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Client SJRA
Project SCW Flood Control Dams **Unit**
Project No. 411500 **File No.**
Title Evaluation of Project Soil Parameters

Computed By P. Turkson
Date 10/25/2024
Approved By David Bentler
Date 12/6/2024
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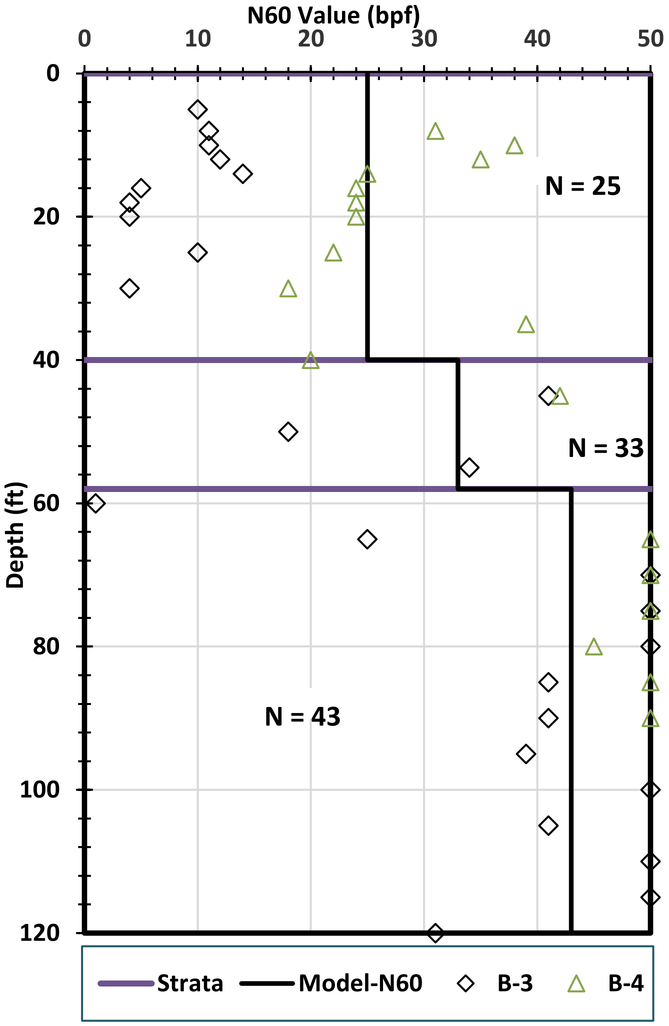

Figure 2. SPT N60 Values for Birch Creek Dam

4.4 Index Properties

4.4.1 Particle Size Distribution

A summary of the size distribution of soil particles statistical analysis and design values for each stratum is included as **Table 9** and **Table 10**Error! Reference source not found.**,** where the basis of the design values are highlighted in bold. Statistical analyses and selection of design values were conducted in accordance with the methods described in **Section 3.3**. For each stratum, the average of the laboratory testing is the preferred basis of design. Plots of the percent finer than sieve number 200 data and design values are included as **Figure 3** and **Figure 4**Error! Reference source not found..

Table 9. Percent Finer than No. 200 Statistical Analysis, Data Comparison, and Design Values— Walnut

Stratum	Laboratory Data					Design Value (%)
	Bottom Depth (feet)	Sample Size (n)	Avg. (%)	Min. (%)	Max (%)	
Silty Sand and Clayey Sand	37	5	19	11	25	**19**
Silty Clay and Sandy Clay	69	5	36	6	67	**36**
Silty Sand and Clayey Sand	82	3	41	17	69	**41**
Silty Clay and Sandy Clay	97	2	36	8	63	**36**
Silty Sand and Clayey Sand	120	2	13	7	19	**13**

Table 10. Percent Finer than No. 200 Statistical Analysis, Data Comparison, and Design Values— Birch

Stratum	Laboratory Data					Design Value (%)
	Bottom Depth (feet)	Sample Size (n)	Avg. (%)	Min. (%)	Max (%)	
Silty Sand and Clayey Sand	40	4	32	21	45	**32**
Silty Clay and Sandy Clay	58	2	46	9	83	**46**
Silty Sand and Clayey Sand	120	8	22	5	64	**22**

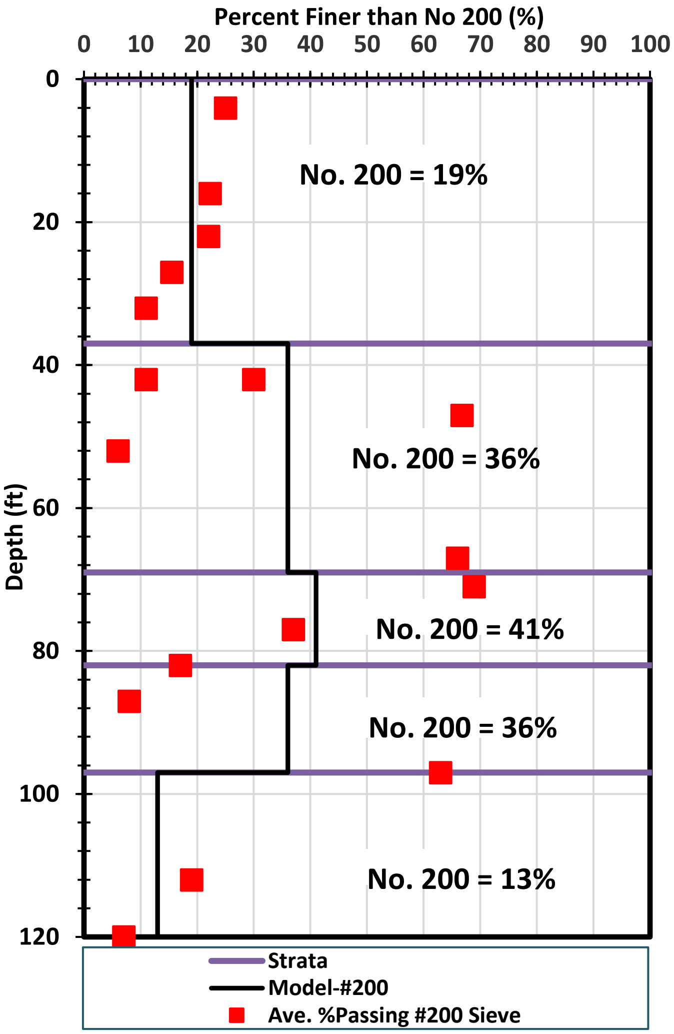

Figure 3. Percent Finer than No. 200 for Walnut Creek Dam

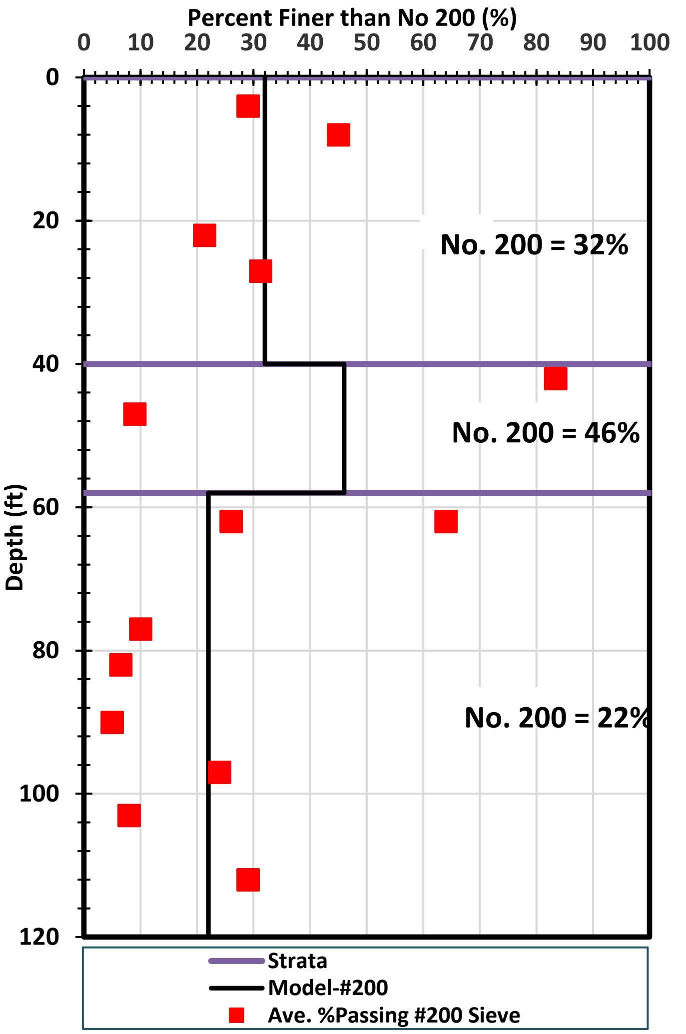

Figure 4. Percent Finer than No. 200 for Birch Creek Dam

4.4.2 Total Unit Weight

A summary of the total unit weight statistical analysis and design values for each stratum is included as **Table 11** and **Table 12,** where the basis of the design values is highlighted in bold. Statistical analyses and selection of design values were conducted in accordance with the methods described in **Section 3.1**.

The average SPT resistance of N_{60} ranges from 18 to 50 bpf for the design sandy layers and from 29 to 43 bpf for the design clay layers at Walnut Creek. At Birch Creek, the average N_{60} range from 25 to 43 bpf for the design sandy layers and is equal to 33 bpf for the clay layers.

Based on the overall average SPT N_{60} range of values and **Reference 18**, the sandy soils are classified as medium dense consistency with total unit weight ranging from 110 to 140 pcf (17 to 22 kN/m^3). Considering the higher confining stresses with soil depth which results in generally denser soils with depth, unit weight for deeper soil units is greater than shallow units for sand layers.

Based on the overall average SPT N_{60} range of values and **Reference 18**, the clayey soils are classified as very stiff to hard consistency with total unit weight ranging from 120 to 140 pcf (18 to 22 kN/m^3). Considering the relatively wide range of soils consistencies, conservatively lower values for total unit weight have been for the clay layers.

Plots of the total unit weight data and design values are included as **Figure 5** and **Figure 6**.

Table 11. Total Unit Weight Statistical Analysis, Data Comparison, and Design Values— Walnut

Stratum	Laboratory Data					Design Value (pcf)
	Bottom Depth (feet)	Sample Size (n)	Avg. (pcf)	Min. (pcf)	Max (pcf)	
Silty Sand and Clayey Sand	37	2	128	127	130	**125**
Silty Clay and Sandy Clay	69	3	127	119	137	**125**
Silty Sand and Clayey Sand	82	1	135	135	135	**130**
Silty Clay and Sandy Clay	97	—	—	—	—	**125**
Silty Sand and Clayey Sand	120	—	–	–	—	**130**

Table 12. Total Unit Weight Statistical Analysis, Data Comparison, and Design Values— Birch

Stratum	Laboratory Data					Design Value (pcf)
	Bottom Depth (feet)	Sample Size (n)	Avg. (pcf)	Min. (pcf)	Max (pcf)	
Silty Sand and Clayey Sand	40	2	139	135	143	**125**
Silty Clay and Sandy Clay	58	1	123	123	123	**123**
Silty Sand and Clayey Sand	120	1	137	137	137	**130**

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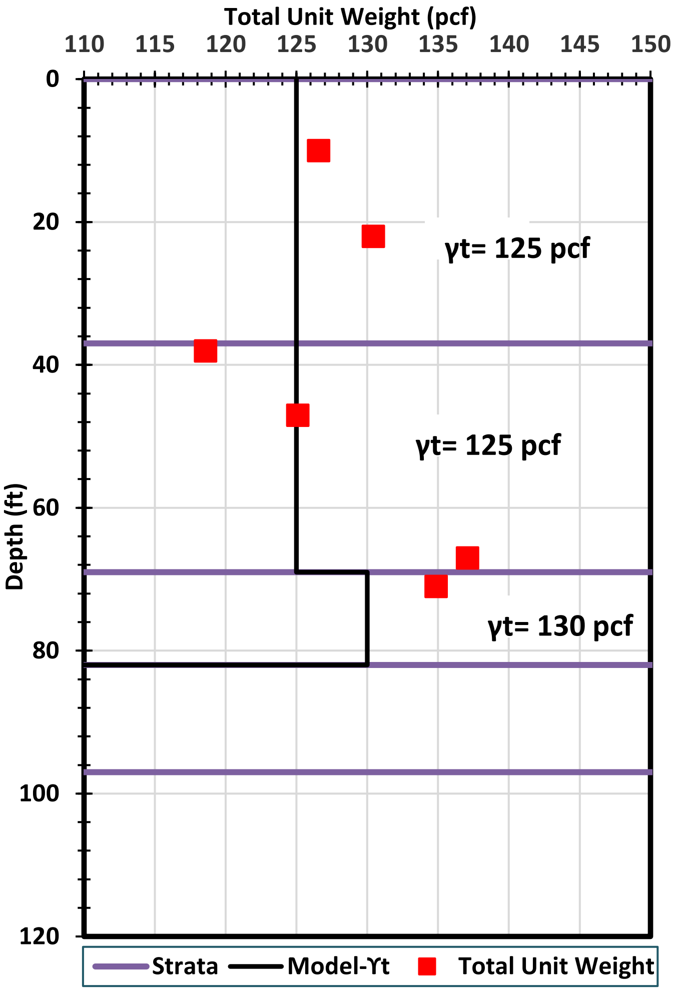


Figure 5. Total Unit Weight Design Profiles for Walnut Creek Dam

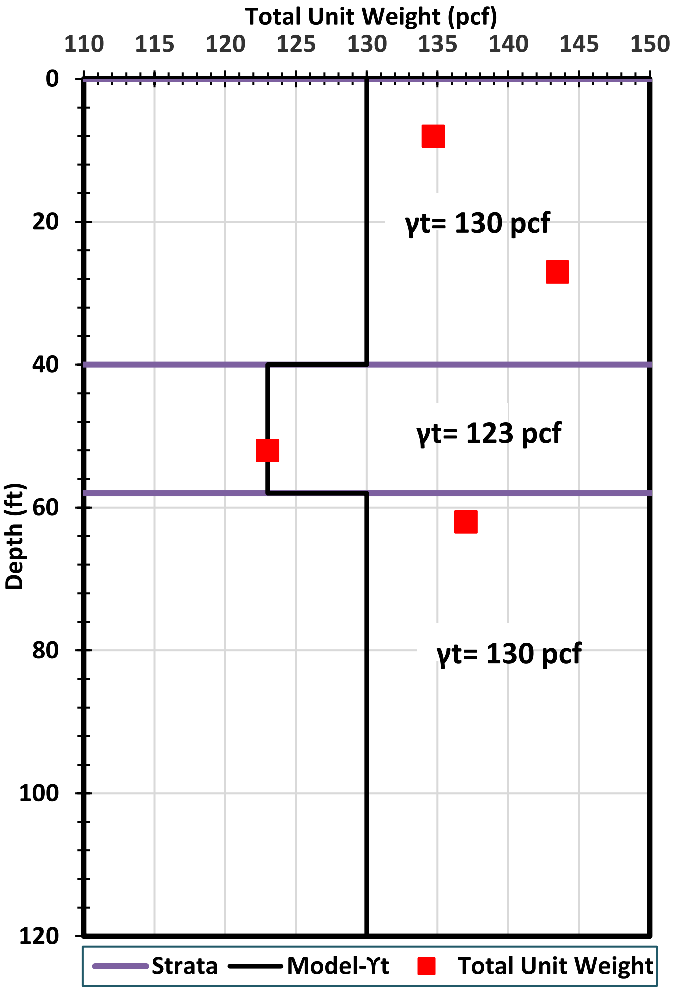


Figure 6. **Total Unit Weight Design Profiles for Birch Creek Dam**

4.4.3 Moisture Content

A summary of the moisture content (MC) statistical analysis and design values for each stratum is included as **Table 13** and **Table 14**, where the basis of the design values are highlighted in bold. Statistical analyses and selection of design values were conducted in accordance with the methods described in **Section 3.4**. The basis of all design values is the average of the laboratory test results. Plots of the moisture content and design values are included as **Figure 7** and **Figure 8**.

Table 13 Moisture Content Statistical Analysis, Data Comparison, and Design Values— Walnut

Stratum	Laboratory Data				Design Value - MC(%)
	Sample Size (n)	Avg. (%)	Min. (%)	Max. (%)	
Silty Sand and Clayey Sand	8	13	6	20	**13**
Silty Clay and Sandy Clay	7	24	16	32	**24**
Silty Sand and Clayey Sand	3	22	20	26	**22**
Silty Clay and Sandy Clay	3	20	20	21	**20**
Silty Sand and Clayey Sand	2	21	20	22	**21**

Table 14 Moisture Content Statistical Analysis, Data Comparison, and Design Values— Birch

Stratum	Laboratory Data				Design Value - MC(%)
	Sample Size (n)	Avg. (%)	Min. (%)	Max. (%)	
Silty Sand and Clayey Sand	7	15	9	23	**15**
Silty Clay and Sandy Clay	4	25	23	27	**25**
Silty Sand and Clayey Sand	10	20	17	23	**20**

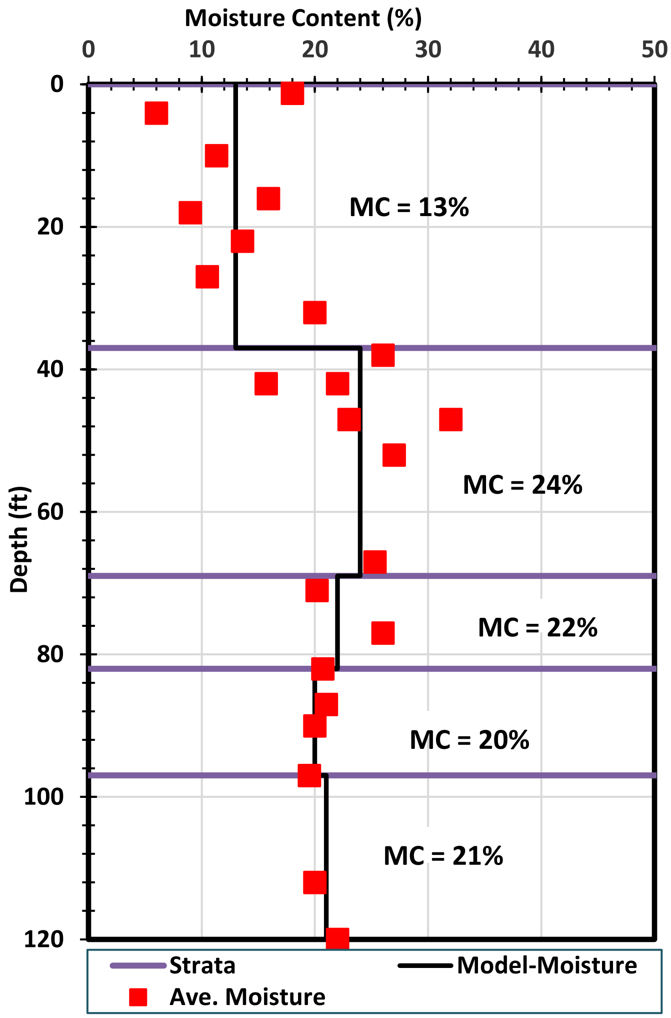


Figure 7. Moisture Content Profiles for Walnut

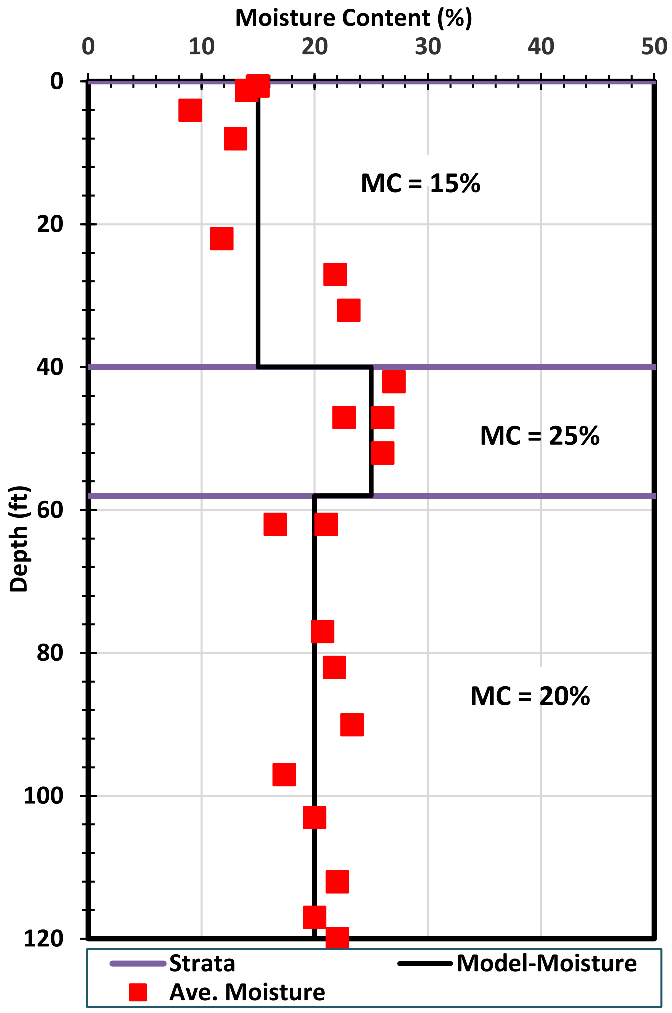


Figure 8. Moisture Content Profiles for Birch

4.4.4 Liquid Limit

A summary of the liquid limit (LL) statistical analysis and design values for each stratum is included as **Table 15** and **Table 16**, where the basis of the design values is highlighted in bold. Statistical analyses and selection of design values were conducted in accordance with the methods described in **Section 3.4**. The basis of all design values is the average of the laboratory test results. Plots of the liquid limit data and design values are included as **Figure 9** and **Figure 10**.

Table 15 Liquid Limit Statistical Analysis and Design Values— Walnut

Stratum	Laboratory Data				Design Value - LL(%)
	Sample Size (n)	Avg. (%)	Min. (%)	Max. (%)	
Silty Sand and Clayey Sand	2	32	26	37	**32**
Silty Clay and Sandy Clay	3	48	26	60	**48**
Silty Sand and Clayey Sand	—	—	—	—	**—**
Silty Clay and Sandy Clay	1	32	32	32	**32**
Silty Sand and Clayey Sand	1	18	18	18	**18**

Table 16 Liquid Limit Statistical Analysis and Design Values— Birch

Stratum	Laboratory Data				Design Value - LL(%)
	Sample Size (n)	Avg. (%)	Min. (%)	Max. (%)	
Silty Sand and Clayey Sand	3	30	21	38	**30**
Silty Clay and Sandy Clay	2	65	64	66	**65**
Silty Sand and Clayey Sand	4	32	23	42	**32**

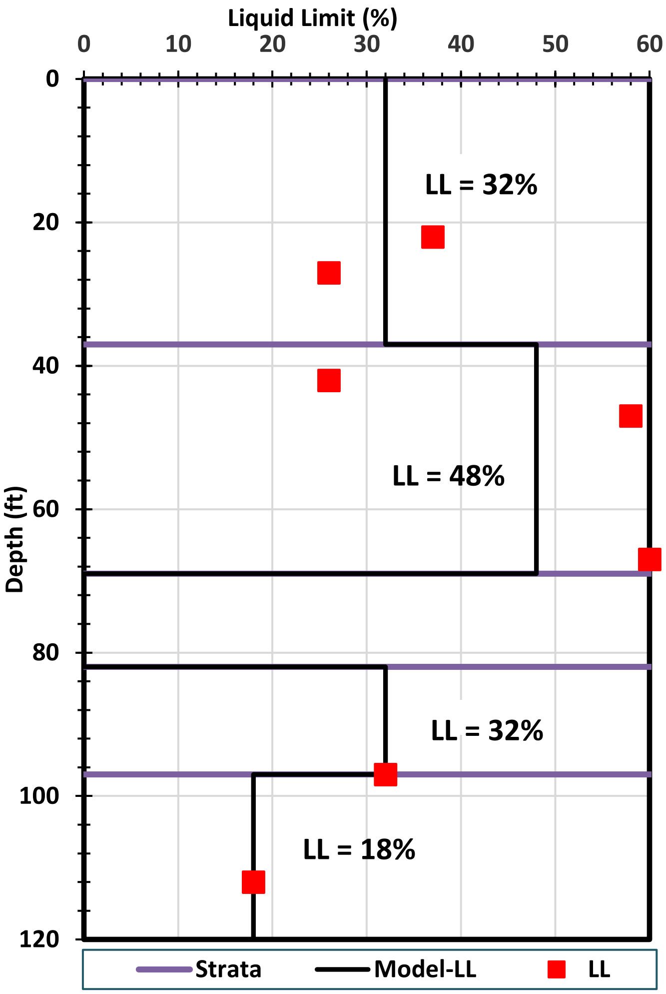


Figure 9. Liquid Limit Profiles for Walnut

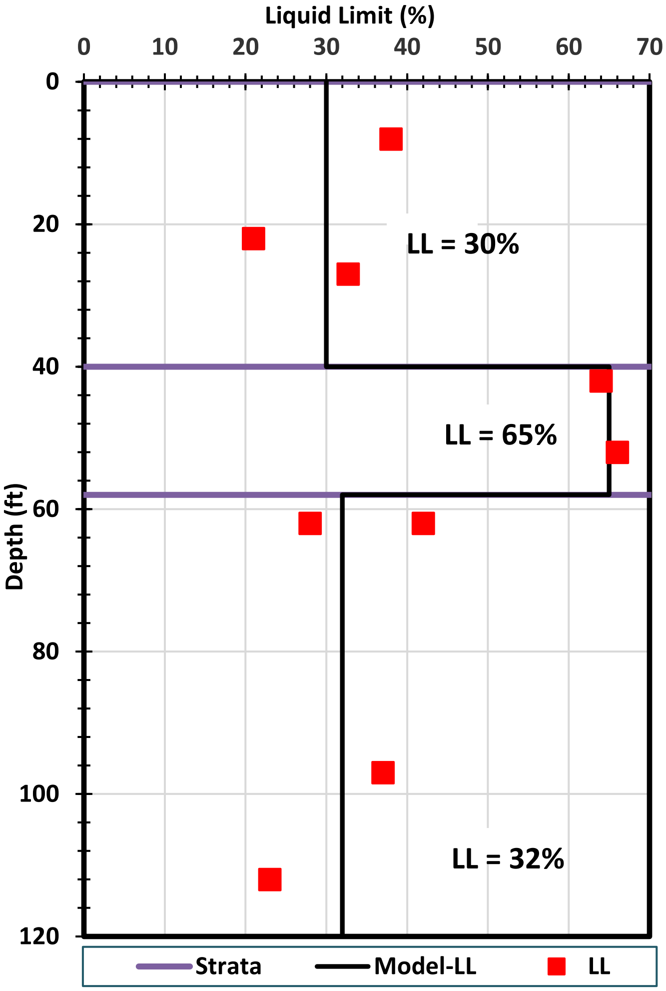


Figure 10. **Liquid Limit Profiles for Birch**

4.4.5 Plasticity Index

A summary of the plasticity index (PI) statistical analysis and design values for each stratum is included as **Table 17** and **Table 18**, where the basis of the design values are highlighted in bold. Statistical analyses and selection of design values were conducted in accordance with the methods described in **Section 3.4**. The basis of all design values is the average of the laboratory test results. Plots of the plasticity index data and design values are included as **Figure 11** and **Figure 12.**

Table 17 Plasticity Index Statistical Analysis and Design Values— Walnut

Stratum	Laboratory Data				Design Value - PI(%)
	Sample Size (n)	Avg. (%)	Min. (%)	Max. (%)	
Silty Sand and Clayey Sand	2	18	13	22	**18**
Silty Clay and Sandy Clay	3	29	13	38	**29**
Silty Sand and Clayey Sand	—	—	—	—	**—**
Silty Clay and Sandy Clay	1	16	16	16	**16**
Silty Sand and Clayey Sand	1	2	2	2	**2**

Table 18 Plasticity Index Statistical Analysis and Design Values— Birch

Stratum	Laboratory Data				Design Value - PI(%)
	Sample Size (n)	Avg. (%)	Min. (%)	Max. (%)	
Silty Sand and Clayey Sand	3	15	4	24	**15**
Silty Clay and Sandy Clay	2	42	40	44	**42**
Silty Sand and Clayey Sand	4	19	12	27	**19**

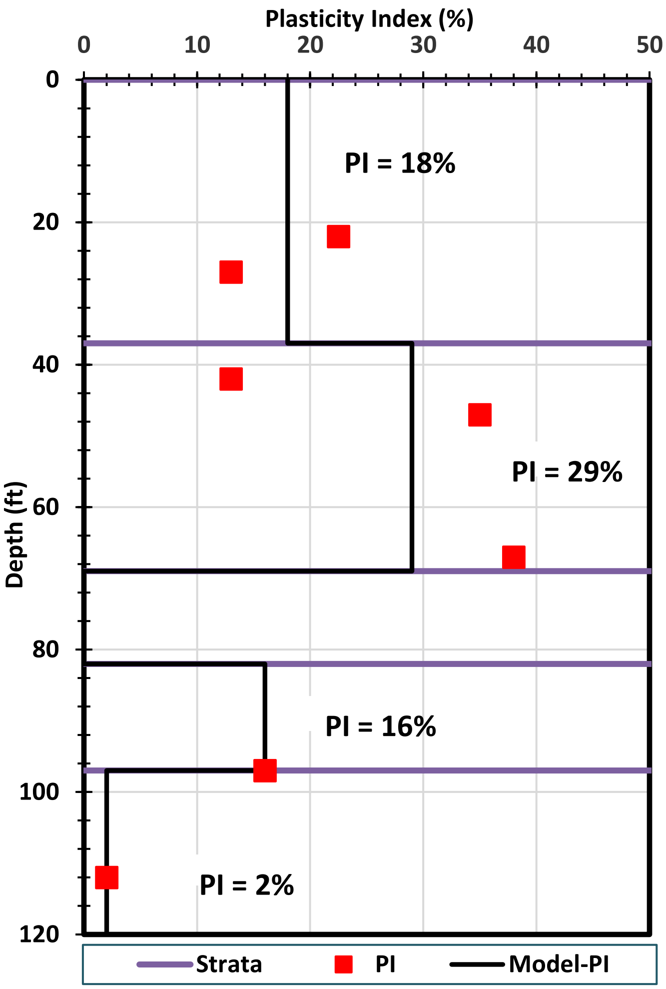


Figure 11 Plasticity Index Profiles for Walnut

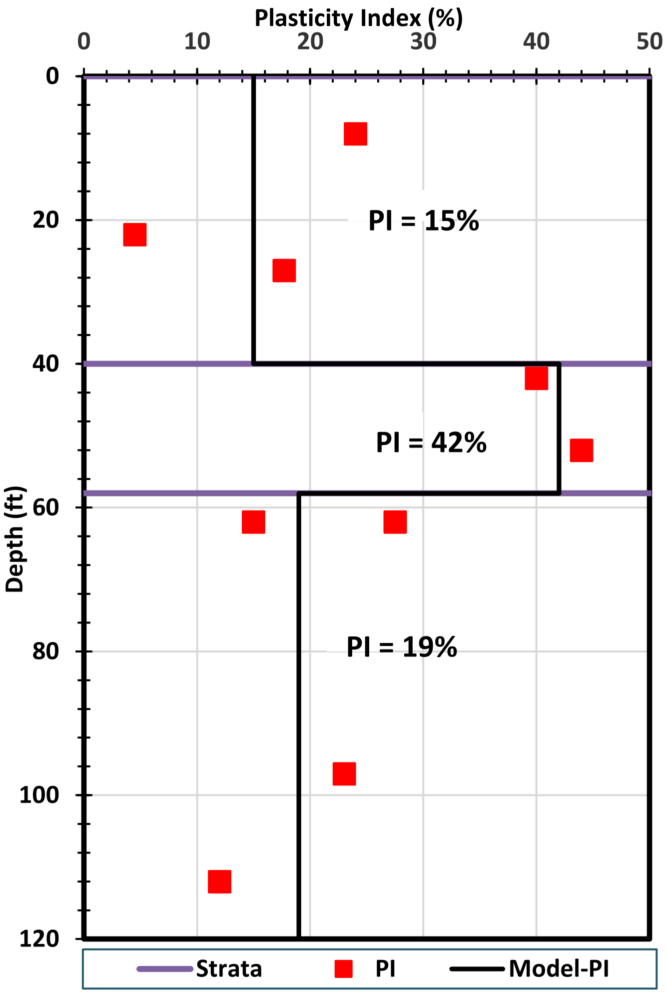


Figure 12 Plasticity Index Profiles for Birch

4.5 Soil Strength Parameters

4.5.1 Q-Case

A summary of the Q-case strength statistical analysis and design values for each stratum is included as **Table 19**, where the basis of the design values are highlighted in bold. Statistical analyses and selection of design values were conducted in accordance with the methods described in **Section 3.5.1**. Two UC tests on samples from relatively shallow subsurface depths up to 14 feet, and four UU tests on samples from deeper subsurface depths were performed for Walnut Creek Dam. Only UC tests were performed for Birch Creek Dam borings. The tests were performed on both clays and clayey sands.

The measured minimum s_u value for clayey sands is the basis of design value with justification based on the average N_{60} value of 25 bpf for Birch Creek. Table 8-10 in **Reference 12** present range of s_u (2000 to 4000 psf) for N values ranging from 15 to 30 bpf.

Plots of the strength data and design values are included as **Figure 13.**

Table 19 **Q-Case Statistical Analysis and Design Values— Walnut**

Stratum	Laboratory (UU and UC)						Design Value (psf)
	Sample Size (n)	Std. deviation (psf)	Min. (psf)	Avg. (psf)	33rd Percentile (psf)	95% Lower Confidence Limit (psf)	
Silty Sand and Clayey Sand[1]	1	—	—	1030	—	—	**1030**
Silty Clay and Sandy Clay[2]	5	1351	690	2400	1924	722	**722**

1. Applicable to all sandy strata.
2. Applicable to all clayey strata.

Table 20 **Q-Case Statistical Analysis and Design Values— Birch**

Stratum	Laboratory (UU and UC)						Design Value (psf)
	Sample Size (n)	Std. deviation (psf)	Min. (psf)	Avg. (psf)	33rd Percentile (psf)	95% Lower Confidence Limit (psf)	
Silty Sand and Clayey Sand[1]	3	335	1090	1327	1149	494	**1000**
Silty Clay and Sandy Clay[2]	—			—	—	—	**722**

1. Applicable to all sandy strata.
2. Adopted from **Table 19** based on similar soil description.

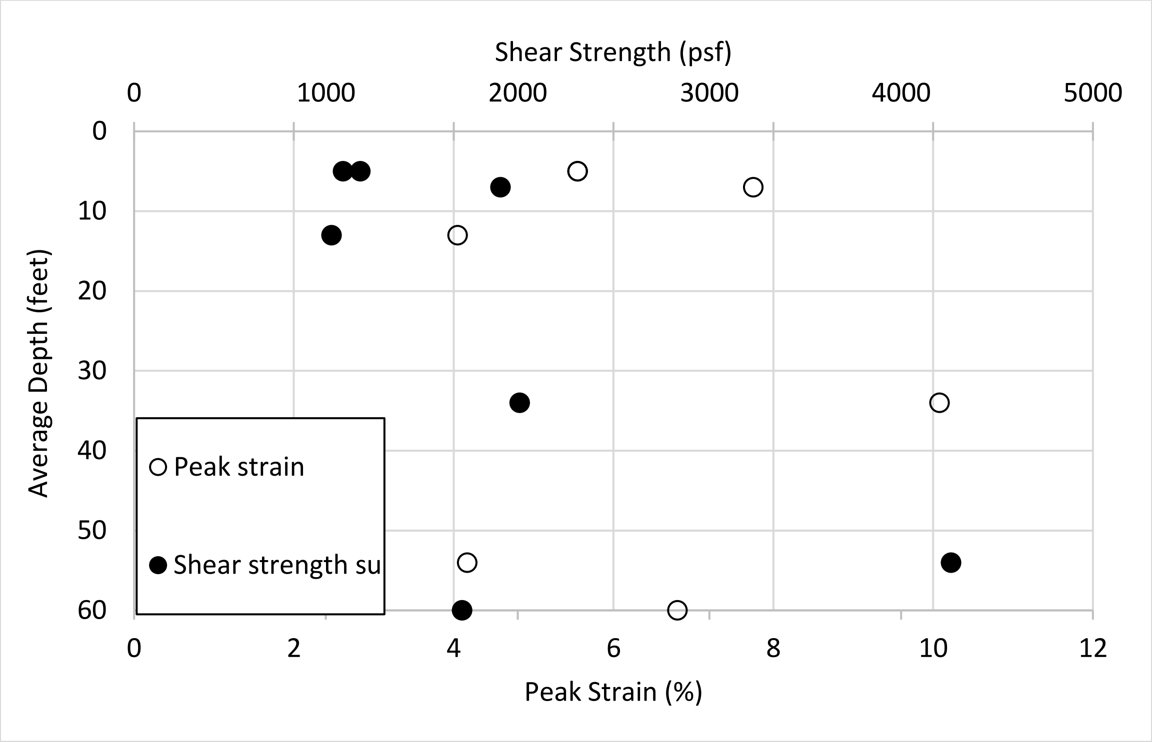


Figure 13. **UC and UU Triaxial Strength versus Depth of Soil Sample for All Borings**

4.5.2 S-Case

A summary of the S-case strength statistical analysis and design values for each stratum is included as **Table 21** and **Table 22**, where the basis of the design values are highlighted in bold. Statistical analyses and selection of ϕ’ design values were conducted in accordance with the methods described in **Section 3.5.2**.

For fine-grained strata, design envelopes were developed from CU laboratory testing of effective friction angle. The CU test data sheets that present the selection of α and calculation of ϕ’ are included as **Attachment 3** of this calculation package.

The average of the two sets of CU triaxial tests performed on clayey soils is the basis of design effective friction for the clay strata. One CU triaxial test was performed on sandy soils, hence a single CU triaxial test is the basis of design effective friction for sand strata. Correlations using PI were considered for comparison. In general, the PI correlation results are closely matched to the design envelope especially for the clay layers.

Recognizing the presence of sand and silt, and the uncertainty in the effective cohesion and its important influence at low normal stress, effective design cohesion for all strata is assumed to be zero (0) psf.

Figure 14 shows design envelopes assuming zero cohesion.

Table 21 **S-Case Analysis – All Strata, Walnut**

Stratum	Laboratory Data				Design Envelope ϕ' (°)	Index Properties ϕ' Correlations (PI)			Design Value (°)
	Sample Size (n)	Avg. CU ϕ' (°)	Min. (°)	Max. (°)		Sample Size (n)	Avg. PI (%)	Sorensen & Okkels (2013) (°)	
Silty Sand and Clayey Sand	1	31	31	31	31	2	18	26	**31**
Silty Clay and Sandy Clay	2	21	18	24.4	21	3	29	24	**21**
Silty Sand and Clayey Sand	1	31	31	31	31	—	—	—	**31**
Silty Clay and Sandy Clay	2	21	18	24.4	21	1	16	27	**21**
Silty Sand and Clayey Sand	1	31	31	31	31	1	2	40	**31**

Table 22 **S-Case Analysis – All Strata, Birch**

Stratum	Laboratory Data				Design Envelope ϕ' (°)	Index Properties ϕ' Correlations (PI)			Design Value (°)
	Sample Size (n)	Avg. CU ϕ' (°)	Min. (°)	Max. (°)		Sample Size (n)	Avg. PI (%)	Sorensen & Okkels (2013) (°)	
Silty Sand and Clayey Sand	1	31	31	31	31	3	15	28	**31**
Silty Clay and Sandy Clay	2	21	18	24.4	21	2	42	21	**21**
Silty Sand and Clayey Sand	1	31	31	31	31	4	19	26	**31**

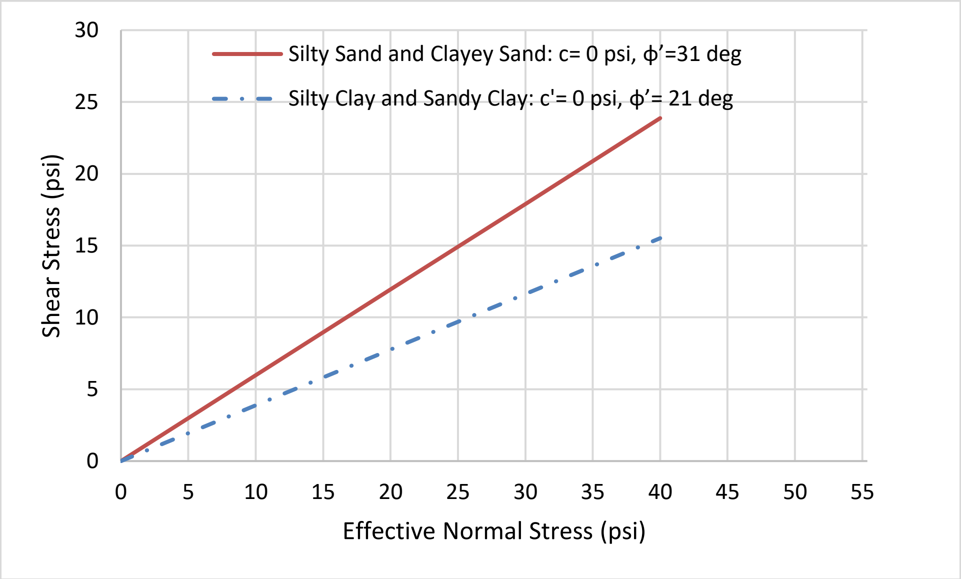


Figure 14. **Effective Stress Shear Strength Design Envelope from CU Tests**

4.5.3 R-Case

A summary of the R-Case ϕ_R and c_R design values for each stratum is included as **Table 23**. The analysis and selection of design values were conducted in accordance with the methods described in **Section 3.5.3**. Two CU tests were available for the clayey soils and one for the sandy soils. The lower of calculated cR and ϕR is the design basis for the clayey soils. cR and ϕR from single test result is the design basis for sandy soils. **Figure 15** shows design R-envelopes.

Table 23 R-Case Design Values— Walnut Creek and Birch Creek

	Laboratory Data					Design Envelope	
		ϕR (°)		CR (psf)			
Stratum	**Sample Size (n)**	**Min.**	**Max.**	**Min.**	**Max.**	**ϕR (°)**	**CR (psf)**
Silty Sand and Clayey Sand[1]	1	23.6	23.6	210	210	**23.6**	**210**
Silty Clay and Sandy Clay[2]	2	14.6	18	240	830	**14.6**	**240**

1. Applicable to all sandy strata.
2. Applicable to all clayey strata.

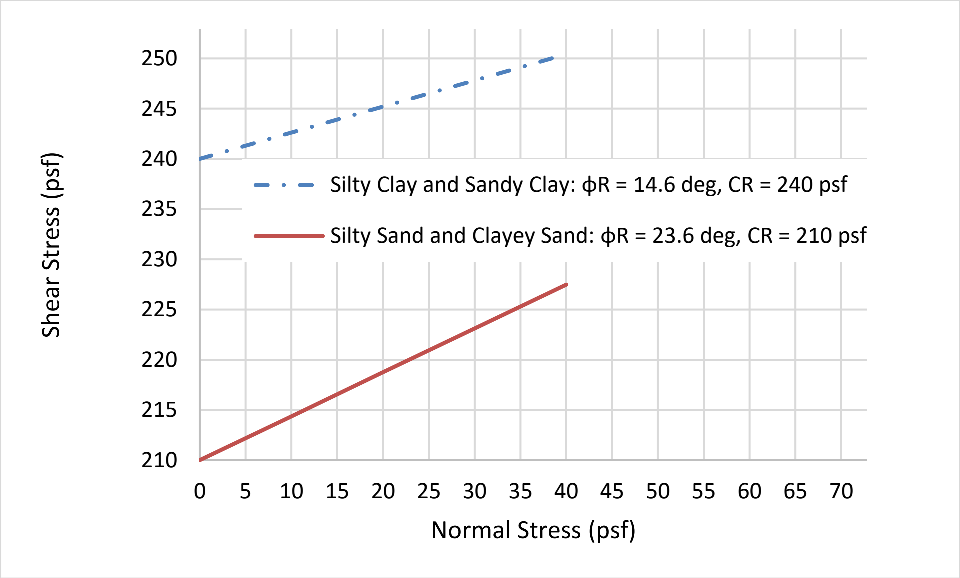


Figure 15. R-Envelope Shear Strength Design Envelope from CU Tests

4.6 Hydraulic Conductivity

A summary of the hydraulic conductivity statistical analysis and design values for each stratum is included as **Table 24**, where the basis of the design values is highlighted in bold. Statistical analyses and selection of design values were conducted in accordance with the methods described in **Section 3.6.1**. Four permeability tests were performed on clayey soils and three on sandy soils. The geometric mean of available permeability laboratory testing is the preferred design basis. The geometric mean for clayey soils permeability is the design value for Silty Clay and Sandy Clay stratum and the geometric mean for sandy soils permeability is the design value for Silty Sand and Clayey Sand stratum. A sensitivity seepage analyses is recommended based on the range of design values.

Table 24 Hydraulic Conductivity Statistical Analysis— Walnut Creek and Birch Creek

Stratum	Laboratory Data				Design Values (cm/s)	Range of Design Values (cm/s)
	Sample Size (n)	Geometric Mean (cm/s)	Min. (cm/s)	Max. (cm/s)		
Silty Sand and Clayey Sand[1]	3	1.20×10^{-7}	1.09×10^{-7}	1.28×10^{-7}	**1×10^{-7}**	**1×10^{-5} — 1×10^{-8}**
Silty Clay and Sandy Clay[2]	4	1.38×10^{-8}	8.89×10^{-9}	2.83×10^{-8}	**1×10^{-8}**	**1×10^{-6} — 1×10^{-9}**

1. Applicable to all sandy strata.
2. Applicable to all clayey strata.

4.7 Consolidation Parameters

4.7.1 Initial Void Ratio (e_0)

A summary of the void ratio statistical analysis and design values for each stratum is included as **Table 25**, where the basis of the design values is highlighted in bold. Statistical analyses and selection of design values were conducted in accordance with the methods described in **Section 3.7.1.** The average of available laboratory testing is the design basis, where typical values from Das (2010) (**Reference 19),** Table 3.2 were provided for comparison. No consolidation testing was performed on the Silty Sand and Clayey Sand stratum, hence void ratio was calculated based moisture content (MC) from **Table 13**, an assumed specific gravity (SG) of 2.7 and an assumed saturation (S) of 100%.

Table 25 Initial Void Ratio Statistical Analysis, Data Comparison, and Design Values— Walnut Creek

Stratum	Typical Values from Das (2010) (Reference 19, Table 3.2)		Laboratory Data				Calculated Value based on MC, SG, S– e_0	Design Value – e_0
	Soil Type	Typical Void Ratio	Sample Size (n)	Avg.	Min.	Max.		
Silty Sand and Clayey Sand	Dense Angular-Grained Silty Sand	0.4	—	—	—	—	0.351	**0.4**
Silty Clay and Sandy Clay	Stiff Clay	0.6	2	0.7093	0.5121	0.9065	—	**0.6**
Silty Sand and Clayey Sand	Dense Angular-Grained Silty Sand	0.4	—	—	—	—	0.594	**0.59**
Silty Clay and Sandy Clay	Stiff Clay	0.6	1	0.5459	0.5459	0.5459	—	**0.5**
Silty Sand and Clayey Sand	Dense Angular-Grained Silty Sand	0.4	—	—	—	—	0.567	**0.56**

Table 26. **Initial Void Ratio Statistical Analysis, Data Comparison, and Design Values—Birch Creek**

	Typical Values from Das (2010) (Reference 19, Table 3.2)		Laboratory Data					
Stratum	**Soil Type**	**Typical Void Ratio**	**Sample Size (n)**	**Avg.**	**Min.**	**Max.**	**Calculated Value based on MC, SG, S– e_0**	**Design Value – e_0**
Silty Sand and Clayey Sand	Dense Angular-Grained Silty Sand	0.4	—	—	—	—	0.405	**0.4**
Silty Clay and Sandy Clay	Stiff Clay	0.6	1	0.7329	0.7329	0.7329	—	**0.7**
Silty Sand and Clayey Sand	Dense Angular-Grained Silty Sand	0.4	—	—	—	—	0.54	**0.54**

4.7.2 Virgin Compression Index (C_c)

A summary of the compression index statistical analysis and design values for each fine-grained stratum is included as **Table 27**, where the basis of the design values is highlighted in bold. Analysis and selection of design values were conducted in general accordance with the methods described in **Subsection 3.7.2**. Index properties used in correlations are the design values specified in **Table 13** and **Table 14** for Moisture Content (*MC*), **Table 15** and **Table 16** for Liquid Limit (*LL*), and **Table 25** and **Table 26** for Void Ratio (e_0).
In strata where consolidation laboratory testing is available, the average of the C_c values from the Casagrande Method are the design basis. Full calculations detailing the Casagrande method for determination of C_c are provided as an **Attachment 4** of this calculation package.

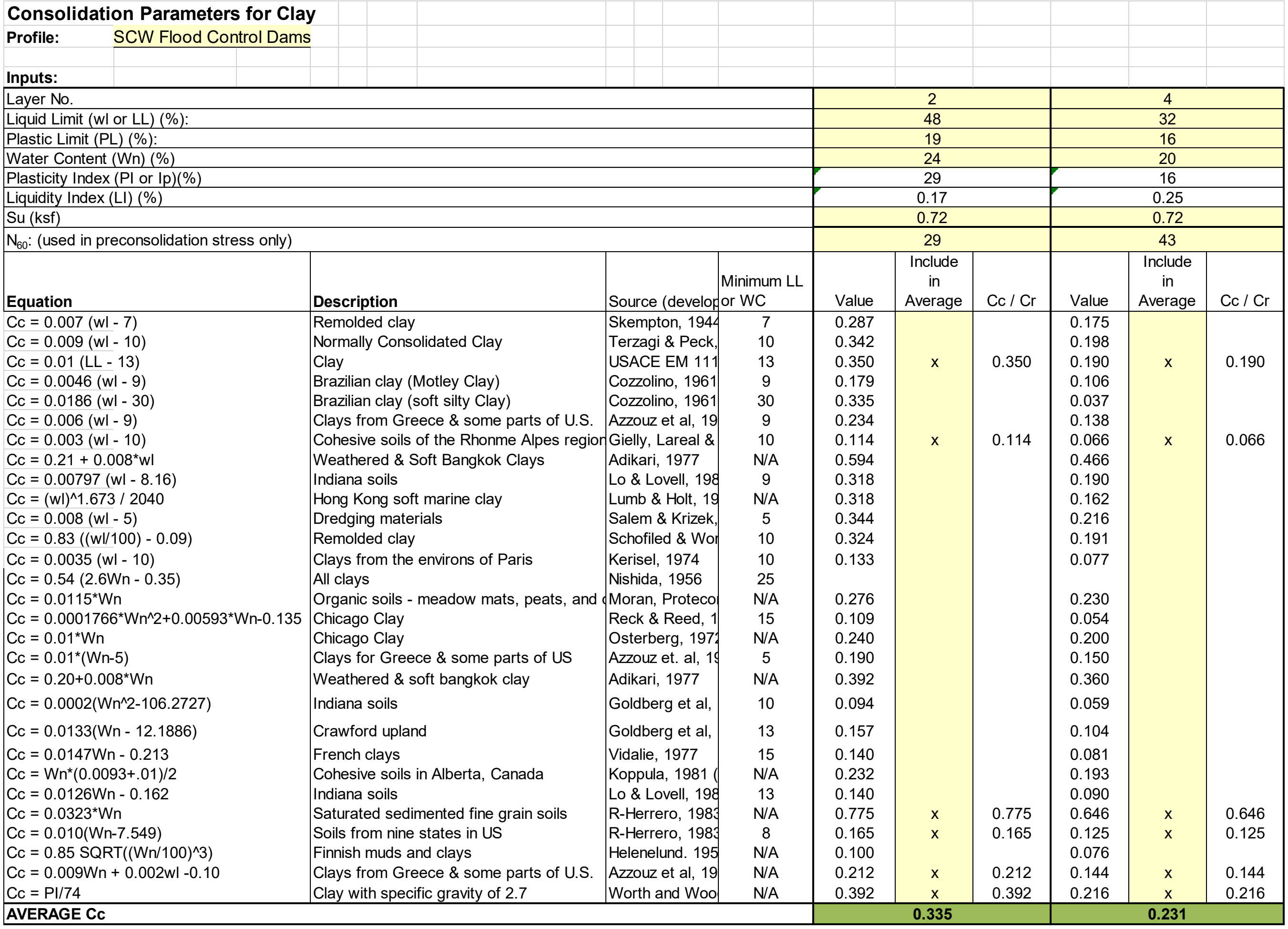
Correlations for C_c based on index properties (*MC, LL, PL*) using BV template (**Reference 15**) is included as **Figure 16** and **Figure 17** for comparison.

Table 27 **Virgin Compression Index Statistical Analysis, Method Comparison, and Design Values— Walnut Creek**

	Casagrande Method (Consolidation Testing)				Correlations (Index Testing)				
Stratum	**Sample Size (n)**	**Avg.**	**Min.**	**Max.**	**Void Ratio**	**MC**	**LL**	**Average of e_0, MC and LL Correlations**	**Design Value - C_c**
Silty Sand and Clayey Sand	—								
Silty Clay and Sandy Clay	2	0.2304	0.1498	0.3109	0.413	0.288	0.35	0.350	**0.23**
Silty Sand and Clayey Sand	—								
Silty Clay and Sandy Clay	1	0.111	0.111	0.111	0.225	0.24	0.19	0.218	**0.11**
Silty Sand and Clayey Sand	—								

Table 28. Virgin Compression Index Statistical Analysis, Method Comparison, and Design Values— Birch Creek

Stratum	Casagrande Method (Consolidation Testing)				Correlations (Index Testing)				Design Value - C_c
	Sample Size (n)	Avg.	Min.	Max.	Void Ratio	MC	LL	Average of e_0, MC and LL Correlations	
Silty Sand and Clayey Sand	—								
Silty Clay and Sandy Clay	1	0.2362	0.2362	0.2362	0.440	0.30	0.52	0.42	**0.24**
Silty Sand and Clayey Sand	—								



Consolidation Parameters for Clay

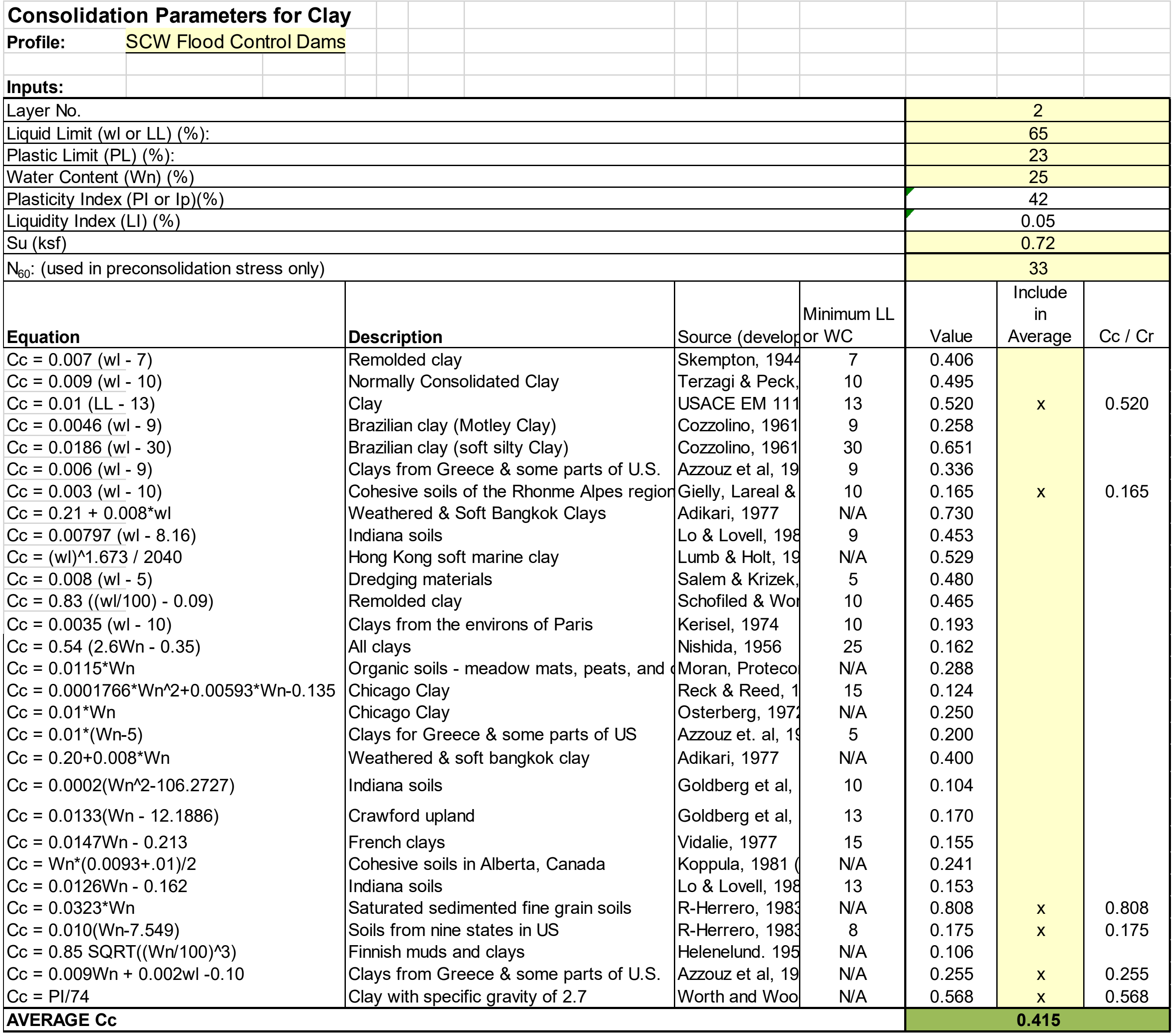
Profile: SCW Flood Control Dams

Inputs:

Layer No.	2	4
Liquid Limit (wl or LL) (%):	48	32
Plastic Limit (PL) (%):	19	16
Water Content (Wn) (%)	24	20
Plasticity Index (PI or Ip)(%)	29	16
Liquidity Index (LI) (%)	0.17	0.25
Su (ksf)	0.72	0.72
N_{60}: (used in preconsolidation stress only)	29	43

Equation	Description	Source (develop	Minimum LL or WC	Value	Include in Average	Cc / Cr	Value	Include in Average	Cc / Cr
Cc = 0.007 (wl - 7)	Remolded clay	Skempton, 1944	7	0.287			0.175		
Cc = 0.009 (wl - 10)	Normally Consolidated Clay	Terzagi & Peck,	10	0.342			0.198		
Cc = 0.01 (LL - 13)	Clay	USACE EM 111	13	0.350	x	0.350	0.190	x	0.190
Cc = 0.0046 (wl - 9)	Brazilian clay (Motley Clay)	Cozzolino, 1961	9	0.179			0.106		
Cc = 0.0186 (wl - 30)	Brazilian clay (soft silty Clay)	Cozzolino, 1961	30	0.335			0.037		
Cc = 0.006 (wl - 9)	Clays from Greece & some parts of U.S.	Azzouz et al, 19	9	0.234			0.138		
Cc = 0.003 (wl - 10)	Cohesive soils of the Rhonme Alpes region	Gielly, Lareal &	10	0.114	x	0.114	0.066	x	0.066
Cc = 0.21 + 0.008*wl	Weathered & Soft Bangkok Clays	Adikari, 1977	N/A	0.594			0.466		
Cc = 0.00797 (wl - 8.16)	Indiana soils	Lo & Lovell, 198	9	0.318			0.190		
Cc = (wl)^1.673 / 2040	Hong Kong soft marine clay	Lumb & Holt, 19	N/A	0.318			0.162		
Cc = 0.008 (wl - 5)	Dredging materials	Salem & Krizek,	5	0.344			0.216		
Cc = 0.83 ((wl/100) - 0.09)	Remolded clay	Schofiled & Wor	10	0.324			0.191		
Cc = 0.0035 (wl - 10)	Clays from the environs of Paris	Kerisel, 1974	10	0.133			0.077		
Cc = 0.54 (2.6Wn - 0.35)	All clays	Nishida, 1956	25						
Cc = 0.0115*Wn	Organic soils - meadow mats, peats, and (	Moran, Protecoi	N/A	0.276			0.230		
Cc = 0.0001766*Wn^2+0.00593*Wn-0.135	Chicago Clay	Reck & Reed, 1	15	0.109			0.054		
Cc = 0.01*Wn	Chicago Clay	Osterberg, 1972	N/A	0.240			0.200		
Cc = 0.01*(Wn-5)	Clays for Greece & some parts of US	Azzouz et. al, 19	5	0.190			0.150		
Cc = 0.20+0.008*Wn	Weathered & soft bangkok clay	Adikari, 1977	N/A	0.392			0.360		
Cc = 0.0002(Wn^2-106.2727)	Indiana soils	Goldberg et al,	10	0.094			0.059		
Cc = 0.0133(Wn - 12.1886)	Crawford upland	Goldberg et al,	13	0.157			0.104		
Cc = 0.0147Wn - 0.213	French clays	Vidalie, 1977	15	0.140			0.081		
Cc = Wn*(0.0093+.01)/2	Cohesive soils in Alberta, Canada	Koppula, 1981 (	N/A	0.232			0.193		
Cc = 0.0126Wn - 0.162	Indiana soils	Lo & Lovell, 198	13	0.140			0.090		
Cc = 0.0323*Wn	Saturated sedimented fine grain soils	R-Herrero, 1983	N/A	0.775	x	0.775	0.646	x	0.646
Cc = 0.010(Wn-7.549)	Soils from nine states in US	R-Herrero, 1983	8	0.165	x	0.165	0.125	x	0.125
Cc = 0.85 SQRT((Wn/100)^3)	Finnish muds and clays	Helenelund. 195	N/A	0.100			0.076		
Cc = 0.009Wn + 0.002wl -0.10	Clays from Greece & some parts of U.S.	Azzouz et al, 19	N/A	0.212	x	0.212	0.144	x	0.144
Cc = PI/74	Clay with specific gravity of 2.7	Worth and Woo	N/A	0.392	x	0.392	0.216	x	0.216
AVERAGE Cc					**0.335**			**0.231**	

Figure 16. Consolidation Settlement Parameters of Clay (Reference 15) — Walnut



Consolidation Parameters for Clay

Profile: SCW Flood Control Dams

Inputs:

Layer No.	2
Liquid Limit (wl or LL) (%):	65
Plastic Limit (PL) (%):	23
Water Content (Wn) (%)	25
Plasticity Index (PI or Ip)(%)	42
Liquidity Index (LI) (%)	0.05
Su (ksf)	0.72
N_{60}: (used in preconsolidation stress only)	33

Equation	Description	Source (develop	Minimum LL or WC	Value	Include in Average	Cc / Cr
Cc = 0.007 (wl - 7)	Remolded clay	Skempton, 1944	7	0.406		
Cc = 0.009 (wl - 10)	Normally Consolidated Clay	Terzagi & Peck,	10	0.495		
Cc = 0.01 (LL - 13)	Clay	USACE EM 111	13	0.520	x	0.520
Cc = 0.0046 (wl - 9)	Brazilian clay (Motley Clay)	Cozzolino, 1961	9	0.258		
Cc = 0.0186 (wl - 30)	Brazilian clay (soft silty Clay)	Cozzolino, 1961	30	0.651		
Cc = 0.006 (wl - 9)	Clays from Greece & some parts of U.S.	Azzouz et al, 19	9	0.336		
Cc = 0.003 (wl - 10)	Cohesive soils of the Rhonme Alpes region	Gielly, Lareal &	10	0.165	x	0.165
Cc = 0.21 + 0.008*wl	Weathered & Soft Bangkok Clays	Adikari, 1977	N/A	0.730		
Cc = 0.00797 (wl - 8.16)	Indiana soils	Lo & Lovell, 198	9	0.453		
Cc = (wl)^1.673 / 2040	Hong Kong soft marine clay	Lumb & Holt, 19	N/A	0.529		
Cc = 0.008 (wl - 5)	Dredging materials	Salem & Krizek,	5	0.480		
Cc = 0.83 ((wl/100) - 0.09)	Remolded clay	Schofiled & Wor	10	0.465		
Cc = 0.0035 (wl - 10)	Clays from the environs of Paris	Kerisel, 1974	10	0.193		
Cc = 0.54 (2.6Wn - 0.35)	All clays	Nishida, 1956	25	0.162		
Cc = 0.0115*Wn	Organic soils - meadow mats, peats, and (	Moran, Protecoı	N/A	0.288		
Cc = 0.0001766*Wn^2+0.00593*Wn-0.135	Chicago Clay	Reck & Reed, 1	15	0.124		
Cc = 0.01*Wn	Chicago Clay	Osterberg, 197	N/A	0.250		
Cc = 0.01*(Wn-5)	Clays for Greece & some parts of US	Azzouz et. al, 1	5	0.200		
Cc = 0.20+0.008*Wn	Weathered & soft bangkok clay	Adikari, 1977	N/A	0.400		
Cc = 0.0002(Wn^2-106.2727)	Indiana soils	Goldberg et al,	10	0.104		
Cc = 0.0133(Wn - 12.1886)	Crawford upland	Goldberg et al,	13	0.170		
Cc = 0.0147Wn - 0.213	French clays	Vidalie, 1977	15	0.155		
Cc = Wn*(0.0093+.01)/2	Cohesive soils in Alberta, Canada	Koppula, 1981 (	N/A	0.241		
Cc = 0.0126Wn - 0.162	Indiana soils	Lo & Lovell, 198	13	0.153		
Cc = 0.0323*Wn	Saturated sedimented fine grain soils	R-Herrero, 198	N/A	0.808	x	0.808
Cc = 0.010(Wn-7.549)	Soils from nine states in US	R-Herrero, 198	8	0.175	x	0.175
Cc = 0.85 SQRT((Wn/100)^3)	Finnish muds and clays	Helenelund. 195	N/A	0.106		
Cc = 0.009Wn + 0.002wl -0.10	Clays from Greece & some parts of U.S.	Azzouz et al, 19	N/A	0.255	x	0.255
Cc = PI/74	Clay with specific gravity of 2.7	Worth and Woo	N/A	0.568	x	0.568
AVERAGE Cc					**0.415**	

Figure 17. Consolidation Settlement Parameters of Clay (Reference 15) — Birch

4.7.3 Recompression Index (C_r)

A summary of the recompression index statistical analysis and design values for each fine-grained stratum is included as **Table 29**, where the basis of the design values is highlighted in bold. Analysis and selection of design values were conducted in accordance with the methods described in **Section 3.7.2**. Where laboratory testing is available, only in clay strata, the average of the C_r values from the Casagrande Method are the design basis. Using BV template (**Reference 15**), C_r from C_c correlations are presented as for comparison as **Figure 18** and **Figure 19**.

Full calculations detailing the Casagrande method for determination of C_r are provided as an **Attachment 4** of this calculation package.

Table 29 **Recompression Index Analysis and Design Values— Walnut Creek**

Stratum	Casagrande Method (Consolidation Testing)				Assumption $C_r = 1/5C_c$ (C_c from Consolidation Testing)	Design Value
	Sample Size (n)	Avg.	Min.	Max.		
Silty Sand and Clayey Sand	—					
Silty Clay and Sandy Clay	2	0.0443	0.0116	0.0769	0.046	**0.044**
Silty Sand and Clayey Sand	—					
Silty Clay and Sandy Clay	1	0.0133	0.0133	0.0133	0.022	**0.013**
Silty Sand and Clayey Sand	—					

Table 30 **Recompression Index Analysis and Design Values— Birch Creek**

Stratum	Casagrande Method (Consolidation Testing)				Assumption $C_r = 1/5C_c$ (C_c from Consolidation Testing)	Design Value
	Sample Size (n)	Avg.	Min.	Max.		
Silty Sand and Clayey Sand	—					
Silty Clay and Sandy Clay	1	0.0341	0.0341	0.0341	0.047	**0.034**
Silty Sand and Clayey Sand	—					

Consolidation Parameters for Clay

Profile: SCW Flood Control Dams

Inputs:

Layer No.				2			4		
Liquid Limit (wl or LL) (%):				48			32		
Plastic Limit (PL) (%):				19			16		
Water Content (Wn) (%)				24			20		
Plasticity Index (PI or Ip)(%)				29			16		
Liquidity Index (LI) (%)				0.17			0.25		
Su (ksf)				0.72			0.72		
N_{60}: (used in preconsolidation stress only)				29			43		
Equation	**Description**	Source (develop	Minimum LL or WC	Value	Include in Average	Cc / Cr	Value	Include in Average	Cc / Cr
Cc = 0.007 (wl - 7)	Remolded clay	Skempton, 194	7	0.287			0.175		
Cc = 0.009 (wl - 10)	Normally Consolidated Clay	Terzagi & Peck,	10	0.342			0.198		
Cc = 0.01 (LL - 13)	Clay	USACE EM 111	13	0.350	x	0.350	0.190	x	0.190
Cc = 0.0046 (wl - 9)	Brazilian clay (Motley Clay)	Cozzolino, 1961	9	0.179			0.106		
Cc = 0.0186 (wl - 30)	Brazilian clay (soft silty Clay)	Cozzolino, 1961	30	0.335			0.037		
Cc = 0.006 (wl - 9)	Clays from Greece & some parts of U.S.	Azzouz et al, 19	9	0.234			0.138		
Cc = 0.003 (wl - 10)	Cohesive soils of the Rhonme Alpes region	Gielly, Lareal &	10	0.114	x	0.114	0.066	x	0.066
Cc = 0.21 + 0.008*wl	Weathered & Soft Bangkok Clays	Adikari, 1977	N/A	0.594			0.466		
Cc = 0.00797 (wl - 8.16)	Indiana soils	Lo & Lovell, 198	9	0.318			0.190		
Cc = (wl)^1.673 / 2040	Hong Kong soft marine clay	Lumb & Holt, 19	N/A	0.318			0.162		
Cc = 0.008 (wl - 5)	Dredging materials	Salem & Krizek,	5	0.344			0.216		
Cc = 0.83 ((wl/100) - 0.09)	Remolded clay	Schofiled & Wor	10	0.324			0.191		
Cc = 0.0035 (wl - 10)	Clays from the environs of Paris	Kerisel, 1974	10	0.133			0.077		
Cc = 0.54 (2.6Wn - 0.35)	All clays	Nishida, 1956	25						
Cc = 0.0115*Wn	Organic soils - meadow mats, peats, and	Moran, Protecol	N/A	0.276			0.230		
Cc = 0.0001766*Wn^2+0.00593*Wn-0.135	Chicago Clay	Reck & Reed, 1	15	0.109			0.054		
Cc = 0.01*Wn	Chicago Clay	Osterberg, 197	N/A	0.240			0.200		
Cc = 0.01*(Wn-5)	Clays for Greece & some parts of US	Azzouz et. al, 1	5	0.190			0.150		
Cc = 0.20+0.008*Wn	Weathered & soft bangkok clay	Adikari, 1977	N/A	0.392			0.360		
Cc = 0.0002(Wn^2-106.2727)	Indiana soils	Goldberg et al,	10	0.094			0.059		
Cc = 0.0133(Wn - 12.1886)	Crawford upland	Goldberg et al,	13	0.157			0.104		
Cc = 0.0147Wn - 0.213	French clays	Vidalie, 1977	15	0.140			0.081		
Cc = Wn*(0.0093+.01)/2	Cohesive soils in Alberta, Canada	Koppula, 1981 (	N/A	0.232			0.193		
Cc = 0.0126Wn - 0.162	Indiana soils	Lo & Lovell, 198	13	0.140			0.090		
Cc = 0.0323*Wn	Saturated sedimented fine grain soils	R-Herrero, 198	N/A	0.775	x	0.775	0.646	x	0.646
Cc = 0.010(Wn-7.549)	Soils from nine states in US	R-Herrero, 198	8	0.165	x	0.165	0.125	x	0.125
Cc = 0.85 SQRT((Wn/100)^3)	Finnish muds and clays	Helenelund. 195	N/A	0.100			0.076		
Cc = 0.009Wn + 0.002wl -0.10	Clays from Greece & some parts of U.S.	Azzouz et al, 19	N/A	0.212	x	0.212	0.144	x	0.144
Cc = PI/74	Clay with specific gravity of 2.7	Worth and Woo	N/A	0.392	x	0.392	0.216	x	0.216
AVERAGE Cc					**0.335**			**0.231**	
Cr = PI/370	Clay with specific gravity of 2.7	Worth and Woo	N/A	0.078	x	0.078	0.043	x	0.043
Cr = 20 percent of Cc	Clay	EPRI	N/A	0.067	x	0.067	0.046	x	0.046
Cr = 10 percent of Cc	Clay	Rule of Thumb	N/A	0.033	x	0.033	0.023	x	0.023
Cr = 5 percent of Cc	Clay	Rule of Thumb	N/A	0.017			0.012		
AVERAGE Cr (recompression)					**0.060**			**0.038**	

Figure 18. Consolidation Settlement Parameters of Clay (Reference 15) — Walnut

Consolidation Parameters for Clay

Profile: SCW Flood Control Dams

Inputs:

Layer No.	2
Liquid Limit (wl or LL) (%):	65
Plastic Limit (PL) (%):	23
Water Content (Wn) (%)	25
Plasticity Index (PI or Ip)(%)	42
Liquidity Index (LI) (%)	0.05
Su (ksf)	0.72
N_{60}: (used in preconsolidation stress only)	33

Equation	Description	Source (develop	Minimum LL or WC	Value	Include in Average	Cc / Cr
Cc = 0.007 (wl - 7)	Remolded clay	Skempton, 194	7	0.406		
Cc = 0.009 (wl - 10)	Normally Consolidated Clay	Terzagi & Peck,	10	0.495		
Cc = 0.01 (LL - 13)	Clay	USACE EM 111	13	0.520	x	0.520
Cc = 0.0046 (wl - 9)	Brazilian clay (Motley Clay)	Cozzolino, 1961	9	0.258		
Cc = 0.0186 (wl - 30)	Brazilian clay (soft silty Clay)	Cozzolino, 1961	30	0.651		
Cc = 0.006 (wl - 9)	Clays from Greece & some parts of U.S.	Azzouz et al, 19	9	0.336		
Cc = 0.003 (wl - 10)	Cohesive soils of the Rhonme Alpes region	Gielly, Lareal &	10	0.165	x	0.165
Cc = 0.21 + 0.008*wl	Weathered & Soft Bangkok Clays	Adikari, 1977	N/A	0.730		
Cc = 0.00797 (wl - 8.16)	Indiana soils	Lo & Lovell, 198	9	0.453		
Cc = (wl)^1.673 / 2040	Hong Kong soft marine clay	Lumb & Holt, 19	N/A	0.529		
Cc = 0.008 (wl - 5)	Dredging materials	Salem & Krizek,	5	0.480		
Cc = 0.83 ((wl/100) - 0.09)	Remolded clay	Schofiled & Wor	10	0.465		
Cc = 0.0035 (wl - 10)	Clays from the environs of Paris	Kerisel, 1974	10	0.193		
Cc = 0.54 (2.6Wn - 0.35)	All clays	Nishida, 1956	25	0.162		
Cc = 0.0115*Wn	Organic soils - meadow mats, peats, and	Moran, Proteco	N/A	0.288		
Cc = 0.0001766*Wn^2+0.00593*Wn-0.135	Chicago Clay	Reck & Reed, 1	15	0.124		
Cc = 0.01*Wn	Chicago Clay	Osterberg, 197	N/A	0.250		
Cc = 0.01*(Wn-5)	Clays for Greece & some parts of US	Azzouz et. al, 1	5	0.200		
Cc = 0.20+0.008*Wn	Weathered & soft bangkok clay	Adikari, 1977	N/A	0.400		
Cc = 0.0002(Wn^2-106.2727)	Indiana soils	Goldberg et al,	10	0.104		
Cc = 0.0133(Wn - 12.1886)	Crawford upland	Goldberg et al,	13	0.170		
Cc = 0.0147Wn - 0.213	French clays	Vidalie, 1977	15	0.155		
Cc = Wn*(0.0093+.01)/2	Cohesive soils in Alberta, Canada	Koppula, 1981 (	N/A	0.241		
Cc = 0.0126Wn - 0.162	Indiana soils	Lo & Lovell, 198	13	0.153		
Cc = 0.0323*Wn	Saturated sedimented fine grain soils	R-Herrero, 198	N/A	0.808	x	0.808
Cc = 0.010(Wn-7.549)	Soils from nine states in US	R-Herrero, 198	8	0.175	x	0.175
Cc = 0.85 SQRT((Wn/100)^3)	Finnish muds and clays	Helenelund. 195	N/A	0.106		
Cc = 0.009Wn + 0.002wl -0.10	Clays from Greece & some parts of U.S.	Azzouz et al, 19	N/A	0.255	x	0.255
Cc = PI/74	Clay with specific gravity of 2.7	Worth and Woo	N/A	0.568	x	0.568
AVERAGE Cc				**0.415**		
Cr = PI/370	Clay with specific gravity of 2.7	Worth and Woo	N/A	0.114	x	0.114
Cr = 20 percent of Cc	Clay	EPRI	N/A	0.083	x	0.083
Cr = 10 percent of Cc	Clay	Rule of Thumb	N/A	0.041	x	0.041
Cr = 5 percent of Cc	Clay	Rule of Thumb	N/A	0.021		
AVERAGE Cr (recompression)				**0.079**		

Figure 19. Consolidation Settlement Parameters of Clay (Reference 15) — Birch

4.7.4 Overconsolidation Ratio (OCR)

A summary of the overconsolidation ratio statistical analysis and design values for each fine-grained stratum is included as **Table 31** and **Table 32**, where the basis of the design values is highlighted in bold. Analysis and selection of design values were conducted in general accordance with the methods described in **Subsection 3.7.2**. Where laboratory testing is available, the average of the *OCR* values from the Casagrande Method are the design basis. OCR with depth trends towards normally consolidated conditions shown as **Figure 20**.

