IFAS AGENCY for the GEF PROJECT

ARAL SEA BASIN PROGRAM
WATER & ENVIRONMENTAL MANAGEMENT PROJECT

COMPONENT C:
DAM SAFETY AND RESERVOIR MANAGEMENT

KAYRAKKUM DAM

SAFETY ASSESSMENT REPORT

MARCH 2000

GIBB

LAWGIBB Group Member

In association with

SMEC

KAYRAKKUM DAM
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CONTENTS

UNITS AND ABBREVIATIONS

ASBP	Aral Sea Basin Program
CA	Central Asia
CMU	Component Management Unit
EA/EIA	Environmental Assessment/Environmental Impact Assessment
EC-IFAS	Executive Committee of IFAS
FSL	Full Storage Level
FSU	Former Soviet Union
FAO/CP	Food and Agriculture Organisation/World Bank Co-operative Programme
GDP	Gross Domestic Product
GEF	Global Environment Facility
ICB	International Competitive Bidding
ICOLD	International Commission on Large Dams
ICWC	Interstate Commission for Water Coordination
IDA	International Development Association of the World Bank
IFAS	International Fund to Save the Aral Sea
JSC	Joint Stock Company
LDL	Lowest Drawdown Level
M & E	Monitoring and Evaluation
NCB	National Competitive Bidding
NGO	Non-governmental Organisation
O & M	Operation and Maintenance
PIP	Project Implementation Plan
PIU	Project Implementation Unit
PMCU	Project Management and Coordination Unit
PMF	Probable Maximum Flood
RE	Resident Engineer
TA	Technical Assistance
TOR	Terms of Reference
SIC	Scientific Information Centre (of the ICWC)
SU	Soviet Union
SW	Small Works
VAT	Value Added Tax
WARMAP	Water Resource Management and Agricultural Production in CA Republics

masl	metres above sea level
Mm^3	million cubic metres
km^3	cubic kilometres = 1000 Mm^3
m^3/s	cubic metres per second
ha	hectare
hr	hour

1 INTRODUCTION

This report is one of ten reports prepared under Component C: Dam and Reservoir Management, of the Water and Environmental Management Project (WAEMP). The WAEMP is supported by a variety of donors, such as the Global Environment Facility (GEF) via the World Bank, the Dutch and Swedish Governments and the European Union, and is being implemented by the IFAS Agency for the GEF Project under the Aral Sea Basin Program.

1.1 Background to Project

In general, the WAEMP aims at addressing the root causes of overuse and degradation of the international waters of the Aral Sea Basin, and to start reducing water consumption, particularly in irrigation. The project also aims to pave the way for increased investment in the water sector by the public and private sectors as well as donors. The project addresses this aim in several components. Dam and Reservoir Management, the assignment with which this report is concerned, is one of them. The other components are: Water and Salt Management, the leading component, to prepare common policy, strategy and action programs; Public Awareness to educate the public to conserve water; Transboundary Water Monitoring to create the capacity to monitor transboundary water flows and quality; Wetlands Restoration to rehabilitate a wetland near the Amu Darya delta; and Project Management. The components have close links with each other.

The Dam and Reservoir Management Component focuses on four activities as follows:

a) Continuing an independent dam safety assessment in the region, improve dam safety, address sedimentation and prepare investment plans;
b) Upgrading of monitoring and warning systems at selected dam sites on a pilot basis;
c) Preparing detailed design studies for priority dam rehabilitation measures; and
d) Gathering priority data and preparation of a program for Lake Sarez.

The activities are grouped for work process purposes into two packages and will be executed simultaneously, according to an agreed schedule of works:

- Dam safety and reservoir management (including activities "a", "b" and "c");
- Lake Sarez safety assessment (covering activity "d").

The Dam Safety and Reservoir Management package covers the following areas: dam safety, natural obstructions, silting of reservoirs, control of river channels etc.

The activity covers the following 10 dams, two in each country:

Kazakhstan: Chardara and Bugun dams;
Kyrgyzstan: Uchkurgan and Toktogul dams;
Tajikistan: Kayrakkum and Nurek dams;
Turkmenistan: Kopetdag and Khauzkhan dams; and
Uzbekistan: Akhangaran and Chimkurgan dams.

Because of the need to safeguard human life, early priority is being given to safety reviews at each of the dams, which is the subject of this report.

1.2 Safety Assessment Procedures

The dam safety assessments are the first stage in the evaluation (including costing and economic justification), analysis, design and implementation of measures aimed at ensuring safe operation of the selected dams. They have been prepared based on a brief reconnaissance visit to each dam, discussions with the operating staff and a perusal of such information and data as was found to be readily available. No attempt has been made at this stage to analyse any of the data. A data collection and cataloguing procedure was initiated before commencement of the assignment but this process (to be carried out by National Teams) is still at an early stage in implementation.

The field visits were made and the reports prepared by a team of international experts specialising in dam engineering and dam safety procedures. The team comprises experts from GIBB Ltd (United Kingdom) and its associate for this assignment, Snowy Mountains Engineering Corporation (SMEC) from Australia, together with members of a team of regional experts who have been contracted as individuals to work with the Consultants for this project. This team is referred to here as the International Consultants (IC). The International Consultants have been supported during the field visits by members of National Teams appointed for this project from each of the five Central Asian republics.

The principal members of the international team, who are the authors of this report, are the following: -

- Jim Halcro-Johnston (GIBB Ltd) – Team Leader
- Gennady Sergeyevich Tsurikov (Uzbekistan) – deputy Team Leader
- Edward Jackson (GIBB Ltd) – Dam Engineering Specialist
- Ljiljana Spasic-Gril (GIBB Ltd) – Geotechnical Engineer/Dam Structures Specialist
- Pavel Kozarovski (SMEC) – Hydrologist/Hydraulic Engineer
- E.V. Gysyn – Dams Specialist (Kazakhstan)
- E.A . Arapov – Hydraulic Structures Specialist (Turkmenistan)
- G.T . Kasymova – Energy Expert (Kyrgyz Republic)
- R. Kayumov – Hydrostructures Specialist (Tajikistan)
- R.G. Vafin – Hydrologist, specialising in reservoir silting (Uzbekistan)
- V.N. Pulyavin – Dam Instrumentation Specialist (Uzbekistan)
- N.A.Buslov – Dam Specialist (Turkmenistan)
- Y.P. Mityulov – Cost and Procurement Expert (Uzbekistan)
- N. Dubonosov – Mechanical Equipment Expert (Kyrgyz Republic)

Most of the above team members have contributed in the preparation of this report.

