Paper No. 176

OPERATION AND MAINTENANCE OF DAMS

Dr. Izhar-ul-Haq

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By

DR. IZHAR-UL-HAQ

INTRODUCTION

Pakistan has about 35 large dames (greater than 15M), a number of small dams and 20 barrages. WAPDA is responsible for the operation and maintenance of the two most important and highest dams Mangla & Tarbela. It looks after Warsak, Hub, and Khanpur Dams also Simly dam is owned and operated by C.D.A but monitored by WAPDA. The rest of the dams are monitored by the Provincial Governments. As Mangla and Tarbela are life line of Pakistan economy, they are being operated and maintained meticulously according to the international Standards. In addition to the site operation and maintenance organizations, WAPDA has a Dams Monitoring Organization (DMO), which acts as a second tier for the safety monitoring of these dams. DMO casries out the Annual inspection of the dams and arranges Periodic Inspection of these dams.

The safety of a dam can be threatened by natural phenomenon such as flood, rock slides, earthquakes and deterioration of the dam body and foundation. In the course of time the dam structure may deteriorate due to ageing and in some cases higher internal pressures, path of seepage and migration may develop. Usually these processes are slow and not readily discerned by routine examination. Continuous monitoring of performance of the dam will ensure detection of any inadequacy either in the design or any problem brewing inside the dam which may cause failure. For preventing possible failure of a dam, it is essential that the stresses, strains, pore pressures, seepage etc., at crucial locations are constantly monitored and compared with the design values. The following parameters are measured constantly: (a) Pore Pressure (b) Earth pressure stresses in the core and filter (c) Internal settlements in the dam embankment and foundation (d) Horizontal movements within the dam embankment: Cracks if any; (e) Horizontal movement and settlement of the embankment surface: (f) Ground acceleration during an earthquake and the seismic response of the structure: (g) Drainage of the seepage water through dam and foundations. The installation of an effective flood warning system on the project is essential for proper reservoir operation so that the dam is not overtopped in case of

sudden flood when the reservoir is already full. The problems encountered in Pakistan in safety evaluation of dams fall broadly in the following catagories:

1. Siltation: Depletion of reservoir capacity and wear and tear of structures due to silty sand.

2. Seepage, sinkholes, migration of fines, Pore pressure and sand boiling.

3. Excessive scour downstream of spillways.

4. High seismic acceleration.

5. Inadequate spillway capacity.

6. Alkali Reactivity.

7. Crack in embankments.

This paper describes briefly the Operation and Maintenance of dams (Embankments and Hydraulic Structures only) in the control of WAPDA. It does not deal the mechanical, electrical equipment and power houses. It also does not include the operation and maintenance of the dams by the provinces. The Operation and Maintenance of the different projects is discussed separately for each project.

MONITORING

The high dams in Pakistan lie above tihickly populaterd areas and rich agricultural lands. Failure of any such dam could spell a disaster and an irripairable loss. The economy of Pakistan largely depends on the dams. The monitoring of these dams is carried out in three tier-I. The first tier monitoring is carried out by the Operation and Maintenance (O&M) Staff of the Project. The major functions of the first tier are :

-- Collection and reduction of data;

-- Preparation of monthly instrumentation reports;

-- Daily visual inspections of structures for detecting & reporting any special observation pertaining to instrument readings, physical chage in concrete and earthworks or any other important phenomenon;

-- Emergency remedial action on first aid basis;

-- Dissemination of information and data;

-- Maintenance and repair of instruments and their readout equipment.

TIER-II.

Second tier monitoring is carried out by Dams Monitoring Organization. The Dams Monitoring Organization, head quartered at Lahore, is a part of Dams Division of WAPDA under the overall control of General Manager Dams and is headed by a Chief Engineer. The DMO is responsible for analysis and long term in terpretation of data. Teams of this organization inspect dams and reservoirs annually. The major functions of the second tier are :-

-- Analysis and interpretation of surveillance data including pore pressures, seepage and movement, etc.

-- Study of seismic records and review of design parameters.

-- Diagnosis of any problem and advice and assistance in implementation of remedial and rehabilitation work.

-- Review of instrument observation schedule and data collection.

-- Arranging and rendering assetance for independent periodic inspections.

-- Making available consolidated reports, description and background histories of specific problems and long term studies of various phenomena and remedial measures.

TIER-III

The third tier start monitoring is the periodic inspection which is usually under taken at an interval of 5 years. These inspections should be organized by the Dam Safety Council of the country but the council has not been established as yet. Therefore this responsibility is also looked after by DMO for Wapda Dams only.

REPORTS

Tarbela Dam Project: In addition to the Design Reports and O&M Manuals (eleven volumes) many reports have been written on each problem and its remedies on Tarbela Dam Project. Many papers have appeared in the National and International Conference. As a routine monthly progress and annual performance reports are written by the site staff organization.

DMO, Wapda reviews the performance data and writes the following reports:

i) Performance Report at Max. Pool Level;

ii) Performance Report at Min. Pool Level.

DMO carries out annual inspections and write reports. So far two annual inspections have been carried out by DMO in 1985 and 1989. First Periodic Inspection was carried out by TAMS in collaboration with Wapda, NESPAK and ACE in 1986-87 and its findings were presented in two volumes.

Mangla Dam Project: In addition to the design report and O&M manuals, many detailed reports were written on each and every aspect of the project. As a routine the Mangla Dam Organization issues the following reports:

i) Monthly Instrumentation Report;

ii) Monthly Movement Control Survey Report;

iii) Monthly O&M Report;

iv) Annual Consolidated Report.

DMO writes quarterly performance reports based on the monitoring data of the project. DMO has carried out 10 Annual Inspections of Mangla Dam Project and written report. First Periodic Inspection was carried out by the Consultants M/s. Binnie and Partners in 1973. Second, Third and Fourth Periodic Inspections were carried out respectively in 1978, 83 and 89 by Wapda in association with NESPAK and ACE.

Hub Dam Project: Design Reports and O&M Manuals were written by the Consultants M/s. ACE. Hub Dam Project Organization issues the Monthly Progress Reports containing the instrument data and O&M works and the Annual Consolidated Reports.

DMO writes quarterly performance reports. DMO has carried out so far 5 Anual Inspections in 1981, 82, 83, 85 and 88 and issued the reports.

Two Periodic Inspections were carried out by Wapda in association with ACE and NESPAK in 1986 and 89 and reports issued.

Khanpur Dam Project: Design Reports and O&M Manuals were written by the Consultants M/s. ACE. Khanpur Dam Project Organization issues the monthly progress reports and the Annual Consolidated Reports. In sends daily and weekly data to DMO Wapda. Site people write reports on special problem such as Turbidity from Relief Wells.

DMO writes quarterly performance reports. So far four Annual Inspections have been carried out by DMO in 1983, 84, 86 and 89 and reports issued. Two Periodic Inspection were carried out by Wapda in association with ACE and NESPAK in 1985 and 88 and reports written.

Warsak Dam Project: Design Reports and O&M Manuals were written by the Consultants. Site Organization writes Monthly Progress Reports and Annual Consolidated Reports. Due to cracks, movement and silt problems M/s. Golders Associates consultants were engaged by CIDA who wrote reports on the problems. Recently M/s. RSW & NESPAK have written report on the rehabilitation cost study of the project.

DMO has so far carried out 8 Annual Inspections during 1978, 79, 1980, 83, 84, 85, 88 and 89 and written report.

Two Periodic Inspections have been carried out by Wapda in association with NESPAK and ACE in 1982 and 1987 and reports written.

SURVEILLANCE AND INSPECTION

SURVEILLANCE

The term "surviellance" covers the instrumental observation of the behaviour of the embankments, their foundations and abutments and the analysis of the collected information. Surveillance is carried out by the surveillance group through instruments intalled in the project features and their foundation and through geodetic survey of bench marks installed throughout the project.

Surveillance icludes patrolling of embankments, visual inspection, recording the movements and monitoring of the instruments embeded in the body of the dam. The following indications give advance warning of deterioration: (i) Development of Cracks or holes on the surface (ii) Development of undulations on the surface (iii) Existence of wet patches (iv) Presence of sediments in spring water and drainage system.

INSPECTION

Visual inspection of the project features, undertaken by the inspection group, is supplemental to the survillance and is a very important element of the operation and maintenance of the project. Visual inspections will be made routinely and regularly by qualified engineering staff, fully trained in assessment of earth and concrete structures and design of the structures. Close collaboration between the surveillance and the inspection groups is necessary for proper monitoring and maintenance operations.

Inspection of the embankment, spillway, intake structures, tunnels, outlet structures, reservoir rim etc. is made at different seasons according to the requirements of the project operations.

EMBANKMENTS

In addition to daily patrol general inspection of embankemtns is carried out at regular intervals (say at one month) and following points are checked:-

i) Settlement, displa- ement or caracking of the crest;

ii) Heaving, subsidence or slips in embankment slopes;

iii) Slips, erosion or cracking at abutments;

iv) Breaching, erosion of riprap and slope protection;

v) Damage to or blockages on berm drains and mitre drains;

iv) Seepage flows at seepage measurement chambers or springs under observation (General condition, not quantity measurement);

vii) New seeps or springs at the downstream toe of the embrankemtn and the downsmtream slopes of natural ground forming part of the reservoir rim;

viii) Condition of concrete structure (Instrument Houses and Drainage Gallery);

ix) Drainange wells.

On lowering of reservoir, the upstream slope of embankment and the reservoir floor is inspected and sink holes if any on earth embankment are covered with the impervious fill or the cracks on the concrete parts are properly treated. Due to wave

action, the rip rap settles or is washed away. This is replenished. The downstream disintegrated stones are replaced. Vegetation on the downstream slope and in the drains is removed periodically.

During the first impounding meticulous and frequent inspections are carried out. During the full reservoir daily inspection is done. In addition to the routine inspections, special inspections are carried out on the following occasions:- After heavy rains; After high winds; At reservoir level below maximum conservation level; After an earthquake. Inspection and maintenance reports of the projects are prepared and submitted to the concerned authorities and also kept in permanent record. Standard forms are available in the manual for preparing the inspection reports of the different components of the project. Maintenance works are planned in advance to suit the reservoir level or closure of outlet works.

SPILLWAYS

The inspection of civil works of spillway includes the follwoing approach channel, Head works, chutes, stilling basin, tail race channel, galleries etc. Concrete surface roughness settlement, bulge, cracks, sound drummy if hollow, joints settlement offset, sealer intact or removed, leakage, wet spots are also checked. After every major earthquake thorough checking of all the structures and the instruments is done.

MANGLA DAM PROJECT

Mangla Dam Project was completed in 1967. The site organization is headed by a Chief Engineer. There is a Resident Engineer (SE) for operation and maintenance of reservoir dams and spillways. A Superintending Engineer alongwith his staff looks after surveillance-instrumentation etc. of the project. The designers produced 11 volumes of operation and maintenance manual dealing with each aspect of project. The main features of civil works operation are discussed below. Fig.1 and 2 show plan and cross section of the project.

Reservoir Operation: The normal conservation level of Mangla dam is 1202 SPD. The crest of the dam is 1235. The maximum probable flood when routed through the reservoir can be passed through the main spillway and emergency spillway with max. water level of 1229 and a free board of 6 ft. Fig.3 shows the reservoir impounding curve for the year 1988 and the rule curve. There is a temptation by the project organisation to impound water above 1202 SPD and produce more electricity but it has not been acceded to by the authorities as it would encroach upon its capability to pass the probable max. flood.

In 1967 the Gross storage of Mangle reserovir was 5.88 MAF. Due to siltation nearly 0.5 MAF storage has been lost. Watershed Management Organization headed by a Dirctor is working to control the silt entry into the reservoir. The sediment delta has crossed Kanshi river junction and is close to main pocket of resrvoir. It is at a distance of 18000 ft. from the Intake.

Pore Pressures: Mangla Dam was built with clay core excavated from Siwalik clays. Quite high pore pressure (pore pressure ratio U=0.7) developed at the end of construction. Over the last 22 years of operation, the entrapped pore pressures have dissipated gradually. Fig.4 present pore pressure in the clay core of main dam. Sukian embankment Jari dam were built with compacted sandstone. Sukian embankment remains in contact with water relatively for a shorter period. Therefore piezometric pressure and a phreatic line has not developed in the embankment. The foundation rocks of the embankments are saturated. The bed rock dip slightly in the downstream direction. The sandstone beds being relatively previous are the water carriers. There are no pressures higher than the filter blanket under the downstream shell of main dam. However sandstone bed between Ch 86 to 89 which is at a depth of about 150 ft from the natural surface at the downstream toe of the dam, has high pressures. These pressures are being dissipated through the overlying beds and alluvium. Downstream of the main dam in the low lying pond area, there has been sand boiling. The pond was filled with gravels. Downstream of Sukian embankment, because of the excavation, there was sand boiling in certain sandstone beds. These bed were treated with inverted filter. In order to avoid development of pore pressures and ensure the stability of intake embankment hill and fort hill, P and T adits were excavated in the abutment. Up and down drains were drilled from the adits. The drains are working as designed and well maintained. Periodically encrustation is removed from these drains.

Seepage: Seepage is measured at a number of points downstream of the project. The seepage varies with the reservoir level. It is increasing slightly for the bed rocks for the same reservoir level with time. Fig.5 shows the cummulative seepage measured at the portals of P and T adits. Seepage is emerging at high hydraulic gradient from several sandstone beds (3, 7, 12, 13, 14, 24 and 27 of Sukian dam and 9 of Intake dam) some of which are close to the downstream toe of dam. Boiling phenomena is being observed in beds 9 and 10 of Intake dam and 6, 12, 24, and 27 of Sukian dam. All the points where boils appear, have been covered with filter. Existing choked filters of Bed-27 were rehabilitated. As recommended in PIMD-1983, trial blanketing of one bed would be done. A start would be made with Bed-27 and on its success other areas could be taken up. The work is planned to be done departmentally by the O&M Organization.

Settlement: Settlement is measured at a number of points on the crest, upstream and downstream shoulders of main dam, Sukian and Jari dams with the help of USBR

guages. Slope indicators have also been installed at a number of points. The main dam being the highest and having the plastic clay core has settled maximum. Provision was made for 3 ft. settlement of the main dam. It has settled 4.8 ft uptill now. Fi.g6 shows the settlement of main dam with time. The settlement of other embankments has been nominal. Howver swelling was reported at Sukian embankment for several years. Later on it was found that the survey marker was disturbed and the data for several years was wrong.

Slope Maintenance: During every drawdown the usptream rip rap is inspected and reinstated or replenished if required. The weathered stones are replaced. Due to severe wave action at station 79 to 80 of main dam there was recurring problem of rip rap displacement. Since it was difficult to replinish the required size of the rip rap at elevation 1180 to 1195, therefore the O&M staff constructed three rows of concrete slabs on trial basis. They are 12' x 5' x 2.5' thick with joints in between. The slabs were cast in situ over the gravels. although the slabs are withstanding the wave action but the filter material underneath is being washed away. The smooth panels increase the run-up of waves. It is better to have the rip rap of proper size.

The downstream slope protection is withstanding the weathering action and rain erosion. A few stones here and there which are disintegrated are replaced. Vegetation on the downstream slope, berms and in the drains is removed periodically.

Unprotected clay/sandstone beds on upstream beyond station 167 of Intake Embankment are being eroded and undermined by the wave action. The slope needs to be cut back to stable angle and protected to avoid any encroachment towards the Intake area. The drains on the downstream of main dam, Sukian embankment and Jari dam which carry the surface run off and seepage water are occasionally damaged due to settlement, erosion and undermining and are repaired.

The narrow ridge between the right abutment of Sukian dam and left of Intake Embankment on the upstream side was being eroded away by reservoir wave action and by discharge of surface run off on the clay bed slope. A mount break wall of gabions was constructed between Ch. 162 to 172 from RL 1198 to 1208 ft. Surface drainage was provided. The softer Swalik rocks were eroded by wave action or surface water erosion are protected by gabions. Occasional repair of the gabions at different places in the project is carried out by the O&M staff.