Table 31 **Overconsolidation Ratio Statistical Analysis, Data Comparison, and Design Values— Walnut**

	Casagrande Method (Consolidation Testing)				
Stratum	**Sample Size (n)**	**Avg.**	**Min.**	**Max.**	**Design Value**
Silty Sand and Clayey Sand	—				
Silty Clay and Sandy Clay	2	3.05	2.9	3.2	**3.05**
Silty Sand and Clayey Sand	—				
Silty Clay and Sandy Clay	1	1.4	1.4	1.4	**1.4**
Silty Sand and Clayey Sand	—				

Table 32 **Overconsolidation Ratio Statistical Analysis, Data Comparison, and Design Values— Birch**

	Casagrande Method (Consolidation Testing)				
Stratum	**Sample Size (n)**	**Avg.**	**Min.**	**Max.**	**Design Value**
Silty Sand and Clayey Sand	—				
Silty Clay and Sandy Clay	1	3.8	3.8	3.8	**3.8**
Silty Sand and Clayey Sand	—				

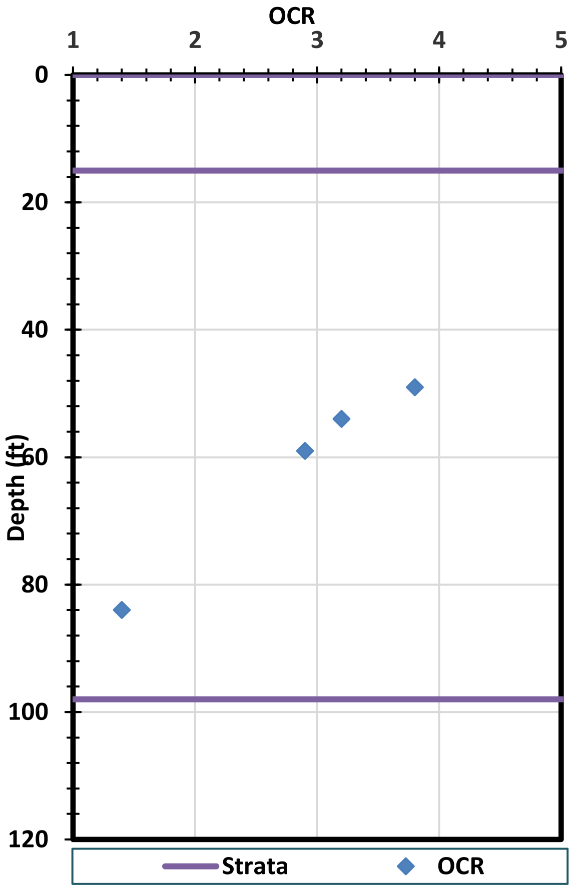


Figure 20 **OCR From Laboratory Consolidation Tests**

4.7.5 Coefficient of Consolidation (C_v)

A summary of the c_v design values for each fine-grained stratum is included as **Table 33**, where the basis of the design values is highlighted in bold. The design c_v were selected from laboratory tests as described in **Subsection 3.7.3**. Where laboratory data was available, the design value for C_v was taken as the geometric mean of C_v evaluated at each load increment.

Table 33 **Summary of Interpolated Coefficient of Consolidation Design Values**

Stratum	Laboratory Tests			Design Value
	Number of Consolidation Tests	Sample Size (n)	Geometric Mean of C_v (m^2/day)	
Walnut				
Silty Sand and Clayey Sand	—			
Silty Clay and Sandy Clay	2	6	9.50×10^{-4}	**9.50×10^{-4}**
Silty Sand and Clayey Sand	—			
Silty Clay and Sandy Clay	1	3	1.55×10^{-2}	**1.55×10^{-2}**
Silty Sand and Clayey Sand	—			
Birch				
Silty Sand and Clayey Sand	—			
Silty Clay and Sandy Clay	1	3	4.57×10^{-4}	**4.57×10^{-4}**
Silty Sand and Clayey Sand	—			

4.7.6 Secondary Compression Index (C_α)

A summary of the correlation results and C_α design values for each fine-grained stratum is included as **Table 34** and **Table 35**, where the basis of the design values is highlighted in bold. Analysis and selection of design values were conducted in general accordance with the methods described in **Subsection 3.7.4**.

Table 34 **Secondary Compression Index Analysis and Design Values— Walnut**

Stratum	EM1110-1-1904 C_α/C_c Correlation (Reference 13)				Design Value
	Soil Type	Mid-Range C_α/C_c	Design C_c	Calculated C_α	
Silty Sand and Clayey Sand	—				
Silty Clay and Sandy Clay	Clay	0.055	0.23	0.013	**0.013**
Silty Sand and Clayey Sand	—				
Silty Clay and Sandy Clay	Clay	0.055	0.11	0.006	**0.006**
Silty Sand and Clayey Sand	—				

Table 35 **Secondary Compression Index Analysis and Design Values— Birch**

Stratum	EM1110-1-1904 C_α/C_c Correlation (Reference 13)				Design Value
	Soil Type	Mid-Range C_α/C_c	Design C_c	Calculated C_α	
Silty Sand and Clayey Sand	—				
Silty Clay and Sandy Clay	Clay	0.055	0.24	0.013	**0.013**
Silty Sand and Clayey Sand	—				

4.7.7 Modulus of Elasticity (E_s)

A summary of the modulus of elasticity statistical analysis and design values for each coarse-grained stratum is included as **Table 36** and **Table 37**, where the basis of the design values is highlighted in bold. Analysis and selection of design values were conducted in accordance with the methods described in **Section 3.7.5**. The average from the SPT N-Value Correlation is the design basis for all strata. It should be noted that range of strain for estimates of E_s from in-situ tests (CPT and SPT) is on the order of 0.1-1%, resulting in a conservative estimate. Modulus values may need to be scaled to match the appropriate range of strain obtained from deformation analyses (**Reference 16**). Correlations for E_s using BV template (**Reference 15**) are presented as **Figure 21** and **Figure 22**.

Table 36 **Modulus of Elasticity Analysis, Data Comparison, and Design Values— Walnut**

Stratum	SPT N-Value Correlation			EM1110-1-1904 Typical E_s Values (Reference 13)			Design Value (ksf)
	Sample Size (n)	Avg. N_{60}	Avg. (ksf)	Soil Type	Lower Bound (ksf)	Upper Bound (ksf)	
Silty Sand and Clayey Sand	16	18		Silty Sand	500	4000	**380**
Silty Clay and Sandy Clay	7	29		—	—	—	—
Silty Sand and Clayey Sand	6	40		Silty Sand	500	4000	**770**
Silty Clay and Sandy Clay	2	43		—	—	—	—
Silty Sand and Clayey Sand	5	50		Silty Sand	500	4000	**940**

Table 37 **Modulus of Elasticity Analysis, Data Comparison, and Design Values— Birch**

Stratum	SPT N-Value Correlation			EM1110-1-1904 Typical E_s Values (Reference 13)			Design Value (ksf)
	Sample Size (n)	Avg. N_{60}	Avg. (ksf)	Soil Type	Lower Bound (ksf)	Upper Bound (ksf)	
Silty Sand and Clayey Sand	12	25		Silty Sand	500	4000	**510**
Silty Clay and Sandy Clay	4	33		—	—	—	—
Silty Sand and Clayey Sand	19	43		Silty Sand	500	4000	**820**

Young's Modulus for Sand/Gravel

1 kg/cm 2.0482 ksf
NC Normally Consolidated
OC Over Consolidated
pa Atmospheric pressure (14.7 psi, 2.116 ksf)

Profile: SCW Flood Control Dams

Inputs:

Layer No.		1			3			5		
N_{60}:		18			40			50		
CONSISTENCY		**Medium Dense**			**Dense**			**Very Dense**		
Equation	**Description**	Value (ksf)	Include in Average	Es (ksf)	Value (ksf)	Include in Average	Es (ksf)	Value (ksf)	Include in Average	Es (ksf)
Es/pa = 5*N_{60}	Sands with Fines	190	x	190	423	x	423	529	x	529
Es/pa = 10*N_{60}	Clean NC Sands	381			846			1058		
Es/pa = 15*N_{60}	Clean OC Sands	571			1270			1587		
Epmt/pa = 9.08*N^0.66	Japanese Sands DMT	129			219			254		
E = 196+7.9*N (in tsf), limit 1500 ksf (N=70)	NC Sand	676			1024			1182		
E = 416+10.9*N (in tsf), limit 1700 ksf (N=40)	Pre-loaded/OC Sand	1224			1700			1700		
E = 5 (N+15) (in tsf)	Submerged SP & SW Sands	330			550			650		
E = 3.3 (N+5) (in tsf)	Submerged SP Clayey Sands	152			297			363		
E = 4N (in tsf)	Silts, Sand Silts, Slightly Cohesive Silt-Sand Mix	144	x	144	320	x	320	400	x	400
E = 7N (in tsf)	Clean, Fine to Medium Sands & Slightly Silty Sands	252	x	252	560	x	560	700	x	700
E = 10N (in tsf)	Course Sand and Sands w\Little Gravel	360			800			1000		
E = 12N (in tsf)	Sandy Gravels and Gravel	432			960			1200		
E = 25N (in kg/cm^2)	Sands	922	x	922	2048	x	2048	2560	x	2560
E = 12(N+6) N<15 (in kg/cm^2)	Gravel (w/Sand)									
E = 40+12(N-6) N>15 (in kg/cm^2)	Gravel (w/Sand)	377			918			1163		
E = 10(N+6) N<15 (in kg/cm^2)	Sand (w/Gravel)									
E = 40+10(N-6) N>15 (in kg/cm^2)	Sand (w/Gravel)	328			778			983		
E = 7(N+6) N<15 (in kg/cm^2)	Coarse Sand									
E = 40+7(N-6) N>15 (in kg/cm^2)	Coarse Sand	254			569			713		
E = 4.5(N+6) N<15 (in kg/cm^2)	Medium Sand									
E = 40+4.5(N-6) N>15 (in kg/cm^2)	Medium Sand	193			395			487		
E = 3.5(N+6) N<15 (in kg/cm^2)	Fine Sand									
E = 40+3.5(N-6) N>15 (in kg/cm^2)	Fine Sand	168	x	168	326	x	326	397	x	397
E = 3(N+6) N<15 (in kg/cm^2)	Silt with Sand									
E = 40+3(N-6) N>15 (in kg/cm^2)	Silt with Sand	156			291			352		
E = 7*N^0.5 (in MPa)	Sand	620	x	620	925	x	925	1034	x	1034
AVERAGE MODULUS OF ELASTICITY (KSF)		**383**			**767**			**937**		
DESIGN MODULUS OF ELASTICITY (KSF) (can be overwritten)		**380**			**770**			**940**		

Figure 21. Elastic Settlement Parameters of Sandy Soils (Reference 15)— Walnut

Young's Modulus for Sand/Gravel

1 kg/cm 2.0482 ksf

NC Normally Consolidated
OC Over Consolidated
pa Atmospheric pressure (14.7 ps

Profile: SCW Flood Control Dams
Inputs:

Layer No.		1			3		
N_{60}:		25			43		
CONSISTENCY		**Medium Dense**			**Dense**		
Equation	**Description**	Value (ksf)	Include in Average	Es (ksf)	Value (ksf)	Include in Average	Es (ksf)
$Es/pa = 5*N_{60}$	Sands with Fines	265	x	265	455	x	455
$Es/pa = 10*N_{60}$	Clean NC Sands	529			910		
$Es/pa = 15*N_{60}$	Clean OC Sands	794			1365		
Epmt/pa = 9.08*N^0.66	Japanese Sands DMT	161			230		
E = 196+7.9*N (in tsf), limit 1500 ksf (N=70)	NC Sand	787			1071		
E = 416+10.9*N (in tsf), limit 1700 ksf (N=40)	Pre-loaded/OC Sand	1377			1700		
E = 5 (N+15) (in tsf)	Submerged SP & SW Sands	400			580		
E = 3.3 (N+5) (in tsf)	Submerged SP Clayey Sands	198			317		
E = 4N (in tsf)	Silts, Sand Silts, Slightly Cohesive Silt-Sand Mix	200	x	200	344	x	344
E = 7N (in tsf)	Clean, Fine to Medium Sands & Slightly Silty Sands	350	x	350	602	x	602
E = 10N (in tsf)	Course Sand and Sands w\Little Gravel	500			860		
E = 12N (in tsf)	Sandy Gravels and Gravel	600			1032		
E = 25N (in kg/cm^2)	Sands	1280	x	1280	2202	x	2202
E = 12(N+6) N<15 (in kg/cm^2)	Gravel (w/Sand)						
E = 40+12(N-6) N>15 (in kg/cm^2)	Gravel (w/Sand)	549			991		
E = 10(N+6) N<15 (in kg/cm^2)	Sand (w/Gravel)						
E = 40+10(N-6) N>15 (in kg/cm^2)	Sand (w/Gravel)	471			840		
E = 7(N+6) N<15 (in kg/cm^2)	Coarse Sand						
E = 40+7(N-6) N>15 (in kg/cm^2)	Coarse Sand	354			612		
E = 4.5(N+6) N<15 (in kg/cm^2)	Medium Sand						
E = 40+4.5(N-6) N>15 (in kg/cm^2)	Medium Sand	257			423		
E = 3.5(N+6) N<15 (in kg/cm^2)	Fine Sand						
E = 40+3.5(N-6) N>15 (in kg/cm^2)	Fine Sand	218	x	218	347	x	347
E = 3(N+6) N<15 (in kg/cm^2)	Silt with Sand						
E = 40+3(N-6) N>15 (in kg/cm^2)	Silt with Sand	199			309		
E = 7*N^0.5 (in MPa)	Sand	731	x	731	959	x	959
AVERAGE MODULUS OF ELASTICITY (KSF)		**507**			**818**		
DESIGN MODULUS OF ELASTICITY (KSF) (can be overwritten)		**510**			**820**		

Figure 22. **Elastic Settlement Parameters of Sandy Soils (Reference 15)— Birch**

4.8 Compacted Fill Properties

4.8.1 Embankment Fill

No test pit samples were collected from potential borrow sources as part of the preliminary field exploration program at the time this work was performed. Embankment fill properties for seepage analysis and slope stability analysis have been adopted from analysis of the field explorations and laboratory test data in this calculation package. It is assumed that embankment fill will be constructed from in situ materials or imported materials of similar properties to in situ materials.

Silty Clay and Sandy Clay stratum is hereafter referred to as Zone A and Silty Sand and Clayey Sand stratum is hereafter referred to as Zone B when used in embankment fill.

Total unit weight for embankment fill is adopted from **Table 11** and **Table 12**, and with guidance from **Reference 20** (**Attachment 5**). The design total unit weight for compacted fill is presented as **Table 38**.

Table 38. Total Unit Weight for Embankment Fill

Zone	Design Value (pcf)
A	**125**
B	**130**

Hydraulic conductivity for embankment fill is adopted from **Table 24** and is presented as **Table 39.** These values have been adopted for seepage analysis for this work. A sensitivity seepage analyses is recommended based on the range of design values.

Table 39. Hydraulic Conductivity for Embankment Fill

Zone	Design Value (cm/s), (ft/s)	Range of Design Value (cm/s)
A	**1×10^{-8}, (3×10^{-10})**	**1×10^{-7} to 1×10^{-9}**
B	**1×10^{-7}, (3×10^{-9})**	**1×10^{-6} to 1×10^{-8}**

Reported undrained strength from **Table 19** is adopted for embankment fill and is presented as **Table 40**. Drained strength parameters for embankment fill reported as **Table 40** is adopted from **Table 21**. The parameter *c′* was assumed to be 0 psf. For the R-Case, R-Envelope values for cohesion intercept (c_R) and friction angle (ϕ_R) presented as **Table 23** have been adopted for embankment fill and is presented as **Table 40**. These values have been adopted for stability analysis for this work.

Table 40 Selected Embankment Fill Design Values

Zone	Undrained Strength (psf)	Drained Strength		R-Envelope	
		c' (psf)	ϕ' (deg)	cR (psf)	ϕR (deg)
A	**720**	**0**	**21**	**240**	**14.6**
B	**1000**	**0**	**31**	**210**	**23.6**

4.8.2 Dispersive Soils

The dispersive characteristics of the subsurface soils were evaluated as part of the laboratory testing program. Test data sheets and test methods are provided in Appendix A of the DBM. Six (6) dispersion tests were performed using the Double Hydrometer test (ASTM D4221) and Crumb test (ASTM D6572). Results for the dispersion tests are presented as **Table 41** and **Table 42**. Crumb test results indicate dispersion potential for both Zone A (Silty Clay and Sandy Clay) and Zone B (Silty Sand and Clayey Sand) materials. Dispersive clay soils are easily eroded and carried away by waterflow under certain conditions. **Reference 21** provides engineering considerations on the use of dispersive soils in embankment fill.

Table 41. Double Hydrometer Test Results

Unit[1]	Dispersion (%)	Dispersive Classification	Remarks
3	24.46	**Non-dispersive**	Soils are classified as Zone B based on soil description
3	29.10	**Non-dispersive**	
3	8.27	**Non-dispersive**	

[1] Unit 3- Clayey Sand (SC). Tests were performed on the clayey portion of the samples.

Table 42. **Crumb Test Results**

Unit[1]	Dispersive Grade	Dispersive Classification	Remarks
2	2	**Intermediate**	Soils are classified as Zone A based on soil description
3	1	**Non-dispersive**	Soils are classified as Zone B based on soil description
7	3	**Dispersive**	

[1] Unit 2- Lean Clay; Unit 3- Clayey Sand (SC); Unit 7- Silty Clayey Sand.

4.8.3 Compaction Properties

As part of the laboratory testing program, standard Proctor compaction tests based on ASTM D 698 Method A standard were performed on composite samples remolded from soils samples which were used to perform other laboratory tests. A total of two tests were performed on composite samples comprising CL from borings B-1 and B-2 together and boring B-3. Results of compaction tests (i.e., the optimum moisture content and maximum dry unit weight) are summarized in **Table 43**.

Table 43. **Standard Proctor Results**

Composite Sample from	Soil Description	Maximum Dry Density (pcf)	Optimum moisture, %
B-1 and B-2	Sandy Lean Clay (CL)	107.2	16.7
B-3	Sandy Lean Clay (CL)	107.1	18.6

4.8.4 Filter, Riprap Rock, and Soil-bentonite cutoff (SBC)

The unit weights for filter and riprap materials have been selected based on records from past work. Table 9-5 of **Reference 22** manual presents reported dry densities and moisture contents from post-construction testing of backfill material from soil-bentonite slurry trench cutoff (SBC). The average dry density and moisture content of 112.9 pcf and 17 percent respectively were used to calculate a total unit weight of 132 pcf. However, given the typically low strength of SBC walls immediately after construction, a dry unit weight of 90 pcf was adopted for design and a saturated unit weight of 100 pcf was for assumed SBC wall.

The permeability of filter materials has been selected based on typical soil permeability from Table 6-3 of **Reference 12**, and the permeability of riprap has been selected based on past work/engineering judgement. The permeability of a completed SBC wall is reported as 10^{-7} cm/s (3.28×10^{-9} ft/s) for walls consisting of ≥ 1 percent bentonite (**Reference 22**). **Table 44** presents a summary of selected unit weight and permeability properties for riprap, filter and SBC wall.

Table 44. **Material Unit Weight and Hydraulic Properties**

Zone	Unit Weight, γ (pcf)		Saturated Hydraulic Conductivity, k_s ft/s (cm/s)
	Moist, γ_{moist}	Saturated, γ_{sat}	
SBC	90	100	$3{\times}10^{-9}$, (10^{-7})
Filter	120	130	$3{\times}10^{-5}$, (0.001)
Riprap	124	140	1, (30.48)

In the absence of strength test data for riprap rockfill and filter material, the shear strength of riprap and filter materials were assumed. Conventionally, riprap and filter materials are considered free-draining and hence the strength properties for the riprap and filter remain the same for both undrained and drained conditions. Strength values for the riprap was assumed based on a study by **Reference 2320** which recommends relationships for drained friction angle of rockfill as a function of confining stress. Based on the assumption of a general riprap material characteristics of loose to medium-dense rockfill at a confining pressure range equivalent to the average depth of maximum flood loading between 7 to 10 pounds per square inch (psi), drained friction angle based on **Reference 23** is between 44 to 47 deg. An assumed drained strength value of 40 degrees was adopted for stability evaluation for the riprap. A drained friction angle of 35 deg was assumed for the filter materials based on past work and engineering judgment.

For stability analysis, **Reference 22** recommends that a soil-bentonite slurry trench cutoff (SBC) should be considered to have zero shear strength and exert only a hydrostatic force to resist failure of an embankment.

Table 45 presents a summary of selected strength properties for riprap, filter and SBC wall.

Table 45. Material Strength Parameters

Zone	Undrained Strength (psf)	Drained Strength		R-Envelope	
		c' (psf)	ϕ' (deg)	cR (psf)	ϕR (deg)
SBC	**No strength**				
Filter	—	**0**	**36**	—	
Riprap	—	**0**	**40**	—	

Attachment 1:

Subsurface Profile

Client SJRA
Project SCW Flood Control Dams **Unit**
Project No. 411500 **File No.**
Title Evaluation of Project Soil Parameters

Computed By P. Turkson
Date 10/25/2024
Approved By David Bentler
Date 12/6/2024
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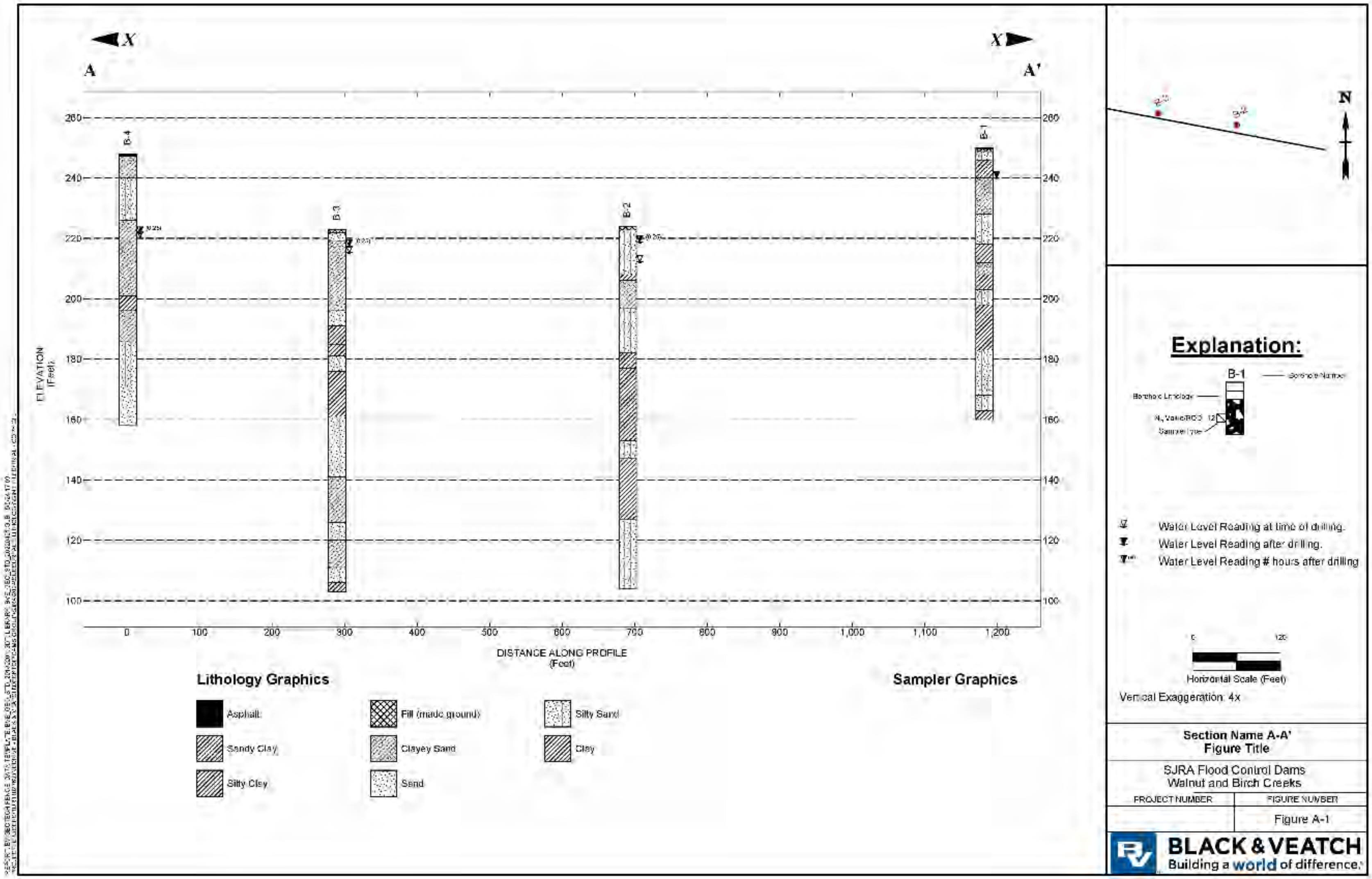

Attachment 2:

Interpreted Subsurface Profile from Boreholes

BLACK & VEATCH

Client SJRA
Project SCW Flood Control Dams **Unit**
Project No. 411500 **File No.**
Title Evaluation of Project Soil Parameters

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Approved By David Bentler
Date 12/6/2024
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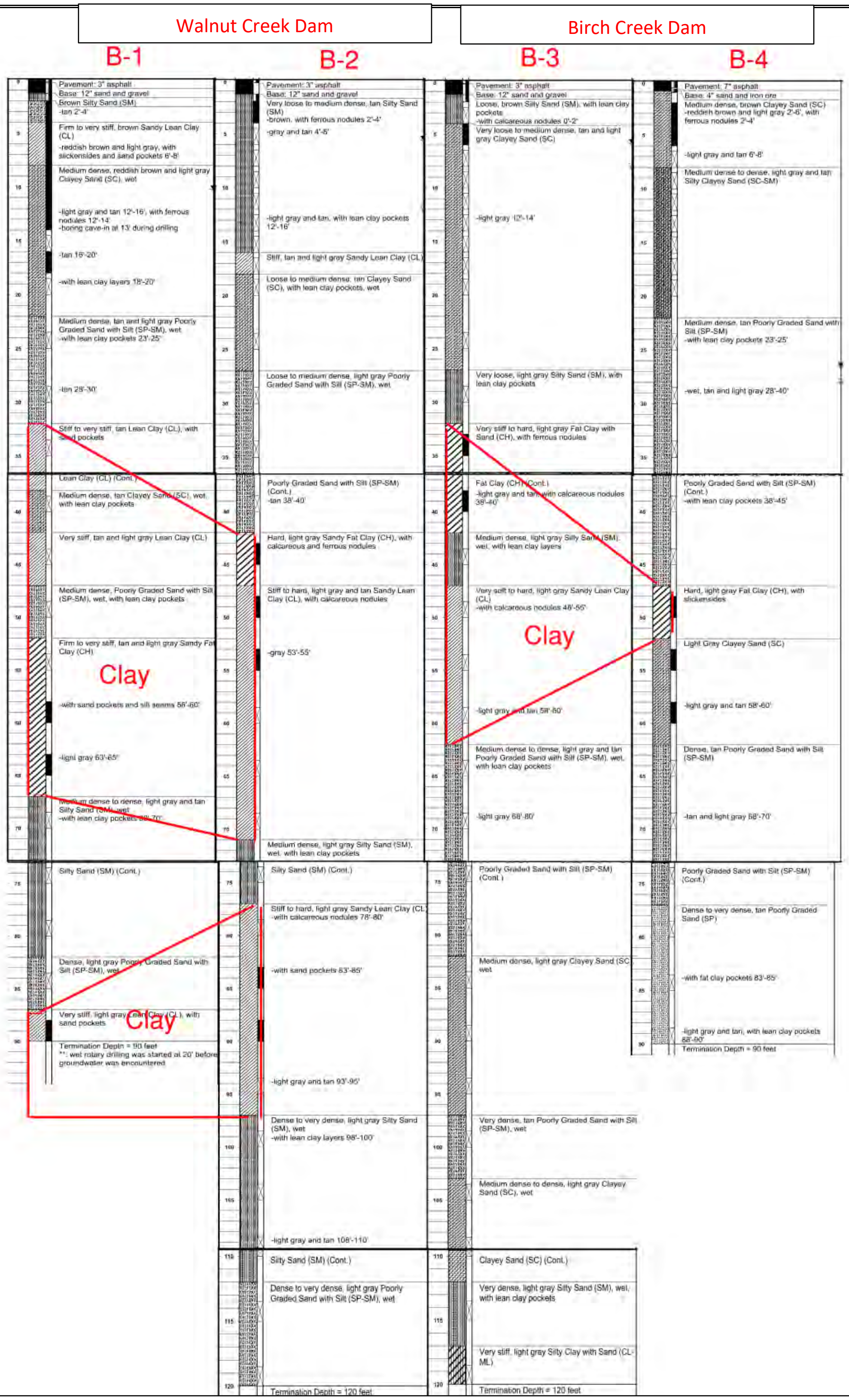


Note: The transition lines are based on general interpolation of the subsurface data and may be different at actual locations.

Attachment 3:

S-Case Selection of ϕ’ for Design Envelopes

G154-21 Boring B-1, 8'-10'

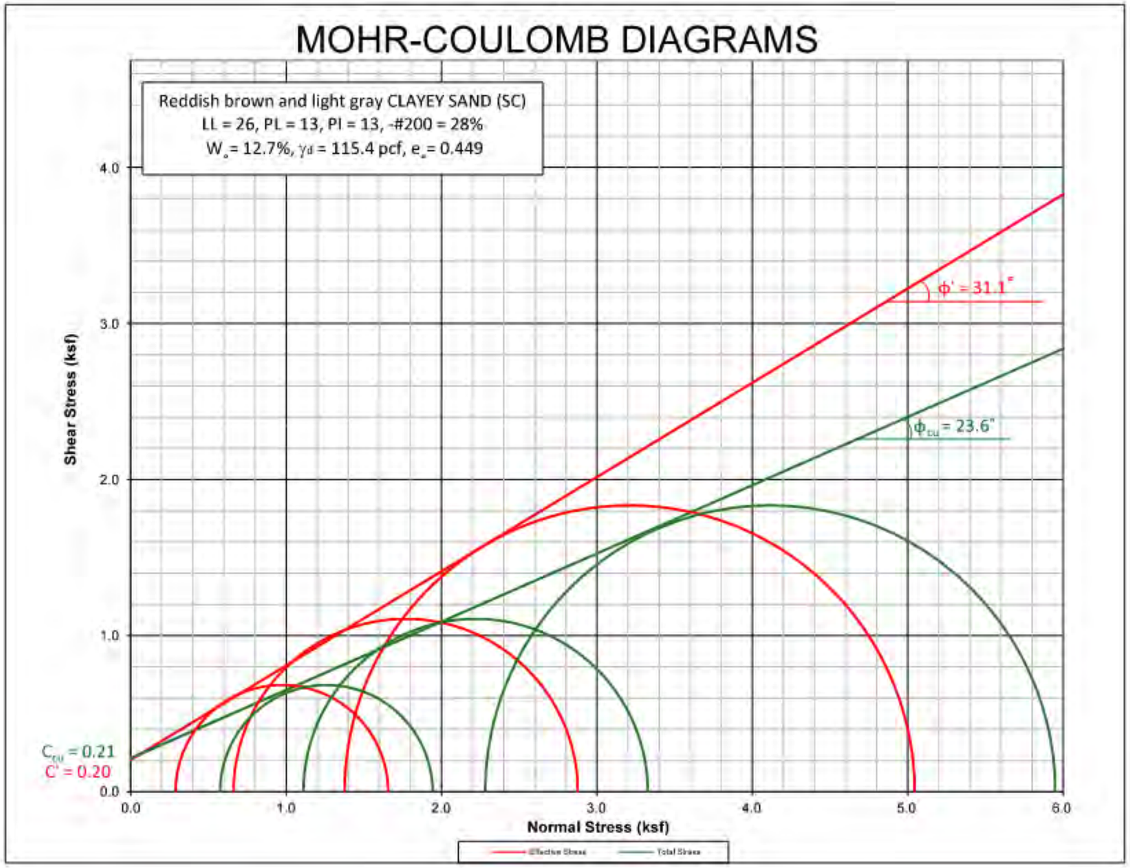
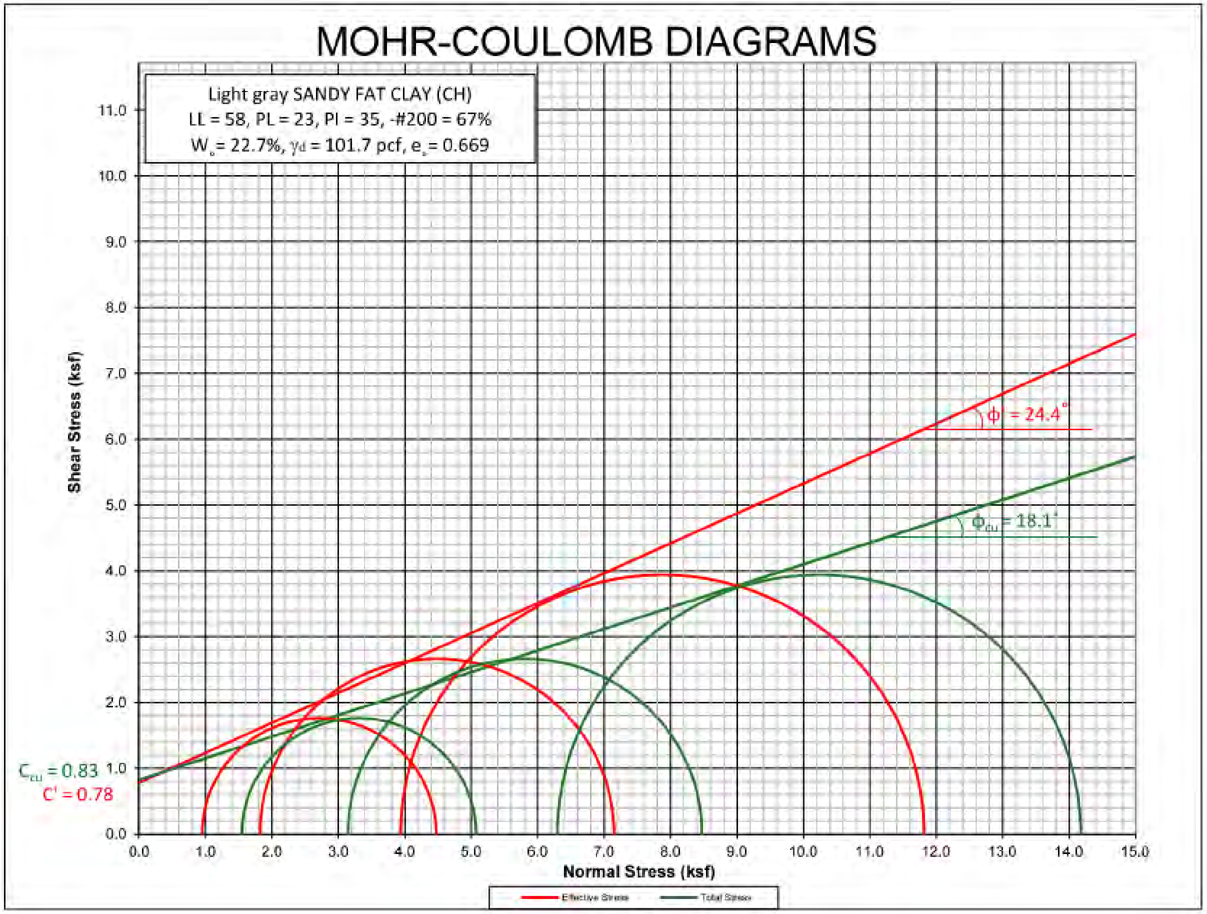


PLATE A-22

G154-21 Boring B-2, 43'-45'



G154-21 Boring B-3, 33'-35'

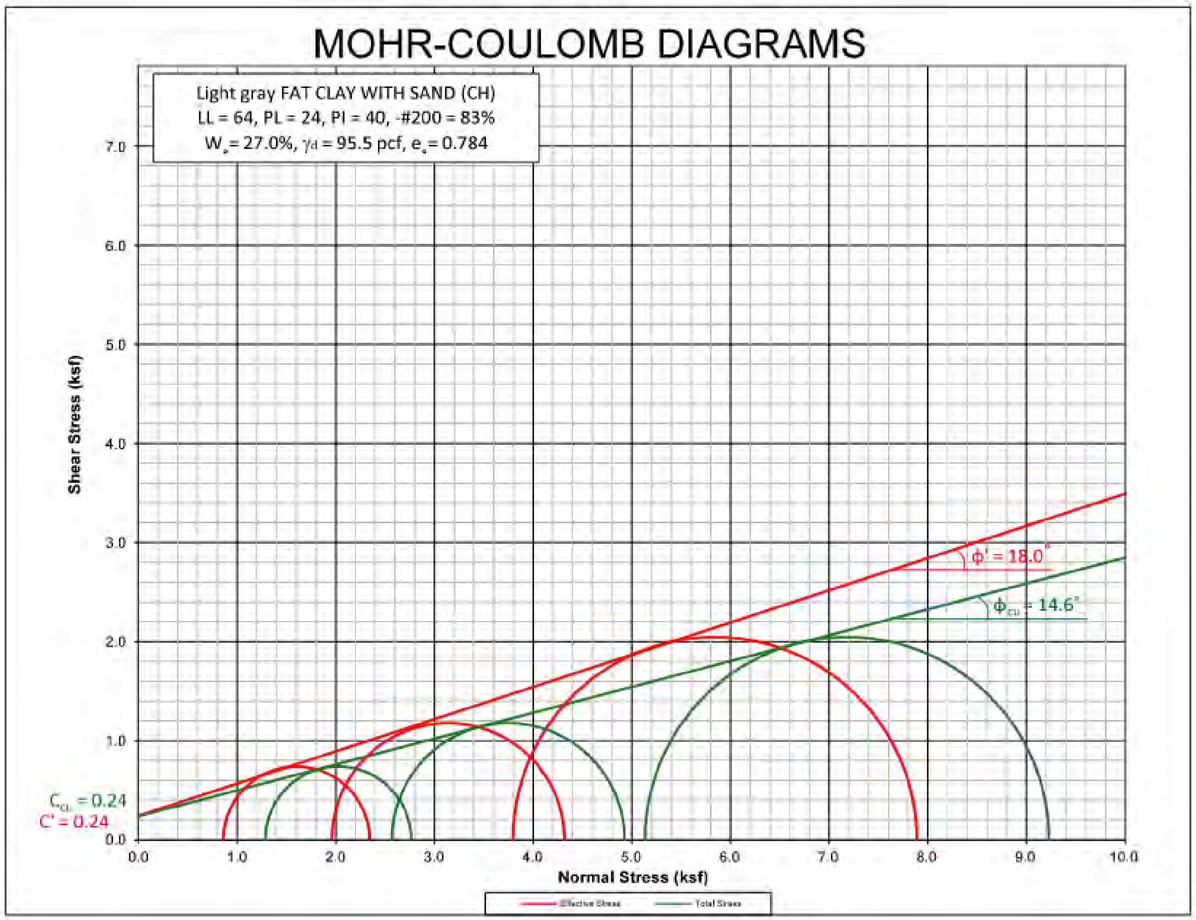
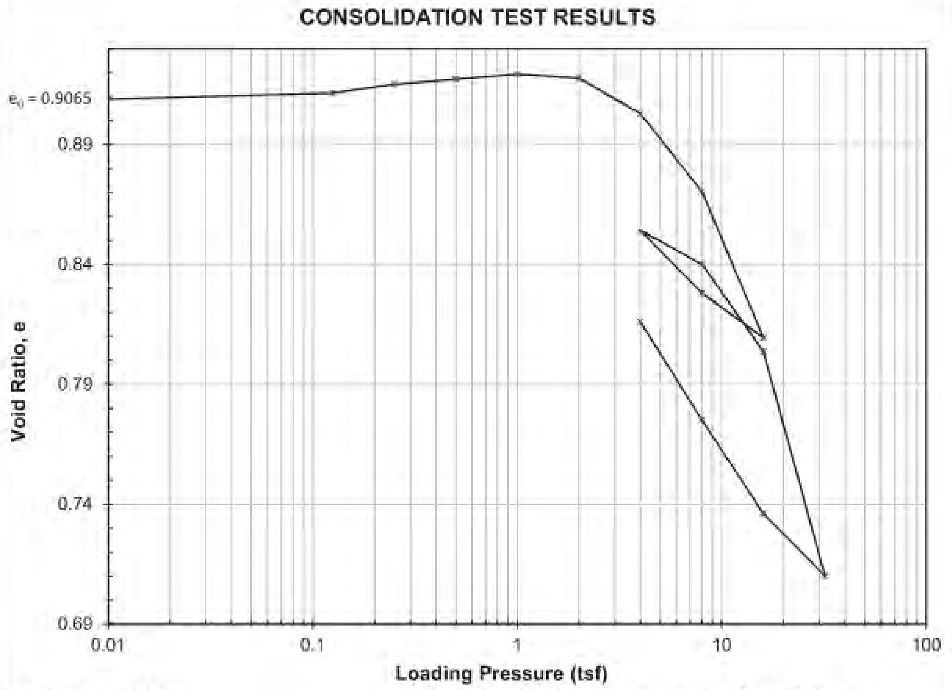


PLATE A-24

Attachment 4:

Casagrande Method for Determination of Consolidation Parameters



Project No.: G154-21
Sample ID: B-1, 58 to 60 feet
Sample Description: Tan and light gray Sandy Fat Clay (CH)
Estimated Consolidation Index (Cc): 0.3109
Estimated OCR: 2.9

Project: SJRA Spring Creek Watershed Feasibility Study
Dry Unit Weight (γ_d): 90.2 pcf
Estimated Recompression Index (Cr): 0.0769
Estimated Preconsolidation Pressure (Pc): 10.1 tsf

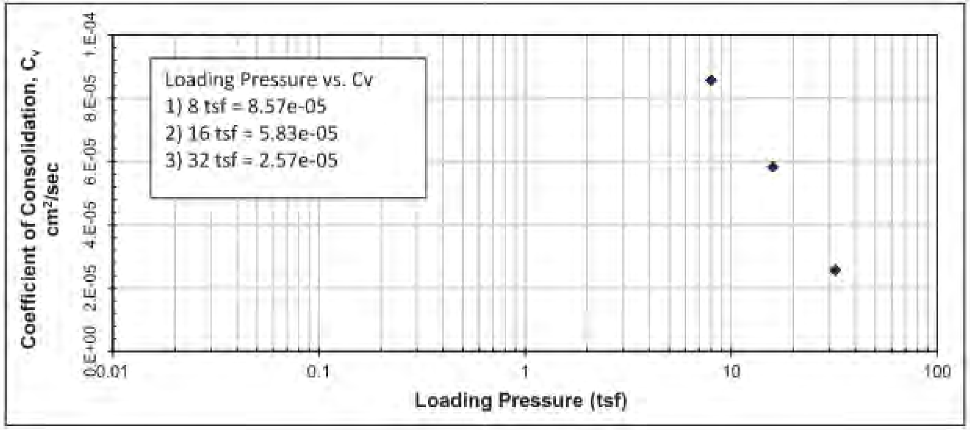
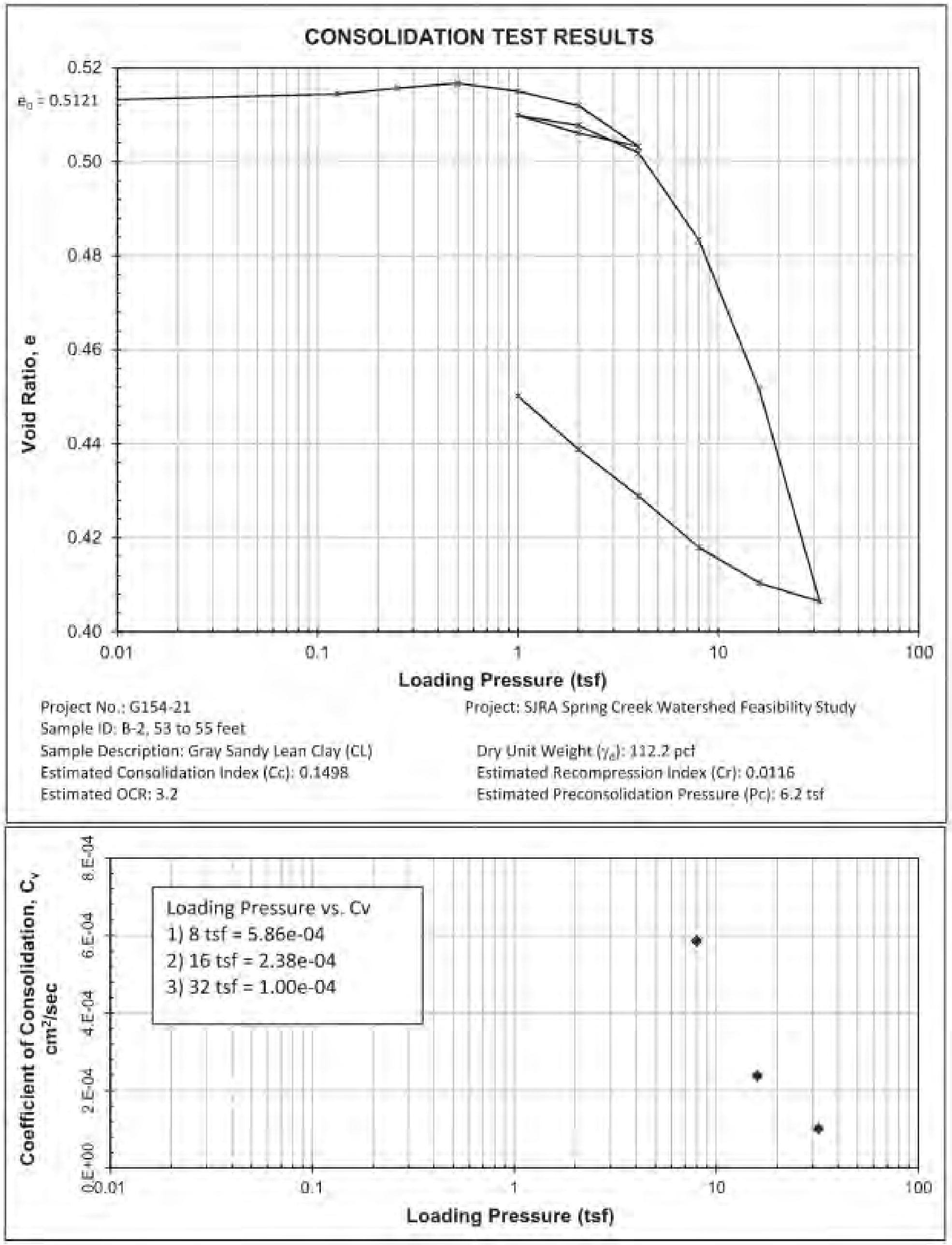


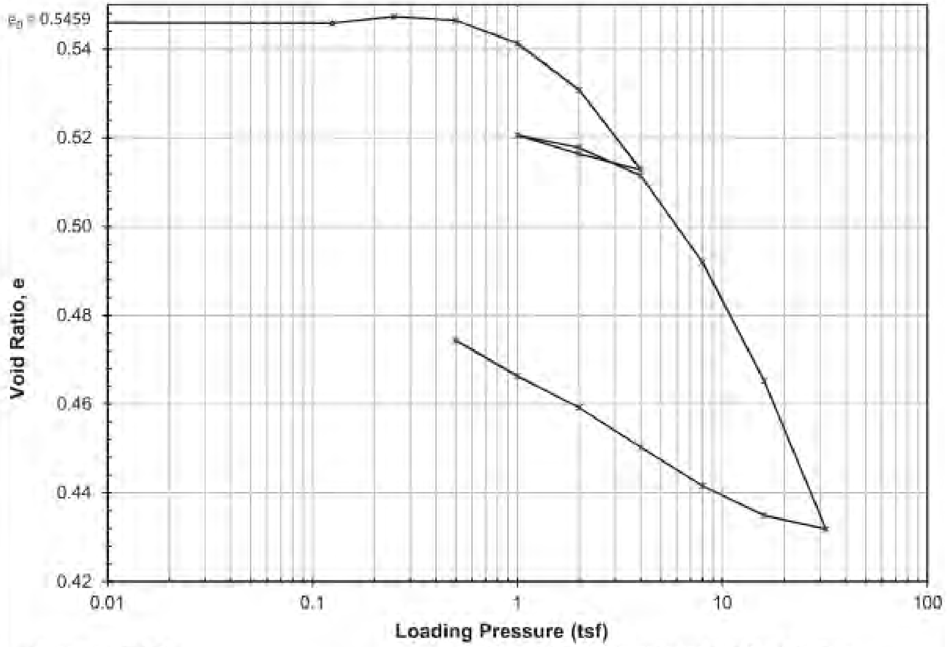
PLATE A-25



Project No.: G154-21
Sample ID: B-2, 53 to 55 feet
Sample Description: Gray Sandy Lean Clay (CL)
Estimated Consolidation Index (Cc): 0.1498
Estimated OCR: 3.2

Project: SJRA Spring Creek Watershed Feasibility Study
Dry Unit Weight (γ_d): 112.2 pcf
Estimated Recompression Index (Cr): 0.0116
Estimated Preconsolidation Pressure (Pc): 6.2 tsf

CONSOLIDATION TEST RESULTS



Project No.: G154-21
Sample ID: B-2, 83 to 85 feet
Sample Description: Light gray Sandy Lean Clay (CL)
Estimated Consolidation Index (Cc): 0.1110
Estimated OCR: 1.4

Project: SJRA Spring Creek Watershed Feasibility Study
Dry Unit Weight (γ_d): 108.5 pcf
Estimated Recompression Index (Cr): 0.0133
Estimated Preconsolidation Pressure (Pc): 3.8 tsf

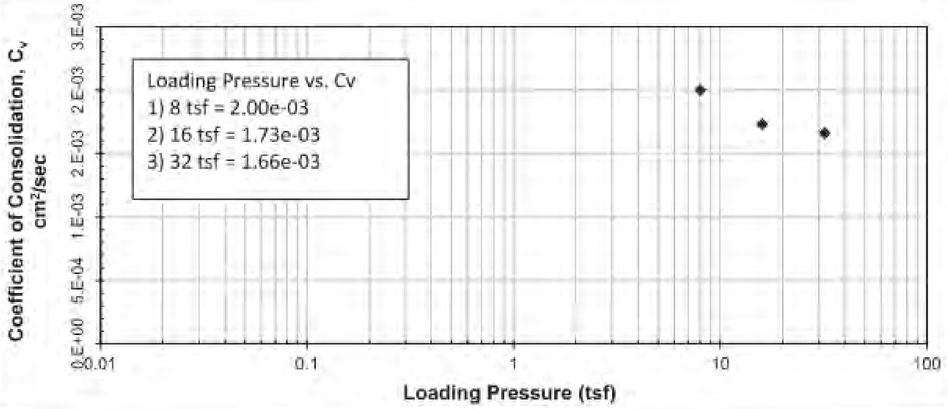


PLATE A-27

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Client SJRA
Project SCW Flood Control Dams **Unit**
Project No. 411500 **File No.**
Title Evaluation of Project Soil Parameters
Computed By P. Turkson
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Approved By David Bentler
Date 12/6/2024
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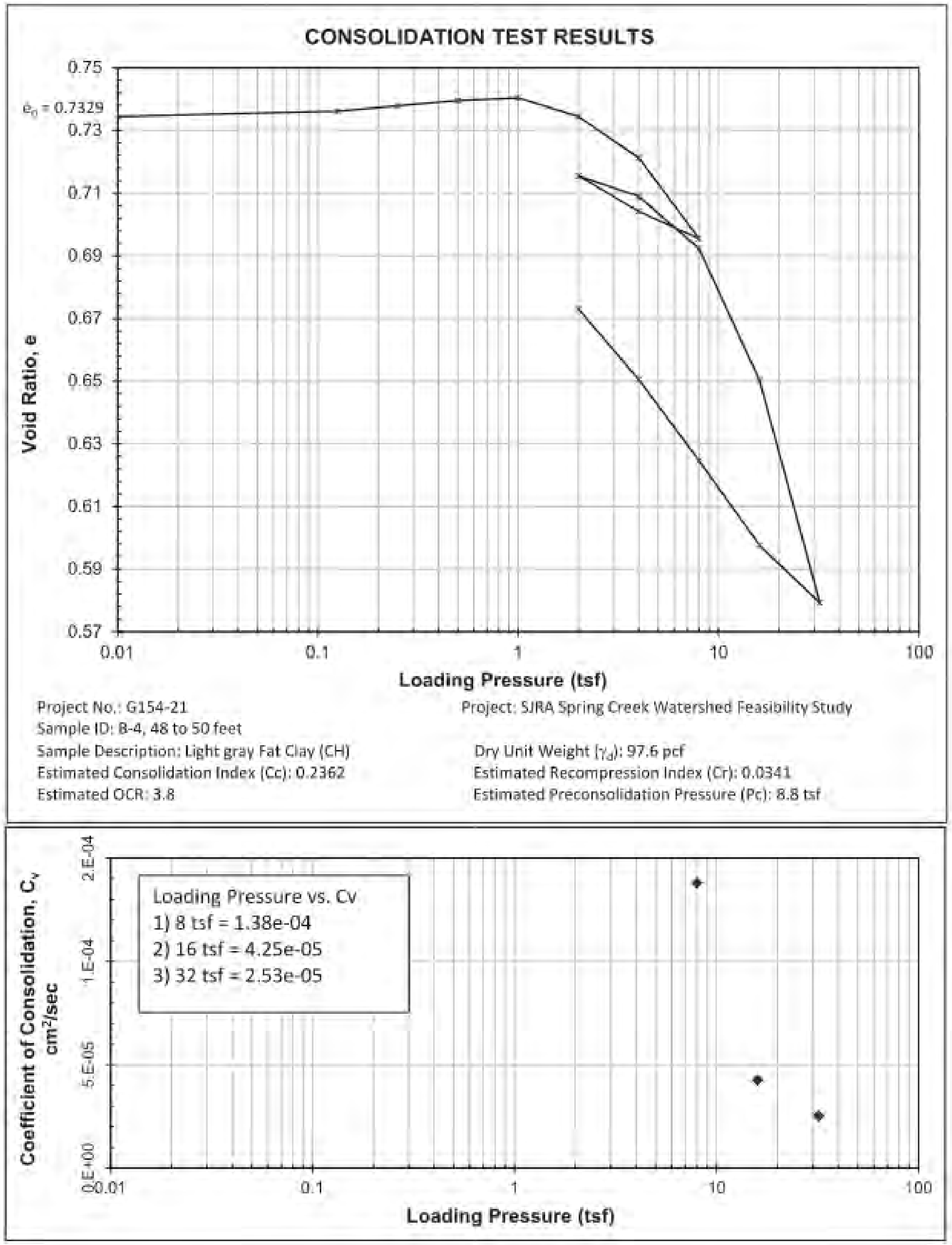


Project No.: G154-21
Sample ID: B-4, 48 to 50 feet
Sample Description: Light gray Fat Clay (CH)
Estimated Consolidation Index (Cc): 0.2362
Estimated OCR: 3.8

Project: SJRA Spring Creek Watershed Feasibility Study
Dry Unit Weight (γ_d): 97.6 pcf
Estimated Recompression Index (Cr): 0.0341
Estimated Preconsolidation Pressure (Pc): 8.8 tsf

Loading Pressure vs. Cv
1) 8 tsf = 1.38e-04
2) 16 tsf = 4.25e-05
3) 32 tsf = 2.53e-05

PLATE A-28

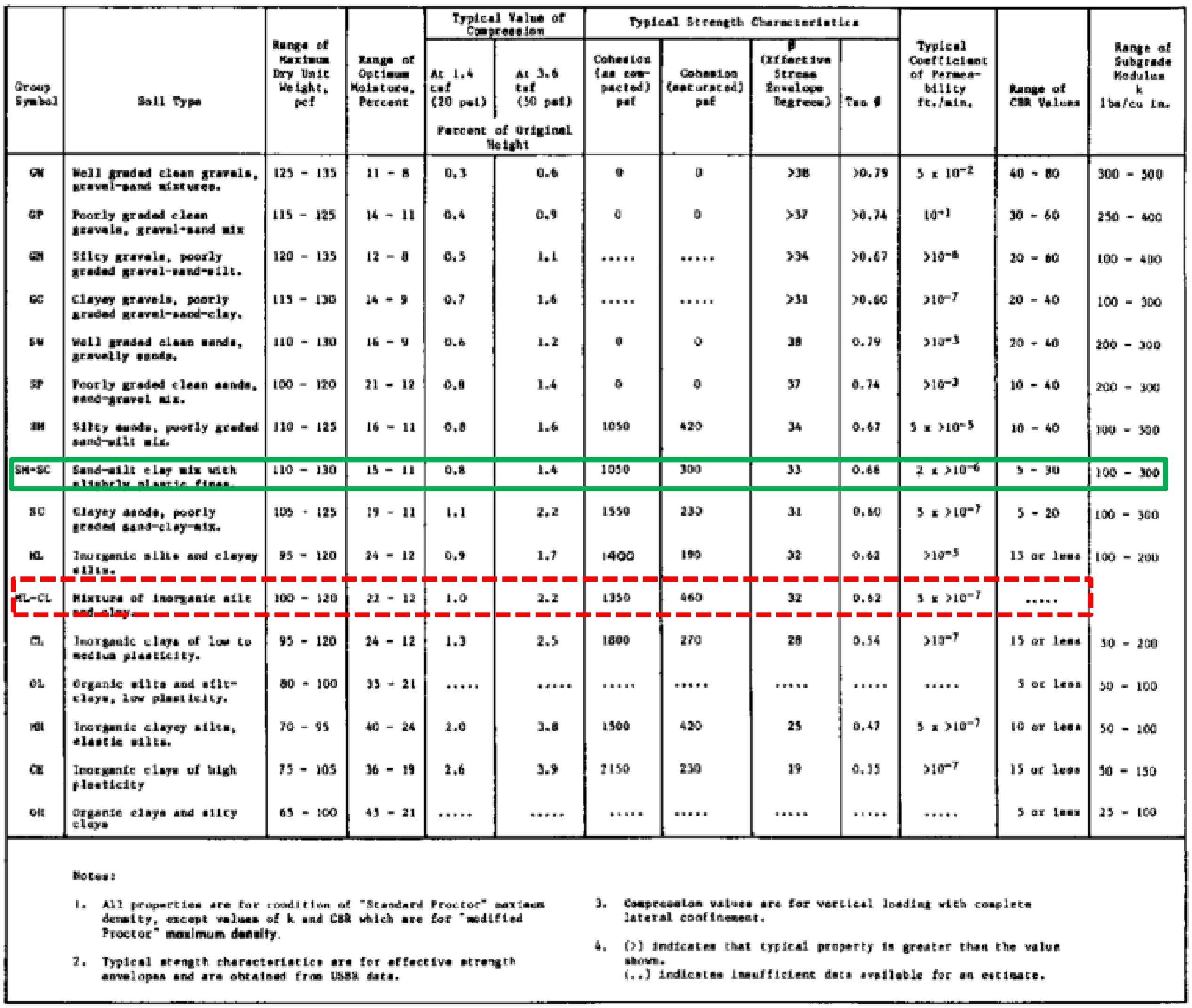
BLACK & VEATCH

Client SJRA		**Computed By**	P. Turkson
Project SCW Flood Control Dams	**Unit**	**Date**	10/25/2024
Project No. 411500	**File No.**	**Approved By**	David Bentler
Title Evaluation of Project Soil Parameters		**Date**	12/6/2024
		Page	67

Attachment 5:

Typical Compacted Fill Properties (Reference 20)

TABLE 1
Typical Properties of Compacted Soils



Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, Percent	Typical Value of Compression At 1.4 tsf (20 psi)	Typical Value of Compression At 3.6 tsf (50 psi)	Typical Strength Characteristics Cohesion (as compacted) psf	Cohesion (saturated) psf	φ (Effective Stress Envelope Degrees)	Tan φ	Typical Coefficient of Permeability ft./min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
				Percent of Original Height								
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	$5 \times >10^{-5}$	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	$2 \times >10^{-6}$	5 - 30	100 - 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	$5 \times >10^{-7}$	5 - 20	100 - 300
ML	Inorganic silts and clayey silts.	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay.	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	$5 \times >10^{-7}$	.....	
CL	Inorganic clays of low to medium plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	$5 \times >10^{-7}$	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	15 or less	50 - 150
OH	Organic clays and silty clays	65 - 100	45 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 - 100

Notes:

1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown.
(..) indicates insufficient data available for an estimate.

Zone A

Zone B

Appendix B-4 Seepage analysis calculation package

Confidential and Proprietary Business Information of Black & Veatch

Client Name SJRA Page 1 of 27

Project Name Spring Creek Watershed (SCW) Flood Control Dams Project No. 411500

Calculation Title Seepage Analysis

Verification Method: ☒ Check and Review ☐ Alternate Calculations

Objective: Evaluate seepage conditions for the Project

Unverified Assumptions Requiring Subsequent Verification

No.	Assumption	Verified By*	Date
1	Estimated soil properties and strength parameters based on information from the Aviles (2024) geotechnical investigations report, literature or past reports are deemed to be appropriate for this design effort.		
2	Long-term seepage conditions represent the most critical seepage conditions within the embankment and foundation.		
3	Selected sections for assessment of seepage rates and exit gradients represent the most critical sections for the overall stability of the embankment.		

Refer to Page _____ of this calculation for additional assumptions.

This Section Used for Software-Generated Calculations

BV Standard Application
Program Name/Version Seep/W , version 24.2.0.298 (GEOSLOPE International Ltd., 2024)

Review and Approval

Rev	Prepared By*	Date	Checked By*	Date	Approved By*	Date
0	P. Turkson, PhD, P.E.	10/29/2024	David Bentler			

*Signature required.

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Table of Contents

1.0 Objective

Evaluate conditions of seepage through Walnut Creek Dam and Birch Creek Dam and their foundation materials as part of the Spring Creek Watershed Flood Control Dams scope of work for SJRA. Factors of safety (FoS) against exit gradients associated with critical embankment and foundation sections, and discharge rates through the embankment body and its foundation have been determined from 2-dimensional finite element analysis using the computer program SEEP/W (version 10.0.2.18035) by GeoStudio.

2.0 References

1. Black & Veatch 2024. “Spring Creek Watershed (SCW) Flood Control Dams Material Calculation Record.” Report prepared for San Jacinto River Authority, dated October 2024.
2. Black & Veatch 2024. “Spring Creek Watershed Flood Control Dams Design Basis Memorandum.” Report prepared for San Jacinto River Authority, dated December 2024.
3. VandenBerge, D. R., Duncan, J. M., & Brandon, T. L. (2015). “Limitations of transient seepage analyses for calculating pore pressures during external water level changes.” JGGE, 141(5), 04015005. https://doi.org/10.1061/(asce)gt.1943-5606.0001283
4. van Genuchten, M. T. (1980). “A closed-form equation for predicting the hydraulic conductivity of unsaturated soils.” Soil Science Society of America Journal, 44(5), 892–898. https://doi.org/10.2136/sssaj1980.03615995004400050002x
5. Daniel, D. E. (1984). “Predicting hydraulic conductivity of clay liners.” JGE, 110(2), 285-300. https://doi.org/10.1061/(ASCE)0733-9410(1984)110:2(285)

3.0 Evaluation Basis

The following section provides the methodology for seepage analysis for Walnut Creek Dam and Birch Creek Dam (hereafter referred to as the Project). Based on the soil boring logs and laboratory testing data from the 2024 Aviles geotechnical investigations, the embankment and foundation zonation were developed, and the respective material properties were selected in **Reference 1**. The selected material properties are used for seepage analysis, and for the determination of factors of safety against exit gradients associated with critical embankment and foundation sections, and discharge rates through the embankment body and its foundation. Embankment geometry and zonation is based on the Design Basis Memorandum (DBM) (**Reference 2**). Three embankment alternative geometries from **Reference 2** have been evaluated for seepage behavior. Sensitivity analysis was performed to determine the influence of various embankment elements and soil properties on the behavior of seepage.

3.1 Design Water Surface Elevations

The basis of design water surface elevations is presented in the DBM based on Hydrologic and Hydraulic Calculations (**Reference 2**). The design water surface elevation for seepage analysis is presented as **Table 1.**

Table 1. Seepage Analysis Design Flood Elevations

Design Condition	**Flood Pool Elevation (ft-msl)**	
	Walnut Creek Dam	Birch Creek Dam
Seepage analysis	261.6	257.1

3.2 General Material Properties and Seepage Control

The general material properties for the embankment and foundation zonation have been adopted from **Reference 1**. The embankment and foundation zonation for seepage control for the three alternatives is presented as **Figure 1**, **Figure 2** and **Figure 3**.

Alternative 1 comprises a 20-foot-deep cutoff trench followed by a sheet pile wall which is keyed a minimum 2 feet into the impervious foundation stratum. Additionally, a vertical chimney drain and horizontal blanket drain is provided to collect seepage through the embankment and foundation and to channel collected seepage water into a ditch which will be located on the downstream toe of the embankment.

In addition to similar seepage control measures to Alternative 1, Alternative 2 comprises an impervious clay core and a filter drain aligned with the core to collect embankment through-seepage.

Alternative 3 comprises a soil-bentonite cutoff (SBC) wall which extends a minimum 6 feet above foundation level and keyed a minimum 2 feet into the impervious foundation stratum. A vertical chimney drain and horizontal blanket drain is provided to collect seepage through the embankment and foundation and to channel collected seepage water into a ditch at the downstream toe of the embankment.

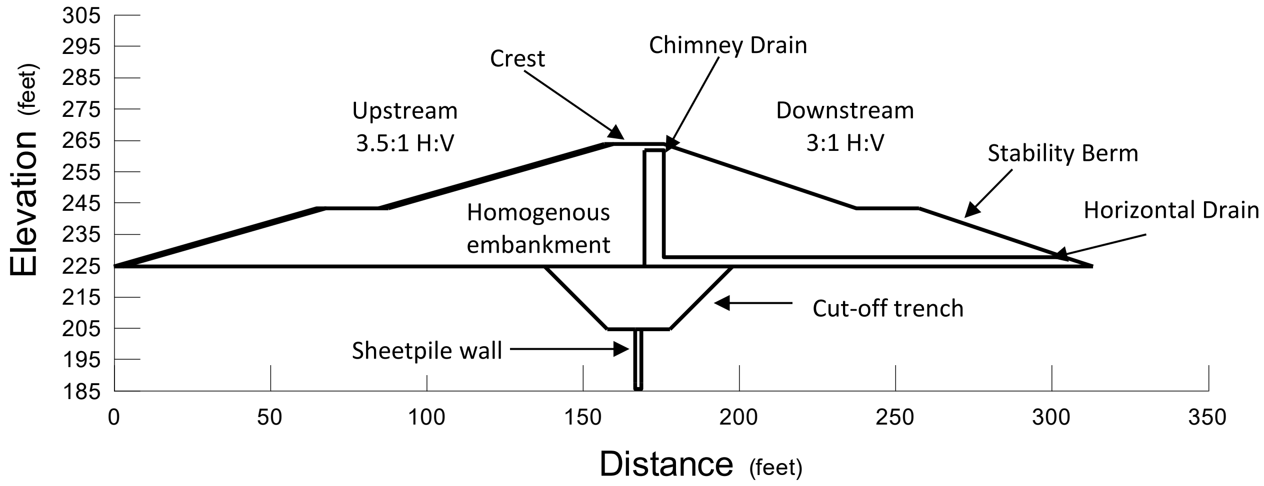

Figure 1. **Alternative 1 Embankment Geometry**

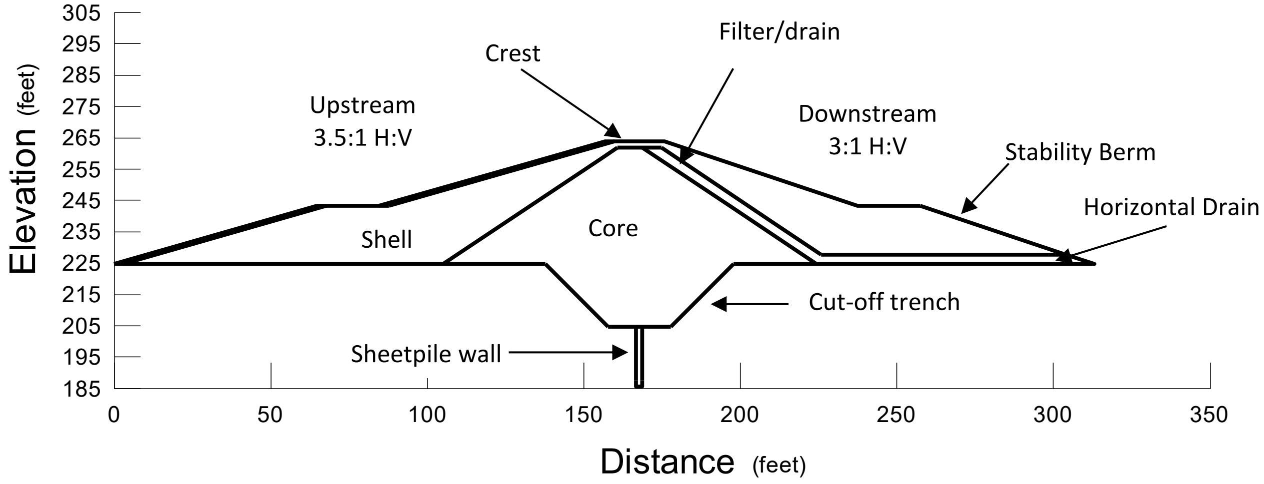

Figure 2. **Alternative 2 Embankment Geometry**

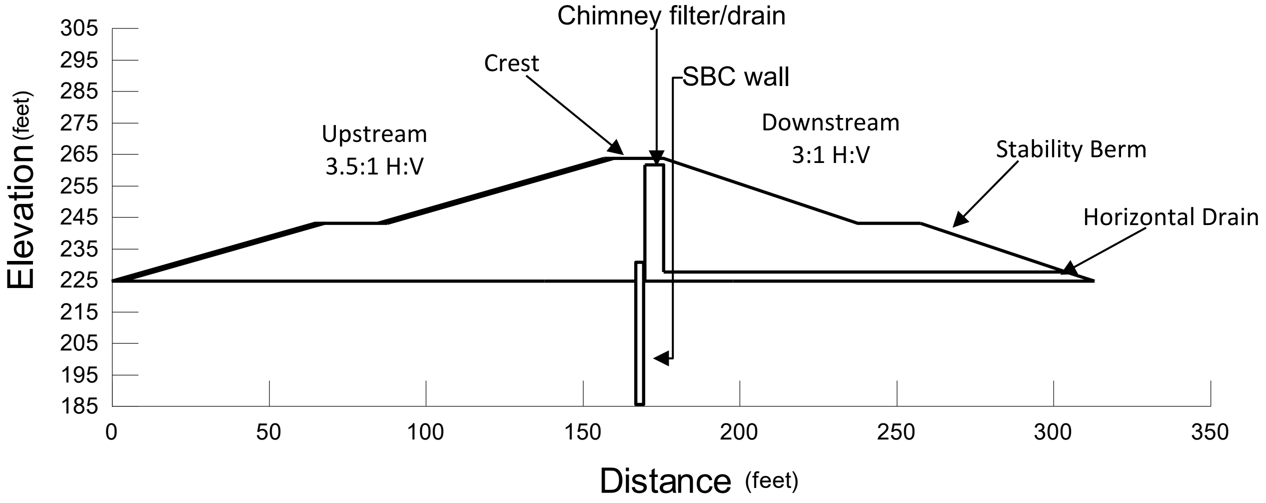

Figure 3. **Alternative 3 Embankment Geometry**

3.3 Hydraulic Properties

The hydraulic properties of the soil required for seepage analysis include the saturated permeability and the unsaturated hydraulic functions of the soil. The unsaturated hydraulic functions include soil-water characteristic curves (SWCCs) and hydraulic conductivity functions (HCFs). The SWCCs for the different soils were estimated using the sample functions method option in the seepage analyses software SEEP/W, and the corresponding HCFs were estimated based on the SWCC using the Van Genuchten method option in SEEP/W. The SWCC for the different embankment zones was estimated based on soil classifications and typical SWCCs in SEEP/W as summarized in **Table 2**. The foundation soils were assumed to be saturated for the seepage analysis. The soil parameters selected for the seepage analyses are summarized in **Table 3**.

In the seepage analysis to establish long-term phreatic surface, volumetric compressibility (m_v) values have been selected such that coefficients of consolidation (c_v) values calculated from m_v, and saturated permeability (k_s) will fall within typical range of values presented in **Reference 3** for the soil or material type and the anticipated function.

The values of field permeability tests are generally 10 to 1000 times higher (**Reference 5**) than would be expected from laboratory tests on either undisturbed or recompacted samples based on the soil types encountered from the geotechnical exploration. A sensitivity seepage analysis to determine seepage through the embankment alternatives has been performed as part of this preliminary design based on the range of values presented in **Table 4**. During detailed design, an updated sensitivity analysis using permeability values obtained from site-specific permeability tests is recommended.

Table 2. SEEP/W Material Sample for SWCC

Zone	SEEP/W Material Sample
A	Silty clay
B	Silty sand
SBC	Clay
Filter	Sand
Riprap	Gravel
Foundation- silty and sandy clays	—
Foundation- silty and clayey sands	—

Table 3. Design Soil Parameters for Seepage Analyses

Zone	Saturated Permeability, k_s ft/s (cm/s)	Anisotropy, kv:kh	Compressibility[1] (1/psf)	Volumetric Water Content[2]
A	3×10^{-10}, (1×10^{-8})	0.11	9×10^{-7}	0.3
B	3×10^{-9}, (1×10^{-7})	0.25	9×10^{-6}	0.3
SBC	3.28×10^{-9}, (10^{-7})	0.11	2×10^{-6}	0.4
Filter	3×10^{-5}, (0.001)	0.25	4×10^{-4}	0.35
Riprap	1, (30.48)	1	4×10^{-9}	0.4
Foundation- silty and sandy clays	3×10^{-10}, $(1\times10^{-8}))$	0.5	5×10^{-8}	0.3
Foundation- silty and clayey sands	3×10^{-9}, (1×10^{-7})	0.5	5×10^{-7}	0.3
Sheetpile wall[3]	N/A			

[1] Compressibility values from typical compressibility from **Reference 3**.

[2] Estimated volumetric water content from **Reference 4**.

[3] Impervious sheetpile wall assigned a no-flow boundary condition.

Table 4. Range of Soil Permeability for Seepage Analysis

Zone	Range of Design Values, ft/s (cm/s)		Compressibility[1] (1/psf)	
	Lower	Upper	Lower	Upper
A	3×10^{-11}, (1×10^{-9})	3×10^{-8}, (1×10^{-6})	9×10^{-8}	9×10^{-5}
B	3×10^{-10}, (1×10^{-8})	3×10^{-7}, (1×10^{-5})	9×10^{-7}	9×10^{-4}
Foundation- silty and sandy clays	3×10^{-11}, $(1\times10^{-9}))$	3×10^{-8}, $(1\times10^{-6}))$	5×10^{-8}	5×10^{-6}
Foundation- silty and clayey sands	3×10^{-10}, (1×10^{-8})	3×10^{-7}, (1×10^{-5})	5×10^{-7}	5×10^{-6}

[1] Compressibility values from typical compressibility from **Reference 3**.

4.0 Seepage Analysis Methodology

The following section describes the methods used for seepage analysis.

4.1.1 Steady-State versus Transient Analysis

Seepage through an embankment dam can be analyzed under steady-state and transient flow conditions. Steady-state seepage represents the long-term operating condition of an impoundment dam. Flow conditions through the embankment body and foundation materials are assumed to be steady (i.e., unchanging). However, for flood protection embankments which typically experience short duration storm surge or flooding, a transient seepage analysis is more appropriate to characterize flow through the dam. In a transient seepage model, both the initial and future hydraulic boundary conditions are specified to determine the response of embankment materials to the change in boundary conditions.

Steady-state analysis was used to evaluate seepage in this preliminary design because prediction of the required pore pressures under changing external boundary loads using transient analysis tends to be complex and inaccurate for many of the commercial seepage analysis computer programs, which makes it difficult to obtain valid results from subsequent effective stress stability analyses of the embankment slopes. Also, uncertainties associated with the future boundary conditions (i.e., flood events) and material properties make the use of a steady-state approach more pragmatic.

Use of steady-state analysis in this preliminary design implies that estimates of seepage quantity are therefore expected to be conservative, i.e. too high, and steady-state analysis provides conservative basis for assessing stability (higher pore pressures).

4.1.2 2-D Seepage Model Set-up

Specified analysis convergence options used for seepage analysis are summarized in **Table 5**. In some cases, the model was simplified by removing the upstream riprap, the significant digits for maximum pressure head difference were increased from two to three, and 0.65 and 0.01 were selected for the initial rate and minimum rate respectively to achieve convergence for some mesh nodes. Illustration of the model geometry with material zones and boundary conditions as well as mesh discretization is shown as **Figure 4**, **Figure 5** and **Figure 6**.

Table 5. Convergence Options Used in Seepage Analyses

Convergence Parameter	SEEP/W Default Value	Specified Value^1
Maximum iterations	500	—
Maximum pressure head difference	0.005 feet	Significant digits were increased from two to three
Maximum number of reviews	10	—
Initial rate	1	0.65
Minimum rate	0.1	0.01
Rate reduction factor	0.65	—
Reduction frequency	10	—
1. Specified values were applied to Alternative 2 models using upper range permeability values from **Table 4**.		

