Appendix B-3 Material calculation package

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Client Name	SJRA	Page	1 of 67
Project Name	Spring Creek Watershed (SCW) Flood Control Dams	Project No.	411500
Calculation Title	Evaluation of Project Soil Parameters		

Verification Method: ☒ Check and Review ☐ Alternate Calculations

Objective: Evaluate the soil parameters that will be used for seepage and stability analysis of the Project

Unverified Assumptions Requiring Subsequent Verification			
No.	**Assumption**	**Verified By***	**Date**

Refer to Page _____ of this calculation for additional assumptions.

This Section Used for Software-Generated Calculations	
BV Standard Application Program Name/Version	Microsoft Excel

Review and Approval						
Rev	**Prepared By***	**Date**	**Checked By***	**Date**	**Approved By***	**Date**
0	P. Turkson, PhD, P.E.	10/25/2024	David Bentler, PhD, P.E.	12/6/2024		

*Signature required.

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1.0 Objective

Evaluate the design soil parameters that will be used for design of the Walnut Creek Dam and Birch Creek Dam as part of the Spring Creek Watershed Flood Control Dams scope of work for SJRA.

2.0 References

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3.0 Design Basis

The following section provides the design methodology for estimating the subsurface profile and selection of soil parameters for the Spring Creek Flood Control Dams which comprise Walnut Creek Dam and Birch Creek Dam (hereafter referred to as the Project). Based on the provided soil borehole logs and laboratory testing results the design stratigraphy was developed, and the required soil properties were selected for use in seepage and static stability analysis of the Project. Additionally, the stratigraphy and soil properties have been developed with guidance from published literature on geological units with similar characteristics.

A design stratigraphy was developed for the Project that includes the following two dams:

- Walnut Creek Dam (37.4 feet high, bottom of dam elevation 226.2 feet to top of crest elevation 263.6 feet).
- Birch Creek Dam (35.4 feet high, bottom of dam elevation 223.7 feet to top of crest elevation 259.1 feet).

The dams are primarily differentiated by the location of each creek. Each dam section is subdivided into strata based on material classification(s), index properties, and strengths. Dam section differentiation is only applied to **Section 4.0** for design strata elevations or depths.

Design soil parameters were evaluated for each stratum defined in the design stratigraphy based on available field and laboratory data. Where available, laboratory testing was used preferentially, and correlations with field testing and/or estimates from the published literature were used where minimal or no laboratory testing was available.

In situ testing for soil consistency and soil strength properties were performed in the four drilled boreholes. Borings B-1 and B-2 were drilled for the Walnut Creek Dam and borings B-3 and B-4 were drilled for the Birch Creek Dam. The Standard Penetration Test (SPT), which primarily targeted granular soils was used to estimate soil density as well as clayey soil consistency. Hand penetrometer (PP) testing was used to evaluate field cohesive soil strength.

Table 1 provides a summary of borehole locations and associated type of borehole in situ testing performed.

Laboratory testing from the 2024 geotechnical explorations was assigned by Aviles Engineering (Aviles) and reviewed by Black & Veatch. Laboratory testing was completed by Aviles. The laboratory testing results and a final geotechnical investigations report applied in this analysis are provided in **Appendix A** of the Design Basis Memorandum (DBM) (**Reference 17**). If new or revised laboratory data are provided that supersede the data applied in this calculation package, the design soil parameter calculations must be reviewed to confirm that the changes do not invalidate the design soil parameters.

Table 1 Borehole Location Summary for the Project

Borehole ID	In Situ Testing Type[2,3,4]		UTM Coordiantes[1] (m)		Surface Elevation (feet)[5]	Borehole Depth (feet)
			Northing	Easting		
	SPT	PP				
B-1	Y	Y	30°11'14.68"N	95°49'49.60"W	250	90
B-2	Y	Y	30°11'20.29"N	95°49'32.18"W	230	120
B-3	Y	Y	30°11'22.52"N	95°49'17.42"W	230	120
B-4	Y	Y	30°11'22.57"N	95°49'6.42"W	245	90

(1) Coordinates are in the UTM Zone 15, WGS 84 coordinate system.
(2) SPT— Standard Penetration Test
(3) PP— Pocket Penetrometer
(4) Y—Yes
(5) Surface elevations are approximate and obtained from Google Earth

3.1 Soil Stratigraphy

The borings drilled around the Project generally encountered soft materials, no rock was encountered. The borings generally encountered alternating layers of silty sands (SM), sandy lean clays (CL), clayey sands (SC), poorly graded sand with silt (SP-SM), sandy fat clay (CH), silty clay with sand (CL-ML) and silty clayey sand (SC-SM).

Based on the results of the geotechnical explorations, the stratigraphic units (from surface to depth) encountered are summarized in **Table 2**. The following subsections in **Section 4.0** provide a general description of the soil stratigraphy encountered.

A longitudinal section of the general stratigraphy of the soil units is shown in **Attachment 1**. Lines designating the interfaces between various strata on the boring logs represent approximate boundaries and the transition between strata. Soil conditions will vary between boring locations.

Table 2 Stratigraphic Units Encountered During Drilling

Unit No.	Description (USCS Classification Symbol)
1	Silty Sand (SM)
2	Lean Clay (CL)
3	Clayey Sand (SC)
4	Sand with Silt (SP-SM)
5	Fat Clay (CH)
6	Sand (SP)
7	Silty Clayey Sand (SC-SM)

The stratigraphic soil unit thicknesses encountered in the boreholes are summarized in **Table 3**Error! Reference source not found.. The thicknesses provided are based on interpretations of the boring logs, and in some cases relatively thin interbedded layers have been combined with the predominant layer as one unit. Appearance of the different units in each drilled hole are not necessarily in the order listed in the summary table. The detailed description of soils as recorded on the boring logs can be found in **Appendix A** in the DBM.

Table 3 **Summary of Stratigraphic Units Encountered in Boreholes**

Borehole ID	Subsurface Unit Thickness (feet)						
	Unit 1	Unit 2	Unit 3	Unit 4	Unit 5	Unit 6	Unit 7
B-1	18	18	18	15	15	—	—
B-2	36	46	9	23	5	—	—
B-3	13	15	47	26	10	—	—
B-4	—	—	17	40	5	13	14

3.2 Groundwater

Groundwater was encountered in all the borings. Groundwater was encountered during drilling in borings B-2, B-3 and B-4 at depths ranging from 8 to 28 feet below ground surface (fBGS). Groundwater in B-1 was only encountered after completion of drilling at a depth of 10.2 fBGS. The groundwater level in the borings 15 mins after completion of drilling was noted at depths ranging from 5.5 to 26.5 feet. A summary of the static groundwater depths by borehole is presented in **Table 4**.

Table 4 **Groundwater Levels**

Borehole ID	Water Level (fBGS)	
	During Drilling	15 mins After Drilling Completion
B-1[1]	Not Encountered	Not measured
B-2	12	5.5
B-3	8	5.8
B-4	28	26.5

1. Groundwater in B-1 encountered at 10.2 fbgs after drilling, no time of measurement recorded.

3.3 SPT N Values

To determine design N values for the soil profile, blow count information from the boreholes (B-1 to B-4) were analyzed. The field-measured blow counts (SPT N-value) are corrected to an equivalent N_{60}, by the following equation (**Reference 1**).

$$N_{60} = \frac{E_m C_b C_s C_R N}{0.60}$$

where, E_m = hammer efficiency

C_b = borehole diameter correction factor

C_s = sampler correction

C_R = rod length correction

N = measured N value

The reported hammer efficiency correction factor ($E_m/60\%$) based on energy data from **Reference 2** is 1.46 for the determination of N_{60} values.

For boreholes between 2.5 to 4.5 inches in diameter, the borehole diameter correction factor is 1.0. Based on the geotechnical report, the inside diameter of the hollow stem auger used for drilling is 4 inches; therefore, a correction factor of 1 is used for the correction. Since a standard sampler was used for the penetration tests, a sampler correction factor of 1 is used.

The rod length (C_R) further modifies this calculation based on the sample depth. The following correction factors are used (**Reference 3**):

- For samples less than 13 feet below grade, a C_R of 0.75.
- For samples between 13 and 20 feet below grade, a C_R of 0.85.

- For samples between 20 and 30 feet below grade, a C_R of 0.95.
- For samples over 30 feet below grade, a C_R of 1.0.

N_{60} values for the proposed design soil profile were calculated as the product of SPT N values and the conversion factors. Design values are taken as approximate average values for the layer. Note that raw N values of 50 were not corrected.

3.4 Index Properties

Design values for total unit weights, liquid limits, plasticity index, and moisture contents for each stratum were selected as the statistical mean value within each stratum. Laboratory index testing results were assigned to the corresponding strata based on sample depth and material type recorded on the boring log.

Unit weights were typically assigned using available laboratory data where available. In strata where laboratory testing was unavailable, published literature of typical values for the soil type from **Reference 18** and **Reference 20** were used to estimate unit weight.

3.5 Soil Strength Parameters

The following section describes the design basis of determining shear strength parameters.

3.5.1 Q-Case

For the Q-Case, or undrained case, undrained shear strength (s_u) design values were evaluated for each fine-grained stratum. Values of s_u within each stratum were evaluated based on Unconfined Compression (UC) tests and Unconsolidated-Undrained (UU) tests; s_u estimates from pocket penetrometer (PP) and torvane (TV) shear tests which are typically high were not considered. The PP and TV test methods are best used as a quick field assessment for soft clays, and the results of the tests are not reliable for clays with sand or silt, or for detailed design. Published correlations between SPT N values and s_u were used to derive undrained shear strengths for comparison to s_u from laboratory tests.

After grouping all the data points for each stratum, statistical evaluation of s_u included the following assessments:

1. Average or mean.
2. Sample standard deviation.
3. 33rd percentile (1/3 rule).
4. 95 percent confidence interval of the mean.

Reference 4 provides guidance on methodology to use for the design strength based on the number of samples (*n*) within each stratum:

- For *n* ≤ 10 samples, the lower 95 percent confidence limit (CL) is the design strength.
- For *n* > 10 samples, the 33rd percentile (1/3 rule) is the design strength.

The 95 percent confidence interval of the mean was calculated as follows:

$$< \mu >_{1-\alpha} = \left[\bar{X} + t_{n-1}\left(\frac{\alpha}{2}\right) * \frac{s}{\sqrt{n}} \; ; \; \bar{X} - t_{n-1}\left(\frac{\alpha}{2}\right) * \frac{s}{\sqrt{n}}\right] \qquad \text{(Equation 1)}$$

Where:

$< \mu >_{1-\alpha}$ = interval for the mean with α = 5 percent (confidence interval of 95 percent)

$\bar{X}$ = sample mean

$t_{n-1}\left(\frac{\alpha}{2}\right)$ = T-Score evaluated at $\frac{\alpha}{2}$ for n-1 degrees of freedom

s = sample standard deviation

n = sample size

Equation 1 is based on a T-Distribution, which considers the greater uncertainty associated with small samples sizes less than or equal to thirty (30). As n increases (>30), the T-Score approaches an equivalent value to a Z-Score from a normal distribution.

3.5.2 S-Case

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, correlations with SPT N-values, and/or based on guidance from published literature where laboratory data estimates are not available.

The value of ϕ' used to develop shear strength envelopes from CU tests is based on the following relationship, where the value for α is measured from stress paths plotted on the CU lab data sheets:

$$tan\ (\alpha) = \sin\ (\phi') \qquad \text{(Equation 2)}$$

The design envelope is the average of the CU tests strength envelopes within each stratum. Where lab testing is not available, correlations from **Reference 5** based on plasticity index (*PI*) were considered for comparison purposes only.

In addition, design values for the effective cohesion intercept (*c'*) for soil strata which are considered fine-grained were evaluated based on CU triaxial testing and with guidance from published literature. The value of *c'* used to develop shear strength envelopes from CU tests is based on the following relationship, where the value for *d'* is measured from stress paths plotted on the CU lab data sheets:

$$c' = \frac{d'}{\cos\ (\phi')} \qquad \text{(Equation 3)}$$

The *c'* for all strata, which are considered free-draining or coarse-grained, is assumed to be 0 pounds per square foot (psf).

For coarse-grained strata, correlations to SPT N-values were used, where SPT testing was available. Correlations from **References 6**, **7**, and **8** were used to select the design value of ϕ' for coarse-grained strata. When using correlations for SPT N-values, ϕ' was capped at 38 degrees. In cases where no laboratory or SPT testing were available, ϕ' was assigned based on typical values from published literature.

The correlation equations are presented as follows:

Fine-grained strata correlations:

Sorensen & Okkels (2013) **(Reference 5)**: $For\ 4 < PI < 50, \phi' = 44 - 14\log_{10} PI$ (Equation 4)
$For\ 50 \leq PI < 150, \phi' = 30 - 6\log_{10} PI$ (Equation 5)

Coarse-grained strata correlations:

Schmertmann (1975) (**Reference 6**): $\phi' = \tan^{-1}\left[\left(\frac{N_{60}}{12.2+\left(20.3\frac{\sigma'_v}{p_a}\right)}\right)^{0.34}\right]$ (Equation 6)

Hatanaka and Uchida (1996) (**Reference 7**): $\phi' = \sqrt{18N_{1,60}} + 20$ (Equation 7)

Peck, Hanson, & Thornburn (1974) (**Reference 8**): $\phi' = 27.1 + 0.3N_{1,60} - 0.00054\left(N_{1,60}\right)^2$ (Equation 8)

Where:

ϕ' = effective stress friction angle

PI = Plasticity index

σ_v' = effective in-situ stress (calculation assumes total unit weight = 120 pcf)

p_a = atmospheric pressure

N_{60} = SPT N-value corrected for field procedures and apparatus to 60% of the theoretical free-fall hammer efficiency

$N_{1,60}$ = SPT N_{60}-value corrected for overburden pressure

3.5.3 R-Case

For the R-Case, which is primarily used for rapid drawdown slope stability analyses, design values for cohesion intercept (c_R) and friction angle (ϕ_R) were developed for fine-grained strata based on the Duncan, Wright, and Wong (1990) procedure (**Reference 9**) detailed in Appendix G of EM-1110-2-1902 (**Reference 10**). These parameters were developed from CU testing. The value of ϕ' used to develop shear strength envelopes in the R-Case is the same ϕ' calculated for the S-Case, based on measurement of α from the CU test data sheets and Equation 2. The design value for c_R is calculated based on results from the CU tests. No more than one CU test was available for the sandy stratum, so single test is the basis of the developed design envelope for the sandy stratum.

3.6 Hydraulic Conductivity

3.6.1 Vertical Hydraulic Conductivity

The design vertical hydraulic conductivities (k_v) were developed for each stratum based on laboratory permeability testing. In strata with more than one permeability test, the geometric mean of the test results is the design value. The geometric mean is appropriate when determining a central value for datasets where the range of values spans multiple orders of magnitude (**Reference 11**). In strata with one permeability test, the test result is the design value. For strata with no testing, either (1) design values from permeability tests within similar materials were used as the design value or (2) typical values within similar materials from published literature were used as the design value. Hydraulic conductivity values generally vary over two orders of magnitude, hence a range of hydraulic conductivity values higher and lower than the design values by one or two orders of magnitude are provided for each stratum.

3.6.2 Anisotropic Ratio

Anisotropic ratio (k_v/k_h) for all strata is assumed based on typical values from values based on typical values from Table 6-6 for natural soils and Table 6-7 for engineered fill (**Reference 12**). Anisotropic ratio of 0.5 was assumed for all foundation stratum. Anisotropic ratio of 0.11 and 0.25 was assumed for clayey stratum and sandy stratum respectively when used as compacted fill. Lower values of anisotropic ratio have been assumed for embankment compacted fill since the embankment will be compacted and placed in horizontal lifts. As such, the vertical hydraulic conductivity will be further reduced than in the horizontal direction. Anisotropic ratio of 1 and 0.25 was assumed for rock riprap and filter sand respectively.

3.7 Consolidation Parameters

The following consolidation parameters were developed as a part of this study and are discussed in detail in the next subsections:

- Initial Void Ratio (e_0).
- Virgin Compression Index (C_c).
- Recompression Index (C_r).
- Overconsolidation Ratio (OCR).
- Coefficient of Consolidation (C_v).

- Coefficient of Secondary Compression (C_α).

3.7.1 Initial Void Ratio (e_0)

The design Initial Void Ratio (e_0) for fine-grained strata were selected based on the average initial void ratios from the available advanced laboratory testing for each stratum.

3.7.2 Virgin Compression Index (C_c), Recompression Index (C_r), and Overconsolidation Ratio (OCR)

The design C_c, C_r, and OCR are calculated for fine-grained strata from laboratory consolidation testing, where available, using the Casagrande Method (**Reference 13**). Laboratory test results were assigned to the corresponding strata based on sample depth and soil type. In strata where laboratory testing was not available, nearby testing results are used if the soil type and index properties are similar.

If consolidation testing was not available or index properties varied significantly from strata with available testing, design values for C_c were evaluated using three correlations with index properties from EM 1110-1-1904 (**Reference 13**), presented below:

1. Void Ratio (e_0): $C_c = 1.15 * (e_0 - 0.35)$ (Equation 8)
2. Moisture Content (MC): $C_c = 0.012 * (MC)$ (Equation 9)
3. Liquid Limit (LL): $C_c = 0.01 * (LL - 13)$ (Equation 10)

The design value (e_0, LL, MC) for the given stratum was inputted into each of the above equations, and the results were compared with laboratory testing, where available. It was noted that the correlation utilizing the void ratio (Equation 8) yielded results that significantly differed from the correlations presented in Equation 9 and 10; therefore, this correlation was not considered for the Project. Once a design value for C_c is selected, the design value of C_r is taken as 1/5 of C_c (**Reference 13**).

3.7.3 Coefficient of Consolidation (c_v)

The design Coefficient of Consolidation (c_v) values were selected for fine-grained strata using the geometric mean of laboratory consolidation testing or the chart from Figure 3-18 of NAVFAC DM 7.1 (**Reference 14**), relating LL to c_v (shown below), where laboratory testing was not available. The LL design value for each stratum and the curve for normally consolidated soils were used to estimate c_v. The selection of the curve for normally consolidated soils is based on the conservative assumption that embankment loading will bring the soils into the virgin compression range of stresses.

