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WARSAK DAM PROJECT PROBLEMS & REMEDIES

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1. INTRODUCTION

Warsak Dam is situated 18 miles from Peshawar City on River Kabul. The hydroelectric project was completed in 1960. It is a concrete gracity dam 76 m high and 180 m long with an overflow spillway and the power house located on the right bank. Fig.1 shows general layout plan. Salient features of the project are given in Table-1. Initially four generators of 40 MW each were installed. Later on two more sets No.5 and 6 were added in 1981. Power House building comprises of two main floors at E1.1132 and E1.1149 SPD and an erection bay at E1.1167. A pipe gallery at E1.1132 and a cable gallery at E1.1149 SPD, run along the main floor at down-stream of the building. Upstream wall is anchored to the hill slope behind the wall at elevation 1157 to 1189. Foundation comprises mainly of granite, schists and phyllites. The two main problems faced by the project are opening up of cracks and joints, misalignment of turbine shaft and wear and tear of all the turbines due to silt.

A couple of years after commissioning of the four units in 1960, the power house showed signs of what was initially considered to be differential settlement in the form of opening of construction joints and cracks that appeared in the floors and walls.DMO Wapda 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 result of alkali-aggregate reaction. The movements are continuing although at a very slow rate. The possible causes of movements and cracks have been studied.

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

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right bank. The sediment level at the wall has reached E1.1240. 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 a problem of cleaning of trash rack also. M/s. Rousseau Sauve Warren Consultants and NESPAK have been engaged for the rehabilitation cost study of the project. The paper describes these problems and possible remedies.

2- MOVEMENT STUDIES

After the development of cracks in concrete a few instruments were installed during 1975 to monitor the development of the cracks and movements in the building. Instrumentation consisted of locally made plumb lines, dial height gauge points, glass tell tales and movements pins etc. The number of these instruments were subsequently increased as required from time to time. The locations and other details are given at Fig.2.

Initially these instruments were being observed occasionally without a planned schedule. However in the early eighties arrangements were made to observe the readings periodically on bi-monthly basis. DMO has been analysing and interpreting the data constantly. In this regard various reports have been issued from time to time.

2.1 Plumb Lines Nos. 1, 2, 3 and 4.

Plumb lines Nos. 1 &2 were installed during Nov. 1975, while Nos.3 and 4 during April, 1982. The data have been plotted at Fig.3. The major movements occur in the upstream and downstream directions, while the movements in east-west directions are negligible. From the trend and pattern of the movements of plumb lines, two types of movements are established in upstream and downstream direction. One may be termed as cyclic and the other one as residual. Cyclic movement corresponds to the seasonal variation in temperature in the area. All the plumb lines move in the upstream (southward) direction during winter and in the downstream (northward) direction during summer having their peaks during Dec. and June respectively.

When compared with the previous years, it was observed that with the passage of time the peaks of cyclic movements vary gradually with a specific trend producing a residual movement in one direction. For example movement of plumb line No.3 is continuously increasing in upstream direction, while at plumb line No.4 it is continuously increasing in downstream direction. The movement of plumb lines No.1 and 2 has no significant trend but on the whole it is slightly in the upstream direction. Thus due to residual movements, the peaks in every subsequent years are greater than the previous years. These types of movements confirm the presence of structural movements in the area.

2.2 Movement Pins L-series.

Six number movement pins designated as L1 to L6 have been installed across the longitudinal construction joints at elvation 1132 floor, downstream and opposite of each unit (generator housing). Out of these pins No.1 to 4 were installed during March 1975, while the remainings No.5 and 6 were installed during Nov.1980. These pins have been showing continuously an opening trend since their installation. This is the area where longitudinal joint opening is maximum as compared to any other area of the building. The magnitude of openings with the passage of time is given in the table below, and shown in Fig.4.

Pin No.	Movement upto 15 Nov. 1980 (Inches)	Movement upto 15 Nov. 1984 (Inches)	Movement upto 15 Nov. 1988 (Inches)	Movement upto 15 Feb. 1989 (Inches)
L1	0.045	0.115	0.175	0.196
L2	0.198	0.382	0.51	0.525
L3	0.245	0.453	0.63	0.655
L4	0.335	0.52	0.725	0.75
L5		0.1075	0.22	0.23
L6		0.025	0.0325	.04

From the set of above readings it is obvious that maximum joint opening has taken place at unit No.4. The magnitude of opening increases along the joint, going from east to west. The movement decreases significantly in the extreme west i.e, opposite unit No.6, however the pin No.5 and 6 were installed 5 years later.

2.3 Dial height gauge points 'UW' series.

These dial height gauge points have been installed across the vertical joints of upstream wall above E1.1149 floor. These gauges monitor the horizontal off-sets at these joints in the upstream and downstream directions. No significant relative movement has occurred at these joints so far. During early eighties an opening trend was observed. Magnitude of opening gradually increased at joints from unit 1 to 4 but substantially decreased opposite unit No.5 i.e, in the western side.

2.4 Movement pins 'UW' series.

These pins have been installed on upstream wall across vertical joints above floor level 1149 SPD. No significant trend of movement is established at these joints. Variation in joint opening generally corresponds to the temperature variation i.e, cyclic variation in movement. However at UW3 and UW4 which are opposite unit No.3 and 4 respectively, a slight opening is continuing with the passage of time.

2.5 Movement pins V-series.

These pins are located across vertical joints, 5 ft.above 1149 floor levels, in the downstream wall. Here also the movement across the joints corresponds the variation in the temperature. A slight opening trend is observed at V3, V3A, V4 which are installed opposite unit No.3 and 4.

2.6 Movement Pins 'TD' series.

These pins were installed on power House transformer deck monolith joints during June, 1981. Since then no appreciable opening has occurred. Generally the opening corresponds to the variation in the temperature, as in the case of other pins. However a continuous opening trend is prevailing at TD1, which is opposite to the erection bay.

2.7 Movement Pins 'RGF'and 'RGC' series.

These pins are located across the cracks in the rock gallery floor/crown of El.1149 floor. These pins exhibit an opening in winter and a closing in summer but with the passage of time an opening trend is developing gradually across all the points. Pins RGF2, RGC2, RGC3 and RGC3 continued to exhibit an opening trend gradually.

2.8 Long Term Movements.

In order to establish absolute displacements of each structural element with respect to a single set of global axes, two such lines were established. The first was located at the toe of the downstream wall of the power house in the dewatered tailrace. The second was located at the base of the upstream face of the generator block in the valve gallery.

Fig.5 shows the movements on a vertical cross section through the upstream wall of the valve gallery, while Fig.6 shows the movements on a vertical cross section through the downstream wall of the power house. The most significant feature of these sections is the large and consistent displacement which occurs within the height of the draft tubes. When reviewed in conjunction with the dominant diagonal cracks observed in the walls of the draft tubes, it is apparent that the building as a whole is being moved outwards by some forces located above 1085 ft. elevation. As there is little to indicate that significant lateral displacement has taken place on the valve gallery side of the concrete blocks housing the scroll case and as there has been no bed rock movement of any significance, the cause of the outward movement in the downstream wall of the power house must lie within the structure itself. The overall lateral movements are shown, together with the upward movement and the major cracking in Fig.7.

3. STATUS OF STRUCTURE

The condition of the various structures and different floors is briefly described in the following paragraphs. For favour of location, the Fig.8 shows a typical cross section of the power house.

3.1 Generator Floor E1.1149 ft.

The most significant feature of structural deformation on this floor is a longitudinal crack measuring in width from 0.2" to 1.5" located just downstream of generator pits. Its greatest width is at units 2 and 3. The crack does not appear in concrete floor that was poured in 1980-81 for units 5 and 6. Between units 1 and 4 on the underside of this floor and adjacent to the crack, spalling of concrete has taken place to the extent that, on fairly large surfaces, the reinforcing bars are exposed. The floor surface in line with the downstream edges of the generator pits of units 1 to 4 has also risen as much as 2.5" immediately above the zones which have spalled underneath.

