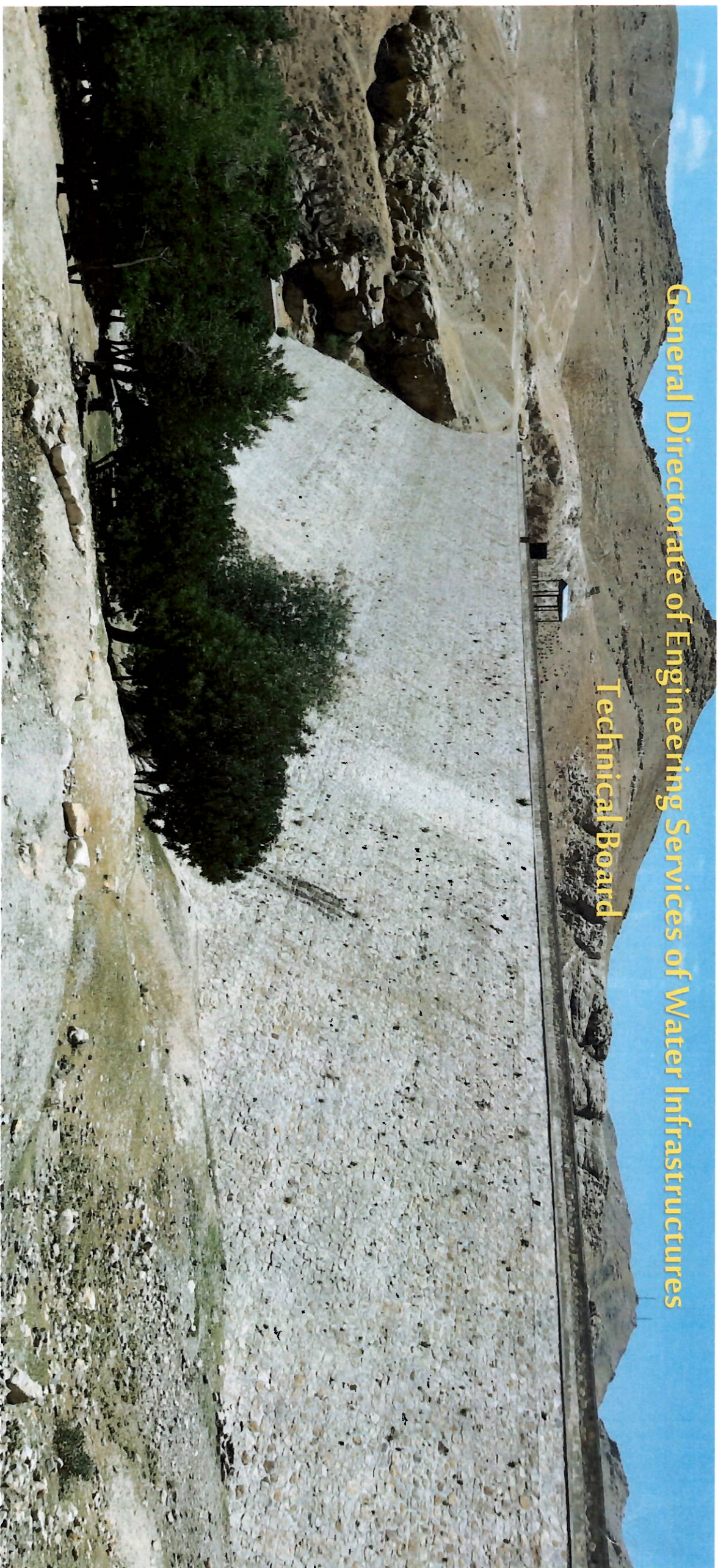
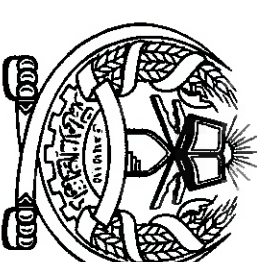




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Deputy Ministry of Water



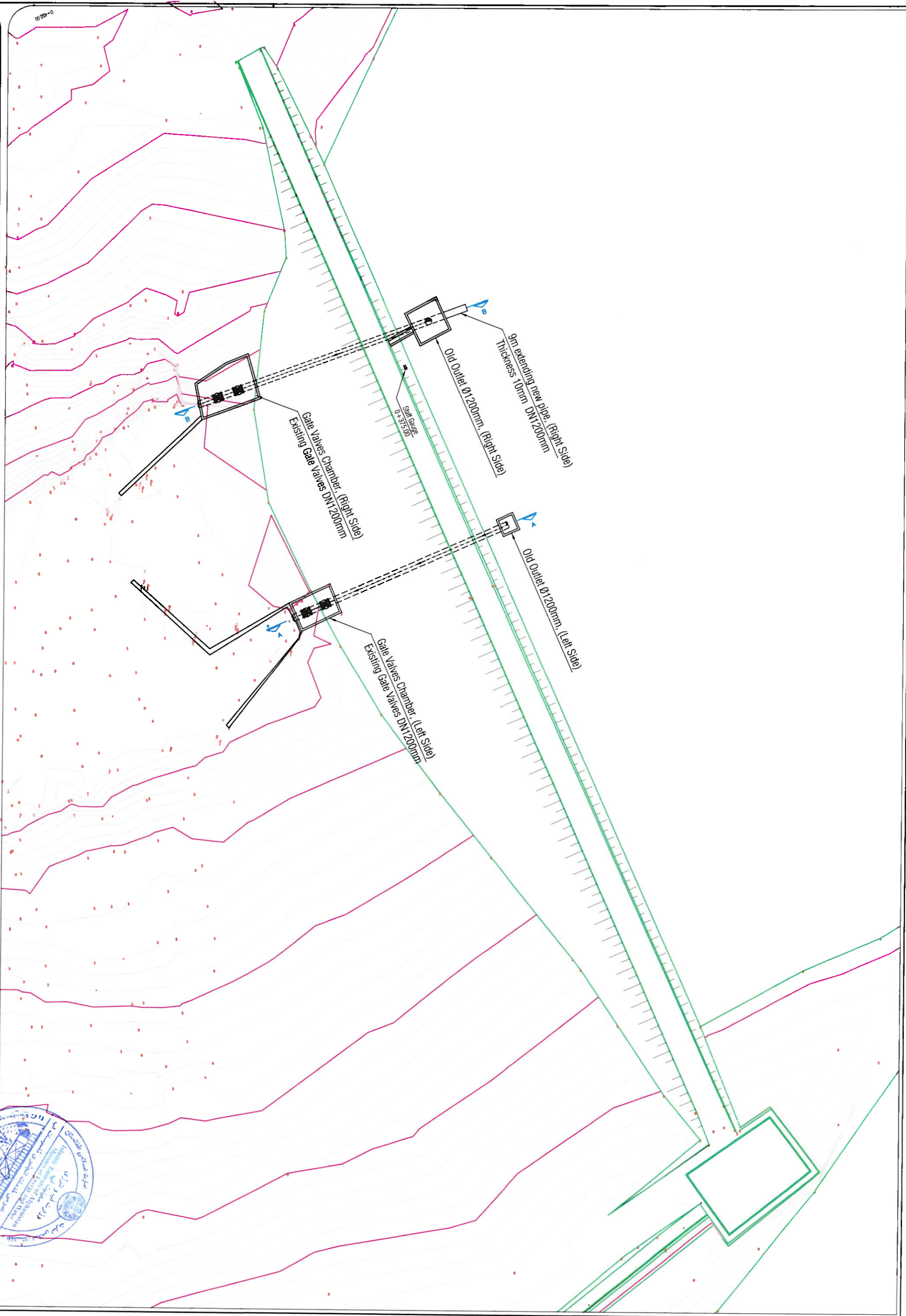
General Directorate of Engineering Services of Water Infrastructures

Technical Board

Rehabilitation of Sultan Dam

Date		October-2023	
Prepared	Checked	Approved	Hydromechanical and instrumentation equipment drawing
Murtaza Barialy	Faridullah Haider	General Directorate of Engineering Services	

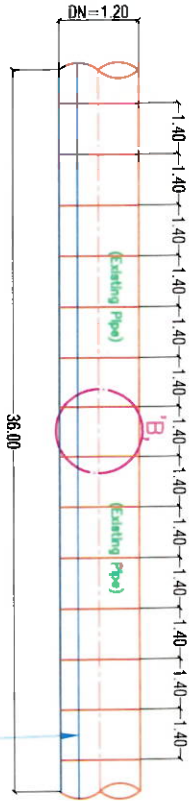
PROJECT NAME	DRAWING	PROVINCE	DISTRICT	DESIGN DATE	SCALE	DESIGNED BY:
SULTAN DAM (REHABILITATION)	SULTAN DAM EXISTING OUTLET PLAN	GHAZNI	KIRKAWA OMARI	OCT-2023	AS SHOWN	ENG MORTAZA BARIALI
						CHECKED BY:
						ENG FARIDULLAH HADARY
						APPROVED BY:
						ENG ABDUL GHAFOR OMARI



In the left gate operating room, the standing bench floor wooden plank using for standing and operating the mechanism and handle is need for repairing as shown in the drawing below.

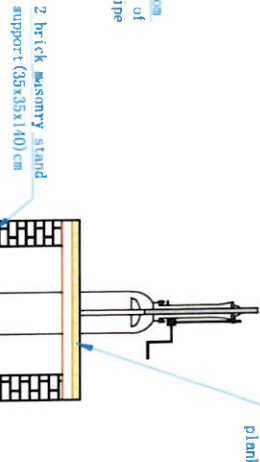
The left side outlet pipe repaired before with bottom support steel (35x0.95x0.02)m. The joints of this support need for some repairing and welding.

Existing bottom support plate of left outlet pipe



14 pieces wooden plank (200x18x5) cm

Existing wall

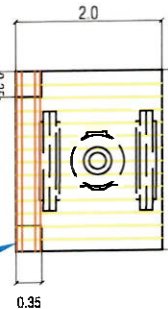


2 brick masonry stand support (35x35x140) cm

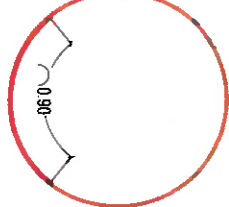
One pair of beam No: 14 L=2x2.5 5m

2384.5 New Protection Dam Parapet wall Level
2381 Existing Dam Crest Level

Existing Stonemasonry Dam Body

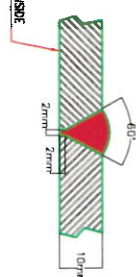


2 brick masonry stand support (35x35x140) cm

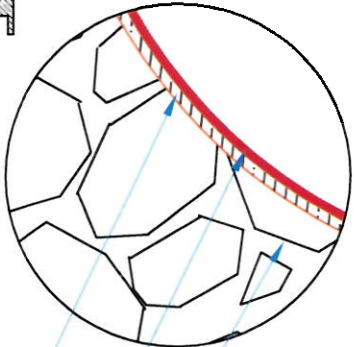


Existing outlet pipe and existing bottom support plate.

Welding Type 3 Detail B



Existing Building



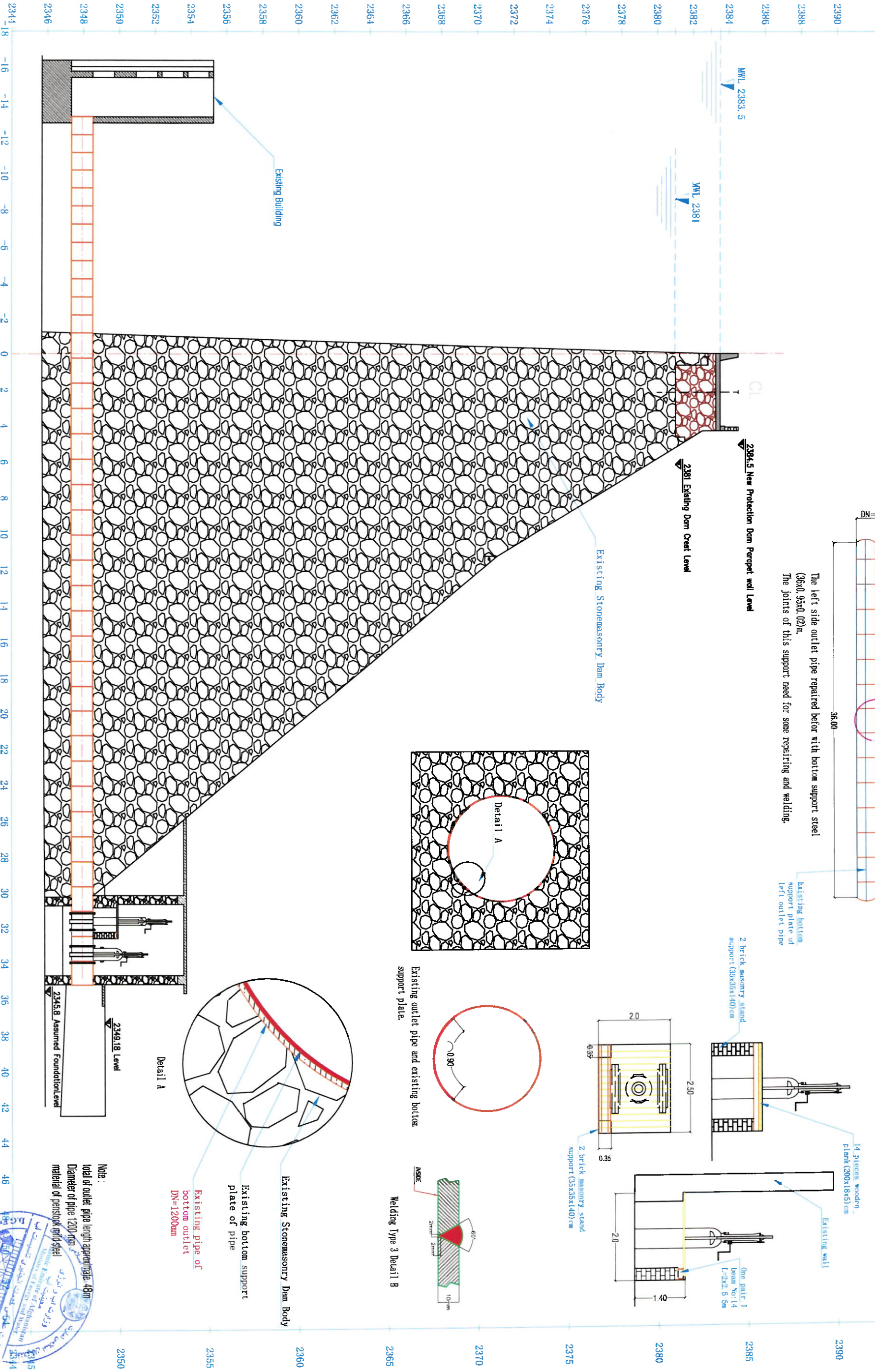
Existing bottom support plate of pipe

Existing pipe of bottom outlet DN=1200mm

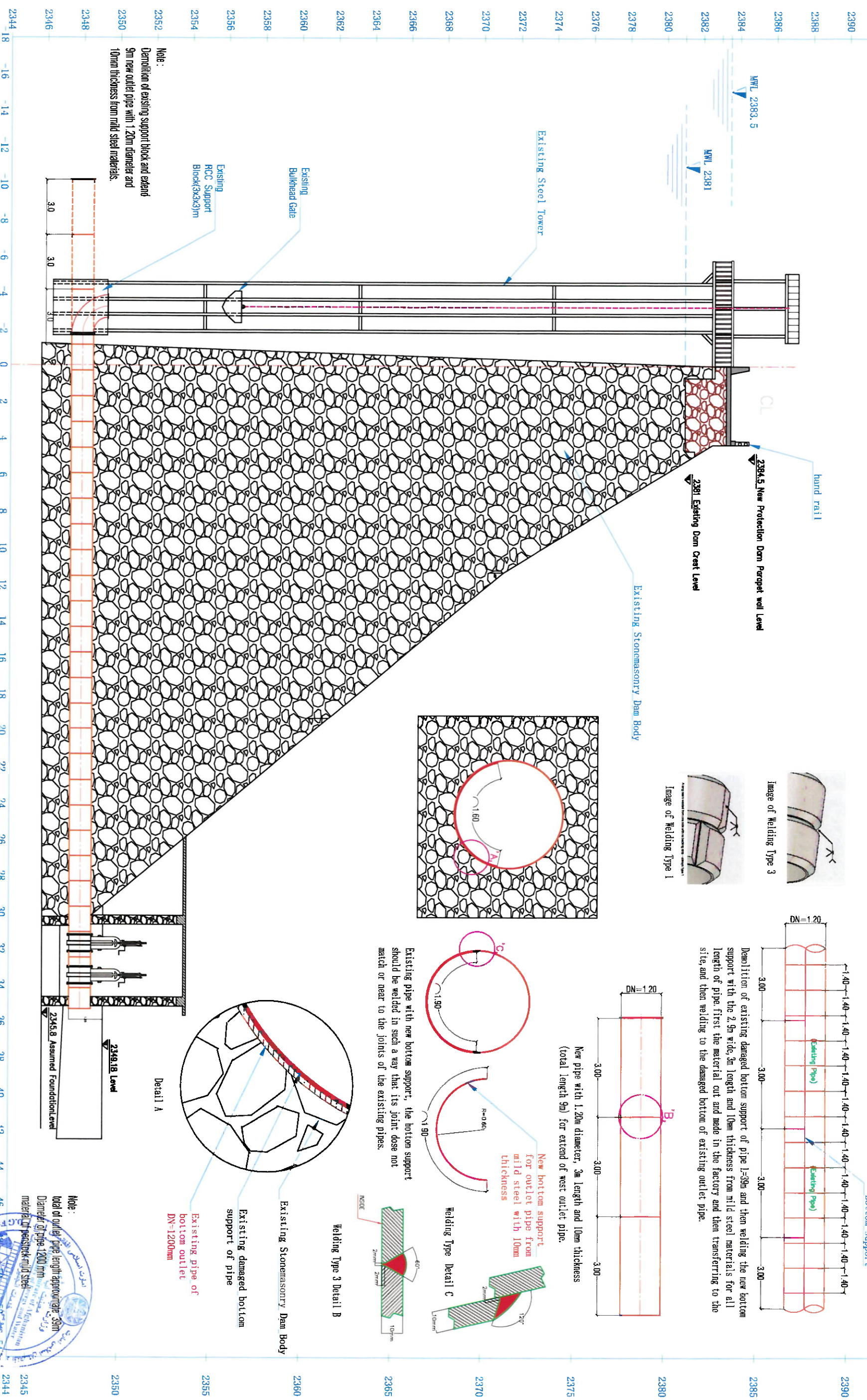
Detail A


Note:


total of outlet pipe length approximate 48m
Diameter of pipe 1200 mm
material of penstock mild steel



0+385.00

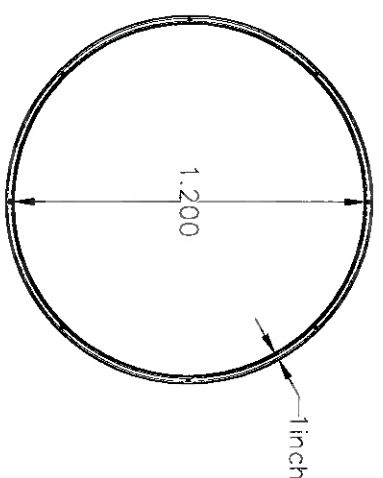


PROJECT NAME	DRAWING	PROVINCE	DISTRICT	DESIGN DATE	SCALE	DESIGNED BY:	ENG MARTAZA BARALY
						CHECKED BY:	ENG FARIDULLAH HADARY
SULTAN DAM (REHABILITATION)	RIGHT SIDE OUTLET DRAWING FOR REHABILITATION	GHAZNI	KIRWALA OMARI	OCT - 2023	AS SHOWN	APPROVED BY:	ENG ABDUL GHAFQOR OMARI
							

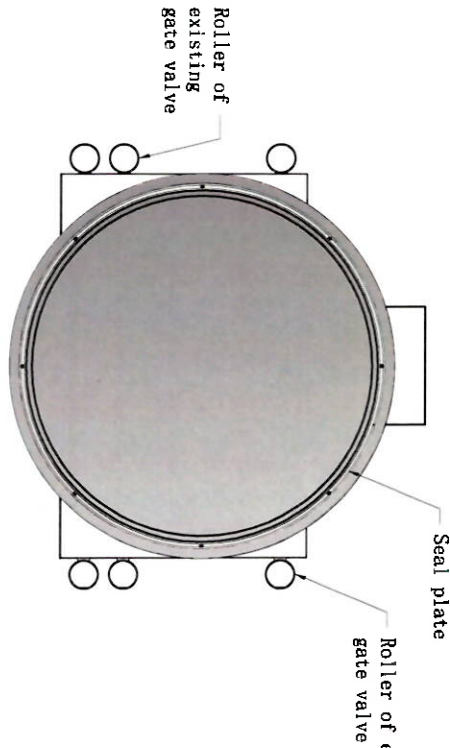


SHEET NO
 3 of 14

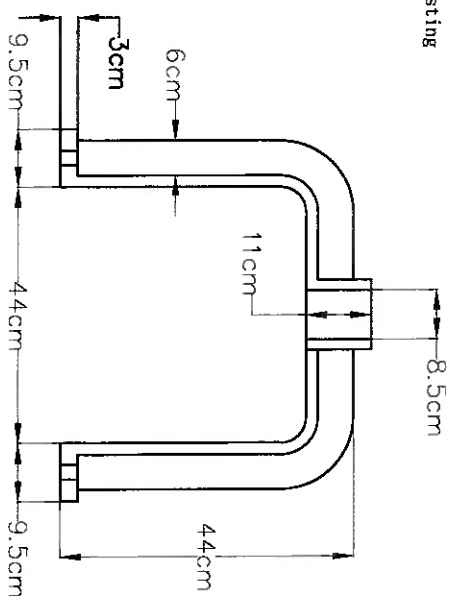
8 pieces seal plate for existing gate valve from bronze or copper materials,internal diameter 120cm wide 1inch and thickness 12mm, with 8 holes at 8mm diameter,with all necessary accessories according to sample.



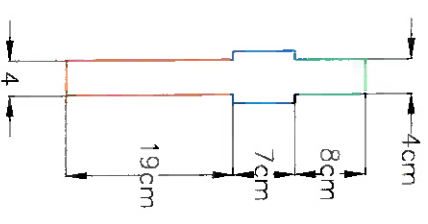
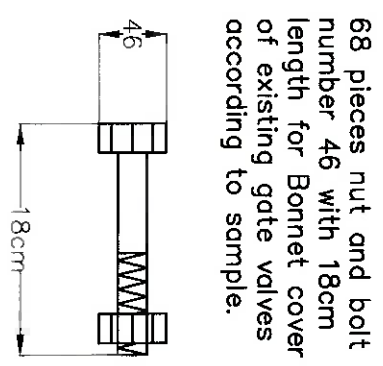
Existing gate valve disc



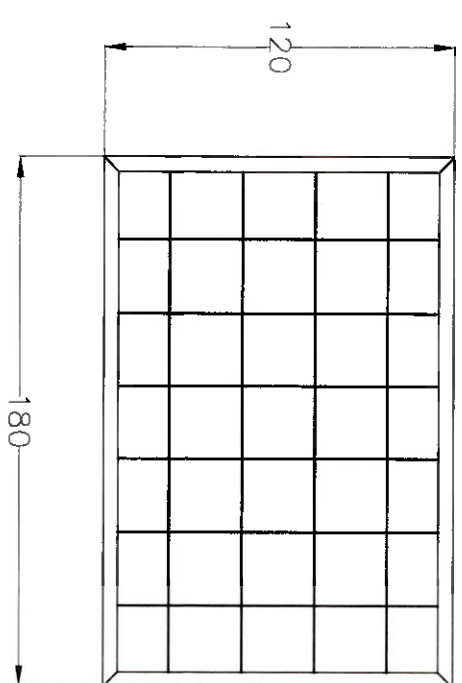
Actuator yoke (OS&Y) (qabza shaft) one piece according to sample with all necessary accessories.



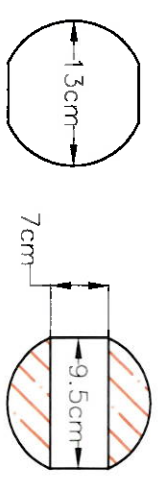
one piece of actuator stem (pine) according to sample with all necessary accessories.



In the windows of the existing gate chambers:the frames of the window are Lauren No. 5 (b = 50mm, t = 4mm, R = 5.5mm, r = 1.8mm), and the net of the windows are 16mm plain reinforcement bars @25cm c/c with all necessary accessories.



Spherical rollers of existing bottom outlet gate valves,12 pieces according to sample.

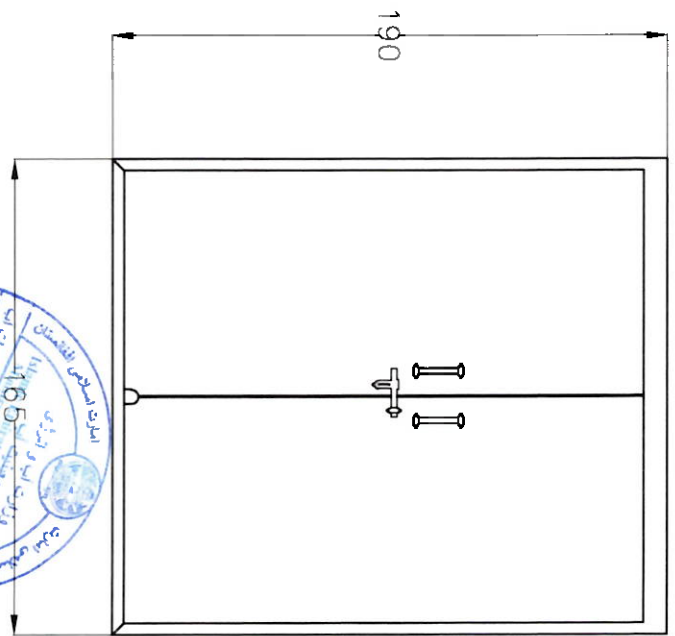
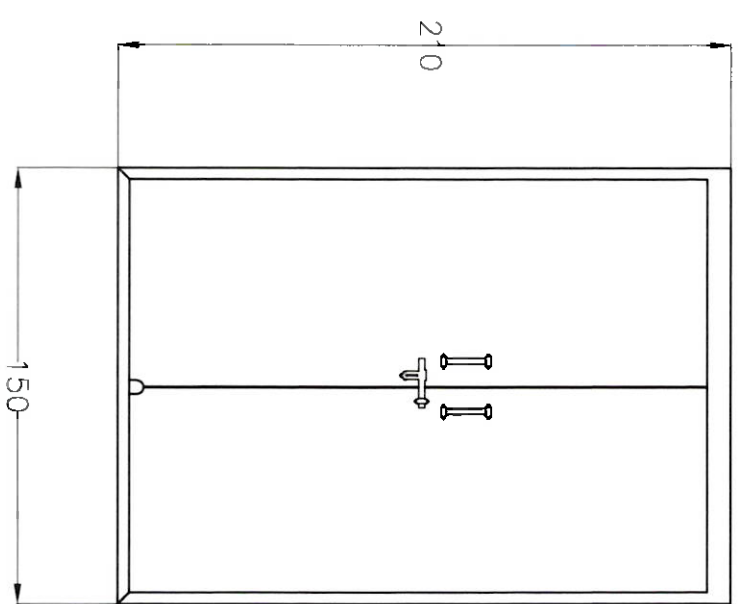
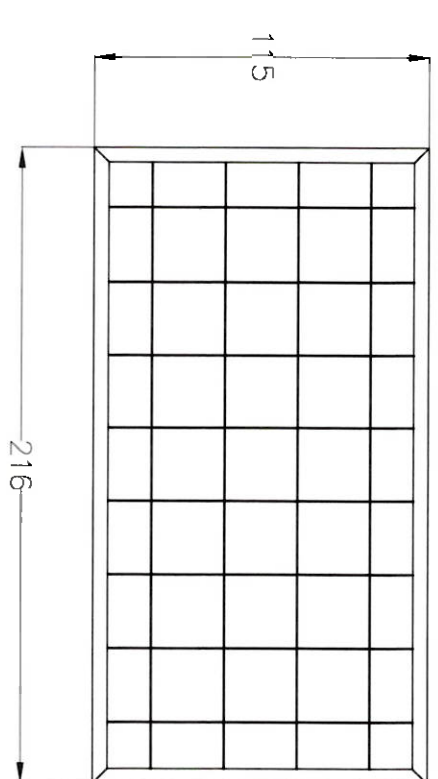


View

Section

Plan

In the doors of the existing gate chambers: the right, left, and bottom frames of the doors are girder U No. 8 (h = 80mm, b = 40mm, s = 4.5mm, R = 6.5mm, r = 2.5mm), and the top frame of the doors are Lauren No. 8 (b = 80mm, t = 5.5mm, R = 9mm, r = 3mm) and the steps of the doors are steel plate 5mm, with all necessary accessories.



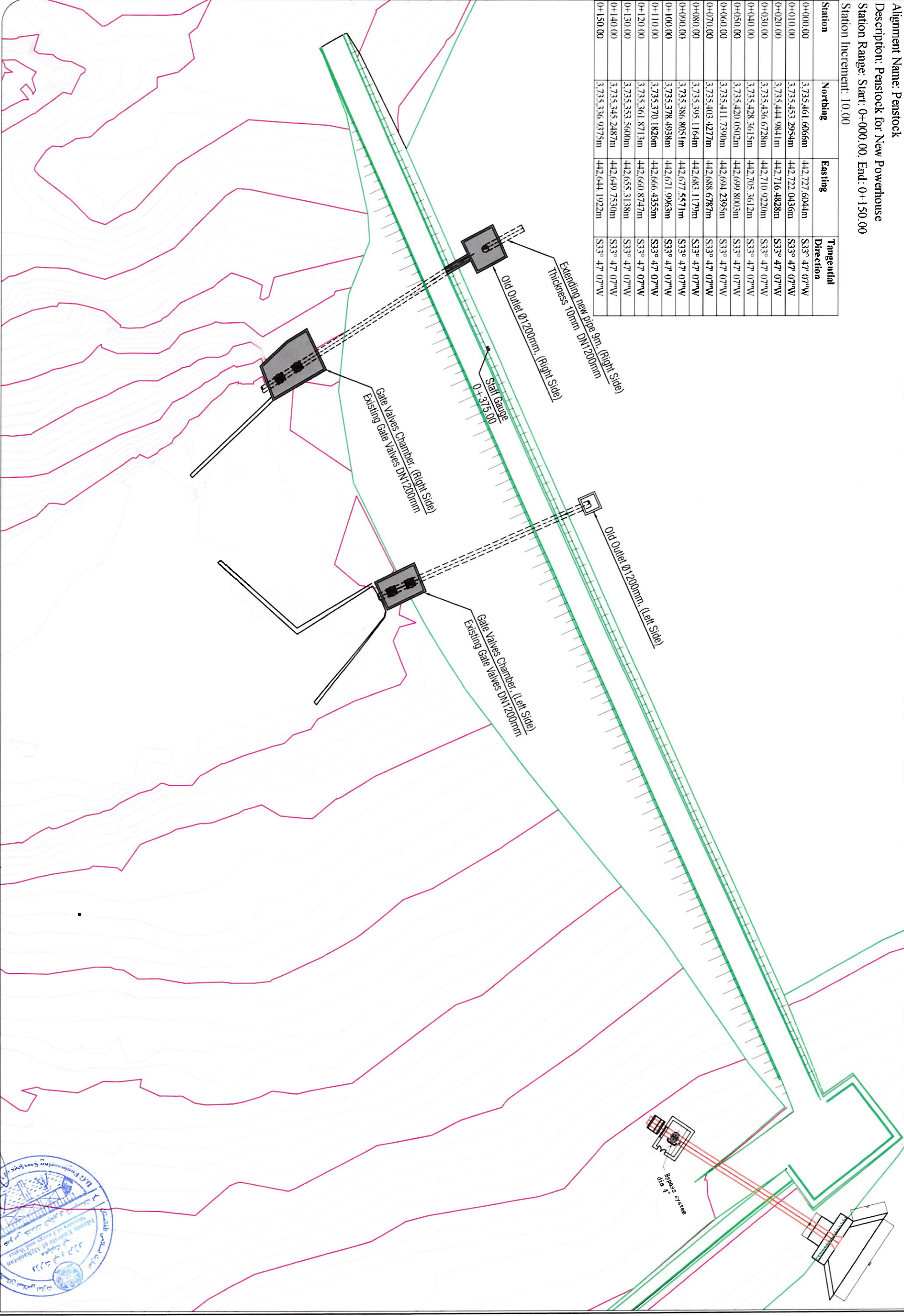
Alignment Name: Penstock

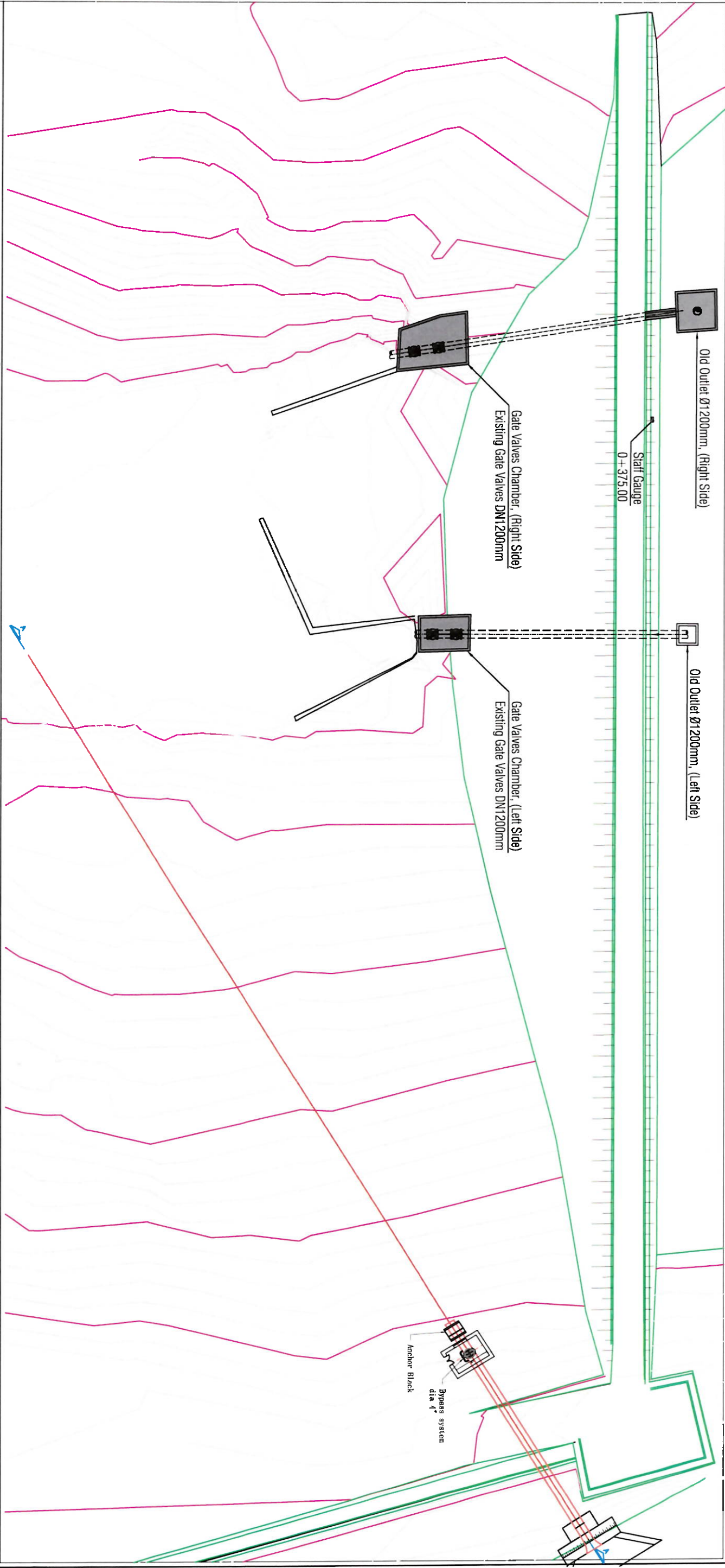
Description: Penstock for New Powerhouse

Station Range: Start: 0+000.00, End: 0+150.00

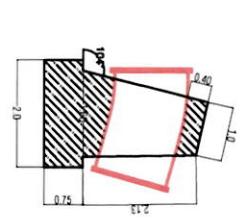
Station Increment: 10.00

Station	Northing	Easting	Tangential Direction
0+000.00	3,735,461.6066m	442,727.6044m	S33° 47' 07"W
0+010.00	3,735,453.2954m	442,722.0436m	S33° 47' 07"W
0+020.00	3,735,444.9841m	442,716.4828m	S33° 47' 07"W
0+030.00	3,735,436.6728m	442,710.9220m	S33° 47' 07"W
0+040.00	3,735,428.3615m	442,705.3612m	S33° 47' 07"W
0+050.00	3,735,420.0502m	442,699.8003m	S33° 47' 07"W
0+060.00	3,735,411.7390m	442,694.2395m	S33° 47' 07"W
0+070.00	3,735,403.4277m	442,688.6787m	S33° 47' 07"W
0+080.00	3,735,395.1164m	442,683.1179m	S33° 47' 07"W
0+090.00	3,735,386.8051m	442,677.5571m	S33° 47' 07"W
0+100.00	3,735,378.4938m	442,671.9963m	S33° 47' 07"W
0+110.00	3,735,370.1826m	442,666.4355m	S33° 47' 07"W
0+120.00	3,735,361.8713m	442,660.8747m	S33° 47' 07"W
0+130.00	3,735,353.5600m	442,655.3138m	S33° 47' 07"W
0+140.00	3,735,345.2487m	442,649.7530m	S33° 47' 07"W
0+150.00	3,735,336.9375m	442,644.1922m	S33° 47' 07"W

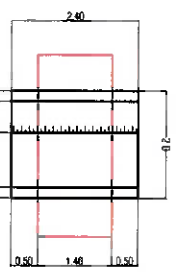




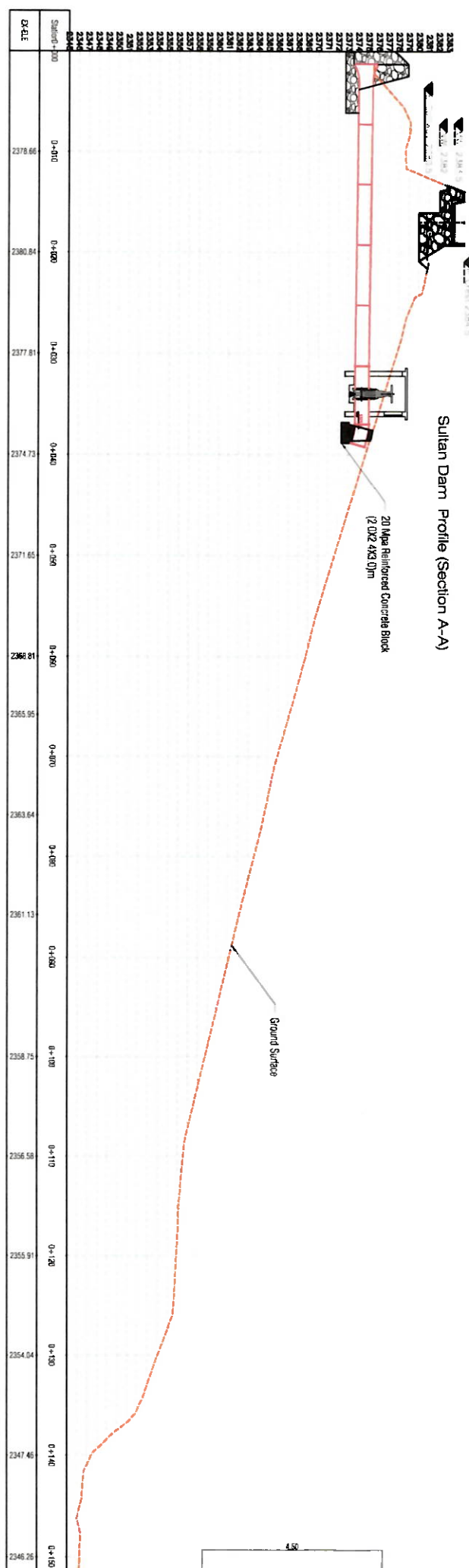
Sultan Dam Profile (Section A-A)



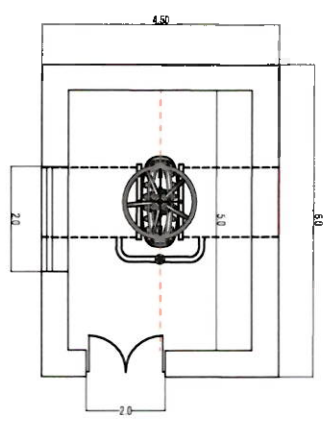
Anchor Block Section



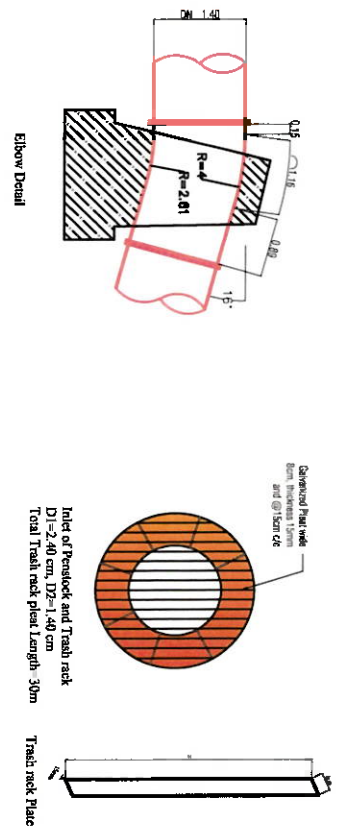
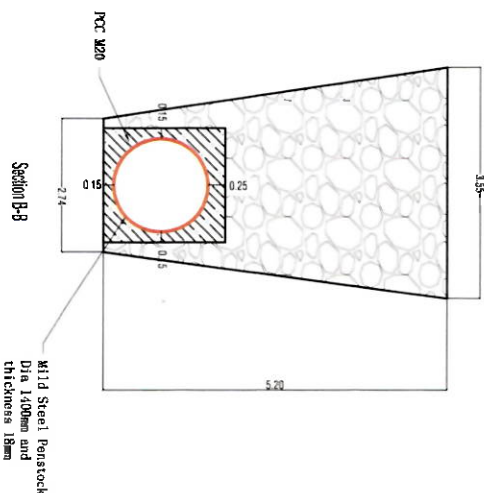
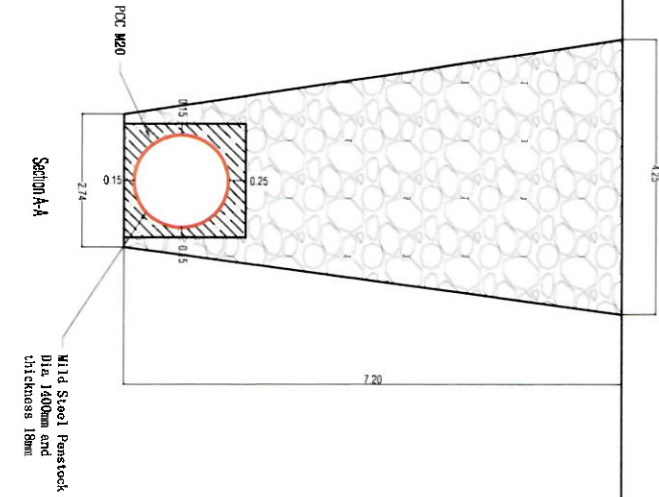
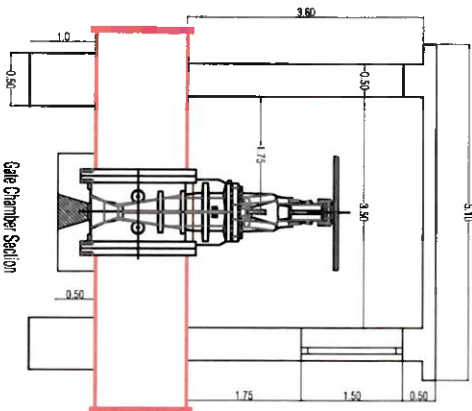
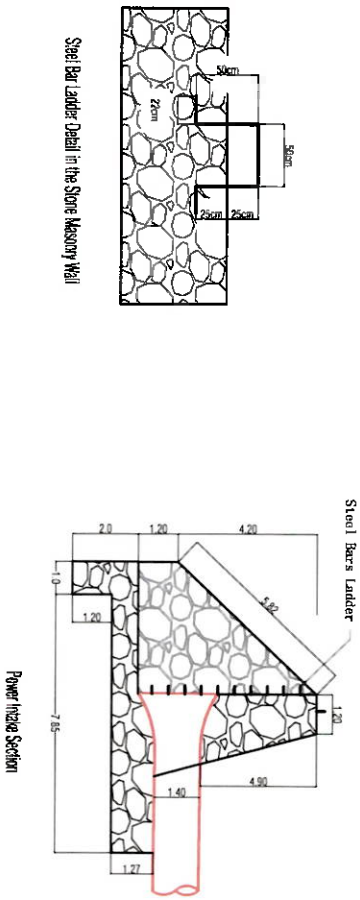
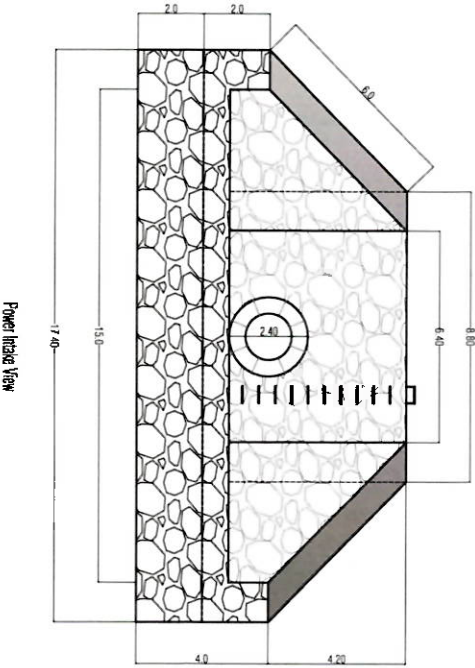
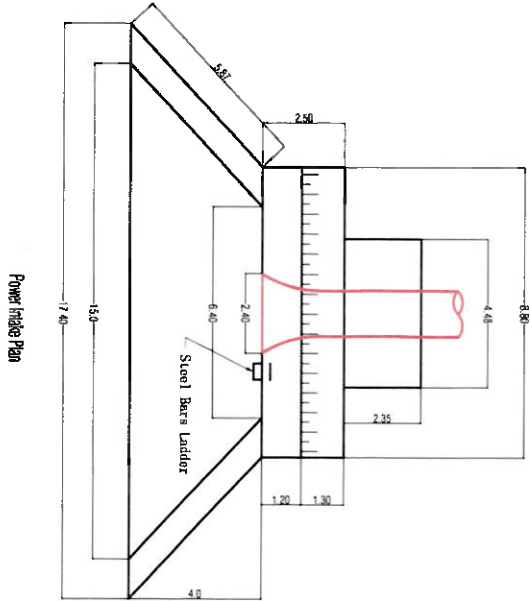
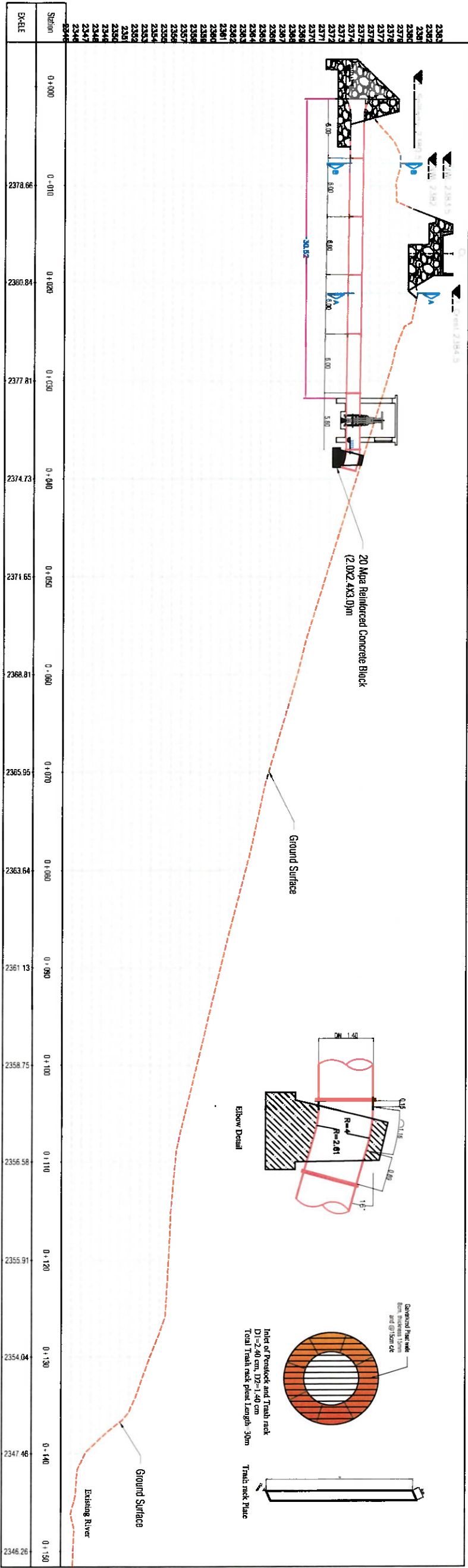
Anchor Block Plan



Gate Chamber Plan



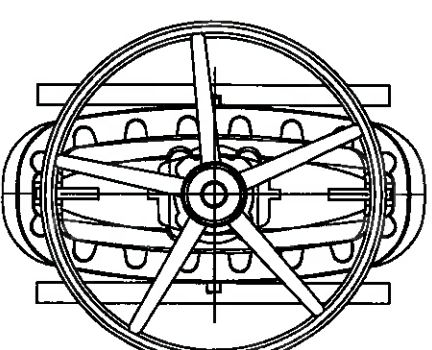
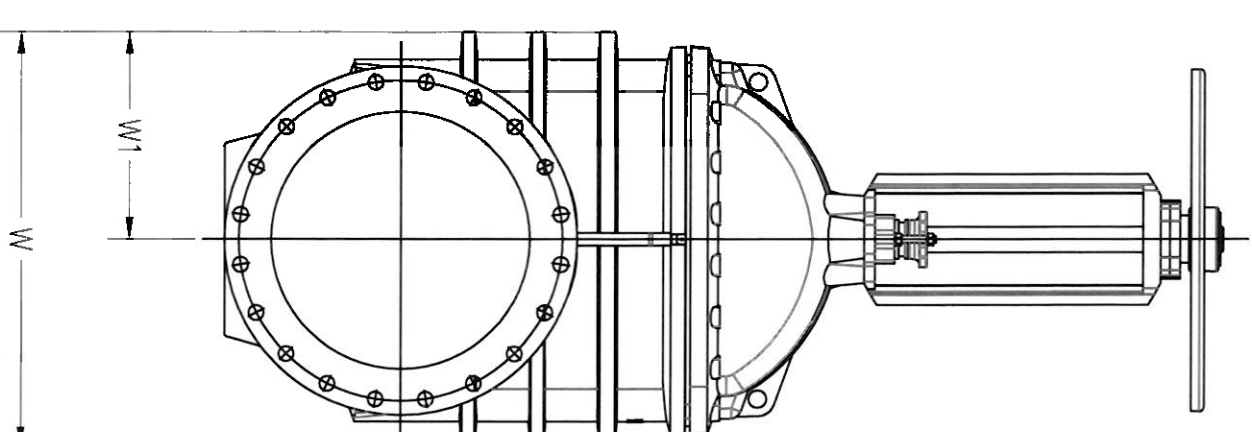
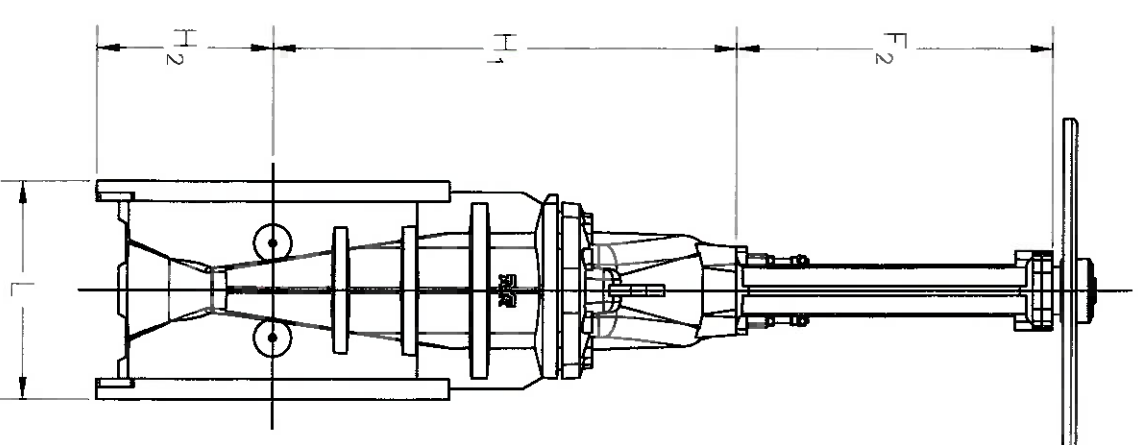
Sultan Dam Profile



Gate valve Drawing and Specification

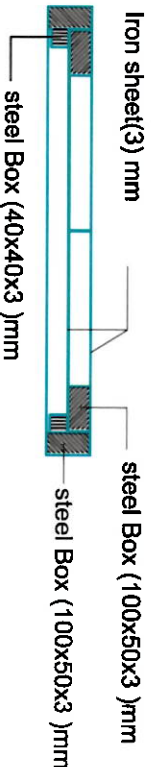
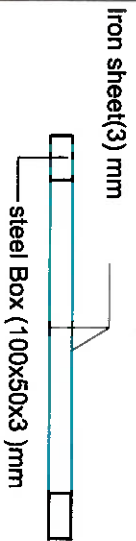
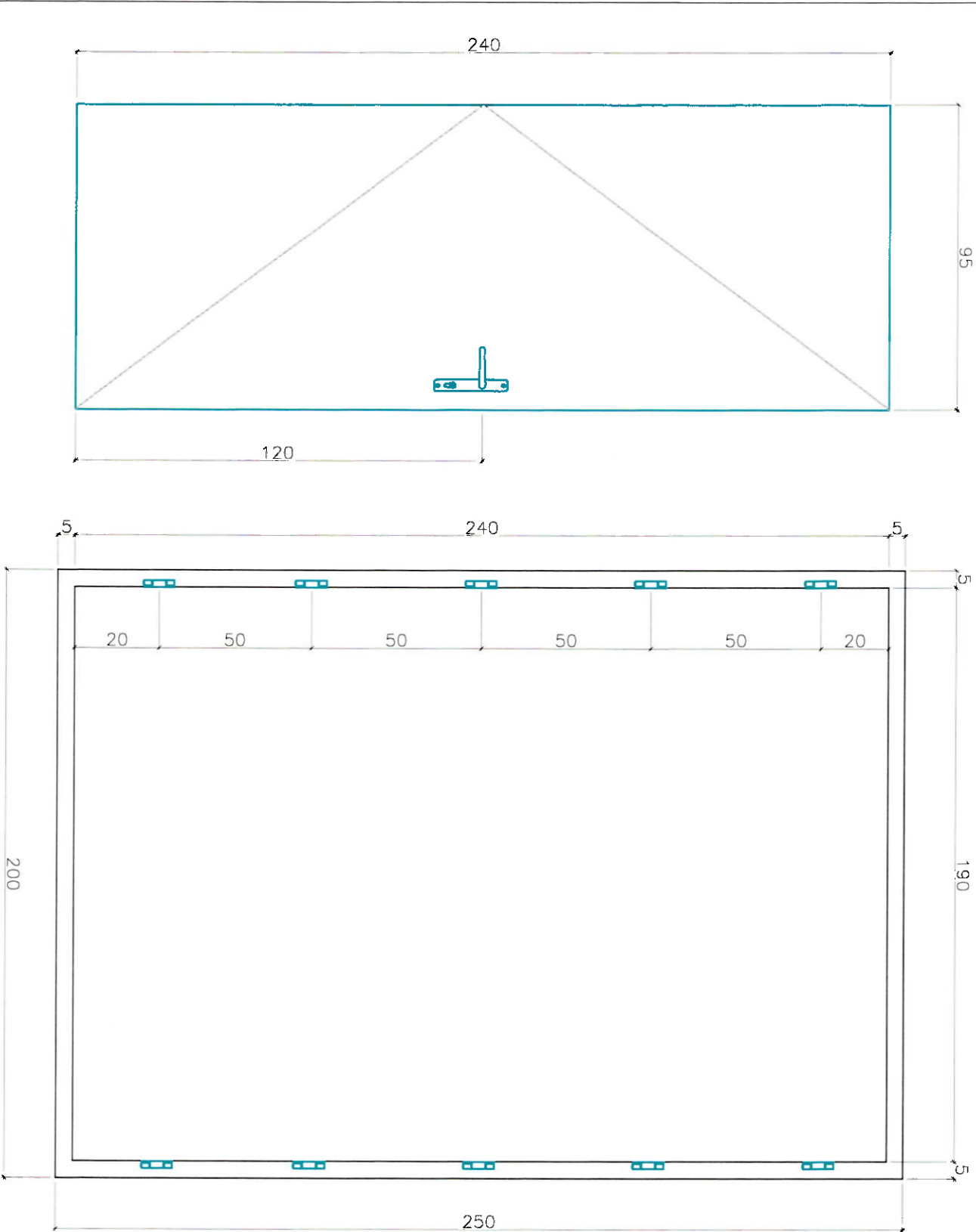
Gate valve Technical Specification

No	Part Name	Material
1-	Body	- Ductile iron
2-	Seat ring	- Aluminum-bronze
3-	Face ring	- Aluminum-bronze
4-	Wedge	- Ductile iron
5-	Wedge nut	- Aluminum-bronze
6-	Stem	- Stainless steel 1.4057(431)
7-	O-cord	- EPDM rubber
8-	Bonnet	- Ductile iron
9-	Air plug	- Stainless steel
10-	Key	- Steel
11-	Bolts	- Steel,hot dip galvanized
12-	Blanking	-Ductile iron
13-	O-ring	- EPDM rubber
14-	Stem cap	-Ductile iron
15-	Seal	-Hot melt glue
16-	Gland flange	-Ductile iron
17-	Thrust collar	-Aluminum-bronze CC331G (AB1)
18-	Gland flange bolt	-Steel,hot dip galvanized
19-	Seal (2)	-Hot melt glue
20-	Bonnet bolt	-Steel,hot dip galvanized
21-	Thrust nut	-Aluminum-bronze CW307G
22-	Distance piece	-Ductile iron
23-	Gland	-Ductile iron
24-	Packing	-PTFE
25-	Stuffing box	-Ductile iron
26-	Ends	-Flanged end to ASME B16.5 RF(300#)
27-	DN	-(1400) mm
28-	PN	-25 bar



Gate Valve Plan

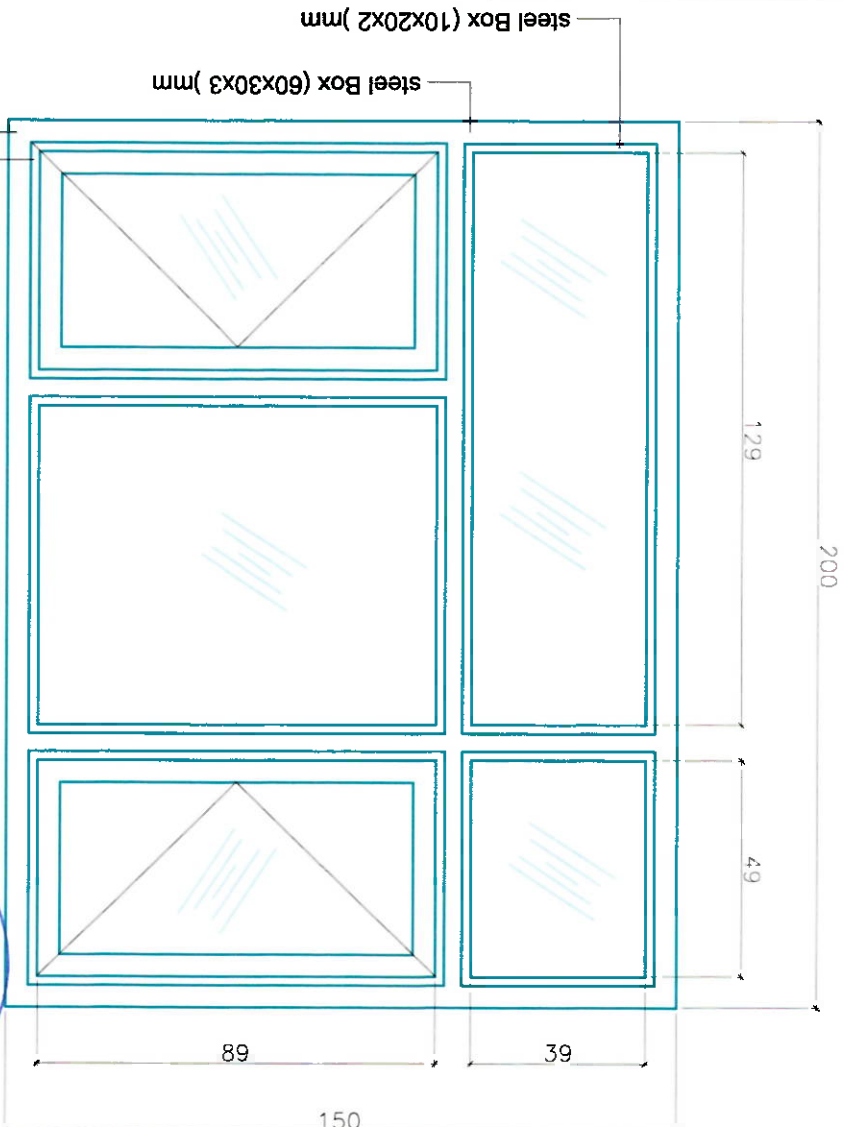
Gate Valve



STEEL DOOR & FRAME

SCALE: 1:50

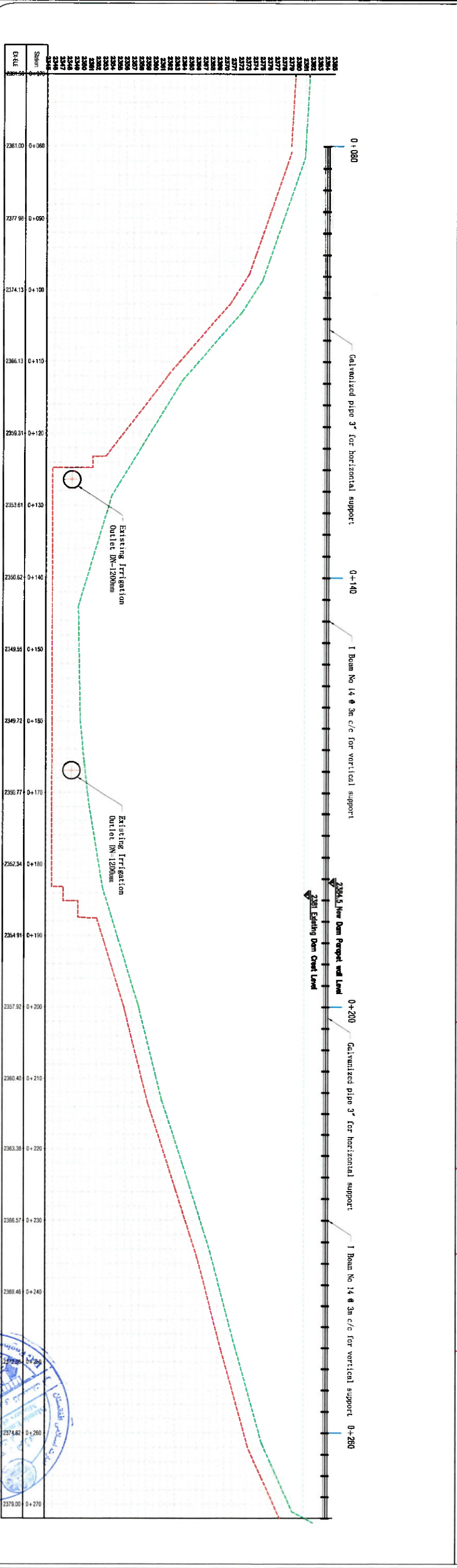
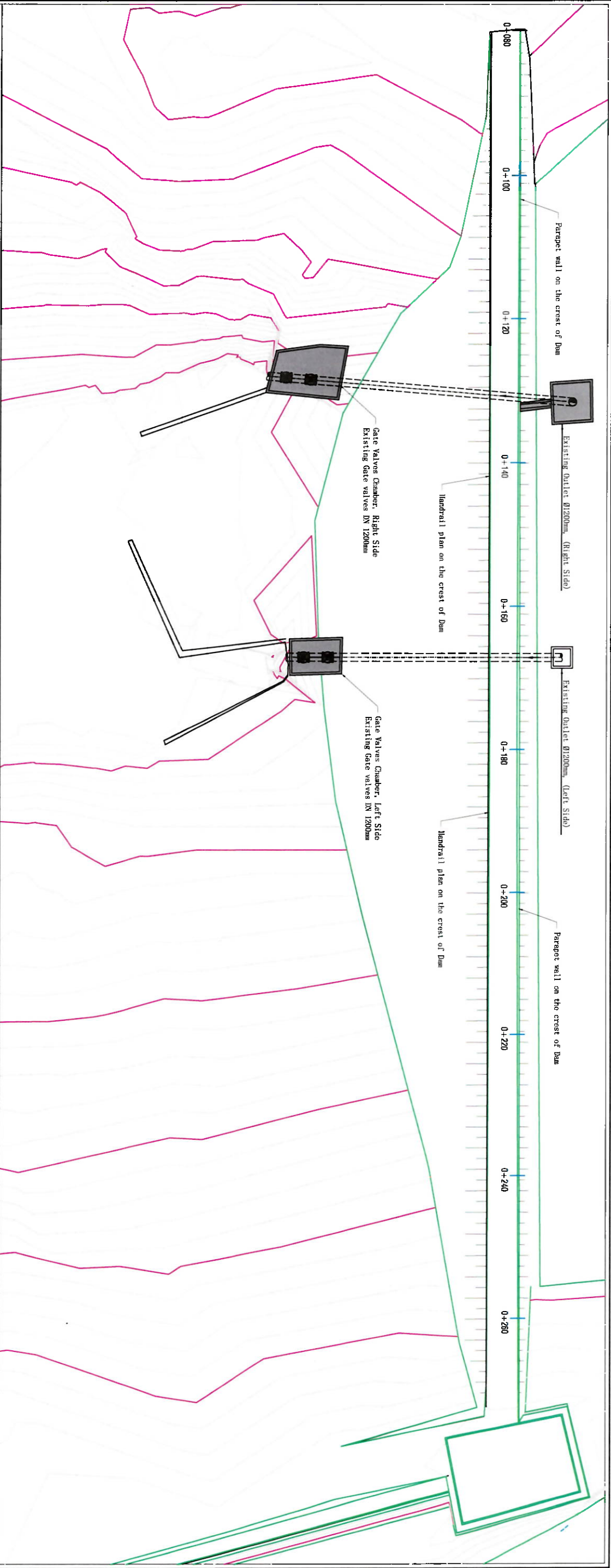
- Surface Preparation:
 - Thoroughly clean the steel surface to remove dirt, grease, rust, and other contaminants.
 - Use appropriate cleaning methods such as solvent cleaning, power washing, or sandblasting to achieve a clean and smooth surface.
 - Repair any surface defects or damage before painting.
- Primer Application:
 - Apply a suitable primer specifically designed for steel surfaces.
 - Ensure that the primer is compatible with the topcoat paint and provides good adhesion and corrosion resistance.
 - Follow the manufacturer's instructions for primer application, including the recommended number of coats and drying times.
- Topcoat Selection:
 - Choose a high-quality paint specifically formulated for exterior or interior steel applications.
 - Consider factors such as durability, weather resistance, color retention, and ease of maintenance.
 - Opt for paints with good adhesion to steel surfaces and resistance to UV radiation, moisture, and chemicals.
- Painting Techniques:
 - Apply the paint evenly and smoothly using a brush, roller, or spray equipment.
 - Follow the recommended paint application thickness and coverage rates.
 - Ensure proper ventilation during painting to allow for proper drying and minimize the risk of fumes or overspray.
- Drying and Curing:
 - Allow sufficient drying and curing time between coats and after the final application.
 - Follow the manufacturer's instructions regarding drying times and environmental conditions (temperature, humidity) for optimal curing.




steel Box (10x20x2) mm
steel Box (60x30x3) mm

STEEL Window & FRAME

SCALE: 1:50



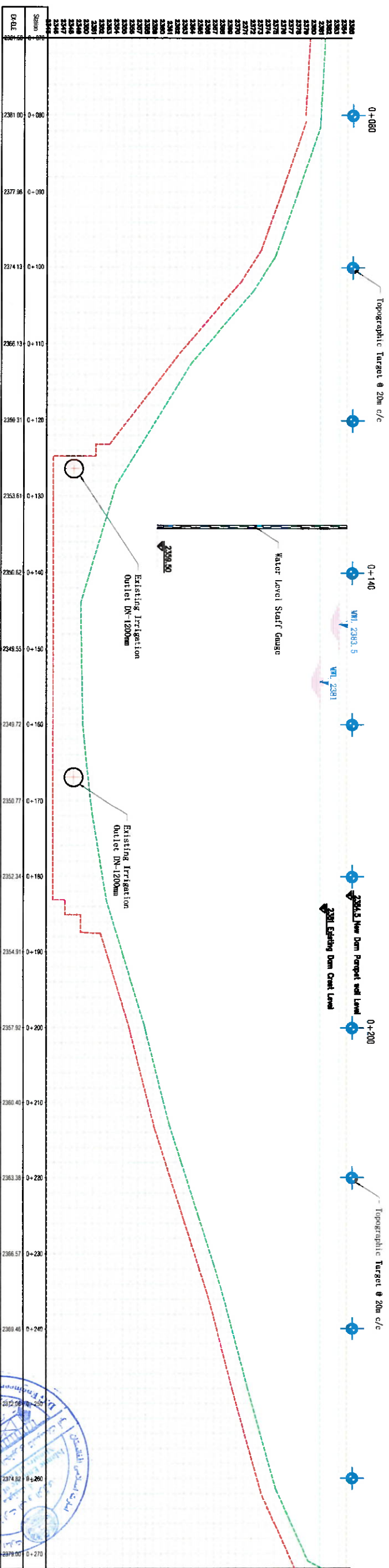
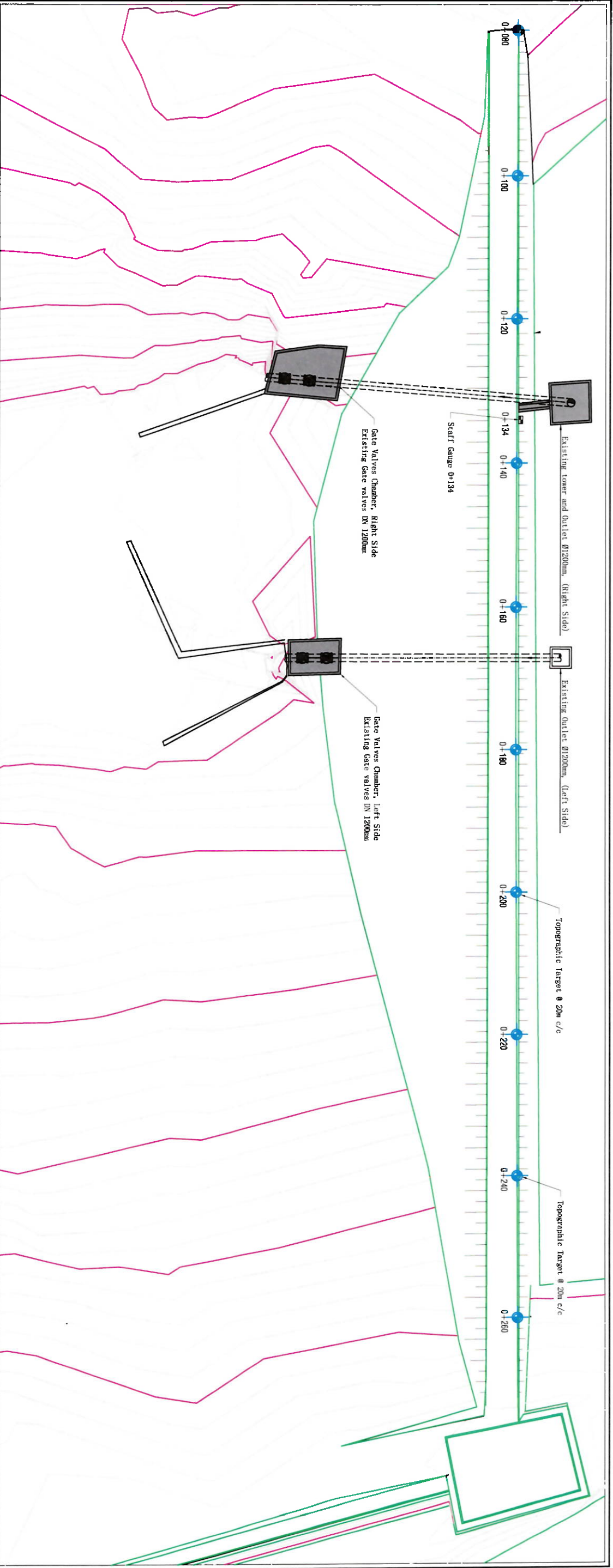


ISLAMIC EMIRATE OF AFGHANISTAN
MINISTRY OF WATER AND ENERGY
DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD

PROJECT NAME	DRAWING	PROVINCE	DISTRICT	DESIGN DATE	SCALE	DESIGNED BY:	CHECKED BY:	APPROVED BY:
SULTAN DAM (REHABILITATION)	THE CREST OF DAM HAND RAIL PLAN AND VIEW	GHAZNI	KHYKHA QAMARI	OCT-2023	AS SHOWN	ENG MURTAZA BARIAL	ENG FARIDULLAH HADARY	ENG ABDUL GHAFOR QAMARI

SHEET NO

11 of 14



ISLAMIC EMIRATE OF AFGHANISTAN
MINISTRY OF WATER AND ENERGY
DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD



PROJECT NAME
SULTAN DAM (REHABILITATION)

DRAWING

INSTRUMENTATION PLAN AND SECTION

PROVINCE

GHAZNI

DISTRICT

KANAKA OMARI

DESIGN DATE

OCT - 2023

SCALE

AS SHOWN

DESIGNED BY:

ENG. FARHULLAH HADARY

CHECKED BY:

ENG. ABDUL GHAFOR OMARI

APPROVED BY:

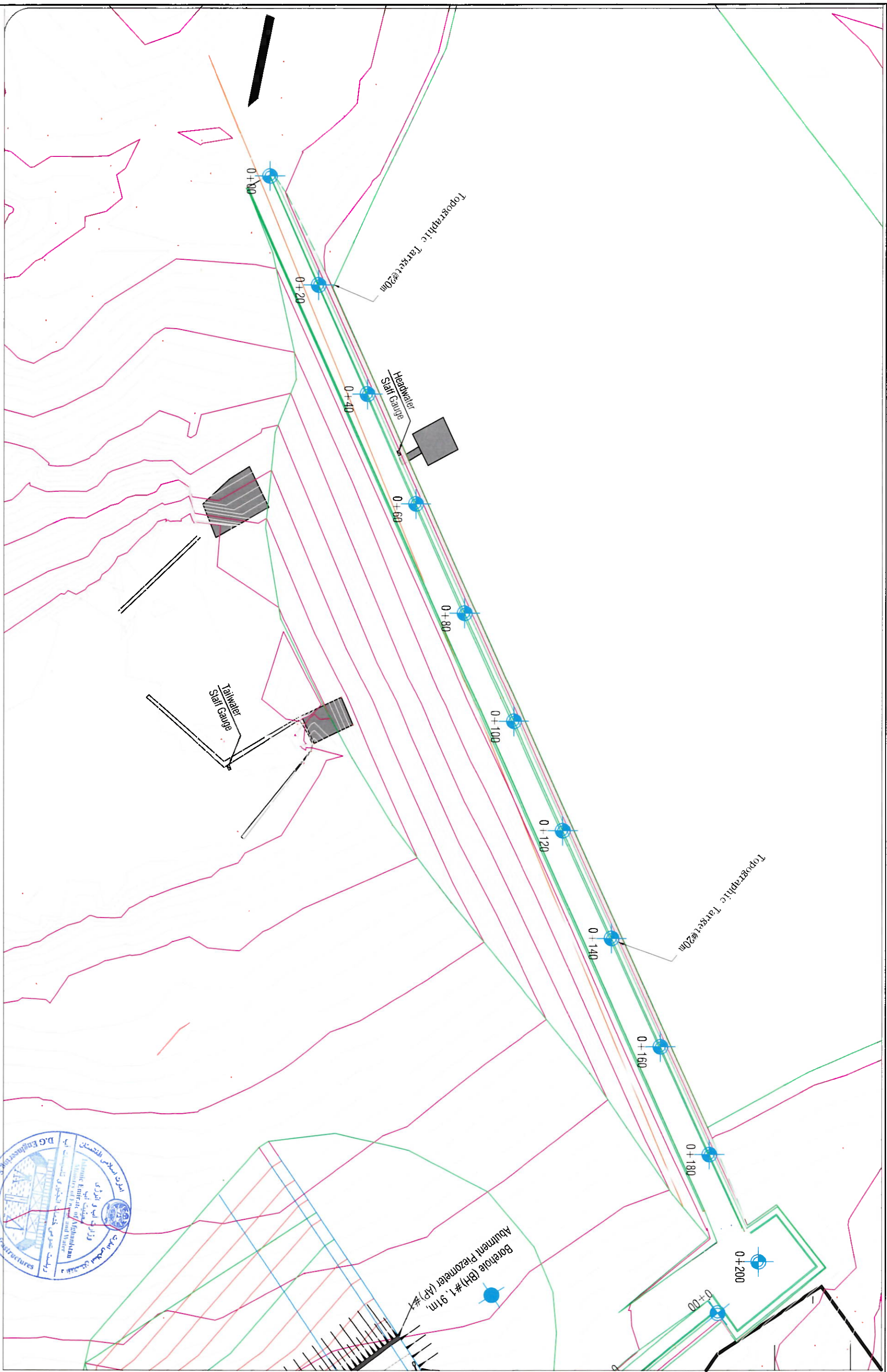
ENG. ABDUL GHAFOR OMARI

SHEET NO

1/3

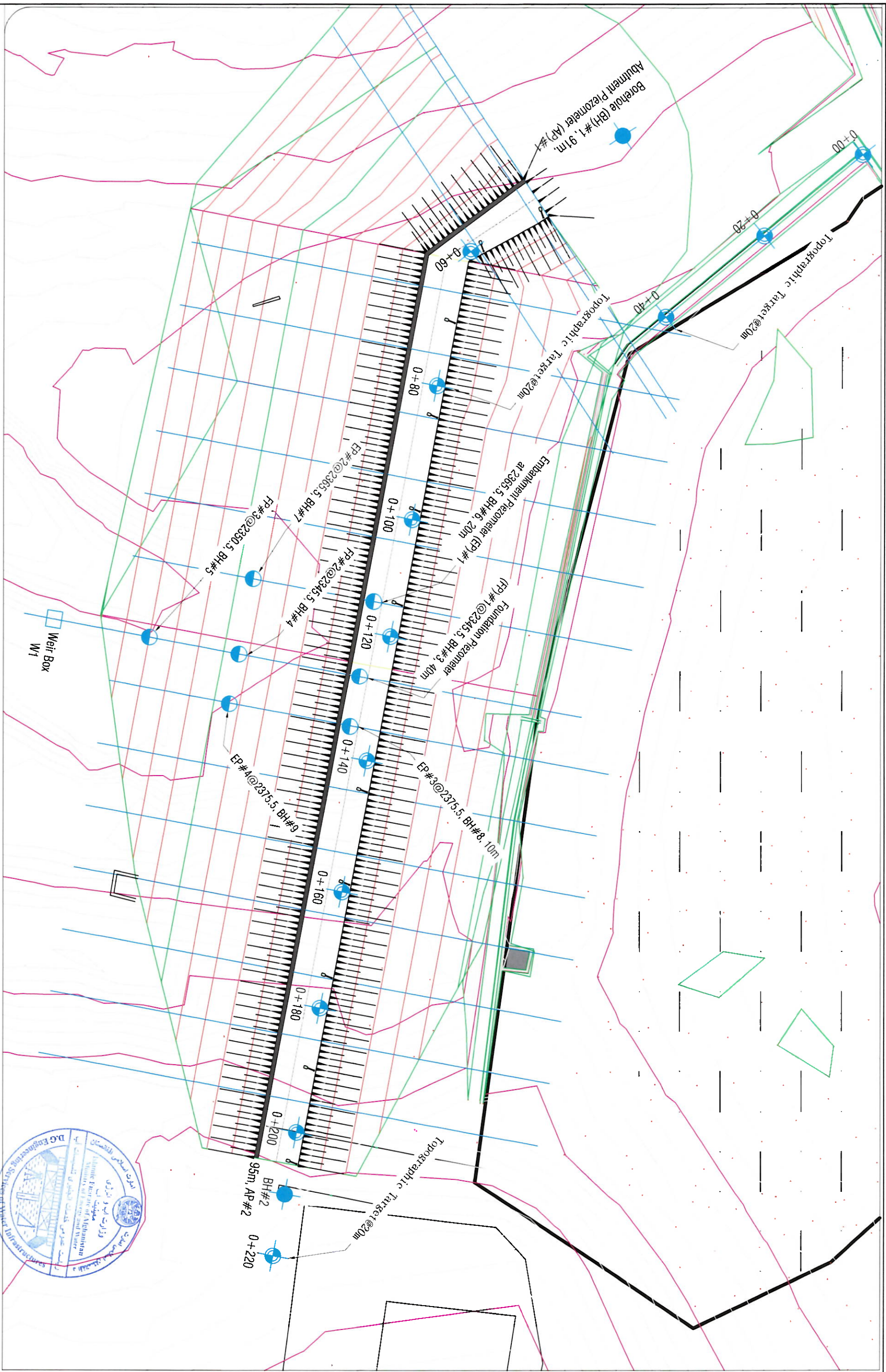
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1/3



PROJECT NAME	DRAWING TITLE	PROVINCE	DISTRICT	DESIGN DATE	SCALE	DESIGNED BY:
SULTAN DAM REHABILITATION WORKS	INSTRUMENTATION PLAN	Ghazni	Khwaja Omari	NOV-2023	AS SHOWN	Eng. FARIDULLAH HAQDARI
						CHECKED BY:
						Eng. MAJID RAZA BAKHAI
						APPROVED BY:



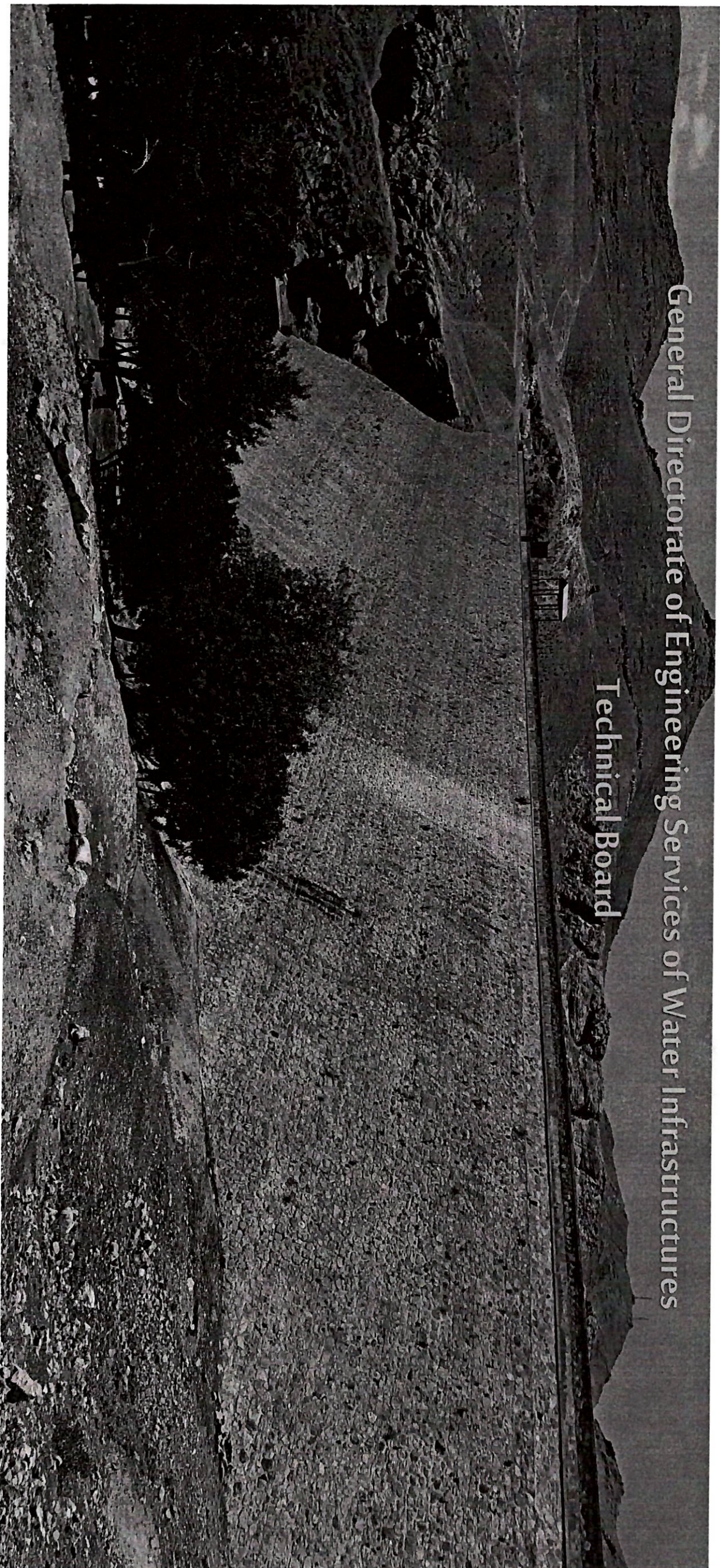
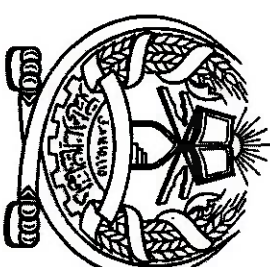


PROJECT NAME	DRAWING TITLE	PROVINCE	DISTRICT	DESIGN DATE	SCALE	DESIGNED BY:	ENGINEER
SULTAN DAM REHABILITATION WORKS	INSTRUMENTATION PLAN	Chazni	Kinwaja Gharri	NOV-2023	AS SHOWN	CHECKED BY:	Eng. FARIDULLAH HADARY
						APPROVED BY:	Eng. MURTIQA BARIALLY





Islamic Emirate of Afghanistan
Ministry of Water and Energy
Deputy Ministry of Water



General Directorate of Engineering Services of Water Infrastructures

Technical Board

Rehabilitation of Sultan Dam

Date		October-2023	
Prepared	Checked	Approved	
Engr. Habib Khan	Engr. Tariq	Abdul Ghafor Omari	

165 m long Earthfill Saddle Dam
Drawings

Description		Page No
Satellite image showing Sultan Dam, and its Saddle dam		
Plan, and Profile of the new Saddle Dam		1 of 11
General Cross Section of the dam Along with other detail		2 of 11
Saddle Dam Cross Sections 0+00, 0+10 m		3 of 11
Saddle Dam Cross Sections 0+20, 0+30 m		4 of 11
Saddle Dam Cross Sections 0+40, 0+50 m		5 of 11
Saddle Dam Cross Sections 0+60, 0+70 m		6 of 11
Saddle Dam Cross Sections 0+80, 0+90 m		7 of 11
Saddle Dam Cross Sections 0+100, 0+110 m		8 of 11
Saddle Dam Cross Sections 0+120, 0+130 m		9 of 11
Saddle Dam Cross Sections 0+140, 0+150 m		10 of 11
Saddle Dam Cross Sections 0+160, 0+1620 m		11 of 11



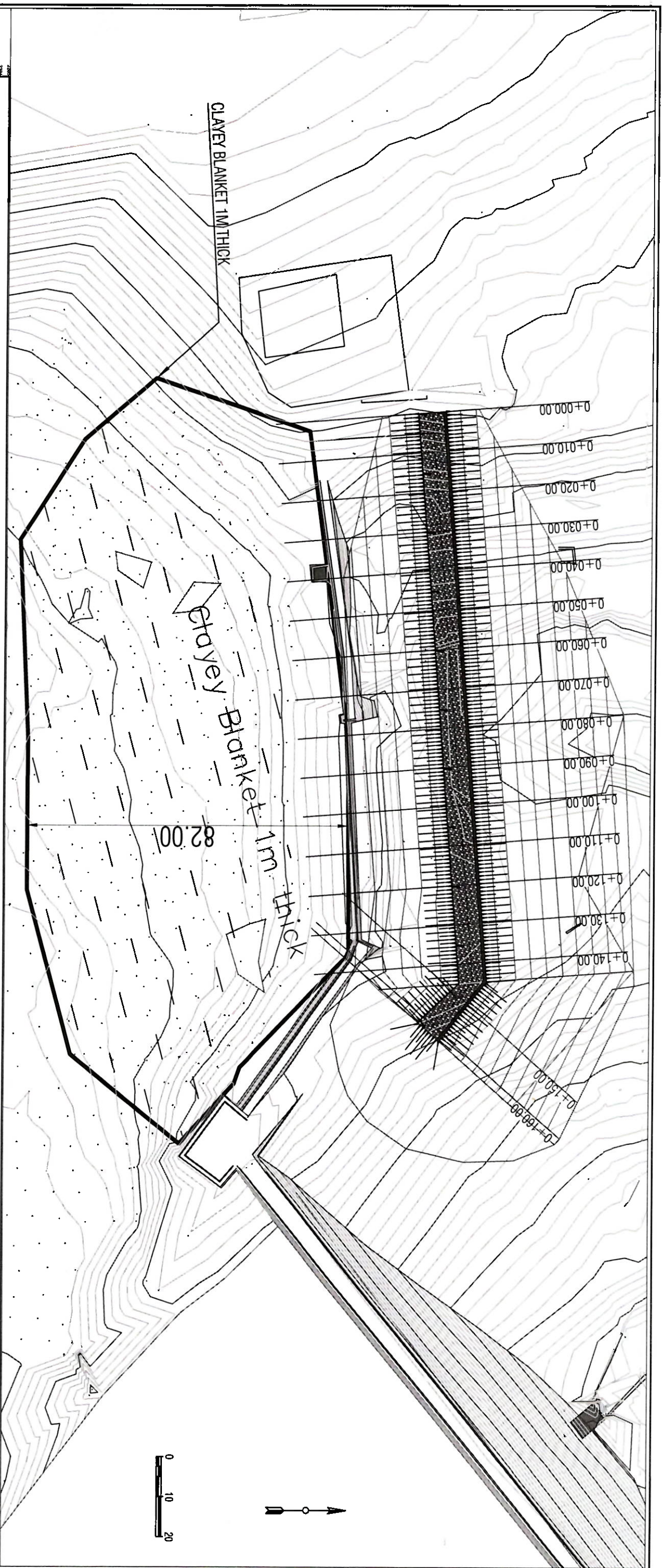
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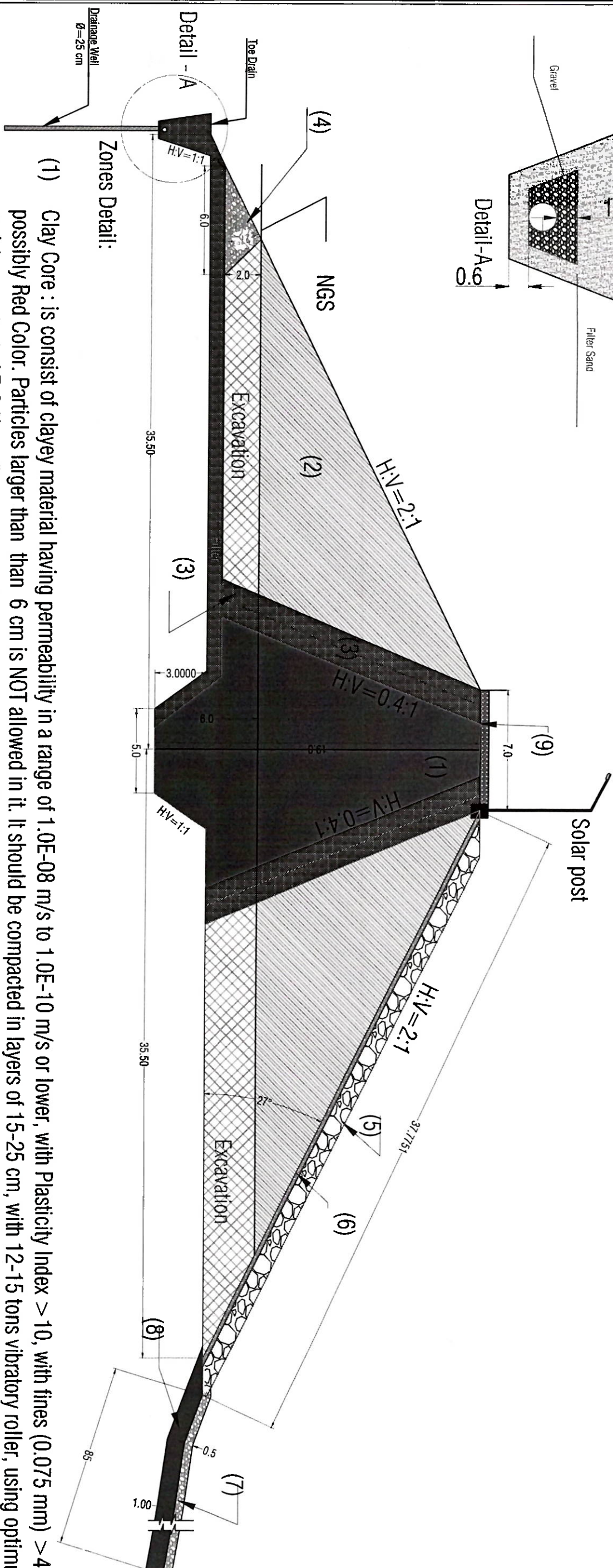
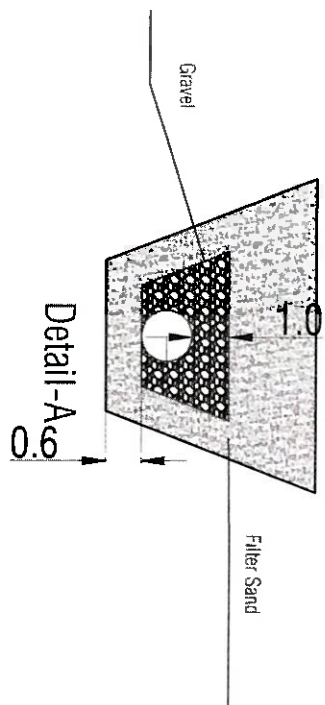
Google Earth Image showing Sultan Dam, and its Saddle dam along with Collapsed Section:



Signature



Station	0+000+000	0+010	0+020	0+030	0+040	0+050	0+060	0+070	0+080	0+090	0+100	0+110	0+120	0+130	0+140
Elevation (m)	2381.688	2377.565	2373.445	2371.123	2371.816	2374.308	2373.906	2373.854	2373.812	2373.769	2373.880	2374.251	2375.448	2375.161	2374.874
Station	2384.93	2380.68	2376.71	2374.33	2375.22	2377.57	2377.06	2376.77	2376.64	2376.69	2376.63	2377.31	2378.57	2378.33	2377.88

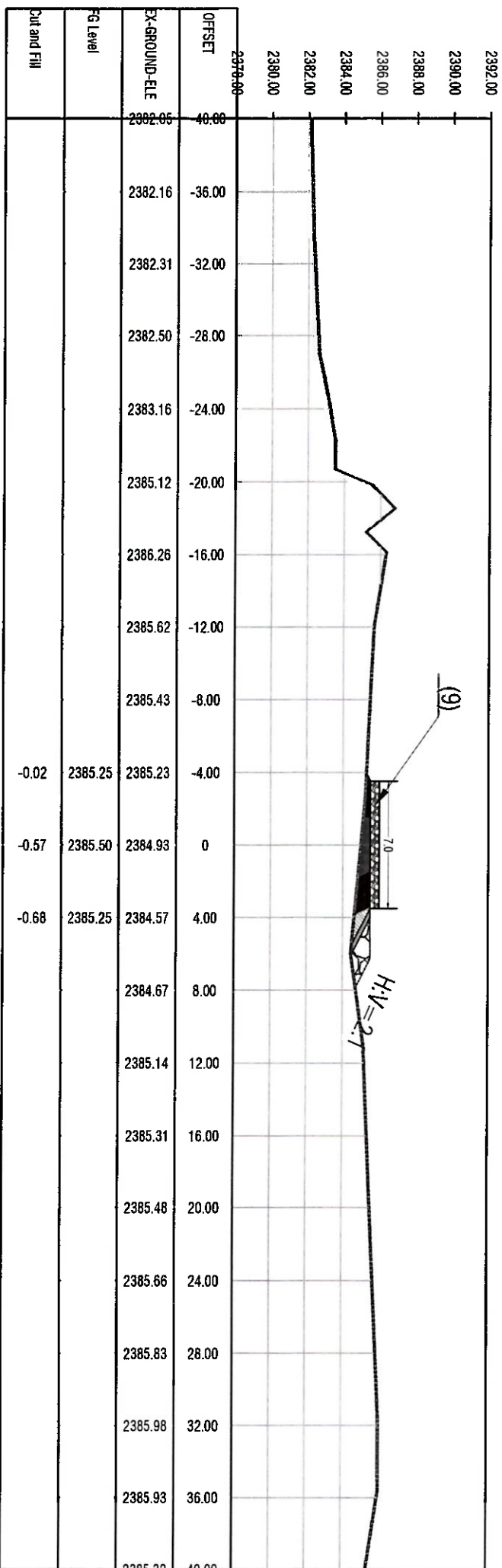


Zones Detail:

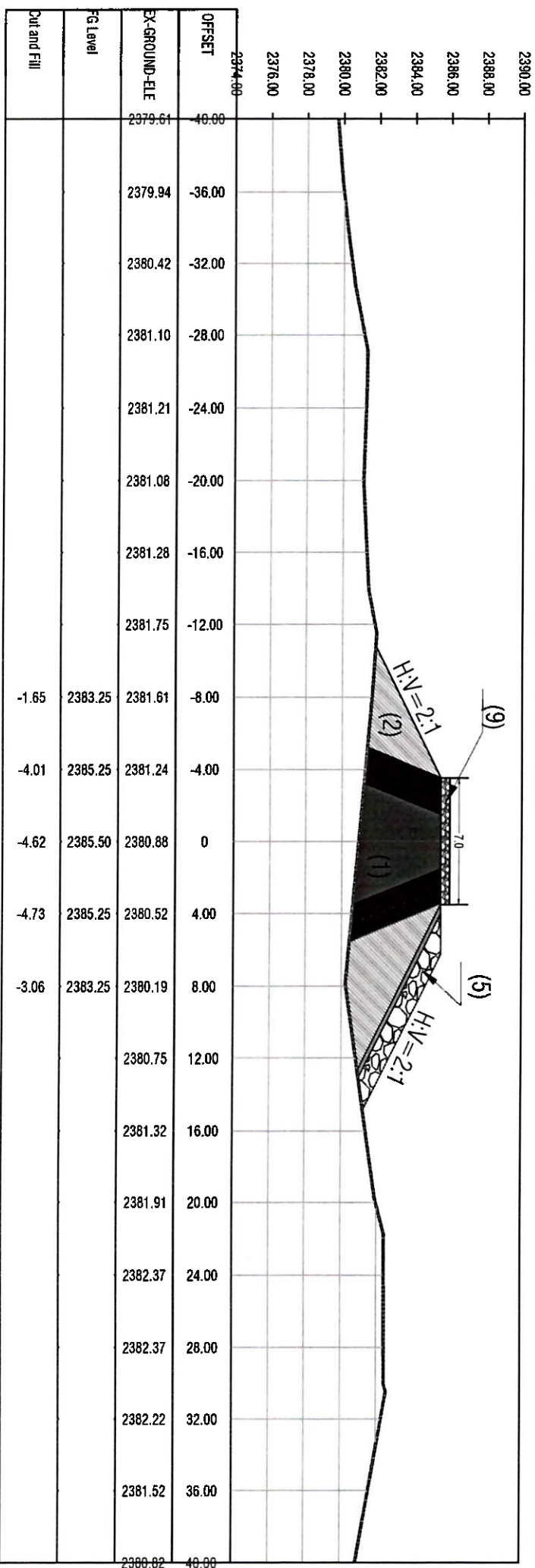
- (1) Clay Core : is consist of clayey material having permeability in a range of 1.0E-08 m/s to 1.0E-10 m/s or lower, with Plasticity Index > 10 , with fines (0.075 mm) $> 45\%$, possibly Red Color. Particles larger than 6 cm is NOT allowed in it. It should be compacted in layers of 15-25 cm, with 12-15 tons vibratory roller, using optimum moisture content of 7-9 % confirmed with Geotechnical Laboratory testing. The side slopes of core is 0.4:1 (H:V). Each single layer should be (monolithic) completed from start - end of the saddle before starting a new layer. Do not use homogenous material to create a sandy or gravel layer in the core
- (2) Shoulders: are consist of alluvial, sand, and gravels compacted in layers of 15-25 cm with 10-12 passes of 12-15 tons vibratory compactors with speed of 3 Km/hr, and optimum moisture content (o give highest dry density) obtained from geotechnical laboratory. Well graded material with Friction Angle > 40 deg is always preferable.
- (3) Filter Zone. Well, graded clean sand and gravel to provide smooth drainage, and is 2 m thick with hydraulic conductivity, $K \geq 1 \times 10^{-3}$ cm/s (1.0E-05 m/s) or higher. Filter layer at the foundation is 1 m thick as shown in the drawing. Filter sand must contain $\leq 5\%$ fines (sand particle size below 0.075 mm) to prevent piping and retain clay core particles and thus prevent washing them, D15 =0.5 mm, with D100 =75 mm. The filter sand has zero plasticity.
- (4) Toe Drain: Containing sand, but mostly gravel to effectively intercept phreatic line/seepages and guided it to the Toe Drain Perforated Pipe as shown in drawing.
- (5) Zone-5: Riprap is consisting of $0.8 \text{ m}^3 - 1 \text{ m}^3$ large blasted stone to protect slopes against waves.
- (6) Zone-6: Sandy Gravel layer of 1 m thick to enhance seepage from u/s face in case of quick drawdown.
- (7) Zone-7: Gravel layer of 0.5 m thick to protect underneath clayey blanket at the upstream within the reservoir
- (8) Zone-8: Upstream 1m thick clayey blanket to prevent seepages.
- (9) Zone-9: 0.5 m thick gravel layer to protect core from freezing and thaw and thus swelling, as well as to provide protection during overtopping.



Saddle Dam Cross Section Station=0+000.00



Saddle Dam Cross Section Station=0+010.00

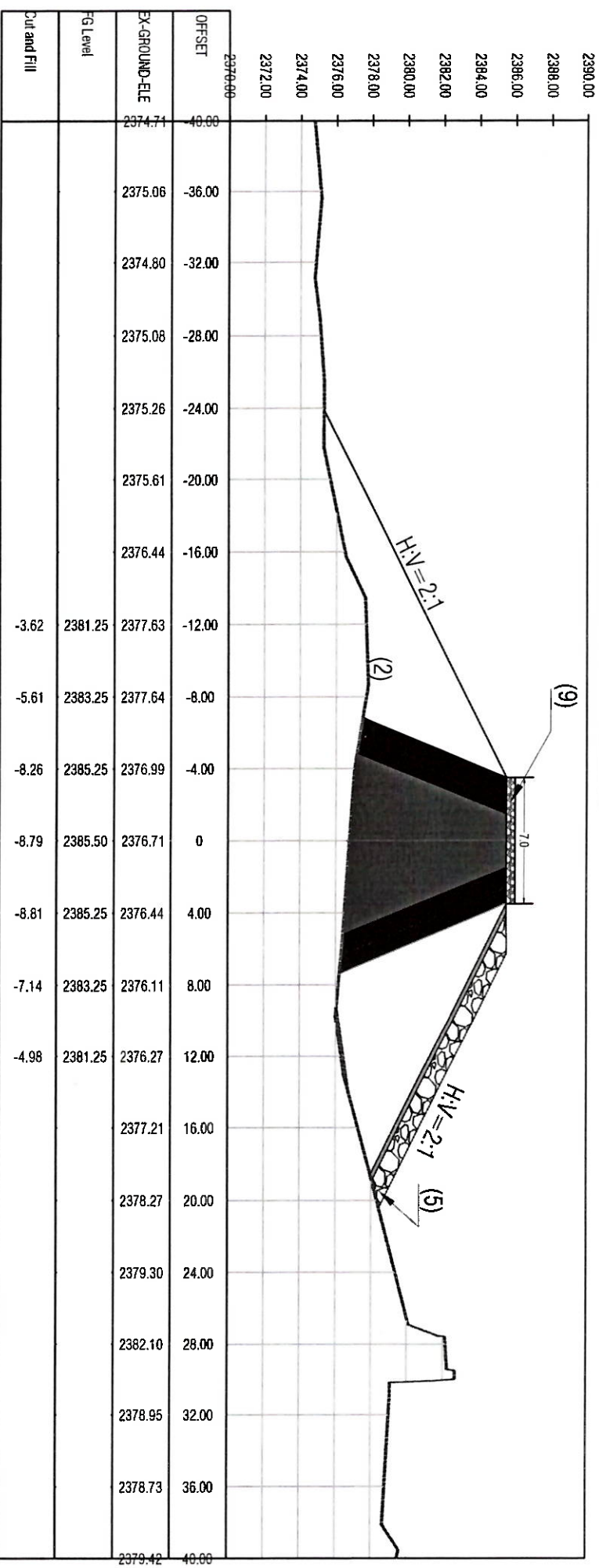


Technical Detail

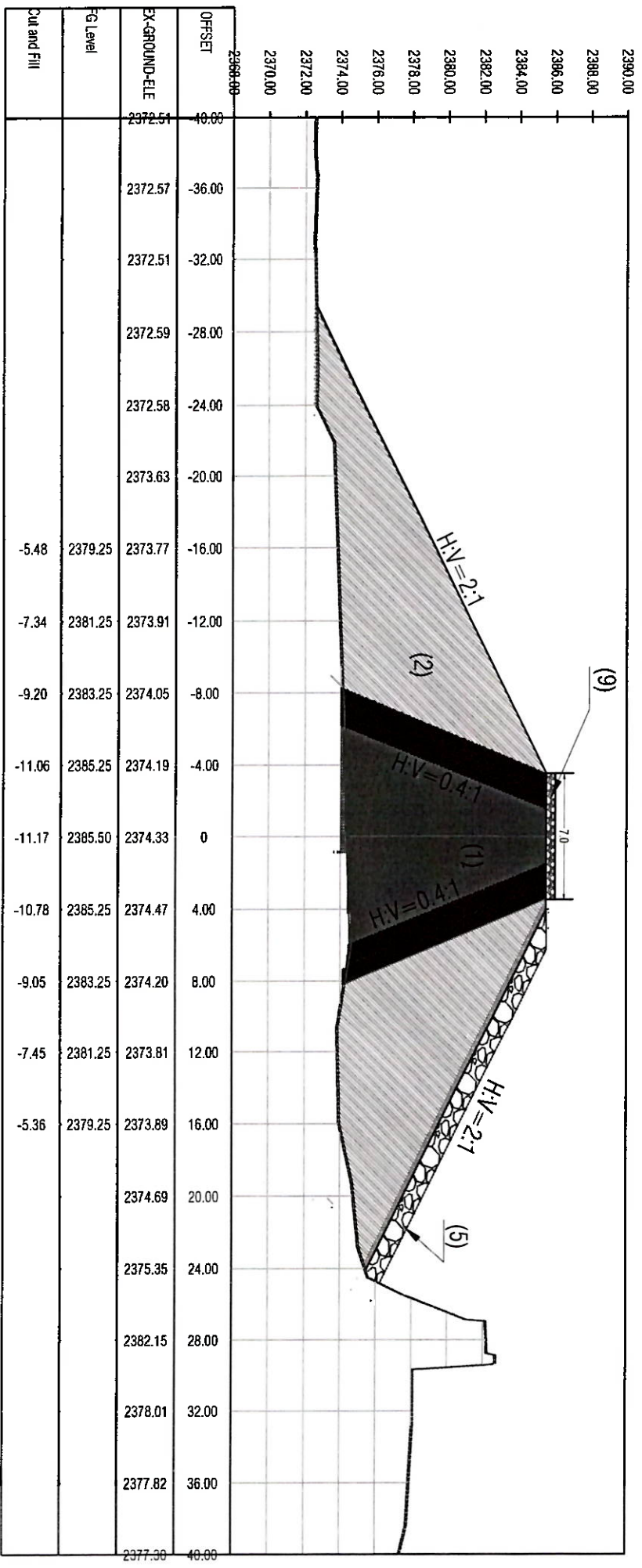
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7. Upstream blanket of 1 m thick clayey blanket (core material), overlayed by 0.5 m thick gravel.
- 8.



Saddle Dam Cross Section Station=0+020.00



Saddle Dam Cross Section Station=0+030.00

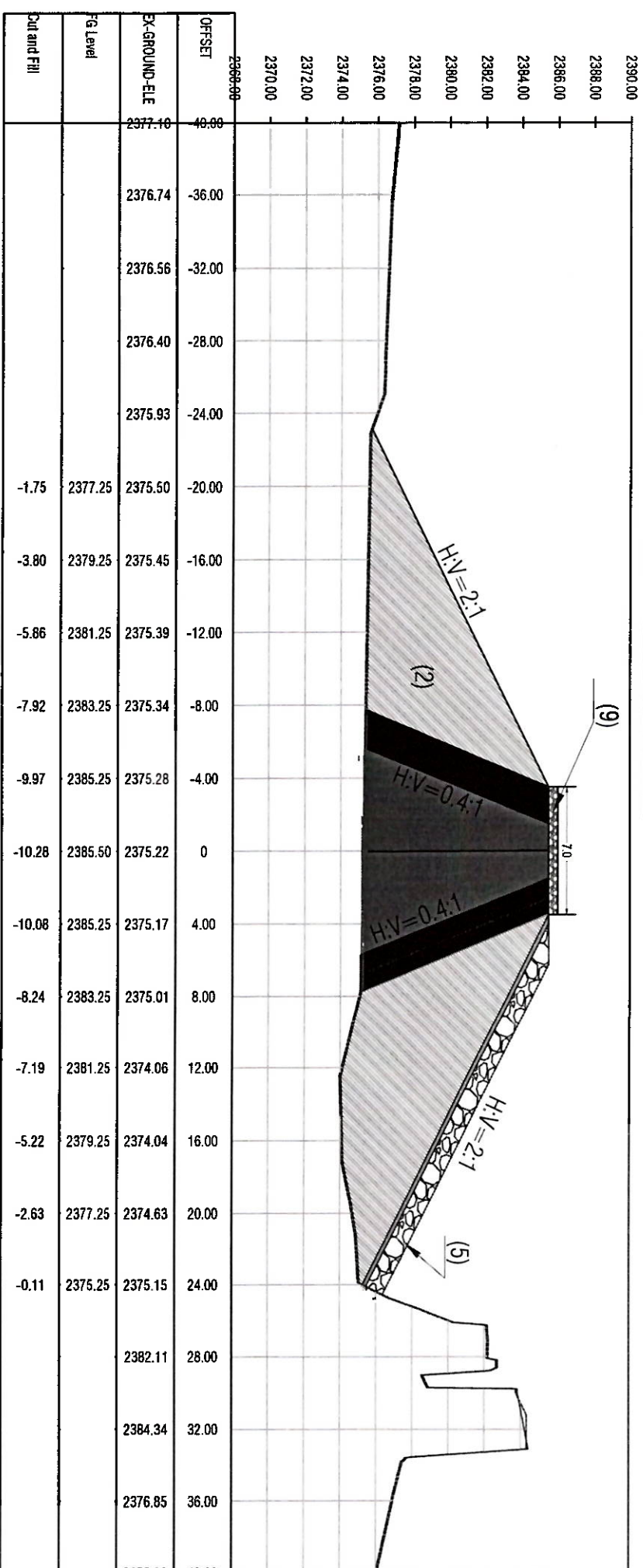


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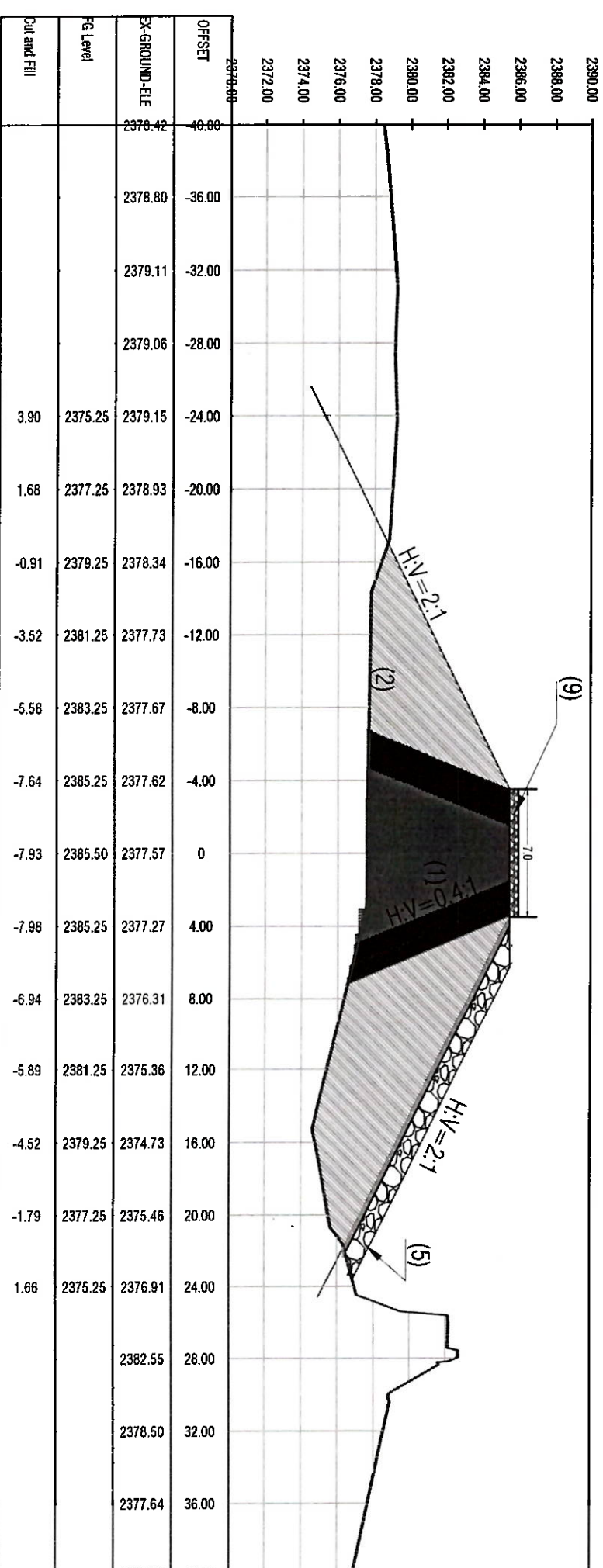
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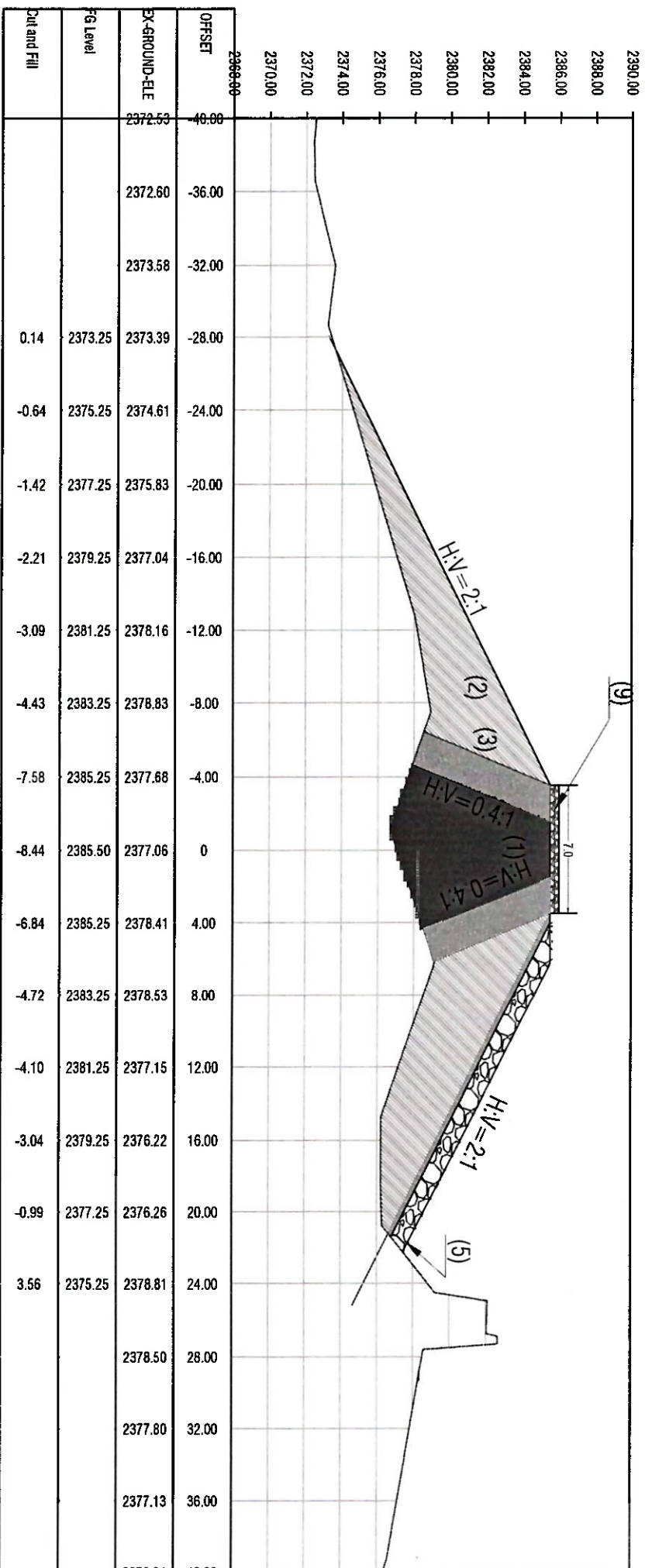
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Saddle Dam Cross Section Station=0+050.00



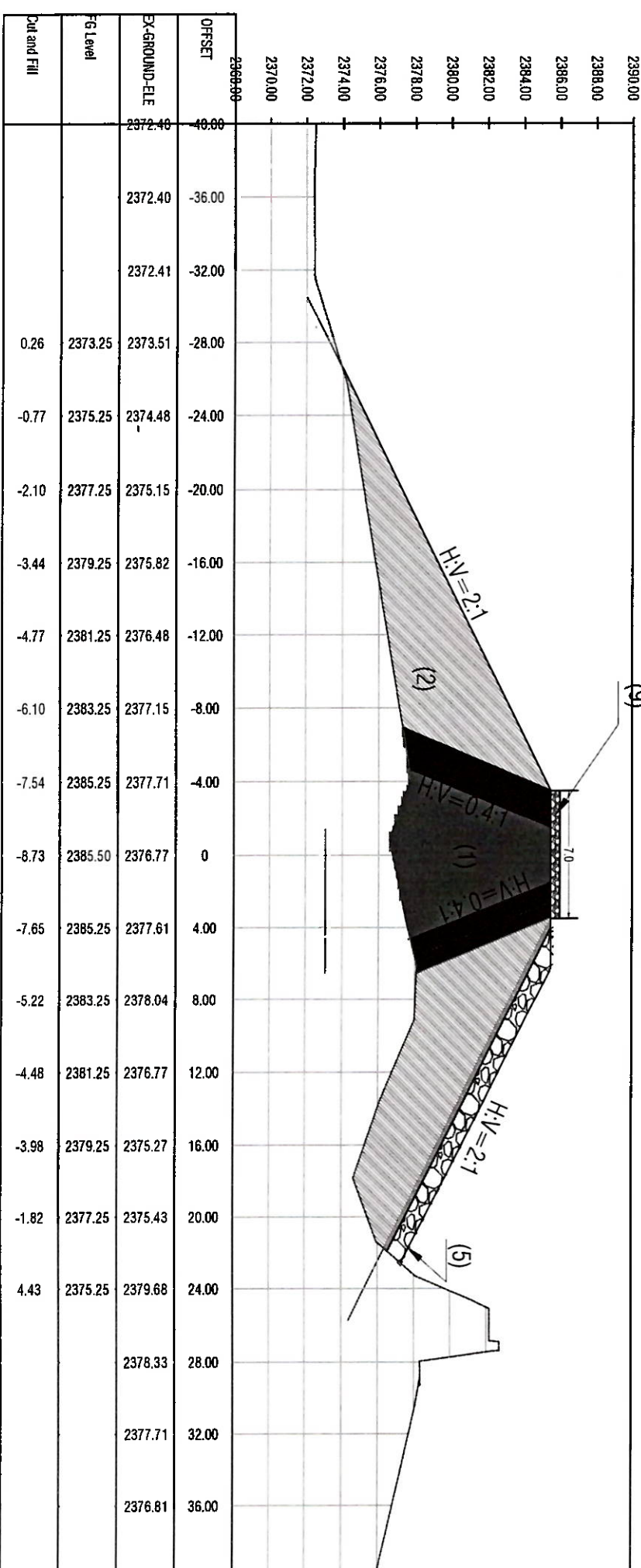
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Technical Detail

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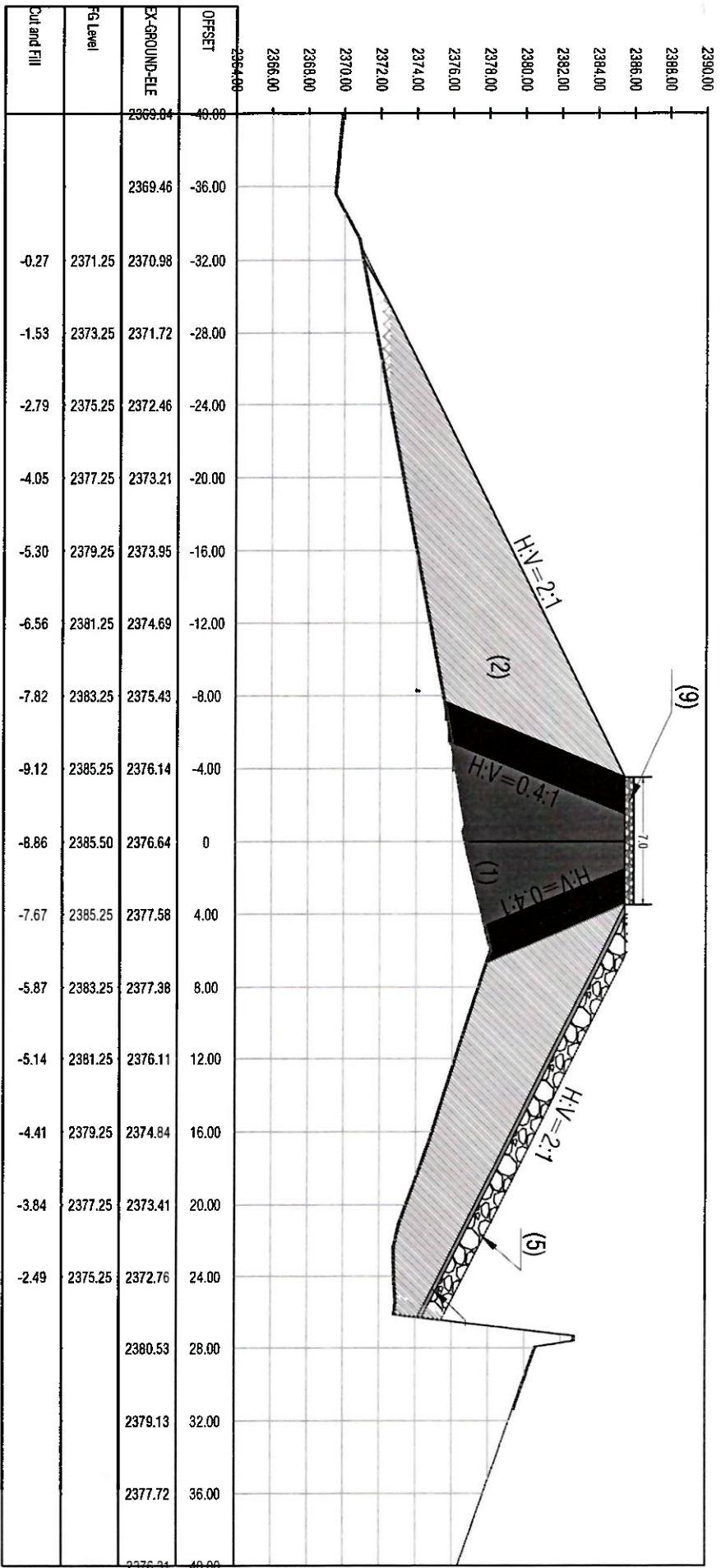


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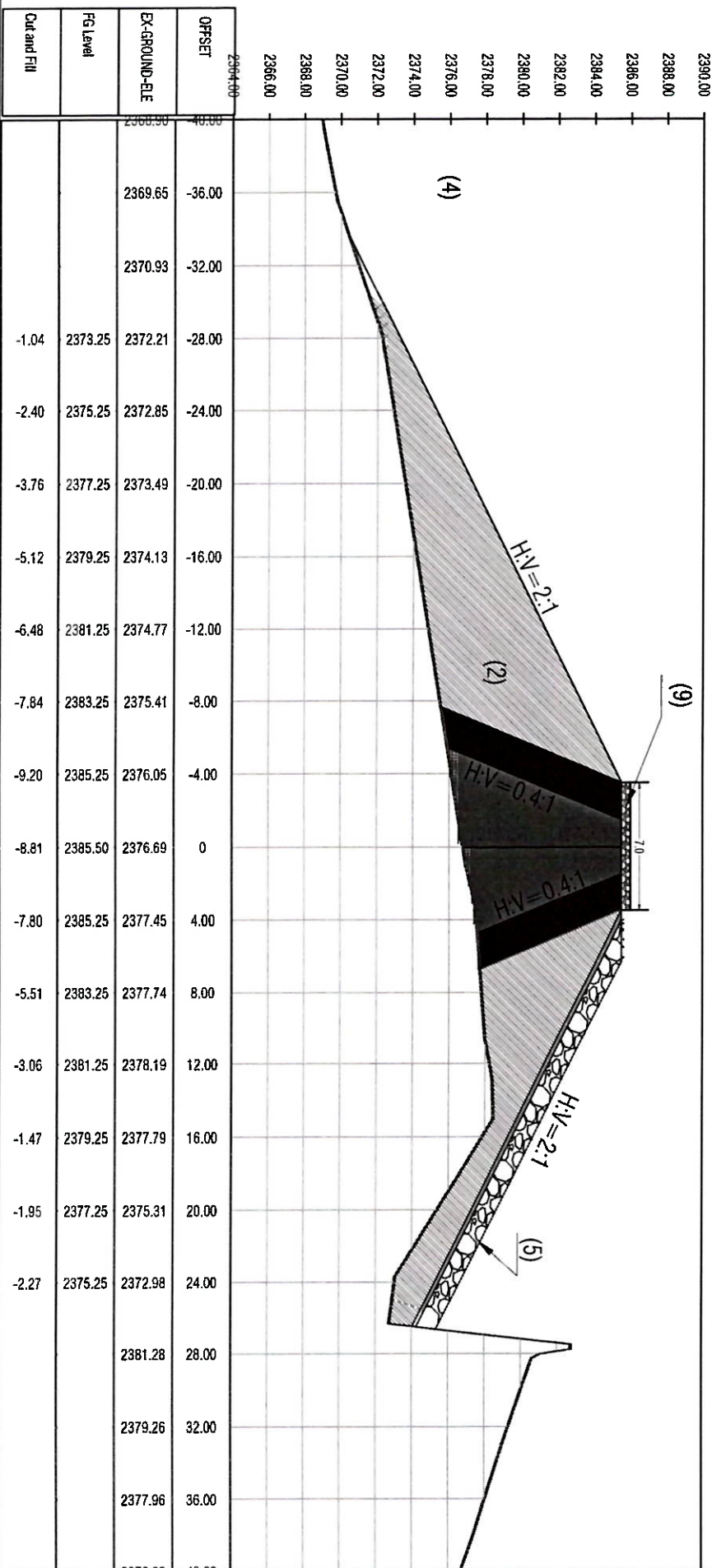
ISLAMIC REPUBLIC OF AFGHANISTAN				DEPUTY MINISTRY OF WATER			
MINISTRY OF WATER AND ENERGY				GENERAL DIRECTORATE OF ENGINEERING SERVICES			
TECHNICAL BOARD							
PROJECT NAME		DRAWING		PROVINCE		DISTRICT	
Saddan Dam-Ghazni (Saddle Dam)		Rehabilitation of 161 m long Saddle Dam		Ghazni		Khwaja Khairi	
DESIGN DATE		SCALE		DESIGNED BY:		CHECKED BY:	
Sept-2023		AS SHOWN		Engr. HABIB KHAN		Engr. Tariq	
APPROVED BY:				APPROVED BY:			
SHEET NO		CS		SHEET NO		CS	
6		1		6		1	



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Saddle Dam Cross Section Station=0+090.00

