PROBLEMS OF PLUNGE POOL DEVELOPMENT AND REMEDIAL MEASURES AT TARBELA DAM SERVICE SPILLWAY

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SYNOPSIS

Tarbela Dam Project has two spillways whose combined capacity is a million and a half cusec. Service spillway with a capacity of 650,000 cusec was conceived as the prime structure for the regulation of the reservoir. The plunge pool development of this spillway took many unanticipated turns which led to serious erosion of the rock on the right side of the bucket. The situation worsened every year and the structure became inoperable in a short span of three years. A huge embayment had occurred on the right hand side. A big reverse current in this embayment was constantly endeavouring to enlarge it. Greater erosion of the rock from one side generated unbalanced geotechnical forces, which caused movement of the structure and put into question the stability of the foundation over which the bucket was resting. This paper describes these unfavourable developments and the various remedies that were evolved over a period of three years. The final shape of the remedies resulted in a pool confined on the three sides by the placement of concrete and rollcrete. The unprecedented situation of stabilizing rock slopes 250 feet high with post-tensioned anchors and the concrete slabs resting over these slopes are discussed. In essence, the paper is a case history of the problems and remedies of Service Spillway of Tarbela Dam Project from 1975 till 1982.

DESCRIPTION OF THE PROJECT

Tarbela Dam is a multipurpose project on River Indus, primarily constructed to meet the growing irrigation needs of Pakistan. The dam is one of the largest structures ever built by man if not the largest.

The Project comprises of a 470 feet high and 9,000 feet long earth-cum-rock fill dam on the main stream of the River Indus about 70 miles north-west of the Capital of Pakistan (Islamabad). There are two earth-cum-rock fill type auxiliary dams close to the main dam on the left rim of the reservoir. These dams are 345 and 220 feet in height, and their lengths are 2,340 and 960 feet respectively. There are three 45 feet diameter and one 36 feet diameter tunnels on the right bank, that were used for diversion and irrigation releases during construction. Of the four tunnels, three are to be used for power generation/ irrigation releases. The fourth tunnel is solely meant for irrigation releases. A fifth tunnel was later on added on the left bank to augment irrigation supplies during the last phase of power installation and to have a substitute or standby arrangement for the irrigation release tunnel on the right bank. The spillway closer to the main dam is called “Service Spillway” and the one apart is called “Auxiliary Spillway”. Figure-1 shows the layout of the project.

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HYDROLOGY OF RIVER INDUS

The Indus River rises in the Tibetan Plateau north of Mansarwar Lake at an elevation of 18,000 feet in the Kailas range of glaciers. It flows for 1,800 miles before draining into Arabian Sea, about 100 miles south-east of Karachi.

The Indus river basin above Tarbela lies in the Karakoram, Kailas, Ladakh ranges and the Great Himalayas that form the chief divide line between Central Asia and the South Asia. The famous peaks like K-2 (28,250 feet), Naga Parbat (26,660 feet), Gasherbrum (26,470 feet) and many others above 20,000 feet, and several important glaciers varying in length from 35 to 45 miles are situated in the catchment area of River Indus.

The topography of the basin is mountainous at the upstream and hilly at the downstream end, and includes some broad valleys. Geologic erosion is predominant in the basin along the youthful Himalayas where there is practically no plant cover. Large alluvial deposits with deep gravel and sand bars are observed in the upper reaches of Indus River and its tributaries.

As the greater portion of the basin lies either above or behind mountain barriers in excess of 15,000 feet elevation, it experiences very little summer precipitation. About 1/4th of the area is perpetual snowfields and glaciers. The annual precipitation in the upper valley at high altitudes in unknown, but at the lower elevations beginning at a point about 100 miles upstream of Tarbela, is as small as 5 inches. The high mountains on both sides of Indus below this point intercept the monsoons and thus create a desert condition within the upper valley. This condition is relieved only in summer by water from the melting of snowfields and glaciers many thousands of feet above the river channel. The accumulated effect of this snowmelt in the 65,500 square miles drainage area above Tarbela is significant and results in a snowmelt hydrograph which extends from April to October and has peak discharges upto 500,000 cusec. The snowmelt runoff forms a base flow which when accentuated by monsoon storms has produced historic floods with peaks upto 860,000 cusec at Attock (August 27-30, 1929). The Indus River follows a dependable seasonal pattern of flow during winter and floods during the summer. The river derives about 90% of its total runoff from melting of snow in the high mountains. The remainder of the runoff is derived from rainfall accruing almost wholly on about 6% of the tributary area (4,000 square miles), which lies immediately upstream of the dam. This is the most critical flood producing area of the basin. The floods in the river are generated by excessive snowmelt, rainfall or failure of natural dams.

A river gauging station at Darband, twenty-two miles upstream of the Dam axis was established in November 1954. Discharge records of the Indus at Attock, twenty-nine miles downstream of Tarbela were available from 1968. Hydrographs of Indus at Darband were derived from Attock records using computed ratios of discharge from the period of overlapping records of the two sites between 1954 and 1964. Discharges at Tarbela were assumed to be the same as at Darband because the annual contribution of the Siran River entering the Indus just upstream of Tarbela is hardly 1% of the annual Indus flow at Darband.

As such 112 years discharge data of Indus river at Tarbela (1868-1980) is available, from which the average annual run-off at Tarbela is derived as 65 million acre feet (MAF) that corresponds to an average depth of runoff of 18.4 inches from 65,500 square miles drainage area. In January/February the river discharge drops to a minimum of 12,000 cusec. The ratio of maximum to minimum discharge is about 1:40.

Spillage Requirements

The spillage requirements in a reservoir are determined by the inflow pattern of the river and the size of the reservoir through which the flows have to be routed. In case of Tarbela, the reservoir happens to be small for the major bulk of the flows that the river brings during the four summer months.

Indus at Tarbela brings 65 MAF of water in an average year. Of this 49 MAF or 75% is concentrated in the four months of May, June, July & August. This is far in excess of the live storage capacity of 9.3 MAF. Sustained flood peaks of 400,000 cusecs occur once every five years.

The following table shows the average discharge (in cusec) for the high flow season:

May	June	July	August	September
59,000	169,000	252,000	228,000	101,000

The release capacity of the low level outlets in the ultimate phase when all the three tunnels on the right are converted to Power and Tunnel 4 & 5 run with gates fully open varies from 163,000 cusec at elevation 1300 feet to 256,000 cusec at elevation 1500 feet at which level the spillways start operating and Tunnel 4 & 5 are shut down.

A study of the flow pattern over the last hundred years shows the occurrence of floods at Tarbela is from three distinct phenomena:

1. Floods caused by storm rainfall.
2. Floods caused by excessive snowmelt.
3. Floods caused by bursting of slides which block the river, sometimes for months.
4. A combination of those above.

From an analysis of past occurrences, it was concluded by the designers that a figure of 600,000 cusec was reasonable to expect from snowmelt including a small factor of safety.

The flood from storm rainfall was calculated for the 4,000 square miles area by using Unit Hydrograph techniques. A recorded high storm was suitably adjusted for hydro- meteorological conditions and it was calculated that a peak of 1,173,000 cusec could arise from this cause.

Imposing the peak of the flood from storm rainfall over the snowmelt results in a peak flow of 1,773,000 cusec, it was further concluded by the Consultants that present day technology would enable advance information about any blockage from land slides and prior lowering of the reservoir could be done to accommodate flows resulting from such an event. Flood routing studies indicates that the ingress on surcharge for the possible sequence of occurrence would be within acceptable limits. A flood of 1,773,000 cusec was accepted as the probable maximum flood for the spillway design. This flood when routed through the reservoir at Normal Full Pool Elevation (1550 feet) would result in an outflow of 1,410,000 cusec over the Spillways with some flows occurring from the Power Units. A surcharge of 2.2 feet (i.e. elevation 1552.20 ft) will increase the flow over the Spillways to 1,490,000 cusec.

SITING OF THE SPILLWAYS

The natural features at the dam site, which could be utilized for spillways were the rock bench at the right abutment (where the power plant is located) or the side valley around the left abutment. The rock on the right abutment was considered unsatisfactory for the foundations of the required overflow gravity structure. For the left abutment three variants were considered. One was to close the side valley with a concrete dam having an overflow spillway section. Here again the rock was considered incompetent to support a structure of the required height. The second variant was to close the side valley at its downstream end with an earth dam and cut a spillway channel in the rock abutment discharging back into the main river downstream of the main dam. This solution was not favoured because the rock conditions were not suitable for economic construction of an adequate stilling basin. The third alternative was to close the side valley at its upper end and to construct two gated spillways discharging into the side valley.

Many considerations favoured a two structures solution. Firstly, there was no suitable location where the entire structure could be economically located. The runoff at Tarbela is far in excess of the reservoir capacity and a good part of it is concentrated in the summer months. The spillways would have to discharge between 300,000 to 400,000 cusec every year and would, therefore, be subjected to severe flow conditions. Thus a spillway sized to pass the maximum flood of record (estimated at 680,000 cusec) would have to operate at a substantial capacity every year. In case such a structure needs repairs extending beyond the available low flow period, a second spillway of equal or greater capacity becomes essential. Subsequent paragraphs would show how fortunate the choice of two spillways proved.

It was feasible to align the chute of one spillway on a fairly long spur to convey water to a point remote from the Project structures. Energy dissipation and consequent erosion, when a good distance away would not cause imminent threat to the safety of the project structures. This spillway was conceived to operate most of the time and is called the "Service Spillway". The second spillway had to be located close to the Auxiliary Dams for reasons of topography and to fit into the overall scheme. The energy dissipation for this spillway had to be close to the auxiliary dams and to the overflow part of the structure itself. Although the conditions for a plunge pool were deemed satisfactory, it was considered prudent to limit this spillway to less frequent use i.e. as 'Auxiliary'. The "Service Spillway" was to operate most of the time.

