Papan dam studies and remedies.

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SYNOPSIS. As part of a major irrigation refurbishment programme in Kyrgystan, seven major dams built in the Soviet era have been examined and rehabilitation measures designed and costed. Perhaps the most complex of these was Papan Dam near the city of Osh on the Silk Road. The dam is a 100m high gravel embankment with a grouted core, set in a very narrow limestone gorge. High regional seismicity and local fault alignments all increase the risk of failure of the works, and the provincial capital downstream is only part of the consequent hazard. This paper describes the inspection and investigation process, and the difficulties of identifying the seepage pattern in a three dimensional context. The justification of remedial measures currently in progress, and comprising a 70m deep diaphragm wall through the upper half of the dam core, is discussed. A short description of the bottom outlet and spillway and their current condition is given, together with comments on the parallel issue of reservoir operation and flood freeboard.

INTRODUCTION

Papan Dam has a height of 100m, above over 20m of alluvium, but is set in an extremely narrow gorge. The crest level is 1290 masl (metres above sea level) but the adjacent cliffs soar another 300m higher, with site access only by tunnel. The 90m crest length reduces to under 20m gorge width at foundation level, and this inner river channel winds within a doglegged and possibly faulted canyon. The embankment dam was constructed in three height stages, to different standards and concepts, and provided with a combined bottom outlet and spillway comprising an intake tower and tunnel.

The Papan water storage project was inspected under the World Bank funded Kyrgyz Irrigation Rehabilitation Project and a dam examination report (DER) issued in June 1999. This report was based on the historic drawings and records made available to the consultant TemelsuGIBB (Joint Venture). The technical and safety evaluation raised concerns in relation to:

- Earthquake resistance and possible fault break
- Flood routing and discharge reliability
- Seepages through the dam and high phreatic surface within the downstream shell
- Operation of hydromechanical equipment.

Long-term benefits and performance of dams. Thomas Telford, London, 2004.

The situation of the dam as understood at this time is described by Jackson & Hinks in Dams 2000 (Reference 1). The recommendations of the Panel of Experts included restricting the maximum normal operating level of the reservoir to 1270 masl, until implementation of rehabilitation works permit safe operation to the full supply level of 1282 masl. The owner had already self-imposed such a reservoir level restriction since 1990, in response to seepages observed on the upper downstream slope at higher reservoir levels.

The site investigations were carried out in 2002 and were immediately followed by preliminary design of full depth core and curtain rehabilitation measures. Monitoring of the expanded piezometer layout over the following reservoir operating cycle gave a better indication of the internal seepage regime. Eventually the measured phreatic surface was used to calibrate a two dimensional seepage model, in which alternative cut-off works could be examined. This led to a contract being let for construction of a 70m deep plastic concrete diaphragm wall within the dam core from crest level, at a much reduced cost compared with treating the full 120m depth. However, the three dimensional reality of the seepage pattern is more difficult to define, and the adequacy of the remedial works will only be confirmed by piezometric monitoring, before and after the diaphragm wall construction expected in 2004.

The long investigation, monitoring and rehabilitation path for the embankment has given time for the other defective operation and safety aspects to be studied and resolved. In particular the bottom outlet capacity is limited as much by downstream interests as operational constraints. Rather than construct a second independent spillway, a compromise maximum reservoir operation level has been selected which will store and reduce the design flood peak while maintaining the irrigation benefit of the project. This resolves the apparent spillway discharge deficit in western eyes whilst improving on the typical Russian reliance on flood storage. Many local dam projects have no spillway at all, since they are oversized reservoirs for current yield and may be raised in future to suit water demand.

SITE INVESTIGATION

The layout and key sections for Papan Dam are shown in Figure 1, and construction took place from 1975 to 1985. The construction stages are indicated and the unusual downstream shell zoning that resulted. The few remaining piezometers had indicated a high phreatic surface across the downstream shell, suggesting a defect in the water tightness of the centrally located core/curtain and an internal hydraulic control under the downstream berm. The lower core was known to consist of a nine-line grouted zone constructed within a selected clean gravel fill, supported by sandy gravel shells. Details of the geometry of later stages relied on an obviously super-

