IFAS AGENCY for the GEF PROJECT

ARAL SEA BASIN PROGRAM
WATER & ENVIRONMENTAL MANAGEMENT PROJECT

COMPONENT C:
DAM SAFETY AND RESERVOIR MANAGEMENT

CHARDARA DAM

SAFETY ASSESSMENT REPORT

MARCH 2000

GIBB
LAWGIBB Group Member

In association with

SMEC

CHARDARA DAM
SAFETY ASSESSMENT REPORT

CONTENTS

DRAWINGS

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

Chardara dam is situated in South-Kazakh region of Kazakh Republic in the end part of middle stream of Syrdariya river to the north of Turkestan mountains, covers the part of Golodniy steppe, Arnasay depression and Syrdariya valley (see Figure 1).

The reservoir is impounded by two dams, one on Syrdariya river at Chardara town and the other partitioning Arnasay depression. Arnasay dam is situated on the border of Kazakhstan and Uzbekistan (see Figure 2).

Access to both of dams is possible at any season. There is an asphalt road Chimkent – Chardara – Arnasay from the north side of water reservoir. From the south it is – Tashkent – Djetysay – Arnasay.

The purpose of the water reservoir is:

- Distribution of winter runoff of Syrdariya river for summer time needs for irrigation of the area about 370 thousand hectares
- Prevent dangerous summer and winter floods that cause flooding of populated areas, irrigated areas and the railway line in Syrdariya valley
- Generation of electric power

The dam was designed in 1955-1967 by Central Asia department of “Hydroproject” Institute in Tashkent.

Construction works were completed in October 1967, and in 1968 full reservoir level was impounded.

2.2 Description of the Dam

The main components of Chardara water resevoir are :

- Chardara dam (see Figure 3)
- Arnasay dam (see Figure 4)
- Flood protecting structures

Chardara dam consists of a hydraulic fill embankment, channel type power station combined in one building with two sluices at the left and right hand side of the power station, and Kyzylkum regulator (see Figure 5) on the left bank of the river.

The dam was constructed by placing the hydraulic fill from two sides. The upstream slope of the embankment is strengthened by reinforced concrete slabs which were placed on a gravel-sandy bed. At the joints of the concrete facing a double-layer of inverted filter was placed. The downstream slope is strengthened by local silty-gravel material. A pipe drain with a triple-layered inverted filter was constructed at the toe of the downstream slope. There are relief wells and a drainage water conduit at the downstream toe. There is a 6m wide asphalt road at the dam crest.

The power station intake works (see Figures 6 and 7) include a clay blanket, reinforced concrete stilling basin with a berm and upstream and downstream retaining walls. The sluices and turbine conduits are equipped with maintenance and service gates. Sizes of the gates are: sluices - 6 x 5 m; turbine conduit - 5 x 5 m. Lifting of the maintenance gates is carried out by gantry crane (capacity 2 x 25 t), and the service gates by hydraulic hoist (capacity 50 t).

The Kyzylkum canal head works consist of an intake structure and three conduits 4.5m x 3.5m placed underneath the embankment. The intake tower is in the reservoir in the upstream shoulder of the dam. The conduits consist of five sections, each 20 m long. Maintenance and service roller gates are housed in the tower. Lifting of service gates is carried out by an electrical hoist (capacity 30 t). Lifting of the maintenance gates is realised by a gantry crane (capacity 30 t).

Arnasay dam consists of an embankment and the draw-off works. The embankment was constructed of compacted silty sands available locally. The upstream slope of the embankment is strengthened by reinforced concrete slabs which were placed on a sandy gravel bed. At the joints of the concrete facing a triple-layer of inverted filter was placed. The upstream slope ends up with a 0.8 m high parapet. A road, railway line and communication cables are accommodated on the dam crest. The main power supply Chardara - Djetysay of 110 kW passes some 24 m from the embankment axis.

Arnasay spillway, placed within the embankment, discharges surplus flood water. The spillway structure is of an open type and consists of a spillway section and stilling basin. The spillway section is a 4-bay weir. Each bay is10m wide and is equipped by 10 m x 8 m maintenance roller gates and 10 m x 6 m service roller gates. All gates are operated by a gantry crane (capacity 2 x 125 t).

Flood protecting structures were destined for protection of north-western part of Golodniny steppe from floods and Chardara reservoir water. The flood protecting structures comprise dykes, a system of 39 vertical drainage holes (depth 50-60 m), a network of collector drains and pump stations.

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 32 points (Table 2.2) puts the Chardara dam in Risk Class IV, that is the highest risk category.

