REPUBLIC OF AZERBAIJAN
Second National Water Supply & Sanitation Project
(SNWSSP)
AMELIORATION AND WATER MANAGEMENT OPEN
JOINT STOCK COMPANY
(AWM OJSC)

FINAL DAM SAFETY ASSESSMENT
FOR VILESHCHAY WATER RESERVOIR (VWR)
IN MASALLI, AZERBAIJAN

Prepared by:

Les Bromwell, P.E., Sc.D.
Civil Engineering Consultant
505 Tulip Lane
Vero Beach, Florida 32963

Walter H. Faulk, Jr., P.E.
Civil Engineering Consultant
945 Deer Hammock Circle
St. Augustine, FL 32080

Hilmi Omuzluoglu, P.E.
Civil Engineering Consultant
31 Hamiyet Yuceses Street 11,
Kadikoy, Istanbul, 34740, Turkey

June 2016
(Revised December 2016)
Document Title : Dam Safety Assessment
Submitted Date : June, 2016 (Revised November 2016)
Revision Date : December, 2016
Project Name : Second National Water Supply and Sanitation Project (SNWSSP) of Republic of Azerbaijan
Client : Amelioration and Water Management Agency of Azerbaijan
Credit Number : 4937 – AZ
Consultancy : Preparation of Dam Safety Assessment Report for Vileshchay Water Reservoir in Masalli
Consultant : Bromwell, Les; Faulk, Walter H.; Omuzluoglu, Hilmi.
Date of Contract : May 21, 2014
Dissemination Level : Second National Water Supply and Sanitation Project Implementation Unit (PIU)
# TABLE OF CONTENTS

1.0 **BACKGROUND** .......................................................................................... 1

2.0 **SCOPE OF THE WORK** .............................................................................. 1

3.0 **PROJECT SITE OVERVIEW** ..................................................................... 2

   3.1 Site Location ............................................................................................... 2

   3.2 Regional Climate ....................................................................................... 2

   3.3 Regional Geology ..................................................................................... 2

   3.4 Vileshchay River Hydrology .................................................................... 3

   3.5 Groundwater ............................................................................................. 3

4.0 **PREVIOUS DAM INSPECTIONS AND MONITORING** ......................... 3

5.0 **PREVIOUS FIELD EXPLORATIONS** ...................................................... 4

6.0 **PREVIOUS LABORATORY TESTING** ....................................................... 4

   6.1 Empirical Estimation of Hydraulic Conductivity ..................................... 5

   6.2 Empirical Estimation of Compression Index and Coefficient of Secondary Compression ................................................................. 6

7.0 **SUBSURFACE STRATIGRAPHY** ............................................................. 6

8.0 **SEEPAGE ANALYSIS** ............................................................................ 8

   8.1 Seepage Model Development ................................................................. 8

   8.2 Seepage Model Elements ....................................................................... 8

   8.3 Seepage Model Hydraulic Input Parameters ....................................... 9

   8.4 Seepage Model Analyses ....................................................................... 10

   8.5 Critical Horizontal and Vertical Seepage Gradients .......................... 10

   8.6 Seepage Model Results ....................................................................... 10

9.0 **SLOPE STABILITY ANALYSIS** ............................................................. 12

   9.1 Slope Stability Model Development .................................................... 12

   9.2 Slope Stability Model Soil and Rock Shear Strength Parameters ....... 13

   9.3 Cross Sections for Slope Stability Analysis ......................................... 14

   9.4 Seismicity ............................................................................................... 14

   9.5 Results of Slope Stability Analysis ....................................................... 15

10.0 **EVALUATION OF DAM FREEBOARD** ................................................ 16

   10.1 Structural Hydraulic Response ........................................................... 16

   10.2 Results of Freeboard Analysis ............................................................. 18

11.0 **UPSTREAM AND DOWNSTREAM SLOPE PROTECTION** ............... 18

12.0 **DESIGN OF GRANULAR FILTER TRANSITIONS** ............................ 20
TABLE OF CONTENTS (continued)

12.1 Retention and Segregation Criteria ............................................................... 20
12.2 Permeability Criterion .................................................................................... 21
12.3 Porosity Criterion ............................................................................................ 21

13.0 SEEPAGE CUT-OFF WALL ............................................................................... 21

14.0 WATER DISCHARGE STRUCTURES ............................................................... 21
14.1 Assessment of the Hydraulic Capacity of Bottom Tunnels and Derivation Pipes 22
14.2 Discharge Spillway .......................................................................................... 32

15.0 EMERGENCY PREPAREDNESS PLAN ......................................................... 34
15.1 Replacement of the Gates for Water Supply Pipes and Discharge Tunnel ......... 35

16.0 CONCLUSIONS .............................................................................................. 35

17.0 RECOMMENDATIONS ..................................................................................... 36
17.1 Phase I Dam ..................................................................................................... 36
17.2 Phase II Dam .................................................................................................... 37
17.3 Costs of the Measures recommended for Phase I Dam ................................. 39

18.0 DISCLAIMER .................................................................................................... 40

19.0 REFERENCES .................................................................................................. 41

LIST OF TABLES

TABLE 1: Subsurface Geologic Soil Delineation and Corresponding Soil and Rock Descriptions ........................................................................................................... 7
TABLE 2: Estimated Soil and Rock Hydraulic Flow Parameters .................................. 9
TABLE 3: Results of Seepage Model Sensitivity Analysis ............................................ 11
TABLE 4: Estimated Effective Soil and Rock Effective Shear Strength Parameters ...... 13
TABLE 5: Representative Regional Seismic Data .......................................................... 14
TABLE 6: Results of Slope Stability Analysis ............................................................. 15
TABLE 7: Results of Freeboard Analysis ................................................................... 18
TABLE 8: Riprap Stone Revetment Nominal Diameter Requirements for Upstream Dam Embankment Slope Protection ................................................................. 19
Table 9 Available Data .................................................................................................. Hata! Yer işareti tanımlanmamış.
Table 10 Friction Loss Calculation ............................................................................ 26
Table 11 Minor Loses ............................................................................................... 26
Table 12 Energy Conservation ................................................................................. 27
Table 13 Available Data...........................................................................................................27

LIST OF TABLES (continued)

Table 14 Friction Loss Calculation..........................................................................................28
Table 15 Minor Loses................................................................................................................28
Table 16 Energy Conservation.................................................................................................28
Table 17 Available Data............................................................................................................29
Table 18 Friction Loss Calculation..........................................................................................30
Table 19 Minor Loses................................................................................................................30
Table 20 Energy Conservation.................................................................................................30
Table 21 Available Data............................................................................................................31
Table 22 Friction Loss Calculation..........................................................................................31
Table 23 Minor Loses................................................................................................................31
Table 24 Energy Conservation.................................................................................................32
Table 25 BoQ for costs of fundamental repair of Spillway .................................................... 39
Table 26 Costs of Measures Recommended for Phase I Dam .............................................. 39

LIST OF FIGURES

FIGURE 1 Site Vicinity Map
FIGURE 2 Vileshchay Water Reservoir Stage-Storage Relationship for 1st Phase
FIGURE 3 Vileshchay Water Reservoir Exceedance Probability-Discharge Relationship for 1st Phase

LIST OF APPENDICES

APPENDIX A Excerpt From Design Report for Vileshchay Water Reservoir (VWR)
APPENDIX B Visual Surveillance Survey Results and Recommendations for VWR Summary of the Dam Safety Condition of Dam
APPENDIX C Minutes of 2015 Inspection by Azerbaijan Government Committee
APPENDIX E Subsurface Geologic Profile Along Longitudinal Axis of Dam
APPENDIX F Results of Seepage Analysis
APPENDIX G Results of Slope Stability Analysis
APPENDIX H Calculations
APPENDIX I Results of Hydrological/Hydraulic Analyses
1.0 BACKGROUND

The Vileshchay Water Reservoir (VWR), located in the administrative region of Masalli of the Republic of Azerbaijan, was designed to supply potable drinking water to Jalilabad and Masalli cities, and to retain water for irrigation of farmland. The site vicinity map and General Layout drawing are shown in Figure 1.

- The reservoir was designed to be constructed in two Phases. Phase I, which was put into operation in 1986, has a crest elevation of 80 meters above sea level (masl\(^1\)), with 46 million cubic meters at normal operating level of 75.5 meters. The reservoir capacity provides irrigation water for approximately 10,000 hectares of farmland. Measured stage level vs. storage volume during operation of the Phase I reservoir is shown in Figure 2.

- The Phase II construction would increase the dam height by 13.5 meters to Elevation 93.5, providing a total of 132 million cubic meters reservoir capacity to serve an additional 15,000 hectares of farmland.

- During the Phase I construction, irrigation pipes, and outlet tunnels were constructed at Station 18+40 and an emergency spillway was constructed on the left bank of the river at Station 8+40. When the dam was built, the irrigation pipes and outlet tunnels were designed and built for Phase II, and the dam was built wider than needed for Phase I in order to facilitate raising it to the Phase II elevation. The positions of the hydraulic facilities were selected to prevent flooding and instability of river banks on which residential settlements are located downstream of the dam.

The Lankaran lowland is very densely populated from the site to the Talysh Mountains to the east, and the relative locations of a number of settlements descend from the elevation of the dam crest down to the valley of the Vileshchay River. These settlements include the Gariblar, Khalfalar, Abbasbayli, Ege, Allahyarli, Arkivan, Shikhlar, and Isi villages. The Isi village was resettled, and the Istisu mineral spring was protected during construction of the dam.

2.0 SCOPE OF THE WORK

Our work for this project consisted of the following objectives:

- Evaluate the condition of the Vileshchay water reservoir and its appurtenant structures based on a review of inspection and monitoring reports.

- Perform an independent technical review of the available design documents to evaluate the safety of the dam and its appurtenant structures in accordance with WB OP/BP 4.37, DIN 4084, EC-7, or other internationally accepted analysis methods.

- Determine if the dam has been inspected regularly and satisfactorily.

- Review the available documents and data and provide recommendations for Phase II construction.

- Develop a dam safety assessment report including any safety concerns and recommendations for remedial action.

\(^1\) All elevations in this report are in meters above sea level (masl) unless otherwise noted.
3.0 PROJECT SITE OVERVIEW

3.1 Site Location

The VWR is located in Masalli Rayon of the Republic of Azerbaijan. It is a reservoir created by flows of the Vileshchay River, which is one of the largest rivers in Azerbaijan that flow into the Caspian Sea. The reservoir is situated between the foothills of the Talysh Mountains and the Lankaran lowland. The lowland extends from the Astara region to the Bilasuvar region, and ultimately to the Caspian Sea.

The Vileshchay River begins at Kuludash peak located in the Talysh Mountains at Elevation 2,203 masl.

3.2 Regional Climate

According to the original design report, a portion of which was translated into English and provided to us (see Appendix A), a range of different climatic conditions is exhibited within the Lankaran lowland. These conditions can range from cold and arid to relatively warm and wet. Generally, cold and arid conditions are found in the mountainous regions at elevations in excess of 2,000 meters above sea level, and relatively warm and dry conditions are found in the lowland areas and adjacent foothills.

Precipitation normally occurs during spring and autumn. The average annual precipitation in lowland areas varies between 540 and 850 millimeters, and the annual precipitation in areas located at elevations of 100 to 500 meters above sea level is 1,250 millimeters.

The average annual temperature exhibited in the Lankaran lowland is 14 degrees Celsius. The minimum and maximum annual temperatures of -23 and 41 degrees Celsius are observed in January and August, respectively. Substantial increases and decreases in temperature are observed at the beginning of April and at the end of November, respectively.

3.3 Regional Geology

As described in the design report in Appendix A, the VWR is located on the border of the alluvial-prolluvial foothill line, and the lower portions of the foothill has been infilled with prolluvial-delluvial sediments as a result of erosion of the Talysh Mountains. The geological nature of the area below and surrounding the reservoir consists predominantly of third and fourth period rocks and sediments.

The Tertiary period rocks consist of argillaceous rocks with siltstones, tuff, and sandstones with thicknesses on the order of a few centimeters to a few dozen centimeters to dozens of meters. The Quaternary sediments are found in various thicknesses and consist primarily of eroded prolluvial-delluvial sediments containing argillaceous rocks and rock fragments. Within the boundaries of the valley, these fourth period sediments are overlain by sands, and sandy and gravelly alluvials. These sandy and gravelly alluvials are in some locations infilled with argillaceous clays.
3.4 Vileshchay River Hydrology

Approximately 70 to 80 percent of the Vileshchay River flow is attributed to floods and torrents, and the remaining 20 to 30 percent is attributed to melting snow. River floods and torrents are observed throughout the year but typically occur during the cooler months; annual maximum flows occur during the spring in March, April and May. The average annual river flow is approximately 1.5 m$^3$/s. Measurements of river flow have been recorded on an annual basis since 1938.

3.5 Groundwater

Infiltration contributes to the flow of groundwater along the bottom of the ancient Vileshchay and Matalichay River valleys. Groundwater has generally been found at depths of 1 to 3 meters within the boundaries of the valley and at depths of 20 to 30 meters along the edges of the valley. More specifically, the groundwater depth varies between 1 and 5 meters at the bed of the Vileshchay River and between 0.5 and 2.5 meters along the bed of the Matalichay River. The groundwater depth beyond the river bed varies between 15 and 50 meters. The direction of the groundwater flow is consistent with the flow of the Vileshchay River, and based on mineralization, the groundwater consists of fresh water.

4.0 PREVIOUS DAM INSPECTIONS AND MONITORING

The Vileshchay Water Reservoir administration has performed periodic assessments of the dam, reservoir, and discharge structures and appurtenances. A summary of visual surveillance survey results and recommendations based on the results of past assessments is provided in Appendix B. The 2015 Inspection Minutes by the Azerbaijan Government are in Appendix C. The key results and recommendations are as follows:

- There are no signs of significant seepage along the downstream embankment slopes of the dam as indicated by lack of excessive moisture or saturation, vegetation changes along the toe, or sloughing of the dam slopes.
- There are no signs of significant seepage below or around the dam discharge structures, as would be indicated by high exit gradients, downstream turbidity, dissolved solids, or variations in pressure differentials and discharge flow quantities. Note that the World Bank Task Team inspection in March 2016 (see Appendix D) found severe leakage on the right side of the spillway connecting to the dam shoulder at the left bank. The task team emphasized that this is a high potential risk for safety of the dam and spillway. The task team recommended the dam operator pay close attention to the development of the leaking point and repair it as early as possible.
- Seepage water was not observed through drainage pipes as no seepage collection pipes were installed within the Phase I dam. The VWR administration indicated that seepage water outlet pipes will be installed during the Phase II construction.
- There are no signs of slope instability along the upstream or downstream embankment slopes, and there are no signs of depressions or sinkholes along the crest of the dam. No changes in the dam fill, including dissolution, loss of plasticity or cementation, susceptibility to shearing, crumbling, nor shrinkage or swelling have been observed.
There are no signs of drainage problems as indicated by the lack of chemical precipitates or deposits, areas of stagnation, or areas of algal or bacterial growth along the toe of the upstream or downstream embankment slopes of the dam.

There is no evidence of settlement, displacement, translation, consolidation, subsidence, buckling, bending, cracking, expansion or contraction, shearing, creeping, or crushing in or around the various dam components, structures, and appurtenances based on the results of surveys and field inspections.

There is no evidence of loss, translation, or deterioration of the upstream or downstream rock face slope protection systems. Rock fill is in good condition.

There is no evidence of consolidation or liquefaction in the dam foundation.

There is no evidence of mechanical error or malfunctions in hydraulic and headwater control structures. There is no evidence of vulnerability to obstruction, cracking, displacement, materials deterioration, cavitation, leakage, etc. to shafts, conduits, or tunnels. (Note that the World Bank Task Team inspection in March 2016 found that the existing four gates, which are more than 30 years old, are in poor condition and should be replaced promptly). The task team was pleased to learn that the dam operator has secured the funds in the 2016 state budget to replace the gates. The task team recommends replacing the old gates as early as possible.

The walls and slab of the discharge spillway require adequate maintenance and rehabilitation. Excessive vegetative growth should be removed from the slab and walls of the spillway to enhance the discharge capacity of the structure. The walls and slab of the spillway require grouting to remediate spalling and some crumbling along the joints; however, no evidence of cracking, settlement, exposure of steel reinforcement or scouring at the downstream river bed has been observed during inspections or reconnaissance studies.

5.0 PREVIOUS FIELD EXPLORATIONS

Geotechnical data were collected during the performance of prior field investigations to characterize the geologic nature of the subsurface soil and rock materials found along the Vileshchay River banks and riverbed in the vicinity of the dam. Boring data were compiled during the investigations from a total of twenty-six Standard Penetration Test (SPT) borings performed prior to dam construction. The relative locations of the SPT borings are shown in Appendix E on the subsurface geologic profile along the centerline (axis) of the dam. The results of the field investigations demonstrate that the subsurface soil and rock strata generally consist of mixtures of gravels and sands, and clay and clay loams. An impervious layer comprised of dense mudstones and sandstones was encountered below the aforementioned mixtures of soil and rock materials.

The hydraulic conductivities of the soil and rock strata were determined by field permeability tests on five water injection wells located along the centerline of the dam.

6.0 PREVIOUS LABORATORY TESTING

A laboratory testing program was performed during dam design to characterize the various subsurface soil and rock formations. The testing program consisted of tests for soil classification, moisture content, particle size analysis, relative density, Atterberg limits,
unconfined compression, natural moisture, and particle size distribution using sieve analysis. The laboratory test results are summarized in Section 4.3.2 of the design report in Appendix A.

6.1 Empirical Estimation of Hydraulic Conductivity

The hydraulic conductivities of the various subsurface geologic materials are expected to be within the range of 0.1 to 100 meters per day based solely on our review of the classifications and particle size analyses of the various subsurface materials.

The hydraulic conductivities of the subsurface geologic materials were estimated using the empirical Kozeny-Carman relationship, which was originally developed by Kozeny (1927), and subsequently modified by Carman (1938, 1956). The relationship assumes D’Arcy (laminar) flow conditions. These assumptions are applicable for the sands and gravelly sands encountered at the site. The equation also assumes that the individual particles are relatively uniform and compact, and electrochemical reactions between the particles and pore fluid are negligible.

The Kozeny-Carman equation can be used predict the hydraulic conductivity of a porous medium as follows:

\[ k = \left( \frac{\gamma}{\mu} \right) \cdot \left( \frac{1}{C_{K-C}} \right) \cdot \left( \frac{1}{S_o^2} \right) \cdot \left[ \frac{e^3}{(1+e)} \right] \]

where:
- \( \gamma \) = unit weight of the permeant;  
- \( \mu \) = viscosity of the permeant;  
- \( C_{K-C} \) = Kozeny-Carman empirical coefficient;  
- \( S_o \) = Specific surface area per unit volume of soil particles (1/cm); and  
- \( e \) = void ratio of soil particles (varies between 0.6 and 0.7)

When the permeant is water with a temperature of 20 degrees Celsius, \( \frac{\gamma}{\mu} \) becomes 9.93x10^-04 1/cms, and the value of \( C_{K-C} \) becomes 4.8+0.3 assuming that the soil particles are uniform spheres. Thus, Equation (1) reduces to the following Equation (2):

\[ k = 1.99 \times 10^{-4} \cdot \left( \frac{1}{S_o^2} \right) \cdot \left[ \frac{e^3}{(1+e)} \right] \]

The shape factor accounts for the angularity of the individual soil particles, and is expressed using the following Equation (3):

\[ S_o = \frac{SF}{D_{eff}} \]

Where: SF varies between 6.0 and 7.7 for spherical and angular particle shapes, respectively, and \( D_{eff} \) is the effective grain size diameter of the soil particles expressed in centimeters.

If the soil particles are assumed to be uniformly spherical, Equation (1) reduces further to the following Equation (4):

\[ k = 552 \cdot (D_{eff})^2 \cdot \left[ \frac{e^3}{(1+e)} \right], \]

which is the commonly used formulation where \( k \) is expressed in centimeters per second.
The gradation test results indicate that the alluvial and alluvial-prolluvial sediments are generally well graded. The grain-size distribution curves were used to estimate the hydraulic conductivity of the subsurface granular materials, and hydraulic conductivities were empirically predicted using the Kozeny-Carman equation provided above. Using Equation (4), predicted hydraulic conductivities range between 0.001 and 100 meters per day. Note that a predicted hydraulic conductivity of 0.001 meter per day is representative of the delluvial-prolluvial clays of Layer 4, and predicted hydraulic conductivities between 2 and 100 meters per day are representative of the alluvial and alluvial-prolluvial sediments of gravel and sand of Layers 2 and 5, respectively.

6.2 Empirical Estimation of Compression Index and Coefficient of Secondary Compression

The compressive index \( C_c \) of the Layer 4 delluvial-prolluvial clays was estimated using empirical relationships developed by Azzouz (1976), Hough (1957), Nashida (1956), and Rendon-Herrero (1980). These relationships are provided below in Equations 5 through 11:

Azzouz (1976):
\[
C_c = 0.4 \cdot (e_0 - 0.25), \tag{5}
\]
\[
C_c = 0.01 \cdot (w_n - 5) \tag{6}
\]

Hough (1957):
\[
C_c = 0.4049 \cdot (e_0 - 0.3216), \tag{7}
\]
\[
C_c = 0.0102 \cdot (w_n - 9.15) \tag{8}
\]

Nashida (1956):
\[
C_c = 0.5217 \cdot (e_0 - 0.20), \tag{9}
\]
\[
C_c = 0.0054 \cdot (2.6w_n - 35) \tag{10}
\]

Rendon-Herrero (1980):
\[
C_c = 0.30 \cdot (e_0 - 0.27), \tag{11}
\]

Using Equations (5) through (11), the predicted values of compression index, \( C_c \), for the Layer 4 clays range between 0.15 and 0.37 with an average value of 0.26. This value of primary compression index is typical for a medium stiff to stiff clay. The ratio of the coefficient of secondary compression to the compression index \( (C_\alpha/C_c) \) for the Layer 4 clays is estimated at 0.04; therefore, the coefficient of secondary compression is equal to approximately 0.01.

7.0 SUBSURFACE STRATIGRAPHY

Based on the results of a review of the design report, six primary generalized soil and rock strata characterize the subsurface geotechnical profile at the site. The approximate subsurface soil and rock delineations, and corresponding geologic descriptions for the primary and secondary strata are summarized below in Table 1.
Dam fill materials are comprised of soil and rock materials borrowed from local quarries and fields. The core of the dam is composed of compacted layers of Layer 4 delluvial-prolluvial clays borrowed from Quarry No.1, which was opened along the left bank of the Vileshchay River at a distance of 0.5 to 1.0 kilometer from the dam body. The Quarry No. 1 clay reserves have been estimated at about 5.3 million cubic meters, and approximately 1.5 million cubic meters was used for the first stage of dam construction.

The portions of the dam located upstream and downstream of the core are comprised of mixtures of gravel and sand borrowed from fields located along the left and right banks of the river near the Khalfalar Village, and between the Arkivan Village and Masalli City. The dam transitions, or filters, are gravel and sand excavated from the field located inside the reservoir near the Shikhlar Village.

### TABLE 1: Subsurface Geologic Soil Delineation and Corresponding Soil and Rock Descriptions

<table>
<thead>
<tr>
<th>Stratum No.</th>
<th>Location(s)</th>
<th>Approximate Elevation Range (masl)</th>
<th>General Soil and Rock Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Central Dam Foundation / Riverbed</td>
<td>45 to 47</td>
<td>Alluvial clay loams with thin intermittent layers of gravel and sand</td>
</tr>
<tr>
<td>2</td>
<td>Central Dam Foundation / Riverbed</td>
<td>40 to 45</td>
<td>Alluvial sediments consisting of sandy loams, clayey loams, gravel, and sandstone</td>
</tr>
<tr>
<td>3</td>
<td>River Banks</td>
<td>Limited</td>
<td>Clay loams with thin intermittent layers of sand and gravel</td>
</tr>
<tr>
<td>3a</td>
<td>River Banks</td>
<td>Limited</td>
<td>Gravel, sand, and stone sediments with clay and sandstone</td>
</tr>
<tr>
<td>4</td>
<td>Left and Right Dam Foundation / River Banks</td>
<td>43 – 127² 47 - 97³</td>
<td>Pellaulvial-prolluvial clays with local intermittent layers of sand, gravel, and sandstone</td>
</tr>
<tr>
<td>4a / 4g</td>
<td>Left and Right Dam Foundation / River Banks</td>
<td>43 – 127² 47 - 97³</td>
<td>Thin lenses of delluvial-prolluvial clay loams with local intermittent layers of gravel and sandstone</td>
</tr>
<tr>
<td>4b</td>
<td>Left and Right Dam Foundation / River Banks</td>
<td>43 – 127² 47 - 97³</td>
<td>Generally sandstone</td>
</tr>
</tbody>
</table>
The bed of the Vileshchay River consists of alluvial and alluvial-prolluvial sediments (Layers 2, 5, and 6). The dam is founded upon alluvial, alluvial-prolluvial and delluvial-prolluvial sediments and clays (Layers 2, 4, 5, and 6).

8.0 SEEPAGE ANALYSIS

8.1 Seepage Model Development

Seepage analyses were performed using SEEP/W, included in the Geostudio© Version 8.12.3.7901 software suite. SEEP/W is a two-dimensional finite element computer program used to generate groundwater seepage flow regimes and resultant pore pressure distributions based on user-defined geologic and hydrogeological flow parameters.

Two-dimensional seepage models were developed to simulate groundwater flow conditions below and through the dam, and facilitate the prediction of seepage flow volumes for low, normal, and maximum reservoir stage levels. Transient or time-dependent model analyses were used to estimate seepage flux quantities during periods of rapid drawdown of the reservoir water level. Steady-state seepage analyses were used to estimate flow quantities subsequent to the equilibration of pore water pressures for low, normal, and maximum reservoir stage levels.

8.2 Seepage Model Elements

The seepage model elements consist of representative west-east cross sections through the dam at Stations 16+00 and 21+00.

<table>
<thead>
<tr>
<th>Stratum No.</th>
<th>Location(s)</th>
<th>Approximate Elevation Range(s) (masl)</th>
<th>General Soil and Rock Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Left, Central and Right Dam Foundation / Riverbed &amp; River Banks</td>
<td>30 – 432, 30 – 404, 30 - 953</td>
<td>Alluvial-prolluvial gravelly and loamy sediments with intermittent layers of semi-consolidated cemented conglomerate</td>
</tr>
<tr>
<td>5a / 5b / 5q</td>
<td>Left and Right Dam Foundation / River Banks</td>
<td>30 – 432, 30 - 953</td>
<td>Silty clays and silty clay loams with intermittent layers of gravel and sand</td>
</tr>
<tr>
<td>6/ 6a</td>
<td>Uniform Impervious Hard Layer</td>
<td>30-1112, 30-473, -15 - 30</td>
<td>Dense mudstones with intermittent argillaceous sandstones, clays, and sandy clay loams</td>
</tr>
</tbody>
</table>

1 Denotes approximate elevations expressed in meters above sea level.
2 Denotes elevations within left river bank.
3 Denotes elevations within right river bank.
4 Denotes elevations below riverbed.
8.3 Seepage Model Hydraulic Input Parameters

The subsurface strata as delineated and described in Table 1 were incorporated into the cross-sectional model elements as user-defined input parameters. Hydraulic flow parameters for each subsurface stratum were estimated using empirical correlations, and the classifications and results of laboratory testing provided in Section 4.3.2 of the Design Report provided in Appendix A. The estimated hydraulic parameters are provided below in Table 2.

<table>
<thead>
<tr>
<th>Stratum Designation</th>
<th>$E'$, kPa</th>
<th>$m_v$ (1/kPa)</th>
<th>$k_n$ (meters/day)</th>
<th>k-ratio $(k_v/k_n)$</th>
<th>Saturated Volumetric Water Content, $\theta_w$</th>
<th>Specific Yield</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>100,000</td>
<td>1.0E-05</td>
<td>50 - 70</td>
<td>1.0</td>
<td>0.25-0.30</td>
<td>0.20</td>
</tr>
<tr>
<td>4</td>
<td>15,000</td>
<td>7.0E-05</td>
<td>0.1 - 0.2</td>
<td>0.1</td>
<td>0.40</td>
<td>0.05</td>
</tr>
<tr>
<td>5</td>
<td>100,000</td>
<td>1.0E-05</td>
<td>1 - 6</td>
<td>1.0</td>
<td>0.25-0.30</td>
<td>0.20</td>
</tr>
<tr>
<td>6$^1$</td>
<td>200,000</td>
<td>5.0E-06</td>
<td>&lt;0.1</td>
<td>1.0</td>
<td>0.40-0.45</td>
<td>0.10</td>
</tr>
<tr>
<td>Gravel-Sand BORROW FILL$^2$</td>
<td>100,000</td>
<td>1.0E-05</td>
<td>1 - 7</td>
<td>1.0</td>
<td>0.25-0.30</td>
<td>0.20</td>
</tr>
<tr>
<td>Clay BORROW FILL$^2$</td>
<td>15,000</td>
<td>7.0E-05</td>
<td>0.2</td>
<td>0.1</td>
<td>0.40</td>
<td>0.05</td>
</tr>
<tr>
<td>Gravel-Sand FILL$^3$</td>
<td>125,000</td>
<td>8.0E-06</td>
<td>1 - 7</td>
<td>1.0</td>
<td>0.25-0.30</td>
<td>0.20</td>
</tr>
<tr>
<td>Clay FILL$^3$</td>
<td>35,000</td>
<td>2.9E-05</td>
<td>0.001-0.002</td>
<td>0.05</td>
<td>0.40</td>
<td>0.03</td>
</tr>
<tr>
<td>Filter Material FILL$^3$</td>
<td>100,000</td>
<td>1.0E-05</td>
<td>1.0</td>
<td>1.0</td>
<td>0.25-0.30</td>
<td>0.15</td>
</tr>
<tr>
<td>Stone FILL</td>
<td>150,000</td>
<td>6.7E-06</td>
<td>200</td>
<td>1.0</td>
<td>0.25-0.30</td>
<td>0.15</td>
</tr>
<tr>
<td>SEEPAGE BARRIER WALL$^4$</td>
<td>N/A</td>
<td>N/A</td>
<td>$2.83x10^{-8}$ - $5.0x10^{-3}$</td>
<td>1.0</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Notes:

$^1$ The surface of Layer 6 (monolithic mudstone with intermittent layers of sandstone) as provided in Table 1 deemed an impervious boundary in the seepage model since its horizontal conductivity is substantially lower than the conductivities of the overlying strata.

$^2$ Denotes hydraulic flow parameters of borrow fill materials in their natural state prior to compaction. Mixtures of gravel and sand borrowed from the Khalfalar & Arkivan Village field quarries. Clay borrowed from Quarry No. 1.

$^3$ Denotes hydraulic flow parameters of borrow fill materials in their compacted state following placement within the dam body. Filter material for dam transitions borrowed from the Shikhlar Village field quarry.

$^4$ Unsaturated hydraulic input parameters were not specified for the “prong” barrier wall since the wall is assumed to be impervious.
8.4 Seepage Model Analyses

Seepage model analyses were performed to quantify potential groundwater seepage quantities, and horizontal and vertical exit gradients for minimum, normal, and maximum reservoir stage level elevations of 62.2, 75.5, and 77.0 meters above sea level.

8.5 Critical Horizontal and Vertical Seepage Gradients

Horizontal exit gradients were calculated using the seepage model analyses performed using SEEP/W. Representative critical horizontal exit gradients were computed using the following Equation 12:

\[ i_{ch} = i_{cv}(\cos b \cdot \tan q - \sin b) \]  

where: \( i_{ch} \) is the critical horizontal exit gradient, \( i_{cv} \) is the critical vertical exit gradient, \( b \) is the angle of seepage flow at the exit face, and \( q \) is the internal friction angle of the soil flow medium. Critical horizontal gradients between 0.01 and 0.40 were calculated using Equation 12 assuming that the angle of seepage flow at the exit face is between 0 and 15 degrees.

The critical vertical gradient was computed using the following Equation 13:

\[ i_{cv} = (\gamma_{sat} \cdot \gamma_w) / \gamma_w \]  

where: \( \gamma_{sat} \) is the saturated unit weight of the soil flow medium, and \( \gamma_w \) is the unit weight of water. Critical vertical gradients between 0.01 and 0.38 were computed using this equation.

The factor of safety against horizontal piping is expressed as the ratio of the critical horizontal seepage gradient to the maximum horizontal exit gradient. According to Harr (1962) and Cedergren (1977), factors of safety for exit gradients within the ranges of 4 to 5 and 2.5 to 3 have, respectively, been proposed; therefore, the maximum allowable horizontal gradient should be on the order of 0.08 to 0.30, with an average horizontal gradient of 0.20.

The factor of safety against uplift is expressed as the ratio of the critical vertical seepage gradient to the maximum vertical exit gradient. According to Harr (1962) and Cedergren (1977), factors of safety for exit gradients within the ranges of 4 to 5 and 2.5 to 3 have, respectively, been proposed; therefore, the maximum allowable vertical gradient should be on the order of 0.16 to 0.40, with an average vertical gradient of 0.30.

8.6 Seepage Model Results

The results of the seepage model sensitivity analyses for minimum, normal, and maximum reservoir stage elevations are summarized below in Table 3. The seepage analysis results for the model scenarios performed using the design conditions (model analysis designations R1 through R3 and R16 through R18) are provided in Appendix F.
## TABLE 3: Results of Seepage Model Sensitivity Analysis

<table>
<thead>
<tr>
<th>Model Analysis Designation</th>
<th>Station</th>
<th>Reservoir Water Level (masl)</th>
<th>Relatively Impervious Central Core Included (Y/N)</th>
<th>Seepage Barrier “Prong” Wall Included (Y/N)</th>
<th>Predicted Seepage Quantity Below Dam Foundation (m³/sec/m)</th>
<th>Predicted Seepage Quantity into Seepage Collector Ditch (m³/sec/m)</th>
<th>Predicted Seepage Quantity Below Seepage Barrier Wall (m³/sec/m)</th>
<th>Predicted Total Seepage Quantity (m³/sec/m)</th>
<th>Predicted Maximum Horizontal Exit Gradient, $i_h$</th>
<th>Predicted Maximum Vertical Exit Gradient, $i_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>16+00</td>
<td>77.0</td>
<td>Y</td>
<td>Y</td>
<td>2.60 E-05</td>
<td>2.00 E-05</td>
<td>4.14 E-06</td>
<td>2.70 E-05</td>
<td>0.035</td>
<td>0.013</td>
</tr>
<tr>
<td>R2</td>
<td>16+00</td>
<td>75.5</td>
<td>Y</td>
<td>Y</td>
<td>2.55 E-05</td>
<td>1.80 E-05</td>
<td>3.95 E-06</td>
<td>2.55 E-05</td>
<td>0.026</td>
<td>0.010</td>
</tr>
<tr>
<td>R3</td>
<td>16+00</td>
<td>62.2</td>
<td>Y</td>
<td>Y</td>
<td>1.43 E-05</td>
<td>1.25 E-05</td>
<td>2.25 E-06</td>
<td>1.43 E-05</td>
<td>0.013</td>
<td>0.005</td>
</tr>
<tr>
<td>R4</td>
<td>16+00</td>
<td>77.0</td>
<td>Y</td>
<td>N</td>
<td>3.27 E-04</td>
<td>3.08 E-04</td>
<td>N/A</td>
<td>3.28 E-04</td>
<td>0.250</td>
<td>0.180</td>
</tr>
<tr>
<td>R5</td>
<td>16+00</td>
<td>75.5</td>
<td>Y</td>
<td>N</td>
<td>3.00 E-04</td>
<td>2.90 E-04</td>
<td>N/A</td>
<td>3.11 E-04</td>
<td>0.220</td>
<td>0.140</td>
</tr>
<tr>
<td>R6</td>
<td>16+00</td>
<td>62.2</td>
<td>Y</td>
<td>N</td>
<td>1.70 E-04</td>
<td>1.61 E-04</td>
<td>N/A</td>
<td>1.71 E-04</td>
<td>0.120</td>
<td>0.090</td>
</tr>
<tr>
<td>R7</td>
<td>16+00</td>
<td>77.0</td>
<td>N</td>
<td>Y</td>
<td>3.77 E-04</td>
<td>4.43 E-04</td>
<td>2.04 E-06</td>
<td>4.67 E-04</td>
<td>0.320</td>
<td>0.150</td>
</tr>
<tr>
<td>R8</td>
<td>16+00</td>
<td>75.5</td>
<td>N</td>
<td>Y</td>
<td>3.47 E-04</td>
<td>3.89 E-04</td>
<td>2.02 E-06</td>
<td>4.11 E-04</td>
<td>0.280</td>
<td>0.100</td>
</tr>
<tr>
<td>R9</td>
<td>16+00</td>
<td>62.2</td>
<td>N</td>
<td>Y</td>
<td>7.95 E-05</td>
<td>9.35 E-05</td>
<td>1.59 E-06</td>
<td>9.99 E-05</td>
<td>0.100</td>
<td>0.050</td>
</tr>
<tr>
<td>R10</td>
<td>16+00</td>
<td>77.0</td>
<td>N</td>
<td>N</td>
<td>5.87 E-04</td>
<td>5.58 E-04</td>
<td>N/A</td>
<td>5.87 E-04</td>
<td>0.400</td>
<td>0.230</td>
</tr>
<tr>
<td>R11</td>
<td>16+00</td>
<td>75.5</td>
<td>N</td>
<td>N</td>
<td>5.31 E-04</td>
<td>5.04 E-04</td>
<td>N/A</td>
<td>5.31 E-04</td>
<td>0.340</td>
<td>0.200</td>
</tr>
<tr>
<td>R12</td>
<td>16+00</td>
<td>62.2</td>
<td>N</td>
<td>N</td>
<td>2.02 E-04</td>
<td>1.89 E-04</td>
<td>N/A</td>
<td>2.02 E-04</td>
<td>0.130</td>
<td>0.100</td>
</tr>
<tr>
<td>R13</td>
<td>16+00</td>
<td>77.0</td>
<td>Y*</td>
<td>Y</td>
<td>8.18 E-05</td>
<td>7.65 E-05</td>
<td>3.81 E-06</td>
<td>8.18 E-05</td>
<td>0.060</td>
<td>0.025</td>
</tr>
<tr>
<td>R14</td>
<td>16+00</td>
<td>75.5</td>
<td>Y*</td>
<td>Y</td>
<td>8.05 E-05</td>
<td>7.52 E-05</td>
<td>3.61 E-06</td>
<td>8.05 E-05</td>
<td>0.055</td>
<td>0.020</td>
</tr>
<tr>
<td>R15</td>
<td>16+00</td>
<td>62.2</td>
<td>Y*</td>
<td>Y</td>
<td>3.27 E-05</td>
<td>3.01 E-05</td>
<td>2.12 E-06</td>
<td>3.27 E-05</td>
<td>0.020</td>
<td>0.010</td>
</tr>
<tr>
<td>R16</td>
<td>21+00</td>
<td>77.0</td>
<td>Y</td>
<td>Y</td>
<td>2.40 E-05</td>
<td>2.06 E-05</td>
<td>3.88 E-06</td>
<td>2.52 E-05</td>
<td>0.140</td>
<td>0.100</td>
</tr>
<tr>
<td>R17</td>
<td>21+00</td>
<td>75.5</td>
<td>Y</td>
<td>Y</td>
<td>2.33 E-05</td>
<td>1.92 E-05</td>
<td>3.69 E-06</td>
<td>2.40 E-05</td>
<td>0.135</td>
<td>0.090</td>
</tr>
<tr>
<td>R18</td>
<td>21+00</td>
<td>62.2</td>
<td>Y</td>
<td>Y</td>
<td>1.31 E-05</td>
<td>1.00 E-05</td>
<td>2.05 E-06</td>
<td>1.31 E-05</td>
<td>0.080</td>
<td>0.068</td>
</tr>
<tr>
<td>R19</td>
<td>21+00</td>
<td>77.0</td>
<td>Y</td>
<td>N</td>
<td>3.04 E-04</td>
<td>2.46 E-04</td>
<td>N/A</td>
<td>3.25 E-04</td>
<td>0.300</td>
<td>0.320</td>
</tr>
<tr>
<td>R20</td>
<td>21+00</td>
<td>75.5</td>
<td>Y</td>
<td>N</td>
<td>2.85 E-04</td>
<td>2.33 E-04</td>
<td>N/A</td>
<td>3.08 E-04</td>
<td>0.260</td>
<td>0.300</td>
</tr>
<tr>
<td>R21</td>
<td>21+00</td>
<td>62.2</td>
<td>Y</td>
<td>N</td>
<td>1.52 E-04</td>
<td>1.40 E-04</td>
<td>N/A</td>
<td>1.59 E-04</td>
<td>0.220</td>
<td>0.240</td>
</tr>
<tr>
<td>R22</td>
<td>21+00</td>
<td>77.0</td>
<td>N</td>
<td>Y</td>
<td>4.00 E-04</td>
<td>3.67 E-04</td>
<td>2.05 E-06</td>
<td>4.75 E-04</td>
<td>0.310</td>
<td>0.360</td>
</tr>
<tr>
<td>R23</td>
<td>21+00</td>
<td>75.5</td>
<td>N</td>
<td>Y</td>
<td>3.73 E-04</td>
<td>3.26 E-04</td>
<td>2.01 E-06</td>
<td>4.19 E-04</td>
<td>0.300</td>
<td>0.340</td>
</tr>
<tr>
<td>R24</td>
<td>21+00</td>
<td>62.2</td>
<td>N</td>
<td>Y</td>
<td>8.75 E-05</td>
<td>8.27 E-05</td>
<td>1.42 E-06</td>
<td>9.30 E-05</td>
<td>0.180</td>
<td>0.190</td>
</tr>
</tbody>
</table>
The results of the seepage sensitivity analysis reveal the following:

- The results of the seepage analyses including the design conditions indicate that approximately 15 to 16 percent of the total seepage can be attributed to seepage below the concrete seepage barrier “prong” wall.

- The results of the sensitivity analyses indicate that both the seepage barrier wall and central dam core are required to reduce the risk of piping at the toe of the dam and uplift downstream of the dam. The maximum allowable horizontal gradient should be on the order of 0.08 to 0.30, with an average horizontal gradient of 0.20 to reduce the risk for piping at the toe of the dam; the maximum allowable vertical gradient should be on the order of 0.16 to 0.40, with an average vertical gradient of 0.30 to preclude the risk for uplift downstream of the dam.

- The results of the seepage analyses indicate that the seepage barrier “prong” wall tip elevation reduces predicted horizontal and vertical exit gradients at the toe and downstream of the dam to acceptable values.

### SLOPE STABILITY ANALYSIS

#### 9.1 Slope Stability Model Development

The slope stability analyses were performed using SLOPE/W, part of the Geostudio 2012 Version 8.12.3.7901 software suite. SLOPE/W is a finite element model that employs limit equilibrium methods to compute both total shear resistance and mobilized shear stress along user-defined slip surfaces.

The soil mass enclosed within a slip surface is discretized into constituent vertical slices, and SLOPE/W computes a local stability factor for each slice. Ultimately, the critical stability factor is computed by SLOPE/W using search optimization techniques. Spencer’s method of slices was employed to compute the critical stability factors for the subject assessment because this
method uses both moment and force limit equilibrium fundamentals to calculate the ratio of available shear resistance to that required for equilibrium.

SLOPE/W integrates pore pressures computed from SEEP/W, granting the user the capability to analyze and evaluate discrete changes in slope stability as a result of variations in transient pore pressure conditions. The phreatic surfaces and pore pressure distributions were generated using SEEP/W for the purposes of the subject stability analyses.

9.2 Slope Stability Model Soil and Rock Shear Strength Parameters

Spencer’s method of slices assumes that the shear strengths of the subsurface materials located along the potential failure surface are governed by linear (Mohr-Coulomb) or non-linear relationships between shear strength and normal stress. Effective linear Mohr-Coulomb soil and rock shear strength relationships were assigned to each of the subsurface strata and are provided below in Table 4.

The shear strength parameters provided in Table 4 are based on the effective shear strength data summarized in Section 4.3.2 of the Design Report provided in Appendix A.

**TABLE 4: Estimated Effective Soil and Rock Effective Shear Strength Parameters**

<table>
<thead>
<tr>
<th>Stratum Designation</th>
<th>General Stratum Description</th>
<th>Dry Unit Weight, $\gamma_d$ (g/cm$^3$)</th>
<th>Saturated Unit Weight, $\gamma_{sat}$ (g/cm$^3$)</th>
<th>Effective Friction Angle, $\phi'$ (degrees)</th>
<th>Effective Cohesion Intercept (kg/cm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Alluvial sediments consisting of sandy loams, clayey loams, gravel, and sandstone</td>
<td>1.9</td>
<td>2.1</td>
<td>33</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>Delluvial-prolluvial clays with local intermittent layers of sand, gravel, and sandstone</td>
<td>1.4</td>
<td>1.8</td>
<td>13-18</td>
<td>0.8</td>
</tr>
<tr>
<td>5</td>
<td>Alluvial-prolluvial gravelly and loamy sediments with intermittent layers of semi-consolidated cemented conglomerate</td>
<td>1.9</td>
<td>2.1</td>
<td>32</td>
<td>0</td>
</tr>
<tr>
<td>6$^1$</td>
<td>Dense mudstones with intermittent argillaceous sandstones, clays, and sandy clay loams</td>
<td>2.1</td>
<td>2.3</td>
<td>36</td>
<td>0</td>
</tr>
<tr>
<td>Gravel-Sand FILL$^2$</td>
<td>Alluvial-prolluvial gravel-sand sediments</td>
<td>1.9</td>
<td>2.1</td>
<td>33</td>
<td>0</td>
</tr>
<tr>
<td>Clay FILL$^2$</td>
<td>Delluvial-prolluvial clays</td>
<td>1.4</td>
<td>1.8</td>
<td>10-18</td>
<td>0.8</td>
</tr>
<tr>
<td>Filter Material FILL$^3$</td>
<td>Alluvial-prolluvial gravel-sand sediments</td>
<td>1.9</td>
<td>2.1</td>
<td>35</td>
<td>0</td>
</tr>
</tbody>
</table>
9.3 Cross Sections for Slope Stability Analysis

Representative west-east cross sections through the dam were generated at Stations 16+00 and 21+00. Transient and steady-state pore pressures as computed using SEEP/W were integrated and used by SLOPE/W to calculate slope stability factors of safety during periods of transient water conditions, i.e., during rapid drawdown, and at equilibrium.

9.4 Seismicity

The reservoir is located in a region characterized by frequent seismic activity, and according to the regulatory document SNIP II-7-81, earthquakes with a magnitude of 7 can potentially occur at the site. According to Table 10.21 included in Appendix A, the estimated maximum potential seismic magnitude is 8.