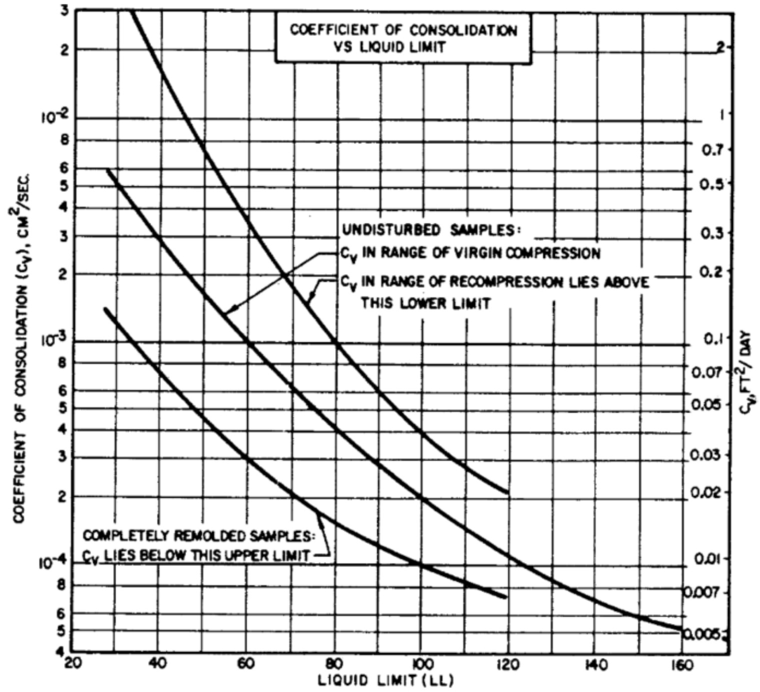


Figure 3-18. Correlations between coefficient of consolidation and liquid limit (NAVFAC DM 7.1)

3.7.4 Coefficient of Secondary Compression (C_α)

The design Coefficient of Secondary Compression (C_α) for the clays were developed from a correlation between C_α/C_C based on soil type from EM 1110-1-1904 (**Reference 13**) and a correlation between moisture content (*MC*) and C_α from the NAVFAC DM 7.1 (**Reference 14**). The C_α design value is based on the midpoint of the C_α/C_C range for corresponding soil types.

Typical ranges of C_α/C_C values are from Table 3-14 in EM 1110-1-1904, shown below (**Reference 13**). A C_α value is developed by multiplying the design C_C value for each stratum with the lower and upper bounds for the corresponding soil type from Table 3-14.

Table 3-14

Coefficient of Secondary Compression C_α
(Data from Item 43)

Soil	C_α/C_c
Clay	0.025 - 0.085
Silt	0.030 - 0.075
Peat	0.030 - 0.085
Muskeg	0.090 - 0.100
Inorganic	0.025 - 0.060

3.7.5 Modulus of Elasticity (E_s)

Design Modulus of Elasticity (E_s) were selected for granular soils using a combination of correlations with SPT N-values, and typical ranges of values based on soil type from Table D-3 in EM 1110-1-1904 (**Reference 13**), depending on the availability of in-situ data. Six(6) correlations with SPT N-Values were selected based on Black & Veatch calculation template (**Reference 15**), and the results averaged. 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**).

Table D-3

Typical Elastic Moduli

Soil	E_s, tsf
Clay	
Very soft clay	5 - 50
Soft clay	50 - 200
Medium clay	200 - 500
Stiff clay, silty clay	500 - 1000
Sandy clay	250 - 2000
Clay shale	1000 - 2000
Sand	
Loose sand	100 - 250
Dense sand	250 - 1000
Dense sand and gravel	1000 - 2000
Silty sand	250 - 2000

4.0 Analysis

The equations and methods presented in **Section 3.0** of this calculation package were used to estimate the design soil parameters for the Project. The laboratory testing results are provided in **Appendix A** of the DBM. (**Reference 17**)

Subsurface profiles illustrating the subsurface conditions have been developed from the information provided by the geotechnical subsurface exploration (**Appendix A** of the DBM). The stratum boundaries were defined based on material classification(s), index properties, and undrained shear strength. **Section 4.1** describes the design stratigraphy for the Project.

4.1 Subsurface Conditions and Profile

4.1.1 Unit 1 — Silty Sand (SM)

Silty sand soils (Unit 1) ranging in density from loose to dense and consisting of pockets lean clay was encountered at all borehole locations except B-4. Unit 1 varied in color from tan to brown and extended to depths ranging from about 1.25 to 16 fBGS. Unit 1 was also observed at deeper depths below ground surface from 27 to 112 fBGS. The thickness of Unit 1 was observed to range from 3 to 15 feet.

4.1.2 Unit 2 — Lean Clay (CL)

Deposits of very soft to hard sandy lean clay (Unit 2) were encountered in all boreholes except B-4 at depths ranging from 4 to 18 fBGS, and at deeper depths from 32 to 97 feet. The thicknesses of Unit 2 were recorded as ranging from 2 to 24 feet.

4.1.3 Unit 3 — Clayey Sand (SC)

Deposits of the clayey sand (SC) were encountered in all boreholes at depths ranging from 1 to 27 fBGS and at deeper depths ranging from 38 to 112 fBGS, with thickness ranging from 4 to 23 feet. In some of the boreholes, deposits of ferrous nodules and pockets of lean clay were recorded. The density of Unit 3 was recorded as ranging from very loose to medium dense.

4.1.4 Unit 4 — Sand with Silt (SP-SM)

Deposits of sand with silt (Unit 4) were encountered in all the boreholes and were interbedded with pockets of lean clay at various depths. Unit 4 varied in thickness between 5 to 25 feet. Unit 4 was recorded at depths from 22 to 47 fBGS and at deeper depths between 62 to 120 fBGS. The density of Unit 4 was recorded as ranging from loose to very dense.

4.1.5 Unit 5 — Fat Clay (CH)

Deposits of clay (Unit 5) were encountered in all boreholes at depths ranging from 32 to 67 fBGS, with thicknesses ranging from 5 to 15 feet. The clay till unit also consisted of pockets of sand and silt seams, and calcareous and ferrous nodules. The consistency of the clay unit was recorded as firm to hard.

4.1.6 Unit 6 — Sand (SP)

Deposits of sand (Unit 6) were observed in only borehole B-4 at a depth of 77 fBGS and thickness of 13 feet. The sand density was recorded as dense to very dense.

4.1.7 Unit 7 — Silty Clayey Sand (SC-SM)

Similar to Unit 6, deposits of the silty clayey sand (Unit 7) were observed in only borehole B-4 but at a relatively shallow depth of 8 fBGS and thickness of 14 feet. The silty clayey sand density was recorded as medium dense to dense.

4.1.8 Design Stratigraphy

A summary of the design stratigraphic units and their design thicknesses is presented in **Table 5** and **Table 6** for Walnut Creek Dam and Birch Creek Dam respectively. The clay units (Unit 2 and Unit 5) have been modeled as a single unit with similar material properties, hereafter referred to as Silty Clay and Sandy Clay unit. The general appearance of the single clay unit has been modeled at depths from 32 to 70 fBGS and from 77 to 97 fBGS for Walnuat Creek Dam. The single clay unit for Birch Creek Dam has been modeled as sandwiched (from 32 to 62 fBGS) between the sandy units (Unit 1, Unit 3, Unit 4, Unit 6 and Unit 7) which have been modeled as a single unit for seepage analysis and slope stability design with the same material properties. The single sandy unit is hereafter referred to as Silty Sand and Clayey Sand unit. The general foundation design profile for each dam is presented as **Attachment 2**. Red lines designating the interfaces between various strata on the foundation design profile represent approximate boundaries and the transition between strata. Soil conditions will vary between boring locations.

Table 5 Summary of Design Stratigraphic Units for Walnut Creek Dam Foundation

Design Elevations/Thicknesses					Basis
Unit No.	Description	Depth (feet), from	Depth (feet), to	Max Thickness (feet)	Justification
1, 3, 4, 6, 7	Silty Sand and Clayey Sand	0	42	42	Differentiated by material change and index properties.
2, 5	Silty Clay and Sandy Clay	32	71	39	Differentiated by material change and index properties.
1, 3, 4, 6, 7	Silty Sand and Clayey Sand	67	87	20	Differentiated by material change and index properties.
2, 5	Silty Clay and Sandy Clay	77	97	20	Differentiated by material change and index properties.
1, 3, 4, 6, 7	Silty Sand and Clayey Sand	97	120	23	Differentiated by material change and index properties.

Table 6 Summary of Design Stratigraphic Units for Birch Creek Dam Foundation

Design Elevations/Thicknesses					Basis
Unit No.	Description	Depth (feet), from	Depth (feet), to	Max Thickness (feet)	Justification
1, 3, 4, 6, 7	Silty Sand and Clayey Sand	0	47	47	Differentiated by material change and index properties.
2, 5	Silty Clay and Sandy Clay	32	62	30	Differentiated by material change and index properties.
1, 3, 4, 6, 7	Silty Sand and Clayey Sand	52	120	68	Differentiated by material change and index properties.

4.2 Groundwater Elevation

The after-drilling groundwater elevations based on records from boring logs ranged between 5.5 to 10.2 fBGS for Walnut Creek Dam and from 5.8 to 26.5 fBGS for Birch Creek Dam. The recorded groundwater appears to follow the ground surface topography considering relatively shallow depths to groundwater at the low-lying borings B-2 and B-3. The overall rise in water levels during the 10-minute wait period indicates the intermittent layers of clay may be inducing a confining condition with a piezometric head higher than its spatial elevation. The design groundwater depth for the Project was set at a conservative 5.5 fBGS. Groundwater levels are controlled by topography and the stratigraphic conditions affecting groundwater flow, and level fluctuations may occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borehole drillings were performed.

4.3 SPT N Value

A summary of the SPT N_{60} statistical analysis and design values for each stratum is included as **Table 7** and **Table 8,** 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 3.3.** Plots of the total unit weight data and design values are included as **Figure 3** and **Figure 2**Error! Reference source not found..

Table 7 SPT N60 Statistical Analysis, Data Comparison, and Design Values for Walnut Creek Dam

Stratum	SPT N60					Design Value (bpf)
	Bottom Depth (feet)	Sample Size (n)	Avg. (bpf)	Min. (bpf)	Max (bpf)	
Silty Sand and Clayey Sand	37	16	18	4	34	**18**
Silty Clay and Sandy Clay	69	7	29	18	42	**29**
Silty Sand and Clayey Sand	82	6	40	16	50	**40**
Silty Clay and Sandy Clay	97	2	43	39	48	**43**
Silty Sand and Clayey Sand	120	5	50	50	50	**50**

Table 8 SPT N60 Statistical Analysis, Data Comparison, and Design Values for Birch Creek Dam

Stratum	SPT N60					Design Value (bpf)
	Bottom Depth (feet)	Sample Size (n)	Avg. (bpf)	Min. (bpf)	Max (bpf)	
Silty Sand and Clayey Sand	40	12	25	4	39	**25**
Silty Clay and Sandy Clay	58	4	33	18	42	**33**
Silty Sand and Clayey Sand	120	19	43	1	50	**43**