1.3 Scope of Safety Assessment

The safety assessments are made based on superficial evidence observed during the site visits, discussions with operating staff and subsequent discussions with members of the National Teams and an examination of supporting design and construction documents as has been made available to the IC for review. (A full list of the documents reviewed is included as Appendix A)

The safety evaluation of the dam has required an assessment of the following factors:

(1) The **characteristics of the reservoir and dam site**, which includes the flood regime
for the river, and the geological conditions at the site;
(2) The **characteristics of the dam,** covering its design and present condition;
(3) The expected **standards of operation and maintenance** of the dams ,its performance, and the implications for safety;
(4) The **effects on the downstream** area resulting from a failure of the dam or an excessive release of water.

The structure of this report reflects the scope of safety assessment. Chapter 2 presents a general description of the dam, including location, purpose, principal dimensions and assessment of its hazard rating in relation to the impact that a safety incident would have on the adjacent community. Chapter 3 discusses the design factors that principally affect the safety of the dam.

Comments on the condition and performance of the dam are given in Chapter 4 and in Chapter 5 an assessment of its safety is given.

Chapter 6 gives recommendations for studies, works and supplies to be undertaken in the interests of ensuring the safety of the dam and the downstream community. Conclusions and recommendations are summarised in Chapter 7.

The recommendations for safety measures given in this report must be regarded as tentative as their precise scope will depend on the outcome of further studies which are outside the scope of the present assignment. No attempts has therefore been made at this stage to evaluate the cost of the required remedial works or to carry out an economic justification for the works proposed, which will be necessary to support an application for funding. This will be carried out when the necessary studies and detail designs have been completed.

2 PRINCIPAL FEATURES AND DIMENSIONS OF THE DAM

2.1 Location, Purpose, and date of Construction

Kayrakkum dam is situated in the central part of the Syrdarya river catchment basin 20 km from Hojent town of Leninabad oblast of the republic of Tajikistan. Access to the dam is available at any season by asphalt road which connects the dam with Hojent town (see Figure 1).

The reservoir is used for seasonal regulation of the river flow in order to provide water for irrigation, and also for power generation.

The dam was designed by SAO GIDROPROEKT Institute in Tashkent. The filling of the reservoir was started in 1956. The reservoir was commissioned in 1959.

2.2 Description of the Dam

The dam consist of (see Figure 2):

- non-overflow hydraulically filled dam
- hydropower station

The non-overflow hydraulically filled dam is made of sand with shells made of gravel and rockfill. The upstream slope has rip-rap facing. The downstream slope is protected by a sandy-gravel layer. An inclined drainage is constructed at the downstream toe of the channel transition part, and pipe drainage is at the flood plain part (see Figure 3).

Protective dykes, 27 km long, were constructed along left the bank of the reservoir as a flood protection together with a system of pumping stations which provide horizontal drainage.

The power plant is located at the left bank of the channel (Figures 4, 5). The power plant adjoins the embankment by a solid abutment. There are six units installed at the power plant with 26 300 kW capacity each. Each unit is arranged in a separate chamber under a spillway. The distance between the units is 17 m. The spillways with total width of 72 m are constructed above the generator chambers to pass 3,960 m^3/s design discharge at MWL = 48.35. 1,080 m^3/s discharge passes through the hydroelectric units. The spillway has two rows of gates - maintenance gates B x H = 12 x 24 m (6 nos.) and operating gates B x H =12 x10 m (6 nos.). All gates are operated by a gantry crane with 2 x 125 t load carrying capacity. There are tunnel type fixed-wheel gates installed at the power conduit, controlled by hydraulic drive, that have 2 x 110 t load carrying capacity.

The gantry crane and hydraulic drive have power feeding from their own power source. If necessary there is a stand-by feed from an outdoor switchyard 220/110 kV from other sources of supply.

The principal dimensions of the reservoir and the various components of the dam are given in Table 2.1.

2.3 Hazard Assessment

In many countries a formal classification system is used to define the risk a dam represents, in terms of the potential for loss of life and/or damage to property which could result in the event of flooding caused by failure of the dam or an extensive release of water. The magnitude of the risk depends partly on the characteristics of the dam and reservoir and partly on the conditions downstream of the dam. Risk factors based on the procedure set out in ICOLD Bulletin 72 (Reference 1) are shown in Tables B1 and B2 in Appendix B.

Based on the Tables in Appendix B, the total risk factor of 34 points (Table 2.2) puts the Kayrakkum dam in Risk Class IV, that is the highest risk category.

Table 2.2 Kayrakkum Dam – Risk Factor

		Points
Reservoir Capacity (Mm^3)	400	6
Dam Height (m)	35	4
Downstream Evacuation Requirements	>1000	12
Potential Damage Downstream	High	12
	TOTAL	34

Table 2.1 Kayrakkum Dam – Principal Dimensions

Main parameters of the reservoir

Total storage capacity (designed)	4,200 Mm^3
Active capacity (designed)	2,300 Mm^3
Dead Storage at LDL (designed)	1,900 Mm^3
Full Storage Level (FSL)	347.50masl
Maximum Water Level (MWL)	348.35masl
Dead storage level	342.5masl
Reservoir surface area at FSL	513 km^2

Main parameters of the dam

Crest Length	1202 m
Crest Level	351.5 m
Crest width	64m
Maximum height of the embankment	32 m
Upstream and Downstream Slopes a) above the berm b) lower of the berm	 1: 4 1: 4
Total maximal capacity of all the structures at FSL:	
at flood of 0,01% of exceedance probability	5040 m^3/sec
turbines	1080 m^3/sec
spillway	3960 m^3/sec

3 DESIGN CONSIDERATIONS

3.1 Hydrology

The Syrdarya river is formed by the confluence of the Naryn and Kara-Darya rivers. Kayrakkum dam was constructed at the site where the river flows out into plain territory. The upstream catchment area is 136,000 km^2. The long term average discharge of the river at the dam site is 520 m^3/s. The runoff of 50% reliability is 15,785 Mm^3. The runoff of the flood period (April - August) makes up to 60 % of the annual runoff.

