The Afghanistan Engineering Support Program assembled this deliverable. It is an approved, official USAID document. Budget information contained herein is for illustrative purposes. All policy, personal, financial, and procurement sensitive information has been removed. Additional information on the report can be obtained from Firouz Rooyani, Tetra Tech Sr. VP International Operations, (703) 387-2151.

ENGINEERING SUPPORT PROGRAM

WO-LT-0009-002

Bamyan Dams, Afghanistan

Shikari Dam and Hydropower Project

Draft Task 3 Report

December 4, 2011

This publication was produced for review by the United States Agency for International Development. It was prepared by Tetra Tech, Inc.

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December 4, 2011

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Re: WO-LT-0009-002 Bamyan Dams, Afghanistan, Draft Task 3 Report

[redacted]

Enclosed is the draft Task 3 Report prepared for WO-LT-0009 Amendment 2 Bamyan Dams – Shikari Site.

I look forward to meeting with you to discuss this report.

Respectfully,

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Cc: [redacted] (USAID-OIEE)

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AFGHANISTAN ENGINEERING SUPPORT PROGRAM

WO-LT-0009-002
BAMYAN DAMS, AFGHANISTAN
SHIKARI DRAFT TASK 3 REPORT

December 4, 2011

Table of Contents

List of Tables

List of Figures

ABBREVIATIONS AND ACRONYMS

AESP	Afghanistan Engineering Support Program
ASTM	American Society of Testing and Materials
BH	Boring Hole
ES	Executive Summary
Dr	Relative Density
FDT	Field Density Test
GP	Poorly Graded Gravel (general group name – UCS System)
GM	Silty Gravel (general group name – UCS System)
Gs	Specific Gravity
GWL	Groundwater Level
GSI	Geological Strength Index
LL	Liquid Limit
NHWL	Normal High Water Level
NAVFAC-DM	Naval Facilities Engineering Command Design Manual
OL	Organic Silt (general group name – UCS System)
PL	Plastic Limit
PI	Plastic Index
PMF	Probable Maximum Flood
Q	Rock Mass Quality
RCC	Roller Compacted Concrete
RQD	Rock Quality Designation
RMR	Rock Mass Rating
Se	Settlement
SM	Silty Sand (general group name – UCS System)
SPT	Standard Penetration Test
Su	Shear Strength
USAID	United States Agency for International Development
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
USCS	Unified Soil Classification System
W	Moisture Content (%)

UNITS OF MEASUREMENT

One jerib = 0.195 ha.(5 jerib = 1 ha. approx.) = 0.49 acres
One foot = 0.3048 meters
One hectare = 10,000 square meters
One acre foot = 1233 cubic meters
One McM or Mm^3 = 1 million cubic meters = 811 acre feet
One cubic meter = 1000 liters
One metric ton = 2205 pounds = 1.1 imperial tons = 1,000 kilograms = 1,000,000 grams
One cubic foot per second = 0.02831 cubic meters per second
One Newton = 0.001 kilonewtons = 1 kilogram per square second = 0.22481 pound-force
Once pascal (Pa) = 1.0×10^{-6} megapascals (mPa) = 1 Newton per square meter
One g (gravitational force) = 9.81 meters per square second

1.0 Executive Summary

Several dam alternatives have been analyzed for hydropower generation at the Shikari Valley site, just east of the town of Bamyan in central Afghanistan. Due to site conditions and constraints, a concrete gravity diversion dam has been recommended for the site. The dam will have an uncontrolled broad crested weir spillway at the existing stream alignment. The foundation for the dam will sit on a prepared bedrock surface after excavation of the overlying soils and weathered bedrock. The dam will have a low level sluiceway that can be drained into the stilling basin, and a higher level outlet drop inlet structure through the dam that will discharge into a concrete settling basin tower. The settling basin will have a sump to allow sediment and coarse grained material to settle out prior to entering the penstock. Additionally, a small discharge outlet has been provided in the tower to discharge minimum downstream flows not entering the penstock. Water flow can be manually controlled with the use of gates at all pipe entrances. A summary of the dam features is shown in Table 1.

The project would also involve a steel penstock pipe, or alternatively, a concrete lined tunnel, to carry flows from the dam to the powerhouse for a distance of approximately 480 meters. A powerhouse would be constructed downstream of the tunnel. The powerhouse will be a steel-framed, metal siding superstructure erected on top of a concrete substructure.

The gross head for hydropower production is assumed to be approximately 14 meters. Considering a 90 percent plant operating efficiency, and a design discharge of 5.0 m^3/s, the maximum theoretical capacity is calculated to be 0.6 MW. The annual energy production is calculated to be 3,061 megawatt-hour (MWH), and the annual average number of days without generation is estimated to be 82 days.

In addition to the low head, and ultimately low power generation capacity, sediment management would also be a complication for this project site due to the estimated high sediment loads from the 2,057 km^2 watershed basin contributing to the site. For these reasons, a low-head dam at the selected project site is not recommended; however, a geotechnical analysis was completed to further assess this recommendation.

The analysis in the report is based on a conceptual level (15%-20%) design and will need to be updated prior to construction of the project. Further information is required, as detailed in Section 8.0 of this report.

TABLE 1
SUMMARY OF FEATURES FOR SHIKARI DAM

Main Dam	
Type:	Concrete Gravity Low Head Diversion
Crest Elevation:	2338 m
Crest Width:	4 m
Crest Length	15 m
Streambed Elev. At Dam Axis	2329.0 m
Lowest Foundation Elev.:	2319.0 m
Valley Floor Elevation at D/S Toe:	2329.0 m
Dam height (dam crest to natural ground surface below crest):	9.0 m
Hydraulic Height (spillway crest to lowest point of original streambed):	5.5 m
Structural Height (dam crest to lowest point in foundation):	19.0 m
Reservoir Storage (at spillway crest):	N/A[1]
Spillway	
Type:	Broad-crested weir
Crest Elevation:	2334.5 m
Spillway Crest Length	15.0 m
Depth at the Spillway Crest	3.5 m
Conveyance at 3.5 m depth	127.4 cms
Design storm:	500-year
Additional freeboard	0.25 m
Outlet Works	
Type:	Gated Intake Structure and Concrete Open Channel
Invert Elevation at Intake Structure:	2432.0 m
Capacity with water surface at spillway crest elevation:	TBD[2]
Low Level Sluice	
Type:	Steel Pipe
Invert Elevation at Intake Structure:	2329.0 m
Capacity with water surface at spillway crest elevation:	TBD[2]

[1] Survey contours do not extend enough upstream of the dam to calculate storage of the proposed reservoir
[2] Capacities of outlet works and low-level sluice to be determined in the next phase

2.0 Introduction

In response to the Afghanistan Engineering Support (AESP) Task Order with USAID (WO-LT-0009-002), Tetra Tech has conducted a Geotechnical Engineering Model to fulfill Task 3 of the June 28, 2011 (Revision 3) Scope of Work for the Shikari dam site. Task 3 is as follows:

> *"After a proposed dam type, size, and configuration is established at a selected site, seepage and stability modeling will be performed using computer programs developed for evaluating proposed dam structures (SEEP/W and SLOPE/W). Slope stability analyses will be performed for static and seismic loads on the dam. Computer simulation runs will be made in order to optimize a final size and configuration for the selected dam type that will provide long-term dam stability under anticipated operational conditions. The simulated analysis will be presented to USAID in a report format for their consideration in determining future dam investigations."*

3.0 Background

According to Tetra Tech's *Engineering Support Program – Preliminary Geological Dam Site Assessment Report Bamyan Province (WO-A-0061)* dated November 25, 2010; the Shikari Valley was identified as a viable location for a hydropower generating facility. The Shikari dam site is approximately 18 km east of the town of Bamyan, as shown in Figure 1. Topographic survey and a geotechnical investigation have been completed per Tasks 1 and 2 of the Scope of Work. Boring Logs from the *Final Geotechnical Report for Shikari Dam Site* completed by Geo Search have been included in Appendix A of this report.

This report includes a summary of the work completed by Tetra Tech for Task 3 for the evaluation of alternative dam configurations and conceptual design of the recommended low head concrete gravity diversion dam with a small reservoir and minimal impact to the existing roadway. Preliminary design includes geologic data, geotechnical analyses, hydrology and hydraulics analysis, conceptual engineering design, and stability and seepage analyses. Additionally, data gaps have been identified within each section in order to provide information that needs to be addressed prior to proceeding to a more detailed level of design. The engineering concepts discussed in this report will be further detailed in conjunction with Tasks 4-6.

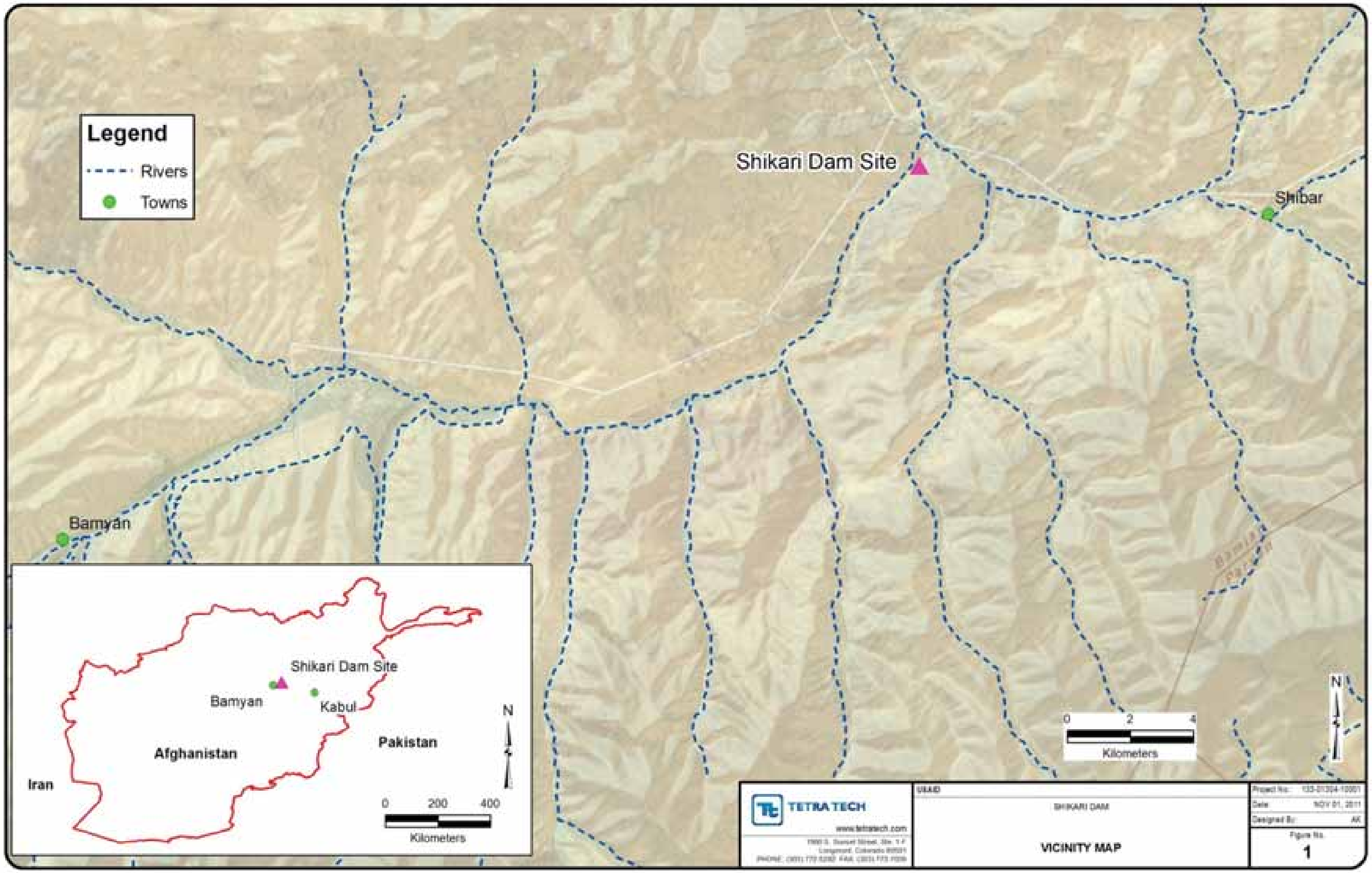


Figure 1 Vicinity Map

4.0 Site Conditions

The site visit, preliminary field and laboratory tests, and initial analyses show that the site conditions for the recommended location of the dam include the following considerations:

- The proposed site is in a U-shaped valley with moderately deep soil deposits (9 to 11 meters) over poor quality bedrock, which has been identified as mica schist. The soils found mainly classify as silty sand and gravels with cobbles, as well as some areas of soft clay lenses. Specifically, the first 2-5 meters of foundation are gravely materials, followed by a lens of silty to clayey soils. In general, the silty to clayey soils have low blow counts, as little as 4 blows per 15 centimeters of penetration, in reference to the Standard Penetration Tests (SPT). Detailed boring logs have been provided in Appendix A of this report. The generally flat valley bottom has steep to near vertical slopes in the west abutment and relatively gradual slope of intact colluvium and bedrock in the east abutment. The soils above the bedrock will be susceptible to settlement under load and, potentially, liquefaction under earthquake loading. If used for foundation, further evaluation of the soils will need to be performed.

- The project region has a high seismic potential with peak ground accelerations estimated between 34% g to 65% g. Liquefaction and deformation of foundation soils for earth-fill and rock-fill alternatives, and stability of gravity dams must be considered in this highly seismic potential region.

- As a result of the limited flow data available (less than 10 years), and the uncertainty of the accuracy and reliability of the data collection procedures, determining a precise flood discharge in the river is not possible. However, based on the available historic data, the 500-year flood peak discharge has been estimated to be approximately 120 (calculated as 116.5) m^3/s.

- Sediment records for Afghanistan, shown in Table 4, indicate potentially large volumes of sediment flow through the project site each year. The sediment composition was not available, but considering the materials found in the borings, it could be inferred that the sediments are mostly sands and silty sands. Gravels and potentially larger materials such as tree logs may be expected in event of higher flood flows. Considering the potential for a large volume of sediment deposits, a small reservoir size, and the necessity to minimize sediment passing through the turbines, sediment management is a primary concern at the site, and must be a design and operational consideration.

Borrow material studies were not specifically conducted; however, limited soil borrow material is available from the dam site. Considering the geology of the site, it may be difficult to obtain aggregate materials locally. Further studies are required to show that aggregate materials are available within a reasonable hauling distance.

5.0 Alternatives

An alternative dam type evaluation was performed by comparing capabilities of several dam types with the conditions at the site. A recommended dam type is presented for these conceptual designs.

5.1 Gravity Dam

A gravity dam can be composed of several materials, depending on the site conditions, project size, and the availability of construction materials and resources. The types to be considered for the Shikari site include; conventional concrete, masonry, and roller compacted concrete (RCC). Gravity dams are generally triangular in cross-section with a vertical upstream face and sloping downstream face. They are generally founded on bedrock due to the rigid behavior of concrete, masonry, and RCC. The downstream face is sloped to provide ease of construction and sufficient dam mass to meet stability requirements. The crest width of a gravity dam is selected to provide access to the dam and its outlet works equipment, as well as to meet stability requirements. The crest elevation is selected to provide sufficient spillway surcharge and access for the dam abutments.

Conventional concrete and masonry dams are typically selected for projects with difficult site access and relatively small volumes of concrete. RCC dams are cost efficient for larger dams where earthmoving equipment can place RCC rapidly, with little influence from the dam site or crest width. Minimum crest widths for RCC dams range from 5 to 7 meters, whereas crest widths for conventional concrete and masonry dams can be as little as 1 meter. Masonry dams are considered if local labor is available for a low cost, and includes stone masons that can provide construction technical expertise. Masonry dams typically take longer to construct than either conventional concrete or RCC gravity dams.

Outlet works for gravity dams typically include an intake structure on the vertical upstream face, a conduit through the dam, either on the bedrock foundation or within the dam mass, and a discharge to either the stream channel (for a sluiceway or for stream discharge) or to a conduit (for a hydropower penstock). Conduit length for gravity dams are relatively short compared to the conduit lengths for earth fill or rock fill dams. The cost savings associated with shorter conduit lengths can be significant.

One advantage of gravity dams is the ability to locate the spillway on the dam, near the original stream channel, thereby eliminating the need for a spillway on the dam abutment. The spillway is designed for a specific flood discharge and includes a crest, chute, and energy dissipater. The crest of a gravity dam can typically provide an erosion resistant structure that can be overtopped by flows exceeding spillway capacity for short durations. This is beneficial for extreme or unpredicted floods.

5.2 Rock Fill Dam

A rock-fill dam is composed largely of fragmented rock with an impervious core. The core is separated from the rock shells by a series of transition zones built of properly graded material. A membrane of concrete, asphalt, or steel plate on the upstream face should be considered in lieu of an impervious earth core only when sufficient impervious material is not available.

It is often desirable to determine the best methods of construction and compaction on the basis of test quarry and test fill results. Free-draining, well-compacted rock fill can be placed with steep slopes if the dam is on a rock foundation. If it is necessary to place rock-fill on an earth or weathered rock foundation, the slopes must be flatter, and transition zones may be required between the foundation and the rock fill. Materials for rock-fill dams range from sound free-draining rock to the more friable materials such as sandstones and silt-shales that break down under handling and compacting to form an impervious to semipervious mass. The latter materials, because they are not completely free-draining and lack the shear strength of sound rock fill, are often termed “random rock” and can be used successfully for dam construction, but, because of stability and seepage considerations, an embankment design using such materials is similar to that of earth dams.

Outlet works for rock fill dams include an intake structure at the upstream toe, a conduit through the dam on the bedrock foundation, and a discharge to either the stream channel (for a sluiceway or for stream discharge) or to a conduit (for a hydropower penstock). In cases where the foundation bedrock is deep, location of the outlet works near an abutment, where the bedrock is shallow, is generally considered. It is good practice to not locate the outlet works on a soil foundation beneath a rock fill dam, due to consolidation of the foundation soils and the associated settlement of the conduit.

Additionally, it is good practice to locate open channel spillways off of the dam on rock abutments or on a gravity concrete dam section located within the rock fill dam. Spillways have been located on rock fill dams by providing an erosion resistant spillway crest, chute, and stilling basin across the crest of the dam; however, these types of spillways are typically constructed for low recurrence floods, and are considered to have higher risk. Conduit spillways are typically considered for rock-fill dams where abutment conditions do not provide an efficient location for an open channel spillway. Tower intakes connected to conduits, founded like outlet conduits have been used on many dams; however, the capacity of these type structures must be carefully selected to reduce the potential for dam overtopping during extreme flows or unpredicted floods.

5.3 Earth Fill Dam

An earth fill dam is composed of suitable soils obtained from borrow areas and compacted in layers by mechanical means. Following preparation of the foundation, earth from borrow areas are transported to the site, dumped, spread in layers of specified depth, and compacted. One advantage of an earth dam is that it can be adapted to a weak foundation, such as the silty sandy soils at this site, provided proper consideration is given to the foundation capability to support the earth fill, resist underseepage, and remain stable under earthquake loading.

Outlet works for earth fill dams include an intake structure at the upstream toe, a conduit through the dam on the bedrock foundation, and a discharge to either the stream channel (for a sluiceway or for stream discharge) or to a conduit (for a hydropower penstock). In cases where the foundation bedrock is deep, location of the outlet works near an abutment, where the bedrock is shallow, is generally considered. It is good practice to not locate the outlet works on a soil foundation beneath a rock-fill dam, due to consolidation of the foundation soils and the associated settlement of the conduit.

Earth fill dams are significantly less erosion resistant than rock-fill dams if overtopped by a flood exceeding the spillway design. In addition, spillways over earth fill dams subject the embankment to erosion if the spillway concrete does not perform as designed and exposes the underlying fill to erosion. Therefore, it is good practice to locate open channel spillways off of earth fill dams on rock foundations/abutments or on a gravity concrete dam section located within the earth fill dam. Spillways have been located on earth fill dams by providing an erosion resistant spillway crest, chute, and stilling basin across the crest of the dam; however, these types of spillways are typically constructed for low recurrence interval floods. Conduit spillways are typically considered for earth fill dams, where abutment conditions do not provide an efficient location for an open channel spillway. Tower intakes connected to conduits, founded like outlet conduits, have been used on many dams; however, the capacity of these types of structures must be carefully selected to reduce the potential for dam overtopping during extreme flows or unpredicted floods.

5.4 Selected Alternative – Concrete Gravity Dam

Considering the cost impacts for each alternative, and due to the site conditions, including weak foundation soils and poor underlying bedrock, a concrete gravity dam was recommended for the project site. The concrete gravity dam is an erosion resistant structure, and thus can handle discharges exceeding spillway capacity, in case of extreme flood events.

Although the earthen dam option reduces the amount of foundation excavation required, extremely poor quality foundation soil at the site creates a risk of substantial differential foundation settlement that can cause seepage through the outlet structures and eventually result in dam failure. Vertical rock masses exist about 40 meters downstream of the proposed dam in a narrow river section, creating a threat of head cutting and erosion migrating upwards to the dam embankment. As a result, the spillway is difficult to locate within the narrowing valley, as well as problematic in providing stilling for the relatively large design flows. Such a design to prevent head cutting is possible; however, will increase costs dramatically. For these reasons, this alternative was decided to be unsuitable for the project.

A conceptual layout of the concrete gravity dam and earthen dam were completed as part of the analysis; however, only a concrete gravity dam has been shown in the Drawings in Appendix E of this report.

The dam design concept is discussed further in Section 6.3.2.

6.0 Engineering Analysis

6.1 Geology

6.1.1 Existing Reports

The geological documents reviewed consisted of:

- Geological and Mineral Resource Map of Afghanistan, US Department of the Interior, USGS, 2006.
- Final Geotechnical Report for Shikari Dam Site. Prepared by Geo Search Geotechnical Lab Department, 2011.

Following is a summary of the subsurface data.

6.1.2 General Geologic Setting

Geologic conditions in the vicinity of the site, as summarized by the United States Geological Survey (USGS), and confirmed by a site geological investigation, consist of conglomerate, sandstone, mica schist, and slate, which are locally faulted. Photographs taken at the site (Appendix B), along with the boring logs suggest mica schist bedrock is present at the dam site. The bedrock forms steep cliffs at each abutment.