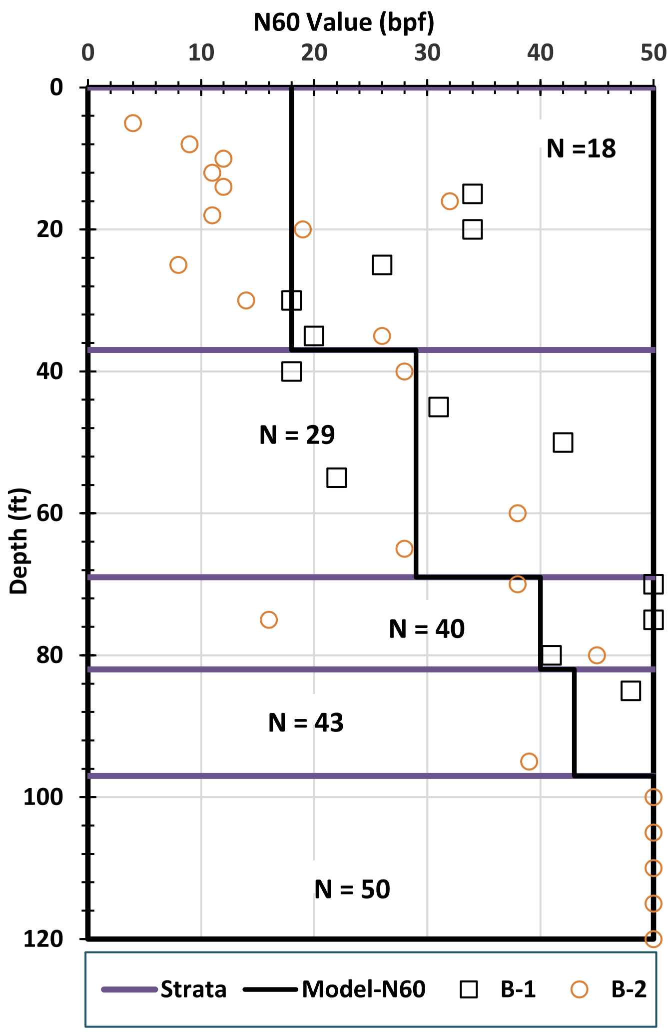


Figure 1. SPT N60 Values for Walnut Creek Dam

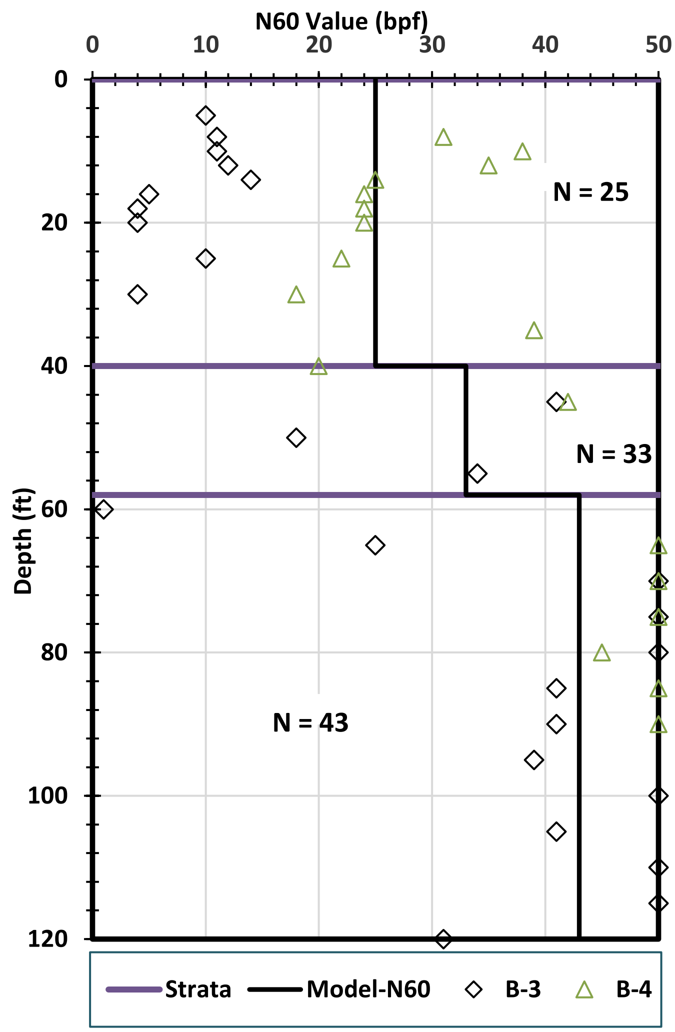


Figure 2. SPT N60 Values for Birch Creek Dam

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**

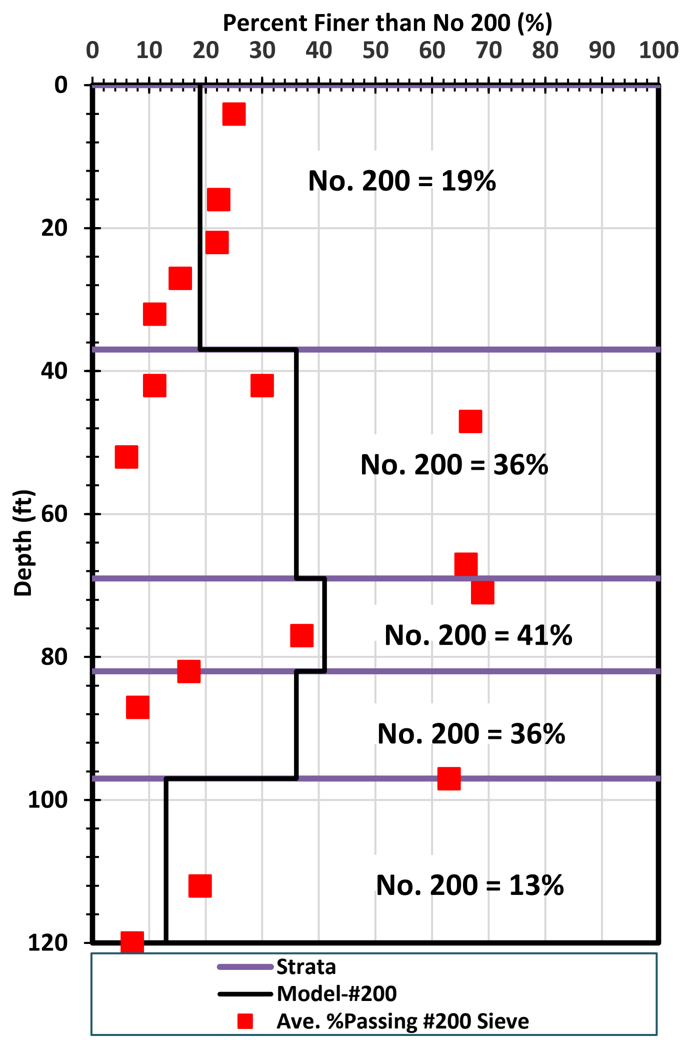


Figure 3. Percent Finer than No. 200 for Walnut Creek Dam

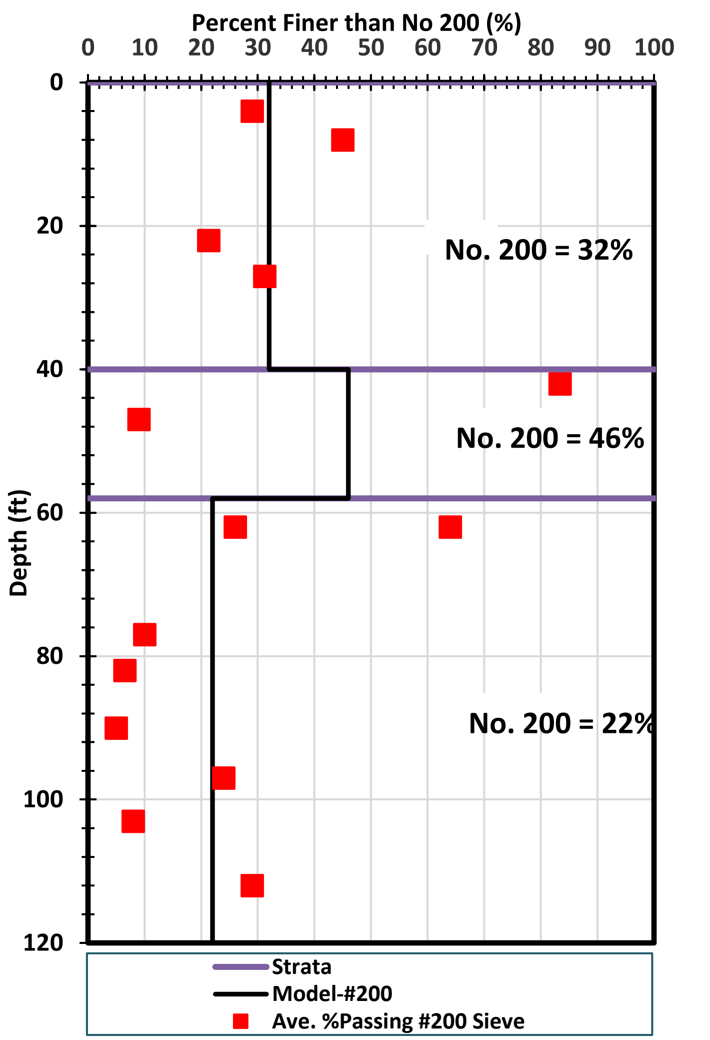


Figure 4. **Percent Finer than No. 200 for Birch Creek Dam**

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**

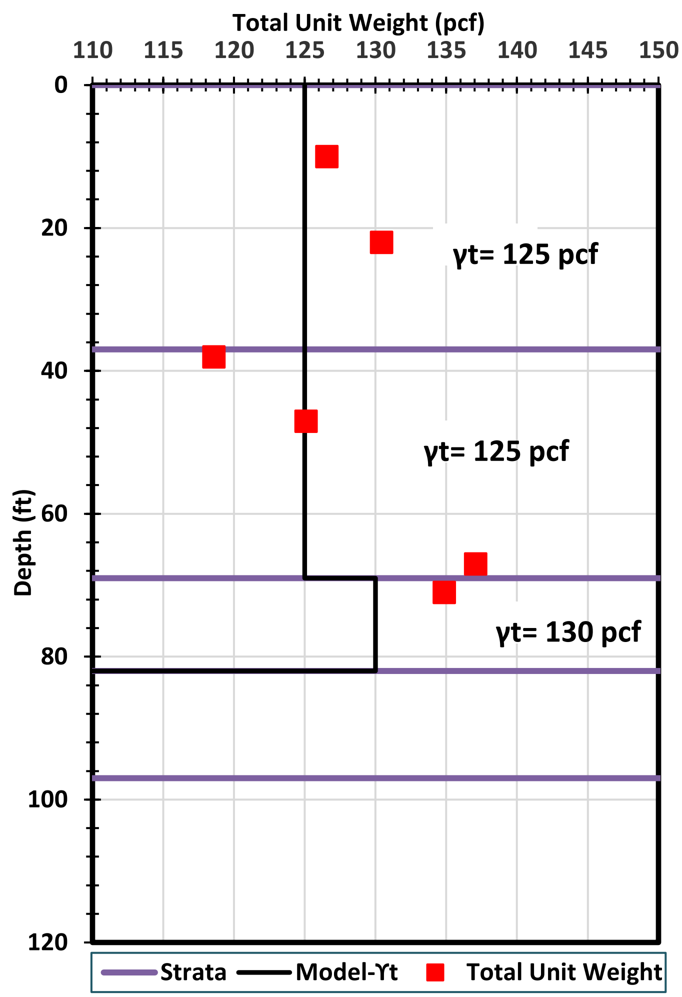


Figure 5. **Total Unit Weight Design Profiles for Walnut Creek Dam**

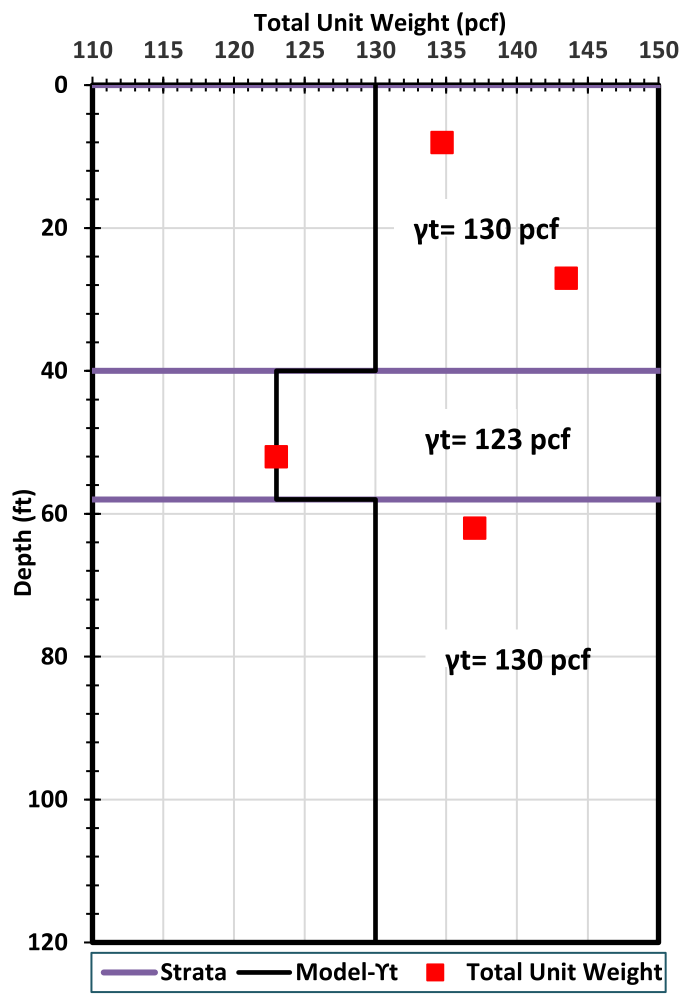


Figure 6. **Total Unit Weight Design Profiles for Birch Creek Dam**

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**

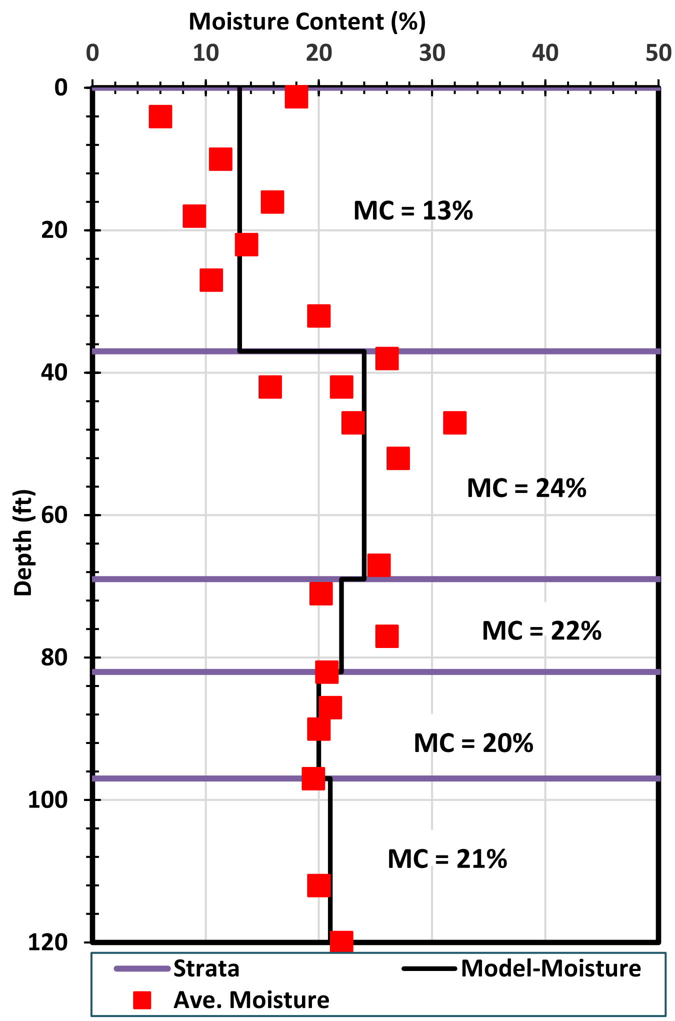


Figure 7. Moisture Content Profiles for Walnut

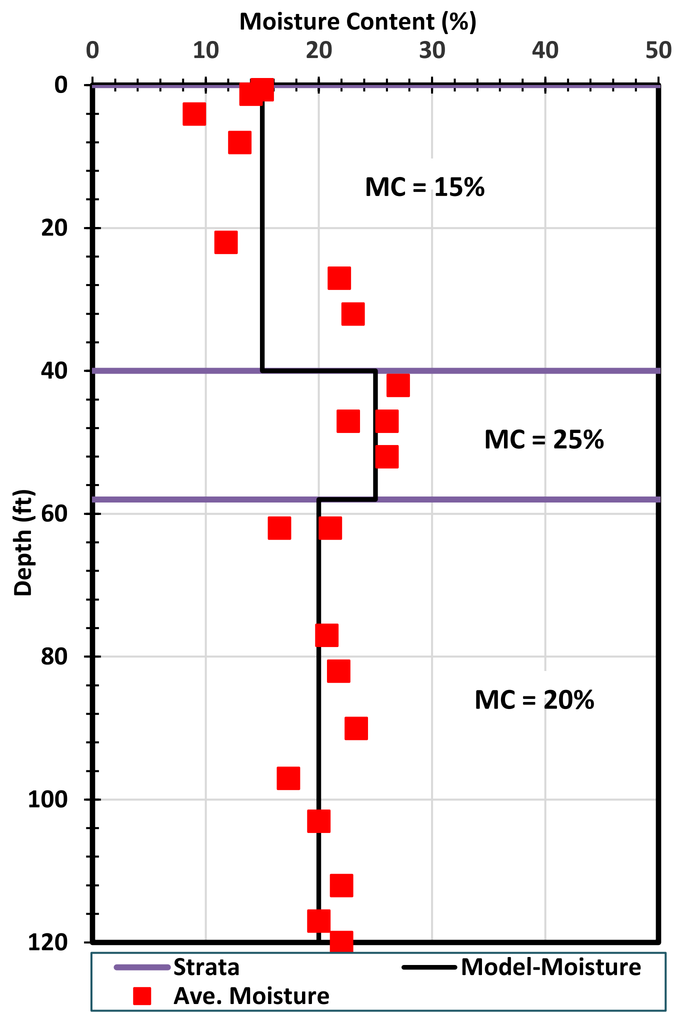


Figure 8. Moisture Content Profiles for Birch

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**

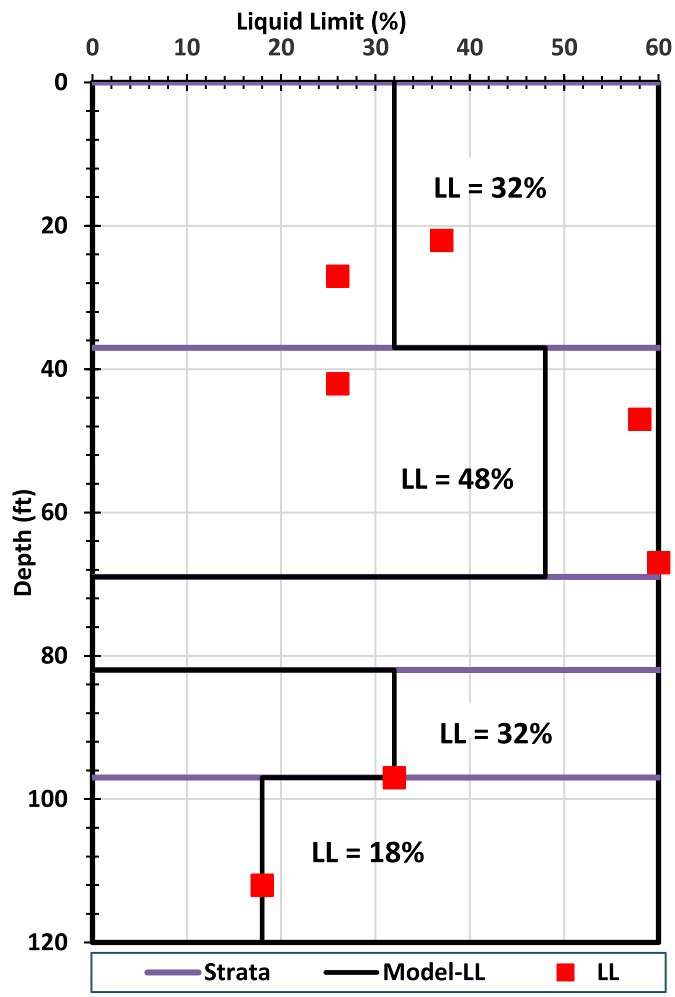


Figure 9. **Liquid Limit Profiles for Walnut**

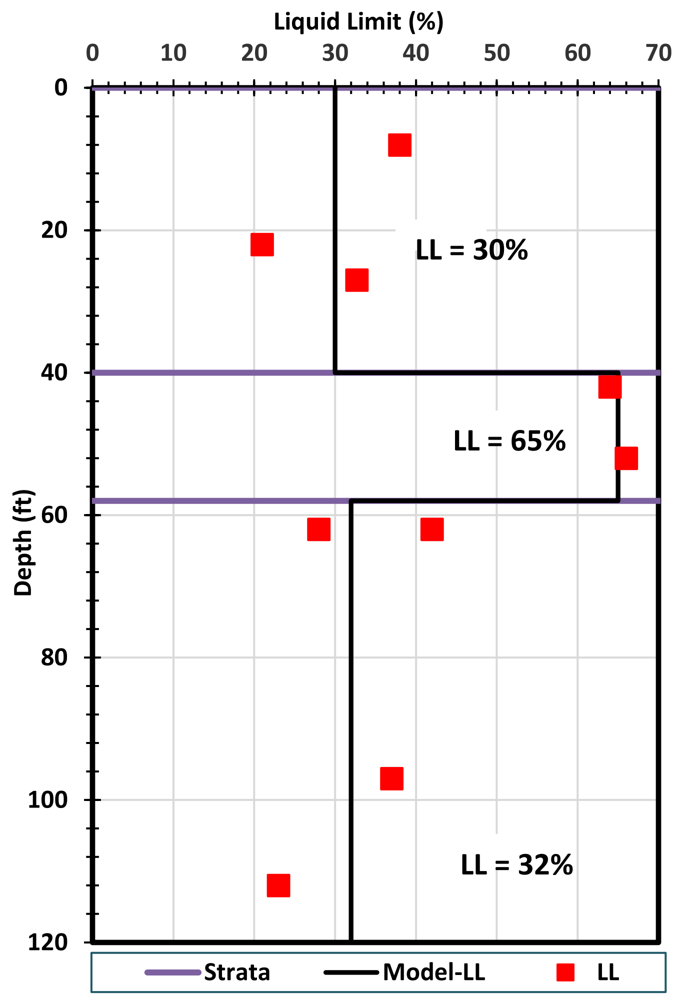


Figure 10. **Liquid Limit Profiles for Birch**

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**

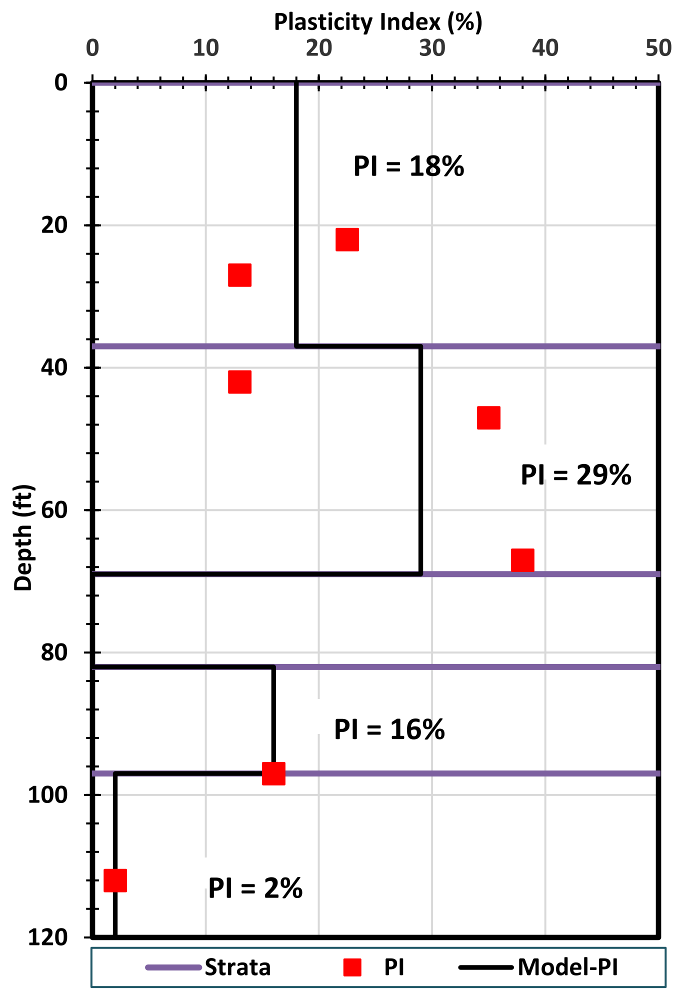


Figure 11 **Plasticity Index Profiles for Walnut**

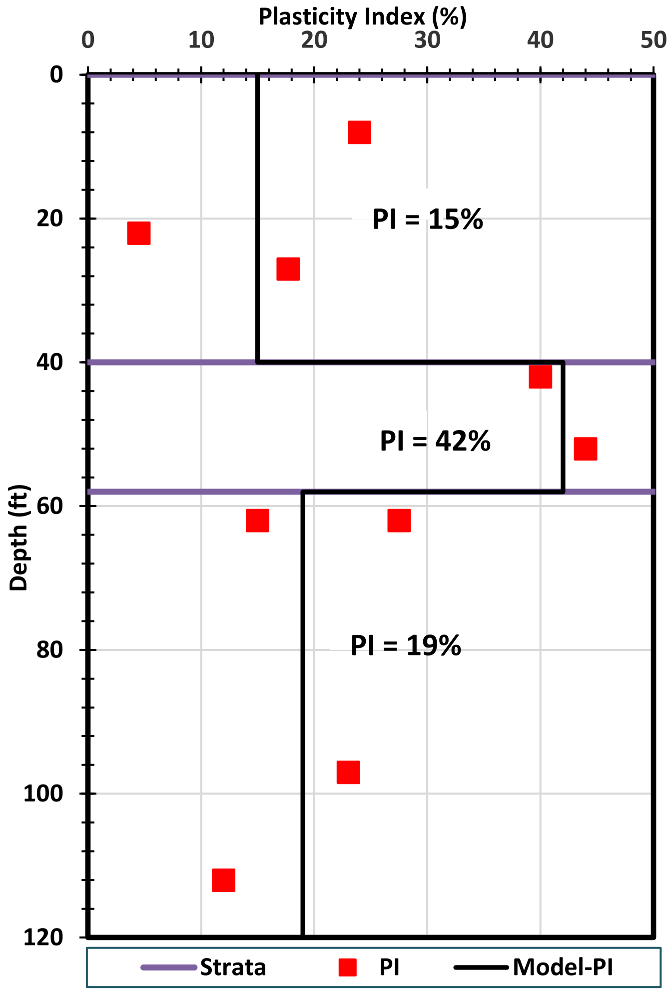


Figure 12 **Plasticity Index Profiles for Birch**

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.

