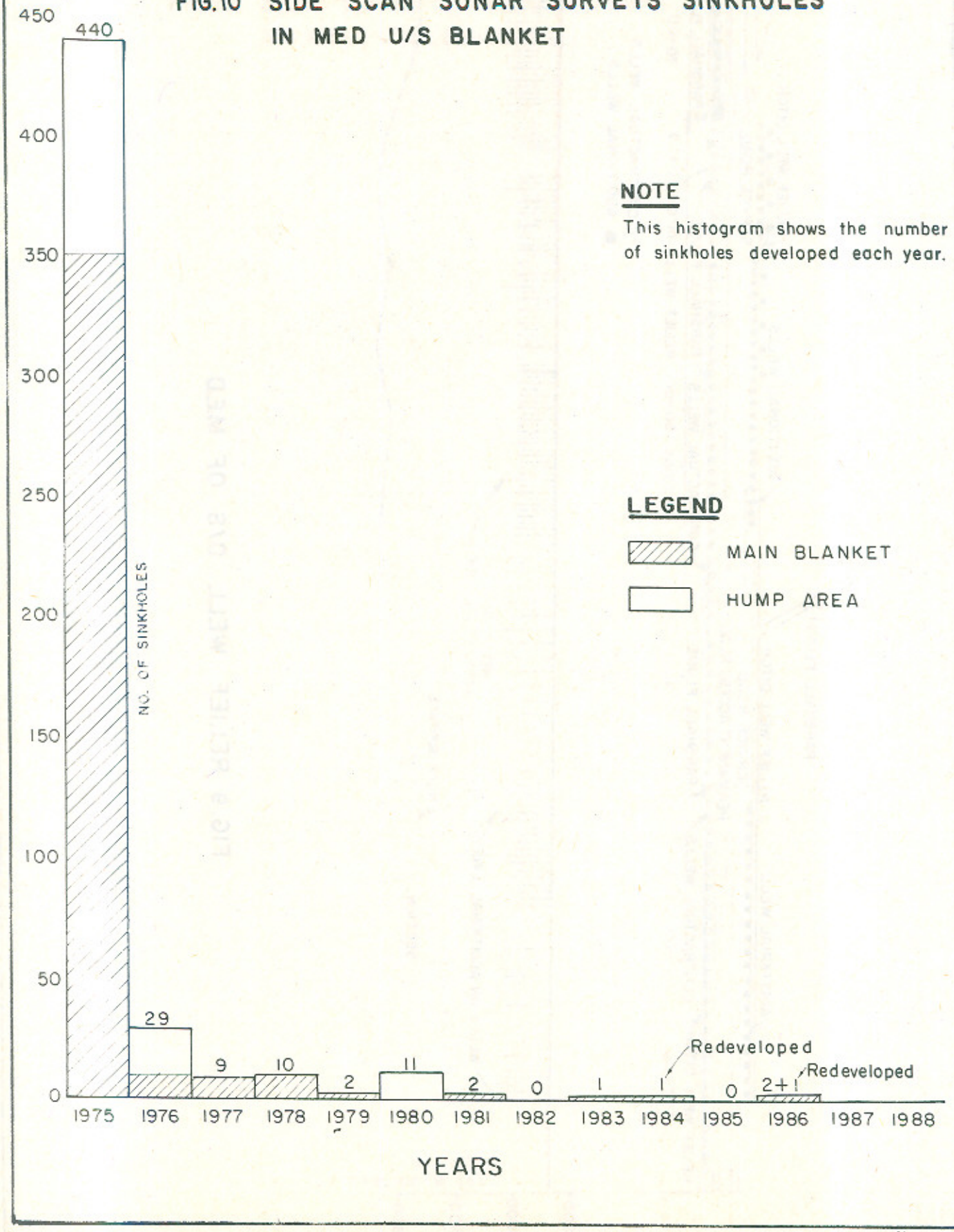


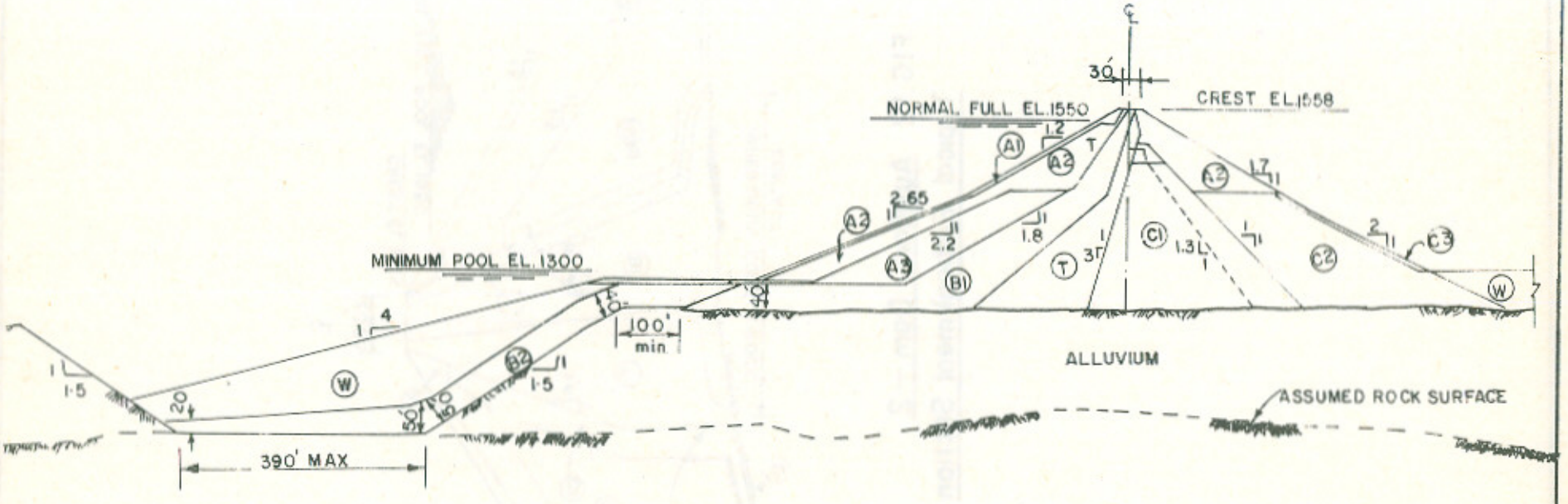
FIG.9 RELIEF WELL D/S OF M.E.D

Naeem

FIG.10 SIDE SCAN SONAR SURVEYS SINKHOLES IN MED U/S BLANKET



Naeem...