The observed maximum discharge is 4,300 m^3/s - April 1969.
The design maximum discharge is for 0.01% = 5,570 m^3/s
for 0.1 % = 4,400 m^3/s
The summer months mean minimal discharge is 122 m^3/s,
The winter months mean minimal discharge is 130 m^3/s.

The actual mass runoff of suspended load for the period from 1956-1969 was 413 million tonnes, equivalent to 31.8 Mm^3/year. The observed maximum of suspended load is 50 kg/m^3.

Maximal discharges : at FSL - 5,040 m^3/s
at MWL – 5,600 m^3/s

The capacity of the downstream river channel is not more than 2,500 - 3,000 m^3/s

3.2 Geology and Seismicity

The Syrdarya river has different appearances of its form along the reservoir site. The right bank represents terraced sandy step adjoining to piedmonts, where there stand out three longitudinal terraces that represent different kind of sands, sandy loam, loam and clay. The left bank appears as rolling plain having a slope in the river direction and represents interlayers of sand, gravel and pebble.

The site of the main structures of the dam is formed by interlayers of clay, sand and soft sandstone. Bedrock forming the power plant foundation is present as brecciated clay and partly cemented sand. The left bank, apart from the power plant, is formed generally of sand.

The construction site seismic intensity is VIII.

3.3 Construction Materials and Properties

The dam was constructed by hydraulic filling using sand obtained from four borrow areas in the river bed and its banks close to the dam site. The material for rip-rap for the upstream slope facing was obtained from borrow areas close to Bekabad town.

Grading characteristics of the embankment materials are summarised in Table 3.1.

Table 3.1 – Kayrakkum Dam – Material Properties

size	gravel mm	Sand mm						
		coarse		medium	fine	Very fine	dust	
	> 2	**2-1**	**1- 0.5**	**0.5 - 0.25**	**0.25 - 0.1**	**0.1 - 0.05**	**0.05 - 0.01**	**<0.01**
%	14.12 ÷ 5.62	2.66 ÷ 1.19	23.83 ÷ 5.43	19.82 ÷ 7.32	52.70 ÷ 39.48	5.92 ÷ 1.56	5.54 ÷ 1.43	5.48 ÷ 3.48
	100	85.88 ÷ 94.38	83.72 ÷ 93.19	65.77 ÷ 86.22	45.92 ÷ 70.92	6.47 ÷ 16.88	4.91 ÷ 11.02	3.48 ÷ 5.48

It was anticipated that a sand density of 1.6 t/m^3 would be achieved during hydraulic filling of the embankment. To provide necessary stability of the embankment special shells of granular material were constructed at upstream and downstream toes.

3.4 Seepage Control Measures

A cut-off sheet pile wall up to 3m deep was constructed under the upstream of the power plant as a seepage control measure for drainage protection purposes.

3.5 Reservoir Draw-off Works

Prior the Soviet Union collapse the reservoir was mainly used for irrigation and partly for power generation. It was operated according to the irrigation schedule during the irrigation season from April up to September. The surplus runoff volume of the flood was saved in the reservoir to produce power in winter using its own power plant.

After the Soviet Union collapse the reservoir has been mainly used to produce power. When the flood inflow is more than 1,080 m^3/s (the capacity of the power conduits), water is discharged through the surface spillways. Combined turbines and surface spillway discharge capacity is 5,040 m^3/s. For 0.01% floods the floods routing is used which with an additional height of 0.85 cm, brings water level in the reservoir to 348.35 masl.

3.6 Performance Monitoring Instrumentation

To control performance of the embankment and the draw-off works the following monitoring is provided (Appendix C):

1. Embankment foundation settlements
2. Settlements of the embankment fill
3. Monitoring of the phreatic surface
4. Seepage
5. Embankment deformations

74 piezometers were installed in 11 monitoring profiles and deep and surface survey benchmarks were installed. The maximum seepage through the embankment was recorded in 1959 and it was 147l/s, that was 24 l/s less than the design discharge.

3.7 Hydropower Facilities

I. Turbines - 6 nos. type ПЛ-495-85-500

1.	Capacity	- 23.6 MW
2.	Design head	- 15 m
3.	Runner diameter	- 4.995 m
4.	Water discharge at design capacity -	- 180 m^3/s
5.	Speed	- 125 rpm

II. Generators - 6 nos. type ВГС –700/100-48

1.	Capacity	- 21 MW
2.	Power factor	- 0.8
3.	Voltage	- 10.5 kV
4	Speed	- 125 rpm

Installed capacity for six units	- 126 MW
Water consumed to generate 1 kWh	- 24 m^3

4 DAM CONDITION AND PERFORMANCE

4.1 Comments Arising out of Inspection

The IC, in company with representatives from the Tajic National Team and Engineers from the site visited the dam on 7 October 1999. Areas inspected included the whole of the embankment and the draw-off works.

The reservoir water level at the time was low.

After the inspection it was found that:

- The rip-rap of the upstream face of the embankment was damaged over a length of 150 m
- Visually and according to the personnel, the hydromechanical and electric equipment of the dam structures are generally in satisfactory condition
- However, electrical cables and communication system are in emergency condition.