Spillway Maintenance: Main spillway of Mangla dam has been designed to pass 870,000 cusecs with reservoir level at 1202 SPD or 1010,000 cuses at RL 1228. The spillway consists of an approach channel, headworks, two stage stilling basin with two chutes and pools separated by a weir and a tailrace channel leading to the river Jhelum.

The gate structure is a 504 ft long reinforced concrete structure, 250 feet wide at the base and 185 feet high. Expansion joints 1 inch wide divide the structure into three monoliths each containing three 36 feet x 40 feet gate orifices, with cill at level 1086 SPD, formed by the 15 - 20 feet thick base slab, 12 feet thick piers and the massive headwall. The gate trunnions are supported by a reinforced concrete beam which is continuous over the three bays in each monolith. Post-tensioned anchors tie the gate trunnion girders to the peirs.

Water released from the gate structure passes down the parabolic upper chute into the upper stilling basin, where there is a deflector to ensure stable hydraulic performance. A concrete weir 45 feet high with crest elevation 1000 SPD terminates the upper stilling basin. Water passing over this weir flows down the lower chute which has a constant gradient, into the lower stilling basin. Baffle blocks 25 feet high and a dentated end sill ensure the effective hydraulic performance of this final stage of energy dissipation. Both stilling basins have sloping sides but reverse flows are prevented by groin walls.

Each of the nine openings in the headworks gate structure is fitted with a 36 feet wide radial gate which closes a 40 feet high orifice. The skinplate assembly of each gate, consisting of a corrosion-resisting clad steel skinplate with horizontal stiffners, is supported by four vertical girders. Each vertical girder is carried on three radial arms which converge to a trunnion assembly. Thus the load on the gate is transmitted to the gate trunnion girder through the four trunnions. Side rollers on each gate run on metal guides embedded in the side of the gate orifice and control lateral movement. Rubber seals are provided at the sides and along the top. The rubber top seal acts only when the gate is closed. Due to weathering and wear and tear of the rubber seals, leakage increased its permissible value. In 1988 these seals were replaced.

Operation of Spillway: The primary function of the main spillway is that of safety value for the reservoir. It has not been designed as an outlet suitable for relasing irrigation demands and should not be used for this prupose except when no other means of satisfying the irrigation demand is available. The satisfactory operation of main spillway depends on its physical condition. The satisfactory operation of main spillway depends on its physical condition. The maintenance of smooth concrete surface is necessary to pass the required flows. Any damage should be made good with epoxy mortar unless the damage is deeper than 3 inches when concrete would be applied.

The main spillway is of unprecedented size and intensity of flow when operating at its full cpacity. It is expected that when operating at high discharges some minor damage may be caused, which if not repaired promptly, might have serious

consequences. It is essential that every year after the monsoon flood has passed and the base flow had dropped to below the irrigation/power requirements, the main spillway should be taken out of service and a careful inpsection made. Any damage found should be repaired before the reservoir again rises to conservation level.

The gates should be raised, starting with the centre bay, proceeding outwards to the end bays. The acceptable sequence of gate operation is then either: - (a) Gates Nos. 5, 6, 4, 7, 3, 8, 2, 9, 1 or (b) Gates Nos. 5, 4, 6, 3, 7, 2, 8, 1, 9. Lowering of gates should being at the end bays (Gates 1 and 9) and proceed inwards to the centre bay in the reverse order to that set out in (a) or (b) above. To rduce the risk of cavitation damage in the water passage downstream fro the gate, the minimum permissible gate opening should be ascertained for the rppropriate reservoir level from the elevation/discharge curve for one gate at partial openings. For example, when the reservoir is at 1186 SPD the minimum gate opening is 5 feet, at conservation level, 1202 SPD the minimum gate opening is 13 feet 6 inches. If it is required to pass smaller quantities of water than those produced by the minimum openings, the fewer gates should be opened at the minimum openings.

To avoid unsatisfactory loadings on gate piers and hydraulic performance of the spillway while the gates are being opened, it is desirable to open the gates in steps of 5 feet, starting in the centre and working out until all gates are open five feet. Before releases are made through the spillway a check must be made to ensure that nobody is in a position to be endangered by the discharge. Gate movements should be kept to the minimum consistent with proper water management.

Drainage: The main spillway has an extensive drainage system designed to control uplift under the structures and to reduce seepage pressures within the earth slopes. There are filter drains under the concrete structures, gravity drainage wells, pumped drainage wells and a pumped drainage gallery. The drainage systems must be checked thoroughly to see that they are functioning correctly. Malfunctioning of the drains will usually be apparent from the records of the discharges, although care must be taken to distinguish the symptoms of malfunction from those of unusual drainage conditions.

Headworks - Joints: The three headworks monoliths are separated by 1 inch wide expansion joints. Flexible waterstops have been provided in these joints to prevent water from seeping right through the headworks structure. If water is observed to be seeping through, an investigation must be made to determine the source of the leak.

Trunnions: The concrete packing between the gate trunnion shoes and trunnion beam must be inspected for looseness or disintegration and for signs of fracture of the anchor bolts.

Upper Chute and Upper Stilling Basin: The paving of the chutes and stilling basin has been constructed with a pattern of joints spaced at 25 feet in both directiions, where controlled cracking is induced by discontinuing alternate reinforcement bars. Seepage through the cracks is prevented by watertsops These joints must be inspected and all spalled edges made good. If leaks develop in the waterstops, increased flow will be observed in the underdrains. From observation of the underdrain flow, it would be possible to locate a leak fairly closely. To stop leaks in the control joints, grout the joints under low pressure so as not to displace the slabs or grout up the under-drains. Any further movement of the slabs will re-open joints and the process must be repeated.

The 2 inch wide joint between the headworks structure and the top of the upper chute has been provided to allow for relative movement between the two sections of the spillwasy, particularly under water loading. The ploysulphide sealer in the top 2 inches of this joint has been eroded by water flow and this space has been left empty. Different sealers, tried so far in this two inches wide joint, have failed and the joint remains open against hydraulic flows. A remedial treatment was proposed in PIMD-1983 but that has not been tried. Part of the joint has been filled recently with a new filler and the rest should be filled up with an inexpensive filler because it is likely to pulled out in time. Careful inspection of this and similar joints will be required to ensure that:-

1- No cavitation or impact damage has occurred to the concrete on the downstream edge. The downstream edge has constructed slightly lower than the upstream edge to alleviate erosion damage.

2- No stones, grit or other obstructions have accumulated in the cavity where they will prevent the joint from closing when necessary.

3- None of the polystrence filler has been pulled out.

The floor uptream of deflector had eroded by the gravel brought in by the flow during early operations of the Spillway but it has been left unrepaired to deteriorate. It must be repaired with concrete of suitable size aggregate which should not be too wet to avoid shrinkage cracks size aggregate which should not be too wet to avoid shrinkage cracks or with epoxy sand mortar. The stilling basin was dewatered after 4 years in 1989. It must be dewatered, inspected, cleared of any debris, and repaired every year.

Lower Chute and Lower Stilling Basin: The normal construction and control joints must be inspected for spalling and the expansion joints must be inpsected to ensure that, if the joint sealer has been eroded, the gap thus left if free of obstructions, the edge are undamaged, and the ploystyrene joint filler is intact.

Lower Stiling Bains: The Basin is under water and concrete surface is covered with algae. An inspection through under water camera and the results reported of an earlier inspection (1985) by driver indicate that the first radial joint on right side has spalled and the floor from toe of chute to the downstream side of Baffle Blocks has eroded, apparently with gravel brought in by the flows. Under water repair should be arranged.

Tail Race Channel: The left side of the channel should be cleared of trees and protruding gravel terrace to even out the flow and also to improve its approach to the Access Brdige.

TARBELA DAM PROJECT

Construction work on Tarbela Dam Project started in March 1969 and first reservoir filling commenced in 1974. Due to certain problems the reservoir was depleted, remedial works were done and the first filling completed in 1975. Although remedial works to different structures were done during the six subsequent years, but the reservoir was impounded and continued to perform its intended functions. Most of the works were done through Tarbela dam contractors. After this Wapda created its own task force to carry out the works. The Consultants (designers) were at site to supervise the constrcution as well as remedial works. The problems occurred not because of the faulty design and construction but because of poor foundation conditions, extra huge size of the project and crossing the frontiers of the known state of the art of the knowledge. Many developments took place in geotechnical and hydraulic engineering. The Consultants left the project on Jan. 1985. The Consultants produced 11 volumes of O&M manuals for the guidance of site staff.

The project is headed by a GM and three Chief Engineers on the civil side and two on the power side. C.E. Project Monitoring Organization is responsible for monitoring the behaviour of the project: recording, plotting and interpretting instrumentation data and advise on remedial measures. Tarbela Dam Project has 4985 instruments installed at various locations. These are read according to their schedule: daily, weekly, fortnightly or monthly. During impounding and full reservoir the surveillance activity is increased considerably. C.E. Operation and Maintenance has the charge of Operation of reservoir, embankments and Spillways and repiar works. The project is stable and running effectively.

Maintenance Main Dam Balanket & Auxiliary Dam: The principal element of the project is the main embankment dam which is the largest in the world. In addition to this two auxiliary embankment dams were required to close a side valley and a saddle in the reservoir rim. The main dam is 470 ft. high earth and rockfill embankemnt with a crest laength of 9000 ft. The main dam has a sloping core which is supported on the upstream and downstream sides by well-graded granular shells.

Continuous with the core, an impervious blanket extends, 7,700 feet in the upstream direction. The blanket is required for underseepage control. Because the foundation fo the dam is composed of deep (up to 700 feet = 200 m) coble gravel and fine sand and alluvial deposits, the construction of a positive cutoff was found to be impractical. Fig. 7 and 8 show plan and cross section of dam.

The upstream shell zones are constructed of good quality granular material; the free draining zone is for stabilization of the upstream slope during drawdowns. Immediately downstream of the core is a transition zone which is rich enough in fines content to serve as a secondary core. The downstream shell adjacent to the transition zone is composed of well-graded, well-compacted granular materials of low compressibility to give firm support to the core. The upstream slope is protected against wave action by free draining slope protection.

The larger of the two auxiliary dams, Auxiliary Dam-1, is similar in design to the main embankment dam. The most important difference is that this dam has only a short upstream impervious blanket terminating in a cutoff trench to bedrock. Axuiliary Dam-2 is situated on a rock foundation and is essentially a centre core dam with free drainage upstream and downstream shells.

Emptying of the reservoir after the 1974 tunnel No.2 collapse revealed a number of sink holes in the upstream blanket. These were repaired with filter material and additional blanket material. After filling of the reservoir additional sink holes appeared, as revealed by side-scan sonar surveys. Barge dumping of material over the sink holes began in July, 1975, and by August 1976 there were no known sink holes remaining. Barge dumping has, however, continued upto the present as a safety precaution and to cover occasional new sink holes and old sink holes which have redevloped. Fig.8 shows histogram of development of sink holes.

The method of treatment developed since 1975 is to dump 50 barge loads on each sinkhole during the high reservoir stage. The fill spreads over a diameter of 200 ft when it reaches the bottom and forms a 2 ft. thickness of fill over the sinkhole which reduces flow into it and forms a reverse filter to trap natural sediment. This method of treatment has satisfactorily stabilized most sinkholes with few redevelopments.

Sonar survey of the main embankment dam blanket is carried out systematically to monitor development new or enlargemnt of existing sinkholes and deposition of slit. A periodic sonar survey of the upstream slope of the main dam and the auxiliary dam is also conducted. Sinkhole formation in blanket has almost ceased after 1984. On the basis of observation and behavoir of the blanket to date it is expected that the development of sinkholes will be minimum during the coming years of reservoir operation. Monitoring of sinkholes development or enlargement of known sinkholes will continue to be carried out by systematic sonar survey of the blanket. The survey

in conjunction with nearby piezometric responses will provide information on the size, depth and the development of sink-holes. It will also indicate the rate at which sedimentation takes place in the blanket.

Sink-hole-like depressions first appeared in the slopes of the Main Embankment Dam in August 1976 and Auxiliary Dam No.1 in September 1977. They were subsequently investigated through test pits and shafts and remedial measures were taken.

In 1984 a sink hole appeared at MB instrumented section of Mian Dam. Because of its similarity with that of sink holes at Auxiliary Dam, it was not investigated. It was backfilled. Some surviving instruments allowed the monitoring of seepage pressures in the vicinity of the affected area. Settlement of the sink hole was measured remotely by extensometer and wire loops installed on the sink hole back fill. During 1988 fill considrable variation in pressure was recorded by the gauges connected to the pneumatic piezo tubes leading to the area. The fluctuations were more rigorous in 1989 filling. Considerable movement was recorded by the extensometer and the wire loops were broken.

The designers TAMS were consulted. They were of the opinion that the event was similar to the one at Auxiliary Dam-1 and the causes were the same. According to TAMS no treatment was required and there was an indication that damage of the kind so far observed would be self healing as seems to have occurred at Auxiliary Dam-1.

In view of the potential significance of sink hole within the dam itself, a second opinion was obtained from Prof. P.R. Vaughan. He concurred with the views of TAMS that the sink hole in the main dam be monitored for the time being rather than treating, in the expectation that this also may become stable. According to Prof. Vaughan, there was only a limited amount of transition fill into which fines can be lost, and this was likely to choke and stabilise. This is also consistant with the general pattern of behaviour at Tarbela. Prof. Vaughan said that there was no possibility of the safety of the dam being at risk through internal erosion of the fill. Although it is not interinsically dangerous, it is sensible to monitor the situation closely.

Seepage under the Main Dam: For control of underseepage beneath the main Dam, a long upstream impervious banket 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 speficications, wheras 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; 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 decreas 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.

On first partial filling in 1974 the seepage was 225 cusecs at RL 1460. On depleting the reservoir the blanket was strengthened and extended. For full reservoir level in 1975, the seepage was 332 cusecs. A second line of relief wells was added downstream of the first line. The major contribution was from the relief wells especially those on the right side where maximum open work gravels were located. Fig 9 shows the seepage with passage of time. Due to siltation the blanket was strengthened and the seepage reduced. For the last two year there has been no seepage at the downstream toe even at the full reservoir. Seepage in both left and right abutments was controlled by additional grouting and drainage from the galleries. The seepage in the abutments has reduced to less than one third that of initial seepage.

Spillways Operation and Maintenance: Two gated spillway structures have been designed to control the elevation of the reservoir in order to serve three purposes: to satisfy need for irrigation, to develop power and to control floods. The spillways control floods. The spillways control the reservoir pool levels between the spillway crest elevation 1492 ft and the normal pool elevation 1550 ft. The service and auxiliary spillways have capacities of 650,000 and 840,000 cfs respectively at spillway design flood elevation 1552.2. Both have concrete gravit-type ogee crest structures with crests at elevation 1492 and radial crest gates 50 feet wide by 58 feet high. There are seven crest gates on the service spillway and nine on the auxiliary spillway. After leaving the ogee crest, the service spillway chute runs 1990 feet (580 m) at a slope of 1 per cent, then drops on a 1 on 2 slope to a flip bucket with lip at elevation 1220 (372 m). The much shorter auxiliary spillway chute slopes at 1 on 2 from near the toe of the ogee to the flip bucket, with the lip also located at elevation 1220 (372 m).

Operation of the spillways began in August 1975. During the first two operating season, the spillways and plunge pools operated without notable incident. In 1977, embayments developed on the left and right sides of the Service Spillway plunge pool. The one on the right became large and started to undermine the right-hand side of the flip bucket and the access road. The resulting unbalanced lateral rock forces on the bucket caused slight movements of the bucket structure to the right.

During the next three dry seasons, extensive remedial work was done: in 1978, removal of unstable rock from the upper slopes of the right side, construction of a rollcrete buttress below the bucket and a groin extending downstream on the right side, plus partial lining on the left side: in 1979, extension of the goin to form a complete right wall and addition of anchored lining and rollcrete on the left; and in 1980, downward extension of proptection works on both sides and at the bucket, and extension of the left side lining and rollcrete around the corner and up the right side of the Dal Darra for a short distance. Similar protection works were carried out for Auxiliar Splillway. The performance of Auxiliary Spillway is better than the service spillway and it is used for greater time.

