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GARGAR
Small Hydropower Project

Feasibility Study

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Acronyms and Abbreviations

/a	per annum (per year)
AMD	Armenian Dram (local currency)
ANPP	Armenian nuclear power plant
B/C ratio	benefit-cost ratio
BMC	biological marginal concentration
BTU	British thermal units
c/kWh	UScents per kilowatt hour
CC	combined cycle power plant
cent	1/100 of a Euro, 1/100 of a Dollar
CIS	Commonwealth of Independent States
CJSC	closed joint stock company
CO_2	carbon dioxide
D/C	direct current
D/S	downstream
DPC	dynamic production cost
DRP	Daily regulation pond
DSCR	debt service coverage ratio
DSCR	debts service coverage ratio
EBRD	European Bank for Reconstruction and Development
EC	European Community, European Commission
EDPC	economic dynamic production cost
EIA	environmental impact assessment
ERC	Energy Regulatory Commission
ESC	Energy Strategy Centre
EU	European Union
Euro, €	Euro, currency of the European Currency Union
FDPC	financial dynamic production cost
g/m^3	gram per cubic meter
GJ	Giga-Joule
GoA	Government of Armenia
GT	gas turbine
GWh	Gigawatt hour (million kWh)
ha	hectare
HPP	hydropower plant
IAEA	International Atomic Energy Agency
IRR	internal rate of return
KfW	Kreditanstalt für Wiederaufbau
kg	kilogram
km	kilometre
kV	kilovolt
kW	kilowatt
kWh	kilowatt hour
l/d	liters per day
l/dp	liters per day per person
l/s	liters per second
LBNP	Line-By-Number-Procedure
LDP	Loriberd Development Project
LIC	Lori Irrigation Channel
LV	Low Voltage
m	meter
m/d	meter/day

m^3	cubic meter
m^3/s	cubic meter per second
masl	meters above sea level
mb	millibar
MBTU	million British thermal units
mg/dm^3	milligram per cubic deci-meter
MHPP	medium hydropower plant
mln.	Million
mm	millimeter
MoE	Ministry of Energy
MUS$	million US Dollar
MW	megawatt
mWC	Meter Water Column
MWh	megawatt hour (1,000 kWh)
NGO	Non-governmental organization
NMRC	Natural Monopolies Regulatory Commission (formerly ERC)
NPP	nuclear power plant
NPV	net present value
PSRC	Public Services Regulatory Commission (formerly NMRC)
RA	Republic of Armenia
ROE	return on equity
RoR	run-of-river
SHPP	small hydropower plant
SSR	Socialist Soviet Republic
t	ton
TACIS	Technical Assistance to the CIS
TOR	terms of reference
TPP	thermal power plant
TUS$	thousand US Dollar
UN	United Nations
USAID	United Stated Agency for International Development
USD, US$	United States Dollar
VAT	value-added tax
w/o	without
WACC	weighted average cost of capital
WB	World Bank
WCD	World Commission on Dams

1

Introduction

1. Introduction

1.1 Project Background

This Feasibility Study for Gargar SHPP was prepared within the framework of the EC-funded project "Substitution to the Nuclear Power through the Development of the Hydropower Capacity of Armenia, EUROPAID/112946/C/SV/AM".

The EC project aims at enhancing the hydropower capacity of Armenia and therefore at allowing an earlier closure of the Armenian nuclear power plant (ANPP) at Medzamor. The ANPP was shut down in 1989 following an earthquake, but in 1995 one unit was restarted in response to the severe energy crisis caused by the closure of the ANPP and by the energy embargo imposed by Armenia's neighboring countries. The European Union (EU) has been supporting Armenia with technical and financial assistance for the shutdown of the ANPP, which the EU is eager to see closed due to safety concerns at the earliest possible date. A precondition for the closure of the ANPP is that there are sufficient other sources of energy, preferably indigenous as the Armenian Government wants to reduce the country's dependency on imports.

Hydropower is Armenia's only indigenous energy source, and it has considerable development potential. Thus, in December 2002 the European Community represented by the Commission of the European Communities (Branch office in Yerevan), on behalf of the Armenian Government, Ministry of Energy, commissioned Fichtner GmbH & Co KG, Stuttgart / Germany, with the preparation of feasibility studies for bankable small and medium hydropower projects (HPP) with a combined capacity of approximately 70 MW as a contribution towards the envisaged replacement of about 400 MW nuclear capacity.

The scope of work for this component of the Consulting Services Contract covered the review of all existing hydropower schemes and feasibility studies available, the identification of the most relevant bankable projects for small and medium hydropower plants for a total capacity of around 70 MW, and the elaboration of feasibility studies for the selected projects.

1.2 Selection of Gargar SHPP

Gargar SHPP was identified by the Consultant during the review and first economical assessment of the original planning of the "Loriberd Cascade Project developed by "ArmHydroEnergoProject" in 1992. The project was planned as Cascade and consisted of following three power projects:

- Loriberd Small Hydropower Project (SHPP)
- Loriberd Hydropower Project (HPP) 1
- Loriberd Hydropower Project (HPP) 2

Loriberd HPP 1and 2 are relevant for the present project Gargar SHPP and are briefly described in this context.

Loriberd HPP 1 would have a small weir downstream the confluence of Kaminka River with Dzoraget River at Stepanavan. It would be a run-of-river plant, diverting the flow via a 10.4 km long tunnel and a 1.6 km long canal to the powerhouse, which was located at Gargar River. This power plant was foreseen to take into consideration the flow of Gargar River and thereby increase the design discharge by 1 m^3/s for the next stage of the Cascade. The total capacity of Loriberd HPP 1 was planned as 8 MW.

The Loriberd HPP 2 project was planned to use the outflow of the Loriberd HPP 1 plant plus additional flow of the Gargar River. The flow was diverted from the newly to be constructed headworks at Gargar River via a 3.3 km long channel to a daily regulation pond (DRP). From the pond a pressure shaft would transmit the water to the powerhouse of Loriberd HPP 2, which was located on the right bank of the Dzoraget River. The total capacity of Loriberd HPP 2 was planned as 49 MW.

The approach used for the analysis of Loriberd HPP 1 was to estimate the additional costs required for all hydropower structures for taking Gargar River flows with the option to combine Loriberd HPP 1 and Loriberd HPP 2 to a one-stage project. The analysis showed, that the additional costs for the incorporation of Loriberd HPP 1 were estimated to be appr. 14.5 million US$.

The additional costs were compared to the additional benefits from the power generation. The additional power generation through installation of Loriberd HPP 1 is due to the additional discharge of 1 m^3/s, which is taken from Gargar River all year round, which is equal to a mean annual power generation of appr. 15.2 GWh. Taking the current maximum tariff of 0.045 US$/kWh the annual revenues from power generation would be equal to 450 TUS$. Under consideration of a discount rate of appr. 10%, which is common for hydropower development, the net present value of revenues was calculated to be appr. 6.8 MUS$. Even under consideration of a future tariff of appr. 0.09 US$/kWh, which is considerably high compared to present tariffs, the net present value of revenues might increase to 13.6 MUS$. However benefits would hardly reach estimated costs in the magnitude of 14.5 MUS$.

Therefore the installation of Loriberd HPP 1 project was considered to be not economical. It decreased the economics of the overall Loriberd Cascade Project. Therefore the development of Loriberd HPP 1 at Gargar River was excluded from the Cascade.

Instead the Consultant proposed the development of a separate small hydropower project at Gargar River, where the investment costs were expected to be considerably smaller than in case of Loriberd HPP 1. Thereby Gargar SHPP was identified by the Consultant and is subject of the present Feasibility Study.

1.3 Objective of This Document

This feasibility study assesses the technical, environmental, economic and financial feasibility of Gargar SHPP and thus provides a sound basis for an investor's decision to develop the project. The study contains the information required by banks for their decision to fund the project and thus can be considered bankable.

2

Project Description

2. Project Description

2.1. Location

2.1.1. General

Gargar SHPP project is located in the Lori district. Lori is the northern district of the Republic of Armenia (RoA). The district is surrounded by Georgia in the North, Tavoush district in the East, Kotayk district in the South-East, Aragatsotn Region in the South-West and Shirak in the West. The total area of the district is approximately 4125 square kilometers.

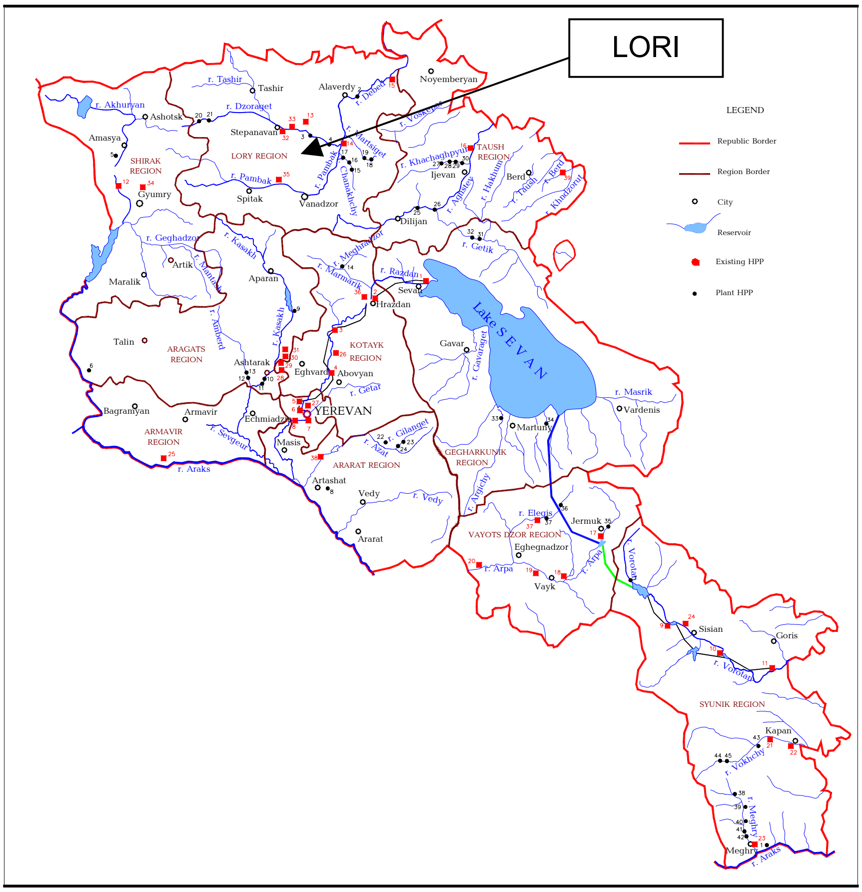

Fig. 2.1: General Location Map of Project Area

Lori Region has high mountain ranges, high altitude plateaus and deep canyons. It lies within the Little Caucasus Range and is surrounded by Dzavacheti Mountains to the West, the Bazum and Pambak to the South with peaks between 2500 and 3000 masl. The northern part of the district consists of the Lori Plateau with an average height of approximately 1400 masl.

2.1.2. Project Area

The project area lies between latitude 41˚-10' and 40˚-50' and longitude 45˚-20' and 45˚-40'. In this reach the Lori Plateau is approximately at a height of 1400 to 1250 masl, declining from Northwest to Southeast. The plateau is cut by the rivers Dzoraget and Gargar, which have formed a deep canyon of approximately 100 - 250 m depth and 10 - 250 m depth respectively.

The Gargar Small Hydropower Project (SHPP) mainly consists of the headworks with appurtenant structures, an embedded penstock and the powerhouse with the hydro-mechanical and electrical equipment.

The headworks of Gargar SHPP are located near by the village Kurtan on the Gargar River. The powerhouse is located near the confluence point between Dzoraget and Gargar Rivers. The waterway consists of an embedded penstock, which is laid along the river gorge.

2.2. Salient Features

Gargar SHPP utilizes a natural head of 223.4 m between the village of Kurtan and the confluence point of the rivers Dzoraget and Gargar. Downstream the village Kurtan a Tyrolean weir will be constructed to divert the flow of Gargar River to the headrace system. Before entering the penstock the flow enters a 35 m long sandtrap consisting of two chambers, each 2 m wide. From the sandtrap the water is conveyed to the 2120 m long embedded penstock with a diameter of 1.0 m, which is laid along the river gorge. Due to potential risks caused by rock- and landslides in the gorge, the penstock was planned to be embedded. The penstock feeds one Pelton turbine with four jets. The installed capacity of Gargar SHPP is equal to 3.2 MW with a design discharge of 1.8 m^3/sec, the mean annual energy generation was calculated to 12.3 GWh.

2.3. Topography

About half of Armenia's area of approximately 29,800 square kilometers has an elevation of at least 2000 meters, and only 3 percent of the country lies below 650 meters. One of the lowest points is in the valley of the Debed River in the far north with an elevation of 430 meters.

Elevations in the Lesser Caucasus vary between 2640 and 3280 meters. To the southwest of the range is the Armenian Plateau, which slopes southwestward toward the Araks River on the Turkish border. The plateau is masked by intermediate mountain ranges and extinct volcanoes. The largest of these, Mount Aragats, 4090 meters high, is also the highest point in Armenia.

The Lori district is situated in the northern part of Armenia, bordered for about 75 km along with Georgia. The district is bordered from north with Somkhet range, from south - Bazoum range separating rivers Dzoraget and Pambak basins, and from the west with Djavakh range. One of the highest points in the district is the Agkasar Mountain with an elevation of 3196 masl. The Lori plateau has a mean altitude of approximately 1400 m. The Plateau is cut by deep canyons and gorges.

The Gargar River, on which the Gargar SHPP is being planned, is a tributary of the Dzoraget River. Both rivers are tributaries of the Debed River, which flows from Southwest to Northeast on Armenian territory. The project area is characterized by the Lori Plateau and the deep and steep cuts by rivers such as Gargar in form of canyons. The plateau has a decreasing elevation from 1270 masl near Kurtan to 1230 masl near the planned powerhouse site. The elevation of the Gargar River decreases from 1250 masl down to 995 masl at the same reach.

2.4. Climate

The climatological conditions can be considered as mild and considerably damp during all seasons of the year. The winter is mild with deep and enduring snow cover. The snowfall starts at the end of November.

At Kurtan, the mean maximum air temperature is 17.1˚ C in July, the hottest month, the minimum is –3.6˚ C in January, the coldest month. The absolute maximum temperatures are 35˚ C in July and -31˚ C in January respectively. The region receives only 687 mm precipitation of which 75% occurs from May to October. The bulk of precipitation occurs in the months of May, June and July.

2.5. Transport and Communication

Armenia relies mainly on aviation that connects the country with the rest of the world and land connections via Georgia and Iran. The nearest seaport is Poti in Georgia, through which Armenia gets access to the countries of the Black Sea region.

Armenia has a well-developed road network, serving all areas of Armenia’s economy with a road density of 3,360 kilometers per 1,000 square kilometers. The road network consists of 7,700 kilometers of interstate roads, inter-republican roads (regional) and local roads.

More than 100 million USD has been invested in the reconstruction of the transport infrastructure of Armenia with the assistance of international organizations (World Bank, TRACECA, UN) and the Armenian Diaspora recently.

Vanadzor as the largest city of Lori district near Kurtan and can be accessed via rail from Yerevan, about 200 km away, as well. The railway is operating seldom but regularly. However access to the village of Kurtan has to be carried out by road.

The closest airport to Kurtan is at Gyumri at a distance of approximately 70 km by road. Gyumri has an airport allowing small aircrafts to fly from Yerevan and Tbilisi.

In regard to communications, the overall situation can be considered as good. Even mobile telephones can be used nowadays in the larger cities such as Vanadzor, Alaverdi, Stepanavan as well as Kurtan Village in the Lori district. The net has been enlarged considerable in recent years.

2.6. Access to the Project Area

The present available access to the project area by road from the city of Vanadzor or Alaverdi to the village of Kurtan is considered to be good. From these roads up to the headworks it is necessary to construct a 0.5 km long access road. The existing access road to the powerhouse shall be enlarged and improved. A bridge shall be constructed over the Gargar River in order to reach the powerhouse located at the left bank of the river.

2.7. Administrative Organization

Lori is one of the ten districts (Marz) constituting the Republic of Armenia. The highest administrative authority is the Governor. The headquarter of the district is located at Vanadzor, where all main administration can be found for the region. The district is divided into several administrative subdivisions, which are the communities of the Marz.

The district has technical support of various departments of the Government of Armenia at the headquarters at Vanadzor, such as Cadastre, Ecology and Health Protection, Transport, Agriculture and Fishing, City Building and Construction, Irrigation etc. Most institutions are located at Vanadzor, the local operating offices are located at Stepanavan, approximately 15 km northwest of Kurtan. The local offices are under directive and supervision of Vanadzor headquarters.

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3

Environmental Impact Assessment

3. Environmental Impact Assessment

3.1 General

3.1.1 Introduction

Extensive investigations were carried out within the elaboration of the Feasibility Study of Gargar SHPP. The main purpose of investigations was to determine the current baseline conditions and to identify the impacts on environment through the development of hydropower projects in the area. Appropriate mitigation measures are proposed in order to minimize impacts or even to improve the environmental conditions. The present assessment is based on extensive desk and field studies, it was carried out by the Yerevan State University of Architecture and Construction.

3.1.2 Location and Project Type

Gargar SHPP was developed from the original planning of Loriberd Cascade Hydropower Project. Since the implementation of Loriberd HPP 1 was not economical, the development of Gargar SHPP was proposed by the Consultant Fichtner. The project is located in the Northern part of Armenia, in the district of Lori Region. The hydropower plant is located near Kurtan village. The power plant is a run-of-river project with a total capacity of approximate 3.2 MW. It is located near the confluence point of Dzoraget and Gargar rivers, at the height of 994 masl. The penstock has a length of 2120 m.

The principal sketch of the layout of the hydropower plant can be seen in the plan view in the Annex to this section.

3.1.3 Environmental and Socio-Economic Conditions

Gargar SHPP is planned to be constructed on Gargar River. In a 3 km long reach, the river flows along a deep gorge. The depth of the gorge varies from 20 m in the upstream part up to 200 m close to the powerhouse. In the main part of the gorge the width varies from 30 m to 60 m.

The average annual air temperature in this region is positive. It is negative in the elevations higher than 3000 masl. The precipitation in Lori catchment area is fluctuating between 600 - 850 mm. The first snow cover in Kurtan appears approximately in the middle of November. The average date of snow cover melting is April 6.

The vicinity of the project is sparsely spread by green and vegetation. There are no natural forestry and green tracts along the river.

In the basin of Dzoraget and Gargar River and its tributaries there is considerable wildlife observed. Over 500 species of animals were encountered in the region, the detailed description of main classes thereof are provided in the section Wildlife.

According to data provided by the administration of Lori region the percentage of production of animal husbandry is considered for about 55% and the plant growing – for 45% in the overall agriculture production.

The population density of Lori Region is between 61 - 104 person per square kilometer. According to data provided by National Statistics this territory can be described as an area with average density of population in Armenia.

3.1.4 Approach to Environmental Impact Assessment

The Environmental Impact Assessment (EIA) Study follows the guidelines for EIA of construction projects set out by the World Bank and the World Commission on Dams (WCD). The EIA identifies the environmental implications of Gargar SHPP. Suggestions for remedial measures to eliminate or minimize any harmful affects as well as additional costs of measures are also incorporated in this section.

3.1.5 Methodology

Desk and field studies were carried out. The desk study has focused on the collection of background information. The socio-economic and environmental data of the project area was collected, processed and analyzed and is distinguished as follows:

- Soil, land, vegetation, farming, irrigation, plantation, forestation and deforestation
- Ecological conditions concerning fisheries and wild life
- Socio-economic conditions concerning the people and their basic necessities
- Landscape zones, natural and cultural heritage

3.2 Baseline Conditions

3.2.1 Landscape Zones

The great part of Lori region is located in the middle mountain circle with considerable climatic contrasts between territories of different altitudes, solar and circulating expositions.

Mountain-steppe, meadow-steppe and mountain-wood landscapes are dominating complexes in the middle mountain circle.

<u>Mountain-Steppes</u>
Steppe landscapes territories occupy Tashir-Dzoraget alluvial plains and hillsides and also plateaus of surrounding mountain ranges. Steppe zone is marked by moderate climatic conditions – not cold snowy winter and not arid summer. Here a great diversity of steppe landscapes in the composition of species is observed. Plain and hillside steppe landscapes

are observed. Plain, steppe complexes occupy the territories of Tashir-Dzoraget accumulative plain and flood plain of Virajdzor mountain range.

From landscape-ecological view they are classified into western (Tashir) and eastern (Dzoraget) parts. The Gargar region belongs to the Dzoraget part. The formation of the first took place on pliocene and quaternary lavas and the formation of the latter took place on andesit-bazalt lava flows, being formed with strong humus-rich chernozems, serving as a grassland. Steppe complexes of this region feel the lack of moisture in the summer period. The types of motley grass and cereal vegetative associations (steppe landscapes in 1600-1700 masl) dominate in top-vegetation. Meadow complexes are developed in moistened grounds and wide flood plains of steppe zone.

Meadow-Steppes
This landscape territory occupies the eastern part of the district, ranging from an elevation of 1600 masl up to 2100 masl. Spread on hillsides and plateau of Dgavakhet massif where leached chernozem meadow-steppe soils are considered as a soil types.

Mountain-Wood Landscapes
They have been well preserved and have favorable ecological conditions. Especially, they have been well preserved eastwards of river Chknah and its catchment area and in river Gargar catchment area ranging up to 2100 masl.

Alpine Zone
This zone is in favorable ecological condition, spreading up to 2600-2700 masl. Humid motley grass cereal types dominate here. Considerable areas of carpet meadows are observed here. Rocky hillsides are represented by poor top-soil and top-vegetation, where considerable areas occupy rocky showers and bare rocks.

3.2.2 Soil

Soil generating rocks in flat territories of Lori region are represented by alluvial-lake and alluvial-dealluvial layer of great magnitude, hillside territories by low magnitude (0.5-1.5 m) dealluvial formations. Considerable areas in Lori region are occupied by alluvial- lake, fluvioglacial and pebbly layers. The formations of the range of landscape zones are clearly observed within the basin. Powerful humus mountain chernozems and chernozem soils develop under steppe groups within 1350-1900m masl. Forest weakly-unsaturated soils are forming under afforestation but chernozem types of soils develop under subalpine meadows.

Morphogenetic and agrophysic characteristic of region types of soils are briefly described below:

Mountain Chernozems
It develops between elevations 1350-1900 masl and is represented by leached subtypes. Segregate by dark coloring, considerable magnitude, and heavy-loamy or mild clay mechanic composition, rich by organic substances (up to 10%), substantially leached. Mature forms of hulin acids

predominate in composition of organic substances. Active reaction of environment fluctuates in the range of 6.6-7.5 pH.

Humus horizons have dust dark-loamy composition. The quantity of humus horizons of arable soils fluctuates within 50-70%. The range of active moisture is high. The bulk weight of humus horizons fluctuates within 0.85-1.2 t/m^3. Allocated weight of soils up to down increases to the range of 2.5-2.7 t/m^3. They need regulation of water regime.

Brown Forest Soils
These soils are generated, substantially, in north hillsides of Bazum crest within 1500-2100m masl under afforestation. These soils are characterized by dark-gray or dark-brown coloring, loamy-clay composition and mean contents of humus (5-8%), environment reaction is mild-acidic or acidic (pH 5.0-6.5). Soils are aggregated (40-50%). Inconsiderable part of these soils are used as grasslands and mowing. Erosion is considerable.

Meadow-Steppe soils
These soils develop between 1400-2300 masl altitudes. They are generated on alluvial-dealluvial rubble layers. They are used for grasslands and mowing. The soils are characterized by crumbly structure, the proportion of humus is high. The soil is characterized by high steadiness from erosion. Soils on the south and east hillsides are eroded and rocky on the surface.

Mountain-Meadow soils
These soils occupy mountain areas higher than 2300 masl, which are the territories of Dgevaxet and Bazum crests (mountain range). The soils are characterized by low power of humus horizons, considerable rubble profile, and a high content of humus (13-18).

Precisely looking, in Gargar catchment area agricultural lands instead of steppes dominate. The main to be observed is the typical black soil, which is shown in the Ecology Map of the Annex.

3.2.3 General Land Use and Agriculture

There is still some forest, grain, industrial crops and mowing. Part of the basin territory, which is not included in industry, construction and land tenure sphere, is intensively exploited as summer grasslands and mowing.

Agriculture
The agriculture production dominates in Lori region since unfavorable conditions emerge for industry development in the present transition period of Armenian economy. In the vicinity of the project two main areas of agriculture development can be observed nowadays:

- cattle breeding
- vegetable gardening

The area used for agriculture is approximately 14 hectares large.

Industry
There is no industry to be observed in the project area. The main industry fields are concentrated in Tashir and Stepanavan districts.

3.2.4 Flora and Wildlife

Flora
As a result of investigations 90 species of superior vascular plants are registered in this region. Investigations determined that all 46 species described in “The Red Book of the Republic of Armenia” and being under vanishing danger, are not directly threatened by construction works of Gargar SHPP, because they grow in mass order out of foreseen construction zone.

Wildlife
Investigations showed, that in the catchment area of the river Gargar and adjoining territories 500 species of animals are numbered, in particular hare, fox, wolf, cabana, squirrel widely can be met. The kinds such as leopard, roe, forest cat and others listed below in the Table 3.1 are met rarely and are included in the Red Book.

Tubemakers are presented by warms and leeches, which are widely spread in the Gargar river catchment area. Arthropoda is presented by different groups and referred to the most numerous animals of Stepanavan district. Phalanx - yellow, many-colored, black scorpions, garden-spiders and ticks are registered among arachnids. Cyclops, crabs are usual among crustaceans.

Rich composition is inherent to insect representatives. Tolstotel Zakharova, Goroljub, Zerkalcenosnij are pointed out among - orthopterous.

Dragonfly class is represented by four-spotted and other species of dragonflies, Cosmos cav., Memocus Desv..

Cicada and psylla species are found among Homoptera. The representatives of hemipterous class are numerous. Lepidopterous class is distinguished by multiplicity. Webbed (-wing) insects are the most widely spread insects in this area. Membranous class is one of the spread insect groups of the basin, a total number of eight groups is known. Diptera is presented by different species of midges. Midges are represented by 8 species and observed in the catchment area of Gargar River. More than 20 species of molluscs are observed on the whole investigated territory.

Vertebrate animals are frequently found in this district. Amphibian animals are represented by lake and Transcaucasian frog, toad, tree-frog. Fauna of reptiles is diversified. Ornitofauna is original (100 species are numbered). 30 species of mammals are numbered in the fauna of mammals.

The animals listed in the Red Book are provided in the Table 3.1 below.

Table 3.1: Animals listed in the Red Book

NN	Name of animal
1	Leopard
2	Forest cat
3	Bear
4	Roe
5	Carnivore
6	Snake
7	Many-colored thrush
11	Rock thrush
12	Woodpecker
13	Sparrow (fly-catching)
14	Falcon (hawk)
15	Sparrow (white-throat)
16	Sparrow (nuthatch)

3.2.5 Aquatic Flora & Fauna

The river Gargar is rather small in size. Investigations state that no fish can be found in the river, only some types of frogs, snails, worms and other aquatic invertebrates.

Due to an ineffective sewerage system of Kurtan village the water from sewerage pipes flows into the river and this results in disappearance of aquatic flora in the river.

3.2.6 Natural and Historical Heritage

While analyzing the impact of the hydropower project on the environment, considerable preference should be given to the investigation of natural, cultural and historical monuments. The mentioned monuments, as specially guarded objects, shall have a strong protection regulation provided by law. They shall be under state control and any human interference shall be strictly prohibited to avoid negative impact on them.

<u>Objects of Cultural Heritage</u>
A church to the north from Kurtan village and a complex of conventual buildings can be observed in the vicinity of the project, as shown in the maps in the Annex. There are no cemeteries to be mentioned in the project area.

<u>Objects of Natural Heritage</u>
There are no objects of natural heritage in the vicinity of the project.

3.2.7 Public Health

The district is considered to be a resort region and problems related to public health are inconsiderable.

The main regional hospital is located in Stepanavan city and the aid points exist in the villages of the region.

Air
Sources of pollution can be described as follows: great deal of everyday garbage is accumulating in the cities and rural areas of Stepanavan and Tashir districts. The majority of populated areas have no sewerage net. Sewerage outflows are presented in the table below.

Table 3.2: Sewerage Outflows

District	Population thous.	Sewerage outflow thous. Tones
Kurtan	2032	1.03

The analysis showed that the sanitary-hygienic situation in the catchment area of Gargar River is close to optimal.

Water
The water is characterized as of hydrocarbonate quality with sulphate and calcium ion predominance. Mineralization fluctuated within 50-150 mg/dm^3. For irrigation purposes the water quality was considered as acceptable. Water does not possess leaching, acid, carbon, sulfate and magnesia aggressiveness. BMC (Biological Marginal Concentration) is about 1.2 mg/dm^3.

3.2.8 Socio-Economics

In the catchment area of Gargar River where Kurtan village is located the population number according to the national statistics is 2032 people[1]. In the year 1988 the number of population in this village was equal to 1907 and in 1999 it was 2069.

According to the information collected the active age group in Kurtan village is about 26% of inhabitants' total number. This is mostly caused by the fact, that the active age group is leaving for seasonal jobs abroad, especially to other CIS countries. The main source of income is agriculture (88%), only a small percentage is service sector (7%) and even less is business sector (5%). The unemployment rate is expected for about 14% of the active age group.

[1] This figure is taken from data of National census implemented by the RA National Statistical Service in 2001.

3.3 Impact Assessment

3.3.1 Physical Impacts

Physical impacts to be expected at the site are limited to earth excavation works, dust and noise caused by construction works as well as by traffic on access roads, destruction of soil and excavation of slopes. Also areas will be needed for dumping of excavated ground and allocation of construction materials.

Two essential principles should be considered for dumping of excavation material.

- land should be state owned;
- area should not be covered by agriculture land.

With respect to this the selected territory is located in the southern part of the project area near the confluence of Dzoraget and Gargar Rivers. The excavated soil can be directly put in the cavities existing on the area.

The construction material will be allocated on the flat territory near the plateau.

Noise and dust will be caused due to construction works as well as traffic on the access roads.

There is no need for construction of a camp as the temporary living space for working staff of the project during the construction phase can be provided in Kurtan village.

Due to construction of the road aside the river in the gorge the river width may be constricted, which may cause considerable damage to the road during flood events.

3.3.2 Biological Impacts

Firstly the impact on water quality of the river should be mentioned in this chapter. As already mentioned before, the environmental condition in the river is not quite good at the moment because of the poor sewerage system existing in Kurtan village. Nevertheless the amount of approximately 0.04 m^3/s water shall be considered as minimum ecological flow. The water reduction will have adverse impact also to the poor vegetation and few invertebrates existing in the river.

The main wildlife is in the forest, which is not going to be affected by the project implementation. The avian species might be forced to leave the territory due to reduction of feed caused by construction works in the area. No considerable damage is expected for the fish as almost no fish can be found in Gargar River.

Obvious changes in the microclimate of the canyon and top-soil are not expected.

There is no damage to be expected due to dumping of excavated material and allocation of construction material, as the proposed land is state owned and is not considered for agriculture use.