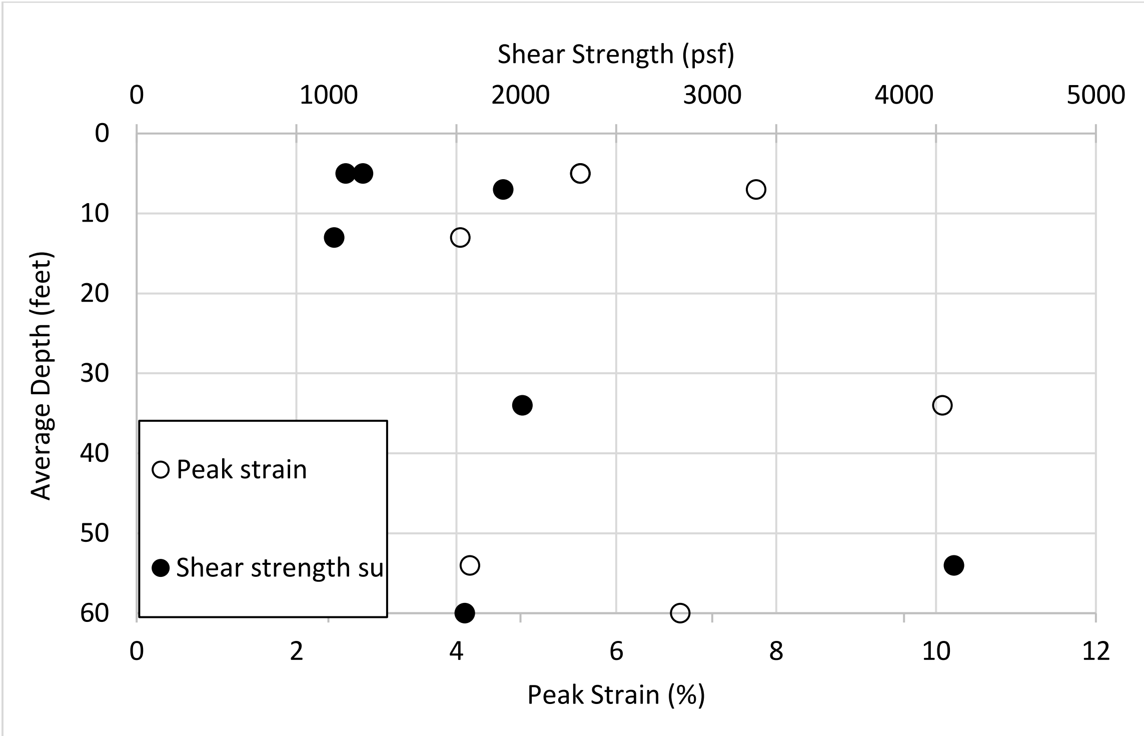


Figure 13. **UC and UU Triaxial Strength versus Depth of Soil Sample for All Borings**

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**

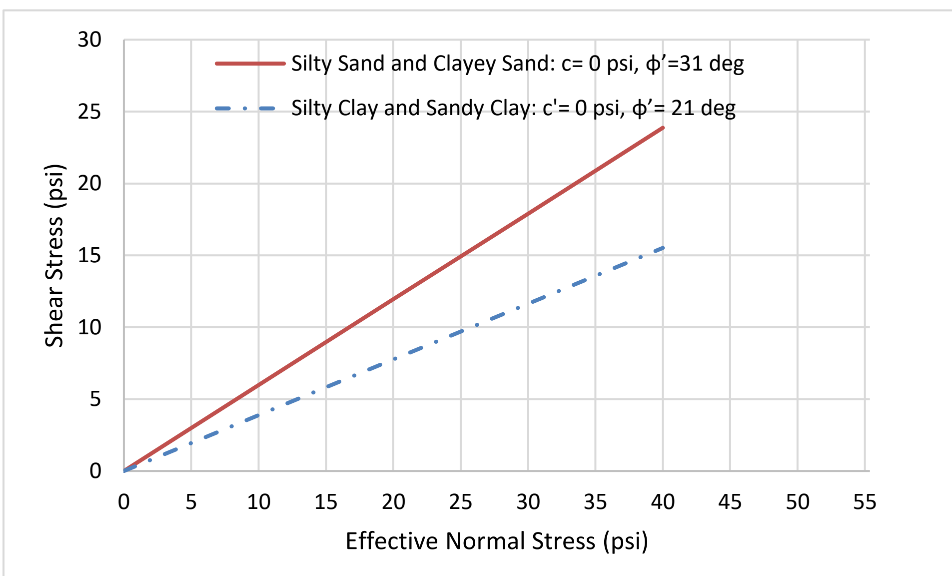


Figure 14. Effective Stress Shear Strength Design Envelope from CU Tests

4.5.3 R-Case

A summary of the R-Case ϕR 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

Stratum	Laboratory Data					Design Envelope	
	Sample Size (n)	ϕR (°)		CR (psf)			
		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.

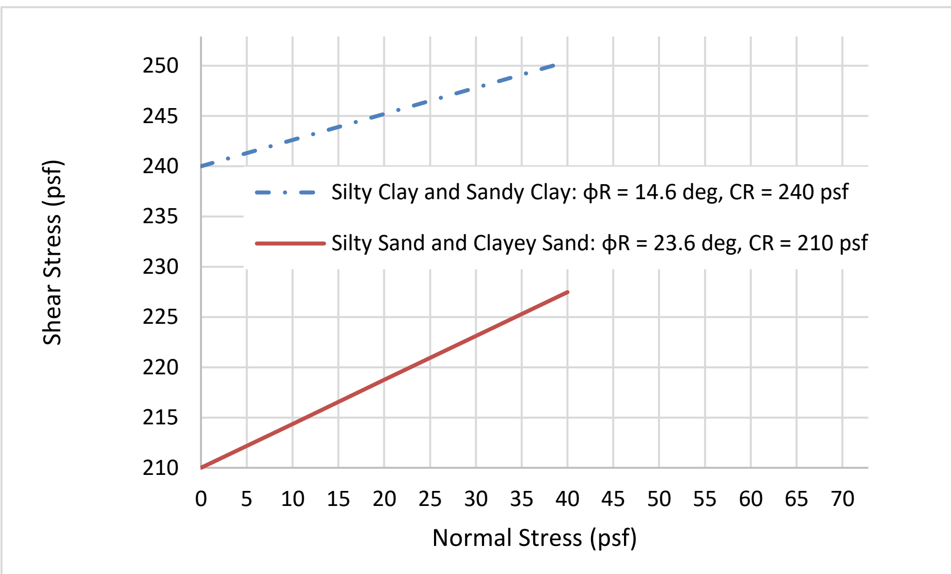


Figure 15. R-Envelope Shear Strength Design Envelope from CU Tests

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	$1.20{\times}10^{-7}$	$1.09{\times}10^{-7}$	$1.28{\times}10^{-7}$	**$1{\times}10^{-7}$**	**$1{\times}10^{-5}$ — $1{\times}10^{-8}$**
Silty Clay and Sandy Clay[2]	4	$1.38{\times}10^{-8}$	$8.89{\times}10^{-9}$	$2.83{\times}10^{-8}$	**$1{\times}10^{-8}$**	**$1{\times}10^{-6}$ — $1{\times}10^{-9}$**

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

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	—	—	—	—	0.405	**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

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	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, Proteco	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, 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, 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**		

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, 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, 19	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, 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, Proteco	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, 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, 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, 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, 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

Stratum	Casagrande Method (Consolidation Testing)				Design Value
	Sample Size (n)	Avg.	Min.	Max.	
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

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

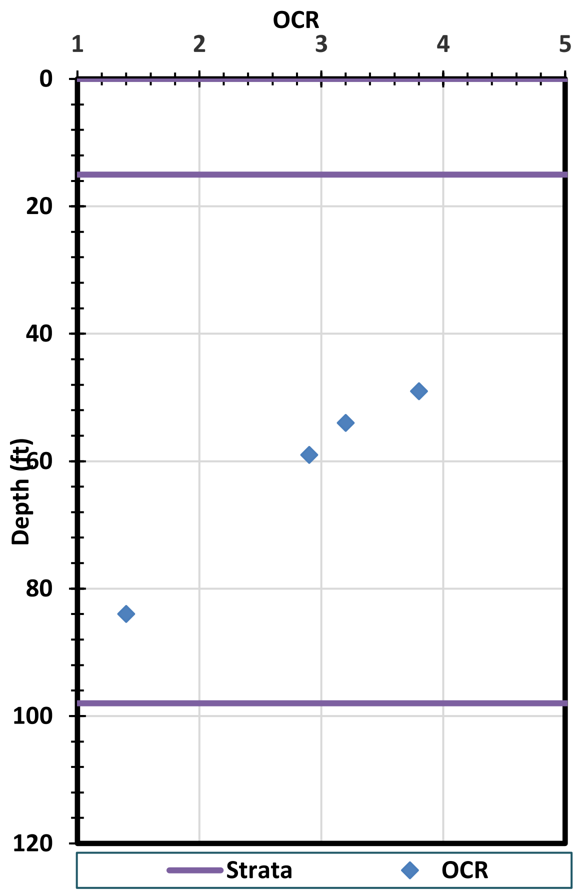


Figure 20 OCR From Laboratory Consolidation Tests

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	3×10^{-9}, (10^{-7})
Filter	120	130	3×10^{-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

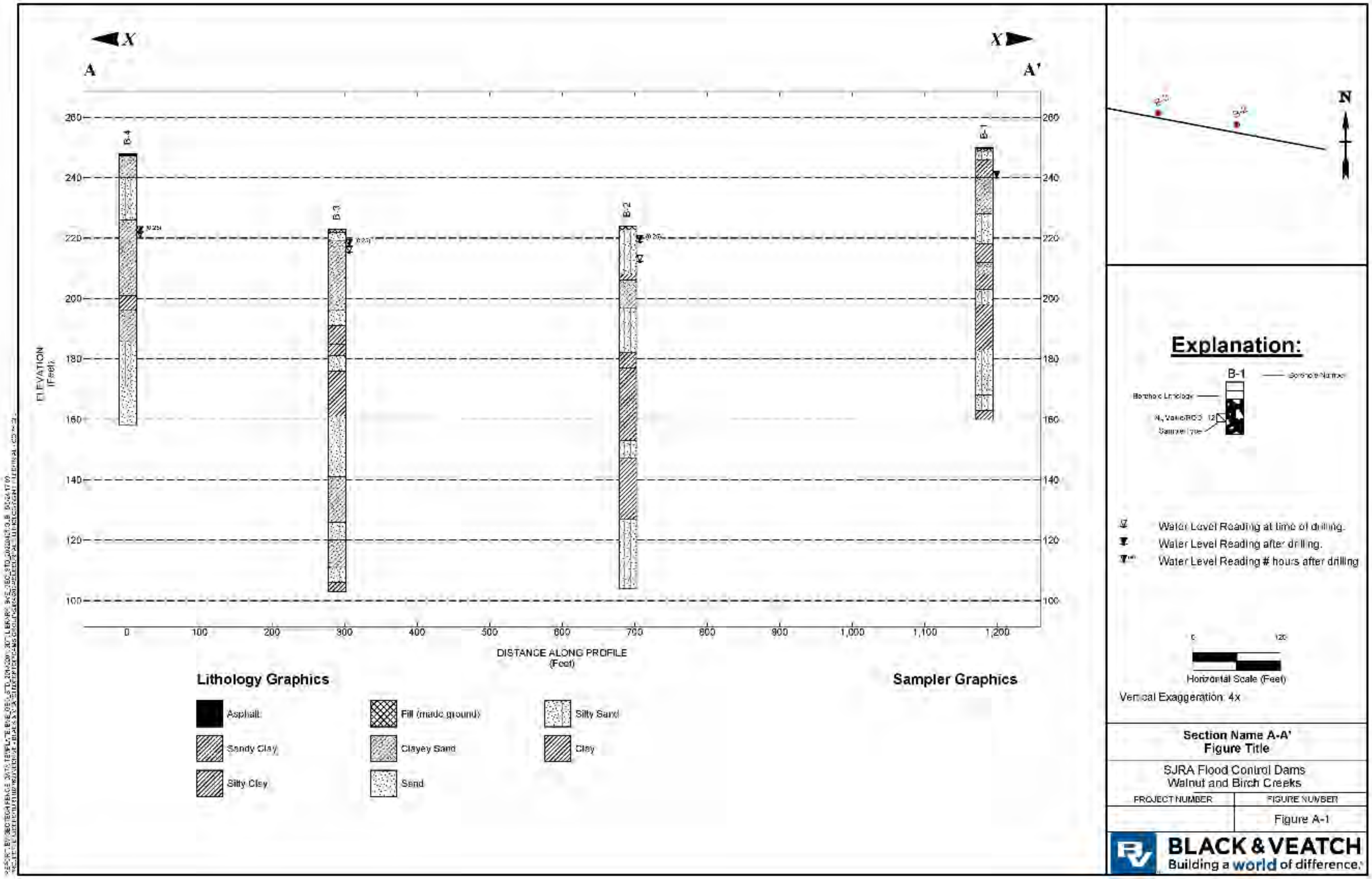
Zone	Undrained Strength (psf)	Drained Strength		R-Envelope	
		c' (psf)	ϕ' (deg)	cR (psf)	ϕR (deg)
SBC	No strength				
Filter	—	0	36	—	
Riprap	—	0	40	—	

Attachment 1:

Subsurface Profile

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	55

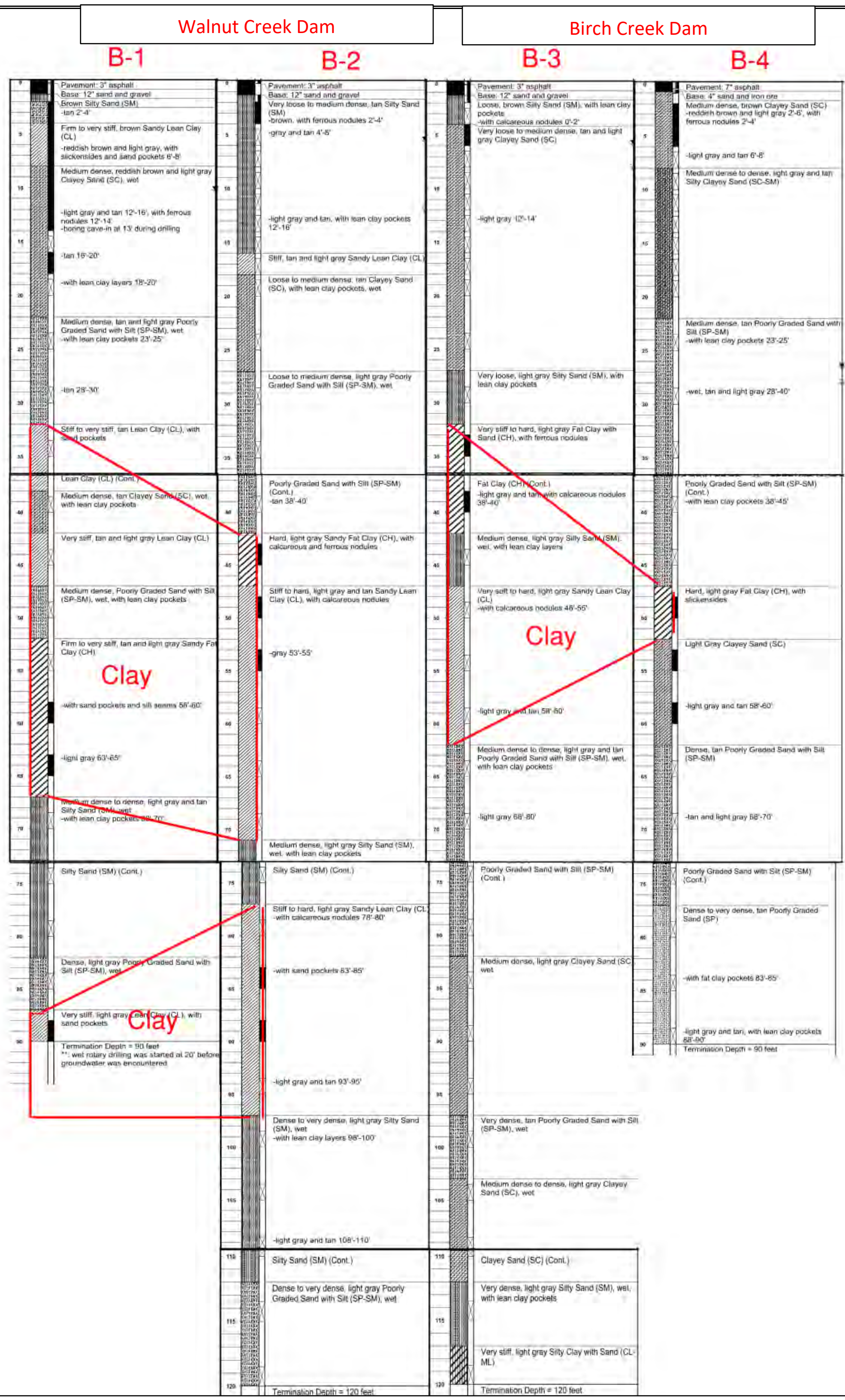


Attachment 2:

