PAPER No. 461

Khanpur Dam Project

BY

Engr. Dr. Izhar-ul-Haq
Director Design, Central Design Office, WAPDA.

Paper No.461

KHANPUR DAM PROJECT

By

Dr. Izhar-ul-Haq (*)

1.0 INTRODUCTION

Khanpur Dam Project derives its name from the village Khanpur located at 8 miles North of Taxila on Haripur road and about 25 miles from Islamabad, the Capital of Pakistan. The dam has been constructed in a narrow gorge on Haro river downstream of the confluence of Haro and a tributary Nilan Kas. It is a multipurpose project. In addition to irrigation supplies (NWFP 110 cusecs and Punjab 87 cusecs), it shall supply drinking water to Islamabad 33 MGD, Rawalpindi 69.37 MGD and Taxila Industrial Complexes 28.5 MGD. Tourism and Fishries shall also be developed at the Project. The Khanpur village will be submerged in reservoir and a new Khanpur town has been built downstream of the dam.

After comparative study of a number of sites on Haro river, the final site was selected in 1962 and detailed studies were started. Initially the Project was conceived to be 137 ft. high but due to shortage of drinking water in Islamabad and Rawalpindi it was decided to raise the height to 167 ft. It would store 1,06,000 acre feet of water. The project consists of main dam, a check dam, two saddles, a gated spillway, diversion tunnel, outlet works and irrigation canals. Layout of the project is shown in Exhibit No.1.

Feasibility and design studies were carried out by Associated Consulting Engineers and reviewed by Wapda. Machinery Pool Organization, Wapda started construction in 1968. Upto 1971 saddle dams, check dam, a part of spillway and main dam upto RL 1960 except for river closure section

---

(*) Director Design, Central Design Office, WAPDA, Lahore.

were completed, when the work was suspended due to certain reasons. The river closure work and completion of dam upto crest was done by Frontier Works Organization from June 1982 to June, 1983. During the Monsoon of 1983, the reservoir was filled upto spillway crest level. The Canal System was complete and irrigation supplies have been started but the municipal supplies have yet to be taken up as the beneficiaries are not ready. The life of the project is about 75 years. The project has been completed at a cost of Rs 1,352 millions.

SALIENT FEATURES

Location	Haro River, Distt. Abbottabad.
Catchment Area	308 sq. miles.
Gross reservoir capacity	107,076 acre feet.
Live storage	91,562 acre feet.
Life of Project.	75 years
Mean Annual run off.	280,000 acre feet.
Max. flood anticipated.	182000 cusecs.

MAIN DAM

Type	Earth and Rockfill
Top of Dam	RL 1992
Max. water level	RL 1982
Deepest river bed level	RL 1825
Length at crest.	1545 ft.
Height.	167 ft.
Length of impervious blanket.	3000 ft.

SADDLE EMBANKMENTS

Length of Right Saddle	2324 ft.
Height of Right Saddle	54 ft.
Length of Left Saddle.	1842 ft.
Height of Left Saddle.	47 ft.

Length of Check Dam.	622 ft.
Height of Check Dam.	70 ft.
Length of Malakanwala Dyke	1555 ft.
Height of Malakanwala Dyke.	15 ft.

SPILLWAY

Capacity	166000 cusecs
Crest	RL. 1947
Gates	5 Radial, 40 ft x 35 ft.
Crest Length	232 ft.

IRRIGATION OUTLET

Capacity	650 cusecs
Sill level	RL 1902
Gross commanded area	50,390 acres.
Culturable command area	36,470 acres.
Municipal & Industrial Supply.	131 mgd = 243 cusecs.

DIVERSION TUNNEL

Type	Horse shoe shaped concrete lined.
Dia	10 ft.
Length	1000 ft.

COST

Total cost	Rs. 1,352 million
Foreign Exchange	Rs. 0.879 million
Benefit Cost Ratio	1:2

2.0 REGIONAL GEOLOGY

The stratigraphic sequence in the catchment of Haro River is as follows:

1. RECENT.	Subrecent to recent.	Alluvium; sand, silt and gravels.

2. CANEOZOIC	Miocene	Sandstone and shales (Murrees)
3. MESOZOIC	Cretaceous Jurrasic.	Sandy Limestone Sandstone, Spitti Shale and Limestone.
4. PALAEOZOIC	Cambrian or Precambrian	Slate Series.

2.1 Geologic Structure

The Project area lies in the Hazara Thrust Fault System. The Margala hill ranges at Khanpur Dam Project are traversed by a system of nearly parallel northeast/southwest tending faults. These faults join the Himalayan thrust along a syntexial bend towards northeast and to Kirthar, Sulaiman fault zone, towards southwest.

The area around Khanpur Dam is tectonically disturbed and is severely folded, faulted and at places overturned. The axes of the major folds on both the abutments are parallel to sub-parallel and have eastwest orientation. The folding and faulting is attributed to the post eocene major regional tectonic activity. The folds are plunging towards west and their axial planes dip towards south.

2.2 Seismicity

The dam is located in an active tectonic area. It is generally believed that the Taxila civilization, only 12 km from the site, was destroyed by an earthquake.

The recorded earthquake at Tarbela, and Nilor (Rawalpindi) are:

Magnitude	Hypocential Distance Approx. (Km)	Event
5.3	5	1
4.9	5	1
4.8	5	1
4.7	5	1

The recurrence of major earthquakes has been as:

Earthquake Magnitude.	Recurrence in years
4	2
5	12
6	66
7	380
7.5	912

From the above it is inferred that a maximum expectable earthquake at Khanpur Dam Project could be of magnitude 6.2 with a return period of 75 years, while for the Maximum Credible Earthquake (MCE) the magnitude would be of the order of 6.8.