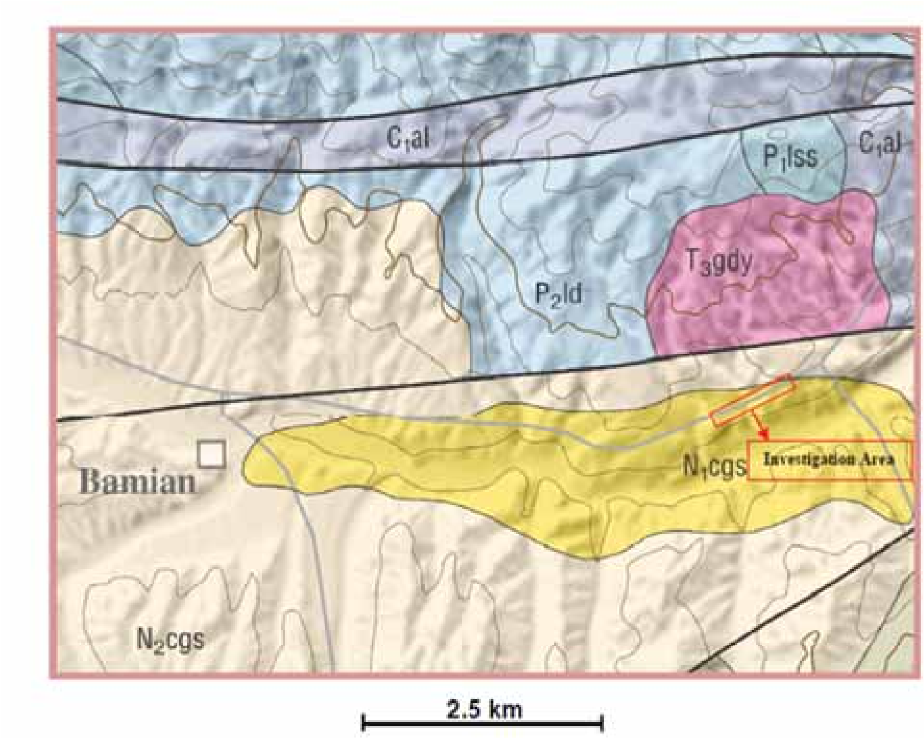


Figure 2 shows the geological location of the site. Descriptions of mapped geologic units are as follows:

N_2cgs *Conglomerate and sandstone (Pliocene)—Gray conglomerate, grit, sandstone more abundant than siltstone, clay, limestone, marl; gypsum, salt; felsic to mafic volcanic rocks*

N_1cgs *Conglomerate and sandstone (Miocene)—Red conglomerate, sandstone more abundant than siltstone, clay; felsic and mafic volcanic rocks; limestone, marl; olivine basalt, trachybasalt, andesitic basalt (Taywara Series).*

T_3gdy *Granodiorite and granosyenite (Late Triassic)—Granodiorite, granosyenite, granophyre, granite.*

P_2ld *Limestone and dolomite (Late Permian)—Limestone, dolomite more abundant than marl, conglomerate, sandstone, siltstone, shale, bauxite and bauxite-bearing rocks.*

P_1lss *Limestone and sandstone (Early Permian)—Limestone and sandstone more abundant than siltstone, argillite, slate*

C_1al *Andesite lava (Early Carboniferous)—Rhyolite to basalt volcanic rocks more abundant than limestone, shale, sandstone, conglomerate*

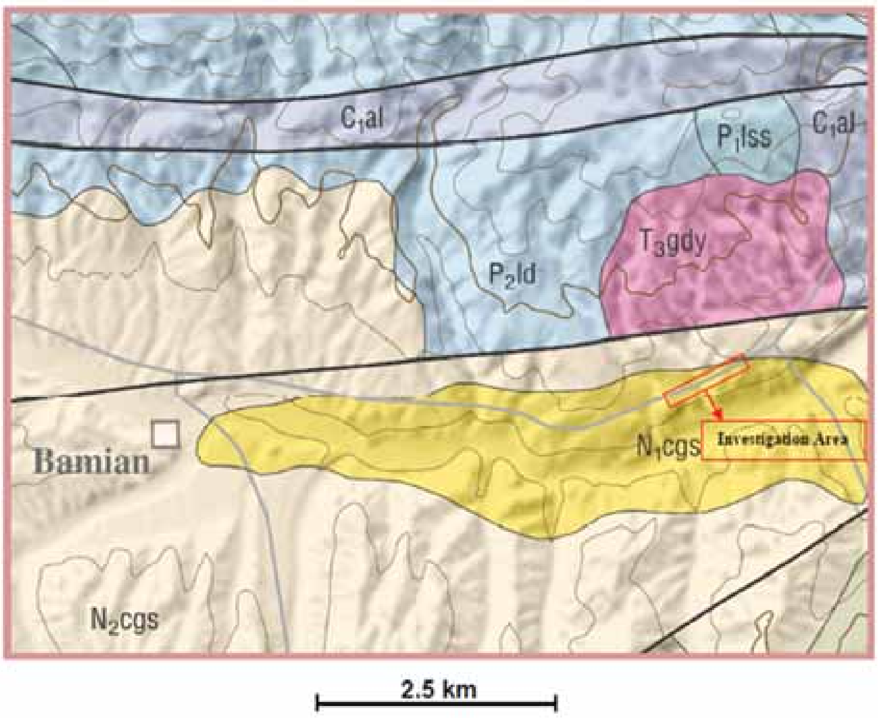


Figure 2 Regional Geology of Shikari Dam Project

(Reference: Final Geotechnical Report for Shikari Dam Site)

As previously mentioned, and as analyzed in the *Final Geotechnical Report for Shikari Dam Site* prepared by Geo Search, the project area is in a seismically active region having estimated peak acceleration between 0.34g to 0.65g, with an intensity VIII rating for potential damage. This rating involves severe shaking, overturned furniture, and unreinforced brick buildings suffering heavy damage. The high risk for seismic activity in this region is a result of the movement on both the Hari Rud and the Andarab fault systems. The Hari Rud fault system extends nearly 600 km in the east-west direction while the Andarab fault system is approximately 150 km in length. While potential seismic magnitudes of these fault systems are unknown, in March 2002, a M 6.1 earthquake in Nahrin, just northeast of the site, caused more than 1,000 fatalities and widespread damage. Figure 3 shows the project site's geographical location (Tectonic domain 4) in relation to these fault systems. Fault systems are categorized as category A (red), category B (green), and category C (blue).

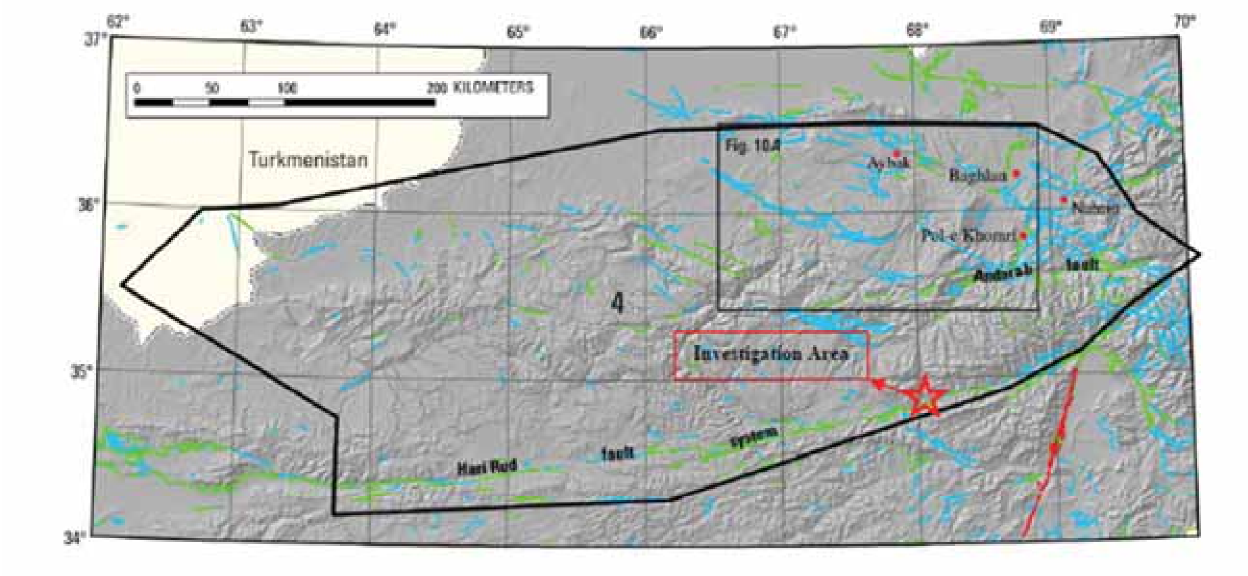


Figure 3 Seismic Location of Shikari Dam

(Reference: Final Geotechnical Report for Shikari Dam Site)

6.2 Hydraulics and Hydrology

6.2.1 Existing Reports

The documents listed below represent the primary source of information used to review hydrology, hydraulics, sedimentation and geomorphology at Shikari Dam.

1. United States Army Corps of Engineers (USACE) 2011. *Afghanistan Watershed Assessment – Bamyan Province*
2. United States Geological Services (USGS) 2011. *Technique for Estimation of Stream flow Statistics in Mineral Area of Interest in Afghanistan.*
3. United States Geological Services (USGS) 2011, *Streamflow Characteristics at Streamgages in Northern Afghanistan and Selected Locations*
4. Raphy Farve and Golam M. Kamal, 2004. *Watershed Atlas of Afghanistan, First Edition.*
5. Stream gage data from USGS website, http://afghanistan.cr.usgs.gov/water.php

6.2.2 Hydrology

The watershed contributing to the proposed dam site has an area of 2057.4 sq. km. Stream gage and precipitation data for the watershed contributing to the proposed dam site was not available. As a result, indirect methods were used to estimate the design discharge.

Two stream gages are located in the watershed; one on each of the two major tributaries contributing to the Bamyan River as shown in Figure 4. The stream gage in the Foladi River has a watershed of 320 km^2, and the stream gage at the Bamyan River in Bamyan has a watershed of 945 km^2. Only eight years of stream data was available, between 1969 and 1977, for each of the stream gages. Percentage exceedance data for annual and monthly discharges for the two stream gages at Foladi River and Bamyan River were collected from the USGS website, *http://afghanistan.cr.usgs.gov/water*.

The watershed contributing to the Shikari project site has two different land cover types. The northwestern portion of the watershed contains desert areas, whereas the southeastern portion of the watershed contains mountainous areas as shown in Figure 4. Of the total watershed area contributing to Shikari Dam site, 914 km^2 is mountainous and 1143.4 km^2 is desert.

Based on topography and aerial photography, the Foladi River watershed is generally mountainous, and the stream flow is mainly generated from melting snowcaps in the mountains. The watershed contributing to Bamyan River at Bamyan has a desert landscape with lower elevations and less vegetation compared to the Foladi River watershed. As a result of these basin characteristics and the proximity to the Shikari site, it was assumed that the stream gages at Foladi River and Bamyan River watershed best represent the hydrology for the Shikari site.

Discharge at the Shikari site is calculated as the area weighted average discharges from mountainous area represented by the Foladi River and from desert areas represented by Bamyan River near Bamyan, as shown in Equation 1, below.

$$Q_T = \frac{Q_F}{A_F} * A_M + \frac{Q_B}{A_B} * A_D \qquad \text{----------} \quad \text{Equation 1}$$

Where,

Q_T = Total stream flow at Shikari Dam Site

Q_F = Stream flow at Foladi River

Q_B = Stream flow at Bamyan River near Bamyan

A_F =Watershed area of Foladi River streamgage

A_D =Watershed area of Bamyan River streamgaged

A_M = Area of mountainous region

A_D = Area of desert region

Exceedance probability discharges for the Foladi River and Bamyan River gages near Bamyan are shown as Table 1 and Table 2 in Appendix D, respectively. Area-averaged annual and monthly exceedance probability discharges for the Shikari site were calculated using Equation 1, and are shown in Table 3 of Appendix D. A graphical summary of calculated annual exceedance probability for Shikari dam site is shown in Figure 5.

For design purposes, a maximum design discharge of 5.0 m^3/s was selected for the hydropower plant, which is equivalent to 23 percent annual probable exceedance discharge. Typically, a range of 15-30 percent exceedance is desirable to optimize the hydroelectric power system, and maintain a higher plant factor The cutoff discharge is defined as 40 percent of the design discharge, which is equivalent to 2.0 m^3/s. In analyzing the annual average number of days without generation, Table 2 shows a total of 82 days that the system will be out of service.

Table 2 Shikari Dam Annual Serviceability

Month	Days Exceeding Low Flow Cutoff	Total Days	Days out of Service
October	96%	31	1
November	75%	30	8
December	74%	31	8
January	80%	31	6
February	76%	28	7
March	67%	31	10
April	43%	30	17
May	76%	31	7
June	93%	30	2
July	79%	31	7
August	78%	31	7
September	95%	30	1
Total			**82**

The field-measured discharge completed in September 2011 was 3.22 m^3/s (Appendix F), falling within the operational range. This discharge is in line with the calculated probability exceedance for the month of September. The 55-percentile exceedance probability discharge for the month of September is 3.25 m^3/s. The calculated discharge-rating curve for the month of September is shown in Figure 6.

The gross head for hydropower production is 15 meters, and for preliminary calculations, the net head is assumed to be 14 meters. Considering a 90 percent plant operating efficiency, the maximum installed capacity is calculated to be 0.6 MW. The annual energy production is calculated to be 3061 megawatt-hour (MWH).

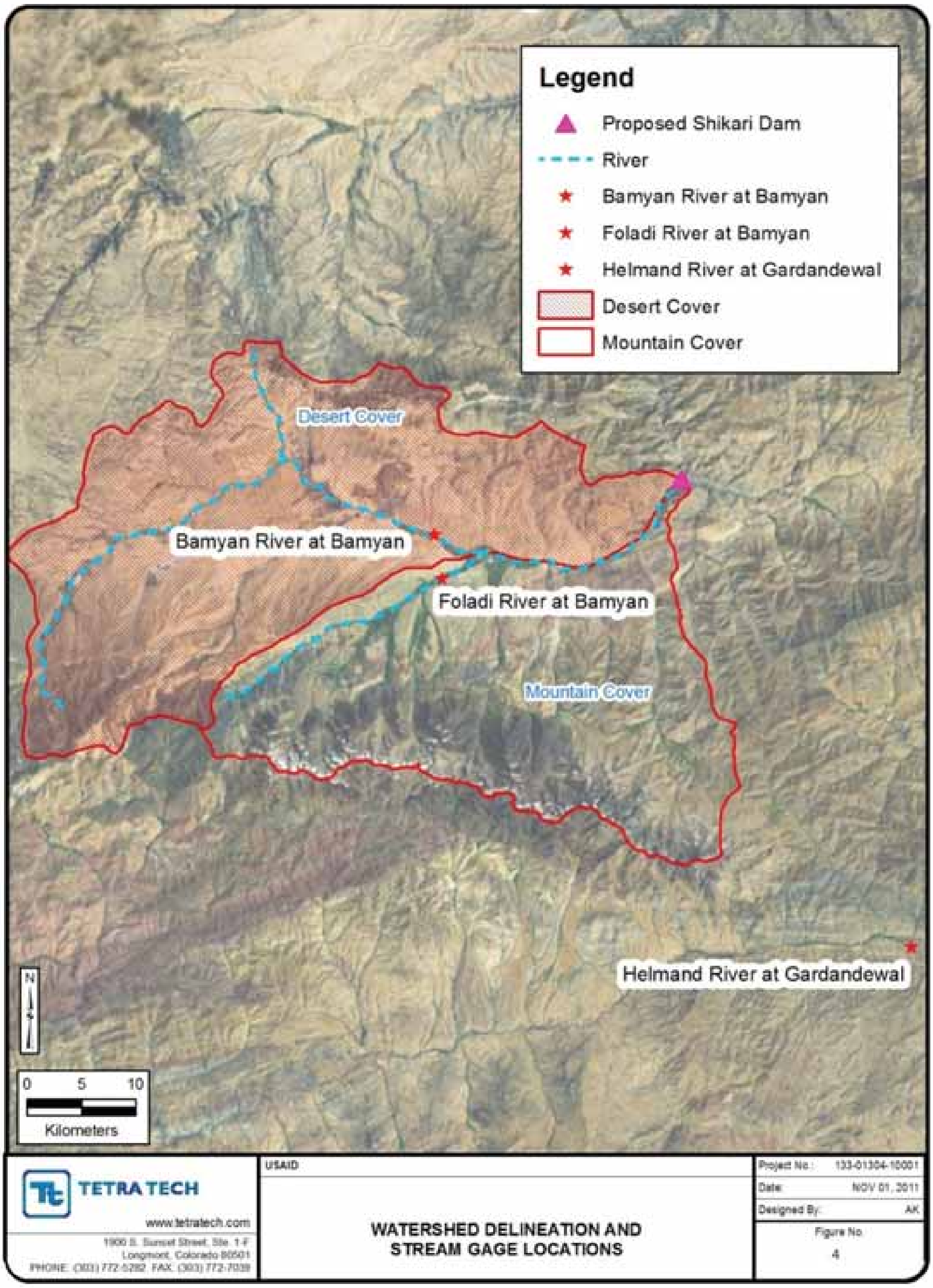


Figure 4 Watershed Delineation and Stream Gage Locations Map

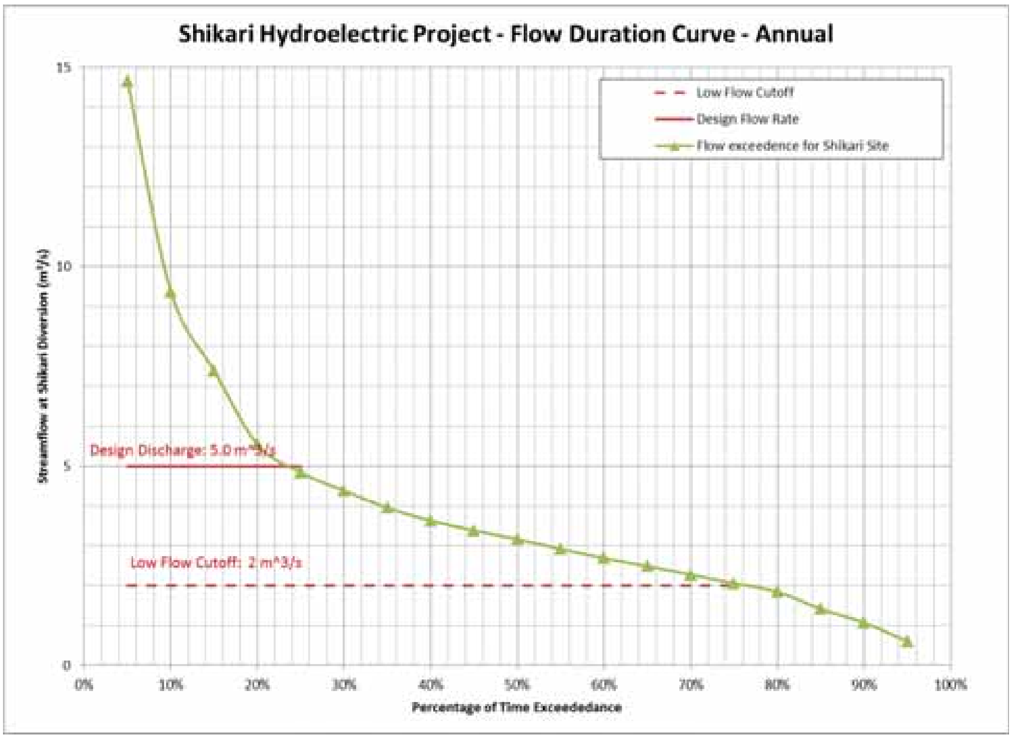


Figure 5 Annual Flow Duration Curve

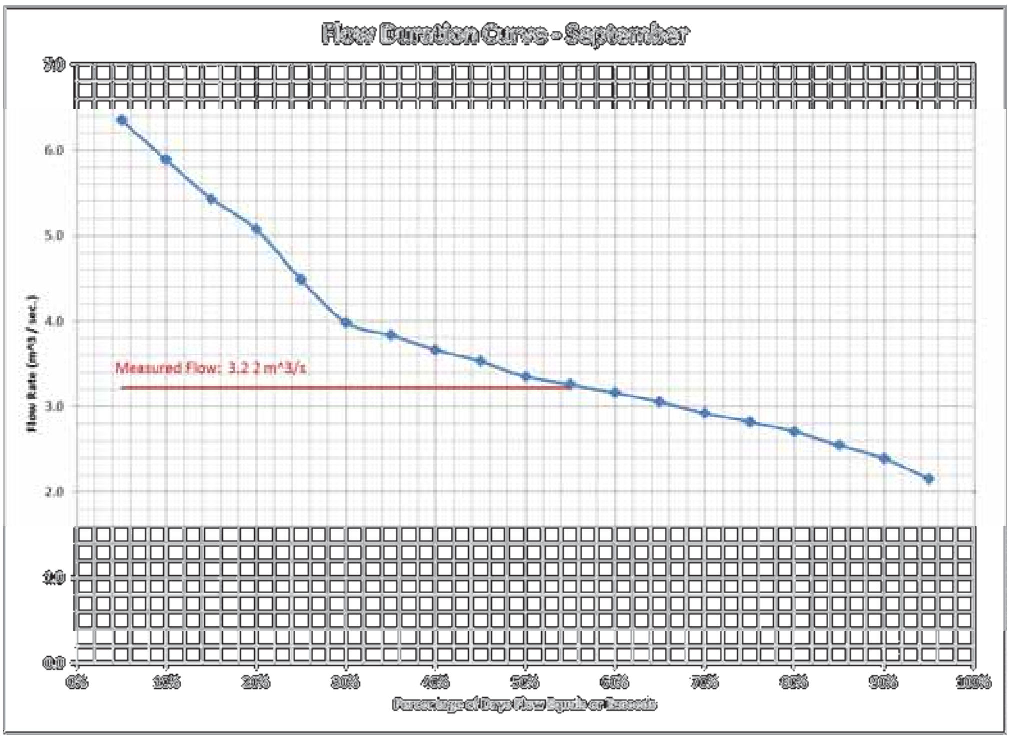


Figure 6 Comparison Flow Duration Curve for the Month of September with Field Measured Flow.

6.2.3 Hydraulics

The spillway is designed to convey a 500-year flood event. Aerial photographs show no human habitation immediately upstream or downstream of the dam. Considering the small reservoir storage volume and limited risk to human life, a 500-year flood protection is deemed sufficient. The 500-year peak discharge was calculated using the annual peak discharges at the Bamyan and Foladi gages by applying a log-Pearson III analysis and the computed probability maximum instantaneous discharge at each gage station was calculated. Only 8 years of gage data is available for the analysis, when the typical minimum accepted period-of-record is 10 years to perform a log-Pearson III analysis. Applying the expected probability correction as detailed in Bulletin 17B – Guidelines for Determining Flood Flow Frequency, the expected probability maximum instantaneous discharges at each gage station were calculated and are shown in Table 3. The 500-year design flood at Shikari site was determined to be 116.5 m^3/s.