Seismic activity with a magnitude of 7 on the Richter Scale corresponds to a Mercalli seismic intensity of VII/VIII, and seismic activity with a magnitude of 8 on the Richter Scale corresponds to a Mercalli Scale intensity of VIII or higher. A summary of representative Mercalli intensities and corresponding relevant seismic data are provided below in Table 5.

### TABLE 5: Representative Regional Seismic Data

<table>
<thead>
<tr>
<th>Richter Scale Seismic Magnitude</th>
<th>Mercalli Scale Seismic Designation</th>
<th>Perceived Shaking</th>
<th>Potential Structural Damage</th>
<th>Peak Ground Acceleration, g</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.0</td>
<td>VII VIII</td>
<td>Very Strong Severe</td>
<td>Moderate/Moderate/Heavy</td>
<td>0.18 to 0.34</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.34 to 0.65</td>
</tr>
<tr>
<td>8.0</td>
<td>VIII IX</td>
<td>Severe Violent</td>
<td>Moderate/Heavy</td>
<td>0.34 to 0.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.65 to 1.24</td>
</tr>
</tbody>
</table>
Based on the data provided in Table 5 and an examination of seismic hazard maps provided in United States Geological Survey (USGS) reports for seismicity of the Earth in the Middle East and vicinity, representative values for peak ground acceleration of 3.2 to 6.4 m/sec$^2$, or 0.32g to 0.65g are applicable for the region. Therefore, representative values for effective peak acceleration, or the maximum ground acceleration to which a structure responds, are between 2.3 and 4.5 m/sec$^2$, or 0.23g to 0.46g, with an average of about 0.35g. Note that the effective ground acceleration is typically within the range of 2/3 to 3/4 of the peak ground acceleration. Also, note that the aforementioned values of maximum and effective ground acceleration are represented as resultant values, and vertical and horizontal component values of the effective ground acceleration were used to calculate the global seismic slope factors of safety provided in Table 6.

9.5 Results of Slope Stability Analysis

Slope stability analyses were performed to assess the stability of the upstream and downstream dam embankments. The upstream embankments are inclined at an angle of 4.5H:1V from Elevation 55 to Elevation 70 meters above sea level, and at 4H:1V from Elevation 70 to Elevation 80 meters above sea level. The surface of the upstream embankments is protected against erosion with a stone revetment system consisting of stones with a thickness of 2.5 meters. The downstream dam embankments are inclined at angles of 2H:1V to 3H:1V, and 5-meter wide stabilization berms were constructed along the downstream embankment slopes at vertical increments of 15 meters.

The results of the sensitivity analysis for the various case model scenarios are summarized below in Table 6. The slope stability analysis results for the model scenarios performed using the design conditions are provided in Appendix G.

### TABLE 6: Results of Slope Stability Analysis

<table>
<thead>
<tr>
<th>Model Analysis Section Designation</th>
<th>Station</th>
<th>Reservoir Water Level (masl)</th>
<th>Relatively Impervious Central Core Included (Y/N)</th>
<th>Seepage Barrier &quot;Prong&quot; Wall Included (Y/N)</th>
<th>Global Static Slope Factor of Safety, F.S.</th>
<th>Local Static Slope Factor of Safety, F.S.</th>
<th>Global Seismic Slope Factor of Safety, F.S.</th>
<th>Global Drawdown Factor of Safety, F.S.</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1 16+00</td>
<td>77.0</td>
<td>Y</td>
<td>Y</td>
<td>1.83</td>
<td>1.72</td>
<td>1.03</td>
<td>1.99</td>
<td></td>
</tr>
<tr>
<td>R2 16+00</td>
<td>75.5</td>
<td>Y</td>
<td>Y</td>
<td>1.87</td>
<td>1.73</td>
<td>1.04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>R3 16+00</td>
<td>62.2</td>
<td>Y</td>
<td>Y</td>
<td>2.03</td>
<td>1.74</td>
<td>1.13</td>
<td></td>
<td></td>
</tr>
<tr>
<td>R4 16+00</td>
<td>77.0</td>
<td>Y</td>
<td>N</td>
<td>1.67</td>
<td>1.62</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>R7 16+00</td>
<td>77.0</td>
<td>N</td>
<td>Y</td>
<td>1.67</td>
<td>1.56</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>R10 16+00</td>
<td>77.0</td>
<td>N</td>
<td>N</td>
<td>1.60</td>
<td>1.55</td>
<td>0.98</td>
<td></td>
<td></td>
</tr>
<tr>
<td>R16 21+00</td>
<td>77.0</td>
<td>Y</td>
<td>Y</td>
<td>2.52</td>
<td>2.41</td>
<td>1.25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>R17 21+00</td>
<td>75.5</td>
<td>Y</td>
<td>Y</td>
<td>2.54</td>
<td>2.42</td>
<td>1.26</td>
<td></td>
<td></td>
</tr>
<tr>
<td>R18 21+00</td>
<td>62.2</td>
<td>Y</td>
<td>Y</td>
<td>2.57</td>
<td>2.43</td>
<td>1.30</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The results summarized above in Table 6 indicate that the 5-meter wide downstream slope stabilization berms spaced at vertical increments of 15 meters are required to maintain stability during large seismic events. The calculated factors of safety are all greater than the minimum values required for slope stability under both static and seismic conditions. Furthermore, the analysis results demonstrate that upstream embankment slopes inclined at 4H:1V to 4.5H:1V are adequate to maintain stability during a rapid drawdown of the reservoir.

10.0 EVALUATION OF DAM FREEBOARD

The wave run-up level was calculated to corroborate the design crest elevation of the dam for the case where no overtopping is allowed. The design crest elevation was determined in accordance with the SNIP 2.06.04-82 regulatory document, and the freeboard was calculated using the wind speed and reservoir water level for the 100-year and 10-year storm events. Design wind speeds of 24 m/s and 10 m/s were adopted for the 100-year and 10-year events with durations of 11.4 and 6 hours, respectively. The following list of viable wind speeds and durations were used for our assessment. These wind speeds are used in the design of wind turbines, wind-generated power plants, and tall structures located in Middle Eastern countries:

- Average annual sustained wind speed: 7 m/sec
- 100-year, 3-second wind gust: 40 m/s
- 475-year, 3-second wind gust: 50 m/s
- 1,000-year, 3-second wind gust: 60 m/s
- 100-year, 10-minute sustained wind speed: 28 m/s
- 100-year, 1-hour sustained wind speed: 26.5 m/s
- 475-year, 10-minute sustained wind speed: 35 m/s
- 475-year, 1-hour sustained wind speed: 34 m/s
- 1,000-year, 10-minute sustained wind speed: 42 m/s
- 1,000-year, 1-hour sustained wind speed: 40 m/s

10.1 Structural Hydraulic Response
The magnitude of the wave run-up (Ru) depends on the height and steepness of the incident wind-generated wave and its interaction with the preceding reflected wave, slope angle of the structural embankment, surface roughness, and the permeability and porosity of the slope material. An increase in the permeability and porosity of the slope material reduces flow velocities along the surface of the embankment, and consequently, the wind setup is reduced.

Wave run-up depends on the type of wave breaking; the type of wave breaking is characterized by the magnitude of the surf-similarity parameter. This parameter is alternatively referred to as the breaker parameter or Iribarren number. The surf-similarity parameter is defined in Equation 14. The wave steepness is defined in Equation 15.

\[ \zeta_{op} = \tan \omega / (s_{op}^{1/2}) \]  
\[ s_{op} = H_s / L_{op} = 2\pi / g \cdot (H_s / T_p^2) \]

where: \( \omega \) is the embankment slope angle, \( s_{op} \) is the wave steepness corresponding to the peak of the wave spectrum, \( H_s \) is the significant wave height of incident waves at the toe of the structure, \( L_{op} \) is the deepwater wavelength corresponding to the peak of the wave spectrum, \( g \) is the gravitational acceleration, and \( T_p \) is the wave period corresponding to the peak of the wave spectrum.

The deep-water significant wave height and wave period are computed using the following Equations 16 and 17.

\[ H_s = 0.000282 \cdot U_A \cdot F_e^{0.5} \]  
\[ T_p = 0.0283 \cdot (U_A \cdot F_e)^{0.33} \]

where: \( U_A \) is the wind stress factor in feet per second and \( F_e \) is the effective fetch length in feet.

The significant wave height and wave period can be estimated using the following Equations 18 and 19.

\[ H_s = 0.00354 \cdot (U_{10}^2 / g)^{0.58} \cdot F_e^{0.42} \]  
\[ T_p = 0.581 \cdot (F_e \cdot U_{10}^2 / g^3)^{1/4} \]

where: \( U_{10} \) is the 10-minute sustained wind speed.

Wave run-up on impermeable and permeable embankment slopes is defined in Equations 20 and 21, respectively.

\[ R_{ui} / H_s = (A\zeta_{op} + C) \cdot \gamma_r \cdot \gamma_b \cdot \gamma_h \cdot \gamma_b \]  
\[ R_{ui} / H_s = A\zeta_{op} \]

where: \( \gamma_r \) is the reduction factor for surface roughness (\( \gamma_r = 1 \) for smooth slopes, \( \gamma_r = 0.9 \) for grassed slopes, \( \gamma_r = 0.5 \) to 0.6 for slopes protected with riprap stone revetment), \( \gamma_b \) is the berm reduction factor (\( \gamma_b = 1 \) for non-bermed profiles), \( \gamma_h \) is the reduction factor for influence of shallow-water conditions where the wave height distribution deviates from the Rayleigh
distribution \(\gamma_{h} = \frac{H}{2H_{2}}\), and \(\gamma_{f}\) is the influence factor for angle of incidence of the waves \(\gamma_{f} = \cos(\beta-10)\) for \(10^{\circ} < \beta < 63^{\circ}\). The coefficient \(A\) for impermeable slopes is equivalent to 1.5 (de Waal and van der Meer (1992)) and 1.6 (Ahrens (1981) and Battjes (1974). The coefficient \(C\) is equivalent to 0 for values of Iribarren number less than 2 to 2.5. The coefficient for permeable slopes is equivalent to 0.96 and 1.12 for exceedance levels of 2 and 0.1 percent, respectively.

### 10.2 Results of Freeboard Analysis

The results of the freeboard analysis are summarized in the following Table 7. Dam freeboard is defined as the distance measured vertically from the maximum reservoir water level to the crest. Dam freeboard values were calculated as the summation of the wave run-up level, wind set-up, and maximum precipitation using wave run-up levels exceeded by 2 percent of the incident waves.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>2,280</td>
<td>827</td>
<td>0.87</td>
<td>2.19</td>
<td>0.76</td>
<td>1.04</td>
<td>0.73</td>
</tr>
<tr>
<td>475</td>
<td>2,280</td>
<td>827</td>
<td>1.04</td>
<td>2.32</td>
<td>0.98</td>
<td>1.20</td>
<td>0.84</td>
</tr>
<tr>
<td>1,000</td>
<td>2,280</td>
<td>827</td>
<td>1.27</td>
<td>2.47</td>
<td>1.21</td>
<td>1.42</td>
<td>1.00</td>
</tr>
</tbody>
</table>

The required dam freeboard based on the assumption that the maximum precipitation at the site is 0.5 meter is as follows:

- For a recurrence interval of 100 years, the required freeboard is 1.5 meters.
- For a recurrence interval of 475 years, the required freeboard is 1.7 meters.
- For a recurrence interval of 1,000 years, the required freeboard is 1.9 meters.

### 11.0 UPSTREAM AND DOWNSTREAM SLOPE PROTECTION

The upstream and downstream stone slope protection was designed to withstand hydraulic loads imposed by wave action expected during large storm events. Calculations used to evaluate the applicability of the slope protection systems are included in Appendix H.

The upstream slope will be subjected to shear stresses and forces due to wind-generated waves predominantly induced by flash flooding and flow surges. Waves are essentially surface features that interact with the banks of dams and hydraulic structures, and the stability of such banks is often compromised by wave action.
The determination of significant wave height ($H_s$) in conjunction with wave period ($T_p$) is a technique used to evaluate wave interaction behavior at a given site location. The significant wave height represents the average wave height of the upper one-third of waves over a given period of time.

The design of revetments subjected to wind-generated wave action is based on a variation of Laplace’s second-order partial differential equation for laminar flow in two dimensions where the potential pressure head induced by structural wave interaction is quantified. Complex derivations of Laplace’s equation can ultimately be used to calculate the uplift pressure exerted on the revetment. The magnitude and direction of the uplift pressure is dependent on the steepness and height of the incoming pressure front, or breaking wave, on the revetment.

Moreover, the wave height and steepness are dependent on factors including significant wave height, wave period, and the angle of slope inclination. The uplift is also dependent on the thickness and permeability of the revetment. However, it should be noted that the uplift pressure is not dependent on permeability if the permeability of the revetment is significantly higher than the permeability of the underlying soil.

The following Equation 22 (Pilarczyk et al 1998) was used in conjunction with Equation 23 to evaluate the design thickness of several potential revetment alternatives.

$$\frac{H_s}{\Delta}{D_{cr}} = \frac{(F\cos\alpha)}{\xi_{op}}^b$$  \hspace{1cm} (22)

$$\xi_{op} = \tan \alpha \div \frac{H_s}{(1.56T_p^2)^{1/2}}$$  \hspace{1cm} (23)

where: $F$ is the revetment stability factor, $H_s$ is the significant wave height (m), $\Delta$ is the relative density of the revetment, $D$ is the revetment thickness (m), $\xi_{op}$ is the surf similarity parameter, or Iribarren number, $b$ is the wave-slope interaction factor as influenced by the roughness and porosity of the revetment, $\alpha$ is the angle of slope inclination, and $T_p$ is the peak wave period (sec).

The approximate values for the revetment stability factor, $F$, are as follows: $F=2.25$ for riprap stone revetment; $F=2.5$ for pitched irregular stone revetment; $F=4.0$ for geomattresses; $F=3.5$ to 5.5 for block revetments; $F=4.0$ to 6.0 for cabled block revetments; $F=6.0$ to 8.0 for gabions; and $F=6.0$ to 10.0 for asphalt or concrete slabs. Typical values for $b$, or wave-slope interaction, are recommended as follows: $b=0.5$ for permeable revetments, e.g., riprap, gabions, and very open block mats, $b=2/3$ for semi-permeable revetments, e.g., pitched stone, placed blocks, block mats, and concrete/sand-filled mattresses, and $b=1.0$ for concrete slabs.

Using Equations 15 and 16, it was determined that the upstream dam embankments should be protected with riprap stone revetment with the following nominal diameter requirements shown in Table 8. The maximum required average diameter of the riprap stone revetment is 0.78 meter, which is substantially less than the 2.5-meter average diameter of the stone revetment specified in the design report.

| TABLE 8: Riprap Stone Revetment Nominal Diameter Requirements for Upstream Dam Embankment Slope Protection |
12.0 DESIGN OF GRANULAR FILTER TRANSITIONS

The dam is 1,200 meters long and comprised of predominantly of gravelly sand, gravel, and stone fill materials. There is a sloping embedded clay core, which is stabilized by granular filter transitional zones on either side.

The core is stabilized with upstream and downstream zones of graded gravel to prevent the migration of clay core material into the upstream sandy gravel and stone fill materials during a rapid drawdown of the reservoir water level. The graded gravel filter zones also prevent the core material from being forced into the downstream sandy gravel and stone fill materials via seepage under full reservoir head conditions.

The upstream and downstream filter zones essentially induce a process of “self-healing” should a transverse crack appear in the core; the upstream and downstream filter zones retain core material and prevent the extension of a transverse crack into the downstream sandy gravel and stone fill zones.

Several criteria developed by J.P. Giroud (1981, 1984) were used to substantiate the design of the graded granular filter transitions. Criteria required to evaluate the suitability of a graded granular filter include retention, permeability, porosity, and thickness. The retention criterion accounts for the grain-size distribution of both the retained soil located adjacent to the filter and graded filter material. The permeability criterion includes pore water pressure gradient and flow rate requirements. The porosity criterion ensures that there are a sufficient number of flow channels per unit area.

12.1 Retention and Segregation Criteria

Retention and segregation are the most critical and complex aspects of filtration. Equations 24 and 25 are used to evaluate the retention and segregation capabilities of the graded filter material, respectively.

\[
d_{15F} / d_{85S} \leq 4 \text{ to } 5
\] (24)
\[
\frac{d'_{60F}}{d'_{10F}} \leq 20
\]  

(25)

12.2 Permeability Criterion

The presence of a graded gravel filter causes an insignificant increase in pore water pressure if the following condition is satisfied:

\[
k_F \geq \text{max}(i_s k_s \text{ or } 25 k_s)
\]  

(26)

where: \(i_s\) represents the hydraulic gradient measured in the soil next to the filter. The typical values of the hydraulic gradient are between 3 and greater than 10 in soil located adjacent to filters in the case of a drain behind a dam clay core. The measured gradient is assumed to be on the order of 10 to 25 to preclude excess pressure development due to potential clogging.

12.3 Porosity Criterion

The porosity criterion ensures that there are a sufficient number of flow channels, or filter openings per unit area. Conservatively, the porosity of a graded granular filter should be equivalent to a value between 0.20 and 0.30.

Using the aforementioned criteria, it is confirmed that the gravel and sand fill materials borrowed from the Shikhlar Village reserve field are suitable for use as filter materials in dam transition zones to retain the clay core materials borrowed from Quarry No. 1.

13.0 SEEPAGE CUT-OFF WALL

A concrete seepage barrier wall comprised of 6 to 7 meter long horizontal sections was constructed at the dam foundation from Station 12+50 to Station 24+50. The wall extends vertically from Elevation 47 meters above sea level to a wall penetration tip elevation of 28 meters above sea level. The bottom 2 meters of wall “key” into the relatively impervious mudstone and sandstone layers of Layer 6. The hydraulic conductivity of the wall material, i.e., concrete, varies between 0.002 and 0.02 meters per day, with an average conductivity of 0.0083 meters per day.

A relatively shallow seepage barrier wall was investigated prior during the design phase and prior to the construction of 1st Phase. The relatively shallow wall “cuts off” the Layer 2 sediments and extends vertically from Elevation 47 meters above sea level to a wall penetration depth of 1.5 to 2 meters into the Layer 5 sediments.

The design report includes the results of the feasibility analysis used to quantify the leakage for the different wall options. The results indicate that the wall that penetrates both the Layer 2 and 5 sediments effectively reduces the leakage by an average of 25 to 27 percent when compared to the alternate option. The results of our analysis corroborate this percent reduction in seepage quantities and confirm the need for a seepage barrier wall that completely penetrates both the Layer 2 and 5 sediments.

14.0 WATER DISCHARGE STRUCTURES
The design report (see Appendix A) states that the water discharge structures were designed to accommodate the 1000-year flood. The flood flow for a recurrence interval of 1,000 years was stated in the design report to be 842 m$^3$/s.

The design distribution for discharging the total flood flow was stated as follows:

- Bottom tunnels of the water discharge facility: 420 m$^3$/s.
- Discharge spillway located along the left bank: 290 m$^3$/s.
- Reservoir storage: 132 m$^3$/s.

We calculated the total flood flow using hydrological data provided in the design report and data provided to us by the Client. The design report contained monthly reservoir flow data for the years 1938 through 1985. We were also provided daily flow data for the years 2000 through 2014. In order to convert the monthly flow data into daily records we used a probabilistic analysis method recommended by the United States Geological Service. The results of the hydrological analysis are provided in Appendix I.

Figure 3 shows a graph of the exceedance probability vs. river flow. The river flood flow for a recurrence interval of 1,000 years (0.01% probability) is 311 m$^3$/s. This flood flow value is significantly less than the value of 842 m$^3$/s in the design report. Because we were not provided calculations from the design report, we cannot reconcile the different values. However, it should be noted that our analysis is based on daily records over a significantly longer period of time (including 15 years of operation) than the design report.

The discharge facilities are located in a region characterized by complex geological and topographical conditions, and densely populated with residential settlements located at a distance of 0.7 to 1.0 kilometers downstream of the dam. The Gariblar Village is situated along the right river bank at a distance of 0.7 to 0.8 kilometers downstream of the dam. The structures associated with this village are located at Elevation 75.0 masl along the left bank and at Elevation 59.0 to 60.0 masl along the right bank. The Khalfalar Village is located at a distance of 1.2 kilometers downstream of the dam, and the horticultural structures associated with this village are located at Elevation 38.0 to 39.0 masl. The dwellings and major structures associated with the Khalfalar and subsequent Yeyenkend and Allahyarli Villages are located between Elevations 44.0 and 50.0 masl. The relative positions of the water discharge facilities were selected to preclude the risk of flooding and consequential catastrophic washout of these residential settlements established along the river banks downstream of the outlet works.

14.1 Assessment of the Hydraulic Capacity of Bottom Tunnels and Derivation Pipes

The water discharge facility consists of two Bottom Tunnels and two Derivation (irrigation) Pipes. The Derivation Pipes are used to convey irrigation water from the reservoir to agricultural areas located along the left and right river banks approximately 2 to 3 kilometers downstream of the dam, at elevations that vary between 48.0 and 80.0 masl. The discharge capacity of the derivation pipes, which are each 1.22 m in diameter, has been calculated as described below, and is insignificant in terms of discharging large flood flows.

The Bottom Tunnels are constructed of concrete and have a width of 4.5 m and height of 5.0 m. The total length of the tunnels is 297 m. Discharge from the tunnels is regulated by the operation of two 12 ton, 4.0 m by 4.0 m sluice gates. The gates are normally controlled using an
electrical/hydro-mechanical drive system; emergency operation is by crane. The controls for the
gates and standby gates are located in the administration, or headwater building.

An energy dissipation facility is located adjacent to the administration building and is comprised
of a throat, a concrete stepped energy dissipation section, and an outlet channel that ultimately
discharges water into an adjoining stilling basin. The stilling basin is approximately 20 hectares
in size and is located downstream of the dam, and east of the Yeyenkend and Allahyarli
Villages.

14.1.1 Results of Hydraulic Analysis for Bottom Tunnels

The bottom tunnel intakes and gates are located within the reservoir at the upstream end of the
tunnel, and therefore are exposed to the full hydraulic pressure of the reservoir water, as shown on Figure 4 in Appendix B. The discharge capacity of the Bottom Tunnels was calculated using equations for flow through a pressurized orifice. Because the tunnel gates are close to the size of the tunnels themselves, no orifice correction was used to calculate the maximum discharge capacity when the gates are fully open. The gate dimensions themselves were used, which is conservative.

The calculations are shown in Appendix H on pages 23 to 26.

The discharge capacity of the two Bottom Tunnels and the Derivation (Irrigation) Pipes involved
use of Bernoulli’s Equation (energy balance equation) incorporating available head based on
the normal operating level of 75.5 masl and the maximum water level of 77 masl. Head loses
were computed using both the Darcy-Weisbach equation and the Hazen-Williams equation, and
a conservative estimate of head loss from the two equations was used. Minor head loss
coefficients were derived based on specific transition characteristics (entry, exit, and bends).

Discharge Capacity of Bottom Tunnels under Normal Operating Levels

Since the Bottom Tunnels are identical in size, one tunnel was analyzed for discharge
capacity and the total capacity was computed as twice the single tunnel values. Table 9
shows the available geometric properties and water level information. The precise
configuration of the tunnels could not be determined from the information we were
provided; therefore, based on the available profile information, two 45-degree long radius
bends were assumed for the purpose of computing bend loses. Table 9 Available

<table>
<thead>
<tr>
<th>Two Identical Concrete Pipes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (L)</td>
</tr>
<tr>
<td>Height (H)</td>
</tr>
<tr>
<td>Width (W)</td>
</tr>
<tr>
<td>Outlet Invert</td>
</tr>
<tr>
<td>Outlet pipe top</td>
</tr>
<tr>
<td>Normal Operating Level</td>
</tr>
</tbody>
</table>
Two categories of head losses, namely, friction head loss and minor head loss, occur as the water flows from the dam out through the pipes. The following paragraphs describe the calculation of these losses.

**Friction Head Loss**

The first step for computation of friction loses in the non-circular pipes involves determining hydraulic diameter. For rectangular sections, the hydraulic diameter ($D_e$) is computed using the following equation.

$$D_e = \frac{2(H \times W)}{(H+W)} \quad (27)$$

The next step is to compute the Reynolds number ($Re$) which is use for calculation of friction factor and eventually friction head loss. By definition $Re$ is defined as

$$Re = \frac{V D_e}{\nu} \quad (28)$$

Where $V$ is the flow velocity (m/s) and $\nu$ is the specific viscosity of water, which is $1E-6$ m$^2$/s (at 20° C). Since at the start of the calculation velocity is unknown the values are computed using an assumed value, which is later iterated so that the velocity assumed in this step and that derived from the application of Bernoulli’s equation is the same.

For concrete pipes the absolute roughness coefficient, $\varepsilon$, was assumed to be $1.2E-3$ m. Using this value and Reynolds number the friction factor can be computed using the Swamee-Jain Equation:

$$f = \frac{0.25}{\left[ \log\left( \frac{\varepsilon D_e}{3.7} \right) + \frac{5.74}{Re^{0.5}} \right]^2} \quad (29)$$

The Darcy-Weisbach Equation can be used to compute the friction head loss from the friction factor and length of the pipe:

$$h_f = \frac{fLV^2}{2gD_e} \quad (30)$$

Where $L$ is the length of pipe in meters and $g$ is the acceleration due to gravity (9.81 m$^2$/s). Additionally, the Hazen-Williams equation (Equation 31) was also used to compute head loss. The Hazen-Williams equation is a dimensionally inconsistent equation, which assumes values in empirical units (English Units). Thus, all parameters used in the Hazen Williams equation were converted into empirical units to determine friction head loss in feet of water, which was subsequently converted into meters. The roughness coefficient $C$ in the following equation was assumed to be 100 (a recommended design parameter for concrete pipes).
\[ h_f(\text{feet}) = \frac{3.022V^{1.85}L}{C^{1.85}D^{1.17}} \]  \hspace{1cm} (31)

Table 10 tabulates the steps and corresponding computed head loss values.

**Minor Head Loss**

The general relationship used for computing minor head losses is shown below (Equation 32)

\[ h_m = k \frac{V^2}{2g} \]  \hspace{1cm} (32)

Where \( k \) is the minor loss coefficient. For the bottom tunnel three type of minor head losses occur, Entrance \((k = 0.5)\), Exit \((k = 1.0)\) and Bend \((2 \text{ bends, } k = 0.2)\)

Table 11 tabulates the computed head loss values for the bottom tunnels.
Table 10  Friction Loss Calculation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic Diameter (De)</td>
<td>4.74 m</td>
</tr>
<tr>
<td>Velocity (initial) V</td>
<td>12 m/s</td>
</tr>
<tr>
<td>Kinematic Viscosity (v)</td>
<td>1.00E-06 m2/s</td>
</tr>
<tr>
<td>Reynolds Number (Re)</td>
<td>5.66E+07</td>
</tr>
<tr>
<td>Roughness Coef. (ε)</td>
<td>1.20E-03 m</td>
</tr>
<tr>
<td>Epsilon/Dia. (ε/D)</td>
<td>2.53E-04</td>
</tr>
<tr>
<td>Friction factor (f)</td>
<td>0.014</td>
</tr>
<tr>
<td>Head Loss - Darcy (h_f)</td>
<td>6.65 m</td>
</tr>
<tr>
<td>Hazen-Williams C</td>
<td>100 (design value)</td>
</tr>
<tr>
<td>Head Loss – Hazen-Williams (h_f)</td>
<td>21.19 ft</td>
</tr>
</tbody>
</table>

Note: Velocity values iterated in table above to make it same as Velocity in energy conservation table (Table 12)

Table 11  Minor Loses

<table>
<thead>
<tr>
<th>Parameter</th>
<th>k</th>
<th>h_m (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bend Loss (2)</td>
<td>0.2</td>
<td>2.94</td>
</tr>
<tr>
<td>Entrance Loss</td>
<td>0.5</td>
<td>3.67</td>
</tr>
<tr>
<td>Exit Loss</td>
<td>1</td>
<td>7.34</td>
</tr>
</tbody>
</table>

Bernoulli Equation (Energy Conservation)

As the flow exits the bottom tunnel, it will undergo contraction, which is about 0.4 times the hydraulic radius of the pipe. Thus, the flow head at the outlet will have to be reduced by this amount. The difference of the head at the reservoir (water level) and at the outlet, based on the principle of conservation of energy, should be equal to the sum of velocity head of the flow at the outlet and the total head losses occurred (friction as well as minor).

The velocity head can be subsequently used to determine flow velocity (Equation 33) and flow rate (Equation 33) can be used to determine the corresponding flow rate (Q m³/s) out of the tunnel. As indicated before, the velocity assumed in Table 10 was iterated manually until its value becomes the same as that derived from Equation 34.

\[
V = \sqrt{\text{Vel. Head} \times 2g} \tag{33}
\]

\[
Q = V \times W \times H \tag{34}
\]

Table 12 tabulates the calculations described above to compute the flow rate of 270 m³/s from one Bottom Tunnel. Overall, a discharge capacity of 540 m³/s can thus be expected under normal operating water level.
Table 12  Energy Conservation

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow Contraction at outlet</td>
<td>0.95 m</td>
<td>0.4 X Radius</td>
</tr>
<tr>
<td>Head at Outlet</td>
<td>47.55 m</td>
<td></td>
</tr>
<tr>
<td>Head Difference</td>
<td>27.95 m</td>
<td>Reservoir vs Outlet</td>
</tr>
<tr>
<td>Head Difference = Velocity Head + Head Losses</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Friction Loss</td>
<td>6.65 m</td>
<td>Using Darcy Head Loss</td>
</tr>
<tr>
<td>Minor Losses</td>
<td>13.94 m</td>
<td></td>
</tr>
<tr>
<td>Velocity Head</td>
<td>7.36 m</td>
<td></td>
</tr>
<tr>
<td>Velocity</td>
<td>12.01 fps</td>
<td>Eq. 7</td>
</tr>
<tr>
<td>Flow (Q per tunnel)</td>
<td>270.32 m³/s</td>
<td>Eq. 8</td>
</tr>
<tr>
<td>Total flow (bottom tunnels)</td>
<td>540.64 m³/s</td>
<td></td>
</tr>
</tbody>
</table>

Discharge Capacity of Bottom Tunnels under Maximum Water Level

To compute flow capacity of the Bottom Tunnels at the maximum water level of 77 masl the calculations described above were repeated with a water level of 77 masl. Tables 13 through 16 tabulate the corresponding calculations.

From Table 16 the computed discharge rate from one bottom tunnel was determined to be 277 m³/s. Thus, an overall discharge rate of 554 m³/s can be expected at the maximum water level 77 masl.

Table 13  Available Data

<table>
<thead>
<tr>
<th>Two Identical Concrete Pipes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (L)</td>
</tr>
<tr>
<td>Height (H)</td>
</tr>
<tr>
<td>Width (W)</td>
</tr>
<tr>
<td>Outlet Invert</td>
</tr>
<tr>
<td>Outlet pipe top</td>
</tr>
<tr>
<td>Maximum Water Level</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bends</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number</td>
</tr>
<tr>
<td>Assuming 45 degree long radius</td>
</tr>
</tbody>
</table>
### Table 14  Friction Loss Calculation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value 1</th>
<th>Unit 1</th>
<th>Value 2</th>
<th>Unit 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic Diameter (De)</td>
<td>4.74</td>
<td>m [Eq. 1]</td>
<td>15.54</td>
<td>ft</td>
</tr>
<tr>
<td>Velocity (initial), V</td>
<td>12.32</td>
<td>m/s</td>
<td>40.42</td>
<td>fps</td>
</tr>
<tr>
<td>Kinematic Viscosity (ν)</td>
<td>1.00E-06</td>
<td>m2/s</td>
<td>(20°C)</td>
<td></td>
</tr>
<tr>
<td>Reynolds Number (Re)</td>
<td>5.81E+07</td>
<td></td>
<td>[Eq. 2]</td>
<td></td>
</tr>
<tr>
<td>Epsilon (ε)</td>
<td>1.20E-03</td>
<td>m</td>
<td>Concrete</td>
<td></td>
</tr>
<tr>
<td>Epsilon/Dia. (ε/De)</td>
<td>2.53E-04</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Friction factor (f)</td>
<td>0.0144</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Head Loss - Darcy</td>
<td>7.00</td>
<td>m [Eq. 4]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hazen-Williams C</td>
<td>100</td>
<td>(design value)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Head Loss - Hazen-Williams</td>
<td>22.24</td>
<td>ft [Eq. 5]</td>
<td>6.78</td>
<td>m</td>
</tr>
</tbody>
</table>

Note: - Velocity values iterated in table above to make it same as Velocity in energy conservation table

### Table 15  Minor Losses

<table>
<thead>
<tr>
<th>Loss Type</th>
<th>Coefficient (k)</th>
<th>Head Loss (h) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bend Loss</td>
<td>0.2</td>
<td>3.09</td>
</tr>
<tr>
<td>Entrance Loss</td>
<td>0.5</td>
<td>3.87</td>
</tr>
<tr>
<td>Exit Loss</td>
<td>1</td>
<td>7.74</td>
</tr>
</tbody>
</table>

### Table 16  Energy Conservation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value 1</th>
<th>Unit 1</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow Contraction at outlet</td>
<td>0.95</td>
<td>m</td>
<td>0.4 X Radius</td>
</tr>
<tr>
<td>Head at Outlet</td>
<td>47.55</td>
<td>m</td>
<td></td>
</tr>
<tr>
<td>Head Difference</td>
<td>29.45</td>
<td>m</td>
<td>Reservoir vs Outlet</td>
</tr>
<tr>
<td>Head Difference = Velocity Head + Head Losses</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Friction Loss</td>
<td>7.00</td>
<td>m</td>
<td>Using Darcy Head Loss</td>
</tr>
<tr>
<td>Minor Losses</td>
<td>14.70</td>
<td>m</td>
<td></td>
</tr>
<tr>
<td>Velocity Head</td>
<td>7.74</td>
<td>m</td>
<td></td>
</tr>
<tr>
<td>Velocity</td>
<td>12.33</td>
<td>fps</td>
<td>Eq. 7</td>
</tr>
<tr>
<td>Flow (Q per pipe)</td>
<td>277.35</td>
<td>m3/s</td>
<td>Eq. 8</td>
</tr>
<tr>
<td>Total flow (bottom tunnels)</td>
<td>554.69</td>
<td>m3/s</td>
<td></td>
</tr>
</tbody>
</table>
Discharge Capacity of Derivation Tunnels under Normal Operating Water Level

To compute flow capacity of the derivation tunnels under normal operating water level of 75.5 masl the calculations described above were repeated. Additionally since the derivation tunnels are circular pipes 1.22 m in diameter and 300 m long, the hydraulic diameter was set equal to the actual diameter and the length was also adjusted. The pipe material was assumed to be plain cast iron. Tables 17 through 20 tabulate the corresponding calculations.

From Table 20 the computed flow rate from one of the derivation tunnel was determined to be 10.5 m$^3$/s. Thus, an overall flow rate of 21 m$^3$/s can be expected under normal operating level conditions from the derivation tunnels.

<table>
<thead>
<tr>
<th>Table 17</th>
<th>Available Data</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Derivation Tunnel Two Identical Pipes</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>300 m</td>
</tr>
<tr>
<td>Diameter</td>
<td>1.22 m</td>
</tr>
<tr>
<td>Outlet Invert</td>
<td>48.5 masl</td>
</tr>
<tr>
<td>Top of Pipe</td>
<td>49.72 masl</td>
</tr>
<tr>
<td>Normal Water Level</td>
<td>75.5 masl</td>
</tr>
<tr>
<td>Bends</td>
<td>2</td>
</tr>
</tbody>
</table>

Assuming 45 degree long radius
Table 18  Friction Loss Calculation

<table>
<thead>
<tr>
<th>Hydraulic Diameter (De)</th>
<th>1.22 m</th>
<th>4.00 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Velocity (initial), V</td>
<td>9.0 m/s</td>
<td>29.54 fps</td>
</tr>
<tr>
<td>Kinematic Viscosity (ν)</td>
<td>1.00E-06 m2/s (@20°C)</td>
<td></td>
</tr>
<tr>
<td>Reynolds Number (Re)</td>
<td>1.09E+07</td>
<td></td>
</tr>
<tr>
<td>Roughness Coef. (ε)</td>
<td>2.40E-04 m Plain Cast Iron</td>
<td></td>
</tr>
<tr>
<td>Epsilon/Dia. (ε/De)</td>
<td>1.97E-04</td>
<td></td>
</tr>
<tr>
<td>Friction factor (f)</td>
<td>0.014</td>
<td></td>
</tr>
<tr>
<td>Head Loss - Darcy</td>
<td>14.05 m</td>
<td></td>
</tr>
<tr>
<td>Hazen-Williams C</td>
<td>100 (design value)</td>
<td></td>
</tr>
<tr>
<td>Head Loss - Hazen-Williams</td>
<td>61.52 ft</td>
<td>18.75 m</td>
</tr>
</tbody>
</table>

Note: Velocity values iterated in table above to make it same as Velocity in energy conservation table

Table 19  Minor Loses

<table>
<thead>
<tr>
<th></th>
<th>k</th>
<th>hf (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bend Loss (2)</td>
<td>0.2</td>
<td>1.65</td>
</tr>
<tr>
<td>Entrance Loss</td>
<td>0.5</td>
<td>2.07</td>
</tr>
<tr>
<td>Exit Loss</td>
<td>1</td>
<td>4.13</td>
</tr>
</tbody>
</table>

Table 20  Energy Conservation

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow Contraction at outlet</td>
<td>0.24 m</td>
<td>0.4 X Radius</td>
</tr>
<tr>
<td>Head at Outlet</td>
<td>49.48 m</td>
<td></td>
</tr>
<tr>
<td>Head Difference</td>
<td>26.02 m Reservoir vs Outlet</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Head Difference = Velocity Head + Head Losses</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Friction Loss</td>
<td>14.05 m Using Darcy Head Loss</td>
<td></td>
</tr>
<tr>
<td>Minor Losses</td>
<td>7.85 m Calculated Above</td>
<td></td>
</tr>
<tr>
<td>Velocity Head</td>
<td>4.13 m</td>
<td></td>
</tr>
<tr>
<td>Velocity</td>
<td>9.00 fps</td>
<td></td>
</tr>
<tr>
<td>Flow (Q per pipe)</td>
<td>10.52 m3/s</td>
<td></td>
</tr>
<tr>
<td>Total flow (deriv. tunnels)</td>
<td>21.03 m3/s</td>
<td></td>
</tr>
</tbody>
</table>

Discharge Capacity of Derivation Tunnels under Maximum Water Level

To compute flow capacity of the derivation tunnels at the maximum water level of 77 masl, the calculations carried out in Section 3.0 were repeated with a water level of 77 masl. Tables 21
through 24 tabulate the corresponding calculations. From Table 24 the computed flow rate from one of the derivation tunnel was determined to be 10.8 m³/s. Thus, an overall flow rate of 21.6 m³/s can be expected under normal operating level conditions from the derivation tunnels.

**Table 21**  
**Available Data**

<table>
<thead>
<tr>
<th>Derivation Tunnel Two Identical Pipes</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Length</strong></td>
<td>300 m</td>
</tr>
<tr>
<td><strong>Diameter</strong></td>
<td>1.22 m</td>
</tr>
<tr>
<td><strong>Outlet Invert</strong></td>
<td>48.5 masl</td>
</tr>
<tr>
<td><strong>Top of Pipe</strong></td>
<td>49.72 masl</td>
</tr>
<tr>
<td><strong>Max Water Level</strong></td>
<td>77 masl</td>
</tr>
<tr>
<td><strong>Bends</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Number</strong></td>
<td>2</td>
</tr>
</tbody>
</table>

Assuming 45 degree long radius

**Table 22**  
**Friction Loss Calculation**

<table>
<thead>
<tr>
<th>Hydraulic Diameter (De)</th>
<th>1.22 m</th>
<th>4.00 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Velocity (initial), V</td>
<td>9.26 m/s</td>
<td>30.38 fps</td>
</tr>
<tr>
<td>Kinematic Viscosity (ν)</td>
<td>1.00E-06</td>
<td>m²/s (@20°C)</td>
</tr>
<tr>
<td>Reynolds Number (Re)</td>
<td>1.13E+07</td>
<td></td>
</tr>
<tr>
<td>Roughness Coef. (ε)</td>
<td>2.40E-04 m</td>
<td>Plain Cast Iron</td>
</tr>
<tr>
<td>Epsilon/Dia. (ε/De)</td>
<td>1.97E-04</td>
<td></td>
</tr>
<tr>
<td>Friction factor (f)</td>
<td>0.014</td>
<td></td>
</tr>
<tr>
<td>Head Loss - Darcy</td>
<td>14.85 m</td>
<td></td>
</tr>
<tr>
<td>Hazen-Williams C</td>
<td>100 (design value)</td>
<td></td>
</tr>
<tr>
<td>Head Loss - Hazen-Williams</td>
<td>64.78 ft</td>
<td>19.75 m</td>
</tr>
</tbody>
</table>

**Note:** Velocity values iterated in table above to make it same as Velocity in energy conservation table

**Table 23**  
**Minor Loses**

<table>
<thead>
<tr>
<th></th>
<th>k</th>
<th>hf (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bend Loss (2)</td>
<td>0.2</td>
<td>1.75</td>
</tr>
<tr>
<td>Entrance Loss</td>
<td>0.5</td>
<td>2.19</td>
</tr>
<tr>
<td>Exit Loss</td>
<td>1</td>
<td>4.37</td>
</tr>
</tbody>
</table>
### Table 24  
Energy Conservation

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow Contraction at outlet</td>
<td>0.24</td>
<td>m</td>
</tr>
<tr>
<td>Head at Outlet</td>
<td>49.48</td>
<td>m</td>
</tr>
<tr>
<td>Head Difference</td>
<td>27.52</td>
<td>m</td>
</tr>
</tbody>
</table>

Note: Head Difference = Velocity Head + Head Losses

<table>
<thead>
<tr>
<th>Type</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Loss</td>
<td>14.85</td>
<td>m</td>
</tr>
<tr>
<td>Minor Losses</td>
<td>8.30</td>
<td>m</td>
</tr>
<tr>
<td>Velocity Head</td>
<td>4.37</td>
<td>m</td>
</tr>
<tr>
<td>Velocity</td>
<td>9.26</td>
<td>fps</td>
</tr>
<tr>
<td>Flow (Q per pipe)</td>
<td>10.83</td>
<td>m³/s</td>
</tr>
<tr>
<td>Total flow (deriv. tunnels)</td>
<td>21.65</td>
<td>m³/s</td>
</tr>
</tbody>
</table>

### 14.2 Discharge Spillway

The discharge spillway is an open channel spillway located on the left abutment of the dam at Station 8+46. The spillway consists of a 200-meter long entrance channel, a 1,000-meter long downstream channel, and a 200-meter long energy dissipation facility located at the end of the downstream channel. The energy dissipation facility consists of a concrete throat located adjacent to the downstream channel at Elevation 40.2 masl, followed by a concrete stepped energy dissipator and an outlet channel that ultimately discharges into a stilling basin located downstream of the outlet channel at Elevation 30.4 masl.

The base of the spillway is sloped at about 3 percent ($S_0=0.027$), and the average depth of the spillway channel is about 4 meters. The foundation of the spillway consists of weak Layer 4 clay sediments, and the applied pressure on the foundation is an average of approximately 1.5 kg/cm². Conservatively, the applied pressure on the foundation does not exceed 2.5 to 3 kg/cm².

An overflow weir is located along the eastern boundary of the spillway entrance channel and extends over a distance of approximately 120 meters. The top of the weir corresponds with the maximum controllable reservoir water elevation, which is 77 meters above sea level.

#### 14.2.1 Results of Hydraulic Analysis for Discharge Spillway

The discharge capacity of the overflow weir was calculated using the Francis weir equation, which is provided below in Equation 35:

$$Q = \frac{2}{3}C_1b \cdot (2g)^{1/2} \cdot H^{3/2}$$  \hspace{1cm} (35)

where: $Q$ is the discharge rate in m³/s, $C$ is the empirical discharge coefficient, $b$ is the width of the weir in meters, $g$ is the gravitational acceleration of the fluid in m/s², and $H$ is the hydrostatic head of the fluid over the weir in meters.

The weir discharge coefficient was approximated using the following Equation 36:
\[ C_1 = 0.602 + 0.083 \cdot (H/Y) \] (36)

where: \( H \) is the hydrostatic head of the fluid flowing over the weir in meters, and \( Y \) is the height of the weir in meters.

The results of the analysis used to calculate the intake capacity of the overflow weir are provided in Appendix I. The results shown in the last figure of Appendix I indicate that the weir is adequately sized to accommodate a volumetric flow rate of 216 m\(^3\)/s (100-year flood) at a reservoir water elevation of 78.1 m, which would maintain a freeboard of 1.9 m. In the event of an emergency, such as inability to lift the bottom tunnel gates, the spillway can discharge the calculated 1000-year flow of 311 m\(^3\)/s and maintain a freeboard of 1.6 m. The 1000-year calculated freeboard is less than the 1.9 meters required for safe operations. This means that the overflow weir spillway can only serve as an emergency discharge facility for such a storm event, with the primary discharge being directed through the bottom tunnels.

The discharge capacity of the spillway outlet chute was calculated using the Gauckler-Manning-Strickler formula. The following Equation 37 is a variant of the Manning equation.

\[ Q = (1.00/n) \cdot AR^{2/3}S^{1/2} \] (37)

where: \( n \) is the Gauckler-Manning roughness coefficient, \( A \) is the flow area (m\(^2\)), \( R \) is the hydraulic radius in meters, and \( S \) is the slope of the energy grade line, which is equivalent to the bed slope of the chute.

The elevation of the specific energy grade line with respect to horizontal distance was calculated using the Bernoulli equation, which is provided below in Equation 38:

\[ E = p/\rho g + v^2/2g + z \] (38)

where: \( p \) is the hydraulic fluid pressure in Pascals, \( \rho \) is the density of the fluid in kg/m\(^3\), \( g \) is the gravitational acceleration of the fluid in m/s\(^2\), and \( z \) is the elevation in meters.

The head loss due to friction, or frictional energy loss was approximated using the following Equations 39 and 40. Equation 39 is a variant of the Manning equation, and Equation 40 is the empirical Hazen-Williams equation.

\[ h_f = \frac{(Ln^2v^2)}{R^{4/3}} \] (39)

where: \( L \) is the total length of the chute in meters, \( n \) is the Manning roughness coefficient, \( v \) is the flow velocity in m/s, and \( R \) is the hydraulic radius in meters.

\[ h_f = \frac{(3.022v^{1.85}L)}{(C^{1.85}D^{1.17})} \] (40)

where: \( h_f \) is the friction loss in feet, \( v \) is the flow velocity in feet per second, \( L \) is the total length in feet, \( C \) is the Hazen-Williams roughness coefficient, and \( D \) is the effective flow diameter in feet.