Various construction activities will have direct impacts in the immediate reach of the project. In the gorge along Gargar Rver there is no forestry to be affected, only a limited number of shrubs will be cut.

Traffic on access roads will cause dust to vegetation in the vicinity. The shrubs will be cut for construction of the access roads.

In the surroundings of Kurtan village the impacts caused by construction works are enumerated in the following:

- Due to the construction of the power plant a number of farmers will not be able to use their land with a total surface of 12.1 ha for plant cultivation.
- The dust caused by construction works may affect the crop in the nearest plants.

3.3.3 Minimum Environmental Flow

The Government of Armenia has recently issued a new resolution on determination of minimum ecological flow for Armenian surface waters. The decree N 592-N published on 22 June 2003 replaces the point 14 of chapter 5 of the article 121 of the RA Water code.

In accordance with the decree the amount of ecological discharge is calculated in the section of surface flow for each water resource by the 75 % of the 95% annual observation probability for each water resource.

Applying the guideline to the available hydrological series a minimum ecological flow of 0.04 m^3/s was calculated for the Gargar River. This minimum ecological discharge shall be spilled via a fishpass, which ensures the ecological patency of the river in future.

3.3.4 Socio-Economic Impact

The project will have no significant impact on both farming and non-farming households. No families will be resettled either permanently or temporarily since the inhabited area in the vicinity is not affected by the project implementation.

During the construction works a certain part of land will be used on permanent basis. No land will be acquired temporarily. The following Table 3.3 shows the land, which shall be acquired permanently for engineering works during the project implementation.

Table 3.3: Temporary and Permanent Land Acquisition for Gargar SHPP

No	Construction name	Area, hectare	
		Temporary	Permanent
1	Headworks	-	2.1
2	Waterway	-	9.7
3	Powerhouse	-	0.3
Total:		-	12.1

A total number of 11 private land plots[2] shall be acquired permanently due to construction works (including 4 land plots affected by the powerhouse, 7 by the waterway). All these land plots belong to one private person on the basis of a 99-years lease agreement. The income rate of this lessee may be affected due to loosing the land plots or parts thereof. No business premises will be affected.

There is no fishery carried out in Gargar River at present stage. However, there is a possibility to establish a fishery in the mentioned area in near future. In fact there is no impact on any fishermen presently but it might be expected for future.

The integrity and safety of historical and cultural monuments are not destructed. There are no cemeteries to be affected in the vicinity of the project.

There are no essential religious and cultural differences to be expected between construction workers and local people of the region.

3.3.5 Health

There is no impact to be considered on the health condition in the area.

3.3.6 Positive Environmental and Socio-Economic Effects

The construction of the hydro power plant will provide new employment opportunities for the local population.

The project implementation will improve the general infrastructure in the area.

Furthermore the project contributes to the planned shut down of Medzamor NPP in future.

[2] This number of land plots is estimated on the basis of the cadastral map provided by the RA State Committee of Cadastre and display the situation in the area at March 29, 2004.

3.4 Mitigation Measures

3.4.1 Physical Measures

Action 1.
Preventing noise and dust
Construction works and traffic on the access roads will cause noise and dust, which can be minimized by spreading water on the road, planting vegetation, particularly special shrubs along the river side.

Action 2.
Rehabilitation of Construction Sites
Various activities on the construction site would destroy the natural vegetation and disturb the soil. Therefore it will be required to rehabilitate the area to pre-construction conditions.

Action 3.
Slope Stability of the river banks in the gorge
The river Gargar in some parts in the gorge is going to be narrowed by the construction by an access road, which may affect the road during flood events. In this respect two different measures can be applied:

- to enlarge the river from the opposite bank equal to the part required for construction of the road;
- to flatten the slope near the river for construction of the road.

3.4.2 Biological Measures

Action 1.
Rehabilitation of the arable land and compensation to the farmers
The agricultural land plots required permanently during the construction phase should be rehabilitated by the arable land of approximately 1-2 meters. The compensation to the farmers affected should be estimated on the basis of maximum outcome that could have been produced from the lands.

Action 2.
Forest vegetation, biodiversity and wildlife survey, and developing linkages between community/agencies, project activities and the forest
The state of the forest and wildlife should be documented at the beginning and the end of project implementation. This will provide control of the project actions towards natural conditions of the area.

Action 3.
Continuous minimum ecological flow
In the framework of Gargar SHPP project a minimum mandatory water discharge of 0.04 m^3/s has been set to maintain the water quality in the river. In order to improve the water quality in the discharged part of the river the construction of a filtration station in Kurtan village might be recommended for the future.

This will conserve micro-flora, invertebrates and maintain the water quality in Gargar River. In case of probable fishery establishment in future the fishermen might be interested in the performance of these measures.

The construction costs of filtration stations vary according to the capacity and type of the station.

3.4.3 Socio-Economic Measures

As already mentioned above, no resettlement is required during project implementation. Only agricultural land plots will be acquired on permanent basis.

During the implementation of the hydropower project the acquisition of land has to be negotiated together with local governmental authorities. The acquisition of land comprises compensation for permanent land acquisition.

The land acquisition can be executed on the basis of:

- Mandatory land acquisition with compensation payment
- Sublease agreement

The costs for the compensation of loss due to agricultural land acquisition are calculated on the basis of the current market price for private land and for the state owned land on the basis of cadastral value[3]. The detailed cost estimation can be found in the Annex. The market value for land is the rate, which will enable the recipient to buy land with equivalent productivity.

As already mentioned above only one landowner will be engaged in negotiations according to data provided by the RA State Cadastre Committee. This procedure should be implemented on the basis of Articles 218-221 of the RA Civil Code. The main principle of negotiations is related to the voluntary basis, in case if no agreement the case will be settled in the court.

Following mitigation actions should be taken:
Action 1.
Timely payment of compensation of the affected families
Adequate and timely arrangement of compensation to the affected families for their loss of land.

Action 2.
Environmental seminar meetings
Seminars shall be organized in the area. The subjects on seminars should comprise issues of air, water and noise pollution, solid waste management, environmental allergens. The estimated cost is considered as USD 1.000 for 2 years.

[3] The cadastral value calculation is made on the basis the RA Government Resolution No: 1746 dated December 24, 2003 on Approval of Regulation of Cadastral Valuation of the RA Residential Land, Location Zoning Coefficients and Boundaries, Part II

Action 3.
Minimization of air, water and noise pollution
For the minimization of air pollution water should be sprayed out in sites where the concentration of suspended particles in the air is high. The workers, who have to work in crushing plants should be supplied with masks against air pollution. People should be instructed against throwing the garbage into the river.

3.5 Public Programs

If the Gargar SHPP is considered for development a public meeting should be organized by the local authority of the district. At the meeting information on the project design and the project implementation should be provided to inhabitants of the area, particularly to the affected families. The objective of the meeting is to receive comments and suggestions and to answer questions concerning the project.

3.6 Institutions

An environmental mitigation program should be implemented with the different institutions, including regional representatives from the RA Ministry of Nature Protection, the RA Ministry of Agriculture, Non Government Organizations (NGO's), representatives from Hunters and Fishermen Union etc, available on local level in the project area. Since the land acquisition is of special importance to the implementation of the hydropower project, local level institutions should be engaged for an efficient and effective implementation, co-ordination and monitoring of the environmental mitigation measures.

3.7 Environmental Monitoring Program

For implementation of such a program a group of experts, including representatives from the RA Ministry of Nature Protection, Yerevan State University of Architecture and Construction and a representative from the investor's part should be established.

The monitoring of environmental and socio-economic impacts is important in order to identify changing conditions since the compilation of EIA as well as to observe the acceptance of the population in the project area with the purpose to amend the mitigation measures to the new boundary conditions. Moreover the proper implementation of mitigation measures as specified in the EIA and tender design should be ensured by means of monitoring.

3.8 Environmental Auditing Program

Audits shall be carried out to assess the actual against the predicted impacts and the efficiency of proposed mitigation measures. Based on the assessment the performance of present and future projects should be improved.

The audits of the environmental mitigation measures should be done one year after the implementation of the project as well as at the end of the construction period. The reason for both dates is, that a year after implementation most compensation should be completed already, while the success of all mitigation measures can be assessed at the end of the project.

4

Data Basis

4.1

Power Market Analysis

4. Data Basis

4.1. Power Market Analysis

4.1.1. Sector Institutions and Legal Framework

In the late 1990s, the Government of Armenia embarked on an energy sector reform program with the long-term objective of developing a competitive environment in the energy sector. The main elements of the sector reform, as laid down in the Energy Law enacted in 1997 and revised in 2001, include:

- creation of an independent energy regulator
- separation of generation, transmission and distribution of electricity
- establishment of a single buyer market
- creation of a national dispatching center.

The Energy Regulatory Commission of Armenia (ERC, now called Public Services Regulatory Commission) was created shortly after the Energy Law was enacted. The large thermal and hydropower plants and the nuclear power plant (NPP) are closed joint stock companies (CJSC), most of them owned by the Armenian government, and some by the Russian government. Since 1999 several existing small hydropower plants have been privatized and are now privately owned. More than 20 licenses have been given for the construction of new SHPP. CJSC High Voltage Electric Networks is responsible for transmission, and CJSC Armenian Electric Networks (ElNetArm) – since 2002 owned and operated by a private company – is responsible for distribution. The single buyer market was established in 2002, with Armenergo acting as "single buyer". Armenergo (the previous monopolistic owner/operator of all energy sector entities) has been phased out in late 2004, and the transmission company now serves as single buyer. Other functions of Armenergo had already been transferred to a Settlement Centre and, in 2003, to an independent system operator (National Dispatch Centre).

Under the Law on Licensing of 2001, production, transmission, distribution and trade of electrical energy as well as electricity import and export require a license. Licenses are granted by the Regulatory Commission. For SHPP development two separate licenses are necessary: a license for construction or rehabilitation of a power plant and, following contruction, a license for operation. Types of licenses and the application procedures are described in the Commission's Regulation on Licensing in the Energy Sector (Decision No. 4 of January 30, 2002).

When reviewing license applications the Commission, in accordance with Art. 34 of the Energy Law, should take into account development programs for the energy sector, need for efficient use of domestic energy resources and protection of the interests of the domestic market consumers. The Commission has the right to reject an application when the project does not fulfill the requirements. The evaluation criteria for license applications are laid down in the regulation confirmed by Decision No. 64 of October 2, 2002. They include technical, environmental and economic feasibility as well as conformity with the objectives of national energy policy.

4.1.2. Generation Capacity

Armenia has a total capacity of about 3,200 MW, with two-thirds provided by thermal and nuclear power (1,756 MW and 380 MW, respectively) and one-third by hydropower (1,030 MW), as shown in the table below. Most of the thermal units are more than thirty years old. Fuel for their operation (natural gas and fuel oil) has to be imported, and shortage of cash for fuel payments has often led to supply disruption.

The nuclear power plant at Medzamor consists of two units. Both units were shut down in 1989 following the 1988 earthquake, but in 1995 one unit was restarted in response to the severe energy crisis caused by the closure of the NPP and by the energy embargo imposed by Armenia's neighboring countries.

The Sevan-Hrazdan cascade accounts for 50% of hydropower capacity (556 MW) and the Vorotan cascade for 40% (404 MW); small hydropower plants provide the remaining 10% (70 MW).

Table 4.1: Generation Capacity in Armenia

Plant	Available Capacity in MW
Hydro	
Sevan-Hrazdan Cascade (7 plants)	556
Vorotan Cascade (3 Plants)	404
Small Hydropower Plants	70
Total Hydro Capacity	1,030
Thermal	
Hrazdan	1,110
Yerevan	550
Vanadzor	96
Total Thermal Capacity	1,756
Nuclear	
Medzamor Unit 2 *)	380
Total Capacity	3,166

*) Available capacity; installed capacity is 415 MW

4.1.3. Energy Supply and Demand

In 2003, 5,501 GWh were generated in Armenian power plants. This was less than half of the energy generation in 1989, before the energy crisis, and also less than in the previous years. The water level of Lake Sevan had been excessively drawn down during the energy crisis. After Medzamor NPP was restarted, it was possible to reduce hydropower generation and thus, the outflow from Lake Sevan.

In recent years Armenia has become a net exporter of energy. In 2003, 307 GWh were imported and 583 GWh were exported, mainly from and to Georgia and Iran. Domestic consumption was 3,654 GWh. Distribution losses still amounted to over 20%, although they have been declining since the establishment of the settlement centre.

Energy is distributed to about 953,000 consumers. About 894,000 thereof are household consumers; they have a share of about one third in total energy consumption.

Demand forecasts in the 1990s assumed a growth in electricity demand of around 5% p.a., based on a fast recovery of the Armenian economy. Although the growth rates were revised downward to 1.3% p.a. in the Least Cost Generation Plan prepared in the year 2000 by the consultant Hagler Bailly, this assumption still proved to be too optimistic. In reality electricity demand decreased since 2000. The latest Least Cost Plan for 2003-2022 (PA Consulting Group, January 2003) is based on an average demand growth of less than 1% p.a., with an increase in peak load of 8-10 MW p.a. thereafter.

4.1.4. Electricity Tariffs

Electricity is sold to residential consumers at a tariff of 25 AMD/kWh (about 4.5 UScents/kWh) including 20% VAT. Consumers supplied at 6(10) kV pay 20 AMD/kWh, and consumers supplied at 35 kV and above pay 16 AMD/kWh including VAT. These tariffs have been unchanged since January 1999, although the general price level increased by 10% between 1999 and 2003. It is expected that – due to the cost increases resulting from change in ownership of some of the large powerplants and privatization of the distribution network – end-user tariffs will have to be raised at the end of 2004.

Export tariffs vary between 9.0 AMD/kWh for export to Iran and (on average) 14.6 AMD/kWh for exports to Georgia.

Generators received an average tariff of 8.9 AMD/kWh (about 1.6 UScents/kWh) in 2003. Due to high fuel cost, tariffs of TPP are much higher than HPP tariffs. Privatized SHPPs received an average 11.2 AMD/kWh (about 2 UScents/kWh). The large TPPs and HPPs have a two-part tariff, comprising an energy charge per kWh and a capacity charge per kW per month. SHPPs are paid per kWh (one part tariff).

By Decree No. 20 of February 9, 2004, the Regulatory Commission set the electricity tariff for new SHPPs constructed on natural water flows at the AMD equivalent of 4.5 UScents/kWh (plus VAT); this tariff will be in effect until 2016. The tariffs of existing SHPPs and new SHPPs on irrigation channels or potable water lines are still determined individually for each SHPP on the basis of actual cost and an appropriate profit, with a maximum of 3 UScents/kWh (plus VAT). The government guarantees that until 2016 all electricity generated by SHPP and other renewable energy sources will be purchased (Art. 59.1 (c) of the Energy Law).

4.1.5. Hydropower in Armenia's Energy System

Fuel security has been a continuing concern for the Armenian government, since all fuel for the thermal power plants has to be imported. A major objective of national energy policy is therefore to reduce the country's dependency on fuel imports.

Since the restart of Unit 2 of Medzamor NPP, the government has been pressured, in particular by the European Union, to finally retire the NPP for safety reasons at the earliest possible date. A precondition for the closure of the NPP is the replacement of nuclear capacity by other energy sources. In line with the government's policy of reducing import dependency, replacement energy should be indigenous.

Looking at costs only, SHPP with their low load factors and high capital investment costs cannot compete with thermal power plants. But hydropower, as the only indigenous energy resource in Armenia, is of strategic importance, and the Government of Armenia explicitly promotes the development of small and medium hydropower plants by the private sector.

Increasing domestic demand, demand for energy export to Georgia and Iran, replacement of thermal energy (to reduce fuel imports) and in the long run the replacement of the NPP require additional capacity and energy supply. It can thus be concluded that the capacity and energy provided by new SHPPs will be absorbed by the Armenian energy system.

4.2

Topography and Surveying

4.2 Topography and Surveying

4.2.1 General

This section comprises data on topographic-geodetic works, carried out by the engineering investigations department of ArmHydroEnergoProject between November 2003 and March 2004 for the project “Gargar SHPP” on Gargar River under subcontract of the Consultant Fichtner.

The works were performed in the local coordinate system, which was established for previous topographic works for the hydropower projects of the Loriberd Cascade on Dzoraget River. The reference elevation system is the Baltic system of 1977.

The following topographic-geodetic works were carried out:

- The creation of supporting elevation geodetic net for the survey basis of topographic maps of 1:500 and 1:1000 scale
- The topographic map in 1:500 scale with relief section through 0.5 m locations of planned headworks on Gargar River in the village Kurtan
- Topographic map in 1:1000 scales with relief section through 1.0 m of the waterway of Gargar SHPP along Gargar River.

Topographic maps of 1:25000 scale with relief horizontal section through 5.0 m, drawn according to the materials of aerial survey of 1979 were available for the inspected region.

The surveyed region is evenly covered with the reference points of the 3-4 class triangulations as well as control points and benchmarks of III class leveling, which were established for the development of Loriberd Hydropower Development Project. These points, as a rule, are located on elevations of 1300-1700 masl. Some of aforementioned points served as basic benchmarks for the plan fixation of the project area. The coordinates of these reference points were extracted from coordinates catalogue, stored in the archive of ArmHydroEnergoProject.

The local coordinates of the benchmarks “Surb Sarqis”, “Surb Gevorg”, “Pir 1020” and “Gr.Pn-1”, established by ArmHydroEnergoProject in 1993, were taken as basic benchmarks for the plan fixation of I and II class traverse points. The coordinates were available in the local coordinate system of rectangular coordinates in the Gaus view in six-grade zone.

“Gr.Pn-3”, located on the daily regulation pond site of the previously planned Loriberd HPP-2, and “Gr.Pn-1”, located on the powerhouse site of the previously planned Loriberd HPP 1, served as the basis for the elevation fixation of the present project Gargar SHPP. These points are included in the system of registered III class leveling, performed by ArmHydroEnergoProject during previous topographic works for previous planning of the Loriberd Cascade. The elevations of basic benchmarks of III class leveling were determined in the Baltic elevation system of 1977.

4.2.2 Topographic-Geodetic Works Carried Out

4.2.2.1 Plan Geodetic Net

The benchmarks of 4th class triangulation of State Net served as the basis for the creation of plan geodetic net in the surveyed region. The plan geodetic net was established by the method of 1st class traverse and was further condensed by distance-local traverse for the survey basis of topographic maps in 1:500 and 1:1000 scales. The central benchmarks were laid in concrete blocks according to the type 6 grade at the depth of 0.6 m.

The angles of 1st class traverse were measured with the theodolite Theo-010B by means of two circular modes; and the angles of distance-local traverse were measured with theodolite 2T5K by means of one circular mode without horizontal closing.

The side lengths of distance-local traverse were measured by a “Carl Zeiss Jena” phototachymeter-002 in direct and reverse directions with relative inaccuracy not more than 1:5000. The coordinates of points were determined in the local coordinate system, established for the planning of the original Loriberd Cascade Project of 1992.

4.2.2.2 Elevation Geodetic Net

The elevation geodetic net of the planned headworks and powerhouse sites was created by IV class leveling. For the waterway alignment a trigonometrical leveling was used. The benchmarks of registered III class leveling, established by ArmHydroEnergoProject for the planning of Loriberd Cascade, served as the basic points for the creation of the elevation geodetic net.

The IV class leveling lines were laid in form of separate strokes, leaning upon basic benchmarks of III class and were fixed on the site by ground and rock benchmarks. One ground and one rock benchmark were established on the territory of the planned headworks site. Two more ground benchmarks were laid on the powerhouse site.

The IV class leveling was performed by leveling staff H-3 with division of cylindrical level 20” on 2 mm (3 m long, centimeter divisions on both sides). The leveling measurement was carried from the center in direct and reverse directions. The length of collimating ray varies in the range between 40-100 m depending on the site relief. The deviation between leveling was not more than 10 mm.

The deviations on the powerhouse site, determined according to black and red sides of staff did not exceed 3 mm. The admissible discrepancy between basic benchmarks was calculated applying the following formula 20mm $\sqrt{L}$, where L is the stroke length in km. The elevation of the IV class leveling reference marks were calculated in Baltic elevation system of 1977. All benchmarks of IV class leveling were included in the plan net.

The catalogue of coordinates and elevation points of local distance-traverse and reference marks is attached to the Annex of this section.

4.2.2.3 Topographic Maps

Topographic maps in different scales were prepared for the further planning of hydropower structures in the project area. The planned headworks site with the small pond on Gargar River in village Kurtan was drawn in the scale 1:500. The area covers the gorge upstream the headworks down to a small wodden bridge crossing the Gargar River. The total area covered is equal to 7.5 ha. The topographic maps of the waterway alignment of Gargar SHPP were drawn in the scale of 1:1000. The map shows the relief with a distance of 1.0 m, it covers a strip from both sides of the river Gargar from the headworks to the confluence point of the rivers Dzoraget and Gargar with an area of approximately 50 ha.

All the maps were drawn in accordance with technical requirements. The survey basis on the site of headworks was determined by the 2^{nd} class traverse method, and on the site of topographic map in 1:1000 scale the survey basis was determined by the local-traverse method. The elevations of the net points were determined by trigeometrical leveling.

The planimetric survey and the survey of characteristic items was carried out in 1:500 and 1:100 scales. The survey was done for all scales by the plan-view and telescopic alidade KH (Tachymeter) from benchmarks of survey basis and transition points.

Topographic maps of all the above-mentioned scales were drawn by fair sketching in obligatory conventional signs; all the tablets have copies on tracing papers in carcass for duplication. Within the present Feasibility Study all relevant existing information was transferred to AutoCad 2004 drawings. The topomaps are available in digital form.

4.2.2.4 Valley Cross Sections Survey

Since different weir locations were to be studied, in addition to above mentioned topographical works the survey of nine cross sections was carried out along the gorge of Gargar River between the village of Vardablur and the village of Kurtan. The distance between sections varied in the range of 180-230 m depending on the relief conditions. The cross sections were limited between the vertical rocks of right and left slopes of Gargar River gorge. For the coordination of cross sections axis a main distance-local traverse with a length of appr. 2 km along the left bank of Gargar River was established. The axis points of the cross sections on the site were marked by red paint with the indication of the cross section number.

The measurement of the main traverse angles was performed by a single complete circular mode with the help of theodolite 2T5K. The sides of main traverse were measured by distance theodolite lines in direct and reverse directions. The coefficient of traverse distance lines was determined on the field comparator with a length of 100 m.

The coordinates of the main traverse points and cross sections axis were determined in the local coordinate system, established for the planning of Loriberd Cascade Project on Dzoraget River. Their elevations were determined by leveling in Baltic elevation system of 1977.

The catalogue of coordinates and elevation of main traverse and cross sections axis is given in the Annex.

The survey of cross sections as performed from the points of main traverse perpendicular to the flow of Gargar River. The elevations of the cross section points were determined by trigonometry leveling. The cross sections were drawn in the 1:500 scales and the original drawings were transferred to digital form to AutoCad 2004.

4.3
Hydrology

4.3 Hydrology

4.3.1 Gargar Catchment Area

4.3.1.1 General

Gargar River is a right-bank tributary of the river Dzoraget. The confluence point is approximately 4 km upstream the confluence point between Dzoraget into Debed River. The catchment area is 129 km^2, the river length is 28 km, the average height of the catchment area is 1680 masl, and the river average slope is 0.044%. Gargar River originates from the eastern slope of Bzovdal mountain range (so called Minor Bzovdal), near the foot of Chakhchan mountain (2364 masl) at the elevation of 2134 masl. At the elevation of 989 masl Gargar River flows into Dzoraget River, which gives a total elevation difference of 1148 m.

The basin of the river Gargar is located on the north of the Republic of Armenia in the Lori District. It is bordered from west and south with Bzovdal range, from north – by watershed between rivers Dzoraget and Gargar. The watershed consists of low mountains which are the continuation of the same range with the highest point Mount Medvejya (1820 masl). Further downstream the mountains are lowering into circular hills and further disappearing near the village of Gyulagarak. Following the direction of Bzovdal mountain range the river flows in latitudinal direction.

The right part of the basin of Gargar River in the upper zones is covered with forest and is the only source, which feeds the river by numerous tributaries, flowing through the slopes of the range. The left part of the basin is very narrow (the maximum width does not exceed 3 km), it is almost woodless and completely waterless.

In the past the whole basin was covered with forests. Nowadays the forest is preserved only in the upper sections of the right part of the basin. Oak trees, hornbeams are quite common; there are pine woods near the village Gyulagarak. The geological structure of the Gargar catchment area is predominantly volcanic. It consists of basalts and andesite-basalts, and the mouth part is composed of tuffs and granites.

The soils in the catchment area are different. Mountain black earth of damp steppes are spread here. The mouth sections are composed of carbonated and typical black earth. Moreover there are also wood-rocky stepped soils. Meadow steppes are predominant. In some places, mainly on the northern slopes of Bazoum range, greenwoods with beech, oak and hornbeam trees can be observed. The major part of the agricultural territories is composed of crop herbs and industrial crops.

4.3.1.2 Climatological Conditions

Introduction
Climatological conditions can be considered as mild and considerably damp during all seasons of the year. The winter is mild with deep and enduring snow cover. The snowfall starts at the end of November.

For the description of the climatological characteristics the long-term observation data of the meteorological stations Gyulagarak and Stepanavan is given in the Table 4.2. At the later station only temperature and precipitation is observed.

Table 4.2: List of meteorological stations

No.	Meteorological Station	Elevation (masl)	Observation period	Number of obser-vation years
1.	Gyulagark	1297	1933-1940, 1968-functioninig	39
2.	Stepanavan	1397	1891-1908, 1927-functioning	93

Temperature
The air temperature of mountainous regions is diverse, it depends on the altitude of area above sea level, on the shape of the relief and exposure of slopes. Average monthly temperature is fluctuating from 7.3 ^{0}C at Gyulagarak to 6.9 ºC at Stepanavan. The coldest month is January with (-3.6 ºC) at Stepanavan and (- 2.8 ºC) at Gyulagarak. The warmest month is July with 16.9 º C at Gyulagarak and 17.1º C at Stepanavan.

The absolute minimum temperature is varying from -26ºC at Gyulagarak and Stepanavan up to -31ºC. The absolute maximum temperature is fluctuating only by 1ºC, from 34ºC at Gyulagarak to 35ºC at Stepanavan.

The transition of average daily temperature through 0ºC takes place usually from March 11 in spring and from November 29th in autumn. The transition of average daily temperature through 5ºC takes place from 11th of April in spring and from 1st of November in autumn. The transition of average daily temperature through 10 ºC takes place from 6th of May in spring and from 9th of October in autumn. The average duration of warm days is 262 per year.

The first autumn frost is observed in the second half of September, and in spring the last frost might occur during the whole April month. The duration of the steady frost period at Stepanavan is 70 days. The average depth of frost penetration into soil is 30 cm and the maximum is 62 cm.

The data on air temperature are shown in the following tables.

Table 4.3: Average annual monthly and yearly air temperature in ºC .

Station	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Average annual
Gyulagarak	-2.8	-2.3	0.9	6.5	10.9	13.8	16.5	16.9	13.1	8.5	3.6	-0.4	7.3
Stepanavan	-3.6	-2.8	0.6	6.7	11.3	14.2	17.1	16.8	13.3	8.2	3.1	-1.8	6.9

Table 4.4: Dates of average daily air temperature - higher and lower of definite limits and number of days with temperature increasing these limits.

Station	Temperature					
	0 °C	Days	5 °C	Days	10 °C	Days
Gyulagarak	14.03/28.11	258	13.04/01.11	201	15.05/07.10	144
Stepanavan	11.03/29.11	262	11.04/01.11	203	06.05/09.10	155

Wind

There is only one wind observation station in the catchment area. Western winds are predominant. Annual wind velocities are not high and equal to 2.4 m/s. The maximum wind velocities with 1% of probability in Stepanavan might reach 51m/s.

In the Tables 4.5 and 4. 6 the data concerning velocity and direction of winds are given.

Table 4.5: Repetition of directions and calms [%]

Station	N	N-E	E	S-E	S	S-W	W	N-W	Calm
Stepanavan	2	2	11	11	3	21	40	10	38

Table 4.6: Average monthly and annual wind velocity [m/s]

Station	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Year
Stepanavan	3.8	4.2	3.1	2.5	1.8	1.4	1.4	1.5	1.5	1.7	2.5	3.1	2.4

Air Humidity

Air humidity mainly depends on the temperature regime and the quantity of precipitation, as well as from the physiographic characteristics of the region.

The air humidity corresponds to the air temperature and reaches maximum values in summer and minimum values in winter. Its annual value is equal to 8.2 mb at Stepanavan. The annual value of the relative air humidity is equal to 73%. Annual oscillations are small and equal to 9% with minimum values in January 68% and maximum values in July-August. The annual value of the saturation deficiency is small and equal to 3.5 mb. Air humidity data are given in the Tables 4. 7 and 4.9.

Table 4.7: Average annual monthly and annual absolute humidity [mb]

Station	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Average annual
Stepanavan	3.4	3.6	4.4	6.7	9.9	12.6	14.6	13.9	11.3	7.9	5.7	3.8	8.2

Table 4.8: Average annual monthly and annual relative humidity [%]

Station	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Average annual
Stepanavan	68	69	71	71	75	78	77	75	76	75	73	69	73

Table 4.9: Average annual monthly and annual deficiency of saturation [mb]

Station	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Average annual
Stepanavan	1.9	1.9	2.2	3.6	4.1	4.8	5.3	6.1	4.6	3.7	2.5	2.1	3.1

Precipitation and Snow Cover

The precipitation of the studied region is illustrated with the pluviometric points. The quantity of precipitation depends on the main wind directions in the catchment area, the height of mountains and the exposure of their slopes. The mean annual rainfall at Stepanavan is equal to 759 mm. Their prevailing quantity occurs during the period of April - July. The wettest month is June, the driest month is December. The maximum observed daily precipitation at Stepanavan is equal to 103 mm.

The first snow cover appears in the middle of November and melts in the first decade of April. The maximum decade height of the snow cover over the winter period in Stepanavan is 85 cm. Precipitation and snow cover data are given in the Tables 4.10 and 4.12.

Table 4.10: Average annual monthly and annual quantity of precipitations with modifications to the indications of the precipitation gauge [mm].

Station	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	XI-III	IV-X	Year
Gyulagarak	22	30	54	76	122	137	84	60	53	53	46	22	174	585	759
Stepanavan	21	26	42	66	122	130	74	58	47	48	33	20	190	497	687

Table 4.11: Maximum annual, monthly and daily quantity of precipitation

Station	For a year		For a month		For a day	
	mm	date	mm	date	mm	date
Stepanavan	952	1963	221	06.1940	103	05. 1944

Table 4.12: Snow cover formation and melting dates

Station	Number of days with snow cover	Snow cover formation dates			Fixed snow cover formation dates		
		Ave	early	late	Ave	early	late
Stepanavan	73	17.11	02.10	22.12	01.01	14.17	-

Snow cover break-up dates			Snow cover melting dates			Quantity of winters with no snow cover
Ave	early	late	Ave	early	late	%
08.03	-	07.04	06.04	12.03	21.04	40

4.3.1.3 Morphological Conditions

The river valley in the beginning cuts a V-shaped canyon to the landscape. The left slopes are steep up to 30-45º degrees, and the right slope is up to 20-30º steep and 200-300 m high. The slopes are covered with grass. Downstream the village Gargar the river valley expands, the slopes become flat and swamps occur near the river.

