DEPARTAMENTO DE LA VIVIENDA
SAN JUAN, PUERTO RICO

DECLARACION DE IMPACTO AMBIENTAL FINAL
JCA-03-001 (DV)

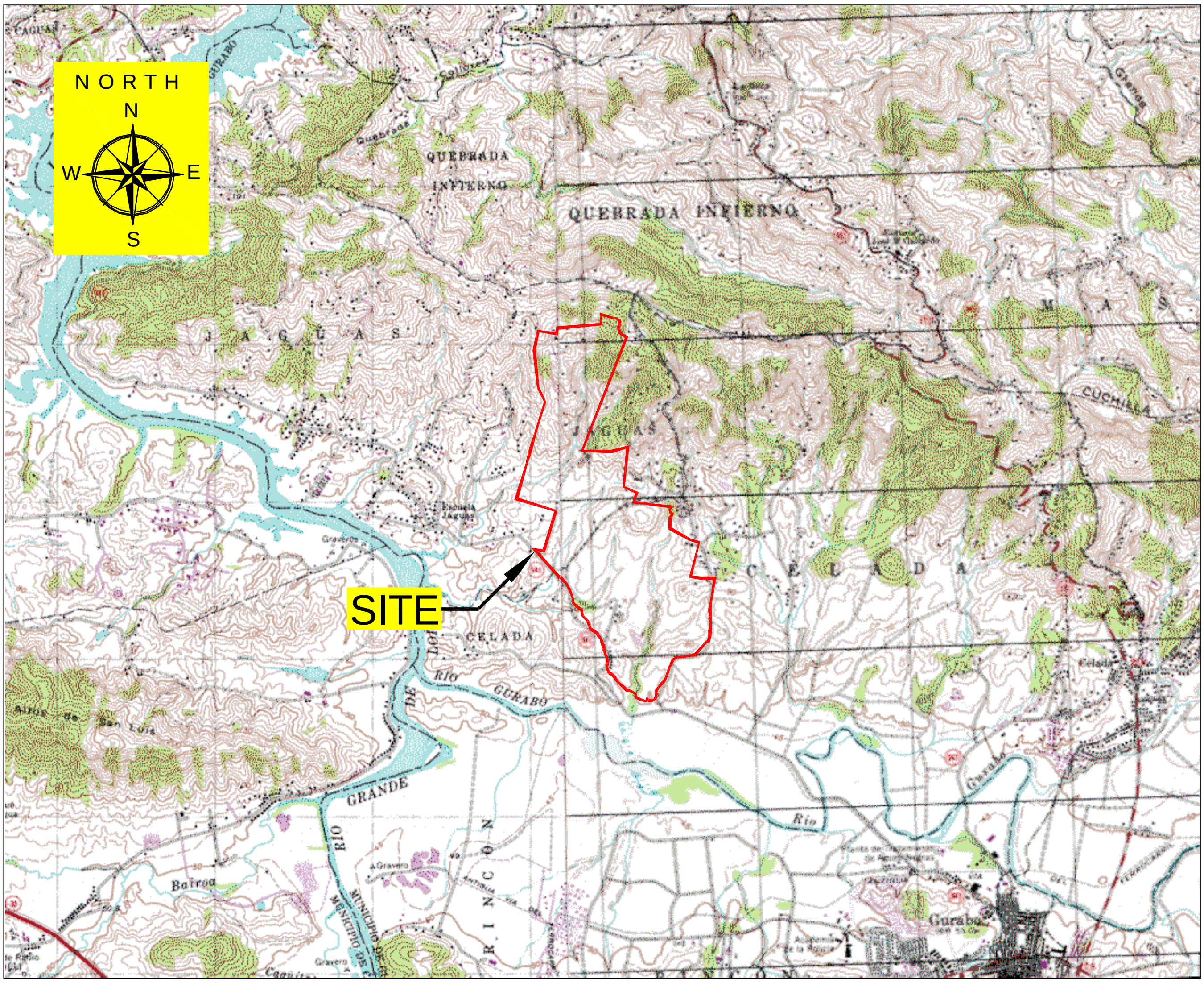
LA AVENTURA
Carretera Estatal PR-941
Barrios Jaguas y Celada
Gurabo, Puerto Rico

Enmienda a Consulta de Ubicación # 2001-47-0705-JPU

VOLUMEN III
APENDICES 9-11

APÉNDICE 9

HYDROLOGIC-HYDRAULIC STUDY
LA AVENTURA
GURABO, P.R.



MAY 2008

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TABLE OF CONTENTS

APPENDIX D HEC-RAS Hydraulic Input and Output Files for Proposed Condition

APPENDIX E Contech Design Details

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

1.1 Project Description and Location

La Aventura is a residential and institutional development proposed to the North of State Road PR-941, at Jaguas Ward in the municipality of Gurabo. **Figure 1** shows the project location on a partial reproduction of the USGS Gurabo topographic quadrangle. The project will consist of 1,833 residential units, commercial area and a school, as illustrated in the proposed project layout presented in **Figure 2**.

1.2 Scope and Purpose of Report

This report summarizes the results of the Hydrologic/Hydraulic (H/H) analysis for the proposed development site. The study evaluates the need for the sizing of stormwater detention structures, in accordance with Puerto Rico Planning Board Regulation #3. An encroachment analysis is performed for various unnamed creeks that cross the project site. Ten (10) bridge crossings are also designed in this study.

1.3 Report Limitations and Warnings

It shall be the responsibility of the site engineer or the project's geotechnical consultant to adapt the hydraulic design recommendations included in this report, to the soil and other conditions at the site on any matters concerning slope stability, conflicts with other infrastructure, etc.

The Department of Natural and Environmental Resources requires a minimum maintenance easement of 20 meters on either side of natural or artificial stream channels. This easement must be deeded to the Department. It shall be the responsibility of the Owner and the Site Engineer to coordinate easement requirements with the Department and incorporate this easement into the design.

1.4 Parties Involved

Project Name:	La Aventura
Owner:	Aventura Development Corporation
Civil Engineer:	Osvaldo Rivera & Associates
H-H Consultant:	Osvaldo Rivera & Associates

1.5 Authorization

Preparation of this report was authorized by means of written contract with Eng. Al Rizek, in representation of Aventura Development Corporation

2. STUDY AREA DESCRIPTION

2.1 Topography and Water Bodies

The topography of the project area presents elevations ranging from 270 m to 50 m. The drainage pattern at the site varies since four (4) unnamed creeks cross the site as illustrated in the project site topography presented in **Figure 3**. The creeks, which have been denominated in this study Unnamed Creeks #1 through #4, flow to the south and the west where three of them discharge into Rio Grande de Loíza and the fourth one discharges to Río Gurabo (see Figure 1). Onsite and offsite runoff tributary to the unnamed creeks create various intermittent reaches within the site.

2.2 Prior Studies and Floodplain Mapping

Figure 4 illustrates a partial reproduction of the FEMA Flood Insurance Rate Map showing the project site location. A small section of the unnamed creek tributary to Rio Gurabo (Unnamed Creek #1) is classified as Zone AE with a water level of 49.8 m. The remaining areas lie outside the flood zone.

2.3 Field Observations

The field visit to the project area was made on August 2006 and it revealed the following:

- The drainage pattern at the project site varies since four unnamed creeks cross the project site area.
- Three of the unnamed creeks are tributary to Río Grande de Loíza while the fourth one is tributary to Río Gurabo.
- Three of the unnamed creeks originate upstream the site.
- Various intermittent reaches throughout the site contribute in runoff to the unnamed creeks.

2.4 Field Data

A topographic survey of the site, referenced to mean sea level was provided by the project owner. Land surveyor José Meléndez prepared the topographic plans for the project site. A copy of the certified topographic survey is included in the back pocket of this report, and is also reproduced as **Figure 3**. All elevations in this report are referenced to mean sea level vertical datum unless otherwise specified.

3. STUDY APPROACH

Two analyses were performed to determine the effect of the proposed project: a detention analysis and a hydraulic analysis of the unnamed creeks.

Both analyses evaluated the study area in pre- and post-development conditions in order to demonstrate the compliance of the project site with the regulations.

Detention Analysis

The existing condition model in the detention analysis determines the peak discharges that produce the project site and the unnamed creeks. Offsite and onsite areas contribute in runoff to the creeks. Since the majority of the project site discharges to the unnamed creeks, the streams were included in the detention analysis.

The post-development model accounts for the change of hydrologic parameters at the proposed site to determine the hydrologic-hydraulic effects in the area. Since the proposed development will produce an increase in site runoff as compared to the existing undeveloped condition, twelve (12) detention structures are provided. The detention ponds will discharge to the unnamed creeks that correspond to the pre-development point of discharge of the majority of the project site runoff. Post-development peak discharges are compared with the existing ones in order to demonstrate the compliance of the proposed project with Puerto Rico Planning Board Regulation # 3.

The stormwater detention analysis was performed using the one-dimensional unsteady-flow adICPR model (Streamline Technologies, 2000). This model dynamically routes storm water through open channels, closed conduits, and detention ponds. Each node in adICPR represents a control volume. Change in storage for each node is calculated based on the difference between inflows and outflows at each time step during the simulation period, and the change in storage is used to determine elevations at each node at the end of each time step. Flow through each link is calculated from the known elevations at each end of the link and the hydraulic properties of the link.

Hydraulic Analysis of Unnamed Creeks

In the hydraulic analysis of the creeks that cross the project site, the pre-development model determines the existing 100-yr water surface levels in the study area. Cross sections were taken at the unnamed creeks and their intermittent tributaries to determine these levels. Cross sections location is illustrated in **Figure 17**.

The post-development model performs an encroachment analysis on the unnamed creeks and its principal tributaries in order to determine the permissible construction limits of the proposed development. Encroachment limits are tested to insure that water surface elevations do not increase by more than 0.15m, in compliance with Planning Board Regulation No.13. This model also presents the design of ten (10) bridge crossings along the unnamed creeks and tributaries.

The hydraulic analysis of the creeks was performed using the one-dimensional steady-flow HEC-RAS model (U.S. Army Corps of Engineers, 2001). This model was considered appropriate given the one-dimensional flow condition in the study reach and the lack of significant storage effects in the area that will be flooded.

4. HYDROLOGIC ANALYSIS

4.1 Methodology

The hydrologic modeling of the project site and the unnamed creeks' watersheds was performed using the Soil Conservation Service Unit Hydrograph method, incorporated in the adICPR model version 3.02. Parameters used in this method include basin area, curve number and time of concentration.

4.2 Watershed Delimitation

Existing Condition

The project site was divided in thirty (30) basins in the existing condition model, according to the pre-development drainage pattern in the area. The majority of these basins are tributary to the unnamed creeks that cross the project site. Twelve (12) offsite areas also contribute in runoff to the unnamed creeks.

The limits for the onsite basins were determined from the project drawings provided by the project owner and are presented in **Figure 5**. The offsite watersheds were determined from the USGS Gurabo topographic quadrangle and are illustrated in **Figure 6**.

In the proposed condition, the project site was divided in twenty-five (25) basins according to the post-development drainage pattern. The majority of the proposed onsite basins will discharge to the proposed detention ponds, while small areas will discharge directly to the unnamed creeks. Twelve (12) detention ponds will be provided as illustrated in **Figure 18**. The ponds will mitigate the increase in runoff produced by the whole development. **Figure 7** presents the limits for the proposed basins.

Table 1 and Table 2 present the areas of the identified basins for existing and proposed conditions, respectively. It is also indicated to which creek or detention pond the basin is or will be tributary.

Table 1: Watershed Areas (acres) for Existing Condition

Basin	Area	Point of Discharge
Existing		
Site 1	8.4	Río Gurabo
Site 2	1.7	Río Gurabo
Site 3A	46.6	Unnamed Creek 1
Site 3B	16.0	Unnamed Creek 1
Site 3C	27	Unnamed Creek 1
Site 3D	26.8	Unnamed Creek 1
Site 4	1.3	D / S Site
Site 5	4.8	D / S Site
Site 6	39.6	Unnamed Creek 2
Site 7	3.2	Unnamed Creek 3
Site 8A1	24.8	Unnamed Creek 3
Site 8A2	24.9	Unnamed Creek 3
Site 8A3	14.9	Unnamed Creek 3
Site 8B	5.1	Unnamed Creek 3
Site 9	1.9	D / S Site
Site 10A	22.5	Unnamed Creek 4
Site 10B	1.1	Unnamed Creek 4

Table 1: Watershed Areas (acres) for Existing Condition (Cont.)

Basin	Area	Point of Discharge
Site 10C	6.9	Unnamed Creek 4
Site 10D	9.4	Unnamed Creek 4
Site 10E	23.8	Unnamed Creek 4
Site 10F	1.8	Unnamed Creek 4
Site 10G	2.3	Unnamed Creek 4
Site 10H	7.0	Unnamed Creek 4
Site 10I	6.0	Unnamed Creek 4
Site 10J	1.1	Unnamed Creek 4
Site 10K	6.6	Unnamed Creek 4
Site 10L	4.9	Unnamed Creek 4
Site 10M	5.3	Unnamed Creek 4
Site 11	5.9	Unnamed Creek 4
Site 12	5.7	Unnamed Creek 4
Offsite 1	7.4	Unnamed Creek 4
Offsite 2	12.8	Unnamed Creek 4
Offsite 3	9.5	Unnamed Creek 4
Offsite 4	8.6	Unnamed Creek 4

Table 1: Watershed Areas (acres) for Existing Condition (Cont.)

Basin	Area	Point of Discharge
Offsite 5	13.3	Unnamed Creek 4
Offsite 6	12.7	Unnamed Creek 4
Offsite 6.5	1.9	Unnamed Creek 4
Offsite 7	62.9	Unnamed Creek 4
Offsite 8	16.7	Unnamed Creek 4
Offsite 9	23.6	Unnamed Creek 4
Offsite 10	14.7	Unnamed Creek 4
Offsite 11	3.0	Unnamed Creek 4

Table 2: Watershed Areas (acres) for Proposed Condition

Basin	Area	Point of Discharge
Site 1A	17.2	Pond 1A
Site 1B	17.9	Pond 1B
Site 1C	4.8	Unnamed Creek 1
Site 1D	14	Pond 1D
Site 2A	33.3	Pond 2A
Site 2B1	8.1	Unnamed Creek 1
Site 2B2	10.4	Intermittent Creek
Site 2C	11.9	Unnamed Creek 1
Site 2D1	13.6	Pond 2D
Site 2D2	2.1	Pond 2D2
Site 2D3	7.8	Pond 2D3
Site 3A1	13.5	Unnamed Creek 2
Site 3A2	7.2	Pond 3A
Site 3B	18.1	Pond 3B
Site 4A	28.9	Pond 4A
Site 4B1	7.0	Unnamed Creek 3
Site 4B2	6.2	Unnamed Creek 3
Site 5A	16.7	Pond 5
Site 5B	10.6	Unnamed Creek 3
Site 5C	3.6	Intt. Reach Trib. To Unnamed Creek3
Site 6A1	23.6	Pond 6A
Site 6A2	33.8	Unnamed Creek 4
Site 6B	36.5	Pond 6B
Site 6C	10.6	Unnamed Creek 4
Site 6D	9.95	Unnamed Creek 4

4.3 Soils and Curve Number

The curve number represents the runoff potential within the watershed and is estimated based on soil type (hydrologic soil group), land use, and antecedent moisture condition II. Soil types within the watershed areas were obtained using the Soil Survey of San Juan Area, Puerto Rico (**Figure 8**), published by the Natural Resources Conservation Service (Boccheciamp, 1977). Table 3 presents the hydrologic soil group classification for the soils within the identified basins and the corresponding curve number of each group. Table 4 and Table 5 present Curve Numbers for the basins for existing and proposed conditions, respectively. Offsite basins do not suffer hydrological changes in the proposed condition.

Table 3: Hydrologic Soil Groups and Corresponding Curve Number

Soil	HSG	Curve Number Existing Condition	Curve Number Proposed Condition
AaB, AaC	B	69	85
NaD2, NaE2	C	79	90
CaF, MxE, RoC2	D	84	92

Table 4: Curve Number for Existing Condition

Basin	HSG	Cover Type	$\overline{CN}$
Site 1	C D	Pasture -0.54 ac Pasture- 7.78 ac	84
Site 2	C D	Pasture -1.29 ac Pasture- 0.4 ac	80
Site 3A	B C D	Pasture -28.4 ac Pasture- 10.4 ac Pasture- 7.7 ac	74
Site 3B	B C	Pasture -3.05 ac Pasture- 12.95 ac	77
Site 3C	B C	Pasture -10 ac Pasture- 16.78 ac	75
Site 3D	B C	Pasture -8.5 ac Pasture- 18.3 ac	76
Site 4	D	Pasture	84
Site 5	B C	Pasture -3.71 ac Pasture- 1.05 ac	71
Site 6	B D	Pasture -23.2 ac Pasture- 16.3 ac	73
Site 7	D	Pasture	84
Site 8A1	B C	Pasture –15.2 ac Pasture- 9.6 ac	72
Site 8A2	B C D	Pasture -7.4 ac Pasture- 14.9 ac Pasture-2.6 ac	77
Site 8A3	B C	Pasture-14.26 Pasture-0.64	69

Table 4: Curve Number for Existing Condition (Cont.)

Basin	HSG	Cover Type	$\overline{CN}$
Site 8B	D	Pasture	84
Site 9	B C	Pasture -1.31 ac Pasture- 0.61 ac	72
Site 10A	B C D	Pasture -6.17 ac Pasture- 2.87 ac Pasture- 13.16 ac	79
Site 10B-Site 10M	D	Pasture	84
Site 11-Site 12	D	Pasture	84
Offsite 1-Offsite 7	D	Pasture	84
Offsite 8	C D	Pasture -4.05 ac Pasture- 12.65 ac	83
Offsite 9	C D	Pasture -4.49 ac Pasture- 19.11 ac	83
Offsite 10	C D	Pasture -0.82 ac Pasture- 13.88 ac	84
Offsite 11	C	Pasture	79

Table 5: Curve Number for Proposed Condition

Basin	HSG	Cover Type	$\overline{CN}$
Site 1A	B C	Developed-4.29 ac Developed-12.81 ac	89
Site 1B	B C	Developed-6.2 ac Developed-11.7 ac	88
Site 1C	B	Developed	85
Site 1D	C	Developed	90
Site 2A	B C D	Developed-20.3 ac Developed-6 ac Developed-7.1 ac	87
Site 2B1	B C	Developed-6 ac Developed-2.1 ac	86
Site 2B2	B C	Developed-2.4 ac Developed-7.96 ac	89
Site 2D1	D	Developed	98
Site 2D2	D	Developed	98
Site 2D3	C,D	Developed	98
Site 3A1	C	Developed	90
Site 3A2	B C	Developed-3.3 ac Developed-3.9 ac	88

Table 5: Curve Number for Proposed Condition (Cont.)

Basin	HSG	Cover Type	$\overline{CN}$
Site 3B	B	Developed	85
Site 4A	B C	Developed-18.6 ac Developed-10.3 ac	87
Site 4B1	B C	Developed-6.4 ac Developed-0.6 ac	85
Site 4B2	B	Developed	85
Site 5A	B C D	Developed-5.77 ac Developed-5.13 ac Developed-5.8 ac	89
Site 5B	C	Developed	90
Site 5C	C D	Developed-0.42 ac Developed-3.2 ac	83
Site 6A1	D	Developed	92
Site 6A2	D	Pasture	84
Site 6B	D	Developed	92
Site 6C	B D	Developed-5.4 ac Developed-5.2 ac	88
Site 6D	B C D	Developed-5.2 ac Developed-3.4 ac Developed-1.35 ac	88

4.4 Time of Concentration

Time of concentration was computed using the Soil Conservation Service method (TR-55).

Time of concentration was estimated with the following equation:

$$t_c = L / (3600 * V)$$

where,

t_c = time of concentration (hr)

L = flow length (ft)

V = average flow velocity (ft / s) from figure 3 of TR-55, and

3600 = conversion factor from seconds to hours.

Table 6 presents time of concentration computed for existing and proposed conditions.

Table 6: Time of Concentration (minutes) for Existing and Proposed Conditions

Basin	Tc
Existing	
Site 1	10
Site 2	9
Site 3A	26
Site 3B	11
Site 3C	10
Site 3D	5
Site 4	5
Site 5	12
Site 6	18
Site 7	8
Site 8A1	18
Site 8A2	11
Site 8A3	13
Site 8B	6
Site 9	9
Site 10A	8
Site 10B	5
Site 10C	5

Table 6: Time of Concentration (minutes) for Existing and Proposed Conditions (Cont.)

Basin	Tc
Site 10D	5
Site 10E	6
Site 10F	5
Site 10G	5
Site 10H	7
Site 10I	5
Site 10J	5
Site 10K	5
Site 10L	5
Site 10M	5
Site 11	7
Site 12	6
Offsite 1	6
Offsite 2	6
Offsite 3	9
Offsite 4	7
Offsite 5	8

Table 6: Time of Concentration (minutes) for Existing and Proposed Conditions (Cont.)

Basin	Tc
Offsite 6	6
Offsite 6.5	5
Offsite 7	10
Offsite 8	8
Offsite 9	9
Offsite 10	8
Offsite 11	10
Proposed	
Site 1A	6
Site 1B	7
Site 1C	5
Site 1D	7
Site 2A	10
Site 2B1	6
Site 2B2	6
Site 2C	5
Site 2D1	5

Table 6: Time of Concentration (minutes) for Existing and Proposed Conditions (Cont.)

Basin	Tc
Site 2D2	5
Site 2D3	5
Site 3A1	5
Site 3A2	6
Site 3B	6
Site 4A	6
Site 4B1	5
Site 4B2	5
Site 5A	6
Site 5B	5
Site 5C	5
Site 6A1	7
Site 6A2	5
Site 6B	8
Site 6C	5
Site 6D	5

4.4 Rainfall

The 24-hour precipitation depths for return periods of 2- and 100-years were obtained from the National Oceanic & Atmospheric Administration, Atlas 14, Volume 3 (Version 3). As illustrated in Appendix A, values of 5.6” and 18.12” were obtained for 2- and 100-yr return periods, respectively.

4.5 Results of Hydrologic Analysis

Table 7 and Table 8 summarize peak discharges for existing and proposed conditions, respectively, of the project site and the offsite watersheds. These results were obtained using the adICPR model for 2- and 100-yr return periods. Input and output files for hydrologic modeling of project site are included in Appendix A and B, respectively.

Table 7: Existing Peak Discharges (cfs) for Watersheds

Basin	2YR	100YR
Site 1	42.1	160.9
Site 2	7.9	32.4
Site 3A	119.6	590.4
Site 3B	66.6	293.3
Site 3C	107.9	494.2
Site 3D	122.2	529.7
Site 4	6.5	24.3
Site 5	16.1	82.0
Site 6	122.1	604.6
Site 7	16.7	63.6
Site 8A1	76.3	377.9
Site 8A2	102.8	452.7
Site 8A3	45.4	245.1
Site 8B	30.0	113.2
Site 9	7.3	35.4
Site 10A	104.0	432.6
Site 10B	5.8	21.7
Site 10C	37.8	141.5
Site 10D	50.6	189.6

Table 7: Existing Peak Discharges (cfs) for Watersheds (Cont.)

Basin	2YR	100YR
Site 10E	128.4	484.0
Site 10F	9.7	36.3
Site 10G	12.8	48.0
Site 10H	36.7	138.9
Site 10I	32.6	122.3
Site 10J	5.7	21.2
Site 10K	36.0	135.0
Site 10L	26.6	99.6
Site 10M	28.6	107.2
Site 11	31.0	117.5
Site 12	30.7	115.7
Offsite 1	40.0	151.0
Offsite 2	68.9	259.6
Offsite 3	48.9	186.2
Offsite 4	45.8	173.2
Offsite 5	69.8	265.6
Offsite 6	68.1	256.6
Offsite 6.5	10.1	37.9

Table 7: Existing Peak Discharges (cfs) for Watersheds (Cont.)

Basin	2YR	100YR
Offsite 7	317.5	1,212.8
Offsite 8	85.7	331.3
Offsite 9	119.4	462.2
Offsite 10	77.0	292.7
Offsite 11	13.5	56.5

Table 8: Proposed Peak Discharges (cfs) for Watersheds

Basin	2YR	100YR
Site 1A	100.5	352.2
Site 1B	101.8	362.9
Site 1C	26.9	99.4
Site 1D	82.6	287.2
Site 2A	179.1	651.3
Site 2B1	45.7	167.1
Site 2B2	61	213.9
Site 2C	70.8	250.5
Site 2D1	23.6	76.7
Site 2D2	13.6	44
Site 2D3	50.6	164.3
Site 3A1	81.4	280.9
Site 3A2	41.7	148.2
Site 3B	99.3	368.5
Site 4A	165	594.2
Site 4B1	39	144
Site 4B2	34.5	127.3
Site 5A	98.1	344
Site 5B	65	224.4
Site 5C	19.3	73.6

Table 8: Proposed Peak Discharges (cfs) for Watersheds

Basin	2YR	100YR
Site 6A1	142.8	485.1
Site 6A2	184.8	692.6
Site 6B	217.7	740.4
Site 6C	62.1	219.9
Site 6D	57.8	204.5

4.6 Verification of Peak Discharge

Peak discharges determined by the ICPR model, were checked for reasonableness using the Rational Method. Table 9 summarizes the Rational Method Parameters for the watersheds and compares peak discharges generated with the adICPR model.

The Rational Method equation has the following form:

$$Q_{100} = CI_{100}A$$

where;

Q_{100} =100- year peak discharge (cfs)

C = runoff coefficient.

I_{100} = Mean precipitation, (in. / hr.)

A = watershed area, (acres)

Table 9: Rational Method Parameters for 100-yr Event

Basin	C	I (in / hr)	Area (acres)	Rational Method (cfs)	adICPR (cfs)
<u>Existing</u>					
Site 1	0.65	13.6	8.3	73.4	160.9
Site 2	0.85	14.0	1.7	20.2	32.4
Site 3A	0.6	9.5	46.5	265.1	590.4
Site 3B	0.65	13.0	16.0	135.2	293.3
Site 3C	0.65	13.6	26.8	236.9	494.2
Site 3D	0.65	18.0	26.8	313.6	529.7
Site 4	0.75	18.0	1.2	16.2	24.3
Site 5	0.65	12.5	4.8	39.0	82.0
Site 6	0.65	10.5	39.5	269.6	604.6
Site 7	0.85	15.0	3.2	40.8	63.6
Site 8A1	0.65	10.5	24.8	169.3	377.9
Site 8A2	0.6	13.0	24.9	194.2	452.7
Site 8A3	0.65	12	14.9	116.2	245.1
Site 8B	0.8	17.0	5.6	76.2	113.2
Site 9	0.65	14.0	1.9	17.3	35.4
Site 10A	0.65	15.0	22.2	216.5	432.6
Site 10B	0.65	18.0	1.1	12.9	21.7
Site 10C	0.65	18.0	6.9	80.7	141.5
Site 10D	0.65	18.0	9.3	108.8	189.6

Table 9: Rational Method Parameters for 100-yr Event (Cont.)