3.2 Turbine Floor E1.1132 ft.

On this floor the most important structural defect is the widening and off-set of the vertical longitudinal construction joint located downstream of the units. When the reservoir is full, the tail water level is about 1136. The joints leak during this period. The silt and fine sand brought by the leakage water indicates the opening of the joint upto the draft tube. The longitudinal crack in the 1149 floor is directly related to the opening of this joint.

3.3 Valve Gallery.

Several horizontal cracks and the upward expansion of the unit blocks were found in the valve gallery. This is consistent with the presence of openings between the tops of the penstocks and the unit block concrete and other indications of severe shear loads on the penstocks, turbine inlet valves and transition pieces.

3.4. Super Structure including walls concrete columns and crane beams.

Some vertical construction joints have opened in the upstream and downstream walls and there is minor opening of the joints between the crane beams and columns. Some horizontal cracks are also visible in the concrete encasement of the steel columns at the levels of the lower flanges of the root girders. These defects are minor and there appears to be no significant structural distress in any elements of the superstructure.

3.5. Super Structure above crane rails, including Steel Columns, Girders and Concrete roof Slabs.

No signs of structural distress or significant cracks are apparent in the underside of the roof structure. All the observed roof leaks occur at the joints in the roof slabs over the main steel girders and not through any cracks between the girders or around drainage pipes.

The upstream connection between the girders and steel columns is rigid whereas the downstream connections was designed to permit relative movement between the column and girder axes. This system is capable of accommodating considerable movement of the column bases without significantly reducing the load carrying capacity of the roof system.

3.6. Wave Wall

The mass concrete wall between the Power House tailrace channel and the spillway dissipation apron is constructed with transverse construction joints. The construction joints are open and many cracks are evident on the surface of the concrete.

3.7. Dam Spillway

Considering the age, the main body of the dam is in reasonably good shape as compared to the power house. The inspection gallery does have some old cracks through which calcium has leached out. For the most part these cracks are tight as demonstrated by the dry hardened calcium deposits.

Near the north end of the dam, the non overflow section, one of the transverse construction joint has opened up and the gap at deck level now measures about 1.5". Many of the gate piers exhibit some surface cracks.

The chute of spillway is eroded and sand blasted. The concrete aggregates are exposed over the entire surface. However the erosion is not deep and no reinforcing bar is exposed.

Although the stilling basin was repaired in 1978-79, the concrete lining has been again destroyed by the grinding action of abrasive sand and stones. At present a lot of debris is lying on the floor of the stilling basin.

4. POSSIBLE CAUSES OF DISTRESS TO POWER HOUSE

The following could be the possible causes of the movements and distress to the Warsak Power House.

4.1. Movements of the U/S Rock Slopes.

The detailed evaluation of slope stability presented in Golder Associates report "Rock Mass Stability Assessment, Warsak Hydro-Electric Power Station," concludes that no major evidence of instability could be found. In addition, extensometers D1 to D3 placed in the rock behind the station failed to show any significant movement at depth which would be the case if continued movement of the slopes were occurring.

4.2. External Water Pressures

The Warsak Power House is equipped with complex system of drains located beneath the building and emptying into an open conduit located on the upstream wall of the valve gallery where flows are channelled to the sump for removal (Fig. 9). Additional drains were also provided from valve gallery floor into rock. The pressures recorded by the piezometers installed by M/s. Golders Associates show that these are not high enough to contribute to the movements of the Power House.

4.3. Internal Water Pressures

In the course of Power House operation, considerable operating difficulties have been experienced as a result of the excess wear on moving turbine parts caused by the silt content of the water flowing through the station. The piezometer observations show that although the pressure do in fact fluctuate indicating that some form of unplanned inter connection occurs, the pressures have not been sufficient to account for the observed deformations nor has there been any direct correlations between the observed water pressures and the observed deformations.

4.4. Earthquake Effects

The Warsak Power House is located in a seismically active area and in the course of the monitoring period, a number of earthquakes have been noted ranging in magnitude upto 7.2, with an epicentre only 300 km from the station. At an early stage of the investigation, it was considered possible that these earthquake effects coupled with the forces which are present during the operation of the facility might be sufficient to cause deformation to occur during active seismic periods.

Although the monitoring period includes times of earthquake activity, no evidence of any connection between the displacements and the earthquake activity has been observed with the exception of short term increase in measured water pressure in a few of the piezometers behind the building persisting for a few days after the earthquake. On the basis of these observations, it is considered unlikely that earthquake effects play a major role in the observed deformations.

4.5. Vibration Effects

It was also considered possible that the effects of silt erosion referred to above might have created some imbalance in the moving machinery and contributed to higher than normal vibration levels which might have been a contributory factor in the deformations. In general, the levels of observed vibrations over the course of the monitoring period have not been considered to be anomalous and there has been no correlation between periods of movement and periods in which machines have been shut down or started up again in the course of routine Power house operation.

4.6. Foundation Movements

A variety of possible reasons for overall foundation movements were considered including nearby tectonic activity, the effects of the major silt load built-up behind the dam, the presence of previously undetected soft seams in the rock beneath the power station building etc.

Geological observations made on the cores recovered from the holes used for instrument installation and the mapping carried out in the slope behind the power station have tended to corroborate the geological mapping which was carried out during the course of the power station design and construction. There is no direct evidence of unexplained or previously undetected weak materials beneath the foundation of the Power house which would give rise to differential settlement.

4.7. Alkali Aggregate Reaction

This phenomenon may be defined as a strong chemical reaction that occurs when the concrete is made either with aggregates which contain a high degree of basic and alkaline particles or with cement that has soluble alkaline and Carbonate sulphates even in relatively small quantities. Under favourable conditions for the reaction to take place, the concrete develops the classic alkali aggregate reaction (AAR) symptoms that are characterised by expansion, swelling, polygonal fissuration on the surface, cracking and spalling due to loss of adhesion between the aggregates and the cement matrix.

4.7.1. Observations :

Examination of drilled cores showed that fracturing was extensive throughout the turbine blocks. In many instances fractures occurred through an aggregate particles. A close examination of the fractured face of one core indicated that a darkened rim was present around many of the fractured aggregates. In addition, white deposits of leached calcium products were visible at nearly all fractured faces. In some areas direct evidence of alkali aggregate reaction was noted characterised by the presence of silica gel.

4.7.2. Analysis of the Effects :

The observed displacements, bulges, deformations and cracks in the power house structure are consistent with the effects of AAR and the following factors :

* below turbine floor level (El. 1132 ft), the concrete is essentially saturated - a necessary condition for AAR;

* the base of the structure is restrained by the foundation rock;

* along with south side and west end, the structure and the high excavated rock faces are stable and are together restrained by the high strength anchors installed through the concrete buttresses of the upstream wall of the power house;

* at the east end (control and service building and erection bay), the structure is restrained by the rock mass on three sides (north, east and south);

* except for the draft tubes, the structural concrete beams, the columns, the buttresses and the generator pit walls, the concrete is relatively lightly reinforced. Most of the heavily reinforced concrete elements are above the saturated zone;

* the pits of units 5 and 6 remained unconcreted from the time of commissioning the first four units in 1961 until units 5 and 6 were installed and commissioned between 1980 and 1981. The west face of unit block 4 and the inner face of the downstream wall above El. 1111 ft at units 5 and 6 were not subject to saturation during this time;

* the wave wall (crest level 1160 ft) between the tailrace and spillway dissipator apron is partly submerged and subject to saturation from spray during periods of high flow.