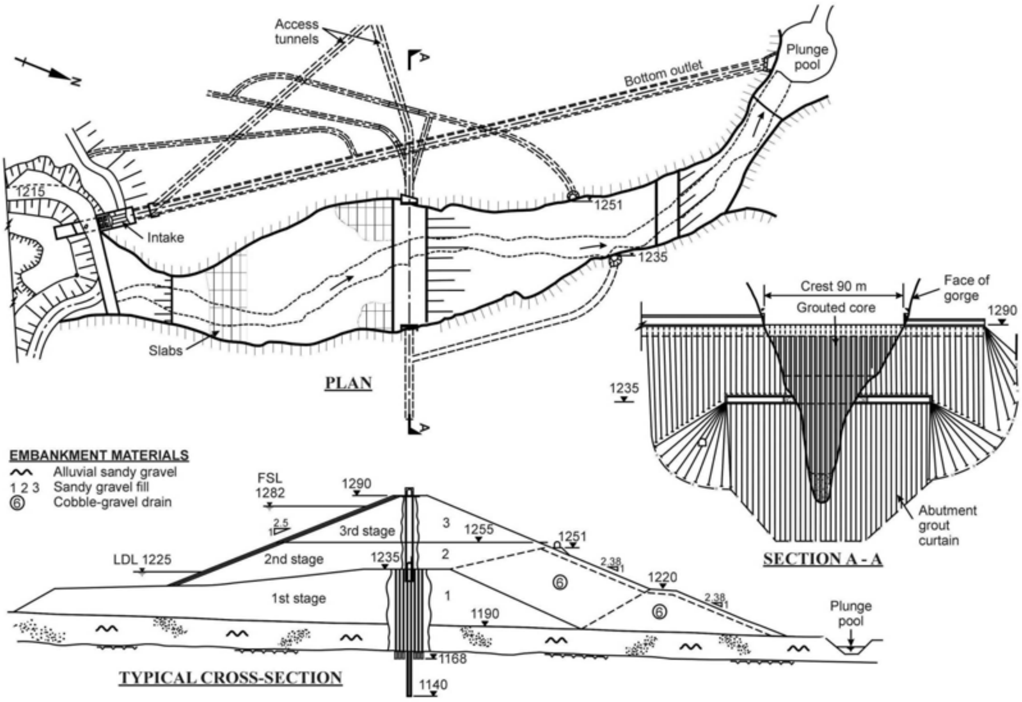

Figure 1. Papan Dam Plan and Sections

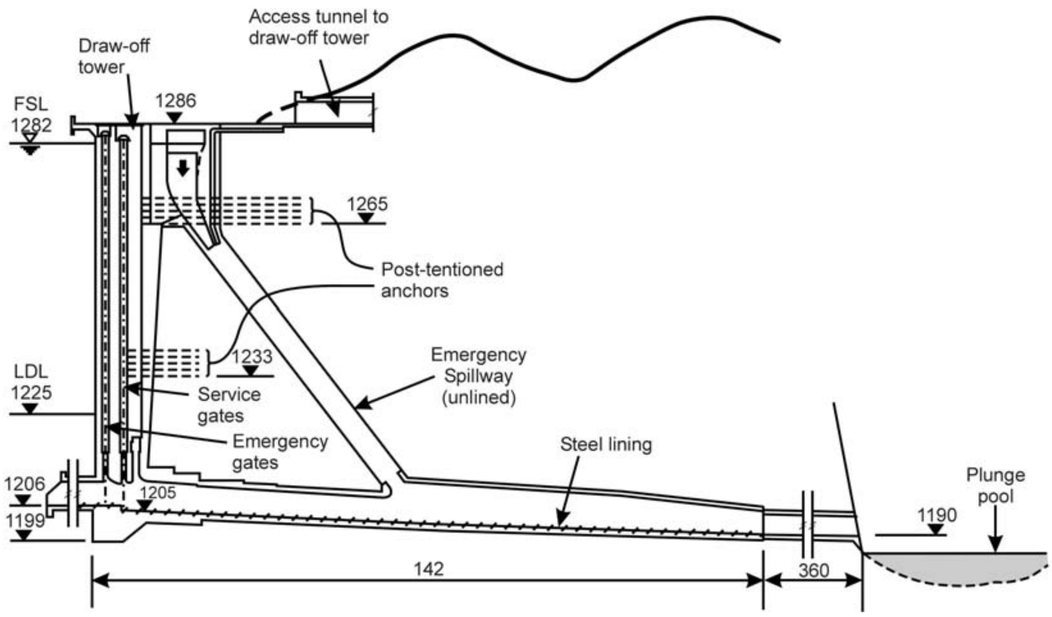

Figure 2. Combined Intake of Bottom Outlet and Spillway of Papan Dam

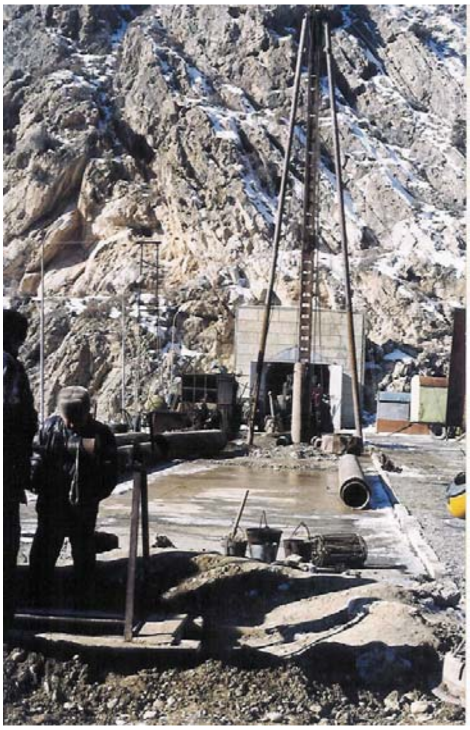

Fig.3. Investigation on Dam Crest

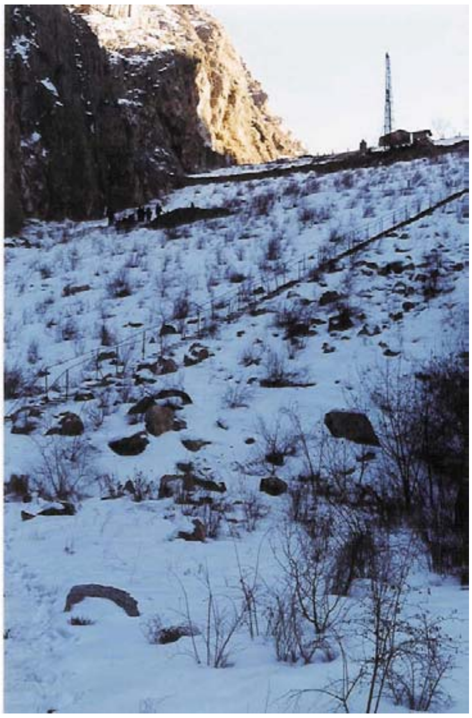

Fig.4. Downstream Slope from Berm

superceded planning drawing, but it was known that the upper core and shells were placed across the full dam width as a single sandy gravel zone followed by three lines of grouting near the dam centerline. The right abutment grout curtain at the upper level had been re-grouted in 1990 to reduce leakage into the gallery from 120 to 52 l/s.

A suitable layout of boreholes and piezometers was instructed with four percussion boreholes defining the cross-section including a 120m deep hole through the core zone. Fifteen rotary holes with piezometers were added in the downstream shell of the dam body, and four 20m deep trial pits - two from the crest into the core. From the tunnel and upper gallery four sets of two boreholes, inclined upstream and downstream of the curtain, were instructed and five minor holes at low levels within the abutments. The local water well drilling organization, called the Kyrgyz Geological Expedition, carried out the work. They provided a heavy-duty percussion rig (Figure 3) and lorry mounted rotary drills, the shafts being subcontracted to the Kyrgyz hydro institute (Kyrgyzhyprovodhoz) who also supervised day to day and carried out the soil testing (density and gradings).

Limitations & Results

Not surprisingly the $100,000 budget and one-month programme had to be doubled and trebled respectively. The recently independent state drilling enterprise had a learning curve on contractual obligations and did a magnificent job with the available equipment. The first deep hole alone took three months and the congestion on the narrow downstream slope prevented simultaneous drilling of more than two or three holes. Although accustomed to pumping out tests for wells, the drilling team found difficulty with constant head and falling head permeability test procedures within extremely deep boreholes. Providing formulae, instructions, occasional supervision and review of calculations is not enough to obtain accurate data from inexperienced testers, as we shall see below.

The steel casing type piezometers and acoustic (non-electrical) sounding equipment endemic to the region are difficult to reconcile with western practice –but the Kyrgyz in turn do not consider a narrow bore plastic pipe and electrical sounder subject to condensation on the pipe walls as reliable in cobbly-gravel fill, with calcareous deposition in all drains or pipework, and subject to frequent seismic shaking. Crucially the steel casing extends two metres below the response length and is capped at the bottom end. This provides a collector 'bucket' for washing out and removal of drilling mud during commissioning of the piezometer, since the rotary drilling depends on mud (and even lorry loads of loess) for stability of the holes within gravel fill. However, it also guarantees a piezometer reading even though the phreatic surface is far lower.