Basin	C	I (in / hr)	Area (acres)	Rational Method (cfs)	adICPR (cfs)
Site 10E	0.65	17.0	23.8	263.0	484.0
Site 10F	0.65	18.0	1.8	21.1	36.3
Site 10G	0.65	18.0	2.3	26.9	48.0
Site 10H	0.65	16.0	6.9	71.8	138.9
Site 10I	0.65	18.0	6.0	70.2	122.3
Site 10J	0.65	18.0	1.1	12.9	21.2
Site 10K	0.65	18.0	6.6	77.2	135.0
Site 10L	0.65	18.0	4.9	57.3	99.6
Site 10M	0.65	18.0	5.2	60.8	107.2
Site 11	0.7	16.0	5.8	65.0	117.5
Site 12	0.8	17.0	5.7	77.5	115.7
Offsite 1	0.8	17.0	7.4	100.6	151.0
Offsite 2	0.75	17.0	12.8	163.2	259.6
Offsite 3	0.75	14.0	9.5	99.8	186.2
Offsite 4	0.75	16.0	8.6	103.2	173.2
Offsite 5	0.75	8.0	13.3	79.8	265.6
Offsite 6	0.75	17.0	12.7	161.9	256.6

Table 9: Rational Method Parameters for 100-yr Event (Cont.)

Basin	C	I (in / hr)	Area (acres)	Rational Method (cfs)	adICPR (cfs)
Offsite 6.5	0.8	18.0	1.9	27.4	37.9
Offsite 7	0.8	13.6	62.9	684.4	1,212.8
Offsite 8	0.8	15.0	16.7	200.4	331.3
Offsite 9	0.8	14.0	23.6	264.3	462.2
Offsite 10	0.8	15.0	14.7	176.4	292.7
Offsite 11	0.65	13.6	3.0	26.5	56.5
Proposed					
Site 1A	0.7	17.0	17.1	203.5	352.2
Site 1B	0.7	16.0	17.8	199.4	362.9
Site 1C	0.7	18.0	4.8	60.5	99.4
Site 1D	0.7	16.0	14	156.8	287.2
Site 2A	0.7	13.6	33.4	318.0	651.3
Site 2B1	0.7	17.0	8.2	97.6	167.1
Site 2B2	0.7	17.0	10.4	123.8	213.9
Site 2C	0.7	18.0	12.1	152.5	250.5
Site 2D1	0.9	18.0	3.65	59.1	76.7
Site 2D2	0.9	18.0	2.1	34.0	44
Site 2D3	0.9	18.0	7.8	126.4	164.3

Table 9: Rational Method Parameters for 100-yr Event (Cont.)

Basin	C	I (in / hr)	Area (acres)	Rational Method (cfs)	adICPR (cfs)
Site 3A1	0.7	18	13.5	170.1	280.9
Site 3A2	0.7	17	7.2	85.7	148.2
Site 3B	0.7	17.0	18.0	214.2	368.5
Site 4A	0.7	17.0	28.9	343.9	594.2
Site 4B1	0.7	18.0	7.1	89.5	144
Site 4B2	0.7	18.0	6.2	78.1	127.3
Site 5A	0.7	17.0	16.7	198.7	344
Site 5B	0.7	18.0	10.8	136.1	224.4
Site 5C	0.7	18.0	3.6	45.4	73.6
Site 6A1	0.7	16.0	23.6	264.3	485.1
Site 6A2	0.7	18.0	33.9	427.1	692.6
Site 6B	0.7	15.0	36.4	382.2	740.4
Site 6C	0.7	18.0	10.6	133.6	219.9
Site 6D	0.7	18.0	9.9	124.7	204.5

Peak discharges determined with the Rational Method are lower than those obtained with the adICPR model. The reason for this is that the Intensity-Duration Curves used in the Rational Method are based of the TP-42, while the ICPR modeling uses the revised rainfall data from the NOAA which presents higher values of rainfall for the Island than the TP-42.

5. HYDRAULIC ANALYSIS

5.1 Models Prepared

Detention Analysis

The following hydrologic-hydraulic models were prepared for the detention analysis of the proposed project site:

Pre-development Condition: This model represents the existing conditions at the project site. Peak discharges are determined for onsite and offsite basins. Cross sections were incorporated in the ICPR model in order to obtain peak discharges at the unnamed creeks. **Figure 9** through **Figure 12** presents the schematic link-node diagrams for this model.

Post-Development Condition Model: The existing condition model was modified to incorporate the proposed land use change and detention structures as illustrated in the link-node diagrams of **Figure 13** through **Figure 16**.

Hydraulic Analysis of Unnamed Creeks

A HEC-RAS model was constructed to define the encroachment limits and flood levels in the unnamed creeks.

Existing Condition Model: This model defines the existing 100-yr water surface profiles along the unnamed creeks. Cross sections were taken at the creeks and at their intermittent tributaries.

Proposed Condition Model: The existing condition model and the 100-yr discharges were used to perform the encroachment analysis for the unnamed creeks and tributaries. The analysis was made using as maximum increase +0.15m. The method of equal conveyance (#4) was used for this analysis. Ten (10) bridge crossings along the creeks and tributaries are proposed.

5.2 Hydraulic Characteristics of Unnamed Creeks

Cross Sections

The location of the cross sections was selected based on field inspection and proposed project layout. **Figure 17** shows their location.

Roughness Coefficients

Values of Manning's hydraulic roughness coefficients (n-values) were based on field inspection and with reference to Chow (1959) and Barnes (1967). N-values of 0.04 and 0.05 were used at the main channel and the overbanks, respectively.

The contraction and expansion coefficients for the natural channel sections are 0.1 and 0.3 respectively, and for bridge entrance and exit are 0.3 and 0.5, respectively.

Starting Water Surface Elevation

For the hydraulic analysis of the creeks, a known water surface level of 49.8m as obtained from FEMA flood map was used as the starting water surface elevation for Unnamed Creek #1 which is tributary to Río Gurabo. For the remaining creeks, the slope area method was used. Appendix C presents the values used.

For the detention analysis, water surface levels obtained from the HEC-RAS modeling were used as boundary condition.

5.3 Detention Analysis

5.3.1 Existing Condition

The detention analysis determines in the pre-development condition the peak discharges produced in the area. These discharges will be compared with the post-development ones in order to demonstrate that no increase in runoff has taken place as a result of the development. Since the point of discharge of almost all basins in the existing condition, and for the whole site in the proposed one corresponds to the unnamed creeks, the comparison of discharges will be made at the last cross section taken at each creek. Table 1 presented the tributary basins of each creek. Table 10 presents the existing 2-yr and 100-yr peak discharges at the most downstream cross section (boundary) of each creek.

Table 10: Existing Peak Discharges (cfs) at Unnamed Creeks

Cross Section	2-yr	100-yr
XS 51 (Unnamed Creek 1)	489	2,226
XS 36 (Unnamed Creek 2)	117	560
XS 24 (Unnamed Creek 3)	335	1,502
XS 1 (Unnamed Creek 4)	1,160	4,567

5.3.2 Proposed Condition

In order to comply with Planning Board Regulation # 3, nine (9) detention ponds and three (3) underground boxes are proposed to mitigate the increase in runoff caused by the proposed project. Their location is presented in **Figure 18**.

Table 1 presented the tributary basin of each detention structure. Some areas of the site will discharge directly to the unnamed creeks. The increase in runoff caused by these basins will be mitigated by the proposed detention structures that will discharge to the unnamed creeks.

The structures will include an orifice and a standpipe to control the 2-yr and 100-yr events, respectively. A concrete apron will be placed in the orifice area of the ponds to prevent vegetation growth. Table 11 presents the design parameters determined for the structures. **Figure 19** through **Figure 30** present their schematic design. As illustrated, Ponds 1D, 2D2 and 2D3 correspond to the underground boxes.

Table 11: Design Parameters for Proposed Detention Ponds

Pond	IE	Top Area (m^2)	Bottom Area (m^2)	Depth (m)	Orifice Diameter (in)	Standpipe Diameter (in)	Outlet Pipe Size	Outlet Pipe Slope (m/m)	100-yr WSE (m)	FreeBoard (m)
1A	74	3,335	1,943	3.7	24	36	48”	0.4	77.4	0.3
1B	64	3,645	2,137	3.7	20	36	48”	0.053	67.4	0.3
1D	64	2,596	2,596	3.0	30	36	48”	0.053	66.6	0.4
2A	54	5,940	3,082	5.3	24	42	60”	0.053	59.0	0.3
2D2	54	276	276	2.6	18	24	30”	0.24	56.3	0.3
2D3	54	1,900	1,900	2	30	42	54	0.24	55.7	0.3
3A	68	1,989	1,367	1.9	18	42	54”	0.18	69.7	0.2
3B	62	2,922	1,336	4.3	18	42	54”	0.13	66.0	0.3

Table 11: Design Parameters for Proposed Detention Ponds (Cont.)

Pond	IE	Top Area (m^2)	Bottom Area (m^2)	Depth (m)	Orifice Diameter (in)	Standpipe Diameter (in)	Outlet Pipe Size	Outlet Pipe Slope (m/m)	100-yr WSE (m)	FreeBoard (m)
4	57	4,900	2,724	4.6	26	36	48”	0.12	61.3	0.3
5	70	3,000	1,995	2.7	40	54	60”	0.26	72.4	0.3
6A	106	3,300	1,896	3.8	36	54	1.52m x 1.52m	0.35	109.5	0.3
6B	65.5	5,900	3,638	4.3	36	48	72”	0.19	69.5	0.3

The detention ponds incorporate an emergency spillway to handle overflow in a controlled manner in case the outlet becomes clogged for any reason, or for an event larger than 100-year return interval. The invert of the emergency spillway is set at the 100-yr water level in the detention ponds (see Appendix B); a freeboard around the ponds was set to allow the 100-year flow to discharge across the spillway, without overtopping any other part of the structures, with the normal outlet completely closed.

Catch basins will be provided to collect stormwater from the site to underground boxes 1D, 2D2 and 2D3. Table 12 presents the minimum inlet area required to collect stormwater at these structures.

Table 12: Minimum Inlet Areas Required to Collect Stormwater at Underground Detention Structures

Pond	Minimum Inlet Area (m^2)
1D	13.4
2D2	2.1
2D3	7.7

Table 13 presents the pre- and post- development peak discharges showing the effect of the detention ponds for the 100-yr and 2-yr events. The comparison is made at the most downstream cross section of the unnamed creeks.

Table 13: Peak Discharge (cfs) for Existing and Proposed Conditions

Point of Analysis	2YR		100YR	
	Existing	Proposed	Existing	Proposed
XS 51 (Unnamed Creek 1)	489	477	2,226	1,984
XS 36 (Unnamed Creek 2)	117	115	560	534
XS 24 (Unnamed Creek 3)	335	298	1,502	1,257
XS 1 (Unnamed Creek 4)	1,160	985	4,567	4,257

Table 14 presents a comparison between inflows at outflows at the detention structures.

Table 14: Comparison between Inflows and Outflows (cfs) at Detention Ponds

Pond	2YR		100YR	
	Inflow	Outflow	Inflow	Outflow
1A	101	34	352	142
1B	102	24	363	155
1D	83	35	287	150
2A	206	44	750	270
2D2	14	12	44	37
2D3	50	30	164	118
3A	42	15	148	97
3B	99	24	369	179
4	165	42	594	216
5	98	56	344	326
6A	143	78	485	284
6B	218	78	740	300

As illustrated in the tables, the proposed detention structures reduce the project peak discharges below the existing peak discharges in compliance with Planning Board Regulation No.3.

5.4 Hydraulic Analysis of Unnamed Creeks

5.4.1 Existing Condition

In order to determine the existing hydraulic conditions in the area, cross sections were taken at the unnamed creeks and tributaries. **Figure 17** presents their location. Peak discharges obtained in the hydrologic analysis were used in the model to obtain water surface elevations. Table 15 presents these results for every other cross section. Complete simulation results can be found in Appendix C.

Table 15: Water Surface Levels (m) for 100-yr Event-Existing Condition

Cross Section	100 YR Water Surface Elevation (m)
<u>Unnamed Creek 1</u>	
XS 78	84.36
XS 76	81.28
XS 74	78.12
XS 72	69.03
XS 70	66.64
XS 68	62.89
XS 66	58.93
XS 64	57.18
XS 62	53.53
XS 60	53.12
XS 58	53.06
XS 56	53.06
XS 54	52.64
XS 52	51.44
XS 87	80.40
XS 85	74.15

Table 15: Water Surface Levels (m) for 100-yr Event-Existing Condition

Cross Section	100 YR Water Surface Elevation (m)
XS 83	66.40
XS 81	61.37
XS 79	58.77
XS 718	78.49
XS 716	66.75
XS 714	56.46
XS 712	56.31
XS 710	53.07
Unnamed Creek 2	
XS 50	90.62
XS 48	74.39
XS 46	66.70
XS 44	62.64
XS 42	59.23
XS 40	58.39
XS 38	54.63
XS 36	52.71

Table 15: Water Surface Levels (m) for 100-yr Event-Existing Condition

Cross Section	100 YR Water Surface Elevation (m)
Unnamed Creek 3	
XS 35	80.73
XS 33	70.69
XS 31	65.47
XS 29	62.63
XS 27	58.71
XS 25	54.25
XS 35.5	142.44
XS 35.3	104.25
XS 35.1	78.21
XS 26.6	76.10
XS 26.4	64.61
XS 26.25	59.07
Unnamed Creek 4	
XS 23	271.36
XS 21	241.3
XS20	200.58

Table 15: Water Surface Levels (m) for 100-yr Event-Existing Condition

Cross Section	100 YR Water Surface Elevation (m)
XS 18.5	157.01
XS 17.5	138.67
XS 16	119.61
XS 15	111.77
XS 13	95.16
XS 11	82.83
XS 9	74.67
XS 7	65.39
XS 5	65.56
XS 3	59.71
XS 1.5	57.26
XS 0.2	232.71
XS 0.09	208.37
XS 0.07	174.65
XS 0.05	155.17
XS 0.03	131.04
XS 0.02	119.39
XS 0.5	232.56

Table 15: Water Surface Levels (m) for 100-yr Event-Existing Condition

Cross Section	100 YR Water Surface Elevation (m)
XS 0.3	206.49
XS 0.24	180.62
XS 0.22	160.88
XS 0.58	310.55
XS 0.56	276.10
XS 0.54	230.76
XS 0.52	182.53
XS 23.07	223.12
XS 23.05	196.57
XS 23.03	160.58
XS 23.01	126.68
XS 23.4	220.35
XS 23.25	180.26
XS 23.1	140.47
XS23.9	111.70
XS 23.7	97.26
XS 23.5	80.94

Table 15: Water Surface Levels (m) for 100-yr Event-Existing Condition

Cross Section	100 YR Water Surface Elevation (m)
XS 0.97	170.57
XS 0.95	113.03
XS 0.93	92.39
XS 0.91	76.65
XS 0.89	120.75
XS 0.87	86.24
XS 0.85	68.48
XS 0.84	62.89

5.4.2 Proposed Condition

In order to determine the construction limit of the proposed project, an encroachment analysis is performed for the unnamed creeks and tributaries. The analysis was made using as maximum increase +0.15m. The method of equal conveyance (#4) was used for this analysis.

The proposed condition also presents the design of ten (10) bridge crossings along the unnamed creeks and tributaries. The Contech Multi-Plate System is proposed at the crossings, specifically the arch shapes. The plates are delivered to the job site and bolted together to form a Multi-Plate structure. The arch shapes are designed for applications where an open invert is desired. They are available with spans from 1.83 m to 7.6 m. Additional information on this system is presented in Appendix E.

Table 16 presents the design parameters of the bridges which location is presented in **Figure 18**. **Figure 31** through **Figure 40** present the schematic design of the bridges.

Table 16: Design Parameters for Proposed Bridge Crossings

Crossing	No. Spans (Archs)	Arch Width (m)	Arch Rise (m)
1	2	5.18	2.7
2	2	4.57	2.36
3	2	5.49	2.72
4	1	3.96	2.06
5	1	5.18	2.69
6	1	3.96	2.06
7	1	3.96	2.06
8	2	5.79	1.93
9	3	6.7	2.7
10	1	3.66	1.52

Table 17 presents a comparison between existing and proposed 100-yr water surface elevations at the unnamed creeks. The proposed condition model includes the encroachment and the proposed hydraulic structures described above.

Table 17: Existing and Proposed 100-yr Water Surface Elevations (m) at Unnamed Creeks

Cross Section	Existing	Proposed	Difference (m)
Unnamed Creek 1			
XS 78	84.36	84.36	0
XS 76	81.28	80.92	-0.36
XS 74	78.12	77.96	-0.16
XS 72	69.03	68.82	-0.21
XS 70	66.64	66.47	-0.17
XS 68	62.89	64.17	1.28
XS 66	58.93	58.98	0.05
XS 64	57.18	57.33	0.15
XS 62	53.53	53.07	-0.46
XS 60	53.12	53.65	0.53
XS 58	53.06	53.32	0.26
XS 56	53.06	53.30	0.24
XS 54	52.64	52.49	-0.15
XS 52	51.44	51.35	-0.09
XS 87	80.40	80.40	0
XS 85	74.15	74.15	0

Table 17: Existing and Proposed 100-yr Water Surface Elevations (m) at Unnamed Creeks

Cross Section	Existing	Proposed	Difference (m)
XS 83	66.40	66.13	-0.27
XS 81	61.37	60.56	-0.81
XS 79	58.77	58.40	-0.37
XS 718	78.49	78.33	-0.16
XS 716	66.75	66.54	-0.21
XS 714	56.46	56.28	-0.18
XS 712	54.31	54.23	-0.08
XS 710	53.07	53.32	0.25
Unnamed Creek 2			
XS 50	90.62	91.32	0.7
XS 48	74.39	74.43	0.04
XS 46	66.70	66.45	-0.25
XS 44	62.64	62.49	-0.15
XS 42	59.23	59.04	-0.19
XS 40	58.39	58.29	-0.1
XS 38	54.63	54.60	-0.03
XS 36	52.71	52.68	-0.03

Table 17: Existing and Proposed 100-yr Water Surface Elevations (m) at Unnamed Creeks (Cont.)

Cross Section	Existing	Proposed	Difference (m)
Unnamed Creek 3			
XS 35	80.73	80.65	-0.08
XS 33	70.69	70.45	-0.24
XS 31	65.47	65.46	-0.01
XS 29	62.63	63.13	0.5
XS 27	58.71	58.65	-0.06
XS 25	54.25	54.20	-0.05
XS 35.5	142.44	142.39	-0.05
XS 35.3	104.25	104.43	0.18
XS 35.1	78.21	78.29	0.08
XS 26.6	76.10	76.07	-0.03
XS 26.4	64.61	64.53	-0.08
XS 26.25	59.07	59.12	0.05
Unnamed Creek 4			
XS 23	271.36	271.36	0
XS 21	241.3	241.3	0
XS 20	200.58	201.13	0.55

Table 17: Existing and Proposed 100-yr Water Surface Elevations (m) at Unnamed Creeks (Cont.)

Cross Section	Existing	Proposed	Difference (m)
XS 18.5	157.01	156.47	-0.54
XS 17.5	138.67	138.53	-0.14
XS 16	119.61	119.29	-0.32
XS 15	111.77	111.83	0.06
XS 13	95.16	95.19	0.03
XS 11	82.83	82.76	-0.07
XS 9	74.67	74.58	-0.09
XS 7	65.39	65.30	-0.09
XS 5	65.56	65.0	-0.56
XS 3	59.71	59.49	-0.22
XS 1.5	57.26	57.06	-0.2
XS 0.2	232.71	232.71	0
XS 0.09	208.37	208.37	0
XS 0.07	174.65	174.31	-0.34
XS 0.05	155.17	154.57	-0.6
XS 0.03	131.04	130.47	-0.57
XS 0.02	119.39	118.49	-0.9

Table 17: Existing and Proposed 100-yr Water Surface Elevations (m) at Unnamed Creeks (Cont.)

Cross Section	Existing	Proposed	Difference (m)
XS 0.5	232.56	232.56	0
XS 0.3	206.49	206.49	0
XS 0.24	180.62	180.54	-0.03
XS 0.22	160.88	160.80	-0.08
XS 0.58	310.55	310.55	0
XS 0.56	276.10	276.10	0
XS 0.54	230.76	230.65	-0.11
XS 0.52	182.53	182.46	-0.07
XS 23.07	223.13	223.13	0
XS 23.05	196.79	196.74	0.18
XS 23.03	160.65	160.60	-0.05
XS 23.01	126.69	126.64	-0.05
XS 23.4	220.35	220.35	0
XS 23.25	180.1	180.1	0
XS 23.1	140.25	140.18	-0.07
XS 23.9	111.71	111.70	-0.01
XS 23.7	97.26	97.26	0

Table 17: Existing and Proposed 100-yr Water Surface Elevations (m) at Unnamed Creeks (Cont.)

Cross Section	Existing	Proposed	Difference (m)
XS 23.5	80.94	80.94	0
XS 0.97	170.58	170.58	0
XS 0.95	113.04	113.04	0
XS 0.93	92.36	92.36	0
XS 0.91	76.6	76.54	-0.06

As can be seen in the results, the proposed 100-yr water surface elevations do not exceed by more than 0.15m the existing elevations outside property limits in compliance with Planning Board Regulation # 13. The increments beyond 0.15 meters occur within property limits without affecting any neighbor. The encroachment limits for the creeks are shown in **Figure 41**. **Figure 42** through **Figure 45** present the 100-yr water surface profiles for the existing and proposed conditions of the unnamed creeks.

6. CONCLUSIONS AND RECOMMENDATIONS

1. Existing and proposed (without detention) 100-yr peak discharges at onsite and offsite basins resulted in the following:

Basin	100YR
Existing	
Site 1	160.9
Site 2	32.4
Site 3A	590.4
Site 3B	293.3
Site 3C	494.2
Site 3D	529.7
Site 4	24.3
Site 5	82.0
Site 6	604.6
Site 7	63.6
Site 8A1	377.9
Site 8A2	452.7
Site 8A3	245.1
Site 8B	113.2
Site 9	35.4
Site 10A	432.6
Site 10B	21.7
Site 10C	141.5
Site 10D	189.6

Basin	100YR
Site 10E	484.0
Site 10F	36.3
Site 10G	48.0
Site 10H	138.9
Site 10I	122.3
Site 10J	21.2
Site 10K	135.0
Site 10L	99.6
Site 10M	107.2
Site 11	117.5
Site 12	115.7
Offsite 1	151.0
Offsite 2	259.6
Offsite 3	186.2
Offsite 4	173.2
Offsite 5	265.6
Offsite 6	256.6
Offsite 6.5	37.9
Offsite 7	1,212.8
Offsite 8	331.3
Offsite 9	462.2
Offsite 10	292.7
Offsite 11	56.5

Basin	100YR
Proposed	
Site 1A	352.2
Site 1B	362.9
Site 1C	99.4
Site 1D	287.2
Site 2A	651.3
Site 2B1	167.1
Site 2B2	213.9
Site 2C	250.5
Site 2D1	76.7
Site 2D2	44
Site 2D3	164.3
Site 3A1	280.9
Site 3A2	148.2
Site 3B	368.5
Site 4A	594.2
Site 4B1	144
Site 4B2	127.3
Site 5A	344
Site 5B	224.4
Site 5C	73.6

Basin	100YR
Site 6A1	485.1
Site 6A2	692.6
Site 6B	740.4
Site 6C	219.9
Site 6D	204.5

2. Twelve (12) detention ponds are proposed to mitigate the increase in peak discharge produced by the development. The ponds will discharge to the unnamed creeks that cross the site. Pre- and post-development peak discharges at the creeks resulted in the following:

Point of Analysis	**2YR**		**100YR**	
	Existing	Proposed	Existing	Proposed
XS 51 (Unnamed Creek 1)	489	477	2,226	1,984
XS 36 (Unnamed Creek 2)	117	115	560	534
XS 24 (Unnamed Creek 3)	335	298	1,502	1,257
XS 1 (Unnamed Creek 4)	1,160	985	4,567	4,257

3. The proposed detention ponds reduce the proposed project peak discharge below the existing ones complying with Planning Board Regulation # 3.
4. The 100-yr water levels within the detention ponds correspond to the following:

Pond	100-yr Water Surface Elevation
1A	77.4
1B	67.4
1D	66.6
2A	59.0
2D2	56.3
2D3	55.7
3A	69.7
3B	66
4	61.3
5	72.4
6A	109.5
6B	69.5

The minimum finished floor elevation for the proposed buildings in the surrounding of the ponds should be set 1.0m higher than the 100-yr water level within them.

5. The site engineer shall design storm water systems for the proposed project site to discharge into the detention structures. The site grading must provide overland flow paths to direct stormwater to the detention ponds.
6. The proposed detention systems and outlet pipes should be inspected periodically to avoid obstruction with debris and to insure the removal of accumulated sediment.
7. An encroachment analysis was performed for the unnamed creeks and tributaries that cross the project site. The encroachment complies with Planning Board Regulation No.13 since water surface levels do not increase by more than 0.15 meters outside property limits in comparison with the existing levels.

8. Ten bridge crossings are proposed along the creeks. The Contech Multi-Plate System is proposed for the crossings which design parameters are presented in Section 5.4.2.