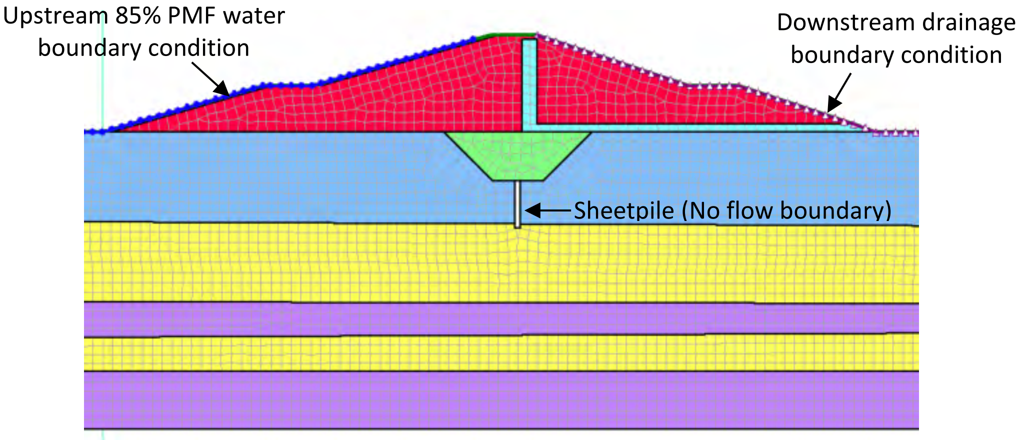

Figure 4. **Alternative 1 Model**

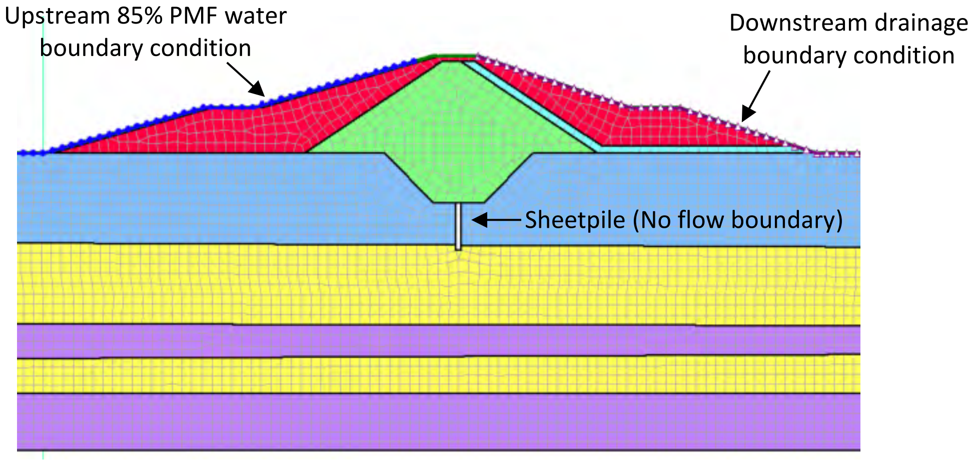

Figure 5. **Alternative 2 Model**

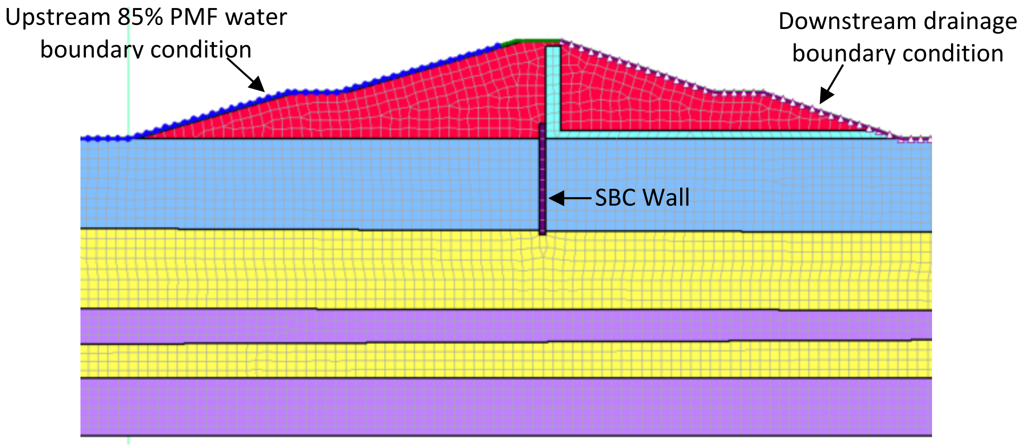

Figure 6. **Alternative 3 Model**

4.1.3 Sensitivity Analysis

To determine the influence of the finite element model length (extents upstream and downstream of dam), anisotropic ratio of the foundation soils, foundation stratigraphy extent, permeability, and depth of SBC wall, five sensitivity analyses were performed on Alternative 1 and Alternative 3 of the selected preliminary design geometries for Walnut Creek Dam. Sensitivity to model length, anisotropic ratio of the foundation soils and thickness of subsurface soils were evaluated for the Alternative 1 model to determine the extent of model and foundation parameters above which there is insignificant change in seepage results. For the purposes of comparison and a baseline estimate, Alternative 1 was modeled without the foundation cut-off trench and sheet pile in the sensitivity analysis exercise. Sensitivity of permeability and depth of SBC wall was performed on Alternative 3 to determine SBC wall parameters above which there is insignificant change in seepage results. These sensitivity analyses were undertaken to examine how the results would vary if the technical assumptions that form the basis of preliminary design were modified to represent upper and lower bounds of credible limits of these factors. The sensitivity analyses will also inform future investigations related to final design and construction planning. Baseline assumptions were made regarding the relevant parameters for the sensitivity analysis as summarized in **Table 6**.

Table 6. Baseline Assumptions for Sensitivity Analysis

Sensitivity Analysis Parameter	Baseline Assumption
Model foundation horizontal extent	360 feet away from both upstream and downstream toes
Anisotropic ratio of foundation soils	$k_V/k_H= 0.5$, $(k_H/k_V= 2)$
Total thickness of subsurface soil	120 feet below bottom of dam
Permeability of SBC wall	$k_s= 3.28\times10^{-9}$ ft/s
Depth of SBC wall below bottom of dam	20 feet below bottom of dam

Each sensitivity analysis was evaluated for steady state seepage conditions at peak design flood elevation of 261.6 feet and based on design soil parameters from **Table 3**. The sensitivity analyses were performed as follows:

i. **Length of the model**: The extent of both upstream and downstream sections of the finite element seepage model may influence seepage results and lead to inaccuracies due to boundary effects. Studies have shown that finite element seepage analysis models with both upstream and downstream extents reaching a minimum of three times the model foundation thickness is sufficient to effectively minimize boundary effects on seepage results. The minimum required length of the seepage models beyond the embankment was determined by extending the SEEP/W models for the preliminary design section to lengths at both the upstream and downstream sides beyond the centerline of the embankment. Both the upstream and downstream boundaries was modeled as three times the model foundation thickness and then varied for five distances beyond the embankment centerline by adding to both sides of the model extents: 250, 500, 1,000, 2,000, and 3,000 feet. The sensitivity analysis results demonstrated that total flow through the embankment do not vary when the model length is extended beyond the baseline length. However, the exit gradient at the downstream toe of the embankment varied slightly when the baseline length of model was extended beyond 2000 feet. The sensitivity analysis results are presented as **Attachment 1**.

ii. **Anisotropic ratio of the foundation soils**: The preliminary design anisotropy (k_V/k_H) of the foundation soil strata underlying the embankment may vary for each soil type. The influence of varying the anisotropy of the foundation soils was evaluated by changing the anisotropy of the foundation soil strata to values of 0.1, 0.25, 1 and 2.5 within the preliminary design section and re-running the seepage models. The results of the seepage analysis were used to determine whether selected preliminary design anisotropy values of the foundation soil strata are conservative. The sensitivity analysis results for the baseline geometry indicated that the total flow through the embankment increases with anisotropy (k_V/k_H), i.e. when vertical permeability increases. Horizontal permeability is generally higher than vertical permeability for stratified deposits. Considering that the geologic formation exposed in the area of the proposed dams comprise the Willis Formation created by the deposition of sediments, the design k-ratio values for the various foundation soil strata are considered reasonable for the preliminary design. The sensitivity analysis results presented in **Attachment 1** show that k_V/k_H values selected for design can influence predicted discharge rates and the required drainage capacities. Hence, seepage models should be updated with site-specific k_V/k_H values from permeability tests during detailed design.

iii. **Thickness of subsurface soils**: Preliminary subsurface investigations may provide limited data on subsurface conditions and may not show depth of bedrock below embankment foundation. The thickness of subsurface soil on top of bedrock may vary at different locations within the preliminary embankment section. Hence, the effect of model bottom extension on seepage flow and exit gradient was evaluated by varying the bottom of the final layer at the depth of borehole termination in increments of 20 feet up to a cumulative total increment of 100 feet. Results of the sensitivity analyses are summarized in **Attachment 1**. The results indicated that increasing the thickness of silty and clayey sands stratum underlying the silty and sandy clay stratum generally resulted in increased quantity of total flow through the embankment foundation. The percent increase in total flow was reduced as the bottom of the silty and clayey sands stratum was lowered to 60 feet downward extension, beyond which the magnitude of percent increase in flow fluctuated. The exit gradient at the downstream toe of the embankment increased with incremental extension of subsurface soil depth up to 40 feet, beyond which the exit gradient does not vary substantially with further subsurface soil depth extension. Based on the sensitivity analysis results, the effective bottom of silty and clayey sands layer of 200 feet below ground surface (bgs) was selected for SEEP/W analyses.

iv. **Permeability of Soil-Bentonite Cutoff Wall**: Selected permeability for the soil-bentonite cutoff wall was varied by an order of magnitude up to four orders of magnitude in the selected design section to determine the influence of permeability of cutoff wall on seepage flow within the embankment section. The sensitivity analysis results indicated that the total flow through the embankment decreases substantially with decrease in SBC wall permeability up to two orders magnitude. Based on the results of the sensitivity analyses, the design permeability value for the SBC wall was selected as 3.28×10^{-11} ft/s. The sensitivity analysis results are presented in **Attachment 1**.

v. **Depth of Soil-Bentonite Cutoff Wall**: Due to the anticipated subsurface geology of relatively porous silty and sandy materials to be encountered within the foundation soil underlying the dam, the depth of the cutoff wall was varied in the design section to determine the influence of depth of cutoff wall on seepage flow and hydraulic gradients within the embankment section. Seepage analyses were completed for 7 conditions: cutoff wall depth from embankment base ranging from 20 feet to 50 feet in 5-foot increments. The sensitivity analysis results presented in **Attachment 1** indicated no change in total seepage flow through the dam with increasing SBC wall depth. Similarly, the exit gradient at the downstream toe of the embankment do not vary when the SBC wall is extended beyond the baseline model depth. Based on the sensitivity analysis results, the design depth of the cutoff along the earth embankment alignment is considered sufficient.

4.1.4 Boundary Conditions

The SEEP/W models require input regarding surface water levels and flow boundaries within the embankment fill and subsurface materials. Each alternative geometry was evaluated for steady state seepage conditions for peak design flood which equates to elevation 261.6 feet for Walnut Creek Dam and 257.1 feet for Birch Creek Dam. Based on the sensitivity analysis results, all seepage analyses were modeled to a minimum additional distance of 500 feet upstream and downstream of the baseline embankment length. Within the foundation soils, the models were extended to a bottom of 200 feet, below which no flow was assumed to occur. The SBC wall along the earth embankment alignment for Alternative

3 was anchored 2 feet into impervious strata below the dam and the SBC wall permeability was assigned 3×10^{-11} feet/second (ft/s).

5.0 Seepage Analysis Results

This section presents seepage analyses results that focus on exit gradients and discharge rates through critical sections of the embankment. Factor of safety against exit gradient (FS_{exit}) was calculated based on the equation:

$$FS_{exit} = \frac{i_c}{i_e}$$

where i_c is the critical gradient and i_e is the exit gradient. The critical gradient is given by the following equation:

$$i_c = \frac{\gamma_t - \gamma_w}{\gamma_w}$$

where γ_t is the total unit weight of soil and γ_w is the unit of weight of water, which is equal to 62.4 pcf.

The critical exit gradient occurs when very high pore pressures exist, resulting in an effective stress of soil equal to zero. This condition allows for upward flow conditions and potential erosion piping. The downstream foundation section is assumed to be the most critical section for exit gradient evaluation, hence a depth of up to 10 feet from the top of the downstream foundation and up to 20 feet away from the downstream toe of the embankment was evaluated for exit gradient. An average vertical gradient was calculated from the node locations at the downstream section of the foundation up to 10 feet from the top of foundation and up to 20 feet away from the downstream toe of the embankment. The assumed saturated unit weight of 135 pcf was used to calculate the critical gradient for the foundation material as 1.16. The calculated average vertical hydraulic gradient is summarized in **Table 7** for each alternative geometry for both Walnut Creek and Birch Creek Dams. A minimum factor of safety (FoS) of 4 was selected as the criteria to check against soil movement because of the exit gradient. The basis of selection of FoS criteria is provided in **Section 3.5.10** of the DBM. The calculated factors of safety against exit gradient are summarized in

Table 8.

Table 7. Average Vertical Hydraulic Gradient

Dam	Permeability	Average Vertical Hydraulic Gradient		
		Alternative 1	**Alternative 2**	**Alternative 3**
Walnut Creek	Lower	0.02	0.03	0.02
	Design	0.03	0.04	0.02
	Upper	0.09	0.05	0.09
Birch Creek	Lower	0.04	0.04	0.04
	Design	0.05	0.05	0.04
	Upper	0.10	0.07	0.10

Table 8. Factor of Safety Against Exit Gradient

Dam	Permeability	Factor of Safety Against Exit Gradient		
		Alternative 1	Alternative 2	Alternative 3
Walnut Creek	Lower	54	44	55
	Design	46	28	50
	Upper	13	22	14
Birch Creek	Lower	29	27	29
	Design	25	26	28
	Upper	11	17	11

Discharge was determined generally at two locations: (i) combined flow through the dam body and foundation, and (ii) flow through filter. The predicted flows are summarized in **Table 9**. Sections where total discharge rates through the embankment body and foundation as well through the blanket drain were obtained are shown on **Attachment 2**.

Table 9. Predicted Flow from Seepage Analysis

Dam	Alternative	Discharge (ft^3/day/ft)					
		Combined Flow Through Dam and Foundation			Flow Through Filter		
		Lower k_s	Design k_s	Upper k_s	Lower k_s	Design k_s	Upper k_s
Walnut Creek	1	0.0006	0.006	0.50	0.0005	0.005	0.40
	2	0.0002	0.002	0.19	0.0001	0.001	0.11
	3	0.0006	0.005	0.48	0.0006	0.005	0.38
Birch Creek	1	0.0006	0.006	0.56	0.0004	0.004	0.36
	2	0.0003	0.003	0.28	0.0001	0.001	0.11
	3	0.0007	0.006	0.55	0.0006	0.005	0.35

6.0 Seepage Analysis Conclusions and Recommendations

The steady-state seepage analyses were completed using inputs, modeling methods, and assumptions previously described in this calculation package. The calculated factor of safety against exit gradient for all alternatives is more than the minimum factors of safety included in the DBM and are acceptable for preliminary design. Comparison of the factors of safety to design criteria shows that the considered embankment alternatives are acceptable against seepage behavior. The estimated combined discharge from the embankment body and foundation is required to determine the seepage capacity of the selected filter material and seepage collection systems for an advanced design. The following recommendations for analysis during detailed/final design are provided for consideration:

- Because the selected soil parameters are based on soil borings outside of the Project footprint, a robust field exploration and soil testing program at the dam sites is recommended to verify and validate selected soil design parameters used in this analysis for an advanced design.
- Following site-specific investigations and verification of preliminary design assumptions, it is recommended that the seepage models be validated after the model uncertainties are reduced.
- Also, considering the primary function of the Project is establishing dry detention dams, considerations for modifications to the foundation treatment methods proposed for the three alternatives with the potential to reduce construction cost are provided in Section 7.0 of the DBM. These modifications should be evaluated using project site specific field exploration and soil testing results.

Attachment 1:

Sensitivity Analysis Summary Results

A1-1. Effect of Model Horizontal Extension on Seepage Flow and Exit Gradient

Model Extent	Alternative 1			
	Combined Flow Through Dam and Foundation (ft^3/day/ft)	Percentage Change of the Flow	Flow Through Filter (ft^3/day/ft)	Exit Gradient
Baseline	0.0088	—	0.0083	0.088
250	0.0088	0%	0.0083	0.088
500	0.0088	0%	0.0083	0.088
1000	0.0088	0%	0.0083	0.088
2000	0.0087	-1%	0.0080	0.087
3000	0.0087	0%	0.0080	0.087

A1-2. Effect of Anisotropic Ratio (k-ratio) of Foundation Soils

Anisotropic Ratio (k-ratio) of Foundation Soils	Combined Flow Through Dam and Foundation (ft^3/day/ft)	Percentage Decrease or Increase of the Flow[1]	Flow Through Filter (ft^3/day/ft)
K_V/k_H=0.5 (Design Values)	0.0078	—	0.0074
k_V/k_H=0.1, (k_H/k_V= 10)	0.0068	-12%	0.0063
K_V/k_H=0.25, (k_H/k_V= 4)	0.0074	8%	0.0069
K_V/k_H=1, (k_H/k_V= 1)	0.0083	13%	0.0079
K_V/k_H=2.5, (k_H/k_V= 0.4)	0.0089	6%	0.0086

1. Negative value denotes decrease of flow and positive value denotes increase of flow.

A1-3. Effect of Model Bottom Extension on Seepage Flow and Exit Gradient

Model Bottom Extent (feet)	Alternative 1			
	Combined Flow Through Dam and Foundation (ft^3/day/ft)	Percentage Increase of the Flow	Flow Through Filter (ft^3/day/ft)	Exit Gradient
Baseline	0.0088	—	0.0083	0.087
-20	0.0091	2%	0.008	0.089
-40	0.0093	3%	0.0084	0.091
-60	0.0095	2%	0.0084	0.092
-80	0.0097	1%	0.0084	0.093
-100	0.0099	2%	0.0084	0.094

A1-4. Effect of Permeability of SBC Wall

Permeability (k) of SBC Wall, ft/s	Combined Flow Through Dam and Foundation (ft^3/day/ft)	Percentage Decrease of the Flow
Design Value 3.28×10^{-9}	0.0036	—
3.28×10^{-10}	0.0031	-14%
3.28×10^{-11}	0.0027	-13%
3.28×10^{-12}	0.0027	0%
3.28×10^{-13}	0.0027	0%

A1-5. Effect of Soil-Bentonite Cut Off Wall Depth on Seepage Flow

SBC Wall Bottom Elevation Extended by (feet)	Alternative 3		
	Combined Flow Through Dam and Foundation (ft^3/day/ft)	Percentage Change in Total Flow	Exit Gradient
Baseline	0.0036	—	0.091
20	0.0036	0%	0.091
25	0.0036	0%	0.091
30	0.0036	0%	0.091
35	0.0036	0%	0.091
40	0.0036	0%	0.091
45	0.0036	0%	0.091
50	0.0036	0%	0.091

Client SJRA
Project SCW Flood Control Dams **Unit**
Project No. 411500 **File No.**
Title SCW Flood Control Dams Seepage Analysis
Computed By P. Turkson
Date 10/29/2024
Approved By
Date
Page 20

Attachment 2:

Seepage Analyses Results Sections for Design Permeability

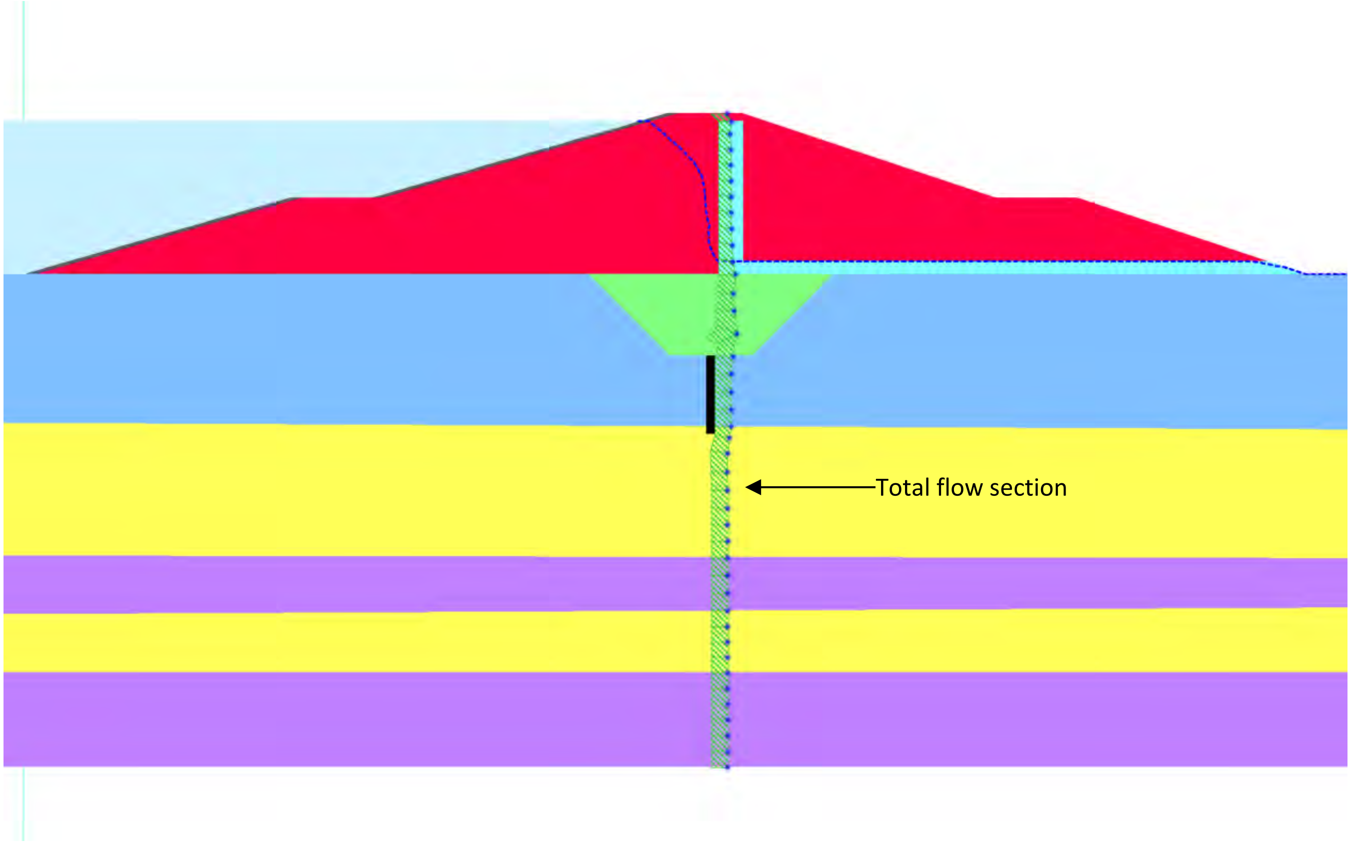

Figure 7. **Walnut Creek Alternative 1 Combined Flow Through Dam and Foundation= 0.006 ft^3/day/ft**

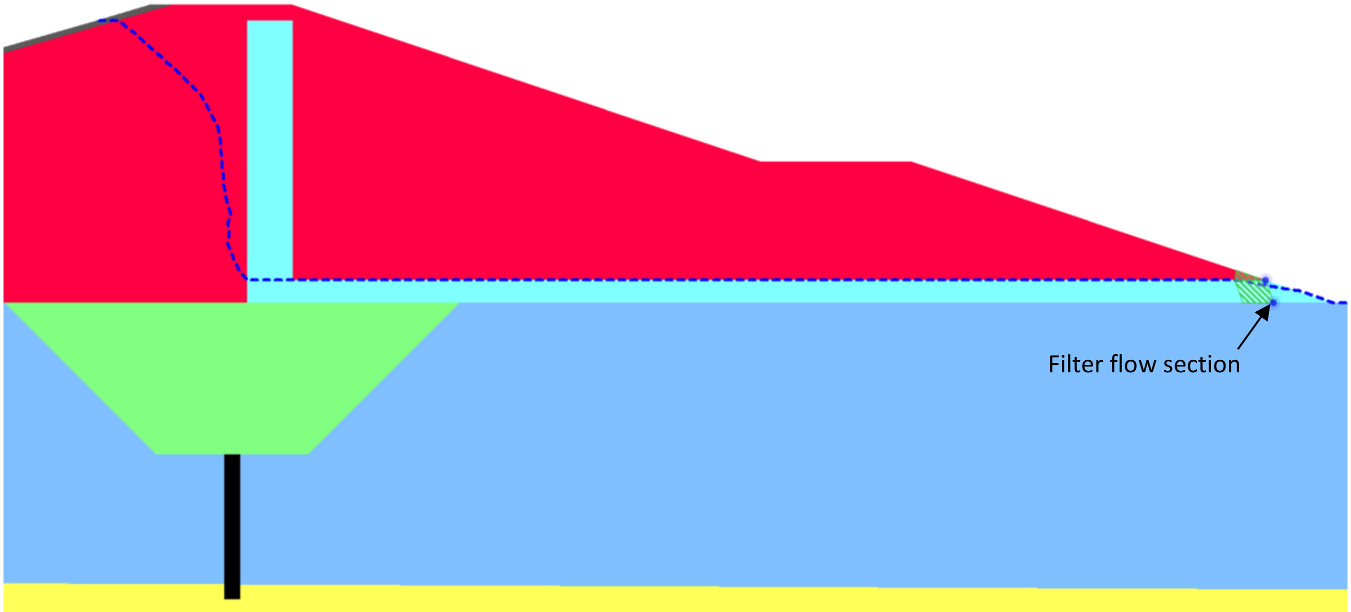

Figure 8. **Walnut Creek Alternative 1 Flow Through Filter = 0.005 ft^3/day/ft**

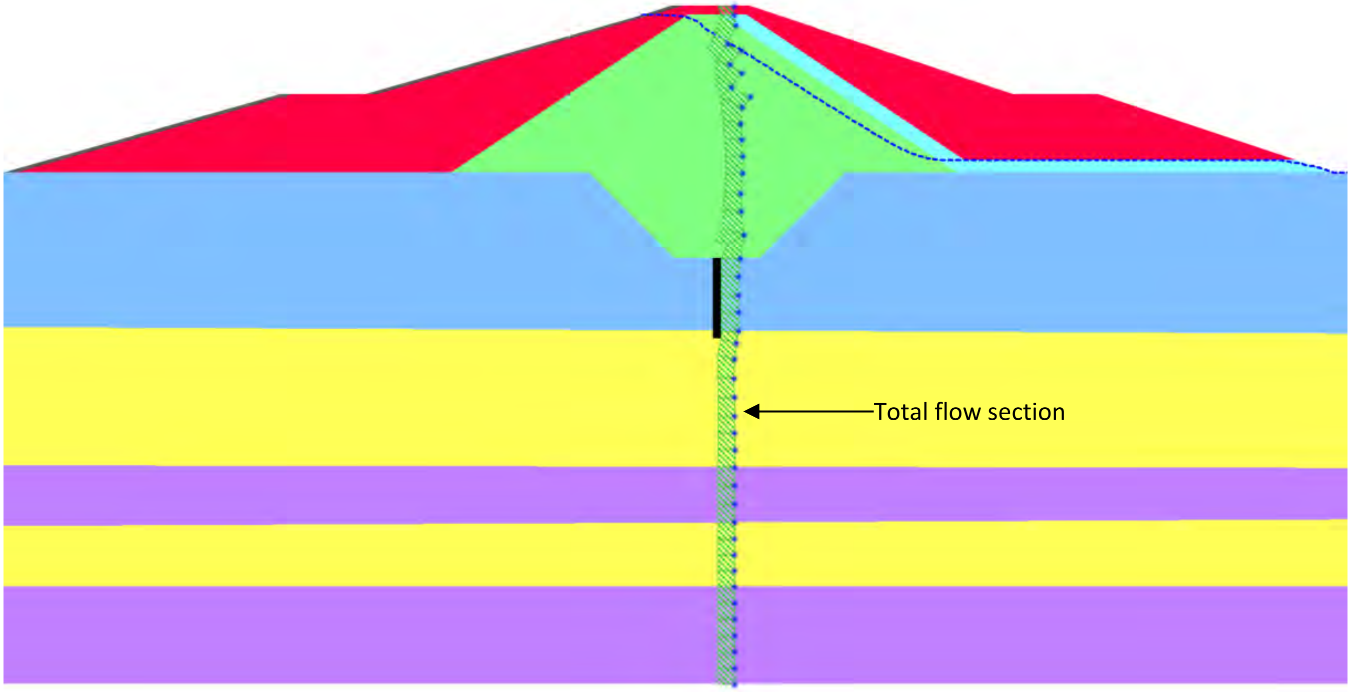

Figure 9. **Walnut Creek Alternative 2 Combined Flow Through Dam and Foundation = 0.002 ft^3/day/ft**

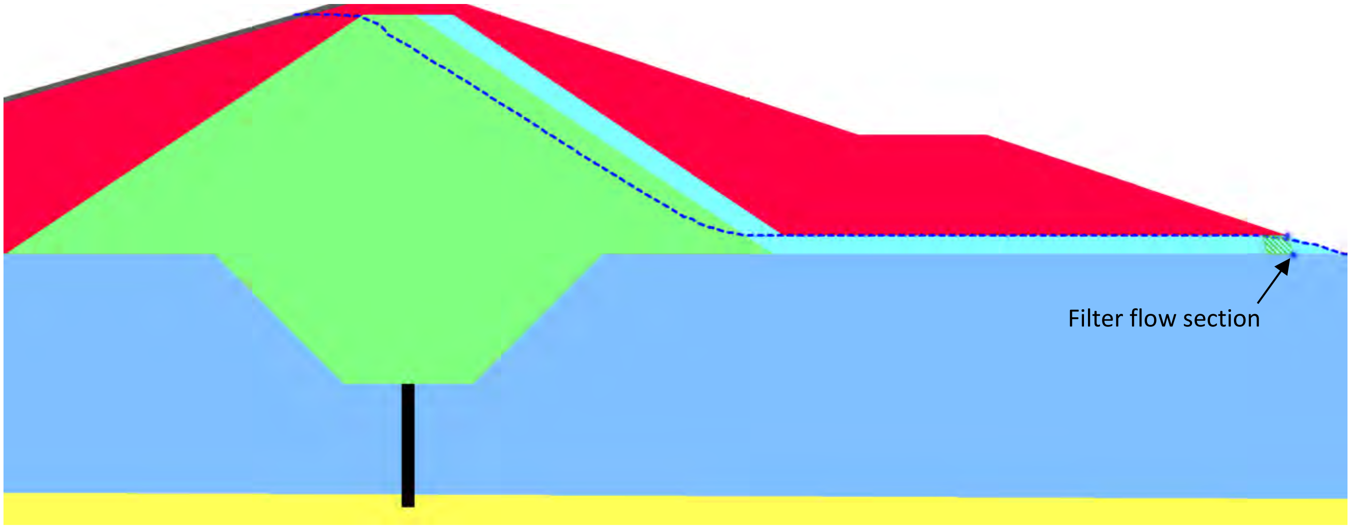

Figure 10 **Walnut Creek Alternative 2 Flow Through Filter = 0.001 ft^3/day/ft**

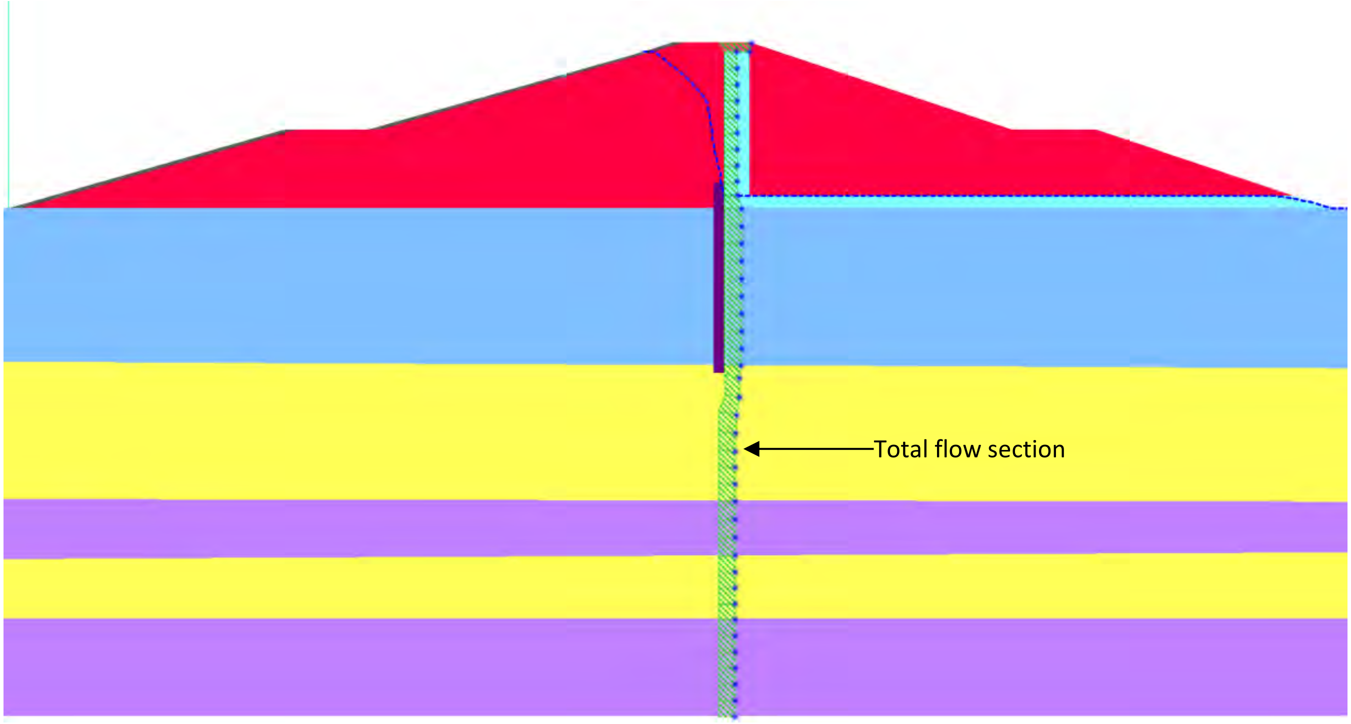

Figure 11. **Walnut Creek Alternative 3 Combined Flow Through Dam and Foundation = 0.005 ft^3/day/ft**

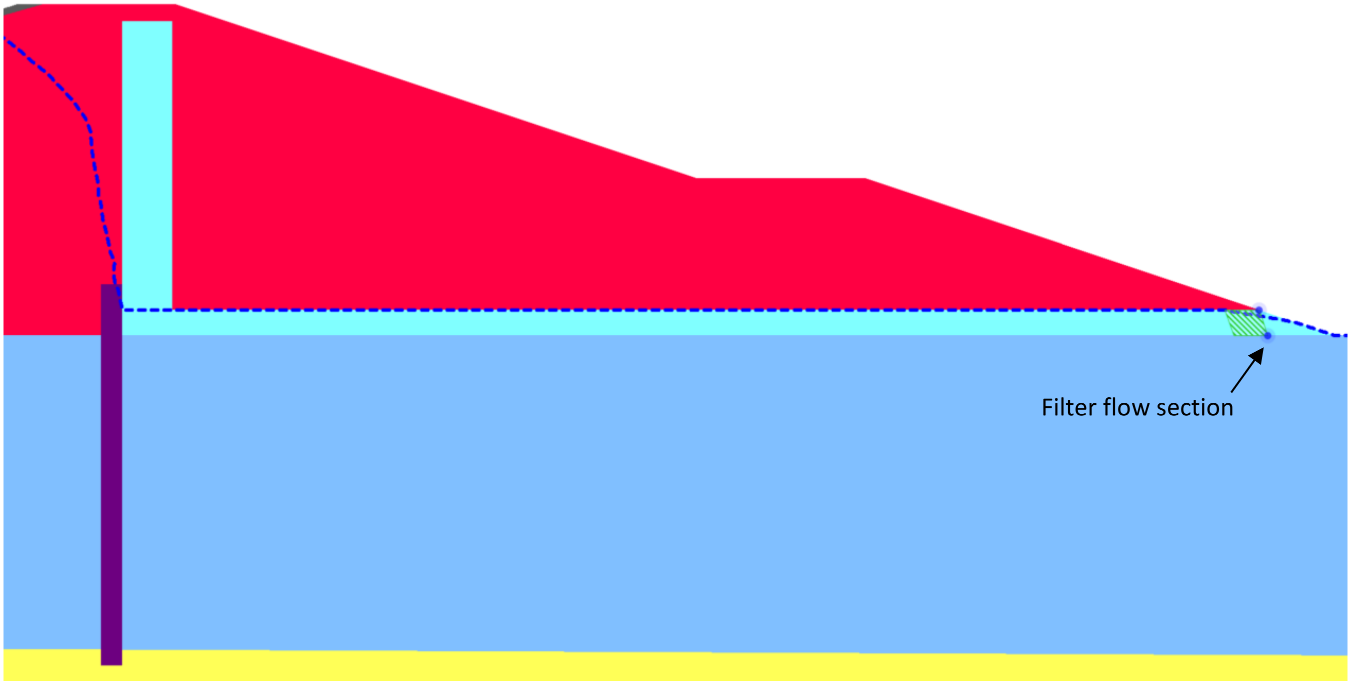

Figure 12 **Walnut Creek Alternative 3 Flow Through Filter = 0.005 ft^3/day/ft**

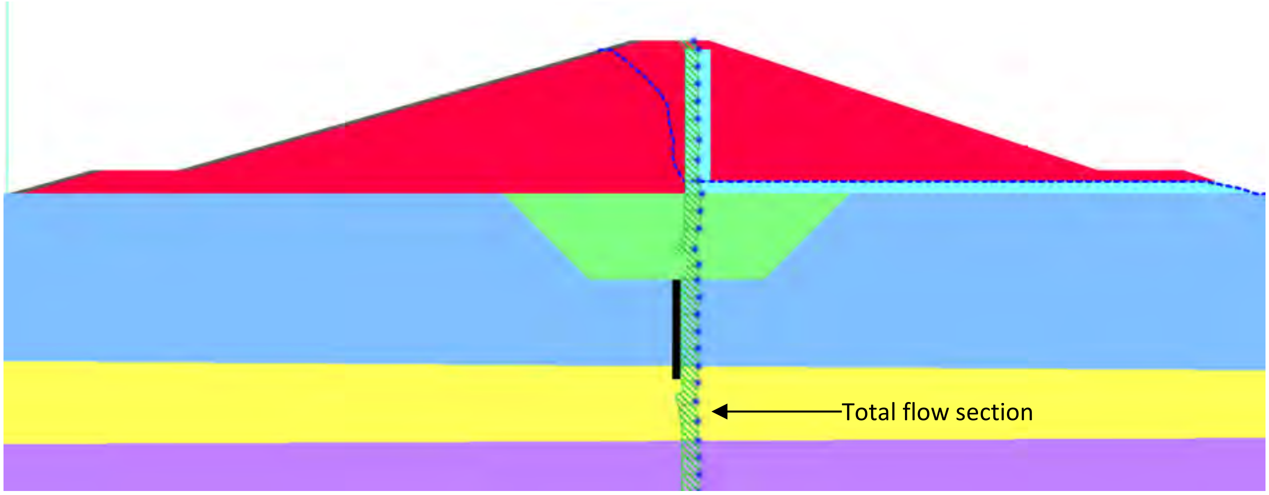

Figure 13. **Birch Creek Alternative 1 Combined Flow Through Dam and Foundation = 0.006 ft^3/day/ft**

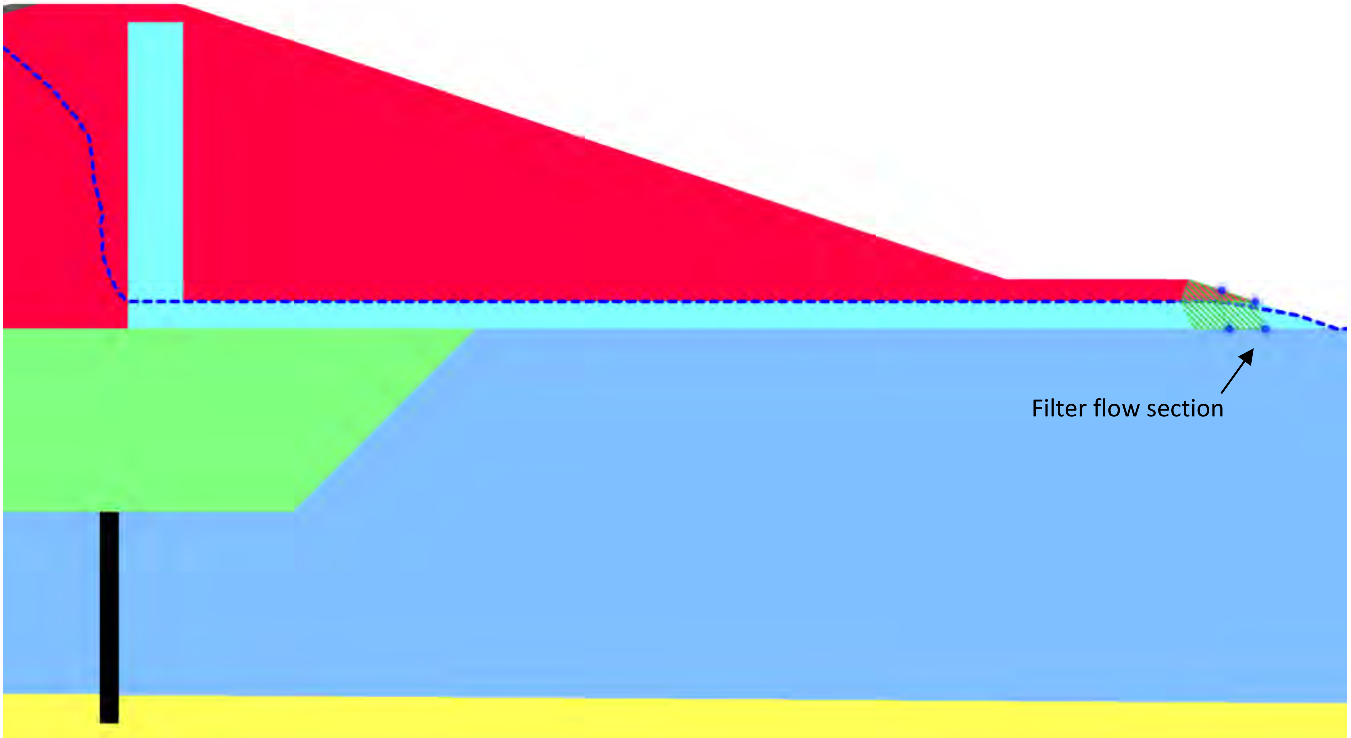

Figure 14. **Birch Creek Alternative 1 Flow Through Filter = 0.004 ft^3/day/ft**

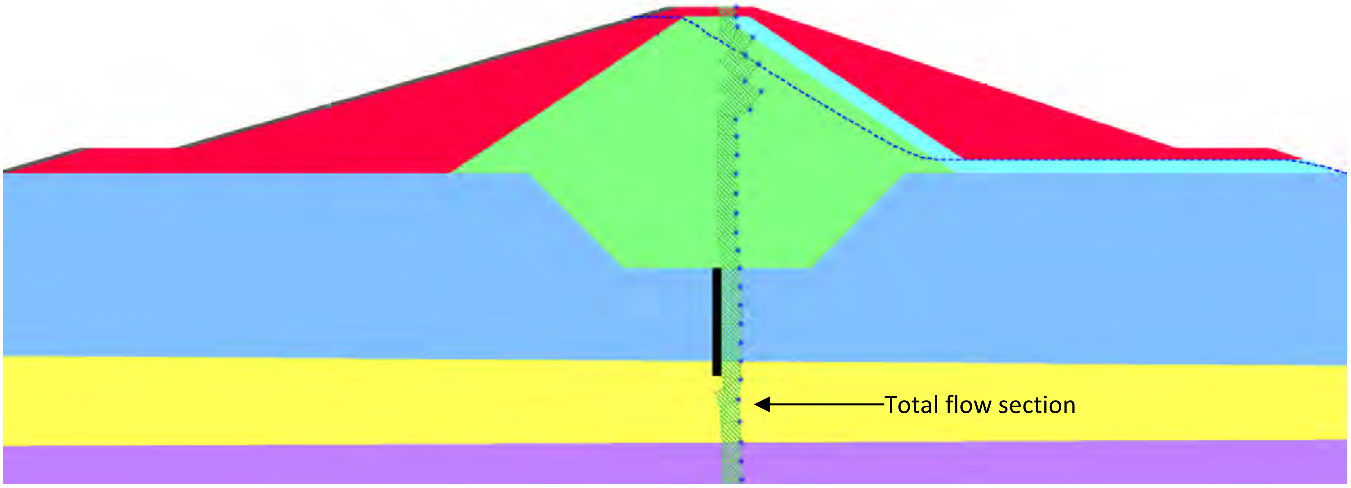

Figure 15. **Birch Creek Alternative 2 Combined Flow Through Dam and Foundation = 0.003 ft^3/day/ft**

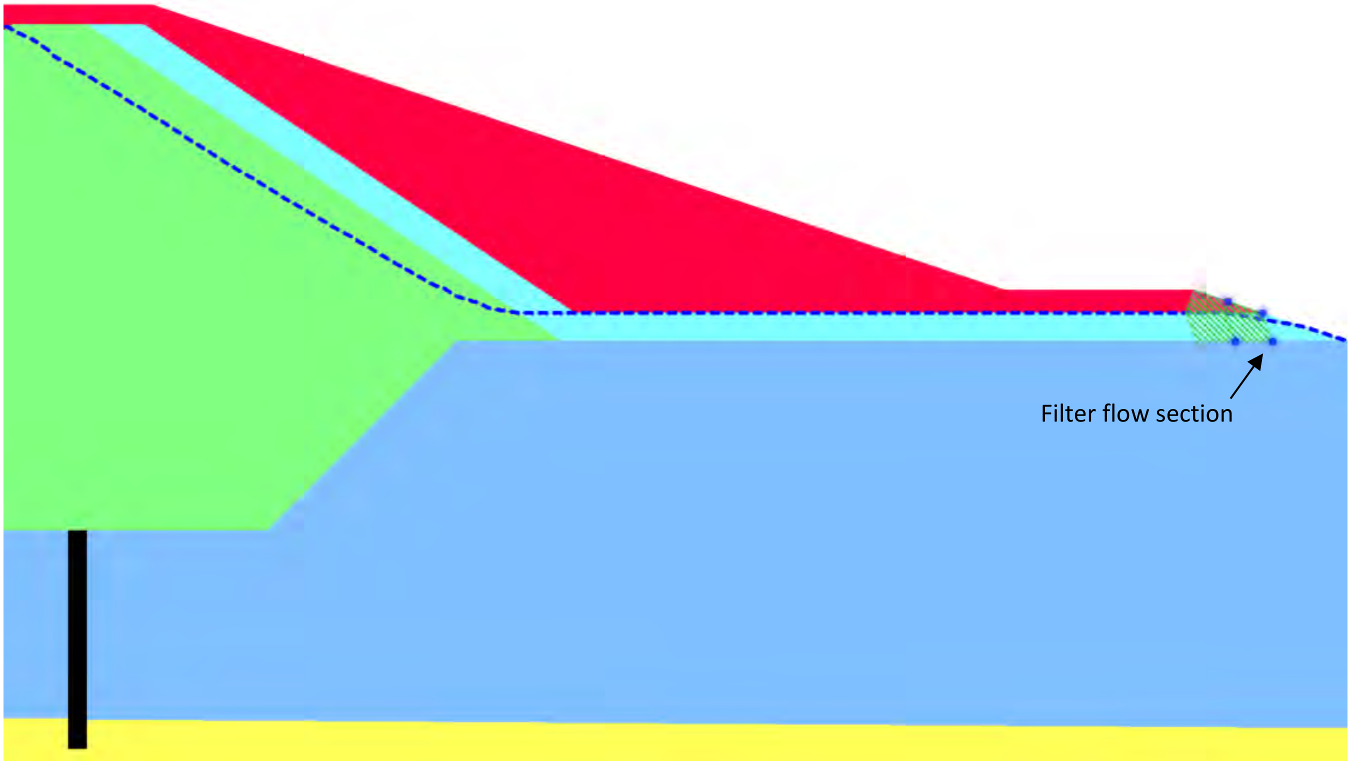

Figure 16. Birch Creek Alternative 2 Flow Through Filter = 0.001 ft^3/day/ft

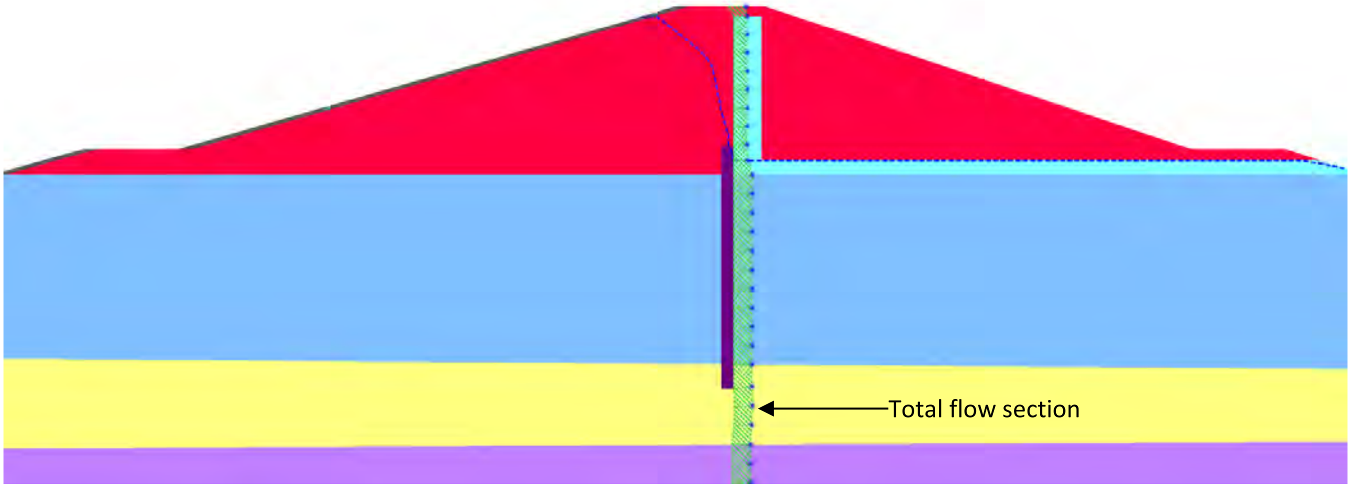

Figure 17. Birch Creek Alternative 3 Combined Flow Through Dam and Foundation = 0.006 ft^3/day/ft

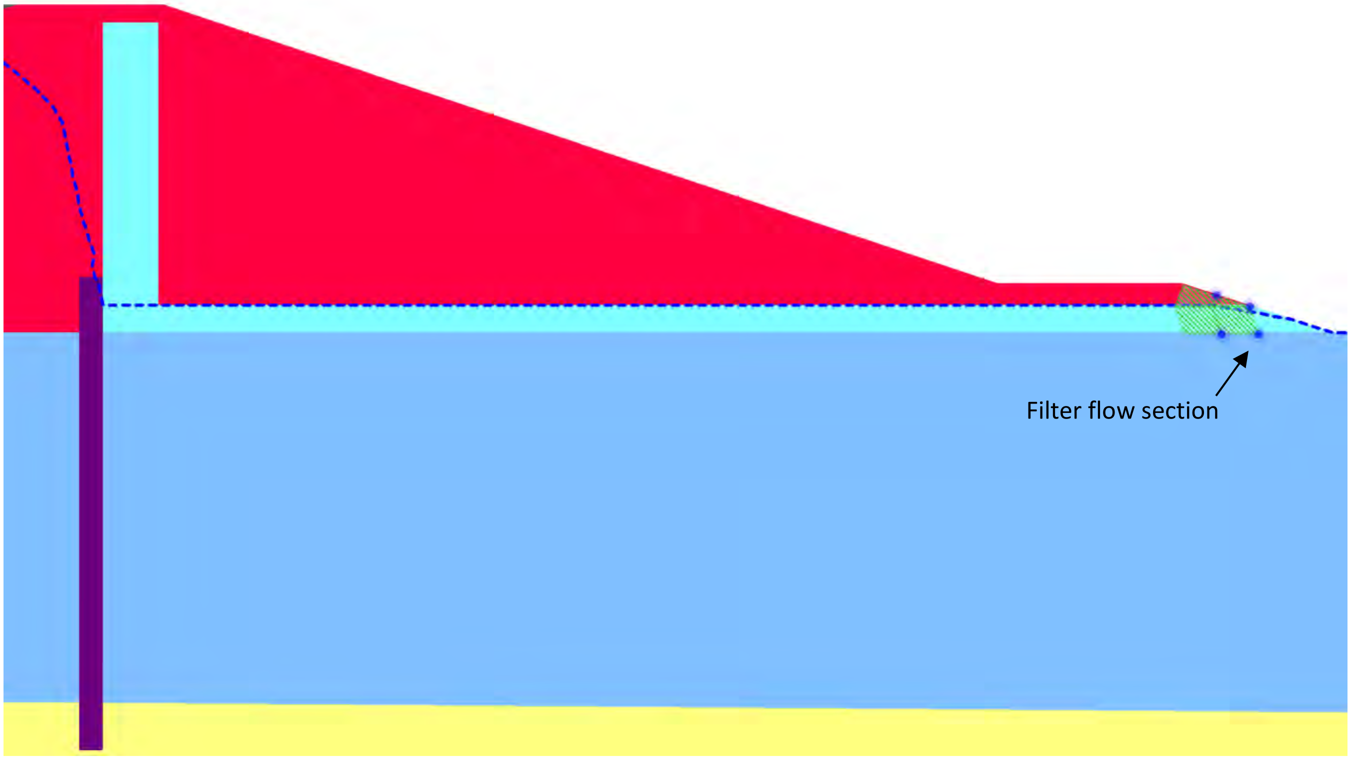

Figure 18. **Birch Creek Alternative 3 Flow Through Filter = 0.005 ft^3/day/ft**

Appendix B-5 Slope stability calculation package

Confidential and Proprietary Business Information of Black & Veatch

Client Name SJRA Page 1 of 34

Project Name Spring Creek Watershed (SCW) Flood Control Dams Project No. 411500

Calculation Title SCW Flood Control Dams Slope Stability Analysis

Verification Method: ☒ Check and Review ☐ Alternate Calculations

Objective: Evaluate static slope stability for the Walnut Creek and Birch Creek Flood Control Dams

Unverified Assumptions Requiring Subsequent Verification

No.	Assumption	Verified By*	Date
1	Estimated soil properties and strength parameters based on information from the Aviles (2024) geotechnical investigations report, literature or past reports are deemed to be appropriate for a preliminary design effort.		
2	.		

Refer to Page _____ of this calculation for additional assumptions.

This Section Used for Software-Generated Calculations

BV Standard Application
Program Name/Version Slope/W , version 10.0.2.18035 (GEOSLOPE International Ltd., 2019)

Review and Approval

Rev	Prepared By*	Date	Checked By*	Date	Approved By*	Date
0	P. Turkson, PhD, P.E.	12/9/2024	D. Bentler, PhD, P.E.	12/9/2024		

*Signature required.

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Table of Contents

1.0 Objective

Evaluate slope stability for the Walnut Creek and Birch Creek Flood Control Dams (hereafter referred to as the Project) for San Jacinto River Authority (SJRA) by determining minimum factors of safety (FoS) and slip surface locations.

2.0 References

1. Aviles Engineering Corporation 2024. "Spring Creek Watershed Flood Control Engineering Feasibility Study Geotechnical Investigation, Report No. G154-21." Report prepared for San Jacinto River Authority.
2. Black & Veatch 2024. "Spring Creek Watershed (SCW) Flood Control Dams Material Calculation Record." Report prepared for San Jacinto River Authority, dated October 2024.
3. Black & Veatch 2024. "Spring Creek Watershed Flood Control Dams Design Basis Memorandum." Report prepared for San Jacinto River Authority, dated November 2024.
4. Texas Commission on Environmental Quality (TCEQ), 2009, Design and Construction Guideline for Dams in Texas, RG-473, August 2009.
5. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13, Chapter 2: Embankment Design, dated December 2012.
6. U.S. Army Corp of Engineers. 2003. *Slope Stability*. EM 1110-2-1902.
7. Unified Facilities Criteria (UFC). 2021.Soil Mechanics. DM 7.1.

3.0 Evaluation Basis

The following section provides the methodology for deterministic static slope stability analysis for the Project. Based on the soil boring logs and laboratory testing data from **Reference 1**, the embankment and foundation zonation was developed, and the respective material properties were selected in **Reference 2**. The selected material properties are used for deterministic static slope stability analysis. Embankment geometry and zonation is based on the Design Basis Memorandum (DBM) by Black & Veatch (**Reference 3**), which was prepared for SJRA. The embankment geometries have been selected in this study to achieve the required global slope stability for the loading cases considered in this work. The loading cases considered for stability analysis in this work are: (i) End of Construction (EOC), (ii) Long-term seepage stability, and (iii) Rapid drawdown (RDD).

Stability analysis was performed for the Project that includes the following three Alternative sections:

- Alternative 1 (homogenous embankment with cutoff trench).
- Alternative 2 (zoned embankment with impervious core and cutoff trench).
- Alternative 3 (homogenous embankment with soil-bentonite cutoff (SBC) wall)

The sections are primarily differentiated by the fill zonation and seepage control features.

The following calculation assumptions have been made:

- Spencer's method adequately captures the minimum factor of safety and critical slip surface; no other limit equilibrium methods were used to compute the factor of safety of trial slip surfaces.
- The minimum factor of safety was considered to be the minimum factor of safety based on either noncircular or circular slip surfaces.
- Applicable analysis method for rapid drawdown is the multi-stage analysis from Duncan et al. (1990).

3.1 General Material Properties

The general material properties for the embankment and foundation zonation have been adopted from **Reference 3**. The natural strata are generally described as comprising alternating layers of silty and clayey sands, and silty and sandy clays. The embankment and foundation zonation for the Project for the three geometry alternatives is shown on **Figure 1**, **Figure 2** and **Figure 3** for Walnut, and **Figure 4**, **Figure 5** and **Figure 6** for Birch.

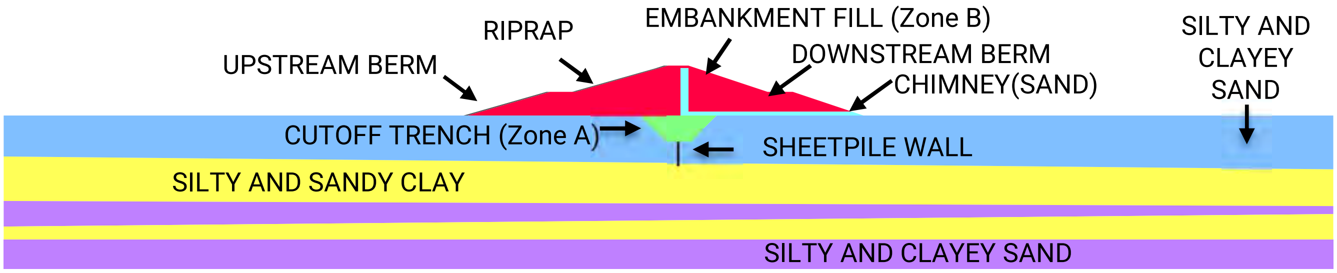

Figure 1. **Alternative 1 Embankment Geometry and Zonation— Walnut**

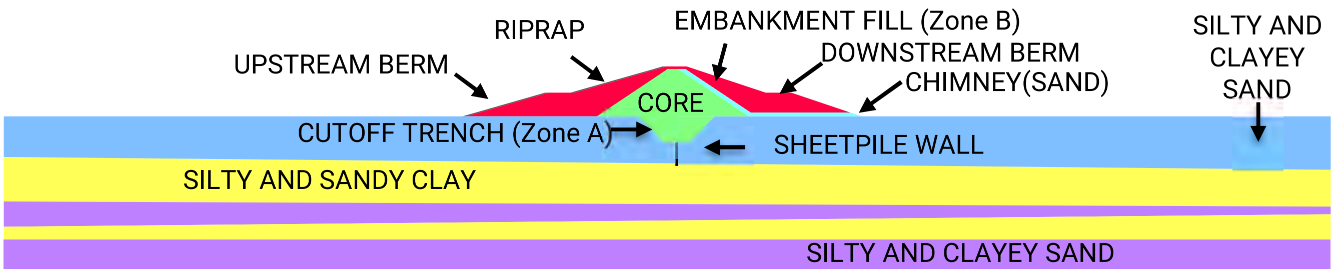

Figure 2. **Alternative 2 Embankment Geometry and Zonation— Walnut**

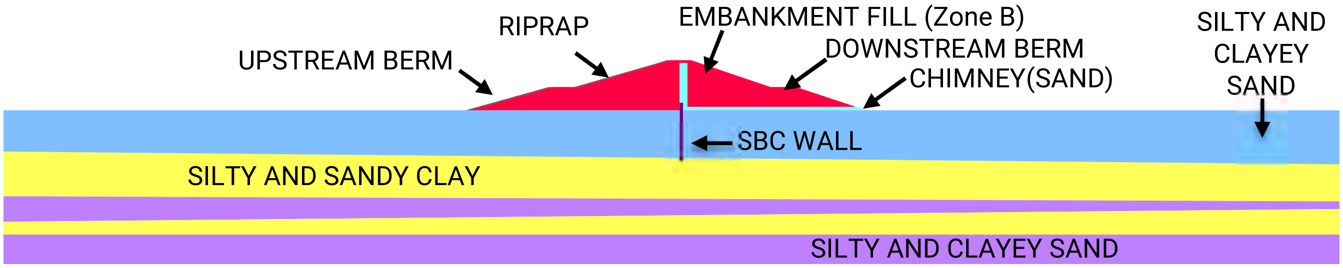

Figure 3. **Alternative 3 Embankment Geometry and Zonation— Walnut**

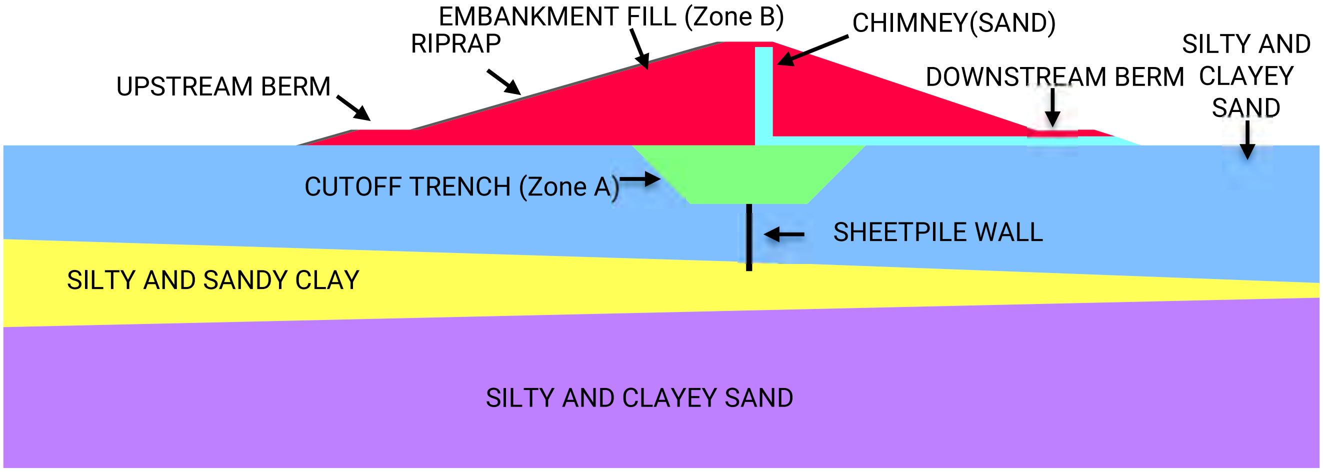

Figure 4. **Alternative 1 Embankment Geometry and Zonation— Birch**

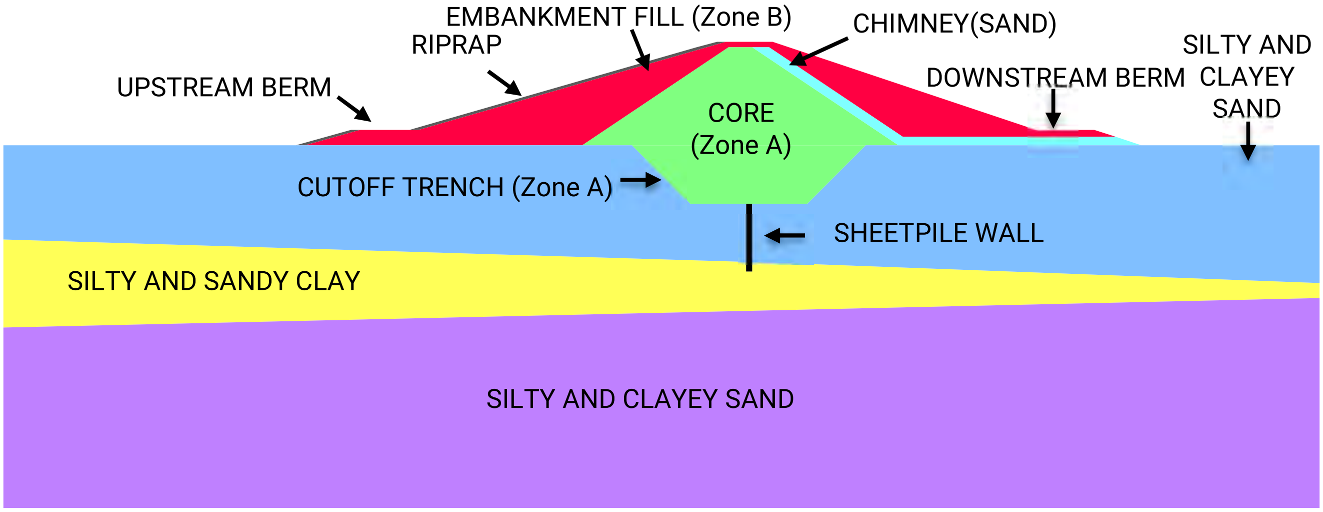

Figure 5. **Alternative 2 Embankment Geometry and Zonation— Birch**

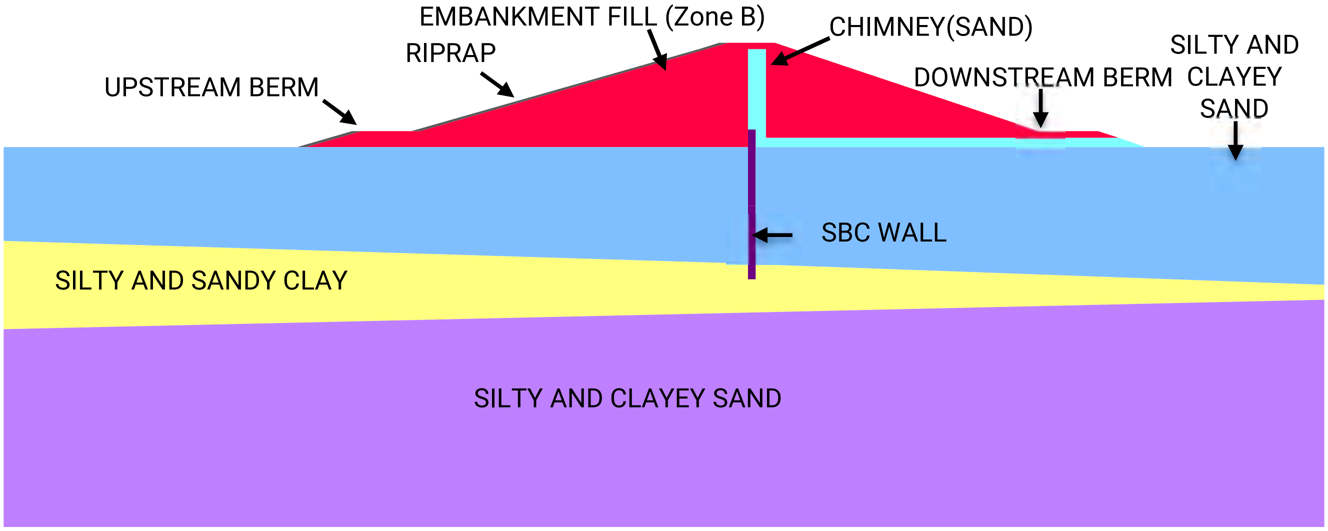

Figure 6. **Alternative 3 Embankment Geometry and Zonation— Birch**

3.2 Index Properties

Values for total unit weights for each material zone were selected from **Reference 2**. Total unit weights assigned to the different material zones are summarized in Table 1. Loading conditions where embankment zone or foundation stratum is anticipated to be saturated require saturated soil unit weight. In some cases, the total unit weight has been assumed to be same as the saturated unit weight.

Table 1. Unit Weights for the Project Material Zones

Material Type	Dam	Unit Weight (pcf)[1]		Loading case for γ_{sat}
		Total, γ_t	Saturated, γ_{sat}[1]	
Zone A	Both	125	125	Long-term, RDD
Zone B		130	130	Long-term, RDD
SBC		90	100	EOC, Long-term, RDD
Filter		120	130	Long-term, RDD
Riprap		124	140	Long-term, RDD
Foundation- silty and sandy clays	Walnut	125	125	EOC, Long-term, RDD
	Birch	123	123	
Foundation- silty and clayey sands[2]	Walnut and Birch	125	125	EOC, Long-term, RDD
	Walnut and Birch	130	130	

1. Unit weight values from **Reference 2**
2. Lower unit weight values for sandy strata directly below dam and higher unit weight for deeper sandy strata.

3.3 Soil Strength Parameters

The following section describes the basis of evaluating shear strength parameters for the different material zones for the Project stability analysis.

3.3.1 Q-Case – Undrained Condition

For the Q-Case, or undrained case, undrained shear strength (s_u) values were evaluated for each fine-grained zone. Values of s_u within each stratum were evaluated based on s_u estimates from the Unconsolidated Undrained (UU) Triaxial Compression (TC) tests, and/or on guidance from past work or published literature as described in **Reference 2**. The values of s_u adopted from **Reference 2** for stability analysis are summarized in Table 2.

Table 2 Q-Case Design Values

Material Type	Dam	Undrained strength (psf)[1]
Zone A	Both	720
Zone B		1000
SBC		No strength
Filter		NA
Riprap		NA
Foundation- silty and sandy clays		722
Foundation- silty and clayey sands	Walnut	1030
	Birch	1000

1. NA — Not Applicable

3.3.2 S-Case – Drained Condition

For the S-Case, or drained case, design values for effective friction angle (ϕ') were evaluated for each stratum based on Consolidated-Undrained (CU) triaxial testing, or based on guidance from past work or published literature where laboratory data are not available as described in **Reference 2**.

In addition, design values for the *c'* for soil strata that are considered fine-grained were evaluated based on CU triaxial testing and with guidance from past work or published literature as described in **Reference 2**. The values of c' and ϕ' adopted from **Reference 2** for stability analysis are summarized in Table 3.

Table 3. Effective Stress Strength Parameters for the Project- Walnut and Birch

Material Type	Effective Stress Strength Parameters	
	c' (kPa)	ϕ' (deg)
Zone A	0	21
Zone B	0	31
SBC	No strength	
Filter	0	36
Riprap	0	40
Foundation- silty and sandy clays	0	21
Foundation- silty and clayey sands	0	31

3.3.3 R-Case

For the R-Case, which is primarily used for RDD slope stability analyses, R-Envelope values for cohesion intercept (c_R) and friction angle (ϕ_R) were developed based on **Reference 2**. These parameters were developed from consolidated undrained (CU) triaxial testing (also referred to as R testing by USACE) in **Reference 1** and are summarized in Table 4.

Table 4. R-Envelope Parameters— Walnut and Birch

Material Type	R-Envelope Strength Parameters	
	cR (kPa)	ϕR (deg)
Zone A	240	14.6
Zone B	210	23.6
SBC	No strength	
Filter	NA	
Riprap		
Foundation- silty and sandy clays	240	14.6
Foundation- silty and clayey sands	210	23.6

4.0 Analysis

4.1 Water Levels Used in Stability Analysis

Hydrologic and hydraulic (H&H) modeling was performed to identify the project design floods (PDF) as part of the current scope of work. The upstream water levels used in this study for stability analysis have been adopted from **Reference 3**. The normal operating pool elevations and the project design flood (PDF) elevation are summarized in Table 5. Only one drawdown elevation is specified in Table 5 for RDD analysis due to the primary function of the Project as detention dams with uncontrolled spillway: full drawdown to existing grade elevation.

The typical position of the steady state phreatic surface in the embankment and the tailwater elevation was specified based on best judgment and experience and is shown on **Figure 7**, **Figure 8** and **Figure 9**. It is noted that the two dams are flood control structures that will only impound water for a several weeks following storm periods, and therefore given the low hydraulic conductivity of the dam embankments steady state seepage conditions may not develop before the flood pool recedes.

Table 5 **Water Levels Used in Stability Analysis for the Project**

Condition	Headwater Elevation (feet)	Tailwater Elevation[2] (feet)
Walnut Creek Dam		
End of Construction	219	
Maximum 100-year Flood (MaxNF)	256.2	Creek bed
Peak Design Flood (PDF)	261.6	
RDD - start of drawdown[1]	256.2 and 261.6	
Birch Creek Dam		
End of Construction	218.2	
Maximum 100-year Flood (MaxNF)	251.1	Creek bed
Peak Design Flood (PDF)	257.1	
RDD - start of drawdown[1]	251.1 and 257.1	

1. Start-of-drawdown water level considered for both MaxNF and PDF.

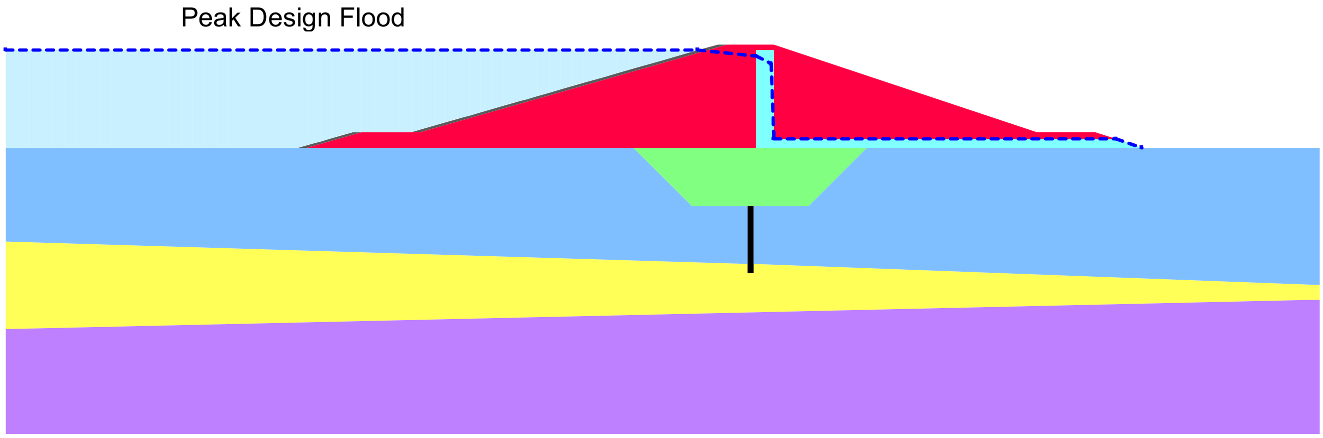

Figure 7. **Alternative 1 Typical Phreatic Surface Location Adopted for this Study**

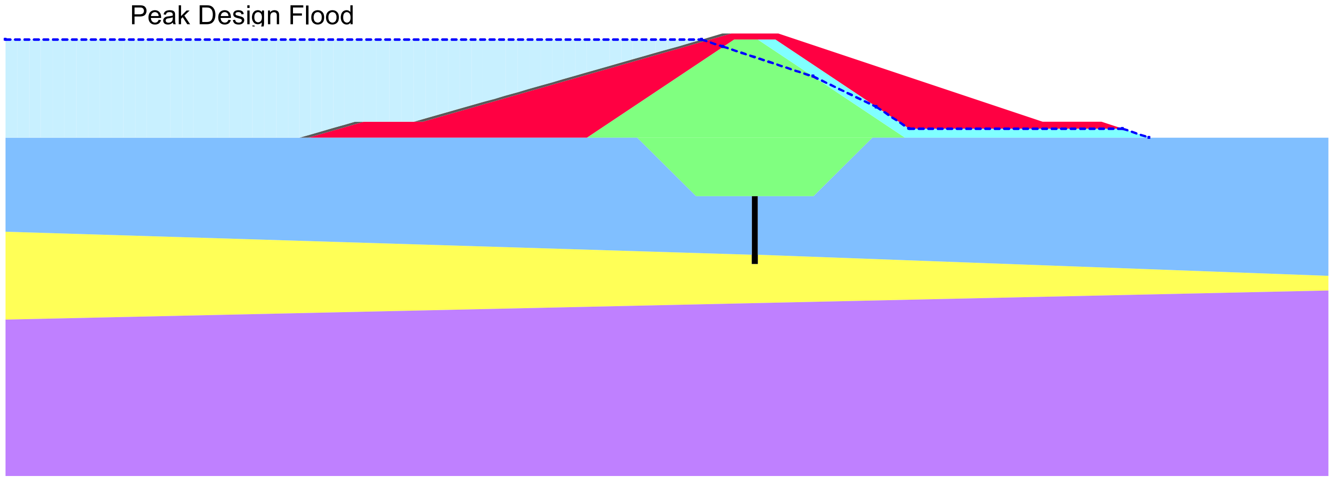

Figure 8. **Alternative 2 Typical Phreatic Surface Location Adopted for this Study**

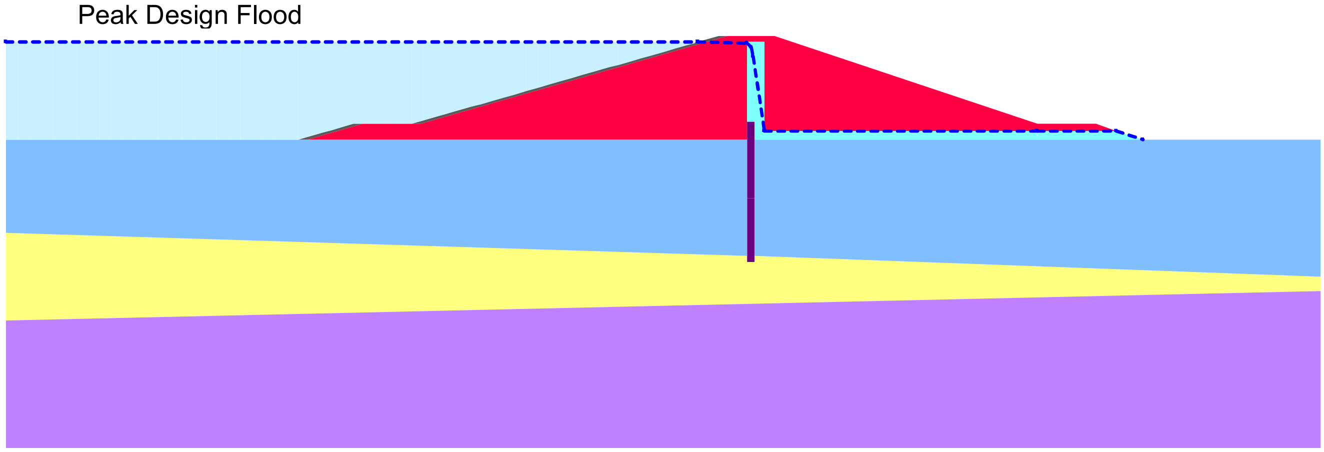

Figure 9. **Alternative 3 Typical Phreatic Surface Location Adopted for this Study**

4.2 Design Criteria

In addition to information provided by **Reference** Error! Reference source not found., **Reference 4** is selected as the design basis for several analyses and activities associated with the Project. **Reference 6Error! Reference source not found.** is included in the design basis for the RDD loading condition.

FoS design criterion for RDD loading generally varies with the estimated frequency of RDD loading. Pumped storage reservoirs and flood control detention dams, for example, are operated with frequent drawdown loadings and require higher computed FoS (greater than 1.3) than reservoirs with less frequent drawdown events. However, the assumption of long-term seepage as the start-of-drawdown phreatic surface for RDD analysis of the detention dams evaluated in this work may be sufficiently conservative. Hence, RDD minimum FoS criterion of 1.3 based on USBR recommendation has been adopted for drawdown to dry creek channel grade.
End of construction, long-term, flood, and RDD loading conditions have been evaluated within the scope of this work. The selected target FoS are summarized in **Table 6**.

Table 6 **Design Minimum Factors of Safety**

Loading Condition	TCEQ Min. FoS	USBR Min. FoS	Design Basis Shear Strength Parameters	Design Basis FoS	Evaluated Slope
End of Construction	1.25	1.3-1.4	Undrained	1.3	Upstream (U/S) and Downstream (D/S)
Long Term (Normal 100-year Flood)	1.5	1.5	Drained	1.5	D/S
Peak Design Flood	—	1.2-1.3	Drained	1.2-1.3	D/S
Full or Partial RDD	1.2	1.2-1.3	Drained Undrained (R-Envelope)	1.3-1.5	U/S

4.3 Stability Analysis

Deterministic calculated FoS from limit equilibrium calculations are presented in this section and are compared against slope stability design criteria. Deterministic analyses that utilize selected design engineering parameters from **Section 3.0** are presented for the loading conditions described in **Section 4.2**.