Interpreted Subsurface Profile from Boreholes

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	57



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

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

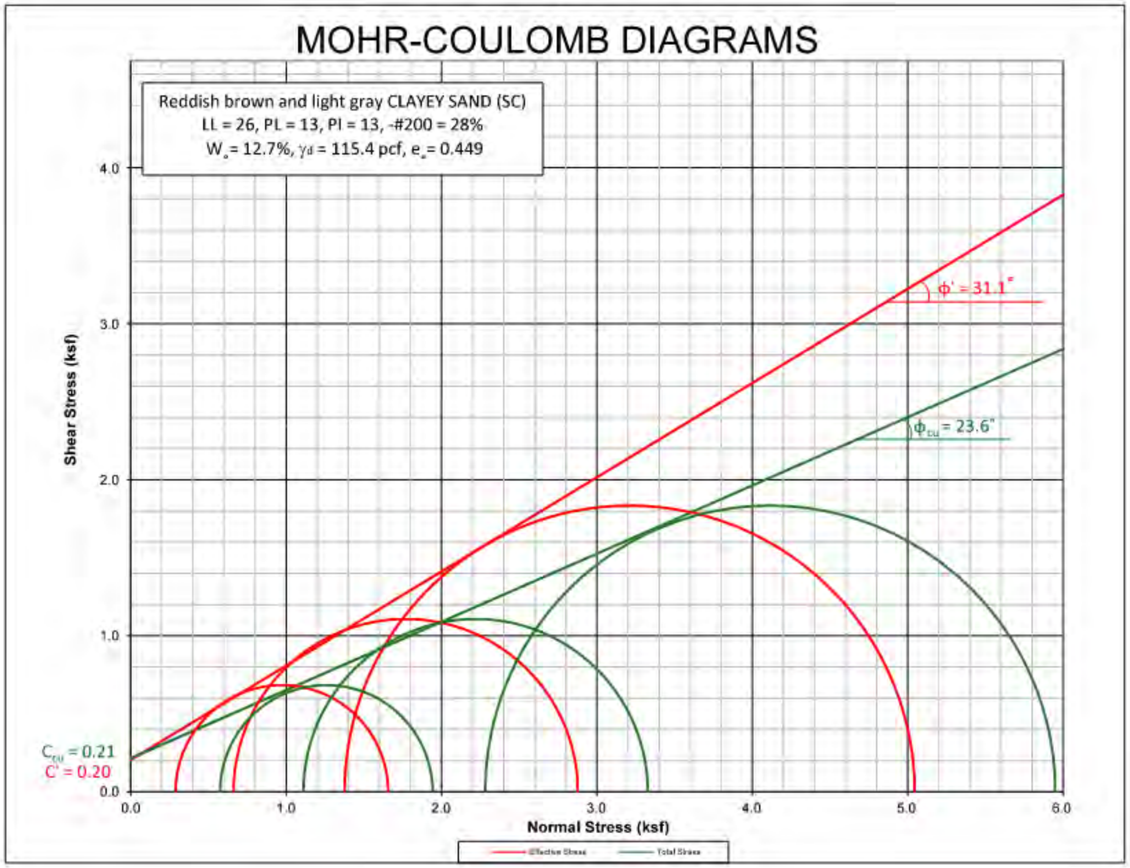


PLATE A-22

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	60

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

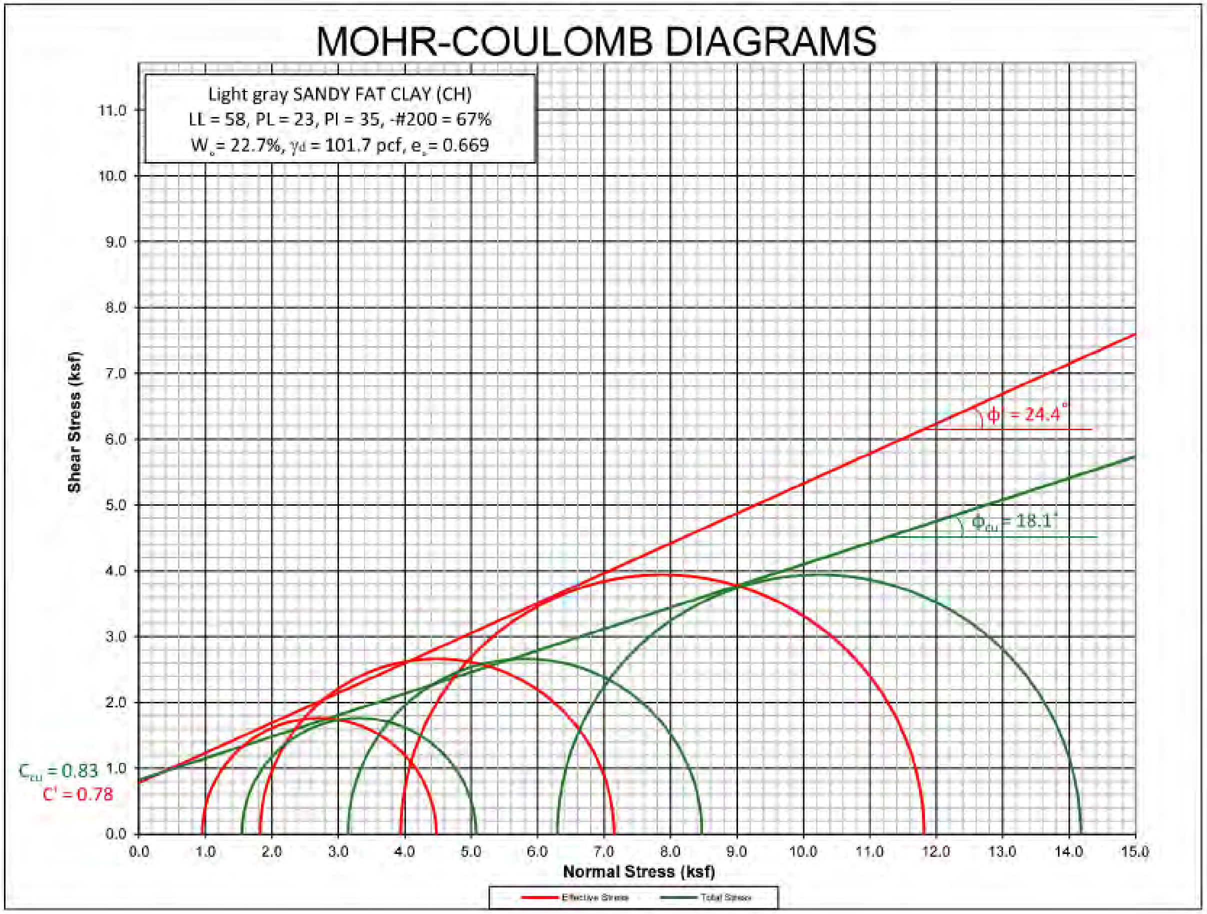
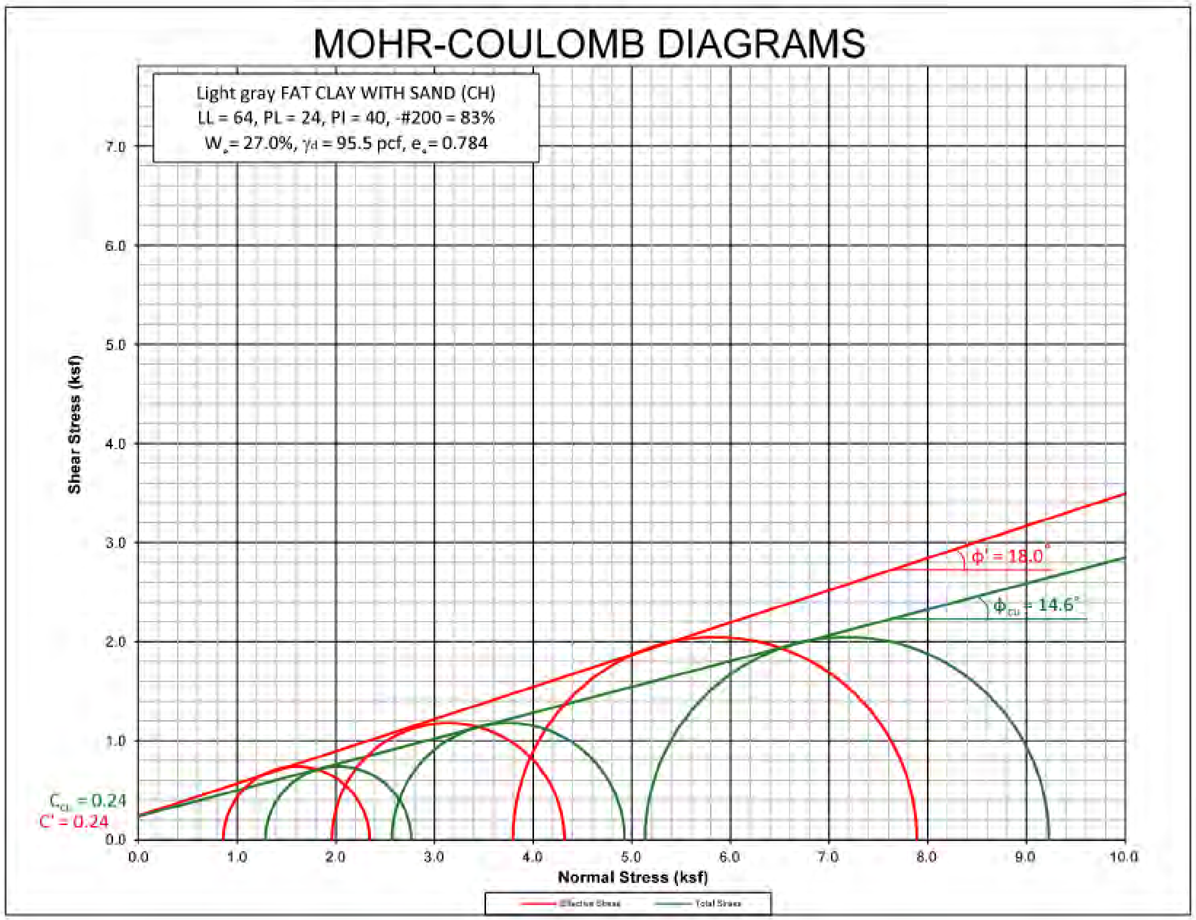


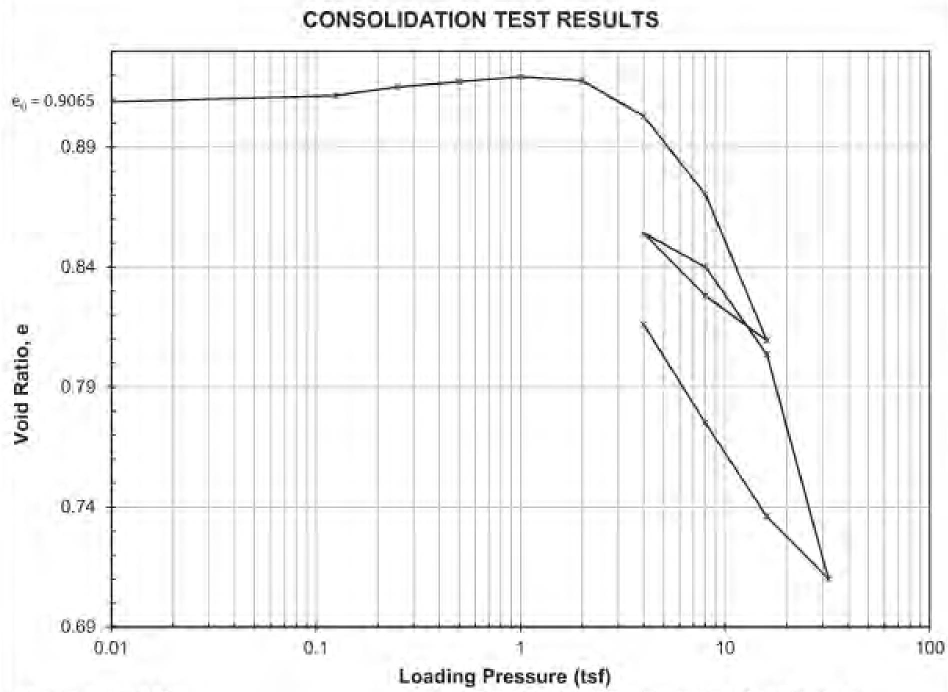
PLATE A-23

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

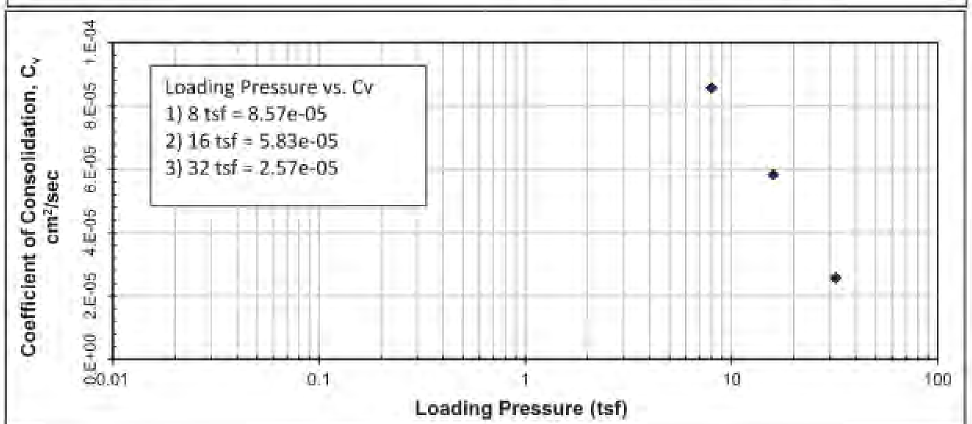


Attachment 4:

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



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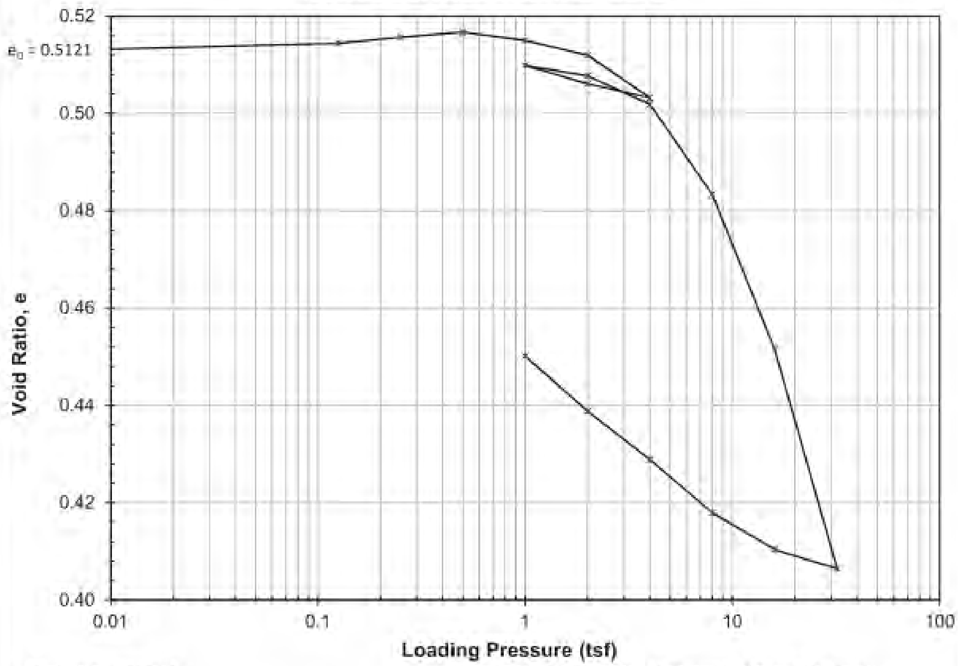
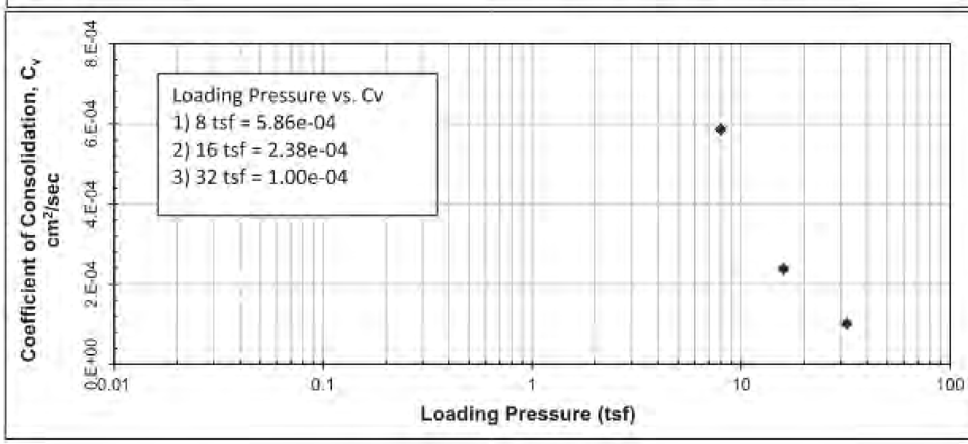
Client	SJRA	**Computed By**	P. Turkson
Project	SCW Flood Control Dams **Unit**	**Date**	10/25/2024
Project No.	411500 **File No.**	**Approved By**	David Bentler
Title	Evaluation of Project Soil Parameters	**Date**	12/6/2024
		Page	64

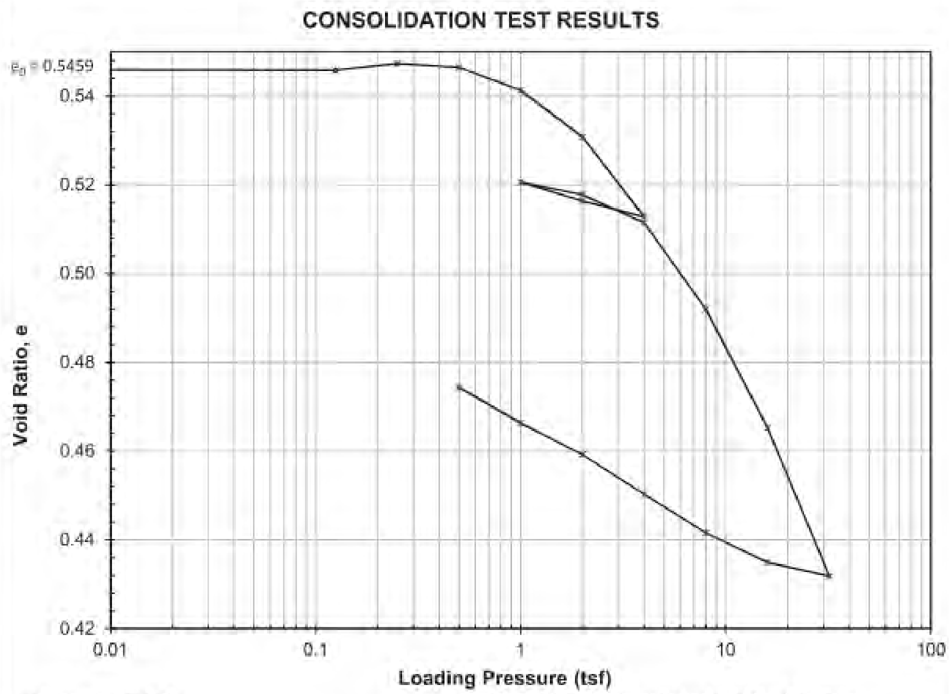
CONSOLIDATION TEST RESULTS



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

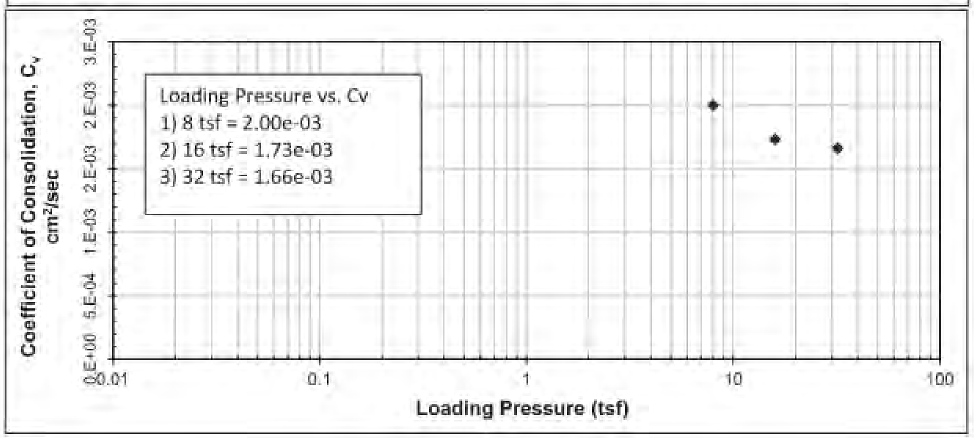




Project No.: G154-21
Sample ID: B-2, 83 to 85 feet
Sample Description: Light gray Sandy Lean Clay (CL)
Estimated Consolidation Index (Cc): 0.1110
Estimated OCR: 1.4