7. REFERENCES

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FIGURES

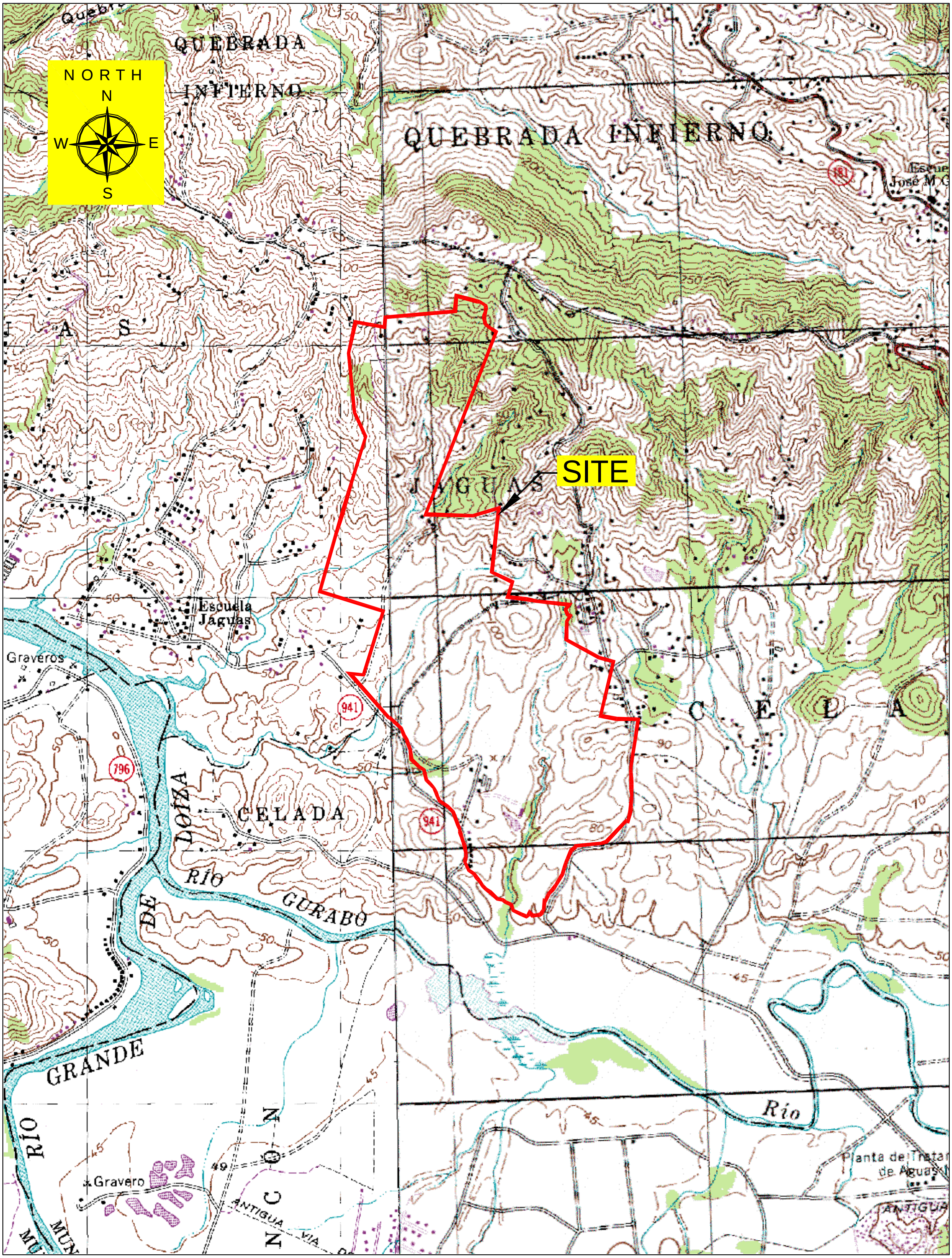


Figura 1: Proposed Project Location on a Partial Reproduction of the U.S.G.S. Topographic Quadrangle of Gurabo

Scale 1: 20,000

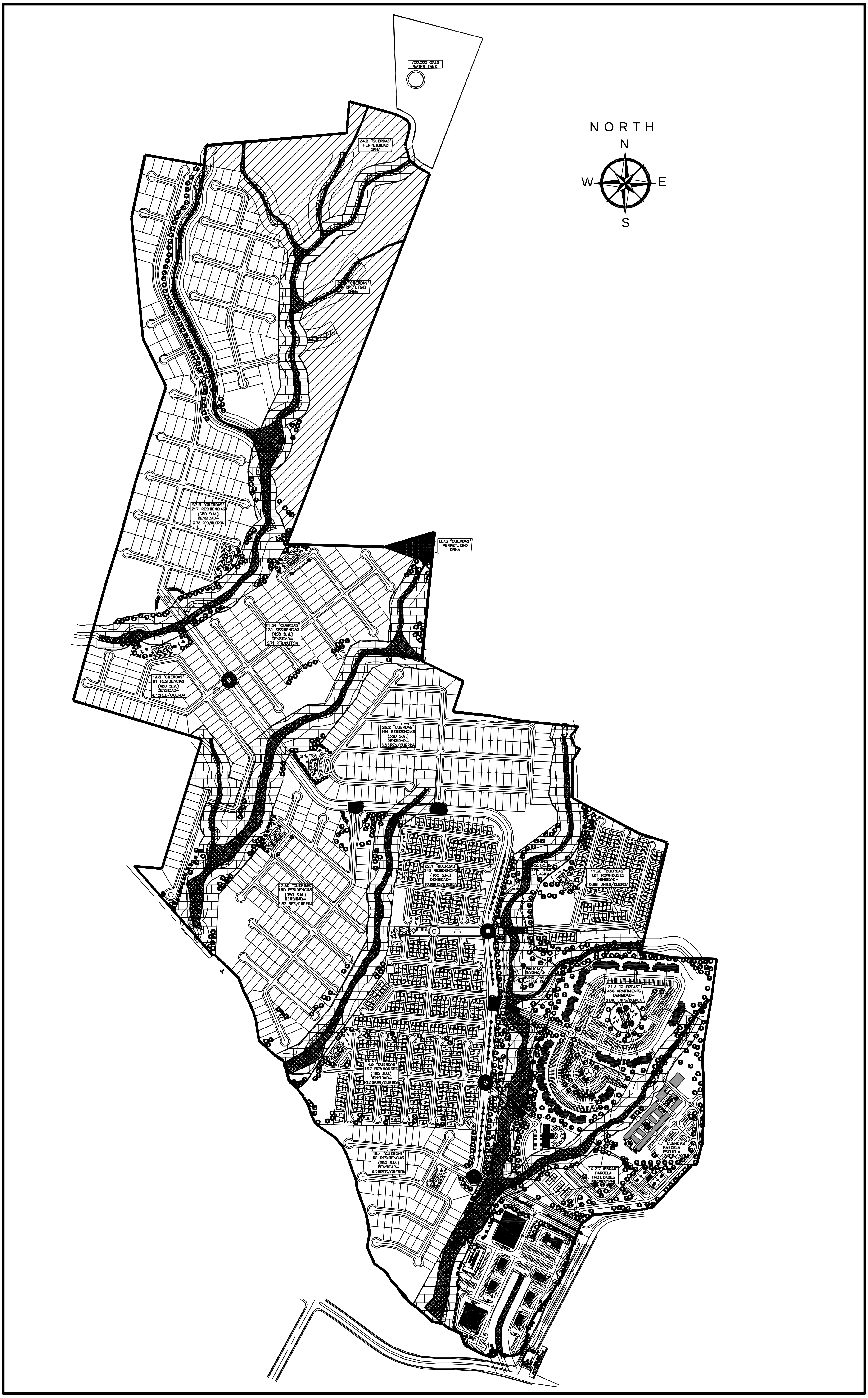


Figure 2: Proposed Project Layout

Scale 1: 7,000

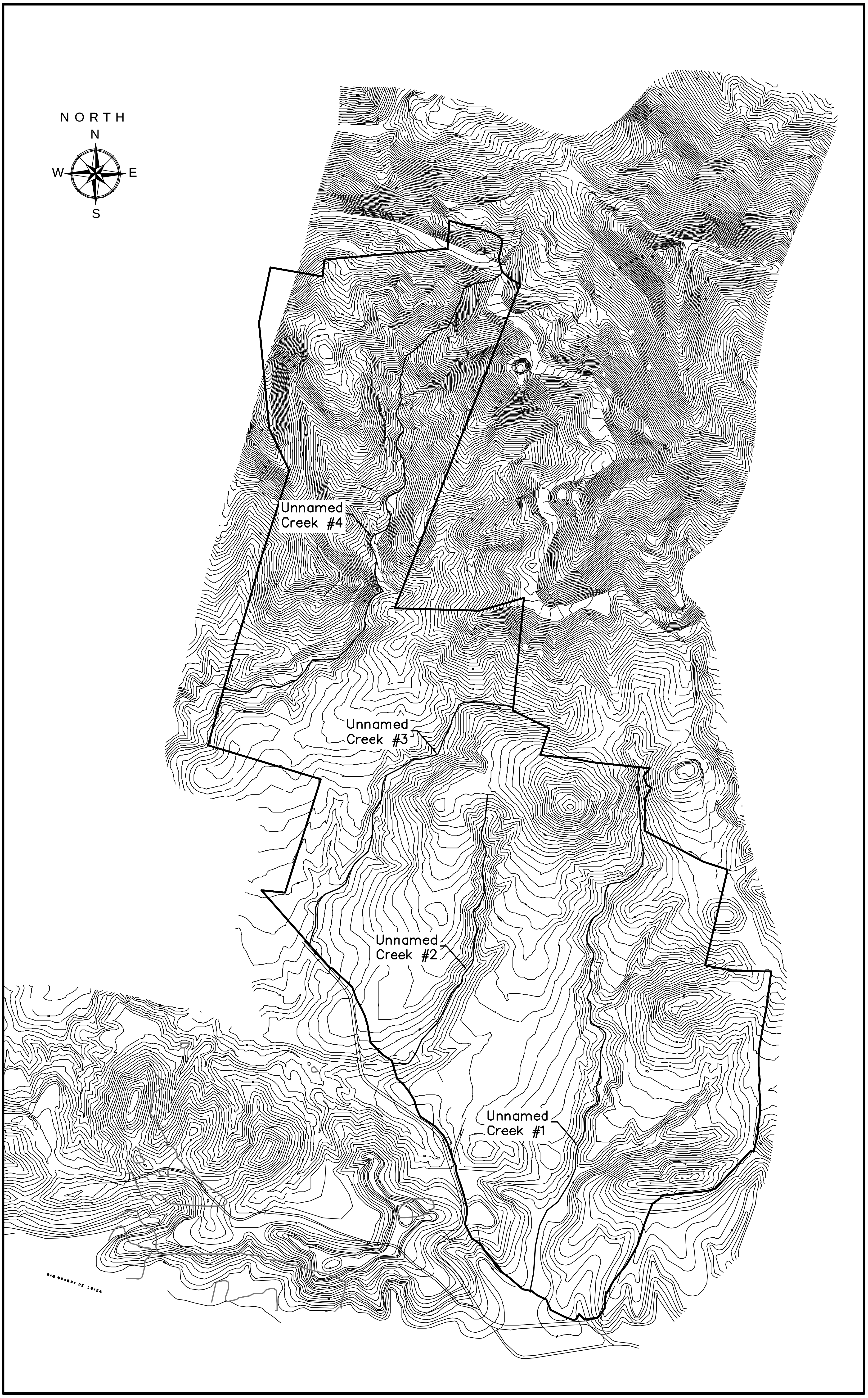


Figure 3: Project Site Topography and Existing Water Bodies

Scale 1: 8,000

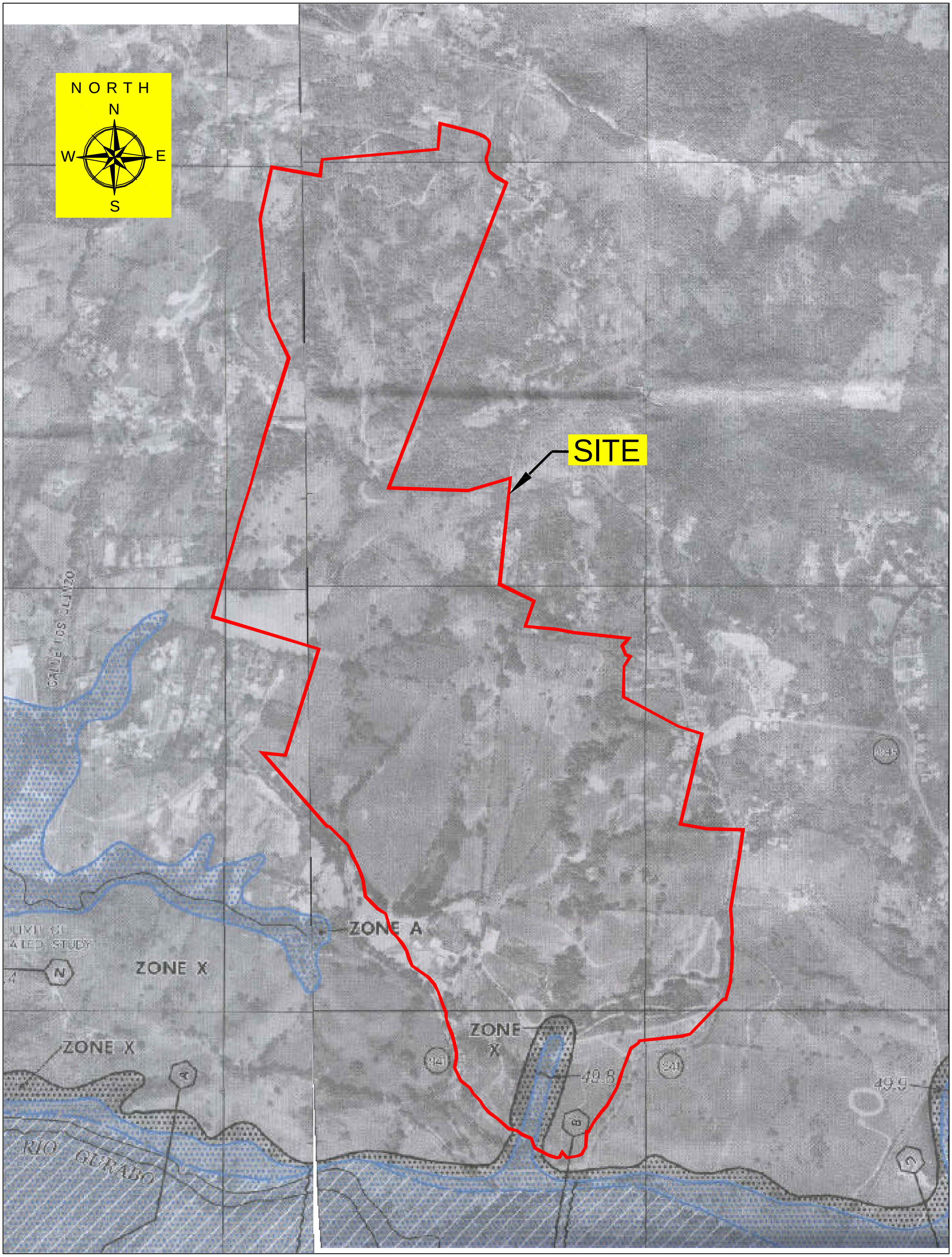


Figura 4: Project Site Area on a Partial Reproduction of the FEMA Firm Map, Panels# 745 & 765
Dated April 19, 2005
Scale 1: 20,000

Figure 5: Watershed Limits for Existing Condition of Project Site

Scale 1: 8,000

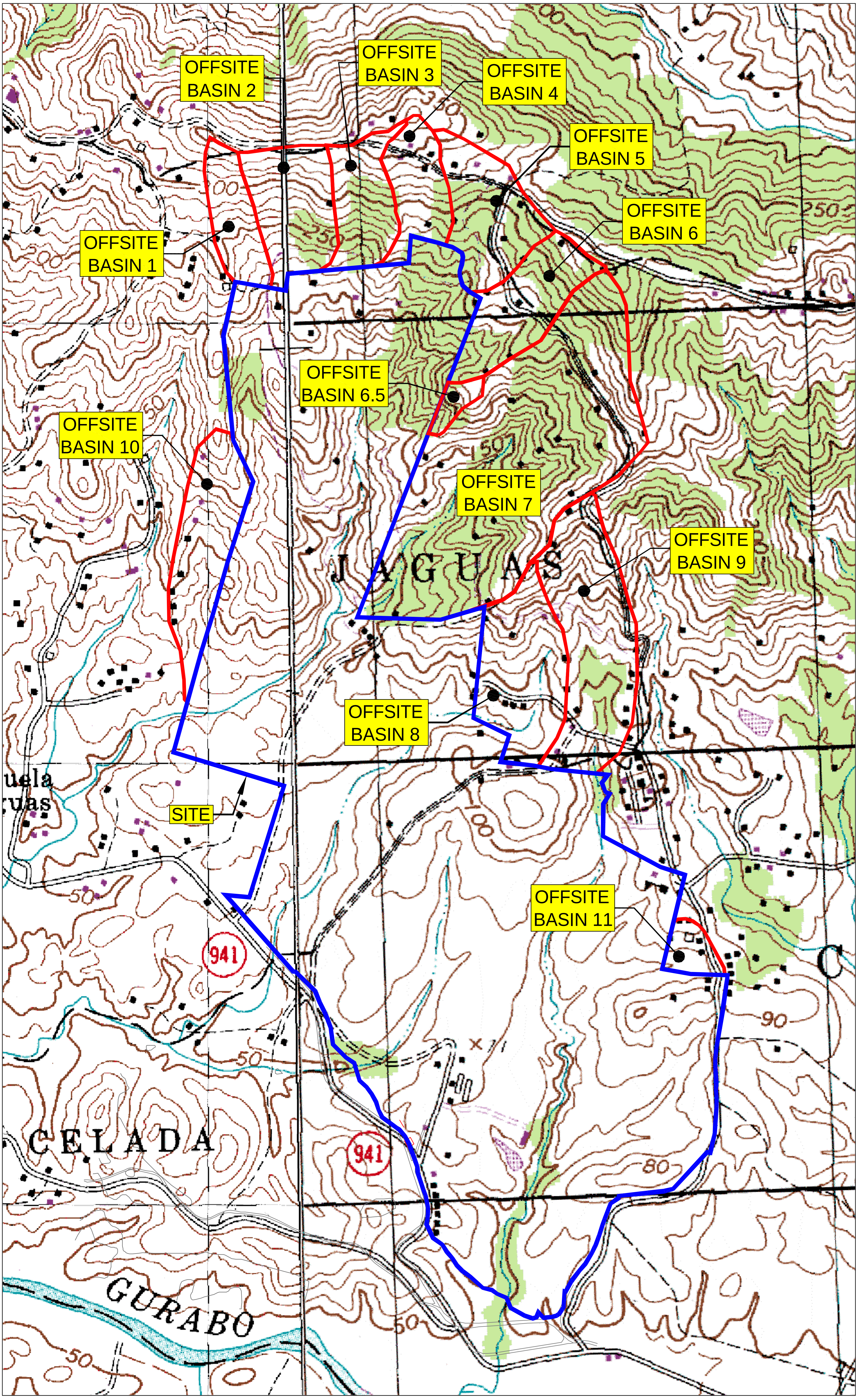


Figure 6: Watershed Limits for Offsite Basins

Scale 1: 8,000

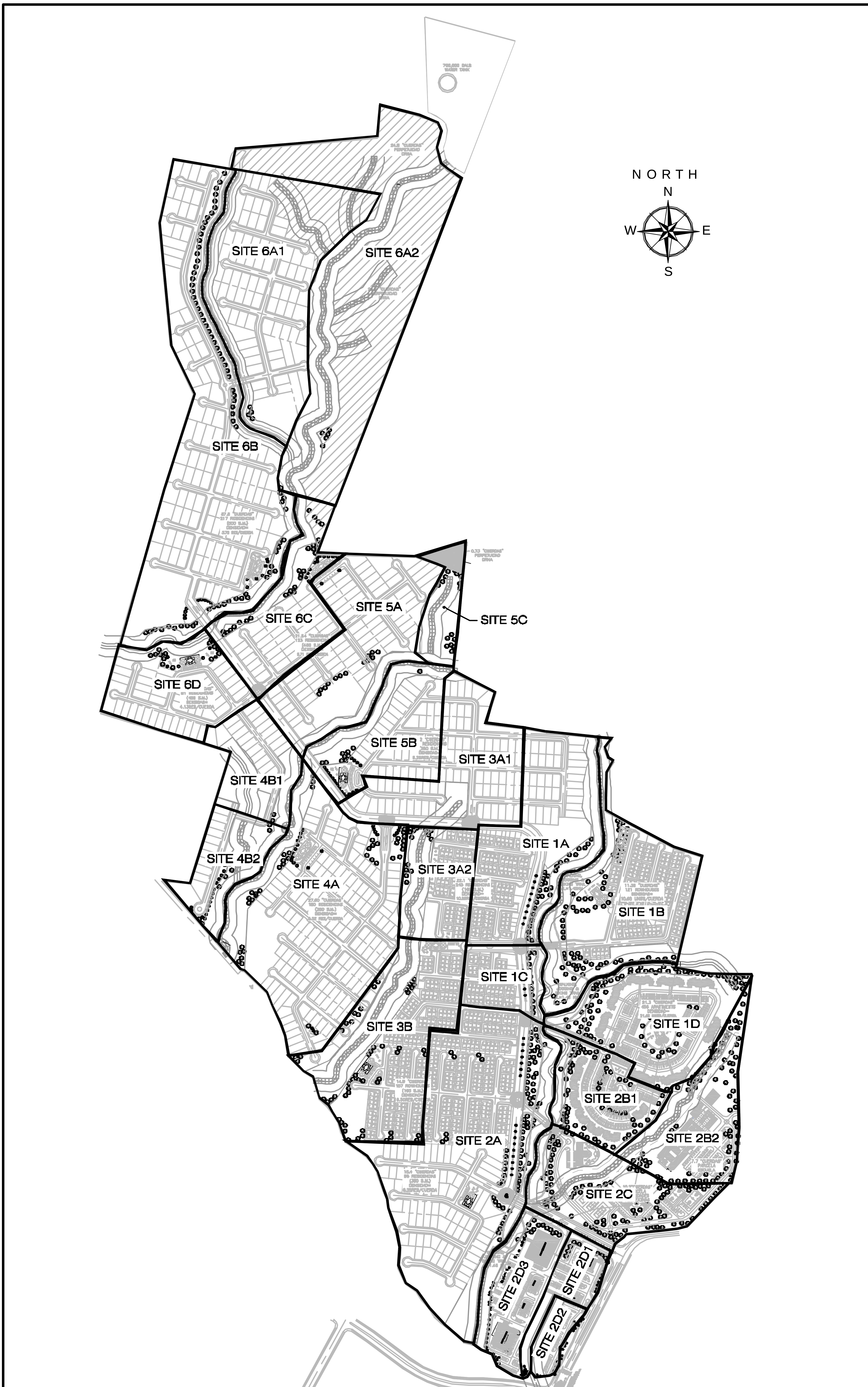


Figure 7: Watershed Limits for Project Site for Proposed Condition Scale 1: 7,000

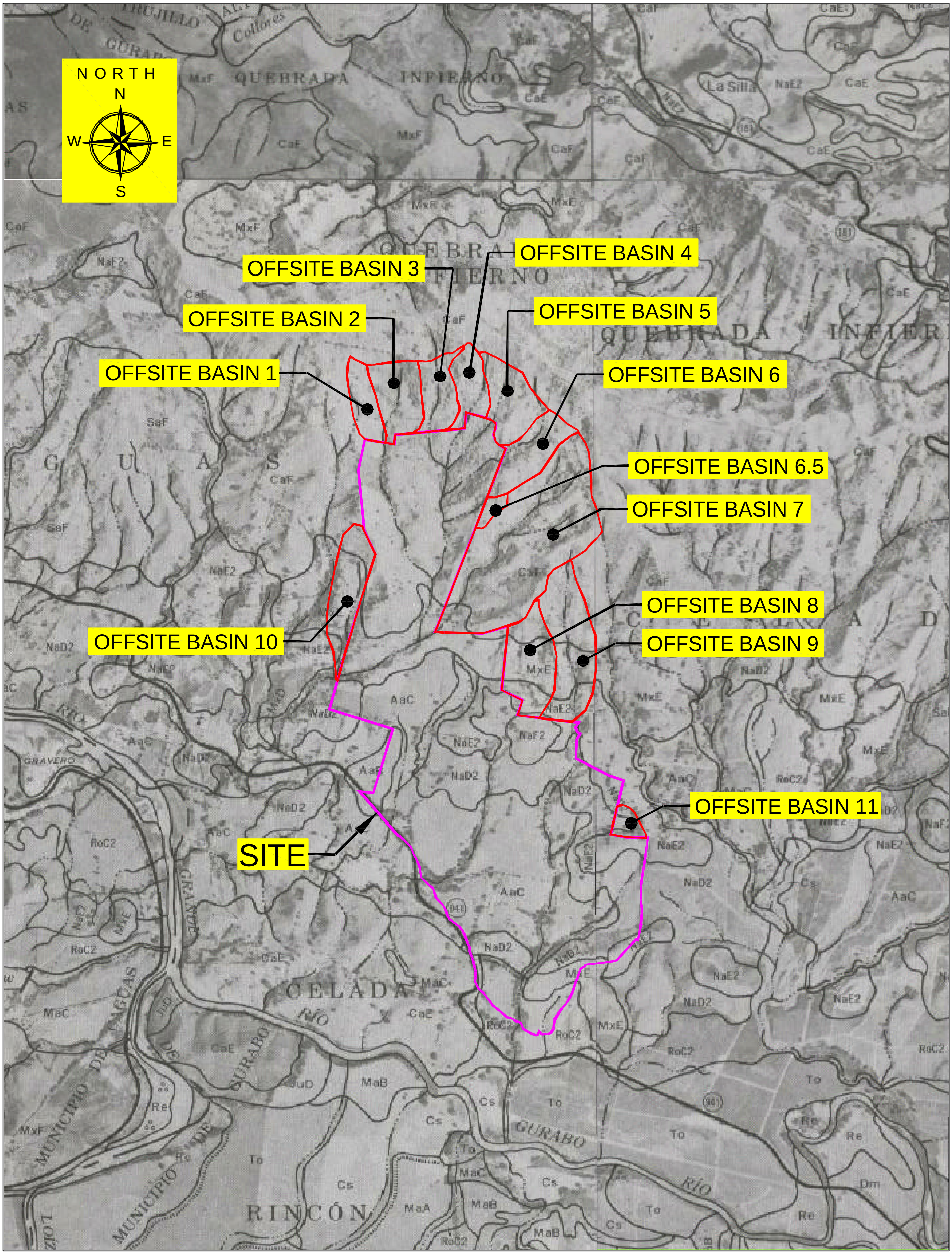


Figure 8: Project Location on a Partial Reproduction of the Soil Survey Map of San Juan Area, Sheet #27 Scale 1: 20,000