With hindsight there were also errors in the much discussed piezometer layout. Basically the arrangement of lines of piezometers yielding dam cross sections, and transverse cross-gorge sections was appropriate, but economies and misconceptions combined to frustrate this simple plan. The diamond shaped zone 6, placed in stage two as the downstream part of the downstream shell, proved to be such coarse cobble-gravel that permeability tests were meaningless and drilling was often curtailed by total loss of drilling fluid – all in the area thought to suffer from an exceptionally high phreatic surface. Setting out on a loose gravel surface (Figure 4), disfigured by a temporary zigzag access road, and confined by near-vertical, irregular canyon walls, was based on offsets from the crest and from a downslope steel access ladder. The narrowness and winding geometry of the inner gorge meant that many 'deep' holes simply hit rock prematurely. The layout had in any case been planned for monitoring seepage conditions for lake levels at or above 1270 masl, because of the high-level seepage reported. Eight out of 18 piezometers installed in the dam body are so shallow that they are never going to register a water level until after rehabilitation. Put simply, every deep piezometer counts for interpretation and there is no redundancy in the system.

The exploration in the dam core was particularly useful in indicating that the grouting had been only partially successful. The deep borehole appeared to indicate a large-scale window of high permeability in the lower core. Together with the 20m deep by one metre square, wood-braced, shafts a good impression of the upper core was also obtained. Eye witness accounts of construction had been useful in establishing that the gravel here was placed in 60 cm layers with non-vibrating roller compaction. The dumping and dozing placing sequence resulted in significant segregation, and only the top of each layer was compacted, resulting in effective horizontal stratification with permeable sub-layers and thin aquicludes. Only occasional boulders, wedges or sills of cemented gravel or plain cement grout takes were found in the upper core constructed by grouting this segregated, sandy gravel material. This scenario is consistent with the seepages observed at high levels on the downstream slope whenever the reservoir rises above about 1270 masl.

Geology & Faults

The strong crystalline Carboniferous/Devonian limestone forming the walls of the gorge is extensively jointed, with variable bedding and locally karst chimneys visible. No new rock cores were extracted as the whole area of the gorge had been extensively explored prior to construction, with addits, geophysical survey and deep coring. There appears to be a halo of stress-relieved, open jointed rock in the canyon walls, and a single line grout curtain on the dam core centerline. A major sub-horizontal, open plane of discontinuity is visible at a level around 1280 masl on the upstream right abutment. Prior to construction the groundwater flowed from right to left across the canyon, and the plateau above the gorge also imposes an intermittent flow towards the gorge. The South Katarsky fault runs along the side of the reservoir and across the upstream toe of the dam, and represents a major regional thrust fault. Overlying intact Middle Quaternary terrace deposits show that movement has not continued since that time. Conceivably the gorge location is determined by an associated tear fault, although its presence was discounted by the original (Tashkent) Design Institute, who opportunely searched for any evidence of breccia zones below the river or differential movement of the walls. The bends of the river gorge would require two suites of short en echelon faults to explain the erosion pattern. This subject was much discussed but not effectively proven as an active fault feature capable of past and future movement rather than a simple joint alignment. The definition of an active fault as defined for New Zealand in Reference 4 was found useful: repeated movements in the last 500,000 years or a single movement in the last 50,000 years. Detailed procedures to determine previous movements were proposed but not rigorously applied, due to the featureless massive limestone and lack of significant terrace remnants within the narrow gorge. This type of dam could in any case withstand minor fault movement without failure.

PRELIMINARY DESIGN

An interpretative report of the site investigation was prepared by GIBB just before the delayed end of the drilling contract and was sufficient to define the various material zones, sub-zones and their characteristics. This left the important piezometric monitoring during the reservoir operating cycle to a later date. The initial phreatic surface, determined on the dam cross-section for a 1248 masl reservoir level on 11/10/2000, confirmed that the inner downstream lower shell of first stage construction was saturated. However it also indicated a near horizontal water level just above the top of foundation alluvium running from the midpoint of the downstream shell to the dam toe. (30m lower than the high levels previously recorded on a single piezometer). Water was also appearing in the right bank lower gallery slightly in advance of and above the water level in the inner downstream shell. At this juncture all parts of the core curtain system were considered as possible culprits for the various leakage phenomena.