In dividing the total spillway capacity it was desired to have the service spillway capacity at least large enough to pass the maximum record flood. While an even distribution of capacity would be natural, the difference in the cost of excavation at the two locations indicated that it was cheaper to make the Service Spillway smaller than the Auxiliary. Fortunately, it was possible to accommodate a spillway of a large enough capacity at the location requiring less excavation, which in combination with flow from turbines would pass the record flood. The Service Spillway was constructed at that location. The great value of stored water justified gated Spillways and the large capacity of the Spillways made very large gates economically desirable.

GEOLOGY OF SPILLWAY SITE

Service Spillway is located in limestone and phyllite formations interlayered in beds of one to two feet thick of Abbottabad age. Beds are striking perpendicular to the chute centerline and dipping steeply downstream (strikes N 60^o E and dips 50^o – 80^o to the SE). Close jointing and fracturing with extensive weathering is prominent. A number of faulted and folded zones cross the centerline of the spillway with almost similar general strikes as the beddings.

Two major dikes of basic igneous intrusion cross the spillway area, one near the bucket and the other at the exit of the getaway channel about 700 feet downstream of the bucket. Numerous small basic intrusions are found within the limestone beds.

The above mentioned limestone/phyllite formations separated from the quartzite formation of Tanawal by a major longitudinal fault which runs just downstream of the mouth of the getaway channel and is associated with secondary faults which run parallel to the main fault.

The rock over which the flip bucket is seated is fractured, contorted and tightly folded, and these characteristics extend up to 400 feet downstream. The orientation of the significant geological features are not consistent and vary along the flip bucket.

The limestone and phyllite beds to the left of the centerline strikes nearly E-W and dips 43^o – 65^o S. There are monoclonal folds superimposed on the bedding, which strike 40^o N and plunge 26^o SW. These folds form buttress, which should normally prevent sliding down the maximum dip direction both to the left and right of the centerline. From the centerline of the flip bucket to the right hand side, the rocks are tightly folded. The fold axes strike N 40^o E and dip 30^o – 54^o SW.

About 300 feet from the right edge of the bucket and along the line of its lip the phyllites and limestone strike N 50^o E and dip 60^o SE. At 250 feet the strike changes N 80^o E and dip 60^o SE. The basic intrusion at SW corner has deformed, altered and sheared these phyllite and limestone beds in a 50 feet wide area. This zone is very soft with many pre-sheared surfaces and with clayey material having low residual shear strength of 11^o – 14^o. This is a weak zone and could be highly susceptible to erosion. It lies on the right of the spillway near the end of spillway chute.

Generally the rocks on the right side are weak and their structural orientation is complex. For details see figure 1A.

PRE–DESIGN MODEL STUDIES

The safe dissipation of enormous energy over fairly long periods every year was the key question in deciding the locations of the Spillway. The other important question was to find the cheapest technically acceptable means of dissipating this energy if the very economic viability of the project was not to be jeopardized. The construction of a flip bucket at the downstream terminus of the spillway chute was found feasible. A 1:120 scale model of the spillways and the Dal Darra conveyance channel was constructed at Irrigation Research Institute (IRI) Nandipur in Pakistan to study the scour in the Dal Darra conveyance channel area in the plunge pool. A model of one bay of the overflow structure on 1:64 scale was constructed at the Lahore (Pakistan) premises of the IRI.

The technical details of the hydraulic designs were evolved from the U. S. Corps of Engineers Publication "Hydraulic Design Criteria". This model was primarily used for studying the hydraulics of the overflow structure and the chute conveying water to the flip bucket. The data obtained from this model (1:64 scale) of the overflow section was used in evolving the final design of the overflow structures.

The experiments on the 1:120 scale model focused attention on the erosion pattern of the Dal Darra Channel and was moulded in 0.12 to 0.4 inch cohesion-less material. Interesting but inconclusive experiments were made for different configurations of a flip bucket. The testing of different configurations was prompted by the geology of the site. Near the downstream end of the spillway channel a strip of competent basic rock (chloritic schist) ran diagonally across the width of spillway channel. It was desired to seat the bucket on this rock or to so treat it as to use this basic rock for defence against any unanticipated erosion of the pool threatening to undermine the bucket.

Two basic design variants were tested (Figure 2). An oblique or a diagonal bucket with a constant lip elevation was tested for constant chute slope and also with slope of the chute that varied from left to right. The basic problem with this arrangement was the considerable variation of discharge over the lip of the bucket. This design was abandoned in favour of a bucket aligned normal to the chute but still making use of the basic rock for the foundations of the bucket. In this variant the steeply sloping part of the chute was divided into two (Figure 3) and three separate channels by construction of divider walls. The buckets staggered in plan; with differing and constant lip elevations were placed at the end of these channels. As a consequence, a 20 feet differential head developed across the divider wall for the two buckets solution with differing lip elevation. Adjustments of chute slopes and lip elevations sufficiently corrected such differentials. It was observed that arrangements for aeration were required at both ends of individual jets. At the time of experimentation the exact location of the sound rock and the character of its qualities was not known fully and further refinements of design were not pursued. Also, preference seemed to have emerged for adopting a more conventional design. The scour patterns in the plunge pool with varying geometries of the buckets were not studied in detail. However, some observations on plunge pool scour were done with a conventional bucket of 50 feet radius. It was noticed that the scour pattern of the pool was lopsided because of the asymmetry of the downstream area. A significant difference in the water level in the pool and under the jet caused a fast current moving upstream towards the low level region, which eroded the left nose of the promontory dividing the two spillway discharge channels. The deepest scour noticed at a discharge of 654,000 cusec was down to elevation 981 feet. These pre-design studies highlighted some basic questions, which had to be answered before further designing could be undertaken.

MODEL STUDIES FOR DESIGNS

The design model studies for the plunge pool were initially undertaken on 1:80 scale model at Nandipur, Pakistan. These studies were further pursued at the Alden Research Laboratories, Worcester Polytechnic Institute USA. The objectives of the studies at both institutes were:

1. What will be the pattern of scour at the flip buckets of the service and auxiliary spillways for the range of operating condition?
2. Will scour tend to undermine the flip bucket monoliths?
3. If protective walls are required to protect one or both the spillway buckets from scour, how deep and extensive should these walls be?

Nandipur Tests

Some preliminary tests were run using cohesion-less material ranging in size from 5/16 to 3/4 inches. The criteria for choice of model material was that it should scour like a rock, be easy to handle and give reproducible results. The results obtained with this material though meeting the criterion partway were deemed unrepresentative since:

After exposure to the design discharge for relatively short periods virtually all the material in the vicinity of the spillway bucket was scoured exposing the fixed bed of the mode.

i. The scour was too rapid and made a systematic study impossible.
ii. The angle of repose was much less than that of the proto-type rock.
iii. The material formed a bar across the Dal Darra raising tail water level so high that the jet leaving the bucket was partially submerged.

A mixture of clay and gravels was then used. The clay available at the model site was pulverized by passing it through a sieve. The mixture was proportionate by mixing 50% of aggregate (5/16 to 3/4 inches) used with 50% of clay by volume. The new material behaved much more like rock, eroded slowly and had a steep angle of repose.

Discharges used

From hydrological records of River Indus, it could be deduced that 400,000 cusec is the maximum discharge that could be expected to pass for extended periods. The same figure of 400,000 cusec was used for the Auxiliary Spillway also. Low discharge with a shorter leap was assumed to be 50,000 cusec.

Results of Tests

The main conclusions drawn from these tests can be summarized as under:

1. A large-scale eddy developed on the left at all tail water levels when the discharge reached 200,000 cusec.
2. The promontory on the left eroded under sustained discharges and the strength of the eroding eddy increased as shown by velocity measurements. This was observed when 400,000 cusec was maintained for several hours.
3. The strength of the eddy also increased when the water level rose because of the deposition of the scour material downstream.
4. At low water level an eddy on the right, of lesser strength than the left was observed. A large cylindrical hole was scoured on the right bank downstream of the bucket.
5. The eddy on the left eroded the rock promontory on that side. The severity of erosion was greater with high tail water level.
6. At low tail water level the base of the bucket was exposed by erosion from the left hand

side.

7. To guard against undermining indicated in (6) above a sloping wall constructed by methods used in mining was installed in front of the buttress. A slope of 1 in 1 was used. After a run at 400,000 cusec the sloped wall was exposed to elevation 1050 feet near the spillway centerline. There was no exposure past the corners of the bucket.
8. The maximum scour during all the test series did not exceed an elevation of 1,000 feet.

Alden Tests

A third series of model tests was undertaken on a model of 1:80 scale at the Alden Research Laboratories, Worcester, Polytechnic Institute, USA from September 1970 to January 1972.

The testing was done in three distinct phases. The first series consisted of studies on a 'fixed bed' model. The second was a 'semi-fixed bed' model and the third was a 'movable bed'. The philosophy of the 'fixed bed' studies was to determine a "stable" scour hole i.e. a scour hole where the peripheral and bottom velocities are low enough not to cause further erosion. The difficulty of knowing the upper limit of a non-scouring velocity was realized and a figure of 20 ft/sec (prototype) was adjudged as reasonable. Three or four sizes of scour holes with appropriate changes in geometries were tested. It was concluded from these tests that a very large scour hole is required if the velocity at all points is to be reduced to 20 ft/sec or lower. Furthermore, as the scour hole is enlarged, an eddy on the left side with a velocity of 35 ft/sec appears, which can cause the erosion of the left promontory in the same way as shown by tests at Nandipur. In the semi-fixed bed tests the sides of the pool were moulded un-erodible and the hole filled with a sand gravel mix to represent the rock (Figure 4). The sides were so placed that a greatly enlarged hole could be formed. A large eddy with a high velocity again formed to the left of the jet attacking the left side.