The slope stability software GeoStudio 2019 Version 10.0.2.180354 was used for this study and the Spencer (1967) slope stability calculation method was specified. The “optimize critical slip surface location” option, which removed constraints on the slip surface shape was selected. Specified analysis convergence options used for slope stability analysis are summarized in **Table 7** and **Table 8**. The typical design sections evaluated for stability against the loading conditions in **Section 4.2** are shown as **Figure 10**, **Figure 11** and **Figure 12.**

Table 7 **Convergence Options Used in Slope Stability Analyses**

Convergence Parameter	Value
Number of slices	30
Minimum slip surface depth	0.1 feet
Tolerable difference in FoS	0.001
Maximum iterations	100
Search method	Root finder
Maximum absolute lambda	2

Table 8 **Optimization Critical Slip Surface**

Search Option	Value
Maximum iterations	2,000
Tolerable difference in FS	1×10^{-7}
Number of points slip surface	Starting: 8/Ending: 16
Number of complete passes	1
Maximum concave angle	Driving side: 5 degrees/Resisting side: 1 degree

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Client SJRA
Project SCW Flood Control Dams **Unit**
Project No. 411500 **File No.**
Title Slope Stability Analysis

Computed By P. Turkson
Date 12/9/2024
Approved By
Date
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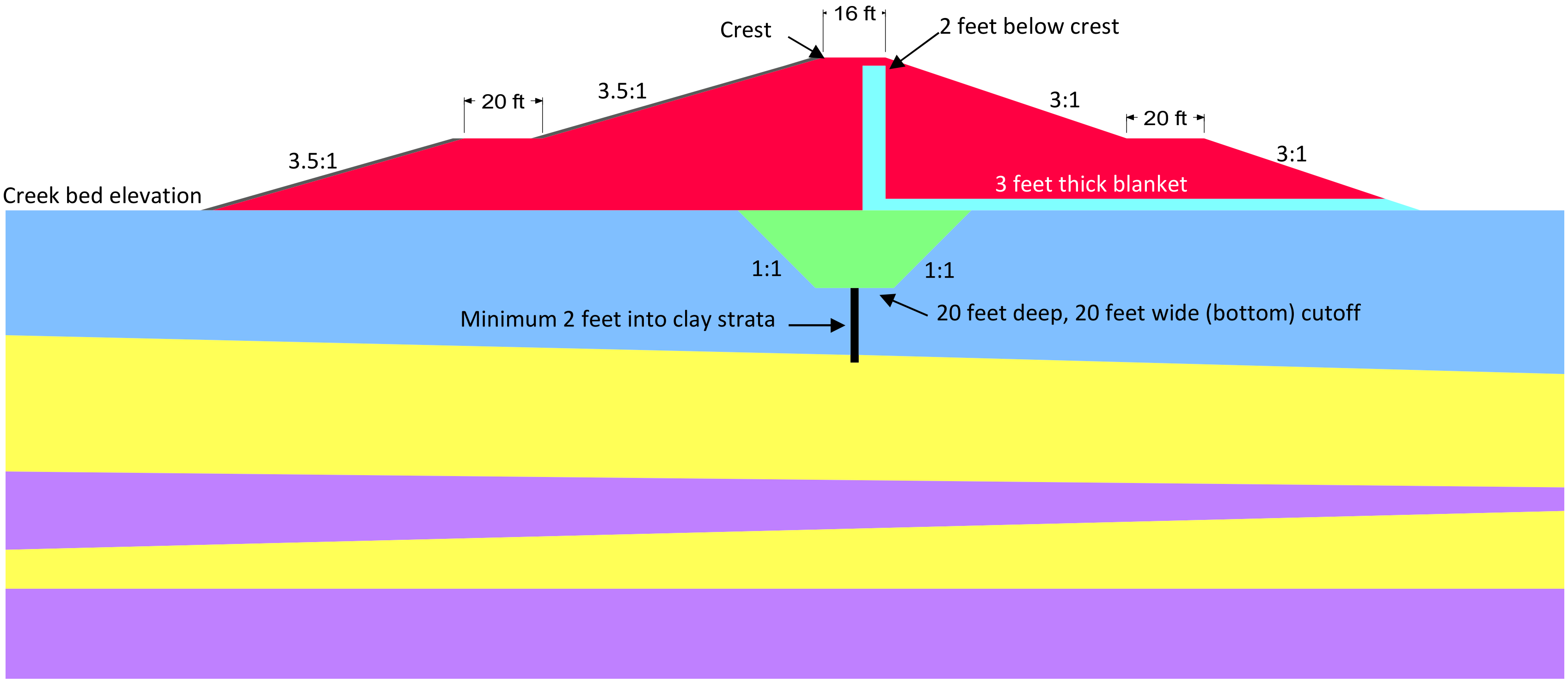

Figure 10 **Alternative 1 Typical Section Used in Slope Stability Analysis**

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Client SJRA
Project SCW Flood Control Dams **Unit**
Project No. 411500 **File No.**
Title Slope Stability Analysis

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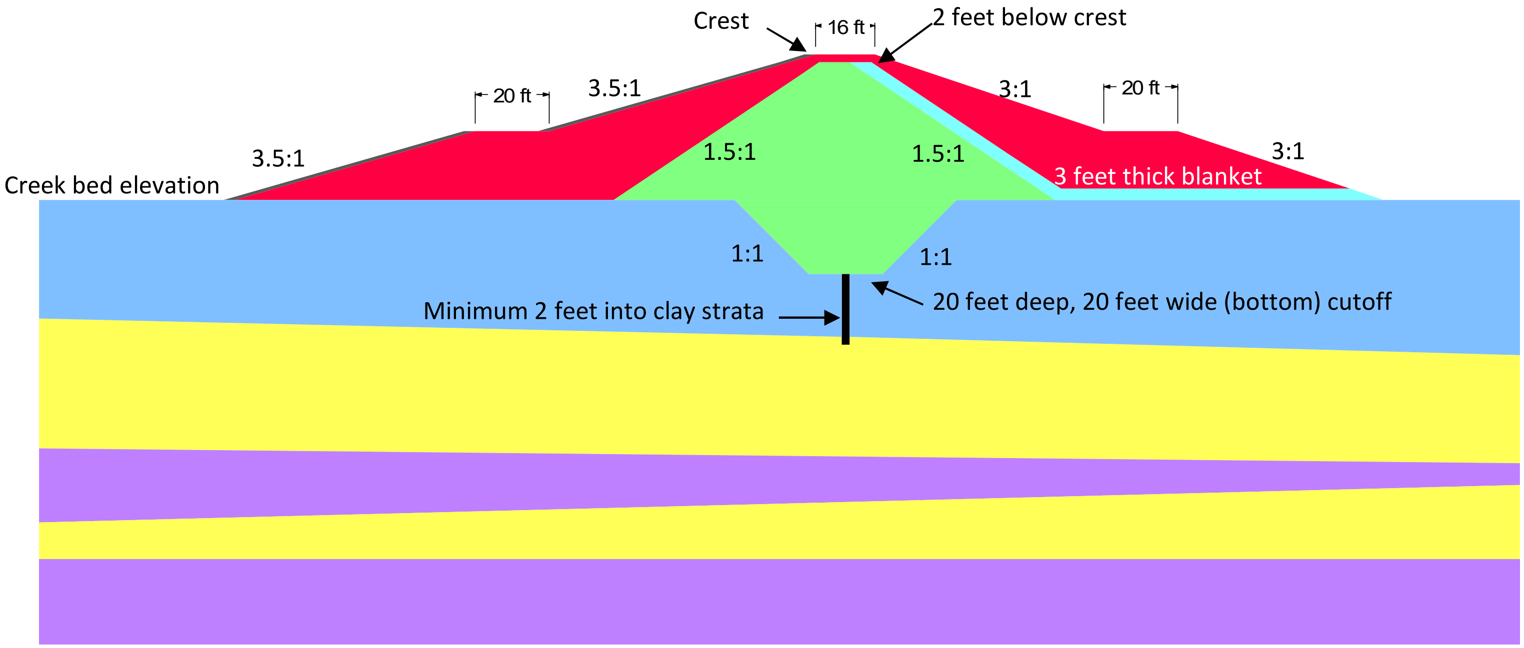

Figure 11 **Alternative 2 Typical Section Used in Slope Stability Analysis**

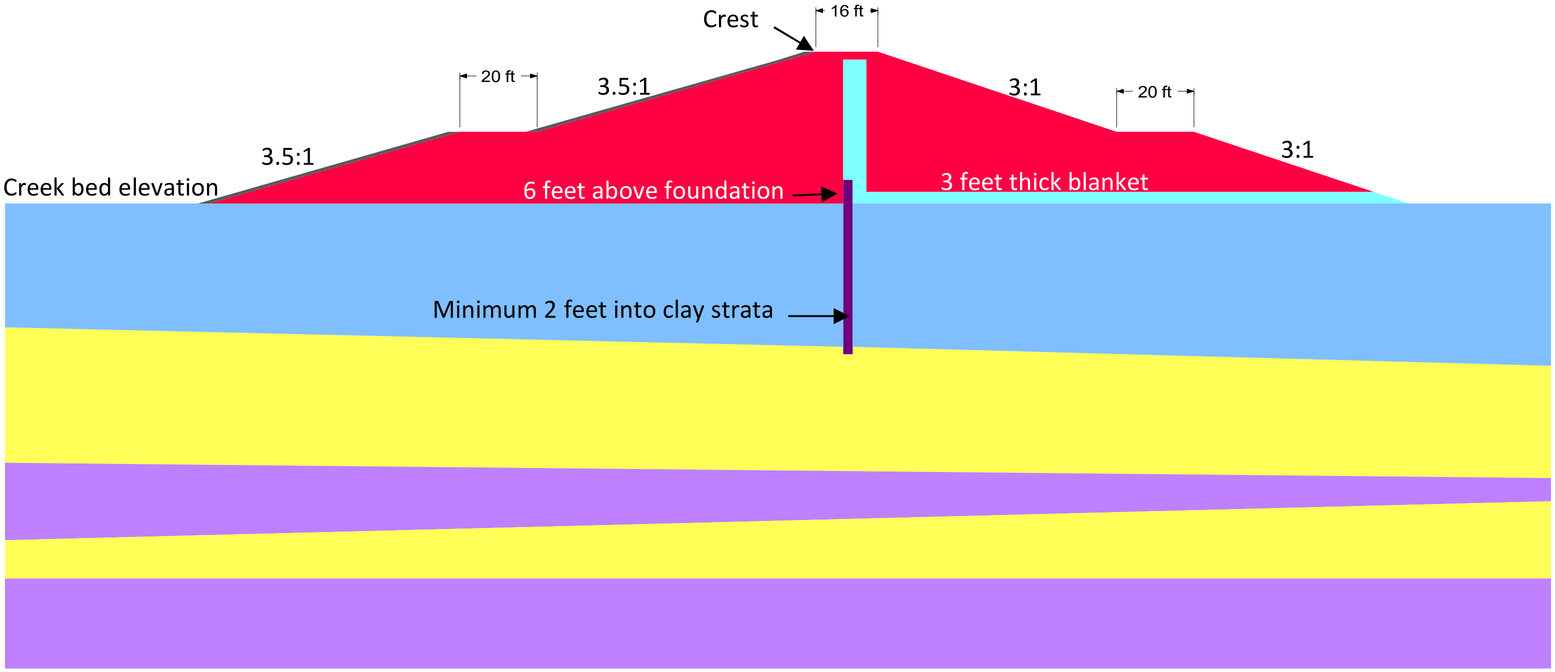

Figure 12. **Alternative 3 Typical Section Used in Slope Stability Analysis**

4.4 Stability Analysis Results

The calculated FoS from static slope stability analysis are summarized in **Table 9**. The results signify that the stability design criteria are satisfied for the proposed embankment geometries except for the end of construction and rapid drawdown loading cases.

The basis of design value for s_u for the silty and sandy clay foundation strata was the 95% lower confidence limit of the test data. This presents significant conservatism in the selected value. The minimum average design Standard Penetration test blow counts (SPT N_{60}-value) for silty and sandy clay foundation strata was 29 blows per foot (bpf) from **Reference 2.** Table 8-10 in **Reference 7** present range of s_u (2000 to 4000 psf) for N values ranging from 15 to 30 bpf. Hence the design s_u value was revised upwards to 1924 psf which is the 33rd Percentile value of the test data.

Also, the design s_u value for Birch Creek silty and clayey sand strata was revised upwards to 1149 psf which is the 33rd Percentile value of the test data considering the design average N_{60} value of 25 bpf for Birch Creek (**Reference 2**).

The end of construction FoS was recalculated, and the results are summarized in Table 10.

The start-of-drawdown phreatic surface in the dam was assumed to be at long-term steady state conditions. This conditions often takes years to be established for dams which impound water continuously over a long time. Considering the primary function of the Project as detention dams, the steady state condition may never be established and the use of such in rapid drawdown analysis presents significant conservatism. When the extent of start-of-drawdown phreatic surface is specified on the upstream slope face or up to 6 feet into the upstream section of the dam (illustrated on **Figure 13**), the calculated FoS is 1.3. Considering the level of conservatism associated with the rapid drawdown analysis, the FoS presented in **Table 9** are considered acceptable for this level of effort. A robust upstream slope protection including rock rip or soil cement including bedding requirements is recommended. Other forms of embankment slope surface protection capable of providing robust slope protection as provided by Section 6.1 of **Reference 4** may also be explored.

The critical slip surface locations for the downstream and upstream slopes are shown on **Attachment 1**.

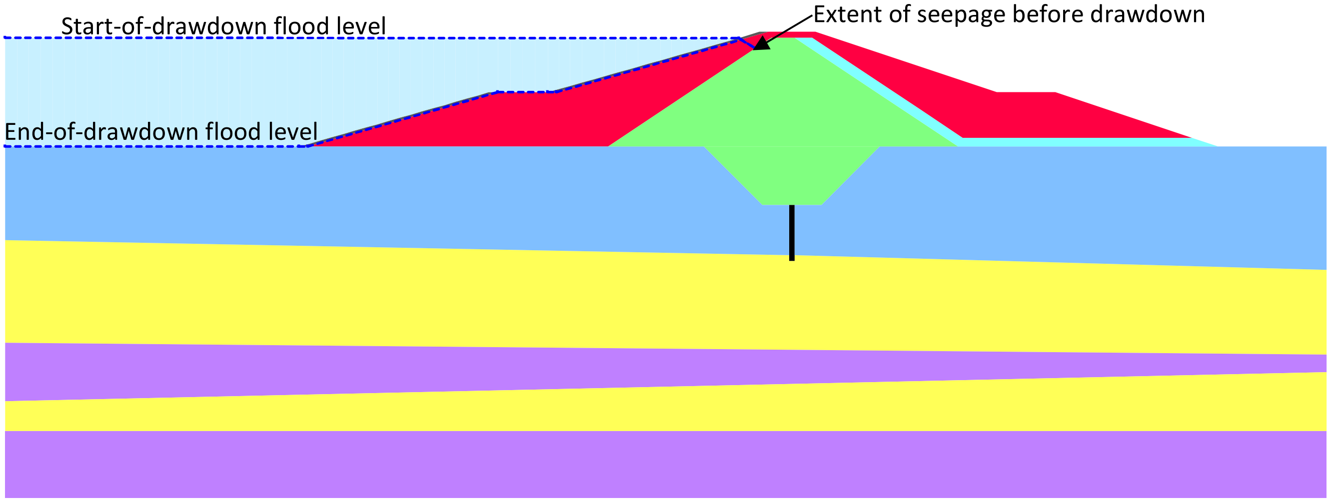

Figure 13. **Illustration of Truncated Phreatic Surface for Rapid Drawdown**

Table 9 **Static Slope Stability Results**

Loading Condition	**Remarks**	**Min. Required FoS (per Table 6)**	**Black & Veatch FoS**		
			Alt. 1	**Alt. 2**	**Alt. 3**
Walnut Creek Dam					
End of Construction^1	—	1.3	1.0 and 1.0 (Note 1)	1.0 and 1.0 (Note 1)	1.0 and 1.0 (Note 1)
Long Term (Normal 100-year Flood)		1.5	1.8	1.9	1.8
Peak Design Flood		1.2-1.3	1.9	1.8	1.8
RDD from Normal 100-year Flood	Drawdown to creek bed	1.3-1.5	1.3	1.3	1.3
RDD from Peak Design Flood			1.3	1.2 (Note 2)	1.3
Birch Creek Dam					
End of Construction^1	—	1.3	1.2 and 1.1 (Note 1)	1.2 and 1.0 (Note 1)	1.2 and 1.1 (Note 1)
Long Term (Normal 100-year Flood)		1.5	1.8	1.6	1.8
Peak Design Flood		1.2-1.3	1.8	1.6	1.8
RDD from Normal 100-year Flood	Drawdown to creek bed	1.3-1.5	1.3	1.3	1.3
RDD from Peak Design Flood			1.2 (Note 2)	1.2 (Note 2)	1.2 (Note 2)

1. FoS for upstream and downstream slope face, respectively.
2. FoS= 1.2 is acceptable based on recommendations provided in this calculation package.

Table 10 **Updated End of Construction Stability Results**

Loading Condition	**Remarks**	**Min. Required FoS (per Table 6)**	**Black & Veatch FoS**		
			Alt. 1	**Alt. 2**	**Alt. 3**
Walnut Creek Dam					
End of Construction	Recalculated FoS using s_u = 1924 psf for clay strata and Zone A	1.3	1.5 and 1.5 (Note 1)	1.7 and 1.7 (Note 1)	1.4 and 1.3 (Note 1)
Birch Creek Dam					
End of Construction	Recalculated FoS using s_u = 1924 psf for clay strata and Zone A, and s_u = 1149 psf for sand strata.	1.3	1.6 and 1.6 (Note 1)	1.8 and 1.3 (Note 1)	1.5 and 1.4 (Note 1)

1. FoS for upstream and downstream slope face, respectively.

5.0 Slope Stability Conclusions and Recommendations

The slope stability analyses were completed using soil design parameters, inputs, modeling methods, and assumptions previously described herein. The calculated FoS generally equal or exceed the minimum FoS criteria stipulated in **Section 4.2** and are considered to be acceptable. In the case of rapid drawdown where calculated FoS is less than the minimum FoS criterion, the calculated FoS are considered acceptable based on the reasons provided and on condition that a robust upstream slope protection method is implemented. Comparison of the FoS to design criteria shows the proposed embankment alternative geometries are acceptable.

The following recommendations have been made for consideration in an advanced design effort:

- The calculated end of construction FoS showed that the characterization and selection of undrained strength is sensitive to embankment stability under rapid loading. Initial end of construction FoS calculation (see **Table 9**) adopted a design undrained strength value which is equal to 95% lower confidence limit of the test data, resulting in a relatively low undrained strength value. The predicted slope instability gives insight to the sensitivity of the embankment stability to both the silty and sandy clay and silty and clayey sand strength, and shows the importance of obtaining strength data from compacted borrow sources specimens and the need for site specific foundation strengths from additional site specific geotechnical field exploration before construction. Furthermore, TCEQ guidelines (Section 4.2) (**Reference 4**) outline potential treatment options for weak foundation materials to include removal of problematic material or improve it in-place, or limit the rate of embankment construction to address instability due to weak foundation.

- The calculated FoS for rapid drawdown was less than the minimum FoS criteria for Alternative 2 of Walnut and for all alternatives of Birch. The conservatism associated with assuming steady state conditions prior to drawdown is significant and resulted in lower calculated FoS. This seepage condition may never be established in a dry detention dam considering that the embankment will only impound water during floods and may experience long periods of dryness. Recognizing the uncertainty associated with flood prediction and soil behavior, a robust upstream slope protection system including rock rip and soil cement with bedding requirements is recommended. Other slope protection systems appropriate for the primary function of the Project may be explored.

Attachment 1:

Critical Slip Surfaces from Static Stability Analysis— Walnut

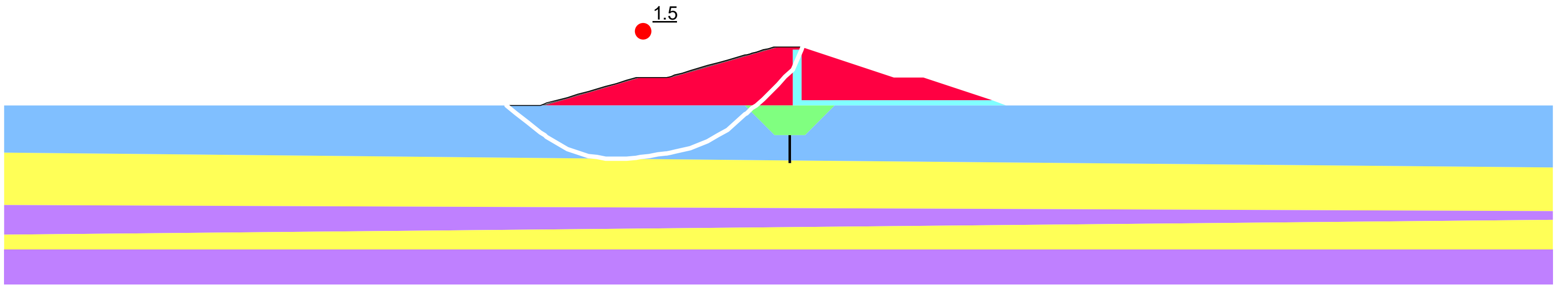

Figure 14 **Alternative 1 End of Construction Upstream Factor of Safety— Walnut**

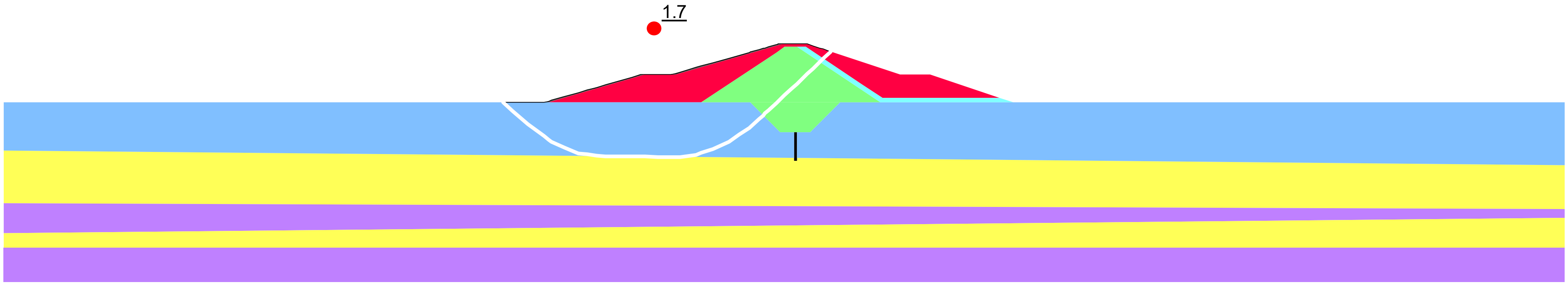

Figure 15 **Alternative 2 End of Construction Upstream Factor of Safety— Walnut**

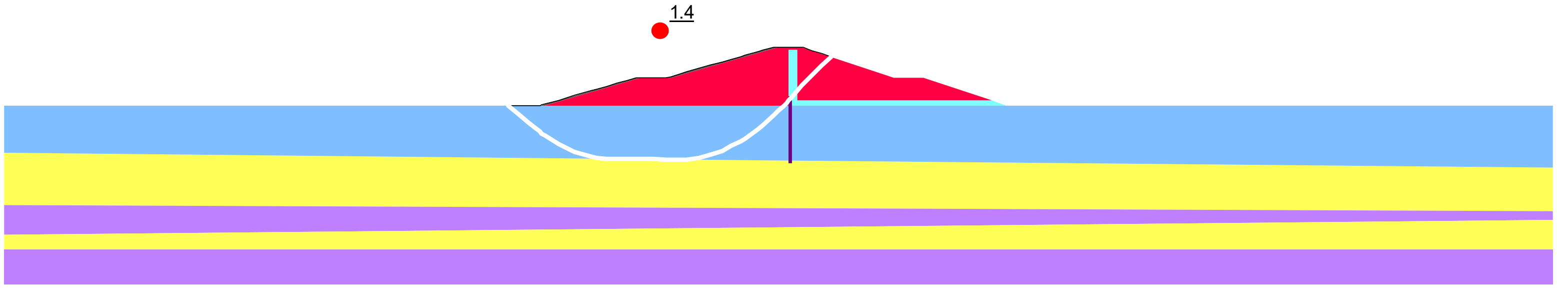

Figure 16 **Alternative 3 End of Construction Upstream Factor of Safety— Walnut**

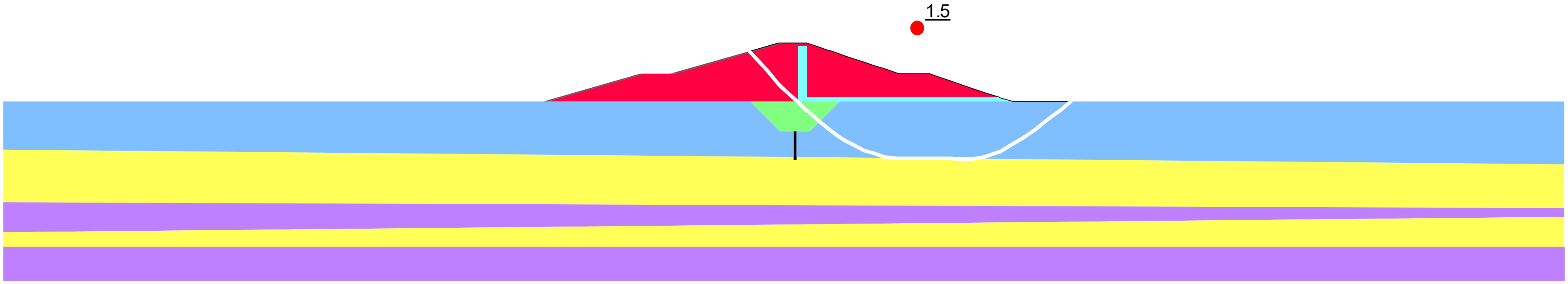

Figure 17 Alternative 1 End of Construction Downstream Factor of Safety— Walnut

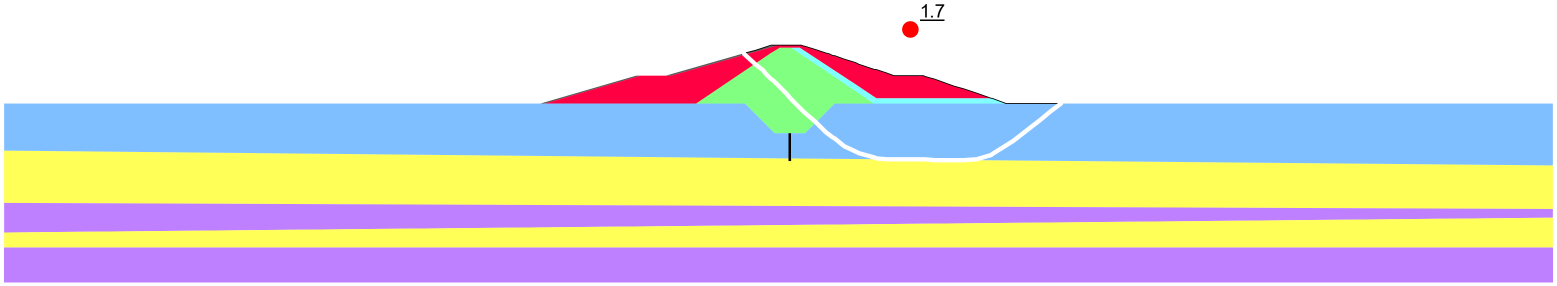

Figure 18 Alternative 2 End of Construction Downstream Factor of Safety— Walnut

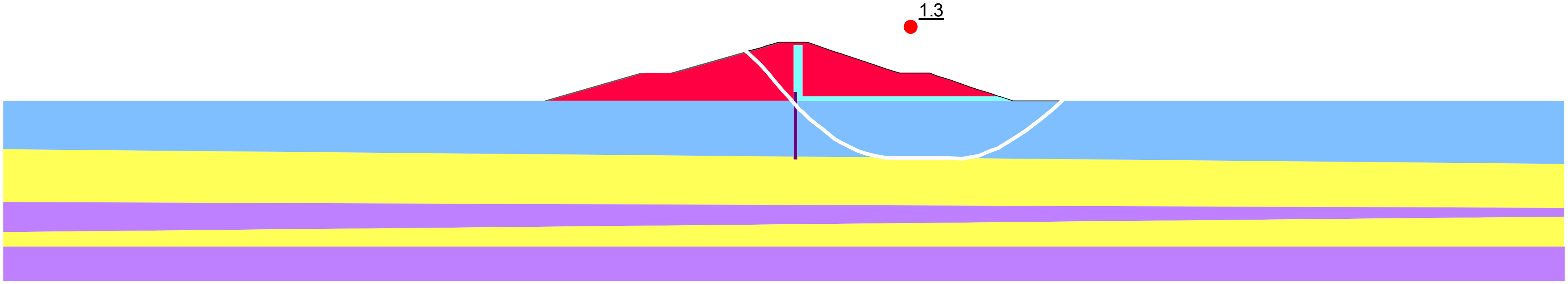

Figure 19 Alternative 3 End of Construction Downstream Factor of Safety— Walnut

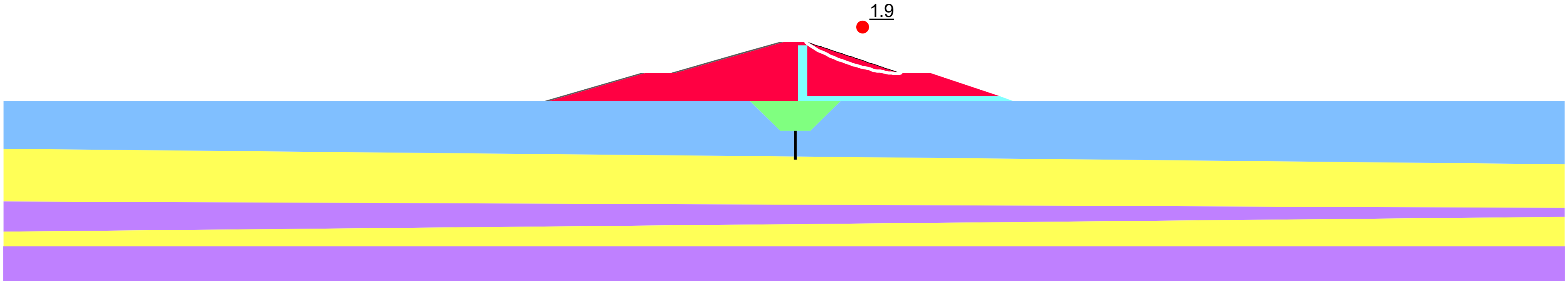

Figure 20 Alternative 1 100-Year Peak Flood Factor of Safety— Walnut

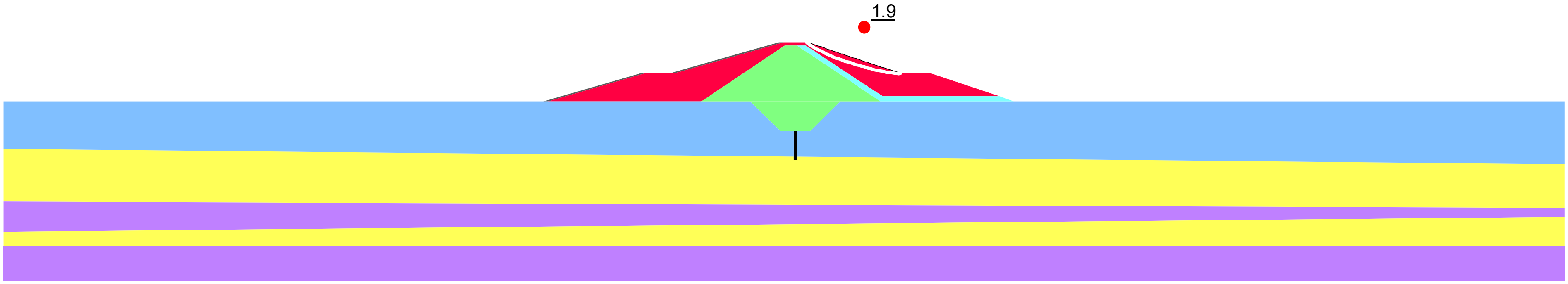

Figure 21 Alternative 2 100-Year Peak Flood Factor of Safety— Walnut

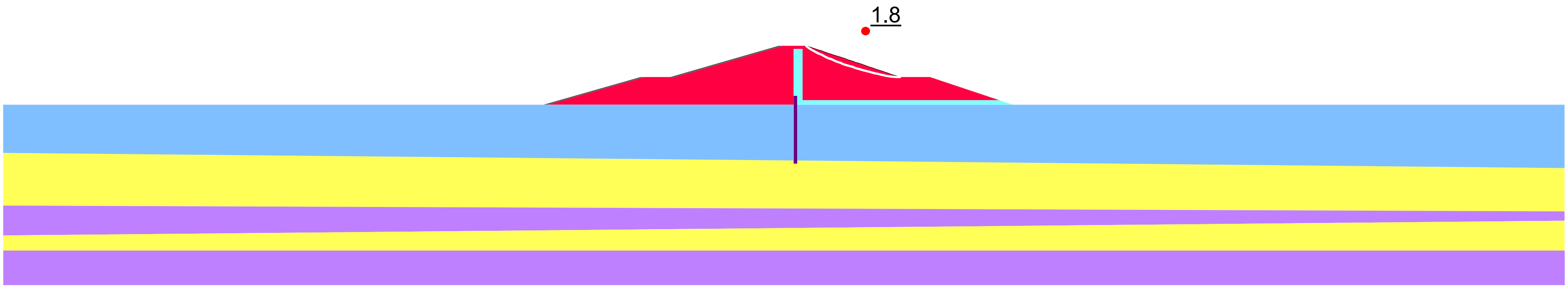

Figure 22 Alternative 3 100-Year Peak Flood Factor of Safety— Walnut

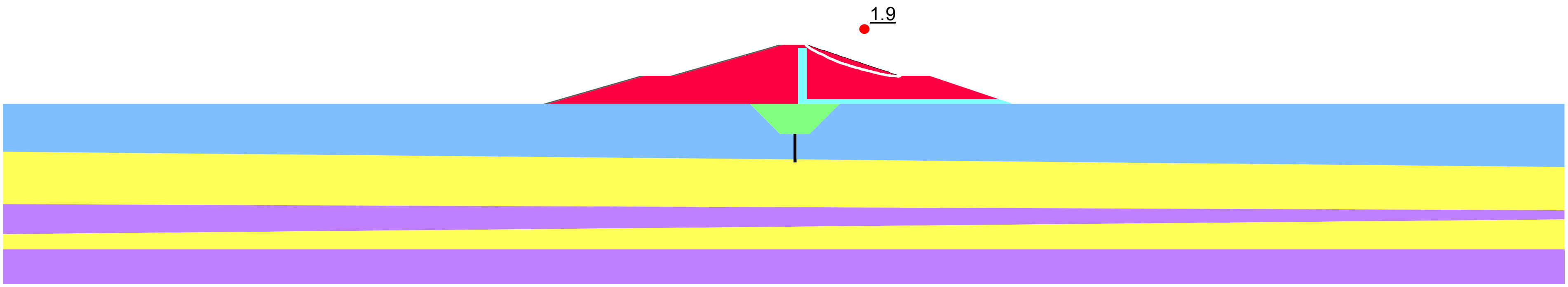

Figure 23 Alternative 1 Peak Design Flood Factor of Safety— Walnut

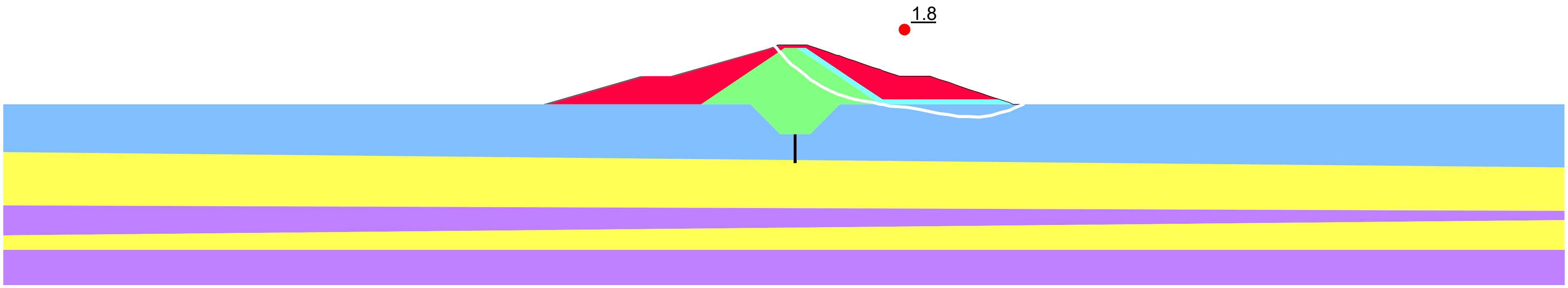

Figure 24 **Alternative 2 Peak Design Flood Factor of Safety— Walnut**

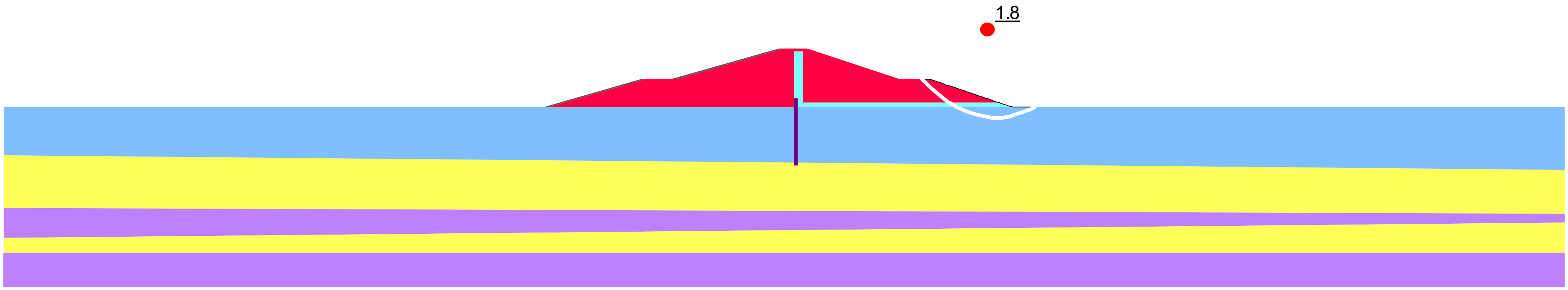

Figure 25 **Alternative 3 Peak Design Flood Factor of Safety— Walnut**

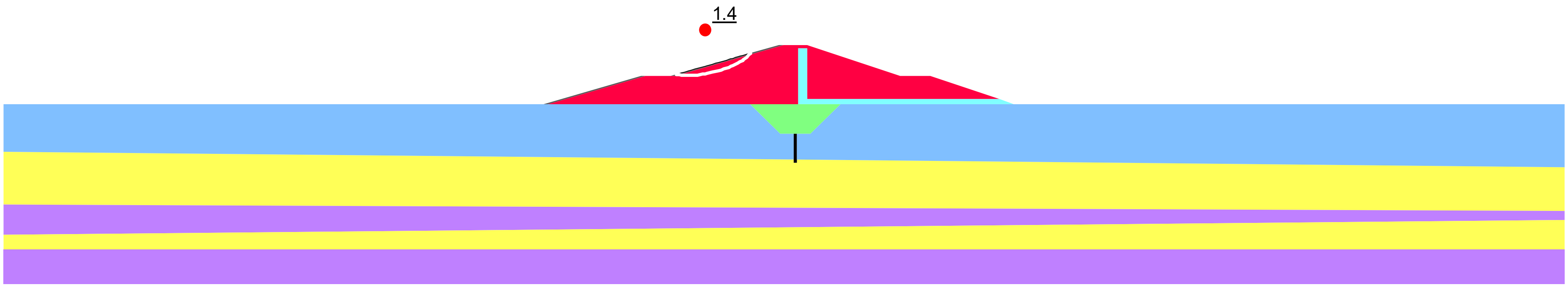

Figure 26 **Alternative 1 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Walnut**

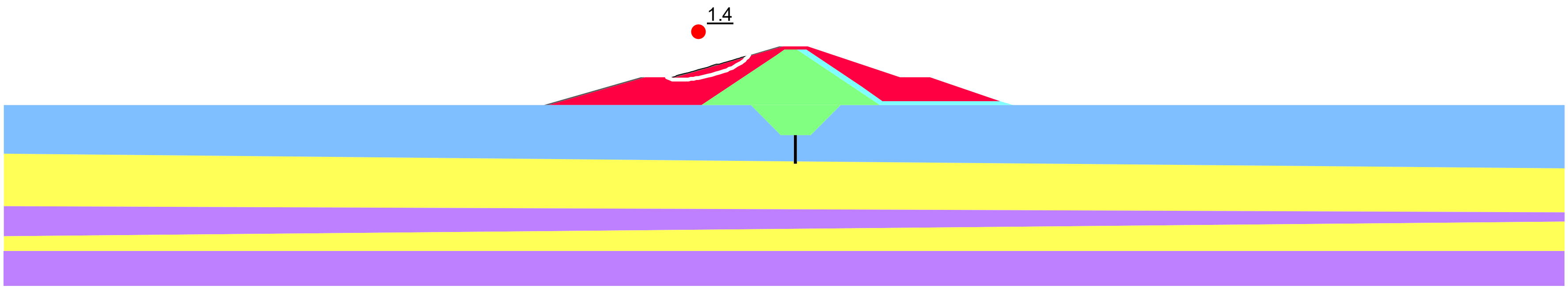

Figure 27 **Alternative 2 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Walnut**

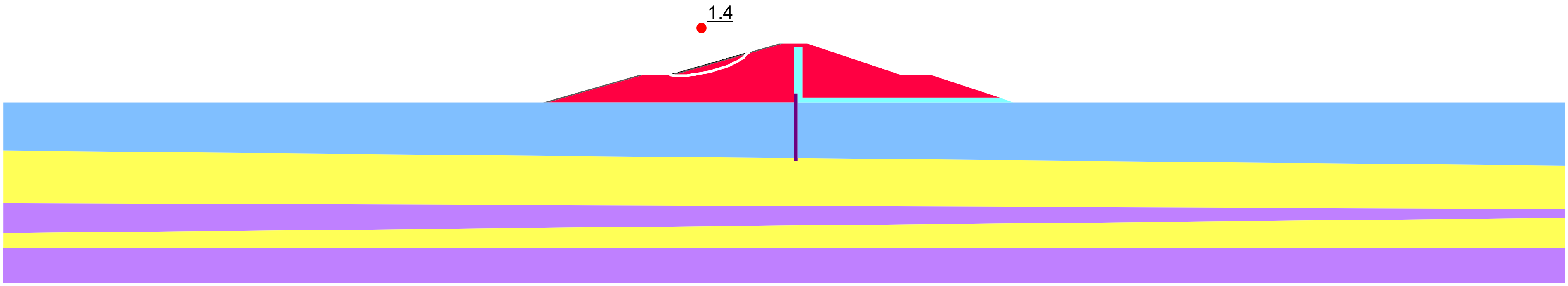

Figure 28 **Alternative 3 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Walnut**

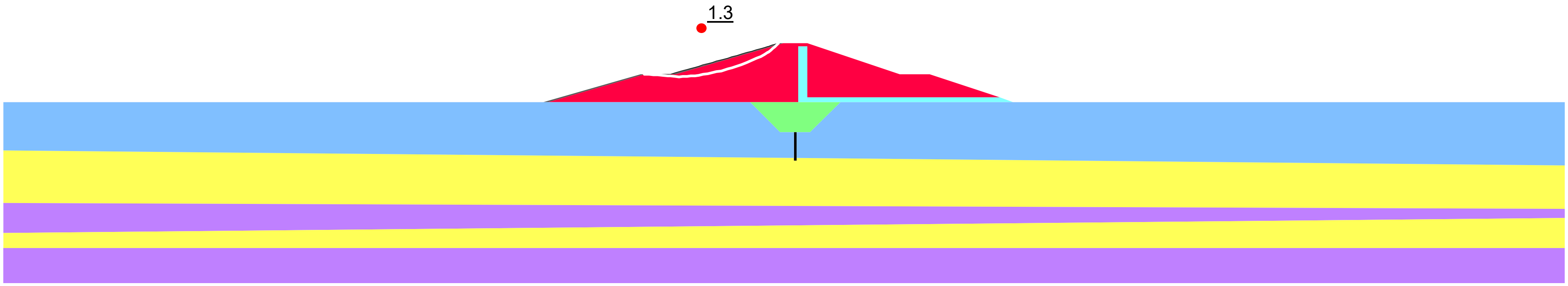

Figure 29 **Alternative 1 Rapid Drawdown from Peak Design Flood Factor of Safety— Walnut**

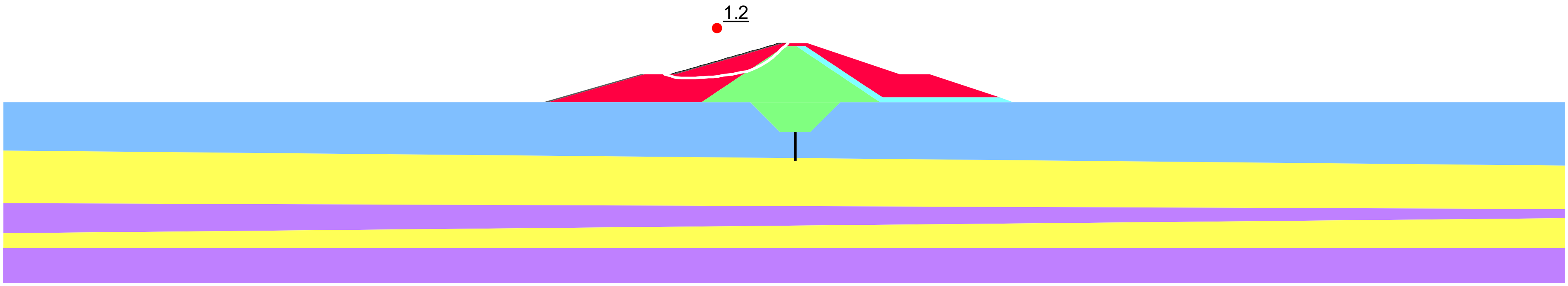

Figure 30 Alternative 2 Rapid Drawdown from Peak Design Flood Factor of Safety— Walnut

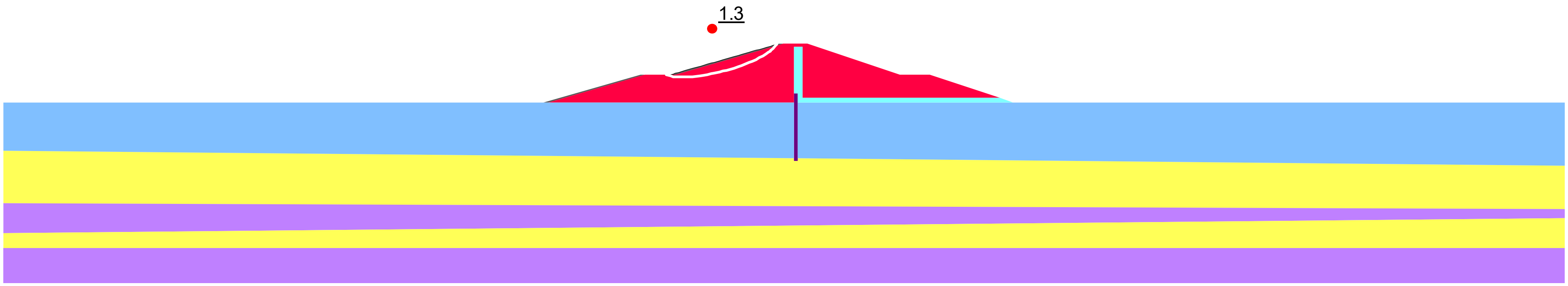

Figure 31 Alternative 3 Rapid Drawdown from Peak Design Flood Factor of Safety— Walnut

Attachment 2:

Critical Slip Surfaces from Static Stability Analysis— Birch

BLACK & VEATCH

Client SJRA
Project SCW Flood Control Dams **Unit**
Project No. 411500 **File No.**
Title Slope Stability Analysis

Computed By P. Turkson
Date 12/9/2024
Approved By David Bentler
Date 12/9/2024
Page 28

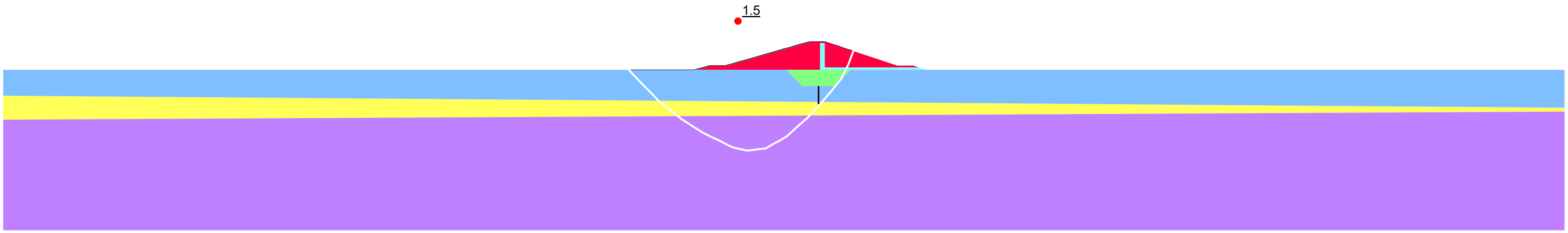

Figure 32 **Alternative 1 End of Construction Upstream Factor of Safety— Birch**

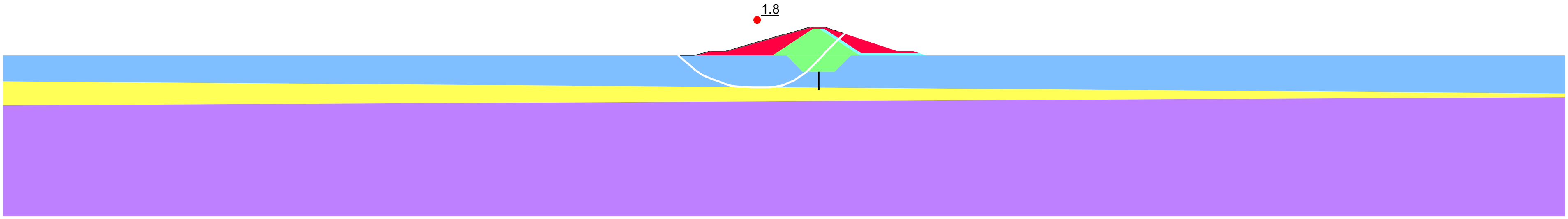

Figure 33 **Alternative 2 End of Construction Upstream Factor of Safety— Birch**

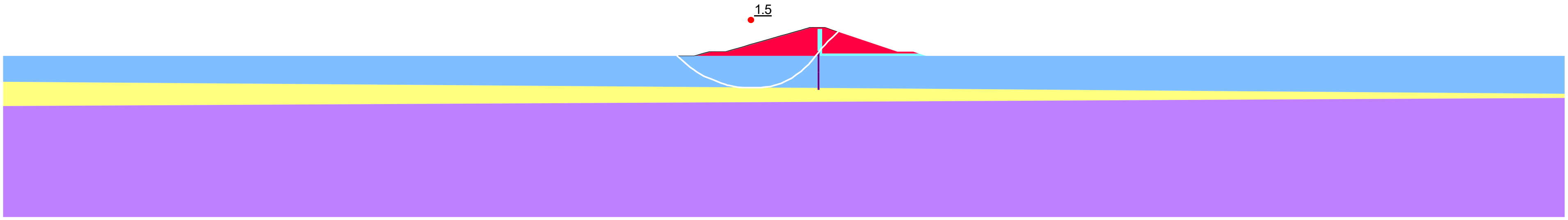

Figure 34 **Alternative 3 End of Construction Upstream Factor of Safety— Birch**

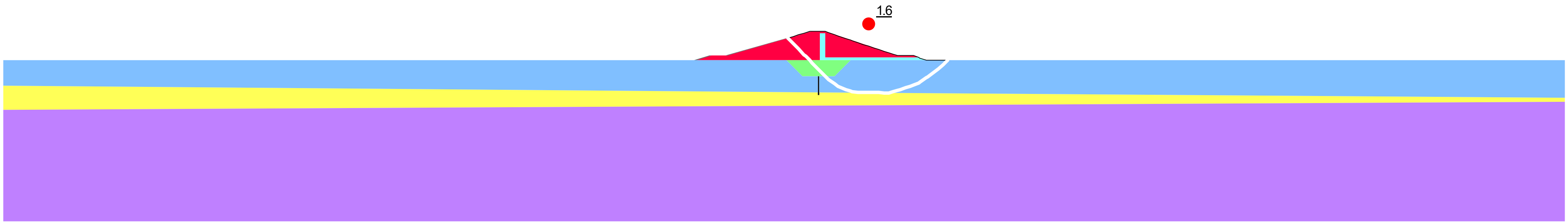

Figure 35 **Alternative 1 End of Construction Downstream Factor of Safety— Birch**

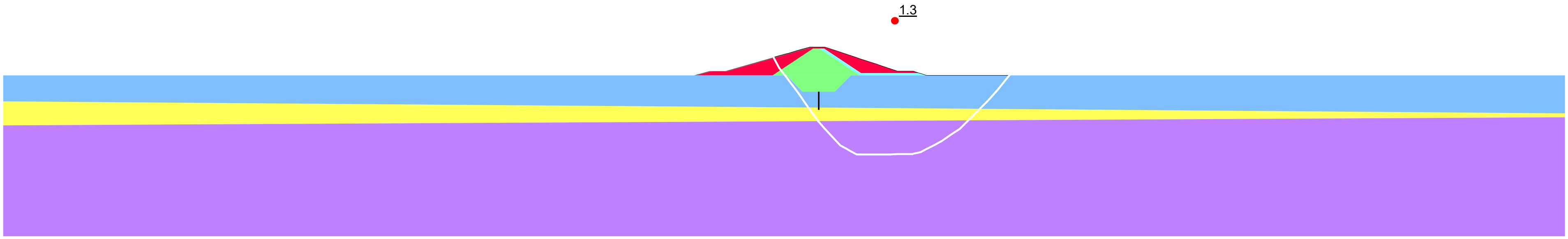

Figure 36 **Alternative 2 End of Construction Downstream Factor of Safety— Birch**

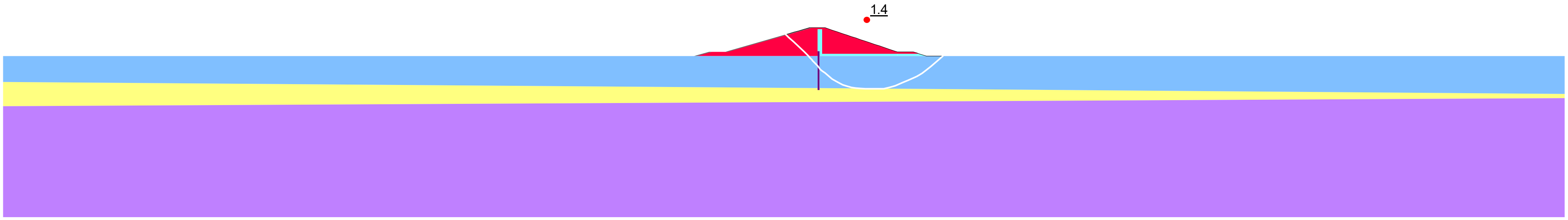

Figure 37 **Alternative 3 End of Construction Downstream Factor of Safety— Birch**

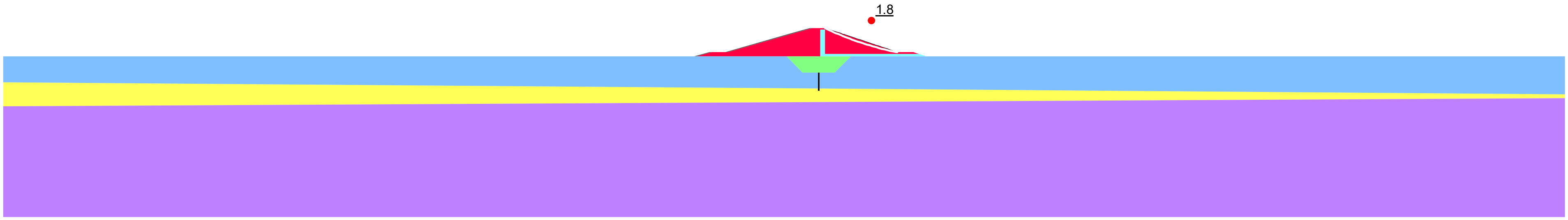

Figure 38 **Alternative 1 100-Year Peak Flood Factor of Safety— Birch**

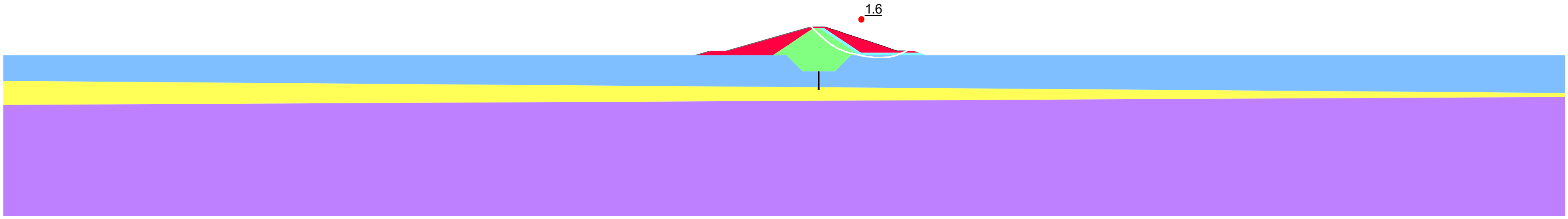

Figure 39 **Alternative 2 100-Year Peak Flood Factor of Safety— Birch**

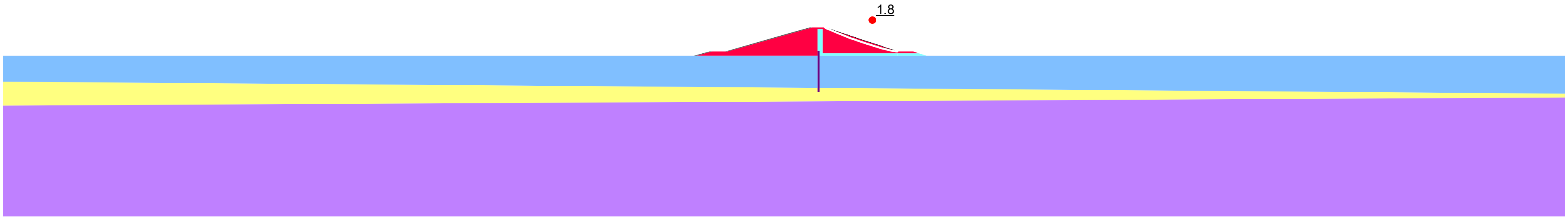

Figure 40 **Alternative 3 100-Year Peak Flood Factor of Safety— Birch**

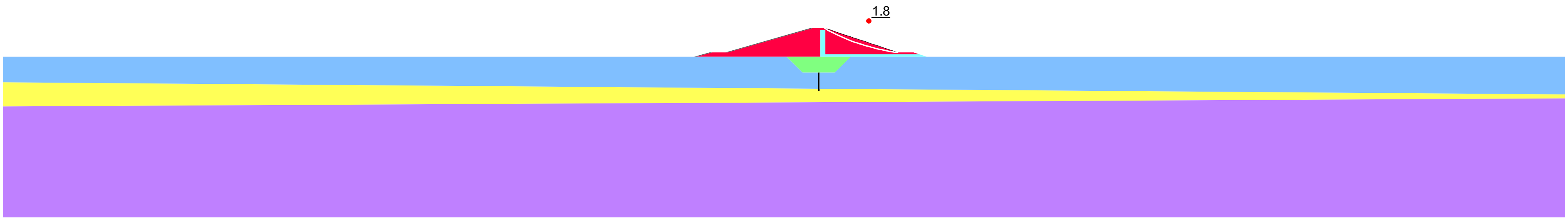

Figure 41 **Alternative 1 Peak Design Flood Factor of Safety— Birch**

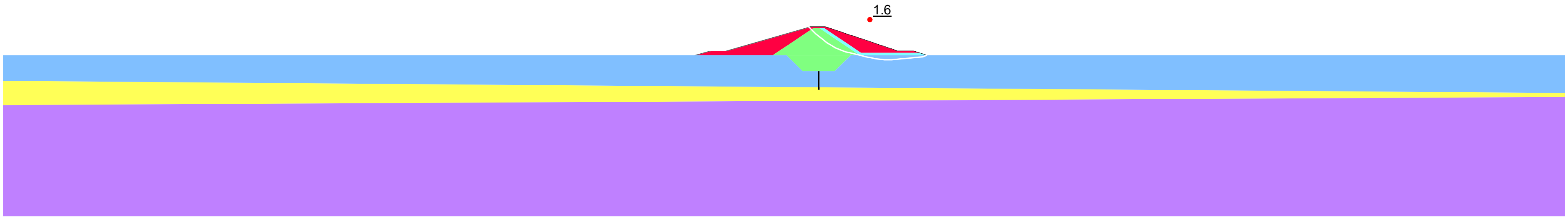

Figure 42 **Alternative 2 Peak Design Flood Factor of Safety— Birch**

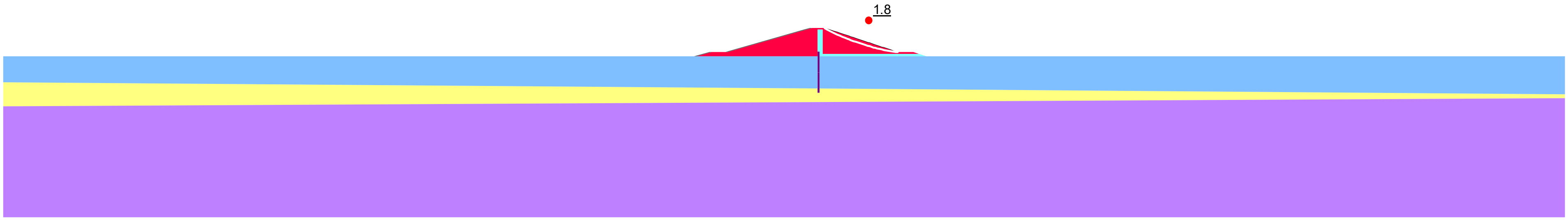

Figure 43 **Alternative 3 Peak Design Flood Factor of Safety— Birch**

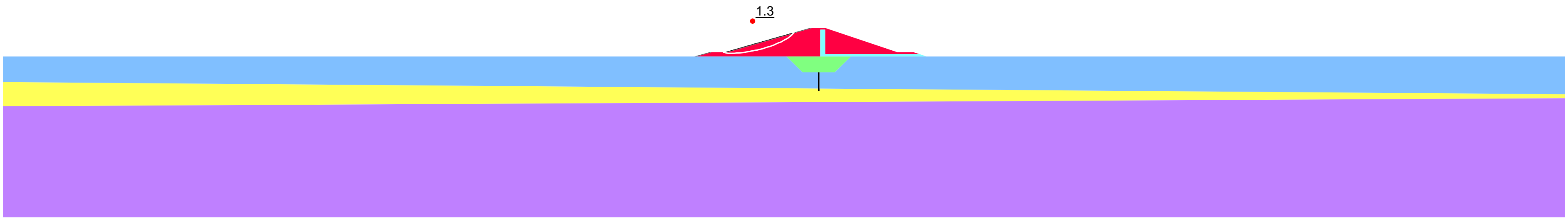

Figure 44 Alternative 1 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Birch

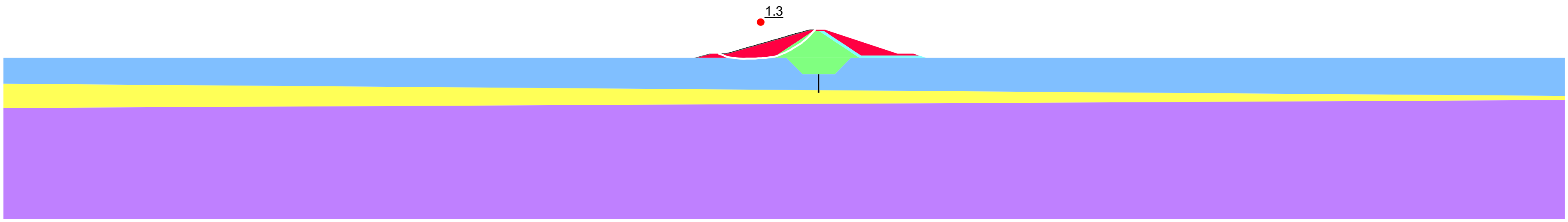

Figure 45 Alternative 2 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Birch

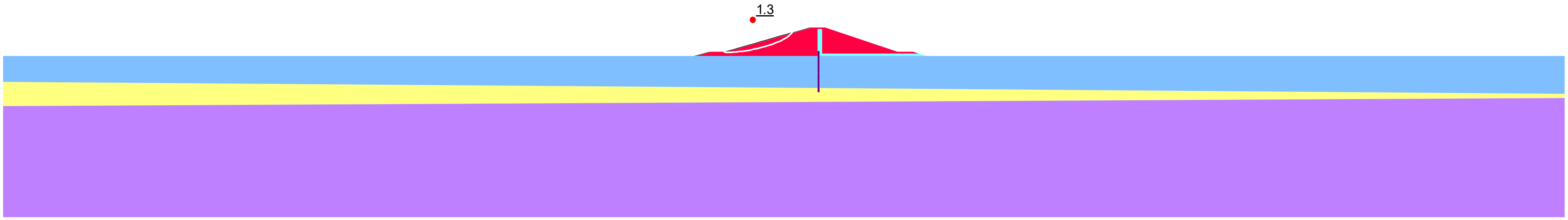

Figure 46 Alternative 3 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Birch

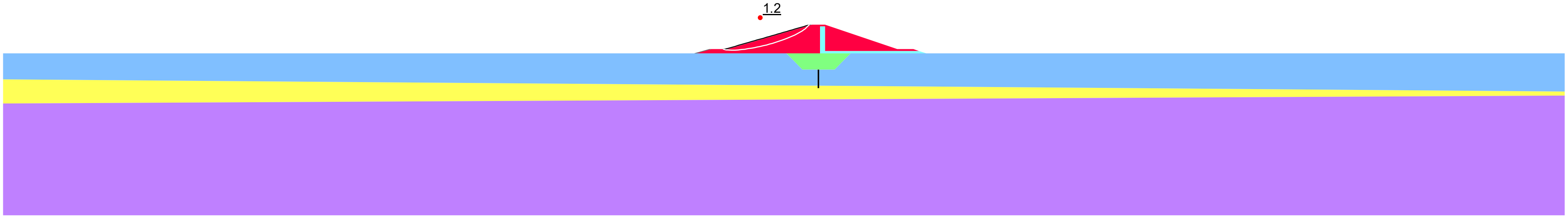

Figure 47 Alternative 1 Rapid Drawdown from Peak Design Flood Factor of Safety— Birch

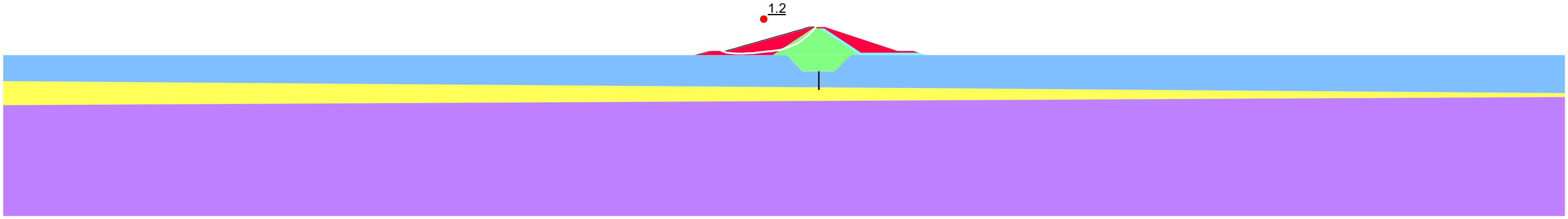

Figure 48 Alternative 2 Rapid Drawdown from Peak Design Flood Factor of Safety— Birch

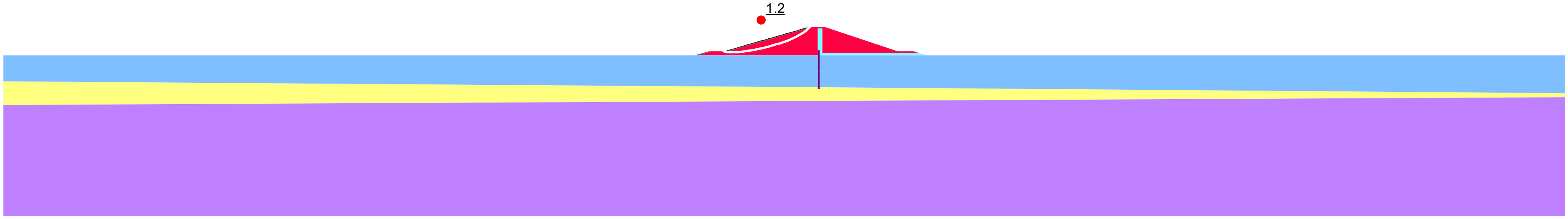

Figure 49 Alternative 3 Rapid Drawdown from Peak Design Flood Factor of Safety— Birch

Appendix B-6 Plans and profiles

WALNUT CREEK DAM

200' 100' 0 200' 400'
HORIZONTAL SCALE 1" = 200'

GENERAL NOTES:

1. THE RIPRAP SIZING AND EXTENTS UPSTREAM AND DOWNSTREAM OF THE SPILLWAY APRON SHOULD BE DETERMINED IN SUBSEQUENT ANALYSIS.
2. THE ASSOCIATED REPORT AND CALCULATIONS ARE DESCRIBED IN THE SPRING CREEK WATERSHED FLOOD CONTROL - CONCEPTUAL BASIS OF DESIGN REPORT. ALIGNMENT 2 IS SHOWN.
3. THE ELEVATION-STORAGE CURVE AND BORROW REQUIREMENTS WERE DERIVED FROM 2018 UPPER COAST LIDAR DATA.
4. VERTICAL DATUM FOR ALL ELEVATIONS ARE IN NAVD88.

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Black & Veatch Corporation
Kansas City, Missouri

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RIVER
AUTHORITY
(SJRA)

SPRING CREEK
WATERSHED FEASIBILITY
STUDY

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DETAILED:	GLC
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APPROVED:	
DATE:	MM/DD/YYYY
PROJECT NO.:	411500

CIVIL

WALNUT CREEK DAM
PLAN

C-00-101 OF

(SCALE BAR IS 4" AT FULL SCALE) 0 1/2 1 2 3 4

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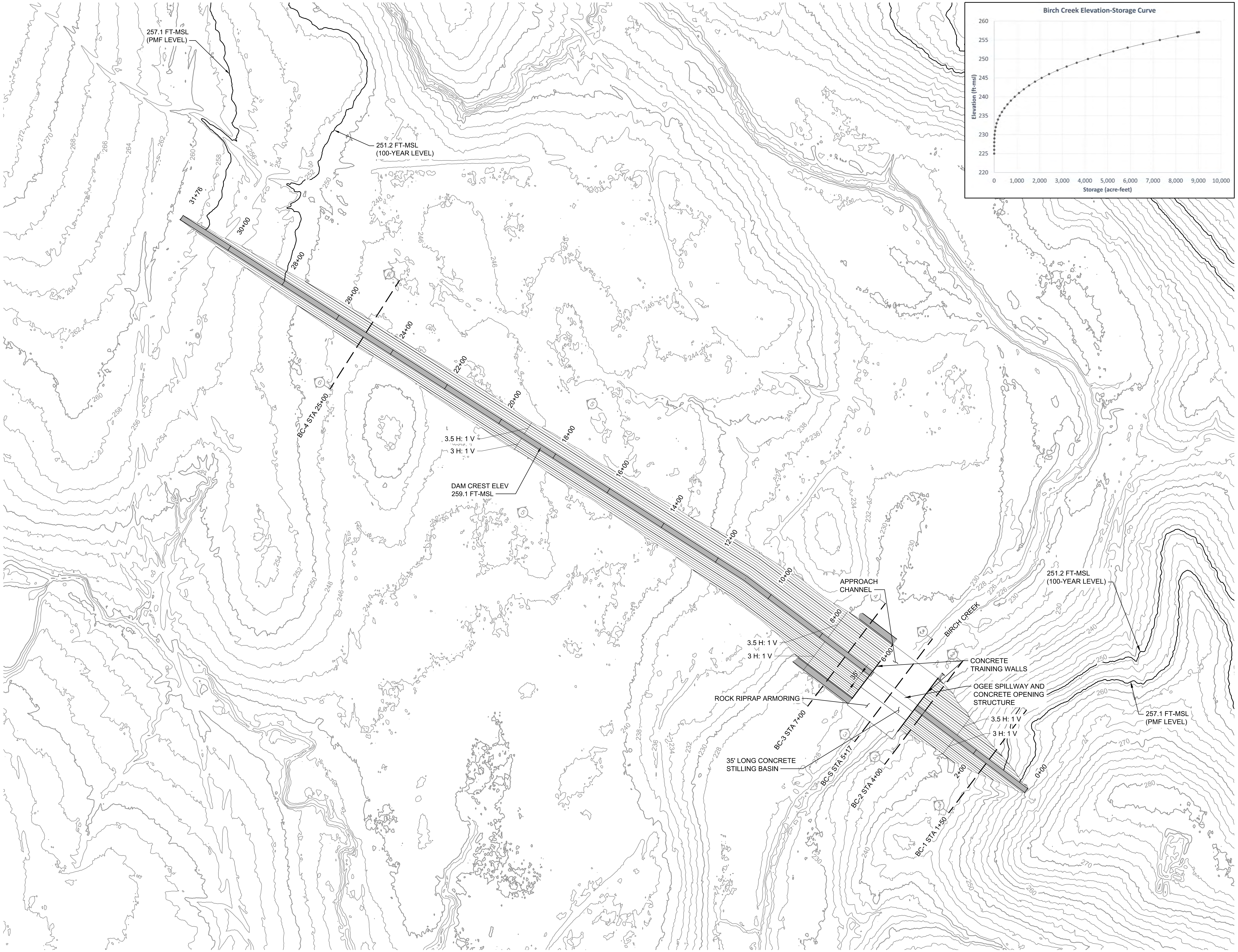

BIRCH CREEK DAM

150' 75' 0 150' 300'
HORIZONTAL SCALE 1" = 150'

GENERAL NOTES:

1. THE RIPRAP SIZING AND EXTENTS UPSTREAM AND DOWNSTREAM OF THE SPILLWAY APRON SHOULD BE DETERMINED IN SUBSEQUENT ANALYSIS.
2. THE ASSOCIATED REPORT AND CALCULATIONS ARE DESCRIBED IN THE SPRING CREEK WATERSHED FLOOD CONTROL - CONCEPTUAL BASIS OF DESIGN REPORT. ALIGNMENT 3 IS SHOWN.
3. THE ELEVATION-STORAGE CURVE AND BORROW REQUIREMENTS WERE DERIVED FROM 2018 UPPER COAST LIDAR DATA.
4. VERTICAL DATUM FOR ALL ELEVATIONS ARE IN NAVD88.

