Paper No. 490

Mangla Dam Project Appraisal of performance of structures and bedrocks

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APPRAISAL OF PERFORMANCE OF STRUCTURES AND BEDROCKS

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SYNOPSIS

Mangla Dam Project structures have performed satisfactorily for almost 20 years since impounding in July 1967. Appraisal of the Performance as per the following aspects has been done. Foundation & rim behaviour has also been examined.

1. Performance of cut slopes and reservoir slopes in weak rock, with details of failures or other stability problems, and remedial measures taken.

2. Piezomoter measurements as related to reservoir operation, particularly drawdown.

3. Performance, maintenance, and replacement of foundation drains.

4. Displacement and/or deformation measurements on main structures.

INTRODUCTION

Mangla Dam Project on Jhelum River, a left bank tributory of Indus was completed in 1967 In addition to Main Dam, with rolled clay core and rolled sandstones/gravel shoulders, with a maximum height 454 feet above core trench, there are three major embankments viz Intake embankment almost in continuation of Main Dam, with a maximum height of 262 ft. a rolled sandstone core with an up slope rolled sandstone blanket on bedrocks, gravel shoulders and five 30' dia tunnels passing underneath; Sukian dyke is almost an homogeneous embankment of rolled sandstone (finer fraction was used as core) with maximum height of 150 ft but mostly is a long embankment (17000 ft), about 75 ft high on a meandering ridge; Jari dam is a remote structure 274' maximum height with a rolled silt core & Siwalik Gravel shoulder. Layout of structures is at Exhibit-1 while the pertinent data is at Table-1. All the dams have cut off to bedrock i.e. founded on bedrocks, with shoulders mostly on alluvium i.e. alluvium under the shoulders not removed as a design requirement.

Mangla Dam area is an highly seismic zone & all the structures have been designed for M.C.E. (Maximum Credible Earthquake) equivalent to event of 1905 of 8.6 MM (Modified Mercalli) occurring at the boundary fault about 50 KM North of Project. Main Dam/Intake/Sukian were designed pseudo statically for 0.1g acceleration whereas Jari dam had an acceleration fraction of 0.13g. In addition the deformations were checked by Newmark method for higher acceleration upto 0.5g for Main Embankment. The concrete structures are designed for design acceleration as above.

Bedrocks are fresh water deposits of Plio-Pleistocene epoch called Siwaliks. Mangla structures are founded on the flatter dipping (10-12°) arm of Changhar monocline, whereas Jari has steep dips upstream 40°-45° with strike parallel to dam axis. The sandstones are weakly cemented fine

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to medium grained with upto 10% hard lenses while the intercalating clays are over consolidated, with clay fraction generally ranging 30-40% but sometime over 60%. Clay beds are gradational & siltstones have been excluded in designations by having clayey silt as part of clay & sandy silt as part of sandstone beds. Sheared zones parallel to bedding were revealed during excavation for foundation. The design parameters are given as Table-2. with also the safety factor criteria as Table-3.

Behaviour of Slopes viz-a-viz Design Slopes.

Slopes in bedrocks close to structures and forming part of structures are generally 1 on 1.75 (30^{o}) but in some cases flatter slopes have been used viz emergency spillway 1 on 2.25 (24^{o}) main spillway approach channel 1 on 2.5 and even 1 on 4.5 (12^{o}) for Intake Pool/slopes. However, if the space does not permit, as steep as 1 on 1.15 slopes above power station has been constructed but it is known that these will deteriorate & may fail in event of a large earthquake. For longterm stability following the discovery of sheared zones in clay beds & consequent redesign of slopes, the slopes for emergency spillway were flattened to 1 on 2.25 from 1 on 1.75. Spillway slopes with dips into basins/pool were provided concrete toes or clays excavated & substituted by gravel fill.

Longterm stability of slopes with shear zones & designed at residual shear strength of 0-18^{o} and for Jari at 0-13^{o} were of interest. Natural slopes, excavated slopes and slopes forming part of the structures have so far performed satisfactorily however most of them have not been subjected to limiting conditions viz saturation/earthquake forces of substantial strength. There has been slides in the reservoir and one in the tailrace of emergency spillway which point out the adequacy of the design and wisdom of designing the structures for longterm residual shear-strength; this after more than 20 years of being subjected to weathering & saturation but only minor shaking.

A slide in tailrace of emergency spillway had occurred in 1973 along bedding plane shear zone at 12^{o}. It occurred along the leftside of the channel which has bedrock dipping into the channel. It was caused by rain water seeping along the shear zone and reducing the effective stresses. Another slip along the same plane occurred in August 1983. Exhibit-2. When the slipped material at toe had been removed to reinstate the embankment. Multi reversal shear box test was carried on the slip surface material and indicate residual values of 12^{o} & $C = 1.0$ psi, at clay fraction of 22%.

The slip S-1 upstream of emergency spillway was a wedge type failure with bedrock dipping into reservoir at 12^{o}. The first slip occurred in March 1968 on drawdown Exhibit – 3. A much bigger slip occurred in February 1982 at the same location. The slip is along a bedding shear zone at 12^{o} but the plane has not been investigated.

There has been no slip in recent times due to earth-quakes. The only notable event being of Kangra 1905 (8.6 MM). The tectonic movements which caused the development of shear zones & consequent lowering of shear strength to residual value at most places along the sheared surface, occurred probably in the middle of Pliestocene-epoch. Subsequently there has been erosion of several thousand feet of deposits and these may have been helped by slips/slide occurring along the sheared planes in bedrock but no such signs i.e. a slip large enough to be noticed now, exists to confirm any slipping along sheared zones to have taken place.

The slips in reservoir are only few, exhibit-3 after 18 impoundings/drawdowns, notably for slopes dipping into the reservoir & having clays with shear zones. Few slips having occurred can be explained as:

i) Only part of the slopes are being subjected to saturation /drawdown.

ii) Wedge type failures have tension cracks/free surface upstream.

iii) Bedding dips are generally less than average residual values of shear strength however Jari dam area (dip 40^{o})/Bangla (Kanshi 40^{o} dip) are an exception. Jari reservoir has silt blanketing & topography is flat & this may explain the lack of slips. Bangala had a recent slip (Aug. 1984 slip S-23) & has been preliminarily related to under cutting of toe.

PORE-PRESSURE MONITORING

Main Dam – Clay Core.

Main dam has a rolled clay core constructed of excavated Siwalik clays which were watered, broken down & stockpiled. It was then laid at a moisture content of O.M.C. (Optimum Moisture Content) to O.M.C. + 2% in 9" layers. The average clay content of clays was of the order of 20-30%. Quite high construction pore-pressures upto 0.7 ru (pore pressure ratio) developed at the end of construction Exhibit-4. Over the last 17 years, the pore pressures in the core have steadily dissipated. As yet no flow net seems to have developed as is apparent, the design flownet is at Exhibit-4. With the reduction in construction pore-pressure, the rate of achivement of flownet pressures is expected to be slower & slower, needing a long time.

The fill of the core is near saturation & the rise of reservoir is reflected by response of piezometer by the pressure on the upstream face & by loading action also i.e. if reservoir rises above the tip location. During drawdowns the core piezometers respond to fall of reservoir with a time lag.

The lack of flownet development is supported by recent analysis of return water from hydraulic piezometers in core which shows large percentage of carbonate as against return water from piezometers with sandstone fill. Table-4

Jari Dam – Silt Core

The behaviour is typical of silt core. Some construction pore-pressures developed & these dissipated during the first few years. Steady state has been almost achieved. The rapid filling of the reservoir when the water goes over the Mirpur divide, register as only slight response at the piezometers to the reservoir. Drawdowns are much slower indicating lesser response.

Sandstone Fill

Sandstone excavated & with larger (above 12") hard lenses removed has been used in the shoulder of main dam and as core of intake embankment while Sukian Dam consists entirely of sandstone fill.

Main dam upstream shoulder piezometers in rolled sandstone follow reservoir closely except one at bottom close to core; keeping in view that sandstone fill being of permeability of 10^{-6} cm/sec, compacted to 100 % procter at 2% dry of optimum. Rolled clay core has consolidated by over 3.0' after construction and consequent load transfer could have affected the upstream shoulder fill. One explaination however is that hydraulic piezometer installation leads crossing from sandstone shoulder to the piezometer in upstream shoulder free draining gravel fill having been installed at same elevation, have not been properly placed. EXhibit-5. Sukian upstream shoulder piezometers do not behave similarly.

Two of the Dams of the Mangla Dam Project have sandstone cores namely Intake Embankment having a sandstone core with shoulders of gravel fill.

Secondly Sukian Dam, is almost an homogenious embankment and behaviour of the sandstone fill used as core as well as in the shoulders is commented. Exhibit-6.

Three aspects of the problems are required to be examined namely:–

1. Pore-pressure developed in the sandstone fill specially the cores during construction & subsequently.

2. The seepage measured at downstream toe has to be examined with regard to the overal permeability of the placed fill.

3. The settlement during construction and post construction has to be seen.

1. Development of pore-pressures in the Intake Embankment.

The maximum pore-pressures developed over the annual impoundments in the Intake Embankment are as per Exhibit-7. Proper flownet in the sandstone core did not develop because the sandstone in the foundation is more pervious than the sandstone fill of the core. The sandstone in the foundation has a permeability of the order of 10^{-3} to 10^{-4} Cm per second, whereas the sandstone fill has the permeability in the range of 10^{-6} Cm/Sec. The grading of the sandstone fill placed is as per Exhibit-8.

The construction pore-pressures were dissipated during the construction.

2. Pore-pressures in the Sukian Dam Fill.

Only one instrumentation section for monitoring of typical sandstone fill was established at Ch. 311+00. The embankment at this location is about 90 ft high and only reservoir above 1150 SPD starts affecting the piezometric heights and in the short interval between elevation 1150 to 1202 the conservation level, flownet does not develop at all, Exhibit-6 shows the maximum of the annual pressures recording. All piezometers indicate negative pore-pressure so far.

3. Seepage through the Intake Embankment.

The seepage as measured at seepage chamber Chainage 134+00 i.e. the maximum of about 50 GPM over the years. The quantity is obviously affected by the rains as all the surface drainage also passes through the seepage chamber. However if this quantity is accepted and looking at the L-Section of the embankment by rough calculations the permeability of the core works out to be of the order of 10^{-4} cm/sec. against possible 10^{-6} cm/sec.

4. Seepage Chambers at Toe of Sukian Dyke

Location of the various chambers is as per Exhibit-9 and the seepages measured varies from 90 Gpm to Zero. Sukian dyke is 17,000 long and seepage recorded through fill is nominal. This is obviously less than for the Intake Embankment whereas the chamber No. 1 at Chainage 190 has contribution by seepage through foundation only because of dam being on ridge.

Sandstone in Foundation

Siwalik sandstones of Mangla area are weakly cemented, can be scratched by hand where the strata is not gradational i.e. the portion of sandy silt, grading is generally fine to medium grained & particle texture sub angular except in the middle portion of Sukian dam. Jari sandstones being younger in age (upper Siwaliks) are more friable & with less hard lenses Sandstone in-situ has a permeability of the order of 10^4 cm/sec but occasionally 10^{-3} cm/sec viz spillway (lowest thick bed) alpha sandstone. The higher permeability at places is due to discontinuities in the form of clay balls/concretion/rarely gravel in sandstone matrix (no gravelly strata exposed at Mangla, however Jari has distinct gravelly horizons), stress release on excavation produces fissuring & for Intake embankment a deep narrow cut off of cement modified fill hand to be provided, also the upstream slope of Intake has a 20 ft sandstone blanketing due to the same reason. This is a particular case where close control of pore-pressures/seepages was desired and other measures viz steel lining of tunnels and removal/weak clays were also taken. Blanketing has also been resorted in controlling pressures through sandstone lenses at Jari rim Works but not for these being fissured, with the general criteria of hydraulic gradient of 1/20 to 1/10 being brought down to 1/20, whereas for the spillway approach channel an upstream impervious membrane of concrete slab has been constructed but partial length considered in design. The fissuring on excavation for left abutment of spillway i.e. junction with main dam was to be treated by grouting after loading the area but the test grout did not indicate that fissures could take cement/bentonite grout. Previously test grouting in Fort indicated that chemical grout could penetrate the fissures.

These pertinent details have been brought out as a reference to the performance of sandstone in foundation, further features as necessary are presented.

Main Dam Boiling in Downstream Pond.

Feeble boiling was noticed at downstream toe chainage 95+66 just north of southern toe weight when a shallow ($\pm$ 1 feet deep) pit was dug by a labourer in 1968. This was treated by weighted filter. Further feebler boiling was noticed intermitantly along the toe of southern toe weight for several years & cured its-self with the sand accumulating from the slope wash however simple G.I. Pipe shallow standpipes were installed for monitoring pressures & did not record any appreciable pressure in the alluvial but only fluctuation with change of reservoir of a few feet with max: levels fraction of a foot above the ground level. Exhibit-10.

In the old river section a pond was formed due to construction procedures. This had been left as such. Of late it was apprehended that seepages to this pond remain unmonitored in quality/ quantity. This situation was disturbing after the reduction in seepages as measured at the V-notch river section from 4 cusecs yearly peak to a maximum of 2.5 cusecs. Seepages measured downstream of pond was over 6 cusecs. There was the possibility of choking of free draining gravels at toe above V-notch. So that the seepages did not appear at the V-Notch. Also sedimentation of reservoir could have affected the seepages but there was no indications to support these apprehensions. It was therefore recommended that the pond be filled. Ideally a filter layer may have been provided all along its upstream periphery however the construction facilities at site & the cost led to the decision of filling it with sandy/silty pit run gravels.

On filling the pond/during the filling which was done upto 2' above normal pond level, boils appeared at several places on the spillway side periphery Exhibit-10. There was also boiling in the collector drain later on. The boils were ringed and are being monitored.

Investigation of the pressures in the sandstone/alluvium along toe of the dam has been done, to confirm the lithology/pressures in the various strata. Three sandstones have pressures higher than ground level. Further drilling is in progress to determine if these sandstone contribute to the boiling which can also happen from the water head along toe of dam, finding a preferential path to pond bottom and developed over the years.

The pressures in the sandstone will be treated by additional drainage wells at toe. Drainage wells have been previously provided downstream of the core at 12½ C/C & in some portion at 6¼' C/C. This location does not help monitor their performance. Also the pressure in the sandstone No. 12 & 13 at toe are inspite of the wells system, although pressure in sandstone-5 does not have the benefit of the drainage curtain of the Intake Embankment.

Sukian Seepages/Boiling in Downstream Nallahs.

Sukian dam has a meandering alignment along a ridge with deep cuts (nallahs) on upstream & downstream. Because of the high gradients in downstream nallah coupled with the discontinuities, in the sandstone bedrock boiling of intensities from vigorous/feeble has been experienced at various locations. These have over the years been monitored & where-ever intensity of boiling was considered to be accompanied by some movement of fines, weighted filters were placed. Exhibit-11.

Piping occurred in 1968 in sandstone Bed-27 exposure on base of slope in nallah. This happened when a dozer had been employed to remove the slight overburden for putting filters. A hole of man size occurred. This was immediately treated with graded filters & was controlled. Subsequently there has not been similar incident. In retrospect, it is conjectured that piping occurred by local heading up of pressures due to the slight overburden cover on cliff face as exposure of sandstone is also in the nallah closeby with steeper hydraulic gradient from reservoir.

Boiling is occurring sometimes at a distance of more than a mile downstream of dam & continues long after the drawdown due to time lag. Gradients along the location of boils are as per Table-5. It is now proposed to blanket upstream exposures of affected sandstones which can be approached when reservoir is drawn down.

Sandstone downstream of Jari have not developed boiling so far, though seepages are apparent downstream of right rimwork. This is mainly due to steep bedrock dips & strike parallel to dam axis, the intercalated clays providing cut off. On first impounding mini mud volcanoes were noticed in nalah downstream of Kakra Dam (an extension of Jari dam) however these stopped & have not since been apparent.

During construction of Jari tunnel, quick sand conditions were met at one location which were attributed to a perched water table in the concerned sandstone bed. The present seepages/soggy area along the access road to control room/stilling basin Exhibit-12 is due to a perched water table caused by rains causing seepage along the steeply upstream dipping sandstone (30-35^{o}). This type of perched table would have caused the quick sand failure in Jari tunnel during construction.

Pressures in Foundation of Main Dam/Intake/Sukian Dam.

Pressures in foundation of main dam are instrumented at Chainage 77+50 & Ch: 86+35. Behaviour over the years is plotted as piezometeric heights Exhibit-13. At Ch: 77+50 pressures in foundation under the downstream shoulder are below filter mattress & have remained so. At Ch: 86+35 on piezometer No. 21 records pressures above filter mattress, however the downstream piezometers at same section do not reflect similar behaviour. Pressures along toe has been discussed earlier.

Intake embankment/power station slopes have been heavily instrumented & typical behaviour is as per Exhibit-14. There has not been any apprehension about the behaviour of these piezometers.

Sukian dyke foundations are instrumented at two locations & recently casagrande pots have been installed in sandstone exhibiting boiling.

Jari has instrument section at Ch: 161+50 & 171+50. Development of pressures in sandstone under the upstream shoulder is apparent however under the downstream shoulder, there should not be any pressures due to the cutoff provided by steeply dipping intercalated clays/sandstone. These strata are not exposed in reservoir but rain effect cannot be eliminated refer remarks regarding incident of quick sand in Jari tunnel/pressure at toe along access road above.

Kakra dam foundations are behaving similar to Jari. Investigational instrumentation is being proposed.

Deformation and/or Displacement Measurements on Main Structures.

Mangla structures are founded on soft rocks of Plio Piectocene series designated as Siwaliks. The bedrocks contain clays and intercalated sandstone with rocks belonging to integradations i.e. siltstone/sandy siltstone/clayey silt. The bedrock clays are over consolidated having bedding plane shears, thrust shear joints & fissures. The shear zones have been traced for continuity for over 1½ miles, although a continuity fo 3 ft was considered in design. There can be several shear zones in one claybed but usual location is below the sandstone contact. The clays have major mineral as montmarrillonite. There has been large excavation (upto 250') and rebound of the order of 0.6 per 100 ft removal of overburden has been measured. Exhibit-15. Tunnels portion with less over burden load have registered extension at the free ends. 4½ inch extension in last 400 ft of tunnel -2 was recorded. Overall extension for the 1900 ft long tunnel was 6 inches. The background is repeated to make the paras on movements self explaining. Details are dealt below:

Settlement of Fill

Main Dam

Settlement of the dam has been monitored by USBR settlement gauges at 1000' interval along the crest. Also when fill load is less, a slight swell has been recorded. Consolidation of fill continues for the clay fill whereas the shoulder of sandstone & gravel settled within first few years of post construction. Relative settlements are at Exhibit-16 & Table-6.

Extension/compression measurements over the full length of the dam has been measured over the left abutment of 0.56' over a length of 200 ft whereas a compression of 0.46' has been measured for the adjacent length of 896 ft. The extension over the left abutment should have produced transverse cracking (generally with extension of 0.1 to 0.3%) but this is not so. However a crack had appeared at location of top edge of core on the access road to spillway in 1969 & produced a distinct depression on the downstream face of the dam in continuation.

INTAKE EMBANKMENT/SUKIAN

Sandstone excavation was easy, although rippers & sometime two dozers pushing the scraper were employed. The ripper was mainly to remove the hard lenses. Most of the settlement took place during construction & post construction settlements are a fraction of foot (0.2 to 0.3 per 100 ft of fill). These stabilized in the first few years.

Rebound

Mention has been made of the rebound during construction & extension of tunnels during excavation. The sandstone/clays have flat dips & have exhibited a uniform elastic behaviour. However during construction, placement of structure on thin lenses of sandstone was avoided.