Regional fault passes only 12 Km from the site. Seismic studies considering the regional fault and its rupture length of 65 Km with the MCE gives 0.28g acceleration at site and 0.2g for max. expectable earthquake.

3.0 FOUNDATION INVESTIGATION

In the first phase 22 bore holes were drilled but as they were not properly located much useful information could not be gathered from them.

In the second phase a geological plan covering the site of the main dam and appurtenant structures was prepared on a scale of 1" = 200'. During this phase 63 more rotary and percussion bore holes were drilled and pits

excavated. Some trenches were done for the purpose of mapping the contacts below the overburden. Few adits were also driven to see the condition of the rock insitu, see Exhibit No.2. Water pressure tests were carried out in the rock while permeability tests were conducted in overburden. A number of pits were dug specially in the burried valley area to investigate the extent of silt cover and evaluate the permeability.

4.0 GEOLOGY OF PROJECT AREA

The rocks exposed in the area belong to the Eocene age and consist of the following:

1. Massive Limestone (Lm)
2. Nodular Limestone (Ln)
3. Argillaceous Limestone (La)
4. Shale (Sh)

As a result of tectonic movements, the strata are tightly folded and faulted at places. The shales being softer have been eroded faster and form deep valleys, the resistant limestone form ridges. The strata have been folded into overturned anticlines and synclines. The major anticline is confined to the valley and synclines occupy the abutments. The Haro River roughly follows the axis of the anticline. After eroding the upper limestone the river has cut the underlying shale to a depth of more than 200 ft. The 'V' shaped valley thus formed was later on filled with alluvial deposits which consist mostly of gravels and silty sand. Exhibit No.3 shows the surface geology of Main Dam and spillway area.

4.1 Valley

The river gorge section consist of alluvial deposits varying from boulders to silty material underlain by shales. The thickness of alluvial deposits exceed 250 ft.

at places and they extend about 600 ft. between the abutments.

The depth of the gravels decrease upstream along the bed of Haro River and the Nilan Kas. At a location, about 3,450 ft. upstream along the Haro River bed, the rock is met with at less than 30 ft. depth and in Nilan Kas the rock is encountered at only 9 ft. depth at a distance of about 3,500 ft.

In between the Haro and Nilan Kas is a saddle formed by outcrops of limestone and a burried valley, which consists of gravel deposits extending over 200 ft. in depth at places. The burried valley at its upstream extremity joins the present course of the river underneath a cliff at Malawalla gardens where there is a vertical exposure over 1,000 ft. in length of gravel deposits.

Parts of the saddle are covered by overburden of clayey silt material. These areas are distinguished by the development of Azizpur and Basharat Gardens.

4.2 Right Abutment

The right bank of the dam area consists of a typical overturned syncline. Shale (Sh-1) occupies the core of the syncline and beds of different limestones flank the shale from three sides. The abutment of the dam is founded on Argillaceous limestone (La-9) Massive Limestone (Lm-8) of the downstream limb of the syncline. Exhibit No.4 shows the geological section along the dam axis and describes the rock units in the area.

General strike of the beds in the abutment varies from normal to about 30° NW with the dam axis and the dip varies from 30° to 80° towards south. The continuation of these beds towards the spillway show change in the direction of strike specially towards the right flank. Minor flexures have also been mapped in the area but no

fault with definite throw has been mapped. A fault in the flip bucket area has been shown, the interpretation of which is based on the structural anamoly where the upstream beds of La-9, Lm-8 and Ln-7 have come against Lm-6. The fault trends E-W with a dip of 28° N and becomes obscure in the valley section. Other faults shown in the upstream side have no implication since they do not travel down to the dam foundation.

4.3 Left Abutment

Lithologically left abutment is very similar to the right abutment and consists of Massive, Nodular and Argillaceous limestone with shale which has limited exposures but has been encountered in the bore holes, diversion tunnels and Adits. The strike of the beds is nearly parallel to the river flow and dip varies from 20° to 50° towards south i.e. away from the river.

4.4 Joints

Joints are well developed in the area specially in Limestone beds. The joints having following orientation are most prominent.

Strike	Dip
1. N 60 W,	50-60 SW (Bedding Joint)
2. NS,	40-65° E
3. NS,	30-35 W

Others have random orientation and are well spread over the area. On the surface the joints are 1 ft. to 6 ft. apart and occasionally less than 1 ft. In the adits the following observations were made:

i) Prominent joints are generally spaced 2 to 5 ft. apart.
ii) About 45% joints have width upto 1" while 34% have an opening of 1"-5". 9% of them are 5" to 10" open and 12% of the joints are tight.

iii) 58% of the joints are either filled with clay or clay associated with calcareous material 31% joints have a filling of only calcareous material while the rest are unfilled.

5.0 CONSTRUCTION MATERIAL

5.1 Impervious Material

Deposits of impervious material quite consistent and uniform in properties are available within one mile reach on either bank, Classification, Proctor compaction, consolidation, shear and permeability tests were carried out on the borrow material.

The gradation tests show that the impervious material cosists of mainly clayey silt; silt (60 to 70%) and clay (20 to 30%). The gradation curves are given in Exhibit No.5. The plasticity index varies from 6 to 16. Maximum Proctor density = 112 lb/ft^3 and optimum moisture content = 15%. The permeability was of the order of 10^{-7} cm/sec. The shear parameters obtained on samples, compacted at 95% Proctor density were C' = 0 to 144 Psf

$\emptyset$ = 28° to 32°

5.2 River Gravels

The gradation of river gravels is shown in Exhibit No.5. The insitu dry density was 139 lbs/ft^3. It was specified that minus 9" material from the river bed shall be used in construction of embankment. The average content of fines passing N°200 sieve was 3.5% and 20% size corresponded to N°4 sieve size. The coefficient of permeability was of the order of 10^{-1} cm/sec. The gravels were assumed to be free draining.