Further downstream village Vardablur, the river gradually runs into volcanic rocks (tuffs and porphyries) and flows though the gorge. The height of the gorge slopes varies from 8-10 m in the beginning and reaches up to 200 m near the confluence point to Dzoraget River near village Kurtan. The width of the gorge on top is 100 - 200 m.

Flood-plain sections are only observed between the villages of Gargar and Vardablur. The width of the flood-plain section reaches 200 - 300 m; they occur on both sides of the river. The bottom of the flood plain consists of pebble-sand. At some places there are small swamped sites, which are covered with dense swamp grass.

Near village Vardablur the left-bank flood plain section is occupied with vegetable gardens and orchards. Two kilometers upstream the river mouth there is also a flood plain, the width is 300 m. The latter is filled up with vast stones and boulders, transported by the river during previous flood events.

In the upper reach of the river the riverbed is curvy. Further downstream the village of Gyulagarak the bends are becoming sharper. At some locations the river breaches, however not more than two breaches are observed. In the lower reach Gargar River flows in a gorge, the riverbed is straight, in the mouth section the riverbed changes its course.

At the upper reaches of the stream the banks are flat and low. The bottom and the banks are composed of pebbles and are filled with boulders. In the flood plain reach the banks are absolutely flat. There are boulders with diameters of 0.5 - 0.8 m in the riverbed. Downstream village Vardablur the riverbed banks are high, steep and stony; the riverbed is filled with boulders with diameters in the range between 0.5 - 1.5 m.

4.3.1.4 Runoff

The flow is caused by underground, snow and pluvial waters. The first two categories are not considerable in volume, especially the last one. Predominant waters are of pluvial origin, although its duration is not so long. The snow does not accumulate in the lower zones of the basin; it gradually melts in the upper sections and penetrates into soil thus flowing into the river mainly in the form of underground water.

The flood period starts at the end of March or in the beginning of April. The flood peak is observed at the end of April or at the beginning of May, consecutively gradually decreasing. The low-water period is observed between June - July. In this period there might occur insignificant floods with the duration of less than 2 - 3 days and with a water level increase between 20 - 30 cm.

In the catchment area as on the mountain slopes as well as near the riverbed there are numerous spring waters. Ice regime on different sites is various. In the upper parts the river is completely covered with ice and even with snow, starting from village Pushkino only ice shores are observed. Frazil ice drift is observed as well.

The river water has a good quality. It is soft and sweet. During summer period the water is polluted by cattle. Earlier the river was used as the energy source by the village mills, located near villages Pushkino, Gargar, Gyulagarak, Vardablur and Kurtan. The river water is also used for the watering of vegetable gardens, spread near the villages of Gyulagarak, Opartsy and Vardablur.

4.3.2 Review of River Flow

4.3.2.1 Data Basis

The regular observations of Gargar River water runoff has started since 1955, when the gauging station in village Kurtan was opened.

Table 4.13: Supporting geometric net

River-point	Distance from the mouth km	Catctment area F, km²	Average weighted height	Functioning period		Graph “zero” elevation masl
				open	closed	
Gargar-Kurtan	4.0	123	1680	1955	Funct.	1232.46

The absolute elevation is given in the Baltic Elevation System.

Expeditionary observations of Gargar River basin were carried out as well; first by the Hydrometstation Department of Armenian SSR, and then by ArmHydroEnergyProject. The methods of observations, the processing of materials and the calculation of flows were performed according to single standards.

Suspended sediment loads in Gargar River were observed since 1971. Bed load movement of the river were not observed. For the description of the annual flow of the river Gargar at gauges of Gargar SHPP the available flow data at the gauging station Gargar- Kurtan were used.

Before starting the calculations for the average flow the reciprocal filling of data gaps, as well as the reciprocal correction of separate annual and average monthly water discharges according to the regression curves between the gauges of Kurtan and M. Gorky on Urut River were made.

Afterwards, the restoration of the natural flow according to the basic gauges was made i.e. in order to calculate actual water discharges. Irrigation and water supply discharges were added taking into account the recommended figures for water losses.

4.3.2.2 Pattern of Flow

A 45 year flow record of mean daily flows was available for analysis at the gauging station Kurtan at Gargar River. The latest data set available of the hydrological series was the year 2001. Average annual water discharges gathered for these years according to all basic gauges were assumed as flow norm. The flow parameters are shown in the Table 4.14. Coefficients of variation and asymmetry were calculated with help of the methodology “Approximate Maximum Probability” of the SniP norm.

The summary of natural average annual and monthly water discharges based on daily discharge figures of Gargar River can be seen in the Annex to this section.

The planned weir location is approximately 500 m downstream the gauging station of Kurtan. Consequently all calculations, which were carried out for the gauge were used for the weir site without any amendment.

Table 4.14: Parameters of probability curve of the average annual water discharges

River - point	**Average height of the catch-ment area Ho (m)**	**Catchment area A (km^2)**	**Flow rate Q (m^3/s)**	**Specific flow M ($l/s,km^2$)**	**Variability ratio Cv**	**Asymmetry ratio Cs**	**Extreme limits of water discharge observations [m^3/s]**	
							max	min
Gargar-Kurtan	1680	123	1.25	10.2	0.32	1.50Cv	2.36	0.45

Table 4.15: Average annual water discharges of different probability [m^3/s]

River - point	Qo	Probability in %								
	m^3/s	**1**	**5**	**10**	**25**	**50**	**75**	**90**	**95**	**99**
Gargar - Kurtan	1.11	2.31	1.96	1.78	1.50	1.22	0.97	0.83	0.75	0.64

The annual distribution of the water flow was determined by the analogical method with the distribution of the actual year. This means that these years, which correspond to 25%, 50% and 75% supply of average annual water discharges were selected from the observed record.

Table 4.16: Annual distribution of the flow for the typical years for the river Gargar at gauging station Kurtan [m^3/s]

Typical years	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Average annual
High-water 1974, 25 %	0.21	0.29	1.41	4.70	3.53	1.45	0.69	1.10	2.47	0.58	0.38	0.40	1.43
Average 1983, 50%	0.32	0.27	0.72	1.93	3.18	1.97	1.25	0.65	0.67	0.55	2.77	0.75	1.25
Low-water 1966, 75 %	0.33	0.35	0.59	1.89	4.68	1.24	1.01	0.31	0.46	0.54	0.29	0.22	0.99

4.3.3 Flood Analysis

Maximum water discharges for the river Gargar are observed usually during spring-summer floods. They are characterized by intensive snow melting in combination with rainfalls. Usually the absolute maximum instantaneous discharge is observed in May. The maximum water discharges for Gargar River are presented in the Table 4.17.

Table 4.17: Absolute maximum water discharges of Gargar River at gauging station Kurtan [m^3/s]

Years	Q m^3/s	Years	Q m^3/s	Years	Q m^3/s	Years	Q m^3/s	Years	Q m^3/s
1957	5.71	1966	26.6	195	12.5	1984	11.6	1993	13.2
1958	11.2	1967	7.95	1976	10.2	1985	11.1	1994	10.2
1959	124	1968	13.0	1977	4.10	1986	12.8	1995	9.30
1960	76.0	1969	26.7	1978	13.8	1987	9.55	1996	9.33
1961	5.44	1970	8.10	1979	16.5	1988	25.0	1997	10.1
1962	5.10	1971	7.10	1980	9.17	1989	9.80	1998	9.90
1963	10.6	1972	13.8	1981	7.81	1990	7.60	1999	11.3
1964	8.20	1973	5.69	1982	12.9	1991	15.5	2000	8.40
1965	13.1	1974	40.5	1983	31.6	1992	-	2001	11.5

According to these records the parameters of the probability curve as well as the maximum water discharges of different probability are presented in the Table 4.18 and 4.19 respectively.

Table 4.18: Parameters of the probability curves of maximum water discharges

River - point	Average height of catchment area Ho (m)	Catchment area F (km^2)	Flow rate Q (m^3/s)	Specific flow M (l/skm^2)	Variability ratio Cv	Asymmetry ratio Cs	Extreme limits of water discharge observations (m^3/s)	
							max	min
Gargar - Village Kurtan	1680	123	16.4	133	1.25	3.5 Cv	124	4.10

Table 4.19: Maximum water discharges of different probability [m^3/s]

River - gauge	Q o m^3/s	Probability in (%)						
		0.1	**0.5**	**1**	**2**	**3**	**5**	**10**
Gargar-Village Kurtan	16.4	194	124	99.1	82.5	65.9	52.8	37.2

4.3.4 Low Flow Analysis

The minimum flow of the river Gargar is observed during winter as well as summer-autumn low water period. In this report the minimum flow is considered in the context of average monthly and daily minimums for winter and summer-autumn low water period.

The duration of summer-autumn low water period is determined from July to October inclusively. The winter low water period covers the period from November to March. Average daily minimum water discharges at Kurtan gauge were selected from the daily water discharges tables for the whole observation period separately according to the above mentioned periods.

Average monthly minimum water discharges were selected from the average monthly and annual water discharges tables according to the same periods. These data were statistically processed and the probability curve parameters, obtained from the records, as well as minimum water discharges of different probability are shown in the tables below.

Table 4.20: Minimum average daily and average monthly water discharges [m³/s]

Years	Average daily minimum		Average monthly minimum		Years	Average daily minimum		Average daily minimum	
	Winter	Summer-autumn	Winter	Summer-autumn		Winter	Summer-autumn	Winter	Summer-autumn
1956	-	-	0.28	0.48	1979	0.15	0.21	0.26	0.37
1957	-	-	0.18	0.23	1980	0.16	0.13	0.25	0.35
1958	0.045	0.02	0.08	0.42	1981	0.25	0.21	0.30	0.46
1959	0.09	0.46	0.14	0.80	1982	0.24	0.27	0.27	0.42
1960	0.18	0.26	0.30	0.41	1983	0.18	0.33	0.27	0.54
1961	0.18	0.10	0.16	0.15	1984	0.22	0.26	0.26	0.32
1962	0.18	0.052	0.19	0.05	1985	0.18	0.21	0.24	0.30
1963	0.69	0.48	0.20	0.82	1986	0.18	0.24	0.24	0.32
1964	0.20	0.42	0.34	0.43	1987	0.68	0.22	0.50	0.39
1965	0.16	0.05	0.21	0.26	1988	0.37	0.79	0.46	1.21
1966	0.15	0.15	0.21	0.36	1989	0.43	0.24	0.65	0.35
1967	0.22	0.37	0.20	0.64	1990	0.30	0.29	0.48	0.43
1968	0.35	0.35	0.52	0.53	1991	0.40	0.38	0.47	0.42
1969	0.18	0.18	0.32	0.35	1993	0.49	0.19	0.66	0.55
1970	0.07	0.18	0.22	0.32	1994	0.43	0.32	0.59	0.56
1971	0.05	0.05	0.20	0.20	1995	0.42	0.29	0.56	0.44
1972	0.085	0.18	0.32	0.48	1996	0.42	0.46	0.51	0.66
1973	0.24	0.24	0.33	0.39	1997	0.40	0.44	0.60	1.08
1974	0.12	0.10	0.10	0.56	1998	0.43	0.38	0.56	0.54
1975	0.22	0.14	0.32	0.34	1999	0.48	0.56	0.57	0.67
1976	0.29	0.29	0.33	0.56	2000	0.49	0.16	0.56	0.28
1977	0.27	0.09	0.30	0.40	2001	0.27	0.18	0.34	0.24
1978	0.22	0.26	0.29	0.61	Ave.	0.273	0.26	0.341	0.46

Table 4.21: Probability curve parameters of minimum average daily and average monthly water discharges for the gauge Kurtan

Minimum type	Average height of catchment area Ho (m)	Catchment area F (km^2)	Flow rate Q (m^3/s)	Specific flow M ($l/s,km^2$)	Variability ratio Cv	Assymmetry ratio Cs	Extreme limits of water discharge observations (m³/s)	
							max	min
Average daily minimum Summer-autumn	1680	123	0.26	2.11	0.58	1.5Cv	0.79	0.02
Winter	1680	123	0.273	2.22	0.58	2.0 Cv	0.69	0.045
Average monthly minimum Summer-autumn	1680	123	0.46	3.74	0.46	1.5Cv	1.21	0.050
Winter	1680	123	0.341	2.77	0.47	2.0 Cv	0.66	0.080

Table 4.22: Average daily and monthly minimum water discharges of different probability for the river Gargar at village Kurtan [m^3/s]

Minimum type	Q o m^3/s	Probability in %									
		1	3	5	10	25	50	75	90	95	97
Average daily minimum Summer-autumn	0.26	0.70	0.60	0.54	0.47	0.35	0.24	0.15	0.086	0.059	0.045
Winter	0.273	0.77	0.64	0.58	0.48	0.36	0.24	0.16	0.10	0.075	0.060
Average monthly minimum Summer-autumn	0.46	1.09	0.93	0.86	0.74	0.58	0.43	0.31	0.22	0.18	0.15
Winter	0.341	0.82	0.70	0.64	0.56	0.43	0.32	0.22	0.16	0.13	0.11

4.3.5 Winter Regime

Average annual air temperature in winter in Gyulagarak, which is the closest point to the headworks of Gargar SHPP is varying between (-0.4ºC - 2.8ºC) Celsius degree.

The earliest ice formations are observed in November and the latest in March. Main ice formations are presented as ice along the banks. Seldom swimming ice on the water surface as well as in the water body is observed. The average number of days during the whole period of observations with shore ice is 37 days, and 41 days with freezing ice. Therefore this can be considered as rare event.

Apart from this in some years ice ways and ice blocks can be observed. The phenomena when water flows on the surface of the ice is very rare. The maximum duration of ice formation in cold years might reach 115 days. During the whole period of observations there were no years without any ice formations.

4.3.6 Ice-Thermal Regime

The temperature of water of Gargar River at the headworks of Gargar SHPP is expected to be almost zero during wintertime. Therefore the small pond might be covered with ice. As the calculations showed the average water temperature under ice cover at the weir site might be gradually changing from 0.9 - 0.8 ºC. Therefore the ice cover on the pond might be mainly observed in cold years.

Ice regime of the penstock was calculated taking into consideration the diameter of penstock with d = 1.2 m and at the continuos operation mode. In accordance with earlier made calculations for similar embedded pipelines with a soil cover of 1.0 m, ice is not expected to be formed on the penstock's walls, because the warmth generating during the friction is expected to be greater than the penstock water losses.

The maximum studge ice discharge during a cold winter, such as in 1973/1974 might reach 0.016 m^3/s at the weir site on Gargar River, during the average winter only 0.012 m^3/s are expected. The total studge ice volume might be equal to 80000 m^3 and 25000 m^3 for a cold and an average winter. The studge ice duration might reach 80 days.

4.3.7 Estimation of Water Levels

In order to prepare the discharge rating curves measurements of cross and longitudinal section with absolute elevation systems were carried out on different sections of the riverbed. On the basis of field data, hydraulic calculations for determination of water discharges at different water levels were done. The stream flow velocity was calculated according to G. Rostova's formula.

$$Vaverage = 11.6H^{0,5} + \frac{0,74}{2,3 + 0,35H^2} S^{0,35} + 2S$$

where:

$V_{average}$- is the flow average velocity, m/s
H – is the flow average depth, m
S – is the river section slope

As a result of these calculations water levels and other main hydraulic parameters of flow at different water discharges were obtained.

In the Annex to this section following cross-sections and plan views of the river Gargar can be seen:

- Plan view of cross-sections at the headworks and powerhouse of Gargar SHPP
- 2 cross-sections at the headworks of Gargar SHPP
- 1 cross-sections on the powerhouse of Gargar SHPP

4.3.8 Chemical Composition of Water

The chemical composition of Dzoraget River water is closely related with physical-geographical conditions, which determine the hydro chemical regime of the catchment. The geologic structure of the region, consisting of magmatic, volcanic-sedimentary rocks, and the hydrogeologic conditions of the underground formation determine the total mineralization of surface waters of the catchment.

During flood period the river feeding is by slightly-mineralized soil-surface waters. The river feeding within low-water period is mainly from underground waters. The water mineralization during low-water period sums up to its maximum values, which are two to three times higher than during flood-period.

The characteristical data of Gargar River water quality, carried out by ArmHydroEnergoProject in the past, can be seen in the following table. On the basis of data from the chemical composition table, the water content is mainly characterized as hydro-carbonated- sulphated and calcium-magnesiumized. According to its hardness the water can be considered as very soft. The river waters have no destroying influence on concrete.

Table 4.23: Chemical composition of Gargar River water

Runoff phases	Sample testing site	Water discharge m^3/sec	Transparency, cm	O_2 mg/l / % saturation	CO_2 mg/l	pH	Unit of measurement
Autumn low-water period 09.91	Weir site				non-destructive	8.2	mg/l
							mg/equiv
					4.4		% equiv

Runoff phases	Sample testing site	Ion Content								Ion sum	Phosphates mgP/l
		Ca	Mg	Na+H	HCO_3	SO_4	Cl	NO_3	NO_2		
Autumn low-water period 09.91	Weir site	69.0	10.3	10.1	213.5	35.5	17.8				
		3.45	0.85	0.44	3.50	0.74	0.50				
		36.4	9.00	4.60	36.9	7.80	5.50				

Runoff phases	Sample testing site	Silicon mgSi/l	Gross Iron mg Fe/l	Roughness		Oxidation mg0/l	
				total	permanent	permanganate	bichromate
Autumn low-water period 09.91	Weir site						
				4.3	0.8		

4.4

Sediment Transport

4.4 Sediment Transport

4.4.1 Available Data

4.4.1.1 Topography

With help of the available topographical maps in the scale of 1:25 000 provided by ArmHydroEnergoProject the longitudinal profile of the river upstream the existing weir location near Kurtan was analyzed. Furthermore the available topographic information including the water level surface upstream the weir location as well as GPS Measurement points were used.

The longitudinal river profile of Gargar River shows a mean gradient of approximately 2.58% upstream the planned weir site. The slope increases from the upper reaches near Vardablur and Kurtan to the confluence point between Gargar and Dzoraget Rivers. The slope shows a considerable increase to 6% - 10% downstream the planned weir location, which is planned to be utilized for power generation. Figure 4.1 shows the longitudinal profile from the GPS measurement points.

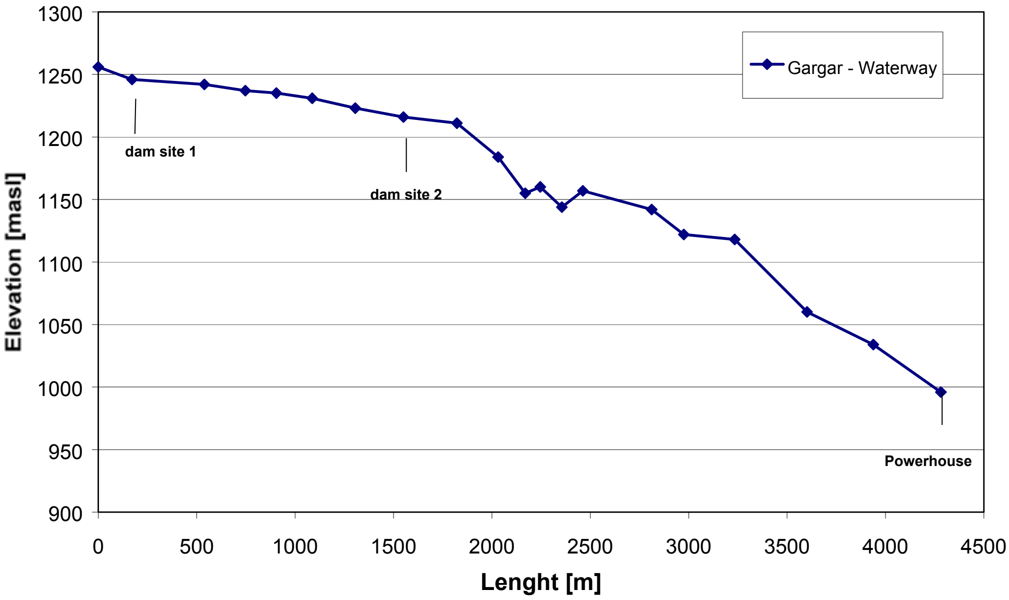

Figure 4.1: River slope of Gargar River

4.4.1.2 Bed Load

An important parameter for all sediment calculations is the grain size distribution of the bed material and its structure in the riverbed. It determines the resistance of the riverbed to the acting shear stresses caused by flow.

The bed of gravel bed rivers such as Gargar River near the planned weir site is characterized by armoring effects. The armour layer of a mountain river consists of coarse material on the surface of the bed, which protects the finer sediment material in the subsurface layer. No bed material measurements were available from hydrological yearbooks or previous investigations.

4.4.1.3 Suspended Load

During low flow period, when suspended loads are hardly measurable or even nil, no measurements were taken at the gauge. Suspended load measurements were carried out on regular basis during mean and high flows at the gauging station at Kurtan. Single-point measurements were taken during flood period. However measured concentrations and corresponding discharges were not published in the hydrological yearbooks, so that no suspended load rating curve could be established. From these measurements grain size distribution curves of suspended load measurements were taken from the publications.

Processed data of single-point measurements were available from the hydrological yearbooks for the series 1976 - 1988. However in the books, only the summary of decade values and their mean values were published. The available data were considered for analysis and estimation of mean annual suspended load transport.

4.4.2 Bed Material

For the grain size distribution of the armour layer at Gargar River, Kurtan, three different measurements were carried out. The final grain distribution curve of the river bed material at Kurtan is shown in the following figure. The grain parameters of the bed material are given in Table 4.24.

Table 4.24: Grain parameters of Gargar River bed material near Kurtan

Location	d_{16} [mm]	d_{30} [mm]	d_{50} [mm]	d_{65} [mm]	d_{84} [mm]	d_{90} [mm]	$\sqrt{d_{84}/d_{16}}$ [-]	d_m [mm]
Kurtan	8.4	31.2	52.6	70.7	98.1	115.0	3.4	51.4

Moreover the specific weight of the bed material was determined through laboratory measurement. For this purpose bed material was collected from the riverbed of Gargar River. A total number of three samples was collected, the test results of the laboratory analysis are shown in the next table. The mean specific weight of the bed material is 2.66 t/m^3.

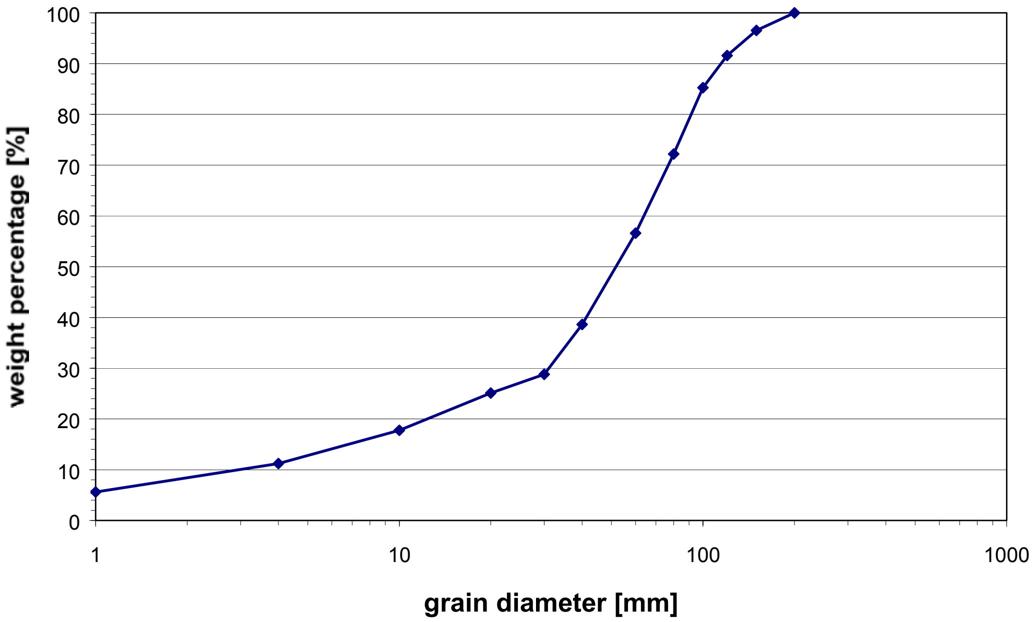

Figure 4.2: Final grain size distribution curve at Kurtan for bed load calculations

Table 4.25: Specific weight of each bed material

Sample No.	Specific Weight [t/m^3]
1	2.66
2	2.65
3	2.66
mean	2.66

4.4.3 Bed Load

Since bed load movement in Gargar River was not directly measured, empirical formula were used on the basis of available and measured parameters required in the formula. Concerning the assessment of bed load two aspects were considered:

- Initiation of motion
- Amount of transported material

4.4.3.1 Initiation of Motion

As proved by field measurements in mountainous regions worldwide, the bed load in nature has to be distinguished between fine gravel in the range between 2 mm < d < 16 mm at the initiation of movement, and the incipient motion of cobbles and boulders of the armour layer.

Considering a mean bed width of Tashir River of approximately 8 m, the water surface slope of S = 2.6% and a grain diameter at 65% passage of weight of 71 mm following threshold discharges can be given for the three different stages of bed load movement:

- no movement: $Q < 1.5\ m^3/s$
- movement of fine gravel (2 mm – 16 mm): $1.5 < Q < 5.1\ m^3/s$
- movement of bed material: $Q > 5.1\ m^3/s$

The calculated discharges reconfirm the observation during various field visits to the site. At discharges below 1.5 m^3/s, no bed load movement was observed. The discharge of 5.1 m^3/s for the beginning of motion of the armour layer indicates under consideration of mean daily discharges, that this phenomenon is a common event at Gargar River. It can already be concluded from present stage of works, that large bed load transport masses are expected at Gargar River during the flood period. It is somehow reconfirmed by the relatively high design flood with a return period of 100 years.

4.4.3.2 Amount of Transported Material

The mean yearly bed load transport of Gargar River at Kurtan was calculated to approximately 800 tons per year. The following table illustrates that only in case of extreme floods large amounts of coarse material are mobilized. Looking on the daily bed loads, these single events can be identified. However bedload movement of coarse particles at Gargar River can be considered as common phenomenon, only extreme events are seldom, such as in the years 1959, 1988 and 1992. Only in some years no coarse material might be transported by the river flow. The bedload characteristics of the Gargar River shall be considered for the design of the headworks.

Table 4.26: Calculation of yearly bed load at Saratovka

Year	G_g fine	G_g coarse	G_g total
[-]	[t/a]	[t/a]	[t/a]
1958	26	0	26
1959	236	3844	4080
1960	69	3323	3392
1961	7	11	18
1962	31	19	50
1963	258	795	1053
1964	145	75	220
1965	73	137	209
1966	89	664	753
1967	152	303	455
1968	205	1277	1481
1969	0	0	0
1970	53	84	137
1971	62	9	72
1972	175	897	1071
1973	163	96	259
1974	117	2414	2531
1975	126	274	399
1976	260	437	697
1977	74	0	74
1978	188	1413	1601
1979	127	179	306
1980	134	99	233
1981	95	85	180
1982	131	158	290
1983	127	573	700
1984	173	366	539
1985	82	17	99
1986	161	636	797
1987	109	177	286
1988	305	3399	3704
1989	24	30	55
1990	220	121	342
1991	124	559	683
1992	300	3582	3882
1993	170	282	452
1994	91	78	169
1995	144	111	255
1996	169	524	694
1997	210	185	395
1998	147	627	774
1999	152	222	374
2000	192	262	454
2001	89	499	587
Mean	136	655	791
Min	0	0	0
Max	305	3844	4080

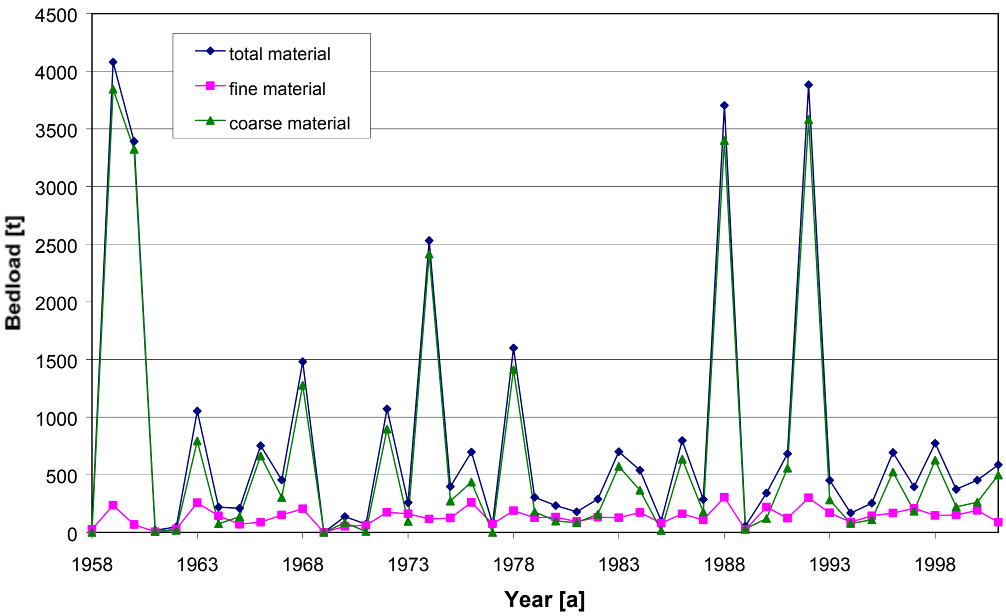

Figure 4.3: Distribution of bedload over the years

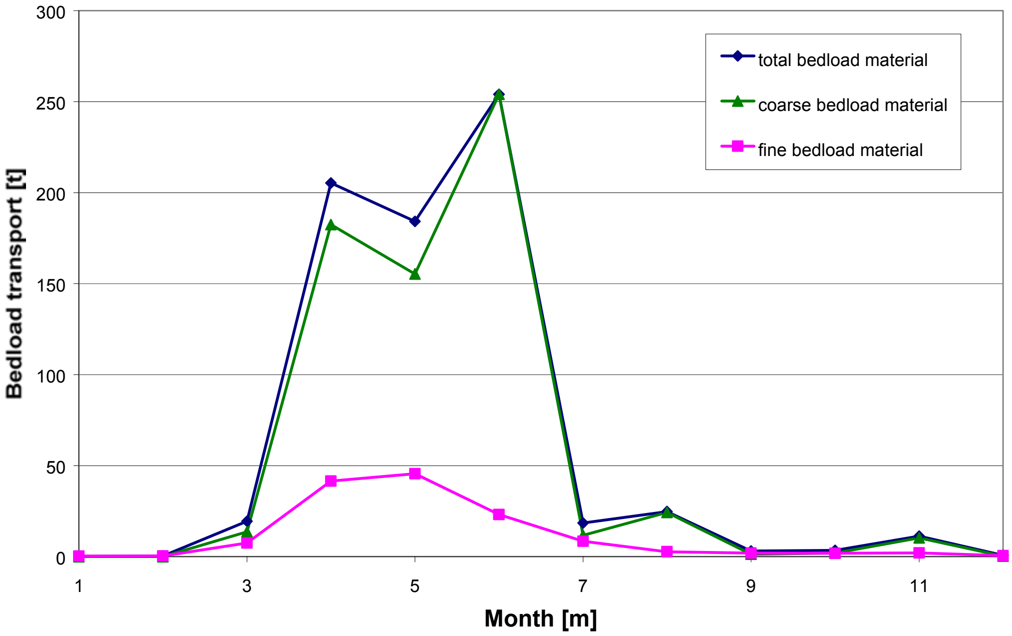

Figure 4.4: Distribution of bedload over one year

4.4.4 Suspended Load

4.4.4.1 Grain Size Distribution

Several grain size distributions of multipoint measurements of the series 1976 – 1988 were available. The summary of grain size distribution of suspended load was analyzed. The grain size is between 0.001 mm and 1 mm. The mean diameter of the grain size transported as suspension is approximately 0.007 mm. It is known from different gauging stations in Armenia, that the grain size distribution changes with the amount of transported suspended load. At high loads, the mean diameter is greater than for mean or low loads of suspended material. However these detailed information was not available from the hydrological yearbooks. It is expected, that the mean grain diameter at high concentrations reaches around 0.2 mm, as it is the case for Loriberd HPP nearby.