Table 2.2 Chardara Dam – Risk Factor

		Points
Reservoir Capacity (Mm^3)	5700	6
Dam Height (m)	28.5	2
Downstream Evacuation Requirements	>1000	12
Potential Damage Downstream	High	12
	TOTAL	32

Table 2.1 Chardara Dam – Principal Dimensions

Principal Dimensions of the Water Reservoir

Total storage capacity	Design	5700 Mm^3
	1977 Survey	5197 Mm^3
Active storage capacity	Design	4700 Mm^3
	1977 Survey	4230 Mm^3
Dead storage capacity	Design	1000 Mm^3
	1977 Survey	967 Mm^3
Full storage level	(FSL)	252 masl.
Maximum water level	(MWL)	253 masl.
Dead storage level	(DSL)	244 masl.
Surface level at FSL	Design	900 km^2
	1977 Survey	783,4 km^2

Principal Dimensions of Chardara Embankment

Crest length		5300 m
Crest level		254.5 masl
Parapet level		255.5 masl
Height of Embankment		28.5 m
Crest width		12.6 m
Upstream slope and downstream slope :	Upper berm	1:4
	Lower berm	1:4.5

Principal Dimensions of Arnasay Dam

Crest length	2020m
Crest level	254masl
Height of Embankment	10,4 m
Crest width	16,5 m
Upstream slope and downstream slope :	1:3
Thickness of upstream slope protection	0.25m

Principal Dimensions of protective dykes

Dyke length	18.5 km
Crest width	4.0m
Height of dyke	5.5 m
Upstream slope and downstream slope:	1:3

Table 2.1 (continued)

Maximum capacity of all structures at 0,01% flood

Chardara Spillway	1282 m^3/sec
Power station	518 m^3/sec
Kyzylkum regulator	200 m^3/sec
Arnasay spillway	2160 m^3/sec

3 DESIGN CONSIDERATIONS

3.1 Hydrology

Syrdaria river is snow fed and is formed from Naryn and Kardaria rivers. The catchment area of the reservoir is 174,000 km^2. Long term annual run-off with 50% reliability is 37.2 km^3, during the flood event - 12,3 km^3. The maximum discharge with 0.01% of exceedance probability is 5,400 m^3/s.

3.2 Geology and Seismicity

The flood plain river part, where the embankment axis is, consists of a silty sandy layer 1.5 m – 2.5 m thick underlain by a layer of fine sands 12m - 17 m thick. The intake structure is founded on bedrock comprising siltstones, marly clay, sand and sandstone with conglomerate. Arnasay embankment foundation consists of silty sand layer 8 m- 10 m thick underlain by a clay layer of 2m - 5 m thickness or a thick layer of fine silty sand.

The ground water table in both areas is at a depth of 0.5m - 2 m from the surface. This water has sulphate aggression to the concrete.

The dam is located in earthquake intensity zone VI. However the embankment and its structures were designed to intensity VII.

3.3 Construction Materials and Properties

The physical and mechanical properties of construction materials and the material in the embankment foundations that were adopted in the design are given in Table 3.1

Table 3.1

Material	Dry density t/m^3	Strength parameters		Notes
		tan φ	kg/cm^2	
Foundations				
Clay	1.5	0.51	0.15	
Silt	1.5	0.51	0.03	
Sand	1.6	0.56	0.158	
Construction material				Supporting mass parameters were determined in 1991y.
River Bed	1.39	0.547	0.122	
Flood-plain area	1.39	0.544	0.124	
Lacustrine area	1.50	0.536	0.117	

Liquefaction of water saturated soils occurs as a result of hydrodynamic processes during seismic acceleration. This type of seismic deformation has been observed in fine cohesionless materials and, depending on earthquake intensity, may cause a partial or full loss of stability of all structures.

The granulometric composition given in Table 3.2 below was obtained from soil samples taken from borrow areas for different parts of the dam.

Table 3.2

Particle size	Flood plain area %	Lacustrine area %
0.1-0.2 mm	45	71
>0.2	55	29
Uniformity	Poor graded, uniform soil	Poor graded, uniform soil
Density	Low	Low

This granulometric composition points out a high potential of liquefaction caused by an earthquake.

3.4 Seepage Control Measures

A sheet pile cut-off wall was constructed underneath the upstream part of the power station down to the bedrock as a seepage control measure. No other seepage control measures were carried out at the dam.

3.5 Reservoir Draw-off Works

Filling and drawdown of the reservoir is carried out in accordance with the Operational schedule and takes into account periods of different water supply, irrigation requirements and requirements of minimum downstream river discharge into Aral Sea (3 km^3 per year).

A rate of filling and drawdown should not exceed 7 cm/day, with a maximum of 10 cm/day.

In order to reduce winter flood damage in downstream parts of Syrdariya river, downstream of Chardara dam, a spilling from the reservoir is normally allowed at 400 m^3/s, and in the absence of reliable forecast at 300 - 350 m^3/s.

The following is the reservoir operational schedule during the maximum floods:

-When the inflow of water exceeds demands of water users and the reservoir level exceeds FSL, water discharge is carried at up to 1500 m^3/s.
-If there is an excess inflow it is spilled in Arnasay until achievement of its full discharge capacity – 2160 m^3/s.
-If there is a further inflow into the reservoir, the excess is accumulated in the reservoir until level 252.7 masl is reached when discharge is carried out at up to 1800 m^3/s.