5.3 Filters

Filter F_1 is medium to coarse sand. Passing

sieve N°4 and not more than 5% passing N° 200. Lawrencepur sand meets the requirement. Filter F_2 varies from 1" to sieve N° 30, Fine river gravels or rock crushed to this size. Filter F_3 varies from 3" to sieve N°4. Coarse river gravels or crushed rock. Average value of dry density was taken as 110 lb/ft³.

5.4 Rockfill

Large quantity of rock was available from excavation of spillway and diversion tunnel which was stock piled and used in the d/s shell of dam. Maximum rock size was 2 ft. and percentage passing No.200 sieve was 5 to 10%. Dry density was 110 lb/ft³ and Bulk Density was 130 lb/ft³.

6.0 MAIN DAM

6.1 Foundation

The foundation of main dam rests on alluvial deposits varying from boulders and sand to silt. The gravel deposits are generally very compact with high densities. From the point of view of strength, the gravel deposits of Haro River provide a stable foundation. There was no continuous layer of silt or clay except a surface deposit of silty clay near the right edge of river.

6.2 Embankment Design

In order to make best use of construction material available at site both from borrow areas and excavation of spillway, a composite earth and rockfill dam was designed. The maximum height of the dam from river bed to crest at RL 1992 is 167 ft. and length at crest is 1,550 ft. In the first instance, the dam was designed with the u/s side consisting of an impervious fill of clayey silt on slope 3 H to 1V which gradually flattened

and finally joined with the upstream blanket and on the d/w side the rockfill on 2.5 H to 1 V slope. The embankment is 30 ft. wide at crest and has 30 ft. wide berm on u/s side at RL 1888 which is also the top of the u/s coffer dam which was merged into the body of main dam.

Transition zones F1, F2 and F3 of graded filters each 8 ft. wide and free draining gravel fill had been placed in between the impervious fill and the d/s rockfill. A horizontal filter blanket connected with the chimney drain was placed under the d/s rock shell. This would convey the seepage from the body of the dam to the d/s toe and intercept seepage emerging from the foundation under the d/s shell. The u/s slope was protected with stone pitching having a transition layer of subgrade material underneath. The u/s slope of coffer dam was protected with random rockfill.

The flood routing calculations for the maximum design flood of 182,000 cusecs indicated a rise of the reservoir to a maximum level of 1982. The elevation of the top of the dam was fixed at 1992 providing a clear free board of 10 ft.

The impervious fill was placed in 9 inches layers and compacted with sheep foot rollers to 95% proctor density at optimum moisture content or within -2% of optimum. The filters were compacted in 9 inches layers with vibratory rollers. The rockfill was compacted in 2 ft. layer with vibratory rollers.

The construction of main dam was completed from ch. 6+25 to ch. 15+45 upto RL 1961 in November, 1971. The top of the partially completed embankment was sloping from ch. 1000 towards the river bed at a slope 2.5H to 1 V. The construction work was suspended in December, 1971 and could not be restarted till 1982 due to certain reasons.

The effect of exposure to weather conditions for more than 10 years was noticeable on the slope towards channel in the form of deep runnels in the impervious fill. However, the u/s slope was protected with stone pitching. During 1980-82 Wapda reviewed the design and construction aspects of Khanpur Dam Project and recommended that gravel shell be provided in the u/s section of closure dam and the already built section.

6.3 Stability Analysis

Stability analysis of u/s and d/s slopes was carried out by using the Bishop's slip circle method of slices in pseudo-static analysis using the following parameters.

Zone	Unit weight (Pcf)		Shear strength	
	Bulk	Saturated	C' psf.	degree
Impervious fill	110	130	0	28
Filter	120	130	0	35
Free draining gravels	130	140	0	35
Rock fill	130	140	0	35
Riprap	140	-	0	40

The following cases were analysed:

6.3.1 Construction Case

The moisture in the impervious fill required for its compaction gives rise to pore pressures during compaction. The stability of the embankment was checked using pore pressures equal to two thirds of the total weight of the overlying embankment and using the total stress parameters for shear strength. The embankments were found to be safe during construction.

6.3.2 Steady Seepage

The stability of d/s slope was examined for the condition of full reservoir and steady seepage. The pore pressures within the impervious zone were assumed to develop for the full vertical height between the seepage line and the base of the slip circle. The above condition was also combined with earthquake and the embankments were found to be safe.

6.3.3 Sudden Drawdown and Earthquake

The water level was assumed to fall rapidly to the lowest possible level, outlet level (RL 1902), such that the water in the impervious zone below the seepage line was not able to drain out, thus developing pore pressures within the impervious zone. The rockfill and filters were assumed to be free draining.

The effect of earthquake was also combined with the sudden drawdown but upto spillway crest R.L. 1947. Initially the dam was designed for a pseudo-static horizontal force of 0.1g due to earthquake but towards its completion it was checked for 0.15g and made safe for it.

Stability analysis of main dam gave the following factors of safety:

U/s slope impervious fill section

Sudden drawdown.	= 1.42
Sudden drawdown combined with 0.1g earthquake.	= 0.88

D/s Slope

Steady seepage	= 2.0
Steady seepage combined with 0.1g earthquake.	= 1.56

U/s Slope modified Section

Sudden drawdown combined with 0.1g earthquake. = 1.20

Sudden drawdown combined with 0.15 g earthquake. = 1.02

U/s Slope Closure Section

Sudden drawdown combined with 0.1g earthquake. = 1.28

Sudden drawdown combined with 0.15g earthquake. = 1.05

6.4 Modification in the Main Dam Cross Section.

The stability analysis showed that all embankments were safe both under the steady seepage and the rapid drawdown conditions.

The saddle embankments were marginally safe for rapid drawdown condition combined with earthquake acceleration of 0.15g.