From the above the nature of the structural deformations may be explained as follows :

* the large volume of concrete surrounding each unit below turbine floor level (1132 ft) has expanded continuously from the time the power house was commissiond and the expansion, a tri-axial phenomenon, is continuing';

* the least resistance to such expansion is in the vertically upward direction. Due to differences in moisture conditions the vertical expansion below elevation 1132 ft. is substantially greater than that of the structural elements above this level;

* there is some lateral expansion of concrete that is founded or rock under the draft tubes as evidenced by the fact that the downstream wall is inclined outward at the bottom. The heel of the draft tubes appears to be acting as an anchor point

* in the upstream direction, the expansion of the concrete surrounding the units is restricted only by the floor systems at El. 1132 ft and 1149 ft. and the penstock-valve-spiral case connections;

* in the downstream direction, expansion is limited by the structural flexibility of the outer wall which is controlled by the following conditions :

a) it is supported on narrow, heavily reinforced draft tube walls which are founded on a solid base slab on rock;

b) below El. 1111 ft. it is essentially monolithic with concrete surrounding the draft tubes beneath the generating units;

c) above El. 1111 ft. it is very thick and is separated from the mass concrete surrounding the turbine by a vertical joint 27 ft. downstream from the centreline of the units;

d) it has a horizontal essentially unreinforced, construction joint at El. 1121 ft. (centreline of turbine spiral case);

e) between El. 1132 ft. and El. 1164, it consists of inner and outer vertical components which are tied together horizontally by beams and slabs at El. 1149 ft. and El. 1165 ft;

f) the top of the downstream concrete wall is lightly restrained by the columns supporting the steel roof girders.

* in the longitudinal direction, unit 1 is restrained on the east side by the 1111 ft. floor which is founded on rock and by floor beams and slabs at El. 1132 ft. and El. 1149 ft. Between units 1 and 4, the unit blocks restrain one another in the longitudinal direction although there was no longitudinal restraint of unit 4 in the west direction until unit 5 was added in 1980;

* the deformations were influenced by the presence of horizontal lift joints which, in the massive sections, were spaced at intervals of about 18". These are obviously planes of weakness where preferential separation has occureed.

4.7.3. Finite Element Analysis

A finite element model of the substructure of the power house was analysed using the "COSMOS/M" computer programme and the deformations and crack patterns thus generatged were compared with those observed at site. (Rehabilittion cost study report by RSW).

The correspondence between the calculated and the observed behaviour of the structure led to the conclusion that expansion of the concrete, while being somewhat restrained at the rock interfaces, is sufficient to cause all the observed movements and cracks in the power house and that the model is capable of reproducing the structural behaviour of the power house with an acceptable degree of precision.

5. MODIFICATION TO MITIGATE THE EFFECT OF AAR AND STRUCTURAL REPAIRS.

The gradual deterioration caused by the AAR within the damp concrete are slow and on going and will not in themselves result in a sudden catastrophic failure. However some of the structural elements are experiencing considerably higher stresses than were contemplated in the original design calculations.

Nothing practical can be done to stop the AAR, hence steps must be taken to ensure that continuing expansion of the substructure concrete will not compromise the stability of the power house, and that it will have a minimal effect on the operation of equipment. To this end the following treatment of the substructure concrete is foreseen :

* Major cracks are flushed clean and grouted ;
* Steel anchors are installed, primarily to help maintain the structural integrity of the badly cracked concrete mass ;
* Slots are cut at appropriate locations in the concrete substructure to relieve existing stresses and accommodate future expansion.
* Realign power house crane rails and replace cracked or badly worn crane wheels.
* Make repairs to concrete on floors and walls where such repairs are necessary to ensure structural integrity and prevent further deterioration.
* Install additional facilities for monitoring structural creep.
* Repair and modify butterfly valves and associated connections to penstocks and spiral cases, in order to :

    relieve the present severe overstresses (failure of one of these components could led to flooding of the power house);

    improve the operation of the valves.
* Modify the generator anchoring arrangements, and realign generating units.

* Realign the draft tube gate guides, restore clearances, and modify draft tube gate crane follower beam, all as needed to permit satisfactory operation of the draft tube gates.

* Modify spillway gates as required to achieve clearances adequate to avoid sticking during operation, and repair gate sills to ensure adequate sealing.

* Re-paint of roof structural steel (otherwise the roof structure appears to be in good condition).

* Repair roof leaks.

6. RESERVOIR SEDIMENTATION

6.1. Sedimentation

Fig. 10 shows Longitudinal section of Main dam and cross section of the spillway. 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 ft. situated some 80 ft. upstream of the intake structure (Fig. 1). 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 sections of the trash racks. Model studies carried out by WAPDA 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 wall 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/or the draft tubes through the cracked concrete of the power house substructure into the turbine floor via the open longitudinal joint downstream of units.

6.2. Silt Exclusion Measures

6.2.1. Skimming Walls in front of the Power Intake :

Sedimentation in front of the skimming wall, upto its crest Elevation 1240, has made this wall in-effective. Thus no purpose is being served by this wall under the present circumstances. Further raising by 10ft. was considered but it is believed that it will meet the same fate soon.

6.2.2. Annual Reservoir Flushing:

Since 1976, WAPDA has introduced a reservoir flushing procedure during the latter stages of the annual high flow period when the flow through the units is interrupted, or substantially reduced, and the bed load sediments are evacuated through the spillway. Flushing is done only for a few hours in a week in an effort to not to reduce the generation. This limited flushing action has no effect.

The flushing operation is an annual feature. Its effect on silt exclusion is limited and does not last over the whole year. Thus there is need to develop a scheme that can ensure silt free supplies to the two intakes all the year round.

6.2.3. Sediment Sluicing:

Several possible alternatives have been studied wnich include lowering of one or more bays of the spillway so that a low level sluicing outlet is available.

Making the diversion tunnel operable was also considered and found not feasible. This will introduce non uniform distribution of flow and would aggravate the damage to stilling basin. The quantity of sediment is so heavy that the area evacuated is filled up in a day.

6.2.4. Alternate Intake:

To draw of low silt concentration surface water outside of this river bend, it is proposed to shift the combined intake of power and irrigation to the right bank near the range line 6 and take the conveyance channel back to the existing intakes of power and irrigation. The rock at the new intake will be removed to E1.1250 in a width of 400 to 500 feet. Sufficient space is available on the right bank for alignment of a large radius unlined conveyance channel between the old and new intake. No concrete work is proposed at the new intake. The channel excavation will be entirely in rock. The conveyance channel will be designed for minimum head loss and to cater for a discharge of 25000 cusecs.

6.2.5. Cellular Coffer dam & Dredging in Pocket:

M/s. NDC & NESPAK Consultants have proposed to construct a Cellular Coffer dam by driving sheet piles in front of the Intake structure enclosing three bays of spillway. A floating dredger installed inside the enclosed area would continue working the year around. The top of the cellular dam would be at RL 1275 while a part of it would have RL 1250 and would serve as concrete overflow crest.

The major draw backs in this proposal are:-

(i) It will adversely affect the working of spillway stilling basin;

(ii) The dredging of the pocket will be a continuous operation and sediment thrown into suspension by dredging operation will be washed into the tunnels.

(iii) It would be very expensive.

6.2.6. Dykes on Right Bank:

A scheme currently under investigation involves the construction of a series of spur dykes on the right bank normal to the reservoir shore line upstream from the power intake and the development of a procedure for systematic flushing of the reservoir during high flow periods. The effect of this system is to force the flow channel away from the right bank and the power intake so that bed load sediments are diverted from the right bank and evacuated by the spillway. Hydraulic model studies are now in progress at the Irrigation Research Institute at Nandipur. It is anticipated that, with this method, bed load sediment flow into the power intake may be reduced from the present 80% of the total to 20% or less.

M/s. Acres International Ltd had made a comparison of the different proposals as reproduced in the table below:-

QUALITATIVE COMPARISON OF REMEDIAL MEASURES

Measure	Initial Cost	Operating Cost	Estimated Effectiveness	Principal Uncertainties
Dredging	High	High	Up to 90 per cent sand removal	- Extent of deepened area which must be maintained.
Hydraulic Sediment Sluice	High	Low	Approximately 24 per cent sand removal estimated.	- Magnitude of by-pass flow necessary. Effectiveness to be expected.
Spillway Manipulation	Nil	Probably Low	Unknown	- Feasibility in view of downstream concrete erosion. - Effectiveness.
Altering Intake Hydraulics	High	Nil	Unknown	- Effectiveness.

7. REPAIRS AND MODIFICATIONS DUE TO SILT PROBLEM

* Replace the two top sections of the trash racks with a new design suitable for use with automated raking equipment and provide a set of mobile automated raking equipment which can be used sequentially on all openings.