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BIRCH CREEK DAM PLAN

C-00-102 OF

(SCALE BAR IS 4" AT FULL SCALE) 0 1/2 1 2 3 4

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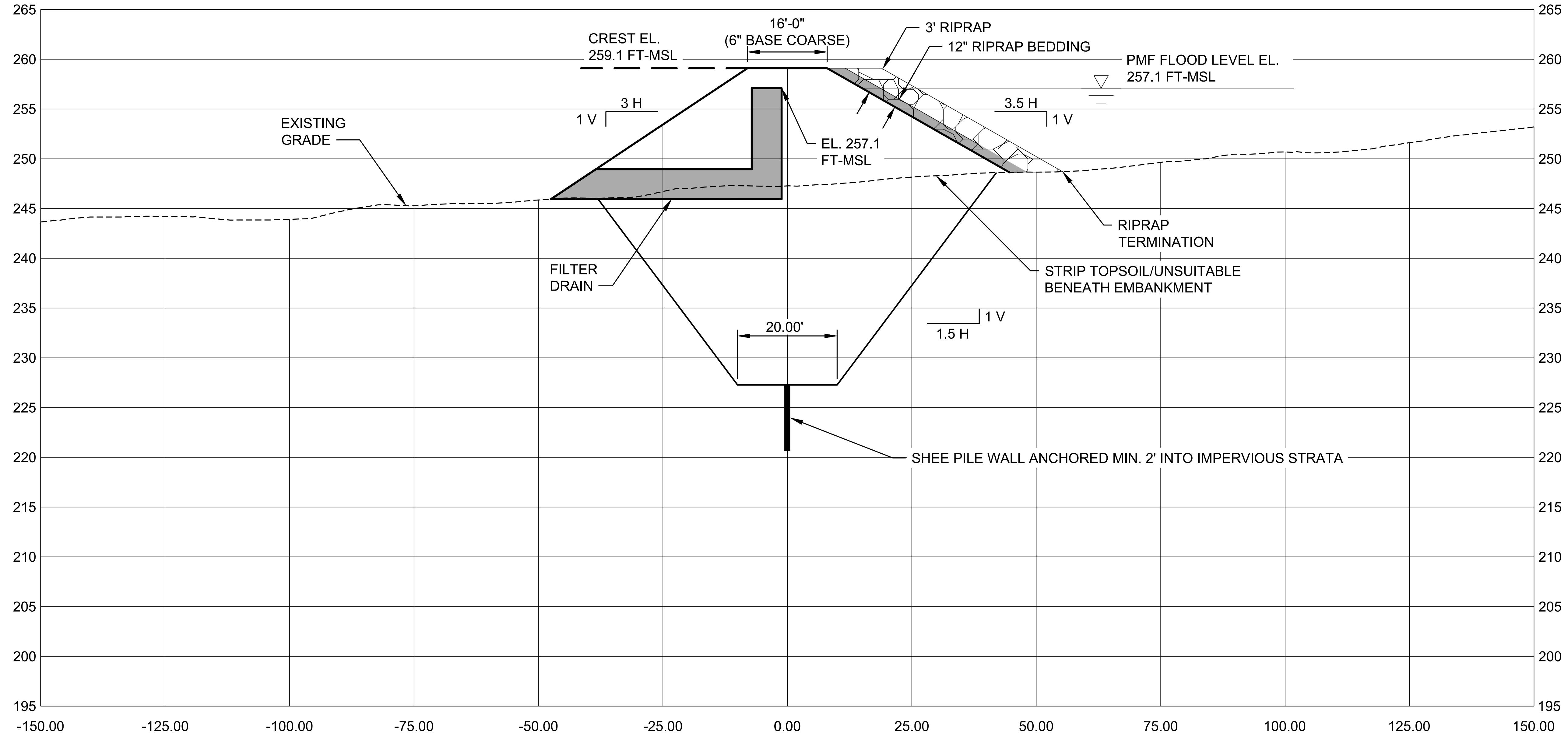
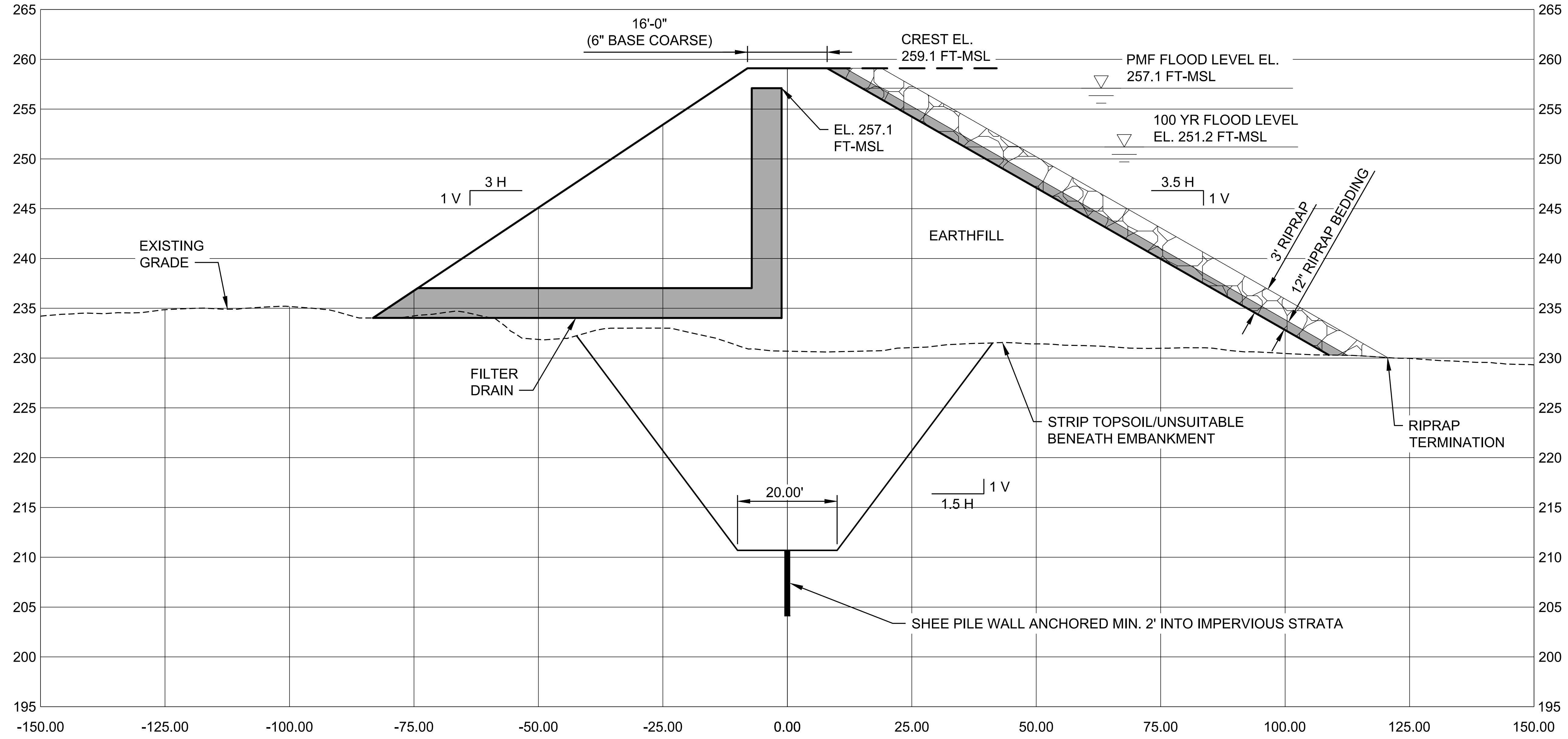


BC-2
SECTION VIEW - STA 4+00



10' 0 10' 20'
VERTICAL SCALE 1"=10'

20' 10' 0 20' 40'
HORIZONTAL SCALE 1"=20'

GENERAL NOTES:

1. VERTICAL DATUM FOR ALL ELEVATIONS ARE IN NAVD88.

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BIRCH CREEK

CIVIL

EMBANKMENT SECTIONS
ALTERNATIVE 1

C-00-201 OF

(SCALE BAR IS 4" AT FULL SCALE) 0 1/2 1 2 3 4

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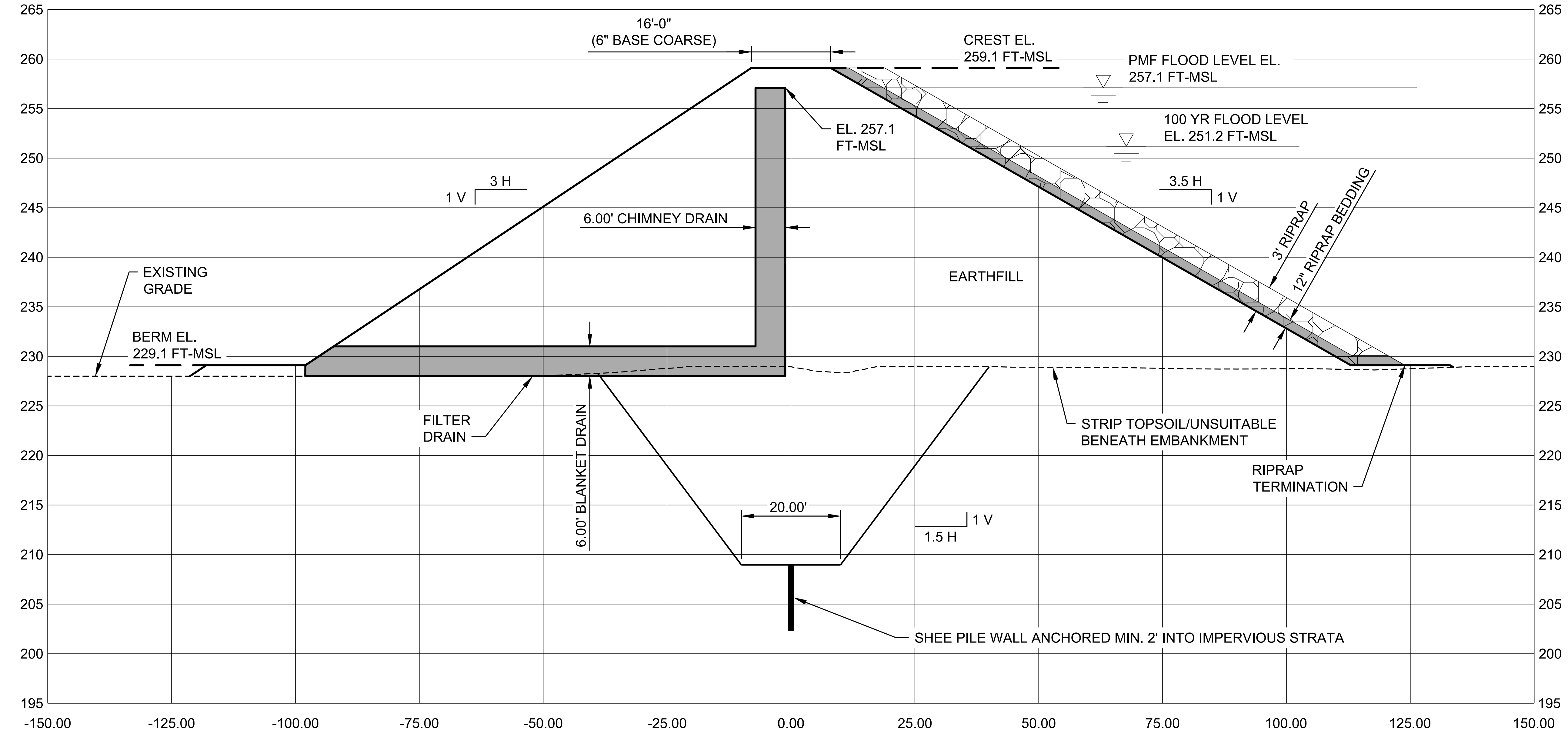
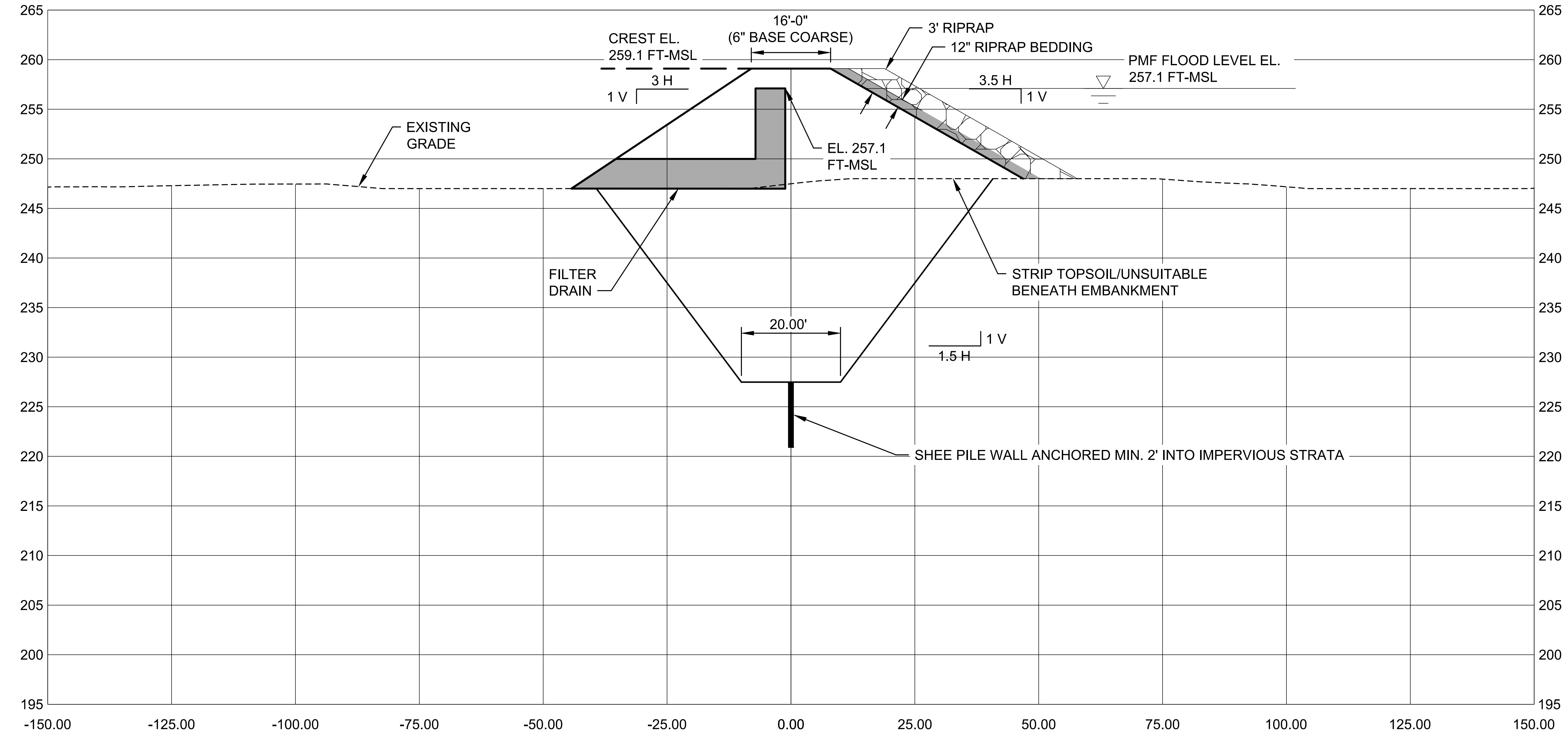
BC-3
SECTION VIEW - STA 7+00



BC-4
SECTION VIEW - STA 25+00



10' 0 10' 20'
VERTICAL SCALE 1"=10'

20' 10' 0 20' 40'
HORIZONTAL SCALE 1"=20'

GENERAL NOTES:

1. VERTICAL DATUM FOR ALL ELEVATIONS ARE IN NAVD88.

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BIRCH CREEK

CIVIL

EMBANKMENT SECTIONS
ALTERNATIVE 1

C-00-202 OF

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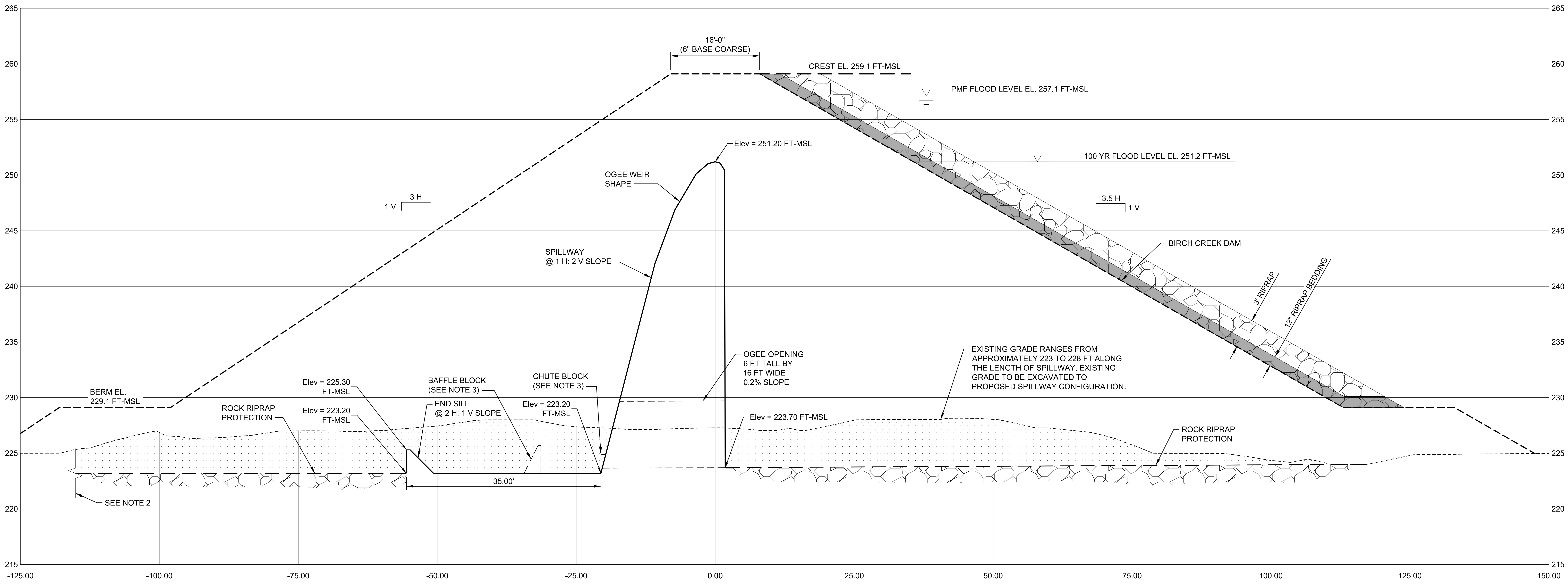
5' 3' 0 5' 10'
VERTICAL SCALE 1"=5'

10' 0 10' 20'
HORIZONTAL SCALE 1"=10'

GENERAL NOTES:

1. VERTICAL DATUM FOR ALL ELEVATIONS ARE IN NAVD88.
2. THE RIPRAP SIZING AND EXTENTS UPSTREAM AND DOWNSTREAM OF THE SPILLWAY APRON SHOULD BE DETERMINED IN SUBSEQUENT ANALYSIS.
3. THE CHUTE AND BAFFLE BLOCKS DIMENSIONS ARE AS FOLLOWS. THE CHUTE BLOCK IS 1.7-FT BY 1.7-FT WIDE, WITH A 1.7-FT LATERAL SPACING. THE BAFFLE BLOCK IS 10.8-FT DOWNSTREAM DOWNSTREAM OF THE CHUTE BLOCK AND IS 2.5-FT TALL BY 1.9-FT WIDE, WITH A 1.9-FT LATERAL SPACING. THE DIMENSIONS FOLLOW TYPE III DESIGN OF SMALL DAMS BASIN GEOMETRY.

BC-S
SECTION VIEW - STA 5+17



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BIRCH CREEK

CIVIL

SPILLWAY SECTION ALTERNATIVE 1

C-00-203 OF

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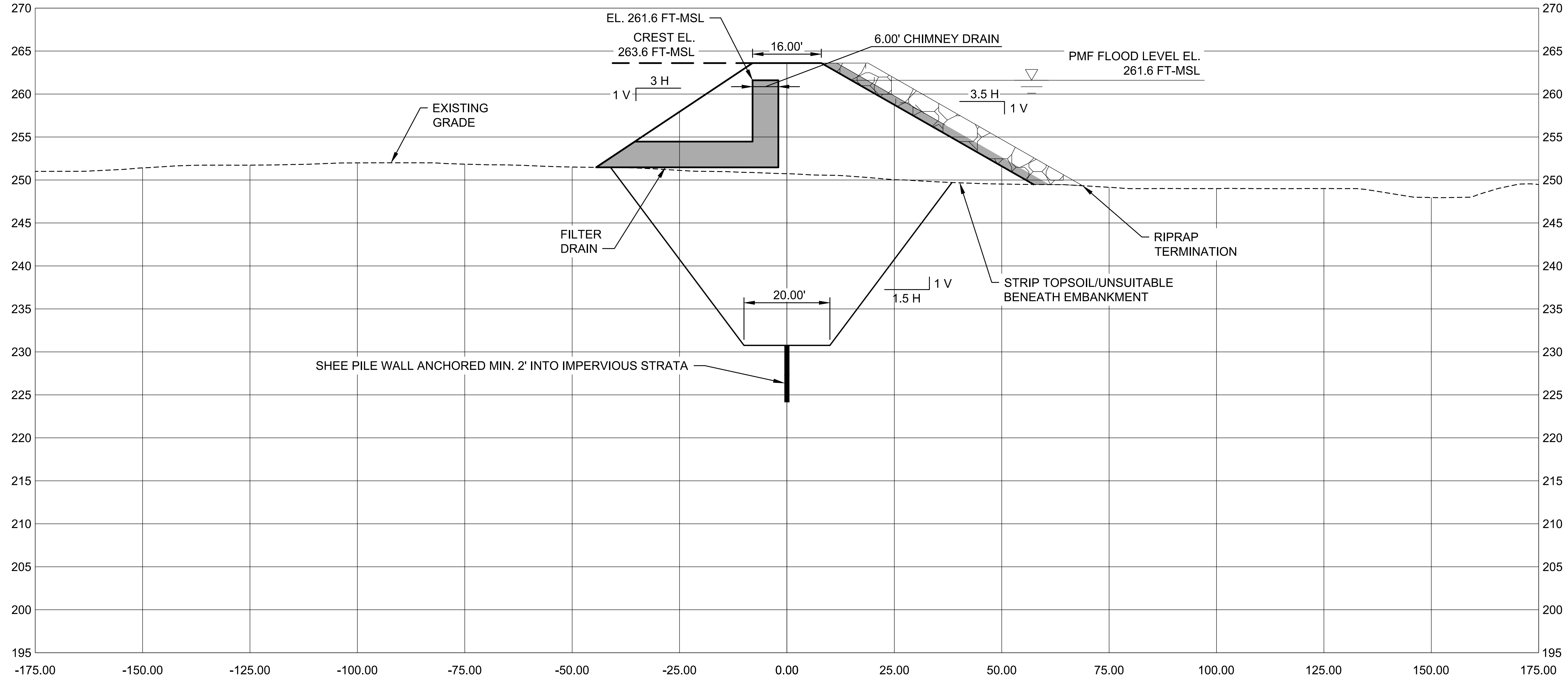
WC-1
SECTION VIEW - STA 6+00



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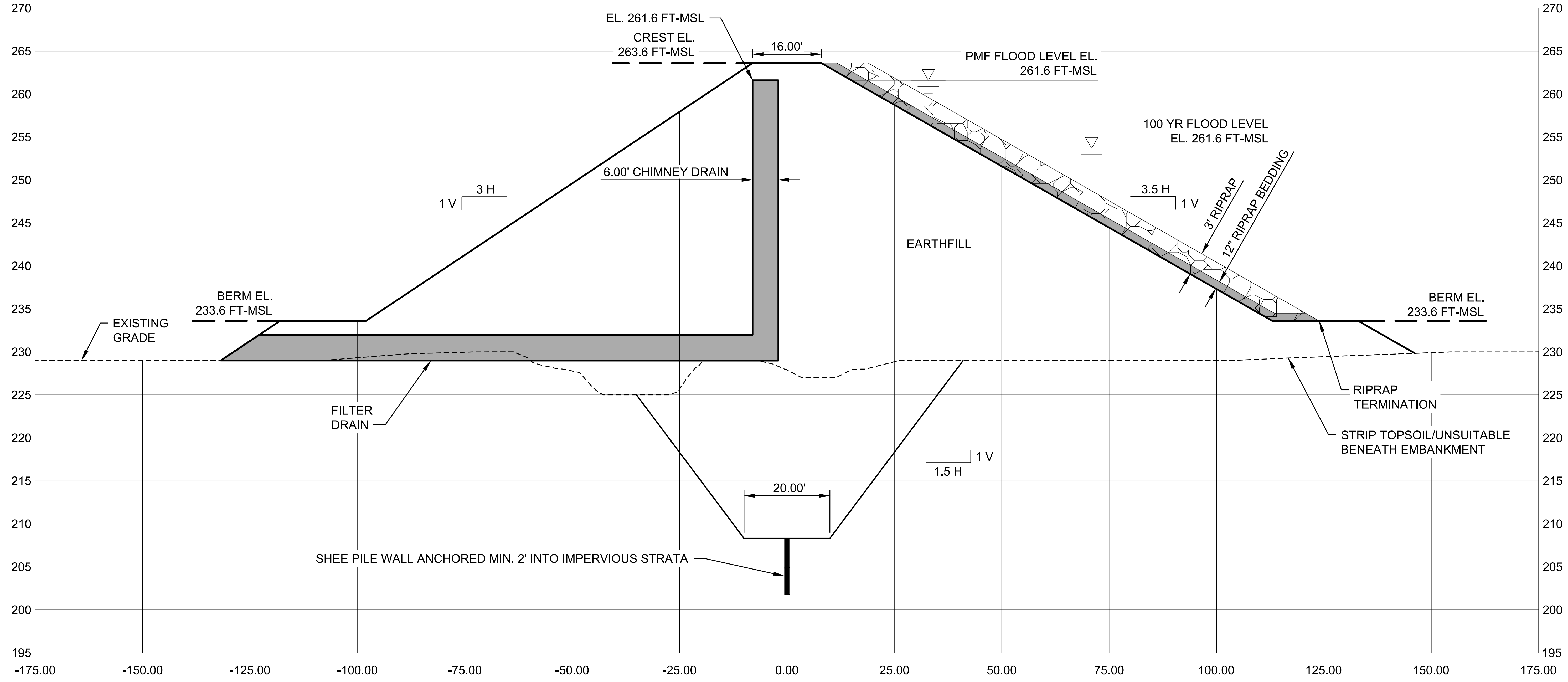
SECTION VIEW - STA 22+00



10' 0 10' 20'
VERTICAL SCALE 1"=10'

20' 10' 0 20' 40'
HORIZONTAL SCALE 1"=20'

GENERAL NOTES:

1. VERTICAL DATUM FOR ALL ELEVATIONS ARE IN NAVD88.

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WALNUT CREEK

CIVIL

EMBANKMENT SECTIONS

C-00-204 OF

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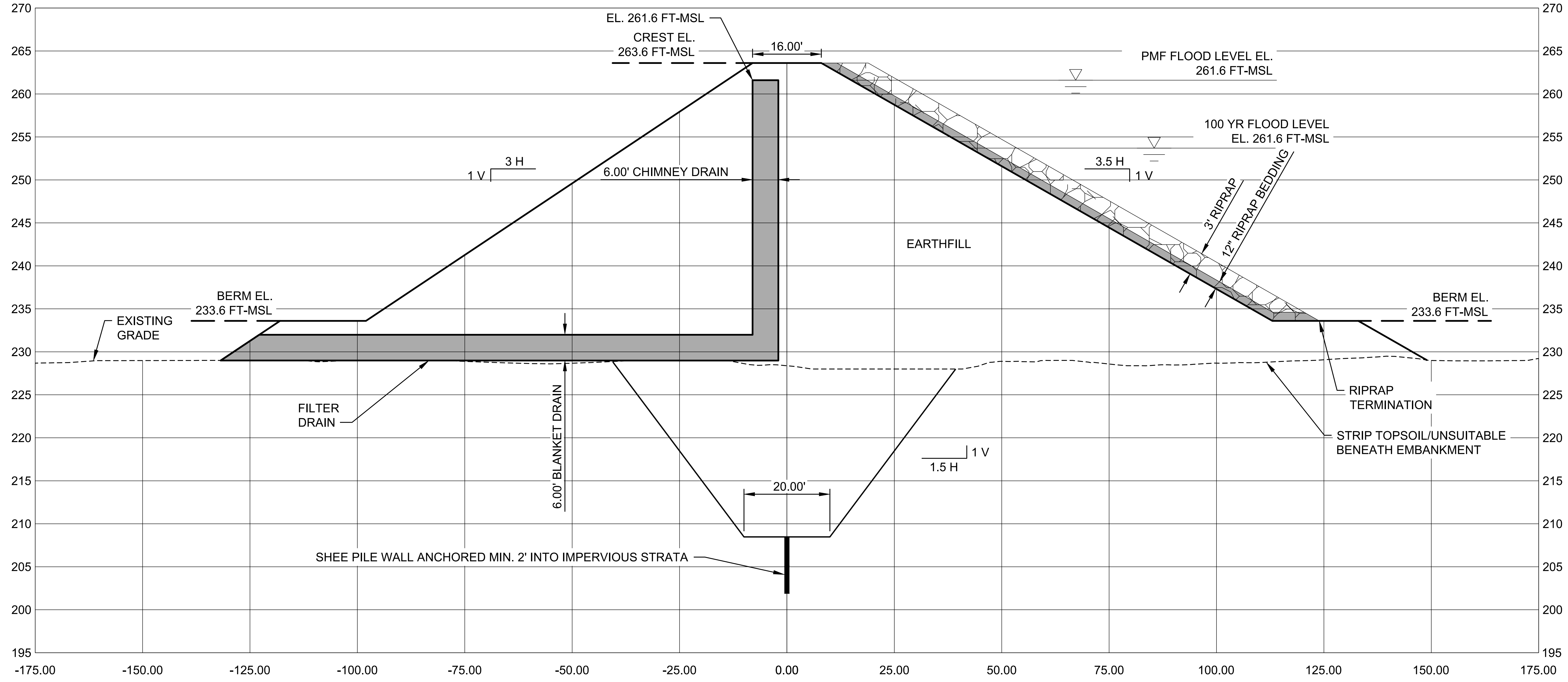
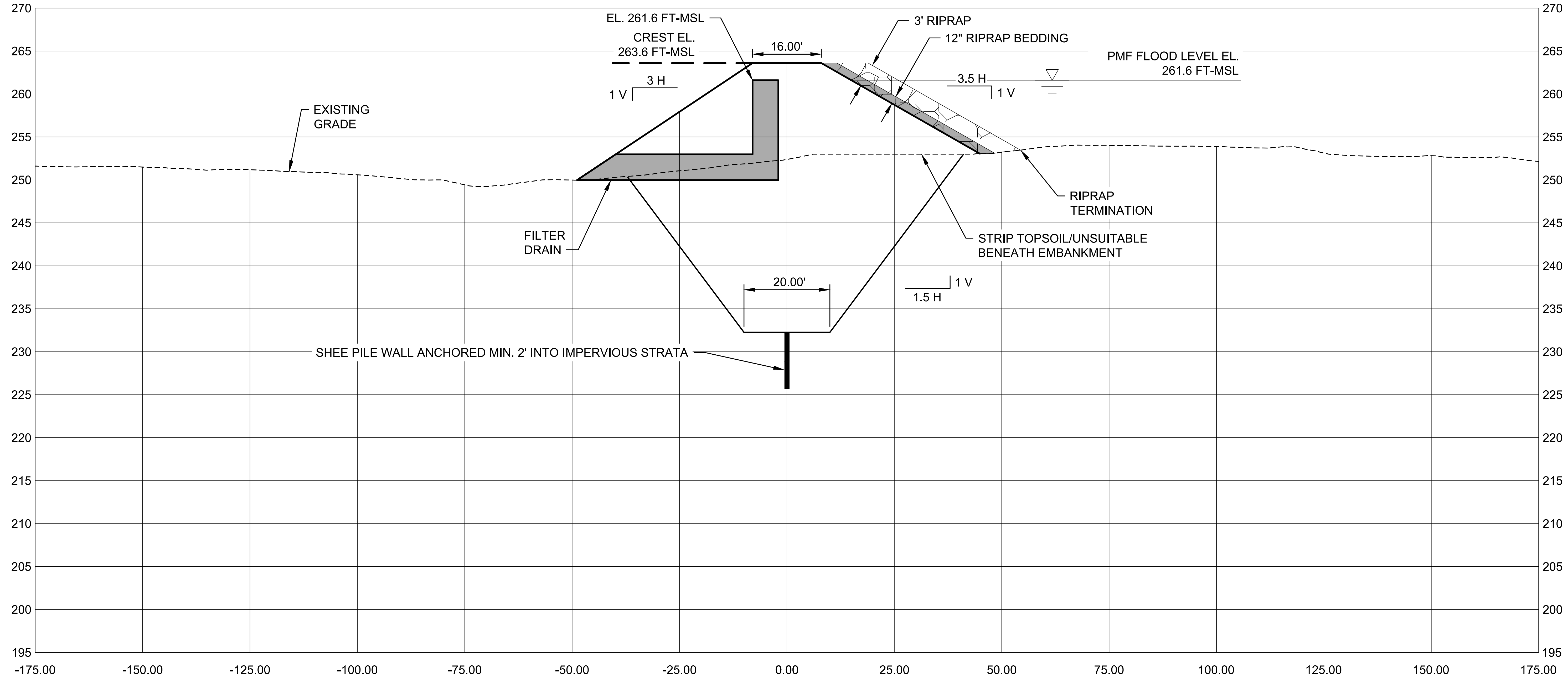
WC-3
SECTION VIEW - STA 24+50



WC-4
SECTION VIEW - STA 30+00



10' 0 10' 20'
VERTICAL SCALE 1"=10'

20' 10' 0 20' 40'
HORIZONTAL SCALE 1"=20'

GENERAL NOTES:

1. VERTICAL DATUM FOR ALL ELEVATIONS ARE IN NAVD88.

BLACK & VEATCH

Black & Veatch Corporation
Kansas City, Missouri

SAN JACINTO
RIVER
AUTHORITY
(SJRA)

SPRING CREEK
WATERSHED FEASIBILITY
STUDY

PRELIMINARY
NOT FOR
CONSTRUCTION

REVISIONS AND RECORD OF ISSUE

DESIGNED:
DETAILED: GLC
CHECKED:
APPROVED:
DATE: MM/DD/YYYY
PROJECT NO.: 411500

WALNUT CREEK

CIVIL

EMBANKMENT SECTIONS

C-00-205 OF

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FD11000
D11000

(SCALE BAR IS 4" AT FULL SCALE) 0 1/2 1 2 3 4

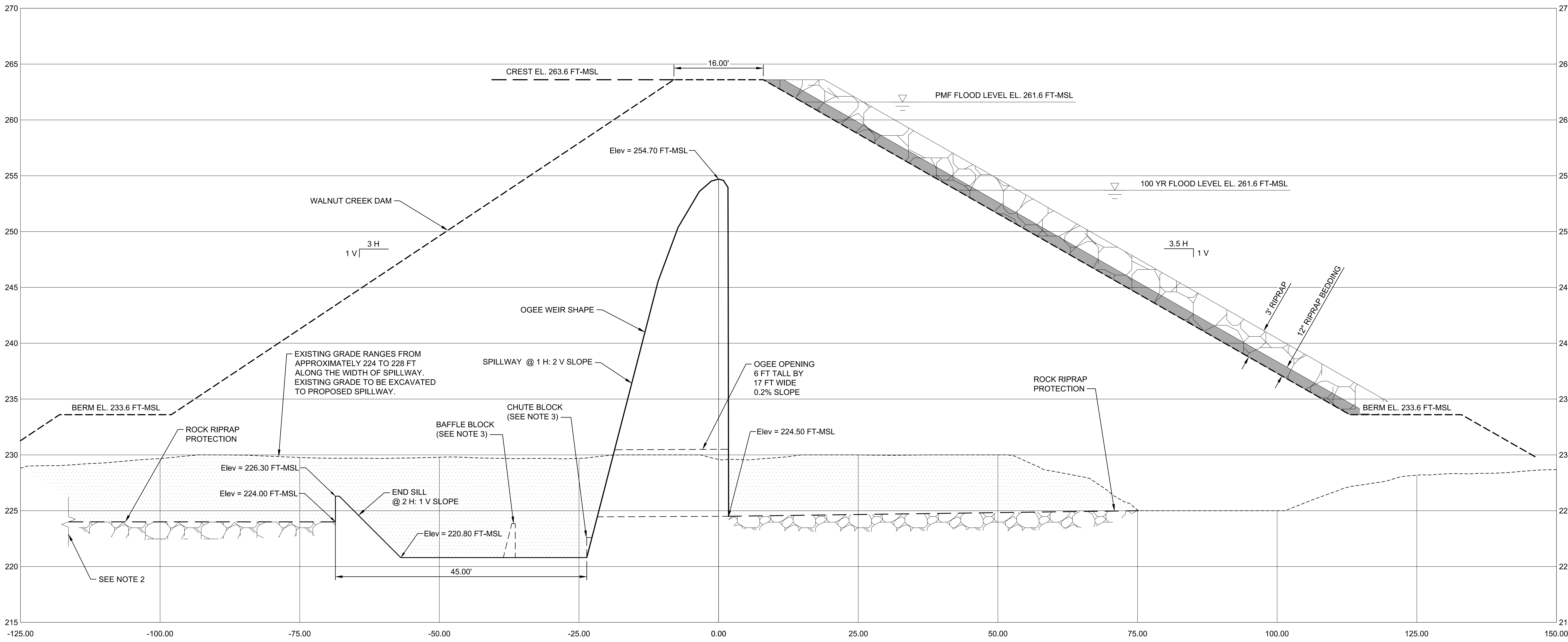
5' 3' 0 5' 10'
VERTICAL SCALE 1"=5'

10' 0 10' 20'
HORIZONTAL SCALE 1"=10'

GENERAL NOTES:

1. VERTICAL DATUM FOR ALL ELEVATIONS ARE IN NAVD88.
2. THE RIPRAP SIZING AND EXTENTS UPSTREAM AND DOWNSTREAM OF THE SPILLWAY APRON SHOULD BE DETERMINED IN SUBSEQUENT ANALYSIS.
3. THE CHUTE AND BAFFLE BLOCKS DIMENSIONS ARE AS FOLLOWS. THE CHUTE BLOCK IS 1.8-FT BY 1.8-FT WIDE, WITH A 1.8-FT LATERAL SPACING. THE BAFFLE BLOCK IS 12.8-FT DOWNSTREAM DOWNSTREAM OF THE CHUTE BLOCK AND IS 3.0-FT TALL BY 2.3-FT WIDE, WITH A 2.3-FT LATERAL SPACING. THE DIMENSIONS FOLLOW TYPE III DESIGN OF SMALL DAMS BASIN GEOMETRY.

WC-S
SECTION VIEW - STA 23+44



BLACK & VEATCH

Black & Veatch Corporation
Kansas City, Missouri

SAN JACINTO RIVER AUTHORITY (SJRA)

SPRING CREEK WATERSHED FEASIBILITY STUDY

PRELIMINARY NOT FOR CONSTRUCTION

REVISIONS AND RECORD OF ISSUE

DESIGNED:	
DETAILED:	GLC
CHECKED:	
APPROVED:	
DATE:	MM/DD/YYYY
PROJECT NO.:	411500

WALNUT CREEK

CIVIL

SPILLWAY SECTION

C-00-206 OF

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FD11000
D11000

(SCALE BAR IS 4" AT FULL SCALE) 0 1/2 1 2 3 4

Appendix B-7 Foundation treatment modification

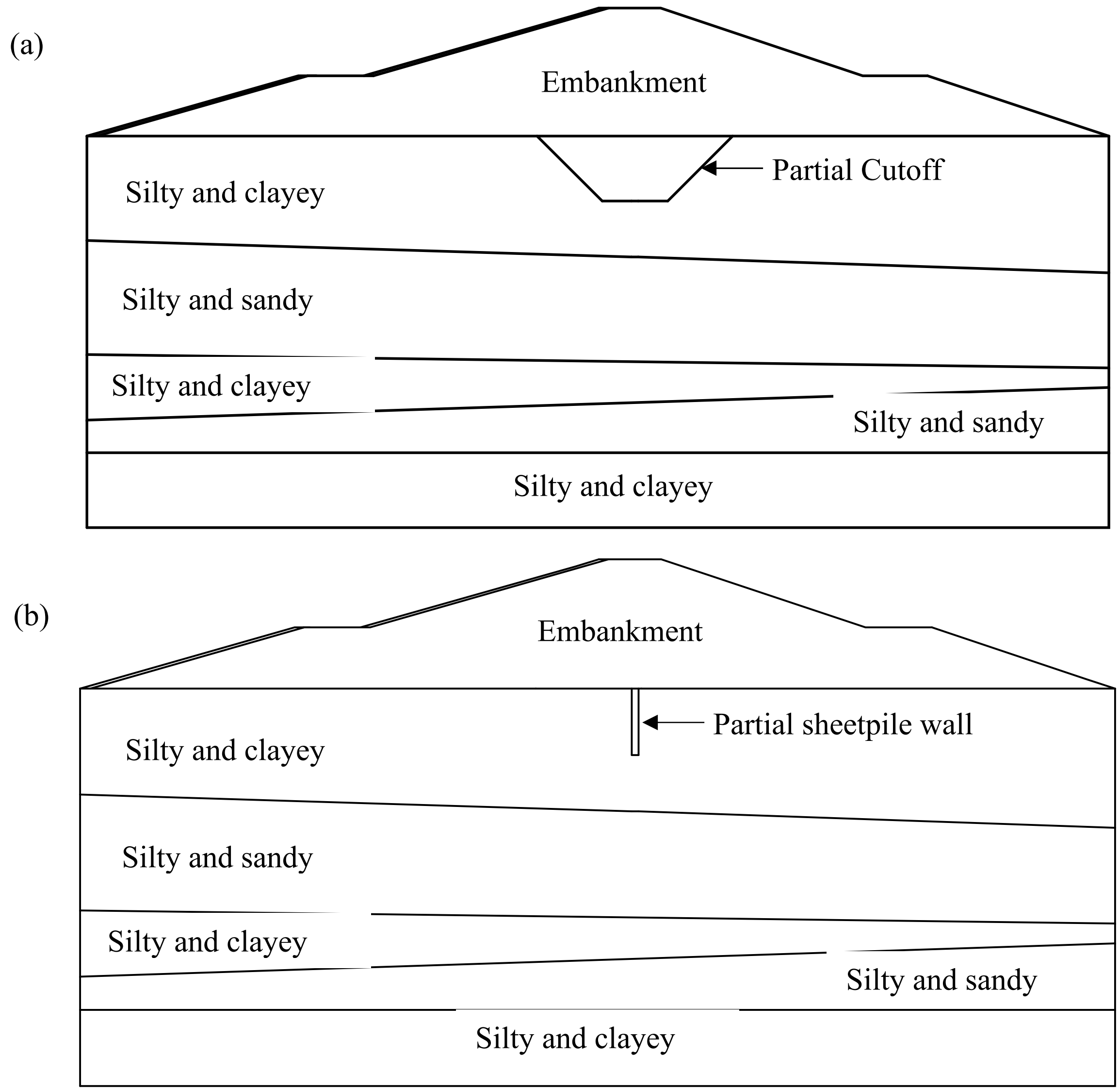

Figure G-1 Cutoff Trench with Sheet Pile Wall Foundation Modification: (a) Partial Cutoff Trench and (b) Partial Sheetpile Wall

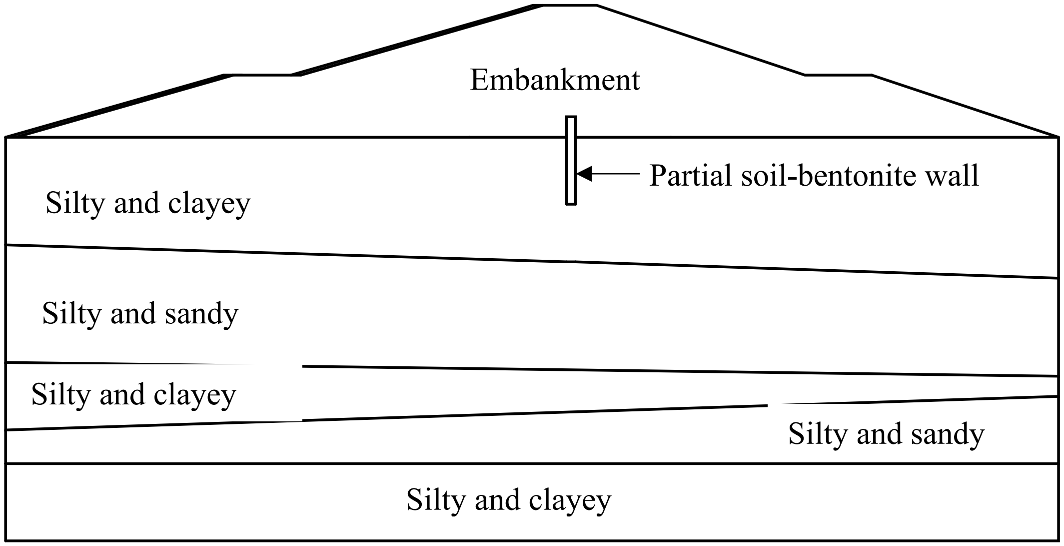

Figure G-2 **Soil-Bentonite Cutoff Wall Foundation Modification**

Appendix B-8 Elevation-storage curves

Walnut Creek Elevation-Area-Storage Data

Elevation (ft-msl)	Area (acres)	Storage (acre-feet)
224.5	0	0
225.5	0	0
226.5	1	1
227.5	1	2
228.5	3	3
229.5	8	8
230.5	21	21
231.5	37	51
232.5	51	95
233.5	65	153
234.5	78	224
235.5	91	308
236.5	107	408
237.5	122	522
238.5	138	652
239.5	156	799
240.5	175	965
241.5	197	1,150
242.5	223	1,360
243.5	250	1,597
244.5	279	1,862
245.5	309	2,155
246.5	344	2,481
247.5	381	2,844
248.5	418	3,243
249.5	456	3,680
250.5	499	4,157
251.5	546	4,679
252.5	594	5,248
253.5	646	5,869
254.5	699	6,541
255.5	753	7,267
256.5	815	8,050
257.5	881	8,898
258.5	948	9,812
259.5	1,021	10,797
260.5	1,101	11,857

Elevation (ft-msl)	Area (acres)	Storage (acre-feet)
261.5	1,187	13,001
261.6	1,196	13,124

Birch Creek Elevation-Area-Storage Data

Elevation (ft-msl)	Area (acres)	Storage (acre-feet)
225	0	0
226	1	1
227	2	2
228	4	5
229	7	10
230	14	20
231	25	40
232	34	69
233	49	109
234	69	168
235	87	246
236	106	342
237	122	456
238	140	586
239	160	736
240	180	906
241	199	1,095
242	222	1,305
243	247	1,539
244	276	1,800
245	310	2,092
246	348	2,421
247	387	2,787
248	428	3,193
249	471	3,642
250	515	4,134
251	559	4,671
252	605	5,252
253	656	5,882
254	710	6,564
255	766	7,301
256	818	8,092
257	875	8,937
257.1	880	9,025

Appendix C:
Cost Analysis Appendix

Spring Creek Watershed Flood Control Dams Conceptual Engineering Feasibility Study

Cost Analysis Appendix

Flood Infrastructure Fund Category 1

Project ID 21-0016

Prepared for:
Texas Water Development Board

Prepared by:
Halff
Levi Hein, P.E., CFM
Rebekah Franz, P.E., CFM
Andrew Moore, P.E., CFM

Table of Contents

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1 Introduction

Following the conceptual design of the detention basins, a cost estimate was developed to determine the potential costs for acquiring land, as well as constructing and maintaining each facility. The costs presented include the opinion of probable construction costs, land acquisition, utility conflicts, and annual maintenance.

1.1 Scope of work

The scope of work for the probable project cost analysis included the following:

- Create Class 4 OPCC estimates for two recommended detention basins, with an accuracy range of -30% to +50% as per AACE International standards. Use comparative costs to differentiate between alternatives at each site, without preparing a complete OPCC for each alternative.
- Estimate land costs based on a combination of land acquisition and flood easement acquisition at various flood pool elevations, using local market prices.
- Use publicly available information to screen utilities in the general locations of the two proposed sites.
- Assume all roads, cemeteries, and utilities will be relocated or raised outside the 100-year flood pool, with houses and buildings purchased and demolished. Estimate costs for utility conflicts and relocations.
- Develop an estimate of environmental mitigation cost based on current regional bank credit costs or similar projects.
- Include annual operations, maintenance, and financing costs over a 30-year period with financing and additional 20 years without financing costs, accounting for environmental permitting schedules.

2 Construction costs

2.1 Cost analysis

The cost estimate totals for both the Walnut Creek and Birch Creek detention basins include all labor, materials and equipment to reflect the current scope of work as defined by the received documents detailed in Basis of Estimate Section of this report. The estimates reflect the preliminary nature of the projects, and costs have been derived using a unit cost estimating approach. The cost estimates include a contingency markup based on unknown project site conditions.

The scope of this task is to provide SJRA a detailed cost estimate for the construction of the Walnut Creek and Brich Creek detention basins, located in Waller County west of Magnolia, Texas. The drawings developed as part of the conceptual design task were utilized to perform detailed take-off and estimate development. On-Screen Takeoff (OST), MS Excel, professional judgment, and manual calculations were used to calculate and record the quantities.

2.1.1 Basis of Estimate

Documents

- Developed by Black and Veatch
    - Draft Spring Creek Watershed Flood Control Design Basis Memorandum
    - C-00-101
    - C-00-102
    - C-00-201
    - C-00-202
    - C-00-203
    - C-00-204
    - C-00-205
    - C-00-206
- Developed by Halff
    - Utilities_Map.
    - Birch_Disturbance
    - Birch_Embankment
    - Birch_Stabilization
    - Walnut_Disturbance
    - Walnut_Embankment
    - Walnut_Stabilization

Unit Cost Sources

- TxDOT Bid Item Averages
- TPWD Projects with related materials and design

2.1.2 Assumptions

The following general assumptions were made in reference to the prepared cost estimates. Assumptions specific to quantities and calculation methods are outlined in Sections 2.2 and 2.3.

- Competitive bidding from medium to large earthwork companies
- Material availability onsite matches the design assumptions
- Contingency markup based on unknown site conditions
- Unit prices include the following markups:
  - Taxes and Fees
  - Direct Cost
  - Subcontractor Markups
    - Jobsite Overhead
    - Home Office Overhead
    - Profit
    - Bonds
  - Prime Contractor Markups
    - Jobsite Overhead
    - Home Office Overhead
    - Profit
    - Bonds

2.2 Walnut Creek opinion of probable construction cost

Quantities and calculations are based off dimensions outlined in the *Draft Spring Creek Watershed Flood Control Dams Conceptual Engineering Feasibility Study Conceptual Design Appendix* prepared by Black and Veatch and the accompanying plan and section exhibits.

Access to the site is along existing ranch roads off the westbound lanes of FM 1488. It is assumed that these roads will be widened to 24-feet and improved/prepared for construction equipment traffic by placing roadway flexbase. The location of the staging and laydown area was selected as existing slopes are relatively flat, it requires minimal clearing, and it is located approximately equidistant from each dam location. It is assumed that the contractor will use this area for field offices, an on-site batch plant, to stockpile materials, and to stage equipment.

Much of the proposed dam alignment is heavily treed and will require substantial clearing and grubbing. For estimating purposes, a cost per acre was determined, assuming the use of large, heavy machinery such as bulldozers and excavators to remove trees and root balls.

The unit cost for the preparation of the Storm Water Pollution Prevention Plan is based on the size of the site. Implementation and maintenance costs of temporary sediment and erosion controls are based off the total limits of disturbance and estimated duration of construction. The unit cost for Care of Water assumes bypass pumping of Walnut Creek for the duration of construction.

It is assumed that the top 2-foot layer of the site contains organics and other materials not suitable for use as fill. Excavation quantities include the stripping of the top 2-feet of all disturbed areas as well as the 20-foot wide, 20-foot deep with 1.5:1 (H:V) side slopes trapezoidal cut-off trench (as shown in Embankment Sections WC-1 – WC-4). To limit hauling off a large amount of material, the stripped material is proposed to be stockpiled on-site for potential reuse as topsoil. It is assumed that the material excavated from the cut-off trench will be suitable for use in the dam embankment. The remaining fill for the embankment will be sourced from two (2) identified borrow sites north and west of the proposed dam location. The volume of the internal drainage system and square footage of the sheetpile walls were determined using dimensions outlined in the conceptual design appendix.

The principal spillway outlet quantities were determined using the Spillway Section (WC-S) and Walnut Creek Dam Alignment conceptual design sheets. Similar elements of the spillway were grouped together for ease of quantification and estimating purposes.

Site stabilization includes a 36-inch layer of rock riprap across the entire back side of the dam (upstream). A soil retention blanket is proposed along the entire front of the dam (downstream) to aid in vegetation establishment. All disturbed areas, including borrow sites, will receive 6-inches of topsoil as well as both temporary and permanent hydromulch seeding.

Table 2-1 Walnut Creek Construction Cost Estimate

No.	DESCRIPTION OF ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED COST
1	Mobilization (5%), Demobilization (3%)	1	LS	$ 4,465,200.00	$ 4,465,200
	Demolition and Temporary Measures				
2	Care of Water	24	MO	$ 15,000.00	$ 360,000
3	SWPPP Preparation	1	LS	$ 20,000.00	$ 20,000
4	Temporary Erosion and Sedimentation Control (2%)	1	LS	$ 1,361,400.00	$ 1,361,400
5	Barricades, Signs and Traffic Handling	24	MO	$ 12,000.00	$ 288,000
6	Site Access	70	STA	$ 3,000.00	$ 210,000
7	Site Prep (Clearing and Grubbing)	90	AC	$ 13,000.00	$ 1,170,000
8	Construction Exit (Install & Remove)	160	SY	$ 60.00	$ 9,600
	Embankment				
9	Excavation (trench)	100,030	CY	$ 25.00	$ 2,500,750
10	Stripping/Remove and Stockpile 24-inches soil	42,080	CY	$ 10.00	$ 420,800
11	Embankment Fill (dam embankment and trench)	356,870	CY	$ 45.00	$ 16,059,150
12	Internal Drainage Aggregate, Embankment - Chimney Drain and Blanket Drain	38,370	CY	$ 120.00	$ 4,604,400
13	Miscellaneous Internal Drainage	1	LS	$ 90,000.00	$ 90,000
14	Cutoff Wall (sheetpiles)	67,480	SFF	$ 75.00	$ 5,061,000
15	Maintenance Road (Flexible Base)	6,950	SY	$ 65.00	$ 451,750
16	Instrumentation, Piezometers	4	EA	$ 30,000.00	$ 120,000
17	Instrumentation, Underdrain System Flow Weir	1	EA	$ 10,000.00	$ 10,000
18	Instrumentation, Surface Reference Monuments	8	EA	$ 2,000.00	$ 16,000
	Spillway				
19	Reinforced Concrete, Principal Spillway	2,950	CY	$ 975.00	$ 2,876,250
20	Reinforced Concrete, Principal Spillway Retaining Wall Stems	4,495	CY	$ 975.00	$ 4,382,625
21	Reinforced Concrete, Principal Spillway Manhole Walls and Deck Slab	260	CY	$ 1,100.00	$ 286,000
22	Reinforced Concrete, Principal Spillway Baffle Blocks and End Sill	190	CY	$ 1,100.00	$ 209,000
23	Reinforced Concrete, Principal Spillway Discharge Apron Slab, 18-in Thickness	1,375	CY	$ 975.00	$ 1,340,625
24	Reinforced Concrete, Principal Spillway Discharge Apron Slab, 36-in Thickness	875	CY	$ 1,050.00	$ 918,750
25	Internal Drainage Aggregate, Spillway	2,400	CY	$ 145.00	$ 348,000
26	Soil Anchors, Principal Spillway	1,250	LF	$ 165.00	$ 206,250
27	Rock Riprap and Bedding	1,415	SY	$ 285.00	$ 403,275
	Site Stabilization				
28	Rock Riprap with bedding stone (thickness = 36")	24,330	CY	$ 260.00	$ 6,325,800
29	Soil Retention Blanket	25,430	SY	$ 6.00	$ 152,580
30	Seeding Hydromulching (Temp. & Permanent)	431,750	SY	$ 5.00	$ 2,158,750
31	6" Top Soil amend onsite/reuse	431,750	SY	$ 8.00	$ 3,454,000
				Probable Construction Cost	**$ 60,279,955**
			Bonds, Insurance and General Conditions	**2.5%**	**$ 1,506,999**
			Contingency	**35%**	**$ 21,097,984**
				Total Probable Construction Cost	**$ 82,884,938**

Since the design professional has no control over the cost of labor, materials, or equipment, or over the contractor's method of determining prices, or over the competitive bidding or market conditions, their opinions of probable cost provided for herein are to be made on the basis of their experience and qualifications. Thes opinions represent their best judgment as a design professional familiar with the construction industry. However, the design professional can not and does not guarantee that proposals, bids, or construction cost will not vary from the opinions of probable cost they have prepared. If the owner wishes greater assurance a construction cost, they shall employ an independent cost estimator.

Client acknowledges and agrees that Halff's preparation of any estimate of probable design and/or construction costs, preliminary or otherwise, and any updated estimates of probable costs prepared by Halff, represent Halff's judgment as a design professional. Client further acknowledges and agrees that Halff has no cont over the cost of labor, materials, or equipment; the Contractor's methods of calculating and estimating bid prices; or competitive bidding, market, or negotiating conditions. Accordingly, Halff cannot and does not warrant or represent that bids or negotiated prices will not vary from Halff's estimate of probable costs (includir any updates thereto) or from Client's budget or from any other estimate or evaluation, prepared or agreed to by Halff.

2.3 Birch Creek opinion of probable construction cost

Quantities and calculations are based off dimensions outlined in the *Draft Spring Creek Watershed Flood Control Dams Conceptual Engineering Feasibility Study Conceptual Design Appendix* prepared by Black and Veatch and the accompanying plan and section exhibits.

Access to the site is along existing ranch roads off the westbound lanes of FM 1488. It is assumed that these roads will be widened to 24-feet and improved/prepared for construction equipment traffic by placing roadway flexbase. The location of the staging and laydown area was selected as existing slopes are relatively flat, it requires minimal clearing, and it is located approximately equidistant from each dam location. It is assumed that the contractor will use this area for field offices, an on-site batch plant, to stockpile materials, and to stage equipment.

Much of the proposed dam alignment is heavily treed and will require substantial clearing and grubbing. For estimating purposes, a cost per acre was determined, assuming the use of large, heavy machinery such as bulldozers and excavators to remove trees and root balls.

The unit cost for the preparation of the Storm Water Pollution Prevention Plan is based on the size of the site. Implementation and maintenance costs of temporary sediment and erosion controls are based off the limits of disturbance and estimated duration of construction. The unit cost for Care of Water assumes bypass pumping of Birch Creek for the duration of construction.

It is assumed that the top 2-foot layer of the site contains organics and other materials not suitable for use as fill. Excavation quantities include the stripping of the top 2-feet of all disturbed areas as well as the 20-foot wide, 20-foot deep with 1.5:1 (H:V) side slopes trapezoidal cut-off trench (as shown in Embankment Sections BC-1 – BC-4). To limit hauling off a large amount of material, the stripped material is proposed to be stockpiled on-site for potential reuse as topsoil. It is assumed that the material excavated from the cut-off trench will be suitable for use in the dam embankment. The remaining fill for the embankment will be sourced from the identified borrow site northwest of the proposed dam location. The volume of the internal drainage system and square footage of the sheetpile walls were determined using dimensions outlined in the design basis memorandum and accompanying exhibits developed by Black and Veatch.

The principal spillway outlet quantities were determined using the Spillway Section (BC-S) and Birch Creek Dam Alignment conceptual design sheets. Similar elements of the spillway were grouped together for ease of quantification and estimating purposes.

Site stabilization includes a 36-inch layer of rock riprap across the entire back side of the dam (upstream). A soil retention blanket is proposed along the entire front of the dam (downstream) to aid in vegetation establishment. All disturbed areas, including borrow sites, will receive 6-inches of topsoil as well as both temporary and permanent hydromulch seeding.

Table 2-2 Birch Creek Construction Cost Estimate

No.	DESCRIPTION OF ITEM	QUANTITY	UNIT	PRICE	COST
1	Mobilization (5%), Demobilization (3%)	1	LS	$ 3,450,200.00	$ 3,450,200
	Demolition and Temporary Measures				
2	Care of Water	24	MO	$ 15,000.00	$ 360,000
3	SWPPP Preparation	1	LS	$ 20,000.00	$ 20,000
4	Temporary Erosion and Sedimentation Control (2%)	1	LS	$ 1,051,900.00	$ 1,051,900
5	Barricades, Signs and Traffic Handling	24	MO	$ 12,000.00	$ 288,000
6	Site Access	90	STA	$ 3,000.00	$ 270,000
7	Site Prep (Clearing and Grubbing)	90	AC	$ 13,000.00	$ 1,170,000
8	Construction Exit (Install & Remove)	160	SY	$ 60.00	$ 9,600
	Embankment				
9	Excavation (trench)	94,210	CY	$ 25.00	$ 2,355,250
10	Stripping/Remove and Stockpile 24-inches soil	30,500	CY	$ 10.00	$ 305,000
11	Embankment Fill (dam embankment and trench)	216,970	CY	$ 45.00	$ 9,763,650
12	Internal Drainage Aggregate, Embankment - Chimney Drain and Blanket Drain	26,220	CY	$ 120.00	$ 3,146,400
13	Miscellaneous Internal Drainage	1	LS	$ 90,000.00	$ 90,000
14	Cutoff Wall (sheetpiles)	63,520	SFF	$ 75.00	$ 4,764,000
15	Maintenance Road (Flexible Base)	5,610	SY	$ 65.00	$ 364,650
16	Instrumentation, Piezometers	4	EA	$ 30,000.00	$ 120,000
17	Instrumentation, Underdrain System Flow Weir	1	EA	$ 10,000.00	$ 10,000
18	Instrumentation, Surface Reference Monuments	8	EA	$ 2,000.00	$ 16,000
	Spillway				
19	Reinforced Concrete, Principal Spillway	2,375	CY	$ 975.00	$ 2,315,625
20	Reinforced Concrete, Principal Spillway Retaining Wall Stems	3,425	CY	$ 975.00	$ 3,339,375
21	Reinforced Concrete, Principal Spillway Manhole Walls and Deck Slab	260	CY	$ 1,100.00	$ 286,000
22	Reinforced Concrete, Principal Spillway Baffle Blocks and End Sill	135	CY	$ 1,100.00	$ 148,500
23	Reinforced Concrete, Principal Spillway Discharge Apron Slab, 18-in Thickness	1,025	CY	$ 975.00	$ 999,375
24	Reinforced Concrete, Principal Spillway Discharge Apron Slab, 36-in Thickness	675	CY	$ 1,050.00	$ 708,750
25	Internal Drainage Aggregate, Spillway	1,885	CY	$ 145.00	$ 273,325
26	Soil Anchors, Principal Spillway	1,050	LF	$ 165.00	$ 173,250
27	Rock Riprap and Bedding	1,250	SY	$ 285.00	$ 356,250
	Site Stabilization				
28	Rock Riprap with bedding stone (thickness = 36")	18,960	CY	$ 260.00	$ 4,929,600
29	Soil Retention Blanket	18,200	SY	$ 6.00	$ 109,200
30	Seeding Hydromulching (Temp. & Permanent)	414,100	SY	$ 5.00	$ 2,070,500
31	6" Top Soil amend onsite/reuse	414,100	SY	$ 8.00	$ 3,312,800
	Probable Construction Cost				**$ 46,577,200**
	Bonds, Insurance and General Conditions			**2.5%**	**$ 1,164,430**
	Contingency			**35%**	**$ 16,302,020**
	Total Probable Construction Cost				**$ 64,043,650**

Since the design professional has no control over the cost of labor, materials, or equipment, or over the contractor's method of determining prices, or over the competitive bidding or market conditions, their opinions of probable cost provided for herein are to be made on the basis of their experience and qualifications. Thes opinions represent their best judgment as a design professional familiar with the construction industry. However, the design professional can not and does not guarantee that proposals, bids, or construction cost will not vary from the opinions of probable cost they have prepared. If the owner wishes greater assurance a construction cost, they shall employ an independent cost estimator.

Client acknowledges and agrees that Halff's preparation of any estimate of probable design and/or construction costs, preliminary or otherwise, and any updated estimates of probable costs prepared by Halff, represent Halff's judgment as a design professional. Client further acknowledges and agrees that Halff has no cont over the cost of labor, materials, or equipment; the Contractor's methods of calculating and estimating bid prices; or competitive bidding, market, or negotiating conditions. Accordingly, Halff cannot and does not warrant or represent that bids or negotiated prices will not vary from Halff's estimate of probable costs (includi any updates thereto) or from Client's budget or from any other estimate or evaluation, prepared or agreed to by Halff.

3 Land costs

3.1 Land considerations

The land acquisition cost for each project accounts for a large percentage of the overall construction cost due to the large acreage needed for the limits of inundation. Much of the land acquisition type and cost will depend on the individual landowner and the negotiations that take place during the acquisition phase (i.e., whole, or partial acquisition, easement or fee ownership). Land costs will vary due to several factors:

- **Inundation Extents:** The extents of the land acquisition will be determined by the flood pool elevations of the two reservoirs and the land below these elevations as well as negotiations with the landowner. In some cases, the full tract may need to be acquired due to future usability of the tract, while in others only a portion will need to be acquired.
- **Land Acquisition Type:** Two types of land acquisition are available for flood control facilities depending on the necessary use of the land.
    - In Fee - If owned in fee, the project owner has full control of the land and will be responsible within the taxing jurisdiction as the underlying landowner. This prevents others from use of the land. However, this ownership will cost more than an easement on the land.
    - Flood Easements - Since these facilities will be dry most of the time, easements can be acquired in lieu of fee to reduce the cost of the occasional use of the land. In this scenario, the underlying owner is still able to use the land with specific restrictions such as placing fill or structures within the easement. Easements are quite common for drainage and flood control facilities.
- **Existing Land Use:** The current and future use of the property can impact the price of the property. Vacant land will tend to be valued lower than land with existing structures or other uses. Structures within the proposed acquisition area would need to be relocated out of the area or demolished prior to construction of the project which will also increase the cost of acquisition.
- **Roadway Access:** Undeveloped properties that have roadway access will be valued higher than those without access.
- **Existing Floodplain Limits:** Development within the floodplain can be costly and therefore properties within an existing floodplain will generally be valued lower than those outside of an identified floodplain. Discounts are usually assessed for the area of an acquisition that is within the floodplain.
- **Future Land Use:** Property owners sometimes have plans for how their property will develop in the future and a perceived value for that future use.
- **Survey:** Inundation extents for the reservoirs are associated with a single elevation based on the proposed flood pool. However, the curvilinear nature of this extent is difficult to accurately survey for acquisition. Therefore, the extent of acquisition is simplified when purchasing an easement or full tract of land. This may result in an acquisition of more total land than necessary to encompass the required area.
- **Eminent Domain:** For large public facilities such as a flood control reservoir, condemnation of the necessary land may be required for some of the properties. This

process increases the time of acquisition as well as the cost since numerous additional steps are needed to obtain the right to use the land.

3.2 Acquisition extents

The land required for the projects consist of the dam footprint as well as the downstream erosion control features needed to dissipate the flow from the spillway. Land acquisition will also be required within the inundation area to accommodate the temporary storage of water during rainfall events.

Several inundation limit scenarios were evaluated to determine the acquisition extents for the inundation limits. The extent of the area needed for acquisition for the 100-year, 500-year, and Probable Maximum Flood (PMF) extents are summarized in the tables below.

Table 3-1 Walnut Creek Acquisition Extent Summary

Storm Event	Water Surface Elevation	Inundation Area (acres)	No. of Impacted Parcels
100-year	254.7	940	69
500-year	259.8	1,210	71
Probable Maximum Flood	261.6	1,370	72

Table 3-2 Birch Creek Acquisition Extent Summary

Storm Event	Water Surface Elevation	Inundation Area (acres)	No. of Impacted Parcels
100-year	251.2	690	17
500-year	255.3	850	19
Probable Maximum Flood	257.1	920	19

While the PMF scenario requires the most area and number of parcels, acquiring this extent provides the future project owner the most amount of property for inundation in case of an extreme flood. These limits would also compensate existing landowners for use of the property. Whether acquired through easements or fee, the acquisition limits were based on the PMF inundation limits.

3.3 Property valuation

Properties were evaluated based on the available 2024 Waller County Appraisal District information including the tract size, land classification, and market value. The land in the area is either classified as large undeveloped tracts, large lot rural areas, or solar farms.

For the large undeveloped tracts (greater than 85 acres), the cost per acre of land varies depending on the variables listed above. For these tracts, the value ranged from $8,000 per acre to $40,000 per acre with a median value of around $20,000 per acre. Since there was a wide variation, standardizing the cost would provide a consistent number and reduce the cost variability of outliers in the estimate. Therefore, the median value of $20,000 per acre was used for cost calculation for the large, undeveloped tracts.

For large lot residential tracts, the cost per acre varied between $14,000 and $130,000 depending on the land use of the tract. The inundation extents vary from lot to lot and with costs having a larger range in value, these lots were evaluated at the market value provided within the appraisal district information.

For all undeveloped land costs, a 2.0 multiplier was included to the market value to account for negotiations and soft costs of fee acquisition. For land where a structure exists, a 3.0 multiplier was used to account for additional soft costs associated with the structure. Easements were assumed to cost half of the market value cost.

3.4 Land cost scenarios

Each of these factors will be considered in the negotiations with individual landowners at the time of acquisition and costs will vary depending on the negotiation timeframe. There will be a wide range of cost scenarios depending how the negotiations occur and therefore two cost scenarios were tabulated to provide the range of potential land costs as summarized below.

- **Full Fee Acquisition:** For this scenario, the project owner would acquire the entire property within the limits of the probable maximum flood. This would provide the owner full access to the property and prevent underlying landowners from accessing or building within the property. This scenario also provides the upper limit of what potential costs may be for acquiring the land.
- **Full Easement Acquisition:** NRCS flood control dams within Texas are typically constructed within permanent easements acquired up to flood pool elevations. This scenario would allow the project owner to construct and operate the project as well as allowing the underlying owner access to the property for agricultural or other permitted uses. This scenario provides the lower limit of potential costs as easements are lesser cost than fee acquisition.

3.5 Walnut Creek detention basin

The PMF extents for the Walnut Creek detention basin cover 72 tracts that include existing floodplain, a large solar farm, and individual landowners with varying levels of development. The limits of acquisition were adjusted beyond the inundation limits to accommodate the ability to survey and document the tracts and easements. There were also some instances where the entire property is recommended to be purchased rather than just the inundated area due to the usability of the remaining tract. The proposed tract acquisition extents are shown in Figure 3-1 and Exhibit 1.

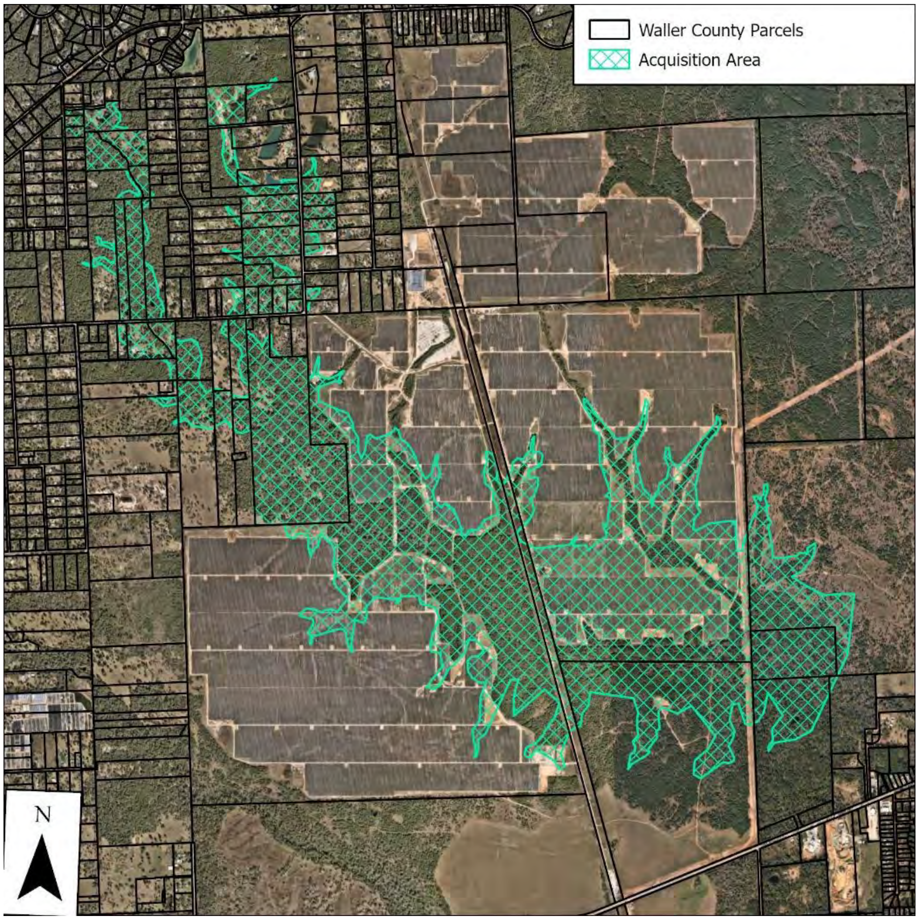


Figure 3-1 Walnut Creek Recommended Acquisition Area

Fee acquisition of the acquisition area would include all or portions of the tracts impacted by the probable maximum flood extents. The total cost for fee acquisition is approximately $122,286,920. Easement acquisitions for the entire inundation extents is estimated to be $68,639,998. The actual cost will be somewhere between this range depending on the type of acquisition, and negotiations with the property owners.

The existing solar panels for the large solar farm will have to be removed and relocated to accommodate the inundation limits. Approximately 880 acres of solar panels will be required to be re-located which is approximately 34% of the total site. At the time of this study, it may cost up to $50 million to remove and relocate these panels. This cost was included in the land acquisition for Walnut Creek. The potential land costs are summarized in Figure 3-2.

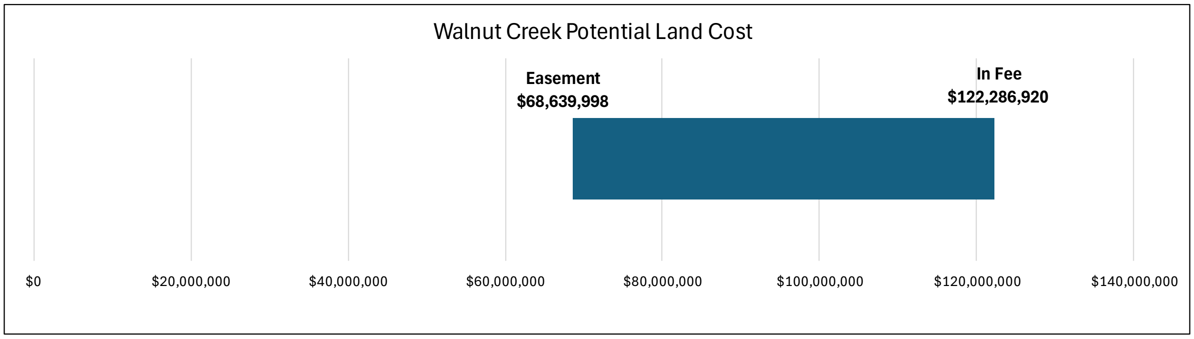


Figure 3-2 Walnut Creek Potential Land Cost Range

3.6 Birch Creek detention basin

The PMF extents cover 19 tracts that include existing floodplain, a potential residential development, and individual landowners with varying levels of development. The limits of acquisition were adjusted beyond the inundation limits to accommodate the ability to survey and document the tracts and easements. There were also some instances where the entire property is recommended to be purchased rather than just the inundated area due to the usability of the remaining tract. The proposed tract acquisition extents are shown in Figure 3-3 and Exhibit 2.

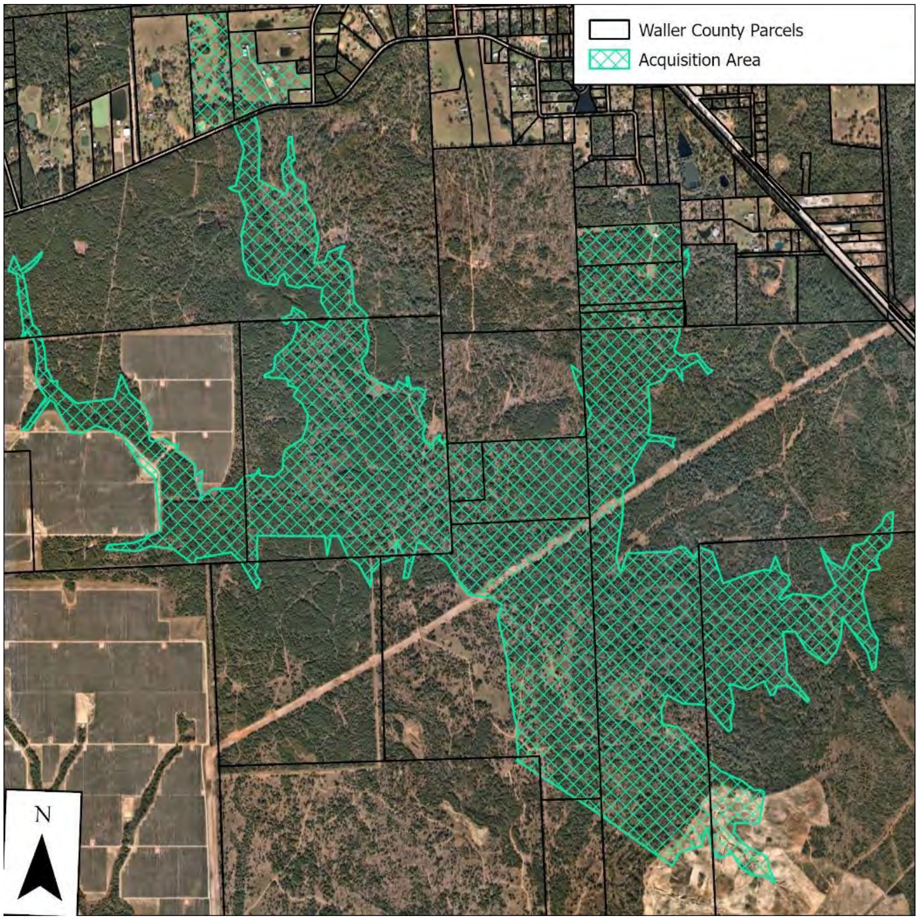


Figure 3-3 Birch Creek Recommended Acquisition Area

Fee acquisition of the proposed acquisition area would include all or portions of the tracts impacted by the probable maximum flood extents. The total cost for fee acquisition is approximately $49,876,968. Easement acquisition for the entire inundation extents is estimated to be $11,748,674. The actual cost will be somewhere between this range depending on the type of acquisition and negotiations with the property owners.

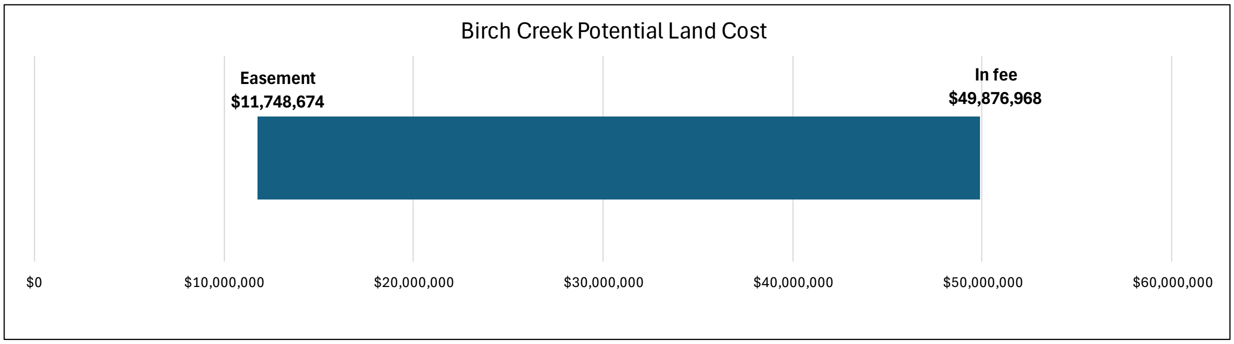


Figure 3-4 Birch Creek Potential Land Cost Range

4 Utility conflicts

4.1 Conflict summary

The site review confirmed no utility conflicts with the proposed project. This desktop assessment utilized the best available data, including a review of the Texas Railroad Commission web viewer for active gas lines and well sites. Easement documents were not reviewed as part of this effort.

The opinion of probable construction cost does not include gas line relocation expenses. If easement restrictions require relocation, the estimated cost is approximately $3 million per mile per line.

Minor overhead utility adjustments may be needed at the proposed construction entrance to meet clearance requirements. An allowance for these adjustments is included in the mobilization cost within the opinion of probable construction cost.

No conflicts with other utilities were identified based on the available data. A detailed subsurface utility investigation was not performed as part of this assessment. Further coordination with utility providers may be required during final design and construction.

5 Environmental mitigation costs

5.1 Impacted areas

5.1.1 Walnut Creek

Based on preliminary design plans, the project would include the construction of an approximately 3,373-foot-long dam that would have an approximately 12.0-acre footprint. Fill material to construct the earthen dam would be collected locally from two borrow pit areas measuring 28.9 acres and 14.7 acres. To support operation and maintenance of the dam, approximately 6,160 feet of 24-foot-wide road improvements would be required to be constructed. A temporary construction laydown area, measuring approximately 17.2 acres will be used throughout construction activities, but will be restored to pre-construction conditions following construction.