Right Bank Tunnels: Water is drawn from the reservoir for irrigation or power generation through four tunnels in the right abutment. When the permanent works are fully developed, Tunnel 4 will serve irrigation. The other three will be used for power generation. In their upstream lenghts, all four tunnels have internal diameters of 45 feet (13.7 m) and are concrete lined. The upstream intake portals of Tunnels 1 and 2 were set at an elevation lower than those of Tunnels 3 and 4. This enabled the former pair of tunnels to develop sufficient discharge capacity to handle low diversion flows without any substantial ponding prejudicial to the security of the nearby length of main embankment.

During first reservoir filling in 1974, the upstream gate of Tunnel No. 2 became stuck in part open position. The waterway downstream of the gate sufered damage, 200 ft length of the tunnel collapsed bringing with it a section of the overlying hill. Repairs were begun in November and completed in July 1975.

Problems with the stilling basins of Tunnels 3 and 4 began in August 1974 with loss of a portion of the steel lining of one of the regulating gate chambers and erosion of the concrete chutes leading to the spillway basins. During the 1974 emergency lowering of the reservoir, both tunnel outlets were operated unsymmetrically so as to favor the damaged gate chambers and chutes. This resulted in abnormal circulation of water in the stilling basins and weakening of the floors through erosion by gravel brought into the basins from downstream. In August 1975, as the outlets operated under increasing reservoir levels, the weakened floor slabs of Tunnel 3 failed and large amounts of the underlying rock were eroded away. Restoration work was completed in April 1976, but a defect in the new floor of the Tunnel 3 basin required further work until April 1977. It thus was necessary to forego emptying of the reservoir during the 1976 dry season.

WAPDA, apprehensive about the reliability of the stilling basin, decided in Mrch 1978 to replace the Tunnel 4 stilling basin with a pair of flip buckets. A contract to carry out the conversion was awarded to Indus River Contractors in April 1981, and the work was completed in May 1984.

Left Bank Tunnels: The purpose of the tunel in the left abutment is to provide irrigation releases and is relatively at low level. Ultimately, when the power house is extended for the installation of Hydroelectric Units 11 through 14, this tunnel will replace the lost irrigation release capacity of Tunnel 3. A flip bucket system dissipates the energy and the water joins the spillway outflow channel to fall back into the river.

Abutments Maintenance: The right abutment of the MED has steep cut slopes at the entrance and a the outlet of the tunnels. Within the right abutment is a network of drainage and grouting adits. The large nullah with a significant catchment area to the right of Tunnel 4 intake has been a potential source of debris. To prevent the debris from reaching the tunnel intake area, two concrete dams were constructed in the nullah. Upstream of these dams the nullah was protected by gabions, riprap and flume to reduce erosion of spoil material dumped into the nullah.

Studies made in 1980-81 indicated that the nullah was a significnt source area for seepage passing through the right abutment. To reduce the seepage volume, additional works were carried out in 1982-83 to seal the surface of the permeable limestones between the lower debris dam and the Gandaf irrigation tunnel gate shaft. Fiber gunite and concrete were used to seal the steep side slopes, while the floor of the nullah, including the debris areas behind both concrete dams, was blanketed with a layer of silt covered by slush-grouted riprap for erosion protection.

The left abutment of the MED also contains drainage and grouting adits. Additionally, there are drainage and grouting adits around the service spillway and the auxiliary spillway. Extensive additional grouting of the right and left abutments of the main embankment dam was performed in 1982-83. The additional grouting was done in the right abutment from adits RGA-3, 4, 5 and 6 along the line of the main grout curtain and from adits RGA-11 and 12 parallel to Tunnels 1 and 2. In the left abutment, the grouting was performed from adits LGA-1, 4 & 5. The seepage and piezometric data from the abutments should be analysed. Over the years, new voids may be opened in the abutments from the gradual deterioraion of the original grout lines, internal migration of fines or gypsum dissolution (in the rights abutment). Drain holes which discharged fines were plugged and replaced by the drain holdes fitted with sandwiched filters.

During 1982 and 1983, following a large rock slide, the high cut slope bordering the outlet area was stabilized by excavaton, drainage, and application of shotcrete.

Slope Maintenance: The upstream slope protection consits of heavy stone riprap. It is generaly durable and in very good condition. There is no deterioration or weathering of stones. There are some minor depressions and disturbances as a result

of beaching from wave action. At instrumented sections MB of main dam and XB of auxiliary dam there are depressions.

The dowsnstream slope protction of cobbles and gravels is intact. Vegetation on the berms and drains is periodically removed.

Settlement: Rate of settlements after the end of construction as measured by surveys of crest points have decreased substantially with time. The maximum settlement of 3 ft has occurred at station 31. A cember of 3 ft was provided in the embankment for the settlement. Fig 10 shows settlement of main dam with time. It has practically stabilized. The upstream shell has settled slightly more than the downstream shell due to saturation. There has been no logitudinal crack on the embankment.

The Double Fluid Settlement Device was used for the first time at Tarbela for settlement measuring which furnishes information continously along the line. In the past, settlement measuring devices have given settlement data only at points, rather than along a line. Eleven loops installed at different levels in the embankment are still working. The movement accuracy is of the order of 0.1 ft.

Horizonatl Movements: Permannent horizontal movements towards downstream have virtually stopped. Current horizontal movement are elastic responses to loading and unloading due to reservoir filling and emptying respectively. Currently recorded movement cycle generally in upstream and downstream direction is of the order of 1 to 2 inches at crest. Permanent horizontal displacements range between 1 to 6 inches at the crest and between 4 and 10 inches at El. 1425. Toe displacements are negligible. The embankment performance from the point of view of movements is satisfactory.

Cracks on Abutments: On the right abutment there is near vertical rock escarpment under the embankment. in 1976 transvrse cracks appeared on the right abutment with max. opening of 2" inches. These cracks were monitored for a couple of years and then treated by filling with sand and sealed with bitumen at top. These cracks have again shown up however the opening is very small. Cracks appeared in 1976 in left abutment. Cracks were also observed on the left abutment during the 1st Periodic Inspection in 1986. Probably these cracks had developed earlier and were noticed on removal of debries. However these cracks did not open up further and diminished away.

RESERVOIR OPERATION: The basic principles for operating the reservoir are: to ensure the safety of the embankment dams against overtopping, to fill the reservoir each high run-off season in so far as possible, and to enhance flushing of sediment when reservoir in so far as possible, and to enhance flushing of sediment when

reservoir is at minimum pool to prolong the life of the reservoir. These requirements lead to the development of attached Rule curve-Q (Fig.11) for reservoir operation. In addition certain guide lines for operation of the outlet works have been established in order to maximise the safety of these works. The spillways are to be operated at minimum discharges to ensure proper flipping at the buckets. At higher discharges, releases are to be shared between the two spillways to reduce the load on a single plunge pool and at reservoir levels above El. 1533 to reduce earthquake loadings on gates of both spillways in so far as possible.

Tunnels 3, 4 and 5 are operated part time at full gate rather than full time at part gate. It is possible to operate the reservoir in a fashion to ameliorate sharp flood peaks. The reservoir can be allowed to rise temporarily above the Rule curve-Q filling line and then brought back to the rule curve after the flood peak has passed or the reservoir can be lowered in advance of flood peak on the basis of upstream flood warning information and flood peack ameliorated by storage created be lowering.

The reservoir is not lowered below El. 1300 ft. the minimum pool level needed for the required net head on the turbines. Rapid or frequent variations in the downstream flow are avoided. Water is possible released through the turbines for generating power. The irrigation tunnels are not operated at reservoir level above El. 1505 except in emergency.

According to rule curve Q, the reservoir is lowered to El. 1300 by 20 May of each year. The reservoir is held at El. 1300 until 20 June unless inflows exceed the low level outlet cpacity and after that allowed to fill to El.1505. Above El.1505 the reservoir is filled at a rate of 1 ft. per day in so far as permitted by inflows and irrigation demands. Draw down of the reservoir is in accordance with the integrated system irrigation demands balanced against the amount of water available from inflows plus storage.

Sedimentation: Tarbela Dam Projct had a gross storage of 11.3 MAF and live storage of 8.9 MAF. During the 13 years of operation, the average annual sediment load deposited in the reservoir has been of the order of 216 million short tons (MST). Storage capacity of about 1 MAF has been lost. A delta of sediment deposits has formed in the lake which at its maximum is about 120 ft high above the river bed. Its pivot point (foreset) is about nine miles upstream of the main dam and is moving at an average rate of half a mile per year. The expected life i.e, depletion of storage is about 70 years. After that 1 MAF would be available continously.

Testing of sediments have proved that the delta is liquifiable, after a severe earthquake. In case the delta is liquified, it can clog the tunnels and deprive the nation of electric power already in short supply. Restoration of power by unclogging of power tunnels may take more than a month.

Under the above circumstances it is necessary that the delta should be kept at a safe distance from the dam which is estimated to be about five miles. The first immediate requirement to halt the movement of delta is to raise the minimum pool level gradually.

KHANPUR DAM PROJECT: Khanpur Project comprises of: Mian Dam 1545 ft long earth and rockfill embankment, 167 ft high with crest at 1992; Check Dam 622 ft long and 70 ft high; Right Saddle homogeneous embankment 2324 ft long and 54 ft high; Left Saddle 1842 ft long and 47 ft high and Malkanwala Dyke 1555 ft long and 15 ft high. The spillway has a capacity of 166000 cusecs with its crest at El. 1947, crest length is 232 ft and has five radial gates each 40 ft x 35 ft high. The outlet 6 ft dia tunnel under the right saddle with cill level at El. 1902 has a capacity of 650 cusecs. Initially the main dam was homogeneous embankemnt on the upstream side but later on during the constructon gravel shell was provided on the upstream in the closure section and above El. 1947 in the prebuilt section.

The dam was completed in 1983. The main dam is founded on alluvial deposits comprising boulders gravels and sand about 150 ft deep. Instead of a positive cut off, an upstream impervious blanket about 3200 ft long was provided. The abutments of main dam comprise of folded limestones. Fig. 12 and 13 show plan and cros section of the dam. Soon after the closure and start of impounding of reservoir, seepage and sand boiling was observed at the downstream toe of the main dam. A weighted toe berm was provided and relief wells were installed at the downstream toe of the dam.

Due to saturation at downstream of saddles, relief wells were provided on downstream both on the left and right saddles. Weighted berm was also added on the downstream of the right saddle. Extensive dental conncreting of all the exposed rocks in the reservoir, which were suspected to be water carriers, was done. Impervious blanket was extended on the upstream of the left saddle. Blanketing and dental concreting was also done on the rocks in the approach channel and its extension of the spillway.

In order to control the seepage and the piezo pressures in the left abutment, grout curtain was extended and strengthened from the diversion tunnel just downstream of the plug and from the surface. The drainage adit and the drainange holes were improved. Drainage holes were also drilled from the diversion tunnel downstream of the drainage adit.

Khanpur Dam Project is under the supervision of a Project Director (SE) and two Executive Engineers: one is responsible for the operation and maintenance of the dam and the spillway while the other looks after the surveillance work.

SURVEILLANCE OF CRACK ON D/S SLOPE OF MAIN DAM: A logitudinal crack on downstream slope of main dam at elevation 1972.7 ft. between RD 1050 and 1150 was observed in 1985. The crack was backfilled and is being monitored by extensometer. The extensometer does not indicate any further movement. There is no physical indication of its opening.

PIEZO PRESSURES & SEEPAGE OBSERVATIONS: Comparison of piezometric pressures of upstream blanket at similar reservoir levels indicates that the pressures are lower than that of 1984. There has been reduction of 3 to 15 ft. The downstream toe piezometers are nearly consistent except those near the left abutment (closure section) which have risen 1 to 3 ft. There is slight rise in the left abutment piezometers also.

Seepage is measured separately for the main dam toe (collective seepage from the toe drain and the relief wells), from the plunge pool when spillway is not working, from the diversion tunnel including that of the drainage adit, from the right saddle, from the left saddle and the left rim seepage. The seepage measured at various points at the same reservoir level has decreased with respect to the previous years which shows the effect of sedimentation and the various remedial measures taken. Seepage at the downstream portal of diversion tunnel has decreased substantially following the strengthening of the grout curtain in the left abutment. In addition to the routine readings of instruments, the instruments are checked for their reliability, soundings are taken whether the depth of the stand pipes is clear, relief wells are checked with CCT Camera to know that they are not choked and working all right.

SURVEILLANCE OF TURBID FLOWS FROM RELIEF WELLS: Tubid flows were observed from four relief wells at the downstream toe of main dam on 4th August, 1986. The burbid flows continued for more than four days and considerable amount of fines were removed. The turbid flows became clear when the relief wells were closed with sand and gravels. The problems has repeated since then each time associated with a storm in the Haro Catchment and sudden rise of reservoir. The following table gives data about the eleven turbidity events. Sonar survey and all efforts to detect the source and path of the turbid flows have not been successful so far. The relief wells have been filled with sand gravels. In order to investigate the pat of the turbid flow, 24 holes have been drilled at different locations and installed with 3" dia perforated (slotted) PVC pipes. These stand pipes permit to sample water and would be helpful to delineate the passage of turbid flows.

TABLE EVENTS OF TURBID FLOW FORM RELIEF WELLS

Event No.	Date	Reservoir Level	Reservoir Rise (Ft.)	Duration (Hrs)	Max. P.P.M.	Flood Peak (Cusecs)	Loss of Fines (In Cfts) Observed at D/S of MED.
1sr	04.08.86	1938	12	105	6% (By Vol.)	-	50,000
2nd	25.08.87	1915	5	20	1,000	-	8
3rd	11.10.87	1928	8	52	10,200	3,000	150
4th	11.03.88	1922	14	16.5	8,500	23,300	248
5th	04.0788	1919	7.29	110	30,000	9,350	8,260
6th	11.07.88	1926	2.0	18	3,900	8,800	618
7th	16.07.88	1940	7.26	76	16,000	8,000	3,308
8th	31.07.88	1953	3.45 (Spillway Flowing)	140	14,600	17,400	13,120
9th	30.07.89	1915	2.72	10	1,277	1,914	1,979
10th	31.0.7.89	1920	3.05	11	2,842	1,920	3,606
11th	31.10.89	1936	4.00	14	3,349	10,000	17,126

MAINTENANCE OF DENTAL CONCRETE: Dental concrete provided on the rock exposures in the reservoir deteriorates with time. It is replenished and strengthed when the rocks are exposed.

MAINTENANCE OF SLOPES: The rip rap is inspected on each draw down and replenished/replaced where required. The upstream rip rap is withstanding all right the wave action. In the closure section of main dam the rip rap is dumped and does not give a uniform slope. The rip rap of check dam and saddle embankemnts is hand packed and gives a uniform slope although the wave run up is more on the smooth surface. The downstream slopes have boulder cobble gravel protection against rain erosion. Vegetation is periodically removed from the downstream slopes and drains.

RESERVOIR OPERATION: Khanpur dam has crest level 1992, max. conservation level 1982 and can pass the max. flood through the spillway, crest level 1947, with some free board. The out let has a max. capacity of 650 cusecs and has invert level 1902. Since its first impounding. in 1983, the reservoir attained the following levels 1950, 1956 and 1966 in 1983, 84 and 1985 respectively. Fig 13 shows reservoir and seepage plots for 1986 to 89. Irrigation water supply was started to both the Punjab and NWFP provinces but the municipal water supplies to Rawalpindi and Islamabad have not yet started because of lack of water supply structures by CDA. During 1986 due to some problem it was decided to keep the spillway gates open and not to impound above it so as not to overstress the dam foundation. About 8 to 10 ft of silt has been deposited at the upstream end of the blanket. Proper upstream gauges and

flood warning system is required for safe operation and passing of max. floods as the downstream area is populated and has important industry.

OPERATION OF OUTLET: The outlet structure can be controlled by fixed wheel bulkhead gate installed in the middle of the conduit at the crest of the saddle embankment and the radial gate at the downstream end of the outlet. On the very first operation of the gate there were some problems. Since then the discharge has been controlled from the downstream gate. The fixed wheel gate is always in the open position and closed only for inspection or repair of the outlet structure. There was some minor damage to steel lining and erosion downstream of the gate which was repaired with acqua plug and is working all right. The stilling basin downstream of the outlet gate was damaged which was repaired after closure.