* Refurbish intake gate seals, add "silt skirts" to the gates to limit accumulation of silt on the gates them- selves, and install a high pressure water system to flush out silt which has accumulated around the gate wheels prior to opening or closing.

* Rebuild turbines, using new components designed specifically to resist erosion. Replace the following major components:

  - discharge rings;
  - head covers;
  - bottom rings;

- guide vanes;
- runners and runner seals;
- shaft seals.

8. CONCLUSIONS AND RECOMMENDATIONS

The Warsak Power House, over the 30 years since it was built, has deteriorated principally as a result of:-

- structural deformation which has been caused by the alkali-aggreagte reactivity (AAR) in the concrete and ;
- erosion in hydraulic equipment due to the quantity and abrasive nature of the silt carried by the Kabul River.

Monitoring of the power house has confirmed that the movements and deterioration are on going at a slow pace and there is no immediate danger to the stability of the structure. Nothing practical can be done to stop AAR. However steps must be taken to ensure that the continuing expansion of the substructure concrete does not compromise the stability of the power house and that it will have minimal effect on the operation of the equipment.

The silt problem is aggravating with the passage of time and causing a lot of wear and tear of turbines. Modifications and repairs due to silt problem as recommended by the Consultants (RSW) need to be carried out. The total cost of the works is estimated as 277 m. Rs. In order to keep the plant available for operation during periods of high flows i.e, from April to September each year, the expected rehabilitation programme would take atleast 4 years site work.

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Table 1.

SALIENT FEATURES WARSAK HYDROELECTRIC PROJECT

1.	Original Gross reservoir capacity upto E1 1269	136000 Af.
2.	Original Gross reservoir capacity upto E1 1259	110000 Af.
3.	Original Live storage 1269 - 1220	62000 Af.
4.	Original River bed level	1108 ft.
5.	Full storage level	1269 ft.
6.	Crest of Spillway	1230 ft.
7.	Dead Storage level	1220 ft.
8.	Recorded peak discharge	236000 cfs.
9.	Spillway design discharge	540000 cfs.
10.	Average annual flow	17.6 MAF
11.	Average annual suspended sediment	20.7 MST
12.	Cill of Power tunnels (39' dia)	1206 ft.
13.	Top of Power Tunnel Intake	1245 ft.
14.	Top of Skimming wall	1240 ft.

15.	Cill level of diversion tunnel (35' dia)	1095 ft.
16.	Intake gates	16.4 ft x 39 ft.
17.	Discharging of capacity tunnels	24000 cusecs
18.	Irrigation tunnels (RL 1240) 10 ft. dia right bank.	4 x 6 left bank

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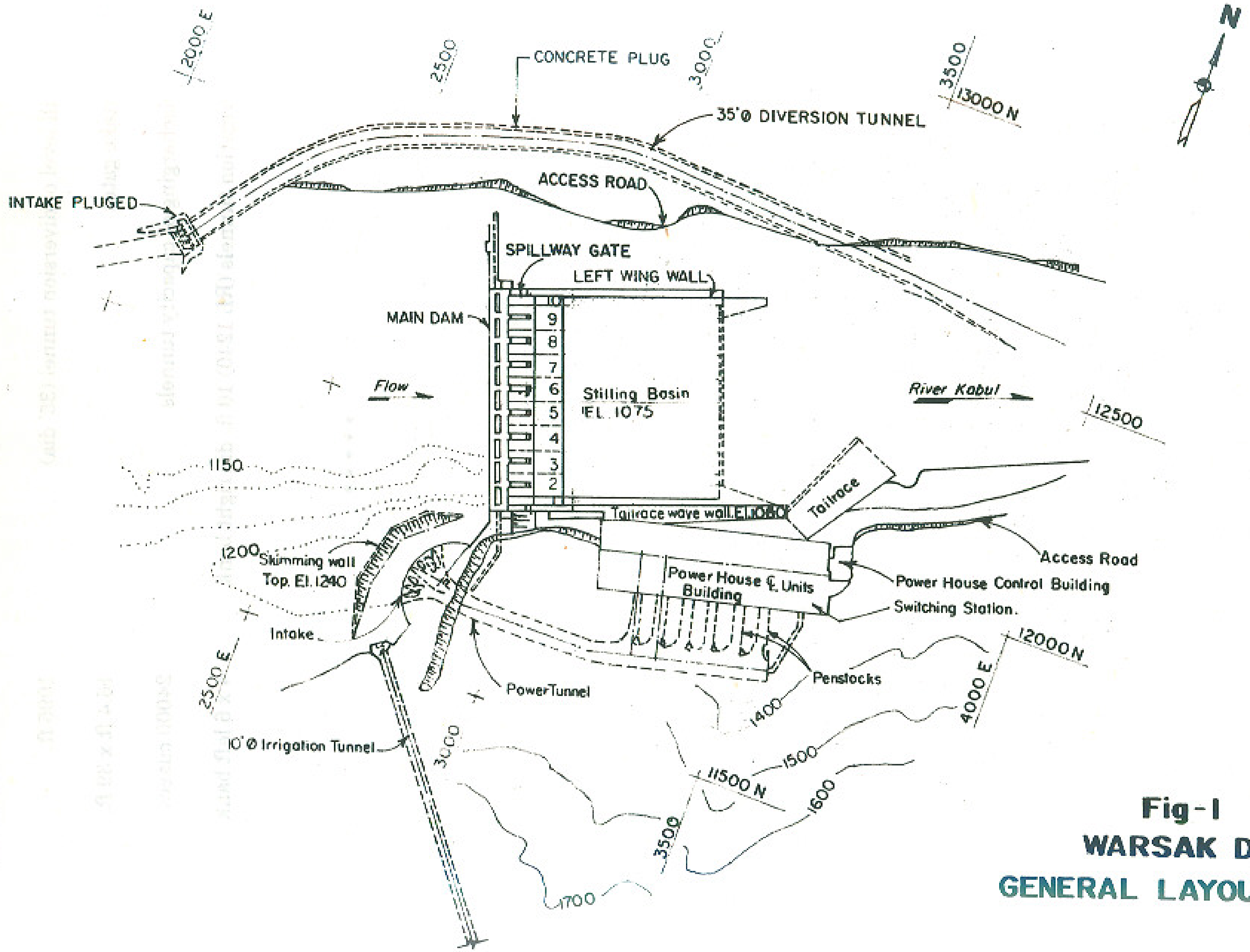