4.2 Assessment of Performance Monitoring Results

The last cycle of geodetic observations for the embankment and power plant deformations was carried out in 1989.

Regular observations for phreatic surface condition are carried out by piezometers. According to the records of 18 August 1999, the internal water levels were measured on only 38 piezometers, that is 51% of the designed number. The measured water levels were plotted at cross sections of the embankment. The actual position of the phreatic surface was lower than the designed one.

The maximum seepage discharge varied from 82 up to 129 l/s for the period from 1958 up to 1999. The designed seepage discharge value is 171 l/s. On that basis it may be concluded that seepage is not a problem .

4.3 Dam Safety Incidents

Since the dam was commissioned there have been no emergency situations.

4.4 Maintenance Procedures and Standards

“Maintenance Manual for Kayrakkum dam and its structures” exists. The manual was prepared based on “ Maintenance Manual for power stations “issued by the Ministry of Energy of the USSR, Moscow 1976, and also “ Operation Manual of Kayrakkum Dam on Syrdarya “

4.5 Existing Early Warning & Emergency Procedures

There is no early warning system. There are public telephone communications and dispatcher service communication of the power plant that give opportunity to communicate with all users of the main canal, with rayon and oblast centres. All operating staff actions are carried out according to the dam manager’s orders.

5 SAFETY ASSESSMENT

5.1 General

The safety assessment is based on the following general criteria:

(1) Structural safety
The dam, along with its foundations and abutments, shall have adequate stability to withstand extreme loads as well as normal design loads.

(2) Safety against floods
The reservoir level shall not rise above the critical level (maximum flood level) for the largest possible flood. Gate mechanism and power units must remain fully operational and accessible at all times.

The dam should have adequate facility for rapid lowering of the reservoir level in case of emergency.

(3) Safety against earthquakes
The dam shall be capable of withstanding ground movements associated with the maximum design earthquake (MDE) without release of the reservoir water. The selection of the appropriate value of MDE is based on an assessment of the consequences of dam failure (Section 2.3).

(4) Surveillance
Arrangements for inspection, surveillance and performance monitoring of the dam should ensure that a danger arising from damage, defect in structural safety or an external threat to safety is recognized as soon as possible, so that all necessary measures can be taken to control the danger.

Adequate emergency planning, early warning and communications facilities shall be in place to ensure the safety of the downstream population in case of emergency.

In the light of the review of the design and performance of the Kayrakkum dam, the findings of the condition assessment, and the review of the hydrological and geological conditions, the following conclusions are drawn regarding the safety of the dam:

5.2 Structural Safety

5.2.1 Embankment

The dam is said to have been operated safely and superficially appears to be basically sound (when inspected at a low reservoir level).

There are, however, a number of defects that detract from the safety of the embankment, as follows:

1) Rip-rap upstream face protection.

The upstream face of the embankment is protected against damage by reservoir waves by means of a 1.1 m thick layer of rock (rip-rap). The design grading curve for the material was not seen but it is reported that the material is required to be graded between 300 – 800 mm size, giving a 50% size (D_{50}) of probably around 600 mm. It is laid on a coarse gravel bed.

Under the attack from waves, which are reported to be up to 2.5 m high, the rip-rap has been displaced and has slumped badly, to the extent that in some areas it no longer fulfils its purpose. 2000 m^3 of rock (obtained from 24 km distance) was replaced over a 150 m length in 1998 but considerable further repair is needed.

There appears to be a high risk of further damage being sustained by the embankment in the event of there being a long period of strong easterly winds (such as can be expected in the spring).

A check on the design of the rip-rap using standard formulae suggests that the basic design criteria are inadequate, viz:

- a wave height of 2.5 m is consistent with a reservoir 'fetch' of 50 km and a wind speed of 19 m/s (Reference 2);
- assuming that a small amount of damage is acceptable the ratio wave height / 50% stone size (H_s/D_{50}) for a slope of m = 4 should be between 2.2 and 2.5 (Reference 3).
- the rip-rap thickness should be around 2 x D_{50}.

These criteria indicate the rip-rap protection for the Kayrakkum dam should comprise a 2 m thick layer of graded rocks having a D_{50} size of about 1 m. With this grading of rip-rap it is also likely that the particle size grading of the present gravel underlay would be insufficient to prevent material washing out through the rip-rap under severe conditions.

It would appear that if the need for frequent repairs is to be avoided the design of the face protection should be substantially modified. Design procedures are well established and options include:

- replace present rip-rap and underlay with more appropriately graded material;
- use of other materials possibly more readily available nearby, e.g. open stone asphalt;
- repair and strengthen the existing surface using asphalt.

The wave protection should extend to the crest of the embankment (in accordance with the design drawings that were seen), but in practice it stops short of the crest. Should alternative wave protection works be considered which have a smoother surface and increased wave run-up, then there may be a necessity to add a wave deflector / parapet wall to prevent overtopping by waves.

2) Embankment Instrumentation

A large number of piezometers in the embankment downstream shoulders are no longer functioning. For proper monitoring of the whole length of the embankment the piezometer installation should be reinstated.

Crest settlements of about 30 mm in 5 years are reported to have occurred, but it is understood that embankment settlement measurements are no longer made, largely due to there being no suitable instruments available to make precise measurements, or suitably trained personnel.

3) Embankment

Apart from the damaged rip-rap referred to above the embankment appears to be in good condition superficially. The downstream drainage is reported to be effective, with no surface seepages. Holes made by burrowing animals are regularly filled in.

Sufficient piezometers remain in working order to obtain information on the internal water levels (phreatic surface) within the embankment, which indicate a water level generally near or slightly below the design line. The design phreatic surface is, however, quite high, and given that the site seismicity is also high (Intensity 8 on MSK scale) it would be prudent to study the embankment stability under the effect of earthquakes, and to assess to what extent the material is susceptible to loss of strength due to liquefaction and its effect on stability. An investigation by means of boreholes, in situ density tests and laboratory testing would be needed to provide material property parameters for such analyses.