HUB DAM PROJECT

Hub Dam is a homogeneous earth embankment, 15640 feet long and maximum height from the river bed is 152 feet. A saddle dam 5740 feet long with average height of 52 feet fills the gap of the valley on the right side. On the extreme right is the spillway, a free overflow concrete gravity weir 6020 feet wide. The Hub Dam Project has a capacity of 0.86 MAF. It supplies municipal water to Karachi city and irrigation water in Baluchistan province. The project was completed in 1979. The crest of the dam is at El.352. The foundation comprises of alternating beds of sandstone, silstaones and shales. The bedding at the dam site is almost horizontal with a 6° to 10° dip to downstream. The main dam is provided with a core trench which penetrate about 5 feet in the rock. The shoulders of both the embankments were founded on stripped foundation. As the dam is built up of silty sand (noncohesive, easily erodable) and without a chimney filter, its stability can be endangered by the appearance of phreatic line at a higher level than the horizontal blanket on the downstream slope. Therefore a gravel shell was added on the downstream slope to ensure its safety. Fig. 13 and 14 show plan and cross section of dam. The project is headed by an SE. There are two Executive Engineers: one is responsible for the operation and maintenance of the dam and the spillway while the other looks after the surveillance and the instrumentation.

Remedial Works for High D/S Pressure: On the very first impounding of reservoir in 1979 (rise only 28 ft), notable pressures were observed at the downstream toe of the main dam and in the alluvial foundation reach of saddle dam. In 1982 reservoir level rose to 306.75 and the downstream piezometer pressures responded. The pressure were 70% of the reservoir rise. At certain areas these were higher than the NSL. The areas became waterlogged. It was extrapolated that at full reservoir level, the uplift pressures at the toe of the dam would be high enough to endanger the stability of the dam.

The following remedial measures were taken :-

- Relief wells were installed along the toe of main dam and saddle dam;

- Grout curtain was attempted in the foundation rock under the completed embankment but it did not ake much grout;

- Section of the burried channel under the saddle dam foundation was loaded/blanketed near the downstream toe by a thick layer of coarse gravels;

- Observational approach was followed involving review of piezometric behaviour by phased filling of the reservoir. For that matter a reservoir level control channel was excavated across the gravel terrace with invert level at El. 305 and with a provision to raise the plug as desired.

The remedial measures proved effective. The reservoir was filled up to max. level by sudden flood equal PMF in 1984. the embankment and all the structures withstood safely. In 1989 the reservoir was again filled to max. level. Although the pressures in some areas downstream of the dam are higher than NSL, but there is no danger to the stability of the dam. The pressures are either perched or in deeper beds and are gradually dissipating. The relief wells are working.

Surveillance of Cracks on Main Dam Crest: Longitudinal cracks appeared on the crest of the dam near the downstream edge from Ch.42 to 49 during December, 1983. The cracks were monitored for more than two years and then sealed on top with a thin concrete pad. It did not develop any moe. Longitudinal cracks also appeared at Ch.112+40 to 115+40 & Ch. 147+50 to154+30 during August, 1988 and December, 1987 respectively. Three pits were made to know the extent of cracks. These cracks are still being monitored. No further distress has been observed in the crack areas. Reservoir loading on saturation of embankment and foundation might have caused the cracks. The cracks have closing trend with the depletion of the reservoir.

RESERVOIR OPERATION

First impounding of the reservoir commenced in 1978/1979. The reservoir level time history plots for six years are shown in Fig.16. The only waterway of the project requiring operation is the irrigation sluice. The discharge of this slouice is regulated as per downstream requirements and it is capable of discharging peack demand of 370 cfs at full pool level of 339 ft. SPD with 1.52 ft. gate opening. The other waterway is a long ungated overflow spillway (crest level 339 ft) which will come into operation automatically when the inflow flood cannot be absorbed in the reservoir upto its normal conservation of 339 ft. SPD. Hub reservoir has a storage capacity of 0.76 MAF which is more than twice the average annual river inflows. Historic river inflow data

show that the frequency of high floods is low so the spillway flow is infrequent. Therefore in the 12 years of reservoir operation the spillway has operated only twice in the year 1984 and 1989 respectively. As a result minor downstream damages were observed which were subsequently repaired.

Operation and Maintenance of Outlet: The water rlease through the conduit have been controlled since its commissioning in 1981, by operating the fixed wheel gate in the middle. On 17 May 1985 the steel lining in the conduit just downstream of the operational gate was damaged. It rolled up after shearing from the concrete surface and caused obstruction inside the conduit. The gate was closed. The suspension of water supply to Karachi, due to this happening, caused a crisis in the city. Rehabilitation of the lining as per desgin was not found practicable mainly due to the problem of the closure time required to carry out the repairs.

WAPDA carried out the repair by removing all the loose and damaged lining and repairing the surface of the concrete where necessary and reopened the conduit as soon as practicable i.e. on 26 May 1985. Since then the steel lining exists at the crown and along the sides above the mid height for about half the lenght at the downstream end of the conduit. Elsewhere it has been totally removed. At the time of re-commissioning it was decided that frequent inspections of the conduit should be done during brief closures to ascertain the condition of concrete. thus the conduit has been operated, since May 1985 without the steel lining. During this period the reservoir has remained close to the maximum conservation level. There has been another operational practice undertaken in this period. During the crisis, a temporary syphon (22 inches dia pipe), discharging into the stilling basin was erected to restore partial water supply to Karachi. It was an uncontrolled syphon without any valve at the inlet or the outlet. Because of the difficulties of priming associated with this syphon it was kept working, even after reopening of the sluice gate as a standy arrnagement in the event of any possible crisis, because the lining was now removed and concrete was exposed to high velocity flow.

With the operation of the syphon, however, the gate opening of the sluice was redued and during this period of high pool levels and continued syphon operation the gate openings remained 4 in. to 8 in. only, resulting in high flow intensities.

WARSAK DAM PROJECT

Warsak Hydroelectric Project was completed in 1960. It is a concrete gravity dam 76m high and 180m long with overflow spilway. The spillway has taintor gates 40 ft x 40 ft high. The power house is located on the right bank. Initially four generators of 40 MW each were installed. Later on two more sets were added in 1981. Power House building comprises of two main floors at El. 1132 and El.1149 SPD. The project is headed by a Chief Engineer, two S.Es for operation of the power house and one XEN

(Civil) to look after the civil works are opening up of cracks and joints, misalignment of a turbine shaft and wear and tear of all turbines due to silty sand.

In order to control the seepage beneath the dam a high pressure grout curtain at elevation 850 ft SPD was made before construction of dam. Two additional curtains at low pressures were provided inclined upstream and downstream from the main grout curtain. The uplift pressures under the dam are released by drainage holes drilled 40 ft centre to centre from the inspection gallery of the dam. Observations of discharges and pressures in theese drain holes are regularly taken on fortnightly basis by the monitoring crew. Fig.17 and 18 show plan and cross section of the Project.

RESERVOIR OPERATION: About 5 years after the commissioning of the project, the reservoir was silted upto the crest of the spillway. The design had envisaged that the sediments would pass through the spillway and a baffle wall was constructed in front of the intake structure. Due to the sharp bend in the river upstream of the dam, the flow is concentrated on the right bank. The sediment level at the wall has reached El. 1240. The intake to the power house is heavily silt ladden which produces the problem of wear and tear of the moving parts of the turbines and the valves. There is problem of cleaning of trash rack also.

The design criteria for the Warsak Project included provision for the ultimate complete sedimentation of the reservoir to spillway crest level (El. 1230 ft). Protection of the power intake structure (semi-circular in plan) was provided in the form of a curved concrete silt baffle wall, with its crest at El. 1240 feet, situated 80 ft upstream of the intake structure. It was apparently presumed that during high flow periods, the bed load sediments would be evacuated through the spillway and that water containing only suspended sediments would pass over the baffle wall and enter the intake structure. However, the configuration of the reservoir is such that, under high flows, the main flow channel is concentrated along the right bank (on the outside of the curve) near the dam so that the direction of the flow of heavy bed load sediments is towards the intake structure before reaching the spillway. The sediments have long since overtopped the baffle wall and have burried the two lower since overtopped the baffle wall and have burried the two lower sections of the trash racks. Model studies carried out by WPADA indicate that as much as 80% of the total bed load sediments pass through the power intake structure at the present time.

In July 1988 the surface of the reservoir sediment deposit at the silt baffle was at El. 1240 ft, only 5 feet below the top of the trash racks, and adjacent to the intake inside this wall, the deposits covered the lower two sections (22.5 ft) of the trash racks. Unit 4 had been stopped temporarily to reduce the intake flow velocities to permit the removal, for cleaning, of the upper trash rack sections which were matted with weeds and silt. The high concentration of bed load sediments in the power house flow is

evidenced by the severe erosion of turbine components, difficulties of operation of the butterfly valves following a unit shutdown, extensive and rapid wear of the turbine shaft seals and the transport of sand-sized material from the tailrace and the draft tubes through the cracked concrete of the power house substructure into the turbine floor via the open longitudinal joint downstream of the units. The chute of spillway is eroded and sand balsted. The concrete aggregates are exposed over the entire surface. However the erosion is not deeper and no reinforcing bar is exposed.

SPILLWAY - STILLING BASIN REPAIRS: To reduce uplift pressures, drains have been provided below the stilling basin floor, these drains connect with the drainage galleries in the basin wing wells. The drainage galleries are at elevation 1120 ft SPD and get submerged. Outlets of these drain holes have been silted up, which need to be raised.

In 1975 the underwater scanning of stilling basin was caried out. Some damages on lower part of the chute and floor of stilling basin were noticed and afterwards confirmed by diver inspection who reported severe damages along the toe of the sloping side wall and floor of the stilling basin. From model study on 1/36 scale complete model of spillway, it was observed that the flow jet did not follow the profile of spillway chute at junction of ogee section with 1:4 slope. It was also noted that flow from extreme side bays were getting deflected by the slopping side walls projectng into the stilling basin and creating undesirable flow conditions along flanks. It was considered desirable to increase the radius of the curve joining the ogee profile with the 1:4 slope. A downstream coffer dam was costructed and repairs/modifications were carried out during December 1978. For monitoring the safety of repaired floor, wire loop indicators were embeded under the floor of stilling basin. It was observed that all of these loop indicators were damaged by the end of 1981 indicating a damaged floor. The coffer dam raised during the said repairs was not removed and it is considered that continuous churning of stones in the stilling basin has damaged the floor. At present a lot of debris is lying on the floor of the stilling basin.

OPERATION AND MAINTENANCE OF POWER HOUSE: A couple of years after commissioning of the four units in 1960, the power house showed signs of what was initialy considered to be differential settlement in the form of opening of construction joints and cracks that appeared in the floor and walls. DMO Wapds started monitoring the cracks. Canadian Consultants M/S Golders Associates were associated from 1984-86 who installed additional instruments. They concluded that cracks and distortions in the power house were the results of alkali-aggregate reaction. Most of the movements are cylic i.e. opening and closing with the season but there is a small residual movement which is continuing between elevation 1125 and 1150. Due to these movements joints are opening and beams, slabs and floors are cracking. There is misalignment of turbine shafts.

There were difficulties in the operation of downdstream gate of draft tube No.1, since late 7OS. In 1986 a cellular coffer dam was erected at the outlet of tailrace channel. For the first time the power house was dewatered and physical inspections upto + 1085 ft SPD and below was carried out. The floor at 1085 was quite intact and no movement signs were observed. Severe sand blasting effect was noted on the walls of draft tube. There was opening of the downstream joint at its erosion. The gate railing had moved out more than 1.50 inch and there was offset in the concrete also. Proper alignment of the gate was restored.

During 1988 movements and an offset of 1 was noted in the penstock of units 1 & 2 and scroll casing joint. The pedestal of penstock was damaged at floor El. 1105. The penstocks are under high compression. It has been decided to provide flexible joint between high compression. It has been decided to provide flexible joint between penstock and scroll casing for units 1 to 4. Penstocks of unit 5 which has flexible joint is experiencing leakage due to movements. In 1989 efforts were made during a partial shut down to inspect the scroll case as well as penstock but due to defective, closure of butterfly valve no such inspection was possible.

There is problem of alignemnt of turbine shaft of unit No.1 and 3. Wapda attempted to restore the centre of turbine to the best centre of stator core by displacing the turbine bottom ring and head cover. Such displacements have attainced valves as high as 1/4 inches.

As the power house is running on the run of the river flows, about 80% of the sediment passes through the power house. There is a lot of wear and tear of the moving parts of the turbine. The silt entry into power tunnels possess serious and expensive maitenance problems of warsak power station. One unit out of the six is out for major overhaul every year.

There is leakage from the roof of the power house from the joints. The bitumen filler has hardened and squeezed out due to weathering. Sinflex 4 has been applied successfully since the last three years as a test patch for a part of the joint.

MAINTENANCE OF CUT SLOPES: Rock slides during excavations for the power house called for protection works. Anchors, drains and berms were provided. A concrete retaining wall was constructed to arrest falling stones and protect the control building, from east side. In 1976 and 1979 rock slides occurred in the east slope remedial measures were taken. Occasional rock falls occur which are cleared.

REMOVAL OF TRASH: At present there is no arrangement to arrest the trash on th upstream side of spillway gates. Log boom originally installed and some 500 ft. upstream of the dam and repaired subsequently for arresting the trash was damaged

and dislodged during high flood. Now floating material reaches the gate bottom and gets stuck up, resultin gin leakages under the gates, damage to seals & cill and it is continuous problem to remove it.

CONCLUSIONS

The largest dams Mangla and Tarbela are very well maintained. They have experienced surveillance staff. These dams are among the best instrumented dams in the world. Instrument data of Tarbela adam has been computerized. Mangla is 23 years old while Tarbela is 16 years old. They were designed, constructed and are being maintained according to the latest state of the art and with the equipment available. Tarbela has a strong operation and maintenance organization. They have an efficient and capable task force handling the situation. They are inspected by experienced and qualified DMO team annually. Intenational expert advice is availed for any problem. Periodic Inspections are carried out by independent expert professional engineers. Follow up action on the recommendations is implemented carefully.

Funds to some extent are also available for the operation and maintenance of these dams. However TAMS observed in their first Periodic Inspection 1987 of Tarbela Dam that "the Water Wing operations and maintenance (O&M) organization at site is considered to have been underfounded the last two years. Considering the importance of the Tarbela Project to the national economy (Tarbela provides about 40 percent of the nation's powe generation and has the capability to provide about 9 million acre-ft of irrigation water), adequate funding for Tarbela O&M should receive the highest priority. Based on experience elsewhere, an appropriate funding level would be about one percent of the original project cost adjusted for inflation. On that basis, the Water Wing's portion of the Tarbela O&M budget for fiscal year 1986/1987 should have been about Rs. 176 million, about Rs. 44 million more than was actually provided."

Warsak is more than 30 years old. It has two major prblems: sediment problem and the cracks due to moveents due to alkali aggregate reactivity. Rehabilitation cost study has been carried out and it is expected to be under thaken with the Canadian assistance.

Hub and Khanpur dams are also important from the point of view of water supply. These dams do not get enough funds for their maintenance. However within the limited means available, these dams are being maintained by Wapda. They are also monitored by DMO Wapda and independent teams periodically. Priorities are fixed according to the necessity of dam safety or routine maintenance.