Accordingly, whilst awaiting more significant monitoring data, a design report was commissioned covering full depth rehabilitation of the dam core/curtain. This involved the feasibility of diaphragm walling or grouting to 120m depth in gravel, technical methodology and cost estimates. The assistance of Mr Gabriel Jorge, ex-S.American manager for Soletanche was obtained and four detailed projects drawn up:

- Full depth bentonite cement and silica gel grouting using tube-a-manchette on 9 lines. (Project 1)
- Full depth plastic concrete diaphragm wall construction with hydrofraise equipment. (Project 2)
- A hybrid project combining an 85m deep diaphragm wall with lower core grouting by angled holes from the ends of the lower gallery. (Project 3)
- Full scale re-grouting of the abutment grout curtain, or part thereof, to supplement the 1m embedment and 10m contact grouting halo included in the other three rehabilitation alternatives. (Project 4)

The estimated construction costs for the three alternatives core rehabilitation projects, with associated investigation and control monitoring, were $US 12.2 Million for core grouting Project 1, $US 7.5 Million for full depth diaphragm wall Project 2, $US 7.5 Million for the Hybrid Project 3. These prices were based on worldwide rates and included a 20% contingency for the isolated location and over half a million $US for mobilization. Each project was programmed to be completed in a single year in view of the snowbound winter conditions. The Hybrid Project 3 was capable of being subdivided into two phases with $US 4.8 Million allocated to the 85m deep diaphragm wall and $US 3.5 Million to lower core grouting from the ends of the existing lower galleries. Project 4 to extend the contact grouting halo to full abutment grout curtain rehabilitation was estimated at $US 3.6 Million, including the same 20% contingency and $US 0.25 Million for mobilization. The possibility of treating only part of the abutments on a pro-rata cost was mooted. The Interim Design Report A covering these matters was issued in March 2001, shortly followed in April 2001 by an initial Monitoring Review Report B from which a decision on the appropriate project to adopt or adapt was expected to emerge.

PIEZOMETRIC MONITORING

The gradual reservoir rise in the winter of 2000/2001 was from a base level of 1230 masl in June up to a peak of 1263 masl in mid-March as the snow-melt season progressed. Thereafter the irrigation releases exceeded inflow, but data up to August 2001 was subsequently analysed and added to the report graphs and figures. Although this reservoir range is rather limited compared with the full range of 1225 to 1282 masl, and has not been amplified in the subsequent years, it was sufficient to derive some surprising conclusions and to illustrate the limitations of the piezometer layout. The data was plotted against time in comparison with reservoir level and onto an idealized dam cross-section plus three gorge sections. These gorge sections were located downstream of the core, at midslope and through the downstream berm, conveniently breaking the data into manageable parcels and focusing on areas of interest. In addition the situation in the left and right abutments were separately analysed. The initial finding of a low-level, near horizontal water table across the downstream half of the downstream shell was confirmed for the range of reservoir levels as a sort of internal stable tailwater level, at around 1195 masl with a crossfall of under 1.5m (downstream gradient less than 0.02 below the berm). This prompted an enquiry into the historical record of high piezometric levels recorded at intervals over many years in the downstream slope. This was traced to a single standpipe piezometer (n8') that, although indicated on the drawing as extending down into the alluvium, in fact terminated at a high level –similar to the ghost readings it had been producing. Pouring ten metres of water into the piezometer resulted in a swift return to the residual level. Therefore all the initial stability checks for the 1999 DER, finding the dam just stable with some crest subsidence under earthquake MDE acceleration of 0.72g, were very conservative, since they were based on a false premise with the downstream phreatic surface 30m higher than reality.

The downstream berm feature is not some temporary cofferdam feature obstructing flow, but a drainage zone. A simple calculation of the flow through the narrow inner river channel alluvium, using measured insitu permeability at the measured low hydraulic gradient, indicates that the discharge is insufficient to maintain equilibrium so the rock walls of the inner gorge are also carrying seepage flows. Looking at the upstream gorge section the water levels in the inner downstream shell zone (first stage construction) are close to 1228 masl and conceivably fed from higher levels on the abutments. The left abutment plots are all based on dry readings, and this whole abutment is considered to be a groundwater sink. On the right abutment the readings are near the bottom of the inclined piezometers from the upper gallery down to 1250 masl (possibly in mud or trapped end sections), and water overflows from the lower gallery piezometers at 1235 masl level. The upstream piezometers faithfully reflect the reservoir level and appear to indicate a head drop of 3 to 8m across the grout curtain.