Further work on fixed and semi-fixed beds was deferred in favour of making changes in the geometry of the bucket, but finally, efforts were directed to find the design parameters by experiments on a moving bed model. This model was moulded in a cement sand mix assumed to represent rock characteristics. Tests on a movable bed model began with a mix of one part of cement added to sixty parts of local sand. The initial run indicated the mix to be too stiff to represent the likely conditions. It was therefore changed to 1:80. This mix when exposed to a flow of 400,000 cusec for about 4 hours caused the collapse of the promontory located at left bank of the spillway outlet channel. Significantly, once the initial embayment occurred on the left bank, the eddy current caused it to enlarge further and further - a situation also witnessed in the Nandipur Model. A tendency for undercutting on the right hand side was also noticed.

The unfavorable conditions demonstrated in this first movable bed were attributed to the location of the plunge pool. The intersection of the centerline of spillway channel and the centerline of Dal Darra was considered too close to the plunge pool. The general direction of the water leaving the pool was forced up to the left side to the Dal Darra and part upstream causing increased differential head between the boil and the water level behind the jet. It appeared that the unfavorable condition could be improved if the pool could be moved upstream away from the channel intersection.

It was decided to shift the bucket 500 feet upstream by shortening the upper one percent slope chute. "The primary objective of the new bucket location was to keep additional natural rock to the left of spillway at the 'nose' between the spillway bucket and the Dal Darra Channel". The upstream location of the spillway bucket increased the distance from the boil area in the Dal Darra left bank to the depressed pool level under the jet. The jet impact area was now well within the service spillway outlet channel for all discharges. Although a small eddy did form near the left bank at the intersection of Dal Darra and Service Spillway discharge channel, no erosion of the bank occurred.

The main observations with the bucket shifted 500 feet upstream were:

1. The small eddy with velocities of 10 ft/sec or less formed along the left nose of the spillway outlet channel but did not cause exposure of the protection apron.
2. Increasing the channel width by increasing the angle of the channel wall with the channel centerline formed eddies on the left and right side of the jet impact area. However, upto 25^{o} cut back, the eddies did not harm in the bucket area.
3. The jet would fall clear at the discharge of 21,000 cusec.
4. The deepest contour observed was of elevation 956 feet at about 800 feet downstream of the bucket.
5. The lateral spread of the jet as it left the bucket lip could be reduced by adding a wedge section at the downstream end of each sidewall of the spillway chute.

DESIGN OPERATIONAL CRITERION

The design of Service Spillway components influencing the plunge pool was finalized on the basis of above study. Figure 5 shows the main components of Service Spillway. The basic design operational criterion was that service spillway would be the prime structure to control the reservoir level in the normal course of events. Sustained flow upto 400,000 cusec for a few days is not an infrequent occurrence and the structure should take care of it without exhibiting any sign of strain. The operational criterion conceived that flows upto the recorded maximum of 600,000 cusec should be within the ambit of the service spillway. Flows in excess of 600,000 cusec shall be shared with the auxiliary spillway.

OPERATIONAL EXPERIENCE

The Service Spillway was first commissioned on 7th August 1975. The reservoir could not be lowered during the winter of 1975-76 as the major damages suffered by Basin-3 and Basin-4 in the summer of 1975 left no outlet through which the irrigation needs of the country could be released except the spillways which routed the inflows. The year 1975 happened to be a very dry year for the Indus River. The Service Spillway was operated 2,543 hours. The survey of the plunge pool after the closure of the structure showed:

1. There was general erosion downstream of the underground curtain wall of about 100 feet.
2. The deepest contour of 1050 feet observed in the end of August was near the extreme right edge of the pool.

3. The sustained passing of low flows during the autumn of 1975-76 caused erosion of the rock cover over the curtain wall and in the area immediately downstream of the curtain wall.

Performance in 1976

Tunnel-5 on the left bank was commissioned in April 1976. During the same month stilling basin-3 suffered another disastrous damage and the reservoir could deplete only to elevation 1476 feet by releases from Tunnel-5. The Service Spillway started operating in the early part of the rising flows. Minor erosion and sloughing started to occur in June 1976 with 100,000 cusec passing and reached the 1500 feet berm on 12th July when the flows were about 200,000 cusec. On July 21, 1976 when the spillway was passing 229,000 cusec a huge slide estimated at half a million cubic yards occurred on the right hand side. A very strong back current developed which started eroding the northwestern slope. This slope proved particularly vulnerable since the beds striking parallel to the bucket were thin and could easily undercut. Within a day or two the 20 feet wide benches between elevation 1200 and 1500 feet had all disappeared and a steeply sloping face was formed. A vein of the basic black rock resisted the outflanking of the deep protective curtain wall on the bucket. This vein proved more resistant to erosion than the surrounding rock. Whereas, the outflanking did not occur during 1976, significant cracks were observed on the bench at elevation 1500 feet, which carried the permanent access road of the Project. The cracks were wide spread and some appeared upstream of the black rock vein. Close monitoring of the cracks was carried out. The cracks towards the west were continuously widening, the permanent access road was abandoned and a temporary diversion was given to the traffic. To reduce the erosion, the flow over the Service Spillway was restricted to less than 180,000 cusec. The balance was passed over the Auxiliary Spillway. The survey of the Service Spillway Plunge Pool conducted after these events indicated:

1. A deepening of the pool on the left. The deepest contour of 1050 feet was now far upstream.
2. The erosion had been continuously deepening and widening the pool. The lower contours continued to open out from left to right.
3. The large slide had created a vertical cliff, which continued to slide with each shutdown of spillway. Deeper pockets appeared on the right inspite of the continuous feeding of this talus.

Figure 6 shows the black rock at the downstream end of the plunge pool. The high crest levels of the black rock on the right with a small gap near the middle and the mid-left apparently provided conditions, which could exacerbate the formation of reverse current on the right. It was felt that if the exit of water out of the pool could be eased by lowering or removing the ledge of the black rock especially on the right side, then there was a likelihood of the strength of the back eddy getting reduced and the erosion controlled. This possible improvement, however, would have to wait until the high flow season was over and the physical work becomes possible.

First Stage Remedial Works

The objectives of the first stage remedial works were:

1. To unload the berms to reduce the destabilizing forces and minimize the latent danger of a deep seated slide.
2. To improve the hydraulic conditions at the outlet of the pool by encouraging longitudinal development and to reduce the strength of the back current by physical removal of the upper part of the black rock on the right and shattering lower parts to make it easily erodible.

Besides these two measures, dental concrete was placed in the seams and joints of the rock above the stagnant pool level adjacent to the bucket to prevent its erosion by the impacting jet at low discharges.

Performance in 1977

The reservoir was lowered to supply water for irrigation and generate power in the 1977-78 season. It started rising in June 1977 and the spillway was opened in the first week of July 1977 when water in the reservoir rose to the operating level.

From 15th of July to the 2nd of August 1977, the discharge over the spillway varied from 112,000 to 192,000 cusec. During this period the monitoring of the plunge pool showed conditions which though not desirable were not unexpected either. It was understood that the pool had to adopt new dimensions for the changed hydraulic conditions brought about by the partial removal of the resistant barrier at the downstream end. Both deepening and progression downstream were anticipated. Some deepening of the upstream part of the pool was not ruled out but it was expected that with the removal of resistant rock downstream, the tendency for expansion in the downstream direction would predominate.

Unfortunately, these expectations of progression and deepening of the pool were not realized in the first two weeks of operation. On the other hand the deepening in the upstream areas of the pool did take place due to back eddies and slides occurred on the northwestern slope. The plunge pool survey did not indicate any cause for alarm, as there was no danger of undermining the protective wall, which had enough rock cover. The deeper parts of the pool were sufficiently remote from bottom end of the protective wall. It was thought that unless discharges higher than 180,000 to 190,000 cusec were passed the downstream part of the pool would not erode and the pool would not develop longitudinally. Unless it did so, no relief would accrue to the erosion occurring upstream due to reverse current. Also the pool had to be conditioned to take the normal annual peaks of 300,000 to 400,000 cusec if the spillway was to perform its intended function as the prime structure for passing the normal high flows.

From August 1977 the discharge was progressively increased from 192,000 cusec to 304,000 cusec in a period of eight days. The surveys on the 7th and 15th August showed considerable erosion of the downstream part. The development of runnel on the left side progressing upstream and turning to the right is noticeable from Figure 7.

The vein of black rock on the right of the bucket, which had resisted erosion in 1976 and consequently prevented the out-flanking of the protective wall or the flip bucket started slumping. However, upto the 15th of August, the erosion indicated no danger of undermining or outflanking of the protective wall. The discharge of 300,000 cusec was maintained for a total of 48 hours between the 11th and 15th of August when it was closed for inspection. The inspection revealed that some major events had occurred during these days and the structure had ceased to be operable any more.