CONSTRUCTION MATERIAL

A1	RIP RAP
A2	FREE DRAINING FILL
B1	IMPERVIOUS BLANKET
C1	CENTRAL ZONE
C2	DOWNSTREAM SHELL
C3	FREE DRAINAGE SLOPE PROTECTION
D1	DRAINAGE MATERIAL
F	FILTER
T	TRANSITION
W	WEIGHTED BERM

FIG. II AUXILIARY DAM–I

TYPICAL EMBANKMENT SECTION ON ALLUVIUM

MASHADI

CONSTRUCTION MATERIALS

A1	RIP RAP
A2	FREE DRAINAGE FILL
B1	IMPERVIOUS BLANKET
C1	CENTRAL ZONE
C3	FREE DRAINAGE SLOPE PROTECTION
D1	DRAINAGE MATERIAL
F	FILTER
T	TRANSITION

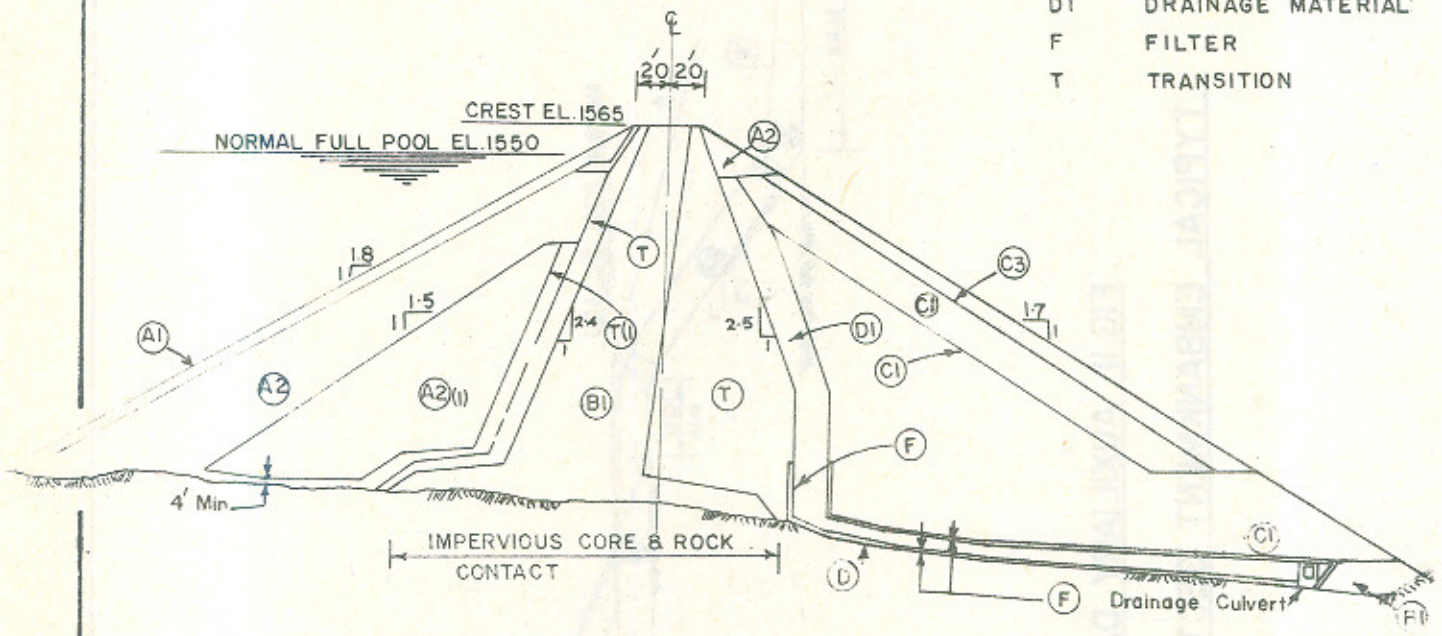


FIG.12 Auxiliary Dam – 2
Typical Embankment Section

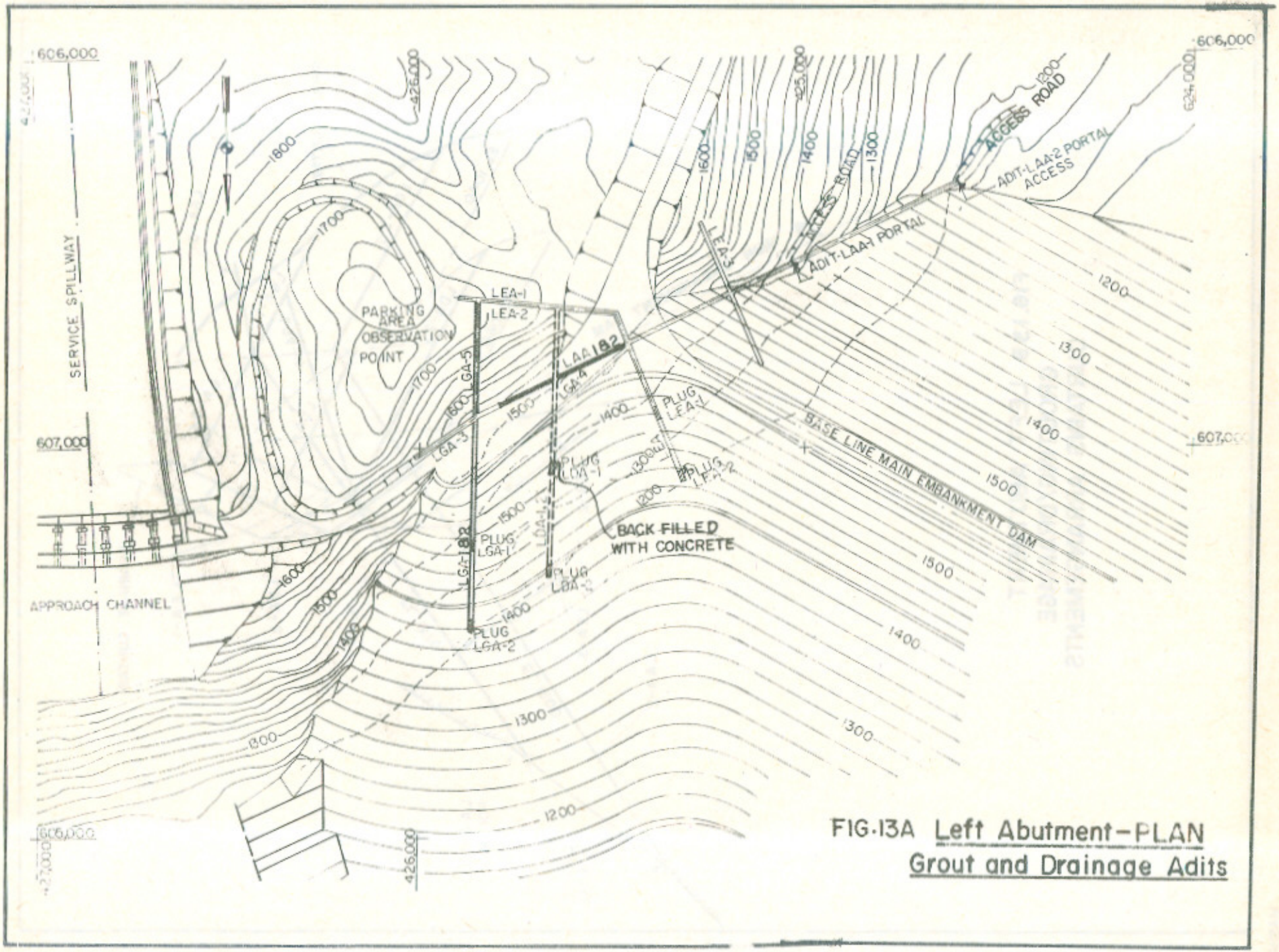


FIG.13A Left Abutment – PLAN
Grout and Drainage Adits