Water is discharging from a number of large (approximately 300 mm dia.) drain pipes emerging from beneath the downstream toe which are reported to be left from the hydraulic filling operations, and should have been grouted. The water appears to be clear, however, and is said to have been flowing for many years. No details are available of the inlets to the pipes, said to be in the core zone, so it is not possible to assess whether they constitute a safety risk in the long term. It would be advisable to grout these pipes nevertheless.

5.2.2 Structures

Much of the structural concrete in the spillway and power station intakes is deteriorating. It would, however, require a more extensive inspection to ascertain whether this constitutes a safety risk or is merely superficial, and what repairs are needed.

5.3 Safety against Floods

Safety against floods depends wholly on the operation of the hydromechanical plant, in particular the gantry crane used to operate the spillway gates, for which a high degree of reliability is obviously essential.

5.3.1 Discussion on the exceedance probability of design hydrographs

The aim of this section is to discuss the conservatism involved during derivation of design hydrographs in accordance with SNIP and how do these hydrographs compare with PMF.

The Kayrakkum outlet structures were designed using 0.1% exceedance probability hydrograph and checked against 0.01% hydrograph. The design flood hydrograph is routed through the dedicated 0.8 km^3 flood storage that is located between 347.5 masl and 348.2 masl.

The design hydrographs are determined through a statistical analysis of historical records. A theoretical curve, based on a 3-parameter gamma distribution, is fitted to maximum annual peak discharge values, and design peak discharges for various exceedance probabilities are determined. The 0.01% discharge value is subject to a correction, which is approximately 20% higher than the original value. The correction itself brings the exceedance probability of the obtained value to approximately 0.005% or 1 in 20,000 years.

The volume of the hydrograph is also defined through a frequency analysis of the annual maxima series. The coincidence of all historical peaks and maximum flood volumes would result in the two variables (peak discharge and flood volume) to be totally dependent, with the exceedance probability of the combined hydrograph equal to the exceedance probability of the peak discharge value. However, the ranked historical peak discharge values do not necessarily coincide with the ranked maximum volumes. In other words these two variables are partially dependent, resulting in a hydrograph with exceedance probability lower than the exceedance probability of the peak discharge.

During the practical fitting of the theoretical frequency curve, a coefficient of asymmetry Cs is calculated from the recorded series of annual maxima. This coefficient is then used to fit an appropriate curve. Higher the coefficient, more skewed is the theoretical curve, resulting in higher discharge values for low probabilities of exceedance. This practice introduced an additional conservatism into the derivation of the design discharge values, which results in some overestimation of the design discharge value.

The above three factors result in the design discharge hydrograph with exceedance probability significantly lower than 0.01% (1 in 10,000 years). It is expected that the resulting exceedance probability of the design hydrograph would be in the range of 0.001% or 1 in 100,000 years. Further investigations into this matter are required to support this statement. If the results confirm the above statement it can be concluded that the conservatism introduced during the design calculations results in the outlet structures of the dams to have been designed for the 1 in 100,000 years events instead of 1 in 10,000 years events, which in general approaches the exceedance probability of a PMF event.

The Uzbekistan “Gidro-Met” (Bureau of Meteorology) provides forecasts of expected streamflows at the beginning of the wet season (early spring). The forecast is based on the snow deposits in the catchments of particular rivers. The Bureau of Meteorology of Uzbekistan is currently developing a methodology for estimation of snow extent and water equivalent using satellite images. Based on the forecast, the central authority, which regulates the dam operation, issues a request for the initial level in the reservoir prior to the beginning of the melting season. In the cases of extremely wet years the requested initial level can be lower than the FSL. This mechanism might introduce an additional storage available for flood routing, increasing the dam safety during extreme floods.

Kayrakkum Dam was designed and constructed prior to the construction of two large reservoirs located upstream of the dam (Toktogul and Andizan). As these two reservoirs have a significant flood storage, their impact on flood peak at Kayrakkum is beneficial, reducing the peak and to a certain degree the volume of a large flood.

5.3.2 Factors which reduce the dam safety during floods

There are several factors that affect the performance of the Kayrakum dam during large flood events. The following factors have been identified during the assessment:

- Estimates of extreme floods used for design of outlet structures are based on statistical analysis of historical records. Analysis of longer records following the dam construction resulted in a 0.01% exceedance probability peak discharge with correction to change from 5,570 m^3/s to 6,750 m^3/s. In order to make meaningful extrapolation of events with exceedance probability of 0.1% the extrapolation would have to be based on regionalized parameters with records longer than 100 years. As this is not the case, the extrapolation beyond 0.1% exceedance probability must be considered to be beyond the credible limit. In order to establish the exact relation between the 0.01% exceedance probability discharge hydrographs developed in accordance with SNIP and the extreme flood hydrographs based on PMF estimates, a PMF study must be undertaken for this site.

- The release of water during extreme flood events is envisaged by the designers to be through 6 turbines (Qturb = 1,080 m^3/s) and through spillways located above the turbines and generators (Qs = 3,960 m^3/s). The release through turbines is based on an assumption that all turbines are operational, the power lines are capable to transfer the generated energy and that the demand centres are able to consume the generated power during the extreme flood event. In order to assess the safety of the dam during an extreme flood, it is reasonable to assume that the turbines will not be operational due to one of the factors mentioned above. In this case the maximum outlet capacity is thus 3,890 m^3/s, assuming that all spillway gates are functional at the period of an extreme flood.

- The 0.01% exceedance probability hydrograph at Kayrakkum has been developed for natural conditions without any water withdrawals for irrigation. The maximum off-take capacity upstream of the dam is 1,000 m^3/s. The seasonal variation has been taken as 20% in April, 50% in May, 80% in June, 100% in July, 80% in August and 20% in September. It is unreasonable to assume that irrigation demand would approach the maximum demand during the extreme flood event. It is also quite likely that the various intake structures could be blocked by sediment and debris and that some of the canals could be breached. The inflow hydrograph was therefore reduced by deducting 0%, 25%, 50% and 75% of the maximum off-take capacity. The resulting reservoir water levels for different capacities of the dam outlet structures are given in Table 5.1 below, with the number of days above the maximum reservoir water level shown in brackets.