The inspection revealed that erosion had outflanked the protective wall on the right hand side by about 20 feet and the lower part of the structure had suffered movements along some joints and developed cracks which cast serious doubts on its stability. The following observations record the happenings:

1. The right side of the protective wall was outflanked. An embayment with its edge 20 feet upstream of the top of the protective wall had occurred.
2. The protective wall was exposed in patches at some locations. Most of the patches were visible 100 feet to the right and 60 feet to the left of the centerline of the spillway between elevations 1170 and 1140 feet.
3. The right wall of the spillway chute is constructed in monolithic sections of 33 feet. Three monoliths of the downstream right wall showed tilting to the right. These movements were large at the top of the wall and were about 0.12 to 0.20 inch relative to one another.
4. An area bounded by these 3 right monoliths and the centerline of the spillway had experienced a general displacement of about 4mm in the downstream direction. This displacement was greatest on the right side and was not visible near the centerline of the spillway.
5. The movements mentioned in (3) above were traceable in the drainage galleries below. Observations in these galleries showed some movement in the further downstream monolith on the left also. There, the wall on the left showed static conditions but the adjacent concrete had moved in the downstream direction.
6. Cracks were noticed in the footing of the upper-most three right wall monoliths, which had tilted.
7. No undermining of the protective wall had occurred anywhere. In general, it still had an overall rock cover of 25 feet.

Naturally, the first task was to instrument the structure to monitor further movements. This structure already had seven standard piezometer holes with two piezometer tips located in each hole at different elevations and four multiple bore holes extensometers in the transverse gallery in the bucket. These were supplemented by:

1. Eleven Rod Extensometers of which 5 were installed in the left abutment, 3 in the right abutment and 3 in the lower transverse gallery in the bucket.
2. Two linear deformation sensors, one in each of the right and left connecting galleries.
3. Surface reference markers for detecting vertical and horizontal movements.
4. Marker pins to measure crack openings by portable strain meters called deformeters.

The readings obtained from rod extensometers were of satisfactory quality and yielded information on movements occurring between the point of anchorage and extensometer head.

Because of the non-availability at site of high quality invar wire and the elastic stretching, the information obtained from linear deformation sensors could not be reliably interpreted.

The movements measured on surface markers by the electronic distance-measuring device were too small for dependable interpretation. The trend, however, could be discerned.

Portable strain meters are a satisfactory device for purpose of measuring changes in crack openings.

These happenings cast serious doubts on the safety of the structure. The principal issues that arose out of the happenings in August 1977 were to:

i) Ascertain the stability of the rock mass on which the bucket was resting.

ii) To restore and ensure this stability.

iii) To safeguard the structure by preventing any further loss of rock from critical areas.

Mechanics of Bucket Movements

The postulates on mechanics of movement varied from expansion of rock caused by loss of confining pressure from the rock in the pool and from area right of the bucket to one of incipient failure. The most logical explanation was provided by Mr. John Lowe-III of TAMS and was broadly agreed to by others who were consulted on the subject.

The design of the work called for the excavation of a getaway channel in front of the bucket where the plunge pool was to form. The getaway channel was excavated to 1210 feet level from an initial level of about 1500 feet. The slopes on the two sides of the channel were 1:1 and were provided with 20 feet wide and 50 feet high benches. No noticeable movements were observed as a consequence of unloading the rock lying above the bed of the channel. As the erosion of the pool occurred, the stresses in the rock must have adjusted themselves to the new equilibrium. These adjustments caused no noticeable movements on the structure. The loss of huge quantities of rock in 1976 and 1977 from the right side of the bucket triggered on elastic rebound causing movements in the structure.

Deep erosion upto elevation 1040 feet occurred on the left side in 1977. It appears that this deep erosion triggered a push from the high-pressure area on the left towards low-pressure area on the right of the bucket. The loss of resisting rock from the right helped this movement to occur. The high horizontal stresses, which had remained, locked in the rock mass since these movements gradually relieved the excavation. In short there developed a thrust from left to right as a consequence of the loss of rock resulting in an elastic rebound. The direction was determined by the fact that much more rock was lost from the right of the bucket. No appreciable rock was lost from the left hand corner. Consequently, higher horizontal stresses prevailed in the left area. The deep runnel downstream of the left hand corner and erosion in front of the bucket and beyond to the right removed the confining rock and provided the opportunity for the relief of these stresses. The massive bucket monoliths along with the thick concrete of slab 68 and the buried curtain wall with post tension anchors tying a part of the rock to the curtain wall, acted like one giant transverse strut pushed from left to right. The closely spaced post-tensioned anchors of the buried curtain wall prevented any buckling of the wall. The rock tied to this wall, however, cannot provide sufficient resistance to prevent movement of this concrete strut. First, there are low friction angle bedding planes dipping at about 50^{o} directly downstream and so resistance from rock under slab 67 cannot be mobilized very well. Secondly, the low angle fault may be present at shallow depths below the bottom of the protection wall and not much resistance can be expected here. The fault running from right to left and passing under the spillway near the right hand side has weak material at its boundary. Any thrust has to pass through this fault before reaching the competent rock at the right. The right end of the buried protection wall was exposed by the last erosion at the end of operation except the bottom 20 feet or so had, therefore, nothing to abut against. The protection wall is closest to the area where stresses have been relieved and receives the greatest effect of elastic rebound towards the right.

As the thrust reached the weak crushed material at the boundary of the fault it could not resist the forces and squashed in the direction of the strike of the fault. Movement along the strike was made possible, as the rock that could have resisted such a movement was no longer there. Erosion has already removed it. The small wedge of rock between the bucket and the fault was pushed out towards the pool as evidenced by a crack at the concrete rock interface.

Second Stage Remedial Works

The above description of mechanism appears logical and explains most of the observations. It, however, could not be concluded for certain that no risk of failure would exist if only further loss of rock could be prevented. The strength of fractured rock is greatly dependent on the state of the stresses. A rock, which suffers expansion or dilation, will lose shear strength. This strength can be restored only if the confining compressive stresses are reintroduced.

Furthermore, the service spillway is a very vital structure and slight movements could jeopardize its ability by causing irregularities on concrete surfaces leading to cavitation. The remedies, therefore, had to be sought in three directions.

1. To restore and strengthen the structural integrity of the foundation rock. The objective would be to prevent any further movement from other possible but undetermined or obscure causes like the development of incipient planes of failure.

2. To safeguard the integrity of the restorative works from destructive hydraulic conditions.

3. To prevent loss of rock from critical areas to obviate possibility of further movements.

With regard to item (1) the concern felt by all the experts was so great that it was unanimously agreed that the final designs of the solution should be prepared by analyzing possible planes or wedges of failure and guarding against such possibilities in preference to seeking remedies in the light of explanations furnished in earlier paras.

Three possible remedies were considered:

a) To provide post tensioned anchors to improve resistance to sliding and to restore any loss in the bearing capacity of the rock resulting from decompression.

b) To place a massive buttress of rollcrete in front of the bucket to provide positive resistance against sliding of the foundation rock.

c) A combination of (a) and (b).

It was recognized earlier that provision of post tensioned anchors as a principal device for ensuring stability might prove a physically impossible task for the size of rock mass involved. Some engineers opined that the decompression of the foundation rock on the right might have weakened the rock to an extent that its bearing strength may need to be restored. This could only be done if an active compressive force is applied. A limited number of anchors on the right side was therefore, agreed to as a matter of abundant caution. More so, time was limiting factor for any such solution.

The principal hope lay in Contractor's ability to place sizable buttress of rollcrete. He had a brilliant record of rollcrete placement during repairs of the intake of Tunnel-2 damaged in 1974. Attention was focused on the design of the buttress. Whatever the design of the buttress might be it had to be guarded against undermining, the chief causative agent for which was the big reverse current.

Structural Design of Second Stage Works

I. A detailed geological examination of rock exposures was carried out. Many possible models of failure were identified and the following criteria were adopted for stability analysis. The objective of the analysis was to calculate the existing margins of safety and the margins of safety for the conditions of pre, during and post-remedial measures. The exercise covered both the static and seismic situations using pseudo-static acceleration of 0.3g.

The analytical investigations were extended for the operational situation and also when the spillway is closed. Figure 8 shows the assumptions used in analysis for pre and during construction situations. Detailed geologic mapping of the area identified sections appropriate for analytical studies.

Out of six sections analyzed, side forces were considered in one only. Five, three-dimensional wedges were analyzed in addition to six two-dimensional sections. The shape and location of these eleven sections and wedges were determined by the existing planes of weakness. Graphical methods were employed for determining forces.

II. Various combinations of remedies were assumed. Buttresses of different dimensions were combined with anchors placed at different elevations providing different degrees of compressive forces. The resistance to sliding was calculated for a number of combinations. Factors of safety were calculated with chosen dimensions of a buttress in combination with anchors. The friction angles chosen ranged from 25° for poor rock to 40° for the rollcrete rock interface. An angle of 50° was assumed for rollcrete proper. Slide resisting force mobilized by sides was estimated from:

$$2\,(T_v\, K \tan\phi \text{ Area}) \qquad \text{for} \qquad K = 0.3 \text{ and } 1.0$$

III. Criterion adopted for design of anchors was that the anchor could extend by one inch. If an anchor is locked off at 0.6 of guaranteed ultimate tensile strength (GUTS), a free length of 45 feet is required to provide one inch lengthening from 0.6 to 0.8 GUTS. In the circumstances of 1977-78, this criteria was not critical since anchors were installed only between El: 1160 and 1185 feet in a length of 150 feet measured from the right corner of the bucket towards the center, and these were kept long enough to cross the fault zone.

IV. A factor of safety of 1.4 was considered satisfactory for static condition. It was concluded from the analysis that 88 feet thick rollcrete section provides the required stabilizing force.