Table 3 Shikari Dam Peak Flood Discharge Summary

		Foladi River (Area = 302 km^2)		**Bamyan River at Bamyan** (Area = 945 km^2)		**Shikari Dam Site** (Area = 2057 km^2)
Exceedance probability	Return Interval (years)	Computed Probability Maximum Instantaneous Discharge (m^3/s)	Expected Probability Maximum Instantaneous Discharge (m^3/s)	Computed Probability Maximum Instantaneous Discharge (m^3/s)	Expected Probability Maximum Instantaneous Discharge (m^3/s)	Expected Probability Maximum Instantaneous Discharge (m^3/s)
0.99	1	2.30	1.25	1.20	0.77	5.0
0.50	2	7.10	7.14	3.44	3.44	24.6
0.20	5	10.1	10.59	5.10	5.42	36.4
0.10	10	11.9	13.10	6.29	7.12	45.0
0.04	25	14.1	15.79	7.49	9.23	54.1
0.02	50	15.7	19.82	9.13	13.11	67.6
0.01	100	17.1	23.37	10.43	17.39	79.4
0.005	200	18.5	27.57	11.78	23.55	93.0
0.002	500	20.3	35.01	13.68	36.53	116.5

The spillway crest has been designed at an elevation of 2334.5 meters, which is 2.5 meters higher than the existing roadway elevation. This additional 2.5 meters rise in elevation generates nearly 20% more hydropower energy compared to the scenario where the roadway is not raised. A broad crested spillway was selected for design considering the ease in concrete placement during construction. The length of the spillway is 15 meters to be in line with the narrow downstream natural channel. The width of the spillway is 4 meters to allow access for service. A coefficient of discharge (C_D) of 1.45 was used for the broad-crested weir. The depth of the spillway for the 500-year flood is 3.25 meters. The dam crest is at an elevation of 2338.0 meters, providing an additional 0.25 meters of freeboard over the 500-year flood level. The spillway-rating curve generated is graphically summarized in Figure 7.

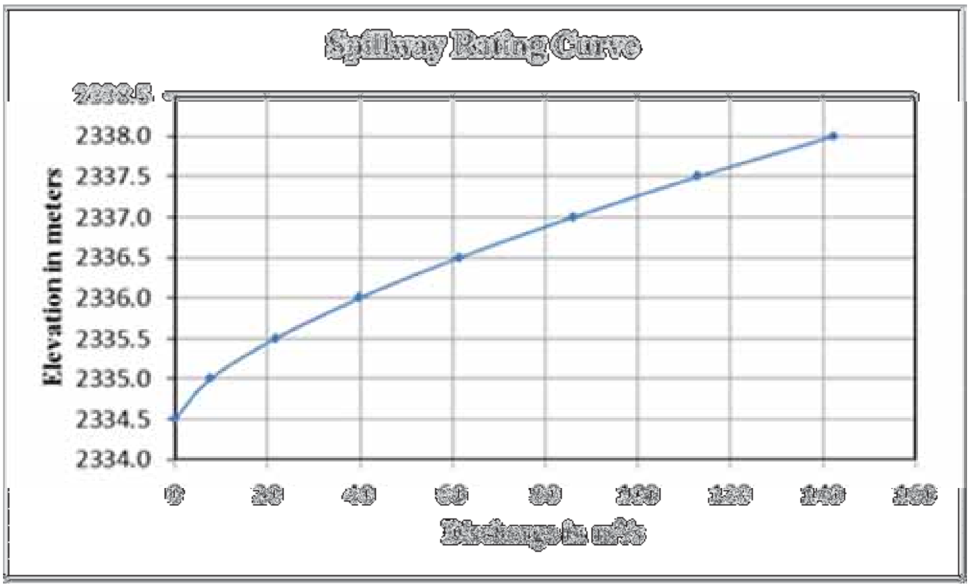


Figure 7 Spillway Rating Curve for Shikari Dam Spillway.

6.2.4 Sedimentation

Management of sediment is a significant challenge for this project. Sediment yield rate at the proposed dam site was not available. Sediment yield for 19 rivers in Afghanistan are available and are shown in Table 4. The high sediment yield is a result of deteriorated watershed conditions. A fraction of this sediment yield is suspended load and the remainder is bed load; however, it cannot be ascertained what fraction is suspended load or bed load. The particle size gradation of sediment is also unknown. The current dam layout includes a low-level sluice for passing sediment through the dam and reservoir site; however, with such a high sediment load, sediment management will be a primary concern if the design moves forwards.

Table 4 Sediment Yields of Afghanistan

River	Location	Sediment Yield	Data Source
Panjshir	Panjshir I	275	(a)
Panjshir	Baghdara	155	(a)
Ghorband	Totumdara	420	(a)
Maidan	Hajian	250	(a)
Logar	Kajab	250	(a)
Logar	Gat	150	(a)
Kabul	Tangi Gharu	148	(a)
Kunar River	not provided	780	(b)
Arghandab	Arghandab Reservoir	250	(c)
Helmand	Kajakai	200	(d)
Ghorband	Pul-i-Ashawa	420	(e)
Hari Rud	Tagau Gaza	270	(e)
Kabul	Naghlu	410	(e)
Kabul	Tangi Saidan	260	(e)
Logar	Kajau	190	(e)
Pajshir	Mouth	455	(e)
Pajshir	Gulbahar	750	(e)
Shakhar Darya	Ak Sahai	273	(e)
Kabul	Nowshera, Pakistan	288	(f)

Sediment Yield Units = tonnes/sq km/year
(a) - Montreal (1980)
(b) - Electrowatt (1977)
(c) - Mort (1973)
(d) - Perkins (1970)
(e) - UN-FAO on-line database, Tkachev et al.
(f) - UN-FAO on-line database (no source, location:33.9967, 72.0131)

(Reference: USACE 2011 Afghanistan Watershed Assessment – Bamyan Province)

6.3 Geotechnical and Dam Engineering Existing Reports

Other than the recent geotechnical investigation report, no existing reports on geotechnical conditions at the Shikari Dam site were available for this study. The geologic report discussed in Section 6.1 was used.

6.3.1 Geotechnical Investigation Analysis

A field investigation was performed on July 3, 2011. Three boreholes were drilled with rotary techniques to the bedrock and split spoon samples were collected. Standard Penetration Test (SPT) blow counts were recorded to measure the in-situ density of the soil. Drilling was advanced 54-115 mm into the bedrock on the left and right abutments, resulting in short sections of rock mass core for the investigation. A geologic investigation was completed using DIPS software for bedrock exposed on the left and right abutments. Boring logs are shown in Appendix A of this report.

A variety of laboratory tests were performed for the investigation including sieve analyses, Atterberg limits, moisture content, specific gravity, direct shear, chemical test, modified compaction, expansion index, consolidation, permeability, and relative density. A summary of the laboratory test results are included in Table 5 (as prepared by Geo Search from the *Final Geotechnical Report for Shikari Dam Site*). Detailed geotechnical information of the Shikari site can also be found in the Geo Search report.

Table 5 Summary of Laboratory Soil Data

Parameters		Minimum Value	Maximum Value
Soils by Geomechanical and Geotechnical Strength		CL	GW
Atterberg Limits	Liquid Limit (LL)	41.22%	NA[3]
	Plastic Limit (PL)	33.82%	NA[1]
	Plastic Index (PI)	8.9	Non-Plastic
Natural Moisture Content (W %)		9.26%	28.40%
Modified Compaction	Max. Dry Density (MDD)	2.07 gr/cc	2.13 gr/cc
	Optimum Moisture Content (OMC)	9.40%	10 %
Specific Gravity (Gs)		2.50	2.78
Expansion Index		0 (non expansive)	04.473 (very low)
Direct Shear	C	0.031 Kg/cm^2	0.8 Kg/cm^2
	φ	20.66	21.56
Field Density	Natural Unit Weight	1.6 gr/cc	1.7 gr/cc
	Dry Unit Weight	1.3 gr/cc	1.5 gr/cc
One Dimensional Consolidation	Cc	0.05	0.06
	Cd	0.016	0.018
	Pc	93 KPa	98 KPa
Unconfined Compressive Strength (UCS)		1.07 Kg/cm^2	1.27 Kg/cm^2
Shear Strength (Su)		0.5 Kg/cm^2	0.63 Kg/cm^2
Hydraulic Conductivity (Falling and Raising Head)		2.97×10^{-4} cm/s	$1.08^{x}10^{-3}$ cm/s
	Sulfate	<5 mg/l	41 mg/l
Chemical Content	Chloride	6 mg/l	10 mg/l
	pH	8.42	8.53
Relative Density (Dr) (Fine Aggregate)	Dry Dr	2.44	2.47
	SSD Dr	2.54	2.56
	Apparent Dr	2.70	2.70
	water Absorption	3.50%	4%
SPT Value (SPT Number)		8	>50 (Refusal)
K Value (Subgrade Soil Reaction)		140 pci	700 pci

6.3.2 Dam Design Concept

The foundation for the concrete gravity dam should be a prepared bedrock surface resulting from removal of soils and weathered bedrock, shaping to provide a uniform surface, and cleaning. Existing borehole information indicates that the bedrock in the valley slopes

[3] Liquid Limit, Plastic Limit and Plasticity Index are not applicable for non-plastic soils

upward from an elevation of about 2320 m (Bore Hole 2), resulting in a maximum overburden depth of up to 12 meters. No bedrock information is available beneath the stream channel, the right (east) side of the valley or the abutments. It is recommended that two to three additional borings be drilled on the right (east) side of the channel to further evaluate bedrock depths and conditions prior to finalizing designs.

An estimated bedrock profile was developed based on the borings provided in the *Final Geotechnical Report for Shikari Dam Site*. An additional 1.0 meter of bedrock may need to be removed to prepare the foundation to meet design requirements. Considering the low head of water in the reservoir and the apparent tight joints in the bedrock exposed on the left and right abutments, foundation grouting is not expected to be required. The suggested foundation profile is shown on Sheet C-04 in Appendix E.

For the design of a Run of the River (RoR) diversion dam, the height of the dam was kept low. This results in a small area of inundation upstream with limited flood attenuation and sediment storage, as well as a smaller length of highway to be relocated or raised. Due to the large watershed draining to the site (2057.4 km^2.), sediment loading in the reservoir will further reduce the active reservoir capacity. Sediment management will be a major concern in future design phases, as well as in maintaining operating efficiency.

The dam spillway is a 15-meter long, broad-crested weir with 3.5 meters of freeboard to the dam crest. The spillway chute will be stair stepped and terminate at a stilling basin to dissipate energy. Considering the potential for the spillway to operate on a frequent basis, a concrete- or masonry-stilling basin should be designed to dissipate spillway flows, prior to releasing the flow to the unprotected stream channel. Additional analysis will be performed during final design to size the stilling basin.

Outlet works should include a low-level sluiceway that allows for periodically flushing sediment from the reservoir and an intake connected to a settling basin that feeds the penstock.

A concrete drop inlet intake structure with a trash rack has been provided at the entrance into the settling basin and the penstock. Water gets into the intake structure as weir flow, thus reducing the amount of sediments. However, the trash rack would need to be periodically cleaned to remove floating debris. An open channel through the dam is suggested for easy observation and cleanout during the life of the dam. The water level may be reduced using the low-level sluice operation. During this operation, the trash rack can be also be cleaned.

A settling basin has been incorporated at the termination of the open channel to allow sediment and coarse material to settle out before entering the penstock. Additionally, a small discharge to the river has been provided to maintain minimum downstream flows and flush sediment from the settling basin and channel. Gates will be installed at each pipe entrance to allow control of water flow rate. Sizes of all dam appurtenances are preliminary, and will need to be verified during final design. A plan of the recommended dam is shown on Sheet C-03 and a typical section is shown on Sheet C-05 in Appendix E.

The 4.0-meter crest width of the gravity dam was selected to provide access to the dam and the outlet works equipment, as well as to meet stability requirements. The crest elevation was selected to provide sufficient spillway surcharge and access from the dam abutments.

6.3.3 Stability Analysis

Stability analyses performed on the proposed dam included three loading conditions: static loading with normal high water level, 500-year flood event, and seismic. Each condition was analyzed for resistance to sliding and overturning. The analyses were performed using the Gravity Wall program v. 10.39 by GEO 5. In addition, hand calculations of free body diagram forces were used to edit the computer generated results. Minimum factors of safety were based on US Army Corps of Engineers (USACE) guidance and are shown in Table 6.

As discussed in Section 6.1.2 of this report, the project site is in a seismically active area. Per the International Building Code, a seismic horizontal acceleration of two-thirds the peak seismic acceleration was used in the analysis (0.23 g).

The water surface elevation varied with the load case. The static analysis and seismic analysis cases assumed a water elevation at the normal high water level (NHWL) at the spillway crest. For the 500-year flood case, the water surface elevation was set at the dam crest.

The following material properties were used in the analysis:

- Concrete dam unit weight of 22.3 KN/m^3.
- Bedrock unit weight of 20.4 KN/m^3, internal friction angle of 35 degrees, and a cohesion value of 95.7 KN/m^2.
- Unit weight of water of 9.81 KN/m^3.
- Friction factor between dam and bedrock of 0.7 (NAVFAC DM-7).
- Sediment (silt) properties:
  - Unit weight of 15.7 KN/m^3
  - Internal friction angle of 28 degrees
  - Cohesionless

The driving forces considered in the dam included:

- Hydrostatic pressure (at 500-yr flood event)
- Sediment (silt) active earth pressure (accumulation of 9 meters upstream of the dam)
- Uplift pressure
- Seismic acceleration of silt and dam (seismic analysis only)

Resisting forces acting on the dam included:

- Bedrock passive pressure at dam toe (no soil backfill pressure was considered)
- Friction force due to dead load of dam

The results of the analyses indicate the proposed dam meets the USACE factors of safety. Calculations are attached in Appendix C. The results are presented in the table below:

Table 6 Factor of Safety Summary

Analysis	Minimum F.S. (Sliding)	Minimum F.S. (Overturning)	Est. F.S. (Sliding)	Est. F.S. (Overturning)
Static Analysis	1.5	1.3	4.3	2.7
500-year Analysis	1.5	1.3	2.7	1.8
Seismic Event Analysis	1.1	1.1	1.13	1.2

6.3.4 Seepage Analysis

Seepage through the dam and foundation was modeled using the Seep/W 7.0 computer program. The seepage model used the maximum dam section founded at a depth of one meter below the top of bedrock. The bedrock was assumed to have a hydraulic conductivity of 5E-8 m/sec. The soils were modeled with a hydraulic conductivity of 5E-6 m/sec. The concrete dam was assumed to have a hydraulic conductivity of 10E-14m/sec.

A computer-generated cross-section summarizing the analysis is shown in Appendix C. Parameters and calculated results, including hydraulic conductivities, water levels and resulting flux under the dam, are shown on the cross-section. The analysis resulted in a flux of about 3.0E-7 cubic meters per second/meter of length (26.0 liters/day/m). Based on the estimated dam length of 50 meters, the seepage is calculated to be 1300 liters per day.

6.3.5 Road Relocation

Because of the dam construction, a section of the existing roadway (~180 m) will need to be relocated. The new road will go up and over the dam crest and tie back to the existing roadway on either side of the dam. A maximum 10% grade was used for design purposes.

7.0 Assessment

The viability of a low-head gravity dam and earthen dam were analyzed as part of this analysis. As previously discussed in Section 5.4, weak foundation soils and other design issues ruled out the possibility of an earthen dam.

Based on site conditions and current site understanding, a low-head gravity dam was analyzed at a conceptual level of design. With this layout, a maximum of approximately 0.6MW of hydroelectricity can be produced. Under the conditions of designing a low-head dam at the selected project site, two major concerns limit the viability of continuing this project:

1.) *Low Head*: A gross head of approximately 14 meters is available with the current design. With such a small difference in elevation change, power output is greatly limited. Due to constraints of inundation areas upstream, including the newly constructed roadway, crop fields, and villages, a larger dam is not appropriate for the site.

2.) *Sediment Management*: As previously mentioned, sediment load is expected to be high. Since only a small reservoir can be constructed, these high sediment loads could cause degradation to the dam's hydraulic systems, including the operation of the penstock. Nearly daily use of the sluice gate would be needed to maintain clear water to the penstock. Although a precise analysis should be calculated during future investigations, this results in an enormous maintenance conflict for the entire life of the dam.

These concerns may be sufficient to make the low head hydroelectric project at this Shikari site uneconomic and should be fully evaluated prior to performing additional investigations or designs.

8.0 Data Gaps

If the Shikari project moves forward, several data gaps will need to be addressed based on the low head diversion dam concept.

8.1 Hydrology and Hydraulics

Additional stream flow data is needed to calculate more accurate recurrence interval flood discharges which will affect the dam design and the final sizing for the power plant. Information such as historical high flow discharges and associated recurrence intervals should be included in data gathering, as well as local input on presence of flooding in agricultural areas. Information on minimum stream flow requirements downstream of the dam should also be derived. A sedimentation analysis would also be desired, but may be difficult to perform in a reasonable time frame. Local inhabitants of the region may be able to provide sufficient information.

8.2 Geotechnical and Geology

Bedrock Information

- Geologic mapping of rock exposed in the area of the dam and abutments should be performed to understand the rock type, structure, and degree of weathering, and to verify the information given in the *Final Geotechnical Report for Shikari Dam Site*.

Borrow Areas

Information needed to proceed with design of the dam includes the following:

- Location for sources of sands and gravels, clays, riprap, aggregate quarry. Borrow areas for alluvial areas should be identified near the project site. Sites used for construction of the Shikari Valley Dam should be evaluated for quantity and quality of materials for each type of material for the dam components. Borrow locations should be reasonably close to the project site, with access conditions that are not too difficult.

- Material classification, durability, weathering. Geologically map and sample the potential borrow materials. Perform sufficient field explorations (test pits) to quantify the materials, recover samples for laboratory testing, and prepare initial mix designs for concrete.

Geology

An additional 2-4 borings should be completed between the existing borings and both abutments to collect bedrock data along the entire dam axis. The additional subsurface exploration should include exploratory borings which penetrate well into the bedrock. The bedrock should be cored with core recoveries and Rock Quality Designations (RQD) recorded for each core interval. Packer permeability testing should be performed in the bedrock and suitable permeability testing should be performed in the soils using field and laboratory methods. The goal of the borings is to characterize the engineering and permeability properties of the bedrock. In addition, the bedrock weathering profile and joints and fractures should be measured and quantified. The exploratory drilling activities should include measurement of groundwater conditions in both unconfined and confined aquifers.

Survey

Limited survey upstream of the dam makes it difficult to understand the limits of inundation and subsequent storage volume of the dam. If the current design moves forward, additional topographic survey information will be required to determine precise inundation areas.

9.0 References

1. *Final Geotechnical Report for Shikari Dam Site – Bamyan Province, Afghanistan*. Prepared by Geo Search.

2. *Final Survey Report; Topographic Survey Work of Shikari Site – Bamyan Province, Afghanistan*. Prepared by Geo Search.

3. *Engineering Support Program – Preliminary Geological Dam Site Assessment Report Bamyan Province (WO-A-0061)* dated November 25, 2010

4. *Afghanistan Watershed Assessment – Bamyan Province*. United States Army Corps of Engineers (USACE) 2011.

5. *Geological and Mineral Resource Map of Afghanistan*, US Department of the Interior, USGS, 2006.

6. *International Building Code 2006*. International Code Council, 2006.

Appendices

Appendix A Boring Logs

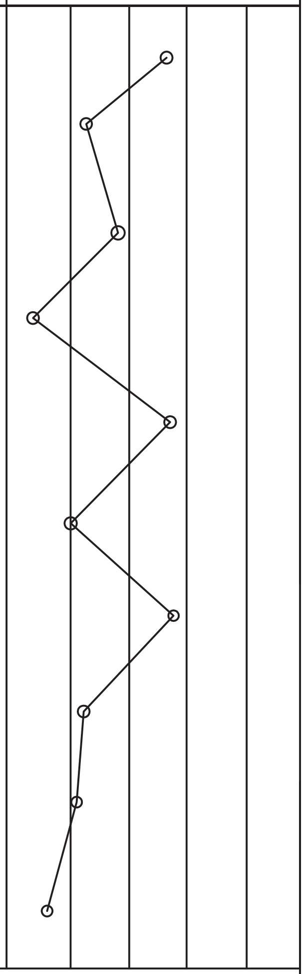
GEO SEARCH GEOTECHNICAL DEPARTMENT

MATERIAL TEASTING LAB

LOG OF BH # 01

Client : USAID	Borehole No : 01	Sampling Method : Continued Sampling by Drilling Machine
Contractor : Tetra tech	Depth : 11M	SPT Hammer : 140-lb/30 inch drop
Project : Shikari Valley dam	Ground Water Table : 4M	SS: Split Spoon via ASTM D-1586
Location : Shikari valley- Bamyan Province		

DEPTH (M)	GRAPHIC LOG	SAMPLING TYPE	MATERIAL DESCRIPTION	USCS CLASSIFICATION	Depth of SPT Test (m)	BLOW COUNTS (N VALUE)		SPT/ (N VALUE) 20 40 60 80
0.3			Top Soil (sand,silt, clay, cobble and root of plant)					
1			silty gravel with sand	GM	1	45->50->50	>50 refusal	
2		☒	silty sand with gravel	SM	2	10-11-13	24	
3			well graded sand with silt	SW-SM	3	15-17-18	35	
4		☒	sandy silt with gravel	ML	4	04-07-07	14	
5			silty gravel with sand	GM	5	26-29-45	>50 refusal	
6		☒	silty sand with gravel	SM	6	07-09-11	20	
7			silty gravel with sand	GM	7	24-27-36	>50 refusal	
8			poorly graded sand with clay or (silty clay)	SP-SC	8	08-10-13	23	
9		☒	silty clayey sand	SC-SM	9	08-09-12	21	
10		☒	slity sand	SM	10	05-08-08	16	
11				Bed Rock	Mica schist (Metamorphic)			



Completion Depth : 11M	Elevation : 2331.131m with station GPS
Date Drilling Started : 02/07/2011	Diameter : 4.5 inch and core 2.1 inch
Date Drilling Completed : 12/07/2011	Geotechnical Eng: [redacted]

Type of drilling : Rotary

Sample Types : Split-Spoon (SS) ☒

Add: House # 511 street # 8 Karte 3, Kabul, Afghanistan
Tell: (+93) 772-983-725 (+93) 707-975-688
Email : geo.search.co@gmail.com

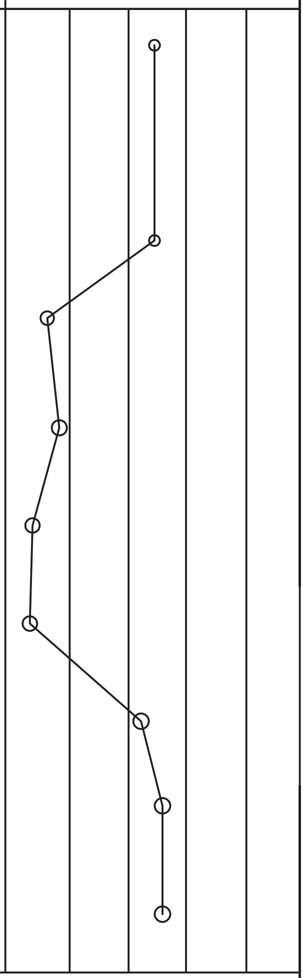
GEO SEARCH GEOTECHNICAL DEPARTMENT

MATERIAL TEASTING LAB

LOG OF BH # 02

Client :	USAID
Contractor :	Tetra tech
Project :	Shikari Valley dam
Location :	Shikari valley- Bamyan Province
Borehole No :	02
Depth :	11M
Ground Water Table :	2M
Sampling Method :	Continued Sampling by Drilling Machine
SPT Hammer :	140-lb/30 inch drop
SS:	Split Spoon via ASTM D-1586