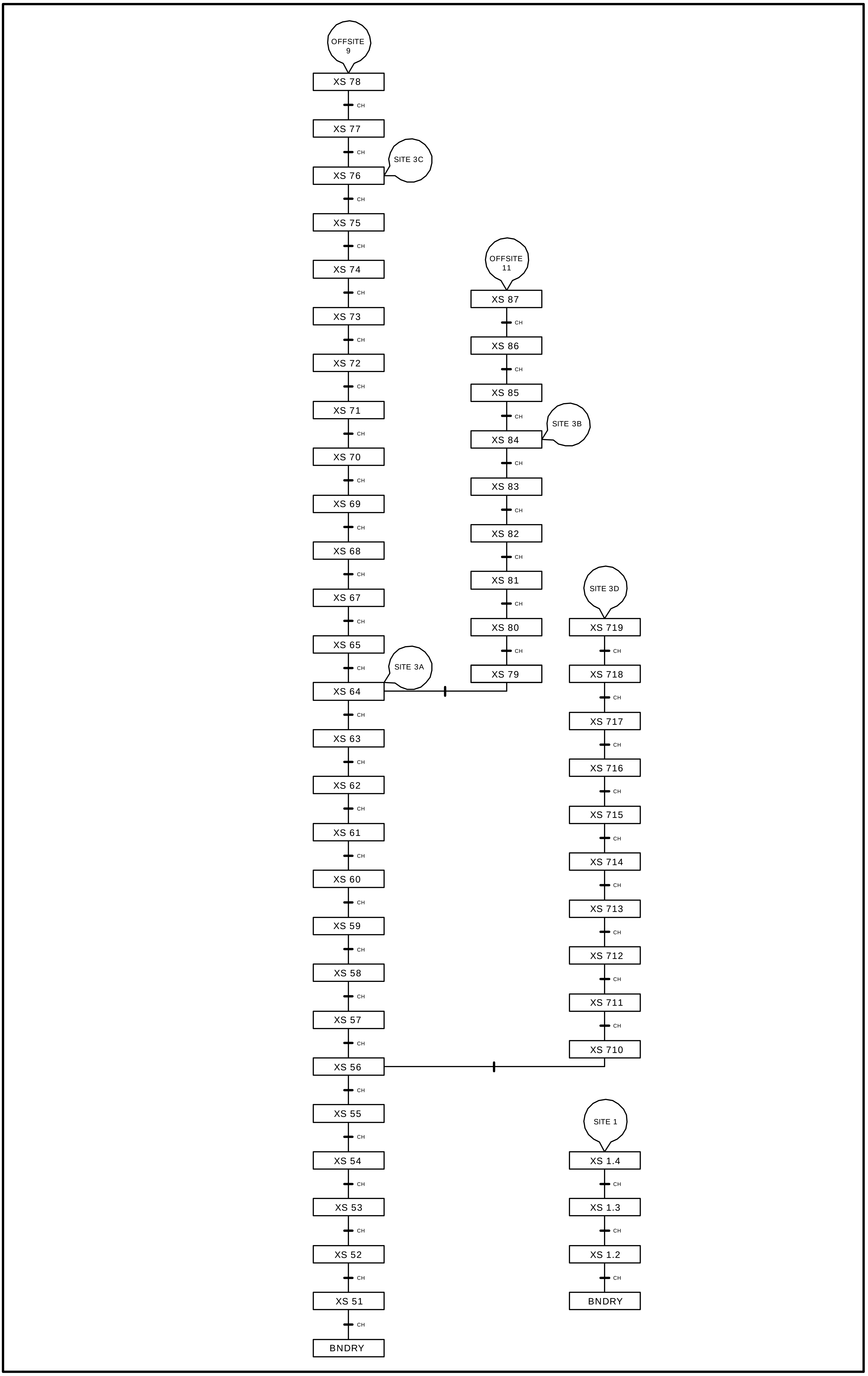


Figure 9: Schematic Link-Node Diagram for Existing Condition - Unnamed Creek 1

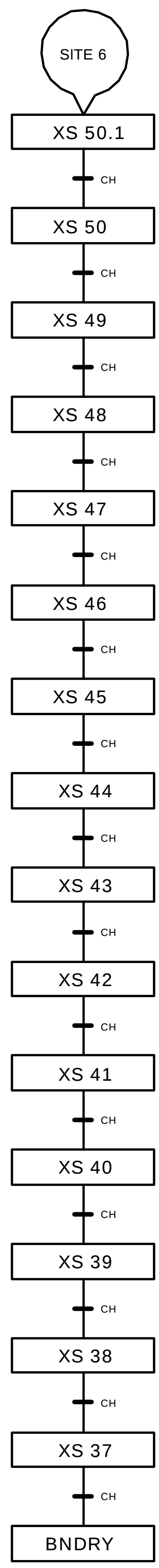


Figure 10: Schematic Link-Node Diagram for Existing Condition - Unnamed Creek 2

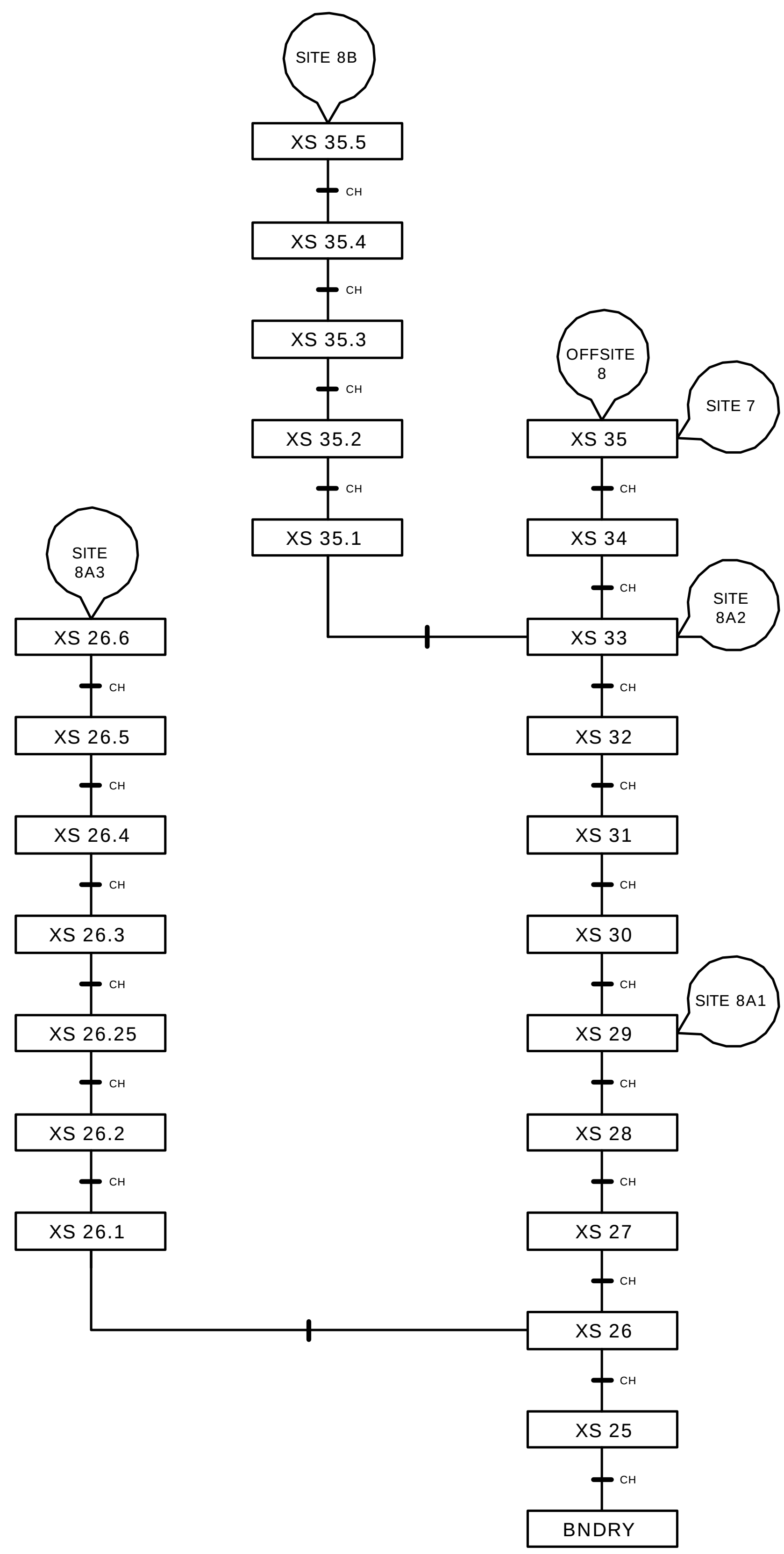


Figure 11: Schematic Link-Node Diagram for Existing Condition - Unnamed Creek 3

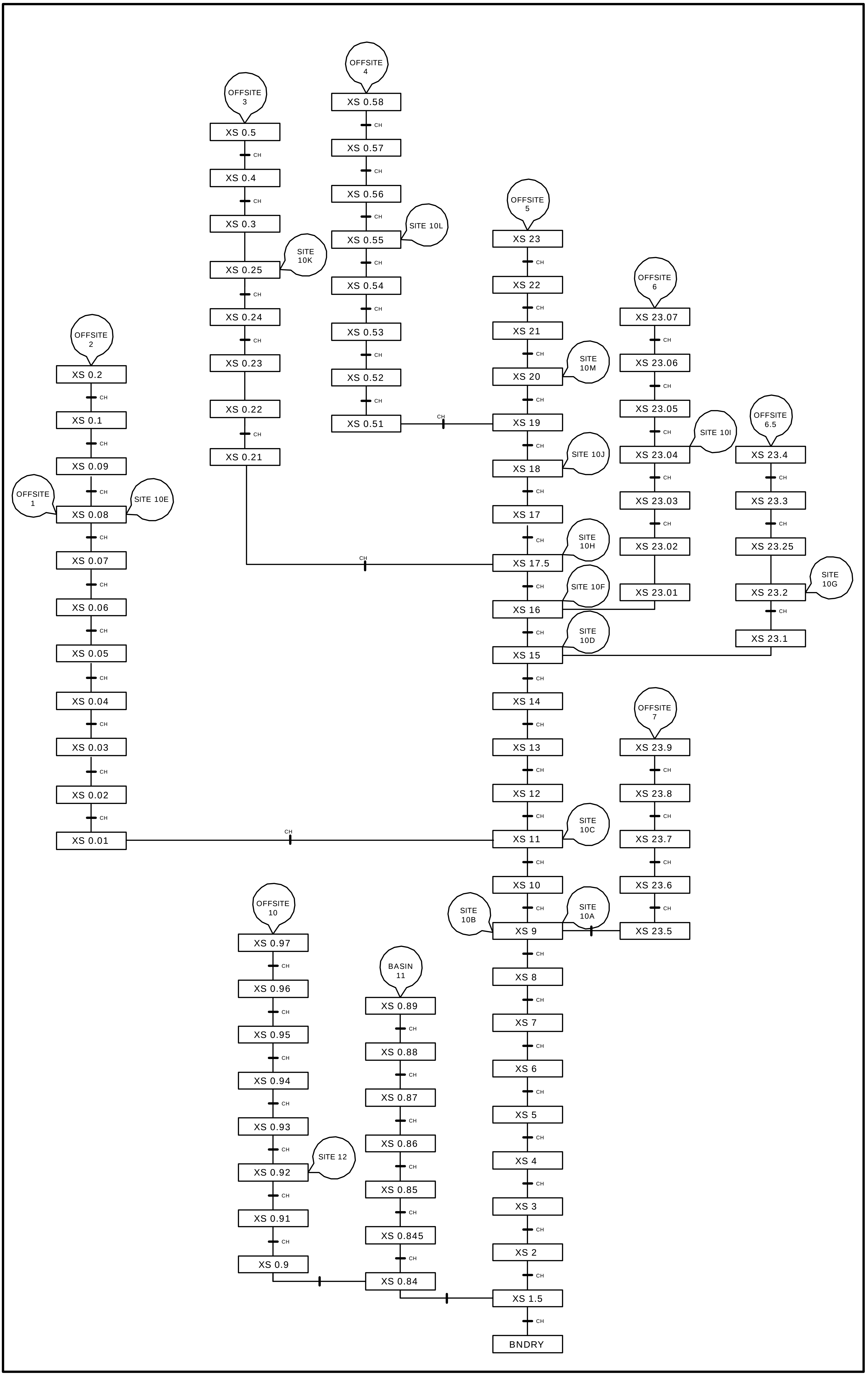


Figure 12: Schematic Link-Node Diagram for Existing Condition - Unnamed Creek 4

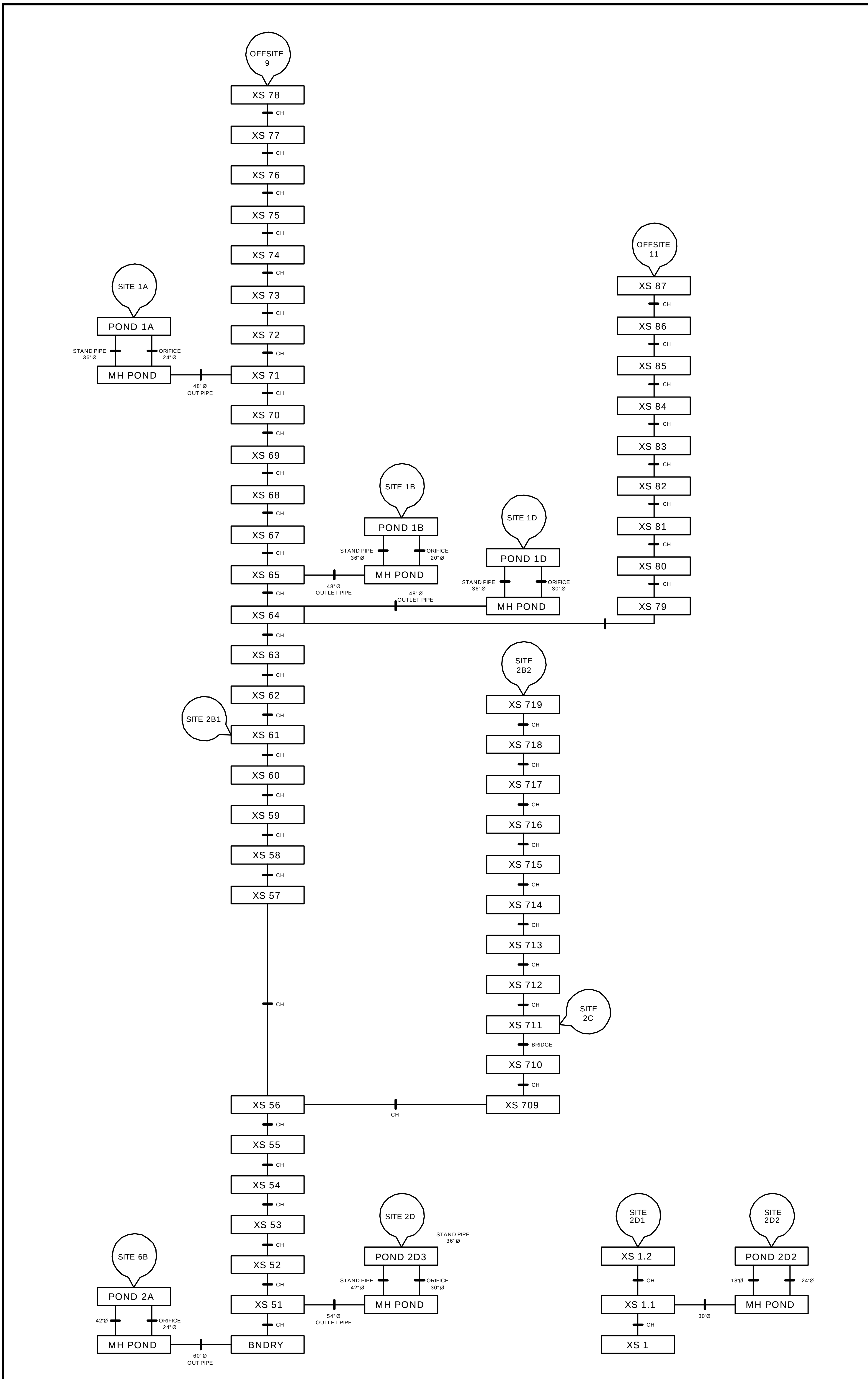


Figure 13: Schematic Link-Node Diagram for Proposed Condition - Unnamed Creek 1

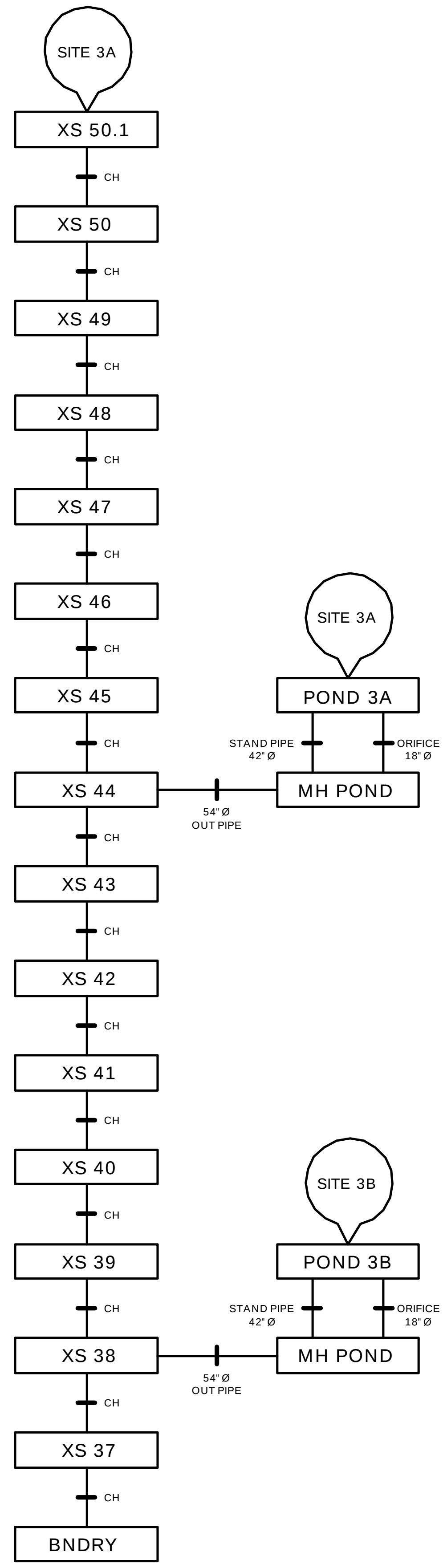


Figure 14: Schematic Link-Node Diagram for Proposed Condition - Unnamed Creek #2

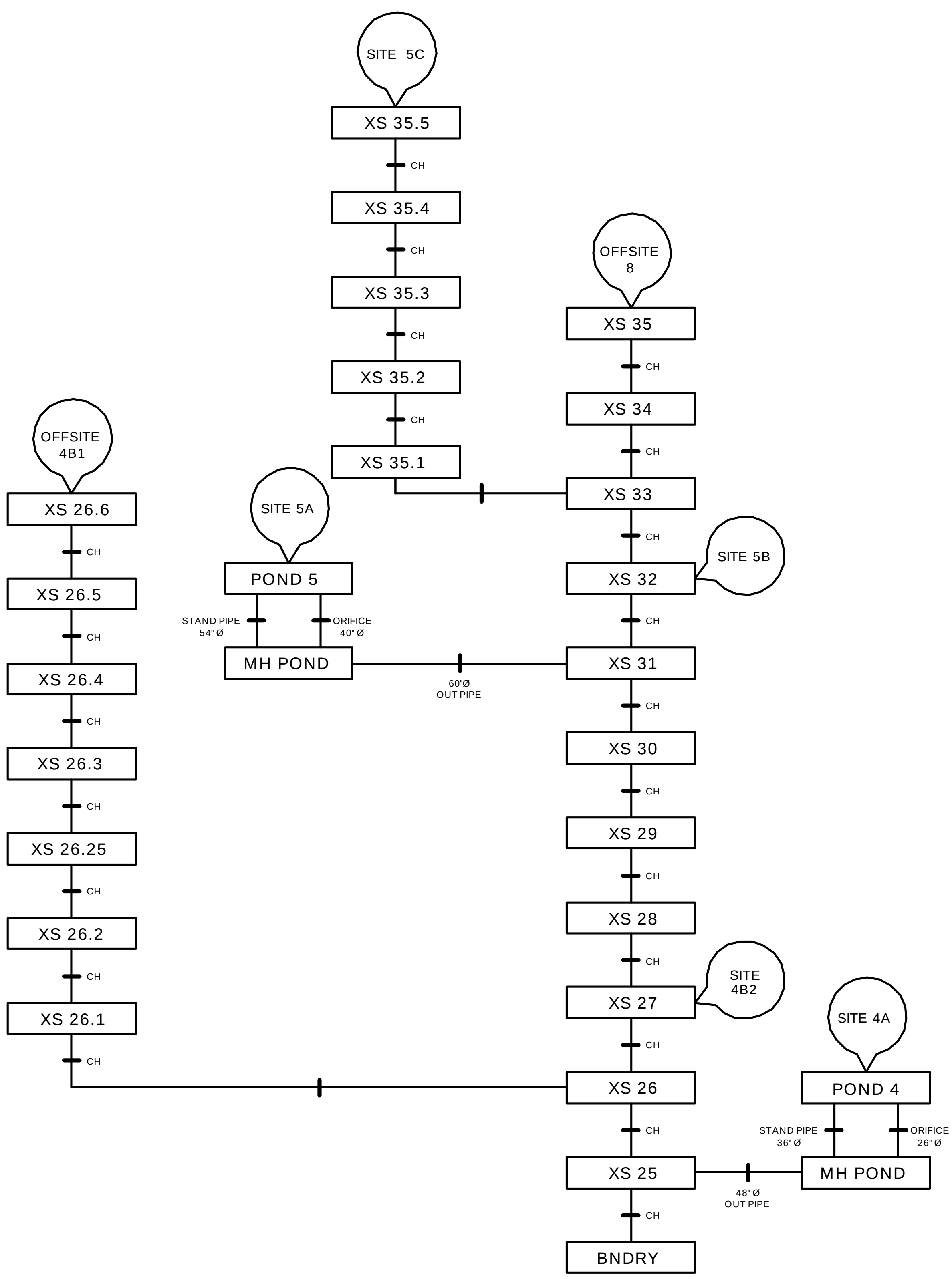