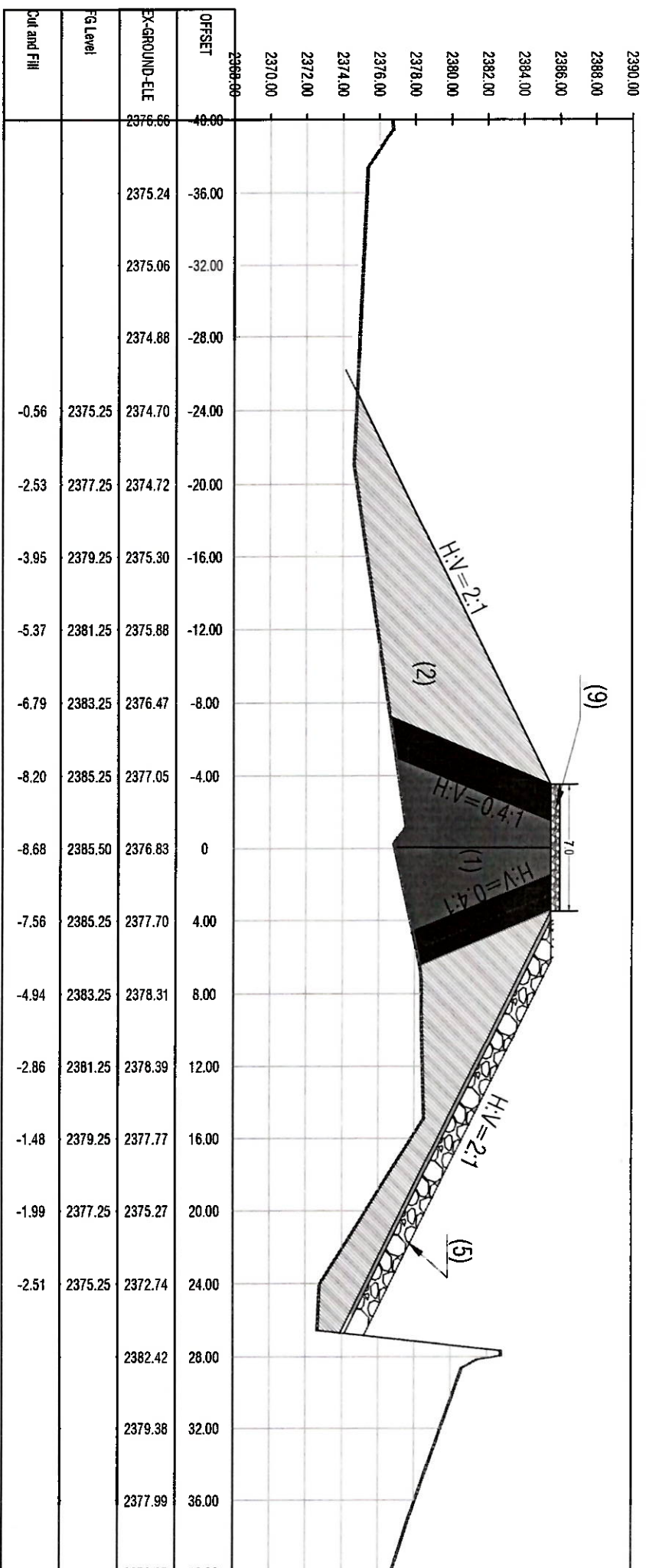


Filter Design (Various Size of Sand-Gravel)

Point	Percentage	Size (mm)
1.	D15 max	0.5
2	D15 min	0.1
3	D60 max	3.0
4.	D60 min	0.6
5.	D5 min	0.075
6.	D100 max	75
7.	D90 Max	20



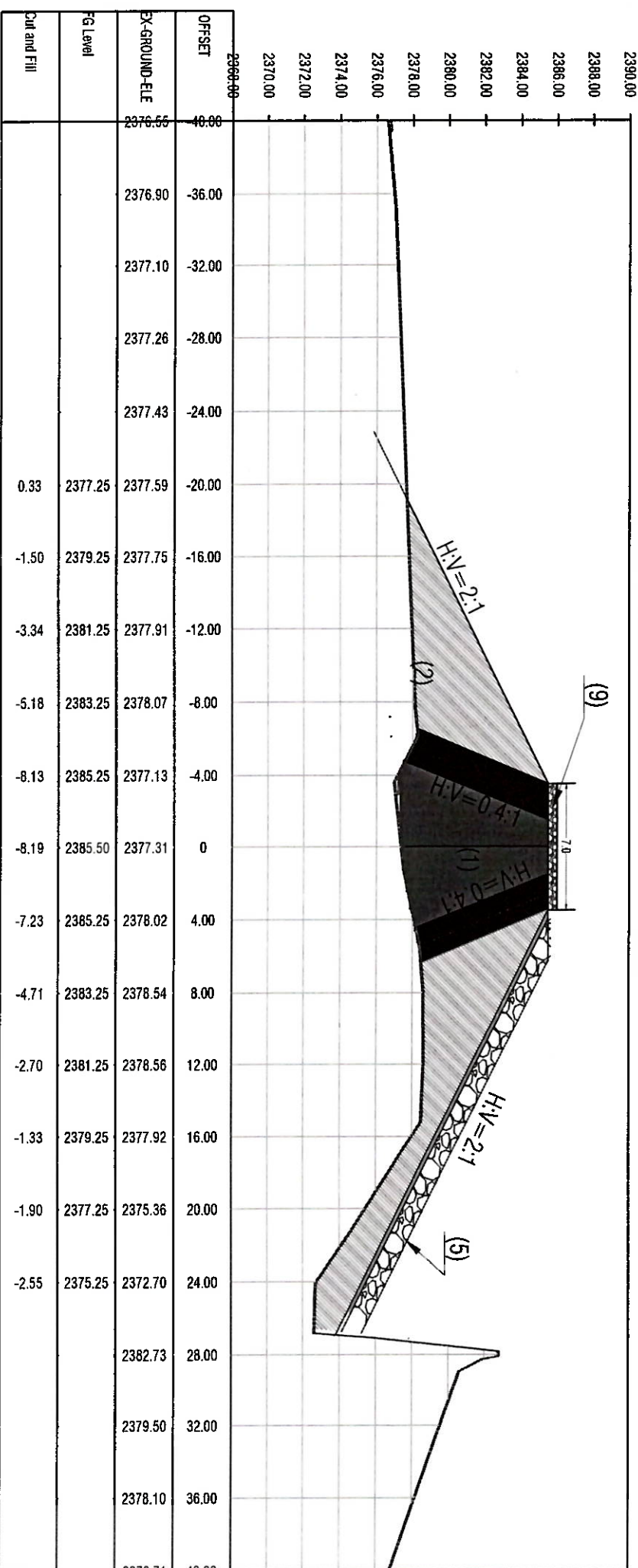
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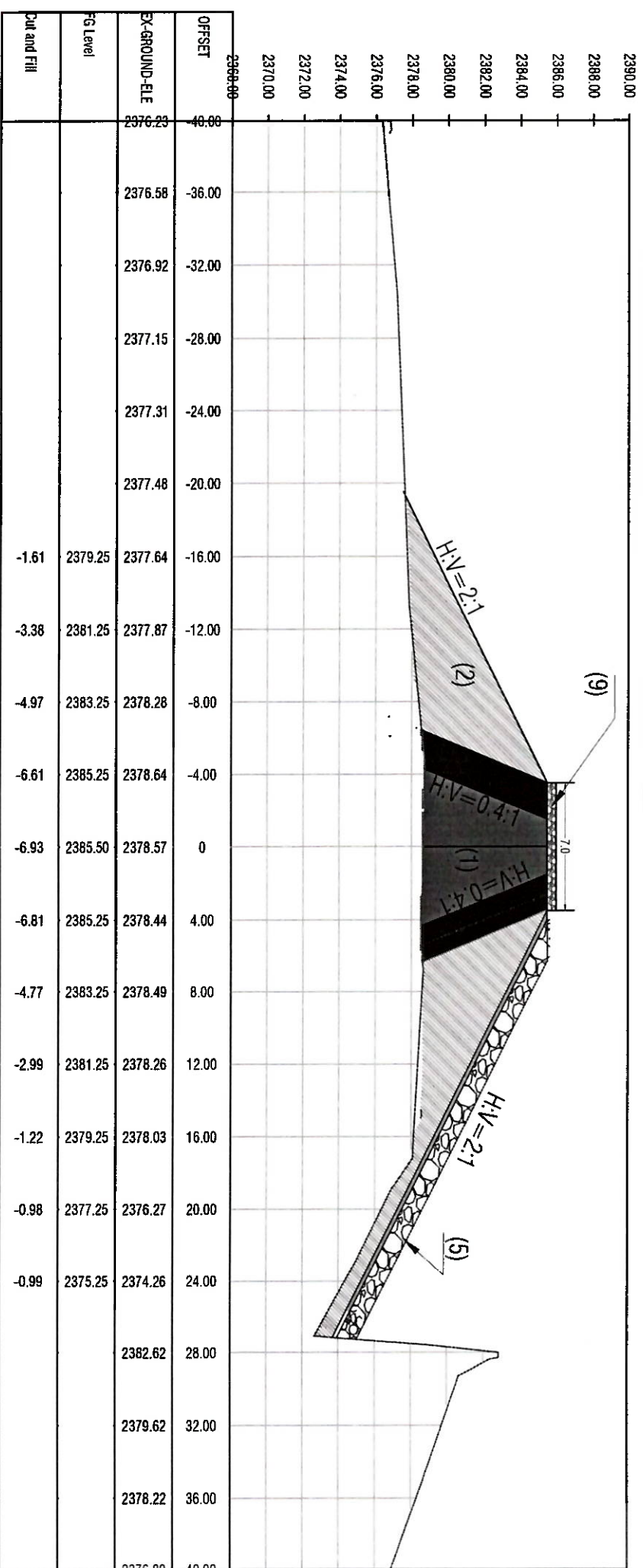
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- Upstream blanket of 1 m thick clayey blanket (core material), overlayed by 0.5 m thick gravel

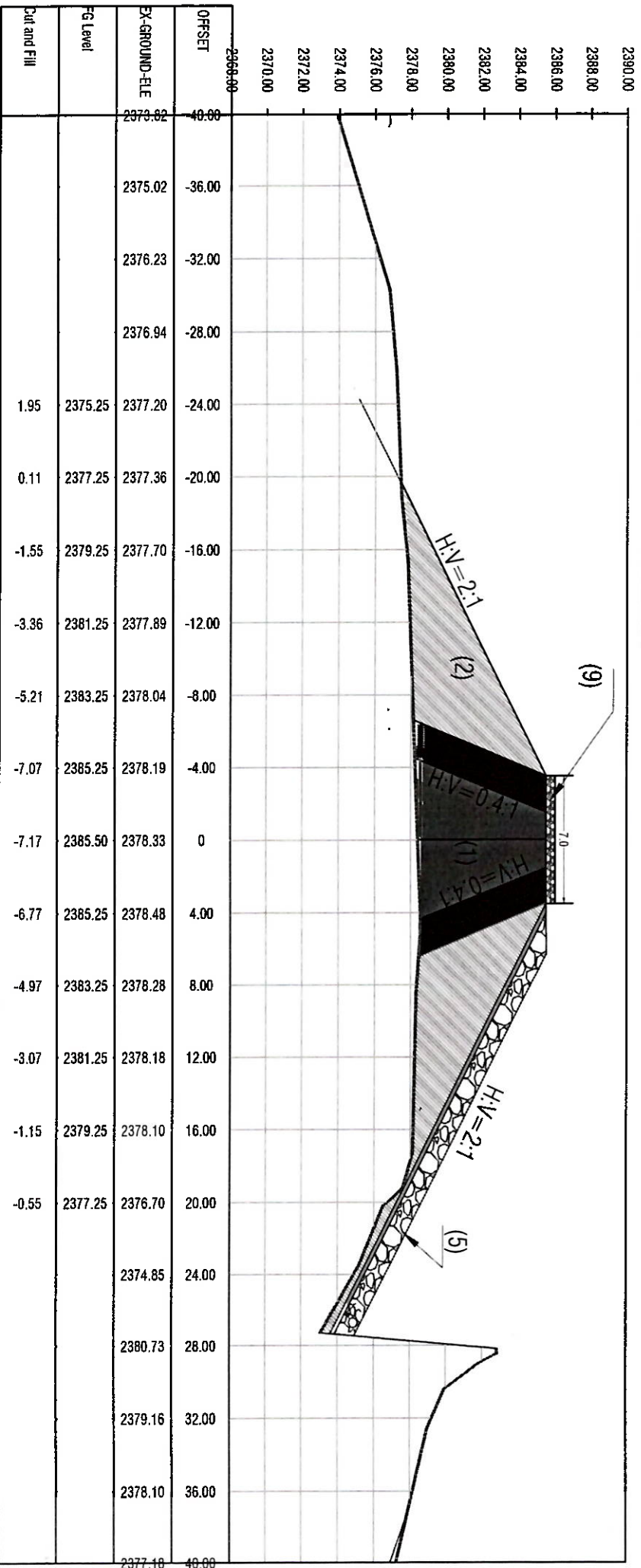
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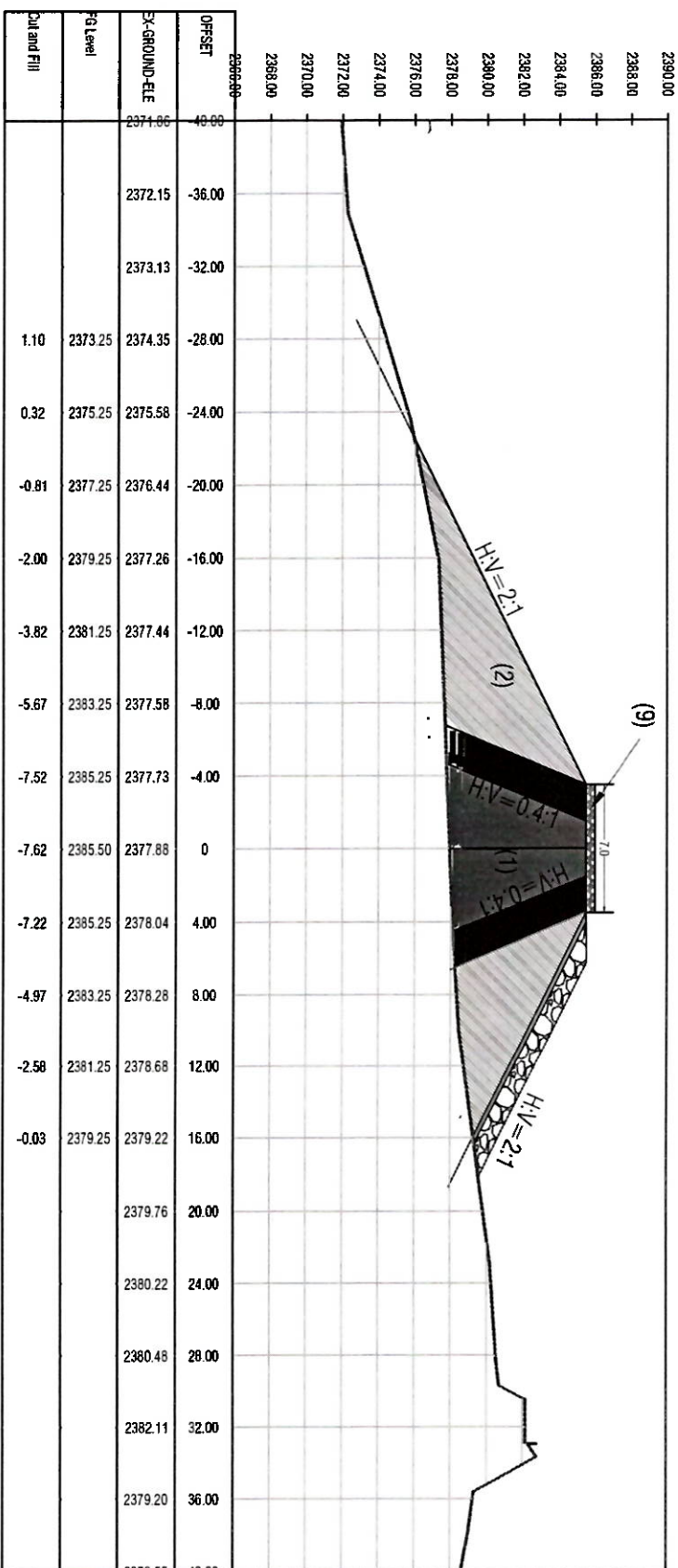


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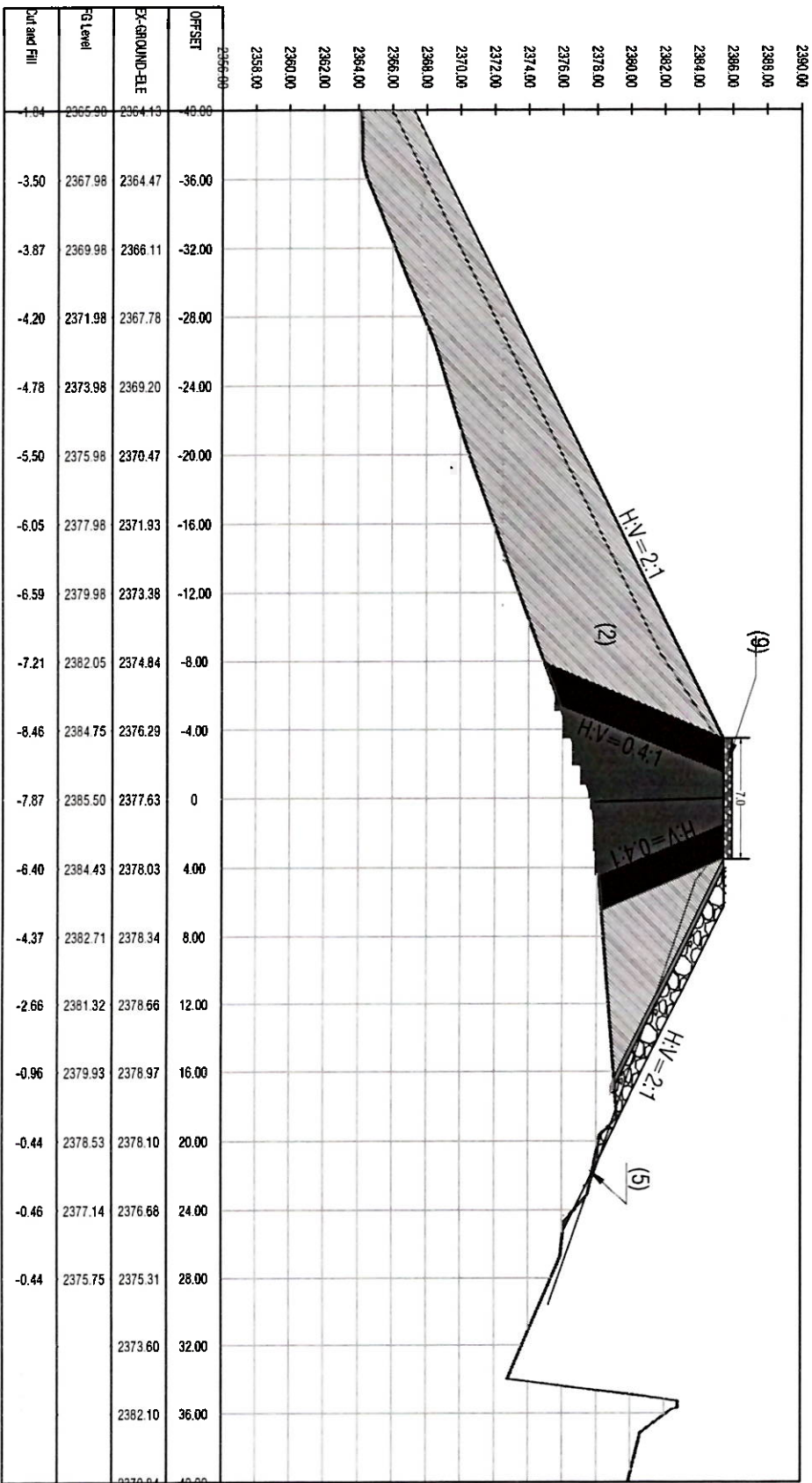
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Saddle Dam Cross Section Station=0+140.00



Saddle Dam Cross Section Station=0+150.00



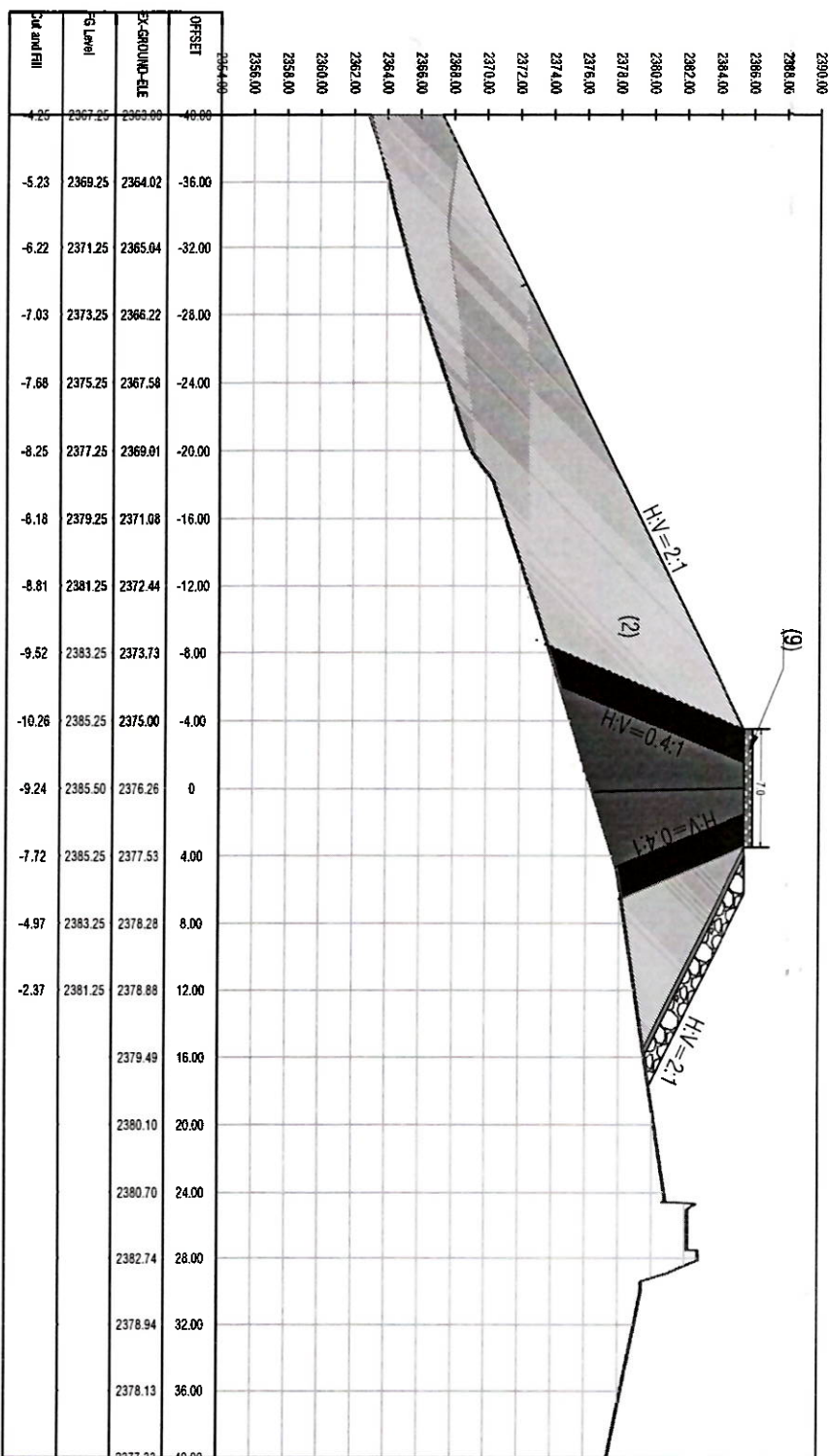
Filter Design (Various Size of Sand-Gravel)		
Point	Percentage	Size (mm)
1.	D15 max	0.5
2.	D15 min	0.1
3.	D60 max	3.0
4.	D60 min	0.6
5.	D5 min	0.075
6.	D100 max	75
7.	D90 Max	20

Technical Detail

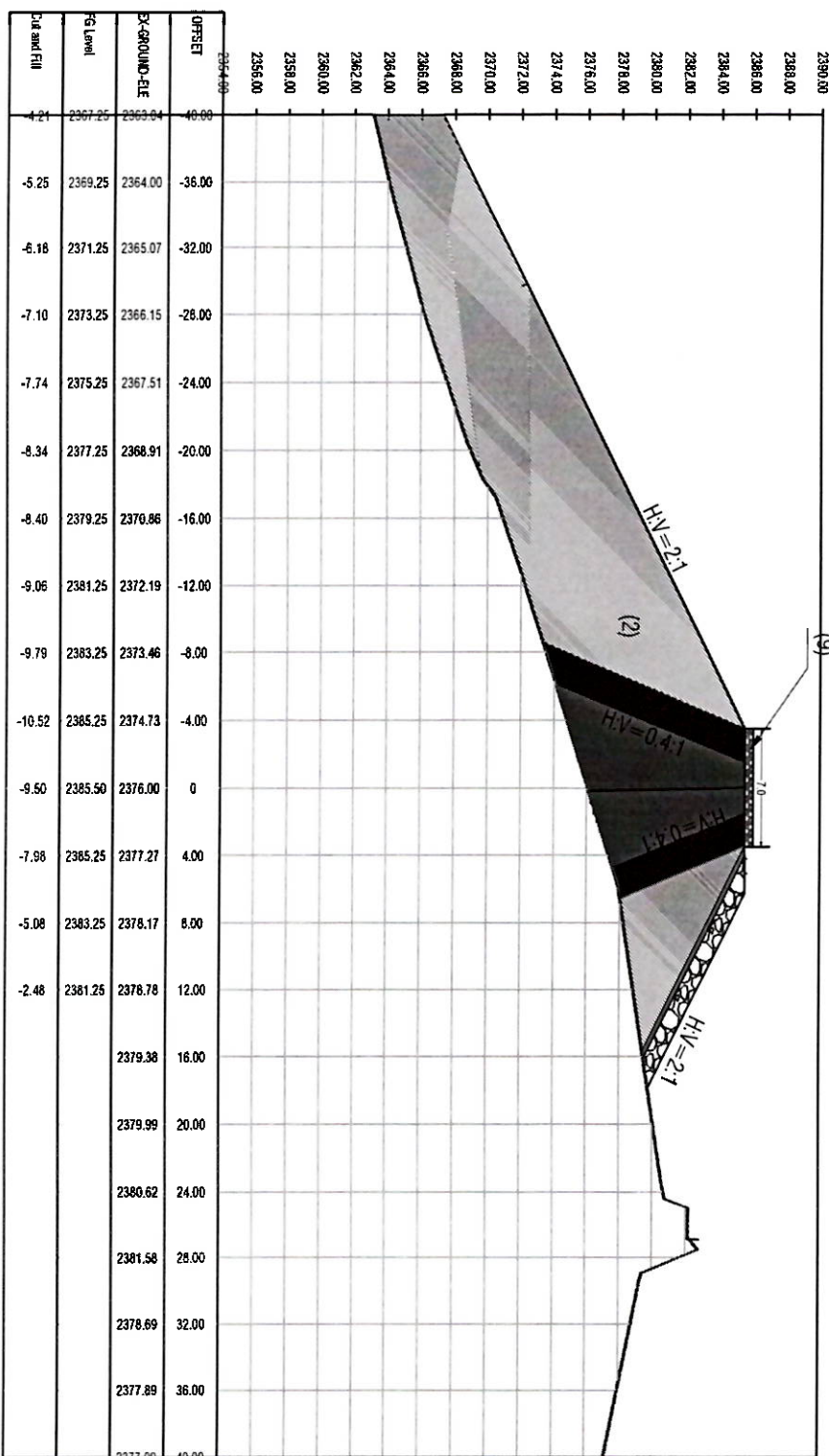
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Saddle Dam Cross Section Station=0+160.00



Saddle Dam Cross Section Station=0+162



Filter Design (Various Size of Sand-Gravel)		
Point	Percentage	Size (mm)
1.	D15 max	0.5
2.	D15 min	0.1
3.	D60 max	3.0
4.	D60 min	0.6
5.	D5 min	0.075
6.	D100 max	75
7.	D90 Max	20

Technical Detail

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ISLAMIC EMIRATE OF AFGHANISTAN
MINISTRY OF WATER AND ENERGY
DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD

PROJECT NAME

DRAMING

PROVINCE

DISTRICT

DESIGN DATE

SCALE

DESIGNED BY:

CHECKED BY:

APPROVED BY:

SHEET NO

Sultan Bani-Ghazni (Saddle Dam)

Rehabilitation of 161 m long Saddle Dam

Ghazni

Khwaib Urman

Sep-2023

AS SHOWN

Engr. HABIB KHAN

Engr. Tariq

11



Islamic Emirate of Afghanistan
Ministry of Water and Energy
Deputy Ministry of Water



General Directorate of Engineering Services of Water Infrastructures
Technical Board



Picture: Saddle dam of Sultan-Dam Ghazni province.

Sultan Dam Rehabilitation -			
Date		July-2023	
Prepared	Checked	Approved	
Habib Khan habib.khan@mew.gov.af	Engr. Tariq Tasal	Earthfill Saddle dam- Design Report	
			



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1. Key features of the project

The existing main dam will be raised by 2.5 meters, and a new saddle 162 m long, and 19 m high will be constructed to replace the existing poorly constructed stone masonry saddle dam.

Table-01: Key features of the Sultan-Dam rehabilitation project is given below.

Sno	Description	Value	Remarks
1	Main Dam type	Gravity	
2	Material Type	Mortar Stone Masonry	
3	Foundation Width	37 m	
4	Crest Length	195m	
5	Crest Width	5.7 m including parapet wall (Clear width is 4.3 m), while the crest of the Earthfill saddle is 7.0 m	
6	(i) Existing Crest elevation (ii) New Crest elevation	2,381 Masl 2,383.5 Masl for main dam 2,385.5+0.5 (camber) for Earthfill saddle	
7	Existing Hydraulic Height of the Dam	34 m + 1.4 m Parapet wall	
8	New Hydraulic Height of the dam	36.5 m + 1 m Parapet wall	
9	Upstream Slope	0	
10	Downstream Slope	0.7:1 (H: V)	
11	Thickness of the Parapet wall	0.8 m	
12	River Bed Elevation	2,347 Masl	
13	Dam Foundation Elevation	2,346 Masl	
14	New Maximum Water Level	2,383.5 Masl (with 2.5 meter raising of the main dam), with water head of 2.5 m above the spillway crest.	
15	Existing Normal Water Level New Normal Water Level	2,370 Masl 2,381 Masl	



16	Dead Storage Level	2,349 masl – as there are two bottom outlet gates which are instated 2m high from the riverbed level.	
17	Existing Gross Storage New Gross Storage Volume	2 M m ³ Approx. 8.0 M m ³ @ 2,381 Masl level 11.28 M m ³ @ 2,384 Masl level	
18	Active Storage Volume	8.0 M m ³	
19	Dead Storage Volume based on 100-year sediment deposition level	0 M m ³ as the bottom outlet gates are used to flush the sediment.	
20	New Earthfill saddle dam, Height, Length and Volume	19 m (high) 162 m long 79,121 m ³ (including all zones)	
21	Upstream 1m thick clayey blanket with 0.5 m thick gravel layer Volume	23,725 m ³	
22	Vol of upstream Riprap	5,832 m ³	
22	Vol of Rockfill Concrete for Raising main dam	3,638 m ³ (for main dam only)	
23	Estimated Cost of Dam Body (saddle only)	Afn 36.5 million	
24	Anticipated Life	More than s 100 year	



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2. Executive Summary

On March, 29 2005, a 55 m long section of the saddle dam of Band-e-Sultan failed in the morning 4:00 am sending flood waves which caused enormous damage to the property within the capital city, as well as to the agricultural land on its way. The Ghazni City is located approx. 25 km in the downstream, and is in direct threat of the flood from this reservoir. A catchment of 1,165 - 1,200 km² is draining to this dam, with two streams coming from left and right directions joining at the dam site. The annual volume of the flow is 50 million m³, while currently the dam can store only 1-2 million m³ in a single filling. The Technical Board of the MEW has decided to raise the main dam by 2.5 m, making the storage capacity of the dam up to 10 million m³, which will triple the agricultural production at downstream cultivable land.

A flood of 135 m³/s is recorded in the past, and has filled the reservoir in time frame of 4 hours. The estimated PMF (probably maximum flood) is 834 m³/s using Log Pearson method. The existing spillway is capable to pass a flood of 85 m³/s only. The newly designed spillway will have a capacity to pass PMF flood (with water column height of 2.5 m over the crest) without overflowing the dam section. The existing stonemasonry dam will be raised by 2.5 m making total height of 36.5 m. Sultan dam has 200 m long stone masonry saddle-dam, and 162 m long section of this saddle will be constructed using locally available Earthfill material, while the remaining part will be constructed with Rock Fill Concrete (RFC). The structural height of the new Earthfill saddle is 19 m. This new embankment saddle will have zones of clay core, filter, and shoulders. Filter zone consists of non-plastic clean river sand and gravel with 5% fines (0.075 mm), D₁₅ of 0.5 mm, and D₁₀₀ = 75 mm. Shoulders are composed of random fill (*with high contents of gravel*) obtained from the nearby borrow area-D. The shoulder material will a *friction angle* greater than 40°.

Modeled seepages is in range of 0.000008 m³/s/m², and total seepages from the 162 m long and 9 m deep section is approximately 12 liters/s, while the hand mad calculation based on Flow-Net diagram shows 9.3 liters/s at a time when reservoir is full. Drainage wells with 12 cm diameter filled with filter sand is provided in each 5 meters in the toe of the dam to prevent building of excessive pressure at the saddle dam foundation.

Slope stability F. S=3.52 using *Morgenstern Price* method, which is the most accurate method among all others. The FS is high but with passage of time the slope might lose its strength. The



shear strength parameters such as $C = 0.53 \text{ kg/cm}^2$, and $\Phi = 16.9^\circ$ values are taken from geotechnical, and borrow area reports.

Site Investigation:

Seven boreholes with depths of 20-25 m were drilled in Saddle dam site, and borehole#7 which is drilled at failed section, shows 5 m thick weathered rock formation from 15-20 m depth below the ground surface beside clayey-gravel layers. The pictures, and other details of all these layers are given this report.

Finite Element (FE) Modeling:

The GoeStudio 2023 FE modeling shows that saddle will go through a settlement of 55 cm on its first full filling and is the worst case scenario. So, it is strongly suggested that dam should not be filled up to an elevation of 2,383 m in a single filling, rather it should be filled in several attempts so that the saddle could have enough time to adjust its self and thus prevent risk of excessive settlement or being overtopped.

Similarly, an earthquake of 0.3g is used to model the response of the embankment dam, and a 5 cm additional settlement is observed with possible cracks in the crest and upstream side. Some liquification is also observed at toe and heel of the saddle as shown in the report here.

Why Earthfill Saddle?

A 19 m high earthfall embankment dam is proposed as saddle dam due to the following reasons.

- i. Clayey material is abundant, and is right at the saddle dam site
- ii. The foundation contains 15-17 m deep alluvial soils, which is appropriate for putting earth-fill saddle
- iii. A large amount of excavated soil could be reused in the construction of Saddle.
- iv. Clayey material is stronger to dump earthquake high frequencies
- v. Central diaphragm made of RCC concrete proposed by our senior dam engineer is not used due to reference studies disapproval, and failure cases elsewhere in the world, as well as lack of bedrock at the site.



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2.1. International statistics on dams

The below graph is taken from the ICOLD website, showing the highest number is of the Earthfill dams constructed internationally, and it convey the message that they are not new in the arena, and have gained an excellent reputation. Similarly, the needs are also classified, and dams constructed for irrigation supply is leading the statistics.

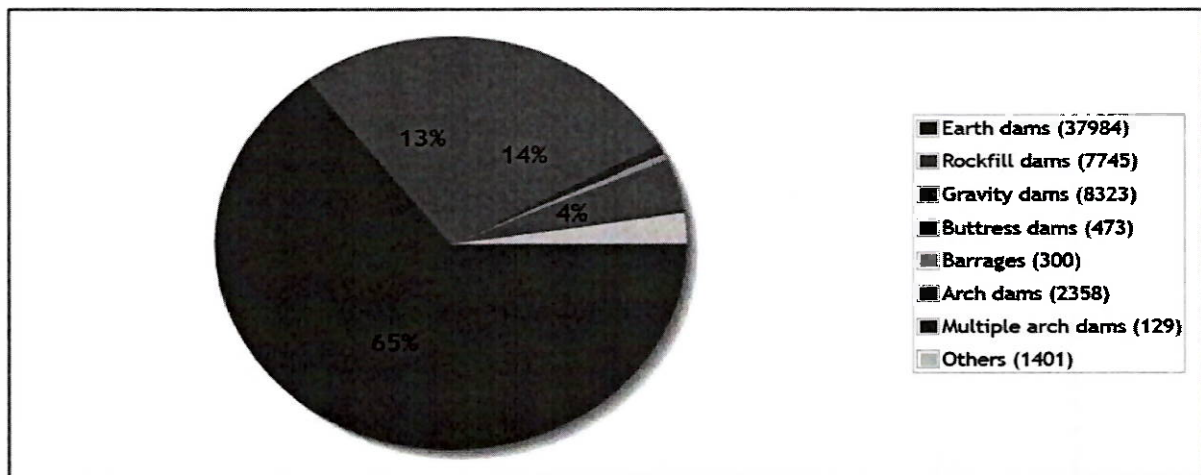


Figure-01: Various types of dams constructed worldwide

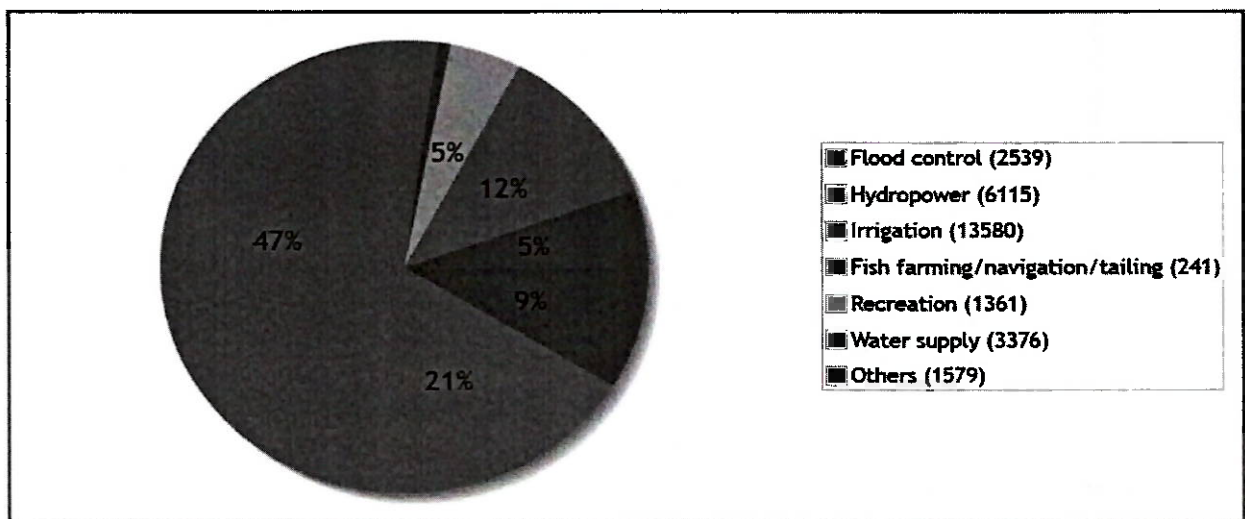


Figure-02: Dams and their intended purposes. Irrigation is the leader.



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3. Introduction:

Sultan Dam is located at 33.7566, 68.38116, i.e. slightly below the confluence of two rivers of Barikab, and Sarab. The distance between the capital city and the dam site is approximately 25 km. Its catchment is approx. 1,200 km². This is an irrigation dam which collect water from floods, and snowmelt for irrigating the agricultural command area at the downstream. The dam height is 34m, and the dam base width is 37m, which will be raised by another 2.5 m making a new height of 36.5 m. The dam will also have 0.8 m high parapet wall. The water storage capacity is greatly reduced to 2 M m³ because the reservoir cannot be filed to its normal level (please see Table-1 for full detail). Currently, the water level in the reservoir is kept below the crest by 10-12 m, because there is a clear risk of saddle failure at some sections e.g. a section close to sofa have large seepages whenever the reservoir level is raised- thus proving a clear risk.

3.1.Existing dam conditions:

Main dam is constructed with *mortar stone masonry*. It is provided with 1 m wide gallery for inspection purposes. On the day of our visit (24,25-July-2023), we went inside the gallery, and observed a 50 cm deep precipitated lime from the dam body. Overall, the main dam looked stable with no apparent settlements, no crakes, and no seepages. Objective amount (3-4 liters/s) of seepages were seen coming down from the roof of the bottom outlet gate hall/house due to the pipe casing breakup within bottom outlet, which needs urgent repair during this coming forth rehabilitation contract.

A large amount of leakages were reported from saddle part near the so called Sofa when the water level is raised in the reservoir.

3.2. The Fear with downstream people:

During the rainfall events the dam supervisor is constantly called from both upstream and downstream people to know the reservoir situation. The upstream people inform the supervisor about the floods probable size coming to the dam site, and that supervisor has to collect people to open the two bottom outlet gates to empty the dam, while the downstream people call the supervisor to know if there is any risk of failure of the dam because of the rainfall event.



During our site visit to the dam site, there was rainfall event, and flood started filling the reservoir. The dam supervisor was constantly called by locals to get the latest information about the flood, and water level in the reservoir, and to make sure that the dam is safe. So, there is a constant fear in the minds of the locals due its previous incident. The dam maintenance and operation crew have to be present all the time at the dam site to open the two bottom outlet gates before arrival of the flood into the dam.

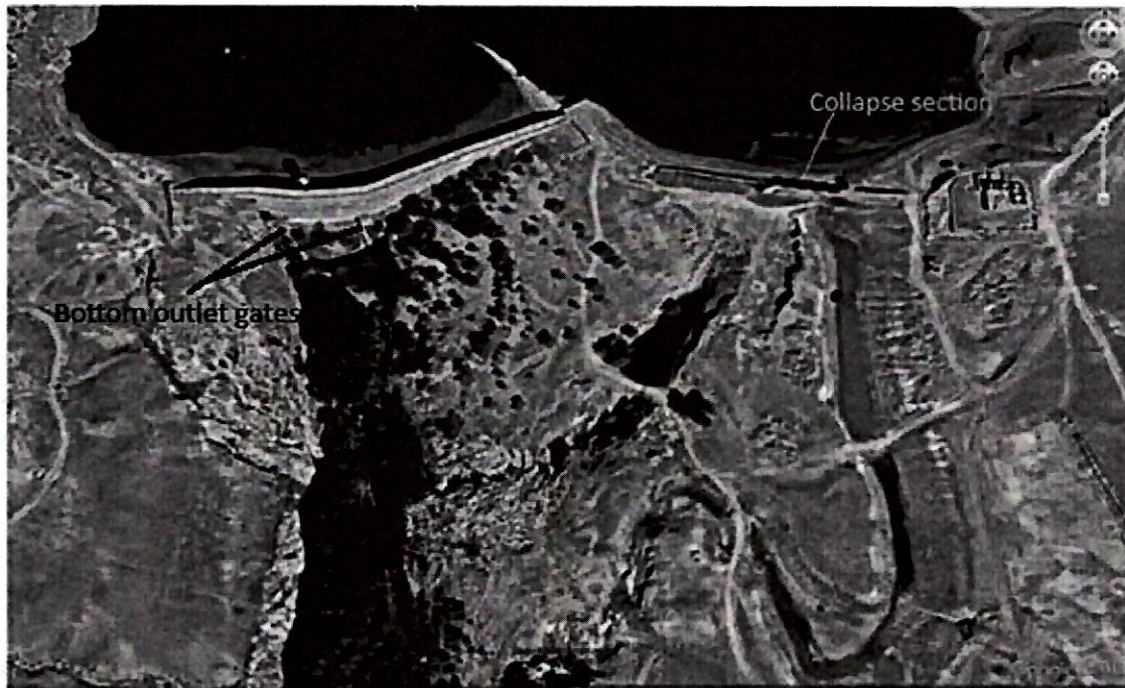


Figure 03: Sultan main dam (on the left) its 200 m long saddle with a long-eroded ditch, and collapsed section of 55 m (Image taken Sep-2022). Only 162 m long section of this will be constructed of Earthfill.



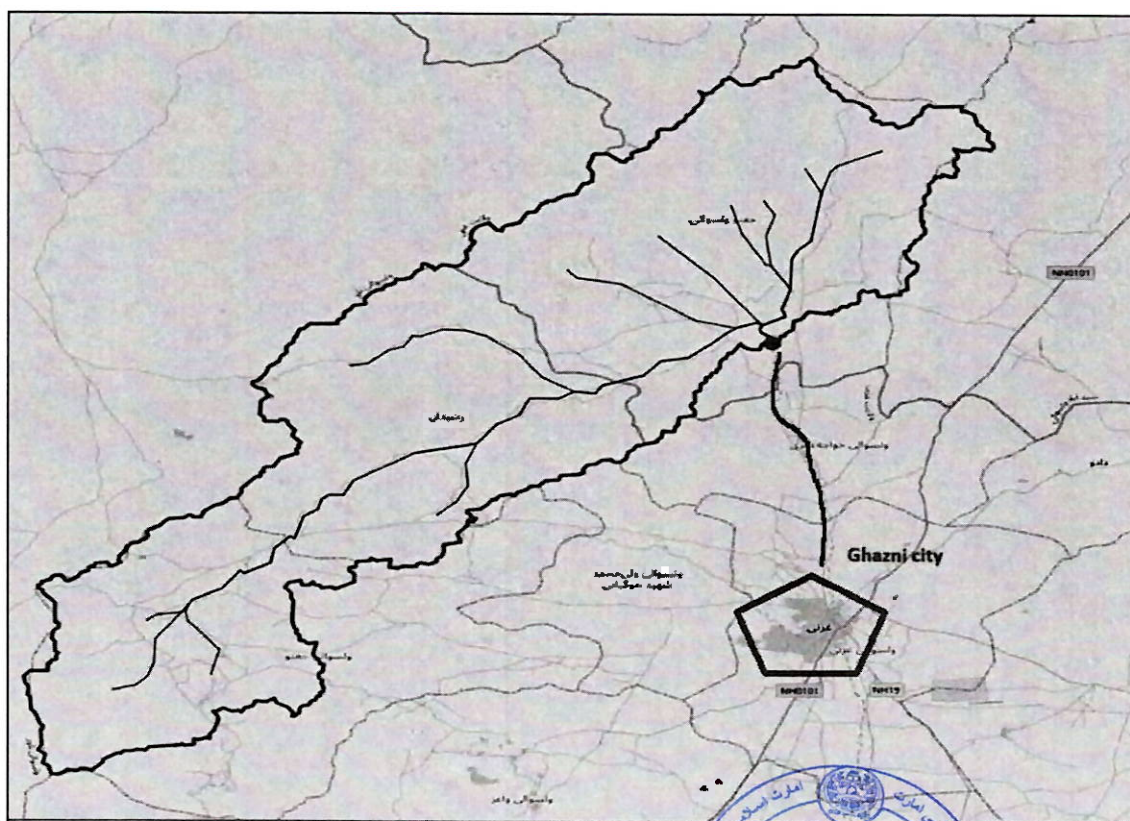


Figure-04: Band-e-Sultan 1,200 km² Catchment, and provincial capital city of Ghazni

3.3. Failure Modes

As shown below in Figure-5, overtopping, and piping failures are the most common types for the embankment dams. This is worth mentioning that the 55 m long section of the saddle failed in 2005 due to piping followed by collapse, and erosion of the section. Poor supervision, and thus O&M is were among the fundamental factors for the failure.

In case of overtopping, the failure starts at toe – shown below by red cricle, where we have provided 2 m thick gravel and cobbles layer at the toe of the new saddle. In fact, this 162 m long Earthfill saddle will never be over topped because the 240 m long section of the main dam will over top much before the Earthfill saddle.

For protection of piping failure, we have provided sufficient size of filter zone, along with drainage wells in each 5 m in the toe of saddle as shown in the general section/ drawing, that will intercept the seepages, and that could be measured and observed at the downstream with safe disposing it off.

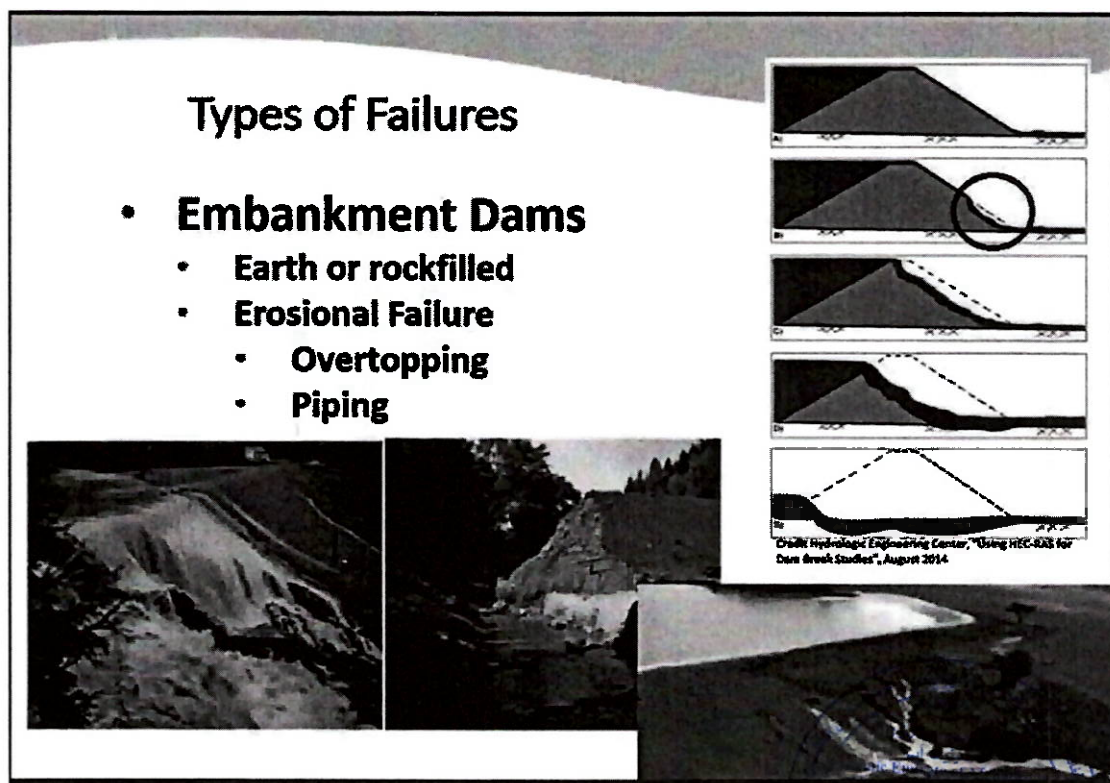


Figure-05: Embankment dam failure modes (overtopping, and piping are the most important ones)



4. The Earthfill Saddle Dam

A 162 m long and 19 m high earth-fill mechanically compacted saddle dam with an earth core, 2 m thick filter zone is evaluated using geotechnical data, and FE model. The factor of stability (FS) against upstream slope failure is 3.56. Similarly, during a highest earthquake the settlement is 5 cm. Overall settlement is estimated to be 55 -60 cm on first filling of the reservoir. A 0.5-0.6 m camber is provided at the top of the Saddle to accommodate the settlement.

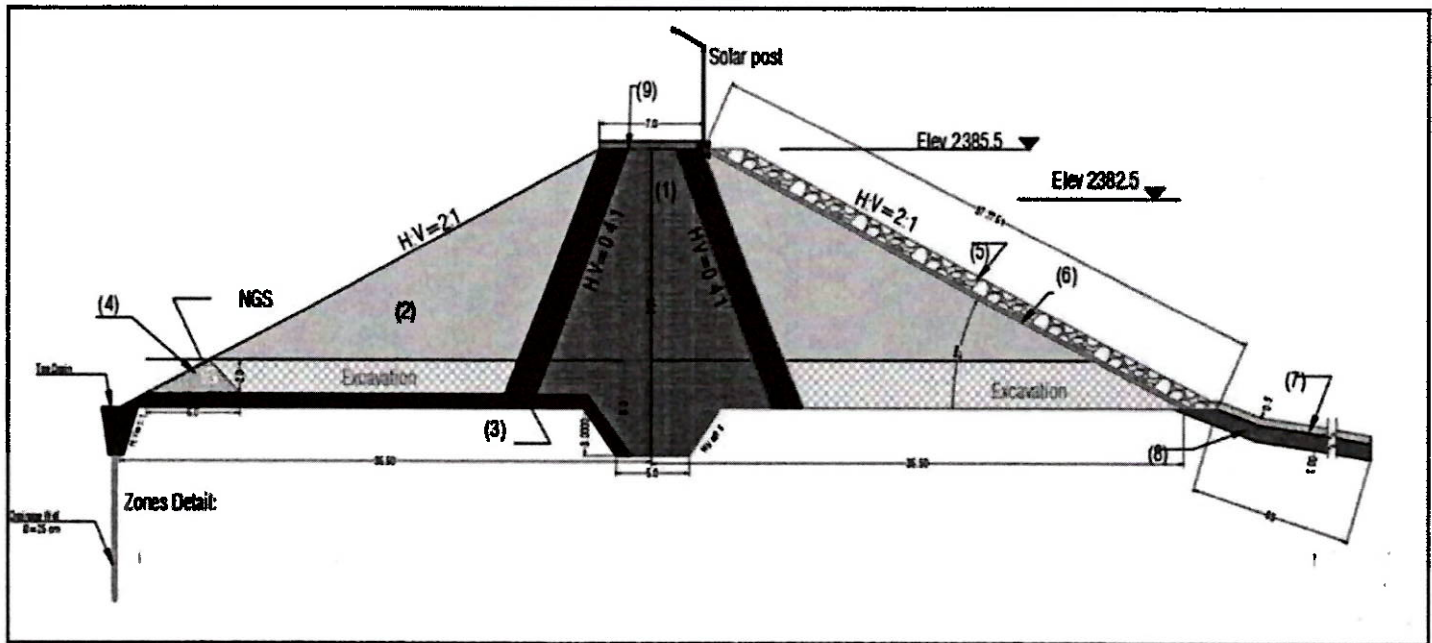
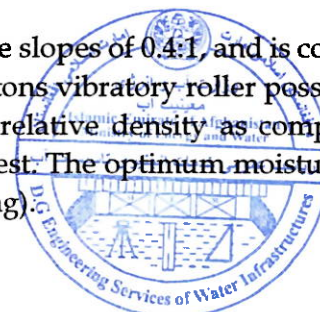


Figure-06: Earthfill saddle general cross section, zones, and dimensions.

The crest width of the saddle is 7 m wide, and it would help if the dam will be raised further in future. Side slopes are 2:1 (H: V). The top width of the clay core is 3 meters, and the side slopes of the core are 0.4: 1 (H: V). The foundation of the clay core is placed 6 m deep from the natural ground surface to block the seepages in the sandy-gravel layers. Other details can be seen at drawing report produced as separate document.

Zone-1: Clay core made of clayey material with side slopes of 0.4:1, and is compacted in layers of 15-20 cm. Compaction should be carried by 12-15 tons vibratory roller possibly two rollers with average speed of 3 km/hr to achieve 95%- 98% relative density as compared to the density achieved in laboratory through modified proctor test. The optimum moisture content should be in the range of 7-10% (confirmed through lab testing).



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Zone-2: Random fill/shoulders consisting of alluvial, sand, and higher contents of gravels compacted in layers of 15-20 cm with 12-15 tons, and optimum moisture content to give highest dry density. The friction angle of the random fill should be more than 40°. Compaction is the same as in Zone-1, to achieve relative density of 95-98%.

Zone-3: Well, graded clean sand and gravel to provide smooth drainage, with fines $\leq 5\%$, Max $D_{15}=0.5\text{mm}$, Max $D_{100} = 75\text{mm}$. The upstream and downstream filter is 2 m thick with $K \geq 1 \times 10^{-3} \text{ cm/s}$.

Zone-4: Toe sand-gravel-cobbles zone to provide smooth drainage, and bearing support and is 6 m wide, and two meter high. This size is based on Seep/W analysis to receive phreatic line.

Zone-5: Riprap is consisting of 0.8m^3 - 1m^3 large blasted stones, and is 1+ plus m thick.

Zone-6: Gravel layer of 1 m thick to enhance seepage from u/s face in case of quick drawdown.

Zone-7: Gravel layer of 0.5 m thick to protect underneath clayey blanket.

Zone-8: Upstream 1m thick clayey blanket to prevent seepages.

Zone-9: 0.5 -0.6 m thick gravel layer to protect underlying core material near the crest, and prevent quick erosion if overtopped.

The upstream & downstream faces of the dam are making 27°, while the friction angle of the compacted random fill is more than 35°, and hence the slope proves to be safe against any sliding.

I have checked the slope stability through Finite Model (FEM), as well as settlement, and the saddle performance is satisfactory.

4.1.Properties and Characteristics of the different zones

1. **Clay Core:** Material type: Clay, Color: Reddish, Size: $<0.002 \text{ mm}$ with 25%-30% percentage, overall, percentage of Fines (that passes #200 mesh or 0.075 mm): $> 45\text{-}50\%$, Plasticity Index ≥ 10 , Grain size distribution: = 0.002 mm to 70 mm (Various Sieve sizes are given in the below table), Hydraulic conductivity $< 10^{-8} \text{ m/s}$, Source Borrow Area A or D (and it needs to be confirmed in the field as borrow area-A is within the reservoir). Borrow Area-A, Borehole-11 i.e. from **1m-10m** depth have fines contents in a range of 60-70% with clay contents of 40% ($<0.005\text{mm}$). With presence of sand -gravel particles, when it is compacted by modern roller it will give us higher friction angle (Φ), which will increase FS against slope failure.



Sieve identification	Opening sizes (mm)
3 in	76
#4	4.75
#8	2.36
#10	2.0
#16	1.18
#20	0.85
#30	0.6
#40	0.425
#50	0.3
#200	0.075

2. **Random Fill (Shell material)** : Material type: alluvial with high clayey contents, Color: Gray to Reddish, Size: with Percentage of Fines (0.075 mm) > 25%, Plasticity Index ≥ 7 , Grain size distribution: = 0.002 mm to 100 mm, frictional angle $> 40^\circ$, Hydraulic conductivity $> 10^{-5}$ m/s, source : Borrow area D.

3. **Filter Zone:** Material type: a mix of Sand+Gravel, Color: Gray/white, Layer_thickness=2m (on the upstream and downstream side of the clay core, while 1.0 m thickness at the foundation), Size: <75 mm, Percentage of Fines: = 5-8%, Grain size distribution: = 0.005 mm to 75 mm (please see filter design section for more detail), Hydraulic conductivity $< 10^{-3}$ m/s. Source = Sorted Riverbed material (located at various locations both on the upstream and downstream of the dam, as shown below). The author of this document has visited these sites. The distance between saddle dam and source is 1.5 km, and 5 km. Filter material on the upstream side is slightly coarser than downstream to help in quick drawdown.



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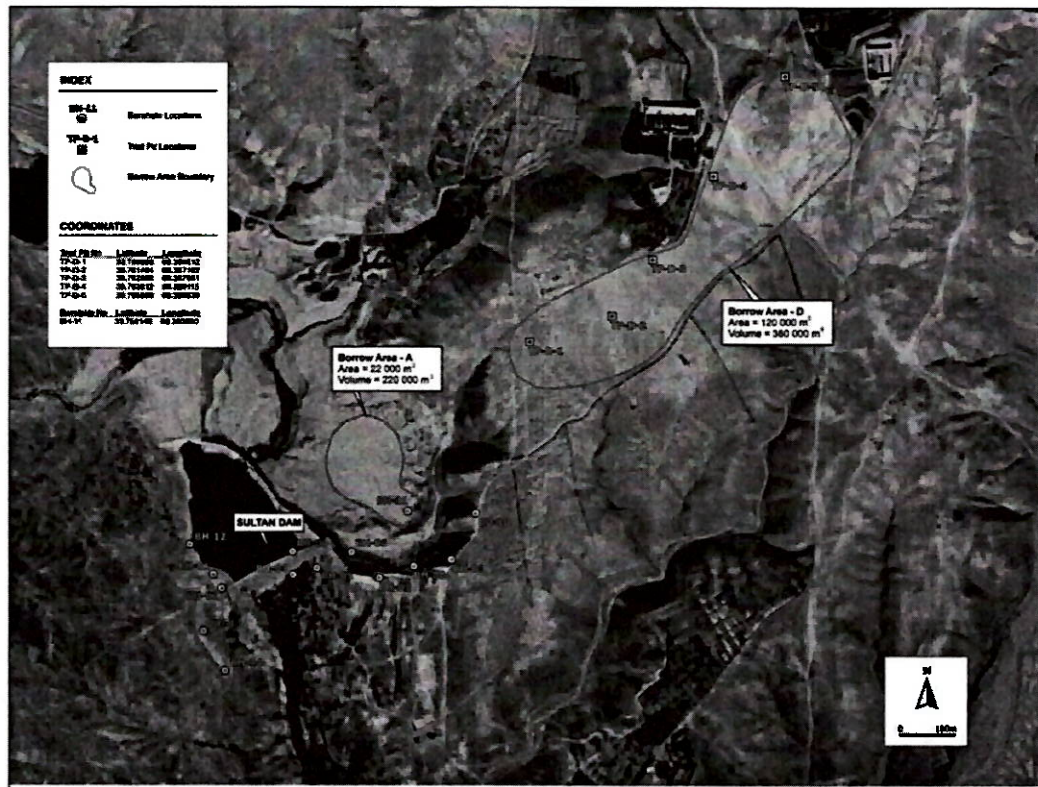


Figure-07: Borrow areas designated for (a) Clay Core – Borrow area A, and (b) Shell material – Borrow area D. Borrow Area -A as shown above has clayey material from 1-10 m depth.



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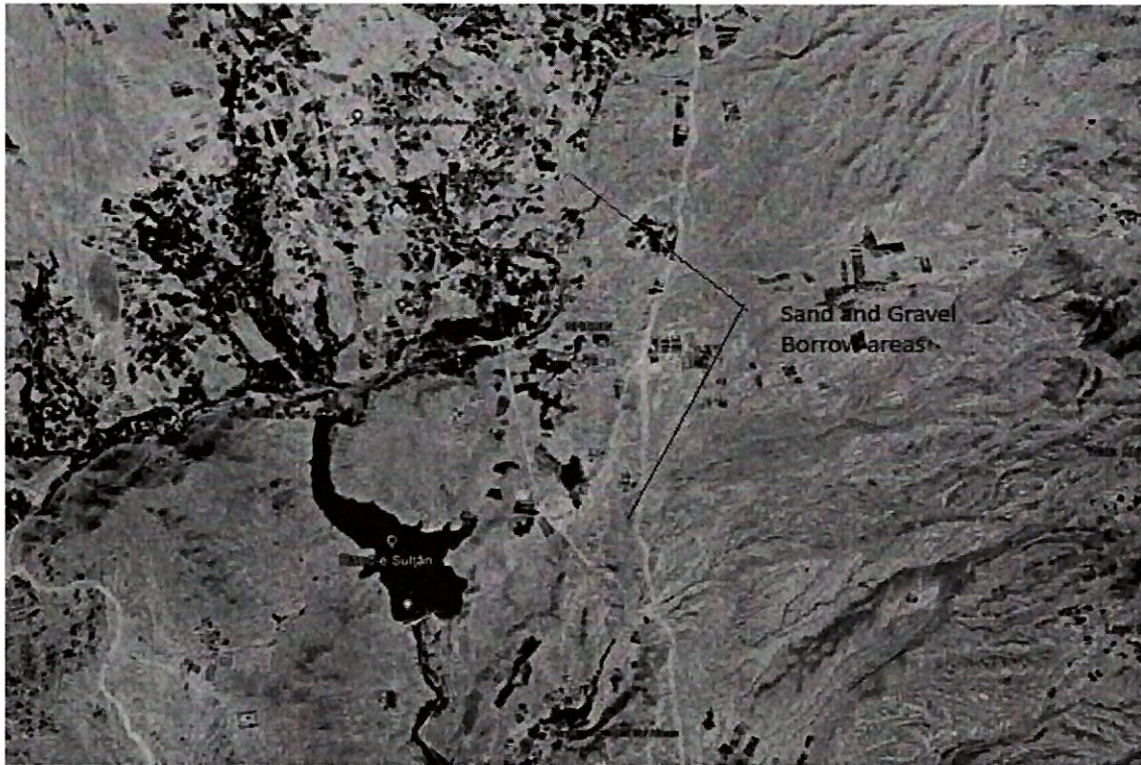
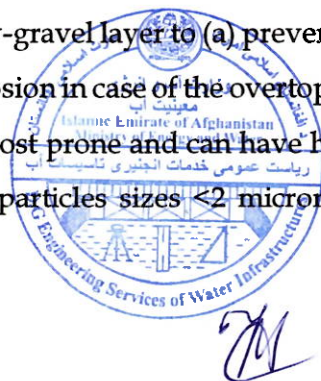


Figure-08: Sand and Gravel borrow areas for filter and transition zones.

Note:

1. Compressibility of the Random fill, and Clay Core, and other zones material should be tested to understand the compactibility through consolidation test.
2. Clay core and Random fill should be compacted in 15 -25 cm layers with *predetermined optimum moisture* content that give us maximum dry density.
3. Core material, filter, and random fill should be processed properly – not allowing oversize stones, or patches of sand or other inappropriate material should be either removed or should be properly mixed to eliminate homogeneity.
4. Small patches of embankment zones e.g., near drain pipes should be compacted with Hand Held small vibratory compaction equipment as shown below.
5. Crest should be covered with at least 50-60 cm sandy-gravel layer to (a) prevent frost front reaching to the clay core, and (b) to prevent quick erosion in case of the overtopping. Sand-Gravel layer are frost resistant, while Silty-clay is frost prone and can have heave rate of 4-5 mm/day. All soils having more than 40% of particles sizes <2 micron are highly susceptible to freeze-thaw.



4.2.Filter Design:

For dry dams which are constructed for flood storage or where the draw down takes more than 10 days, filter is not need. In case of Sultan Dam, a draw down is quicker and it may take a day or two (for draw down) in the summer when irrigation demand is high, so filter is needed.

Filter zone on the upstream of the core is provided to prevent slope failure during the rapid drawdown. Similarly, perforated pipes of 10 inches will be used at the toe drain to dispose of seepages in safe manner. Perforation in the collector pipes must also be less or equal to 0.5 mm to prevent movement of the adjacent filter materials into the perforation.

Filter material consists of sand-gravel with 5% fines (sizes less than 0.075 mm), and $D_{15}=0.5$ mm. Filters will basically do two functions of (i) **To filter** small particles and this will not let them (clay particles) wash downstream (ii) **Permeability** – to dispose off the seepages safely, and to prevent building of any seepage forces called Excessive Pore Pressure which is the main cause of internal erosion.

- (i) **Filtration:** modern filter consist of sand and gravel sizes should be such that the eroded base material from foundation or core of the saddle, coming from crakes will deposit at the surface of filter. The 5% fines in the sand is actually performing this function.
- (ii) **Permeability:** Filter most important criterion is to collect, and carry water to a safe outlet at a low gradient or without pressure. Sand particles D_{15} will perform this function.

Both, base material from the foundation soils (BH-7, BH-8), and core materials (BH-11) were plotted against the filter material as shown in the Figures-11, and 12, and it shows that piping failure will NOT happen both from foundation soils and clay core as the Filter Envelop is on the right-hand side and thus retaining the finest materials.



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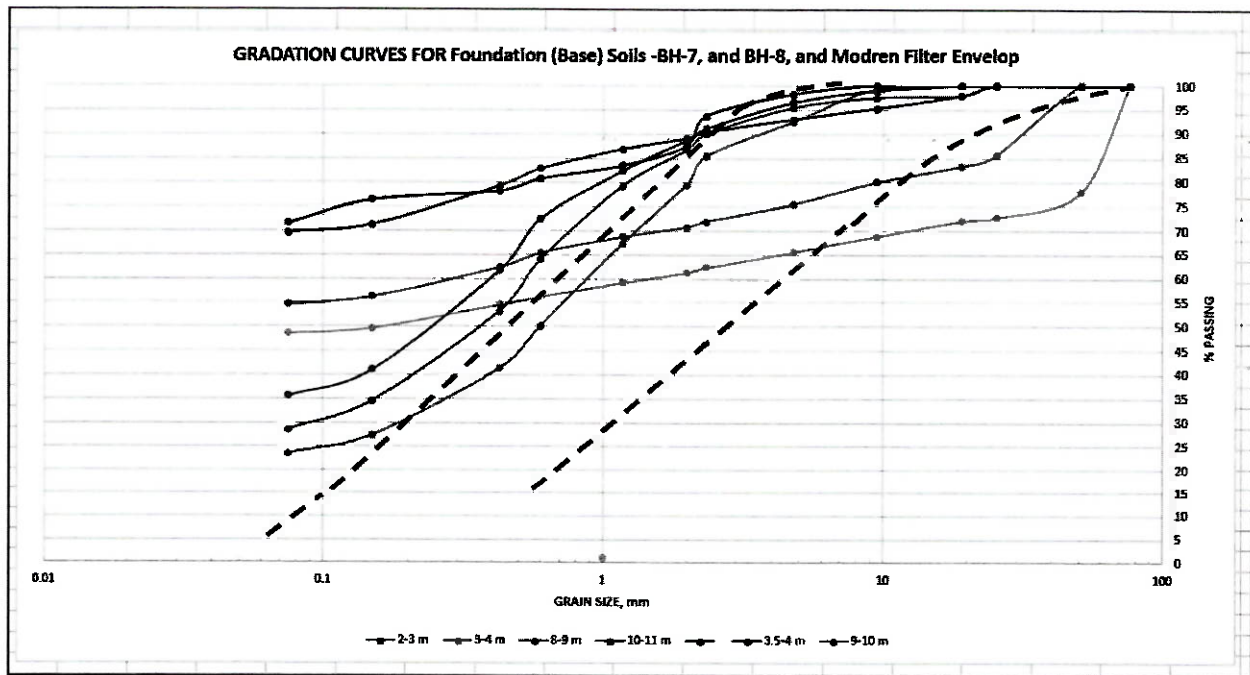


Figure-09: Gradation curves for foundation (Base) soil of BH-7, and BH-8, and Modern Filter material envelop.

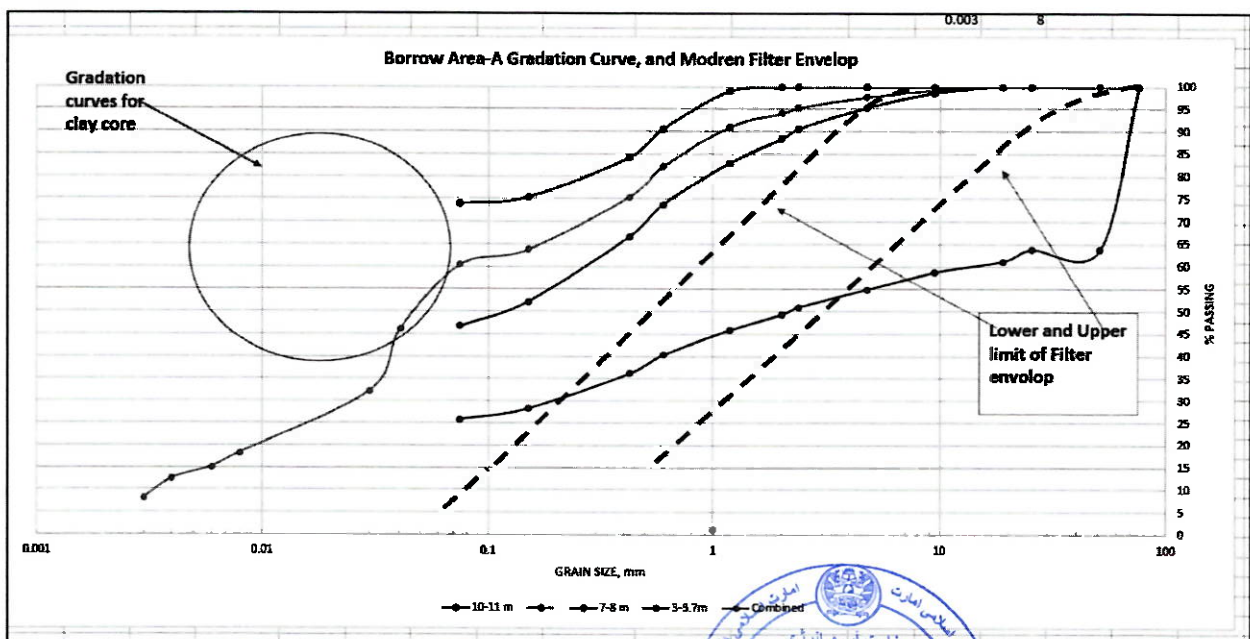


Figure-10: Gradation curves for Clay Core obtained from BH-11, and Modern Filter material.



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Modern Filter Design Criterion:

1. Step-1. Plotted the gradations/ particle sizes both for i. Clay core material from BH-11, and ii. Base material BH-7 & 8.
2. Step-2: Regraded the soil as the core soil has larger particle than #4 sieve
3. Step-3: The borrow area-A, depth 3.0 - 3.7m and 7-8 m, has fines of 62%, and the soil sets in Category-2 (sands, silt, clays, clayey sands).
4. Step-4. Filtering Requirement: $\text{Max } D_{15} \leq 0.7 \text{ mm}$. In our case it is $= 0.7 \text{ mm}$ as there is no risk of dispersive soils, as confirmed by the geotechnical investigation report.
5. Step-5: Permeability Requirement: $\text{Min } D_{15} \geq 5 d_{15} = 5 \times 0.005 = 0.025 \text{ mm}$ which is less than 0.1, hence $= 0.1 \text{ mm}$
6. Step-6: $\text{Max } D_{15} / \text{Min } D_{15} = 0.7 / 0.1 = 7 < \neq 5$, so change the value of $\text{Max } D_{15} = 0.5$, so that the ratio of two is ≤ 5 .
7. Step-7: Calculate $\text{Max } D_{10} = \text{Max } D_{15} / 1.2 = 0.5 / 1.2 = 0.42 \text{ mm}$.
8. Step-8: Calculate $\text{Max } D_{60} = 6 * \text{Max } D_{15} = 6 \times 0.5 = 3 \text{ mm}$.
9. Step-9: Calculate $\text{Min } D_{60} = \text{Max } D_{60} / 5 = 3 / 5 = 0.6 \text{ mm}$
10. Step-10: $\text{Min } D_5 = 0.075 \text{ mm}$
11. Step-11: $\text{Max } D_{100} = 75 \text{ mm}$

Point	Percentage of Filter Material	Size (mm)
1	$D_{15} \text{ max (filter requirement)}$	0.5
2	$D_{15} \text{ min (Permeability Requirement)}$	0.1
3	$D_{60} \text{ max}$	3.0
4	$D_{60} \text{ min}$	0.6
5	$D_5 \text{ min}$	0.075
6	$D_{100} \text{ max}$	75
7	$D_{90} \text{ max}$	20

Table-1: Filter material's various sizes and their percentage by weight.



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4.3.Composition of Filter Material by weight

In the light of above estimation, a 100 kg by weight of the Sultan's Saddle Filter material will contain the following proportions of various sizes sand and gravels.

5 kg = 0.075 mm or smaller size sand.

10 Kg = 0.1-0.5 mm

45 kg = 0.6 mm – 3mm

30 kg = 4 mm -20 mm

10 kg= 21 mm – 75mm.

4.4.Drainage pipe perforation size.

The perforation in the drainage pipes should be less than 0.5 mm as per code. The layout of the 162 m long drainage pipe along with filter sand-gravel is given in the figure-12. Hand-held compactor as shown on the right should be used to compact filter zone, as well as local spots to eliminate looseness.



Figure-11: Small Hand-Held Vibratory Compaction Equipment, the one on the right side is easily available.



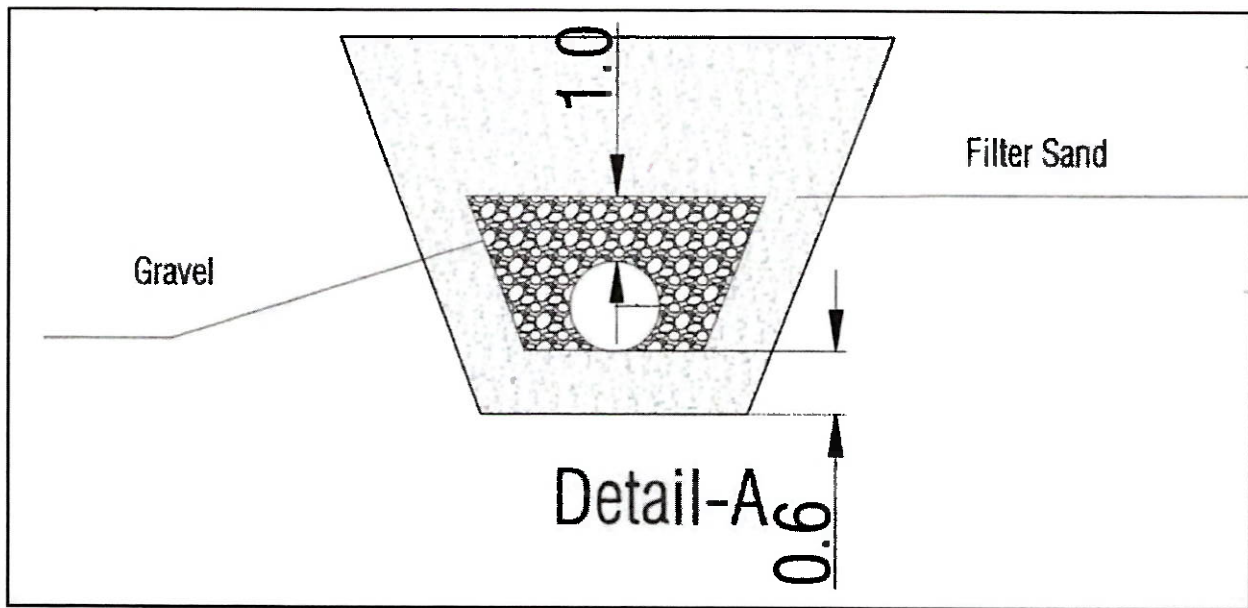


Figure-12: Toe drains along with various dimensions e.g. filter sand, gravel, and side slopes of 1.5: 1.

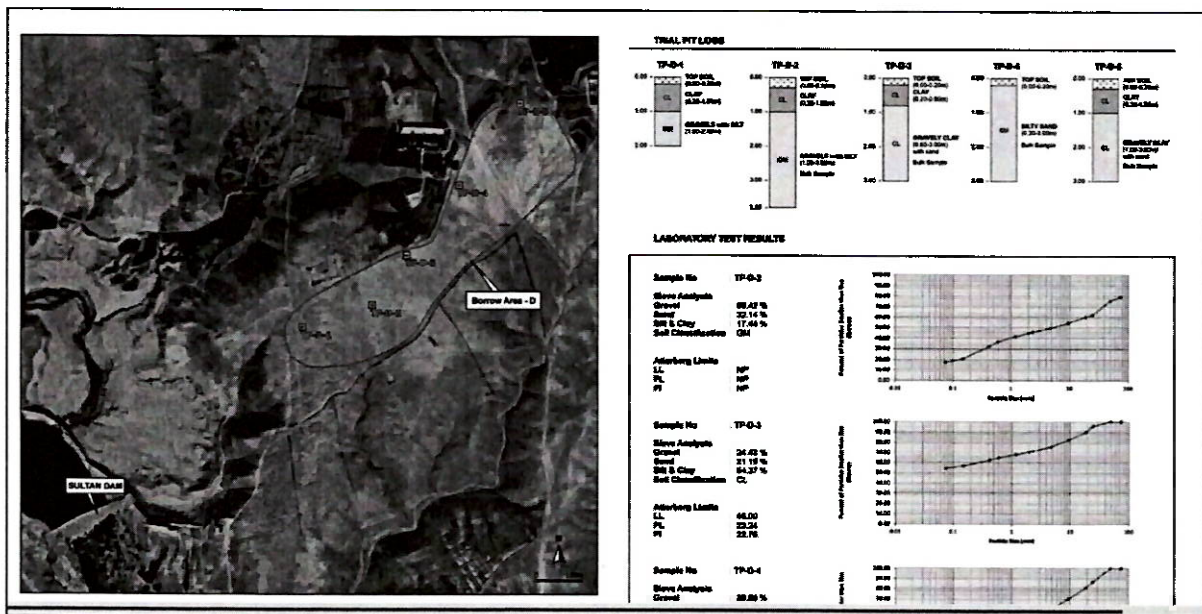


Figure-13: Borrow area- D, with 4 Test pits and their soil types, which is suitable for shoulder of the embankment dam as it has high contents of gravel, which is free draining material, and will have high friction angle.



4.4.1. Filter zone construction and compaction:

Vibratory roller or vibratory plate compactors shown in Figure-09 should be used in compacting clean sand filters. A 5-ton vibratory compactor using thin lifts of 20-25 cm would give good results (Source. *Embankment Dam Filters*, Page 5-128). 2-3 passes of the Roller will obtain 75-80 percent relative density for the sand evaluated, and should not be less than 70-percent relative density using Sand Cone Test in the field. The test frequency is 1 test per 500-1000 m³ compacted sand at any random place. The maximum density of the filter sand can be obtained if it is saturated (wet). The percentage of moisture to give maximum compaction can be obtained from Laboratory test results.

5. Geotechnical characteristics of the site

As shown below in the image, BH-6, BH-7, and BH-8 are right drilled at the Saddle area. The SPT values shows refusal after 17th meter, and it means that there is bedrock.



Figure-14: Bore holes plan at the Sultan Dam site.

To prevent seepages, there is need either to excavate a trench up to 17-meter depth (which is not easy from construction point of view), or to spread a clay blanket on the upstream. Kindly,



see below core pictures for visual understanding of various material at the foundation of saddle dam.

5.1.Core pictures from Borehole-7 (at failed section of the saddle)

Below are the pictures of the cores from 1-25 m deep below the ground surface. This gives us a good idea of why the saddle failed in 2005 floods.

DESIGN OF RECONSTRUCTION AND REHABILITATION WORKS OF SULTAN DAM

BOREHOLE NO : BH-07
CORE BOX NO : 1/5
DEPTH : 0.00-5.00m



DESIGN OF RECONSTRUCTION AND REHABILITATION WORKS OF SULTAN DAM

BOREHOLE NO : BH-07
CORE BOX NO : 2/5
DEPTH : 5.00-10.00m



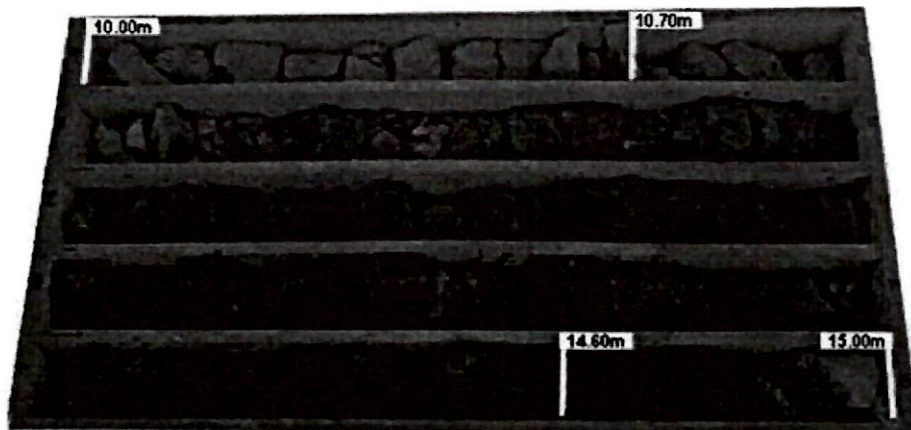
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BH-07

2/3

**DESIGN OF RECONSTRUCTION AND REHABILITATION
WORKS OF SULTAN DAM**

BOREHOLE NO : BH-07
CORE BOX NO : 3/5
DEPTH : 10.00-15.00m



**DESIGN OF RECONSTRUCTION AND REHABILITATION
WORKS OF SULTAN DAM**

BOREHOLE NO : BH-07
CORE BOX NO : 4/5
DEPTH : 15.00-20.00m



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**DESIGN OF RECONSTRUCTION AND REHABILITATION
WORKS OF SULTAN DAM**

BOREHOLE NO : BH-07
CORE BOX NO : 5/5
DEPTH : 20.00-25.00m

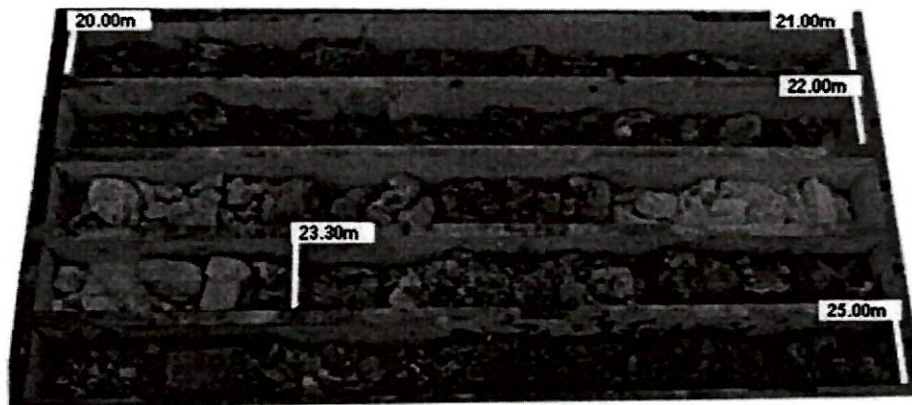
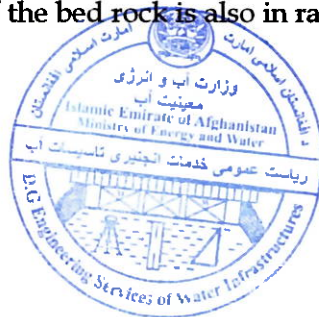


Figure-15: Core samples obtained from BH-07 at the saddle dam.

As shown above in the Core samples, 1-10 m depth is responsible for leakages, and thus failure of the saddle. RQD of the bed rock is also in range of 20-45% which is low and is highly prone to seepages.



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5.2. Boreholes, depth, and soil samples with their engineering properties.

Summary of Laboratory Test Results																				
Geotechnical Investigation for Rehabilitation Sultan Dam																				
National Water Affairs Regulation Authority(NWARA)																				
Gulf Extreme / EKASU																				
Ghazal Province																				
BH No.	Depth(m)	Particle Size Analysis (%) (ASTM D 422)			Atterberg Limits (ASTM D 4318)			Hydrometer Analysis (ASTM D 422)		Soil Classification		Moisture Content (ASTM D 2216) (%)	Specific gravity (ASTM D 854)	One Dimensional Consolidation (ASTM D2436)			Unit weight (ASTM D 7263)		Consolidated Undrained Triaxial Compression	
		% Gravel (>4.75mm)	% Sand (0.075-4.75mm)	% <0.075mm	LL (%)	PL (%)	PI (%)	SS size 0.075-0.005 (%)	Clay size<0.005 (%)	USCS (ASTM D 2487)	AASHTO (M 145)			Cc	Cu	Pc(kPa)	wet Density (g/cc)	Dry Density (g/cc)	C (kN/m ²)	φ (Degree)
6	0-1	15.03	26.31	58.65	35.05	19.01	16.04	33.42	23.23	CL	-	19.74	2.679	-	-	-	-	-	-	
6	1-2	0.00	18.17	81.83	44.37	28.6	16.77	46.54	36.19	CL	-	33.16	2.638	-	-	1.8	1.37	-	-	
6	2.5-3	0.00	22.66	77.34	37.6	24.25	13.35	44.86	32.46	CL	-	32.26	2.709	0.102	0.039	125	-	-	0.53	
6	3-6	0.00	1.6	98.4	36.72	21.66	17.06	81.2	17.2	CL	-	23.94	2.684	-	-	-	-	-	-	
6	7-8	0.00	25.71	74.29	44.8	26.43	18.17	31.2	43.09	CL	-	27.43	2.69	-	-	-	-	-	-	
6	12-13	0.10	10.08	89.82	24.24	15.96	8.28	44.9	44.9	CL	-	23.26	2.694	-	-	-	-	-	-	
7	1-2	0.99	23.38	69.64	34.6	22.79	12.01	48.05	21.59	CL	-	16.25	2.654	-	-	-	-	-	-	
7	2-3	24.56	20.85	54.8	32.4	21.73	10.67	42.04	12.56	CL	-	38.47	2.716	-	-	-	-	-	-	
7	3-4	34.42	16.95	48.64	35.72	18.76	16.94	32.83	15.81	GC	-	18.6	2.667	0.119	0.029	140	2.09	1.85	-	
7	8-9	3.51	60.87	35.82	36.16	20.26	15.89	29.35	6.27	SC	-	17.31	2.665	-	-	-	-	-	-	
7	10-11	4.58	65.85	29.57	28.73	16.23	10.5	21.86	6.71	SC	-	12.21	2.687	-	-	-	-	-	-	
8	3.5-4	1.79	28.63	71.58	29.22	20.37	8.85	-	-	CL	-	22.76	2.704	0.112	0.026	80	1.94	1.58	-	
8	9-10	7.59	68.85	23.56	Non plastic	Non plastic	Non plastic	21.44	2.12	SM	-	4.28	2.662	-	-	-	-	-	-	
8	4-5	11.46	67.56	20.98	Non plastic	Non plastic	Non plastic	14	6.89	SM	-	8.89	2.691	-	-	-	-	-	-	
8	6-9	29.64	49.3	21.06	Non plastic	Non plastic	Non plastic	17.4	3.66	SM	-	19.82	2.698	-	-	-	-	-	-	
9	13-14	32.65	42.18	25.28	29.2	21.74	7.46	16.43	8.85	SC	-	8.1	2.666	-	-	-	-	-	-	
10	3-4	60.87	28.48	10.64	21.04	16.98	4.06	7.86	2.88	GP-GC	-	2.81	2.692	-	-	-	-	-	-	
10	8-9	1.56	51.48	46.98	35	21.32	13.68	26.07	20.81	SC	-	19.69	2.686	-	-	-	1.93	1.68	-	
11	3-3.7	0.00	28.1	73.9	39.5	24.71	14.79	33.62	40.28	CL	-	36.89	2.706	0.024	130	1.86	1.39	-	-	
11	7-8	4.78	48.45	46.76	33.31	23.86	9.43	20.39	26.37	SC	-	34.72	2.676	-	-	-	-	-	-	
11	10-11	45.06	29.14	25.8	25.02	16.07	8.95	16.90	8.90	GC	-	11.95	2.705	-	-	-	-	-	-	

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Table-02: Foundation soil engineering properties

5.3.Sultan-Dam Bed-Rock characteristics:

Geosearch Construction&Engineering Company													Geo Search	
Summary of Laboratory Test Results														
Geotechnical Investigation for Rehabilitation of Sultan Dam														
National Water Affairs Regulation Authority (NWARA)														
Gulf Extreme /EKASU														
Ghazni Province														
BH No.	Depth(m)	Density (g/cc)	Specific Gravity (ASTM D 3148)	Water Absorption by weight (ASTM D 3148)	Triaxial Compressive Strength (ASTM D 3148)		Unconfined Compressive Strength (ASTM D 2938)	Splitting Tensile Strength (ASTM D 3987)		Uniaxial Compressive Strength (ASTM D 7012)(ASTM D 3148)				
					C (Mpa)	Φ (Degree)		UCS (Mpa)	Maximum applied load (kN)	Splitting tensile strength (Mpa)	Uniaxial Compressive Strength (Mpa)	Young Modulus (Gpa)	Poisson Ratio (%)	
BHM03	1.8 - 2.0	2.576	-	-	-	-	44.08	-	-	-	-	-	-	-
	15.4 - 15.55	2.616	-	-	-	-	127.995	-	-	-	-	-	-	-
	23 - 23.2	2.581	-	-	-	-	34.372	-	-	-	-	-	-	-
	45.4 - 45.466	-	-	-	-	-	-	44.28	5.478	-	-	-	-	-
	45.466-45.98	-	-	-	18.3	28.3	-	-	-	-	-	-	-	-
	47 - 47.083	-	-	-	3.5	48.5	-	25.71	3.181	-	-	-	-	-
	9.1 - 9.26	2.63	2.83	0.39	-	-	-	-	-	80.76	-	-	0.14	-
	18 - 18.063	-	-	-	-	-	-	71.5	8.883	-	-	-	-	-
	18.063 - 18.6	-	-	-	15.6	50.4	-	-	-	-	-	-	-	-
	23 - 23.7	-	-	-	32.1	49.9	-	-	-	-	-	-	-	-
BHM04	23.7 - 23.87	2.64	2.84	0.27	-	-	-	-	-	-	80.32	86.75	0.23	-
	34 - 34.9	-	-	-	0.0	58.7	-	-	-	-	-	-	-	-
	35 - 35.059	-	-	-	-	-	-	17.78	2.865	-	-	-	-	-
	45.4 - 45.468	-	-	-	-	-	-	83.17	10.928	-	-	-	-	-
BHM05	9.8 - 9.93	2.57	2.57	0.84	-	-	-	-	-	22.38	40.88	0.44	-	-
BHM12	19.7 - 19.84	2.58	2.58	1.12	-	-	-	-	-	50.53	106.1	0.18	-	-
BHM13	8.2 - 9.35	2.54	2.54	2.42	-	-	-	-	-	23.15	4.42	0.19	-	-

Table-03: Bedrock (15-20 m deep) engineering properties.

6. Modelling of Earthfill embankment dam using GeoStudio V 23.

Handmade calculations, and FE modeling is based on engineering properties and stratification of BH-7, and BH-8 which is right at the place of failure.

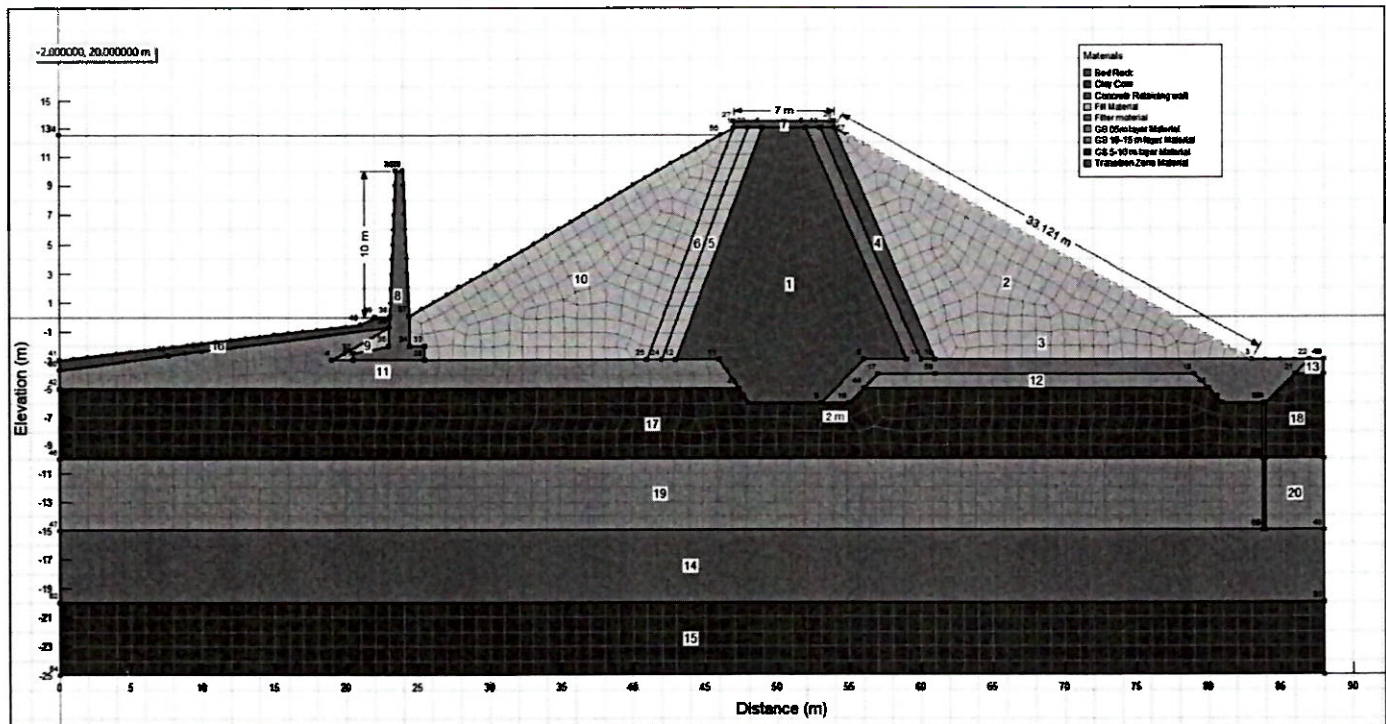


Figure-16: Existing RCC retaining wall type of saddle, new embankment type saddle dam cross-section, and various zones material, foundation soils (taken from Geotechnical report), drainage well, and with upstream clay blanket (1.0 m thick) to prevent seepages

6.1. Seepage analysis:

Lefranc tests conducted on the BH-6, and BH-7 is showing a vertical hydraulic conductivity in range of $K_{\text{vertical}} = 6.0 \times 10^{-4} \text{ cm/s}$, and $K_{\text{Horizontal}} = 3.3 \times 10^{-5} \text{ cm/s}$, and it means that the horizontal permeability is high than the vertical one, and should be because of the presence of over consolidated layers, as well as presence of 5 m thick rock at a depth of 15-20m. I have used $100 \times K_{\text{Horizontal}}$ in the Seep/W, because of the uncertainty in the values measured. For example, at the failed segment of the saddle i.e., at BH-7, and 0-5 m depth $K_{\text{Horizontal}} = 1.7 \times 10^{-4} \text{ cm/s}$, but I have considered it $1.7 \times 10^{-2} \text{ cm/s}$ ($1.7 \times 10^{-3} \text{ m/s}$) in Seep/W. This layer and the



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subsequent layer are the culprit and were responsible for the saddle failure in 2005. With application of upstream clayey blanket maximum seepages are reduced from 0.00026 m³/s/m² to 0.000008 m³/s/m². Drainage wells of 12 cm diameters are also drilled up to a depth of 15 m from ground surface, and are filled with filter-sand.

Permeability of 5 m thick rock layer at a depth of 15-20 m is Not recorded, although in other boreholes the permeability is in range of 6-20 lugeons which is fairly permeable.

Bore Hole number	Depth (m)	K _{vertical} (cm/s)	K _{Horizontal} (cm/s)	Comments
6	5-10	8.0E-04	4.3E-05	Water table is 12.8 m below the ground surface
6	15-19.5	4.5E-04	2.1E-05	
7	0-5	3.25E-03	1.7E-04	1.7 E-02 cm/s is considered in Seep/W
7	10-15	6.0E-04	3.3E-05	
7	20-25	7.0E-05	3.0E-06	

Table-04: Hydraulic conductivity at various depths in BH-6, and BH-7.

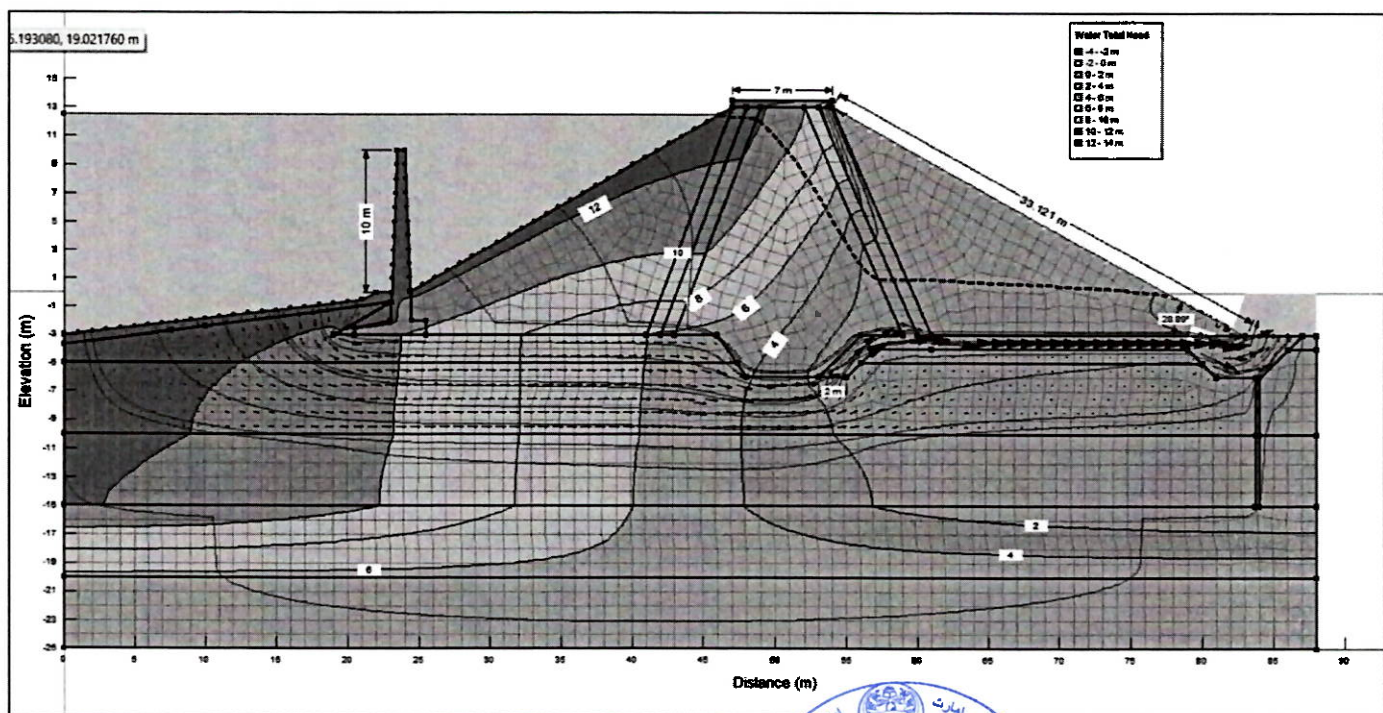
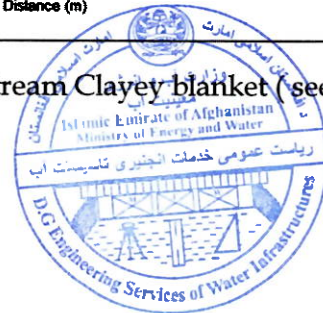


Figure-17: Seepages in the foundation if there is no upstream Clayey blanket (seepages shown in dotted blue arrows)



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Node	Node	X (m)	Y (m)	Z (m)	Water Pressure (kPa)	Water Total Head (m)	Water Pressure Head (m)	Water Rate (m³/sec)	Water Mass Rate (kg/sec)	Water X Flux (m³/sec/m²)	Water Y Flux (m³/sec/m²)	Water Flux (m³/sec/m²)	Water Mass X Flux (kg/sec/m²)	Water Mass Y Flux (kg/sec/m²)	Water Mass Flux (kg/sec/m²)
1530	1,530	50	-15	0	178.8	3.2	18.2	-2.94E-20	-2.94E-17	1.20E-06	2.27E-08	1.20E-06	0.001202455	2.27E-05	0.001202668
1532	1,532	50	-14	0	168.9	3.2	17.2	-6.78E-21	-6.78E-18	2.43E-06	9.11E-08	2.43E-06	0.002427028	9.11E-05	0.002427837
1533	1,533	50	-13	0	158.7	3.2	16.2	4.24E-20	4.24E-17	2.50E-06	1.87E-07	2.51E-06	0.00249855	0.000186766	0.002505521
1534	1,534	50	-12	0	148.3	3.1	15.1	-6.35E-20	-6.35E-17	2.63E-06	2.93E-07	2.65E-06	0.002633447	0.000292854	0.00264968
1535	1,535	50	-11	0	137.7	3.0	14.0	-5.17E-20	-5.17E-17	2.86E-06	4.17E-07	2.89E-06	0.002862694	0.000416706	0.002892863
1536	1,536	50	-10	0	126.7	2.9	12.9	-1.22E-19	-1.22E-16	2.92E-05	1.21E-06	2.93E-05	0.029245602	0.001211473	0.029270683
1537	1,537	50	-9.00278	0	116.7	2.9	11.9	1.79E-18	1.79E-15	5.72E-05	3.54E-06	5.73E-05	0.057223449	0.003538536	0.05730755
1540	1,540	50	-8.00045	0	106.2	2.8	10.8	-1.36E-18	-1.36E-15	6.30E-05	5.92E-06	6.33E-05	0.06284658	0.005916468	0.06326193
1545	1,545	50	-7.00603	0	95.6	2.7	9.7	3.25E-19	3.25E-16	7.63E-05	5.02E-06	7.64E-05	0.076227346	0.005017959	0.076437234
1553	1,553	50	-6	0	85.2	2.7	8.7	8.13E-20	8.13E-17	4.24E-05	1.68E-06	4.24E-05	0.042389411	0.001678353	0.042422624

Table-05: Seepages when there no upstream blanket (please note the Node number for comparison)

Node	Node	X (m)	Y (m)	Z (m)	Water Pressure (kPa)	Water Total Head (m)	Water Pressure Head (m)	Water Rate (m³/sec)	Water Mass Rate (kg/sec)	Water X Flux (m³/sec/m²)	Water Y Flux (m³/sec/m²)	Water Flux (m³/sec/m²)	Water Mass X Flux (kg/sec/m²)	Water Mass Y Flux (kg/sec/m²)	Water Mass Flux (kg/sec/m²)
1529	1,529	50.84603	-15.997	0	158.0	0.1	16.1	1.22E-23	1.22E-20	5.80E-12	5.81E-12	8.21E-12	5.80E-09	5.81E-09	8.21E-09
1530	1,530	50.84603	-15	0	148.1	0.1	15.1	-8.31E-21	-8.31E-18	3.70E-08	6.43E-10	3.70E-08	3.70E-05	6.43E-07	3.70E-05
1532	1,532	50.84845	-14	0	138.3	0.1	14.1	-9.49E-20	-9.49E-17	7.47E-08	2.59E-09	7.47E-08	7.47E-05	2.59E-06	7.47E-05
1533	1,533	50.85087	-13	0	128.5	0.1	13.1	6.10E-20	6.10E-17	7.71E-08	5.94E-09	7.73E-08	7.71E-05	5.94E-06	7.73E-05
1534	1,534	50.85329	-12	0	118.6	0.1	12.1	6.61E-20	6.61E-17	8.16E-08	8.44E-09	8.20E-08	8.16E-05	8.44E-06	8.20E-05
1535	1,535	50.85571	-11	0	108.8	0.1	11.1	9.74E-20	9.74E-17	8.91E-08	1.21E-08	9.00E-08	8.92E-05	1.21E-05	9.00E-05
1536	1,536	50.85813	-10	0	99.0	0.1	10.1	3.74E-19	3.74E-16	9.16E-07	3.66E-08	9.16E-07	0.000915769	3.66E-05	0.000916501
1537	1,537	50.85813	-9.00278	0	89.2	0.1	9.1	-2.17E-19	-2.17E-16	1.79E-06	1.09E-07	1.80E-06	0.00179276	0.000108695	0.001796052
1540	1,540	50.91485	-8.00045	0	79.3	0.1	8.1	1.63E-19	1.63E-16	1.98E-06	1.82E-07	1.98E-06	0.00197541	0.000182492	0.001983821
1545	1,545	50.9642	-7.00603	0	69.6	0.1	7.1	6.10E-19	6.10E-16	2.39E-06	1.54E-07	2.40E-06	0.002394436	0.000154074	0.002399388
1553	1,553	51	-6	0	59.7	0.1	6.1	-9.00E-20	-9.00E-17	1.33E-06	4.93E-08	1.33E-06	0.00133132	4.93E-05	0.00133232

Table-06: Seepages with upstream blanket (please note the Node number 1530, and 1535 for comparison).

Note: with clayey blanket on the upstream side within the reservoir, seepages are reduced by half, and similarly, the amount of seepage forces are reduced by more less 50%.

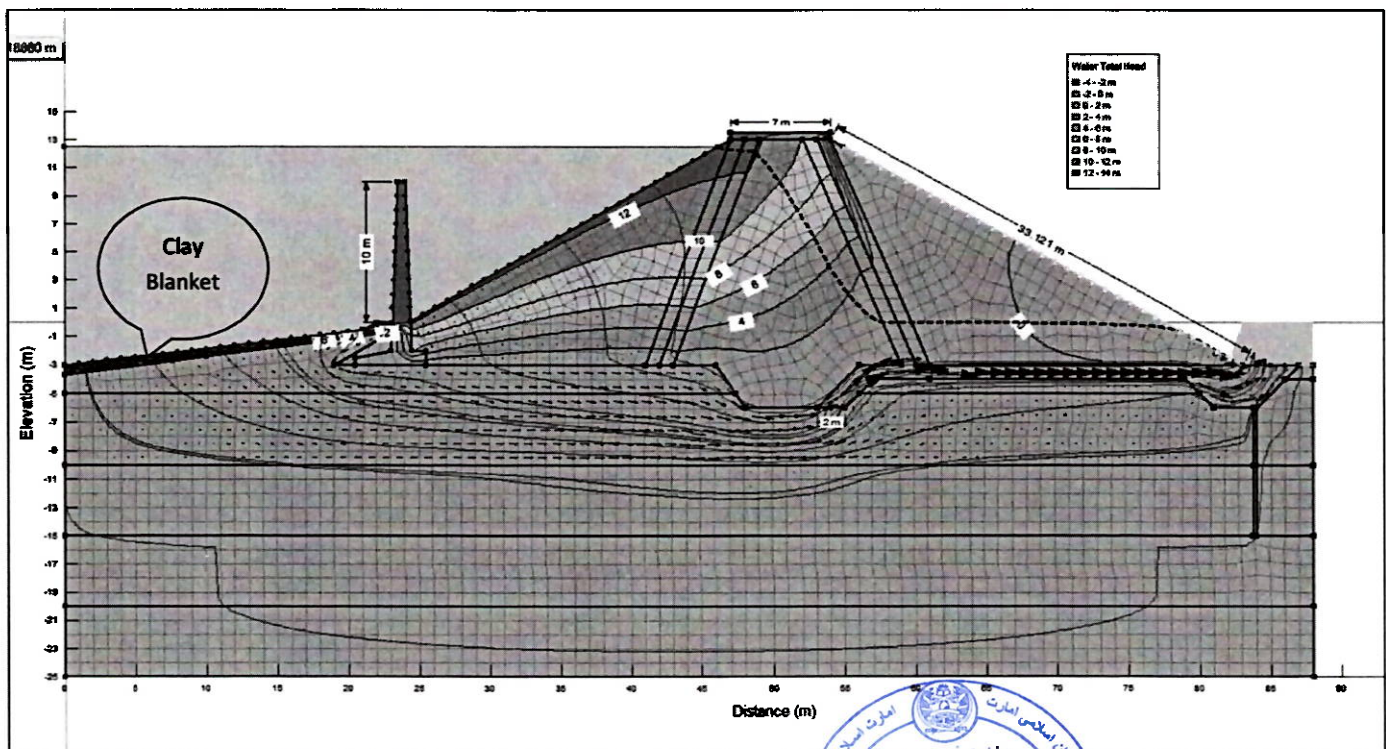


Figure-18: Upper clayey blanket, and seepages are reduced as it can be seen that 12 m Water Head is missing below the clayey blanket.



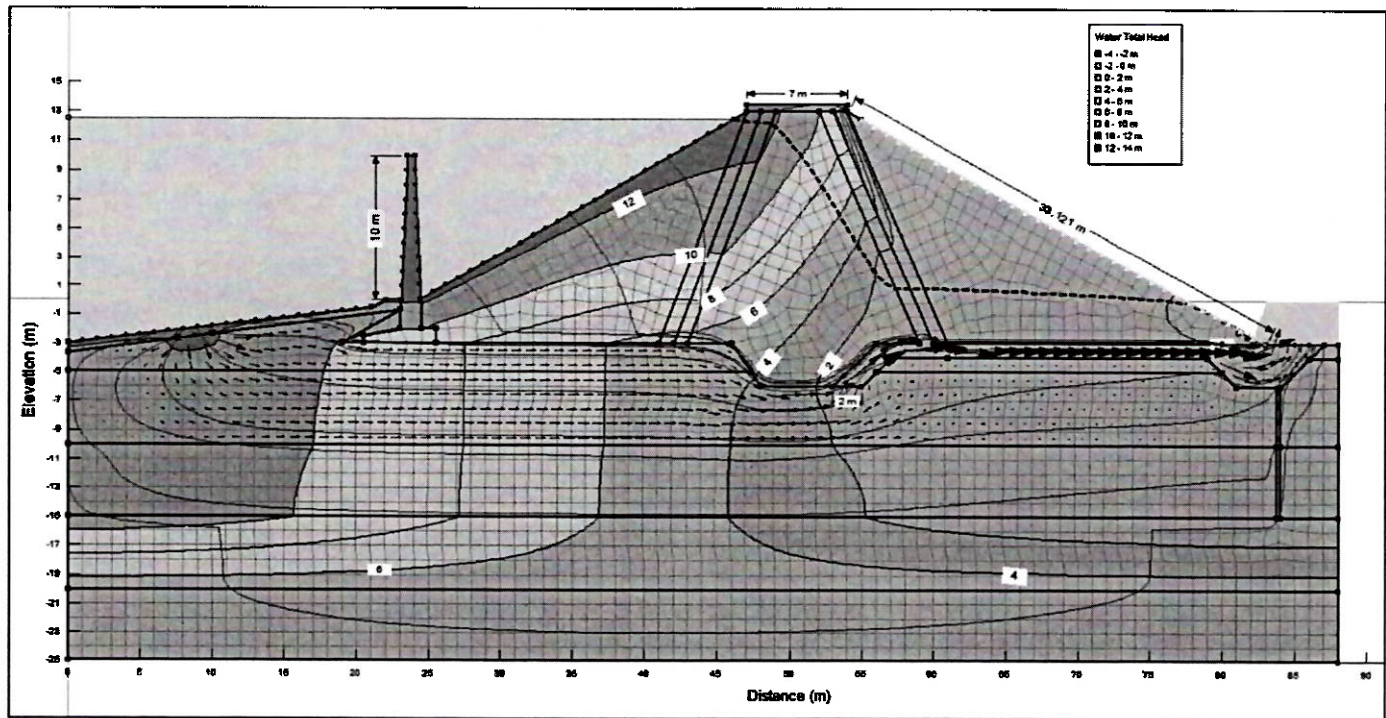
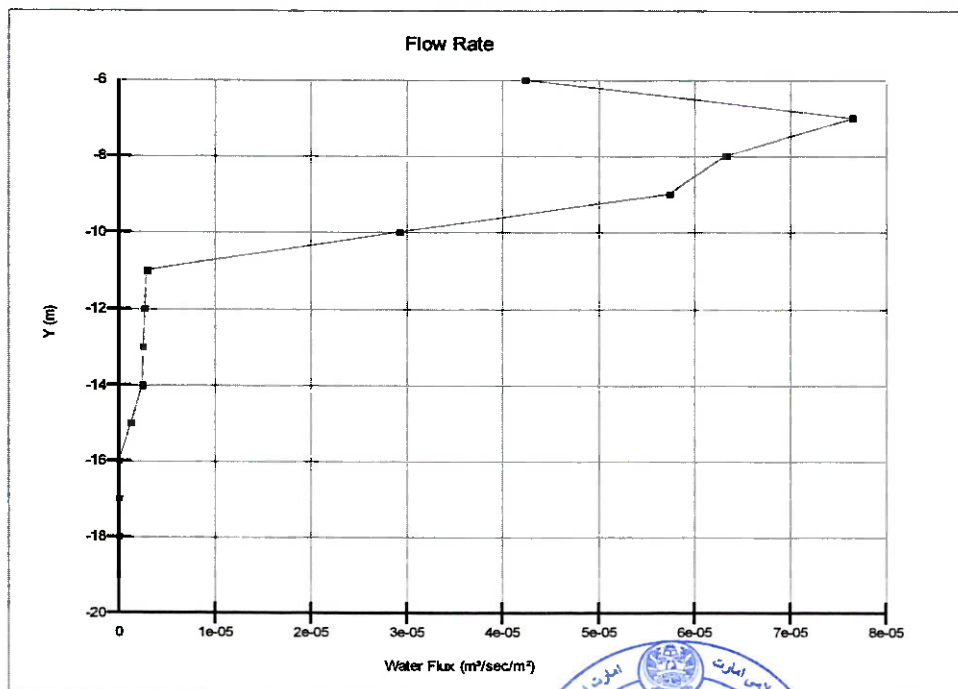


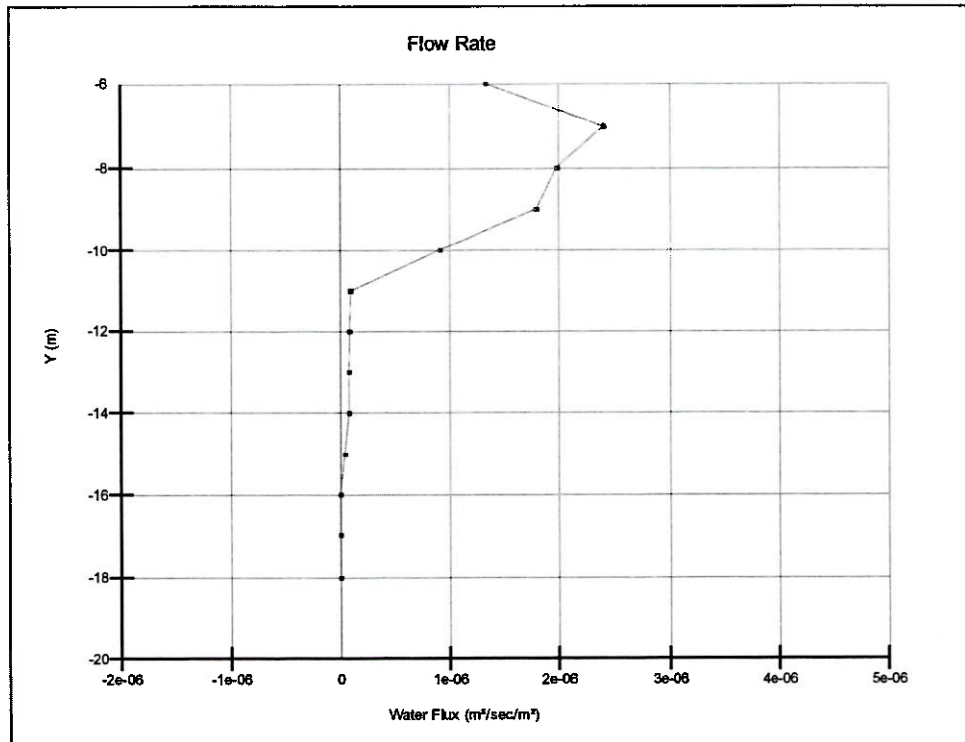
Figure19: Crake in the upper blanket and the seepages increased



Graph-01. Flow rate (m³/sec-m²) when there is no upstream clay-blanket



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Graph-02. Flow rate (m³/sec-m²) when upstream 0.6 m -1.0 m clay-blanket is applied

6.2. Seepages calculation through Flow-Net Diagram (Handmade calculations):

As shown below in the figure, there are 10 equipotential lines (Nd) , and 4 flow lines (Nf). The seepages can be calculated through following equation.

$$Q = \frac{N_f}{N_d} \cdot k \cdot H \cdot L$$

$$Q = \text{cm}^3/\text{s},$$

$$N_f = 10$$

$$N_d = 4$$

$$L \text{ (length of saddle dam)} = 162 \text{ m} = 16,200 \text{ cm}.$$

$$K = 1.7 \text{ E-4 cm/s}, H \text{ (water head)} = 13.5 \text{ m} = 1350 \text{ cm}$$

$$Q = 10/4 \cdot 1.7 \times 10^{-4} \text{ cm/s} \times 1350 \text{ cm} \times 16,200 \text{ cm} = 9,294.75 \text{ cm}^3/\text{s} = 9.3 \text{ liters/s}$$



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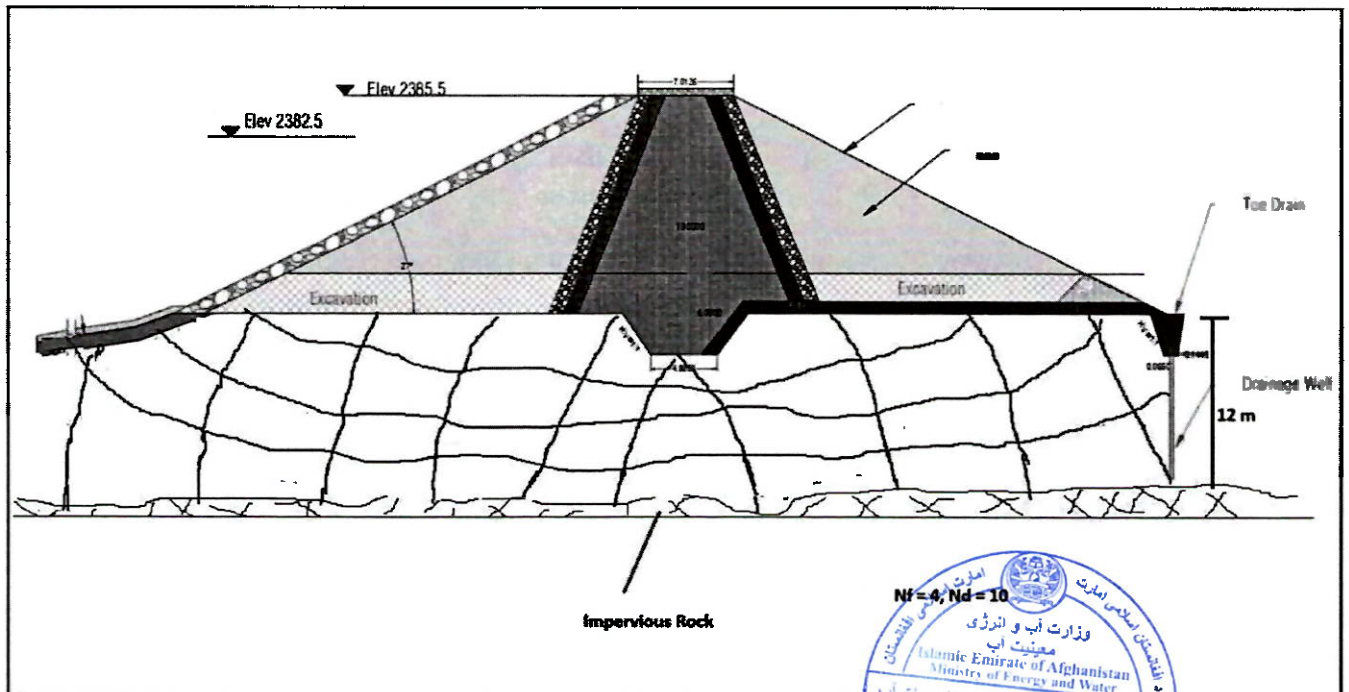
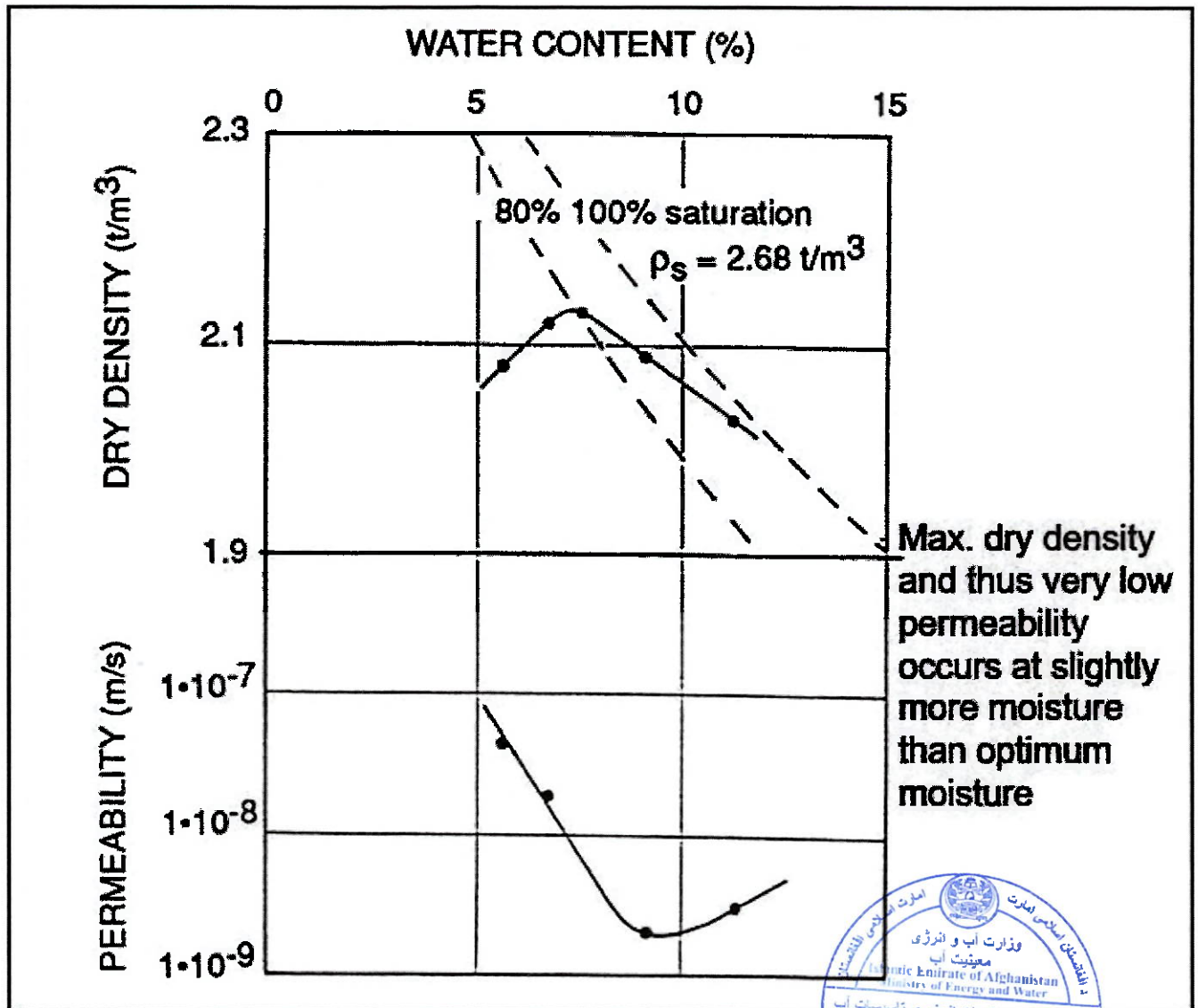


Figure-20: Seepages measurement through Flow net diagram.

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6.3. Permeability Vs Dry Density at optimum moisture content.

As shown below, the highest dry density can be achieved at optimum moisture content of approximately 7%, while very low permeability could be achieved with the same compaction effort (e.g., 10-12 passes of 15-ton vibrating roller) at a moisture little more than optimum moisture content as shown in the below graph.



Graph-03. Clay core Dry density and Permeability vs Moisture content

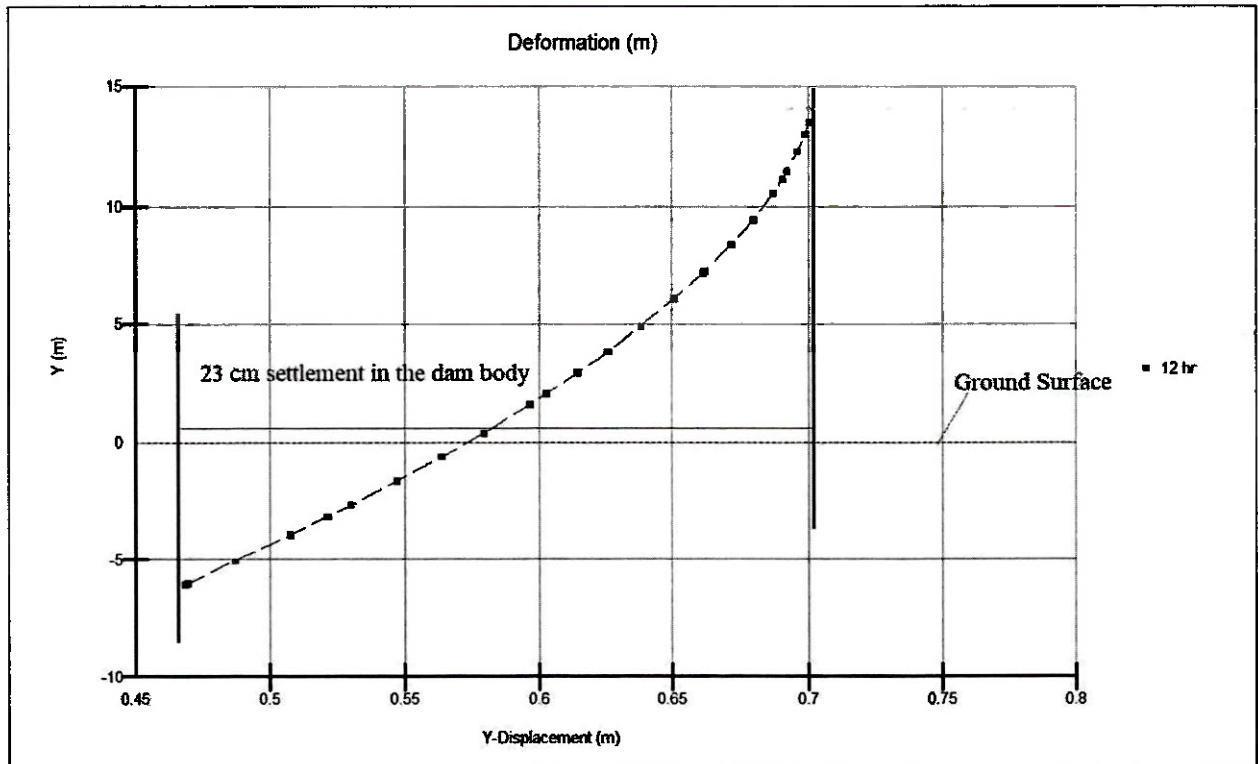


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Slope/W is used to estimate factor of safety. Clay core, Shell material, Filter, and Sub-surface profile data is used from the geotechnical report. Mohr Coulomb material model, and Morgenstern-Price analysis is used to find the most critical slip surface (shown by the white color). The F.S were found to be 3.56, which is very safe, and should be as there a big city in the downstream.



Figure-22: Slope stability of the saddle.



Graph-04: Settlement of 23 cm is recorded in 19 m high embankment (saddle dam) on its first filling. Total settlement is 60 cm, so the rest of settlement will occur in the foundation layers.