Table 5.1 - Kayrakkum maximum water levels (masl)

	Irrigation Demand as a percentage of the maximum demand			
Scenario Description	**0%**	**25%**	**50%**	**75%**
Qspill+Qturbines	348.7 (4)	348.6 (3)	348.4 (1)	348.25 (0)
Qs	349.65 (12)	349.5 (9)	349.3 (8)	349.15 (6)
5/6*Qs (5 out of 6 gates are opened)	350.6 (44)	350.4 (28)	350.2 (16)	349.9 (13)

Note: values in brackets represent the number of days when the water level in the reservoir exceeded the maximum reservoir water level.

It can be seen from the table that the maximum reservoir levels are always below the crest of the dam (351.5 masl), with the lowest freeboard of 0.9 m; the design freeboard is 3.2 m. This implies that the dam is safe during a design 0.01% event only if all turbines and gates are opened and the irrigation withdrawals are at least 75% of the maximum capacity. In all other cases there is a possibility for the dam crest to be overtopped by waves. It must be stated that the above analyses were conducted without taking into account the impact of the upstream dams, however, due to the large volume of the incoming flood the impact of the upstream dams is expected to be small.

5.3.3 Conclusions and recommendations

It can be concluded in general that the adopted design procedure in accordance with SNIP provides a relatively conservative estimate of large floods. The exceedance probability of the design flood is lower than 0.01% and is expected to approach 0.001% or 1 in 100,000 years. Kayrakkum dam was constructed prior to the design of Toktogul and Andizan dams, so the attenuating effect of these dams increases marginally the Kayrakkum dam safety during extreme floods.

The assumption that the turbines will be operational during an extreme flood event is over-optimistic, so during a 0.01% flood event with turbines closed, the water level would still remain approximately 2.0 m below the crest level. The most critical scenario with significant impact on dam safety is when one of the spillway gates is not operational.

It is recommended that:

- PMF study be conducted, taking into account the combined effect of an extreme snow (glacier) melt, an extreme rainfall (PMP) and the attenuating effect of the upstream reservoirs.
- Analysis of irrigation demand and the capability of the off-take structures and canals be undertaken to identify the most likely water withdrawals during extreme flood events. The PMF hydrograph should be accordingly reduced.
- The obtained PMF hydrograph be routed through the storage using the spillway only, commencing at FSL. The maximum reservoir water levels be identified and the dam stability for that level be assessed. An analysis of the reservoir behaviour if one of the six gates is blocked must be undertaken including the maximum wind wave height. A new parapet wall along the dam crest might be an acceptable solution, if other factors such as stability and filtration are acceptable.

5.4 Provision for Emergency Draw-down

Draw-down of the reservoir in case of emergency could be achieved by means of the spillway gates, and if practical by the turbines, though large releases (in excess of about 2,500 m^3/s) are said to cause flooding downstream. The total discharge capacity of the six 12 m wide gates is about 4,000 m^3/s, giving an initial draw-down rate from full storage level of about 0.7 m/day.

Should such an emergency release of water be approved, however, the risk to the downstream population could be substantially mitigated if an effective emergency plan could be put into operation rapidly.

5.5 Safety against Earthquakes

5.5.1 Seismic design criteria

In the original design seismic input parameters and stability analysis in seismic condition are assumed to have been carried out in accordance with procedure given in the Russian Seismic Standards (Reference 4). According to the Russian Seismic Standard, a seismic design coefficient (k_g) is derived for a site based on MSK earthquake intensity scale. The coefficients are derived based on 1:500 year earthquake. The required minimum factor of safety in seismic condition is always greater than unity.

However, the current practice based on the guidelines given in ICOLD Bulletin 72 (Reference 1) is to assess dam safety against two representative design earthquakes that are as follows:

OBE - Operating Basis Earthquake
MDE - Maximum Design Earthquake

Where:

- OBE, or “no damage earthquake” is the earthquake which is liable to occur on average not more than once during the expected life of the structure (of not less than 100 years). During an OBE, the dam and its ancillary works should remain functional but may need repair. The required minimum factor of safety for the OBE earthquake should be greater than unity.

- MDE or “no failure earthquake” is the earthquake that will produce the most severe level of ground motion under which the safety of the dam against catastrophic failure should be ensured. For dams which are classified to be Risk Class IV a recommended return period of MDE is 30,000 years (Reference 5). For this earthquake displacements of the crest are assessed and compared with the allowable wave freeboard

Although the seismicity of the Kayrakkum site is quite high (Intensity Zone 8 on MSK Scale) the dam safety has not been assessed for OBE and MDE earthquakes and it is recommended to carry out additional engineering studies (see Section 6.2.4) to evaluate its performance in those conditions.

As part of safety assessment a check should also be carried out to evaluate the height of seismic waves (seismic seiche) on the reservoir which could occur during a seismic event and which requires additional height to be added to the standard “static” freeboard.

5.5.2 Liquefaction of fill and foundation materials

It is well known that low density saturated sands and silts in a hydraulic fill embankment are highly susceptible to loss of strength due to seismic shaking. Kayrakkum Dam is no exception and the risk that the material in the dam and its foundations might liquefy during a severe seismic event is high bearing in mind the type of the dam, and the type of fill material used.

It is therefore recommended to carry out further in-situ testing to verify the properties of the embankment and foundation materials in order to assess soil strength reduction and displacements that could occur during strong earthquakes.

5.6 Other Safety Matters

A number of other matters will need further examination as part of a more comprehensive safety assessment than has been possible during the present study, for instance:

5.6.1 Security of access

The dam can be accessed from both sides of the river and the chances that extreme events (e.g. floods, earthquake) would completely sever both are remote, unless the roads are cut due to washouts, collapsed culverts etc.