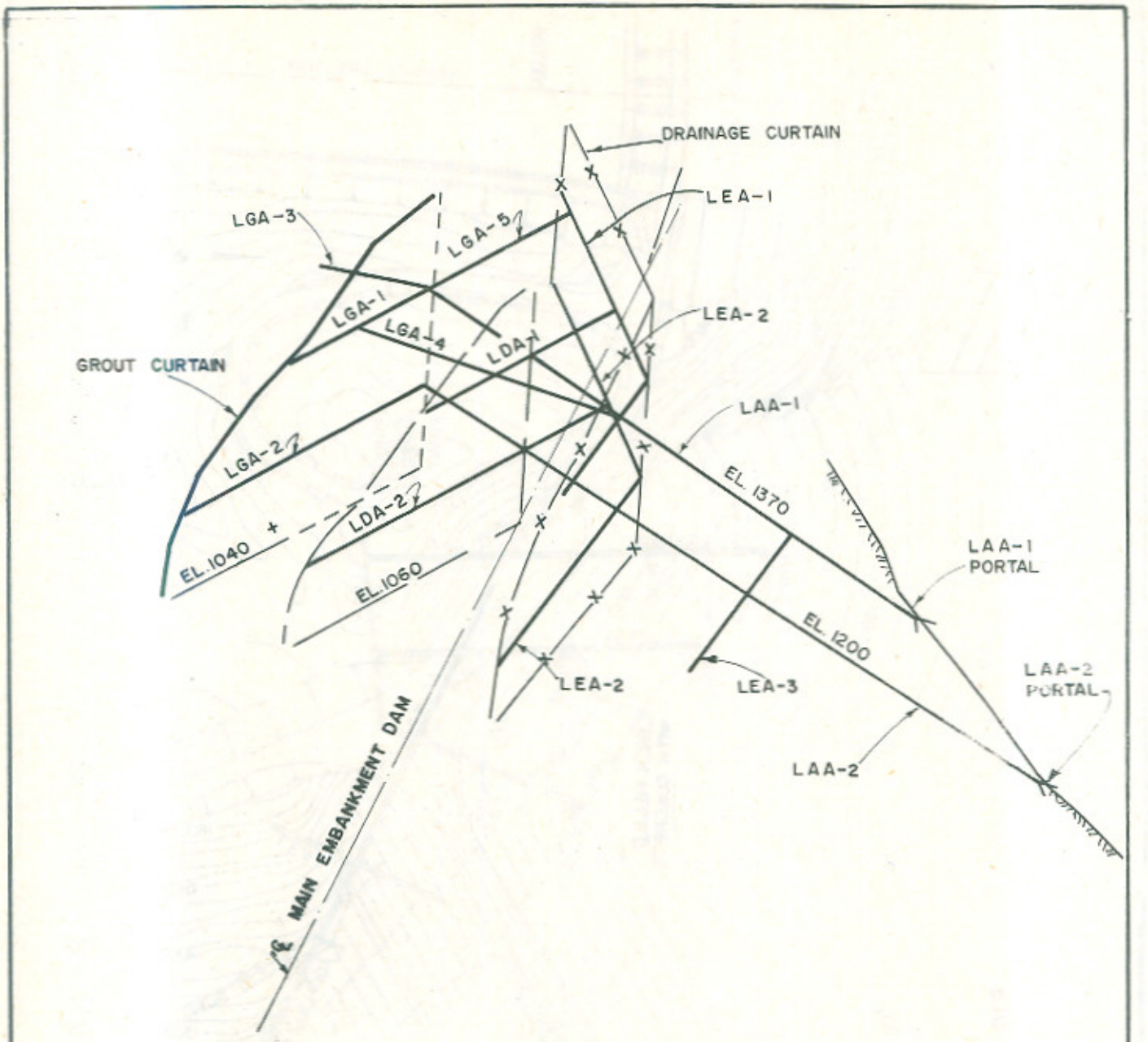


FIG. 13B LEFT ABUTMENT
GROUTING/DRAINAGE
CURTAINS ARRANGEMENTS

MASHADI

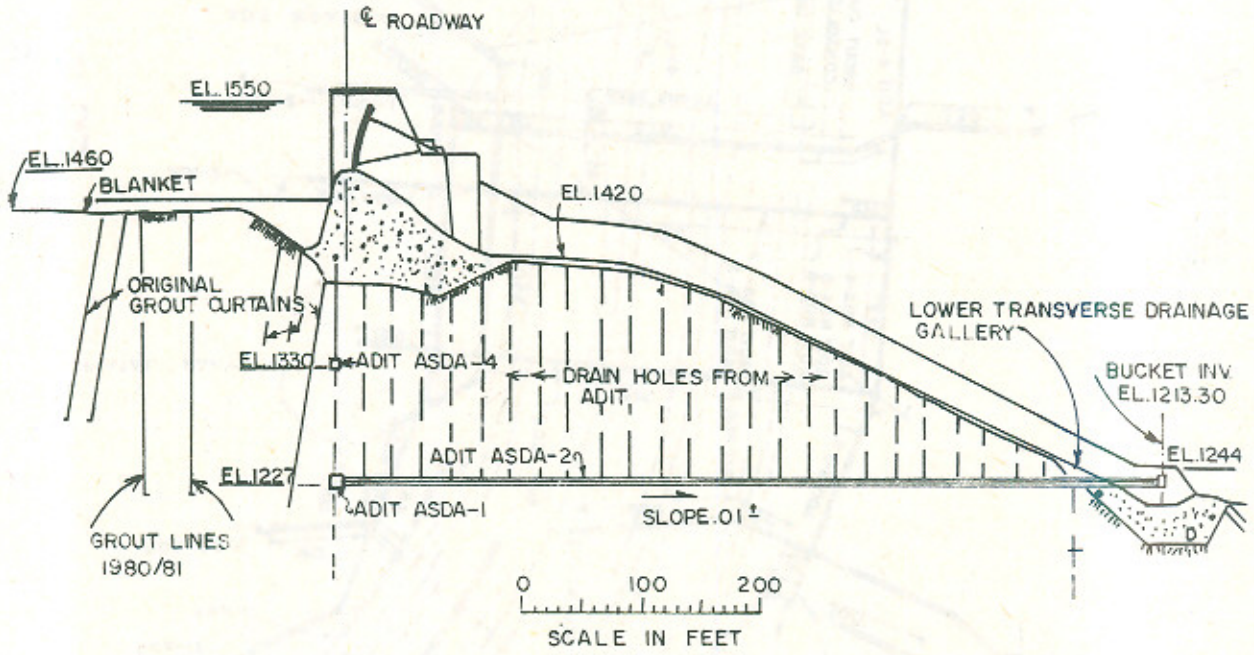


FIG. 14 Auxiliary Spillway Section
Drainage and Grout Curtain

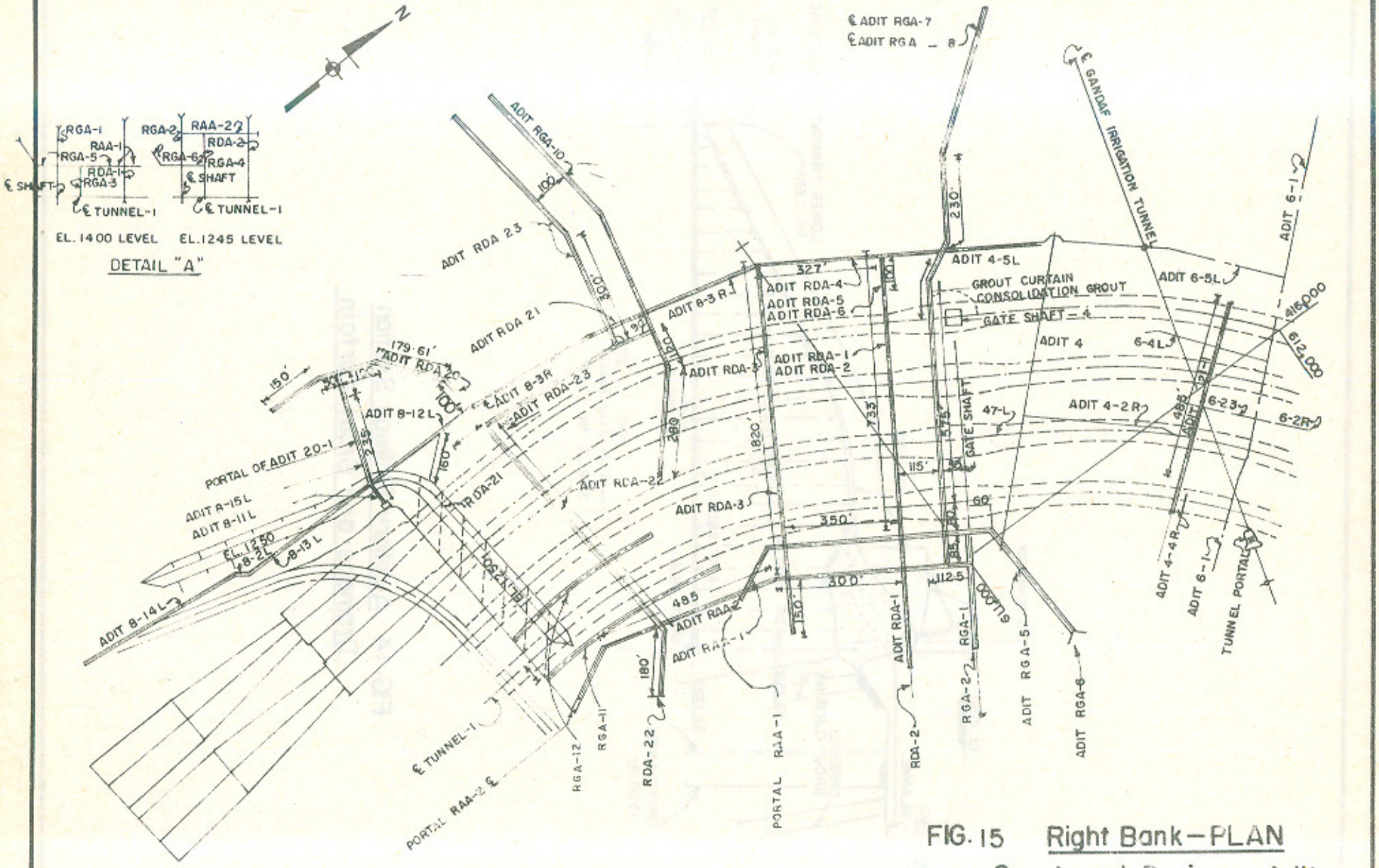
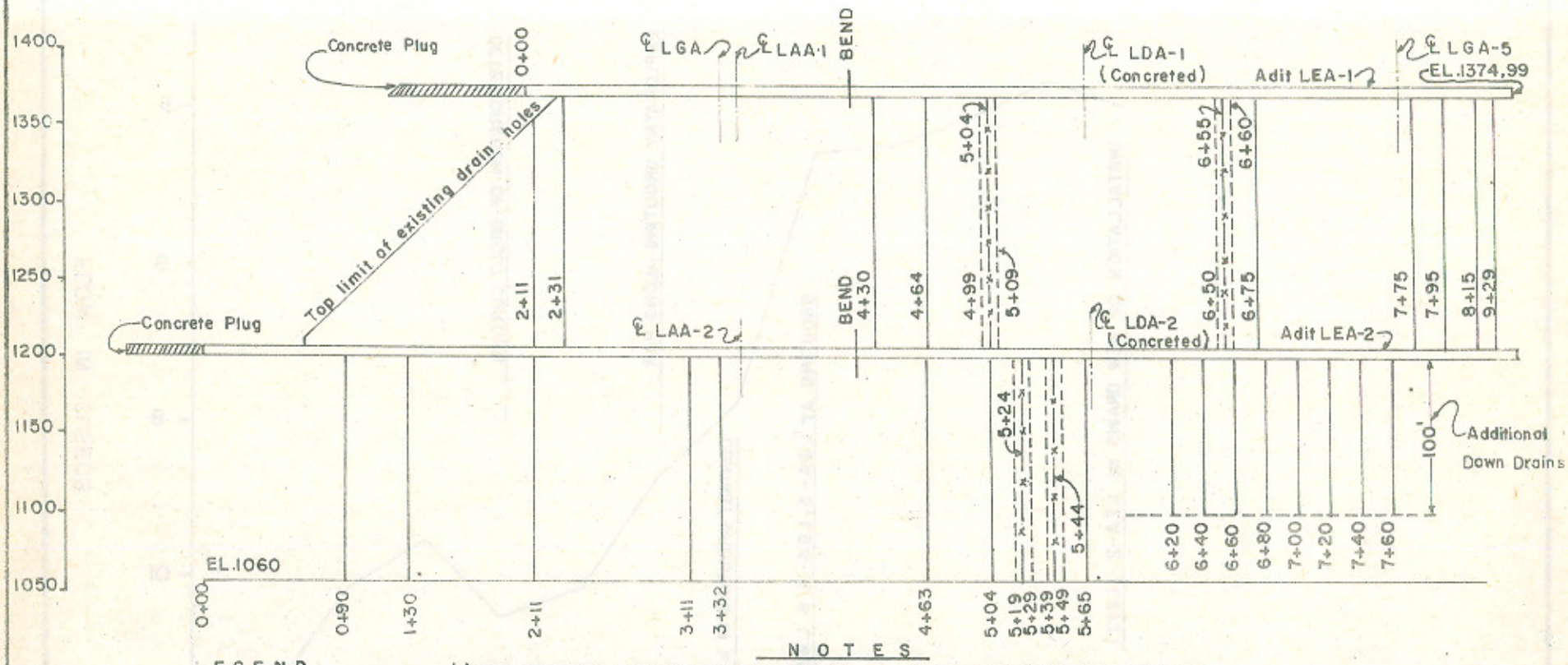