The results of the hydraulic analysis used to calculate the discharge capacity of the spillway channel indicate that the spillway is adequately sized to accommodate volumetric flow rates of 650 to 1,300 m\(^3\)/s with Manning roughness coefficients varying between 0.022 and 0.011.
respectively. Note that all of these flow rates exceed the calculated 1,000-year flood discharge of 311 m³/s.

The results of the hydraulic analysis indicate that the elevation of the hydraulic grade line at the juncture between the spillway outlet channel and the stilling basin is equivalent to about 41.3 to 42.0 masl, which is less than the respective elevations of the dwellings and major structures associated with the Khalfalar and subsequent Yeyenkend and Allahyarli Villages. These structures are located between Elevations 44.0 and 50.0 masl.

14.2.2 Settlement of Spillway Foundation Clays

As previously mentioned, the foundation of the spillway consists of weak Layer 4 clay sediments, and the applied pressure on the foundation is an average of approximately 1.5 kg/cm². Conservatively, the applied pressure on the foundation does not exceed 2.5 to 3 kg/cm².

The predicted values of compression index and coefficient of secondary compression, $C_c$ and $C_\alpha$, for the Layer 4 clays are, respectively, 0.26 and 0.01. These values are common for a medium stiff to stiff clay material.

The results of the settlement analysis indicate that the summation of the consolidation and secondary settlement of Layer 4 clays subjected to an applied optimum pressure between 2.5 and 3.0 kg/cm² can vary between approximately 46 and 54 centimeters over a recurrence interval of 100 years. Similarly, the predicted settlement over a recurrence interval of 475 years can vary between 63 and 73 centimeters. Hence, the differential settlement over 200 meters, which is the length of the entrance channel, for recurrence intervals of 100 and 475 years is equivalent to 4.0E-04, or 0.04 percent, and 5.0E-04, or 0.05 percent, respectively.

15.0 EMERGENCY PREPAREDNESS PLAN

The Emergency Preparedness Plan (EPP) for the Vileshchay Dam, shown in Appendix J, includes four scenarios of emergency situations: (a) destruction and landslides due to an earthquake; (b) a fire accident; (c) attack by an enemy or terrorism; and (d) outbreak of disease or other health condition.

Based on the Azerbaijan dam safety management regulations, the EPP should be prepared and approved by the regional office of the Ministry of Emergency Situations every five years. The most recent EPP for Vileshchay Dam covers the period of from 2011 to 2015. The MES has sent an official letter to Dam operator to prepare the revised EPP for 2016 to 2020 and send it to MES. According to the World Bank Task Team, the revised EPP is under preparation by the Dam operator. The task team has recommended improvements in the EPP as follows:

(i) Add a scenario of water poisoning because water supply has been added to the reservoir’s function by the World Bank.

(ii) Update the EPP every year. The EPP has been reviewed and approved every five years. During the five years, there may be some uncertainty including social, economic, technical and economic conditions; to include these changed situations, the EPP should be updated every year.

(iii) The revised EPP can be improved by including the recommendations given by World Bank Task Force, see Appendix D.

The Consultants found that the existing four gates (which are more than 30 years old) are in poor condition and should be replaced promptly. The task team was pleased to learn that the dam operator has secured the funds in the 2016 state budget to replace the gates. The Consultants recommend replacing the old gates as early as possible.

16.0 CONCLUSIONS

The following conclusions can be drawn based on a review of the information provided to us, and the results of the engineering analysis and evaluations presented herein:

- The existing dam crest width of 15 meters is adequate to promote structural stability of the dam and ample vehicular access during inspections and maintenance operations.
- The existing five-meter wide berms are required to stabilize the 2 horizontal:1 vertical and 3 horizontal:1 vertical downstream embankment slopes and increase the structural stability of the downstream dam embankment slopes. These berms are essential especially during periods of seismic activity.
- The central core and upstream and downstream transition zones are necessary to provide adequate structural stability of the dam and to reduce seepage exit gradients at the toe of the downstream embankment slopes.
- The granular transitional, or filter zones prevent migration of the clay core materials into the gravelly-sand fill zones located upstream and downstream of the core. The gravelly sand materials borrowed from the Shikhlar Village reserve field are adequately sized to retain the clay core material and permit the seepage flow through the filter. The current filter arrangement is sufficient to provide the necessary protection against internal migration or erosion and to control the seepage flow.
- A transverse crack could potentially propagate through the entire width of the central core during periods of substantial seismic activity. The upstream and downstream filter zones are adequate to retain core material and prevent the extension of a transverse crack into the downstream sandy gravel and stone fill zones.
- The “prong” concrete seepage barrier wall is required to prevent seepage flow through the highly permeable Layer 2 sediments in the dam foundation, and to eliminate the risk of piping of materials through the sediments by reducing the magnitude of horizontal and vertical exit gradients along the toe of the downstream embankment slope. The current cutoff wall is sufficient for these purposes.
- The 4 horizontal:1 vertical upstream embankment slopes are adequately stable during the event of rapid drawdown; the upstream and downstream stone slope revetment materials are sized adequately to efficiently and expeditiously dissipate excess pore pressures during rapid drawdown of the reservoir water level.
- Settlement of the Layer 2, 5, and 6 sediments occurred during and immediately subsequent to construction. Settlement contributions from consolidation and secondary compression should be minimal throughout the remainder of the project’s design life. Secondary settlement, or creep of the Layer 4 sediments, however, is expected to be substantial over the reminder of the design life. These sediments exist below the spillway discharge structure and portions of the dam along the left river bank located between Stations 0+00 and 14+00.
• A liquefaction analysis of the Layer 2 sediments and dam fill materials was not conducted during this assessment because copies of the boring logs and raw laboratory test results were not provided.

• The spillway overflow weir located along the eastern boundary of the entrance channel is adequately sized to accommodate a flood flow of 216 m³/s (100-year event) while maintaining a freeboard of 1.9 meters. In an emergency situation, the spillway can discharge 313 m³/s (1000-year event) and maintain a freeboard of 1.6 meters.

• The spillway discharge chute is sized adequately to accommodate a flood flow that exceeds the 1000-year event.

• The energy dissipation facility for the spillway was designed adequately to prevent flooding of downstream residential settlements.

• The two bottom tunnels combined are capable of providing a discharge flow rate of approximately 540 m³/s at the operating level of 75.5 masl. At the maximum reservoir level of 77 meters, they can discharge 554 m³/s. Both discharges are much larger than the estimated 1000-year flood flow of 313 m³/s.

17.0 RECOMMENDATIONS

We recommend the following based on the results of the Vileshchay Water Reservoir dam assessment as documented herein:

17.1 Phase I Dam

• Inspections of the dams should be conducted on daily, quarterly, and annual schedules to monitor the condition of the dam, reservoir, river banks, water discharge structures and appurtenances, and surrounding areas with respect to time. Standard forms should be developed that include pertinent data and observations to be collected and recorded during the inspections to promote proper and adequate long-term maintenance and operation of the facility. Daily measurements should at a minimum include the following: reservoir water level, stilling basin water level, and flow rates from water discharge facilities. General reconnaissance of the dam slopes, dam crest, abutments, discharge structures, and stilling basin should be performed, and pertinent observations should be recorded. All changes in the general condition of the dam, discharge structures, abutments, and surrounding areas should be documented. Such changes may include changes in seepage water quantity and quality, and vegetative growth changes at the downstream toe of dam; and indications of settlement or slope instability, which may be evidenced in the form of sloughing, cracking, sliding, or subsidence.

• The quarterly inspection reports should include a summary of the observations made during the daily inspections, and a summary of the data recorded.

• Annual inspection reports should include all of the data and observations recorded during the quarterly inspections, in addition to pertinent survey data. Annual bathymetric surveys should be conducted within the reservoir and stilling basin. In addition, general surveys should be conducted along pre-designated cross sections located perpendicular to the dam centerline to measure angles of upstream and downstream slope inclination, crest width, and locations and dimensions of downstream slope stabilization berms. General surveys should also include the locations and corresponding elevations of settlement monitoring devices, piezometers, and groundwater monitoring wells.
• An annual dam safety inspection should be performed to assess and evaluate the condition of the dam, water discharge structures and appurtenances, river banks, reservoir, and downstream residential areas. The inspections should be conducted by a professional engineer competent in dam design engineering, dam construction, and post-construction dam assessments.

• The spillway channel should be rehabilitated to reduce the Manning roughness coefficient from a current estimated value of about 0.018 to a value equivalent to or less than 0.014. This will require sediments and vegetation to be removed from the spillway. In addition, spalling and weathering of the concrete surfaces of the spillway should be repaired and finished to a smooth surface to enhance the efficiency of the structure.

• The existing gates for the Derivation Pipes and Bottom Tunnels (which are more than 30 years old) are in poor condition and need to be replaced. These gates should be replaced as early as possible in order to avoid a potential failure condition.

• The severe leakage on the right side of the spillway connecting to the dam shoulder is a potential risk for safety of the dam and spillway. Soil erosion could lead to an uncontrolled piping condition that could endanger the integrity of the dam. The dam operator should pay close attention to any increase in the leakage and repair it as early as possible.

• The original design report recommended installing monitoring instruments, consisting of piezometers and settlement points, at various locations on the dam and abutments. The instrumentation has not been installed. The current monitoring by visual observations is not sufficient for effective dam safety management. The dam operator should install the recommended monitoring system to improve the monitoring of the dam.

• Seismographic instrumentation should be installed at the site to measure seismic activity during earthquake events.

17.2 Phase II Dam

• The interface between the existing Phase I Dam and the new Phase II expansion must be carefully designed and constructed in order to avoid weak zones and incompatible materials that could lead to excessive seepage and reduced stability.

• During final design of the Phase II dam, consideration should be given to the potential need for additional seepage control at the downstream toe of the dam; e.g., installation of a pervious toe drain with a seepage collection pipe that would allow measurement and monitoring of seepage flow rates.

• The initial filling of the reservoir must be done slowly, at a rate determined during the design phase, so that observations and measurements can be made to ensure that the performance of the dam is satisfactory.

• The monitoring system recommended in the design report should be reviewed and supplemented as needed for the Phase II construction. A sufficient quantity of piezometers and groundwater monitoring wells should be installed prior to initial filling of the Phase II reservoir. The instruments should be installed along the centerline of the dam crest and in the abutment areas to measure pore water pressures within and beneath the dam and in the abutments. An adequate number of piezometers should also be installed along the downstream toe of the dam, and along the downstream stabilization berms. Clusters of piezometers should be installed at each location to
different depths within the various dam fill materials and foundation sediments. The piezometer data should be used to calibrate the seepage model and the hydraulic conductivity of the various fill and subsurface geologic strata.

- Similarly, additional ground movement instrumentation should be installed at appropriate locations along the dam crest and in the abutments. The instruments should include settlement points and inclinometers in order to measure both surface movements and movements in the body of the dam.

- Inspections of the dam should be conducted on daily, quarterly, and annual schedules to monitor the condition of the dam, reservoir, river banks, water discharge structures and appurtenances, and surrounding areas with respect to time. Standard forms should be developed that include pertinent data and observations to be collected and recorded during the inspections to promote proper and adequate long-term maintenance and operation of the facility. Daily measurements should at a minimum include the following: reservoir water level, stilling basin water level, and flow rates from water discharge facilities. General reconnaissance of the dam slopes, dam crest, abutments, discharge structures, and stilling basin should be performed, and pertinent observations should be recorded. All changes in the general condition of the dam, discharge structures, abutments, and surrounding areas should be documented. Such changes may include changes in seepage water quantity and quality and vegetative growth changes at the downstream toe of dam; and indications of settlement or slope instability, which may be evidenced in the form of sloughing, cracking, sliding, or subsidence. The monthly inspection reports should include a summary of the observations made during the daily inspections, and a summary of the data recorded.

- Settlement of the Layer 2, 5, and 6 sediments occurred during and immediately subsequent to construction. Settlement contributions from consolidation and secondary compression should be minimal throughout the remainder of the project’s design life. Secondary settlement, or creep of the Layer 4 sediments, however, is expected to be substantial over the remainder of the design life. These sediments exist below the spillway discharge structure and portions of the dam along the left river bank located between Stations 0+00 and 14+00. Instrumentation should be installed to measure the total settlement and differential settlements at these areas during and following construction of the Phase II.

- A probabilistic risk assessment of the following potential failure modes should be conducted for the dam and water discharge structures and appurtenances:
  1. Failure of seepage barrier “prong” wall
  2. Liquefaction of Layer 2 sediments during seismic events
  3. Upstream and downstream slope instability during seismic events
  4. Continued settlement of spillway and dam founded atop Layer 4 clays
  5. Failure of relatively impervious central dam core

- A liquefaction analysis of the Layer 2 sediments should be conducted to evaluate the potential for substantial loss in shear resistance of these sediments when exposed to seismic forces. Dam translation may occur along its base if these sediments should liquefy. Borings and pertinent laboratory tests should be completed to perform the liquefaction analysis.
- The seepage and slope stability models should be calibrated using piezometer data. The hydraulic conductivity and strength parameters of the various fill materials and foundation sediments should be modified based on additional borings, field and laboratory tests, and piezometer data during the calibration of the model elements. More accurate estimates of slope stability factors of safety and seepage quantities can be made once the model elements are calibrated.

17.3 COSTS OF MEASURES RECOMMENDED FOR PHASE I DAM

A cost estimation that has been conducted for recommended measures of further safety of Phase I Dam, is given in the following Tables.

Table 25: Costs of the Fundamental Repair works of the Spillway

<table>
<thead>
<tr>
<th>Retaining walls and slab of Spillway</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Wall Surface (m²)</th>
<th>Unit Price (US$)</th>
<th>Total Price (US$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Entrance channel - interior faces - surface treatment &amp; coating w/reinforced layer -25 cm</td>
<td>300.00</td>
<td>6.00</td>
<td>1,800.00</td>
<td>123.00</td>
<td>221,400.00</td>
</tr>
<tr>
<td>Entrance channel - exterior faces - surface treatment &amp; coating w/reinforced layer - 25 cm</td>
<td>300.00</td>
<td>1.00</td>
<td>300.00</td>
<td>123.00</td>
<td>36,900.00</td>
</tr>
<tr>
<td>Entrance channel - surface treatment &amp; coping</td>
<td>336.00</td>
<td>0.80</td>
<td>268.80</td>
<td>114.00</td>
<td>30,643.20</td>
</tr>
<tr>
<td>Chute Spillway - interior faces - surface treatment &amp; coating w/reinforced layer - 25 cm</td>
<td>2,800.00</td>
<td>6.00</td>
<td>16,800.00</td>
<td>123.00</td>
<td>2,066,400.00</td>
</tr>
<tr>
<td>Chute Spillway - exterior faces - surface treatment &amp; coating w/reinforced layer - 25 cm</td>
<td>2,800.00</td>
<td>2.50</td>
<td>7,000.00</td>
<td>123.00</td>
<td>861,000.00</td>
</tr>
<tr>
<td>Chute Spillway - surface treatment &amp; coping</td>
<td>2,960.00</td>
<td>0.80</td>
<td>2,368.00</td>
<td>114.00</td>
<td>269,952.00</td>
</tr>
<tr>
<td>Slab - surface treatment &amp; reinforced concrete layer -25 cm thick</td>
<td>1,550.00</td>
<td>12.00</td>
<td>18,600.00</td>
<td>130.00</td>
<td>2,418,000.00</td>
</tr>
<tr>
<td>Sealings for Concrete Joints</td>
<td>1,780.00</td>
<td>0.00</td>
<td>0.00</td>
<td>45.00</td>
<td>80,100.00</td>
</tr>
<tr>
<td><strong>Total costs of Fundamental Repair Works</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>5,984,395.20</strong></td>
</tr>
</tbody>
</table>

Table 26: Total Costs of Measures Recommended for Phase I Dam Safety

<table>
<thead>
<tr>
<th>Cost Items</th>
<th>Unit</th>
<th>Unit Price (US$)</th>
<th>Qty</th>
<th>Estimated Cost of Measures for further safety of Phase I Dam (US$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>General Surveys for annual inspection, to measure angles of upstream and downstream slope inclination, crest width, the locations and corresponding elevations of seepage outlet pipes, water discharge facilities and appurtenances, settlement monitoring devices, piezometers, groundwater monitoring wells and locations, and dimensions of downstream slope stabilization berms (by an experienced dam design engineer once a year)</td>
<td>LS</td>
<td>32,500</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>Bathymetric Surveys for annual inspection once a year, and each flood event</td>
<td>LS</td>
<td>17,500</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>Fundamental repair of the spillway * see BoQ in Table 25</td>
<td>BoQ</td>
<td>5,984,395</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>Replacing the four gates for water supply and discharge tunnel, including installation of electrical</td>
<td>Nos</td>
<td>155,000</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>Installing monitoring Instruments, consisting of piezometers and settlement points, at various locations on the dam and abutments</td>
<td>LS</td>
<td>125,000</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>Seismographic instrumentation</td>
<td>LS</td>
<td>24,000</td>
<td>1</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
As seen above table 88% of the budget for costs should be allotted for spillway fundamental repair works.

18.0 DISCLAIMER

This assessment report was prepared using documents provided by the Client: specifically “Data and Information given by VWR Administration on the Condition of Dam and Operation" and a partial translation of the design basis report, as listed in the Reference section herein. The following data and documentation were not available during the preparation of this dam safety assessment report:

- Boring logs and other field explorations
- Calculations
- Design drawings with complete cross sections
- Laboratory test data on soils and rocks

The following engineering analyses and evaluations were not conducted because the aforementioned data and documentation were not provided for the preparation of this report:

- Dam fill and foundation liquefaction analysis
- Calibration of seepage model elements
- Uplift of water discharge facility at Station 18+40

The following engineering analysis and evaluations were conducted using assumed or empirically determined parameters based on the information contained within the translation of the design report and the document titled “Data and Information given by VWR Administration on Summary of the Dam Safety Condition of Dam and Operation:":

- Evaluation of primary compression index, $C_c$
- Evaluation of coefficient of secondary compression
- Partial evaluation of seismic peak ground acceleration and velocity
- Evaluation of wind/wave analysis including the maximum precipitation

The hydrological analysis was conducted using the literature, data, schematics, and details as provided in the document titled “Data and Information given by VWR Administration on Summary of the Dam Safety Condition of Dam and Operation," and as provided in daily and monthly spreadsheets of Reservoir_Inflow_Outflow Data, shown in Appendix I.
19.0 REFERENCES

1. Ahrens, J. P. (1981a). “Irregular Wave Runup on Smooth Slopes,” Technical Aid No. 81-17, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.


FIGURES

FIGURE 1   Site Vicinity Map
FIGURE 2   Vileshchay Water Reservoir Stage-Storage Relationship for 1st Phase
FIGURE 3   Vileshchay Water Reservoir Exceedance Probability-Discharge Relationship for 1st Phase
APPENDIX A

Excerpt From Design Report for Vileshchay Water Reservoir (VWR)

Summary of the Dam Safety Condition of Dam
APPENDIX B

Visual Surveillance Survey Results and Recommendations for VWR

Summary of the Dam Safety Condition of Dam
APPENDIX C

Minutes of 2015 Inspection by Azerbaijan Government Committee
APPENDIX D

APPENDIX E

Subsurface Geologic Profile Along Longitudinal Axis of Dam

Design Cross Sections Located at Stations 16+00 and 21+00
APPENDIX F

Results of Seepage Analysis
APPENDIX G

Results of Slope Stability Analysis
## APPENDIX H

Calculations

### TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>SUBJECT</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Empirical Prediction of Hydraulic Conductivities</td>
<td>1</td>
</tr>
<tr>
<td>Empirical Estimation of Primary Compression Index</td>
<td>3</td>
</tr>
<tr>
<td>Wind/Wave Analysis</td>
<td>4</td>
</tr>
<tr>
<td>Required Thickness of Upstream Dam Stone Revetment</td>
<td>12</td>
</tr>
<tr>
<td>Design of Dam Granular Filter/Transition Zones</td>
<td>15</td>
</tr>
<tr>
<td>Hydraulic Flow Analysis for Bottom Tunnels</td>
<td>23</td>
</tr>
<tr>
<td>Hydraulic Flow Analysis for Derivation Tunnels</td>
<td>25</td>
</tr>
<tr>
<td>Hydraulic Flow Analysis for Spillway Outlet</td>
<td>27</td>
</tr>
<tr>
<td>Consolidation Settlement of Spillway Foundation Clays</td>
<td>30</td>
</tr>
<tr>
<td>Secondary Settlement of Spillway Foundation Clays</td>
<td>33</td>
</tr>
</tbody>
</table>
APPENDIX I

Results of Hydrological/Hydraulic Analyses
APPENDIX J

Emergency Preparedness Plan
### FIGURES

| FIGURE 1 | Site Vicinity Map |
| FIGURE 2 | Vileshchay Water Reservoir Stage-Storage Relationship for 1st Phase |
| FIGURE 3 | Vileshchay Water Reservoir Exceedance Probability-Discharge Relationship for 1st Phase |
FIGURE 1
DAM SAFETY ASSESSMENT REPORT
VILESHCHAY WATER RESERVOIR

Les Bromwell, P.E., Sc.D.
Engineering Consultant
505 Tulip Lane
Vero Beach, Florida 32963
FIGURE 2
DAM SAFETY ASSESSMENT REPORT
VILESHCHAY WATER RESERVOIR
FIGURE 3
DAM SAFETY ASSESSMENT REPORT
VILESHCHAY WATER RESERVOIR

VILESHCHAY WATER RESERVOIR EXCEEDANCE PROBABILITY DISCHARGE RELATIONSHIP FOR CONSTRUCTION PHASE ONE

Vileshchay Water Reservoir Exceedance Probability-Discharge Relationship for Construction Phase One

recurrence interval = 100/probability (%)
APPENDIX A

Excerpt From Design Report for Vileshchay Water Reservoir (VWR)

Summary of the Dam Safety Condition of Dam
4. NATURAL CONDITIONS OF THE CONSTRUCTION REGION

The Vilash River reservoir is located in the administrative region of Masalli of the Republic of Azerbaijan, between the foothills of the Talysh Mountains and Lankaran lowland. The mountainous landscape extends 2-2.5 km down towards the eastern side of the fold of the reservoir and turns into a gently sloping lowland towards the Caspian Sea. The entire lowland surrounded by the Talysh Mountains in the west and Caspian Sea in the east belongs to the subtropical zone of Lankaran and extends from Astara region to Bilasuvar region in the form of a thin strip.

The Talysh Mountains consist mainly of lower mountains. Their separate heights reach 2350-2470 m. The watersheds and slopes of the mountains are almost deprived of vegetation. Parts below the heights of 900-700 m are covered with forests.

The rivers in this area begin in these mountains and relatively large ones among them are the Lankaran River, the Astara River and the Vilash River.

4.1. The Climate of the Region

Lankaran zone has different climates ranging from relatively warm, quite wet and mild winters, hot and dry summers, autumns and winters with high precipitation to cold semi-desert climate. Relatively warm and dry summer climate dominates in the lowland and adjacent foothill lanes. The annual precipitation amount in this area is 320-1440 mm and occurs mainly in the spring and autumn months.

Cold semi-desert climate is found in the mountains over 2000 m in height.

Air temperature. The annual average air temperature of Lankaran lowland is 13.9-14°C. January is the coldest month of the year (with an average temperature of +2.9°C) and the hottest is July (with an average temperature of +25.1°C).

The dramatic increase in the air temperature occurs at the beginning of April, while the dramatic decrease occurs at the end of November. The lowest temperature of -23°C is observed in January, while the highest temperature of + 41°C is observed in August.

Precipitation. The distribution of precipitation in the region varies depending on the distribution of mountain ranges and the distance from the sea. The average annual precipitation amount in the parts of the lowland located in Masalli region
varies between 540-850 mm from east to west. The average annual precipitation amount in the parts over 100-500 m in height reaches 1250 mm.

**Air humidity.** The humidity in the area varies between 60% in July and 89% in November and December.

**Wind regime.** South-east and north-west winds prevail throughout the year. Of these two winds, the former is observed from February to October, while the latter is observed in November and January for short periods of time. Information on the winds with speed above 15 mph is given in Table 6.4.

### 4.2. The Hydrology of the Vilash River

The Vilash River starts at 2203 m high Kuludash peak, from the north-western slopes of the Talysh Mountains. Seven rivers join the Vilash River by the fold of the dam (Khalfalar station), the largest of which is its left tributary – the Matali River and flows into the Vilash River in the direction of the dam’s arrow fold. The bed of the Vilash River is U-shaped throughout its entire flow. Down the fold of the dam and starting from the place it flows into the Caspian Sea, the Vilash River becomes a lowland river. This part of the river bed has lost its form and takes the form of a gorge down the city of Masalli.

**Water regime.** The Vilash River is related to floodable rivers, with 70-80% of its flow contributed by flood waters and 30-20% by melting snow. Floods and torrents can occur in the basin throughout the year, but are mainly observed during cold seasons. Annual maximums occur in May and autumn.

The average flow is formed in January and continues for 35-45 days.

**The river flow.** The area of the river basin is 785 km² in the upper parts of the area where the reservoir is built and the length of the river is 62.5 km. The basin of the Matali River on the other hand is 79.3 km² and its length is 21.1 km.

The measurements over the river flow have been carried out on a regular basis since 1938 and and the gaps left in 1943, 1944 and 1945 have been calculated by the method of analog and restored.

Data on the observations over the river flow carried out during a 48-year period (1938-1985) is given in Table 8.4.
## Table 8.4

<table>
<thead>
<tr>
<th>#</th>
<th>YEAR</th>
<th>FLOW (monthly)</th>
<th>Flow (year)</th>
<th>Provision of flow</th>
<th>Flow - May to August (monthly)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>mln. m³</td>
<td>(mln. m³)</td>
<td>as % of total</td>
<td>(mln. m³)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1939</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>2</td>
<td>1940</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>3</td>
<td>1941</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>4</td>
<td>1942</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>5</td>
<td>1943</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>6</td>
<td>1944</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>7</td>
<td>1945</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>8</td>
<td>1946</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>9</td>
<td>1947</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>10</td>
<td>1948</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>11</td>
<td>1949</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>12</td>
<td>1950</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>13</td>
<td>1951</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>14</td>
<td>1952</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>15</td>
<td>1953</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>16</td>
<td>1954</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>17</td>
<td>1955</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
</tbody>
</table>

As can be seen from the table, the river’s average long-term flow is 181.35 million m³ and varies between 326.82 - 50.07 million m³.

The flow of eroded materials (soil, rocks) brought by floods. The observations on this flow have been carried out on the Matali River as well as its tributaries. The study of the flood flows was carried out over 21 years during the period projection
of the 1st and 2nd stages of the reservoir. These studies revealed that the eroded materials brought to the Vilash River constitute 5.4 kg/h with turbidity of 40 g/m³, while the eroded materials brought to the Matali River constitute 0.2 kg/h with turbidity of 0.23 g/m³. The general amount of the flow of the eroded materials is accepted to be 5.6 kg/h and this has been used in the calculation of the "dead volume" of the reservoir.

The granulometric composition of dependent eroded materials is given in Table 10.4.

<table>
<thead>
<tr>
<th>illegible</th>
<th># of measurements</th>
<th>Particle Content - %, Diameter - mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.0-0.5</td>
<td>0.5-0.2</td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>illegible</td>
<td></td>
<td></td>
</tr>
<tr>
<td>illegible</td>
<td>99</td>
<td>1.2</td>
</tr>
<tr>
<td>illegible</td>
<td>31</td>
<td></td>
</tr>
<tr>
<td>illegible</td>
<td></td>
<td></td>
</tr>
<tr>
<td>illegible</td>
<td>44</td>
<td></td>
</tr>
<tr>
<td>illegible</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Average Number</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: The highlighted cells could not be read and/or confusing.

The granulometric composition of contributed sediments reaches 300-400 mm. The sediments reach 30% of the volume of the eroded materials. The average annual figures on the flows of the eroded materials brought by the rivers of Vilash and Matali are given in Table 11.4.

<table>
<thead>
<tr>
<th>INTENSIVE FLOW OF THE RIVER IN DAM SITE</th>
<th>Table 11.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>NAME</td>
<td>Silts (monthly)</td>
</tr>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Flow rate of suspended silts - R (kg/sec)</td>
<td>1.8</td>
</tr>
<tr>
<td>Volume of suspended silts - W, (thousand ton)</td>
<td>4.8</td>
</tr>
<tr>
<td>Volume of bed silts - W, (thousand ton)</td>
<td>1.4</td>
</tr>
<tr>
<td>Total volume of silt flow (thousand ton)</td>
<td>6.2</td>
</tr>
</tbody>
</table>

### 4.3. Geological Condition of the Construction Site

The Vilash River reservoir is located on the border of the foothills of the Talysh Mountains and alluvial-prollovall foothill lane. The width of the lower parts of the relatively wide foothill is 3-6 km and is fragmented by the hydrographic networks
of the rivers and dry valleys and is filled with proluvial-delluvial sediments as a result of the mountain erosion.

Such eroded parts can be found on the right bank of the river near the village Gariblar. The geological structure of the area of the projected reservoir consists mainly of the third and fourth period sediments.

The third period rooty rocks consist of clay rocks with alevrit layers and tuff and sandstones with the thickness of from a few centimeters to a few dozen centimeters and dozens of meters in some places and occasional cases.

The fourth period sediments are found almost everywhere in various thicknesses and consist mainly of proluvial-delluvial sediments containing argillaceous rocks and rooty rock fragments.

Sandy and gravelly sediments with clump stones lie along the valley of the right bank of the Vilash River right down to the village Gariblar in the construction area of the 1st stage of the reservoir, from the existing bridge to Masalli-Yardimli road. Their main fillers are argillaceous clays and in some places sands and sandstones.

At the areas below the reservoir, the surface of rooty rocks is covered with the fourth period sediments. Within the boundaries of the valley, the fourth period sediments are themselves covered with modern period sand and sand stone fillings and sand and gravel alluvials.

Groundwater, formed by the infiltration from the rivers of Vilash and Matai, flows along the bottom of the river’s ancient valley. These waters have been found in the depths of 1.0-3.0 m within the boundaries of the valley and in the depths of 20.0-30.0 m in the edges.

Engineering and geological research and study have been carried out both for the 1st and 2nd stages of the dam construction.

4.3.1. Engineering and Geological Characteristics of the Dam Foundation
(Bottom of the Foundation)

The bed of the Vilash River consists of alluvial sediments (layer 2) with the thickness of up to 5.0 m lying on ancient alluvial sediments (layer 5 and 5a) with
the thickness of 16-17 m and these in turn are lying on half-rocky rooty rocks (host rocks).

The edges (sides) of the river consists of terrace sediments containing layers 3 and 3a with the thickness of up to 5.0 m and layer 4 with the thickness of up to 40 m.

They are lying on layer 6 containing rooty rocks along the river's left bank (on the relative height of 95 m) and in the relatively lower parts on both banks, they are lying on layer 5 containing ancient alluvial sediments with the thickness of 40.0 m. Ancinet alluvial sediments are lying everywhere on rooty rocks.

The results of tests on rooty rocks show that everywhere they contain cracks with uncertain directions. According to the fractionation categories, they are related to the 3rd category. The average distance between the cracks is 0.5-1.0 m.

The sediments contained in the foundation of the dam consist mainly of layer 2, 4 and 5 and sometimes layer 6 sediments.

So, the structure of the foundation where the body of the dam will be located is composed of layers 2, 4, 5 and 6. Other layers are involved on a very small scale and have very little effect based on their physico-mechanical and filtration properties.

### 4.3.2. Physico-mechanical Properties of Rocks

**Layer 1** – grayish-brown alluvial clays containing single gravel and fine sand layers. Its thickness is 0.5-1.0 m and is spread in few areas of the river bed and everywhere contains plant roots and remanants.

These sediments have been removed from the foundation of the dam everywhere. **Layer 2** – Modern alluvial sediments of the Vilash River. These sediments consist of sand and gravel, contain clump stones, their fillers are sand stones.

The average granulometric composition is as follows:

<table>
<thead>
<tr>
<th>The size of fractions</th>
<th>〉120 mm</th>
<th>10.2%</th>
</tr>
</thead>
<tbody>
<tr>
<td>120-60</td>
<td>29.8</td>
<td></td>
</tr>
<tr>
<td>60-20</td>
<td>25.3</td>
<td></td>
</tr>
<tr>
<td>20-2</td>
<td>19.7</td>
<td></td>
</tr>
<tr>
<td>2-0.1</td>
<td>8.7%</td>
<td></td>
</tr>
<tr>
<td>〈0.1</td>
<td>2.4</td>
<td></td>
</tr>
</tbody>
</table>
Its specific weight varies between 1.95-2.35 and the average rate is 2.1 g/cm³, its porosity varies between 25-28% and the average rate is 26%. Its estimated internal friction angle is 33°.

**Layer 3** – Dark-gray solid clays with thin sand layers. It lies on the 1st sub-bed terrace. Its natural humidity is 29%, has high porosity – 45% and low density – 1.3 g/cm³. The granulometric composition of these clays is as follows: clay – 45-65%, dust – 38-45%, sand – 7-10%. The plasticity ratio of the clay is 24. Internal friction angle of clays is 16-17 ° and cohesive force is 0.4-0.55 g /cm².

These sediments are limited and are not used in the foundation of the dam.

**Layer 3a** – Gravel, sand and clump stone sediments with fillings of clay and sand stones. These sediments are of alluvial origin and mainly analogous with layer 2.

These sediments are limited and are mostly found in the right slope. They are not found in the foundation of the dam and equipment.

**Layer 4** – Proluvial-delluvial clays with the thickness of 10-40 m. This layer is not homogeneous. It contains local sand layers, sandstones and gravels. These rocks are widespread along the area.

The natural humidity of the layer – 20.1%, natural density – 1.97 g/cm³, the density of the dry sediment – 1.64 g/cm³, the density of sediment particles – 2.71 g/cm³, porosity – 40.1%, porosity ratio – 0.67, plasticity rate – 22.4%. It contains clay particles – 40%, dust particles – 23.3% and sand – 36.7%.

The compression tests of these sediments show that in various depth intervals, they are inflatable under loads of up to 1.5 kg/cm² and up to 3 kg/cm² in rare cases.

Its estimated internal friction angle is 13° and cohesive force is 0.77 kg/cm².

**Layer 4 a.** Delluvial-prollevial clays spread inside the layer 4 in the shape of lens with low thickness (up to 0.5 m).

It contains clay particles – 24.5%, dust particles – 55.5% and sand – 20%.
The natural humidity of the layer – 23%, natural density – 1.85 g/cm³, the density of the dry sediment – 1.50 g/cm³, porosity – 40%, plasticity rate – 14%. Its estimated internal friction angle is 20° and cohesive force is 2.28 g/cm².

**Layer 4 b** – Sand stones spread inside the layer 4 in the shape of closed lens with the thickness of up to 3.0 m.

It contains gravel particles – 13%, sand – 53%, dust – 23% and clay – 11%.

**Layer 4 q** – Delluvial-prolluvial sediments containing gravel and clump stones with weak clay and sand stone fillings. It is found in layer 4 in the shape of lenses with high thickness (5-10 m). It is mostly widespread on the right bank of the Vilash River.

It contains clump stones and large sands (larger than 100 mm) – 17%, small sands (100-20 mm) – 40%, gravel – 24%, sand (2-0.5 mm) – 11%, dust and clay – 8%.

**Layer 5** – Alluvial-prolluvial sediments containing poor cemented layers of gravel and sand. Its average granulometric composition is as follows:

<table>
<thead>
<tr>
<th>Fraction</th>
<th>Size Range</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pebbles and clump stones</td>
<td>&gt;120 mm</td>
<td>12%</td>
</tr>
<tr>
<td>Pebbles</td>
<td>120-60</td>
<td>18.5%</td>
</tr>
<tr>
<td>Sand stones and gravel</td>
<td>60-2</td>
<td>2?%</td>
</tr>
<tr>
<td>Sand</td>
<td>2.0-0.05</td>
<td>16.5%</td>
</tr>
<tr>
<td>Dust</td>
<td>0.05-0.005</td>
<td>10.5%</td>
</tr>
<tr>
<td>Clay</td>
<td>&lt;0.005</td>
<td>9%</td>
</tr>
</tbody>
</table>

The specific weight of the sediment particles – 2.7 g/cm³, average humidity – 11.5%, the volume weight of the sediment – 2.1 g/cm³, porosity – 27-28%. Its estimated internal friction angle is 32°.

**Layer 5a** – Bluish-gray clays with sparse gravel, sand and dust particles. The average humidity of the clay – 23-24%, the volume weight of the wet sediment – 1.9 g/cm³, the volume weight of the sediment skeleton – 1.61 g/cm³, porosity – 41%, plasticity rate – 23% . Its estimated internal friction angle is 11° and cohesive force is 0.85 kg/cm².

Its granulometric composition is dominated by clays: sand – 4%, dust – 46% and clay – 50%.
Layer 6 – It consists of blue-gray clay rocks and contains tuff and sand stones and tuff-alevrit layers in the ratio of 6:1:1.

The strength of tuff and sand stones varies between 410-450 kg/cm² in dry conditions and between 300-330 kg/cm² when saturated with water. Its volume weight in natural lying position is 2.3-2.5 g/cm³.

The strength of tuff-alevrit rocks in dry conditions is a bit lower: between 70-350 kg/cm² and between 180-210 kg/cm² when saturated with water. Its volume weight in natural lying position is 2.4-2.6 g/cm³.

Clay rocks, when coming to the earth's surface, are exposed to intensive wearing as a result of the effect of the atmosphere. The cementing material of the rock contains clay, montmorillonite and hydromica.

Massive clay rocks show themselves as semi-rocky rocks.

The volume weight of monolithic clay rocks varies between 2.3-2.5 g/cm³ and their strength is 25 kg/cm².

Layer 7 – It is used in the support prism of the 1st and 2nd stage of the dam and flows from the gravel, sand and stone sediments with sand, sand stone and clay fillings. The volume weight of these rocks – 2.15 g/cm³, density – 2.06 g / cm³, internal friction angle – 35°.

4.3.3. Filtration Properties of Rocks

The filtration properties of rocks have been determined by the methods of water injection, water withdrawal and water casting applied on rocks taken from different depths.

The filtration coefficients (Kf) of the rocks used in the foundation of the dam are given below in Table 15.4.
Table 15.4.

<table>
<thead>
<tr>
<th>Line number</th>
<th>Layer number</th>
<th>Change interval of ( K_f ) (m/day)</th>
<th>Estimated figure, m/day</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>minimum</td>
<td>maximum</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>54.0</td>
<td>66.0</td>
<td>63.0</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>0.14</td>
<td>2.20</td>
<td>Has not been studied</td>
</tr>
<tr>
<td>3</td>
<td>3a</td>
<td>0.25</td>
<td>0.22</td>
<td>The research work of the 1st stage</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>0.62</td>
<td>0.72</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>4a</td>
<td>0.20</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>4b</td>
<td>1.10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>4q</td>
<td>1.16</td>
<td>6.00</td>
<td>3.80</td>
</tr>
<tr>
<td>8</td>
<td>5</td>
<td>0.06</td>
<td>6.50</td>
<td>2.20</td>
</tr>
<tr>
<td>9</td>
<td>5a</td>
<td>0.10</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>6</td>
<td>0.04</td>
<td>0.34</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>7</td>
<td>0.20</td>
<td>2.50</td>
<td></td>
</tr>
</tbody>
</table>

4.3.4. The Cup (Bowl) of the Reservoir

The reservoir cup has a complex configuration and it is due to the flow of two rivers into it and a lot of ravines.

From the hydrogeological standpoint, groundwater is disseminated everywhere of the reservoir.

Groundwater depth varies between 1.0 m- 5.0 m at the bed of the Vilash River and 0.5-2.5 m at the bed of the Matali River. The groundwater beyond the river bed lies deeper – at 15-50 m.

The direction of the groundwater flow is consistent with the flow of the Vilash River. Based on mineralization, groundwater is fresh water.

In the cup of the 2nd stage of the reservoir, there are not springs, fountains or the outpouring of groundwater to the surface.

0.8-1.0 km above the tail section of the cup of the 2nd stage of the reservoir, at an altitude of 120 m, on the right bank of the Vilash River, mineral water comes to the surface. During the projection period of the 1st stage of the reservoir, its consumption has been determined to be 6.5 l/sec. These mineral waters come from
the depth of the ground and are not related to the groundwater, instead they are thermal waters.

The geological structure of the reservoir cup is composed of clays (layer 4), gravelly-sandy sediments with clay and sand stone fillings (layer 2), gravelly-sandy sediments with clay fillings (layer 5) and clay rocks with tuff stones and tuff alevrit layers.

The thickness of layer 4 varies between 5- 30 meters and lies on the rooty rocks. The bed of the Matali River is as a deep ravine in the form of a narrow strip. The upper part of the bed is formed of gravelly-sandy sediments with clay and sand stone fillings and its thickness is 1.5-2 m.

In separate locations, it lies on layer 5 at the thickness of 2.0 m.

The physico-mechanical properties of the cup rocks are identical with the rocks located at the fold of the cup.

Below are the characteristics of the rocks of the reservoir cup:

The main part of the cup is covered with layer 4 clay rocks. The indicators of these rocks are the following:

Natural humidity – 13.1%, the volume weight of the wet sediment – 1.93 g/cm³, the volume weight of the skeleton – 1.76 g/cm³, porosity ratio – 0.562, plasticity rate – 24.3%, the amount of the clay fraction – 44.3%, dust – 45.5 and sand – 10.2% .

The internal friction angle of these clays is 18° and the cohesive force is 0.30 g/cm².

Modern alluvial sediments have almost been completely removed during the construction of the reservoir, and left only locally in insignificant amounts in some areas.

Its granulometric composition is as follows:

<table>
<thead>
<tr>
<th>Pebbles and clump stones</th>
<th>&gt;120 mm</th>
<th>7.0%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pebbles</td>
<td>120-60</td>
<td>25%</td>
</tr>
<tr>
<td>Sand stones</td>
<td>60-20</td>
<td>25%</td>
</tr>
<tr>
<td>Gravel</td>
<td>20-2.0</td>
<td>17%</td>
</tr>
<tr>
<td>Sand</td>
<td>2.0-0.05</td>
<td>19%</td>
</tr>
<tr>
<td>Dust</td>
<td>0.05-0.005</td>
<td>6.5%</td>
</tr>
<tr>
<td>Clay</td>
<td>&lt;0.005</td>
<td>4.5%</td>
</tr>
</tbody>
</table>
The volume weight – 2.1 g/cm³, porosity – 20%, internal friction angle – 35°.

The granulometric composition of alluvial-proluvial sediments is as follows:

<table>
<thead>
<tr>
<th>Pebbles and clump stones</th>
<th>&gt;120 mm</th>
<th>12%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pebbles</td>
<td>120-60</td>
<td>24.5%</td>
</tr>
<tr>
<td>Sand stones and gravel</td>
<td>60-2.0</td>
<td>27.5%</td>
</tr>
<tr>
<td>Sand</td>
<td>2.0-0.05</td>
<td>15%</td>
</tr>
<tr>
<td>Dust</td>
<td>0.05-0.005</td>
<td>11%</td>
</tr>
<tr>
<td>Clay</td>
<td>&lt;0.005</td>
<td>9%</td>
</tr>
</tbody>
</table>

The volume weight – 2.1 g/cm³, porosity – 27-28%, internal friction angle – 32°.

Layers 4b and 4g are found separately in the shape of closed lens.

Clays (layer 4b) are characterized by containing clay particles – 13%, sand particles – 53%, dust – 23% and clay – 11%.

The granulometric composition of gravelly-sandy sediments is as follows:

<table>
<thead>
<tr>
<th>Pebbles and clump stones</th>
<th>&gt;100 mm</th>
<th>16%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pebbles</td>
<td>100-20</td>
<td>39%</td>
</tr>
<tr>
<td>Sand stones and gravel</td>
<td>20-2.0</td>
<td>20%</td>
</tr>
<tr>
<td>Sand</td>
<td>2.0-0.05</td>
<td>10%</td>
</tr>
<tr>
<td>Dust</td>
<td>0.05-0.005</td>
<td>7%</td>
</tr>
<tr>
<td>Clay</td>
<td>&lt;0.005</td>
<td>8%</td>
</tr>
</tbody>
</table>

Its internal friction angle is 30°.

The analysis of the filtration properties of the rocks of the reservoir cup and their hydrogeological conditions indicates that rooty rocks (host rocks) lying at the depth of 40 m can be adopted as a regional water support.

### 4.4. Local Construction Materials

The reservoir region is poor in terms of the local materials, and therefore a large part of its resources has been used in the construction of the 1st stage of the dam.

Despite their wide spread across the region, clays have not been used as a construction material. These sediments lie either in the shape of separate lens or in the form of a coating with the thickness of 0.5-1.0 m and contain plant roots and
gravelly-sandy particles.

High quality clays are clays that contain 25-35% or more clay particles. The reserves of separate sediments are not a lot and since their thicknesses are low, their precise boundaries are unknown.

A similar situation is typical for the entire Lankaran zone.

Therefore, clays in layer 4 can be considered as waterproof elements of the dam.

As a manufactory of such clays, quarry No. 1 was opened on the left bank of the Vilash River, about 0.5-1.0 km far from the contact point of the dam.

The sediments of the quarry have hard mechanical composition containing clays – up to 53%, dust – 43%, and sand – 4%.

These sediments are composed of yellowish-gray clays containing white limestone crumbs.

The natural humidity of the sediments varies between 14.5-30%. At the higher part of 0.5 m, humidity varies around 20%, and within the range of 5-10 m depth, it varies around 30%.

The volume weight of the sediment in natural lying position is 1.99 g/cm³ and its density is 1.63 g/cm³. Average porosity ratio is 0.685 and water absorption ratio is 0.81.

These sediments are capable of swelling in some cases and their swelling ratio is 1%.

The clay reserves of the quarry No. 1 have been estimated to be 5.3 million m³ and 1.5 million m³ of this was used in the construction of the first stage.

Currently, the useful clay reserves here have been estimated to be 1.56 million m³ and 1.34 million m³ clay is needed for the construction of the 2nd stage.

For gravel-sand materials, a couple of fields have been identified. One of them is located on the left bank of the river in the Khalfalar village, while the other is located on the right bank of the river near the brick factory. The main field of
gravel and sand materials is in the Arkivan field located between the Arkivan village and Masalli city.

All of these are considered to be the main field and the materials will be used in the construction of the prism of the dam.

For the filter construction, the field in the reservoir cup in the village Shikhlar is conserved.

Considering the losses in the transportation process of materials and the amount of materials to be used by local organizations, local construction materials in these fields have been determined to be sufficient for the construction of the dam.

The granulometric composition and characteristics of the materials in the fields are given below in Table 16.4.