Change of the reservoir operating regime, especially in case of an eminent threat to safety and structures, is only possible under orders of the responsible personnel in charge in the dam with notification of higher organisation and local administration (Water Resource Committee).

3.6 Performance Monitoring Instrumentation

The following instruments are used for performance monitoring of Chardara dam (see AppendixC).

Power station:	Bench marks for horizontal deformations - 3 nos Foundation Bench marks - 3 nos Surface concrete bench marks - 31 nos Extensometers - 14 nos
Kyzylkum regulator	Surface concrete bench marks - 4 nos.
Chardara embankment	Deep bench marks -33 nos, Surface concrete bench marks - 27 nos, Piezometers - 40 nos, Seepage measuring weirs - 6 nos Extensometers – 14 nos
Arnasay embankment	Foundation bench marks - 6 nos, Deep bench marks - 6 nos Surface concrete bench marks –6nos Piezometers - 9 nos
Arnasay outlet	Surface concrete bench marks - 4 nos Extensometers - 15 nos

3.7 Hydropower Facilities

The power equipment of the power station is represented by four vertical hydroelectric generators with the following parameters:

Turbine type	PL –661 –B6-500
Shaft capacity	2600 kW
Speed of rotation	115.7 r/min
Design head	15.8m
Generator	OB4 790 /106-52

Capacity 31,250kw, 25000kvt

The power station works in the irrigation regime. An average electricity generation is 337 Million kWh.

4 DAM CONDITION AND PERFORMANCE

4.1 Comments Arising out of Inspection

The IC, in company with representatives from the Kazak National Team and Engineers from the site visited the dam on 30 September 1999. Areas inspected included the whole of the main embankment, the power station, the head works for the Kyzil Kum canal and the Arnasai embankment and its draw-off works.

The reservoir level at the time of the inspection was lower dead storage level, approximately at 242 -243 masl.

During the inspection it was find out that:

1. The condition of the downstream slope from Ch 30 to Ch 48+75 is not satisfactory. Some soil sliding was noticed in front of the catchment conduit.
2. Settlement of the crest of 3 cm to 5cm and 3m in diameter was noticed at the location of Kyzylkum canal conduits.
3. The joints of concrete facing of upstream slopes are found to be open, the timber joint filler separating one slab from another has rotted through. There are voids at the joints between the ground and concrete slabs
4. There is erosion on the downstream slope, caused by precipitation.

According to the site personnel the following was found during inspections in 1992 and 1998,:

1. The pipe toe drain does not function.
2. Drainage wells do not function.
3. The phreatic surface is some 2m to 3 m above the toe drain (see measurements form 1996 at sections 3.3 and 4).
4. Subsidence area on the dam crest is backfilled by sand every 1 - 2 years, and joints between pipe sections are repaired annually by wooden wedges.
5. Underwater inspection of the draw-off works in 1997 showed that there was damage to the sills, grooves, walls, weir and denudation of reinforcement steel. Underwater repairs in 1998 did not give any results.
6. At a spillway discharge of 1,000 m^3/s there is strong vibration of the gate shafts and powerhouse, which is why the operating department does not recommend to increase the discharge more than 1,000 m^3/s.
7. Some of the piezometers are jammed with stones and silt.

4.2 Assessment of Performance Monitoring Results

The dam Operating Department keeps data related to dam repair works previously undertaken. They also hold the results on variation of the phreatic surface with the reservoir water levels, settlements of the embankment and structures as well as the displacements of structures. However the IC was not able to obtain copies of this material. It is believed that the National Team of GEF Agency will collect and analyse these data.

4.3 Dam Safety Incidents

There was a pre-emergency situation on Chardara dam in 1987.

During excavation for foundation of the power station and the access roads at the right bank and excavation for the service building a part of the right bank slope was under cut. The slope comprised clay beds which slopped towards the river at 4-5^{O}.

Filling of water reservoir and uncontrolled irrigation of farmland at the top of the cut slope caused intensive seepage towards the river, which triggered a landslide process on the slope. The landslide had a volume of 400,000 - 450,000 m^3 and a maximum thickness 20 - 30 m.

Stabilisation measures that were implemented at the time included:

-flattening of the slope by excavating 130 000 m^3 of material,

-construction of a deep drainage system (5m - 9m deep, 600 m total length)

-drainage works and installation of drainage wells 1,020 mm in diameter, from 15.3m to 21.2 m deep,

The implemented stabization measures prevented further landslide propagation with subsequent consequences.

Regular instrumentation observations of the landslide slope deformations are carried out at present. Deformation measured in 1993 of 0-28 mm and 1-89mm, in vertical and horizontal direction respectively were less by 3% than deformations measured in 1987.

4.4 Maintenance Procedures and Standards

An exploitation procedure for Chardara dam was prepared by «Tashhydroproject» Institute in 1993. «Standard exploitation procedures for reservoir with capacity 10 Mm^3 and more» (Minvodhoz USSR 1987) together with design studies were used for establishment of the exploitation procedure for Chardara dam.
"The procedure…" determines principal exploitation rules which meet the requirements of main water users and guarantees safety of the dam structures.
« The procedure…» is the guiding document for all organizations and departments which are related to the use of reservoirs regardless their departmental classification.