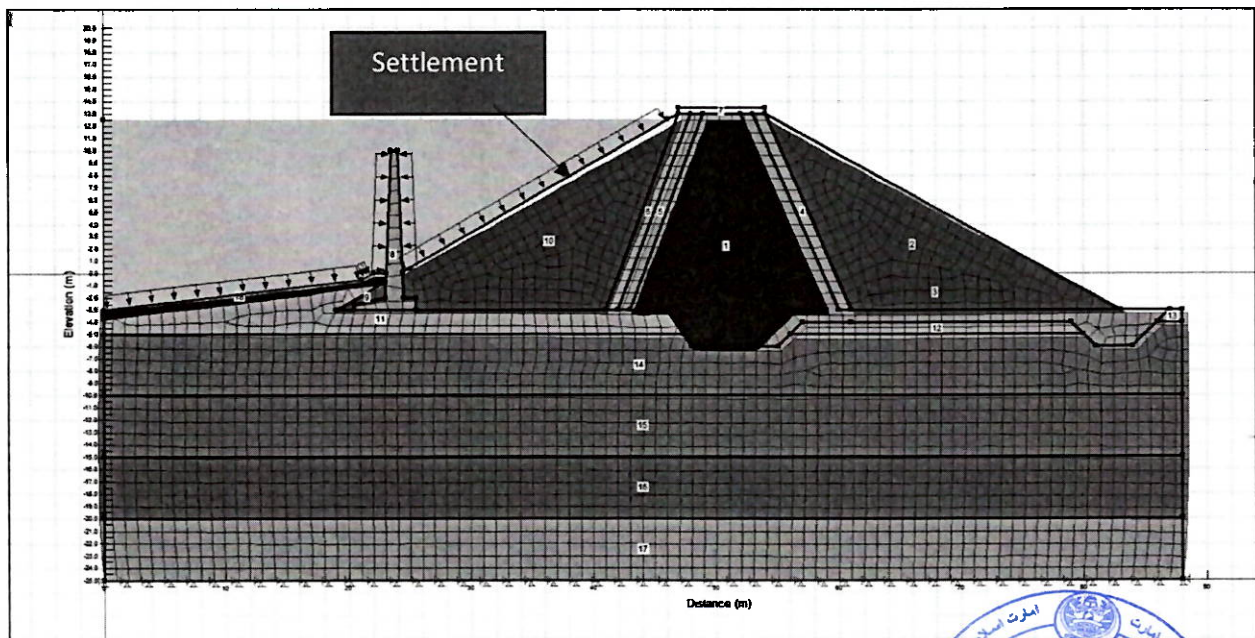


Figure-23: Settlement of the saddle dam shown as white-space at the shoulders.



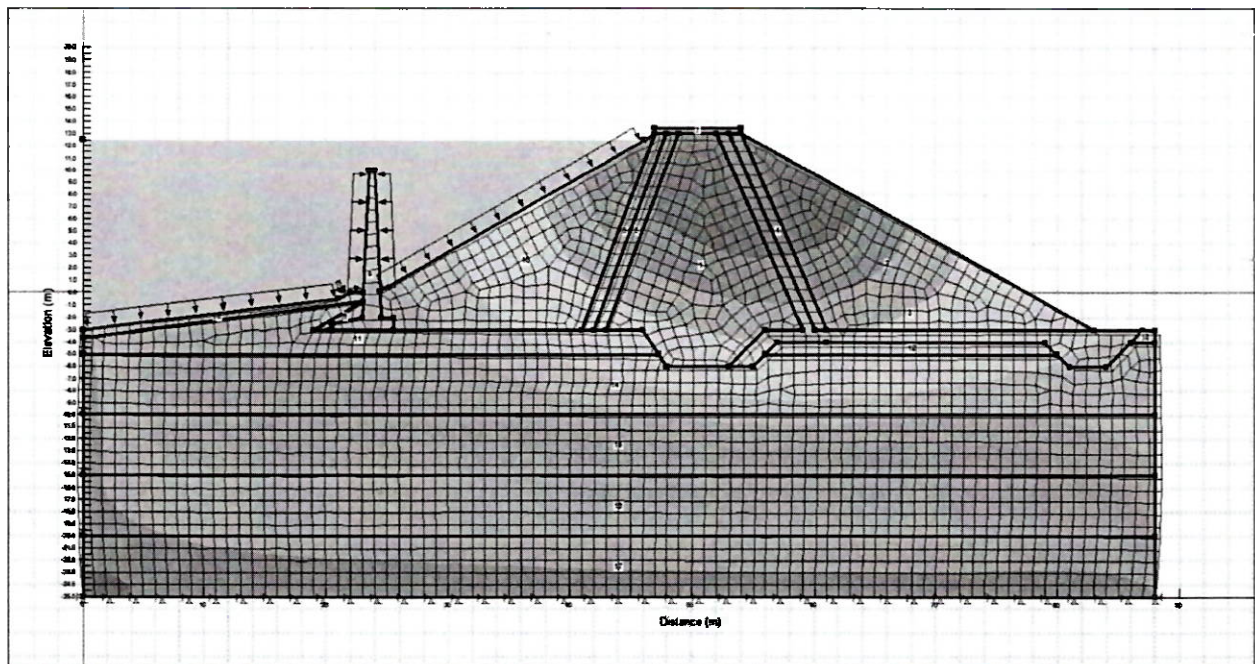


Figure-24: An overall Settlement 0.6 m is recorded on the dam first filling, along with excessive stresses development towards the downstream.

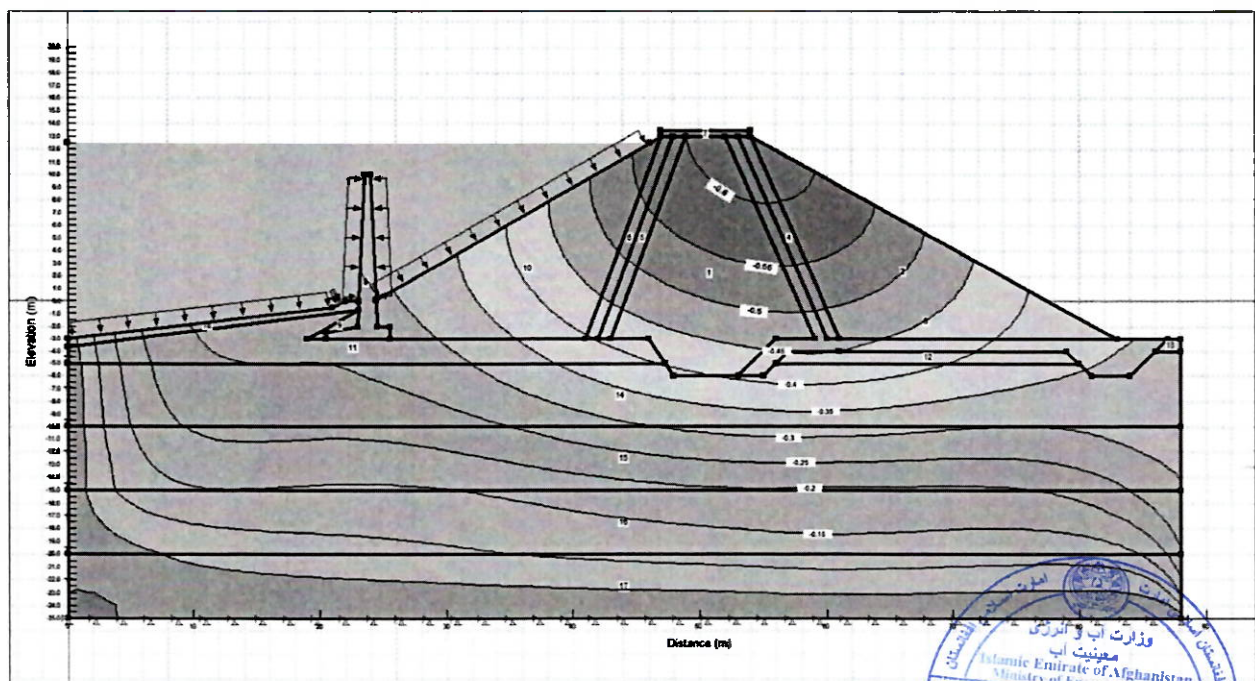
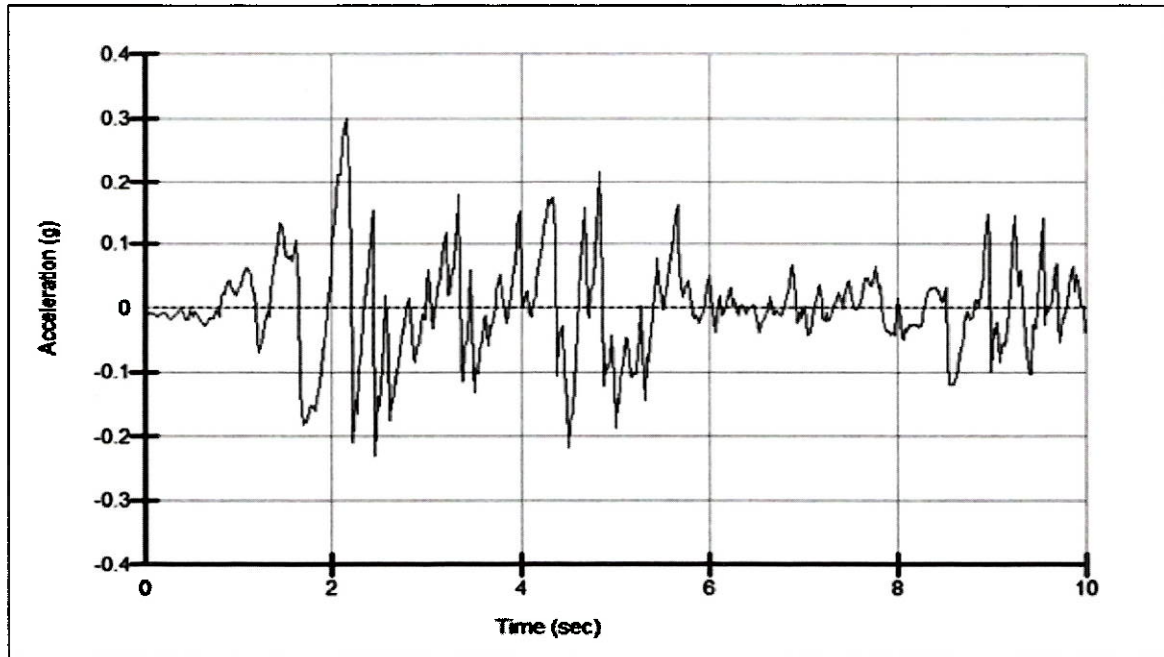


Figure-25: Settlement of the saddle dam in the form of contours.

6.5. Earth Quake, and its impact over the Earthfill saddle

The maximum horizontal earthquake acceleration is 0.3g for this site as shown below. The earthquake event is for 10 seconds, although the aftershocks with low acceleration could be for longer time, which will have no affects except it can keep the computer busy for quite some time.



Graph-05: earth quake spectrum for Sultan Saddle dam of 19 m

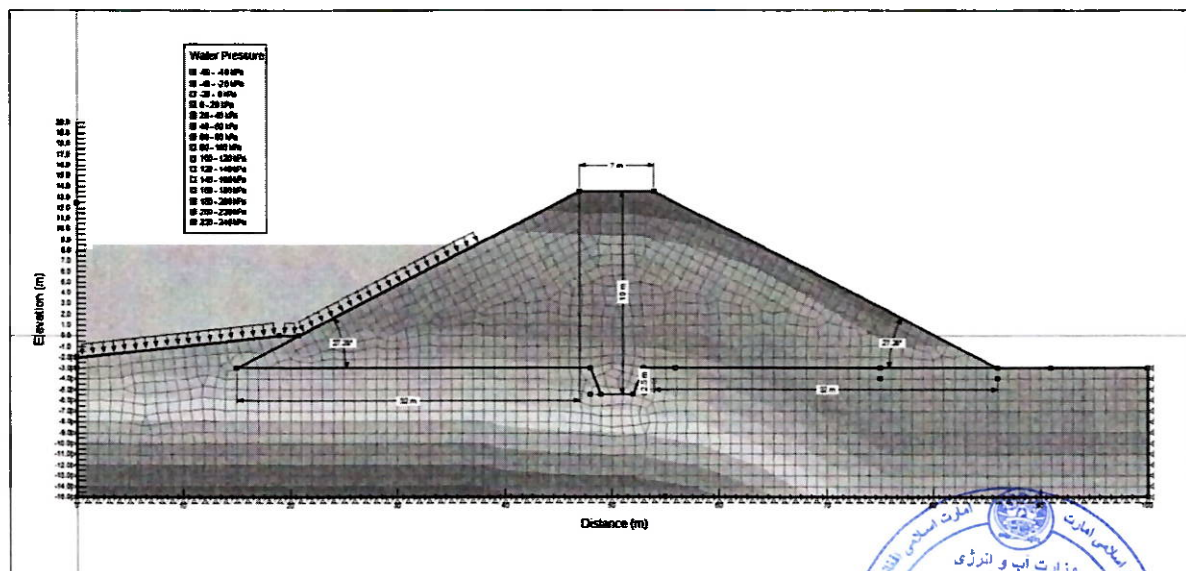


Figure-26: Pore water pressure development during earthquake event. Maximum pressure of 220 KPa is shown in the foundation layers.



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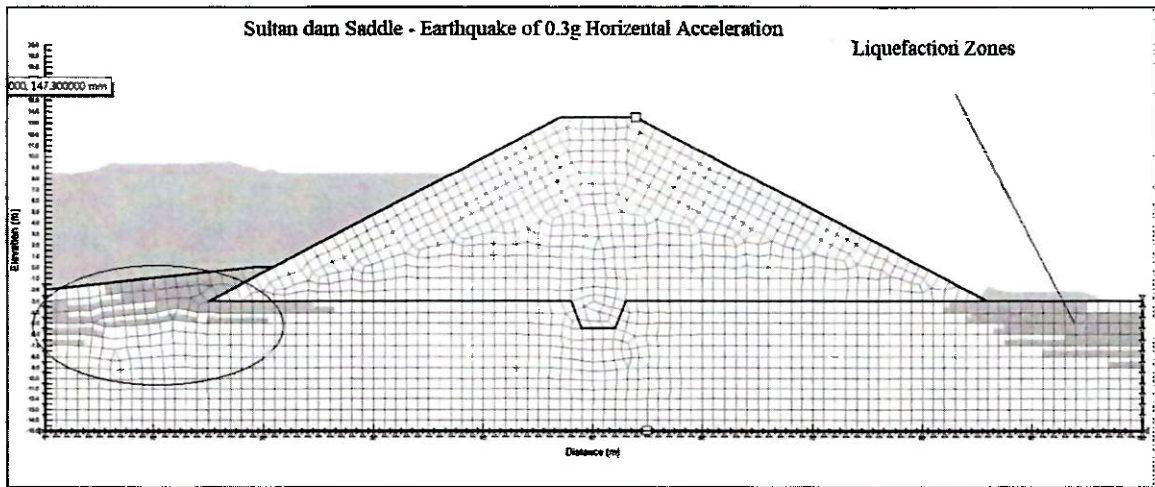


Figure-27: Liquefaction of the saddle at the heel and toe of the dam

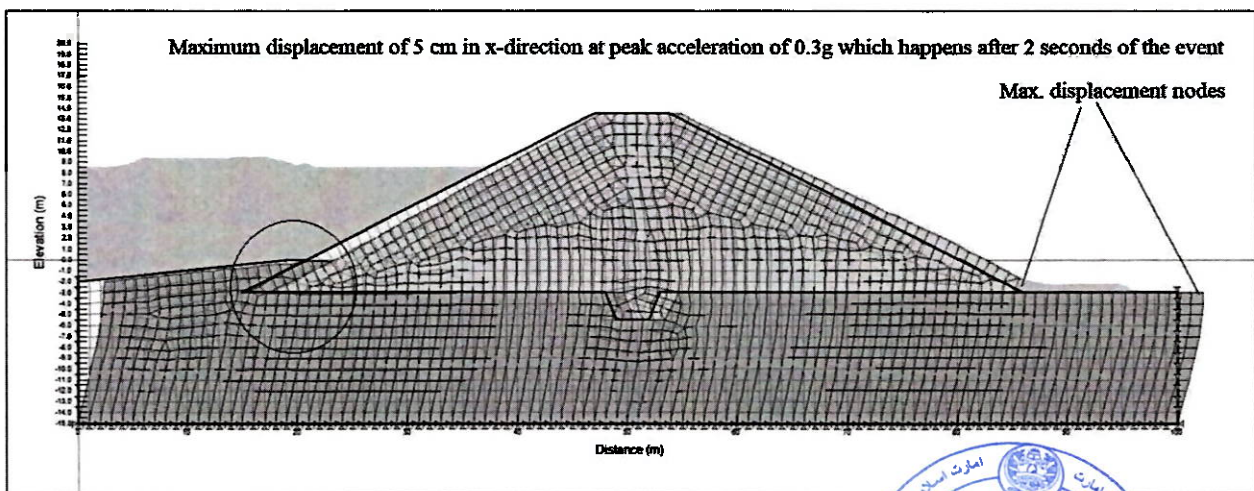


Figure-28: area of maximum settlement/displacement of 5 cm during the earthquake.



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6.6. Design Aspects of the Embankment dams.

The design of embankment dams is more based on experience, empirical approach, and engineering judgment. We as Technical Board will imply different approaches to study the dam analytically by using the software, and handmade calculations to prove that the saddle dam is safe against sliding, and to make sure there is no excessive seepages that will not cause major deformations.

On potential challenge in the Sultan Dam site is to connect the Stone masonry/ RFC section with new Earthfill dam.

The rockfill portion at the saddle where the new embankment and RFC are had to connect with each other should be first completed followed by saddle, and a hand-held compactor should be used to carry proper compaction.

6.7. Width of Dam Crest:

The dam crest should have enough width to accumulate construction machinery and crew members. European standard recommends a minimum width of 5 m. The width of the crest of Sultan Saddle dam is estimated through the following equation. The crest width is considered to be 7m in case the dam has to be raised in future so that they have enough crest width,

$$\text{Dam Height (H)} = 19 \text{ meters}$$

$$W \text{ (m)} = 1.65(H+1.5)^{1/3} = \text{approx. 5 meters}$$

6.8. Free Board

The freeboard is calculated by summing up the maximum flood level, wind tide, and wave run-up. Similarly, the freeboard to the top of the dam relative to the design flood water level has been set equal to 2.5 meters as a minimum. Here in Sultan dam, the maximum design flood level is 2383.5 m, while the crest level is 2385 plus a 0.5 m high gravel layer (camber, to accommodate future settlement).



6.9. Loads due to Water and Wind

Water in the reservoir causes basic external loads on the dam body which may damage or cause failure of the dam. In case of the Rockfill dams, pore pressure, and seepages exert no pressure as it is free draining material, except the excess leakages can cause erosion both at the foundation and also to some extent at the downstream most part of the dam i.e at the exit point. So, at the foundation proper filter in terms of spreading of sand-gravel layers will be carried, and at the downstream most part (heel) a pipe of 10 inches dia, embedded in a filter-sand-gravel will collect all the flow and will dispose of 50-100 m away from the dam. The leakages will also be measured to know if there is any increase in the leakage with the passage of time.

The weight of the dam at any level should be more than the overall forces exerted by the water standing in the reservoir.

6.9.1. Wave Run-up and Wind Tide

Waves hitting the upstream face slope will run up to a height above the mean water level as shown below. The FB above the maximum pool level is calculated through following equations.

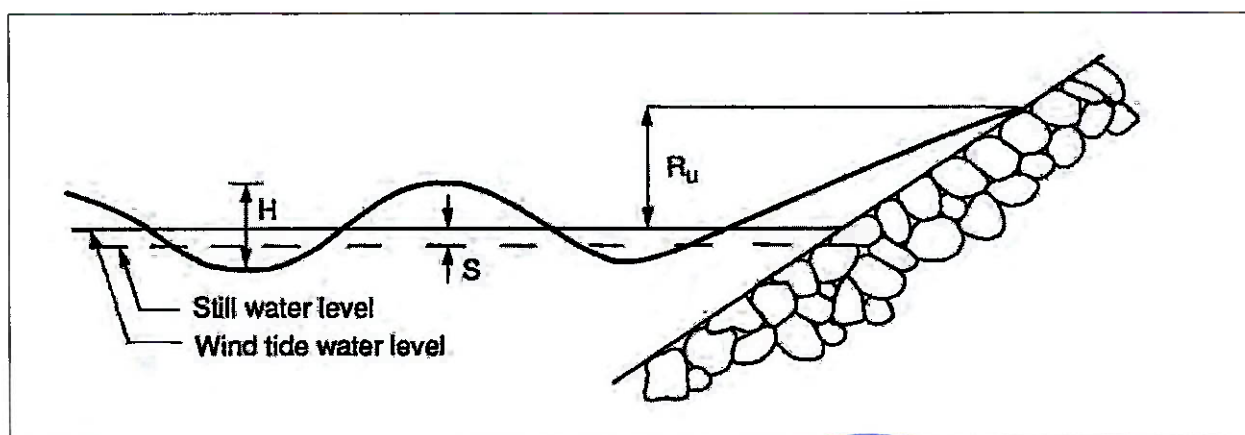


Figure-29: Wave height H , wind tide S , and run-up, R_u at the upstream slope

Run-up exceeded only by 2% of the waves in a storm is given by



27/

$$Ru=rH_s$$

Where, Ru: run-up, vertical height above still water level

r: attenuation factor, = 2.0 taken from a graph

H_s: significant wave height, m

Where H_s is given by the following formula

$$\frac{gH_s}{U_*^2} = 0.0506 \left[\frac{gFe}{U_*^2} \right]^{1/2}$$

Where, g = 9.81 m/s²,

U* = is friction velocity, and can be calculated from U₁₀ i.e wind speed 10 m above the ground. U₁₀ for Sultan dam site = 90 km/hr = 23.5 m/s,

$$U_* = (0.0008 + 0.000065 U_{10}) * U_{10}^2, = 2.0$$

Fe= fetch i.e. distance in Km from the opposite shore of the dam, 1.0 km in this case

$$H_s = 1.30 \text{ m}$$

Ru = 2.0 * 1.30 = 2.6 m, and in our case Free Board is 3.0 m, hence no overtopping, but if the wind speed increases to 110 Km/hr, then

Ru = 2.3*1.50 = 3.3 m, and there are chances that some waves might reach crest level,

Note: Since, there is 1m big riprap boulders on the upstream which will splash the wave before reaching the top of the saddle crest.



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Figure-30: Placement of Riprap as slope stability on the upstream side.

6.10. Grouting Requirements

Geotechnical investigation shows alluvial formation about 15-17 m below the ground surface, and excavating a trench for clay cut off at a this much depth is difficult task. Similarly, grout might not be an efficient way in alluvial formation to stop seepages.

The bedrock is 5 m thick, and is located 15 below the ground surface, and is highly fractured. So, an attempt was made to spread compacted clay blanket of 1 m thick, overlayed by 0.5 m thick gravel to prevent seepages as shown in the drawing below.



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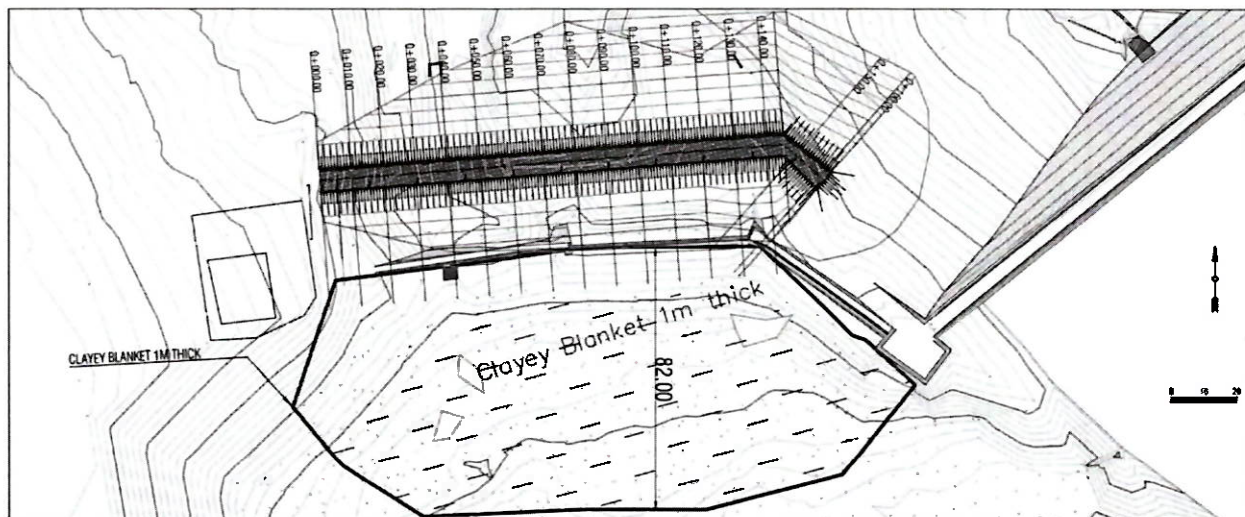
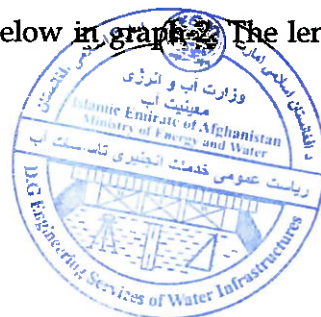


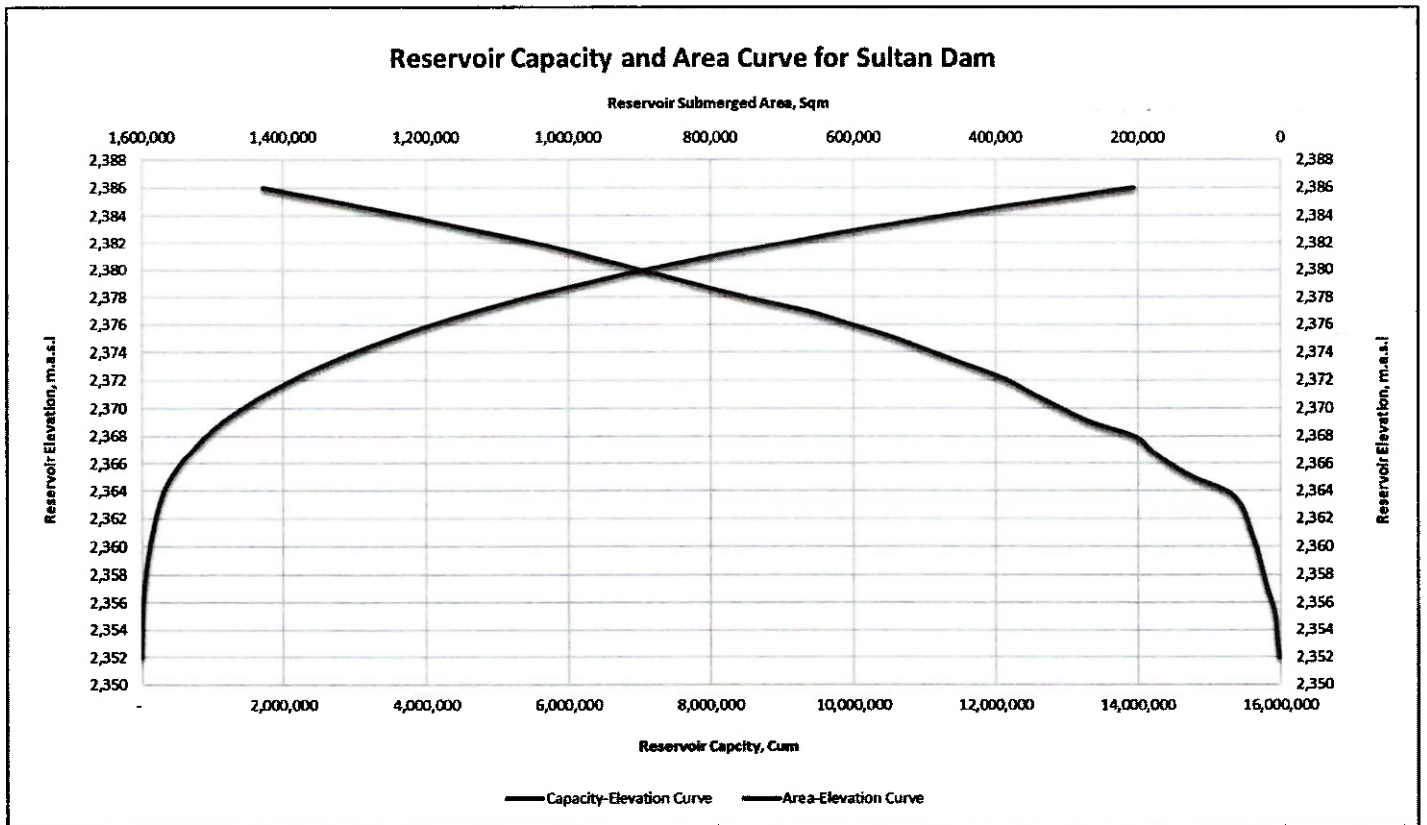
Figure-31: showing drawing of the new Saddle Dam plan, and upstream clayey blanket.

7. Dam storage capacity

At a height of 37 m above river bed level i.e. at an elevation of 2347, this century old stone masonry dam stores about 11.28 million m^3 irrigation water, while the annual average hydrology is 50 million m^3 . The dam will also have bottom outlet gates to flush sediment from time to time. The submerged area of the reservoir is about 2.14 Km^2 as shown below in graph-2. The length of reservoir is about 3.5 km upstream from the dam body.



[Handwritten signature]



Graph-2: Reservoir Capacity & Area Curve for Sultan Dam, with 11 million m³ storage at an elevation of 2384 (37 m), and 10 m³ at elevation of 2383.5 (36.5).



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8. Cost of the 19m high, and 162 m long Saddle

The preliminary estimation for the project is given below. Total cut volume is estimated to be 55,282 m³, while total fill volume is to be 167,000 m³ and that I have used 1.25 as fill factor because the loose soil will be compacted and thus required more soil to construct the saddle_dam body.

Total cost of the dam body construction is Afn 36.5 million.

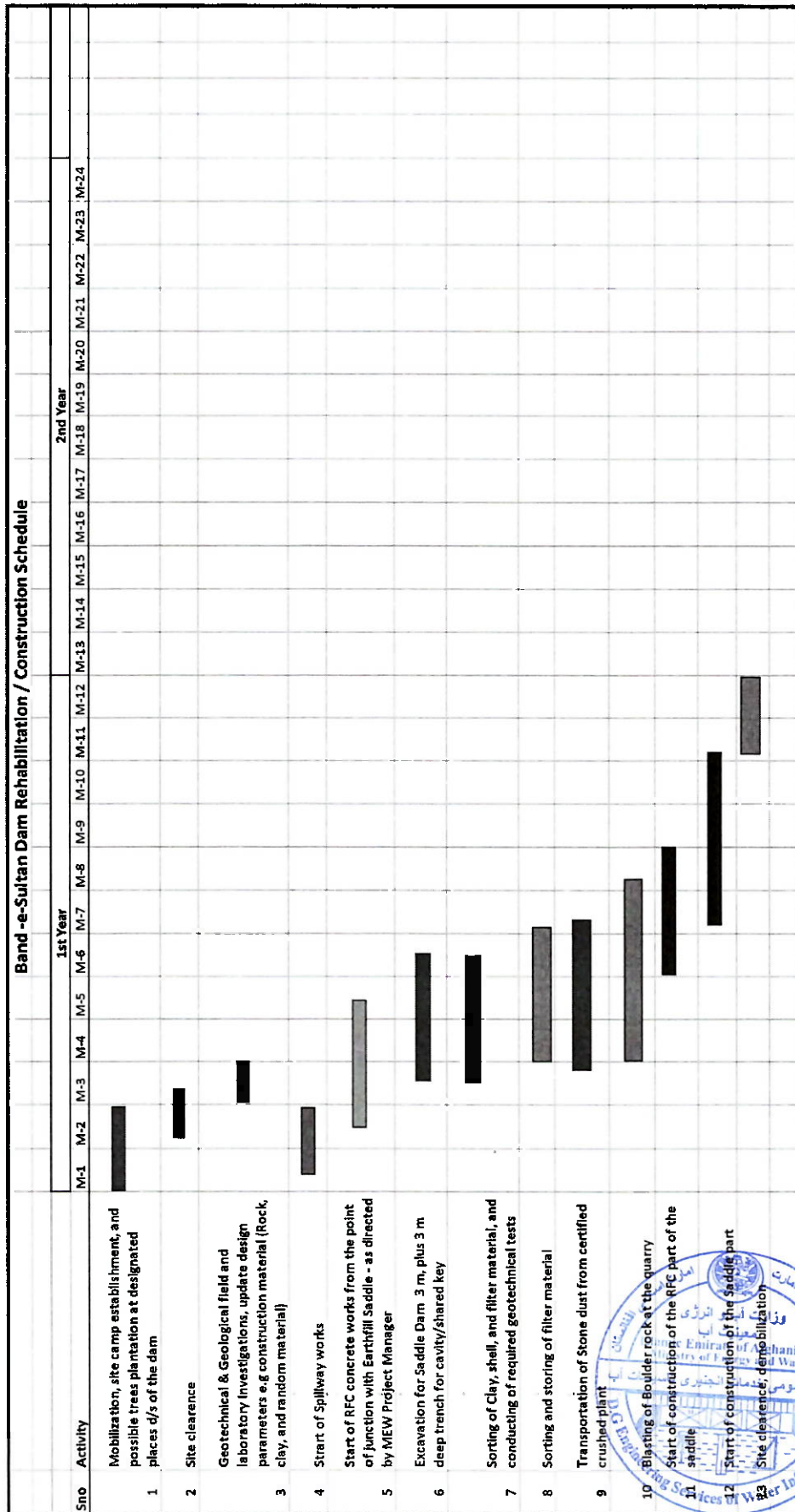
Bill of Quantity of Sultan Dam's EarthFill 162 m long Saddle:						
Sub-Items	Area	Length	Fill/Cut Factors	Quantity	Unit Rate	Amount (Afn)
(i) Removing the top 3 meters	213	162	1.2	41,407	150	6,211,080
(ii) Cavity Excavation in ordinary soil	21	162	1.2	4,082	150	612,360
(iii) Removing the top 0.5 meters at u/s reservoir area (for clay blanket)	13,000.0	0.5	1.2	7,800	150	1,170,000
(iv) Removal of top soil at various locations	693	10	1.2	8,316	150	1,247,400
				61,606		-
(i) Shoulders (random fill)	139	162	1.25	28,148	200	5,629,500
(ii) Core (Clay)	57	162	1.25	11,543	280	3,231,900
(iii) Filter (clean sand and gravel))	35	162	1.15	6,521	1,000	6,520,500
(iv) Gravel Layer at up stream face of the embankment	8	162	1.15	1,490	200	298,080
(v) Toe drain gravel	6.5	162	1.15	1,211	200	242,190
(vi) Gravel layer (0.5 m thick) at crest	3.5	162	1.15	652	200	130,410
(vii) Upstream Riprap (1m thick)	36	162	1.0	5,832	1,000	5,832,000
(viii) Upstream Clayey blanket at Reservoir	13,000.0	1	1.25	16,250	200	3,250,000
(ix) Upstream Gravel blanket overlayed the c	13,000.0	0.5	1.15	7,475	200	1,495,000
				79,121		-
Stone masonry parapet wall						-
(i) Perforated Toe Drain pipe (dia 25 cm)		162	1	162	1000	162,000
(ii) Perforated PVC pipe (dia 12 cm)		291.6	1	292	300	87,480
(iii) Drilling of drainage wells at d/s		291.6	1	292	250	72,900
(iv) Gravel for Drainage wells	0.0029	291.6	1	0.832	600	499
				-		-
(i) Solar Pannel				16	3,000.0	48,000
(ii) Steel post with lights				8	20,000.0	160,000
(iii) Batery and Charge controller				8	12,000.0	96,000
(iv) Foundation				8	3,000.0	24,000
(v) Anchor Bolts				32	500	16,000
						36,537,299

Table-07: Showing cost of 19 m high compacted earth-fill saddle dam



9. Construction Schedule.

The construction will start with mobilization, establishment of site camp, and start of spillway works followed by other activities as are shown below in the schedule.



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10. References

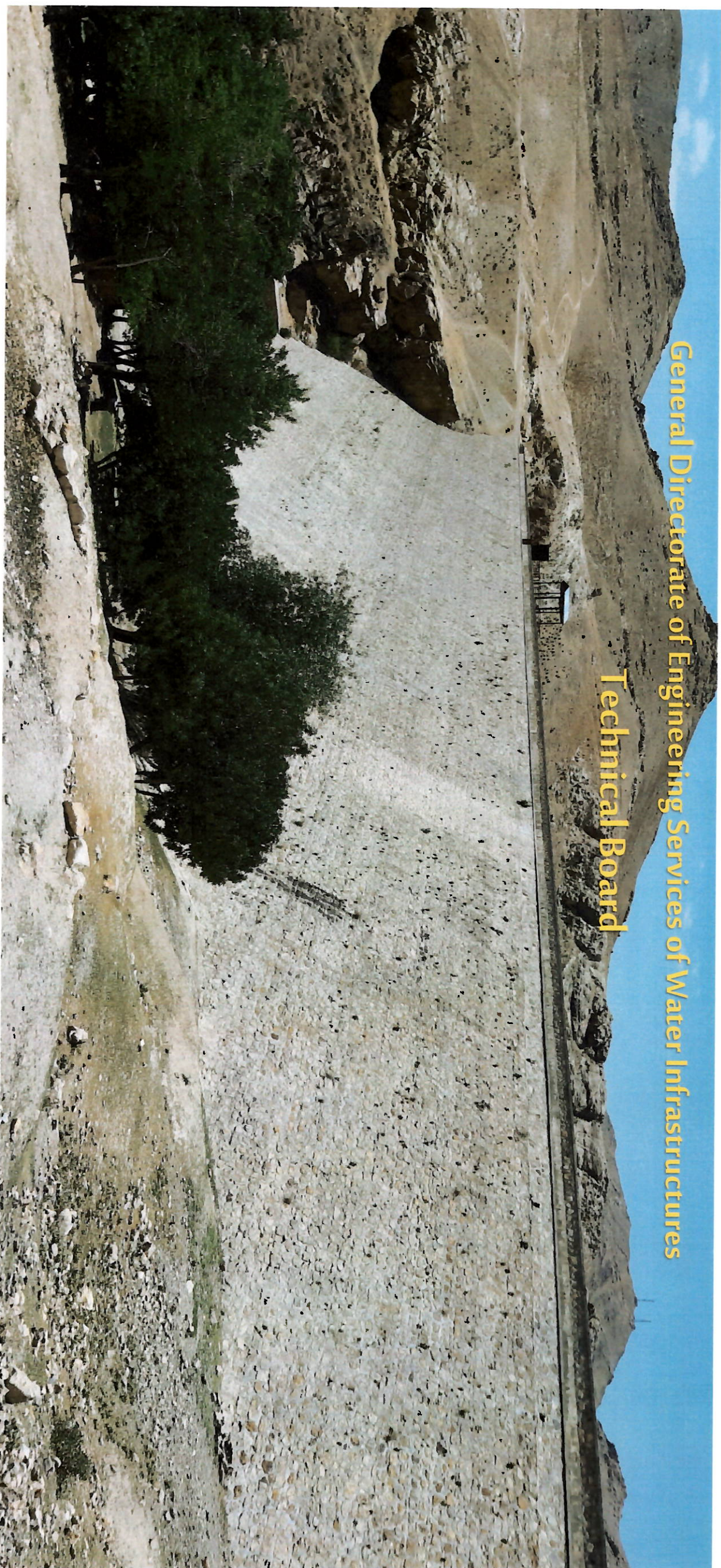
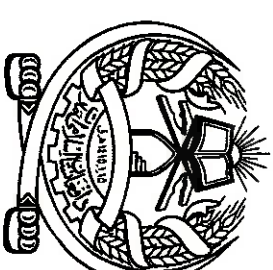
1. Embankment /Rockfill dam by Kaare Hoeg, and Tore valstad - NTNU
2. Subgrade Soil Frost Susceptibility Assessment for Pavement Design in Manitoba
3. Embankment dam – Design Standard No. 13 USBR. Protective Filters
4. Small Dam – USBR
5. Steps in Filter Design – Chapter 26 – USBR



Thanks for the reading,



Islamic Emirate of Afghanistan
Ministry of Water and Energy
Deputy Ministry of Water

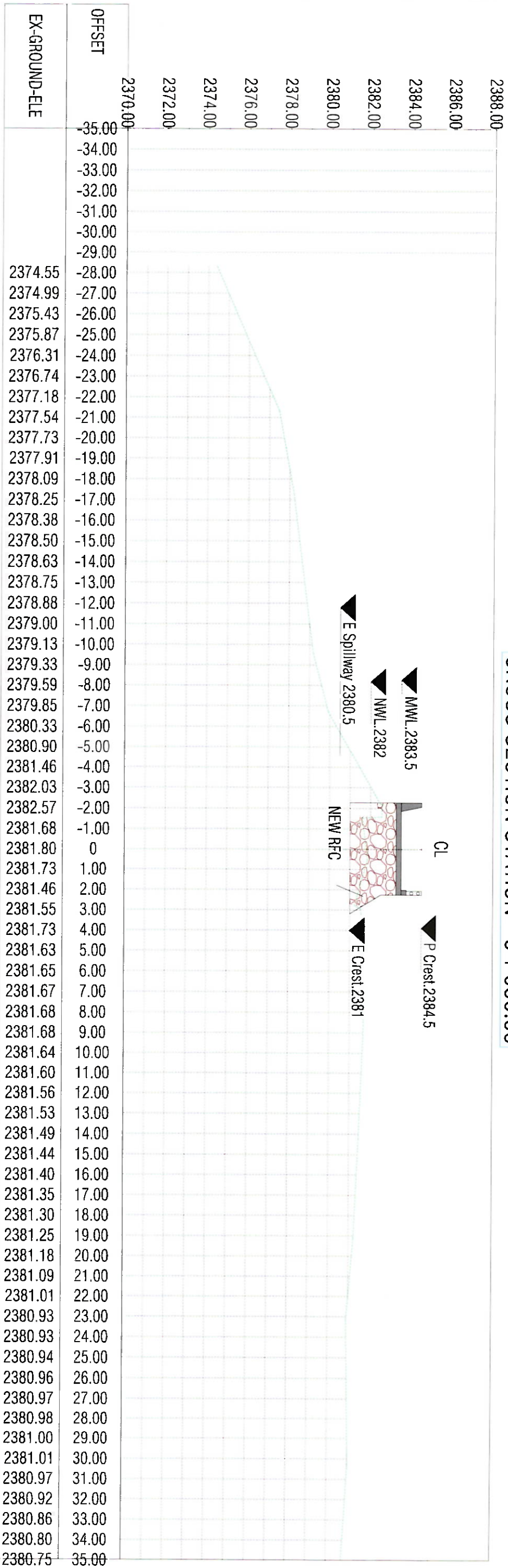


General Directorate of Engineering Services of Water Infrastructures
Technical Board

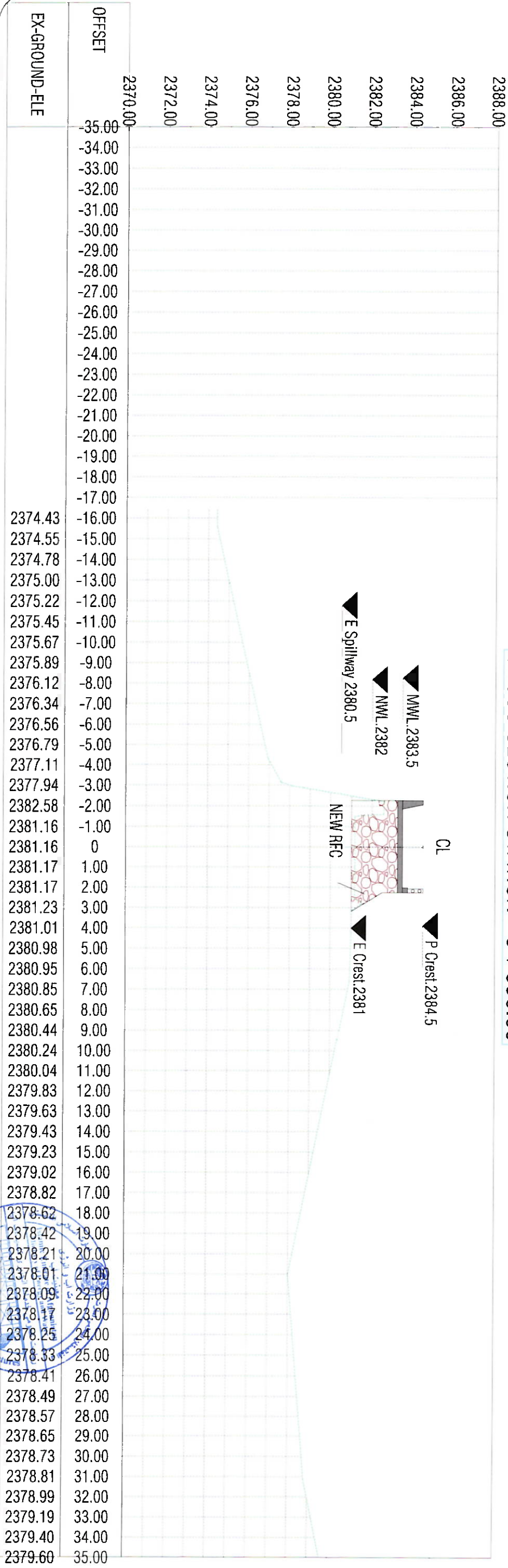
Rehabilitation of Sultan Dam

Date		October-2023
Prepared	Checked	Approved
Naqibullah Ghubar	Tariq Tasal	Abdul Chafor Qmani
Gravity Section Drawing		

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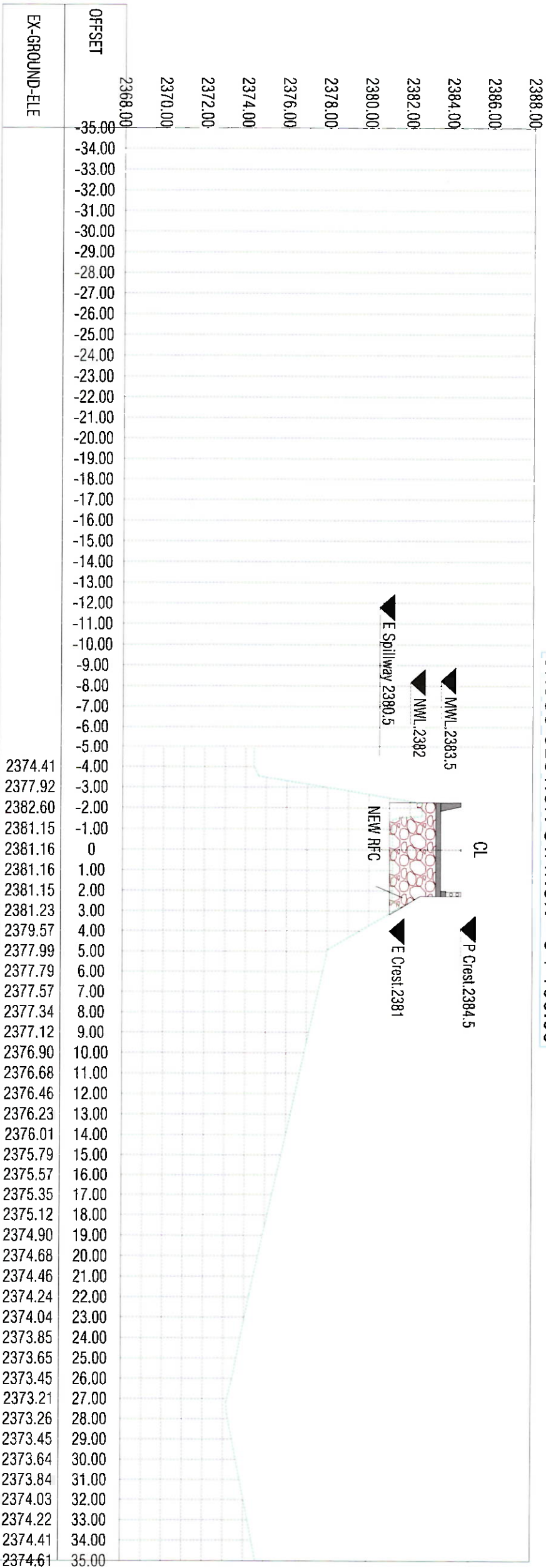


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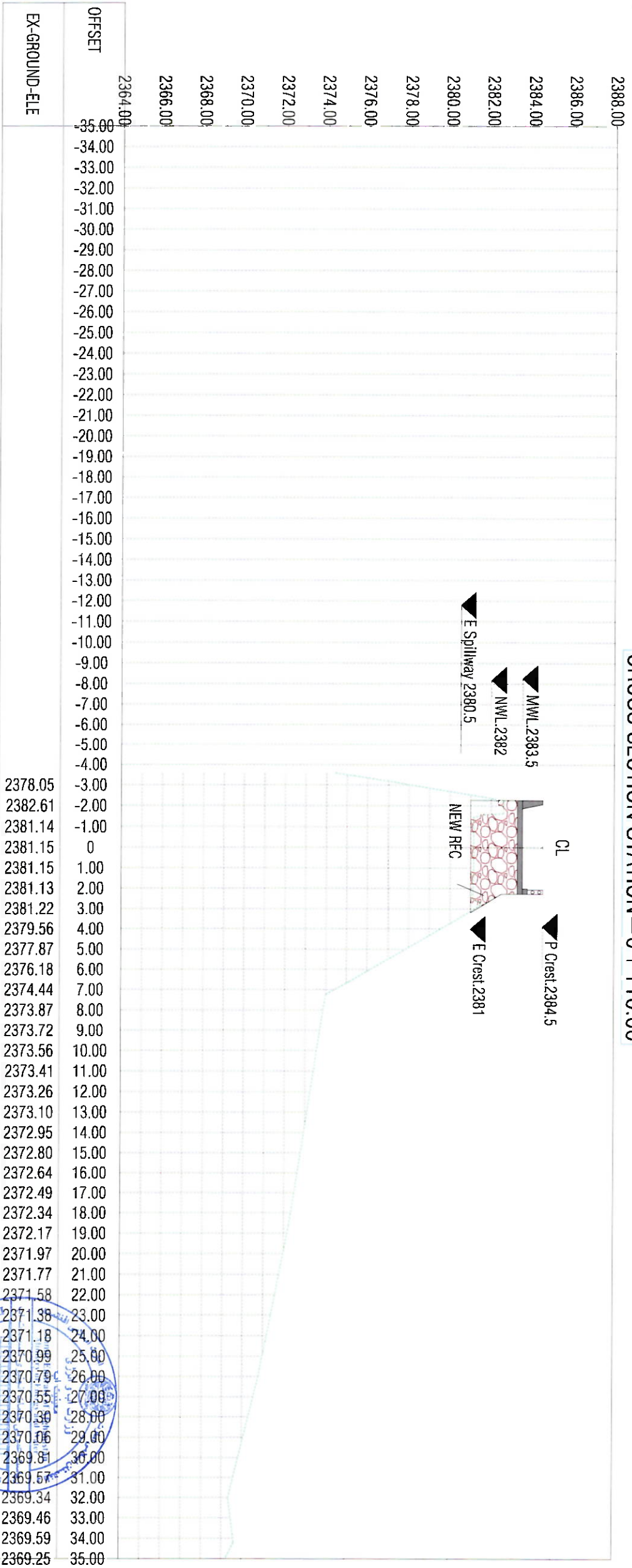


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SHEET NO		CS		1		19													

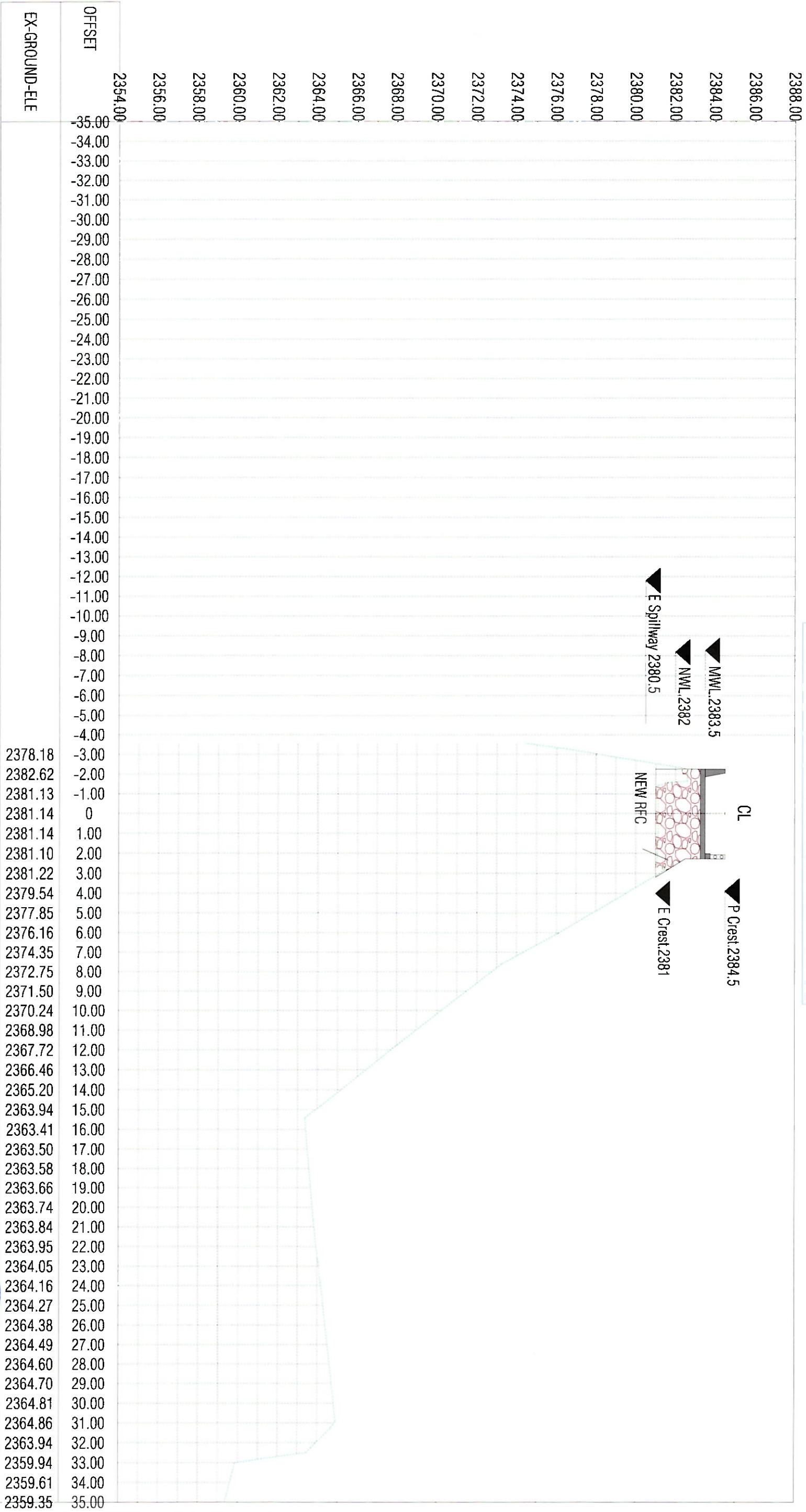
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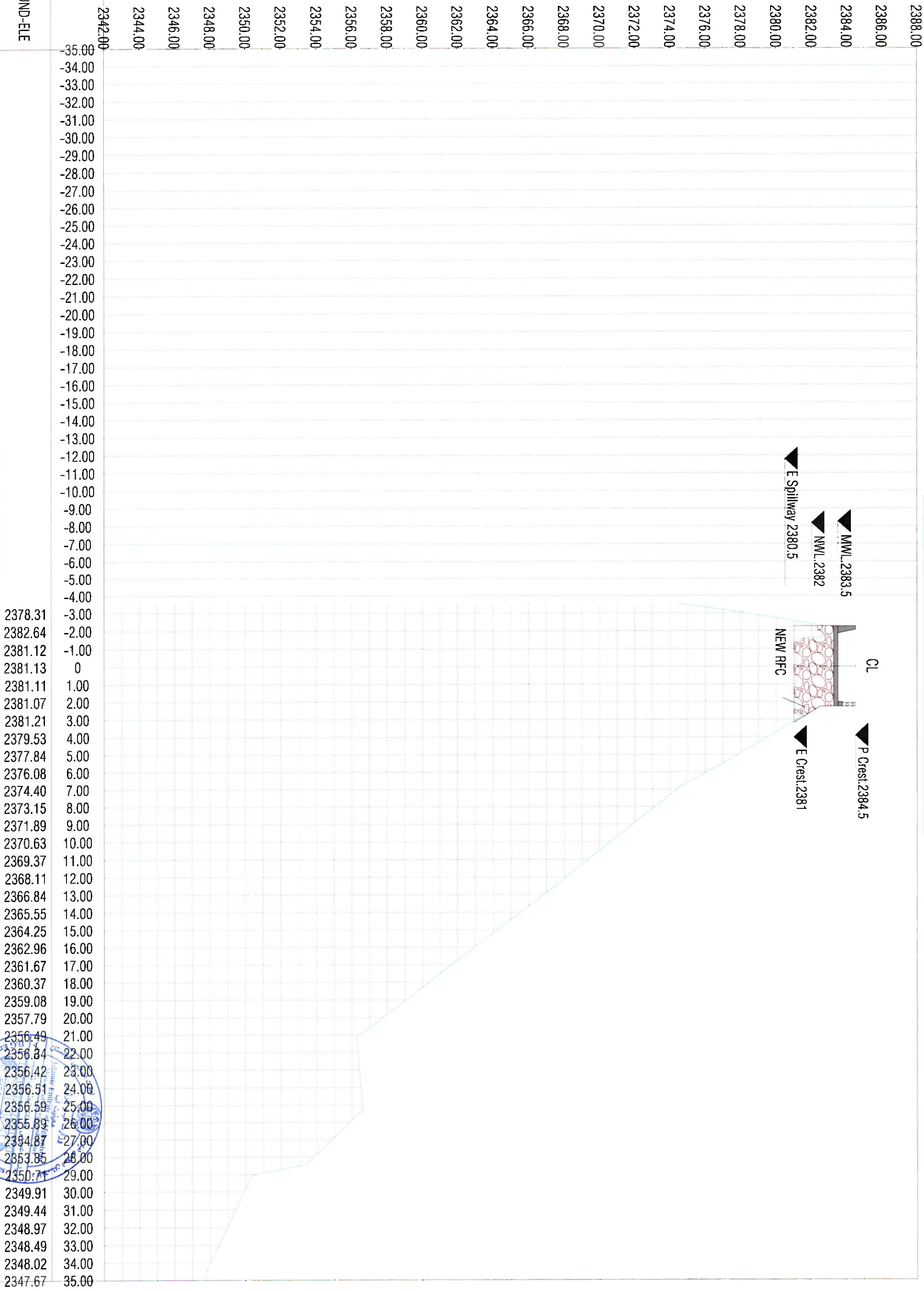
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DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD

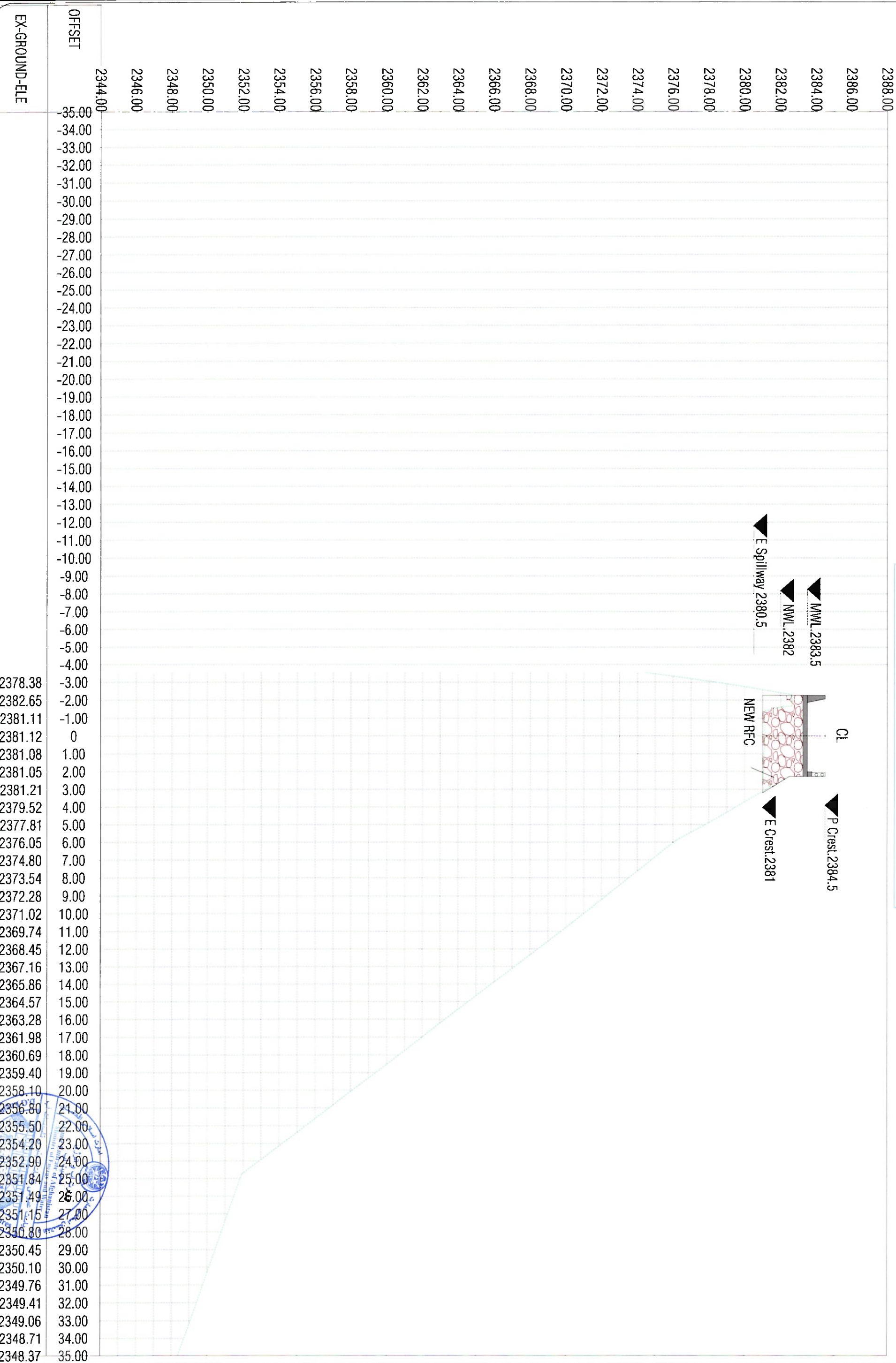
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SHEET NO



19

CROSS SECTION STATION=0+140.00





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PROJECT NAME
SULTAN DAM REHABILITATION

DRAWING
INCREMENTAL STATIONING
CROSS SECTION

PROVINCE
WARDAK

DISTRICT
JAGHATO

DESIGN DATE
NOV 2023


SCALE
AS SHOWN

DESIGNED BY:
ENG. MAJIDULLAH GHOBARI

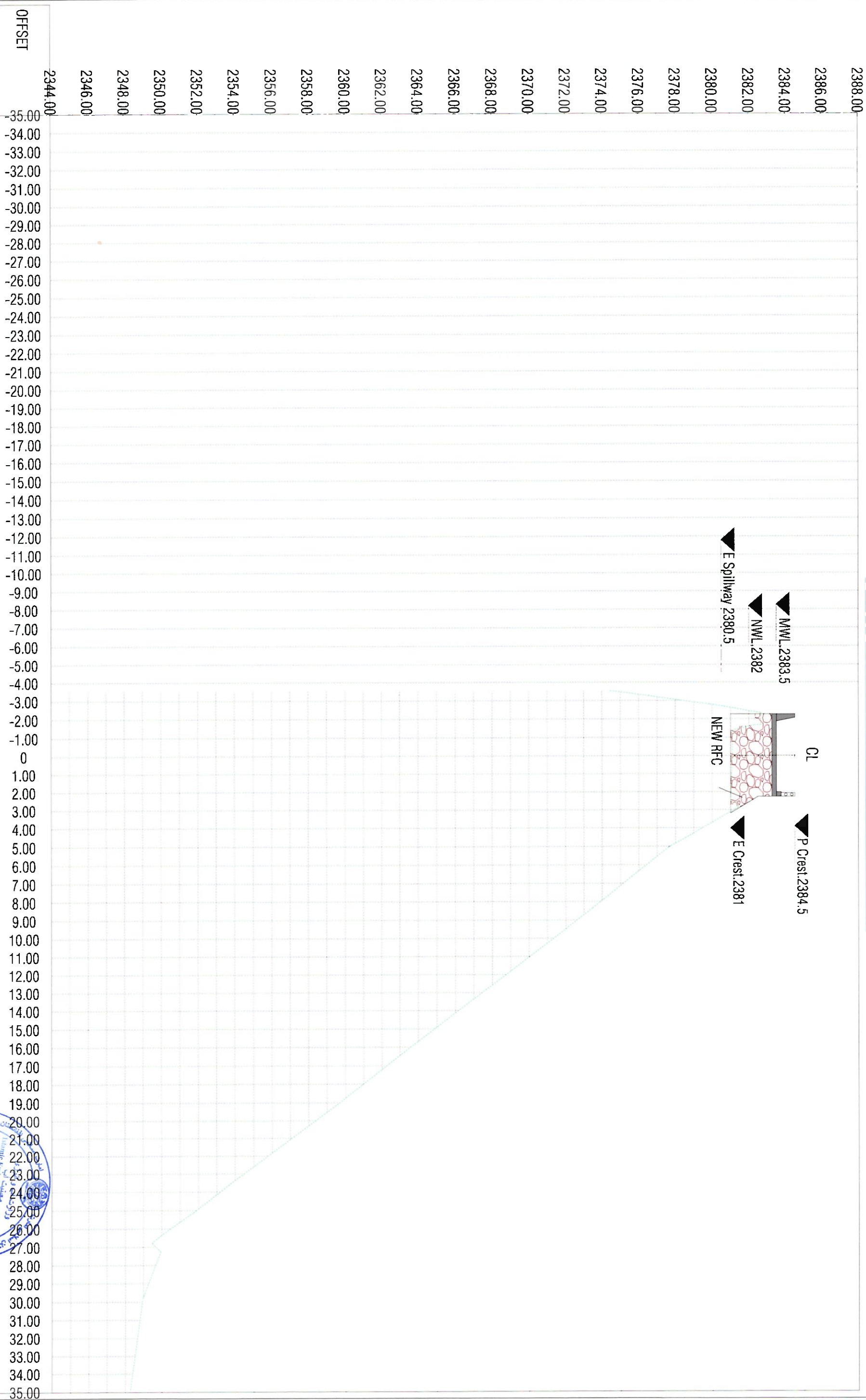
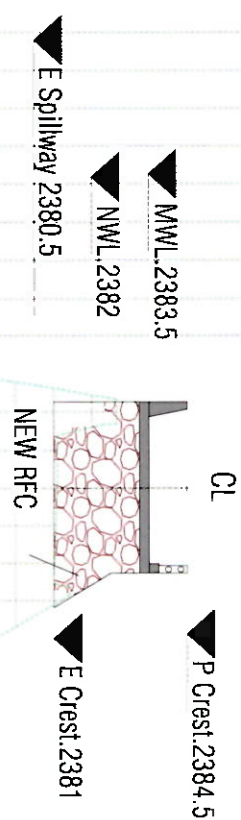
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APPROVED BY:

SHEET NO
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19



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PROJECT NAME
SULTAN DAM REHABILITATION

DRAWING

INCREMENTAL STATIONING

CROSS SECTION

WARDAK	PROVINCE
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DISTRICT
JAGHATON

DESIGN DATE	NOV 2023
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SCALE	AS SHOWN
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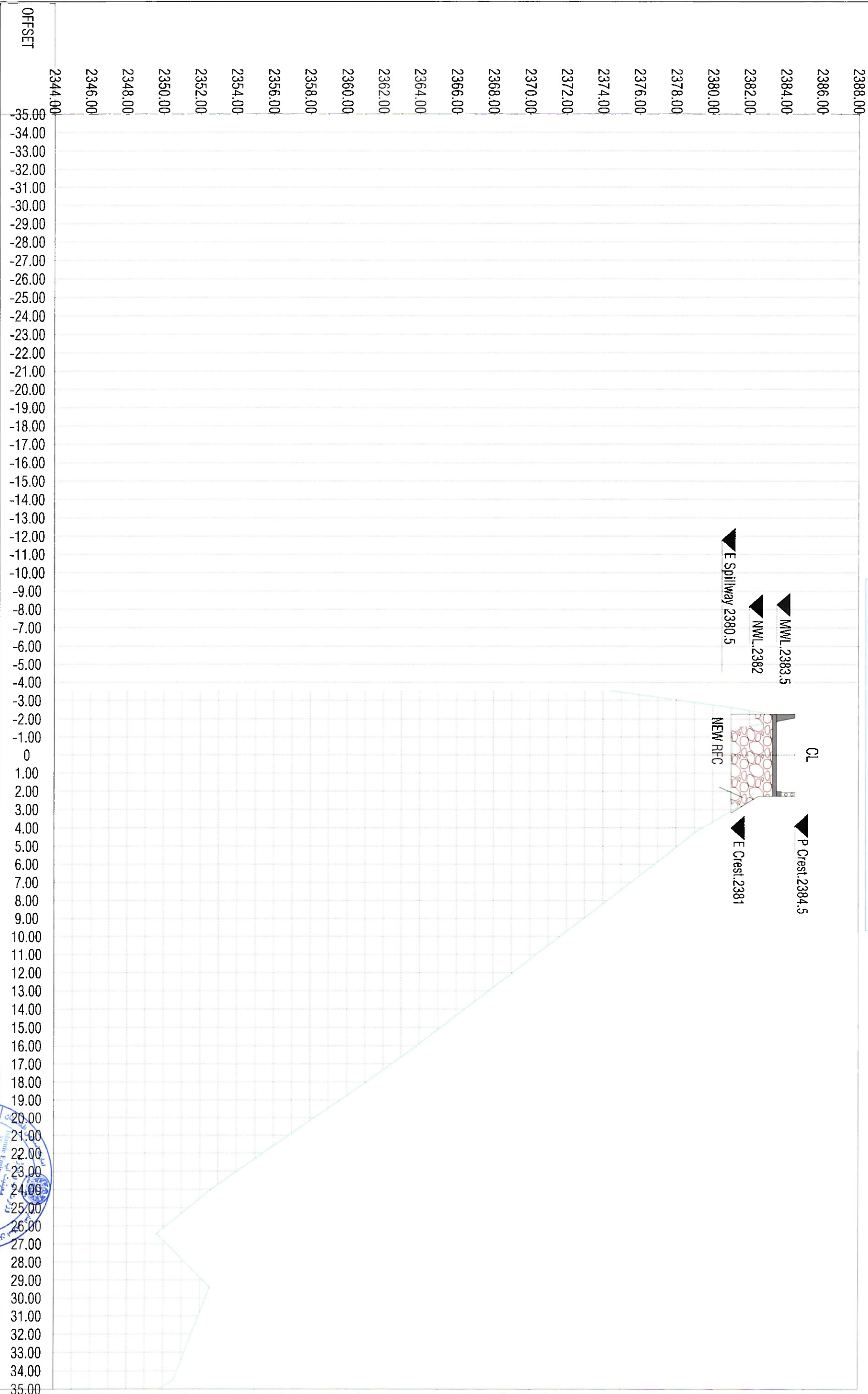
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ENG MARIQ TASAL


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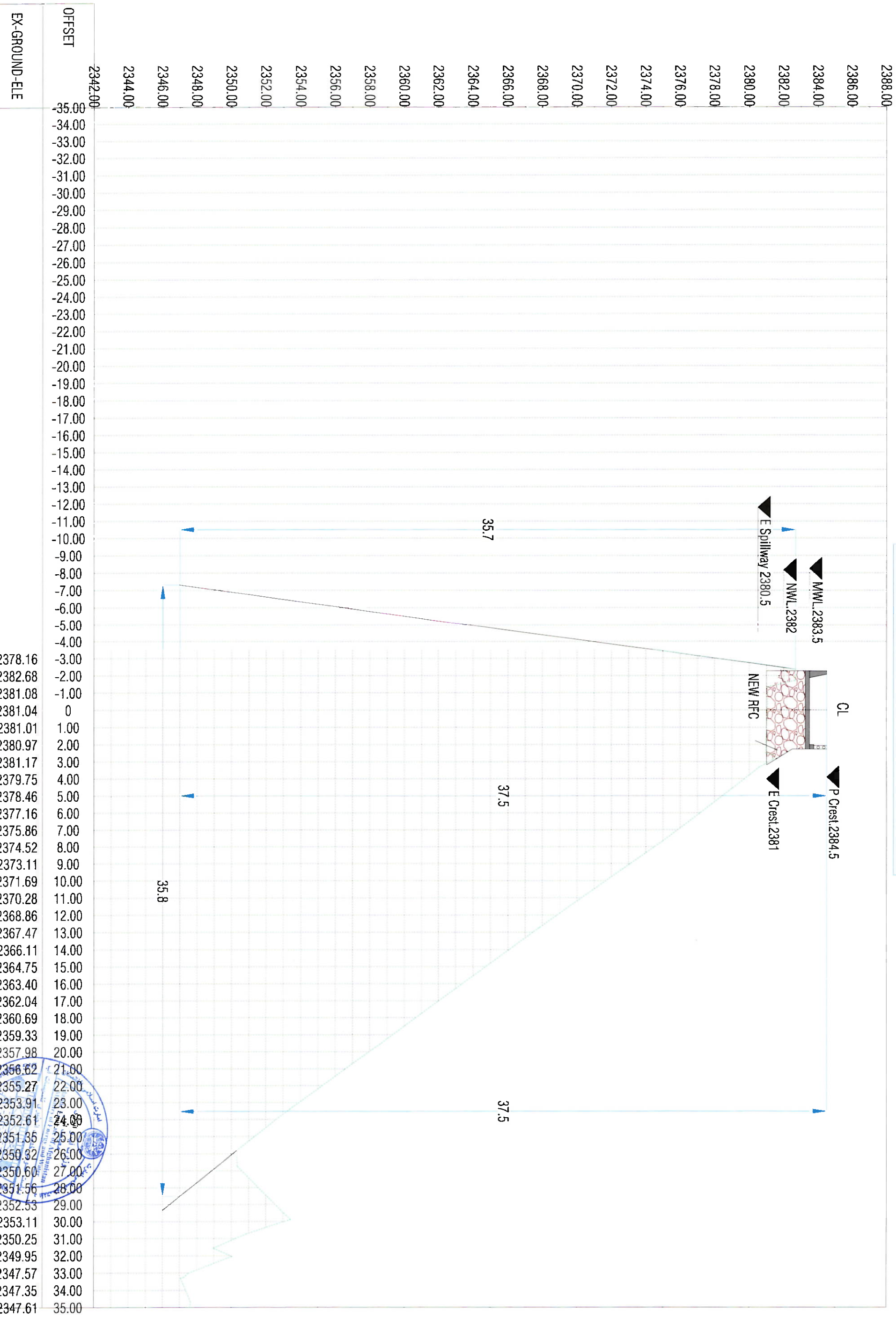
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ISLAMIC EMIRATE OF AFGHANISTAN
MINISTRY OF WATER AND ENERGY
DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD

PROJECT NAME
SULTAN DAM REHABILITATION

DRAWING
INCREMENTAL STATIONING
CROSS SECTION

PROVINCE
WARDAK

DISTRICT
JAGHATO

DESIGN DATE
NOV 2023

SCALE
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DESIGNED BY:
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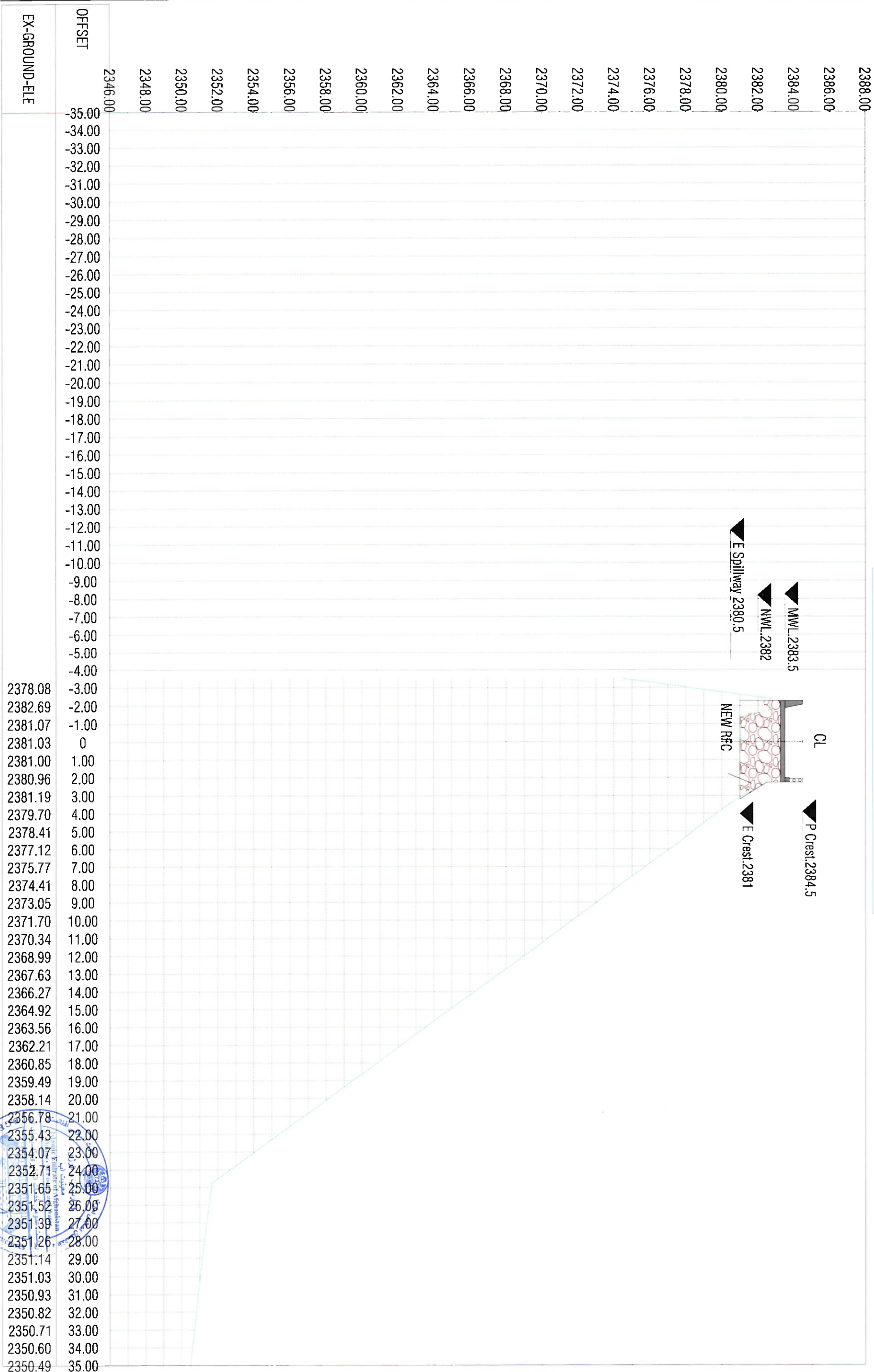
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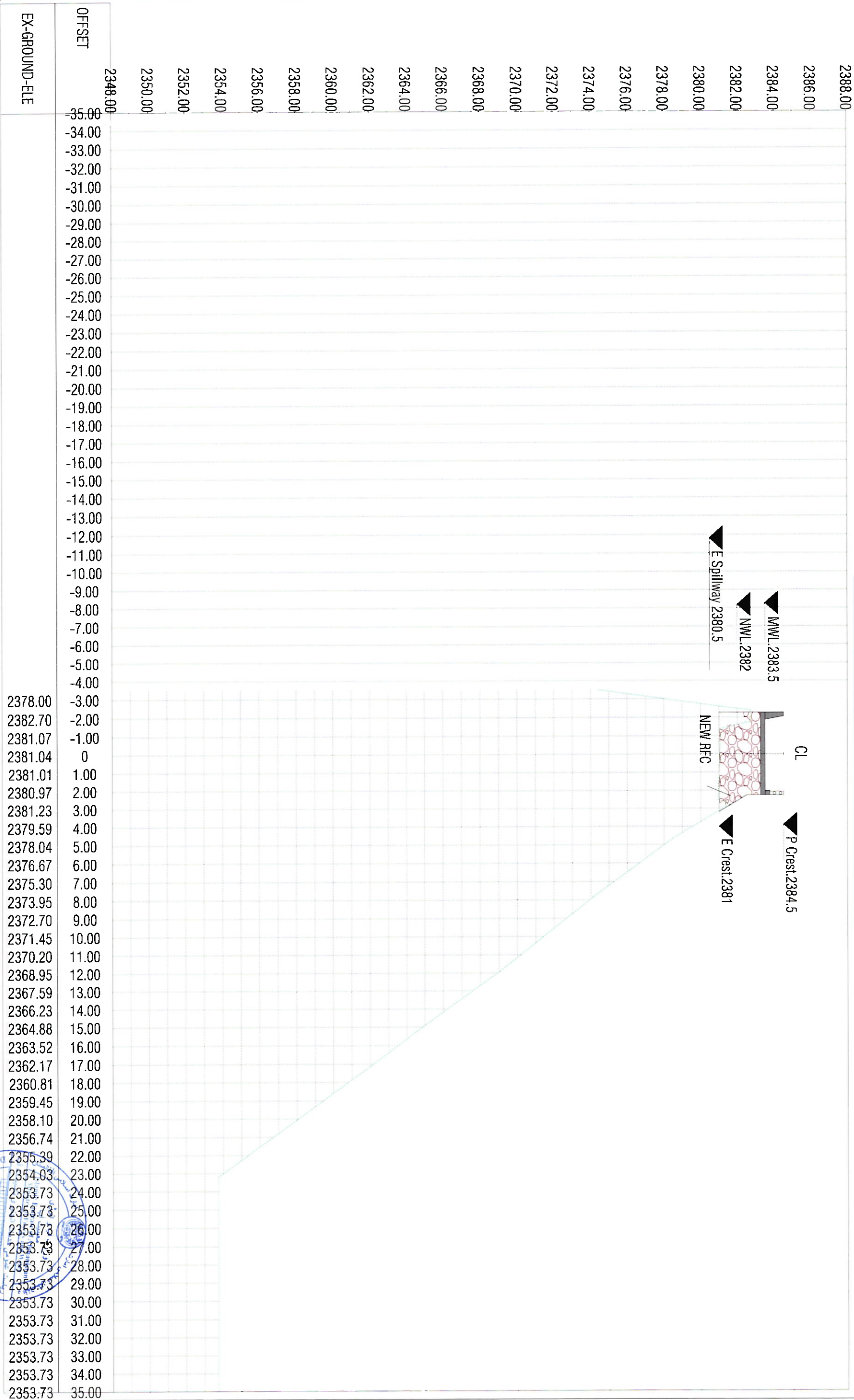
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EX-GROUND-ELE

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DRAWING

INCREMENTAL STATIONING

CROSS SECTION

WARDAK	PROVINCE
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DISTRICT	JAGHATO
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SCALE	AS SHOWN
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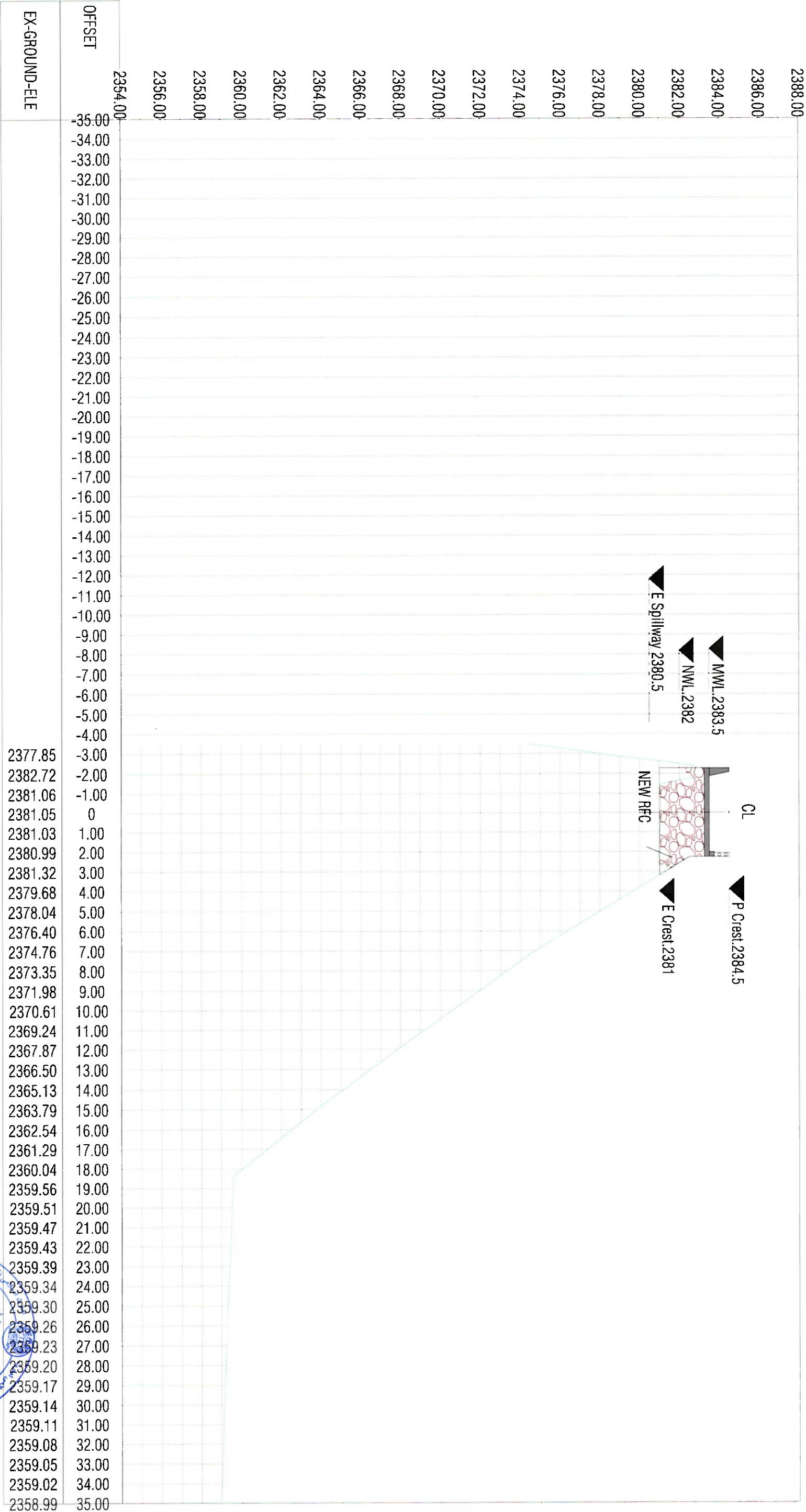
ENG NAQIBULLAH GHUBAR
ENG M TARIO TASAL


12

SHEET NO



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DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD

PROJECT NAME
SULTAN DAM REHABILITATION

DRAWING
INCREMENTAL STATIONING
CROSS SECTION

PROVINCE
WARDAK

DISTRICT
JACHATO

DESIGN DATE
NOV 2023

SCALE
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DESIGNED BY:
ENG. NAQIBULLAH GHUBAR

CHECKED BY:
ENG. M. TARIQ TASHAL

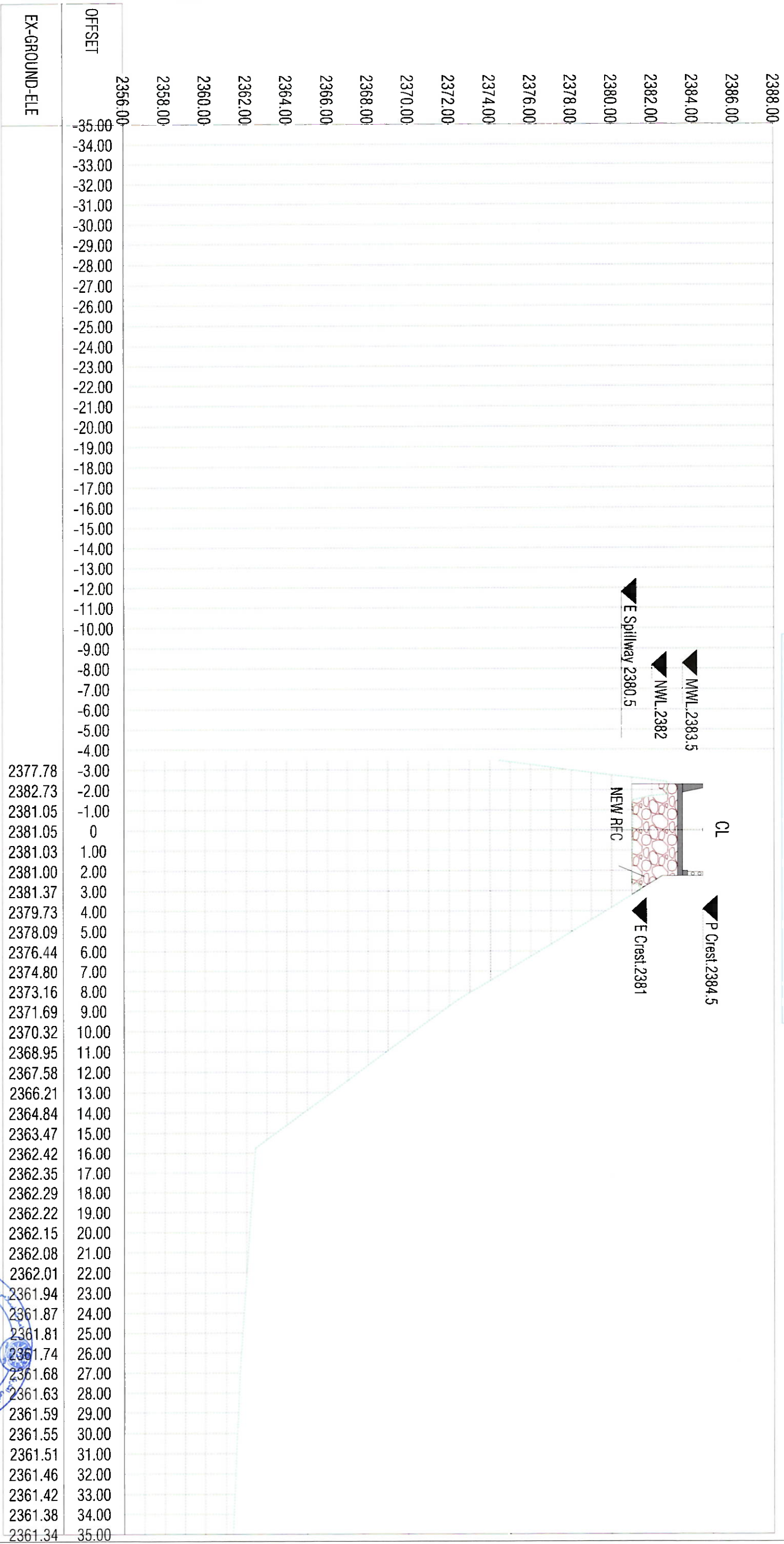
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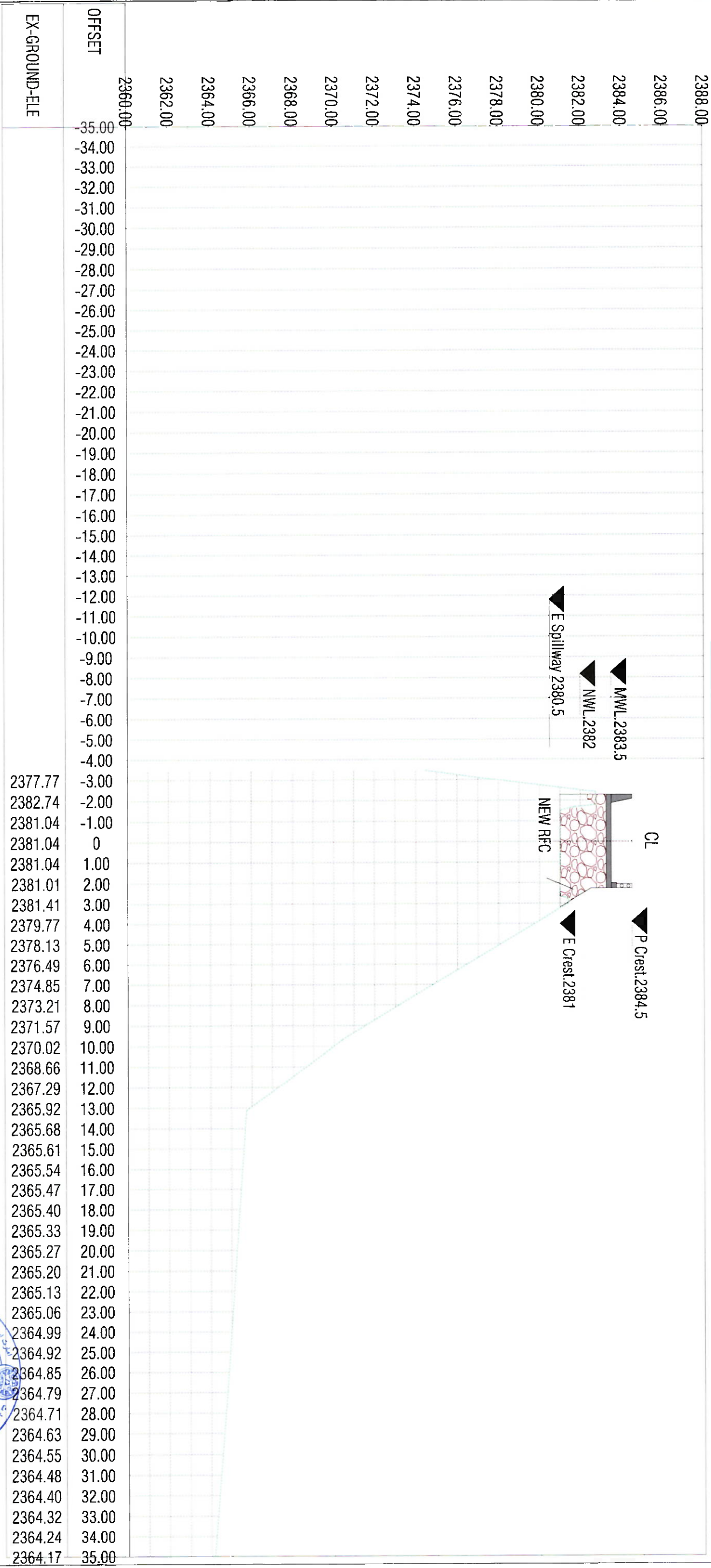
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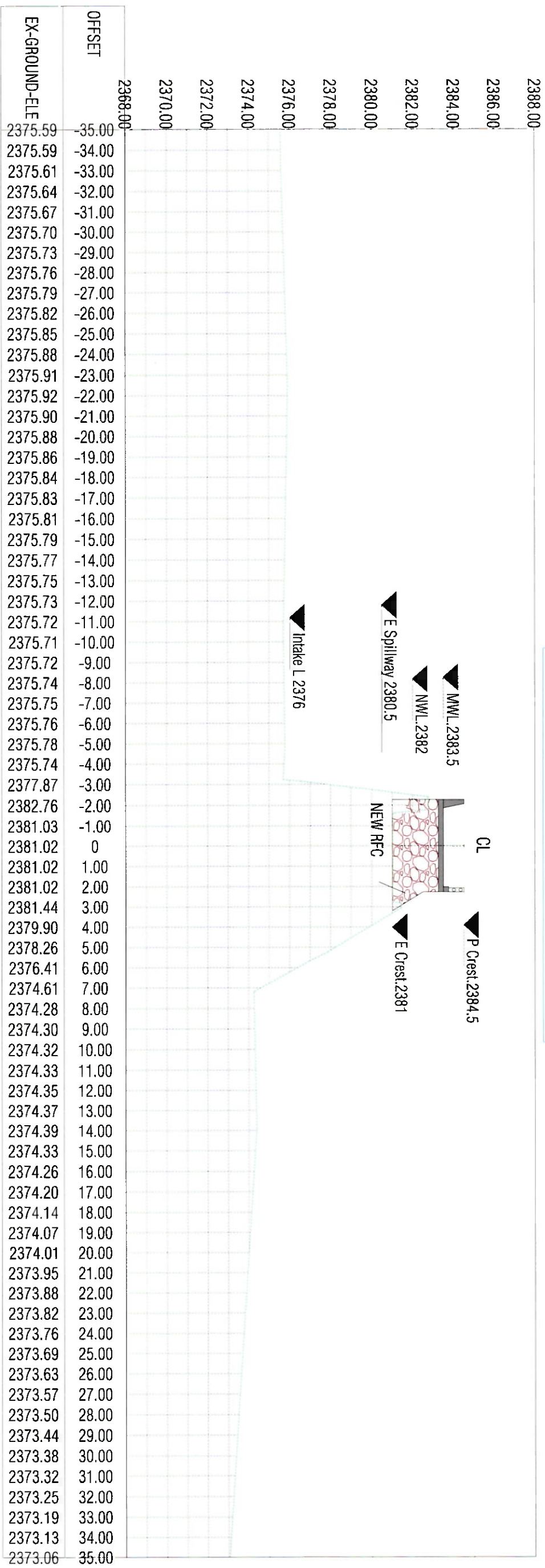


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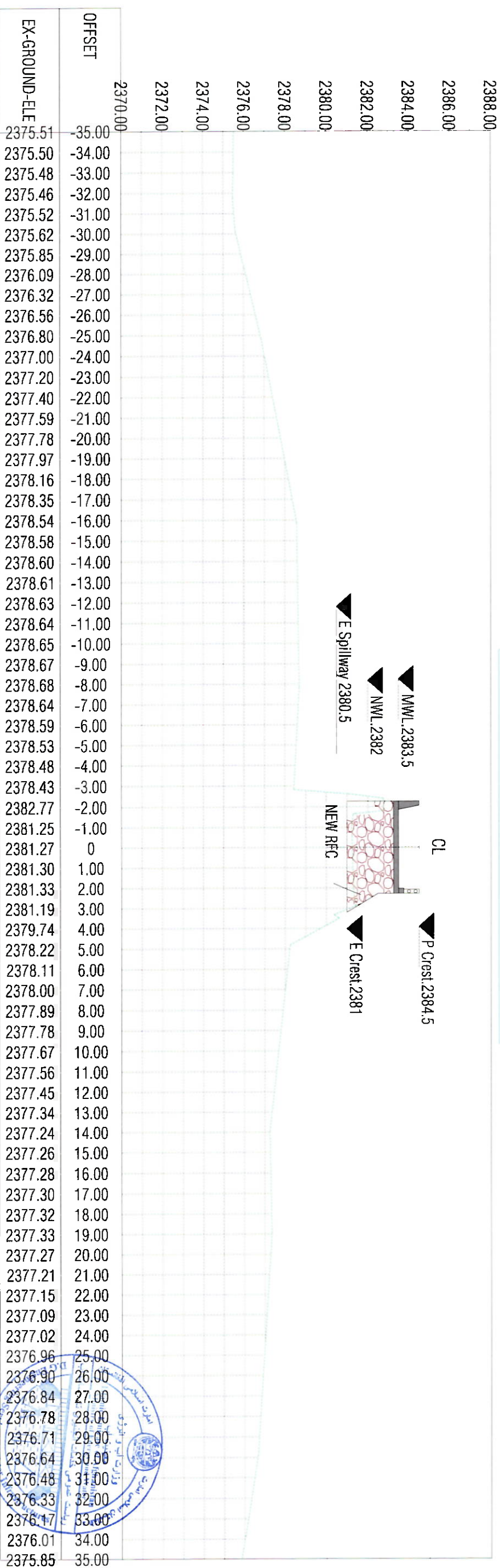


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CROSS SECTION STATION=0+270.00



ISLAMIC EMIRATE OF AFGHANISTAN
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GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD

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PROVINCE
WARDAK

DISTRICT
JAGHATO

DESIGN DATE
NOV 2023

SCALE
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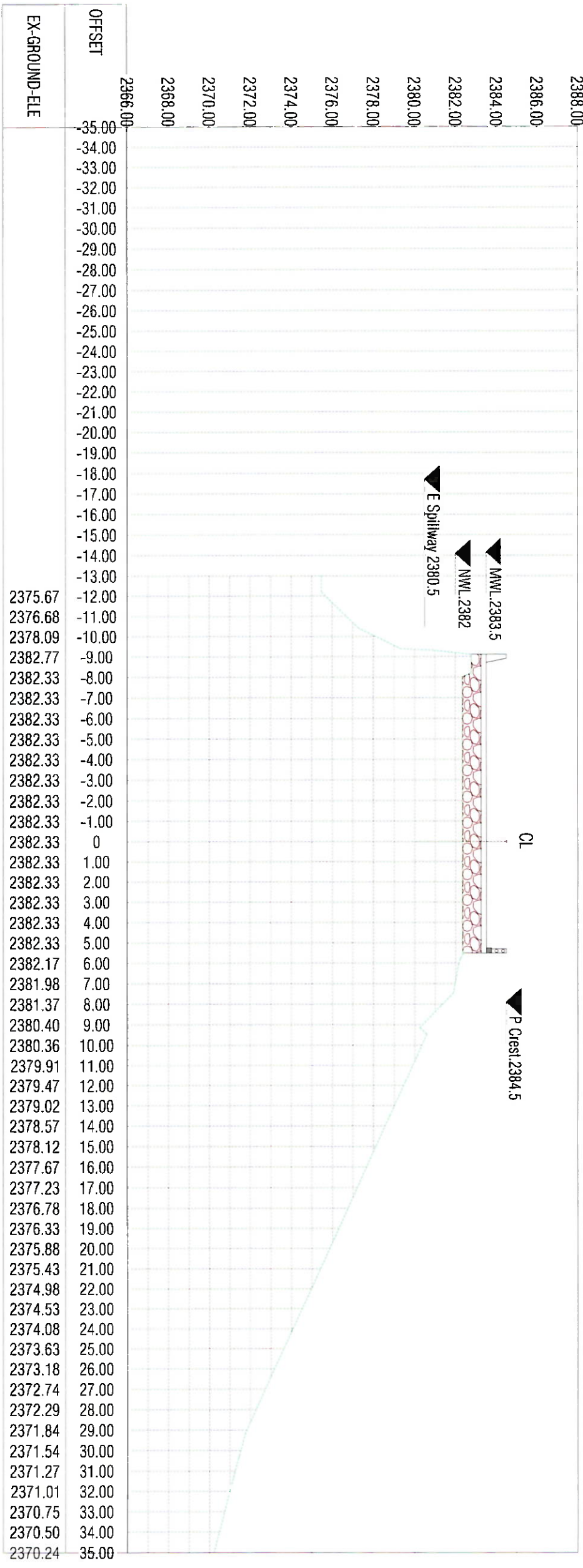
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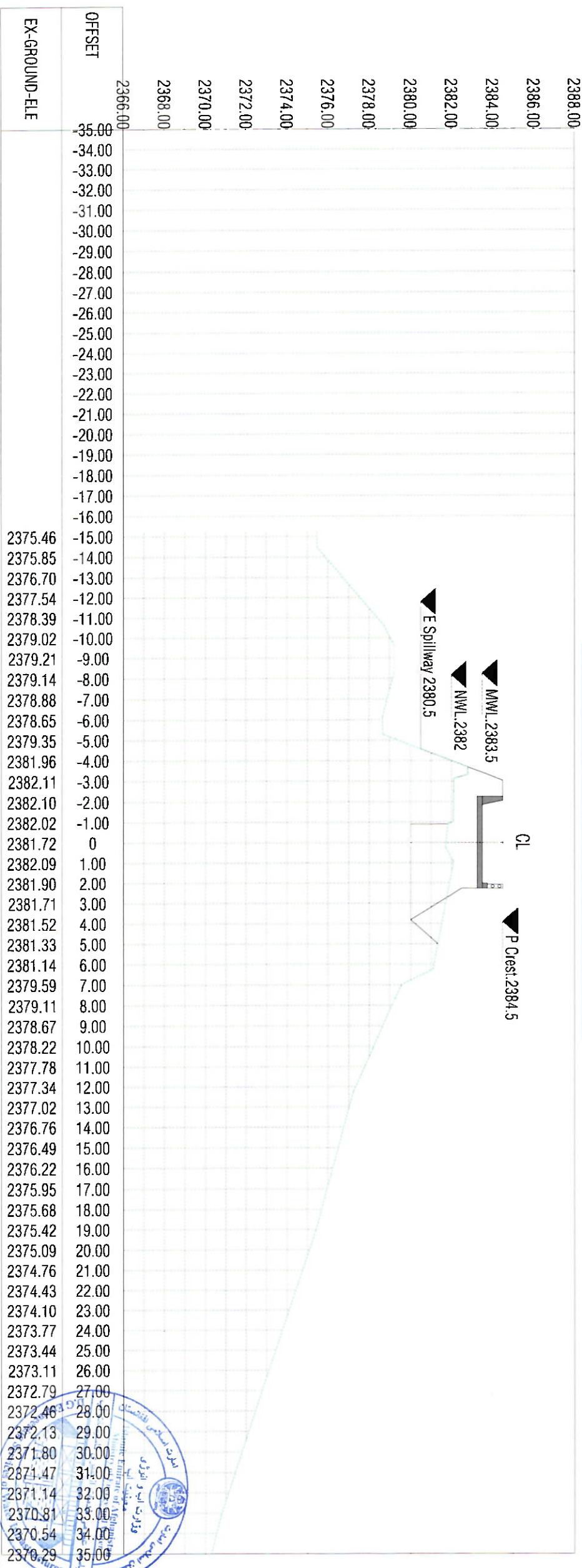
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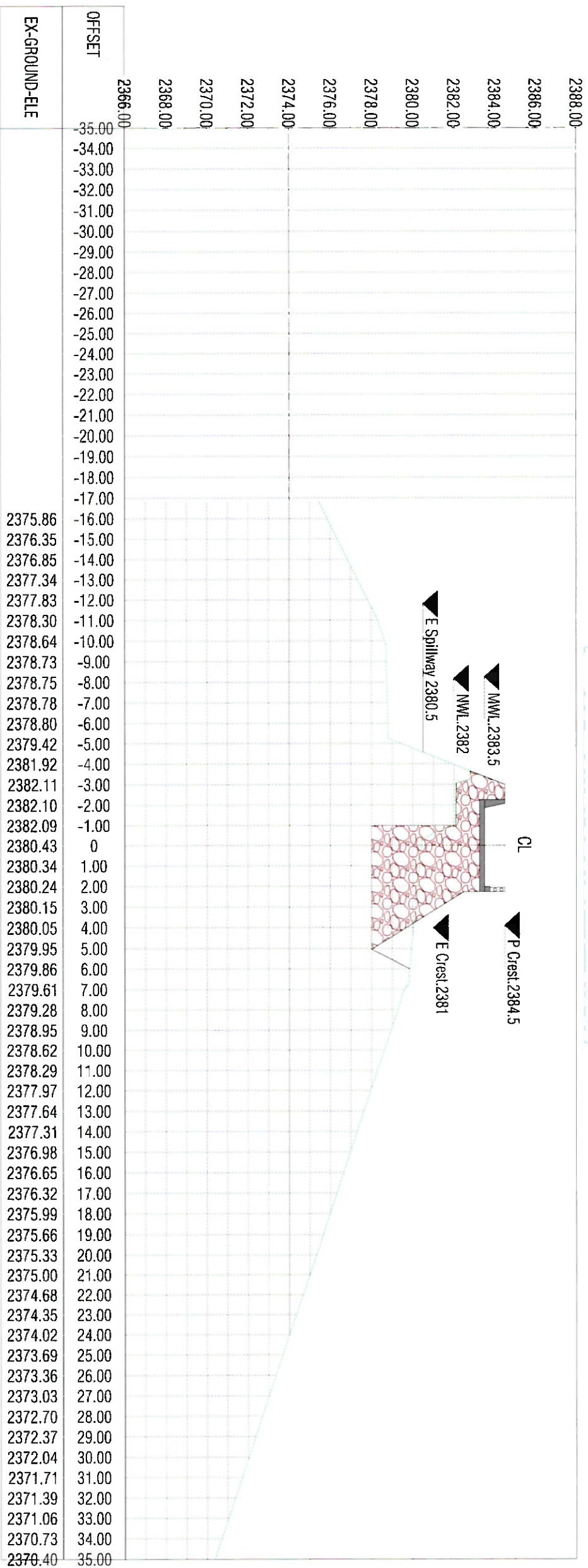
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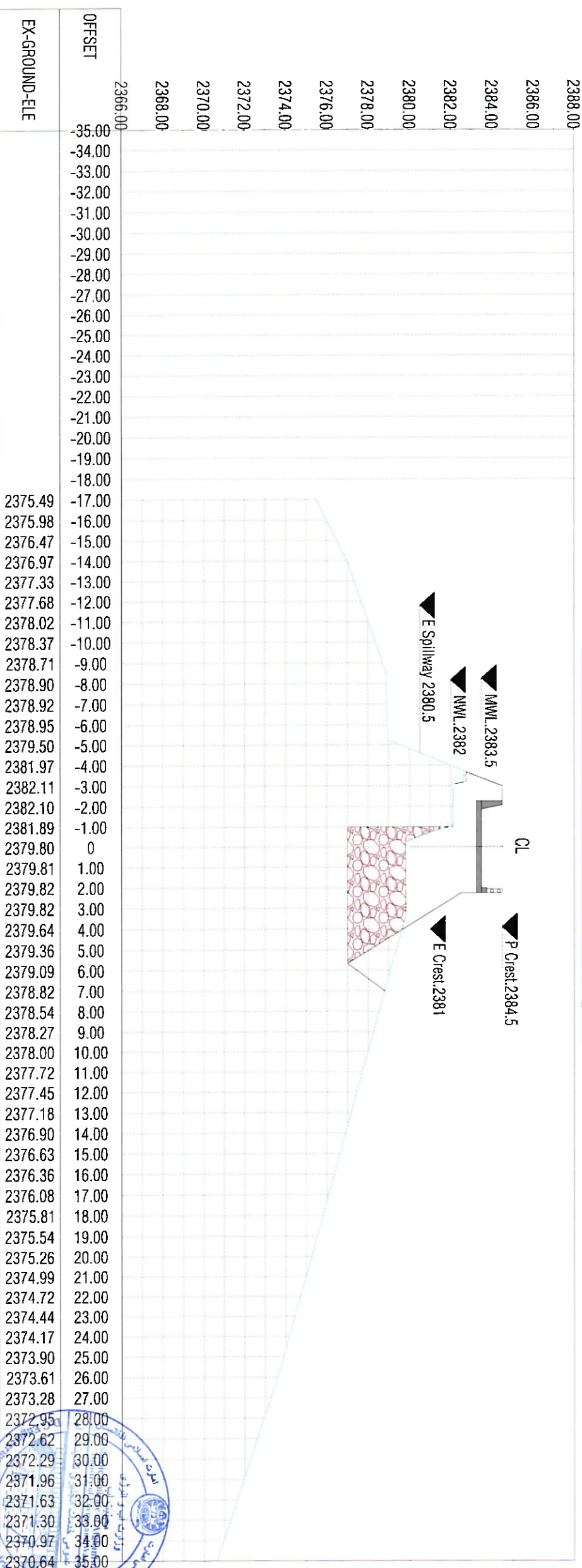
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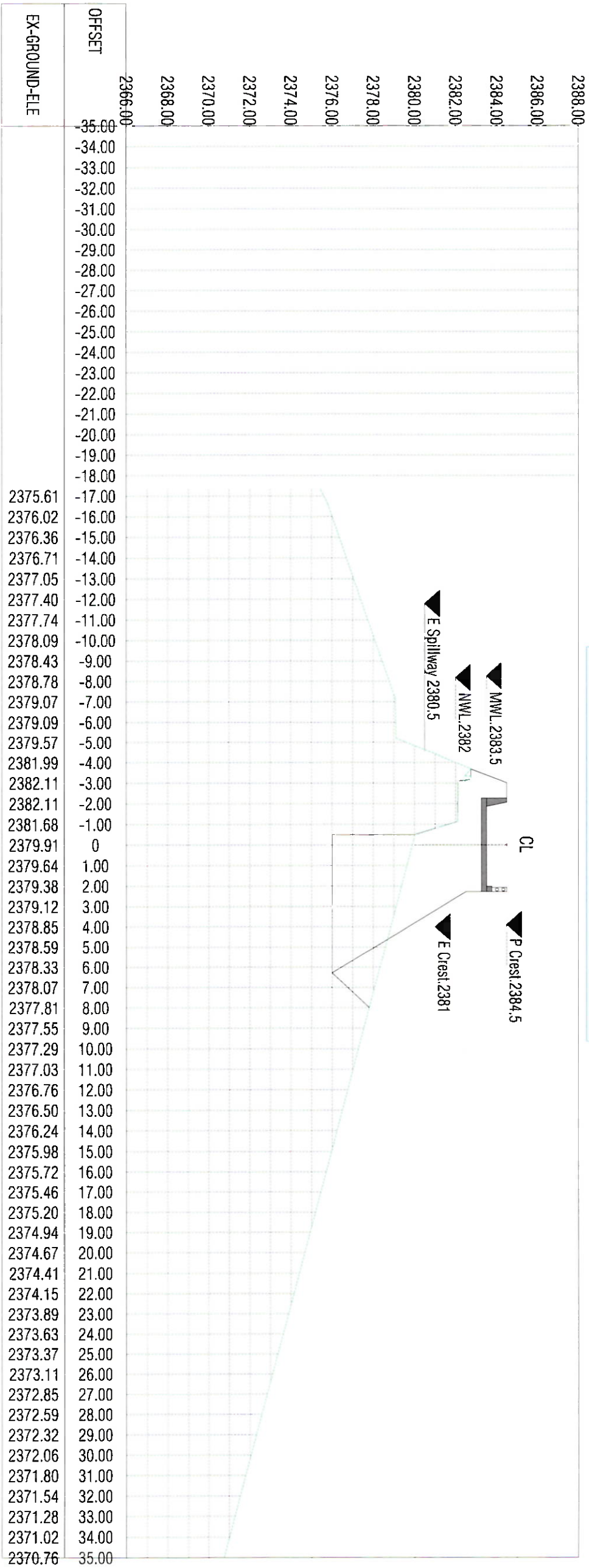
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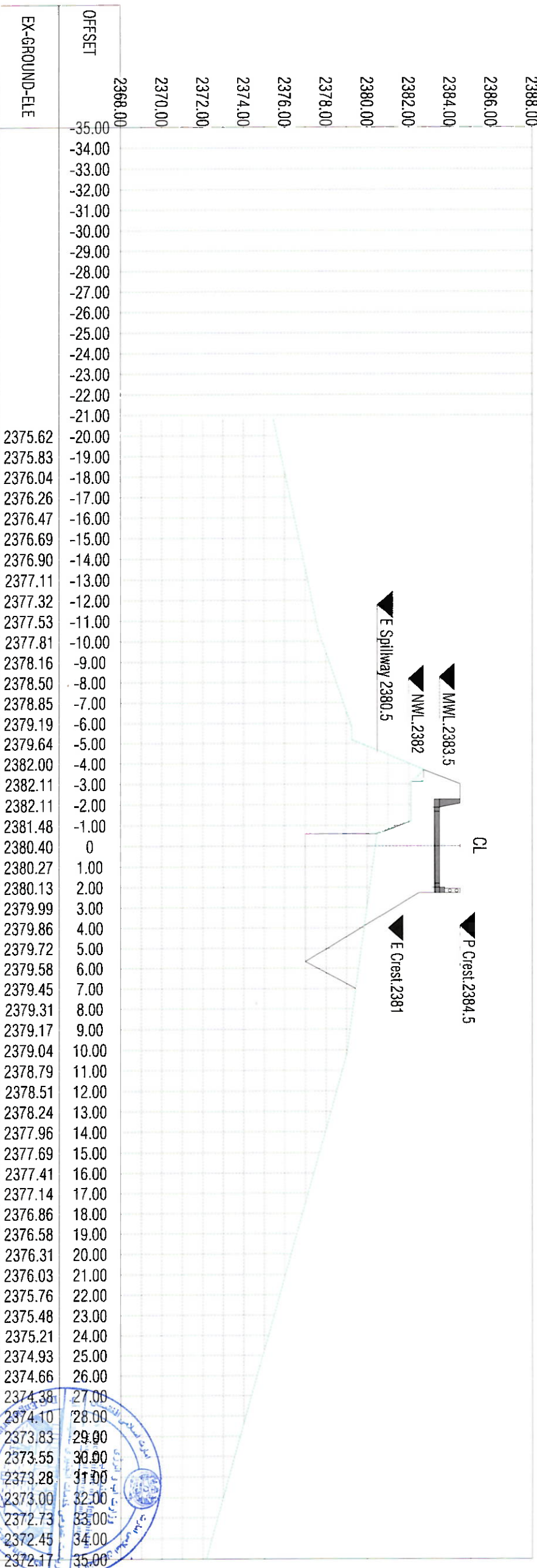
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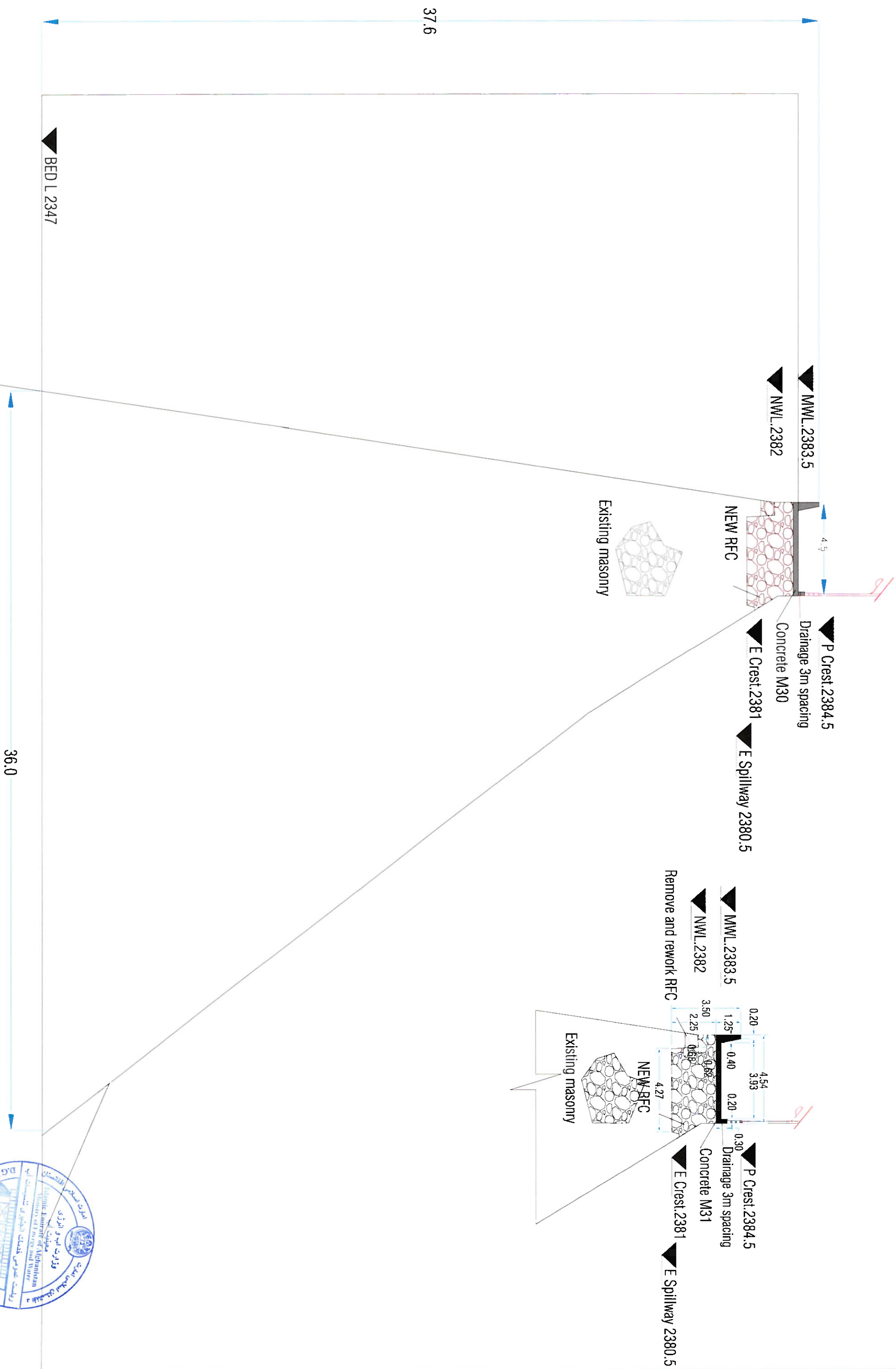


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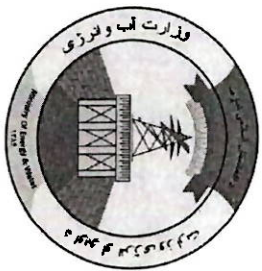
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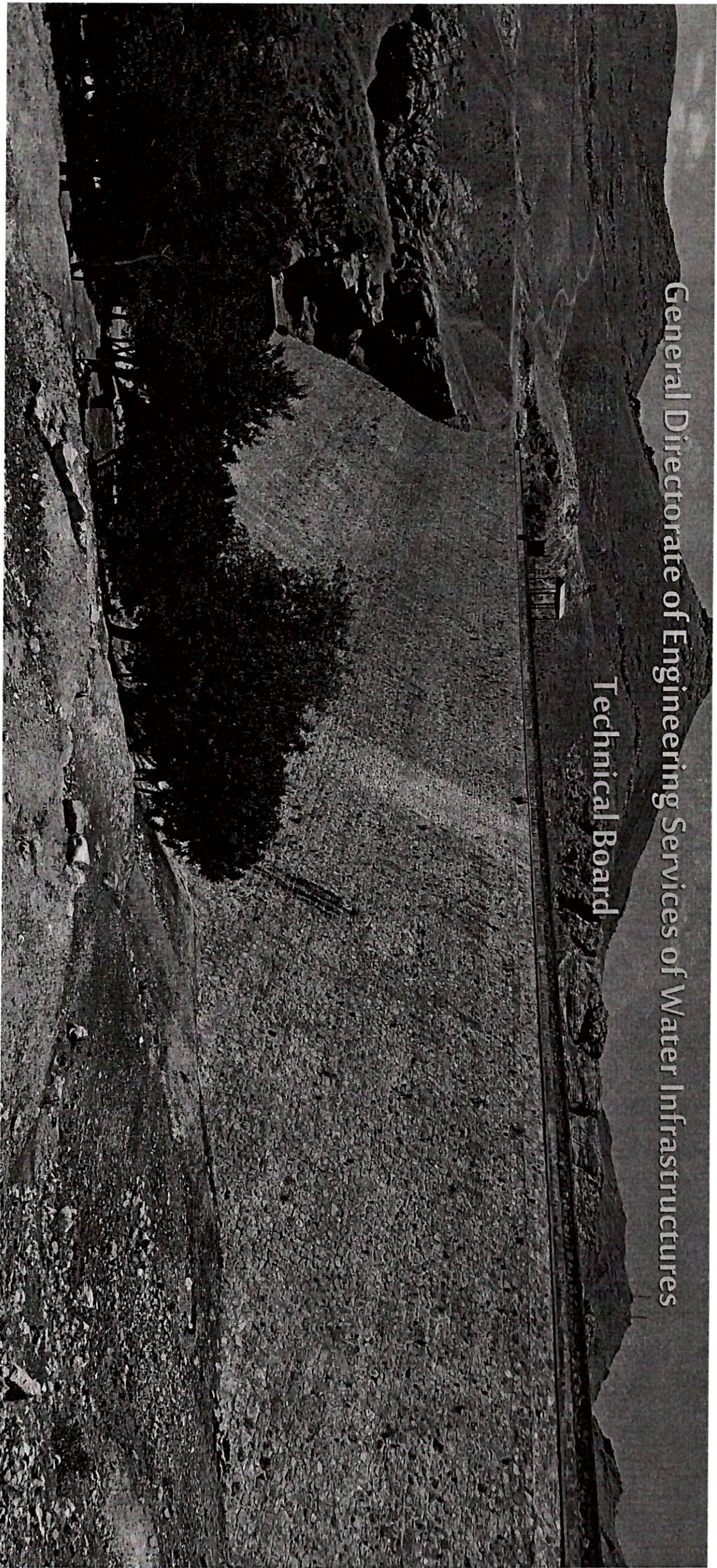
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ISLAMIC EMIRATE OF AFGHANISTAN
MINISTRY OF WATER AND ENERGY
DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD

PROJECT NAME	DRAWING	PROVINCE	DISTRICT	DESIGN DATE	SCALE	DESIGNED BY:	ENG. MAJIBULLAH GHUBAR
SULTAN DAM REHABILITATION	TYPICAL CROSS SECTION	WAPDAK	JAGHATO	NOV 2023	AS SHOWN	CHECKED BY:	ENG. M. TARIQ TASHAL
						APPROVED BY:	
							 
							SHEET NO.
							