DEPTH (M)	GRAPHIC LOG	SAMPLING TYPE	MATERIAL DESCRIPTION	USCS CLASSIFICATION	Depth of SPT Test (m)	BLOW COUNTS (N VALUE)		SPT/ (N VALUE) 20 40 60 80
0.3			Top Soil (sand,silt, clay, cobble and root of plant)					
1					1	25->42->50	>50 refusal	
2			Silty calyey gravel with sand	GC-GM	2	31-35->50	>50 refusal	
3			Silty gravel with sand	GM	3	31-42-45	>50 refusal	
4		☒	Sandy lean clay with gravel	CL	4	02-04-07	11	
5		☒	Silty clay	CL-ML	5	04-07-07	14	
6		☒	Silt	ML	6	05-05-05	10	
7		☒	Silty sand with garavel	SM	7	03-04-05	09	
8			Silty gravel with sand	GM	8	15-17-25	42	
9			Well graded gravel with sand	GW	9	25-34->50	>50 refusal	
10			Poorly graded gravel with silt	GP-GM	10	24-35-37	>50 refusal	
11				Bed Rock	Mica schist (Metamorphic)			



Completion Depth : 11M
Date Drilling Started : 02/07/2011
Date Drilling Completed : 12/07/2011

Elevation : 2330.397m with station GPS
Diameter : 4.5 inch and core 2.1 inch
Geotechnical Eng: [redacted]

Type of drilling : Rotary
Sample Types : Split-Spoon (SS) ☒

Add: House # 511 street # 8 Karte 3, Kabul, Afghanistan
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Email : geo.search.co@gmail.com

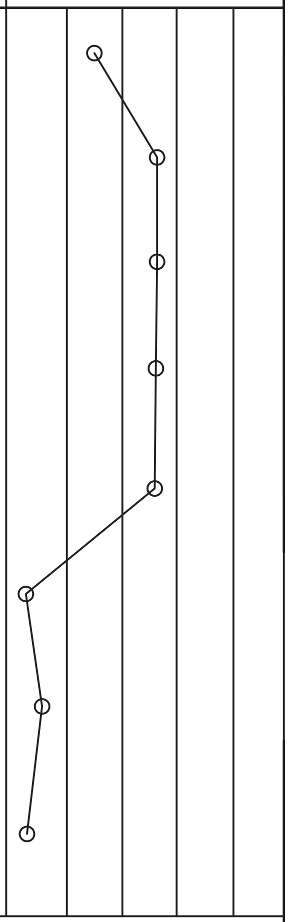
GEO SEARCH GEOTECHNICAL DEPARTMENT

MATERIAL TEASTING LAB

LOG OF BH # 03

Client : USAID	Borehole No : 03	Sampling Method : Continued Sampling by Drilling Machine
Contractor : Tetra tech	Depth : 09M	SPT Hammer : 140-lb/30 inch drop
Project : Shikari Valley dam	Ground Water Table: In the river	SS: Split Spoon via ASTM D-1586
Location : Shikari valley- Bamyan Province		

DEPTH (M)	GRAPHIC LOG	SAMPLING TYPE	MATERIAL DESCRIPTION	USCS CLASSIFICATION	Depth of SPT Test (m)	BLOW COUNTS (N VALUE)		SPT/ (N VALUE) 20 40 60 80
			Top Soil (sand,silt, clay, cobble and root of plant)					
1		☒	Silty sand with gravel	SM	1	05-07-08	15	
2			poorly graded gravel with sand	GP-GM	2	28-31-45	>50 refusal	
3			poorly graded gravel with silt and sand	GP-GM	3	35-45-48	>50 refusal	
4			silty gravel with sand	GM	4	31-36-48	>50 refusal	
5			silty, clayey, gravel with sand	GC-GM	5	36-42-45	>50 refusal	
6		☒	sandy silt	ML	6	02-03-05	08	
7		☒	Silty sand with garavel	SM	7	02-04-05	09	
8		☒	poorly graded sand with silt and gravel	SP-SM	8	03-04-04	08	
9				Bed Rock	Mica schist (Metamorphic)			



Completion Depth : 09M
Date Drilling Started : 02/07/2011
Date Drilling Completed : 12/07/2011

Elevation : 2329.409m with station GPS
Diameter : 4.5 inch and core 2.1 inch
Geotechnical Eng: [redacted]

Type of drilling : Rotary
Sample Types : Split-Spoon (SS) ☒

Add: House # 511 street # 8 Karte 3, Kabul, Afghanistan
Tell: (+93) 772-983-725 (+93) 707-975-688
Email : geo.search.co@gmail.com

Appendix B Photographs

Photo 1: Looking upstream towards the proposed dam location. An almost vertical rock mass rock mass is observed to the left. Date of photography: June 8, 2011

Photo 2: Looking downstream of the proposed dam location. Date of photography: June 8, 2011

Photo 3: Looking downstream at stream. The rock mass on the left dips into the valley. Date of photography: June 8, 2011

Photo 4: Looking upstream of the proposed dam location. Date of photography: June 8, 2011


Photo 5: Drilling Operation at dam site. Date of photography: June 8, 2011

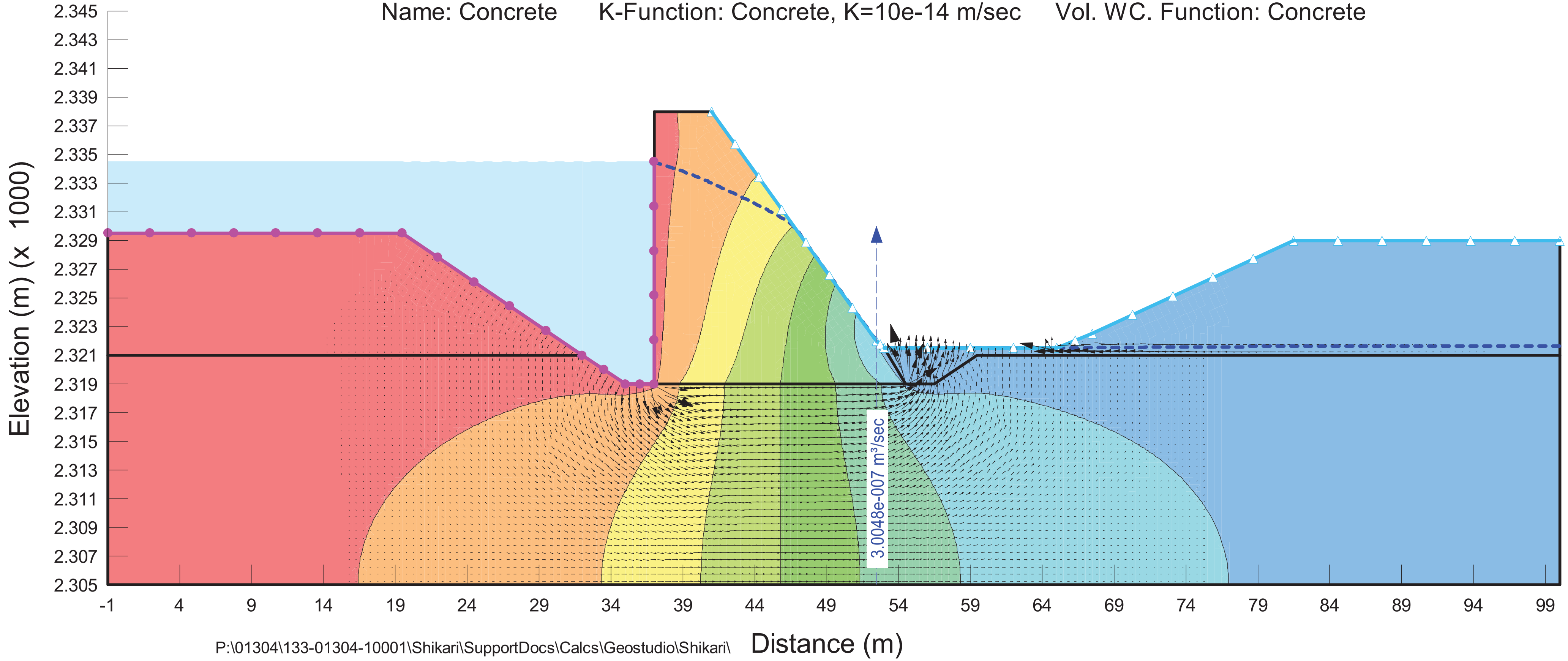
Appendix C Stability and Seepage Analyses

Appendix C-1 Seepage Analysis

Shikari Valley

Steady State Seepage
Typical Section at Spillway

Name: Bedrock K-Function: Bedrock, K=5e-8 m/sec Vol. WC. Function: Mica Schist
Name: Cover Soil K-Function: Cover Soil, K=5e-6 m/sec Vol. WC. Function: Cover Soil
Name: Concrete K-Function: Concrete, K=10e-14 m/sec Vol. WC. Function: Concrete



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Appendix C-2 Dam Stability Hand Calculations

CALCULATION COVER SHEET

Client: USAID Project No.: 133-01304-10001

Project Name: Shikari Dam

Title: Dam Stability Analysis – Static, Seismic, and PMF Scenarios

Total Number of Pages (including cover sheet): 14

Total Number of Computer Runs: 0

Prepared by: FJD Date: 11/17/2011

Checked by: DJ Date: 11/17/2011

Description and Purpose:

To calculate the dam stability under static, seismic, and PMF conditions.

Design Basis/References/Assumptions:

Factors of Safety:

1. Sliding – 1.5
2. Overturning – 1.3
3. Seismic – 1.1 (both sliding and overturning)

Material Properties:

1. Concrete dam unit weight of 22.3 KN/m3.
2. Bedrock unit weight of 20.4 KN/m3, internal friction angle of 35 degrees, and a cohesion value of 95.7 KN/m2.
3. Unit weight of water of 9.81 KN/m3.
4. Static friction factor between dam and bedrock of 0.7 (NAVFAC DM-7).
5. Silt accumulation of 9.3 meters upstream of the dam with the following properties:
    a. Unit weight of 15.7 KN/m3
    b. Internal friction angle of 28 degrees
    c. Cohesionless
6. Horizontal ground acceleration = 0.23 g (seismic analysis only)

Remarks/Conclusions/Results:

Analysis	Acceptable F.S. (Sliding)	Acceptable F.S. (Overturning)	Est. F.S. (Sliding)	Est. F.S. (Overturning)
Static Analysis	1.5	1.3	4.3	2.7
500-year Analysis	1.5	1.3	2.7	1.8
Seismic Event Analysis	1.1	1.1	1.13	1.2

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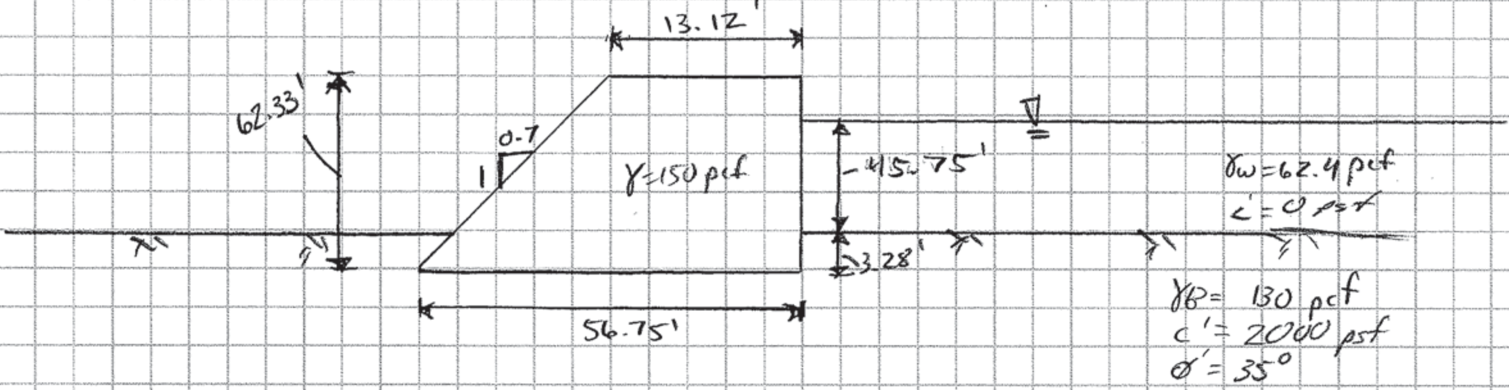
Client: USAID   Job No.: 133-01304-10001   Sheet 1 of ___

Description: Shikari Dam Stability Analysis   Designed By: FJD   Date: 11/14/11

Checked By: SES   Date: 11/18/11

SHIKARI DAM CROSS-SECTION (N.T.S)   (SEISMIC CONDITIONS)



Static Friction Factor Coefficient = 0.7

Peak Seismic Acceleration = 0.34g ; use 2/3 = 0.23g = A

Seismic Horizontal Coefficient = 1.5(A) = 1.5(0.23) = 0.35

Active Earth Pressure Coefficient = K_{ae} = 0.53
(interpolated from table 7.6)

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Client: USAID | Job No.: 133-01304-10001 | Sheet 2 of

Description: Shikari Dam Stability Analysis | Designed By: FJD | Date: 11/14/11

Checked By: SES | Date: 11/18/11

Dead Load of Dam:

$D_L = \gamma_c(\text{height})(\text{top width}) + \frac{1}{2}(\gamma_c)(\text{height})(\text{bottom width} - \text{top width})$

$= 150\ lb/ft^3 (62.33\ ft)(13.12\ ft) + \frac{1}{2}(150\ lb/ft^3)(56.75\ ft - 13.12\ ft)(62.33\ ft)$

$= 122,665\ lb/ft + 203,960\ lb/ft$

$= 326,625\ lb/ft$

$= 326.6\ kip/ft$

Hydrostatic Pressure:

$P_W = \frac{1}{2}(\text{water depth} * \gamma_w)(\text{water depth})$

$= \frac{1}{2}(45.75\ ft)^2(62.4\ lb/ft^3)$

$= 65,304\ lb/ft$

$= 65.3\ kip/ft$

Seismic Force on Dam from Water per Unit length:

$P_{AE} = \frac{1}{2}(\gamma_w)(\text{water depth})^2 (1 - k_v)(K_{AE})$

$= \frac{1}{2}(62.4\ lb/ft^3)(45.75\ ft)^2 (1)(0.53)$

$= 34,611\ lb/ft$

$= 34.6\ kip/ft$

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Client: US AID   Job No.: 133-01304-10001   Sheet 3 of

Description: Shikari Dam Stability Analysis   Designed By: FSD   Date: 11/14/11

Checked By: SBS   Date: 11/18/11

Seismic Inertia force of concrete dam:

$I_{DAM} = K_{AE} (D_L)$

$= 0.53 (326.6 \text{ kip/ft})$

$= 173 \text{ kip/ft}$

Mica Schist Bedrock passive pressure

$K_p = \tan^2(45 + \phi/2) = \tan^2(45 + 35/2) = 3.7$

$P_p = 1/2 (\gamma_B)(\text{height})^2(K_p) + 2(c')(\text{height})(K_p)^2$

$= 1/2 (130 \text{ lb/ft}^3)(3.28 \text{ ft})^2(3.7) + 2(2000 \text{ lb/ft}^2)(3.28)(3.7)^2$

$= 2,587 \text{ lb/ft} + 179,613 \text{ lb/ft}$

$= 182,200 \text{ lb/ft}$

$= 182.2 \text{ kip/ft}$

Uplift Pressure

$P_u = 1/2 (\gamma_w)(\text{water depth})(\text{bottom width})$

$= 1/2 (62.4 \text{ lb/ft}^3)(45.75')(56.75')$

$= 81,005 \text{ lb/ft}$

$= 81 \text{ kip/ft}$

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Client: USAID Job No.: 133-01304-10001 Sheet 4 of ___

Description: Shikari Dam Stability Analysis Designed By: FJD Date: 11/14/11

Checked By: SES Date: 11/18/11

Resistance force due to friction

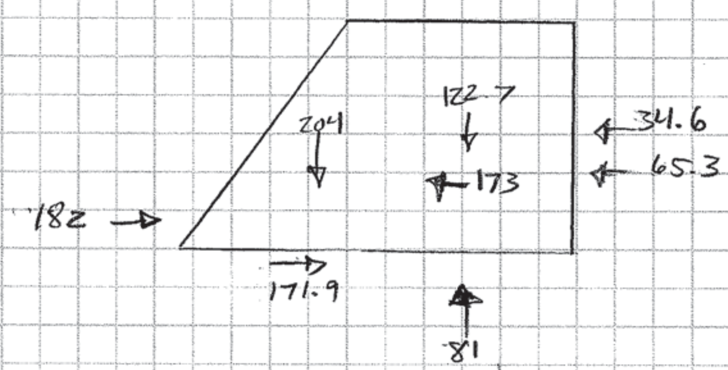
$\Delta F_z = 326.6 \text{ kip/ft} - 81 \text{ kip/ft}$

$\Delta F_z = 245.6 \text{ kip/ft}$

$P_F = (\Delta F_z)(\mu_s) = (245.6 \text{ kip/ft})(0.7)$

$P_F = 171.9 \text{ kip/ft}$

Free Body Diagram



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Client: USAID Job No.: 133-01304-10001 Sheet 5 of

Description: Shikari Dam Stability Analysis Designed By: FJD Date: 11/14/11

Checked By: SES Date: 11/18/11

SLIDING CHECK

$$F.S. = \frac{\text{Resisting Forces}}{\text{Driving Forces}} = \frac{182 + 171.9}{173 + 65.3 + 34.6} = \frac{353.9}{272.9}$$

F.S. = 1.3 > 1.1 OK for sliding

OVERTURNING CHECK

* Moments are calculated about the downstream toe of the dam.

Dead Load $M = (204 \text{ kip/ft})(29.1 \text{ ft}) + (122.7 \text{ kip/ft})(50.19 \text{ ft})$

$= 12{,}095$ kip ft/ft

Hydrostatic $M = (65.3 \text{ kip/ft})(18.53 \text{ ft})$

$= -1210$ kip-ft/ft

Bedrock Passive $M = (182 \text{ kip/ft})(1.1 \text{ ft})$

$= 200.2$ kip·ft/ft

I_{Dam} $M = (173 \text{ kip/ft})(24.7 \text{ ft})$

$= -4{,}273$ kip-ft/ft

Uplift Pressure $M = (81 \text{ kip/ft})(37.8 \text{ ft})$

$= -3{,}062$ kip-ft/ft

$P_{AE\,water}$ $M = (34.6 \text{ kip/ft})(3.28 \text{ ft} + 0.6(45.75 \text{ ft}))$

$= -1063$ kip·ft/ft

$$F.S. = \frac{\text{Resisting Moments}}{\text{Driving Moments}}$$

$$= \frac{112{,}095 + 200.2}{1210 + 4273 + 3062 + 1063}$$

$$= \frac{12{,}295.2}{9{,}608}$$

$F.S. = 1.3 > 1.1$ OK for Overturning

Now, Assume silt is deposited upstream of the dam a height of 2/3 of the NHWL.