<table>
<thead>
<tr>
<th>Fields</th>
<th>Volume weight. g/cm³</th>
<th>Porosity %</th>
<th>Amount of fractions</th>
<th>120</th>
<th>120-60</th>
<th>60-20</th>
<th>20-2</th>
<th>2-0,1</th>
<th>0,1-0,005</th>
<th>&lt;0,005</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 Main fields</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Khalfalar 1</td>
<td>1,9</td>
<td>28</td>
<td>17</td>
<td>21</td>
<td>11,3</td>
<td>23,7</td>
<td>11,8</td>
<td>5,5</td>
<td>3,5</td>
<td></td>
</tr>
<tr>
<td>Khalfalar 2</td>
<td>2,0</td>
<td>-</td>
<td>18</td>
<td>20,8</td>
<td>13,5</td>
<td>20,3</td>
<td>13,7</td>
<td>4,1</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>Khalfalar 3</td>
<td>2,0</td>
<td>-</td>
<td>16,3</td>
<td>18,3</td>
<td>17,1</td>
<td>18,7</td>
<td>14,5</td>
<td>3,5</td>
<td>6,3</td>
<td></td>
</tr>
<tr>
<td>Arkivan</td>
<td>2,0</td>
<td>-</td>
<td>12,7</td>
<td>20,1</td>
<td>19,2</td>
<td>20,2</td>
<td>17,6</td>
<td>5,6</td>
<td>5,1</td>
<td></td>
</tr>
<tr>
<td>Reserve fields</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shikhlar</td>
<td>2,0</td>
<td>14,1</td>
<td>33,5</td>
<td>18</td>
<td>17,1</td>
<td>14,7</td>
<td>6,2</td>
<td>2,0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The filtration ratios of gravelly-sandy sediments in natural lying position vary between 7.0-1.0 m/day. The average filtration ratio during the tests of the sediments of the 1st stage was determined to be 1.0 m/day.

As a reserve field, Arkivan field can be expanded towards the road of Yardimli. In this case, 400-600 thousand m³ of gravel-sand can be obtained additionally.

All clay and gravel-sand quarries will be remedied after the completion of the construction work.

**5. Location of the Dam Junction**

A number of factors have affected the location of the dam junction along the selected dam line and the major ones are given below:
5.1. The construction sequence

The construction of the Vilash River reservoir is planned to be at two stages: the 1st stage construction will be according to the 1974 project and have the capacity of 46 million m³ with a pressure of 33.8 m and will irrigate 11275 hectares of land.

The 2nd stage construction aims at enhancing the capacity up to 138 million m³ by raising the dam height based on the current project plan and increasing the pressure up to 51.3 m and the irrigation area up to 33000 hectares.

Two-stage construction of such structures is beneficial when the construction of the 2nd stage can benefit from the operation of the 1st stage.

This key provision is reflected in the resolution of all reservoir issues, and first of all in the full location of the hydro system.

Two-stage construction requires such a solution of junction facilities and their locations from the project plan that they can be adjusted to the 2nd stage of the reservoir construction at normal operation conditions with minimum time and resource losses.

In particular, the redevelopment of the facilities or their parts associated with the reservoir discharge and the halt of the operation of the 1st and 2nd stages for a certain time period should be immediately resolved at both stages and in the 1st and 2nd stage construction project plans.

One such facility is the operation-construction water discharge facility. The redevelopment of the water discharge facility while the dam is located in the river would not be possible without separating the dam into sections to discharge the water consumption into the river.

Therefore, the water discharge facility was designed and built taking into consideration the 2nd stage.

Since a water discharge facility cannot fully discharge the operation flood consumption, the construction of a second flood spillway is required. In this case, the construction and location of the spillway should allow a minimum time restriction during the 2nd stage construction. During this time, by halting or shortening the operation of the 1st stage, the transmission to the horizon of the 2nd stage construction of the reservoir should be completed.

The adopted design of the flood spillway with a ditch allows the transfer to the capacity of the 2nd stage construction within a year and in this case, irrigation in the 1st stage construction is reduced by up to 50%.
5.2. The Locations of the Residential Areas

From the hills of the Talysh Mountains to the east, Lankaran lowland is very densely populated. In this sense, the location region of the Vilash River reservoir is no exception. A number of settlements start directly from the dam line and many of them descend toward the valley of the Vilash River.

Except the Gariblar village located a little above the dam line, on the right bank of the river, there is also the Isi village resettled in accordance with the 1st stage construction. Also, the resort district on the basis of Istitu mineral spring is located here and because of the impossibility of its resettlement, it has been protected in the 1st stage project.

1.2 km down from the dam line, on the left bank of the Vilash River, the Khalbalar village is located, followed by the Abbasbayli and Ege villages.

Further down, on the right bank of the river, the Allahari village is located.

The positions of these residential areas require such locations of all the hydro system facilities that the washout of the riverbanks on which villages are located becomes impossible.

In particular, this case applies to the Gariblar village and the resettled Isi village. The main issue here is the remaining of these areas under the water of the 2nd stage horizon of the reservoir.

5.3. Natural conditions

The conditions affecting the locations of the structure junctions are the topographical and engineering-geological conditions of the dam.

5.4. The location of the junction

So, after considering all the conditions, only one option in the location of the dam junction facilities is left.

This option consists of the following:

Thus, tunnel-type water discharge facility in the dam line is excluded; the irrigation water consumption can only be discharged through the bottom pipes in the dam body.

The construction of separate pipes is sensible for the discharge of these operation consumptions. Joint facilities are considered more economic in terms of initial costs and more efficient in terms of operation conditions.
The joint construction-operation water discharge facility is designed and built on the riverbed of the dam by cutting it perpendicular to the 18 + 40 picket.

The resettlement of the water discharge facility towards the right bank of the river has allowed the execution of the work until the completion of the main parts of the water discharge facility and direction of the construction consumption to it in the first phase of the 1st stage construction. The construction works at this stage were carried out with the help of low protection dikes in the riverbed and simple anti-flood measures in the form of walls. At the same time, sediment casting works have been strengthened on the left bank joint of the dam after construction consumptions are directed to the water discharge facility. The majority of the works is at the foundation of the dam, including the excavation-ditch walls.

Emergency flood spillway is located on the left bank, in the merging point of the 8+40 picket of the 1st stage construction. The position of the spillway allows the direction of the release of the river’s main consumption towards the empty places on the right bank. The leading bed of the spillway is along this bank.

It is considered that, the construction of the spillway is not limited in such a situation and it can simultaneously start and end with the dam construction. However, a number of reasons prolong the spillway construction.

The head junction of the right bank channel is located at the bottom of the right bank with the help of railway trenches. The one on the left bank is located in the same way. However, this is due to the resettlement of the water discharge facility and the banks of the reservoir cup.

Thus, while considering the options of the spillway and the water transfer facility, the tunnel type among them has not been adopted on one occasion.

- According to geological conditions, crossing at the large cross-section (at the area of 100 m² over the sediments).

During the construction period, it is not possible to create a safe area over the excavation area in low terraces, the large cross-sections of the excavation, and low mechanical indicators of the sediments. In such circumstances, the tunnel construction needs to be carried out in open excavations and in this case the large-scale soil and reinforced concrete works are required.

The cost of such a structure is very high, the execution is very complex and due to the opposite casting of the large amounts of land, the reliability is very low.
- According to the landscape of the area. The construction of the tunnel facility on the dam line is only possible with a curve in the plan. In this case and in particular during the construction period, the normal delivery of water in the low horizons is very difficult to create. This, in turn, makes it difficult to cast the dam because of the inevitable water rise.

The use of the right bank as the emergency spillway location is excluded. In this case, either the screen part of the right bank in the area of 38 pickets should be cut, or the beginning of the spillway should be located in the resort district. It requires a great length of the facility and will direct the emergency consumption towards the Khalfalar village on the left bank. This, in turn, can lead to washout in the village area.

From the geological and constructive standpoint, it is better to locate the operation-construction spillway within the lower part of the Vilash River valley. Here the structure foundation consists of the layer 2 and 5 sediments and the related works were done during the 1st stage construction.

The emergency spillway on the left bank is almost in a similar geological condition and it was located due to the reliability of the water delivery to the receiving part, the minimum volume of its construction work and the favorableness of its operation. The spillway passes through the riverbed the absolute level of which is below the water discharge facility. That is why, within the boundaries of the Vilash River bed, the left bank channel is in the form of an aqueduct with metal support pillars. With the help of these pillars, the channel approaches the rapidly flowing part of the flood spillway ditch, crosses over it and continues its flow through the reinforced concrete trenches in the shape of the right bank channel along the left bank.

The 1st stage dam starts from the axis of the flood spillway in the 8+46 picket, turns up in the direction of the flow and creates the screen on the right bank till the 40+00 picket. Here the dam leans against the foundation of the Isi promontory.

The 2nd stage dam starts from the 3+20 picket, passes through the surface of the Isi promontory and ends in the 42+00 picket.

Up from the Isi promontory, in the resort district, the thickening of the banks is planned. The thickening will be carried out up to the absolute level of 95 m in consideration with the 2nd stage construction work.
Access to the dam’s brow is by means of the bridge over the flood spillway from the left bank side. Access from the right bank is planned to be in the 42+00 picket from the resort district side.

6. The water economy calculations and and the capacity of the reservoir.

6.1. The graph of the water consumption project.

The 1st stage of the reservoir is planned to irrigate 11275 hectares of land and in this case, the total annual water volume for irrigation is 47.11 million m³. In this case, the efficiency rate of the system was adopted to be 0.90. In the current project phase, the efficiency rate for both the 1st and 2nd stage constructions has been specified and adopted to be 0.94.

Thus, in the current phase, the total water consumption amount for the 1st stage has been specified as below:

\[
47.11 \times 0.90/0.94 = 45.14 \text{ million m}^3/\text{year}
\]

In the current project, the 2nd stage irrigation is planned to use the total water consumption of 83.91 million m³/year for 20958 hectares of land.

Thus, the total water consumption for 32233 hectares of land is 129.03 91 million m³.

Table 17.6

<table>
<thead>
<tr>
<th>#</th>
<th>Irrigation for I stage mn. m³</th>
<th>Irrigation for II stage mn. m³</th>
<th>Irrigation for I &amp; II stages mn. m³</th>
<th>OutFlow of Reservoir m³/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>monthly</td>
<td>cumulative total</td>
<td>monthly</td>
<td>cumulative total</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>0.45</td>
<td>0.45</td>
<td>0.84</td>
<td>0.84</td>
</tr>
<tr>
<td>4</td>
<td>2.44</td>
<td>2.89</td>
<td>4.53</td>
<td>5.37</td>
</tr>
<tr>
<td>5</td>
<td>2.59</td>
<td>5.48</td>
<td>4.81</td>
<td>10.18</td>
</tr>
<tr>
<td>6</td>
<td>10.89</td>
<td>16.37</td>
<td>20.25</td>
<td>30.43</td>
</tr>
<tr>
<td>7</td>
<td>16.64</td>
<td>33.01</td>
<td>30.93</td>
<td>61.36</td>
</tr>
<tr>
<td>8</td>
<td>7.44</td>
<td>40.45</td>
<td>13.83</td>
<td>75.19</td>
</tr>
<tr>
<td>9</td>
<td>3.07</td>
<td>43.52</td>
<td>5.71</td>
<td>80.9</td>
</tr>
<tr>
<td>10</td>
<td>1.62</td>
<td>45.14</td>
<td>3.01</td>
<td>83.91</td>
</tr>
<tr>
<td>11</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Yearly</td>
<td>45.15</td>
<td>83.91</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
6.2. The determination of the regulation regime.

The regime of the Vilash River is characterized by large inequality. As such, during the 48-year observation period, average long-term flow was 181.35 million m³. During the projection of the 1st stage of the reservoir, average long-term flow consumption has been determined in accordance with the 34-year observation period.

However, the first flows recorded in the observation years of 1934-1947 differ even from the later observations till 1985.

It can be explained by the irregularity of the observations in early years. The determination of the flow in these years was based on calculations in similar rivers rather than direct observations. The reliability of these flows is not high and these rates are not consistent with the conformity of the flows after the spring period. However, in the perennial cutting, the average long-term flow indicates that these numbers distort the general situation; since, both the maximum and the minimum flows are observed in the first decades.

The list of flow indicators for the period of 1938-1983 are given in Table 18.6. The share of the flow in the summer months is noted here. The average long-term consumption in the first decade is 137.05 million m³. In this case, the average long-term consumption during the next 38 years is 193.10 million m³.

<table>
<thead>
<tr>
<th>#</th>
<th>Years</th>
<th>Summer, May-August</th>
<th>Yearly</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>flow min. m³</td>
<td>probability, %</td>
</tr>
<tr>
<td>1</td>
<td>1938</td>
<td>4.53</td>
<td>52.5</td>
</tr>
<tr>
<td>2</td>
<td>1939</td>
<td>14.04</td>
<td>66.7</td>
</tr>
<tr>
<td>3</td>
<td>1940</td>
<td>33.23</td>
<td>37.5</td>
</tr>
<tr>
<td>4</td>
<td>1941</td>
<td>7.37</td>
<td>95.8</td>
</tr>
<tr>
<td>5</td>
<td>1942</td>
<td>11.74</td>
<td>77.1</td>
</tr>
<tr>
<td>6</td>
<td>1943</td>
<td>26.23</td>
<td>32.1</td>
</tr>
<tr>
<td>7</td>
<td>1944</td>
<td>50.92</td>
<td>18.7</td>
</tr>
<tr>
<td>8</td>
<td>1945</td>
<td>11.76</td>
<td>72.9</td>
</tr>
<tr>
<td>9</td>
<td>1946</td>
<td>11.47</td>
<td>81.2</td>
</tr>
<tr>
<td>10</td>
<td>1947</td>
<td>9.58</td>
<td>87.5</td>
</tr>
<tr>
<td>11</td>
<td>1948</td>
<td>11.34</td>
<td>82.3</td>
</tr>
<tr>
<td>12</td>
<td>1949</td>
<td>23.25</td>
<td>47.9</td>
</tr>
<tr>
<td>13</td>
<td>1950</td>
<td>11.5</td>
<td>79.1</td>
</tr>
<tr>
<td>14</td>
<td>1951</td>
<td>5.12</td>
<td>97.9</td>
</tr>
</tbody>
</table>
The analysis of this table shows the following:

Water consumption in the main irrigation season of summer, including the 7th and the 8th months, is 74.0-106.0 million m³. In this period, even if the irrigation rate is reduced up to 70% compared to the optimal rate, irrigation in the natural flow is ensured for a five-year period. However, optimal irrigation rate is not ensured in any year of the period.

For this reason, it is not possible to ensure irrigation without the regulation of the flow.
6.3. Determination of the loss from the reservoir

The loss from the reservoir is equal to the sum of the losses derived from the evaporation of water from the surface of the reservoir and filtration. During the first operation period, losses at the time of the flow regulation will be high enough. However, these losses are very small and as the operational service gains experience in the consumption regulation, its volume will decrease even further. For this reason, the last losses are not taken into account in the report.

Evaporation depends on the reservoir area and time of the year.

Leakage losses consist of the sum of two types of losses. First, leakages to the slopes and banks of the reservoir, and later constant leakages to abutment below and neighboring valleys.

The first loss will occur in the early years of the reservoir filling, later its volume will sharply decrease and the volume of the leakage losses will almost has no impact. Considering the operation of the 2\textsuperscript{nd} stage of the reservoir and its filling start 8-10 years ago, it is possible not to take these losses into account.

As for the permanent leakages, they depend on the level of the reservoir. Of course, considering the Vilash River has large eroded sediments and they deposit in the reservoir, these losses will decrease as well. However, it is impossible to predict this decrease in advance. For this reason, this “reserve” is not taken into account in the reports and the leakage losses are considered permanent.

Considering that the main filtration indicators of the sediments forming the foundation of the dam are almost constant, the volume of losses from the reservoir has been kept the same in the 1\textsuperscript{st} stage projection.

The main sediments inside the dam and at the banks of the reservoir are alluvial-deluvial clump stone –sand- gravel sediments of layer 5. In this case, they are covered with modern alluvial sediments of layer 2a in the riverbed and the thick deluvial-proluvial sediments of the layer 4 along the banks.

The filtration rate of layer 2 reaches 50-60 m/day and the rate of 20 m/day is adopted in calculations. The filtration rate of layer 5 varies between 0.16-0 m/day and 2.0 m/day is adopted in calculations.

As for the different sediments of layer 4, different filtration rates are adopted for them.

The calculation of filtration in the 1\textsuperscript{st} stage projection phase was carried out with the help of pronges plunged 1.5-2.0 m deep to the rooty rocks.
Filtration losses are given in the table.

Table 19.6.

<table>
<thead>
<tr>
<th>Elevation of Water</th>
<th>Loss by Evaporation (monthly) m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td>1, 2, 3, 10, 12</td>
</tr>
<tr>
<td>55</td>
<td>-0.615</td>
</tr>
<tr>
<td>65</td>
<td>-0.055</td>
</tr>
<tr>
<td>75</td>
<td>-0.09</td>
</tr>
<tr>
<td>85</td>
<td>-0.14</td>
</tr>
<tr>
<td>95</td>
<td>-0.195</td>
</tr>
</tbody>
</table>

6.4. The dead capacity of the reservoir.

The solid flow of the river, as well as the coastal development materials above the normal pressure level, take part in the creation of the dead capacity of the reservoir.

Sediments brought by eroded materials to the Vilash River and its tributary the Matali River were studied for the period of 33 years. It is accepted that the bottom sediments (sludge) make up 30% of the dependent sediments brought by eroded materials.

The distribution of the solid flow volume in the Vilash River during the year is given in Section 4.2.

In the low-water summer months, when the reservoir is in operating and the water discharge is not fully done, all eroded sediments will deposit in the reservoir.

From September till March, during the flood times, water will be discharged and will be equal to the following:

- For the 1st stage reservoir: 181- 47 =134 million m³/year.
- For the 2nd stage reservoir based on the current project plan: 181-129 = 52 million m³/year.

Fractions smaller than 0.01mm and 50% of fractions between 0.01-0.05 mm will be discharged to the lower abutment during the water discharge process.

According to Table 10.4, fractions smaller than 0.01 mm and fractions between 0.01-0.05 mm make up 50% of dependent sediments. The volume of the eroded sediments discharged into the reservoir is:

- For the 1st stage construction: 134/181 x 50.0 = 37%
- For the 2\textsuperscript{nd} stage construction: \(52/181 \times 50.0 = 14.5\%\)

Considering that dependent eroded sediments make up 77\% of the total volume of 227.2 thousand tons of the solid flow, the amount of the solid flow deposited in the 1\textsuperscript{st} and 2\textsuperscript{nd} stage reservoirs is as follows:

\[-W1 = 227.2 \times (1.0-0.37 \times 0.77) = 162.0\ \text{thousand tons}\]

\[-W2 = 227.2 \times (1.0-0.145 \times 0.77) = 202.0\ \text{thousand tons}\]

The share of the wearing of the banks is very little in the creation of the dead capacity. The reason for this is relatively smaller length of the steep slopes in the reservoir. In addition, the main winds are blowing along the length of the reservoir in the direction of the dam and the expulsion length of the water is very small.

### 6.5. The capacity of the reservoir

The capacity of the reservoir was determined during the years 1946-1985 for a 38-year period. In the 1\textsuperscript{st} projection phase, the capacity of the reservoir was determined for a 34-year period. At that time, considering the relative humidity of the zone, climate conditions and optimal calculation norms especially in the low-water years, the annual calculation rate of 1\% was adopted. In optimal or calculation irrigation norms with 100\% insurance, 10\% decrease is allowed in the low-water years.

The calculations with these conditions in the 1\textsuperscript{st} projection phase were conducted in the section "The scheme of the main plan for determining the full capacity of the 2\textsuperscript{nd} stage reservoir".

This capacity was determined to be 130 million m\(^3\) and at that time the limit allowed in accordance with the local and topographic conditions was adopted.

The control of the capacity of 130 million m\(^3\) in the current phase shows that the adopted starting condition of the irrigation norm has not changed.

It should be noted that the sanitary discharge of the river in the 1\textsuperscript{st} projection phase is planned only for 6-year summer months. In the current phase, the sanitary discharge in accordance with the requirements of the water cadaster is adopted to be once a year.

The area of the reservoir mirror is 532 hectares in the NPH figures of the 2\textsuperscript{nd} stage and 64 hectares in the DVL figures. The working prism of the reservoir is: 93.0-58.0 35 meters. The greatest depth of the reservoir is 51.3 m and the average depth is: \(130.5 / 5.32 = 24.5\) m.
The calculation of the reservoir capacity based on the work balance during a 38-year period is given in the tables of this section.

7. Measures on the formation of the reservoir cup

7.1. Short characteristics of the reservoir and its supply sources.

The area of the 1st stage reservoir at a normal pressure horizon of 75.50 m is 350 hectares and of the 2nd stage reservoir at a normal pressure horizon of 93 m is 532 hectares. A large part of this area is covered by forests.

On the right bank of the Vilash River, at an absolute altitude of 95.0 meters within the reservoir cup, the Isi village is located. This village used to be located above the flooding area of the 1st stage. However, since the village located on the promontory extending towards the bank had steep, unstable banks, it was removed from the boundaries of the 2nd stage flooding area on the right bank.

Separate parts of the right bank above riverbed terrace in the reservoir cup are used for agricultural farming. Their total areas are several hectares. These areas have been removed aside and currently quarry No. 2 is under operation in their places.

On the left bank of the river, above the Matali River, an old cemetry has been discovered. The total area of the cemetry is 3.6 hectares.

During the 1st stage construction, the cemetry was removed aside.

On the right bank of the river, near the existing bridge, there is a pioneer camp. At the current stage, its removal aside is planned.

On the right bank, above the pioneer camp, Istisu springs are located. The springs are located outside the reservoir, at a distance of 4.5 km from the dam’s riverbed at an absolute altitude of 115-125 m.

The following works have been achieved during the 1st stage reservoir construction process:

1. The Isi village was resettled.

2. The 2nd stage reservoir cup was cleaned of trees and bushes at 60 hectares of land.

3. Works at 3.6 hectares of land in the cemetry.
7.2. The resettlement of the Isi village

The Isi village is located on the right bank of the river, at an absolute altitude of 80-85 m, on the promontory extending towards the river. At the 1st stage of the reservoir, the village is not left under water, but the steep banks of its promontory can get washed out and slide.

Taking these into account, the Isi village consisting of 17 yards was resettled in along the Masalli-Yardimli road.

7.3. The cleaning of the reservoir cup area from trees.

The reservoir cup is completely covered with forests and bushes. At the 1st stage, only 80 hectares of land is covered with trees. At the 2nd stage, it is 243.3 hectares of land.

The total cleaning of the reservoir cup from plants covered 323.3 hectares of land up to an absolute altitude of 94.0 m. Trees wet cut and removed aside by leaving them with small stumps (20-25 cm). There are not valuable trees of industrial importance.

7.4. Works in the cemetery area.

Two cemeteries have been found within the reservoir cup, at the foundation of the left bank of the dam. Initially found areas of cemeteries are 1.16 and 0.34 hectares.

In this case, the larger cemetery is not in use. The second cemetery is relatively new.

It was removed aside during the 1st stage construction with the help of a special brigade and under the control of the sanitary and epidemiological station.

The groundwater has been removed from the reservoir up to the depth of 2.0 m and the land surface was twice cleaned with chlorine lime.

8. The forecast of the flooding of the area

The construction of the Vilash River reservoir is carried out in complex geological and topographical conditions, in a very densely populated region and with residential areas being at a distance of 0.7-1.0 km from the dam.
Layer 5 is spread around the dam and 2-3 km down along the riverbed. This layer consists of clump stone-sand-gravel sediments containing various kinds of fillings. The permeability of these sediments is different and, depending on their thickness and the quality of their fillings, varies between 0.2-6.0 m/day.

This layer mainly makes up the ancient riverbed of the Vilash River and as a rule lies on rooty rocks. Rooty rocks consist of clay rocks containing sandstones or alevrolit paddings and are almost impermeable with the average filtration rate of less than 0.1 m/day.

On the banks of the river, this layer lies under the thick deluvial-proluvial sediments of layer 4 (more than 30 m). On the banks, they consist of clays containing different sediment paddings. In the riverbed, they are covered by modern alluvial sediments with the thickness of 2.5-3.5 m.

Under the top layers of the left bank terrace, the transverse spread of this layer is relatively small and is 300-400 m on the dam line. In this case, the width of the layer decreases towards the Khalfalar village. Its maximum thickness on the left bank is 10-12 m.

In the water-meadow of the Vilash River, in the district of the dam's line, layer 5 has a maximum thickness of 15-16 m. Up along the river, in the bridge region over the Masalli-Yardimli highway, this layer wedges completely in the riverbed. However, in the sediments of the right bank terrace, the thickness of this layer reaches up to 28-30 m.

Groundwater, as a rule, is in the sediments of layer 5 and feeds the riverbed with less inclination towards the river in the area of the reservoir cup. This inclination increases dramatically towards the depth in contact with rooty rocks.

On the right bank of the Vilash River, at a distance of 0.7-0.8 km from the right bank axis of the dam, the Qaraatbulag riverbed is located. The Gariblar village is located on the bank of this river.

The buildings in this village are located at lower altitudes with an absolute altitude of 75.0 m on the western edge and 59-60.0 m on the eastern edge.
On the left bank of the Vilash River, at a distance of 1.2 km from the dam line, the Khalfalar village is located. The farming and greenhouse buildings of this village descend down an absolute altitude of 38.0-39.0 m towards the river. The houses and major buildings of this village and the subsequent Abbasbayli village are located slightly above, at an absolute altitude of 45.0-50.0 m on the bank terrace.

The irrigation area begins 2-3 km down the reservoir and is located on the right and left bank terraces of the river. However, the area is at a sufficient height in this part and its absolute altitude is 48-80 m.

Taking into account the above mentioned points, the issue of forecast solution of the hydrogeological conditions of the reservoir region for the 1st and 2nd stage constructions was set.

This issue for the 1st and 2nd stage constructions was resolved in 1974 in the meliorative hydrogeology laboratory of the University of Kiev under the guidance of Professor I.E. Jarkov. At that time, the water horizon in the considered dam line for the 1st stage construction was adopted as 80.0 m in the reports and the possible limit for the 2nd stage construction was set as 93.0 m.

Two options were considered for the 2nd stage reservoir:

- The first variant – A deep prong in the form of an excavation- concrete wall plunged 2 m deep into the rooty rocks at the foundation of the screen. In this case, taking into account the construction conditions and the structure of the excavation-concrete wall consisting of a row of piles cutting each other, the filtration rate from this wall was adopted to be 0.1 m/day with current reserves. This wall was later built in the form of a concrete-gravel ditch with each section having the length of 6-7 m.

This further improved the quality of the wall and thus decreased the filtration rate by 5-10 times compared to the adopted figures in the reports.

- The second option – A shallow clay prong plunged 1.5-2.0 m deep into the layer 5 rocks by cutting only the alluvial sediments of layer 2.

The performed reports show that the use of the deep a wall-shaped prong reduces the leakage by 25-27% compared to the shallow clay prong presented in the second option. It should be taken into account in this case that the losses directly from the prongs were adopted at an increased level. This, in turn, allows for the reduction in
the filtration rates in the water economy reports of the reservoir area. The relative reduction in the filtration in the prong adopted in the project plan occurs at the 1100 m-length section. Here the dam crosses the riverbed between the 13-24 pickets. The difference between the losses is 35% at this section.

The difference between the leakage losses in the other parts of the dam, including the bank combinations is 12% at a ratio of the deep prong to the shallow prong.

Taking all these into consideration, a concrete-ditch wall was adopted at the dam foundation from the 12+50 picket to the 24 + 50 picket at the 1st and 2nd stage construction. According to the results of the numerous control tests, the filtration rate of the wall material is 0.0083 m/day on average. This is 12 times less than the figures adopted in the project reports. The filtration rate of the wall material varies between 0.002-0.02 m/day.

According to the drawn forecast hydro contour lines, the area adjacent to the reservoir is divided into areas with varying ascension degrees of groundwater.

All the area below the reservoir is divided into 3 zones with different degrees of change of the groundwater level.

- The first zone – a zone in which the level of groundwater rises from 10 m to 20 m compared to their natural levels. On the right bank, this zone is located up the western edge of the Gariblar village and above the 27+00 picket. On the left bank, this zone covers the area with a width of 300-400 m and the area till the junction along the left bank of the dam after the 13+00 picket.

- The second zone – a zone covering the area where water rises up to 3-10 m. These areas are directly adjacent to the first zone. It covers the area till the eastern borders of the Gariblar village on the right bank and 100-200 m down the first zone on the left bank.

- The third zone – a zone extending 3.0-3.5 km down along the river down the second zone and the dam. The groundwater horizon rises 0-3.0 m in this zone.

The hydrogeological conditions down this zone will not change in relation to the reservoir.

The most unfavorable hydrogeological conditions for residential areas are the following:
- The groundwater horizon in the south-western end of the Gariblar village rises up to 4-5 m above the surface, this figure reduces to 3.0 m towards the Qaraatbulaq River bed.

- The groundwater level in the Khalfalar village, within the boundaries of the first above the riverbed terrace rises up to a depth of 3-4 m above the surface.

The remaining parts of the adjacent territories have enough good conditions. The groundwater level in the territories including the Istisu resort district will be 20 m above the surface.

Thus, considering the impact of the reservoir projected on the basis of forecasts, the hydrogeological conditions can be considered quite satisfactory. Existing buildings and structures are not in any danger of flooding. The projected reservoir has almost no impact on the hydrogeological conditions of the irrigation areas and large residential areas including the city of Masalli located below.

Later, it is planned to conduct studies on the specification of the forecast of hydrogeological conditions. This specification is carried out with consideration of the results of the engineering and geological search works done during the projection.


Up from the reservoir, on the banks of the Vilash and the Matali Rivers, the fountain of mineralized, hot sulfur-hydrogen water mainly with a temperature of 45-55 degrees is observed.

It was not possible to accurately measure the consumption of all springs. Separate springs have a consumption of a few g/sec. However, the estimated consumption of all springs can be 14-15 l g/sec. During the year, the change in consumption is very small.

“Isti sular”, the largest spring, is located on the right bank of the Vilash River, at a distance of 2.5 km from the riverbed of the dam. This spring comes up to the earth's surface at an absolute altitude of 115-120 m with a consumption of up to 6 l g/sec. The water temperature at its issue place is 47 degrees. There is a resort on the basis of this spring near the village of Isi.

This spring has very high mineralization. However, when mixed with river waters, the mineralization falls sharply and depends on the river consumption.
The study on the mineralization of river waters was conducted at 14 stations over the Vilash River and 2 stations over the Matali River.

According to the results of the numerous analysis on river waters carried out mainly during the low-water periods (summer and winter), the dependence of mineralization on the river consumption became clear.

In separate cases, variations to both rules have been observed. In this case, with a sharp rise in consumption, the mineralization has fallen very little. However, these rare cases should be related to the start of the increase in floods. At this time, the washout of insoluble salts from the heights of the riverbed starts as a result of the rise of the horizon.

The sum of all ions in the Vilash River varies between 2826 mg/l at a consumption rate of 0.29 m³/sec and 270 mg/l at a consumption rate of 31 m³/sec. The sum of all ions at a consumption rate of 2.5 m³/sec is ............ mg / l.

10. The justification of the main calculation parameters of the dam junction.

The justification for all the calculation indicators and parameters of the dam were carried out in the 1st and 2nd stage projections. However, a number of requirements in the regulatory documents have changed during this time period, and therefore, they are considered in the current projection phase.

10.1. The substantiality class of the hydro system.

In general, the main structure of the reservoir determining the substantiality class of the hydro system is considered dam.

The maximum height of the dam for the 1st stage is 80.0-41.7=38.3 m. This figure is rounded up to 39.0 m in calculations. The height of the dam for the 2nd stage construction is 96.0-41.7=53.3 m.

According to Table 3 (river hydro-technical facilities) of the regulatory document SNIP II-I.62 which was in force during the projection for the 1st stage construction, the substantiality of the dam and all its main facilities were related to class 2. Currently, the document SNIP 2.06.01-86 is in force. According to Table 1 of Appendix 2, in case the dam foundation consists of sand-gravel and hard clay
sediments with large crumbs and its height is 35-75 m, the substantiability of the hydro system is related to class 2.

Generally near-dam facilities – operation-construction water discharge facility and emergency flood spillway work in relation to the consumption and horizon in the reservoir and provide enough security and reliability when discharging the flood waters. Therefore, similar to the dam, the substantiability of these structures is related to class 2.

The substantiability of other facilities not related to the safety of the dam, (right and left bank channel ditches, bridges and passages) is related to class 3.

10.2. The calculation consumption of the river.

During the projection of the 1st stage reservoir construction, in accordance with the regulatory document SNIP II.1.7-65, the river calculation consumption for class 2 substantiability was adopted to be 842 m³/sec with 0.1% insurance. Construction consumption was adopted to be 320 842 m³/sec with 10% insurance.

Currently, the regulatory document SNIP 2.06.01-86 is in force for this part.

According to Table 1 of this regulatory document, for class 2 substantiability, consumption with 1% insurance is calculation consumption and consumption with 0.1% insurance is considered to be control consumption.

According to this regulatory document, construction consumption is projected with 5% insurance.

The main facility of the hydro system working for construction consumption is a joint water discharge facility. This facility is fully completed in the 1st stage construction. Therefore, changing the construction consumption at this stage is meaningless.

At the current stage, the redeveloped flood spillway completely depends on the river calculation consumption. This facility should be monitored to the issue of the consumption with 0.1% insurance. Therefore, 842 m³/sec consumption is maintained for this facility. In this case, water passed through the water discharge facility with open bolts and water gathered in the reservoir is taken into account.
Thus, the distribution of the river's control consumption over the facilities is as follows:

- On completely open bolts over the two lines of the water discharge facility - 420 m$^3$/sec.

- From the 1.2 m tall automatic flood spillway of the water passage - 290 m$^3$/sec.

- Gathering in the reservoir cup with 1.10 m layer -132 m$^3$/sec.

It should be noted that with 10% insurance, the flood consumption is 320 m$^3$/sec and with 5% insurance, it is 401 m$^3$/sec, which is 25% more.

During the period of holding the 2$^{nd}$ stage flood consumption and discharging the consumption of the 1$^{st}$ stage, the filling of the reservoir was calculated based on this consumption.

10.3. Calculation seismicity.

According to the regulatory document SNIP II-7-81, the projected reservoir is located in the zone with a magnitude of 7. The construction region is exposed to earthquakes quite often.

Information on earthquakes in Lankaran zone has been gathered after the devastating earthquake Ararat on January 22, 1840. The earthquake which caused great destructions in the Ararat district had a magnitude of 5 in Lankaran.

Later, 84 earthquakes, occurring almost every year, were recorded in Lankaran zone between the years of 1840-1937. Seven of these earthquakes had a magnitude of 7, four had a magnitude of 6, and thirteen had a magnitude of 5. The intensity of the other 49 earthquakes was not determined, but they mostly had a magnitude of 4-5 and did not pass a magnitude of 6.

The frequency of the earthquakes noted was observed directly in Lankaran city. Earthquakes were rarely recorded to the north from Lankaran city, as well as in the reservoir region. They were recorded to have less magnitude compared to Lankaran city.

The earthquakes of this zone can be divided into 3 types:
- The first type – the effect of earthquakes mainly in Iran and Turkey. Despite the destructive power of these earthquakes in their epicenters, they reach the construction region in a weak form and mainly have a magnitude of less than 5.

- The second type – these earthquakes also occur outside the construction region. However, since these earthquakes occur at great depths, they cause vibrations in large areas of the republic and go very far from the construction region. This type of seismicity is weak in the construction region too.

- The third type – these earthquakes are considered completely local. These earthquakes occur at great depths of 50-55 km and are related to deep Lesser Caucasus front and the activities of Lahij-Gizilagaj fractures. These earthquakes are the most powerful and frequent.

Despite the high frequency of earthquakes, they have no destructive power in Lankaran zone.

There also local earthquake sources which become active once in 2-3 or even 11 years. The epicenters of these earthquakes are at a distance of 50-60 km from the projected reservoir.

The seismic micro-districtization of the Vilash River reservoir construction area and the determination of the calculation magnitude of its seismicity were carried out in 1973.

Special seismic expedition conducted several studies and did the following works:

- Engineering studies on the conditions of buildings and structures in Masally city and adjacent villages were conducted.

- The ancient buildings and sediments at their foundations in the construction area and adjacent locations were studied. These studies were conducted in the administrative regions of Masalli, Lankaran, Astara and Yardimli as well.

- The engineering-geological and hydrogeological conditions of the construction area were studied.
- The relative seismicity of the sediments at the foundations of buildings and structures was determined on the basis of the indicators of stronger earthquakes.

- The relative seismicity of the sediments comprising the dam foundation was determined on the basis of the measurement of the expansion rate of transverse and longitudinal waves.

The last works were determined with the help of the device complex in the seismic station conducting the engineering seismic measurements.

The study of the relative seismicity of the reservoir sediments with variety of devices allowed the evaluation of the seismicity of sediments. The increase of magnitude because of the mellowness of the sediments after the construction of the reservoir was calculated using the formula of S.V. Medvedev.

The results for all sediments as a result of these calculations are given in Table 10.21.

<table>
<thead>
<tr>
<th>№</th>
<th>Metering point of waves</th>
<th>Soil characteristics by geological report</th>
<th>Layer thickness m</th>
<th>Compressional wave velocity m/sec</th>
<th>Initial seismic activity ball</th>
<th>Incremental size due to watering ball</th>
<th>Estimated seismic magnitude subject to water saturation ball</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dam gate</td>
<td>Clay of layer 4</td>
<td>&gt;20m</td>
<td>465</td>
<td>7</td>
<td>1</td>
<td>8</td>
<td>The layers 4, 2 and 5 that cover the layer 6 comprise the basis of the dam</td>
</tr>
<tr>
<td>2</td>
<td>-</td>
<td>Gravel with clayey filler - layer 2</td>
<td>10</td>
<td>470</td>
<td>7</td>
<td>1</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>-</td>
<td>Pebbles with sandy fillers - layer 2</td>
<td>5</td>
<td>725</td>
<td>7</td>
<td>1</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>-</td>
<td>also</td>
<td>5</td>
<td>426</td>
<td>7</td>
<td>1</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Location across Vilesh river</td>
<td>aggregates with interlayers of tuff sandstone - layer 6</td>
<td>&gt;100m</td>
<td>2167</td>
<td>6</td>
<td>-</td>
<td>6</td>
<td></td>
</tr>
</tbody>
</table>

Thus, as a result of the works conducted, the calculation seismicity of the dam area was determined to have a magnitude of 8. The changing of the hydro-geological conditions during the filling of the reservoir was also taken into account.

13. Selection of the flood spillway option.

The surface of the reservoir flood spillway is in the form of a ditch.

Only an option of a flood spillway was considered in the 1st stage technical project plan in which the main plan of the structure in full development was resolved. In this option, the beginning of the spillway is redeveloped till the curve of the 2nd
stage construction. In this case, the head part of the spillway built till the curve for the 1\textsuperscript{st} stage reservoir is divided into separated parts.

In the current phase, the option of the redevelopment of the flood spillway without dismantling it has been considered.

The description of the redevelopment of the flood spillway and the comparison of options are given below. According to the recommendations of the current project, the description of the structure of the redeveloped spillway is given in the section “Flood spillway”.

13.1. The constructive solution of the redevelopment of the flood spillway.

The spillway in the 1\textsuperscript{st} stage construction consists of a one-sided water supply facility and 150 m-long water receiving ditch. This later turns into a rapidly-flowing ditch. The rapidly-flowing ditch, in its turn, ends in an inclined trampoline consisting of an extinguisher and water basher well.

The normal pressure level of the 2\textsuperscript{nd} stage construction is 17.5 m higher compared to the 1\textsuperscript{st} stage level. The redevelopment of the head of the flood spillway in the form of a mine spillway with a closed main tract in the lower abutment of the dam is planned by using this height.

Considering that the foundation of the spillway structure consists of the weak sediments of layer 4 (clays with thin sandstone paddings and crumb materials), the whole area of the foundation of the flood spillway ditch should be used. The burden on the foundation in this case is no more than 2.5-3 kg/cm\textsuperscript{2}.

A similar construction for the ditch of the water receiver with a mine spillway is adapted to the reservoir built on the Arpa River.

The structure of the water receiver consists of two joint water receiving ditches with a leading mine. It is followed by the 1\textsuperscript{st} stage spillway containing two-sided pipe tract in the water receiving ditch. In the vertical mine in the middle of the spillway, directing columns are installed for a better water supply. These columns direct all the consumption to the vertical mine.

Along the axis of the mine, on both sides of the ditches there are special air-transmitting cameras. They are kept closed to exclude the water entrance.
The structure of the spillway for the Arpa River reservoir was calculated for the reception and discharge of 350 $\text{m}^3/\text{sec}$ of water on a 1.5 m-height span.

On a 1.1 m-height span of the water receiving ditch, during the full maintainence of the structure, the flood spillway dispatches 265 $\text{m}^3/\text{sec}$ of water. Thus, all the 842 $\text{m}^3/\text{sec}$ of flood consumption of the river with 0.1% insurance is distributed as follows:

- 416 $\text{m}^3/\text{sec}$ of water is discharged through the two lines of the bottom water discharge facility in the horizon of the reservoir at an absolute altitude of 93.0 + 1.1 = 94.1 m.

- 265 $\text{m}^3/\text{sec}$ of water is discharged directly through the flood spillway.

-161 $\text{m}^3/\text{sec}$ of residual water is gathered in the reservoir at the expense of 1.1 m water layer.

However, with this option, the construction of the spillway becomes difficult, because its construction is being carried out in the facilities which are operating. Therefore, condition should be created so that flood waters do not enter the 1st stage spillway. This is possible only in one case: during the construction of the spillway, the reservoir should not be filled above the absolute altitude of 68.0 m.

During the filling up to this altitude, the irrigation of 11.3 hectares of land at the 1st stage is not possible.

In accordance with the regulations, the construction period of the spillway with a capacity of 43 thousand $\text{m}^3$ of concrete and reinforced concrete is 30 months.

The casting of the dam in the spillway interval with a capacity of 250 thousand $\text{m}^3$ requires 10-12 months. Given the coincidence of the separate works, a total of 36 months is accepted, with 24 months for the construction of the spillway and 12 months for the casting of the dam.

13.2. The constructive solution of the redevelopment of the flood spillway and the comparison of options.

According to the current project plan, the description of the structure of the redeveloped spillway is given in the spillway section.
In this section, the following should be noted:

Surface-type spillway has a bilateral water receiving head with a developed row of water dischargers. The width of the ditch with a rapid flow is reduced to 12 m and this is related to the reduction of the flood consumption at the expense of the increase of the consumption accumulation.

This spillway can be constructed parallel with the 2nd stage dam, regardless of the filling of the reservoir. The casting of the dam is carried out within the boundaries of the cleft of the 1st stage spillway. This cleft in the body of the dam should be filled within a year. During this period, the junction part of the new part of the spillway with the 1st stage spillway is built. To do this, 2.6 thousand m³ of reinforced concrete works should be done and 2.0 thousand m³ of trenches of the 1st stage construction should be dismantled.

The main technical and economic indicators of the compared options are given below in Table 26.13.

<table>
<thead>
<tr>
<th>#</th>
<th>Indicators</th>
<th>Measuring instrument</th>
<th>Version of reconstruction for shaft spill</th>
<th>Version of reconstruction for trench spill</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Height of overflow water</td>
<td>m</td>
<td>1.1</td>
<td>0.90</td>
</tr>
<tr>
<td>2</td>
<td>Outflow of output tract</td>
<td>m³/sec</td>
<td>285</td>
<td>296</td>
</tr>
<tr>
<td>3</td>
<td>Outflow transformation</td>
<td>-</td>
<td>161</td>
<td>131</td>
</tr>
<tr>
<td>4</td>
<td>Ground excavation under spill</td>
<td>thousand m³</td>
<td>-</td>
<td>650.00</td>
</tr>
<tr>
<td>5</td>
<td>Reinforced iron of shaft spillway</td>
<td>-</td>
<td>4,799</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>Reinforced iron of output tract</td>
<td>-</td>
<td>35,363</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>Reinforced iron chute</td>
<td>-</td>
<td>-</td>
<td>49,110</td>
</tr>
<tr>
<td>8</td>
<td>Facing with reinforced iron</td>
<td>-</td>
<td>2.31</td>
<td>-</td>
</tr>
<tr>
<td>9</td>
<td>Further embankment, including</td>
<td>-</td>
<td>167.4</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>clayey ground</td>
<td>-</td>
<td>48.4</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>gravel-pebble</td>
<td>-</td>
<td>119</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>Construction period</td>
<td>year</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>11</td>
<td>Expenditure for construction of water outlet</td>
<td>min. rouble</td>
<td>4.09</td>
<td>3.87</td>
</tr>
<tr>
<td>12</td>
<td>Loss of net income from the area of 5.8 thousand hectare</td>
<td>-</td>
<td>13.92</td>
<td>4.64</td>
</tr>
<tr>
<td>13</td>
<td>Full costs</td>
<td>min. rouble</td>
<td>18.01</td>
<td>8.51</td>
</tr>
<tr>
<td>14</td>
<td>Relative cost</td>
<td>unit</td>
<td>2.19</td>
<td>1</td>
</tr>
</tbody>
</table>

So, as you can see from the table, the construction cost of the mine spillway is almost the same as the redevelopment cost of the flood spillway by reaching its NPL to the 2nd stage rate.
However, the construction of the mine spillway in the body of the operating 1\textsuperscript{st} stage spillway is more complex than the self-dependent spillway. Large-scale wooden mold work is required during its construction.

Since during the construction period of the mine spillway, the 1\textsuperscript{st} stage spillway will not operate, the reservoir can be filled up to an absolute level of 60.0 m.

In theory, the construction period of a mine spillway is three years. However, this period may increase two-fold. Under such a circumstance, only a half of the irrigation area can be irrigated.

In this case, during the construction of the ditch spillway over the new line, the 1\textsuperscript{st} stage reservoir is not filled during the joining of the 1\textsuperscript{st} and 2\textsuperscript{nd} stage reservoirs, and a year is required for it.

Thus, the mine spillway construction alone costs 13.92 million manats; the redevelopment of the ditch spillway costs 8.51 million manats.

Considering the difference in costs, as well as the simplicity of the construction, the ditch spillway is preferred.

14. The dam

As can be seen from the scheme of the head plan of the structure developed in 1974, the construction of this structure for irrigation in the area of 32 hectares is a long-term and quite difficult measure.

Therefore, the structure is divided into two parts in the head plan scheme. The construction period of each one is 5-6 years.

The construction period of the 2\textsuperscript{nd} stage reservoir is considered real in this case so that the operation of the 1\textsuperscript{st} stage and the construction of the 2\textsuperscript{nd} stage are possible at the same time. The selected type and design of the dam allows for raising the 2\textsuperscript{nd} stage reservoir in the water level of the NPL rate of the 1\textsuperscript{st} stage reservoir.