4.5 Existing Early Warning & Emergency Procedures

After the break up of the USSR the early warning system in a situation of floods or works upstream of the reservoirs has not been available. In the Operating Department there is a telephone which allows contact with regional and republic organizations. Communication between the site personnel and security is realized by an internal phone system. Actions of the operating personnel in an emergency situation are instructed by the Operating Department.

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 recognised 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 Chardara 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

Chardara Main Embankment

The dam appears to have been operated successfully for some 30 years and superficially appears to be basically sound (when inspected at a low reservoir level).

There are, however, a number of matters that detract from the safety of this hydraulic fill embankment and to which urgent attention needs to be given, namely:

(1) From the limited information available on the grading of the hydraulic embankment fill the IC is of the opinion that there is a high risk that material in the embankment and its foundations would liquefy during a severe earthquake, which would result in reduction of strength of the material. Displacements that could occur in this situation should be accommodated by the provision of sufficient freeboard in order to avoid overtopping of the crest of the embankment.

It is important that further data are obtained on soil properties so that possible strength reduction and slope deformation during liquefaction can be assessed.

(2) At many locations joints between the upstream wave protection concrete slabs have been found to be open. This increases a risk of seepage and material suffosion that could reduce the ability of the embankment to withstand severe reservoir wave conditions.

(3) The drainage ‘prism’ at the downstream slope of the embankment appears to have been blocked for many years and additional drainage relief holes were constructed (Ø120 mm and at 220 m centres). However, most of the drainage holes have also become blocked which has resulted in seepage emerging from the toe of the embankment at high reservoir levels. This reduces the factor of safety of the downstream slope against sliding. There are zones of erosion and settlements on the downstream slope.

(4) No piezometer records were made available concerning the internal water levels in the embankment and it is reported that many piezometers are blocked or broken. Similarly, no current settlement or seepage records were available which would confirm, or otherwise, the satisfactory performance of the embankment.

(5) Slope erosion was reported to have occurred on the right bank, upstream of the dam which could have blocked the entrance to the inlet structure. The unstable material was removed, and the entrance to the inlet structure cleared. However, it is recommended to obtain records of the repair works that were carried out.

(6) A major landslide occurred in 1987 at the right bank, just downstream of the dam, near the switchyard. The landslide mass was estimated to be 400,000 - 450,000 m^3. Some remediation measures were undertaken to stabilise the slope. However, the instruments installed still measure some continuing deformation of the slope. It is recommended to carry out full topographical survey and investigation of material in this area in order to establish whether further stabilisation measures are needed, since further sliding could encroach on the area adjacent to the outlet canal.

Arnasai Embankment

Superficially the embankment appears to be sound. However, no records on embankment materials and performance monitoring instruments were made available to the IC. It is essential that these records are examined and embankment safety reviewed.

Dykes Surrounding the Reservoir

No records on materials and performance of the dykes were made available to the IC. It is recommended that the dykes are inspected, any such records are examined and the performance of the dykes reviewed.

Draw-off Works at the Power Station

Two pairs of two sluices with a maximum total design discharge capacity of 1,100 m^3/s are located in the Powerhouse. However, due to cavities which were formed under the gates and destroyed steel lining, significant vibration occurs when discharges exceed 500 m^3/s. This is an extremely dangerous situation in case the maximum floods cannot be routed through the reservoir and discharged via Arnasai spillway works only.

Intake for Kyzilkum Canal

It appears that seepage and erosion of fine soil particles occur through open joints in the intake conduits underneath the dam which resulted in a subsidence of the dam crest about 3m in diameter and 3 - 5 cm deep. The conduits are made of segmental units and it is reported that joints between the segments are opening causing seepage and erosion of the material.

The concrete of the intake structure is in a poor condition. Concrete cover to the reinforcement is damaged in many areas. While this in itself does not constitute a threat to the safety of the dam, it is possible that, if not repaired, further deterioration of concrete could endanger operation of parts of the headworks.

Draw-off Works at Arnasai Dam

The Arnasai gated spillway structure is in a reasonable condition and no important safety risks were noted.

5.3 Safety against Floods

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.

Chardara outlet structure was 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 R.L. 252.0 masl and R.L. 253.0 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. For example change from Cs=3Cv to Cs=4Cv increases the peak and volume by 10 to 15%. 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 case 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.

Chardara Dam was designed and constructed prior to the construction of two large reservoir located upstream of the dam (Toktogul and Andizan). As these two reservoirs have a flood storage role, their impact on flood peak at Chardara 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 Chardara 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 the 0.01% exceedance probability peak discharge with correction to change from 5,400 m^3/s to 7,450 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 4 turbines (Qturb=520 m^3/s), through bottom outlets located on each side of the HEP (Qs=1,280 m^3/s, currently limited to 500 m^3/s due to vibrations), through Arnasay outlet (Qarnasay=2,160 m^3/s) and through Kyzilkum canal off-take (Qkyz=200 m^3/s), totaling 4,160 m^3/s. The release through the turbines is based on an assumption that all turbines are operational, the power lines are capable of transferring 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 3,640 m^3/s, assuming that all gates are functional during an extreme flood event.