Silt Parameters

$\gamma = 100\ lb/ft^3$
$c = 0\ psf$
$\phi' = 28^\circ$

$K_a = \tan^2(45 - \phi'/2) = \tan^2(45 - 28/2)$

$K_a = 0.36$

Active Pressure

$P_A = 1/2(\gamma_s)(height)^2(K_a) + \cancel{2(c)(height)(K_a)^2}$

$= 1/2(100\ lb/ft^3)(30.5ft)^2(0.36)$

$= 16,745\ lb/ft$

$= 16.7\ kip/ft$

Seismic force on Dam from silt

$P_{AE\,silt} = 1/2(\gamma_s)(height)^2(1 - k_v)(K_{AE})$

$= 1/2(100\ lb/ft^3)(30.5ft)^2(0.53) = 24,651\ lb/ft$

$= 24.7\ kip/ft$

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Client: USAID Job No.: 133-01304-10001 Sheet 8 of

Description: Shikari Dam Stability Analysis Designed By: FSD Date: 11/15/11

Checked By: SES Date: 11/18/11

P_A $M = (16.7 \text{ kip/ft})(3.28 \text{ ft} + \frac{1}{3}(30.5 \text{ ft}))$

$= 224.6 \text{ kip-ft/ft}$

$P_{AE_{silt}}$ $M = (24.7 \text{ kip/ft})(3.28 + 0.6(30.5 \text{ ft}))$

$= 533 \text{ kip-ft/ft}$

Sliding $F.S. = \dfrac{353.9}{272.9 + 16.7 + 24.7} = \dfrac{353.9}{314.3}$

$= 1.13$

Overturning $F.S. = \dfrac{12,295.2}{9608 + 224.6 + 533} = \dfrac{12,295.2}{10,365.6}$

$= 1.2$

STATIC ANALYSIS

* Cross - Section drawn in the seismic analysis applies

From Seismic Analysis, static forces:

Dead Load of Dam = 326.6 Kip/ft

Hydrostatic Pressure = 65.3 Kip/ft

Mica Schist Bedrock Passive Pressure = 182.2 Kip/ft

Uplift Pressure = 81 Kip/ft

Resistance Force due to Friction = 171.9 Kip/ft

Silt Active Pressure = 16.7 Kip/ft

From Seismic Analysis, Static Moments:

Dead Load Moment = 12,095 Kip-ft/ft

Hydro static Moment = -1,210 Kip-ft/ft

Bedrock Passive Moment = 200.2 Kip-ft/ft

Uplift Pressure Moment = -3,062 Kip-ft/ft

Silt Active Pressure Moment = -224.6 Kip-ft/ft

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Client: USAID | Job No.: 133-01304-10001 | Sheet 10 of
Description: Shikari Dam Stability Analysis | Designed By: FJD | Date: 11/17/11
Checked By: SES | Date: 11/18/11

Static sliding Check:

$$F.S. = \frac{182.2 + 171.9}{65.3 + 16.7} = \frac{354.1}{82}$$

$$F.S. = 4.3 > 1.5$$

OK for sliding

Static Overturning Check

$$F.S. = \frac{12,095 + 200.2}{1,210 + 3,062 + 224.6} = \frac{12,295.2}{4497}$$

$$F.S. = 2.7 > 1.3$$

OK for overturning

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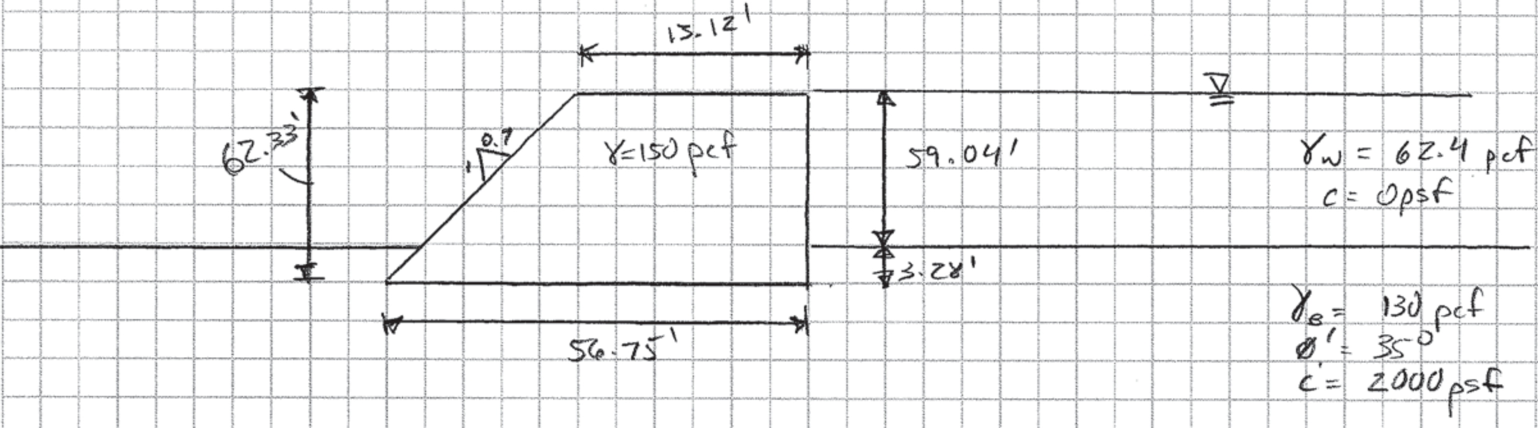
Client: USAID Job No.: 133-01304-10001 Sheet 11 of

Description: Shikori Dam Stability Analysis Designed By: FJD Date: 11/17/11

Checked By: SES Date: 11/18/11

500-Yr Flood Analysis



Silt upstream of Dam is 2/3 of NHWL = 30.5'

From Static Analysis:

Dead Load of Dam = 326.6 Kip-ft/ft

Mica Schist Bedrock Passive Pressure = 182.2 Kip/ft

Silt Active Pressure = 16.7 Kip/ft

Dead Load Moment = 12,095 Kip-ft/ft

Bedrock Passive Force Moment = 200.2 Kip-ft/ft

Silt Active Pressure Moment = -224.6 Kip-ft/ft

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Client: USAID Job No.: 135-01304-10001 Sheet 12 of

Description: Shikari Dam Stability Analysis Designed By: FJD Date: 11/17/11

Checked By: SES Date: 11/18/11

Hydrostatic Force

$P_W = \frac{1}{2}(62.4\ lb/ft^3)(59.04\ ft)^2 = 108{,}754\ lbs/ft$

$= 108.8\ Kip/ft$

Hydrostatic Moment

$P_w M = (108.8\ Kip/ft)(3.28 + \frac{1}{3}(59.04\ ft))$

$= 2{,}498\ Kip\text{-}ft/ft$

Uplift Pressure

$P_u = \frac{1}{2}(62.4\ ft)(59.04\ ft)(56.75\ ft) = 104{,}536\ lbs/ft$

$= 104.5\ Kip/ft$

Uplift Pressure Moment

$P_u M = (104.5\ Kip/ft)(37.8\ ft)$

$= 3{,}950\ Kip\text{-}ft/ft$

Resistance Force due to friction

$P_F = (326.6 - 104.5)(0.7)$

$P_F = 155.5\ Kip/ft$

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Client: USAID | Job No.: 133-01304-10001 | Sheet 13 of

Description: Shikari Dam Stability Analysis | Designed By: FJD | Date: 11/17/11

Checked By: SES | Date: 11/18/11

500 YR FLOOD SLIDING CHECK

$$F.S. = \frac{182.2 + 155.5}{16.7 + 108.8} = \frac{337.7}{125.5}$$

$$F.S. = 2.7 \quad > \quad 1.5$$

OK for Sliding

500 YR FLOOD OVERTURNING CHECK

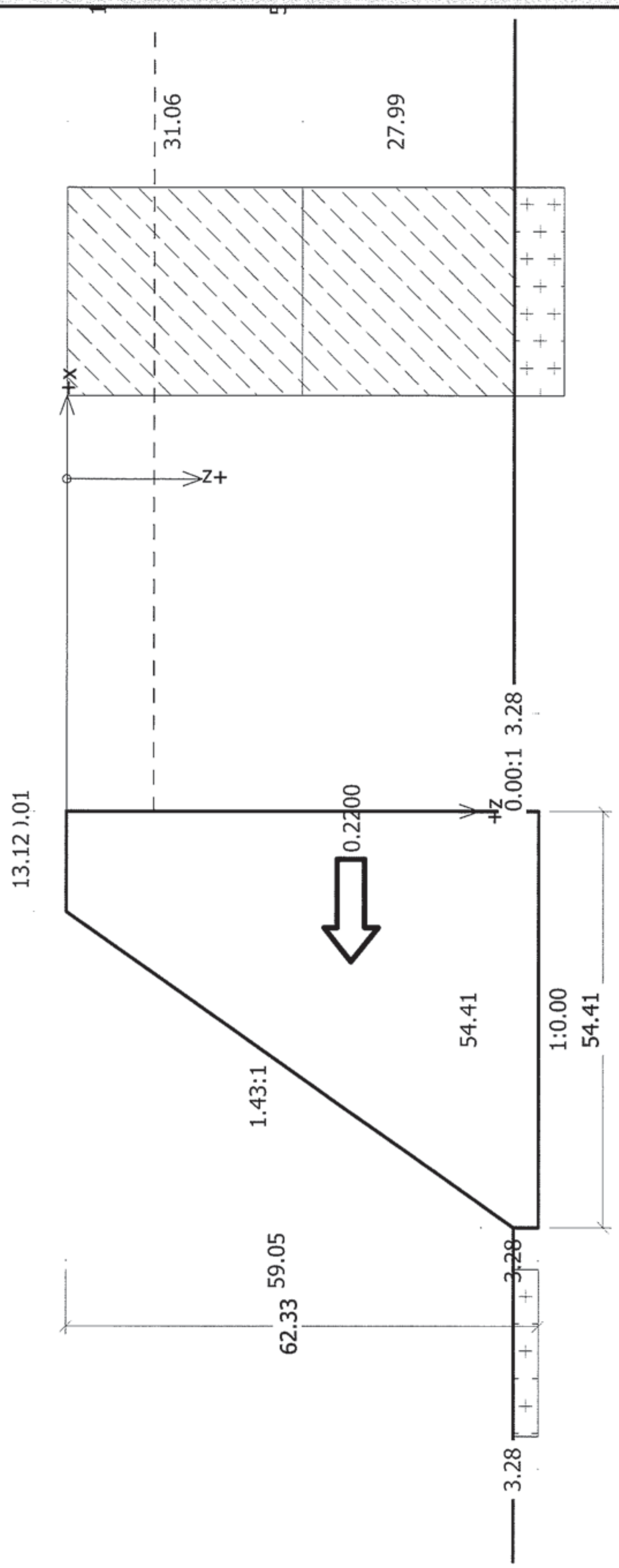
$$F.S. = \frac{12{,}095 + 200.2}{224.6 + 2{,}498 + 3{,}950} = \frac{12{,}295.2}{6{,}673}$$

$$F.S. = 1.8 \quad > \quad 1.3$$

OK for Overturning

Appendix C-3 Dam Stability With Seismic Loading

Name :	Project	Stage : 1



(see Free body diagram, App.C-2)

Gravity wall analysis

Input data

Project

Task : Shikari Dam Stability
Descript. : Seismic Conditions
Author : FJD
Customer : USAID
Date : 11/11/2011

Material of structure

Unit weight γ = 150.0 pcf
Analysis of concrete structures carried out according to the standard ACI 31802.

Concrete : ACI
Compressive strength $f_{c'}$ = 3000.0 psi
Tensile-bending strength f_r = 410.8 psi
Elasticity modulus E_{cm} = 3122.0 ksi

Longitudinal steel : A615/40
Tensile strength f_y = 40000.0 psi
Elasticity modulus E_s = 29000.0 ksi

Geometry of structure

No.	Coordinate X [ft]	Depth Z [ft]
1	0.00	0.00
2	0.00	59.05
3	0.00	62.33
4	-54.41	62.33
5	-54.41	59.05
6	-13.12	0.00

The origin [0,0] is located at the most upper right point of the wall.
Wall section area = 2172.41 ft^2.

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Mica Schist		35.00	2000.0	130.0	87.5	35.00
2	Silt		28.00	0.0	100.0	37.5	0.00

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters
Mica Schist

Unit weight : γ = 130.0 pcf
Stress-state : effective
Angle of internal friction : φ_{ef} = 35.00 °
Cohesion of soil : c_{ef} = 2000.0 psf
Angle of friction struc.-soil : δ = 35.00 °
Soil : cohesionless
Saturated unit weight : γ_{sat} = 150.0 pcf

Silt

Unit weight :	γ = 100.0 pcf
Stress-state :	effective
Angle of internal friction :	φ_{ef} = 28.00 °
Cohesion of soil :	c_{ef} = 0.0 psf
Angle of friction struc.-soil :	δ = 0.00 °
Soil :	cohesionless
Saturated unit weight :	γ_{sat} = 100.0 pcf

Geological profile and assigned soils

No.	Layer [ft]	Assigned soil	Pattern
1	31.06	Silt	
2	27.99	Silt	
3	-	Mica Schist	

Terrain profile

No.	Coordinate X [ft]	Depth Z [ft]
1	0.00	0.00
2	0.01	0.00
3	0.01	59.05
4	1.01	59.05

Origin [0,0] is located in upper right edge of construction.
Positive coordinate +z has downward direction.

Water influence

GWT behind the structure lies at a depth of 11.48 ft
Uplift in foot. bottom due to different pressures is considered as linear.

Resistance on front face of the structure

Resistance on front face of the structure: at rest
Soil on front face of the structure - Mica Schist
Soil thickness in front of structure h = 3.28 ft
Terrain in front of structure is flat.

Earthquake

Horizontal seismic coefficient K_h = 0.2200
Vertical seismic coefficient K_v = 0.0000

Water below the GWT is restricted.

Global settings

Verification methodology : Classical way
Active earth pressure calculation - Coulomb
Passive earth pressure calculation - Caqout-Kerisel
Earthquake analysis theory - Mononobe-Okabe
Standard for concrete structures - ACI 31802

Max. eccentricity of normal force e = 129.49 in
Maximum allowable eccentricity e_{alw} = 215.48 in

Eccentricity of the normal force is SATISFACTORY

Footing bottom bearing capacity verification
Max. stress at footing bottom σ = 7293.1 psf
Bearing capacity of foundation soil R_d = 12000.0 psf

Safety factor = 1.65 > 1.10
Bearing capacity of foundation soil is SATISFACTORY

Overall verification - bearing capacity of found. soil is SATISFACTORY

Dimensioning No. 1

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. Z [ft]	F_{vert} [lbf/ft]	App.Pt. X [ft]	Design coefficient
Weight - wall	0.0	-23.51	299079.9	35.43	1.000
Earthq.- constr.	65797.6	-23.51	0.0	35.43	1.000
Active pressure	15538.0	-22.29	0.0	54.41	1.000
Water pressure	70712.1	-15.86	0.0	54.41	1.000
Uplift pressure	0.0	0.00	-80886.0	36.28	1.000
Earthq.- act.pressure	15321.8	-35.31	0.0	54.41	1.000

Wall stem check

Cross-section depth h = 54.41 ft

Shear : V_u = 167369.4 lbf/ft < ϕV_n = 314721.2 lbf/ft

Utilization is 53.18 %

Combination of flexure and axial load - tension face is decisive

M/S-P/A = 75.207
5fi*sqrt(fc) = 150.624

Utilization of tension face is 49.93 %
Wall bearing capacity at the joint is SATISFACTORY

Settings of the stage of construction

Analysis carried out according to classical theory (safety factor)

Safety factor for slip = 1.10
Safety factor for overturning = 1.10
Factor of safety for bearing capacity = 1.10

Masonry friction reduction factor base-soil μ = 0.70

Verification No. 1

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. Z [ft]	F_{vert} [lbf/ft]	App.Pt. X [ft]	Design coefficient
Weight - wall	0.0	-24.72	325861.4	34.75	1.000
Earthq.- constr.	71689.5	-24.72	0.0	34.75	1.000
FF resistance	-298.2	-1.09	0.0	0.00	1.000
Active pressure	15538.5	-25.57	0.0	54.41	1.000
Water pressure	80803.8	-16.95	0.0	54.41	1.000
Uplift pressure	0.0	0.00	-86466.8	36.28	1.000
Earthq.- act.pressure	19820.4	-36.35	47.6	54.41	1.000

Verification of complete wall

Check for overturning stability
Resisting moment M_{res} = 8189918.2 lbfft/ft
Overturning moment M_{ovr} = 4259301.5 lbfft/ft

Safety factor = 1.92 > 1.10
Wall for overturning is SATISFACTORY

Check for slip
Resisting horizontal force H_{res} = 163325.5 lbf/ft
Active horizontal force H_{act} = 187554.1 lbf/ft

Safety factor = 0.~~87~~ < 1.10
Wall for slip is ~~NOT~~ SATISFACTORY
1.3 > 1.1

Edited per hand calculation (see App. C-2)

Forces acting at the centre of footing bottom
Overall moment M = 2583852.5 lbfft/ft
Normal force N = 239442.2 lbf/ft
Shear force Q = 187554.1 lbf/ft

Overall check - WALL is ~~NOT~~ SATISFACTORY

Bearing capacity of foundation soil

Forces acting at the centre of the footing bottom

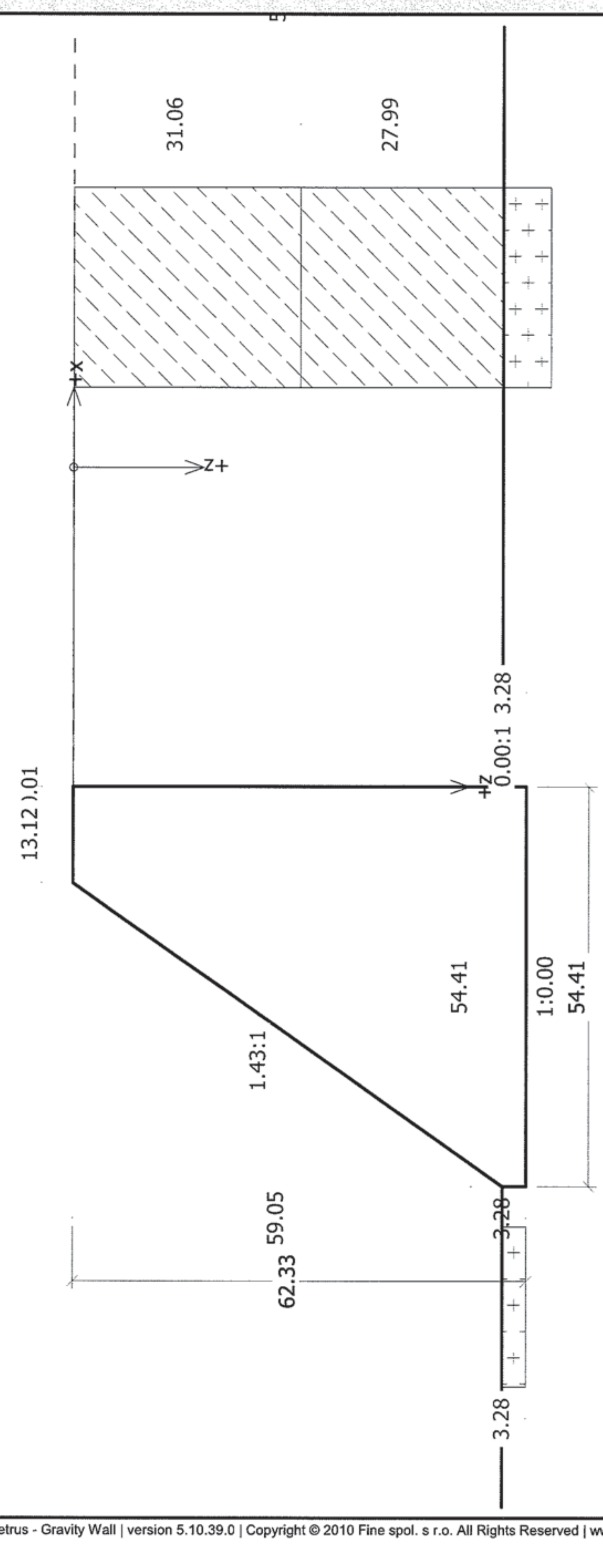
Number	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]	Eccentricity [ft]	Stress [psf]
1	2583852.5	239442.2	187554.1	10.79	7293.1

Bearing capacity of foundation soil check

Eccentricity verification

Appendix C-4 Dam Stability With 500-year Flood Hydrostatic Loading

Name :	Project	Stage : 1



(See Free body diagram, App. C-2)

Gravity wall analysis

Input data

Project
Task : Shikari Dam Stability
Descript. : PMF Conditions
Author : FJD
Customer : USAID
Date : 11/11/2011

Material of structure

Unit weight γ = 150.0 pcf
Analysis of concrete structures carried out according to the standard ACI 31802.

Concrete : ACI
Compressive strength $f_{c'}$ = 3000.0 psi
Tensile-bending strength f_r = 410.8 psi
Elasticity modulus E_{cm} = 3122.0 ksi

Longitudinal steel : A615/40
Tensile strength f_y = 40000.0 psi
Elasticity modulus E_s = 29000.0 ksi

Geometry of structure

No.	Coordinate X [ft]	Depth Z [ft]
1	0.00	0.00
2	0.00	59.05
3	0.00	62.33
4	-54.41	62.33
5	-54.41	59.05
6	-13.12	0.00

The origin [0,0] is located at the most upper right point of the wall.
Wall section area = 2172.41 ft^2.

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Mica Schist		35.00	2000.0	130.0	87.5	35.00
2	Silt		28.00	0.0	100.0	37.5	0.00

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters
Mica Schist
Unit weight : γ = 130.0 pcf
Stress-state : effective
Angle of internal friction : φ_{ef} = 35.00 °
Cohesion of soil : c_{ef} = 2000.0 psf
Angle of friction struc.-soil : δ = 35.00 °
Soil : cohesionless
Saturated unit weight : γ_{sat} = 150.0 pcf

Silt

Unit weight :	γ = 100.0 pcf
Stress-state :	effective
Angle of internal friction :	φ_{ef} = 28.00 °
Cohesion of soil :	c_{ef} = 0.0 psf
Angle of friction struc.-soil :	δ = 0.00 °
Soil :	cohesionless
Saturated unit weight :	γ_{sat} = 100.0 pcf

Geological profile and assigned soils

No.	Layer [ft]	Assigned soil	Pattern
1	31.06	Silt	
2	27.99	Silt	
3	-	Mica Schist	

Terrain profile

No.	Coordinate X [ft]	Depth Z [ft]
1	0.00	0.00
2	0.01	0.00
3	0.01	59.05
4	1.01	59.05

Origin [0,0] is located in upper right edge of construction.
Positive coordinate +z has downward direction.

Water influence

GWT behind the structure lies at a depth of 0.00 ft
Uplift in foot. bottom due to different pressures is considered as linear.

Resistance on front face of the structure

Resistance on front face of the structure: at rest
Soil on front face of the structure - Mica Schist
Soil thickness in front of structure h = 3.28 ft
Terrain in front of structure is flat.