To avoid the rebound effect of deep excavations, frozen bench marks were established close to the structure/within the premises. Closure pours, sometimes of one year duration were scheduled within the structure, so as not to stress the structure by settlements/rebound; the excavation levels were kept lower than designed to compensate for the rebound likely; the rebound measured were closely observed at the two main centres of deep excavation i.e. Power Station tailrace/main spillway so that proper allowance can be made.

There has been continuous rebound of 1000 elevation berm road above Power Station. This is a case of intercalation of hard sandstone & soft clay. It has been conjectured that clay is squeezing but this is a very slow process.

Swelling:

Clays are dominantly montmorillonites and after few years of impounding, swelling was recorded,particularly at Sukian Dam,a long low dyke of mostly 75 ft.height.Also clay bedrocks have generally clay fraction from 40 to 60%. The swelling caused by increase of moisture content of clay after impoundings/seepages developing through the intercalated sandstone beds & in some cases helped by more rapid travel of seepage through the discontinuities, viz sheared zones. (Also refer pressures in clays on flanks of spillway). Uplift has been recorded by bottom (1st)X arm of the settlement gauges is still continuing, although the rates of swelling is not significant to be of structural distress in the near future. Swelling pressures upto 12 tons/sq foot were measured in the Laboratory Table-7. A close network of crest markers were installed along entire crest. No significant uplift due to swelling has been apparent since 1978 but there is cyclic effect of reservoir fluctuation.

Swelling has been recorded elsewhere, where fill height is low viz Jari Rim works Table-8. The only structural distress attributed to swelling is at emergency spillway & further details appear in succeeding paras.

Horizontal Movements/Deformations

Horizontal movements/deformations has been measured by an elaborate network of movement control system by surface survey & instrumentally by slope indicators/plumblines (spillway Headworks monoliths only).

Horizontal Movements by Survey

An elaborate network, with station well away from the influence of reservoir pondings/unloadings/construction activities/traffic was installed before first impoundment. There has not been any interesting feature except the movements during first impounding of Forthill which did not further develop. The movement of main dam crest & marks on downstream slope is also shown. The recorded maximum movement was of 3.22' downstream by mid markers which were installed/observed earlier and hence had an effect of construction causing spreading. The crest maximum is 1.23 ft. The movements stabilized between 1975-79. The correspondence between observation of inclinometers is obvious but slope indicator have a limit of measuring upto 12° off vertical & were not successful for such observation in clay fill because of larger deformation.

Slope Indicators

Slope Indicators have been installed on slope of power station, downstream slopes of intake embankment/Forthill, as also slopes of spillway. None of these have recorded any movements & frequency has been lowered to ½ yearly observation. Slope Indicator No. 1 in approach channel demonstrates the limitation of slope indicator. It has some portion close to 9°-10° off vertical & with observation accuracy of 1/1000, was indicating apparent movements without physical evidence.

Spillway Plumblines

5 Sets of normal and inverted plumblines have been installed in the three spillway headworks monoliths & one each of the abutment blocks. The normal plumblines show a upstream movement at maximum reservoir level & revert to original observation on drawdown. The maximum observations are not necessarily at the maximum conservation level but usually occur before the level is reached. Over the years the maximum upstream movement has increased from 16 mm to 19.5 mm.

There has been change of couple of millimeters & these have set new maxima. The trend has to be watched/confirmed by closer observation at greater freduency/instruments checked for reliability.

Inverted plumblines of headworks monolith move in upstream direction 2-3 mm & revert to original on drawdown.

Movement of Block L3-L4--L5 of Left Abutment.

Typical abutment block (45' each) section is at Exhibit-17. Blocks are separated by contraction joints, block L5, showed a rotationery movement on first impounding of about 1 inch that reverted on drawdown,however no such movement took place again over the years. Plumblines No.1 in this block have been showing a retention of its upstream movements on drawdown Exhibit-18 & a tendency to record more upstream movement on impoundings. The inverted plumbline No. 1 also has off late shown movement in upstream direction. During 1983, these blocks L3 to L5 again moved upstream as shown by offsets of ½" at the contraction joints. The blocks are founded on clay and a concentration of loading may occur on the upstream toe causing on upstream tilting. There is the remote possibility of sliding but this effect should be apparent on drawdow & not on impounding. The sliding can be on base of the block along a shear zone, however this would have registered on the inverted plumbline which are anchored deep in foundation. A plansible explaination is that the height of the block varies along the contreline of roadway and the water pressures on impounding are varying along the block, giving them a rotational tilting with inter block friction carying all the three blocks.

Concrete Structures Joints

Main spillway joints system & details of the joints are shown at Exhibit-19 & 20. Movement pins were installed and do not show any movement of significance. The open joint downstream of headworks showed a maximum of 0.25 inch movement as against 2" provided and 5" considered during design to accommodate possible displacement on Ist impounding.

The expansion joint between weir & lower chute Exhibit-21, with provision of movement along the chute (after a closure pour left unpoured for a year) and shear failure of weir toe & in almost all the blocks of weir occurred due to settlement as any vertical movements, would lead to concrete failure by shear. The same happened due to swelling of clay bedrock for weir toe of emergency spillway.

The provision of keys between adjacenct pours, subjected to differential settlement between each other, lead to spalling of concrete. This has happened to concrete slab adjacenct to groin walls but is not significant being available for repairs, being at the back of groin walls. Such incidents also happened on the side slab along the upstream of weir due to the key provided, obstructing the free movements of the construction joint in a vertical plane.

The construction control joint with corrugated sheet for controlled cracking is not desirable as cracks do not occur on top of the sheet but some times stray & spalling takes place.

R7, monolity of right training wall of upper chute showed movements and spalling due to clay bed-rock swelling from seepages/rain water getting access to the foundation. Possibly fissuring took place of the intercalated clays/sandstone on excavation of right drainage tunnel & nearby the ingress of water, caused recent movements.

On excavation of emergency spillway outlet/priming tunnel, rebound in clay was recorded but no movements took place, once the structure was in place. There has been opening of construction joints of chute slab of emergency spillway (1978) of the order of 1/4", possibly due to temperature variation as further movement did not take place.

Power Station Movements/Tunnel Movements

There has been movement of downstream wall of power station towards upstream at diversion stage as the river sand deposited in tailrace & this caused the deflection of the wall. No subsequent movements took place. There is a longitudinal joint right through the generator blocks & the roof is like a cap on top i.e. placed on the walls, however there has not been any significant differential movements to cause deflection.

Recently, some cracking has been noticed but is not of distress so for, as all the power house substructure/walls are heavily reinforced. Tilting of generator blocks so far is maximum of 140 thousand in 40' length and affects the original units 1-4 probably as explained above.

The articulated joints between the intakes & tunnel on the culvert section were designed for freedom of 1^{o}, however leakages took place at diversion/impounding stage and the downstream joints of all tunnels have sealing rings installed. Subsequently sealing rings on upstream joint has been placed in tunnel 1,2,4 as large leakages occurred (Over 30 Cusecs in Tunnel-4).

Foundation Drains

A network of foundation drains have been provided under the spillway slabs & components Exhibit-46. Details of the drains are at Exhibit-22.

Seepage & uplift features of headworks, consist of an upstream impervious slab, about 200' along with a further 100' of sloping slab, (but only 85 ft considered for the flownet); headworks drainage wells (at upstream and of headworks) drain into gallery with lowest portion at 1051.75 elevation, while the pressure points under the approach slab are at 1063 elevation (Numbers 8,10,12 & 14 of series a, f & g). The approach slab is placed on bedrocks, with no weep-holes and drains. There is a second line of drainage wells which drain into a low level adit at $\pm$ 900 elevation, with wells also going down from it. The impervious slab is 1 ft thick. The approach pit geology is mainly sandstone refer Exhibit-23. The headworks drains/drainage well system are designed to independently control the uplift pressure to acceptable limits.

Pressure points of f & g series under the impervious slab & numbering 8, 10, 12 & 14 do not show a uniform drop of head from upstream to downstream i.e. g-series higher heads & f-series lower. All these pressure points fluctuate with the reservoir but with hardly any time lag, typically. However the a-series (elevation 1055) do not record/record significant pressures & hence confirm the complete effectiveness of system of impervious slab/drains & contribution by headworks drainage wells. There has been an improvement trend as shown by observation of a-series. Headworks wells have over the years have now achieved a more or less optimum condition in conjunction with DU wells. The headwork wells flow pattern at maximum reservor levels is at Exhibit-24 & this would support the view that large flows from drain RA1 are seepages from Beta sandstone & not leakages. Refer further paras on page–29 for details.

Under Drain Performance

An elaborate network of under drain totalling 192 has been provided under the spillway slab/ structures as below. Exhibit-22 shows the flowing drain marked:

Headwork	17	17
Upper Chute		72
Upper Stilling Basin		66
Lower Chute		29
Lower Stilling Basin		8

The drainage system has so far performed satisfactorily & there has not been any potential in-adequacy. There have been several maintenance problems i.e.

i) Encrustation of drains
ii) Movement of filters.
iii) Movement of filters/fines.

i) Encrustation of Drain

The sandstone bedrock contains on the average 20% carbonate as binder. This is removed as bicarbonate by the seepage water and deposited in the 8" diameter, hume concrete pipes conveying the seepages/leakages from the drain to the various galleries. The concrete pipes have been placed with open joints & also bend into the gallery. Encrustations are hard & have to be chisilled & this may damage the drain, causing filter migration/exposure of foundation with movement of fines by seepages if large enough. Cleaning with use of sulphamic acid will attack the concrete pipes & the filters which is undesirable Exhibit-22. Drains of the central gallery have two ends accessable & cleaning with water is possible. The side drains are adjacent to bedrocks & the ones getting encrusted. Encrustation is being removed but there has not been indication of inadequacy, and need for additional so far. Effected drains are at Exhibit-22.

ii) **Movement of Filters**

Filters were noticed in a numbers of drain Exhibit-22. The filters come through the open joints of concrete pipes & may be some pipes were crushed during construction. The drains have minor flows & filters in these pipes do not impair their efficiency. However these have to be watched & it is proposed to fill the pipe with pea gravel and monitor for any movement of fines.

iii) **Movement of Filters/Sandstone.**

This is a phenomena peculiar to drains of lower stilling basin which is all the time under water. On dewatering heaps of sand/filters are seen near each drain, typically as below:—

Ref: Drain	Discharge Liter/sec.	Sand at outlet
LS1	4.77	One bucket
LS2	1.95	One bucket
LS3	0.60	1 Cft
LS4		
LS5		
LS7		

Sleeves have been put, to stop the movement of fines, from the open jinted concrete pipes.

iv) There has been incidence of a drain responding to opening of contraction joints during winter. The replacement of joint sealer on the joint has controlled the leakage.

Emergency Spillway Drains

A network of drain have been provided. The emergency spillway has not operated so far. There are some seepage flow after rains. No encrustation or filter movement has been noticed.

Drainage Well System

Foundation of Main Spillway, Intake Embankment have an elaborate system of drainage wells Exhibit-25. In addition 10 wells have been provided below tunnels and wells at 12½′ centre are provided downstream of the core of main dam which are not accessable. These are B-type well. Their performance and inadequacies have been dealt in para page-10.

Drainage wells layout for main spilway & intake are at Exhibit-26. A total of 403 wells have been installed.

Spillway Wells

Main Spillway Headworks have a line of wells (66 Nos) under the upstream toe & draining in the headwork gallery. Another line of wells (DU Wells 26 Nos.) was provided at downstream toe of Headworks following the discovery of sheared clay zones draining to a low level adit, with wells going below the adit (DL wells19 Nos.). The DU wells also cover the left flank of upper chute. Initially the headworks wells took up most of the seepages but gradually the DU wells have taken the larger share. DL wells flows remain more or less constant. Refer table below:

Maximum Flow

Year	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983
Head-works. Wells	172.7 GPM Sept.	83.0 GPM Sept.	153.1 GPM Sept.	156.4 GPM Oct.	135.4 GPM Oct.	128 GPM Spt.	120 GPM Spt.	121 GPM Spt.	128 GPM Spt.	116.5 GPM Spt.	122 GPM Oct.
D.U. Wells	114.3 GPM Oct.	122.15 GPM Oct.	133.2 GPM Nov.	156.6 GPM Oct.	171.6 GPM Nov.	177 GPM Dec.	189 GPM Spt.	193 GPM Oct.	201.5 GPM Aug	211.0 GPM Aug.	192 GPM Oct.
D.L. Wells											15.8 GPM Oct.

There has been problem of carbonate encrustation & one of the wells(DU 26)got completely filled within first few months of impounding & was cleared by salphamic acid. All the wells are viewed periodically by C.C.T.V. Camera & cleared of encrustation by a sewage rodder. No chemical clearing, as for DU 26 has been necessary. The drainage well system is adequately draining the headwork foundation & has not deteriorated. Also refer Exhibit-23 for pressure across the wells.

Spillway pumped Wells

Pumped wells LP1-LP9 on left flank & RP1-RP7 on right flank, (one in weir gallery RP3) were provided to cut off seepages to the spillway from main dam & rain effect from slopes on right flank. The left flank wells have suffered with iron encrustation, fortunately it does not stick but chokes the riser (about 70 lbs is removed from one well). The wells only puncture the top sandstone strata & are consequently, inadequate in keeping pressures behind the stilling basin walls to acceptable limit. More wells have been recommeded to be put as the pressure in base sandstone (alpha) are also high & pumping rate is inadequate in lowerng the ground water to design limits.

Intake Embankment Foundation.

A line of wells draining into an adit (T-adit) was provided about 200' downstream of centreline, going right through the Forthill into the main foundation. On the left flank it ends with the left abutment. Another line of wells (P-adit) 175' upstream of centreline with flanks on either side enclose the centre portion of the intake embankment. The wells have encrustation by carbonate & are cleaned/viewed by CCTV. There has been problem of outflanking of wells of left flank which terminate within the left abutment. However the seepage did not develop to the extent to go past the last well although all the wells of left flank are operating. The upward going wells are in-accessable from top and these can be cleaned with difficulty. Also the cleaning operation causes filter to settle & the finer component moves through the well screen. This is not a desirable situation. Also cleaning by salphamic acid may also break down the filter. There has been incidence of filter loss in downward wells which were not properly developed. Subsequently filter material was put in & no further loss has been registered. A comprehensive network of instrumentation was done to monitor the development of pore-pressure in intake embankment foundation & also monitor well adequacy. There has not been any sign of inadequacy except the left flank of T-adit described above.

Wells below tunnel also suffer from carbonate encrustation.

MANGLA DAM PROJECT

(Salient Features)

MAIN DAM & INTAKE EMBANKMENT

Type	: Earthfill	
Maximum Heigh	: 454 feet	= 138.38m
Crest Elevation	: 1,234 ft. SPD	= 376.12m SPD
Length of Crest	: 11,000 ft.	= 3352.8m
Excavation	: 15,500,000 cu. yds.	= 11,842,000 cu. meters
Fill	: 85,000,000 cu. yds.	= 64,940,000 cu. meters

SUKIAN DAM

Type	: Earthfill	
Maximum Height	: 144 feet	= 43.89m
Length at Crest	: 16,900 feet	= 5,151.12m
Excavation	: 7,000,000 cu. yds.	= 5,348,000 cu. meters
Fill	: 12,500,000 cu. yds.	= 9,550,000 cu. meters

JARI DAM

Type	: Earthfill	
Maximum Height	: 274 ft.	= 83.52m
Length at Crest	: 14,500 ft.	= 4,419.6m
Excavation	: 16,500,000 cu. yds.	= 12,606,000 cu. meters
Fill	: 43,000,000 cu. yds.	= 32,852,000 cu. meters

MAIN SPILLWAY

Type	: Submerged Orifice	
Capacity	: 870,000 cusecs (at pool el. of 1202'SPD)	= 24,360 m^3/second
Type of Gate	: Tainter	
Number of Gates	: 9 (nine)	
Size of Orifice	: 30' wide & 40' high	= 9.14 m x 12.19m
Excavation	: 33,000,000 cu. yds.	= 25,212,000 cu. meters
Fill	: 890,000 cu. yds.	= 679,960 cu. meters
Concrete	: 1,200,000 cu. yds.	= 916,800 cu. meters

EMERGENCY SPILLWAY

Type	: Unregulated Weir	
Capacity	: 230,000 cusecs (at pool el. of 1228'SPD)	= 6,440 cu. meters/sec.
Excavation	: 14,000,000 cu. yds.	= 10,696,000 cu. meters
Fill	: 630,000 cu. yds.	= 481,320 cu. meters
Concrete	: 75,000 cu. yds.	= 57,300 cu. meters

BONG CANAL & NEW BONG ESCAPE

Capacity	: 49,000 cusecs	= 1,372 cu. meters/sec.
Length	: 25,000 ft.	= 7,620 meters
Bed width	: 80 ft.	= 24.38 meters
Depth	: 44.5 ft.	= 13.56 meters

RESERVOIR OPERATION

Normal Conservation Level	: 1202 ft. SPD (above mean sea level)	= 366.37 m
Minimum Operating Level	: 1,040 ft. SPD (above mean sea level)	= 317 m
Gross Storage	: 5.88 MAF	= 0.725692 Hectare meters
Live Storage	: 5.34 MAF	= 0.658956 hectare meters
Surface Area	: 100 Square Miles	= 259 Square Kilo-meters
Population affected	: 81,000	
Cost	: Rs.3,200 million	= US $ 674 million

POWER STATION

Number of Sets		
Present development	: 8	
Ultimate development	: 10	
Nominal Size (Each set)	: 100 MW	
Turbines		
Type	: Vertical Francis	
Design Head	: 295 ft.	= 89.92m
Head Range	: 192 ft. to 429 ft.	= 58.52m to 130.76m
Inlet Valves		
Type	: Butterfly	
Internal Diameter	: 16 ft.	= 4.88m
Effective Closing Time	: 240 Secs.	
Weight of Valve	: 192 tons	
Irrigation Valves		
Type	: Howell-Bunger	
Internal Diameter	: 8 ft. & 6 inches	= 2.59m

TABLE-2

Final design bedrock parameters

Position	Clay bedrock: Along bedding			Clay bedrock: Across bedding			Bentonite			Sandstone bedrock		
	c' lb/sq. in	φ	R %	c' lb/sq. in.	φ	R %	c' lb/sq. in	φ	R %	c' lb/sq. in	φ	R %
Mangla Dam	o	18	100	6	26°	50	–	–	–			
Intake embankment	o	18°	100	0	24°	100	–	–	–			
Sukian Dam (western half)	0	18°	100	6	25°	50	0	12°	100			
Sukian Dam (eastern half)	0	16°	100	6	25°	50	0	12°	100	o	38°	100
Jari Dam	0	13°	100	5	20°	50	–	–	–			
Jari rimworks	0	13°	100	0	16°	100	–	–	–			
Main spillway (headworks and upper chute)	0	18°	100	0	24°	100	–	–	–			
Main spillway (stilling basins)	0	18°	100	3	26°	75	–	–	–	3	38.5°	75
										(above water-table)		
										0	38.5°	75
										(below water-table)		

Effective shear strength parameters of fill materials

Material	c' lb/sq ft	φ
Rolled clay	450	22°
Rolled silt	0	30°
Rolled sandstone	0	36°
Sandstone/clay	450	28°
Gravel fill / Washed gravel fill / Free-draining gravel	0	40°

Contract design bedrock parameters

Position	Clay bedrock: c' lb/sq. in	φ	Sandstone bedrock: c lb/sq. in	φ
Mangla Dam	3.5	32°		
Intake embankment	0	30°		
Sukian Dam	0	31°	0	38°
Jari Dam and rimworks	0	28°		
Main spillway	3.5	32°		