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



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	66

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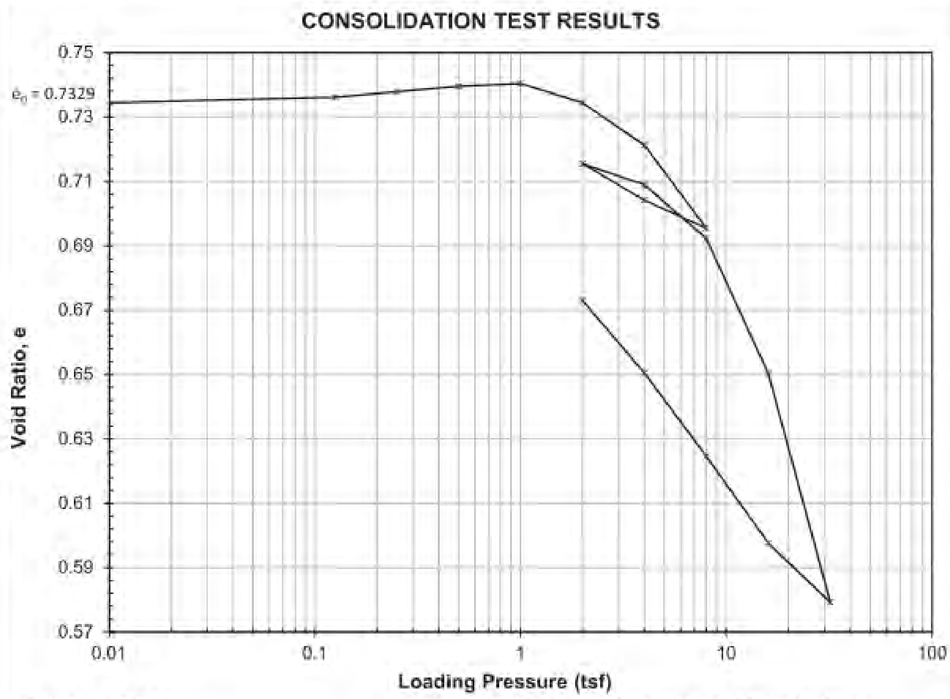

Project No.: G154-21
Sample ID: B-4, 48 to 50 feet
Sample Description: Light gray Fat Clay (CH)
Estimated Consolidation Index (Cc): 0.2362
Estimated OCR: 3.8

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

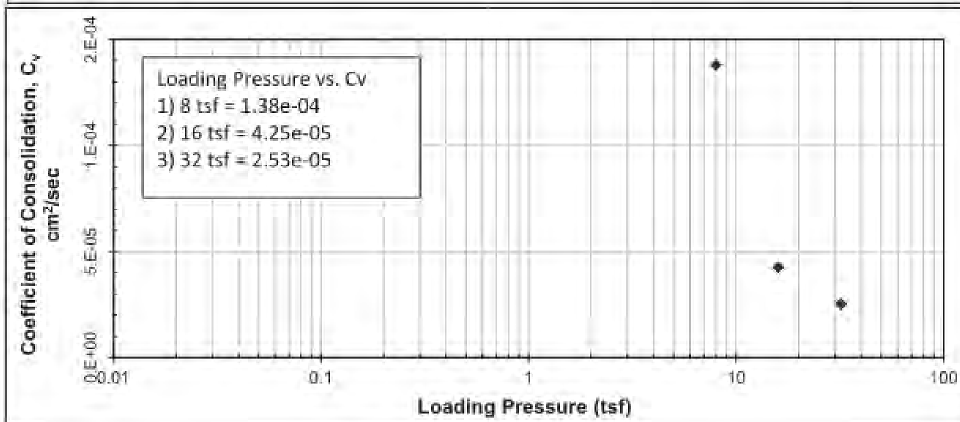

PLATE A-28

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	35 - 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.

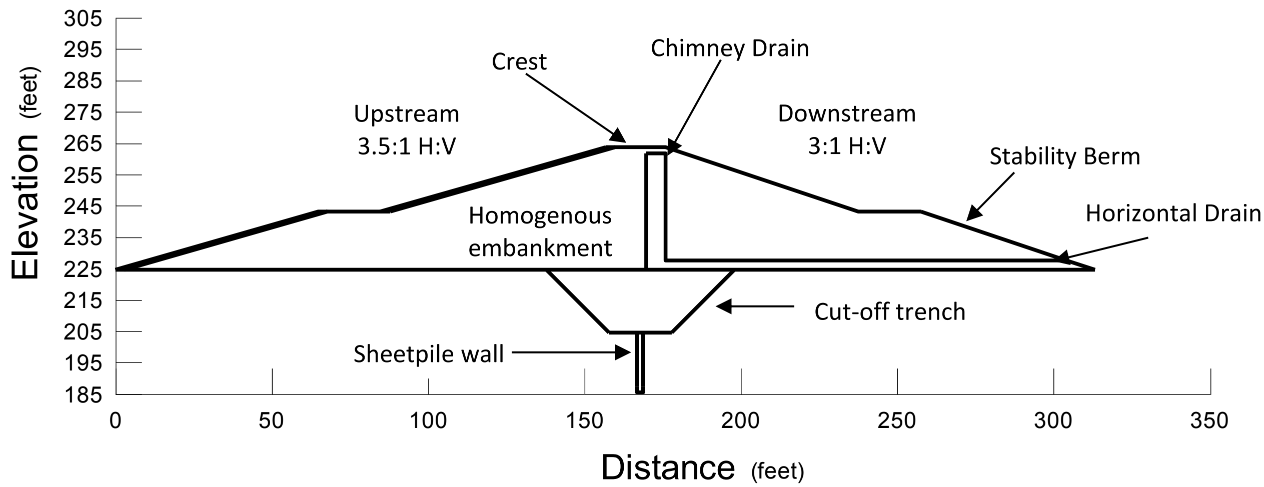

Figure 1. Alternative 1 Embankment Geometry

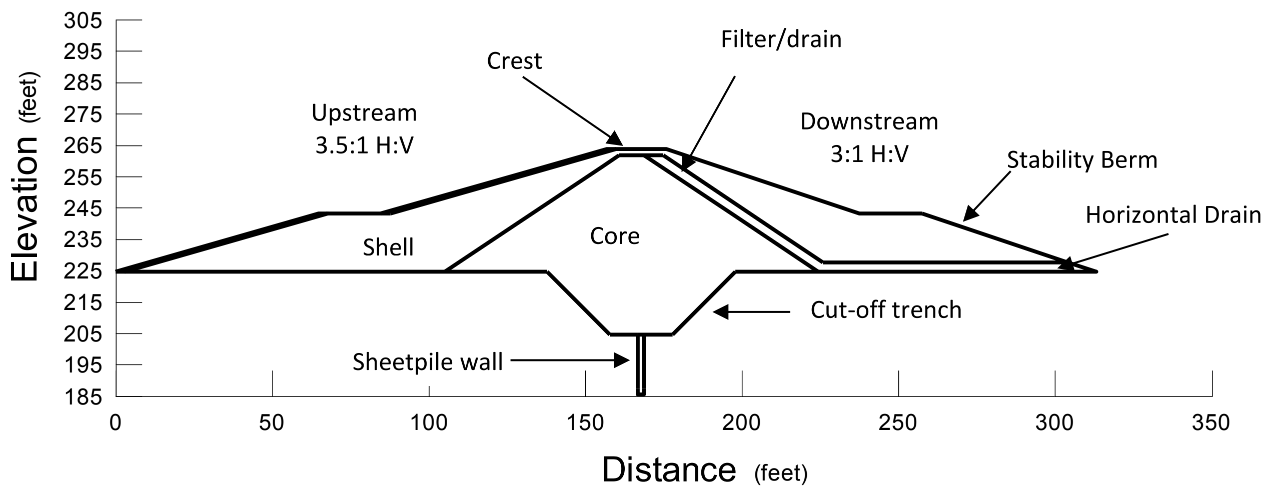

Figure 2. Alternative 2 Embankment Geometry

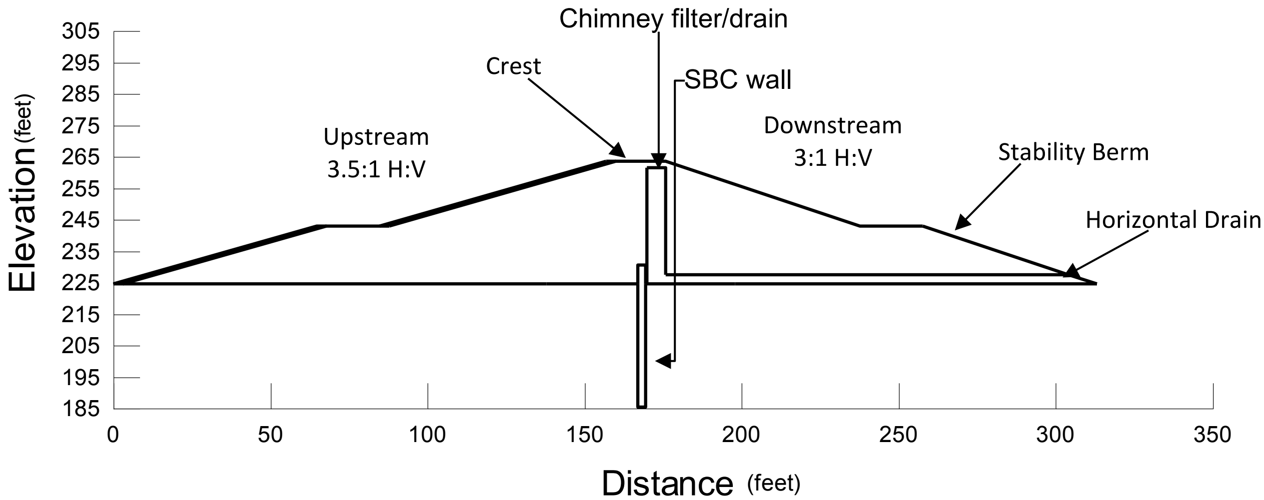

Figure 3. Alternative 3 Embankment Geometry

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}, (1×10^{-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}, (1×10^{-9}))	3×10^{-8}, (1×10^{-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**.

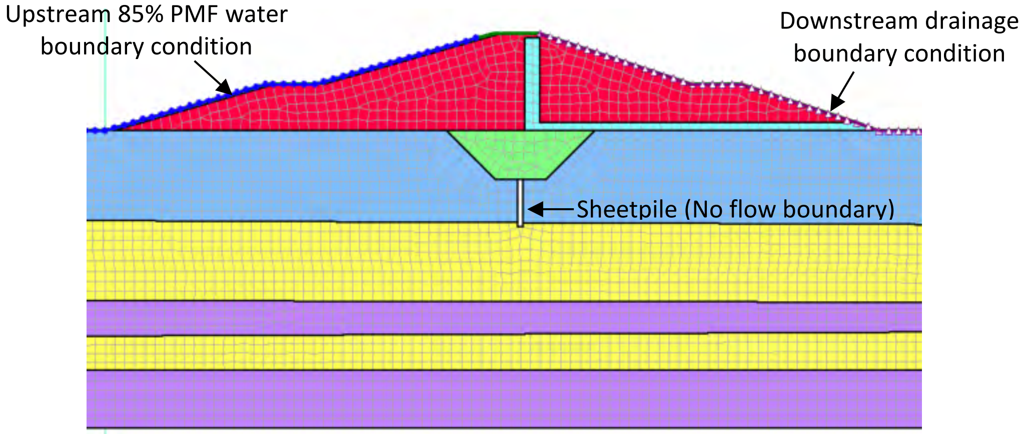

Figure 4. Alternative 1 Model

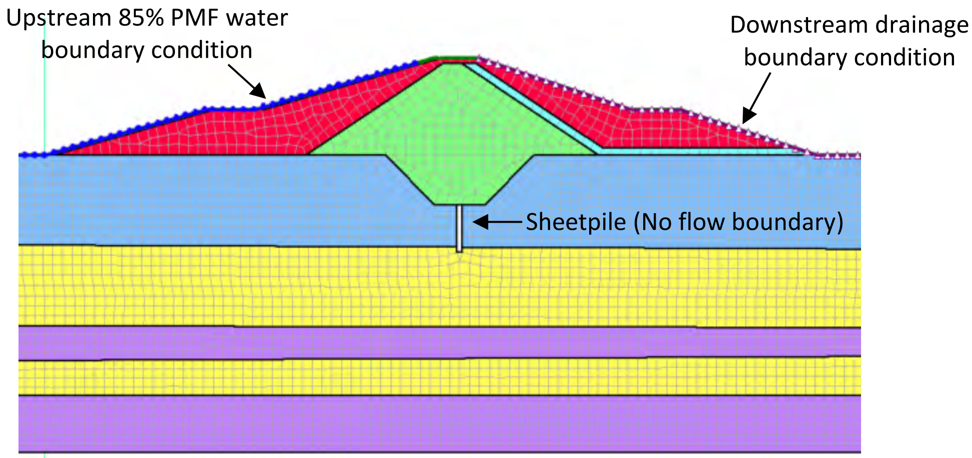

Figure 5. Alternative 2 Model

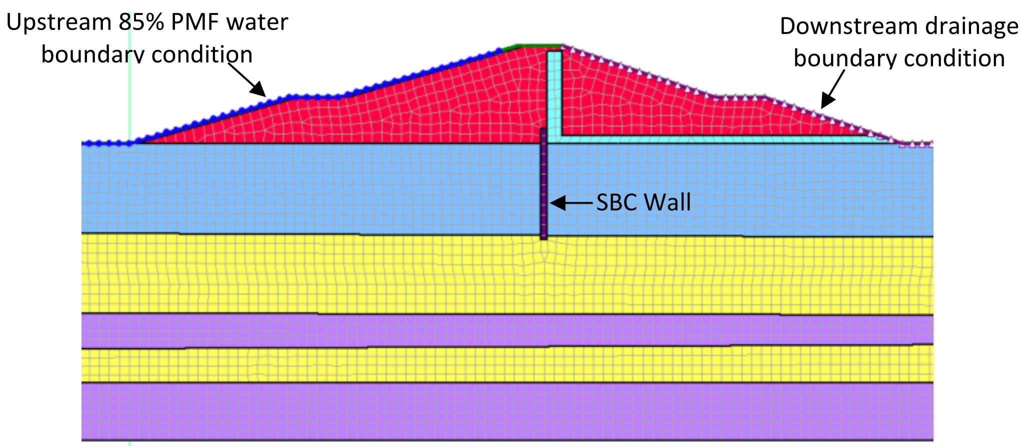

Figure 6. Alternative 3 Model

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= 3.28×10^{-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

Attachment 2:

Seepage Analyses Results Sections for Design Permeability

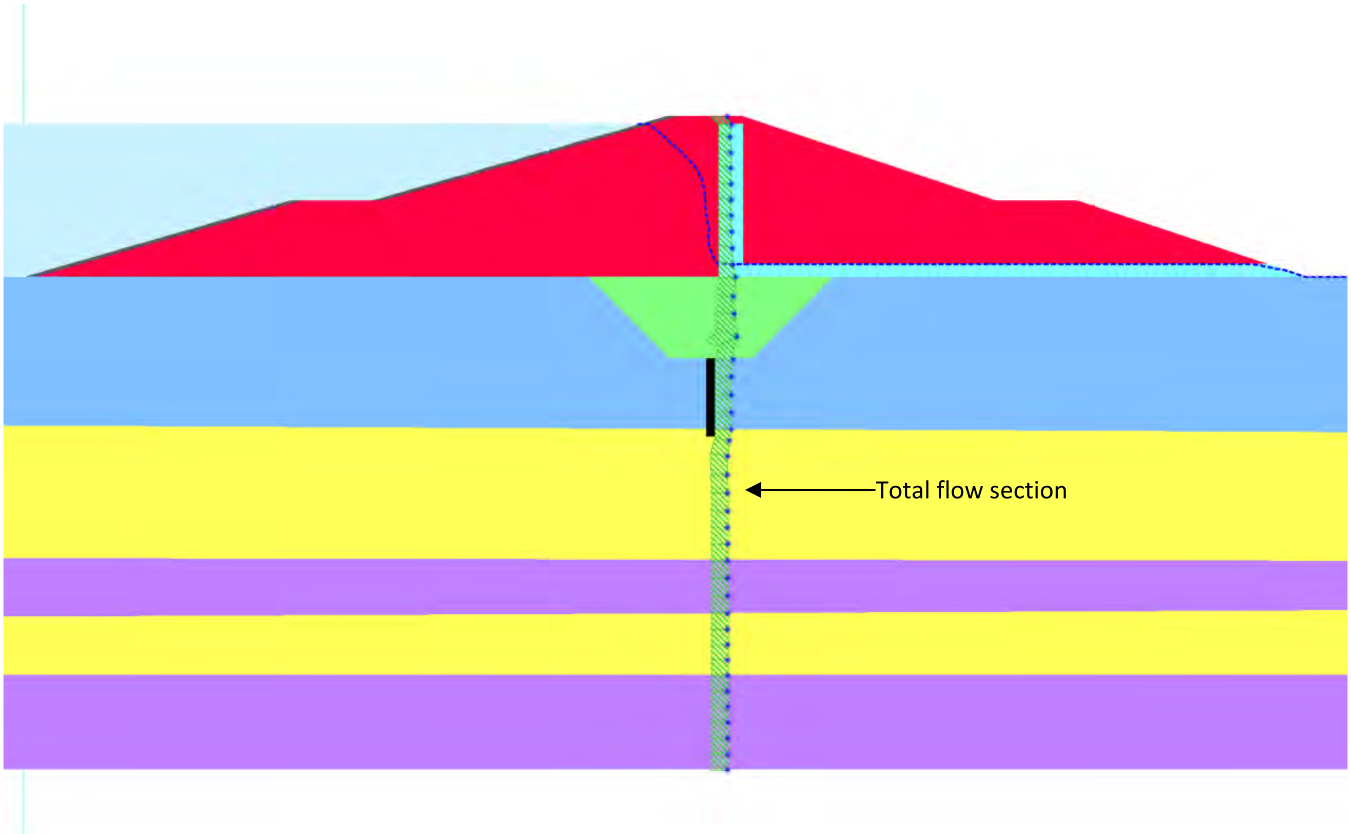

Figure 7. Walnut Creek Alternative 1 Combined Flow Through Dam and Foundation= 0.006 ft^3/day/ft