Global settings

Verification methodology : Classical way
Active earth pressure calculation - Coulomb
Passive earth pressure calculation - Caqout-Kerisel
Standard for concrete structures - ACI 31802

Settings of the stage of construction

Analysis carried out according to classical theory (safety factor)

Safety factor for slip = 1.50
Safety factor for overturning = 1.30
Factor of safety for bearing capacity = 2.00

Masonry friction reduction factor base-soil μ = 0.70

Verification No. 1

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. Z [ft]	F_{vert} [lbf/ft]	App.Pt. X [ft]	Design coefficient
Weight - wall	0.0	-24.72	325861.4	34.75	1.000
FF resistance	-298.2	-1.09	0.0	0.00	1.000
Active pressure	9810.2	-22.97	0.0	54.41	1.000
Water pressure	121407.2	-20.78	0.0	54.41	1.000
Uplift pressure	0.0	0.00	-105987.7	36.28	1.000

Verification of complete wall

Check for overturning stability
Resisting moment M_{res} = 7479193.8 lbfft/ft
Overturning moment M_{ovr} = 2747418.3 lbfft/ft

Safety factor = 2.72 > 1.30
Wall for overturning is SATISFACTORY

Check for slip
Resisting horizontal force H_{res} = 168027.3 lbf/ft
Active horizontal force H_{act} = 130919.2 lbf/ft

Safety factor = ~~1.28~~ < 1.50
Wall for slip is ~~NOT~~ SATISFACTORY
2.7 > 1.5

Edited per hand calculation (see App. C-2)

Forces acting at the centre of footing bottom
Overall moment M = 1250297.3 lbfft/ft
Normal force N = 219873.7 lbf/ft
Shear force Q = 130919.2 lbf/ft

Overall check - WALL is ~~NOT~~ SATISFACTORY

Bearing capacity of foundation soil

Forces acting at the centre of the footing bottom

Number	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]	Eccentricity [ft]	Stress [psf]
1	1250297.3	219873.7	130919.2	5.69	5108.5

Bearing capacity of foundation soil check

Eccentricity verification
Max. eccentricity of normal force e = 68.24 in
Maximum allowable eccentricity e_{alw} = 215.48 in

Eccentricity of the normal force is SATISFACTORY

Footing bottom bearing capacity verification
Max. stress at footing bottom σ = 5108.5 psf
Bearing capacity of foundation soil R_d = 12000.0 psf

Safety factor = 2.35 > 2.00

Bearing capacity of foundation soil is SATISFACTORY

Overall verification - bearing capacity of found. soil is SATISFACTORY

Dimensioning No. 1

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. Z [ft]	F_{vert} [lbf/ft]	App.Pt. X [ft]	Design coefficient
Weight - wall	0.0	-23.51	299079.9	35.43	1.000
Active pressure	9809.8	-19.69	0.0	54.41	1.000
Water pressure	108961.2	-19.68	0.0	54.41	1.000
Uplift pressure	0.0	0.00	-100406.6	36.28	1.000

Wall stem check

Cross-section depth h = 54.41 ft

Shear : V_u = 118771.0 lbf/ft < ϕV_n = 314721.2 lbf/ft

Utilization is 37.74 %

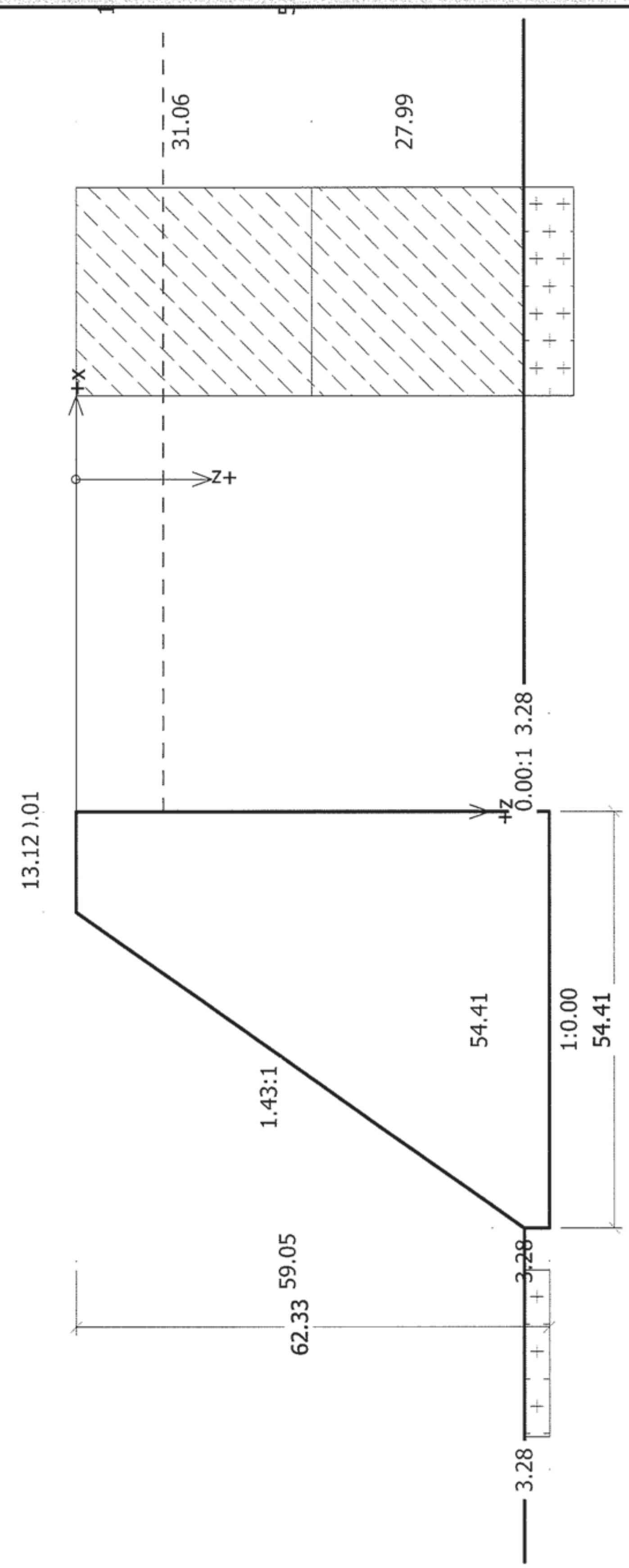
Combination of flexure and axial load - tension face is decisive

M/S-P/A = 19.125
5fi*sqrt(fc) = 150.624

Utilization of tension face is 12.70 %
Wall bearing capacity at the joint is SATISFACTORY

Appendix C-5 Dam Stability With Static Loading

Name :	Project	Stage : 1



(see Free body diagram, App. C-2)

Gravity wall analysis

Input data

Project

Task : Shikari Dam Stability
Descript. : Static Conditions
Author : FJD
Customer : USAID
Date : 11/11/2011

Material of structure

Unit weight γ = 150.0 pcf
Analysis of concrete structures carried out according to the standard ACI 31802.

Concrete : ACI
Compressive strength $f_{c'}$ = 3000.0 psi
Tensile-bending strength f_r = 410.8 psi
Elasticity modulus E_{cm} = 3122.0 ksi

Longitudinal steel : A615/40
Tensile strength f_y = 40000.0 psi
Elasticity modulus E_s = 29000.0 ksi

Geometry of structure

No.	Coordinate X [ft]	Depth Z [ft]
1	0.00	0.00
2	0.00	59.05
3	0.00	62.33
4	-54.41	62.33
5	-54.41	59.05
6	-13.12	0.00

The origin [0,0] is located at the most upper right point of the wall.
Wall section area = 2172.41 ft^2.

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Mica Schist		35.00	2000.0	130.0	87.5	35.00
2	Silt		28.00	0.0	100.0	37.5	0.00

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Mica Schist

Unit weight : γ = 130.0 pcf
Stress-state : effective
Angle of internal friction : φ_{ef} = 35.00 °
Cohesion of soil : c_{ef} = 2000.0 psf
Angle of friction struc.-soil : δ = 35.00 °
Soil : cohesionless
Saturated unit weight : γ_{sat} = 150.0 pcf

Silt

Unit weight :	γ = 100.0 pcf
Stress-state :	effective
Angle of internal friction :	φ_{ef} = 28.00 °
Cohesion of soil :	c_{ef} = 0.0 psf
Angle of friction struc.-soil :	δ = 0.00 °
Soil :	cohesionless
Saturated unit weight :	γ_{sat} = 100.0 pcf

Geological profile and assigned soils

No.	Layer [ft]	Assigned soil	Pattern
1	31.06	Silt	
2	27.99	Silt	
3	-	Mica Schist	

Terrain profile

No.	Coordinate X [ft]	Depth Z [ft]
1	0.00	0.00
2	0.01	0.00
3	0.01	59.05
4	1.01	59.05

Origin [0,0] is located in upper right edge of construction.
Positive coordinate +z has downward direction.

Water influence

GWT behind the structure lies at a depth of 11.48 ft
Uplift in foot. bottom due to different pressures is considered as linear.

Resistance on front face of the structure

Resistance on front face of the structure: at rest
Soil on front face of the structure - Mica Schist
Soil thickness in front of structure h = 3.28 ft
Terrain in front of structure is flat.

Global settings

Verification methodology : Classical way
Active earth pressure calculation - Coulomb
Passive earth pressure calculation - Caqout-Kerisel
Standard for concrete structures - ACI 31802

Settings of the stage of construction

Analysis carried out according to classical theory (safety factor)

Safety factor for slip = 1.50
Safety factor for overturning = 1.30
Factor of safety for bearing capacity = 2.00

Masonry friction reduction factor base-soil μ = 0.70

Verification No. 1

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. Z [ft]	F_{vert} [lbf/ft]	App.Pt. X [ft]	Design coefficient
Weight - wall	0.0	-24.72	325861.4	34.75	1.000
FF resistance	-298.2	-1.09	0.0	0.00	1.000
Active pressure	15538.5	-25.57	0.0	54.41	1.000
Water pressure	80803.8	-16.95	0.0	54.41	1.000
Uplift pressure	0.0	0.00	-86466.8	36.28	1.000

Verification of complete wall

Check for overturning stability
Resisting moment M_{res} = 8187330.8 lbfft/ft
Overturning moment M_{ovr} = 1766648.6 lbfft/ft

Safety factor = 4.63 > 1.30
Wall for overturning is SATISFACTORY

Check for slip
Resisting horizontal force H_{res} = 192435.5 lbf/ft
Active horizontal force H_{act} = 96044.2 lbf/ft

Safety factor = 2.00 > 1.50
Wall for slip is SATISFACTORY

Forces acting at the centre of footing bottom
Overall moment M = 92493.3 lbfft/ft
Normal force N = 239394.7 lbf/ft
Shear force Q = 96044.2 lbf/ft

Overall check - WALL is SATISFACTORY

Bearing capacity of foundation soil

Forces acting at the centre of the footing bottom

Numbe	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]	Eccentricity [ft]	Stress [psf]
1	92493.3	239394.7	96044.2	0.39	4462.9

Bearing capacity of foundation soil check

Eccentricity verification
Max. eccentricity of normal force e = 4.64 in
Maximum allowable eccentricity e_{alw} = 215.48 in

Eccentricity of the normal force is SATISFACTORY

Footing bottom bearing capacity verification
Max. stress at footing bottom σ = 4462.9 psf
Bearing capacity of foundation soil R_d = 12000.0 psf

Safety factor = 2.69 > 2.00

Bearing capacity of foundation soil is SATISFACTORY

Overall verification - bearing capacity of found. soil is SATISFACTORY

Dimensioning No. 1

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. Z [ft]	F_{vert} [lbf/ft]	App.Pt. X [ft]	Design coefficient
Weight - wall	0.0	-23.51	299079.9	35.43	1.000
Active pressure	15538.0	-22.29	0.0	54.41	1.000
Water pressure	70712.1	-15.86	0.0	54.41	1.000
Uplift pressure	0.0	0.00	-80886.0	36.28	1.000

Wall stem check

Cross-section depth h = 54.41 ft

Shear : V_u = 86250.1 lbf/ft < ϕV_n = 314721.2 lbf/ft

Utilization is 27.41 %

Combination of flexure and axial load - compression face is decisive

Flexure : M_u = 3084675.0 lbfft ; ϕM_n = 1.759E+09 lbfin
Pressure : P_u = 218193.9 lbf/ft ; ϕP_n = 12928492.2 lbf/ft

Utilization of compressive face is 1.86 %
Wall bearing capacity at the joint is SATISFACTORY

Appendix D Hydrology Calculations

Table 1. USGS Gage Flow Data for Foladi River

Percentage of days discharge equaled or exceeded	Month												Annual
	October	November	December	January	February	March	April	May	June	July	August	September	
	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)
95	0.49	0.20	0.16	0.51	0.39	0.24	0.03	0.04	0.44	0.09	0.08	0.52	0.09
90	0.50	0.28	0.23	0.52	0.42	0.26	0.06	0.06	0.61	0.10	0.09	0.57	0.22
85	0.60	0.32	0.27	0.53	0.47	0.31	0.08	0.16	0.89	0.22	0.41	0.62	0.32
80	0.71	0.37	0.44	0.57	0.49	0.34	0.09	0.26	1.27	0.46	0.46	0.66	0.45
75	0.72	0.40	0.48	0.59	0.54	0.40	0.11	0.49	1.68	0.82	0.50	0.70	0.50
70	0.73	0.47	0.68	0.64	0.57	0.43	0.15	0.56	2.01	0.90	0.52	0.73	0.55
65	0.75	0.50	0.73	0.67	0.60	0.48	0.20	0.92	2.28	0.95	0.54	0.75	0.60
60	0.76	0.58	0.77	0.70	0.64	0.51	0.22	1.33	2.52	1.00	0.59	0.78	0.64
55	0.90	0.68	0.83	0.75	0.67	0.54	0.24	1.57	2.74	1.17	0.61	0.80	0.70
50	0.97	0.72	0.92	0.79	0.73	0.57	0.27	1.72	2.94	1.36	0.63	0.83	0.76
45	0.99	0.73	0.96	0.81	0.74	0.60	0.31	1.83	3.57	1.46	0.65	0.87	0.82
40	1.03	0.75	0.99	0.86	0.76	0.62	0.40	1.94	4.13	1.78	0.73	0.91	0.89
35	1.19	0.77	1.04	0.91	0.79	0.65	0.45	2.10	4.57	1.98	0.82	0.96	0.97
30	1.20	0.81	1.13	0.98	0.84	0.69	0.55	2.32	5.37	2.37	0.93	1.01	1.09
25	1.22	0.90	1.18	1.01	0.88	0.75	0.71	2.66	6.13	2.45	1.17	1.18	1.23
20	1.23	1.14	1.21	1.02	0.92	0.81	0.81	3.22	6.77	2.54	1.25	1.38	1.43
15	1.25	1.21	1.23	1.04	0.96	0.84	0.93	3.68	7.71	2.64	1.32	1.46	1.99
10	1.28	1.24	1.26	1.22	0.99	0.88	1.32	4.34	9.73	2.99	2.11	1.56	2.52
5	1.33	1.28	1.51	1.33	1.18	0.96	1.7	5.39	11.1	4.14	ng	1.71	4.14

Table 2. USGS Gage Flow Data for Bamyan River near Bamyan

Percentage of days discharge equaled or exceeded	Month												Annual
	October	November	December	January	February	March	April	May	June	July	August	September	
	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)
95	0.68	0.44	0.32	0.21	0.21	0.38	0.19	0.35	0.42	0.19	0.24	0.55	0.29
90	0.73	0.53	0.36	0.28	0.30	0.42	0.30	0.48	0.45	0.25	0.30	0.63	0.36
85	0.75	0.61	0.37	0.29	0.31	0.43	0.33	0.54	0.47	0.30	0.42	0.64	0.41
80	0.79	0.64	0.37	0.30	0.37	0.46	0.39	0.59	0.48	0.38	0.49	0.68	0.46
75	0.81	0.70	0.39	0.34	0.41	0.49	0.44	0.63	0.57	0.46	0.56	0.68	0.52
70	0.87	0.75	0.41	0.37	0.42	0.53	0.58	0.69	0.61	0.53	0.63	0.69	0.58
65	0.89	0.84	0.42	0.42	0.43	0.58	0.63	0.76	0.65	0.61	0.64	0.75	0.64
60	0.91	0.89	0.48	0.49	0.56	0.69	0.66	0.85	0.70	0.67	0.67	0.77	0.71
55	0.93	0.97	0.54	0.55	0.61	0.82	0.69	0.95	0.76	0.71	0.73	0.80	0.76
50	0.95	1.00	0.61	0.57	0.62	0.88	0.72	1.03	0.80	0.73	0.77	0.81	0.82
45	1.04	1.03	0.67	0.60	0.67	0.94	0.77	1.12	0.87	0.76	0.79	0.86	0.86
40	1.09	1.10	0.74	0.63	0.69	0.97	0.88	1.27	0.94	0.79	0.81	0.88	0.90
35	1.12	1.14	0.82	0.73	0.76	1.02	1.01	1.53	1.01	0.82	0.82	0.90	0.98
30	1.13	1.29	0.96	0.83	0.79	1.09	1.12	1.67	1.12	0.96	0.84	0.91	1.05
25	1.15	1.35	1.12	0.98	0.81	1.17	1.41	2.02	1.27	1.09	0.89	0.92	1.10
20	1.16	1.40	1.32	1.03	0.85	1.51	1.76	2.31	1.82	1.16	0.92	0.94	1.21
15	1.32	1.45	1.39	1.10	0.90	1.60	2.15	2.67	2.42	1.37	1.02	1.04	1.41
10	1.88	1.87	1.44	1.25	0.96	1.99	2.49	2.90	3.72	1.79	1.10	1.18	1.79
5	2.01	1.99	1.65	1.54	1.14	2.31	2.83	3.19	4.52	2.15	1.14	1.21	2.33

Table 3. Flow-Duration Estimates at Shikari Dam based on weighted area averages from Foladi River and Bamyan River

Percentage of days discharge equaled or exceeded	Month												Annual
	October	November	December	January	February	March	April	May	June	July	August	September	
	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)
95%	2.22	1.10	0.84	1.71	1.37	1.15	0.32	0.54	1.76	0.49	0.52	2.15	0.61
90%	2.31	1.44	1.09	1.82	1.56	1.25	0.53	0.75	2.29	0.59	0.62	2.39	1.06
85%	2.62	1.65	1.22	1.86	1.72	1.41	0.63	1.11	3.11	0.99	1.68	2.55	1.41
80%	2.98	1.83	1.70	1.99	1.85	1.53	0.73	1.46	4.21	1.77	1.91	2.71	1.84
75%	3.04	1.99	1.84	2.10	2.04	1.74	0.85	2.16	5.49	2.90	2.11	2.82	2.06
70%	3.14	2.25	2.44	2.28	2.14	1.87	1.13	2.43	6.48	3.21	2.25	2.92	2.27
65%	3.22	2.44	2.59	2.42	2.23	2.07	1.33	3.55	7.30	3.45	2.32	3.05	2.49
60%	3.27	2.73	2.78	2.59	2.51	2.29	1.43	4.83	8.04	3.67	2.50	3.16	2.69
55%	3.70	3.12	3.02	2.81	2.65	2.53	1.52	5.63	8.75	4.20	2.63	3.25	2.92
50%	3.92	3.27	3.37	2.95	2.84	2.69	1.64	6.16	9.37	4.77	2.73	3.35	3.16
45%	4.09	3.33	3.55	3.04	2.92	2.85	1.82	6.58	11.25	5.09	2.81	3.53	3.38
40%	4.26	3.47	3.72	3.22	3.01	2.94	2.21	7.08	12.93	6.04	3.07	3.66	3.63
35%	4.75	3.58	3.96	3.48	3.18	3.09	2.51	7.85	14.28	6.65	3.33	3.83	3.96
30%	4.79	3.87	4.39	3.80	3.36	3.29	2.93	8.65	16.69	7.93	3.67	3.99	4.38
25%	4.88	4.20	4.73	4.07	3.49	3.56	3.73	10.04	19.05	8.32	4.42	4.48	4.84
20%	4.92	4.95	5.05	4.16	3.66	4.14	4.44	11.99	21.54	8.66	4.68	5.08	5.55
15%	5.17	5.21	5.20	4.30	3.83	4.34	5.26	13.74	24.95	9.20	5.00	5.43	7.39
10%	5.93	5.80	5.34	5.00	3.99	4.92	6.78	15.90	32.29	10.71	7.36	5.88	9.36
5%	6.23	6.06	6.31	5.66	4.75	5.54	8.28	19.25	37.17	14.43	ng	6.35	14.64

Appendix E Conceptual Design Drawings

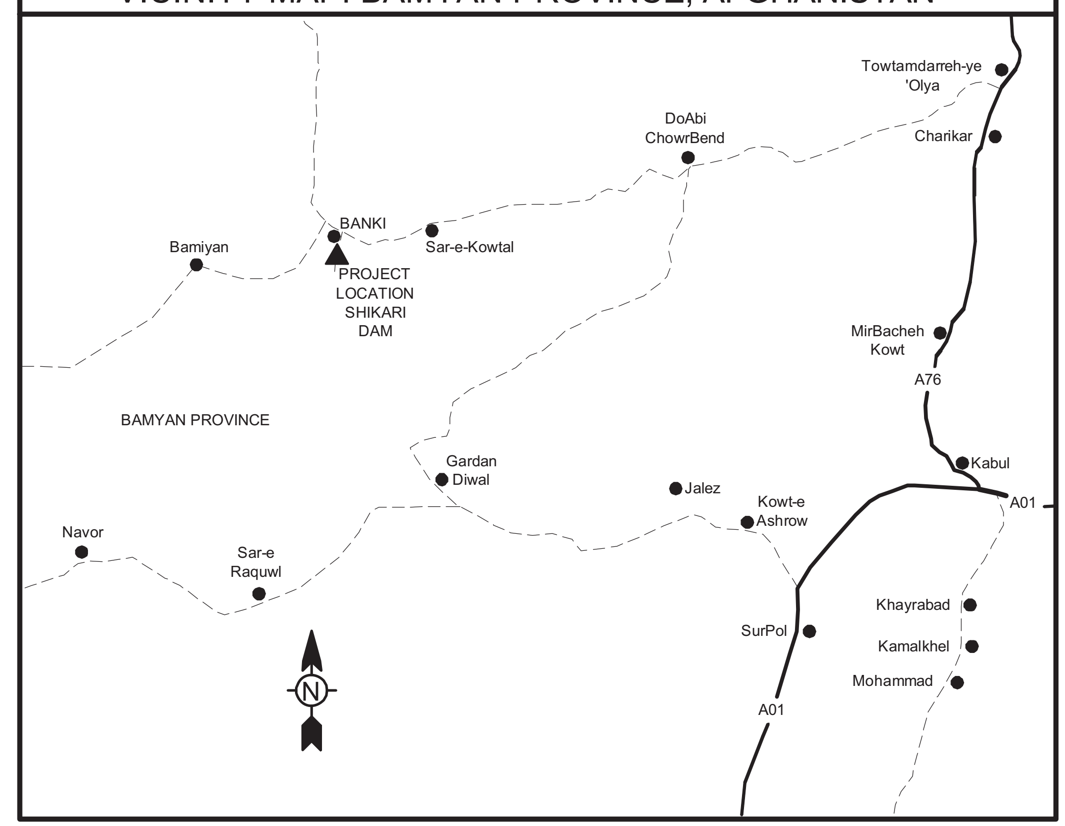
LEGEND

- BENCHMARK
- BOREHOLE — BH-3
- CENTERLINE ROAD
- DESIGN MAJOR CONTOUR
- DESIGN MINOR CONTOUR
- EXISTING ROAD
- EXISTING MAJOR CONTOUR
- EXISTING MINOR CONTOUR
- GATE VALVE
- RIVER CENTERLINE
- RIVER EDGE
- ROAD IMPROVEMENTS

SHEET INDEX

SHT #	SHEET TITLE
G-01	COVER SHEET
C-01	EXISTING CONDITIONS
C-02	DAM AND PENSTOCK PLAN
C-03	CONCRETE DAM PLAN VIEW
C-04	CENTERLINE DAM PROFILE
C-05	CONCRETE DAM SECTIONS
C-06	FOUNDATION PLAN
C-07	FOUNDATION SECTION
C-08	BORE HOLE LOGS BH-01 AND BH-02
C-09	BORE HOLE LOG BH-03

VICINITY MAP: BAMYAN PROVINCE, AFGHANISTAN



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CONCEPT DESIGN

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	WO-LT-0009-002 SUBMITTAL		

DESIGNED BY: AK
DATE: 11/18/11
DWN BY: MAM
SUBMITTED BY: TETRA TECH
CHK BY: RJT
FILE NO.: G-1 Cover Sheet Shekari Dam