FIG. 15 Right Bank – PLAN

Grout and Drainage Adits

S. MANSOOR.H.



LEGEND

——— HOLE REAMED

—x—x— HOLE GROUTED

------ NEW DRAINS

NOTES

(1) FOR UP DRAINS GROUTING WAS DONE IN TWO STAGES FIRST STAGE WITH 0·6 W/C GROUT MIX AND 2nd STAGE WITH 1·5 W/C GROUT MIX WITH 2% BENTONITE.

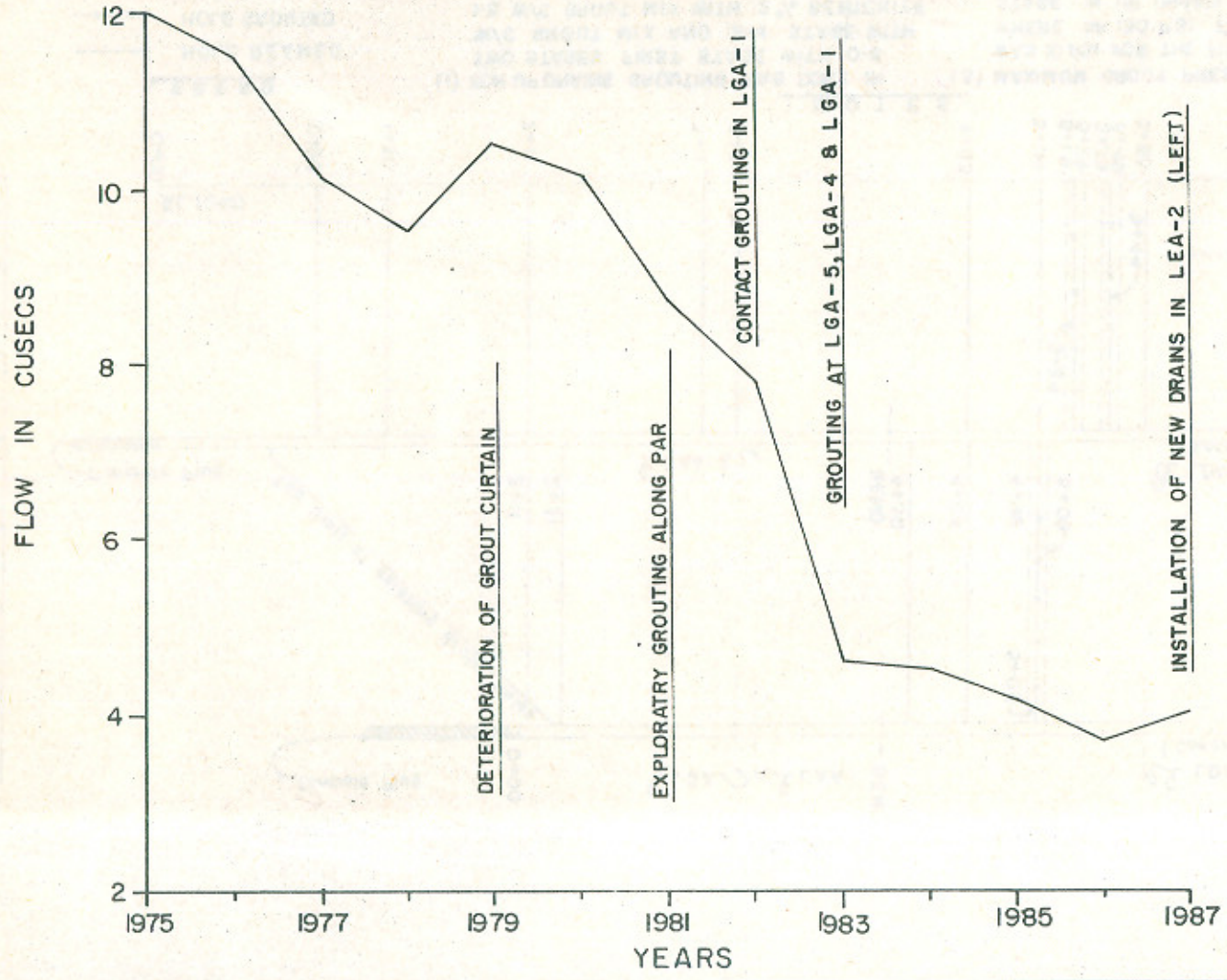
(2) PACKER WAS PLACED AT CONCRETE OF ADIT ROOF FOR THE FIRST STAGE WHERE AS AT 20' FOR THE SECOND STAGE.

(3) MAXIMUM GROUT PRESSURE WAS 10 PSI FOR THE FIRST STAGE WHERE AS 130 PSI FOR THE 2nd STAGE IN UP DRAINS.