Figure 15: Schematic Link-Node Diagram for Proposed Condition - Unnamed Creek #3

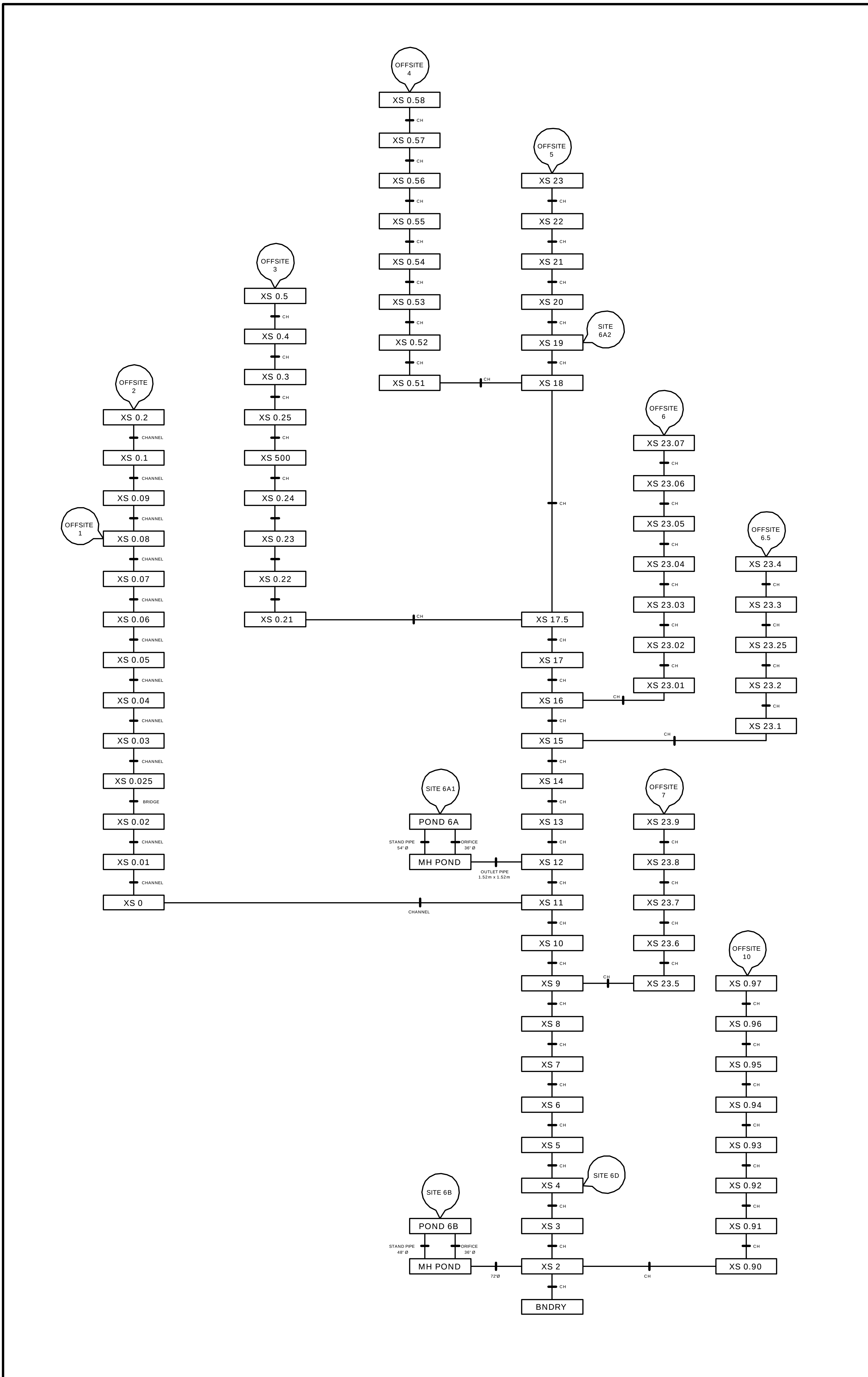


Figure 16: Schematic Link-Node Diagram for Proposed Condition - Unnamed Creek 4

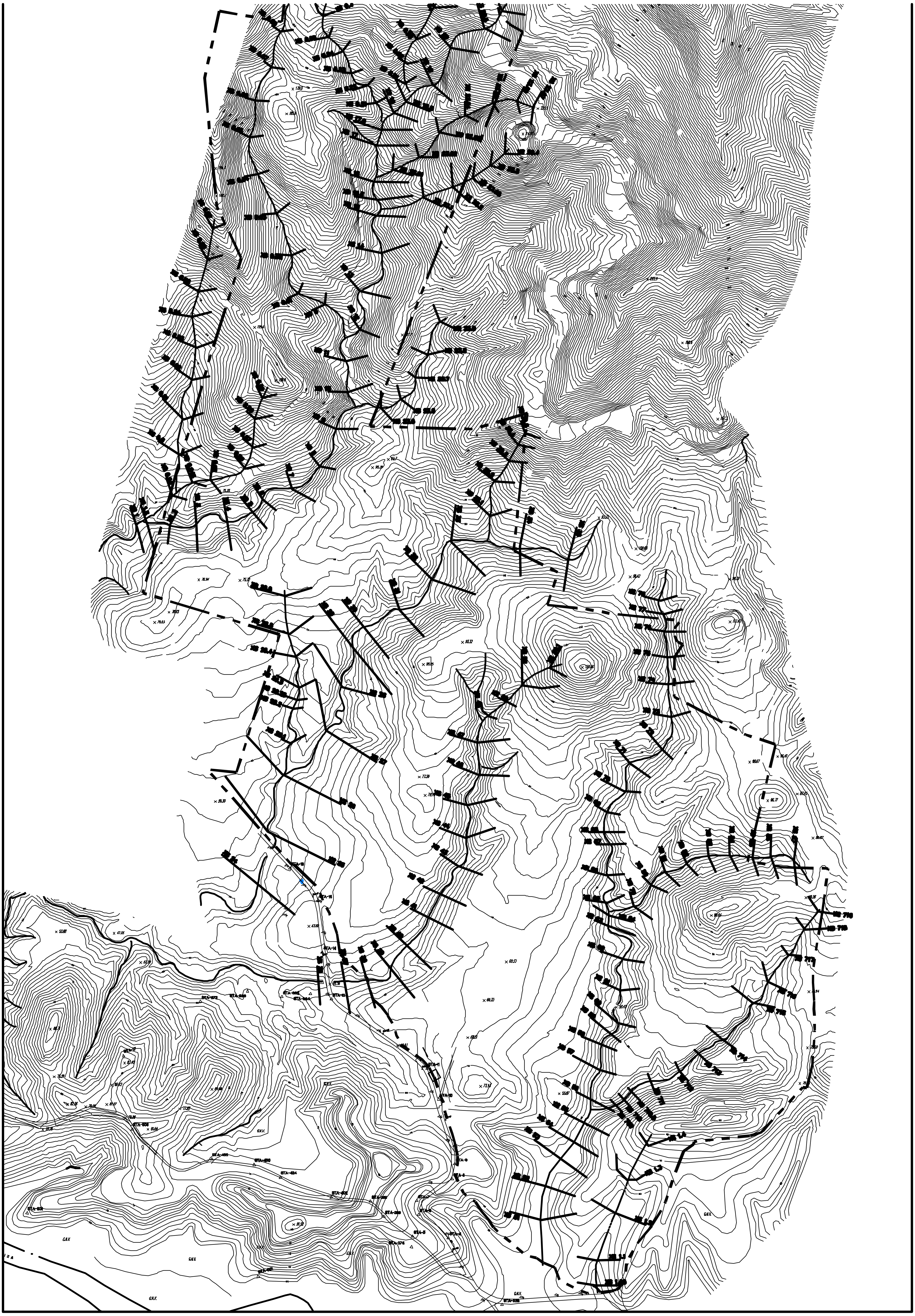


Figure 17: Cross Sections Location

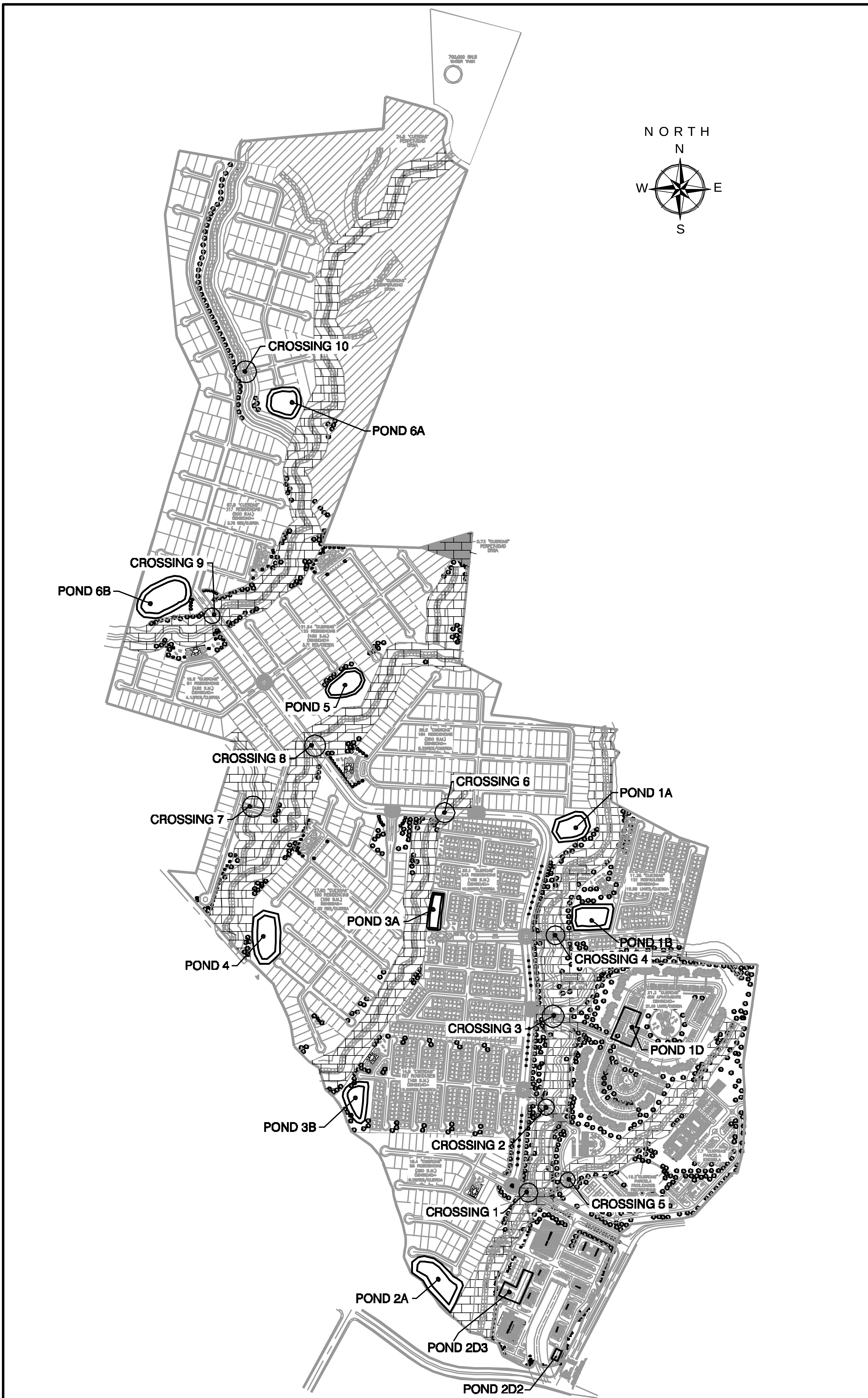


Figure 18: Location of Proposed Hydraulic Structures

Scale 1: 7,000

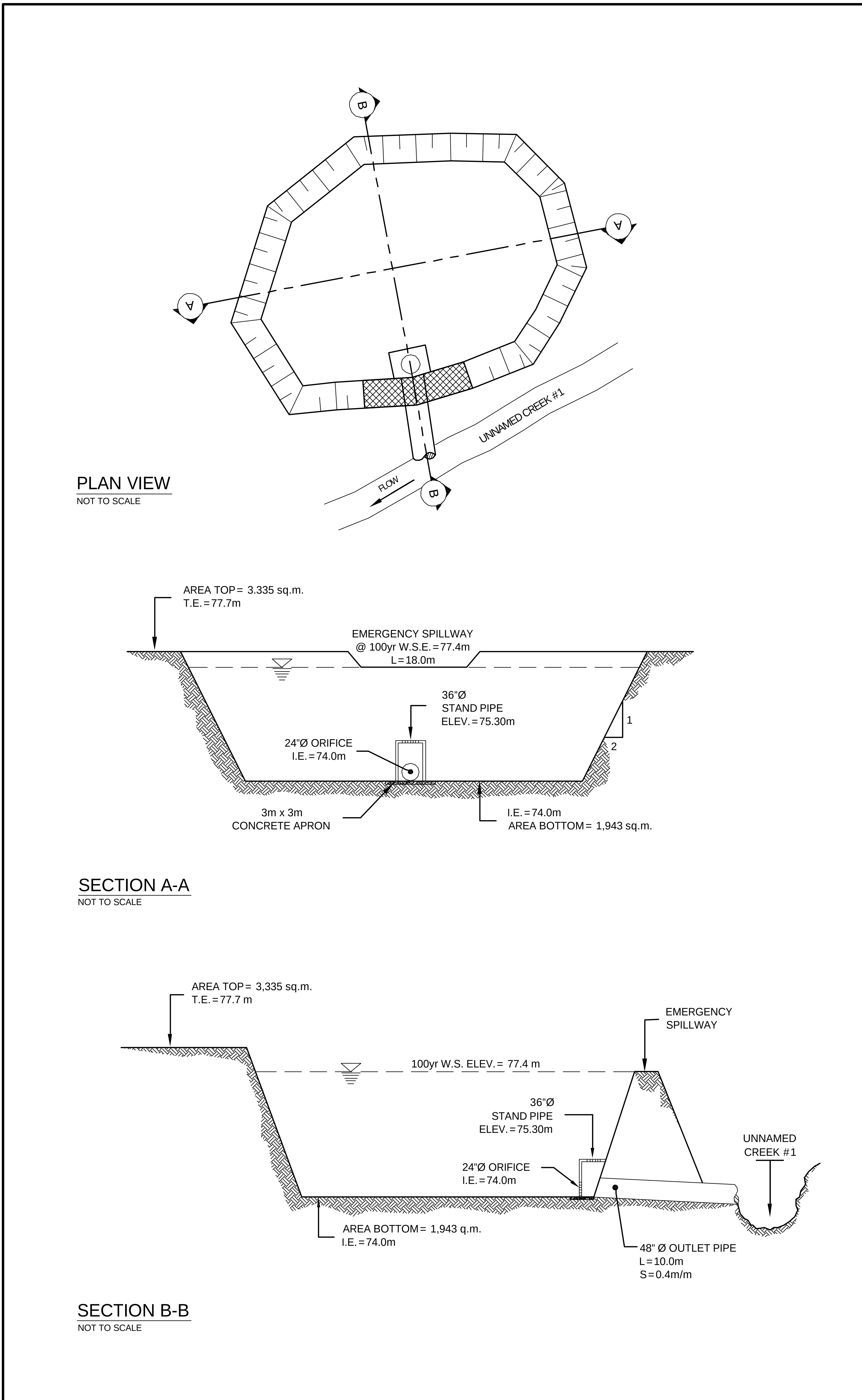


Figure 19: Schematic Design of Detention Pond 1A

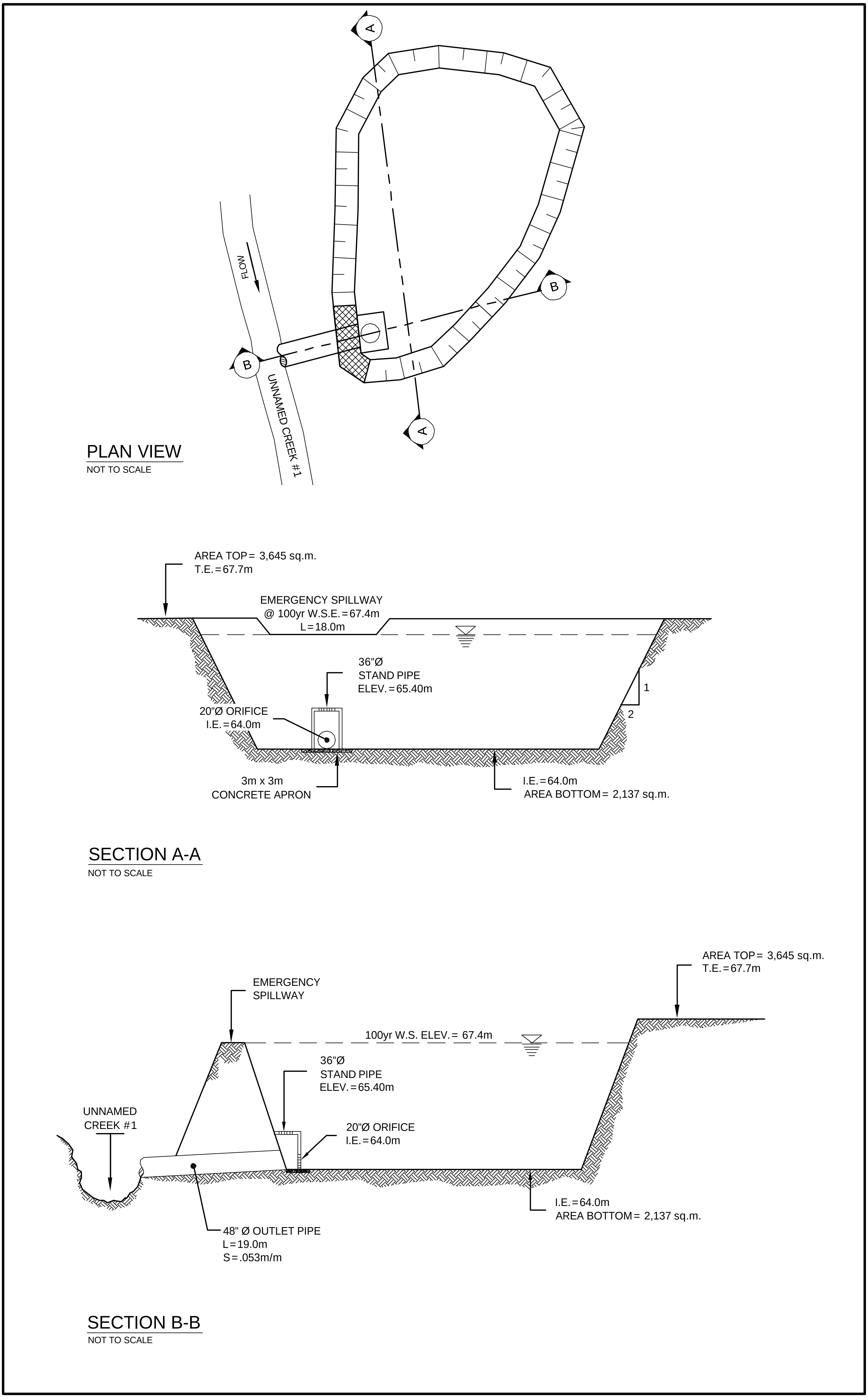


Figure 20: Schematic Design of Detention Pond 1B

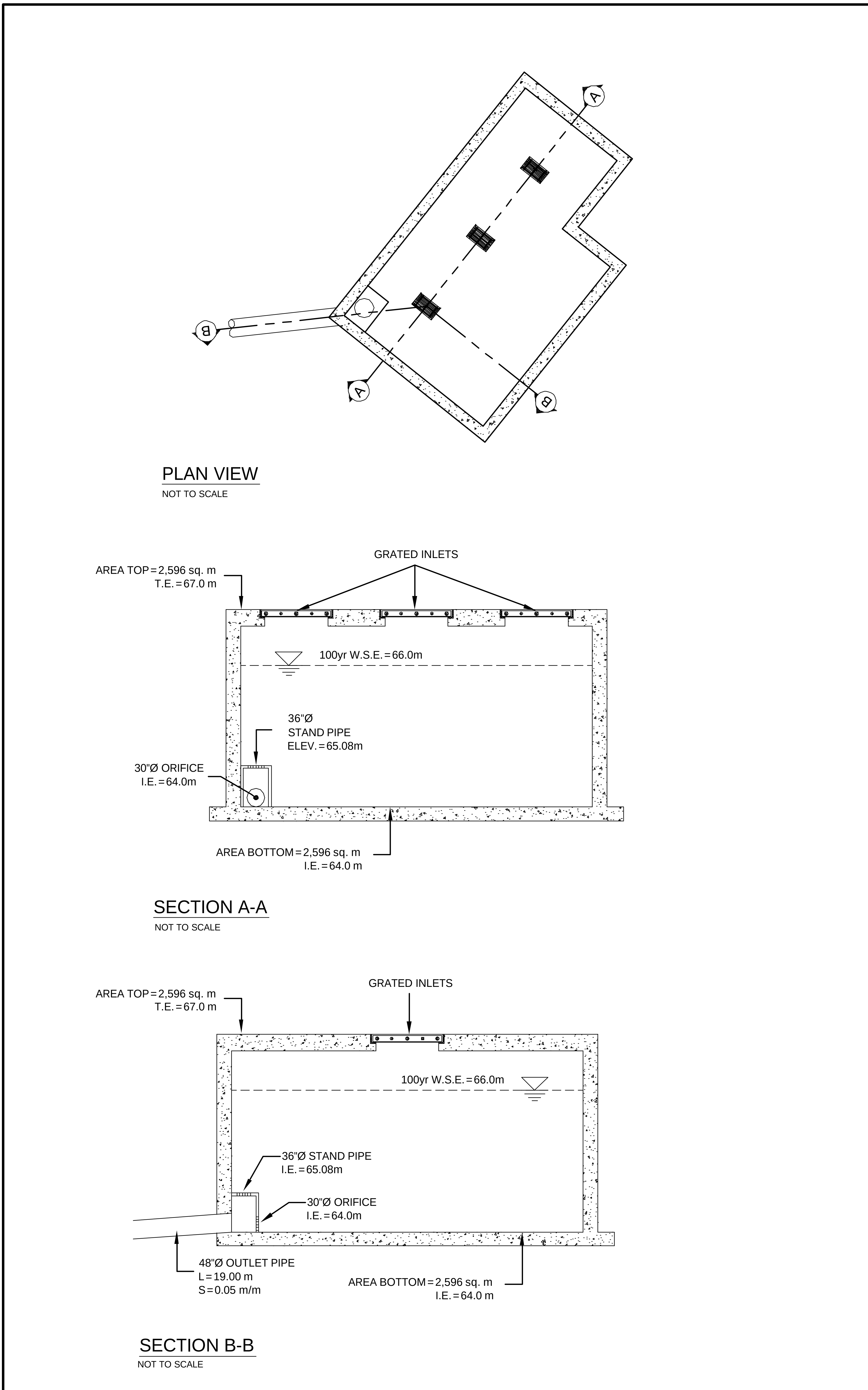
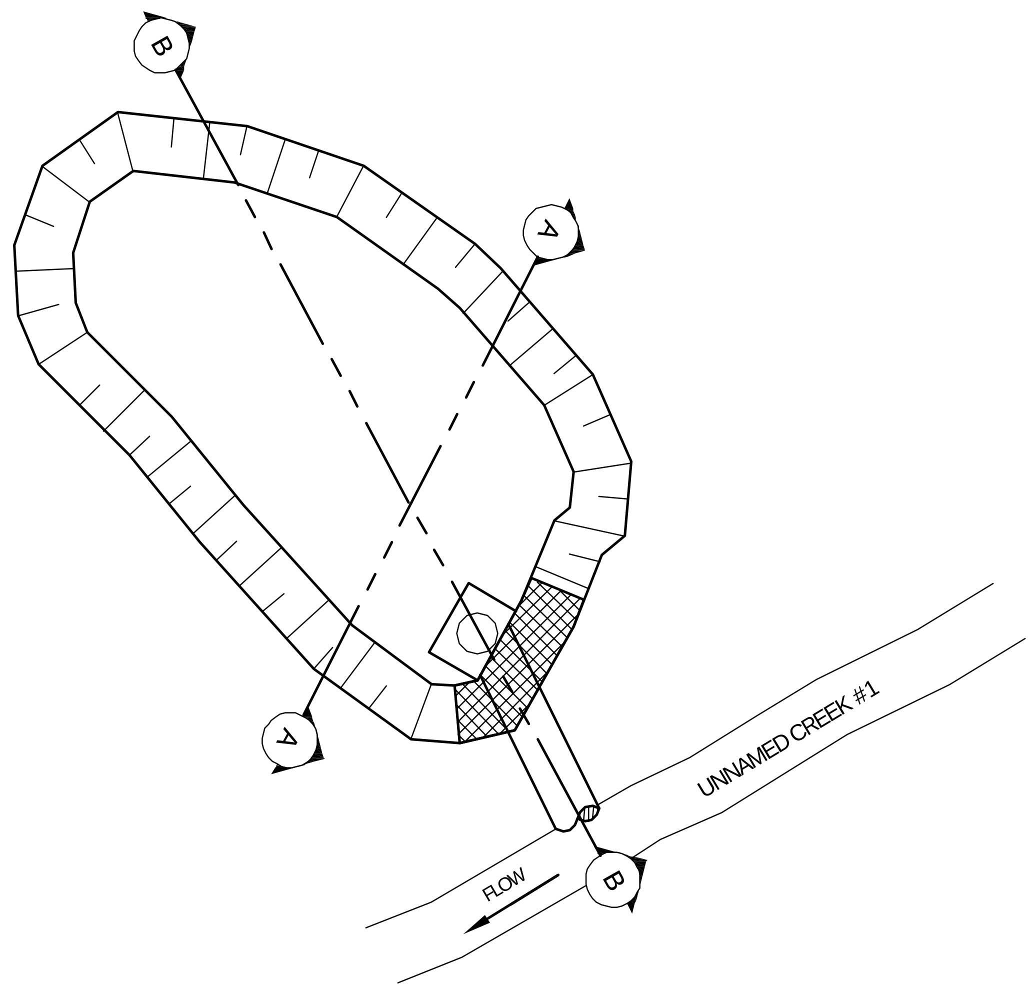
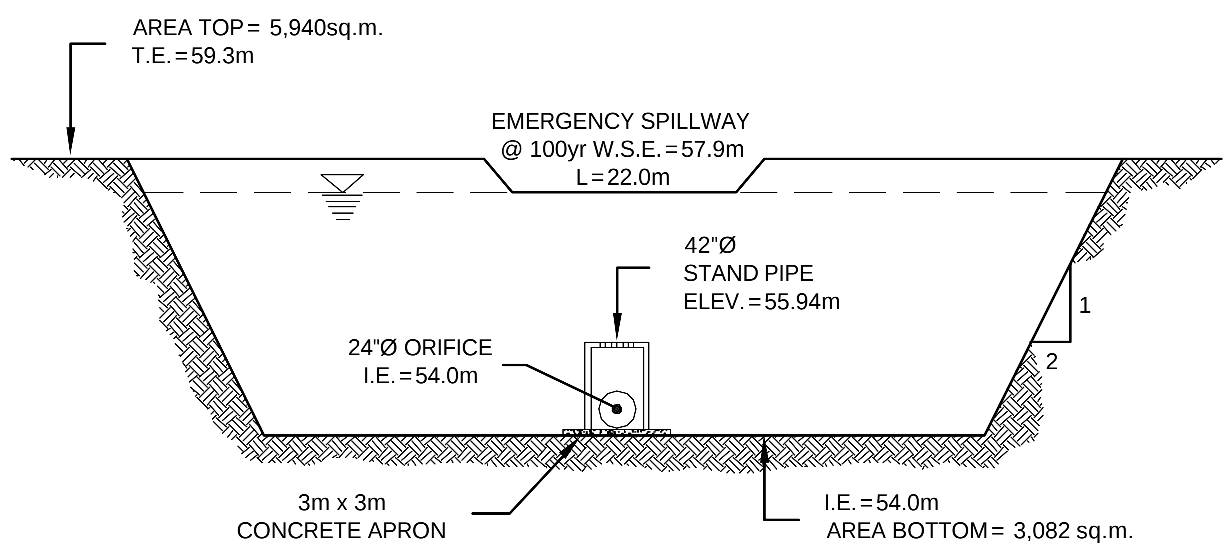


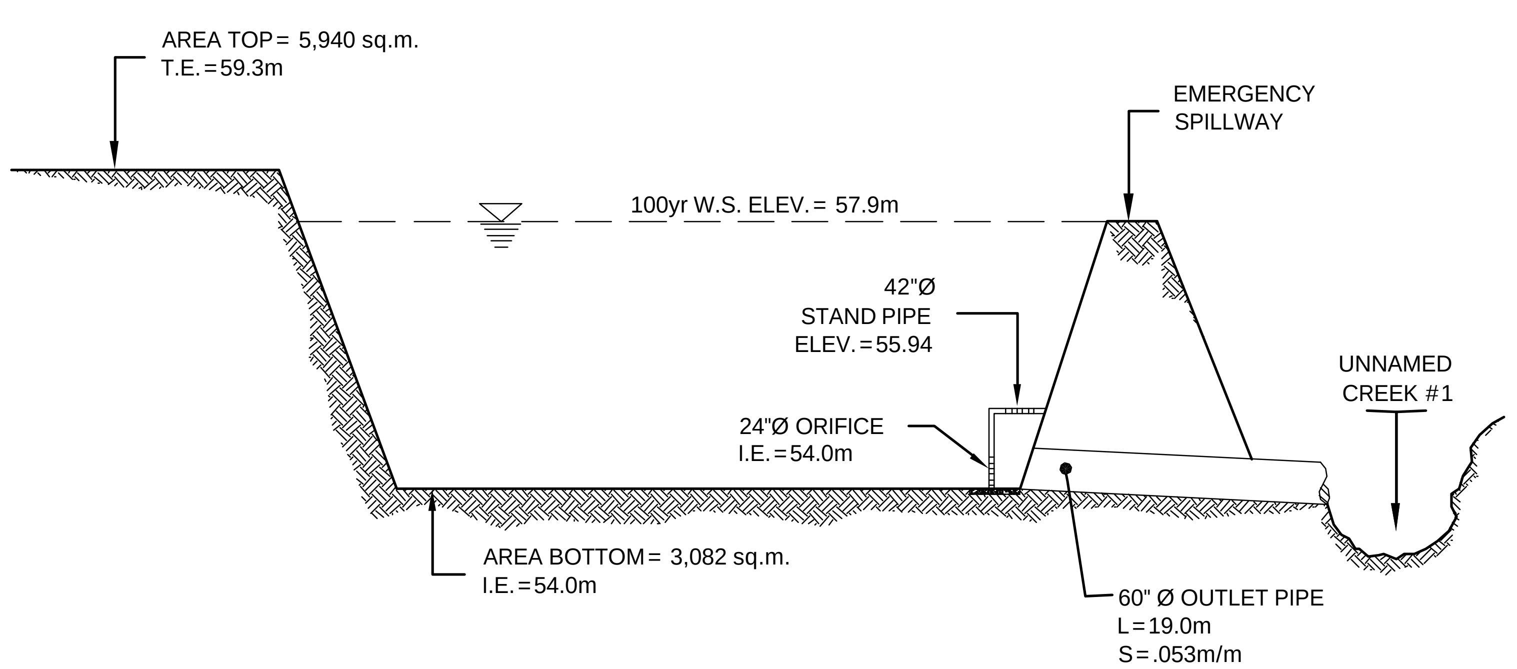
Figure 21: Schematic Design of Detention Pond 1D