The second factor in the selection of the dam design is climate condition of the construction region. Here, quite a lot of atmospheric precipitations fall for 6 months continuously. The dam design should allow for the pile of constructive elements separately in this part of the valley. Since, elements containing stone, sand and gravel can be piled throughout the year. In this case, the clayey screen can be founded in dry weather for 5-6 months in a year.

Upon the completion of the 1\textsuperscript{st} stage in the construction region, the situation with local building materials has become complicated.
Pure clay stones exist in small amount in separate irrigation areas. The situation with clay is better; its reserves provide enough clay materials to construct the screen.

The situation with local sand and gravel is a little difficult, because their reserves are limited and the demand for the region’s needs is a lot.

As for the stone materials, they usually consist of weak and crumbled rocks. They can be used after a series of conditions are met.

If the design of the screen or the foundation of the dam depends on the construction sequence or the climatic factor, their profile placements have been determined by the existence of local sediments and their physical and mechanical properties.

In the end, the key factor in the dam design solution is that the construction company implementing the current project has extensive experience in founding the dam with local building materials. However, this company does not have experience in building the individual elements of the dam from materials other than sediments.

All of these conditions were the basis for the selection of the dam design.

14.1 Consideration of the dam options and the selection of the main option at the 1st stage

During the projection of the 1st stage of the reservoir, the possibility of the airtightness of local clay caused enough doubts. For this reason, as well as in order to eliminate the possibility of creeping of clay, advance liming of the clay of the screen in its own way was recommended. Considering the implementation of this measure is both complex and expensive, another option of the dam was also considered. This option offers a dam with counter-filtration element and which contains non-sediment materials. This is in the form of the shallow screen containing hydraulic multi-layered asphalt concrete.

This option retains all the advantages of the sediment screen due to the construction sequence and climatic conditions. At the same time, it corresponds to the configuration of the transverse profile of the dam along its entire length. However, it has its own shortcomings.

These include:
- The installation of such a screen along the entire height of the dam is possible only after its raise is completed.

- The necessity of additional fencing up to the height of 54.0-55.0 m of the dam and the intensive removal of water from the riverbed during the installation of the screen.

- Relatively light asphalt-concrete screen does not allow the placement of a depression curve enough above the reservoir horizon. This case is possible when the reservoir operates very fast.

In addition to the asphalt-concrete screen, the following options were also recommended:

- A 0.7 m-thick core of bitumen mastic. It is raised in layers equal to the thickness of sand and gravel layer of the dam prism.

- A core in the form of a multi-rowed mixture of cement.

In this case, it should be noted that without closing the right bank and turning the dam in the 27th picket, the core option of the dam is only possible over a “straight line”. With this option, a part of the Gariblar village and Masalli-Yardimli highway remain under water. Of course, the core of the dam option is more compressed and requires less material for the raise.

However, the general technical and economic comparisons of the options should be done by taking into account the importance of covering and thickening of the right bank, the existence of the Gariblar village and its remaining under water and the need to carry out measures on the foundation of the dam.

Taking into account all these, four options of the dam were considered in the 1st stage construction.

**The 1st option.** This option has been accepted. A 1200 m-long, stone and soil dam with embedded sloping clay screen and concrete wall in the riverbed.

**The 2nd option.** This option does not differ from the 1st option in the plan. The counter-filtration element of the dam consists of 300 mm-thick three-layered screen made of hydraulic asphalt-concrete. This screen leans against the head of the excavation-concrete wall on the dam foundation. The wall is made up of 600 mm-diameter intersecting piles.
The body of the dam is made up of sand and gravel materials in the riverbed and local stones.

The 3rd option. This option was considered according to the proposal of the VNIIQ named after Vedeyenev. This option offers a dam containing sand and gravel sediments and with a slope covered with stones. In this case, the central core consists of bitumen mastic with a minimum thickness of 0.7 m and an average thickness of 1.15 m.

This option is only possible over the "straight line". Therefore, in order to protect a part of the Gariblar village from remaining under water, along with the installation of a drainage, an excavation-concrete wall should be installed on the foundation of the dam to the right bank.

Thickening of banks in the areas from the dam line to the Isi village are also taken into account in this option.

The dam is 3660 m long in this option.

The 4th option. This option was also given by the VNIIQ named after Vedenev and differs from the 3rd option by the structure of the dam’s core. The core is created in the central part of the sand and gravel elements of the body with the help of 4 and 3 rows of cement membranes. An excavation-concrete wall was adopted on the foundation along the entire length of the dam.

There is no experience in the republic in raising the dam options described in the 2nd, 3rd and 4th options. All of these options require large-scale work for the installation of the excavation-concrete wall on the dam foundation which is costly and labourous. In addition, as can be seen from experiences, the excavation-concrete wall installed at a depth of 50-60 m over the aligned line on the right bank does not guarantee the full covering of drilling joints between separate piles. A concrete-ditch wall is considered to be more reliable, but its installation depth is limited to only 20 m.

In the accepted 1st option, on the dam foundation along the entire length, the 90-82% of the wall depth does not exceed the depth of the installed wall which has a quality guarantee. This percentage decreases by up to 70-75% in the 2nd option and up to 30-35% in the 3rd and 4th options.
The 2\textsuperscript{nd} and 3\textsuperscript{rd} options require large amounts of imported materials and additional production for the preparation and installation of the asphalt-concrete and mastic on the spot. At the same time, the installation work is very complex in the bank join parts in both options.

In the 3\textsuperscript{rd} and 4\textsuperscript{th} options, the removal of a part of the Gariblar village would be required and a difficult situation would be created for the sanatory. In addition, it would require the far removal of the Masalli-Yardimli highway.

The construction cost and amount of work alone was calculated in 1969 during the 1\textsuperscript{st} stage projection.

The results of these calculations are given in Table 27.14 with 1984 prices.

<table>
<thead>
<tr>
<th>Name</th>
<th>Measure</th>
<th>Amount on options</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Excavation of foundation dam</td>
<td>thousand cubic metre</td>
<td>5174</td>
</tr>
<tr>
<td>Embankment of the dam</td>
<td>&quot;</td>
<td>18221</td>
</tr>
<tr>
<td>Construction cost</td>
<td>mln. rouble</td>
<td>88.04</td>
</tr>
<tr>
<td>Relative cost</td>
<td></td>
<td>1.00</td>
</tr>
</tbody>
</table>

As can be seen from the table, the 1\textsuperscript{st} option is the most economical, that is why it was recommended, approved for the 1\textsuperscript{st} stage and built.

14.2. The design of the adopted 1\textsuperscript{st} stage dam

The adopted and built option of the 1\textsuperscript{st} stage dam is a part of the profile with a sloping core embedded in the form of a screen. The length of the dam along its brow is 3154 m. The dam consists of four areas along its length.

The first area is from the bank joint to the flood spillway line in the 14+00 picket. This is the area where the dam foundation changes from the Vilash River bed sediments to the clayey layer 4 of the left bank terrace in the left bank joint of the dam. The dam height is reduced by 5-6 m between the 14+00 – 12+00 pickets in this area.

The second area is the riverbed of the dam and is between the 14+00 – 21+00 pickets. In this area, the 1\textsuperscript{st} stage of the dam has a maximum height of 38.3 m. Here, the dam foundation consists of the Vilash River bed sediments.
The third area is between the 21+00 – 27+00 pickets. The dam foundation in this area rises from a low ledge to the coastal terrace made up of the queueing sediments of layer 4a. The dam screen passes to sediments covering the right bank.

The last area begins from the 27+00 picket. Here, the dam line turns up along the right bank in the direction of the river flow and ends by entering the coastal promontory of the Isi village in the 40+00 picket.

The type of the dam has been adopted as homogenous along its entire length. However, depending on the landscape of the area, general shape of individual elements of the dam changes a little.

So, from the 8+46 picket to the 27+00 picket, the lower slope of the dam screen has been adopted with a gradient of 1:1.5, and the upper slope in this area with a gradient of 1:2.0.

Further down, the screen from the 27+00 picket to the end is in a gradient of 1: 2.0 in the lower slope and in a gradient of 1: 2.25 in the upper slope.

The thickness of the screen has been calculated in such a way that the pressure gradient in a less reliable cutting for the 2nd stage dam does not pass 3.2. The contact of the lower slope of the screen with sand and gravel of the prism along the entire length of the dam is formed with the help of the transition layer of sand and gravel sorted out in sizes of 0-80 mm. The thickness of the transition layer in the horizontal direction is 3.6 m.

Transition layer is not required on the upper slope of the screen.

The upper and lower prisms of the dam consist of the sand and gravel sediments of the Vilash River.

The upper slope is protected with 2.5 m-thick stones.

Along the lower slope, in accordance with height, 5 m-wide berms are planned every 15 m. The gradients of the slopes were adopted as 1:2.5 and 1:3.0.

The gradient of the upper slope was adopted as 1:2.5 till 57.00 m, then 1: 4.5 and above the height of 70.0 m as 1:4.0.
The width of the dam’s brow is 10 m.

14.3. The design of the 2\textsuperscript{nd} stage raise of the dam.

The raising of the dam is planned without changing the profile. As such, the profile of the dam screen is completely reserved. This, in turn, allows the use of the reservoir during the whole period of the 2\textsuperscript{nd} stage construction.

The dam’s brow is cut by 3.0 m and the gradient of the screen is reduced to 1: 0.5 above 77.00 m. The screen in the 2\textsuperscript{nd} stage dam’s brow is 4 m thick.

On the lower slope of the screen, a 0-80 mm-high transition layer containing sorted sand and gravel is installed.

The upper screen is covered with a riverbed layer of sand and gravel with a 1:2.75-gradient external slope. On its surface, 20 cm-thick reinforced concrete cladding is installed. This cladding is installed on a 30 cm-thick filter layer with sizes of 0-60 mm.

In the dam raise, upper prism makes up the main weight. This is built from local weak stones up to an absolute altitude of 64 m and sand-gravel mixture of the riverbed above this altitude. Stones made up of weak rocks are protected from destruction and erosion on the slope both from upper part and the loading part.

A 7.0 m-wide shelf is installed on the upper slope of the dam, at an absolute altitude of 77.0 m. This shelf acts as a support for the cover of the upper slope.

The gradient of the upper slope of the dam is 2.75 above 77.00 m.

The lower slope of the dam has a gradient of 1: 2.5. There are 5 m-wide berms at altitudes of 50.5 m, 66.0 m and 81.0 m.

The raising of the 2\textsuperscript{nd} stage dam is carried out by building the prism with layers by an altitude of 72.0 m of the lower prism, cutting the “cap” of the 1\textsuperscript{st} stage dam and foundry work along the entire profile of the dam including the screen of the 2\textsuperscript{nd} stage and the upper sand-gravel prism.
14.4. The attachment of the dam brow

The attachment of the dam brow has been determined in accordance with the SNIP 2.06.04-82 regulatory document.

To calculate the dam brow attachment up the water level with 1% insurance, the wind with 1% insurance was adopted as the computing wind. For the discharge of the maximum flood consumption as the control and at NPL of 93.3 m of the reservoir, wind with 10% insurance was adopted. The expulsion length of the waves is the most unfavourable in the direction of from west to east.

The advancement of waves towards the slope of the dam at 2.75 gradient covered with concrete was calculated with the following formula:

\[ H_{\text{run}} \cdot 1\% = kr \cdot kp \cdot ksp \cdot kr_{\text{run}} \cdot h_{1\%} \]

The wind speed was adopted to be 24 m/sec with 1% insurance and 10 m/sec with 10% insurance. The durations of the effect of the wind are 11.4 and 6.0 hours accordingly.

The wind expulsion height is calculated with this formula:

\[ \Delta h = 2 \times 10^{-6} \times VL1/gh \]

Here,

L1 – the length of the reservoir on its mirror equal to 4.5 km

h – the computational depth of the reservoir equal to 24.5 m

The calculated results of the dam brow are given in the table and they were calculated by using the following formula:

\[ d = h_{\text{run}} + \Delta h + a \]

h run and \( \Delta h \) are given above.

a – the reserve of the dam brow not less than 0.5 m.
Table 28.14

<table>
<thead>
<tr>
<th>Horizons</th>
<th>Mark of horizon</th>
<th>Height of design wave</th>
<th>Height of pileup</th>
<th>Height of... (invisible)</th>
<th>Invisible</th>
<th>Invisible</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m</td>
<td>m</td>
<td>m</td>
<td>m</td>
<td>18</td>
<td>invisible</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>Normal water level</td>
<td>93.00</td>
<td>0.82</td>
<td>2.02</td>
<td>0.02</td>
<td>3.00</td>
<td>0.91</td>
</tr>
<tr>
<td>Maximum water level</td>
<td>93.35</td>
<td>0.82</td>
<td>2.02</td>
<td>2.02</td>
<td>2.65</td>
<td>0.56</td>
</tr>
<tr>
<td>Maximum water level</td>
<td>93.90</td>
<td>0.67</td>
<td>1.46</td>
<td>0.01</td>
<td>2.10</td>
<td>0.63</td>
</tr>
</tbody>
</table>

14.5. Calculation of the dam stability

The dam stability for the most serious conditions was calculated for a number of transverse profiles of the dam. In this case, areas were selected at various constructive cuttings of the dam and in different soil conditions at the foundation.

The following indicators of the sediments of the dam and its foundation were adopted.

Table 29.14

<table>
<thead>
<tr>
<th>Construction</th>
<th>... (invisible) of soil</th>
<th>Soil name</th>
<th>Volume weight of soil</th>
<th>Calculation angle of interior ground</th>
<th>Calculation of tangents angle</th>
<th>Bond calculation t/sq.m</th>
<th>... (invisible) coefficient</th>
<th>Bed silt mg/sq.cm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>invisible</td>
<td>invisible</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
<td>8</td>
<td>9</td>
</tr>
<tr>
<td>Body of the dam</td>
<td>Stone</td>
<td>1.45</td>
<td>2.02</td>
<td>40</td>
<td>0.84</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Gravel-pebblestone</td>
<td>2.15</td>
<td>2.26</td>
<td>35</td>
<td>0.70</td>
<td>0</td>
<td>34000</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Clay</td>
<td>1.89</td>
<td>1.96</td>
<td>7</td>
<td>0.123</td>
<td>8.0</td>
<td></td>
</tr>
<tr>
<td>Bedding</td>
<td>4</td>
<td>Clay with interlayers</td>
<td>1.97</td>
<td>2.27</td>
<td>14</td>
<td>0.25</td>
<td>8.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Gravel-pebblestone</td>
<td>2.15</td>
<td>2.26</td>
<td>35</td>
<td>0.70</td>
<td>0</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>Gravel-pebblestone</td>
<td>2.15</td>
<td>2.26</td>
<td>32</td>
<td>0.625</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

The following calculation cases were adopted.

For the upper slope:

1. Taking into account the normal operation of the reservoir and the adaptation of both main and specific (seismic) loads.
2. The rapid discharge of the reservoir under the influence of main load.

For the lower slope:

1. Maximum filling of the reservoir up to the normal pressure level and the established level of the depression curve in the dam body, by taking into account the adaptation of the main and specific (seismic) loads.
2. Contamination of the lower part of the lower prism and the rise of the depression curve level.

The results of the calculated stabilities are given below in Tables 30.14 and 31.14.
### Table 30.14

<table>
<thead>
<tr>
<th>Invisible</th>
<th>Slope</th>
<th>Static forces assessment</th>
<th>Seismic forces assessment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>12+00</td>
<td>Downstream</td>
<td>1.72</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td>Up-river</td>
<td>1.64</td>
<td>1.27</td>
</tr>
<tr>
<td>21+50</td>
<td>Downstream</td>
<td>1.70</td>
<td>1.37</td>
</tr>
<tr>
<td></td>
<td>Up-river</td>
<td>1.80</td>
<td>1.22</td>
</tr>
</tbody>
</table>

#### Table 31.14

<table>
<thead>
<tr>
<th>Slope</th>
<th>Static forces assessment</th>
<th>Seismic forces assessment</th>
<th>Fast level drawdown</th>
<th>... (invisible) of filters</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Downstream</td>
<td>1.668</td>
<td>1.38</td>
<td>-</td>
<td>1.561</td>
</tr>
<tr>
<td>Up-river</td>
<td>1.039</td>
<td>1.249</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Downstream</td>
<td>1.668</td>
<td>1.38</td>
<td>1.441</td>
<td></td>
</tr>
<tr>
<td>Up-river</td>
<td>1.639</td>
<td>1.249</td>
<td>1.414</td>
<td>1.531</td>
</tr>
<tr>
<td>Downstream</td>
<td>1.551</td>
<td>1.112</td>
<td>-</td>
<td>1.396</td>
</tr>
<tr>
<td>Up-river</td>
<td>1.545</td>
<td>1.117</td>
<td>1.396</td>
<td>-</td>
</tr>
</tbody>
</table>

### 14.6. The base of the dam and bank joints.

Similar to the dam foundation, in the riverbed and right bank joint, there are the sand and gravel sediments of layers 2 and 5. Along the right bank joint, from the 27+00 picket to the end of the dam, the right bank of the Vilash River containing the sediments of layer 4 plays the role of support prism.

In the other bank joint, from the beginning of the dam to the 14+50 picket, the dam foundation consists of the sediments of layer 4. This layer contains sand and gravel lenses with clay and clay-stone fillers.

The dam screen consists of the ancient alluvial sediments of layer 5 along its entire length. This layer consists of sand and gravel sediments with clay and clay-stone fillers. Its filtration rate is much lower than the alluvial sediments of the modern layer.

The end of the right bank enters the Isi promontory towards the riverbed in the 40+41 picket.

### 14.7. Counter-filtration measures
The right bank sediments located higher from the river bottom was reliably isolated with the dam screen. In this case, modern alluvial sand and gravel sediments of layer 2 are cut with the screen prong.

However, since the sediments of layer 5 which reach 12-15 m in the riverbed and 25 m on the right bank are relatively thicker, they cannot be cut with the prong of the screen. At the same time, the sediments of layer 5 are not homogeneous both in depth and area. Since, even they are sand and gravel sediments, their fillers vary from small-grained sand to clay and clay-stones. This defines their permeability varying between a few centimeters and 6 m/day.

In this case, it is difficult to detect a pattern in the distribution of filtration rate over the depth. In some places, a high filtration rate is observed in great depths.

However, it is simpler to detect a pattern in the distribution of filtration rate over the areas. Thus, the highest permeability in layer 5 is in the riverbed between the pickets of 14+00 – 22+00. Here, the filtration rate of layer 5 varies between 0.9-6.0 m/day.

The filtration rate of layer 5 is quite low after the 22+00 picket and usually does not exceed 0.5 m/day.

The filtration rate on the left bank is even less and do not exceed 0.2 m/day.

Thus, the highest permeability in layer 5 is in the riverbed and here the pressure gradient has the highest rate.

On the right and left bank joints, where the filtration way is long and the filtration rate is lower, the pressure gradient does not exceed 0.15.

In this case, the pressure gradient in the riverbed is 0.8-1.0 for the 1st stage dam and 1.2-1.5 for the 2nd stage dam.

In this case, it should be taken into account that the filtration rate determined for depth for a 5-10 meter zone has a middle or slightly lower rate. This is because there may be sediment paddings with large and small filtration rates in the testing zone.

According to a number of calculations, filtration losses from the reservoir are almost low without any measures on the foundation. However, the high pressure gradient in the riverbed of the dam is very dangerous. In this case, the washout of the screen base and filtration concentrated from separate paddings of the sediment is possible.
Therefore, during the 1st stage dam projection, the creation of a deep prong was considered useful in the riverbed. This prong containing excavation-concrete wall is well-established in rooty clay rock soils. Its filtration rate is around 0.1 m/day and was adopted as a waterproof layer.

The prong-support is constructed in the length of 1200 m from the 12+50 picket to the 24+50 picket in the riverbed of the dam.

In all initial reports, the filtration rate of the wall was adopted as 0.1 m/day (specific water absorption is between 0.04-0.05). Such a high rate of filtration is because of the wall constructed in the shape of the intersection of 600 mm-long piles. All filtration reports were done on the basis of this filtration rate.

Later, however, concrete-ditch wall was included in the plan with 6-7 m long sections intersected. This has sharply reduced the number of seams in contact with the wall (12 times). In addition, the control signals of the wall show that its specific water absorption is between 0.02-0.001, so the filtration rate of the wall can be taken 005002 m/day.

Thus, the goal of the concrete-ditch wall is not only the 2.0-2.5 times reduction of the loss from the reservoir, but also and mainly the equalization of the filtration consumption at the dam foundation and the possible prevention of the soil erosion from thin layers.

The thickness of the wall was taken 600 mm. The wall enters the screen in its upper part and that is why it is in the form of a derivative screen at a height of 4-6 m.

14.8. Filling of the left bank ravine

On the left bank, from the tributary of the Matali River, a ravine starts at the reservoir cup, a little up the dam in the 14+00 picket. This ravine rises almost parallel to the dam towards the depths of the left bank.

The distance from the edge of the ravine to the axis of the 2nd stage dam is 170-270 m at an absolute altitude of 75-76 m. This distance is 80 m closer to the bank joint near the Matali River.

During the 1st stage construction, the ravine was filled with the sediments extracted from the construction pits of the flood spillway and the dam itself. In order to prevent these sediments from entering the cup when the reservoir is operating, the ravine was filled with the help of a special dike containing sand-gravel and stone materials. This dike cut the ravine parallel to the Matali River course. The gradient
of the lower slope of the dike is 1:4.5 and the upper slope filled with sediments is 1:135 m.

17. The discharge of flood waters

The construction of the tunnel spillway was not possible under the circumstance of the adopted valve of the dam. This is because, a large tunnel should be opened in the dam body, the construction of which is very complex and is mainly completed in two stages.

A relatively reasonable option is the construction of an automatic surface spillway. Spillways with such a design have been used in a number of reservoirs and have proven their reliabilities.

The most serious drawback of this spillway is the construction of its head facility at the 2nd stage.

The 1st stage spillway was constructed in the form of an open ditch, one-sidedly pouring the water into the intake ditch.

The estimated depth of the ditch is 1.5 m for discharging flood waters from the fully open parapet with estimated 0.1% insurance. In this case, 426 m³/sec of flood waters can be discharged.

The length of the water intake ditch of the 1st stage spillway is 150 m.

The width of the intake ditch varies between 10-16 m and joins the conduit. The ditch has the gradient of 0.02.

It is followed by a fast-flowing facility with the gradient of 0.995 and the length of 286.2 m. This part is then followed by a part with the small gradient of 0.025 and the length of 166.5 m.

Then a 41 m-long sloping grid-like extinguisher is located. The spillway is completed with a 71 m-long stilling basin.

The width of the part with the dimensions “41+71=112 m” has been carried out with 16 m at the beginning and 50 m at the end of the stilling basin. The depth of the basin is 4.5 m. Cushions containing 18 concrete hobs with the dimensions of 1.0x1.0x0.2 m are planned at the end of the basin.

The sloping grid-like extinguisher consists of a hob with square holes. This hob is strengthened with the help of a row of columns.
The water intake part of the spillway is in the form of a covered conduit. Water enters the conduit from all sides. The water intake part consists of 4 sections and the length of the water suspender reaches 242 running meters.

The width of the water intake conduit is 16 m, of the longitudinal water intake ditch is 4.0 m and of the transverse ditches are 5.0 m.

In the report of the ditch-like spillway, it was adopted that 90% of the possible 420 m³/sec of consumption will be discharged by the bottom water discharge facility. The remaining 422 m³/sec of consumption will be divided as follows: 290 m³/sec will be discharged by the flood spillway and 132 m³/sec will be transferred to the reservoir.

The water intake conduit is followed by ten 20 m-long conduit sections. These sections consist of the curved bends with a canvas of 200.5 m. The widths of the conduits decrease along their lengths from 16.0 m to 12.0 m in this part of the spillway. The longitudinal gradient of the area is 0.01.

The surface of the 6th section of the curved conduits is covered to make a contact with the passage between the bridge and the dam crest.

This is followed by the fast-flowing part of the conduit consisting of 15 sections (300 m) with each having the gradient of 0.045 and the length of 20 m. The 26 conduits (520 m) are located with the gradient of 0.01.

The underneath of the 1st stage conduit is rubbed and filled with clay and this in turn is combined with the dam screen.

The surface of the clayey soil is filled with sand and gravel.

19. Control-measurement devices

Piezometers and surface marks (rappers) are considered the main control-measurement devices of the dam. The construction of the depth marks (rappers) was not considered appropriate because the height of the dam is low.

The ground marks are installed on the dam crest and the lower coast of the slope.

To measure the level of groundwater within the boundaries of the dam, piezometers are planned to be installed in the following way. 7 piezometers are planned from the crest of the dam towards the lower slope. The same number of piezometers is planned in the middle of the lower slope of the dam and 4

Azerbaijan State Water Management Agency
Dam Safety Assessment Report - Vilesbchay Water Reservoir

August 2015
APPENDIX A
piezometers are planned in the riverbed of the dam. Thus, 18 piezometers are planned in the body of the dam.

In addition to these, to study the filtration and the groundwater on the banks, 9 piezometers are planned on the left bank and 8 piezometers are planned on the right bank.

A total of 35 piezometers are planned.

In addition, 2 support rappers are planned near the dam on both banks.

The water level in the piezometers will be measured by special level measuring devices.

The collapse of the dam will be determined as a result of the measurements across all marks (rappers).

21. Considerations on the construction of the 2nd stage of the dam junction

During the development of the "The technical project of the 1st stage", the two-stage construction of the reservoir was planned according to the head plan of the structure.

At the 1st stage, the capacity of 46.0 million m³ and the irrigation area of 11.2 thousand hectares should be achieved and at the 2nd stage, the irrigation area should reach 32.5 thousand hectares by achieving the capacity of 130.0 million m³.

This will be applied both to the main structure of the dam and the facilities of the hydro system.

The construction of the water discharge facility is planned at the 1st stage, since after the construction of the 1st stage, its reconstruction or amendments to it is not possible.

The expansion of the dam starting from the bottom is planned at the 2nd stage.

The upper part of the dam at an absolute altitude of 77.0 m will be strengthened in the form of a 9 m-wide shelf with monolithic reinforced concrete hobs. Reinforced concrete pillars are located in this shelf for strengthening the slope of the 2nd stage dam.

The construction of the dam will begin in the riverbed and continue towards the banks.
At the same time, the flood spillway of the 2\textsuperscript{nd} stage and the water discharge facility of the left bank channel will be constructed.

The main and the relatively hard work is the construction of the dam at the place of the head facility of the 1\textsuperscript{st} stage flood spillway and the joining of the 2\textsuperscript{nd} stage spillway to the 1\textsuperscript{st} stage spillway.

For this purpose, 50 m-wide passages from the axis of the spillway to all sides will be built.

Upon the completion of the construction in all parts of the dam, the filling of the passage in 12-15 months and its joining to the 1\textsuperscript{st} stage spillway will be started. For this, the water level in the reservoir should be at an absolute altitude of 67-68 m within one year.

The following measures must be implemented within 12-15 months:

- The installation of a concrete plug into the 1\textsuperscript{st} stage ditch
- The filling of the dam by pouring without the covering of the slope
- Dismantling of the five sections of the 1\textsuperscript{st} stage spillway
- Concreting over the joining part (section) of the 1\textsuperscript{st} and the 2\textsuperscript{nd} stages

The following works should be carried out for this:

- The filling of the dam body in the capacity of 260 thousand m\textsuperscript{3} in the passage
- Dismantling of the reinforced concrete structure of the 1\textsuperscript{st} stage conduit in the capacity of 3.2 thousand m\textsuperscript{3}
- The construction of the reinforced concrete conduit in the capacity of 3.0 million m\textsuperscript{3} at the joining place of the stages

In the end, the covering works of the passage slope will be completed.

The table below shows the amounts of work of the 2\textsuperscript{nd} stage construction of the reservoir.
<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>Measure</th>
<th>Amount</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dam embankment, including</td>
<td>thousand cubic metre</td>
<td>8782.98</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Stone</td>
<td>&quot;</td>
<td>1501.81</td>
<td>1.6 t/cubic metre</td>
</tr>
<tr>
<td>3</td>
<td>Gravelly pebblestones</td>
<td>&quot;</td>
<td>5695.50</td>
<td>2.06 ton/cubic metre</td>
</tr>
<tr>
<td>4</td>
<td>Membrane clay</td>
<td>&quot;</td>
<td>1335.87</td>
<td>1.601/cubic metre</td>
</tr>
<tr>
<td>5</td>
<td>Gravel filter</td>
<td>thousand cubic metre</td>
<td>349.60</td>
<td>1.901/cubic metre</td>
</tr>
<tr>
<td>6</td>
<td>Slope planning under concrete paving</td>
<td>thousand cubic metre</td>
<td>227.85</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Trench excavation under lining toe</td>
<td>thousand cubic metre</td>
<td>4.20</td>
<td>invisible</td>
</tr>
<tr>
<td>8</td>
<td>Backfilling with sealing</td>
<td>&quot;</td>
<td>1.40</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Reinforced concrete of section tooth 1.0x0.6m</td>
<td>&quot;</td>
<td>2.77</td>
<td>2.50 B-4, MP3-50</td>
</tr>
<tr>
<td>10</td>
<td>Reinforced concrete tiling</td>
<td>thousand cubic metre</td>
<td>42.77</td>
<td>250, B-8,</td>
</tr>
<tr>
<td></td>
<td>category A-I</td>
<td>t</td>
<td>121.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>category A-III</td>
<td>&quot;</td>
<td>43.52</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Reinforced concrete tiling</td>
<td>thousand cubic metre</td>
<td>3693.79</td>
<td>welded fabric mat</td>
</tr>
<tr>
<td></td>
<td>monolithic slabs 10x10x0.2</td>
<td>&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>Plain reinforcement:</td>
<td>cubic metre</td>
<td>328.00</td>
<td>fine asphaltic (invisible)</td>
</tr>
<tr>
<td></td>
<td>category A-I</td>
<td>&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>Coverage along the dam coping</td>
<td>thousand cubic metre</td>
<td>3.78</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1) base of gravel, thickness 15 cm</td>
<td>&quot;</td>
<td>1.51</td>
<td>with double surface treatment</td>
</tr>
<tr>
<td></td>
<td>2) black gravel, thickness 6 cm</td>
<td>&quot;</td>
<td>1.51</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>Curbstone</td>
<td>cubic metre</td>
<td>410</td>
<td>Concrete M-200</td>
</tr>
<tr>
<td>15</td>
<td>Concrete mattress</td>
<td>thousand cubic metre</td>
<td>0.78</td>
<td>Concrete M-200</td>
</tr>
<tr>
<td>16</td>
<td>Precast concrete blocks of canals-walkways</td>
<td>&quot;</td>
<td>1.84</td>
<td>Concrete M-200</td>
</tr>
<tr>
<td>17</td>
<td>Reinforcement:</td>
<td>&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>category A-I</td>
<td>t</td>
<td>165.1</td>
<td>carcass welding</td>
</tr>
<tr>
<td></td>
<td>category A-II</td>
<td>&quot;</td>
<td>116.7</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>Asphaltic concrete of walkways, thickness 20mm</td>
<td>so.m</td>
<td>6825</td>
<td>poured asphaltic concrete</td>
</tr>
<tr>
<td>19</td>
<td>Breast rail metal</td>
<td>t</td>
<td>162.7</td>
<td>welded</td>
</tr>
<tr>
<td>20</td>
<td>Painting of rails</td>
<td>so.m</td>
<td>1330</td>
<td>for two times</td>
</tr>
<tr>
<td>21</td>
<td>Ground excavation under pillar</td>
<td>cubic metre</td>
<td>80</td>
<td>in gravel-pebblestones</td>
</tr>
<tr>
<td>22</td>
<td>Backfilling</td>
<td>cubic metre</td>
<td>70</td>
<td>with compression</td>
</tr>
<tr>
<td>23</td>
<td>Installation of reinforced concrete pillars</td>
<td>piece/cubic metre</td>
<td>400/14.4</td>
<td>Concrete M-250</td>
</tr>
<tr>
<td>24</td>
<td>Water diversion ditch along the downstream toe of the dam</td>
<td>thousand cubic metre</td>
<td>3.50</td>
<td>Layers 4 and 2</td>
</tr>
<tr>
<td>25</td>
<td>Filling of discharge chute from (invisible)</td>
<td>&quot;</td>
<td>13.20</td>
<td>2.86 t/cubic metre</td>
</tr>
<tr>
<td>26</td>
<td>Controlling and measuring apparatus, including:</td>
<td>piece</td>
<td>70</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1) invisible</td>
<td>piece</td>
<td>35</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2) Surface marks</td>
<td></td>
<td>33</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3) supporting benchmark</td>
<td></td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>27</td>
<td>Ground excavation</td>
<td>thousand cubic metre</td>
<td>984.97</td>
<td>Layer</td>
</tr>
<tr>
<td>28</td>
<td>Backfilling</td>
<td>&quot;</td>
<td>339.96</td>
<td>with compression</td>
</tr>
<tr>
<td>29</td>
<td>Concrete mattress, thickness 10 cm</td>
<td>&quot;</td>
<td>1.76</td>
<td>Concrete M-100</td>
</tr>
<tr>
<td>30</td>
<td>Reinforced concrete of wall</td>
<td>&quot;</td>
<td>18.27</td>
<td>250, B-8, MIP-50</td>
</tr>
</tbody>
</table>

**Table 32.21**

Azerbaijan State Water Management Agency
Dam Safety Assessment Report - Vileschchay Water Reservoir
August 2015
APPENDIX A
<table>
<thead>
<tr>
<th>No.</th>
<th>Item Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>31</td>
<td>Reinforced concrete of wall</td>
<td>&quot;</td>
<td>30.77</td>
<td>250, B=8, Mp=50</td>
</tr>
<tr>
<td>32</td>
<td>Reinforced concrete of column</td>
<td>&quot;</td>
<td>0.07</td>
<td>250, B=8, Mp=50</td>
</tr>
<tr>
<td>33</td>
<td>Slope tiling</td>
<td>&quot;</td>
<td>2.83</td>
<td>250, B=8, Mp=50</td>
</tr>
<tr>
<td>34</td>
<td>Reinforcement of discharge chute:</td>
<td>&quot;</td>
<td>1471.3</td>
<td>reinforcing fabric and reinforcing cage</td>
</tr>
<tr>
<td></td>
<td>category A-I</td>
<td>&quot;</td>
<td>4413.8</td>
<td>reinforcing cage</td>
</tr>
<tr>
<td></td>
<td>category A-II</td>
<td>invisible</td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>Reinforcement of column</td>
<td>&quot;</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>category A-I</td>
<td>&quot;</td>
<td>6.00</td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>Lining reinforcing fabric</td>
<td>&quot;</td>
<td>313.7</td>
<td></td>
</tr>
<tr>
<td>37</td>
<td>Gravel mattress under reinforcement, thickness 30 cm</td>
<td>thousand cubic metre</td>
<td>3.80</td>
<td>Dmax≤60 mm</td>
</tr>
<tr>
<td>38</td>
<td>Devices of movement joint of chute sections</td>
<td>sq m</td>
<td>2386.3</td>
<td></td>
</tr>
<tr>
<td>39</td>
<td>Precast concrete blocks of bridge</td>
<td>cubic metre</td>
<td>42.80</td>
<td>Concrete M-800</td>
</tr>
<tr>
<td>40</td>
<td>Precast concrete blocks of walkways</td>
<td>cubic metre</td>
<td>2.69</td>
<td>Concrete M-200</td>
</tr>
<tr>
<td>41</td>
<td>Concrete of protective layer and drain triangle</td>
<td>cubic metre</td>
<td>9.5</td>
<td>Concrete M-200</td>
</tr>
<tr>
<td>42</td>
<td>Curbstone concrete</td>
<td>invisible</td>
<td>1.7</td>
<td>Concrete M-300</td>
</tr>
<tr>
<td>43</td>
<td>Railings concrete</td>
<td>&quot;</td>
<td>1.74</td>
<td></td>
</tr>
<tr>
<td>44</td>
<td>Joint concreting</td>
<td>&quot;</td>
<td>4.83</td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>Cement concrete lining</td>
<td>&quot;</td>
<td>9.4</td>
<td></td>
</tr>
<tr>
<td>46</td>
<td>Reinforcing steel</td>
<td>&quot;</td>
<td>1.85</td>
<td>reinforcing cage</td>
</tr>
<tr>
<td></td>
<td>category A-I</td>
<td>invisible</td>
<td>10.20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>category A-II</td>
<td>invisible</td>
<td>0.29</td>
<td></td>
</tr>
<tr>
<td>47</td>
<td>Rails metal</td>
<td>&quot;</td>
<td>1.33</td>
<td>Steel St. 3</td>
</tr>
<tr>
<td>48</td>
<td>Painting of rails</td>
<td>sq m</td>
<td>11.00</td>
<td>for 2 times</td>
</tr>
<tr>
<td>49</td>
<td>Rockfill embankment</td>
<td>cubic metre</td>
<td>33</td>
<td>Fineness</td>
</tr>
<tr>
<td>50</td>
<td>Ground excavation for drainage making</td>
<td>thousand cubic metre</td>
<td>2.20</td>
<td>Layer 4</td>
</tr>
<tr>
<td>51</td>
<td>Backfilling</td>
<td>thousand cubic metre</td>
<td>2.100</td>
<td></td>
</tr>
<tr>
<td>52</td>
<td>Surface planning</td>
<td>thousand sq.m</td>
<td>2.60</td>
<td></td>
</tr>
<tr>
<td>53</td>
<td>Wells of precast concrete</td>
<td>cubic metre</td>
<td>2.6</td>
<td>B=200</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&quot;</td>
<td></td>
<td>Mp=50</td>
</tr>
<tr>
<td>54</td>
<td>Wellhead structure of precast concrete</td>
<td>&quot;</td>
<td>0.41</td>
<td></td>
</tr>
<tr>
<td>55</td>
<td>Drain tile, 200 mm</td>
<td>piece</td>
<td>2415</td>
<td></td>
</tr>
<tr>
<td>56</td>
<td>Polyethylene clutch</td>
<td>invisible</td>
<td>2415</td>
<td></td>
</tr>
<tr>
<td>57</td>
<td>Asbestos-cement pipe</td>
<td>invisible m/piece</td>
<td>196/49</td>
<td></td>
</tr>
<tr>
<td></td>
<td>360 mm, 4cm</td>
<td>invisible m/piece</td>
<td>196/49</td>
<td></td>
</tr>
<tr>
<td>58</td>
<td>Mattress of gravel-pebble soil</td>
<td>cubic metre</td>
<td>79</td>
<td></td>
</tr>
<tr>
<td>59</td>
<td>Invisible</td>
<td>&quot;</td>
<td>610</td>
<td>Dmax≤60 mm</td>
</tr>
<tr>
<td>60</td>
<td>Dismantling of reinforced concrete chute for I stage</td>
<td>thousand cubic metre</td>
<td>3.22</td>
<td>Reinforced concrete M=250</td>
</tr>
<tr>
<td>61</td>
<td>Demountable chute reinforcement</td>
<td>t</td>
<td>233.4</td>
<td></td>
</tr>
<tr>
<td>62</td>
<td>Auto road surface, including:</td>
<td>thousand cubic metre</td>
<td>14.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1) Cushion course 15 cm</td>
<td>cubic metre</td>
<td>2113</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2) Gravel bed, 15 cm</td>
<td>&quot;</td>
<td>2115</td>
<td>optimum mixture</td>
</tr>
<tr>
<td></td>
<td>3) black gravel, thickness 8 cm, with double surface treatment</td>
<td>&quot;</td>
<td>846</td>
<td></td>
</tr>
<tr>
<td>63</td>
<td>Device for ditch excavation along road</td>
<td>thousand cubic metre</td>
<td>1.67</td>
<td>Layer 4</td>
</tr>
<tr>
<td>64</td>
<td>Surface planning</td>
<td>thousand sq.m</td>
<td>5.75</td>
<td></td>
</tr>
<tr>
<td>65</td>
<td>Concrete lining of ditch, thickness 15 cm</td>
<td>cubic metre</td>
<td>882.5</td>
<td>Concrete M-150</td>
</tr>
<tr>
<td>66</td>
<td>Excavation under pillar</td>
<td>cubic metre</td>
<td>50</td>
<td>Layer 4</td>
</tr>
<tr>
<td>67</td>
<td>Backfilling</td>
<td>invisible</td>
<td>45</td>
<td>with compression</td>
</tr>
<tr>
<td>68</td>
<td>Installation of typical reinforced concrete pillars</td>
<td>piece/cubic metre</td>
<td>250/8.96</td>
<td>Concrete M-250</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&quot;</td>
<td>&quot;</td>
<td></td>
</tr>
<tr>
<td>II.</td>
<td>DISCHARGE INTO LABORATORY CANAL</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>69</td>
<td>Foundation pit excavation</td>
<td>thousand cubic metre</td>
<td>0.52</td>
<td>Layer 2</td>
</tr>
<tr>
<td>70</td>
<td>Backfilling</td>
<td>&quot;</td>
<td>0.15</td>
<td>with compression</td>
</tr>
<tr>
<td>71</td>
<td>Concrete mattress, thickness 10 cm</td>
<td>cubic metre</td>
<td>120</td>
<td>Concrete M-200</td>
</tr>
<tr>
<td>72</td>
<td>Reinforced-concrete slab</td>
<td>cubic metre</td>
<td>68</td>
<td>250, B=8, Mp=50</td>
</tr>
<tr>
<td>73</td>
<td>Reinforced-concrete of coulumn</td>
<td>&quot;</td>
<td>54</td>
<td></td>
</tr>
<tr>
<td>74</td>
<td>Reinforced-concrete of the bed</td>
<td>&quot;</td>
<td>66</td>
<td></td>
</tr>
<tr>
<td>75</td>
<td>Reinforced concrete of ... (invisible)</td>
<td>&quot;</td>
<td>137</td>
<td></td>
</tr>
</tbody>
</table>
22. Considerations on the operation of the hydro system

The operation of the dam system facilities should be started with the initial filling of the reservoir. The operation with the initial filling is a very important stage, because only at this stage the errors occurred during the dam construction can be seen.

Therefore, specific instructions on how to fill the reservoir were prepared and given to the organized operation service.

The main principle here is to fill the reservoir up to the allowable level at this stage and to control the filling speed. The filling speed should not exceed 50 cm/day.

After the beginning of the 2nd stage construction, the main stage of the operation will start, because the main issue of the operation is its conformity with the construction.

The main issue at all stages of the operation is the timely detection of any defects and their removal as soon as possible.

The following observations should be carried out in accordance with the operation stages of the reservoir:

A. During the filling period of the 1st or the 2nd stages:

<table>
<thead>
<tr>
<th>No.</th>
<th>Material/Component</th>
<th>Unit</th>
<th>Amount 1</th>
<th>Amount 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>76</td>
<td>Reinforced-concrete of damming and consoles</td>
<td></td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>77</td>
<td>Reinforced-concrete of water measuring well</td>
<td></td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>78</td>
<td>Reinforcing steel category A-I</td>
<td>t</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>79</td>
<td>Reinforcing steel category A-II</td>
<td>t</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>Facing metal, thickness 6 mm</td>
<td>t</td>
<td>1.55</td>
<td>Steel st. 3</td>
</tr>
<tr>
<td>81</td>
<td>Railing metal</td>
<td>invisible</td>
<td>1.94</td>
<td></td>
</tr>
<tr>
<td>82</td>
<td>Lining metal, thickness 6 mm</td>
<td>cubic metre</td>
<td>72.1</td>
<td></td>
</tr>
<tr>
<td>83</td>
<td>Painting of rails</td>
<td>sq.m</td>
<td>16</td>
<td>for 2 times</td>
</tr>
<tr>
<td>84</td>
<td>Vertical drainage holes, depth 15 m</td>
<td>piece</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>85</td>
<td>Cutting slopes with planning until dam slope 1:2</td>
<td>thousand cubic metre</td>
<td>18.50</td>
<td>Layer 4</td>
</tr>
<tr>
<td>86</td>
<td>Embankment device with compaction</td>
<td></td>
<td>18.50</td>
<td></td>
</tr>
<tr>
<td>87</td>
<td>Slopes planning</td>
<td>thousand cubic metre</td>
<td>7.50</td>
<td>m²</td>
</tr>
<tr>
<td>88</td>
<td>Excavation of cutoff trench</td>
<td>thousand cubic metre</td>
<td>1.60</td>
<td>Layer 4</td>
</tr>
<tr>
<td>89</td>
<td>Backfilling with compression</td>
<td></td>
<td>1.50</td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>Gravel cushion, thickness 20 cm</td>
<td></td>
<td>1.50</td>
<td>Dmax ≤ 60 mm</td>
</tr>
<tr>
<td>91</td>
<td>Concrete of slope lining, 15 cm</td>
<td></td>
<td>1.13</td>
<td>200, B-4</td>
</tr>
<tr>
<td>92</td>
<td>Reinforcement category A-I</td>
<td>t</td>
<td>31.4</td>
<td>welded fabric mat</td>
</tr>
<tr>
<td>93</td>
<td>Reinforced-concrete slab</td>
<td>cubic metre</td>
<td>43</td>
<td>Concrete M-200</td>
</tr>
<tr>
<td>94</td>
<td>Reinforcement category A-I</td>
<td>t</td>
<td>4.5</td>
<td>welded carcass</td>
</tr>
<tr>
<td>95</td>
<td>Steel railing</td>
<td>invisible</td>
<td>15.0</td>
<td>Steel st. 3</td>
</tr>
<tr>
<td>96</td>
<td>Painting of rails</td>
<td>sq.m</td>
<td>16</td>
<td>for 2 times</td>
</tr>
</tbody>
</table>
1. To avoid the speed and the level of the filling of the reservoir to exceed the specified amounts in the instructions
2. To examine the condition of the dam and the banks, to detect the filtration places and to determine the turbidity and the consumption of water there
3. To determine the possibility of the landslide on the reservoir shores

B. During the construction period of the 2\textsuperscript{nd} stage:

1. To comply with the conditions of the demolition of the old part of the dam and the joining of the old and the new parts
2. To strictly follow the condition on the level of water in the reservoir according to the construction conditions based on the specific instructions on the reservoir operation
3. To strictly comply with the sequence and timing of the separate construction works in accordance with the project documents

C. During the clean operation period after the completion of the reservoir:

1. To examine the filling of the dead capacity of the reservoir, the erosion of the shores, the collapse of the shores and the landslide. The operation service should not allow the pollution of the reservoir with industrial and domestic wastes, the growth of the plants, as well as the creation of the small ponds in the tail section of the reservoir.
2. To control the water level in the dam body, its collapse and the deformation of the banks by means of control-measurement devices
3. To observe the emergence of springs on the shores and lower slopes of the dam, their consumption and turbidity
4. To observe the cleanliness of the flood spillway and its always being ready to accept water
5. To observe in detail all the elements of the water discharge facility from the head tower to conduits in the channels

All the equipments, measuring devices, the power grid, the electrical appliances, etc. should be regularly examined and adjusted if needed.