Seismic coefficients used in design

Mangla main embankment	(a) Overall	0.11 g
	(b) Top 130 ft	0.15 g
	(c) Bedrock	0.11 g
Intake embankment	(a) Fill	0.14 g
	(b) Bedrock	0.10 g
Sukian Dam	Fill and bedrock	0.12 g
Jari Dam	(a) Overall	0.13 g
	(b) Top 60 ft	0.17 g
	(c) Bedrock	0.13 g

TABLE-3

Design factors of safety

Condition	Factor of safety (equal to or greater than)
Upstream slope	
Rapid drawdown	1.4
Rapid drawdown with earthquake	1.0
Downstream slope	
End of construction	1.4
Earthquake (surfaces breaching the core)	1.2
Earthquake (surfaces not breaching the core)	1.0

TABLE - 4

CHEMICAL ANALYSIS OF RETURN WATER FROM PIEZOMETERS

Source	MILLI	EDUIVALENTS						TOTAL	p^H
Piezo No.	Ca	Mg	Na	CO_3	CI	SO_4	Catons	Anions	
117-Core	0.94	0.66	1.85	1.80	1.00	0.70	3.45	3.52	7.8
114-Core	0.94	0.46	0.20	1.40	0.20	0.10	1.60	1.72	7.8
145-Core	2.50	1.10	1.70	3.60	1.80	0.10	5.42	5.52	7.5
120-U/S SST	0.52	0.48	0.41	1.10	0.20	0.20	1.51	1.61	7.6
147-U/S SST	0.73	0.07	0.17	0.70	0.20	0.10	0.97	1.06	7.5
148-U/S SST	0.63	0.07	0.16	0.60	0.20	0.10	0.85	0.93	7.5
185-D/S SST	0.31	0.15	0.15	0.40	0.20	0.10	0.65	0.75	7.3
186-D/S	0.31	0.09	0.28	0.40	0.20	0.10	0.68	0.73	7.2
Reservoir	0.39	–	–	1.35	0.05	0.60	–	–	7.6

T = Traces, K, Fe, R, & Co_3 = Nil

TABLE-8

PHENOMENA OF SWELLING – MANGLA DAM

St.No.	Site	Station	Max. Ft.	Up Lift Inches
1.	Emergency Spillway	SD 4.77	0.305	3.66
2.	Power Station Area	SD 3.55	0.593	7.12
3.	Sukian Dam	DM 6.4	0.219	2.63
4.	JARI			
	i) Mid Height	–	0.557	6.69
	ii) Toe	–	0.220	2.64
5.	Kakra Dam			
	i) Toe	–	0.357	4.28
6.	Now Bong Escape	–	0.374	0.89

Table–5

MANGLA DAM

BOILING POINTS SUKIAN DAM AND INTAKE AREA

Nullah No.	G. Pt. No.	Elv: SPD	STRATA	Length (L) Ft.	RL at 1202'	RL at 1110'	IL at RL-1202	IL at RL-1110'
B_2	BL1-9	941.16	Intake SST-9	400	260.84	168.84	1/1.54	1/2.4
B_3	BL1-6	973.32	Sukian SST-6	5500	228.68	138.68	1/24	1/40
B_3	BL1-12	1074.46	Sukian SST-12	2000	127.84	35.87	1/16	1/56
B_7	BL1-27	1086.92	Sukian SST-27	2250	133.08	45.08	1/17	1/55
-do-	BL2-27	1091.89	–do–	1250	110.11	18.11	1/11	1/89
-do-	BL5-27	1090.72	–do–	1250	111.28	19.28	1/11	1/65
-do-	BL2-24	1043.50	Sukian SST-24	2000	158.50	66.50	1/13	1/30
-do-	BL3-24	1038.20	–do–	2000	168.80	71.80	1/12	1/28
-do-	BL1-10	920.91	Intake SST-10	5750	281.09	189.09	1/23	1/30
-do-	BL2-10	924.51	–do–	5500	277.49	185.49	1/19	1/29
-do-	BL3-10	929.11	–do–	5250	272.89	180.89	1/19	1/28
-do-	BL4-10	930.20	–do–	5000	271.80	179.80	1/18	1/28

Table–6

MANGLA MAIN DAM
SETTLEMENT IN FILL MATERIAL

Sr. No.	Description of Material	From 1967 to 9/77		From 9/77 to 12/79		From 12/80 to 12/81		From 12/81 to 12/82		From 12/82 to 12/83		Total From 1967 to 1983		The end of construction i.e.1967	
		Total Ft.	Per. 100′	Total Ft.	Per 100	Total Ft.	Per 100′	Total Ft.	Per 100′	Total Ft.	Per 100	Total Ft.	Per 100′	Total Ft.	Per 100′
1	Rolled Clay Core	5.02	0.64	0.27	0.04	0.12	0.02	0.16	0.03	0.09	0.01	5.66	0.74	23.72	3.04
2.	Washed Gravel Fill	1.05	0.14	0.09	0.01	-0.03	–	0.10	0.02	-0.01	–	1.24	0.17	3.21	0.31
3.	Rolled S.St. Type 'A'														
	a) U/s Shoulder	1.25	0.19	0.23	0.03	-0.02	–	0.03	0.006	-0.01	–	1.51	0.23	13.36	2.16
	b) D/s Shoulder	1.81	0.14	0.12	0.02	0.05	0.007	-0.07	–	0.10	0.01	2.08	0.18	14.67	1.12
4	Rolled St/Clay	1.70	0.44	0.07	0.02	0.04	0.01	0.06	0.02	-0.05	–	1.87	0.49	10.60	2.75

Table—7

MANGIA CLAY BEDROCK – SWELLING PRESSURE TESTS

Sample	LL No.	Clay Fraction	FIELD CONDITIONS M/C	D Lb/C.f.	SWELLING FEATURE Saturation % (Approx.)	T Sq. Ft.	Lb./ Sq.Ft.	Location
63355	45	27	17.0	115	98	0.20	3.1	Intake Core trench. Bed 8 minus. Same bed as Block-II about 100′ away.
63358	45	35	11.9	130	96	0.75	11.7	Intake Core Trench. Bed 9A minus. Same location as Orange-II
63359	51	37	17.3	111	92	0.40	6.2	Intake Embankment Bed II minus Soft clay at Block-I.
62218 x	58	44	16.3	123	100	0.30	4.7	Main Dam core Trench. Bed 217. Same location as purple sample 62218.
58133	62	(49)	14.6	(119)	(95)	0.75	11.7	"Black" Clay, Trench Se-26.
58137	59		21,9			0.35	5.4	"Red" Clay, Core Trench Excavation.
58138	44		10.8			0.58	9.0	"Brown" Clay, Old Road Cutting.
58136	85		55			0.70	10.9	Bentonite, Core Trench Excavation.
58147	138	(79)	44.9	(69)		0.94	14.6	"Red" Bentonite (As for shear box test).
					Results from previous tests shown in brackets.			
48603A	31	11	7.4	129	97	8.78	12	Buttress between tunnel Portal 4 & 5.
48746A	46	20	13.9	133	100	2.0	31	P/Sta. excavation slope on berm.
48742A	36	21	7.0	136	88	2.5	39	" " 70′down.
48743	33	14	4.3	138	53	2.5	39	" " 75′down.
47954	38		6.6	139	81	1.5	23	West of Tunnel Portal 5.
47952	38	63	5.7	135	72	6.5 (12.2)*	100 (198)	Tunnel Portal No. 5. West Wall
47901	47	64	9.0	138	100	6.5 (11.2)*	100 (170)	" " "
479018	42	64	8.2	133	100	7.5 (10.4)*	117 (162)	" " "

*Figures in brackets are after Odeometer correction.

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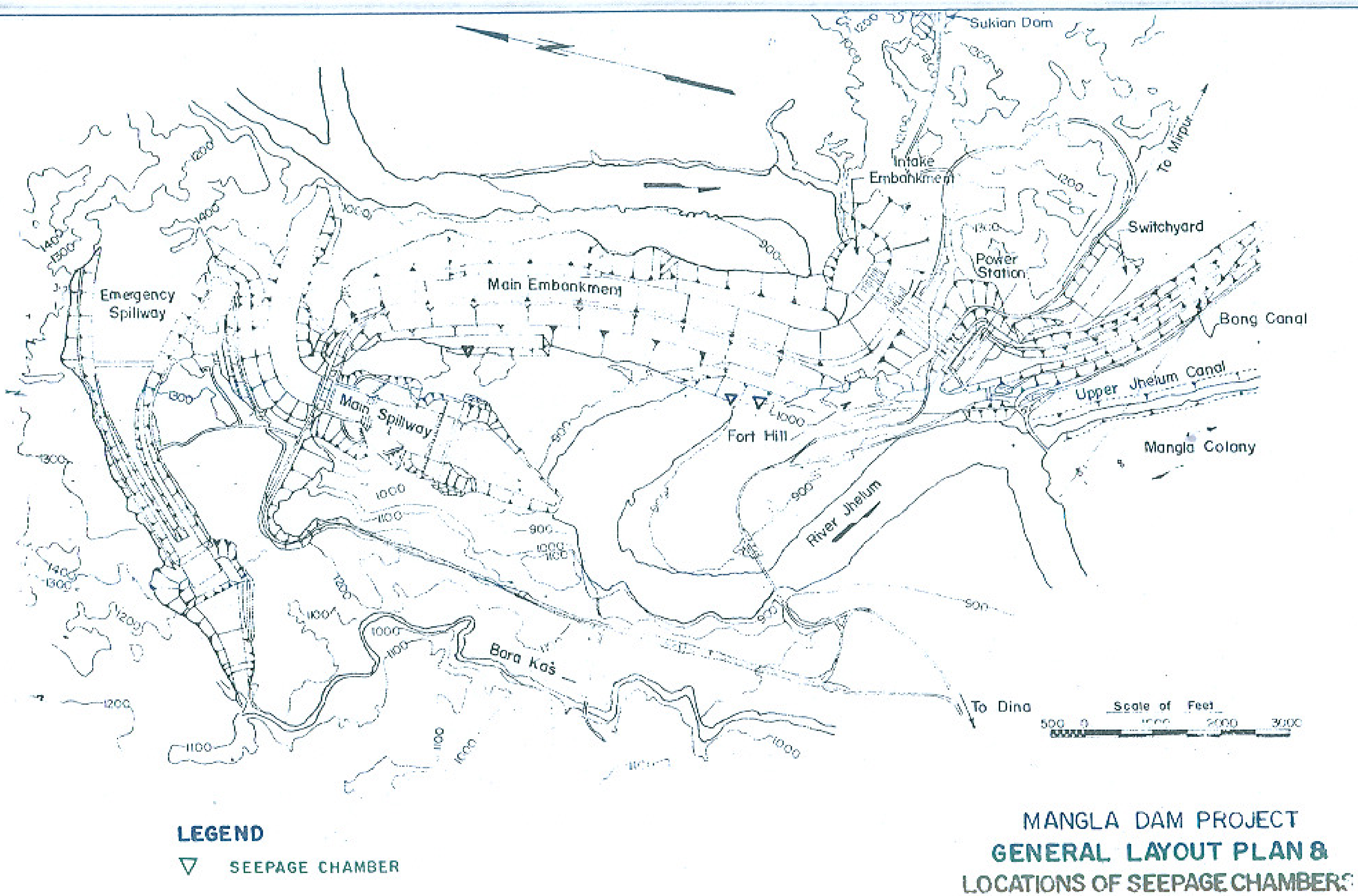
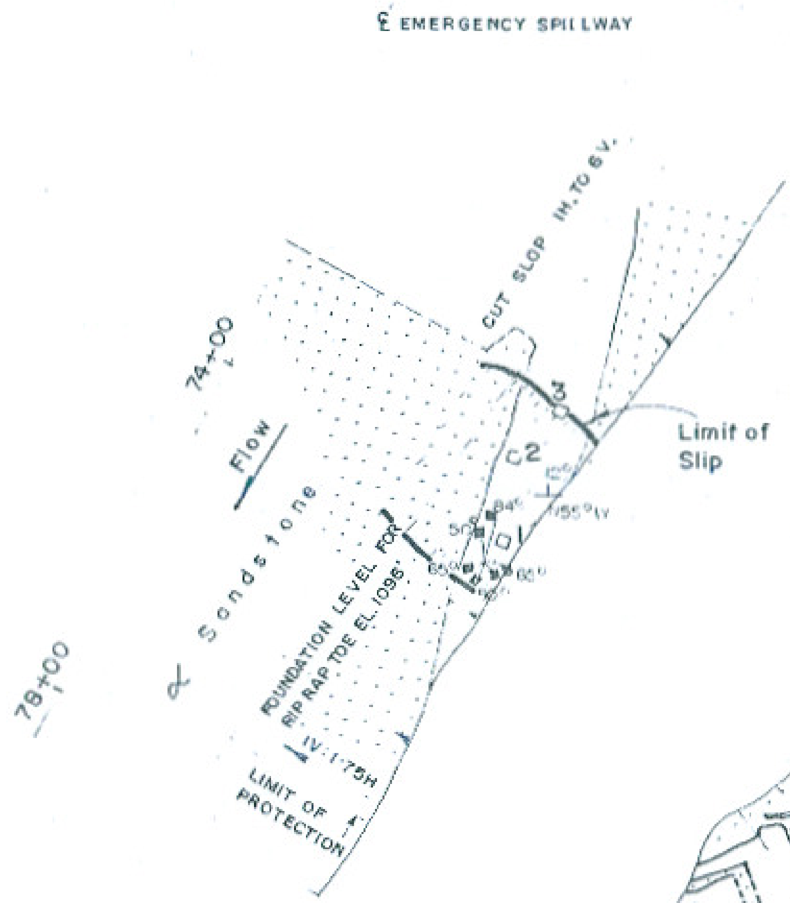


MANGLA DAM PROJECT
GENERAL LAYOUT PLAN &
LOCATIONS OF SEEPAGE CHAMBERS

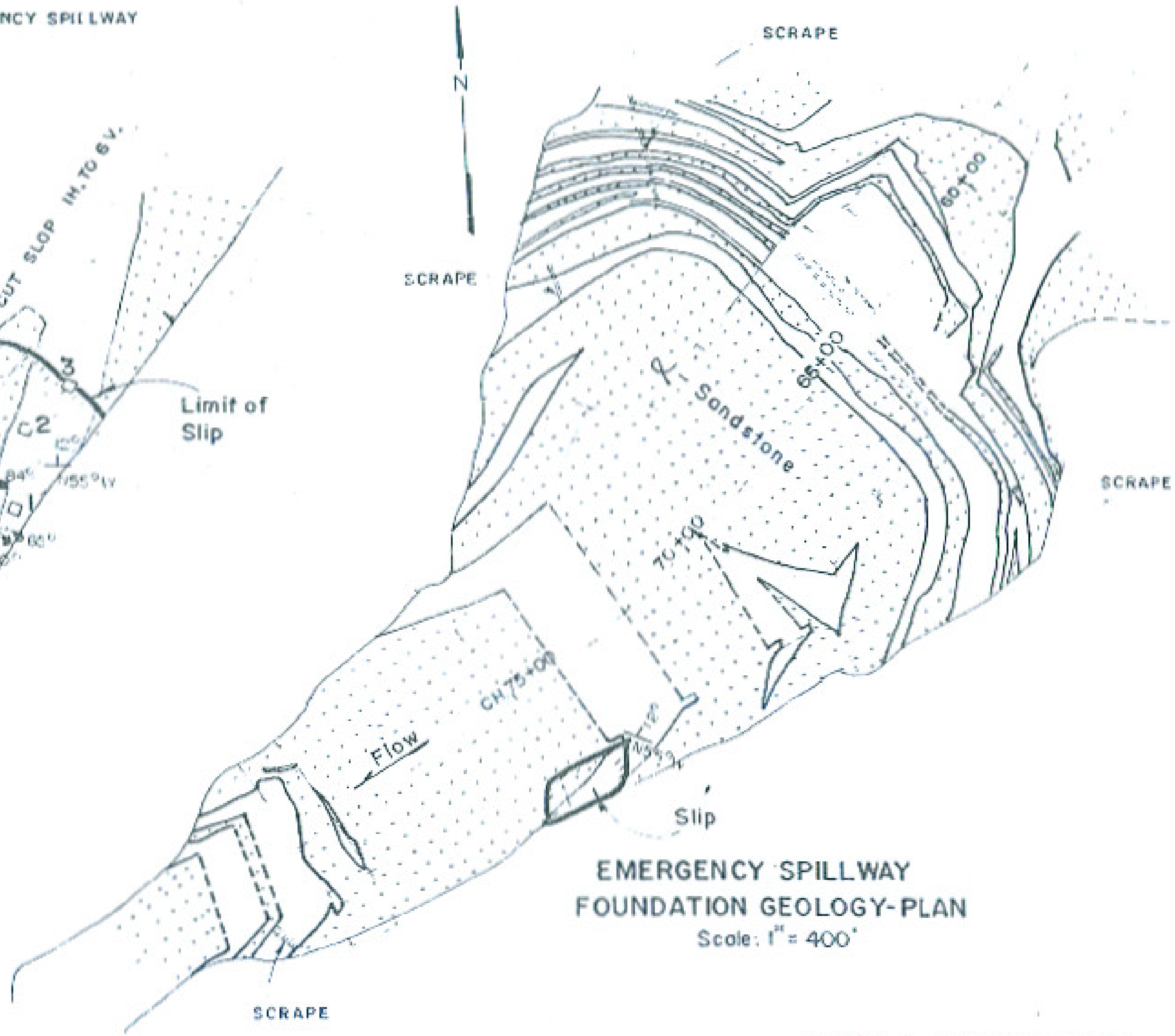
DMO/M/I-1

Tariq / 3-85



ENLARGED VIEW SHOWING LOCATION OF SLIP

Scale: 1" = 80'



EMERGENCY SPILLWAY FOUNDATION GEOLOGY-PLAN

Scale: 1" = 400'