The parameters used in the adopted design are tabulated below. The sliding plane is assumed along a relatively low inclined plane, 30° inclination with strike in E-W direction and dip of 30° to the south.

Angle of Friction ϕ, on sliding planes	Friction angle for rollcrete	Friction angle for rollcrete rock interface	Acceleration due to earthquake	Stretch in anchors (inches)	Factor of safety
15°	50°	35°	0.5g	0.69″	1.35
20°	50°	35°	0.3g	0.30″	1.61

V. Fifty-six post-tensioned anchors of 500 kips were agreed to be provided under the right half of the bucket in addition to the rollcrete buttress. It was estimated that a confining force of 80 tons per meter run has been lost from the right half of the bucket. The 56 anchors re-introduced a force of 90 tons per meter. All anchors crossed the fault zone and the bonded length of 20 feet, is in undisturbed rock. A 6 feet thick concrete pad was placed against the trimmed rock to provide a proper surface for placement of the anchor block and to transmit the load to the rock. To increase resistance to movements, a rollcrete strut was added as an extension of the buttress to the right. This section provides added passive resistance to the left to right thrust.

Hydraulic Solution for preventing undermining of the buttress

Once the contractor gave an assurance that he could construct the buttress within the limited time of eight months, the problem of stability became one of finding reliable safeguards for the buttress against undermining. There was no time available to undertake a research on models to find answer to the difficult problem of scour. It was obvious that the best assurance against undermining would be to take the foundations deeper than anticipated scour. In the given circumstances we would neither determine the possible scour levels nor could we physically go to those depths.

Naturally, the first thoughts turned towards the more familiar and conventional forms of protection like self-launching flexible aprons. An apron of concrete blocks stacked in layers at the toe of the buttress was studied. Blocks, which could slide down the slope of buttress, were also considered.

Model tests on 1:80 scale fixed bed model at Nandipur

The model at 1:80 scale was reactivated as a fixed bed model for determination of the size of blocks. Besides, it was intended to find if some relief would accrue by raising the tail water level, lowering the crest of the resistant rock at the mouth of the pool to some practicable value, widening the mouth at the outlet end and extending the pool into the getaway Dal Darra Channel. Should these measures proved beneficial, it was possible to implement them at least partially. The model was moulded for the existing geometry of the pool along-with the proposed buttress. The following observations were recorded:

1. The raising of tail water reduced the velocities in the critical areas on the right. The effect was less obvious at lower discharges.
2. The lowering of the resistant rock ridge to elevation 1020 feet reduced the velocities, especially at lower discharges.
3. The pool mouth widened by 270 feet to the right, but did not appear to produce any benefits in the critical area. Increase in velocities at 400,000 cusec was indicated.
4. Lengthening of the pool had no effect on the velocities in the critical area.
5. The flexible apron of blocks was tested for three different sizes, but was concluded as unreliable.

In fact, none of the tests provided any positive line of approach to tackle the undermining problem of the buttress. The Irrigation Research Institute successfully tested a massive groyne configuration, which effectively shielded the main buttress but experienced erosive velocities at its nose.

Works Proposed for the Hydraulic Solution

(I) It was concluded that the best means of protecting the buttress against undermining is the groyne. Some protection to the nose of the groyne could be provided by:

- i. Placing its foundations (specially the nose part of it) as low as the contractor could manage (hopefully lower than the conjectured scour level).
- ii. Restricting the spillway discharge to 200,000 cusec, and passing the balance over the auxiliary spillway for the next season by which time a more definitive solution may have become available.
- iii. Shattering the black rock at the mouth of the pool to elevation 1020 feet, which causes some reduction in the back current velocities.

Also, it was thought that even if some undermining of the nose does occur, it would not be destructive to the buttress as scouring in vicinity of the nose would be sufficiently remote from the buttress. The model tests also demonstrated the beneficial influence of the spur in reducing the velocities in the northwestern area of the pool where the buttress extension (the rollcrete strut) is located.

(II) There is a natural weir of basic rock across the Dal Darra Channel 500 feet downstream of the plunge pool outlet. This weir in combination with high rock surface in the left part of Dal Darra Channel controls the levels in the plunge pool. Operation of the spillway had caused a breach in the central part of this rock weir. The bed level of the breached part was about 40 feet lower than the sides. This and another small breach to the left were plugged with concrete. The intention was to ensure as deep a cushion in the pool as could be managed. The plugging of the gap prevented further lowering of the Dal Darra bed and even raised the tail water level.

Protecting the rock on the left hand side immediately downstream of the bucket and plugging the sizable cove on the left hand side was necessary. There were fears that the left hand cove could enlarge and lead to serious problems if not protected. The difficulty, however, was that any rollcrete that could be placed there would be subject to undermining unless its foundations were taken below the scour depths, which were yet to be determined on the models. It was decided to place the rollcrete plug and take the foundations as deep as the contractor could manage and accept the risk of undermining. Subsequent experience proved the propriety of the judgment. No serious undermining of any part took place in the 1978-operating season.

(III) To monitor the behaviour of the pool in the critical areas, during spillway operation, a system of electric wire loops was installed. The wire loop consisted of an electrical circuit, which would break if the monitored spot is eroded. This system had successfully worked in Basin-3 and proved its dependability in the Service Spillway.

Performance during 1978

During 1978 season, the service spillway operated for 1502 hours with the discharges restricted to 200,000 cusec. No unexpected event occurred during the season except for scouring at the nose and left hand side of the groyne, which was greater than indicated by the new 1:50 scale model then operative. The rest of observations were more positive. The spillway was then run at 300,000 cusec for 24 hours to flush out debris.

A pleasant surprise ensued with minimal abrasion of the concrete surface. The inspection of the concrete surfaces of the pool showed an abrasion of 2 to 3 inches on the horizontal steps of the concrete forms. Small protruding lengths of reinforcement used to hold the forms, were found bent and the ribs on the steel bars were smoothed out in the direction of the back currents. Mostly, the abraded concrete surface looked like a lightly overworked surface for a green cut. An interesting observation was that there was practically no abrasion at the nose of the groyne where the model showed high velocities and where severe abrasion was expected.

Final Stage Remedial Works

The Service Spillway at Tarbela is unique in one respect. Spillways of most dams are very seldom, if ever, subjected to flows of about half of their design capacity. The exposure to such flows, if it occurs, is for limited duration reckoned in hours. In case of Tarbela the spillway has to cope with half the design discharge for days on end.

In June/July 1978 the inflow to the reservoir remained above 350,000 cusec for eleven consecutive days. For eight consecutive days the flow was ±400,000 cusec, which is nearly two-third of the design discharge. Luckily this flow occurred when the reservoir was below the spillway crest. Scour is a time dependent phenomenon and serious problems had arisen in a short operational history of three seasons. The satisfactory behaviour of the groyne during 1978 season provided evidence that the buttress, the groyne and the dyke in the left hand cove could be adequately protected against undermining by deepening of their foundations, and then these works could very well constitute the permanent solution. Such a solution was baptized as the 'short groyne solution'.

There was, however, another school of thought, which favoured cutting off the embayment on the right altogether and eliminating the back eddy, the very source and cause of troubles. This school of thought argued that the existing embayment would enlarge if the pool is subjected to prolong or higher flows and will continuously feed abrasive materials for years to come. This will harm the 5 feet thick jacket of concrete. It was thought that the likely extent of embayment could not be assuredly ascertained from tests. It was possible that in the long run it might outflank the rollcrete strut on the right and the situation faced now could be reproduced. Also, readjustment of stresses in the rock masses would be necessitated by measurable changes in the geometry of the pool. These readjustments could possibly cause the problems to re-appear. Similar problems could arise on the left side where the action has been delayed by more favourable orientation of bedding planes but essentially it was a question of time. A confining dyke up to the pool outlet on the right or what was also called the 'long groyne solution' would create a symmetrical pool and provide clean sweep out characteristics. There could be a piling up of material on the buttress as indicated by some preliminary model tests because of the symmetry in the pool. It could eliminate runnels and reduce scour depths.

The advocates of 'short groyne solution' did not share the apprehensions at least not to the extent believed by others. They were not impressed by the advantages listed. In their view, erosion, remote from the structure could do no harm. If at any stage that erosion were to approach near the structures, then there would be sufficient time to take remedial steps. In their view restricting and jacketing the pool by confining dykes is a step against the natural tendencies of the pool. For confining the pool, the concrete structures will have to be placed opposite to jet impaction and the high boil areas, where the structure will have to sustain high dynamic loads. In their opinion problems arising with a confined dyke solution will be more difficult to handle. They thought that deepening the foundations of the buttress, the groyne and the rollcrete dyke in the left hand cove below the anticipated scour level provided a satisfactory solution. They advocated the concept that auxiliary spillway should share the discharge when flow exceeds three hundred thousand cusec. Operated in conjunction with the auxiliary spillway, the service spillway may never experience the design discharge. Exhaustive research was undertaken to determine the merits of each solution.

Model Tests on 1:50 scale at Nandipur

A 1:50 scale model was constructed at the IRI and a very extensive programme of testing was launched. The objectives of the tests were:

1. To find out a model material which could faithfully reproduce the scour pattern.
2. To determine the scour depths for protecting the structures.
3. To evaluate the effects on scour by changes in the geometry of the bucket like removal of fillets from the sidewalls, installation of a dentate bucket etc.
4. To ascertain the influence of the downstream resistant rock on the scour patterns in the pool.
5. Measurements of dynamic pressures in relevant areas.