The Check Dam was just marginally unsafe for the severe conditions with the conservative parameters taken into consideration.

The saddle embankments and check dam are of less height as compared to the main dam and were already completed in all respects.

The main dam was built from Ch.6+25 to 15+45 upto RL 1960. The closure section in part of the active river bed was yet to be built.

Several ruts had been formed in the already built main dam section which had been left exposed for the last 10 years. The dam was cut back to the depth of ruts, upto fresh surface, and on a slope not steeper than 2 H to 1 V.

It was decided to place gravel shell 130 ft. wide u/s of impervious core in the already built section of main dam starting from RL 1947, the crest of spillway, the level upto which the drawdown would be rapid as shown in Exhibit No.6a.

The closure section was built with a proper free draining gravel section on the u/s as shown in Exhibit No.6b. This section is safe for rapid drawdown combined with the 0.15g earthquake.

The d/s section of the closure dam was the same as the already built dam.

For integration of the closure dam having a free draining gravel shell with the already built part which had only the impervious fill, a well graded transition which was compatible with both the above materials was placed between the two. The transition material was made by blending the silt with processed river run sandy gravels.

6.5 Dynamic Analysis

The effect of an earthquake is to induce inertia in the embankment which change in direction many times during the period of the earthquake. These inertia forces act in conjunction with the dead load stresses acting on the dam at any instant of time.

Dynamic Response Analysis were carried out for Khanpur Dam Project for MCE = 6.8 and A max = 0.28g of 25 seconds duration using the computor programme "SHAKE".

The following parameters were used:

Poison ratio:

Impervious fill	= 0.3	Rockfill	= 0.2
Foundation gravels.	= 0.25	Bed rock	= 0.2

G max = Shear modulus at low strain (10^{-4}) was determined analytically.

G max = 3000 Kips/ft^2 was used in the design.

Damping value was taken 25% for fill and 5% for foundation material.

6.5.1 Max. Acceleration

For simulated Khanpur earthquake with 0.28g excitation at base of modified section of main dam, the maximum acceleration at crest works out to be 0.42g.

6.5.2 Deformation

Since the Khanpur Dam embankments have been designed to withstand dead load stresses and the inertia forces induced by the earthquake, transient in nature, the ground motion will induce permanent displacement only when they induce stresses exceeding the yield strength of the material comprising the body of the dam. However, as the inertia forces reverse in direction or the acceleration is reduced, the deformation is arrested and the f.o.s, exceeds unity. Thus the total deformation is the cumulative sum of displacements due to the above condition over the earthquake. The deformation depends on the time history of inertia forces.

For the main dam the maximum permanent displacement of 8 inches in the upper part while negligible displacements would occur in the lower parts. For Check Dam and saddle embankments founded on silt, the maximum permanent displacement would be of the order of 14 inches.

Such displacements are in no way catastrophic. There would be some encroachment on the free board which can be set right. The insitu densities of silt under the saddle embankments were in the medium range. The dynamic

shear tests were also carried out on the intact silt samples which did not indicate the liquefaction potential under severe shaking.

7.0 SADDLE EMBANKMENTS

Four saddle embankments have been constructed to close the gaps in the periphery of the reservoir. Total fill required for the saddle embankments was 1.2 million cu.yards. All the saddles were homogenous earthfill embankments except check dam which was earth and rockfill.

7.1 Left Bank Saddle Embankment

The embankment had been constructed to plug a gap in the left flank of the reservoir, part of it runs parallel to the Nilan Kas, see Exhibit No.7a. The embankment had already been constructed prior to the decision of raising the height of the dam from RL 1965 to RL 1992. The toe drain and turfing from the rear slope was removed to accommodate the revised design.

7.1.1 Geology and Foundation

Major portion of the L-shaped saddle embankment was founded on alluvial soil consisting of clayey silt and the remaining portion founded on rock. Massive Limestone was exposed in the ridge between RD 620 and 950. It had numerous joints of varying sizes along bedding planes which ran in eastwest direction and a few transverse joints also, alongwith numerous pot holes and cavities. The direction of bedding joints was practically parallel to the axis of the saddle embankment.

The top elevation of the ridge after removal of overburden and clearing of rock was 1978 which was 4 ft. below the reservoir level. A cut off trench, 5 ft. deep and 5 ft. wide at the base with side slopes of 1:1

was provided. At RD 950 this trench joined with shale bed. Beyond this RD, the alignment followed the natural valley along the continuation of this shale bed which provided a natural cut off upto left abutment.

Near the left abutment, the alignment ran through alternate bands of shale and Massive limestone from RD 00 to RD 100. From RD 100 to RD 620 the alignment crossed the natural valley having clayey silt in the foundation. The massive limestone had several discontinuities which were dentally treated.

7.1.2 Embankment Design

The embankment was designed as homogeneous earth fill section with a downstream horizontal filter blanket. The width of the embankment at the crest is 25 ft. Exhibit No.7b shows the typical x-section.

Soil Parameters		Main Features	
C'	= 0	Length of embankment	= 1842 ft.
ø'	= 28°	Maximum height	= 45 ft.
γ_d	= 110 pcf	Upstream slope	= 3:1
γ_{sat}	= 130 pcf	Downstream slope	= 3:1

7.2 Check Dam

The Check Dam is located in the valley between the spillway and the ridge where Project Colony is situated. The dam was constructed to plug a low saddle having a minimum foundation level of 1922. It also provided a cover over the right abutment of spillway where limestone was exposed. The surface protection of the above area was accomplished by wrapping the slope with random rockfill and stone pitching in continuation of the check dam embankment.

7.2.1 Geology and Foundation

The dam was founded on shale (sh.1). The left abutment consisted of limestone and Argillaceous limestone bed overlain by Massive Limestone. The Massive Limestone exhibited higher permeabilities therefore it was considered necessary that the grout curtain under the spillway be extended into the right abutment of Check Dam. Exhibit No.8 shows plan and typical x-section of Check Dam.