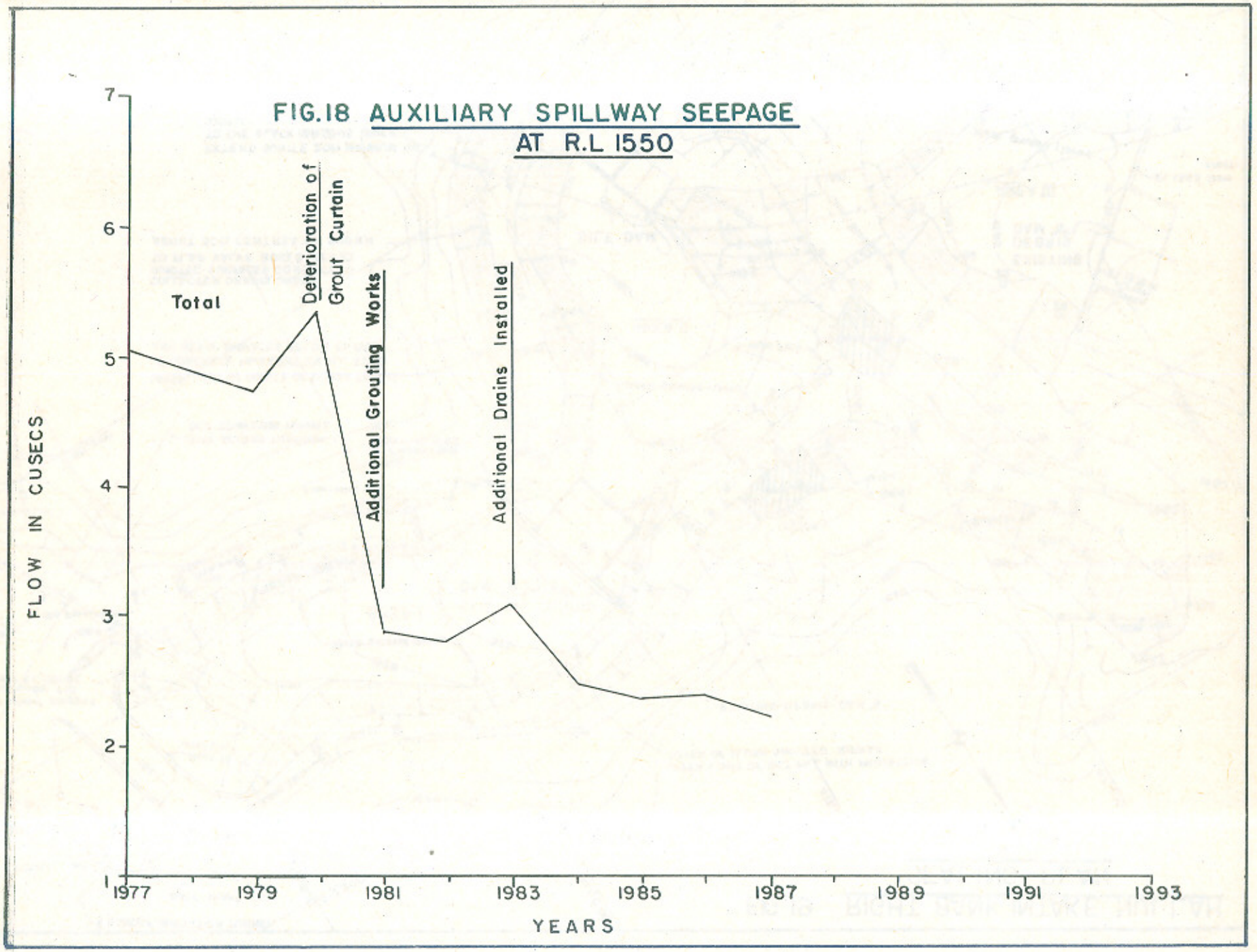
(4) DOWN DRAINS WERE GROUTED IN ONE STAGE WITH 0.6 GROUT MIX AT GRAVITY PRESSURE.

FIG.16 M.E.D. LEFT ABUTMENT, REPLACEMENT DRAINS, 1987

FIG. 17 TOTAL LEFT ABUTMENT SEEPAGE AT RES. LEVEL 1550'



S.MANSOOR.


FIG.18 AUXILIARY SPILLWAY SEEPAGE
AT R.L 1550
FLOW IN CUSECS
7
6
5
4
3
2
1
Total
Deterioration of
Grout Curtain
Additional Grouting Works
Additional Drains Installed
1977
1979
1981
1983
1985
1987
1989
1991
1993
YEARS