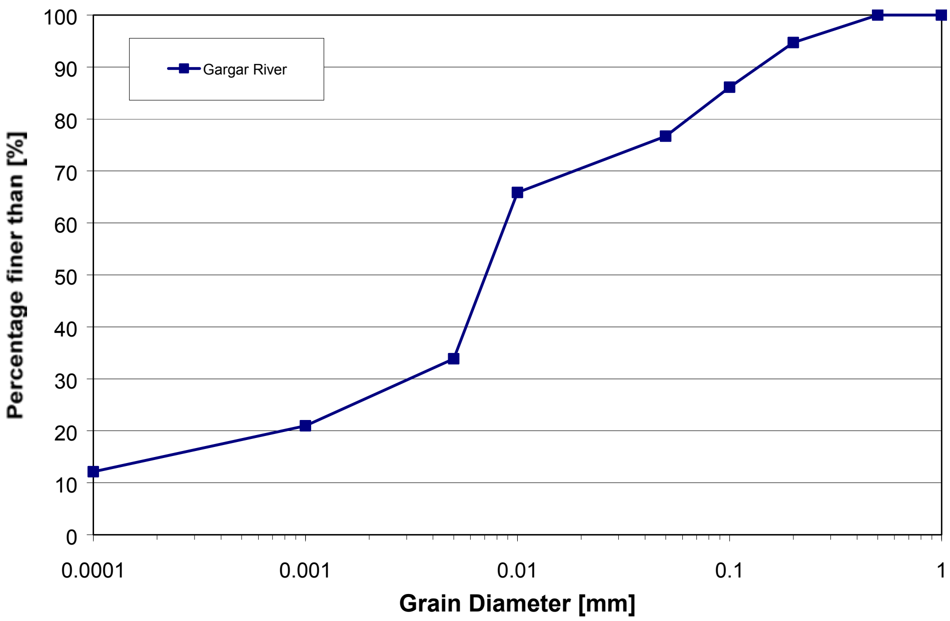

Fig. 4.5: Grain size distribution of mean monthly suspended load transport

The size of grains has to be considered for the design of the headworks and appurtenant structures of Gargar SHPP to avoid the entry of these particles to the turbine. The grains might cause severe abrasion effects on hydraulic steel structures and turbine wheels under large heads, such as in case of Gargar SHPP.

Similar to the performance of bed load, the variations between each year are considerable high, as explained in detail in the following paragraph. Moreover the analysis of decade data shows, that the large are transported loads during the flood season. During these seasons turbine runners might be affected through quartz particle as part of the suspended sediments.

4.4.4.2 Amount of Transported Material

The suspended load concentration was measured during the flood period (March to June/July) at the gauging station Kurtan for the time series between 1976 – 1988. Therefore the available decade data for suspended load transport were used for the estimation of mean expected transport.

According to the decade data the mean annual suspended load transport was calculated to 4500 tons/year. Figure 4.6 shows the variation of suspended load transport throughout the year. It can be seen, that the transport is high during the flood season, while during low flow the transport of suspended loads is expected to be marginal.

Table 4.27: Calculation of yearly suspended load at Kurtan, Gargar

Year	Load
	[t/a]
1976	5748
1977	659
1978	11973
1979	2231
1980	807
1981	1115
1982	1237
1983	546
1984	3266
1985	3560
1986	8046
1987	1555
1988	18090
Mean	4526
Min	546
Max	18090

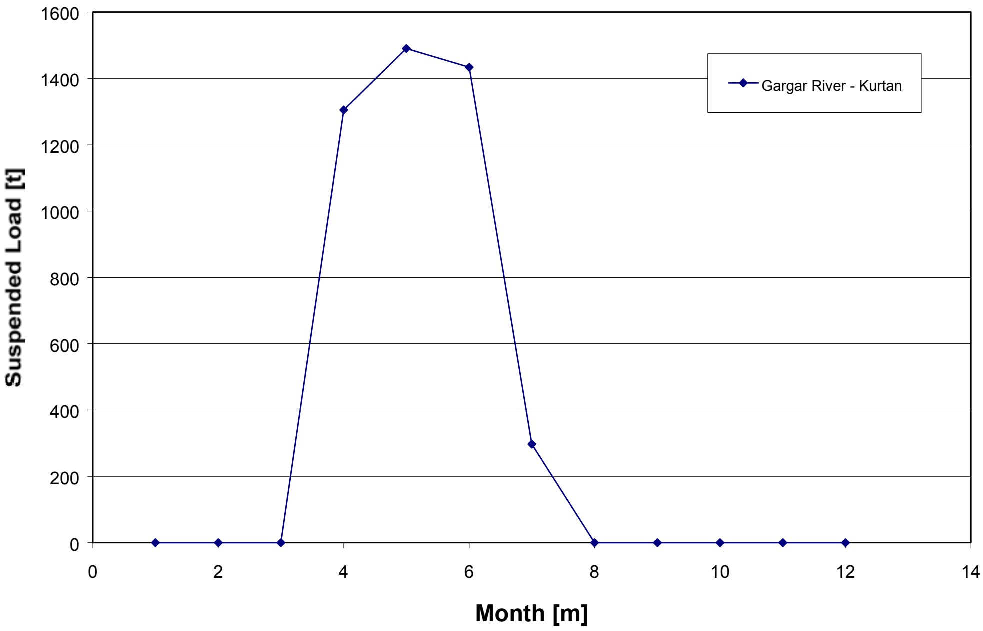

Figure 4.6: Distribution of suspended loads in a year

4.5

Geology

4.5 Geology

4.5.1. Introduction

Gargar SHPP is planned on the right-bank tributary Gargar of the river Dzoraget. The Gargar SHPP consists of the following structures:

- Headworks (weir, sandtrap and gravel trap)
- Penstock
- SHPP powerhouse

The headworks are located near village Kurtan. The SHPP powerhouse is situated on the left bank of river Gargar 300 m upstream the confluence point of Gargar River with Dzoraget River. The penstock is constructed both along right and left banks of the Gargar River crossing the river three times.

The engineering-geological survey was carried by the engineering-survey department of ArmHydroEnergoProject in July 2004 under subcontract of the Consultant. Following types of engineering-geological works were performed:

- Engineering-geological survey of the hydropower structures sites
- Engineering-geological mapping in the scale 1:1000
- Drilling works with total depth of 90 m; in total 6 boreholes, each 15 m deep

Nevertheless the drilling locations were beyond the boundaries of the structures sites due to inadequate drilling equipment. This report comprises results of the above-mentioned works as well as general geological and engineering-geological investigation results of the past years.

4.5.2. Geological Conditions of the Project Area

4.5.2.1. Orohydrography and Climate

The area of the planned Gargar SHPP structures comprises Gargar River valley reach downstream village Kurtan. The headworks are located near the village Kurtan, the powerhouse site is planned to be located downstream the crossing point of the gas pipeline with Gargar River, appr. 300 m upstream of the confluence point of the rivers Gargar and Dzoraget.

The described territory is a piedmont part of the Bazoum range, stretching to latitudinal direction with river Gargar on the right side and Lori Plateau on the left.

Gargar River, which is the right-bank tributary of the main water source of the region river Dzoraget, originates from the northeastern ledges of the Bazoum fault and flows in latitudinal direction. The feeding of the river is mixed: snowy, pluvial and spring waters. The flood period is usually observed in March-May.

The climate of the region is characterized by mild and snowy winters. The average annual air temperature is appr. +6.6ºC. The annual precipitation is equal to appr. 770 mm in this region. At the end of spring and beginning of the summer the precipitation is mostly in form of rains and storms.

Village Kurtan is the largest settlement in the project area.

4.5.2.2. Geological Studies

The first geological reconnaissance of the region was carried out in the mid of 19th century. Furthermore more detailed engineering geological investigations were the first time carried out in the described region by “ArmHydroEnergoProject” in 1955-1957. Additional engineering geological works were performed in 1965-69 and further in 1992-93.

4.5.2.3. Geological Structure

The rocks of volcanic-sedimentary formation thickness of Paleocene and lower-Eocene represent the oldest rocks in the region of the hydropower structures sites of the planned Gargar SHPP. These rocks consist of metasandstones, tuffs, porphyries and other rocks. These rocks are outcropping on the right slope of Gargar River gorge downstream the weirsite.

The rocks of volcanic-sedimentary formation thickness are mainly hydrothermally modified. They are inclined to northeast with separate stresses with a thickness of 70 m. The rocks in the zones of stresses as a result of mylonitization were transformed into white flour-like substance (mylonite).

At the weirsite, upper reaches and on the entire left slope of the gorge the above mentioned rocks are covered with Pliocene and lower-Quaternary basalts, which are fully outcropping in form of vertical shear cliff. On the left slope of Gargar River gorge, near the village of Kurtan these basalts compose 7 streams, which can be seen on Photo No.1, attached in the Annex to this section. The streams are layered on each other horizontally without essential intervals in-between. The brown volcanic sands are the basis for the lava streams.

Within the riverbed and at the valley slopes both basalts as well as the rocks of volcanic-sedimentary formation thickness are covered with young alluvial-coalluvial sediments, sliding dealluvial-coalluvial load sediments and other formations. The thickness of latter may reach several tens of meters. The size of separate boulders of these formations (especially in sliding sediments) is expected to reach 3 - 4 m.

There are no traces of young tectonic stresses in Pliocene and lower-Quaternary basalts, covering older rocks of the region.

4.5.2.4. Tectonics and Seismicity

According to the "Seismic Zonation Map of Armenia" the investigated region is located at the border of the second and third seismic zone, where the overall seismicity is estimated as > 9 scale according to MSK-64 scale (A = 0.4g, v = 32 cm/s).

4.5.2.5. Hydrogeological Conditions

The ground waters at the hydropower structures site are connected with the waters of the Gargar River.

According to their chemical composition, these waters are sweet, they mainly contain hydro carbonated natrium and calcium. These waters are not expected to cause corrosion to concrete. The data are presented in the Annex to this section.

There are practically no ground waters on the slopes of the valley except for a small spring, which is flowing near the zone of tectonic splitting of rocks of volcanic-sedimentary thickness. There are also leakages of technical waters, flowing from the plateau upstream the weirsite, where the houses of the residents of village Kurtan are located on the plateau.

4.5.2.6. Geomorphology and Physical-Geological Phenomena

Weathering, rockslides and mudflows from the steep slopes of the gorges are the main physical-geological processes on the territory. The landslide processes are observed on the slopes, composed of basalts and rocks of volcanic-sedimentary formation thickness. Moreover these slides and rockfalls are nowadays at an advanced stage of development.

A landslide, developed on the right slope of Gargar River is caused due to spring water, which comes out from a tectonic fault. The phenomena can be seen on Photo 2 in the Annex. The slides are mainly a mixture of stones and water. They are intensive and have a short duration.

4.5.3. Physical-Mechanical and Filtration Parameters of Soil and Rock

The physical-mechanical and filtration parameters of main types of soil and rock spread on the structures site of the planned Gargar SHPP as well as the layer numbers were determined the same as for the study of Loriberd HPP since the rocks on the SHPP and HPP sites are mainly similar.

<u>De-alluvial-colluvial Sediments (1st layer)</u>
De-alluvial-colluvial sediments consist of boulders. The size of boulders might reach 2-3 m. The filling consists of silty-clay and silty-sand material up to 30%. In some places there is no filling material.

- density of de-alluvial-colluvial sediments: 1900-2100 kg/m³
- rock particle density: 2800 kg/m³
- internal friction coefficient: tg φ = 0.700
- specific cohesion: 0.001 MPa

- admissible foundation design pressure: 0.5 MPa

These sediments are highly water-permeable, especially when there is little filling material. The average filtration coefficient of these sediments was determined to be appr. 10 m/d if there is a filling. Without filling material the coefficient might reach a multiple of this value.

Sliding-loose rocks (1^a layer)
Sliding rocks comprise big boulders, which may reach a size of 4.0 m, detritus and gruss from basalts, porphyries, tuffs and metasandstones and other rocks. The filling consists of sand-silty-sand up to 20-30%. In some places there is no filling material.

- density of these sediments: 1900-2100 kg/m³
- rock particle density: 2800 kg/m³
- internal friction coefficient: tg φ = 0.700
- specific cohesion: 0.001 MPa
- admissible foundation design pressure: 0.5 MPa

The coefficient of filtration varies between 10 m/d and more.

Alluvial-colluvial sediments (2^{nd} layer)
Alluvial-colluvial sediments of this layer are different according to grain-size distribution and lithological characteristics. These sediments consist of big boulders, pebble and gravel as well as detritus and gruss from different bedrocks. The filling consists of silty-sand material up to 25%.

- density of 2^{nd} layer sediments: 1800-2000 kg/m³
- rock particle density: 2700 kg/m³
- internal friction coefficient: tg φ = 0.700
- specific cohesion: 0.001 MPa
- admissible foundation design pressure: 0.5 MPa

The coefficient of filtration is equal to 5.0 m/d.

Dealluvial –proalluvial sediments (3^{rd} layer)
Dealluvial-proalluvial sediments of this layer consist of silty-sands and fine-grained sands with filling of boulders, gruss and detritus from different bedrocks up to 40-45%.

- density of 2^{nd} layer sediments: 1700 kg/m³
- rock particle density: 2700 kg/m³
- internal friction coefficient: tg φ = 0.466
- specific cohesion: 0.02 MPa
- admissible foundation design pressure: 0.25 MPa

The coefficient of filtration of this layer is equal to 0.05 m/d.

Lower Quaternary lava (5^{th} layer)
Lower Quaternary lava consists of basalts. The lower Quaternary basalts show a doleritic structure. Very often the basalts are separated in boulders, they show an irregular direction of cracks. In some places thin layers of volcanic ashes can be observed.

- density of basalts: 2600 kg/m³
- rock particle density: 2930 kg/m³
- static deformation modulus: 30000 MPa

- internal friction coefficient: tg φ = 0.839
- Specific cohesion: 0.3 MPa
- Pressure testing for dry sample : 92.5 MPa
- Pressure testing for saturated sample: 61.0 MPa
- Filtration coefficient: appr. 20 m/d

Volcanic sand (5^a layer)
The volcanic sands, occupying the lower part of Lower-Quaternary basalt stream are fine-grained, weathered and loose, they can be viewed on Photo 3 in the Annex to this section

- density of 5^a layer sediments: 1800 kg/m³
- rock particle density: 2800 kg/m³
- internal friction coefficient: tg φ = 0.532
- specific cohesion: 0.002 MPa
- admissible foundation design pressure: 0.2 MPa

The coefficient of filtration of this layer is equal to 5 m/d.

Effusive-sedimentary rocks (9^a layers)
These rocks consist of different porphyries (plagioclase), tuffs, tuff-conglomerates, metasandstones and sands. These rocks are thick, highly fractured and hydrothermally altered.

- density of 9^a layer rocks: 2400-2500 kg/m³
- rock particle density: 2700 kg/m³
- static deformation modulus: 25000 MPa
- internal friction coefficient: tg φ = 0.781
- Specific cohesion: 0.05 MPa

The coefficient of filtration of this layer was determined to be 0.1 m/d.

Tectonic splitting zone (11^{th} layers)
The rocks in this zones of stresses as a result of mylonitization were transformed into white flour-like substance (mylonite).

- density of this layer rocks: 1400 kg/m³
- rock particle density: 2600 kg/m³
- static deformation modulus: 50 MPa
- internal friction coefficient: tg φ = 0.577
- Specific cohesion: 0.01 MPa

The coefficient of filtration of this layer is equal to 0.05 m/d.

The main indexes of physical-mechanical, filtration and construction parameters of all above-mentioned rocks are given in the table in the Annex to this section.

4.6

Transport and Access Facilities

4.6 Transport and Access Facilities

4.6.1 Access to the Site

4.6.1.1 Headworks

The construction site of headworks can be reached by road via Stepanavan-Kirov-Gyulagarak-Vardablur-Kurtan, which exists along the right bank of Dzoraget River on the Lori Plateau. The road is asphalted with a length of 28 km and a width of 6 m; it is in acceptable condition. The gradient of the above-mentioned roads is smaller than 8 %. The above-mentioned motor-road crosses the bridge of Kurtan village, which can handle the passage of heavy machinery. Before accessing the headworks a new access road should be constructed. The length of the latter is app. 400 m; the width is 8 m and the gradient is 8 %. Considerable blasting is expected, since the access road needs to pass a slide area between the Plateau and the gorge. The slide is in the river reach, where the river slope changes abruptly and becomes steep down to Dzoraget River.

4.6.1.2 The Waterway

The waterway of Gargar SHPP is planned as an embedded penstock with a diameter of 1.2 m. If manufactured in Yerevan, the penstock pieces should be transported via the intergovernmental motor-road between Yerevan and Stepanavan. The length is 225 km. Before entering to the city of Stepanavan the existing motor-road passes:

- Spitak passes
  Width: 15 m
  Gradient: <10 %
  Cover: Asphalt
- Spitak city bridge
  Width: ca. 20 m
  Design load: 60 t
- Tunnel, which passes through Pushkin passages
  Length: 2 km
  Height: 5.5 m
  Width: 6 m

The penstock pieces from city Stepanavan are transported to the construction site via the same existing road, which shall be used for the construction of headworks. There are no roads along the gorge of Gargar River.

In order to mantle the penstock a temporary access road of 4.5 km length and 8 m width needs to be constructed, which is in accordance to present SniP norms. The gradient of the road is app. 6%.

4.6.1.3 Powerhouse

From Vanadzor railway station the hydro-mechanical equipment of Gargar SHPP shall be transported to the powerhouse site via the existing Vanadzor-Gyulagarak 20 km long and 8 m wide main highway. Approximately 20 km before entering the city of Stepanavan the existing road passes through a tunnel, which has a length of 2 km, a width of 6 m and a height of 5.5 m.

The tunnel allows the passage of heavy cargo such as turbines and generators (the maximum height is 4 m). In this case it would be necessary to stop the cars coming from the opposite direction.

Further the existing Gyulagarak-Vardablur-Kurtan road, which has a length of 8 km, a width of 6 m and a slope up to 4%, passes the bridge of Kurtan village and leads to the planned powerhouse area of Gargar SHPP. The existing road is unpaved; it is necessary to apply harsh covering (asphalt).

It is also required to carry out additional fixing works of the bridge of Kurtan village, since its design load will not allow to transport of heavy loads. Further from Kurtan village up to the powerhouse area of Gargar SHPP the existing 4.5 km long and 6 m wide motor road can be used. In order to approach to the powerhouse site it is required to construct a new paved access road with a length of 0.65 km and a width of 6 m. The gradient of the new access road is appr. 9%.

4.6.2 Transportation of the Equipment

4.6.2.1 By Sea

If the hydro-mechanical equipment of Gargar SHPP is manufactured in Europe it can be transported by shipping via Black Sea up to the seaport Poti, which is in Georgia. During the transportation the heavy-load carrying capacity of the ship must be considered. The existing dimensions of the baggage compartments of the ship should coincide with the dimensions of the heavy equipment.

4.6.2.2 By Rail

Having reached Poti seaport the equipment is further transported by railway. Prior to any transport a preliminary notice on the weight and dimensions of the equipment must be given to the railway department. They provide freight wagons of the train corresponding to the characteristics of the equipment. Poti-Gori-Tbilisi-Yerevan railway with a length of approximately 600 km is presently functioning. It is in good condition and is capable to transport any loads.

In order to transport the hydro-mechanical equipment of Gargar SHPP the Poti-Gori-Tbilisi-Alaverdi-Vanadzor railway with a length of 500 km is used. Further the equipment is transported to the powerhouse area of Gargar SHPP as described below.

4.6.2.3 By Road

There are three layout alternatives to reach the site.

- After the unloading of the cargo at Sanahin (Alaverdi) station the equipment can be transported by trucks on the existing motor-road via Alaverdi-Odzun-Koges-Yagdan-Agarak-Stepanavan. The road has a length of 30 km and a width of 6 m; it is semi-asphalted, at some places destroyed. There are three bridges on the way (at Koges, Yagdan and Agarak villages). It is not possible to transport heavy equipment across these bridges.

  From the city of Stepanavan the existing main road, which is laid along the right bank of Dzoraget River, leads to Kurtan village. The latter is non-asphalted with a length of 28 km and a width of 6 m; it is in acceptable condition. The bridge of Kurtan village can handle the passage of heavy machinery. The gradient of the above-mentioned existing road is 8 %.
- The equipment can be transported up to Tunamyam station by railway; from there the equipment is transported via an existing road, which goes up to the village Dzoragyugh. From Dzoragyugh village a road leads to the village Kurtan. Although the latter road option is relatively short, nevertheless it passes through serpentines with very steep gradient (< 10%). Moreover it is in a very bad condition.
- Further after the unloading of the cargo at Vanadzor railway station, the equipment can be also transported to the powerhouse site of Gargar SHPP by a ca. 30 km long and 6 m wide existing motor-road by trucks via the cities/villages Vanadzor-Gyulagarak-Kurtan.

The third alternative is recommended as the best route for the transportation of the equipment due to following reasons:

- the road planned for the transportation of the equipment is the shortest one
- app. 20 km of this state motor-road is asphalted
- there is only one bridge on the way

The map of all the above-mentioned intergovernmental motor-roads and highways is given in the Annex.

5

Description and Evaluation of Layout Alternatives

5. Description and Evaluation of Layout Alternatives

5.1 Methodology

The analysis for the determination of the most economic project layout was carried out under consideration of the investment costs, the installed capacity and the expected energy production.

The investment costs consist mainly of civil works, hydromechanical and electrical equipment. Cost estimates for civil works, electrical equipment and transmission lines are prepared on basis of local prices. The costs for the hydro-mechanical components are based on information from international turbine manufacturers.

The installed capacity as well as the annual energy production was calculated based on a design discharge in the magnitude of 2 m^3/s. For the present purpose of layout alternative screening, the selected design discharge of 2 m^3/s can be considered as appropriate figure, since it is in-between the mean monthly minimum and maximum flows of Gargar River. The ultimate determination of design discharge is carried out in section 6 through an optimization procedure. Present energy calculations were carried out on basis of the mean daily discharges for the hydrological series between 1958 – 2001. Water demand for irrigation and water supply purposes were also considered in the present calculations.

The waterway with a length of approximately 1.5 – 6 km was considered as the most expensive part of the project of different layout alternatives. Therefore calculations of investment costs for screening purposes concentrated on the determination of costs of the headrace system, such as the tunnel, the covered open channel and the penstock. Other hydropower structures, such as the weir, the powerhouse, hydromechanical as well as electrical equipment, were not used for the present screening of project alternatives, since their difference in costs is considered as insignificant.

5.2 Selected Project Alternatives

The relief of the project area allows the development of the hydropower potential by three different layout alternatives. They differ in terms of the hydraulics as well as the alignment and hydraulic structures of the waterway. The layout alternatives are:

- Layout A: Penstock along the gorge
- Layout B: Open channel along the plateau
- Layout C: Pipetunnel through the plateau

In the Layout A a penstock is planned to be laid in a trench along the river. Due to geological conditions along the gorge the pipeline needs to cross the river several times.

Layout Alternative B was derived from the original planning of the Loriberd Cascade Project, where the weir of the second stage powerhouse was located at Gargar River and the flow was diverted via an covered open channel towards a daily regulation pond placed at the edge of the Lori Plateau near Dzoraget River. In a similar way the waterway is planned in this layout alternative as covered open channel on Lori Plateau, along the village of Kurtan. The flow is diverted to a headpond, where the intake for the penstock is placed. The penstock diverts the water to the powerhouse, which is located in the vicinity of the planned Loriberd Development Project elaborated by the Consultant. The penstock of Gargar SHPP is planned to be constructed as open surface penstock.

The third alternative, Layout C, for the utilization of the hydropower potential is the construction of a pipetunnel through the massif of Lori Plateau. The tunnel diverts the flow of Gargar River towards Dzoraget River. From the low pressure pipetunnel a penstock conveys the water to the powerhouse, which is located at the right bank of Dzoraget River.

For all selected layouts the location of the weir is identical, near the village of Vardablur, approximately 5.9 km upstream the confluence point with Dzoraget River. The location was selected from the previous weir location of the second stage project of the Loriberd Cascade. From this point the headrace of alternatives differs and shall be briefly described in section 5.3. The principle sketch of all developed layout alternatives are shown in the Annex to this section.

5.3 Brief Description of Project Alternatives

5.3.1 Penstock along Gorge

From the weir site it is envisaged to construct an embedded penstock along the riverbed. The conduit will cross the river several times. A new access road for construction works needs to be established. Along the planned alignment of the waterway in the gorge no structures are observed. Furthermore the land is not used for agricultural purposes. The powerhouse is planned to be placed on left bank of the river, 420 m upstream the confluence point.

From the headworks the water is conveyed via a 5525 m long embedded penstock. The alignment of the penstock follows the gorge. An optimization was carried out to determine the most economic diameter for the penstock. The calculations were carried out under following conditions:

- The annual costs of energy losses in the penstock corresponding to different diameters were compared against the annual construction costs of the penstock depending on the relevant diameter under consideration of a capital recovery factor of 10.37%
- The calculation of annual costs of energy losses was based on the present tariff of 0.045 US$/kWh for power generation
- The most economic penstock diameter was evaluated where the total annual cost of the penstock was a minimum. The corresponding graph can be seen in the following figure.

As it can be seen in Figure 5.1 the most economic penstock diameter was determined to be between 1.0 m and 1.2 m. The difference of annual costs between both diameters is marginal. The absolute minimum was reached at 1.2 m and was taken for the present layout.

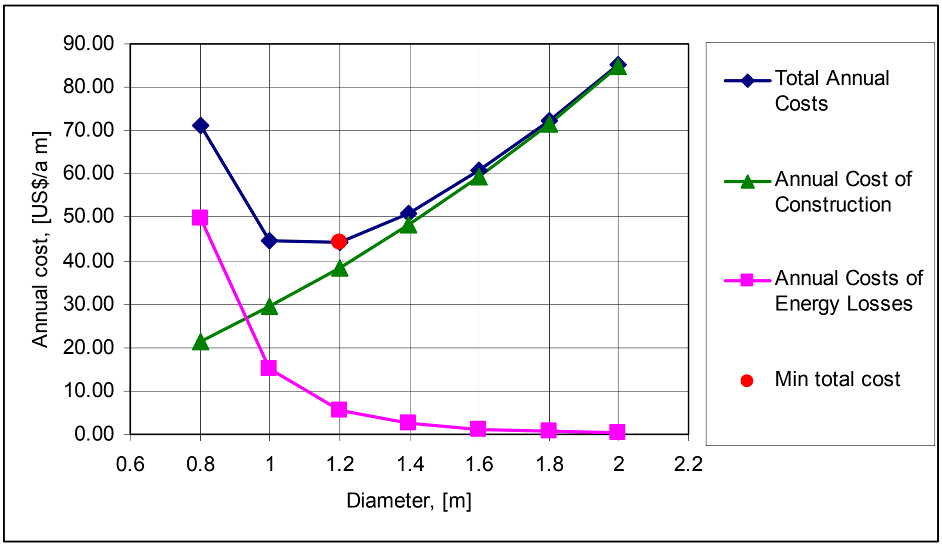

Figure 5.1: Annual cost of penstock depending on its diameter for Layout A

It is planned to lay the penstock in a trench. The width of the trench will be equal to the diameter of the pipe plus 0.5 m for each side. The depth of the trench from the ground surface will be equal to the penstock diameter plus 1 m, which is equal to the depth of soil freezing. The side slopes of the excavated trench are taken as 1:1. A bedding sand layer with a thickness of 20 cm is planned below the penstock. The pipeline shall be mounted and finally the backfill shall be carried out. For the cost calculations the width of the trench for the penstock was considered as 2.2 m and the depth as 2.4 m.

The powerhouse was planned to be located upstream the confluence point at an elevation of about 997 masl. The turbined water shall be spilled back to the river Gargar via a tailrace channel.

With a rated net head of 267 m and the design discharge of 2 m^3/s the operation range was considered as typical application for a Pelton turbine set. The diameter of the turbine as well as its setting was calculated in order to determine the machine hall elevation and construction costs.

The capacity of the plant was determined to be 4.7 MW, and the mean annual energy production was calculated to 17.12 GWh. The main characteristic technical data and corresponding power and energy calculations are given in the Annex. The following table shows the summarized key data of Layout A.

Table 5.1: Key Data of Layout A

Key Data	Layout A
Normal operating level [m]	1275
Turbine axis elevation [m]	997
Gross head [m]	281
Design head [m]	266.7
Design discharge [m^3/s]	2
Turbine type	Pelton
Installed capacity [MW]	4.71
Mean annual energy production [GWh]	17.12
Plant Factor [%]	41

5.3.2 Open Channel along the Plateau

In case of Layout Alternative B the weir site as well as the powerhouse site are identical to aforementioned Layout A. The waterway consists of an covered open channel, which conveys the water from the weir to the headpond. The headpond is located at the edge of the Lori Plateau on an elevation of about 1235 masl. From there an open surface penstock spills the water to the turbine generator set placed in the powerhouse on the left bank of the Gargar River.

The length of the gravity-flow open channel of covered type is 4250 m. The channel was planned with a rectangular section with a width and height of 1.1 m x 1.1 m. The geometrical dimensions were determined by hydraulic calculations under consideration of a design discharge of 2 m^3/s. The wall thickness of the channel was selected to be equal to 40 cm.

At the upper section on a length of appr. 800 - 1000 m the channel is planned to be constructed along the left bank of the Gargar gorge. In this section the slope is almost vertical, the gorge has a height of 20 - 30 m. Rockslides and soil erosion from the banks are expected during storm events. Consequently it is recommended to cover the open channel by a concrete slab in order to protect the waterway from sliding stones and entry of eroded soils.