LEGEND

- Sandstone
- Clay
- 2 □ Rock sample

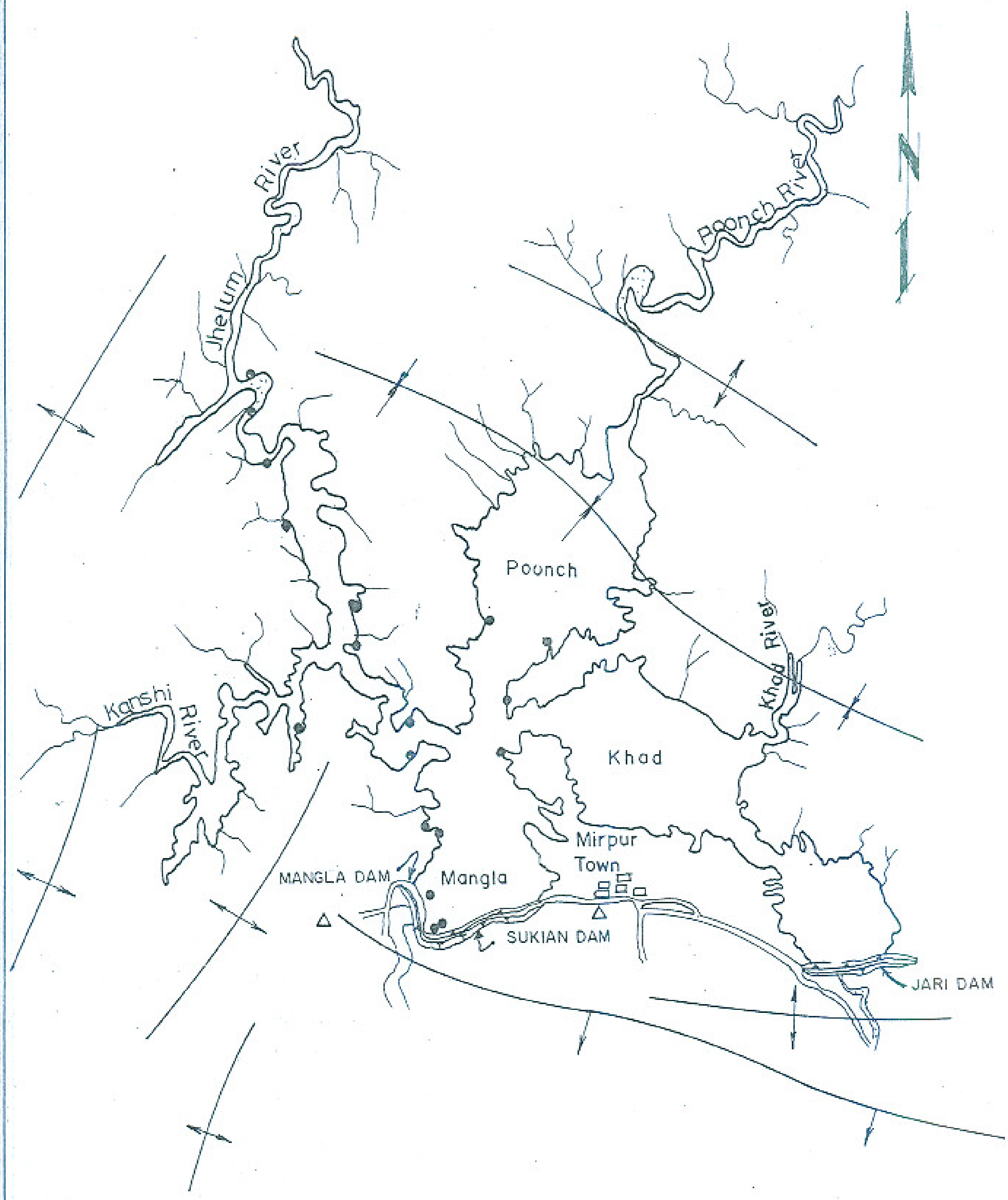
MANGLA DAM PROJECT

EMERGENCY SPILLWAY-TAILRACE

EMBANKMENT SLIP LOCATION

Khalil / 2-85

Paper No. 490 EXHIBIT- 3

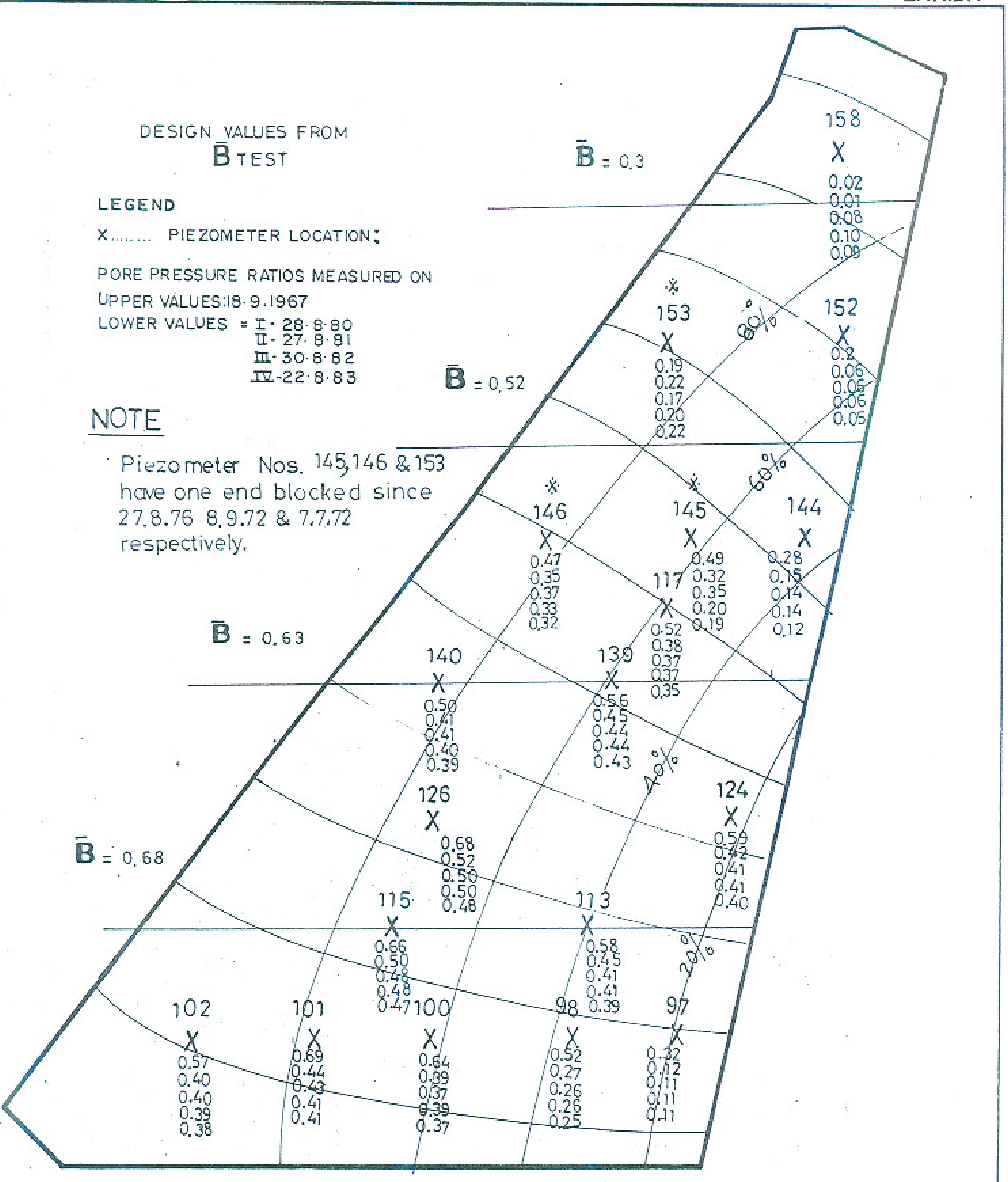


LEGEND

ANTICLINE
SYNCLINE
MONOCLINE
LOCATION OF SLIPS
SEISMIC OBS

MANGLA DAM PROJECT

RESERVOIR PLAN SHOWING LOCATION OF SLIPS

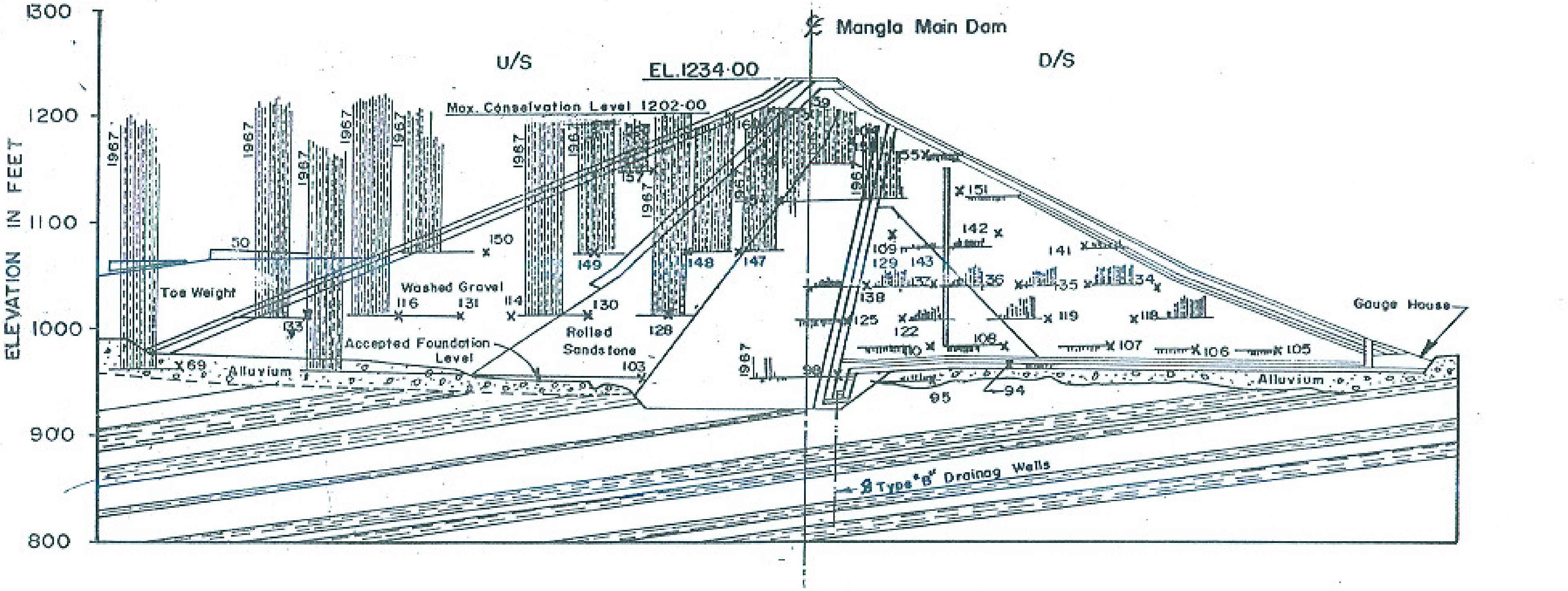


MAIN DAM ROLLED CLAY CORE PORE PRESSURE RATIO (ru)

Scale: 1" = 33.3 FT

DMO / M / I - 10

Riaz



X-SECTION AT CH.77+50

Scale: 0 100 200 Ft.
1" = 100'

LEGEND

X Hydraulic piezometer
+ Electrical piezometer
Sandstone
Clay

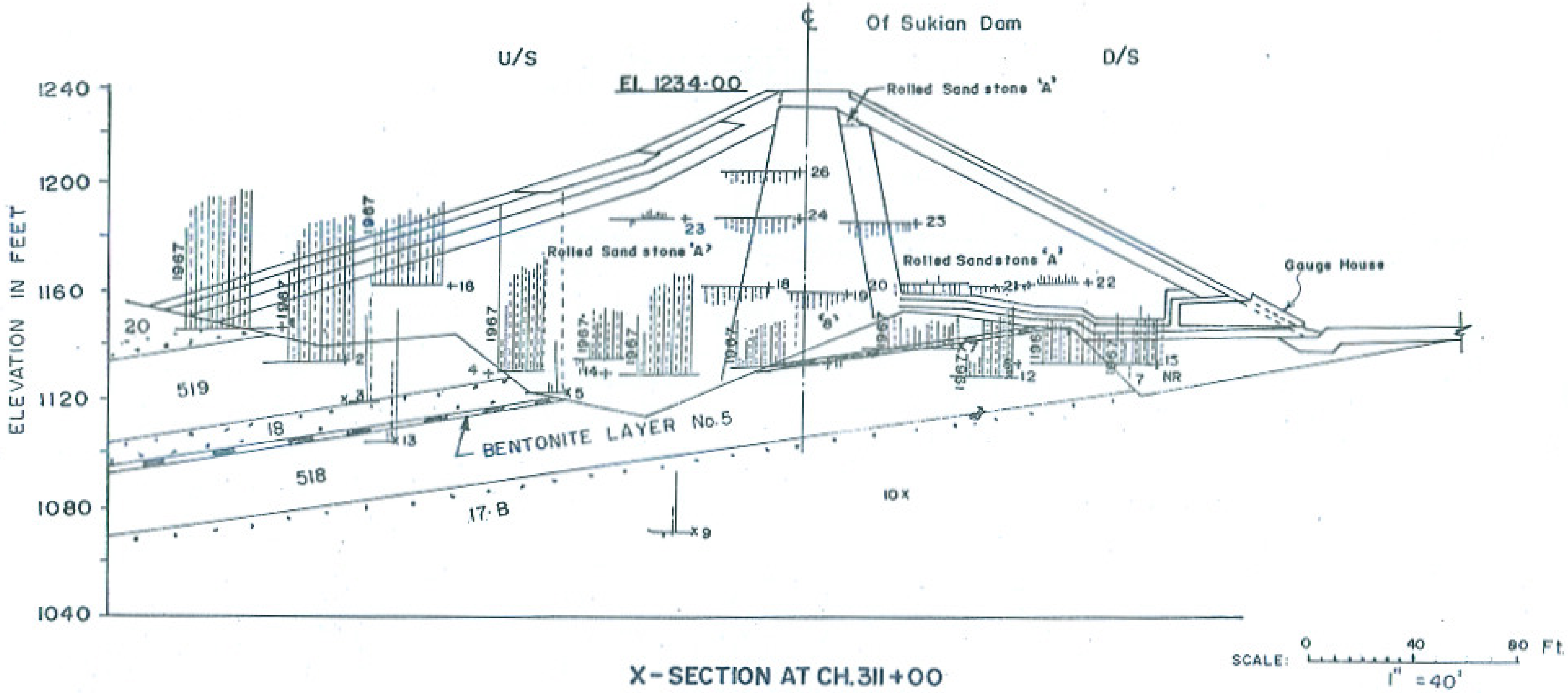
Maximum piezometric height at Reservoir Level·1202 during

1967	1977
1967	1978
1969	1980
1972	1981
1973	1982
1975	1983
1976	

MANGLA—MAIN DAM

GEOLOGICAL X-SECTION AT CH.77+50 SHOWING U/S & D/S SHOULDER PIEZO LEVELS

Ghafoor/ 7.65



X-SECTION AT CH. 311+00

LEGEND

X Hydraulic piezometer
\+ Electrical piezometer
Sand stone.
519 Clay

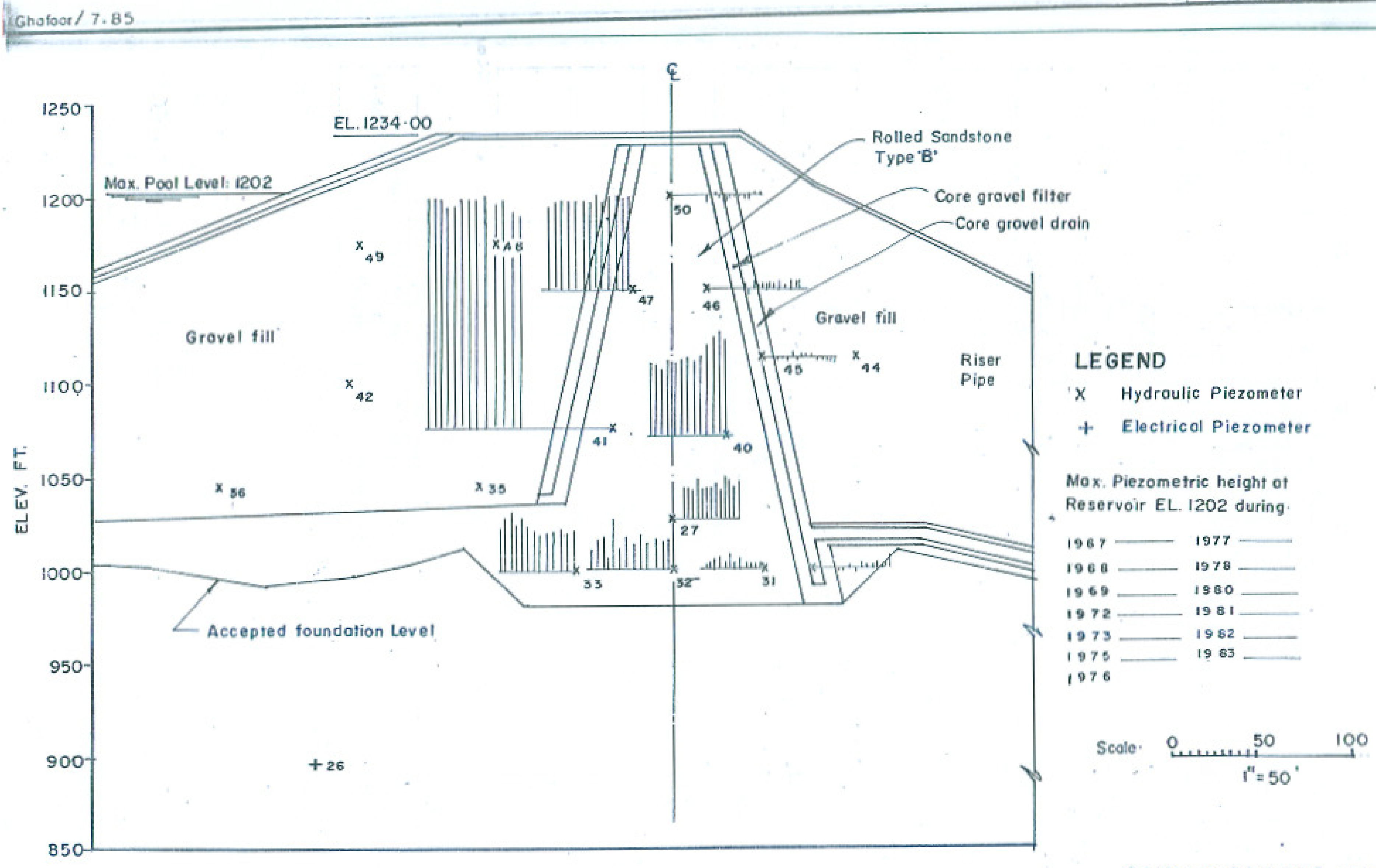
Maximum piezometric heights at Reservoir Level: 1202 during