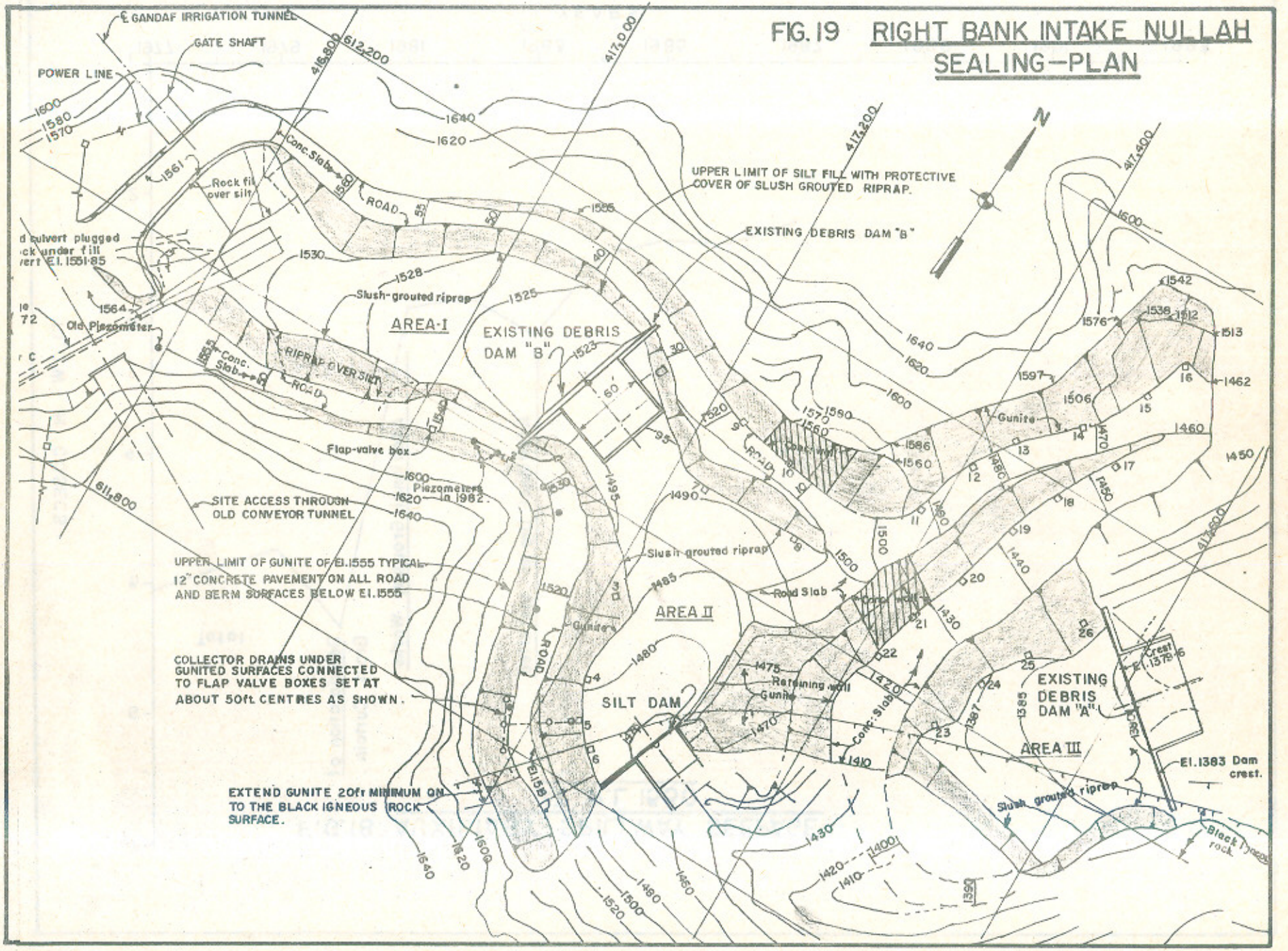
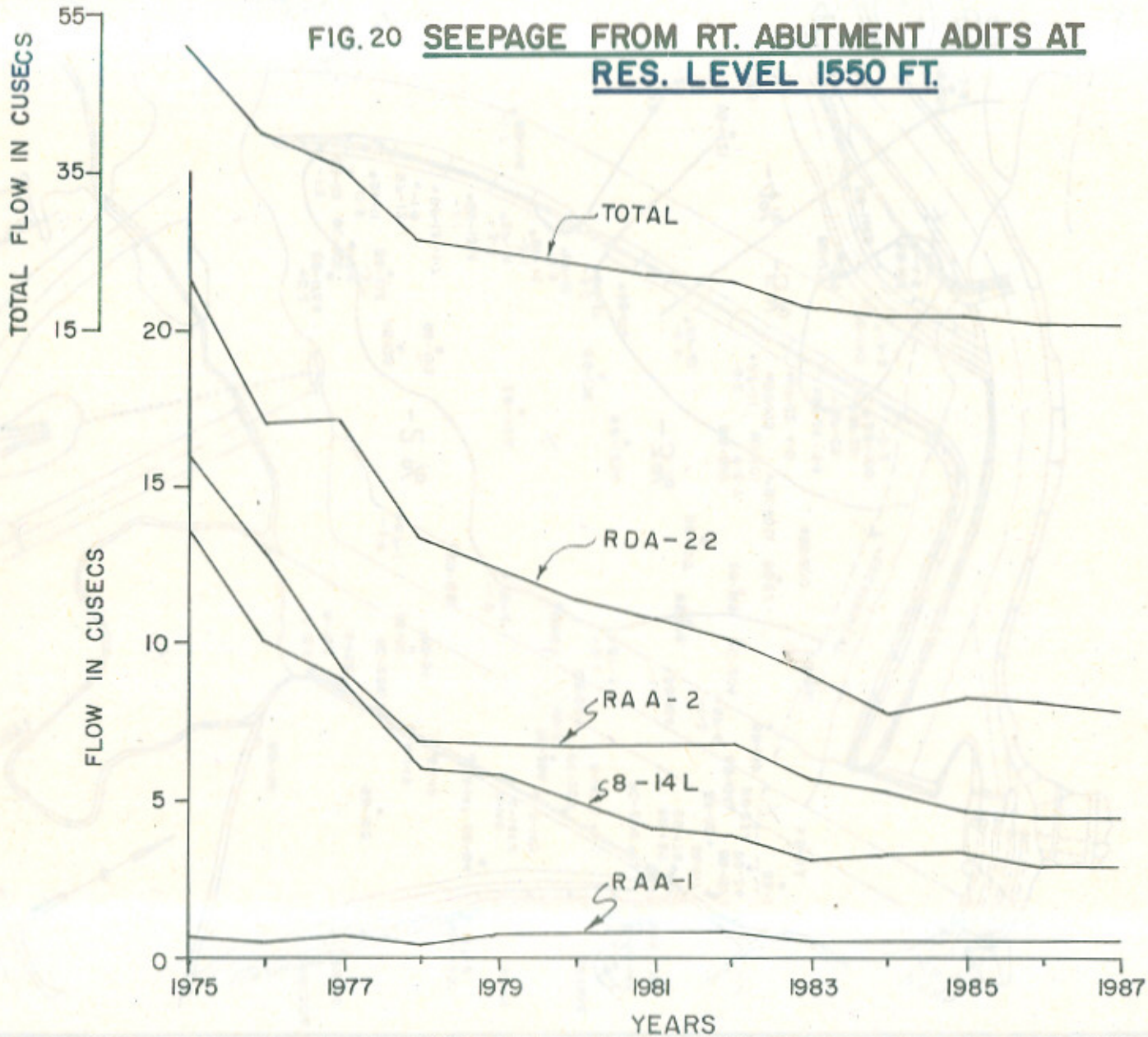

FIG. 19 RIGHT BANK INTAKE NULLAH
SEALING—PLAN
℄ GANDAF IRRIGATION TUNNEL
GATE SHAFT
POWER LINE
416,800
612,200
417,000
417,200
417,400
611,800
N
UPPER LIMIT OF SILT FILL WITH PROTECTIVE COVER OF SLUSH GROUTED RIPRAP.
EXISTING DEBRIS DAM "B"
AREA-I
AREA II
AREA III
SILT DAM
EXISTING DEBRIS DAM "A"
Conc. Slab
ROAD
Rock fill over silt
Slush-grouted riprap
RIPRAP OVER SILT
Old Piezometer
Flap-valve box
Piezometers in 1962.
SITE ACCESS THROUGH OLD CONVEYOR TUNNEL
Gunite
Road Slab
Retaining wall
Crest El. 1379.6
El. 1383 Dam crest.
Black igneous rock
UPPER LIMIT OF GUNITE OF EL.1555 TYPICAL
12" CONCRETE PAVEMENT ON ALL ROAD AND BERM SURFACES BELOW EL.1555
COLLECTOR DRAINS UNDER GUNITED SURFACES CONNECTED TO FLAP VALVE BOXES SET AT ABOUT 50ft. CENTRES AS SHOWN.
EXTEND GUNITE 20ft MINIMUM ON TO THE BLACK IGNEOUS ROCK SURFACE.
S. MANSOOR. H.

FIG. 20 SEEPAGE FROM RT. ABUTMENT ADITS AT RES. LEVEL 1550 FT.