- The 0.01% exceedance probability hydrograph at Chardara has been developed for natural conditions without any water withdrawals for irrigation. The maximum off-take capacity upstream of the dam is 2,200 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. Inflow hydrograph was therefore reduced by deducting 0%, 25%, 50% and 75% of the maximum off-take capacity for each season. 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 -Chardara maximum water levels (masl)

	Irrigation Demand as a percentage of the maximum demand			
Scenario Description	0%	25%	50%	75%
Qt+Qb+Qarnasay+Qkyz	257.9 (93)	256.2 (65)	254.6 (46)	253.1 (0)
Qb+Qarnasay+Qkyz	259.8 (93)	258.0 (78)	256.4 (64)	253.6 (16)
Qb+Qarnasay	261.0 (100)	259.1 (89)	257.4 (75)	255.9 (61)

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 above the crest of the dam (254.5 masl), except for two cases. 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. Chardara dam was constructed prior to the design of Toktogul and Andizan dams, so the attenuating effect of these dams increases marginally the Chardara dam safety during extreme floods.

Flood routing studies indicate that the flood discharge capacity of the Chardara dam is only sufficient to control an extreme flood if the rate of irrigation abstraction upstream of the dam is more than 75% of the maximum rate when the turbines are not operating, or more than 50% of the maximum rate when the turbines are operating at full capacity. In each case both the other outlets (Kyzyl Kum and Arnasay) would need to be discharging at full capacity, if overtopping is to be avoided. This can not be regarded as complying with normally accepted safety standards.

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 Arnasay and the bottom outlets only.
- The possible requirement for new spillway capacity in the range of 2,500 to 3,000 m^3/s to be investigated.
- The existing bottom outlets be repaired and brought to the design capacity. Possibilities for increasing the bottom outlet capacities should be investigated in parallel with the design of a new spillway.

5.4 Provision for Emergency Draw-down

The dam has no surface spillway and floods are controlled by the outlets at the main dam (max flow restricted to 1000 m^3/s), and the outlet at the Arnasai dam (max. capacity 2,160 m^3/s)

It is reported that in February 1999 some 3 x 10^9 m^3 of water discharged into the Arnasai depression through the draw-off works of the Arnasai dam.

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 2). According to the Russian Seismic Standard, a seismic design coefficient (k_g) is derived for a site based on the 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 3). For this earthquake displacements of the crest are assessed and compared with the allowable wave freeboard

Although the seismicity of the site is low (Intensity Zone 6 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 dam performance in those conditions.

As a part of safety assessment a check shall be carried out to evaluate the height of seismic waves (seismic seiche) of the reservoir which may occur during a seismic event and which requires the 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. Chardara 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 dam, type of fill material used and its density (see Appendix A, Report 3.)

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 more comprehensive safety assessment than has been possible during the present study, for instance:

5.6.1 Safety 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 for gate operation can be assured in all circumstances, and standby generators to operate the crane gantry at the spillways in emergency are recommended.

5.7 Safety Assessment – Summary

5.7.1 Principal matters of concern

The IC see the following as the principal matters of concern as regards the safety of Chardara dam:

(1) Flood routing studies indicate that the flood discharge capacity of the Chardara dam is insufficient to control an extreme flood, depending on the rate of irrigation abstraction upstream of the dam, even with both the Kyzyl Kum and Arnasay outlets discharging at full capacity. In these circumstances, there is a serious risk of overtopping of the embankment.

(2) The present condition of the bottom outlets in the power station structure, which are limited to less than 40% of their nominal capacity by vibrations and other operating constraints, severely accentuates the flood routing situation.

(2) Piezometer readings combined with significant seepage from the surface of the embankment above the downstream toe indicate that water levels in the extreme downstream shoulder are high, which might possibly lead to local instability and erosion of the slope.

(3) The risk of liquefaction of the saturated sands and silts in the embankment and foundations under the effect of a severe earthquake is high with consequent risk of large deformations or partial collapse.

(4) There are strong indications that seepage is occurring into the open joints in Kzyl Kum outlet culvert, which could be causing the development of cavities in the surrounding fill, leading ultimately to local collapse of the embankment.

(5) Deficiencies in the embankment performance monitoring system.

5.7.2 Safety statement

From examination of the dam and the data made available, and discussions with the engineers responsible for the dam, the IC conclude that, until recommended investigations and studies reveal otherwise, the dam should be regarded as being at risk from the following dangers, and cannot be regarded as meeting normal safety standards.

1) Danger of overtopping of the embankment in the case of an extreme flood.

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

3) **Danger from internal erosion** with the risk of local collapse of the embankment in the vicinity of the Kzylkum outlet culvert due to continuing seepage through open joints on the structure.