The following is a very brief description of the type of tests done and the major observations. To evaluate the results of the enormous volume of data involving different geometries for the works in the pool, and of bucket with varying model materials and discharges, the technique of developing a scour envelope was evolved. At points of interest the scour caused by different tests was recorded. The worst was chosen. This exercise was done for every point of interest. The design parameters were decided by the values of this scour envelope.

Salient Observations & Conclusions of model tests

(a) The first series of tests was done with a gravel clay mix. Two to eight mm gravels were mixed by 25% of its volume with local sieved pulverized clay. Water was added to make the mix workable and was tamped with pieces of wood. The mix was stiff enough to stand for sufficient time on about ½ horizontal to 1 vertical slope. The downstream resistant rock was moulded un-erodible. The pool bed and sides were moulded as per topography before the start of 1977 operation season.

The model was run for 24 hours. Allowing for the basic deficiencies in the state of the art, the chosen mix reproduced the proto condition to a satisfactory degree. The location of the deepest contour (1020 feet in case of proto and 1010 feet in case of model) and the area enclosed by it, the runnel proceeding upstream and turning to the right resembled fairly close to the runnel seen on the prototype. Had time been available, the mix could be adjusted to give even closer results. To ascertain probable scours, a full series of tests was carried out up to 400,000 cusec on the solution adopted for 1978 operational season.

(b) To save time base tests were run on non-cohesive material i.e. 2 to 8 mm gravel. Figure 9 shows the gradation of gravel used. Tests were also done with material which had d_{50} = 0.38 mm. Following conclusions were drawn:

i. For the tested geometry, cohesive material showed less scour near the concrete boundaries than the non-cohesive material. The reason may be shear drag.

ii. The difference in the scour levels indicated by the two non-cohesive model materials is marginal near the concrete boundaries. The pluses and minuses of the two-materials are almost equally distributed in a scour envelope situation.

Changing the geometry of the bucket, another series of tests was run both on 1:50 and 1:80 scale models. Broad conclusions of these tests were:

- The small changes brought about in the lateral spread of the jet by removal of fillets on the side walls did not result in any quantum change in the scour near the concrete boundaries.

- The serrated buckets aerated the jet, and to an extent, the point of jet impact was also dispersed. The scours were generally reduced by about 15 to 20 feet when compared to scours with existing bucket at many locations. At others there was no significant difference. The proposal was not accepted for fear of cavitations at the corners. Also, there is no precedent for a serrated bucket.

(c) An exhaustive series of tests was undertaken to determine scour patterns with different configurations of dykes inside the pool. The configurations of dykes inside the pool included tests with different slopes of existing groyne, different lengths, inside flares, curves and face slopes and widths of outlet mouths etc. All these were tested in combination with geometry of buckets discussed earlier. The main conclusions are as under:

i. At any given point there were considerable variations in the observed scours for a given discharge. The difference in indicated scours ranged up to 60 feet.

ii. The scours at a given point increased with discharge in most of the observations. This statement is more relevant up to 400,000 cusec.

iii. The fully confining dyke on the right created the most favourable conditions of scour in front of the buttress. The best results were obtained with a confining dyke and serrated bucket.

iv. In the upstream areas, the critical discharge was 400,000 cusec.

v. The highest discharge of 600,000 cusec caused a deep scour hole (deepest point at El: 937 feet) on the left side near the downstream end of the pool. This was particularly the case with a fully confining dyke on the right.

vi. The results obtained using fine sand (d_{50} = 0.36mm) on the 1:50 scale model were considered conservative.

vii. The removal of the resistant black rock downstream up to and below elevation 940 feet noticeably improved the scour conditions in all locations both for the “short” and “long” groyne solutions.

viii. The results of dynamic pressure measurements were not definitive. Pressure measurements were done only for two alternatives. In one case the variations were plus 12 to minus 5 feet above the average static water level. In the other case fluctuations of 17 to 32 feet were recorded above the static water level, though in both alternatives the displacement of transducer surfaces from the centre of the spillway was approximately the same.

Choice of Permanent Solution

Generally speaking the model tests by themselves did not lead to any preferred solution. The choice of the final solution was, therefore, a matter of judgment based on evaluation of model tests, assessment of hydraulic conditions not demonstrable on the model and the geo-technical conditions. Hydraulically, a long dyke on the right (Figure 10) in combination with the buttress and the rollcrete plug in the left cove provided assured means against uncontrolled erosion of the poolsides.

A confining dyke was adjudged a better solution against abrasion than a short groyne embayment. It was considered that the material left in the pool from construction would get ground down before the abrasion of concrete became serious. In the years to follow, the pool would be fairly clean as there is little chance for the materials to enter the Service Spillway pool from operation of the Auxiliary Spillway.

Such a solution virtually lines the pool on three sides and, therefore, practically overcomes all the difficulties in the hydraulic behaviour of the pool. Hydraulically, a long dyke had the obvious merit of causing shallower scours in critical areas. Elimination of scour in the area of rollcrete strut and shallower scour near the buttress reduces the potential for the movements of bucket. Enlargement of the right embayment or a deeper hole near the nose of the groyne would impair the support given to the bucket by these structures.

The fears about the dynamic pressures were allayed by the model test results. The problems with such a solution were, therefore, basically structural. The concrete had to be taken to depths below the anticipated scour level, which required deep excavations invoking slope stability problems. The Contractor's resources and the time available for construction determined the outer

limits for the excavation. These limits admitted only small variations in the slopes if the concrete surfaces were to be kept away from the direct impact of the jet.

Flatter slopes ease the stability problem but are susceptible to greater abrasion. The vertical surfaces suffer far less abrasion than horizontal surfaces in a given hydraulic environment. Conflicting requirements have, therefore, to be balanced.

The concrete in the left hand cove covered only a part of the left face of the pool. A way had to be found to protect the downstream edge of this concrete if the siliceous white and the basic black rock downstream were to erode. Large chunks had got detached from this black rock on the left hand side during the 1997 operation, which cast doubts on its long-term durability.

The concrete lining of the sides must be securely fastened to the rock underneath. The lining must withstand backpressures resulting from the accumulation of the water behind the lining. Subject to the satisfactory resolution of these basic structural problems the alternative of 'long groyne' solution was chosen.

DESIGN CONSIDERATIONS

The following considerations governed the formulation of design concepts for the chosen solution.

i. The bottom of protective works should be lower than the scour levels anticipated from model tests.

ii. The scour depths anticipated from models should be adjusted for the ultimate condition when the rock weir in the Dal Darra Channel gets eroded. The erosion of this weir will reduce the cushion of the water in the pool by about 40 feet. It was adjudged that scour in the pool will deepen by the same depth.

iii. The scour at all points should be determined from the scour envelope adjusted for Dal Darra erosion.

iv. As far as possible the concrete surfaces should be outside the area of direct impact of the jet.

v. To minimize abrasion of concrete the protection work should be as steeply sloped as possible.

vi. Thickness of concrete should be adequate to last sufficiently long before a replacement becomes necessary.

vii. Water from the spray of the jet will accumulate in the rock underlying the concrete lining. The water table in the rock can, therefore, rise above the pool level. This difference will cause pressure at the back of the lining trying to push the concrete away from the rock. A differential pressure can also arise when the level in the pool falls rapidly following the shutdown of the spillway and the water in the rock fails to drain as fast. The lining has to withstand this differential pressure.

viii. The concrete should be crack-free and the joints properly water-stopped to avoid possibility that concrete might crack and pulsating pressures penetrate the rock and cause high pressure to act from the back.

ix. The rock underneath the lining should be properly drained to reduce the backpressures.

x. The critical consideration was the stability of the spillway sides. The topography of the pool, the constraints of available time for construction and some of the considerations mentioned above narrowly restricted the choice of slopes for the sides of the plunge pool. Preliminary calculations showed that the rock would need pre-stressing to ensure stability.

The importance of the Service Spillway demanded the greatest caution in adoption of the most appropriate means to ensure the stability of the sides of the plunge pool. Had there been no constraints of time, the preference would have most naturally swung to cutting the sides at stable angles and covering them with concrete. In such a solution the drawback of a higher risk of abrasion on flatter slopes was to be weighed against the drawbacks of the means, which could enable the adoption of steeper slopes. The solution of flatter slopes has the obvious advantage of permanency over any solution, which depends upon mechanical means such as post tensioned anchors, even for a noticeable difference in cost. The problem in hand was that a very competent contractor found it impossible to execute within the prescribed time, a programme based on excavations at stable angles.

The difficulties of the situation compounded by the envisaged depths of the cuts. The stabilization of 250 feet high slopes by mechanical means is unprecedented. Once again the shear size of the project forced upon the profession the adoption of unprecedented means.

DESIGN PARAMETERS IN ANALYSIS OF SLOPE STABILITY

(a) A key parameter in slope stability analysis is the choice of the friction angle. The rocks in the Service Spillway can best be divided into three groups:

i. The thinly bedded poor quality limestone with little resistance to erosion.
ii. Basic rock intrusion occurring as a large sill and as small veins within limestone.
iii. Massive siliceous limestone on either side of the basic rock sill.

The limestone and basic rock are jointed but a factor of greater importance is the direction of the bedding planes. The bedding in the limestone generally strike N60^o and dips 60^o to 80^o SE, whereas the right and left slope faces are oriented approximately NS. The strength of a particular rock depends on the mode of failure. The limestone/phyllite would exhibit different strength when failure occurs through intake rock or along discontinuities than when failure develops along the bedding planes. The main buttress and the strut is oriented nearly parallel to the bedding, thus the stability of these structures will be controlled by consideration of sliding along bedding planes. On the left hand side the orientation and spacing of shear zones and joints do not suggest the presence of a sizeable three dimensional wedge and the entire slope is considered homogenous and subject to plain strain condition. Stability consideration in the basic rock was those of a homogenous rock taking into account the better quality of the rock.