The alignment of the channel changes the direction towards the Lori Plateau approximately 600 m upstream the village of Kurtan. Along the plateau the alignment crosses fertile lands on a length of approximately 3300 m. Since the land is intensively utilized for agriculture, the open channel shall be covered in this section as well. It is planned to construct the channel in a trench with a width of 2.9 m and a depth of 3.1 m with side slopes in the ratio of 1:1. A sand layer of 20 cm thickness is planned as bedding material for the bottom slab of the channel. After the concrete works are completed, the excavated material is filled back in order to restore the arable lands of the project area. At the end of the covered open channel a headpond is planned.

From the headpond to the powerhouse a penstock shall be constructed, which shall be of open surface type for the upper section and of embedded type for the lower section. The construction type of the penstock depends on the topography of the Gargar gorge. The upper section is almost vertical and has a length of approximately of 85 m. The consecutive reach has a slope of 30º - 45º degree and a length of 150 m. The reach from the foot of the Gargar gorge to the powerhouse is more or less flat. The total length of the penstock is 1150 m. For the determination of the penstock diameter the same methodology was used as described above for Layout A. The results for costs per m penstock were identical with the optimum at 1.2 m.

The powerhouse was planned to be at the same location as in case of Layout A. Again the net head as well as the strongly varying available flows are suitable for the selection of a turbine generator set of Pelton type. The diameter of the turbine as well as its setting was calculated in order to determine the machine hall elevation and construction costs. The turbined water shall be spilled back to the river Gargar via a tailrace channel.

With a rated net head of 244 m and the design discharge of 2 m^3/s the capacity of the plant was determined to be 4.3 MW. The mean annual energy production was calculated to 15.5 GWh. The main characteristic technical data and corresponding power and energy calculations are given in the Annex. The following table shows the summarized key data of Layout B.

Table 5.2: Key Data of Layout B

Key Data	Layout B
NOL [m]	1275
Turbine axis elevation [m]	997
Gross head [m]	281
Design head [m]	243.8
Design discharge [m^3/s]	2
Turbine type	Pelton
Installed capacity [MW]	4.3
Average annual el. energy production [GWh]	15.46
Plant Factor [%]	41

5.3.3 Pipetunnel through Massif of Lori Plateau

In Layout Alternative C the flow is diverted from Gargar River and spilled back to Dzoraget River. The weir site is located at the same place as in case of other alternatives. The powerhouse is planned on the right bank of the Dzoraget River gorge on an elevation of 1077.5 masl, approximately 7 km upstream the confluence point of both rivers. The powerhouse site is opposite the village of Koges.

With this alignment the length of the waterway is reduced considerably to 1375 m only. However the head is also reduced by 84 m, which is approximately 30% of the available head of Layout Alternatives A and B.

Since the elevation of Lori Plateau is approximately 40 m higher than the normal operation level of the weir of Gargar SHPP the construction of an embedded penstock is impossible. Therefore the penstock shall be laid in a tunnel, which is constructed trough the massif of Lori Plateau. The planned pipetunnel has a width and height of 4 m x 4 m. It was assumed, that the area consists of Basalt rock on the entire length of the tunnel. However this assumption implies a certain risk, since the clay cover on top of the Lori Plateau might reach a thickness between 20 - 30 m, as indicated by drilling works during the elaboration of Loriberd Hydropower Development Project by the Consultant. The presence of loamy soil along the tunnel axis would cause extraordinary tunnel costs. Under the prerequisite of the presence of Basalt rock only primary support is foreseen for the tunnel with a concrete bottom slab of 40 cm thickness, placed on top of a 20 cm thick sand-bedding layer.

Similar to previous layout alternatives an optimization analysis was carried out to determine the most economic diameter for the penstock. As it can be seen in Figure 5.2 the most economic penstock diameter was determined to be 1.2 m.

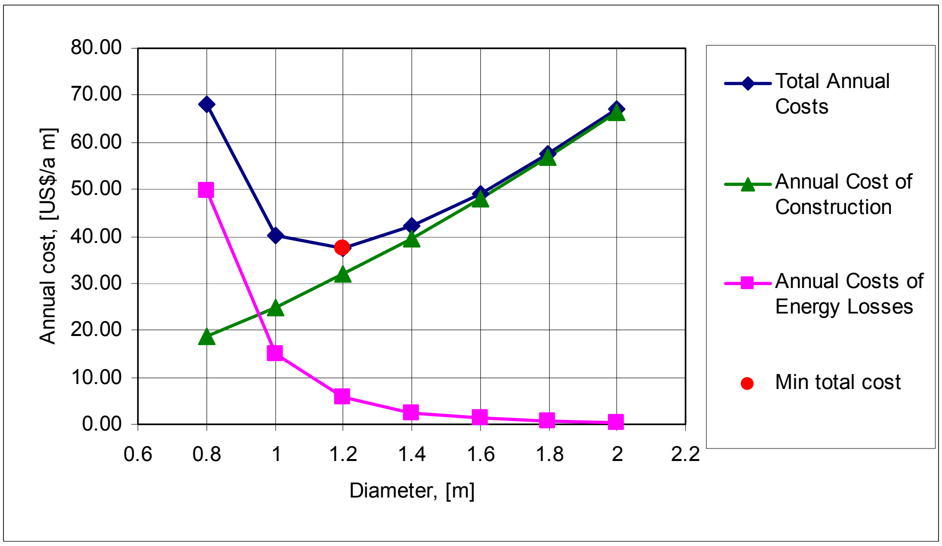

Figure 5.2: Annual cost of penstock depending on its diameter for Layout C

The turbine type was determined under consideration of the rated net head of 191 m and the design discharge of 2 m^3/s. Basically two turbine types, Pelton and Francis might be appropriate for the calculated design parameters. In light of the changing natural flows of the Gargar River a Pelton turbine set with several nozzles might be a more suitable technical solution. The turbined water shall be spilled back to Dzoraget River and join the natural flows of Gargar River at the confluence point between both rivers further downstream. The slight increase of flows from the tailrace to the confluence point in the Dzoraget River is considered to be of minor importance, since the riverbed of Dzoraget River usually has much higher discharges.

The installed capacity at the powerhouse would reach 3.4 MW under a net rated head of 191 m. The mean annual energy generation would be 12.1 GWh. A principle sketch of the layout alternative in the plan view as well as the main technical data are given in the Annex.

Table 5.3: Key Data of Layout C

Key Data	Layout C
NOL [m]	1275
Turbine axis elevation [m]	1080.5
Gross head [m]	197.5
Design head [m]	191.1
Design discharge [m^3/s]	2
Turbine type	Pelton
Installed capacity [MW]	3.37
Mean annual energy production [GWh]	12.15
Plant Factor [%]	41

5.4 Layout Evaluation

5.4.1 Methodology

The objective of the layout evaluation is to identify the most economic layout alternative for the present project. The project screening was carried out in a step-by-step approach, which is described in the following:

In the first step all possible layout alternatives for the Gargar SHPP were identified. These were already described in section 5.3 of this report and are the basis for the present project screening process.

The alternatives were evaluated in terms of costs per installed capacity as well as cost per annual energy production. As mentioned in section 5.1 calculations of investment costs for screening purposes concentrated on the determination of costs of the headrace system, since the difference in other costs of other hydropower structures were considered as insignificant. The corresponding construction costs of the waterways are provided in the Annex to this section.

The ratio between the construction costs and the installed capacity is called the unit cost per MW. The ratio between the construction costs per average annual energy production is the unit cost per kilowatthour without consideration of interest rates. Both provide a first indication which alternative would be the most economical one. The selected alternative needs to be investigated further with a detailed cost estimate and the preparation of the bill of quantities.

The installed capacity and the annual energy production figures for each layout alternative were taken from the calculations carried out and described in section 5.3.

5.4.2 Results

The developed layout alternatives differ from each other not only in construction costs but also in terms of power and energy parameters. Due to this the comparison of all three layouts and the selection of the most economic layout was carried out proceeding from the best economic parameters. Table 5.4 shows the main technical and economical parameters of the comparison between the layouts.

Table 5.4: Comparison of main technical and economical parameters of layout alternatives

Layout	"A"	"B"	"C"
Design discharge, [m^3/s]	2	2	2
Pressure conduit length, [m]	5525	1150	1375
Free flow conduit length, [m]	-	4250	-
Upstream elevation, [m]	1275	1275	1275
Downstream elevation, [m]	994	994	1077.5
Optimum diameter of pipe, [m]	1.2	1.2	1.2
Free flow channel width, [m]	-	1.1	-
Free flow channel height, [m]	-	1.1	-
Pipe tunnel width, [m]	-	-	4
Pipe tunnel height, [m]	-	-	4
Capacity, [kW]	4.7	4.3	3.4
Energy, [GWh]	17.1	15.5	12.1
Cost, [$US]	3502816.5	3874734.8	2888070.3
Specific cost per kWh	**0.20**	**0.25**	**0.24**
Specific cost per kW	**743.7**	**899.4**	**856.0**
Construction time, [year]	2	2	2
Cost Distribution 1-st year	40%	40%	40%
Cost Distribution 2-nd year	60%	60%	60%
Tariff, [$US]	0.045	0.045	0.045
Operation & repair cost	1.5%	1.5%	1.5%
Discount Rate	10%	10%	10%
Life time, [year]	30	30	30
IDC Factor	0.04	0.04	0.04
CRF	0.11	0.11	0.11
Operation & repair cost, [$US]	52542.2	58121.0	43321.1
Investment Cost including IDC, [$US]	3642929.2	4029724.2	3003593.1
Revenues, [$US]	7261974	6563521.1	5152258.7
Cost & O & R, [$US]	3998127.8	4422636.7	3296454.2
B/C	1.8	1.5	1.6
Annual cost, $	438981.4	485591.1	361940.0
DUC per 1kWh	**0.026**	**0.031**	**0.030**

Under consideration of the installed capacity, which is given in Table 5.4 for each layout alternative, the specific costs were calculated. The same table shows the investment costs per installed Megawatt. The figures vary from 744 US$/kW to 900 US$/kW. However it is important to mention, that these costs cover the waterway only. The calculation basis of the waterway costs is presented in the Annex to this section. Layout Alternatives A and B have the highest investment costs per MW. The most attractive layout was the penstock along the Gargar River with specific costs in the range of 744 US$/kW.

The second criterion used during the layout screening process was the specific generation costs. The investment costs and the mean annual energy production were used to calculate the specific parameter. For the present project the specific dynamic unit costs were determined, which do take into account the discount rate as well as the lifetime of the project. A discount rate of 10% and a lifetime of 30 years were assumed for the present screening process of Gargar SHPP. Furthermore operation and maintenance costs as well as interest during construction was considered. Again it is of mayor importance to notice, that the DUC do include only the waterway, which can not be compared to the final DUC of Gargar SHPP discussed in section 13 of the present Feasibility Study.

Table 5.4 also includes the mean annual energy production of all developed layouts. The mean annual energy calculations vary between 12 GWh and 17 GWh.

The specific dynamic unit costs for power generation are governed by the differences in costs and energy. The costs of the open channel option and the pipetunnel option are similar; the penstock is the least cost layout alternative. As a consequence the least specific dynamic unit costs are given for the penstock alternative laid along the gorge. Layout B and C have appr. 0.4 – 0.5 cents/kWh higher costs than the penstock development.

Under consideration of both specific cost parameters calculated above the most attractive layout alternative was determined to be the penstock. The results showed, that this layout is the most economic one, in terms of investment per installed capacity as well as dynamic power generation costs.

6
Optimization of Design

6. Optimization of Design

6.1 Optimization of Selected Layout

After the selection of the penstock development along the Gargar River, Layout Alternative A, as the most economic waterway a further optimization was carried out. The aim of the present optimization was to identify the final weir site location in order to determine the best economic solution for the development of the hydropower potential of Gargar River.

For this reason the river reach between the previous selected weir site A and the powerhouse was investigated in more detail. As it can be seen in Figure 6.1 the longitudinal river slope increases with the flow direction. The total length of the investigated river reach is approximately 6.2 km long. The various river reaches and their corresponding slopes can be summarized as follows:

- River reach 0.000 km – 1.987 km: head 30.3 m, slope 1.5 %
- River reach 1.987 km – 3.438 km: head 28 m, slope 1.93 %
- River reach 3.438 km – 5.835 km: head 221 m, slope 9.2 %
- River reach 5.835 km – 6.255 km: head 12 m, slope 2.8 %

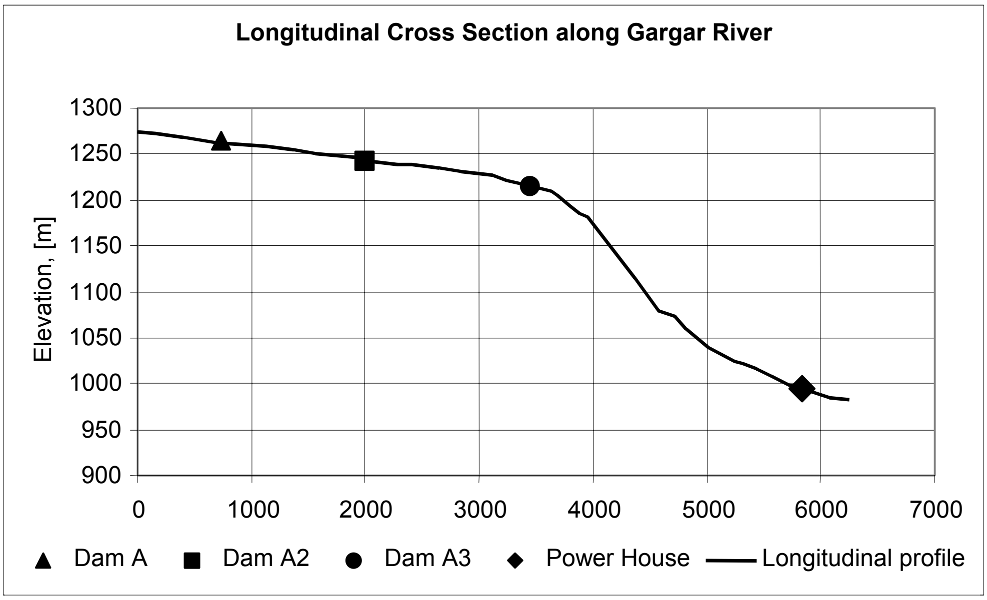

Figure 6.1: Location of weir sites for Layout A along Gargar River

On basis of the investigated river reach a total number of three weir locations for the penstock development along the gorge were identified, namely A, A_2 and A_3. They are shown in Figure 6.1. The technical data of the different weir locations are summarized in Table 6.1.

The most upstream location A is identical to the weir site mentioned in section 5 of this report. It was derived from the original planning of the Loriberd Cascade Project. Access conditions to the weir site as well as to the gorge between weir site A and A_2 are unfavorable. The construction of a new access road in deep excavations may be required since the gorge consists of almost vertical slopes along the whole length of the first section.

The second weir location A_2 is was determined due to good access conditions. The existing bridge over Gargar River at the village of Kurtan can be used for the access of the second weir site, which was planned to be located 200 m downstream the bridge. The river width at the weir location is approximately 30 m wide. The river reach between the weir sites A_2 and A_3 however is characterized by steep slopes, access is considered to be more difficult than to the headworks. The depth of the gorge varies between 20 – 40 m, the slope consists mainly of basalt rocks, partly covered by soil. A footpath along the right side bank exists, which may be used as a basis for the establishment of an access road for construction works on the penstock.

The third weir location is governed by the topographical conditions of the river Gargar. The location is given by the fact, that the river slope increases suddenly from 2 % to 9 %. Consequently the section between the weir location A_3 and the powerhouse provides the maximum head along the river on the shortest distance. Access to the weir site can be established by the construction of a new short access road from the right slope of the river. The access road diverts from the existing asphalted road on the plateau on the right side of Gargar River. The length of the access road is 300 m.

The drawings for all Layouts A, A_2 and A_3 are attached to the Annex 5.

Table 6.1: Comparison of technical data for different weir locations

Key Data	Layout „A“	Layout „A_2“	Layout „A_3“
NOL [m]	1275	1243.5	1215
Turbine axis elevation [m]	997	997	997
Gross head [m]	281	249.5	221
Design head [m]	266.7	238.4	212.6
Design discharge [m^3/s]	2	2	2
Waterway Length [m]	5525	3848	2397
Waterway Costs [MUS$]	3.503	2.439	1.519
Turbine type	Pelton	Pelton	Pelton
Installed capacity [MW]	4.71	4.21	3.75
Mean annual energy production [GWh]	17.12	15.25	13.56
Plant Factor [%]	41	41	41

Comparing the different weir locations with each other following conclusions can be given. The shift of the weir site from A to A_2 results in a reduction of the waterway length by 1677 m, the loss of gross head is equal to 31 m. This gives a reduction of mean annual energy in the magnitude of 1.87 GWh, which is equal to a net present value of appr. 0.793 MUS$ (discount rate 10%, lifetime 30 years). In comparison to this the reduction of costs for the waterway are 1.064 MUS$. Since the reduction of waterway costs is greater than the loss of revenues through power generation, weir location A_2 is more economical than A.

The comparison between weir locations A_2 and A_3 can be summarized as follows. The waterway length is reduced by 1451 m with the consequence of costs reduction in the order of 0.920 MUS$. The loss of energy would reach 1.7 GWh, which is equal to a net present value of approximately 0.721 MUS$. Consequently weir location A_3 is more economical than the weir location A_2.

The calculation of the waterway costs of layouts A_2 and A_3 is presented in the Annex to this section. The civil costs for the construction of headworks and powerhouse were not considered in the calculation since the costs for headworks and powerhouse would differ insignificantly.

Finally it can be stated, that the weir location A_3 is determined as the most economic weir location for the development of a penstock solution along the river gorge. This layout is the basis for any further optimization procedures discussed in the next paragraph and detailed costs estimation carried out in section 11 of the present report.

6.2 Selection of Design Discharge

6.2.1 Results

After the determination of final and most economic layout the design discharge is optimized. The optimization of the design discharge is based on maximizing the benefit cost ratio. The ratio is equal to the benefits from power production divided by the total construction costs. The methodology requires the estimation of benefits and costs and the selection of evaluation parameters.

For this purpose technical and economical calculations were carried out for various design discharges. The discharge varied between 1.0 m³/s - 3.4 m³/s in steps of 0.2 m³/s. For each discharge the capacity, mean annual energy production as well as costs of the scheme including turbine-generator set were determined.

The calculation of civil costs included headworks, the waterway as well as the powerhouse. The works for the construction of headworks including weir, gravel trap, fishpass and sandtrap of appropriate size were considered. The calculation of the waterway included works of trench excavation, blinding layer, steel pipeline and trench backfill. Furthermore the civil costs for the powerhouse were taken into account. While the cost calculations did not consider the variability of headworks the powerhouse costs were estimated in accordance with the optimum turbine diameter and number of turbines for various design discharges. The variability of the headworks is considered as insignificant in comparison with the costs of the waterway and hydro mechanical equipment. The results of costs calculations are presented in the Table 6.2.

The economic parameters are shown in the Table 6.3. Figure 6.2 shows the B/C curve depending on the design discharge and Figure 6.3 shows the curve of dynamic unit cost per 1 kWh generated energy depending on the magnitude of the design discharge.

For the economical calculations the present tariff of 0.045 $US/kWh for small hydropower development projects was used. The calculations are carried out on basis of a lifetime of 30 years and a discount rate of 10 %. The total investment costs included physical contingencies in the magnitude of 10% of the direct costs. Other costs items, such as environmental mitigation costs, preliminary and general, engineering and supervision and duties were not considered for the present level of development. Therefore the calculated dynamic unit costs cannot be compared to the final financial calculations carried out in section 13 on basis of a detailed and comprehensive cost estimation.

Table 6.2: Results of Costs and Energy Calculations

Design Discharge [m^3/s]	Optimum diameter, [m]	Civil Works, [TUS$]	Hydro-mechanical Equipm., [TUS$]	Electrical Equipm., [TUS$]	Direct cost, [TUS$]	Total cost, incl. Phys. Cont. [TUS$]
1.0	0.8	1339	936	93	2369	2605
1.2	0.9	1455	993	98	2547	2801
1.4	0.9	1466	1173	102	2740	3014
1.6	1	1595	1262	107	2964	3261
1.8	1	1605	1319	111	3035	3338
2.0	1.1	1726	1399	115	3240	3564
2.2	1.1	1736	1677	119	3532	3885
2.4	1.2	1886	1764	122	3772	4150
2.6	1.2	1896	2028	126	4051	4456
2.8	1.2	1907	2084	130	4122	4534
3.0	1.3	2049	2203	135	4386	4825
3.2	1.3	2059	2326	139	4524	4976
3.4	1.3	2088	2448	143	4679	5147

Table 6.3: Results of Economic Calculations

Design Discharge [m^3/s]	Capacity, [kW]	Energy, [GWh]	Revenues [TUS/a]	Benefits [TUS$]	Costs [TUS$]	B/C [-]	DPC per 1kWh [US$/kWh]
1.0	1731	9.51	428	4034	2605	1.548	0.0291
1.2	2100	10.29	463	4365	2801	1.558	0.0289
1.4	2418	11.40	513	4836	3014	1.604	0.0280
1.6	2799	12.09	544	5129	3261	1.573	0.0286
1.8	3118	12.70	572	5387	3338	1.614	0.0279
2.0	3506	13.51	608	5731	3564	1.608	0.0280
2.2	3829	14.02	631	5947	3885	1.531	0.0294
2.4	4221	14.46	651	6134	4150	1.478	0.0304
2.6	4548	14.86	669	6304	4456	1.415	0.0318
2.8	4869	15.21	684	6452	4534	1.423	0.0316
3.0	5273	15.74	708	6677	4825	1.384	0.0325
3.2	5600	16.03	721	6800	4976	1.367	0.0329
3.4	5922	16.29	733	6910	5147	1.343	0.0335

Table 6.2 shows, that the mean annual energy production varies between 9.5 GWh – 16.3 GWh. This is equal to annual benefits from power generation between 0.428 MUS$/a – 0.733 MUS$/a. Under consideration of a lifetime of 30 years and a discount rate of 10%, which is common in hydropower development, the revenues are between 4.033 MUS$ - 6.910 MUS$.

The investment costs, which include only physical contingencies in the magnitude of 10% at present stage of works, range between 2.60 MUS$ - 5.15 MUS$ for discharges between 1.0 m³/s - 3.4 m³/s.

Consequently dynamic unit costs reach the minimum at this discharge with 0.0279 US$/kWh. Consequently the benefit-cost (B/C) ratio for the selected design discharges varies between 1.34 – 1.61. The maximum is reached at a design discharge of 1.8 m^3/s. Both economic parameters are shown in Figure 6.2 and 6.3.

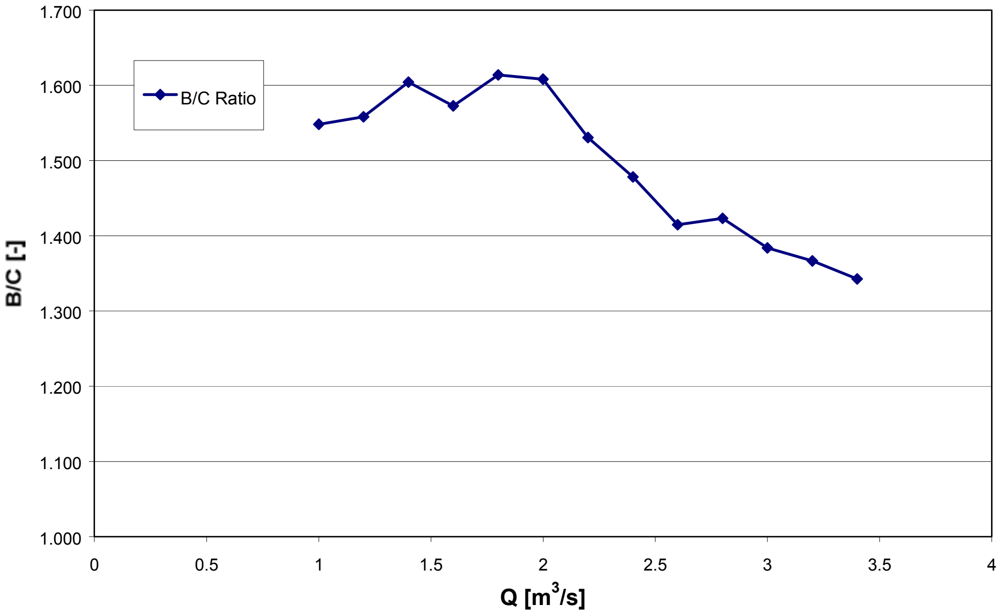

Figure 6.2: B/C ratio depending on the design discharge

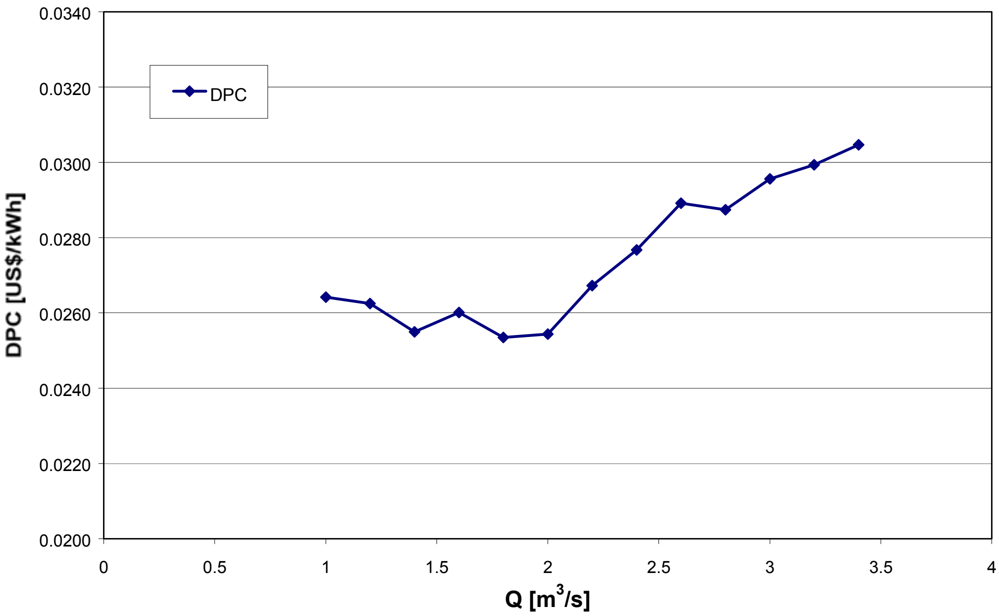

Figure 6.3: DUC depending on the design discharge

Both curves are characterized by a certain scatter, the performance of the graph can not be considered as smooth. This is mainly caused by the differences in the most economic diameters of the penstock, which have steps of 0.1 m difference.

It is recommended by the Consultant to develop the scheme for a design discharge of 1.8 m^3/s. The installed capacity reaches approximately 3.1 MW, the mean annual energy generation is equal to approximately 12.7 GWh. The final main technical data of Gegharot SHPP are given in section 7 of this report.

6.2.2 Comments

The selection of the design discharge determined the layout and the design of a hydropower scheme. It fixes the installed capacity, the expected mean annual energy generation as well as the costs of the scheme.

On basis of the benefit-cost ratio graph different point of views shall be discussed here. Except from the optimum design discharge in the magnitude of 1.8 m^3/s the benefit cost graph in Figure 6.2 shows two more peaks, at Q = 1.4 m^3/s and at Q = 2.8 m^3/s. The corresponding B/C ratios are 1.60 and 1.42 respectively. The phenomenon is also reflected at the DUC, which are 0.028 US$/kWh and 0.032 US$/kWh. While the lower design discharge of 1.4 m^3/s reaches nearly the same dynamic productioon costs as the optimum, the greater design discharge of 2.8 m^3/s has 0.4 UScent/kWh higher generation costs than in case of the optimum. It shows, that both points are not the most economic solutions for the site, however since the difference to the optimum point is relatively small they might also be considered for the development of the hydropower potential of the Gargar River. Other criteria apart form benefit cost may play a role for the decision maker/investor for the selection of the design discharge. The two different types of investors and their sources of financing distinguished here are

- Public Financing through Government
- Private Financing through Private Investors

Since hydropower is the most promising renewable energy resource in Armenia, the Government of Armenia (GoA) promotes hydropower development in order to reduce the dependency on fuel imports. Furthermore it is intended to provide alternative capacity and energy for the final closure of the nuclear power station Medzamor. In light of both reasons a higher design discharge of 2.8 m^3/s might be more appropriate from Governments point of view. Thereby the capacity would be increased by 1.75 MW (56%) and the annual energy production by 2.51 GWh (20%) compared to the optimum of 1.8 m^3/s. However costs would also be increased by appr. 1.2 MUS$, which is 36% more than at the optimum point.

The selection of a higher design discharge causes additional costs, especially for the turbine generator set with high investment costs. These additional costs might create difficulties for a private investor in financing the project or even in the cash flow of the first years of the development of the scheme. From his point of view a smaller investment with a short payback period might be of more interest. For this reason he might consider the lower design discharge in the magnitude of 1.4 m^3/s. In this case the capacity would be decreased by 0.7 MW (22%) and the annual energy production by 1.3 GWh (10.2%). Consequently investment would be only 3.01 MUS$, which is 9.7% less than at the optimum point.

The selection of any design discharge apart from the optimum means, that one characteristic of the hydropower scheme is improved while another characteristic is made worse. A higher design discharge than the optimum improves the energy production but reduces the profitability of the project. A lower design discharge reduces the investment costs but also reduces the energy production. The potential of the river is not fully utilized for the economy of Armenia and the profitability is reduced as well.

For the present Feasibility Study the Consultant recommends to take the maximum benefit cost ratio for the development of the hydropower potential of the Gargar River. Thus a design discharge of 1.8 m^3/s was selected.

However, it should be pointed out, that there are basically two ways in order to find a compromise between the interest of the Government in producing more energy and the private investor in receiving the largest profit. Both ways are closely connected with the granting of construction and operating licenses for the small hydropower schemes, which is in the control of the Regulatory Commission of RoA.

In the first approach the Regulatory Commission might tender the hydropower project. Different tenderers are expected to submit proposals in form of a Pre-Feasibility Study for the development of the scheme, which shall include the basic technical and economical parameters of the project, such as installed capacity, expected mean annual energy production as well as estimated costs. The bidder, who proposes a scheme with a higher energy production at the given tariff for SHPP's gets the construction and operating license of the Regulatory Commission.

In the second approach the Regulatory Commission itself should determine either a range or a minimum design discharge for the development of the natural hydropower potential. Thereby a compromise between the interests of the Government of the Republic of Armenia and the future private investors is ensured. The fixation of the design discharge of the hydropower project shall be based on the analysis of a benefit cost curve as shown in Figure 6.2. The decision shall be taken by technical and economic experts in the Regulatory Commission.

Both ways described above ensure an economical development of the hydropower potential in Armenia under private investments. However, in order to provide the Regulatory Commission with these rights the legal framework has to be amended in future.

7

Power and Energy Potential

7. Power and Energy Potential

7.1 Availability Discharge

7.1.1 Observed and Natural Flows

For power and energy calculations mean daily discharges for the time series 1958-2001 were used. The figures were derived from hydrological yearbooks and were transferred by the Consultant to digital form in order to carry out calculations by computer application. For the calculations the gauging station at the village of Kurtan was used. The gauiging station is located appr. 500 m upstream the selected weir location, therefore no conversion ratios were applied for the estimation of flows at the weir site.