23. Conclusion and Summaries

As a result of the conducted technical and economic studies and the examination of the reservoir options based on three valves, it was determined that a relatively affordable option is to raise the height of the 1\textsuperscript{st} stage dam.

None of the other considered options are economically viable.
Thus, no change has been done in the project developed in 1974.

The only serious issue is the provision of the construction with sand and gravel materials, since the construction zone lacks sufficient reserves of these building materials. Therefore, the use of the sophisticated Arkivan field has been planned.

No significant changes have been planned in the head plan project of the 2nd stage construction. Taking into account the previous observations on the Vilash River, the reservoir will provide 32.5 hectares of land covering the entire territory of the Masalli administrative region with irrigation water.

As a result of the study of all the information and materials, the construction of the 2nd stage of the Vilash River reservoir is recommended.
APPENDIX B

Visual Surveillance Survey Results and Recommendations for VWR

Summary of the Dam Safety Condition of Dam
APPENDIX B1

Previous Visual Surveillance Survey Results and Recommendations for VWR

Summary of the Dam Safety Condition of Dam
Summary of the Dam Safety Condition of Dam

Surface Water Diversion (SWD) from the Vileshchay Water Reservoir (VWR) for providing the drinking raw water requirements of Masalli and Jalalabad city has been selected by the Feasibility study team in consultation with the AWM OJSC and relevant Public Utility Departments of the rayon.

The total raw water to be diverted from VWR for two cities (Masalli and Jalilabad) is as follows;

<table>
<thead>
<tr>
<th>Total raw water to be diverted from VWR for two cities</th>
<th>l/s</th>
<th>cum/day</th>
<th>cum/year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw water requirement for Jalilabad city</td>
<td>148.38</td>
<td>12.820.03</td>
<td>4.679.311.68</td>
</tr>
<tr>
<td>Raw water requirement for Masalli city</td>
<td>98.78</td>
<td>8.534.59</td>
<td>3.115.126.08</td>
</tr>
<tr>
<td>Total</td>
<td>247.16</td>
<td>21.354.62</td>
<td>7.794.437.76</td>
</tr>
</tbody>
</table>

Objective of the Study

The objectives of this assignment are to;

- Inspect and assess dam of VWR and its appurtenant structures with respect of primarily safety and other required technical standards are improved in accordance with WB OP / BP 4.37.

- Assess and conform that there would be no risk or negligible risk of significant adverse impacts due to potential failure of the structure (VWR) to local communities and assets, including assets to be financed as part of this project when VWR involves in the project.

- Ensure that the dam is inspected and maintained regularly and satisfactorily.

- Develop a dam safety assessment report including any safety concerns and recommendation of remedial measures.

Vileshchay River

The Vileshchay river travels along the entire Yardimli and Masalli rayon areas and flows directly to Caspian Sea within the Kızılaagach bay. The river is sourced from the Guludash peak (2200 masl) of Talish range nearby the Iran border at the western part of the rayon. Its length is 115 km, the basin area 935 square km. It runs through Peshteser range in the upper stream, Burovar range in the middle stream, and flows through the Lankaran Plain in the final 25 km. Its main branches are the Sheratuk River in the right, the Meteli River in the left. Its area of upstream collection basin is 9.35 square km.

The Vileshchay river is fed mostly by storm water (70%), partially subsurface (20%), and snow (10%) at its upstreams. The average annual water flow is 5,47 m3/sec. Its 45% passes in the spring, 6% in the summer, 25% in the autumn, and 24% in the winter. However, at its low streams at the Masalli section,
surface run-off from the atmospheric precipitation (73 %) and sub-surface lateral inflows (27 %) play the main role in its flow volume.

The vileshchay flow is routinely measured on two Hydrometeorological stations, which are Yardimli and Sixlar in Masalli. Yardimli Hydrometeorological station is being operated since 1936, and Sixlar since 1930.

There is no water reservoir on its upstreams for flood control, irrigation purposes, or water supply. A water reservoir is constructed for irrigation purpose of the farmlands of Masalli rayon on the course of the river at the upper levels of Masalli city in 1986.

**Vileshchay Water Reservoir (VWR) and Earth Embankment Dam**

The VWR was planned to retain water both for irrigation scheme of Masalli farmlands and Drinkable Water Supply to Jalilabad and Masalli city in early 1980s. The construction of the first phase of VWR has been completed in 1986. At a later stage, it is intended that the dam height will be increased by 15 m to raise the storage capacity to 130 million cum. The reservoir has a storage capacity of 46 million cum, a dam body height of 36.5 m, and a crest length of 1.4 km.

The irrigation scheme of Masalli is developed over the course of several years to an approximately 9,892 hectares irrigated land in 2013.

The water levels in the reservoir are measured twice a day routinely with a sequence of five observation posts located in the reservoir.
Assessment

The upstream face of the dam body is covered by rock fill and downstream face by earth fill. The embankment is a zoned rockfill and transition zone with a permeable layer in upstream, central clay core (impermeable) layer, a drainage blanket layer and transition zone with a permeable layer and earth fill at downstream side. Central clay core layet is slightly inclined toward upstream.

The downstream slope surface of the dam is dry, and no seepage is observed.

The water level within the reservoir is relatively secured with a flood freeboard of 3 meters.

The concrete surfaces of tunnels and gate room have been inspected together with the technical staff of VWR administration on June 09 – 10, 2014 during the field inspection. No pattern crazing and cracking, leaching, frost action, abrasion, spalling, and crumbling and structural cracking are observed.

Concrete surfaces and elements inspected in gate room, tunnels, and outlet structure are in adequate and not problematic condition. However, concrete surfaces in tunnel for derivation pipes and structure require small touch ups and cleaning.

The walls and slab of the spillway requires adequately repair, groutings, and rehabilitation after cleaning the vegetation on the walls and slab. However it can be assessed that the spillway structure has enough stability safety, and in no problematic condition.

Rock fill at the upstream face of dam body is in good condition, and there is no reason for any concern.

There are no signs of change in soil characteristics or signs of soil movements on and around the soils of the dam.

No deterioration from electrolysis, metal fatigue, stress-accelerated corrosion, tearing and rupture is detected on the metal dam components during the surveys and inspection, except a little corrosion on the derivation pipes and metal components in the gate house. They require cleaning and painting.

However, all metal components are in adequate condition, and have enough stability safety, and in no problematic condition as there are no signs of deterioration on them.

Seepage water is observed only at the outlets of the pipes coming from upstream toe of the dam body. According to the records of the VWR Administration, seepage water coming from toe drain is about 2 l/sec for each of the pipe outlet, and the water is generally clean. The seepage flow from upstream to drain is normal for such dam.

There are no problems detected with stability are evidenced in concrete and steel structures.

Technical Analyses Required

The technical / engineering analyses (geotechnical, structural or hydro-technical) will include;

(i) conducting stability analysis and resulting factors of safety for normal, unusual and extreme loading conditions, using internationally acceptable method for the dam embankment and other associated structures, and if required reservoir rim slopes.
(ii) To secure adequate hydrological safety undertaking assessment of flood hydrology to check the flood discharge volume for 100 years and 1000 years return periods, including frequency analysis with additional data during 28 years of operation.

(iii) The inflow volume should be calculated based on combined discharge volume and reservoir volume change. Then, frequency analysis of the reservoir inflow should be conducted.

(iv) Checking the effectiveness of foundation treatment and seepage control measures, to support conclusions and recommendations of the assessments.

(v) Conduct the slope stability analyses using an internationally accepted method and software in accordance with DIN 4084 for general stability and EC-7 for safety factors of normal, unusual, and extreme loading conditions, considering of horizontal and vertical seismic acceleration, and analysis of infiltration systems for a variety of frequencies (recurrence intervals).

(vi) The discharge capacity of the side spillway has been checked as 566 m³/s, but, the peak volume of 1000 year flood should be established checking the maximum water level at the spillway.

(vii) Check the rating curve of the spillway and the surcharge water level in the case of 100 year flood first, and then 1,000 year flood based on the above frequency analysis.

(viii) Check the required freeboard based on the anticipated wind speed and wave run up during large storms and then compared with the actual freeboard, i.e. embankment crest elevation minus surcharge water level as aforementioned.

(ix) Check the rating curve of the bottom outlet.

(x) Check the bottom outlet capacity (380 m³/s) in a similar manner to the spillway. The frequency analysis should be made for the total inflow volume of the dam.

**Key Data of the Earth Embankment Dam and Reservoir**

The key data of the earth embankment dam body and reservoir are as follows;

**A. RESERVOIR**

The key data of the reservoir is given in the following Table.

<table>
<thead>
<tr>
<th>Table 4: Hydrological &amp; Hydraulic Data on Reservoir</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Water Storage Capacity</td>
</tr>
<tr>
<td>Dead storage Volume</td>
</tr>
<tr>
<td>Spillway Capacity</td>
</tr>
<tr>
<td>Bottom outlet gate</td>
</tr>
<tr>
<td>Derivation Tunnels – 2 Nos (for irrigation)</td>
</tr>
<tr>
<td>Total Discharge Capacity</td>
</tr>
<tr>
<td>--------------------------</td>
</tr>
<tr>
<td>A 100 year discharge capacity</td>
</tr>
<tr>
<td>Planned Sedimentation Volume</td>
</tr>
<tr>
<td>Actual Sedimentation</td>
</tr>
<tr>
<td>Max. Controllable Water Level</td>
</tr>
<tr>
<td>Normal Water Level</td>
</tr>
<tr>
<td>Lowest Operational Water Level</td>
</tr>
<tr>
<td>Flood Freeboard</td>
</tr>
<tr>
<td>Hyrometeorological posts</td>
</tr>
<tr>
<td>Frequencies of water intake measurements</td>
</tr>
<tr>
<td>Frequency of measurement of water level in overflowing condition</td>
</tr>
<tr>
<td>Frequency of Water Quality Analysis</td>
</tr>
</tbody>
</table>

According to VWR administration, irrigation time span a year is 4 – 5 months between May – August (September).

A simple gravity system is used because the reservoir level is higher than the level of the fields in the irrigation scheme. Simply, the operational criteria of the scheme is in the manner of continuous flow of water for crop production during the irrigation season.

In addition, additional data obtained from Institute of Azdovsuteslayihe in January 29, 2014 regarding the original design of the VWR is given below;

- Average annual flow of Vilashchay $181.35$ million m$^3$
- Minimum average annual flow of Vilashchay $52.07$ million m$^3$
- Maximum average annual flow of Vilashchay $362.86$ million m$^3$
- 75% probability of Vilashchay $135.56$ million m$^3$
- 85% probability of Vilashchay $109.85$ million m$^3$
- 95% probability of Vilashchay $69.46$ million m$^3$
- Maximum flood discharge of 0.1% probability (for 1000 years) of Vilashchay $842.0$ m$^3$/sec

Water level observation posts are located in reservoir with a sequence of the following five levels;
VWR Administration reports that water level observations are conducted twice a day routinely in the reservoir.

B. EARTH EMBANKMENT DAM BODY

No berm exists at the upstream face of the dam body.

The key data about the earth embankment dam is given in the following Table.

**Table 8: Key Data on Earth Embankment Dam Body**

<table>
<thead>
<tr>
<th>Dam Body</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Construction</strong></td>
<td>1980 - 1986</td>
</tr>
<tr>
<td><strong>Type</strong></td>
<td>Zoned Earth Embankment Dam Body; upstream face is covered by rock and lowstream face is covered by earth fill</td>
</tr>
<tr>
<td><strong>Impermeable Layer</strong></td>
<td>Clay Core</td>
</tr>
<tr>
<td><strong>Height</strong></td>
<td>36.5 m</td>
</tr>
<tr>
<td><strong>Embankment Length</strong></td>
<td>1,400 m</td>
</tr>
<tr>
<td><strong>Crest Width</strong></td>
<td>10 m</td>
</tr>
<tr>
<td><strong>Crest Level</strong></td>
<td>80 masl *</td>
</tr>
<tr>
<td><strong>Freeboard</strong></td>
<td>1.5 -2 m</td>
</tr>
<tr>
<td><strong>Upstream Toe Level</strong></td>
<td>46.15 masl *</td>
</tr>
<tr>
<td><strong>Hydraulic Height</strong></td>
<td>35.45 m</td>
</tr>
<tr>
<td><strong>Structural Height</strong></td>
<td>38.45 m</td>
</tr>
<tr>
<td><strong>Slope – upstream face</strong></td>
<td>Vertical 2: Horizontal 3</td>
</tr>
<tr>
<td><strong>Slope – downstream face</strong></td>
<td>Vertical 1: Horizontal 2</td>
</tr>
<tr>
<td><strong>Top width of the clay core zone</strong></td>
<td>7.00 m</td>
</tr>
</tbody>
</table>
| Seepage Control Features | Downstream Chimney drains; made with permeable fills  
|                          | Downstream Drainage Blanket; made with permeable materials  
|                          | Upstream Toe; Piping from upstream Toe to river flow channel  
|                          | Downstream with two pipes; each of them 2 lt/sec.  
| Derivation Tunnels       | 2 Nos, Metal Pipes;  
|                          | Diameter: 1.22 m  
|                          | Length: Each of them is 300 m  
| Bottom Gates             | 4 Nos; 2 are stand-by; Each of them (4x4) m Sluice Gates; Weight of one gate: 12 tn  
|                          | Operation; Normal Operation : Electrical / hydro-mechanical drive System  
|                          | Emergency Operation: by Crane; operation by remote control and upstairs panel  
|                          | Release Capacity: Each Gate: 200 m³ / sec  
| Bottom Tunnel            | 2 Nos; Concrete (4.50x5.00) m.  
|                          | Length: Each of them: 297 m  

Figure 4: Details of Dam Body and Associated Structures
The spillway of the system is concrete chute (open channel). The spillway is not on embankment, and has the following features:

- The spillway consists of entrance channel in length of 200 m at its upstream end, and energy dissipator at the end of downstream channel.
- The entrance channel is excavated open channel, through fairly slope (3%) and placed along left abutment. Spillway consists of simplicity of design. Surplus water is conveyed from the reservoir to the river below the dam through an open channel.

The spillway is not on embankment, and has the following features:

- The spillway consists of entrance channel in length of 200 m at its upstream end, and energy dissipator at the end of downstream channel.
- The entrance channel is excavated open channel, through fairly slope (3%) and placed along left abutment. Spillway consists of simplicity of design. Surplus water is conveyed from the reservoir to the river below the dam through an open channel.
Key data on spillway structure is given in the following Table and Figures.

Table 9: Key Data on Spillway

<table>
<thead>
<tr>
<th>Component</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Chute Spillway with reinforced concrete open channel</td>
</tr>
<tr>
<td>Channel Width</td>
<td>12 m</td>
</tr>
<tr>
<td>Depth (average)</td>
<td>4 m</td>
</tr>
<tr>
<td>Total Channel Length</td>
<td>1400 m</td>
</tr>
<tr>
<td>Entrance Channel</td>
<td>200 m</td>
</tr>
<tr>
<td>Downstream Channel</td>
<td>1000 m</td>
</tr>
<tr>
<td>Energy Dissipator</td>
<td>200 m</td>
</tr>
<tr>
<td>Slope of downstream chute</td>
<td>approx. 3%</td>
</tr>
<tr>
<td>Stilling Basin from dissipator outlet to river flow channel</td>
<td>Pond in (500x50) m</td>
</tr>
<tr>
<td>Crest of Spillway</td>
<td>77 masl</td>
</tr>
<tr>
<td>Spillway Toe level</td>
<td>39.70</td>
</tr>
</tbody>
</table>

3.2.3 Energy Dissipation Facilities and Stilling Basin

There are two emergency energy dissipation facility within the system, one of which is located at the downstream end of the chute spillway South of the Yeyenkend village (see Figures below), other one is located at the end of bottom outlet of the reservoir.

The latter facility for bottom outlet begins with a throat adjacent to headwater building, and has a concrete stepped dissipator section and an outlet channel conned toward stilling basin.

Spillway’s dissipation facility has also a throat, a concrete stepped dissipator section, and an outlet channel coned toward its small pond before joining stilling basin.

Stilling basin serves jointly for this outlet and spillway outlet, and has approximately 20 hectares area between downstream of dam, Yeyenkend village, and Allahyarli village. Relief of the stilling basin varies between 39.50 masl in front of the bottom outlet dissipation facility, 29.50 masl at the end of spillway’s dissipation facility. Southern part of Yeyenkend village is located at about 44 masl nearby the stilling basin. Allahyarli village is also located 45 – 50 masl along the river basin downstream of stilling basin.
In addition, there is sufficient distance to Masalli city with a large river basin which serves for stilling of large spillway discharge. This area provides an approximately 1.7 - 2 sq. km flat sloped river basin (3.8x0.55 km), which is sufficient to attenuate the flood effects of large discharge.

**Energy Dissipation Facilities for Bottom Outlet**

**Figure 5: Details of Bottom Outlet Dissipation Facility**

**Spillway - Energy Dissipation and Stilling Basin**

**Figure 6: Details of Spillway Dissipation Facility**
APPENDIX B2

Visual Surveillance Survey Results and Recommendations for VWR
Visual Surveillance Survey Results and Recommendations for VWR

1. Monitoring & Instrumentation

Topographic Surveys

VWR administration reports that topographic surveys of the dam surface are conducted twice a year.

Piezometer Wells

Piezometer wells are not constructed within the first phase of the construction of the Dam. Reportedly, they will be constructed within the second phase of the construction. However, no certain program is developed by the AWM OJSC for the second phase of construction of the dam yet.

Seepage Control Features

There are two upstream toe drains with piping to downstream of the dam, and chimney drain and drainage blanket at the downstream face of the dam body. Chimney drain and drainage blanket is made with permeable material fill. Chimney drain is 2 m in width horizontally and 17 m vertically, and drainage blanket is 1 m in depth and 105 m in length toward downstream toe of the dam body.

According to report of the VWR Administration the seepage flow coming from the pipe outlets is measured weekly.

Seismic Analysis

According to VWR administration, no seismic analyses are conducted within the last 20 years of operation. However, design data of the dam shows that dam body and associated structures are planned to stand by 10 richter scale earthquake activity, which is the extreme earthquake event to consider for the region nearby the northeast Talish fault.

Landslide Analysis

VWR administration reports that no landslide analysis is conducted within the the last 20 years of operation. However, no signs of slide either on and around the dam body and reservoir coasts are detected during the field surveys and site inspections.
2. Assessments on Earth Embankment Dam Body, Reservoir and Appurtenance Structures

2.1 General

The dam is rather steep on downstream face, and its slope is 1:2 (V.H). The surface is protected by a 4 m deep earth layer, and the face is in adequate conditions. No indications of major settlements or slope instabilities are observed. The surface is dry, and no seepage is observed.

The upstream face of the dam body is covered with rock fill in 2 m depth, and is in good condition. its slope is 2:3 (V.H). No indication for dam settlements or instabilities is observed. This face gives no reason for concerns.

The water level within the reservoir is relatively secured with a flood freeboard of 3 meters.

2.2 Changes in Constituent Materials

In general, a look for evidence of defective, inferior, unsuited, or deteriorated materials is conducted during the field surveys and inspections.

The condition of individual components of the dam with respect of quality and durability is determined in order to assess the condition of the structure as a whole.

Concrete Surfaces

*Bottom outlet, tunnel for derivation pipes and bottom tunnels;*

The concrete surfaces of tunnels and gate room have been inspected together with the technical staff of VWR administration on June 09 – 10, 2014 during the field inspection. No pattern crazing and cracking, leaching, frost action, abrasion, spalling, and crumbling and structural cracking are observed.

The structure is massive, and no stability problem is reported.

Concrete surfaces and elements inspected in gate room, tunnels, and outlet structure are in adequate and not problematic condition. However, concrete surfaces in tunnel for derivation pipes and structure require small touch ups and cleaning.

*Spillway*

The spillway has been inspected together with the technical staff of VWR administration on June 09 – 10, 2014 during field inspection. Although no cracking is observed on the concrete surfaces of the walls of spillway, quite high abrasion, spalling, and in somewhere crumblings and un-tight
joints are detected as no grouting protection exists on the reinforced concrete walls bottom slab of the spillway. In addition, over vegetation is also observed in some parts of the reinforced concrete slab.

The walls and slab of the spillway require adequately repair, groutings, and rehabilitation after cleaning the vegetation on the walls and slab.

However it can be assessed that the spillway structure has enough stability safety, and in no problematic condition. The spillway condition gives no reason for any concern in a short-term period.

**Energy Dissipation Facilities**

The walls and slab of the energy dissipation facilities require adequately repair, groutings, and rehabilitation after cleaning the vegetation on the walls and slab. No cracks, settlements, and exposure of reinforcement bar, and/or scouring on the downstream river bed are observed during the reconnaissance surveys and site visits.

However they can be assessed that they have enough stability safety, and in no problematic condition. Their conditions give no reason for any concern in a short-term period.

**Rock**

Rock is used only to cover the gravel – pebble layer at the upstream face of the dam body, and no changes are observed in rock composition include disintegration, exfoliation, softening, and dissolution during the survey and inspection of the dam body.

Rock fill at the upstream face of dam body is in good condition, and there is no reason for any concern.

**Soils**

No soil changes that may cause stability and maintenance problems for dam include degradation, dissolution, loss of plasticity or cementation, mineralogical shift, susceptibility to shearing, crumbling, and shrinkage–swelling are detected on and around the soil of the dam body. There are no signs of change in soil characteristics or signs of soil movements on and around the soils of the dam.

**Metals**

No deterioration from electrolysis, metal fatigue, stress-accelerated corrosion, tearing, and rupture is detected on the metal dam components during the surveys and inspection, except a little corrosion on the derivation pipes and metal components in the gate house. They require cleaning and painting.
However, all metal components are in adequate condition, and have enough stability safety, and in no problematic condition as there are no signs of deterioration on them.

2.3 Generic Changes

Though construction materials and design features may differ, the signs of generic changes are similar, as are the remedies.

Seepage & Leakage

Seepage control features within the dam body are chimney drain and drainage blanket at the downstream of the dam body, and piping from upstream toe of dam body to river flow channel at the downstream side of the dam. The inclined surface of the dam body at the downstream side is dry, and there is no sign of what shows excessive seepage from the inclined downstream face. Seepage water is observed only at the outlets of the pipes coming from upstream toe of the dam body. According to the records of the VWR Administration, seepage water coming from toe drain is about 2 lt/sec for each of the pipe outlet, and the water is generally clean.

The seepage flow from upstream to drain is normal for such dam.

According to VWR administration, there is no sign for seepage and leakage under or around the dam detected by observing changes in water quality indicators such as downstream turbidity, dissolved solids, temperature, and color; and physical indicators such as changes in discharge stage relation, pressure differentials, vegetation change on the downstream toe of the dam, and quantity of flow.

This shows that the transition zones within the dam body meet the accepted filter criteria, and are sufficiently wide to ensure that they are continuous.

Inadequate Seepage

There is no other visible evidence and clues on and around the dam body for wet spots; new vegetal growth, seepage, and for leakage; boils; saturation patterns on slopes, hillsides, and in streambeds; depressions and sinkholes; and evidence of high escape gradients.

Ice Damage

There no signs to show the drainage problems which note the presence of chemical precipitates and deposits, areas of stagnation, and areas of algal or bacterial growth at or around the upstream and downstream of the dam.

Stress & Strain

There is no visible evidence as a result of surveys and field inspections that certain components of the dam are being stressed beyond their design capabilities includes (in concrete) cracks, crushing,
displacements, and offsets, shearing and creeping; (in steel) cracks, extensions or contractions, bending, and buckling; and (in rock and soil) cracks, displacements, settling, consolidation, subsidence, and zones of extension and compression.

**Stability**

There are no problems detected with stability are evidenced by (in concrete and steel structures) tilting, tipping, sliding, and overturning; (in embankments, natural slopes, and cuts) bulging, sloughing, slumping, sliding, shrinkage, cracking and undercutting of escarpments, and sinking; (in rocks structures, slopes and foundations) slumping, sliding, falls, bulging, and cracking.

**Erosion Control**

There is no visible evidence for loss, displacement, and deterioration of upstream face rock fill, underlayment, and downstream face slope protection.

**Foundation**

There are no visible weaknesses in the foundation occurred through consolidation or liquefaction potential.

**Hydraulic Control Structures**

There are no malfunctions in hydraulic control structures include stability, retention of capacity rating, erosion at toe, unauthorized installations on crest, raising storage level, and decreasing spilling capacity, gate piers, and siphon prime settings.

**Headwater Control**

There are no mechanical errors in the headwater control occurred by unauthorized position, wedging, gate trunnion displacements, loss of gate anchorage post tensioning, undesirable eccentric loads from variable positions of adjacent gates, gate-seal binding, and erosive seal leakage.

**Shafts, Conduits, & Tunnels**

There are no weaknesses include vulnerability to obstruction; evidence of excessive external overloading pressure jets, contorted cross-sections, cracks, displacements, and circumferential joints; serviceability of linings (concrete and steel), materials deterioration, cavitation, and erosion; rockfalls; and severe leakage about tunnel plugs.
2.4 Environs

Reservoir

There are no noteworthy items at time of field inspection and site surveys; depressions and sinkholes in exposed reservoir basin surfaces; massive water displacing slide potentials - leaning trees, escarpments, and hillside distortions; flood pool encroachments; and siltation adversely affecting loading on dam and forming approach channel and waterway obstructions.

Downstream Proximity

There are no noteworthy items at time of field inspection and site surveys, reservoir connected springs; endangering seepage or leakage regardless of source; and river obstructions creating unanticipated tailwater elevations or interference with outfall channel capacities of the spillway and outlets.

Watershed

There are no noteworthy items include surface changes that might materially affect runoff characteristics.

Regional Vicinity

There are no noteworthy items include subsidence indications - sinkholes, trenches, subsidence surveys, settlements of buildings, highways, and other structures in the region; assessment of land forms and regional geologic structure; and records of mineral, hydrocarbon, and ground-water extractions, locations, producing horizons, accumulated production, and current rate of production.

Downstream Floodplain

There are no noteworthy items include the limits of natural, improved, or leveed channel; areas of potential inundation for spillway design flood and for hypothetical failure; proximity of developed areas; and habitation, population, communication, and transportation corridors.

There is sufficient distance to Masalli city with a large river basin which serves for stilling of large spillway discharge. This area provides an approximately 1.7 - 2 sq. km flat sloped river basin (3.8x0.55 km), which is sufficient to attenuate the flood effect.

The flood waves which will be born from the sudden rupture of the dam body leading to outburst of 46 million cum of water in an uncontrolled manner, would not reach the downstream villages/houses because of the downstream river flood plain will serve as a large stilling basin.
3. Spillway Capacity

According to hydraulic design of the dam, the spillway’s maximum discharge capacity is 550 m$^3$/s (see Table 4 and 9). The spillway is designed ungated typically in skimming flow regime for eliminating of any possibility of operator error. This causes that the spillway is unaffected by the massive amounts of debris commonly accompanying high flows. See following Figure.

Spillway discharge capacity is analyzed according to following formula.

$$Q = C_D \cdot B_{max} \cdot \sqrt[3]{g \cdot (2/3 \cdot H_1)^3}$$

Where:

<table>
<thead>
<tr>
<th>$Q$</th>
<th>Discharge capacity of chute spillway</th>
<th>566.09 m$^3$/s (calculated with the formula)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_D$</td>
<td>Dimensionless discharge coefficient</td>
<td>0.66</td>
</tr>
<tr>
<td>$B_{max}$</td>
<td>Breach free surface width for skimming overflow water</td>
<td>120 m</td>
</tr>
<tr>
<td>$g$</td>
<td>Gravitational acceleration</td>
<td>9.81 m/s$^2$</td>
</tr>
<tr>
<td>$H_1$</td>
<td>$(H - H_2) - \mu$</td>
<td>2.6 m</td>
</tr>
<tr>
<td>$H$</td>
<td>Dam Crest</td>
<td>80 masl</td>
</tr>
<tr>
<td>$H_2$</td>
<td>Spillway crest</td>
<td>77 masl</td>
</tr>
<tr>
<td>$\mu$</td>
<td>Wave coefficient to be deducted</td>
<td>0.4 m</td>
</tr>
</tbody>
</table>

As a result of this calculation; Spillway discharge capacity for skimming overflow water is 566.09 m$^3$/s, that this capacity is higher than the one which is given by VWR administration.
Partial Layout of Spillway
Operational Problems

The spillway will never fail to discharge even when unattended and under the most extreme conditions such as earthquake damage together with power failure and broken communication lines or under panic conditions during large floods as it is ungated.

4. Bottom Outlet

As seen in Table 8 and Figure 4, dam’s bottom outlet consists of two concrete channel, each of which is 297 m in length. It has the discharge capacity of 380 m³/s by 2 gates for flood discharge.

A back-up power facility for providing non-stop gate operations exists, and is maintained routinely.

5. Recommendations

5.1 Spillway

The spillway is in adequate and full, unrestricted operational condition.

However, the walls and slab of the spillway requires adequately repair, groutings, and rehabilitation after cleaning the vegetation on the walls and slab. The outlet of the downstream chute and energy dissipator section at the end of the downstream of the chute also require adequately repair, groutings, and rehabilitation after cleaning the vegetation.

5.2 Dam Embankments, Slopes – Up and Down Stream Faces

The upstream and downstream face of the dam body is in good condition, and there are no reasons for concerns.

The dam embankment shows no indications of irregularities. The slope surfaces are in proper condition. The design of slope inclination is in accordance with rules as usually adopted worldwide for zoned earth fill dams with a downstream gravel/rockfill shoulder - as is case also for the VWR dam.

Operation level is limited, freeboard assumed in range of 1.5 - 2 m.

In general, although the dam is safe, some exemplary stability calculations are conducted by the consultant for verification of the dam stability.
5.3 Constituent Materials

5.3.1 Concrete Surfaces

No pattern crazing and cracking, leaching, frost action, abrasion, spalling, and crumbling and structural cracking are observed.

Although concrete surfaces and elements inspected in gate room, tunnels, and outlet structure are in adequate and not problematic condition, they require small touch ups and cleaning.

5.3.2 Metals

Although no deterioration from electrolysis, metal fatigue, stress-accelerated corrosion, tearing, and rupture is detected on the metal dam components during the surveys and inspection, except a little corrosion on the derivation pipes and metal components in the gate house, they require comprehensive cleaning and adequate painting.

5.4 Monitoring & Instrumentation

Generally, instrumentation is not adequate in the dam.

Topographic Surveys

Topographic surveys of the dam surface are conducted twice a year.

However, it should be conducted quarterly (once a three month), and in high water level conditions (in case of spilling) it should be conducted monthly.

Piezometers and Benchmarking

Installing 2 benchmarks each on left and right abutment of the dam, and 2 benchmarks immediate downstream of the toe of the dam is immediate need, to control eventual movements of the flanks of the dam abutment.

Installing five piezometers gauges on the downstream face of the dam is immediate need as no piezometer wells are constructed within the first construction phase of Dam, and installing them on the crest is not possible at this stage, is immediate need. Installing system for them is proposed below;

- A first level of installation line about 15 m below crest - at station 150 m / 250 m / 350 m; and
- Second level of installation line about 30m below crest – at station 200 m / 400 m.
Piezometer readings should be monthly.

The head of these piezometers can be used as benchmark points.

In addition, installing at least one piezometer on downstream toe of the dam at the deepest level is required for downstream groundwater monitoring.

Any anomalies, such as sand boiling, sink hole, and sloughing on the embankment surface should be regularly observed during operation.

**Bathymetric Surveys for Monitoring of Sediment Deposits Regularly**

Regular bathymetric surveys should be conducted with a boat, GPS and sonar devices with an interval of at least one year.

**5.5 Analyses to be Conducted Periodically**

**Slope Stability Analysis**

The embankment of the dam should be cross checked once a 5 year (at least) operation period using with an general accepted method and software.

**Seismic Analysis**

Though design data of the dam shows that dam body and associated structures are planned to stand by 10 richter scale earthquake activity, a seismic analysis for the dam body should be conducted at least once a 5 year, taking into account no seismic and seismic loading conditions both for upstream and downstream reservoir empty and full conditions.

**Installing Seismometer on Dam Body**

A seismometers should be installed on dam body for monitoring of seismic activities nearby area.

**5.6 Hydraulic Control Structures**

There are no recommendations for the hydraulic control structures at this time as there are no malfunctions in them include stability, retention of capacity rating, erosion at toe, unauthorized installations on crest, raising storage level, and decreasing spilling capacity, gate piers, and siphon prime settings.

**5.7 Head Water Control**

There are no recommendations on head water control at this time as there are no mechanical errors in the headwater control occurred by unauthorized position, wedging, gate trunnion displacements,
loss of gate anchorage post tensioning, undesirable eccentric loads from variable positions of adjacent gates, gate-seal binding, and erosive seal leakage.

5.8 O&M Plan and Monitoring and Operation Documentation

Generally, documentation of VWR administration is good. However, since instrumentation in the dam is not adequate for the time being, the dam operation, monitoring etc. shall be documented regularly and adequately (log book with daily entrances; half year dam report with record data of reservoir level (daily) and results of topographic surveys and piezometer readings; including recording of drain water flows).

According to consultant’s professional judgement, O&M plan operation procedures for reservoir / gate operation, regular surveillance, and periodic inspection procedures for the VWR are adequate.

Nevertheless, the following example standard sheets for dam documentation and monitoring are annexed to this report for recommending to use in the VWR during the operation of the dam.

1. Project Data Sheet,
2. Annual Inspection Form
3. Data Forms for Monitoring Recording;
   – Seepage monitoring recording form
   – Observation well data form, and
   – Settlement / Movement monitoring data form

5.9 Energy Dissipation Facilities

The walls and slab of the energy dissipation facilities require adequately repair, groutings, and rehabilitation after cleaning the vegetation on the walls and slab.
APPENDIX B3

Site Photos 2015
Meeting with technical staff from VWR Administration
H. Omuzluoglu and Mahir Aliyev, Chief Eng.
VWR Administration

VWR Administration Building at site

Meeting with technical staff from VWR Administration
H. Omuzluoglu and Valiyev Ilham, VWR Administration

VWR Administration Building from Dam Crest
Inside the tunnel, derivation pipes on the right and left

Gate room and Gates

Outlet of Tunnel

Upstream Face of the Dam
Downstream Face of the Dam Body

One of Upstream Toe Drain Pipes

Entrance Channel of the Spillway

Downstream Part of the Spillway

Spillway – Energy Dissipator Outlet

Gate Room
Dam Crest and Right Abutment

Left Abutment
APPENDIX C

Minutes of 2015 Inspection by Azerbaijan Government Committee
Masalli raion, Geribler village

December 16, 2015

We are the committee members signing below:

Nuriyev Seymur Malik – Deputy Head of Raion Executive Power (REP), head of the department of social-economic analysis and forecasting, chairman of the committee
Agayev Agabey Huseyn- chief of the II Vilashchay Reservoir, deputy head
Imanli Orkan – senior advisor of the Division of Reservoirs and Maintenance of Water Intakes in Amelioration and Water Management Open Joint Stock Company, member of the committee
Gafarli Adil – Deputy Director of Azerbaijan State Water Management Project Institute of the Amelioration and Water Management Open Joint Stock Company, member
Fazil Huseynov – Head of the Department of Land Acquisition in Azerbaijan State Water Management Project Institute of the Amelioration and Water Management Open Joint Stock Company, member
Agil Ismayilov – senior engineer of Registration, Water Use and Conservation Control of the Amelioration and Irrigation Objects Office in Amelioration and Water Management Open Joint Stock Company, member
Mammadov Eyneli Panjali – advisor in local office # 15 of the State Committee of Property Issues, member
Huseyli Rafiq – staff member of the Geodesy and Cartography State Agency under the Ministry of Ecology and Natural Resources, member
Gasimov Rovshan – Chief of South Regional Construction Inspection Office in Construction Safety Control State Agency under MoES, member
Mikayilov Abbas Latif- Head of the Department of Architecture and Construction in Raion Executive Power Office (REPO), member
Maharramov Adem Imanverdi- Head of the Department of Work with Local and Self-management Offices in Raion Executive Power Office, member
Huseynov Mammadhuseyn Rizvan – Head of the Legal Department of REPO, member
Jafarov Famil Soltanaga – senior advisor of the Department of Social and Economic Development Analysis and Forecasting in REPO, member
Mirzyeyev Ingilab Balakhan – lead advisor in the Department of Architecture and Construction in REPO, member
Imamyarov Elnur Ispandiyyar – Representative from Garibler administrative territorial division in Massalli REP, member
Agayev Mushvig Vasif – Head of the Geribler municipality, member

In accordance with the order of the Masalli Head of Raion Executive Power number 172 dated December 2, 2015 and in agreement with the relevant agencies, a committee was established to investigate the status of the land use in conservation area of the Vilashchay Reservoir and Geribler administrative territorial division. Preliminarily, as a result of the committee’s monitoring in the reservoir and conservation are, the following were found out:
Vilashchay dam type reservoir built for irrigation purposes in Masalli raion, Geribler administrative territorial division, which will supply potable water to Masalli and Jalilabad raions
in near future has been operational since 1986. In accordance with the technical specifications of the reservoir, the length of the dam is 3807 m, maximum height is 33.85 m, and its water sources are the Vilash and Metere rivers. At normal water pressure the full volume of the reservoir is 46.0 mln. m³, and a useful volume is 38.0 mln. m³. Irrigation system furnished by the reservoir is functioning through closed irrigation network and main canal on the right bank and open soil canal on the left bank. Currently, the irrigation type reservoir drains an area of 9900 ha. The dam is equipped with two-section water releasing pipes functioning by electric doors with 380 m³/sec water releasing capacity and on the left bank it is equipped with automatic emergency water pushers and ferro-concrete fast flowers.

In accordance with the Decree of the Cabinet of Ministers of the RA number 109 dated August 21, 2003 “Classification of the List, Conservation Regime and Safety of Public and Local Hydro-Technical Plants” and “Rules on Dimension, Borders and Use of Conservation Areas” were approved. The water basin of Vilashchay Reservoir at normal water pressure was indicated 2.44 sq.km in its technical specifications submitted to the committee. In accordance with paragraph 6.1.1.2 of the “Rules on Dimension, Borders and Use of Conservation Areas” approved by the Decree of the CM of the RA number 109 dated August 21, 2003, a conservation area of the reservoirs with more than 2 sq. km water area is specified 500 m. In accordance with paragraphs 9, 10, 11 of the same rules the borders of conservation areas should be defined in their projects as well as dimensions and borders of the conservation areas of hydro-technical plants in the settlements should be included into the approved master plans considering planning and construction conditions and the relevant border signs should be placed by the amelioration and water management organizations. In addition, paragraph 13 of the same rules mentions that ministries and committees specified in paragraph 11 of the same rules should define the minimum conservation area of hydro-technical plants and they should be included into the general construction plans, land use plans and other plan-cartographic materials of the cities and other settlements until the projects of the conservation areas of hydro-technical plants are drafted and approved.

In accordance with the State Program of the RA on Social and Economic Development of the Regions for 2009-2013 reconstruction of water supply and sanitation systems in Masalli are progressing under the relevant project. The Project intends to supply drinkable water of the city from the Vilashchay Reservoir and for this purpose, a water regulator with daily power capacity of 22,000 cubic meters and a reservoir with a volume of 5000 cubic meters were constructed on the down bank. As a result of the World Bank-financed project potable water supply of Masalli and later Jalilabad will be provided from the Vilashchay Reservoir in near future. Therefore, using the water of the Vilashchay Reservoir for irrigation and domestic purposes, made the second phase of construction and assessment of the conservation regime necessary.

Despite of the aforementioned issues, the conservation area project papers defining the borders of protected areas of the reservoir were not available during the assessment, and the relevant border signs were not found out in the area. During the observation it was found out that the reservoir was surrounded by forests from north-west and south-west, and agricultural land plots and settlements from south and south-east. Taking into consideration that in fact maps of the area are not available, REP applied to the local office # 8 of the State Service of Real Estate Register under the State Committee of Property Issues. The real condition was assessed by the committee until acquiring any maps of the area and taking non-availability of the border signs into consideration, the signs covering 200-400 m area within 500 m conservation area specified for this type of reservoirs at
normal water pressure were included into the draft master plan of reservoir presented by the maintainer organization of the reservoir. Taking into consideration that the signs covering 400 m area in the master plan in some places are crossing the Masalli -Yardimli highway of national importance, preliminarily the committee assessed the situation in the area of 200 m. During the preliminary assessment, the committee conducted monitoring in the buildings on the right and left sides of the road in distance of 100 m from normal water pressure level on the south-east bank in Garibler village. During the monitoring on the right side of the mentioned road (western direction), it was observed that in fact 30 private houses, 6 incomplete buildings and 10 empty land plots were fenced in. Besides, on the left side of the mentioned road 40 private houses, 6 lands plots were fenced in. The monitoring were mostly conducted in the distance of 200 m from normal water pressure level. In addition, we would like to mention that during the construction of Vilashchay reservoir, Isi village were moved from its old location and private houses were built for the residents out of the proceeds of the public funds along the Masalli -Yardimli highway. However, it was found out that some private houses and recreation centers remained in the old location of Isi village. We would like to mention that there are 6 recreation centers in the forests on the south bank of the reservoir and these centers are seasonably functional. 

During the preliminary monitoring, it was found out that there was no living in some private houses and as some residents had not presented their property documents, the documents of only 12 lands plots were collected. Four of them are related to agricultural lands, but there are private houses in these lands as well. The committee members, taking into consideration the aforementioned issues and the results of the preliminary monitoring, consider it advisable to raise the issue in the relevant agencies in order to take complex measures and conduct serious assessments in this area.

We approve this act by our signatures:

S. Nuriyev  
A. Agayev  
O. Imanli  
A. Gafarli  
F. Huseynov  
A. Ismayilov  
E. Mammadov  
R. Huseynli  
R. Gasimov  
A. Mikayilov  
A. Maharramov  
M. Huseynov  
F. Jafarov  
I. Mirzayev  
E. Imamyarov  
M. Agayev
APPENDIX D

Report of Vilashchay Dam

1. The Bank task team visited Baku and Massali from March 1-4, 2016 to review the safety status and operation and maintenance of Vilashchay Dam. The mission visited the dam site in Massali, inspected the status of dam seepage, monitoring instrumentation, gate operation, and discussed the operation and maintenance procedures and emergency preparedness with the dam owner/operator. The task team also reviewed the dam safety documents provided by PIU, dam operator and design institute; and discussed technical issues of proposed extension of the dam with the dam owner/operator, Azerbaijan State Institute on Design of Water Farm Objects, and consulting firm Aim Texas Trading, LLC. The findings and recommendations are summarized below.

2. Documents provided include: original design report of the dam; hydrological data about water level, inflow and outflow (daily and month average, from 1900 to 2014); operation and maintenance manuals, emergency and preparedness plan for 2016. The task team was informed that official inspection was taken place in September 2015, draft minutes have been prepared and the final signed minutes is expected to be ready by the end of March, 2016. The task team recommends the PIU to provide the signed minutes to the Bank for information once the signed minutes are ready.

3. Design of the dam. The existing Vilashchay Dam is in fact at its phase one stage. The dam was design to be built in two phases. In first phase, which is the existing dam, the dam height was 28 meters with 46 million cubic meters reservoir capacity serving irrigation for 10,000 hectors farmland. For the second phase, which is planned to be built in the future, the dam height would be increased by 13.5 meters with totally 132 million cubic meters reservoir capacity serving for 15,000 hectares more farmland. When the dam was built from 1979 to 1986, irrigation tunnel and pipes were designed and built for phase II, and dam was built wider than phase I needed considering easy extension to phase II.

4. Hydrological calculation. The Bank task team has discussed with the consulting firm about hydrological calculation for design of the dam extension. Constrained by the available hydrological data, the former calculation of the consulting firm are not sufficient, the task team recommends the consulting firm to update its calculation and analysis by using the new collected data, and revise the dam safety report to the Bank for further review by March 31, 2016. When doing the freeboard calculation, wave run-up, wind backwater, and surcharge should be fully considered.

5. Operation and maintenance (O&M). The dam is being operated by Vilashchay Water Dam Operation and Maintenance Department (dam operator department). Funding of the O&M is coming from government budget. The dam operator department has about 80 staff, of which 22 are being directly in charge of the O&M of Vilashchay Dam. 6 of the 22 people have engineering degree. There are 9 observers doing daily patrolling and observation on site and another 9 stay at the operating building during 24 hours a day (3 shifts with 3 staff per shift). The Ministry of Emergency Situations (MES) provides training on dam operation and maintenance and emergency preparedness one to two times a year as per approved schedule of training sessions attached to annual emergency preparedness plan. Operation and maintenance manuals are existed but
documented in the shelf. The mission recommends the dam operator department to improve its management by:

(i) inviting competent technical agency and experts to provide training on the dam operation and maintenance; and

(ii) print out the operation and maintenance procedures and stick them on the wall in the operating building.

6. Emergency preparedness plan (EPP). Based on the Azerbaijan dam safety management regulations, emergency preparedness plan should be prepared and approved by regional office of the Ministry of Emergency Situations every five years. The most recent EPP of Vilashchay Dam covers the period of from 2011 to 2015. The EPP includes three scenarios of emergency situations: (a) destruction and landslides happen at the office of administrative building and the dam due to earthquake; (b) unexpected fire accident; (c) terrorism and enemy’s attack; and (d) unstable epidemiology. The MES has sent an official letter to Dam operator requesting to prepare the revised EPP for 2016 to 2020 and send it to MES by March 15, 2016. The revised EPP is under preparation by the dam operator department. The task team recommends that the EPP could be improved by:

(i) Adding a scenario of water poisoning should be included because water supply function has been added to the reservoir by the Bank project.

(ii) The EPP should be updated every year. The EPP was reviewed and approved every five years. Among the five years, there may be some uncertainty including social, economic, technical and economic conditions; to counter these changed situations, the EPP should be updated every year.

(iii) The revised EPP could be improved by using the attached guidelines.

7. Replacement of the gates for water supply pipes and discharge tunnel. The task team found that the existing four gates (which are more than 30 years old) are in poor conditions and should be replaced promptly. The task team was pleased to learn that the dam operator has secured the funds in 2016 state budget to replace the gates. The task team recommends to replace the old gates as early as possible.

8. Monitoring instrumentation. The task team did not observe the use of any monitoring devices/facilities on the dam control the deformation, sedimentation, seepage etc. It is understood that these parameters are mainly monitored through visual inspections by chief engineer and patrolling team. This is not sufficient for effective dam safety management. The task team recommends the dam operator department to install monitoring system to improve the monitoring of the dam.