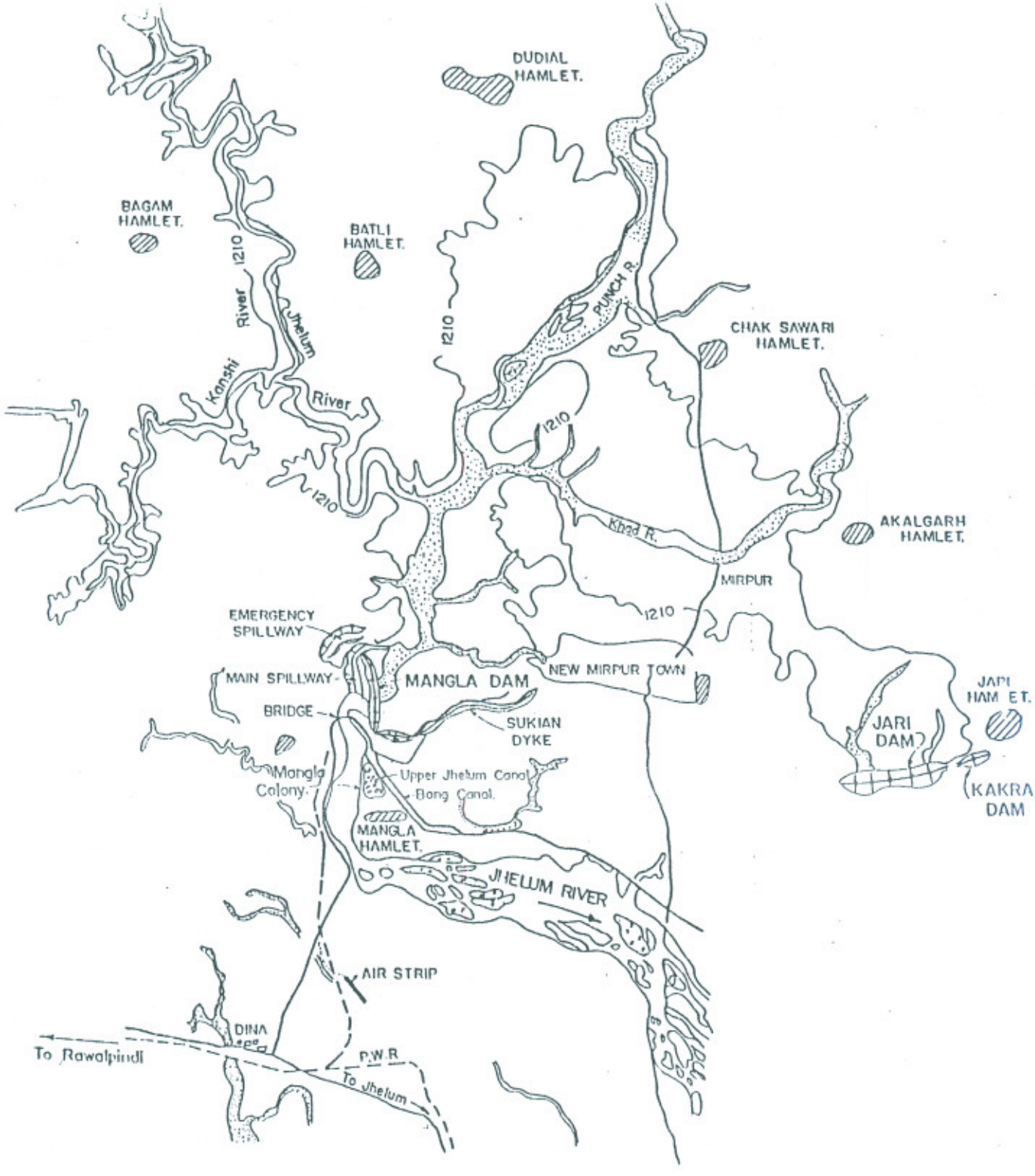


FIG.1 GENERAL PLAN OF MANGLA DAM

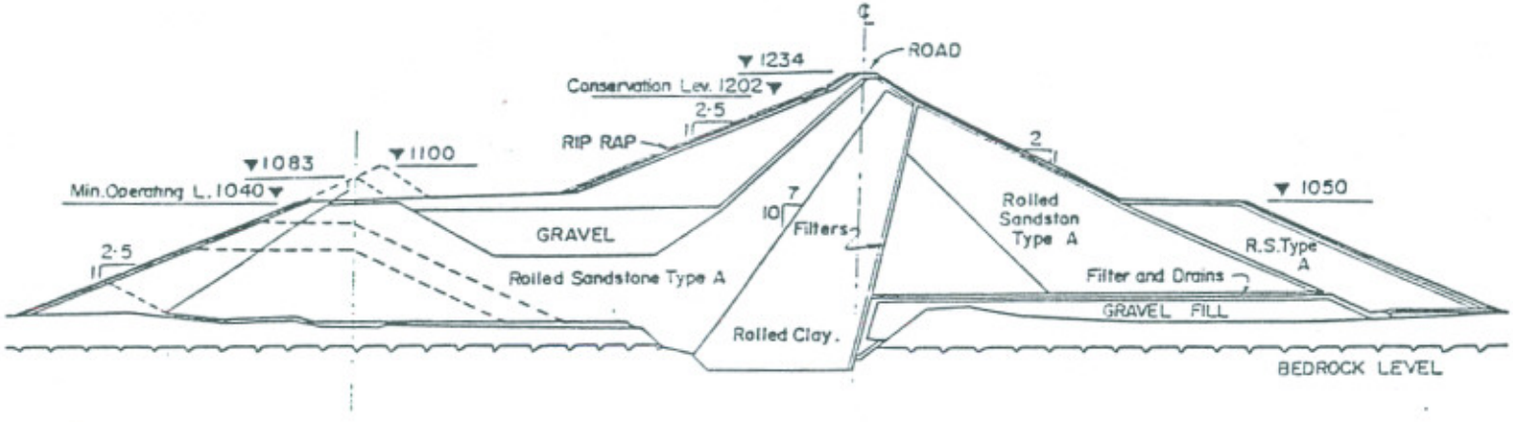


FIG.2 TYPICAL SECTION OF MAIN DAM

MANGLA DAM

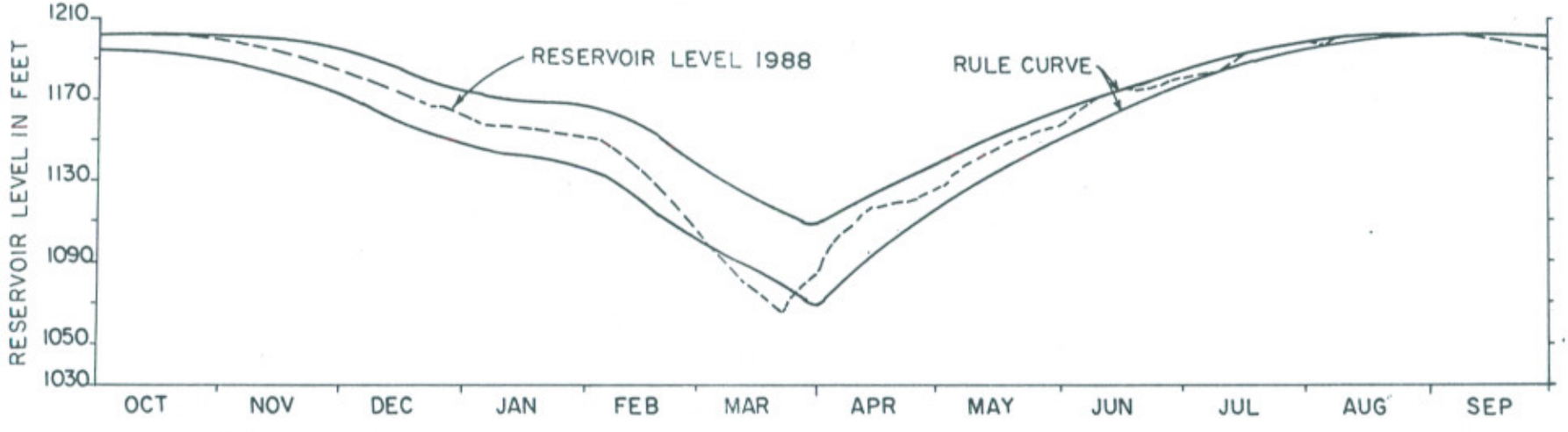


FIG.3 MANGLA DAM PROJECT

RESERVOIR OPERATION

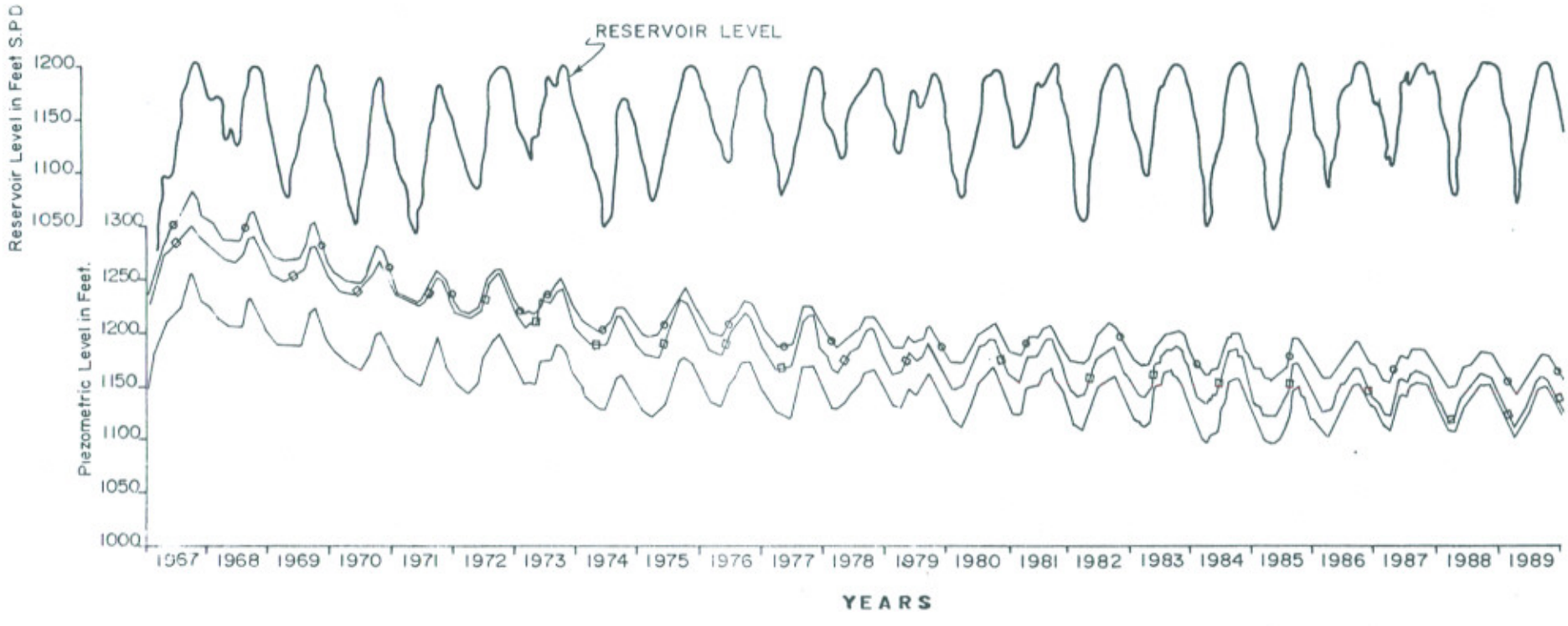


FIG.4 MANGLA DAM PROJECT

MAIN DAM

CORE PIEZOMETERS

LEGEND

PIEZO No.	SYMBOL
101	
102	
103	

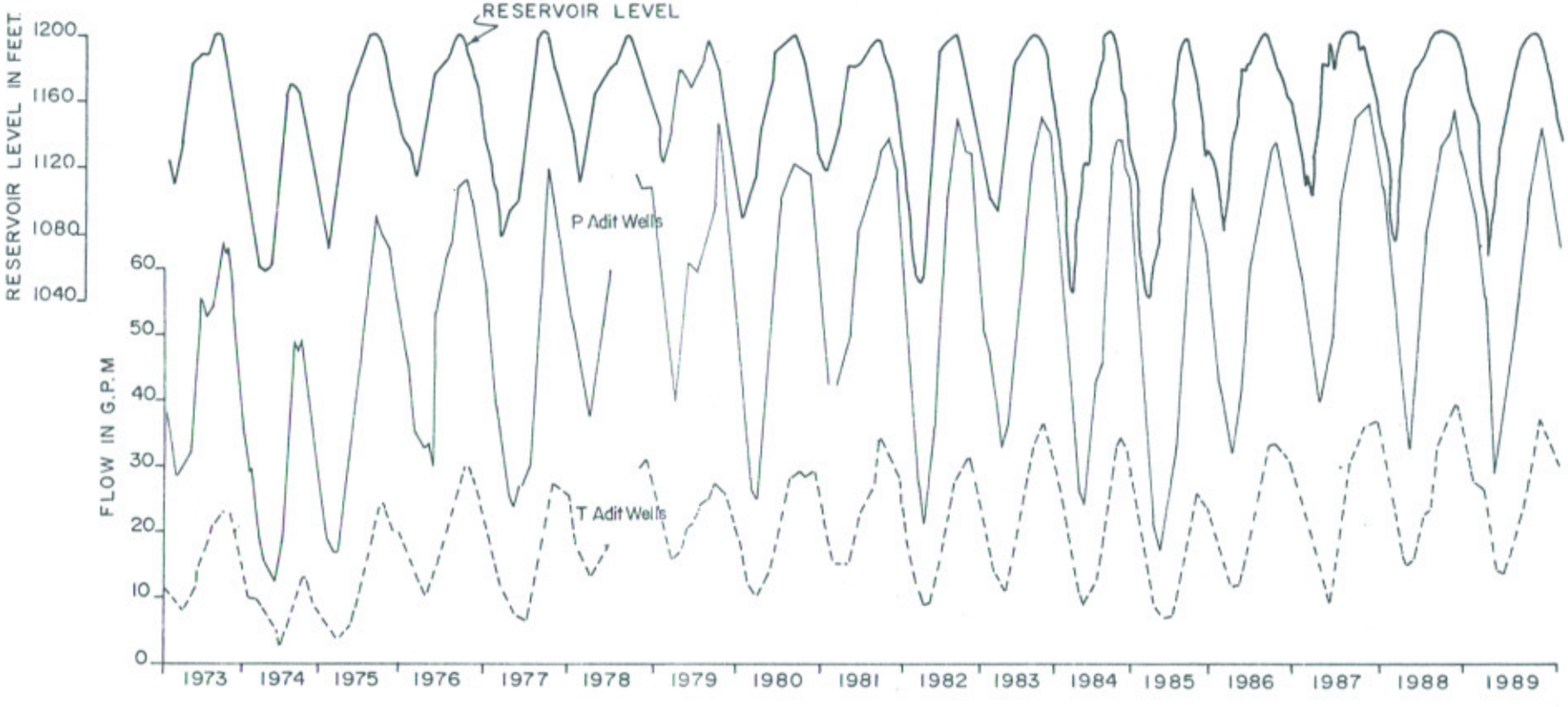