PLAN VIEW
NOT TO SCALE



SECTION A-A
NOT TO SCALE



SECTION B-B
NOT TO SCALE

Figure 22: Schematic Design of Detention Pond 2A

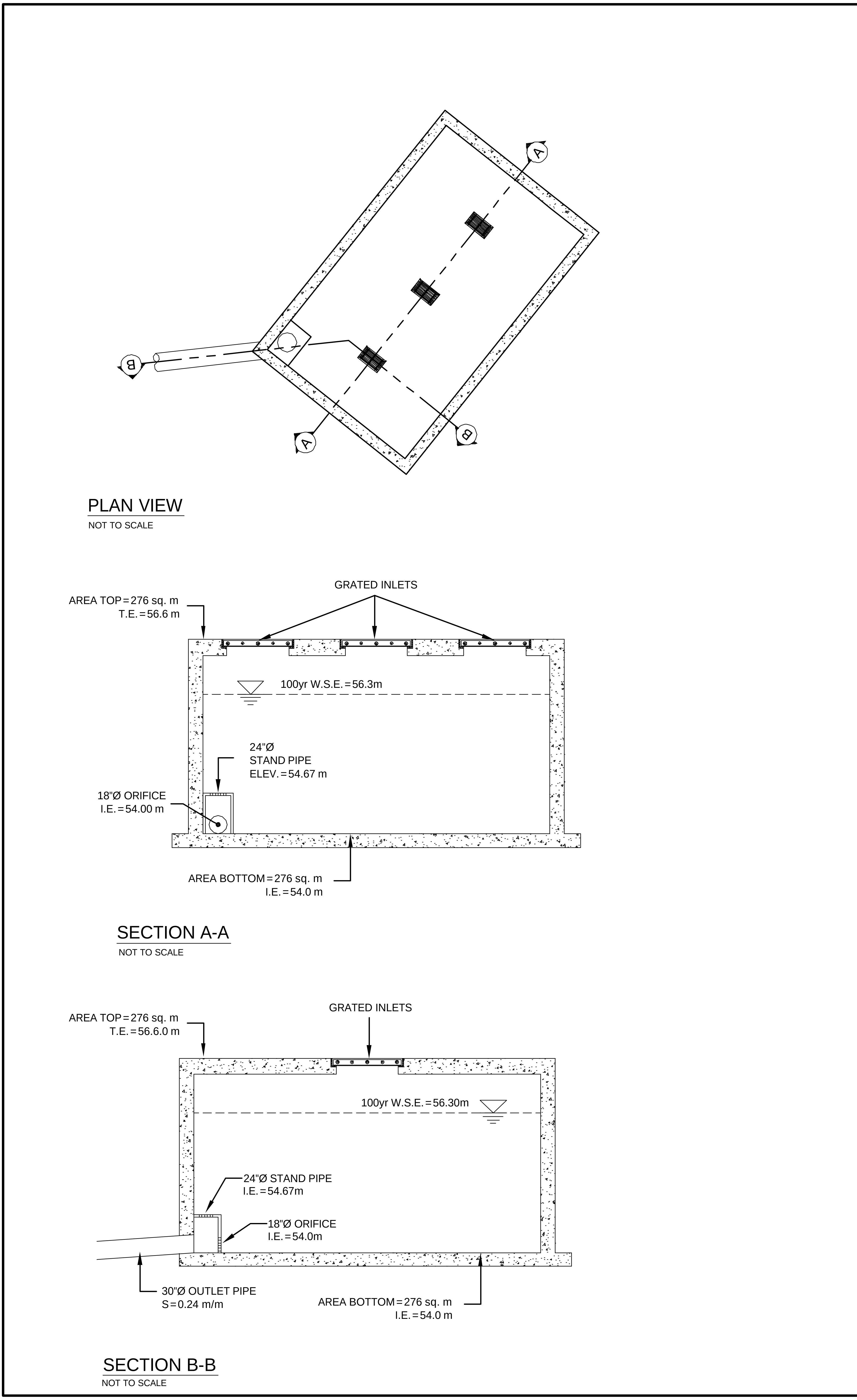


Figure 23: Schematic Design of Detention Pond 2D

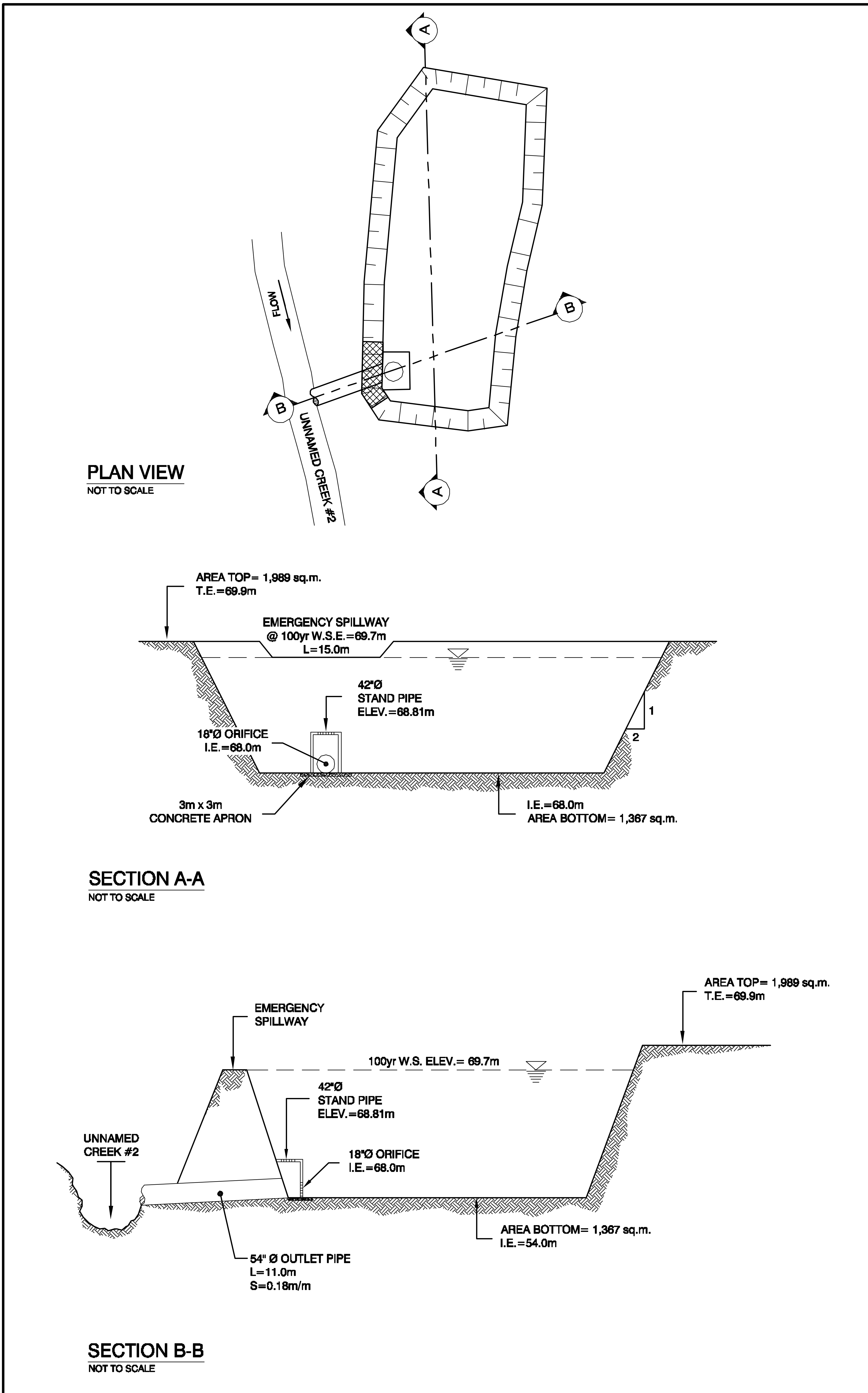


Figure 24: Schematic Design of Detention Pond 3A

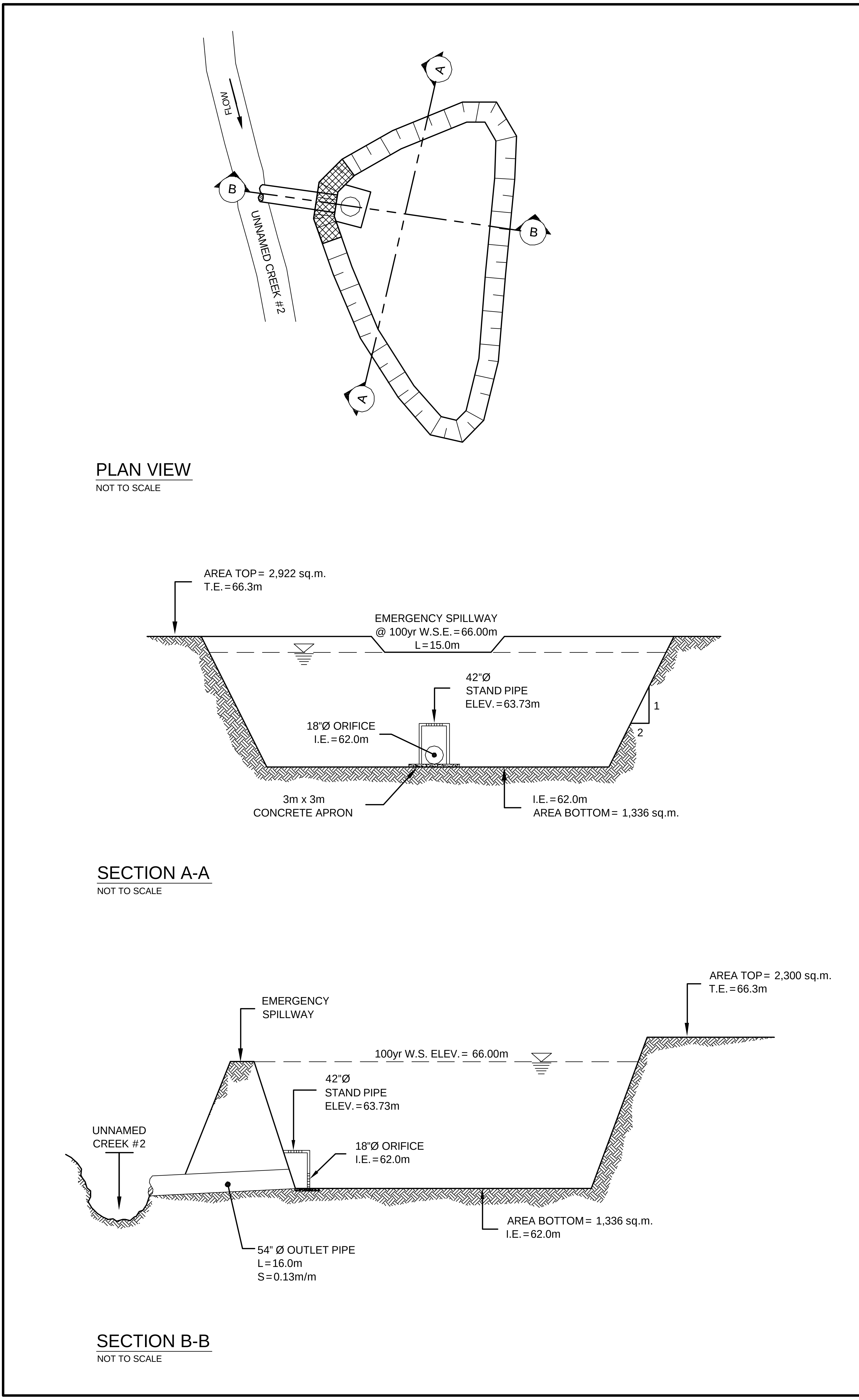


Figure 25: Schematic Design of Detention Pond 3B

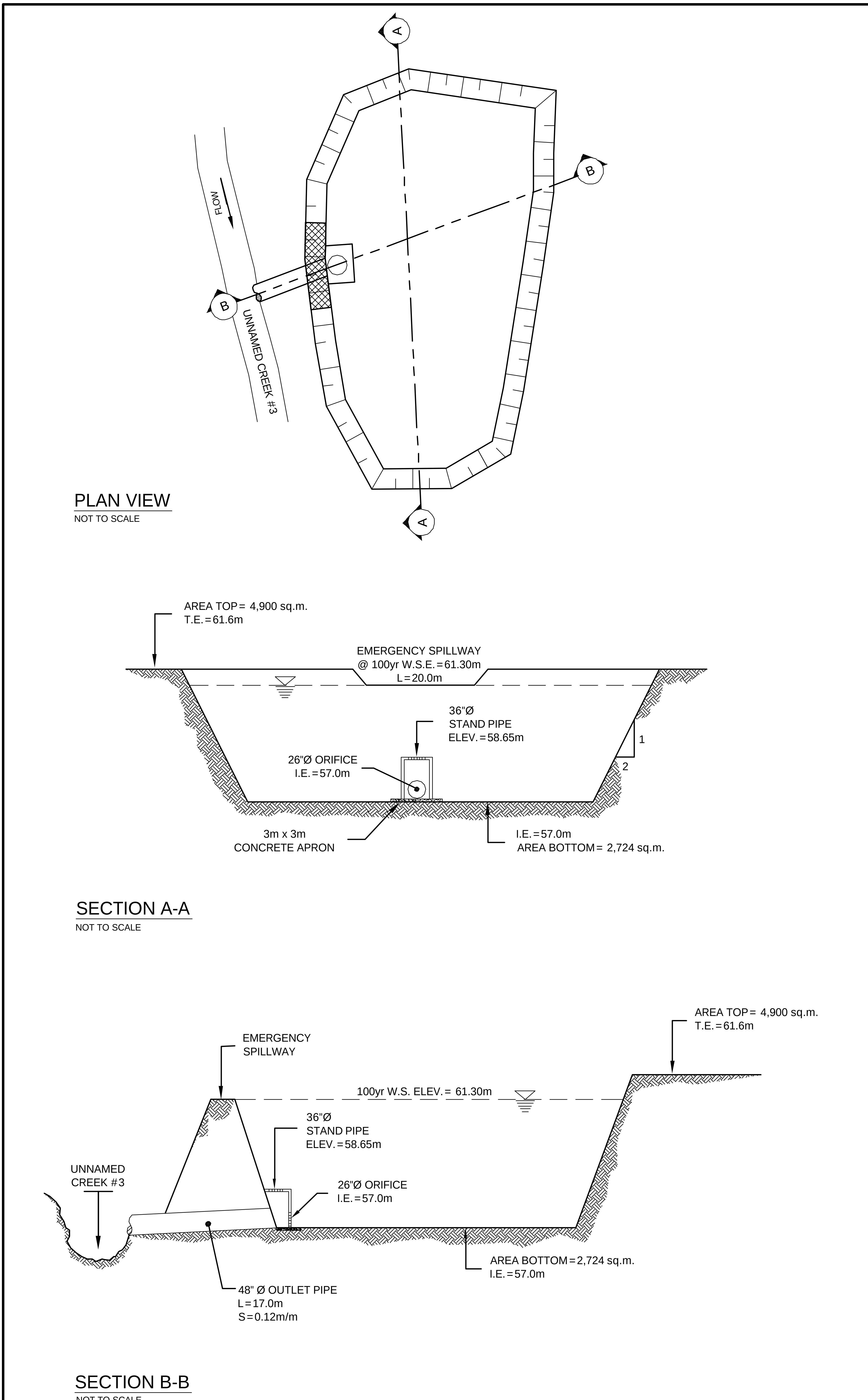
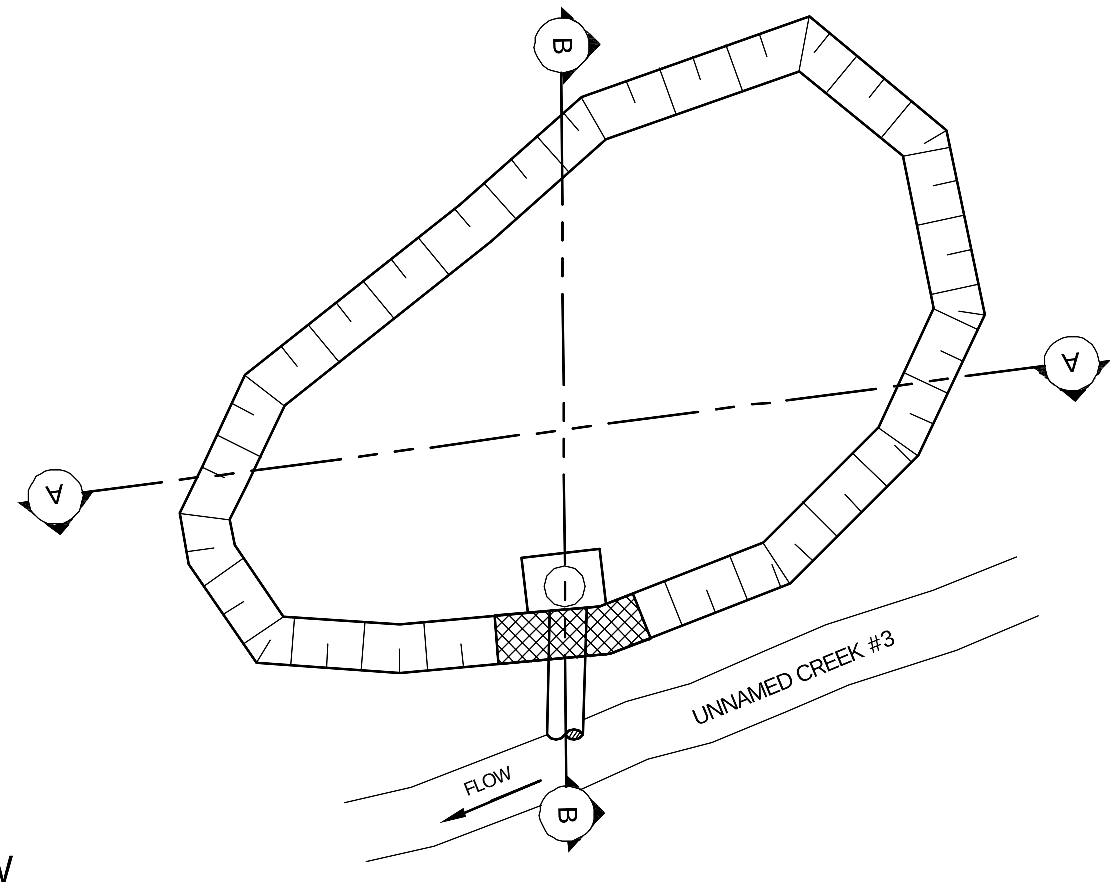
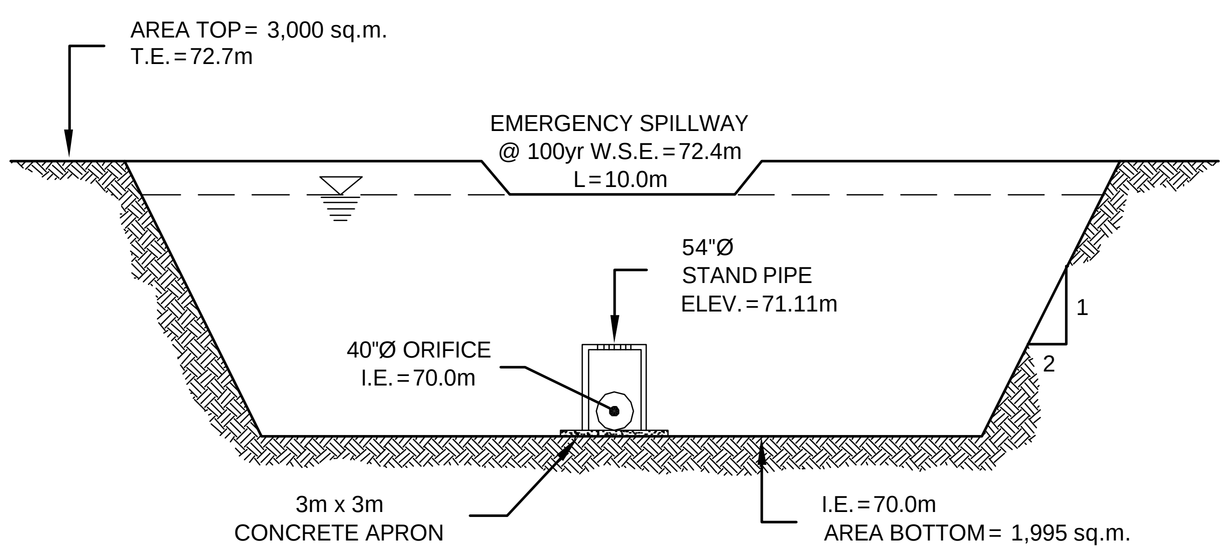


Figure 26: Schematic Design of Detention Pond 4



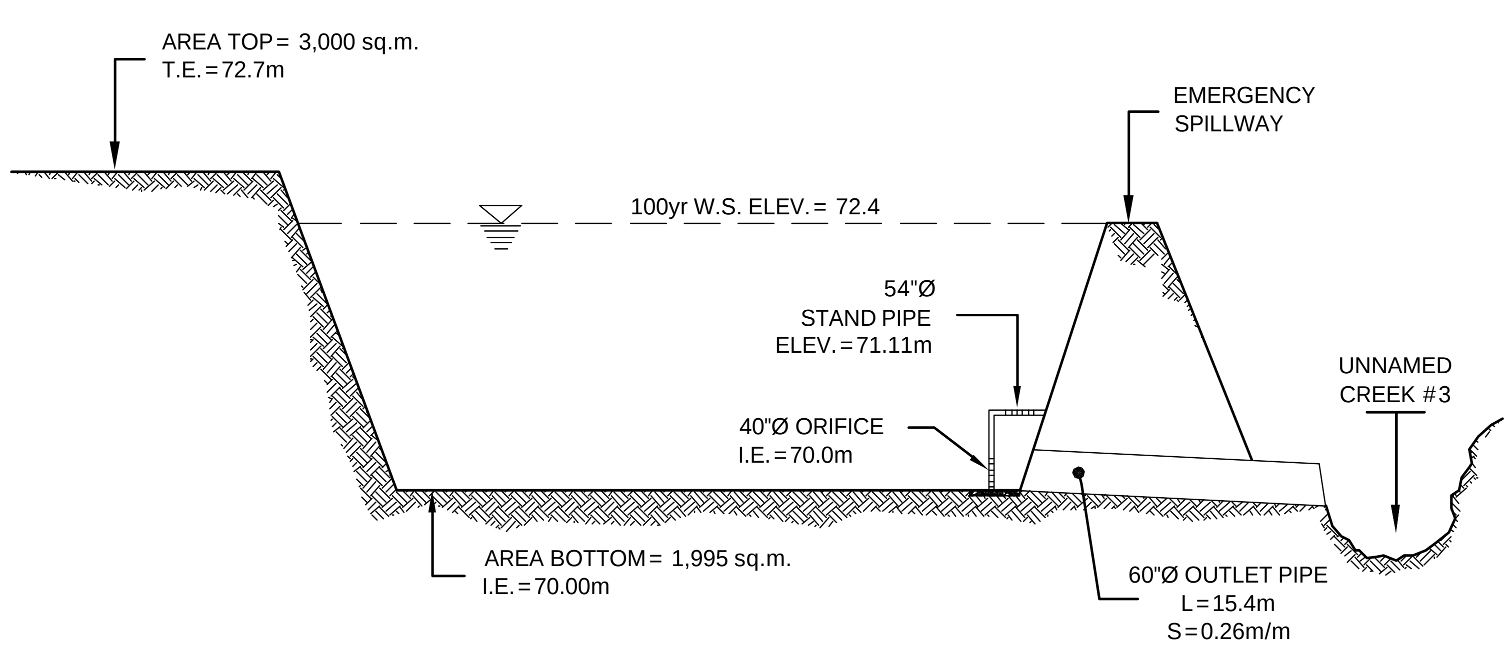
PLAN VIEW

NOT TO SCALE



SECTION A-A

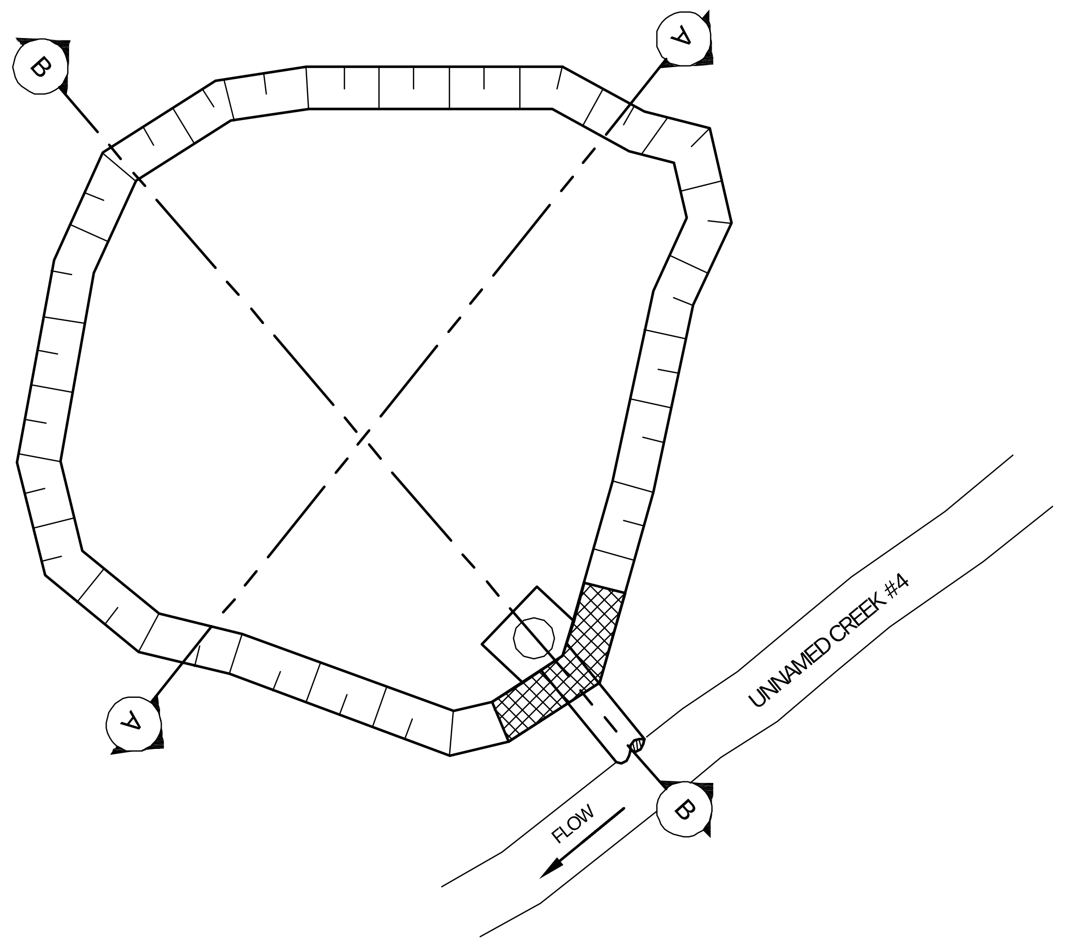
NOT TO SCALE



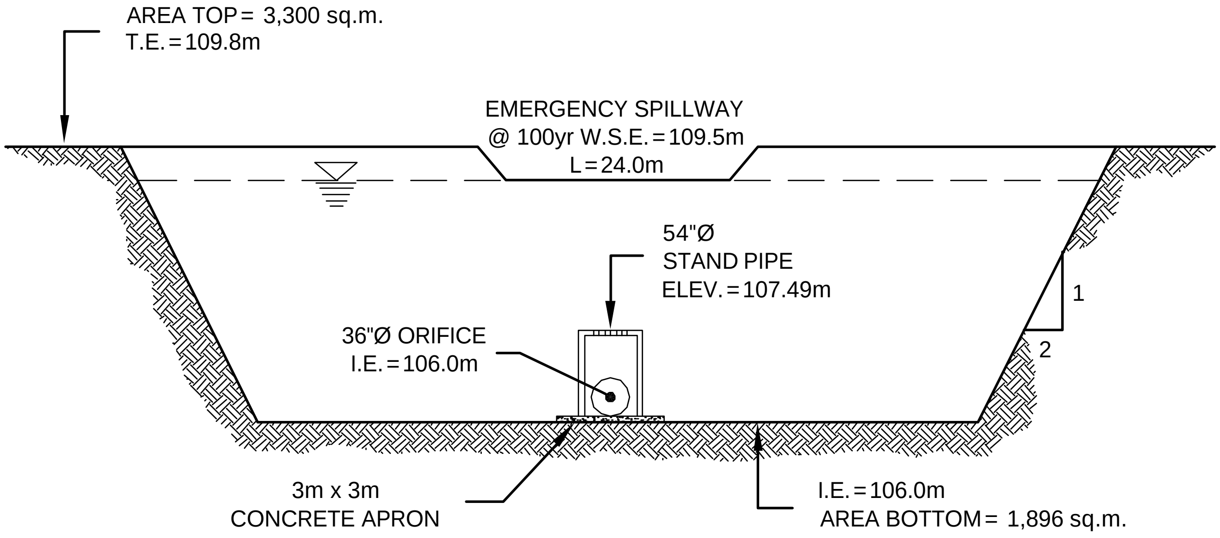
SECTION B-B

NOT TO SCALE

Figure 27: Schematic Design of Detention Pond 5



PLAN VIEW
NOT TO SCALE



SECTION A-A
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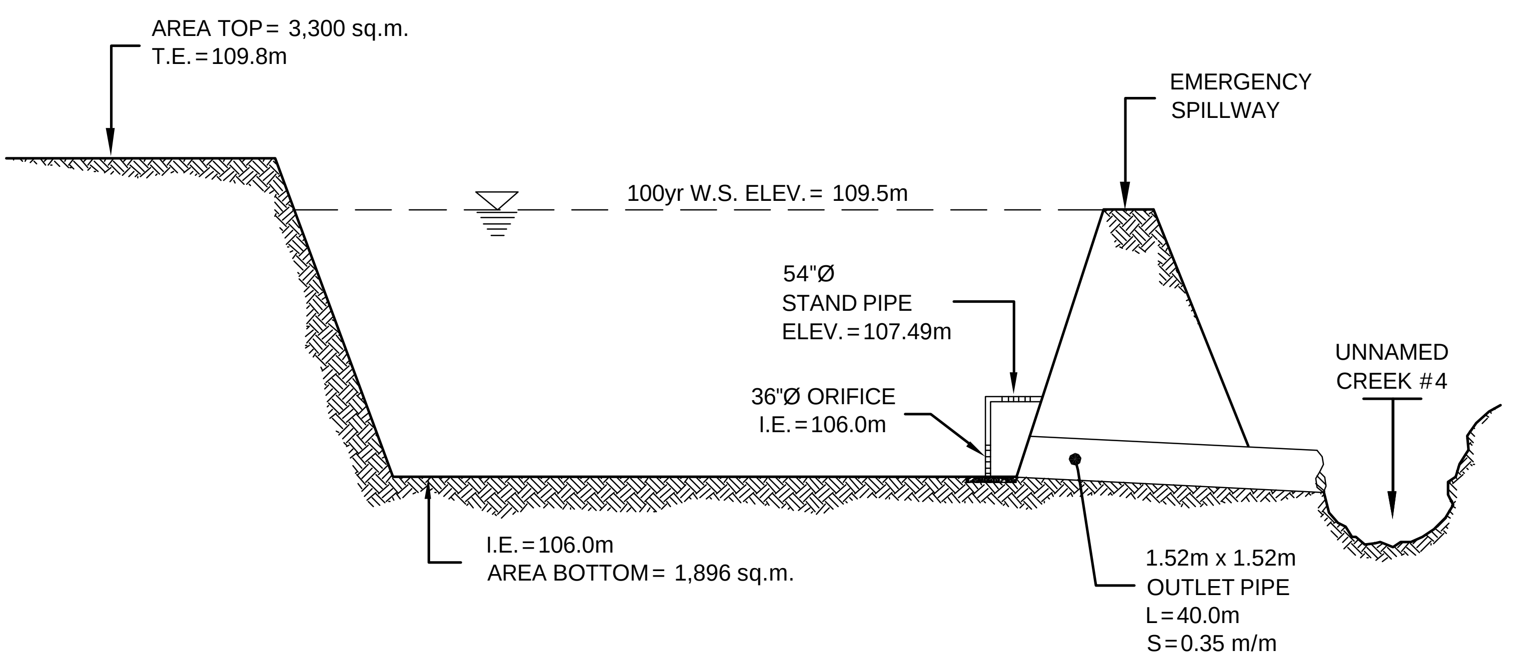