7.2.2 Embankment Design

The upstream impervious fill was obtained from the borrow area while the rock excavation from spillway was utilized in the rockfill zone of embankment. Filters F_1 and F_2 was brought from Lawrencepur while F_3 and sub-grade material were processed from Haro River. The major consideration in designing the section of Check Dam were unusual topographic features of the valley and the economic consideration in utilizing the maximum quantity of the rock available from spillway excavation. Due to the higher topography on the d/s side, the drainage of the downstream slope and that of d/s filter blanket was collected in the toe drain and disposed of through a tunnel which opened into the right flank of the spillway chute.

Main Features		Soil Parameters	
Length of the Dam	= 622.5 ft.	C'	= 0
Height (Max)	= 77 ft.	Ø'	= 28°
U/s Slope	= 3.5:1	γ_d	= 112 Pcf
D/s Slope	= 3:1	γ_{sat}	= 130 Pcf
Top width	= 25 ft.	K of impervious fill.	= 5×10^{-6} cm/sec

7.3 Right Bank Saddle Embankment No.1

The saddle embankment spans the low lying area on the right bank of Haro River. It connects ridge of Project

Colony with a minor ridge to its north.

7.3.1 Geology and Foundation

The saddle embankment rested on the alluvial foundation of clayey silt same as that of the embankment. The alluvial strata was underlain by limestone and shale. Both abutments consists of outcrop of Massive Limestone.

Soil Parameters		Main Features	
C'	= 0	Length	= 2324 ft.
ø'	= 28°	Max. height	= 54 ft.
γ_d	= 112 Pcf.	U/s Slope	= 3.5:1
γ_{sat}	= 130 Pcf.	D/s Slope	= 3:1

7.3.2 Embankment Design

The original design of this embankment with crest level at 1965, comprised of a homogeneous earthfill embankment with upstream and downstream slopes of 3:1 and 2.5:1 respectively with a rockfill toe. Work on this embankment including construction of rockfill toe and protective filters and revetment of upstream slope had practically been completed. While redesigning the modified higher embankment, an effort was made to retain as far as possible the work that had been already executed specially the rockfill toe and filters. A horizontal filter blanket forming an extension of the rock fill drain was provided. The riprap and gravel sub-grade on the upstream slope were removed and relaid, Exhibit No.9a.

7.4 Malkanwala Dyke

This saddle has a maximum height of only 11 feet and connects the right bank ridge of saddle No.1 with higher ground beyond the habitation known as "Malkanwala Mora". In view of the small height of this saddle, it was constructed of a homogenous embankment of clayey silt

with upstream and downstream slopes of 2.5:1 and 2:1 respectively. Since the lowest ground level in this area was 1981 to 1982 i.e. about the same as the maximum water level, the embankment will not be retaining any water and no arrangement for internal drainage was therefore required. For protection against wave splash and against erosion due to rainfall nominal 12" thick riprap and stone pitching was provided on the upstream and downstream slopes, Exhibit No. 9b.

8.0 SEEPAGE CONTROL

In order to store water in the reservoir created by the dam it was necessary to prevent excessive seepage through alluvial deposits under the river bed and from the rim of the reservoir. Studies were carried out to check the effectiveness of partial cut off, positive cut off and an upstream impervious blanket. It was found that impervious blanket was superior to partial cut off. However, the provision of a positive cut off involved huge expenditure of foreign exchange, the difficulty of arranging the same and the Technical know-how available at that time in the country for such a deep cut off was the biggest argument against the provision of positive cut off. As such it was decided to lay an impervious blanket in continuation of the core.

Permeability tests were carried out in the valley section and burried channel which gave following average values:

K horizontal = 2.5×10^{-1} cm/sec.

K vertical = 4.1×10^{-3} cm/sec.

8.1 Impervious Blanket

The blanket is 3,000 ft. long and constructed of impervious fill material (clayey silt) used in the

embankment. Before laying the blanket, a transition was formed by blending the river gravels with siltysand material so that the blanket was not washed into the open work gravels. At the toe of the dam the blanket was 20 ft. thick and sloped down to 10 ft. in a distance of 100 ft. The thickness of the blanket was governed by the hydraulic design considerations and reduced to a minimum of 5 ft. at far extremity. A key trench of the impervious blanket was made at the terminal point in the stream bed. The blanket was placed in 9 inch layers and compacted to 95% proctor density. The top blanket surface was covered with 2 ft. of gravel layer for protection.

The blanket was tied with the abutments by means of carefully prepared trenches so as to attain a creep length equal to 1/3 of the differential head at that point. These trenches were properly backfilled with impervious fill after cleaning and suitable dental treatment to rock surface. Where impervious rocks such as shale or argillaceous limestone were not met and the limestone was exposed, shallow grout curtain 50 ft. deep using clay cement grout was installed to reduce leakage of water through open joints.

The topographical irregularities coming within the limits of the blanket have been treated so as to give a uniform slope of 1:3, or even flatter to the finished surface of the blanket.

Construction of the blanket was phased in two stages In the first stage the blanket was constructed on areas above the active stream beds and in a diversion channel leading to the diversion tunnel. In the second stage the streams were diverted through the channel and the blanket was laid in the active stream beds. Before the final impounding of reservoir, the eroded blanket in the diversion channel was restored by rediverting the streams.

With the above length of the blanket if it works all right, the exit gradients at the toe of the dam would be quite low and there would be no problem of sand boiling. However, in order to relieve pressure at the d/s toe of the dam, relief wells about 100 ft. deep at an initial spacing of 100 ft. were designed. The performance of impervious blanket would also improve with siltation with the passage of time.