Fig - I
WARSAK DAM
GENERAL LAYOUT PLAN

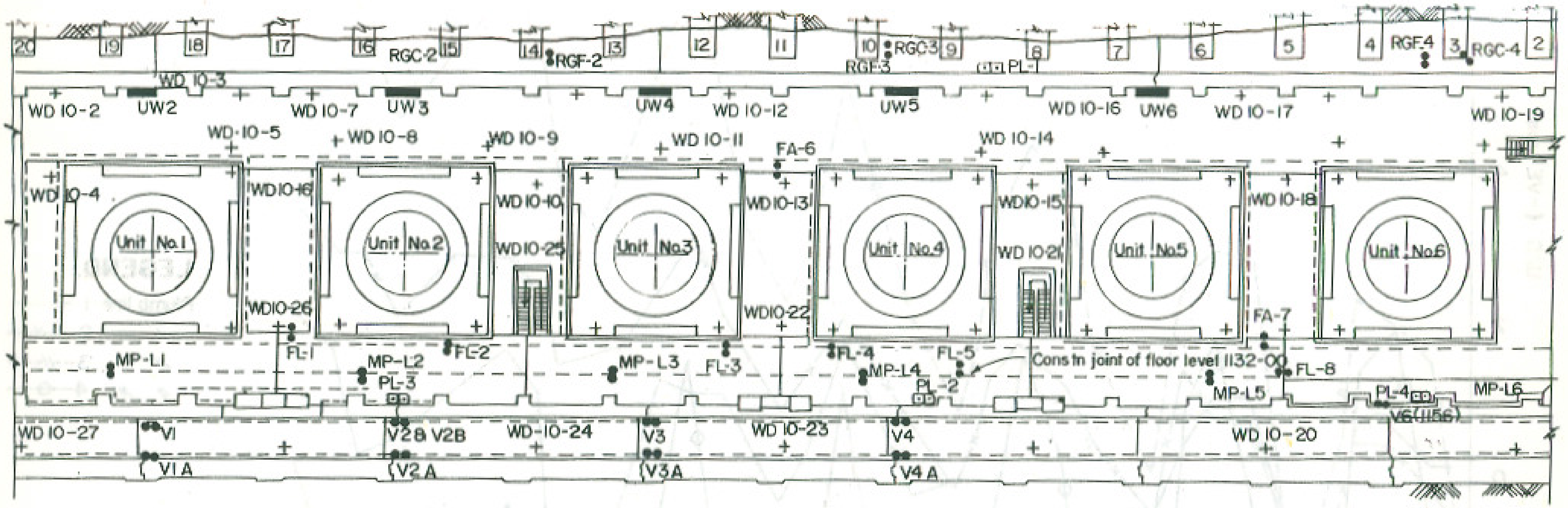


Fig-2
LOCATION OF INSTRUMENT
EL.1149 FLOOR
WARSAK POWER STATION

LEGEND

- ● ● MOVEMENT PINS
- [●●] PLUMB LINES
- ▬ OFFSET PLATE(-UW)
- FL FLOOR CRACKS LONGITUDIANAL
- FA FLOOR CRACKING AXIAL
- +WD VERTICIALITY.POINTS

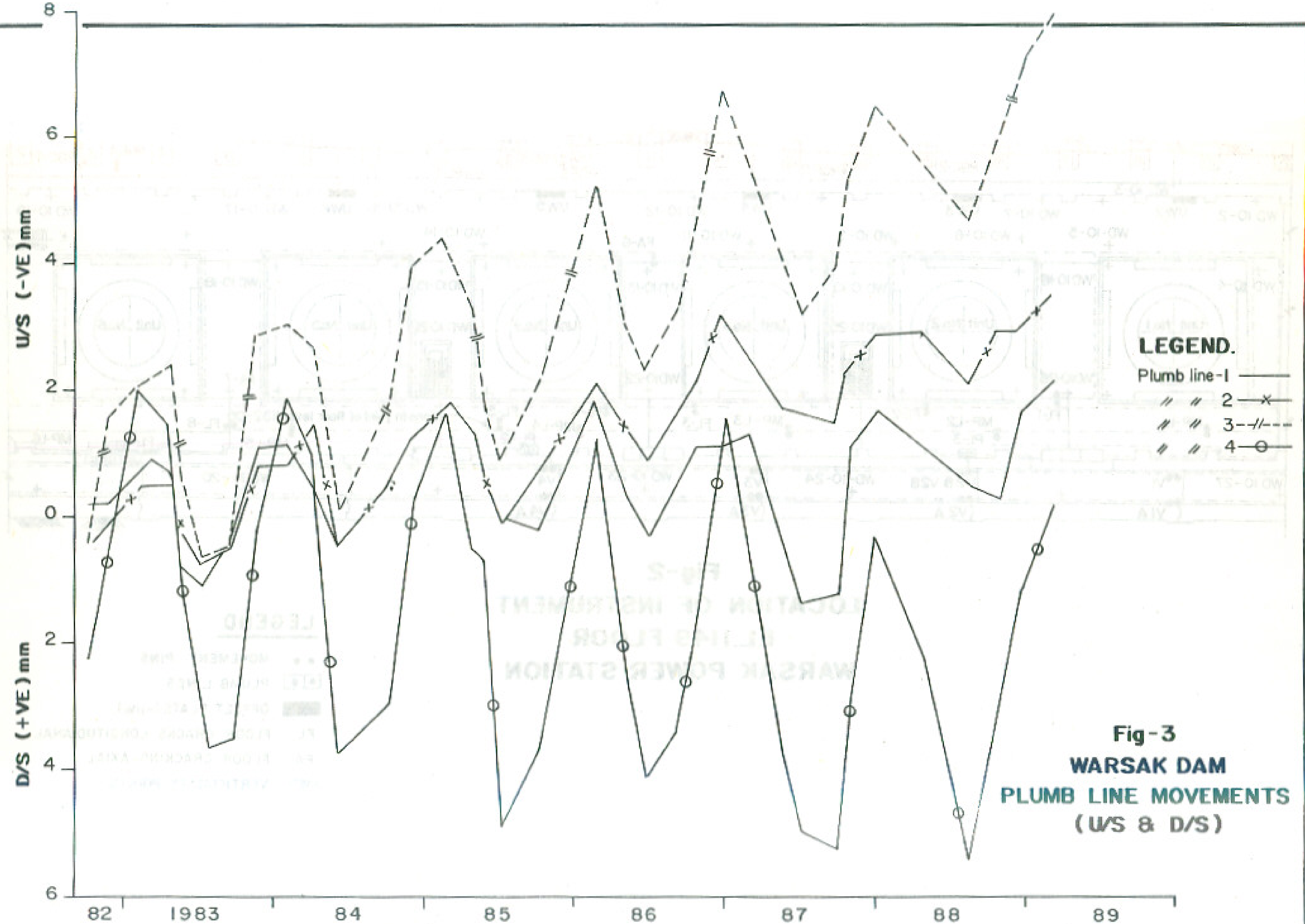


Fig-3
WARSAK DAM
PLUMB LINE MOVEMENTS
(U/S & D/S)

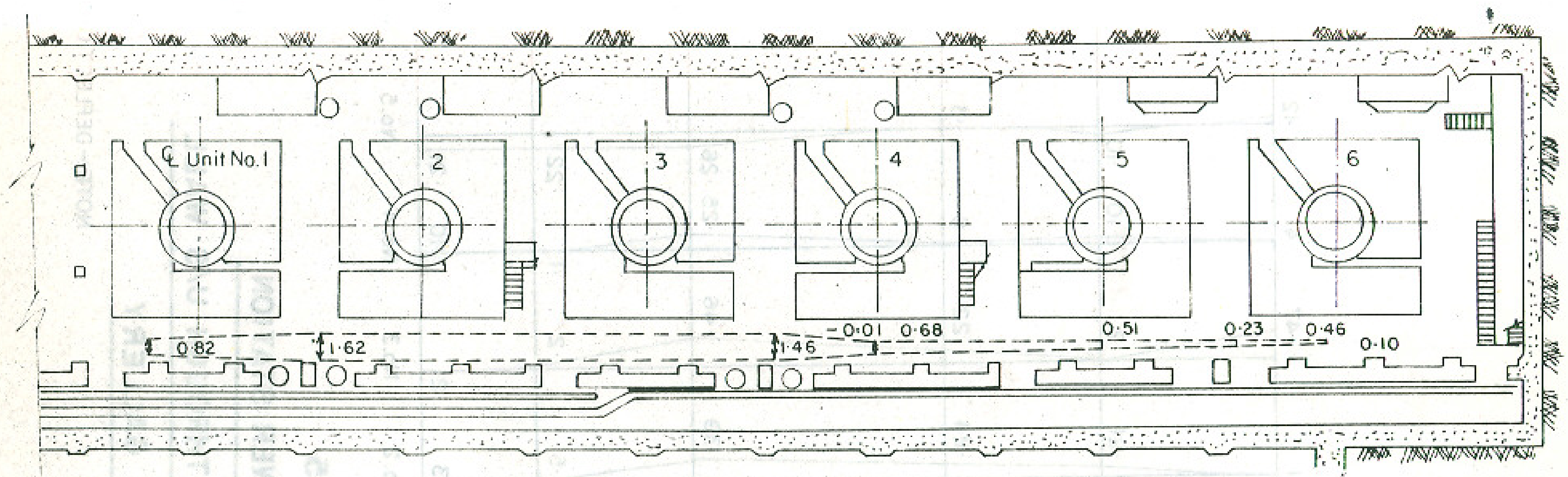


Fig. 4

WARSAK POWER HOUSE

MOVEMENT ACROSS MAJOR CRACK (In mm)