SECTION B-B
NOT TO SCALE

Figure 28: Schematic Design of Detention Pond 6A

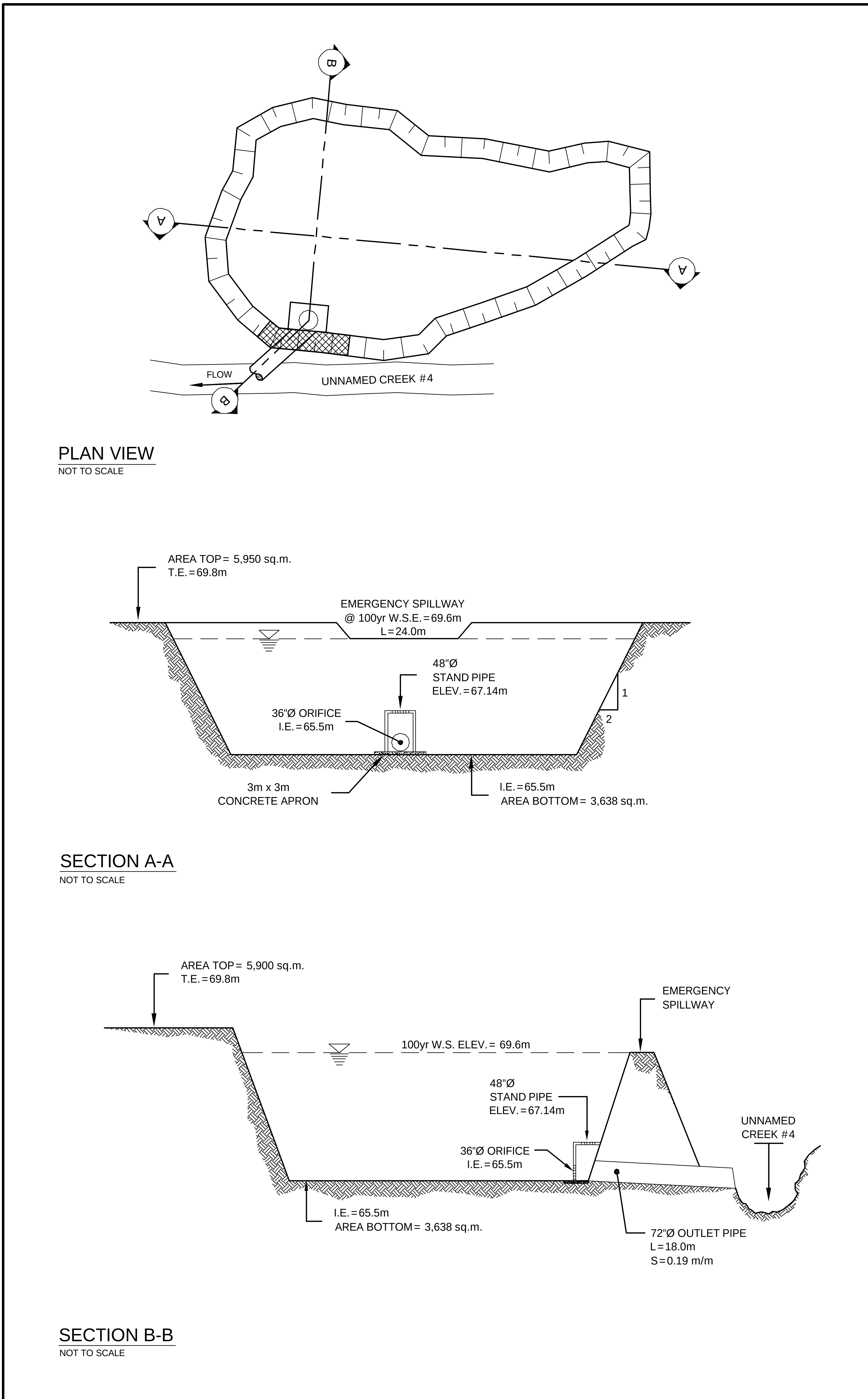

Figure 29: Schematic Design of Detention Pond 6B

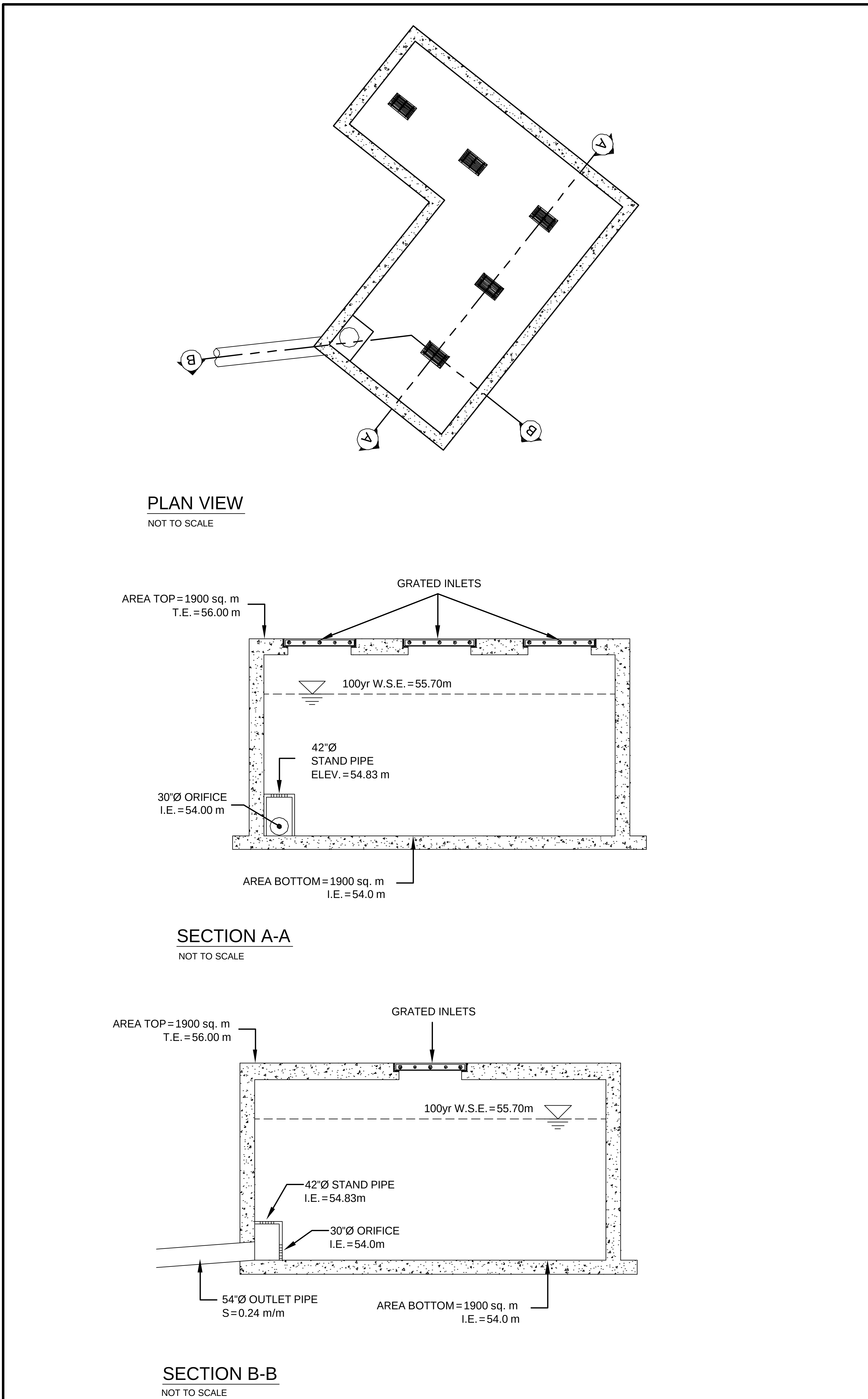

Figure 30: Schematic Design of Detention Structure 2D2

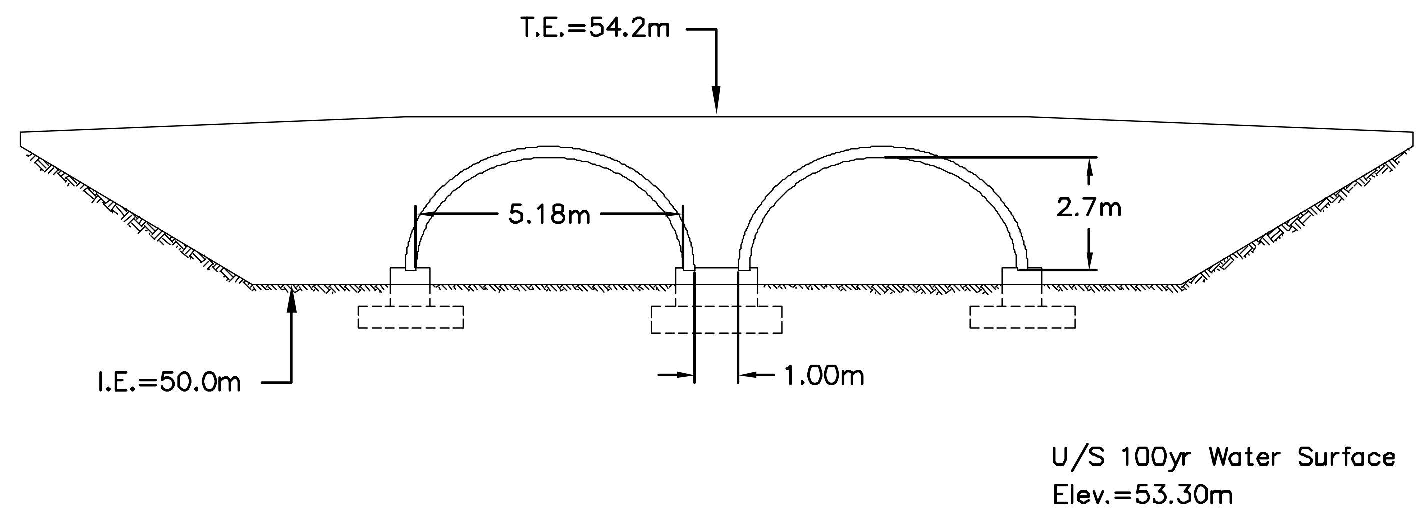

Figure 31: Schematic Design of Bridge Crossing #1

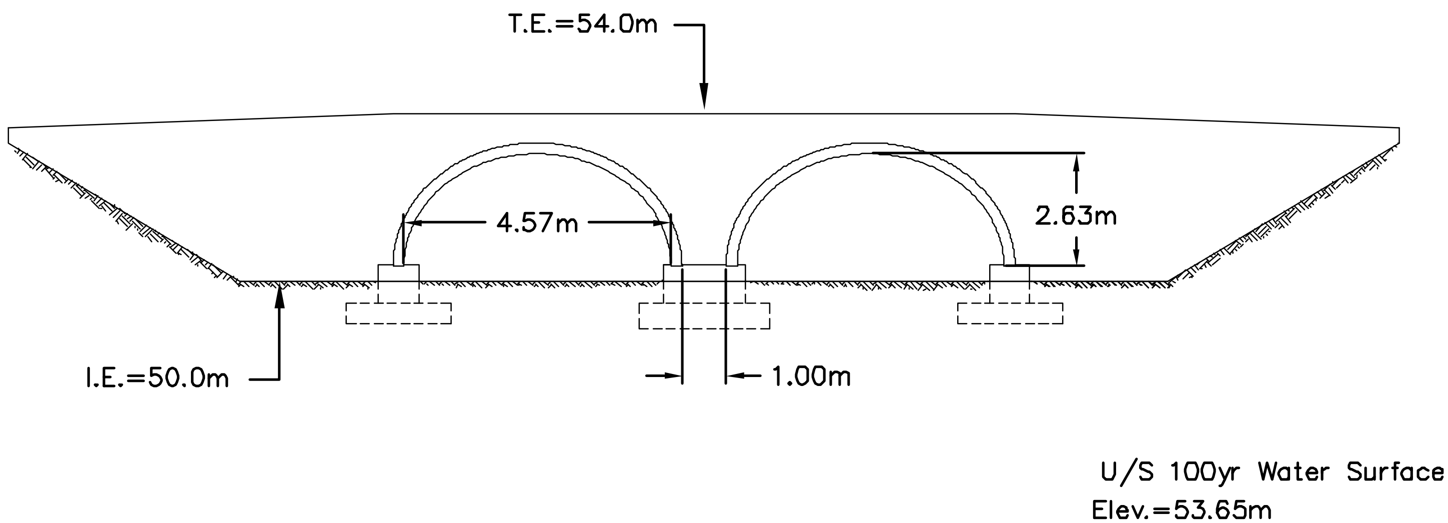

Figura 32: Schematic Design of Bridge Crossing #2

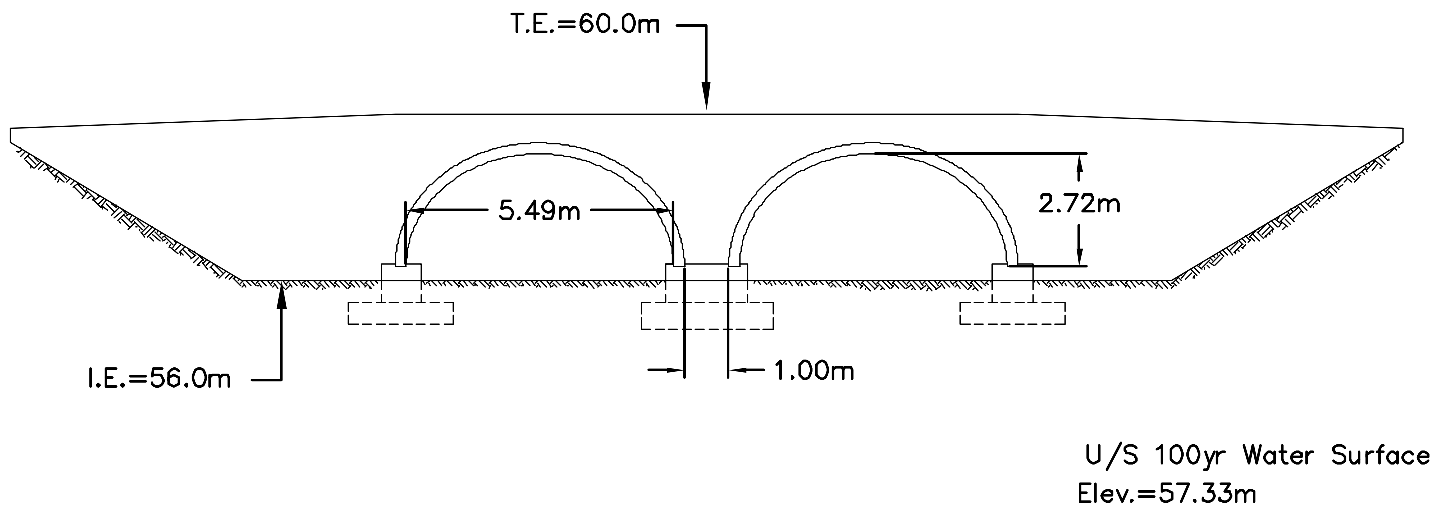

Figura 33: Schematic Design of Bridge Crossing #3

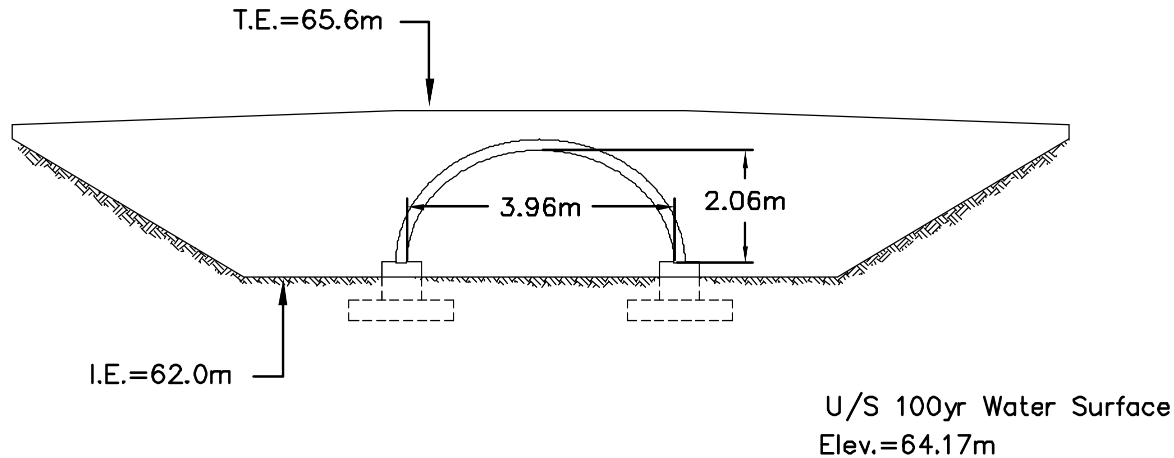

Figura 34: Schematic Design of Bridge Crossing #4

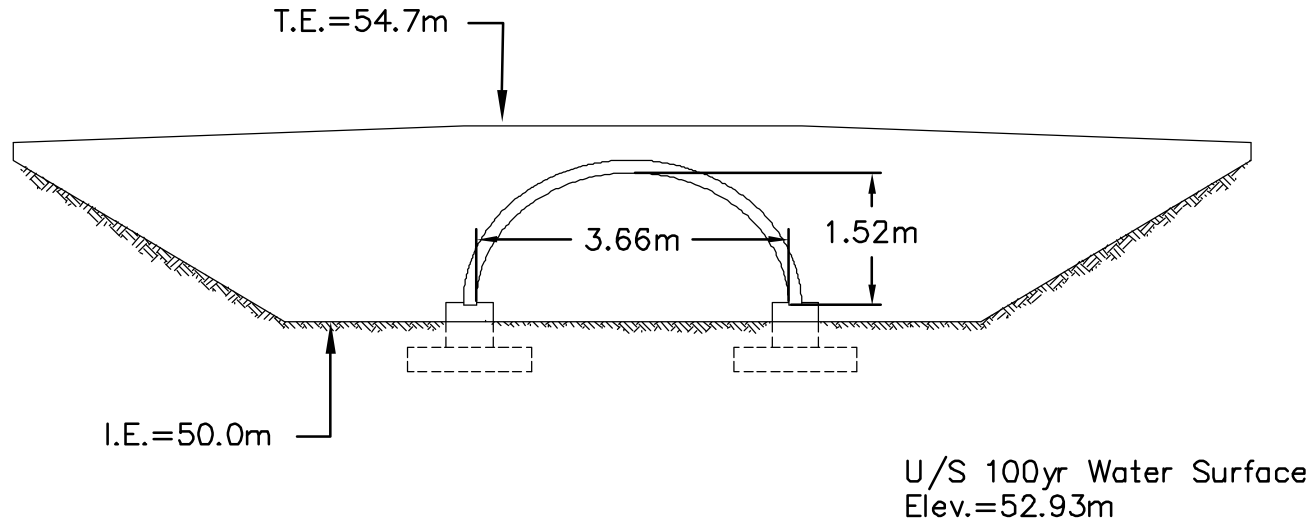

Figura 35: Schematic Design of Bridge Crossing #5

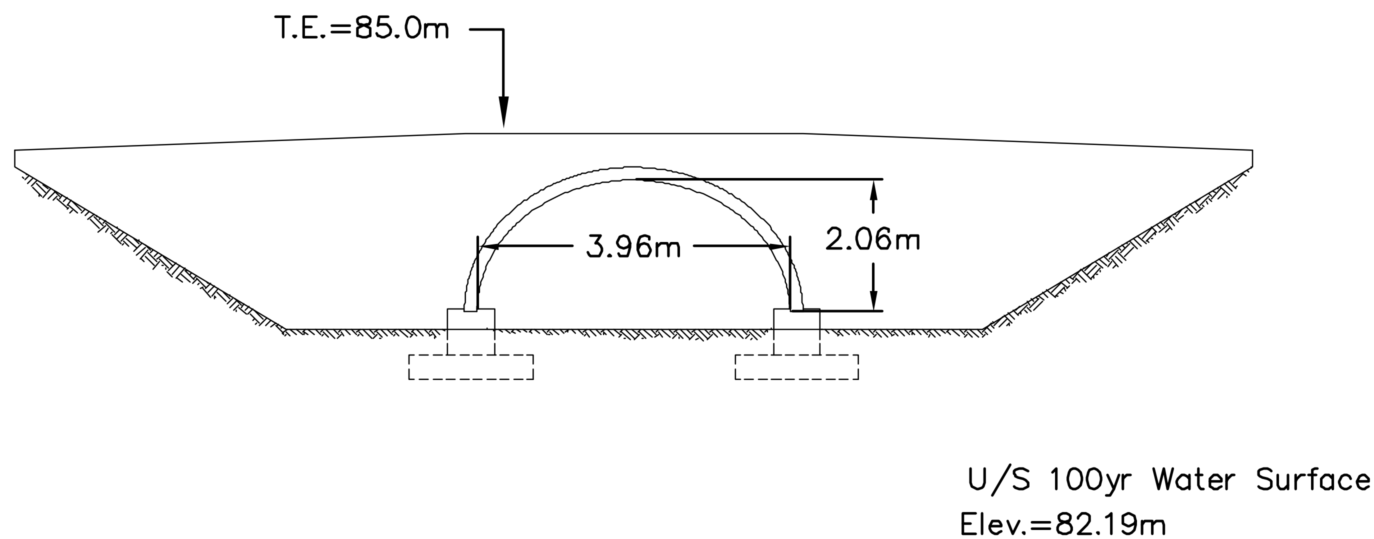

Figura 36: Schematic Design of Bridge Crossing #6

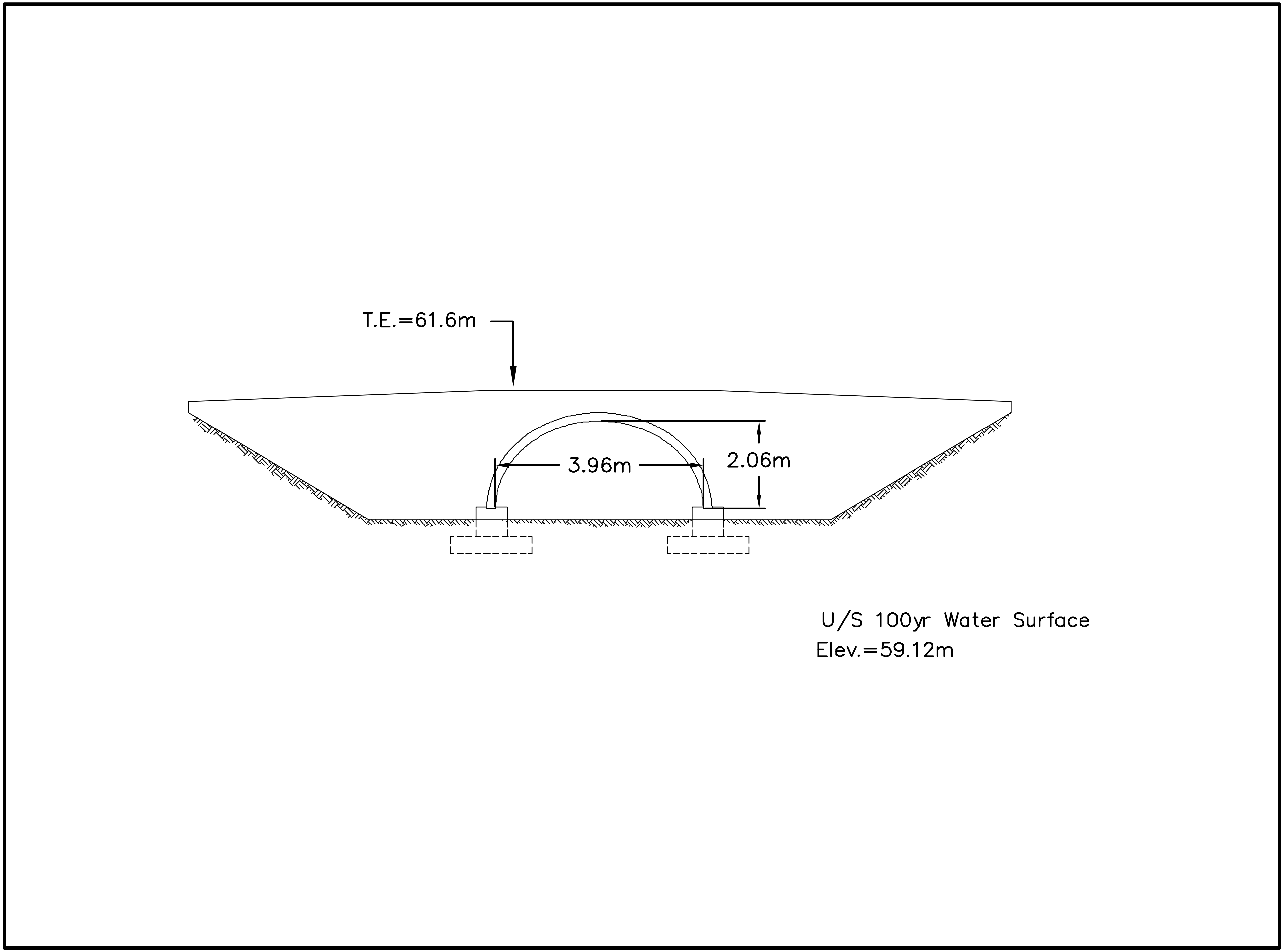