As part of the selected rehabilitation measures these raking piezometers from the high level will be extended downwards to overlap with the lower gallery and thus give a reliable reading in the critical range for intermediate reservoir levels. Additional rock drainage measures will probably convert them to permanently dry piezometers, unless right abutment seepage is a major source for maintaining the water level in the adjacent shell zone. The midslope gorge section shows a water level in the shell at 1201 masl, just 7m above the internal tailwater level and giving the overall downstream gradient as 0.05. There is thus an internal waterfall 27m high close to the downstream slope of the first stage construction. This was picked up during drilling as a high permeability (75m/d) sub-zone, and may simply represent the leading edge of the second-stage, diamond-shaped, internal drainage zone 6 of cobble-sized gravel or a rock boulder layer of rejects or riprap on the then downstream face. By accident or intention an inclined internal chimney drain has resulted downstream of which the internal phreatic surface is insensitive to reservoir level fluctuations.

Finally it was discovered that the line of existing piezometers immediately downstream of the 25m wide crest, again indicated on the drawing provided, was actually just one defunct piezometer. The reading on piezometer n2', thought to be on this line, actually corresponds to the gorge section 40m downstream of the dam core axis on which the line of new piezometers were installed (supposedly to fill the gap). There is thus still a gap in the critical area just downstream of the core where piezometric readings might indicate whether the seepage is passing through upper, middle or lower core. This may be indicated as a concave or convex phreatic surface joining reservoir level at the single upstream shell piezometer to the established upstream row of piezometer readings. Again this defect in the piezometer layout will be remedied with a new row of instruments installed to control the diaphragm wall performance. The seepage contribution of the much grouted right abutment and of the alluvium below the core (also grouted on just three lines) also remain unknown factors. Before taking a decision on rehabilitation measures and priority areas to be treated, a computer seepage model was commissioned in an attempt to resolve these issues.

SEEPAGE STUDIES

The two-dimensional seepage model was prepared using the program SEEP/W and the assistance of Clare Glenton. We started with the zone boundaries and permeabilities from the Site Investigation interpretation, and this created a phreatic surface as for a homogenous section, exiting onto the downstream slope near the top of berm level 1220 masl. This was plainly due to the supposed window in the lower core. Of course the water test results from one borehole towards the back of the core does not imply the window cuts through the whole 9-line grout curtain. Taking a probability view on the mass permeability of the lower core the window was closed down to 10% of its potential flow. Thereafter the model required 8 further iterations in order to adjust the phreatic surface to fit the monitored version. Each time the permeability of one or other key zone was modified, and in some cases the vertical/horizontal permeability ratios. This calibration process intrinsically accepted the overall seepage pattern of the piezometric data in preference to measured in situ point values of permeability. Even supposing that the insitu tests were accurate, they need not be representative of complex zones subjected to years of high gradients and particle migration (suffosion). Of course the zone interpretation in terms of geometry, permeability relative to adjacent zones, material type and placing method were respected as far as possible.

To avoid potential errors at the beginning, peak or end of the monitoring series, three calibration curves of phreatic surface were used corresponding to reservoir levels:

- 1262 masl – either side of the peak 1263 masl.
- 1250 masl – descending reservoir stage, and when it became available
- 1242 masl.-close to the August 2001 minimum of 1241.5 masl.

The model based on the first two of these phreatic surfaces, and incorporating significant modifications of initial permeability values, was then able to predict the third. The model replicates in two dimensions what is in truth a three-dimensional seepage pattern, with probable transfers from the right abutment and to the left abutment rock. It is thus superior to a simple zone/permeability model lacking calibration and a reasonable predictive tool, which was named the basic revised seepage model. However, it is not a unique model since the permeability of the ultimately controlling upper core zone is not affecting the calibration against the observed phreatic surfaces. This point was difficult to communicate since the original hypothesis of a downstream hydraulic control below the berm had permitted simple extrapolation of phreatic surface data to the highest flood levels. Nevertheless the model was able to simulate the observed seepages on the upper downstream slope at high reservoir levels, which was the primary concern of the owner. This was only possible because the particular program indicates flows above the zero pressure line. Hydraulic gradient contours were produced for the 1286 masl reservoir flood level, and values of 6.5 occur in the second stage shell adjacent to the top of the drainage zone, compromising the filter relationship between sandy gravel and rockfill. Even higher gradients are registered at the adjacent external slope around 1250 masl, indicating sloughing may be expected where seepage losses had indeed been observed at high reservoir levels. It also estimated unit width seepage flows at chosen sections, but care has to be used in deriving three-dimensional flows in the gorge situation.