FIG.5 MANGLA DAM PROJECT

SEEPAGE P & T ADITS

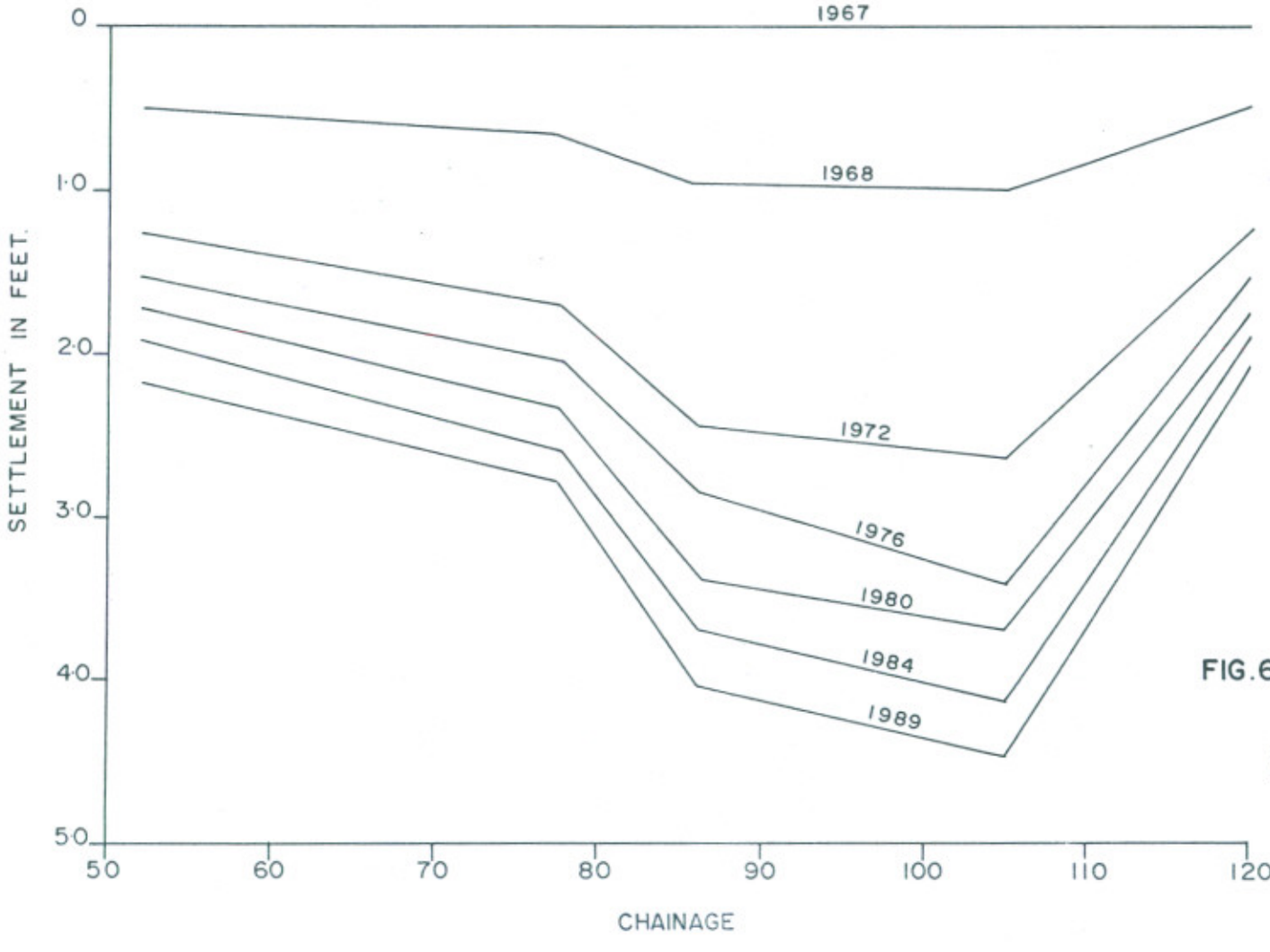


FIG.6 MANGLA DAM PROJECT

MAIN DAM

CREST SETTLEMENT

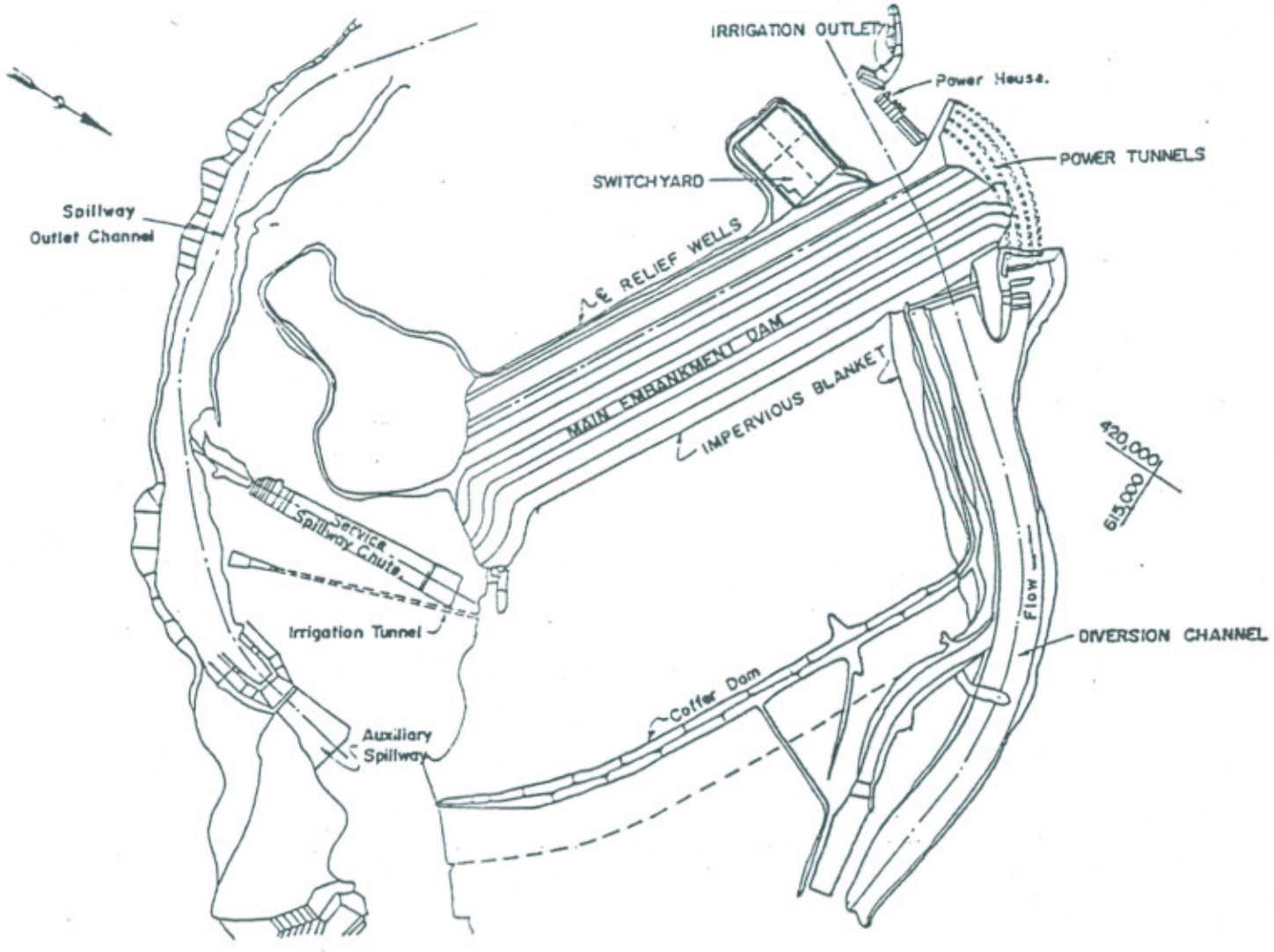


FIG.7 PLAN OF
TARBELA DAM

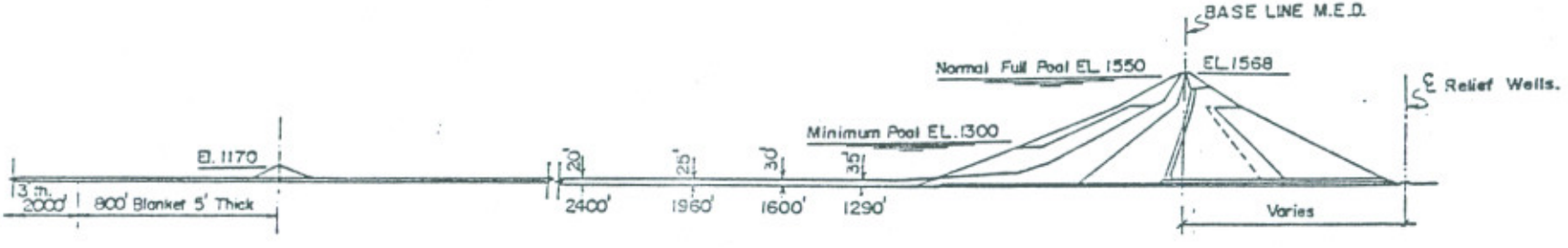


FIG. 8 TYPICAL SECTION

TARBELA DAM INCLUDING BLANKET

FIG.8A SIDE SCAN SONAR SURVEYS SINKHOLES IN MED U/S BLANKET

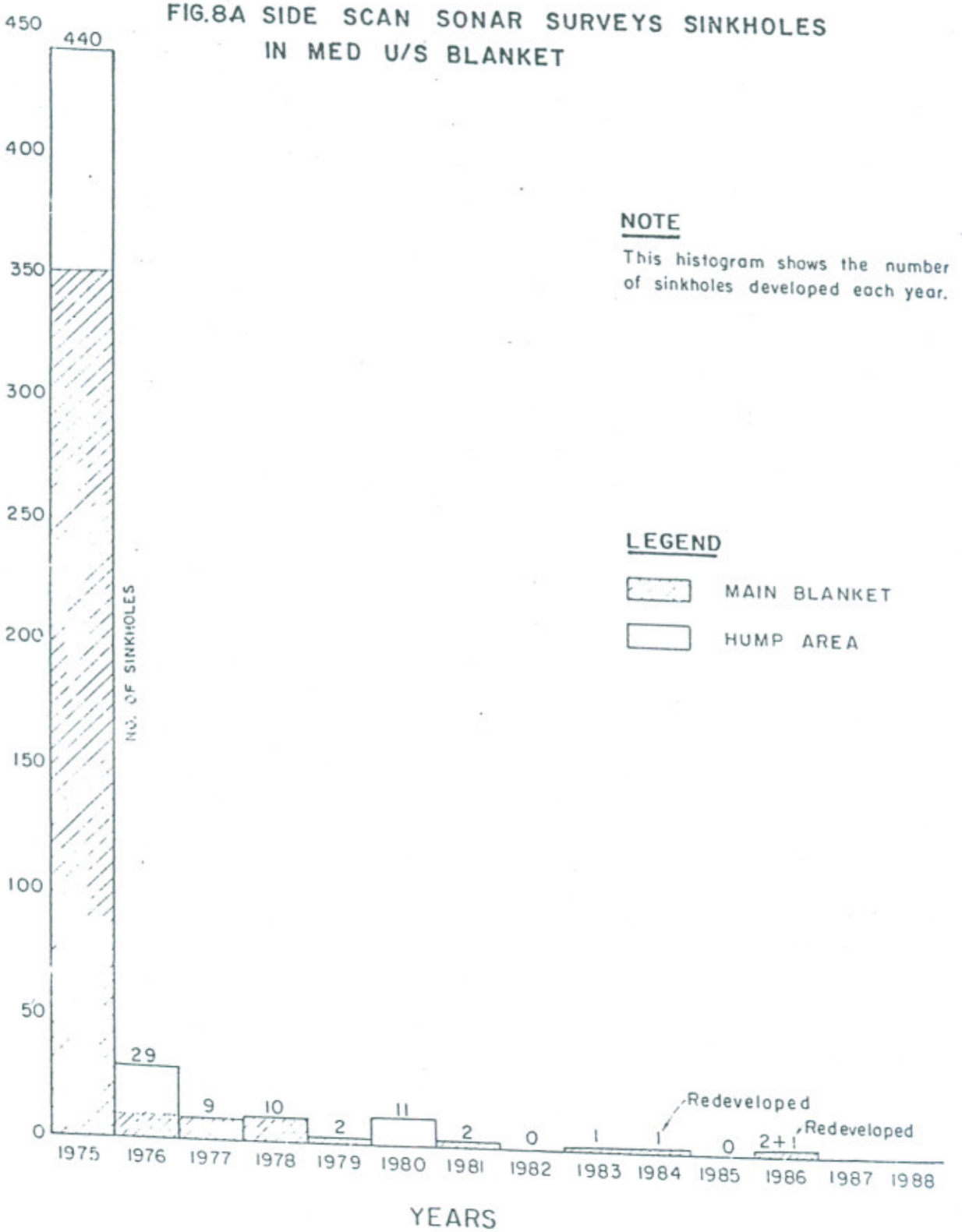
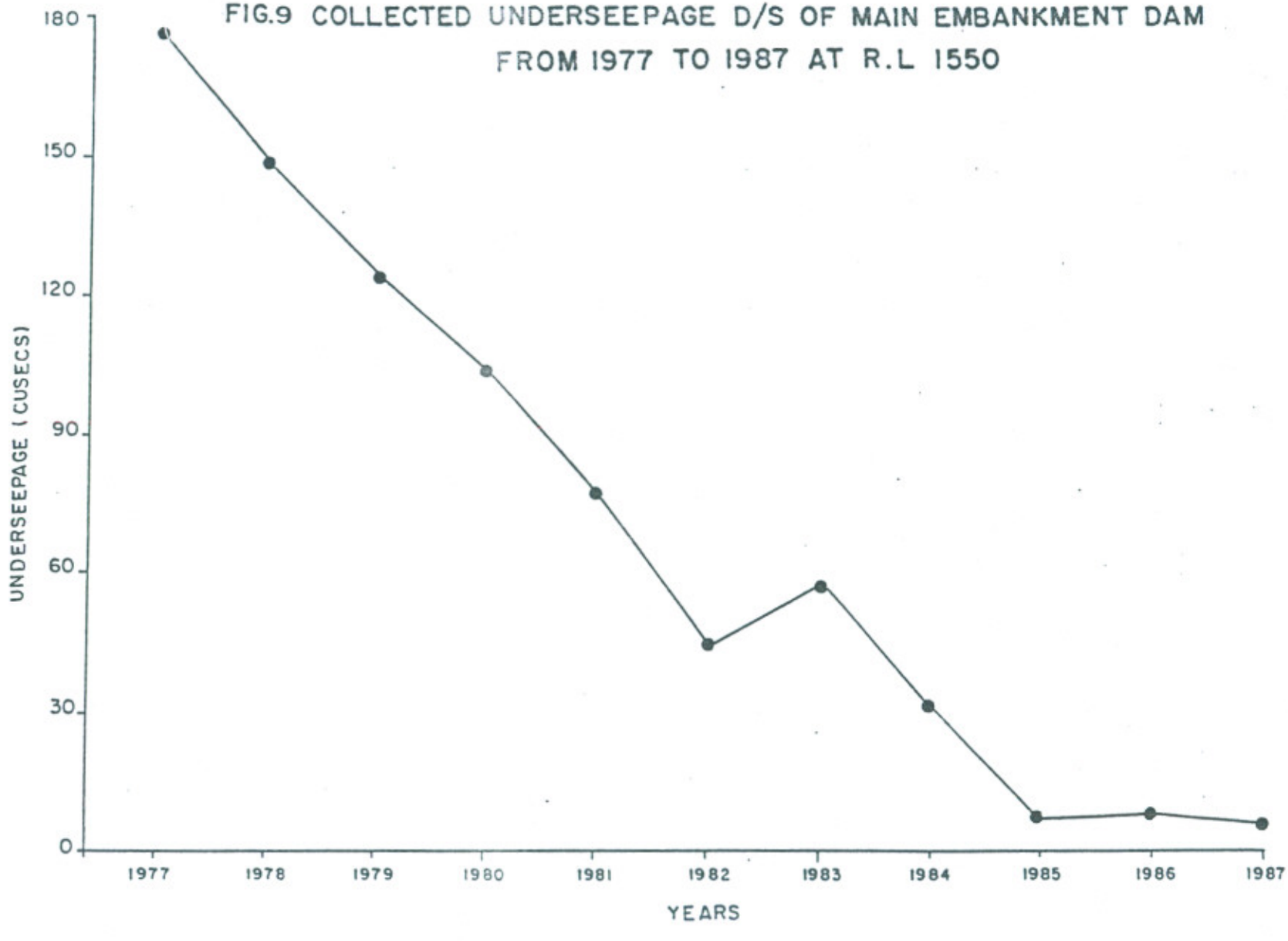