The natural flow at the gauging station was restored by historical data for irrigation and water supply available from hydrological yearbooks. For estimation of available flows for power generation future demands for irrigation as well as water supply were considered from the information obtained by the RA Committee of Water Resources, as described in the following paragraphs.

7.1.2 Water Demand for Irrigation

On request from the Consultant data on future water demand for irrigation from the Gargar River were submitted by the RA Committee of Water Resources. Table 7.1 shows the total future demand of irrigation waters taken off by all irrigation canals located upstream the weirsite of Gargar SHPP. More detailed information can be found in the Annex to this section.

Table 7.1: Future water demand for irrigation at Gargar SHPP

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Irrigation	-	-	-	-	-	0.345	0.345	0.345	-	-	-	-

7.1.3 Water Demand for Water Supply

The Committee of Water Resources of RA was also asked by the Consultant to provide data for the future demand for water supply upstream the planned weir site of Gargar SHPP. The information received from the Authority was, that the future demand would be nil. However the latest data available from hydrological yearbooks indicated a constant demand for water supply in the magnitude of 0.0078 m^3/s. The figures were constant for several years.

In analogy to the approach used by the Consultant for Loriberd Hydropower Development Project the future water demand was estimated. The approach is based on the assumption, that the population is more or less constant over the next 20 years, due to following population development.

- 2004 – 2009: – 1.0%
- 2009 – 2014: + 0.5%
- 2014 – 2024: + 1.0%

The approach is in accordance with the KfW report, which was approved by the local authorities, such as the State Committee of Water Resources, etc.

At several site visits and talks with local Authorities at Stepanavan from the Consultant it could not be confirmed, that the , that the water supply was nil in recent years. Therefore a constant value of 0.008 m^3/s was taken for the calculations. Table 7.2 shows the distribution of future demand.

Table 7.2: Future water demand for water supply

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Irrigation	0.008	0.008	0.008	0.008	0.008	0.008	0.008	0.008	0.008	0.008	0.008	0.008

7.1.4 Minimum Environmental Flow

Apart from the future water demand for irrigation as well as water supply, the minimum ecological flow is remaining constantly in the original riverbed.

The Government of Armenia has recently issued a new resolution on determination of minimum ecological flow for Armenian surface waters. As already mentioned in section 3 of the present report the decree N 592-N published on 22 June 2003 replaces point 14 of chapter 5 of the article 121 of the RA Water code.

In accordance with the decree the amount of ecological discharge is calculated in the section of surface flow for each water resource by the 75 % of the 95% annual observation probability for each water resource. Applying the norm to the available hydrological series a minimum ecological flow of 0.04 m^3/s was calculated for the Gargar River. The determination of this magnitude is shown on the probability curve, the data are provided in the in Annex 7.

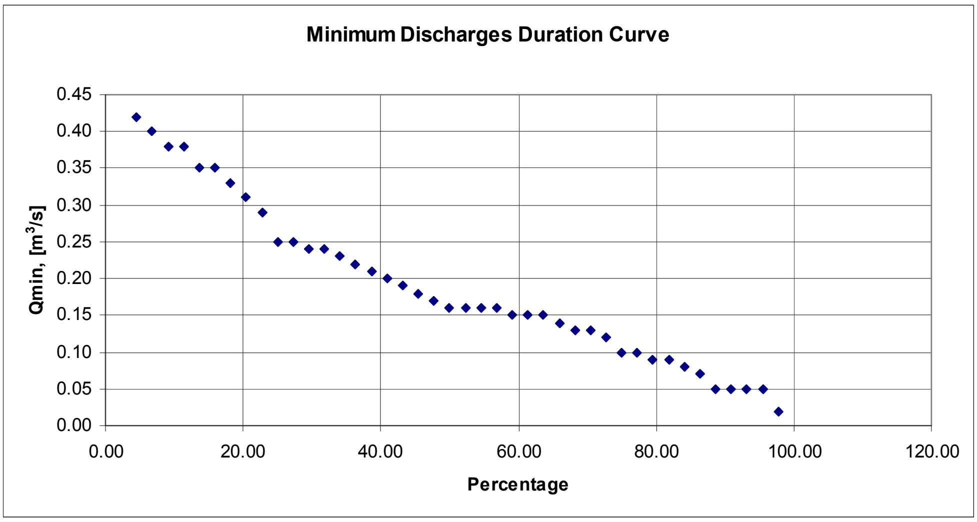

Figure 7.1: Probability Curve of minimum flow at Gargar River

The Ministry of Nature Protection of the RA was informed by the Consultant, that the present Armenian norm does not reflect the state of the art. On international basis the minimum ecological flow is recommended to be changing in accordance with the natural flow dynamics of the river. Moreover the Ministry was informed, that the calculated minimum ecological flow is relatively low compared to other international standards. The Ministry appreciated the recommendations given by the Consultant, however insisted on the application of the present Armenian norm. The Consultant followed this directive.

7.2 Net Head

The net head depends on the varying head- and tailwater levels as well as the hydraulic losses of the waterway. The gross head of Gargar SHPP was calculated to 223 m.

The headwater level has been calculated under consideration of the foreseen shape of the crest of the weir at the headworks. At design discharge the normal headwater elevation is at 1213.0 masl. Daily variations of the upstream water level were not considered in the present study.

At Gargar SHPP the installation of a Pelton turbine set is foreseen. The tailwater level is calculated on basis of the measured cross sections and corresponding water levels of the year 2004. The elevation of the river bottom in the powerhouse site is 989.6 masl. The discharge rating curve at the powerhouse location is shown in the Annex 4.3 of this report.

The turbine axis of the Pelton wheels was set to 993.5 masl, considering a minimum setting of 1.5 m. The maximum water level at the design flood with a return period of 100 years is equal to an elevation of 992 masl at the powerhouse.

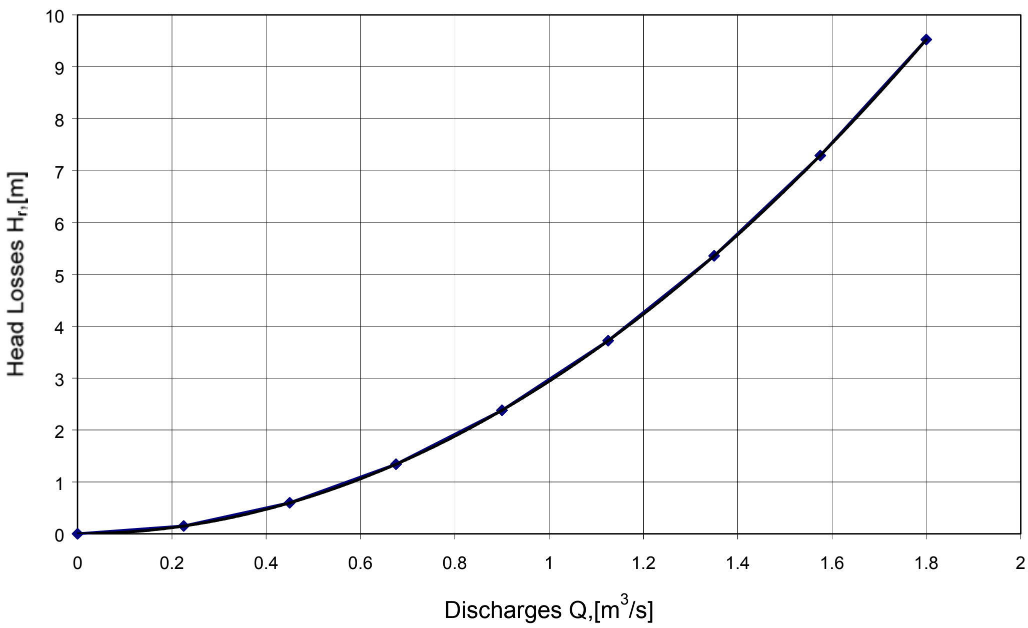

Figure 7.2: Head losses as a function of the discharge

For the determination of the net head, head losses at design discharge were calculated. Figure 6.3 shows the head losses depending on the discharge. The detailed hydraulic calculations are shown in Annex 7. The daily head values were determined by deduction of daily head losses from daily gross head and the setting elevation of the turbine runner. The operating head varies in the range between 210.0 - 219.5 m, the design head is equal to 210 m.

7.3 Efficiency of Equipment

It was planned to install turbine and generator sets from international Market as approved by the Ministry of Energy. Equipment supplied from turbine manufacturers from Western Countries was expected to have higher efficiencies than equipment form eastern Europe and Russia.

The reliability and lifetime of the equipment was expected to be higher, outage times due to maintenance works are considerable smaller. With less outage times the power generation shall reach calculated mean annual figures.

The overall efficiency was calculated based on:

- Generator efficiency: 98%
- Transformer efficiency: 99%
- Turbine efficiency: 88% at design discharge

The Pelton turbine runner can be considered as ideal for varying discharges as in case of the Gargar SHPP. Due to the possibility to operate with a limited number of jets of the complete turbine, the natural available discharges are utilized to a maximum extent. Efficiencies are expected to reach still about 85% at 20% of the design discharge for a turbine with one jet. One turbine set with four jets can operate down to discharges in the range of 5% of the design discharge.

7.4 Results

The power and energy calculation were carried out on a daily basis with help of historic mean daily discharge data and average daily discharge data for available head.

The summary of mean energy calculations for the years 1958 – 2001 is shown in the following tables. The details are attached to Annex 7. The technical data, capacity and energy calculations for the recommended run-of-river plant Gargar SHPP are enclosed summarized:

Design capacity P_d: 3.16 MW
Mean annual energy E_m: 12.19 GWh
Plant factor: 44 %
based on following design parameters:
Design Discharge Q_d: 1.8 m^3/s
Net Head at Q_d H_n: 210.0 m
Type of Turbines: Pelton
Number of Units: 1

Number of Jets: 4

The following tables and graph show the distribution of the energy output of Gargar SHPP. All the basic data and main results of the power and energy calculations are shown in the Annex to this section. Tables of mean monthly power and generated energy are also presented in the Annex.

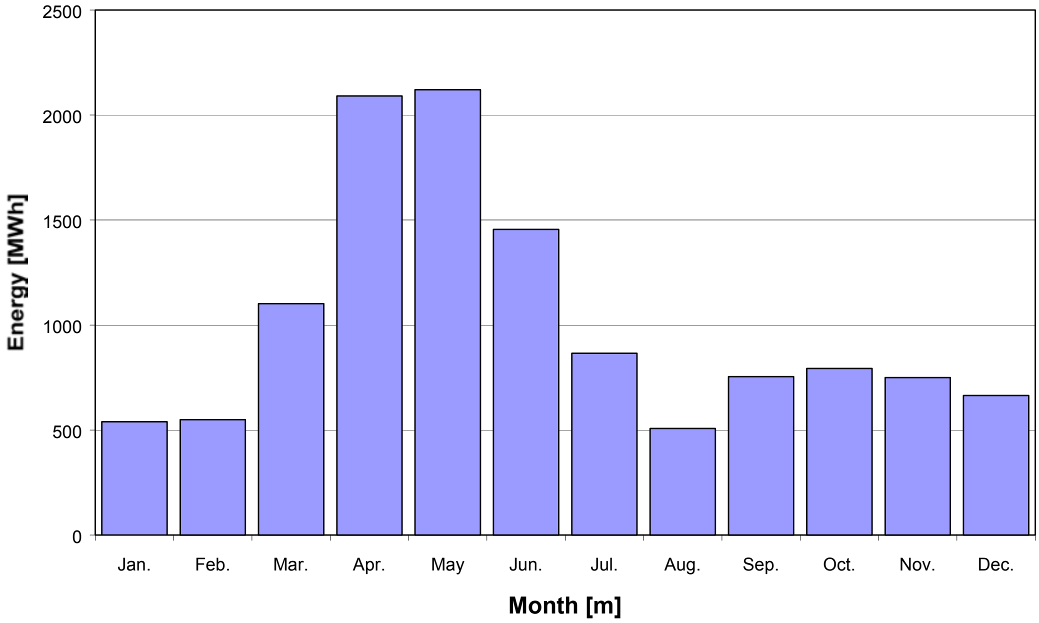

Figure 7.3: Monthly Energy Distribution

Table 7.3: Yearly Distribution of Energy [MWh]

Year	Annual energy	Year	Annual energy
-	MWh	-	MWh
1958	5958.6	1982	11284.8
1959	16121.9	1983	12749.1
1960	10971.7	1984	10821.8
1961	4830.4	1985	9642.6
1962	6639.1	1986	11315.3
1963	15761.0	1987	12150.6
1964	12550.7	1988	19652.1
1965	10726.5	1989	10948.9
1966	9287.4	1990	13284.7
1967	13381.0	1991	11595.5
1968	15389.6	1992	27701.4
1970	7339.2	1993	13825.1
1971	8224.7	1994	13615.1
1972	12382.6	1995	14347.1
1973	11230.8	1996	13018.8
1974	12331.2	1997	16381.9
1975	11037.6	1998	14560.5
1976	13075.9	1999	14645.5
1977	9920.4	2000	11903.1
1978	13101.6	2001	10525.1
1979	12067.3	Mean	12194.8
1980	7405.7	Max	27701.4
1981	10671.3	Min	4830.4

Table 7.4: Monthly Energy Distribution [MWh]

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Energy	540.	549.3	1102.	2090.	2120.	1455.	866.2	508.0	754.8	793.7	749.3	665.0	12194.

8

Civil Design

8. Civil Design

8.1 Final Project Layout

Based on the analysis of alternative layouts for Gargar SHPP in sections 5, the penstock development along the Gargar Gorge was considered as the most economic solution for the development of the site. The further optimization carried out in section 6 determined the most appropriate and economic weir location.

The overall project layout in plan and section view are shown in Annex 8. The project comprises the following principal features:

- Headworks with appurtenant structures

The headworks convey the water to the headrace system. No daily storage device was foreseen at the headworks. The weir is of Tyrolean Type. The water is conveyed from the weir to a gravel trap and further on to a sandtrap, which are both located at the right side of the weir. On the left side of the headworks a fishladder is placed. The intake to the pressure conduit is located at the end of the sandtrap.

- Embedded Penstock

The embedded penstock is constructed along the Gargar River from the headworks to the powerhouse. The penstock crosses Gargar River three times. It starts on the right bank and ends near the powerhouse on the left bank. Due to unfavorable topographical conditions there is no possibility for the installation of a surge chamber. The penstock should be embedded, since sliding of weathered rocks might occur from the steep Gargar Gorge during the lifetime of the project.

- Powerhouse with appurtenant structures

The open surface powerhouse is located on the left bank side of Gargar River. The location is approximately 400 m upstream the confluence point between Gargar and Dzoraget Rivers. The powerhouse accommodates one generating unit of Pelton type with four jets. The governing and control system for the operation of the equipment as well as the electrical equipment are also mounted in the powerhouse.

- Tailrace

The tailrace is planned for the conveyance of turbined water back to the river. The channel starts below the Pelton turbine set in the powerhouse and is laid up to the river. The channel has a trapezoidal shape.

The following paragraphs contain the engineering description of the main structural components of the project. The design parameters, assumptions and results of analysis are presented in this section. Furthermore the river diversion structures required during the construction of the weir and appurtenant structures are described.

8.2 Headworks and Appurtenant Structures

8.2.1 River Diversion during Construction

River diversion structures have to be provided for the construction of the headworks. The diversion of the river shall be planned in the following sequence:

- Construction of a cofferdam on the right side of the riverbank in order to prepare a dry construction pit for the civil works on the sandtrap.
- Construction of the sandtrap including its flushing channels, however the upper face wall of the sandtrap should be kept open.
- The construction of an open diversion channel from the cofferdam site to the upper face wall of the sandtrap. In accordance with Armenian Standards the channel shall be designed for a design flood with a return period of 10 years.
- The construction of cofferdam on the left side of the river is required in order to direct the river flow via the channel into the sandtrap and further downstream. Thereby a dry construction pit for the works on the weir, the apron, stilling basin, armor, gravel trap and fish ladder is ensured.

8.2.2 Engineering Geological Conditions

The head structures of Gargar SHPP including weir, sandtrap and gravel trap are located in the flood-plain section of the Gargar River. The site is south of the village Kurtan, 1.5 km downstream the bridge, which crosses Gargar River on the Stepanavan-Alaverdi motorroad.

Taking into consideration the uncovering of bedrocks on slopes as well as the inaccessible terrain for the implementation of drilling works, the geological map in 1:1000 scale was drawn for the headworks region. On the basis of the map the geological-lithological sections were prepared.

At this reach the river Gargar valley shows steep slopes. The weir site consists of lower-Quaternary doleritic basalts (5^{th} layer), which are covered with young alluvial-coaluvial sediments of 2^{nd} layer in the flood-plain section of Gargar River. According to upstream boreholes the thickness of alluvial-coalluvial sediments at the weirsite reaches 6 - 7m.

The compact basalts are almost fully outcropping on the slopes. These basalts are porous and fractured to different degree. The alluvial-coalluvial sediments of Gargar River are composed of pebble and gravel from different bedrocks, as well as big boulders of basalts with silty-clay and silty-sand filling to 30%.

The sandtrap as well as the gravel trap sites also consist of lower-Quaternary basalts of 5^{th} layer, which are mainly covered with alluvial-coalluvial sediments of 2^{nd} layer. The thickness of latter is 4.5 m.

The ground waters on the headworks site are connected with the riverbed flow. According to the chemical composition these waters are sweet, they contain hydrocarbonate-calcium. The underground waters are not expected to cause any corrosion of concrete.

Finally it can be concluded, that the engineering-geological conditions at the headworks site are quite favorable.

8.2.3 Design

8.2.3.1 Tyrolean Weir and Intake

The intake structure is located 1500 m downstream the bridge of Gargar River at the village Kurtan. The coordinates of the weir site are defined by following reference points:

Reference Point	X [m]	Y [m]
A	129769.09	108348.73
B	129789.76	108363.91

The length of the weir crest is 18.5 m. It is planned for a design flood discharge with a return period of 100 years, which is equal to 99.1 m^3/s. The height of the weir is equal to 2.1 m. The upstream wall of the weir is vertical, the downstream wall is inclined by 45º degree to the horizon. The elevation of the Tyrolean weir crest is 1213.7 masl, at the third spilling section the elevation is 1214.5 m. The weir foundation elevation is 1210 m. The top elevation of the piers is 1216.4 masl. A footbridge with a width of 2 m is planned on top of the piers.

An apron with a length of 6 m is planned to be constructed upstream the weir. The width of the apron is equal to the entire length of the headworks, which consist of the gravel trap, the sandtrap, the weir body and the fishpass.

The energy dissipation is ensured by a stilling basin with a length of 12.0 m. The width of the stilling basin is equal to the length of the weir and the gravel trap. The surface elevation of the stilling basin is 1211.2 masl. For the transition of the flow between the stilling basin and the riverbed an approximately 20 m long riprap is planned. The elevation of the riprap is 1212.2 m.

The Tyrolean type intake has two bays, each 5.5 m long, 1.5 m wide and 0.3 - 1.3 m deep. The chamber of the intake is covered with an 10% inclined rack. The space between steel bars is 6 mm. The bars have a circular section.

The hydraulic calculations determining the main dimensions of the headworks are enclosed to the Annex of this section.

The main data are as follows:

Weir length, m	18.5
Number of bays	3
Length of one weir section, m	5.5
Flood design discharge, m^3/s	99.1
Design discharge of Tyrolean intake, m^3/s	1.8

Weir crest elevation at the Tyrolean intake, masl	1213.7
Weir crest elevation at spilling section, masl	1214.0
Elevation of the highest water level, masl	1215.8
Elevation of the weir bottom, masl	1210.1
Elevation of the weir crest, masl	1216.4
Weir width in the basement, m	7.0
Apron length, m	6
Length of the stilling basin, m	12.1
Elevation of the stilling basin, masl	1211.2
Riprap length, m	16.5
Elevation riprap, masl	1212.2

Between the intake channel of the Tyrolean weir and the gravel trap as well as to the sandtrap two intake gates of vertical sliding type are installed. The height of the gates is 1.0 m and the width is 1.5 m.

The required hydraulic steel structures can be summarized as follows:

Number of Units	2
Type of Gates	Sliding Gates
Width, m	1.5
Height, m	1.0
Pressure at Bottom	0.5 bar

A drawing of the Tyrolean weir in plan and several sections can be found in Annex 8.

8.2.3.2 Gravel Trap

From the Tyrolean intake the water is conveyed via its collecting channel into the gravel trap with the consecutive spill. The gravel trap is located on the right side of the weir, the dimensions of the trap are of 2 x 16.3 m. The trap is planned for accumulation of fine bed load material in the range of 2 mm – 6 mm, which is expected to enter through the intake rack.

For occasional flushing of the fine bed load material vertical flushing gates with the dimensions 1.5 x 1.5 m are planned to be installed on the upstream and downstream end of the trap. With help of these gates it will be possible to flush the entered fine material to the downstream section of the riverbed. The upstream gate can also be used as intake structure during winter time.

The main dimensions of the gravel trap are the following:

Gravel trap width, m	2
Gravel trap length, m	16.3
Capacity of the gravel spill, m^3/s	8.5
Bottom Elevation, masl	1211.5
Crest Elevation, masl	1216.4

At the upstream and downstream side of the gravel trap two gates of vertical sliding type are installed. The height of the gates is 1.5 m and the width is 1.5 m.

The required hydraulic steel structures can be summarized as follows:

Number of Units	2
Type of Gates	Sliding Gates
Width, m	1.5
Height, m	1.5
Pressure at Bottom	0.5 bar

A drawing of the gravel trap in plan and section view can be found in Annex 8.

8.2.3.3 Sandtrap

To eliminate sediments with grain sizes greater than 0.2 mm, a sandtrap with two chambers and a total length of 35 m is required to be constructed on the right river bank. The alignment of the sandtrap is defined by following reference points:

Reference Point	X [m]	Y [m]
A	129769.09	108348.73
D	129800.00	108391.73

The total length includes a 3.5 m long transition from the intake to the total depth of the sandtrap. The cross section of each chambers is 2 m wide and maximum 3.9 m deep at the upper section and 5.1 m at the downstream cross section. The mean water depth at design discharge is 2.2 m. The bottom of the sandtrap is 2 % inclined in order to flush deposited material. The structure is of reinforced concrete with sidewalls of 1 m thickness.

The main dimensions of the sandtrap are summarized for one chamber as follows:

Characteristic grain size of suspended particles, mm	0.2
Design discharge of the Tyrolean intake, m^3/s	1.8
Flow velocity, m/s	0.20
Number of chambers	2
Length, m	35
Width, m	6.5
Width of each chamber, m	2
Maximum water depth, m	5.1
Minimum water depth, m	1.4
Bed slope, [%]	2
U/S Foundation Level, masl	1210.1l
D/S Foundation Level, masl	1208.9l
Length of the flushing canal, m	63
Width of the flushing canal, m	1.5
Height of the flushing canal, m	1.0

At the upstream side of the sandtrap two gates are installed with a width of 2 m and a height of 4 m. At the downstream end of the chamber an appr. 10 m long and 6.5 m wide gate chamber is situated. The gate chamber is equipped with two intake gates with a width of 1.5 m and a height of 1.5 m. Moreover two flushing gates with a width of 1.5 m and a height of 1.5 m are installed there. The deposited suspended loads, are removed to the river through a flushing channel under pressure with a length of 50 m. The channel is of rectangular shape with a height of 1 m and a width of 1.5 m.

The sediment free water enters via the intake chamber to the intake of the penstock. The intake is equipped with a vertical roller gate of a height of 1.5 m and a width of 1.5 m. The bottom elevation of the intake gate is 1210.8 masl. In front of the intake gate an inclined fine rack is installed in order to avoid entry of fine debris to the penstock. The fine trash rack is required to be removed from debris manually by the operating staff.

Following main dimensions are required for the hydraulic steel structures of the sandtrap and the consecutive intake.

U/S Vertical Sliding Gates for Intake:

Number of units	2
Width:	2 m
Height:	4 m
Pressure at bottom	0.5 bar

D/S Vertical Sliding Gates for Intake:

Number of units	2
Width:	1.5 m
Height:	1.5 m
Pressure at bottom	0.5 bar

D/S Vertical Sliding Gates for Flushing:

Number of units	2
Width:	1.5 m
Height:	1.5 m
Pressure at bottom	0.5 bar

D/S Vertical Roller Gates for Intake:

Number of units	1
Width:	1.5 m
Height:	1.5 m
Pressure at bottom	0.5 bar

D/S Stoplog for Intake Gate:

Number of units	1
Width:	1.5 m
Height:	1.5 m
Pressure at bottom	0.5 bar

At design discharge the water level is equal to 1213.0 masl. In case the discharge in the river is decreased 20 cm water level decrease in the sandtrap is permitted. This means that it is required to maintain the water level in the sandtrap no lower than at the elevation of 1212.8 m. This can be done by maneuvering the governing devices of turbines according to the signals from level sensor, mounted near intake orifice of the penstock. During flood period the water level in the sandtrap might exceed above 1213.0 masl. Due to latter the elevation of the sandtrap wall crest was determined to be on 1215 masl.

Downstream the flushing chamber of the sandtrap an open channel connects the sandtrap with the entry portal of the headrace tunnel. The first part of the open channel is a transition from the appr. 20 m wide section to the 3.8 m wide free-flow section towards the tunnel entry portal. The length of the transition is 33.4 m long. After the transition the open channel has a width of 3.8 m. The open channel has a length of 150 m. The channel might be constructed in the cut and cover construction technology. The shape of the transition channel is rectangular.

A drawing of the sandtrap and the consecutive intake area in plan and section view can be found in Annex 8.

8.2.3.4 Fishpass

The fishpass is located on the left side of the weir. The design discharge was selected in accordance to the minimum ecological flow of 0.04 m^3/s. A technical fishladder of the vertical slot type was designed. The total length of the fischpass is 19 m. The alignment of the fishpass axis are defined by by following reference points:

Reference Point	X [m]	Y [m]
B	129789.76	108363.91
C	129800.70	108348.99

The ladder consists of 7 basins, the elevation difference at each basin is equal to 0.2 m at design discharge. The length of each basin is 2.5 m and the width is 1.2 m, the mean depth of the basin is 0.5 m. At the upstream and downstream end a stop log is placed in order to close the fishpass in case of repair works. The fishpass as well as the dividing walls are of reinforced concrete. Each dividing wall is equipped with a vertical slot of 0.15 m width on the left side of the wall, which spills the minimum ecological flow to the downstream riverbed. At the bottom of the fishpass a layer of bed material with a thickness of 0.1 m is placed. The substrate shall have a similar grain size distribution as the original riverbed ($d_{50} \approx 53$ mm) in order to enable passage of invertebrates and other small aquatic fauna.

The main parameters of the fishpass are the following:

Type	Vertical Slot
Design discharge, m^3/sec	0.04
Number of basins	7
Length of the basins, m	2.5
Width of the basins, m	1.2
Slot width, m	0.15
Flow velocity at the slots, m/sec	1.79
Crest elevation, masl	1214.3

A drawing of the fishpass in plan and section view can be found in Annex 8.

8.3 Penstock

8.3.1 Engineering Geological Conditions

The waterway alignment is constructed along the left and right banks of Gargar River crossing the river three times. The geological map and geological-lithological sections along the waterway alignment were drawn on the basis of available engineering-geological survey and material from investigations carried out in the past years.

In the beginning short section of appr. 7m length the alignment of the penstock crosses the basalts of 5th layer. Further dowsntream to the third crossing of the penstock of Gargar River (upstream symbol Yr 12 on drawings in Annex 4.5) the penstock alignment will be laid in sliding-loose rocks (1^{a} layer). A short length of 25 m on aforementioned reach will be constructed along the right bank in hydro thermally modified volcanic-sedimentary rocks of 9^{a} layer. At the consecutive length of about 80 m the penstock alignment is along the left bank between the first and second river crossings. This reach is composed of dealluvial-coalluvial sediments (1st layer).

The sliding-loose rocks of 1^{a} layer consist of big boulders and detritus from basalts, porphyries, tuffs, metasandstones and other rocks with silty-sand filling to 20-30%. The size of boulders may reach 4.0 m. In the first 120 m length there is no filling in the above-mentioned rocks. The accumulation of big boulders of basalts is shown on photo 4 in Annex 4.5.

On the right bank of Gargar River, where the penstock is constructed along the central part of relatively stable sliding massif, the upper steep slope consists of volcanic-sedimentary rocks. On the site, where the slope is getting relatively flat, at the steep edge there is a fracture. The length of the fracture is 100 m, the width is 2 - 3 m and the depth is 3 – 4 m. The upper part of the fracture is composed of silty-clays and the lower part consists of volcanic-sedimentary rocks. In case of seismic activity or other stresses a rockfall from great height consisting of large boulders to the penstock might be expected. It can not be forecasted, whether the sliding-loose massif will be shifted after the slope’s collapse or not. Taking into consideration that there is a big accumulation of sliding-loose material a shifting and consequently a dislocation of the penstock is not expected. Nevertheless the pipeline is expected to be covered with loose rocks, which are very thick. Therefore in any case it is recommended to undertake some measures, minimizing the risk. In particular it is recommended to embed the pipeline into a deep trench and to backfill it from the surface with a layer of silty-clay and silty-sand material with a thickness of 2.0 m. Thereby the damage of the penstock by big rocks and stones rolling down from the slope is limited.

Washout processes and collapse of banks close to the river are also observed on the above-mentioned territory together with the development of sliding processes.

Downstream the third and last river crossing the penstock is laid along the left bank in dealluvial-coalluvial sediments (1st layer). (downstream symbol Yr 15 on drawings in Annex 4.5). On the consecutive length of 150 m the penstock alignment passes through alluvial-coalluvial sediments (2nd layer) and then again though dealluvial-coalluvial sediments of 1st layer. At the next reach of appr. 70 m the alignment crosses a tectonic stress zone (11th layer), consisting of volcanic-sedimentary rocks, transformed into the white flour-like substance mylonite as a result of mylonitization.

Further downstream on a 270 m long reach the waterway alignment passes in dealluvial-proalluvial silty-clays and silty-sands (3rd layer), then (upstream symbol Yr 21 on drawings in Annex 4.5) through alluvial-coalluvial sediments of 2nd layer. At the consecutive 95 m the waterway is laid in dealluvial-proalluvial silty-clays and silty-sands of 3rd layer.

The final section of the penstock with a length of appr. 80 m to the powerhouse is constructed in alluvial-coalluvial sediments of Gargar River. These sediments are composed of large boulders, pebble, gravel and detritus from basalts, porphyries, tuffs, metasandstones and other rocks with silty-sand and sand filling up to 25% (2nd layer).

Finally it has to be concluded, that the engineering-geological conditions of the waterway alignment are quite complicated.

8.3.2 Design

The intake to the penstock has already been discussed in the paragraph 8.2.4. Under consideration of the optimum penstock diameter, calculated to 1.2 m the bottom elevation of the penstock is at 1210.3 masl. At the beginning section of the closed conduit the penstock is laid in concrete. The alignment of the penstock is defined by following reference points:

Reference Point	X [m]	Y [m]
D	129800.00	108391.73
E	131758.00	108353.00

The appr. 2160 m long penstock along the Gargar River shall be constructed as embedded penstock in order to avoid any damage by sliding or falling weathered rocks from the Gargar Gorge. The penstock is planned to be laid appr. 1 m below the ground surface to be saved against freezing ground. The penstock is laid on bedding material.