8.2 Grout Curtain

Due to the consideration that the limestone specially Massive Limestone and Nodular Limestones have numerous discontinuities which may allow the seepage and may cause uplift pressures, a grout curtain in each abutment was constructed. Prior to taking up the grouting, a trench was excavated down to the sound rock and a 3 ft. thick grout cap was also provided.

8.2.1 Right Abutment Grout Curtain

The foundation rock in the abutment is mostly Massive Limestone (LM-8). Overlying Nodular Limestone (LN-7) and underlying Argillaceous Limestone (La-9) have limited exposures. To check/reduce the seepage a grout curtain had been provided. In addition to trenching and grout cap, dental treatment was also provided on the upstream region of right abutment due to the presence of solution cavities and interlinked open joints visible at the surfaces. A few of these discontinuities were normal to the dam axis. On the right abutment the curtain starts from its junction with the periphery curtain at RD-1255 and goes upto RD 1545, and it extends under the spillway and check dam.

The curtain initially had two rows spaced at 5 ft. from RD 1255 to 1545. The depth of the holes

varies from 100 to 142 ft. During the grouting operation of primary holes which were spaced at 10 ft, there was heavy intake. Three holes indicated open zone conditions almost throughout their depth. Holes of the secondary phase at 5 ft. interval indicated improvement but still 5% of these holes showed open conditions. Consequently further holes of the tertiary series were drilled and grouted along the middle line upto the full depth of curtain. To improve the condition further, holes in the fourth phase were drilled at 2.5 ft. spacing. The curtain had not been tied to any impervious rock.

8.2.2 Left Abutment Grout Curtain

The foundation rocks in the left abutment are shale, Massive Limestone and Nodular Limestone. A grout curtain has been provided in this abutment which extends from RD 10 to RD 355 where it is tied to underlying shale bed. The spacing of the two grout rows was kept at 3.5 ft but later on by improving the technique, the spacing was increased to 5 ft. Portion of the grout curtain which falls in the vicinity of diversion tunnel was treated separately. The grout holes from the top were terminated 20 ft. above the tunnel and further drilling and grouting was done from inside the tunnel through radial holes. Depth of curtain varies from 50 ft. at lower end to 185 ft. near the top. A few prominent bedding joints and open solution channels were also encountered on this abutment. They generally follow the strike and open up in drainage gallery.

8.3 Drainage Curtain

Drainage curtains were constructed in each abutment of the Main Dam and along the crest line of spillway a short distance d/s of the grout curtains. Grout curtains

on abutments are vertical and are along the axis of the dam. Drainage curtain consists of inclined holes which have been drilled through the drainage galleries. These galleries were developed from exploratory adits 40 ft. to 100 ft. d/s of the dam axis. Drainage curtain has been constructed along the centre line and the holes open up in the gallery.

Drainage Galleries

Permanent drainage galleries had been provided at downstream side of both the abutments as well as under the spillway through which drainage holes were drilled. Drainage holes were placed upwards and downwards from these galleries so as to relieve any uplift pressures which may develop d/s of grout curtain and to intercept any leakage. P.V.C. strainers with shrouding material were placed in all the drainage holes.

8.3.1 Drainage Gallery Under Left Abutment

The exploratory adit No.1 was enlarged for the purpose of drainage. The size of this gallery after lining was 6 ft. x 8 ft. and its length was 243 ft. The inlet of the adit which was within the main dam fill was plugged with concrete and access to the gallery was made through the exit of the diversion tunnel.

8.3.2 Drainage Gallery Under Right Abutment

A part of the exploratory adit No.5 was used for this gallery alongwith a new extension of 155 ft. excavated towards the spillway crest. The entrance of adit No.5 was within the base of main dam fill therefore a suitable structure was constructed for access.

Dr. Izhar-ul-Haq

9.0 IRRIGATION OUTLET STRUCTURE

The Irrigation Outlet Structure is the Structure through which water will be released from the reservoir into the canal system. It is located in the Right Bank Saddle Embankment at chainage 4+00. This outlet will cater irrigation demands of the Right Bank and the Left Bank command areas as well as the Municipal requirements of twin cities of Rawalpindi and Islamabad besides meeting the industrial needs of the various Engineering Complexes situated at Taxila and Wah Cantt.

The discharge capacity of the outlet is 650 cusecs. The structure consists of an intake tower, a control tower, a conduit, radial gates and a stilling basin. The regulation will be done by radial gates at downstream end of conduit. The layout and longitudinal section of irrigation outlet strcutre is given in Exhibit No.10. The salient features are:

9.1 R.C.C. Conduit

The length of cut and cover conduit is 665 ft. out of which 303.66 ft. on the upstream side of control tower is of circular shape with an internal diameter of 6.5 ft. The downstream portion of 361.34 ft. is of rectangular shape with 5.5 ft. width and 6.5 ft. height. The conduit was placed on fresh cut rock. Where rock was deeper, the alluvium was excavated and replaced with lean concrete. Concrete cut off collars were provided around the conduit and compacted impervious backfill was placed around the collar.

9.2 Intake Tower

An open approach channel leads the water from the reservoir at the end of which an intake tower with

trash rack measuring 49.5 ft. x 25.5 ft. and 35 ft. high is provided. The entire structure is submerged under a maximum head of 80 ft. at full reservoir level of 1982 and is designed for adequate inflow area, assuming 50% clogging due to trash. Provision was made for closing of the part of Intake Tower with stop logs upto which it was silted up.

9.3 Control Tower

The control tower, 93 ft. high was located at RD 307.67 of right saddle embankment. Control Tower was designed as a rigid R.C.C. box structure embedded in the embankment. Arrangement of hoisting the bulk head gates was provided at the top. The gates operate in a groove 6"x6" provided in the entire height of the tower. Bulk Head Gate 8'x5'-6" and fixed wheel gate 6'x5'-6" were provided so that one serves as an emergency gate in case the service gate requires repairs.