AT FLOOR PLAN ELEVATION 1132·0 S.P.D

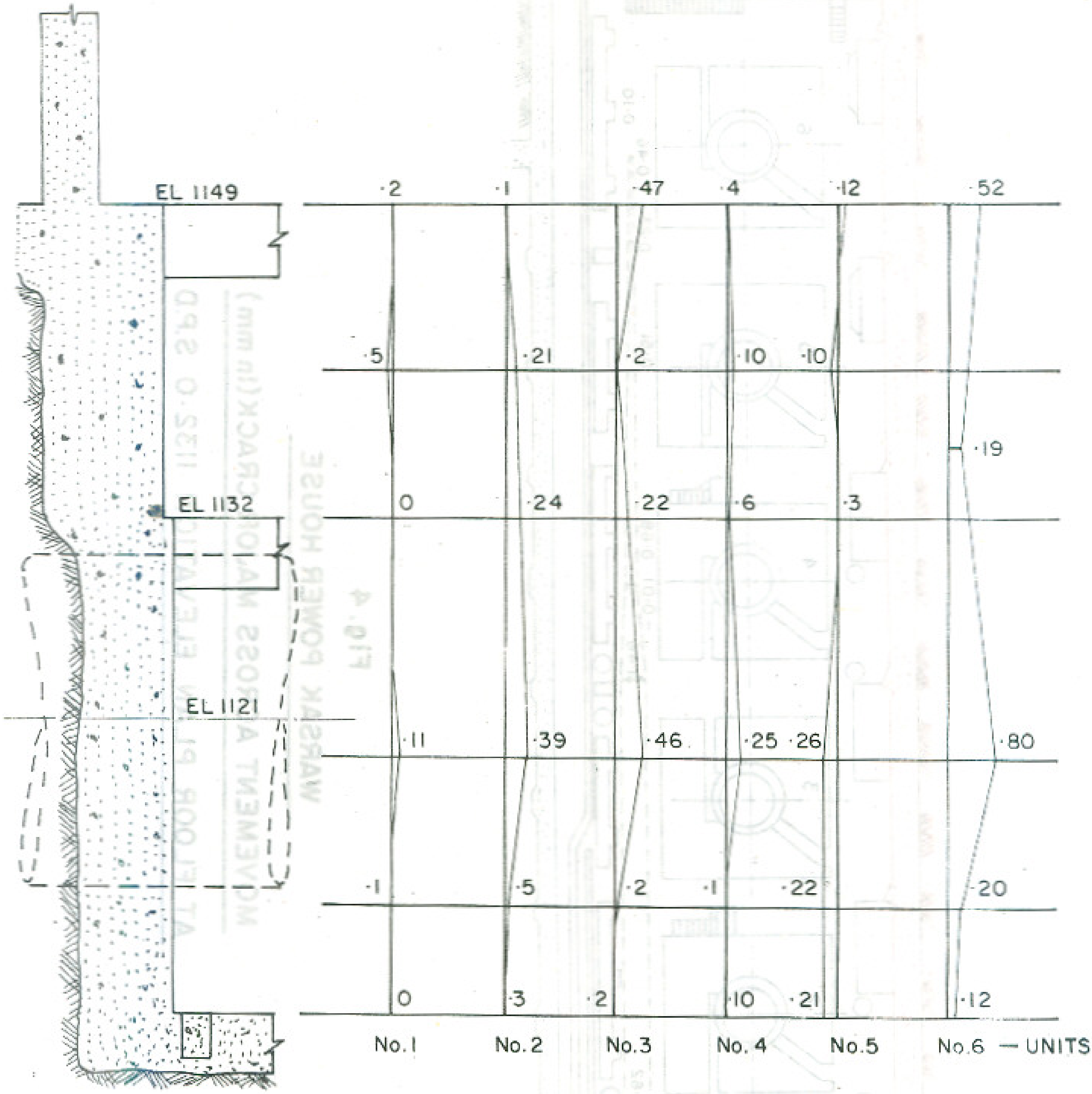


Fig-5

WARSAK POWER STATION

VERTICAL SECTION THROUGH U/S WALL

OF VALVE GALLERY

NOTE :- DEFLECTIONS ARE IN m.m.