FIG.9 COLLECTED UNDERSEEPAGE D/S OF MAIN EMBANKMENT DAM FROM 1977 TO 1987 AT R.L 1550



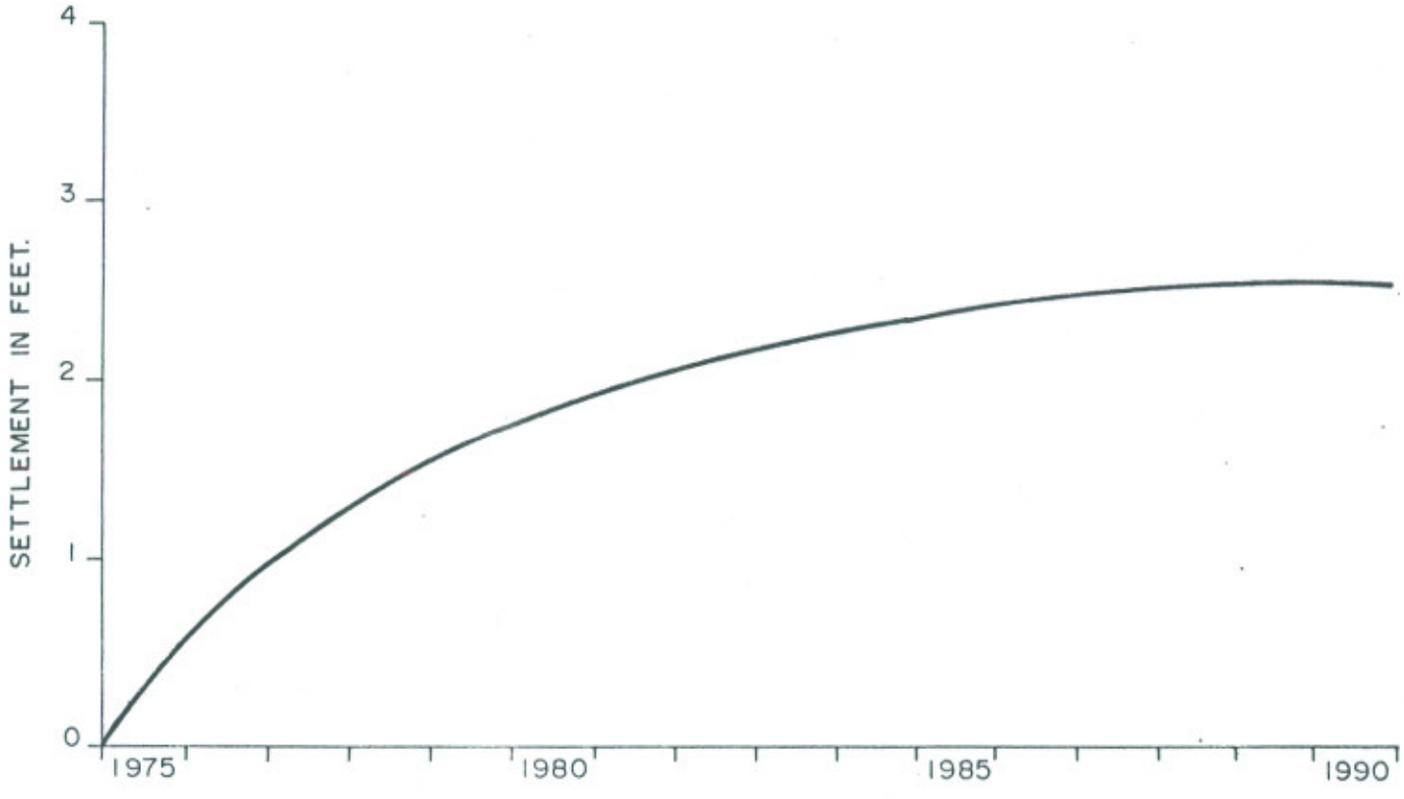
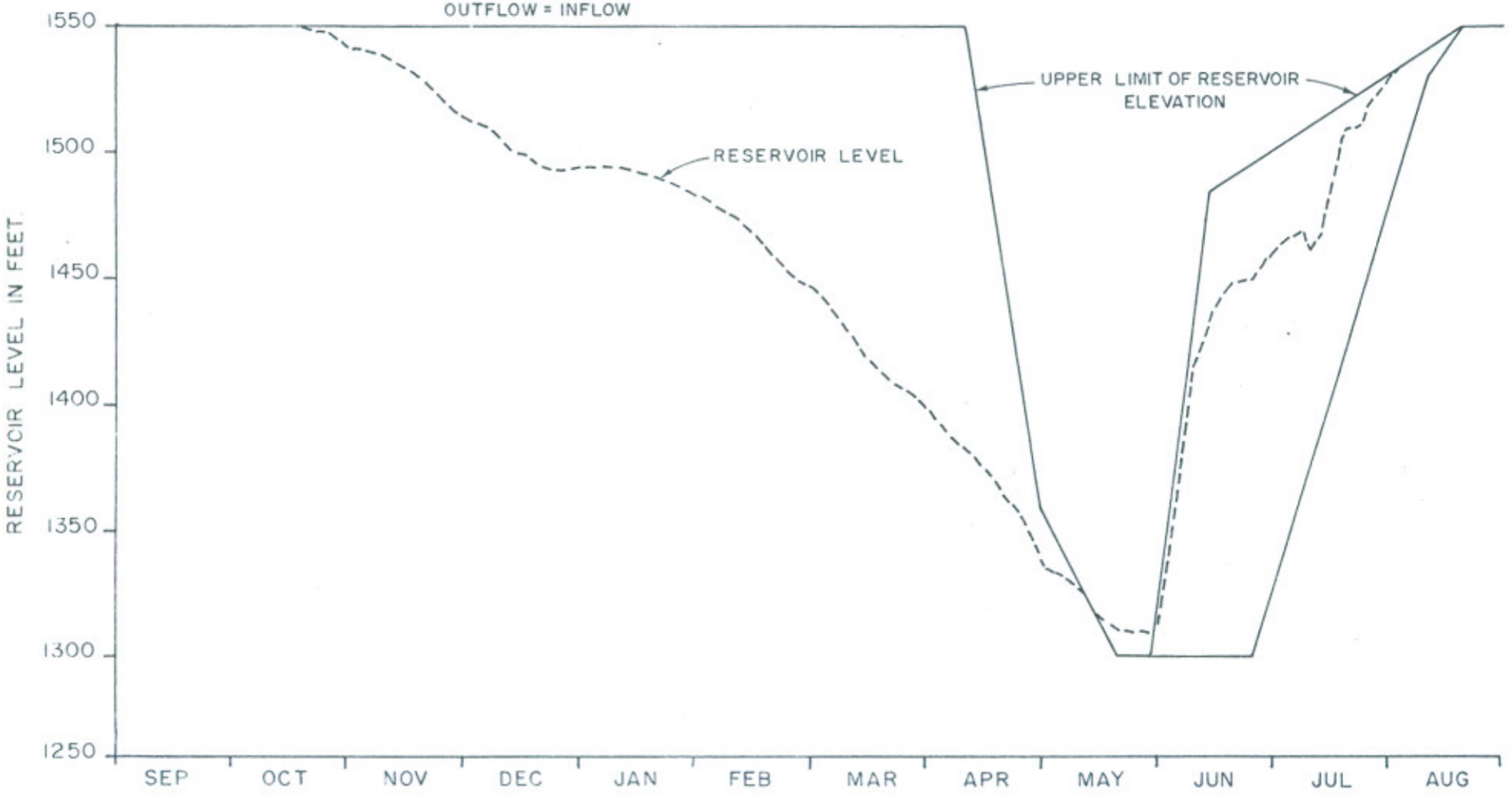


FIG.10 TARBELA DAM PROJECT

MAIN DAM SETTLEMENT

FIG.11 TARBELA DAM

RESERVOIR OPERATION



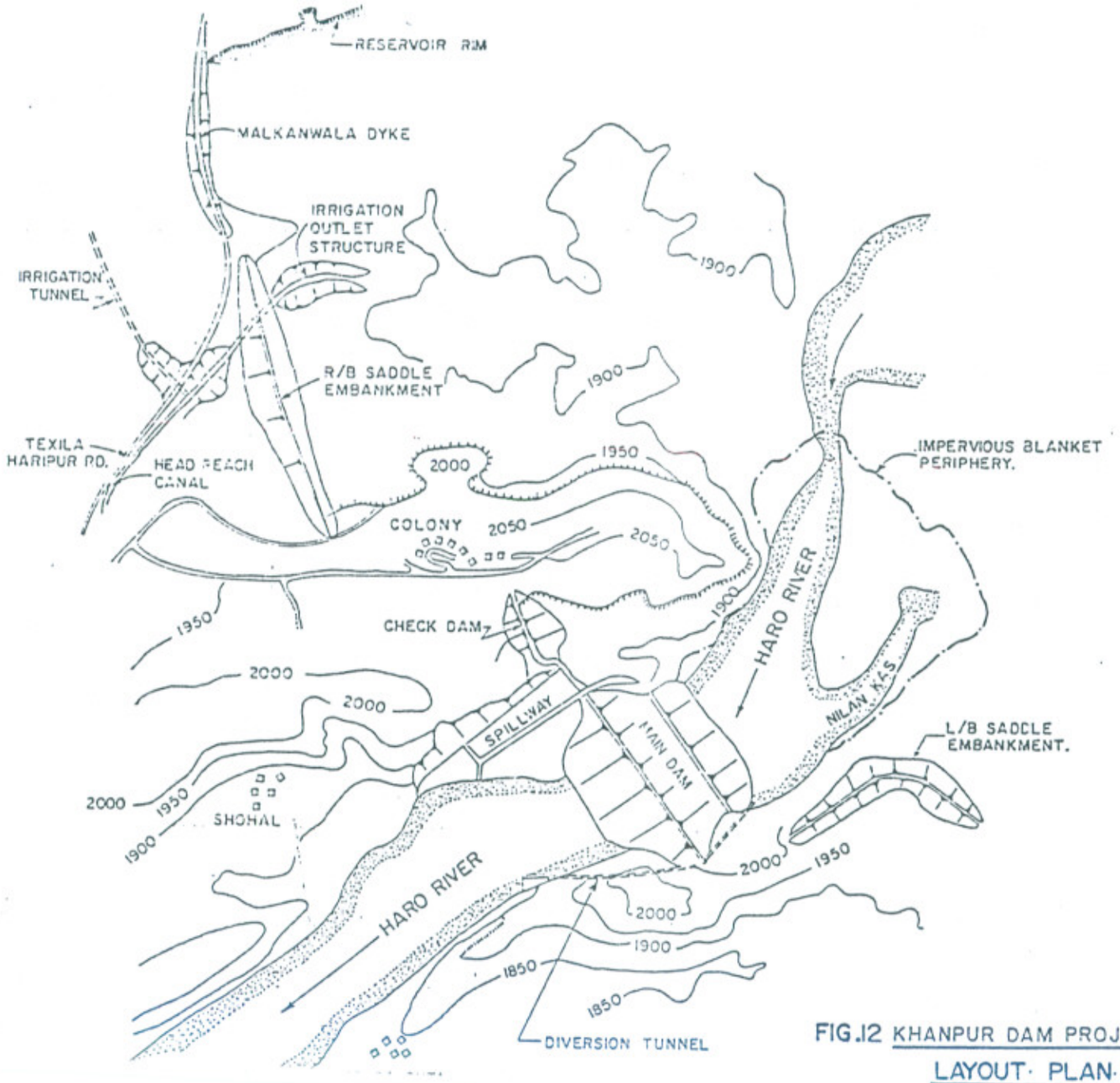


FIG.12 KHANPUR DAM PROJECT
LAYOUT· PLAN·

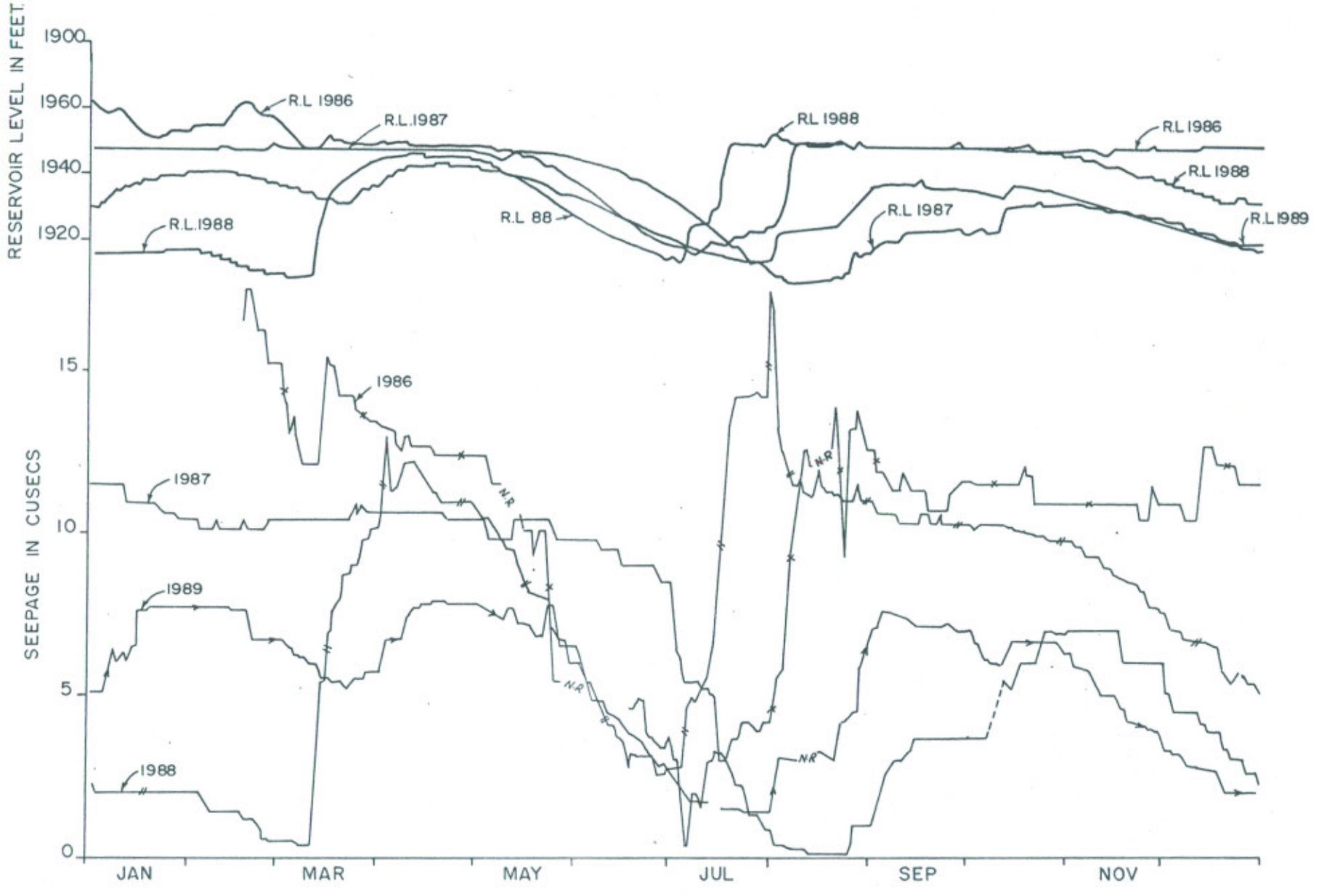


FIG.13 KHANPUR DAM PROJECT
RESERVOIR LEVEL & SEEPAGE
OF MAIN DAM FOUNDATION

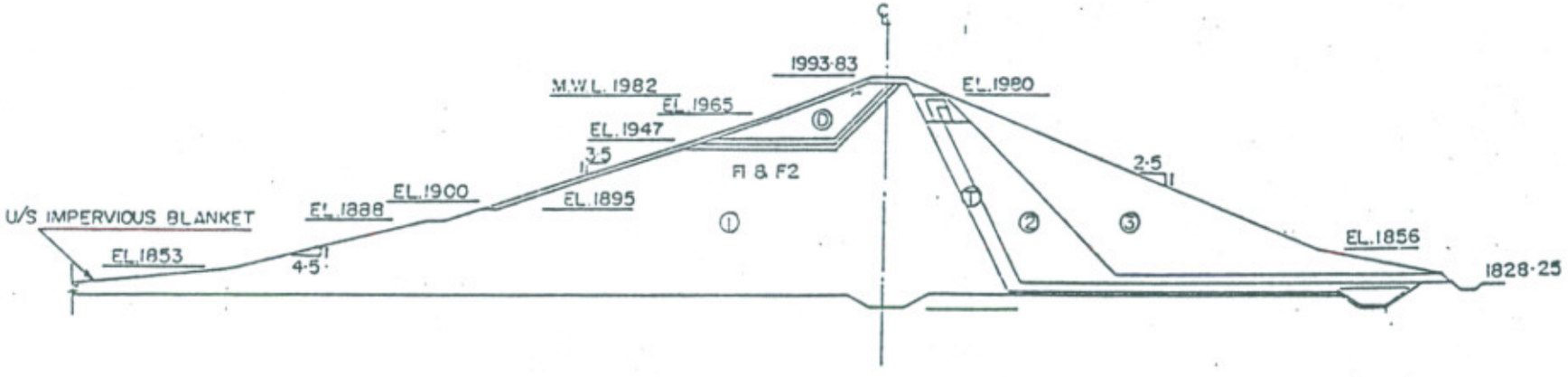


FIG. 13-A KHANPUR DAM PROJECT
TYPICAL CROSS SECTION

LEGEND

① ROLLED CLAYEY SILT
② GRAVEL FILL.
③ ROCK FILL.
Ⓣ GRADED FILTER.
Ⓓ FREE DRAINAGE FILL.
F1 FINE & COURSE FILTER & F2.