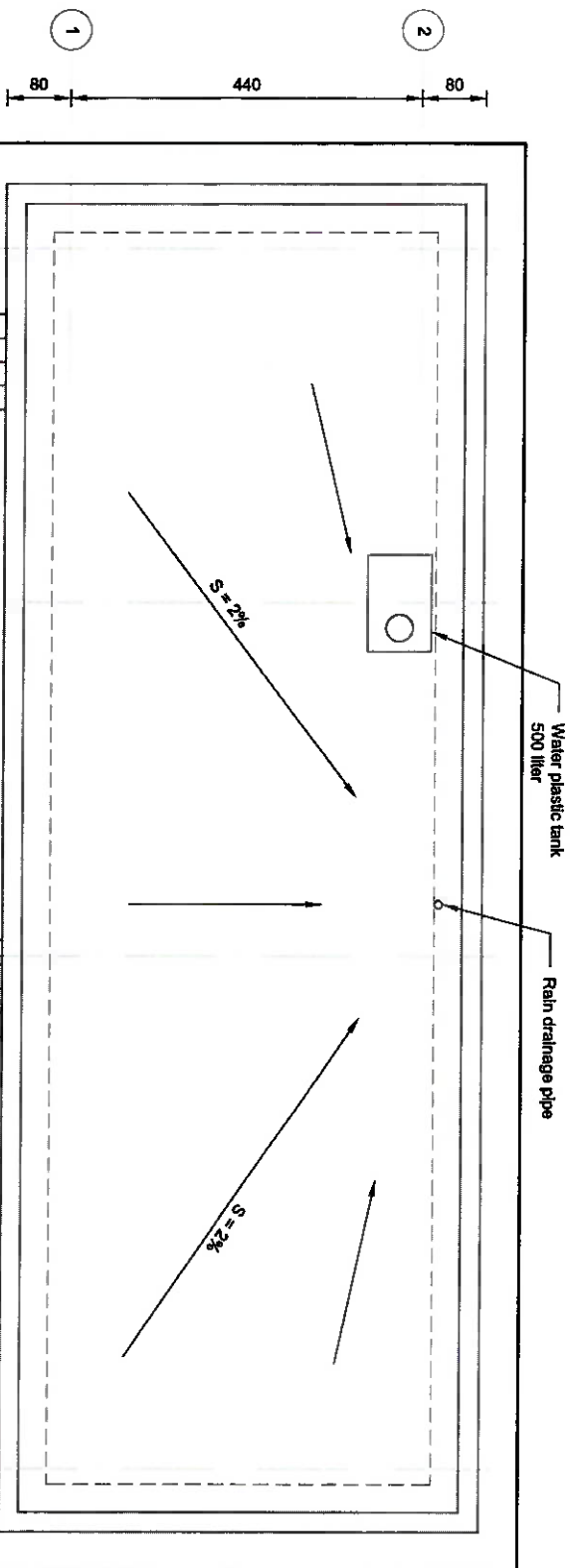
Islamic Emirate of Afghanistan
Ministry of Water and Energy
Deputy Ministry of Water



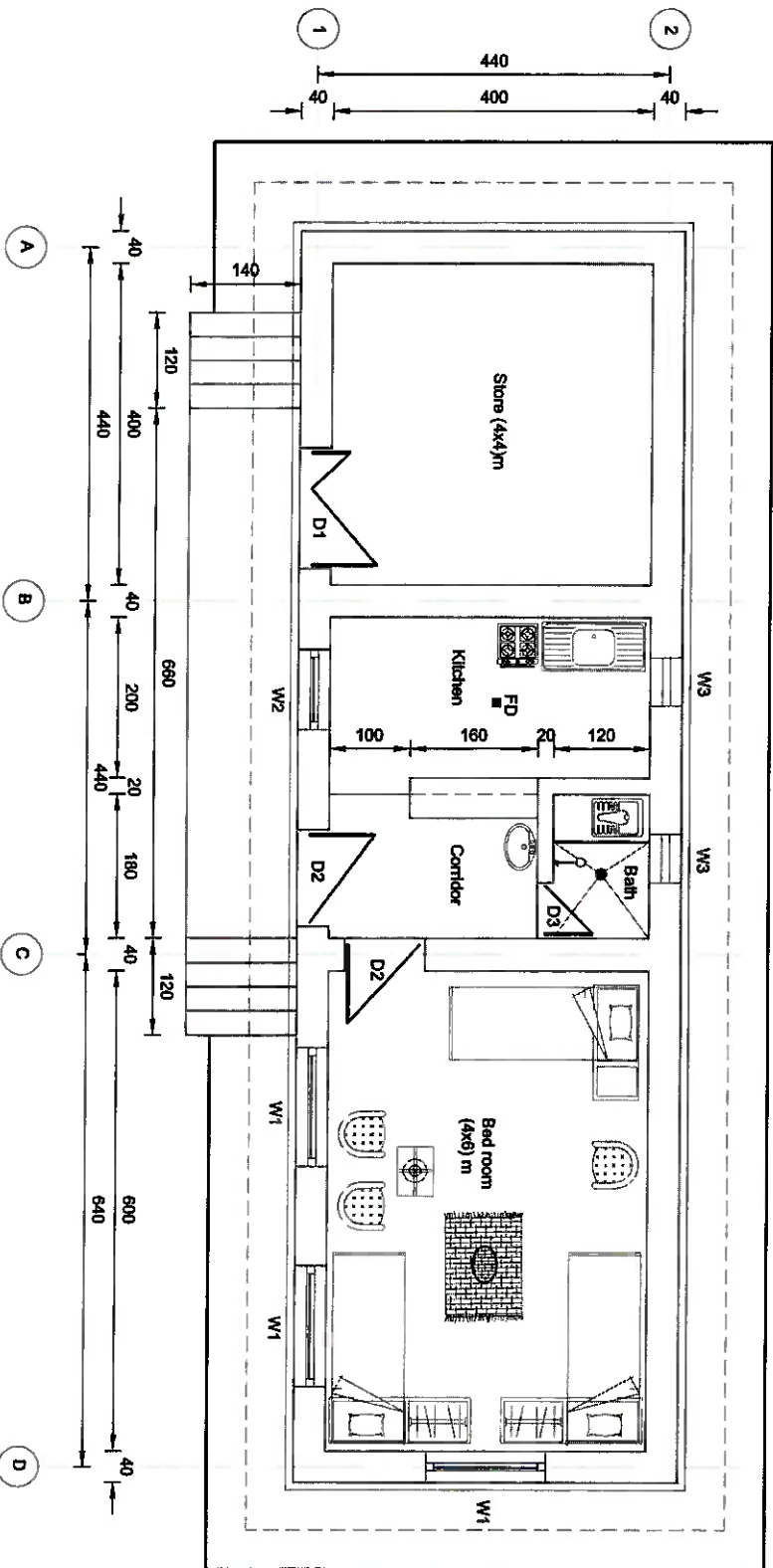
Rehabilitation of Sultan Dam

Date		Nov-2023	
Designed	Checked	Approved	General Directorate of Engineering Services
Qasim.M.Haqyar	Shafuallah		

Guard Room drawing



Roof Plan
Scale: 1:100

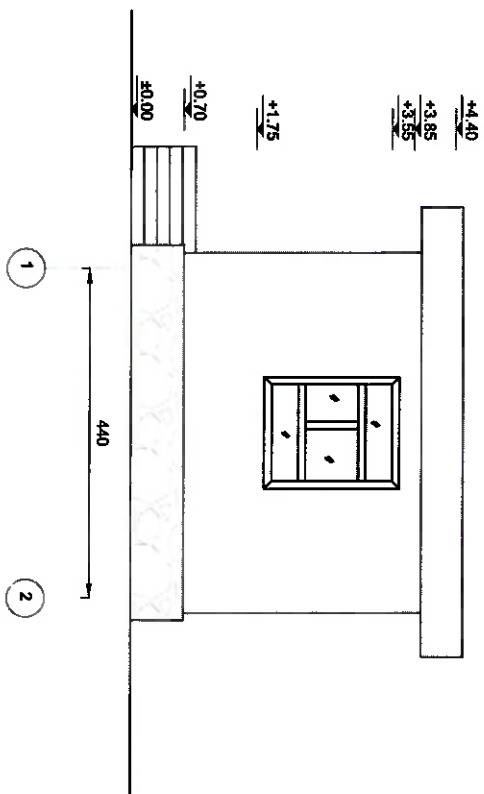
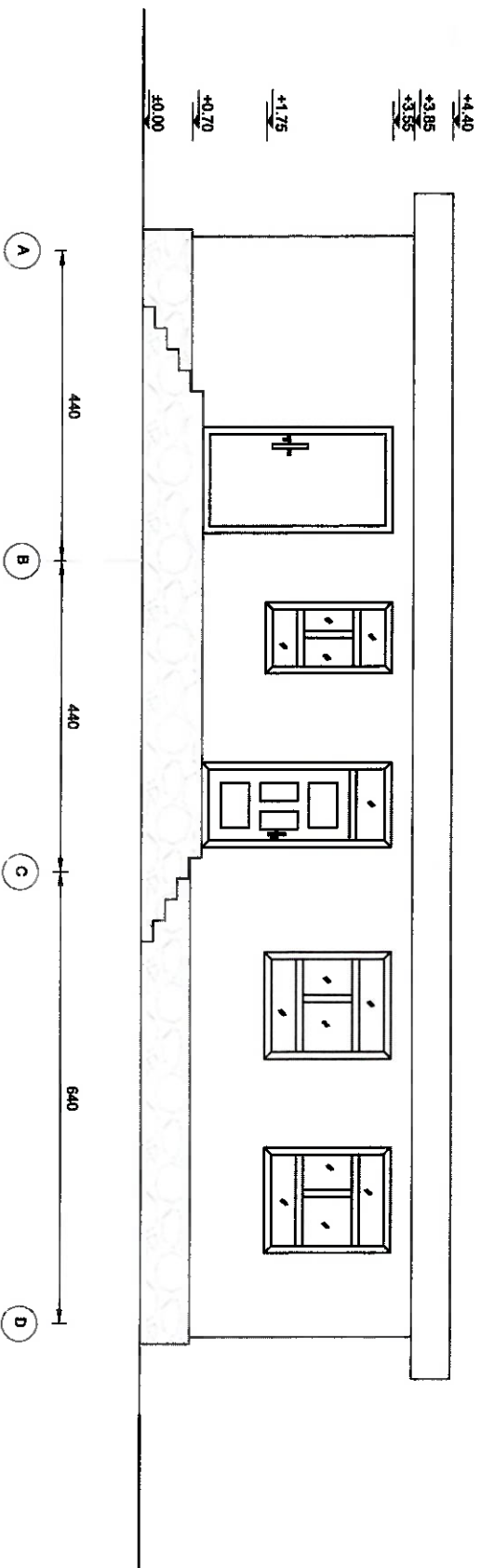
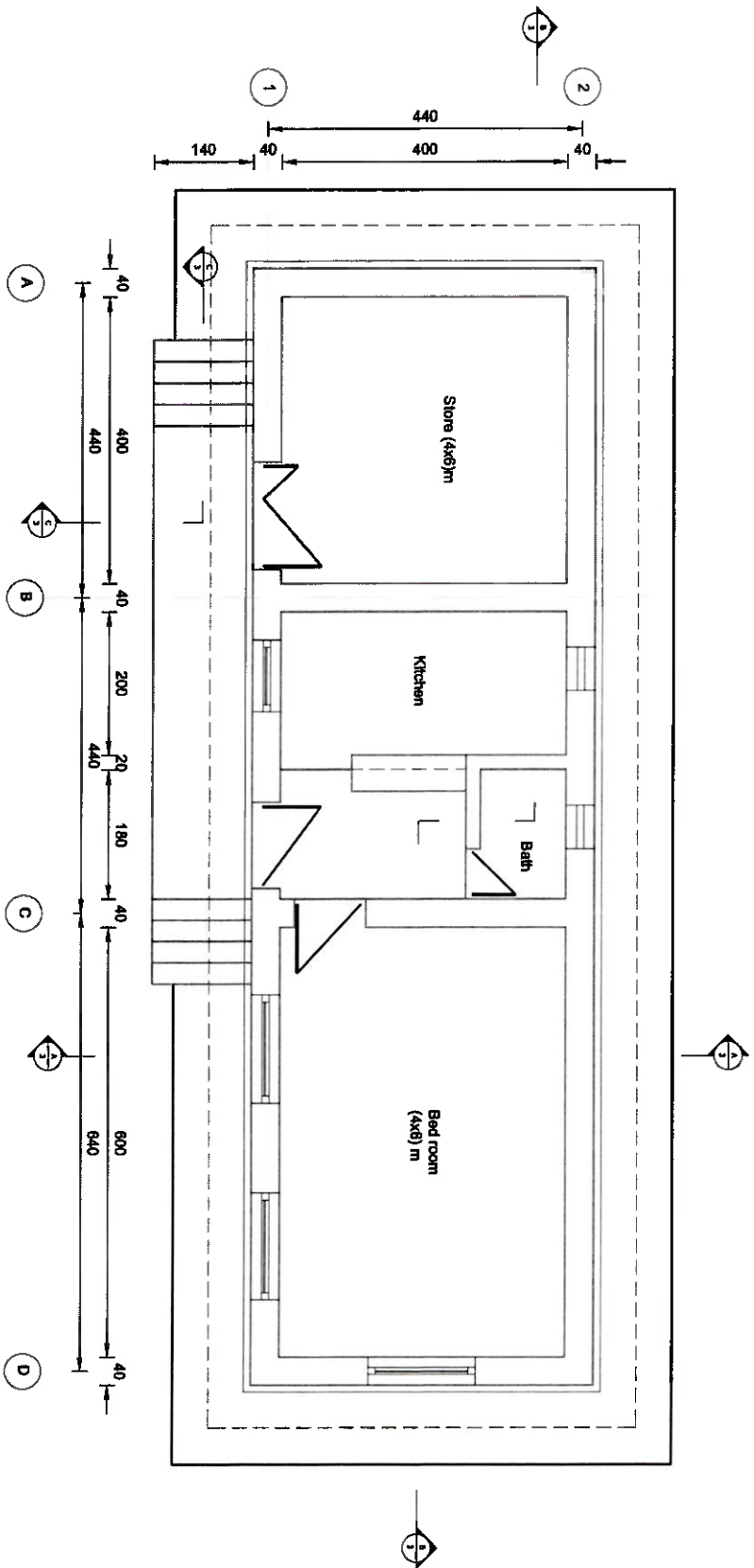



Furniture Plan
Scale: 1:100

Note:
1- The layout shall be adjusted as per site condition to have good sunlight.

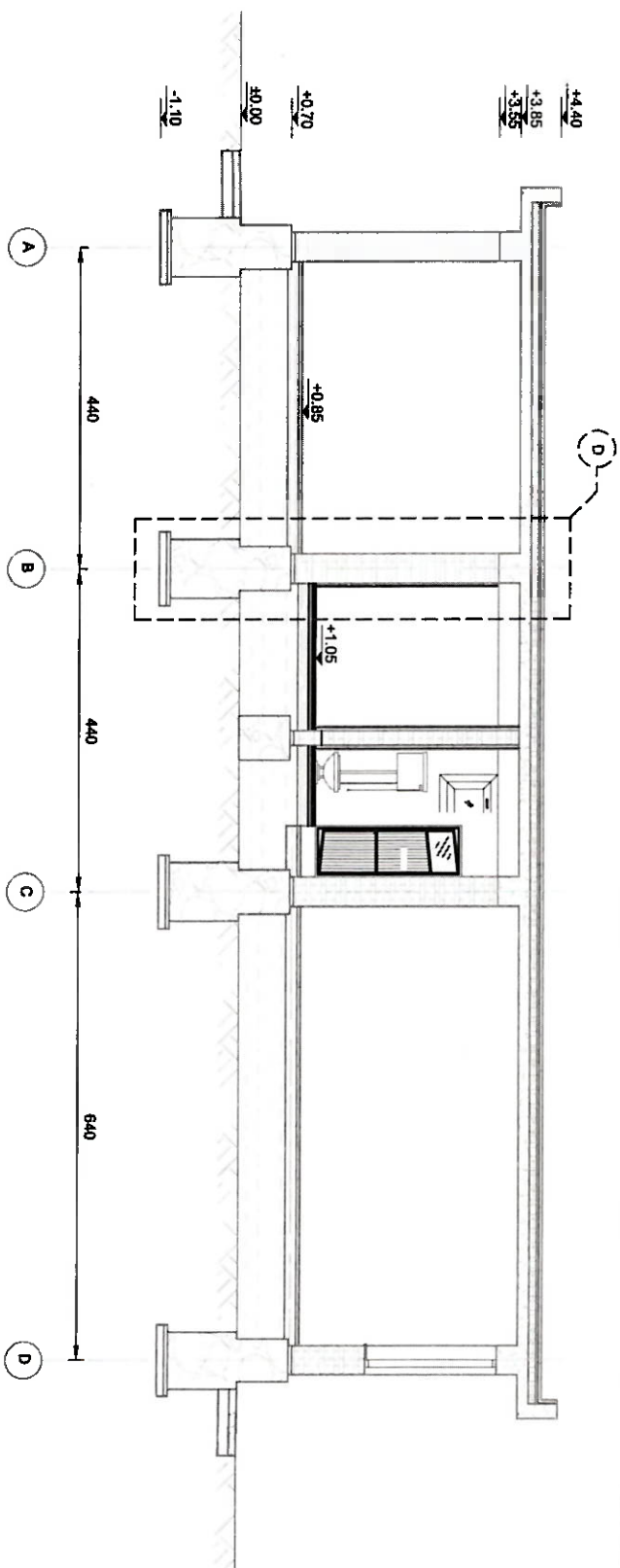
PROJECT NAME	DRAWING	PROVINCE	DISTRICT	DESIGN DATE	SCALE	DESIGNED BY:	CHECKED BY:	APPROVED BY:
Sultan Band Ghazni	Guard Room	Ghazni	Khwaib Umai	Nov-2023	AS SHOWN	Eng. Basim M. Hagar	Eng. Sharifah	



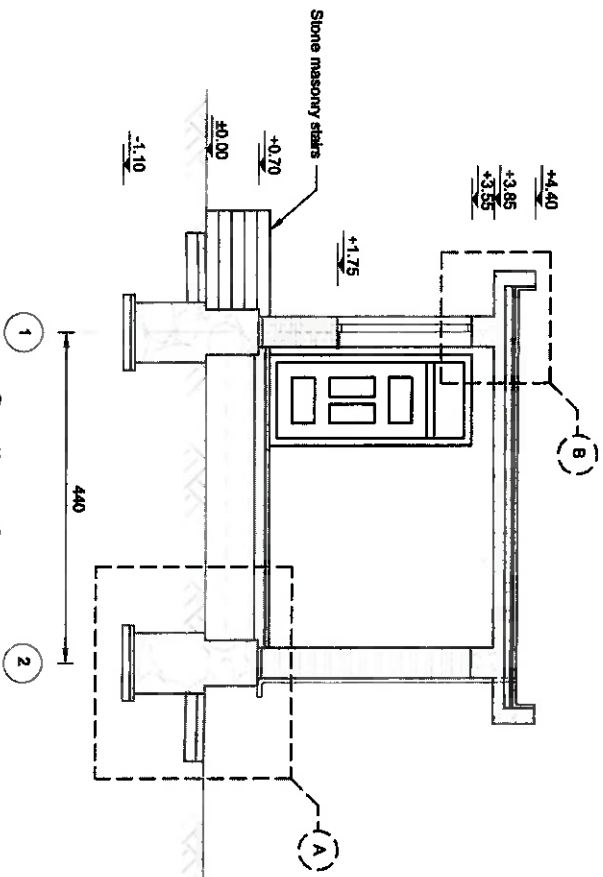


 ISLAMIC EMIRATE OF AFGHANISTAN MINISTRY OF WATER AND ENERGY DEPUTY MINISTRY OF WATER GENERAL DIRECTORATE OF ENGINEERING SERVICES TECHNICAL BOARD		PROJECT NAME	DRAWING	PROVINCE	DISTRICT	DESIGN DATE	SCALE	DESIGNED BY:	CHECKED BY:	APPROVED BY:	SHEET NO
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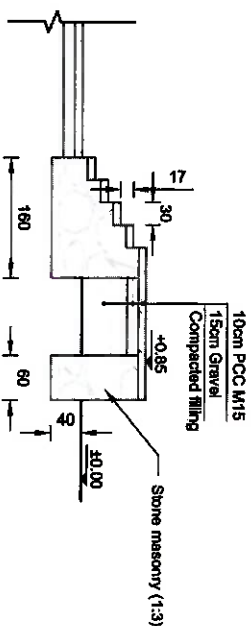




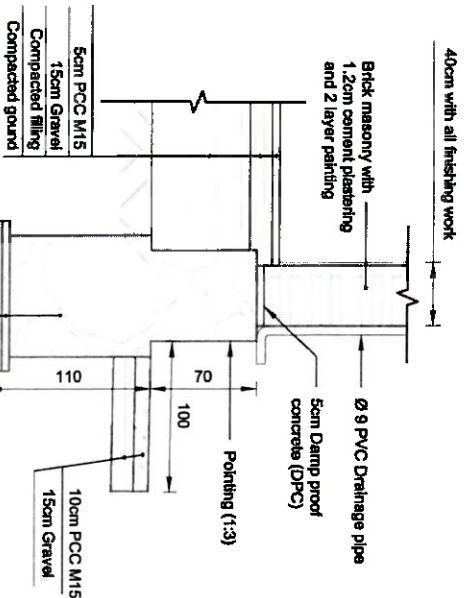
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Scale: 1:100



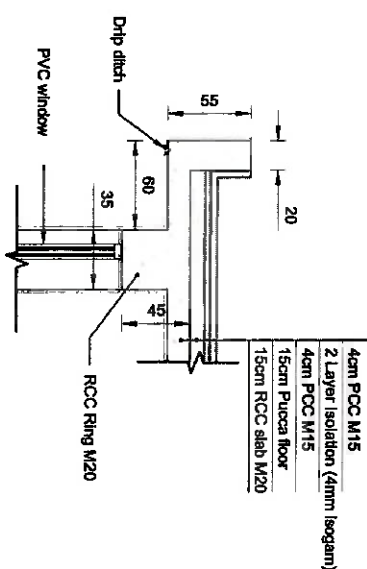
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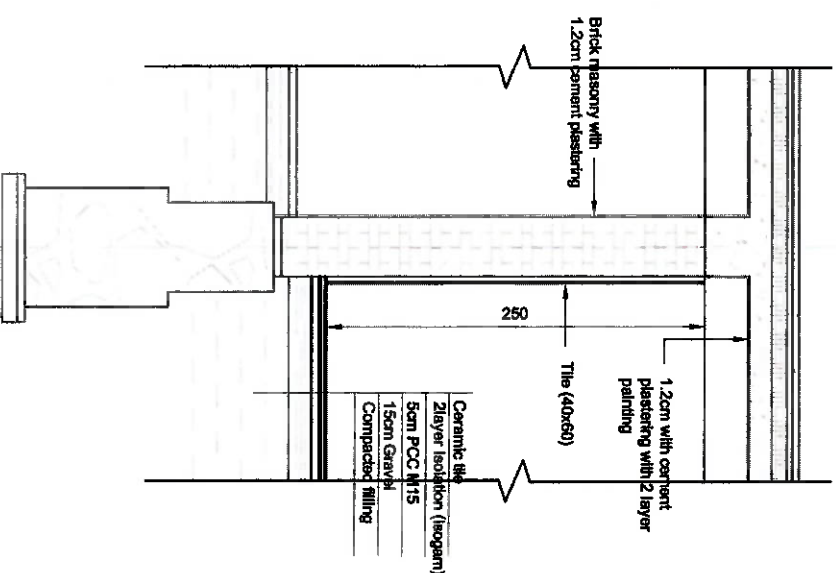
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Scale: 1:50



Detail - B
Scale: 1:50

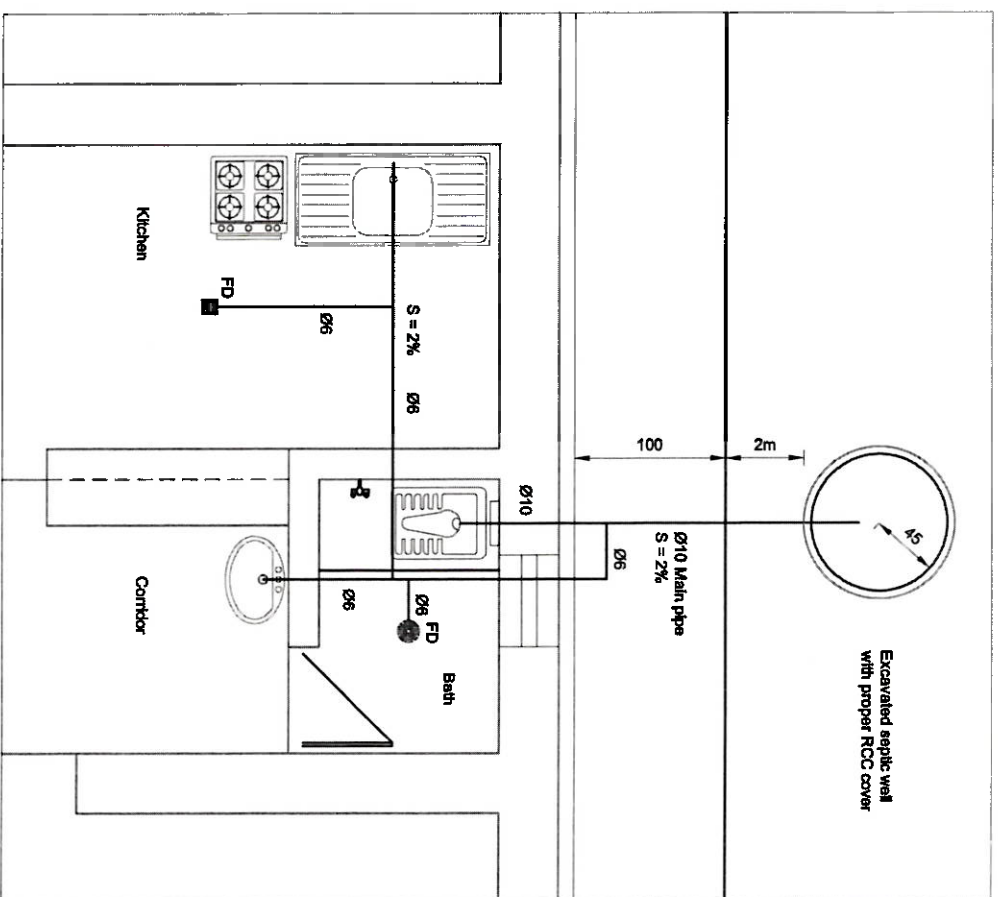


Detail - C
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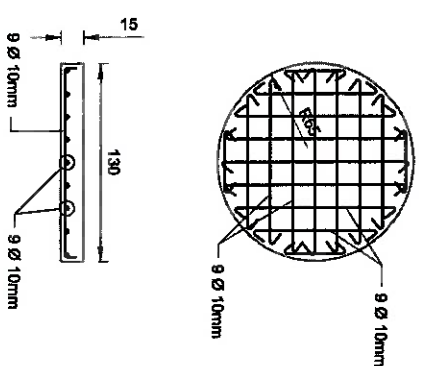
Detail - D
Scale: 1:50

Not: In case of unsuitable foundation, RCC ring with dimensions (0.4x0.3)m can be applied on the top of stone masonry.



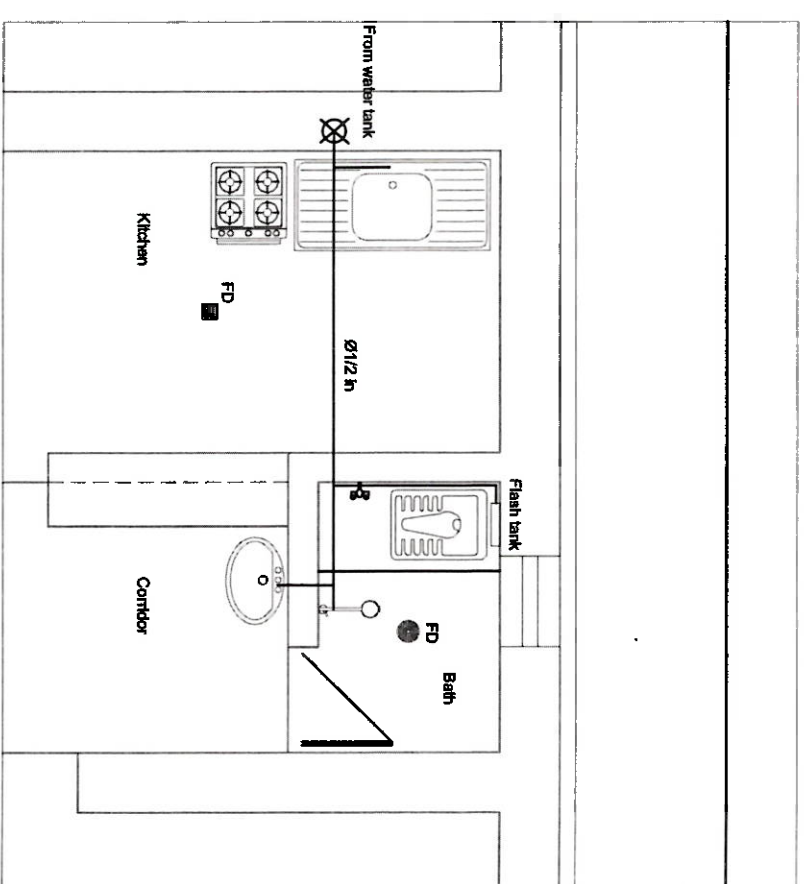
Sewer system plan

Scale: 1:100



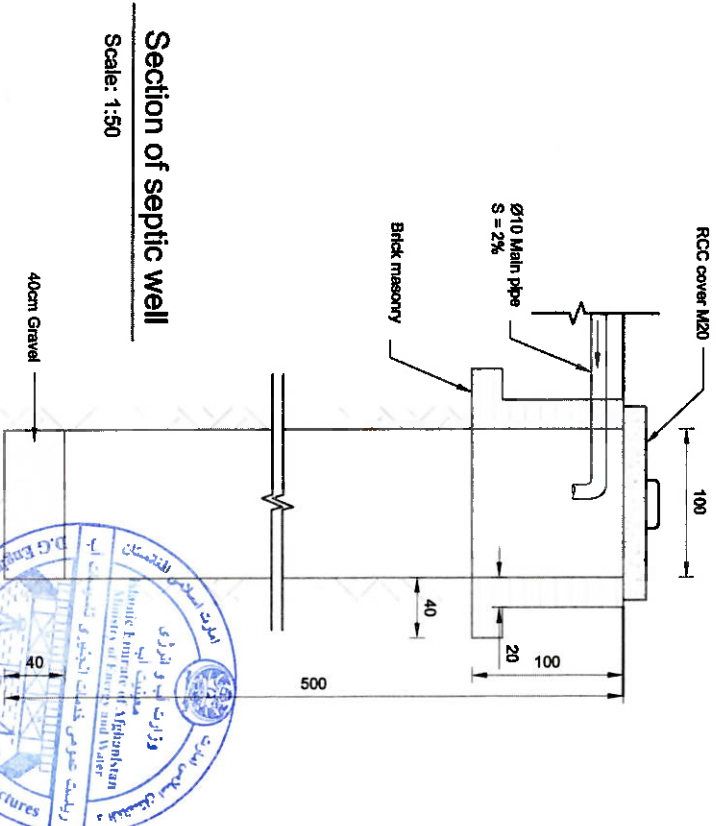
Plan & section of RCC cover

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Water system plan

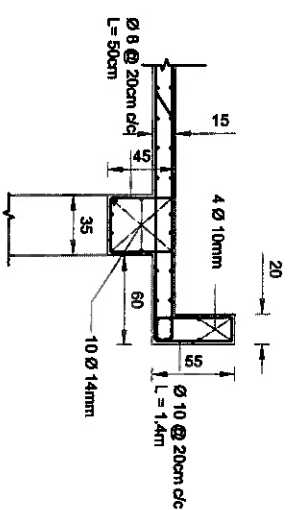
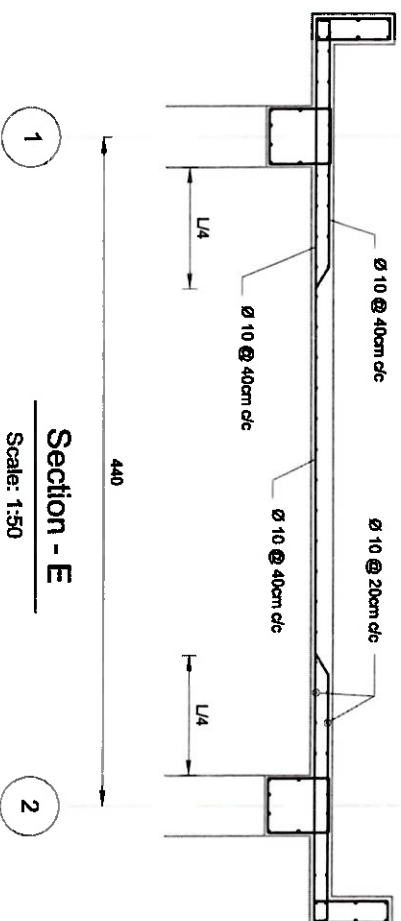
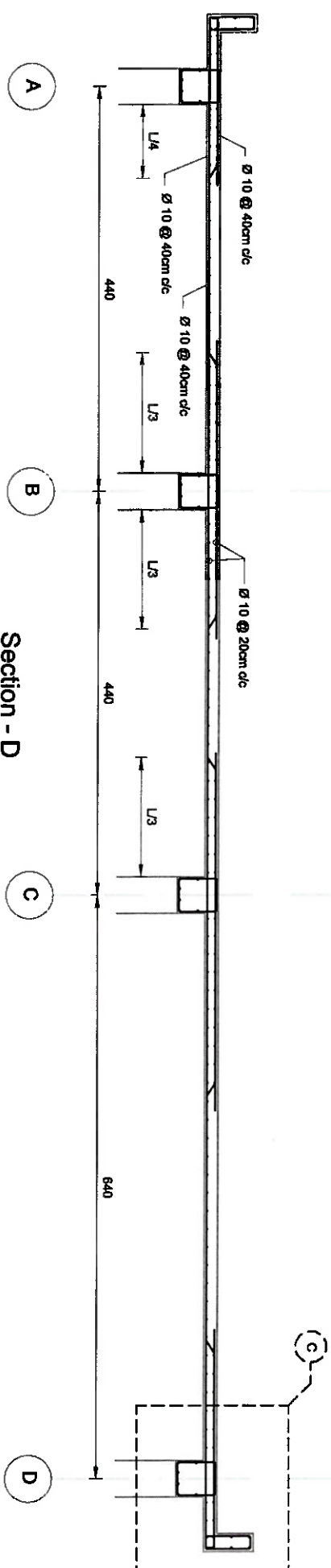
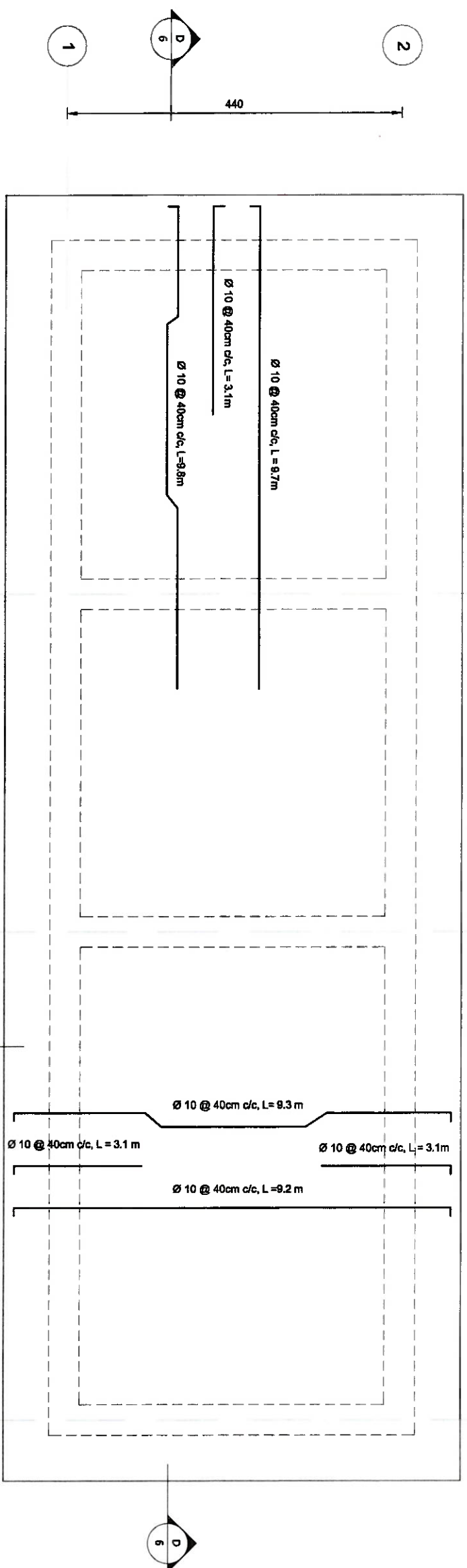
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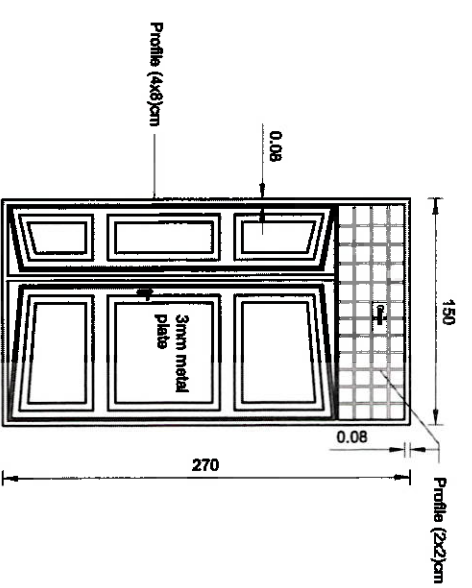
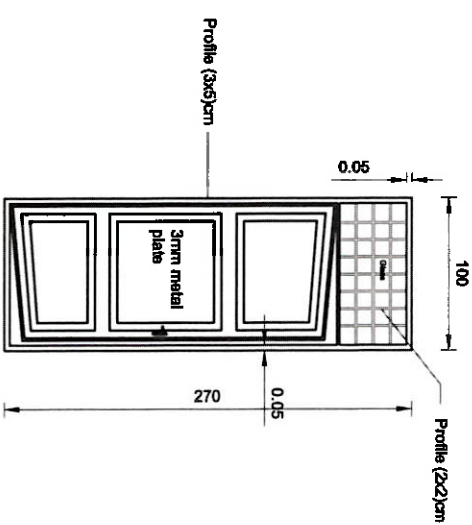
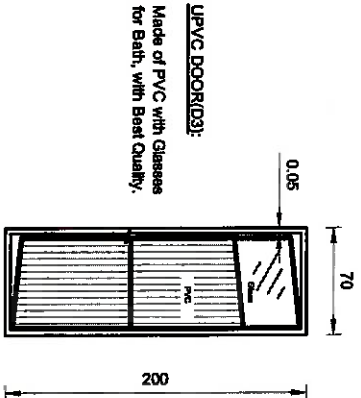


Section of septic well

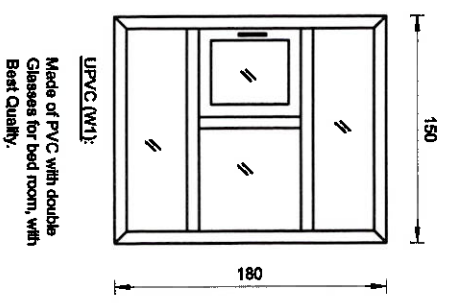
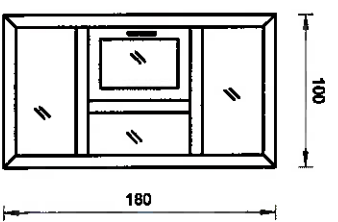
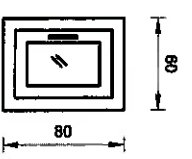
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Door details:



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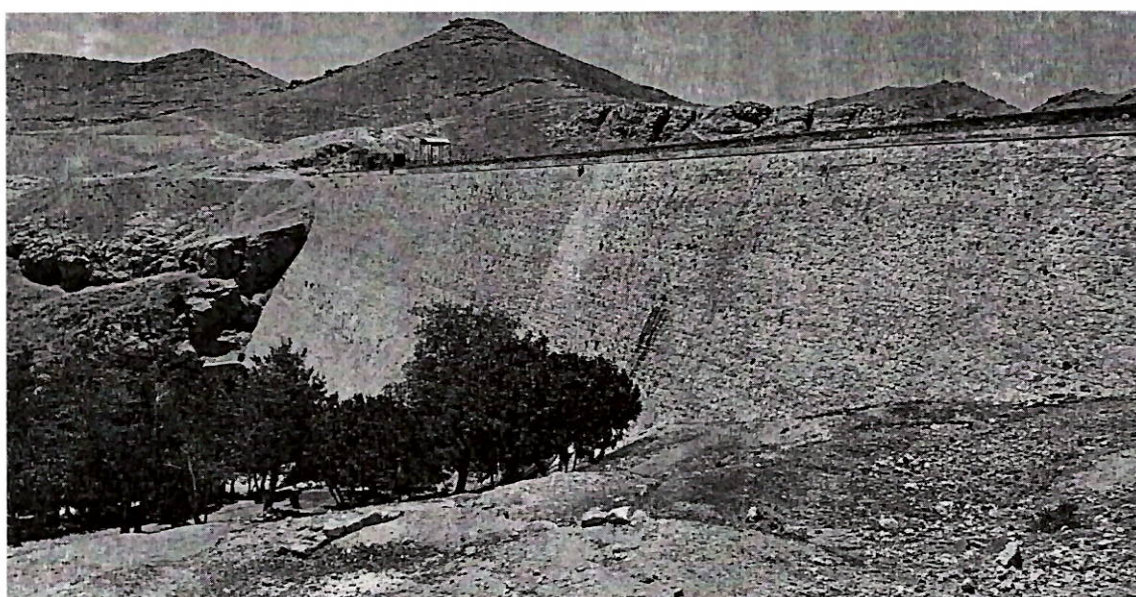




Islamic Emirate of Afghanistan
Ministry of Energy and Water
Deputy Ministry of Water



General Directorate of Engineering Services of Water Infrastructures
Technical Board



Rehabilitation of Sultan Dam			
Date		Sep-2023	Hydraulic Design Report of Spillway
Prepared	Checked	Approved	
Ahmad Sohail Noori	Shafiullah Naseri	Abdul Ghafor Omari	

Hydraulic Design Report of Spillway

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Introduction

The Band-e-Sultan Dam is situated on the river Ghazni, north of the Ghazni town, in the province Wardak, District Jaghato. The dam collects water due to the inflow of Ghazni river and its two tributaries, river Sarab and river Barikab, merging at the upstream end of the Band-e-Sultan Reservoir. River flow is fed from rainfall and snow melting mainly during the spring periods.

The Band-e-Sultan Dam was constructed in 1901 by British Engineers for irrigation of downstream areas. The dam was built at a place of an ancient dam, the ruins of which are still existing in the upstream reservoir.

The dam is a Lime Mortar Stone Masonry Dam with following main dimensions:

- 34 m high
- 200 m long
- 32 m long side spillway on natural rock without stilling basin
- about 200 m long shoulder dam (with 4 m to 10 m high stone masonry over ground)
- 2 pipe outlets each of 1.2 m diameter and equipped with manually driven operating and emergency valves.

Spillway

A spillway is a hydraulic structure that passes normal (operational) and/or flood flows in a manner that protects the structural integrity of the dam and/or dikes (reservoir impoundment structures). Spillways are hydraulically sized to safely pass the Inflow Design Flood (IDF). The IDF will be equal to, or less than, the Probable Maximum Flood (PMF).

For Rehabilitation of Sultan Dam many types and options of controlled and uncontrolled spillways are tried and finally the most suitable and economical option is proposed.

As per literature review and considering topography and geology of the site the flowing options examined:

- 1- Overflow spillway with fixed weir sill, curved, 40 m long and 8 fuse gate elements, each 4.30 m by 2.20 m; no automation system and external power supply required; (Recommended by FITCHNER 2006)
- 2- Gated Spillway within the dam body the includes 4*(6.7mX4.5m) gates (Recommended by EKASU 2021)
- 3- Finally, extension of existing spillway as an uncontrolled chute spillway in right abutment, is considered to be technically and economically the most feasible and safe option. Therefore, in this report we focus on this option.

Scope

The object of spillway design, which involves two steps, is to provide a safe and adequate spillway structure for the lowest combined cost of the spillway and the dam. The first step in the design involves

determining the type and overall size of the spillway structure to suit the anticipated requirements and conditions of the site. A detailed hydraulic and structural design of the spillway structure is the next step. This report is concerned with the general procedure of an overall design. An evaluation of the basic data should be the first step in the preparation of the design. This includes the topography and geology as well as flood hydrography, storage, and release requirements. The type, size, and elevation of the crest and whether it will be controlled can also be tentatively decided. Several alternative arrangements might be possible and a final layout could be created on the basis of economic analysis.

Uncontrolled Chute Spillway

Shore or river bank spillways may be on one side or both sides to the dam abutments or within the bank of reservoir where adequate topographic condition is available. Shore spillways may also be classified by the flow pattern into the types of chutes, side channel, drop inlet, siphon. Chute (proper open channel or trough) spillways is one of the most prevalent types in which control weir is placed approximately perpendicular to the adjoining spillway discharge channel.

By the chute spillway, the flood discharge is conveyed from the reservoir to the downstream river reach through an open channel, placed either along a dam abutment or through a reservoir bank saddle, which might be called as chute-, open-channel-, or trough-type spillway. Factors in favor of the selection of chute spillways are the simplicity in their design and construction, their adaptability to almost any foundation condition, and their overall economy often attributable to the use of large amounts of spillway excavation in the embankment rockfill. Chute spillways have been constructed successfully on all types of foundation materials, ranging from hard rock to soft clay.

A chute spillway normally consists of an entrance (approaching) channel, a control structure, a discharge channel, a terminal structure, and an outlet channel (tail race channel).

The control structure of uncontrolled chute spillway with ogee profile has a control weir that is ogee-shaped (S-shaped) in profile. The upper curve of the ogee structure ordinarily conforms closely to the profile of the lower nappe of a ventilated sheet falling from a sharp-crested weir. Flow over the crest adheres to the face of the profile by preventing access of air to the underside of the sheet. The profile below the upper curve of the ogee is continued tangent along a slope to support the sheet on the face of the overflow. A reverse curve at the bottom of the slope turns the flow onto the apron of a stilling basin or into the spillway discharge channel.

Area- Volume- Height Curve

The reservoir volume calculated from Topographic Survey of Site. The volume and area curve of Paltoni dam is shown below. Normal water level based on Hydrology and Demand of downstream is selected to be 2272.00 m.a.s.l. as shown in Figure-2.



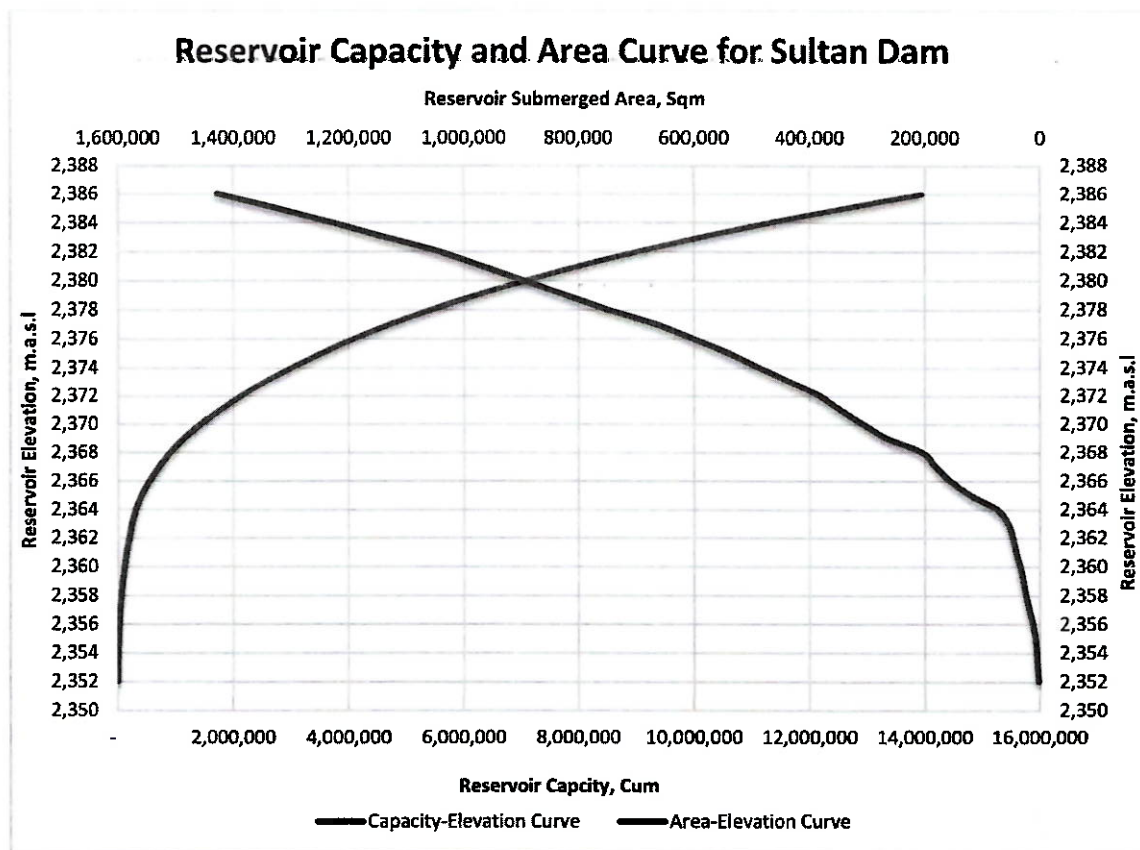


Figure 1: Reservoir Capacity and Area Curve for Paltoni Dam

Spillway Design Flood

Dams impounding large reservoirs on principal rivers with high runoff potential should unquestionably be considered to be in the high-hazard category. For such developments, conservative design criteria should be selected because failure could involve the loss of life or damages of disastrous proportions. Conversely, small dams built on isolated streams in rural areas where failure would neither jeopardize human life nor create damages beyond the sponsor's financial capabilities may be considered to be in a low-hazard category.

Design flood is required for design of spillway which is the structure to pass it safely in the downstream. A cautious and judicious approach is required in selecting the design flood. An over-estimation will result in costly structure and underestimation may endanger the safety of structure. These are based on the size of dam which is defined in terms of hydraulic head (from normal or annual average flood level in the downstream to the maximum water level) and the gross storage. The overall size of dam would be greater of that indicated by either of the two parameters.

Table 1: Predicted Discharges with Different Return Period [Hydrology Report]

Predicted Discharges Summary Table (M ³ /Sec) for Sultan Dam - Ghazni Province					
Return Period (years)	Log Pearson-III (m ³ /sec)	Gumbel (m ³ /sec)	EV1 (m ³ /sec)	Log Normal (m ³ /sec)	Average PMF (m ³ /sec)
2	198	197	195	197	197
5	247	261	246	227	245
10	283	304	279	284	288
25	322	357	321	324	331
50	350	397	353	353	363
100	376	437	384	381	395
200	402	476	415	409	426
500	364	528	456	434	446
1,000	462	568	487	473	498
5,000	515	659	559	536	567
10,000	545	698	590	566	600
PMF	834	1068	903	866	918

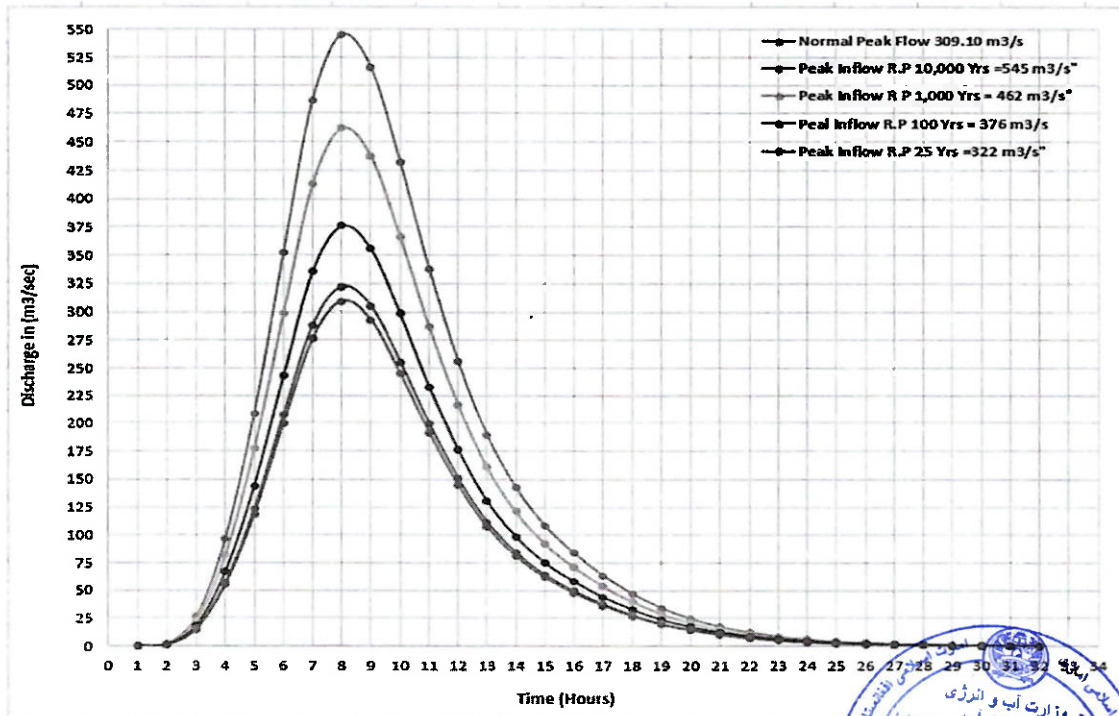


Figure 3: Inflow Design Flood Hydrographs

Table 2: The guidelines for the selection of design flood [Recent Advances in Dam Engineering, B. N. Asthana • Deepak Khare]

Type of dam		Gross storage (Mcm)	Hydraulic head (m)	Design flood
(i)	Small	0.5 to 10	7.5 to 12	in 100 year flood
(ii)	Intermediate	10 to 60	12 to 30	SPF
(iii)	Large	Greater than 60	Greater than 30	PMF

The guidelines for the selection of design flood as per IS:11223 are as below: These are based on the size of dam which is defined in terms of hydraulic head (from normal or annual average flood level in the downstream to the maximum water level) and the gross storage. The overall size of dam would be greater of that indicated by either of the two parameters.

The PMF is used in situations where the failure of the structure would result in loss of life and catastrophic damage and as such complete security from potential floods is sought. On the other hand, SPF is often used where the failure of a structure would cause less severe damages. Typically, SPF is about 40 to 69% of the PMF for the same drainage basin. The criteria used for selecting the design flood for various hydraulic structure vary from one country to another. Table-3 give a brief summary of the guidelines adopted by Central Water Commission (CWC) India, to select design floods.

Table 3: The guidelines for the selection of design flood CWC India.

S. No.	Structure	Recommended design flood
1.	Spillways for major and medium projects with storages more than 60 Mm ³	(a) PMF determined by unit hydrograph and probable maximum precipitation (PMP) (b) If (a) is not applicable or possible flood-frequency method with $T=1000$ years
2.	Permanent barrage and minor dams with capacity less than 60 Mm ³	(a) SPF determined by unit hydrograph and standard project storm (SPS) which is usually the largest recorded storm in the region (b) Flood with a return period of 100 years. (a) or (b) whichever gives higher value.
3.	Pickup weirs	Flood with a return period of 100 or 50 years depending on the importance of the project.
4.	Aqueducts (a) Waterway (b) Foundations and free board	Flood with $T=50$ years Flood with $T=100$ years Empirical formulae
5.	Project with very scanty or inadequate data	

Considering above tables, the reservoir volume is less than 10 M. Cum, however, its height is more than 30m, so, its failure will cause sever damages to downstream villages and towns. As a result, considering economy and life of project, the flood with return period of $T=10,000$ years is selected as

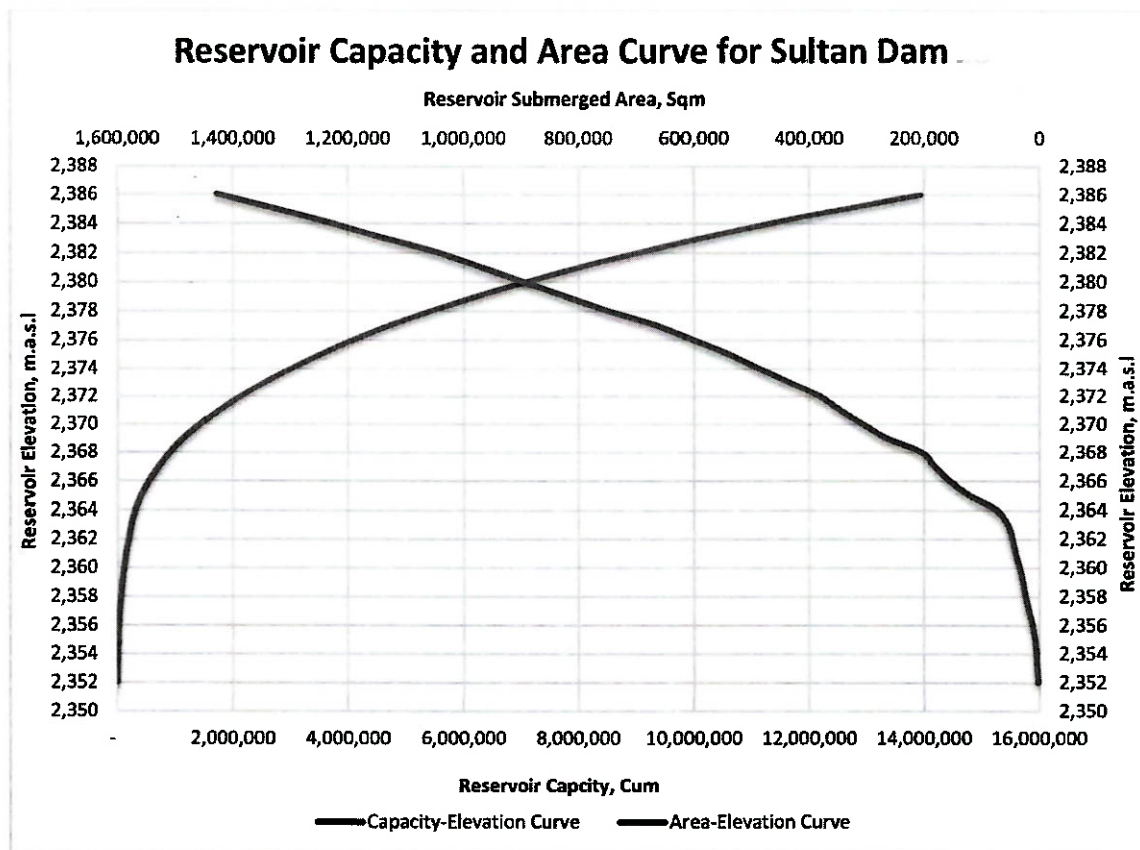


Figure 1: Reservoir Capacity and Area Curve for Paltoni Dam

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design flood which is greater than SPF and 65% of PMF. Additionally, the performance of spillway is checked for PMF, so, the dam designer will ensure that the dam is safe during PMF considering flood water level. Water Resources Council of United States uses the Log-Pearson Type III distribution as a base method.

The risk of failure for flood with $T=10,000$ years is calculated as follow:

$$R_{sk} = 1 - \left(1 - \frac{1}{T}\right)^n = 1 - \left(1 - \frac{1}{10,000}\right)^{100} = 0.0099 \approx 1\%$$

n =life period of dam, 100 years

T =return period of design flood, 1000 years

Spillway Design

Designs of Paltoni Dam's spillway are carried out in accordance with the following issues listed below.

Design Criteria

The reference books and standards that are followed for the design are as follow:

- 1) Design Of Small Dams - United States Department of The Interior - Bureau of Reclamation (USBR).
- 2) Hydraulic Design of Spillways - Engineer Manual 1110-2-1603.
- 3) USBR Design Standards No.14, Appurtenant Structures for Dams (Spillway and Outlet Works).
- 4) Recent Advances in Dam Engineering (B.N. Asthana, Deepak Khare).
- 5) Hydraulic Structures (Sheng-Hong Chen).
- 6) Hydraulics of Spillways and Energy Dissipators (R. M. Khatsuria)
- 7) Hydraulic Structures and Irrigation Engineering (S. K. Garg)
- 8) Irrigation Engineering and Hydraulic Structures (S. K. Ukurande)

Discharge Capacity Calculations

Discharge Capacity Service Spillway (Ogee Spillway)

Design discharge of an overflow spillway can be determined by integrating the velocity distribution over the cross-sectional flow area on the spillway from the crest to the free surface. The resulting equation is given below,

$$Q = C L H_0^{3/2}$$

Q = discharge,

C = variable discharge coefficient,

L = effective length of crest, and

H_a = actual head being considered on the crest, including velocity of approach head, h_a.

Restriction of Head: The design head of spillway is restricted to 2m for keeping more irrigation water writing reservoir. It means that higher head over crest of uncontrolled spillway will strongly affect the active volume of reservoir which defect the profitability and overall economy of dam. So, the aforementioned restrictions caused lengthy spillways with lower head.

The preliminary length of spillway in the first trial calculated as follow:

$$L = \frac{Q}{C \times H^{3/2}} = \frac{545}{2.12 \times 2.5^{3/2}} = 65, \text{ Accep, } C=2.12 \text{ assumed.}$$

The right abutment of dam is a hill of hard rock, so, lengthy spillway will cause huge quantity of hard cutting. Therefore, a Duckbill spillway is proposed for economy of the project.

The duckbill spillway is trapezoidal plan form and corner angel of 120° and total length of 70m, however its width is 47m in plan.

The first step is to calculated the Approach Channel Losses:

Since $\frac{P}{H_0} = \frac{1}{2.5} = 0.40 < 1.33$, so, the spillway is considered to be low spillway and the approach velocity has effects. Therefore, for width B=47m, height H=2.5m, Manning's n=0.025 and average length of approach is=30m the effect of approach velocity is calculated as below:

To evaluate the approach channel losses a value of C=2.12, P=1m and H=2.5m is assumed to obtain the approximate approach velocity.

The design discharge per unit of crest length is equal to $q=545/47=11.6 \text{ m}^3/\text{sec}$. $H_e+P=2.5+1=3.5\text{m}$.

$$\text{Velocity of Approach, } V_a = \frac{q}{H_e+P} = \frac{11.6}{3.5} = 3.314 \text{ m/sec}$$

$$\text{Approach Velocity Head, } h_a = \frac{V_a^2}{2g} = \frac{3.314^2}{2 \times 9.81} = 0.56 \text{ m}$$

Area of Approach Channel, $A=(2.5+1) \times 47=164.5 \text{ sqm}$, Wetted Perimeter, $P=54\text{m}$, Hydraulic Radius, $R=3\text{m}$, Manning $n=0.025$, so the friction losses calculated as follow:

$$h_L = \frac{n^2 \times V^2 \times L}{R^{4/3}} = \frac{0.025^2 \times 3.314^2 \times 30}{3^{4/3}} = 0.0476 \text{ m}$$

Entrance Losses equal to $0.1 \times h_a = 0.056$

Total Approach Losses = $0.0476 + 0.056 = 0.1 \text{ m}$



The effective head is equal to $H_e = 2.5 - 0.1 = 2.4\text{m}$

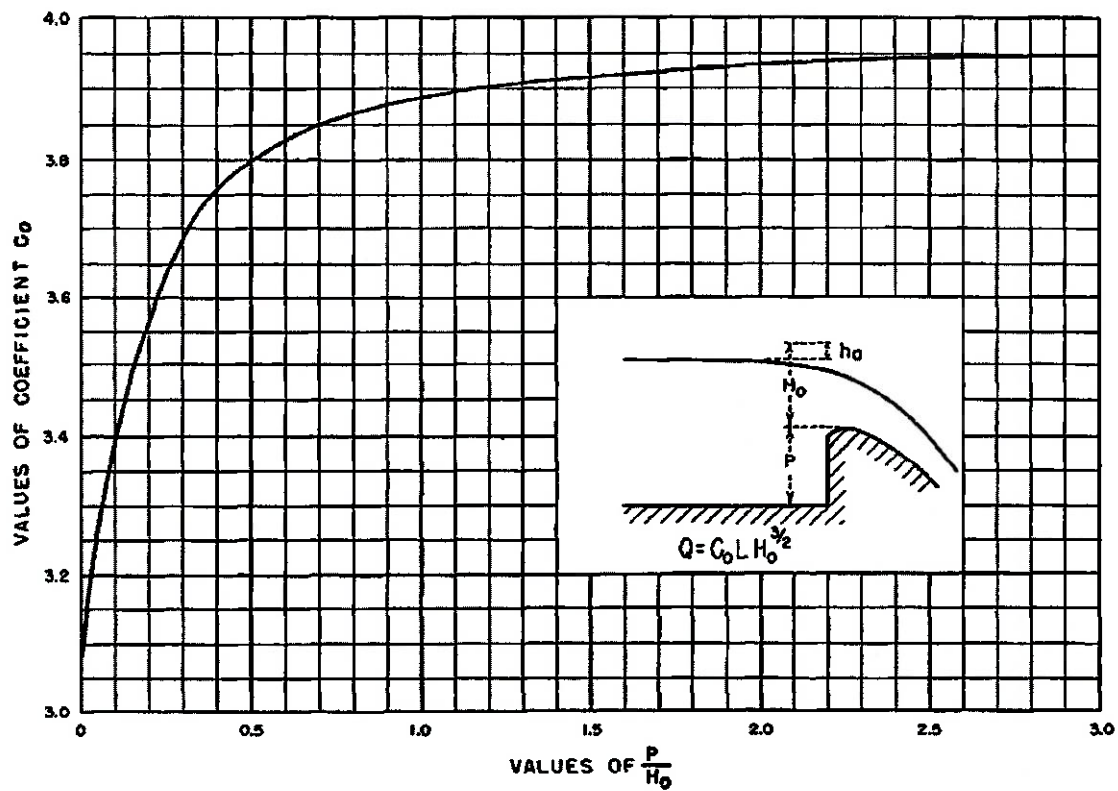


Figure 2: - Spillway Discharge Coefficient for Vertical Faced Crest (USBR, 1987)

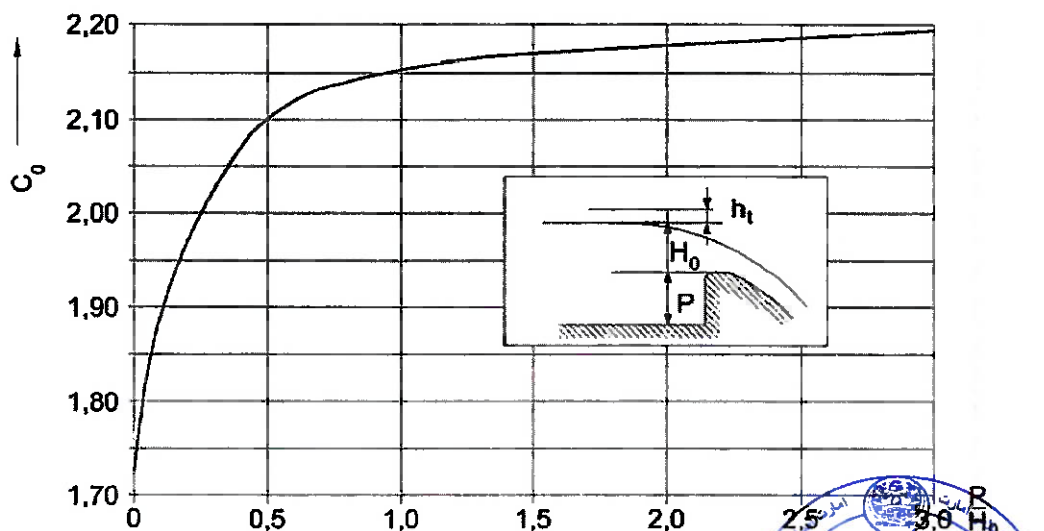


Figure 3: - Coefficients of discharge \times relation P/H_0 (Chow 1959)

From figure 1.2 for $\frac{P}{H_0} = \frac{1}{2.4} \cong 0.416$, $C_0=3.79$ (from Figure 2) $=3.77*0.552*=2.081$ (Metric)

Discharge coefficient is converted to metric system in all spillway hydraulic calculations; however, the metric version is available in other references (Figure 3).

Upstream slope of spillway is 3:3, so the effect of upstream face slop is considered as follow:

$$\frac{P}{H_0} = 0.416, \frac{C_{inclined}}{C_{vertical}} = 1.018 \text{ (from Figure 4)} \Rightarrow C_{inclined} = 2.09 * 1.018 \cong 2.12$$

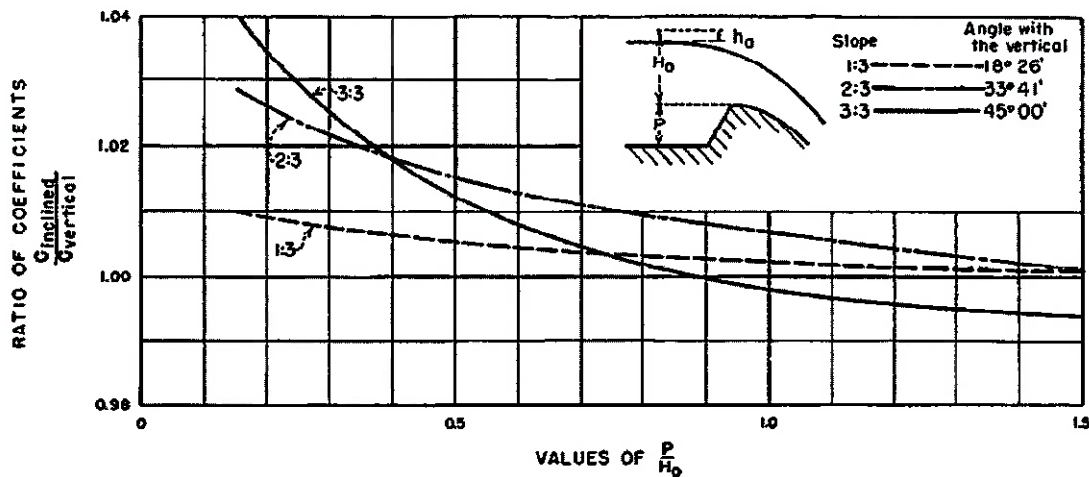


Figure 4: - Spillway Discharge Coefficient for Sloping Upstream Face (USBR,1987)

The effective length of spillway, L_e (m) can be calculated from the following formula:

$$L_e = L' - 2 \times (N K_p + K_a) \times H_0$$

where L' is the net crest length. Contractions in flow induced by the presence of piers and abutments are considered in coefficients K_p and K_a , respectively and N is the number of piers. The pier contraction coefficient K_p depends upon shape and location of pier nose, thickness of the pier and approach velocity. The abutment contraction coefficient K_a depends upon shape of abutment, angle between upstream approach wall and the axis of flow and approach velocity. Based on the recommendations given in USBR,1987, the pier contraction coefficient, K_p , and the abutment contraction coefficient, K_a , are assumed to be 0.01 and 0.1, respectively.

$$L'=70m, N=0, K_p=0.01, K_a=0.1 \Rightarrow L_e = 70 - 2 * (0 * 0.01 + 0.1) * 2.4 = 69.52m$$

Effect of downstream apron interference and downstream submergence is considered from the following graph as below:

$$\frac{h_d + d}{H_e} = \frac{4.5}{2.4} = 1.875$$

$$v = \sqrt{2 \times g \times (H - h)} = \sqrt{2 \times 9.81 \times (2.5 + 1 + 1 - 1.5)} = 7.67 \text{ m/s} \quad d = \frac{q}{v} = \frac{11.6}{7.67} = 1.5$$

$$h_d = 4.5 - 1.5 = 3 \text{ m}, \quad \frac{h_d}{H_e} = 1.25$$

The decrease in the coefficient of discharge is 0%.

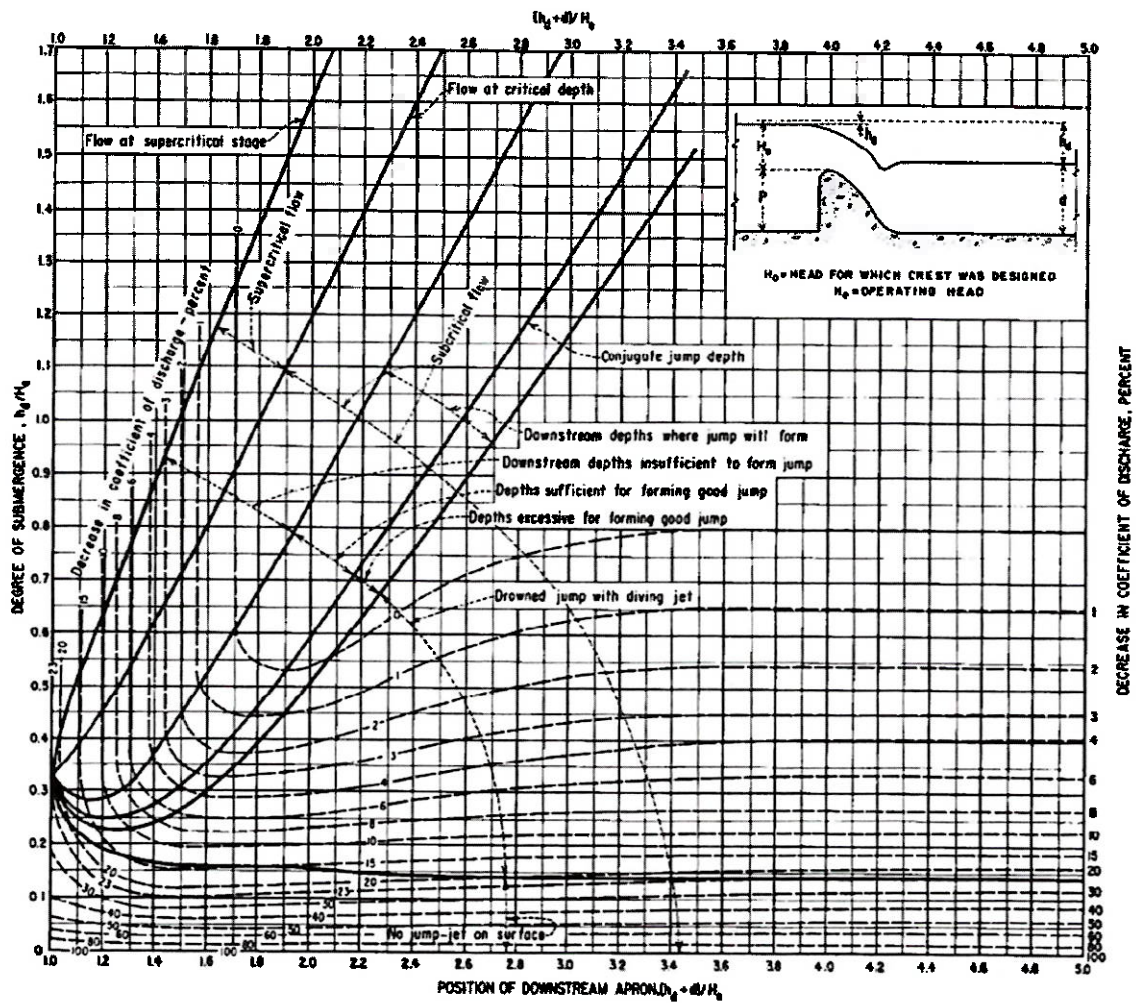


Figure 5: Effects of downstream influences on flow over weir crests.

The downstream effect considered as follow:

$$\frac{h_d + d}{H_e} = \frac{4.5}{2.4} = 1.875, \quad \frac{C_s}{C_0} = 1 \text{ (from Figure 7)} \Rightarrow C_s = 2.12 \times 1 = 2.12$$



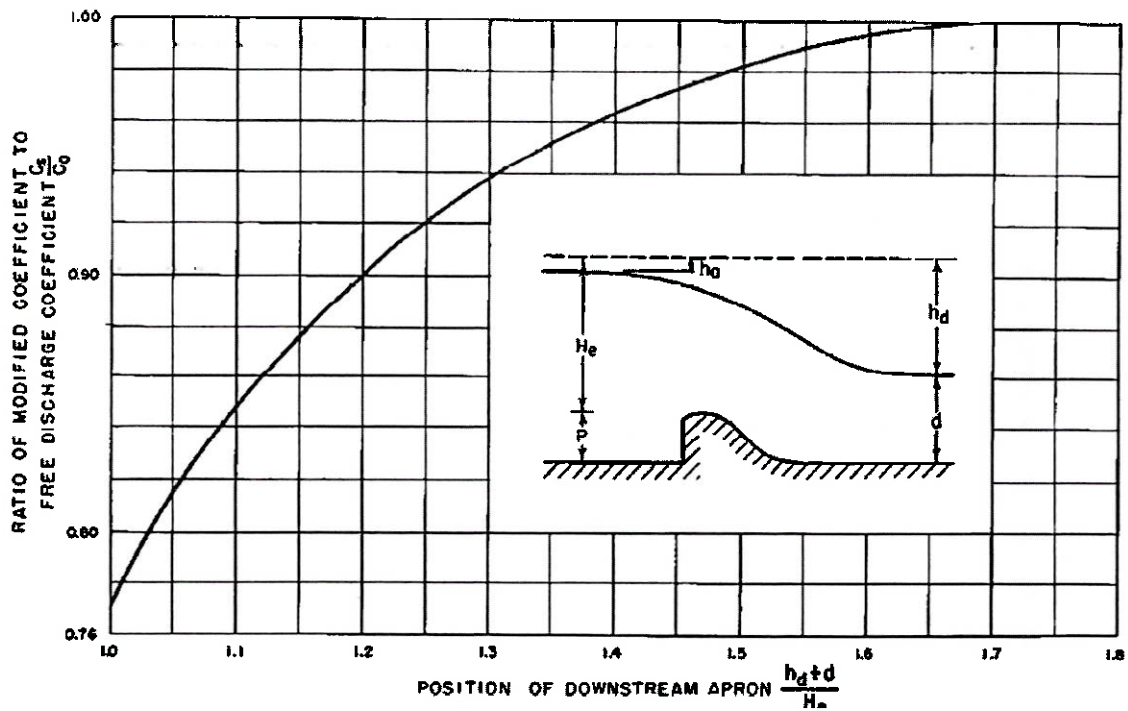


Figure 6: - Ratio of discharge coefficients resulting from apron effects. (USBR, 1987)

The Discharge Capacity of Duckbill Spillway is calculated as Follow:

The Geometry of Duckbill spillway is provided as below:

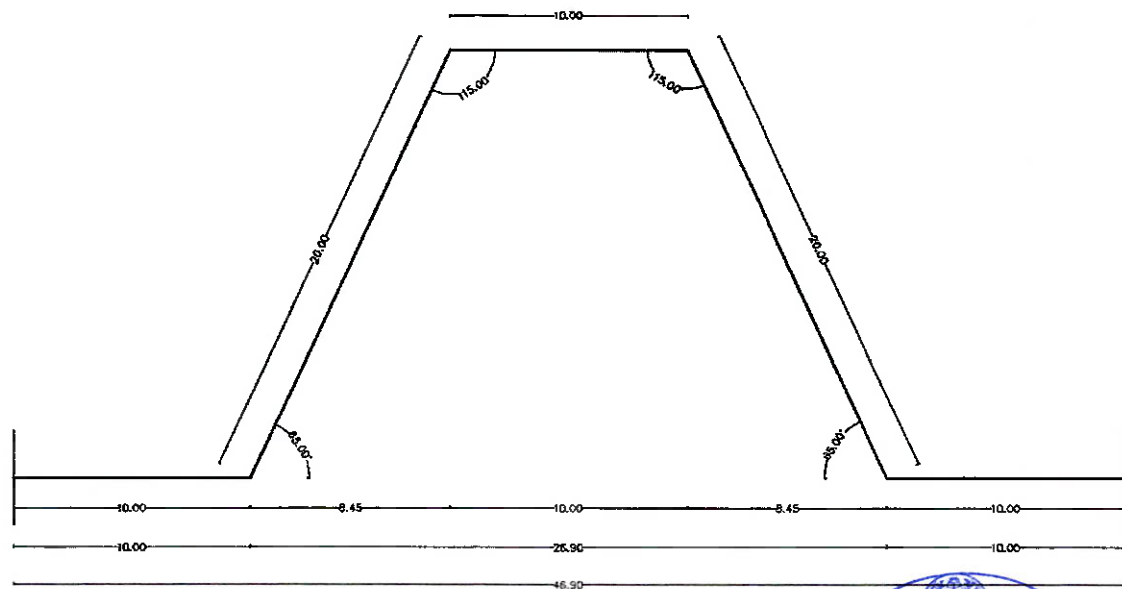


Figure 7: Plan form of Trapezoidal Duckbill Spillway

$$\text{Total Length, } L = 10 + 20 + 10 + 20 + 10 = 70\text{m}$$

$$\text{Duckbill Length, } L_1 = 20 + 10 + 20 = 50\text{m}$$

For trapezoidal plan form and corner angles of 120° , $e=0.25$ and $f=0.88$ value is readable from below figures:

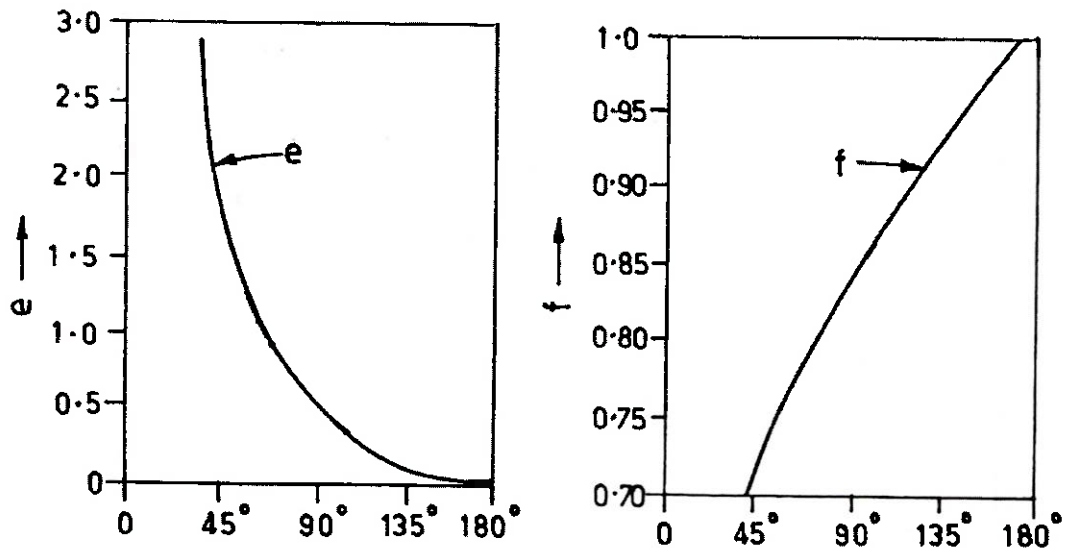


Figure 8: Coefficient of corner effects

Work out corner effect $ld = [e/(1-f)]$, where e and f are functions of corner angle ϕ of the duckbill spillway:

$$ld = \frac{e}{1-f} = \frac{0.25}{1-0.88} = 2.083$$

Work out reduction coefficient C_r as:

$$C_r = 1 - \frac{2 \times ld}{L_1} = 1 - \frac{2 \times 2.083}{50} = 0.916$$

$$Q = C_r \times C_{ds} \times L_1 \times H^{1.5} + C \times L_2 \times H^{1.5}$$

$$Q_{Design} = 0.916 \times 2.12 \times 50 \times 2.5^{1.5} + 2.12 \times 20 \times 2.5^{1.5} = 551.4 \text{ Cumecs}$$

$$\Rightarrow Q_{Design} = 551.4 \text{ Cumecs} > 545 = Q_{10,000}$$

The maximum outflow capacity of spillway is revised after flood routing carried out for $T=10,000$ flood and PMF.

Design of Service Spillway discharge capacity is summarized in the below table:

Table 4: Design of Uncontrolled Ogee Spillway

Discharge Calculations of Service Spillway

Service Spillway																
Crest Length: 60m																
Crest Elevation: 238.10																
Head Over Crest, H_o : 2.50																
Normal Capacity: 400 m ³ /s																
H_o/H_o	H_o , m	C/C_o	C_i	b_o+d	b_o+dH_o	C_o/C	C_i	$C_i H_o^{3/2}$	H_o+P	V_o (approx.)	h_o	Friction Losses	Entrance loss 0.1 h_o	Total approach losses, m	Gross head, m	Service Spillway $Q=C_i C_o H_o^{3/2}$, m ³ /s
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.0000	0.00	0
0.20	0.50	0.85	1.81	2.50	5.02	1.00	1.81	0.63	1.50	0.42	0.009	0.0021	0.0009	0.0030	0.50	44
0.40	0.99	0.90	1.91	2.99	3.02	1.00	1.91	1.88	1.99	0.94	0.045	0.0073	0.0045	0.0119	1.00	130
0.60	1.50	0.94	2.00	3.50	2.34	1.00	2.00	3.66	2.50	1.46	0.109	0.0134	0.0109	0.0243	1.52	254
0.80	2.00	0.97	2.07	4.00	2.00	1.00	2.07	5.83	3.00	1.95	0.193	0.0189	0.0193	0.0382	2.03	405
1.00	2.45	1.00	2.13	4.45	1.82	1.00	2.13	8.15	3.45	2.36	0.285	0.0236	0.0285	0.0521	2.50	566
1.20	2.93	1.02	2.18	4.93	1.68	1.00	2.18	10.94	3.93	2.78	0.395	0.0280	0.0395	0.0675	3.00	761
1.40	3.42	1.05	2.23	5.42	1.59	1.00	2.23	14.05	4.42	3.18	0.516	0.0321	0.0516	0.0837	3.50	977



Flood Routing

Reservoir routing is a process of calculating changes in reservoir levels, its volume and outflows for a given inflow hydrograph representing a flood approaching from catchment area.

After many trials of flood routing maximum water level and length of spillway is adjusted to the required dimensions. Design head and spillway length is achieved as follow

Table 5: Summary of Flood Routing for proposed spillway

Flood Routing Result for $T=10,000$ Flood	Flood Routing Result for Average PMF
$Q_{10,000}=545\text{m}^3/\text{sec}$	$Q_{\text{PMF}}=918\text{m}^3/\text{sec}$
$Q_{\text{outflow}}=498.98\text{m}^3/\text{sec}$	$Q_{\text{outflow}}=866.24\text{m}^3/\text{sec}$
Available Length, $L=70\text{m}$	Available Length, $L=70\text{m}$
Head= 2.34m	Head= 3.38m
Max. Water Level= 2383.34 m.a.s.l	Max. Water Level= 2384.38 m.a.s.l
Available Freeboard= $1.16\text{m}>\text{Min.Fb}=0.91\text{m}$	Available Freeboard= 0.12m ($\sqrt{\text{Not Overflowing}}$)

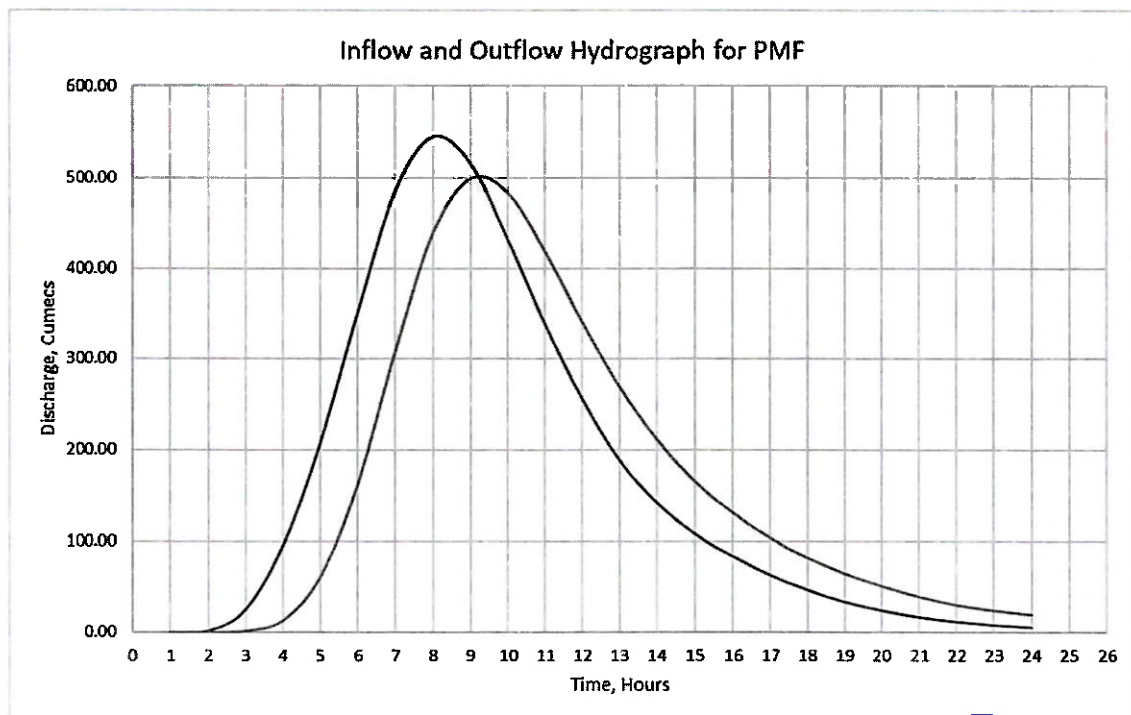


Figure 9: Inflow and Outflow Hydrograph for $L=50\text{m}$ Ogee Spillway, $T=10,000$ Years.

Flood Routing Computation for 10,000 years flood

$$\left(\frac{I_1 + I_2}{2} \right) \Delta t - \left(\frac{O_1 + O_2}{2} \right) \Delta t = S_2 - S_1$$

1	2	3	4	5	6	7	8	9	10	11	12	13
Time, hours	Δt , hours	I Inflow at time t_i cumecs	$(I_1 + I_2)/2$ Average Rate of Inflow Q_i at for Δt , Cumecs	$S_i = \Delta t \times (I_1 + I_2)/2$ Inflow Volume, Cum	Total Reservoir Storage Elevation at time t_i	O Outflow at time t_i , Cum	$(O_1 + O_2)/2$ Average Rate of Outflow Q_o at for Δt , Cumecs	$S_i = \Delta t \times (O_1 + O_2)/2$ Outflow Volume, Cum	$S_2 - S_1$ Incremental Storage, ΔS_i , Cum	Total Storage, Cum	Reservoir Elevation at the end of Δt	Remarks
1	0	0.00	0.00	-	2,381.00	-	-	-	-	-	2,381.00	
2	1	1.95	0.97	3,507	2,381.00	-	-	-	3,507	3,507	2,381.00	
3	1	26.01	13.98	50,329	2,381.05	1.56	0.78	2,805	47,523	51,030	2,381.04	
4	1	96.37	61.19	220,292	2,381.21	13.42	7.49	26,952	193,339	244,369	2,381.21	
5	1	208.90	152.64	549,492	2,381.58	61.58	37.50	134,982	414,510	658,879	2,381.58	
6	1	351.91	280.41	1,009,465	2,382.11	163.02	112.30	404,275	605,189	1,264,069	2,382.11	
7	1	487.02	419.46	1,510,072	2,382.70	308.98	236.00	849,612	660,460	1,974,529	2,382.69	
8	1	545.00	516.01	1,857,630	2,383.15	439.46	374.22	1,347,201	510,428	2,434,957	2,383.14	Freeboard
9	1	515.89	530.45	1,909,608	2,383.34	498.98	469.22	1,689,201	220,407	2,655,364	2,383.34	1.16
10	1	432.13	474.01	1,706,437	2,383.29	483.08	491.03	1,767,709	(61,272)	2,594,092	2,383.28	
11	1	337.82	384.97	1,385,900	2,383.08	418.17	450.63	1,622,252	(236,353)	2,357,739	2,383.07	
12	1	255.54	296.68	1,068,044	2,382.81	339.45	378.81	1,363,731	(295,687)	2,062,052	2,382.81	
13	1	189.31	222.42	800,724	2,382.45	269.00	304.23	1,095,236	(294,502)	1,767,550	2,382.55	
14	1	142.78	166.04	597,758	2,382.32	211.41	240.21	864,745	(266,987)	1,500,563	2,382.32	
15	1	108.52	125.65	452,341	2,382.12	165.23	188.32	677,951	(225,610)	1,274,953	2,382.12	
16	1	83.65	96.09	345,908	2,381.96	131.12	148.18	533,491	(187,523)	1,087,429	2,381.96	
17	1	63.25	73.45	264,433	2,381.82	103.51	117.32	422,335	(157,902)	929,527	2,381.82	
18	1	47.10	55.18	198,634	2,381.70	81.64	92.58	333,273	(134,639)	794,888	2,381.70	
19	1	34.03	40.57	146,036	2,381.60	64.79	73.21	263,571	(117,535)	677,353	2,381.60	
20	1	24.52	29.28	105,402	2,381.51	50.77	57.78	208,005	(102,604)	574,749	2,381.51	
21	1	17.30	20.91	75,287	2,381.43	39.31	45.04	162,140	(86,853)	487,896	2,381.43	
22	1	12.26	14.78	53,217	2,381.36	30.11	34.71	124,951	(71,734)	416,162	2,381.37	
23	1	8.48	10.37	37,334	2,381.31	24.06	27.09	97,508	(60,174)	355,989	2,381.31	
24	1	5.96	7.22	24,989	2,381.27	19.56	21.81	78,512	(52,523)	303,466	2,381.27	

From above table the following results can be achieved:

- Service Spillway Crest Level=2381.00 m.a.s.l
- The normal or operating water level, N.W.L=2381.00 m.a.s.l
- Design Water Level for $Q_{10,000}$, D.W.L=2383.50
- Maximum Water Level for Average PMF, M.W. L=2384.38 m.a.s.l

Discharge Rating curve of spillway are shown as follow:

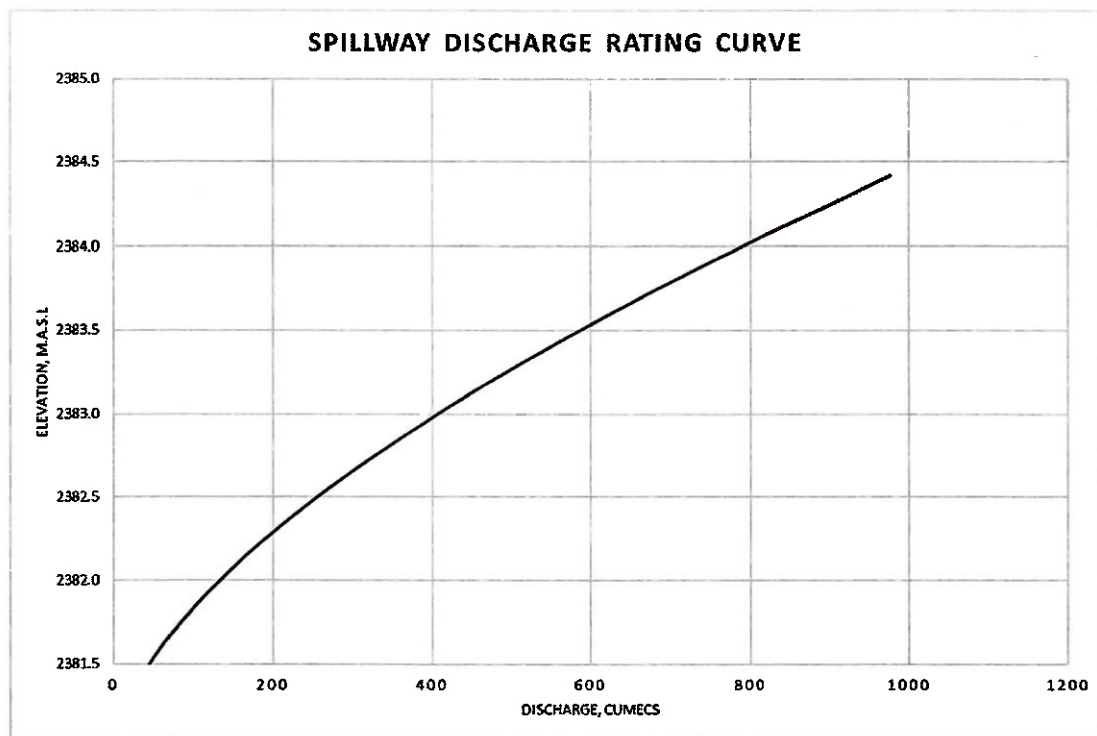


Figure 10: Discharge Rating Curve of Spillways

Shape of Uncontrolled Ogee Crest

Crest shapes that approximate the profile of the under nappe of a jet flowing over a sharp-crested weir provide the ideal form for obtaining optimum discharges. The shape of such a profile depends upon the head H_0 , the inclination of the upstream face of the overflow section, and the height of the overflow section above the floor of the entrance channel, P .

Calculation of the ogee crest shape of the spillway shall be carried out based on the 100-year return flood only. The shape of an ogee profile depends upon the head, the inclination of the upstream face of the overflow section, and the height of the overflow section above the floor of the entrance channel.



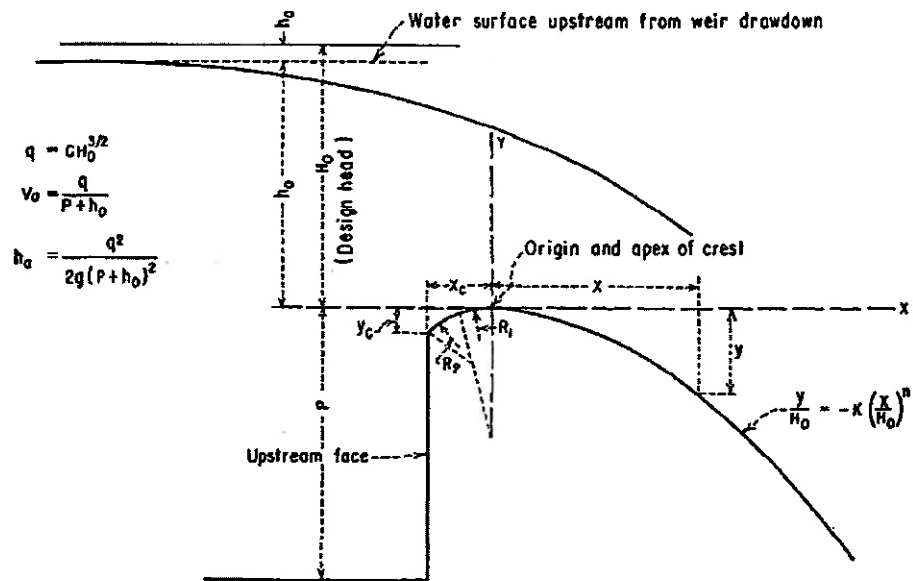


Figure 11: - Elements of Nappe-Shaped Crest Profiles

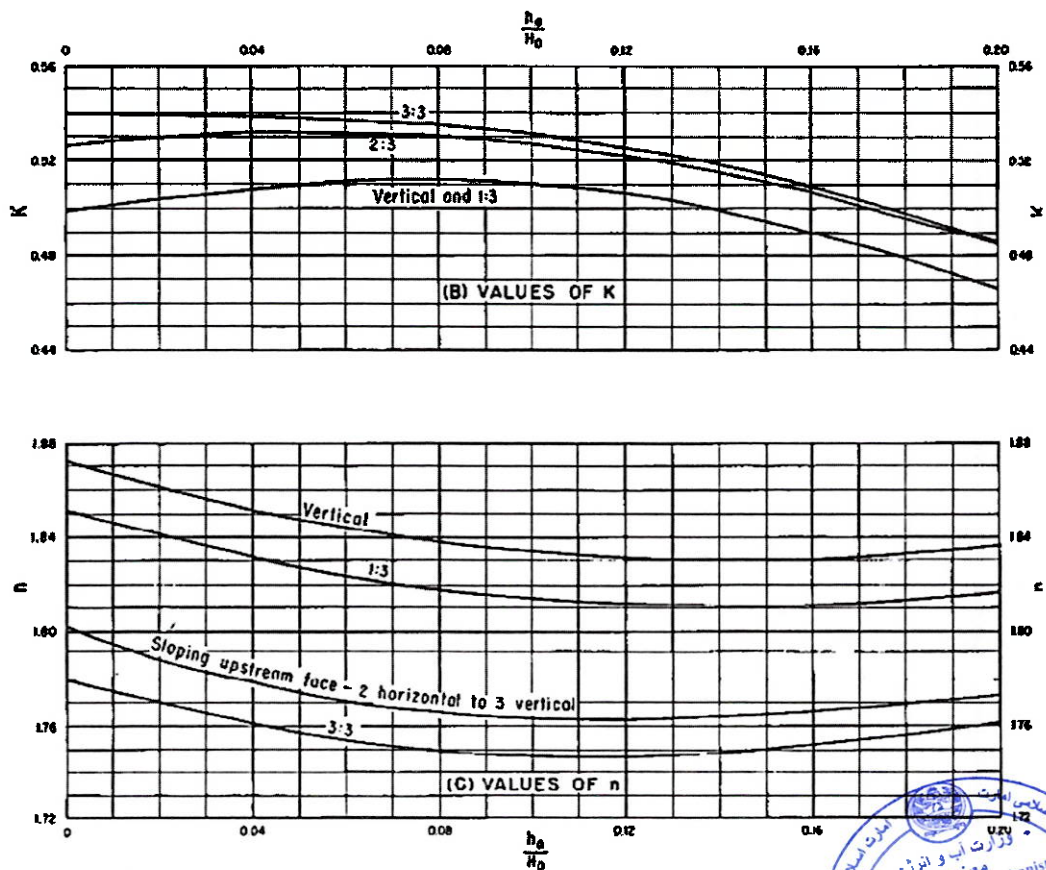


Figure 12: -Factors for definition of nappe-shaped crest profiles (Value of K & n).

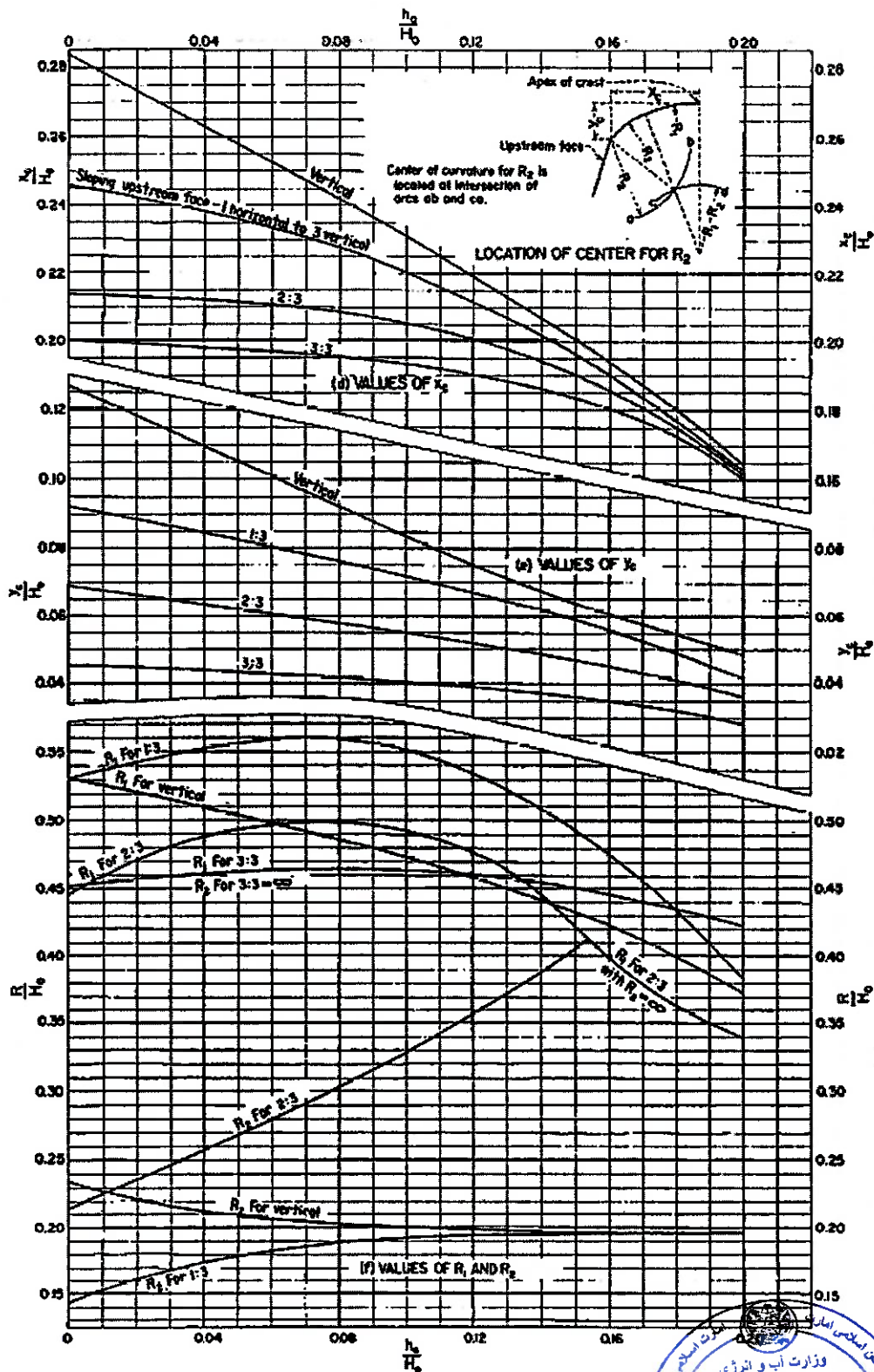


Figure 13:- Factors for definition of nappe-shaped crest profiles (X_c , Y_c , R_1 & R_2)

Elements of nappe shaped crest profiles can be determined from below 1-8 & 1-9 figure where the profile is defined as it relates to axes at the apex of the crest. The upstream portion from the origin is defined as both a single curve and a tangent or as a compound circular curve. The portion downstream is defined by the following equation:

$$\frac{y}{H_o} = -K \left(\frac{x}{H_o} \right)^n$$

in which K and n are constants whose values depend on the upstream inclination and on the velocity of approach. Figure 1.8 (B) and (C) gives values of these constants for different conditions.

The radii and centre points of downstream portion at the apex of the crest can be evaluated from the Figure 1.8 (D), (E) and (F).

From the above 1-8 figures (B, C, D, & E) the following factors of nappe-shaped crest profile is derived:

Table 6: Elements of Ogee Crest Profile and Nappe Shaped Profile Ordinates.

Ogee Crest Profile Elements			X	Y
H _c	2.20	m	0.30	-0.04
h _a	0.23	m	0.60	-0.12
H ₀	2.20	m	0.90	-0.25
h _a /H ₀	0.10		1.20	-0.41
n	1.738		1.50	-0.60
K	0.53		1.80	-0.82
X _c	0.42	m	2.10	-1.08
Y _c	0.09	m	2.40	-1.36
R ₁	1.03	m	2.70	-1.66
R ₂	Infinity	m	3.00	-2.00

From above factors the equation of downstream profile is defined by the equation:

$$\frac{y}{H_o} = -K \left(\frac{x}{H_o} \right)^n$$

$$\Rightarrow \frac{y}{2.2} = -0.53 \left(\frac{x}{2.2} \right)^{1.738} \Rightarrow y = -0.06 x^{1.738}$$

By entering the values for ordinate of x we can obtain ordinates y shown table

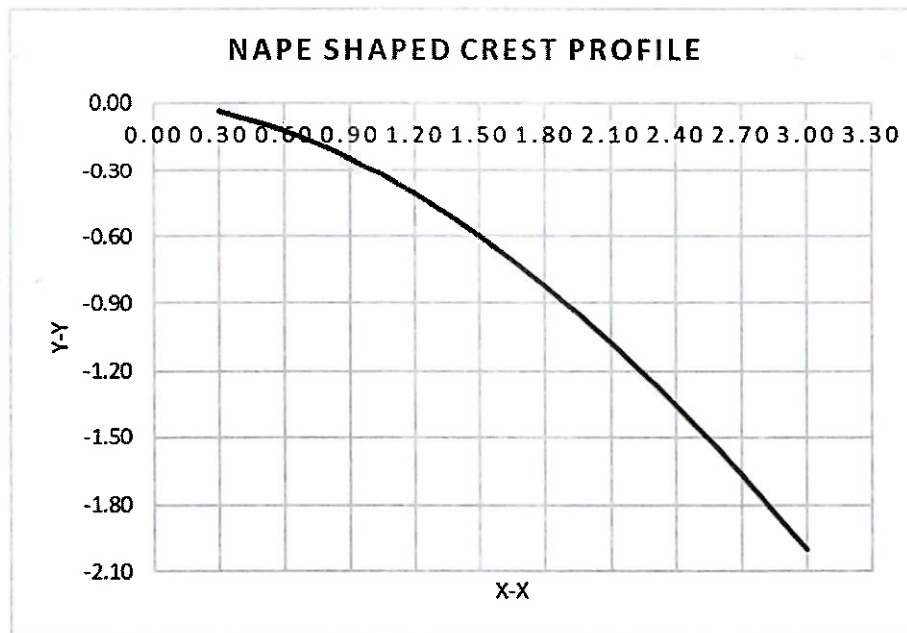


Figure 14: - Ogee Crest Profile of Downstream

Stability Analysis of Spillway

Methods of determining the load-resisting capacity of the dam or spillway should be the most accurate available. All uncertainties regarding loads or load-carrying capacity should be resolved as far as practicable by field or laboratory tests and by thorough exploration and inspection of the foundation. Thus, the safety factor should be as accurate an evaluation as possible of the ability of the structure to resist applied loads.

Although somewhat lower safety factors may be permitted for limited local areas within the foundation, overall safety factors for the dam and its foundation, including the contributions from any remedial treatment, should meet requirements for the load combination analysed.

Safety factors for gravity dams or spillway are based on the use of the gravity method of analysis, and those for foundation sliding stability are based on an assumption of uniform stress distribution on the plane being analysed

Load Combinations:

Gravity dams or spillway should be designed for all appropriate load combinations, using the proper safety factor for each. Combinations of transitory loads, each of which has only a remote probability of occurrence at

any given time, have less probability of simultaneous occurrence and should not be considered as appropriate load combinations.

Gravity dams should be designed for the following load combinations using the corresponding safety factors.

1. Usual load combinations. -Normal design reservoir elevation with appropriate dead loads, uplift, silt, ice, and tailwater. If temperature loads are applicable to the specific sites, use minimum usual temperatures occurring at that time.
2. Unusual load combinations. -Maximum design reservoir elevation with appropriate dead loads, silt, tailwater, uplift, and minimum usual temperatures occurring at that time, if applicable.
3. Extreme load combinations. -The usual loading plus the effects of the MCE.
4. Other loads and investigations:
 - The usual or unusual load combination with drains inoperative.
 - Dead load.
 - Any other load combination that, the designer thinks should be analysed for a particular dam.

Stability Requirements:

The concepts used to develop the structural stability requirements contained are to establish safety factors or safety provisions for the three prescribed load condition categories of usual, unusual, and extreme such that the risk of a failure is kept to an acceptably low level and such that performance objectives are achieved.

Table 7: Load Condition Probabilities [EM 1110-2-2100]

Load Condition Categories	Annual Probability (p)	Return Period (t_r)
Usual	Greater than or equal to 0.10	Less than or equal to 10 years
Unusual	Less than 0.10 but greater than or equal to 0.0033	Greater than 10 years but less than or equal to 300 years
Extreme	Less than 0.0033	Greater than 300 years

Civil works structures, for the purpose of establishing safety factors or safety provisions for use in stability analyses, are to be designated as either critical or normal. Structures designated as critical are those structures on high hazard projects whose failure will result in loss of life.



Factors of Safety for Sliding:

factor of safety is required in sliding analyses to provide a suitable margin of safety between the loads that can cause instability and the strength of the materials along potential failure planes that can be mobilized to prevent instability.

$$FS_S = \frac{N \times \tan \phi + c \times L}{T}$$

N = force acting normal to the sliding failure plane under the structural wedge.

ϕ = angle of internal friction of the foundation material under the structural wedge.

c = cohesive strength of the foundation material under the structural wedge.

L = length of the structural wedge in contact with the foundation.

T = shear force acting parallel to the base of the structural wedge.

The required factors of safety for sliding stability for critical structures and for normal structures are presented in below Tables:

Table 8: Required Factors of Safety for Sliding - Critical Structures [EM 1110-2-2100]

Load Condition Categories			
Site Information Category	Usual	Unusual	Extreme
Well Defined	1.7	1.3	1.1
Ordinary	2.0	1.5*	1.1*
Limited**	-	-	-

*For preliminary seismic analysis without detailed site-specific ground motion, use FS=1.7 for unusual and FS=1.3 for extreme. See further explanation in section 3.11 b.

**Limited site information is not permitted for critical structures

Table 9: Required Factors of Safety for Sliding - Normal Structures [EM 1110-2-2100]

Load Condition Categories			
Site Information Category	Usual	Unusual	Extreme
Well Defined	1.4	1.2	1.1
Ordinary	1.5	1.3	1.1
Limited	3.0	2.6	2.2



Factors of Safety for Flotation

A factor of safety is required for flotation to provide a suitable margin of safety between the loads that can cause instability and the weights of materials that resist flotation.

$$FS_f = \frac{W_s + W_c + S}{U - W_G}$$

W_s = weight of the structure, including weights of the fixed equipment and soil above the top surface of the structure. The moist or saturated unit weight should be used for soil above the groundwater table and the submerged unit weight should be used for soil below the groundwater table.

W_c = weight of the water contained within the structure

S = surcharge loads

U = uplift forces acting on the base of the structure

W_G = weight of water above top surface of the structure.

The required factors of safety for *flotation* are presented in below Table.

Table 10: Required Factors of Safety for Flotation – All Structures [EM 1110-2-2100]

Site Information Category	Load Condition Categories		
	Usual	Unusual	Extreme
All Categories	1.3	1.2	1.1

Overturing Stability and Limits on Resultant Location

When the resultant of all forces acting above any horizontal plane through a dam intersects that plane outside the middle third, a non- compression zone will result. For usual loading conditions, it is generally required that the resultant along the plane of study remain within the middle third to maintain compressive stresses in the concrete. For unusual loading conditions, the resultant must remain within the middle half of the base. For the extreme load conditions, the resultant must remain sufficiently within the base to assure that base pressures are within prescribed limits.



Table 11: Stability and stress criteria [EM 1110-2-2200]

Load Condition	Resultant Location at Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress	
				Compressive	Tensile
Usual	Middle 1/3	2.0	\leq allowable	$0.3 f'_c$	0
Unusual	Middle 1/2	1.7	\leq allowable	$0.5 f'_c$	$0.6 f'_c{}^{2/3}$
Extreme	Within base	1.3	$\leq 1.33 \times$ allowable	$0.9 f'_c$	$1.5 f'_c{}^{2/3}$

Rotational behaviour is evaluated by determining the location of the resultant of all applied forces with respect to the potential failure plane. This location can be determined through static analysis. Limits on the location of the resultant are provided in below Table.

Table 12: Requirements for Location of the Resultant – All Structures [EM 1110-2-2100]

Load Condition Categories			
Site Information Category	Usual	Unusual	Extreme
All Categories	100% of Base in Compression	75% of Base in Compression	Resultant Within Base

Allowable Base Pressure

The allowable bearing capacity value is defined as the maximum pressure that can be permitted on soil or rock giving consideration to all pertinent factors with adequate safety against rupture of the soil or rock mass, or movement of the foundation of such magnitude that the structure is impaired.

The maximum computed base pressure should be equal to or less than the allowable bearing capacity for the usual and unusual load conditions. For extreme loading condition, the maximum bearing pressure should be equal to or less than 1.33 times the allowable bearing capacity.

Seismic Stability

The factors of safety given in Tables 9 include $FS=1.5$ for unusual and $FS = 1.1$ for extreme load conditions, for ordinary site information. The ordinary site information and related factor of safety must be used in the seismic coefficient method. These factors of safety are based on use of extreme loads with very low probabilities of being exceeded. When factors of safety for seismic loadings are being calculated using the coefficient method, the MCE loads are usually not based on detailed site-specific seismic data. Since the loads would be based on less precise data, there would be greater probability that the predicted extreme loads could be exceeded, therefore, it is appropriate to use higher factor of safety for such analyses. For such analyses, use a factor of safety of 1.7 for unusual and 1.3 for extreme, as stated in the notes following the above table.



Stability Calculations:

Based on above mentioned criteria for stability of concrete structures and gravity dams as per USBR- Design of Small Dams, EM 1110-2-2100, and EM 1110-2-2200; the following calculations are summarized:

1. Section Properties:

Table 13: Properties of Spillway Section

Spillway Type	Ogee Over flow	
Crest Length	70.00	m
Crest Elevation	2381.00	m.a.s.l
Approach Channel Elevation	2380.00	m.a.s.l
Operational Dam Water Level	2381.00	m.a.s.l
Max. Water Level	2383.50	m.a.s.l
Area	9.792	sqm
Perimeter	21.355	m
Width	9.12	m

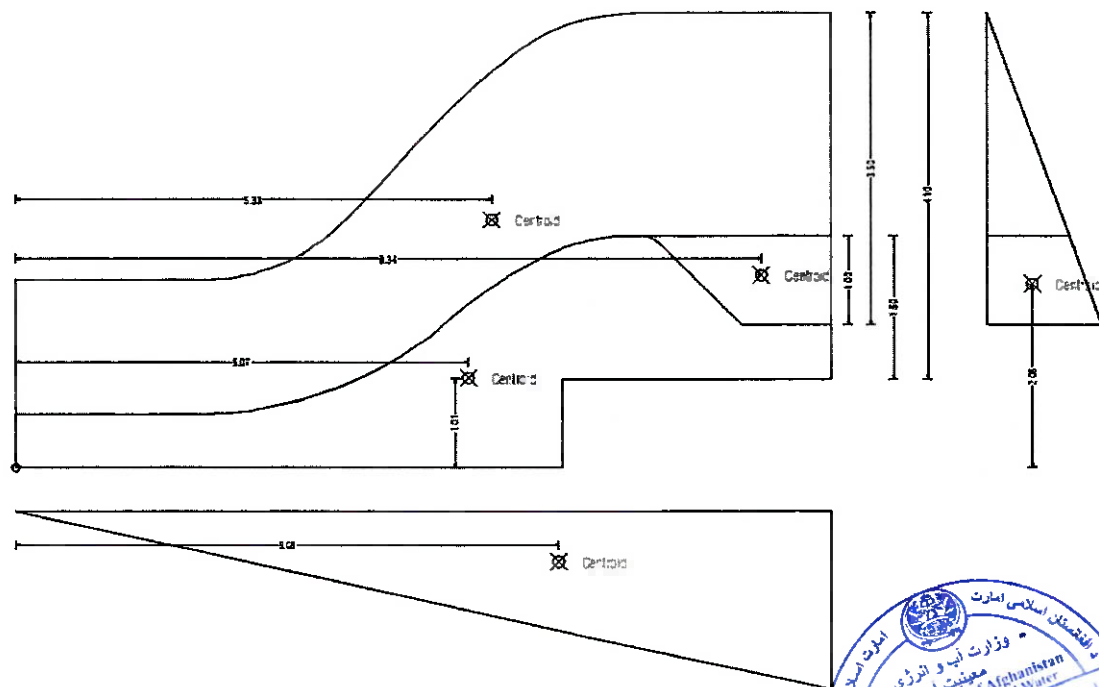


Figure 15: Geometry of Section and Loads with its centroids and lever arms

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2. Load Calculations:

Hydrostatic Forces

Normal Water Level	2,381.00	m
Flood Water Level	2,383.50	m
Approach Level	2,380.00	m
River Bed Level	2,379.00	m
Normal Head	1.00	m
Maximum head	2.50	m
Hydrostatic Force at Normal Water Level, P_{min}	4.91	KN/m
Hydrostatic Force at Maximum Water Level, P_{max}	19.62	KN/m

Earth Quak Forces

Horizontal PGA, g	0.30	
Vertical PGA, g	0.20	
Body Mass	235.01	KN
Horizontal Component of Earthquak Forc, P_{EH}	70.50	KN
Vertical Component of Earthquak Force, P_{EV}	47.00	KN

Hydrodynamic Forces

a	0.30	
g	9.81	m/sec ²
λ_w	9.81	KN
Water Depth, h	2.50	m
Desired Water Level, y	1.00	m
θ	45.00	degree
C_m	0.37	



C by formula	0.26	
y/h	0.40	
C from Graph	0.30	
P_e	2.21	KN
F earthquake	4.01	KN-m
M earthquake	4.14	KN-m

Uplift Forces

Normal Head	1.60	m
Maximum head, US	4.10	m
Maximum Depth, DS	1.50	m
Normal Case	71.57	KN/m
Flood Case	183.41	KN/m

Weight of Ogee Structure

W_G	235.01	KN
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Weight of water

Normal Case	14.73	KN
Flood Case	179.64	KN

3. Result of Stability Analysis:

Figure 16: Input Data for Stability Analysis

ϕ = angle of internal friction of the foundation material under the structural wedge.	35 Degree
c = cohesive strength of the foundation material under the structural wedge.	16,300 Kpa
Allowable Bearing Capacity of Foundation Rock, q_a	3370 Kpa
Horizontal P.G.A (M.C.E, T=2475 years)	0.305g
UCS=100 Mpa, $\tau=0.011*UCS$	710 Kpa

Vertical P.G.A (M.C.E, T=2475 years)	0.201 g
--------------------------------------	---------

Figure 17: Result of Stability Analysis for Usual Load Combination

Item	Forces, KN/m	Magnitude, KN	Lever Arm, m	Moment about Toe, KN-M
A Vertical				
1	Body Weight	235.01	5.07	1,191.49
2	Water Weight	14.73	8.34	122.89
3	Normal Uplift Pressure	71.57	6.08	435.17
4				
B Horizontal				
1	Static Water Pressure	4.91	1.93	9.48
2	Sediment Load	-	-	-
3	Rock passive pressure	-	-	-
4				
Total Vertical Load				178.17
Total Horizontal Load				9.48
Stabilizing Moment				1,314.38
Disturbing Moment				444.65
Algebraic Sum of Moments				869.73
Factor of Safety against sliding, FSS				13.16 >2 OK
Factor of Safety against overturning, FSO				2.96 >2.5 OK
Factor of Safety against Flotation, FSF				4.39 >1.3 OK
X				4.88
e				-0.32 <B/6 OK
P _{toe}				15.40
P _{heel}				23.67
α				0.00
σ				15.40 <3370 OK
τ				0.00 <710 OK

Figure 18: Result of Stability Analysis for Unusual Load Combination

Item	Forces KN/m	Magnitude	Lever Arm, m	Moment about Toe, KN-M
A	Vertical			
1	Body Weight	235.01	5.07	1,191.49
2	Water Weight	179.64	5.33	957.50
4	Uplift Pressure	183.41	6.08	1,115.12
B	Horizontal			
1	Static Water Pressure	19.62	2.06	40.38
2	Sediment Load	-	0.53	-
3	Rock passive pressure	-	0.40	-
4				
Total Vertical Load				231.24
Total Horizontal Load				19.62
Stabilizing Moment				2,148.99
Disturbing Moment				1,155.50
Algebraic Sum of Moments				993.49
Factor of Safety against sliding, FSS				8.25 >1.5 OK
Factor of Safety against overturning, FSO				1.86 >1.5 OK
Factor of Safety against Flotation, FSF				1.36 >1.2 OK
X				4.30
e				0.26 <B/6 OK
P _{toe}				29.75
P _{heel}				20.96
α				0.00
σ				29.75 <3370 OK
τ				0.00 <710 OK

Figure 19: Result of Stability Analysis for Extreme Load Combination

Item	Forces KN/m	Magnitude	Lever Arm, m	Moment about Toe, KN-M
A	Vertical			
1	Body Weight	235.01	5.07	1,191.49



2	Water Weight	14.73	8.34	122.89
3	Earthquake, E_v	47.00	5.07	238.30
4	Uplift Pressure	71.57	6.08	435.17
5				
B	Horizontal			
1	Static Water Pressure	4.91	1.60	7.85
2	Earthquake, E_H	70.50	1.01	71.21
3	Sediment Load	-	-	-
4	Shear Key passive pressure	-	-	-
5	Hydrodynamic Pressure	4.01		4.14
Total Vertical Load				
131.17				
Total Horizontal Load				
79.41				
Stabilizing Moment				
1,314.38				
Disturbing Moment				
756.66				
Algebraic Sum of Moments				
557.72				
Factor of Safety against sliding, FSS				
1.16				
Factor of Safety against overturning, FSO				
1.74				
Factor of Safety against Flotation, FSF				
3.05				
X				4.25
e				0.31
P_{toe}				0.55
P_{heel}				0.36
α				0.00
σ				0.55
t				0.00

Appendix A: Section of Spillway

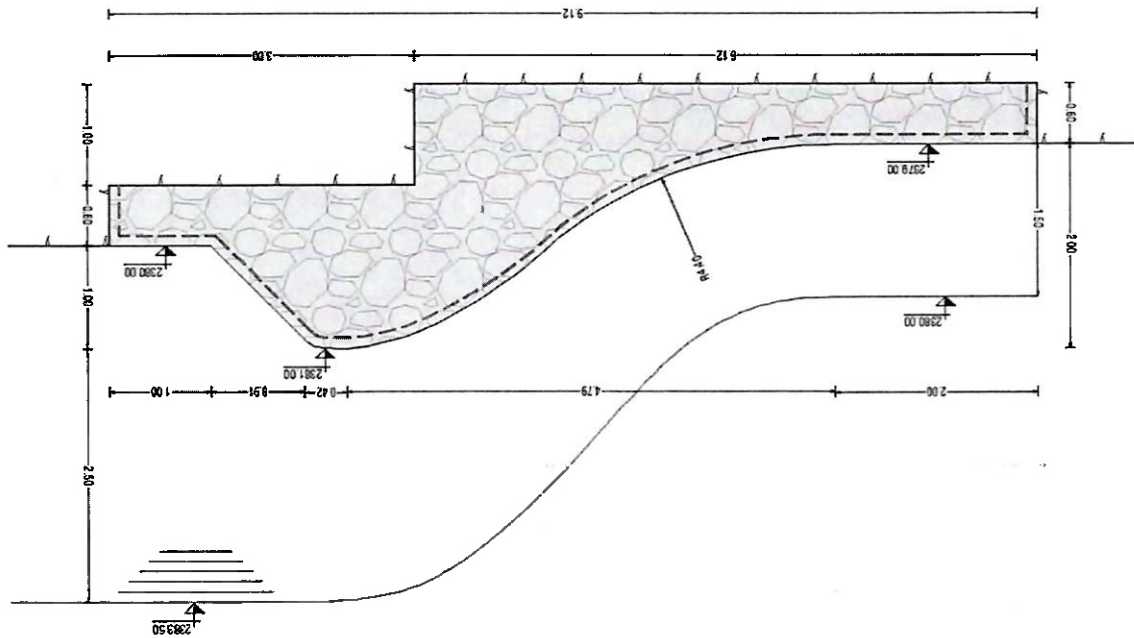


Figure 20: - Plan and Section of Service Spillway.

Table 14:- Bill of Quantity of Duckbill Spillway

S. N	Description of Work	Unit	Quantity	Remarks
1	Earth Work in Excavation of Hard Cutting	Cu.m	13,894	
2	Rock-Filled Concrete of Spillway Ogee Structures	Cu.m	680	
3	Rock-Filled Concrete of Spillway Ogee Structures	Cu.m	134	
4	Stone Masonry of Side Wall	Cu.m	84	
5	PCC under footing and top of Side Wall	Cu.m	4	
6	Water Stop for Contraction Joint	Meters	63	

The End





Islamic Emirate of Afghanistan
Ministry of Water and Energy
Deputy Ministry of Water



General Directorate of Engineering Services of Water Infrastructures

Technical Board



Rehabilitation of Sultan Dam			
Date		July 2023	
Prepared	Checked	Approved	Hydrology Report of Sultan Dam
Faridoon Danesh	Ahmad Sohail Noori	Abdul Ghafor Omari	



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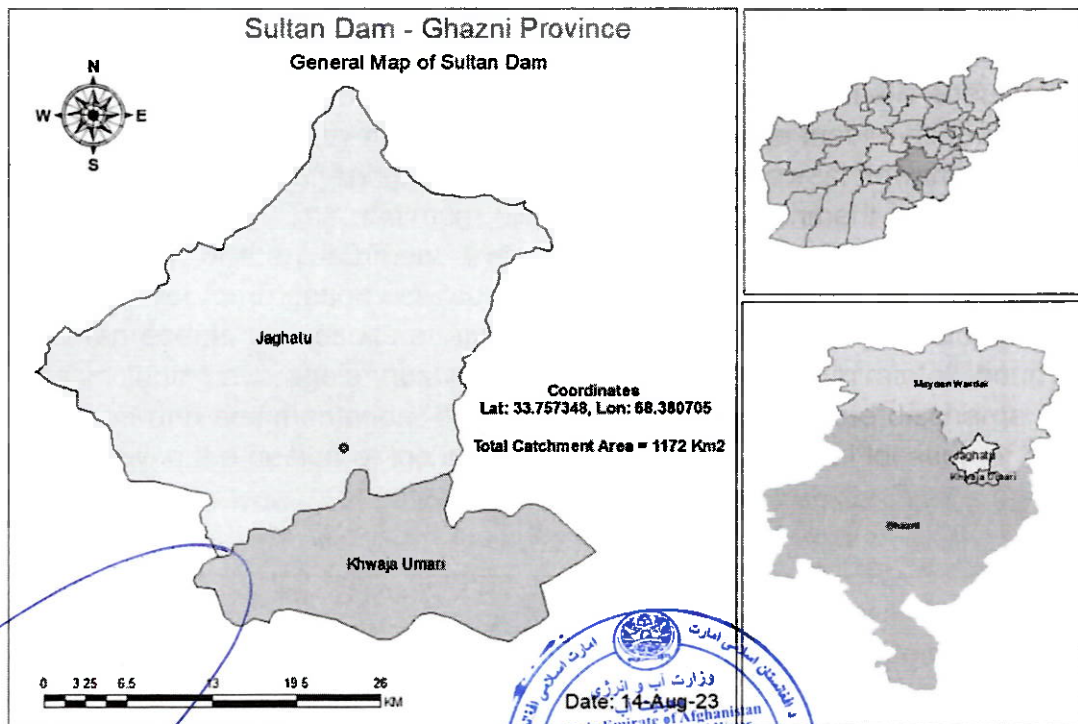


Chapter One General

1.1 Introduction

Sultan dam Project is located near Kala Manar village, Jaghato district of Maidan Wardak Province. Latitude and Longitude of the Project Site are 33.757348°N and 68.380705°E. The distance from Ghazni city to the proposed dam site is about 24 km. The Ghazni Stream is fed mostly by rain and less by snow melt, so that it has the highest flow in January, February, March, April and May months and the lowest flow in, June, July and August, October months. The minimum elevation of the catchment is 2365 m.a.s.l and maximum elevation of the catchment area is above 4562 m.a.s.l. The proposed project would provide water for irrigation use, sub-ground water recharge, and climate change.

This report represents the results of analyzing different meteorological and hydrological parameters, including average annual precipitation, 24-hr maximum rainfall, hourly rainfall, discharge, flood and sedimentation, estimated for refinement of the discharge and flood parameters used in the design of the dam. Figure 1 shows general location of the project area.



1.2 Afghanistan Climate

Afghanistan has an arid to semi-arid climate, with cold winters and dry summers. Northeast mountains of this country have subarctic climates with cold and dry winters. In mountains on Pakistan border, a divergent fringe effect of the monsoon, generally from southeast, brings tropical air masses affecting the climate from July to September. These air masses sometimes progress towards the center and south of Afghanistan and increase in the humidity and precipitation.

Wind is not so much powerful in the plateau enclosed by the mountains; however, some intensive wind storms occur in Sistan catchment area, mainly from December to February. In the southern and western areas, the wind blowing from the north which is known as the "120-days wind" occurs during summer time, from June to September. This wind is almost accompanied by severe heat, dryness and sand storms, causing difficulties to the inhabitants of steppe lands and deserts. Dust and whirlwind occur mainly during summer in the southern flat lands. Such dusty winds start in the mid-day with a speed of 97 to 177 km/hr causing high clouds of dust. Temperature and precipitation are controlled in Afghanistan through exchange of air masses. The highest and lowest precipitations are dominant in the arid southern plateau, extending beyond the Iran and Pakistan borders.

Central mounts with higher peaks, whose height decreases to the Pamir Knot, are indicatives of a totally different climate. From Kohe Baba mount down to the Pamir Knot, temperature may go down to (-35°C) or even lower in higher mountainous areas during January. However, it varies from 0 to 26°C during July, depending on the height. In mountains, precipitation, mostly as snowfall, is increased towards the east, with its maximum occurring in Kohe Baba, western part of Pamir Knot and eastern Hindukosh. In these areas and in the eastern monsoon, precipitation is almost 400 mm/yr. The eastern monsoon area includes parts of eastern boundary with Pakistan, eastern parts of Afghanistan from the north of Asmar to the north of Darkh-e-Yahya and sometimes Kabul valley, as its western limit. In the Wakhan corridor, although temperatures vary between (9°C) in the summer and -21°C in the winter, annual precipitation is less than 100 mm. Highest peaks are covered by permanent snow. In the mountainous areas adjacent to the north of Pakistan, the winter snow height is almost above 2 meters. Valleys are usually trapping the snow when rapid winds take snow from the mountain peaks and divides.



Observed Climatology of Mean-Temperature 1991-2020 Afghanistan

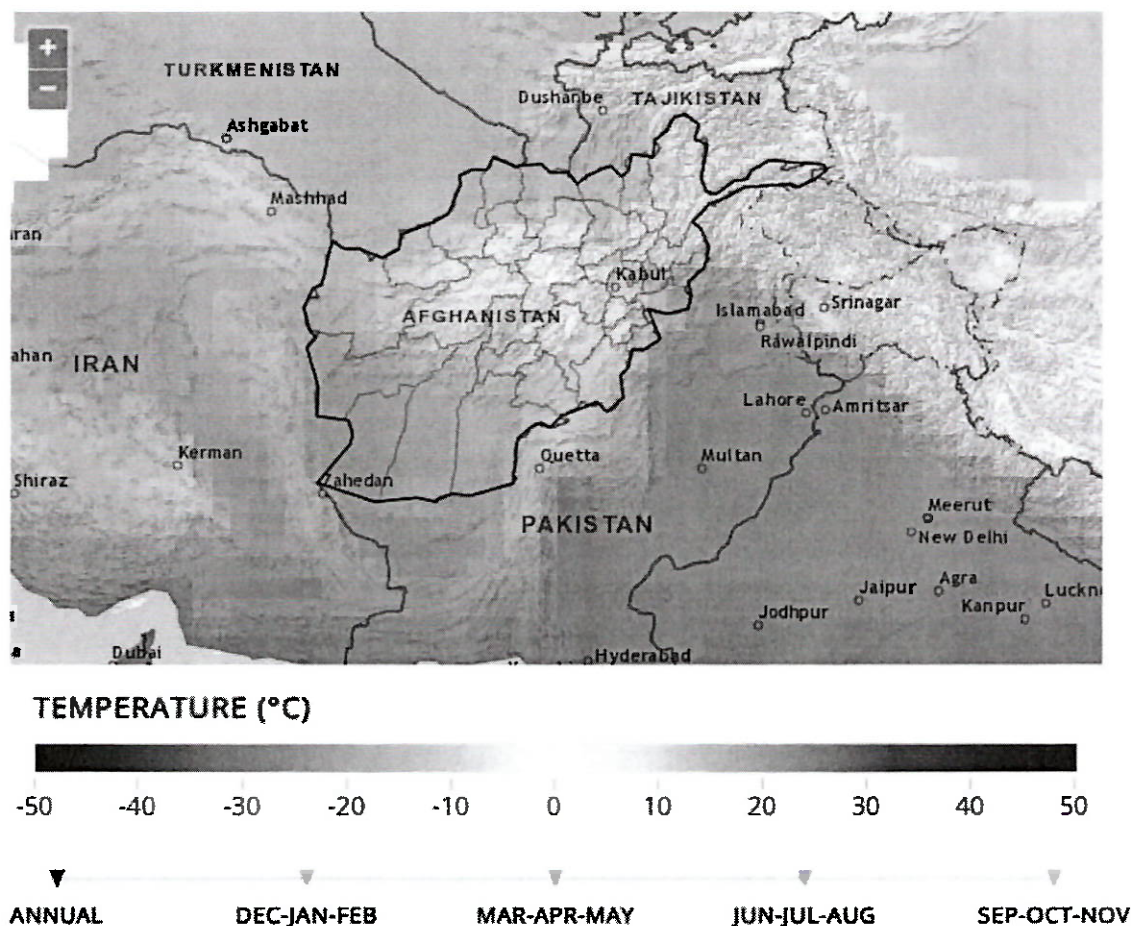


Figure 2 shows Afghanistan Climatology Map during the years (1991 – 2020)

In all parts of the country, precipitation is considerably varying during the year. Intensive, unexpected rains change seasonal rivers and waterways from potholes to ravines which shocks everybody. In Turkistan plains, extending northwards from northern hills, the climate Decreased, dryness is increased and temperature rises up, reaching to its maximum in the Lower Amu Darya and in the western parts of the plain.

In Afghanistan, climate is highly affected by cold northern air masses and low-pressure air masses of the Atlantic Ocean from the northwest, during winter and early spring, which cause snowfall and intensive coldness in highest areas and precipitation in lower areas. As already mentioned, in the mountains at the Pakistan border, a divergent fringe effect of the monsoon mainly coming from the southeast, brings tropical air masses which affect the climate from July to September. Sometimes, these air masses progress towards center and south of Afghanistan and increase humidity and precipitation. Figure 2 shows major air masses affecting Afghanistan during the year.