Figura 37: Schematic Design of Bridge Crossing #7

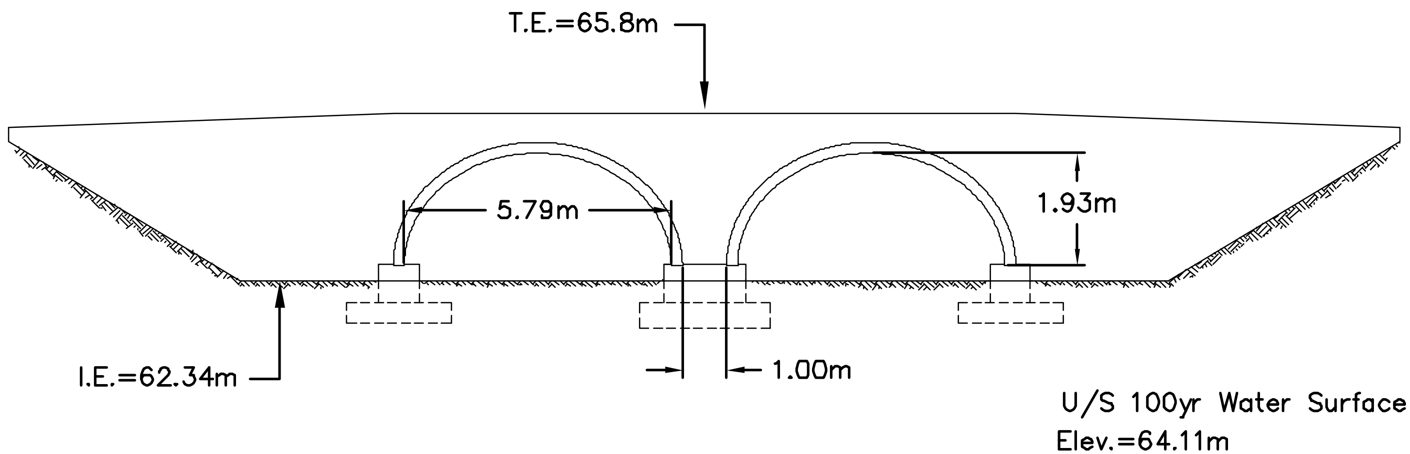

Figura 38: Schematic Design of Bridge Crossing #8

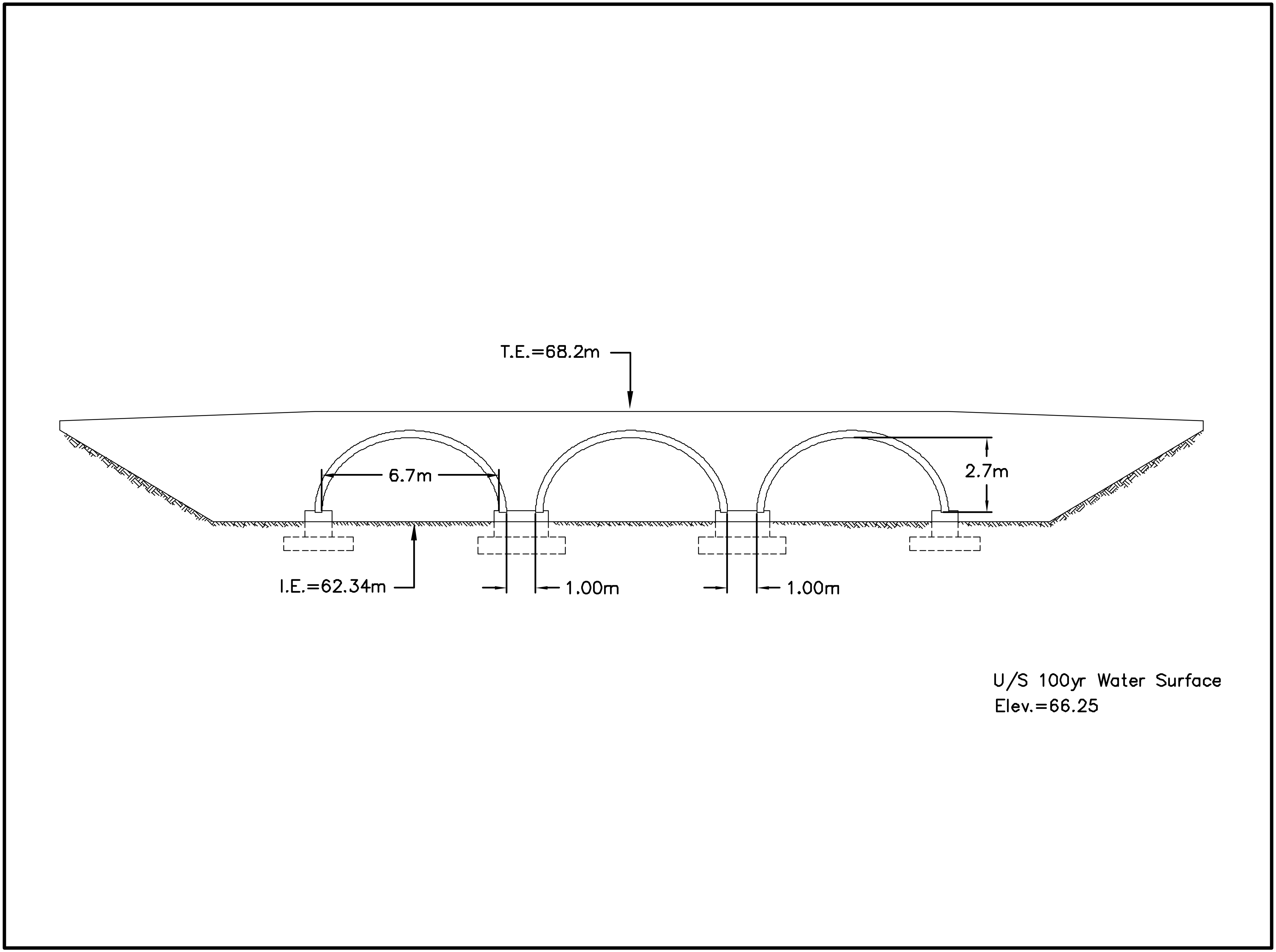

Figura 39: Schematic Design of Bridge Crossing #9

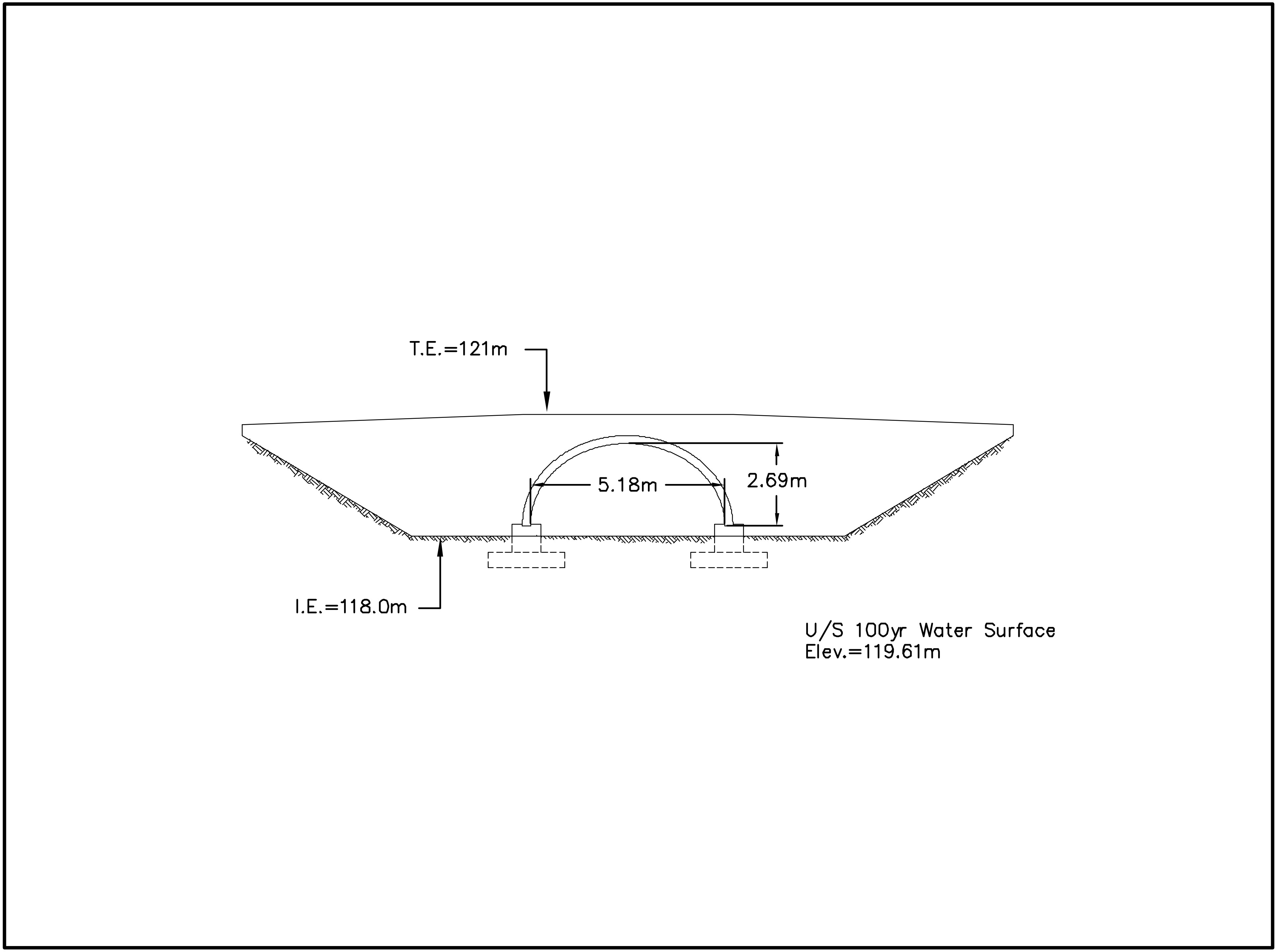

Figura 40: Schematic Design of Bridge Crossing #10

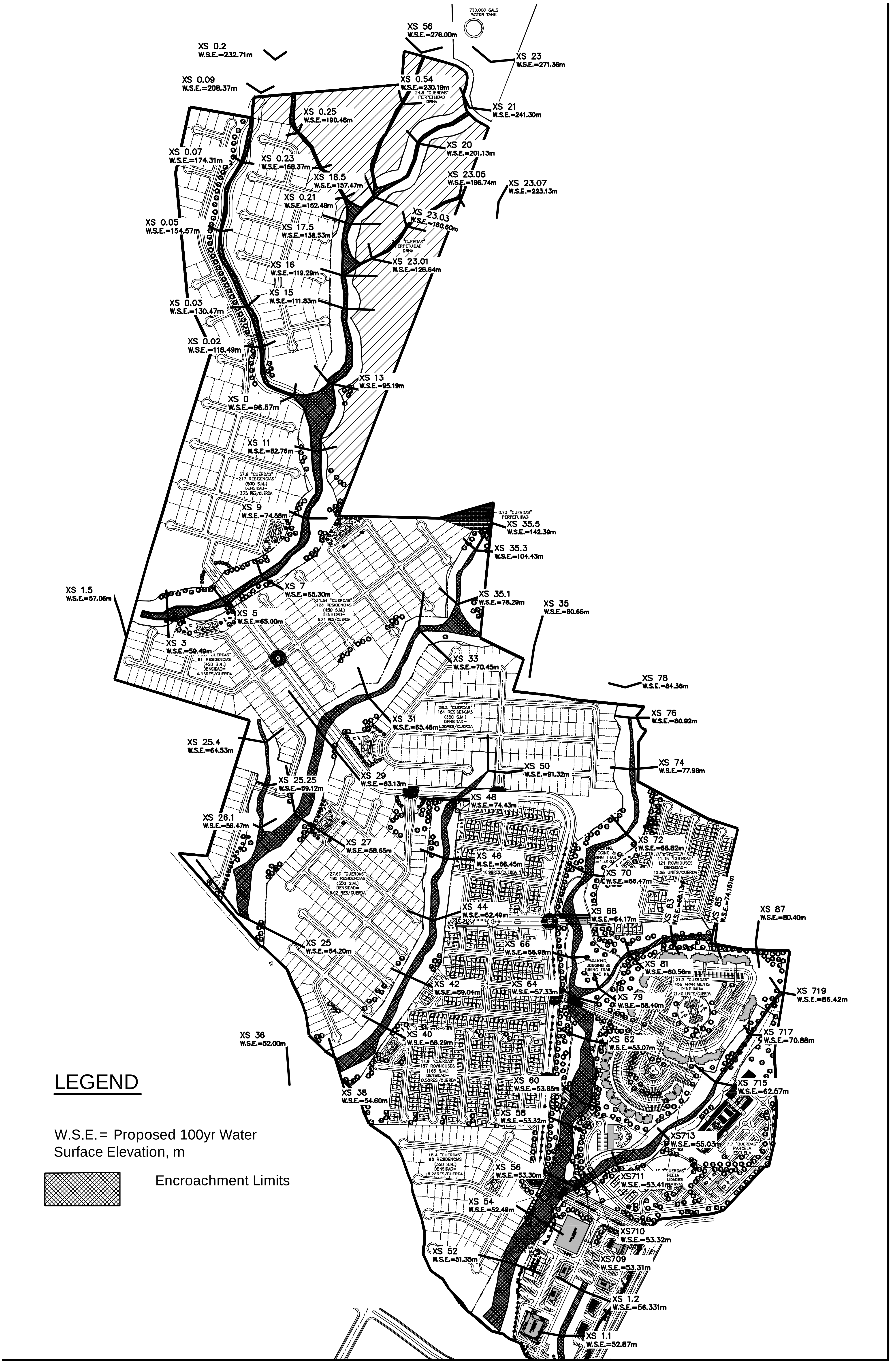

Figure 41:Encroachment Limits For Unnamed Creeks

Not To Scale

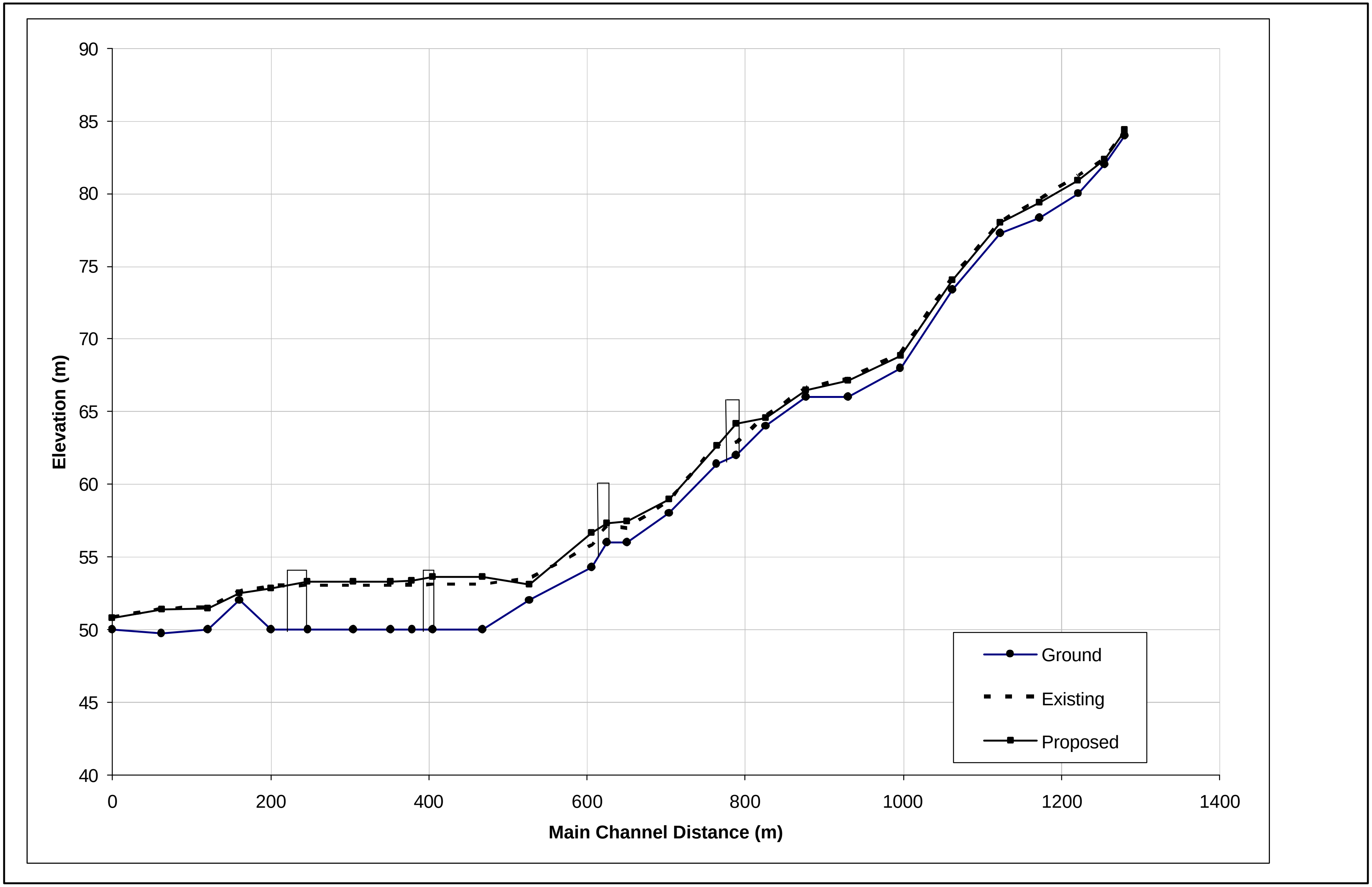

Figure 42: Existing and Encroached 100-yr Water Surface Profile for Unn. Creek #1

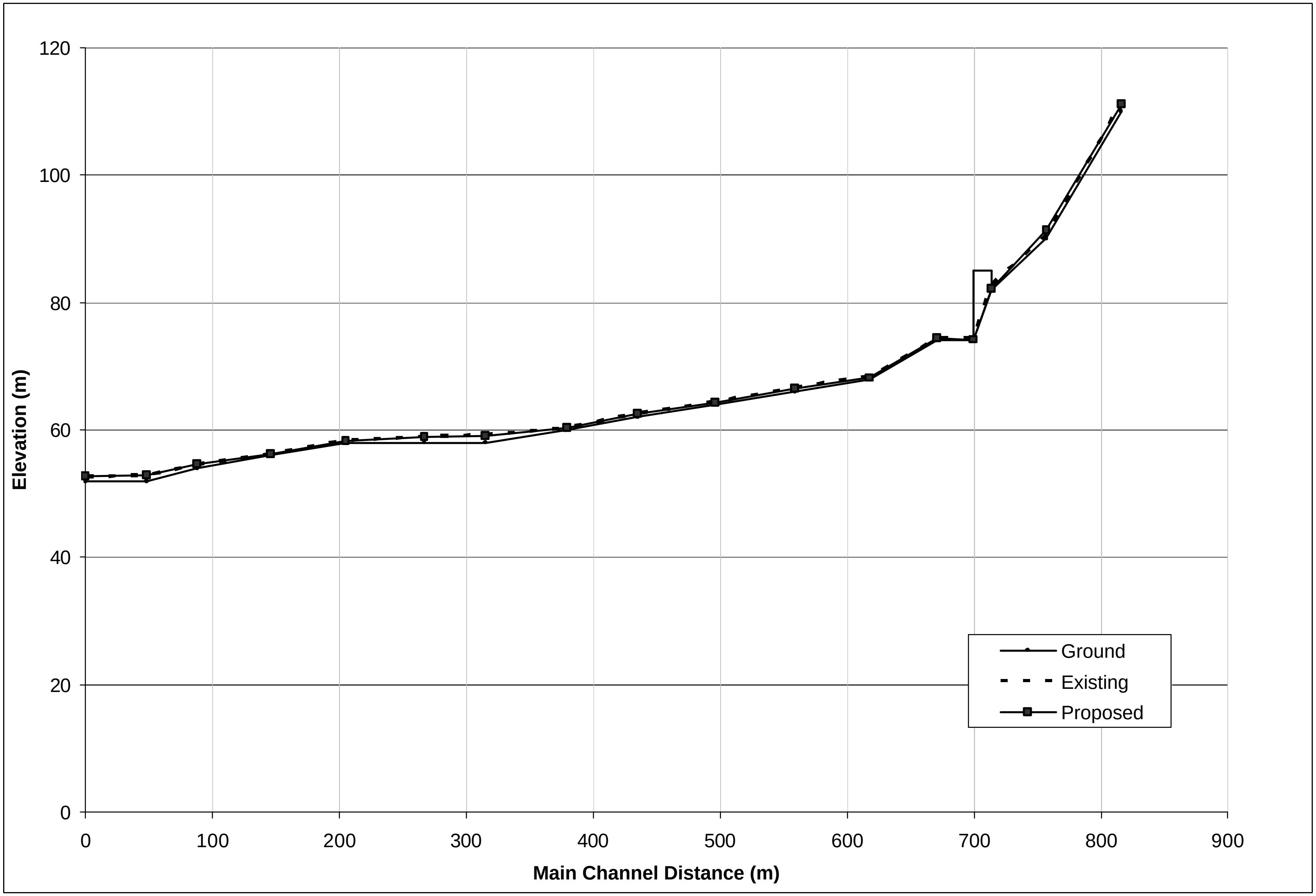

Figure 43: Existing and Encroached 100-yr Water Surface Profile for Unn. Creek #2

Figure 44: Existing and Encroached 100-yr Water Surface Profile for Unn. Creek #3

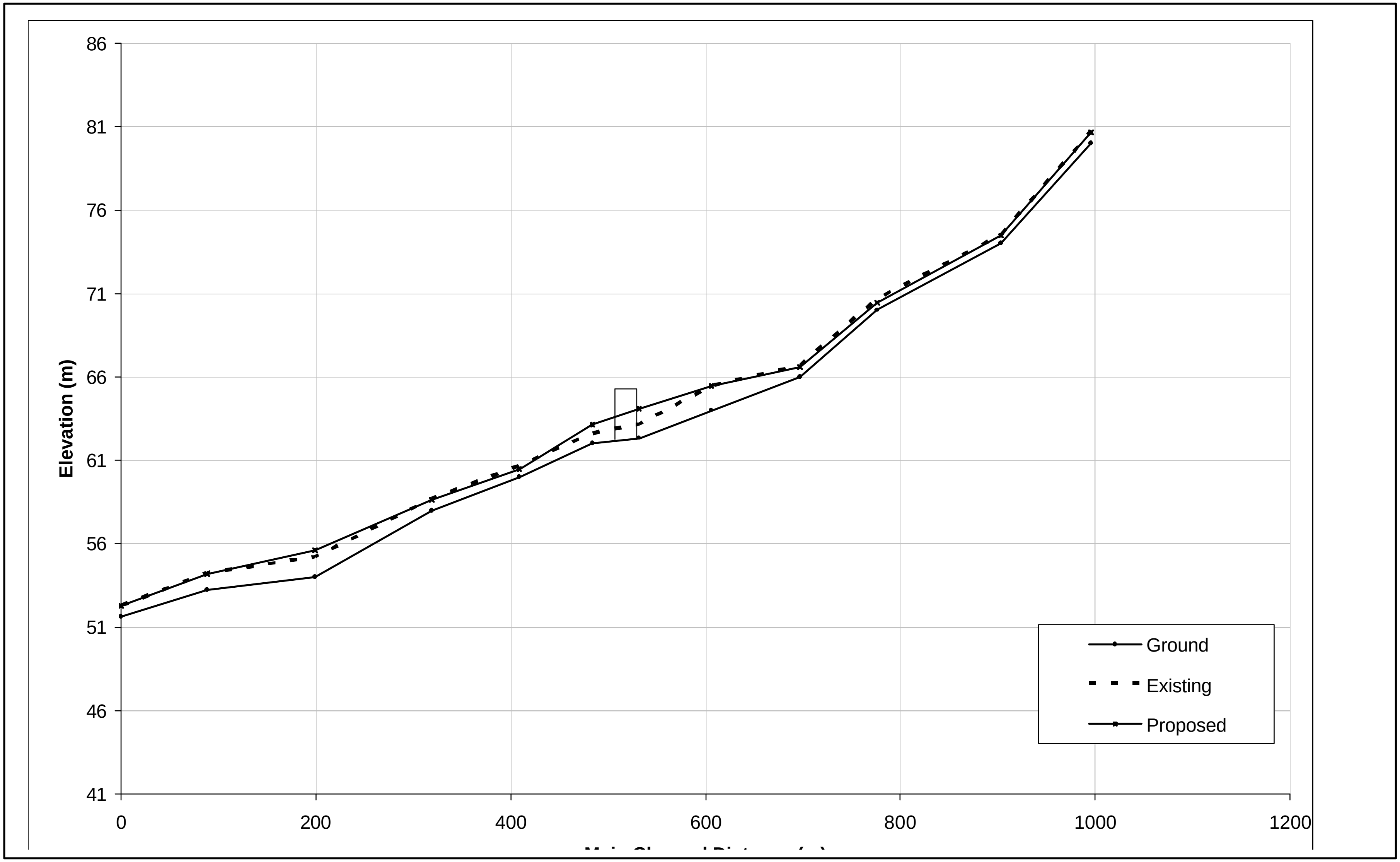

Figure 45: Existing and Encroached 100-yr Water Surface Profile for Unn. Creek #4