5.1.2 Birch Creek

The proposed Birch Creek dam project's preliminary design involves the construction of an approximately 3,168-foot-long dam covering an approximately 8.7-acre footprint. Fill material to construct the earthen dam would be collected locally from an adjacent 53-acre borrow pit. To support operation and maintenance of the dam, approximately 7,410 feet of 24-foot-wide road improvements would be required to be constructed. The same temporary construction laydown area (approximately 17.2 acres) would be used throughout construction activities but will be restored to pre-construction conditions following construction.

5.2 Aquatic feature impacts

Halff estimated the potential wetland and waterbody impacts by comparing the most recent project layouts to current aerial images and wetland and waterbody alignments from the most recent U.S. Geological Survey digital orthographic quarter quadrangle topographic maps (Magnolia West 2023 and Waller NW 2022)) and U.S. Fish and Wildlife Service (USFWS) National Wetlands Inventory (NWI) data (USFWS 2024) and U.S. Geological Survey (USGS) National Hydrography Dataset (NHD) data (USGS 2024). The NWI and NHD data were selected to estimate impacts because they are modeled data that are generally appropriate at broad geographic scale; however, it is critical to note that actual aquatic resource features would have to be delimited from a field survey.

Based on these data, construction of the Walnut Creek dam project would involve impacts to 3.5 acres of wetlands and 295 linear feet of stream. Similarly, the Birch Creek dam project would involve impacts to 0.9 acre of wetlands and 267 linear feet of stream.

5.3 Potential mitigation costs

At the time of writing, eight mitigation providers can provide mitigation credits for the projects (Table 5-1). The projects fall within the primary service area of four of these banks, but in the secondary service area of the remaining four mitigation banks.

Table 5-1 Mitigation Options for Impacts Associated with the Proposed Spring Creek Dam Projects

Bank Name	Service Area	Wetland Suites	Stream	Current Price
Katy Prairie Stream	Primary	–	19,537	$375/linear foot
Houston Conroe Stream	Primary	–	32,011	$325/linear foot
Sand Hill	Primary	TBD	TBD	$220,000/acre $350/linear foot
Tarkington Bayou	Primary	127.5 (Forested) 65.7 (Non-forested)	3,591	$214,500/acre $325/linear foot
West Montgomery	Secondary	67.7 (Forested)	–	$153,000/acre
Spellbottom	Secondary	31.1 (Forested)	–	$150,000/acre
Mill Creek	Secondary	13.9 (Forested)	–	$220,000/acre
Lake Houston	Secondary	14.4 (Forested)	12,048	$153,000/acre $310/linear foot

Assuming that the wetlands associated with the proposed impact areas are typical of the region, each acre of wetland loss would require 1.8 to 2.4 credit suites per acre of impact (i.e., complete loss of wetland function). However, conversion of wetlands or other incremental decreases in functional value and/or reductions in acreage would reduce credit needs. Stream impacts are calculated based on the linear feet of the project and the type of alteration to the project. Purchasing credits from a secondary service area would increase mitigation costs by 50%.

Considering the price range of wetland credits, purchasing credits necessary to mitigate for the complete loss of the wetlands associated with the Walnut Creek detention basin would cost between $1,386,000 (i.e., 3.5 acres x 1.8 functional loss x $220,000) and $1,848,000 (i.e., 3.5 acres x 2.4 functional loss x $220,000) if completed within the primary service area of these banks. Depending on the extent of stream impacts, the project would also require between $110,625 (i.e., 295 linear feet x $375 x 1.0 impact factor) and $442,500 (i.e., 295 linear feet x $375 x 4.0 impact factor). Similarly, wetlands impacted by the Birch Creek detention basin would cost between $356,400 and $475,200, if completed within the primary service area of these banks. Depending on the extent of stream impacts, the project would also require between $100,125 and $400,500.

These projections should be considered maximum probable estimates of costs for mitigation. A complete delineation of waters of the United States, functional assessment, and determination of functional impacts associated with construction need to be conducted during future phases of the project lifecycle and would provide additional guidance as to potential mitigation costs. Additionally, per credit costs are negotiable with individual banks.

6 Maintenance

Developing a comprehensive operational and maintenance (O&M) cost plan for dry detention basins involves considering various activities essential for the basin's functionality and safety. Below is a breakdown of typical maintenance activities, their recommended frequencies, and considerations for cost estimation:

Vegetation Maintenance (Mowing and Tree Removal):

- **Frequency:** Mow side slopes, and embankments at least twice per year. Regular mowing prevents woody vegetation establishment and maintains accessibility. Tree removal should be conducted as needed to prevent root systems from compromising structural integrity.
- **Cost Considerations:** Costs include labor, equipment operation, and debris disposal. Factors influencing costs are basin size, slope steepness, and vegetation density. Utilizing native plantings can reduce mowing frequency and associated costs.

Debris and Litter Removal:

- **Frequency:** Inspect and remove debris from inlets, outlets, and basin areas during each mowing session and after significant storm events.
- **Cost Considerations:** Costs involve labor for inspections and debris removal, as well as proper disposal methods. Regular removal prevents blockages and maintains basin functionality.

Post-Storm Event Inspections:

- **Frequency:** Conduct inspections after major storm events to assess structural integrity and identify immediate maintenance needs.
- **Cost Considerations:** Costs include labor for thorough inspections and documentation. Timely inspections help in early detection of issues, potentially reducing long-term repair costs.

Annual Comprehensive Inspections:

- **Frequency:** Perform detailed inspections annually to evaluate the overall condition of the basin, including embankments, spillways, and mechanical components.
- **Cost Considerations:** Could require qualified dam safety personnel and appropriate equipment. Comprehensive inspections ensure compliance with safety standards and identify areas needing minor maintenance.

Minor Maintenance Projects (e.g., Addressing Animal Damage):

- **Frequency:** As needed, based on inspection findings.
- **Cost Considerations:** Includes materials and labor for repairs such as filling burrows or repairing minor erosion. Proactive management can prevent minor issues from escalating into major problems.

Five-Year Dam Safety Inspections:

- **Frequency:** Every five years.
- **Cost Considerations:** Will require dam safety engineers. Costs are higher due to the specialized nature of the inspection but are crucial for ensuring long-term structural integrity and safety compliance.

Additional Considerations:

- **Sediment Removal:** Monitor sediment accumulation and plan for removal when storage capacity is significantly reduced, typically every 5 to 10 years. Costs depend on sediment volume, disposal requirements, and accessibility.
- **Erosion Control:** Regularly inspect for erosion on embankments and basin outlet. Implement corrective measures, such as reseeding, installing erosion control blankets, additional riprap, as needed.

Cost Estimation:

Annual maintenance costs for dry detention basins are estimated to be approximately 2-5% of the initial construction cost. For this cost analysis, maintenance was assumed to be approximately 3.4%. This percentage accounts for routine activities such as inspections, vegetation management, and minor repairs. Non-routine maintenance, like sediment removal or significant structural repairs, will incur additional costs and should be budgeted for separately.

Implementing a detailed maintenance plan with scheduled activities and allocated budgets will help ensure the detention basin operates effectively and remains compliant with safety regulations.

7 Total costs

As outlined in the sections above, the total cost to construct each detention basin is dependent on and inclusive of several factors. While lands and easement acquisitions, utility relocations, environmental requirements, and operations and maintenance all impact overall project cost, the primary cost driving factor is the absence of site-specific geotechnical information. Unknowns in subsurface conditions have direct effects on the assumptions made regarding on-site properties and applicability, seepage control design, foundation design, and groundwater levels. Additional design information is required to establish the applicability of the selected seepage control design – sheet pile cut off wall. Geotechnical information is essential to further tailor costs for each dam. The breakdown and total cost for each project is included in Table 7-1.

Table 7-1 Detention Basin Total Cost

	Walnut Creek	**Birch Creek**
Construction	$82,884,938	$64,043,650
Engineering[1]	$12,432,740	$9,606,547
Land Acquisition[2]	$95,463,459	$30,812,821
Environmental	$2,290,500	$875,700
Utilities	$0	$0
Total	**$193,071,637**	**$105,338,718**
Annual Maintenance	$2,800,000	$2,100,000

[1] Engineering including geotechnical, survey, design, and construction management is assumed to be 15% of the total construction cost

[2] The 50% mark of the land cost range was used for the total cost estimate

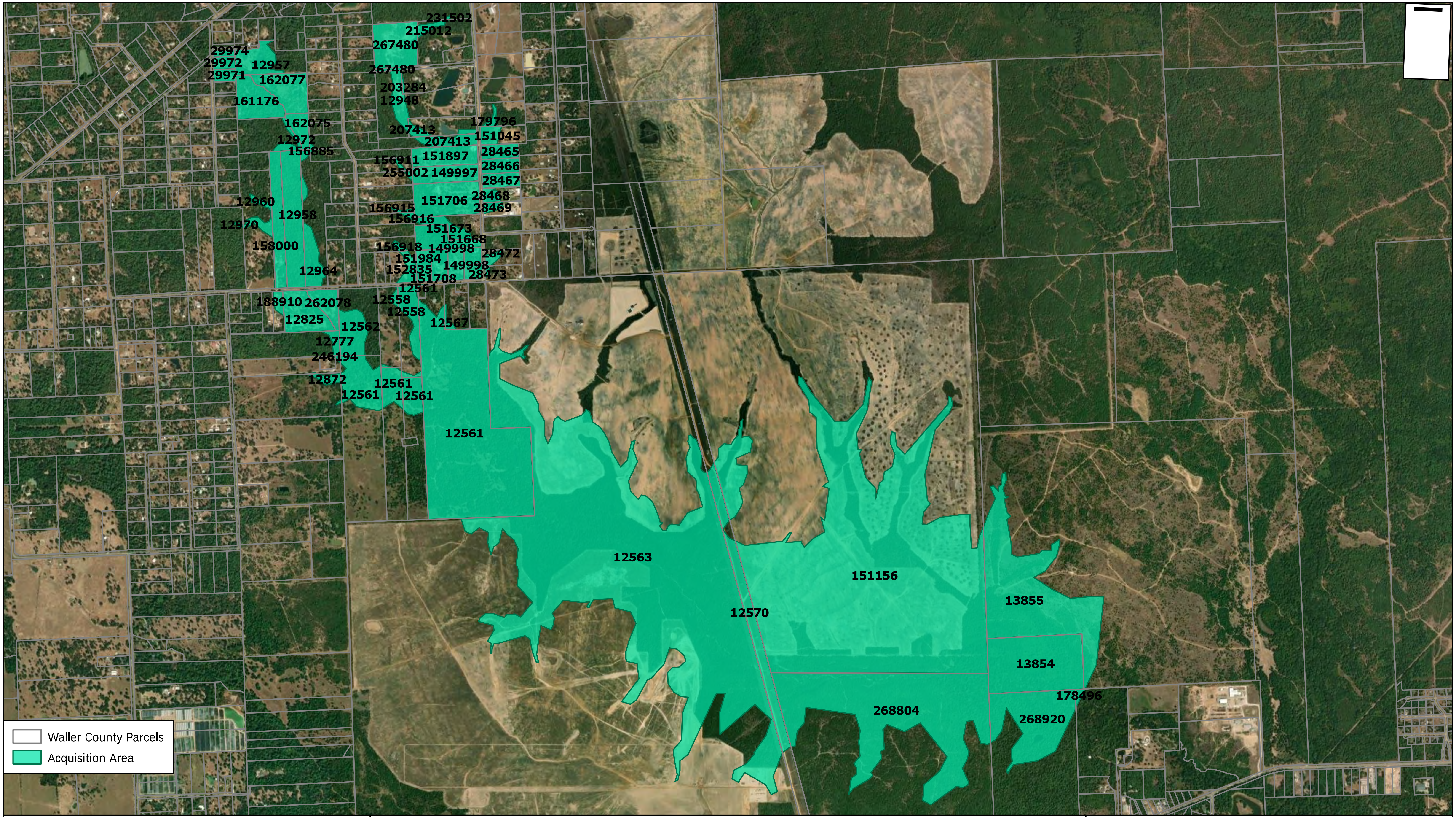
The purpose of this lifecycle cost estimate is to assess the full financial commitment associated with the two projects, including both construction and long-term maintenance costs. The analysis calculates total annual costs over a 50-year project life, which includes 30 years of O&M plus debt service followed by 20 years of continued operations and maintenance. Each project is evaluated independently with its own financing structure and O&M obligations. The results provide a clear, long-range financial outlook to support decision-making and resource planning.

Each project is assumed to be financed independently using a 30-year term loan at a fixed interest rate of 4.00%, which aligns with recent rates available to public entities (e.g., AA-rated municipal bonds). Level debt service is assumed, meaning the same payment is made each year, simplifying long-term financial planning. This structure assumes no refinancing, variable rates, or early payoff. All cost figures are presented in 2025 dollars, with no inflation applied. The debt service amounts are calculated using a standard amortization formula, annual payments over the loan period.

Table 7-2 Project Cost

	Walnut Creek	**Birch Creek**
Construction	**$193,071,637**	**$105,338,718**
Annual Maintenance	$2,800,000	$2,100,000
Debt Service Factor	0.05783	0.05783
Annual Debt Service	$11,165,000	$6,092,000
30-Year Debt Service Total	$334,950,000	$182,760,000
50-Year Operations & Maintenance Total	$140,000,000	$105,000,000
50-Year Lifecycle Cost	**$474,950,000**	**$287,760,000**

Initial construction cost, Annual debt service (30-year loan), Annual operations & maintenance (O&M), A total cost view for both the 30-year financing period and the full 50-year project lifecycle



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5/7/2025

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Exhibit 1
Walnut Creek Parcel Acquisitions

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study

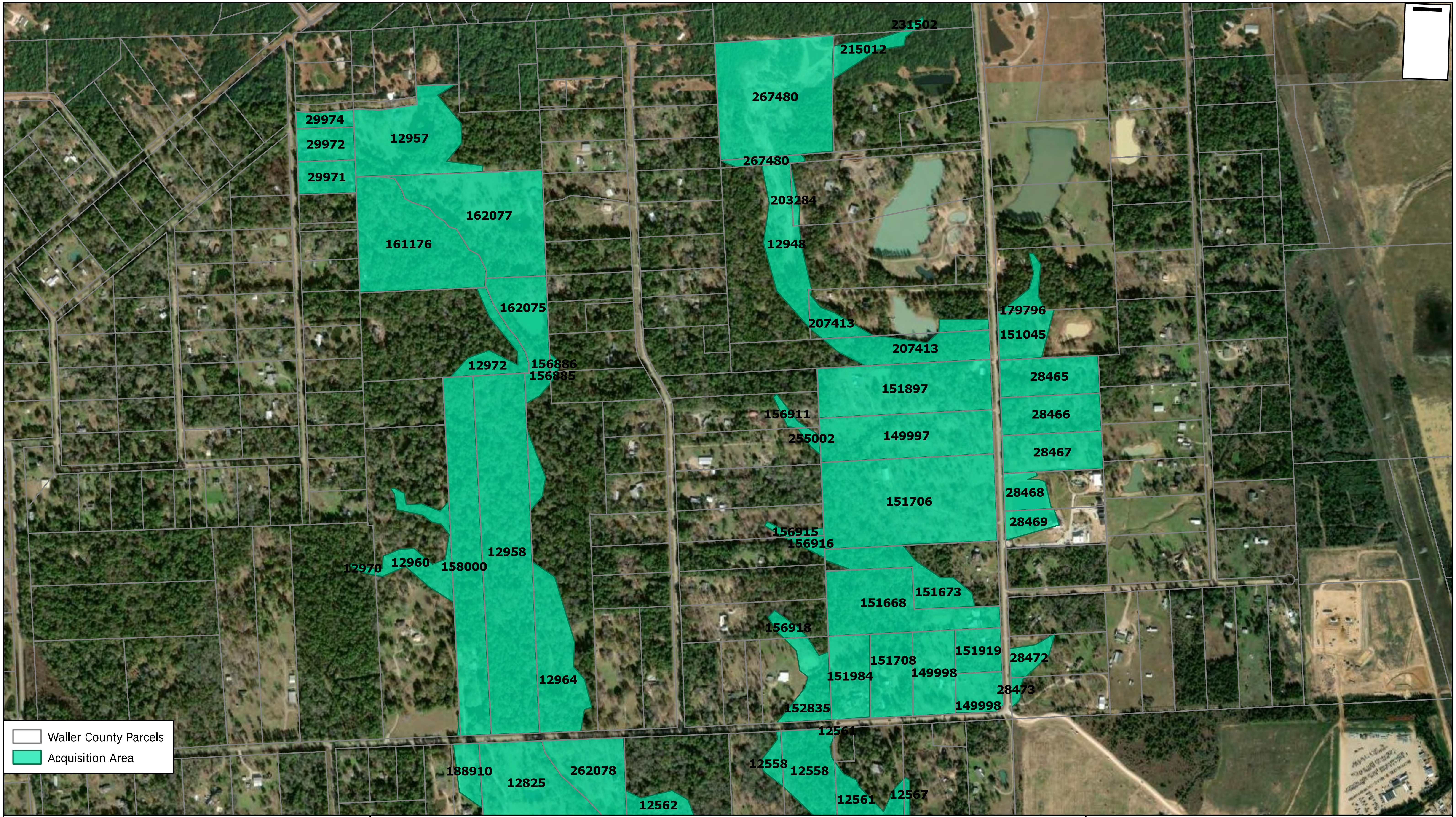

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Exhibit 1
Walnut Creek Parcel Acquisitions

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study

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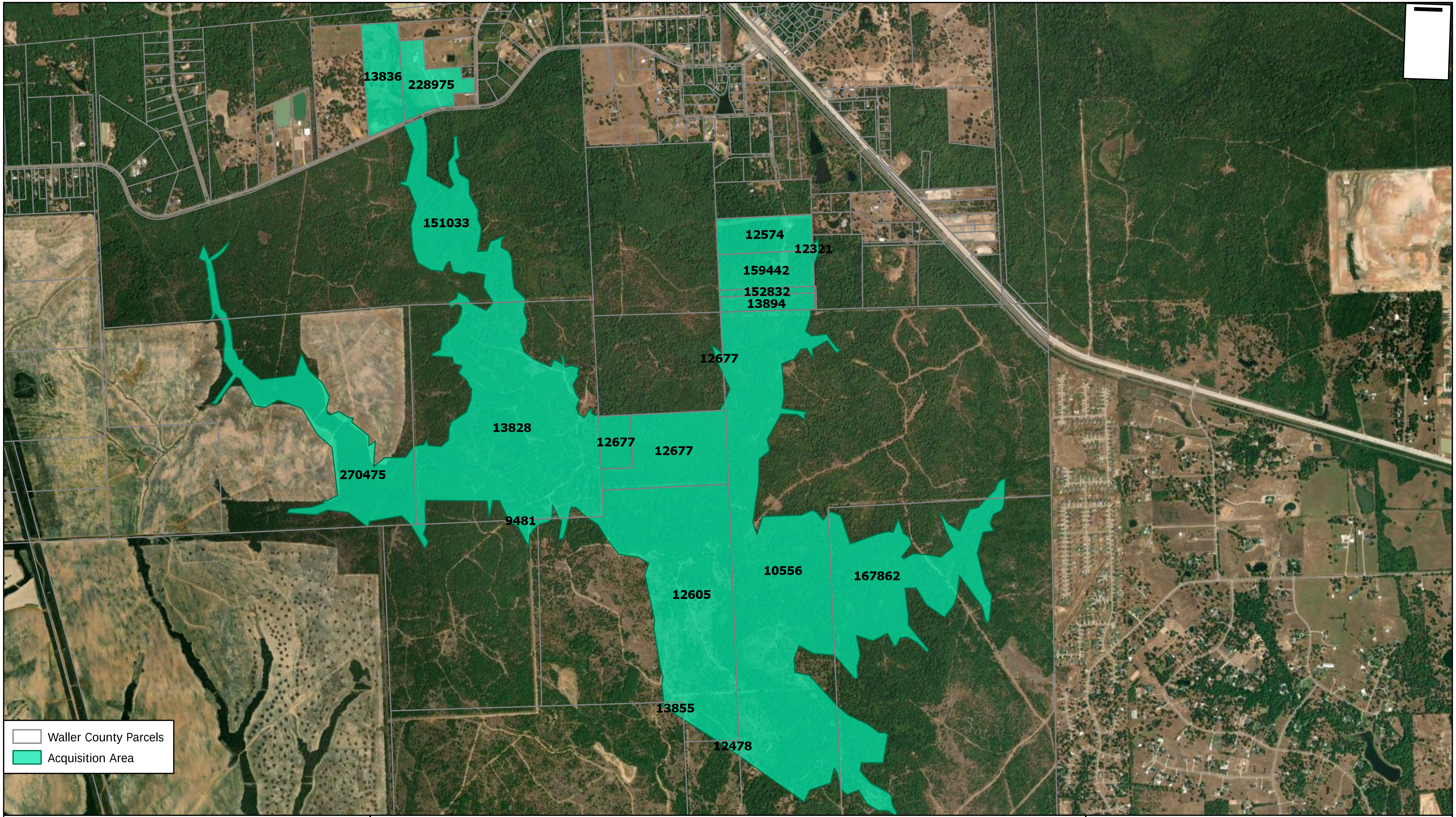

☐	Waller County Parcels
🟩	Acquisition Area

5/7/2025

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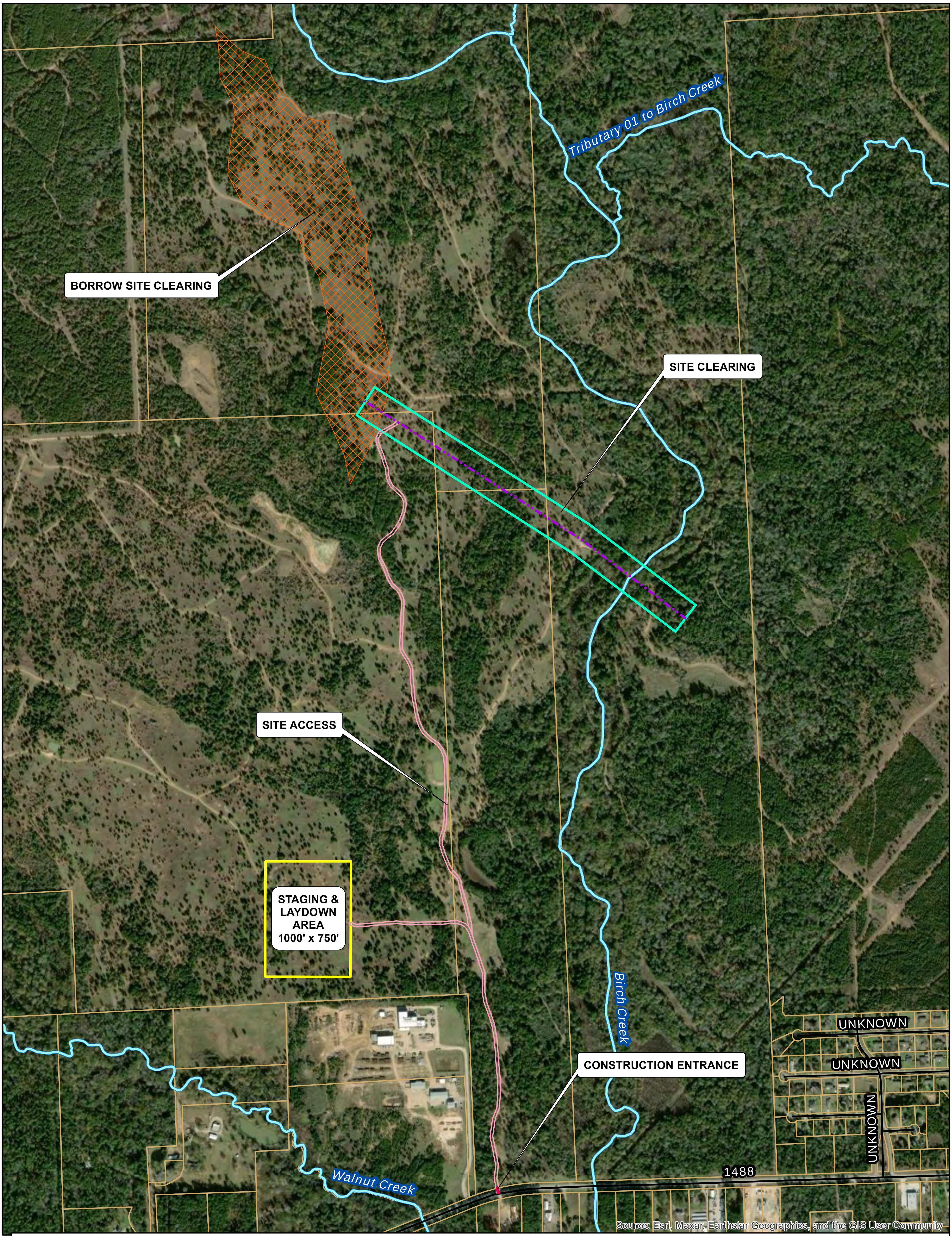
Exhibit 2
Birch Creek Parcel Acquisitions

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study



Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

Spring Creek Watershed Flood Control Dams

Conceptual Engineering Feasibility Study - Birch Creek

Disturbance Limits

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SJRA SAN JACINTO RIVER AUTHORITY

Legend

-··	Birch Alignment	▭	Site Clearing - 200' L & R
■	Construction Entrance/Exit	▨	Borrow Site
▭	Site Access - 24' Wide	═	Stream Centerline
▭	Staging/Laydown Yard	▭	Parcels

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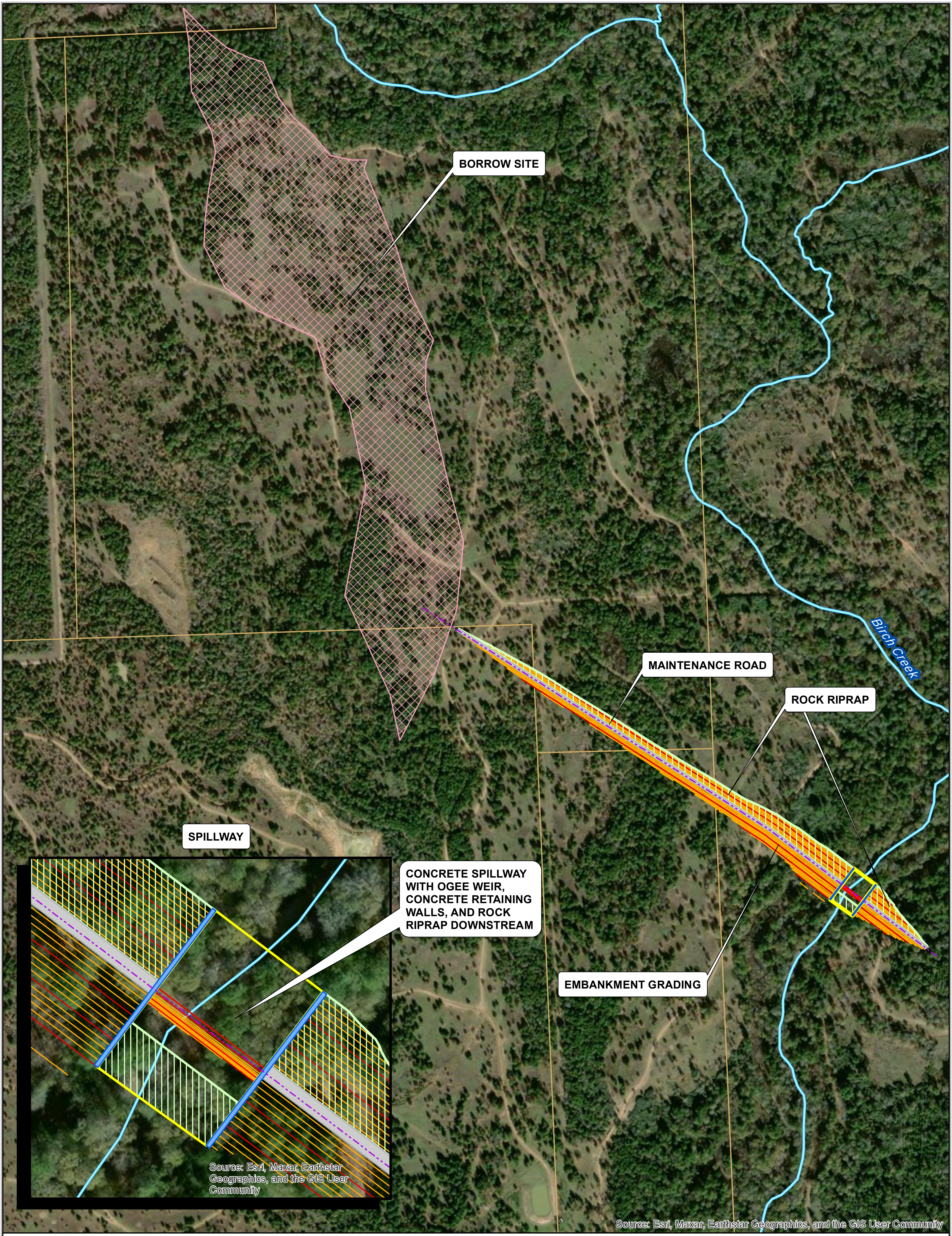
Feet
1 inch = 800 feet



Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

Spring Creek Watershed Flood Control Dams

Conceptual Engineering Feasibility Study - Birch Creek

Embankment & Outlet

halff

SJRA
SAN JACINTO RIVER AUTHORITY

Legend

-·-·-	Birch Alignment	▨	Borrow Site
🟦	Concrete Retaining Walls	—	Proposed Minor Contours - 2'
⬜	Maintenance Road	—	Proposed Major Contours - 10'
🟨	Concrete Spillway	═	Stream Centerline
▥	Rock Riprap Limits	☐	Parcels

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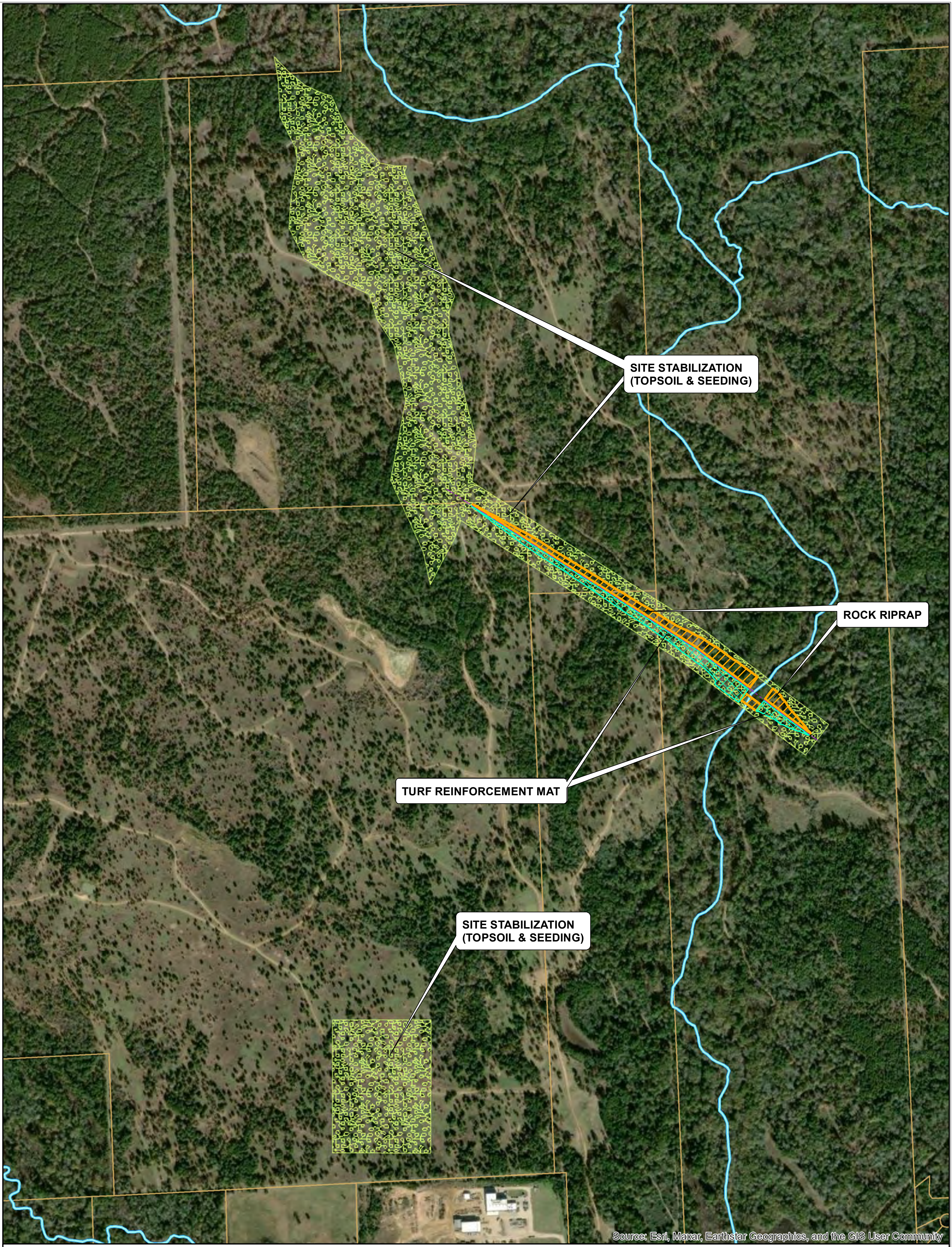

Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

Spring Creek Watershed Flood Control Dams

Conceptual Engineering Feasibility Study - Birch Creek

Site Stabilization

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SJRA SAN JACINTO RIVER AUTHORITY

Legend

- -·-·- Birch Alignment
- ▩ Turf Reinforcement Mat
- ▨ Rock Riprap Limits
- ▦ Site Stabilization Limits
- ═ Stream Centerline
- ☐ Parcels

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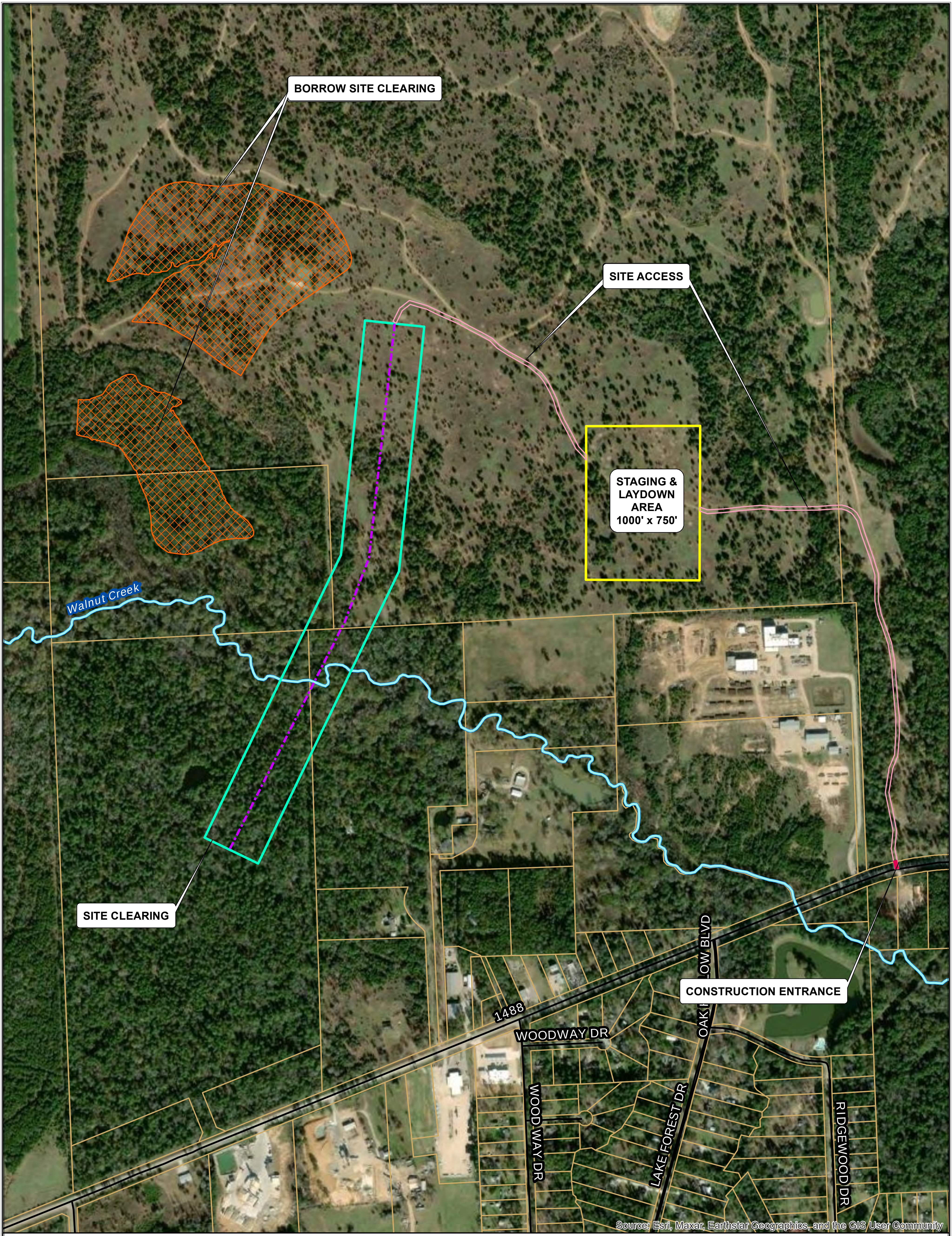


Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

Spring Creek Watershed Flood Control Dams

Conceptual Engineering Feasibility Study - Walnut Creek

Disturbance Limits

halff

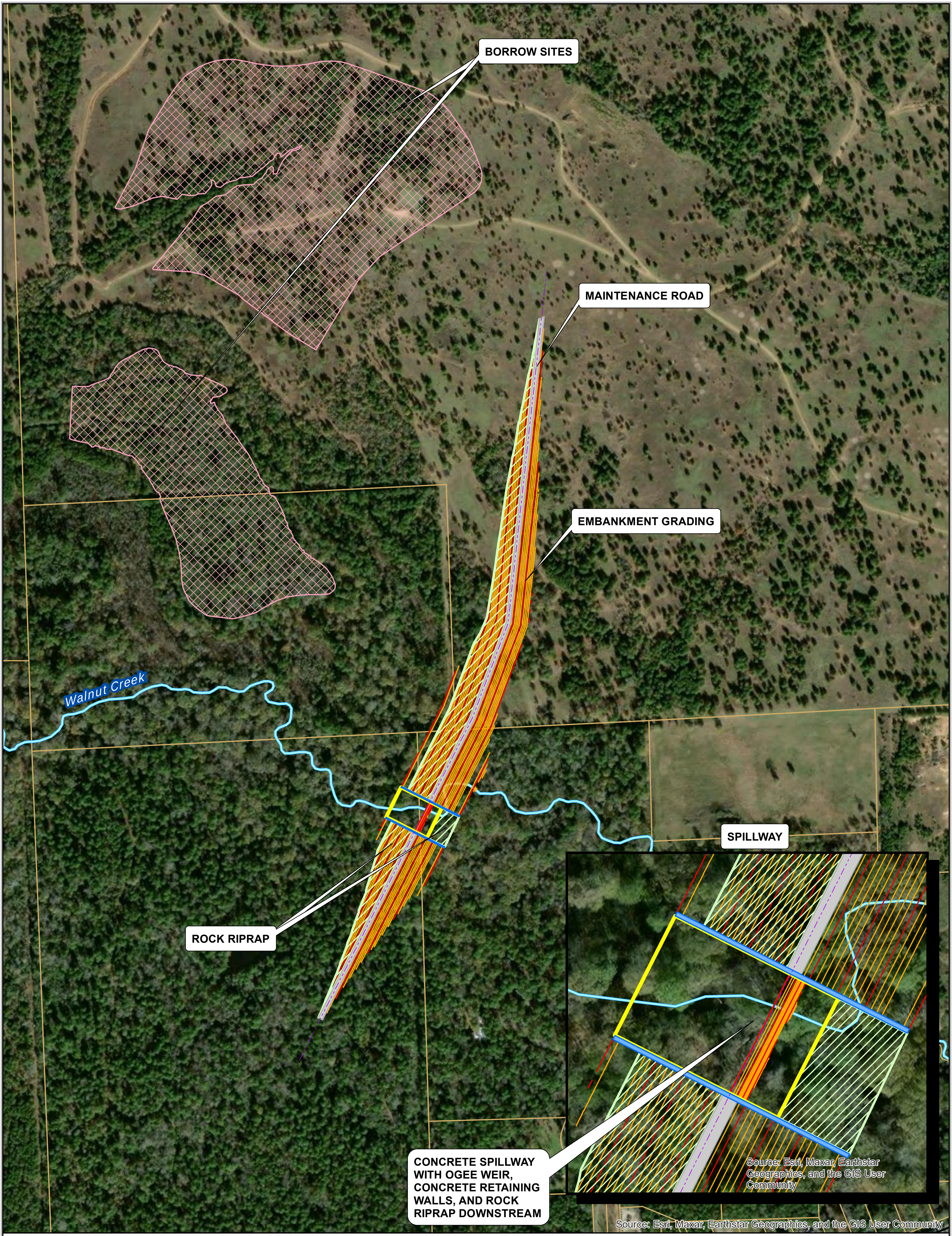
SJRA SAN JACINTO RIVER AUTHORITY

Legend

-·- Walnut Alignment
■ Construction Entrance/Exit
☐ Site Access - 24' Wide
☐ Staging/Laydown Yard
☐ Site Clearing - 200' L & R
▨ Borrow Site
═ Stream Centerline
☐ Parcels

0 150 300 600 900 1,200 Feet

1 inch = 600 feet



Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

Spring Creek Watershed Flood Control Dams

Conceptual Engineering Feasibility Study - Walnut Creek

Embankment & Outlet

halff

SJRA
SAN JACINTO RIVER AUTHORITY

Legend

- Walnut Alignment
- Concrete Retaining Walls
- Maintenance Road
- Concrete Spillway
- Rock Riprap Limits
- Borrow Site
- Proposed Minor Contours - 2'
- Proposed Major Contours - 10'
- Stream Centerline
- Parcels

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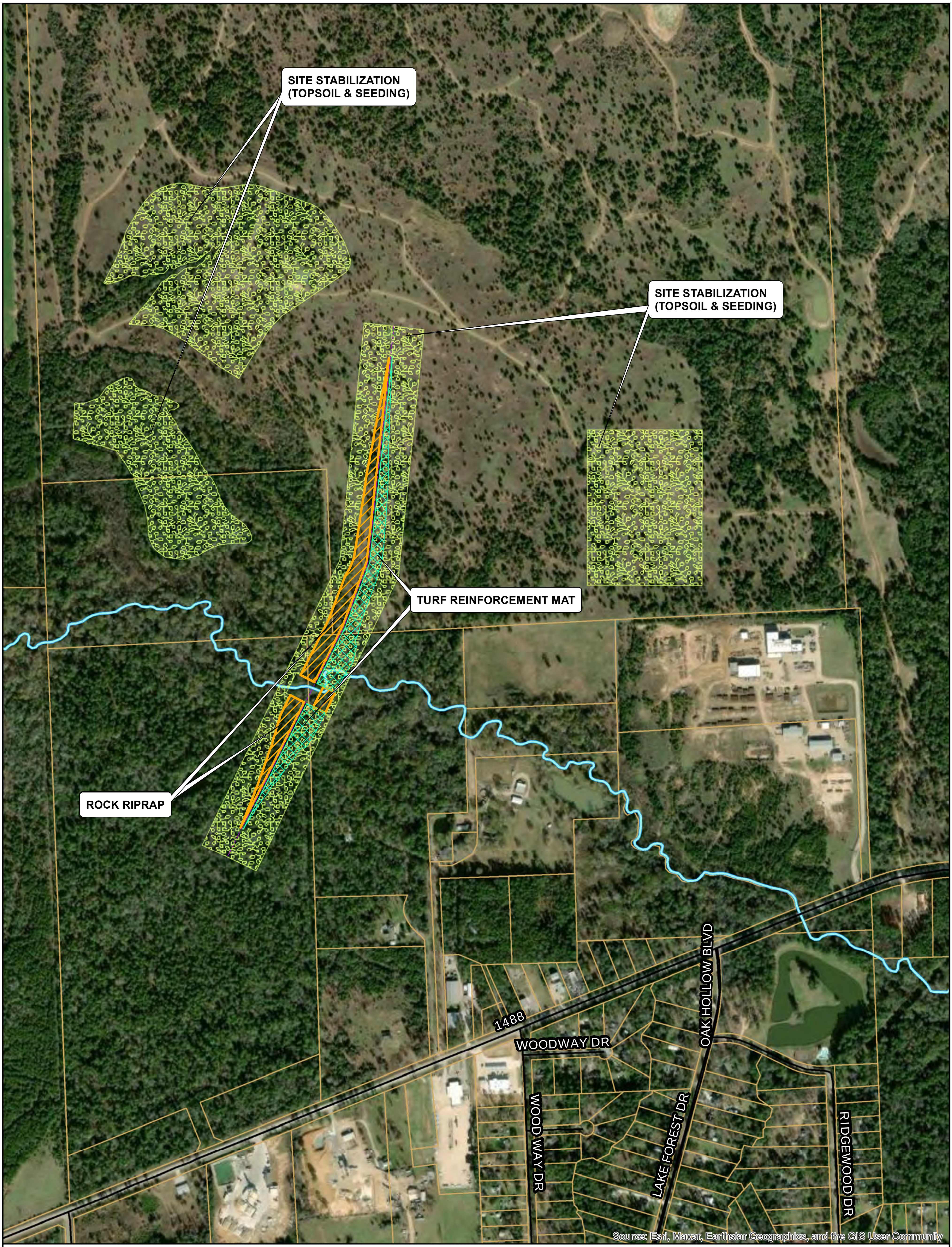

Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

Spring Creek Watershed Flood Control Dams

Conceptual Engineering Feasibility Study - Walnut Creek

Site Stabilization

halff

SJRA
SAN JACINTO RIVER AUTHORITY

Legend

- Walnut Alignment
- ▨ Turf Reinforcement Mat
- ▨ Rock Riprap Limits
- ▨ Site Stabilization Limits
- Stream Centerline
- ☐ Parcels

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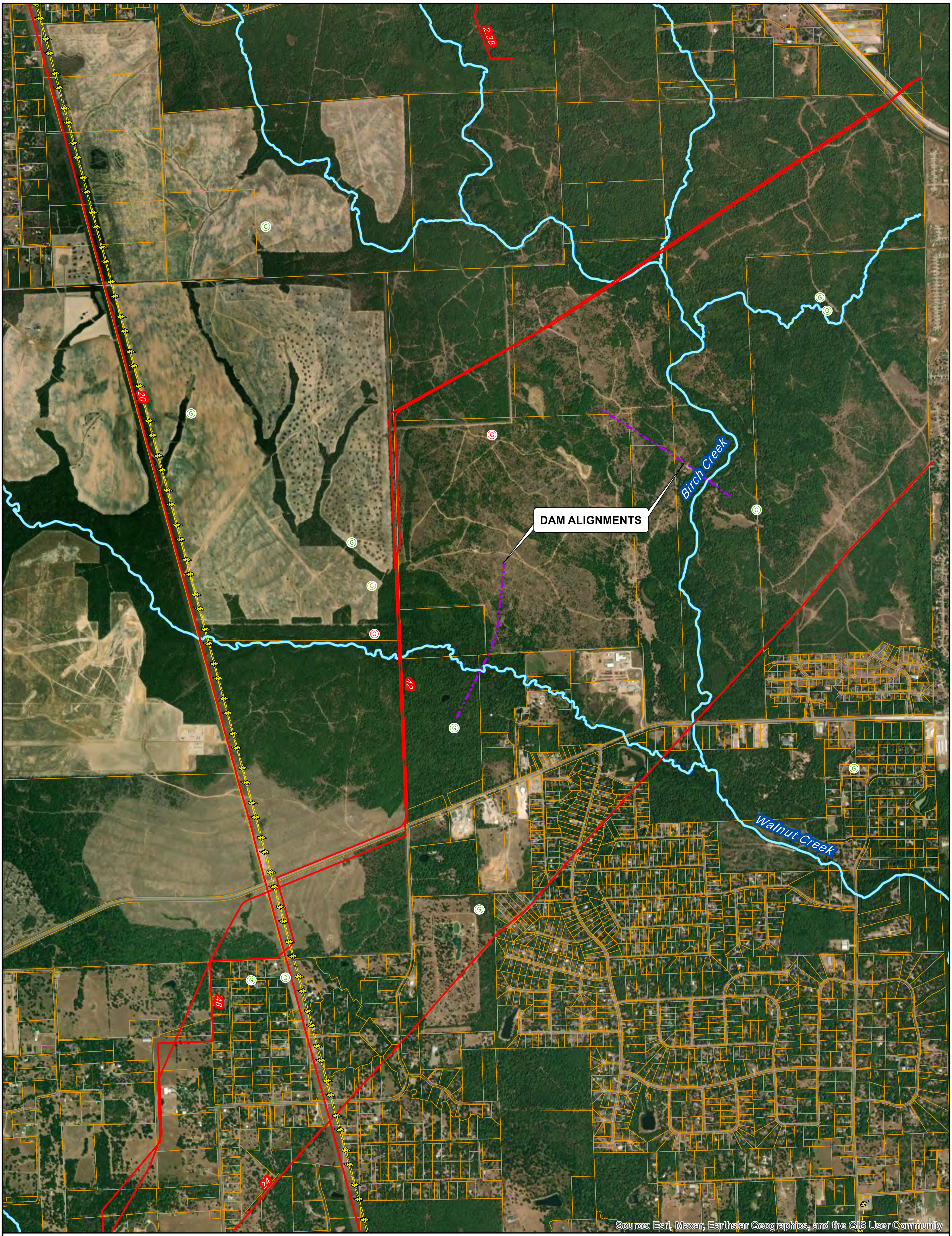
Feet

1 inch = 600 feet



Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

Spring Creek Watershed Flood Control Dams

Conceptual Engineering Feasibility Study

Utilities

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SJRA SAN JACINTO RIVER AUTHORITY

Legend

Gas Wells

- Ⓖ Dry Hole
- Ⓖ In Operation
- Ⓖ Plugged

- Electric Transmission Lines
- Natural Gas Transmission Lines
- Dam Alignments
- Stream Centerline
- ☐ Parcels

N W E S

0 500 1,000 2,000 3,000 4,000 Feet

1 inch = 2,000 feet

Appendix D:
Hydrology & Hydraulics BCR Appendix

Spring Creek Watershed Flood Control Dams Conceptual Engineering Feasibility Study

Hydrology & Hydraulics BCR Appendix

Flood Infrastructure Fund Category 1

Project ID 21-0016

Prepared for:
Texas Water Development Board

Prepared by:
Halff
C. Andrew Moore, P.E., CFM
Sam Hinojosa, P.E., CFM
Cynthia Rodriguez, EIT
Brandon Huggett, EIT, CFM

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Exhibits

1 Introduction and background

A hydrologic and hydraulic analysis provided the basis for sizing the two detention basins on each of the creeks, identifying the inundation limits upstream of the detention basins, and determining the downstream benefits. Hydrology was conducted using HEC-HMS version 4.8 and hydraulics using HEC-RAS version 5.0.7. Following the sizing of the structures and completion of the analysis, benefits were calculated based on the FEMA Benefit Cost Analysis (BCA) guidelines and tool.

1.1 Scope of work

The scope of work for hydrologic, hydraulic, and benefit cost analysis included the following:

- Development of hydrologic and hydraulic models for the Birch Creek, Walnut Creek, and Spring Creek watersheds to determine flood storage and hydraulic benefits provided by the proposed detention basins.
- Simulation of the models for two historical storm events to confirm calibration with observed conditions.
- Development of a structural database from publicly available information for structures potentially benefitting from the projects.
- Quantification of benefits from the projects both individually and in combination.
- Determination of the benefit cost ratio for the projects using the FEMA BCA toolkit over a 50-year period.

1.2 San Jacinto Regional Watershed Master Drainage Plan (SJRWMDP)

The San Jacinto River Regional Watershed Master Drainage Plan (SJRWMDP) was a comprehensive regional study led by the Harris County Flood Control District (HCFCD), Montgomery County, City of Houston (City), and the San Jacinto River Authority (SJRA). The study was completed in December 2020. The study goals were to identify existing flood risk within the San Jacinto River basin upstream of the Lake Houston dam and evaluate flood risk reduction alternatives on a regional basis. The study included development of hydrologic and hydraulic models for the major streams of the upper San Jacinto River watershed including Spring Creek.

Recommendations from the study for the Spring Creek watershed included two detention facilities on the Walnut Creek and Birch Creek tributaries. These recommended projects targeted flood reduction along Spring Creek as well as future mitigation for conveyance improvement projects in the watershed. The Walnut Creek and Birch Creek detention basin projects are further explored in the current analysis.

1.2.1 Walnut Creek detention basin

The proposed inline detention basin is located on Walnut Creek, a tributary to Spring Creek, approximately 0.6 miles north of the FM 1488 crossing and 5.5 miles west of Magnolia, Texas. The detention basin is in the upper half of the Spring Creek watershed and captures flow from a drainage area of approximately 21 square miles. The location of the proposed detention basins is shown in Figure 1-1.

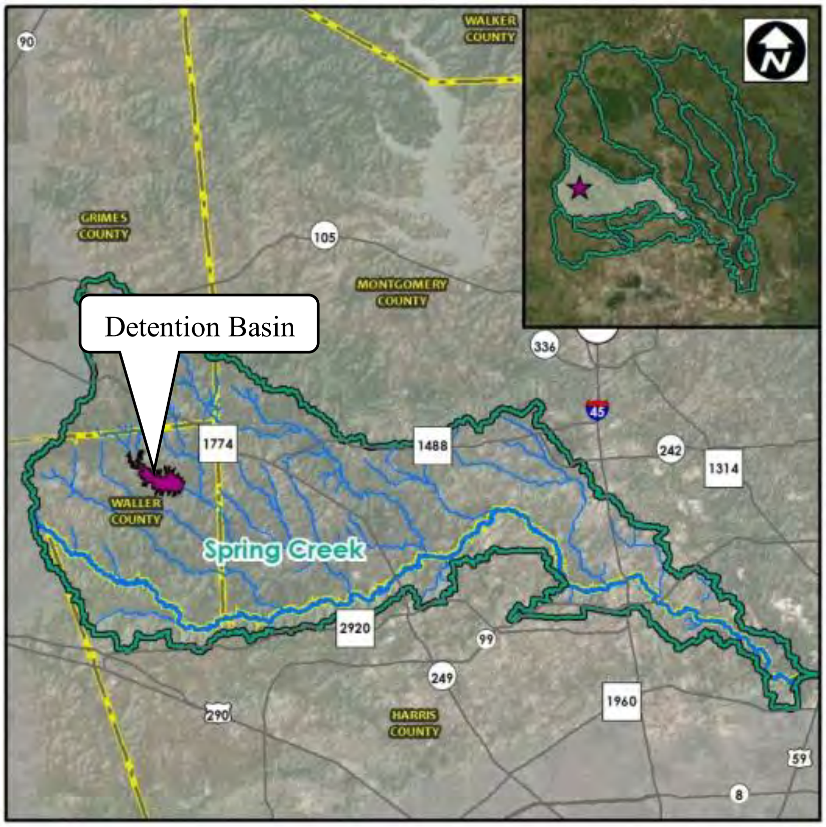

Figure 1-1 Walnut Creek Detention Basin Location (from SJRWMDP)

The proposed project includes a dry detention basin that reduces flows within the watershed. The control structure is a 46-foot-high earth dam with a concrete cap with a primary outfall consisting of 2 – 4’ x 4’ reinforced concrete boxes and a secondary ogee spillway approximately 200 feet in length. The impoundment requires approximately 0.7 million cubic yards of embankment. At the 1% ACE (Annual Chance Exceedance) water surface elevation the detention basin encompasses an area of 1,218 acres and detains over 12,000 acre-feet of storage.

1.2.2 Birch Creek detention basin

The proposed inline detention basin is located on Birch Creek, a tributary to Spring Creek, approximately 1 mile north of the FM 1488 crossing and 3.5 miles west of Magnolia, Texas. The detention basin is in the upper half of the Spring Creek watershed and captures flow from a drainage area of approximately 13 square miles. The location of the proposed detention basins is shown in Figure 1-2.

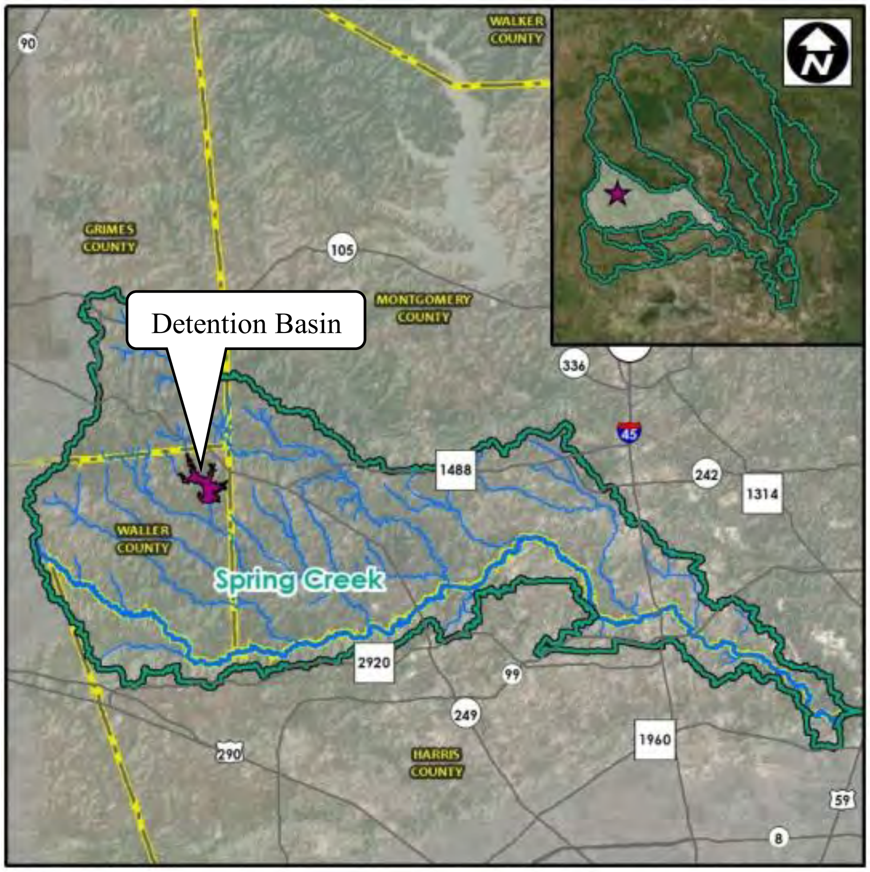

Figure 1-2 Birch Creek Detention Basin Location (from SJRWMDP)

The proposed project includes a dry detention basin that reduces flow within the watershed. The control structure is a 35-foot- high earth dam with a concrete cap with a primary outfall consisting of 2 – 4' x 3' reinforced concrete boxes and a secondary ogee spillway approximately 200 feet in length. The impoundment will require approximately 0.46 million cubic yards of embankment. At the 1% ACE water surface elevation the detention basin encompasses an area of 873 acres and detains over 7,700 acre-feet.

2 Data collection

Various data sources were acquired, reviewed, and adjusted as part of the hydrologic and hydraulic analysis. Data types included terrain, gages, historical rainfall, previous studies, and modeling.

2.1 HCFCD modeling

The Modeling Assessment & Awareness Project (MAAPnext), led by the Harris County Flood Control District (HCFCD) in partnership with FEMA, involved the development of new modeling and updated floodplain mapping for Harris County's 22 major watersheds, including the Spring Creek watershed. The effort incorporated most current terrain and rainfall data and utilized new hydrologic and hydraulic modeling methodologies to better depict flood risk in the region. The feasibility study leveraged the following HCFCD models and supporting documentation:

- HEC-RAS (v5.0.7) model for the Spring Creek Watershed including simulations for both the frequency and historical storm events including Hurricane Harvey (2017), Memorial Day (2016), and Tax Day (2016).
- HEC-HMS (v4.3) model for the Spring Creek Watershed including simulations for both the frequency and historical storm events

2.2 Terrain

The terrain developed as part of the HCFCD mapping effort was used as the basis for the analysis. The terrain was developed in 2018 by the Texas Strategic Mapping (StratMap) Program on the North American Vertical Datum of 1988 (NAVD88), Geoid 12B (cell size is 3 feet x 3 feet). The terrain also incorporated channel bathymetry for the Spring Creek channel from Kuykendahl Road to the confluence with the West Fork San Jacinto River developed under MAAPnext.

2.3 Structural database

A structural database was developed and used for the calculation of damages for the benefit cost analysis. The database included building footprints, finished floor elevations, square footage, and building type.

- Building footprints were provided by the Texas Water Development Board (TWDB) and screened to include structures within 1,000 feet of the 500-year floodplain. Structures smaller than 500 square feet were assumed to be sheds or other non-habitable structures and were removed from the database.
- Finished floor elevations were tabulated for each structure based on the underlying terrain elevations. Each structure's finished floor elevation was estimated to be one foot above the terrain elevation at the centroid of the structure.
- Square footage was obtained from the residing structure's county appraisal district (Waller, Montgomery, or Harris).
- Building types were set based on the TWDB information including either residential, industrial, or commercial.

3 Hydrology

The HEC-HMS models prepared by the HCFCD were used as the basis to develop runoff hydrographs for the watershed. These models were updated as needed to incorporate the proposed projects. Updates included changes to the drainage basins within the vicinity of the proposed projects as well as parameters associated with the basin changes. HEC-HMS v4.8 was used for the analysis.

3.1 Rainfall data

Rainfall data for the frequency storm events was obtained from HCFCD Rainfall Depths and Intensities White Paper for Harris County Hydrologic Region No. 1, which encompasses the Spring Creek watershed. Table 3-1 below provides the Atlas 14 rainfall depth, duration, and frequency data used.

Table 3-1 Atlas 14 Rainfall Depths

Duration	10 % AEP[1]	2 % AEP	1% AEP	0.2 % AEP
Min	0.81	1.07	1.19	1.49
15 Min	1.62	2.13	2.36	2.95
1 Hour	3.07	4.06	4.51	5.87
2 Hour	4.03	5.67	6.49	9.04
3 Hour	4.66	6.84	7.99	11.50
6 Hour	5.79	8.94	10.70	15.90
12 Hour	6.95	11.10	13.40	20.10
1 Day	8.22	13.40	16.30	24.20

[1] Annual Exceedance Probability

3.2 Hydrology updates

Drainage areas were obtained from the HCFCD model and verified with the topography and land use. Areas near the proposed project sites were subdivided and adjusted to include additional detail upstream of the proposed detention basins. Exhibit 2 shows the drainage areas from the prior study and drainage areas for the updated analysis. The HCFCD drainage area and updated drainage area sizes are shown in Table 3-2.

Table 3-2 Drainage Area Sizes

HCFCD Drainage Areas	Area (ac)	Updated Drainage Areas	Area (ac)
J501_06	1,888.95	J501_06	1,118.23
		J501_06_02	770.72
J503_04	2,041.67	J503_04_01	888.95
		J503_04_02	1,152.72
J503_05	2,216.47	J503_05_01	894.15
		J503_05_02	489.72
		J503_05_03	847.44

The hydrologic losses, impervious cover, and transform methodology were updated for the subdivided drainage areas. The methods from the HCFCD study were used to determine these parameters.

3.2.1 Hydrologic losses

Rainfall losses were calculated using the Green & Ampt method for all drainage areas. The Green & Ampt methodology requires suction and hydraulic conductivity values, which are based on soil type. The Canopy Loss Method was used in conjunction with Green and Ampt to account for losses due to vegetation. The values used in the HEC-HMS model are based on the HCFCD study and presented below in Table 3-3.

Table 3-3 Hydrologic Loss Parameters

Predominate Soil	Hydrologic soil group	Initial Canopy (in)	Max. Canopy (in)	Crop Coefficient	Initial Moisture Content	Saturated Content	Suction (in)	Conductivity (in/h)
Sandy Loam	B	0.0	0.5	1.0	0.059	0.46	2.286	0.181

3.2.2 Impervious cover

Impervious cover values were assigned based on the underlying land use type and the values from the HCFCD study as presented in Table 3-4.

Table 3-4 HCFCD Typical Impervious Cover Values

Land Use	Description	Percent Impervious
Undeveloped	Unimproved, natural, or agricultural	0%
Residential – Rural Lots	≥ 5-acre ranch or farm	5%
Residential – Large Lots	> 1/2 acre new residential with storm sewers or roadside ditches with adequate capacity, OR > 1/4 acre older neighborhoods with limited capacity roadside ditches	25%
Residential – Small Lots	≤ 1/4 acre	40%
School	School with non-paved areas	40%
Developed Green Areas	Parks or golf courses	15%
Light Industrial/Commercial	Office, parks, nurseries, airports, warehouses, or manufacturing with non-paved areas	65%
High Density	Commercial, business, industrial, or apartments	85%
Isolated Transportation	Highway or major thoroughfare corridors	80%
Water	Detention basins, lake, and channels	100%

Impervious cover values were recalculated for the subdivided drainage areas using the same GIS-based impervious cover layer developed in the HCFCD study. Table 3-5 shows the original and recalculated impervious cover values for the updated drainage areas.

Table 3-5 Calculated Impervious Cover

Original Subbasins	Original Impervious	Updated Subbasins	Updated Impervious
J501_06	6.35%	J501_06	2.58%
		J501_06_02	11.82%
J503_04	4.85%	J503_04_01	7.66%
		J503_04_02	2.68%
J503_05	10.75%	J503_05_01	14.59%
		J503_05_02	4.83%
		J503_05_03	10.50%

3.2.3 Transform method

The Clark Unit Hydrograph Method was used for the hydrograph transform method and uses both a time of concentration factor (Tc) and a storage coefficient (R). The Tc and R values for the updated drainage areas were computed from the Basin Development Factor (BDF) method in the HCFCD study. Table 3-6 shows the original and recalculated Tc and R values for the updated drainage areas.

Table 3-6 Calculated Tc and R Values

Original Subbasins	Original TC	Original R	Updated Subbasins	Updated TC	Updated R
J501_06	2.03	5.40	J501_06	1.52	4.16
			J501_06_02	1.18	3.20
J503_04	1.82	4.85	J503_04_01	1.14	3.10
			J503_04_02	1.45	3.96
J503_05	2.14	5.68	J503_05_01	1.02	2.79
			J503_05_02	1.07	3.02
			J503_05_03	1.28	3.53

3.3 HEC-HMS results

The HEC-HMS model was simulated for the frequency and historical storm events to develop the peak flows and hydrographs for the updated drainage areas. The results for the original and updated peak discharges for the 10% ACE, 2% ACE, 1% ACE, and 0.2% ACE events are shown in Table 3-7. The peak discharges remained unchanged for drainage areas that were not subdivided.

Table 3-7 HEC-HMS Peak Discharge

Original Subbasins	Peak Discharge (cfs)				Updated Subbasins	Peak Discharge (cfs)			
	10%	2%	1%	0.2%		10%	2%	1%	0.2%
J501_06	1,092	1,816	2,222	3,433	J501_06	792	1,292	1,568	2,388
					J501_06_02	688	1,090	1,303	1,942
J503_04	1,305	2,136	2,598	3,984	J503_04_01	802	1,274	1,525	2,271
					J503_04_02	849	1,382	1,670	2,540
J503_05	1,266	2,092	2,550	3,933	J503_05_01	897	1,395	1,655	2,439
					J503_05_02	450	710	850	1,267
					J503_05_03	298	494	601	1,088

4 Existing hydraulic model

The HEC-RAS models prepared by the HCFCD were used as a basis for the hydraulic analysis and updated as needed to reflect changes in the topography and land use as well as re-configured for the analysis of the two projects.

4.1 HCFCD model

The HCFCD HEC-RAS model consisted of a 1D/2D model of the entire watershed. The Spring Creek main stem was modeled with 1D cross sections for flows within the main channel and 2D zones for the floodplain. Northern tributaries in the model included Panther Branch, Mill Creek, Walnut Creek, Birch Creek, and Threemile Creek all of which were modeled using 1D cross sections. Southern tributaries included several HCFCD channels noted as J109, M101, J121, J131, J157, J158, J231 which were modeled using combined 1D/2D sections. The layout of the 1D/2D hydraulic model for the Spring Creek watershed is shown in Figure 4-1.

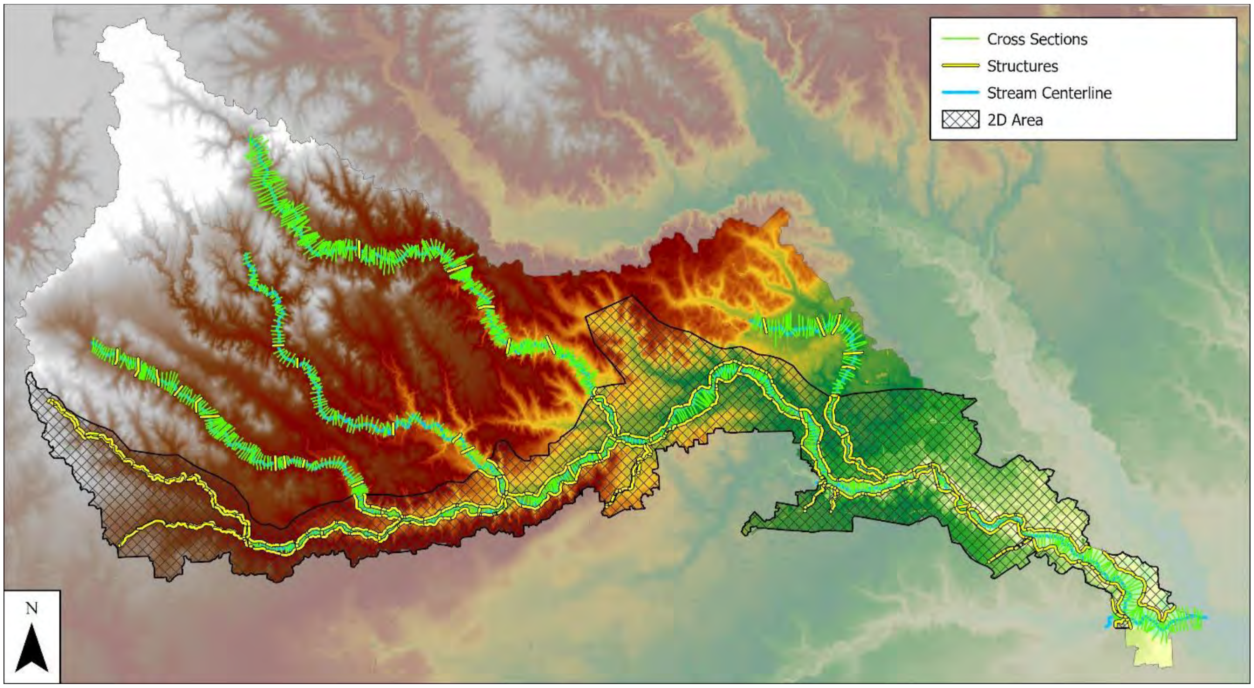

Figure 4-1 1D/2D Spring Creek Hydraulic Model Layout

4.2 Model adjustments

Since the HEC-RAS model was recently developed and calibrated, updates to the model were focused on Birch Creek and Walnut Creek to both accurately assess the existing conditions of the creeks as well as prepare for modeling the proposed projects. The layout of the revised 1D/2D hydraulic model for the Spring Creek watershed is shown in Exhibit 3.

4.2.1 2D area

The HCFCD HEC-RAS model consisted of 1D cross sections along Walnut Creek that extended into Birch Creek upstream of FM 1488. The cross sections upstream of FM 1488 were removed and replaced with a 2D area that covered the upstream portion of Walnut Creek and Birch Creek. A 2D area was used instead of 1D cross sections so that the alignment of the projects could be relocated as necessary and to best account for the footprint of the proposed projects. The 2D area boundary, shown in Figure 4-2, was delineated based on the upstream drainage area boundaries for Birch Creek and Walnut Creek.

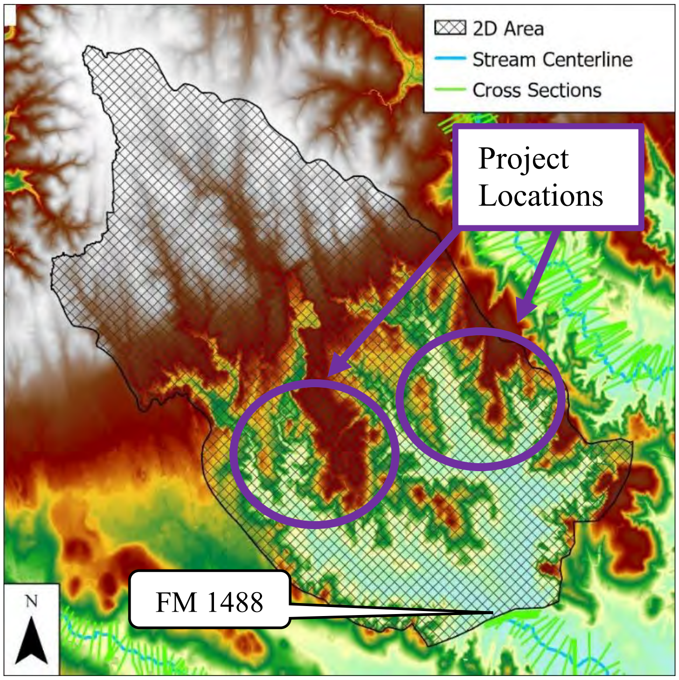

Figure 4-2 Walnut and Birch Creek 2D Area

4.2.2 Internal boundary conditions

The hydrographs for the drainage areas within the 2D area were added as internal boundary conditions. On the upstream end of the 2D area the boundary conditions were placed perpendicularly to the stream to simulate the upstream cross section of the stream. The boundary conditions for the drainage areas along the stream were placed following the stream centerline. The internal boundary conditions within the 2D area are shown in Figure 4-3.

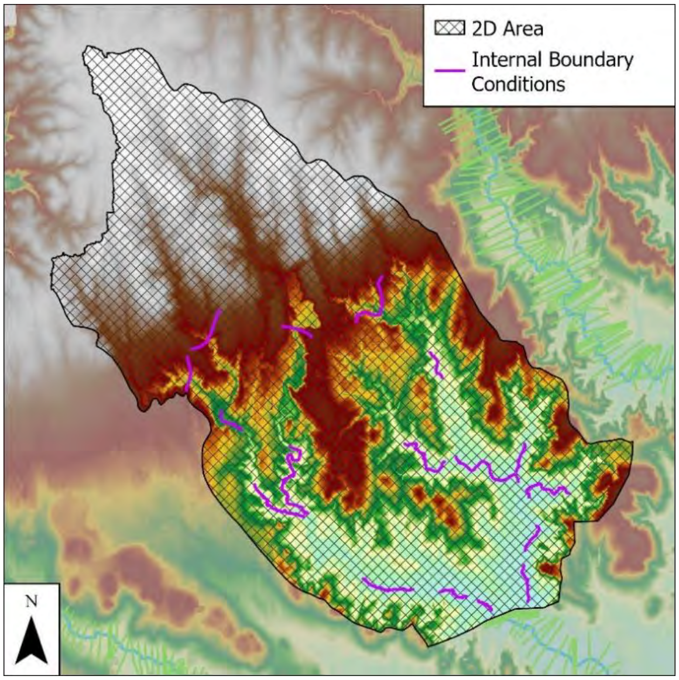

Figure 4-3 2D Area Internal Boundary Conditions

4.2.3 Breaklines

Breaklines were added to outline the centerlines of streams located within the 2D area. The breaklines oriented the cell alignments to best match the flow patterns for each creek. Figure 4-4 shows the breaklines that were added to the 2D area.

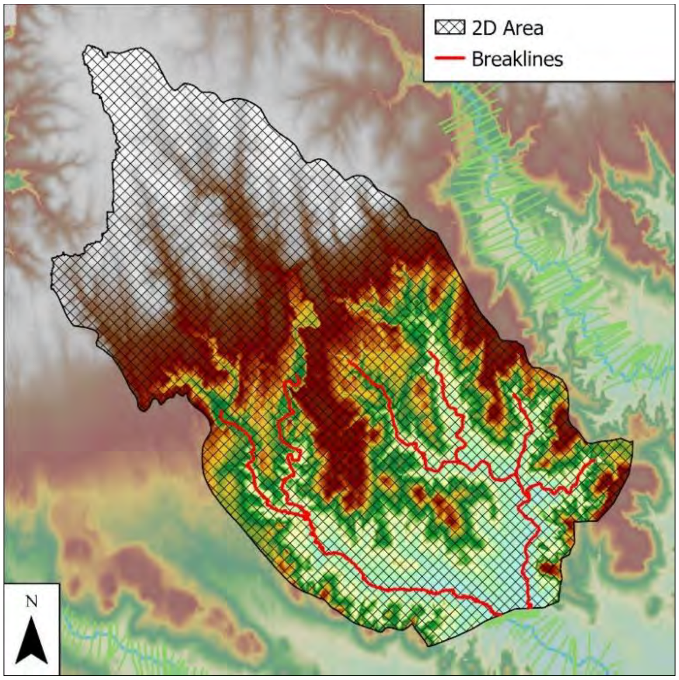

Figure 4-4 2D Area Breaklines

4.2.4 Cross sections adjustments

Cross sections along Walnut Creek downstream of FM 1488 were extended to contain the entire 0.2% ACE extents. Cross section elevations were obtained from the terrain data and roughness values corresponded to the existing land use values used within the HCFCD study. The adjusted cross sections are shown in Figure 4-5 below.

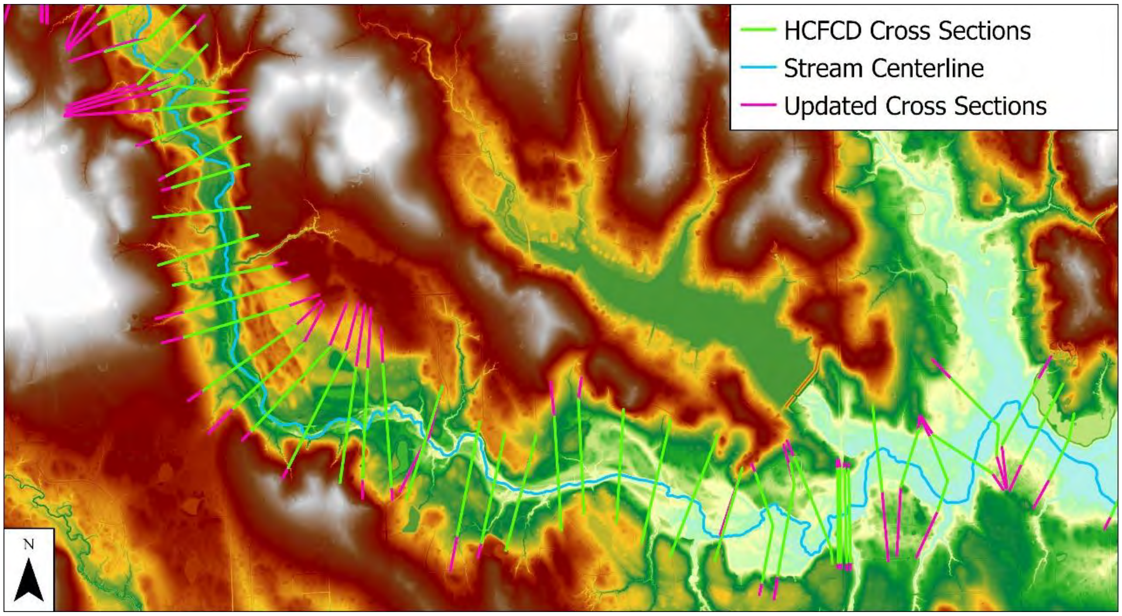

Figure 4-5 Walnut Creek Adjusted Cross Sections

Cross sections were also added along Walnut Creek from FM 1488 to the confluence with Birch Creek. The cross sections along Birch Creek Upstream of FM 1488 were removed and replaced with a 2D area. Cross section elevations were obtained from the terrain data and roughness values corresponded to the existing land use values used within the previous study. The cross sections that were added and adjusted on Walnut Creek are shown in Figure 4-6.