At first 154 m the penstock alignment passes along the right bank of the river at riverbed elevation. Further downstream the penstock crosses the river by two concrete support blocks on each side of the river with a distance of 17 m. The consecutive section of the alignment, which is constructed along the left bank has a length of 77 m. At the end of this section one more river crossing is planned. The width between both foundations on each side of the river is 15 m. From the alignment station point +496 m the penstock is planned along the right bank down to the foot of the gorge. Further downstream the penstock again needs to cross the river on a river width of 19 m. From this crossing onwards the final section of 1342 m length of the alignment continues along the left bank up to the powerhouse.

In accordance with Armenian standards the wall thickness of the penstock at the end section is 10 mm taking into consideration the absolute hydraulic hammer effect in the magnitude of 2.6 MPa, if a steel quality with a yield stress of 250 N/mm^2 is taken. The pressure in the order of 2.6 MPa may be observed in the penstock under a sudden load break at the powerhouse. The closing time of jet nozzles of both turbines would be equal to 30 seconds at this condition.

The thickness of 10 mm is required at a distance of 450 m. For the most upstream reach with a length of 100m a wall thickness of 8 mm is sufficient, the center reach has a wall thickness of 9 mm on a length of appr. 710 m.

For the calculation of the wall thickness 2 mm for corrosion was considered.

The embedded part of the penstock is constructed in a trench with a depth of 1.8 m and a width of 2.1 m. The trench is constructed from a new access road to be constructed along the Gargar River. The width of the access road was considered to be 10 m wide and the top layer should consist of a 15 cm thick layer of crushed stones, a so-called unpaved access road. After fixation of the penstock the excavated material shall be filled back.

At the three locations, where the penstock crosses the river two concrete support blocks shall be constructed on each bank. During the construction phase the pit needs to be protected by a cofferdam. After the construction of the concrete support blocks the dams should be dismantled. During the operation of the plant the steel supporting structures of the penstock allow the sliding on the horizontal flatness along the steel plate, which is fixed on the concrete support blocks. Due to this compensatory pieces along the penstock alignment are planned. In case the sliding is not ensured at all crossings the installation of contraction compensators might be required. The necessity of compensatory pieces shall be investigated during the detailed design of the project. The issues of installation of valves at crossings for the emptying of penstock and devices for the air relief shall also be determined in a latter stage.

The main penstock parameters are the following:

Type	Embedded
Material	Steel
Length, m	2160
Design discharge, m^3/s	1.8
Diameter, m	1.0
Flow velocity, m/s	2.3
Maximum pressure, MPa	2.6
Static pressure, MPa	2.2
Maximum wall thickness, mm	10

A drawing of the penstock in plan and section view can be found in the Annex 8.

8.4 Powerhouse and Appurtenant Structures

8.4.1 Engineering Geological Conditions

It is planned to construct the powerhouse of Gargar SHPP on the left bank of the river at the flat area within the limits of the flood plain, some 300 m upstream the confluence point of the rivers Gargar and Dzoraget. The location of the powerhouse area is available on photo 5, attached in the Annex 4.5. The powerhouse site is mainly composed of alluvial-coalluvial sediments, which are covered from surface with silty-clays and silty-sands. The thickness of latter is not large.

The alluvial-coalluvial sediments are composed of pebble and gravel as well as of big boulders, detritus and gruss from different bedrocks with silty-sand filling up to 25% (2nd layer). At this section the thickness of the alluvial-coalluvial sediments exceeds 10.0 m. The alluvial-coalluvial sediments will serve as the basis for the powerhouse.

The ground waters are circulating in alluvial-coalluvial sediments. The level of ground waters is connected with the water level of the river Gargar. It is slightly inclined to the left-bank side to alluvial-coalluvial sediments, which have high filtration coefficients.

According to their chemical composition both ground waters of powerhouse area and Gargar River are soft. These waters contain hydro carbonated calcium and do not cause the corrosion of concrete.

Finally it can be concluded, that the engineering-geological conditions at the powerhouse site are quite favorable.

8.4.2 Design

The powerhouse is located on the left bank of the river, 300 m upstream the confluence point of Gargar and Dzoraget Rivers. The main dimensions of the powerhouse are determined by the size of the hydro-mechanical equipment and from operation, assembly and dismantling conditions in the powerhouse. Furthermore the transportation of the equipment by the machine hall crane was considered. The erection site, which is adjacent to the block, is the continuation of the units block and is maintained with the help of the machine hall crane. The dimensions in the plan-view are 15 m x 18 m. The powerhouse comprises also rooms for operation personnel and lavatory arrangements. A crane with load-carrying capacity of 30 t is mounted in the powerhouse for maintenance works. Beneath the deepest floor level a sump shall be placed for collecting leakage water. The pit needs to be equipped with pumps.

The powerhouse floor is designed at elevation 992.7 masl, appr0.7 m above the maximum flood level of 992.0 masl, which corresponds to a flood of Q=99 m^3/s with a return period of 100 years.

The most appropriate solution for the hydro-mechanical equipment was found to be a vertical Pelton turbine with four nozzles with a design discharge of 1.8 m^3/s. One valve with a diameter of 0.6 m is placed in the powerhouse. The turbine axis of the Pelton turbines are installed at an elevation of 993.5 masl, which is 3 m higher than the riverbed elevation. The maximum water depth in the river is 2 m.

The powerhouse area covers a space by 30 m x 50 m, it is located at the elevation of 994.5 masl and it is protected by a fence. On the spot the switchyard with an area 6 m x 10 m is located, where the transformer of 6.3/35 kVA type is planned to be placed.

The main powerhouse parameters are given as follows:

Powerhouse type:	External
Total width, m:	15
Total height, m	11.6
Total length, m	18
Machine hall level, masl	992.7

Foundation level, masl	989.8
Turbine type,	Pelton
No. of units	1
Bearing type	Vertical
Design discharge, m^3/s	1.8
No. of nozzles,	4
Turbine elevation, masl	993.5

A drawing of the powerhouse in plan and section view can be found in the Annex to this section.

8.5 Tailrace

A tailrace canal of about 50 m length conveys the turbined water back to the Gargar River. The tailrace is designed as open channel.

The tailrace canal has a rectangular section at the beginning, which changes to a trapezoidal shape further downstream. The bed width of the trapezoidal section is 1.1 m and the side slopes are 1:1. The depth of the tailrace channel is 1.5 m, the bed slope is 0.1%. With a flow area of 1.8 m^2 the velocity in the tailrace canal shall be 1.0 m/s at design discharge with a water depth of 0.9 m.

The tailrace will be of reinforced concrete with following main dimensions:

Cross Section,	Trapezoidal
Design discharge, m^3/sec	1.8
Bottom width, m	1.1
Top width, m	4.1
Water depth, m	0.9
Depth, m	1.5
Side slope	1:1
Slope, %	0.1
Channel length	50
Flow velocity, m/sec	1.0

A drawing of the tailrace in plan and section view can be found in the Annex to this section.

9

Hydromechanical Equipment

9. Hydromechanical Equipment

9.1 General

Gargar HPP is a typical run-of-river HPP for a mountainous region, with no storage at the dam site or daily reservoir.

Key design parameters are as follows:

Intake
Max. operating level in sand trap basin: 1215.4 masl
Min. operating level in sand trap basin: 1212.9 masl
Design discharge for run-of-river operating mode: 1.8 m³/s

Tailrace
Probable maximum flood level: 991.9 masl
Head losses at design discharge 1 unit: 9.68 m

9.2 Turbine

As a typical run-of-river plant in the mountains as noted, at Gargar HPP the river water discharge is very high over 3 months and is less than 50% of the maximum discharge over the remaining 9 months.

Selection of the turbine type and number of turbine-generator sets for plants that perform no regulating function in the grid but operate only in parallel to the grid depends essentially on the following parameters:

- head
- water discharge fluctuations in the river, particularly days with very high and very low water levels
- economic parameters.

Under consideration of these parameters, for a net head of 210 m and a design discharge of 1.8 m³/s, a four-jet Pelton turbine is selected as the best technical and most favorable economic option. Thanks to its four nozzles, this turbine can be operated at part load down to 5% or 0.09 m³/s and provide the necessary flexibility at part-load operation of the plant.

Incoming water to the turbine flows from the penstock into a spiral case around the runner. This spiral case is arranged horizontally and equipped with four nozzles that direct water jets at the runner buckets. Installed above the runner on a support structure is the generator, with the generator shaft directly coupled to the turbine runner. The entire turbine-generator unit is supported by the generator thrust bearings. Additionally, both turbine and generator are provided with guide bearings.

The purpose of the nozzles is not only to precisely direct the waterjet onto the runner buckets. In combination with the governor and nozzle control by servomotor, the flow through the nozzles is directly matched to the water availability in the river. Additionally, to prevent the water hammer in the water pipe during rapid control actions, the water jet is regulated by jet deflectors and directed past the turbine runner. This means the turbine has a double regulation arrangement by regulating needles and jet deflectors, so avoiding water-hammer in the penstock.

Turbine design parameters are as follows:

Table 9.1: Parameters of turbine:

Parameter	Abbreviation	Value
Number of units	-	1
Number of nozzles per unit	-	4
Rated net head	H_{rated} [m]	210
Rated flow	Q_{rated} [m³/s]	1.8
Rated output	P_{rated} [kW]	3,260
Speed	n [rpm]	600
Runner pitch diameter	D_1 [m]	1.2
Turbine center line	[masl]	991,9

The Turbine consists of the following components:

- turbine runner
- turbine shaft
- guide bearing
- shaft seal
- turbine housing
- spiral case with 4 nozzles
- 4 jet deflectors
- all necessary auxiliary equipment.

9.3 Governor

In order to establish how the plant is to be controlled, the modes of operation must be specified. This plant will only be operated in parallel to the grid, and island operation (operation in isolated mode) is not necessary. This eliminates the need for speed regulation, although speed monitoring for protection of the plant is still necessary. Furthermore, power regulation to a schedule is not possible, as there is no water storage for this purpose. The available water discharge of the river should be completely and optimally exploited in the turbine. The essential tasks of the regulator are then as follows:

- level regulation in the sand trap (controlled variable)
- power monitoring, with emergency trip if necessary
- speed monitoring, with emergency trip if necessary.

The water level in the sand trap may not drop below the specified minimum, so no air bubbles will be drawn into the penstock. Depending on this water level, the nozzles will be so controlled that the highest possible plant efficiencies will be attained. During part-load operation, the nozzles will be shut down in small steps. For minimum load, there is a transition to operation with one nozzle.

For each turbine, one governor is provided, consisting of:

- oil system
- digital governor
- control valves and pipes.

The governor regulates power depending on the water level in the sand trap through electrical circuits that issue a signal to an electro-hydraulic transducer. This controls the main oil distributing valves to direct pressure oil to the servomotors to position the needles and jet deflectors.

The main components of the oil system are sump tank, oil pressure pumps, pressure accumulator, control board and other auxiliary devices.

The hardware of the digital turbine governor consists of the programmable electronic modules, the control panel for local control, the output amplifiers as well as additional devices and signal decoupling and/or transformation.

The governor is designed and equipped for starting and stopping the turbine manually locally as well as automatically under remote control from the control room. Furthermore, the monitoring of important operating parameters with issue of prewarning signals and emergency tripping, etc. is provided. The key-operated switches for the functions “manual / automatic” and “local / remote” are arranged on the control panel.

In addition, the governing system incorporates emergency shutdown for full closure of the turbine and turbine inlet valve.

9.4 Auxiliary Equipment

9.4.1 Turbine Inlet Valve

The unit will be fitted with a hydraulically operated spherical shut-off valve to provide emergency shutdown of the unit and also enable routine inspection, repairs and maintenance of the unit without draining the penstock. The valve with its operating mechanism and accessories will be designed for installation in the available space in the powerhouse, as shown in the drawings. An important consideration for valve design is that it must be possible to assemble and disassemble the valve body, operating gear and dismantling joint using the powerhouse crane.

The valve body must be designed for a maximum water head of approximately 250 m and has a diameter of 0.6 m. The oil-hydraulically operated servomotor is integrated into the oil supply unit of the turbine governor.

9.4.2 Powerhouse Crane

One powerhouse bridge crane will be provided for installation, maintenance and repair of the turbine, generator and for general powerhouse use. The crane bridge consists of a double box beam structure that can run on rails along the length of the hall. The crane trolley is mounted between the beams, so saving space. The geared motors for the beam structure and trolley are provided with soft starting and stopping equipment, allowing very quiet and precise traversing of the crane. The hoisting capacity of the crane will be 30.0 tonne. Bridge and trolley travel will both be controlled by hand-operated pendant chains.

9.4.3 Cooling Water System

A cooling water system is foreseen for cooling all bearings, governor oil and other auxiliary equipment. The complete cooling water equipment will consist of two pumps, filters, pipes and valves, and will extract water from the turbine discharge channel.

9.4.4 Drainage and Dewatering System

Any drainage and leakage water will be directed by means of embedded pipes to the tailrace channels. Floor drains from various points of the powerhouse will likewise be directed to these channels. However, water draining from the fitting area will first be directed to an oil separator, so that the receiving waters will not be polluted.

9.4.5 Air Conditioning and Ventilation

Rooms or spaces without any cooling load but accessible by maintenance staff shall be ventilated at a fresh air rate of at least two air changes per hour to prevent build-up of harmful gases.

The central control room shall be air-conditioned. The air conditioning unit shall comprise all equipment for filtering, cooling, heating, and dehumidifying the air. For cooling and dehumidifying the air, a direct expansion cooling coil shall be installed.

9.4.6 Workshop and Stores Equipment

Provision will be made to equip a small workshop and a store inside the powerhouse. This will consist of tool and spare part racks to store all tools and spare parts supplied by the manufacturers of the various equipment. It will also contain a workbench with vice, drill press, grinding wheel and gas electric welding equipment.

10

Electrical Equipment

10. Electrical Equipment

10.1 General

It is planned to install one vertical hydro unit in Gargar SHPP. The turbine shall have a capacity of around 3.26 MW. According the power capacity of the plant, the isolated load regulating mode is not foreseen. The power plant will be operating parallel to the grid only.

The grid voltage on the connection place is 35 kV. The diagram of the main electrical connections is chosen according to the significance of the plant. The main scheme is shown in Annex 10.

The following components for the hydro power plant and connection to the grid shall be provided:

- Complete generator including excitation and generator control system
- One main power transformer 35/6 kV
- One station service voltage transformer 6/0.42 kV
- One set station service equipment: 0.4 kV station service power centre with distribution boards
- One set 24 V DC system: rectifier and batteries
- Instrumentation, Protection and Control system
- All required power- and control-cabling systems
- Domestic power system
- Earthing and lighting protection systems

10.2 Generator

According to the type of turbine, one vertical shaft three-phase synchronous generator directly coupled to a Pelton turbine shall be foreseen for Gargar SHPP.

According the turbine speed and depending of manufacturer, the generator speed may be selected to 600 min^{-1}.

The main features of the generator will be as follows:

Power output	3170 kW / 3965 kVA
Rated voltage	6 kV plus and minus 5%
Number of poles	10
Power factor	0.8
Protection class	IP 23

Brush-less exciter shall be mounted directly on the generator shaft. Excitation power supply shall be taken from the generator main terminals. Two automatic voltage regulators to be provided each assembled in separate panels and shall include automatic power factor regulation as well.

The generator and exciter shall be designed with a self-ventilating open air cooling system.

The generator shall be provided with complete control and indicating system for local remote operation, and with all protection and alarm systems necessary for a safe and reliable operation. All required auxiliary equipment to make the generator complete are to be provided as well.

The location and disposition of the generators can be seen on the respective civil drawings.

10.3 Automation, control, signaling and protection

The circuits of control current, circuits of switching on and switching off of the 6 kV and 35 kV circuit breakers, automatic synchronizing equipment for the 6 kV generator breaker as well as supply, automation, protection and signaling circuits and generator excitation system are related to the control circuits.

Digital data processing station of turbine-generator shall be provided to ensure full automation of the turbine-generator operation, control of auxiliary devices as well as of servo-drives of the turbine-generator. They will work together with the excitation and synchronization regulator and electric protections of generator. Their further task is to ensure automatic start-up of the turbine-generator set with time control. Also cutting-off of the turbo-generator set will be performed automatically, subject to the order from the plant control system, through the action of a fault relay or manually by actuation of an emergency push-button. The processing station performs diagnostics of the technology equipment as well as filing of records (to a limited extent) that describe operation.

Control system shall be installed at the data processing station. The number of I/O modules shall be selected with respect to the number of inputs and outputs counted up in the delivered documents:

digital inputs 24V DC,
digital outputs 24V/0.5A DC,
analogue inputs 4-20 mA.

Communication with protection means will be performed by means of digital signals.

The automatic control system of the unit and auxiliary devices provides:

- Automatic start and stop with one impulse from the unit control board.
- Centralized control of the work of different blocks and devices in HPP Building
- Warning signaling at the failure of mechanisms, if the work is still possible
- Warning signaling at the failure of mechanisms, if the work is impossible

- The following relay protections shall be provided for the generator feeder (and for transformer feeder where mentioned):
    - over-current protection, instantaneous and inverse-time (both for generator and transformer individually)
    - differential protection (separate for generator and transformer)
    - earth-fault protection (both for generator and transformer individually)
    - buchholz protection (transformer only)
    - temperature protection (both for generator and transformer individually)
    - protection from asynchronous operation
    - over- and under-voltage protection
    - over- and under-frequency protection
    - over-load protection
    - unbalanced-load protection
    - reverse power protection

All generator- and transformer-protections are acting on the unit trip.

10.4 Transformer

Transformers to be provided:

- Main transformer: One 4000 kVA, 35/6 kV, plus 3 and minus 3 steps each 2.5% (tapselector), ONAN-cooling type, sealing tank, outdoor installation
- Auxiliary transformer: One 160 kVA, 6/0.42 kV, plus 2 and minus 2 steps each 2.5% (tapselector), AN-cooling type, dry-type with protection housing, indoor-erection.

10.5 Swithgear

The 6 kV switchgear shall be of metal-clad compartmented design for indoor installation. The cubicles are provided with a single bus bar system and draw-out type circuit breaker feeder panels and fused load-break switch feeder panels.

The switchgear to be capable of being operated locally and also remote from the control room. The general arrangement of the switchgear can be seen on the relevant building drawing.

The 0.4 kV switchgear is designed as an indoor metal-clad switchgear with plug-in feeders.

10.6 DC System

The 24 V DC system shall consists of two 100% rectifiers and one 100% battery. Each rectifier must be design for a capacity sufficient to supply all consumers of all units and additionally to charge the battery set at the same time. A maintenance free lead acid battery shall be provided.

10.7 Power and Lighting Installation

A complete earthing and lighting protection system shall be provided.

For MV cables XLPE insulation material shall be used, for LV cables PVC insulation material shall be used.

For internal lighting fluorescent lamps will be provided. The internal lighting shall be designed so that lighting illumination densities of 250 lx for the powerhouse and 500 Lx for the control room will be achieved.

10.8 Connection to the grid

It is foreseen to connect the plant to the electrical network of RA with one 35 kV over-head line.

For the transfer of 6 kV generator voltage to the 35 kV over-head line it is foreseen, to construct a substation of 6/35 kV near by HPP Building. One power transformers – 4000 kVA and the 6 kV and 35 kV circuit breaker provide the connection to RA grid.

The connection to the grid is through a new double 35 kV transmission line to the existing transmission lines “Vardablur”. The existing high voltage line runs not far from the gorge, on the plateau on the left bank of the river Dzoraget. The new line which has to be built crosses the valley in the area where the rivers Dzoraget and Gargar flow together, then crosses the river Dzoraget and leads out of the gorge onto the high plateau where it connects to the existing line. The new line has a total length of 2.5 km.

11

Project Quantities and Costs

11. Project Quantities and Costs

11.1 Project Quantities and Costs

The cost estimates used in this report are based on unit and lump sum prices applied to the quantities of major work items.

11.2 Documents Used

An estimate of the expected investment cost has been prepared by the Consultant based on price indications for the compact hydro set and hydraulic steel structures as well as on present local unit prices for civil works.

Materials and quantities required have been roughly computed using the engineering principle drawings consisting of plans and sections of all the components of the project given in the Annex 8.

11.3 Cost Estimates

11.3.1 Overview

The summary of cost estimates for the Gargar SHPP consists of following parts:

- Environmental Mitigation Costs
- Preliminary Works
- Civil Works
- Hydraulic Steel Structures
- Hydro-mechanical Equipment
- Electrical Equipment
- Transmission Line

The cost estimation for civil works and hydraulic steel structures is based upon the bill of quantities, prepared on the principle drawings shown in the Annex 9. Unit prices were taken from a local data basis used for cost estimations carried out by “ArmHydroEnergoProject”. The approach and its methodology are explained in more detail in section 11.3.2.

For hydromechanical equipment the cost estimates are based on tentative quotations from qualified manufacturers and suppliers.

For electrical equipment and the transmission line to the next substation, cost estimates are based on local cost data basis also used for cost estimations carried out by “ArmHydroEnergoProject”.

11.3.2 Local Unit Prices

11.3.2.1 Methodology

The methodology is elaborated by the Ministry for Urban Development (MoUD) of the Republic of Armenia and is confirmed by the decree N46 as of April 28, 1998. It is coordinated with the Ministry of Finance and Economy (MoFE) of RA by the decree N 10-470 as of April 15, 1998. It is registered on May 21, 1998 by the State register and the registration number is N199800129.

The current methodology was elaborated for the construction of new buildings and for the reconstruction, rehabilitation, extension, improvement and maintenance of existing buildings and constructions on the territory of RA. Terms and definitions are applied in the methodology in accordance with ISO 1.0-93 “National Standardized System of RA. Main Items”.

11.3.2.2 Cost Items

In order to determine the costs of civil works and hydraulic steel structures according to functioning construction, industrial, estimate norms and rules (SniP IV-1 ÷ 16-84, SniP is the technical requirements, functioning in Armenia since Soviet period) the following expenditures articles are selected: workers labor expenses, sample types, working hours of machines and mechanisms, the constructions and materials requirements.

The calculation of the construction cost is determined by the following expenses:

- Wages
- The cost of machines and mechanisms operation
- Materials cost
- Overhead costs
- Income
- Other expenditures
- Taxes

11.3.2.3 Wages

The wages are determined applying the following formula:

W=L x R x C, where

W = the wage amount
L = labour expenses in person/hours (is determined according to the realized volumes of works and the time norms of the certain work unit)
R = is the rate of 1 hour (is determined by dividing the average monthly salary by 173.1, and the amount of average monthly salary is fixed by the MoFE of RA)
C = is the coefficient of Social Security

11.3.2.4 Costs of Machines and Operation

The cost of machines and mechanisms operation is determined applying the following formula:

MO= $(T_1 x C_1 + T_2 x C_2 + ... + T_n x C_n) x 1.03$, where

MO = machines and mechanisms operation cost
$T_{1,} T_{2,} T_n$ = time of machines and mechanisms operation
$C_{1,} C_2 C_n$ = cost of 1 hour of machines and mechanism operation
1.03 = coefficient of considered operations of other machines

If there are no rates of labor expenses the wages and the cost of machines and operation can be determined according to the note NSB-26/1622 as of 07.08.98 of MCB.

A sample for average monthly salary is 100.000 AMD
The wages index can be determined by

$$\left[\frac{(100.000 \div 1.325 - 102.12) x 1.17 + 102.12}{102.12}\right] x 1.25 = 1080.66$$

1.325 = coefficient of minimum tariff rate
102.12 = average tariff salary according to functioning SniP
1.17 = Social Security coefficient
1.25 = transition coefficient into functioning prices

The index of machine operation can be determined by
(0.15x2.2+0.37x784.3+0.24x1125.05+0.24x864.53) x1.7=1305.63

0.15, 0.37 = operation expenses portions correspondingly for depreciation, spare parts according to the present SniP
0.24, 0.24 = operation expenses portions correspondingly for fuel and wages according to the present SniP
0.22, 784.3 = coefficients for the changes correspondingly for depreciation, spare parts
1178.95, 270.27 = coefficients for the changes correspondingly for fuel-lubrication materials and wages
1178.95 = coefficient of fuel-lubrication materials is determined monthly
1.7 = transition coefficient into functioning prices

11.3.2.5 Material Costs

The cost of materials is determined with the help of the following formula:

Mat= $(M_{1,} x P_1 + M_2 x P_2 + ... + M_n x P_n) x 1.05 x 1.05 x 1.02$, where

Mat = materials costs
$M_1, M_{2,} M_n$ = materials consumption according to SniP
$P_{1,} P_{2,} P_n$ = prices of materials and constructions (are assumed according to bulletin of information center and according to the pricing of MoUD monthly)

1.05	=	coefficient considering the amount of other materials
1.05	=	coefficient of the transportation expenditures
1.02	=	coefficient of manufacturing-storage charges

11.3.2.6 Overhead Costs

The overheads (overheads include control, organization and domestic service of the construction production) are determined from total cost of direct expenses at the rate of 5.3%.

$$O = DE \times 0.053$$

O = overhead costs
DE = direct expenses

11.3.2.7 Income

The income is determined from total cost of all works at the rate of 10%

$$Income = (DE + O) \times 0.1$$

DE = direct expenses
O = overhead costs

11.3.2.8 Other Expenditures

Other expenses are determined applying the following formula:

$$Other = Temp. + Win. + Add.\text{ , where}$$

Temp. = the temporary buildings and constructions (mobile or mantled-dismantled buildings, temporary engineering services, (electrical energy supply, waters, access roads and others). They are determined by the norms of MoUD of RA depending on the construction type.)
Win. = is the average annual additional expenses during winter period of construction (the protection of ground from freezing, the storage of reinforced concrete and concrete constructions under required temperatures (the electrical energy supply, waters, access roads and others), which are determined by the norms of MoUD of RA depending on the construction type.)
Add. = additional expenses are determined by the decree of the government of RA

11.3.2.9 Taxes

The taxes are determined from the cost of civil works according to the legislation of RA

11.3.3 Environmental Mitigation Costs

As already mentioned in the framework of environmental considerations, the environmental mitigation costs are mainly caused by temporary land requirements during the construction period and corresponding mitigation costs. Minor costs are caused by permanent land acquisition and by tree compensation and afforestation. The details of cost estimate for environment and socio-economical mitigation measures are given in Annex 3.

11.3.4 Preliminary Works

The preliminary works consist of the access to the site, which is the construction of temporary roads as well as a single line access road to the headworks and to the powerhouse.

The details of the cost estimation of the structures for the access roads to the site are provided in the Annex 11.

11.3.5 Civil Works

The unit prices of civil works are based on the local unit prices used in Armenia in the year 2004. They were determined in accordance with present Armenian standards, which are explained in detail in section 11.3.2.

A detailed cost estimate was carried out for the direct civil works, which are:

- Earth works
- Temporary civil works during construction
- Concrete and Reinforced Concrete Works
- Reinforcement
- Bedding material

A detailed description of main quantities of civil works for each structure is given in the Annex 11.

11.3.6 Hydraulic Steel Structures

The costs of hydraulic steel structures are local prices in Armenia on the price level of the year 2004. The same approach as for civil works was used. The sharp increase in steel prices worldwide in the year 2004 was reflected in the selected unit prices.

The details of the cost estimation of hydraulic steel structures required at certain hydropower structures are provided in the Annex 11.

11.3.7 Hydro-Mechanical Equipment

The costs of hydromechanical equipment are based on budgetary prices from reputed international manufactures and suppliers and on the Consultant's actual cost statistics of comparable projects. Costs for erection, commission and testing charges for hydromechanical equipment were included in the costs estimates of concerned positions.

It was recommended by the Consultant to foresee European equipment with high efficiencies and reliability during operation, which was approved by the Ministry of Energy.

The details of the cost estimation of the hydro-mechanical equipment are provided in the Annex 11.

11.3.8 Electrical Equipment

The costs of electrical equipment are based on local budgetary prices in Armenia on price level of the year 2004. The same data bank is also used by "Armhydroenergoproject" for estimation of costs for electrical equipment.

Costs for erection, commission and testing charges for electrical equipment were included in the costs estimates of concerned positions.

The details of the cost estimation of the electrical equipment are provided in the Annex 11.

11.3.9 Transmission Line

The costs of the transmission line were calculated on the basis of local unit prices in Armenia on price level of the year 2004. The same approach as for electrical equipment has been applied.

11.3.10 Physical Contingencies

An effort has been made to project a cost estimation as realistic as it is feasible. To cover some of the unforeseens that may occur over the period of construction, provision for contingencies has been made as for separate positions as follows:

- Preliminary Works 5%
- Civil Works 5%
- Hydraulic Steel Structures 5%
- Hydromechanical Equipment 5%
- Electrical Equipment 5%
- Transmission Line 5%

11.3.11 Total Project Costs

The summary of all above mentioned costs comprise the total costs for the project and was calculated in detail for the final layout described in the present report. The detailed cost estimates are given in the Annex, the summary of cost estimates can be found in the following table. The summary shows already the distribution between local and foreign investment, expressed in US$ respectively.

11.4 Cost Basis

11.4.1 Price Level

All costs have been estimated for the price level of September 2004.

11.4.2 Currency and Conversion Rates

All costs are given in US $. The total costs were split in local (AMD) and foreign (US $) components. Following exchange rates were applied:

1 US $ = 500 AMD
1 US $ = 1.20 €

11.4.3 Local and Foreign Costs

The costs were split into local and foreign components according to the availability of locally produced materials.

It was agreed that only the hydromechanical equipment should be imported from abroad. All other costs are based on local level.

11.4.4 Investment Costs on Local Price Level

The following table shows the summary of cost estimates for the final layout alternative of Gargar SHPP. The detailed cost estimate is attached to the Annex of this section. The above-described methodology of the MoUD for determination of local prices was applied for two reasons mentioned below

- Even private companies in Armenia, not yet familiar with price setting in a market economy, still revert to this methodology for the preparation of tenders.
- For approving tariffs, the Regulator has installed a monitoring commission, which checks the capital costs of projects above 10 million AMD for plausibility. Local costs are reviewed with reference to the MoUD methodology.

However quotations for local cost components for mayor civil works and hydraulic steel structures indicated, that the local market can be considered as uncertain at present. Local prices on free market were tending to be slightly higher than the administered prices described in detail above. Therefore the Consultant decided to prepare two alternative cost estimates for Gargar SHPP, one based on the local methodology (see Table 11.1), and the other based on international prices, which is briefly explained in the following paragraph.