(b) The strength values used in the slope stability analysis were determined by back calculation of existing slopes using limit equilibrium assumptions. Analysis of the steep and high slopes on the right, and of the slopes on other parts of the spillways, with factor of safety of unity and where the bedding planes do not control the mode of failure, indicated that at low values of effective normal stresses a cohesion of 3.7 kips per square foot is justified with a 63° angle of friction. For normal stresses higher than 5 kips per square foot the friction angle of 55° was derived. The back calculated strength envelope is shown in Figure 11. An angle of 55° was adopted for design purposes. For basic rock cohesion of 2000 lbs/ft^2 and Ø = 55° was adopted. In general, the findings of the study were that shear zones are not planer but undulating, yielding high angles of shear resistance.

(c) Water was expected to enter the rock mass and create differential pressures between the back of the strengthened rock/rollcrete facing and the pool. This difference would also act at the back of the concrete lining covering the strengthened rock. No analytic approach was possible to determine as to how high this differential head would be. The maximum head against which a practicable design could be prepared was about 20 feet. Reliance was placed on drainage of the rock for any excessive rise of pressures.

(d) The dynamic pressures to which the concrete surfaces were to be subjected when impacted upon by a submerged jet were calculated from a formula proposed by Beltaos.

$$\frac{p_s}{w} = 8H\,\frac{D}{L}\sin\phi$$

where, $\frac{p_s}{w}$ = pressure head at impact point less the static head.

H = Velocity head in the jet at the tailwater level.

D = Thickness of jet at tailwater level.

L = Length measured along submerged jet to the point of impact

= Depth of impact point below tailwater / Sin α

where, α is the angle between the direction of submerged jet and the horizontal.

ϕ = Angle between direction of submerged jet and the surface of concrete.

To obtain the values of the variable, the following assumptions were made:

i. The jet remains essentially a solid stream throughout its entire length.
ii. The main cause of the energy loss is the drag force exerted by the atmosphere on the surfaces of the jet.
iii. At any point along the jet, the drag force, per unit of surface area is proportional of the velocity head at that point.

The drag effects on the jet, determine the energy loss between the lip of the bucket and when the jet enters the tailwater in the pool, the X co-ordinates of the jet for the given Y co-ordinate, and the angle between the tangent to the jet and the horizontal tailwater. The drag effects were taken into account by measuring the co-ordinates of the proto jet and determining the co-efficient of proportionality. This co-efficient was estimated at 0.006 since it must be recognized that the measurements from which this co-efficient was derived are very approximate as the enormous spray obscures the jet. Dimensionless plots were developed to determine the X co-ordinates for the known Y co-ordinates, the tangent and the energy loss. The computations of these studies are given in

Table-1. The table shows computations for two values of tailwater, i.e. for the Dal Darra Weir intact and eroded.

(e) The debris circulating in the pool would cause abrasion of concrete. Most of the material could be ejected out of the pool if the spillway was run at high discharges and some material would get ground to reducing its abrasive power. Some material, however, would find its way inside the pool from the operation of Tunnel-5 and Auxiliary Spillway. It was appreciated that concrete cover over the anchor head should never be less than 5 feet and repairs must be done before this stage arrives. An additional 10 feet thickness would hopefully provide an adequately long interval before major repairs need to be undertaken. A thickness of 15 feet for concrete was therefore adopted.

(f) To guard against the possibility of transmittal of high pulsating pressures the concrete was to be as crack free as possible. The placement temperatures of concrete did not exceed 50^{o} F even during the hotter part of construction season. The concrete lining was to be reinforced, panel-led, and properly water-stopped.

Drainage galleries running parallel to the sides of the pool daylight into the pool behind the main jet where the water level is about 10 to 15 feet lower than the boil area downstream of the jet. Connecting the galleries to the low-pressure area would thus relieve the high pressures. At the back of the lining, the network of drains under each strip was brought to one elevation along which a series of drain holes were put in, to connect with the gallery. The rock mass behind the lining was drained by two series of holes, one horizontal and the other vertical, each at 20 feet centers. The spacing, angles and depths were chosen to intersect the entire bedding plane. These drains were cased with slotted PVC pipes. The area of the basic rock was provided with drain holes about 20 feet deep discharging into the pool.

SLOPE STABILITY ANALYSIS

The stability of slopes is governed by two requirements. Firstly, there must be enough rock attached to the concrete lining to act monolithically to counter balance the design unbalanced water pressure of 20 feet. Secondly, there must be enough confinement of the rock to ensure an acceptable ratio of the major and minor principal stresses $\frac{\delta_1}{\delta_3}$

Since, $$\frac{\delta_1}{\delta_3} = \frac{1 + \sin \phi_D}{1 - \sin \phi_D}$$

where, ϕ_D is the developed angle of friction, i.e. for a unit factor of safety.

$$\phi_D = \frac{\tan 55^{o}}{1.5}$$

The acceptable $$\frac{\delta_1}{\delta_3} = \frac{1 + \sin 43^{o}}{1 - \sin 43^{o}}$$

The first requirement determined the length of the anchor and the second requirement determined the force to be provided by the anchors. Preliminary trials showed that 65 feet long anchors would meet the requirements even if 10 feet is assumed as de-bonded and the remaining effective length is 55 feet.

In a conventional analysis of slopes, the distribution of stresses along the failure plane is not given any special attention. The important factor is to know the total force mobilized to resist sliding. Since post tensioned anchors were to be used to stabilize the spillway slopes, it was, therefore, necessary to examine the distribution of stresses so as to properly distribute anchors. A special limit equilibrium analysis was devised.

The strength of the rock is determined by the ratio of principal stresses from a Mohr's–Coulomb diagram. In the case of spillway plunge pool slopes; post-tensioned anchors provide the external force. The required minor principal stress (anchor force) can be determined only after the major principal stress has been sound. The following assumptions are made to calculate the major principal stress and subsequently the anchor force.

1. The installation of anchors provides a reinforced rock mass extending the sloped face to a parallel plane 65 feet away from the water when the anchors are bonded.
2. The major principal stress in the slope is assumed to act parallel to the slope face. The assumption is conservative and sufficiently accurate for the steep slopes of 2 vertical on 1 horizontal.
3. A critical potential failure surface through the rock will extend on a plane from the toe upward at an angle of $45 + \frac{\phi_D}{2}$ degrees from the assumed major principal plane f_e in Figure 12.
4. Above the point where the plane described in (3) intersects the rear of the anchored zone, the critical surface is assumed to be along the rear of the anchored zone (Plane dc in the figure). The stress distribution on the plane is assumed to follow the Coulomb's equation.

$$T = c + \delta' \tan \phi_D$$

Where, T = Shear stress

δ' = Effective normal stress

ϕ_D = Angle of friction for unit factor of safety.

5. The state of stress on the assumed major principal plane does not change with increasing distance from the slope face.
6. Above the point of intersection mentioned in (3) and the rear of the anchored zone point c in Figure 12, the confining stress δ_3 required to be supplied by the anchors is

$$\delta_3 = \delta_1 \tan^2\left(45 - \frac{\phi_D}{2}\right)$$

$$- 2c\tan^2\left(45 - \frac{\phi_D}{2}\right) \qquad \text{(c = Cohesion intercept)}$$

Below point c the confining strength requirements are assumed constant.

7. The distribution of stress is assumed as triangular and trapezoidal above and below point c. At the top point the stress is assumed zero and increases linearly to twice the

average at c. In the lower portion cf, the stress at f is assumed to equal the weight of the concrete above this point. The stress along the face is assumed to increase linearly to a value so as to yield an average on the face as determined from rigid body analysis.

DESIGN CALCULATIONS

Following these assumptions the calculation of the anchor force is a straightforward exercise in construction of force polygon. Calculation sheet is presented as Figure 13 to illustrate the method. In the limestone the cohesion was assumed as zero. Figure 14 shows the calculations for the main buttress. Crosscheck on these calculations was carried out by analyzing the forces on the plane failure by the standard conventional method. It will be observed from the calculation sheet (Figure 15) that the force polygon closes for block 1 and an anchor force of 400 kips per foot is required for closing the polygon for block 2. Special limit equilibrium analysis (Figure 13) estimates this force at 438 kips per foot for anchors below 1050 feet elevation when only such anchors as cross the plane of failure by 10 feet or more are assumed to contribute to stability. The figures testify the propriety of the method used. The calculations were for static conditions with a factor of safety of 1.5.

Dynamic Analysis for Seismic Conditions

The dynamic studies on the plunge pool slopes were examined for the criterion that the slopes should not get displaced by more than one inch under accelerations impacted by the Maximum Credible Event. The Maximum Credible Event is defined as one, which has a return period of 100 years. The approach to calculations was:

i. Select a representative accelerogram for input motion.

ii. Determine the yield acceleration (the acceleration under which the body will just move) of the slope in question by determining the pseudo-static acceleration, which will cause the yielding of the slope in a conventional slope stability analysis.

iii. Calculate by double integration, the cumulative displacement caused by pulses of acceleration higher than the yield acceleration.

The accelerogram adopted (Figure 16) for the site was developed by Prof. Bolton Seed who thought the earthquake resulting from the Darband fault could reasonably be represented by a modified form of earthquake felt at Paicoma Dam. Pseudo Static Analysis with a rollcrete reinforced rock thickness of 65 feet show the critical acceleration to be 0.20g (Figure 17).