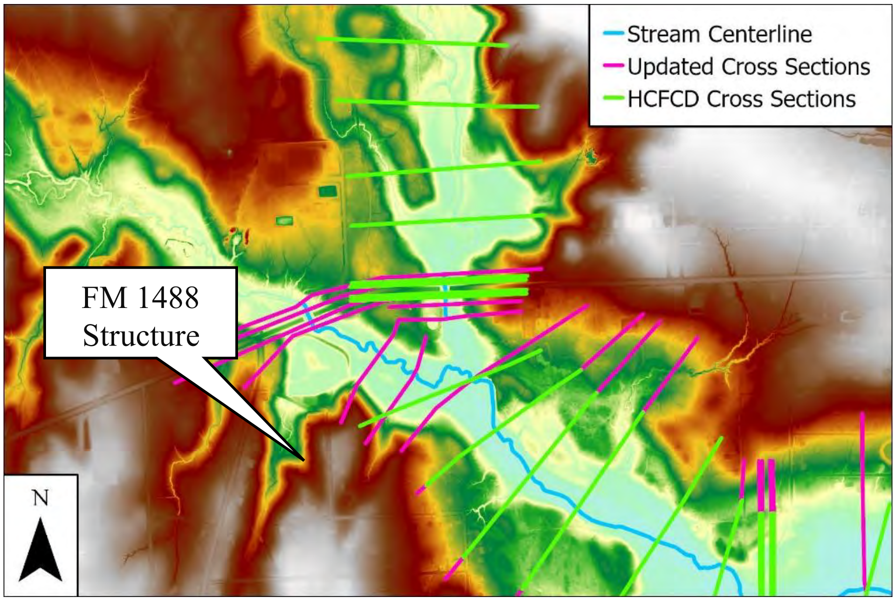

Figure 4-6 Walnut Creek Additional Cross Sections

4.2.5 *Additional structure*

The HCFCD HEC-RAS model included the FM 1488 crossing on Birch Creek, but not along Walnut Creek. The FM 1488 crossing along Walnut Creek was included from field survey provided by Waller County. Figure 4-7 shows the additional crossing location on FM 1488.

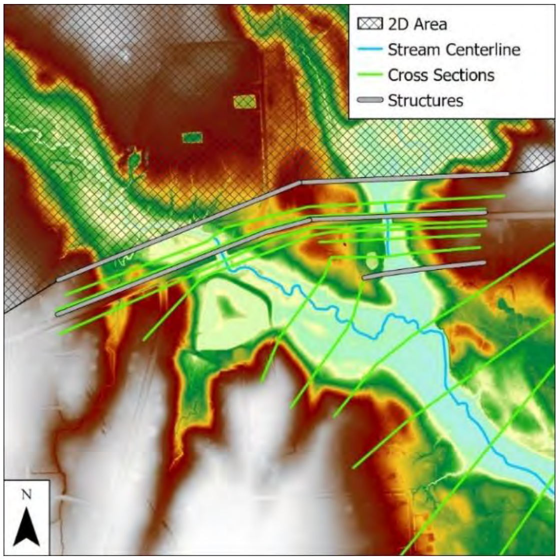

Figure 4-7 Walnut Creek Additional Structure

4.2.6 *1D/2D connections*

At the downstream end of the 2D area, two 2D connections were placed along Walnut Creek and Birch Creek. The 2D connections connected the 2D area and storage areas located on the upstream end of Walnut Creek and Birch Creek. This allowed for flow to go from the 2D area to the 1D sections of Walnut and Birch Creek. The two 2D connections and storage areas are shown in Figure 4-8.

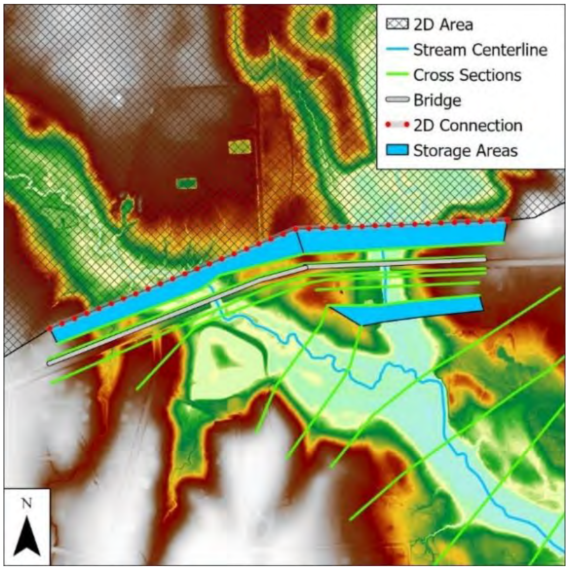

Figure 4-8 1D/2D Connections on Walnut and Birch Creek

5 Calibration

The existing conditions model was simulated for two historical storm events that were previously calibrated in the HCFCD study, and results were compared to ensure the model would provide reasonable results when compared to observed conditions. Table 5-1 below shows the Harvey (2017) observed water surface elevations, as well as discharge and water surface elevations for the HCFCD model and the revised existing conditions model.

Table 5-1 Harvey (2017) WSE and Discharge Comparisons

	SH 249	FM2978	Kuykendahl	I-45
HCFCD Discharge	55,315	80,021	80,522	97,444
Revised Discharge	53,774	75,857	76,638	95,019
HCFCD WSEL	165.61	154.19	141.00	111.19
Revised WSEL	165.37	153.76	140.79	111.81
Observed WSEL	165.08	153.74	140.62	111.40

Table 5-2 below shows the Memorial Day (2016) observed water surface elevations, as well as discharge and water surface elevations for the HCFCD model and the revised existing conditions model.

Table 5-2 Memorial Day (2016) WSE and Discharge Comparisons

	SH 249	FM2978	Kuykendahl	I-45
HCFCD Discharge	45,954	65,310	63,959	67,631
Revised Discharge	46,839	63,941	62,511	66,918
HCFCD WSEL	164.68	152.96	138.39	108.14
Revised WSEL	164.12	152.37	138.61	108.61
Observed WSEL	164.66	152.90	139.19	108.25

The revised existing conditions model has similar results to the previous HCFCD calibration as well as the observed conditions. These results showed that with the changes to the model, it remained calibrated and appropriate for the benefit analysis.

6 Existing conditions results

The calibrated models were simulated for the 10% ACE, 2% ACE, 1% ACE, and 0.2% ACE events to determine discharges and water surface elevations throughout the watershed.

6.1 Spring Creek watershed summary

The Spring Creek watershed has over 392 square miles of drainage area that consists of flows from Grimes, Waller, Montgomery, and Harris Counties. Most of the runoff reaches the creek through the four major northern tributaries: Threemile Creek, Walnut Creek, Mill Creek, and Panther Branch. Peak flows for the 1% ACE in Spring Creek are over 70,000 cfs at the confluence with the West Fork, making it one of the higher flow watersheds within the San Jacinto River basin. Figure 6-1 shows how the flows combine throughout the watershed and the 1% ACE peak discharges at key locations in the creek.

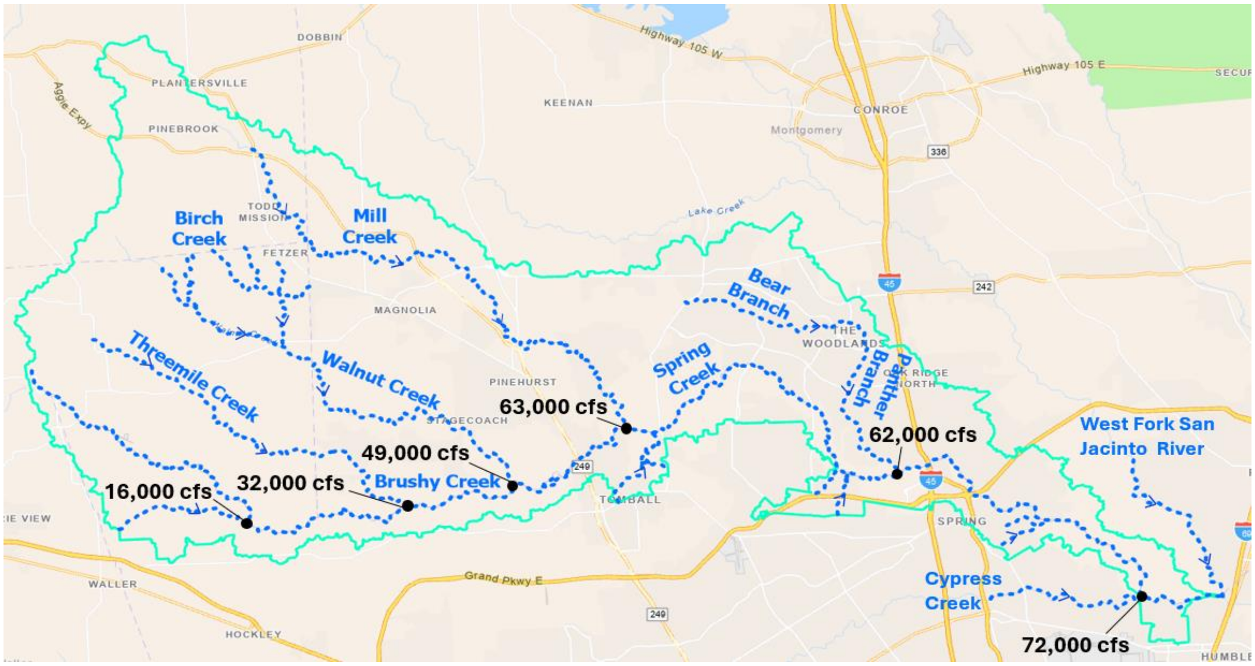

Figure 6-1 1% ACE (100-year) Flows Throughout Spring Creek

6.2 Discharge comparisons

Discharges for the 1% ACE event were compared between the effective FEMA model, HCFCD model, and the revised model used for the study to identify major changes. In general, the revised model discharges are higher than the effective FEMA model due to the application of Atlas 14 rainfall in the watershed but match well with the HCFCD discharges. The increases in discharges from the effective modeling indicate that flood risk may be higher than those shown on current FEMA maps (which are based on pre-Atlas 14 rainfall values).

Table 6-1 1% ACE (100-year) Existing Conditions Discharge Comparisons

	On Walnut Creek	Walnut Creek Confluence	SH 249	Kuykendahl	Gosling	I-45	West Fork Confluence
Effective Discharge	-	44,311	44,311	54,138	49,790	57,889	76,749
HCFCD Discharge	23,646	53,004	49,458	60,143	56,818	63,757	70,074
Revised Discharge	18,334	48,330	46,808	58,220	56,087	60,814	69,337

6.3 Water surface elevation comparisons

Water surface elevations for the 1% ACE event were compared between the effective FEMA model, HCFCD model and the revised model used for the study to identify major changes. In general, the revised model elevations are higher than the effective FEMA model due to the application of Atlas 14 rainfall in the watershed. The increases in elevation show that the watershed has more potential for flood risk than that shown on current FEMA maps (which are based on pre-Atlas 14 rainfall values).

Table 6-2 1% ACE (100-year) Existing Conditions WSE Comparisons

	On Walnut Creek	Walnut Creek Confluence	SH 249	Kuykendahl	Gosling	I-45	West Fork Confluence
Effective WSEL	-	168.75	161.87	136.99	126.00	107.24	67.10
HCFCD WSEL	187.54	170.46	164.53	138.76	127.81	111.26	71.42
Revised WSEL	186.95	170.06	164.09	138.44	127.52	111.07	71.29

6.4 Structure flooding summary

The resulting water surface elevations from the revised model were compared to assumed building finished floor elevations to identify the number of structures potentially flooded in each storm event. Spring Creek has a wide and deep floodplain and in general does not experience significant structural flooding until it reaches the 2% ACE event. This indicates that structural flooding is infrequent; however, when large storm events occur, there is the potential for widespread damages.

Exhibit 4 shows the structures that are potentially flooded for the 10% ACE, 2% ACE, 1% ACE, and 0.2% ACE events. The number of potentially flooded structures in Spring Creek for the 10% ACE, 2% ACE, 1% ACE, and 0.2% ACE events are in Table 6-3.

Table 6-3 Potentially Flooded Structures

Event	Potentially Flooded Structures
10% ACE	42
2% ACE	292
1% ACE	848
0.2% ACE	9,603

While damages occur throughout the floodplain of Spring Creek, concentrations of flood damages tend to occur in the following areas:

- Walnut Creek – There are nearly a hundred structures within the Walnut Creek floodplain that are most single-family residential housing in rural subdivisions. Most structures are older homes likely built prior to floodplain regulations and are subject to frequent flooding due to the creek.
- SH 249 – In this location there are low lying older neighborhoods that are susceptible to flooding in the 50-year event, as well as a large amount of commercial and industrial facilities that are inundated in the larger events. Most structures here reside in Montgomery County.
- FM 2978 – There are multiple residential structures and commercial/industrial facilities in Montgomery County that are susceptible to flooding in the larger events. This includes communities on Dobbin-Huffsmith Road and sections of the Northgrove neighborhood.
- Kuykendahl road – This area is mostly residential structures in Harris County that are susceptible to flooding in the 500-year event including the Creekside and Timmarron Lakes neighborhoods of The Woodlands.
- Between Gosling Rd and I-45 – There are multiple residential structures and a few commercial/industrial sites in Montgomery County that are susceptible to flooding in the larger events. Notable neighborhoods include Grogan's Point, Timber Lakes, and the commercial districts near Rayford Road.
- Grand Parkway – There are many residential structures around Grand Parkway in Montgomery County that are susceptible to flooding in the 500-year event including the Forest Village, Spring Trails, Fox Run, and Benders Landing neighborhoods.

The number of potentially flooded structures in Spring Creek for the 10% ACE, 2% ACE, 1% ACE, and 0.2% ACE events for each county are in Table 6-4.

Table 6-4 Potentially Flooded Structures

Event	Waller	Montgomery	Harris
10% ACE	4	30	8
2% ACE	17	251	24
1% ACE	32	743	73
0.2% ACE	60	7,575	1,968

7 Proposed projects

As recommended in the San Jacinto Regional Watershed Master Drainage Plan, two projects are proposed within the Walnut Creek watershed to provide flood mitigation along Spring Creek. One project is proposed on Walnut Creek upstream of FM 1488. The second project is proposed along the Birch Creek tributary also upstream of FM 1488.

7.1 Modeling approach

The proposed projects are located within the 2D area on the upstream end of Walnut Creek. The detention basins were modeled by adding 2D connections along the proposed project alignments. The 2D connections are shown in Figure 7-1.

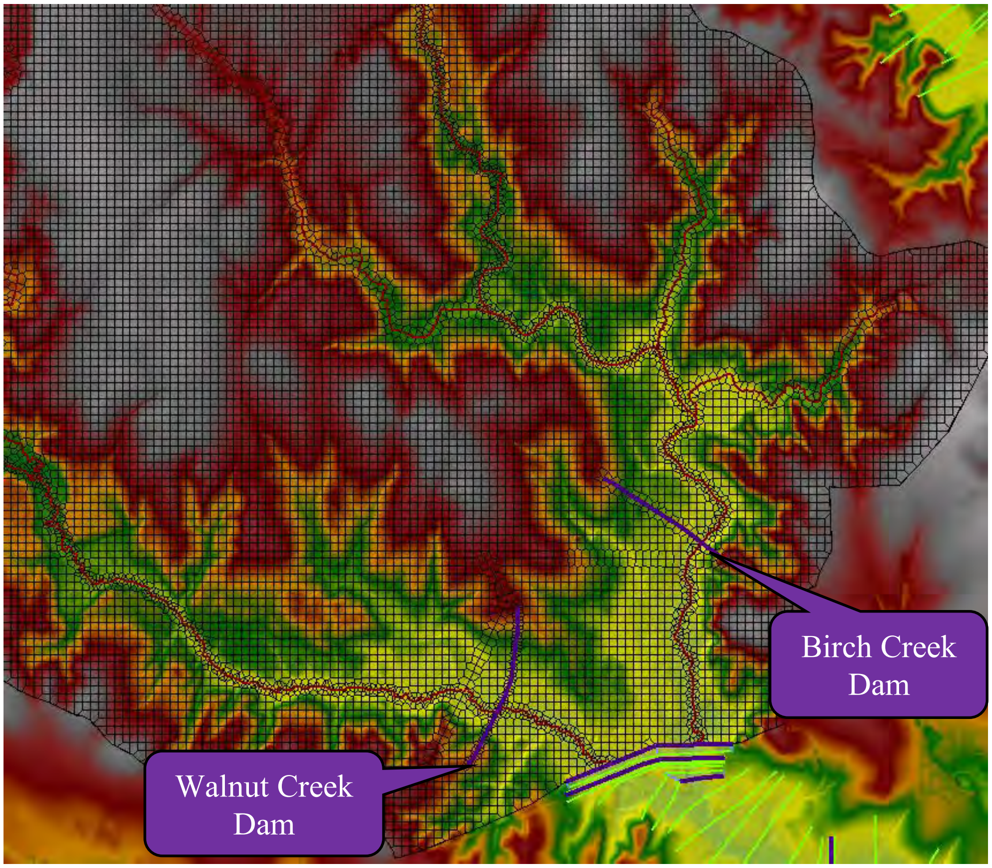

Figure 7-1 2D Connections Modeling Proposed Detention Basins

The 2D connections were modeled as ogee weirs and the weir elevations matched the top of dam and spillway elevations. The project outlets were modeled as large culvert openings at the flowline of the streams. A cross section view of the 2D connections for the proposed Walnut and Birch Creek projects are shown in Figure 7-2 and Figure 7-3.

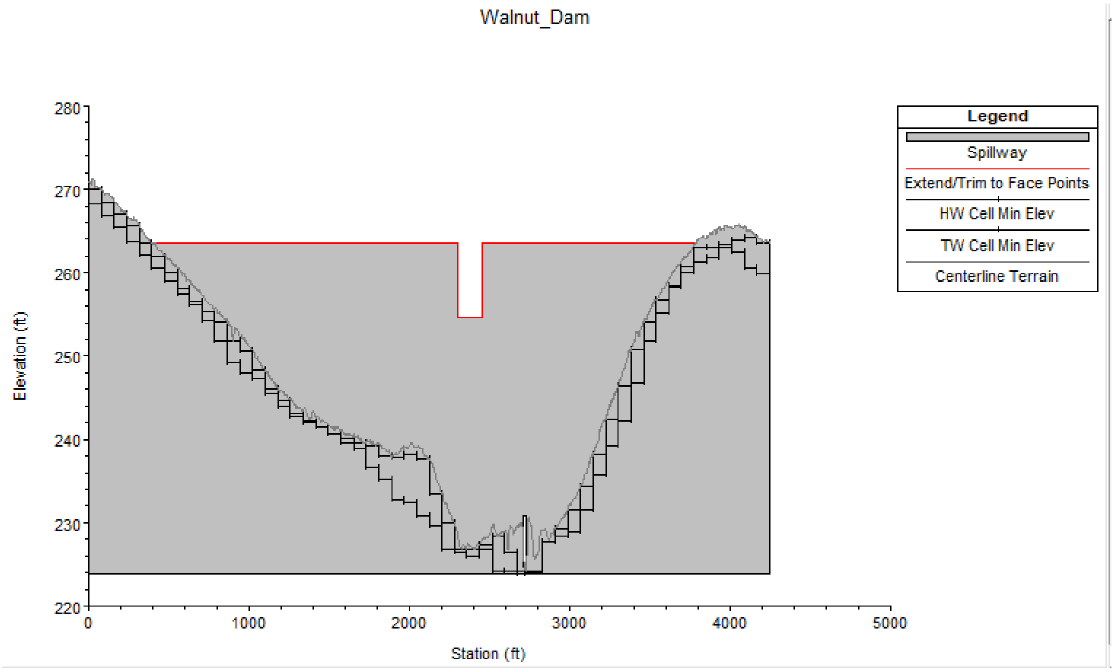

Figure 7-2 Walnut Dam Cross Section View of Dam 2D Connection

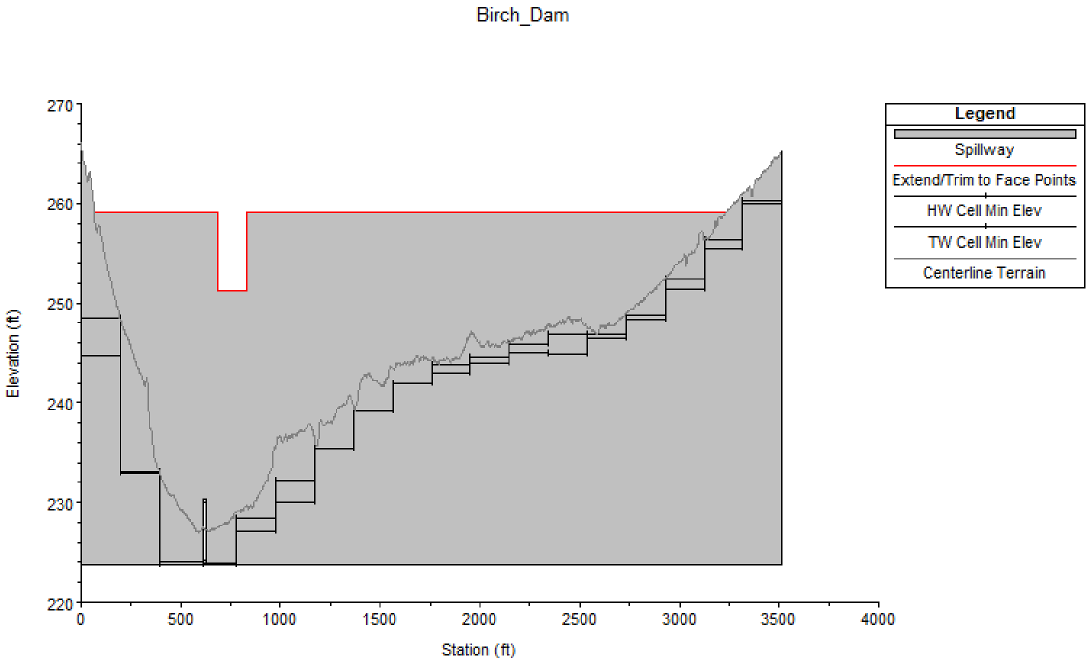

Figure 7-3 Birch Dam Cross Section View of Dam 2D Connection

7.2 Optimization

The detention basin elevations and footprints as presented in the SJRWMDP were initially simulated within the revised models to identify the design and benefits of the features. The models showed that the project areas were not completely full during large events and the size of the detention footprint could be reduced while providing similar benefits.

An optimization analysis was performed to determine the optimal volume within both the Birch and Walnut Creek detention basins that would minimize cost while still providing benefits along Spring Creek. Several different volume iterations for each dam were simulated and resulting water surface elevations compared at Kuykendahl Road. The Birch Creek comparisons are shown in Figure 7-4.

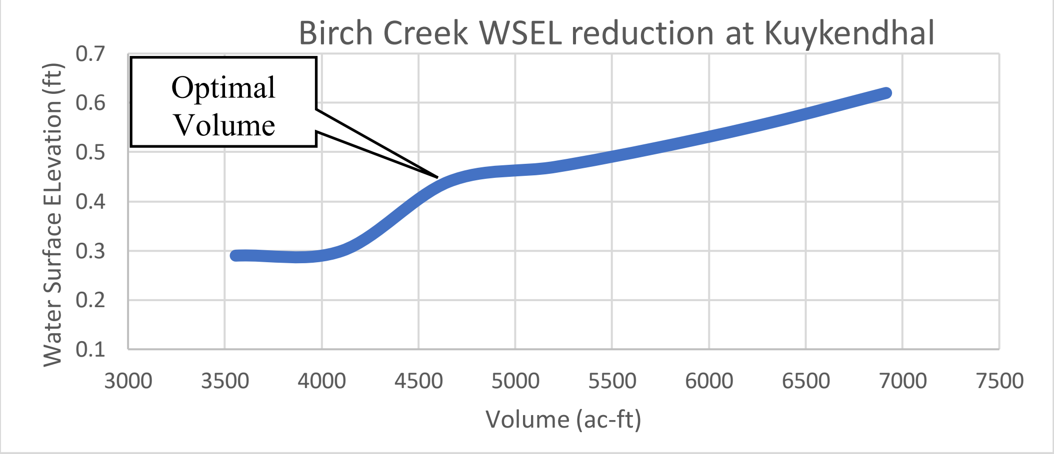

Figure 7-4 Birch Creek Volume Vs WSE Reduction Comparison

The optimization for Birch Creek showed that a detention volume of approximately 4,700 acre-feet would provide an optimized solution where the water surface elevation reductions are maximized while the volume is minimized. This volume became the new target volume for the project area behind the dam.

The Walnut Creek comparisons are shown in Figure 7-5. The optimization for Walnut Creek showed that a detention volume of approximately 6,600 acre-feet would provide an optimized solution where the water surface elevation reductions are maximized while the volume is minimized. This volume became the new target volume for the project area behind the dam.

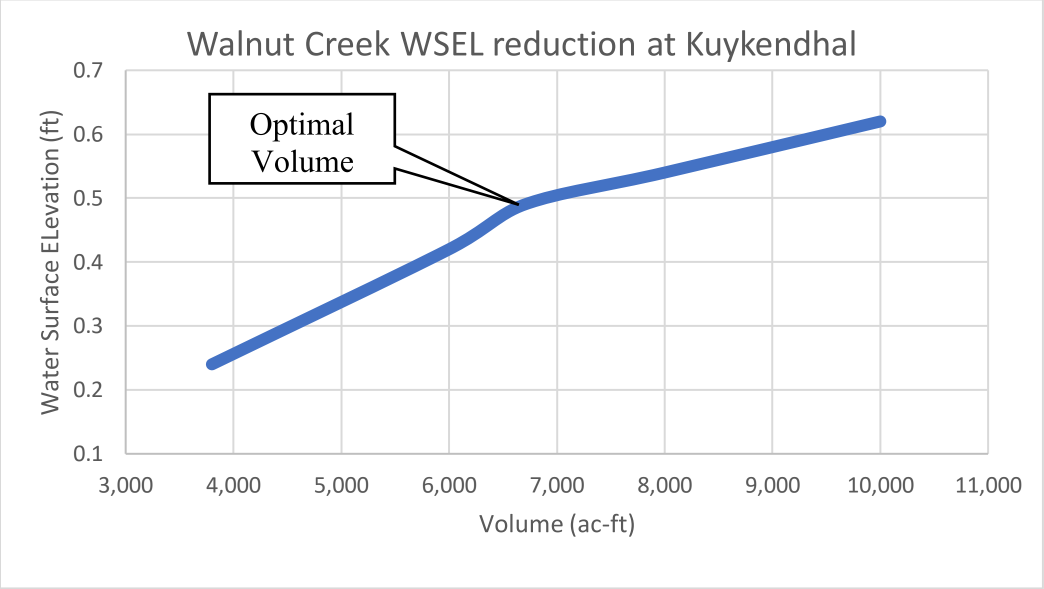

Figure 7-5 Walnut Creek Volume Vs WSE Reduction Comparison

7.3 Walnut Creek detention basin

7.3.1 Description

The detention basin will be a dry detention basin that passes low flows and everyday rain events to match existing conditions and detains water during larger storm events. Table 7-1 shows a variety of parameters detailing the size of the detention basin.

Table 7-1 Walnut Creek Detention Basin Parameters

	Dam Design Configuration
Spillway Elevation	254.7 ft
Spillway Length	175 ft
Top of Dam	263.6 ft
Max Dam Height	39.1 ft
1% ACE Inundation Area	940 ac
1% ACE Storage Capacity	7,300 ac-ft
Opening Size	6’ x 17’ RCB

7.3.2 Hydraulic results

The proposed Walnut Creek detention basin reduces the flow and water surface elevations throughout Spring Creek. Table 7-2 and Table 7-3 show the reduction in flow and water surface elevations between the proposed conditions and revised existing conditions for the 10% ACE, 2% ACE, 1% ACE, and 0.2% ACE events, respectively.

Table 7-2 Walnut Creek Flow Difference

Flow Difference (cfs)							
	On Walnut Creek	Walnut Creek Confluence	SH 249	Kuykendahl	Gosling	I-45	West Fork Confluence
10% ACE	-1,452	-334	-773	-450	-431	-223	0
2% ACE	-4,047	-903	-2,837	-1,954	-1,690	-1,582	-1,200
1% ACE	-6,381	-1,214	-3,397	-2,724	-3,409	-3,319	-1,296
0.2% ACE	-9,874	-1,956	-2,719	-1,982	-2,201	-1,324	-708

Table 7-3 Walnut Creek Water Surface Elevation Difference

Water Surface Elevation Difference (ft)							
	On Walnut Creek	Walnut Creek Confluence	SH 249	Kuykendahl	Gosling	I-45	West Fork Confluence
10% ACE	-1.15	-0.30	-0.29	-0.14	-0.12	-0.08	0.00
2% ACE	-2.1	-0.59	-0.58	-0.41	-0.37	-0.34	-0.25
1% ACE	-2.8	-0.75	-0.77	-0.54	-0.50	-0.38	-0.22
0.2% ACE	-3.03	-0.78	-0.71	-0.56	-0.41	-0.09	-0.15

7.4 Birch Creek detention basin

7.4.1 Description

The detention basin will be a dry detention basin that passes low flows and everyday rain events to match existing conditions and detains water during larger storm events. Table 7-4 shows a variety of parameters detailing the size of the dam.

Table 7-4 Birch Creek Detention Basins Parameters

	Dam Design Configuration
Spillway Elevation	251.2 ft
Spillway Length	175 ft
Top of Dam	259.1 ft
Max Dam Height	35.4 ft
1% ACE Inundation Area	690 ac
1% ACE Storage Capacity	4,800 ac-ft
Opening Size	6’ x 16’ RCB

7.4.2 Hydraulic results

The proposed Birch Creek detention basin reduces the flow and water surface elevations throughout Spring Creek. Table 7-5 and Table 7-6 show the reduction in flow and water surface elevations between the proposed conditions and revised existing conditions for the 10% ACE, 2% ACE, 1% ACE, and 0.2% ACE events, respectively.

Table 7-5 Birch Creek Flow Difference

	Flow Difference (cfs)						
	On Walnut Creek	Walnut Creek Confluence	SH 249	Kuykendahl	Gosling	I-45	West Fork Confluence
10% ACE	-979	-272	-678	-449	-417	-294	-1
2% ACE	-2,587	-639	-1,860	-1,230	-1,012	-850	-556
1% ACE	-4,399	-868	-2,349	-1,901	-1,988	-2,562	-811
0.2% ACE	-6,235	-1,384	-1,896	-1,263	-1,438	-826	-428

Table 7-6 Birch Creek Water Surface Elevation Difference

	Water Surface Elevation Difference (ft)						
	On Walnut Creek	Walnut Creek Confluence	SH 249	Kuykendahl	Gosling	I-45	West Fork Confluence
10% ACE	-0.79	-0.26	-0.26	-0.14	-0.12	-0.11	0.00
2% ACE	-1.36	-0.40	-0.38	-0.25	-0.22	-0.18	-0.11
1% ACE	-1.99	-0.52	-0.54	-0.36	-0.33	-0.23	-0.14
0.2% ACE	-1.87	-0.54	-0.49	-0.37	-0.26	-0.06	-0.09

7.5 Combined detention basin hydraulic results

The proposed Birch Creek and Walnut Creek detention basins together reduce the flow and water surface elevations throughout Spring Creek. Table 7-7 and Table 7-8 show the reduction in flow and water surface elevations between the proposed conditions and revised existing conditions for the 10% ACE, 2% ACE, 1% ACE, and 0.2% ACE events, respectively.

Table 7-7 Combined Detention Basins Flow Difference

	Flow Difference (cfs)						
	On Walnut Creek	Walnut Creek Confluence	SH 249	Kuykendahl	Gosling	I-45	West Fork Confluence
10% ACE	-2,309	-597	-1,467	-888	-821	-489	0
2% ACE	-7,601	-1,514	-4,699	-3,189	-2,726	-2,379	-1,775
1% ACE	-10,626	-1,917	-5,441	-4,614	-5,137	-4,689	-2,134
0.2% ACE	-17,676	-3,190	-4,596	-3,442	-3,792	-2,683	-1,191

Table 7-8 Combined Detention Basins Water Surface Elevation Difference

	Water Surface Elevation Difference (ft)						
	On Walnut Creek	Walnut Creek Confluence	SH 249	Kuykendahl	Gosling	I-45	West Fork Confluence
10% ACE	-1.96	-0.57	-0.56	-0.29	-0.25	-0.18	0.00
2% ACE	-2.90	-1.00	-1.00	-0.68	-0.61	-0.53	-0.37
1% ACE	-3.64	-1.18	-1.20	-0.88	-0.82	-0.67	-0.36
0.2% ACE	-4.47	-1.31	-1.20	-0.93	-0.70	-0.15	-0.26

8 Structure benefit analysis

Using the structure database as well as the hydraulic model results, an analysis of benefits was conducted to determine the number and frequency of structures that would benefit from the detention basins. The detention basins were evaluated both independently and in a combined scenario to understand the benefits for both the frequency storms as well as the historical storm events.

8.1 Frequency storms

The proposed Birch Creek and Walnut Creek detention basins reduce the number of structures impacted throughout Spring Creek for each of the modeled frequency events. Table 8-1 show the benefited structures for the 10% ACE, 2% ACE, 1% ACE, and 0.2% ACE events with the Birch Creek detention basin, Walnut Creek detention basin, and combined detention basin scenarios.

Table 8-1 Benefited Structures

	Birch		**Walnut**		**Birch + Walnut**	
	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]
10% ACE	37	2	36	5	30	11
2% ACE	252	48	230	70	199	101
1% ACE	802	160	738	225	629	335
0.2% ACE	9,207	303	9,032	484	8,762	795

[1] Structures that are still in the inundation area but the depth of flooding at the structure was reduced
[2] Structures that would no longer flood

The results show the detention basins have widespread benefit in reducing water surface elevations for all storm events due to the large detention volume provided. Over 9,000 structures show some type of benefit from each detention basin, including several hundred showing removal from the 1% ACE floodplain.

8.2 Historical storms

The proposed Birch Creek and Walnut Creek facilities were modeled with historical rainfall to determine the potential structural benefit if the facilities had been in operation prior to the events. Table 8-2 show the potential benefited structures for Hurricane Harvey (2017), Memorial Day (2016), and Tax Day (2016) with the Birch Creek detention basin, Walnut Creek detention basin, and combined detention basin scenarios.

Table 8-2 Potential Structural Benefits for Historical Storms

	Birch		Walnut		Birch + Walnut	
	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]
Harvey	3,749	254	5,081	321	5,351	542
Memorial Day	1,230	160	1,234	233	1,237	359
Tax Day	241	14	235	13	286	93

[1] Structures that are still in the inundation area but the depth of flooding at the structure was reduced
[2] Structures that would no longer flood

Table 8-3 shows the potential benefited structures for Hurricane Harvey (2017) within each county precinct.

Table 8-3 Potential Structural Benefits for Hurricane Harvey (2017) by Precinct

	Birch		Walnut		Birch + Walnut	
	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]
Harris County Precinct 3	662	45	701	53	701	90
Harris County Precinct 4	11	1	11	2	11	2
Montgomery County Precinct 2	423	45	423	50	432	74
Montgomery County Precinct 3	2,631	154	3,924	203	4,185	358
Waller County Precinct 2	22	9	22	13	22	18

[1] Structures that are still in the inundation area but the depth of flooding at the structure was reduced
[2] Structures that would no longer flood

Table 8-4 show the potential benefited structures for Memorial Day (2016) within each county precinct.

Table 8-4 Potential Structural Benefits for Memorial Day (2016) by Precinct

	Birch		Walnut		Birch + Walnut	
	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]
Harris County Precinct 3	120	19	120	23	120	33
Harris County Precinct 4	8	0	8	1	8	1
Montgomery County Precinct 2	361	41	361	64	361	95
Montgomery County Precinct 3	712	85	716	126	719	206
Waller County Precinct 2	29	15	29	19	29	24

[1] Structures that are still in the inundation area but the depth of flooding at the structure was reduced

[2] Structures that would no longer flood

Table 8-5 show the potential benefited structures for Tax Day (2016) within each county precinct.

Table 8-5 Potential Structural Benefits for Tax Day (2016) by Precinct

	Birch		Walnut		Birch + Walnut	
	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]
Harris County Precinct 3	18	6	18	6	18	7
Harris County Precinct 4	3	0	3	0	5	2
Montgomery County Precinct 2	126	0	123	0	151	47
Montgomery County Precinct 3	85	8	84	7	97	26
Waller County Precinct 2	9	0	7	0	15	11

[1] Structures that are still in the inundation area but the depth of flooding at the structure was reduced

[2] Structures that would no longer flood

9 Benefit cost analysis

For each of the evaluated detention basin alternatives described in Section 7, benefit-cost analyses (BCA) were performed to evaluate flood damage benefits for structures within the watershed of Spring Creek. The analyses utilized the FEMA BCA Toolkit version 6.0 to evaluate cost-effectiveness, adhering to accepted FEMA Benefit Cost Analysis practices. Base data was gathered and analyzed based on the methodologies described in Section 2.2.

9.1 Methodology

Information from the hydraulic models including water surface elevations for both existing conditions and each of the proposed detention basin alternatives were extracted to perform the analysis. In addition, base data such as residential and non-residential building footprints, building type and use, location, terrain, and building square footage were used within the analysis.

9.1.1 Period of analysis

The period of analysis was based on 50 years, which is the typical benefit period for dam projects which are in service for several decades. The costs over the 50-year period account for estimated environmental mitigation, property acquisition, and capital costs. Project cost for the individual as well as combined detention basins are summarized in Table 9-1.

Table 9-1 Project Costs Per Detention Basin Alternative

Project	Cost
Birch Creek	$105 M
Walnut Creek	$193 M
Combined	$298 M

9.1.2 Interest rates and price levels

All economic damages, benefits, and costs for drainage improvement projects use base fiscal year (FY) 2024. Future damages, benefits, and costs use the FEMA-default discount rate of 3.1 percent over the 50-year period of analysis.

9.1.3 Affected structures

Following the completed analysis of the existing condition results, the structures showing inundation based on the estimated finished floor elevations and the modeled inundation areas were inventoried into affected structures. This inventory captured all residential and non-residential buildings within 1,000 feet of the Spring Creek 0.2% ACE floodplain. Structures under 500 square feet were removed from this analysis. The finished floor elevations were determined by the elevation of the Lidar data at the centroid of the structure with an additional 1 foot. Affected structures were assigned flood depths for each of the modeled frequency events under existing conditions and each of the proposed alternatives.

9.1.4 Depth-damage functions

Depth-damage functions (DDF) are used to link the hydraulic data inputs, structure value, content value, and flood elevations to determine the monetary value of flood damages. These functions identify the percentage of the total damage value that correspond to the severity of flooding. Functions for damages to residential property structures and contents were obtained from the USACE EGM 04-01. Functions for non-residential properties are specific to building type and use; therefore, this study used the FEMA toolkit's default DDF by building type. Table 9-2 presents the Generic USACE depth-damage functions used for all residential structures in this study.

Table 9-2 USACE Residential Generic Depth-Damage Function

Flood Depth (ft.)	One Story, No basement Mean of Damages		Two or More Stories, No basement Mean of Damages	
	Structure	Contents	Structure	Contents
-2	0%	0%	0%	0%
-1	2.5%	2.4%	3.0%	1.0%
0	13.4%	8.1%	9.3%	5.0%
1	23.3%	13.3%	15.2%	8.7%
2	32.1%	17.9%	20.9%	12.2%
3	40.1%	22.0%	26.3%	15.5%
4	47.1%	25.7%	31.4%	18.5%
5	53.2%	28.8%	36.2%	21.3%
6	58.6%	31.5%	40.7%	23.9%
7	63.2%	33.8%	44.9%	26.3%
8	67.2%	35.7%	48.8%	28.4%
9	70.5%	37.2%	52.4%	30.3%
10	73.2%	38.4%	55.7%	32.0%
11	75.4%	39.2%	58.7%	33.4%
12	77.2%	39.7%	61.4%	34.7%
13	78.5%	40.0%	63.8%	35.6%
14	79.5%	40.0%	65.9%	36.4%
15	80.2%	40.0%	67.7%	36.9%
16	80.7%	40.0%	69.2%	37.2%

9.1.5 Building size

The building size is defined as the entire finished and livable space with disregard to unfinished basements, porches, attached garages, and other outside areas. For non-residential structures, the building size is equivalent to the first-floor area as it is assumed that only the first-floor area will sustain damages to the building and contents in a flood event. The first-floor area is a parameter to avoid inputting the full building square footage for a multi-story building.

The building sizes are sourced from the County Appraisal District databases for Harris, Montgomery, and Waller Counties.

9.1.6 Building replacement value and costs

The Building Replacement Value (BRV) is defined as the cost per square foot to replace an affected structure with a functionally equivalent building. This value is not the equivalent of the current market or assessed value of the structure. The BRV considers only the current cost of labor and the replacement materials. The FEMA default BRV of $100 per square foot was used in this analysis.

The total building replacement cost is found from the product of the BRV and the building size. The building replacement cost and the structure data are applied to a DDF as shown in Table 9-2 to calculate the expected annual losses (damages) in relation to the water surface elevations (depth) modeled at a given structure.

9.1.7 Content value

Content values for all residential and non-residential structures were calculated using the FEMA default method as a percentage of the BRV.

- For residential structures, the default method considers the contents to be equivalent to 100% of the building replacement costs mentioned in the BRV section. This total content value, which does not include permanent utilities such as plumbing and electrical systems, is applied to the contents section of a DDF as shown in Table 9-2.
- For non-residential structures, the content values are determined by the product of the first-floor area, the BRV, and an economic percentage value multiplier based on the building type and use. It was undetermined if the buildings were pre-engineered; therefore, all non-residential buildings were considered engineered buildings for a conservative approach. Table 9-3 displays the FEMA BCA Toolkit's default economic values by non-residential building type and use.

Table 9-3 Contents – Economic Percentage Values

Building Use	Value Multiplier (Engineered Building)	Value Multiplier (Pre-engineered Building)
Apartment	10%	12%
Clothing, Retail	29%	36%
Industrial Light	38%	47%
Office One-Story	12%	14%
Service Station	66%	83%
Warehouse, Non-Refrigerated	36%	43%

9.1.8 Additional benefits

A major component in determining benefits for flood mitigation projects is based on the effects of depth reduction and evaluated monetary damages, described in the previous sections. Additional benefits may be incorporated into the overall benefits including the effects the project may have on services and residents.

Displacement

An additional part of the standard benefits calculation includes the residential and non-residential displacement costs after a flood event. Residential displacement losses represent the cost to residents caused by being out of their home after a storm event causes damage to the structure. The cost of residential displacement was calculated using the method and the recommended values in the FEMA BCA Toolkit. These costs account for temporary lodging for each displaced household and increased meal costs associated with eating out of the home for each displaced resident. The unit costs are sourced from the U.S. General Services Administration's (GSA) "FY 2025 per diem rates for Texas."[1]

Expected annual benefits depend on the number of displaced residents per the depth of flooding at the structures. The total benefits associated with the avoidance of residential displacement costs are summarized in Table 9-4. The meal cost per person on a daily basis is found by the difference of the U.S. GSA's unit costs of meals per day per capita and the daily meal cost of eating at home per person.

For this study, the population per household was not accounted for from the appraisal districts. The U.S. Census Bureau's "QuickFacts for Texas" states the average persons per households is 2.7; therefore, the number of residents per structure was rounded to 3 persons per household for the BCA input requirement.[2]

Table 9-4 Residential Displacement Unit Cost

County	Meals per Day per Capita	Cost of Eating at Home per Day	Meal Cost per Person per Day	Hotel per Day per Family, up to 5 People
Waller (standard rate)	$68	$10	$58	$110
Harris and Montgomery	$80	$10	$70	$128

Nonresidential displacement losses represent the rental costs and one-time costs that an owner would experience for loss of function. Rental costs consider that the non-residential structure will rent the same amount of space required for the damaged building use. One-time costs consider the costs required to transport relevant items to the alternate rental locations and other pertinent costs due to displacement.

The rental costs and the one-time costs are determined by the building type and building use, and the FEMA default values are summarized in Table 9-5

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[1] U.S. General Services Administration FY 2025 per diem rates for Texas

[2] U.S. Census Bureau QuickFacts Texas Table

Table 9-5 Non-Residential Displacement Unit Cost

Label	Occupancy Class	Rental Cost per sq.ft. per day	One-time Cost per sq.ft.
COM3	Personal and Repair Services	$ 1.83	$ 1.28
COM4	Professional/Technical/Business	$ 1.83	$ 1.28
IND2	Light	$ 0.37	$ 1.28
AGR1	Agriculture	$ 0.91	$ 0.91
GOV1	General Services	$ 1.37	$ 1.28

Social benefits

Flooding can be a mental stress and added anxiety to residents experiencing natural disasters. Social benefits are based on FEMA BCA standard values which include $2,443 for treatment of mental stress and anxiety for each resident of a home benefitted by the project and $8,736 for the loss of production for full-time workers impacted by the flooding.

To standardize the social benefits, the same assumption for number of residents from the displacement section was made. For number of working residents, the U.S. Census Bureau provides Texas's total employment and the total households to provide an average of 1 working resident per household[1]. With the FEMA standard values and the assumptions made for residents and working residents, the social benefits applied is equal to $16,065 per benefitted residential structure.

9.2 Walnut Creek results

A benefit-cost analysis was performed for the Walnut Creek Detention Basin Alternative using the water surface elevation results described in Section 7.3 with the parameters described in Section 9.1. The benefit value derived for this alternative was used along with the engineering opinion of probable project cost to generate the final benefit-cost ratio for the Walnut Creek Detention Basin, as shown in Table 9-6.

Table 9-6 Walnut Creek Detention Basin BCA Results

Building Type	Benefits		Total
	Standard	Social	
Residential	$42,899,652	$141,420,195	$184,319,847
Non-Residential	$17,467,588	$0	$17,467,588
		Total Mitigation Benefits	**$201,787,435**
		Total Project Cost	**$193,071,637**
		Project BCR	**1.05**

Using the total mitigation benefits and the associated project cost, the Walnut Creek Detention Basin has a benefit cost ratio (BCR) of 1.05.

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[1] U.S. Census Bureau QuickFacts Texas Table

9.3 Birch Creek results

A benefit-cost analysis was performed for the Birch Creek Detention Basin Alternative using the results described in Section 7.4 with the parameters described in Section 9.1. The benefit value derived for this alternative was used along with the engineering opinion of probable project cost to generate the final benefit-cost ratio for the Birch Creek Detention Basin, as shown in Table 9-7.

Table 9-7 Birch Creek Detention Basin BCA Results

Building Type	Benefits		Total
	Standard	Social	
Residential	$33,369,403	$141,163,155	$174,532,558
Non-Residential	$10,814,136	$0	$10,814,136
		Total Mitigation Benefits	**$185,346,694**
		Total Project Cost	**$105,338,718**
		Project BCR	**1.76**

Using the total mitigation benefits and the associated project cost, the Birch Creek Detention Basin has a benefit cost ratio of 1.76.

9.4 Combined Detention Basin results

A benefit-cost analysis was performed for the Combined Detention Basins Alternative using the results described in Section 7.5 with the parameters described in Section 9.1. The benefit value derived for this alternative was used along with the engineering opinion of probable project cost to generate the final benefit-cost ratio for the Combined Detention Basins Alternatives, as shown in Table 9-8.

Table 9-8 Birch-Walnut Creek Detention Basins BCA Results

Building Type	Benefits		Total
	Standard	Social	
Residential	$49,527,304	$141,709,365	$191,236,669
Non-Residential	$20,504,771	$0	$20,504,771
		Total Mitigation Benefits	**$211,741,440**
		Total Project Cost	**$298,410,355**
		Project BCR	**0.71**

Using the total mitigation benefits and the associated project cost if constructed together, the Combined Birch Creek and Walnut Creek Detention Basins have a benefit cost ratio of 0.71. The lower BCR in comparison to the BCRs of the independent detention basins is attributed to the significant cost increase of a project for two detention basins and due to the nature of social benefits being attributed to the number of affected structures rather than the changes in depth-to-damage. For the combined case, social benefits are still only counted per the structures benefiting so remain the same as if only one basin was being constructed.

10 Potential funding opportunities

Due to the size of the projects, funding for the detention basins will likely require a combination of multiple funding sources from both the local entities as well as partnerships with the state and federal governments. Each funding source may have specific requirements for meeting the source and stipulations as to the types of projects or parts of projects that it can fund. Below is a summary of current potential funding sources separated out by potential agency.

10.1 Federal Emergency Management Agency (FEMA)

Assuming both projects retain a benefit cost ratio greater than 1.0 in subsequent detailed design efforts, FEMA funding can be a source for project design and construction. FEMA has a variety of funding opportunities with eligible activities that range from Hazard Mitigation Planning to conveyance and detention improvements to flood warning system enhancements. The entity that applies must have an adopted Hazard Mitigation Plan.

10.1.1 Flood Mitigation Assistance (FMA)

- Project Type: Planning, Engineering, Design, Construction
- Maximum Funding: $25 million
- Cost Share: 75% FEMA, 25% local
- Frequency: Annually
- Administrator: Texas Water Development Board
- Restrictions: BCR > 1.0

10.1.2 Hazard Mitigation Grant Program (HMGP)

- Project Type: Planning, Engineering, Design, Construction
- Maximum Funding: $25 million
- Cost Share: 75% FEMA, 25% local
- Frequency: After federally-declared disaster
- Administrator: Texas Division of Emergency Management
- Restrictions: BCR > 1.0

10.2 US Housing and Urban Development Funding (HUD/GLO)

The HUD Community Development Block Grants (CDBG) provide opportunities for communities following a major disaster. HUD funding is administered through the General Land Office (GLO) for Texas and can also be filtered through the local council of governments (Houston-Galveston Area Council [HGAC] for our region). HUD funding generally does not have a BCR requirement but may have a low-moderate income emphasis for the applying entity. Funding opportunities may have different thresholds of percent Low-Moderate Income (LMI) benefitting from the project.

10.2.1 Community Development Block Grant – Disaster Relief (CDBG-DR)

- Project Type: Planning, Engineering, Design, Construction
- Maximum Funding: Varies
- Cost Share: 100% HUD
- Frequency: After federally-declared disaster
- Administrator: General Land Office
- Restrictions: Large emphasis on LMI communities

10.2.2 Community Development Block Grant – Mitigation (CDBG-MIT)

- Project Type: Planning, Engineering, Design, Construction
- Maximum Funding: Varies
- Cost Share: 100% HUD
- Frequency: After federally-declared disaster
- Administrator: General Land Office
- Restrictions: Large emphasis on LMI communities

10.3 Natural Resource Conservation Service (NRCS)

NRCS's natural resources conservation programs help people reduce soil erosion, enhance water supplies, improve water quality, increase wildlife habitat, and reduce damages caused by floods and other natural disasters. NRCS funds have been used locally for conservation efforts or repair of damaged infrastructure. The funding requires projects to be completed relatively quickly.

10.3.1 Watershed and Flood Prevent Operations (WFPO)

- Project Type: Planning, Engineering, Design, Construction
- Maximum Funding: $5 million (unless otherwise approved by Congress)
- Cost Share: Varies
- Frequency: Annually
- Administrator: NRCS (US Department of Agriculture)
- Restrictions: Benefit area must include 20% agriculture

10.4 Congressional Allocation

Congress can directly allocate funding for a drainage infrastructure project through the annual appropriations process or by authorizing specific funding in legislation. This typically involves a member of Congress submitting a request—often in the form of a Community Project Funding (CPF) or earmark—for a particular project in their district or state. If approved, the request is included in one of the appropriations bills passed by Congress and signed into law by the President. Alternatively, Congress can include funding for such projects in larger infrastructure or disaster relief bills, directing federal agencies such as the Army Corps of Engineers or the Environmental Protection Agency to administer the funds. This process ensures that federal dollars are designated for targeted improvements, like stormwater management systems or flood mitigation infrastructure, that address local needs and protect communities. Projects funded with direct allocation may have to follow the rules of the funding agency such as that USACE funding cannot be used for land acquisition.

10.5 Texas Water Development Board (TWDB)

The TWDB has several sources of funding available for flood mitigation projects and has recently increased awareness of these projects and programs through the regional flood planning initiative. These two projects were included in the latest amendment of the plan which will make them eligible for state funding. Some of these funding sources are relatively new and standard requirements may be subject to change.

10.5.1 Flood Infrastructure Fund (FIF)

Project Type:	Planning, Engineering, Design, Construction
Maximum Funding:	$19 million (current cycle)
Cost Share:	30%-75%, low interest loans
Frequency:	Bi-annually
Administrator:	TWDB
Restrictions:	Subject to state legislature funding the program

10.6 Local funding

Local funds will need to be raised for the local share required in most state and federal sources as well as for the long-term operations and maintenance of the basins.

10.6.1 Bonds

Bond funding can be used for flood protection and management projects. Bonds typically provide project specific financing that requires proposed improvements to be ready for design and construction and meet the priorities set by the funder. Although repayment terms can offer low or no interest financing, these sources do require full repayment.

10.6.2 Fees and ad valorem taxes

A development impact mitigation fee is a tax that is imposed as a precondition for the privilege of developing land. Since the proposed projects address existing conditions and are not meant for mitigating developing land, imposing a fee on new development to address pre-existing flooding conditions may be difficult to implement. Ad valorem taxes are based on the value of a transaction of a property. Sales taxes or property taxes are ad valorem taxes that could be considered for funding the projects.

10.6.3 Public private partnerships

While there is not an identified stream of funding available for private investment, it may be considered as an option if the opportunity is presented. The detention basins will provide ample space for recreational activities outside of storm events and dual use of the basins should be explored. The watershed also includes several different industrial and commercial developments that were significantly damaged in recent flood events and whose owners may be looking for opportunities to reduce flood risk in the area.

11 Recommendations and next steps

The hydrologic and hydraulic analyses show that the Walnut Creek and Birch Creek Detention Basins both individually, as well as combined, would provide a widespread benefit to the Spring Creek watershed. By detaining flows within each tributary, the detention basins reduce the overall flow in Walnut Creek and Spring Creek. The structural analysis showed that the reductions in both flow and water surface elevation translate to reductions in flooding throughout the watershed for both the frequency as well as historical storm events.

Table 11-1 Benefitted Structures

	Birch		**Walnut**		**Birch + Walnut**	
	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]	Reduced[1]	Removed[2]
10% ACE	37	2	36	5	30	11
2% ACE	252	48	230	70	199	101
1% ACE	802	160	738	225	629	335
0.2% ACE	9,207	303	9,032	484	8,762	795

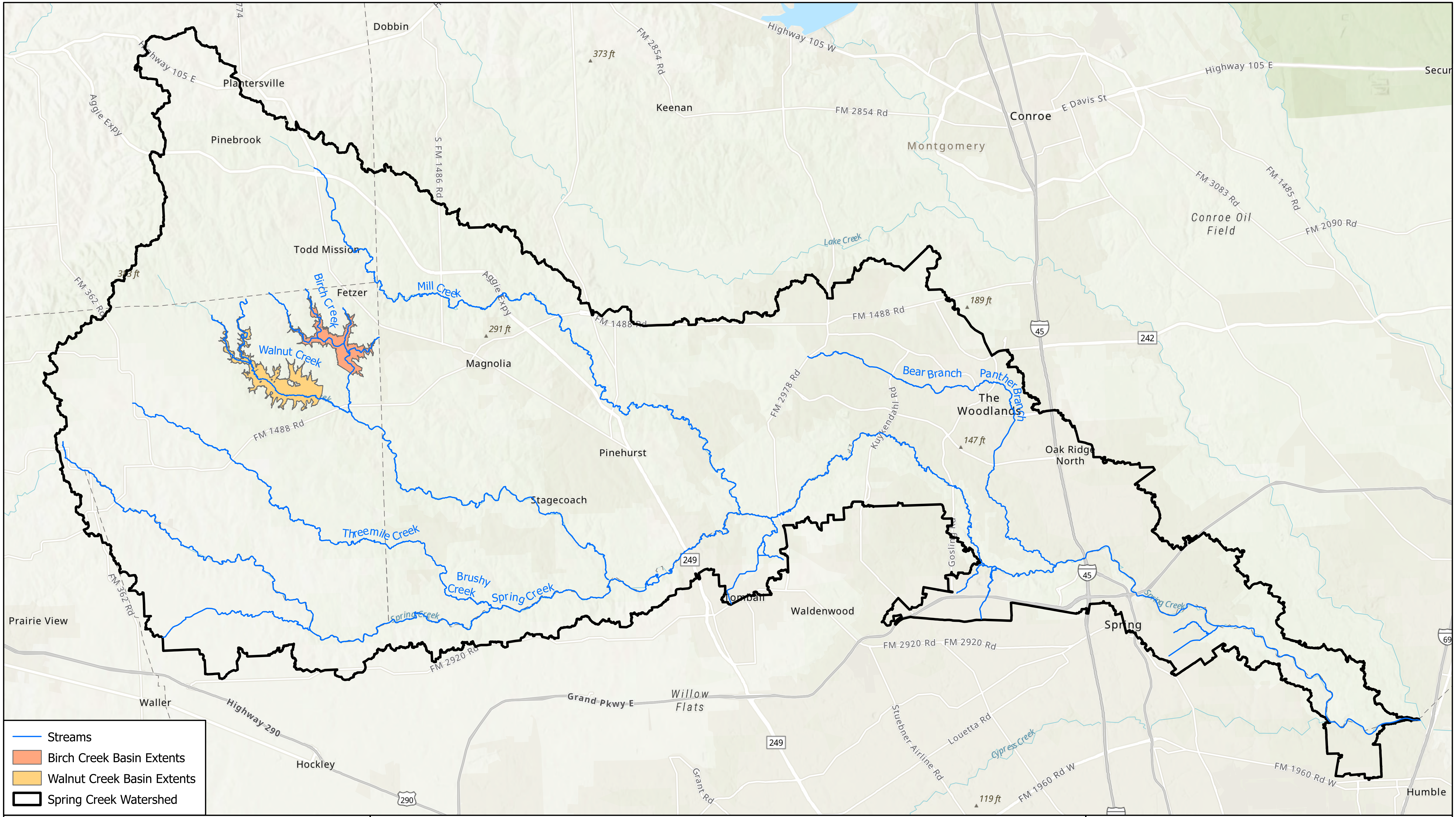
[1] Structures that are still in the inundation area but the depth of flooding at the structure was reduced
[2] Structures that would no longer flood

The benefit cost analysis shows that both detention basins have a positive benefit cost ratio when analyzed individually and below a 1.0 benefit cost when analyzed together due to the application of social benefits. Applications for these projects should keep these projects as separate in order to maximize the benefit cost ratio.

Table 11-2 Final Benefit Cost Ratio

	Cost	**Benefit**	**BCR**
Birch Creek	$105,338,718	$185,346,694	1.76
Walnut Creek	$193,071,637	$201,787,435	1.05
Combined	$298,410,355	$211,741,440	0.71

Several funding sources are available for project funding, and construction as well as operations and maintenance will likely require a combination of federal, state, and local funding sources. The project owner should begin funding discussions with local, state, and federal agencies to determine the most probable source and begin funding applications.



halff

Exhibit 1
Spring Creek Watershed

Spring Creek Watershed Flood Control Dams Engineering Feasibility Study

Path: A:\42000s\42682\001\GIS\Working\PM\Exhibit APRXs\Exhibit_1_Watershed_Layout.aprx

2/14/2025

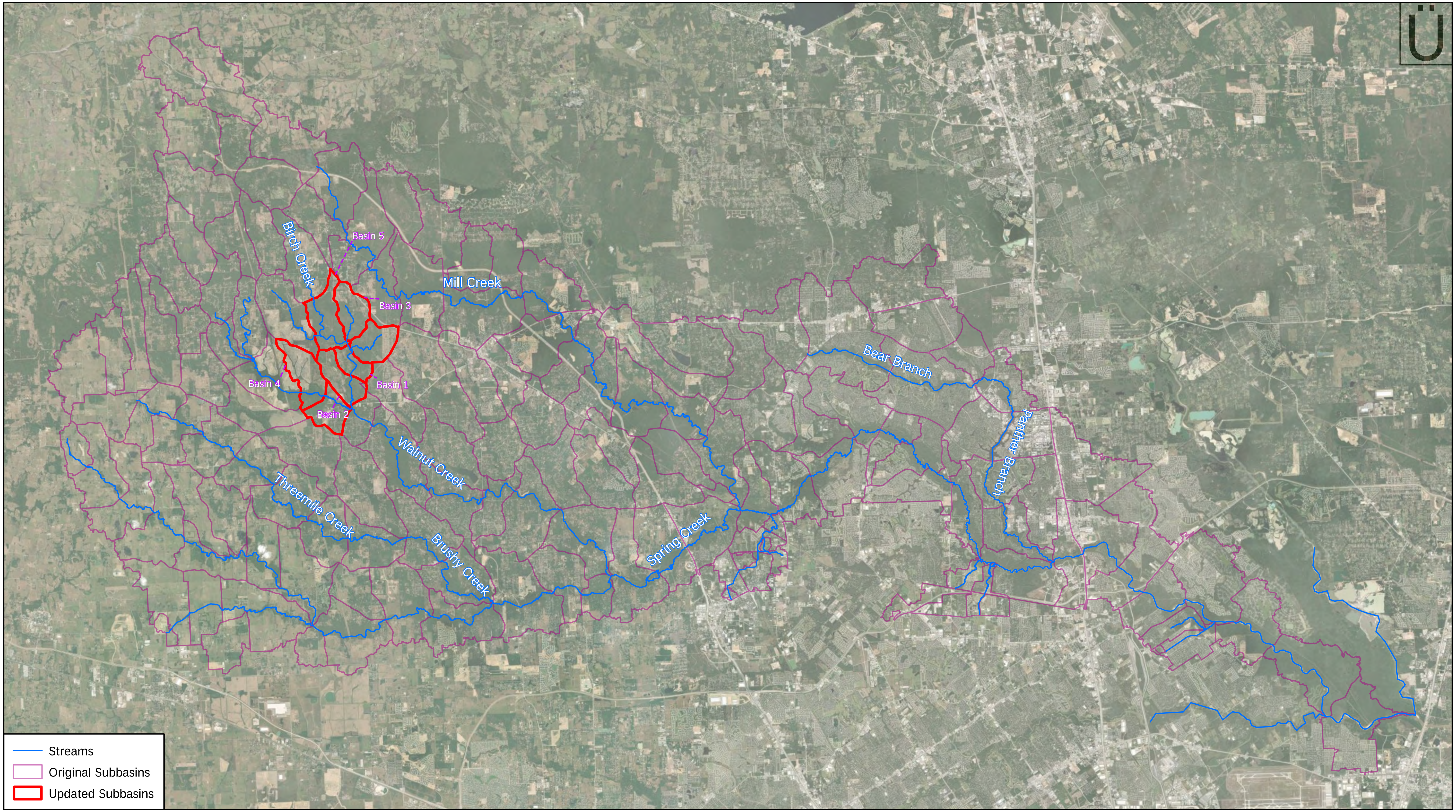


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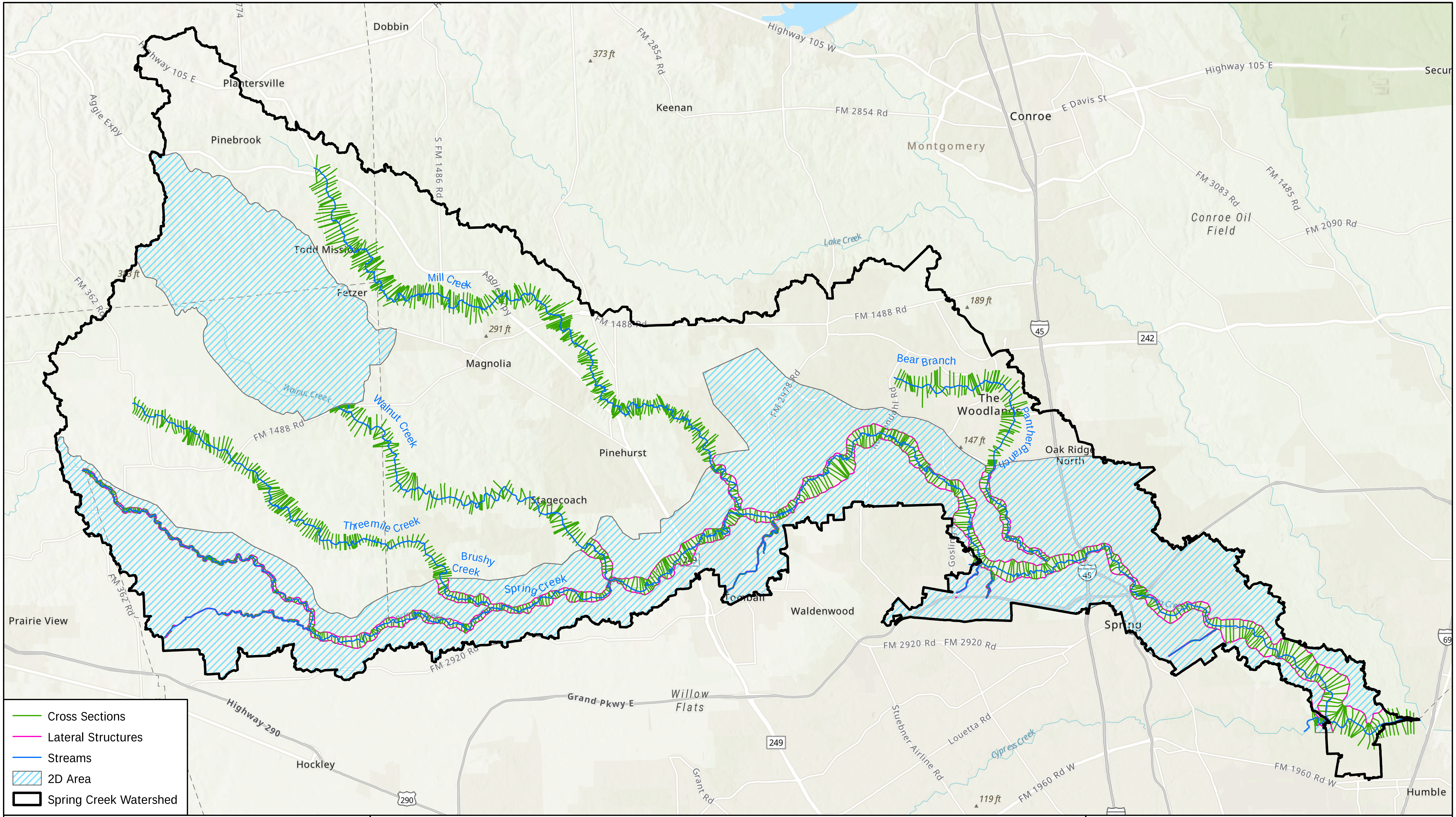
Exhibit 2
Spring Creek Watershed Subbasins

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study



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5/7/2025

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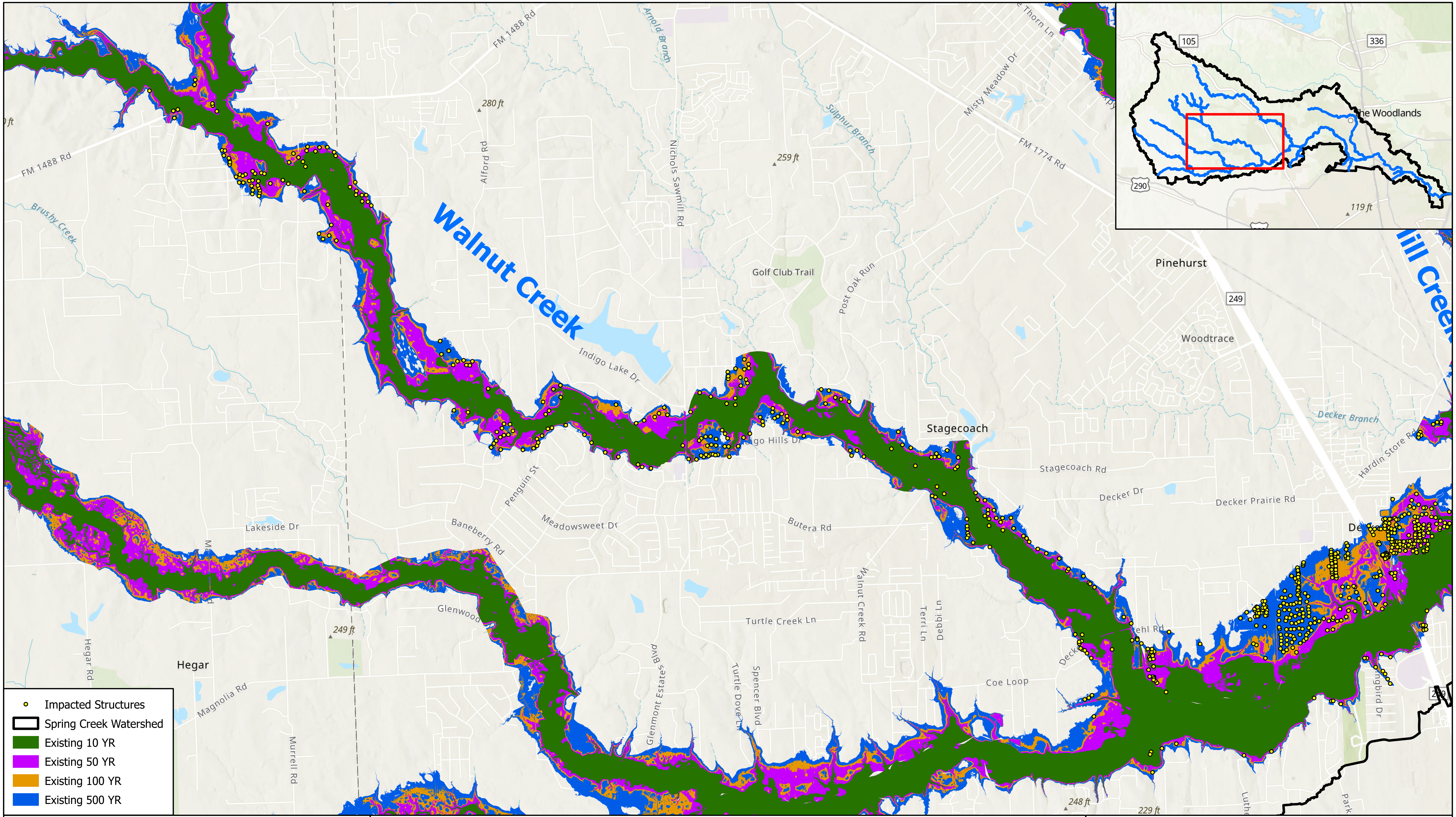
Exhibit 3
Modeling Extents & Layout

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study



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2/13/2025

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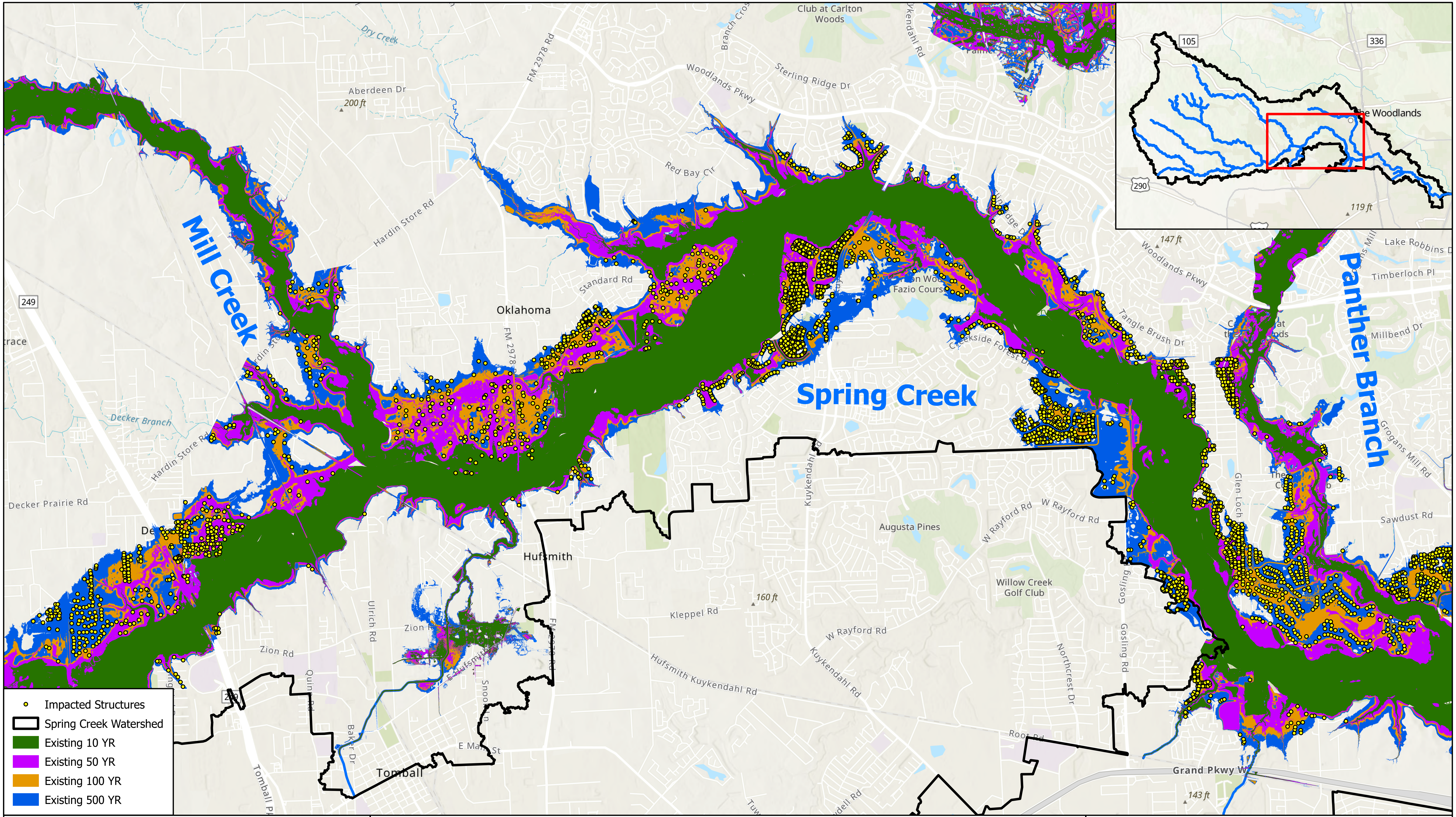
Exhibit 4
Existing Impacted Structures

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study



Path: A:\42000s\42682\001\GIS\Working\PM\Exhibit APRXs\Exhibit_4_Existing_Impacted_Structures.aprx

2/13/2025

halff

Exhibit 4
Existing Impacted Structures

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study

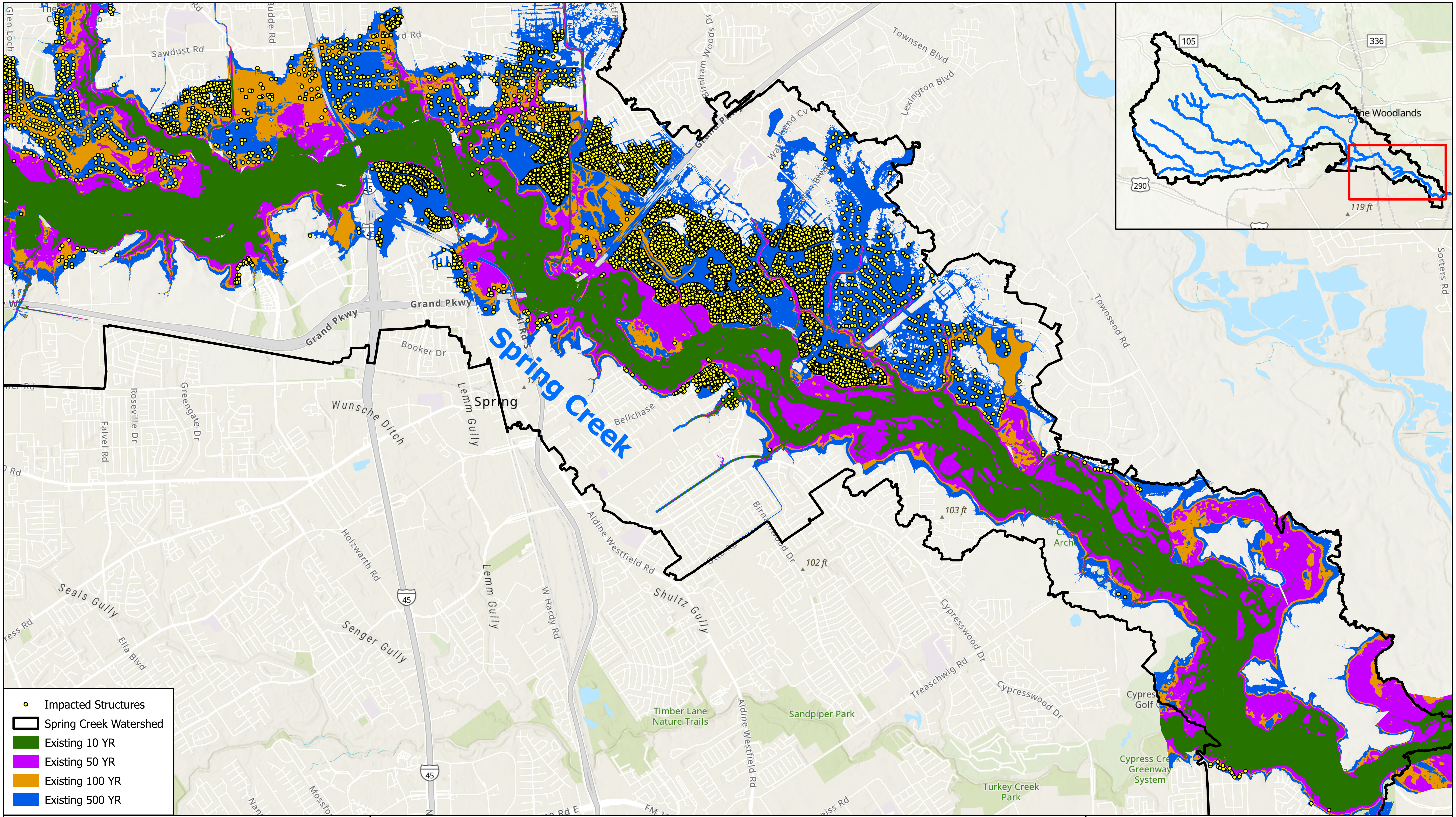


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Exhibit 4
Existing Impacted Structures