1967, 1967, 1969, 1972, 1973, 1975, 1976, 1977, 1978, 1980, 1981, 1982, 1983

MANGLA - SUKIAN DAM
LOCATION OF PIEZOMETERS AT CH. 311+00

Ghafoor / 7.85

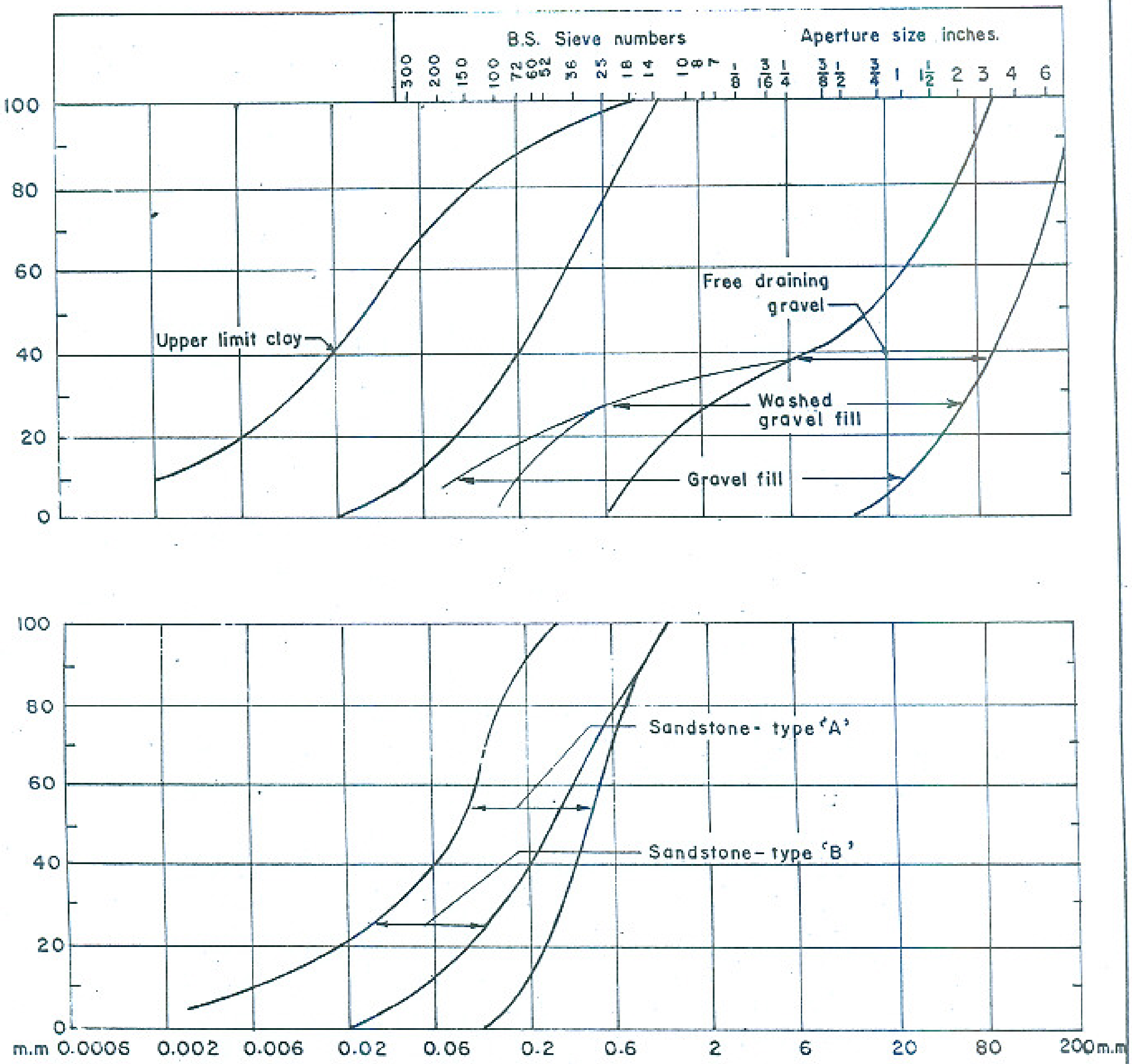
Paper No. 490 EXHIBIT- 7



X- SECTION AT CH. 133+81

MANGLA DAM PROJECT
INTAKE EMBANKMENT (CORE)
PIEZOMETRIC HEIGHTS
SECTION AT CH. 133 + 81

Chalid / 3·85

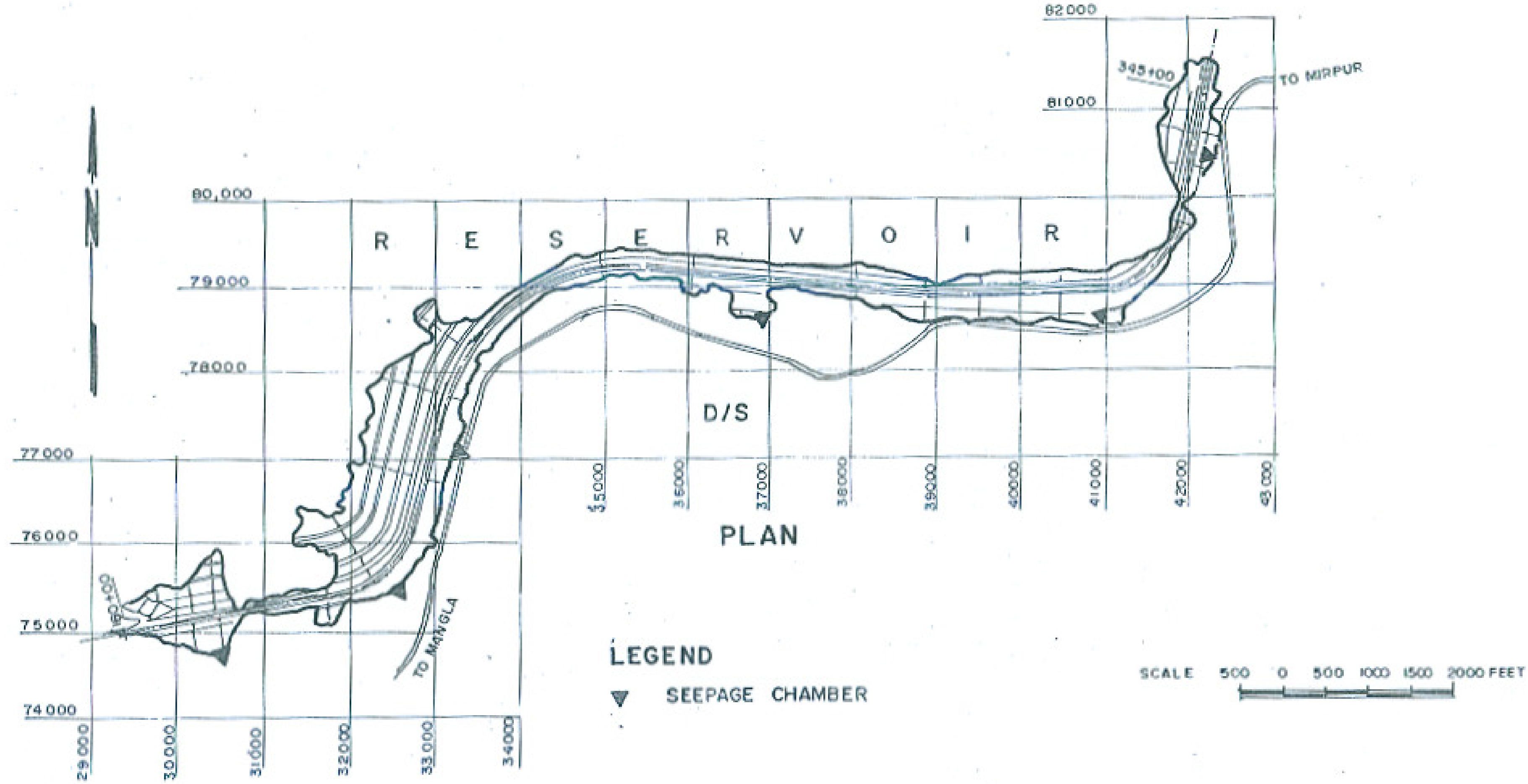


Clay fraction	Silt fraction	Fine (Sand fraction)	Medium (Sand fraction)	Coarse (Sand fraction)	Fine (Gravel fraction)	Medium (Gravel fraction)	Coarse (Gravel fraction)	Cobbles

MANGLA DAM PROJECT
MAIN DAM
FILL MATERIALS

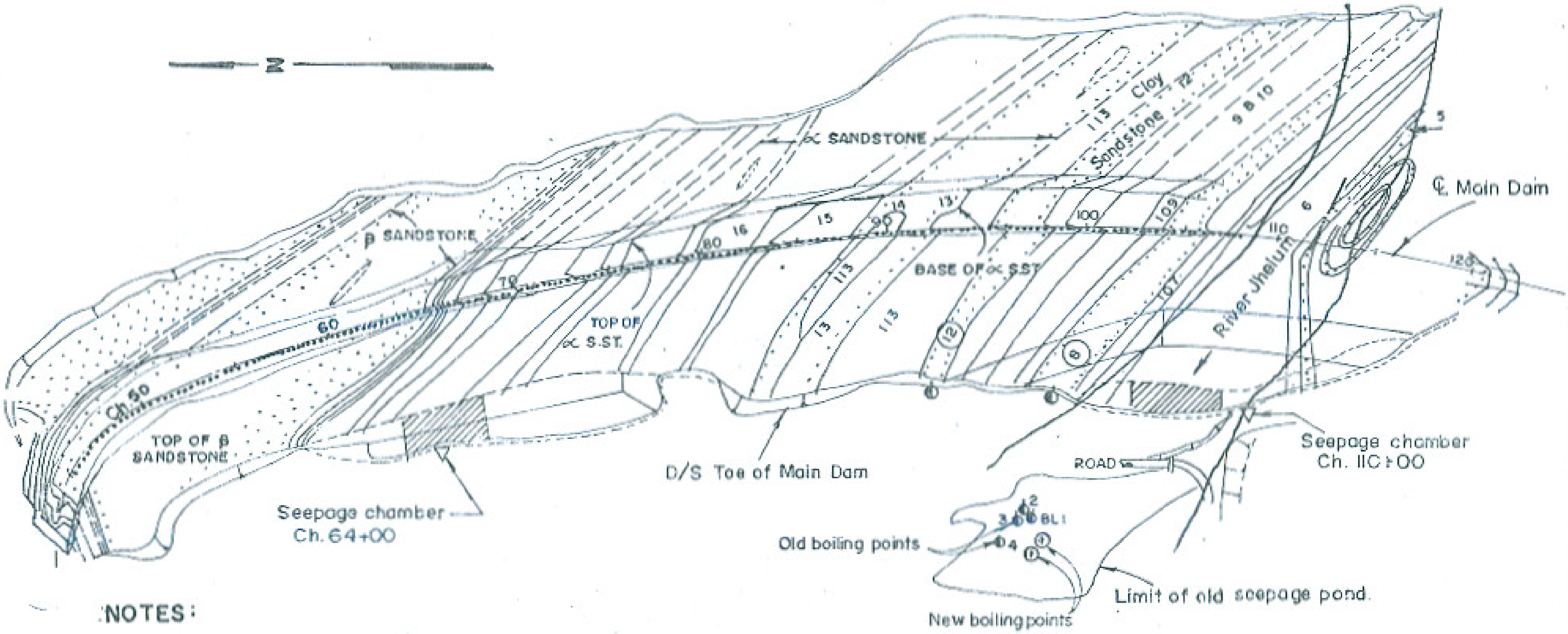
Reference: Mangla completion report vol. VI, fig. 5

Paper No. 490 EXHIBIT- 9



MANGLA DAM PROJECT
SUKIAN DAM GENERAL PLAN
SEEPAGE CHAMBER LOCATIONS

KHALIL / 5·85



Scale: 1" = 800'

NOTES:

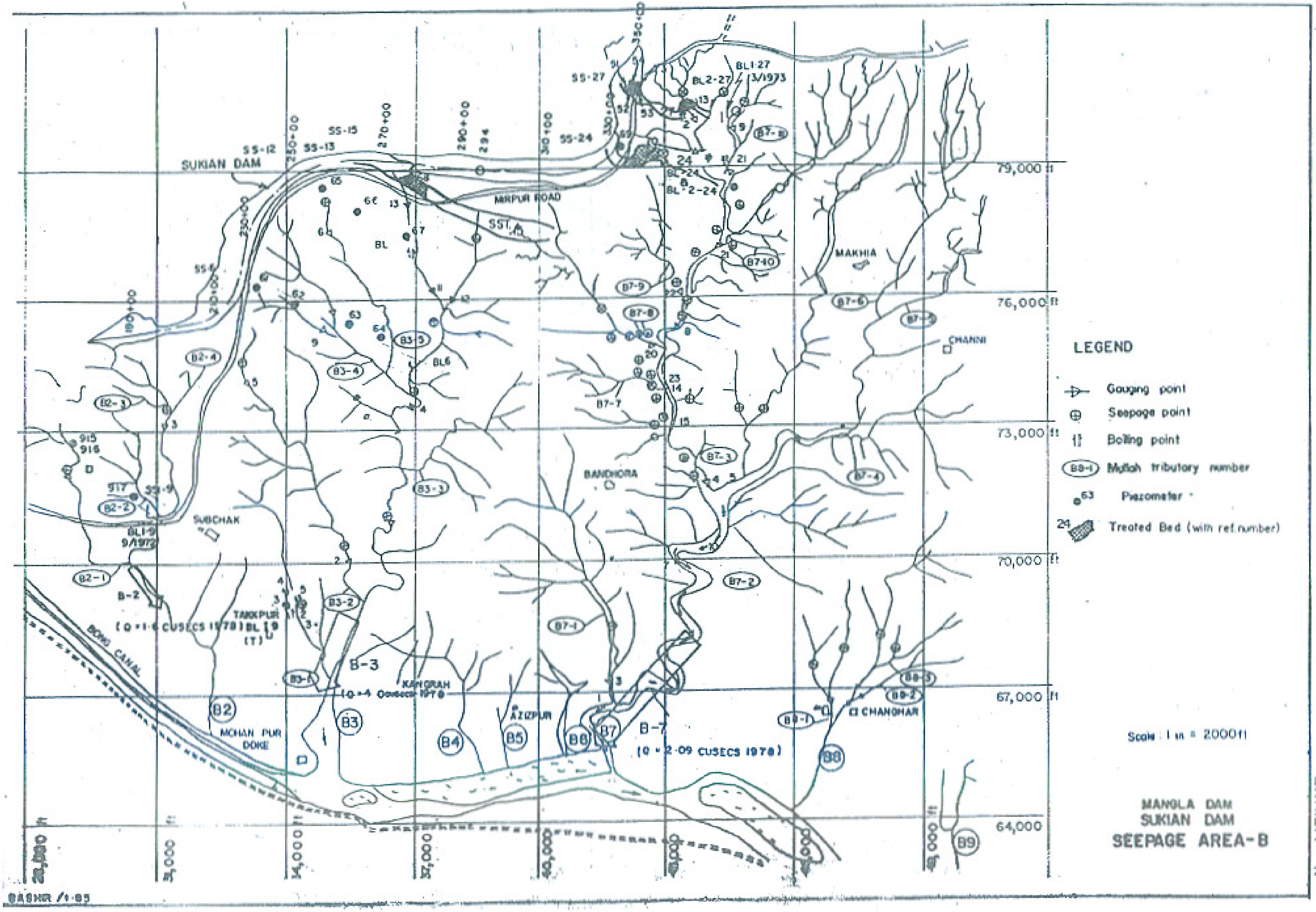
1. Wells were normally spaced at 25ft centers.
2. Additional intermediate type 'B' wells were drilled in main dam at the position given in table.

INTERMEDIATE WELLS (TYPE 'B')	
Stations	Depth in ft.
CH. 42+00 TO 112+25	155 FT. DEEP EACH.
45+00 TO 46+87.5	DEPTH VARIES BETWEEN 88.5 & 95.65
45+00 TO 47+00	84 FT. — 90 FT.
49+50 TO 53+00	65 FT. — 80 FT.
62+00 TO 65+00	155 FT.
65+00 TO 66+00	155 FT.
81+00 TO 84+50	155 FT

MANGLA DAM PROJECT
MAIN DAM-GENERAL GEOLOGY AND D/S BOILING LOCATIONS

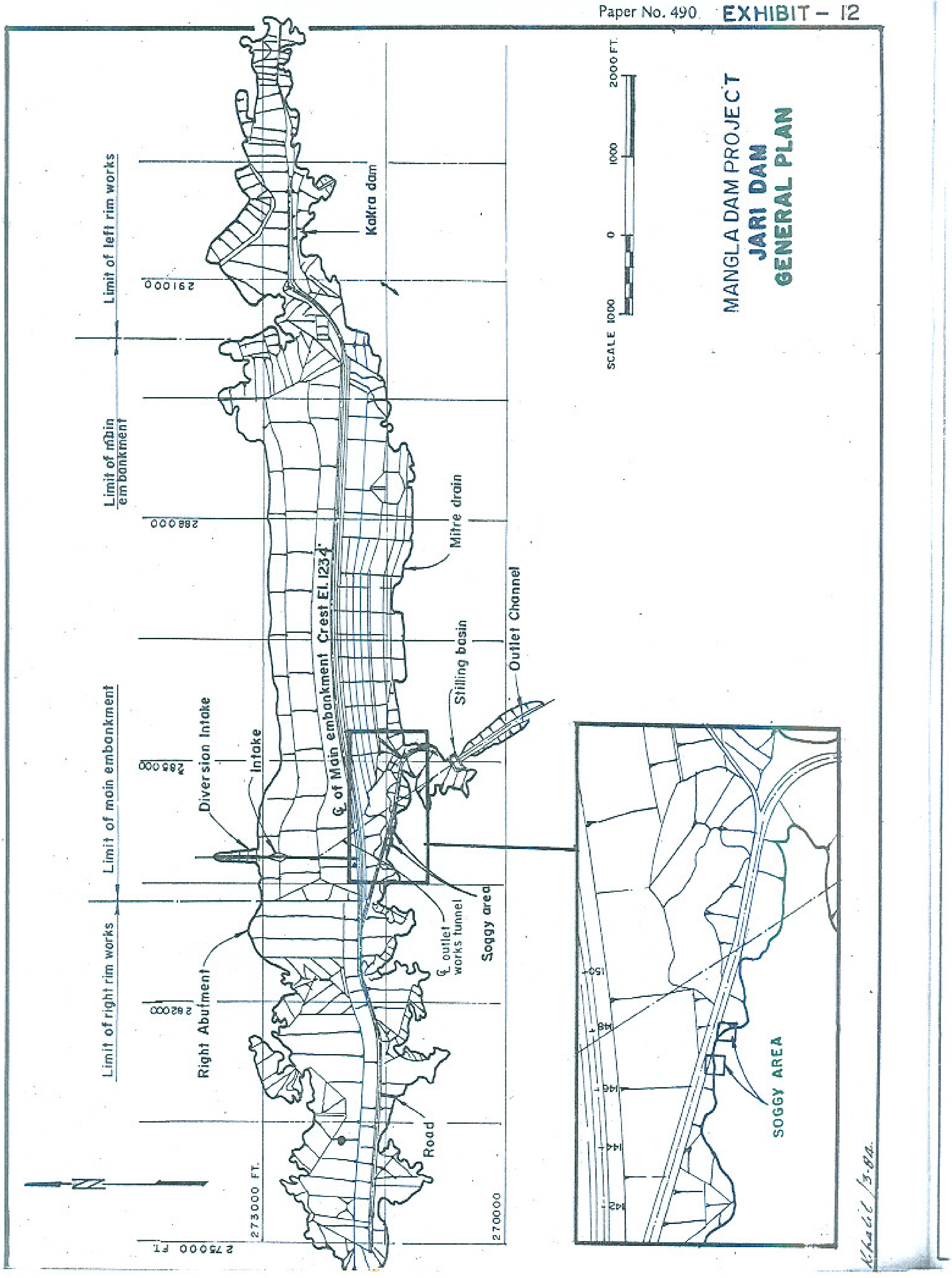
Khalil / 2.85

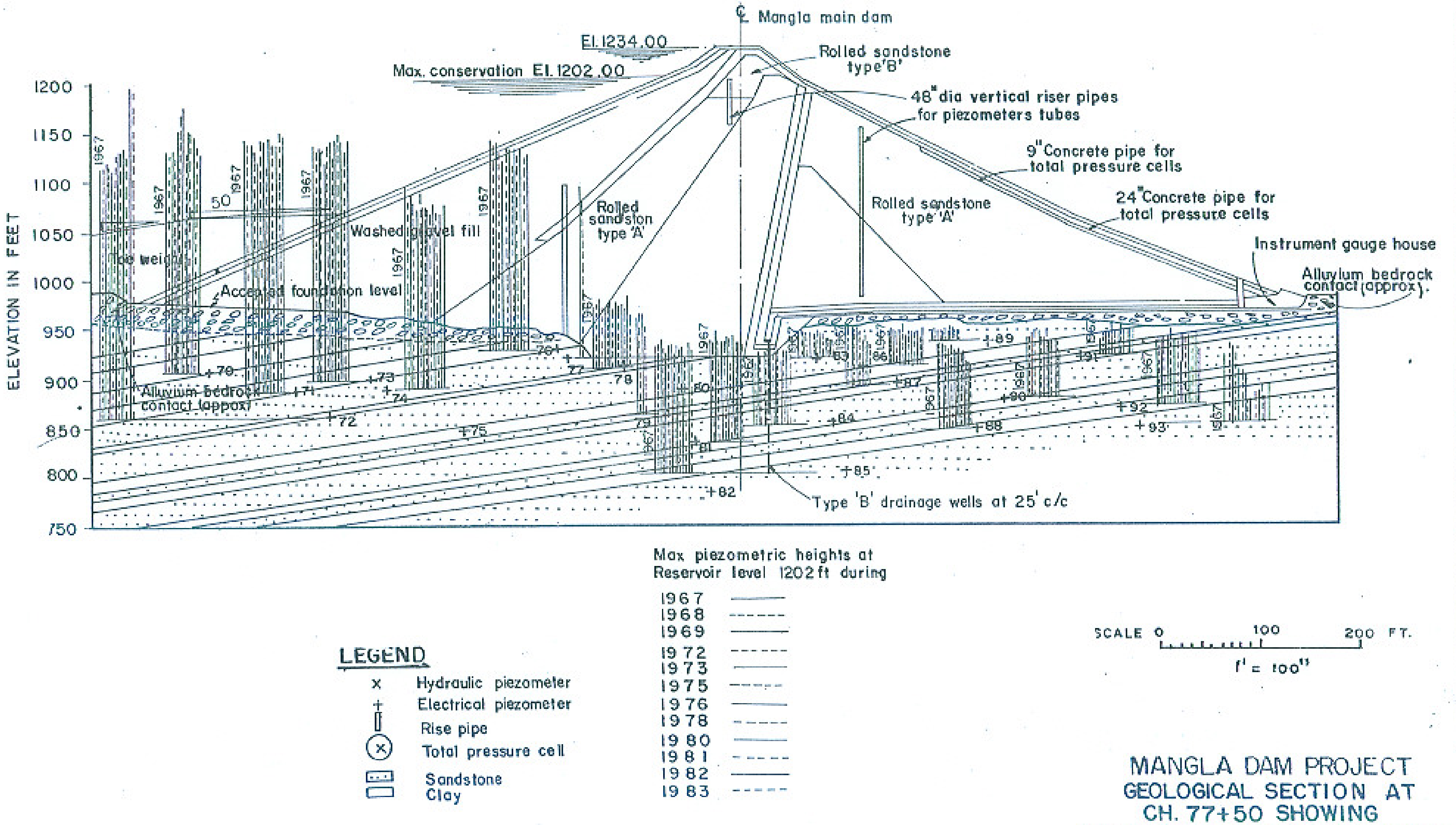
EXHIBIT - II



LEGEND
Gauging point
Seepage point
Boiling point
Mullah tributary number
Piezometer
Treated Bed (with ref.number)
Scale : 1 in = 2000ft
MANGLA DAM
SUKIAN DAM
SEEPAGE AREA-B
SUKIAN DAM
MIRPUR ROAD
MAKHIA
CHANNI
BANDHORA
SUBCHAK
TAKKPUR
KANGRAH
AZIZPUR
MOHAN PUR DOKE
CHANGHAR
BONG CANAL
B-2 (Q = 1.6 CUSECS 1978)
B-3 (Q = 4 Cusecs 1978)
B-7 (Q = 2.09 CUSECS 1978)
BL1-27 3/1973
BL1-9 9/1972
79,000 ft
76,000 ft
73,000 ft
70,000 ft
67,000 ft
64,000
BASHIR /1-85