Leaking on the right side of spillway. The task team spotted severe leakage on the right side of the spillway connecting to the dam shoulder at left bank. The task team emphasized that this is a highly potential risk for safety of the dam and spillway. The task team recommends the dam operator to pay close attention to the development of the leaking point and repair it as early as possible.
Attachment:

**Guidelines for Emergency Preparedness Plan (EPP)**

- Distribution list, list of revision
- Purpose, description of dam
- Personnel authority, responsibilities and duties
- Emergency identification and evaluation process
- Preventative actions (where applicable)
- Notification procedures and flowcharts
- Communication systems, access to site
- Emergency sources of equipment, materials and power
- Inundation maps and tables
- Warning system (if applicable)
- Testing and upgrading EPP
- Training
APPENDIX E

Subsurface Geologic Profile Along Longitudinal Axis of Dam

Design Cross Sections Located at Pickets 16+00 and 21+00
APPENDIX F

Results of Seepage Analysis
SEEPAGE ANALYSIS RESULTS

Figure D-1
Picket 16+00
Steady-State Seepage Analysis Results
Reservoir Water Level: 77 meters above sea level (masl)
Figure D-2
Picket 16+00
Steady-State Seepage Analysis Results
Reservoir Water Level: 75.5 meters above sea level (masl)
Figure D-3
Picket 16+00
Steady-State Seepage Analysis Results
Reservoir Water Level: 62.2 meters above sea level (masl)
SEEPAGE ANALYSIS RESULTS

Figure D-4
Picket 21+00
Steady-State Seepage Analysis Results
Reservoir Water Level: 77 meters above sea level (masl)
Figure D-5
Picket 21+00
Steady-State Seepage Analysis Results
Reservoir Water Level: 75.5 meters above sea level (masl)
SEEPAGE ANALYSIS RESULTS

Figure D-6
Picket 21+00
Steady-State Seepage Analysis Results
Reservoir Water Level: 62.2 meters above sea level (masl)
APPENDIX G

Results of Slope Stability Analysis
STABILITY ANALYSIS RESULTS

Figure E-1
Picket 16+00
Downstream Static Slope Stability Analysis Results
Reservoir Water Level: 77 meters above sea level (masl)
Figure E-2
Picket 16+00
Downstream Seismic Slope Stability Analysis Results
Reservoir Water Level: 77.0 meters above sea level (masl)
STABILITY ANALYSIS RESULTS

Figure E-3
Picket 16+00
Downstream Static Slope Stability Analysis Results
Reservoir Water Level: 75.5 meters above sea level (masl)

FIGURE E3
DAM SAFETY ASSESSMENT REPORT
VILESHCHAY WATER RESERVOIR
STABILITY ANALYSIS RESULTS

Figure E-4
Picket 16+00
Downstream Seismic Slope Stability Analysis Results
Reservoir Water Level: 75.5 meters above sea level (masl)

FIGURE E4
DAM SAFETY ASSESSMENT REPORT
VILESHCHAY WATER RESERVOIR

Les Bromwell, P.E., Sc.D.
Engineering Consultant
505 Tulip Lane
Vero Beach, Florida 32963

DATE: August 2015
SCALE: As Shown

APPENDIX G
STABILITY ANALYSIS RESULTS

DAM SAFETY ASSESSMENT REPORT
VILESHCHAY WATER RESERVOIR

DATE: August 2015
SCALE: As Shown

Les Bromwell, P.E., Sc.D.
Engineering Consultant
505 Tulip Lane
Vero Beach, Florida 32963

APPENDIX G
Figure E-6
Picket 16+00
Downstream Seismic Slope Stability Analysis Results
Reservoir Water Level: 62.2 meters above sea level (masl)
STABILITY ANALYSIS RESULTS

Figure E-7
Picket 21+00
Downstream Static Slope Stability Analysis Results
Reservoir Water Level: 77 meters above sea level (masl)

Elevation, masl

Distance, meters

2.517

Les Bromwell, P.E., Sc.D.
Engineering Consultant
505 Tulip Lane
Vero Beach, Florida 32963

DATE: August 2015
SCALE: As Shown

APPENDIX G
Figure E-8
Picket 21+00
Downstream Seismic Slope Stability Analysis Results
Reservoir Water Level: 77 meters above sea level (masl)
Figure E-9
Picket 21+00
Downstream Static Slope Stability Analysis Results
Reservoir Water Level: 75.5 meters above sea level (masl)
STABILITY ANALYSIS RESULTS

Figure E-10
Picket 21+00
Downstream Seismic Slope Stability Analysis Results
Reservoir Water Level: 75.5 meters above sea level (masl)

FIGURE E10
DAM SAFETY ASSESSMENT REPORT
VILESHCHAY WATER RESERVOIR
Figure E-11
Picket 21+00
Downstream Static Slope Stability Analysis Results
Reservoir Water Level: 62.2 meters above sea level (masl)
STABILITY ANALYSIS RESULTS

Figure E-12
Picket 21+00
Downstream Seismic Slope Stability Analysis Results
Reservoir Water Level: 62.2 meters above sea level (masl)
Figure E-13
Picket 21+00
Rapid Drawdown Analysis Results
Upstream Embankment Slope Stability
Reservoir Water Level: 77.0 meters above sea level (masl)
## APPENDIX H

Calculations

### TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>SUBJECT</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Empirical Prediction of Hydraulic Conductivities</td>
<td>1</td>
</tr>
<tr>
<td>Empirical Estimation of Primary Compression Index</td>
<td>3</td>
</tr>
<tr>
<td>Wind/Wave Analysis</td>
<td>4</td>
</tr>
<tr>
<td>Required Thickness of Upstream Dam Stone Revetment</td>
<td>12</td>
</tr>
<tr>
<td>Design of Dam Granular Filter/Transition Zones</td>
<td>15</td>
</tr>
<tr>
<td>Hydraulic Flow Analysis for Bottom Tunnels</td>
<td>23</td>
</tr>
<tr>
<td>Hydraulic Flow Analysis for Derivation Tunnels</td>
<td>25</td>
</tr>
<tr>
<td>Hydraulic Flow Analysis for Spillway Outlet</td>
<td>27</td>
</tr>
<tr>
<td>Consolidation Settlement of Spillway Foundation Clays</td>
<td>30</td>
</tr>
<tr>
<td>Secondary Settlement of Spillway Foundation Clays</td>
<td>33</td>
</tr>
</tbody>
</table>
Empirical Predictors of Hydraulic Conductivity

Kozlen-Carman:

\[ k = \left( \frac{y}{n} \right) \cdot \left( \frac{1}{h_w} \right) \cdot \left( \frac{1}{S^2} \right) \left[ \frac{c^3}{1 + c} \right] \]

where

\[ f = \frac{998 \text{ kg}}{\text{m}^3} \text{ at } 20^\circ \text{C} \]

\[ \mu = \text{dynamic viscosity (N.s/m}^2\text{)} \]

\[ 1N = 1 \text{ kg} : \frac{m}{s^2} = 1 \times 10^{-3} \text{ N/m}^2 \]

\[ \gamma_w = 9.98 \times 10^{-4} \text{ N/m} \text{ at } 20^\circ \text{C} \]

\[ c_{ref} = 4.8 \times 10^{-4} \text{ cm/sec} \]

\[ S_0 = \frac{5F}{D_{eff}} \]

\[ S_i = \frac{6.0}{s_i} \text{ spherical particles} \]

\[ S_F = \frac{7.7}{s_F} \text{ angular particles} \]

\[ D_{eff} = \text{effective grain diameter of particles in centimeters} \]

\[ D_{eff} = 7.7 \times \frac{1}{0.02} + 0.08 \text{ cm} = 385 \text{ to 96 cm} \]

\[ h = 9.98 \times 10^{-4} \cdot \left( \frac{1}{4.8} \right) \cdot \left( \frac{1}{S^2} \right) \left[ \frac{0.35^3}{1 + 0.35} \right] = \]

\[ = 7.2 \times 10^{-3} \text{ cm/sec} \]

\[ k = 5.52 \cdot \left( \frac{D_{eff}}{S} \right)^2 \cdot \left[ \frac{c^3}{1 + c} \right] \]

\[ = 0.007 \text{ cm/sec} = 7.01 \times 10^{-3} \text{ m/sec} \]

\[ = 0.1115 \text{ cm/sec} = 0.001115 \text{ m/sec} = 16 \text{ m/hr} \]

\[ = 96 \text{ m/hr} \]

\[ = 9.6 \text{ ft/hr} \]

Layer 2

Good.
\[ a = \frac{e}{1 + e} \]
\[ 0.4 = \frac{e}{1 + e} \]
\[ 0.4 + 0.4e = e \]
\[ e = 0.67 \]

\[ b = 552 \cdot (D_{eff})^2 \cdot \left[ \frac{e^3}{1 + e} \right] \]
\[ = 552 \cdot (1 \times 10^{-4} \text{cm})^2 \cdot \left[ \frac{0.67^3}{1 + 0.67} \right] \]
\[ = 9.44 \times 10^{-7} \text{ cm/sec} \]
\[ = 0.10 \text{ m/day} \]

\[ 5.9 \times 10^{-8} \text{ m/sec} \]
\[ = 8.6 \times 10^{-4} \text{ m/day} \]

\[ 1 \times 10^{-3} \text{ m/day} \]
\[ = (0.001 \text{ m/day}) \]

\[ = (0.002 \text{ m/day}) \]

\[ = 0.001 \text{ m/day} \]

\[ = 0.002 \text{ m/day} \]

\[ = 0.001 \text{ m/day} \]

\[ = 0.002 \text{ m/day} \]

\[ \text{USE VALUE IN REPORT} \]

\[ = \text{USE FOR REPORT} \]

\[ = 0.2 \text{ m/day} \]

\[ = 0.28 \text{ m/day} \]

\[ = 0.72 \text{ m/day} \]

\[ = 0.28 \]

\[ = 0.4 \]

\[ \text{O.K.} \]

\[ \text{NOT EFFECTIVE FOR CLAYS.} \]

\[ \text{USE FOR REPORT} \]

\[ \text{1.2 mm (2 mm max.)} \]

\[ e = 0.4 \]

\[ 0.36 \text{ cm/sec} \]

\[ 3.13 \text{ m/day} \]

\[ 0.08 \text{ m/day} \]

\[ 0.1 \text{ m/day} \]

\[ 2.1 \text{ m/day} \]
Empirical Estimation of Primary Compression Index, $C_c$

Typical Values of Primary Compression Index

- Firm Clay: $0.03 - 0.06$
- Stiff Clay: $0.06 - 0.15$
- Medium-Soft Clay: $0.15 - 1.0$

$C_c = 0.54(e_0 - 0.35)$

$C_c = 0.0054(2W_h - 35)$

$C_c = 0.5217(e_0 - 0.2) = 0.245$

Nashida (1956) for undisturbed clay:

$C_c = 0.6$ to $0.7$

$W_h = 36\%$ to $52\%$

$C_c = 0.173$ (Nashida)

$C_c = 0.37$ to $0.32$ (Nashida)

$C_c = 0.245$

Layer 4 Clay:

$\gamma = 0.4$

$e = 0.6$ to $0.7$

$W_h = 36\%$ to $52\%$

$C_c = 0.173$ (Nashida)

$C_c = 0.37$ to $0.32$ (Nashida)

$C_c = 0.245$

$\ell_0 = w_{bf}$

$C_c = 0.4(e_0 - 0.25)$

$C_c = 0.01(W_h - 5)$

$\ell_0 = w_{bf}$

Avg. $= 0.2465$

Avg. $= 0.23$

$Hough (1957):$

$C_c = 0.4049(e_0 - 0.3216) = 0.15$

$C_c = 0.0102(W_h - 9.15) = 0.26$

Rendel - Henrvon (1980):

$C_c = 0.30(e_0 - 0.27) = 0.12$

Serajuddin (1987):

$C_c = 0.0102(W_h - 9.15) = 0.26$

$C_0/C_c = 0.04$  $C_c = 0.010$
### Wind/Wave Analysis: Vilesh Chay Water Reservoir

**Fetch Calculations**

**Maximum Fetch:** 7480 ft \(= 1.42 \text{ mi} \) or 2279.9 m

**Effective Fetch:**

<table>
<thead>
<tr>
<th>( t )</th>
<th>( C_1 )</th>
<th>( t - 1.42 )</th>
<th>( C_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>42</td>
<td>0.7431</td>
<td>19.00</td>
<td>14.119</td>
</tr>
<tr>
<td>36</td>
<td>0.8090</td>
<td>20.42</td>
<td>16.579</td>
</tr>
<tr>
<td>30</td>
<td>0.8660</td>
<td>24.22</td>
<td>20.975</td>
</tr>
<tr>
<td>24</td>
<td>0.9135</td>
<td>26.26</td>
<td>23.894</td>
</tr>
<tr>
<td>18</td>
<td>0.9511</td>
<td>27.07</td>
<td>25.741</td>
</tr>
<tr>
<td>12</td>
<td>0.9781</td>
<td>27.83</td>
<td>26.719</td>
</tr>
<tr>
<td>6</td>
<td>0.9945</td>
<td>7.050</td>
<td>69.665</td>
</tr>
<tr>
<td>0</td>
<td>1.00</td>
<td>7.480</td>
<td>74.80</td>
</tr>
<tr>
<td>12</td>
<td>0.9781</td>
<td>21.73</td>
<td>21.254</td>
</tr>
<tr>
<td>18</td>
<td>0.9511</td>
<td>13.30</td>
<td>12.649</td>
</tr>
<tr>
<td>24</td>
<td>0.9135</td>
<td>9.38</td>
<td>8.569</td>
</tr>
<tr>
<td>30</td>
<td>0.8660</td>
<td>7.84</td>
<td>6.789</td>
</tr>
<tr>
<td>36</td>
<td>0.8090</td>
<td>7.60</td>
<td>6.148</td>
</tr>
<tr>
<td>42</td>
<td>0.7431</td>
<td>7.60</td>
<td>5.647</td>
</tr>
</tbody>
</table>

\[ F_e = \frac{\sqrt{\frac{X_{\text{wind}}}{2}}}{\sqrt{\frac{X_{\text{wind}}}{2}}} = \frac{36,650.9}{13.5106} = 2,712.7' = 826.9 \text{ m} = 0.8269 \text{ km} \]

**Viable Wind Speeds/ Velocities:**

- **3-sec, 100-yr Wind Speed:** 40 m/sec \(= 90 \text{ mph} - 100 \text{ mph} \)*
- **3-sec, 475-yr Wind Speed:** 50 m/sec \(= 115 \text{ mph} \)*
- **Avg. Annual Wind Speed:** 7 m/sec \(= 15 \text{ mph} \)*
- **10-yr, 100-yr Wind Speed:** 30 m/sec \(= 68 \text{ mph} \)*
- **3-sec, 1000-yr Wind Speed:** 134 mph \(= 135 \text{ mph} \)*

*Note: The aforementioned wind speeds are used to design wind turbines, wind power plants, and tall structures in Middle Eastern countries.*
### Vileshchay Water Reservoir

#### Wind/Wave Analysis - Freeboard Determination

**Effective Fetch Calculation**

<table>
<thead>
<tr>
<th>alpha</th>
<th>alpha (rad)</th>
<th>cos alpha</th>
<th>x, feet</th>
<th>xcos alpha</th>
</tr>
</thead>
<tbody>
<tr>
<td>42</td>
<td>0.733037667</td>
<td>0.74314524</td>
<td>1900</td>
<td>1411.975956</td>
</tr>
<tr>
<td>36</td>
<td>0.628318</td>
<td>0.809017306</td>
<td>2042</td>
<td>1652.01334</td>
</tr>
<tr>
<td>30</td>
<td>0.523598333</td>
<td>0.866025625</td>
<td>2422</td>
<td>2097.514064</td>
</tr>
<tr>
<td>24</td>
<td>0.418878667</td>
<td>0.913545602</td>
<td>2612</td>
<td>2386.181111</td>
</tr>
<tr>
<td>18</td>
<td>0.314159</td>
<td>0.951056598</td>
<td>2707</td>
<td>2574.510212</td>
</tr>
<tr>
<td>12</td>
<td>0.209439333</td>
<td>0.978147638</td>
<td>2731</td>
<td>2671.321198</td>
</tr>
<tr>
<td>6</td>
<td>0.104719667</td>
<td>0.994521905</td>
<td>7005</td>
<td>6966.625942</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>1</td>
<td>7480</td>
<td>7480</td>
</tr>
<tr>
<td>6</td>
<td>0.104719667</td>
<td>0.994521905</td>
<td>3324</td>
<td>3305.790811</td>
</tr>
<tr>
<td>12</td>
<td>0.209439333</td>
<td>0.978147638</td>
<td>2173</td>
<td>2125.514816</td>
</tr>
<tr>
<td>18</td>
<td>0.314159</td>
<td>0.951056598</td>
<td>1330</td>
<td>1264.905276</td>
</tr>
<tr>
<td>24</td>
<td>0.418878667</td>
<td>0.913545602</td>
<td>938</td>
<td>856.905743</td>
</tr>
<tr>
<td>30</td>
<td>0.523598333</td>
<td>0.866025625</td>
<td>784</td>
<td>678.9640899</td>
</tr>
<tr>
<td>36</td>
<td>0.628318</td>
<td>0.809017306</td>
<td>760</td>
<td>614.8531528</td>
</tr>
<tr>
<td>42</td>
<td>0.733037667</td>
<td>0.74314524</td>
<td>760</td>
<td>564.7903822</td>
</tr>
</tbody>
</table>

Max. Fetch = 13.51091983 ft = 36651.86612 ft

**Effective Fetch, feet =**

- 2712.758761 feet
- 826.8589249 meters
- 0.513780068 miles
- 0.826858925 kilometers

Maximum Fetch, \( \frac{\text{ft}}{\text{ft}} \) = 7480 ft = 2279.9 meters = 1.42 miles = 2.28 kilometers

\[ \begin{align*}
\text{Angle at } \theta = 96.29' + 1' = 97.29' \\
\text{Angle at } \theta = 97.29' = 3.46 \text{ degree in I. D. formula} \quad \frac{\theta}{\theta} \\
\text{Angle at } \theta = 1.0 \\
\frac{\text{M} \theta}{\text{M} \theta} = 1.0 \quad \frac{\theta}{\theta} = 3.43'
\end{align*} \]
WIND/WAVE ANALYSIS - VILESH CHAY RESERVOIR

Total Annual Average Precipitation: 270 mm (10 in)
Avg. Monthly May Monthly Precipitation: 30 mm (November)
Max. Daily Precip.: 13 mm
Max. Annual Precip.: 55.15 (1400 mm) 1000 mm (39")
60 in/yr

Surf-similarity parameter (Tribeke number)

\[ \theta_0 = \frac{\tan \theta}{\sqrt{\theta}} \]

\[ \theta_0 = \frac{H_0}{L_0} \]

\[ L_0 = \frac{a^2}{g} \]

\[ H_0 = \text{deep water wave height} \]

\[ \theta_0 = \frac{H_0}{L_0} \]

\[ \theta_0 = \frac{2\pi H_s}{gT^2} \]

\[ H_s = \text{significant wave height} \]

\[ \theta_0 = \frac{1}{H_s} \]

\[ \frac{H_s}{\theta} = 1.0 \]

\[ \theta = 0 \]

\[ \frac{H_s}{\theta} = 1.6 \]

\[ \theta = 0.6 \]

\[ \frac{H_s}{\theta} = 1.0 \]

\[ \theta = 0.0 \]

\[ \frac{H_s}{\theta} = 0.6 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]

\[ \theta = 0.0 \]
For permeable slopes: Delft Hydraulic Lab

\[ P = 0.4 \]

\[ P = 0.5 \]

\[ R_{uv} = \frac{A \cdot \frac{3}{2}}{B \cdot \left(\frac{3}{2}\right)} \]

\[ H_s = \frac{D}{3} \text{ for } 1.0 \leq \frac{3}{2} \leq 1.5 \]

\[ H_s = \frac{D}{3} \text{ for } 1.5 \leq \frac{3}{2} \leq \left(\frac{D}{3}\right)^{1/2} \]

\[ H_s \leq \frac{D}{3} \text{ for } \left(\frac{D}{3}\right)^{1/2} = \frac{D}{3} \leq 7.5 \]

\[ A = 0.1 \]

\[ B = 1.34 \]

\[ C = 0.55 \]

\[ D = 2.58 \]

\[ P = 0.4 \]

\[ P = 0.1 \]

\[ \text{deep water condition} / \text{Rayleigh wave effect} \]

For permeable slopes w/ impermeable core: Delft Hydraulic Lab

\[ R_{uv} = \frac{A \cdot \frac{3}{2}}{B \cdot \left(\frac{3}{2}\right)} \]

\[ H_s = \frac{D}{3} \text{ for } 1.0 \leq \frac{3}{2} \leq 1.5 \]

\[ P = 0.1 \]

\[ \frac{R_{uv}}{H_s} = B \left(\frac{3}{2}\right) \]

\[ f_c (\frac{3}{2}) \]

\[ \frac{3}{2} \]

\[ 1.0 \leq \frac{3}{2} \leq 1.5 \]

\[ H_s \]

\[ = 0.03 \cdot 0.25 \cdot (\frac{40}{g})^{0.58} \]

\[ F_e = 0.42 \]

\[ = 0.03 \cdot 0.25 \cdot (\frac{40}{g})^{0.58} \]

\[ (\frac{827}{5.2})^{0.58} \]

\[ = 1 m = 3.1 \text{ ft, 1.3 m, 1.3 ft. \checkmark} \]

\[ 1.17 m = 3.85' \]

\[ T_e = 0.581 \cdot \left(\frac{10}{9}\right)^{0.58} \]

\[ = 0.581 \cdot \left(\frac{827}{5.2}\right)^{0.58} \]

\[ = 2.5 \text{ sec, acc.} \]

\[ H_s = 1.3 \]

\[ H_s = 1.3 \cdot 3.2' = 4.2 \text{ ft. \checkmark} \]

\[ H_s = 1.3 \cdot 3.85' = 5 \text{ ft. AVG.} \]

\[ H_s = 1.3 \cdot 3.85' = 5 \text{ MAX} \]

\[ 3.525 \]

\[ 4.2 \text{ ft.} \]

\[ 4.2 \text{ ft.} \]

\[ 5 \text{ ft.} \]

\[ 4.2 \text{ ft.} \]

\[ 1.3 \]

\[ 1.3 \cdot 3.85' = 5 \text{ ft.} \]

\[ 1.3 \cdot 3.2' = 4.2 \text{ ft.} \]

\[ \text{1000-yr recurrence interval:} \]

\[ H_s = 0.03 \cdot 0.25 \cdot (\frac{40}{g})^{0.58} \]

\[ F_e = 0.42 \]

\[ = 0.03 \cdot 0.25 \cdot (\frac{40}{g})^{0.58} \]

\[ (827)^{0.58} \]

\[ = 1.14 m = 3.8 \text{ ft.} \]

\[ 1.3 \]

\[ 1.3 \cdot 3.85' = 5 \text{ ft.} \]

\[ 5.25 \text{ ft.} \]

\[ 4.9 \text{ ft.} \]
Vileshchay Water Reservoir

Determine Design Wind Speed (100-yr recurrence interval)

50-yr 3-sec wind speed: ____________
100-yr 3-sec wind speed: 100 mph / 92 mph
100-yr 1-hr wind speed: 60 mph / 66 mph

\[
\begin{align*}
\frac{U_3}{U_{3000}} &= 1.277 + 0.296 \cdot \text{tanh} (0.9 \cdot \log_{10} \left( \frac{72}{30} \right)) \\
\frac{U_3}{U_{3000}} &= 1.277 + 0.296 \cdot \text{tanh} \left( 4.76 \right) = 1.509 \\

\text{Wind speed over water (1-hr)} &= 66 \text{mph} \cdot 1.2 = 79.47 \text{mph}
\end{align*}
\]

fetch-limited duration required: \( t = \frac{112 \cdot F_0}{V_{3000}} = 16.19 \)

\[
\begin{align*}
U_3 / U_{3000} &= 1.277 + 0.296 \cdot \text{tanh} \left( 4.76 \right) = 1.03 \\
\Rightarrow 81.85 \text{mph}
\end{align*}
\]

Stability correction does not apply as \( F < 10 \text{min} \) (0.5 < 10 min)

wind stress factor, \( U_a \) (mph): \( U_a = 0.5891 \cdot U_{3000} \cdot 1.2 = 132.6 \text{mph} = 194.82 \text{m/s} \)

Design Wind Speed (475-yr recurrence interval)

475-yr 3-sec wind speed: 115 mph / 93 mph \cdot 1.23 = 114.39 mph
475-yr 1-hr wind speed: 114.39 mph / 1.32 = 86.58 mph

wind speed over water (1-hr) = 90.94 mph / 1.32 = 75.76 mph

fetch-limited duration: \( t = \frac{112 \cdot F_0}{V_{3000}} = 16.45 \text{min} \)

\[
\begin{align*}
U_3 / U_{3000} &= 1.277 + 0.296 \cdot \text{tanh} \left( 4.76 \right) \\
\frac{U_3}{U_{3000}} &= 1.277 + 0.296 \cdot \text{tanh} \left( 4.76 \right) = 1.03 \\
U_3 / U_{3000} &= 1.277 + 0.296 \cdot \text{tanh} \left( 4.76 \right) = 1.03 \\
\Rightarrow 157.1 \text{mph}
\end{align*}
\]

wind stress factor, \( U_a \) (mph): \( U_a = 0.5891 \cdot U_{3000} \cdot 1.2 = 183.68 \text{mph} = 230.47 \text{m/s} \)
Deep Water Significant Wave Height (1):

\[ H_{m0} = 0.00282 \cdot V_a \cdot F_c^{0.5} = 2.86' \] 100-yr, 3-sec wind.

\[ H_{m0} = 0.00282 \cdot V_a \cdot F_c^{0.5} = 3.38' \] 475-yr, 3-sec wind.

Deep Water Wave Period (T):

\[ T = 0.0283 \cdot (V_a \cdot F_c)^{0.33} \] 100-yr, 3-sec wind.

\[ T = 0.0283 \cdot (194.82/116.7 \cdot 2712.7)^{0.33} = 2.19\sec \]

\[ T = 0.0283 \cdot (V_a \cdot F_c)^{0.33} \] 475-yr, 3-sec wind.

\[ T = 0.0283 \cdot (230.47/116.7 \cdot 2712.7)^{0.33} = 2.31\sec \]

Deep Water Wavelength (λ):

\[ 2\lambda = 5.12 \cdot T^2 = 27.32' \] (475-yr)

\[ 2\lambda = 24.5' \] (100-yr)

Relative Depth \(d/H_0\):

\[ d/H_0 = \frac{1}{100} \cdot \frac{27.32}{24.5'} = 0.360 \]

\[ d/H_0 = \frac{1}{100} \cdot \frac{4.07}{3.92\min} = 0.50 \]

So, deep water conditions present.

Wave Height & Probability of Exceedence:

100-yr, 3-sec: 2.86' / 1.4' = 4' \(H_{52\%}\)

475-yr, 3-sec: 3.38' / 1.4' = 4.76' \(H_{52\%}\)
\[ R_{20\%} / H_s = (A_1 \% + G) \cdot Y_b \cdot Y_h Y_B \]

\[ Y_r = 0.9 \text{rips} \]

\[ Y_b = 0.55 \text{ grano} \]

\[ \beta = 10.24 \]

\[ \gamma = 1.5 \]

\[ h = 2.86 \]

\[ H_s = \frac{2g \cdot H_s}{g \cdot H_s \cdot (2.19) \cdot (0.22)} = 0.116 \text{ - 100-yr recurrence period} \]

\[ \frac{H_0}{L_0} = \frac{2.86}{24.57} = 0.116 \text{ - 100-yr recurrence period} \]

\[ \frac{H_0}{L_0} = 3.38 \]

\[ 27.32 = 0.124 \text{ - 475-yr recurrence period} \]

\[ \frac{\sin \theta}{\cos \theta} = \tan \phi \]

\[ \beta = 17.35^\circ \]

\[ \phi = 41.03^\circ \]

\[ \theta = 14.08^\circ \]

\[ \phi = 18.43^\circ \]

\[ y = 67.5 \pm 60 \]

\[ 50 - 55 \]

\[ \gamma = 3.5 \text{ to 1 Avg.} \]

\[ \delta = 3.1 \]

\[ 0.976 \]

\[ \delta = 3.2 \]

\[ 0.917 \]

\[ \delta = 4.1 \]

\[ 0.734 \]

\[ \delta = 3.1 \]

\[ 0.946 \]

\[ \delta = 3.2 \]

\[ 0.887 \]

\[ 0.7099 \]

Dam Safety Assessment Report (DSAR)

APPENDIX H

Page 11

100-yr recurrence interval.

3-yr, 100-yr wind speed: 60 m/s assume 2 = 135 mph gurt

10-min, 1000-yr wind speed:

\[ \frac{V_2}{V_{300}} = 1.277 + 0.296 \tanh \left(0.910 \log_\frac{45}{3} \right) \]

\[ = 1.5094 \]

\[ = 128.6 \text{ m/s} \]

\[ = 135 \text{ mph} / 1.5094 = 89.44 \text{ mph} \]

Wind speed over water (1-hr):

\[ = 107.33 \text{ mph} \]

fetch-limited velocity: \( t = 112 \text{ F} = 0.67 \text{ N} / 0.34 \)

\[ t = 112 \cdot \frac{2712.76}{5280 \text{ ft}} = 0.67 \left( \frac{107.33 \text{ mph}}{5280 \text{ ft}} \right)^{0.34} \]

\[ t = 14.62 \text{ m/s} \]

\[ \frac{U_2}{V_{300}} = 1.277 + 0.296 \tanh \left(0.910 \log \frac{45}{3} \right) \]

\[ = 1.0338 \]

\[ U_2 = 0.5891 \cdot U_2 \]

\[ V_4 = 0.5891 \cdot 110.97 \text{ mph} \]

\[ = 193.0977 \text{ mph} \]

\[ = 193 \text{ mi} / 5280 \text{ ft} = 0.1 \text{ L} \]

\[ = 283.07 \text{ ft} / 60 \text{ sec} \]

\[ = 0.6 \text{ mph} \]

\[ 416 = \frac{97^2}{27} = (32.2 \frac{45}{3}) (2.485 \text{ sec})^2 \]

\[ = 31.52' \]

\[ 416 = \frac{144}{27} = \frac{4.16}{31.52'} \]

\[ = 0.13197' \]

\[ \frac{140362}{\tan 14.0362} \]

\[ = 0.68818 > 0.5 < 5 < 20 \]

\[ v = 4.05' + 1.5' = 4.15' \]
VERIFICATION OF REQUIRED THICKNESS OF UPSTREAM/DOWNSTREAM
REVESTMENT SYSTEMS

Design for Wave Action - Via Pilarczyk (1998)

\[
\frac{H_s}{\Delta d} = \frac{F_{op}}{\delta_{op}}
\]

- \( b = 0.5 \) for permeable revetment
- \( b = 0.27 \) for semi-permeable revetment

\[
\Delta = \text{relative density} = \frac{1.89 \text{m}^3 - 10.9 \text{m}^3}{1.0 \text{m}^3}
\]

- \( F = 2.5 \) for pitched stone
- \( F = 2.25 \) for riprap
- \( F = 3.5 \) to 5.5 for block revetment
- \( F = 6 \) to 10 for concrete slabs

\( H_s = 2.9 \) 100-yr recurrence interval
\( H_s = 3.4 \) ft. 475-yr recurrence interval

\( \delta_{op} = 0.734 \) 100-yr recurrence
\( \delta_{op} = 0.7095 \) 475-yr recurrence

\[
\begin{align*}
103.63 \times 0.89 \text{m}^3 \cdot D &= 2.25 \cdot 0.7095 \times 0.5 \\
2.073 D &= 103.63 \\
1.7463 D &= 87.31
\end{align*}
\]

\( D = 50 \) cm. 20 in. 1.64 = Dref

\( D_{50} = 500 \) cm.

\( 50 \) mm. avg.

\( \frac{50 \text{ cm}}{100 \text{ cm}} = 0.5 \) mm. reg'd.

\( D_{10} : 50 \) cm. nominal, minimal

\( D_{50} : 500 \) cm. avg.

\[
\begin{align*}
\Delta t &= (1 - \gamma) \cdot \Delta \\
(1 - 0.25) &\cdot 0.8 \\
\Delta t &= 0.64 \cdot 0.6
\end{align*}
\]

\[
\begin{align*}
50 \text{ cm} &= \left( \frac{M_{50}}{P_5} \right)^{1/3} \\
50 \text{ cm} &= \left( \frac{M_{50}}{1.89 \text{m}^3} \right)^{1/3}
\end{align*}
\]
Revetment Thickness Read.

\[ D_{\text{nom}} = 50 \text{ cm} = 19.7 \text{ m} = 64.4 \text{ ft.} \]

\[ D_h = \left( \frac{M_{50}}{\rho_s} \right)^{1/3} \]

\[ 50 \text{ cm} = \left( \frac{M_{50}}{1.83 \text{ g/cm}^3} \right) \]

\[ M_{50} = 225,000 \text{ g.} \]

\[ \gamma = \frac{M}{V} \]

\[ 1.83 \text{ g/cm}^3 = \frac{225,000 \text{ g}}{V} \]

\[ V = 125,000 \text{ cm}^3 \]

\[ V = \frac{4}{3} \pi R^3 \]

\[ 125,000 \text{ cm}^3 = \frac{4}{3} \pi R^3 \]

\[ R = 31.0 \text{ cm} \]

\[ D = 2R = 62 \text{ cm.} \]

\[ 65 \text{ cm.} = 2.6 \text{ m.} \]

\[ 2.13 \text{ ft.} / 2.3 \text{ ft.} \]

\[ 0.64 \text{ to } 0.7 \text{ meter} < 2.5 \text{ m. stone per design report.} \]

\[ \checkmark \ 0.8 \text{ meter} < 2.5 \text{ m. stone.} \]

Report states that upper slope is protected with 2.5 m thin stone. \[ \checkmark \]
<table>
<thead>
<tr>
<th>Y (pcf)</th>
<th>Y (g/cm³)</th>
<th>Δ</th>
<th>D, cm</th>
<th>nom.</th>
<th>H, cm</th>
<th>R, cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>115</td>
<td>1.84</td>
<td>0.84</td>
<td>47.6</td>
<td>59.05cm</td>
<td>59.95cm</td>
<td>74.38 cm</td>
</tr>
<tr>
<td>130</td>
<td>2.08</td>
<td>1.08</td>
<td>37</td>
<td>45.9 cm</td>
<td>46.63 cm</td>
<td>57.85 cm</td>
</tr>
<tr>
<td>120</td>
<td>1.92</td>
<td>0.92</td>
<td>44</td>
<td>54.6 cm</td>
<td>54.74 cm</td>
<td>67.91 cm</td>
</tr>
<tr>
<td>125</td>
<td>2.00</td>
<td>1.00</td>
<td>40</td>
<td>49.6 cm</td>
<td>50.3 cm</td>
<td>62.48 cm</td>
</tr>
<tr>
<td>135</td>
<td>2.16</td>
<td>1.16</td>
<td>34.5</td>
<td>47.8 cm</td>
<td>43.41 cm</td>
<td>53.84 cm</td>
</tr>
<tr>
<td>140</td>
<td>2.24</td>
<td>1.24</td>
<td>32.26</td>
<td>40.6 cm</td>
<td>40.61 cm</td>
<td>50.38 cm</td>
</tr>
<tr>
<td>145</td>
<td>2.32</td>
<td>1.32</td>
<td>30.3</td>
<td>37.6 cm</td>
<td>38.15 cm</td>
<td>47.33 cm</td>
</tr>
<tr>
<td>150</td>
<td>2.40</td>
<td>1.40</td>
<td>28.6</td>
<td>35.5 cm</td>
<td>35.97 cm</td>
<td>44.63 cm</td>
</tr>
<tr>
<td>155</td>
<td>2.48</td>
<td>1.48</td>
<td>27</td>
<td>33.5 cm</td>
<td>34.03 cm</td>
<td>42.2 cm</td>
</tr>
</tbody>
</table>

\[
\frac{\text{D}}{\text{D}} = \frac{2.1889}{0.76899^{0.5}}
\]

\[
87.814 = \sqrt{\frac{N_{50}}{1.84}}
\]

\[
N_{50} = 198.44 \text{ g} \text{ cm}^3
\]

\[
y = \frac{1.84}{1.64 \text{ cm}^2} = 198.44 \text{ g} \text{ cm}^3
\]

\[
V = \frac{4}{3} \pi R^3
\]

\[
107,850 = \frac{4}{3} \pi R^3 \quad R = 29.52 \text{ cm} \quad D = 59.05 \text{ cm}
\]
DESIGN OF FILTER/TRANSITION ZONES IN DAMS:

CHARACTERISTICS OF FILTER LAYER:

BORROW MATERIAL: SHIKHLAR VILLAGE BORROW QUARRY
CONSISTING OF GRAVEL & SAND MATERIALS

\[ V = 2.0 \text{ g/cm}^3, \phi = 35^\circ, C' = 0.5 \text{ kg/cm}^2, n = 0.25-0.30 \]

\[ k = 1.5 \text{ m/day} \]

\[ k = 1.5 \times 4 \times 10^{-5} \text{ m/sec}. \]

\[ D_{50} = 0.2 \text{ mm (No. 70)} \]

\[ D_{50} = 0.21 \text{ mm} \]

\[ D_{50} = \frac{7}{8} \text{ inch} \]

\[ 2.3 \text{ cm} / 1 \text{ cm} = 22.2 \text{ mm} \]

\[ D_{50} = 50 \text{ mm} / 55 \text{ mm} \]

\[ D_{10} = 0.05 \text{ mm} \]

CHARACTERISTICS OF CLAY CORE:

\[ D_{85} = 0.05 \text{ mm} / 0.075 \text{ mm} / 0.04 \text{ mm} \]

\[ D_{50} = 0.0065 \text{ mm} / 0.008 \text{ mm} \]

\[ D_{15} = 0.0012 / 0.0011 \text{ mm} \]

Fine Migration:

\[ \frac{D_{15}}{D_{85}} < 4-5 \]

0.21 mm

0.05 mm = 4.2

0.21

0.04 mm = 5

0.21

0.075 = 3

Permeability:

\[ \frac{D_{50}}{D_{15}} > 4-5 \]

0.21

0.0012 = 175

Segregation:

\[ \frac{D_{60}}{D_{10}} < 60 \]

50 mm

0.05 mm
Check of Granular Filter Criteria

Terzaghi: $d_{50} \geq 4.5d_{15}$

- $0.21 \geq 4.5(0.0012\text{mm})$
- $0.21 \geq 0.0054 \checkmark$
- $0.15 \leq 4.5d_{85}$

- $0.1 \leq 0.345 \checkmark$
- $0.21 \leq 0.375 \checkmark$
- $0.21 \leq 0.3 \checkmark$
- $0.07 \leq 0.15$

$s_{15} \leq 4.5 \times 0.03\text{mm}$

- $0.21 \leq 0.135 \text{mm}$
- $0.07 \leq 0.075$

Filter Thickness: 1m - 2m, \( \text{1000 mm - 2000 mm} \)

- $k_f \geq 25 \text{ks}$
- $1.15 \times 10^{-5} \text{m/s sec} \geq 25 \text{ ks}$
- $1.15 \times 10^{-5} \text{m/s sec} \geq 5.78 \times 10^{-5} \text{m/s sec}$
- $25 \times 0.002 \text{m/day}$

- $0.05 \text{ m/day}$
- $1.0 \text{ m/day} \geq 0.05 \text{ m/day}$
- $7.0 \text{ m/day} \geq 0.05 \text{ m/day}$

Alt. $k_f \geq 15 \text{ks}$

- $10 \times k_f = 0.02 < 1.0 \times 10^{-9} \text{ m/sec} \checkmark$
Check of Granular Filter Criteria Via Giroud, J.P. (cont.)

\[ d_{15F} = 5 \cdot d_{85S} \quad \text{(Terzaghi)} \]

\[
\begin{align*}
0.2 & \geq 5.0.001 \\
0.2 & \geq 0.005 \\
\text{Factor} &= 200.
\end{align*}
\]

Giroud, J.P.

Retention Criterion:

\[ d_{15F} \geq \frac{d_{85S}}{} \]

\[ d_{15F} = 5 \cdot d_{85S} \]

\[ 0.21 \text{mm} \leq 5 \cdot 0.05 \text{mm} \]

\[ 0.21 \text{mm} \leq 0.25 \text{mm}. \]

\[ 0.21 \text{mm} \leq 0.2 \text{mm}. \quad \text{OK}. \]

\[ 0.2 = 5 \cdot 0.04 \text{mm} \]

\[ 0.2 \leq 0.2 \text{mm}. \quad \text{OK}. \quad \checkmark \quad \text{Good for retention.} \]

Permeability:

\[ K_F \geq \max\left( i_s, k_s, 25 \text{ks} \right) \]

\[ i_s = 3 \Rightarrow 710 \]

\[ K_F \geq 25 \cdot \text{ks} \]

\[ K_F \geq 25 \cdot 0.2 \text{ m/d}. \]

\[ K_F \geq 5 \text{ m/d}. \quad \text{Uncompacted.} \]

\[ 1.0 \text{m/d} - 7.0 \text{m/d} \geq 5 \text{ m/d}. \quad \text{Uncompacted.} \]

Computed: \( h_{	ext{max}} = 0.002 \text{ m/d} \)

Compressible, \( h_{	ext{max}} \leq 0.002 \text{ m/d} \)

Keeps shear stress less than 2\% of total stress of 10.04 m/d.
### APPENDIX H

#### SECOND NATIONAL WATER SUPPLY & SANITATION PROJECT (SWSSP) OF REPUBLIC OF AZERBAIJAN

**Dam Safety Assessment Report (DSAR)**

---

**U.S. STANDARD SIEVE OPENING IN INCHES**

<table>
<thead>
<tr>
<th>100</th>
<th>80</th>
<th>60</th>
<th>40</th>
<th>30</th>
<th>20</th>
<th>16</th>
<th>12</th>
<th>10</th>
<th>8</th>
<th>6</th>
<th>5</th>
<th>4</th>
<th>3</th>
<th>2</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100</td>
<td>80</td>
<td>60</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>16</td>
<td>12</td>
<td>10</td>
<td>8</td>
<td>6</td>
<td>5</td>
<td>4</td>
<td>3</td>
<td>2</td>
</tr>
</tbody>
</table>

**U.S. STANDARD SIEVE NUMBERS**

- 100
- 80
- 60
- 40
- 30
- 20
- 16
- 12
- 10
- 8
- 6
- 5
- 4
- 3
- 2
- 1

**GRAIN SIZE IN MILLIMETERS**

<table>
<thead>
<tr>
<th>500</th>
<th>100</th>
<th>50</th>
<th>10</th>
<th>5</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**PERCENT FINER BY WEIGHT**

- 100
- 90
- 80
- 70
- 60
- 50
- 40
- 30
- 20
- 10
- 5
- 1

**PERCENT COARSER BY WEIGHT**

- 0.001
- 0.01
- 0.015
- 0.02
- 0.03
- 0.05
- 0.1
- 0.5
- 1
- 1.5
- 2
- 2.5
- 3
- 5
- 10
- 20
- 30
- 40
- 50
- 60
- 70
- 80
- 90
- 100

---

**APPENDIX H**

<table>
<thead>
<tr>
<th>COBBLES</th>
<th>GRAVEL</th>
<th>SAND</th>
<th>2. PROJECT</th>
<th>3. AREA</th>
<th>4. BORING NUMBER</th>
<th>5. DATE (YYYYMMDD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ELEVATION OR DEPTH</td>
<td>CLASSIFICATION</td>
<td>4%</td>
<td>Project Name</td>
<td>Area</td>
<td>Boring Number</td>
<td>Date</td>
</tr>
<tr>
<td>SAMPLE NUMBER</td>
<td>1.</td>
<td>2.</td>
<td>3.</td>
<td>4.</td>
<td>5.</td>
<td></td>
</tr>
<tr>
<td>1.</td>
<td>Layer 2</td>
<td>Alluvial sediments of</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(ALUVIAL</td>
<td>the Vilesh River consisting</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SEDIMENTS)</td>
<td>of sand, gravel, and</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>various stones</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**GRADATION CURVES**

For use of this form, see EM 1110-2-1908; the proponent agency is CECW-EG.

---

**ENG FORM 2087, 1 MAY 1963**

PREVIOUS EDITIONS ARE OBSOLETE. VERSION 1.0
APPENDIX H


Dam Safety Assessment Report (DSAR)

U.S. STANDARD SIEVE OPENING IN INCHES

U.S. STANDARD SIEVE NUMBERS

PERCENT FINER BY WEIGHT

PERCENT COARSER BY WEIGHT

GRAIN SIZE IN MILLIMETERS

COBBLES

GRANULAR

SAND

COARSE

MEDIUM

FINE

SAMPLE NUMBER

ELEVATION OR DEPTH

CLASSIFICATION

NAT W %

LL

PL

PI

2. PROJECT

Dam Safety Assessment for Vileshehyan Water Reservoir

SILT OR CLAY

3. AREA

Full

Barren

Layer 5 - Foundation "CUP"

4. BORING NUMBER

Varies

5. DATE (YYYYMMDD)

For use of this form, see EM 1110-2-1906; the proponent agency is CECW/EG.

ENG FORM 2087, 1 MAY 1963

PREVIOUS EDITIONS ARE OBSOLETE.

VERSION 1.0
### Dam Safety Assessment Report (DSAR)

**APPENDIX H**

**Second National Water Supply & Sanitation Project (SWSSP) of Republic of Azerbaijan**

#### Table 1: Gradation Curves

<table>
<thead>
<tr>
<th>COBBLES</th>
<th>GRAVEL</th>
<th>SAND</th>
<th>V = 1.9 - 2.0 (\frac{y}{cm^3})</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAMPLE NUMBER</td>
<td>ELEVATION OR DEPTH</td>
<td>CLASSIFICATION</td>
<td>NAT W%</td>
</tr>
<tr>
<td>KHALFALAR</td>
<td>GRAVEL-SAND</td>
<td>y = 0.15 - 0.30</td>
<td></td>
</tr>
<tr>
<td>ARKIVAN</td>
<td>FILL MATERIALS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>VILLAGE</td>
<td>EBORROW QUARRIES</td>
<td></td>
<td></td>
</tr>
<tr>
<td>QUARRIES</td>
<td>MAIN FIELDS</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

SILT OR CLAY: \(k = 1.0 - 7.0 \text{ m}^3/\text{day}\)

For use of this form, see EM 1110-2:1905; the proponent agency is CECW-EG.

---

**ENG FORM 2087, 1 MAY 1963**

PREVIOUS EDITIONS ARE OBSOLETE.
APPENDIX H

GRADATION CURVES

For use of this form, see EM 1110-2-1906; the proponent agency is CECW-EG.