5.6.2 Security of electricity supply

It is unlikely that 100% security of electricity supply can be assured in all circumstances, and a standby generator to operate the crane gantry in emergency is recommended.

5.7 Safety Assessment – Summary

5.7.1 Principal matters of concern

The IC see the following as being the principal matters of concern as regards the safety of the Kayrakkum dam.

(1) The embankment is at risk from damage by reservoir waves due to the inadequate upstream slope protection.
(2) In common with other hydraulic fill dams there is a high risk of liquefaction and loss of strength of the fill material under the effect of severe seismic shaking. The risk of large deformations occurring under seismic loading is enhanced by the rather high internal water levels in the embankment.
(3) There are deficiencies in the embankment performance monitoring system.

5.7.2 Safety Statement

From an examination of the data made available, and discussions with the engineers responsible for the dam, the IC concludes that the Kayrakkum dam cannot be regarded as complying with all normal safety standards, and is faced with significant dangers, that is:

(1) **Structural Danger** from reservoir waves, due to inadequate protection of the upstream face.

(2) **Damage from earthquakes**, which could cause liquefaction of the saturated fine sand in the embankment and foundations, leading to large deformations or partial collapse.

6 RECOMMENDED STUDIES, WORKS AND SUPPLIES

6.1 General

The review of the design of the dam together with information obtained during the site inspections, and discussions with the site manager has enabled the IC to arrive at certain conclusions regarding the safety of the dam, which are discussed in Section 5. These conclusions, along with considerations of requirements for emergency management have provided the basis for an assessment of the need for additional studies, investigations, construction works and supplies necessary to bring the dam to an acceptable and sustainable standard of safety. However, it must be recognized that the need for further work might still become evident as an outcome of this work, as the preliminary conclusions are refined.

A more detailed specification and methodology for the work described in this Section is presented in the report `Methodology for Design of Priority Rehabilitation Measures'.

6.2 Additional Surveys, Investigations and Inspections

6.2.1 General

To provide the basic data for designing the works described below and for refining the conclusions of the safety assessment, additional information is required which is outside the scope of the present study. This work is described under the following headings:

- surveys
- ground investigations and inspections
- engineering studies

In addition, it is recommended that a dossier of 'as constructed' record drawings and other essential information relating to the design, construction and performance of the dam be assembled and regularly updated. Where original drawings have deteriorated they should be retraced or preferably re-drawn using a computer system. The dossier would comprise the basic source of information to be referred to when carrying out inspections or undertaking modifications in the future.

6.2.2 Surveys

(1) Topographic Surveys

To provide essential data for the dossier basic information the following ground surveys are recommended:-

Main embankment

- longitudinal crest profile, on crest road;
- typical cross sections of the embankment, (upstream and downstream faces);
- locations of piezometers and drainage works.

(2) Reservoir bed survey

The IC was given some information on the present extent of siltation in the reservoir. It is recommended that further measurements are made to validate the present estimates.

6.2.3 Ground Investigations and Inspections

The following investigations and surveys are recommended:

1) Investigations at main embankment

- drilling with in situ permeability tests and static sounding tests (or geophysical density profiling), sampling and laboratory testing to obtain data for liquefaction analysis;
- investigation of possible borrow areas for material for rip-rap and underlayer(s);
- investigate by trial pits the condition, grading and thickness of existing rip-rap slope protection and underlayer.

2) Inspections

To provide information on which to base a detailed assessment of required repairs and equipment, it is recommended that a detailed inspection should be carried out and an inventory of defects, materials and repairs required prepared, covering:

- Area over which improved upstream face protection is required;
- improvements to embankment drainage (inspect for seepages when reservoir is at high level);
- repairs to embankment downstream face protection and surface water drainage works;
- concrete works;
- electrical wiring etc., and lighting;
- gates and operating equipment;
- steelwork (e.g. stairways and landings);

6.2.4 Engineering Studies

The following engineering/hydrological studies are recommended:

1) Study options for repair/renewal of upstream wave protection;

2) Review the estimates of extreme flood inflows to the reservoir, taking into account effect of possible actions (intentional or unauthorised) at upstream dams, and sedimentation;

3) Review reservoir management procedures, giving first priority to ensuring the safety of the dam.

4) Assess susceptibility of embankment materials to liquefaction and loss of strength, and carry out static and seismic stability analyses.

6.3 Construction Works

A preliminary assessment of the required construction works is made on the basis of the safety assessment and available data. Final details will depend on the outcome of the studies described above.

1) Embankment

- repair / renew upstream face protection

Although the embankment appears to be generally in good condition it is essential that its performance is properly monitored. The performance monitoring installation should be reinstated where necessary. The following is proposed:

- install new standpipe piezometers where the existing tubes are blocked;
- install surface settlement measurement markers and fixed beacons, for precise measurement of vertical displacements;
- Install seepage flow measurement devices.

2) Structural works

- Carry out other structural repairs as found to be necessary.

3) Hydromechanical Equipment

The safety of the dam relies heavily on the proper operation of the hydromechanical equipment. All necessary repairs, electrical wiring renewals, etc., should be undertaken immediately, and adequate standby generating plant provided.

4) Miscellaneous

Other defects discovered during the detailed inspection should be rectified.

6.4 Equipment and Supplies

A preliminary assessment of equipment and supplies required for the rehabilitation of the dam is as follows:

(1) Embankment instrumentation comprising:

- Inclinometers, with measuring probe
- Piezometers
- Surface movement markers

(2) Standby generator

(3) Early warning and communications equipment

6.5 Emergency Planning Studies

The Kayrakkum dam is an important dam impounding a very large reservoir, and a failure could have catastrophic consequences. To be able to respond to an emergency situation a comprehensive emergency plan supported by an efficient organization, communications and alarm system, is therefore essential. Inundation and flood hazard maps showing dambreak wave arrival time and duration of inundation should be prepared, based on dambreak modelling and simulation of dambreak wave propagation in the downstream areas. Flood damage estimates and potential loss of life should be developed on the basis of the above results.