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PRT SUPPORT PROGRAM
SHIKARI VALLEY DAM
COVER SHEET

SHEET REFERENCE NUMBER:
G-1

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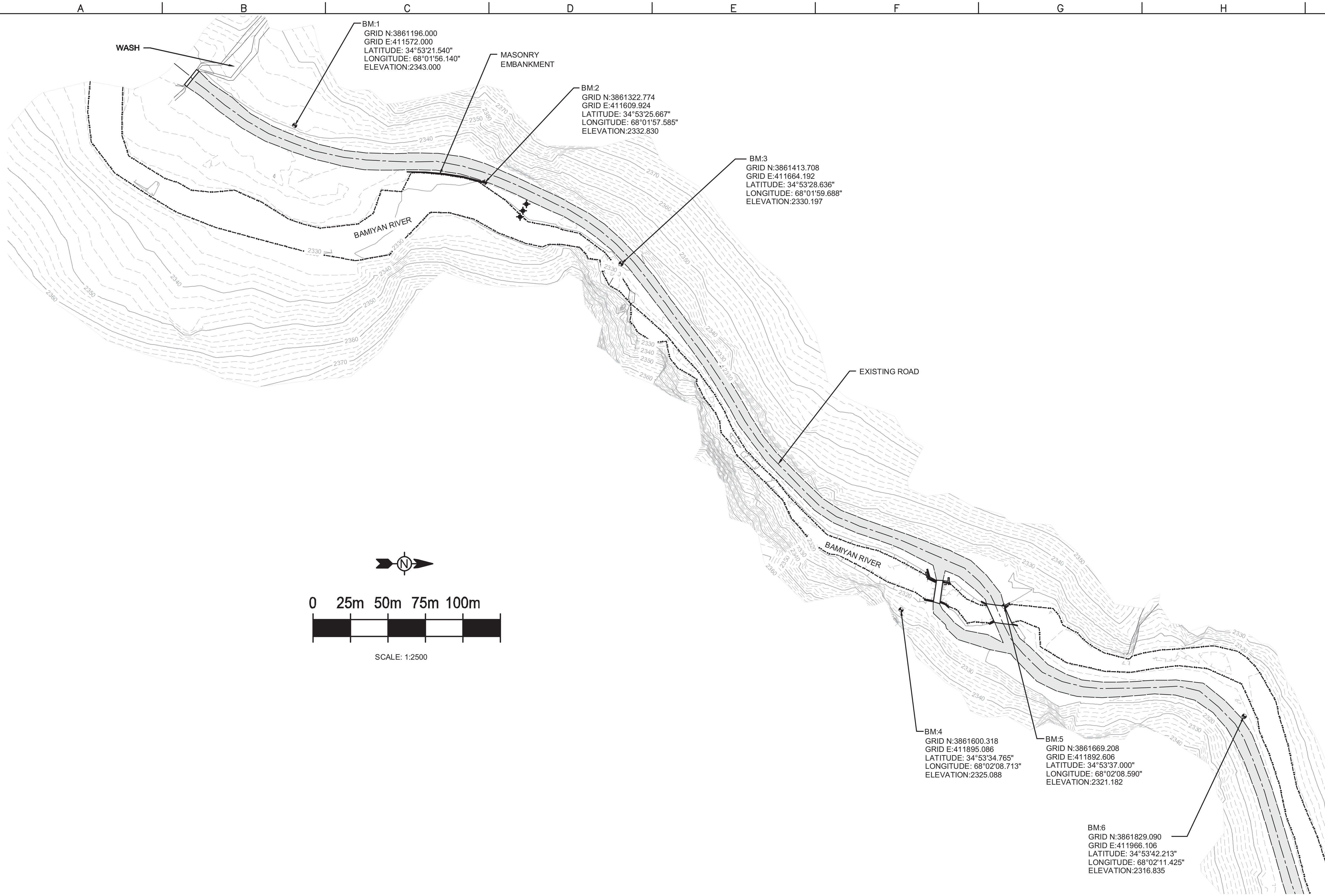

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	WO-LT-0009-002 SUBMITTAL		

DESIGNED BY: AK
DATE: 11/18/11
DWN BY: MAM
SUBMITTED BY: TETRA TECH
CHK BY: RJT
FILE NO.: C-01 EXISTING CONDITIONS REVISED

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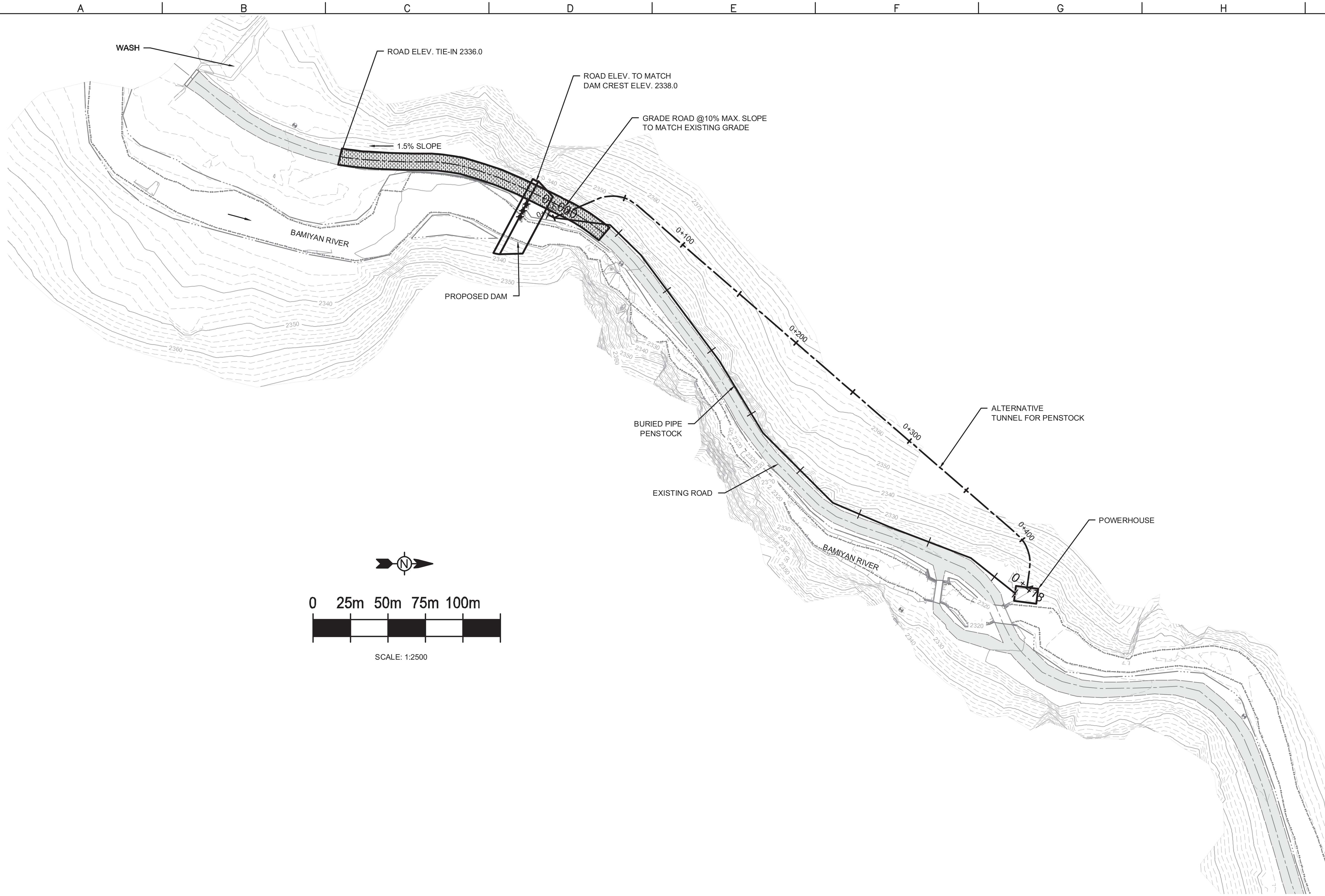
SHIKARI VALLEY DAM
EXISTING CONDITIONS

SHEET
REFERENCE
NUMBER:
C-01



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	WO-LT-0009-002 SUBMITTAL		

DESIGNED BY: AK	DATE: 11/18/11	
DWN BY: MAM	SUBMITTED BY: TETRA TECH	
CHK BY: RJT	FILE NO.: C-02 DAM AND TUNNEL PLAN REVISED	

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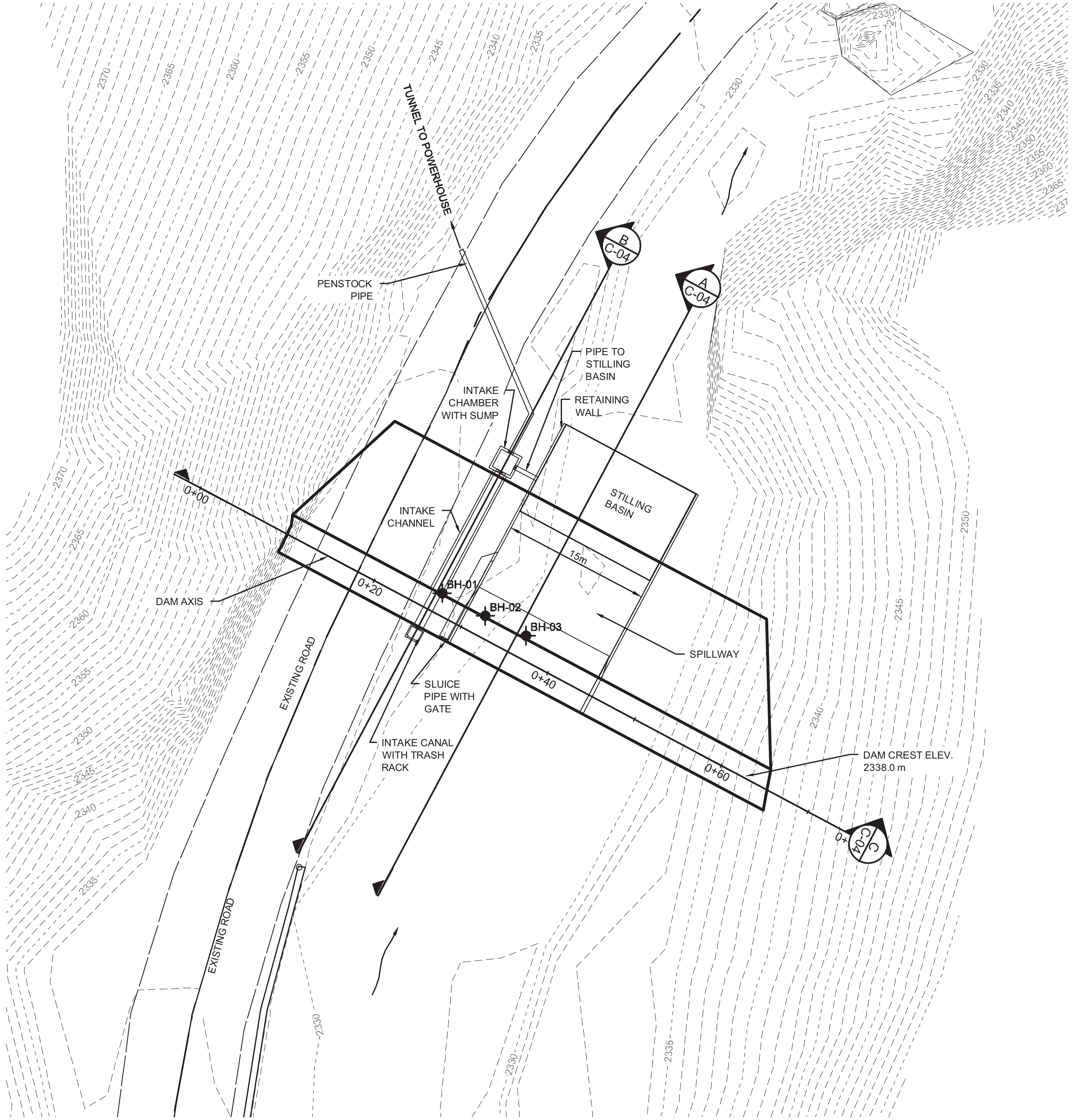
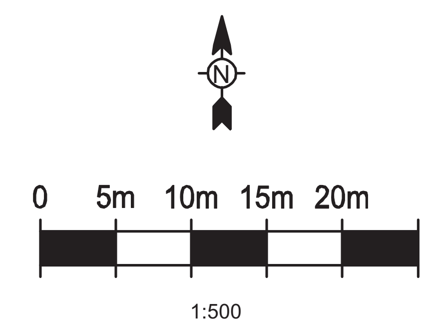
NOT FOR CONSTRUCTION

USAID/OIEE-AFGHANISTAN
PRT SUPPORT PROGRAM
SHIKARI VALLEY DAM
DAM AND PENSTOCK PLAN

SHEET REFERENCE NUMBER:
C-02



CONCRETE DAM PLAN VIEW



UNLESS OTHERWISE NOTED, ALL
DIMENSIONS SHOWN ARE IN METERS.

CONCEPT DESIGN

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SYMB	DESCRIPTION	DATE	APR
	TASK 3 SUBMITTAL		
	WO-LT-0009-002 SUBMITTAL		

DESIGNED BY: AK
DATE: 11/18/11
DWN BY: MAM
SUBMITTED BY: TETRA TECH
CHK BY: RJT
FILE NO.: C-03 CONCRETE DAM PLAN VIEW

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PRT SUPPORT PROGRAM
SHIKARI VALLEY DAM
CONCRETE DAM PLAN VIEW

SHEET REFERENCE NUMBER:
C-03

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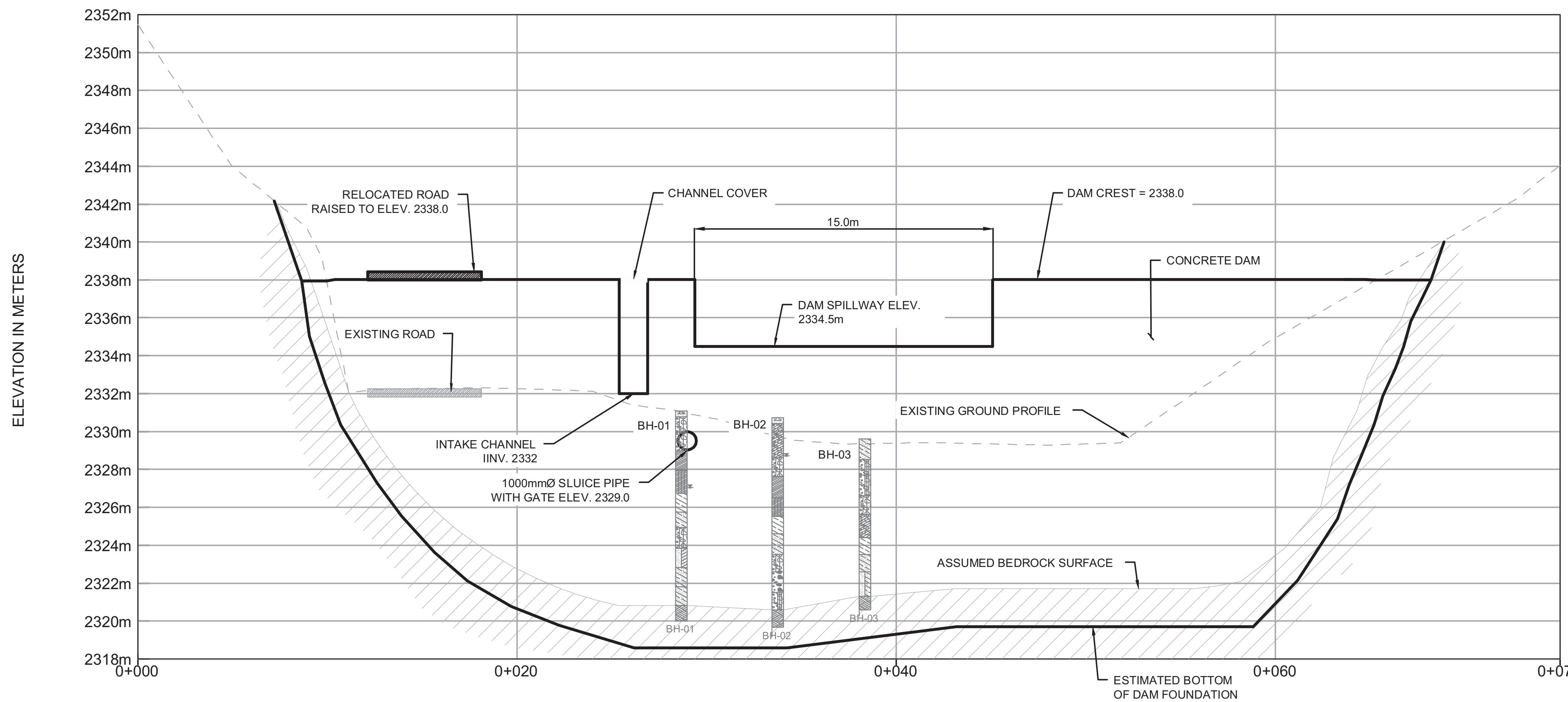

C
C-03
PROFILE ALONG DAM AXIS

LEGEND

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

- TOPSOIL
- CL: USCS Low Plasticity Clay
- CL-ML: USCS Low Plasticity Silty Clay
- ML: USCS Silt
- SM: USCS Silty Sand
- SP-SC: USCS Poorly-graded Sand with Clay
- SP-SM: USCS Poorly-graded Sand with Silt
- GC: USCS Clayey Gravel
- GM: USCS Silty Gravel
- GP-GM: USCS Poorly-graded Gravel with Silt
- GW: USCS Well-graded Gravel
- BEDROCK: MICA SCHIST Bedrock
- Water Level at End of Drilling, or as Shown

0 3m 6m 9m 12m

SCALE: 1:300

UNLESS OTHERWISE NOTED, ALL DIMENSIONS SHOWN ARE IN METERS.

CONCEPT DESIGN

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	TASK 3 SUBMITTAL		
	WO-LT-0009-002 SUBMITTAL		

DESIGNED BY: AK | DATE: 11/18/11
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SHIKARI VALLEY DAM
CENTERLINE DAM PROFILE

SHEET REFERENCE NUMBER:
C-04

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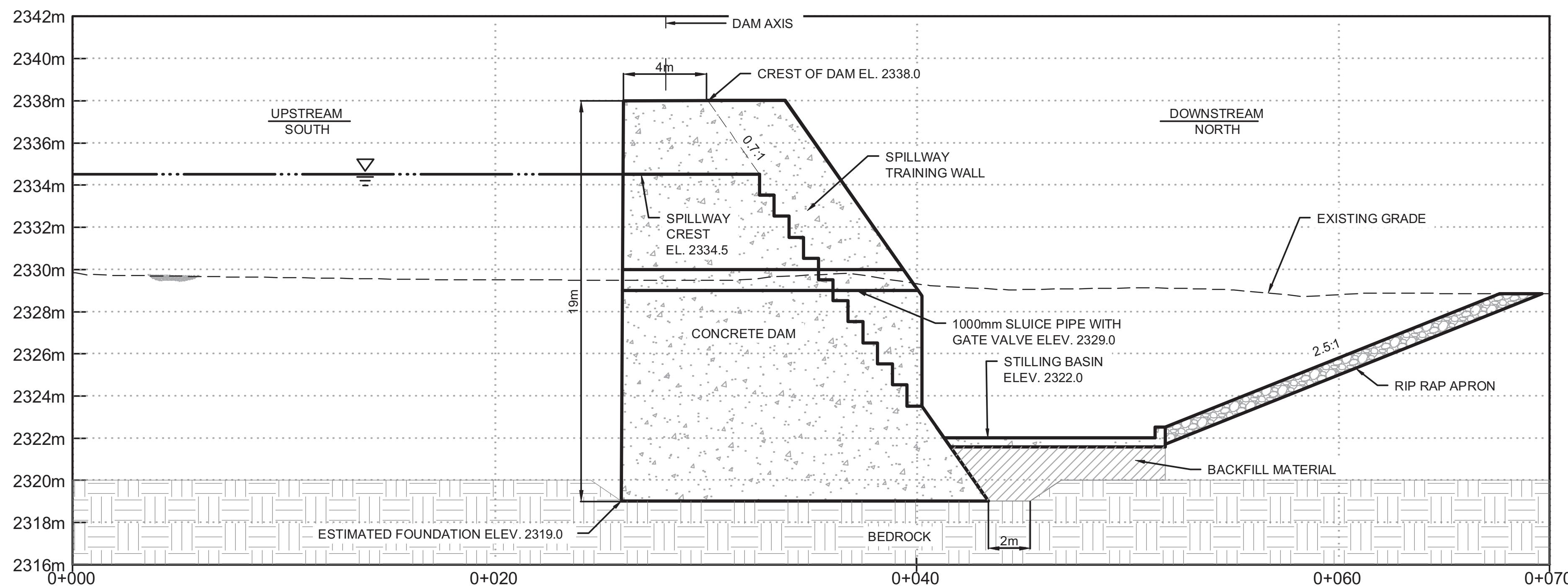
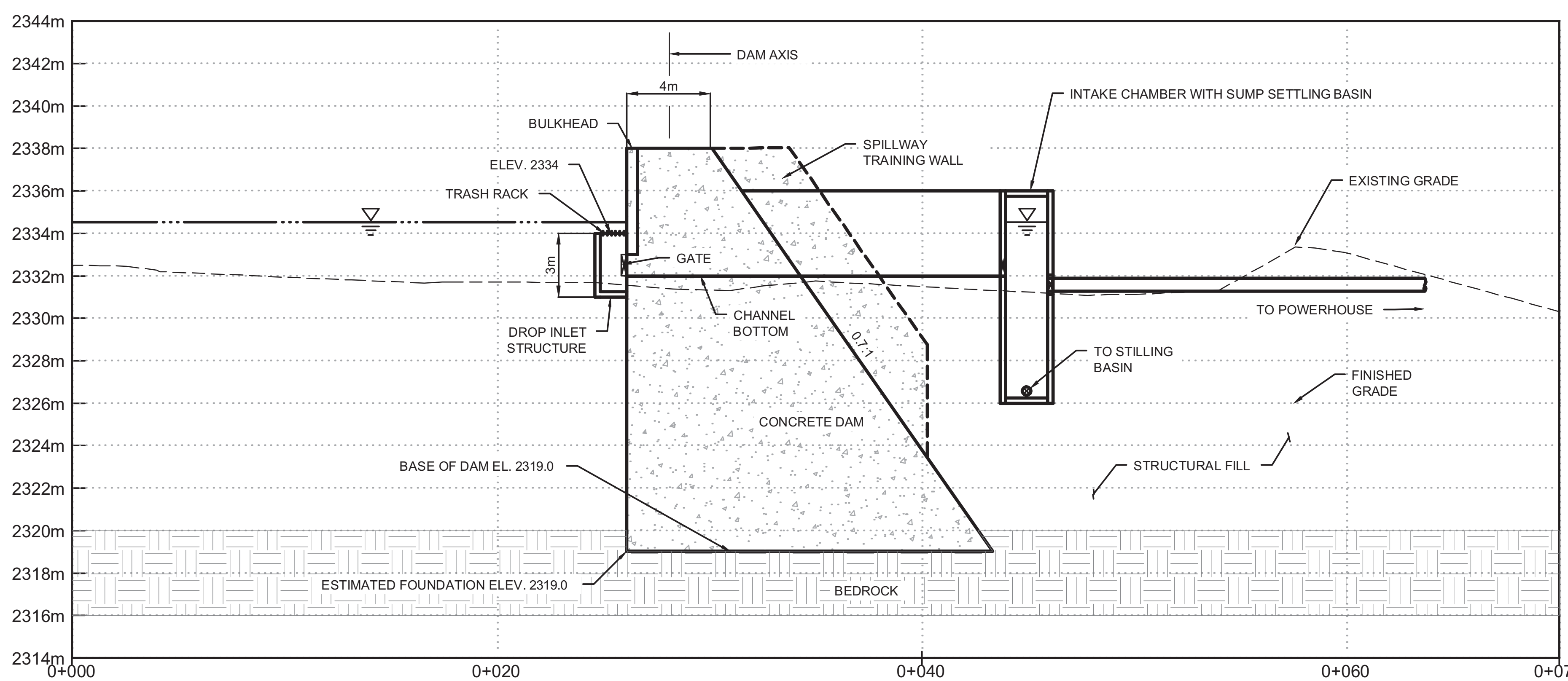
CONCEPT DESIGN