Anchor requirements at various sections of the slopes were worked out by drawing the polygon of forces for horizontal acceleration of 0.15g, 0.25g and 0.35g. The dynamic force was calculated by using total weight. It was found that anchorages provided for the static conditions (FS = 1.5) met the requirements for accelerations less than 0.30g. Figure 18 shows the illustration of calculations and the force polygon to determine the anchor requirements.

Displacements were calculated assuming different values for critical accelerations in the Paicoma modified accelerogram. The criterion for limiting the total displacement to one inch for the critical acceleration of 0.20g is satisfied for the case in study.

In conclusion, the dynamic analysis showed that the design evolved for the static case was satisfactory for earthquake with a return period of a hundred years. Consideration was given to the hydrodynamic forces generated by the water in the pool.

Westergaard equation,

$$F_{max} = C \gamma_w \frac{\alpha_{max}}{g} h^2$$

In which, γ_w = Unit weight of water

α_{max} = Maximum ground acceleration

h = Height of water

g = Acceleration due to Gravity

C = A co–efficient

yielded very conservative values if the value of C was taken for the rigid wall case. A finite element analysis indicated these values to be half of Westergaard's value. The co-efficient C was practically independent of the height of water and slope seem to have a minor influence of 10% on the coefficient for slope of ½ on 1 to ¼ on 1. It was interesting to note that the values of the maximum hydrodynamic force computed by finite element analysis for different heights of water were remarkably close to Westergaard values if the co-efficient C was put at 0.00019.

It was deemed that the forces generated by Westergaard effects in the pool would be counter-balanced by the negative pore pressures created due to dilation along the failure plane.

Anchor Capacity and Spacing

In the repairs of stilling basin, the Contractor had successfully installed anchors of 500 kips at 60% of Guaranteed Ultimate Tensile Strength (GUTS). A one-inch extension increases the stress to 80% GUTS. At the rock surface, the bearing capacity of rock surface was found adequate for approximately 51 x 49 inches base of the anchor blocks. Each tendon consisted of 15 strands, 0.60-inch diameter high strength steel placed in 5-inch diameter hole. Twenty feet length was bonded. For ease of construction the vertical spacing of 10 feet was kept in almost all areas. The required force was adjusted by adopting horizontal spacing of 8 feet and 13 feet. Concrete was placed in panels, 30 feet high and 40 feet wide.

Stability of Concrete Lining

The possibility of sliding of the concrete lining would arise, if its base were undermined. Care was taken to the lining below the expected scour level. Also, the placement of 30 feet thick rollcrete pad in front of it would provide added assurance. Assuming 45° friction angle at the concrete rock interface, the factor of safety against sliding was found to be 1.5 and 1.0 for the static and earthquake conditions, assuming its toe undermined. The force acting under seismic condition included the force provided by the anchors as the lining was securely tied to the anchors with a spider of steel reinforcement running under the anchor block and meshing with the steel of the concrete lining. For the static condition, the resisting force was simply taken equal to the effective weight normal to the surface times tan 45^o.

Finite element analysis was done for estimating thermal stresses and the possible cracking resulting from such stresses. Assumptions were made that concrete will be restrained by anchor blocks and at ridges of good quality rock. The analysis indicated the principal tensile stresses higher than the tensile strength of the mass concrete specified for the job. A mat of reinforcement was placed near the rock face to control such cracks.

Long Term Stability of Slopes

The stability of the slopes is dependent upon the anchors and unfortunately very little information is available on the long-term performance of post-tensioned anchors. The longest experience is on Chauerfas Dam in Algeria where anchors were installed in 1933. These anchors are reported to have lost 5% load. The 120 feet high Muda Dam in Malaysia has 205 anchors installed in 1967/68. About 10 of these have suffered losses in load up to 30%. The analysis at Muda Dam show that six anchors have probably suffered from corrosion, the others lost load due to other reasons.

The anchors in the Service Spillway of Tarbela Dam cannot be monitored. To get some idea of the long term performance of the 1600 anchors installed, six anchors 90 feet long were installed from the berm at El: 1200 feet on the left hand side of the Spillway. They are placed in an area where the rock is weakest and which best represents the geological environment for most of the anchors.

The programme of surveillance of test anchors was intended to investigate changes in anchor loads over a long period and displacements of bonded length of the anchors. The test anchors and the surrounding rock have been provided with following instruments for measurements:

- i. Load measuring cells under the anchor heads.
- ii. Reference marker for measuring movement of anchor block.
- iii. Reference marker for measuring movement of strands in the bonded part of the anchor.
- iv. Reference marker for measuring movement of the bond grout.
- v. Reference Marker for measuring movement of adjacent rock.

DESCRIPTION AND TYPES OF INSTRUMENTS

Glotzal Load Cells

These have been mounted beneath each head to measure anchor load. The Glotzal load cell system for anchors has been developed in West Germany by “INTERFELS”. This system works on hydraulic principles. It is very robust in construction and simple in installation. These cells can measure anchor loads to an accuracy of 1%.

Rod Extensometers

Extensometers have been installed to measure for each anchor, displacement of the anchor strands and the surrounding grout. Five rod extensometers have been installed at various depths for monitoring the movement of the surrounding grout. The entire bonded length was covered with two extensometers at the bottom, two in the middle and one at the top. One extensometer has been installed five feet above the bond length.

Similarly, they have been installed at various depths for the measurement of movement of the strand in the bonded part. In addition, displacement of the adjacent rock is measured by installing rod extensometers in the center of each triangle of the three anchors. They are read with dial gauge with a measuring accuracy of ± 0.1mm.

Surface Survey Measurement

The displacement of the anchor blocks is determined by survey referred to a fixed point.

Stand Pipe Piezometers

The standpipe piezometers were installed to measure ground water depth, temperature and to do water sampling for chemical analysis.

Besides, to check long-term behaviour of anchors, instruments were installed to verify other elements of design assumptions. These included electrical piezometers to determine the differential pressures immediately behind the concrete lining and standpipe piezometers in the general body of rock. To determine the dynamic pressures, transducers were installed during 1978/79 on the left side and in 1979/80 on the right side. In addition, erosion detection wire loops have been installed to monitor the abrasion of concrete.

MONITORING OF SERVICE SPILLWAY

The Service Spillway was closely monitored during construction. Some instruments would continue to provide useful information in the past construction period. There is a category of instruments, which shall provide information for future designs.

The type and number of instruments installed at the Service Spillway were.

1.	Rod Extensometers	15 Nos.
2.	Electrical Piezometers	18 Nos.
3.	Standpipe Piezometers	12 Nos.
4.	Pressure Transducers	17 Nos.
5.	Erosion Detectors	158 Nos.

Most of the above had been installed at the time of writing this paper. A brief description of the instruments is given below:

Rod Extensometers

These instruments are meant to measure the movements, if any, of the rock slopes around the Plunge Pool during and after construction. In an extensometer, the relative movement between two points is transmitted by means of a rod to a gauge from which a direct reading can be made.

Seven extensometers have been installed on the left hand side, two on the buttress and seven on the right hand side. The lengths of the rock extensometers vary from 60 to 85 feet.

Piezometers

The purpose of the Piezometers is to record the hydrostatic pressures behind the concrete lining and to monitor the performance of the drainage system in the rock mass and at the back of the concrete lining. Seventeen instruments were installed on the left side, three are placed on the buttress side and ten on the right hand side. Out of the above instruments, twelve are standpipe piezometers and the rest are the electrical piezometers.

Pressure Transducers

The pressure transducers have been installed to determine the magnitude of the hydraulic forces caused by the plugging jet. It is recognized that the zones of high pulsating pressures lie between station 6+00 and 8+00. Consequently the transducers have been fixed between these stations. Ten are on the left side and seven on the right side. The right side dyke is farther from the right edge of the jet.

Wire Loop Erosion Detectors

A network of wire loops has been laid in the concrete facing to monitor the erosion of concrete at selected spots. This instrument was developed by the instrumentation Division of Tarbela Dam Project and has been described earlier. The instruments on the left hand side were installed during 1978/79 construction season.

Some Preliminary Remarks On Instrument Readings

All instruments have satisfactorily responded. The extensometers did not show any general movement of the rock although some local movement was observed while the excavation was in progress. Thereafter, the movements ceased. The piezometers have shown quick response to the closing and opening of the spillway gates. The piezometer readings during 1979 operations have established that the drainage system works satisfactorily and no adverse differential pressures exist between the back of concrete and the plunge pool.

The pressure transducers remained buried under an access ramp of fill material constructed by the Contractor for placing concrete at the higher elevations. This ramp did not erode during the spillway operation of 1979. The readings taken on the transducers indicate a RMS fluctuation of about 2 feet. Since the theoretical calculations indicate substantially higher values, it appears that the fill material attenuated the pressure fluctuations.

CONCLUDING REMARKS

Plunge Pool formation in poor quality rock is uncertain and unpredictable. Erosion can lead to serious problems of stability and outflanking of the structure. These risks increase manifold if the structure has to be intensively used to pass on appreciable percentage (say 40 to 50%) of the design discharge. No model test technique is available which can reliably take into account the different characteristics of erodibility. In a plunge pool bounded mainly by easily erodible rock on intrusion of hard rock can cause serious local erosion of the softer material. Reverse currents are necessary concomitants of a plunging jet. These currents could be the cause of major troubles. For poor quality rock it is best to conduct model tests with cohesion-less materials of appropriate gradation. Some engineers think that a pre-excavated plunge pool may prove less expensive than later remedies. A plunge pool in poor quality rock leaves many questions to be answered.

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