By this stage the Panel of Experts including Professor Raymond Lafitte, Jonathan Hinks and Professor Bektur Chukin had co-opted a grouting specialist from Moscow, with records of the original lower core grouting. Based on his evidence they inclined to suspect that seepage loss flows were more likely to be crossing the core/curtain alignment through the upper core than elsewhere. The model was used to determine the effect of a diaphragm wall cutting off the upper core zone (first phase of the hybrid Project 3). Distinct depths of penetration of 85, 75, 70 and 60m were modeled and hydraulic gradients and seepage flows derived. All the plots were similar. Gradients behind the toe of the wall and within the downstream shell do not exceed 2.5, which should avoid suffosion effects, or at any rate avoid significantly increased particle migration over the current situation. Total leakage predictions are subjective, but unit width flows decrease only gradually with depth of cut-off wall and by up to 12% in comparison with the no cut-off version. The real benefit is thus only to cut off the horizontal seepage on privileged paths through the segregated sandy-gravel fill in the upper shoulder of the dam.

BOTTOM OUTLET & SPILLWAY

The combined spillway and bottom outlet system were described in Reference 1 and are shown here on Figure 2. TemelsuGIBB calculations for flood routing in the DER 1999 were based on western concepts such as passing the full PMF over a spillway, ignoring the bottom outlet. Dambreak would not only destroy the provincial capital Osh but also pass through the main agricultural zone of the Ferganah Valley, and cross several national borders as a flood wave on the Syrdarya River (Reference 2). There are numerous local precedents for designing dam projects with massive freeboard so as to absorb flood peaks as reservoir storage. In the unusual case of Orto Tokoi dam the spillway capacity had to be severely restricted to increase the live storage and water yield. In most other Kyrgyz dams additional spillway provision was recommended, and in the case of Papan this implied an expensive separate tunnel spillway with the possible addition of a mini-hydro station to augment the two 9m long weirs at the intake tower (Figures 2 and 5).

Understandably the client stalled on this issue until the necessary embankment rehabilitation was decided and permissible dam operating level was established. The possibility of glacial lake bursts within the far upstream catchment was mooted (Reference 1), but dismissed after examination of possible natural dams originating from landslides, fault movements or moraine deposits. Reservoir routing of PMF and 1:10,000 year floods had been carried out for the temporary maximum reservoir level of 1270 masl and for the normal maximum of 1282 masl (spillway crest level). The flood hydrographs comprise a narrow 2-day peak of 652 and 464 m^3/s respectively superimposed on snowmelt base flows of around 150 m^3/s. For the 1282 masl reservoir starting level, dam crest level of 1290 masl and the intake platform at 1286 masl were compromised by the extreme cases even with maximum gate discharge of 260 m^3/s. The two huge wagon gates would be difficult to adjust in a flood situation and only a 1:1,000 year flood could be accommodated at the irrigation setting of 20 m^3/s, although in reality the combined tunnel becomes the limiting factor with a 345 m^3/s maximum open channel flow. The inclined spillway shaft linking to the tunnel was said to be a textbook design, but no calculations or model tests were provided. The condition and presence of the shaft lining have not been confirmed (no access), the steel tunnel lining downstream of the gates has previously been repaired, the gates have vibration and aeration problems, and the tunnel outlet and plunge pool are sub-standard.

The reasons for this situation being tolerated only gradually became clear. First the maximum gate discharge is only 160 m^3/s as permitted by the gate manufacturer not 260 m^3/s as calculated for two fully opened gates. There is a continuous minimum discharge of 20 m^3/s compensation water for Osh city potable water. Fundamentally there is no benefit in filling the reservoir and even after rehabilitation a 1275 masl normal top water level will be applied. Flood routing then becomes much simplified with 45Mm3 extra flood storage, doubling that for the 1282-88 masl flood range. The gate vibrations occur at small openings and have now been measured and found to be within acceptable limits of the Soviet code (SNIP). The obstruction to the aeration shaft has been lifted and an aeration slot will be created, and minor welded insitu reinforcement of gates and linings added, rather than proposed major off-site gate modification. The tunnel outlet rock-support wall and plunge pool amplification at the inaccessible dam toe will be carried out, after diverting the compensation flow by a temporary adit linked to the tunnel.