9.4 Stilling Basin

At R.D. 675 the conduit ends and stilling basin starts which has a length of 67 ft. upto offtake of the Canal. The sides of the stilling basin were designed as R.C.C. retaining walls.

10.0 HYDROLOGY

Haro River rises in the Galian and Murree range of hills and has a catchment area of 308 sq. miles upto Khanpur Dam Site, out of which 202 sq. miles i.e. 66% is above 3,500 contour. The mean weighted rainfall in the catchment area amounts to 49.14 inches. The precipitation is distributed throughout the year though two periods January to April and July to September are distinguished by higher precipitation. The run off is characterised by

the high flows when the monsoons are active. The river is mainly fed by monsoon rains with small contributions by snow melt. The base flow of the river is of the order of a hundred cusecs, which in a dry year can be half the value. Taking 35% as a mean coefficient of run off from the catchment gives an average annual depth of 17.5", which is equivalent to a run off volume of 300,000 acre ft. The figure of 256,000 acre ft/year was however taken in the design as the maximum.

Majority of the floods are of short duration and flashy in nature. These floods have sharp high peaks lasting about a day from rise to fall. In an average flood, the peak can rise to 5,000 cusecs though peaks as high as 50,000 cusecs have been observed. The studies carried out for the size of the reservoir and capacity of spillway show that maximum probable flood is 182,000 cusecs. A historic maximum discharge of one Lac cusecs has been estimated in the year 1929 computed on the basis of flood marks on the Railway bridge on Taxila Havelian Railway embankment. Exhibit No.11 shows the unit hydrograph for probable maximum flood.

11.0 SPILLWAY

The spillway of Khanpur Dam Project is located on the right abutment between the Main Dam and Check Dam. A number of alignments and different proposals were considered and finally an ogee crest weir with gate at the above site was selected. A flip bucket at the downstream end of glacis to dissipate the energy was considered economical as compared to a stilling basin. Exhibit No.12 shows the plan and cross section of spillway.

The rocks exposed in the spillway area consist mainly of shale, Massive Limestone, Argillaceous Limestone

and Nodular Limestones. Due to the geological conditions, it was tfied that the control structure and discharge chute be founded on sound rock still a small outcrop of Argillaceous Limestone falls in chute area. There were shale beds in the approach channel area and the crest axis was offset by 30 ft. d/s from the main dam axis to avoid these shale beds.

In the approach channel of spillway, the beds dip towards d/s and are in proper sequence that is younger beds overlying the older beds but further d/s the older beds were seen overlying the younger beds indicating an overturned syncline which was confirmed by deep bore hole.

11.1 Approach Channel

The limestone (Lm6, Ln7 & Lm8) are exposed in the approach channel. These limestones have cavities and open discontinuities. Due to the presence of an overturned syncline and the grout curtain under the crest of spillway not being a positive cut off the water may find an access from the u/s and create uplift pressures under the chute of spillway and the flip bucket. Therefore, it was decided to blanket the rocks at lower levels and provide dental treatment at higher levels. The blanketing of the beds at higher levels would have required considerable excavation, huge quantities of blanket material and its protection because of variation of water level above the crest of spillway.

The bed level of Khanpur Dam spillway approach channel was fixed at 1933.0 just upstream of the crest. This provided the optimum conditions for max. discharge over spillway crest with crest level at 1947.0 Approach channel elevation was lined where max. water velocity was 10 ft./sec. The thickness of RCC lining

both in the bed and walls was 18 inches, with anchor bars grouted 8 ft. deep in bed rock. Reinforcement steel at the rate of 0.4% of gross concrete area distributed between upper and lower faces in the base slab was provided. The thickness of apron lining adjacent to the crest structure was gradually increased from 18" to 36" to withstand the uplift which might develop due to hydraulic drop in water surface when the gates were fully opened. The reinforcement was continuous through all joints with a minimum lap of 24 x dia. PVC water stops were provided at all joints in the approach channel lining and control structure.

11.1.1 Approach Channel Left Bank Guide Wall

The guide wall of the spillway approach channel was extended about 200 ft. u/s from the axis of the spillway on the left side. It is a massive gravity structure. Maximum permissible pressure on foundation was taken as 15T/sft. Following loadings were considered in design:

1. Dead loads consisting of the weight of the structures, with unit weight of concrete equal to 150 lb/ft^3.
2. Lateral earth pressure imposed by main dam embankment fill.
3. Hydrostatic uplift pressure.
4. Lateral differential hydrostatic pressure.
5. Lateral hydrodynamic pressures.
6. A horizontally directed earthquake force equal to 10% of weight of the wall.

The assumed design criteria and the various load combinations were as following:

1. The structure subject to the loading of self weight and water loads during conditions of maximum discharge listed as loadings 1,3,4 and 5. Water level on main dam side equal to 1983.5 and on spillway

approach channel side 1981.5. Factor of safety against overturning and sliding equal to or greater than 1.5.

2. The structure subject to the loadings of self weight hydrostatic uplift pressure and earthquake forces listed as loadings 1,3 & 6. Water level at RL 1982 on each side of the wall. F.O.S. against overturning and sliding equal to or greater than 1.5.

3. The structure subject to loadings of self weight, water loads during condition of maximum discharge and earthquake forces listed as loading 1,3,4,5 and 6. F.O.S. against overturning and sliding equal to or greater than 1.0.

11.2. Control Structure

11.2.1 Crest

An ogee shaped crest was constructed at elevation 1947.0. The net width of the crest was kept as 200.00 ft. The control structure consists of 5 radial gates of 40 ft. x 35 ft. with 4 piers of 8 ft. width each. The u/s slope of the crest was fixed at 1:1.