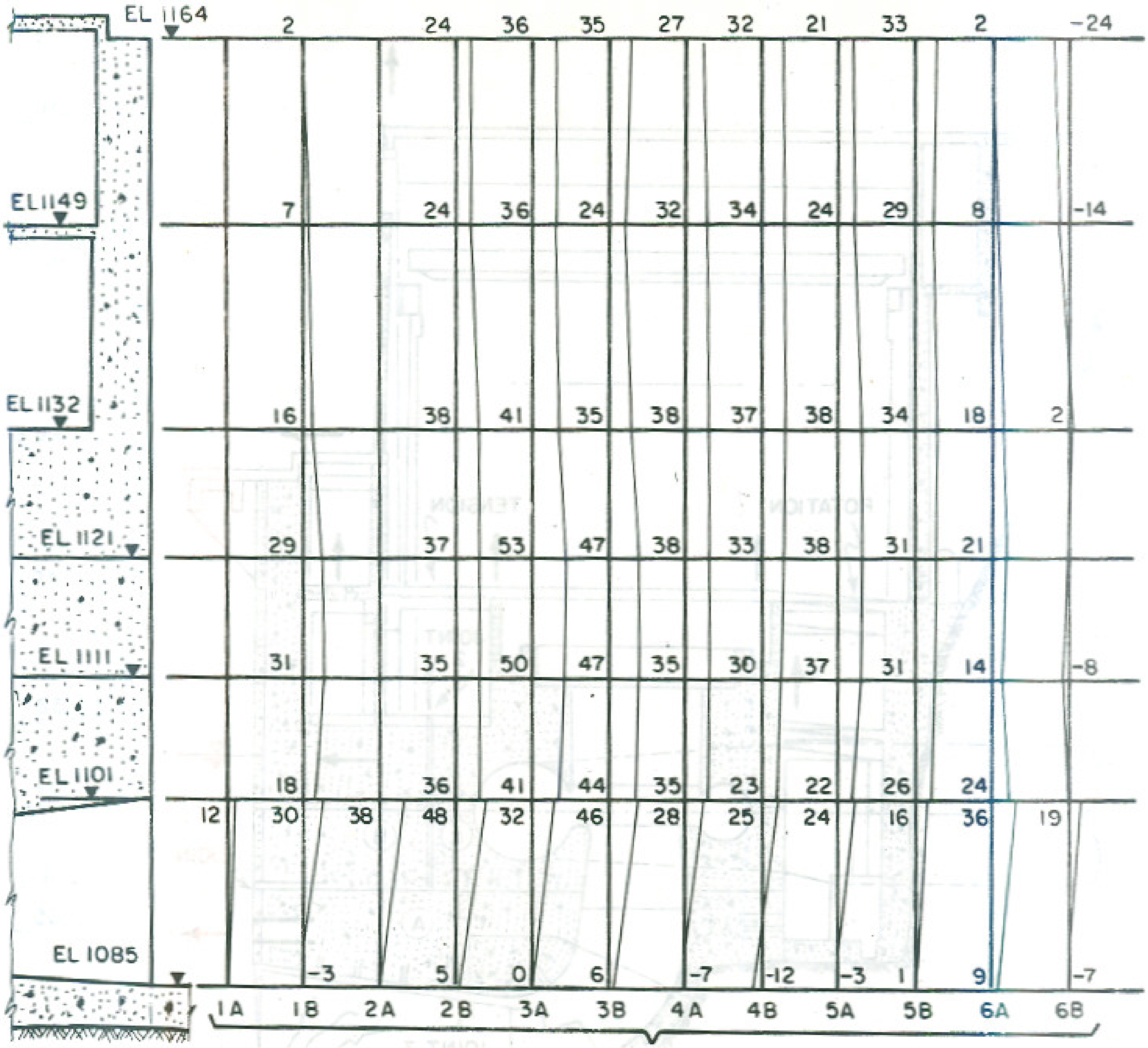


Fig-6

WARSAK POWER STATION

VERTICAL SECTION THROUGH D/S WALL

OF POWERHOUSE

NOTE :- DEFLECTIONS ARE IN mm

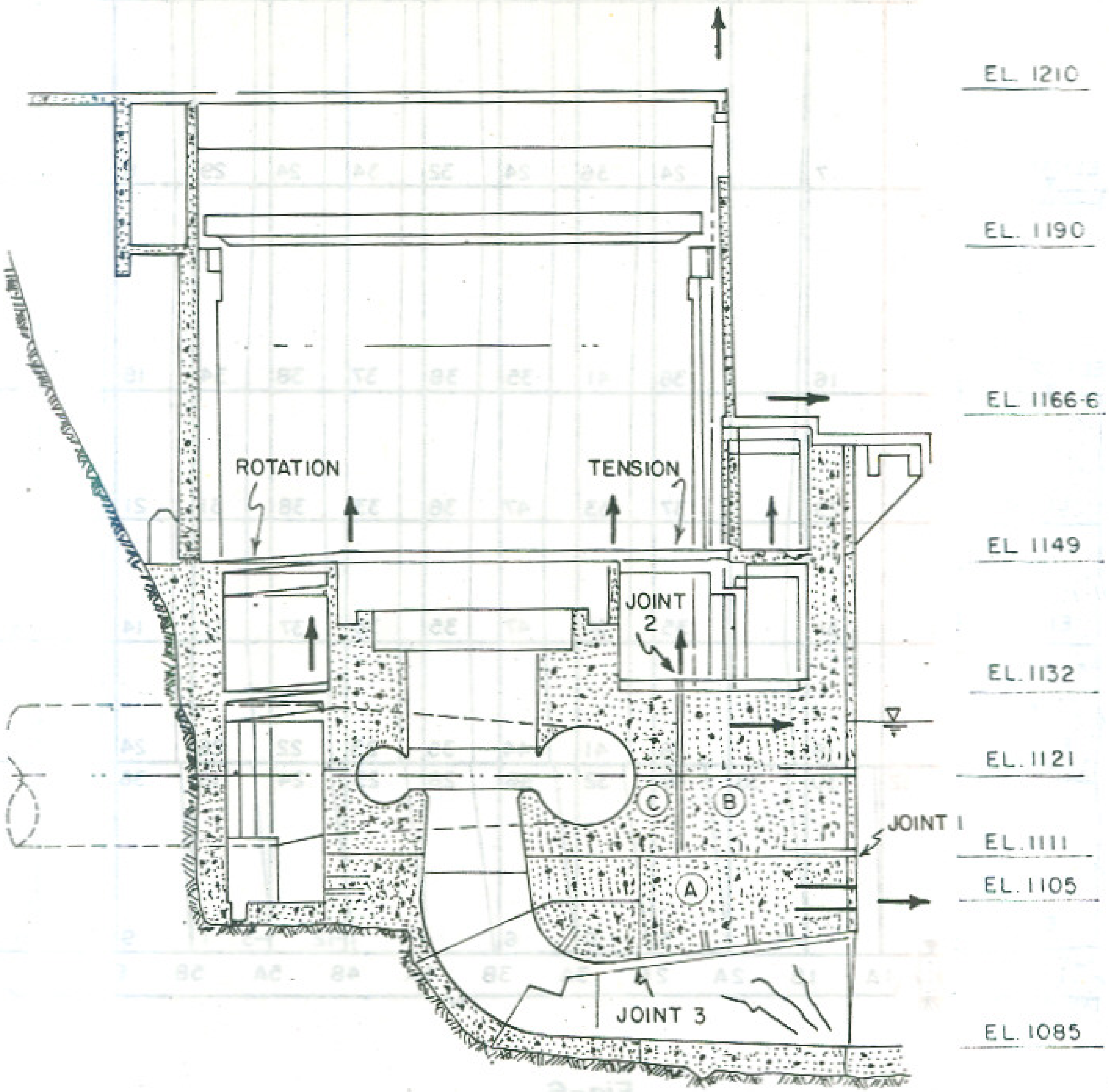


Fig - 7

CROSS SECTION THROUGH WARSAK POWER HOUSE

&

STRUCTURAL DISPLACEMENTS AND CRACKS

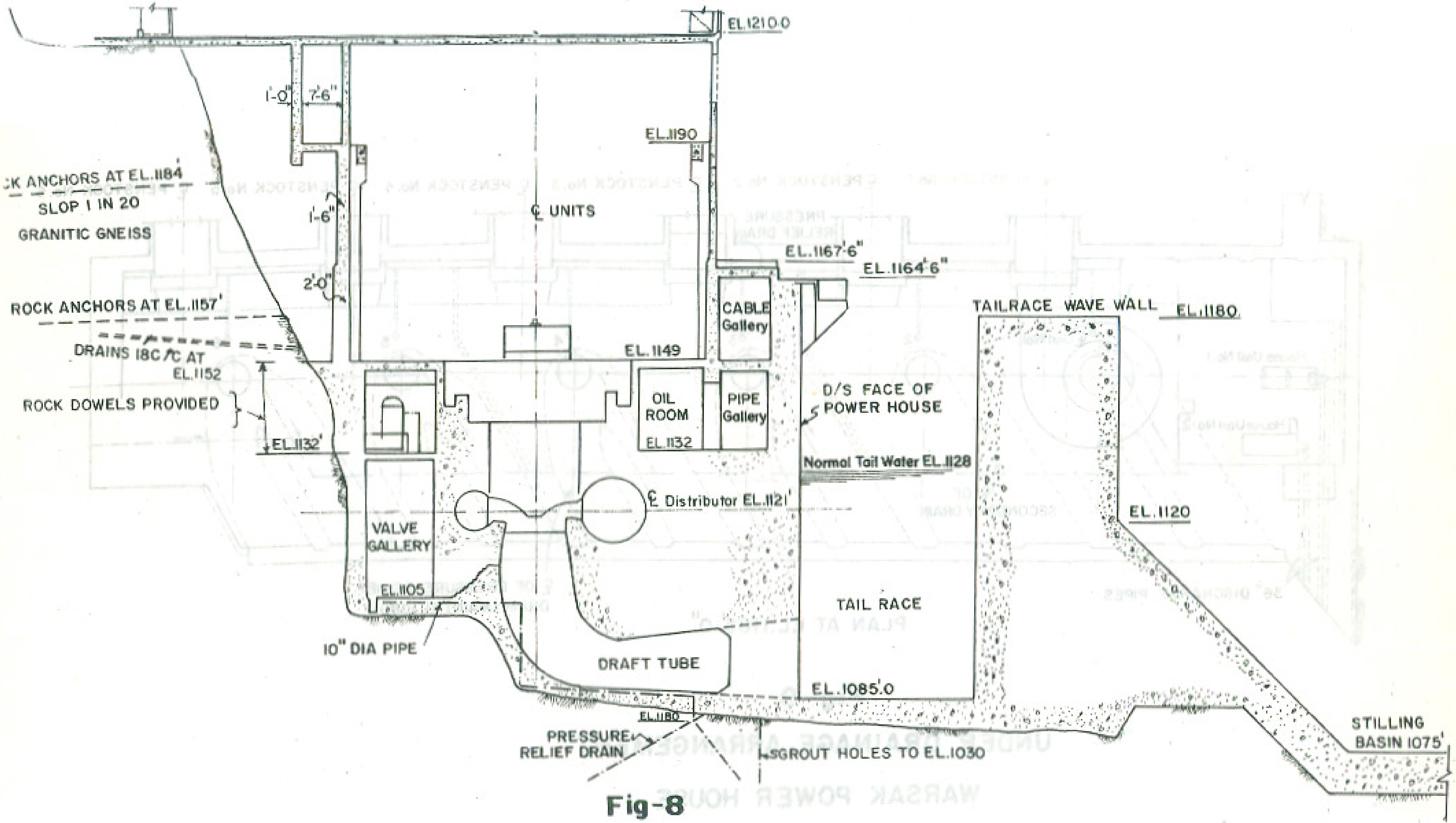


Fig-8