Paper No. 490. **EXHIBIT – 12**

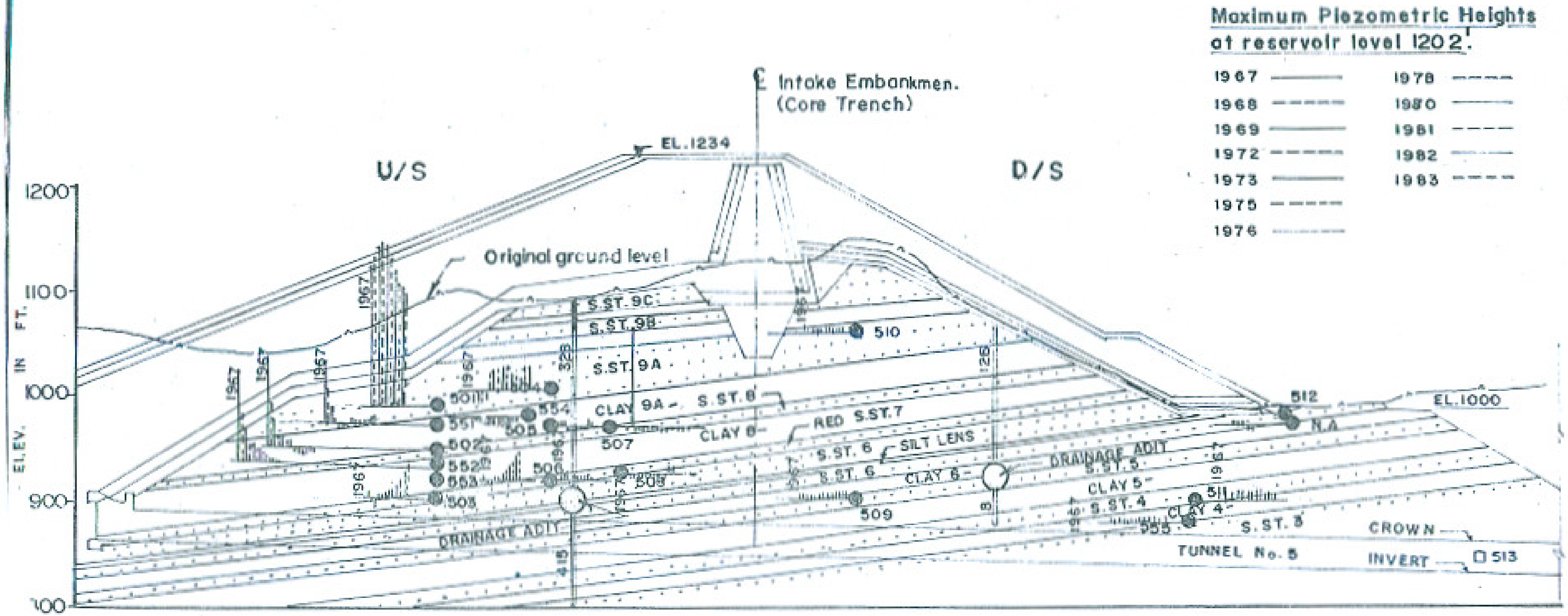


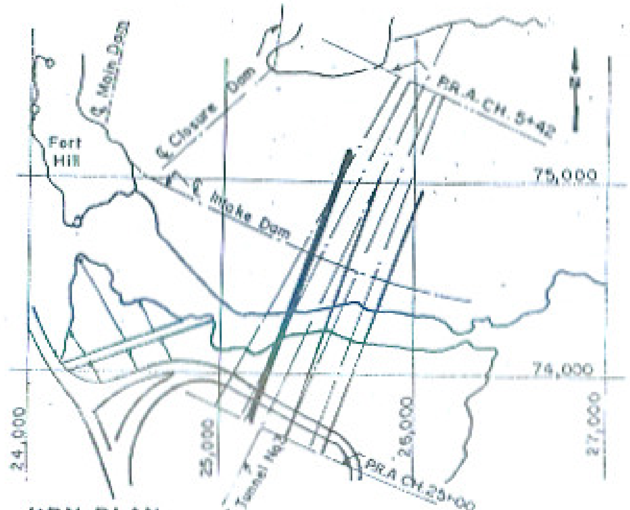


MANGLA DAM PROJECT
GEOLOGICAL SECTION AT CH. 77+50 SHOWING FOUNDATION PIEZOMTRIC HEIGHTS

Zahid / 7-85

Paper No. 490 EXHIBIT I-13





KEY PLAN
Scale 1:1000

LEGEND

- Sandstone
- Clay
- Piezometer

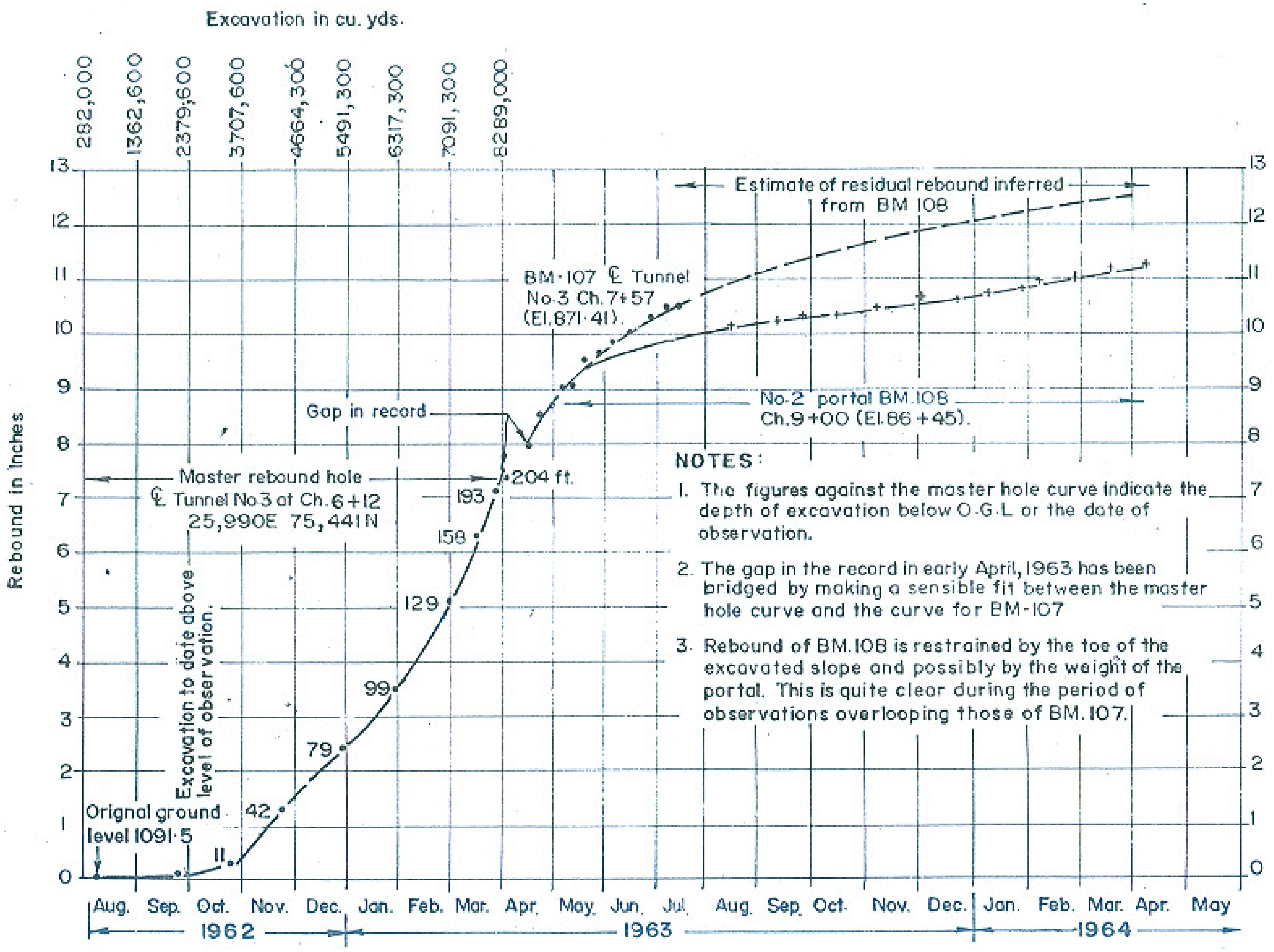
Scale : 1" = 160'

MANGLA DAM PROJECT
INTAKE EMBANKMENT
FOUNDATION PIEZOS. SEC.AT.Ch:136+55

KHALIL / 3.85

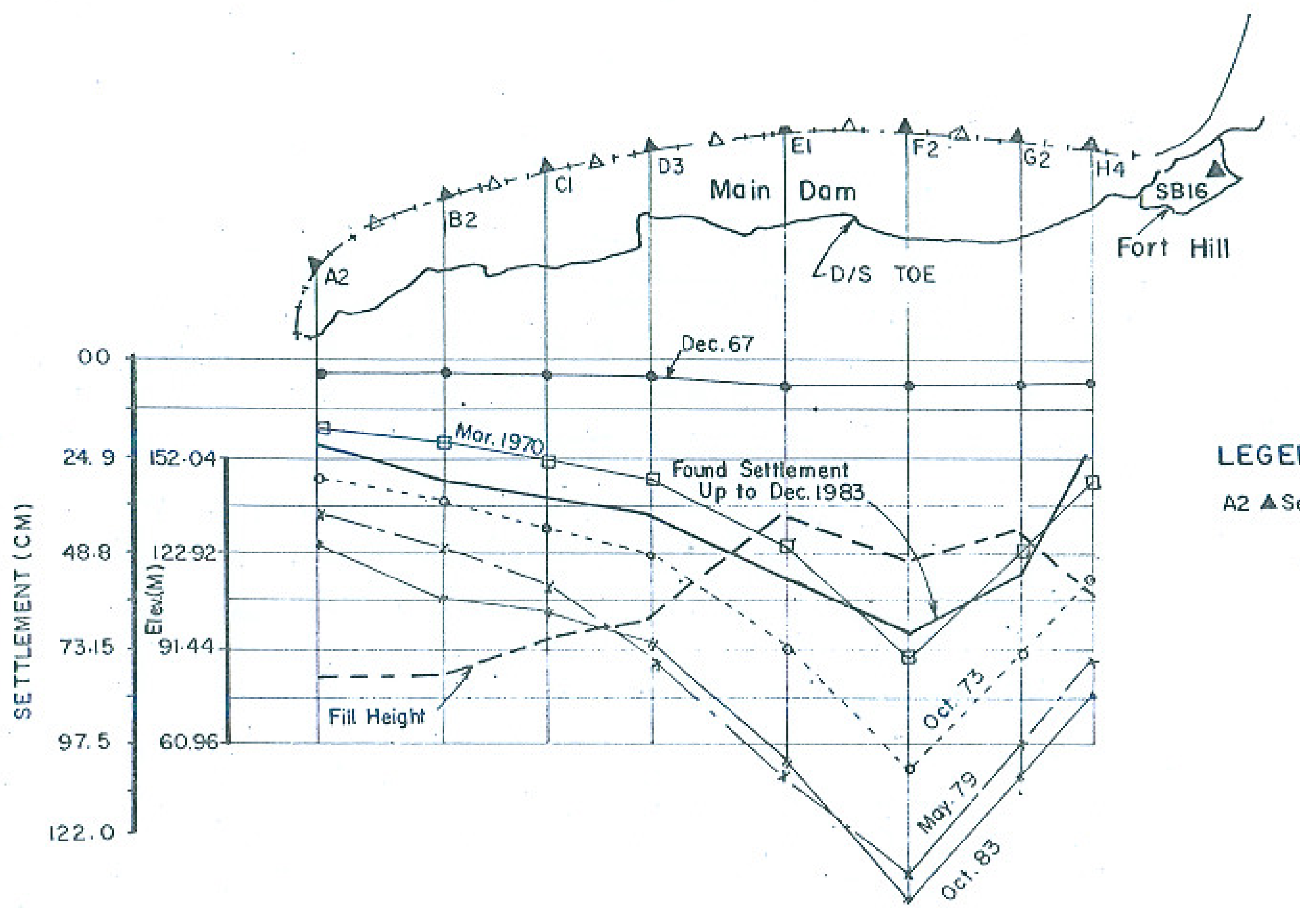
EXHIBIT-15

KEY PLAN
Scale 1:1000
KHALIL / 3.85



REBOUND OBSERVATIONS, INTAKE AREA.

REFERENCE Mangla Completion Report Vol. XV Fig. 16.