Table 11.1: Total Investment Costs – Local Price Level

500	Dram=1 US$, Price Index September 2004	LOCAL	FOREIGN	TOTAL
ITEM	**DESCRIPTION**	[US$]	[US$]	[US$]
I	**Environmental Mitigation Costs**	133,400	0	133,400
II	**Preliminary and General**	79,492	0	79,492
III	**Civil Works**	974,619	0	974,619
	Subtotal I - III	**1,187,512**	**0**	**1,187,512**
IV	**Hydraulic Steel Structures**	715,603	0	715,603
V	**Hydromechanical Equipment**	0	1,268,000	1,268,000
VI	**Electrical Equipment**	130,000	0	130,000
VII	**Transmission Line**	62,500	0	62,500
	Subtotal I-VII	**2,095,615**	**1,268,000**	**3,363,615**
VIII	**Physical Contingencies**			
5	% of Preliminary Works	3,975	0	3,975
5	% of Civil Works	48,731	0	48,731
5	% of Hydraulic Steel Structures	35,780	0	35,780
5	% of Hydromechanical Equipment	0	63,400	63,400
5	% of Electrical Equipment	6,500	0	6,500
5	% of Transmission Line	3,125	0	3,125
	Subtotal VIII	**98,111**	**63,400**	**161,511**
IX	**Engineering & Supervision**			
	% of Invest. Cost (Subtotal III-VII)	150,000	0	150,000
X	**Client's Costs**			
	% of Invest. Cost (Subtotal III-VII)	0	0	0
XI	**Miscelleneous**			
	unknown	0	0	0
XI	**Total Base Cost**	**2,343,726**	**1,331,400**	**3,675,126**
XIII	**Duties**			
10	% on Imported Goods	133,140	0	133,140
XIV	**Total Project Cost**	**2,476,866**	**1,331,400**	**3,808,266**

11.4.5 Investment Costs on International Price Level

In Armenia, the construction sector is presently in transition from a centrally planned economy with prices of state owned companies set by the government and a market economy with prices of private companies determined by the market.

Market prices vary considerably, and without actually tendering for the project, it is impossible to identify the unit costs at the level of reliability required for feasibility studies.

The Consultant asked for quotations from local construction companies for main civil works components, such as excavation, concrete and hydraulic steel structures, such as penstocks of different sizes and gates to confirm the price level of the local estimate. The quoted prices were higher than the estimated local prices, but lower than present international prices of the Consultants data bank for similar hydropower projects. This shows that local prices are tending towards the international level.

Investors have to be aware that hydropower development in Armenia will become more costly in future. In the past few years, a large supply of new and second-hand cheap pipes (diameter 0.8-1.2 meters) from abandoned sewerage and other projects kept costs for penstocks low, but this supply will run out in near future. Recent trends in international steel prices are also expected to lead to a sharp increase of unit costs for hydraulic steel structures such as penstocks and gates.

In order to reflect the expected development of local prices, the Consultant decided to carry out a cost estimation on basis of international costs for all hydropower components, including civil works, hydraulic steel structures, hydromechanical and electrical equipment and the transmission line.

While the local cost estimate (see Table 11.1) is relevant for SHPP development today, the international cost estimate shown in Table 11.2 will become relevant in the future.

Table 11.2: Total Investment Costs – International Price Level

500	Dram=1 US$, Price Index September 2004	LOCAL	FOREIGN	TOTAL
ITEM	**DESCRIPTION**	[US$]	[US$]	[US$]
I	**Environmental Mitigation Costs**	133,400	0	133,400
II	**Preliminary and General**	98,238	0	98,238
III	**Civil Works**	1,911,882	0	1,911,882
	Subtotal I - III	**2,143,519**	**0**	**2,143,519**
IV	**Hydraulic Steel Structures**	1,337,682	0	1,337,682
V	**Hydromechanical Equipment**	0	1,268,000	1,268,000
VI	**Electrical Equipment**	419,000	0	419,000
VII	**Transmission Line**	135,000	0	135,000
	Subtotal I-VII	**4,035,201**	**1,268,000**	**5,303,201**
VIII	**Physical Contingencies**			
5	% of Preliminary Works	4,912	0	4,912
5	% of Civil Works	95,594	0	95,594
5	% of Hydraulic Steel Structures	66,884	0	66,884
5	% of Hydromechanical Equipment	0	63,400	63,400
5	% of Electrical Equipment	20,950	0	20,950
5	% of Transmission Line	6,750	0	6,750
	Subtotal VIII	**195,090**	**63,400**	**258,490**
IX	**Engineering & Supervision**			
	% of Invest. Cost (Subtotal III-VII)	150,000	0	150,000
X	**Client's Costs**			
	% of Invest. Cost (Subtotal III-VII)	0	0	0
XI	**Miscelleneous**			
	unknown	0	0	0
XI	**Total Base Cost**	**4,380,291**	**1,331,400**	**5,711,691**
XIII	**Duties**			
10	% on Imported Goods	133,140	0	133,140
XIV	**Total Project Cost**	**4,513,431**	**1,331,400**	**5,844,831**

12

Project Implementation

12. Project Implementation

12.1 Main Assumptions and Considerations

12.1.1 General

The climatological conditions of the project area need to be considered for the elaboration of the foreseen project implementation schedule. The winter season in the Lori region can be considered as relatively mild, however snow and ice might be observed in severe winters. Intensive snowfall and considerable snow covers might be observed in the period between from December 15th to February 20th. Consequently temperature related construction works, such as concrete works, excavation of soil, etc. shall be carried out mainly in the warm season.

The preparatory works before construction of hydraulic structures include the construction of new access roads, such as to the headworks, to the powerhouse as well as along the penstock alignment. Especially the access road along the waterway is of mayor importance. It is desirable to reconcile these works with the flood period as mentioned in the schedule.

The construction works of Gargar SHPP are expected to be of limited extent compared to a large scheme project. Therefore the preliminary and general items, such as workshops for daily service and maintenance, deposit sites, etc. are considered to be relatively small as well.

Due to the relatively short duration of construction period and limited requirements of resources it is recommended by the Consultant to use the existing construction capacities of the local industry of the Stepanavan region, which utilization ratio is limited at present. This shall also provide job opportunities in the region.

The establishment of a concrete mixing plant on the construction site is obligatory. By the use of local quarries the dependency on ready made concrete from factories as well as the unit costs of concrete are reduced. The preparation time of the concrete on site by the concrete mixing plant before in situ works is not more than 1 hour. The formwork shall be mainly carried out by panels with an area of up to 10 m^2. Favorable seasons for the concrete works are the periods with an air temperature not lower than +5 C in average for a five-day week.

Gargar River is a typical mountainous river. Due to high velocities during flood-period the construction of safe cofferdams for the protection of hydraulic structures are difficult and consequently non-desirable. It is preferable to complete the construction of separate civil structures, especially the headworks, before flood periods.

12.1.2 Progress at Main Works

12.1.2.1 Weir, Intake and Sandtrap

Access to the headworks need to be established by the construction of a 400 m long access road from the existing motorway.

The headworks shall be constructed according to the classical river diversion: first the right bank side is protected and the sandtrap is constructed. The river flows in the left-bank section of the riverbed. By the time of the next flood period the soil cofferdam is dismantled and after flood the left-bank trench is protected, where the rest of the headworks is constructed.

Before construction works are started the area needs to be cleaned from shrubs and bushes by means of bulldozers, stubbing machines and graders. The progress of this work is approximately 44.5m^2/hour.

All soil and rock excavation works shall be carried out with the excavators, bulldozers and dump trucks. Rocks shall be preliminary loosened by drilling-shooting operations. The progress of concrete works is according to present regulations in Armenia. It is foreseen to use concrete pumps mounted on trucks in order to place the concrete in situ. For the erection and dismantling of formworks the present standards of “Common Norms and Prices” (EniR) are used.

12.1.2.2 Penstock

The works of penstock consist of the construction of a single steel pipeline. The construction of the access road for the works of the penstock is carried out in parallel.

Loosening of bedrock foundations along the waterway alignment shall be carried out by the borehole drillings and consecutive blasting. The loosened bedrock shall be taken by excavators with a bucket capacity between 1.25m^3 and 1.5m^3. The material is loaded on the dump truck and transported to the dump place.

The mean progress of works of the excavators in terms of excavation and preparation of unpaved access roads on site is defined on the basis of present Armenian norms and standards, such as SNiP, EniR, and other sources.

Table 12.1: Mean progress of works of excavators

Mechanism type	Bucket capacity	Soil type	
		Rock	Loose soil
Backdigger	0.65	-	313m^3/day
Front acting shovel	1.0	168 m^3/day	375m^3/day
Front acting shovel	1.5	320 m^3/day	423m^3/day

The erection of the penstock along the waterway alignment shall be executed with two truck cranes, each with a carrying capacity of 12.5 tons. The sections of pipeline are delivered and laid exactly at the sites, where the joints will be welded to a length of 30 m – 40 m up to a maximum weight of 15 tons. It is desirable to perform assembly and isolation of the penstock during warm periods of the year as it can be seen in the implementation schedule. The erection is carried out by mean of the truck cranes. After the quality of the weld joints and anticorrosive covering has been approved and tested, the backfilling of the trench shall be done by a bulldozer of 98 ton capacity.

12.1.2.3 Powerhouse and Appurtenant Structures

Access to the headworks need to be established by the construction of a 300 m long access road from an existing road. The access road shall be paved for future operation and maintenance requirements.

Before construction works are started the area needs to be cleaned from shrubs and bushes by means of bulldozers, stubbing machines and graders. The area is flat, so that progress of works is expected to be greater than in case of the headworks.

All soil and rock excavation works shall be carried out with the excavators, bulldozers and dump trucks. Rocks shall be preliminary loosened by drilling-shooting operations.

Before beginning the main concrete works for the powerhouse substructure installation of the manifold should be carried out. Progress and technology is similar to the one discussed in the section of the headworks. As soon as the groundwater is reached at the construction site, the columns and crane beams will be constructed. While the powerhouse crane is being erected the remaining works of concrete support structures is carried out.

The installation of the hydro-mechanical and electrical equipment in the machine hall shall be executed by means of the powerhouse crane.

12.1.3 Construction Equipment

The requirements of machine resources is determined by the bill of quantities, the mean progress of works and the local conditions on the site.

Table 12.2: Type and number of required Construction Equipment

No	Equipment	Quantity
1.	Caterpillar unitized excavator - bucket capacity 1.0 m^3	2
2.	Caterpillar unitized excavator - bucket capacity 1.25 m^3	1
3.	Wheel unitized excavator - bucket capacity 0.5 m^3	2
4.	Dump truck - lifting capacity 8-10 ton	18-20
5.	Autoconcrete pump	3
6.	Lorry – carrying capacity 10 ton	2-3
7.	Semitrailer cart-horse – carrying capacity 13.5-20 ton	2
8.	Truck tractor	4
9.	Bulldozer with the capacity of 96 kW with the hydraulic drive of the rock spoil	4
10.	Truck crane – carrying capacity 10-16 ton	3 + 2
11.	Welding transformer 34 kVA	4
12.	Overhead mobile compressor – productivity 10 m^3/min, 8 bar	3
13.	Boring rig with the submersible hammer – diameter 105 mm	8
14.	Perforator/coal hammer/manual - average diameter 42 mm	8
15.	Concrete mixing facility – productivity 15 m^3/h /two mixers – capacity 0.5-0.75m^3/	1
16.	Mobile, silo cement storage – capacity 25 ton	2
17.	Drainage pumps – 20-50m^3/h	4
18.	Manual vibrator – capacity up to 1.0 kW	6-8

12.2 Implementation Schedule

Based on main assumptions and considerations regarding progress of work given above a tentative implementation schedule for the present Gargar SHPP containing all major activities is given in Annex 12.

The total construction period was calculated to 20 months. According to the schedule the construction work can start at the earliest at the beginning of 2005. If the works start at that time, the construction is expected to be completed at the end of 2006.

The prerequisite for the presented time schedule is, that detailed design, preparation of tender documents as well as tender evaluation will be completed by the end of March 2005.

The production of the hydro-mechanical and electrical equipment shall take place between 2005 and mid of 2006. The installation of equipment is expected to take place at the end of 2006, so that the testing and commissioning of all equipment might be carried immediately after this. Consequently the project is expected to be connected to the grid and to be handed over to the Client at the end of 2006.

12.3 Disbursement Schedule

The disbursement of costs for the project is shown in the following table. At present the start of construction works in the beginning of 2005 is considered as planning basis.

Table 12.3: Disbursement of Costs

Disbursement			
%	-2	-1	Total
Civil Works	65%	35%	100%
Hydraulic Steel Structures	50%	50%	100%
Mechanical Equipment	30%	70%	100%
Electrical Equipment	30%	70%	100%
Transmission Line	20%	80%	100%
Environmental Mitigation Cost	100%	0%	100%
Engineering, Supervision	70%	30%	100%

13

Financial Analysis

13. Financial Analysis

13.1 Financial Analysis

Currently SHPPs can be developed in Armenia at a comparatively low price, but markets are in transition, and prices are tending towards the international level. Therefore two cost estimates have been prepared for Gargar SHPP, one at local price level and the other at international price level. While the local cost estimate is relevant for SHPP development today, the international cost estimate will become relevant in the future. In the financial analysis both cost estimates are considered.

Capital investment costs are estimated as 3,808 TUS$ (local prices) and 5,845 TUS$ (international prices) – see Chapter 11. These costs include direct costs, physical contingencies, engineering & supervision, and duties for imported equipment. When price contingencies are added (assuming 2% annual inflation until start of construction), the costs increase to 3,884 TUS$ (local prices) and to 5,962 TUS$ (international prices).

Key technical and economic parameters for the financial analysis are summarized in the following table.

Table 13.1: Key Technical and Economic Parameters

Item	Unit	Parameter	
Installed capacity	MW	3.2	
Energy generation	MWh	12,190	
Station use (1%)	MWh	122	
Transmission losses (1%)	MWh	121	
Useful output	MWh	11,947	
Construction period	Years	2.0 (incl. 4 months planning)	
Economic life civil works	Years	30	
Economic life equipment	Years	30	
Operating period	Years	30	
Tariff	c/kWh	4.5	
		local prices	internat. prices
Investment cost (w/o VAT)	000 US$	3,844	5,962
Specific investment cost	US$/kW	1,229	1,887
Specific investment cost	US$/kWh	0.32	0.49
Annual O&M cost (1% of direct cost)	000 US$	35	54

The tariff for SHPPs at natural flows has been set at a constant rate of 4.5 c/kWh (without VAT) by the Regulatory Commission. This tariff is guaranteed until 2016 only. For the purpose of the financial analysis it is assumed that this tariff will apply during the entire operating period of 30 years.

Financial indicators have been calculated over the project life comprising a planning & construction period of 2 years and an operating period of 30 years at discount rates of 10%, 12% and 14%, as is standard practice in Armenia. In addition, a discount rate of 8% has been applied - as required by some international donors - and a discount rate of 16% - equivalent to the profit norm commonly accepted for SHPPs by the Regulatory Commission.

The analysis for the cost estimate in local prices leads to the following results, as shown in the table below:

- At a discount rate of 10%, the project has dynamic production costs (DPC) of 3.9 c/kWh, a net present value (NPV) of 415 TUS$ and a benefit/cost (B/C) ratio of 1.15. At a discount rate of 12% the DPC are 4.6 c/kWh, the NPV is -33 TUS$ and the B/C Ratio is 0.99.
- The internal rate of return (IRR) of the project is 11.8%.

Whether the project can be considered feasible based on these indicators, depends on the financing conditions of the project: If the IRR is higher than the weighted average cost of capital (WACC) of the project (i.e interest rate of bank loan and investor's return on equity ROE), the project is financially feasible. Local interest rates have been very high, and therefore the Regulatory Commission has in the past accepted a profit norm of around 16%, which reflects the high WACC, for SHPPs. Assuming that financing can be arranged at more favorable conditions, the Regulatory Commission now expects the WACC to be in the range of 10-12%.

With an IRR of 11.8%, Gargar SHPP would thus be considered financially feasible, provided that financing can be arranged at an appropriate WACC.

Table 13.2: Key Financial Indicators (local price basis)

Discount Rate	Unit	8%	10%	12%	14%	16%
DPC	c/kWh	3.3	3.9	4.6	5.2	6.0
NPV	TUS$	1,107	415	-33	-325	-513
B/C Ratio	-	1.37	1.15	0.99	0.86	0.76
IRR	%	11.8%				

13.2 Sensitivity Analysis

A sensitivity analysis was carried out to test the effect of changes in investment costs on the key financial indicators. The following three cases were considered: a reduction in costs (local prices) by 10%, an increase by 10%, and an increase to international price level which is about 50% above the local price level. The results in the table below show that:

- a 10% reduction in investment costs (local prices) increases the IRR to 13.2%;
- a 10% increase in investment costs (local prices) reduces the IRR to 10.7%, a rate at which the project can no longer be considered financially feasible;
- when the investment costs reach international price level, the IRR is only 6.7%, and the project is clearly not feasible.

Table 13.3: IRR for Different Cost Levels

	Unit	Base Case	Cost (local) -10%	Cost (local) +10%	Internat. Prices
IRR	%	11.8%	13.2%	10.7%	6.7%

The key indicators for the project at international prices are shown in more detail in the table below.

Table 13.4: Key Financial Indicators (international price basis)

Discount Rate	Unit	8%	10%	12%	14%	16%
DPC	c/kWh	5.1	6.0	7.0	8.1	9.2
NPV	TUS$	-516	-1,057	-1,376	-1,554	-1,643
B/C Ratio	-	0.89	0.75	0.64	0.56	0.49
IRR	%	6.7%				

In addition, another sensitivity case was investigated, based on the assumption that inexpensive second hand pipes can be used for the penstock. According to verbal information from a potential investor, who purchased such pipes already, the price for pipes with a diameter of 1.2 meters is US$ 300 per ton. The assumed reduction in costs for the penstock brings total investment cost (local prices) down by about 12% from 3,808 TUS$ to 3,346 TUS$. Considering the increased pipe diameter, annual energy is increased to 12.39 MWh. As a result, the IRR of the project increases to 13.9%.

The IRR was also calculated for different tariff levels. The results show that:

- in the case of local price level, a rise in the tariff to 5.0 c/kWh would bring the IRR up to 13.3% and increase the profitability of the project;
- in the case of international price level, the IRR is still only 7.9% at a tariff of 5.0 c/kWh, so that the project cannot be considered feasible today.

Table 13.5: IRR for Different Tariff Levels

Tariff (c/kWh)	Unit	4.5	5.0	5.5	6.0
Local prices	%	11.8%	13.3%	14.7%	16.1%
Internat. Prices	%	6.7%	7.9%	8.9%	10.0%

13.3 Project Financing

Whether the project is attractive for an investor depends on the financing arrangements and the profit expectations of the investor, which together determine the WACC. For an investor, the ROE is the more relevant indicator than the IRR.

Interest rates in Armenia are currently so high (up to 20% p.a.) that a project with an IRR of 11.8% provides no return on equity (ROE) for the investor after the loan has been repaid. The investment costs of 3.8 million US$, however, make it unlikely that the project can be developed with equity capital only. To overcome problems like this, international donors are planning to establish one or more revolving funds for the financing of renewable energy projects. Although financing conditions have not been worked out yet, it may be assumed that these funds can provide loans at interest rates of about 8-9% with loan terms of 7-10 years.

Total financing requirements for Gargar SHPP, including financing fees and interest during construction, then amount to 4,158 TUS$ (local prices) and 6,388 TUS$ (international prices).

In order to assess the financial performance of the project with financing considered, financial statements (cash flow statement, income statement, balance sheet) have been set up over the operating period of 30 years. The key assumptions concerning financing of the project are summarized in the table below.

Table 13.6: Key Financing Assumptions

Item	Parameter
Debt/equity ratio	70/30
Interest	9%
Loan term	10 years
Grace period	construction period
Interest on overdraft	12%
Interest received	5%
Accounts receivable	30 days
Accounts payable	30 days
Inventory	180 days
Profit tax	20%

The financial indicators calculated with the help of the financial statements are presented in the tables below, separately for the case of local prices and international prices, and for different tariffs.

Table 13.7: Financial Performance (Local Prices)

Tariff (c/kWh)	Unit	**4.5**	5.0	5.5	6.0
IRR	%	**11.8%**	13.3%	14.7%	16.1%
ROE pre-tax	%	**12.5%**	15.3%	17.9%	20.5%
ROE post-tax	%	**10.7%**	12.7%	14.7%	16.8%
Investor's payback period	years	**12.5**	11.0	10.0	8.2
Minimum DSCR	-	**0.93**	1.04	1.15	1.26
Maximum DSCR	-	**1.65**	1.85	2.05	2.24
Average DSCR	-	**1.19**	1.37	1.52	1.67

Table 13.8: Financial Performance (International Prices)

Tariff (c/kWh)	Unit	**4.5**	5.0	5.5	6.0
IRR	%	**6.7%**	7.9%	8.9%	10.0%
ROE pre-tax	%	**0%**	4.0%	6.6%	8.8%
ROE post-tax	%	**0%**	4.4%	6.6%	8.2%
Investor's payback period	years	**>30**	23.2	18.1	15.5
Minimum DSCR	-	**0.56**	0.65	0.73	0.80
Maximum DSCR	-	**0.58**	0.72	0.90	1.11
Average DSCR	-	**0.57**	0.68	0.79	0.92

The results show that at local prices:

- the investor's ROE (pre-tax) is only 12.5%, although the assumed interest rate of 9% is below the project's IRR of 11.8%; this is because in the early years the revenue is not sufficient to service the debt, and an additional loan (overdraft) at a higher interest rate is required. During the loan term of 10 years, the debt is serviced first, and the dividend payments to the investor are postponed to later years which reduces the ROE;
- the profit tax of 20% reduces the investor's ROE considerably;

- at a tariff of 4.5 c/kWh the project has an ROE (post tax) of 10.7% and a payback period of 12.5 years; investors may not consider this attractive.

At a tariff of 5.0 c/kWh, the financial indicators are more favorable (ROE post-tax of 12.7% and investor's payback period of 11 years). These values may already be in an acceptable range, but a tariff of 5.5 c/kWh would be required to make the project attractive for investors (ROE of 14.7% and payback period of 10 years). The project would also be more attractive to investors, if financing was available at a lower interest rate than the assumed 9% p.a. or – more importantly – with longer repayment periods than the assumed ten years.

At international prices the project is not financially feasible even at a tariff of 6.0 c/kWh.

Under the assumption that the project costs (local prices) can be reduced by installing inexpensive pipes, the ROE (post-tax) increases to 14.4% and the investor's payback period decreases to 11.2 years at a tariff of 4.5 c/kWh. Although these results are more encouraging than for the base scenario, it will still require further efforts (such as favorable financing arrangements or a higher tariff) to make the project attractive for investors.

Table 13.8 Financial Performance (Local Prices – Alternative Scenarios)

		Base Scenario	Low Cost Scenario
Tariff (c/kWh)	Unit	4.5	4.5
IRR	%	11.8%	13.9%
ROE pre-tax	%	12.5%	16.5%
ROE post-tax	%	10.7%	14.4%
Investor's payback period	years	12.5	11.2

13.4 Commercial Risk Analysis

When making his investment decision, the investor has to consider the following commercial risks:

Tariff risk: The tariff of 4.5 c/kWh is only guaranteed until the year 2016. Thereafter, the tariff will be determined by the market. It is likely that the tariffs for power plants in Armenia will increase in the future when new power plants are generating at distinctly higher prices than the existing plants. SHPPs may therefore be able to get cost-covering tariffs for the remaining operating period.

Market risk: The Energy Law (Art. 59.1) guarantees only until 2016 that all energy generated by SHPPs will be purchased. Thereafter, the SHPP has to compete with other plants.

Licensing risk: The operating license is normally issued for a period of 15 years. There is a risk that the license will not be extended for the remaining 15 years, but this has been rarely the case in the past.

Hydrological risk: The actual energy generation in any given year may differ from the estimated average annual generation. When less energy is generated, the operating revenues may not be sufficient to cover the costs. The hydrological risk may be considered small in the case of Gargar SHPP. The plant may build up reserves in years with higher than average generation for use in years with below average generation.

Permit risk: The investor may not get the required permits, such as water permits, or may face conflicting water rights of other parties. In Armenia drinking water supply and irrigation have a higher priority than hydropower. For Gargar SHPP this risk can be considered low.

Payment risk: The investor may have difficulties in receiving the payment for the project output. The Single Buyer Armenergo was notorious for the delay of payments to the energy generators. With the restructuring of the energy market (privatization of the distribution companies, establishment of a payment settlements centre, phasing out of Armenergo and transfer of the single buyer function to the transmission company) this problem has been addressed. The time lag between energy sales and payments is now already considerably shorter and is expected to decrease further.

14

Summary and Conclusion

14. Summary and Conclusion

14.1 Summary

Gargar SHPP is a byproduct of the analysis carried out by the Consultant for the original planning of the Loriberd Cascade Project elaborated by ArmHydroEnergoProject. The analysis showed, that the development of the first stage powerhouse Loriberd HPP 1 would not be economical. Therefore the Consultant Fichtner recommended to develop the scheme as a separate small hydropower development project, which utilizes the natural head of the Gargar River between the villages of Vardablur and Kurtan down to the confluence point of the Dzoraget and Gargar Rivers. The gross head of the Gargar River in this reach is approximately 250 m. In this way the Consultant identified the new project “Gargar SHPP”.

For the present Feasibility Study a sound data basis was elaborated by the Consultant. Comprehensive desk and field studies were carried out in order to ensure the definition of reliable design parameters for the development of the scheme. The main results can be found in the relevant sections and annexes to of the present Feasibility Study report:

- Topography
- Hydrology
- Sediment Transport
- Geology
- Environmental Impact Assessment

During the elaboration of the Feasibility Study three different layout alternatives were considered for the determination of the most economic layout. For the layout screening the headworks for all these alternatives was selected to be on the same site, all layout alternatives were developed as run-of-river project. The considered alternatives are summarized below:

- Penstock along the Gorge of Gargar River
- Free Flow tunnel along the Lori Plateau;
- Pipetunnel crossing the Lori Plateau massif towards Dzoraget River

During the screening process the layout alternatives were evaluated in terms of hydraulics, energy production, expected benefits and costs. As a result the penstock solution along the river gorge was found out to be the most economical solution.

In the next step of the elaboration of the study an optimization of the selected layout was carried out. The optimization was focused on the most economic reach along the river to be developed for power generation as well as on the selection of the design discharge.

For this purpose the powerhouse was selected on one site and three different weir locations were investigated in depth. Thereby three different length of the waterway were considered:

- length of 5525 m
- length of 3848m
- length of 2397m

All alternatives comprise the steepest parts of the river. Corresponding parameters of the reaches, such as construction costs, installed capacity and power generation were calculated in depth. The most economic reach was determined by a benefit cost analysis. As a result the shortest penstock solution along the river gorge was found out to be the most economic one. On basis of detailed topographical maps in the scale 1:500, geological reconnaissance and site visits the weir site was decided to be shifted further downstream so that the waterway was finally reduced to 2160 m.

In a second step the design discharge was selected. For this determination power and energy calculations for various discharges between 1.0 m^3/s and 3.2 m^3/s were calculated. Together with the corresponding costs the benefit cost ratio was calculated, which was the basis for the selection of the design discharge. The final design discharge was selected for Gargar SHPP as 1.8 m^3/s.

Based on the final layout the hydraulic design was carried out for Gargar SHPP, which consists of following structures and equipment:

- Weir with the length of 18.5 m and a height of 3.9 m. The length of the front wall is 16.5 m consisting of 3 bays, each with a length of 5.5 m. The Tyrolean weir with a collecting channel width of 1.5 m is designed on the weir crest in two bays.
- The width of the gravel trap is equal to 3m. The length is 16.3 m and the height is 4.9 m. The gravel trap is provided with the vertical gates of 1.5 m x 1.5 m from upstream and downstream. The gravel trap has two functions
  - Trap of fine grain size bed load, entering through the intake rack of the Tyrolean weir and 1.5 x 1.0 m vertical sliding gate located in the left wall of the weir gallery
  - Flushing of deposited bedload via the downstream gate
- Two chamber sandtrap with a length of 35 m. Each chamber has a width of 2 m and is equipped with the vertical gates for alternate flushing of accumulated suspended loads through a flushing gate with the dimensions of 1.5 m x 1.5 m. The loads will be removed by a flushing channel under pressure with the length of 63 m. The section of the flushing channel is rectangular with a width and height of 1.4 m and 1.0 m respectively.
- The suspended sediment free water is conveyed to the chamber, where the intake to the penstock is foreseen. At the intake a vertical roller gate is installed with a size of 1.5 m height and width.
- Fishpass on the left side of the weir, consisting of 7 chamber with a total length of 19 m and a width of 1.2 m
- Embedded penstock with a length of 2160 m and a diameter of 1.0 m, which crosses the Gargar River three times
- Surface powerhouse with one Pelton unit with four jets and a gross capacity of 3.16 MW. A 6/35 kV switchyard is located on the powerhouse area
- Tailrace channel with the length of 50 m. The section of the open channel is trapezoidal with the bottom width of 1.1 m, a height of 1.5 m and side slopes in the ratio 1:1

The installed capacity of the planned hydropower scheme is 3.16 MW. The expected mean annual energy production is 12.19 GWh.

Cost estimates for civil works, hydraulic steel structures, electrical equipment and the transmission line were based on bill of quantities on a local price level. The hydro-mechanical equipment cost estimates consider the Consultant's data bank information and budget quotation. As requested from the Ministry of Energy it was based on the assumption that the hydro-mechanical equipment should be imported from Western countries and is therefore based on an international price level. The total investment cost including physical contingencies in the magnitude of 5% for all works and items were estimated as 3.81 MUS$ at mid 2004 price level.

The total expected construction time is 20 months. The project might be implemented at the earliest by March 2005. The earliest commissioning is expected at the end of 2006.

At the current tariff of 4.5 c/kWh, the Gargar SHPP has a comparatively low IRR of 11.8%. If developed by the Government, the project is feasible at this IRR. But private development involves some financial risks, and further efforts may be required to make the project attractive for investors (such as favorable financing conditions, tax privileges, or a tariff of at least 5 c/kWh).

Considering the price trends in Armenia, it is not unlikely that construction costs will rise to international price level in the future. If this is the case, Gargar SHPP will clearly not be feasible, unless the tariff is increased considerably.

14.2 Conclusion

In the present Feasibility Study the most economic layout alternatives for developing the hydropower potential of Gargar River between the village of Vardablur and its confluence with Dzoraget River were elaborated.

Within the Feasibility Study extensive desk and field investigations were carried out in order to establish a sound data basis for a reliable cost estimation.

Based on the investigations and analysis carried out it can be concluded, that the identified project Gargar SHPP is technically feasible, no major obstacles are expected during the implementation of the project. The project might also be developed in two phases. In the first phase only one turbine generator unit might be installed while the civil works shall already be constructed for the final design of the project. This has a positive effect for the cash flow of a private investor.

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Considering the price trends in Armenia, it is not unlikely that construction costs will rise to international price level in the future. If this is the case, Gargar SHPP will clearly not be feasible, unless the tariff is increased considerably.

Besides the monetary benefits of the implementation of Gargar SHPP secondary benefits shall be mentioned here as well. The two year long implementation period provides considerable job opportunities in the Lori district, which has a high unemployment rate at present. This may reduce the migration of the labor force either to the capital Yerevan or to Russia, and improve the general economic situation in the city of Stepanavan and the surrounding villages.

Other benefits of the project include the avoidance of greenhouse gas emissions from alternative thermal power plants, savings in foreign currency in particular for fuel imports. The scheme provides an alternative renewable energy resource, which makes Armenia less dependent on imported fuel.