A precise emergency plan instruction document should be prepared as soon as possible, giving detailed instruction to the dam site manager, regional engineers and civil authorities.

6.6 Safety Measures-Priorities

The safety measures identified above are listed in Table 6.1 and assigned to one of three priority levels (I, II or III).

The proposed Priority levels are:

I - high priority; work to be carried out immediately
II - intermediate priority; work to be completed within next three years
III - low priority; the need to be kept under review.

Table 6.1 Kayrakkum Dam - Dam Safety
Priorities for Studies, Works and Supplies

Item	Studies etc	Construction Works and Supplies		
		Priority I	Priority II	Priority III
1. Surveys (6.2.2)	☐			
2. Investigations and Inspections (6.2.3)	☐			
3. Engineering Studies (6.2.4)	☐			
4. Construction Works (6.3)				
• Upstream face		☐		
• Instrumentation		☐		
• Hydromechanical equipment				☐
• Miscellaneous Repairs				☐
5. Supplies (6.4)				
• Piezometers and deformation monitoring equipment		☐		
• Standby Generator		☐		
• Early warning and communications equipment		☐		
6. Emergency Planning Studies (6.5)	☐			

7 CONCLUSIONS

The IC conclude that on the basis of the information received and a brief inspection the Kayrakkum dam is in an unsatisfactory state and does not comply with required safety standards.

High priority should be given to the following activities;

(a) Repair of upstream slope wave protection;

(b) reinstatement of piezometers and installation of a comprehensive deformation monitoring system, and thereafter regular monitoring or pore pressures, deformations and seepages;

(c) ground investigation

(d) review of flood management procedure

(e) establishment of a reliable early warning system for the downstream population in the event of an emergency, supported by an efficient organization and communications system.

(f) Provision of reliable standby generation facilities.

(g) Carry out assessment of seismic stability of the embankment

REFERENCES

1. ICOLD Bulletin 72,1989

2. 'Floods and reservoir safety' Institution of Civil Engineers, UK1996

3. 'Design of rip-rap slopes protection against wind waves' UK Construction Industry Research and Information Association, Report No 61, December 1976.

4. SNIP II-7-81, Russian standard for Seismic Design

5. 'An Engineering Guide to Seismic Risk to Dams in the United Kingdom', Building Research Establishment (BRE) UK,1991

APPENDIX A

KAYRAKKUM DAM

LIST OF DATA EXAMINED

Kayrakkum Dam

Appendix A – List of Data Examined

1. World Bank June Mission, 1997.

APPENDIX B

HAZARD ASSESSMENT PROCEDURE

APPENDIX B – HAZARD ASSESSMENT PROCEDURE

Table B1 Classification Factors

	Classification Factor			
Capacity ($10^6 m^3$)	>120 (6)	120-1 (4)	1-0.1 (2)	<0.1 (0)
Height (m)	>45 (6)	45-30 (4)	30-15 (2)	<15 (0)
Evacuation requirements (No of persons)	>1000 (12)	1000-100 (8)	100-1 (4)	None (0)
Potential downstream Damage	High (12)	Moderate (8)	Low (4)	None (0)

Table B2 Dam Category

Total Classification factor	Dam Category
(0-6)	I
(7-18)	II
(19-30)	III
(31-36)	IV

APPENDIX C

KAYRAKKUM DAM INSTRUMENTATION

REPORT BY SPECIALIST MR V N PULYAVIN

October 1999

Inspection of instrumentation condition and dam structures observations
KAYRAKUM DAM

The process of the Kayrakkum dam safety control includes regular, monthly observations for the dam seepage regime. Last cycle of geodetic observations for deformations of a dam and power station house was executed in 1989.

Installation of piezometers in 11 monitoring profiles at the dam and 74 piezometers at power station was envisaged by the design to observe water levels. Accordingly to the readings of 18 August 1999 measurements weres only carried out on 38 piezometers , that is 51 % of the design number. At two monitoring profiles only two piezometers were operational in each profile (33 %), and 3 piezometers are operational in each of the four profiles (50 %). The operational staff of the reservoir plots the actual, measured phreatic surfaces regularly once a month. The actual phreatic surfaces are below than maximum designed one

A rectangular weir was provided downstream of the embankment for measurement of seepages. Besides, the seepage water that filters through the embankment springs at two other places, at the toe drain - through a 200mm diameter pipe and from an other spring. In both cases the discharges are not measured. Seepage water flows into the basin of a pumping station and is then pumped out back into the river. Total seepage is estimated from the volume of the pumped water. It is necessary to recognize that these estimates are insufficiently reliable: It is therefore necessary to equip all the existing outlets of seepage by measuring weirs. The maximal seepage for the period from 1958 up to 1999 varied from 82 l /s to up to 129 l /s. The designed seepage discharge is 171 l /s.

Last geodetic survey for monitoring of settlement of the dam and the power station was carried out 10 years ago. Over the same period an extensive scour of the upstream slope has occurred accompanied with the settlements.

Based on the information obtained at the site, both geodetic survey and seepage monitoring are found to be unsatisfactory as they not allow to carry out reliable dam safety control.

Recommendations:

1. Installation of measuring weirs at all seepage outlets
2. Replace piezometers that are out of order
3. Representatives of maintenance organization (Customer) must supervise piezometer filter installations. It is recommended to use modern synthetic materials (geofabrics) for the filters.
4. It is necessary to install additional survey marks in the zone of embankment extensive deformation and to carry out settlement measurements with a frequency not less than once in three months. Settlement monitoring for the other parts of the embankment should be carried out at least once a year.
5. Involve specialists from design or research institutes for analyse the results of seepage monitoring.
6. Define safe trigger values for the monitored parameters (phreatic surface, seepage discharge and deformation of the dam and power house).

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DRAWINGS