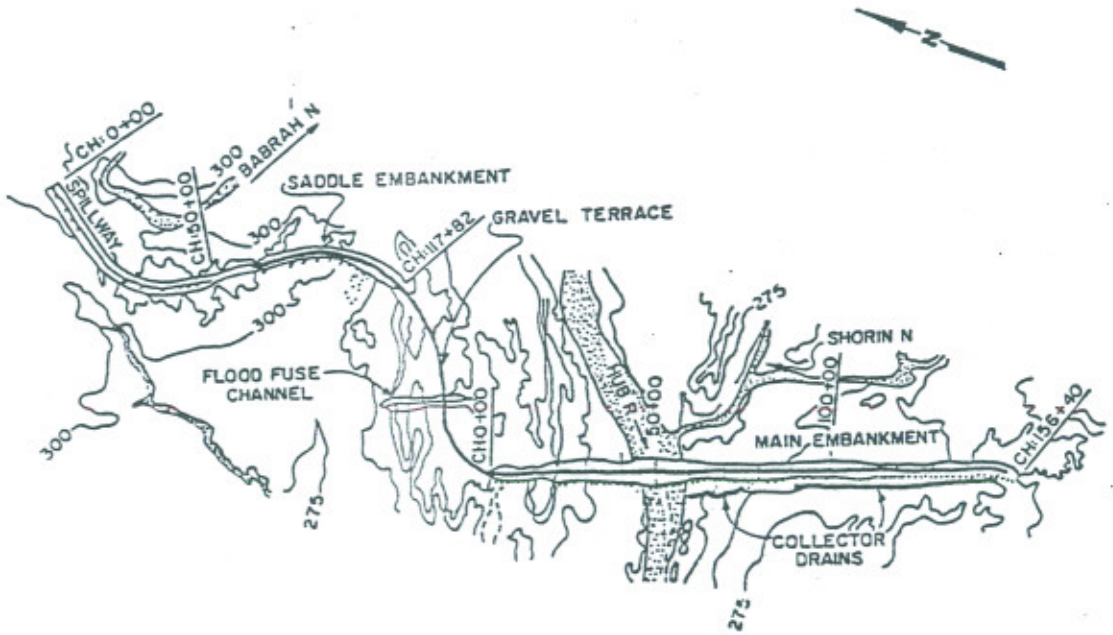


FIG.14 LAYOUT PLAN

HUB DAM

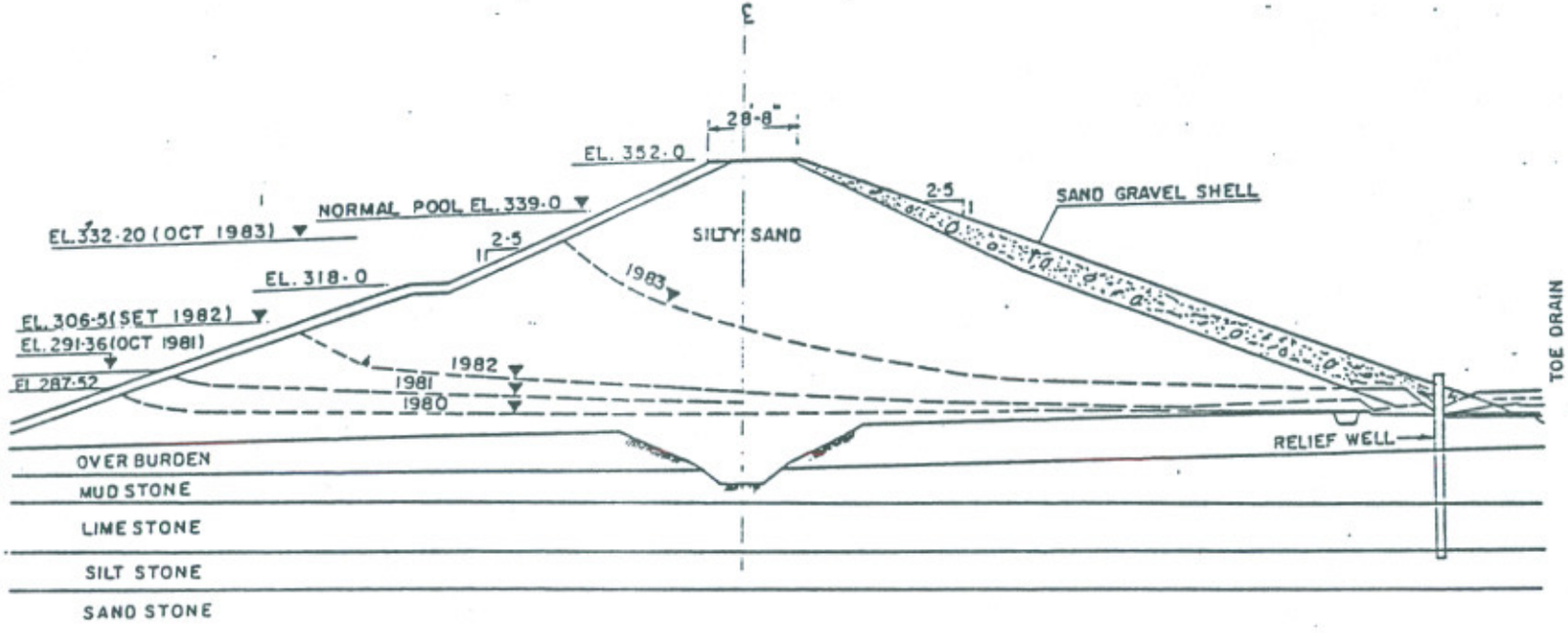


FIG.15 TYPICAL SECTION OF HUB DAM

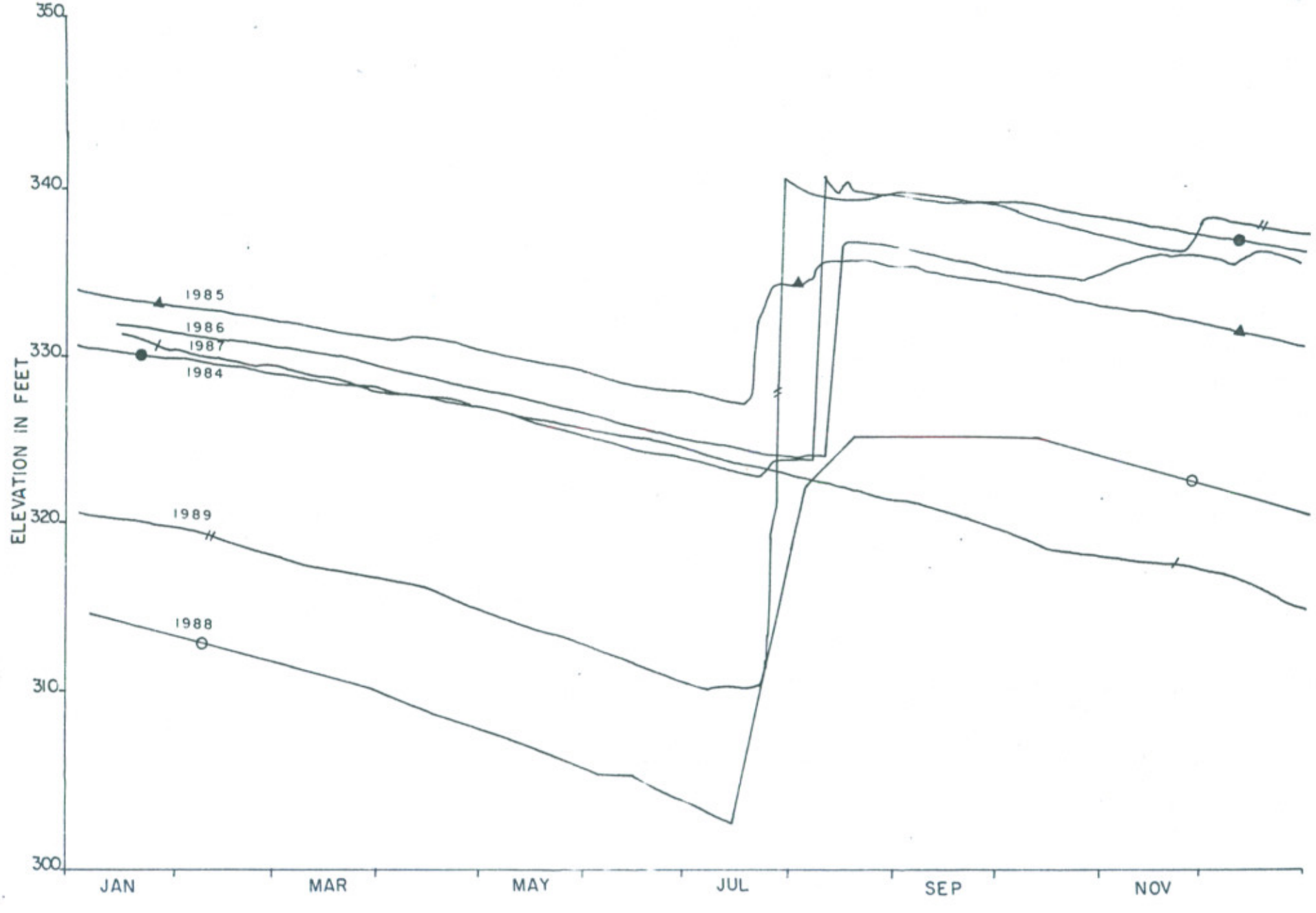


FIG.16 HUB DAM PROJECT

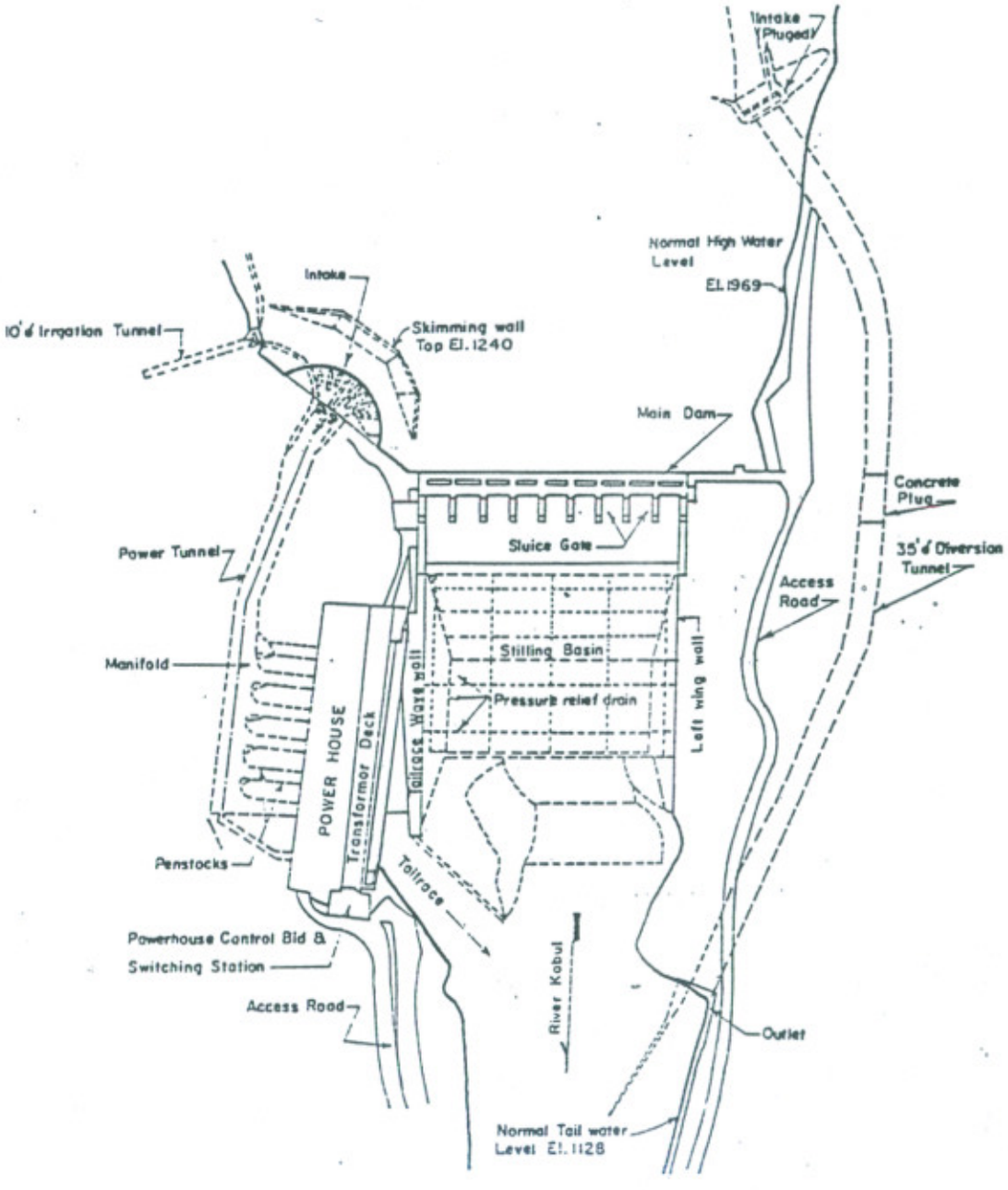


FIG.17 PLAN OF WARSAK DAM

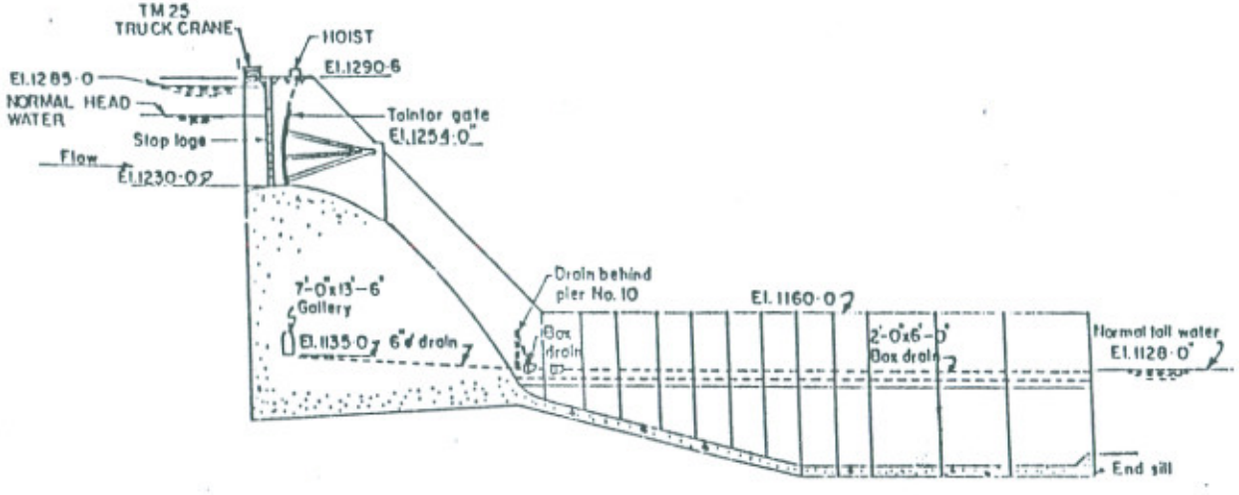


FIG.18 TYPICAL SECTION OF WARSAK DAM