A
C-03
CONCRETE DAM SECTION THROUGH SPILLWAY

0 3m 6m 9m 12m

HOR. SCALE= 1:300
VERT. SCALE= 1:300



B
C-03
INTAKE STRUCTURE SECTION

0 3m 6m 9m 12m

HOR. SCALE= 1:300
VERT. SCALE= 1:300

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SYMB	DESCRIPTION	DATE	APR
	TASK 3 SUBMITTAL		
	WO-LT-0009-002 SUBMITTAL		

DESIGNED BY: AK	DATE: 11/18/11	
DWN BY: MAM	SUBMITTED BY: TETRA TECH	
CHK BY: RJT	FILE NO.: C-04 CONCRETE DAM SECTION REVISED	

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SHIKARI VALLEY DAM
CONCRETE DAM SECTIONS

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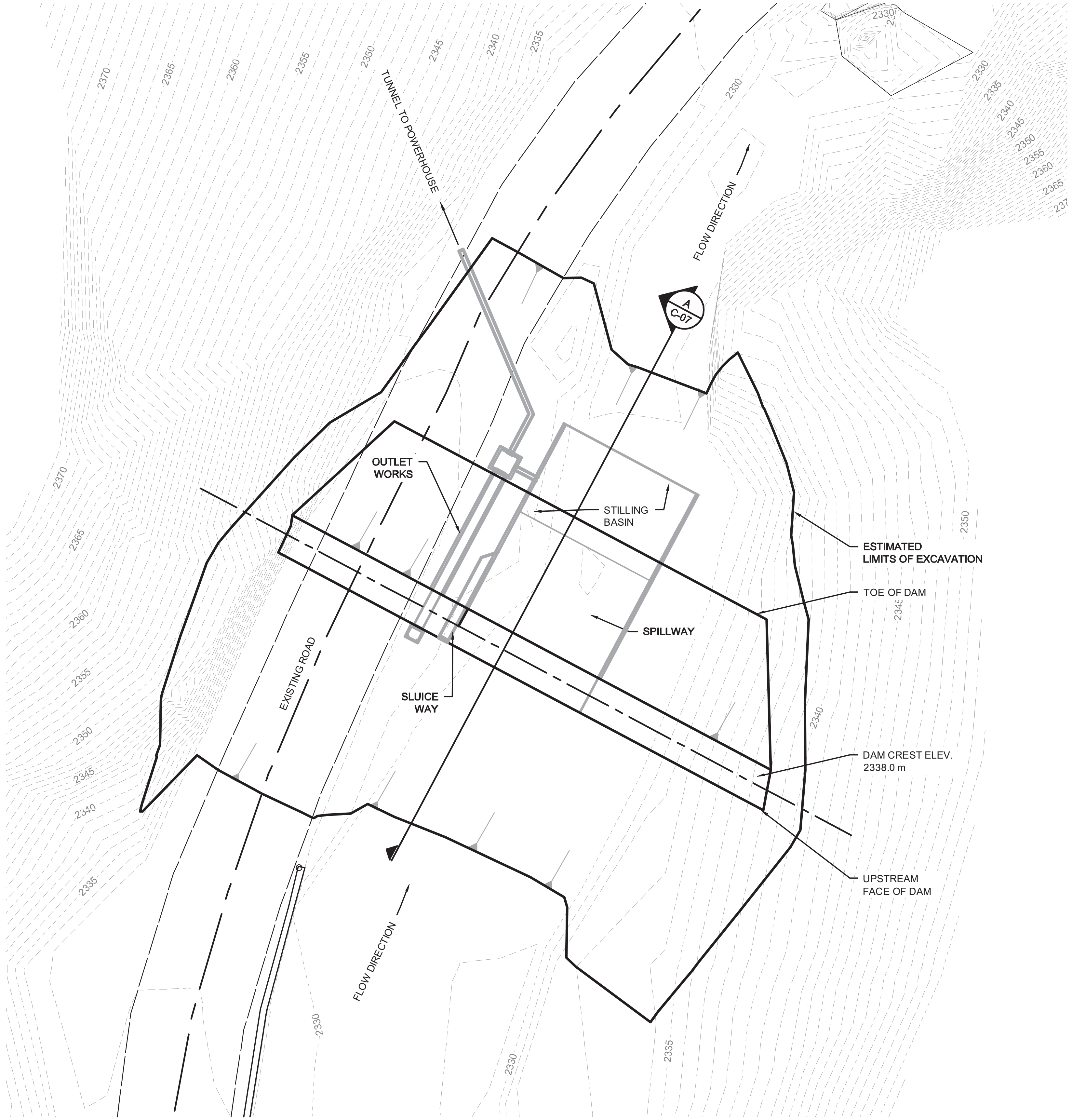
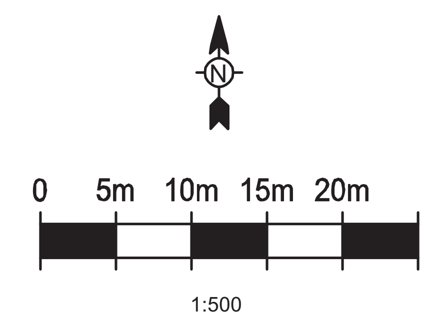
C-05



CONCRETE DAM PLAN VIEW



NOTE:

1. STREAM DIVERSION DURING CONSTRUCTION IS TO BE RESPONSIBILITY OF THE CONSTRUCTION CONTRACTOR. THE CONTRACTOR SHOULD PROVIDE DIVERSION CAPACITY EQUAL TO THE 5-YEAR RETURN INTERVAL FLOOD FOR THE SEASON(S) DURING WHICH THE CONSTRUCTION IS BEING PERFORMED. METHODS FOR DIVERSION DEVELOPED BY THE CONTRACTOR SHALL BE SUBMITTED FOR REVIEW AND COMMENT PRIOR TO DIVERSION CONSTRUCTION. METHODS FOR DIVERSION HAVE BEEN CONSIDERED. ONE SUCH METHOD IS TO PROVIDE CONTAINMENT OF THE STREAM ON THE RIGHT SIDE OF THE VALLEY EITHER IN A CONSTRUCTED CHANNEL OR A PIPE. THE LEFT VALLEY WOULD BE EXCAVATED TO FOUNDATION DEPTHS FROM THE LEFT ABUTMENT TO ABOUT THE SPILLWAY LOCATION, SUFFICIENT TO CONSTRUCT THE DAM AND THE SLUICEWAY PIPE. ONCE THE SLUICEWAY IS COMPLETED AND THE DAM IS CONSTRUCTED TO A LEVEL SUFFICIENT TO CONTAIN THE STREAM FLOW, THE STREAM COULD BE ROUTED THROUGH THE SLUICEWAY AND THE RIGHT SIDE OF THE VALLEY CAN BE EXCAVATED.

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CHK BY: RJT | FILE NO.: C-06 FOUNDATION PLAN

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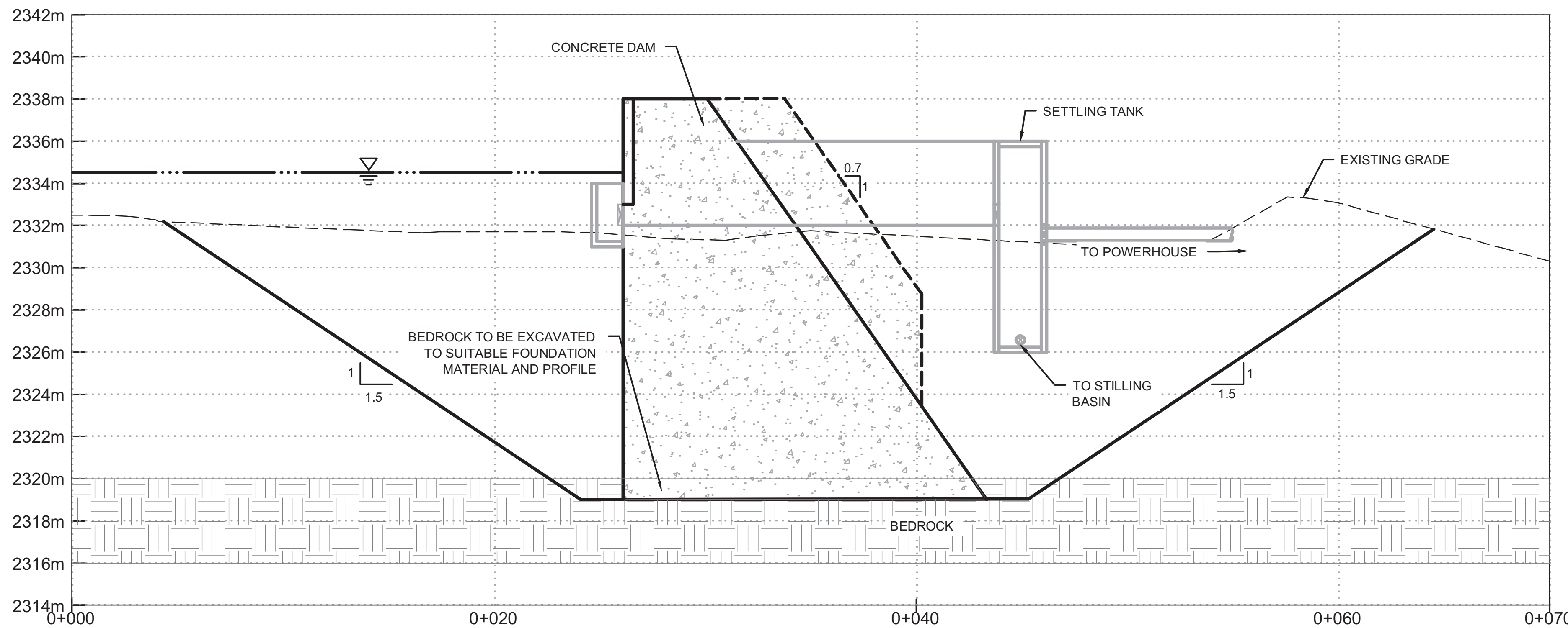
PRT SUPPORT PROGRAM
SHIKARI VALLEY DAM
FOUNDATION PLAN

SHEET REFERENCE NUMBER:
C-06



A C-06 FOUNDATION SECTION

0 3m 6m 9m 12m

SCALE: 1:300

UNLESS OTHERWISE NOTED, ALL DIMENSIONS SHOWN ARE IN METERS.

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SYMB	DESCRIPTION	DATE	APR
	TASK 3 SUBMITTAL		
	WO-LT-0009-002 SUBMITTAL		

DESIGNED BY: AK
DATE: 11/18/11
DWN BY: MAM
SUBMITTED BY: TETRA TECH
CHK BY: RJT
FILE NO.: C-08 FOUNDATION SECTION

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PRT SUPPORT PROGRAM
SHIKARI VALLEY DAM
FOUNDATION SECTION

SHEET REFERENCE NUMBER:
C-07

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P:\01304\133-01304-10001\CAD\SheetFiles\Shikari\G-2 BORE HOLE LOGS.dwg 11/18/2011 9:32:51 AM Weatherl, Laura

Tetra Tech Inc
1900 South Sunset Street 1-F
Longmont, CO. 80501
Telephone: (303) 772-5282
Fax: (303) 772-7039

BOREHOLE BH-01

PAGE 1 OF 1

CLIENT USAID
PROJECT NAME Shikari Valley Dam
PROJECT NUMBER 133-01304-10001
PROJECT LOCATION Shikari Valley, Bamyan Province, Afghanistan
DATE/TIME STARTED 7/2/2011 COMPLETED 7/12/2011
GROUND ELEVATION 2331.131 m HOLE SIZE 4.5
DRILLING CONTRACTOR Geo search
GROUND WATER LEVELS:
DRILLING METHOD 11.4 cm Rotary Core & 5.3 cm wireline Core
AT TIME OF DRILLING ---
LOGGED BY H. Jawid CHECKED BY R. Tocher
▼ AT END OF DRILLING 4.00 m / Elev 2327.13 m
NOTES
COORDINATES 0N, 1 E

DEPTH (m)	U.S.C.S.	MATERIAL DESCRIPTION	Elev.
0.3			2330.8
1.1	GM	silty gravel with sand, very dense	2330.0
2.1	SM	silty sand with gravel, medium dense	2329.0
3.1	SW-SM	well graded sand with silt, dense	2328.0
4.3	ML	sandy silt with gravel, medium dense	2326.8
5.3	GM	silty gravel with sand, very dense	2325.8
6.1	SM	silty sand with gravel, meduim dense	2325.0
7.2	GM	silty gravel with sand, very dense	2323.9
8.2	SP-SC	poorly graded sand with clay or (silty clay), very stiff	2322.9
9.2	SC-SM	silty clayey sand, very stiff	2321.9
10.2	SM	silty sand, medium dense	2320.9
11.0		bedrock, mica schist	2320.1

SAMPLE TYPE NUMBER	BLOW COUNTS/15 cm (N VALUE)
SS	45-50-50/15cm
SS	10-11-13/15cm
SS	15-17-18/15cm
SS	4-7-7/15cm
SS	26-29-45/15cm
SS	7-9-11/15cm
SS	24-27-36/15cm
SS	8-10-13/15cm
SS	8-9-12/15cm
SS	5-8-8/15cm

Bottom of borehole at 11.0 meters.

TETRA TECH STANDARD LOG BORING AND TP - TETRA TECH 04-29-10.GDT - 11/17/11 16:16 - P:\01304\133-01304-10001\SHIKARI\SUPPORTDOCS\EXISTSITEDATA\JULY 2011 DRILLING\SHIKARI VALLEY DAM.GPJ

0 .75m 1.5m 2.25m 3m

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Fax: (303) 772-7039

BOREHOLE BH-02

PAGE 1 OF 1

CLIENT USAID
PROJECT NAME Shikari Valley Dam
PROJECT NUMBER 133-01304-10001
PROJECT LOCATION Shikari Valley, Bamyan Province, Afghanistan
DATE/TIME STARTED 7/2/2011 COMPLETED 7/12/2011
GROUND ELEVATION 2330.397 m HOLE SIZE 4.5
DRILLING CONTRACTOR Geo search
GROUND WATER LEVELS:
DRILLING METHOD 11.4 cm Rotary Core & 5.3 cm wireline Core
AT TIME OF DRILLING ---
LOGGED BY H. Jawid CHECKED BY R. Tocher
▼ AT END OF DRILLING 2.00 m / Elev 2328.40 m
NOTES
COORDINATES 0N, 2 E

DEPTH (m)	U.S.C.S.	MATERIAL DESCRIPTION	Elev.
0.3			2330.1
2.1	GC	silty clayey gravel with sand, very dense	2328.3
3.1	GM	silty gravel with sand, very dense	2327.3
4.2	CL	sandy lean clay with gravel, stiff	2326.2
5.2	CL-ML	silty clay, stiff	2325.2
6.1	ML	silt, medium dense	2324.3
7.2	SM	silty sand with gravel, medium dense	2323.2
8.1	GM	silty gravel with sand, very dense	2322.3
9.2	GW	well graded gravel with sand, very dense	2321.2
10.1	GP-GM	poorly graded gravel with silt, very dense	2320.3
11.0		bedrock, mica schist	2319.4

SAMPLE TYPE NUMBER	BLOW COUNTS/15 cm (N VALUE)
SS	25-42-50/15cm
SS	31-35-50/15cm
SS	31-42-45/15cm
SS	2-4-7/15cm
SS	4-7-7/15cm
SS	5-5-5/15cm
SS	3-4-5/15cm
SS	15-17-25/15cm
SS	25-34-50/15cm
SS	24-35-37/15cm

Bottom of borehole at 11.0 meters.

TETRA TECH STANDARD LOG BORING AND TP - TETRA TECH 04-29-10.GDT - 11/17/11 16:16 - P:\01304\133-01304-10001\SHIKARI\SUPPORTDOCS\EXISTSITEDATA\JULY 2011 DRILLING\SHIKARI VALLEY DAM.GPJ

0 .75m 1.5m 2.25m 3m

UNLESS OTHERWISE NOTED, ALL DIMENSIONS SHOWN ARE IN METERS.

CONCEPT DESIGN

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SYMB	DESCRIPTION	DATE	APR
	TASK 3 SUBMITTAL		
	WO-LT-0009-002 SUBMITTAL		

DESIGNED BY: AK | DATE: 11/18/11
DWN BY: MAM | SUBMITTED BY: TETRA TECH
CHK BY: RJT | FILE NO.: G-2 BORE HOLE LOGS

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PRT SUPPORT PROGRAM
SHIKARI VALLEY DAM
BORE HOLE LOGS
BH-01 AND BH-02

SHEET REFERENCE NUMBER: C-08

BOREHOLE BH-03

PAGE 1 OF 1

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CLIENT USAID
PROJECT NAME Shikari Valley Dam
PROJECT NUMBER 133-01304-10001
PROJECT LOCATION Shikari Valley, Bamyan Province, Afghanistan
DATE/TIME STARTED 7/2/2011 COMPLETED 7/12/2011
GROUND ELEVATION 2329.409 m HOLE SIZE 4.5
DRILLING CONTRACTOR Geo search
GROUND WATER LEVELS:
DRILLING METHOD 11.4 cm Rotary Core & 5.3 cm wireline Core
AT TIME OF DRILLING —
LOGGED BY H. Jawid CHECKED BY R. Tocher
AT END OF DRILLING — bore hole in river
NOTES
— —
COORDINATES 0N, 3 E

DEPTH (m)	SAMPLE TYPE NUMBER	BLOW COUNTS/15 cm (N VALUE)	U.S.C.S.	WATER LEVEL	GRAPHIC LOG	MATERIAL DESCRIPTION	
0			SM			silty sand with gravel, medium dense	
1.1	SS	5-7-8/15cm	GP-GM			poorly graded gravel with sand, very dense	2328.3
1.9	SS	28-31-45/15cm	GP-GM			poorly graded gravel with silt and sand, very dense	2327.5
3.0	SS	35-45-48/15cm	GM			silty gravel with sand, very dense	2326.4
4.0	SS	31-36-48/15cm	GC			silty, clayey, gravel with sand, very dense	2325.4
	SS	36-42-45/15cm					
5.2			ML			sandy silt, loose	2324.2
6.1	SS	2-3-5/15cm	SM			silty sand with gravel, medium dense	2323.3
7.0	SS	2-4-5/15cm	SP-SM			poorly graded sand with silt and gravel, loose	2322.4
	SS	3-4-4/15cm					
8.3						bedrock, mica schist	2321.1
9.0							2320.4

Bottom of borehole at 9.0 meters.

TETRA TECH STANDARD LOG BORING AND TP - TETRA TECH 04-29-10.GDT - 11/17/11 16:16 - P:\01304\133-01304-10001\SHIKARI\SUPPORTDOCS\EXISTINGDATA\JULY 2011 DRILLINGS\SHIKARI VALLEY DAM.GPJ

0 .75m 1.5m 2.25m 3m

LEGEND

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

- TOPSOIL
- CL: USCS Low Plasticity Clay
- CL-ML: USCS Low Plasticity Silty Clay
- ML: USCS Silt
- SM: USCS Silty Sand
- SP-SC: USCS Poorly-graded Sand with Clay
- SP-SM: USCS Poorly-graded Sand with Silt
- GC: USCS Clayey Gravel
- GM: USCS Silty Gravel
- GP-GM: USCS Poorly-graded Gravel with Silt
- GW: USCS Well-graded Gravel
- BEDROCK: MICA SCHIST Bedrock

SAMPLER SYMBOLS

- Split Spoon

ABBREVIATIONS

- LL - LIQUID LIMIT (%)
- PI - PLASTIC INDEX (%)
- W - MOISTURE CONTENT (%)
- DD - DRY DENSITY (PCF)
- NP - NON PLASTIC
- -200 - PERCENT PASSING NO. 200 SIEVE
- Water Level at Time Drilling, or as Shown
- Water Level at End of Drilling, or as Shown
- Water Level After 24 Hours, or as Shown

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CONCEPT DESIGN

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SYMB	DESCRIPTION	DATE	APR
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	WO-LT-0009-002 SUBMITTAL		

DESIGNED BY: AK | DATE: 11/18/11
DWN BY: MAM | SUBMITTED BY: TETRA TECH
CHK BY: RJT | FILE NO.: G-2 BORE HOLE LOGS

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PRT SUPPORT PROGRAM
SHIKARI VALLEY DAM
BORE HOLE LOG
BH-03

SHEET REFERENCE NUMBER:
C-09

Appendix F Flow Measurement

Shikari #1

DATA FORM FOR CALCULATING FLOW

Solving the equation: $\text{Flow} = \dfrac{A\,L\,C}{T}$

Where:

A = Average cross-sectional area of the stream. L = Length of the stream reach measured (usually 20 ft.).
C = A coefficient or correction factor (0.8 for rocky-bottom streams or 0.9 for muddy-bottom streams). T = Time, in seconds, for the float to travel the length of L.

A: Average Cross-Sectional Area

Transect #1 (upstream)

	Interval width (feet)	Depth (feet)	
A to B	= 10	0.98	(at B)
B to C	= 10	1.02	(at C)
C to D	= 10	1.35	(at D)
D to E	= 10	0	(shoreline)
Totals	40	3.35	÷ 4

= Avg. depth 0.84 ft

Cross-sectional area of Transect #1

= Total width (ft) X Avg. depth (ft)

40 X 0.84 = 33.5 ft²

Transect #2 (downstream)

	Interval width (feet)	Depth (feet)	
A to B	= 10	1.41	(at B)
B to C	= 10	1.21	(at C)
C to D	= 10	0.89	(at D)
D to E	= 10	0	(shoreline)
Totals	40	3.51	÷ 4

= Avg. depth 0.88 ft

Cross-sectional area of Transect #2

= Total width (ft) X Avg. depth (ft)

40 X 0.88 = 35.1 ft²

(Cross-sectional area of Transect #1 + Cross-sectional area of Transect #2) ÷ 2 = Average Cross-sectional area

$A = (\boxed{33.5}\ ft^2 + \boxed{35.1}\ ft^2) \div 2 = \boxed{34.3}\ ft^2$

L: Length of Stream Reach

40 ft

C: Coefficient

0.8

T: Travel Time

Travel Time of Float (sec.)

Trial #1	9	#4	9	#7	8
Trial #2	12	#5	10	#8	13
Trial #3	8	#6	9	#9	8

Total 86 ÷ ~~3~~ 9

= Avg. time 9.6 sec.

Flow = $\dfrac{A\,L\,C}{T} = \dfrac{\boxed{34.3}\ \boxed{40}\ \boxed{0.8}}{\boxed{9.6}} = \boxed{114}$ **ft³/sec.**

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