The crest is founded on nodular limestone and massive limestone. Although these rocks have bearing capacity well above the superimposed loads but as there were some cavities in them, consolidation grouting through 30 feet deep holes was done in an area 260 ft. x 90 ft.

One hundred feet deep grout curtain was provided under the spillway crest in continuation of the main dam and check dam curtain on the other side. The curtain remains within Lm-6 and Ln-7 in most of the area.

A drainage curtain through a drainage gallery was also provided under the crest of spillway d/s of the grout curtain. The drainage holes were provided with proper filters and strainers. In the design of spillway for P/H ratio of 0.343, a value of coefficient of dischrge 'C' equal to 3.75 was used. To minimise the effect of the sub-atmospheric pressure which may develop along the crest profile at partial opening of gates, the radial gates were fixed 5 ft. d/s from the apex of the crest. The max. head at the crest is 35 ft. whose one tenth is equal to 3.5 ft. Had the gates been fixed at the apex of the crest, 3.5 ft. would have been the max. sub-atmospheric pressure on the crest, but now this pressure would be reduced due to shifted position of the gates. The uplift pressure due to seepage was considered in the design of crest. Dissipation of 25% of total uplift head was considered at the location of drainage gallery which was quite conservative. The structural design of crest was based on the following assumptions/ conditions:

1. Reservoir level at 1982.0 and all gates closed.
2. A horizontally directed earthquake acceleration of 0.1g and earth movement towards upstream.

11.2.2 Pier Walls

The structural designs of pier walls was based on the following assumptions/conditions:

a. Full reservoir level and all gates closed.
b. A horizontally directed earthquake acceleration of 0.1g intensity acting parallel to the apillway axis, earth movement towards upstream.
c. Radial gates just raised so that the load is transferred to hoist.
d. Uplift as determined by the hydraulic gradient.

e. No live load on the service bridge.

Lateral Stress Analysis

i. Water surface level in reservoir at 1982.0. Both gates adjacent to pier closed. A horizontally directed lateral earthquake acceleration of 0.1g.

ii. Water surface level in reservoir at 1982.0. Both gates adjacent to pier closed. Horizontally directed lateral earthquake acceleration of 0.1g.

11.2.2 Abutment Wall

The backfill behind the abutment walls is of pervious nature. The active pressure of the backfill was taken as 80 lb/ft^3. In the stability analysis following loads were considered:

a. Self weight.
b. Rockfill load.
c. Uplift after rapid drawdown, assuming saturation level at RD 1970 behind wall due to seepage.
d. Earthquake force.

Permissible Stability Factors Used

a. Sliding

i. Sliding Factor (S.F) $= \frac{\text{Horizontal Force}}{\text{Vertical loads-uplift}} = 0.7$

ii. Shear friction factor $Q = \frac{f(\text{Vertical loads-uplift}) + rSaA}{\text{Horizontal Force.}} = 5$

where f= ratio of friction of concrete against rock = 0.65.
r= ratio of average to max. shearing stress at the base. = 0.5
Sa= Unit shearing strength of the base contact = 200 psi.
A= Area of base.

b. Overturning

Overturning factor = $\frac{\text{Total Balancing Moment}}{\text{Total overturning Moment}}$
= 1.5 for normal loading.
= 1.25 for earthquake loading.

Dr. Izhar-ul-Haq

The chute of Khanpur Dam spillway has an u/s slope of 1:3.4 and contraction ratio of 0.75 while the velocity at the full diacharge will be 95 ft/sec. In the model studies it was observed that the flow follows all along the chute without any cross waves except diamond pattern of waves which is a typical characteristics of this type of spillway.

In general 18 inches thick reinforced concrete base slab was provided. The thickness was increased near the crest and the flip bucket to allow for some possible up-lift pressures and to increase stability of the chute in the flip bucket area. The reinforcement was provided in a ratio of 0.4% of the gross concrete area, and was carried across the panel joints. Anchor bars spaced at approximately 7 to 8½ ft. centre to centre were provided. Depth of anchors was fixed generally at 8 ft. Greater depth was provided near crest and flip bucket in argillaceous zone.

11.3 Flip Bucket

The chute of the spillway terminates in a solid flip bucket. It was considered to be economical as compared to a conventional stilling basin. Hydraulic model studies were carried out and it was observed that a significant percentage of energy was dissipated in surface boils in the bucket.

The flip bucket was founded on massive and nodular limestone. Consolidation grouting was done upto RL 1790 under the flip bucket.

The various design parameters of the flip bucket were:

Routed design discharge	1,66,000 cusecs.
Radius	26 ft.
Lowest Elevation	RL 1825.38

Tail Water Level	RL 1833.00
Lip angle.	45°
Lip elevation.	RL 1833.0

The following design criteria were used for the flip bucket:

i) Hydrostatic and hydrodynamic loads on flip bucket floor for a water depth of 10 ft. and velocity of 100 ft/sec.

ii) A horizontally directed earthquake of 0.1g intensity.

iii) Uplift on structure as determined by the assumed hydraulic gradient line of sub-surface seepage.

iv) The rock bed d/s of flip bucket was assumed to be eroded upto the cut off base level.

Perforated 4 inches dia pipe drains along each lateral joint draining into three 9 inches diameter longitudinal drains were provided. Two intermediate outlets were provided near the upstream and downstream end of the counterfort retaining wall on the left flank. In the argillaceous zone, a graded gravel blanket drain was provided.

11.4 Plunge Pool

The flow passing down the spillway chuts would be deflected by the flip bucket into the tail channel which ultimately joins the river. Protective measures in the tail channel below the bucket were necessary to prevent possible erosion and scour caused by high velocity flows as indicated by the model studies. The works involved were: Sloping glacis, the pool, right flank protection, left flank protection and spur dyke. Exhibit No.13 shows the plan and cross sections.