DIAPHRAGM WALL CONSTRUCTION

TemelsuGIBB first proposed a no-action strategy based on maintaining a normal top water level of 1270 masl. The only example of diaphragm wall construction through a high dam core in a gorge of this magnitude appears to be Mud Mountain Dam in the USA, where problems due to arching of the core occurred (Reference 3). A separate Kyrgyzhyprovodhoz study had shown that the irrigation benefit of the extra 12m depth to 1282 masl (75Mm3 extra reservoir volume) was zero for the agricultural areas developed, due to unreliability of supply in most years. The Panel of Experts however did not accept no-action, since flood rises will temporarily affect the upper dam and long-term storage levels might not respect present agreements as the responsible personnel retire or change. The owner agreed that the opportunity to introduce a partial cut-off at limited expense from current funding should not be missed. The options of 60 or 70m depth were further explored, with consideration of later phases should lower core or right abutment grouting subsequently prove necessary. A key factor was that the Hybrid Project 3, with drillholes angled sidewards and downwards fom the confined and plugged ends of the lower galleries to achieve a nine-line curtain was simply impracticable. At this stage, after 20 years of consolidation of embankment gravel fill, the gallery ends could be simply joined by excavating across the partially grouted core – permitting sub-vertical groutholes and minimal overlap with the diaphragm wall. A diaphragm wall depth of 70m (down to 1220 masl) was eventually selected to give a generous 15m penetration into the lower core.

Contract documents were prepared based on the Engineer's scheme, but asking the Contractor to carry out final design to suit his capacity and plant for excavation and production, delivery and placing of plastic concrete mixes. The Iranian firm JTMA Co. was awarded the diaphragm wall construction contract early in 2003 at a sum close to the estimate. Competitors had priced the works at up to twice this sum, anticipating difficulties with the site location and inhospitability. The Contractor has reduced the typical panel length to 3.0m and volume to 210 m3, from the French based 8.8m and 616 m3. This gives a more manageable task in the congested area of the short dam crest with tunnel access only. The other advantage is that to achieve a nominal 1.0m depth key into rock, the hydrofraise has to cut a square panel base jutting into abutment rock as a triangle. Given the extremely high rock strength (estimated at 200 MN/m^2) the advantage of smaller triangles of the hard, siliceous, crystalline limestone to grind away is evident. The joint detail has also been simplified, cutting into the preceding panel and inserting a groutpipe rather than drilling separately.

Maintaining a one-metre wall thickness for the reduced wall depth assists the joint overlap being guaranteed against lateral deviation during excavation. The alignment of the wall has been moved a few metres downstream, to the back of the lower core, for fear of the hydrofraise being severely damaged by bumping into old steel grouthole casings left in the fill. Not surprisingly given the logistics of reaching Osh, of fabricating batching plant in Tehran and upgrading the hydrofraise excavation rigs in Italy, progress this year has been slow and limited to preparatory and auxillary works. It is now anticipated that the diaphragm wall construction will proceed in the spring of 2004. Contractual niceties come second to physical possibilities in remote locations and extreme winter weather.

CONCLUSIONS

The Final Dam Design Report was compiled to record this four-year study and issued in October 2002. While every effort was made to speed the process, the assumptions made to facilitate design packages were often proved unsatisfactory in the long term. The difficulty in obtaining and then verifying information and even drawings should not be underestimated. Piezometer layout sections were extremely misleading, and 10 years of hand plotted phreatic surfaces in full reservoir conditions simply erroneous. As for dinosaur research, one must first assemble an awful lot of fossil bones, and then ask the right questions in order to correctly interpret them functionally. In these days when safety inspections of 60 dams at a time are required, with two days input allotted for each and only limited access to and around the site, the inspector must explore the 'facts' and try to understand the mindset of the owner/operators. The painstaking investigation, monitoring and design review process applied at Papan dam were a rare opportunity to focus on the individuality and idiosyncrasies of a standard Soviet dam design applied to a narrow canyon. There were no short cuts, and only perseverance by all parties concerned will lead to satisfactory rehabilitation works. The very active role of the client, Project Implementation Unit for Kyrgyz Irrigation Rehabilitation (Ministry of Agriculture, Water Resources and Processing Industry) and invaluable assistance of Mr Fedotov (Kyrgyzhyprovodhoz) in pursuing these technical issues, providing information in difficult circumstances, and obtaining value for money solutions is acknowledged.

Figure 5. Papan dam – Reservoir and Intake Tower

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