LEGEND

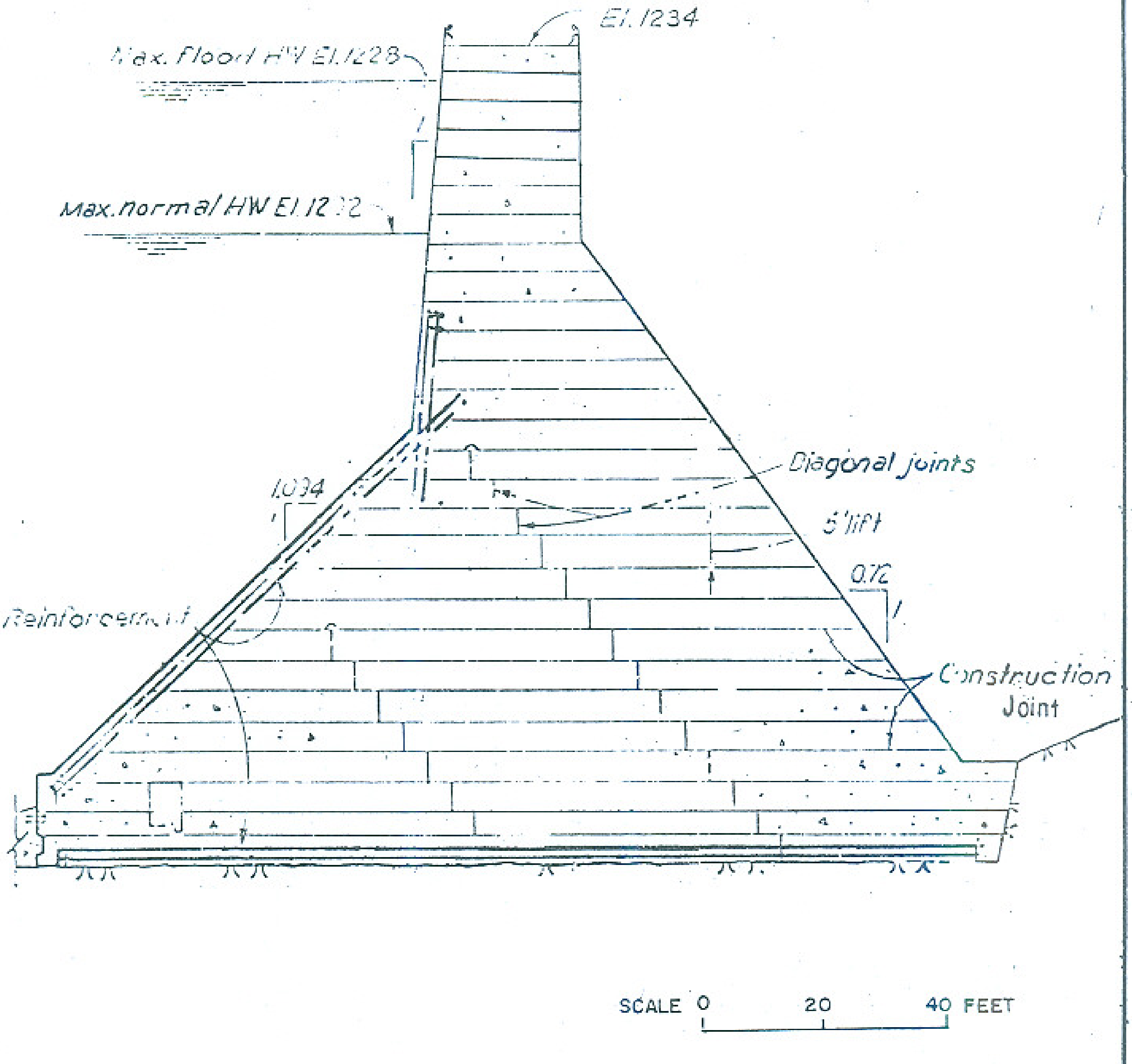
A2 ▲ Settlement Gauge / number

MANGLA DAM PROJECT

MAIN DAM

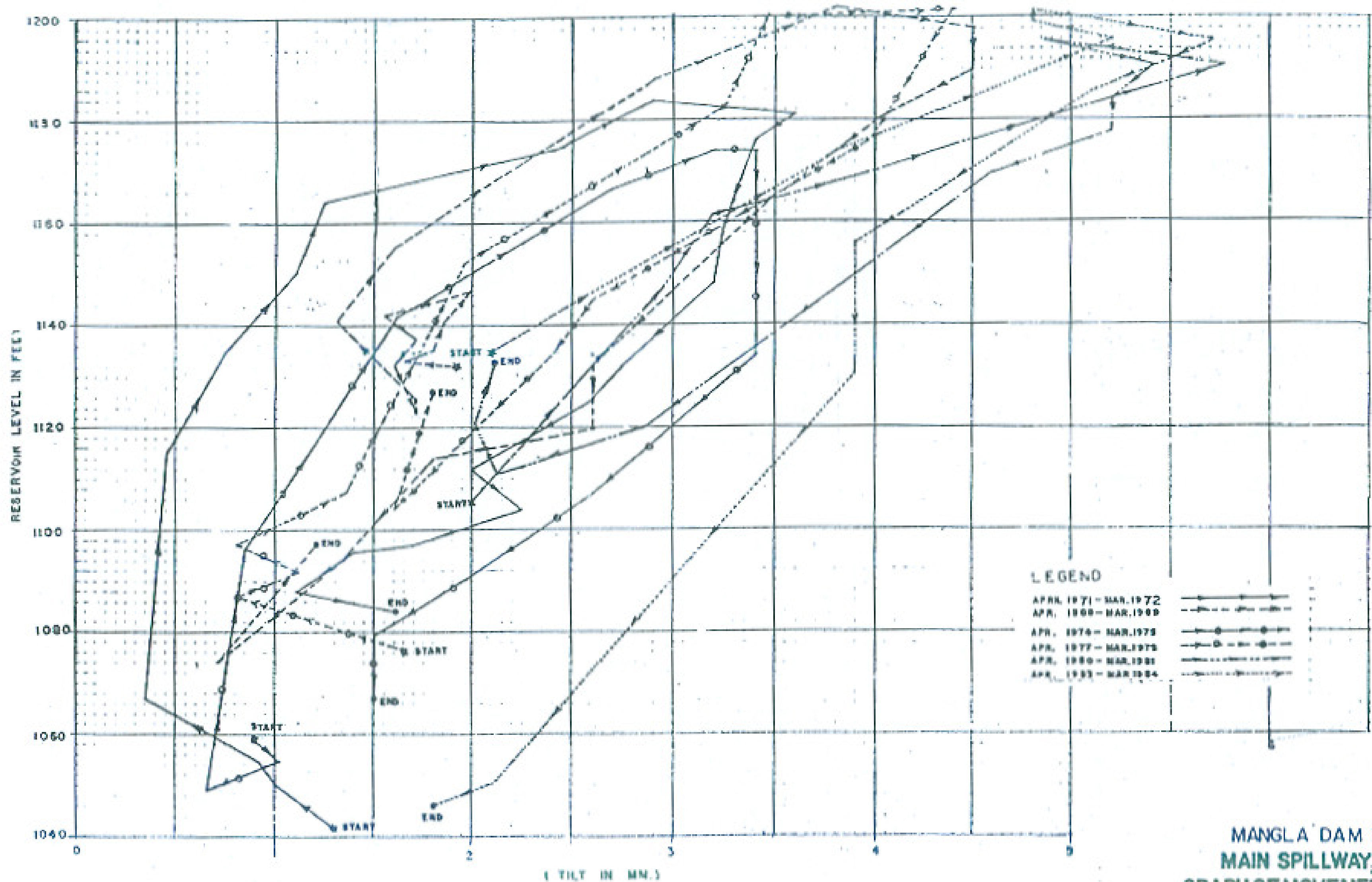
SETTLEMENT PATTERN

KHALIL/ 3 85



MANGLA SPILLWAY HEADWORKS

CROSS SECTION OF ABUTMENT MONOLITH



MANGLA DAM PROJECT

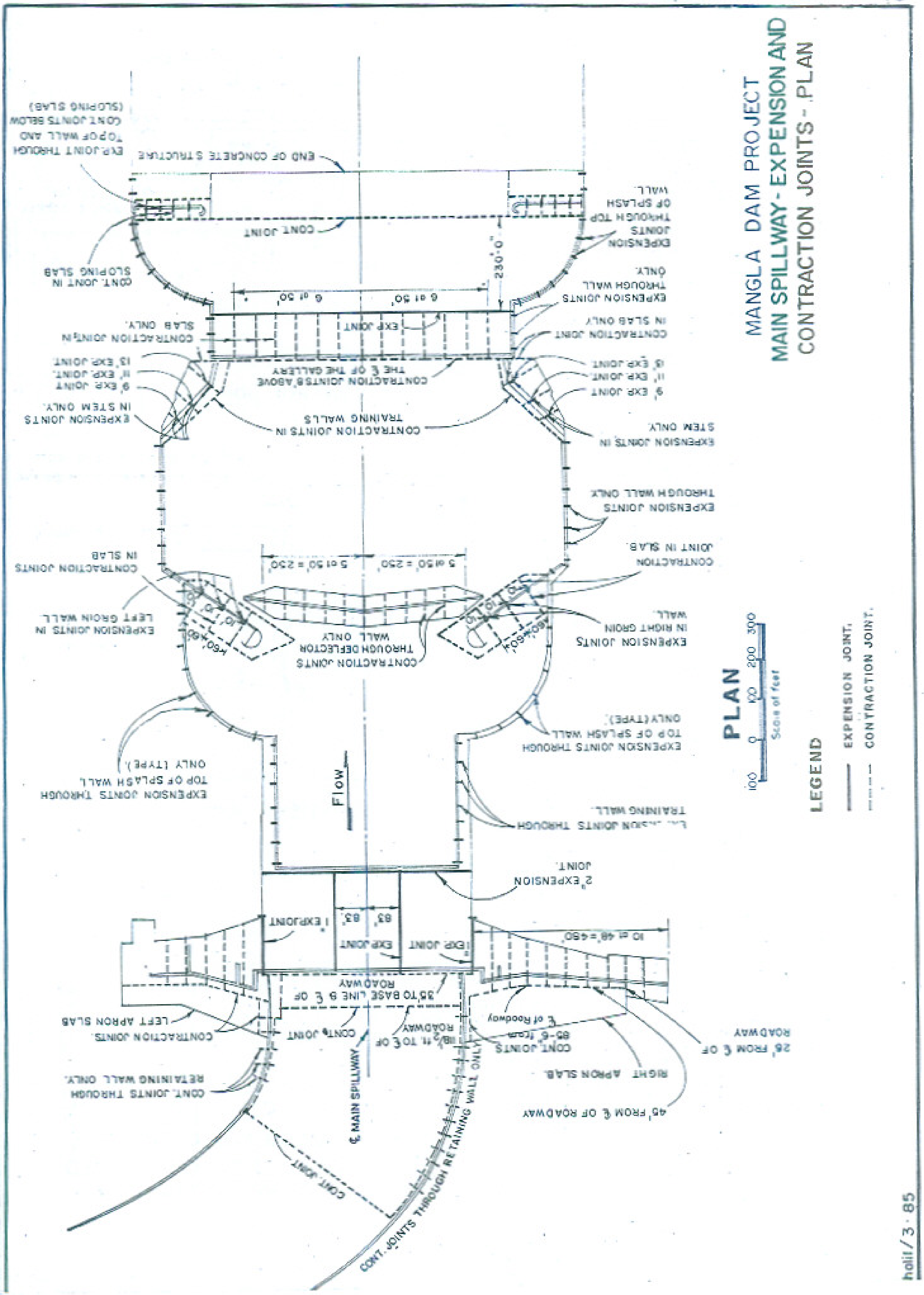
MAIN SPILLWAY, MAIN DAM

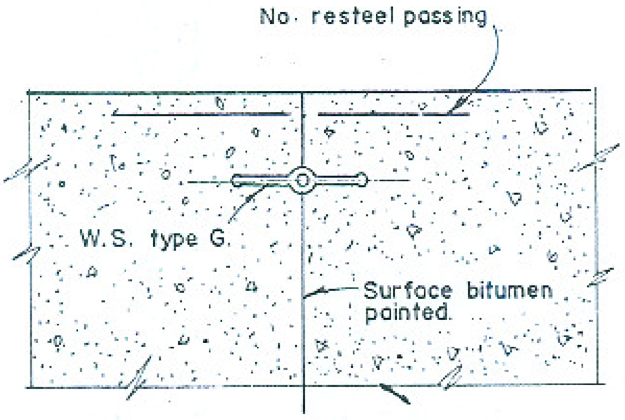
GRAPH OF MOVEMENT OF PLUMBLINE

(NORMAL) No- I

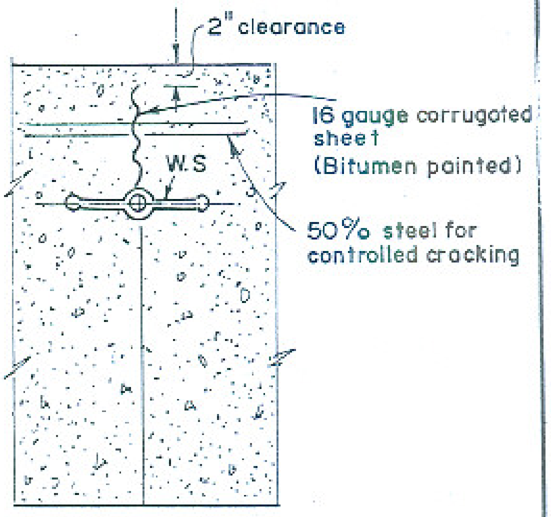
EXHIBIT - 18

hmed / 6.84

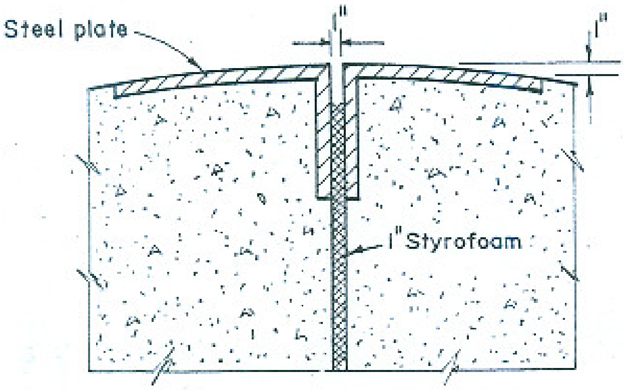

MANGLA DAM PROJECT
MAIN SPILLWAY - EXPENSION AND
CONTRACTION JOINTS - PLAN
PLAN
Scale of feet
100 0 100 200 300
LEGEND
EXPENSION JOINT.
CONTRACTION JOINT.
END OF CONCRETE STRUCTURE
EXP. JOINT THROUGH TOP OF WALL AND CONT JOINTS BELOW (SLOPING SLAB)
CONT. JOINT
CONT. JOINT IN SLOPING SLAB
EXPENSION JOINTS THROUGH TOP OF SPLASH WALL.
EXPENSION JOINTS THROUGH WALL ONLY.
230'-0"
6 at 50'
EXP JOINT
CONTRACTION JOINTS IN SLAB ONLY.
CONTRACTION JOINT IN SLAB ONLY.
CONTRACTION JOINTS ABOVE THE ℄ OF THE GALLERY
13' EXP. JOINT
11' EXP. JOINT.
9' EXP. JOINT
EXPENSION JOINTS IN STEM ONLY.
CONTRACTION JOINTS IN TRAINING WALLS
EXPENSION JOINTS THROUGH WALL ONLY.
CONTRACTION JOINTS IN SLAB
CONTRACTION JOINT IN SLAB.
5 at 50' = 250'
EXPENSION JOINTS IN LEFT GROIN WALL.
EXPENSION JOINTS IN RIGHT GROIN WALL.
CONTRACTION JOINTS THROUGH DEFLECTION WALL ONLY
EXPENSION JOINTS THROUGH TOP OF SPLASH WALL ONLY (TYPE).
Flow
EXPENSION JOINTS THROUGH TRAINING WALL.
2' EXPENSION JOINT.
1' EXP JOINT
83' 83'
EXP JOINT
10 at 48' = 480'
35' TO BASE LINE & ℄ OF ROADWAY
LEFT APRON SLAB
CONTRACTION JOINTS.
CONT. JOINT
118½ ft. TO ℄ OF ROADWAY
CONT. JOINTS 85'-6" from ℄ of Roadway
26' FROM ℄ OF ROADWAY
RIGHT APRON SLAB.
45' FROM ℄ OF ROADWAY
CONT. JOINTS THROUGH RETAINING WALL ONLY.
℄ MAIN SPILLWAY
CONT. JOINTS THROUGH RETAINING WALL ONLY
CONT. JOINT
halil/3·85