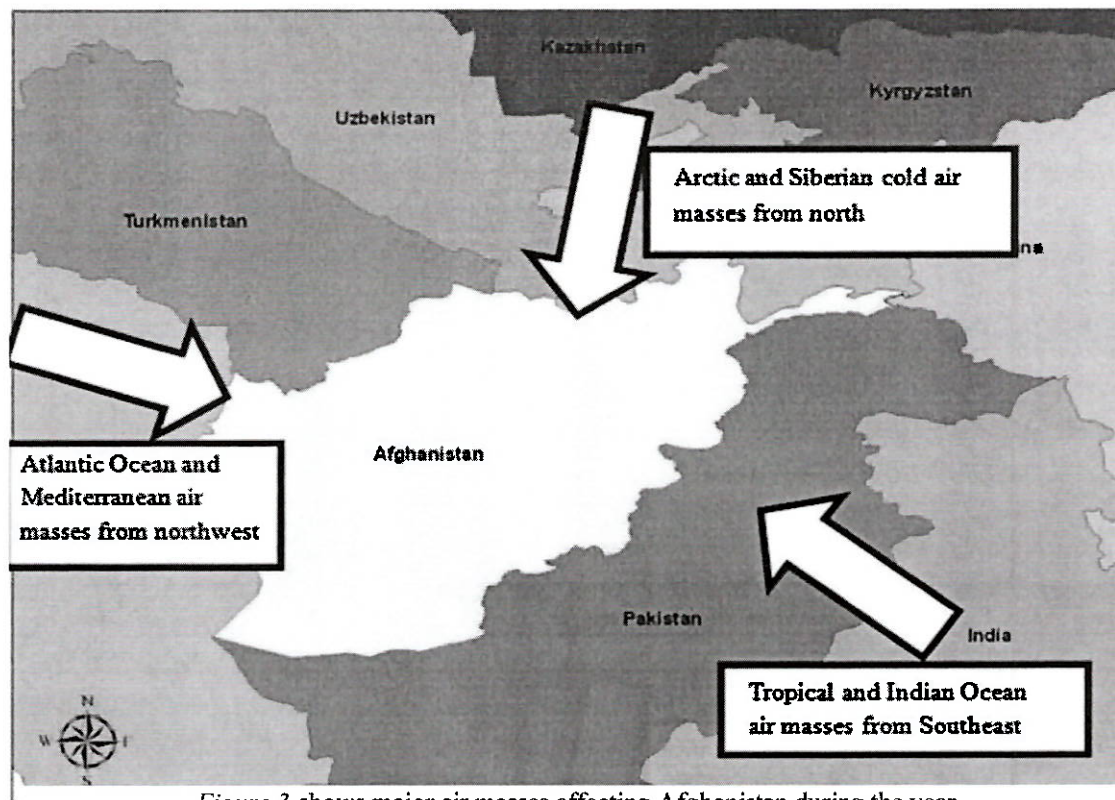


Figure 3-shows major air masses affecting Afghanistan during the year

1.3 Objective

The main object of the current hydro-climatologic studies is to estimate meteorological and hydrological parameters required for revising water resources planning studies of the Ghazni stream and consequently revise of the Sultan dam and appurtenant structures.

1.4 Methodology

Collection and processing of the required data in Afghanistan, with regard to the long-term war and insecurity in the country for more than 43 years, is not Satisfactory. Recording of the data has not been carried out continuously and furthermore a large amount of the previously recorded data has been lost. As the result, the required data are rarely available. The data has been collected from different sources such as General Water directorate of Ministry of Energy and Water of the country (as paper calendars and computer files for the years 2009 to 2022), the older meteorological calendars of Afghanistan (1948 to 1980).

the daily Precipitation series of Pul-i-Ghazni station, as the nearest station to the studied catchment area, has been used in estimation of daily precipitation with different return periods, using various record distributions. For estimation of the long-term discharge time series in the area stations and generalizing it to the dam site, the available data from Pul-i-Ghazni station is used, for calculation of Maximum Discharge at the dam axis the GIS



and HEC- HMS software are utilized.

1.5 Source of Data and Station Network

Station network for measuring climatic parameters in Afghanistan used to include more than 200 stations in the past; however, lots of these stations and their data have been lost in the long-term war in the country. This is why there are no coherent station records in Afghanistan. In order to cover the shortcomings, the data have been collected from different sources such as general directorate of water (as paper calendars from 2009 to 2022, Pul-i-Ghazni station). List of meteorological stations used in these studies are indicated in table1; locations of these stations are shown in below table.

Table 1-Shows the Pul-i-Ghazni and Gardiz Stations Coordinates

No	Station	Long.	Lat.	Elevation	Comment
1	Pul-i-Ghazni	68.4200222	33.5555866	2181	
2	Gardiz-AWS	69.2229472	33.5992666	2312	

1.6 New Station Suggested

According to the World Meteorological Organization (WMO) standard for appropriate spacing and density of the rain gauge station network, as shown in table 2, the number of stations in the studied area is not sufficient throughout the catchment area.

Table 2-Shows WMO Standard on Needed Gauge Stations

Climate	Minimum density of rain gauge stations			
	Minimum range of network density		Acceptable range in hard conditions	
	Area of each station (Km2)	Spacing between stations (Km)	Area of each station (Km2)	Spacing between stations (Km)
Mediterranean, tropical and moderate regions	600 – 900	25 – 30	900 – 3000	30 – 55
Mediterranean, tropical and moderate mountainous	100 – 200	10 – 16	350 – 1000	16 – 32
Polar and arid areas except for large plains	1500 – 10000	39 – 100	---	---



Chapter Two Rainfall

2.1 Introduction

Precipitation is of high importance in terms of hydrological, water resources, irrigation and drainage studies. Precipitation in the studied area mainly occurs in cold seasons and is due to the activity of air masses from the Atlantic Ocean and Mediterranean from the northwest. Tropical and Indian Ocean air masses from the southeast which affect parts of Afghanistan adjacent to Pakistan, cause inconsiderable precipitation in the project area.

2.2 Data Precision

The data which we received from the related organization (General Directorate of water) are used in hydrology calculation and data are daily precipitation.

2.3 Annual Rainfall

Regarding the available records of stations as well as the wet and dry periods, the 14-years' time period from 2009 to 2022 has been selected for study of rainfall. Table 3 provides the long-term average values of many stations records Afghanistan.

Table 3-Long Terms Yearly Average Precipitation Values in Afghanistan Stations (mm)

No	Station Name	Mean. Annual Precipitation (mm)
1	Tarnak	135
2	Jalalabad	232
3	Gardiz	239
4	Sarobi	283
5	Baghlan	294
6	Kabul Airport	317
7	Ghazni	333
8	Aiback	358
9	Karizmir	393
10	Drosh	627
11	North Salang	1050
12	South Salang	1075

2.3.1 Variation in Annual Rainfall with Height

Unless in specific conditions, as a general rule, in a rainfall regime rainfall increases with an increase in the height of the area. Such positive gradients justify the reason for the aerographic ascend event, or ascend of the air coming through with mountains. In specific conditions where effects of distance from the sea and humidity erosion compensate the aerographic ascend; the gradient would be zero or even negative. Based on the long-term mean precipitation parameter in the studied stations (indicated in table 3), variations of precipitation with height have been shown in figure 4. As indicated in the figure, precipitation is increased with increase in the height with the same regional gradient, as shown in dashes.

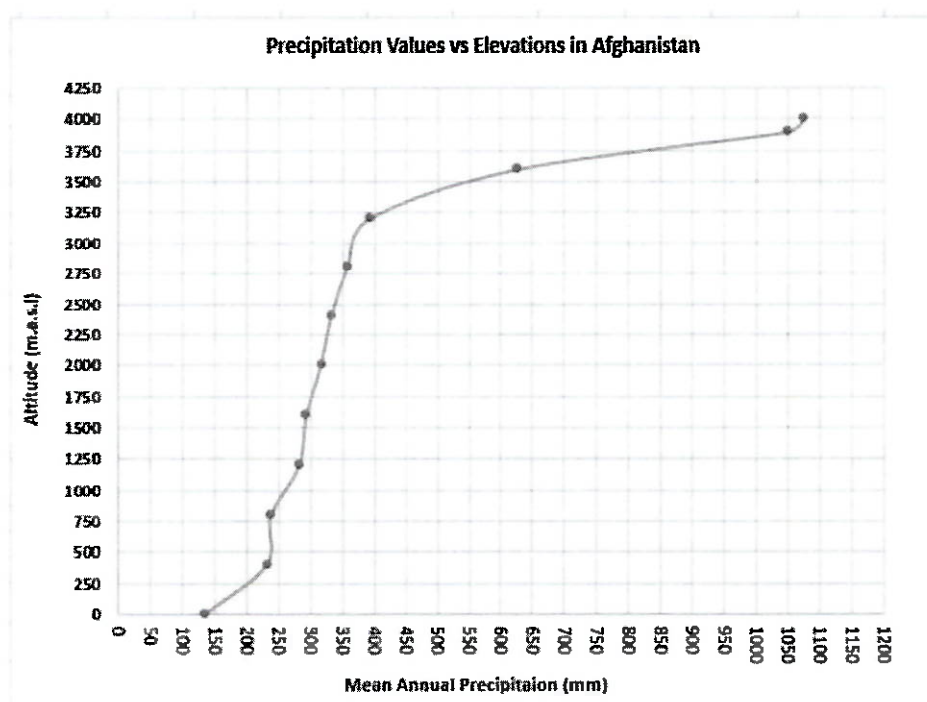


Figure 4-Illustrate Precipitation Values Vs Elevation

2.4 Monthly and Annual Rainfall Precipitation

Monthly precipitation distribution in the studied area has been the highest based on Pul-i-Ghazni station. Accordingly, maximum percentage of monthly rainfall belongs to January and is equal to 23.42 percent of annual precipitation. Meanwhile, minimum monthly precipitation belongs to September to 0.0 percent of annual precipitation per month. Based on seasonal distribution of precipitation in Pul-i-Ghazni station, winter is the highest-precipitation season of the year, which attributes to 46 percent of annual precipitation. Meanwhile, summer is the lowest-precipitation season of the year, which attributes to only 2 percent of annual precipitation.



precipitation values in Pul-i-Ghazni station from 2009–2022 have been provided in figures. Variations in the precipitation monthly percentage and seasonal precipitation percentage have been provided.

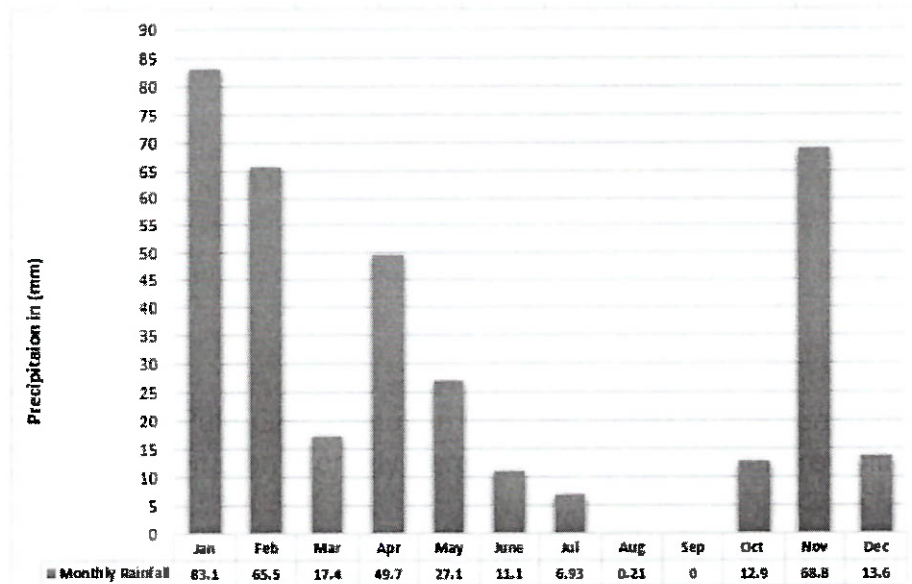


Figure 5-Illustrate Monthly Precipitation Values (2009-2022)

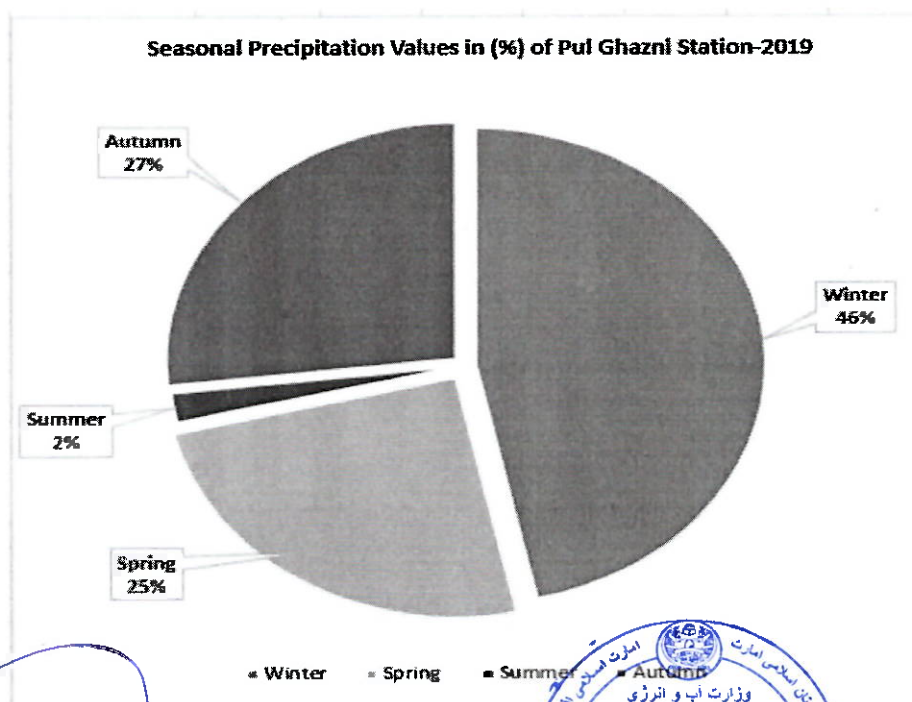


Figure 6-Illustrate Seasonal Ave. Precipitation Percentage in Pul-i-Ghazni Station (2009-2022)

2.5

Maximum 24-hr Precipitation



The maximum daily precipitation in Pul-i-Ghazni station. Based on the existing recorded data for 14 years, the maximum occurred daily Precipitation is 44 mm in 2016. The total yearly rainfall from 2009 – 2022 is tabulated below.

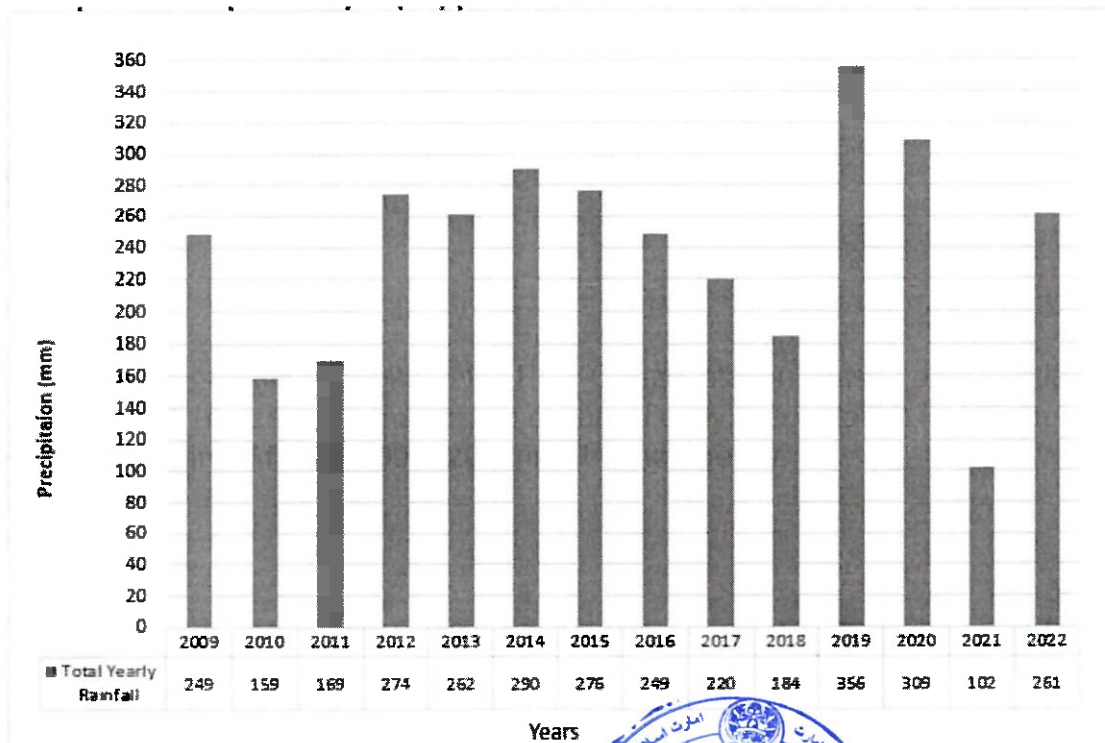


Figure 7 Illustrate Total 24-hrs Yearly Maximum Precipitation for Pul-i-Ghazni Station (2009-2022)



Table 5 provides the mean monthly and annual humidity values at Pul-i-Ghazni station. Average relative humidity at this station is 47.09 percent, varying from 26.9 percent in June to 76.2 percent in February.

Table 5 Average Monthly and Yearly Relative Humidity in Pul-i-Ghazni Station

Parameter	Month												Annual (%)
	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	
Mean Relative Humidity	50.8	76.1	70.7	52.7	41.3	26.9	42.8	45.4	38.8	32.7	43.2	43.7	47.09

3.4 Sunshine Hours

Sun radiation is a determinant factor in the evaporation. This parameter is measured as sunshine hours using a heliograph. Average sunshine hours at Gardiz station have been provided in table 6. Average total of annual sunshine hours at Gardiz station is 3487 hours. Minimum average of sunshine hours in January is 219 hours and its maximum value in July is 362 hours. Due to the missing data in Pul-i-Ghazni station, the Gardiz station is utilized.

Table 6 Monthly and Yearly Sunshine Hours for Gardiz Station

Parameter	Month												Annual
	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	
Average Sunshine Hours	219	258	248	297	344	351	362	333	290	291	246	239	3478.00

3.5 Wind Speed

Horizontal or sub horizontal movement of the air is called wind. Wind functions as a cause of other processes such as perspiration, scattering of clouds, variations in temperature and wind erosion in arid climate. Ground topography, sun radiation and man-made facilities affect air movement and make for different wind speeds on the surface. Wind is one of the meteorological factors with specific importance due to its effect on evaporation, perspiration and making waves in dam reservoirs and so, it is used in hydraulic designs. Average monthly and annual wind speed values at Gardiz station have been used in relevant estimates as this station is adjacent to the dam site. Table 7 provides the average monthly and annual wind speed at Gardiz station. Average wind speed at Gardiz station is 1.52 m/s, varying from an average of 1.33 m/s in October to 1.69 m/s in June. Due to the missing data in Pul-i-Ghazni station, the Gardiz station is utilized.

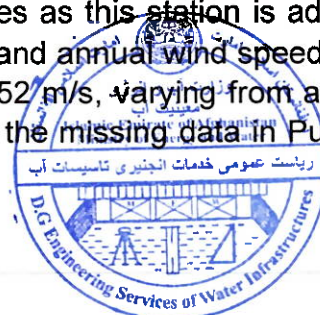


Table 7 Max. Min. and Average Wind Speed for Gardiz Station

Parameter	Month												Annual (m/s)
	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct	Nov	Dec	
Max. Wind Speed	6.32	5.66	4.33	5.99	5.14	4.6	4.71	4.74	4.7	5.07	4.17	4.06	6.32
Average Wind Speed	1.42	1.61	1.57	1.67	1.63	1.69	1.68	1.6	1.47	1.33	1.27	1.35	1.52
Min. Wind Speed	0.23	0.24	0.24	0.29	0.28	0.27	0.39	0.25	0.27	0.25	0.22	0.24	0.22

3.6 Open Water Evaporation

Open water evaporation at the dam site has been calculated in Empirical method and using climatic parameters at Gardiz and Pul-i-Ghazni stations. figure 8 shows the results of such estimates. Total annual evaporation from water free level at the Sultan dam site is **977.09 mm**, varying from 12.37 mm in February to 182.5 in June.

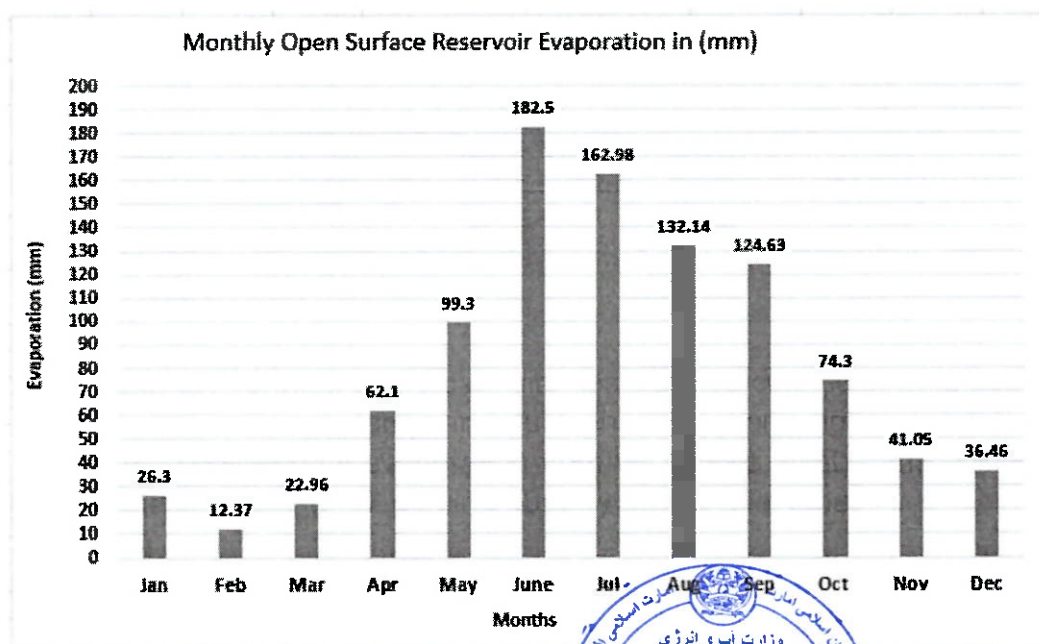


Figure 8- Illustrate Monthly Open Surface Reservoir Evaporation (mm)



Chapter Four Specification of the Catchment Area, Rainfall and Flood

4.1 River System and Specification of the Catchment Area

The Ghazni stream originates at an altitude of 4562 m in the North Mountains the main stream flows northward for a length of about 58 km up to Sultan dam axis. The catchment area has a plain slope which show in figure 9. The catchment area of the Sultan dam up to the proposed dam site is **1172 km²**.

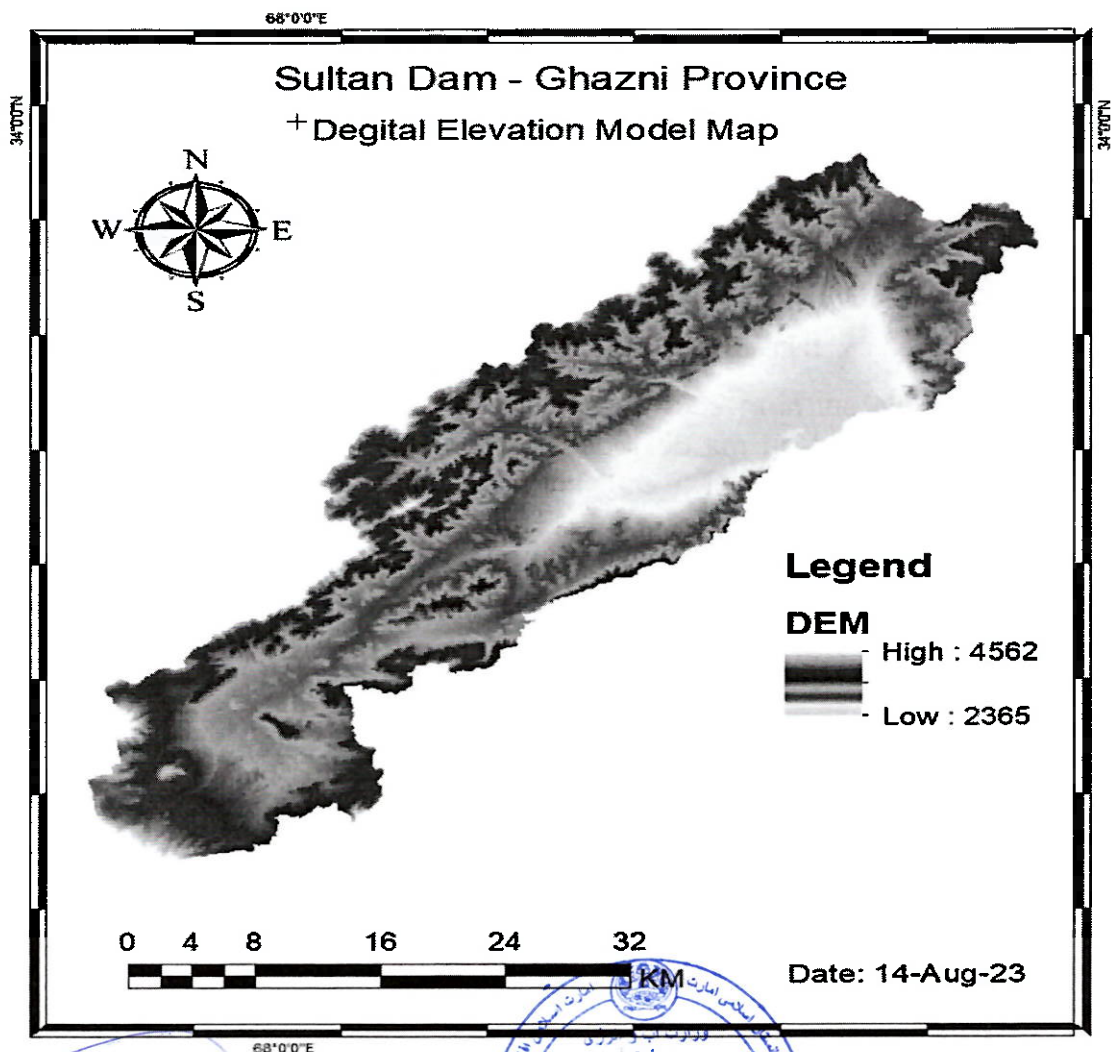


Figure 9-DEM General Map Catchment

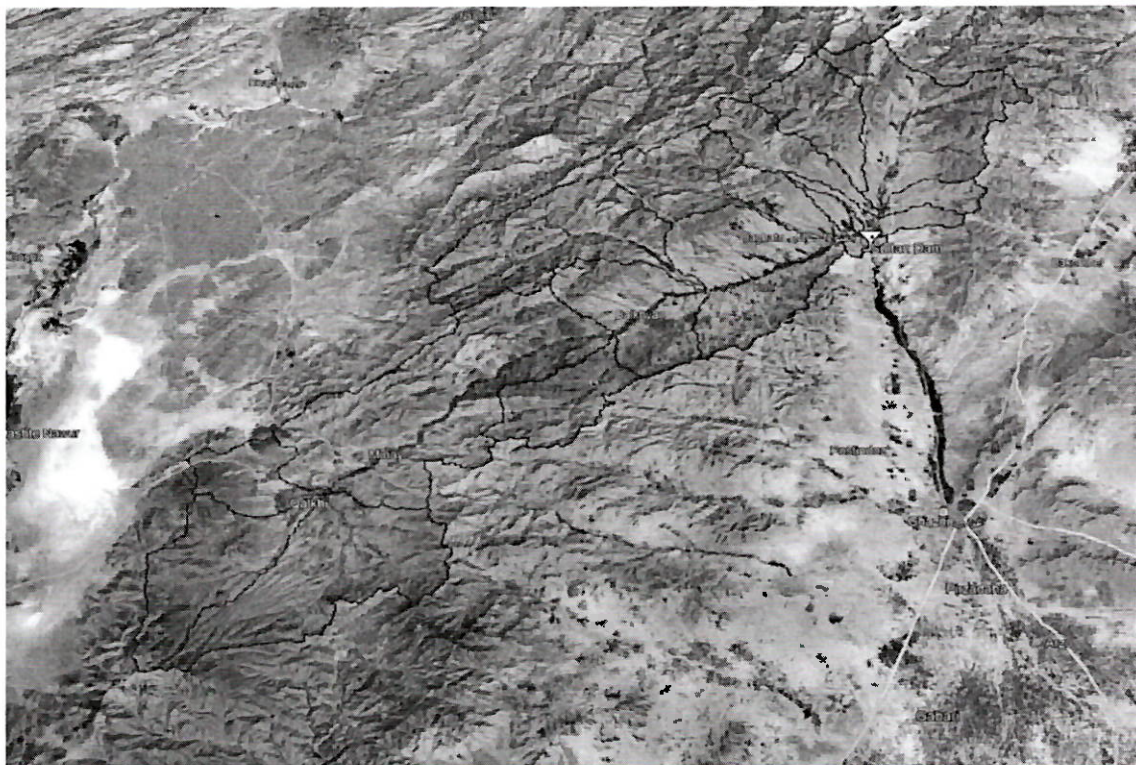


Figure 10- Google Map of the Sultan Dam

The Sultan catchment has many sub-rivers which generate the total discharge in outlet of the catchment and the sub-rivers shows in figure 11.

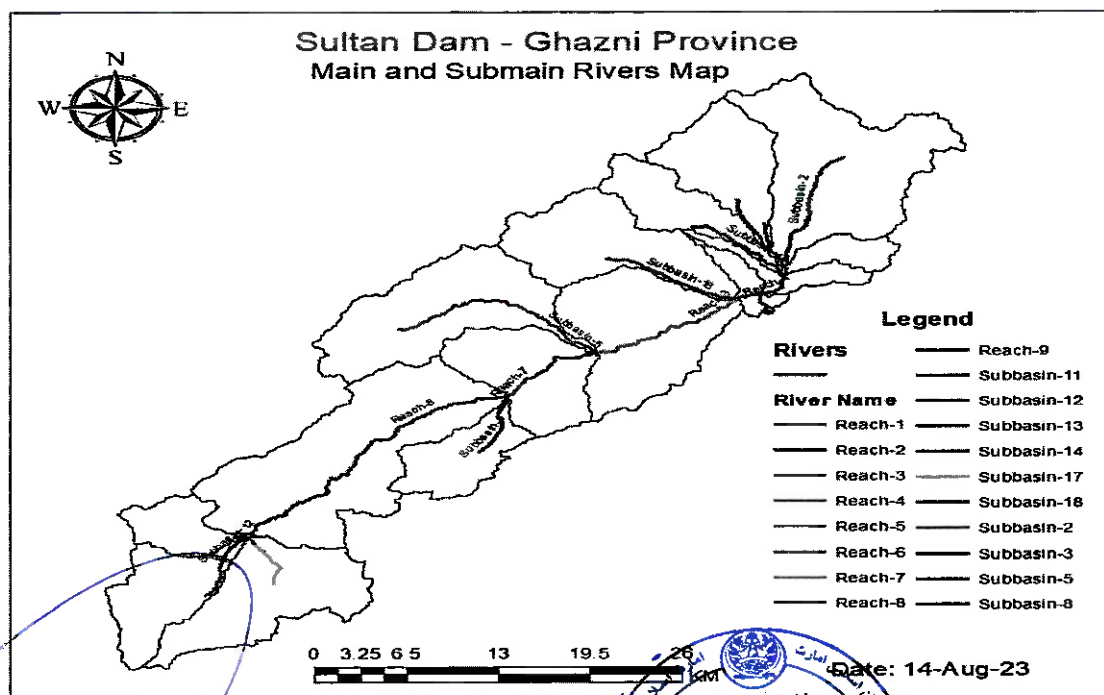


Figure 11-Sultan Dam Watershed Main and Sub-Rivers

[Basin 1]

Filter: **—None—** Sorting: **Alphabetic**

Reach	Length (KM)	Slope (M/M)	Relief (M)	Sinuosity
Reach-1	1.91897	0.00469	9.00000	1.32610
Reach-2	2.08461	0.00816	17.00000	1.20595
Reach-3	13.82210	0.00818	113.00000	1.23243
Reach-4	3.32643	0.01052	35.00000	1.32656
Reach-5	1.28487	0.01401	18.00000	1.28709
Reach-6	2.51777	0.01668	42.00000	1.19847
Reach-7	9.78523	0.00848	83.00000	1.24980
Reach-8	28.68710	0.00760	218.00000	1.27911
Reach-9	0.45671	0.02190	10.00000	1.18157

Re-compute Ap... Cl...

According to the FAO Land Cover Atlas-2016 and the Sultan catchment contains many types of soils which show in figure 13.

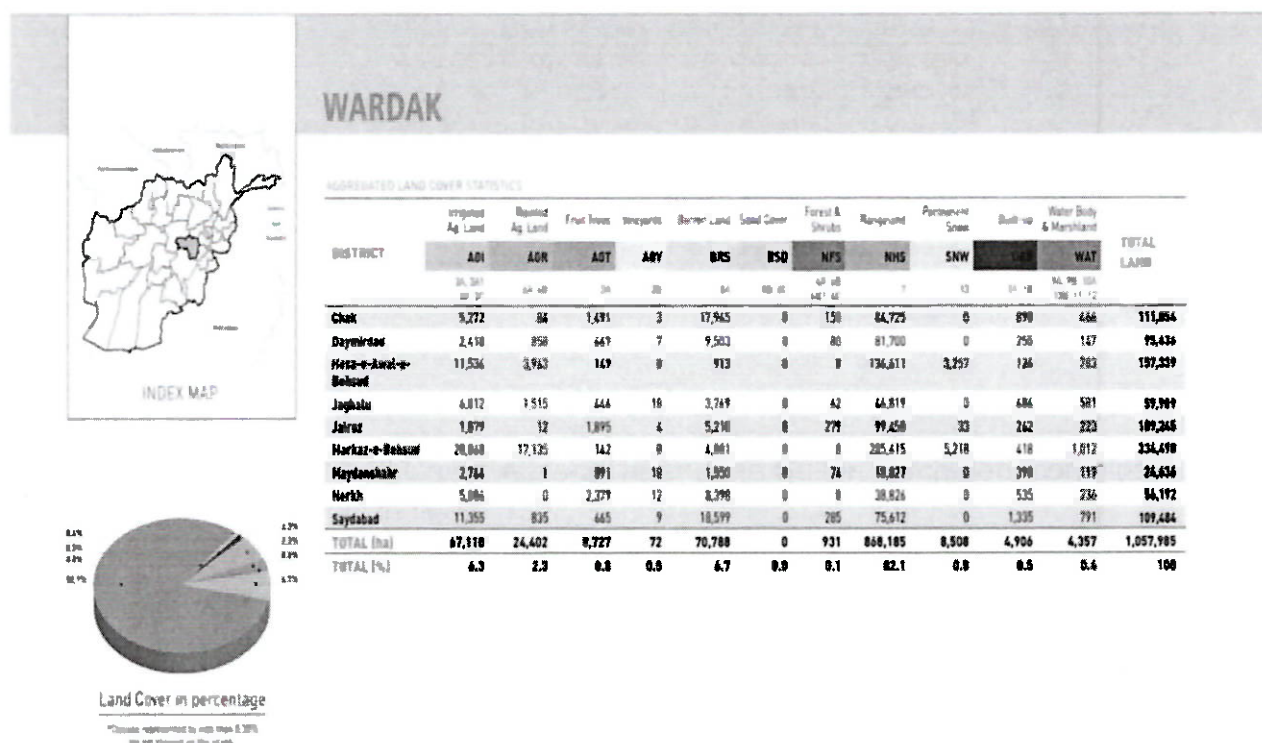


Figure 13- Illustrate FAO Land Cover Maidan Wardak Province Characteristics

The Sultan catchments divided in to 15 sub catchments which generates total discharge of basin and all sub basins are shown in below figure.



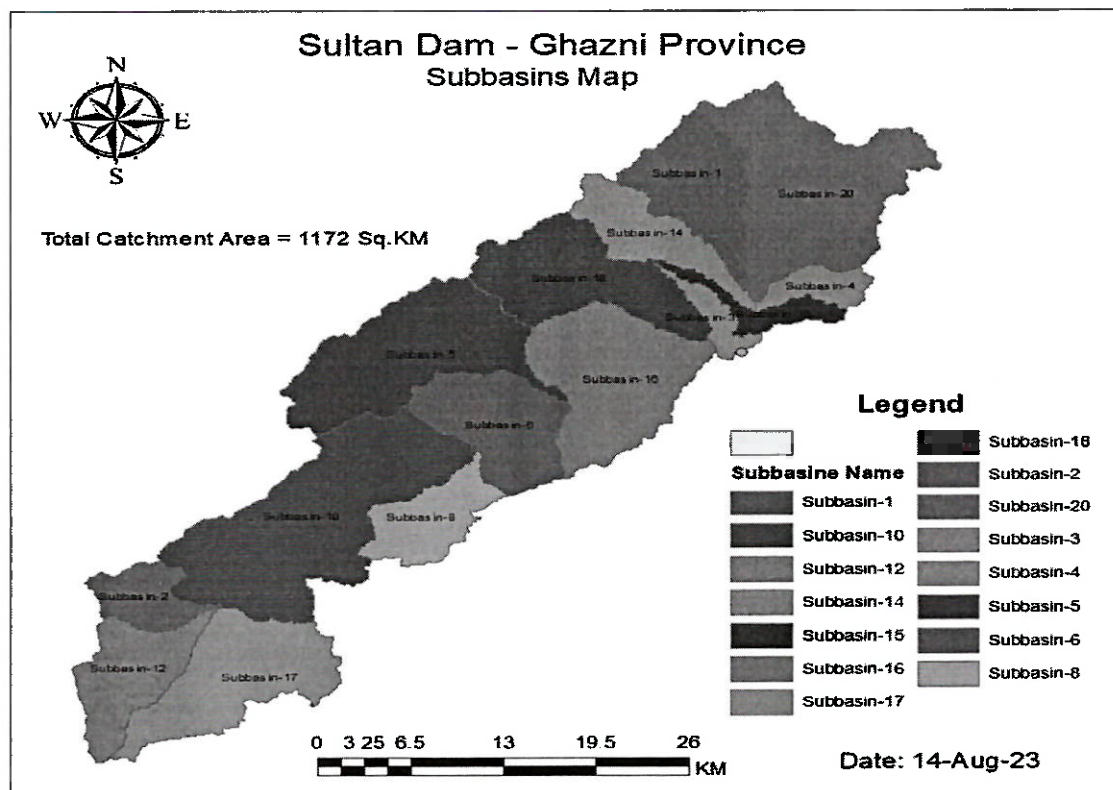


Figure 14-Sultan Dam Watershed Subbasins

And all subbasins characteristics are tabulated in below table

Table 9- All Physical Characteristics of Sultan Dam Subbasins

Subbasin Characteristics [Basin 1]

File... -None- Sort... Alphab...

Subbasin	Longest Flowpath Length (KM)	Longest Flowpath Slope (M/M)	Centroidal Flowpath Length (KM)	Centroidal Flowpath Slope (M/M)	10-85 Flowpath Length (KM)	10-85 Flowpath Slope (M/M)	Basin Slope (M/M)	Basin Relief (M)	Relief Ratio	Elongation Ratio	Drainage Density (KM/KM ²)
Subbasin-1	19.04118	0.06176	13.18235	0.02321	14.28088	0.02925	0.32567	1304.00000	0.06848	0.49099	0.13710
Subbasin-10	33.66622	0.03553	14.81833	0.00884	25.24967	0.01108	0.31506	1235.00000	0.03668	0.46239	0.15065
Subbasin-12	19.62719	0.08521	11.35654	0.03377	14.72039	0.06039	0.23854	1748.00000	0.08906	0.45022	0.13425
Subbasin-14	21.45353	0.05216	12.92092	0.02366	16.09015	0.03522	0.31659	1370.00000	0.06386	0.37046	0.20298
Subbasin-15	13.71575	0.01976	3.69746	0.01569	10.28681	0.01903	0.08122	358.00000	0.02610	0.37123	0.16267
Subbasin-16	21.16698	0.04391	9.48381	0.00733	15.87524	0.01891	0.14702	930.00000	0.04394	0.57699	0.11781
Subbasin-17	19.69918	0.07269	6.75719	0.01909	14.77438	0.05363	0.18881	1432.00000	0.07269	0.56663	0.06644
Subbasin-18	22.00775	0.04669	11.77306	0.02298	16.50581	0.02965	0.30930	1267.00000	0.05757	0.50264	0.11846
Subbasin-2	13.86557	0.05496	5.51537	0.03010	10.39917	0.04057	0.20294	1101.00000	0.07941	0.45464	0.06441
Subbasin-20	25.68209	0.03520	13.41356	0.01558	19.26156	0.02824	0.23376	1287.00000	0.05011	0.54080	0.11117
Subbasin-3	12.00057	0.02204	4.12442	0.01519	9.00043	0.02299	0.09025	266.00000	0.02217	0.36607	0.26507
Subbasin-4	11.91901	0.04640	6.59038	0.01942	8.93926	0.03274	0.15950	647.00000	0.05428	0.42080	0.06431
Subbasin-5	27.99758	0.03977	15.61017	0.01880	20.99819	0.02472	0.39555	1245.00000	0.04447	0.45645	0.15315
Subbasin-6	14.91940	0.04092	6.78763	0.00818	11.18955	0.01655	0.19243	986.00000	0.06609	0.66417	0.12670
Subbasin-8	16.69290	0.04673	8.39585	0.02251	12.51968	0.02608	0.26651	897.00000	0.05374	0.46848	0.14612

Re-comp...



4.2 Basin Mode Component

The basin model represents the physical watershed. The user develops a basin model by adding and connecting elements. Hydrological elements use mathematical models to describe physical processes in the watershed. The descriptions of available hydrological elements are:

4.2.1 Subbasin: the subbasin element is used to present the physical watershed. Given precipitation, outflow from the subbasin element is calculated by subtracting precipitation losses, transforming excess precipitation to stream flow at the subbasin outlet, and adding baseflow.

4.2.2 Reach: the reach element is used to convey stream flow downstream in the basin model. Inflow into the reach element can come from one or many upstream hydrological elements. Outflow is simply calculated by summing all inflows and assuming no storage at the junction.

4.2.3 Junction: the junction element is used to combine stream flows from hydrological elements located upstream of the junction element. Inflow into the junction element can come from one or many upstream elements. Outflow is simply calculated by summing all inflows and assuming no storage at the junction.

4.2.4 Source: the source element is used to introduce flow into the basin model. The source element has no inflow. Outflow from the source element is defined by the user.

4.2.5 Sink: the sink element is used to represent the outlet of the physical watershed. Inflow into the sink element can come from one or many upstream hydrological elements. There is no outflow from the sink element.

4.2.6 Reservoir: the reservoir element is used to model the detention and attenuation of a hydrograph caused by a reservoir or detention pond. Inflow into the reservoir element can come from one or many upstream hydrological elements. Outflow from the reservoir element can be calculated three ways. The user can enter a storage -outflow, elevation-storage -outflow, or elevation-area-outflow relationship, or the user can enter an elevation-storage or elevation-area relationship and define one or more outlet structures, or the user can specify a time-series of outflow.

4.3 Area Reduction Factor.

Area-reduction functions describe the relationship between the point maximum precipitation observed in a storm pattern, and the average depth of precipitation averaged over a larger area. It is a highly-idealized means of describing the spatial pattern of precipitation in space. Area reduction factors (ARFs; also known as depth-area relationships) are used to convert point rainfall estimates to area-averaged estimates and are central to conventional flood risk estimation in ungauged watersheds.



4.3 Data Collection and Model

With regard to the lack of data from the studied area, the discharge of the area is calculated from daily rainfall with HEC-HMS model. This stream is mostly a seasonal stream and the source of water is mostly rainfall and less snow melting, and this is an ungauged stream. In an ungauged stream, the discharge calculates by empirical methods, catchment area ratio method, or generates models with HEC-HMS, GIS. Using model with HEC-HMS is the best and accurate method than others and the USACE recommend it.

The Hydrological Engineering Center - Hydrologic Modeling System (HEC-HMS) is designed to simulate the complete hydrologic processes of dendritic watershed systems. The software includes many traditional hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing. HEC-HMS also includes procedures necessary for continuous simulation including evapo-transpiration, snowmelt, and soil moisture accounting. The software features a completely integrated work environment including a database, data entry utilities, computation engine, and results reporting tools. A graphical user interface allows the user seamless movement between the different parts of the software.

The model result is directly connected with input data to the model, if the input rainfall duration data is in less time, the accuracy of the model is getting high. For example; hourly rainfall data accuracy is better than daily rainfall data, and minutely rainfall data is better than hourly rainfall data. Finally, we officially received the daily rainfall data from GDOWR for the Pul-i-Ghazni and Gardiz meteorological stations (Pul-i-Ghazni and Gardiz stations are the nearest meteorological stations for the Sultan dam axis).

All hydrological and metrological data which were needed in Sultan dam model are from Pul-i-Ghazni and Gardiz stations. Pul-i-Ghazni station is far about 24 km from the dam axis the Gardiz station is far about 81 Km from the dam axis.

For the current model for Loss calculation the Deficit and Constant method is selected, and for transform calculation the Clark Unit Hydrograph method is selected, for the baseflow the Linear Reservoir method is selected, and for the river routing the Muskingum method is selected.

In addition to the above explanations, sultan dam has three hydrological stations and right now they are inactive, due to the 45 years of civil war in the country all those stations are damaged are completely destroyed. The last data which are recorded by these stations are from 1948–1970, due to the climate changes and other factors, these data are ignored in this report.



4.4 Reaches Routing

For better analysis and reach the optimum discharge values in HEC-HMS model, reaches routing is necessary and the below table shows the reaches characteristics and design factors, for the reaches routing the Muskingum method is utilized. The X values range is between (0.1 - 0.5) and for this project is assumed 0.25.

Table 10-Reaches Characteristics K and X Values

Muskingum (Basin 1)

Filter: --None-- Sorting: Alphabetic

Reach	Initial Type	Initial Discharge (M3/S)	Muskingum K (HR)	Muskingum X	Number of Subreaches
Reach-1	Discharge = Inflow		0.0264	0.25	1
Reach-2	Discharge = Inflow		0.0928	0.25	1
Reach-3	Discharge = Inflow		0.2773	0.25	1
Reach-4	Discharge = Inflow		2.8132	0.25	3
Reach-5	Discharge = Inflow		0.9084	0.25	1
Reach-6	Discharge = Inflow		0.1667	0.25	1
Reach-7	Discharge = Inflow		1.3065	0.25	1
Reach-8	Discharge = Inflow		0.1973	0.25	1
Reach-9	Discharge = Inflow		0.2396	0.25	1

Compute: All Elements Calculator... Apply Close

4.6 Total Yearly and Monthly Flood and Base Flow Volume

For calculation of the flood water volume the HEC-HMS hourly precipitation result is exact and accurate because the input data is hourly. For the calculation of water volume for irrigation purpose, the hourly precipitation is utilized and the result is tabulated as below.

Table 11-Average Monthly and Yearly Water Volume

Total Monthly Water Volume (M.m3) for Sultan Dam												
Years/Months	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2009	5.26	0.00	23.03	20.86	7.26	2.65	5.281	1.369	1.586	1.586	5.429	6.819
2010	10.295	13.143	14.308	9.151	7.434	1.735	8.298	1.282	0.494	0.494	5.232	0.166
2011	1.227	14.322	12.663	18.555	7.897	2.637	0.494	0.380	7.330	5.675	4.268	0.467
2012	14.633	25.429	13.952	12.355	9.753	3.021	1.020	1.859	11.094	2.143	7.258	21.161
2013	9.514	17.633	21.966	31.266	3.425	4.831	1.445	3.079	0.138	0.076	16.952	7.368
2014	8.809	27.206	44.061	15.455	13.890	0.273	0.328	0.052	0.000	2.554	4.393	11.543
2015	8.291	21.724	18.217	41.099	2.250	0.439	12.863	2.208	0.353	6.691	10.240	0.173
2016	2.710	26.203	20.705	15.365	15.369	8.574	9.407	2.561	0.888	0.166	0.055	0.556
2017	13.599	49.870	12.072	5.706	2.775	0.114	0.052	0.183	1.134	0.353	3.266	1.946
2018	1.182	16.907	11.011	8.982	12.625	3.867	4.949	0.325	0.114	4.092	4.209	12.227
2019	33.679	30.091	7.862	21.507	12.34	6.29	2.85	1.31	0.22	5.08	30.73	6.89
2020	20.263	16.437	46.829	15.752	9.61	0.23	1.19	8.91	1.41	0.31	12.69	2.94
2021	0.740	5.723	15.438	4.178	17.04	0.24	0.09	0.68	0.00	0.00	0.00	2.32
2022	22.575	3.242	15.697	4.448	5.64	6.38	12.80	37.33	1.61	0.34	1.98	5.72
Ave. Flood Vol. =	10.91	19.14	19.84	16.05	9.09	2.95	4.36	4.40	1.88	2.11	7.62	5.74

Average Yearly Volume = 104.10



4.7 Deficit and Constant for Loss Method:

The Deficit and Constant loss method is very similar to the Initial and Constant loss method in that a hypothetical single soil layer is used to account for changes in moisture content. However, the deficit and constant method allows for continuous simulation when used in combination with a canopy method that will extract water from the soil in response to potential evapotranspiration computed in the meteorologic model. Between precipitation events, the soil layer will lose moisture as the canopy extracts infiltrated water. Unless a canopy method is selected, no soil water extraction will occur. This method may also be used in combination with a surface method that will hold water on the land surface. The water in surface storage can infiltrate into the soil layer and/or be removed through evapotranspiration. The infiltration rate is determined by the capacity of the soil layer to accept water.

If the moisture deficit is greater than zero, water will infiltrate into the soil layer. Until the moisture deficit has been satisfied, no percolation out of the bottom of the soil layer will occur. After the moisture deficit has been satisfied, the rate of infiltration into the soil layer is defined by the constant rate. The percolation rate out of the bottom of the soil layer is also defined by the constant rate while the soil layer remains saturated. Percolation stops as soon as the soil layer drops below saturation (moisture deficit greater than zero). Moisture deficit increases in response to the canopy extracting soil water to meet the potential evapotranspiration demand. Parameters that are required to utilize this method within HEC-HMS include the Initial Deficit (inches or millimeters), Maximum Deficit [inches or millimeters], and Constant Rate (in/hr or mm/hr). The Directly Connected Impervious Area (percent) is an optional parameter and can be specified by the user.

Table 12-Deficit and Constant Method Parameters

Deficit and Constant [Basin 1]

Filter: --None-- Sorting: Alphabetic

Subbasin	Initial Deficit (MM)	Maximum Storage (MM)	Constant Rate (MM/HR)	Impervious (%)
Subbasin-1	76.2	203.2	8	0.0
Subbasin-10	76.2	203.2	8	0.0
Subbasin-12	76.2	203.2	8	0.0
Subbasin-14	76.2	203.2	8	0.0
Subbasin-15	76.2	203.2	8	0.0
Subbasin-16	76.2	203.2	8	0.0
Subbasin-17	76.2	203.2	8	0.0
Subbasin-18	76.2	203.2	8	0.0
Subbasin-2	76.2	203.2	8	0.0
Subbasin-20	76.2	203.2	8	0.0
Subbasin-3	76.2	203.2	8	0.0
Subbasin-4	76.2	203.2	8	0.0
Subbasin-5	76.2	203.2	8	0.0
Subbasin-6	76.2	203.2	8	0.0
Subbasin-8	76.2	203.2	8	0.0

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4.8 Clark Unit Hydrograph for Transform Method:

The unit hydrograph is a technique for modeling the transformation of excess precipitation to runoff at the watershed scale. When the watershed has both precipitation and flow data available for a storm event, the unit hydrograph relationship can be derived directly. When the watershed is ungauged, the unit hydrograph relationship cannot be derived directly; therefore, many synthetic unit hydrograph methods have been developed to help with ungauged watersheds (though not limited for use to just ungauged watersheds). There are different synthetic unit hydrograph methods available in HEC-HMS, such as the Snyder and SCS Unit Hydrograph Methods, for which the user can choose. In this project the Clark unit hydrograph (UH) method is applied.

Table 13-The Clark Unit Hydrograph Parameters

Clark Unit Hydrograph (Basin 1)

Filter: --None-- Sorting: Alphabetic

Standard Variable Maricopa County AZ USA

Subbasin	Time of Concentration (HR)	Storage Coefficient (HR)	Time Area Method
Subbasin-1	4.07	7.57	Default
Subbasin-10	5.79	10.8	Default
Subbasin-12	3.53	6.55	Default
Subbasin-14	4.08	7.58	Default
Subbasin-15	2.69	4.99	Default
Subbasin-16	4.07	7.55	Default
Subbasin-17	3.08	5.71	Default
Subbasin-18	4.1	7.62	Default
Subbasin-2	2.72	5.04	Default
Subbasin-20	4.5	8.36	Default
Subbasin-3	2.6	4.82	Default
Subbasin-4	2.83	5.25	Default
Subbasin-5	4.93	9.16	Default
Subbasin-6	3.38	6.28	Default
Subbasin-8	3.48	6.46	Default

Compute: All Elements Calculator... Ap... Cl...

4.3 Discharge Calculation

For Discharge calculation as mention previous, the HEC-HMS software with the daily rainfall data is used. The maximum instantaneous Discharge **309.1 m³/sec** is occurred in February 04, 2017. the yearly peak discharges from 2009 to 2022 are tabulated in below table. Peak discharges which are generated from the HEC-HMS model.



Table 14-Peak Yearly Discharges from HEC-HMS (2009 – 2022)

No	Year	Peak Discharges (m3/sec)
1	2009	147.10
2	2010	124.60
3	2011	146.50
4	2012	202.40
5	2013	209.70
6	2014	235.50
7	2015	240.50
8	2016	298.70
9	2017	309.10
10	2018	142.90
11	2019	216.30
12	2020	206.40
13	2021	150.80
14	2022	230.10
Average=		204.33

The peak discharges values are come from the result of HEC-HMS software from 2009 to 2022.

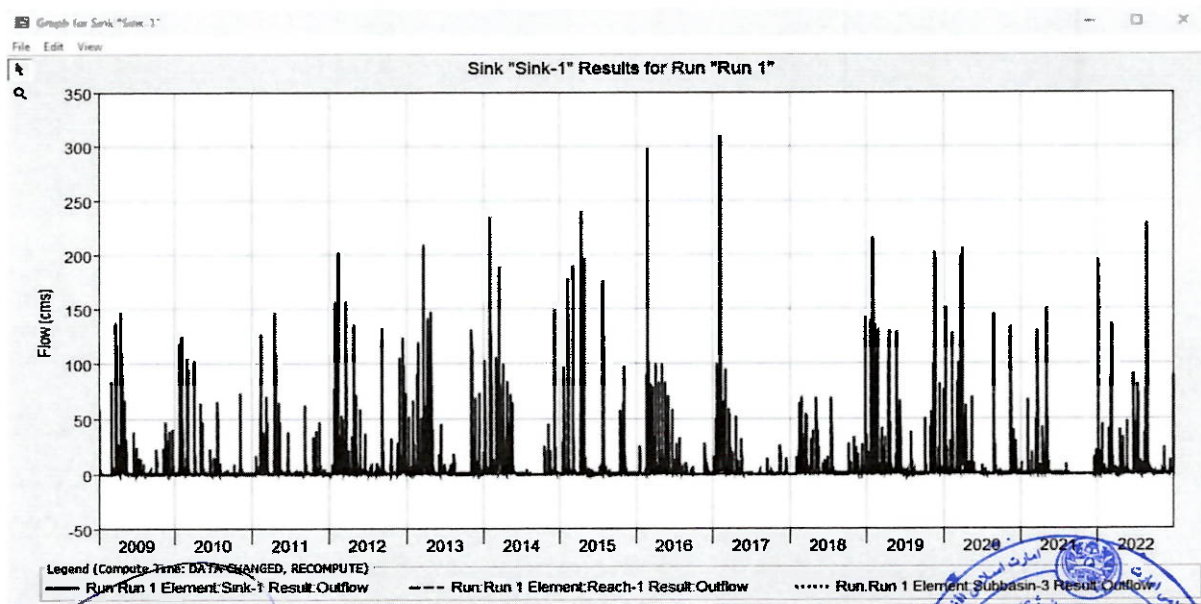


Figure 15-Shows Peak Discharges (m3/sec) from HEC-HMS Model 2009-2022



The global summary result for the outlet is

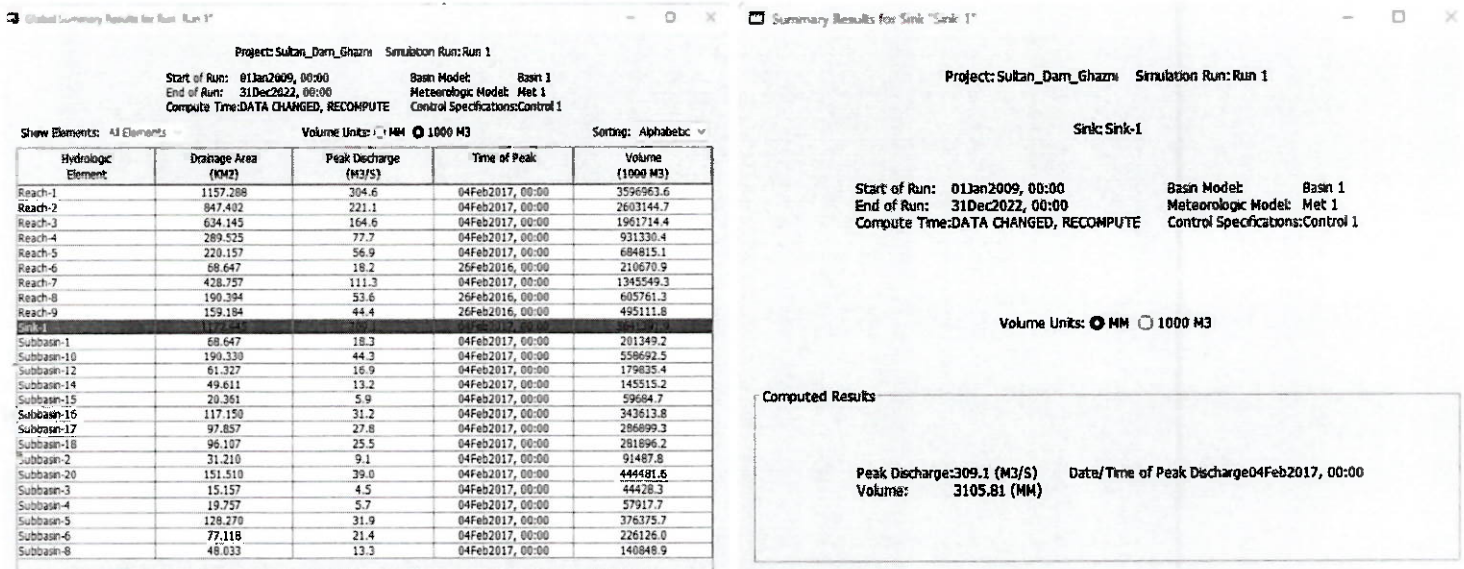


Figure 16-Shows Summary Result of Sink

For getting the high accuracy result in HEC- HMS model the Sultan dam catchment is divided into 15 sub-basins. The smaller the watersheds, the better the accuracy of the model result, and the discharge for all 15 sub-basins are calculated by model, and only for sample one sub-basin W10 discharge model calculation shown in figures.

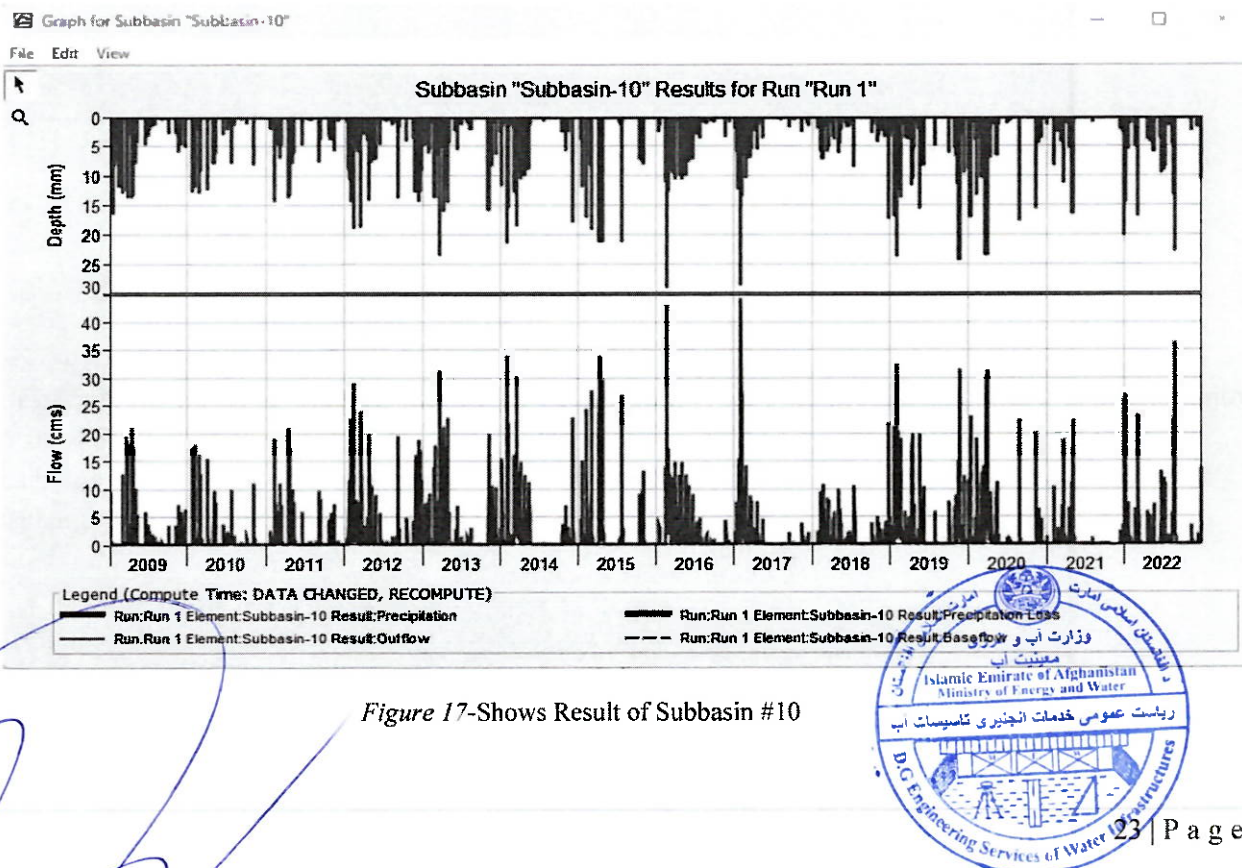


Figure 17-Shows Result of Subbasin #10

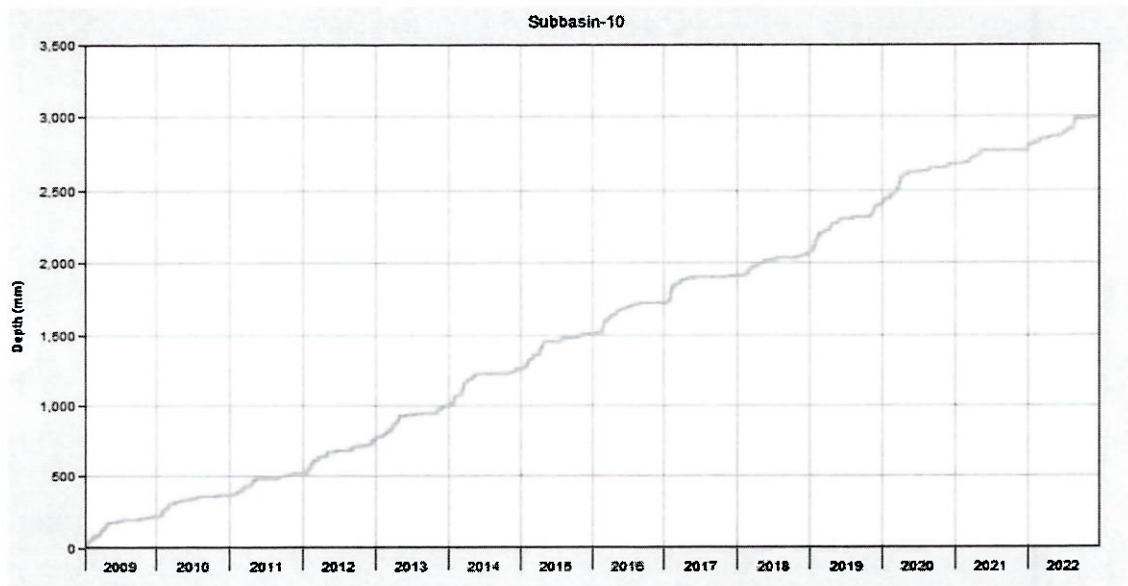


Figure 18-Cumulative Precipitation in Subbasin #10

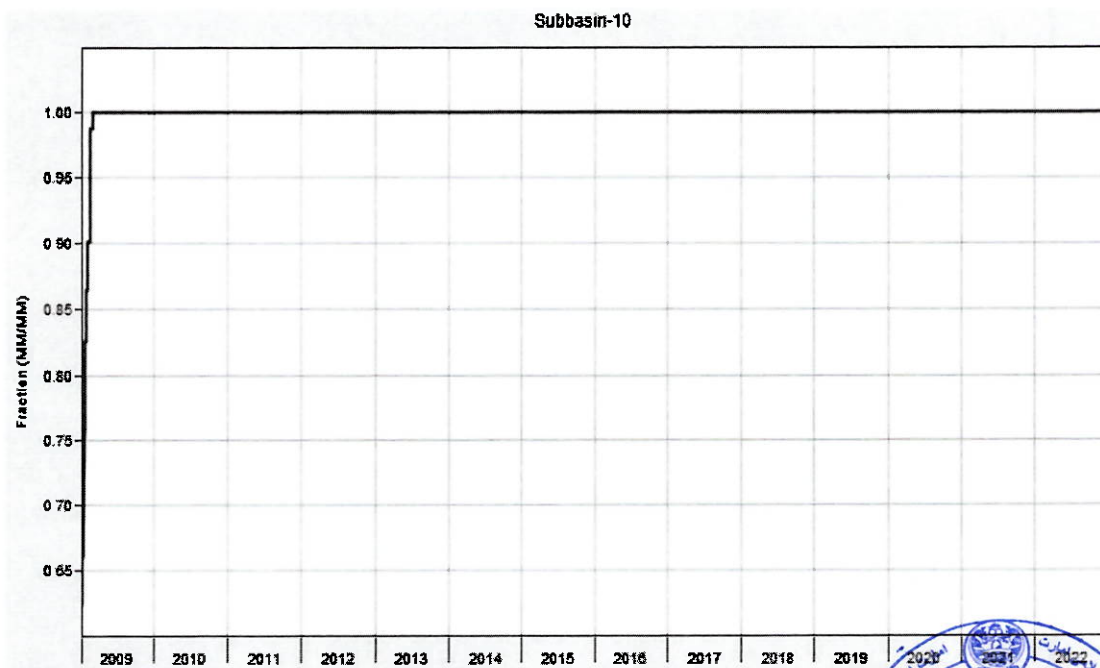


Figure 19-Saturation Fraction in Subbasin #10



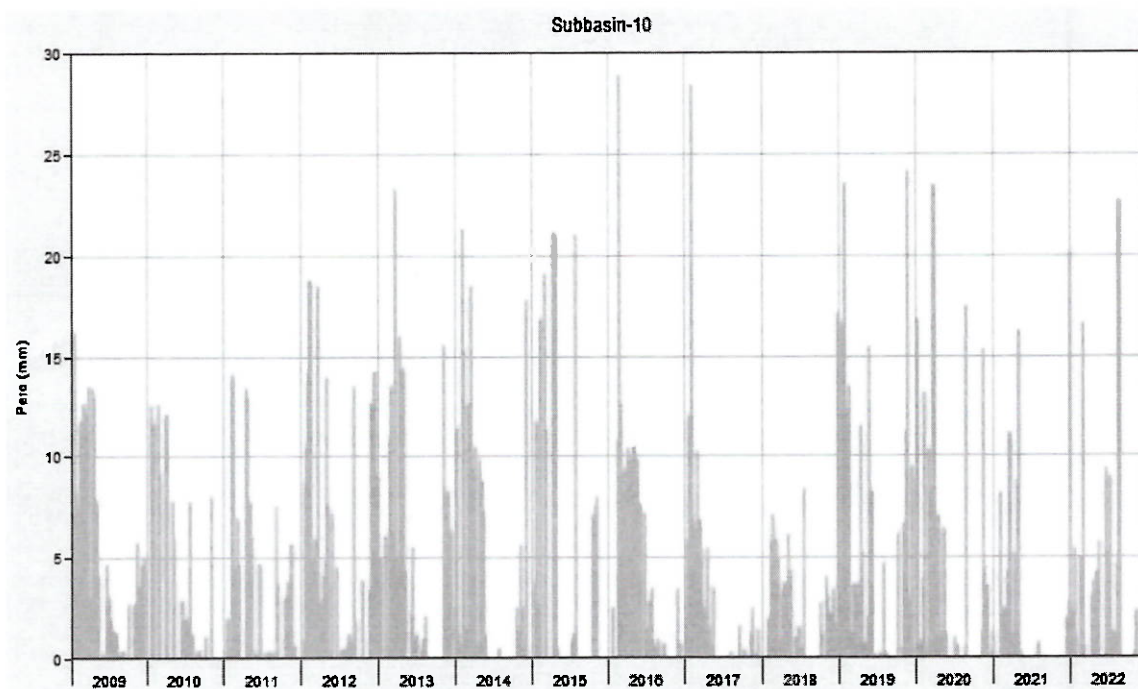


Figure 20-Soil Infiltration in Subbasin #10

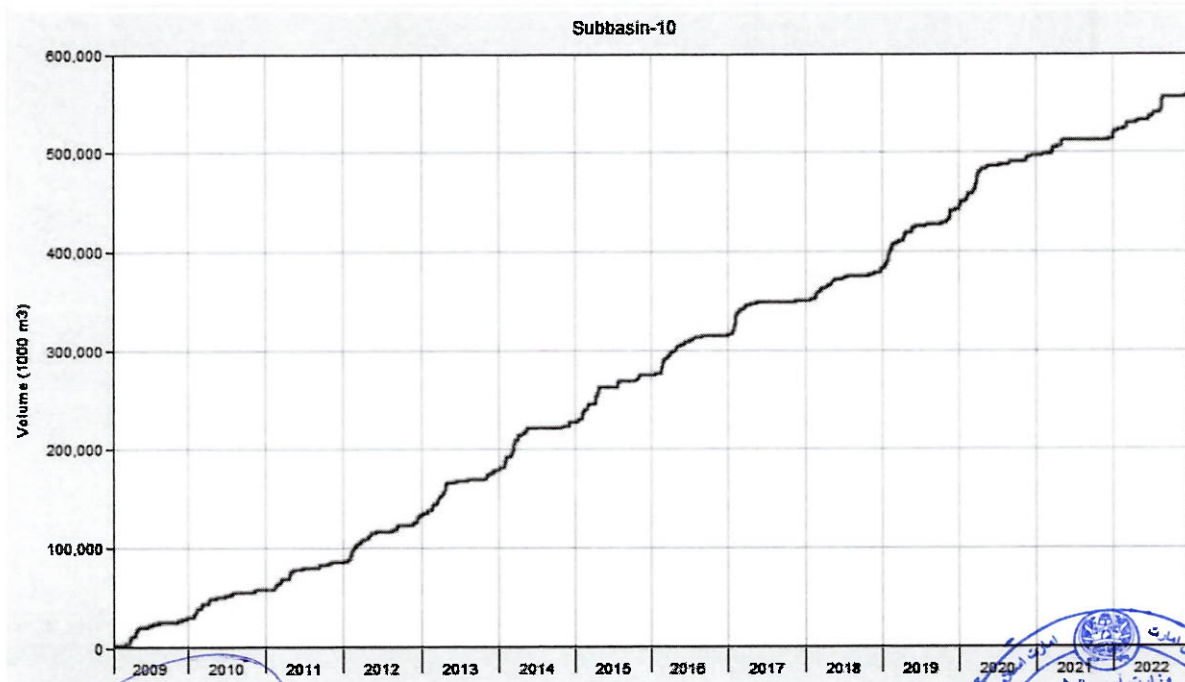


Figure 21-Cumulative Outflow in Subbasin #10

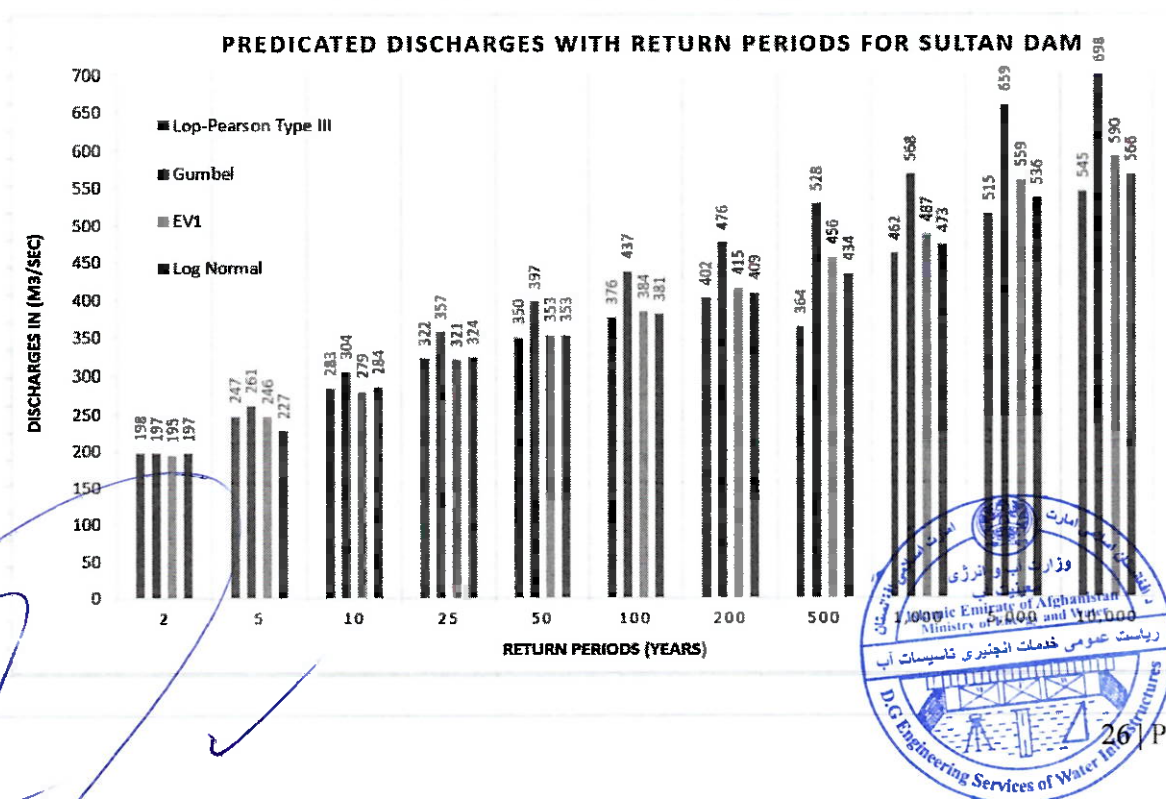


For the design of spillway structure, the predicated discharge is needed and for this purpose the 10 years calculated discharges should be predicated for the return periods of 2, 5, 10, 25, 50, 100, 200, 500, 1,000, 10,000 years and the Gumbel, Log-Pearson type 3, and EV1, Log Normal methods are used and tabulated in below table, and USBR recommendation is the Log Pearson-III.

Table 15 Predicated Discharges Summary Table

Return Period (years)	Log Pearson-III (m³/sec)	Gumbel (m³/sec)	EV1 (m³/sec)	Log Normal (m³/sec)
2	198	197	195	197
5	247	261	246	227
10	283	304	279	284
25	322	357	321	324
50	350	397	353	353
100	376	437	384	381
200	402	476	415	409
500	364	528	456	434
1,000	462	568	487	473
5,000	515	659	559	536
10,000	545	698	590	566
PMF	834	1068	903	866

Table 22-Predicated Discharges Summary Graph



4.9 Peak Discharges Calculation by Empirical Methods.

In addition, to the modeling of the catchment, the catchment peak inflow discharge to the dam axis is calculated by other empirical methods too, and these empirical methods are function of catchment area, catchment slope, land use and land cover of the basins, river slope and other characteristics of the catchment, and mostly function is catchment area. Some empirical equations are utilized and tabulated as below.

Table 16 Peak Discharges Calculation by Empirical Equations

No	Formula	Peak Discharge (m ³ /sec)
1	Deckin's	4,006.14
2	Fanning for USA	752.92
3	Inglis	4,226.65
4	Coutange for France	5,135.17
5	Mayer for USA	5,991.03
6	Jung Bahadur	4,602.88
7	Horton	2,437.50
8	Fuller	752.52

4.8 Hydrograph

A hydrograph is a graph showing the rate of flow versus time past a specific point in a river, channel, or conduit carrying flow. The rate of flow is typically expressed in cubic meters or cubic feet per second. A good way for drawing hydrograph is gauge station and recording the floods vs time. Unfortunately, the Sultan dam is an ungauged catchment and the hydrograph is derived from daily precipitation. The hydrograph with return periods is in figure.

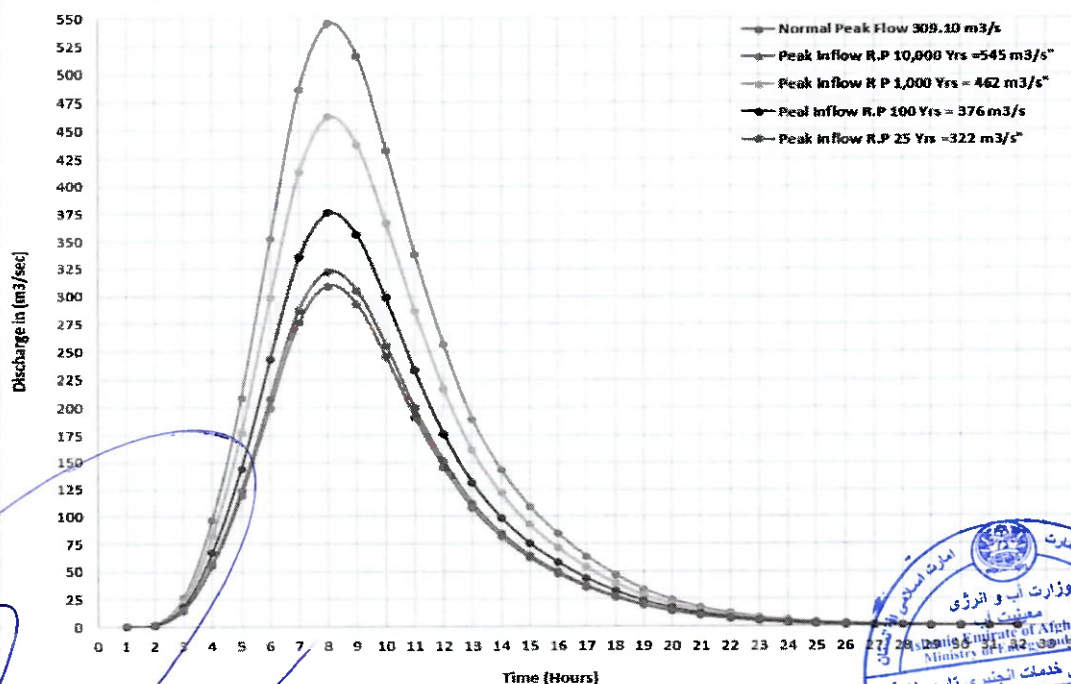


Figure 23-Hydrograph with Predicated Discharges



Flood routing and hydrograph generation form the key elements of a watershed hydrologic model. Stream channels, floodplains, and reservoirs can have a significant impact on the delivery of water to any location along a stream network. Flood routing impacts the magnitude of the peak discharge, the time of the peak discharge, depth and extent of flooding, and environmental factors such as stream bank erosion, floodplain scour, sediment transport, and deposition. Flood routing is variously defined as follows:

Routing, flow—A mathematical procedure that predicts the changing magnitude, speed, and shape of a flood wave as a function of time at one or more points along a watercourse (Maidment 1993).

These definitions relate to flood routing in streams and rivers. Definitions related to reservoirs and breaching dams are:

Routing, reservoir—This procedure derives the outflow hydrograph from a reservoir from the inflow hydrograph into the reservoir with consideration of elevation, storage, and discharge characteristics of the reservoir and spillways. The conservation of mass equation is solved with the assumption that outflow discharge and volume of storage are directly related.

The continuity equation used in reservoir routing methods is that for the conservation of mass: for a given time interval, the volume of inflow minus the volume of outflow equals the change in volume of storage. The principal assumption is that the water in the reservoir level (level pool routing). For PMF and return period of 10,000 years flood routing of Sultan dam, the ogee spillway and broad crested type is selected $L_1 = 50 \text{ m}$, $L_2 = 200 \text{ m}$, $H = 1.7 \text{ m}$, $H_{\max} = 3.041 \text{ m}$, and reservoir water surface area is about 1.095 sq.km . The Sultan reservoir routing result shown in below figure.

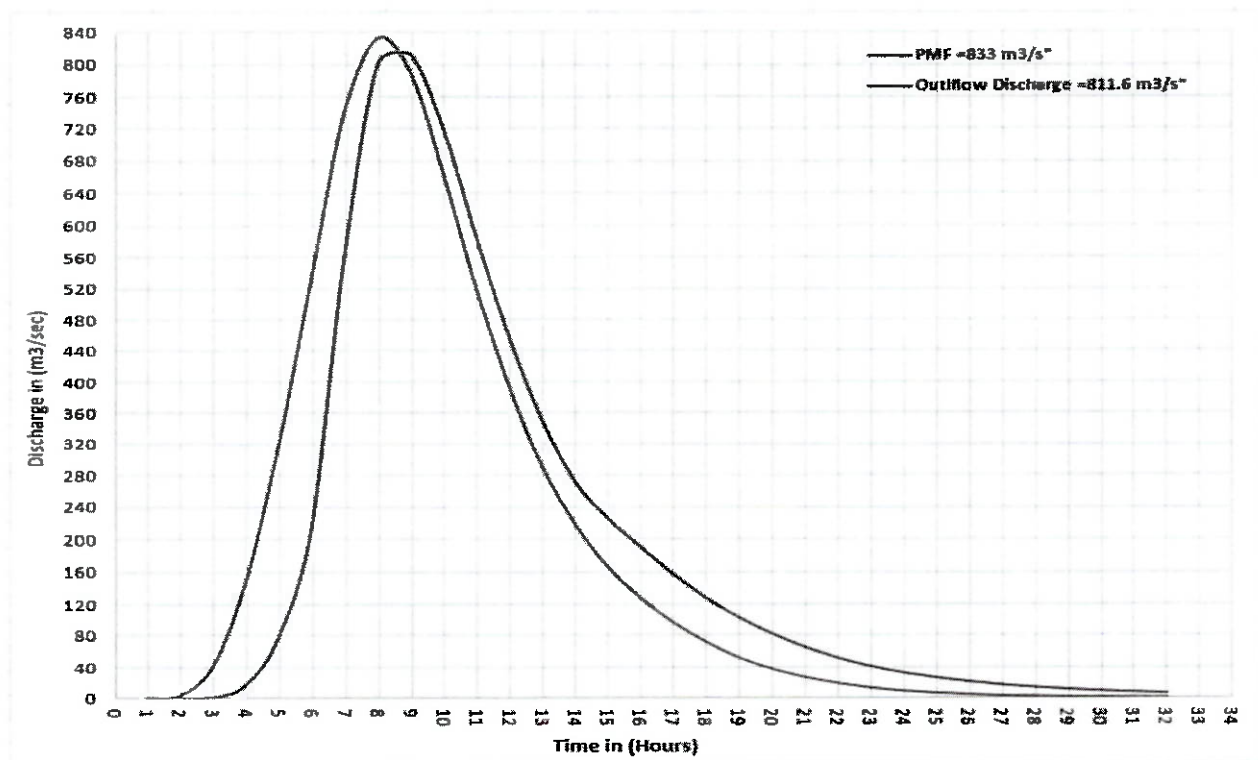


Figure 24-Inflow and Outflow Discharge for PMF

For the second option, for return period of 10,000 years flood routing of Sultan dam is utilized, the ogee spillway and broad crest type is selected $L_1 = 50$ m, $L_2 = 200$ m, $C = 1.7$ $H_{max} = 2.26$ m. The Sultan reservoir routing result shown in below figure.

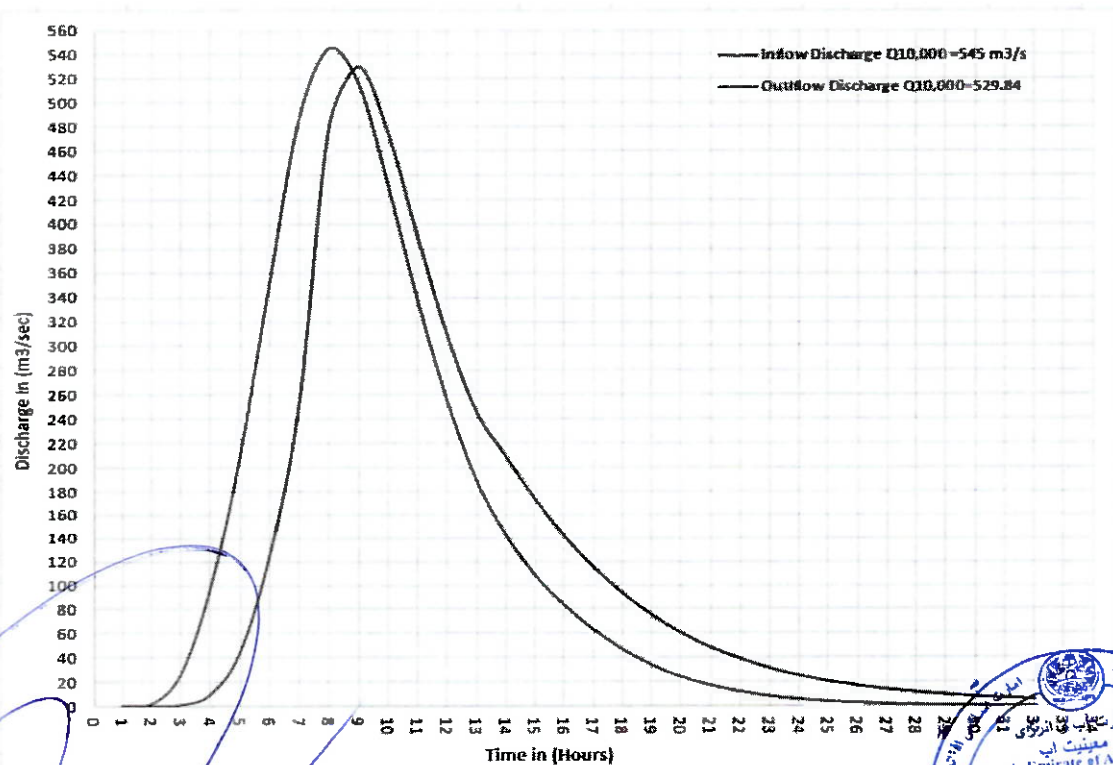


Figure 25 Inflow and Outflow Discharges for Return Period of 10,000 years

Chapter Five Sedimentation

5.1 introduction

Runoff resulted from rainfall throughout catchment areas and river flows are always accompanied by soil erosion and sediment takes. As the dead capacity of reservoirs and de-silting activities are affected from sediment data, it is of high importance to determine this parameter.

5.2 Sediment Calculation Methods: There are number of methods in order to calculate the amounts of sediments deposited at the reservoir area, in the first method, the data should obtain from at least one of the gauging stations within the project area. This data must involve the periodical measurements of sedimentation in accordance with the international standards. In basin basis the amount of suspended and accumulated material must be known. Unfortunately, no gauge station is available in the project site to capable sediment measurement.

The second method is using the empirical equations, The third method is using the FAO and (MONENCO 1980) tables. And the fourth one is using EKASU consultancy selected sediment amount in this project last methods will be used and select the best one.

The following empirical equations have been developed by several investigators, for estimating the annual sediment yield of a reservoir.

5.2.1 Swami's Regression Equation.

Swamy and Grade (1977) have proposed a relation correlating the cumulative volume inflow of sediment deposited in a reservoir with the cumulative volume of water inflow, and the initial bed slope, as

$$V_s = C \times B \times (V_{ci})^{0.94} \times (S_o)^{0.84}$$

Where V_s = Cumulative vol. of sediment deposited in the reservoir in M. m³

C = Regression constant with safe value of the order of 1.16.

B = width of the reservoir at full reservoir level in m, and S_o = bed slope of the river

V_{ci} = cumulative vol. of inflow per unit width B of the reservoir

For the current project, catchment area = 1172 km², bed slope = 0.0076, reservoir average width is 560 m, the annual yearly inflow into the reservoir in M.m³ is



Table 17-Yearly Inflow to the Reservoir in (M.m3)

2009	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020	2021	2022
81.14	72.03	75.91	123.68	117.69	128.56	124.55	102.56	91.07	80.49	158.86	136.58	46.46	117.76

Table 18-Computation of sediment Load by Swamy's Regression Method

S. No	Year	Inflow (M.m ³)	Cumu. Inflow (M.m ³)	Vci = Cumu. Inflow per unit with B of resrv. =Col (4)/B	Vs=10.78 (Vci) ^{0.94} (M.m ³)
1	2	3	4	5	6
1	2009	81.14	81.14	0.145	1.75
2	2010	72.03	153.18	0.274	3.19
3	2011	75.91	229.09	0.409	4.65
4	2012	123.68	352.77	0.630	6.98
5	2013	117.69	470.46	0.840	9.15
6	2014	128.56	599.02	1.070	11.48
7	2015	124.55	723.57	1.292	13.71
8	2016	102.56	826.13	1.475	15.53
9	2017	91.07	917.20	1.638	17.14
10	2018	80.49	997.69	1.782	18.55
11	2019	158.86	1156.55	2.065	21.31
12	2020	136.58	1293.12	2.309	23.67
13	2021	46.46	1339.58	2.392	24.47
14	2022	117.76	1457.34	2.602	26.48

$Q_s = 1,344 \text{ m}^3/\text{km}^2/\text{year}$, this amount of annual sediment yield is too high and it is negligible.

5.2.2 Jogelkar's Equation. An equation proposed by Jogelakar (1960) is given as:

$$Q_s = 0.59 \times A^{-0.24}$$

Where Q_s = annual silting rate in M .m3 per 100 sq. km of catchment area

$$Q_s = 0.59 \times 1172^{(-0.24)} = 902 \text{ m}^3/\text{km}^2/\text{Year}$$

this amount is also too high and it is negligible.



5.2.3 Khosla's Equation. Khosla (1953) has proposed the following empirical equation.

$$Q_s = 0.323 \times A^{-0.28}$$

Where Q_s = annual silting rate in M. m³/100 sq.km/year

$$Q_s = 0.323 \times 1172^{-0.28} = 372 \text{ m}^3/\text{km}^2/\text{year}$$

This amount of annual yield sediment is better than two above one, and it is high and negligible.

5.2.4 FAO and MONENCO 1980 Tables. Sediment data in Afghanistan is almost non-exist the scanty data obtained from FAO's world river database is shown in (Table 20). Other source of sediment yield data (Table 21) in Afghanistan is by Montreal Engineering Company (MONENCO 1980). These data will be used to determine sediment transport and reservoir sedimentation at the dam site.

Table 19-Afghanistan Rivers Sediments Yields - FAO

River	Area	Monitoring		R	Ro	Y	Ref. Citation
	km ²	Start Date	End Date	mm/yr	(mm/yr)	(t/km ² /yr)	(Data Source)
River		Start	End	Rainfall	runoff	Yield	Source
Ghorband	4440	1962	1963	900	191	420	Tkachev et al, 1965
Hari Rud	11700	1962	1963	350	121	270	Tkachev et al, 1965
Kabul	130010	1962	1963	530	149	410	Tkachev et al, 1965
Kabul	1900	1962	1963	350	100	280	Tkachev et al, 1965
Logar	4720	1962	1963	350	70	190	Tkachev et al, 1965
Pajshir	13810	1962	1963	900	274	455	
Panjshir	5610	1962	1963	900	351	750	Tkachev et al, 1965
Shakhar Darya	1060	1962	1963		125	273	
					Average	381	



Table 20 Sediment Yield Data in Afghanistan (MONENCO 1980)

River	Location	Sediment Yield	Data Source
Panjehir	Panjehir I	275	(a)
Panjehir	Baghdara	455	(a)
Ghorband	Totumdara	420	(a)
Maidan	Hajian	250	(a)
Logar	Kajab	250	(a)
Logar	Gat	150	(a)
Kabul	Tangi Gharu	140	(a)
Kunar River	not provided	780	(b)
Arghandab	Arghandab Reservoir	250	(c)
Helmand	Kajakai	200	(d)
Ghorband	Pul-i-Ashrawa	420	(e)
Hari Rud	Tagau Gaza	270	(e)
Kabul	Naghu	410	(e)
Kabul	Tangi Saidan	280	(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)

The MONENCO (1980) data is more comprehensive; the average sediment yield of all measurements is **342 ton/km²/year**. Sediment yield is highly variable from catchment to catchment according to the topography, geologic and geo-morphologic situation.

5.2.5 EKASU Turkish Consultancy Company. The EKASU company selected **40.66 M³/km²/year** sediment annual yield amount in their hydrology report for this catchment area and this amount is adopted for this project catchment.

This amount of sediment annual yield is looking logical because it has been more than a century that this old dam is built and still it has not been filled of sediment.

The catchment area at the dam site is 1172 km², therefore, the total sediment transport per year is **47,653-M³/year**.

The capacity inflow ratio of Sultan dam is about 0.072 with mean annual flow 104 Mm³ and reservoir capacity is **7.535 Mm³** at the crest elevation of 2380.5 m (Figure 29). The sediment trapping efficiency of the reservoir, therefore, is about 0.87 from Brune's Curve. Therefore, about 87% of the annual sediment reaching the reservoir will be deposited.

calculation 87% trapping efficiency, the annual sediment deposition at the reservoir will be 41,458 m³. In addition, about 15% of bed load is assumed to be transported therefore, the total annual sediment deposition will be **47,677 m³**.

In 50 years, the total sediment deposition in the reservoir will be **2.383 Mm³**, and in 100 years, the total sediments deposition in the reservoir will be **4.768 Mm³**

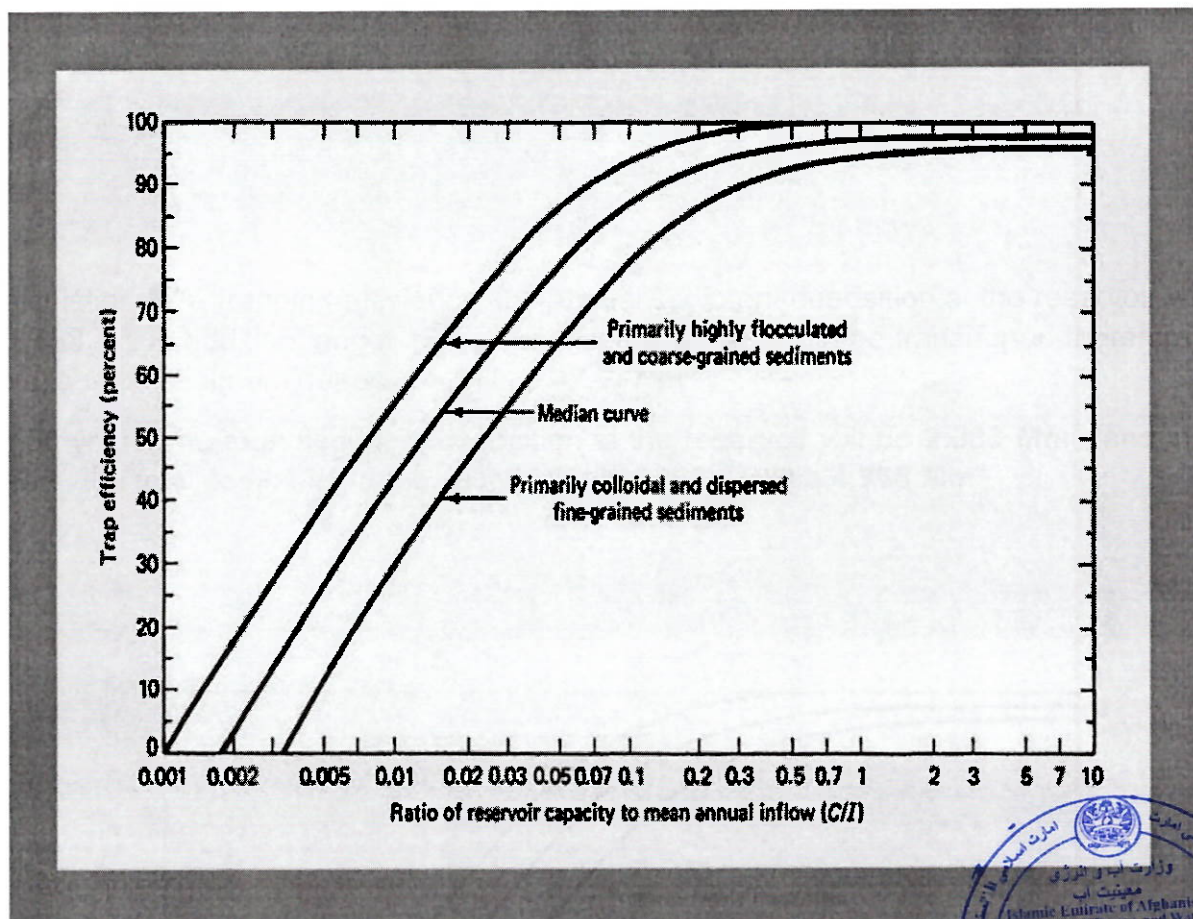


Figure 26-Capacity Inflow Ratio (Brun's Curve 1953)



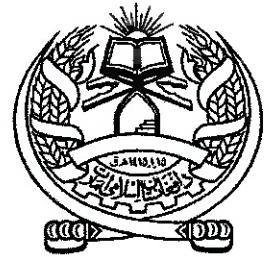
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Islamic Emirate of Afghanistan
Ministry of Water and Energy
Deputy Ministry of Water



General Directorate of Engineering Services of Water Infrastructures
Technical Board



Rehabilitation of Sultan Dam			
Date		Dec 2023	Report of hydro mechanical and Instrumentations
Prepared	Checked	Approved	
Faridullah Haidary	Murtaza Barialy	General Directorate of Engineering Services	



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1. Project Location and Information

The Sultan Dam or Band-e Sultan is located on the Ghazni river in the Kwaja Omari district of Ghazni province in Afghanistan. The dam site lays estimated 35 km north from Ghazni town. It can be reached on an unpaved roadway along the Ghazni river valley.

The dam site is located at a height of 2,400 m above sea level. Higher hills and mountains are around this location.

The dam is a lime mortar stone masonry dam with following main dimensions:

- 34 m high
- 200 m long/Crest Length
- 4.90 m Crest Width
- 32 m long side spillway on natural rock with no downstream stilling basin
- about 200 m long shoulder dam (with 6 m to 10 m high stone masonry over ground)
- Two outlet pipes each of 1.2 m diameter and equipped with manually driven operating and emergency valves.

2. Bottom Outlet Pipes

This section covers the repairing of the two existing bottom outlet pipes and the rehabilitation of manually driven operating and emergency gate valves in the main dam. The rehabilitation of two existing bottom outlet pipes and gate valves will allow for fine regulation of the reservoir's outflow and will prevent water loss, as is currently experienced with the existing outlets and gate valves, which are leaky and no longer watertight.

The existing right bottom outlet pipe must remain open for the majority of the construction duration in order to pass the river discharge.

The existing left bottom outlet pipe will only be operational for a few weeks during the initial construction of the saddle dam and during repairing of the right bottom outlet pipe.

The rehabilitation of the intake structure has been considered on the right bottom outlet to decrease the acting water pressure on bottom outlet gate valves.

As part of the civil works, the space between existing outlet pipes and dam body (stone masonry) have to be filled segment-wise and complete with a non-shrinking concrete mix with compression strength of B 25 (25 N/mm²). Finally, grouting must be carried out and, if necessary, repeated until a watertight structure is attained without any remaining gap between pipes and stone masonry walls. If necessary, the grouting has to be repeated until no penetration of water can be observed. The purpose of this work is to avoid vibration and guarantee water tightness along the existing outlet pipes.

The lower portions (half of pipe's lower section) of existing bottom outlets with an inner diameter of 1200 mm have been damaged due to corrosion and erosion, which will be repaired by installation of new half steel pipe with an inner diameter of 1180 mm and a

thickness of 10 mm. The new half of the pipes must be welded to the existing pipes at the two edges and can run through the existing pipes and coexist.

Construction and rehabilitation of the bottom outlets requires that the reservoir be completely emptied and the silt and clay sediments be excavated from the reservoir. The bottom of the reservoir is covered by wet clay to a depth of several meters.

The old right-bottom outlet pipe must be kept open to pass the remaining permanent river discharge (up to 5 m³/s has to be considered during construction) around the construction site.

Cofferdams (sandbags) and pumps are required to keep the work areas dry during the installation of the new right-bottom outlet pipe.

The half of pipe's lower sections will be made of coated mild steel, DN 1,180 mm, t = 10 mm, to be delivered in standard lengths of 3,000 mm, and all connections shall be welded joints. The allowed pipe pressure shall be PN 10 bar.

On the downstream side of the main dam, the existing dual gate valves (DN 1200 mm, PN 10 bar) for each outlet pipe, which are leaking for the time being, shall be repaired. The valves are housed in concrete structures with removable roof slabs, allowing mobile cranes to replace them in the future. The housing will include steel platforms and ladders for valve maintenance and operation via manual drives. The two housings will be closed by extremely strong steel doors with heavy-duty locks.

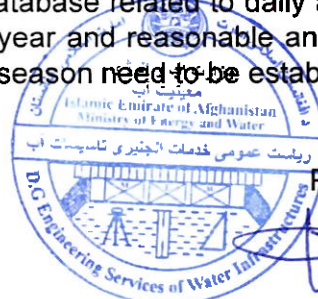
On the downstream side of the valves, a roadway of 4.85 m in width will allow vehicles access to the final outlet structure stabilized by reinforced concrete walls.

In order to prevent sedimentation in the closed pipes and for maintenance purposes, each pipe shall be closed on the upstream side with a gate, which is manually operated from the existing intake tower.

3. Hydropower Plant

Whenever efforts are made to study the feasibility of installing fully automated equipment or upgrading dams and reservoirs, the viability of implementing a small hydropower plant is justified.

In addition to the two existing bottom outlet pipes, it is considered necessary to install a new penstock taking water from level 2383.5 to level 2373.5 and connecting it to the turbine generator sets in order to transform the acting pressure head into electrical energy. The generated power may not only be used for operating gates, valves and automated instrumentation but also for local power supply to the villages in the vicinity of the dam. In order to be able to presenting reliable figures concerning installed capacity and expected electricity generation, an improved database related to daily and monthly inflow data, reservoir level records throughout the year and reasonable and applicable irrigation water supply program during the irrigation season need to be established in the future.



3.1. Hydraulic Calculation of Penstock Pipe

Available Data:

Total Discharge $Q = 4.50 \text{ m}^3 / \text{sec}$ for tow Turbine.

Discharge $Q = 2.25 \text{ m}^3 / \text{sec}$ for one Turbine.

Length of penstock pipe from inlet up to Y branch $L_{\text{penstock}} = 124.20\text{m}$, $D = 1.40\text{m}$.

Length of penstock pipe from Y branch up to power house $L_{\text{penstock}} = 43\text{m}$, $D_1 = 1.40\text{m}$, $D_2 = 1.0\text{m}$.

Maximum head $h_{\text{max}} = 2383.5 - 2348 = 35.5 \text{ m}$.

Gross head $h_{\text{gross}} = 32.5 \text{ m}$.

Number of horizontal bends $n = 1$, $\Theta = 16^\circ$.

Number of vertical bends $n = 1$, $\Theta = 16^\circ$.

Number of gate valves $n = 1$.

Number of butterfly valves $n = 0$.

Velocity in penstock $V_P = 2.92 \text{ m / s}$

Manning Coefficient for welded steel $n = 0.012$

Penstocks materials: Mild steel flat rolled and site welded.

$$L_{\text{pipe}} = \sqrt{L_{\text{Horizontal}}^2 + h_{\text{gross}}^2} =$$

3.1.1. Calculate of Penstock Diameter

$$d_{\text{penstock}} = 2.69 \left(\frac{n^2 \times Q^2 \times L_{\text{penstock}}}{H_{\text{gross}}} \right)^{0.1875} = 2.69 \left(\frac{0.012^2 \times 4.50^2 \times 167}{32.5} \right)^{0.1875} = 1.22 \text{ m}$$

$d_{\text{penstock}} = 1.22\text{m}$ we select the pipe diameter 1.40m for another calculation, $d_{\text{penstock}} = 1.40\text{m}$ OK

3.1.2. Calculate the Velocity of Water in the Penstock

Permissible velocity, $V = 0.125(2 \times g \times H_{\text{gross}})^{0.5} \text{ m/sec}$, [Ref: USBR (1961)]

$$V = 0.125(2 \times g \times H_{\text{gross}})^{0.5} = 0.125(2 \times 9.81 \times 32.5)^{0.5} = 3.16 \text{ m/sec}$$

$$V = \frac{4Q_{\text{max}}}{\pi d^2} = \frac{4 \times 4.50}{3.14 \times 1.40^2} = 2.92 \text{ m/sec}$$

$$V = 2.92 \text{ m/sec}, (2.92 < 3.16) \text{ m/sec} \dots \dots \text{OK}$$



K (roughness value) for normal mild steel is = 0.1mm from table,

Roughness values, k mm		Table 3.11.2		
Use 'normal condition' for design purposes				
Material	Age/condition			
	Good (< 5 years)	Normal (5-15 years)	Poor (>15 years)	
Smooth pipes				
PVC, HDPE, MDPE, Glass fibre	0.003	0.01	0.05	
Concrete	0.06	0.15	1.5	
Mild steel - Uncoated	0.01	0.1	0.5	
- Galvanized	0.06	0.15	0.3	
Cast iron				
New	0.15	0.3	0.6	
Old - Slight corrosion	0.6	1.5	3.0	
- Moderate corrosion	1.5	3.0	6.0	
- Severe corrosion	6.0	10.0	20.0	

Table 1: Roughness values, k mm

3.1.3. Calculate of Wall Friction and Turbulence Losses

$$1.2 \frac{Q}{d} = 1.2 \frac{4.50}{1.40} = 3.85$$

$$\frac{K}{d} = \frac{0.1}{1400} = 0.0000714 \quad f_{\text{friction}} = 0.012$$

head loss due to pipe wall friction ($h_{\text{wall loss}}$),

$$h_{\text{wall loss}} = \frac{f_{\text{friction}} \times L_{\text{pipe}} \times 0.083Q^2}{d^5} = \frac{0.012 \times 167 \times 0.083(4.50)^2}{1.40^5} = 0.626 \text{ m}$$

The total turbulence head loss ($h_{\text{turbulence loss}}$) is given by.

$$h_{\text{turbulence loss}} = \frac{V^2}{2g} (K_{\text{entrance}} + K_{\text{bend}} + K_{\text{valve}} + K_{\text{trash rack}} + K_{\text{construction}})$$



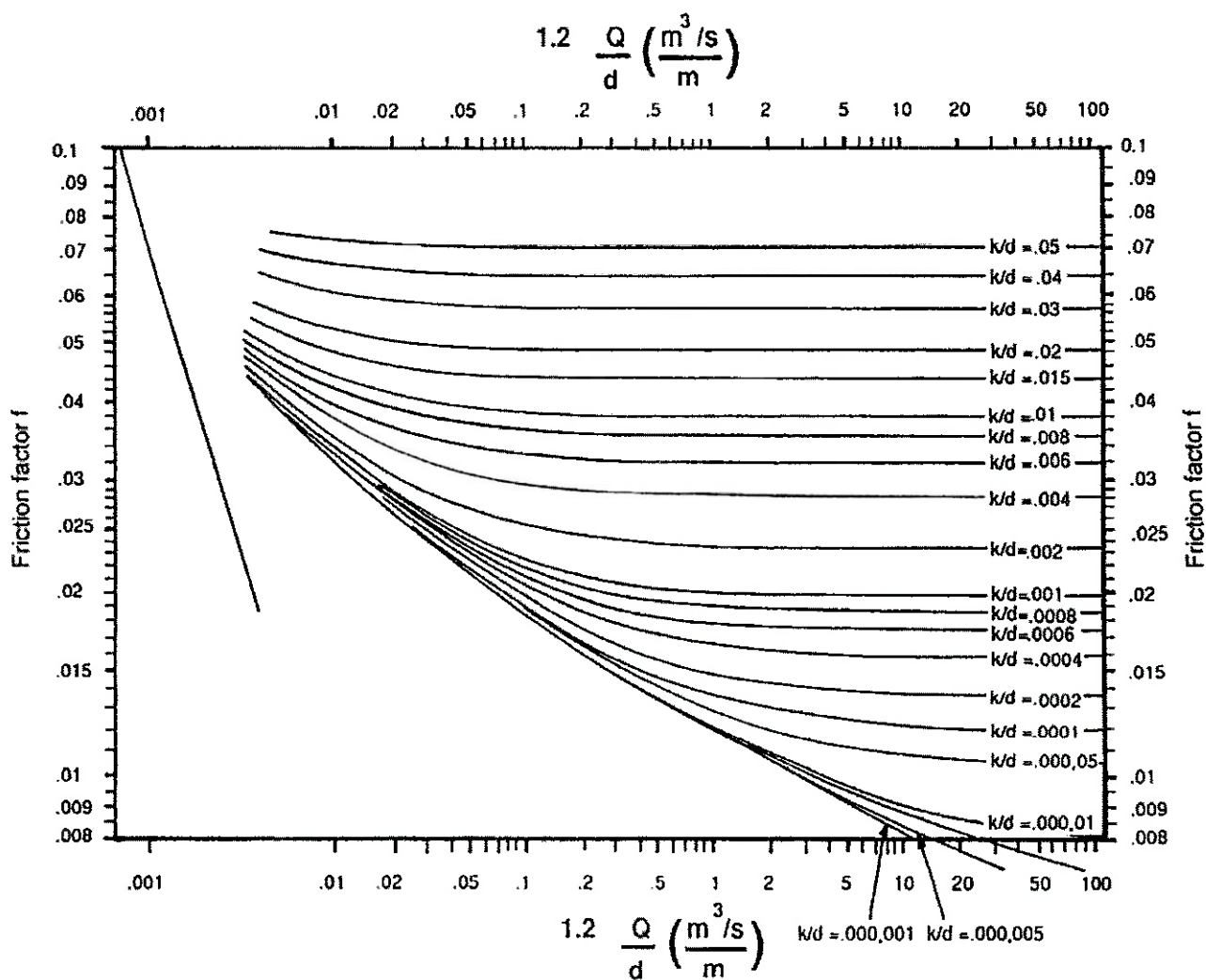


Figure 1: Moody chart

$$h_{\text{entrance loss}} = K_e \frac{V^2}{2g} = 0.2 \frac{(2.92)^2}{2 \times 9.81} = 0.0869 \text{ m}$$

$$h_{\text{bend loss}} = K_b \frac{V^2}{2g} = (2 \times 0.25) \times \frac{(2.92)^2}{2 \times 9.81} = 0.217 \text{ m}$$

$$h_{\text{valve loss}} = K_v \frac{V^2}{2g} = (2 \times 0.3) \times \frac{(2.92)^2}{2 \times 9.81} = 0.26 \text{ m}$$

Turbulence losses in penstocks

• 3.11.3

The total turbulence head loss ($h_{\text{turb loss}}$) is given by

$$h_{\text{turb loss}} = \frac{v^2}{2g} (K_{\text{entrance}} + K_{\text{bend}} + K_{\text{contraction}} + K_{\text{valve}})$$

Note that there may be more than one bend or contraction. K in all cases represents a head loss coefficient.

Head loss coefficients for intakes (K_{entrance})

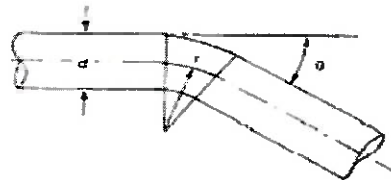
Entrance profile



K_{entrance}	1.0	0.8	0.5	0.2
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Head loss coefficients for bends (K_{bend})

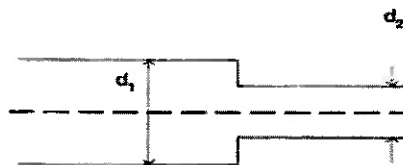
Bend profile



r/d		1	2	3	5
K_{bend}	($\Theta = 20^\circ$)	0.36	0.25	0.20	0.15
K_{bend}	($\Theta = 45^\circ$)	0.45	0.38	0.30	0.23
K_{bend}	($\Theta = 90^\circ$)	0.60	0.50	0.40	0.30

Head loss coefficients for sudden contractions ($K_{\text{contraction}}$)

Contraction profile



d_1/d_2	1.0	1.5	2.0	2.5	5.0
$K_{\text{contraction}}$	0	0.25	0.35	0.40	0.50

Head loss coefficients for valves (K_{valve})

Type of valve	Spherical	Gate	Butterfly
K_{valve}	0	0.1	0.3

Table 2: Turbulence losses in penstock

$$h_{\text{contraction loss}} = K_c \frac{v^2}{2g} = (0.25) \frac{(2.92)^2}{2 \times 9.81} = 0.1086\text{m}$$

$$h_{\text{trash rack loss}} = K_t \frac{v_{\text{section}}^2}{2g} = 1.8 \frac{(1.10)^2}{2 \times 9.81} = 0.111\text{m}$$

$$\text{or } h_{\text{trash rack loss}} = K_t \left(\frac{t}{b} \right)^{\frac{4}{3}} \left(\frac{V_o^2}{2g} \right) \sin \alpha$$

$$h_{\text{trash rack loss}} = 1.8 \left(\frac{15}{150} \right)^{\frac{4}{3}} \left(\frac{1.10^2}{2 \times 9.81} \right) \sin 90^\circ = 0.00518 \text{ m}$$

$$A_{\text{era bar}} = 0.015 \times 2.0 \times 15 = 0.45 \text{ m}^2$$

$$A_{\text{era section}} = \frac{\pi D^2}{4} = \frac{3.14 \times 2.4^2}{4} = 4.52 \text{ m}^2$$

$$A_r = \frac{A_{\text{era bar}}}{A_{\text{era section}}} = \frac{0.45}{4.52} = 0.10$$

According to experience the velocity at the entrance of the rack should be between 0.25 m/s and 1.0 m/s.

$$\text{Net Aera section} = A_{\text{era section}} - A_{\text{era bar}} = 4.52 - 0.45 = 4.07 \text{ m}^2$$

$$V_{\text{section}} = \frac{Q_{\text{Max}}}{A_{\text{era section}}} = \frac{4.50}{4.07} = 1.10 \text{ m/sec}$$

$$h_{\text{turbulence loss}} = (0.087 + 0.217 + 0.261 + 0.112 + 0.109) = 0.786 \text{ m}$$

$$h_{\text{friction loss}} = h_{\text{wall loss}} + h_{\text{turbulence loss}}$$

$$h_{\text{friction loss}} = 0.626 + 0.786 = 1.412 \text{ m}$$

percentage losses of head due to friction,

$$\% \text{ Loss} = \frac{h_{\text{friction}}}{h_{\text{gross}}} \times 100 \quad h_{\text{net}} = h_{\text{gross}} - h_{\text{friction}} =$$

$$\% \text{ Loss} = \frac{1.412}{32.5} \times 100 = 4.34 \% \quad h_{\text{net}} = 32.5 - 1.412 = 31.088 \text{ m}$$

3.1.4. Wall Thickness of Penstock

Calculation wall thickness of penstock from inlet up to power house:

Available Data:

Total Discharge $Q = 4.50 \text{ m}^3 / \text{sec}$ for tow Turbine.



Length of penstock pipe $L_{\text{penstock}} = 167 \text{ m}$.

Diameter of pipe $d_{\text{pipe}} = 1.40 \text{ m}$.

Gross head $h_{\text{gross}} = 32.5 \text{ m}$.

Net head $h_{\text{net}} = 31.088 \text{ m}$.

Number of vertical bends $n = 1$, $\Theta = 16^\circ$.

Number of horizontal bends $n = 1$, $\Theta = 16^\circ$.

Number of valves $n = 2$.

Velocity in penstock $V_P = 2.92 \text{ m / s}$

Manning Coefficient for welded steel $n = 0.012$

Penstocks materials: Mild steel flat rolled and site welded.

pressure wave velocity (a),

$$a = \frac{1400}{\sqrt{1 + \frac{2.1 \times 10^9 \times d}{E \times t}}} = \frac{1400}{\sqrt{1 + \frac{2.1 \times 10^9 \times 1.40}{200 \times 10^9 \times 0.018}}} = 1038.7 \text{ m/sec}$$

Velocity, surge head, and total head,

$$V = \frac{4 \times Q_{\text{max}}}{\pi d^2} = \frac{4 \times 3.51}{3.14 \times (1.22)^2} = 2.92 \text{ m/sec}$$

$$h_{\text{surge}} = \frac{a \times V}{g} = \frac{1038.7 \times 2.92}{9.81} = 309.17 \text{ m}$$

$$h_{\text{total}} = h_{\text{gross}} + h_{\text{surge}} = 32.5 + 309.17 = 341.67 \text{ m}$$

Safety factor (SF):

$$SF = \frac{t_{\text{effective}} \times S}{5 \times h_{\text{total}} \times 10^3 \times d} = \frac{0.018 \times 400 \times 10^6}{5 \times 341.67 \times 10^3 \times 1.40} = 3.01$$

$$SF = 3.01 > 3.0 \dots \text{OK}$$

If the safety factor is below 3, reject this penstock option and repeat the above calculation for stronger walled options. In certain circumstances it is legitimate to accept safety factors of above 2, if all of the following condition are met:

- 1- Staff experienced with similar pressures and materials.
- 2- Slow closing valves.
- 3- Low damage costs (gentle stopes, low head).



4- Careful pressure testing to total head.

Physical characteristics of common materials				Table 3.11.4
Material	Young's Modulus (E) N/m ²	Coefficient of linear expansion (α) m/m °C	Ultimate tensile strength (S) N/m ²	Density (ρ) kg/m ³
Steel	200 × 10 ⁹	12 × 10 ⁻⁶	400 × 10 ⁶ *	7.8 × 10 ³
uPVC	2.8 × 10 ⁹	54 × 10 ⁻⁶	28 × 10 ⁶	1.4 × 10 ³
HDPE/MDPE	0.2 – 0.8 × 10 ⁹	140 × 10 ⁻⁶	6 – 9 × 10 ⁶	0.9 × 10 ³
Ductile iron	170 × 10 ⁹	11 × 10 ⁻⁶	350 × 10 ⁶	0.7 × 10 ³
Cast iron	100 × 10 ⁹	10 × 10 ⁻⁶	140 × 10 ⁶	7.2 × 10 ³
Asbestos cement	variable	8 × 10 ⁻⁶	variable	1.6 – 2.1 × 10 ³
Concrete	20 × 10 ⁹	10 × 10 ⁻⁶	variable	1.8 – 2.5 × 10 ³

* In some countries steel of a lower strength may be supplied. If you are uncertain it is possible ask for samples to be tested by university laboratories. Other materials such as PVC can also be independently checked for strength (S) and elasticity (E).

Table 3: Physical characteristics of common materials

3.1.5. Stress due to Internal Pressure

$$\sigma_1 = \frac{\{P_1 \times (D + \varepsilon)\}}{2 \times (t_0 - \varepsilon) \times \varphi} = \frac{\{34.167 \times (140 + 0.1)\}}{2 \times (1.8 - 0.1) \times 0.9} = 1565 \text{ Kg/cm}^2 < 2100 \text{ Kg/cm}^2 \text{ SAFE}$$

Welding Factor $\varphi = 0.9$

Design load P_1 = Hydrostatic load at the penstock axis + Water Hammer = 32.5+309.17=341.67m

$$P_1 = \gamma \times h_{\text{total}} = 1000 \text{ kg/m}^3 \times 341.67\text{m} = 341670 \text{ kg/m}^2 = 34.167 \text{ kg/cm}^2$$

$$t = \frac{P_1 \times D}{2 \times \sigma_1 \times \varphi} + e_s = \frac{34.167 \times 1400}{2 \times 1565 \times 0.9} + 1 = \frac{47833.8}{2817} + 1 = 18 \text{ mm}$$

If the pipe is steel, it is subject to corrosion and to welding or rolling defects. Its effective thickness is therefore less than the nominal thickness quoted by the manufacturer. to find effective thickness for steel pipes.

For welding ($t_{\text{effective}} = 1.1 - 1.2 \times t$)

$$t_{\text{effective}} = 1.0 \times t = 1.0 \times 18 = 15 \text{ mm} = 18 \text{ mm}$$



$$P_{\text{pressure}} = 0.1 \times \rho \times a \times \frac{V}{g} = 0.1 \times 1 \times 1038.7 \times \frac{2.92}{9.81} = 30.9 \text{ bar}$$

The penstock Gate valve and Butterfly Valves are produced the factory according to the following parameters.

- 1- Max discharge $Q_{\text{max}} = 4.50 \text{ m}^3/\text{sec}$.
- 2- Penstock diameter $d_{\text{penstock}} = 1.40 \text{ m}$.
- 3- Wave pressure velocity $a = 1038.7 \text{ m/sec}$.
- 4- Gross head $h_{\text{gross}} = 32.5 \text{ m}$.
- 5- Surge head $h_{\text{surge}} = 309.17 \text{ m}$.
- 6- Total head $h_{\text{total}} = 341.67 \text{ m}$.
- 7- Total pressure $P_{\text{pressure}} = 30.9 \text{ bar}$

We select one Gate Valve and tow butterfly valves for penstock pipe.

3.1.6. Gate Valve Specification

No	Part Name	Material
1.	Body	Ductile iron GJS-500-7
2.	Seat ring	Aluminum-bronze CC331 G (AB1)
3.	Face ring	Aluminum-bronze CC331 G (AB1)
4.	Wedge	Ductile iron GJS-500-7
5.	Wedge nut	Alu-bronze CC333 G (AB2)
6.	Stem	Stainless Steel 1.4057 (431)
7.	O-Cord	EPDM rubber
8.	Bonnet	Ductile iron GJS-500-7
9.	Air plug	Stainless Steel
10.	Key	Steel
11.	Bolts	Steel, hot dip galvanized
12.	Blanking	Ductile iron GJS-500-7
13.	O-ring	EPDM rubber
14.	Stem cap	Ductile iron GJS-500-7
15.	Seal	Hot melt glue
16.	Gland Flange	Ductile iron GJS-500-7



17.	Thrust Collar	Alu-bronze CC331 G (AB1)
18.	Gland Flange bolt	Steel, hot dip galvanized
19.	Seal (2)	Hot melt glue
20.	Bonnet bolt	Steel, hot dip galvanized
21.	Thrust nut	Alu-bronze CW 307G
22.	Distance Piece	Ductile iron GJS-500-7
23.	Gland	Ductile iron GJS-500-7
24.	Packing	PTFE
25.	End	Flanged end to ASME B 16.5 RF (300#)
26.	DN	(55) inch and (1400)mm

3.2. Trash Rack Design

The rack is an important protection element against the inflow of solid material that can provoke damage in turbines. Longitudinal bars lean against crossbars with or not transversal reinforcement beams characterize each rack element. The rack is located at inlet entrance and can be of fixed or movable type. The rack can be defined by its spacebar, a , length in flow direction, b , thickness, c , and the total cross-section, S .

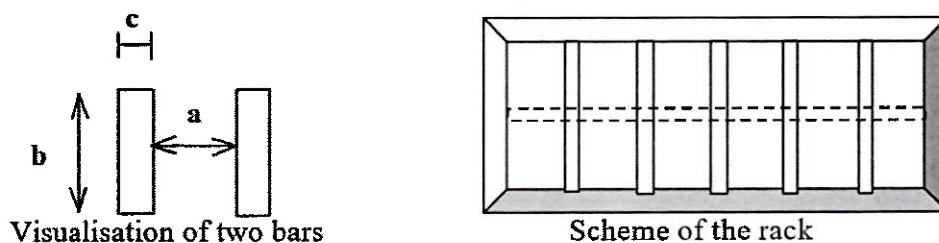


Figure 2: Scheme of trash rack

In order to avoid rack obstructions by solid materials, namely when installed in an approach canal, it will appropriate to provide it with an auto trash rack. The rack must be specified in order to avoid excessive head loss by grid obstruction if the spacebars are too small, neither to allow driving solid material into the turbine flow if the spacebars are very large.

For a submerged rack system, the maximum flow approach velocity is about 0.80 to 1.00 m/s. This velocity is based on the kinematic law:

3.2.1. Velocity based on the Kinematic Law

$$V = \frac{Q}{S} = \text{m/sec}$$

where Q is the turbine discharge value (m³/s); S is the cross-section of the rack (m²)

$$Q = 4.50 \text{ m}^3/\text{sec}$$

$$S = \frac{\pi \times D^2}{4} = \frac{3.14 \times (2.4)^2}{4} = 4.52 \text{ m}^2 - 0.45 = 4.07 \text{ m}^2$$

$$V = \frac{Q}{S} = \frac{4.50}{4.07} = 1.10 \text{ m/sec}$$

The flow through the rack can induce severe vibrations due to detached swirls with a frequency, f_s which should be different of the bars frequency, f_b , in order to avoid resonance phenomena and the collapse of the rack. According to the stability criteria, f_b must obey to the following condition:

$$f_b \geq 1.5 f_s$$

3.2.2. Calculate of Swirl Frequency, f_s (Hz)

$$f_s = \frac{St \times V}{c} = \frac{0.17 \times 1.10}{0.015} = 12.50 \text{ (Hz)}$$

Where St is Strouhal number

3.2.3. Calculate of Structural Frequency

The calculation of the structural frequency of the bar of a rack with fixed extremities, is based on the following equation:

$$f_b = 3.6 \frac{c}{3.46 \times L} \sqrt{\frac{g \times E_b}{\gamma_b + \frac{a}{c} \gamma}} = 3.6 \frac{0.015}{3.46 \times 2.4} \sqrt{\frac{9.81 \times 2.1 \times 10^{11}}{78000 + \frac{0.15}{0.015} \times 9800}} =$$

$$f_b = 22.24 \text{ (Hz)}$$

$$f_b \geq 1.5 f_s = 22.24 \geq 1.5 \times 15.50 = 22.24 \geq 18.755$$

$$22.24 \geq 18.75 \text{ SAFE}$$

being L the distance between bar supports (m); g the gravity acceleration (9.8 m/sec²). E_b is the elasticity modulus (N/m²), γ_b the specific weight (N/m³) of the bar material and γ is the water specific weight (9800 N/m³)



In order to guarantee the rack structural stability, the dimension L must be reduced until the above equation be verified. Sometimes the solution leads to consider a transversal bar in order to obtain a lower bar length.







Type of bar	$b=c$	$b=2.8 c$	Diameter= c	$b=2.8 c$	$b=2.8 c$	$b=5 c$
						
S_t	0.130	0.155	0.200	0.255	0.265	0.275

Table 4: Strouhal number for different type of bars (LENCASTRE-1987)

$(a+c)/c$	1.50	2.00	2.50	3.00	4.00	5.00
F	2.15	1.70	1.40	1.20	1.05	1.01

Table 5: Safety coefficient for Strouhal number (LENCASTRE-1987)

(LENCASTRE, 1991)

Turbine type	Kaplan	Fast Francis	Slow Francis	Pelton	Small Pumps
a (m)	0.10 - 0.15	0.08 - 0.10	0.06 - 0.09	0.03 - 0.05	0.02

Table 6: Spacebar for Different Turbine Type (LENCASTRE-1991)

4. Monitoring Instrumentations

Instrumentation of dams is the use of devices to measure safety parameters of such dams which together with visual inspections and other measurements made at the site provide powerful tools to evaluate the performance and discover early signs of abnormal behavior. Careful examination of instrumentation data on a continuing basis may reveal a possible critical condition. Conversely, instrumentation may be a mean of assuring that an observed condition is not serious and does not require immediate remedial measures.

All installed instruments require; however, constant attention to insure their good operation and prevents malfunctioning and giving false alarms.

Typical instruments are used for checking certain areas of concern or performance aspects which generally include the following:

1. Seepage flows,
2. Seepage water turbidity, its salt content and temperature.

3. Piezometric levels.
4. Water levels.
5. Deformations or movements.
6. Pressures.
7. Loading conditions.
8. Temperature variations within dams' body; and
9. Accelerations experienced by the dam during an earthquake.

Dam safety concerns need to be checked by both visual inspections and instrument measurement.

For the final instrumentation, additional vertical holes must be drilled through the main wall and into the bedrock. The drill holes will house the instrumentation to observe the dam, such as inclinometers and piezometers.

Furthermore, multiple brass bolts shall be installed as geodetic measurement fix points on the top of the walls to observe possible movements with a precise laser total station.

Adequate instrumentation and permanent registration of movements and water pressure will ensure the safety of the entire dam and provide early warning of any potential problems.

Seepage through a dam or its foundation is visible and shows progressive dissolution or erosion in a dam foundation or abutment. Weirs, flowerets, and calibrated catch containers are used for seepage monitoring.

Uplift pressure is important from foundations stability point of view. Measuring water pressures in a rock foundation is always a challenging problem, since it can change over short distances because of jointing and fissuration. Open pipe piezometers, standpipe with water level indicator are used to measure water pressure.

Description and Location of Instruments

Description of Instrument	Position of Instrument		
	In the dam body	Under the dam body	Near the dam body
Target Point	X		
Staff Gauge	X		
Piezometer	X		
Weir Box			X

Initial measurements on all instrumentations shall be taken during and after the first fill of the reservoir following completion of the repair works. Systematic measurements and controls are essential for safe future operation of the dam. The observations must be documented in a dam record book by the responsible water department in Ghazni. Especially after exceptional events, such as extreme water levels and/or earthquakes, control measurements must be repeated and compared with previous readings.



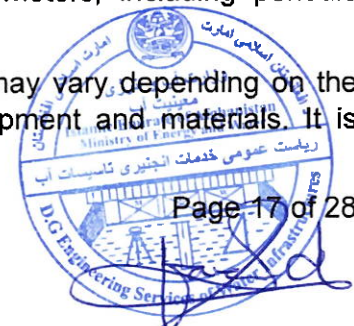
4.1. Piezometer

Piezometers are used to measure pore water pressure, the water pressure in the soil, earth/rock-fills, foundations and concrete structures i.e. piezometric level in the ground, the soil, earth/rock fills, foundations and concrete structures. It provides significant quantitative data on the magnitude and distribution of pore pressure and its variations with time. It also helps in evaluating the pattern of seepage, zones of potential piping and the effectiveness of seepage control measures undertaken.