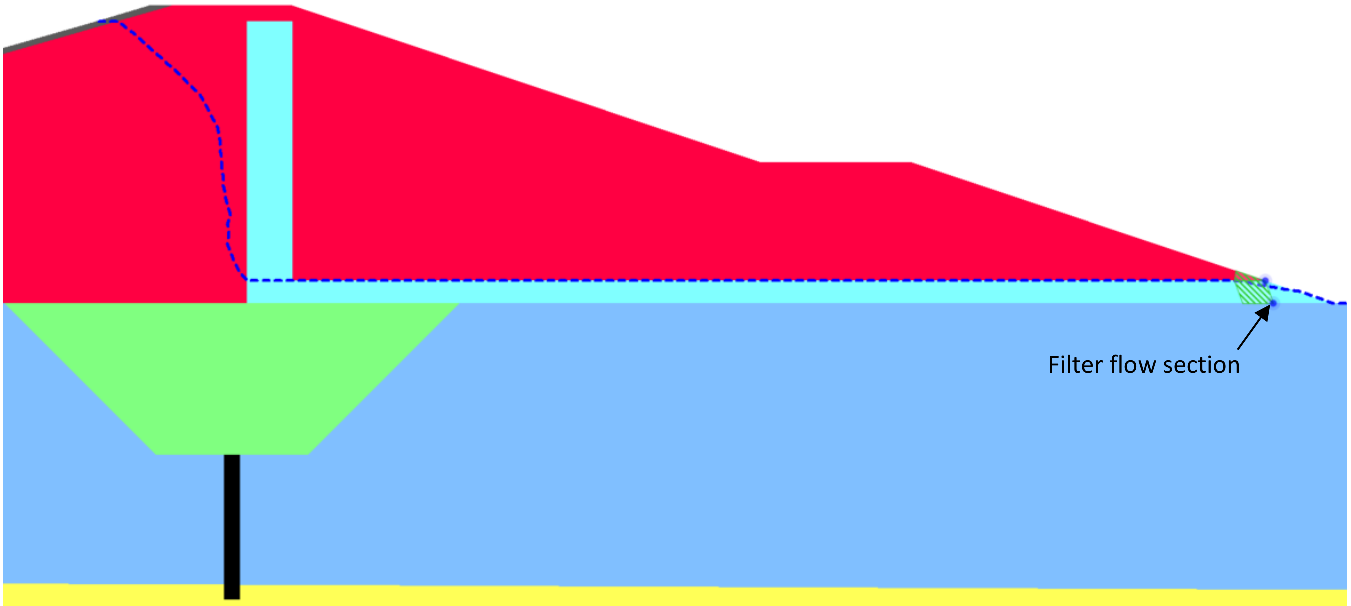

Figure 8. Walnut Creek Alternative 1 Flow Through Filter = 0.005 ft^3/day/ft

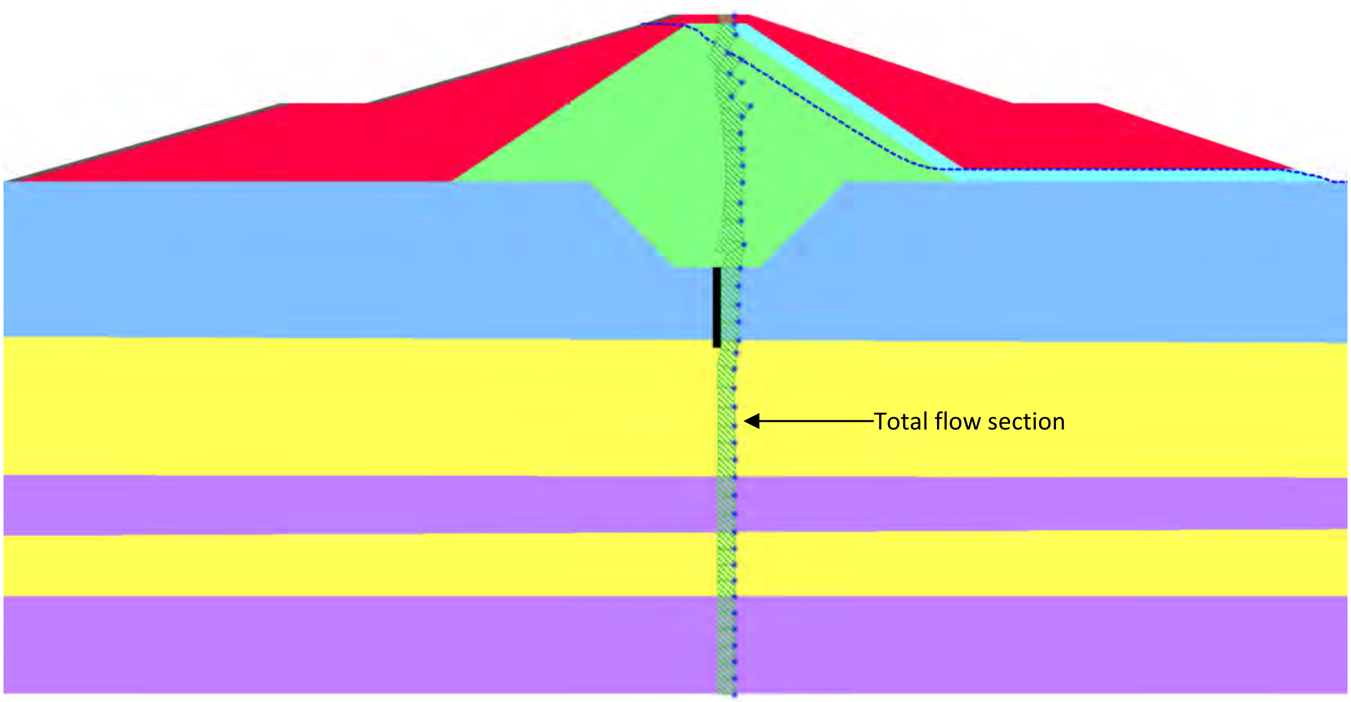

Figure 9. Walnut Creek Alternative 2 Combined Flow Through Dam and Foundation = 0.002 ft^3/day/ft

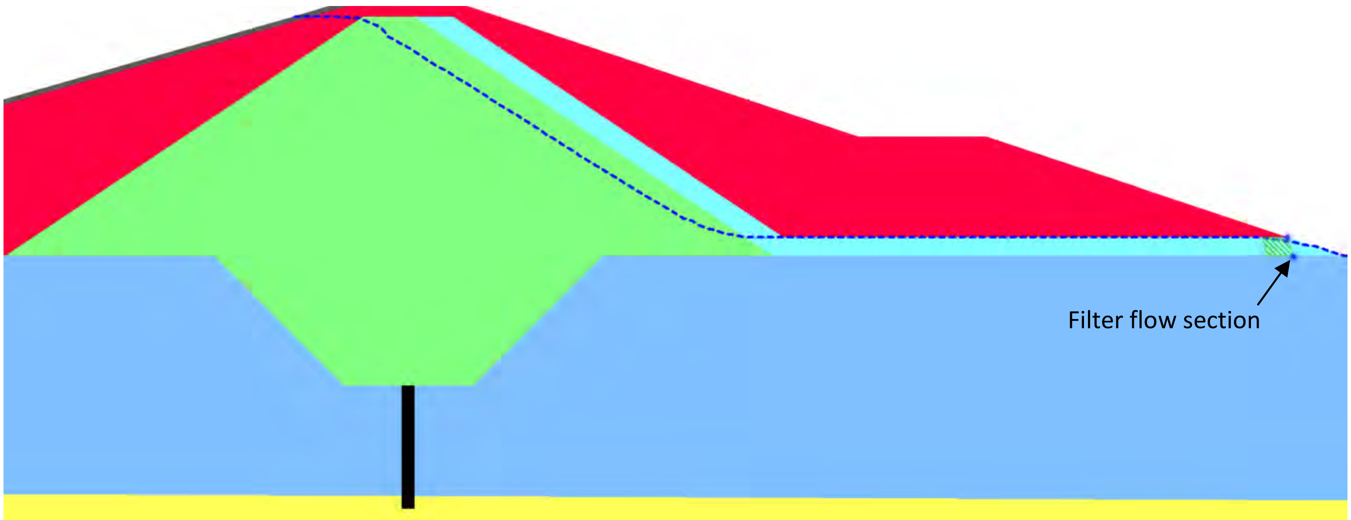

Figure 10 **Walnut Creek Alternative 2 Flow Through Filter = 0.001 ft^3/day/ft**

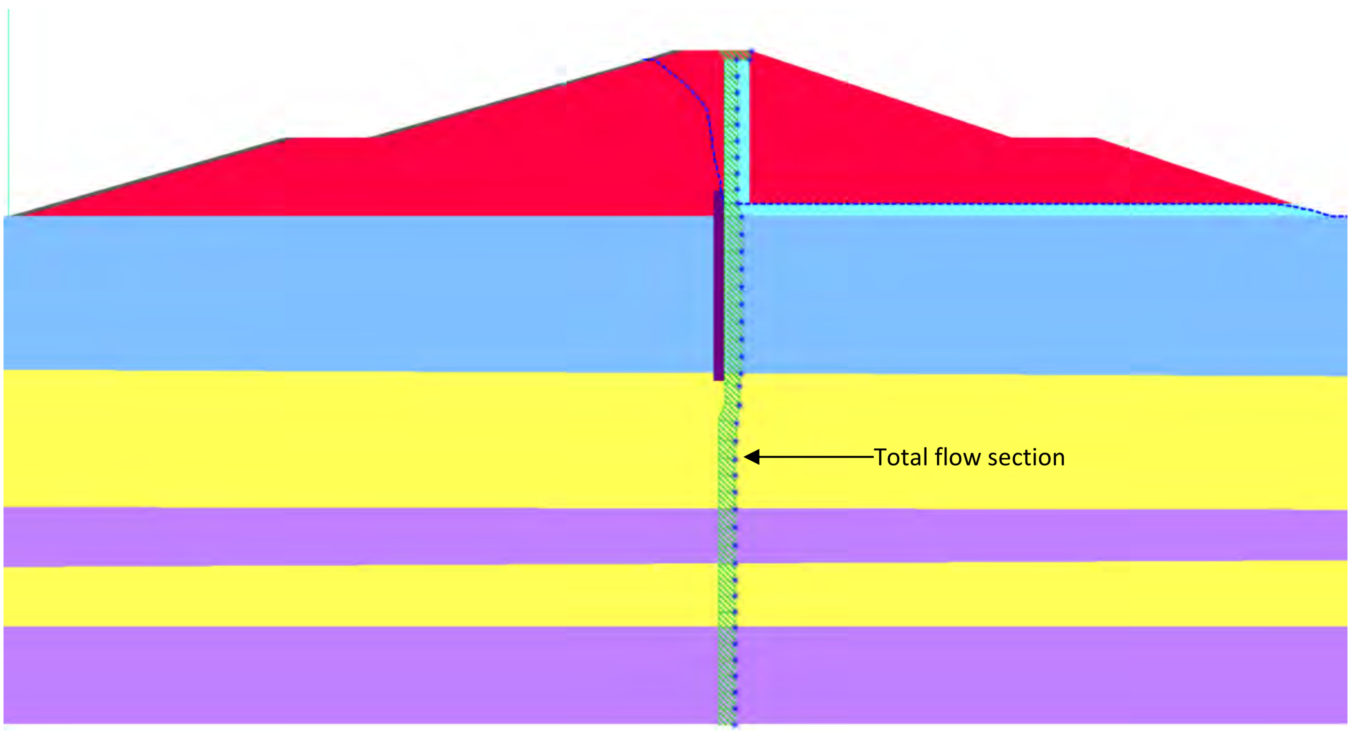

Figure 11. **Walnut Creek Alternative 3 Combined Flow Through Dam and Foundation = 0.005 ft^3/day/ft**

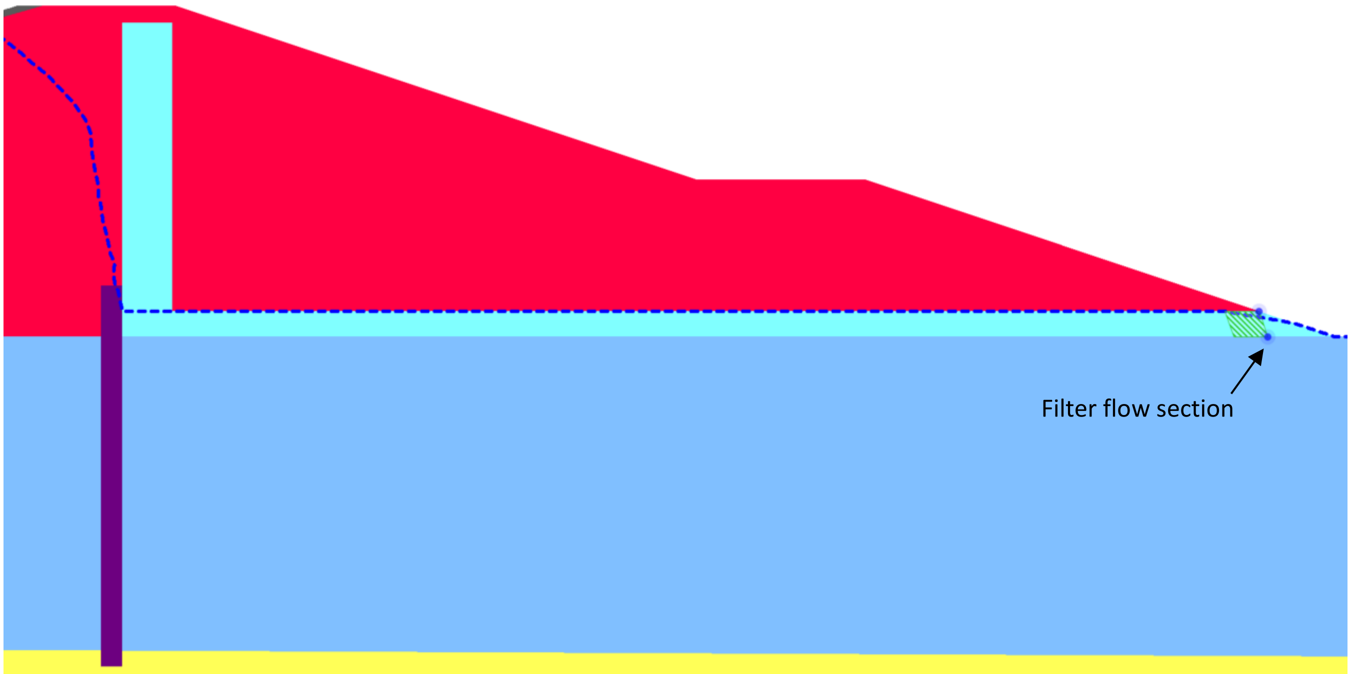

Figure 12 **Walnut Creek Alternative 3 Flow Through Filter = 0.005 ft^3/day/ft**

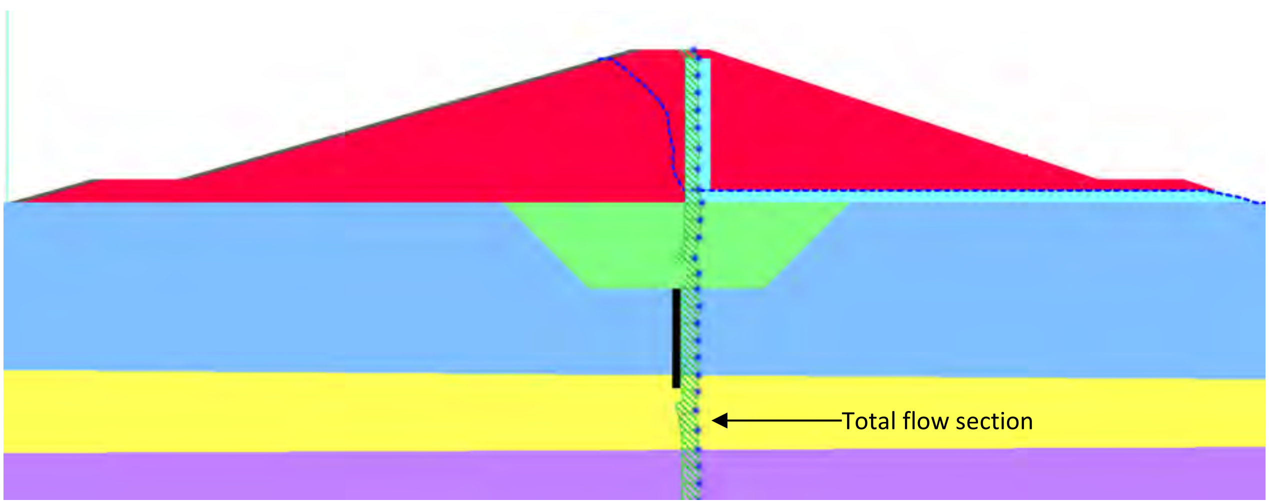

Figure 13. **Birch Creek Alternative 1 Combined Flow Through Dam and Foundation = 0.006 ft^3/day/ft**