ENG FORM 2087, 1 MAY 1983

PREVIOUS EDITIONS ARE OBSOLETE.
### Bottom Tunnels - Normal Operating Level Scenario

<table>
<thead>
<tr>
<th>Available Data</th>
<th>Friction Loss Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (m)</td>
<td>297</td>
</tr>
<tr>
<td>Height (m)</td>
<td>5.0</td>
</tr>
<tr>
<td>Width (m)</td>
<td>4.5</td>
</tr>
<tr>
<td>Outlet Invert (m)</td>
<td>43.5</td>
</tr>
<tr>
<td>Top of Pipe (m)</td>
<td>48.5</td>
</tr>
<tr>
<td>Normal Water Level (masl)</td>
<td>75.5</td>
</tr>
</tbody>
</table>

#### Bends

| Number                  | 2                         |
|                        | Assuming 45 degree long radius |

| Roughness Coef. (ε)     | 1.20E-03                  |
|                        | 2.53E-04                  |
|                        | 0.014                     |
|                        | 6.65                      |
|                        | 100                       |
|                        | 21.19                     |

| Note: Velocity values iterated in table above to make it same as Velocity in energy conservation table |

<table>
<thead>
<tr>
<th>Minor Losses</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>k</td>
<td>0.2</td>
</tr>
<tr>
<td>Entrance Loss</td>
<td>0.5</td>
</tr>
<tr>
<td>Exit Loss</td>
<td>1.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Energy Conservation</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow Contraction at outlet</td>
<td>0.95</td>
</tr>
<tr>
<td>Head at Outlet</td>
<td>47.55</td>
</tr>
<tr>
<td>Head Difference</td>
<td>27.95</td>
</tr>
<tr>
<td>Friction Loss</td>
<td>6.65 m</td>
</tr>
<tr>
<td>Minor Losses</td>
<td>13.94 m</td>
</tr>
<tr>
<td>Velocity Head</td>
<td>7.36 m</td>
</tr>
<tr>
<td>Velocity</td>
<td>12.03 fps</td>
</tr>
<tr>
<td>Flow (Q) per pipe</td>
<td>270.32 m/3</td>
</tr>
<tr>
<td>Total flow (bottom tunnels)</td>
<td>540.64 m/3</td>
</tr>
</tbody>
</table>

---

**Source**

- Table 16.1 Lindeburg 11th Ed
- Eq 16.22(a) Lindeburg 11th Ed
- Table 17.2 Lindeburg 11th Ed
- Eq 17.21 Lindeburg 11th Edition
- Eq 17.28 Lindeburg 11th Edition
- Appendix 17-A Lindeburg 11th Ed
- Eq. 17.30 Lindeburg 11th Edition
### Bottom Tunnels - Maximum Water Level Scenario

#### Available Data

<table>
<thead>
<tr>
<th>Two Identical Concrete Pipes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
</tr>
<tr>
<td>Height</td>
</tr>
<tr>
<td>Width</td>
</tr>
<tr>
<td>Outlet invert</td>
</tr>
<tr>
<td>Outlet pipe top</td>
</tr>
<tr>
<td>Maximum Water Level</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bends</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number</td>
</tr>
</tbody>
</table>

Assuming 45 degree long radius

#### Friction Loss Calculation

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic Diameter (De)</td>
<td>4.74 m</td>
<td>15.54 ft</td>
</tr>
<tr>
<td>Velocity (initial), V</td>
<td>12.32 m/s</td>
<td>40.42 fps</td>
</tr>
<tr>
<td>Kinematic Viscosity (ν)</td>
<td>1.00E-06 m²/s</td>
<td>(at 20°C)</td>
</tr>
<tr>
<td>Reynolds Number (Re)</td>
<td>5.81E+07</td>
<td></td>
</tr>
<tr>
<td>Roughness Coef. (ε)</td>
<td>1.20E-03 m Concrete</td>
<td></td>
</tr>
<tr>
<td>Epsilon/Dia. (ε/D)</td>
<td>2.53E-04</td>
<td></td>
</tr>
<tr>
<td>Friction factor (f)</td>
<td>0.0344</td>
<td></td>
</tr>
<tr>
<td>Head Loss - Darcy</td>
<td>7.00 m</td>
<td></td>
</tr>
<tr>
<td>Hazen William C</td>
<td>100 (design value)</td>
<td></td>
</tr>
<tr>
<td>Head Loss - Hazen-Williams</td>
<td>22.24 ft</td>
<td>6.78 m</td>
</tr>
</tbody>
</table>

Note: Velocity values iterated in table above to make it same as Velocity in energy conservation table

#### Minor Losses

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>k</td>
<td>hF (m)</td>
</tr>
<tr>
<td>Bend Loss</td>
<td>0.2</td>
</tr>
<tr>
<td>Entrance Loss</td>
<td>0.5</td>
</tr>
<tr>
<td>Exit Loss</td>
<td>1</td>
</tr>
</tbody>
</table>

#### Energy Conservation

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow Contraction at outlet</td>
<td>0.95 m</td>
</tr>
<tr>
<td>Head at Outlet</td>
<td>47.55 m</td>
</tr>
<tr>
<td>Head Difference</td>
<td>29.45 m</td>
</tr>
</tbody>
</table>

| Head Difference = Velocity Head + Head Losses |
|---|---|
| Friction Loss | 7.00 m | Using Darcy Head Loss |
| Minor Losses | 14.70 m | Calculated Above |
| Velocity Head | 7.74 m |
| Velocity | 12.32 fps |
| Flow (Q per pipe) | 277.35 m³/s |
| Total flow (bottom tunnels) | 554.69 m³/s |
### DERIVATION PIPES - NORMAL OPERATING LEVEL SCENARIO

<table>
<thead>
<tr>
<th>Available Data</th>
<th>Friction Loss Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Derivation Pipes - Two Identical Pipes</strong></td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td>300 m 984.24 ft</td>
</tr>
<tr>
<td>Diameter</td>
<td>1.22 m 4.00 ft</td>
</tr>
<tr>
<td>Outlet Invert</td>
<td>48.5 m asl</td>
</tr>
<tr>
<td>Top of Pipe</td>
<td>49.72 m asl</td>
</tr>
<tr>
<td>Normal Water Level</td>
<td>75.5 m asl</td>
</tr>
<tr>
<td>Bends</td>
<td></td>
</tr>
<tr>
<td>Number</td>
<td>2</td>
</tr>
<tr>
<td><strong>Assuming 45 degree long radius</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Source</strong></td>
<td></td>
</tr>
<tr>
<td>Table 16.1 Lindeburg 11th Ed</td>
<td></td>
</tr>
<tr>
<td>Starting with initial value</td>
<td></td>
</tr>
<tr>
<td>Eq 16.22(a) Lindeburg 11th Ed</td>
<td></td>
</tr>
<tr>
<td>Table 17.2 Lindeburg 11th Ed</td>
<td></td>
</tr>
<tr>
<td>Eq 17.21 Lindeburg 11th Edition</td>
<td></td>
</tr>
<tr>
<td>Eq 17.28 Lindeburg 11th Edition</td>
<td></td>
</tr>
<tr>
<td>Eq. 17.30 Lindeburg 11th Edition</td>
<td></td>
</tr>
<tr>
<td><strong>Note:</strong> Velocity values iterated in table above to make it same as Velocity in energy conservation table</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Minor Losses</strong></th>
<th><strong>hf (m)</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Bend Loss (2)</td>
<td>0.2</td>
</tr>
<tr>
<td>Entrance Loss</td>
<td>0.5</td>
</tr>
<tr>
<td>Exit Loss</td>
<td>1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Energy Conservation</strong></th>
<th><strong>Value</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow Contraction at outlet</td>
<td>0.24 m</td>
</tr>
<tr>
<td>Head at Outlet</td>
<td>49.48 m</td>
</tr>
<tr>
<td>Head Difference</td>
<td>26.02 m</td>
</tr>
<tr>
<td>Head Difference = Velocity Head + Head Losses</td>
<td></td>
</tr>
<tr>
<td>Friction Loss</td>
<td>14.05 m</td>
</tr>
<tr>
<td>Minor Losses</td>
<td>7.35 m</td>
</tr>
<tr>
<td>Velocity Head</td>
<td>4.13 m</td>
</tr>
<tr>
<td>Velocity</td>
<td>9.00 fps</td>
</tr>
<tr>
<td>Flow (Q per pipe)</td>
<td>10.52 m³/s</td>
</tr>
<tr>
<td>Total flow (derivation pipes)</td>
<td>21.03 m³/s</td>
</tr>
</tbody>
</table>

| **Paper Source**          | **Data from** |

*Flow values iterated in table above to make it same as Velocity in energy conservation table.*
### DERIVATION PIPES - MAXIMUM WATER LEVEL SCENARIO

<table>
<thead>
<tr>
<th>Available Data</th>
<th>Friction Loss Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Derivation Pipes - Two Identical Pipes</strong></td>
<td><strong>Hydraulic Diameter (De)</strong></td>
</tr>
<tr>
<td>Length</td>
<td><strong>Velocity (initial), V</strong></td>
</tr>
<tr>
<td>Diameter</td>
<td><strong>Kinematic Viscosity (ν)</strong></td>
</tr>
<tr>
<td>Outlet Invert</td>
<td><strong>Reynolds Number (Re)</strong></td>
</tr>
<tr>
<td>Top of Pipe</td>
<td><strong>Roughness Coef. (ε)</strong></td>
</tr>
<tr>
<td>Max Water Level</td>
<td><strong>Epsilon/De, (ε/De)</strong></td>
</tr>
<tr>
<td>Bends</td>
<td><strong>Friction factor (f)</strong></td>
</tr>
<tr>
<td>Number</td>
<td><strong>Head Loss - Darcy</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Hazen-Williams C</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Head Loss - Hazen-Williams</strong></td>
</tr>
</tbody>
</table>

Note: Velocity values iterated in table above to make it same as Velocity in energy conservation table.

<table>
<thead>
<tr>
<th>Minor Losses</th>
<th>( k )</th>
<th>( hf ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bend Loss (2)</td>
<td>0.2</td>
<td>1.75</td>
</tr>
<tr>
<td>Entrance Loss</td>
<td>0.5</td>
<td>2.19</td>
</tr>
<tr>
<td>Exit Loss</td>
<td>1</td>
<td>4.37</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Energy Conservation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow Contraction at outlet</td>
</tr>
<tr>
<td>Head at Outlet</td>
</tr>
<tr>
<td>Head Difference</td>
</tr>
<tr>
<td>Head Difference = Velocity head + Head Losses</td>
</tr>
<tr>
<td>Friction Loss</td>
</tr>
<tr>
<td>Minor Losses</td>
</tr>
<tr>
<td>Velocity Head</td>
</tr>
<tr>
<td>Velocity</td>
</tr>
<tr>
<td>Flow (Q per pipe)</td>
</tr>
<tr>
<td>Total flow (derivation pipes)</td>
</tr>
<tr>
<td>Spillway</td>
</tr>
<tr>
<td>----------</td>
</tr>
<tr>
<td>550</td>
</tr>
<tr>
<td>931 (95)</td>
</tr>
<tr>
<td>1000</td>
</tr>
</tbody>
</table>

**Design Spillway LAPP**

\[ v = \left( \frac{1000}{n} \right)^{0.25} \sqrt{\frac{L}{S_0}} \]

\[ v = \frac{1000}{0.016} \left( \frac{950}{1.31} \right)^{0.25} \sqrt{0.0273} \]

\[ v = 22.783 \text{ cm/s} \]

\[ \alpha = 1093.6 \text{ m}^2 \text{ sec for } n = 0.016 \]

\[ \alpha = 1093.6 \left( \frac{2.4}{10} \right)^{0.25} \sqrt{0.016} \]

\[ \alpha = 18.51 \text{ m}^2 \text{ sec for } n = 0.016 \]

\[ \alpha = 888.5 \text{ m}^2 \text{ sec for } n = 0.016 \]

**Conclusion:** Spillway should accommodate the 100-year discharge without the inclusion of the bottom tunnels. Require a cross-sectional area of \( 50 \text{ m}^2 \) to accommodate 100-year discharge assuming \( v = 0.016 \text{ cm/s} \).

\[ S_0 \text{ is actually } 0.0273 \]

Entrance channel \( L = 190 \text{ m} \)

Downstream section \( L = 1000 \text{ m} \)

\[ S_0 = \frac{70 \text{ m} + 40 \text{ m} + 2 \text{ m}^2}{1190 \text{ m}} = 0.02756 = S_0 \sqrt{S_0} = 0.166 \]

<table>
<thead>
<tr>
<th>Spillway</th>
<th>Capacity</th>
<th>Width, W (m)</th>
<th>Depth, D (m)</th>
<th>Net Efficient Area (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>550</td>
<td>8</td>
<td>4.05</td>
<td>3.09</td>
<td>32.5</td>
</tr>
<tr>
<td>8931 (95)</td>
<td>9.8</td>
<td>4.9</td>
<td>3.7</td>
<td>48 ( \checkmark )</td>
</tr>
<tr>
<td>1000</td>
<td>10.2</td>
<td>5.1</td>
<td>3.84</td>
<td>52 ( \checkmark )</td>
</tr>
<tr>
<td>590</td>
<td>7.5</td>
<td>3.75</td>
<td>2.85</td>
<td>28 ( \checkmark )</td>
</tr>
<tr>
<td>100</td>
<td>9.05</td>
<td>4.525</td>
<td>3.92</td>
<td>40.95 ( \checkmark )</td>
</tr>
</tbody>
</table>
SETTLEMENT/CONSOLIDATION OF LAYERED CLAYS BELOW SPILLWAY & LEFT DAM BANKS

\[ q = 2.5 - 3 \text{ kN/m}^2 \]

\[ q' = q' \]

\[ \Delta = \text{7 meters (4 m away along length)} \]

\[ C_c = 0.26/1.20 \quad \phi = 13 - 16^\circ \]

\[ C_s = 0.01 \quad \varepsilon = 0.8 \text{kN/m}^2 \]

\[ \gamma_a = 1.6 \text{ kN/m}^3 \quad \gamma_f = 0.4 \]

\[ Y_{sat} = 1.9 \text{ kN/m}^3 \quad k_n = 0.2 \text{ in/day} \quad k_{mp} = 0.002 \text{ in/day} \]

\[ = 35 \text{ m/} \]

\[ \text{Rock} \]

\[ H_v' = (81 \text{m-70m})(1.6 \frac{c}{c^2}) + (1.9 \frac{c}{c^2} - 1.5 \frac{c}{c^2})(1800 \text{cm}) \]

\[ 1760 \frac{c}{c^2} + 1620 \frac{c}{c^2} = 3380 \frac{c}{c^2} = \gamma_c' \]

\[ p = (100 \text{cm})(800 \text{ cm})(2.99 \frac{c}{c^2}) + (10 \text{cm})(2 \text{ cm})(2.99 \frac{c}{c^2}) + (100 \text{cm})(500 \text{cm})(2.49 \frac{c}{c^2}) \]

\[ 3840 \frac{c}{c^2} + 400 \frac{c}{c^2} = 4240 \frac{c}{c^2} \]

\[ = 4240 \frac{c}{c^2} \]

\[ \frac{w}{\text{buoyant weight:}} \]

\[ p = \frac{1}{2}(108 \text{cm})(1000 \text{cm})(2.5 \frac{c}{c^2} - 1.5 \frac{c}{c^2}) + (1000 \text{cm})(200 \text{cm})(3.4 \frac{c}{c^2} - 10 \frac{c}{c^2}) \]

\[ = 224000 \frac{c}{c^2} + 280000 \frac{c}{c^2} = 504000 \frac{c}{c^2} \]

\[ = 1.3 \frac{c}{c^2} \text{buoyant} \]

\[ \frac{\text{Concrete Weight}}{(2 \text{cm})(8 \text{cm})(2.40 \frac{c}{c^2})} = 41550 \frac{c}{c^2} \]

\[ \frac{\text{Water Weight}}{(1.9 \frac{c}{c^2})(2 \text{cm})(8 \text{cm})} = 2080 \text{Kg/m}^3 \]

\[ 1.1 \text{Kg/cm}^2 \]

\[ (1 \text{m})(8 \text{m})(2.40 \frac{c}{c^2}) \]

\[ = 51140 \frac{c}{c^2} \]

\[ = 1.8 \text{Kg/cm}^2 \text{ max} = \rho_{\text{max}} \]
SETTLEMENT OF SPILLWAY CURVE

\[
\frac{(100 \text{ cm})(900 \text{ cm})^{2}}{(2.40 \text{ km}^3 - 1.0 \text{ km}^3)} + \frac{(12 \text{ m})(10 \text{ m})}{(2.40 \text{ km}^3 - 1.0 \text{ km}^3)}
\]

\[
257,000 \text{ m}^3 + 280,000 \text{ m}^3
\]

\[
700,000 \text{ m}^3
\]

\[
Z = 1,232,000 \text{ m}^3
\]

\[
\text{Pressure area of spillway} = \frac{1,232,000 \text{ m}^3}{1200 \text{ m}} = 1027 \text{ m}^2
\]

\[
1.03 \text{ m}^2 \text{ per min} \leq
\]

\[
(\text{100 km})(900 \text{ km})(2)(2.40 \text{ km}^3) + (\text{260 km})(1000 \text{ km})(2.40 \text{ km}^3)
\]

\[
+ 700,000 \text{ km}^3
\]

\[
= 1,612,000 \text{ km}^3
\]

\[
= 1.34 \text{ km}^3 \text{ per min} \leq
\]

\[
(\text{200 km})(900 \text{ km})(2)(2.40 \text{ km}^3) + (\text{200 km})(1000 \text{ km})(2.40 \text{ km}^3)
\]

\[
+ 700,000 \text{ km}^3
\]

\[
= 2,284,000 \text{ km}^3
\]

\[
2,284,000 \text{ km}^3 - (1.9 \text{ km}^3 - 1.0 \text{ km}^3)
\]

\[
\frac{1300 \text{ km}}{1.9 \text{ km}^3}
\]\n
\[
= \frac{2.86 \text{ km}^2}{2.85 \text{ km}^2}
\]

\[
\text{Concrete}
\]

\[
(\text{300 km})(2)(900 \text{ km})(2.40 \text{ km}^3 - 1.0 \text{ km}^3)
\]

\[
+ (\text{260 km})(300 \text{ km})(2.40 \text{ km}^3 - 1.0 \text{ km}^3)
\]

\[
+ (1.9 \text{ km}^3)(260 \text{ km})(700 \text{ km})
\]

\[
= \frac{3.414,000 \text{ km}^3}{3000 \text{ km}}
\]

\[
= 113.9 \text{ km}^3
\]

\[
= 1.14 \text{ km}^2 \leq 1.5 \text{ km}^2
\]
Dam Safety Assessment Report (DSAR)

APPENDIX H

\[ P = 2.5 \times 1 = 2.5 \, \text{kN/m}^2 \]
\[ P = 3 \times 1 = 3 \, \text{kN/m}^2 \]
\[ P_{avg} = 1.5 \times 2.5 = 3.7 \, \text{kN/m}^2 \]

\[ \Delta V = \frac{a \times b}{(a + b)(b + 2)} \]

\[ T_v = \frac{3 \times 15}{(12 + 17)(1 + 17)} \]
\[ T_v = 0.07, \quad T_l = 0.05, \quad T_s = 0.06 \, \text{kN/m}^2 \]

\[ T_v = \frac{a \times b}{(a + b)(b + 2)} \]

\[ \Delta T = T_v + T_l + T_s \]

\[ T_v = \frac{3400 \text{ cm}}{1 + 0.07} \left( \frac{3380 \text{ kN/m}^2 + 70.5 \text{ kN/m}^2}{3380 \text{ kN/m}^2} \right) = 4.71 \text{ cm}, \quad 1.85'' \]

\[ T_l = 47.1 \text{ mm}, \quad 1.85'' \]

\[ T_s = 40.5 \text{ cm}, \quad 1.5'' \]

\[ \Delta V = \frac{a \times b}{(a + b)(b + 2)} = 3400 \text{ cm}, \quad 0.10 \log \left( \frac{1}{a} \right) \]

\[ C_e = \frac{a \times b}{a \times b - 0.02} \]

Spillway will settle due to both primary and secondary settlement factors.

Dam Safety Assessment Report (DSAR)

APPENDIX H

Page 3

\[ \Delta T = \frac{a \times b}{(a + \pi)(b + \pi)} \]

\[ = \frac{3^{1/2} \text{cm}^2}{(12 + 1.3)(1 + 1.3)} = 0.09 \text{ cm} \]

\[ P_c = \frac{H_i}{1 + e^p} \left( C \log \left( \frac{Y_0 + \Delta Y}{Y_c} \right) \right) \]

\[ = \frac{2850 \text{ cm}}{1 + 0.40} \left( (0.26) \log \left( \frac{3627.5 \text{ cm}^2 + 70 \text{ cm}^2}{3627.5 \text{ cm}^2} \right) \right) \]

\[ = 4.72 \text{ cm} = 1.96 \text{ in.} \]

\[ \phi = \frac{C_p}{1 + e^p} H_p \log \left( \frac{t}{t_p} \right) \]

\[ = \left( \frac{0.051}{1 + 0.28} \right) 2845 \text{ cm} \log 50 \Rightarrow 41.5 \text{ cm.} \]

\[ 59.8 \text{ cm} \]

\[ 23 \text{ in.} \]

\[ 2.3 \text{ in.} \]

\[ 54.54 \text{ cm} = 2.16 \text{ in.} \]

\[ 53.71 \text{ cm} = 2.11 \text{ in.} \]

\[ \text{differential settlement} = \frac{100}{\text{yr.}} \]

\[ = 4 \times 10^{-7} \text{ in.} \]

\[ = 0.036 \% \approx 0.04 \% \text{ possible low.} \]

Therefore, no cracking expected. However, spillway capacity could be

\[ \text{reduced by about 0.55 meters.} \]

\[ 1.63 \text{ m or about 1.8 to 2 feet.} \]

\[ \text{differential settlement} = \frac{73.3 \text{ cm} - 62.12}{20.9} \]

\[ = 5.29 \times 10^{-4} \text{ in.} \]

\[ 0.053 \% \text{ over 475 years.} \]

\[ \text{or about } 2 \text{ to } 2.4 \text{ ft over 475 years.} \]
APPENDIX I

Results of Hydrological/Hydraulic Analyses
Vilashchay Water Reservoir in Masalli, Azerbaijan

Outflow (cubic meters per second) and water level (meters)

Inflow (cubic meters per second)

Vileshchay Water Reservoir Hydrological/Hydraulic Analysis

APPENDIX I


Dam Safety Assessment Report (DSAR)
Vileshehay Dam Monthly Inflow
Historical Record (1948 to 1985) Compared to Current (2000 to 2014)
Vilashchay Water Reservoir in Masalli, Azerbaijan

plot date: 4/8/2016
file: FreqCurves_ver306 Vileshehay Synthetic.xlsm

exceedance probability
flow (cms)

plot position: Weibull
recurrence interval = 100/probability
<table>
<thead>
<tr>
<th>recurrence (years)</th>
<th>Q (cms)</th>
<th>Q$_5$ (cms)</th>
<th>Q$_{95}$ (cms)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000</td>
<td>311</td>
<td>416</td>
<td>249</td>
</tr>
<tr>
<td>500</td>
<td>281</td>
<td>370</td>
<td>228</td>
</tr>
<tr>
<td>200</td>
<td>243</td>
<td>313</td>
<td>200</td>
</tr>
<tr>
<td>100</td>
<td>216</td>
<td>273</td>
<td>180</td>
</tr>
<tr>
<td>50</td>
<td>189</td>
<td>235</td>
<td>160</td>
</tr>
<tr>
<td>25</td>
<td>163</td>
<td>198</td>
<td>140</td>
</tr>
<tr>
<td>20</td>
<td>155</td>
<td>187</td>
<td>133</td>
</tr>
<tr>
<td>10</td>
<td>129</td>
<td>152</td>
<td>113</td>
</tr>
<tr>
<td>5</td>
<td>104</td>
<td>119</td>
<td>92</td>
</tr>
<tr>
<td>3.333</td>
<td>88</td>
<td>100</td>
<td>79</td>
</tr>
<tr>
<td>2.5</td>
<td>77</td>
<td>86</td>
<td>69</td>
</tr>
<tr>
<td>2</td>
<td>67</td>
<td>75</td>
<td>60</td>
</tr>
<tr>
<td>1.667</td>
<td>59</td>
<td>66</td>
<td>53</td>
</tr>
<tr>
<td>1.429</td>
<td>51</td>
<td>57</td>
<td>45</td>
</tr>
<tr>
<td>1.250</td>
<td>43</td>
<td>49</td>
<td>38</td>
</tr>
<tr>
<td>1.111</td>
<td>34</td>
<td>39</td>
<td>29</td>
</tr>
<tr>
<td>1.053</td>
<td>28</td>
<td>32</td>
<td>23</td>
</tr>
<tr>
<td>1.020</td>
<td>22</td>
<td>27</td>
<td>18</td>
</tr>
<tr>
<td>1.010</td>
<td>19</td>
<td>23</td>
<td>15</td>
</tr>
</tbody>
</table>
## Vilashchay Water Reservoir in Masalli, Azerbaijan

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weir Crest</td>
<td>77 masl</td>
</tr>
<tr>
<td>Weir Length</td>
<td>120 meters</td>
</tr>
<tr>
<td>1000-year inflow</td>
<td>311 cms</td>
</tr>
<tr>
<td>100-year inflow</td>
<td>316 cms</td>
</tr>
<tr>
<td>Design Report Spillway Flow</td>
<td>240 cms</td>
</tr>
<tr>
<td>Starting water level</td>
<td>75.5 masl</td>
</tr>
<tr>
<td>Rainfall</td>
<td>1.1 meters</td>
</tr>
<tr>
<td>Water Level</td>
<td>76.6 masl</td>
</tr>
<tr>
<td>Dam Crest</td>
<td>80 meters</td>
</tr>
<tr>
<td>Max Weir Spillway Flow</td>
<td>958 cms</td>
</tr>
<tr>
<td>1000-Year Stage</td>
<td>78.4 masl</td>
</tr>
<tr>
<td>100-Year Stage</td>
<td>78.1 masl</td>
</tr>
<tr>
<td>Design Report 1000 Year Stage</td>
<td>78.3 masl</td>
</tr>
</tbody>
</table>

### Weir Spillway Calculations

\[ Q = CH^{3/2} \]

\[ C = 1.6 \]

### Cross Section Diagram

- **Max. Controllable Water Level**: 77 masl
- **Entrance Channel**: Width unspecified
- **Discharge Part of Chute Spillway**: Width unspecified
- **Down Stream**: Width unspecified
- **Up Stream**: Width unspecified
APPENDIX J

The PLAN
of the Vilashchay Reservoir’s Operations Office on civil defence measures for the year of 2016
## 2016

<table>
<thead>
<tr>
<th>#</th>
<th>Name of the event</th>
<th>Responsible executors</th>
<th>Involved persons</th>
<th>January</th>
<th>February</th>
<th>March</th>
<th>April</th>
<th>May</th>
<th>June</th>
<th>July</th>
<th>August</th>
<th>September</th>
<th>October</th>
<th>November</th>
<th>December</th>
<th>Note on implementation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>I. Measures on civil defence office</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Control to organization and rendering methodological assistance to civil defence measures to be held at local executive power agencies, institutions, enterprises and organizations</td>
<td>Head of the civil defence office</td>
<td>Heads of division, service and unit of the Civil Defence (CD) offices. Heads of the Regional Emergency and CD division</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>During the year</td>
</tr>
<tr>
<td>2</td>
<td>Control to conducting laboratory tests of the existing Personal Protection Equipment (PPE) at offices and enterprises located in the territory covered by the Emergency and CD offices</td>
<td>Head of the division on radiation, chemical and medical-biological protection of population and territories</td>
<td>Chief of CD staff of city (region) objects</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>During the year</td>
</tr>
</tbody>
</table>
### II. Measures of Lenkaran regional Emergency and CD division

<table>
<thead>
<tr>
<th>Civil defence training of the object</th>
<th>Head of the Lenkaran Regional Emergency and CD division</th>
<th>Head of the Object. Commander-chief staff. Civil defence forces, personnel not involved to the forces</th>
<th>15</th>
</tr>
</thead>
</table>

### III. Measures on the Executive Power of the Masalli region

<table>
<thead>
<tr>
<th>1</th>
<th>Organization of methodological meeting on results of 2016 academic year and challenges ahead in the year of 2017</th>
<th>Head of the Executive power of Masalli region</th>
<th>Senior staff based on the decision of head of the Region's executive power</th>
<th>December</th>
</tr>
</thead>
</table>

### IV. Measures of the Operations office of the Vilashchay Reservoir of Masalli region

<table>
<thead>
<tr>
<th>1</th>
<th>Preparation of the object for conducting CD training</th>
<th>Head of the Operations office of the Vilashchay Reservoir</th>
<th>Senior management staff. Emergency Committee, CD headquarters</th>
<th>17</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Assembly and warning training with non-militarized CD forces</td>
<td>Head of the Operations office of the Vilashchay Reservoir</td>
<td>Personnel of the Office's non-militarized CD forces</td>
<td>22</td>
</tr>
<tr>
<td>3</td>
<td>Organization of events in connection with the &quot;World Civil Defence Day&quot;</td>
<td>Head of the Operations office of the Vilashchay Reservoir, Masalli region</td>
<td>Employees of the office</td>
<td>01</td>
</tr>
<tr>
<td>4</td>
<td>Training of senior staff on 15 hour program</td>
<td>Head of the Operations office of the Vilashchay Reservoir</td>
<td>Senior staff of the office</td>
<td>In accordance with the training schedules</td>
</tr>
<tr>
<td>#</td>
<td>Activity</td>
<td>Responsible Person</td>
<td>Additional Details</td>
<td></td>
</tr>
<tr>
<td>----</td>
<td>-------------------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Training of the non-militarized forces' personnel of CD on 15 hours program</td>
<td>Head of the Operations office of the Vilashchay Reservoir</td>
<td>Personnel of the Office's non-militarized CD forces</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>In accordance with the training schedules</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Training of employees not being part of non-militarized forces of CD on 12 hours program</td>
<td>Head of the Operations office of the Vilashchay Reservoir, Masalli region</td>
<td>Employees that are not part of non-militarized CD forces of the office</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>In accordance with the training schedules</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Training of senior staff at Central Civil Defence courses of the Ministry of Emergency Situations (MES)</td>
<td>Head of the Operations office of the Vilashchay Reservoir, Masalli region</td>
<td>Senior staff of the office</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Based on the combined plan of the Central CD courses</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Submission of monthly information to the CD headquarters at the Region's Executive Power (REP) about the training process on CD</td>
<td>Head of the Operations office of the Vilashchay Reservoir, Masalli region</td>
<td>CD headquarters of the office</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>until the 25th date of each month</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Presenting the annual final information on the state of CD to the CD headquarters of the REP</td>
<td>Head of the Operations office of the Vilashchay Reservoir, Masalli region</td>
<td>CD headquarters of the office</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>December</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Organization of methodological meeting on results of 2016 academic year and challenges ahead in the year of 2017</td>
<td>Head of the Operations office of the Vilashchay Reservoir, Masalli region</td>
<td>Senior staff of the office. EC, CD headquarters. Commander-chief staff</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>December</td>
<td></td>
</tr>
</tbody>
</table>

Head of the Civil Defence HQ's of the Vilashchay Reservoir’s operations office, Masalli region:  

Mahir Aliyev
The Civil Defence PLAN of operations office of the Vilashchay Reservoir of Masalli region

Part 1: Brief geographical and socio-economical characteristics, probable conditions

1.1. Economical characteristics:

Masalli region is has 50 km border with Lerik region on the south-west, 21 km with Jalilabad region on the north-west, 35 km with Neftchala region on the north and 35 km with Jalilabad, Lenkaran regions.

 Territory of Masalli region is 721 km$^2$, population number is 211877. It is a sub-tropical region.

The mountainous part of the region and the western part are covered with forests. The forest area is 16.6 hectares.

The Vilashchay reservoir is located at the region. The territory of reservoir is 520 hectares, water volume is 46 million cubic meters and height of the dam is 28 meters.

Masalli Vilashchay reservoir is one of the important objects located in the Masalli region. The reservoir is located at the Qeribler village of the region and the main objective is to ensure irrigation of agricultural crops. Employees of the office serve as persons on duty at the reservoir. Besides this, the reservoir is guarded by the police officers.

The four stored water release building has been built on the Masalli Vilashchay reservoir. The reservoir is working on continuous operating mode. Number of workers is 21 and an overall number the office’s employee is 205 persons.

“The Emergency Situations” of the Vilashchay reservoir is carrying out its operations at the permanent work place.

1.2. There is no explosive and dangerous to fire substances at operations office of the Masalli regions Vilashchay reservoir.

1.3. Short assessment of probable situation in case of natural calamities, industrial accidents and destructions:

- Destructions and landslides can happen at the office of administrative building and the dam due to earthquake. As well as communication lines can be damaged. As a result
of destruction of the dam, certain part of the region can get under water. In this case, death and injury can happen among employees of the office and the population.

- Fire can happen in cases of when equipments are not properly used and security measures not taken while use of electric sources. In case of fire, the communication equipment and documents can be burnt down, injures and death can happen among the employees.

- The territory of office and employees can be faced with huge destructions and loss in case of enemy’s attack with conventional weapons and weapons of mass destructions. The electricity network is totally or partly gets out of order, big human casualties can happen due to destruction and explosion, there is a probability of secondary damaging factor sources of fire and poisoning.

- Unstable epidemiological and low level sanitary condition can be a source for contagious diseases and consequently, employees of the office can be infected to these contagious diseases. In case of outbreak of mass contagious diseases among employees, it’s considered to immediately locate them in Medical institutions.

1.4 The prevention measures are being carried out in order to prevent natural calamities, industrial accidents, destructions and also in order to decrease the volume of measures, rescue and other urgent actions on protections of employees’ material and cultural resources.

Part II: Implementation of measures in situations of natural calamities, industrial accidents, destruction and in cases of their threats.

1. The measures taken in case of threat of natural calamities, industrial accidents and destructions (higher level of readiness mode).

When head of the CD office (chairman of the emergency commission) receives the warning alarm about the threat of emergency occurrence, he/she applies higher level of readiness mode. In order to timely inform employees about the emergency situation taken place or possibly to happen and carrying out measures to prevent outcome of the emergency situation, the proper information is obtained from the management of relevant agencies, Emergency, CD and data is exchanged.

Depending on the situation, implementation of the below mentioned measures are organized:

- To notify the objects’ employees and forces, assembly of senior staff and provision of precise tasks to them within 30 minutes;
- Gathering, processing and exchange of information about the emergency situation within 1.5 hours;
- Carrying out exploration and observation in the territory of the object for 1 hour;
- Arrangement of daily duty of the senior staff within 15 minutes;
- To put on alert non militarized CD forces of 40 persons within 1 hour;
- To specify the emergency and civil defence plan within 20 minutes;
- To make ready the health post to provide medical aid to victims within 30 minutes;
- Arrangement of personnel’s nutrition by the senior staff;
- The warehouse of the office is prepared for distribution of personal protection equipment by the distribution station;
- The motor vehicles and area for resettlement of people at the courtyard of the office are made ready by head of the civil defence forces (CDF);
- Anti-epidemic and health protection measures are carried out for employees of the office by doctors of the Masalli central hospital;
- Preparatory measures are carried out for trouble-free suspension of production (the work process) and anti-fire prevention by the fire brigade, superintendent and the CDF head;

The warning on occurrence of the emergency situation is issued by the responsible person on duty with use of loudspeakers and telephone channels.

Recommendations on rules of conduct in specific extreme situations and on emergency threat cases are provided in each and every message transmitted.

Based on the information and data received from the Emergency and CD management agencies and relevant higher authorities, the local situation is assessed, forecasting is being carried out and proposals are worked out to prevent outbreak of the emergency situation.

The control dispatcher service is reinforced with involvement of workers of the security service.

The automobile transport is made ready. Medical and anti-epidemic protection measures are carried out with support from the Masalli central hospital. Fire protection measures are organized.

Life support systems are made ready for accident free suspension.

Protection installations are made ready to accommodate employees.

2. Measures to be taken when natural calamities, industrial accidents and destructions happen (emergency mode)

The warning is in the case of emergency situation is issued in accordance with scheme. Investigation is conducted to clarify correctness of the decision made earlier about carrying out works to study the situation in the emergency area, rescue and other urgent measures, based on the investigation data the real situation is assessed and further developments are forecasted.

The non militarized civil defence forces consisted of 40 people and 6 teams are brought to the area of rescue and other urgent work, in order to eliminate consequences of emergency situation.

Special attention is given to medical aid and rescue works for protection of workers from damaging factors during emergency situations. Besides the health staff of unit, workers of medical institutions located nearby are planned to involve to the implementation of treatment and rescue measures.

It is important to teach first aid skills to employees at the civil defence lessons.

When the fire starts in the object:

- Immediately calling the city fire service via 101 or 112 phone numbers;
- Head of CDF, communication unit and members of the fire unit, arranges warning of the employees, and evacuation of them from dangerous area within 5 minutes;
- Organization of assembly of senior staff (members of emergency commission), informing them about the situation and making tasks;
- Head of CDF organizes investigation of fire source together with fire unit within 10 minutes;
- Assessment of situation and making the decision to extinguish fire within 5 minutes, based on investigation data;
- Extinguishing of fire is done by fire unit under the guidance of head of the CDF;
- The protection of public order unit, rescue unit, health post, fire unit, communication unit and distribution station for personal protection equipment are made ready;
- Health post establishes medical aid station to provide aid for those injured from the fire and carbon monoxide;
- Head of CD organizes mutual action with neighbourhood objects’ forces for rendering support to extinguish the fire.

Part III. Provision of resources involved to the implementation of the rescue and other urgent works and carrying out measures on protection of workers, material and cultural resources.

Prevention of emergency situations and ensuring operation of resources involved to elimination of its outcomes, are the main important conditions in successful execution of rescue works and are the main tasks of the emergency commission. Successful execution of rescue works is ensured by full and timely preparation for emergency situations, continuous control of the situation in surrounding area and the area of accident as well as by medical, technical, material and metrological provisions.

Investigation/exploration

Investigation is organized by the emergency and civil defence headquarters in order to identify characteristics and the level of destruction of the communication network and engineering facilities, the volume, priority and methods of exploration, rescue and other urgent actions as well as changes in the environment (chemical poisoning, radioactive contamination). In order to conduct general exploration, an operative group is created with involvement of specialists from the emergency commission and a task is given to explore the situation in the damaged area and in surrounding territory.

In order to obtain full information about the general condition, chairman of the emergency commission organizes mutual action with local Emergency and CD management agencies.

Medical provision

First aid is carried out by means of medical staff of the Masalli central hospital, resources of city emergency aid station, as well as by means provision of self and mutual aid.

Provision of medical aid is considered by doctors of Masalli central hospital and hospital reception of the central hospital located in the region.

Hygiene and anti-epidemic measures are provided with assistance of doctors of the local hygiene and epidemiological centre and the Masalli central hospital.

Preventive measures are provided by the medical staff of the Masalli central hospital under the guidance of heads of the iodine prophylaxis structural units.

Transportation provision

Despatching rescue forces and facilities to the damaged area are provided with transport of Masalli passenger OJSC’s. Evacuation of people and despatching to medical institutions also planned with this transportation.

Material provision
Arranges timely and full provision of CD forces and employees with necessary possessions, products, water, fuel, lubricants and other means. Solution of material welfare is assigned to the head of material technical supply office.

All reserved material technical supply is used and relevant measures are taken to immediately fill the lacking reserves. Material means are located in the reserve warehouse. CD personnel are provided with meals/food.

**Meteorological provision**

Collections of information on the weather condition, dangerous changes in hydrometeorological regime, and on other information in emergency situations, for the purpose of conducting assessment of situation and making decisions, is organized by the headquarters of Emergency and CD. Based on the level of the emergency situation tasks are given for taking measures for security of employees in emergency condition as well as in protection of material and spiritual wealth.

Employees are provided with personal protection equipment and provided with instruction for use.

Transmission of warning alarms, information on rules of conduct ensures timely implementation of protection measures.

**Conduction of rescue and other urgent works on health of employees and elimination of danger to the life of employees and restoration of life support systems.**

The main task in the process of elimination of outcomes of the emergency situation is to save people, provision of medical aid, as well as limitation of fire and accident cases.

For the purpose of assessment of emergency situation outcome, explorations are conducted by specialist of the civil defence exploration forces that have better understanding of the area and the location of engineering communication places.

Operations group is formulated from the members of the emergency commission for general exploration. The operation group specifies the condition, location of the injured, their number and the volume of destruction.

CD forces immediately start rescue and other urgent works, if needed, technical service specialists are involved.

The following civil defence forces are created for carrying out rescue and urgent works:

- Protection of public order unit – 1/8
- Rescue unit – 1/15
- Health post – 1 /4
- Fire unit – 1 /6
- Communication unit – 1 /4
- PPE distribution station – 1 /3

The purpose of rescue works is to find out the injured, rescue and provision of firs medical aid. Rescuers clear out destruction area, make a way to hiding places, take out those injured and render medical aid to those needed.

Besides the civil defence forces, other units and services are involved into urgent rescue works (takes part in fire extinguish, eliminates damages in communal energy industry, in water supply and sanitation network and etc.).
In order to support civil defence forces, other forces not part of civil defence are also involved.

The mutual activity of resources involved in prevention of emergency situations and elimination of its outcome.

In the process of organization of mutual activity, the rule of obtaining and exchange of information about the emergency situation is defined.

Initial information about the emergency situation in neighbouring object can enter from emergency and civil defence division, emergency commission of Masalli region or from other object’s emergency commissions.

Organization of mutual communication in this manner, allows carrying out measures to inform employees and students, prevention of state of emergency without delay and elimination of outcomes.

Part IV. Management of operations and of the implemented activities during the emergency situations

During the emergency situations management of activities is planned at the management point and the civil defense manager (chairperson of the emergency commission) manages the operations on elimination of the consequences of emergency.

The management point is composed of the members of emergency commission department and technical service specialists able to make decisions.

An operation group is sent to the place of accident out of members of emergency commission with the aim of assessing the situation and preparing proposals.

Based on the decision of the civil protection manager (chairperson of the emergency commission) a communication is organized to ensure non-stop and continuous management and interaction between forces and means during the implementation of rescue works.

Wired and mobile communication means are used to facilitate the management.

Civil defense manager (chairperson of the emergency commission) controls the distribution of information based on an urgent (termed) information table.

If a threat of emergency situation is popping up or happening, a responsible person on duty informs the management of the emergency commission. Other senior staff members, services and commander-chief officers of the civil protection groups are informed by a person on duty at night, and managers of the related structural departments at daytime.

Civil protection manager (chairperson of the emergency commission) implements the general management of the rescue and other urgent works.

During the elimination of the consequences of emergency the management of activities is implemented by civil defense service managers of an object (working apparatus of emergency commission) via the usage of operating daily communication channels and radio-telephone communication.

Notification of the department employees is implemented by a dispatcher service on duty based on notification scheme. In case of an unavailability of the telephone communication radio
communication means should be used in the very first turn to establish a communication with an emergency area.

Attachments:

1. Calendar plan of key activities in case of a threat or occurrence of a natural calamity, industrial accident and destruction

2. Supposed conditions, or situation that might occur during the emergency

3. A report on forces and means involved in the activities on prevention of emergency situations and elimination of their consequences in case of a threat or occurrence of a natural disaster, industrial accident and destruction

4. The table on provision of employees with personal protection equipment, chemical exploration and dosimetrical control devices

5. The table on provision of employees with collective protection equipment

6. The organisation of management, notification and communication, in case of a threat or occurrence of a natural calamity, industrial accident and destruction

Head of civil defence headquarters Mahir Aliyev
Attachment 6. The scheme for organization of management, notification and communication in case of a threat or occurrence of a natural calamity, industrial accident and destruction

Duty of Water and Irrigation Ministry
Phone: 493 – 11 - 76

Duty operation office of CD MES
Phone: 492 – 07 - 11

Lenkaran regional Emergency and CD division
Phone: 4 – 12 - 67

Head of the Vilashchay Reservoir’s operations office, Masalli region:
Aghabey Aghayev
Address: Seybatin village
Phone: 5 – 31 – 87 office, 050 325 – 58 – 91 (cell)

Head of civil defence headquarters
Mahir Aliyev

Assistant to head of CDF:
Valiyev Ilham Ibad
Address: Shixlar village
Phone: 21 – 41–6-34 (office)
070 337 – 76 - ** (cell)

Chairman of the emergency commission:
Aliyev Mahir Arif
Address: Xocavar village
Phone: 21 – 79 – 7 - 32 (office)
050 342 – 53 – 24 (cell)

Deputy head of CDF:
Aghabeyov Sadraddin Qurbaqulu
Address: Sharafa village
Phone: 050 631 – 44 – * (cell)

Region Emergency Commission
Phone: 5 – 31 – 13

Workers and servants

Head of CDF:
Rufullayev Dayyan Rufulla
Address: Chaxirli village
Phone: 050 525 15 22 (cell)
The REPORT of resources involved for eradication of outcome and prevention of emergency situations, natural calamities, industrial accidents and accidents’ threat and at their occurrence.

<table>
<thead>
<tr>
<th>#</th>
<th>Name of structural units</th>
<th>general forces</th>
<th>well prepared forces</th>
<th>from them</th>
<th>Special forces</th>
<th>well prepared forces</th>
<th>from them</th>
<th>specialize d forces</th>
<th>well prepared forces</th>
<th>from them</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>num. prs.</td>
<td>num. prs.</td>
<td>object</td>
<td>territory</td>
<td>num. prs.</td>
<td>object</td>
<td>num. prs.</td>
<td>num. prs.</td>
<td>num. prs.</td>
</tr>
<tr>
<td>1</td>
<td>Operation office of the Vilashchay reservoir</td>
<td>6   40</td>
<td>- -</td>
<td>6   40</td>
<td>- -</td>
<td>- -</td>
<td>- -</td>
<td>- -</td>
<td>- -</td>
<td>- -</td>
</tr>
</tbody>
</table>

Head of civil defence headquarters Mahir Aliyev
The table on provision of employees with personal protection equipment, chemical exploration and dosimetal control devices

<table>
<thead>
<tr>
<th>#</th>
<th>Names of structural units</th>
<th>Number of employees</th>
<th>Personal protection equipment</th>
<th>Chemical exploration and dosimetal control devices</th>
<th>Medical tools</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Gas mask (in accordance with production characteristics)</td>
<td>Protection clothing (in accordance with production characteristics)</td>
<td>Roentgenmeter</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Respirator</td>
<td></td>
<td>Required</td>
</tr>
<tr>
<td>1</td>
<td>Operations office of the Vilashchay Reservoir</td>
<td>201</td>
<td>60</td>
<td>30</td>
<td>201</td>
</tr>
</tbody>
</table>

Head of civil defence headquarters

Mahir Aliyev