Piezometers are commonly used in earth dams to measure pore water pressure and monitor the stability of the dam. Installing piezometers in an earth dam involves several steps. Here's a general procedure for installing piezometers in an earth dam:

1. **Site Investigation:** Conduct a thorough site investigation to identify the locations where piezometers need to be installed. This involves studying the geology, hydrogeology, and engineering properties of the dam.
2. **Design and Planning:** Based on the site investigation data, design the piezometer installation plan. Determine the number of piezometers needed, their depths, and their spacing. Consider factors such as dam height, foundation conditions, and anticipated pore water pressure zones.
3. **Drilling:** Drill boreholes at the predetermined locations within the dam. The borehole diameter should accommodate the piezometer assembly. The depth of the borehole should extend to the desired monitoring zone.
4. **Piezometer Installation:** Install the piezometer assembly into the drilled borehole. The assembly typically consists of a riser pipe, filter pack, and piezometer tip. The riser pipe is usually made of PVC or stainless steel and has openings at the desired monitoring depths. The filter pack, composed of gravel or sand, surrounds the riser pipe and prevents soil particles from entering the piezometer. The piezometer tip measures the pore water pressure and is connected to the riser pipe.
5. **Backfilling:** Carefully backfill the borehole with suitable material, such as bentonite or sand, to ensure good hydraulic contact between the surrounding soil and the filter pack. Proper backfilling is essential to minimize any disturbance to the soil and ensure accurate measurements.
6. **Manifold and Data Acquisition:** Install a manifold system to connect multiple piezometers for simultaneous monitoring. The manifold consists of pipes and valves that allow for pressure equalization and data collection from each piezometer. Connect the manifold to a data acquisition system to record and analyze the measured pore water pressures over time.
7. **Monitoring and Maintenance:** Regularly monitor the piezometers to measure and record pore water pressure variations within the dam. Implement a maintenance plan to ensure the proper functioning of the piezometers, including periodic inspection, cleaning, and calibration.

It's important to note that the specific installation process may vary depending on the design requirements, local regulations, and available equipment and materials. It is



recommended to consult with geotechnical engineers or dam construction specialists for detailed guidance and adherence to safety protocols during the installation of piezometers in an earth dam.

4.1.1. Monitoring and Analysis

Once the piezometers are installed, regular monitoring and data analysis are crucial for dam safety assessment. Monitoring frequency may vary depending on the project requirements, but it is typically done at regular intervals (e.g., monthly, quarterly, or annually). Analysis of the collected data helps identify trends, evaluate pore water pressure variations, and assess the stability of the dam.

By monitoring pore water pressures, engineers can detect potential issues such as excessive seepage, internal erosion, and changes in dam behavior. This information is vital for making informed decisions regarding dam operations, maintenance, and remedial measures if necessary.

It's important to consult with geotechnical engineers or dam specialists for specific guidance tailored to the dam project in question. They can provide detailed recommendations based on the site conditions, dam design, and monitoring objectives.

4.1.2. Types of Piezometers

1. **Standpipe Piezometers:** These are the most commonly used type of piezometers in earth dams. They consist of a riser pipe with multiple openings (usually slotted) at different depths. The openings allow water to enter the piezometer and equilibrate with the pore water pressure in the surrounding soil.
2. **Vibrating Wire Piezometers:** These piezometers use a vibrating wire sensor to measure pore water pressure. The vibrating wire is attached to a diaphragm, which deforms with changing pore water pressure, causing a change in the frequency of vibration. This change in frequency is then measured and correlated to the pore water pressure.
3. **Pneumatic Piezometers:** Pneumatic piezometers measure pore water pressure by using compressed air or gas. The pressure in the piezometer casing varies with the pore water pressure, and this pressure is measured using a pressure transducer.

Selection: Since the saddle dam is earth-filled and the open standpipe piezometer is easy to use and reasonably priced, we have chosen it for this project.

4.1.3. Open Standpipe Piezometer

Open standpipe piezometers are the standard against which all other piezometers are judged. They are simple, reliable, inexpensive, and easy to monitor. Water level readings are typically obtained with a water level indicator. Typical applications include:

1. Monitoring pore-water pressure to determine slope stability
2. Monitoring seepage and ground water movement in embankments and dams
3. Monitoring the effectiveness of dewatering schemes



The piezometer, which consists of a filter tip and a riser pipe, is installed in a borehole. The zone around the filter tip is backfilled with sand and a bentonite seal is placed above that to isolate the intake zone. The remainder of the borehole is backfilled with bentonite-cement grout. Pore-water flows into the standpipe until pressure equilibrium is reached. The water level in the pipe then represents the pore-water pressure in the soil around the intake zone. Readings are taken with a water level indicator.

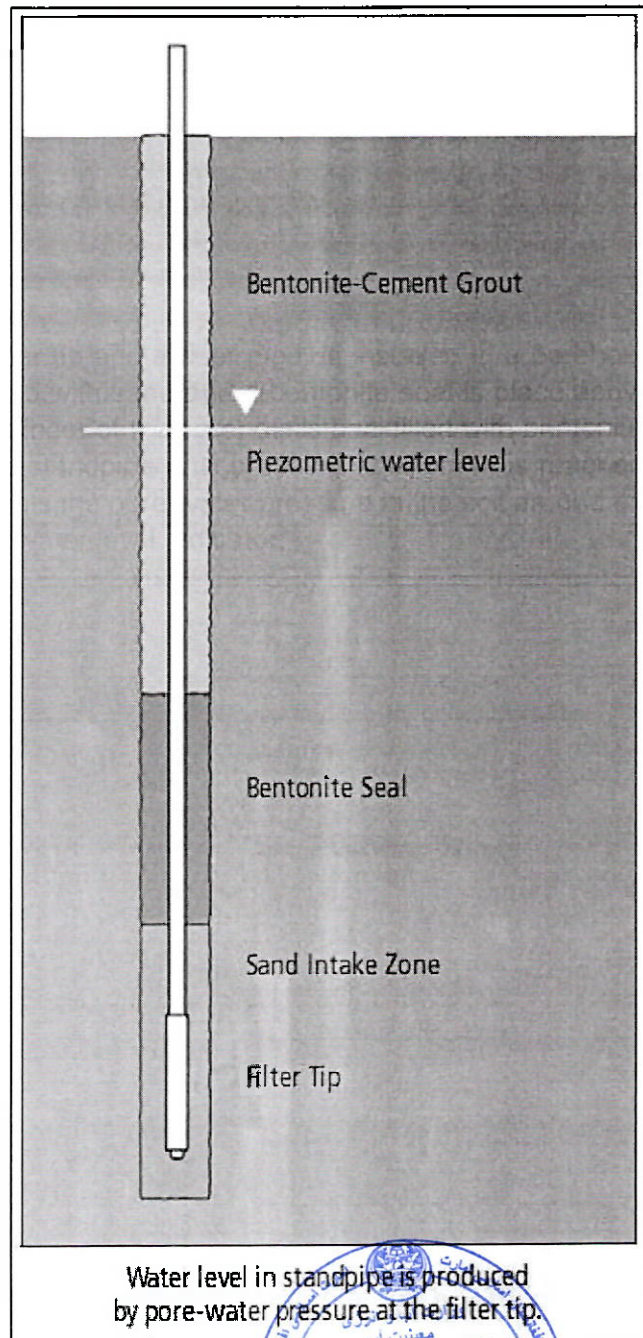


4.1.3.1. Components

The open standpipe piezometer consists of a filter tip joined to a riser pipe. The filter has 60 to 70 micron pores and is made from polyethylene or porous stone. The riser pipe is typically made from PVC plastic pipe. A water level indicator is used to monitor the piezometric water level.

4.1.3.2. Installation

After the borehole is drilled, the filter tip and riser pipe are assembled and installed downhole. Sand is tremied to the bottom of the borehole to form a sand intake zone around the filter tip. A bentonite seal is placed above the intake zone and the borehole is backfilled with a bentonite-cement grout. The riser pipe is terminated above ground level and capped to prevent entry of rain water.



4.1.3.3. Operation

Pore-water pressure around the intake zone drives water into the standpipe. The water level in the standpipe rises or falls with changes in pore-water pressure.

A water level indicator is used to monitor the changes.

4.1.3.4. Methodology and Procedure

The 2 inch PVC pipe piezometer at the boring points shall be installed in the dam site (abutments, downstream toes). Then the PVC pipe shall be connected together to make it filter based on sieve analyses results in the field then install it in the boreholes. The metal boxes shall be installed on the piezometer to measure water level changes after completion of borings.

According to ASTM D 5092, section 6, the primary filter pack should consist of an inert granular material (generally ranging from gravel to very fine sand, depending grain size distribution) of selected grain size that is installed in the annulus between the well screen and the borehole wall washed and screened silica sand and gravels with less than 5% non-siliceous material, should be specified. Whereas in the proposed depth for piezometer installation, if soil layer is not encountered, there is no need for sieve analysis.

The filters diameter suggested 1 mm for this diameter cause of that we use (2-5 mm) washed gravel for filter packing.

After gravel packing we use sealing material (bentonite sealing) and after sealing all gaps between riser pipe and borehole wall will be backfilled with grout materials.

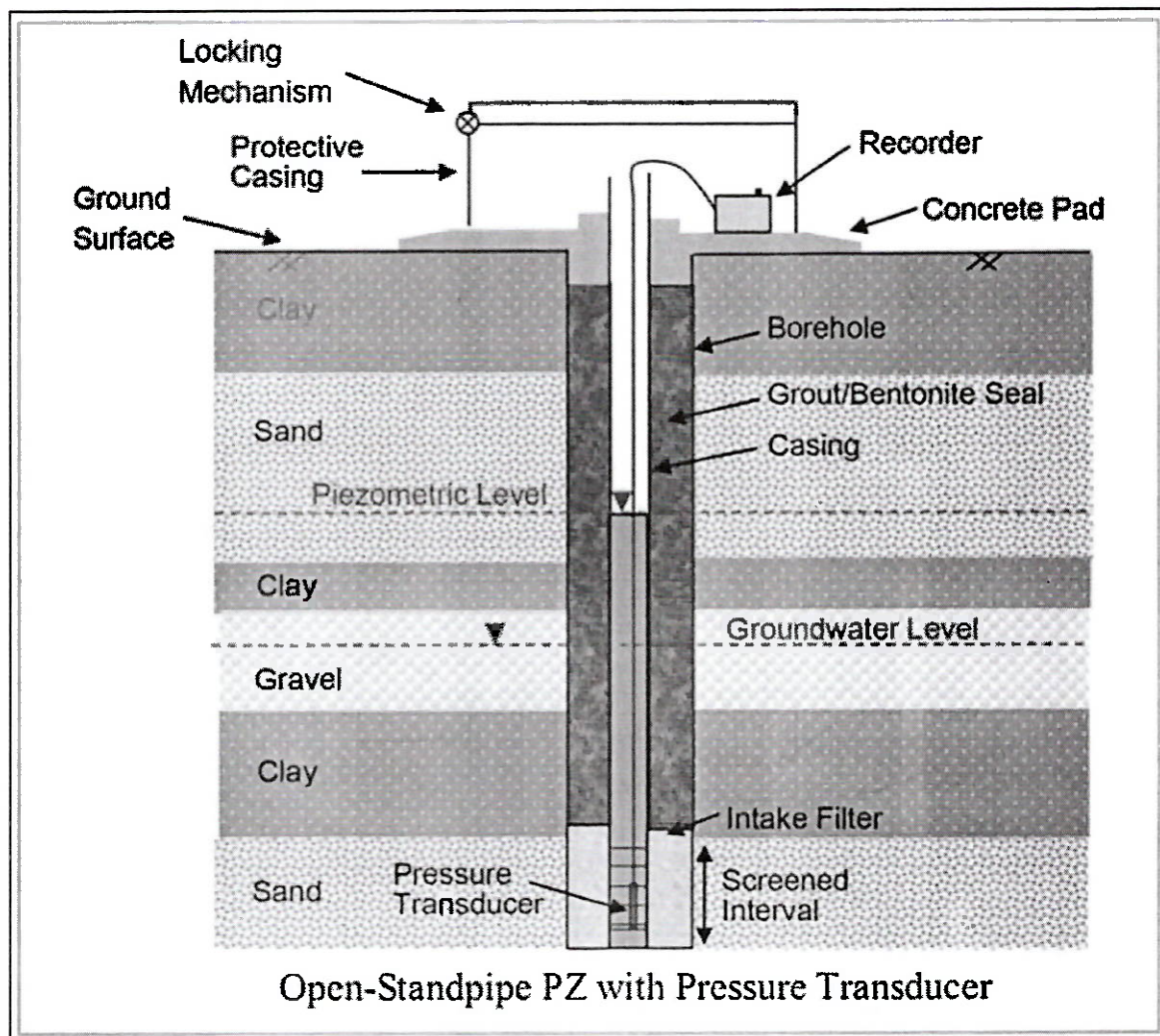
The metal protect cover boxes on the piezometer boreholes shall be installed to keep the hole safe and after measuring the water level then it shall be closed and locked in order to keep the hole safe.



Protect Cover Box



Tape line (water level indicator)

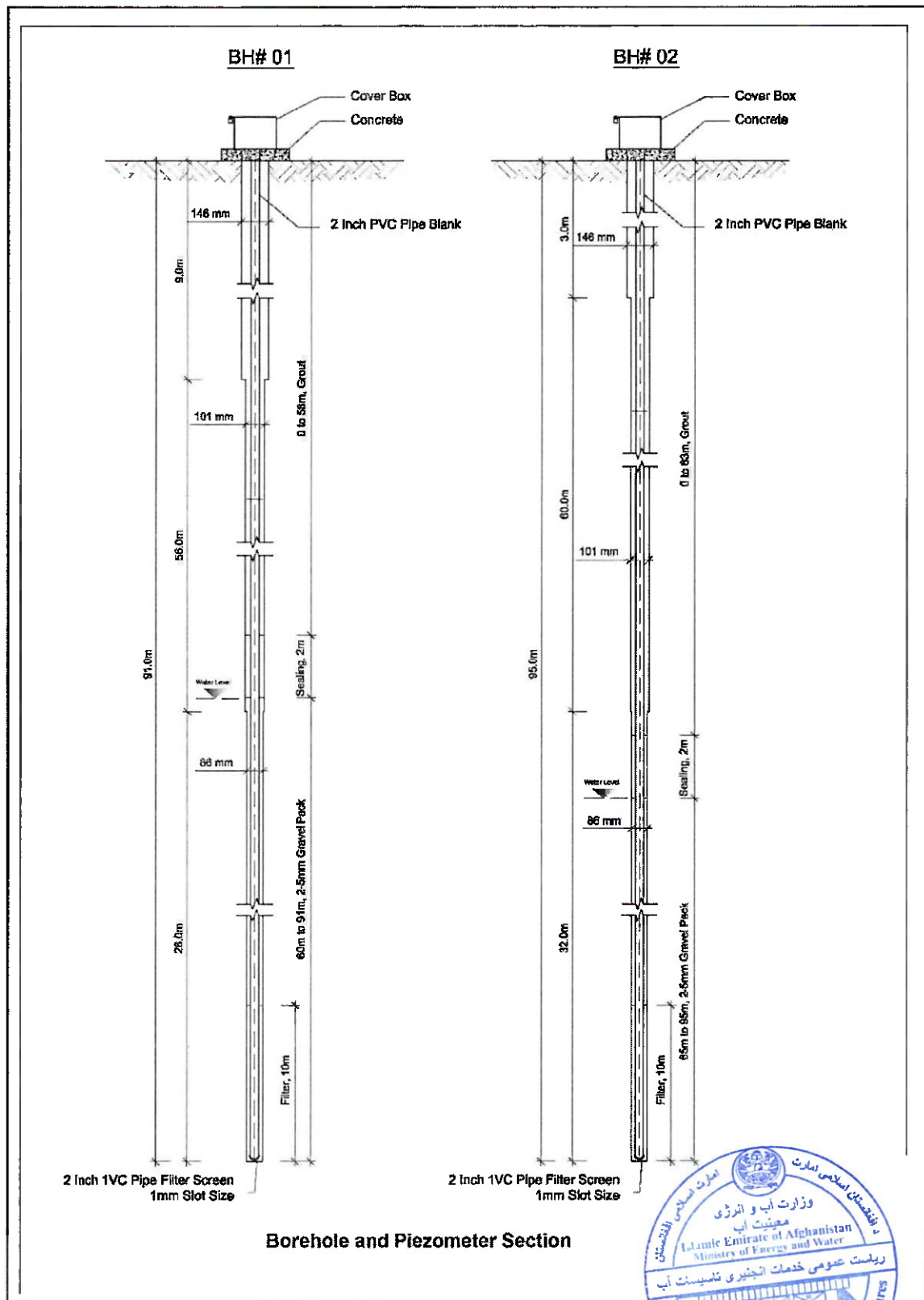


In every borehole at first, 0.5meter fine sand or 2-5mm gravel in the borehole, then the piezometer slotted pipe (filter) with their riser pipe according to suggested model "4590 casagrande" open standpipe piezometer placed into the borehole, and centralized. After that until up to water layer (aquifer layer) (2-5mm) gravels shall be placed into borehole and then sealing to be done (bentonite pellets or sealing placed into borehole, in order to seal the installation level).

After sealing, all existing gaps between borehole wall and riser pipe shall be backfilled with grout up to land surface according to piezometers installation standard.

Finally, borehole shall be filled/closed with concrete and protector cover box. After all, completing installation, piezometer measurements can be read with water level Indicator.

The reading that every 24hrs done by experienced and skilled personnel to minimize the installation time and reading errors, the reading of water level shall be recorded in every borehole.



Piezometer/Hole Number:		Piezometer Installation Log		LOCATION:	
				Northing:	
Drilling Contractor:		Project Name:		Easting:	
				Date/Time Started:	
Geologist:		Project Number:		Date/Time Completed:	
Driller:		Drilling Method:		Ground Surface Elev. _____ ft.msl	
				Top of Piezometer Casing Elev. _____ ft.msl	

1. Stick-Up of Protective Casing (If Any) Above Ground: _____ ft.
2. Stick-Up Piezometer Casing Above Ground: _____ ft.
3. Depth to Base of Surface Seal/Protective Casing: _____ ft.
4. Depth to Static Water Level: _____ ft.
5. Depth to Top of Screen Interval Seal: _____ ft.
6. Depth to Bottom Screen Interval Seal/Top of Sand Pack: _____ ft.
7. Depth to Top of Piezometer Screen: _____ ft.
8. Total Length of Blank Piezometer Casing: _____ ft.
9. Depth to Bottom of Piezometer Screen/Cap: _____ ft.
10. Depth to Base of Sand Pack/Top of Bottom Hole Seal: _____ ft.
11. Depth to Base of Bottom Hole Seal: _____ ft.
12. Depth to Base of Hole: _____ ft.
13. Protective Casing ☐ Yes ☐ No Locking Cap: ☐ Yes ☐ No
14. Concrete Pad: ☐ Yes ☐ No
15. Type of Protective Casing: _____ Diameter: _____ in.
16. Type of Surface Seal: _____
17. Borehole Diameter: _____ in.
18. Type of Annular Seal: _____
19. Type of Piezometer Casing: _____ Diameter: _____ in.
20. Type of Screen Interval Seal: _____
21. Type/Size of Sand Filter Pack: _____ Diameter: _____ in.
22. Type of Screen Material: _____ Slot Size: _____ in
23. Type of Bottom Hole Seal: _____
24. Borehole Backfill Material and Thickness: _____

Recorded By _____	Date _____	Checked By _____	Date _____
-------------------	------------	------------------	------------

EM 1110-2-1908 • 30 November 2020



Also, readings can be made by sounding with a water-level indicator, with a pressure transducer placed in the standpipe below the lowest piezometric level as shown in above figure, or with an ultrasonic sensor.

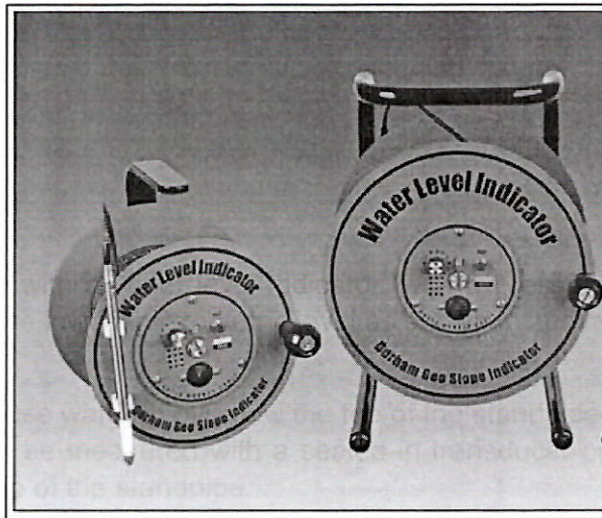
Note: If piezometric level is sufficient to cause water to overflow the top of the standpipe, the pipe may be sealed and pressure can be measured with a sealed-in transducer or using a pressure gauge mounted on the top of the standpipe.

4.2. Water Level Meter (Indicator)

Water level indicators are used to monitor water levels in standpipes and wells.

4.2.1. Operation

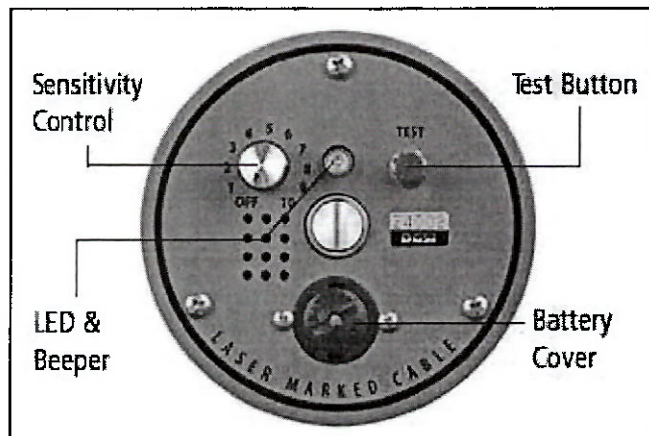
The indicator consists of a probe, a cable with laser-marked graduations, and a cable reel. The hub of the cable reel contains batteries, electronics, a bright LED lamp, and a beeper. The operator lowers the probe into the standpipe or well. When the probe contacts the surface of the water, the LED illuminates and the beeper sounds. The operator then reads the depth-to-water measurement from the graduations on the cable.



4.2.2. Water Level Indicator Specifications

Probe: Stainless steel body and tip, polyethylene insulator. 10 x 170 mm (3/8" x 6.6").

Cable: 3.2 mm (1/8") diameter polyurethane jacket with two copper-clad, steel conductors inside. Graduations are marked with laser and cannot be rubbed off. Clean cable with laboratory grade detergent, such as Alconox® or Liquinox®.



Reel Construction: Heavy-gage aluminum plate sides, PVC spool, rotating knob. Larger reels are equipped with a stand made of strong steel tubing, a probe holder, and a reel brake.

Sensitivity Control provides consistent results in different well and water conditions and helps eliminates false triggering.

LED and beeper provide a positive indication of contact with water. Test button is used to check the batteries, beeper, and LED. Battery cover provides easy access to two AA batteries. Low-power circuits provide excellent battery life.

4.3. Target/ Survey Points

Target points at the top of the dam crest are used to take survey of vertical and horizontal coordinates and measure probable displacement at the vertical and horizontal axes. All required Survey points according to corresponding drawings and the respective item in the BOQ shall be installed ready for use.

Two types are distinguished, as follows:

- For dam crest deformation monitoring
- For abutment deformation monitoring
- Reference point installed on top of concrete structures

Target points shall be:

- Manufactured from stainless steel, grade 316 S13 to BS 970: Part 1.
- Fixed to the dam body in the designed location.
- Protective galvanized steel caps shall be included.

4.4. Staff Gauge

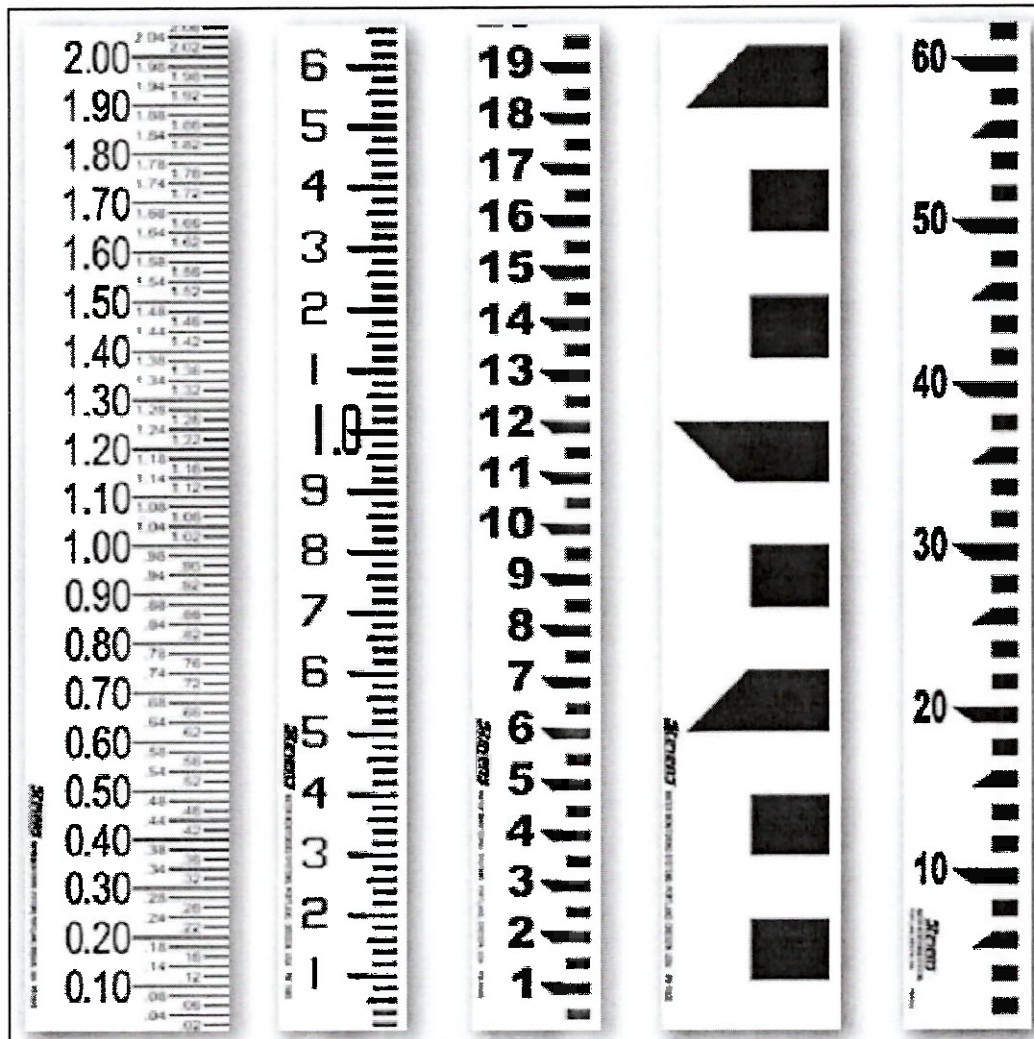
The staff gauge is used for measuring headrace (reservoir) and tailrace water levels. The staff gauge shall be of such a design that it can be effectively used under the prevailing environmental conditions. It needs to be readable for all possible water levels at site.

The staff gauge shall be of a sturdy construction. The staff gauge shall be easy to operate and maintain. All materials on the staff gauge shall be non-corrosive.

The staff gauge shall have following specifications:

- Enameled: steel or stable FRP plate
- Colors: white background, deep blue or black graduations
- Width: 100 to 200 mm
- Thickness: 1.5 to 2.5 mm
- Length: 1000, 1500 or 2000 mm
- Smallest graduation: 0.01 m
- Accuracy of graduations: 0.5 mm
- Distinction: cm, dm and m, index at 5 cm mark
- Temperature: 0 to 60°C, the accuracy shall be maintained over the full temperature range





Staff Gauge

4.5. Weir Box

A weir box, also known as a weir chamber or flow measurement chamber, is a hydraulic structure used to measure or control the flow of water in open channels or pipes. It consists of a rectangular or trapezoidal-shaped enclosure with a notch or opening through which water flows.

Weir boxes are commonly used to measure the flow rate of water in open channels. The notch or opening in the weir box creates a constriction point where water levels can be measured or calibrated to determine the flow rate.

Various types of weir notches, such as rectangular (with and without contractions), V-notch, Cipolletti, and broad-crested weirs, can be used depending on the application and flow conditions.

Seepage is measured by a weir located approximately at the downstream toe of the maximum section.



Dam Seepage Measuring Stainless Steel Weir Box

A stainless steel weir box measuring dam seepage for the Portland Water Bureau. Flow spills into the weir box from an elevated pipe, passes through the box, and then spills off the end into the downstream basin.

While weirs are easy to construct and use, particular care must be taken to ensure that the crest of the weir – the surface over which the seepage will flow – is kept clean, free of debris, and in good repair.

Growth or damage to the weir crest can dramatically affect how the seepage will flow over the crest, which will in turn affect system accuracy. To minimize the potential for damage, weirs should be constructed of stainless steel where possible.

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Deputy Ministry of Water

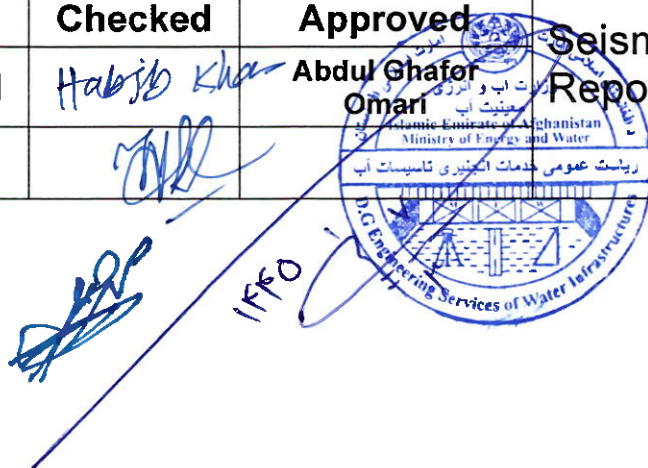


General Directorate of Engineering Services of Water Infrastructures

Technical Board



Rehabilitation of Sultan Dam			
Date		Nov 2023	
Prepared	Checked	Approved	Seismology/Earthquake Report
M.Hanif Jawid	Habib Khan	Abdul Ghafor Omar	



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APPENDIX

APP 1. Records

DESIGN OF RECONSTRUCTION AND REHABILITATION WORKS OF SULTAN DAM SEISMIC RISK ANALYSIS

1. INTRODUCTION

1.1. Scope

The seismic risk analysis covers the seismic evaluation of "Sultan Dam" from the engineering point of view.

The seismic risk is defined as the exceedence of the parameters related to the soil characteristics and earthquake intensity values of a project site in a certain time. The aim of this analysis is to define the deterministic and probabilistic values of the project area by considering the historical records, geological characteristics, sismological and statistical factors.

1.2. Project Site

The project area is located on Gazni River at north west of the Gazni province. The project area is located about 35 km north west of Gazni. The area is a mountainous region. The elevation of Sultan Dam is 2400 meters above sea level. Jaghatu village is located in upstream of the dam axis while Petawak village is located in downstream of the dam axis.



The geographical coordinates of Sultan Dam is; 33.757151° north latitude and 68.380269° east longitude.

The location of the project area is given in Figure 1 and the satellite image of project area is given in Figure 2.

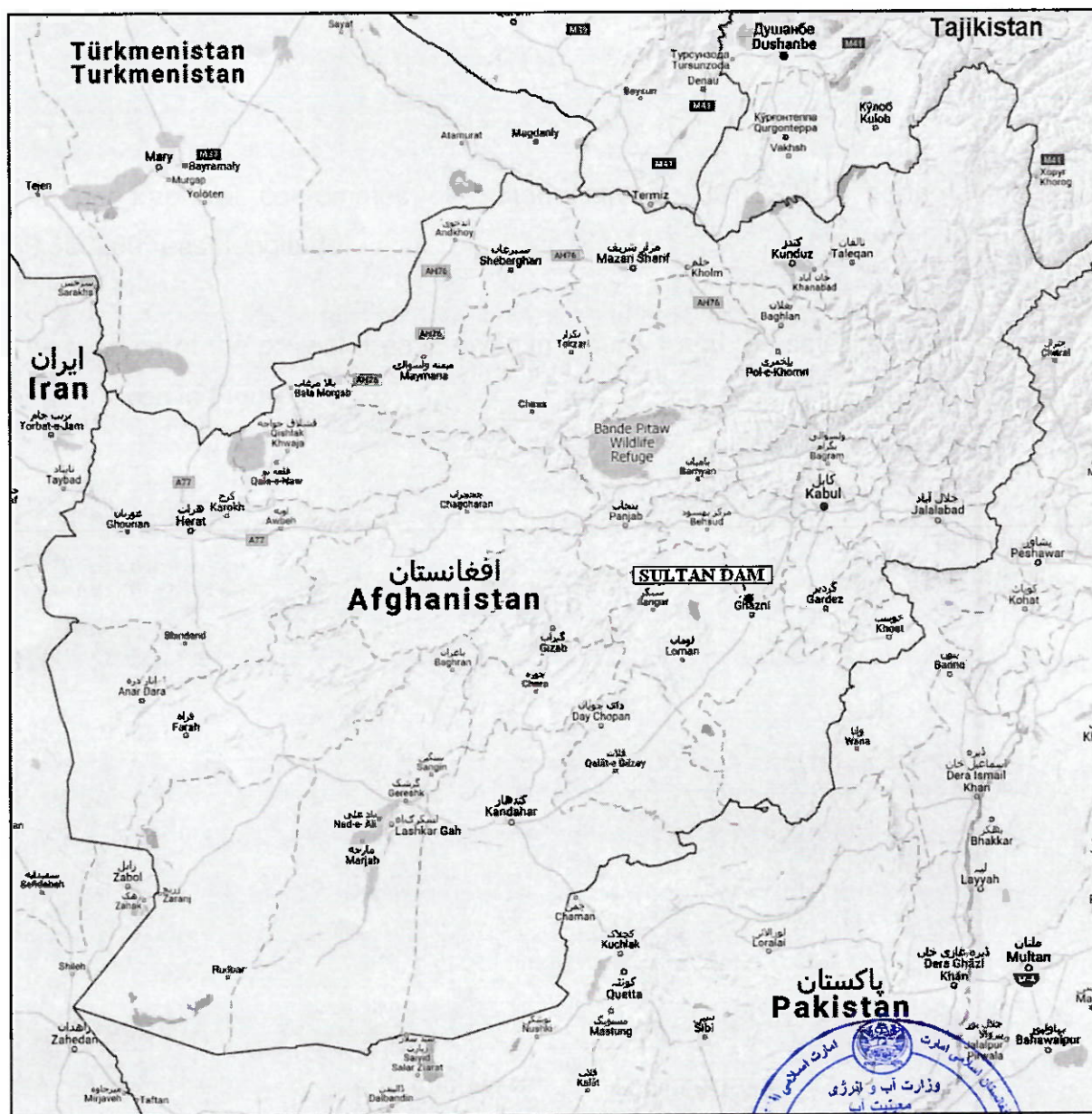


Figure 1. Location Map of Sultan Dam



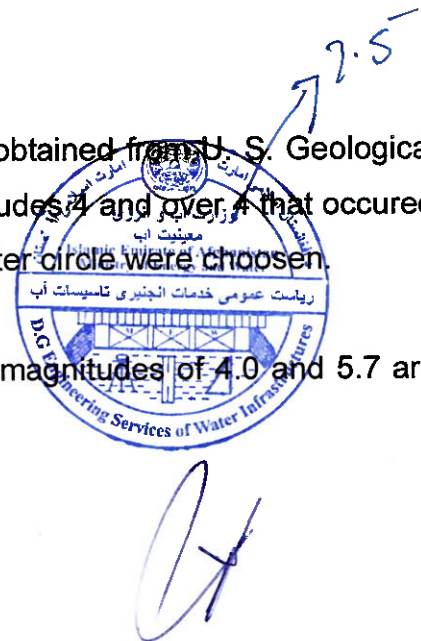
Figure 2. Satellite Image of Sultan Dam

2. SEISMOLOGICAL AND TECTONIC DATA

2.1. Seismological Data

The sismological data of Sultan Dam project area is obtained from U. S. Geological Survey (USGS) records. The earthquakes with magnitudes 4 and over 4 that occurred between years 1900 and 2020 with in a 100 km diameter circle were choosen.

The seismic records for 31 earthquakes with varying magnitudes of 4.0 and 5.7 are given appendix as a table.



2.2. Tectonic Data

Afghanistan is located in a seismological active part of the world where the northward-moving Indian plate is colliding with southern part of the Eurasian plate. Earthquakes in Afghanistan are most abundant in and near the northeastern part of the country where the effects of the plate collision between India and Asia are most pronounced.

Afghanistan occupies a southward-projecting, relatively stable promontory of the Eurasian tectonic plate. Active plate boundaries, however, surround Afghanistan on the west, south, and east.

To the west, the Arabian plate moves northward relative to Eurasia at about 3 cm/yr. The active plate boundary trends northwestward through the Zagros region of southwestern Iran. Deformation is accommodated throughout the territory of Iran; major structures include several north-south-trending, right-lateral strike-slip fault systems in the east and, farther to the north, a series of east-west-trending reverse and strike-slip faults. This deformation apparently does not cross the border into relatively stable western Afghanistan. In the east, the Indian plate moves northward relative to Eurasia at a rate of about 4 cm/yr. A broad, transpressional plate-boundary zone extends into eastern Afghanistan, trending southwestward from the Hindu Kush in northeast Afghanistan, through Kabul, and along the Afghanistan-Pakistan border. Deformation here is expressed as a belt of major, north-northeast-trending, left-lateral strike-slip faults and abundant seismicity. The seismicity intensifies farther to the northeast and includes a prominent zone of deep earthquakes associated with northward subduction of the Indian plate beneath Eurasia that extends beneath the Hindu Kush and Pamirs Mountains.

Main active faults in Afghanistan;

Chaman Fault

The Chaman fault system is more than 1000 km long, extending from the Hindu Kush region in northeastern Afghanistan south-southwestward through eastern Afghanistan into western Pakistan.



The Chaman fault is the major strike-slip structural boundary between the India and Eurasia plates. Despite sinistral slip rates similar to the North America-Pacific plate boundary, no major ($>M7$) earthquakes have been documented along the Chaman fault, indicating that the fault either creeps aseismically or is at a late stage in its seismic cycle. Several large historical earthquakes have produced surface rupture on the fault in Afghanistan. In 1505, an earthquake having an estimated magnitude of M_s 7.3 occurred near Kabul. An earthquake in 1892 occurred near $31^\circ N$, producing 60-75 cm of left-lateral movement and dropping the west side of the fault by 20-30 cm. A moment-magnitude (M_w) 6.4 earthquake in 1975 near $30^\circ N$ produced 5 km of surface rupture, 4 cm of left-lateral offset, and a small amount of east-side-down slip.

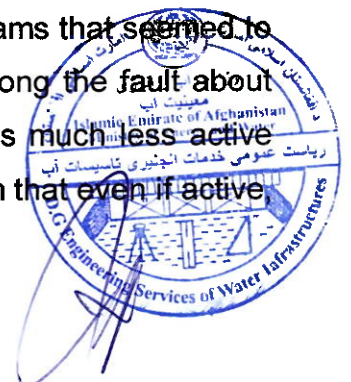
Despite the large uncertainty in the estimated slip rate, the Chaman fault system poses a significant seismic hazard.

Hari Rud Fault

The 730 km long, right-lateral Hari Rud fault extends from its intersection with the Chaman fault north of Kabul westward to the Iran border.

The fault has an exceptional geomorphic expression on the landscape, due in large part to the region's arid climate and much of the fault being located in hard bedrock, but other evidence for active faulting remains controversial. According to Quittmeyer, this is a right-lateral strike-slip feature with a probable history of movement throughout the Cenozoic. Ambraseys state that slip on this fault would be insignificant in accommodating the north-south convergence thus explaining absence of seismic activity during the historic and instrumental period. Verma have concluded that the Hari Rud fault is seismically inactive barring its portion that trends northeast towards the Hindukush.

Quite conversely, Wellman classified the fault as being 'active' citing significant evidence of dextral topographical displacement (60-100m) of streams that seemed to have originated more than 10,000 years ago, at two locations along the fault about 200 km east and 500 km west of Kabul. Geologically, the fault is much less active than the faults of east Iran and recent GPS measurements confirm that even if active, the estimated rate of motion is low.



Central Badakhshan Fault

Could not found a published slip rate for the Central Badakhshan fault. Assuming that the slip rate is conserved at the junction of the Hari Rud and Chanman faults, can be assigned a slip rate of 12 mm/yr for the Central Badakhshan fault.

Darvaz Fault

The 380 km long, left-lateral Darvaz fault parallels the Central Badakhshan fault in northeastern Afghanistan and, like it, extends northward into Tajikistan.

Historical and instrumental period seismicity of Afghanistan (734-2002 AD, $M > 5$) is given in Table 1 and Figure 3.

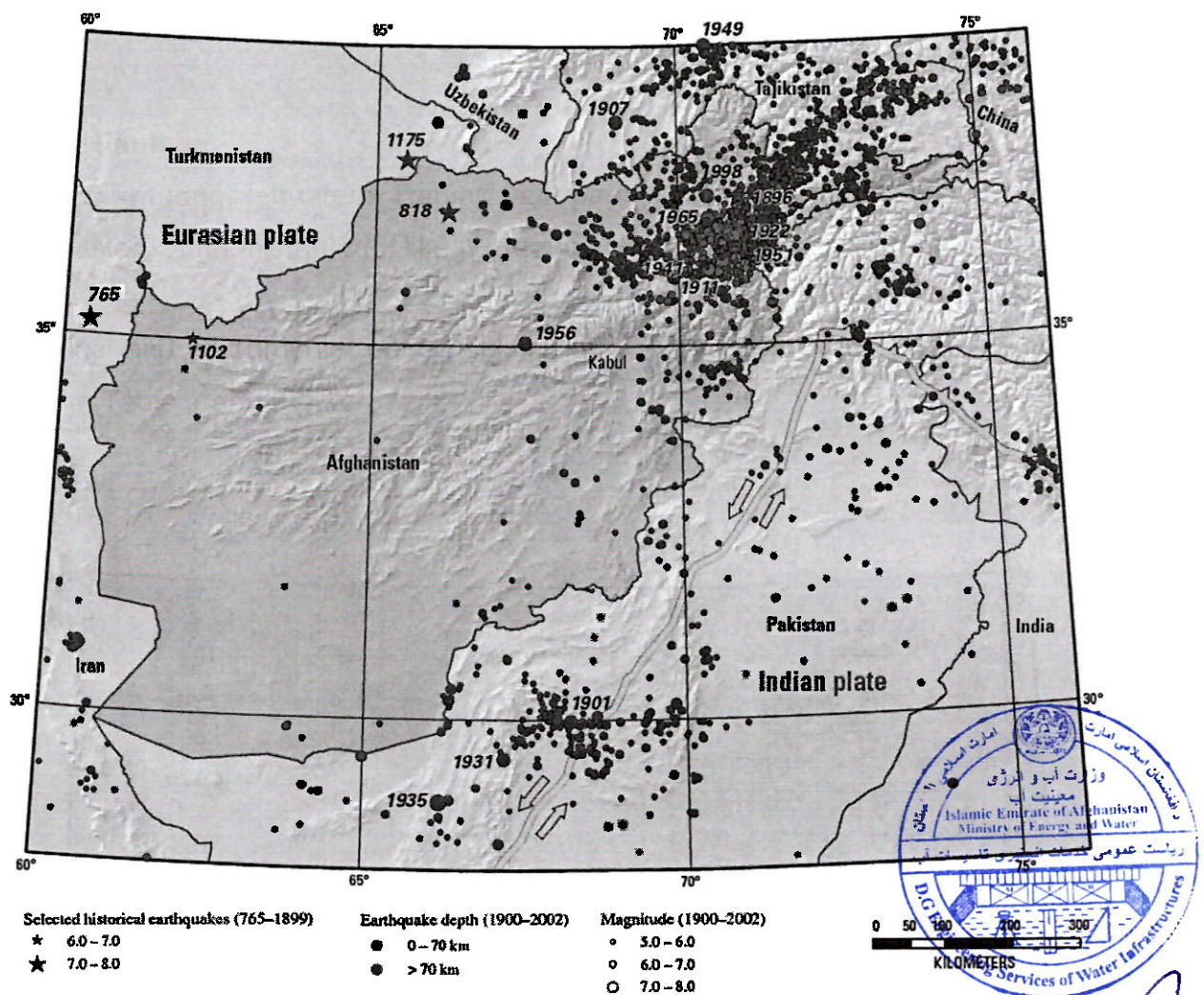


Figure 3. Historical and Instrumental Period Seismicity of Afghanistan

Date, A.D.	Magnitud	Latitude	Longitude	Description
819	7.4	36.40	65.40	The earliest well-documented earthquake occurred in northern Afghanistan, ~150 km west of Mazar-e Sharif.
849	5.3	34.30	62.20	These earthquakes caused heavy damage and some casualties in Herat.
1102	5.3	34.40	62.20	
1364	5.8	34.90	61.70	
1505	7.3	34.50	69.10	This earthquake caused damage in Kabul, and heavy casualties and damage in Paghman (~20 km west of Kabul) and nearby villages.
22 Jan 1832	7.4	36.50	71.00	Centered in the Badakhshan district of northeast Afghanistan, this large earthquake reportedly destroyed many villages and killed thousands of people.
18 Oct 1874	7	35.10	69.20	This earthquake caused heavy damage and casualties in villages ~70km north of Kabul.
20 Dec 1892	6.5	30.90	66.50	Located near the Pakistan border, ~90 km northwest of Quetta, this earthquake caused surface rupture on the Chaman fault.
7 Jul 1909	7.5	36.50	70.50	Strong damage in the Badakhshan region.
1 Jan 1911	7.1	36.50	66.50	This earthquake was widely felt in northern Afghanistan and adjacent Tajikistan.
30 May 1935	7.7	28.90	66.40	This major earthquake was centered across the southeastern border of Afghanistan, near Quetta, Pakistan. Some researchers associate this earthquake with the Ghazaband fault zone.
9 Jun 1956	7.4	35.10	67.50	Centered ~160 km northwest of Kabul in a sparsely populated region of northern Afghanistan.
3 Oct 1975	6.8	30.20	66.30	This earthquake occurred in a sparsely populated region of southeast Afghanistan. Apparent co-seismic slip on the Chaman fault, ~4 cm of left-lateral displacement with a minor component of up-to-the-west dip slip, extended for a distance of ~5 km.
16 Dec 1982	6.5	36.10	69.00	Centered in the mountains ~170 km north of Kabul.
4 Feb 1998	5.9	37.10	70.10	This earthquake was located in the mountains ~300 km north of Kabul.
30 May 1998	6.5	37.10	70.10	Located near the site of 4 Feb 1998 shock.
3 Mar 2002	7.4	36.50	70.48	Centered in the Hindu Kush region, this is an example of a deep earthquake (~200 km depth) that caused damage and fatalities.

Table 1. Significant Historical and Instrumental Period Earthquakes of Afghanistan

The faults and fault systems exist in the vicinity of Sultan Dam project area is given in Figure 4.



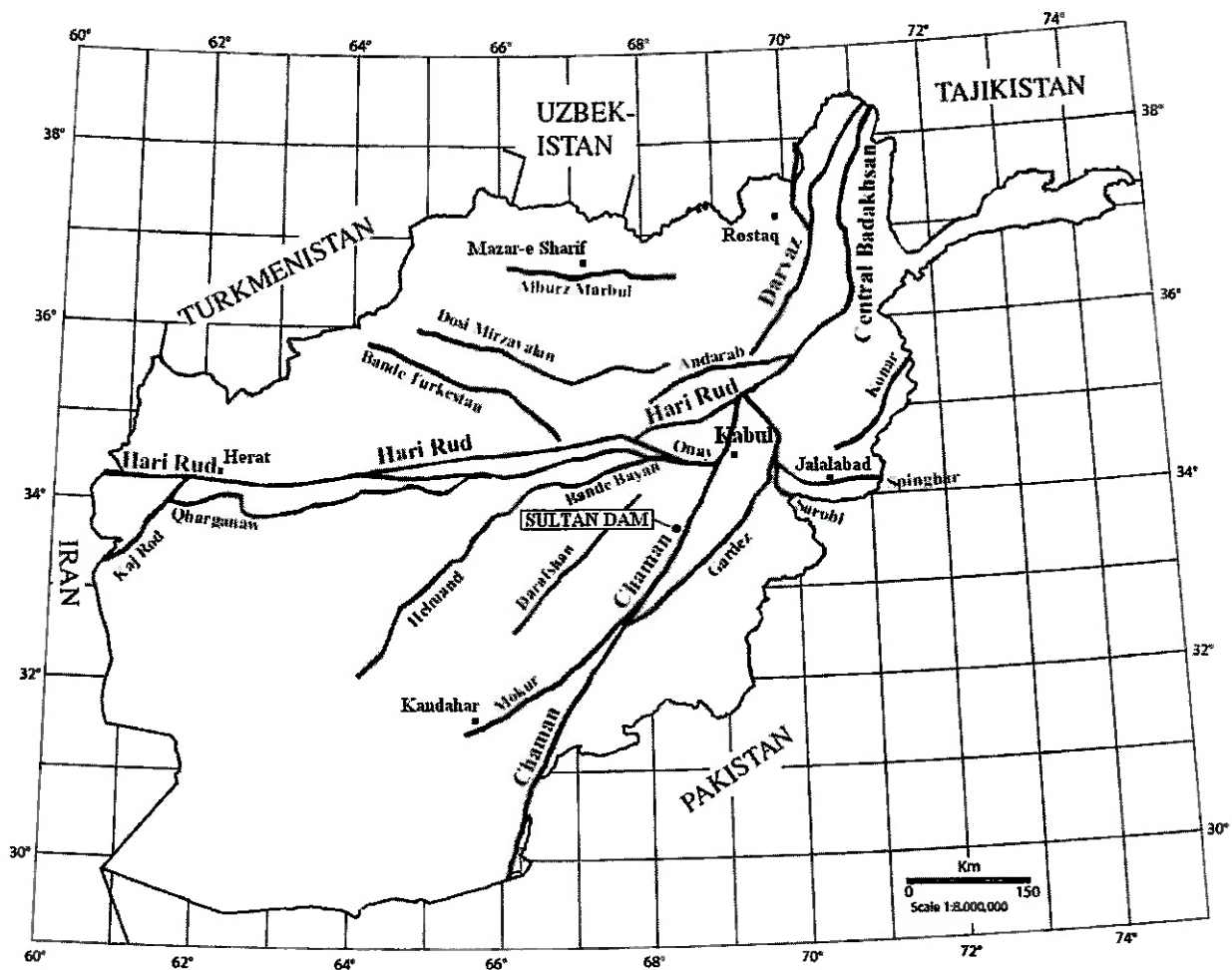


Figure 4. Faults And Fault Zones Around The Sultan Dam

Considering the distances to faults, Chaman Fault can possess a seismic risk for the Sultan Dam project area. The distance to the Sultan Dam project area is approximately 14 kilometers. Therefore, it is necessary to consider this fault in the seismic risk analysis.

3. EARTHQUAKE DESIGN CONCEPT

The earthquake or earthquakes used for the design of a dam have been specified by a variety of terms such as the maximum credible earthquake (MCE), the maximum design earthquake (MDE) and the operating basis earthquake (OBE).



Operating Basis Earthquake (OBE):

The OBE is an earthquake that produces ground motions at the site that can reasonably be expected to occur within the service life of the project. The associated performance requirement is that the project function with little or no damage, and without interruption of function. The purpose of the OBE is to protect against economic losses from damage or loss of service. Therefore, the return period for the OBE may be based on economic considerations.

For the OBE, the repetition period of earthquake has been accepted as 144 years, in other words, the OBE refers to an earthquake corresponding to 100 years of economic life and 50% probability of exceedance.

Maximum Design Earthquake (MDE) or Safety Evaluation Earthquake (SEE):

This is the earthquake that produces the maximum level of ground motion for which a structure is to be designed or evaluated. The MDE or SEE may be set equal to the MCE or to a design earthquake less than the MCE, depending on the circumstances. Factors to consider in establishing the size of MDE or SEE are the hazard potential classification of the dam, criticality of the project function (water supply, recreation, flood control, etc.) and the turnaround time to restore the facility to operability.

For the MDE or SEE, the repetition period of earthquake has been accepted as 475 years, in other words, the MDE or SEE refers to an earthquake corresponding to 50 years of economic life and 10% probability of exceedance.

Maximum Credible Earthquake (MCE):

The MCE is the largest earthquake magnitude that could occur along a recognized fault or within a particular seismotectonic source area under the current tectonic framework.

For the MCE, the repetition period of earthquake has been accepted as 2475 years, in other words, the MCE refers to an earthquake corresponding to 50 years of economic life and 2% probability of exceedance.



4. SEISMIC RISK ANALYSIS

Both probabilistic and deterministic methods are utilized in the seismic risk analysis.

4.1. Probabilistic Method

The seismic risk analysis by probabilistic method aims the evaluation of seismic conditions in the uncertainties that encounter in the variations of earthquake dimensions, locations, occurrence frequencies, intensities and local characteristics of a certain area. By this analysis the defined uncertainties are able to be evaluated quantitatively and this more proper presentation of probable seismic risk will be possible (Kramer, 1956). In order to understand the mechanism of this analysis, the fundamental concepts of the probabilistic theory must be known. The probabilistic seismic risk analysis may be defined as a four stage procedure (Reiter, 1990).

1- The first step is to define the earthquake sources and their characteristics. A uniform probabilistic distribution is applied to each source zone. This means that the probability of an earthquake occurrence is same at all over the source area. These distributions will be later combined with the source geometry in order to obtain the probabilistic distribution of source-area distances.

2- In the following step, distribution of the seismicity and earthquake repeating times are defined. The repeating relations that defines the average ratio of the exceeding of a certain earthquake intensity is used for the evaluation of each source zone.

3- The ground motion that is produced by an earthquake that occurred at any point of the source area with any intensity, must be defined by using attenuation relations

4- The last step is to estimate the earthquake location, intensity and ground motion. The uncertainties related to estimations are gathered and the exceeding probability of the ground motion parameters in a certain time interval is obtained.



The source zoning is done by considering the mid point of the project area as a center (33.757151° north latitude and 68.380269° east longitude) for a 100 km radius circle. By the guiding of epicenter points that fall in this circle the zones were determined.

The location of the project area, active faults and the seismotectonic map that shows the epicenter distribution of the earthquakes occurred in the past is given in Figure 5.



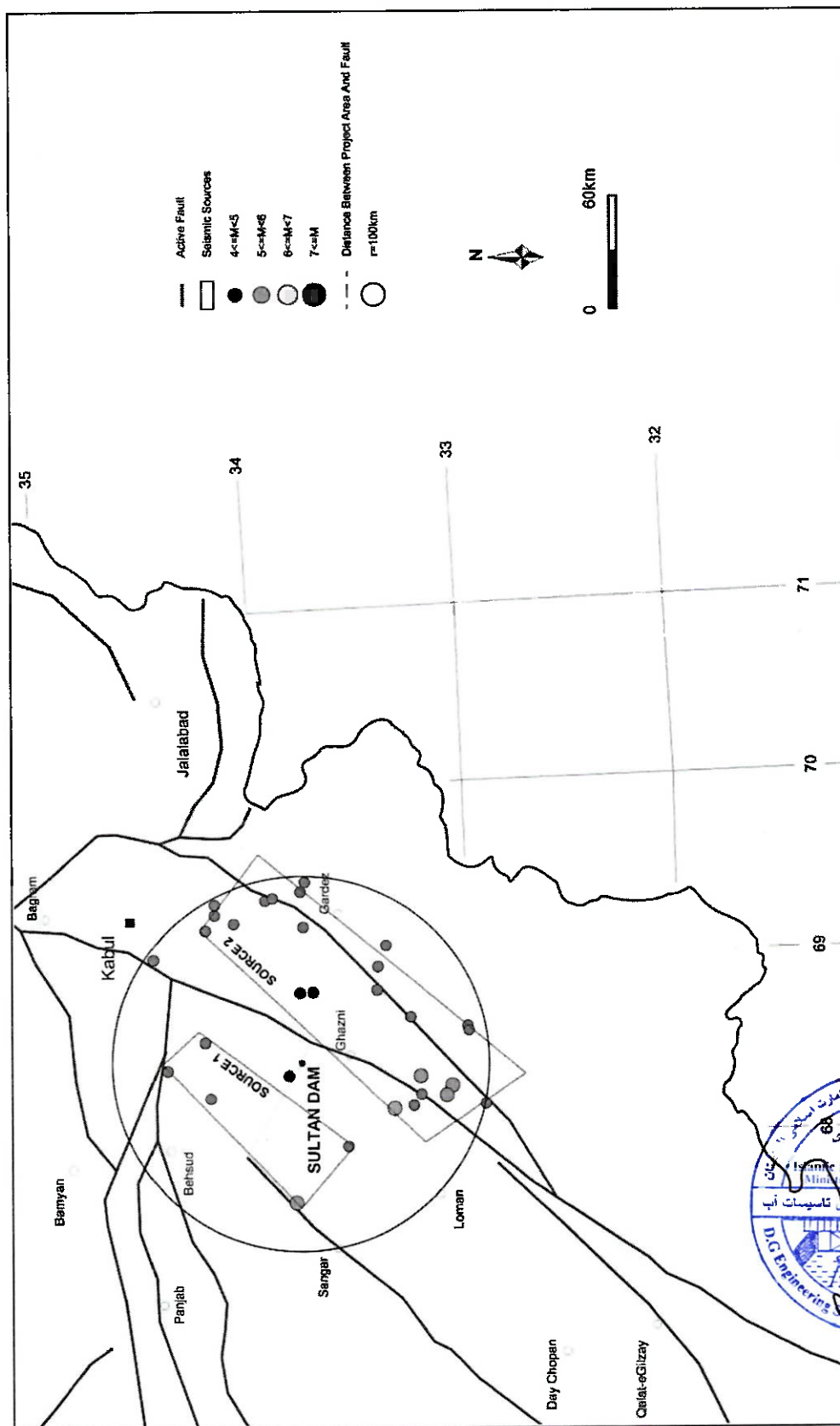


Figure 5. Seismotectonic Map Of Sultan Dam



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4.1.2. Earthquake Magnitude – Recurrence Modelling

In order to define the earthquake influences of seismic sources, the relation of recurrences of these sources must be known. This relation is given by Gutenberg-Richter with the below formula;

$$\log N = a + bM$$

a, b= regression coefficients

M= magnitude

N= cumulative number of earthquakes

“a” coefficient is the average number of the yearly earthquakes occurred in the area that depends on the size of the area, duration of the observation and the seismic activity during observation period. “b” coefficient is a seismotectonic parameter that specifies the probability of relative occurrences of the minor and major earthquakes. It also depends on the tectonic characteristics of the region.

Gutenberg-Richter relations of the seismic sources that influence the Sultan Dam is given in Table 2.

Seismic Source	M _{min}	M _{max}	a	b	λ	β
Seismic Source - 1	4.0	5.1	4.3540	0.8451	0.1667	1.9459
Seismic Source - 2	4.1	5.7	4.5181	0.7429	0.3934	1.7107

Table 2. Properties Of Seismic Sources Around Sultan Dam

4.1.3. Results of Probabilistic Method

The maximum horizontal ground acceleration values for different economic lives of the structures and for the probability of exceedences are defined by the probabilistic seismic risk analysis.



In this study “Crisis 2015” software program was used and the parameters defined in this report were utilized by using the attenuation relationships given by “Chiou and Youngs, 2014” and “Boore, Stewart, Seyhan and Atkinson, 2014”. In the calculations, in order to be on the safe side, the average shear wave velocity in the upper 30 m (V_{S30}) was taken as 360 m/sn for very dense soil and soft rock in the current NEHRP provisions.

Repetition Period (year)	Maximum Horizontal Ground Acceleration (g)		
	Chiou and Youngs 2014	Boore, Stewart, Seyhan and Atkinson 2014	Average
144	0.101	0.097	0.100
475	0.176	0.172	0.174
975	0.230	0.231	0.231
2475	0.302	0.307	0.305

4.2. Deterministic Method

In the deterministic risk analysis a certain seismic scenario is developed and the evaluation of the ground motion is done accordingly.

The prerequisite of the scenario earthquake is the occurrence of an earthquake at a certain location and a certain intensity. This method may be typically defined in four stages (Reiter, 1990).

1. To identify the all earthquake sources that may cause ground motions, at the project area. The geometry and the potential of each individual earthquake source must be defined.
2. The selection of source-area distance parameter of each source. Mostly the minimum distance is chosen between the source and project area.
3. The determinative earthquake that is defined by a certain ground motion parameter, at the project area is selected. The correlation of the earthquake at the first step and the earthquakes assumed to be occurred in the second step will be done according to their intensity levels. The determinative earthquake (that is generally defined as moment magnitude) is described by its size and its distance to the project area.
4. The risk at the project area is generally defined as the ground motion the area that caused by the determinative earthquake.

4.2.1. Selection of Maximum Magnitude and Distance

Considering the distances of active fault that may create ground motions which may effect the Sultan Dam project area, Chaman Fault is the nearest source with 14 km. This fault is considered as a source that produce a ground motion which may effect the project area.

The maximum magnitude value that will generate by this fault is computed by the emprical relation suggested by "Wells and Coppersmith, 1994" and correlated with the existing records for using in the deterministic evaluation.

The maximum magnitude of this fault was computed and considering the historical earthquake activities, the magnitude was accepted as $M=7.5$.



4.2.2. Result of Deterministic Method

In the deterministic method, the acceleration attenuation relations suggested by "Chiou and Youngs, 2014" and "Boore, Stewart, Seyhan and Atkinson, 2014" were utilized in the computation of maximum horizontal ground acceleration value. In the calculations, in order to be on the safe side, the average shear wave velocity in the upper 30 m (V_{s30}) was taken as 360 m/sn for very dense soil and soft rock in the current NEHRP provisions.

The results of the computations are given in Table 4.

Active Fault	R (km)	M_{max}	Maximum Horizontal Ground Acceleration (g)		
			Chiou and Youngs 2014	Boore, Stewart, Seyhan and Atkinson 2014	Average
Chaman Fault	14.0	7.5	0.322	0.307	0.315

Table 4. Result of Deterministic Method

In the deterministic method, the seismic risk is determined due to the results of an earthquake with an intensity of 7.5 and with a distance of 14 km. The maximum horizontal ground acceleration at the Sultan Dam area, that will generate by this earthquake was computed as "**0.315 g**".

5. DETERMINATION OF SPECTRUM CHARACTERISTICS FOR DESIGN BASIS

The spectral accelerations for different periods for the Sultan Dam project area is computed by the probabilistic and the deterministic method utilizing "Crisis 2015" programme and using acceleration attenuation relations suggested by "Chiou and Youngs, 2014" (Table 5).



Spectral Periods	Spectral Accerelations (g) Chiou and Youngs, 2014				
	100 year %50 exceedance 144 year	50 year %10 exceedance 475 year	50 year %5 exceedance 975 year	50 year %2 exceedance 2475 year	Deterministic R=14km M=7.5
PGA (T=0.00sec)	0.101	0.176	0.230	0.302	0.322
Short Period S _s (T=0.20sec)	0.252	0.435	0.573	0.725	0.738
Long Period S ₁ (T=1.00sec)	0.062	0.101	0.134	0.185	0.326

Table 5. Spectral Accelerations of Qadis Khordak Storage Dam

The axis values of the desing spectrums are calculated by acceleration attenuation relations suggested by "Chiou and Youngs, 2014". The axis values are given in Table 6 and its graphics are given in the following figures.

Period	100 year %50 exceedance 144 year	50 year %10 exceedance 475 year	50 year %5 exceedance 975 year	50 year %2 exceedance 2475 year	Deterministic R=14km M=7.5
T (sn)	SA (g)	SA (g)	SA (g)	SA (g)	SA (g)
0.00	0.101	0.176	0.230	0.302	0.322
0.10	0.228	0.367	0.485	0.556	0.575
0.20	0.252	0.435	0.573	0.725	0.738
0.40	0.194	0.338	0.468	0.550	0.700
0.50	0.161	0.281	0.382	0.474	0.636
0.75	0.106	0.183	0.244	0.326	0.460
1.00	0.062	0.101	0.134	0.185	0.326
1.50	0.036	0.070	0.086	0.106	0.175
2.00	0.021	0.032	0.049	0.076	0.110

Table 6. The Axis Values of The Design Spectrums

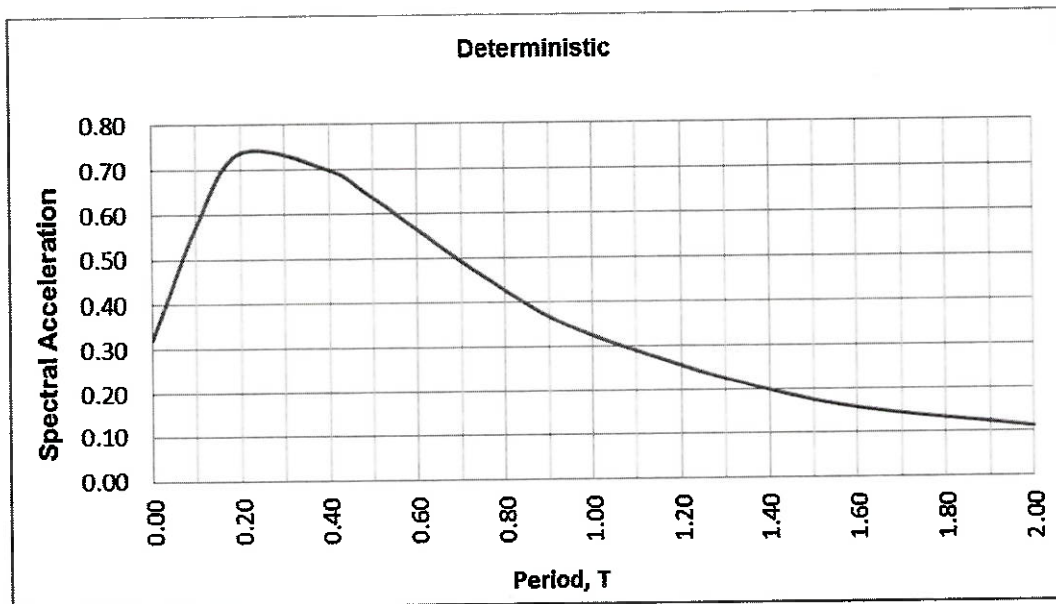


Figure 6. Design Spectrum Graph Calculated By Deterministic Method

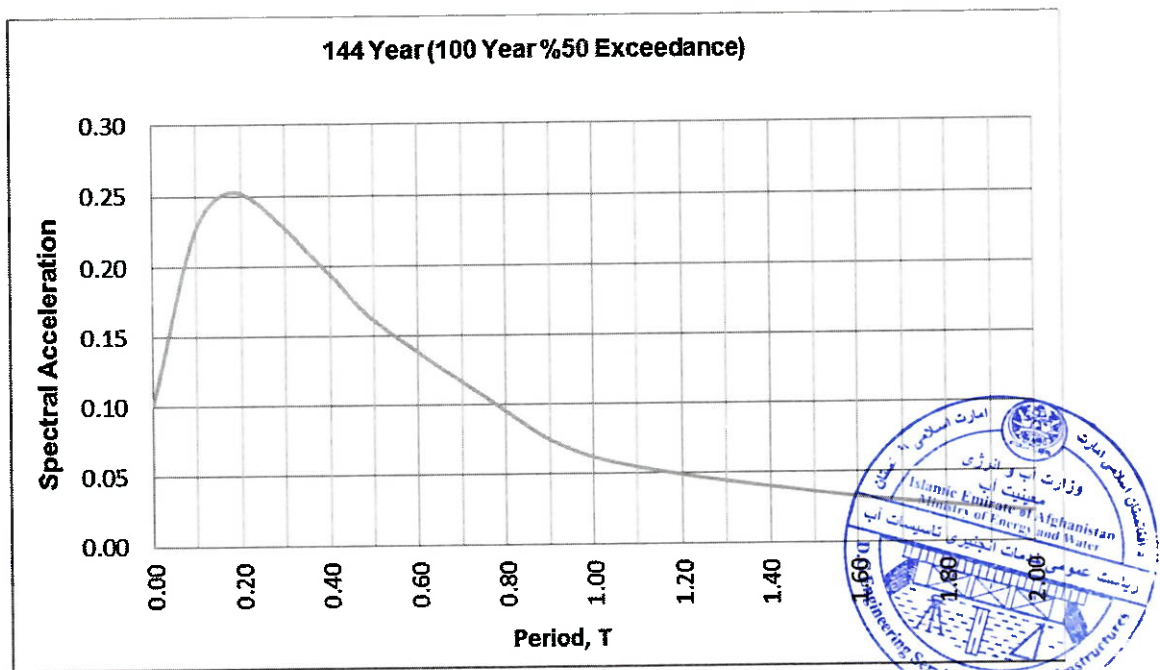


Figure 7. Design Spectrum Graph Calculated By Probabilistic Method
(100 year, %50 exceedance; 144 year recurrence period)

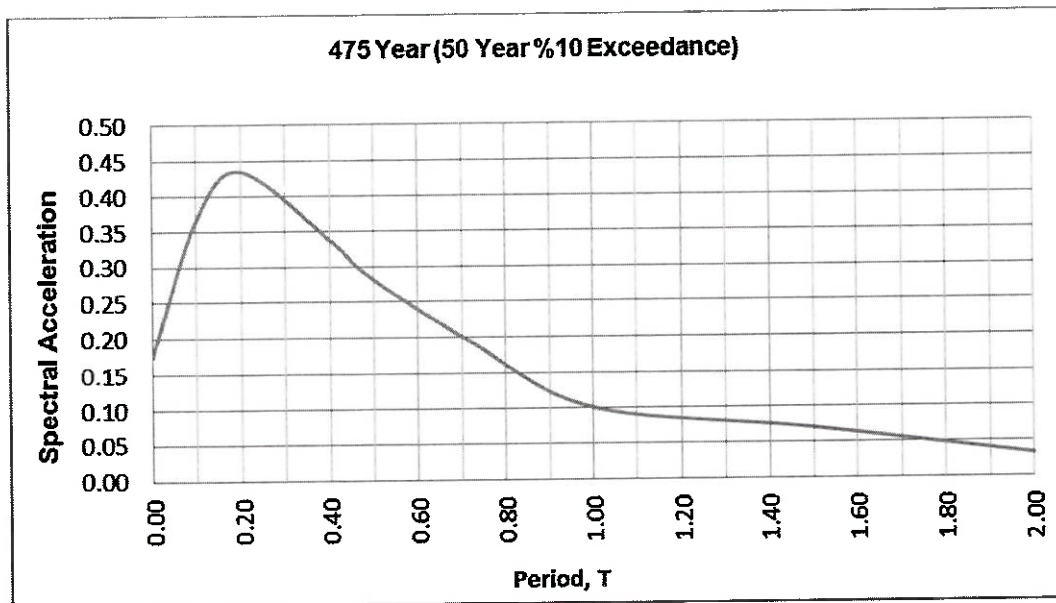


Figure 8. Design Spectrum Graph Calculated By Probabilistic Method
(50 year, %10 exceedance; 475 year recurrence period)

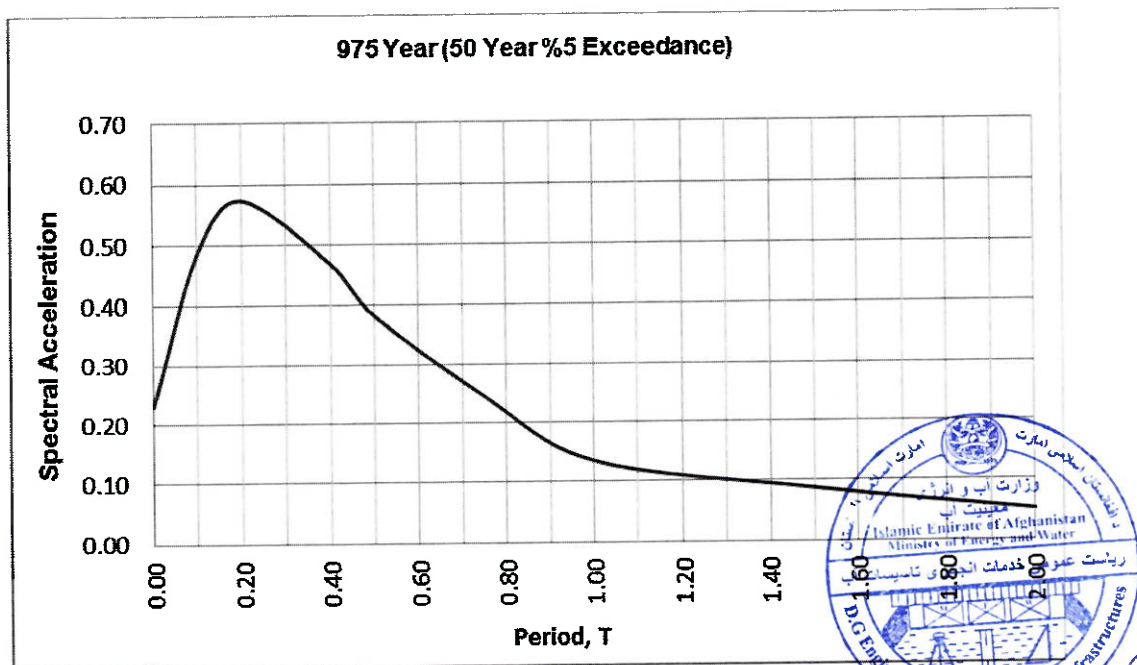


Figure 9. Design Spectrum Graph Calculated By Probabilistic Method
(50 year, %5 exceedance; 975 year recurrence period)

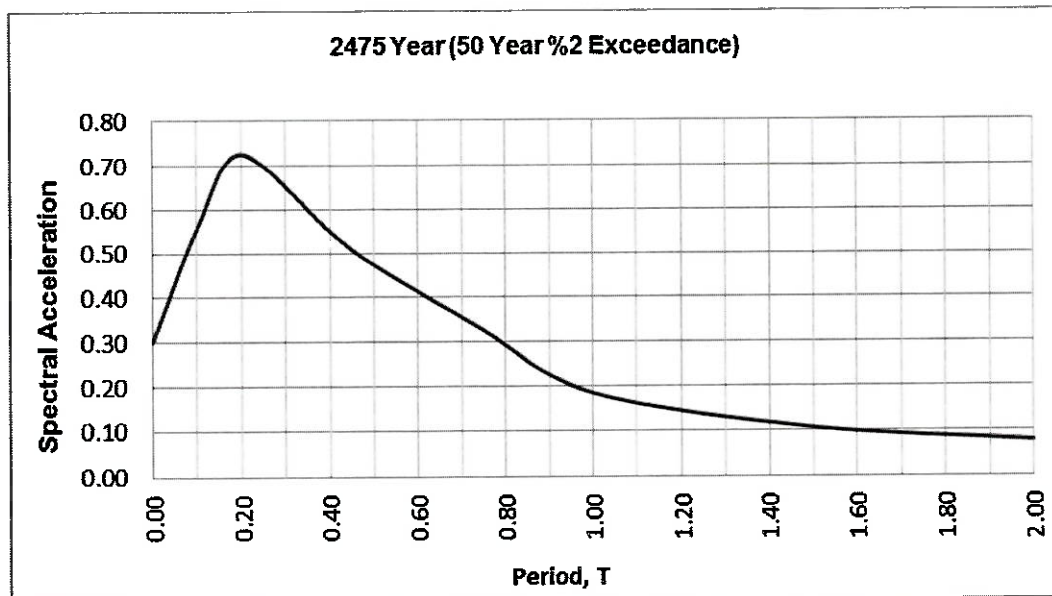


Figure 10. Design Spectrum Graph Calculated By Probabilistic Method
(50 year, %2 exceedance; 2475 year recurrence period)

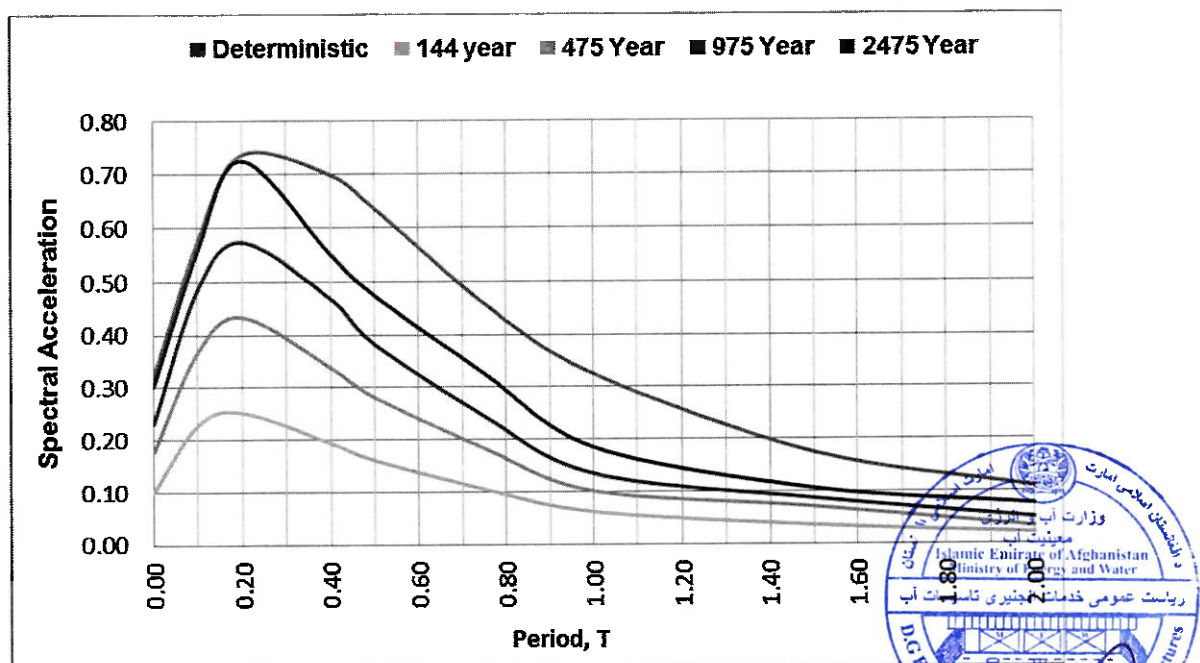


Figure 11. Comparison Graph of Design Spectrums
Calculated By Deterministic and Probabilistic Method



6. CONCLUSIONS

Sultan Dam is located on Gazni River at the north west of the Gazni province. The project area is located about 35 km north west of Gazni.

During the seismic risk analysis, the seismic sources were defined by considering the seismic characteristics of the region. The mid point of the project area (33.757151° north latitude and 68.380269° east longitude) is selected as the center. The circle of 100 km radius with the above center is used in order to determine the seismic source zones. The distribution of epicenters were evaluated by using U.S. Geological Survey (USGS) catalog values related to the earthquakes that occurred between years 1900 and 2020 with intensities $M \geq 4.0$.

The seismic risk of Sultan Dam project area is defined by using both deterministic and probabilistic methods and seismic parameters were evaluated for design purposes. The analysis were done by utilizing Crisis 2015 software, and by using the acceleration attenuation relations suggested by "Chion and Youngs, 2014" and "Boore, Stewart, Seyhan and Atkinson, 2014". The average values obtained from these analysis are defined and considered as the maximum horizontal ground acceleration value.

According to the results of "Deterministic Seismic Risk Analysis", the Chaman Fault is the nearest earthquake source in the investigation area and causes more risk than other sources. The seismic risk is defined for an earthquake with an intensity of 7.5 and 14 km away from the project area. The maximum horizontal ground acceleration that will generate by this earthquake is computed as **0.315 g**.

According to the results of "Probabilistic Seismic Risk Analysis", the maximum horizontal ground acceleration was computed as **0.101 g** for 100 years of economical life with 50% of exceedance and for a 144 years of reoccurrence period. It was computed as **0.176 g** for 50 years of economical life with 10% exceedance and 475 years of reoccurrence period. **0.302 g** for 50 years of economical life with 2% exceedance and 2475 years of reoccurrence period.



chosen as $k_f = 0.15$




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A handwritten signature in blue ink, consisting of a large, stylized capital letter 'A' followed by a smaller, cursive-like flourish.

APPENDIX

A handwritten signature in blue ink, consisting of a large, stylized 'D' followed by a vertical line and a small flourish.

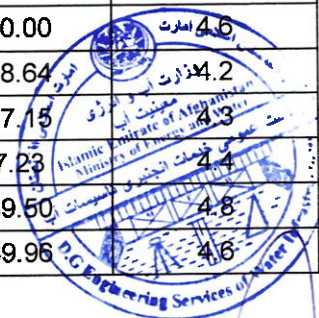
APP 1. RECORDS

EARTHQUAKE LIST OF SOURCE-1 (1900-2020.Nov, $M \geq 4.0$, <https://earthquake.usgs.gov>)

NO	TIME		LATITUDE	LONGITUDE	DEPTH (km)	MAGNITUDE
1	09.12.1979	00:29:46	34.2170	68.4910	24.60	4.1
2	27.09.1992	12:28:38	33.8260	68.3030	33.00	4.5
3	17.11.1992	02:38:50	33.7820	67.5740	33.10	5.1
4	20.09.2009	00:17:36	33.5450	67.9020	10.00	4.4
5	27.05.2010	01:30:00	34.1940	68.1730	7.30	4.4
6	27.05.2010	01:45:31	34.3930	68.3290	13.80	4.0
7	18.05.2020	08:26:07	34.4604	68.9654	10.00	4.3

EARTHQUAKE LIST OF SOURCE-2 (1900-2020.Nov, $M \geq 4.0$, <https://earthquake.usgs.gov>)

NO	TIME		LATITUDE	LONGITUDE	DEPTH (km)	MAGNITUDE
1	08.04.1958	09:59:24	33.0400	68.2390	35.00	5.6
2	19.05.1959	15:17:46	33.0680	68.1830	25.00	5.7
3	17.04.1973	03:37:48	33.3200	68.1090	40.00	5.1
4	06.09.1977	02:11:13	33.3660	69.0460	44.00	4.4
5	04.12.1981	12:13:42	33.9010	69.3140	33.00	4.8
6	14.09.1988	09:44:10	33.2480	68.6350	33.00	4.7
7	02.04.1990	18:19:51	33.4050	68.9230	33.00	4.4
8	02.04.1990	19:06:10	33.1920	68.2940	12.60	5.1
9	20.06.1992	21:18:34	33.7710	68.7730	33.00	4.2
10	23.03.1995	08:07:51	32.8890	68.1430	33.00	4.2
11	20.05.2004	15:39:24	33.1990	68.1950	10.00	4.5
12	20.06.2006	01:46:32	33.7520	69.4060	35.00	4.1
13	12.07.2006	05:16:26	33.9370	69.3030	33.30	4.7
14	13.08.2011	16:58:48	33.4110	68.7880	35.00	4.3
15	24.11.2011	20:38:21	33.7570	69.1500	10.00	4.3
16	13.08.2013	01:43:08	33.2348	68.1353	15.91	4.5
17	20.03.2015	08:24:27	33.7730	69.3536	10.00	4.2
18	27.01.2016	10:41:06	33.7119	68.7772	25.24	4.4
19	08.12.2016	10:01:59	34.1753	69.2760	10.00	4.6
20	08.12.2016	14:34:51	34.2167	69.1294	18.64	4.2
21	17.05.2017	15:05:27	34.1767	69.2214	17.15	4.3
22	03.10.2017	16:28:59	34.0848	69.1716	7.23	4.4
23	08.05.2018	18:17:55	32.9750	68.5817	39.50	4.8
24	28.05.2018	19:19:42	32.9691	68.5616	39.96	4.6





Islamic Emirate of Afghanistan
Ministry of Energy and Water
Deputy Ministry of Water



General Directorate of Engineering Services of Water Infrastructures

Technical Board



Sultan Dam Rehabilitation		
Date		Dec 2023
Prepared	Checked	Approved
Eng. Hedayatullah Omari	Eng. Faraidon Danish	Abdul Ghafor Omari

Survey Report

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Introduction

Ghazni Province is one of thirty-four provinces of Afghanistan located at the southern side of Afghanistan, from north connected to Maidan-wardak from east connected to paktika and Ghazni from west connected to urozgan and dikundi and for south connected to Zabul, Ghazni is the capital of Ghazni province lies at a distance of 100 km south of Kabul, Almost 90 percent of the citizens are engaged in agriculture and livestock and thereby meet their household needs, Drought has increased in recent years and people are forced to use solar pumps to grow their agricultural products and horticulture.

Sultan Dam Project is located in this province specifically in Jaghato District. The dam site is located 25 km from province capital, Ghazni city, some site specifications are:

Administration: Province: Ghazni

District: Jaghato

River basin: Helmand

Sub-basin: Ghazni

Coordinates: Easting: 442601

Northing: 3735380

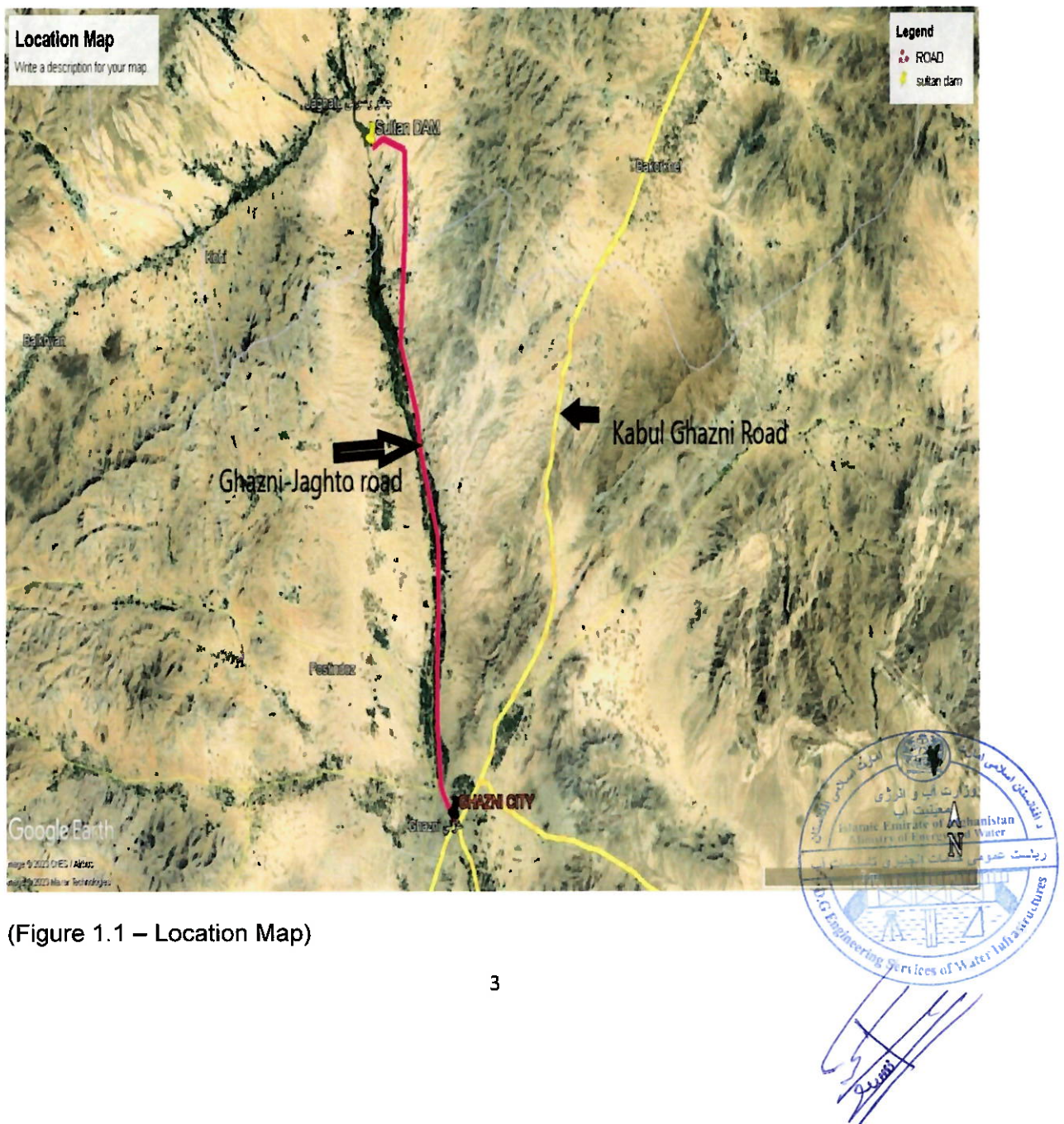


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Location Map

Sultan Dam Project is located Ghazni province specifically in Jaghato District. The dam site is located 25 km from province capital, Ghazni city,

Dam location Easting: 442601 Northing: 3735380
Coordinates:



(Figure 1.1 – Location Map)

Catchment Area

The sources of the Sultan River are snowmelt runoff and rainfall. The catchment area at the dam site was estimated to be about 1172 km². The watershed is covered with sparse grass and appreciable shrub or tree cover. Figure 1.2 presents the catchment area at the proposed dam site.

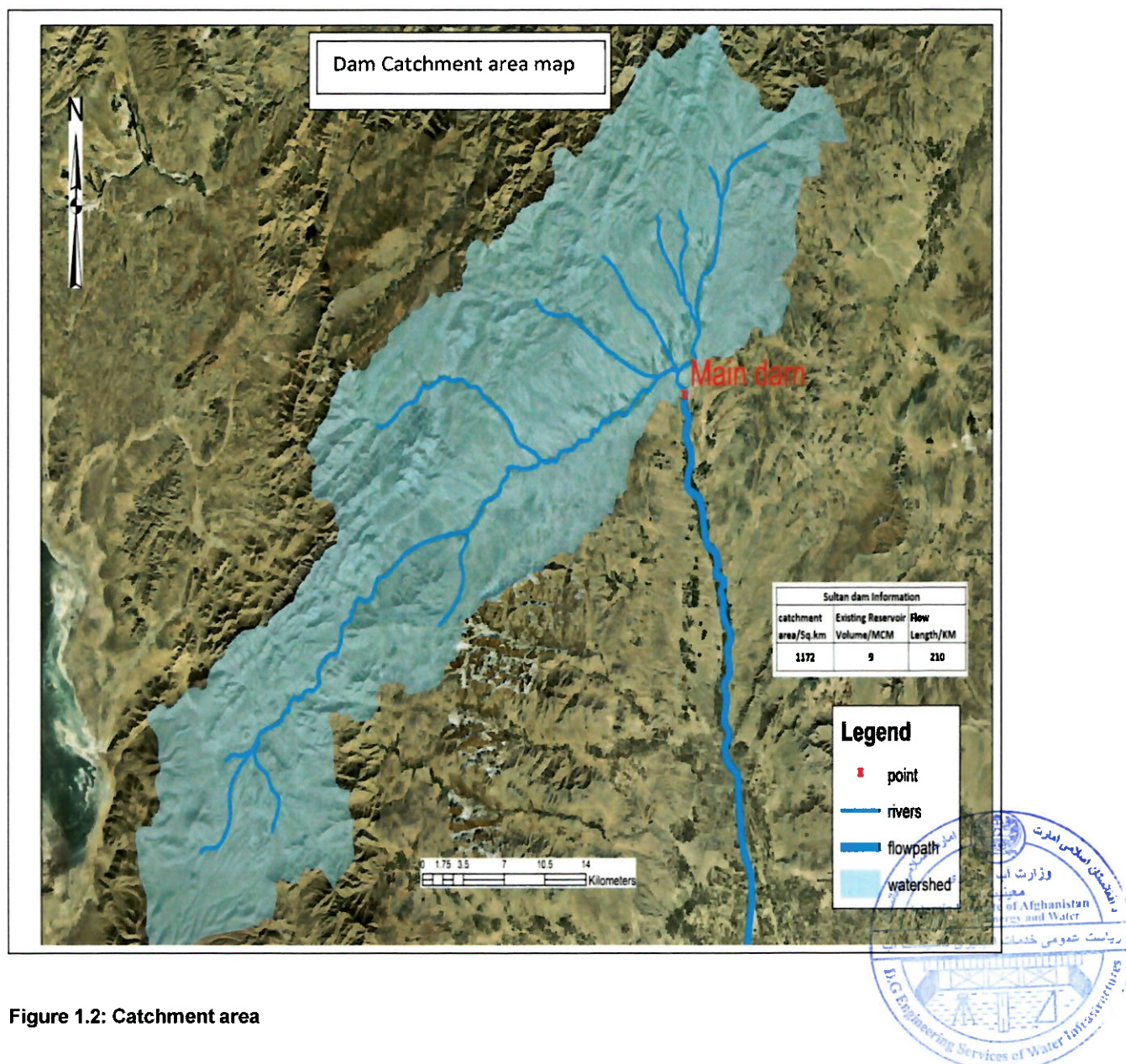
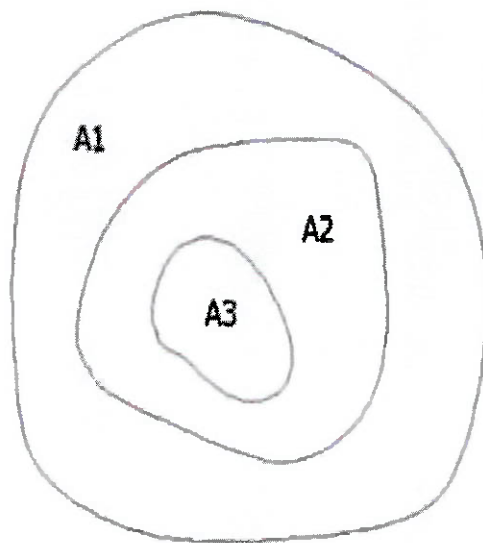


Figure 1.2: Catchment area

Reservoir Volume

Reservoir Volume calculated from on ground survey data and contours with 1 meter interval by civil 3d software which base is the below formula.

$$\text{Volume} = \left(\frac{A_1 + A_2 + \dots + A_n}{n} \right) \times ((n - 1) \times d)$$



A1,A2,A3 = area
n = number of area
d = contour interval



Sultan DAM TOP VIEW

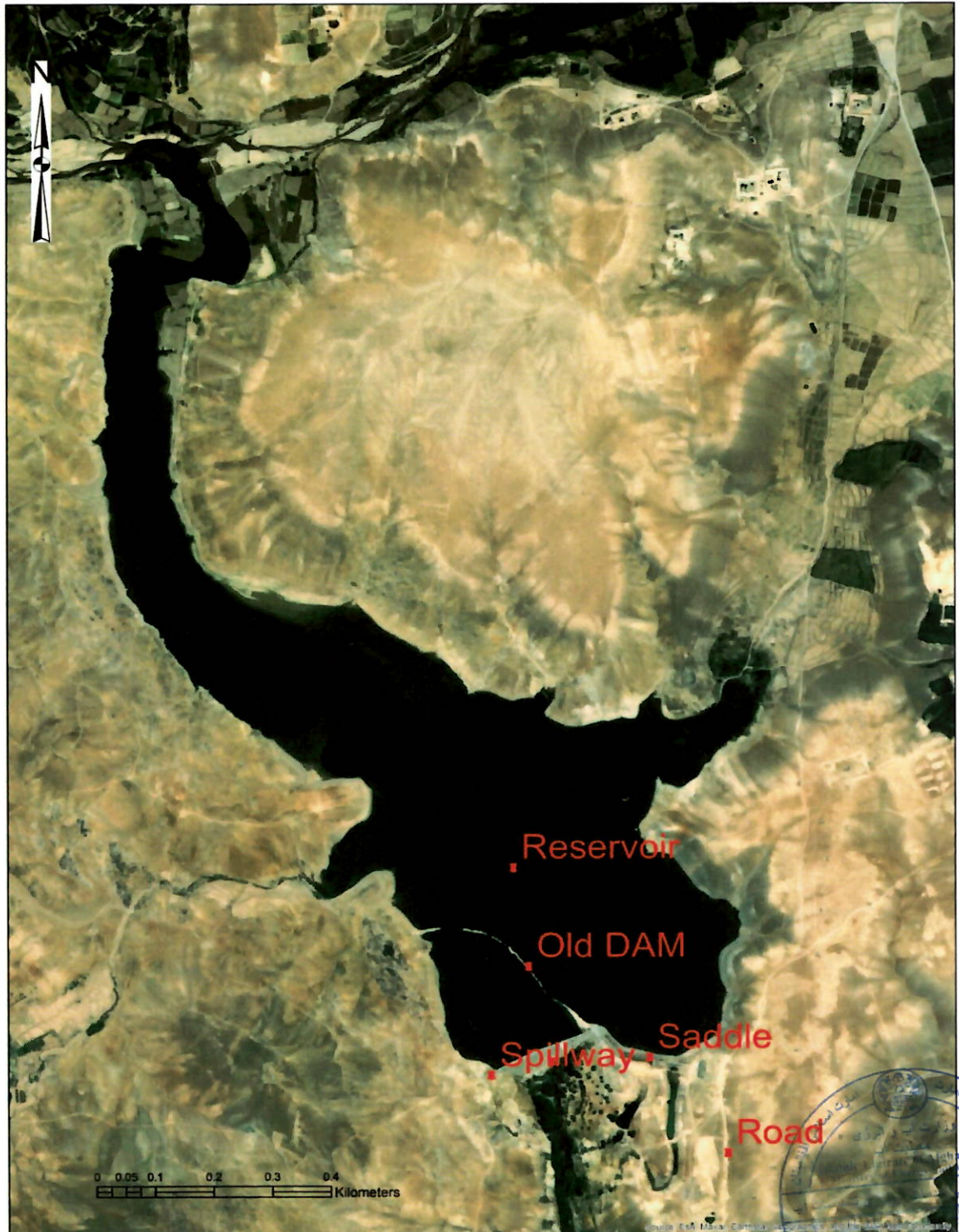


Figure 1.3: water reservoir

Sultan dam volume Report				
Contour Elevation	Contour Area(sq.m)	Depth (m)	Incremental Volume Avg.End (cu.m)	Cumulative Volume Avg.End (cu.m)
2,352.00	1,814.66	0	0	0
2,353.00	4,279.92	1	3047.29	3047.29
2,354.00	5,622.26	1	4951.09	7998.38
2,355.00	7,236.31	1	6429.285	14427.665
2,356.00	11,932.79	1	9584.55	24012.215
2,357.00	18,431.76	1	15182.275	39194.49
2,358.00	23,785.53	1	21108.645	60303.135
2,359.00	28,664.21	1	26224.87	86528.005
2,360.00	34,175.30	1	31419.755	117947.76
2,361.00	40,902.27	1	37538.785	155486.545
2,362.00	47,074.21	1	43988.24	199474.785
2,363.00	55,225.43	1	51149.82	250624.605
2,364.00	74,728.41	1	64976.92	315601.525
2,365.00	120,886.12	1	97807.265	413408.79
2,366.00	155,171.50	1	138028.81	551437.6
2,367.00	181,478.33	1	168324.915	719762.515
2,368.00	205,035.63	1	193256.98	913019.495
2,369.00	265,798.99	1	235417.31	1148436.805
2,370.00	307,651.00	1	286724.995	1435161.8
2,371.00	347,026.93	1	327338.965	1762500.765
2,372.00	382,977.63	1	365002.28	2127503.045
2,373.00	435,089.25	1	409033.44	2536536.485
2,374.00	488,498.74	1	461793.995	2998330.48
2,375.00	539,813.70	1	514156.22	3512486.7
2,376.00	601,769.81	1	570791.755	4083278.455
2,377.00	664,640.28	1	633205.045	4716483.5
2,378.00	748,673.99	1	706657.135	5423140.635
2,379.00	823,805.50	1	786239.745	6209380.38
2,380.00	897,310.69	1	860558.095	7069938.475
2,381.00	971,104.25	1	934207.47	8004145.945
2,382.00	1,044,234.94	1	1007669.595	9011815.54
2,383.00	1,137,710.15	1	1090972.545	10102788.09
2,384.00	1,229,103.49	1	1183406.82	11286194.91
2,385.00	1,325,614.79	1	1277359.14	12563554.05
2,386.00	1,428,275.98	1	1376945.385	13940499.43

TOPOGRAPHIC SURVEY METHODOLOGY:

A topographic survey gathers data about the natural and man-made features of the land, as well as its terrain. Permanent features such as buildings, fences, trees and streams accurately define the ground and its boundaries. Land contours and spot levels show the elevation of the terrain. Followings describe the hardware and field procedures associated with collecting survey data that we have used to meet the requirements of the project:

SURVEY EQUIPMENT:

The GPS and Total station are the main tools that have a direct influence of the quality and quantity of the data to be collected. Following equipment were used to collect the necessary data:

- GPS Unit
- Sokkia CX105 Total station
- Tripod
- Reflector
- Meter
- Camera
- Nails
- Spray
- Cements
- Shovel
- Hummer
- Tape



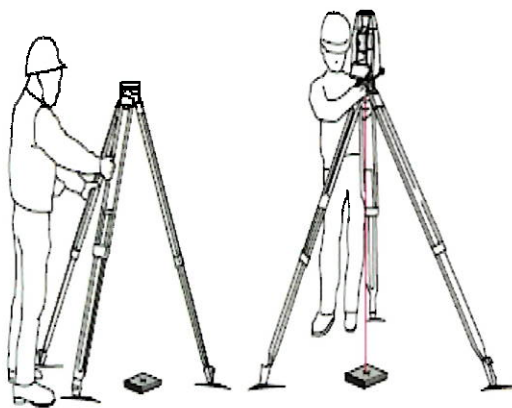
- Note book
- wire less
- Lap top Computers



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BMs AND COORDINATE SYSTEM:

To perform the topographic survey, we established numbers of permanent benchmarks with good quality of materials . Especially we established 3 main high qualities permanent benchmark near the existing dam.



On the first working day we installed 4 physical concrete benchmarks in appropriate locations. On the second day, we started GPS observation in order to get coordinates for first station that we named Station1 and also we get second point coordinates for backside, the total station is settled on station 1 and adjusted and the reflector is settle on backside point, after instrument preparation we start topography survey and measured on concrete BM and recorded its coordinates and elevation . The coordinate system we used is WGS84 UTM Zone 42N Coordinate system.. The BMs coordinate are shown in BMs table 1-2. These points are used as our reference points for all topography survey.



BM Table				
No	Northing	Easting	Elevation	Description
1	3735380.37	442758.14	2378.69	BM1
2	3735341.67	442550.25	2381.35	BM2
3	3735295.24	442665.67	2363.44	BM3

Table 1-2

Survey Code and abbreviations

Deferent survey code is used in topography survey, but it's often belonging to site features and we used following codes in our topography survey.

Survey Codes and abbreviations	
SPW	Spillway
DC	Diversion Canal
DAXIS	Dam Axis
NSL	Natural Surface Level
ER	Edge River
WL	Water Level
WCL	Water Center Line
CP	Change Point
NWL:	Normal Water Level (In the reservoir)
MWL:	Maximum Water Level (In the reservoir)
HROCK	Hard Rock
RB	River Bed Level
BM	Bench marks
BV	Bed of Volley
EV	Edge of Volley
HOUSE	House
AG	Agricultural Land
HFL	High Flood Level
TREE	TREE

Table1-3

Topography survey data:

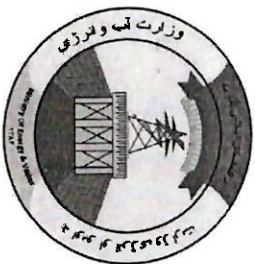
Survey data is collected by survey team with GPS (Garman) and total station (sokkia cx 105) instruments.

The data of dam related to structures like (Dam axis, spillway, powerhouse, access road,)

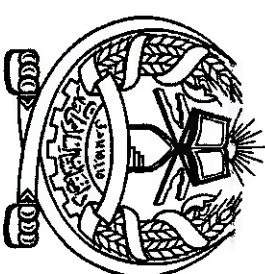
The data format is point No, Northing, Easting, Elevation and descriptions (PNEZD)

Survey data showed in soft





Islamic Emirate of Afghanistan
Ministry of Water and Energy
Deputy Ministry of Water



General Directorate of Engineering Services of Water Infrastructures

Technical Board

Rehabilitation of Sultan Dam

Date		October-2023	
Prepared	Checked	Approved	
Ahmad Sohail Noori	Shafiqullah Naseri	Abdul Ghafor Omari	

Spillway Drawings

SHEET INDEX OF SPILLWAY DRAWINGS

S.NO.	SHEET NO.	SHEET TITLE
01	S-01	Sheet Index
02	S-02	General Notes
03	S-03	General Notes
04	S-04	General Notes
05	S-05	Site Plan
06	S-06	Excavation Plan
07	S-07	Coordinates Tables
08	S-08	Profiles
09	S-09	Chainage Plan
10	S-10	Spillway Section 0+000 to 0+015
11	S-11	Spillway Section 0+020 to 0+035
12	S-12	Spillway Section 0+040 to 0+055
13	S-13	Spillway Section 0+060 to 0+075
14	S-14	Spillway Section 0+080 to 0+095
15	S-15	Spillway Section 0+100 to 0+105
16	S-16	Plan and Section of Duckbill Spillway
17	S-17	Sections of Oggee Structures
18	S-18	Duckbill Section and Water Stop Details
19	S-19	
20	S-20	
21	S-21	
22	S-22	
23	S-23	
24	S-24	
25	S-25	



GENERAL

- * UNLESS NOTED OR OTHERWISE SHOWN ON THE DRAWINGS OR OTHERWISE DIRECTED, THE FOLLOWING GENERAL NOTES AND DETAILS APPLY TO ALL STRUCTURES.
- * ALL STANDARDS MENTIONED ARE TO BE APPLIED IN THEIR LATEST EDITION.
- * ALL DIMENSIONS AND ELEVATIONS/LEVELS ARE IN METERS AND MILLIMETERS UNLESS OTHERWISE NOTED.
- * ALL DIMENSIONS AND COORDINATES AFFECTING THE WORK AT JOB SITE SHALL BE VERIFIED AS PER DRAWING OR APPROVED BY THE ENGINEER.
- * DIMENSIONS MENTIONED ON DRAWING SHOULD BE FOLLOWED IN PREFERENCE TO SCALE MEASUREMENTS.
- * STRUCTURE COORDINATES ARE SUBJECT TO CONFIRMATION AT SITE BY THE ENGINEER.
- * FOR LOCATIONS AND DIMENSIONS OF GROOVES, SLEEVES, CURBS, OPENINGS, EMBEDDED OR ATTACHED ITEMS REFER LAYOUT DRAWINGS.

1-STRUCTURAL STEEL AND METALWORK

- * DESIGN, DETAILING AND FABRICATION OF STRUCTURAL STEEL WORK SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF THE "SPECIFICATION FOR THE DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS" AISC (AMERICAN INSTITUTE OF STEEL CONSTRUCTION) MANUAL OF STEEL CONSTRUCTION.
- * STRUCTURAL STEEL CONSTRUCTION SHALL BE WELDED EXCEPT AS NOTED.
- * ALL WELDING SHALL BE IN ACCORDANCE WITH THE APPLICABLE REQUIREMENTS OF THE BS 4871 AND BS 5135 CODES.
- * STRUCTURAL STEEL AND EMBEDDED STEEL SHALL CONFORM TO THE REQUIREMENTS A36 UNLESS OTHERWISE INDICATED.
- * NO HOLES, CUTS OR COPIES WILL BE PERMITTED EXCEPT AS INDICATED ON DRAWINGS. NO CUTTING OR BURNING OF STEEL SHALL BE PERMITTED WITHOUT APPROVAL OF THE ENGINEER.
- * FOR ALLOWABLE STRESSES REFER "SPECIFICATION FOR THE DESIGN FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS" AISC MANUAL OF STEEL CONSTRUCTION.
- * EMBEDDED METAL SUCH AS PIPING, FITTING, VALVES AND DRAINS, FOR DIFFERENT SYSTEMS SUCH AS DRAINAGE SYSTEM, WATER SYSTEM, UNWATERING SYSTEM, OIL HANDLING SYSTEM, COMPRESSED AIR SYSTEM, FIRE PROTECTION SYSTEM ETC. AND VENTS TURBINE, DRAFT TUBE HVAC ETC. ARE SHOWN PARTLY AND SUBJECT TO ADDITIONS AND ALTERATIONS AND APPROVAL OF THE ENGINEER.

2-CONCRETE

A. DESIGN OF STRUCTURES

DESIGN OF STRUCTURES IS IN ACCORDANCE WITH "BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE (ACI 318-14) AND COMMENTARY (ACI 318R-14)", AMERICAN CONCRETE INSTITUTE, U.S.A.

B. DESIGN OF CONCRETE:

* DESIGN OF STRUCTURAL CONCRETE , IN GENERAL, IS BASED ON THE FOLLOWING

GRADE	MIN. COMPRESSIVE CYLINDER STRENGTH AT 28 DAYS (MPa)	TYPICAL LOCATION	MAX. AGG. SIZE (mm.)
G35	35	STRUCTURES EXPOSED TO RAPID FLOW	40
G31	31	WATER AND EARTH RETAINING	40
G28	28	STRUCTURES, NORMAL REINF. CONC.	40
G25	25	STRUCTURES.	50
G15	15 AT 90 DAYS	MASS CONC. (EXTERIOR) UNREINFORCED STRUCTURES NOT SUBJECTED TO FLOWING WATER	150
G15	15	LEAN CONC. (INTERIOR) AND FILL CONC.	75 TO 150

* STAIRS WILL BE CAST-IN-PLACE REINFORCED CONC. WITH NON-SKID MONOLITHIC TREADS.

- * BEFORE PLACING CONCRETE, ALL ITEMS FOR EMBEDMENT SHALL BE IN POSITION AND SECURELY FASTENED IN PLACE. ALL ITEMS FOR EMBEDMENT ARE NOT SHOWN ON THE CONCRETE OUTLINE DRAWINGS. SEE APPLICABLE MECHANICAL ELECTRICAL AND EQUIPMENT SUPPLIERS AND SHOP DRAWINGS FOR ADDITIONAL INFORMATION FOR LOCATION, SETTING, AND ARRANGEMENT OF ITEMS FOR EMBEDMENT
- * ELECTRIC CONDUITS AND PIPING EMBEDMENT IN STRUCTURAL CONCRETE SHALL MEET THE APPLICABLE REQUIREMENTS FOR PLACEMENT OF ACI "BUILDING CODE" (318-99) FOR REINFORCEMENT CONCRETE
- * ALL REINFORCING STEEL AND DOWELS SHALL BE WELL SECURED IN POSITION PRIOR TO POURING CONCRETE.
- * RUBBER AND PVC WATERSTOP SHALL BE USED AS PER SPECIFICATION AND APPROVED BY THE ENGINEER.
- * WATERSTOPS SHALL BE SUPPORTED/PROTECTED FROM DAMAGE AND EXPOSURE.
- * WATER STOPS WILL BE PROVIDED AS FOLLOWS:
 1. BELOW MAX TAIL WATER LEVEL AND BACKFILL SURFACE, RESPECTIVELY.
 - a) DOUBLE POLYVINYL CHLORIDE WATER STOPS IN VERTICAL CONSTRUCTION JOINTS,
 - b) SINGLE POLYVINYL CHLORIDE WATERSTOPS IN ALL VERTICAL CONSTRUCTION JOINTS: SINGLE WATERSTOPS IN HORIZONTAL (LIFT) JOINTS.
 2. ABOVE MAX TAIL WATER LEVEL AND BACKFILL SURFACE, RESPECTIVELY SINGLE POLYVINYL CHLORIDE WATERSTOPS IN ALL VERTICAL CONTRACTION AND EXPANSION JOINTS.
- * BEAMS, COLUMNS AND WALLS ARE CENTERED ON REFERENCE LINES EXCEPT AS NOTED. BEAMS/SLAB, DEPTHS AND DEPTH OF RECESSES SHALL BE MEASURED FROM THE TOP OF THE STRUCTURAL SLAB.

E. LIFT HEIGHT AND TIME BETWEEN PLACEMENT

S.#	LOCATION	MAX.DEPH OF CONCRETE PLACED IN ONE LIFT	MIN.TIME ELAPSING BETWEEN PLACING OF SUCCESSIVE LIFTS
1	CONCRETE IN BASE SLABS AND IN WALLS COLUMNS AND PIERS WHOSE THICKNESS DOES NOT EXCEED 3 m (10FT.)	3.0 m(10FT.)	120 hr
2	GATE GUIDE BLACKOUTS	4.0 m(15FT.)	8 hr
3	ALL OTHER CONCRETE	1.5 m(5FT.)	120 hr

F. CONSTRUCTION AND CONTRACTION JOINTS

- * CONSTRUCTION JOINTS IF NOT SHOWN ON THE DRAWINGS SHALL BE AS APPROVED BY THE ENGINEER.
- * SEQUENCE OF POURING SHALL BE AS APPROVED BY THE ENGINEER.
- * CONSTRUCTION JOINTS SHALL BE TREATED IN ACCORDANCE WITH THE SPECIFICATIONS OR APPROVED BY THE ENGINEER.
- * REINFORCEMENT SHOULD BE TERMINATED 50mm FROM THE FACE OF JOINTS.
- * EXPANSION JOINT, 25mm THICK AS INDICATED ON DRAWINGS.

G. WATERSTOPS

POLYVINYL CHLORIDE OR RUBBER WATERSTOPS WILL BE PROVIDED IN THE CONSTRUCTION, EXPANSION AND CONTRACTION JOINTS TO PREVENT SEEPAGE THROUGH THE JOINTS.

H. FINISHES AND MISCELLANEOUS

- * LOCATION AND CLASSES OF FINISHES FOR FORMED AND UNFORMED SURFACES SHALL BE IN ACCORDANCE WITH SPECIFICATIONS CLAUSES 8.4.23 AND 8.4.25

3. FOUNDATIONS

- * ROCK/NATURAL SOIL SURFACE AND BOTTOM LINES OF CONCRETE ARE SHOWN ON THE DRAWINGS.
- * ROCK SURFACE AND FOUNDATION LINES SHOWN ON THE DRAWING ARE BASED ON PRESENT KNOWLEDGE OF FOUNDATION CONDITIONS. FOUNDATION LEVELS MAY BE MODIFIED BY THE ENGINEER. INFERIOR ROCK SHALL BE REPLACED BY FILL CONCRETE OR DENTAL CONCRETE AS DIRECTED BY THE ENGINEER.
- * ROCK/NATURAL SOIL WILL NOT BE PERMITTED TO PROJECT MORE THAN 152mm WITHIN THE THEOCRITICAL CONCRETE OUTLINES (EXCEPT FILL CONC.) SHOWN ON THE DRAWINGS.



4. REINFORCING STEEL

- * UNLESS OTHERWISE SHOWN ON THE REINFORCEMENT DRAWINGS, OR OTHERWISE DIRECTED, THE FOLLOWING STANDARD GENERAL NOTES APPLY TO ALL REINFORCED CONCRETE STRUCTURES.

A. DIMENSIONS

- * SPACING OF BARS AND DIAMETER IS GIVEN IN MILLIMETER
- * IF SHOWN BAR # IN NOT AVAILABLE, THE BAR DIAMETER AND SPACING MAY BE REVISED TO NEXT SMALLER OR LARGER DIAMETER KEEPING SAME STEEL CROSS SECTION SUBJECT TO MINIMUM / MAXIMUM SPACING REQUIREMENTS.

B. MATERIAL

ALL REINFORCEMENT STEEL BARS SHALL BE DEFORMED BARS STEEL BARS, CONFORMING TO ASTM-A615 GRADE 60 AND 40.

C. PLACING REINFORCING BARS

- * ALL REINFORCING BARS SHALL BE PLACED IN ACCORDANCE WITH THE REQUIREMENTS OF ACI BUILDING CODE (318-14)
- * REINFORCEMENT MAY BE ADJUSTED AS NECESSARY TO CLEAR PIPES, SEALS, RECESSES, EMBEDDED METALWORK AND CONDUITS EXCEPT IN AREAS OF THE EMBEDDED MATERIALS MUST BE CONSIDERED.
- * REINFORCEMENT BARS SHALL NOT BE BENT OR STRAIGHTENED IN A MANNER THAT WILL INJURE THE MATERIAL.
- * THE FIRST AND LAST BARS IN SLABS AND WALLS, SHALL BE PLACED AT A MAXIMUM OF ONE HALF OF THE BARS SPACING SHOWN UNLESS OTHERWISE NOTED.
- * THE CLEAR DISTANCE BETWEEN PARALLEL BARS IN A LAYER SHALL NOT BE LESS THAN THE NOMINAL DIAMETER OF THE BAR.
- * THE CLEAR DISTANCE BETWEEN LONGITUDINAL BARS IN COLUMNS SHALL NOT BE LESS THAN 1½ TIMES THE NOMINAL BARS DIAMETER NOR 40mm A UNIT OF BUNDLED BARS SHALL BE TREATED AS A SINGLE BAR OF A DIAMETER DERIVED FROM THE EQUIVALENT TOTAL DIAMETER.
- * THE CLEAR DISTANCE LIMITATION BETWEEN BARS SHALL ALSO APPLY TO CLEAR DISTANCE BETWEEN A CONTACT LAP SPUCE AND ADJACENT SPUCES OF BARS.

D. SPACING

- * THE FIRST AND LAST BARS IN SLABS AND WALLS, SHALL BE PLACED AT A MAXIMUM OF ONE HALF OF THE BARS SPACING SHOWN UNLESS OTHERWISE NOTED.
- * THE CLEAR DISTANCE BETWEEN PARALLEL BARS IN A LAYER SHALL NOT BE LESS THAN THE NOMINAL DIAMETER OF THE BAR, NOR 25 mm.
- * THE CLEAR DISTANCE BETWEEN LONGITUDINAL BARS IN COLUMNS SHALL NOT BE LESS THAN 1½ TIMES THE NOMINAL BARS DIAMETER NOR 40 mm. A UNIT OF BUNDLED BARS SHALL BE TREATED AS A SINGLE BAR OF A DIAMETER DERIVED FROM THE EQUIVALENT TOTAL DIAMETER.
- * THE CLEAR DISTANCE LIMITATION BETWEEN BARS SHALL ALSO APPLY TO CLEAR DISTANCE BETWEEN A CONTACT LAP SPUCE AND ADJACENT SPUCES OF BARS.

E. SPUCES

- * LOCATION OF SPUCES IS SHOWN ON THE DRAWINGS.
- * SPUCES NOT SHOWN ON THE DRAWINGS SHALL BE IN ACCORDANCE WITH THE APPLICABLE REQUIREMENTS FOR DETAILING REINFORCEMENT AND AS APPROVED BY THE ENGINEER.
- * WHEN BARS OF DIFFERENT SIZE SPUCE, THE LAP LENGTH SHALL BE GOVERNED BY THE SMALLER BAR UNLESS OTHERWISE NOTED.

- * WALL AND SLAB REINFORCEMENT SPUCES ARE TO BE MADE SO THAT GIVEN DISTANCES TO THE FACE OF WALL, SLAB, OR REFERENCE LINES WILL BE MAINTAINED.

- * ALL SPUCING OF REINFORCEMENT BARS SHALL BE DONE BY LAPPING FLEXURAL MEMBERS SHALL NOT BE SPACED TRANSVERSELY FARTHER APART THAN 1/5 THE REQUIRED LAP LENGTH NOR 152mm.

- * TENSION LAP SPUCES SHALL BE STAGGERED LONGITUDINALLY SO THAT NO SECTION WITHIN THE BARS ARE LAPPED LESS THAN REQUIRED LAP LENGTH.

DEVELOPMENT LENGTH (mm) FOR BARS IN TENSION												
CONCRETE SPECIFICATION	BAR SIZE (mm)											
	10	12	14	16	18	20	22	25	28	30	32	36
35 MPa	350	415	480	550	615	685	720	980	1110	1245	1350	1480
31 MPa	370	440	510	580	650	720	980	1110	1245	1350	1420	1550
28 MPa	385	465	540	615	680	760	1030	1170	1310	1400	1490	1630
25 MPa	410	490	570	650	725	805	1090	1240	1390	1480	1580	1725

- NOTES. FOR SPUCE LENGTH THE VALUES OF DEVELOPMENT LENGTH SHOULD BE MULTIPLIED WITH FACTOR 1.3.

DEVELOPMENT LENGTH (mm) FOR BARS IN COMPRESSION												
CONCRETE SPECIFICATION	BAR SIZE (mm)											
	10	12	14	16	18	20	22	25	28	30	32	36
35 MPa	200	230	265	290	340	365	410	480	520	550	590	640
31 MPa	200	230	265	310	335	370	415	465	530	560	600	645
28 MPa	210	240	285	320	360	390	435	495	550	565	620	680
25 MPa	210	260	300	340	380	420	460	520	580	630	660	720

- NOTES. FOR SPUCE LENGTH THE VALUES OF DEVELOPMENT LENGTH SHOULD BE MULTIPLIED WITH FACTOR 1.3.

DEVELOPMENT LENGTH (mm) FOR BARS WITH STANDARD HOOKS												
CONCRETE SPECIFICATION	BAR SIZE (mm)											
	10	12	14	16	18	20	22	25	28	30	32	36
35 MPa	325	400	460	520	590	660	720	820	920	980	1050	1150
31 MPa	350	420	490	560	625	695	765	870	970	1050	1120	1220
28 MPa	365	440	510	580	660	730	800	910	1020	1100	1180	1280
25 MPa	390	465	540	620	700	770	850	970	1100	1160	1240	1350

- ALL BARS MARKED CONTINUOUS SHALL BE PROPERLY LAPPED AT SPUCES, AND CORNERS AND HOOKED AT NON CONTINUOUS ENDS.

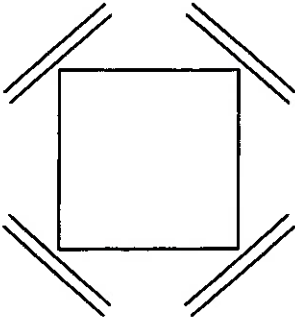
F. CLEAR CONCRETE COVER AND OPENING

- * CLEAR CONCRETE COVER FOR REINFORCING BARS CAST IN PLACE CONCRETE SHALL BE PROVIDED, UNLESS OTHERWISE NOTED.

S#	TYPE	COVER (mm)
1.	BOTTOM OF FOUNDATIONS AND FOOTINGS	100
2.	BACKFILLED SURFACES, WATER RETAINING SURFACES AND SURFACES SUBJECTED TO SUBMERGENCE OR RAPID FLOW	100
3.	OTHER EXTERIOR WALLS, BEAMS SLABS AND COLUMNS	75
	1. ELEMENTS GREATER THAN 915mm THICK	50
	2. ELEMENTS 610 TO 915mm THICK	40
	3. ELEMENTS LESS THAN 610mm THICK	40
4.	INTERIOR WALLS, BEAMS, GIRDERS AND COLUMNS	40
5.	INTERIOR WALLS AND SLABS	25

- * IN NO CASE SHALL THE CONCRETE COVER BE LESS THAN 1.5 TIMES THE NOMINAL SIZE OF AGGREGATE, OR 2 TIMES THE MAX. DIAMETER OF REINFORCEMENT.

- * FOR OPENING MINIMUM SIZE 610mm x 610mm OR MORE, PROVIDE 2 #16 AS BELOW JOIN AT LEAST DEFORMED BARS AT EACH CORNER IF NOT SPECIFIED.

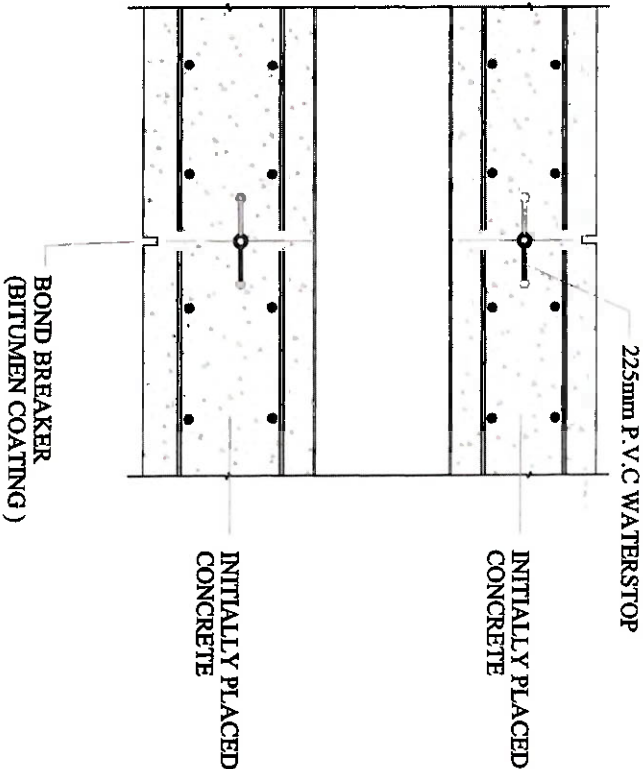
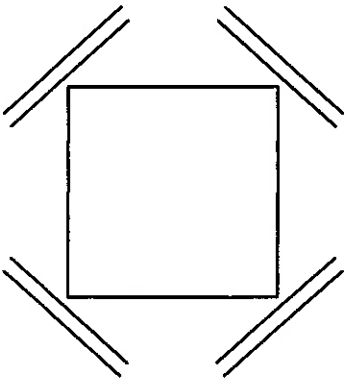


F. CLEAR CONCRETE COVER AND OPENING

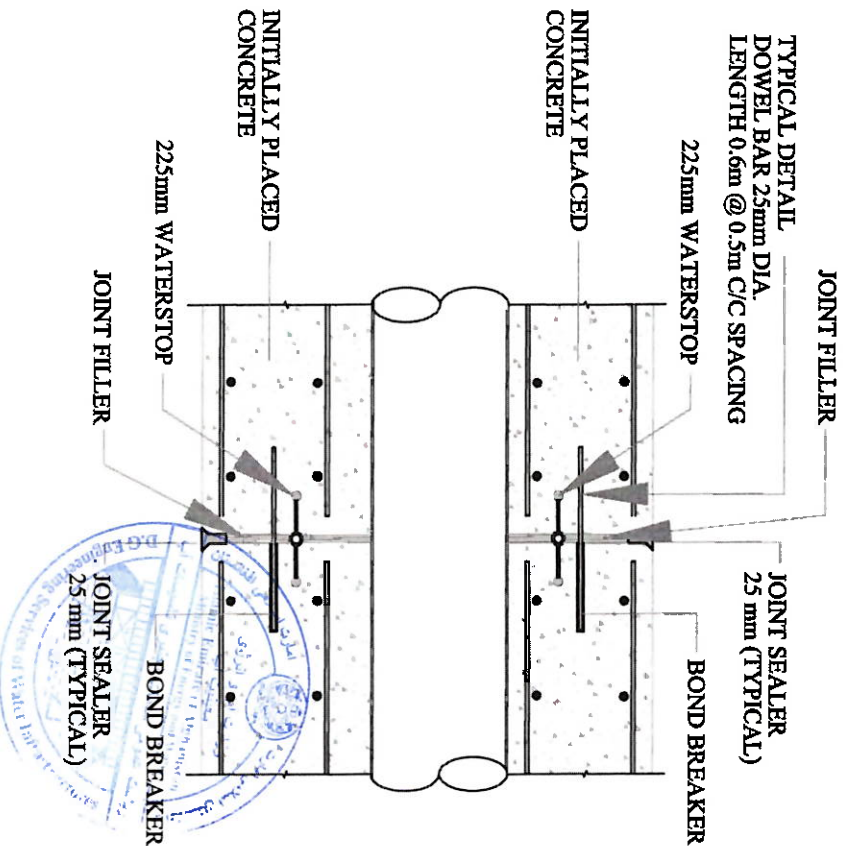
* CLEAR CONCRETE COVER FOR REINFORCING BARS CAST IN PLACE CONCRETE SHALL BE PROVIDED, UNLESS OTHERWISE NOTED.

S.#	TYPE	COVER (mm)
1.	BOTTOM OF FOUNDATIONS AND FOOTINGS	100
2.	BACKFILLED SURFACES, WATER RETAINING SURFACES AND SURFACES SUBJECTED TO SUBMERGENCE OR RAPID FLOW	100
3.	OTHER EXTERIOR WALLS, BEAMS SLABS AND COLUMNS	75 50 40
4.	INTERIOR WALLS, BEAMS, GIRDERS AND COLUMNS	40
5.	INTERIOR WALLS AND SLABS	25

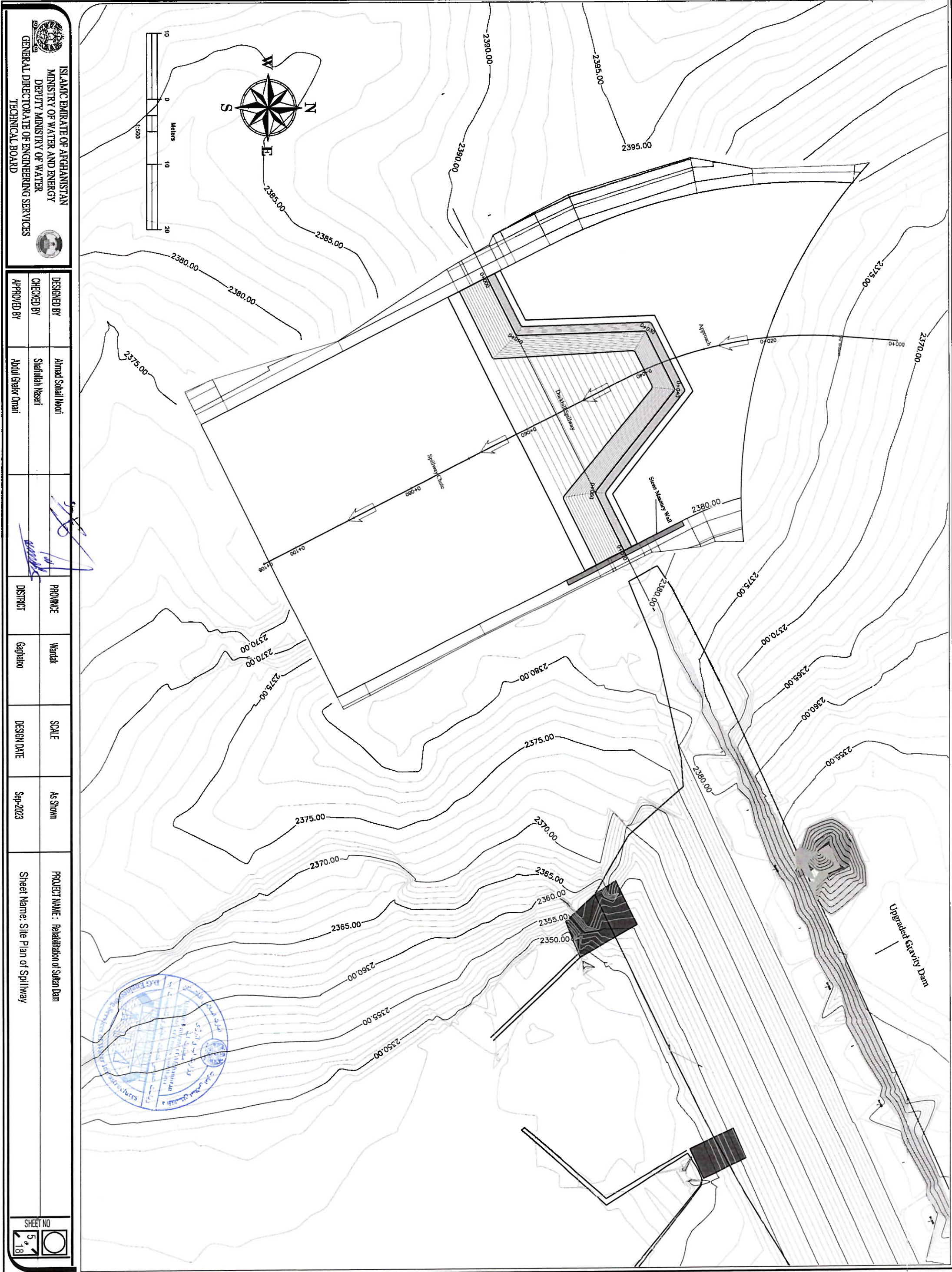
- * IN NO CASE SHALL THE CONCRETE COVER BE LESS THAN 1.5 TIMES THE NOMINAL SIZE OF AGGREGATE, OR 2 TIMES THE MAX. DIAMETER OF REINFORCEMENT .
- * FOR OPENING MINIMUM SIZE 610mm x 610mm OR MORE, PROVIDE 2 Ø16 AS BELOW JOIN AT LEAST DEFORMED BARS AT EACH CORNER IF NOT SPECIFIED.