4) **Danger from internal erosion** from seepages emerging from the lower part of downstream shoulders of the main embankment under conditions of high reservoir level, due to ineffective drainage.

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, Inspections and Studies

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 and undertaking modification in the future.

6.2.2 Surveys

1) Topographic Surveys

To provide essential information for the dossier of basic information concerning the dam the following ground surveys are recommended:

<u>Main embankment</u>
- Embankment longitudinal crest profile, on crest road;
- Typical cross sections of the embankment, to verify the 'as constructed' profile;

- Area downstream of the embankment, including longitudinal section on downstream drain channel and details of culverts and drainage discharge works;
- Landslide area downstream of the right flank.

Arnasai Dam
- Embankment longitudinal profile on crest road;
- Typical cross sections of the embankment, to verify the 'as constructed' profile.

In addition, it is proposed to carry out a topographic survey for the site of a possible new spillway structure at the left bank of the main dam.

2) Reservoir Bed Survey

The IC understand that reservoir sediment measurements were last made in 1997. It is recommended that further measurements are made within five years or so to validate the estimates of sediment volume.

6.2.3 Ground Investigations and Inspections

The following investigations and inspections are recommended:

1) Investigations at main dam

The IC are in general agreement with the proposals for investigations at the main dam set out in the November, 1998, Final Report on the Safety of Chardara Dams (Ref. 3) which comprise:

- Drilling into the embankment from the crest and downstream berm, to depths of up to 50m, with in situ permeability tests and static sounding tests (or geophysical profiling) to ascertain in situ densities;
- Investigation of possible borrow areas for construction materials and concrete aggregates by means of trial pits.

Details of the proposed investigations are set out in the 1998 report referred to, and are not repeated here.

2) Investigations at Arnasai dam

The Arnasai dam was constructed of compacted earth fill by conventional methods, and is not giving particular cause for concern. It would, however, be advisable to install new piezometers to monitor the phreatic surface in the downstream shoulder and it is recommended that advantage be taken of the drilling required to carry out in situ permeability tests to take samples of laboratory analysis to verify the design parameters.

3) Inspections

It has already been recommended in the 1998 Report on the Safety of Chardara dam (Ref. 3) that major works should be undertaken to improve the drainage conditions in the downstream shoulder of the main dam, and to prevent further loss of fine fill material through open joints in the Kzyl Kum irrigation outlet culvert.

To provide information on which to base a more detailed assessment of the requirements for other repairs and equipment than is possible in the present report it is recommended that a detailed inspection of both the embankments and associated structures be carried out. An inventory of defects, materials and repairs required should be prepared covering:

- Embankment upstream face (inspect when reservoir is at a low level);
- Embankment downstream face erosion protection and surface water drainage works;
- Structural and concrete works;
- Gates and operating equipment;
- Electrical wiring and lighting;
- Steelworks (e.g. ladders and handrailing).

Of particular importance, is the need to dewater and inspect the bottom outlet sluices at the power station structure in the dry. This is not presently possible because there are no means of isolating the sluices from the power station tailrace channel, and it is understood that it would not be feasible to close down the power station, even for a short period of time, because it is the only electrical power source in the region. Arrangements therefore need to be made for supplying and fitting a watertight bulkhead or installing downstream stoplogs.

6.2.4 Engineering Studies

1) Hydrological and flood routing investigations and studies as listed in Section 5.3.3.

2) Main embankment

The IC have reviewed and are in general agreement with the outline of the studies proposed in the 1998 Report on the Safety of Chardara dam, involving:

- Ground investigations and laboratory testing,
- Assessment of susceptibility of materials to liquefaction under seismic shaking (OBE and MDE);
- Seepage and stability studies for various rehabilitation options.

Options to improve safety proposed in the 1998 report comprise a central diaphragm wall cut-off and/or improvements to the downstream seepage interception works. The central cut-off would be very costly, however, and given that the downstream groundwater level is near the surface the IC are of the opinion that unless the cut-off extends through the full depth of the foundation alluvium its effect in reducing pore pressures beneath the downstream shoulder would be small, and unlikely to justify the expense.

The works suggested for improving seepage interception at the downstream toe would probably be effective but would also be costly. However, the records show that the general level of the phreatic surface in the shoulder is not particularly high and there appears to be little scope for a substantial general reduction in the water level. The IC suggest that a lower cost option could probably be devised, directed simply at avoiding the seepage breaking the surface as is reported to occur at present. Reinstating the downstream drainage wells combined with a weighted filter berm at the downstream toe might be a cost effective option.

All practical stabilizing options should be compared and the most favourable selected.

3) Review reservoir management procedures, with emphasis on achieving the safety of the dam in all circumstances.

4) In view of the time it is likely to take to implement the safety measures presently envisaged, it is recommended that studies are carried out immediately to ascertain the maximum level to which the reservoir can safely be impounded in the meantime with the dam in its present state.