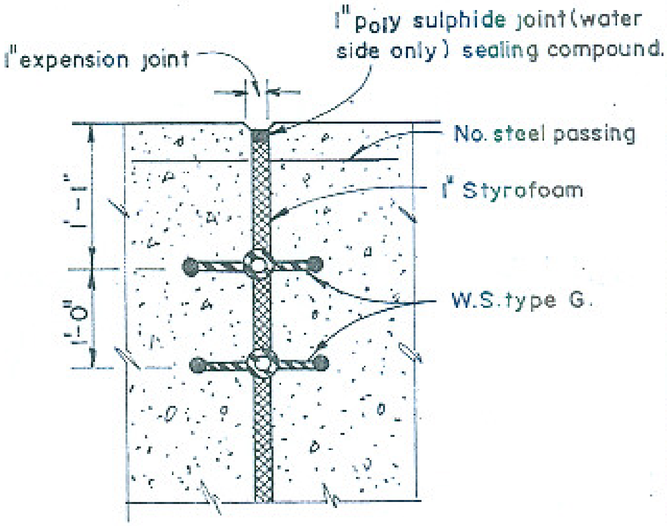
CONTRACTION JOINT



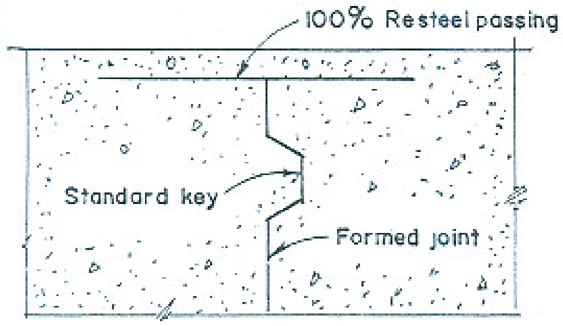
CONTROL JOINT



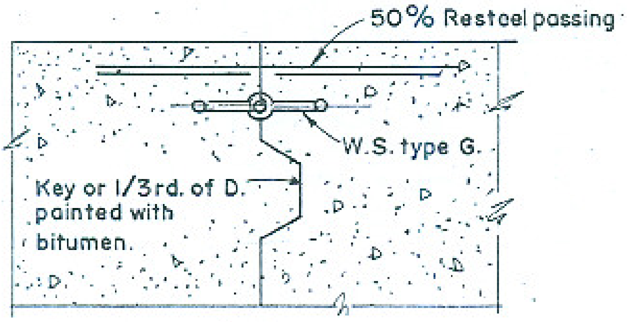
OPEN JOINT HEADWORKS MONOLITH



EXPANSION JOINT



CONSTRUCTION JOINT

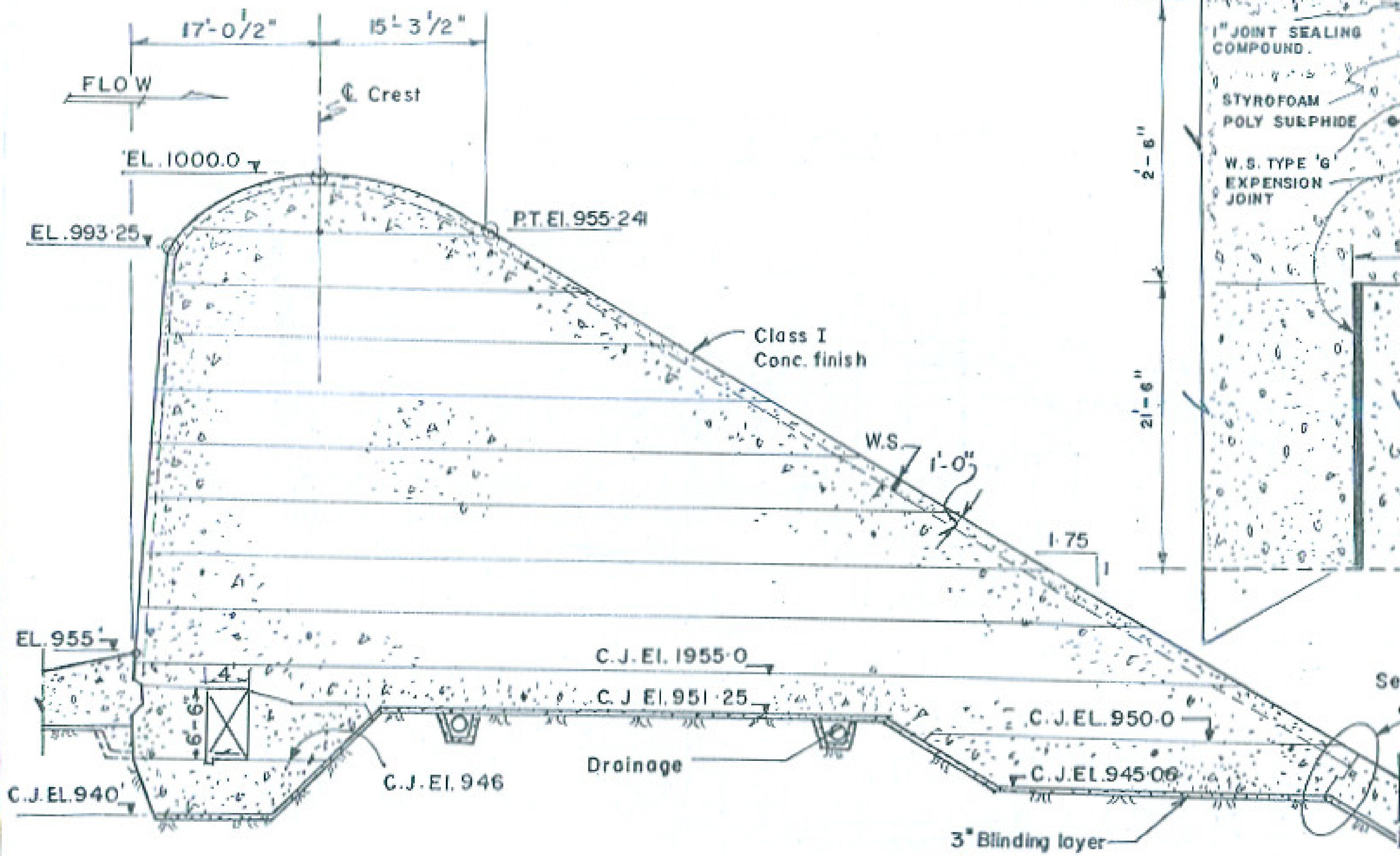


CONSTRUCTION CONTROL JOINT

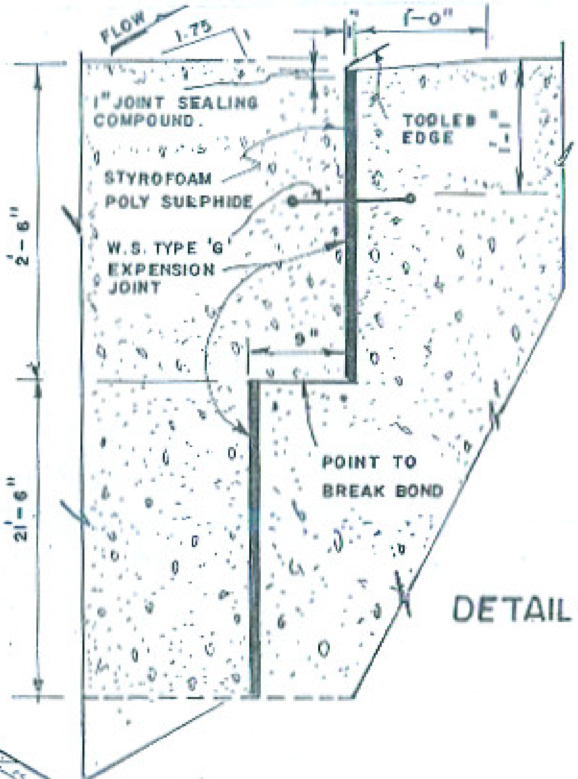
MANGLA DAM PROJECT

DETAILS OF JOINTS USED IN MAIN SPILLWAY

KHALIL / 3-85



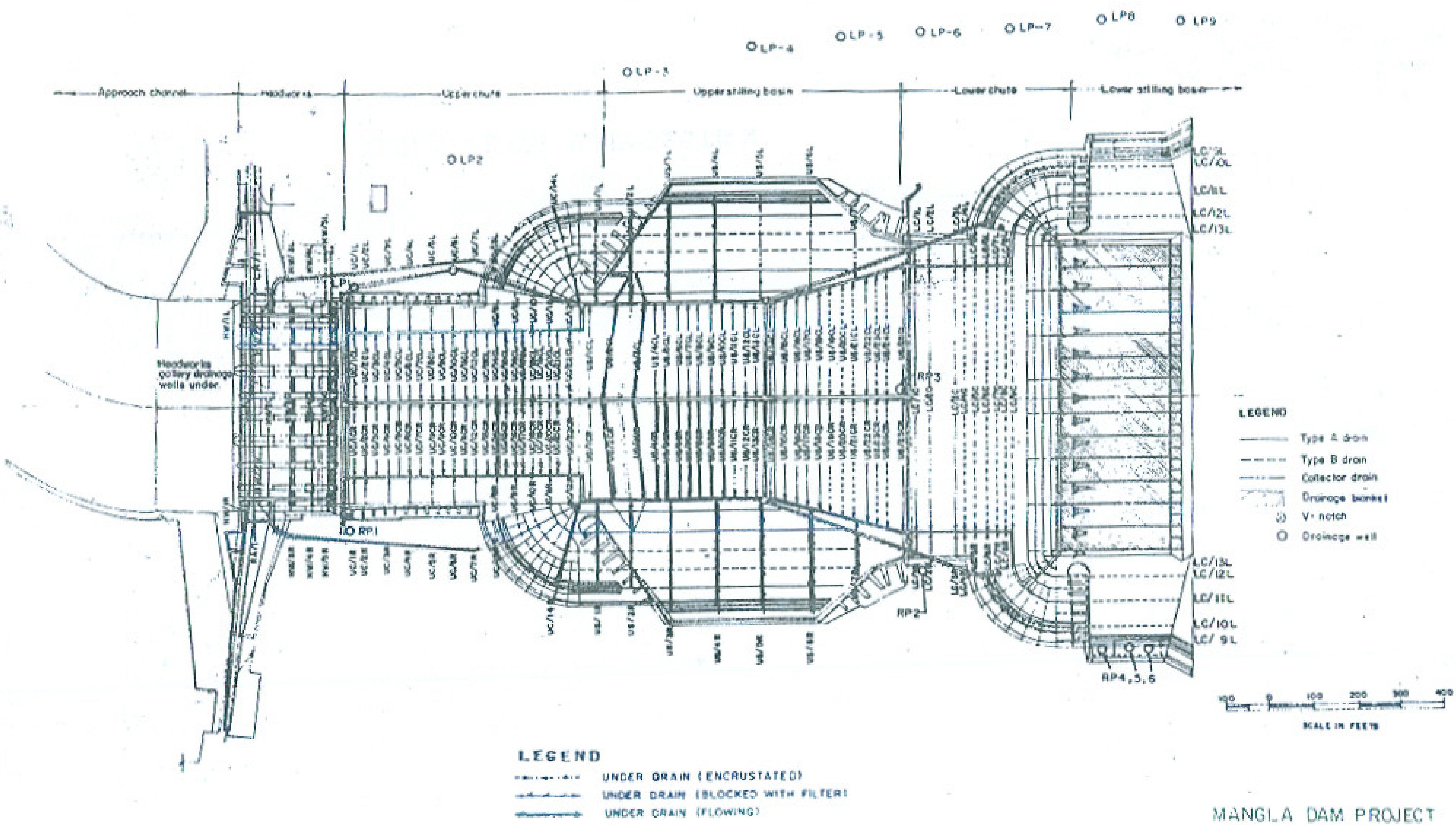
MAIN SPILLWAY WEIR SECTION



DETAIL-'A'

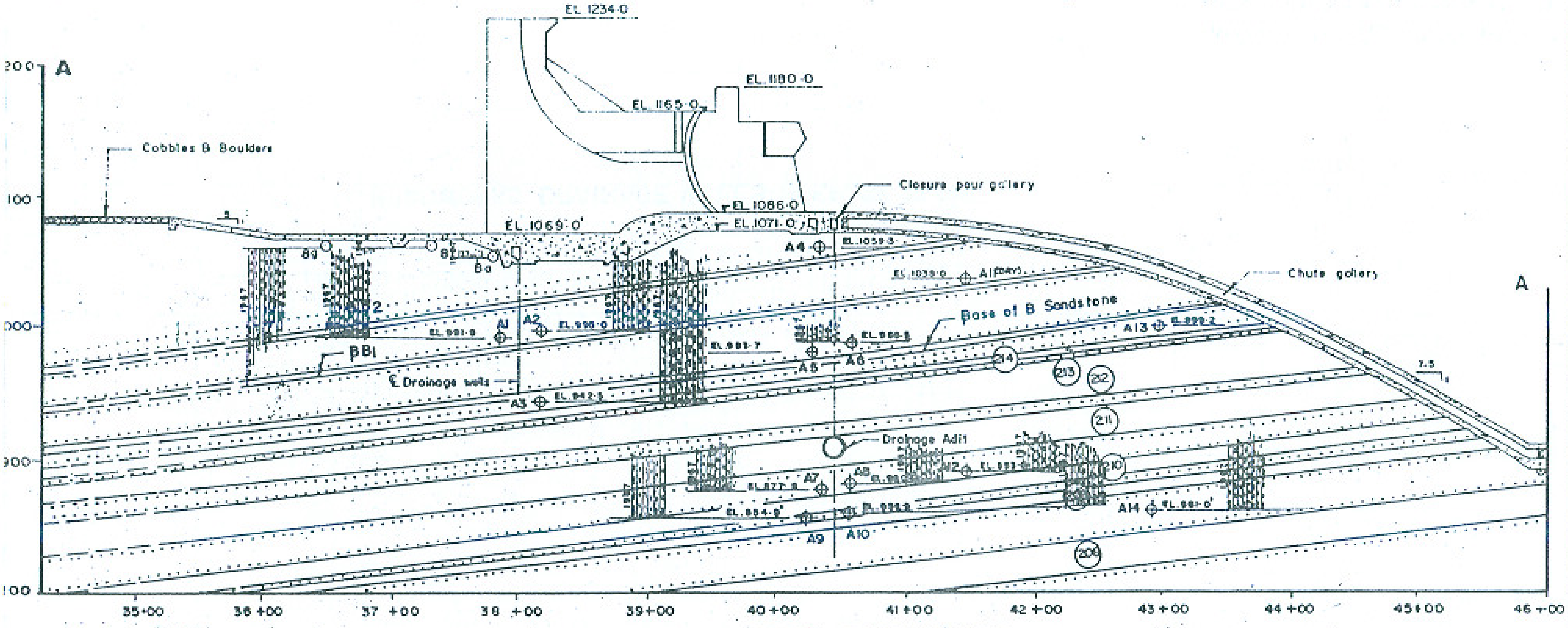
MANGLA-MAIN SPILLWAY
CROSS SECTION OF WEIR SHOWING
DETAIL OF EXPENSION JOINT BETWEEN
LOWER CHUTE & WEIR

Khalid / 6.85



MANGLA DAM PROJECT
MAIN SPILLWAY
UNDERDRAIN SYSTEM- PLAN

Zahid / 3.85



L-SECTION A-A AT LT. OF ℄ OF MAIN SPILLWAY

LEGEND

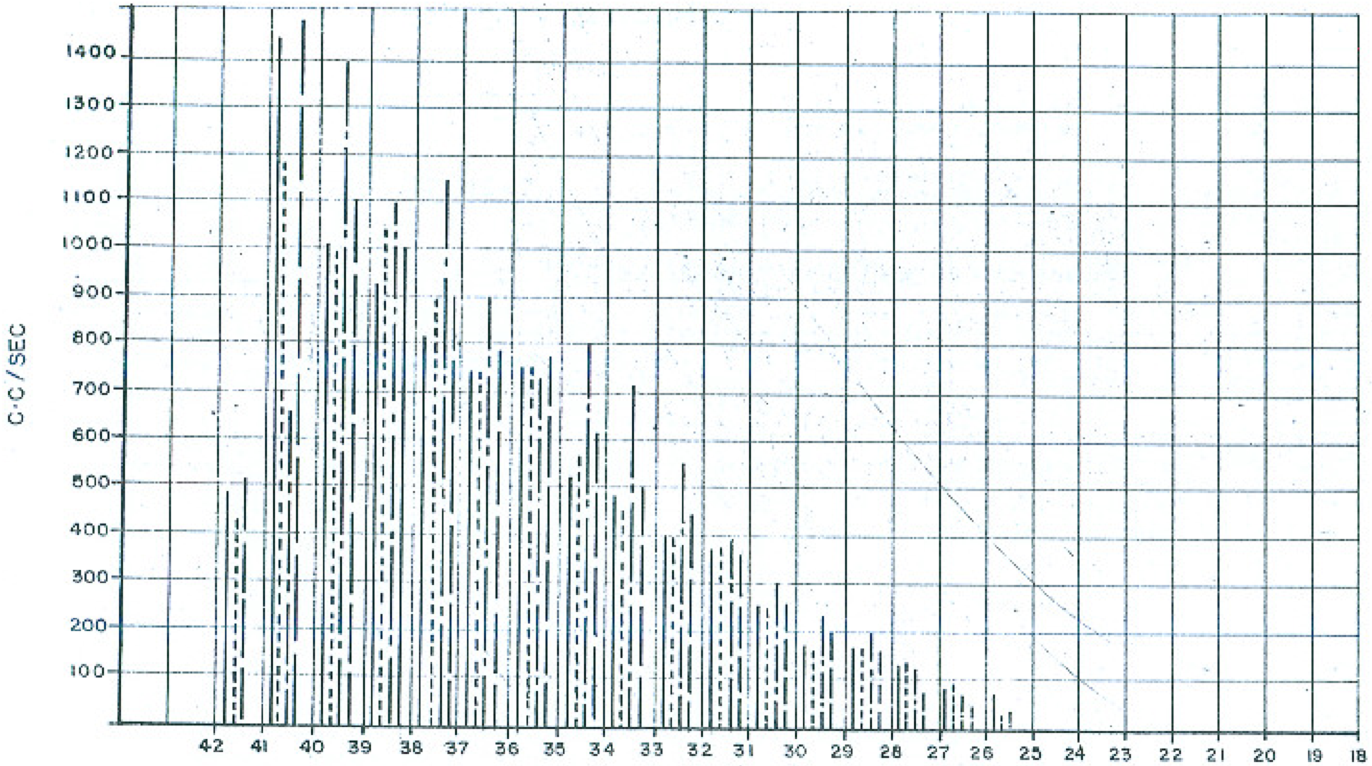
- Sandstone
- Clay
- Piezometer

Max. piezometric height at Reservoir level 1202 during 1967 to 1983

Scale: 1" = 80'

MANGLA DAM PROJECT
MAIN SPILLWAY
HEADWORKS/UPPER CHUTE
GEOLOGICAL SECTION
PIEZO. HIGHTS

lq/8.85



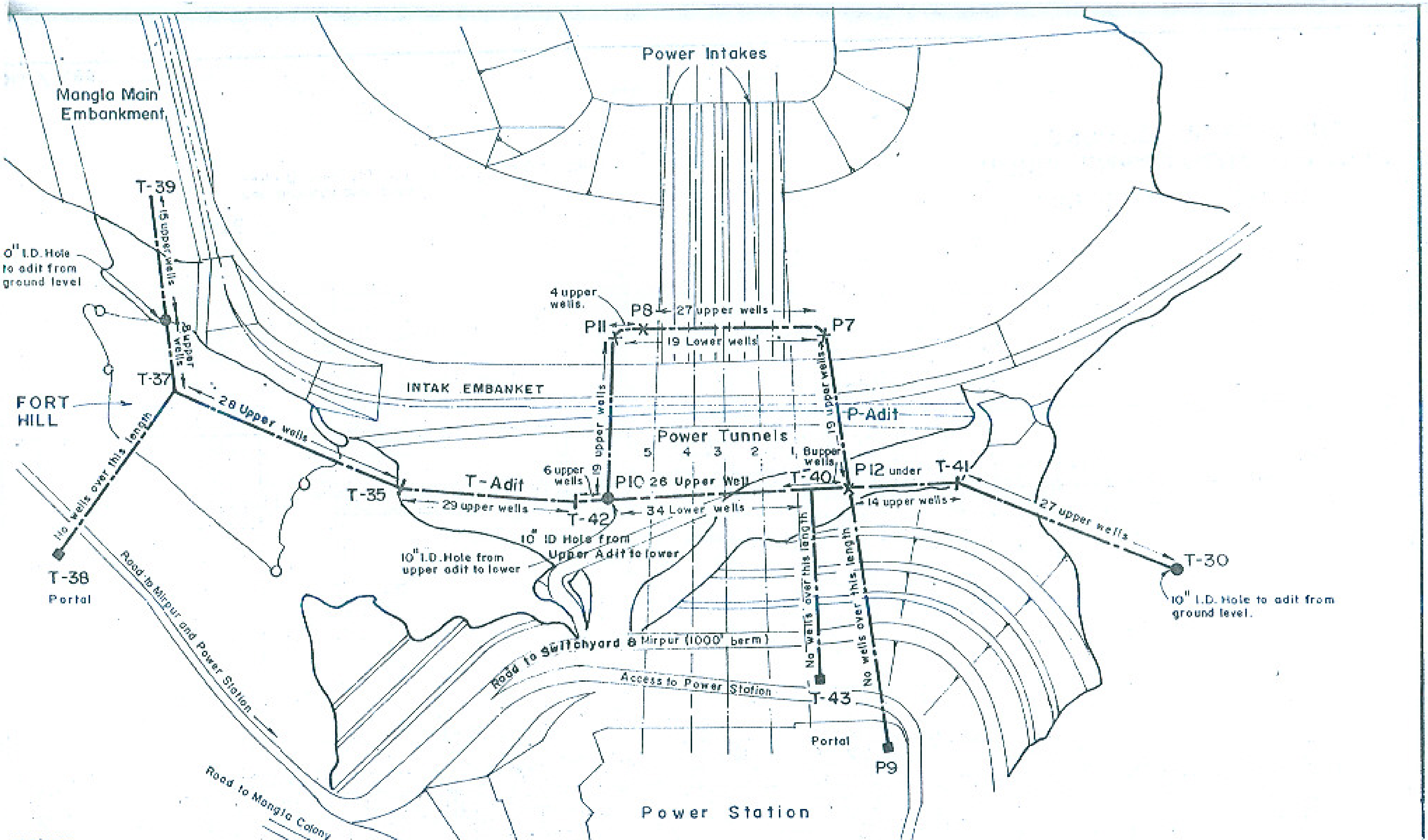
HEADWORKS DRAINAGE WELLS SEEPAGE FLOW·

MANGLA DAM PROJECT

HEADWORKS DRAINAGE WELLS

MAX· FLOW INTENSITY GRAPHS

Tariq / 3. 85



NOTES

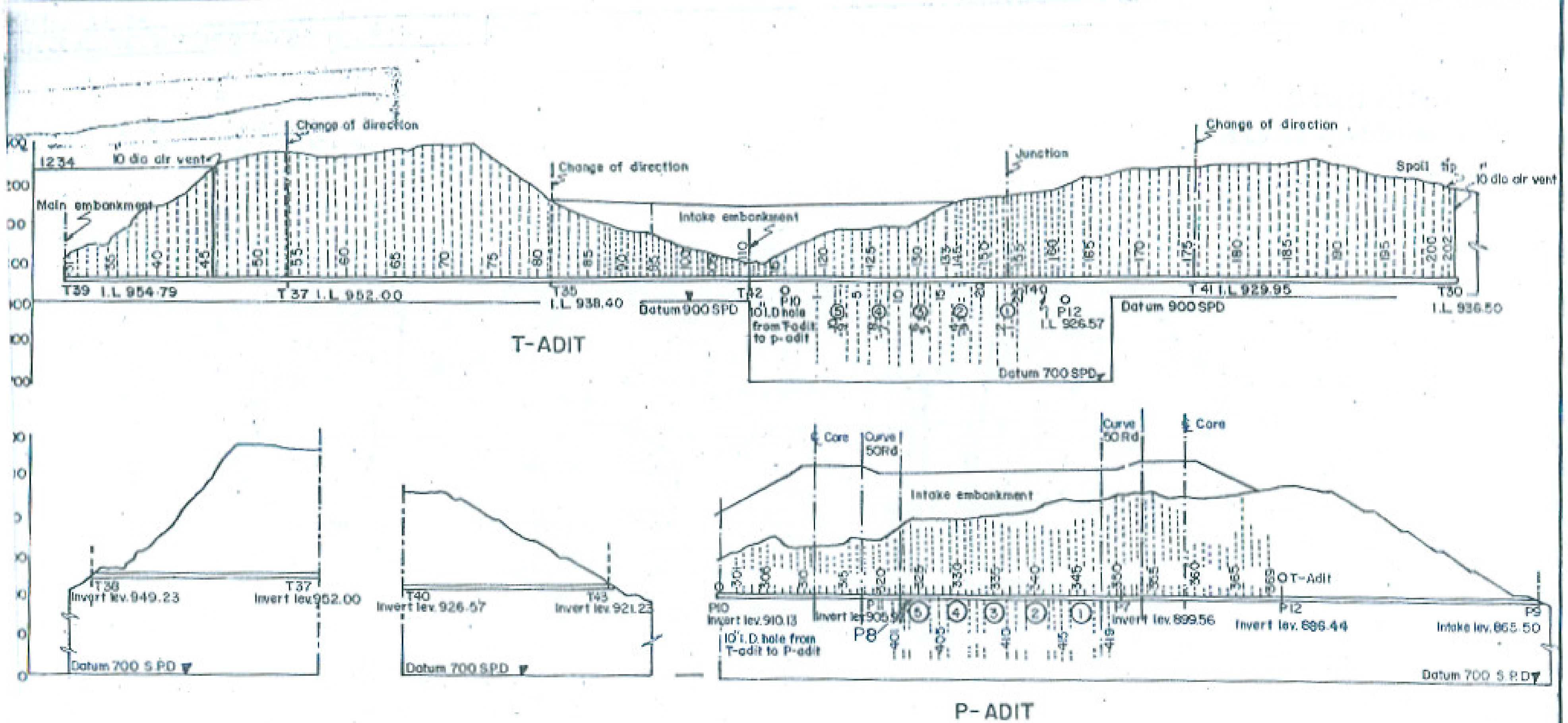
1. SEE DRG No DMO/M/I- FOR DETAILS OF WELL POSITIONING.
2. THE POINTS LABELLED T-41 & P-7 ETC. INDICATE ADIT TERMINAL, CHANGE OF DIRECTION OR CHANGE OF SLOPE POINTS.

MANGLA DAM PROJECT

INTAKE EMBANKMENT ADITS PLAN

REFERENCE: Mangla O & M. Vol. 10 Fig. 20

HALIL / 3.85



NOTE

For drainage adit plan
refer Drawing No. DMO/M/I 81 exhibit 54a

MANGLA DAM PROJECT

INTAKE EMBANKMENT T&P ADITS

LONGITUDINAL SECTIONS

EXHIBIT- 26

Paper No. 49?

AHID / 3.85