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study

Path: A:\42000s\42682\001\GIS\Working\PM\Exhibit APRXs\Exhibit_4_Existing_Impacted_Structures.aprx

2/13/2025

Path: A:\42000s\42682\001\GIS\Working\PM\Exhibit APRXs\Exhibit_5_Benefited_Structures_Walnut.aprx

2/14/2025

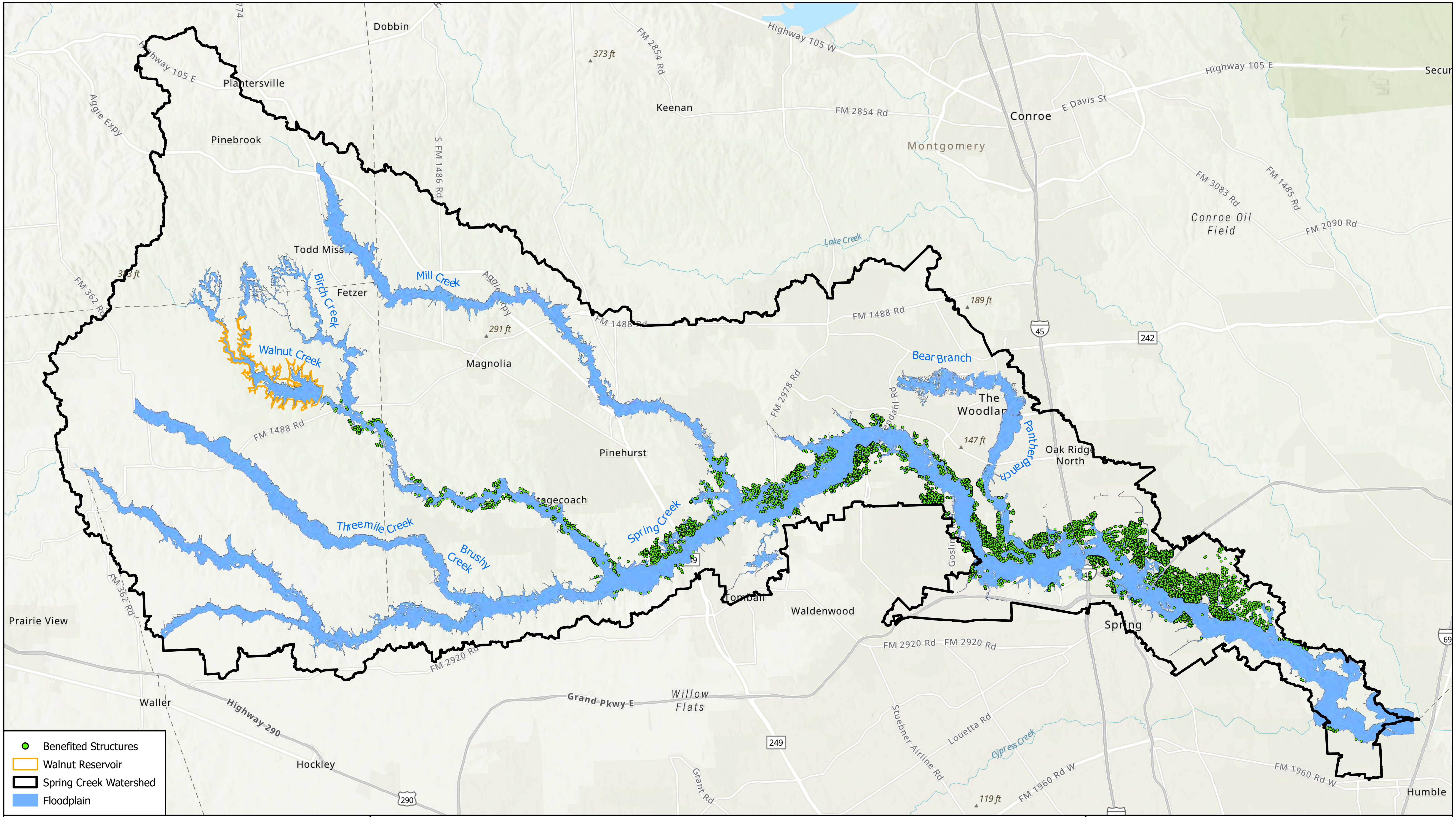

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Exhibit 5
Benefited Structures Walnut

Spring Creek Watershed Flood Control Dams Engineering Feasibility Study

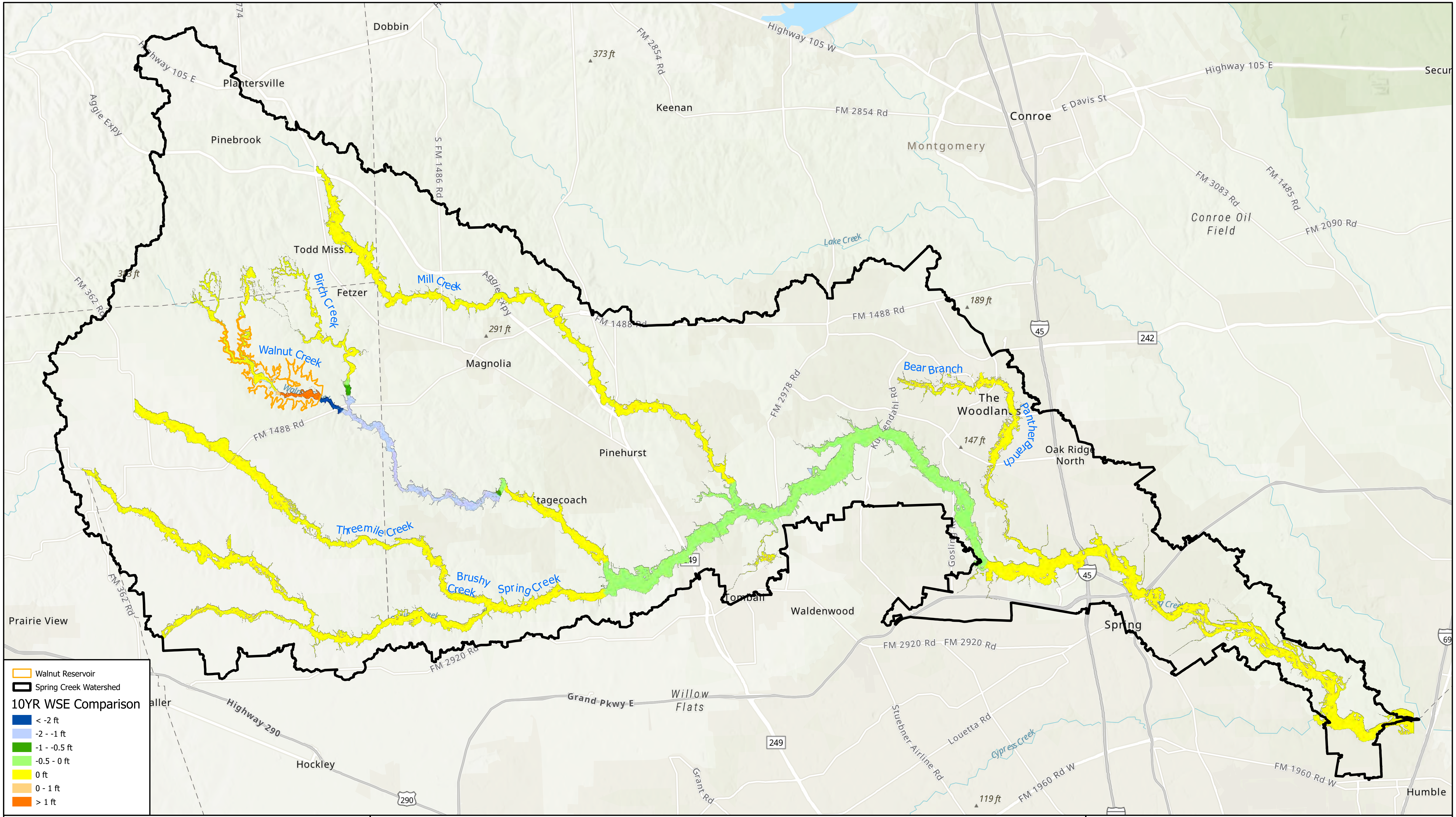

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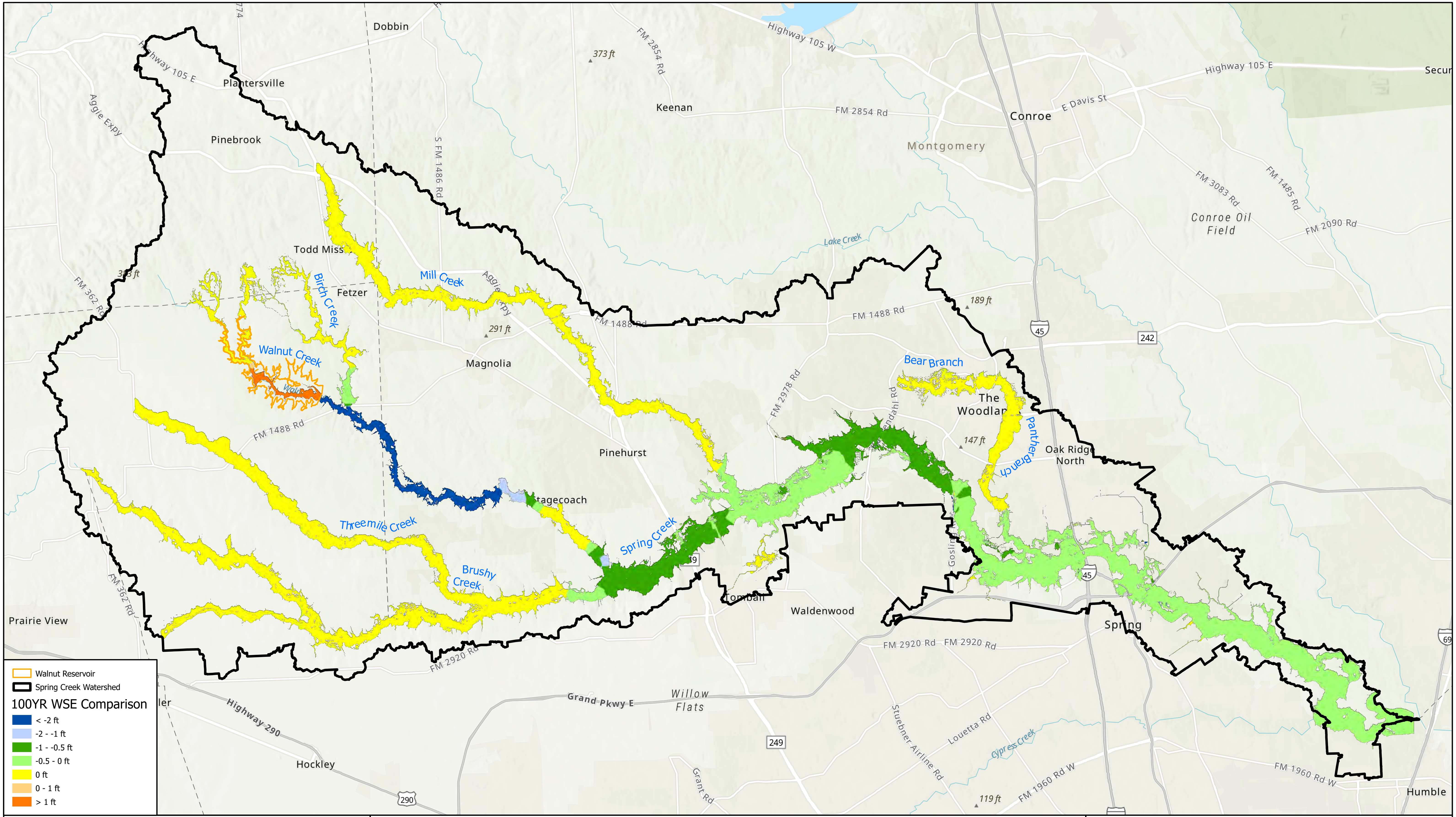
Exhibit 6
Proposed Walnut vs Existing
10 YR Comparison

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study

3/7/2025

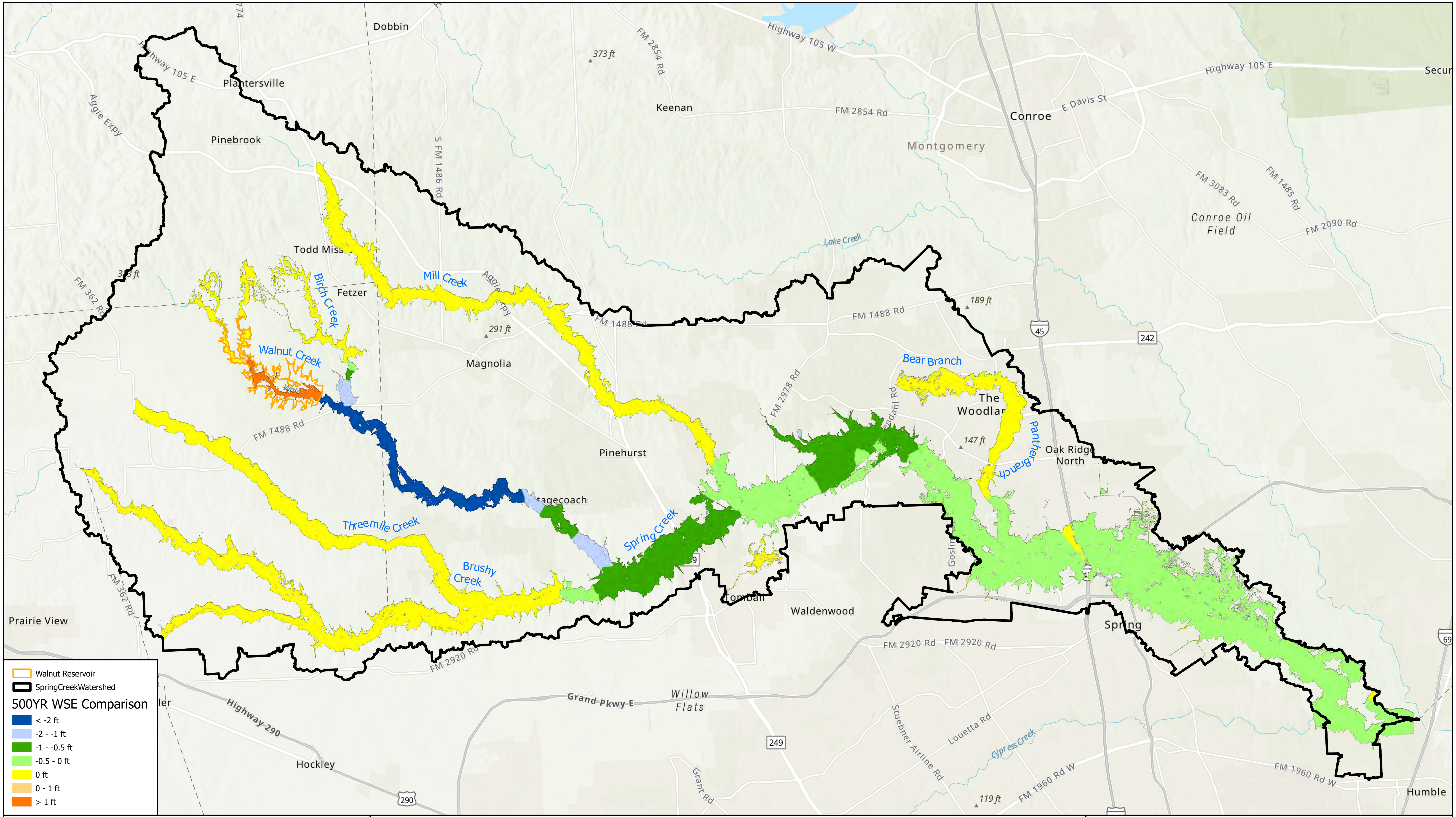


3/7/2025

halff

Exhibit 7
Proposed Walnut vs Existing
100 YR Comparison

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study

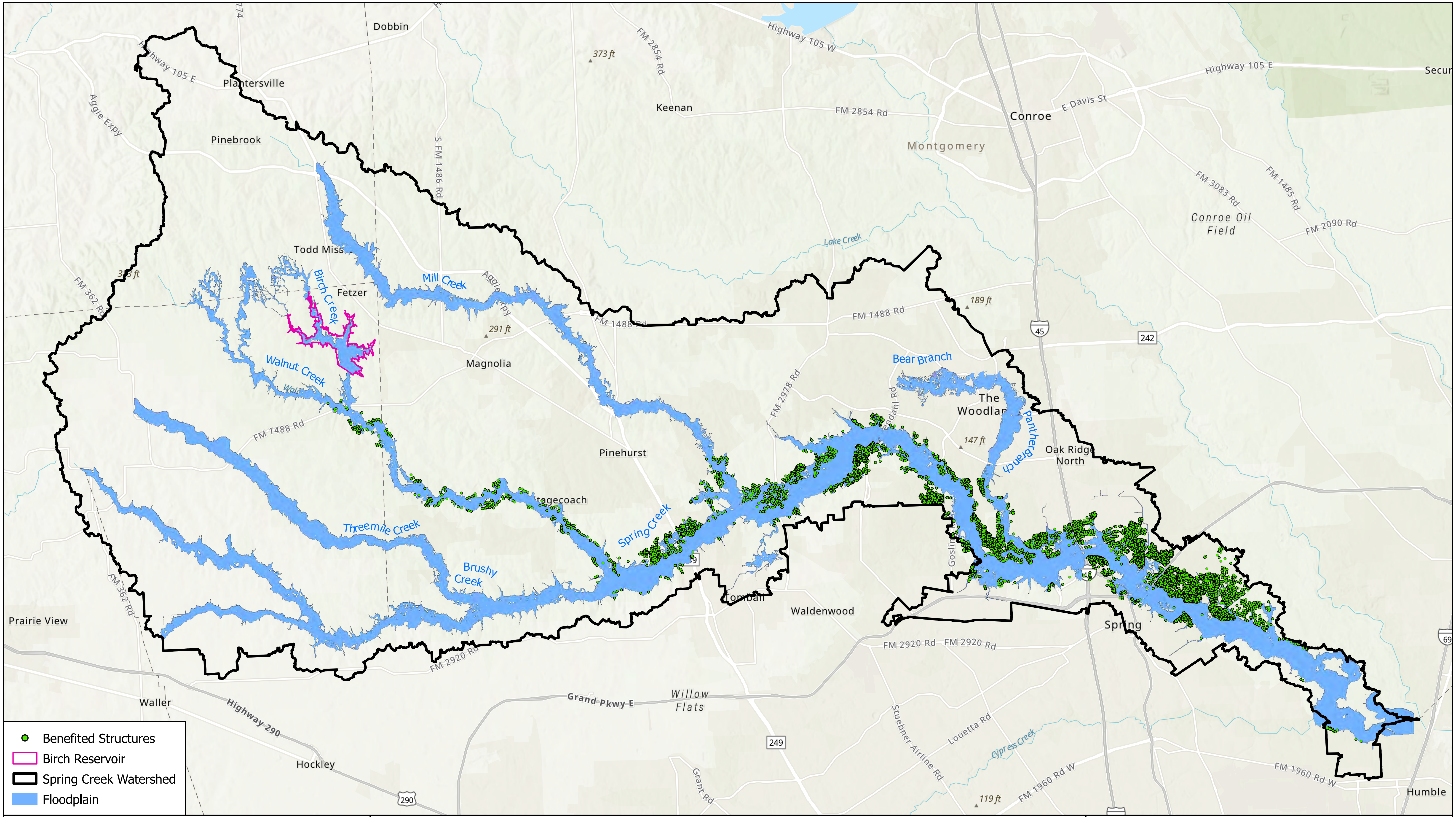


3/7/2025

halff

Exhibit 8
Proposed Walnut vs Existing 500 YR Comparison

Spring Creek Watershed Flood Control Dams Engineering Feasibility Study



Path: A:\42000s\42682\001\GIS\Working\PM\Exhibit APRXs\Exhibit_9_Benefited_Structures_Birch.aprx

2/14/2025

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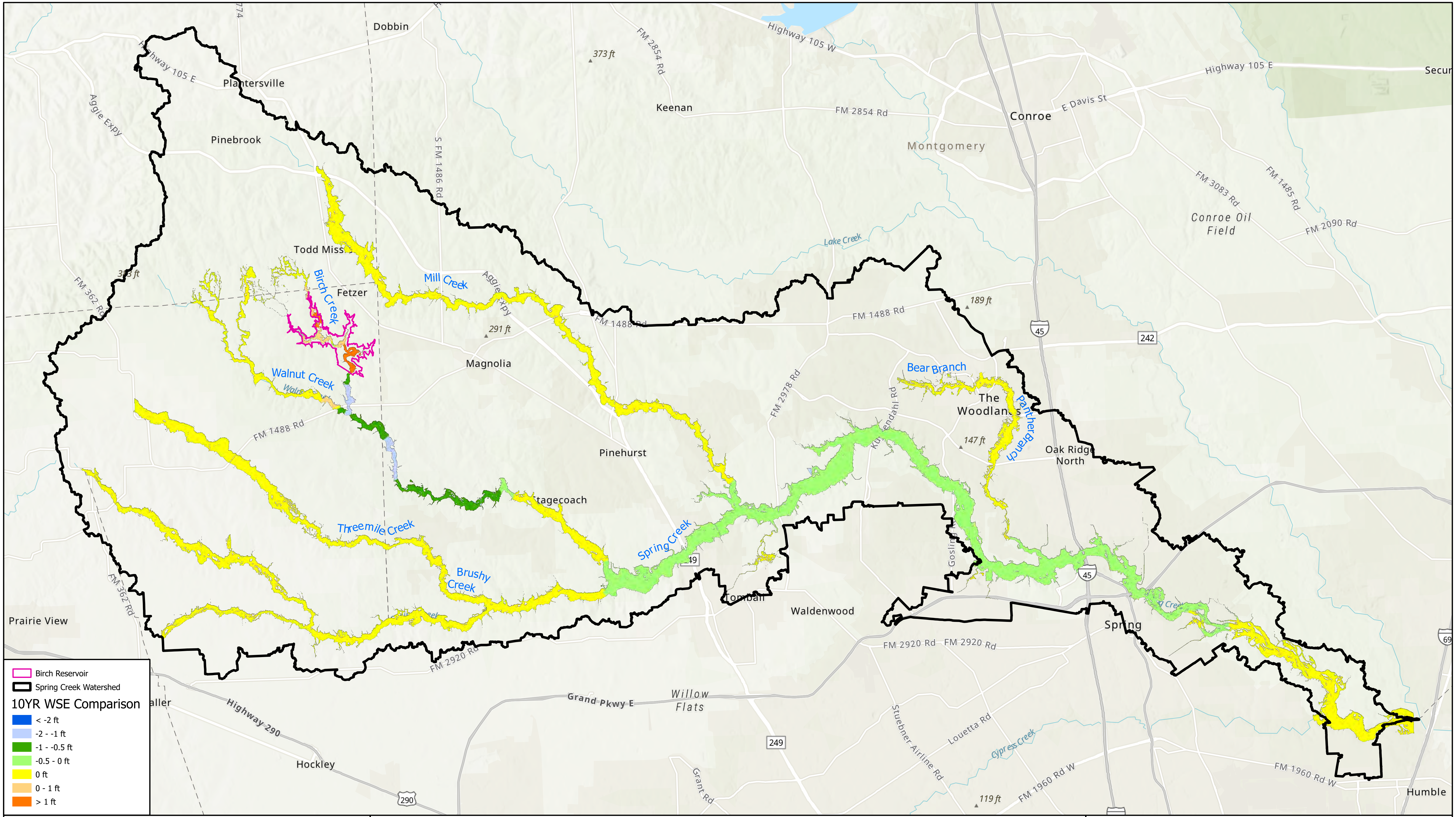
Exhibit 9
Benefited Structures Birch

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study



3/7/2025

Exhibit 10
Proposed Birch vs Existing
10 YR Comparison

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study

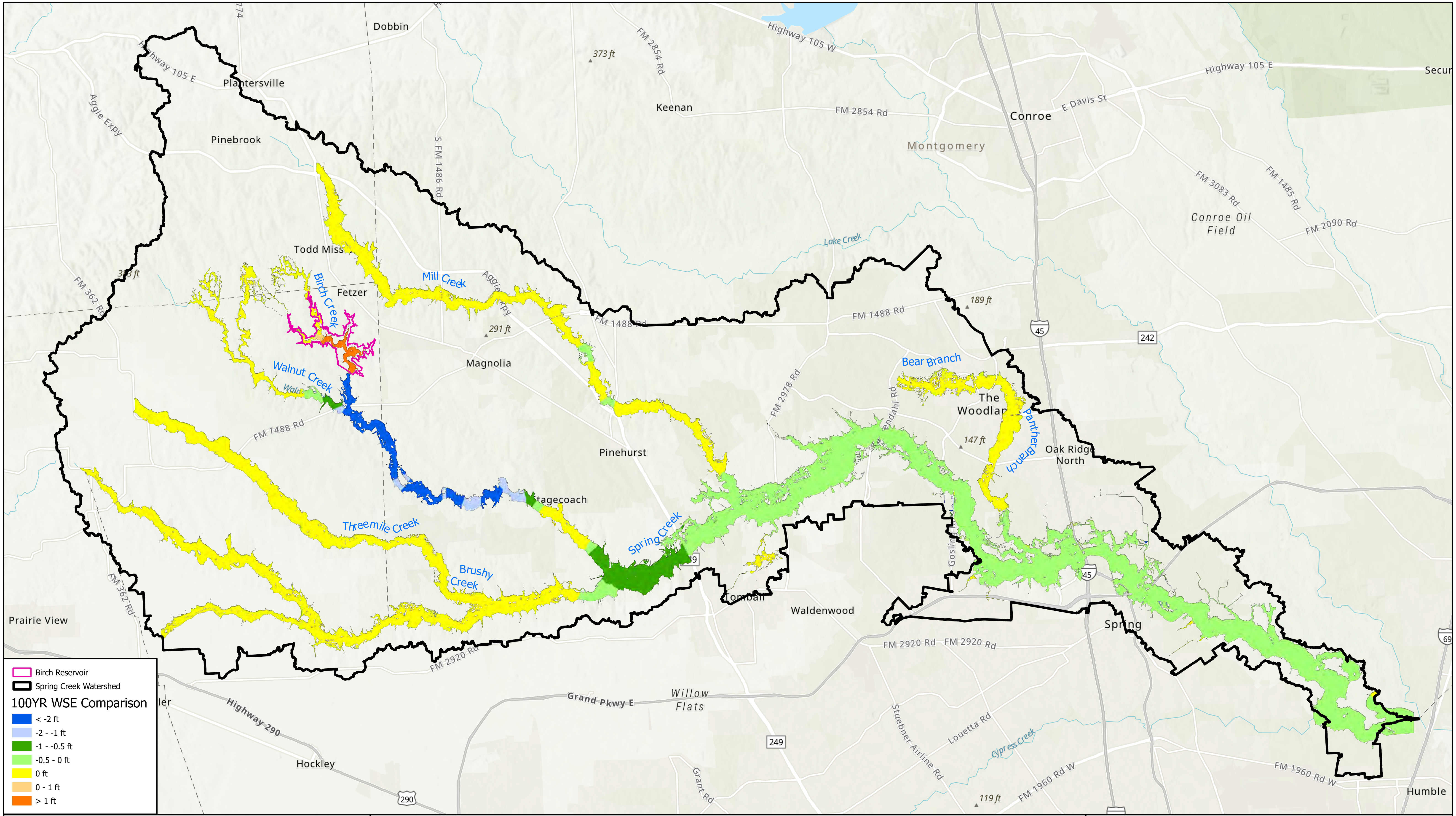


Exhibit 11
Proposed Birch vs Existing
100 YR Comparison

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study

3/7/2025

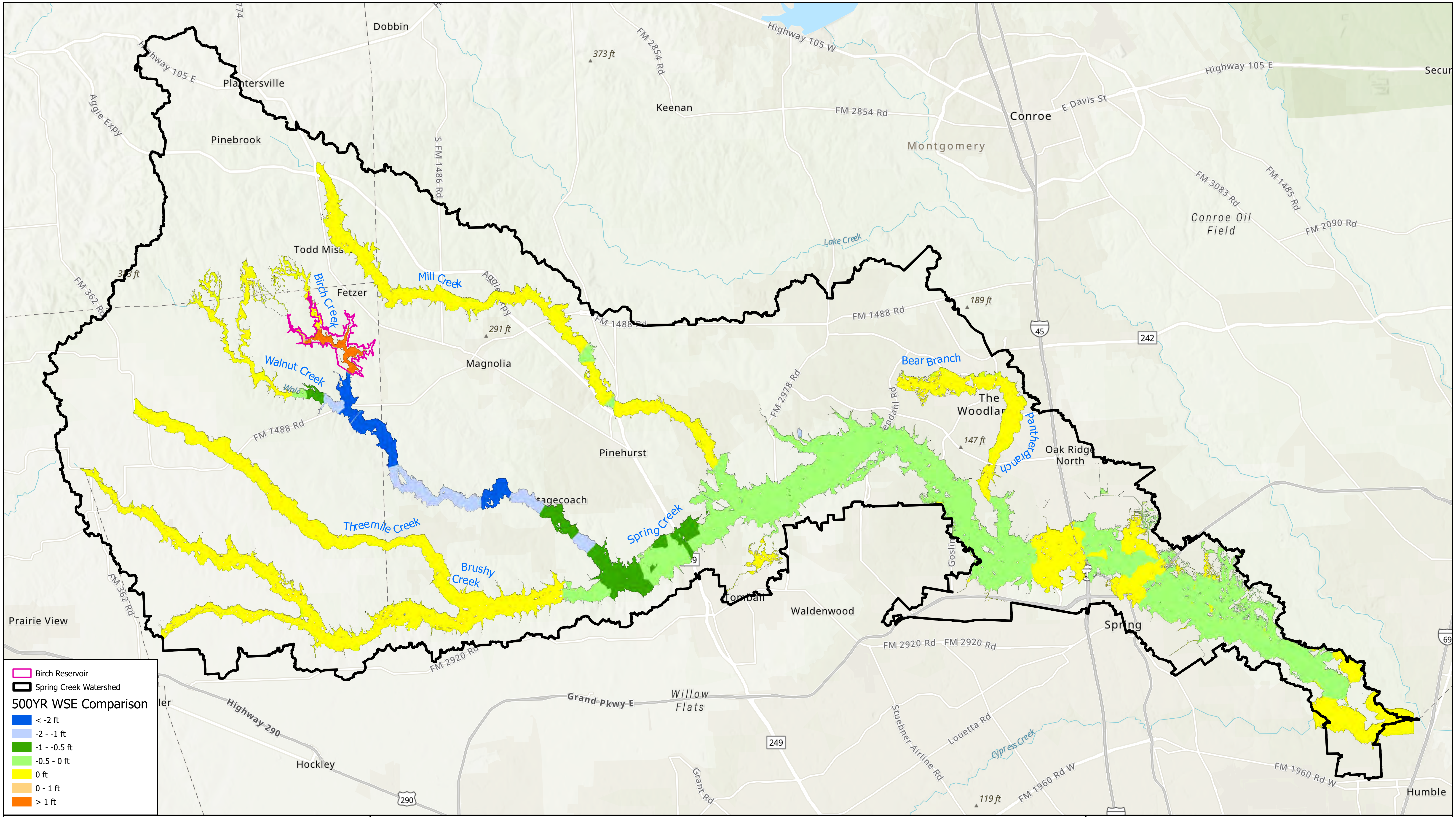


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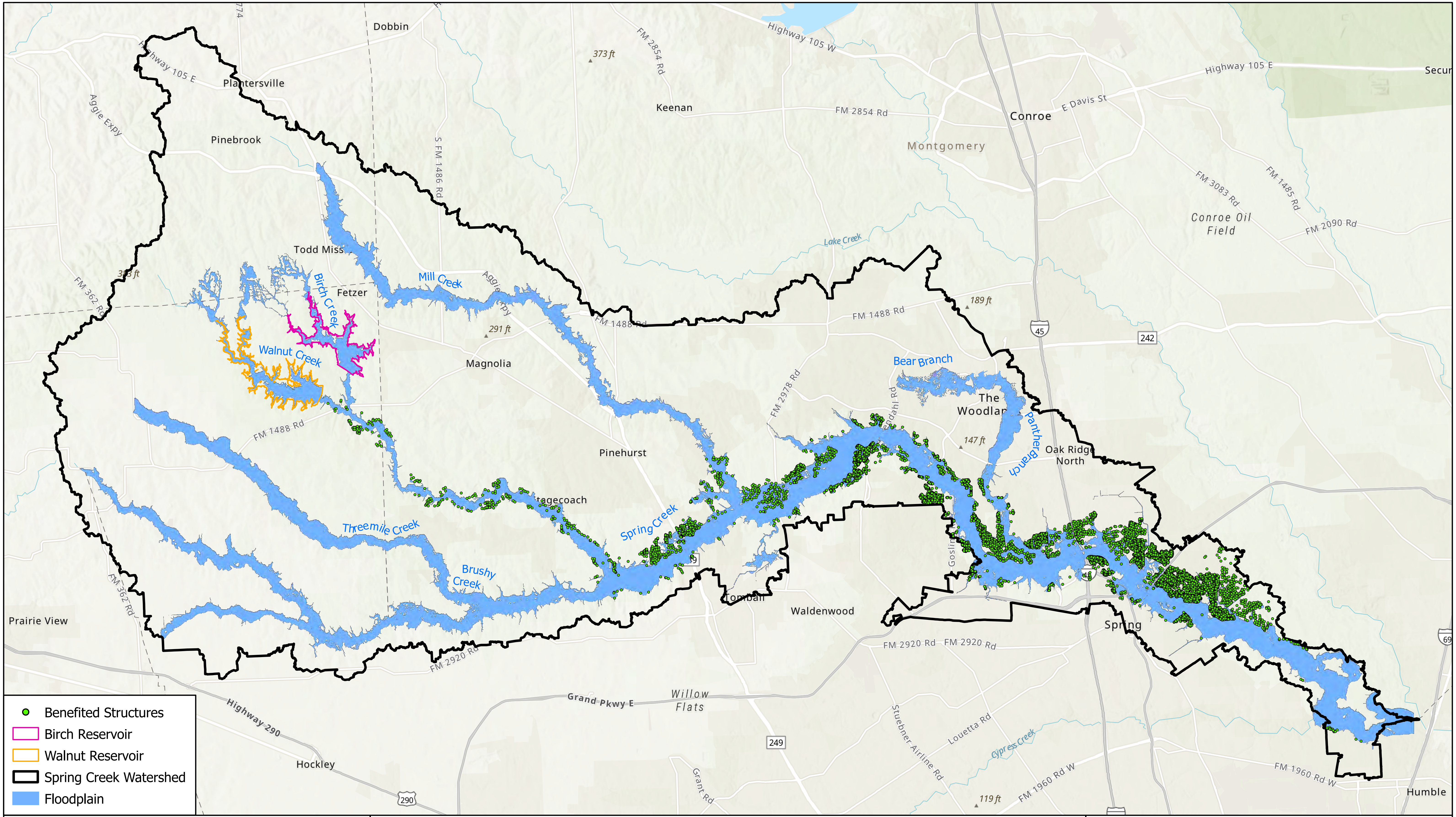
Exhibit 12
Proposed Birch vs Existing
500 YR Comparison

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study

3/7/2025



● Benefited Structures

☐ Birch Reservoir

☐ Walnut Reservoir

☐ Spring Creek Watershed

■ Floodplain

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2/14/2025

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Exhibit 13

Benefited Structures Combination

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study

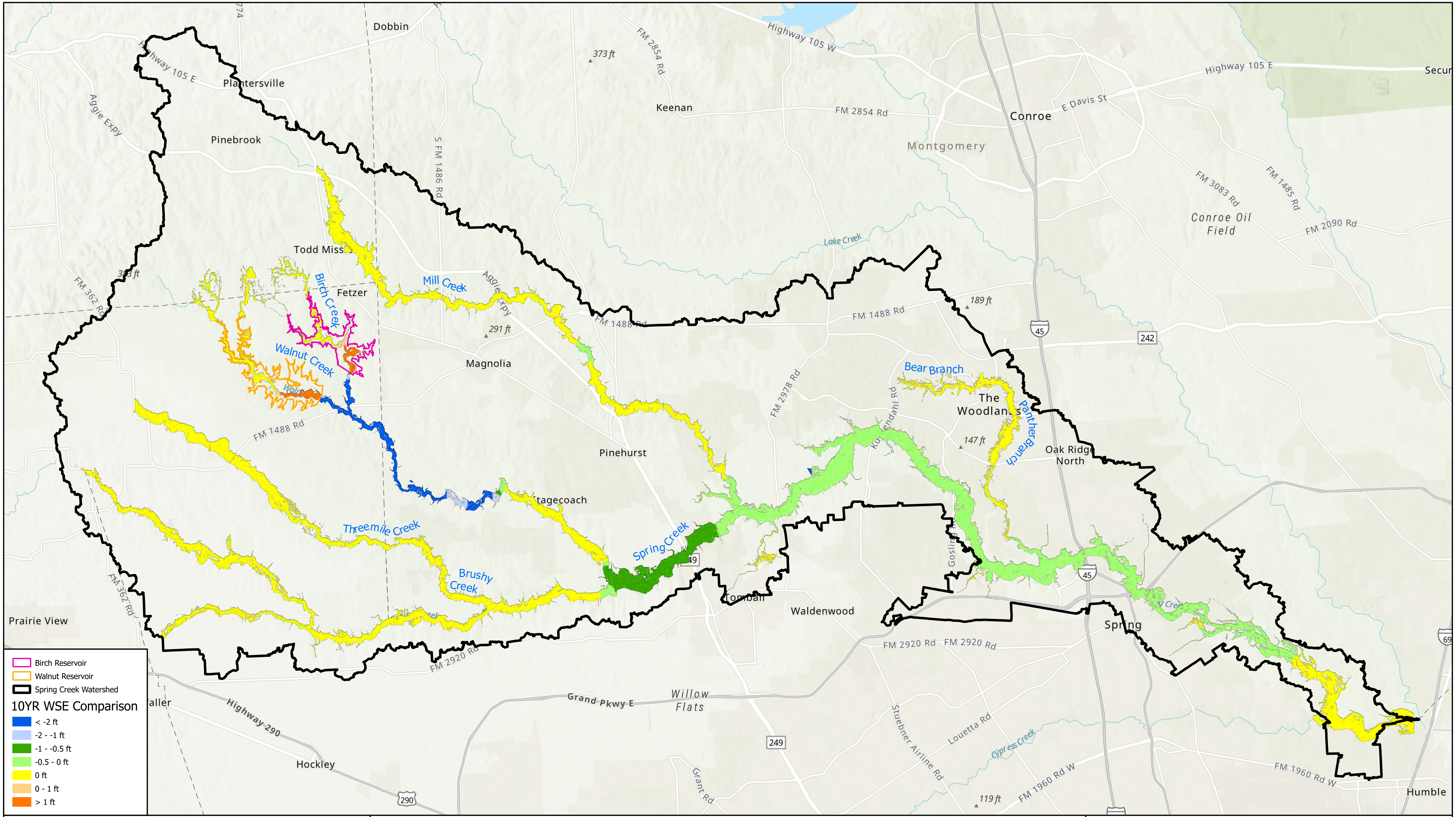

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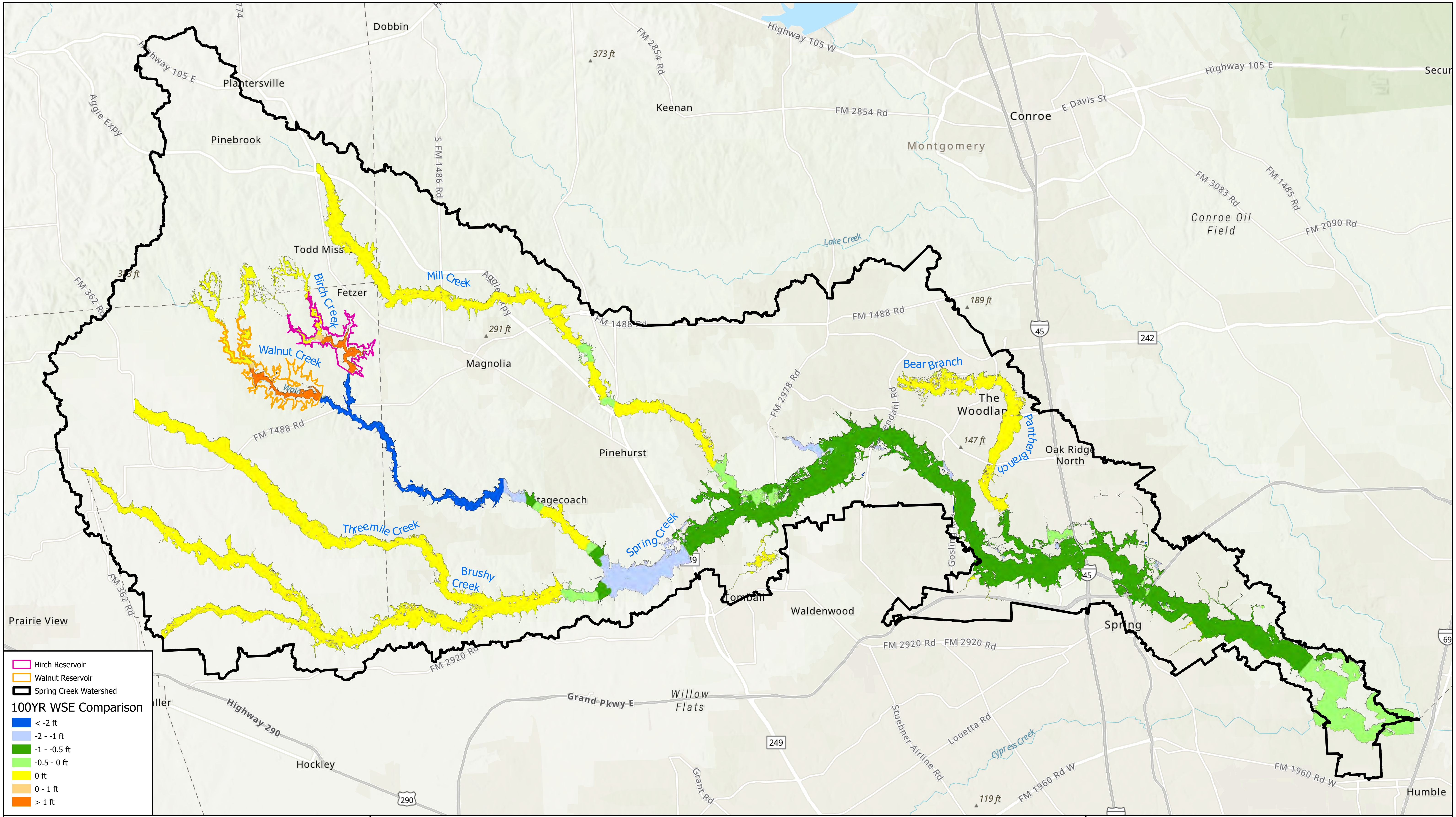
Exhibit 14
Proposed Combined vs Existing
10 YR Comparison

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study



3/7/2025

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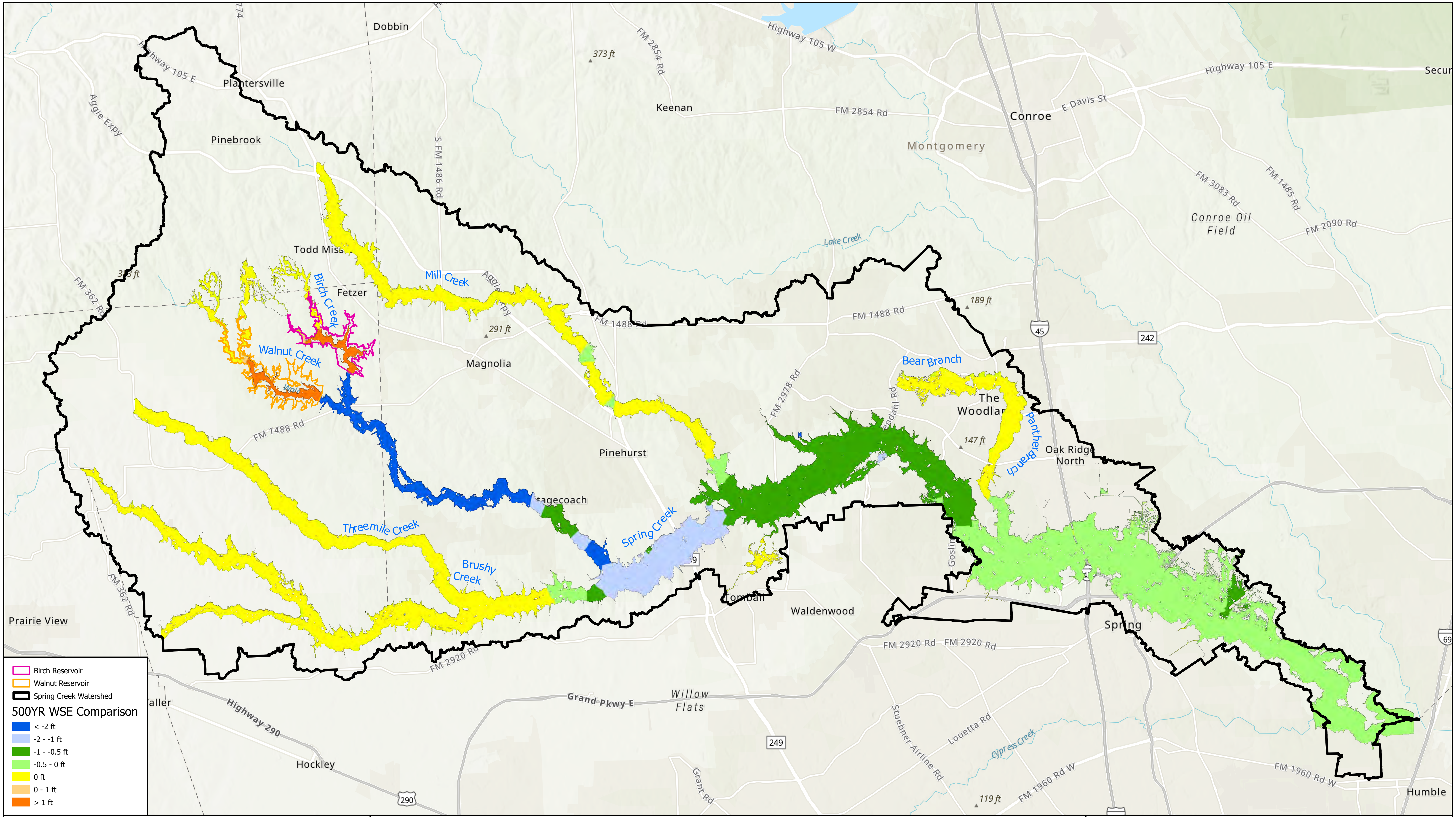
Exhibit 15
Proposed Combined vs Existing
100 YR Comparison

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study



3/7/2025

Exhibit 16
Proposed Combined vs Existing
500 YR Comparison

Spring Creek Watershed
Flood Control Dams
Engineering Feasibility Study

Appendix E:
Public Engagement

Spring Creek Flood Control Dams Feasibility Study
Written Comment Form/Formulario de Comentarios Escritos

ARE YOU A PUBLIC OFFICIAL? */ ¿ES USTED UN FUNCIONARIO PÚBLICO?*

☒ **NO**/NO ☐ **YES**/ SÍ **If yes, position**/ En caso afirmativo, puesto: ______________________

First and Last Name/Nombre y Apellido
Ana Cosio

Mailing Address/Dirección de Envío
82 N. Country Gate Circle

City, State, Zip Code/Ciudad, Estado, Código Postal
The Woodlands, TX 77384

Email Address/Correo Electrónico
scubamom1234@hotmail.com

Affiliation/Afiliación

How did you learn about this public meeting?
¿Cómo se enteró de esta reunión pública?

☒ **Email**/Correo Electrónico
☐ **Website**/Sitio Web
☐ **Facebook/Twitter**
☐ **Mail**/Correo
☐ **Other (please explain)**/Otro (por favor de explicar) ______________________

Do you support the study?/Usted apoya el estudio?
YES - Buy the LAND NOW

☒ **I support the study**/Sí, apoyo el estudo - in its original -
☐ **I do not support the study**/No apoyo el estudio

Do not lower dam ht.

How would you prefer to receive information about this study? (Please check one)/¿Cómo prefiere recibir información sobre el estudio? (Por favor marque uno)

☐ **Website**/Sitio web ☐ **Mail**/Correo ☒ **Email**/Correo Electrónico ☐ **Facebook/Twitter**

☐ **Other (please explain)**/Otro (por favor de explicar)______________________

COMMENTS ***(Please make additional comments on the back, if needed)***
COMENTARIOS *(Por favor haga comentarios adicionales en la parte posterior, si es necesario)*

- Why are we basing results on a 100 yr. study when Harvey was a 500 year flood event?
- DO NOT LOWER the DAM, DO NOT MINIMIZE overflow FLOOD PLAIN BEHIND DAM
- FEMAS FORMULAS ARE OUTDATED!
- Were Atlas 14 rainfall predictors used?
- Buy the LAND - All of it - NOW -

that has been the unending battle - by the time you find out the area floods, it's too late + peoples' lives are changed.

- Don't let upstream development build - it will raise land level, more concrete - further causing downstrea floodi

Spring Creek Flood Control Dams Feasibility Study
Written Comment Form/Formulario de Comentarios Escritos

ARE YOU A PUBLIC OFFICIAL? / ¿ES USTED UN FUNCIONARIO PÚBLICO?

☒ **NO**/NO ☐ **YES**/ SÍ **If yes, position**/ En caso afirmativo, puesto: ____________________

First and Last Name/Nombre y Apellido
Barbi Petty

Mailing Address/Dirección de Envío
31 W Tallowberry Dr.

City, State, Zip Code/Ciudad, Estado, Código Postal
The Woodlands TX 77381

Email Address/Correo Electrónico
barbipetty@yahoo.com.

Affiliation/Afiliación

How did you learn about this public meeting?
¿Cómo se enteró de esta reunión pública?

☒ **Email**/Correo Electrónico

☐ **Website**/Sitio Web

☐ **Facebook/Twitter**

☐ **Mail**/Correo

☐ **Other (please explain)**/Otro (por favor de explicar)

Do you support the study?/Usted apoya el estudio?

☑ **I support the study**/Sí, apoyo el estudo

☐ **I do not support the study**/No apoyo el estudio

How would you prefer to receive information about this study? (Please check one)/¿Cómo prefiere recibir información sobre el estudio? (Por favor marque uno)

☐ **Website**/Sitio web ☐ **Mail**/Correo ☒ **Email**/Correo Electrónico ☐ **Facebook/Twitter**

☐ **Other (please explain)**/Otro (por favor de explicar)____________________

COMMENTS ***(Please make additional comments on the back, if needed)***
COMENTARIOS *(Por favor haga comentarios adicionales en la parte posterior, si es necesario)*

The presentation was informative BUT at the same time disheartening because it doesnt seem as if a solution can be found in the near future to keep our homes from flooding again. When I bought my home it wasnt in a flood plain now my entire property is! Home flooded in 2016 + 2017. Montgomery County should DO something to reduce the over development + sinking of our property.

Hopefully a solution can be found QUICKLY to avoid future flooding.

Spring Creek Flood Control Dams Feasibility Study
Written Comment Form/Formulario de Comentarios Escritos

ARE YOU A PUBLIC OFFICIAL? / ¿ES USTED UN FUNCIONARIO PÚBLICO?

☑ **NO**/NO ☐ **YES**/ SÍ **If yes, position**/ En caso afirmativo, puesto: ____________________

First and Last Name/Nombre y Apellido
Marta Lafaver

Mailing Address/Dirección de Envío
28433 Riley Rd

City, State, Zip Code/Ciudad, Estado, Código Postal
Waller TX 77484

Email Address/Correo Electrónico
marta.lafaver@gmail.com

Affiliation/Afiliación

How did you learn about this public meeting?
¿Cómo se enteró de esta reunión pública?

☑ **Email**/Correo Electrónico

☐ **Website**/Sitio Web

☐ **Facebook/Twitter**

☐ **Mail**/Correo

☐ **Other (please explain)**/Otro (por favor de explicar)

____

Do you support the study?/Usted apoya el estudio?
☐ **I support the study**/Sí, apoyo el estudo
☑ **I do not support the study**/No apoyo el estudio

How would you prefer to receive information about this study? (Please check one)/¿Cómo prefiere recibir información sobre el estudio? (Por favor marque uno)

☐ **Website**/Sitio web ☐ **Mail**/Correo ☑ **Email**/Correo Electrónico ☐ **Facebook/Twitter**

☐ **Other (please explain)**/Otro (por favor de explicar)____________________

COMMENTS ***(Please make additional comments on the back, if needed)***
COMENTARIOS *(Por favor haga comentarios adicionales en la parte posterior, si es necesario)*

As a resident who will be directly impacted by the Walnut Creek proposed dam, I am gravely concerned by the impact of this project. If the water rises up to Riley Rd, which it already normally does WITHOUT the dam, I may be forced to stay on my higher property instead of being able to go to work. If I can't leave my property, I will lose it. This project may displace me from my Home!!! Please take road improvements into consideration of this project! Please! My other major concern is that road improvements will be placed on the county, which will lead to unfair tax hikes on us - the community already paying the price for improvements to another city's well being. If these concerns are taken into consideration and solutions are put in place, we will withdraw our opposition to this project.

Spring Creek Flood Control Dams Feasibility Study
Written Comment Form/Formulario de Comentarios Escritos

ARE YOU A PUBLIC OFFICIAL? / ¿ES USTED UN FUNCIONARIO PÚBLICO?

☐ **NO**/NO ☐ **YES**/ SÍ **If yes, position**/ En caso afirmativo, puesto: ______________________

First and Last Name/Nombre y Apellido
Tyle L Rao

Mailing Address/Dirección de Envío
67263 Lenox Hill Dr. 77382

City, State, Zip Code/Ciudad, Estado, Código Postal

Email Address/Correo Electrónico

Affiliation/Afiliación

How did you learn about this public meeting?
¿Cómo se enteró de esta reunión pública?

☐ **Email**/Correo Electrónico

☐ **Website**/Sitio Web

☐ **Facebook/Twitter**

☐ **Mail**/Correo

☐ **Other (please explain)**/Otro (por favor de explicar)

____

Do you support the study?/Usted apoya el estudio?
☐ **I support the study**/Sí, apoyo el estudo
☐ **I do not support the study**/No apoyo el estudio

How would you prefer to receive information about this study? (Please check one)/¿Cómo prefiere recibir información sobre el estudio? (Por favor marque uno)

☐ **Website**/Sitio web ☐ **Mail**/Correo ☐ **Email**/Correo Electrónico ☐ **Facebook/Twitter**

☐ **Other (please explain)**/Otro (por favor de explicar)______________________

COMMENTS ***(Please make additional comments on the back, if needed)***
COMENTARIOS *(Por favor haga comentarios adicionales en la parte posterior, si es necesario)*

① The water levels rise with (backflow) not (foreflow)

② the Harris County Channel from the Spring Creek is choked with 'O' capacity. Please increase the capacity. (Behind Lenox Hill)

Spring Creek Flood Control Dams Feasibility Study
Written Comment Form/Formulario de Comentarios Escritos

ARE YOU A PUBLIC OFFICIAL? / ¿ES USTED UN FUNCIONARIO PÚBLICO?

☑ **NO**/NO ☐ **YES**/ SÍ **If yes, position**/ En caso afirmativo, puesto: ____________________

First and Last Name/Nombre y Apellido
Kenneth Barron

Mailing Address/Dirección de Envío
28446 Riley Rd

City, State, Zip Code/Ciudad, Estado, Código Postal
Waller, TX 77484

Email Address/Correo Electrónico
Kenneth.barrontx@gmail.com

Affiliation/Afiliación

How did you learn about this public meeting?
¿Cómo se enteró de esta reunión pública?

☐ **Email**/Correo Electrónico
☐ **Website**/Sitio Web
☐ **Facebook/Twitter**
☐ **Mail**/Correo
☐ **Other (please explain)**/Otro (por favor **de** explicar)

Do you support the study?/Usted apoya el estudio?
☐ **I support the study/**Sí, apoyo el estudo
☑ **I [✓ do not] support the study/**No apoyo el estudio

How would you prefer to receive information about this study? (Please check one)/¿Cómo prefiere recibir información sobre el estudio? (Por favor marque uno)

☐ **Website**/Sitio web ☑ **Mail**/Correo ☐ **Email**/Correo Electrónico ☐ **Facebook/Twitter**

☑ **Other (please explain)**/Otro (por favor de explicar) Through our attorney

COMMENTS ***(Please make additional comments on the back, if needed)***
COMENTARIOS *(Por favor haga comentarios adicionales en la parte posterior, si es necesario)*

We have lived in our home on 22acres for over 15years. A tributary of Walnut Creek runs through our property. Our home has NEVER flooded.
We did our due diligence before building with elevation levels to insure we were buiding in a safe area. We are now placed in the impact zone for flooding because of this project.
Because builders and Counties have not considered flooding before completing projects we are being penlized for their negligane.
Our attorney information is on file with you.
Our 22 acres should be impacted because of others incompetance and greed.

From: Mariah Najmuddin
To: Connor Stokes
Subject: Fwd: New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "
Date: Wednesday, April 26, 2023 7:23:07 PM

What’s the protocol for responses?

Sent from my iPhone

Begin forwarded message:

> **From:** HubSpot Forms <noreply@hubspot.com>
> **Date:** April 26, 2023 at 11:15:45 AM CDT
> **To:** Mariah Najmuddin <Mariah@hollawayenv.com>
> **Subject: New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "**
> **Reply-To:** HubSpot Forms <noreply@hubspot.com>

CAUTION: Email from outside Hollaway

HubSpot

New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "

Page submitted on: Contact Us - Spring Creek Feasibility Study

First name:
Sylvia A

Last name:
Centanni

Email:
centannis2@gmail.com

Subject:

Flood control and aquifer recharge

Message:
Have you considered retention ponds with recharge into the aquifer we pull water from in The Woodlands?

It takes years to recharge an aquifer but if you drill down to a level that feeds into the aquifer, it will take less time. This is not an immediate fix for flooding, but it would remediate some flooding because of the ponds and hopefully recharge the aquifer for years down the road.

View in HubSpot

CONTACT
Sylvia A Centanni

From: Mariah Najmuddin
To: Connor Stokes
Subject: Fwd: New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "
Date: Wednesday, April 26, 2023 7:23:24 PM

Sent from my iPhone

Begin forwarded message:

> **From:** HubSpot Forms <noreply@hubspot.com>
> **Date:** April 26, 2023 at 9:28:43 AM CDT
> **To:** Mariah Najmuddin <Mariah@hollawayenv.com>
> **Subject: New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "**
> **Reply-To:** HubSpot Forms <noreply@hubspot.com>

CAUTION: Email from outside Hollaway

HubSpot

New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "

Page submitted on: Contact Us - Spring Creek Feasibility Study

First name:
Ryan

Last name:
Quigley

Email:
ryan.quigley@sbcglobal.net

Subject:
Walnut and Birch dams

Message:

Groundwater availability is a very hot topic in Montgomery County. Could the impounded water from these dams be used to recharge the aquifer? For instance, in California, they use these flood control dams to protect property and recharge aquifers. I doubt we have the geology needed but might be worth looking into.

View in HubSpot

CONTACT
Ryan Quigley

From: HubSpot Forms
To: Mariah Najmuddin
Subject: New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "
Date: Tuesday, May 2, 2023 7:03:18 PM

HubSpot

New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "

Page submitted on: Contact Us - Spring Creek Feasibility Study

First name:
Leah

Last name:
Raney

Email:
lraney99@gmail.com

Subject:
Spring Creek Flood Control Feasibility Study

Message:
Is there a way to include planned construction developments, particularly residential developments, in the study area and how they will impact the modeled benefits of the dams under study? It would be helpful to have a tool similar to the Montgomery county interactive flood maps that would allow residents to see if their property might benefit from the construction of either or

both dams. The maps used in the public meeting are helpful, but too small to see where your property is located in respect the creek. Thank you.

View in HubSpot

CONTACT
Leah Raney

From: HubSpot Forms
To: Mariah Najmuddin
Subject: New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "
Date: Tuesday, May 2, 2023 6:56:42 PM

CAUTION: Email from outside Hollaway

HubSpot

New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "

Page submitted on: Contact Us - Spring Creek Feasibility Study

First name:
Jace

Last name:
Houston

Subject:
Spring Creek dam presentation

Message:
That Matt Barrett guy did a really good job.

View in HubSpot

CONTACT
Jace Houston

From: HubSpot Forms
To: Mariah Najmuddin
Subject: New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "
Date: Tuesday, May 2, 2023 6:49:27 PM

CAUTION: Email from outside Hollaway

HubSpot

New submission on HubSpot Form "New contact us form (March 24, 2022 3:39:31 PM) "

Page submitted on: Contact Us - Spring Creek Feasibility Study

First name:
Stuart L

Last name:
Schroeder

Email:
aggiebob@consolidated.net

Subject:
Spring Creek Watershed

Message:
Can you email the powerpoint presentation? There was a tremendous amount of information contained therein.

View in HubSpot

CONTACT
Stuart L Schroeder

springcreekstudy.com (Hub ID: 21548652)

Spring Creek Flood Control Dams Feasibility Study
Written Comment Form/Formulario de Comentarios Escritos

ARE YOU A PUBLIC OFFICIAL? */ ¿ES USTED UN FUNCIONARIO PÚBLICO?*

☑ **NO**/NO ☐ **YES**/ SÍ **If yes, position**/ En caso afirmativo, puesto: ______________________

First and Last Name/Nombre y Apellido
John GRAZIANO

Mailing Address/Dirección de Envío

City, State, Zip Code/Ciudad, Estado, Código Postal
MAGNOLIA 77354

Email Address/Correo Electrónico
6253-31@outlook.Com

Affiliation/Afiliación

How did you learn about this public meeting?
¿Cómo se enteró de esta reunión pública?

☑ **Email**/Correo Electrónico

☐ **Website**/Sitio Web

☐ **Facebook/Twitter**

☐ **Mail**/Correo

☐ **Other (please explain)**/Otro (por favor de explicar)

____

Do you support the study?/Usted apoya el estudio?
☑ **I support the study**/Sí, apoyo el estudo
☐ **I do not support the study**/No apoyo el estudio

How would you prefer to receive information about this study? (Please check one)/¿Cómo prefiere recibir información sobre el estudio? (Por favor marque uno)

☐ **Website**/Sitio web ☐ **Mail**/Correo ☑ **Email**/Correo Electrónico ☐ **Facebook/Twitter**

☐ **Other (please explain)**/Otro (por favor de explicar)______________________

COMMENTS ***(Please make additional comments on the back, if needed)***
COMENTARIOS *(Por favor haga comentarios adicionales en la parte posterior, si es necesario)*

ProTecT Private Property RighTS &
Protect FARMS & Ranches.
NO Public TRAILS ON private property Rights

Appendix A

Public Comment – Jamie LeBlanc

Spring Creek Flood Control Dams Feasibility Study
Written Comment Form/Formulario de Comentarios Escritos

ARE YOU A PUBLIC OFFICIAL? / ¿ES USTED UN FUNCIONARIO PÚBLICO?

☑ **NO**/NO ☐ **YES**/ SÍ **If yes, position**/ En caso afirmativo, puesto: ______________________________

First and Last Name/Nombre y Apellido
Jamie LeBlanc

Mailing Address/Dirección de Envío
28300 Mustang Dr.

City, State, Zip Code/Ciudad, Estado, Código Postal
Waller, TX 77484

Email Address/Correo Electrónico
jleblanc@gilbaneco.com

Affiliation/Afiliación
land owner / resident

How did you learn about this public meeting?
¿Cómo se enteró de esta reunión pública?

☐ **Email**/Correo Electrónico

☐ **Website**/Sitio Web

☒ **Facebook/Twitter** Judge Duhon

☐ **Mail**/Correo

☐ **Other (please explain)**/Otro (por favor de explicar) ______________________

How would you prefer to receive information about this study? (Please check one)/¿Cómo prefiere recibir información sobre el estudio? (Por favor marque uno)

☐ **Website**/Sitio web ☐ **Mail**/Correo ☑ **Email**/Correo Electrónico ☑ **Facebook/Twitter**

☐ **Other (please explain)**/Otro (por favor de explicar)______________________________

COMMENTS ***(Please make additional comments on the back, if needed)***
COMENTARIOS *(Por favor haga comentarios adicionales en la parte posterior, si es necesario)*

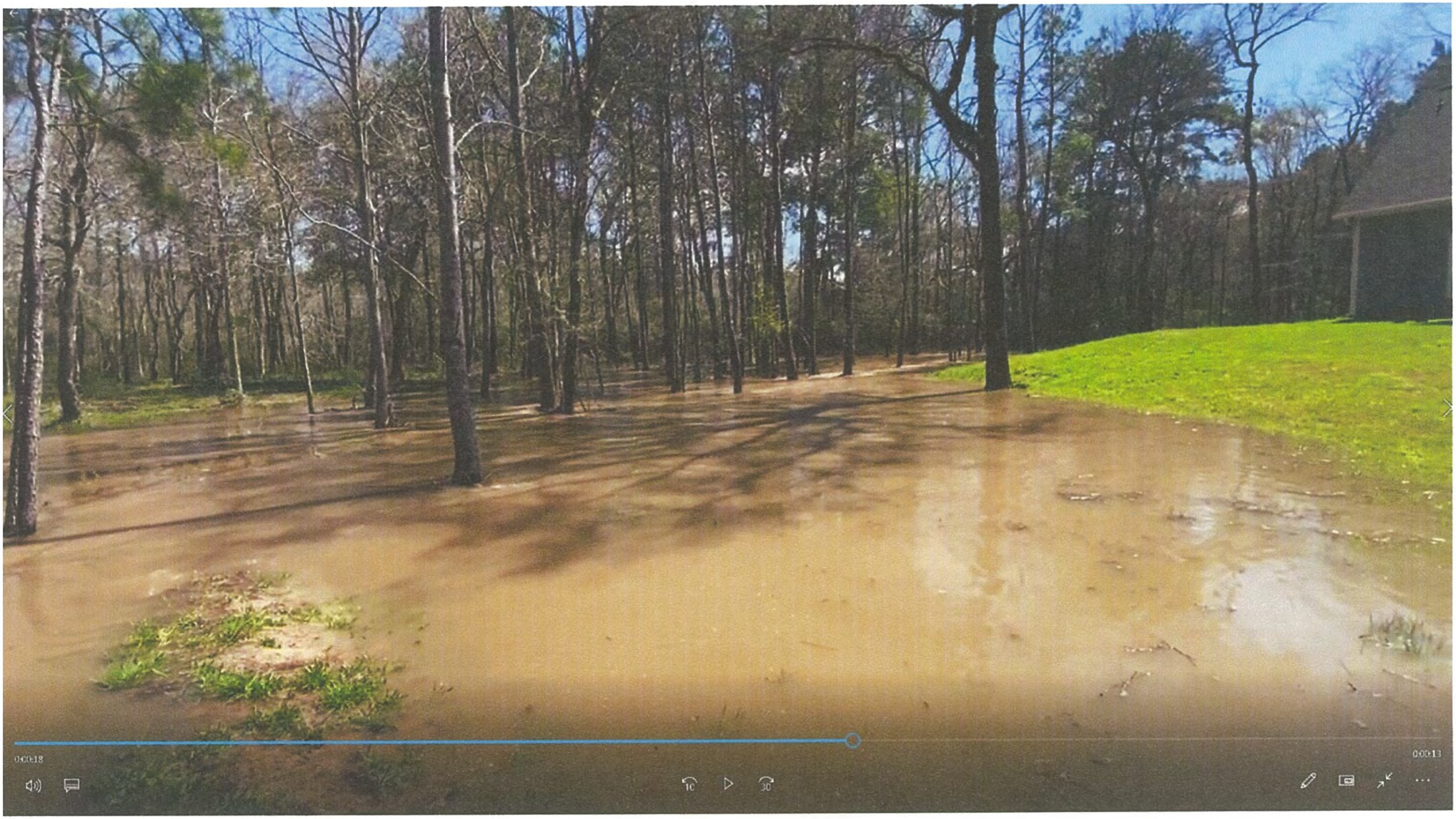
We have a lot of drainage issues in our subdivision, Saddle Creek Forest, and our neighbors are concerned about egress & flooding in/around our homes. I would ask that SJRA take a look at the impact of these projects on Saddle Creek Forest. I have attached photos & a map of the area in/around my home.

on N. Reids Prarie which crosses a floodway

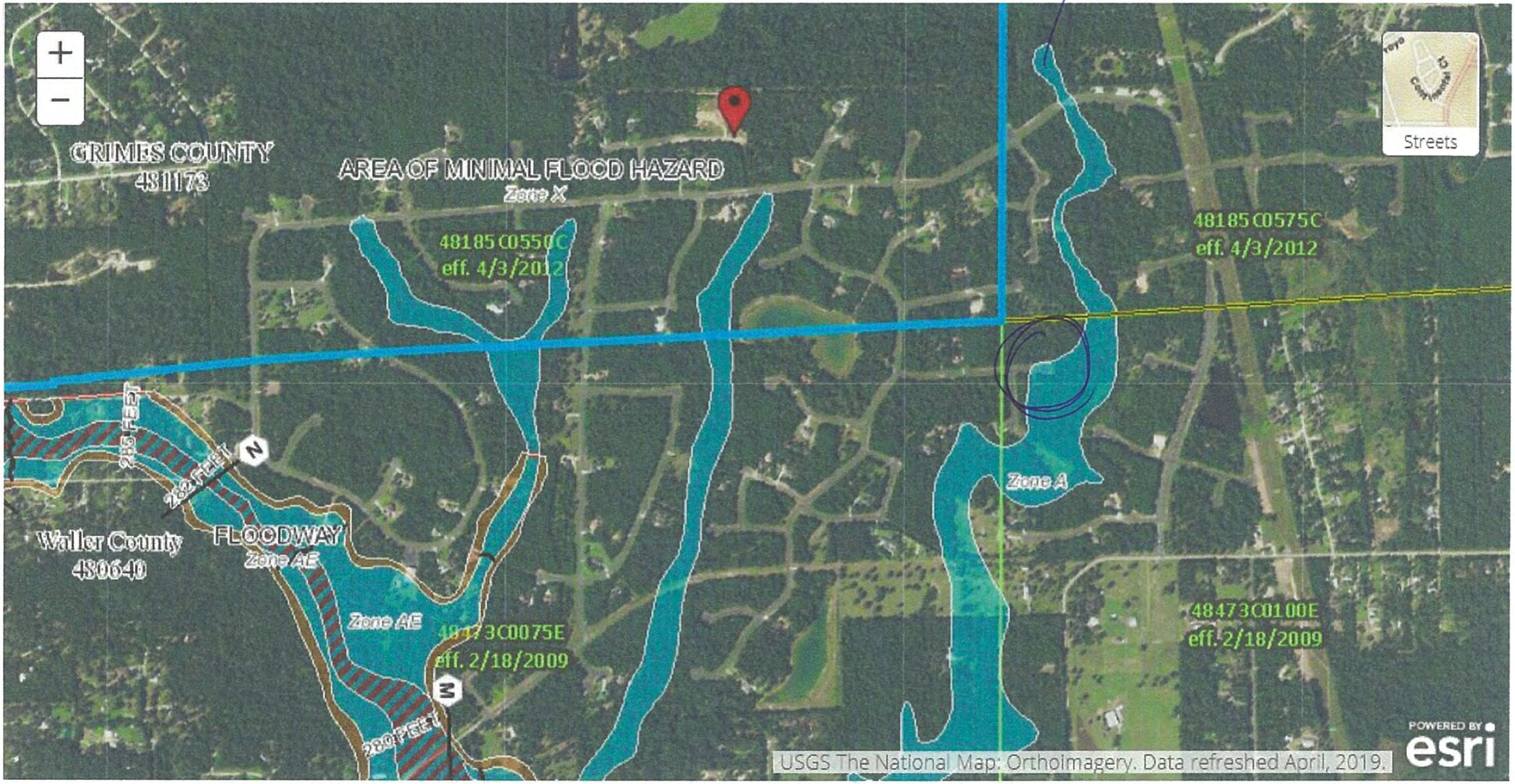
LeBlanc, Jamie 832-392-7824
28300 Mustang Dr. Waller 77484

Typical Day

5'-6" Rain



Walnut Creek Tributary



USGS The National Map: Orthoimagery. Data refreshed April, 2019.

Appendix B

Public Comments on Project Map

Note:
Dam locations and inundation limits are preliminary and subject to revision during the study.

Flood during rain events (Jerry Duane SFPOA)

Binkley (John ~~B~~) would inundate / cause backup in drainage ditches

Note:
Dam locations and inundation limits are preliminary and subject to revision during the study.

IF RILEY RD
IS FLOODED
AND REIDS PRAIRIE
RD. IS WASHED OUT
HOW WILL PEOPLE BE ABLE
TO ACCESS THEIR PROPE
BETWEEN. WE WOULD HA
NO ACCESS.

How will riley rd
be impacted?

Legend

Stream

Impacted Parcel

Proposed Flood Control Reservoir Inundation Limits

100-Year Limits

Probable Maximum Flood Limits

FEMA Effective Floodplain (Pre Atlas 14)

🟦 100-Year Existing Floodplain

🟧 500-Year Existing Floodplain

Spring Creek Study – Comment/Response Matrix
May 2025

#	NAME	EMAIL ADDRESS	FORMAT RECEIVED	SUPPORTS STUDY	TOPIC	COMMENT	DATE RECEIVED	COMMENT RESPONSE
1	Janet Duane	jan.duane@outlook.com	Website Comment Form		Walnut Creek Dam Feasibility Study	Having attended the most recent public hearing as well as the previous one, I noticed that the edge/limits of the survey area do not extend north beyond the Waller/Grimes county line. As a resident of Saddle Creek Forest, I can tell you that there are 4 tributaries to Walnut Creek that that run through SCF that originate in Grimes County, resulting in delayed road flooding from rainfall across the watershed in Grimes County. You study fails to address effects of any planned developments in extreme southern Grimes County, especially given the speed at which developments are spreading northward in Waller County. Also, Grimes County seems to have much less activity, involvement or regulation in flood control and mitigation relating to development than Waller. I feel this topic should be included as potentially having a significant effect on the proposed dam in the 10+ year time frame.	10-May-25	
2	Michael Michalski	mbm804@icloud.com	Website Comment Form		Aquifer Considerations	What implications for the local aquifers do the two dams present?	28-Apr-25	
3	Michael Sullivan	msullivan@bleylengineering.com	Website Comment Form		The Zero Cost No Action option	You have no considered the Zero Cost, Do Nothing, option that saves everyone without creating unnecessary burden for an agency that can't even be identified. Require the MoCo FLOOD CONTROL DISTRICT to do their job and RESTRICT construction of homes in areas defined as high hazard. The study is now completed and actually shows WHICH 275 homes will be impacted. Why keep allowing citizens to build in hazardous locations? The Barker Cypress / Addicks reservoir is a perfect example of County Flood Control failing to CONTROL building in hazardous areas. There are many other potential uses, but just because a developer purchased it does not mean the county is required to allow them to put lives at stake for profit. Really - negative impact and $5000,000 cost on the entire county to benefit only 275 homes? The fact no one has been identified to be a sponsor should be a good indicator of what the problem is. Cost Ratio for dual project is only 0.71 - you failed to state that in the presentation. Barely 1.0 based on made up benefit values with no backup. Obviously not beneficial. Cut losses now and STOP WORK.	24-Apr-25	
4	Mike Bernelle	lmike@swbell.net	Website Comment Form		Spring Creek Study - Walnut and Birch Creeks Detention	We live in Saddle Creek Forest. It is north of Riley road with parts of our neighborhood in Waller county and some in Grimes county and is between Birch Creek to the east and Walnut Creek to the west. Our past experiences with rain events and flooding in this area are a concern. We are stranded when bridges and roads are flooded and we cannot get out and outside services (like EMS) are needed but they cannot get in. We heard about this project via social media and were interested if it would provide any relief to these flood events. We attended the 4/21/25, meeting at the Field Store Community Center. The documentation and presentation both inform us that there be no positive result for us if this project, in it's current scope, goes forward. In our area that is impacted by these floods, the water levels do not remain at that stage for long after the rain stops. Currently, it moves thru (downstream) in usually less that a day. But at the meeting we were told that these two dams would cause a minimal increase in water levels but would take longer to drain - days longer. As stated in your documentation "Runoff impounded upstream of the dams could take up to a week to completely drain after an extreme rainfall event". Before the presentation when we were looking at maps, we asked that question to the person that did the presentation, (before we knew he was the person that was doing it) he replied we would probably get a little more water and it would take longer to drain. I am totally against this project. It will make a bad situation worse. It has no mention of building higher bridges over those creeks so we are not stranded. All the estimated benefits of "No Longer Flooded" are for those downstream of the proposed dams - at the expense of us upstream of the proposed dams.	23-Apr-25	
5	Marfa Lafaver	marta.lafaver@gmail.com	Website Comment Form		Walnut Creek & Birch Creek Reservoir Road Concerns	I believe it may be a grave concern and huge mistake to not include the cost of improving the Riley Road improvements needed for this project to go ahead. If both of these reservoirs are built and full of water, as intended, then you will be trapping the residents of Riley Road with no means of leaving or getting emergency services to their property. This is the only road in our area, there are no other options. Riley Road already floods during non-major storm events for at least a full day. This will only make it worse. The majority of residents work off of their property and must leave to earn a living. The bridges crossing the creeks at all points MUST be improved and built higher due to this project. This is not a cost that should be pushed onto Waller County since it is YOUR project causing this effect. This is a significant oversight of the project to not include this cost in your Cost to Benefits Analysis.	21-Apr-25	
6	Stephen Lafaver	slafav@gmail.com	Website Comment Form		Road Flooding Along Riley Road	Roads and bridges along Kyle, Riley, and countless other roads in the affected area already suffer from significant flooding during normal rain events. Not including the cost of improvements to these areas within this project is not only irresponsible but poses a significant risk of trapping residents in their homes for the entire duration that water is being held back by the new dams. Assuming that Waller County will fund such improvements in support of a project that only does harm to local residents seems naive at best and should be the sole responsibility of those who actual benefit from the project.	21-Apr-25	
7	John Graziano	6253-31@outlook.com	Public Meeting Comment Form		Private Property Rights	Protect private property rights Protect farms and ranches. NO public trails on private property rights.	28-Apr-25	