6.3 Construction Works

A preliminary assessment of the recommended construction works is made on the basis of the safety assessment and the available data, as follows:

1) Main embankment

- Pending completion of the investigations and design studies an immediate start should be made on carrying out simple low cost improvements to the embankment drainage, comprising for instance:
  - Cleaning and possibly deepening the downstream toe channel drain, ensuring free flow at its outlet;
- Carry out more extensive stabilizing works on completion of the investigations and design studies, along with a full rehabilitation of the performance monitoring installation, comprising:
  - Reinstatement of piezometers
  - Network of horizontal and vertical movement markers.
  - Seepage flow measurement devices
- Repair upstream concrete facing.

2) Structural works

- Carry out repairs to prevent further seepage and loss of material into the Kzylkum outlet culvert as a matter of urgency.
- 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. Al necessary repairs, electrical wiring renewals, etc, should be undertaken immediately, and adequate standby generating plant provided. One particular matter needed immediate attention on the repair or replacement of the service gates at he Kzylkum irrigation outlet.

4) Power Station Bottom Outlets

To permit inspection of the sluices and associated hydromechanical equipment in the dry, it is necessary to provide and install a watertight downstream bulkhead or downstream stoplogs to both pair of sluices.

5) Miscellaneous

Other defects will be discovered during the detailed inspections and these will need to 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 (as proposed in the 1998 Report on the Safety of the Chardara dam, to be reviewed):

• Inclinometers, 30 m long	30 m
• Measuring probe	1 nr
• Piezometers	20 nr
• Surface movement markers	20 nr

(2)	Watertight bulkheads or stoplogs for the power station sluices	4 nr
(3)	Standby generators	2 nr
(4)	Early warning and communications equipment	

6.5 Emergency Planning

In view of the present unsatisfactory condition of the dam and the possible catastrophic consequences of failure it is essential that plans for dealing with an emergency situation should be well prepared, and an efficient organization, communications and alarm system in place. 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 detailed emergency plan instruction document should be prepared as a matter of urgency, setting out the procedures to be followed, and the responsibilities of the site manager, regional engineers and civil authorities.

6.6 Safety Measures-Priorities

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

The proposed Priority levels are:

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

Table 6.1 Chardara 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)				
• Instrumentation		☐		
• Repairs to upstream slop		☐		
• Repairs to drainage works				☐
• Repairs to Kyzylkum outlet		☐	☐	
• Cut-off wall				☐
• Assembling and disassembling of Hydropower units and power generator		☐		
• Spillway structure				☐
• Electromechanical hydromechanical equipment		☐		
• Miscellaneous Repairs		☐		
5. Supplies (6.4)				
• Piezometers and deformation monitoring equipment		☐		
• Standby generator(s)		☐		
• Watertight bulkhead or stoplogs for the power station sluices		☐		
• Early warning and communications equipment		☐		
6. Emergency Planning Studies (6.5)	☐			

7 CONCLUSIONS

On the basis of the information received, the 1998 Report on the Safety of Chardara dam, and a brief inspection, the IC are of the opinion that the Chardara dam cannot be regarded as meeting normally accepted safety standards, viz:

- Flood routing studies indicate that the flood discharge capacity of the Chardara dam is insufficient to control an extreme flood, depending on the rate of irrigation abstraction upstream of the dam, even with all outlets discharging at full capacity. In these circumstances, there is a serious risk of overtopping of the embankment. Further hydrological investigations are required but it appears that it may be necessary to provide additional spill capacity.

- There is an urgent requirement to restore the capacity of the bottom outlets at the power station which are presently limited to less than 40% of their capacity because of vibrations and other operational constraints. An essential prerequisite will be the fitting of downstream watertight bulkheads to the sluiceways to allow the sluices and associated hydromechanical equipment to be inspected in the dry.

- There appears to be a serious risk of substantial deformation or possibly partial collapse of the embankment being caused by a strong earthquake, due to liquefaction;

- Substantial cavities could be developing and could already exist within the embankment as a result of loss of material through open joints in the Kzyl Kum outlet, which if allowed to continue could ultimately lead to local collapse;

- Deficiencies in the instrumentation system have resulted in insufficient information being available to allow the behaviour of the embankment to be properly monitored.

- There is a risk of erosion and instability due to the seepages resulting from the high water level in the main embankment downstream toe.

The safety risks could be considerably mitigated if the reservoir was to be held to a level below its normal full storage level of 252.00 masl. Until such time as further studies have demonstrated that it is safe to impound to full storage level, or improvement works have been completed, it is tentatively suggested that the reservoir should be held down to elevation 250.00 masl (this level should be reviewed following further study).

High priority should be given to:

a) Supplying and fitting bulkheads to allow the power station sluices to be inspected in the dry, and then repairing the steel linings and rehabilitating the other hydromechanical equipment at the power station,
b) Repairing the joints in the Kzyl Kum outlet culvert,
c) Carrying out the investigation works followed by engineering studies relating to the stability analysis and remedial measures necessary to ensure the safety of the embankment, and to implementing such works as are found to be necessary,
d) Instituting a formal programme of inspections and reporting on the safety of the dam.