CONTRACTION JOINT
(PLACED EVERY 9 m)



EXPANSION JOINT
(PLACED EVERY 30 m)



ISLAMIC EMIRATE OF ARGHANISTAN
MINISTRY OF WATER AND ENERGY
DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD

DESIGNED BY: Ahmed Sahai Noori
CHECKED BY: Shafiqullah Nazeri
APPROVED BY: Abdul Ghafor Ghani

PROVINCE: Wardak
DISTRICT: Gajghatoo
SCALE: As Shown
DESIGN DATE: Sep-2023

PROJECT NAME: Rehabilitation of Sultan Dam
Sheet Name: Site Plan of Spillway



DESIGNED BY	Ahmed Shah Noori	PROVINCE	Wardak	SCALE	As Shown	PROJECT NAME : Rehabilitation of Sultan Dam
CHECKED BY	Shahmullah Hazeri	DISTRICT	Gajinabo	DESIGN DATE	Sep-2023	Sheet Name: Excavation Plan
APPROVED BY	Abdul Ghafor Omari					



Alignment Incremental Station Report

Chute Spillway's Center Line

Station	Northing	Easting	Tangential Direction
0+000.00	3,735,416.7401m	442,491.7245m	S4° 38' 14"W
0+020.00	3,735,396.7831m	442,491.0956m	S5° 22' 17"E
0+040.00	3,735,377.4718m	442,495.9855m	S23° 02' 50"E
0+060.00	3,735,359.5478m	442,504.8530m	S26° 42' 01"E
0+080.00	3,735,341.6804m	442,513.8395m	S26° 42' 01"E
0+100.00	3,735,323.8130m	442,522.8260m	S26° 42' 01"E
0+105.53	3,735,318.8738m	442,525.3102m	S26° 42' 01"E

Alignment Incremental Station Report

Chute Spillway's Right Offset (23.5m)

Station	Northing	Easting	Tangential Direction
0+000.00	3,735,418.6400m	442,468.3014m	S4° 38' 14"W
0+020.00	3,735,398.6778m	442,467.4096m	S2° 42' 31"E
0+040.00	3,735,378.9769m	442,470.5988m	S15° 40' 55"E
0+060.00	3,735,360.4713m	442,478.0836m	S26° 42' 01"E
0+080.00	3,735,342.6039m	442,487.0701m	S26° 42' 01"E
0+100.00	3,735,324.7365m	442,496.0566m	S26° 42' 01"E
0+118.38	3,735,308.3164m	442,504.3152m	S26° 42' 01"E

Excavation Plan Coordinates

Point Table				
Point #	Raw Description	Elevation	Northing	Easting
17649	E1	2379.400	3735355.5495	442480.6124
17650	E2	2379.400	3735359.3786	442488.2256
17651	E3	2379.400	3735379.3698	442487.6322
17652	E4	2379.400	3735385.1913	442499.2067
17653	E5	2379.400	3735372.7958	442514.9023
17654	E6	2379.400	3735376.6249	442522.5156
17655	E7	2378.400	3735352.8694	442481.9603
17656	E8	2378.400	3735357.5573	442491.2810
17657	E9	2378.400	3735377.5485	442490.6876
17658	E10	2378.400	3735381.6524	442498.8473
17659	E11	2378.400	3735369.2569	442514.5429
17660	E12	2378.400	3735373.9448	442523.8636
17661	E13	2378.400	3735347.4020	442484.7102
17662	E14	2378.400	3735357.9397	442505.6618
17663	E15	2378.400	3735368.4774	442526.6134

Alignment Incremental Station Report

Spillway Crest Coordinates

Station	Northing	Easting	Tangential Direction
0+000.00	3,735,353.4769m	442,481.6548m	N63° 17' 59"E
0+010.00	3,735,358.0483m	442,490.5256m	N26° 47' 54"E
0+020.00	3,735,368.0355m	442,490.2897m	N1° 42' 01"W
0+030.00	3,735,377.9975m	442,490.1374m	N42° 48' 14"E
0+040.00	3,735,382.3149m	442,499.0865m	S64° 11' 35"E
0+050.00	3,735,376.1270m	442,506.9409m	S51° 42' 01"E
0+060.00	3,735,370.1860m	442,514.8732m	N67° 47' 23"E
0+070.00	3,735,374.5520m	442,523.5574m	N63° 17' 59"E

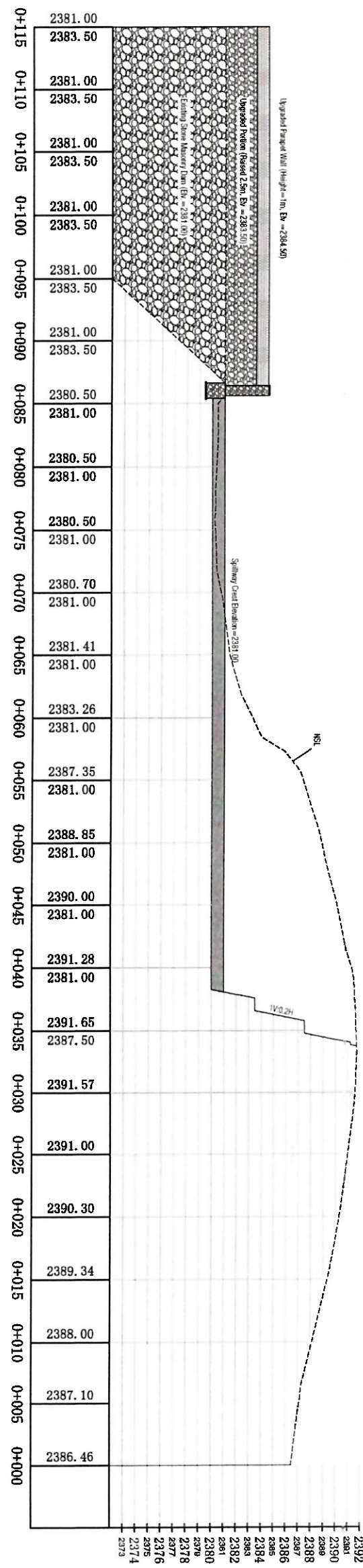
Alignment Incremental Station Report

Chute Spillway's Left Offset (23.5m)

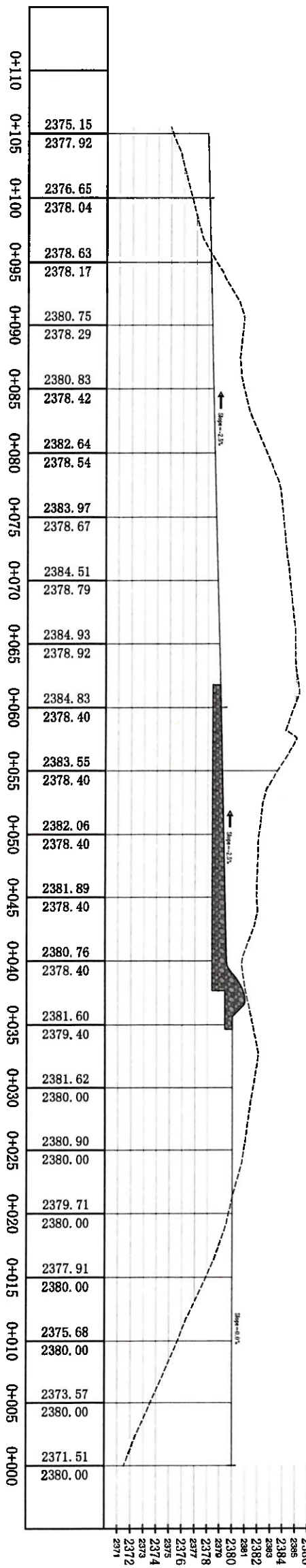
Station	Northing	Easting	Tangential Direction
0+000.00	3,735,414.8402m	442,515.1476m	S4° 38' 14"W
0+020.00	3,735,394.9218m	442,515.0789m	S11° 03' 45"E
0+040.00	3,735,376.4916m	442,522.6360m	S26° 42' 01"E
0+060.00	3,735,358.6243m	442,531.6224m	S26° 42' 01"E
0+080.00	3,735,340.7569m	442,540.6089m	S26° 42' 01"E
0+092.68	3,735,329.4329m	442,546.3044m	S26° 42' 01"E



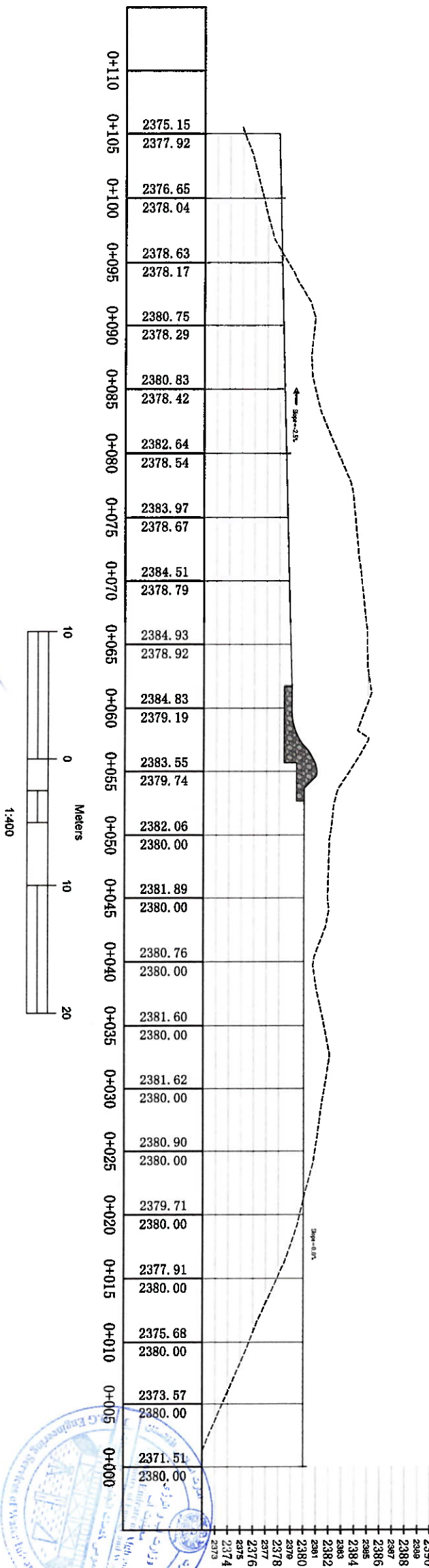
Spillway and Dam PROFILE



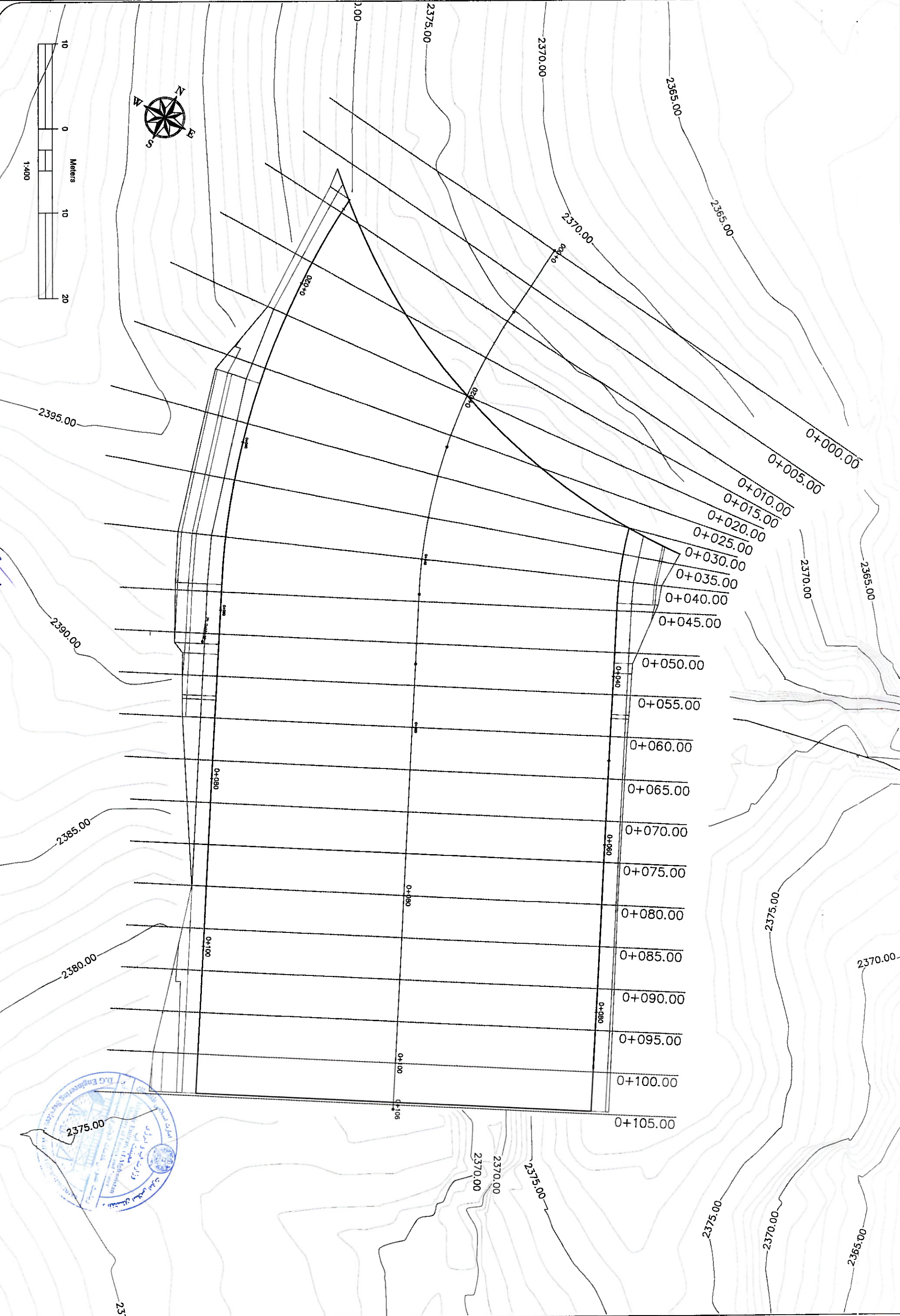
Chute Spillway PROFILE (Duckbill Portion)

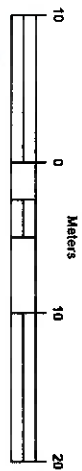
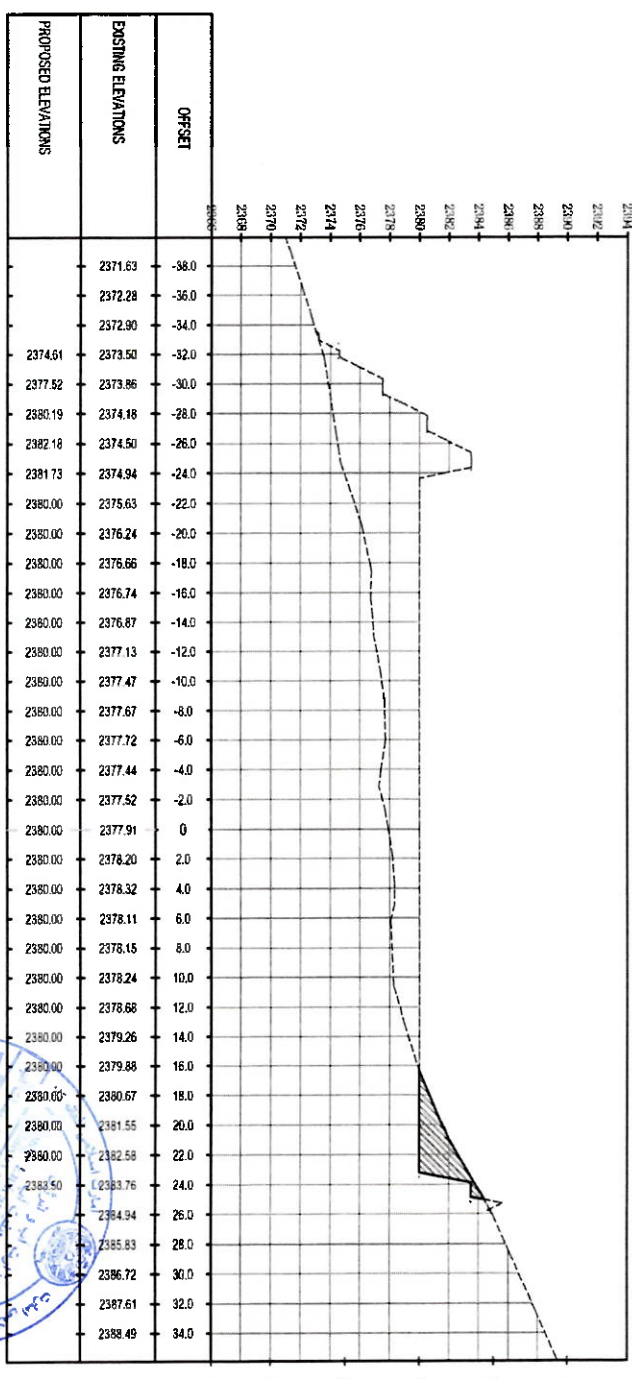
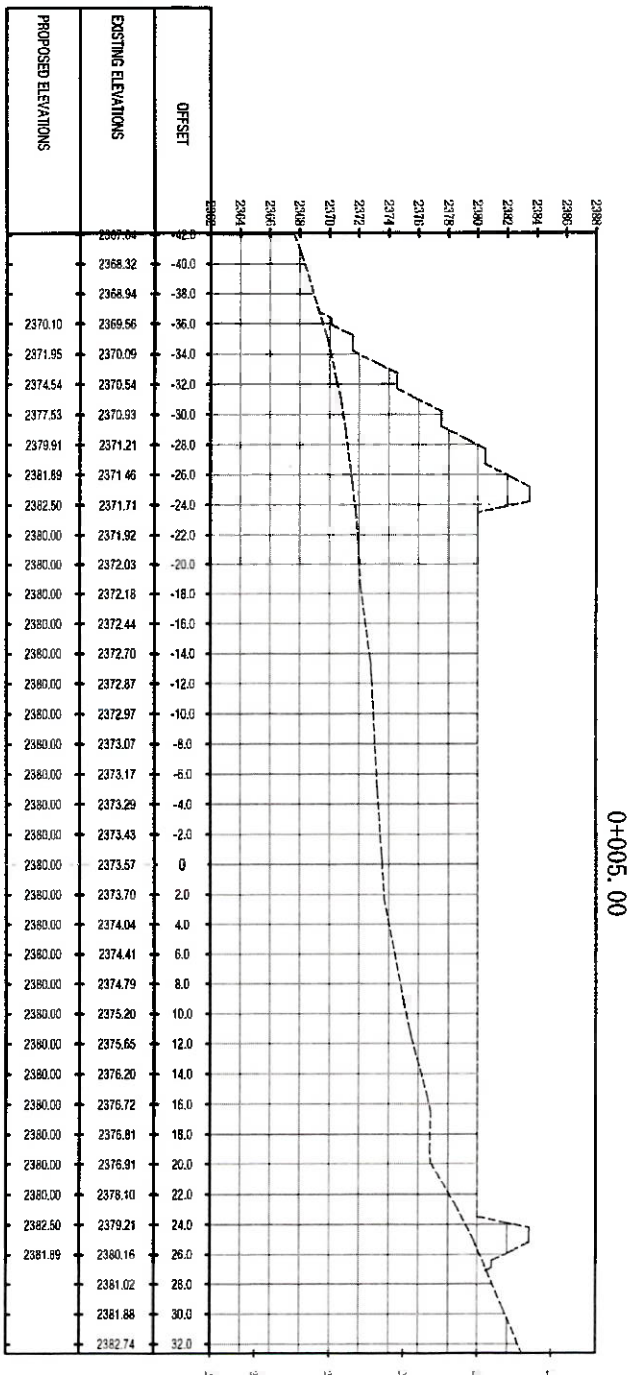
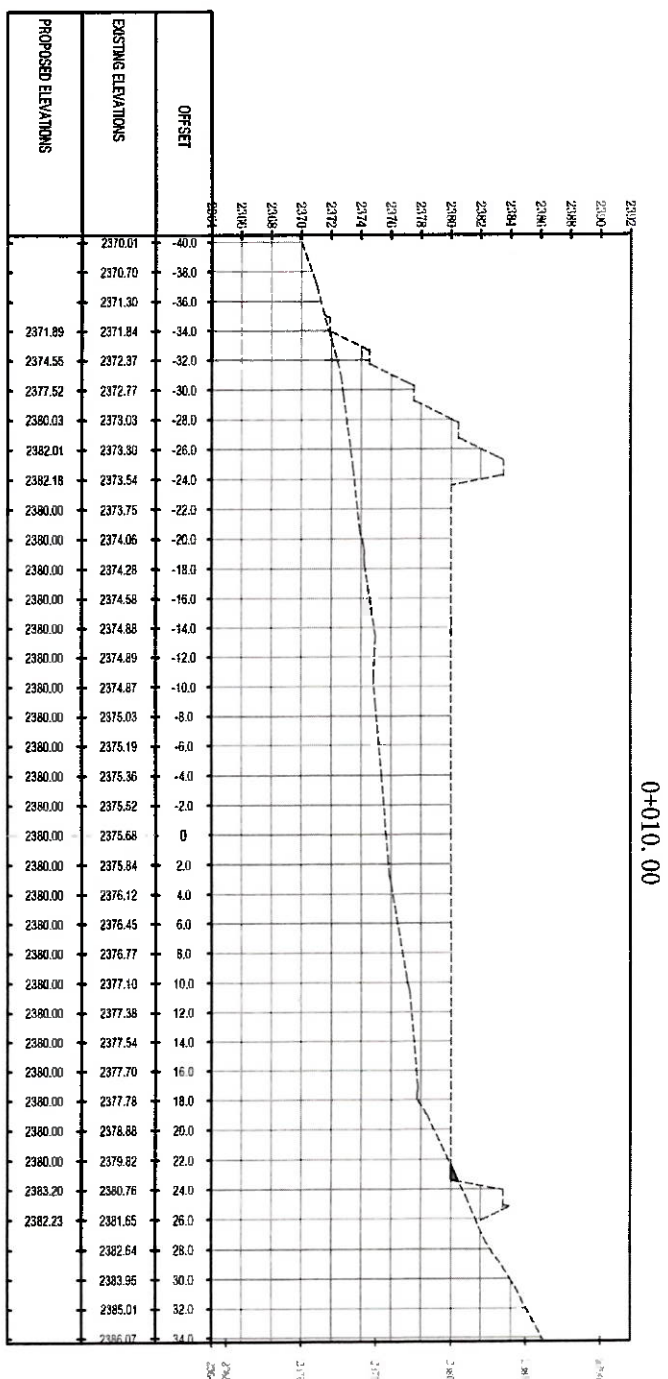
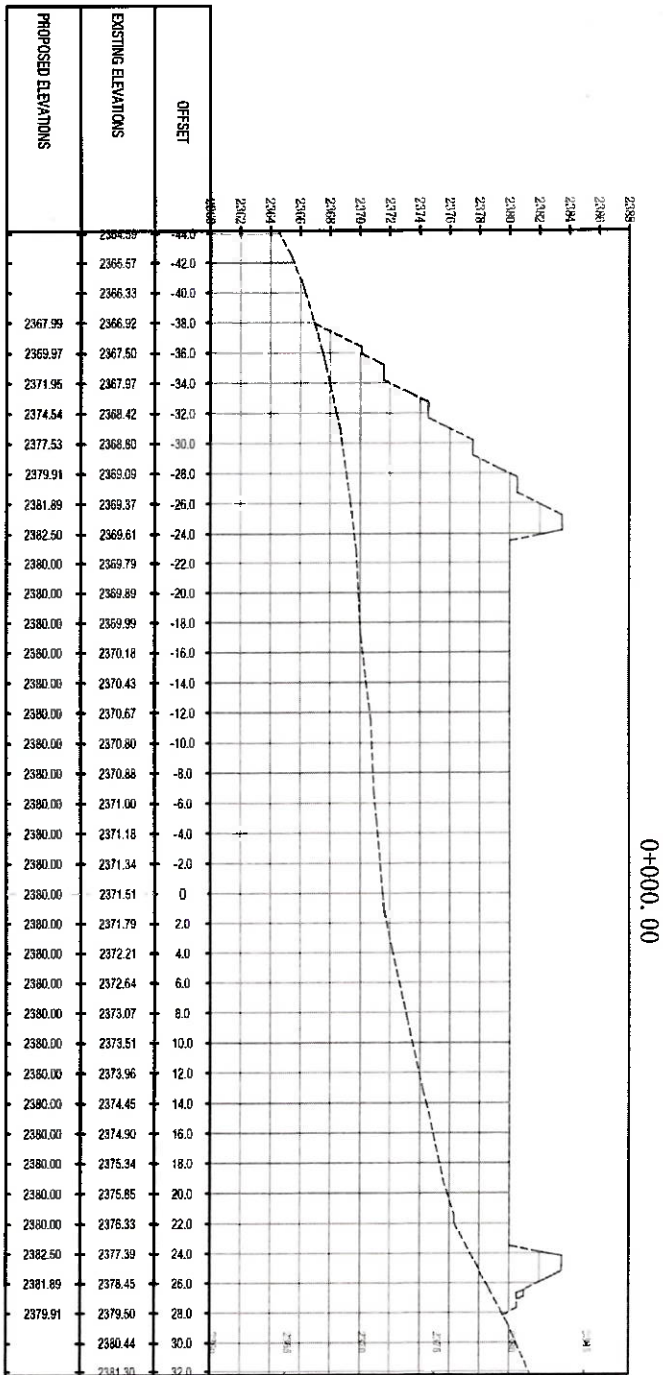


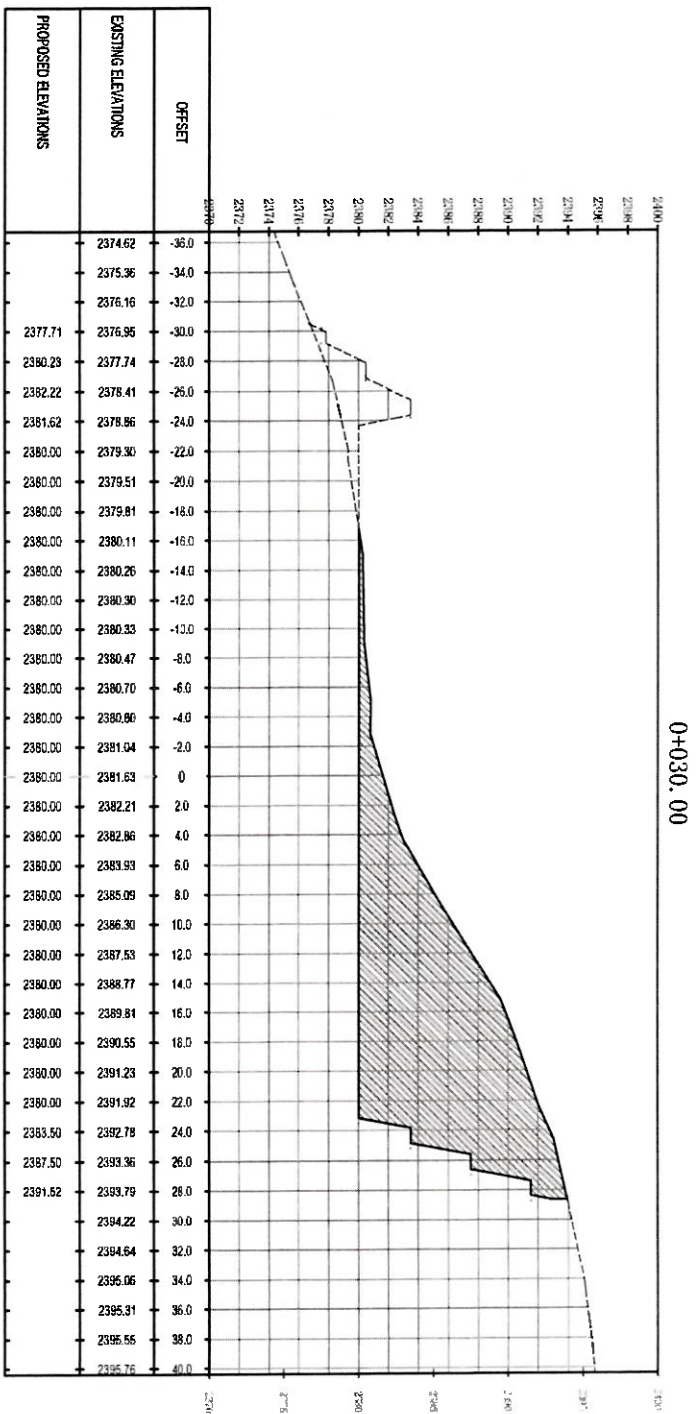
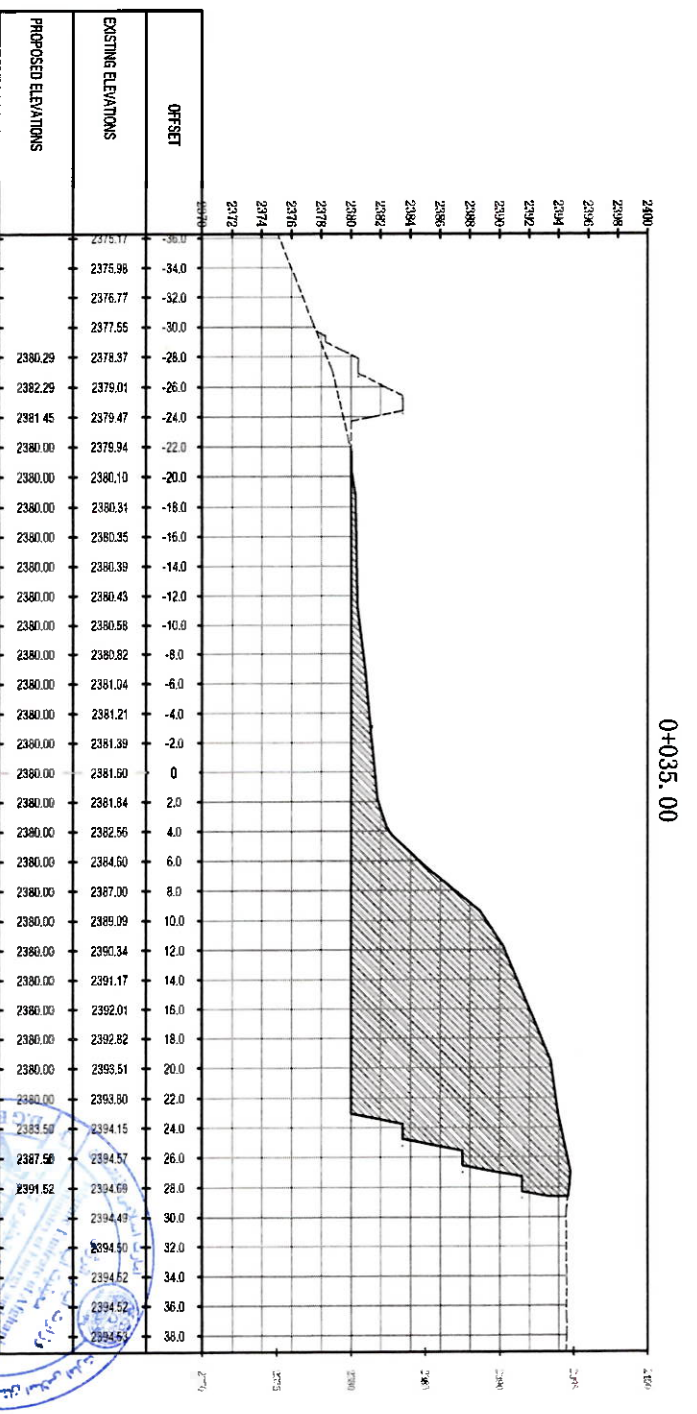
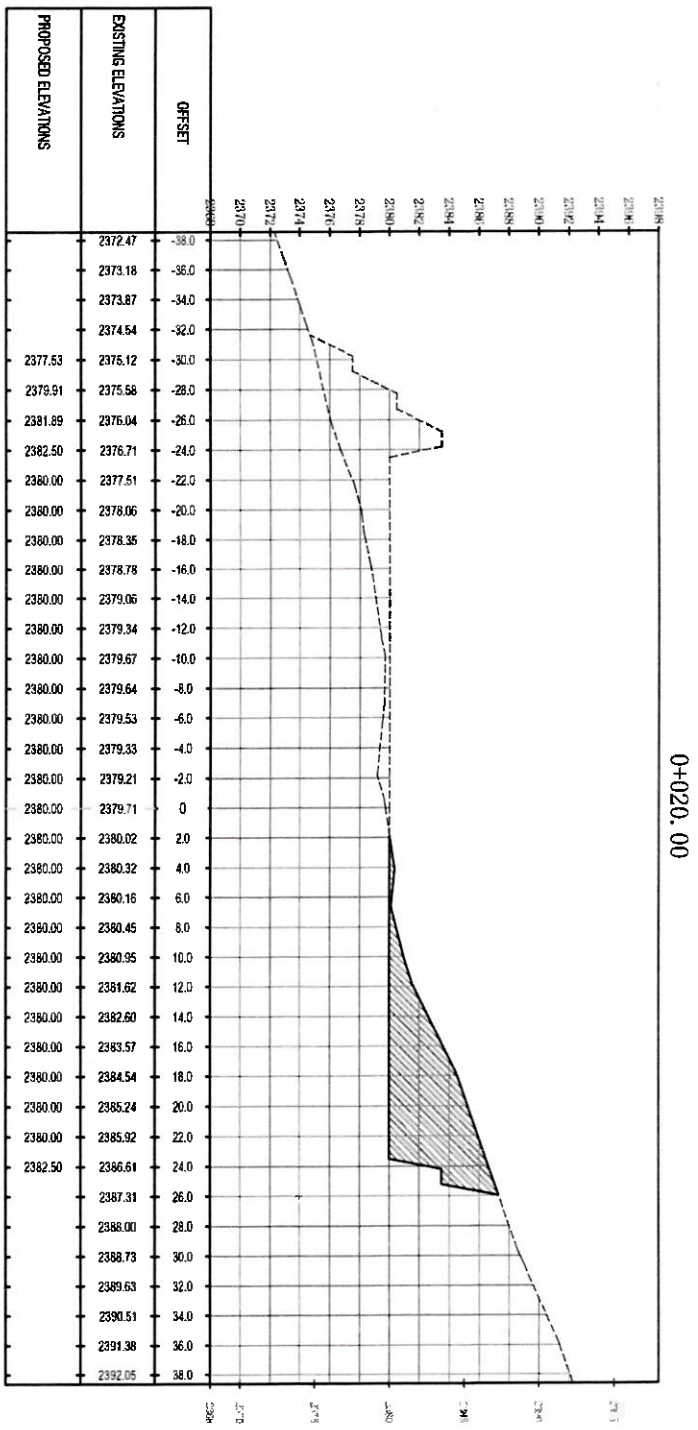
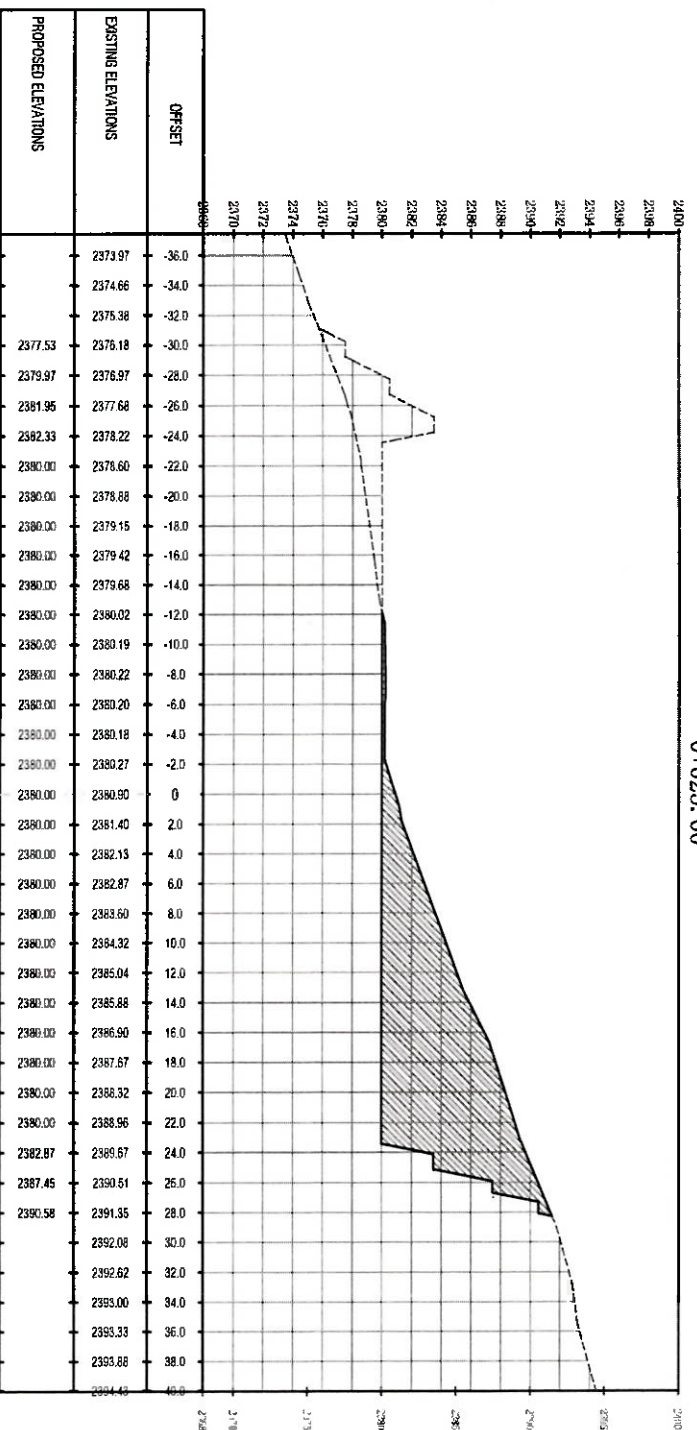
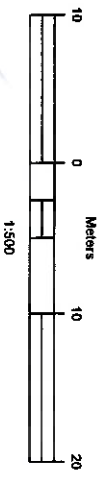
Chute Spillway PROFILE (Straight Portion)



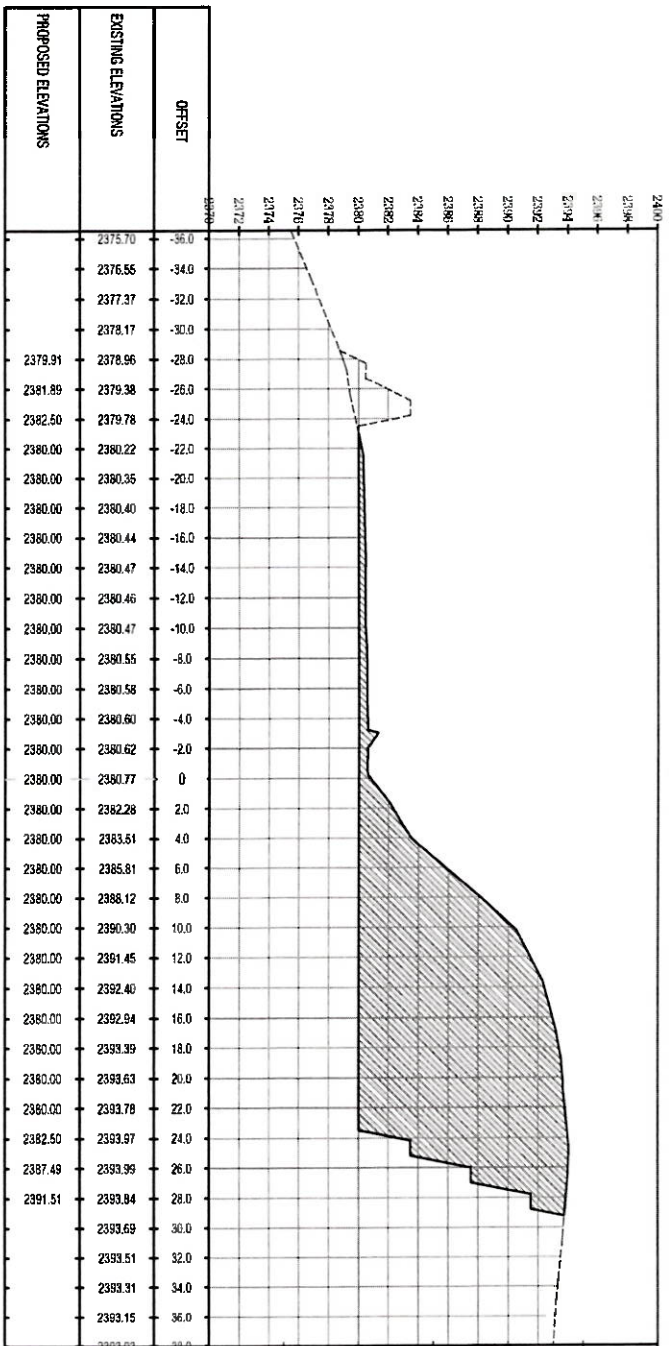
DESIGNED BY	Abdul Sobhan Noori	PROVINCE	Wardak	SCALE	As Shown	PROJECT NAME	Rehabilitation of Sultan Dam
CHECKED BY	Shafiqullah Nasseh	DISTRICT	Gardiz	DESIGN DATE	Sep-2023	SHEET NAME	Chainage Plan
APPROVED BY	Abdul Ghafur Ghani						



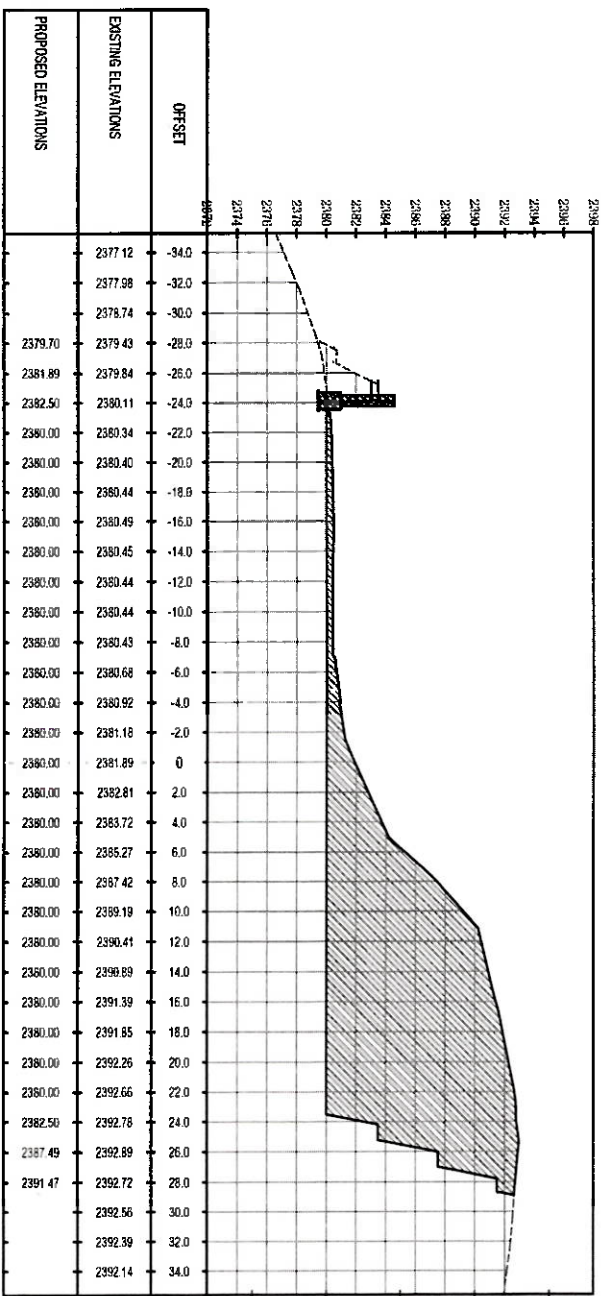




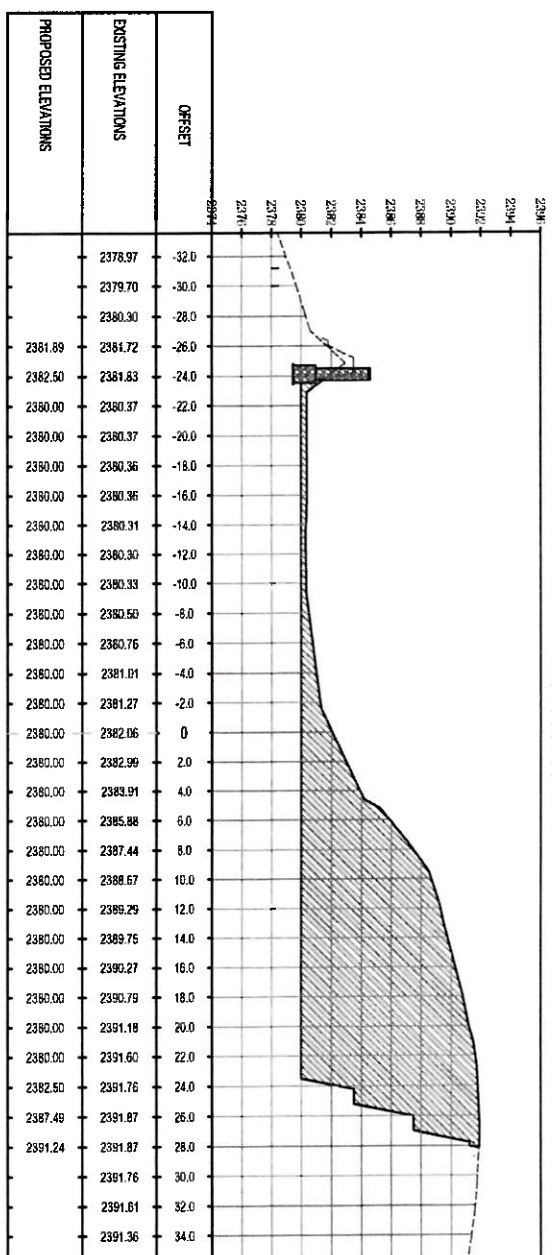
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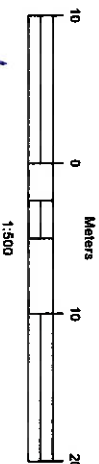
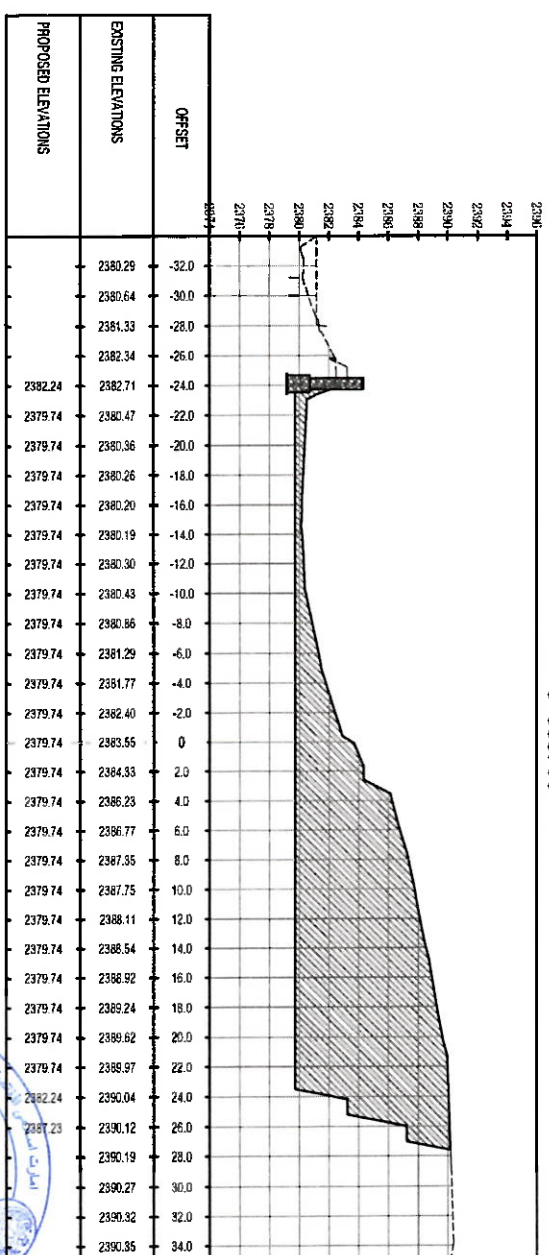
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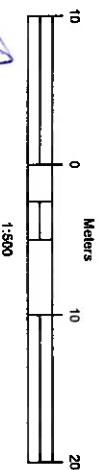
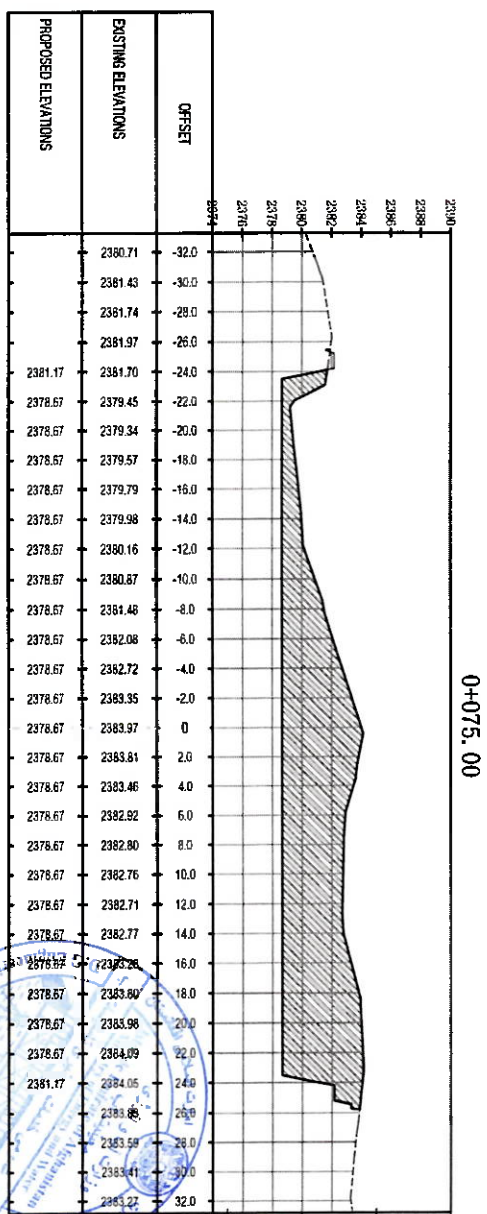
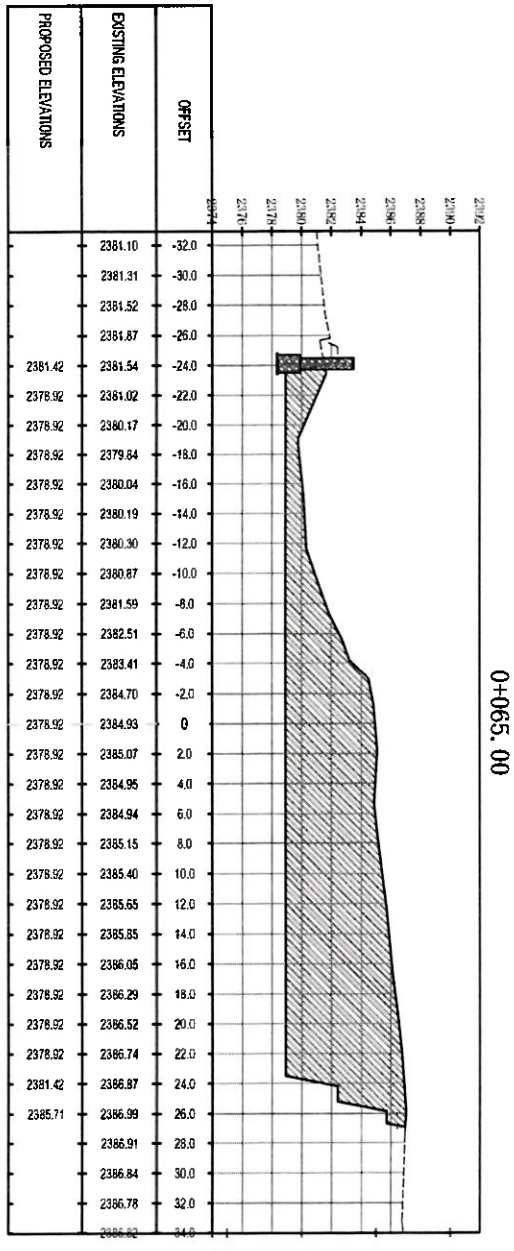
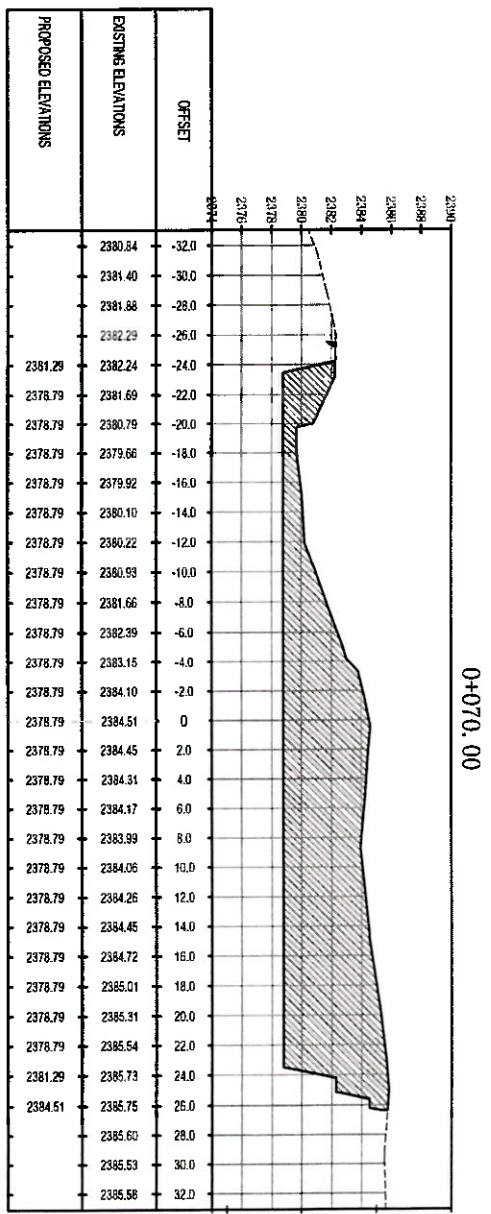
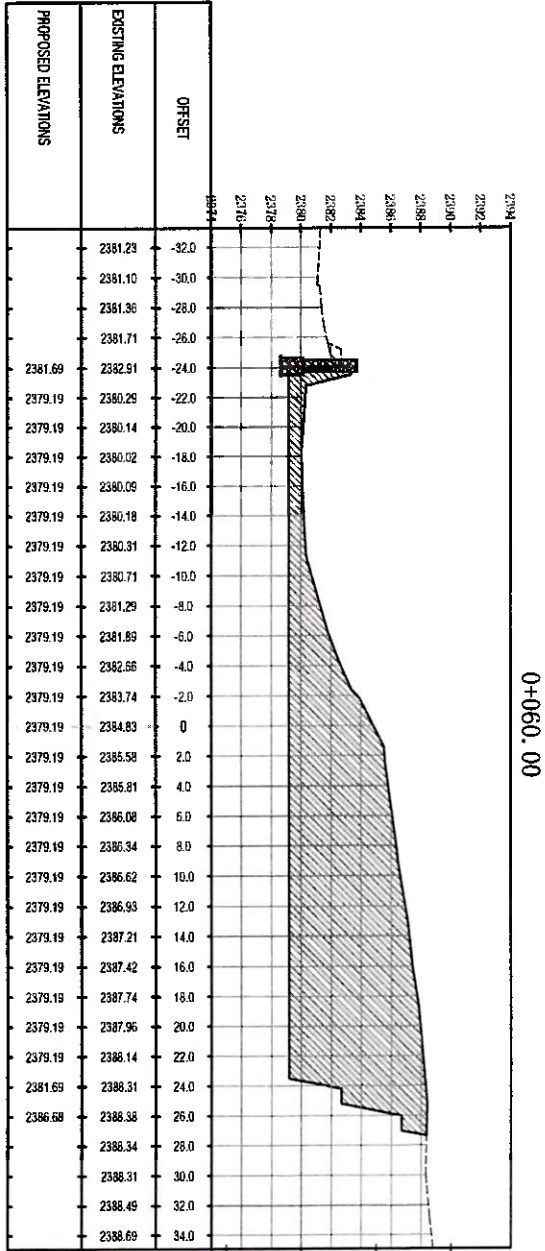


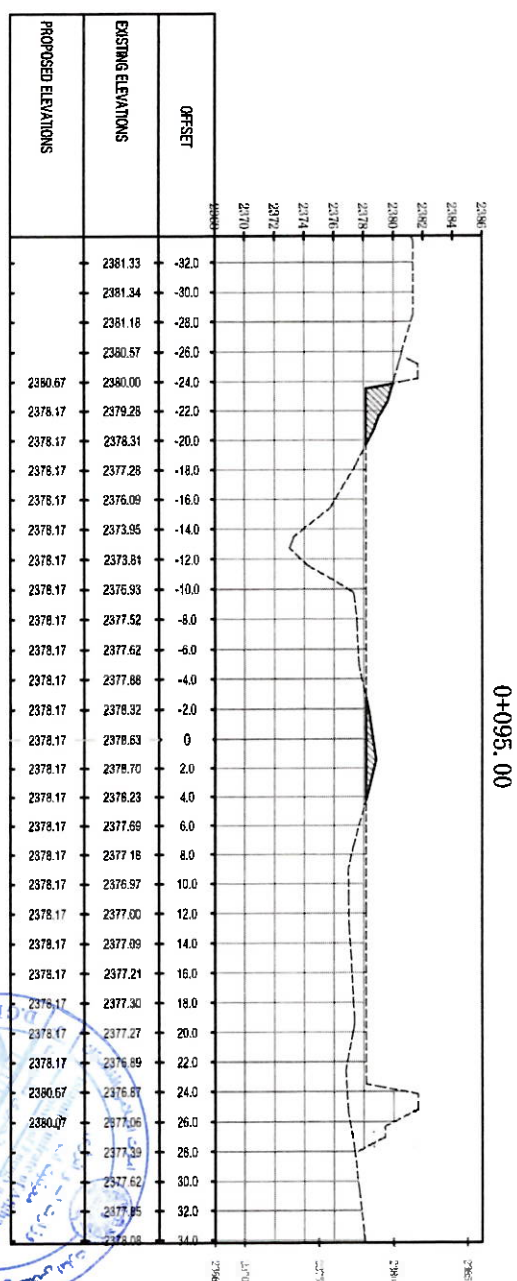
ISLAMIC EMIRATE OF AFGHANISTAN
MINISTRY OF WATER AND ENERGY
DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD

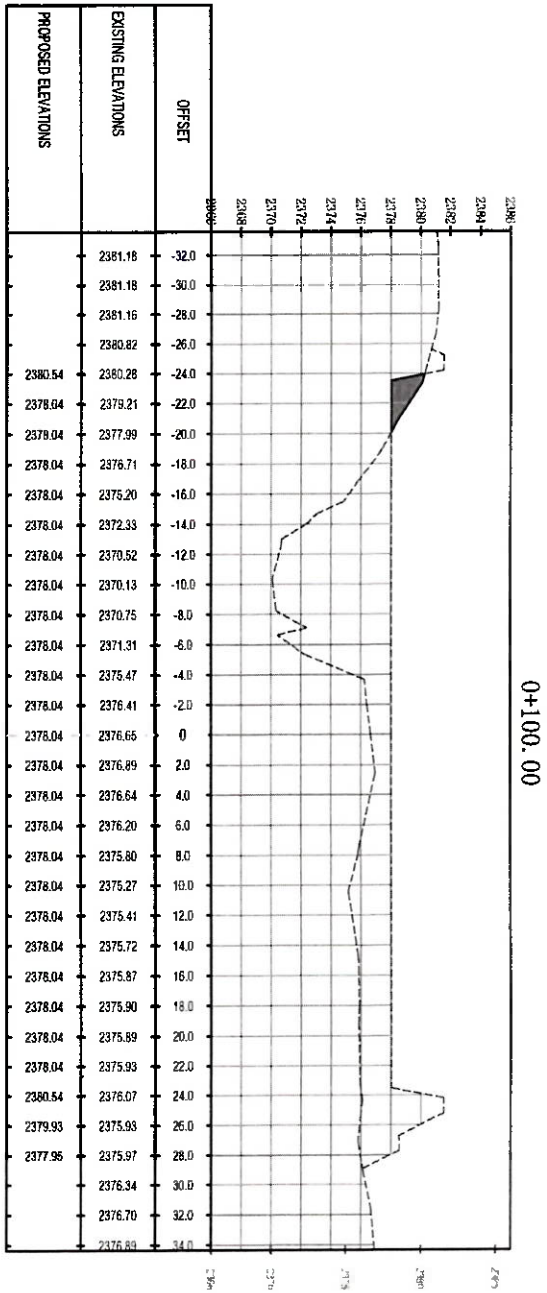
DESIGNED BY: Ahmad Shafiq Noori
CHECKED BY: Shahidullah Nassei
APPROVED BY: Abdul Ghafar Omar

PROVINCE: Wardak
DISTRICT: Gajaluddin
SCALE: As Shown
DESIGN DATE: Sep-2023

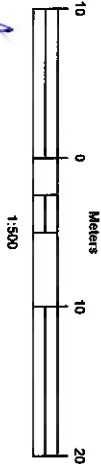
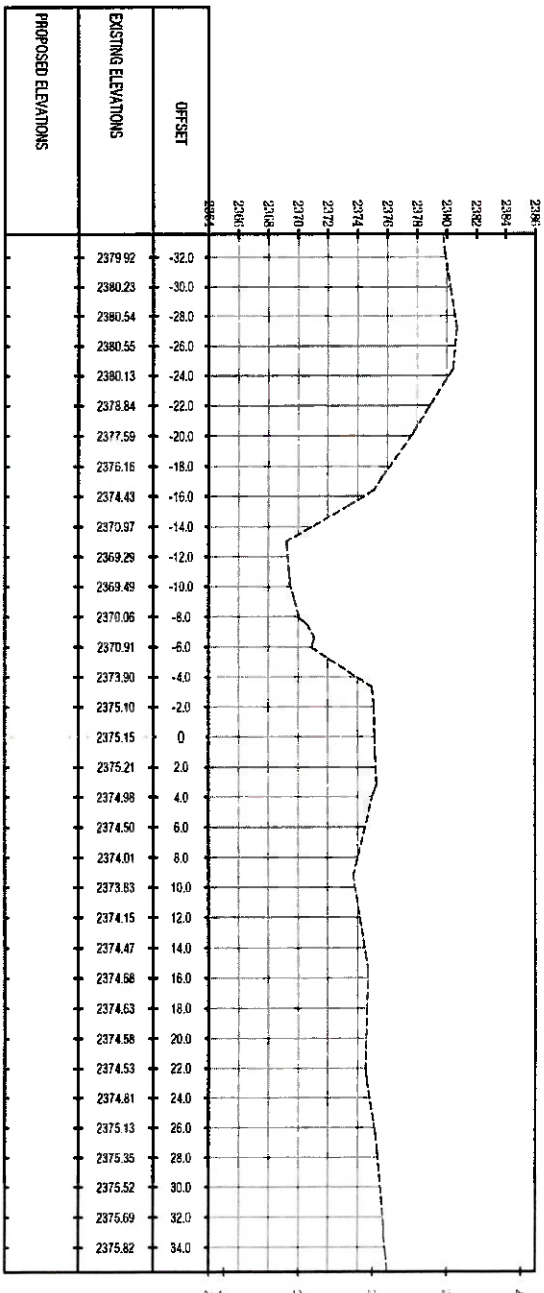
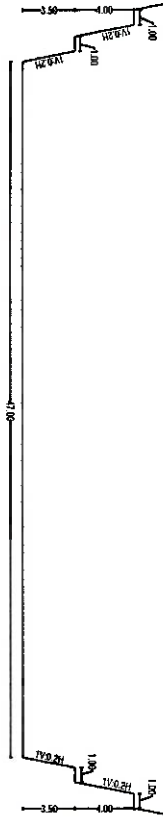
PROJECT NAME: Rehabilitation of Sultan Dam
Sheet Name: Spillway Sections 0+040 to 0+055

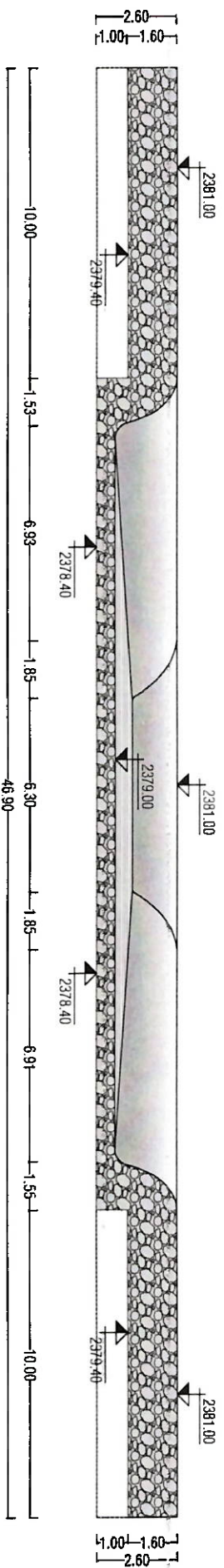




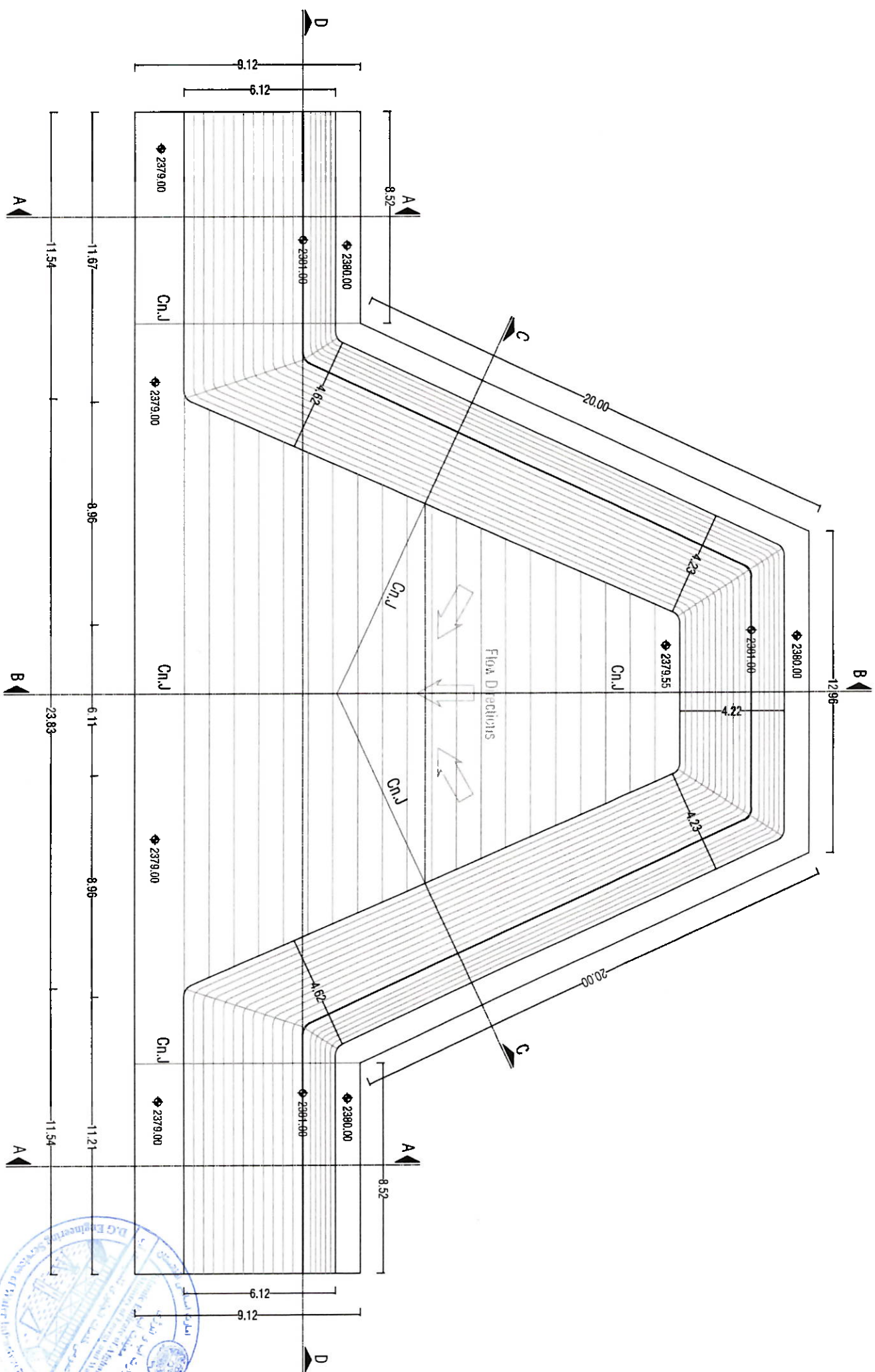


Typical Section of Cutting

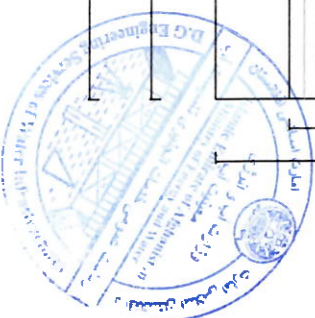
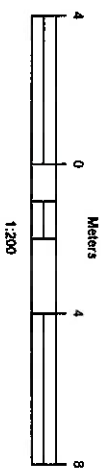


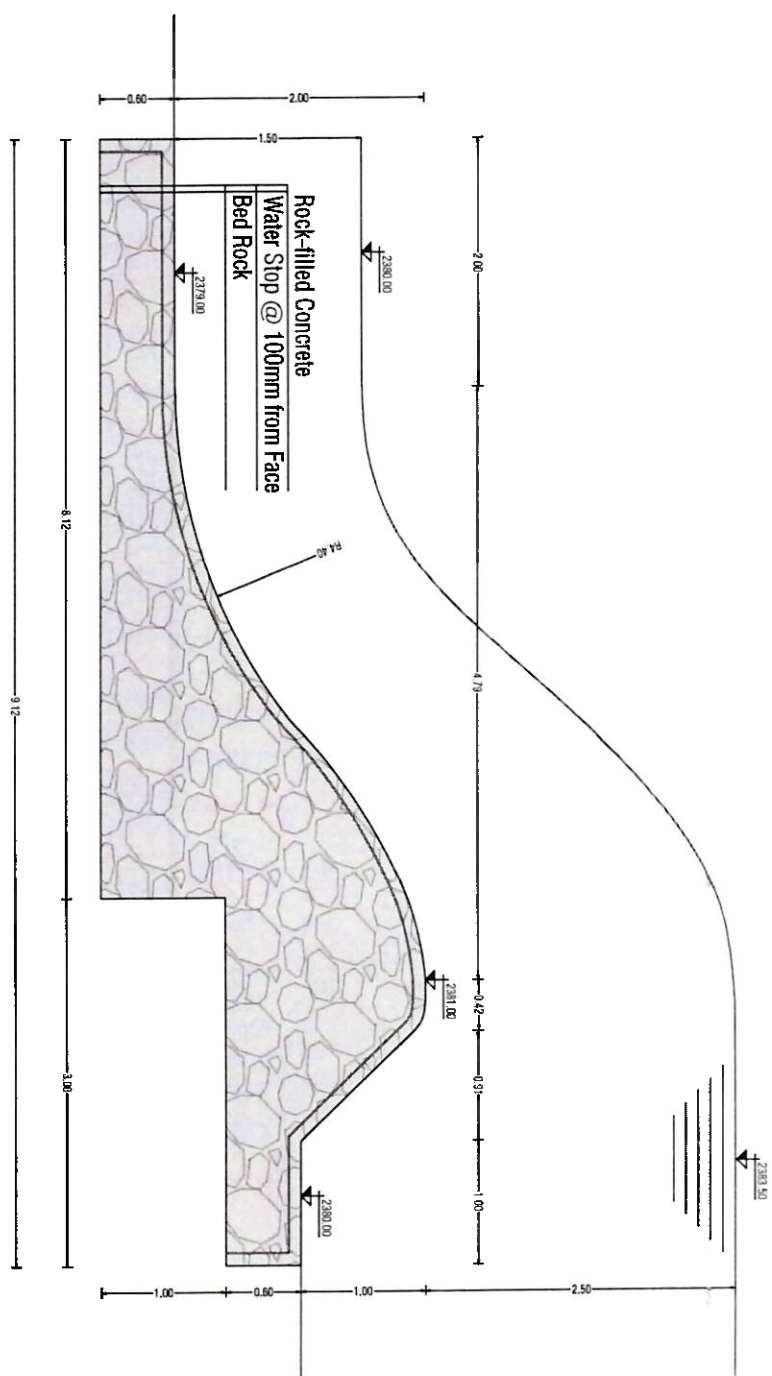
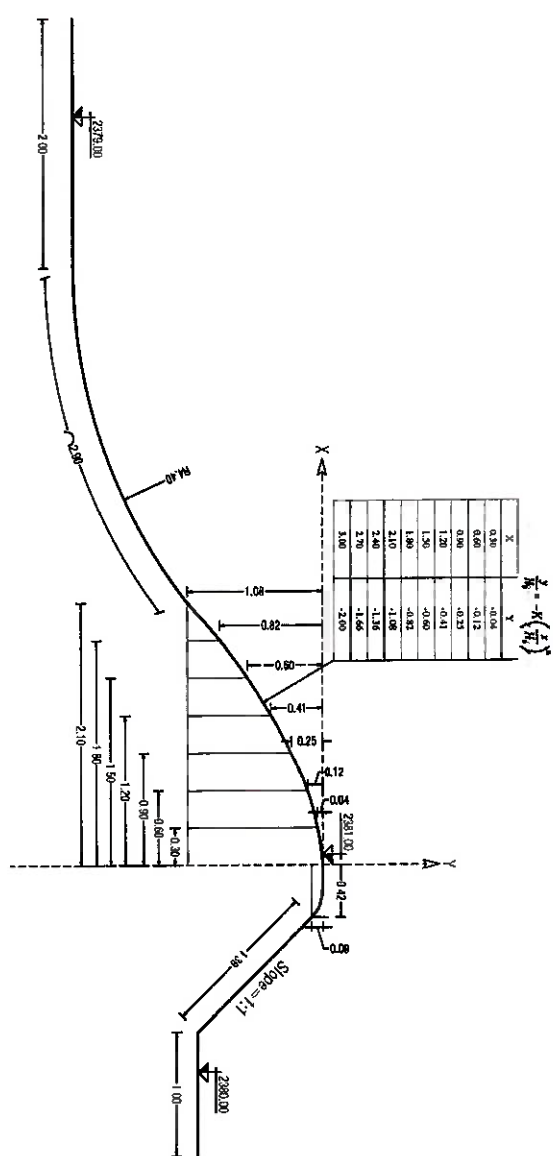


Section D-D (Long Section and Elevation of Duckbill Spillway)

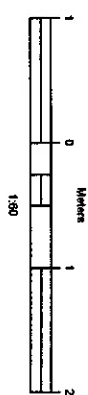


Plan of Duckbill Spillway

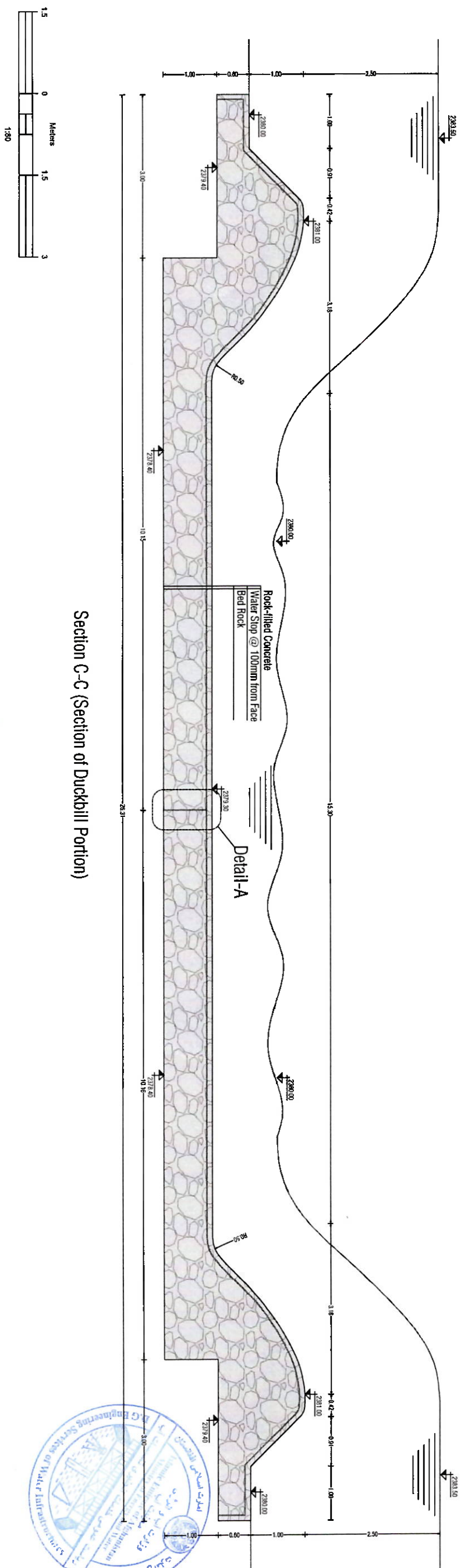
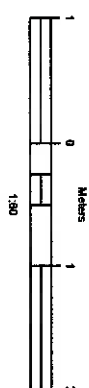




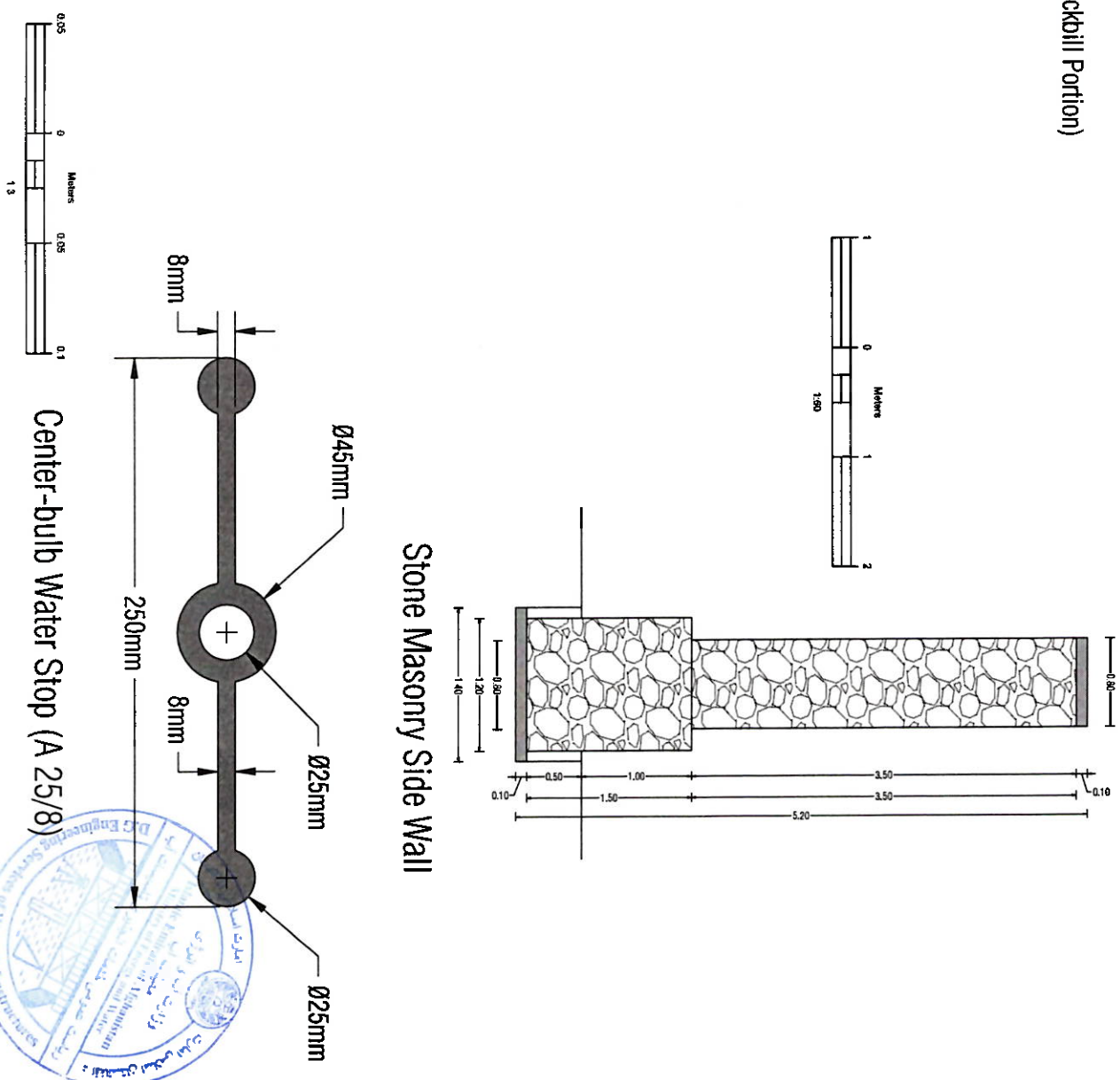
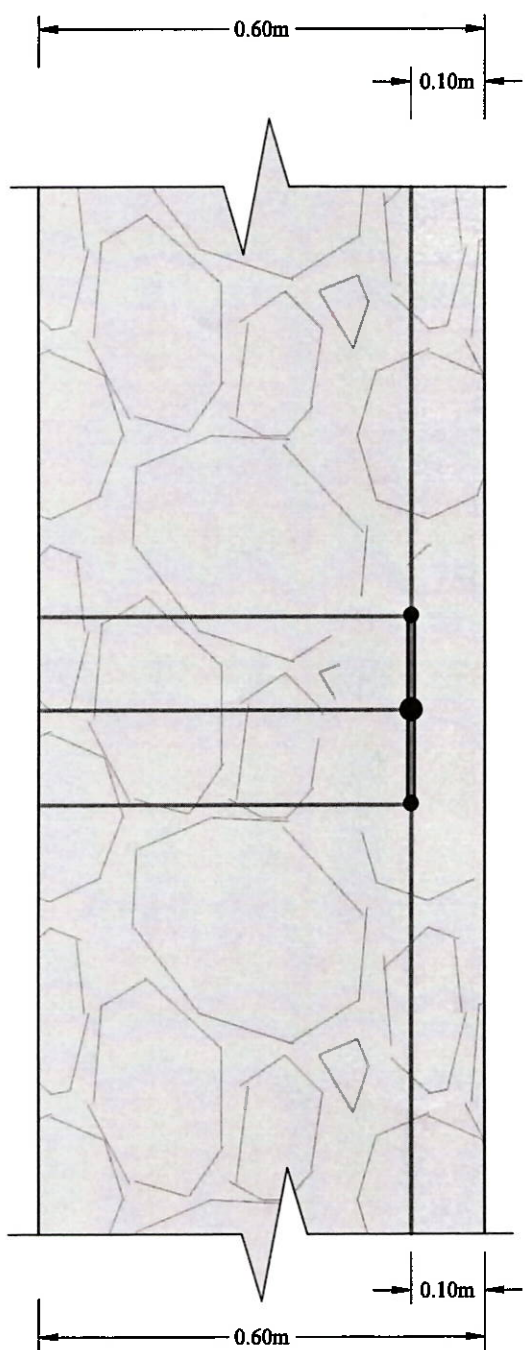
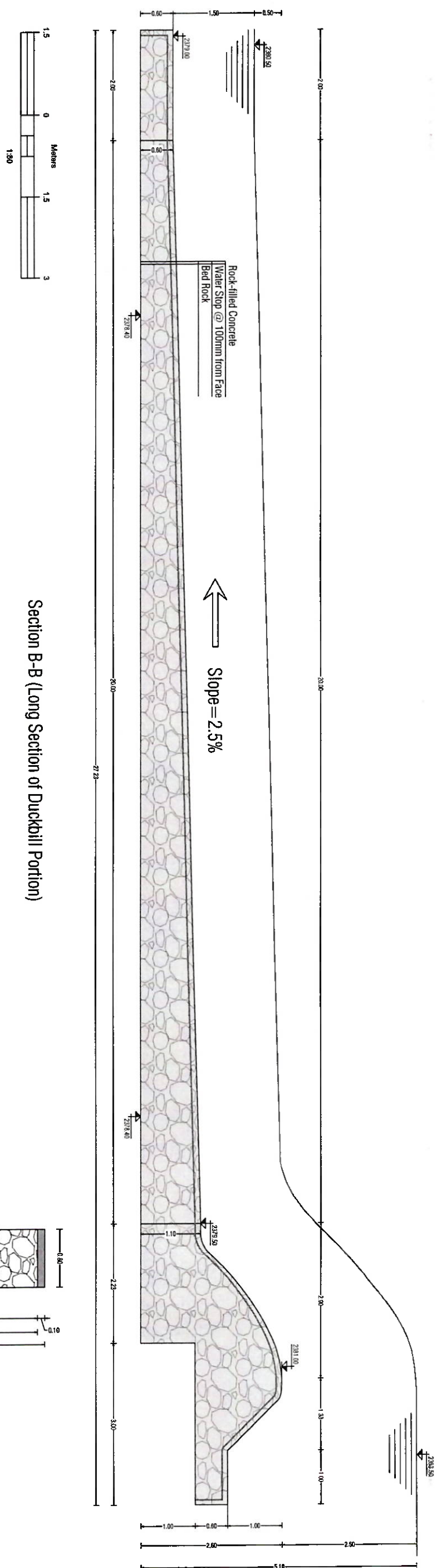
Geometry of Oggee Structure



Section A-A (Cross Section of Straight Portion)



Section C-C (Section of Duckbill Portion)





Islamic Emirate of Afghanistan
Ministry of Water and Energy
Deputy Ministry of Water



General Directorate of Engineering Services of Water Infrastructures

Technical Board



Rehabilitation of Sultan Dam			
Date		Oct 2023	
Prepared	Checked	Approved	
M. Tariq Tasal		Abdul Ghafoor Omar	Stability & stress analysis report using CADAM 2D



Table of Contents

Stability and Stress Analysis of Sultan dam	2
1. Introduction	2
2. Stability and stress analysis of the Sultan dam	3
2.1 Geometric dimensions of the dam	3
2.2 Material Properties	4
2.3 Earthquake Characteristics of the site	5
2.4 Loads applied on Dam	6
2.5 Analysis Result	17
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3. Conclusion	30

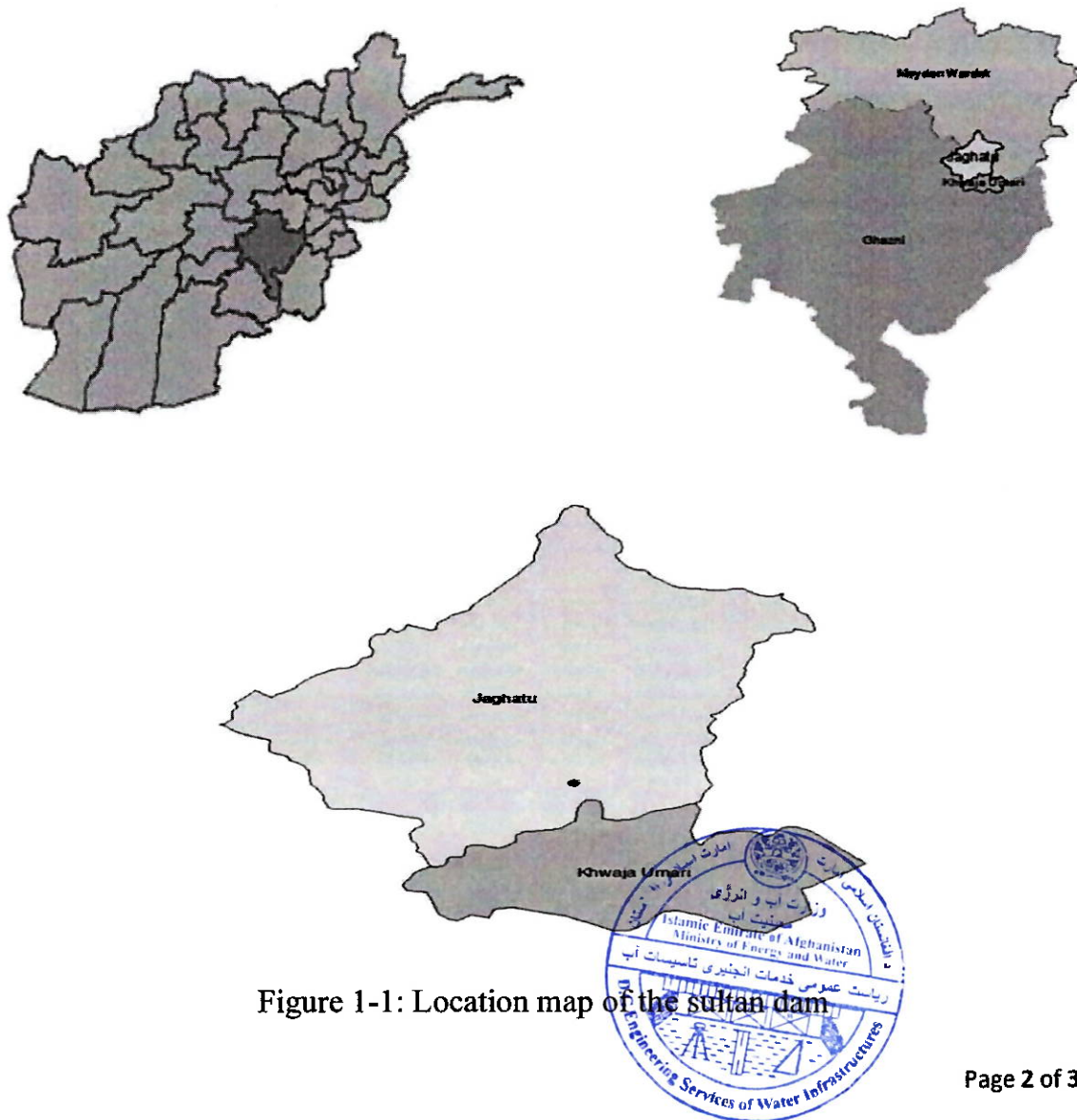


Stability and Stress Analysis of Sultan dam

1. Introduction

The Ministry of Energy and Water (MEW) in the Islamic Emirate of Afghanistan is currently rehabilitating and constructing new small scale water resources infrastructure in the provinces, through the technical design team. One of the projects to rehabilitate by MEW is Sultan dam.

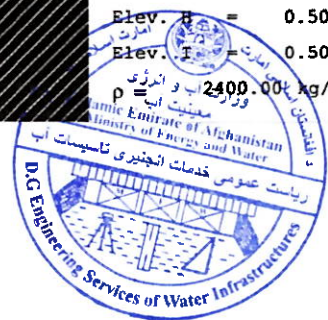
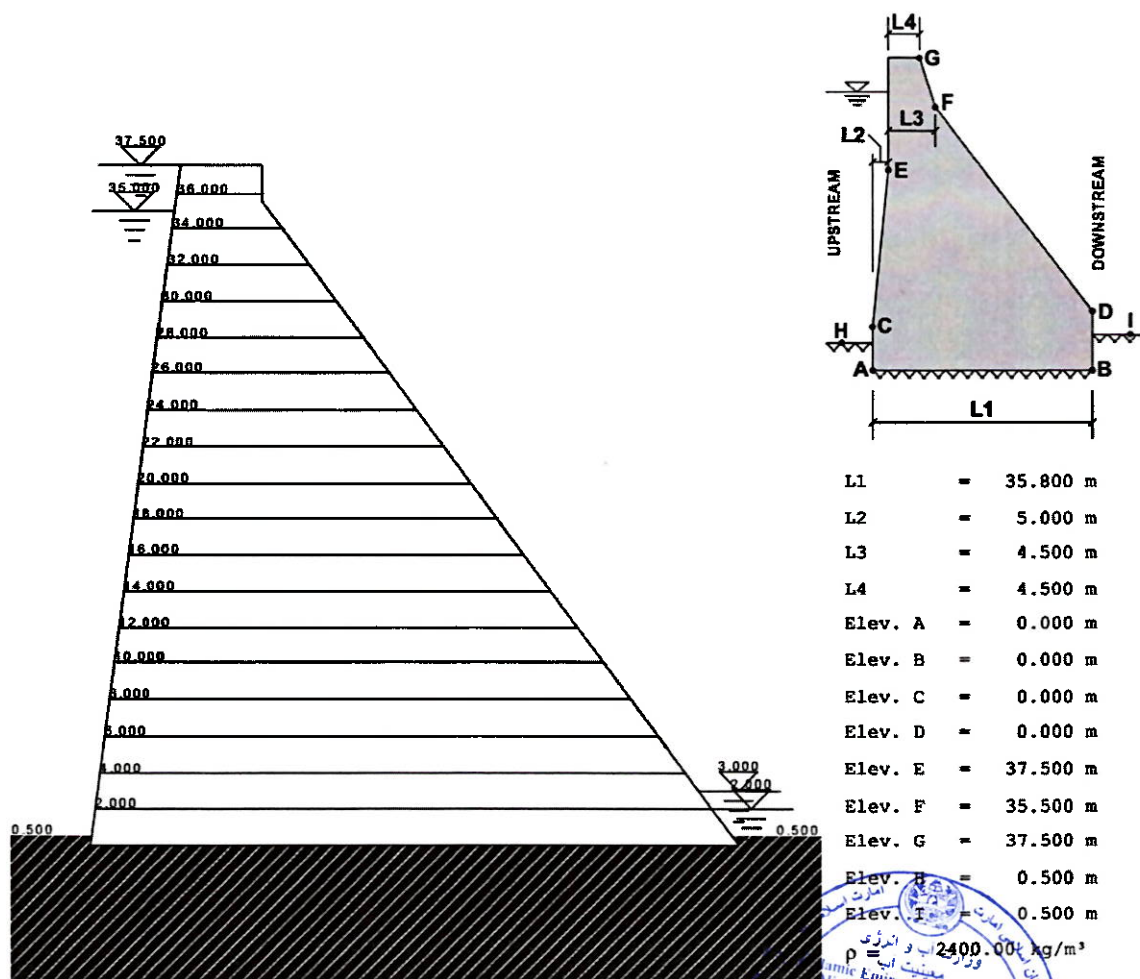
Sultan dam Project is located near Kala Manar village, Jaghato district of Maidan Wardak Province. Latitude and Longitude of the Project Site are 33.757348°N and 68.380705°E. The distance from Ghazni city to the proposed dam site is about 24 km. This project would provide water for irrigation use, sub-ground water recharge, and climate change. Location map of the project shown in (figure 1-1).



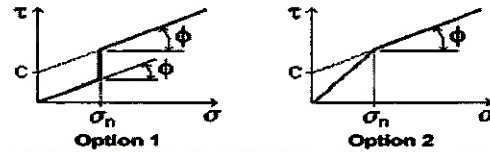
2. Stability and stress analysis of the Sultan dam

Stability and stress analysis are the most important elements that require rigorous consideration in design of a dam structure. Stability of dams against sliding and overturning is crucial due to the substantial horizontal load that requires sufficient and safe resistance to develop by mobilization of adequate shearing forces along the base of the dam foundation. In the current report, a software (CADAM 2d) was used to analyze stresses in the body of the dam and foundation and also the stability of the dam.

2.1 Geometric dimensions of the dam



2.2 Material Properties



Material name	Concrete strength		Shear strength			
	f_c (kPa)	f_t (kPa)	Cohesion (kPa)	Angle (ϕ) (deg)	σ_n (kPa)	Curve type (kPa)
concrete	30000.0	0.000	Peak: 0.000	55.000	0.000	Option 2
			Residual: 0.000	45.000	0.000	Option 2
Base joint	30000.0	3000.000	Peak: 0.000	55.000	0.000	Option 2
			Residual: 0.000	45.000	0.000	Option 2

Rock Passive Shear Strength Properties:

Consideration of passive shear strength	Unit mass (kg/m ³)	Cohesion (kPa)	Friction angle (deg)	Failure plane angle (deg)
No	2400.000	0.000	30.000	30.000

Lift Joint(s):

ID	Material name	Upstream end		Downstream end		Length (m)	Inertia (m ⁴)
		Elevation (m)	Position x (m)	Elevation (m)	Position x (m)		
1	concrete	36.000	4.800	36.000	9.500	4.700	8.6519167
2	concrete	34.000	4.533	34.000	10.611	6.078	18.710559
3	concrete	32.000	4.267	32.000	12.093	7.826	39.947237
4	concrete	30.000	4.000	30.000	13.575	9.575	73.145429
5	concrete	28.000	3.733	28.000	15.056	11.323	120.97728
6	concrete	26.000	3.467	26.000	16.538	13.071	186.11494
7	concrete	24.000	3.200	24.000	18.020	14.820	271.23055
8	concrete	22.000	2.933	22.000	19.501	16.568	378.99626
9	concrete	20.000	2.667	20.000	20.983	18.316	512.08421
10	concrete	18.000	2.400	18.000	22.465	20.065	673.16655
11	concrete	16.000	2.133	16.000	23.946	21.813	864.91543
12	concrete	14.000	1.867	14.000	25.428	23.562	1090.0030
13	concrete	12.000	1.600	12.000	26.910	25.310	1351.1014
14	concrete	10.000	1.333	10.000	28.392	27.058	1650.8828
15	concrete	8.000	1.067	8.000	29.873	28.807	1992.0192
16	concrete	6.000	0.800	6.000	31.355	30.555	2377.1830
17	concrete	4.000	0.533	4.000	32.837	32.303	2809.0462
18	concrete	2.000	0.267	2.000	34.318	34.052	3290.2809
19	Base joint	0.000	0.000	0.000	35.800	35.800	3823.5593

Reservoirs, Ice & Silt:

Water volumetric weight		Crest overtopping pressure		Ice	
$\gamma =$	9.810 kN/m ³	Upstream pressure =	100.00%	Load =	0.000 kN
Operating reservoir elevations		Downstream pressure =	50.00%	Thickness =	0.000 m
Upstream	Downstream	Silt		Elevation =	35.000 m
35.000 m	2.000 m	Elev. =	0.500 m	Load =	0.000 kN
Flood reservoir elevations		$\gamma' =$	7.000 kN/m ³	Apply at elev. =	-1.000 m
Upstream	Downstream	$\phi =$	15.000 deg	Max. elevation =	2.000 m
37.500 m	3.000 m	Assumption: At rest		Floating debris	

Uplift pressures:

Uplift pressures are considered as an external load (linearisation of effective stresses)



2.3 Earthquake Characteristics of the site

Pseudo-Static (seismic coefficient):

Earthquake return period = 2500 years

Ground accelerations:

Horizontal Peak Ground Acceleration (HPGA) =	0.300 g
Vertical Peak Ground Acceleration (VPGA) =	0.200 g
Horizontal Sustained Acceleration (HSA) =	0.150 g
Vertical Sustained Acceleration (VSA) =	0.100 g

Apply correction for water compressibility: No

Earthquake accelerogram period (te) = 1.000 sec

Westergaard correction for inclined surfaces: Generalized Westergaard

Reservoir depth where pressure remains constant 0000.000 m

Pseudo-Dynamic (spectral method):

Earthquake return period = 2500 years

Ground accelerations (for higher modes and vertical response):

Horizontal Peak Ground Acceleration (HPGA) =	0.3000 g
Vertical Peak Ground Acceleration (VPGA) =	0.2000 g
Horizontal Sustained Acceleration (HSA) =	0.1500 g
Vertical Sustained Acceleration (VSA) =	0.1000 g

Spectral accelerations (for 1st mode of vibration in horizontal direction):

Horizontal Peak Spectral Acceleration (HPSA) =	0.3200 g
Horizontal Sustained Spectral Acceleration (HSSA) =	0.1700 g
Horizontal period of vibration (dam-foundation-reservoir) =	0000.000 sec
Damping (dam-foundation-reservoir) =	0000.000 of critical

Dam:

Dam divisions for analysis =	201
Dam damping on rigid foundation without reservoir =	0.0500 of critical
Concrete Young's modulus (dynamic) =	27400.0 MPa

Reservoir:

Wave reflection coefficient for reservoir bottom materials =	0.500
Velocity of pressure waves in water =	1440.000 m/sec

Foundation:

Foundation constant hysteretic damping =	0.100
Foundation Young's modulus (dynamic) =	27400.0 MPa

Modal combination:

SRSS combination



Dam Self-Weight (D) & Added Mass(es) Weight (Mv):

Lift joints		Dam			Added mass(es)	
ID	U/S Elevation (m)	Weight (D) (kN)	position (x-axis) (m)	Elevation (m)	Weight (Mv) (kN)	position (x-axis) (m)
1	36.000	-162.454	7.200	36.745	0.000	0.000
2	34.000	-409.668	7.263	35.663	0.000	0.000
3	32.000	-737.029	7.541	34.461	0.000	0.000
4	30.000	-1146.717	7.881	33.213	0.000	0.000
5	28.000	-1638.731	8.247	31.940	0.000	0.000
6	26.000	-2213.072	8.626	30.651	0.000	0.000
7	24.000	-2869.740	9.012	29.353	0.000	0.000
8	22.000	-3608.734	9.402	28.049	0.000	0.000
9	20.000	-4430.055	9.796	26.739	0.000	0.000
10	18.000	-5333.703	10.192	25.425	0.000	0.000
11	16.000	-6319.677	10.590	24.108	0.000	0.000
12	14.000	-7387.977	10.988	22.789	0.000	0.000
13	12.000	-8538.605	11.388	21.469	0.000	0.000
14	10.000	-9771.559	11.789	20.146	0.000	0.000
15	8.000	-11086.839	12.190	18.823	0.000	0.000
16	6.000	-12484.446	12.591	17.498	0.000	0.000
17	4.000	-13964.380	12.993	16.173	0.000	0.000
18	2.000	-15526.641	13.395	14.846	0.000	0.000
19	Base	-17171.228	13.798	13.519	0.000	0.000

Upstream Hydrostatic Forces - Operating level (Hnu & Vnu):

Reservoir normal operating elevation (upstream side) = 35.000 m

Lift joints		Horizontal		Vertical	
ID	Elevation (m)	Force (Hnu) (kN)	Elevation (m)	Force (Vnu) (kN)	position (x-axis) (m)
2	34.000	4.905	34.333	-0.654	4.578
3	32.000	44.145	33.000	-5.886	4.400
4	30.000	122.625	31.667	-16.350	4.222
5	28.000	240.345	30.333	-32.046	4.044
6	26.000	397.305	29.000	-52.974	3.867
7	24.000	593.505	27.667	-79.134	3.689
8	22.000	828.945	26.333	-110.526	3.511
9	20.000	1103.625	25.000	-147.150	3.333
10	18.000	1417.545	23.667	-189.006	3.156
11	16.000	1770.705	22.333	-236.094	2.978
12	14.000	2163.105	21.000	-288.414	2.800
13	12.000	2594.745	19.667	-345.966	2.622
14	10.000	3065.625	18.333	-408.750	2.444
15	8.000	3575.745	17.000	-476.766	2.267
16	6.000	4125.105	15.667	-550.014	2.089
17	4.000	4713.705	14.333	-628.494	1.911
18	2.000	5341.545	13.000	-712.206	1.733
19	Base	6008.625	11.667	-801.150	1.556



Upstream Hydrostatic Forces - Flood level (Hfu & Vfu):

Reservoir flood elevation (upstream side) = 37.500 m

Joint ID	Elevation (m)	Horizontal		Vertical	
		Force (Hfu) (kN)	Elevation (m)	Force (Vfu) (kN)	position (x-axis) (m)
1	36.000	11.036	36.500	-1.472	4.867
2	34.000	60.086	35.167	-8.012	4.689
3	32.000	148.376	33.833	-19.784	4.511
4	30.000	275.906	32.500	-36.788	4.333
5	28.000	442.676	31.167	-59.023	4.156
6	26.000	648.686	29.833	-86.492	3.978
7	24.000	893.936	28.500	-119.191	3.800
8	22.000	1178.426	27.167	-157.124	3.622
9	20.000	1502.156	25.833	-200.288	3.444
10	18.000	1865.126	24.500	-248.684	3.267
11	16.000	2267.336	23.167	-302.312	3.089
12	14.000	2708.786	21.833	-361.172	2.911
13	12.000	3189.476	20.500	-425.264	2.733
14	10.000	3709.406	19.167	-494.588	2.556
15	8.000	4268.576	17.833	-569.144	2.378
16	6.000	4866.986	16.500	-648.932	2.200
17	4.000	5504.636	15.167	-733.952	2.022
18	2.000	6181.526	13.833	-824.204	1.844
19	Base	6897.656	12.500	-919.688	1.667

Downstream Hydrostatic Forces - Operating level (Hnd & Vnd):

Reservoir normal operating elevation (downstream side) = 2.000 m

Joint ID	Elevation (m)	Horizontal		Vertical	
		Force (Hnd) (kN)	Elevation (m)	Force (Vnd) (kN)	position (x-axis) (m)
19	Base	-19.620	0.667	-14.535	35.306

Downstream Hydrostatic Forces - Flood level (Hfd & Vfd):

Reservoir flood elevation (downstream side) = 3.000 m

Joint ID	Elevation (m)	Horizontal		Vertical	
		Force (Hfd) (kN)	Elevation (m)	Force (Vfd) (kN)	position (x-axis) (m)
18	2.000	-4.905	2.333	-3.634	34.071
19	Base	-44.145	1.000	-32.705	35.059

Uplift forces (Un & Uf):

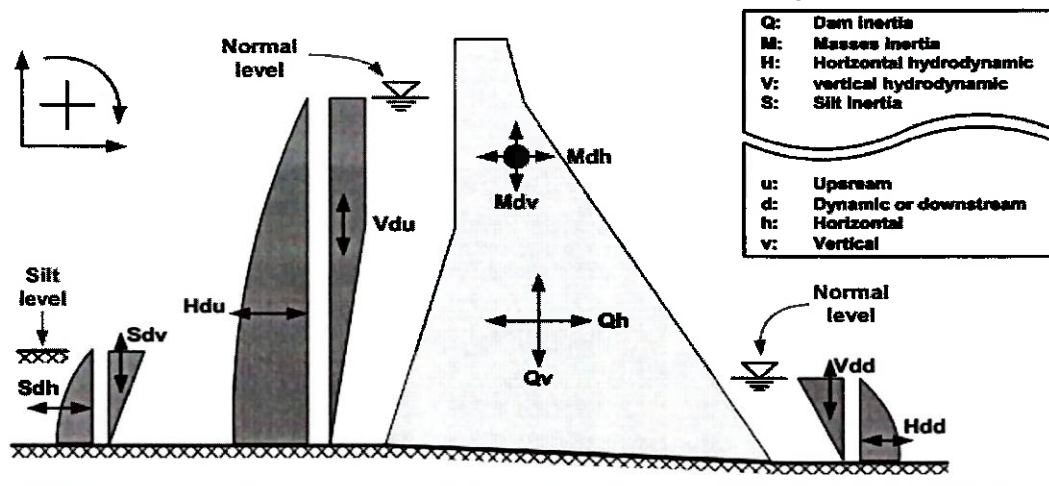
Joint ID	Elevation (m)	Normal operating level			Flood level		
		Force (Un) (kN)	position (I-axis) (m)	p factor (USBR)	Force (Uf) (kN)	position (I-axis) (m)	p factor (USBR)
1	36.000	0.000	0.000		34.580	1.567	
2	34.000	29.812	2.026		104.343	2.026	
3	32.000	115.164	2.609		211.134	2.609	
4	30.000	234.818	3.192		352.227	3.192	
5	28.000	388.775	3.774		527.624	3.774	
6	26.000	577.035	4.357		737.323	4.357	



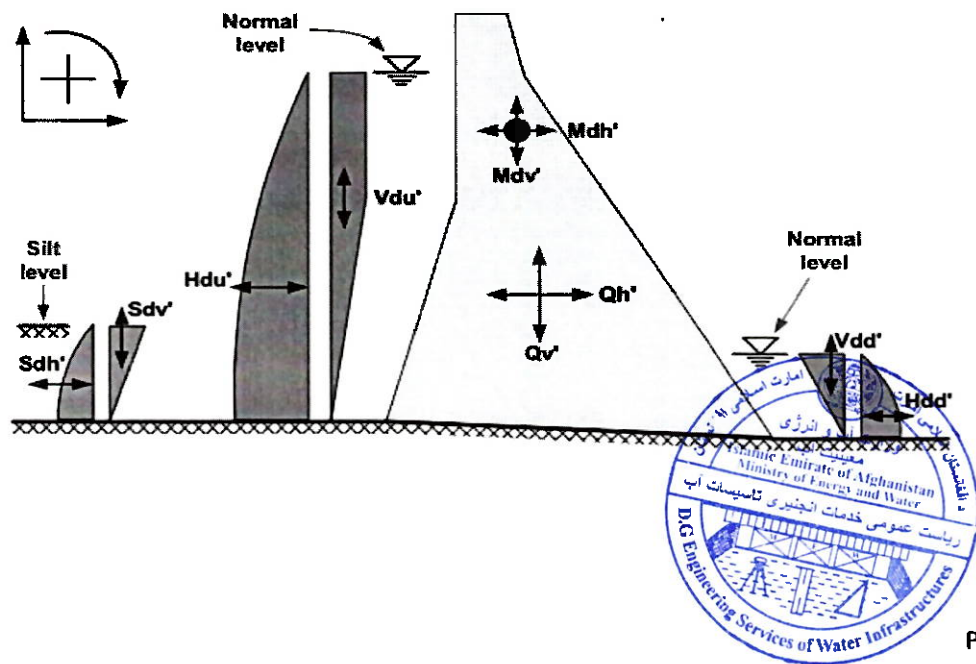
7	24.000	799.598	4.940	981.325	4.940
8	22.000	1056.463	5.523	1259.629	5.523
9	20.000	1347.631	6.105	1572.237	6.105
10	18.000	1673.102	6.688	1919.147	6.688
11	16.000	2032.876	7.271	2300.360	7.271
12	14.000	2426.953	7.854	2715.875	7.854
13	12.000	2855.332	8.437	3165.694	8.437
14	10.000	3318.014	9.019	3649.815	9.019
15	8.000	3814.998	9.602	4168.239	9.602
16	6.000	4346.286	10.185	4720.966	10.185
17	4.000	4911.876	10.768	5307.995	10.768
18	2.000	5511.769	11.351	6096.351	11.662
19	Base	6497.163	12.578	7111.760	12.817

Pseudo-static Loads (seismic coefficient method):

Stress Analysis (Peak accelerations)



Stability Analysis (Sustained accelerations)



Pseudo-static (Stress Analysis) Dam + Masses Inertia forces (Qh, Qv, Mdh & Mdv):

Joint ID	Elevation (m)	Dam (Horizontal)		Dam (Vertical)		Masses (Horizontal)		Masses (Vertical)	
		Force (Qh) (kN)	Elevation (m)	Force (Qv) (kN)	position (x) (m)	Force (Mdh) (kN)	Elevation (m)	Force (Mdv) (kN)	position (x) (m)
1	36.000	-48.736	36.745	-32.491	7.200	0.000	0.000	0.000	0.000
2	34.000	-122.901	35.663	-81.934	7.263	0.000	0.000	0.000	0.000
3	32.000	-221.109	34.461	-147.406	7.541	0.000	0.000	0.000	0.000
4	30.000	-344.015	33.213	-229.343	7.881	0.000	0.000	0.000	0.000
5	28.000	-491.619	31.940	-327.746	8.247	0.000	0.000	0.000	0.000
6	26.000	-663.922	30.651	-442.614	8.626	0.000	0.000	0.000	0.000
7	24.000	-860.922	29.353	-573.948	9.012	0.000	0.000	0.000	0.000
8	22.000	-1082.620	28.049	-721.747	9.402	0.000	0.000	0.000	0.000
9	20.000	-1329.017	26.739	-886.011	9.796	0.000	0.000	0.000	0.000
10	18.000	-1600.111	25.425	-1066.741	10.192	0.000	0.000	0.000	0.000
11	16.000	-1895.903	24.108	-1263.935	10.590	0.000	0.000	0.000	0.000
12	14.000	-2216.393	22.789	-1477.595	10.988	0.000	0.000	0.000	0.000
13	12.000	-2561.581	21.469	-1707.721	11.388	0.000	0.000	0.000	0.000
14	10.000	-2931.468	20.146	-1954.312	11.789	0.000	0.000	0.000	0.000
15	8.000	-3326.052	18.823	-2217.368	12.190	0.000	0.000	0.000	0.000
16	6.000	-3745.334	17.498	-2496.889	12.591	0.000	0.000	0.000	0.000
17	4.000	-4189.314	16.173	-2792.876	12.993	0.000	0.000	0.000	0.000
18	2.000	-4657.992	14.846	-3105.328	13.395	0.000	0.000	0.000	0.000
19	Base	-5151.368	13.519	-3434.246	13.798	0.000	0.000	0.000	0.000

Pseudo-static (Stress Analysis) Reservoir forces (Hdu, Vdu, Hdd & Vdd):

Joint ID	Elevation (m)	Upstream (Horizontal)		Upstream (Vertical)		Downstream (Horizontal)		Downstream (Vertical)	
		Force (Hdu) (kN)	Elevation (m)	Force (Vdu) (kN)	position (x) (m)	Force (Hdd) (kN)	Elevation (m)	Force (Vdd) (kN)	position (x) (m)
2	34.000	-8.403	34.400	-0.131	4.578	0.000	0.000	0.000	0.000
3	32.000	-43.662	33.200	-1.177	4.400	0.000	0.000	0.000	0.000
4	30.000	-93.945	32.000	-3.270	4.222	0.000	0.000	0.000	0.000
5	28.000	-155.620	30.800	-6.409	4.044	0.000	0.000	0.000	0.000
6	26.000	-226.872	29.600	-10.595	3.867	0.000	0.000	0.000	0.000
7	24.000	-306.554	28.400	-15.827	3.689	0.000	0.000	0.000	0.000
8	22.000	-393.852	27.200	-22.105	3.511	0.000	0.000	0.000	0.000
9	20.000	-488.152	26.000	-29.430	3.333	0.000	0.000	0.000	0.000
10	18.000	-588.968	24.800	-37.801	3.156	0.000	0.000	0.000	0.000
11	16.000	-695.903	23.600	-47.219	2.978	0.000	0.000	0.000	0.000
12	14.000	-808.625	22.400	-57.683	2.800	0.000	0.000	0.000	0.000
13	12.000	-926.851	21.200	-69.193	2.622	0.000	0.000	0.000	0.000
14	10.000	-1050.335	20.000	-81.750	2.444	0.000	0.000	0.000	0.000
15	8.000	-1178.864	18.800	-95.353	2.267	0.000	0.000	0.000	0.000
16	6.000	-1312.245	17.600	-110.003	2.089	0.000	0.000	0.000	0.000
17	4.000	-1450.309	16.400	-125.699	1.911	0.000	0.000	0.000	0.000
18	2.000	-1592.902	15.200	-142.441	1.733	0.000	0.000	0.000	0.000
19	Base	-1702.733	14.300	-160.230	1.556	-1.175	1.100	-3.773	35.232



Pseudo-static (Stability Analysis) Dam + Masses Inertia forces (Qh', Qv', Mdh' & Mdv'):

Joint ID	Elevation (m)	Dam (Horizontal)		Dam (Vertical)		Masses (Horizontal)		Masses (Vertical)	
		Force (Qh') (kN)	Elevation (m)	Force (Qv') (kN)	position (x) (m)	Force (Mdh') (kN)	Elevation (m)	Force (Mdv') (kN)	position (x) (m)
1	36.000	-24.368	36.745	-16.245	7.200	0.000	0.000	0.000	0.000
2	34.000	-61.450	35.663	-40.967	7.263	0.000	0.000	0.000	0.000
3	32.000	-110.554	34.461	-73.703	7.541	0.000	0.000	0.000	0.000
4	30.000	-172.008	33.213	-114.672	7.881	0.000	0.000	0.000	0.000
5	28.000	-245.810	31.940	-163.873	8.247	0.000	0.000	0.000	0.000
6	26.000	-331.961	30.651	-221.307	8.626	0.000	0.000	0.000	0.000
7	24.000	-430.461	29.353	-286.974	9.012	0.000	0.000	0.000	0.000
8	22.000	-541.310	28.049	-360.873	9.402	0.000	0.000	0.000	0.000
9	20.000	-664.508	26.739	-443.006	9.796	0.000	0.000	0.000	0.000
10	18.000	-800.055	25.425	-533.370	10.192	0.000	0.000	0.000	0.000
11	16.000	-947.951	24.108	-631.968	10.590	0.000	0.000	0.000	0.000
12	14.000	-1108.197	22.789	-738.798	10.988	0.000	0.000	0.000	0.000
13	12.000	-1280.791	21.469	-853.860	11.388	0.000	0.000	0.000	0.000
14	10.000	-1465.734	20.146	-977.156	11.789	0.000	0.000	0.000	0.000
15	8.000	-1663.026	18.823	-1108.684	12.190	0.000	0.000	0.000	0.000
16	6.000	-1872.667	17.498	-1248.445	12.591	0.000	0.000	0.000	0.000
17	4.000	-2094.657	16.173	-1396.438	12.993	0.000	0.000	0.000	0.000
18	2.000	-2328.996	14.846	-1552.664	13.395	0.000	0.000	0.000	0.000
19	Base	-2575.684	13.519	-1717.123	13.798	0.000	0.000	0.000	0.000

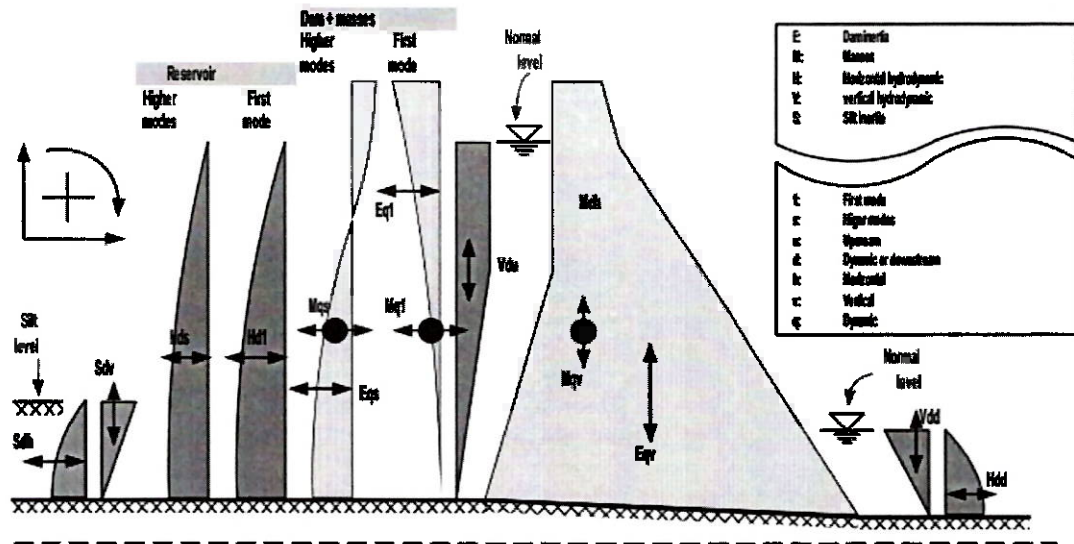
Pseudo-static (Stability Analysis) Upstream reservoir forces (Hdu' & Vdu'):

Joint ID	Elevation (m)	Upstream (Horizontal)		Upstream (Vertical)		Downstream (Horizontal)		Downstream (Vertical)	
		Force (Hdu') (kN)	Elevation (m)	Force (Vdu') (kN)	position (x) (m)	Force (Hdd') (kN)	Elevation (m)	Force (Vdd') (kN)	position (x) (m)
2	34.000	-4.201	34.400	-0.065	4.578	0.000	0.000	0.000	0.000
3	32.000	-21.831	33.200	-0.589	4.400	0.000	0.000	0.000	0.000
4	30.000	-46.972	32.000	-1.635	4.222	0.000	0.000	0.000	0.000
5	28.000	-77.810	30.800	-3.205	4.044	0.000	0.000	0.000	0.000
6	26.000	-113.436	29.600	-5.297	3.867	0.000	0.000	0.000	0.000
7	24.000	-153.277	28.400	-7.913	3.689	0.000	0.000	0.000	0.000
8	22.000	-196.926	27.200	-11.053	3.511	0.000	0.000	0.000	0.000
9	20.000	-244.076	26.000	-14.715	3.333	0.000	0.000	0.000	0.000
10	18.000	-294.484	24.800	-18.901	3.156	0.000	0.000	0.000	0.000
11	16.000	-347.951	23.600	-23.609	2.978	0.000	0.000	0.000	0.000
12	14.000	-404.312	22.400	-28.841	2.800	0.000	0.000	0.000	0.000
13	12.000	-463.425	21.200	-34.597	2.622	0.000	0.000	0.000	0.000
14	10.000	-525.168	20.000	-40.875	2.444	0.000	0.000	0.000	0.000
15	8.000	-589.432	18.800	-47.677	2.267	0.000	0.000	0.000	0.000
16	6.000	-656.123	17.600	-55.001	2.089	0.000	0.000	0.000	0.000
17	4.000	-725.155	16.400	-62.849	1.911	0.000	0.000	0.000	0.000
18	2.000	-796.451	15.200	-71.221	1.733	0.000	0.000	0.000	0.000
19	Base	-851.367	14.300	-80.115	1.556	-0.587	1.100	-1.889	35.232

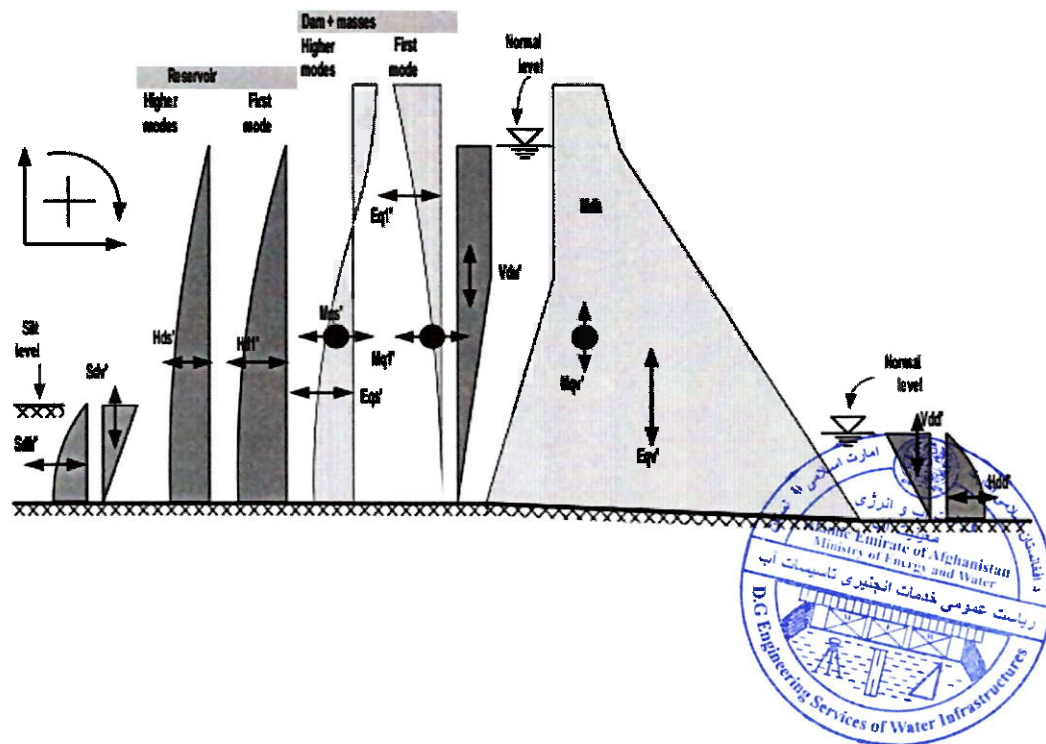


Pseudo-dynamic Loads (Chopra's method):

Stress Analysis (Peak accelerations)



Stability Analysis (Sustained accelerations)



Pseudo-dynamic - constants results:

Fundamental periods			
Dam	Reservoir	Dam / reservoir	Dam/reservoir/foundation
$T_1 = 0.0861 \text{ sec}$	$T_1^r = 0.0958 \text{ sec}$	$\bar{T}_r = 0.1039 \text{ sec}$	$\bar{T}_1 = 0.1233 \text{ sec}$
Period Lengthening ratios		Reservoir related data	
Reservoir	Foundation	$R_w = 0.9225$	$A_p = 0.2672$
$R_r = 1.2067$	$R_f = 1.1871$	$F_{st} = 5838.18 \text{ kN}$	$B_1 = 100.74 \text{ ton}$
Viscous damping ratios			
Dam	Reservoir	Foundation	Dam/reservoir/foundation
$\xi_1 = 0.0500$	$\xi_r = 0.0399$	$\xi_f = 0.0677$	$\bar{\xi}_1 = 0.1325$
Generalized masses		Earthquake force coefficients	
$M_1 = 115.220 \text{ ton}$	$\bar{M}_1 = 167.767 \text{ ton}$	$L_1 = 293.796 \text{ ton}$	$\bar{L}_1 = 428.383 \text{ ton}$

Pseudo-dynamic (Stress Analysis) First mode forces (Eq1, Hd1 & Em1):

Joint ID	Elevation (m)	Dam		Reservoir		Dam + Reservoir	
		Force (Eq1) (kN)	Elevation (m)	Force (Hd1) (kN)	Elevation (m)	Force (Em1) (kN)	Elevation (m)
1	36.000	-125.855	36.759	0.000	0.000	-125.855	36.759
2	34.000	-291.101	35.750	-6.043	34.342	-297.144	35.721
3	32.000	-474.171	34.683	-44.267	33.081	-518.439	34.547
4	30.000	-666.650	33.618	-101.886	31.883	-768.536	33.388
5	28.000	-862.394	32.570	-168.830	30.734	-1031.224	32.269
6	26.000	-1056.744	31.546	-240.495	29.619	-1297.239	31.189
7	24.000	-1245.422	30.555	-314.748	28.528	-1560.169	30.146
8	22.000	-1424.515	29.607	-390.305	27.458	-1814.820	29.145
9	20.000	-1590.839	28.709	-466.036	26.408	-2056.875	28.187
10	18.000	-1742.146	27.867	-540.832	25.384	-2282.977	27.279
11	16.000	-1877.024	27.087	-613.728	24.389	-2490.752	26.423
12	14.000	-1994.630	26.376	-684.092	23.424	-2678.721	25.622
13	12.000	-2094.518	25.740	-751.706	22.487	-2846.224	24.881
14	10.000	-2176.733	25.184	-816.705	21.573	-2993.438	24.199
15	8.000	-2242.020	24.714	-879.366	20.678	-3121.386	23.577
16	6.000	-2291.762	24.331	-939.833	19.798	-3231.595	23.013
17	4.000	-2327.231	24.037	-997.964	18.937	-3325.195	22.506
18	2.000	-2348.477	23.848	-1053.507	18.097	-3401.984	22.067
19	Base	-2354.995	23.786	-1106.951	17.272	-3461.946	21.703

Pseudo-dynamic (Stress Analysis) Superior modes forces (Eqs, Hds & Ems):

Joint ID	Elevation (m)	Dam		Reservoir		Dam + Reservoir	
		Force (Eqs) (kN)	Elevation (m)	Force (Hds) (kN)	Elevation (m)	Force (Ems) (kN)	Elevation (m)
1	36.000	69.084	36.769	0.000	0.000	69.084	36.769
2	34.000	149.597	35.821	25.318	34.566	174.915	35.640
3	32.000	222.779	34.904	55.602	33.774	278.381	34.678
4	30.000	280.069	34.117	67.168	33.339	347.237	33.966
5	28.000	315.718	33.551	62.708	33.687	378.426	33.574
6	26.000	325.364	33.372	44.052	36.571	369.416	33.753



7	24.000	305.001	33.948	12.158	67.101	317.160	35.219
8	22.000	250.969	36.328	-32.603	6.488	218.366	40.784
9	20.000	160.283	45.040	-90.086	15.725	70.196	82.663
10	18.000	30.840	154.552	-160.102	17.144	-129.263	-15.639
11	16.000	-138.681	-13.638	-242.299	17.087	-380.979	5.903
12	14.000	-349.070	3.603	-336.138	16.499	-685.208	9.929
13	12.000	-600.744	7.528	-440.891	15.664	-1041.634	10.972
14	10.000	-893.661	8.659	-555.594	14.698	-1449.256	10.974
15	8.000	-1227.124	8.746	-679.014	13.660	-1906.138	10.497
16	6.000	-1599.839	8.335	-809.743	12.584	-2409.582	9.763
17	4.000	-2010.614	7.651	-946.566	11.486	-2957.180	8.878
18	2.000	-2459.401	6.799	-1088.990	10.376	-3548.392	7.897
19	Base	-2946.650	5.838	-1237.192	9.252	-4183.842	6.848

Pseudo-dynamic (Stress Analysis) Modal combination (Emc):