TYPICAL CROSS SECTION THROUGH ℄ UNIT No. 4

WARSAK POWER HOUSE

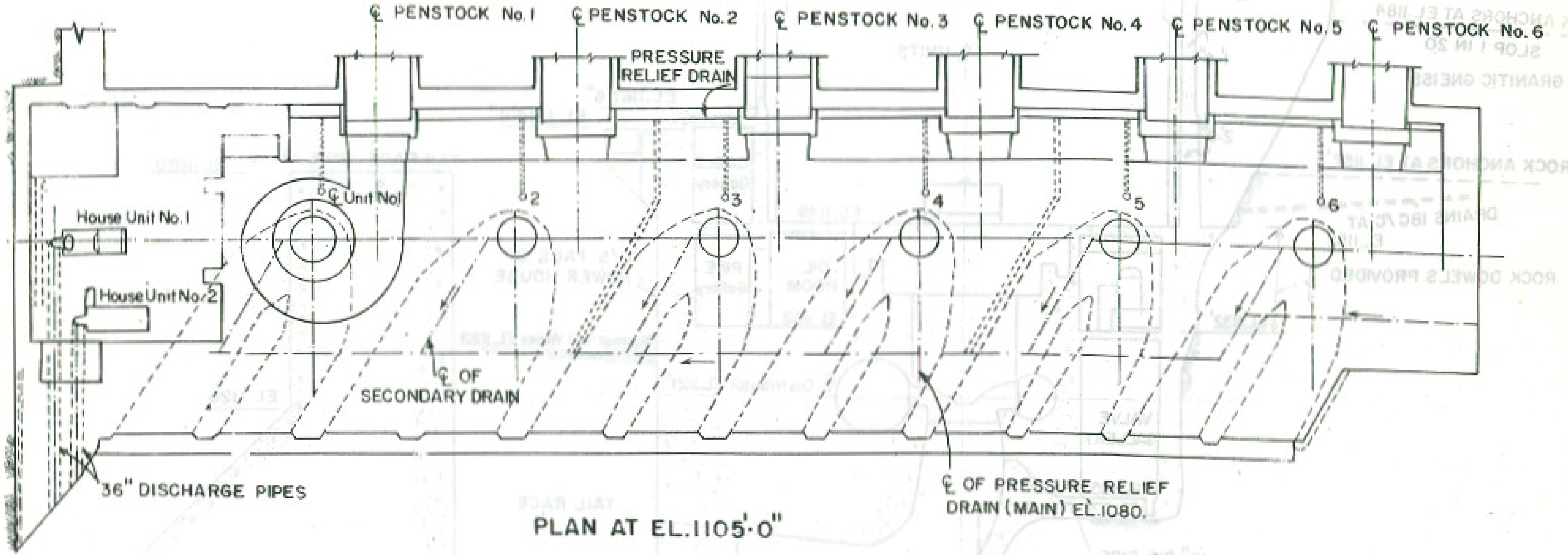
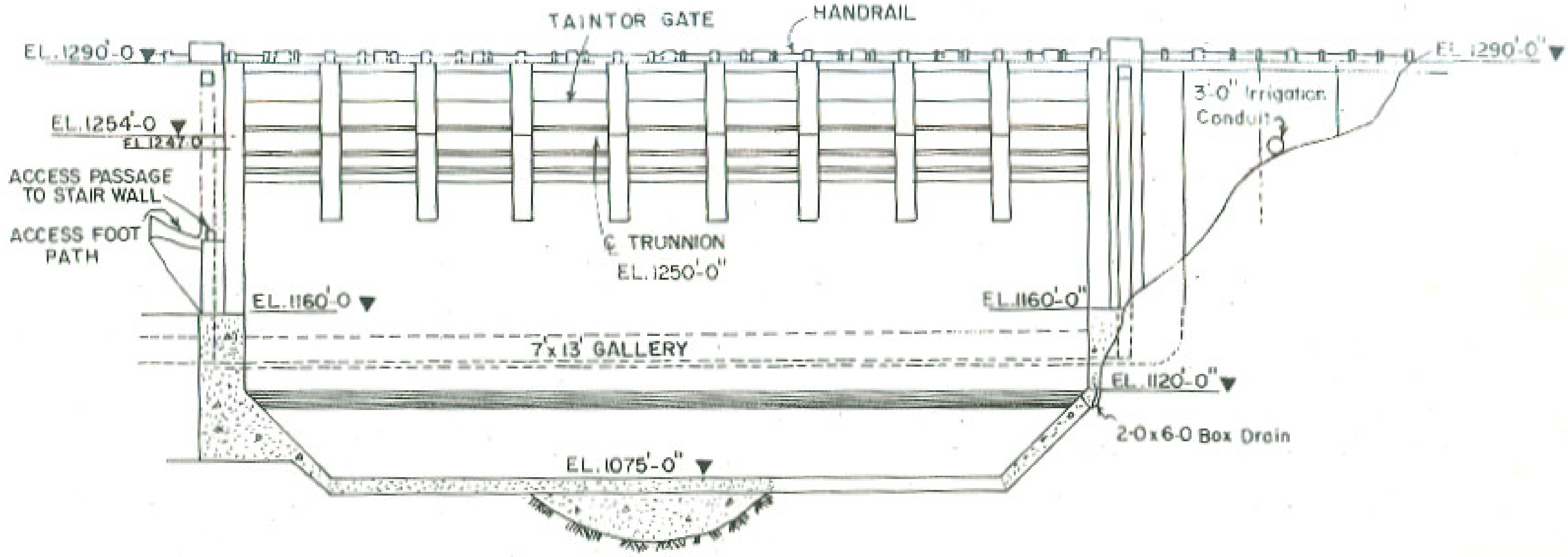


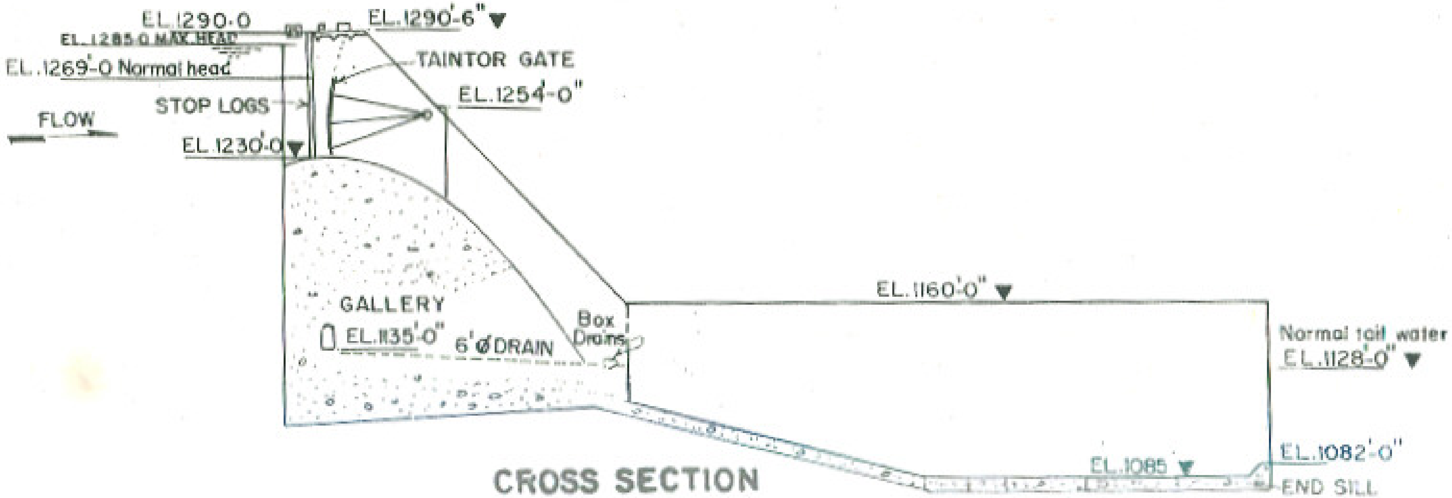
Fig-9

UNDER DRAINAGE ARRANGEMENT

WARSAK POWER HOUSE



LONG SECTION
MAIN DAM & ABUTMENTS



CROSS SECTION

Fig-10 MAIN DAM SPILLWAY CHUTE STILLING BASIN
WARSAK DAM