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

In view of the importance of the Chardara dam and the serious consequences of a possible failure, the IC recommend that an independent panel of experienced dam engineers (Panel of Experts) be appointed at an early date to monitor the rehabilitation works and to advise the Kazakhstan Government on matters relating to the safety of the dam.

REFERENCES

1. ICOLD Bulletin 72, 1989

2. SNIP 11-7-81, Russian standard for Seismic Design

3. ‘An Engineering Guide to Seismic Risk to dams in the United Kingdom’, Building Research Establishment (BRE) UK, 1991

4. L. Wang, ‘Zonation on seismic Geotechnical hazards in loess areas of China’, Manual for zonation on Seismic Geotechnical Hazards, 1999 Technical committee for earthquake Geotechnical Engineers, ISSMGE

APPENDIX A

CHARDARA DAM

LIST OF DATA EXAMINED

Chardara Dam

Appendix A – List of Data Examined

1. Summary of design,
2. World Bank June Mission, 1997.
3. CES/Sogreah Report on Safety of Chardara Dam 1998,

APPENDIX B

HAZARD ASSESSMENT PROCEDURE

APPENDIX B – HAZARD ASSESSMENT PROCEDURE

TableB2.1 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 B.2 Dam Category	
Total Classification factor	**Dam Category**
(0-6)	I
(7-18)	II
(19-30)	III
(31-36)	IV

Ref: ICOLD Bulletin 72

APPENDIX C

CHARDARA DAM INSTRUMENTATION

REPORT BY SPECIALIST MR V. N.PULYAVIN

OCTOBER 1999

Inspection of instrumentation condition and dam structures observations Chardara dam

Installation of large quantity of instrumentation was proposed in the design for control of Chardara dam condition, including:

Power station	
Foundation bench marks	- 2
Surface bench marks	- 59
slotmeters	- 21
piezometers under structure	- 19
ground piezometers	- 14
reinforce dynamometer	- 30
tensometers in concrete	- 30
ground dynanometers	- 37
coordinate-measuring unit	- 2
plumbs	- 2
Hydraulic filled dam	
piezometers	- 63 (in 14 sections)
foundation bench marks	- 20 (in 10 sections)
compilation marks	- 42 (in 10 sections)

Instrumentation was out of action repeatedly during the operation period of Chardara dam. Accordingly to Kazgiprovodhoz the last reconstruction of instrumentation used for measurement of deformation of the dam and seepage was carried out in 1993.
After that, for observations could be used the following equipment:

Power station

- marks for observations for horizontal displacements -3
- marks in foundation - 3
- geodetic marks in concrete - 31
- extensometers - 14

Chardara Main Embankment

- piesometer - 40
-geodetic marks in concrete - 27
- ground geodetic marks -33
- measuring weir - 6
- extensometers - 14

Kyzylkum irrigation outlet

- geodetic marks in concrete - 4

Arnasay embankment

- piezometers - 9
- survey bench marks in foundation - 3
- survey bench marks in concrete - 6
- ground survey bench marks - 6

Arnasay spillway

- survey bench marks in concrete - 4
- extensometers - 15
-

Right bank slope of power station

- piezometer -35
- survey bench marks in concrete -24
- ground survey benchc marks - 35

During the visit of the IC on 30 September 1999, dam operation staff informed us that the last topographical survey of deformations were carried out in 1987.
Instrumentation observations data concerning the technical conditions of water reservoir structures had not been given to the IC due to the some reason of organizational character.
There is an exact information that 35 piezometers were used in 1996 for analysis of dam condition.
Thus, on the basis available information at the present time it can be noted:

- Sufficient number of instrumentation for control of conditions of water reservoir structures was envisaged by the design;
- it was not succeeded to define the number of instrumentation used for control of structures conditions due to the absence of recording registration book;
- it is impossible to define appropriateness of functioning instrumentation due to the above mentioned reason and also because of absence of graphic material instrumentation.For implementation of my mission it is necessary to acquaint with aforesaid data.

  Preliminary based on visual examination, constructional features and age of the structure, and also at first hand, I can presume the following:
- string transcriber placed in power station building are out of order and for monitoring of constructions of power station building can be used only part of geodetic marks;most part of piezometer are silted.

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Recommendations:

1. It is necessary to carry out dynamic survey concerning frequency characteristics of the check points for determination of the reasons of an intensive vibration of power station spillways.
2. It is necessary to install several concrete remote sensors for concrete destruction detention during scour reparation works in concrete, outside the spillway gates. They allow control of concrete deterioration.
3. Carry out repair works of existent piezometers and if it is impossible construct new piezometers.
4. Installation of filters in boreholes should be carried out with obligatory presence of Employer representative. It is recommended to use modern synthetic materials for the filters . Equip the spillways on the drainage.
5. Reestablished lost survey bench marks.
6. Get experts of scientific-research and planning organizations to take part in data analysis.

DRAWINGS