Joint ID	Elevation (m)	Mode 1 and superior modes combined	
		Force (Emc) (kN)	Elevation (m)
1	36.000	-143.569	36.761
2	34.000	-344.804	35.700
3	32.000	-588.451	34.576
4	30.000	-843.339	33.487
5	28.000	-1098.467	32.427
6	26.000	-1348.813	31.388
7	24.000	-1592.080	30.364
8	22.000	-1827.910	29.343
9	20.000	-2058.073	28.312
10	18.000	-2286.634	27.249
11	16.000	-2519.720	26.134
12	14.000	-2764.970	24.945
13	12.000	-3030.840	23.667
14	10.000	-3325.810	22.300
15	8.000	-3657.378	20.852
16	6.000	-4031.041	19.350
17	4.000	-4449.925	17.823
18	2.000	-4915.748	16.301
19	Base	-5430.433	14.807

Pseudo-dynamic (Stress Analysis) Vertical forces (Eqv & Vdu):

Joint ID	Elevation (m)	Dam		Reservoir	
		Force (EQ) (kN)	position (x-axis) (m)	Force (VD) (kN)	position (x-axis) (m)
1	36.000	-32.491	7.200	0.000	0.000
2	34.000	-81.934	7.263	-0.131	4.578
3	32.000	-147.406	7.541	-1.177	4.400
4	30.000	-229.343	7.881	-3.270	4.222
5	28.000	-327.746	8.247	-6.409	4.044
6	26.000	-442.614	8.626	-10.595	3.867
7	24.000	-573.948	9.012	-15.827	3.689
8	22.000	-721.747	9.402	-22.105	3.511
9	20.000	-886.011	9.796	-29.430	3.333
10	18.000	-1066.741	10.192	-37.801	3.156
11	16.000	-1263.935	10.590	-47.219	2.978
12	14.000	-1477.595	10.988	-57.683	2.800
13	12.000	-1707.721	11.388	-69.193	2.622



14	10.000	-1954.312	11.789	-81.750	2.444
15	8.000	-2217.368	12.190	-95.353	2.267
16	6.000	-2496.889	12.591	-110.003	2.089
17	4.000	-2792.876	12.993	-125.699	1.911
18	2.000	-3105.328	13.395	-142.441	1.733
19	Base	-3434.246	13.798	-160.230	1.556

Pseudo-dynamic (Stress Analysis) Downstream reservoir forces (Hdd & Vdd):

Joint ID	Elevation (m)	Horizontal		Vertical	
		Force (Hdd) (kN)	Elevation (m)	Force (Vdd) (kN)	position (x-axis) (m)
19	Base	-1.175	1.100	-3.777	35.232

Pseudo-dynamic (Stability Analysis) First mode forces (Eq1', Hd1' & Em1'):

Joint ID	Elevation (m)	Dam		Reservoir		Dam + Reservoir	
		Force (Eq1') (kN)	Elevation (m)	Force (Hd1') (kN)	Elevation (m)	Force (Em1') (kN)	Elevation (m)
1	36.000	-66.860	36.759	0.000	0.000	-66.860	36.759
2	34.000	-154.648	35.750	-3.210	34.342	-157.858	35.721
3	32.000	-251.904	34.683	-23.517	33.081	-275.421	34.547
4	30.000	-354.158	33.618	-54.127	31.883	-408.285	33.388
5	28.000	-458.147	32.570	-89.691	30.734	-547.838	32.269
6	26.000	-561.395	31.546	-127.763	29.619	-689.158	31.189
7	24.000	-661.630	30.555	-167.210	28.528	-828.840	30.146
8	22.000	-756.774	29.607	-207.349	27.458	-964.123	29.145
9	20.000	-845.133	28.709	-247.582	26.408	-1092.715	28.187
10	18.000	-925.515	27.867	-287.317	25.384	-1212.832	27.279
11	16.000	-997.169	27.087	-326.043	24.389	-1323.212	26.423
12	14.000	-1059.647	26.376	-363.424	23.424	-1423.071	25.622
13	12.000	-1112.713	25.740	-399.344	22.487	-1512.056	24.881
14	10.000	-1156.389	25.184	-433.875	21.573	-1590.264	24.199
15	8.000	-1191.073	24.714	-467.163	20.678	-1658.236	23.577
16	6.000	-1217.498	24.331	-499.286	19.798	-1716.785	23.013
17	4.000	-1236.341	24.037	-530.169	18.937	-1766.510	22.506
18	2.000	-1247.628	23.848	-559.676	18.097	-1807.304	22.067
19	Base	-1251.091	23.786	-588.068	17.272	-1839.159	21.703

Pseudo-dynamic (Stability Analysis) Sup. modes forces (Eqs', Hds' & Ems'):

Joint ID	Elevation (m)	Dam		Reservoir		Dam + Reservoir	
		Force (Eqs') (kN)	Elevation (m)	Force (Hds') (kN)	Elevation (m)	Force (Ems') (kN)	Elevation (m)
1	36.000	34.542	36.769	0.000	0.000	34.542	36.769
2	34.000	74.798	35.821	12.659	34.566	87.458	35.640
3	32.000	111.390	34.904	27.801	33.774	139.191	34.678
4	30.000	140.034	34.117	33.584	33.339	173.618	33.966
5	28.000	157.859	33.551	31.354	33.687	189.213	33.574
6	26.000	162.682	33.372	22.026	36.571	184.708	33.753
7	24.000	152.501	33.948	6.079	67.101	158.580	35.219
8	22.000	125.484	36.328	-16.302	6.488	109.183	40.784
9	20.000	80.141	45.040	-45.043	15.725	35.098	82.663
10	18.000	15.420	154.552	-80.051	17.144	-64.631	-15.639
11	16.000	-69.340	-13.638	-121.149	17.087	-190.490	-3.803
12	14.000	-174.535	3.603	-168.069	16.499	-342.604	10.947
13	12.000	-300.372	7.528	-220.445	15.664	-520.817	10.947



14	10.000	-446.831	8.659	-277.797	14.698	-724.628	10.974
15	8.000	-613.562	8.746	-339.507	13.660	-953.069	10.497
16	6.000	-799.920	8.335	-404.871	12.584	-1204.791	9.763
17	4.000	-1005.307	7.651	-473.283	11.486	-1478.590	8.878
18	2.000	-1229.701	6.799	-544.495	10.376	-1774.196	7.897
19	Base	-1473.325	5.838	-618.596	9.252	-2091.921	6.848

Pseudo-dynamic (Stability Analysis) Modal combination (Emc'):

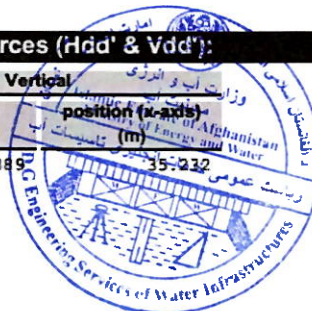
Joint ID	Elevation (m)	Mode 1 and superior modes combined	
		Force (Emc') (kN)	Elevation (m)
1	36.000	-75.256	36.761
2	34.000	-180.466	35.702
3	32.000	-308.595	34.573
4	30.000	-443.666	33.477
5	28.000	-579.593	32.411
6	26.000	-713.481	31.367
7	24.000	-843.874	30.340
8	22.000	-970.286	29.321
9	20.000	-1093.279	28.298
10	18.000	-1214.553	27.253
11	16.000	-1336.853	26.166
12	14.000	-1463.731	25.019
13	12.000	-1599.239	23.794
14	10.000	-1747.577	22.486
15	8.000	-1912.613	21.100
16	6.000	-2097.349	19.654
17	4.000	-2303.646	18.175
18	2.000	-2532.611	16.691
19	Base	-2785.433	15.225

Pseudo-dynamic (Stability Analysis) Vertical forces (Eqv' & Vdu'):

Joint ID	Elevation (m)	Dam		Reservoir	
		Force (Eqv') (kN)	position (x-axis) (m)	Force (Vdu') (kN)	position (x-axis) (m)
1	36.000	-16.245	7.200	0.000	0.000
2	34.000	-40.967	7.263	-0.065	4.578
3	32.000	-73.703	7.541	-0.589	4.400
4	30.000	-114.672	7.881	-1.635	4.222
5	28.000	-163.873	8.247	-3.205	4.044
6	26.000	-221.307	8.626	-5.297	3.867
7	24.000	-286.974	9.012	-7.913	3.689
8	22.000	-360.873	9.402	-11.053	3.511
9	20.000	-443.006	9.796	-14.715	3.333
10	18.000	-533.370	10.192	-18.901	3.156
11	16.000	-631.968	10.590	-23.609	2.978
12	14.000	-738.798	10.988	-28.841	2.800
13	12.000	-853.860	11.388	-34.597	2.622
14	10.000	-977.156	11.789	-40.875	2.444
15	8.000	-1108.684	12.190	-47.677	2.267
16	6.000	-1248.445	12.591	-55.001	2.089
17	4.000	-1396.438	12.993	-62.849	1.911
18	2.000	-1552.664	13.395	-71.221	1.733
19	Base	-1717.123	13.798	-80.115	1.556

Pseudo-dynamic (Stability analysis) Downstream reservoir forces (Hdd' & Vdd'):

Joint ID	Elevation (m)	Horizontal		Vertical	
		Force (Hdd') (kN)	Elevation (m)	Force (Vdd') (kN)	position (x-axis) (m)
19	Base	-0.587	1.100	-1.889	35.232



2.5 Analysis Result

Load Combination Factors:

	Usual	Flood	Seismic #1	Seismic #2	Post-seismic #1
Self-weight	1.0000	1.0000	1.0000	1.0000	1.0000
Hydrostatic (upstream)	1.0000	1.0000	1.0000	1.0000	1.0000
Hydrostatic (downstream)	1.0000	1.0000	1.0000	1.0000	1.0000
Uplift pressures					1.0000
Silts					
Ice					
post-tensioning					
Applied forces					
Floating debris					
Seismic (horizontal)			-0.6000	-1.0000	
Seismic (vertical)			-0.6000	-1.0000	

Combination Required Safety Factors:

	Usual	Flood	Seismic #1	Seismic #2	Post-seismic #1
Peak sliding factor	3.0000	2.0000	1.3000	1.3000	2.0000
Residual sliding factor	1.5000	1.3000	1.0000	1.0000	1.1000
Overturning factor	1.2000	1.1000	1.1000	1.1000	1.1000
Uplifting factor	1.2000	1.1000	1.1000	1.1000	1.1000

Combination allowable stresses:

	Usual	Flood	Seismic #1	Seismic #2	Post-seismic #1
Tension (% of ft)	0.0	50.0	90.9	90.9	66.7
Compression (% of fc)	33.3	50.0	90.9	90.9	66.7

Usual Combination (Stresses):

ID	Joint	U/S elevation (m)	Cracking		Normal stresses		Allowable normal stress		Shear stresses			
			Upstream (%) of joint	Downstream (%) of joint	Upstream (kPa)	Downstream (kPa)	Tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at (%) of joint	Downstream (kPa)
1		36.000			-32.374	-36.755	0.000	-9990.000				
2		34.000			-88.169	-46.852	0.000	-9990.000	11.756	-11.385	41.470	34.710
3		32.000			-138.921	-50.931	0.000	-9990.000	18.523	-6.286	42.881	37.732
4		30.000			-180.988	-61.960	0.000	-9990.000	24.132	0.813	41.831	45.902
5		28.000			-217.361	-77.751	0.000	-9990.000	28.982	8.647	39.191	57.601
6		26.000			-249.911	-96.808	0.000	-9990.000	33.321	16.556	35.538	71.720
7		24.000			-279.796	-118.170	0.000	-9990.000	37.306	24.160	31.291	87.546
8		22.000			-307.761	-141.207	0.000	-9990.000	41.035	31.247	26.753	104.612
9		20.000			-334.298	-165.494	0.000	-9990.000	44.573	37.713	22.134	122.606
10		18.000			-359.749	-190.739	0.000	-9990.000	47.966	43.521	17.575	141.308
11		16.000			-384.352	-216.732	0.000	-9990.000	51.247	48.676	13.163	160.565
12		14.000			-408.285	-243.320	0.000	-9990.000	54.438	53.210	8.951	180.262
13		12.000			-431.676	-270.388	0.000	-9990.000	57.557	57.166	4.966	200.316
14		10.000			-454.624	-297.851	0.000	-9990.000	60.616	60.592	1.220	220.661
15		8.000			-477.205	-325.640	0.000	-9990.000	63.627	63.245	0.000	241.249
16		6.000			-499.479	-353.703	0.000	-9990.000	66.597	66.039	0.000	262.039
17		4.000			-521.494	-381.998	0.000	-9990.000	69.536	69.001	0.000	283.001
18		2.000			-543.287	-410.491	0.000	-9990.000	72.438	72.000	0.000	304.110
19	Base joint				-564.172	-440.663	0.000	-9990.000	75.223	75.000	0.000	326.478



Usual Combination (Stability):

Joint		Safety factors					Resultants over ligament				Final uplift	Rock wedge
ID	U/S elevation (m)	Sliding		Overturning		Uplifting	Normal (kN)	Shear (kN)	Moment (kN-m)	Position % of joint	Normal (kN)	Resistance (kN)
		Peak	Residual	toward U/S	toward D/S							
1	36.000	> 100	> 100	> 100	> 100	> 100	-162.45	0.00	8.1	51.05612	0.00	0.000
2	34.000	> 100	83.65390	> 100	> 100	> 100	-410.32	4.91	-127.2	44.89987	29.81	0.000
3	32.000	24.03428	16.82898	> 100	77.02802	> 100	-742.92	44.15	-449.1	42.27555	115.16	0.000
4	30.000	13.54562	9.48475	> 100	32.69317	> 100	-1163.07	122.63	-909.3	41.83444	234.82	0.000
5	28.000	9.92789	6.95158	> 100	20.52683	> 100	-1670.78	240.34	-1491.6	42.11537	388.78	0.000
6	26.000	8.14550	5.70354	> 100	15.25445	> 100	-2266.05	397.31	-2179.9	42.64041	577.04	0.000
7	24.000	7.09586	4.96857	> 100	12.40014	> 100	-2948.87	593.50	-2958.1	43.23116	799.60	0.000
8	22.000	6.40773	4.48674	> 100	10.63795	> 100	-3719.26	828.95	-3809.9	43.81714	1056.46	0.000
9	20.000	5.92314	4.14743	> 100	9.45185	> 100	-4577.21	1103.63	-4719.4	44.37086	1347.63	0.000
10	18.000	5.56402	3.89597	> 100	8.60339	> 100	-5522.71	1417.55	-5670.2	44.88304	1673.10	0.000
11	16.000	5.28750	3.70235	> 100	7.96844	> 100	-6555.77	1770.71	-6646.3	45.35227	2032.88	0.000
12	14.000	5.06819	3.54878	> 100	7.47649	> 100	-7676.39	2163.11	-7631.6	45.78054	2426.95	0.000
13	12.000	4.89087	3.42406	> 100	7.08472	> 100	-8884.57	2594.75	-8809.9	46.17111	2855.33	0.000
14	10.000	4.74259	3.32079	> 100	6.76570	> 100	-10180.31	3065.63	-9565.1	46.52761	3318.01	0.000
15	8.000	4.61849	3.23390	> 100	6.50110	> 100	-11563.61	3575.75	-10481.0	46.85359	3815.00	0.000
16	6.000	4.51265	3.15979	> 100	6.27823	> 100	-13034.46	4125.11	-11341.4	47.15231	4346.29	0.000
17	4.000	4.42132	3.09584	> 100	6.08803	> 100	-14592.87	4713.70	-12130.3	47.42673	4911.88	0.000
18	2.000	4.34172	3.04010	> 100	5.92386	> 100	-16238.85	5341.55	-12831.6	47.67947	5511.77	0.000
19	Base joint	4.28919	3.00332	> 100	5.78105	> 100	-17986.91	5989.01	-13189.1	47.95179	6497.16	0.000
Required:		3.000	1.500	1.200	1.200	1.200						

Flood Combination (Stresses):

Joint		Cracking		Normal stresses		Allowable normal stress		Shear stresses over the ligament			
ID	U/S elevation (m)	Upstream (% of joint)	Downstream (% of joint)	Upstream (kPa)	Downstream (kPa)	Tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at (% of joint)	Downstream (kPa)
1	36.000			-32.101	-37.654	0.000	-15000.000	4.280	4.280	0.000	0.000
2	34.000			-81.693	-55.748	0.000	-15000.000	10.892	-0.596	34.368	41.301
3	32.000			-123.304	-70.099	0.000	-15000.000	16.441	7.898	30.577	51.932
4	30.000			-157.193	-90.024	0.000	-15000.000	20.959	15.505	24.609	66.694
5	28.000			-186.835	-113.042	0.000	-15000.000	24.911	22.010	17.815	83.747
6	26.000			-213.894	-137.954	0.000	-15000.000	28.519	27.382	10.975	102.202
7	24.000			-239.265	-164.107	0.000	-15000.000	31.902	31.704	4.486	121.578
8	22.000			-263.474	-191.118	0.000	-15000.000	35.130	141.589	100.000	141.589
9	20.000			-286.846	-218.748	0.000	-15000.000	38.246	162.058	100.000	162.058
10	18.000			-309.599	-246.837	0.000	-15000.000	41.280	182.868	100.000	182.868
11	16.000			-331.878	-275.278	0.000	-15000.000	44.250	203.938	100.000	203.938
12	14.000			-353.788	-303.893	0.000	-15000.000	47.172	225.211	100.000	225.211
13	12.000			-375.404	-332.926	0.000	-15000.000	50.054	246.646	100.000	246.646
14	10.000			-396.783	-362.036	0.000	-15000.000	52.904	268.212	100.000	268.212
15	8.000			-417.968	-391.291	0.000	-15000.000	55.729	289.886	100.000	289.886
16	6.000			-438.991	-420.666	0.000	-15000.000	58.532	311.648	100.000	311.648
17	4.000			-459.879	-450.142	0.000	-15000.000	61.317	333.486	100.000	333.486
18	2.000			-480.451	-480.118	0.000	-15000.000	64.060	355.693	100.000	355.693
19	Base joint			-499.819	-512.674	1500.000	-15000.000	66.643	379.812	100.000	379.812

Flood Combination (Stability):

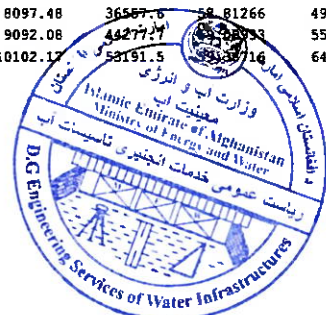
Joint		Safety factors					Resultants over ligament				Final uplift	Rock wedge
ID	U/S elevation (m)	Sliding		Overturning		Uplifting	Normal (kN)	Shear (kN)	Moment (kN-m)	Position % of joint	Uplift (kN)	Resistance (kN)
		Peak	Residual	toward U/S	toward D/S							
1	36.000	21.21276	14.85333	> 100	68.95822	> 100	-163.93	11.04	10.2	51.32676	34.58	0.000
2	34.000	9.92754	6.95134	> 100	20.24640	> 100	-417.68	60.09	-79.9	46.85384	104.34	0.000
3	32.000	7.28446	5.10063	> 100	12.88536	> 100	-756.81	148.38	-271.6	45.41495	211.13	0.000
4	30.000	6.12607	4.28952	> 100	9.95805	> 100	-1183.50	275.91	-513.1	45.47167	352.23	0.000
5	28.000	5.47724	3.83521	> 100	8.41917	> 100	-1697.75	442.68	-788.4	45.89876	527.62	0.000
6	26.000	5.06272	3.54496	> 100	7.47884	> 100	-2299.56	648.69	-1081.3	46.40280	737.32	0.000
7	24.000	4.77510	3.34356	> 100	6.84757	> 100	-2988.93	893.94	-1375.5	46.89458	981.32	0.000
8	22.000	4.56389	3.19567	> 100	6.39566	> 100	-3765.86	1178.43	-1655.1	47.34725	1259.63	0.000
9	20.000	4.40221	3.08246	> 100	6.05668	> 100	-4630.34	1502.16	-1903.9	47.75517	1572.24	0.000
10	18.000	4.27450	2.99303	> 100	5.79327	> 100	-5582.39	1865.13	-2105.6	48.12022	1919.15	0.000
11	16.000	4.17105	2.92060	> 100	5.58283	> 100	-6621.99	2267.34	-2444.3	48.40630	2300.36	0.000
12	14.000	4.08557	2.86075	> 100	5.41091	> 100	-7749.15	2708.79	-2903.6	48.70088	2715.88	0.000
13	12.000	4.01374	2.81045	> 100	5.26789	> 100	-8963.87	3189.48	-3267.6	49.00000	3163.69	0.000
14	10.000	3.95254	2.76760	> 100	5.14705	> 100	-10266.15	3709.41	-4120.0	49.23581	3644.12	0.000
15	8.000	3.89977	2.73065	> 100	5.04364	> 100	-11655.98	4268.58	-4844.3	49.43506	4168.24	0.000
16	6.000	3.85380	2.69846	> 100	4.95415	> 100	-13133.38	4866.99	-5644.7	49.60472	4720.97	0.000
17	4.000	3.81340	2.67017	> 100	4.87595	> 100	-14698.33	5504.64	-6499.3	49.75280	5308.00	0.000
18	2.000	3.78145	2.64780	> 100	4.80708	> 100	-16354.48	6176.64	-7521.9	49.89422	6036.35	0.000
19	Base joint	3.77664	2.64443	> 100	4.74666	> 100	-18123.62	6953.55	-8732.9	50.00000	6711.76	0.000
Required:		2.000	1.300	1.100	1.100	1.100						

Seismic #1 Combination - Peak accelerations analysis (Stresses):

Joint		Cracking		Normal stresses		Allowable normal stress		Shear stresses over the ligament			
ID	U/S elevation (m)	Upstream (% of joint)	Downstream (% of joint)	Upstream (kPa)	Downstream (kPa)	Tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at (% of joint)	Downstream (kPa)
1	36.000			-22.576	-38.258	0.000	-27270.000	3.010	8.660	44.582	0.000
2	34.000			-57.315	-61.503	0.000	-27270.000	7.642	2.683	25.377	45.564
3	32.000			-86.664	-80.405	0.000	-27270.000	11.555	11.146	8.419	59.568
4	30.000			-106.883	-106.911	0.000	-27270.000	14.251	79.204	100.000	79.204
5	28.000			-121.511	-138.187	0.000	-27270.000	16.202	102.375	100.000	102.375
6	26.000			-132.623	-172.490	0.000	-27270.000	17.683	127.789	100.000	127.789
7	24.000			-141.429	-208.781	0.000	-27270.000	18.857	154.675	100.000	154.675
8	22.000			-148.668	-246.423	0.000	-27270.000	19.822	182.561	100.000	182.561
9	20.000			-154.810	-285.007	0.000	-27270.000	20.641	211.146	100.000	211.146
10	18.000			-160.162	-324.267	0.000	-27270.000	21.355	240.232	100.000	240.232
11	16.000			-164.935	-364.020	0.000	-27270.000	21.991	269.682	100.000	269.682
12	14.000			-169.275	-404.137	0.000	-27270.000	22.570	299.403	100.000	299.403
13	12.000			-173.287	-444.529	0.000	-27270.000	23.105	329.327	100.000	329.327
14	10.000			-177.049	-485.129	0.000	-27270.000	23.607	359.405	100.000	359.405
15	8.000			-180.617	-525.886	0.000	-27270.000	24.082	389.600	100.000	389.600
16	6.000			-184.035	-566.765	0.000	-27270.000	24.538	419.885	100.000	419.885
17	4.000			-187.335	-607.737	0.000	-27270.000	24.978	450.239	100.000	450.239
18	2.000			-190.544	-648.781	0.000	-27270.000	25.406	480.646	100.000	480.646
19	Base joint			-193.106	-691.138	2727.000	-27270.000	25.747	512.026	100.000	512.026

Seismic #1 Combination - Peak accelerations analysis (Stability):

Joint		Safety factors					Resultants over ligament				Final uplift	Rock wedge
ID	U/S elevation (m)	Sliding		Overturning		Uplifting	Normal (kN)	Shear (kN)	Moment (kN-m)	Position % of joint	Uplift (kN)	Resistance (kN)
		Peak	Residual	toward U/S	toward D/S							
1	36.000	6.98206	4.88889	8.79876	5.60975	8.33333	-142.96	29.24	28.9	54.29649	0.00	0.000
2	34.000	6.16203	4.31470	9.27436	4.72198	8.33333	-361.08	83.69	12.9	50.58755	29.81	0.000
3	32.000	4.59921	3.20040	9.72159	4.19711	8.33333	-653.77	203.01	-31.9	49.37565	115.16	0.000
4	30.000	3.79269	2.65567	10.16726	3.74940	8.33333	-1023.50	395.40	0.2	50.00219	234.82	0.000
5	28.000	3.33994	2.33865	10.56590	3.42026	8.33333	-1470.28	628.69	178.2	51.07019	388.78	0.000
6	26.000	3.05640	2.14012	10.90865	3.18050	8.33333	-1994.12	931.78	567.6	52.17775	577.04	0.000
7	24.000	2.66405	2.00343	11.20019	3.00199	8.33333	-2595.01	1293.99	1232.7	53.20534	799.60	0.000
8	22.000	2.42579	1.90862	11.44828	2.86549	8.33333	-3272.95	1714.83	2236.1	54.12370	1056.46	0.000
9	20.000	2.26201	1.83595	11.66045	2.75847	8.33333	-4027.94	2193.93	3640.0	54.93376	1347.63	0.000
10	18.000	2.14149	1.77957	11.84312	2.67270	8.33333	-4859.98	2730.99	5505.7	55.64599	1673.10	0.000
11	16.000	2.07734	1.73465	12.00152	2.60265	8.33333	-5769.08	3325.79	7893.9	56.27291	2032.88	0.000
12	14.000	2.02513	1.69810	12.13982	2.54451	8.33333	-6755.22	3978.12	10865.2	56.82647	2426.95	0.000
13	12.000	2.38190	1.66782	12.26139	2.49558	8.33333	-7818.42	4687.80	14479.5	57.31722	2855.33	0.000
14	10.000	2.14555	1.64237	12.36892	2.45389	8.33333	-8958.67	5454.71	18796.6	57.75420	3318.01	0.000
15	8.000	2.31462	1.62071	12.46458	2.41799	8.33333	-10175.97	6278.69	23875.9	58.14502	3815.00	0.000
16	6.000	2.28801	1.60208	12.55013	2.38680	8.33333	-11470.33	7159.65	29776.5	58.49606	4346.29	0.000
17	4.000	2.26489	1.58589	12.62703	2.35946	8.33333	-12841.73	8097.48	36587.6	58.81266	4911.88	0.000
18	2.000	2.24465	1.57172	12.69645	2.33534	8.33333	-14290.19	9092.08	44277.4	59.09933	5511.77	0.000
19	Base joint	2.23761	1.56679	12.73577	2.31487	8.33132	-15827.96	10102.17	58191.5	59.39716	6497.16	0.000
Required:		1.300	1.000	1.100	1.100	1.100						



Seismic #1 Combination - Sustained accelerations analysis (Stresses)

Joint		Cracking		Normal stresses		Allowable normal stress		Shear stresses over the ligament			
ID	U/S elevation (m)	Upstream (% of joint)	Downstream (% of joint)	Upstream (kPa)	Downstream (kPa)	Tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at (% of joint)	Downstream (kPa)
1	36.000			-27.475	-37.506	0.000	-27270.000	3.663	4.188	26.134	0.000
2	34.000			-72.742	-54.177	0.000	-27270.000	9.699	-3.717	35.612	40.137
3	32.000			-112.792	-65.668	0.000	-27270.000	15.039	4.831	32.554	48.650
4	30.000			-143.935	-84.435	0.000	-27270.000	19.191	13.896	24.806	62.553
5	28.000			-169.436	-107.969	0.000	-27270.000	22.592	21.289	12.966	79.988
6	26.000			-191.267	-134.649	0.000	-27270.000	25.502	99.754	100.000	99.754
7	24.000			-210.613	-163.476	0.000	-27270.000	28.082	121.110	100.000	121.110
8	22.000			-228.215	-193.815	0.000	-27270.000	30.429	143.587	100.000	143.587
9	20.000			-244.554	-225.251	0.000	-27270.000	32.607	166.876	100.000	166.876
10	18.000			-259.955	-257.503	0.000	-27270.000	34.661	190.770	100.000	190.770
11	16.000			-274.644	-290.376	0.000	-27270.000	36.619	215.123	100.000	215.123
12	14.000			-288.780	-323.729	0.000	-27270.000	38.504	239.833	100.000	239.833
13	12.000			-302.481	-357.459	0.000	-27270.000	40.331	264.822	100.000	264.822
14	10.000			-315.836	-391.490	0.000	-27270.000	42.112	290.033	100.000	290.033
15	8.000			-328.911	-425.763	0.000	-27270.000	43.855	315.425	100.000	315.425
16	6.000			-341.757	-460.234	0.000	-27270.000	45.568	340.962	100.000	340.962
17	4.000			-354.414	-494.868	0.000	-27270.000	47.255	366.620	100.000	366.620
18	2.000			-366.915	-529.636	0.000	-27270.000	48.922	392.378	100.000	392.378
19	Base joint			-378.639	-565.910	2727.000	-27270.000	50.485	419.252	100.000	419.252

Seismic #1 Combination - Sustained accelerations analysis (Stability)

Joint		Safety factors					Resultants over ligament				Final uplift	Rock wedge
(kPa)	U/S elevation (m)	Sliding		Overturning		Uplifting	Normal (kN)	Shear (kN)	Moment (kN-m)	Position % of joint	Uplift (kN)	Resistance (kN)
		Peak	Residual	toward U/S	toward D/S							
1	36.000	14.91621	10.44444	17.13209	11.21949	16.66667	-152.71	14.62	18.5	52.57289	0.00	0.000
2	34.000	12.43547	8.70741	17.61988	9.39127	16.66667	-385.70	44.30	-57.1	47.56219	29.81	0.000
3	32.000	8.07060	5.65110	18.20732	7.96047	16.66667	-698.34	123.58	-240.5	45.59900	115.16	0.000
4	30.000	6.14681	4.30404	18.88295	6.72729	16.66667	-1093.28	254.01	-454.5	45.65764	234.82	0.000
5	28.000	5.16194	3.61443	19.53020	5.86352	16.66667	-1570.53	434.52	-656.7	46.30699	388.78	0.000
6	26.000	4.57769	3.20533	20.11035	5.26356	16.66667	-2130.08	664.54	-806.1	47.10470	577.04	0.000
7	24.000	4.19470	2.93716	20.61837	4.83376	16.66667	-2771.94	943.75	-862.7	47.89992	799.60	0.000
8	22.000	3.92563	2.74875	21.06038	4.51484	16.66667	-3496.10	1271.89	-786.9	48.64149	1056.46	0.000
9	20.000	3.72683	2.60956	21.44524	4.27059	16.66667	-4302.57	1648.78	-539.7	49.31520	1347.63	0.000
10	18.000	3.57428	2.50274	21.78161	4.07842	16.66667	-5191.35	2074.27	-82.3	49.92102	1673.10	0.000
11	16.000	3.45369	2.41830	22.07709	3.92374	16.66667	-6162.42	2548.25	623.8	50.46406	2032.88	0.000
12	14.000	3.35609	2.34996	22.33808	3.79683	16.66667	-7215.81	3070.61	1616.8	50.95097	2426.95	0.000
13	12.000	3.27555	2.29356	22.56987	3.69101	16.66667	-8351.50	3641.27	2934.8	51.38844	2855.33	0.000
14	10.000	3.20801	2.24627	22.77684	3.60152	16.66667	-9569.49	4260.17	4615.8	51.78261	3318.01	0.000
15	8.000	3.15059	2.20607	22.96256	3.52493	16.66667	-10869.79	4927.22	6697.5	52.13894	3815.00	0.000
16	6.000	3.10122	2.17149	23.13001	3.45870	16.66667	-12252.39	5642.38	9217.6	52.46215	4346.29	0.000
17	4.000	3.05832	2.14146	23.28166	3.40089	16.66667	-13717.30	6405.59	12213.6	52.75631	4911.88	0.000
18	2.000	3.02072	2.11513	23.41955	3.35002	16.66667	-15264.52	7216.81	15723.1	53.02494	5511.77	0.000
19	Base joint	3.00119	2.10145	23.49401	3.30514	16.66263	-16907.44	8045.59	20001.2	53.30442	6497.16	0.000
Required:		1.300	1.000	1.100	1.100	1.100						

Seismic #2 Combination - Peak accelerations analysis (Stresses):

Joint		Cracking		Normal stresses		Allowable normal stress		Shear stresses over the ligament			
ID	U/S elevation (m)	Upstream (% of joint)	Downstream (% of joint)	Upstream (kPa)	Downstream (kPa)	Tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at (% of joint)	Downstream (kPa)
1	36.000			-16.043	-39.260	0.000	-27270.000	2.139	15.040	48.083	0.000
2	34.000			-36.745	-71.271	0.000	-27270.000	4.899	52.801	100.000	52.801
3	32.000			-51.826	-100.055	0.000	-27270.000	6.910	74.125	100.000	74.125
4	30.000			-57.479	-136.878	0.000	-27270.000	7.664	101.406	100.000	101.406
5	28.000			-57.612	-178.478	0.000	-27270.000	7.682	132.225	100.000	132.225
6	26.000			-54.430	-222.945	0.000	-27270.000	7.257	165.168	100.000	165.168
7	24.000			-49.184	-269.189	0.000	-27270.000	6.558	199.427	100.000	199.427
8	22.000			-42.607	-316.567	0.000	-27270.000	5.681	234.527	100.000	234.527
9	20.000			-35.151	-364.683	0.000	-27270.000	4.687	270.173	100.000	270.173
10	18.000			-27.104	-413.286	0.000	-27270.000	3.614	306.181	100.000	306.181
11	16.000			-18.656	-462.211	0.000	-27270.000	2.487	342.427	100.000	342.427
12	14.000			-9.935	-511.349	0.000	-27270.000	1.325	378.830	100.000	378.830
13	12.000			-1.028	-560.623	0.000	-27270.000	0.137	415.335	100.000	415.335
14	10.000	1.32912		0.000	-610.088	0.000	-27270.000				
15	8.000	2.66365		0.000	-659.952	0.000	-27270.000				
16	6.000	3.84758		0.000	-709.858	0.000	-27270.000				
17	4.000	4.90288		-0.001	-760.057	0.000	-27270.000				
18	2.000	5.84763		0.000	-810.412	0.000	-27270.000				
19	Base joint			54.272	-858.108	2727.000	-27270.000	7.236	635.725	100.000	635.725

Seismic #2 Combination - Peak accelerations analysis (Stability):

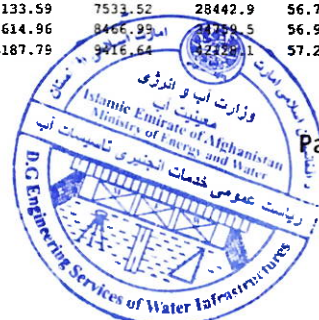
Joint		Safety factors					Resultants over ligament				Final uplift	Rock wedge
ID	U/S elevation (m)	Sliding		Overturning		Uplifting	Normal (kN)	Shear (kN)	Moment (kN-m)	Position % of joint	Uplift (kN)	Resistance (kN)
		Peak	Residual	toward U/S	toward D/S							
1	36.000	1.00819	2.06667	5.46542	3.36595	5.00000	-129.96	48.74	42.7	56.99680	0.00	0.000
2	34.000	3.44180	2.40897	5.93616	2.85896	5.00000	-328.26	136.21	106.3	55.32729	29.81	0.000
3	32.000	2.74766	1.92399	6.52730	2.57438	5.00000	-594.33	308.92	246.2	55.29241	115.16	0.000
4	30.000	2.37043	1.65979	6.68098	2.45790	5.00000	-930.45	560.59	606.6	56.80865	234.82	0.000
5	28.000	2.15066	1.50594	6.96018	2.19670	5.00000	-1336.62	887.58	1291.4	58.53254	388.78	0.000
6	26.000	2.00934	1.40737	7.22797	2.08193	5.00000	-1812.84	1288.10	2399.4	60.12554	577.04	0.000
7	24.000	1.91302	1.33965	7.47291	1.99432	5.00000	-2359.10	1760.98	4026.5	61.51715	799.60	0.000
8	22.000	1.84119	1.29062	7.60341	1.92692	5.00000	-2975.41	2305.42	6266.8	62.71249	1056.46	0.000
9	20.000	1.79045	1.25709	7.74654	1.87483	5.00000	-3661.76	2920.79	9212.9	63.73618	1347.63	0.000
10	18.000	1.74950	1.22501	7.86773	1.83117	5.00000	-4418.17	3606.62	12956.2	64.61512	1673.10	0.000
11	16.000	1.71692	1.20270	7.97119	1.79607	5.00000	-5244.62	4362.51	17587.5	65.37344	2032.88	0.000
12	14.000	1.69048	1.18169	8.06052	1.76730	5.00000	-6141.11	5188.12	23196.5	66.03140	2426.95	0.000
13	12.000	1.66867	1.16841	8.13800	1.74292	5.00000	-7107.66	6083.18	29872.5	66.60564	2855.33	0.000
14	10.000	1.65042	1.15563	8.20575	1.72218	5.00000	-8144.25	7047.43	36239.9	67.10969	3318.01	0.000
15	8.000	1.63497	1.14482	8.26539	1.70436	5.00000	-9250.88	8080.66	43231.3	67.55455	3815.00	0.000
16	6.000	1.62176	1.13557	8.31618	1.68991	5.00000	-10427.57	9182.68	51059.1	67.94918	4346.29	0.000
17	4.000	1.61016	1.12759	8.36517	1.67540	5.00000	-11674.30	10353.33	59771.3	68.30094	4911.88	0.000
18	2.000	1.60046	1.12065	8.40721	1.66353	5.00000	-12991.08	11592.44	69416.6	68.61587	5511.77	0.000
19	Base joint	1.59987	1.11024	8.42972	1.65314	4.99879	-14388.66	12844.28	97445.2	68.91720	6497.16	0.000
Required:		1.300	1.000	1.100	1.100	1.100						

Seismic #2 Combination - Sustained accelerations analysis (Stresses)

Joint		Cracking		Normal stresses		Allowable normal stress		Shear stresses over the ligament			
ID	U/S elevation (m)	Upstream (%) of joint	Downstream (%) of joint	Upstream (kPa)	Downstream (kPa)	Tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at (% of joint)	Downstream (kPa)
1	36.000			-24.209	-38.007	0.000	-27270.000	3.228	7.092	42.467	0.000
2	34.000			-62.457	-59.061	0.000	-27270.000	8.328	0.769	29.544	43.755
3	32.000			-95.373	-75.493	0.000	-27270.000	12.716	10.061	19.396	55.928
4	30.000			-119.234	-99.419	0.000	-27270.000	15.898	73.654	100.000	73.654
5	28.000			-137.486	-128.114	0.000	-27270.000	18.332	94.913	100.000	94.913
6	26.000			-152.171	-159.877	0.000	-27270.000	20.289	118.444	100.000	118.444
7	24.000			-164.490	-193.680	0.000	-27270.000	21.932	143.487	100.000	143.487
8	22.000			-175.184	-228.887	0.000	-27270.000	23.358	169.569	100.000	169.569
9	20.000			-184.725	-265.088	0.000	-27270.000	24.630	196.389	100.000	196.389
10	18.000			-193.426	-302.012	0.000	-27270.000	25.790	223.744	100.000	223.744
11	16.000			-201.504	-339.472	0.000	-27270.000	26.867	251.496	100.000	251.496
12	14.000			-209.110	-377.334	0.000	-27270.000	27.881	279.546	100.000	279.546
13	12.000			-216.352	-415.506	0.000	-27270.000	28.847	307.825	100.000	307.825
14	10.000	1.32912		-223.311	-453.916	0.000	-27270.000	29.775	336.281	100.000	336.281
15	8.000	2.66365		-230.048	-492.512	0.000	-27270.000	30.673	364.875	100.000	364.875
16	6.000	3.84758		-236.609	-531.255	0.000	-27270.000	31.548	393.578	100.000	393.578
17	4.000	4.90288		-243.028	-570.114	0.000	-27270.000	32.404	422.366	100.000	422.366
18	2.000	5.84763		-249.334	-609.066	0.000	-27270.000	33.245	451.223	100.000	451.223
19	Base joint			-254.950	-649.395	2727.000	-27270.000	33.993	481.101	100.000	481.101

Seismic #2 Combination - Sustained accelerations analysis (Stability)

Joint		Safety factors					Resultants over ligament				Final uplift	Rock wedge
ID	U/S elevation (m)	Sliding		Overturning		Uplifting	Normal (kN)	Shear (kN)	Moment (kN-m)	Position % of joint	Uplift (kN)	Resistance (kN)
		Peak	Residual	toward U/S	toward D/S							
1	36.000	8.56889	6.00000	10.46542	6.73170	10.00000	-146.21	24.37	25.4	53.69642	0.00	0.000
2	34.000	7.47486	5.23396	10.94347	5.66003	10.00000	-369.29	70.56	-10.5	49.53428	29.81	0.000
3	32.000	5.40924	3.78759	11.41874	4.98224	10.00000	-668.62	176.53	-101.5	48.06082	115.16	0.000
4	30.000	4.37619	3.06424	11.91040	4.39840	10.00000	-1046.76	341.61	-151.4	48.48965	234.82	0.000
5	28.000	3.80787	2.66630	12.35876	3.97195	10.00000	-1503.70	563.96	-100.1	49.41189	388.78	0.000
6	26.000	3.45629	2.42012	12.74899	3.66382	10.00000	-2039.44	842.70	109.7	50.41158	577.04	0.000
7	24.000	3.21963	2.25441	13.08382	3.43602	10.00000	-2653.99	1177.24	534.2	51.35827	799.60	0.000
8	22.000	3.05037	2.13589	13.37070	3.26281	10.00000	-3347.33	1567.18	1228.5	52.21507	1056.46	0.000
9	20.000	2.92377	2.04724	13.61741	3.12761	10.00000	-4119.48	2012.21	2246.8	52.97767	1347.63	0.000
10	18.000	2.82575	1.97861	13.83082	3.01963	10.00000	-4970.44	2512.08	3643.0	53.65285	1673.10	0.000
11	16.000	2.74778	1.92401	14.01663	2.93168	10.00000	-5900.19	3066.61	5470.6	54.25057	2032.88	0.000
12	14.000	2.68437	1.87962	14.17947	2.85883	10.00000	-6908.75	3675.61	7782.4	54.78092	2426.95	0.000
13	12.000	2.63188	1.84286	14.32309	2.79760	10.00000	-7996.11	4338.96	10631.3	55.25313	2855.33	0.000
14	10.000	2.58776	1.81197	14.45050	2.74551	10.00000	-9162.28	5056.53	14069.7	55.67521	3318.01	0.000
15	8.000	2.55020	1.78567	14.56418	2.70069	10.00000	-10407.24	5828.20	18149.8	56.05401	3815.00	0.000
16	6.000	2.51787	1.76303	14.66611	2.66177	10.00000	-11731.01	6653.89	22923.6	56.39536	4346.29	0.000
17	4.000	2.48977	1.74335	14.75795	2.62768	10.00000	-13133.59	7533.52	28442.9	56.70415	4911.88	0.000
18	2.000	2.46514	1.72611	14.84107	2.59760	10.00000	-14614.96	8466.99	34799.5	56.98454	5511.77	0.000
19	Base joint	2.45507	1.71906	14.88820	2.57106	9.99758	-16187.79	9416.64	42729.1	57.26944	6497.16	0.000
Required:		1.300	1.000	1.100	1.100	1.100						



Post-Seismic Combination (Stresses):

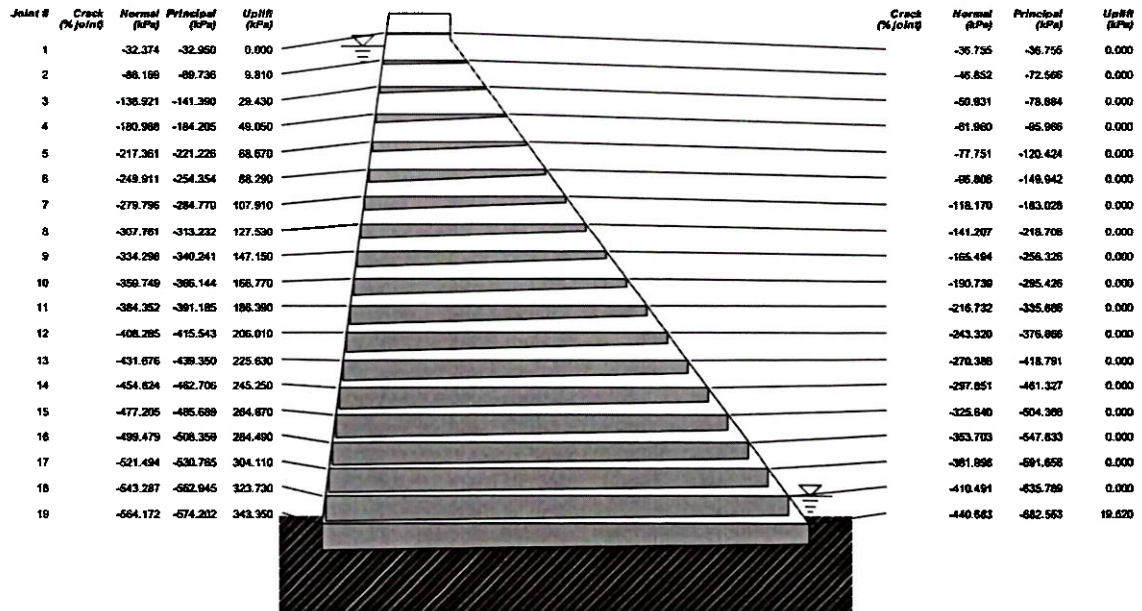
Joint		Cracking		Normal stresses		Allowable normal stress		Shear stresses over the ligament			
ID	U/S elevation (m)	Upstream (% of joint)	Downstream (% of joint)	Upstream (kPa)	Downstream (kPa)	Tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at (% of joint)	Downstream (kPa)
1	1			-32.374	-36.755	0.000	-20010.000				
2	2			-78.359	-46.852	0.000	-20010.000	10.448	-11.205	40.714	34.710
3	3			-109.491	-50.931	0.000	-20010.000	14.599	-5.708	40.608	37.732
4	4			-131.938	-61.960	0.000	-20010.000	17.592	1.574	37.544	45.902
5	5			-148.691	-77.751	0.000	-20010.000	19.826	9.083	31.998	57.601
6	6			-161.621	-96.808	0.000	-20010.000	21.549	15.817	24.255	71.720
7	7			-171.886	-118.170	0.000	-20010.000	22.918	20.995	14.530	87.546
8	8			-180.231	-141.207	0.000	-20010.000	24.031	23.953	3.004	104.612
9	9			-187.148	-165.494	0.000	-20010.000	24.953	122.606	100.000	122.606
10	10			-192.979	-190.739	0.000	-20010.000	25.730	141.308	100.000	141.308
11	11			-197.962	-216.732	0.000	-20010.000	26.395	160.565	100.000	160.565
12	12			-202.275	-243.320	0.000	-20010.000	26.970	180.262	100.000	180.262
13	13			-206.046	-270.388	0.000	-20010.000	27.473	200.316	100.000	200.316
14	14			-209.374	-297.851	0.000	-20010.000	27.916	220.661	100.000	220.661
15	15			-212.335	-325.640	0.000	-20010.000	28.311	241.249	100.000	241.249
16	16			-214.989	-353.703	0.000	-20010.000	28.665	262.039	100.000	262.039
17	17			-217.384	-381.998	0.000	-20010.000	28.984	283.001	100.000	283.001
18	18			-219.557	-410.491	0.000	-20010.000	29.274	304.110	100.000	304.110
19	19			-220.822	-421.063	2001.000	-20010.000	29.443	311.943	100.000	311.943

Post-Seismic Combination (Stability):

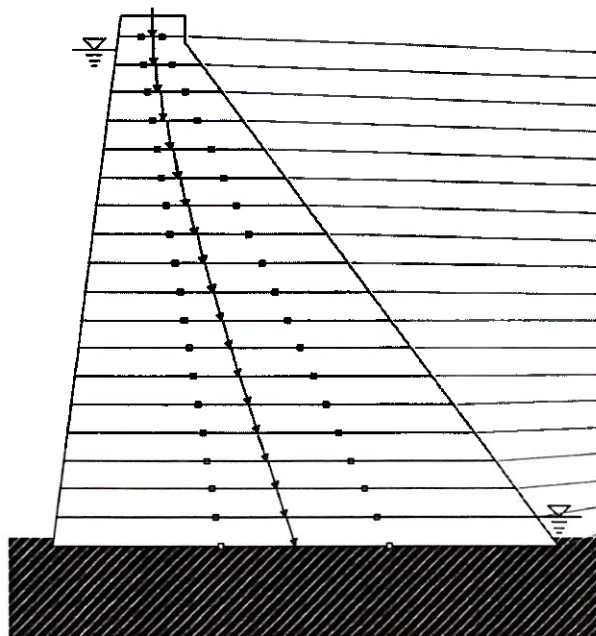
Joint		Safety factors					Resultants over ligament				Final uplift	Rock wedge
ID	U/S elevation (m)	Sliding		Overturning		Uplifting	Normal (kN)	Shear (kN)	Moment (kN-m)	Position % of joint	Uplift (kN)	Resistance (kN)
		Peak	Residual	toward U/S	toward D/S							
1	36.000	> 100	> 100	> 100	> 100	> 100	-162.45	0.00	8.1	51.05612	0.00	0.000
2	34.000	> 100	77.57596	18.53943	11.23704	13.76354	-380.51	4.91	-97.0	45.80608	29.81	0.000
3	32.000	20.30858	14.22022	8.18151	5.27181	6.45094	-627.75	44.15	-298.9	43.91604	115.16	0.000
4	30.000	10.81082	7.56982	6.21625	3.92291	4.95305	-928.25	122.63	-534.6	43.98496	234.82	0.000
5	28.000	7.61775	5.33401	5.42977	3.29321	4.29754	-1282.00	240.34	-757.9	44.77858	388.78	0.000
6	26.000	6.07130	4.25117	5.02353	2.92299	3.92705	-1689.01	397.31	-922.8	45.82008	577.04	0.000
7	24.000	5.17179	3.62133	4.78303	2.67813	3.68795	-2149.28	593.50	-983.1	46.91346	799.60	0.000
8	22.000	4.58760	3.21227	4.62772	2.50391	3.52048	-2662.80	828.95	-892.7	47.97659	1056.46	0.000
9	20.000	4.17924	2.92633	4.52114	2.37356	3.39648	-3229.57	1103.63	-605.4	48.97659	1347.63	0.000
10	18.000	3.87840	2.71569	4.44460	2.27235	3.30088	-3849.61	1417.55	-75.1	49.90273	1673.10	0.000
11	16.000	3.64790	2.55429	4.38766	2.19149	3.22487	-4522.89	1770.71	744.2	50.75435	2032.88	0.000
12	14.000	3.46584	2.42681	4.34408	2.12541	3.16298	-5249.44	2163.11	1898.8	51.53521	2426.95	0.000
13	12.000	3.31849	2.32363	4.30994	2.07041	3.11157	-6029.24	2594.75	3434.8	52.25084	2855.33	0.000
14	10.000	3.19686	2.23847	4.28267	2.02392	3.06819	-6862.29	3065.63	5398.2	52.90723	3318.01	0.000
15	8.000	3.09478	2.16699	4.26052	1.98410	3.03109	-7748.61	3575.75	7835.2	53.51023	3815.00	0.000
16	6.000	3.00792	2.10617	4.24228	1.94963	2.99899	-8688.17	4125.11	10792.0	54.06829	4365.29	0.000
17	4.000	2.93313	2.05380	4.22705	1.91948	2.97094	-9681.00	4713.70	14314.6	54.57733	4911.86	0.000
18	2.000	2.86806	2.00823	4.21422	1.89291	2.94621	-10727.08	5341.55	18449.2	55.05078	5511.77	0.000
19	Base joint	2.73986	1.91847	3.77785	1.83394	2.76843	-11489.75	5989.01	21386.4	55.19329	6487.16	0.000
		2.000	1.100	1.100	1.100	1.100						

2.6 Stability drawings for various load combinations

Usual combination (effective stress analysis)



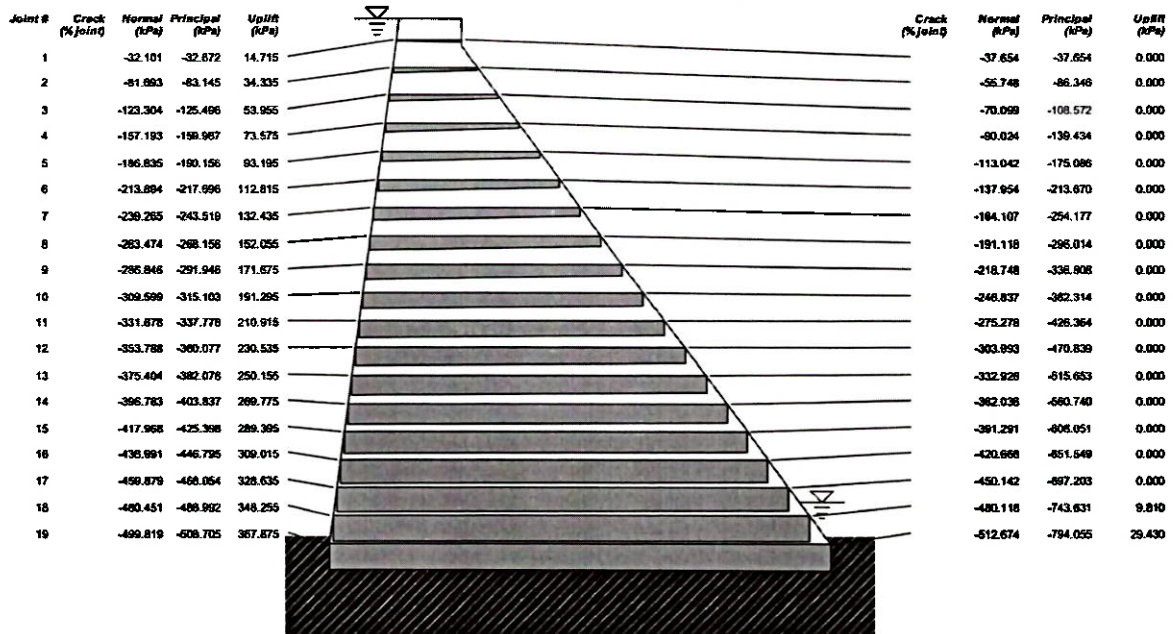
Usual combination (stability analysis)



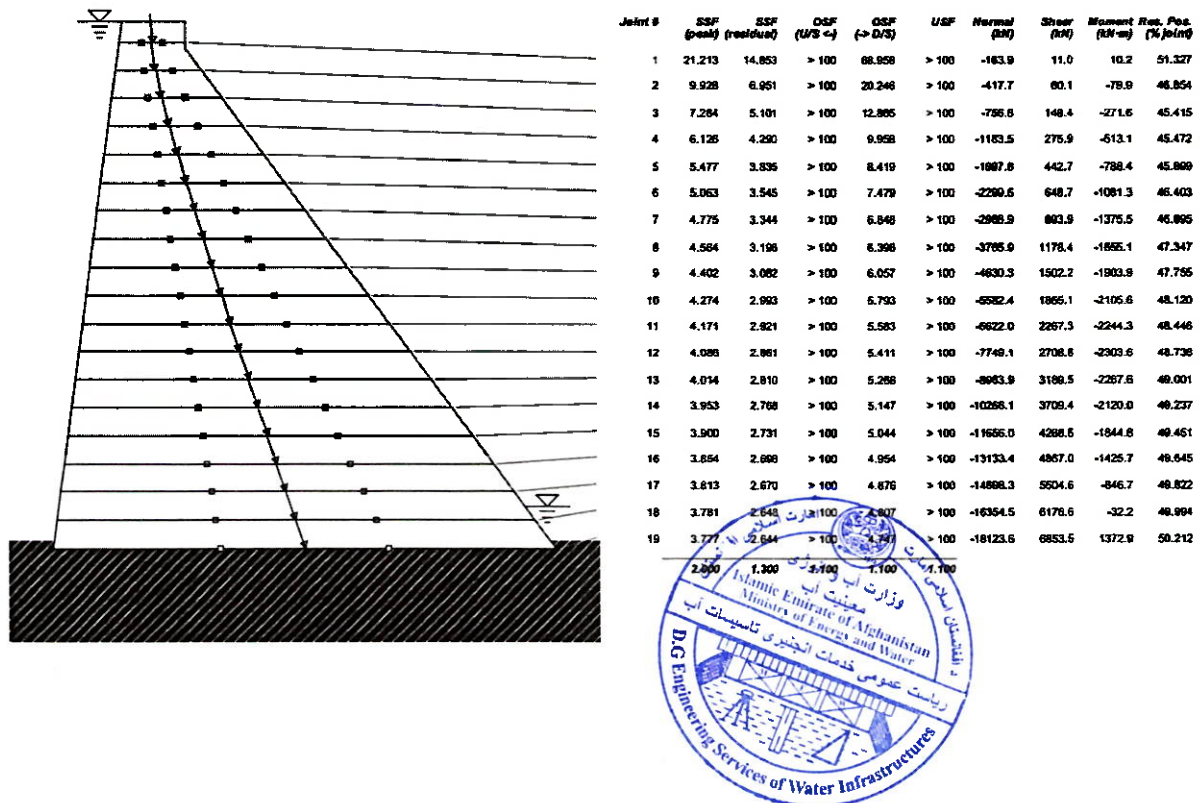
Joint #	S&F (peak)	S&F (residual)	OSF (U/S <)	OSF (> D/S)	USF	Normal (kN)	Shear (kN)	Moment (kN-m)	Res. Pos. (% joint)
1	> 100	> 100	> 100	> 100	> 100	-162.5	0.0	8.1	51.058
2	> 100	83.654	> 100	> 100	> 100	-410.3	4.9	-127.2	44.900
3	24.034	16.829	> 100	77.028	> 100	-742.9	44.1	-449.1	42.276
4	13.546	9.485	> 100	32.990	> 100	-1183.1	122.6	-908.3	41.834
5	9.928	6.952	> 100	20.527	> 100	-1670.8	240.3	-1491.6	42.115
6	8.146	5.704	> 100	15.254	> 100	-2269.0	387.3	-2179.9	42.640
7	7.096	4.909	> 100	12.400	> 100	-2948.9	583.5	-2958.1	43.231
8	6.406	4.487	> 100	10.638	> 100	-3719.3	828.9	-3909.9	43.617
9	5.923	4.147	> 100	9.452	> 100	-4577.2	1103.6	-4719.4	44.371
10	5.564	3.896	> 100	8.803	> 100	-5522.7	1417.5	-5670.2	44.883
11	5.286	3.702	> 100	7.998	> 100	-6655.8	1770.7	-6646.3	45.352
12	5.068	3.649	> 100	7.476	> 100	-7678.4	2183.1	-7831.6	45.781
13	4.890	3.424	> 100	7.085	> 100	-8884.8	2594.7	-8509.9	46.171
14	4.743	3.321	> 100	6.766	> 100	-10180.3	3065.6	-9595.1	46.528
15	4.618	3.234	> 100	6.501	> 100	-11563.6	3575.7	-10481.0	46.854
16	4.513	3.160	> 100	6.278	> 100	-13034.5	4125.1	-11341.4	47.152
17	4.421	3.095	> 100	6.098	> 100	-14502.9	4713.7	-12130.3	47.427
18	4.342	3.040	> 100	5.924	> 100	-16238.8	5341.5	-12831.6	47.679
19	4.289	3.003	> 100	5.781	> 100	-17996.9	5989.0	-13189.1	47.952
	2.000	1.500	1.200	1.000					



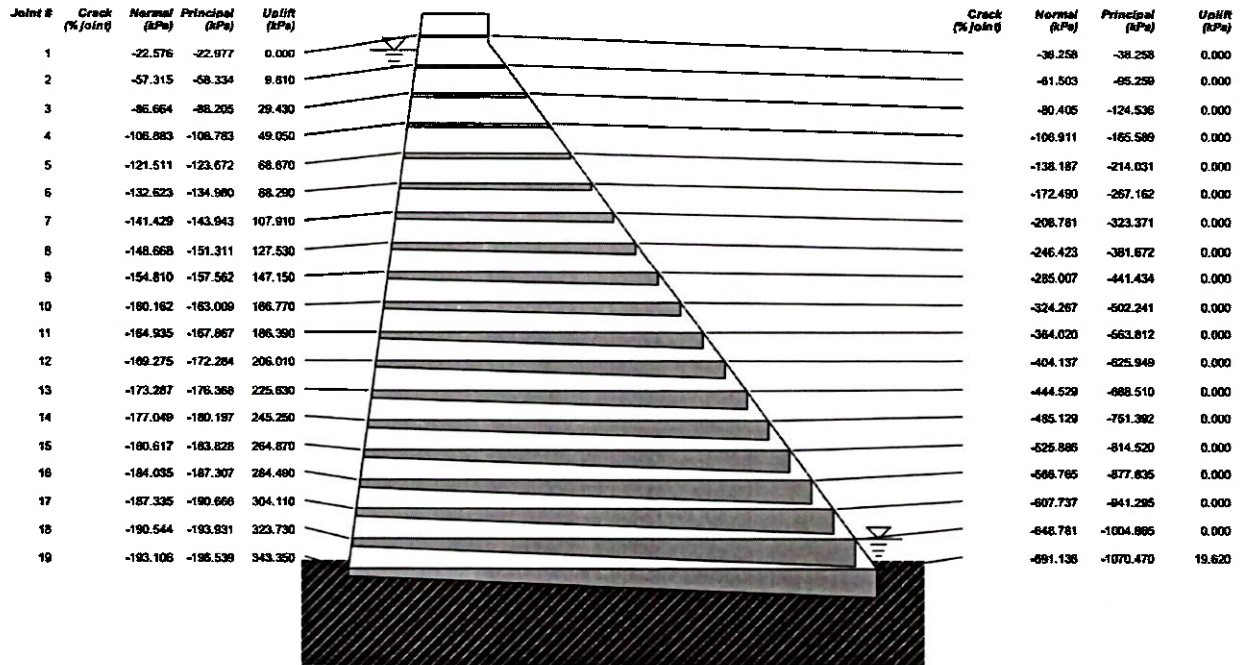
Flood combination (effective stress analysis)



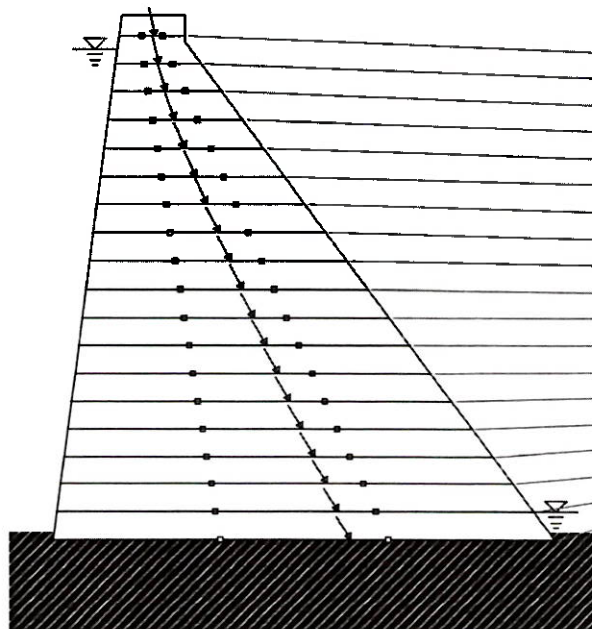
Flood combination (stability analysis)



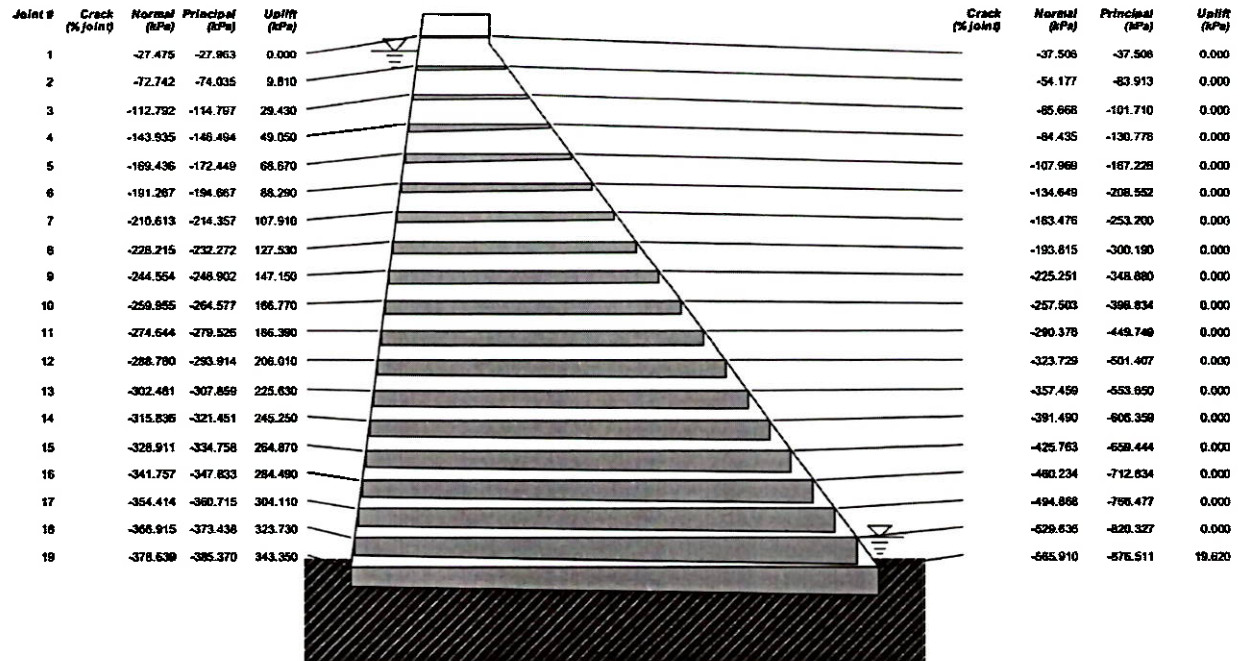
Seismic #1 combination - Peak accelerations (stress analysis) (effective stress analysis)



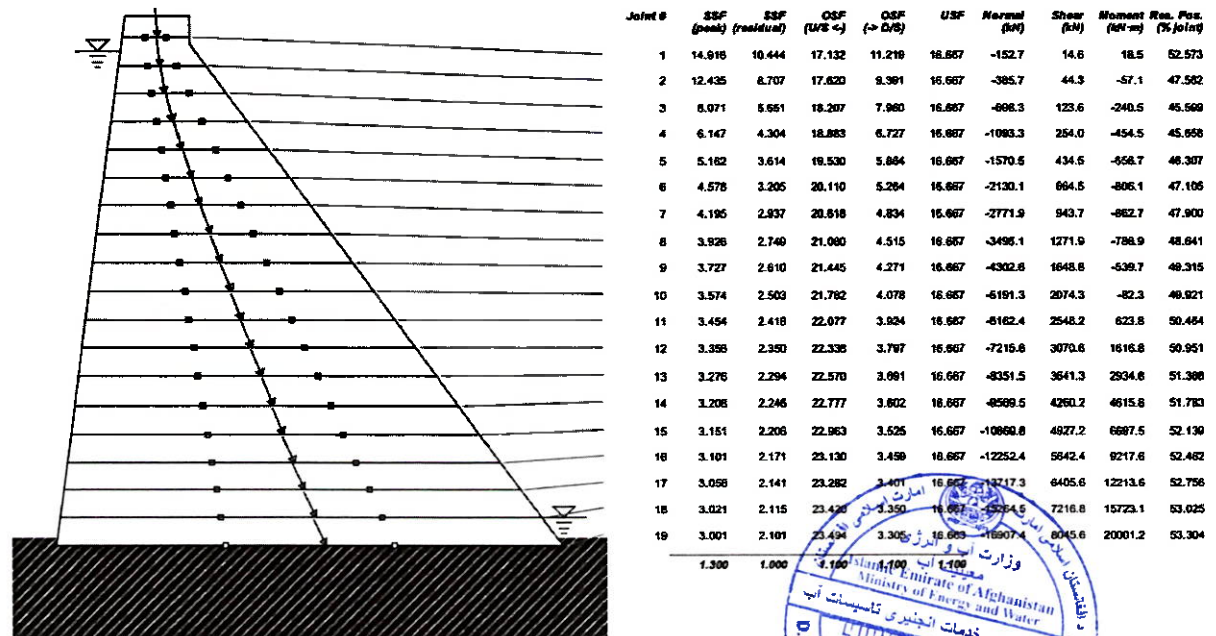
Seismic #1 combination - Peak accelerations (stress analysis) (stability analysis)



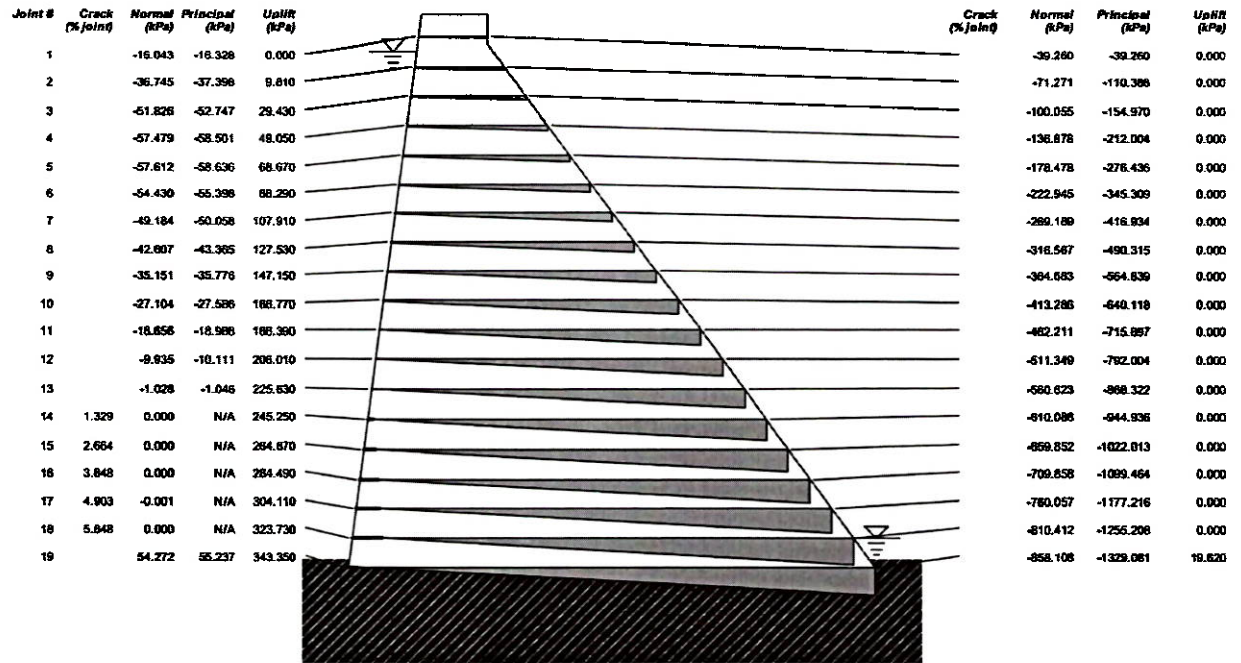
Seismic #1 combination - Sustained accelerations (stability analysis) (effective stress analysis)



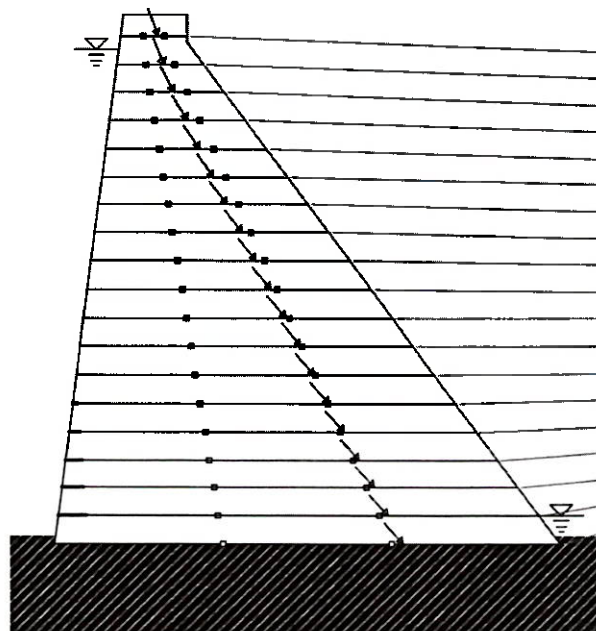
Seismic #1 combination - Sustained accelerations (stability analysis) (stability analysis)



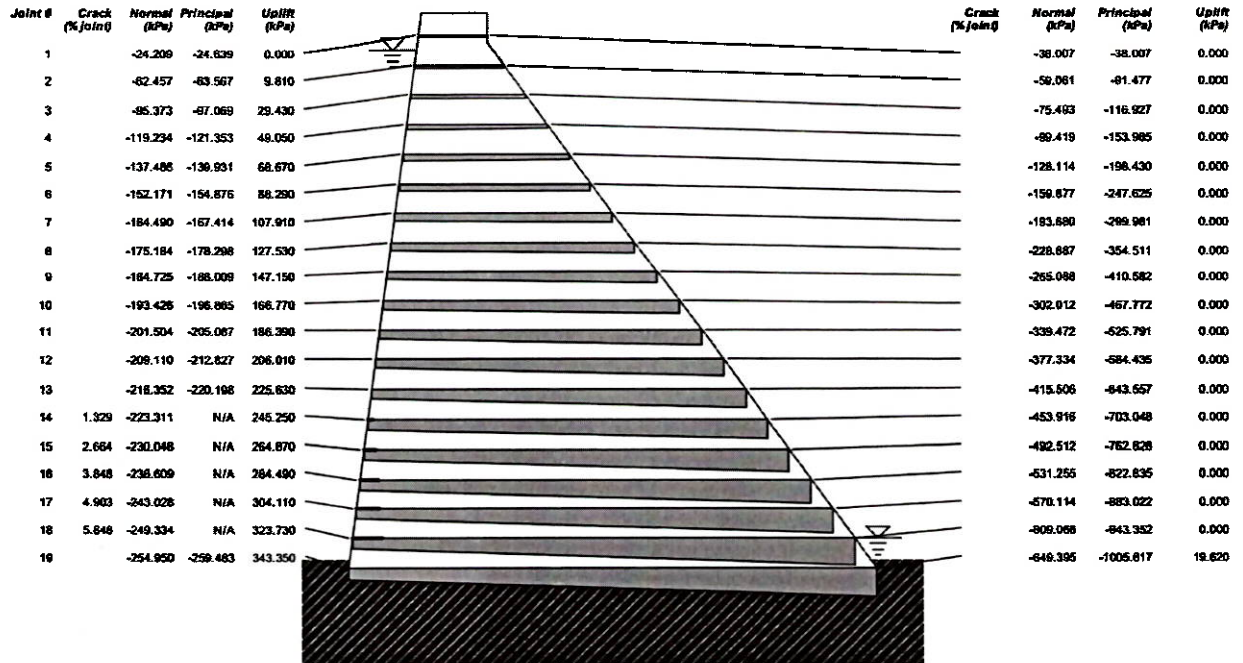
Seismic #2 combination - Peak accelerations (stress analysis) (effective stress analysis)



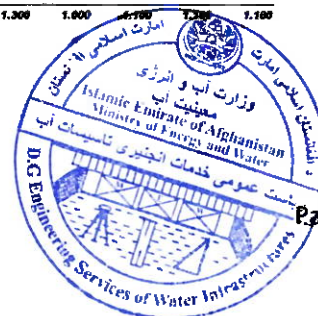
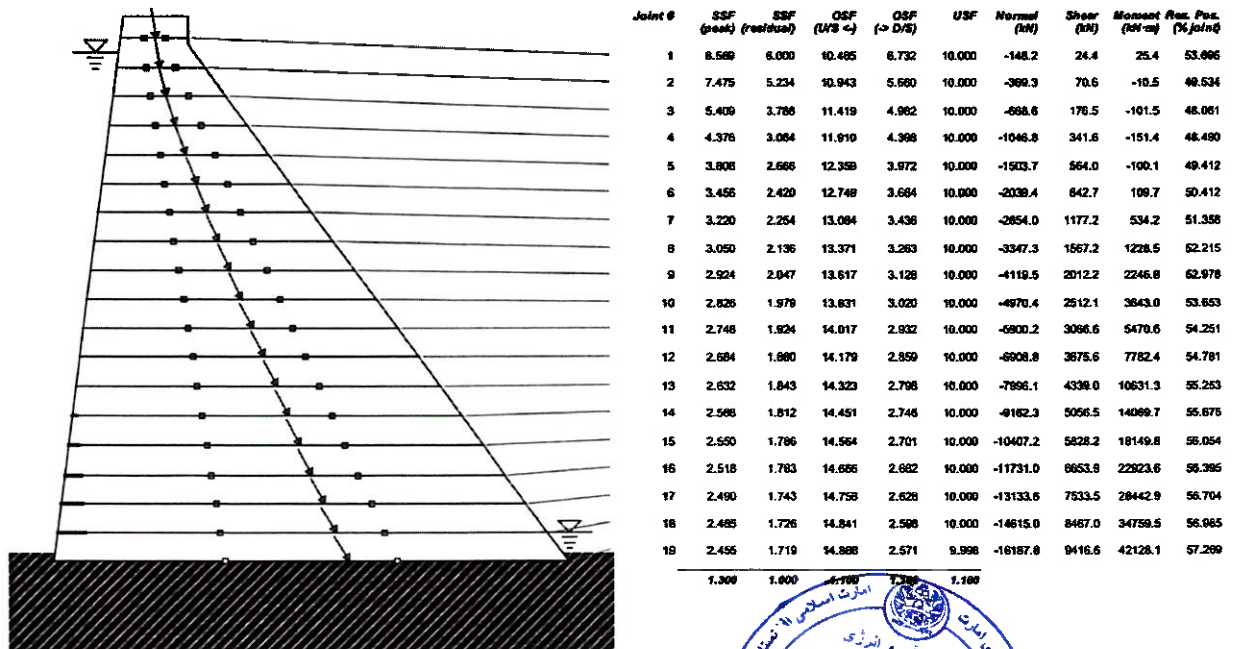
Seismic #2 combination - Peak accelerations (stress analysis) (stability analysis)



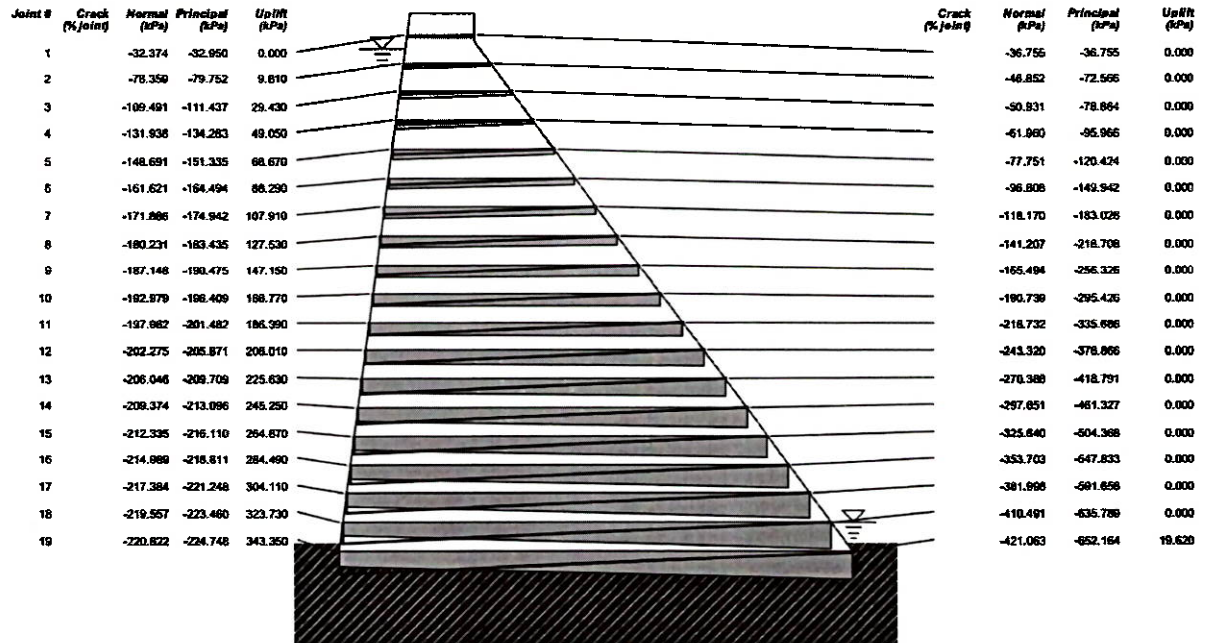
Seismic #2 combination - Sustained accelerations (stability analysis) (effective stress analysis)



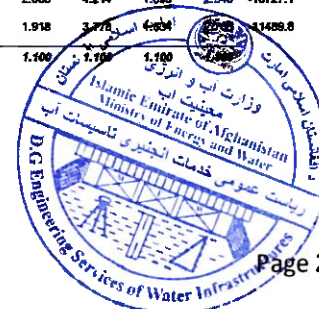
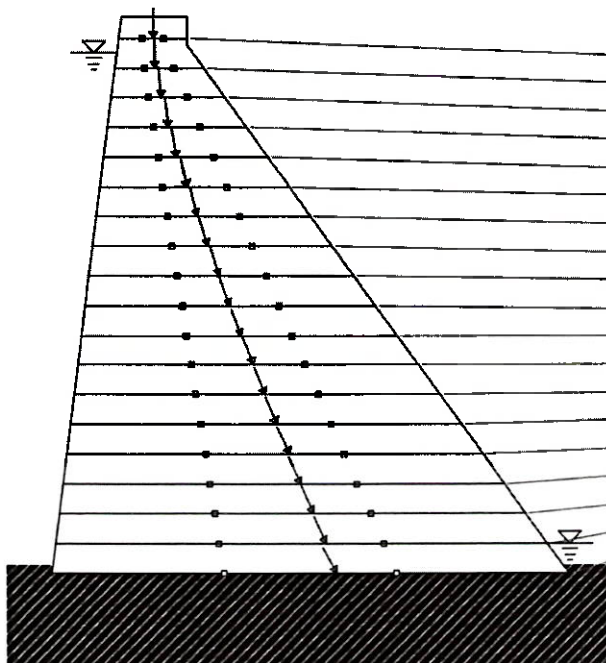
Seismic #2 combination - Sustained accelerations (stability analysis) (stability analysis)



Post-seismic #1 (effective stress analysis)



Post-seismic #1 (stability analysis)



3. Conclusion

Final result show that dam will be safe against sliding and overturning except MCE load condition and developed stresses also within the limit. During the MCE, cracks will be appeared in the bottom of the Upstream side of the dam which are acceptable in the case of MCE.





Islamic Emirate of Afghanistan
Ministry of Water and Energy
Deputy Ministry of Water



General Directorate of Engineering Services of Water Infrastructures

Technical Board

Rehabilitation of Sultan Dam			
Date		October-2023	
Prepared	Checked	Approved	
Hedayatullah Omari		Abdul Ghafor Omari	


Topography Survey Drawings

SULTAN DAM TOP VIEW




0 0.05 0.1 0.2 0.3 0.4 Kilometers

Source: Esri, Maxar, Earthstar Geographics and the GIS User Community



ISLAMIC EMIRATE OF AFGHANISTAN
MINISTRY OF WATER AND ENERGY
DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD




PROJECT NAME:	DRAWING:	PROVINCE	DISTRICT	DATE	SCALE	SURVEYED BY	ENGINEER
Sultan DAM	Top view	Ghazni	Khushk Dargah	OCT.2023	as shown	CHECKED BY	ENG. HEBAWILLAH OMARI
						APPROVED BY	

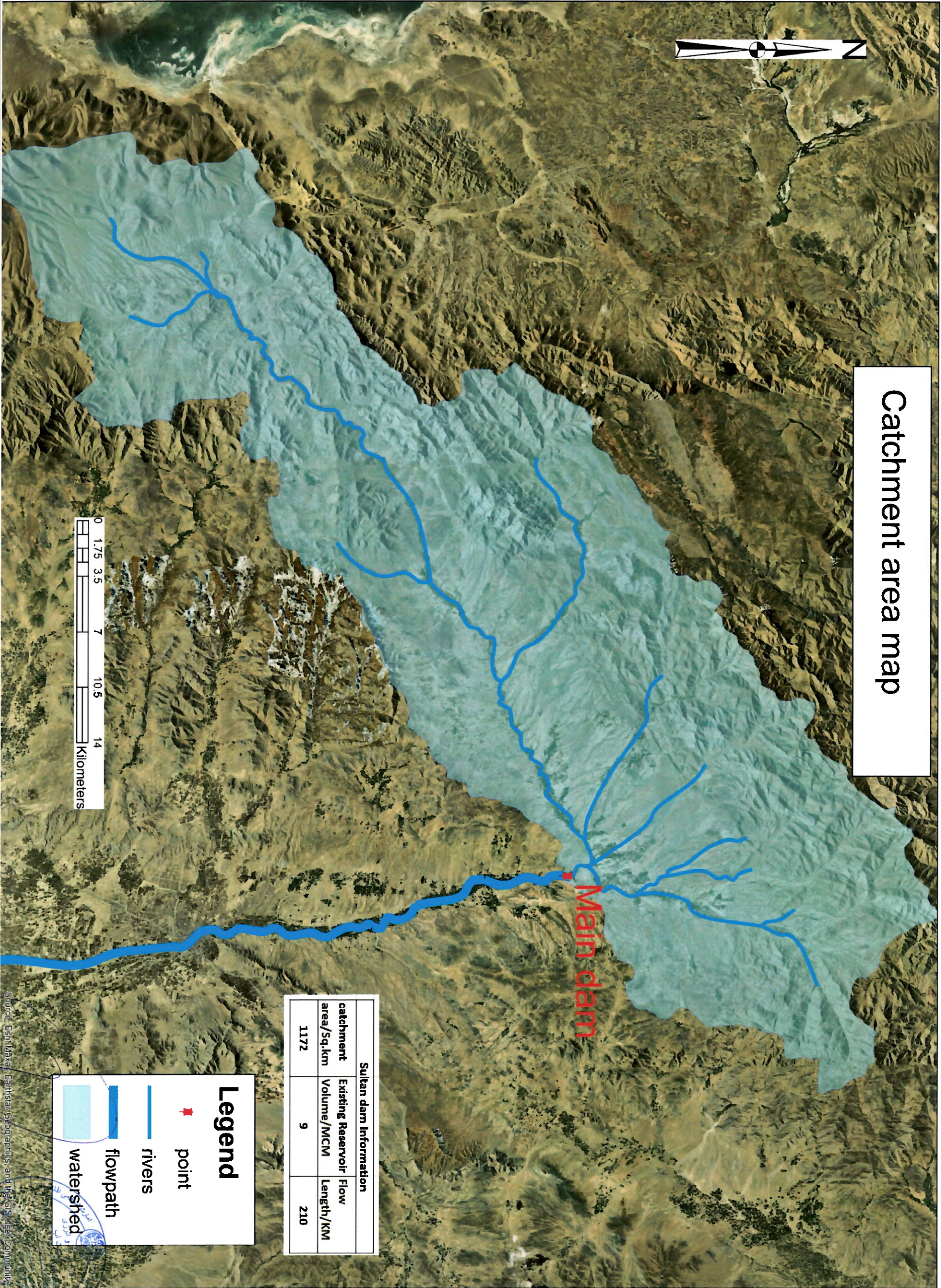
SHEET NO

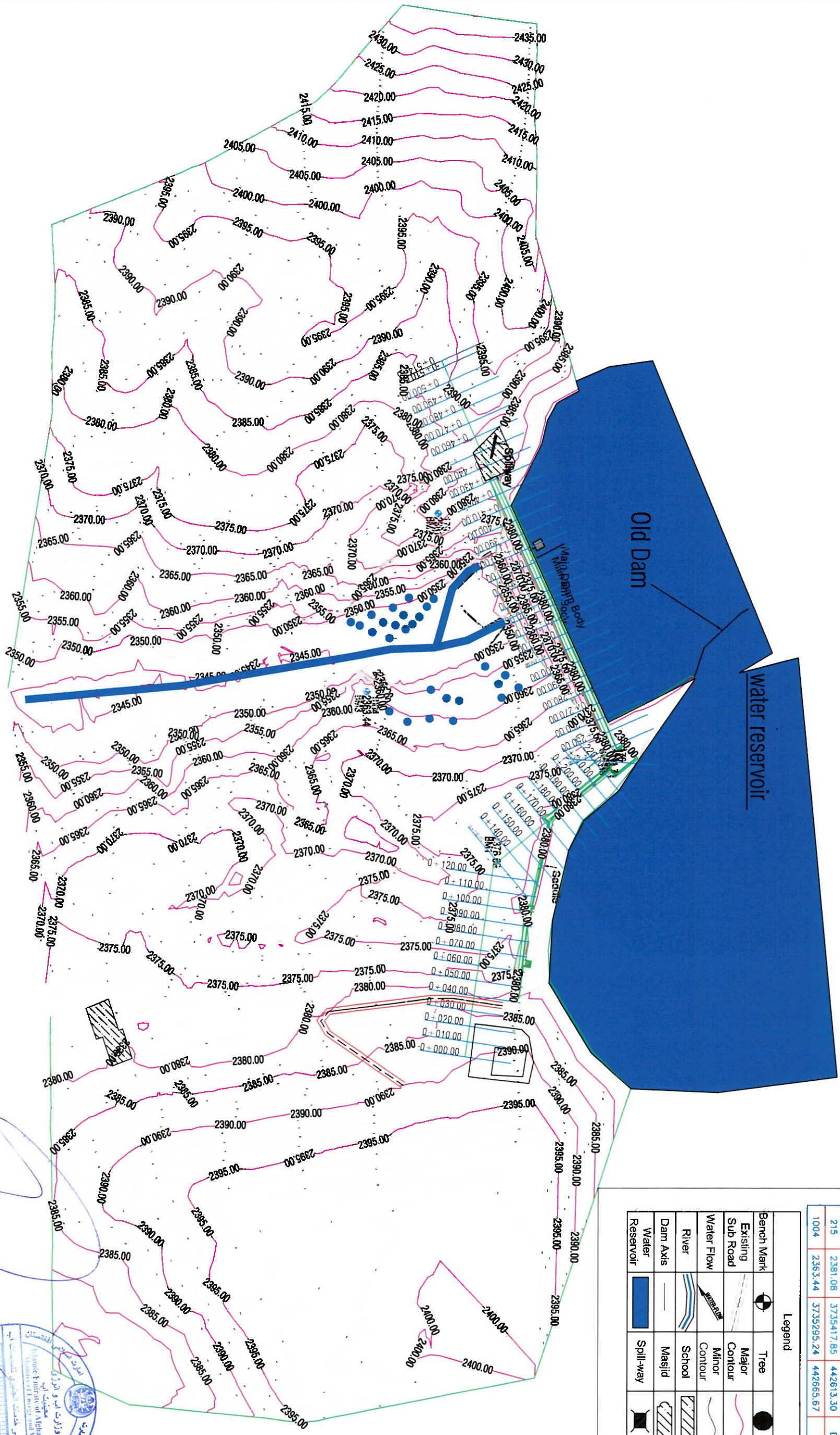
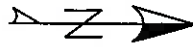
1

11



Catchment area map





Point Table			
Point #	Elevation	Northing	Eastng
1	2378.69	3735380.37	442758.14
7	2380.51	3735381.41	442507.14
61	2380.47	3735377.10	442513.20
179	2381.35	3735341.67	442550.25
215	2381.08	3735417.85	442613.30
1004	2363.44	3735295.24	442665.67

Legend			
Bench Mark		Tree	
Existing Sub Road		Major Contour	
Water Flow		Minor Contour	
River		School	
Dam Axis		Masjid	
Water Reservoir		Spill-way	

ISLAMIC EMIRATE OF AFGHANISTAN
MINISTRY OF WATER AND ENERGY
DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD

PROJECT NAME:	DRAWING:	PROVINCE:	DISTRICT:	DATE:	SCALE:	SURVEYED BY:	ENGINEER/ARCHITECT/PLANNER:
Sultan DAM	Topography Map	Ghazni	Khujja Ormai	OCT 2023	1:2500	CHECKED BY:	
						APPROVED BY:	

PROJECT NAME:		DRAWING:		PROVINCE	DISTRICT	DATE	SCALE	SURVEYED BY	ENGINEER
Sultan DAM		Catchment Area		Ghazni	Khujja Ormai	OCT.2023	1:1500	CHECKED BY	ENG. HEDAYATULLAH QAMARI
								APPROVED BY	
SHEET NO		TCS							
4-1-1									



EX-ELE	Station
2389.15	0+000
2387.98	0+010
2386.89	0+020
2384.12	0+030
2380.11	0+040
2376.01	0+050
2373.79	0+060
2375.57	0+070
2377.93	0+080
2376.81	0+090
2376.60	0+100
2376.41	0+110
2376.45	0+120
2376.94	0+130
2378.41	0+140
2378.29	0+150
2376.99	0+160
2373.81	0+170
2382.11	0+180
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2382.11	0+200
2382.11	0+210
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2381.51	0+240
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2381.04	0+280
2381.04	0+290
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2381.02	0+340
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2381.12	0+380
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2381.15	0+400
2381.16	0+410
2381.17	0+420
2381.17	0+430
2380.09	0+440
2380.69	0+450
2384.81	0+460
2387.70	0+470
2388.94	0+480
2390.06	0+490
2390.19	0+500
2388.05	0+510
	0+520
	0+530

Sultan Dam Profile

PROJECT NAME:
Sultan DAM

DRAWING:
SECTIONS

PROVINCE
Ghazni

DISTRICT
Khyabab District

DATE
OCT.2023

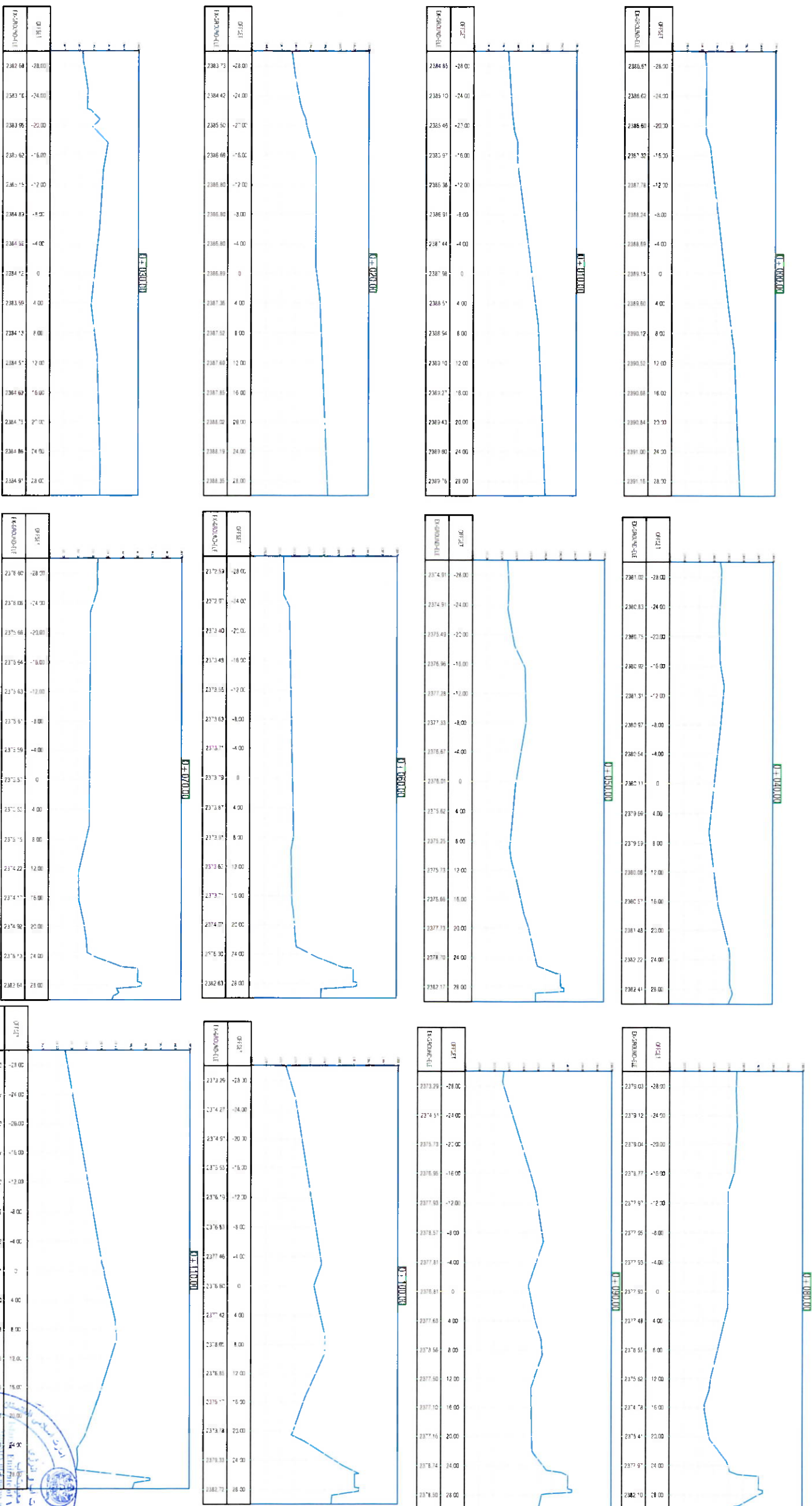
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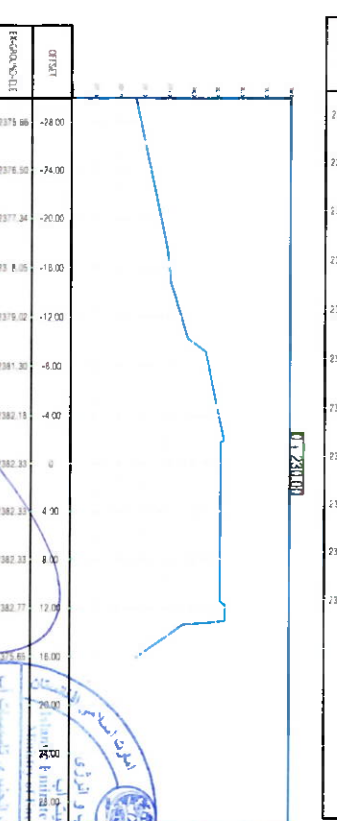
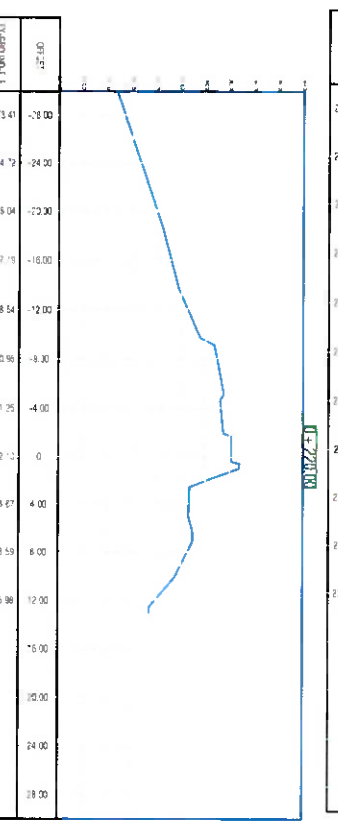
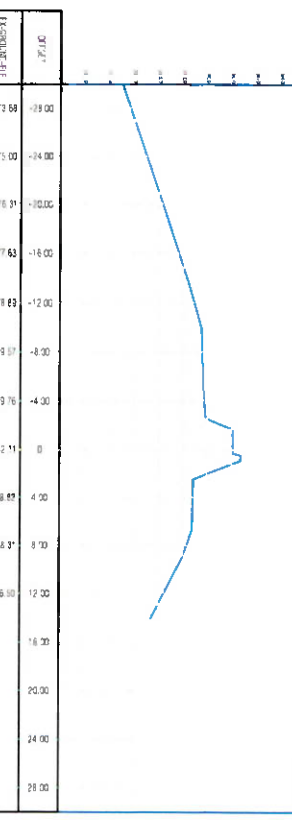
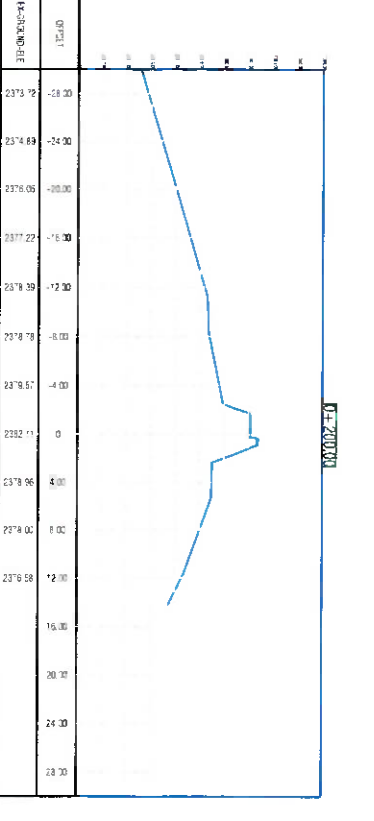
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CHECKED BY

ENG. HEBAATULLAH OMARI

APPROVED BY

SHEET NO
5
11



[illegible]

PROJECT NAME:
Sultan DAM

DRAWING:
SECTIONS

PROVINCE
Ghazni

DISTRICT
Khatla Omari

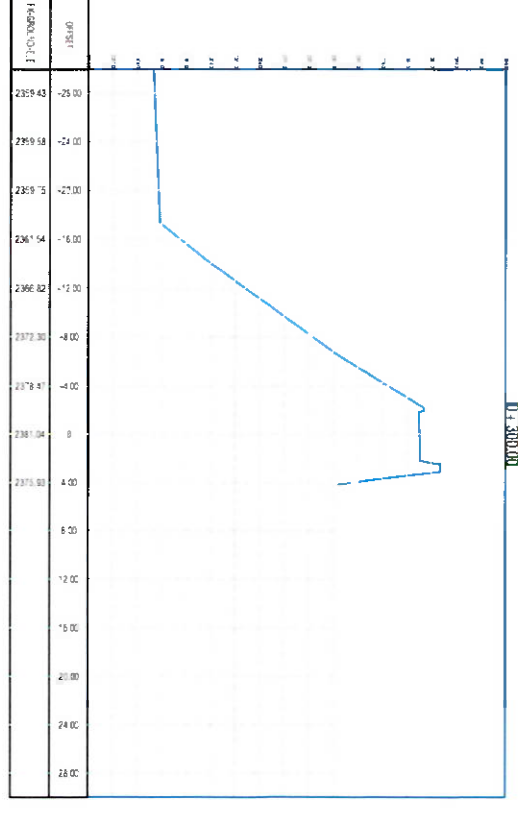
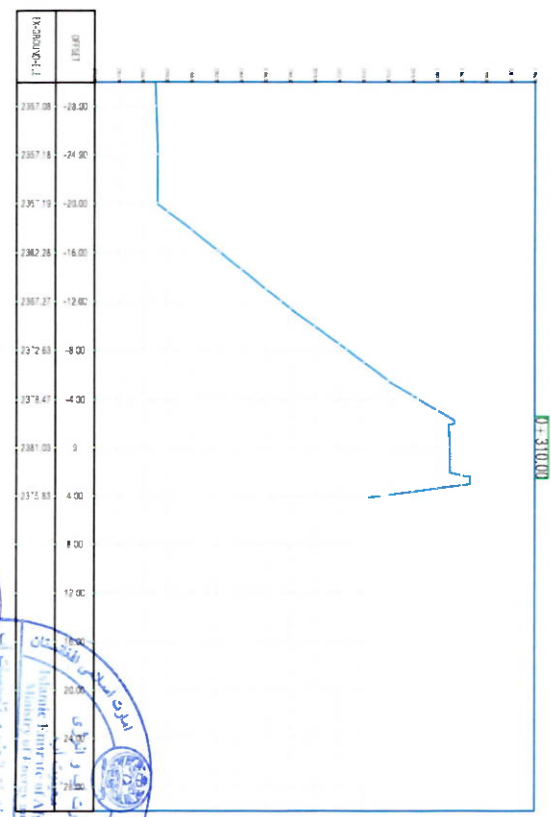
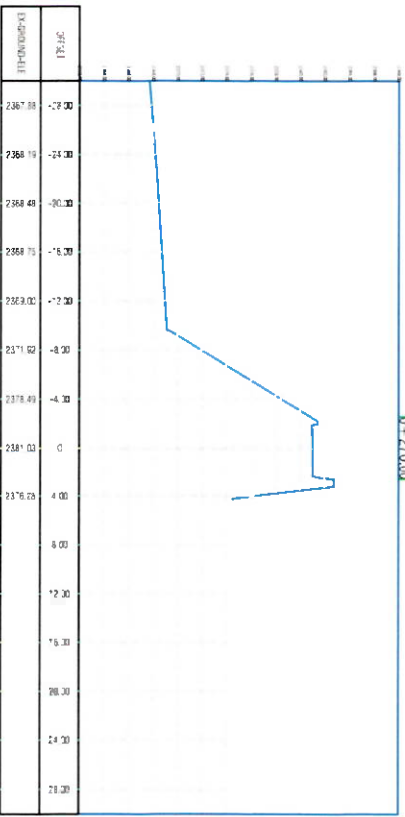
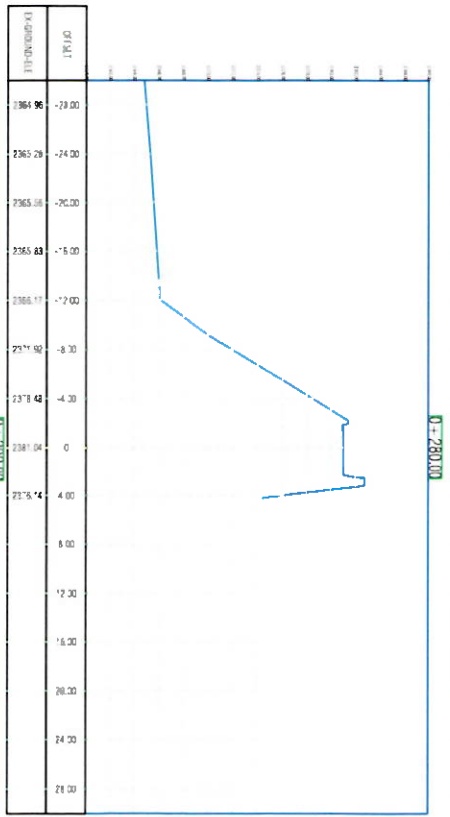
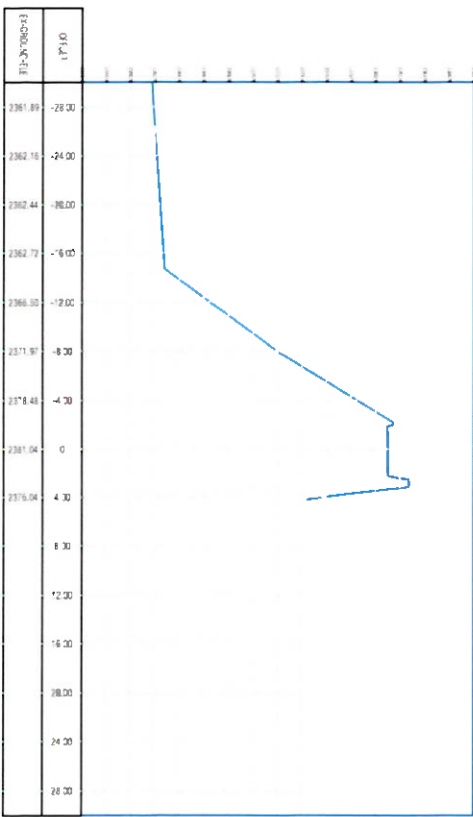
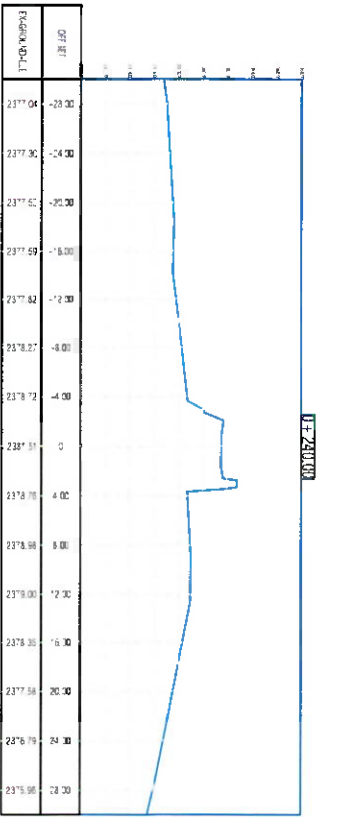
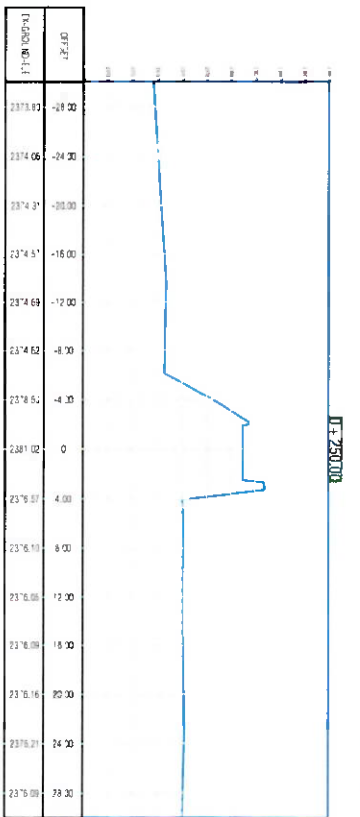
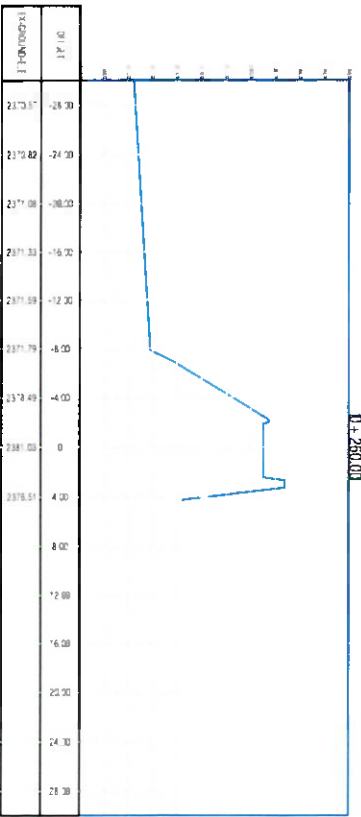
DATE
OCT.2023

SCALE
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SUBMITTED BY
CHECKED BY
APPROVED BY

ENG. HEDAYATULLAH OMARI

SHEET NO
7
11



PROJECT NAME:
Sulani DAM

DRAWING:
SECTIONS

PROVINCE
Ghazni

DISTRICT
Khaaja Omari

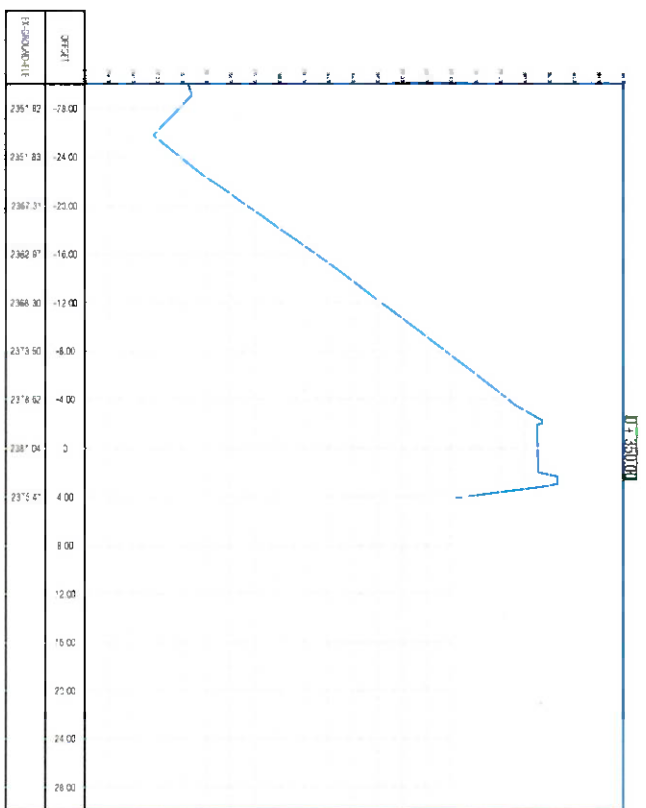
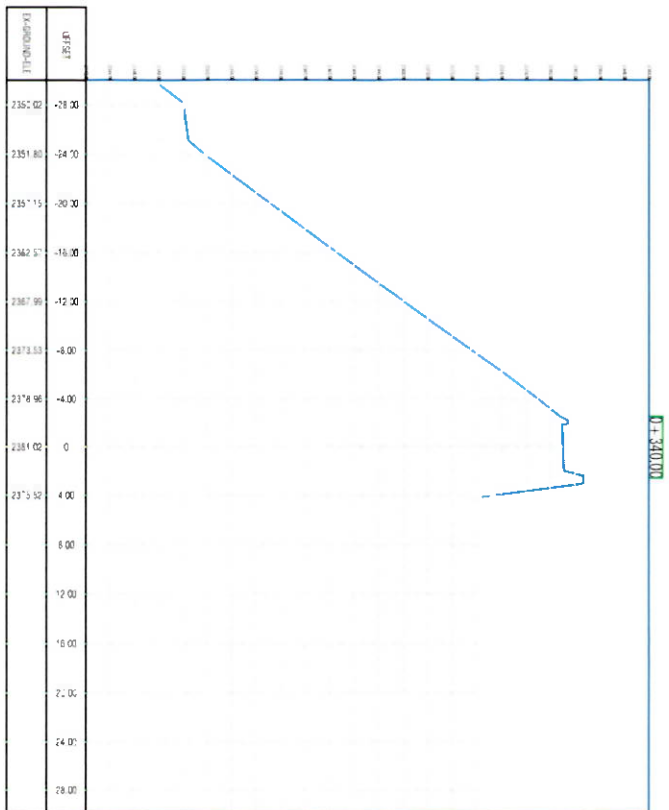
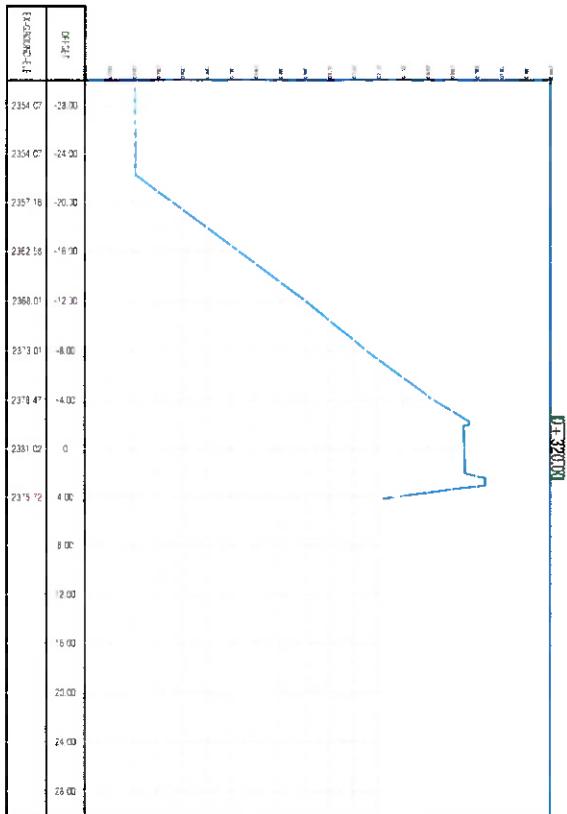
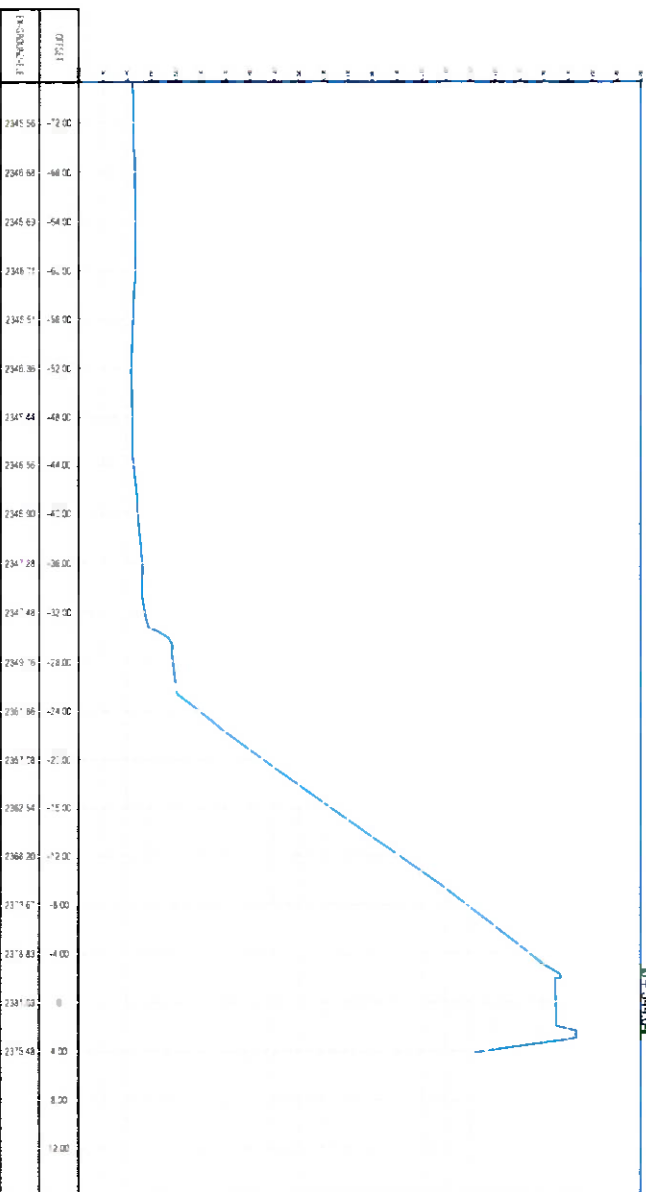
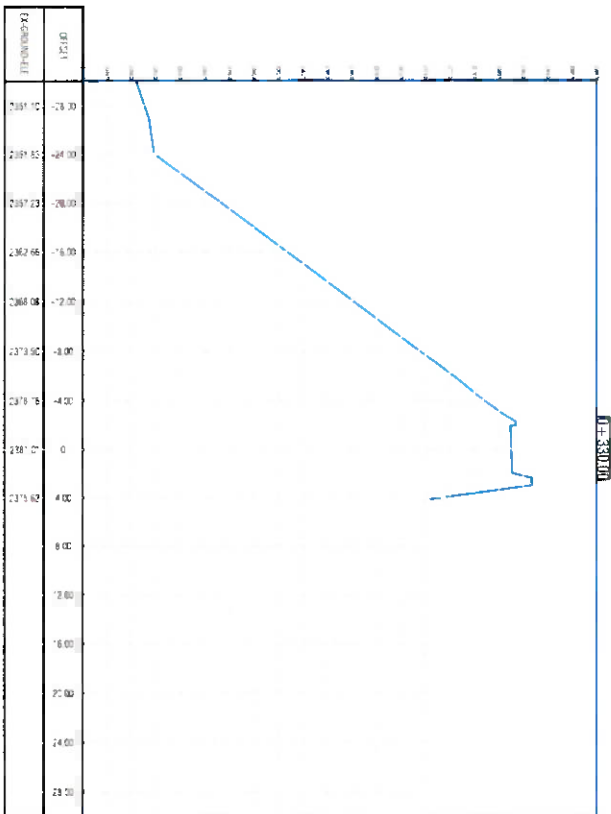
DATE
OCT/2023

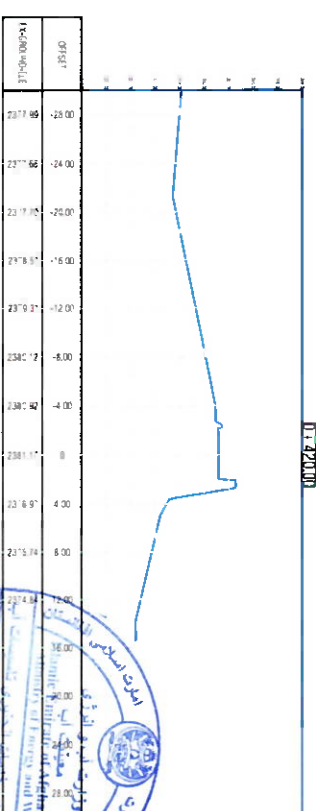
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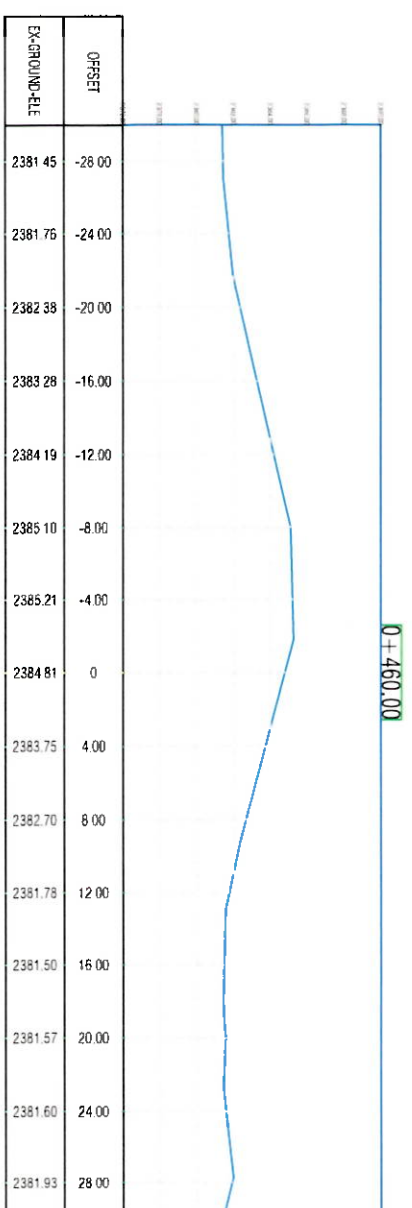
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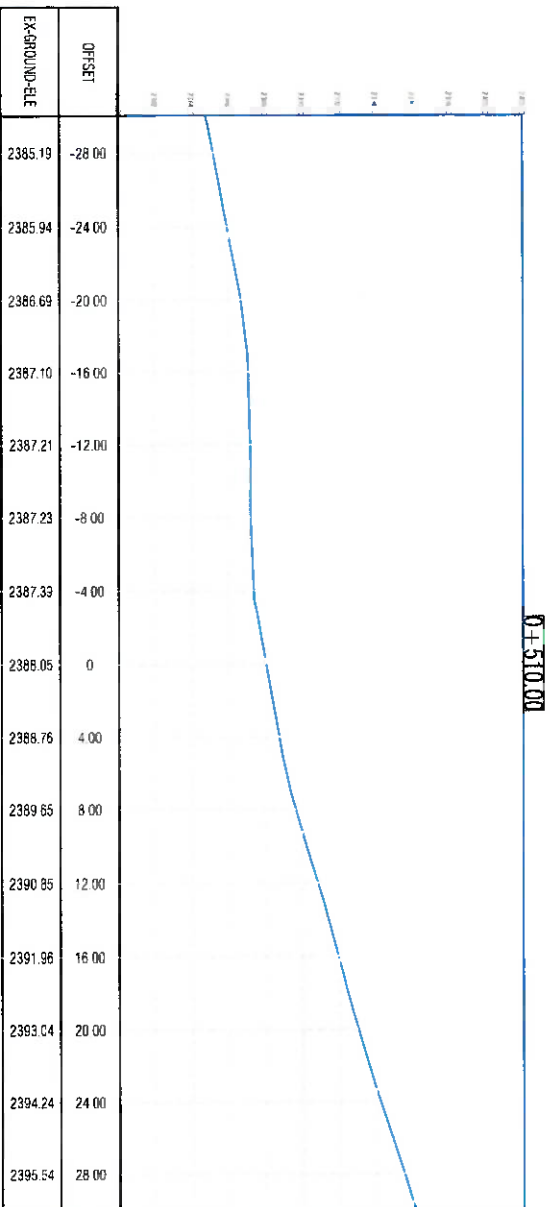
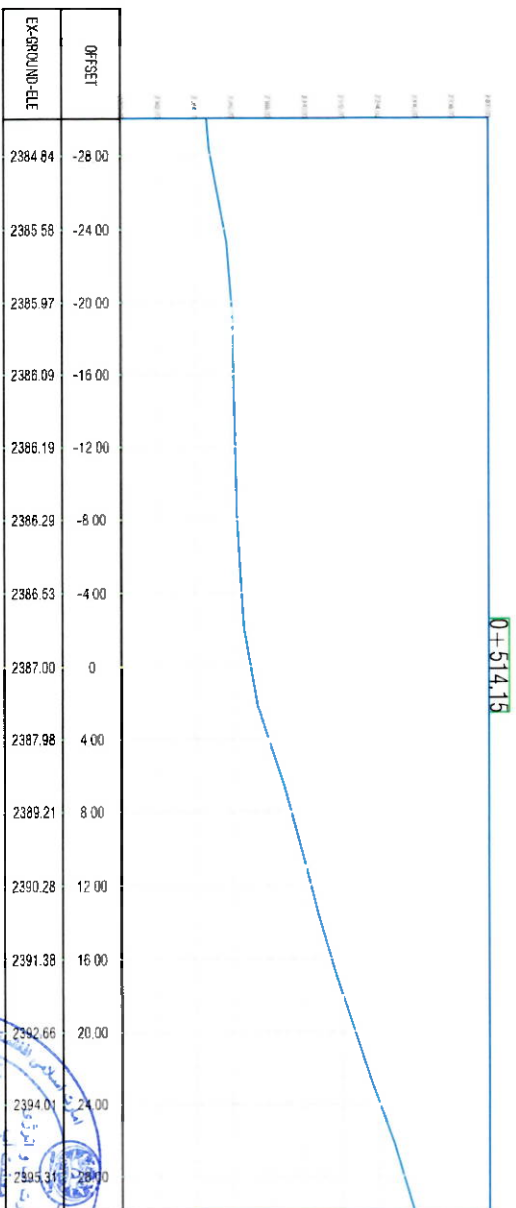
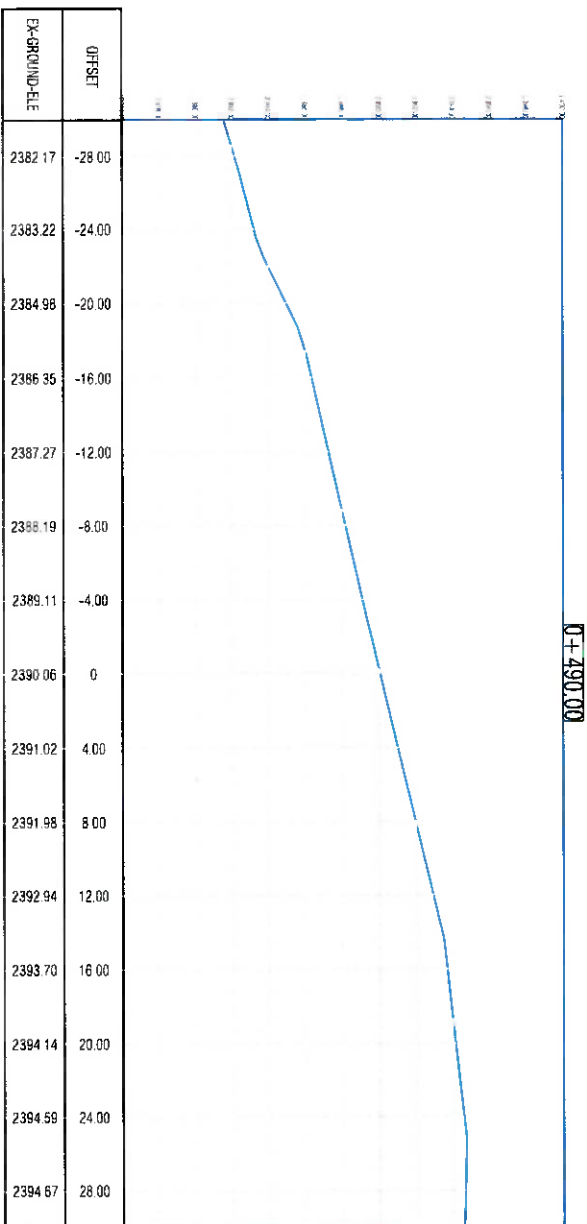
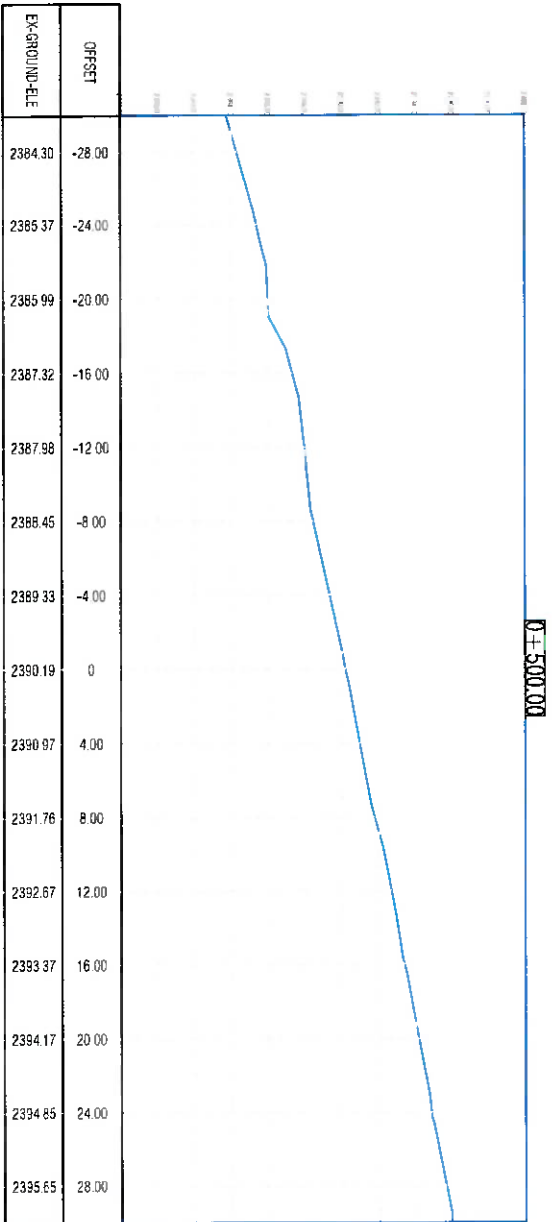
ENG. HEDAYATULLAH OMARI


SHEET NO
8
11











ISLAMIC EMIRATE OF AFGHANISTAN
MINISTRY OF WATER AND ENERGY
DEPUTY MINISTRY OF WATER
GENERAL DIRECTORATE OF ENGINEERING SERVICES
TECHNICAL BOARD

PROJECT NAME:
Sultan DAM

DRAWING:
SECTIONS

PROVINCE
Ghazni

DISTRICT
Khanjeh Ghazni

DATE
OCT 2023

SCALE
1:400

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SHEET NO
11