S. MANSOOR.

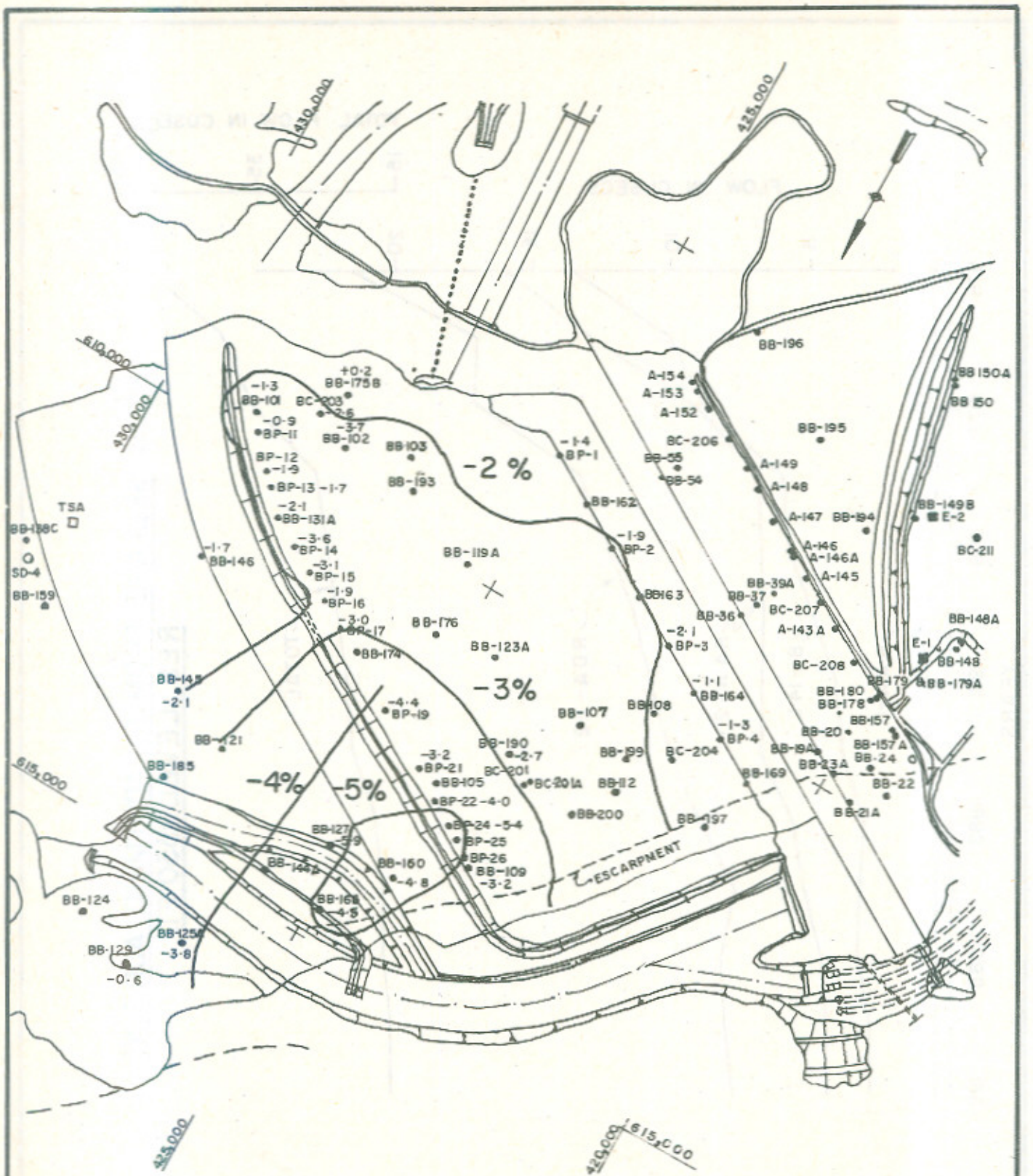


FIG.21 BLANKET PIEZOMETER POTENTIAL CHANGE 1984 – 1985

S.MANSOOR. H.

PERMANENT INSTRUMENTATION

TABLE-2

IDENTIFICATION CHART

TYPE OF MEASURMENT	INSTRUMENT DESCRIPTION			INSTRUMENT LOCATION								TOTAL INSTALLED	TOTAL WORKING	% WORKING
	NAME	MAKE, TYPE	DESIGNATION	AUXILIARY DAM-1		AUXILIARY DAM-2		M.E.D		U/S	BLANKET			
				IN	WO	IN	WO	IN	WO	IN	WO			
A	PIEZOMETER	GEONOR (VIB. WIRE)	P1					13	4			13	4	31
	PIEZOMETER	TELEMAC (VIB. WIRE)	P2	2	1			49	41	32	16	83	58	70
	PIEZOMETER	HOKE (PNEUMATIC)	P3					19	0			19	0	0
	PIEZOMETER	GLOTZL (HYDRAULIC)	P4	29	19			155	34	7	0	191	53	28
	PIEZOMETER	KYOWA (RESISTIVITY)	P5					12	0			12	0	0
	PIEZOMETER	MAIHAK (VIB, WIRE)	P6	38	24			26	18	66	31	130	73	56
	PIEZOMETER	CASAGRANDE	P7							7	4	7	4	57
	PIEZOMETER	OPEN STAND PIPE	P8	25	20	7	6	72	50			104	76	73
B	SURFACE MARKERS + SURVEY REFERENCE POINTS		SM+SR	46	33	14	13	104	90			164	136	83
	DFSD	TAMS (DOUBLE FLUID)	D	16	4			44	15			60	19	32
	TIC	TELEMAC (VERTICAL)	TI	9	8	2	2	19	14			30	24	80
	FIXED INCLINOMETER	TELEMAC (ELECTRICAL)	FI					2	0			2	0	0
	BIAXIAL TILTMETER	SINCO (ELECTROLYTIC)	I3					10	10			10	10	100
	VLDS	TELEMAC (ELECTRICAL)	VI					3	3			3	3	100
	HLDS	TELEMAC (ELECTRICAL)	HI					4	4			4	4	100
	EXTENSOMETER	TELEMAC (MECHANICAL)	E	6	4			18	18			24	22	92
	EXTENSOMETER	MAIHAK (ELECTRICAL)	M4					7	7			7	7	100
	CREST EXTENSOMETER	SINCO (RESISTIVITY)	CEI	1	1							1	1	100
C	SOIL STRESS CELLS	GLOTZL (HYDRAULIC)	ST	30	13			30	18			60	31	52
D	WEIRS			1	1	2	2					3	3	100
	PARSHALL FLUMES							6	6			6	6	100
	VENTURIMETER			1	1							1	1	100
E	THERMOMETER	TELEMAC (ELECTRICAL)	Th1	9	0			32	0	4	1	45	1	2
	THERMOMETER	MAIHAK (ELECTRICAL)	Th2							20	7	20	7	35

A. Water Pressure

B. Deformation

C. Stress

D. Flow

E. Temperature

IN = Installed Instrument

WO = Working Instrument