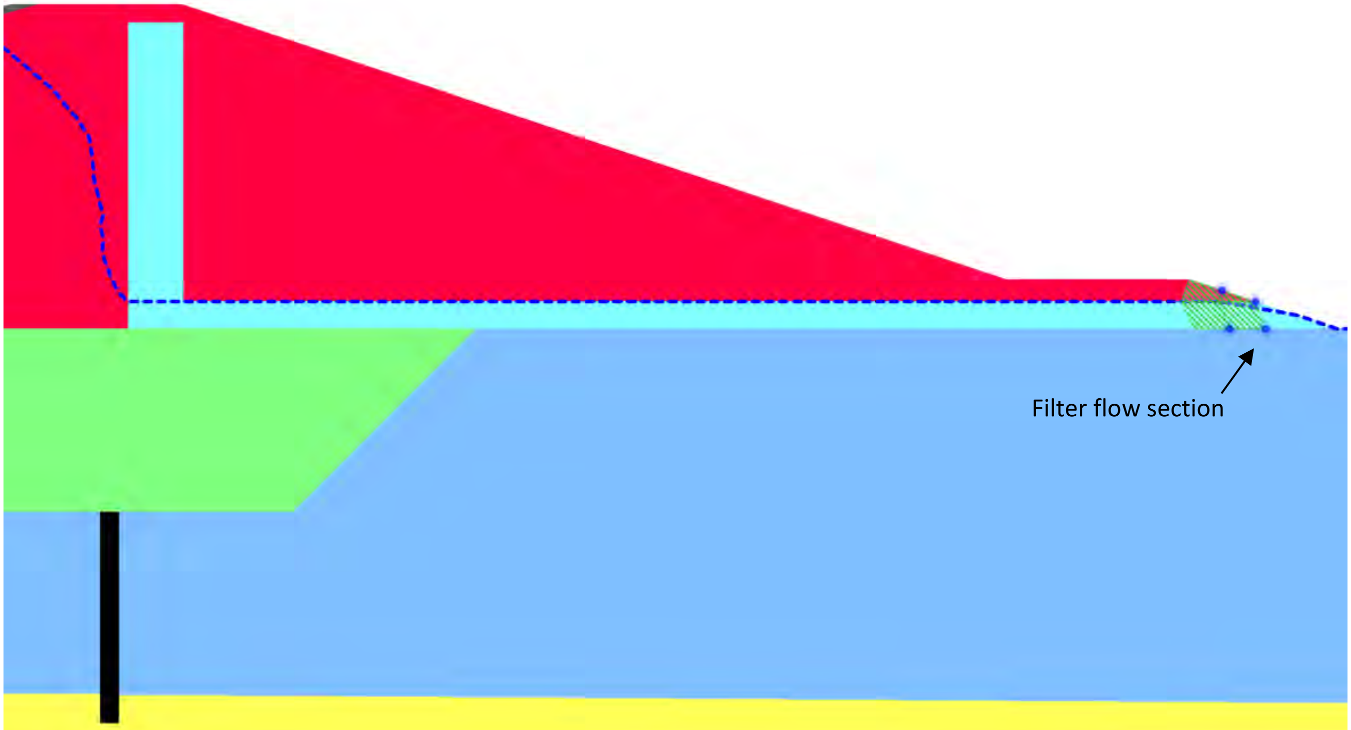

Figure 14. Birch Creek Alternative 1 Flow Through Filter = 0.004 ft^3/day/ft

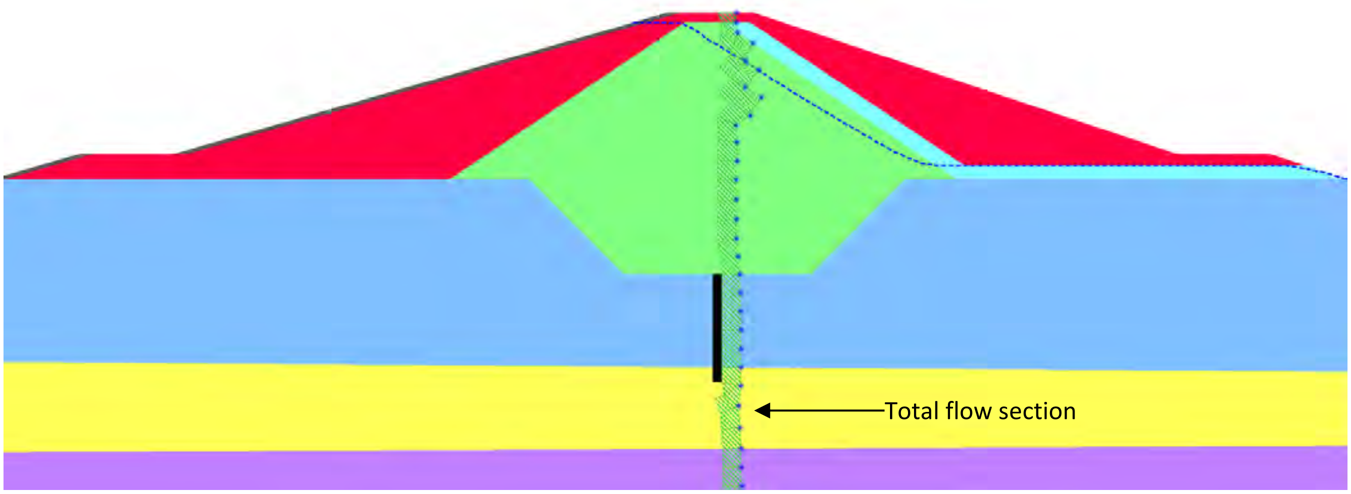

Figure 15. Birch Creek Alternative 2 Combined Flow Through Dam and Foundation = 0.003 ft^3/day/ft

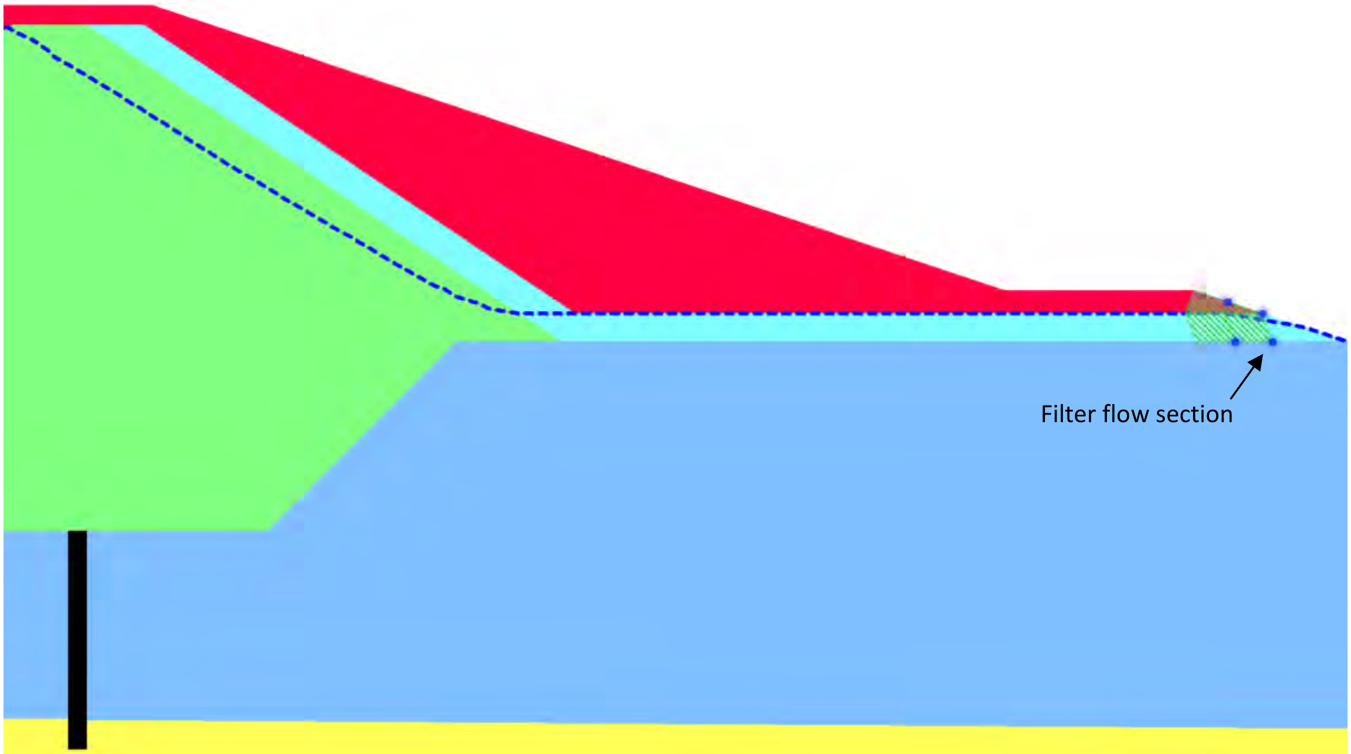

Figure 16. **Birch Creek Alternative 2 Flow Through Filter = 0.001 ft^3/day/ft**

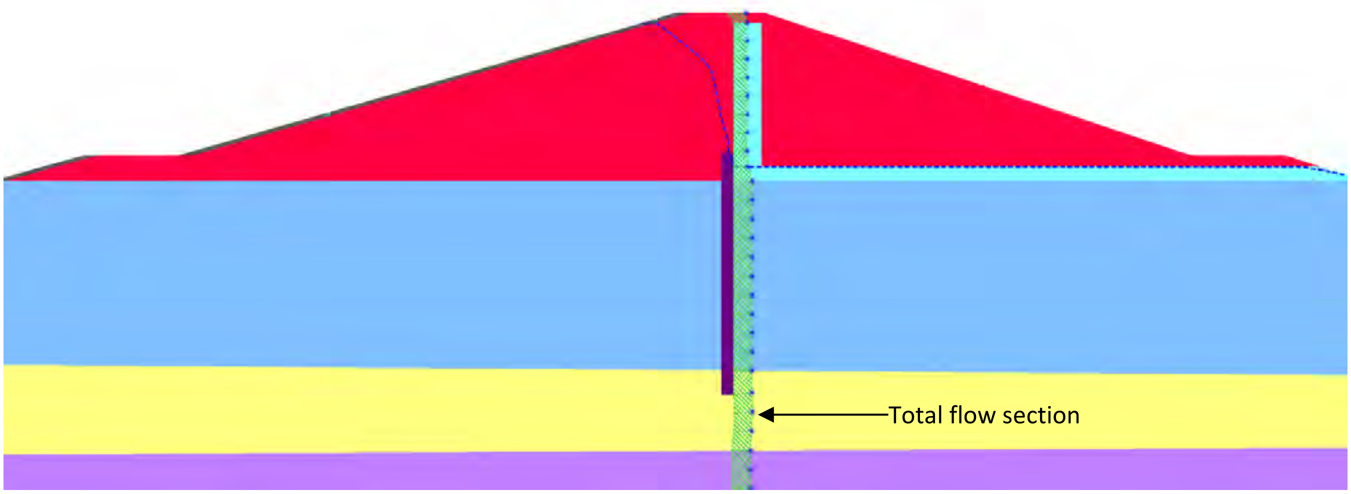

Figure 17. **Birch Creek Alternative 3 Combined Flow Through Dam and Foundation = 0.006 ft^3/day/ft**

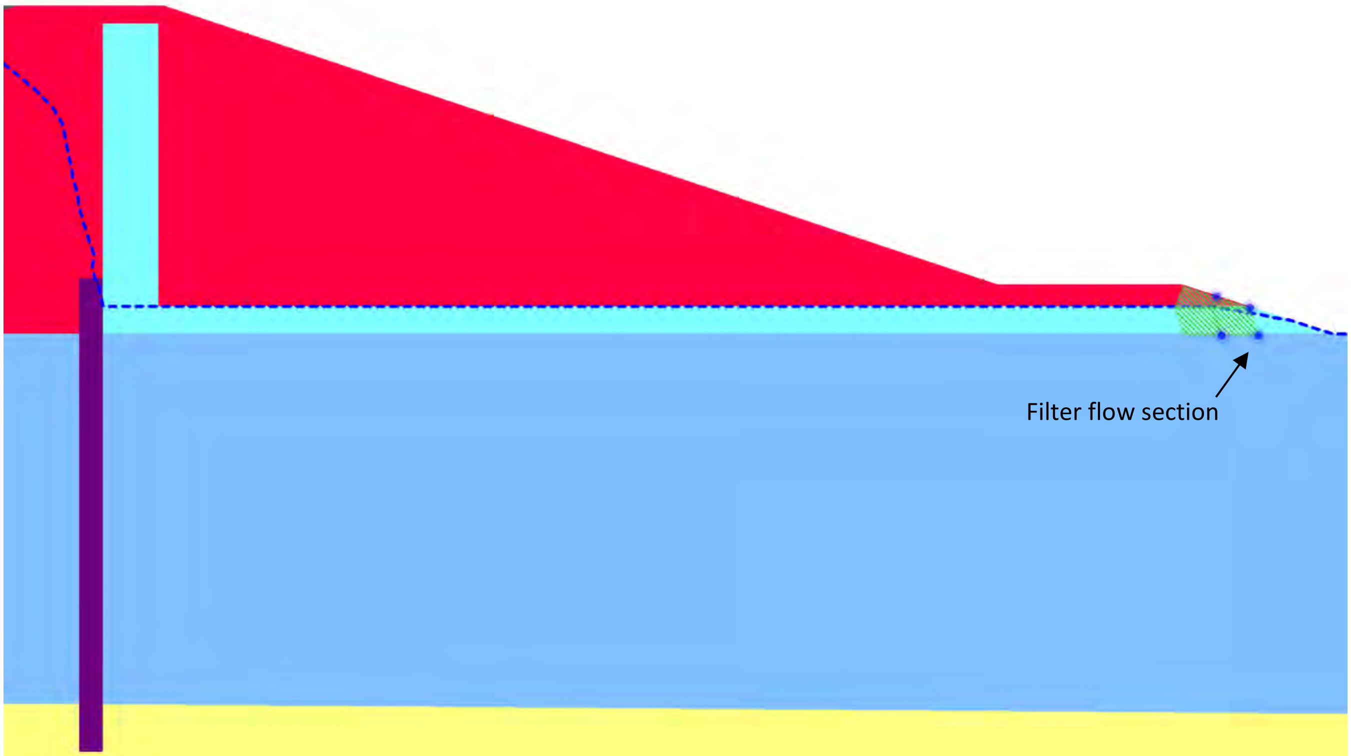

Figure 18. Birch Creek Alternative 3 Flow Through Filter = 0.005 ft^3/day/ft

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.

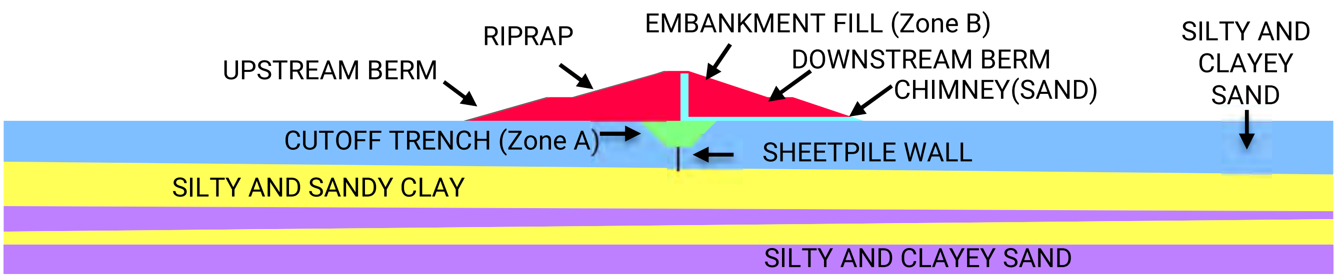

Figure 1. Alternative 1 Embankment Geometry and Zonation— Walnut

Client	SJRA	**Computed By**	P. Turkson
Project	SCW Flood Control Dams **Unit**	**Date**	12/9/2024
Project No.	411500 **File No.**	**Approved By**	David Bentler
Title	Slope Stability Analysis	**Date**	12/9/2024
		Page	5

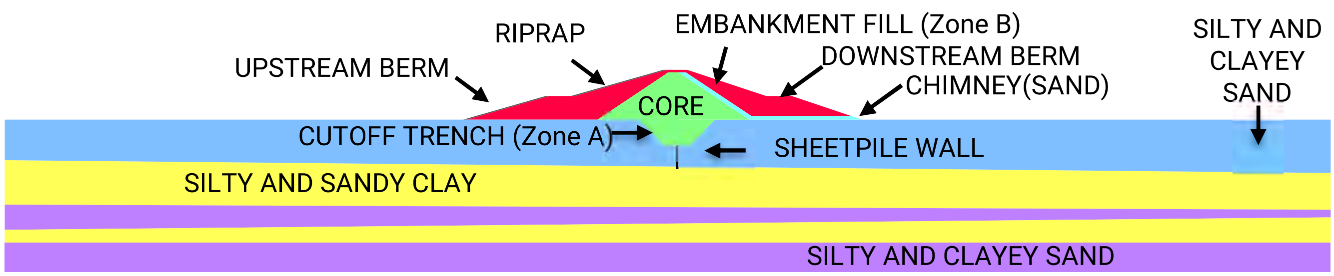

Figure 2. **Alternative 2 Embankment Geometry and Zonation— Walnut**

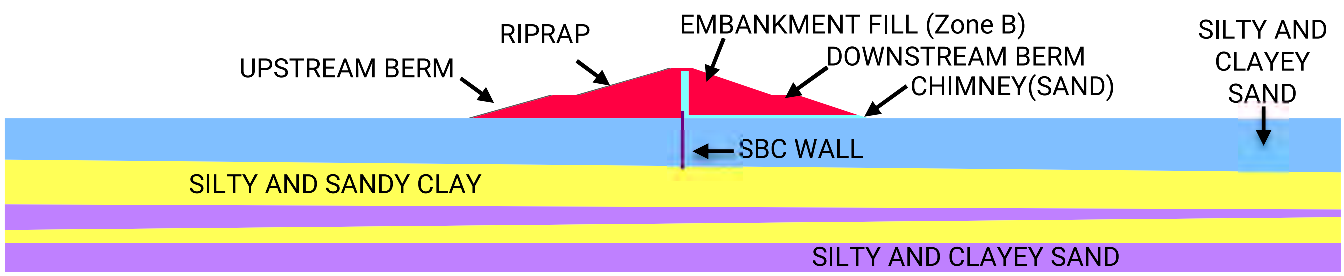

Figure 3. **Alternative 3 Embankment Geometry and Zonation— Walnut**

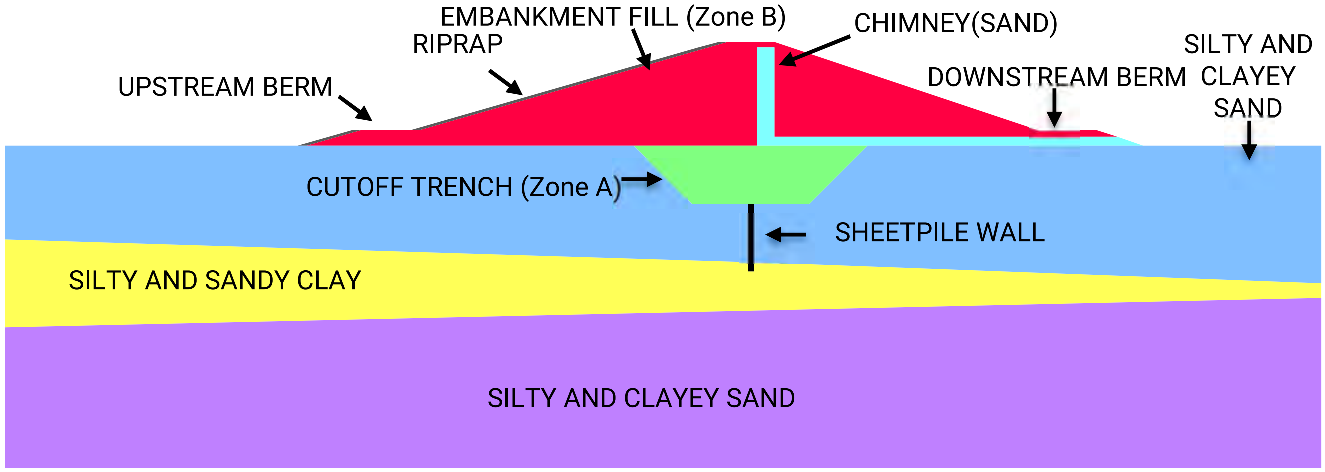

Figure 4. **Alternative 1 Embankment Geometry and Zonation— Birch**

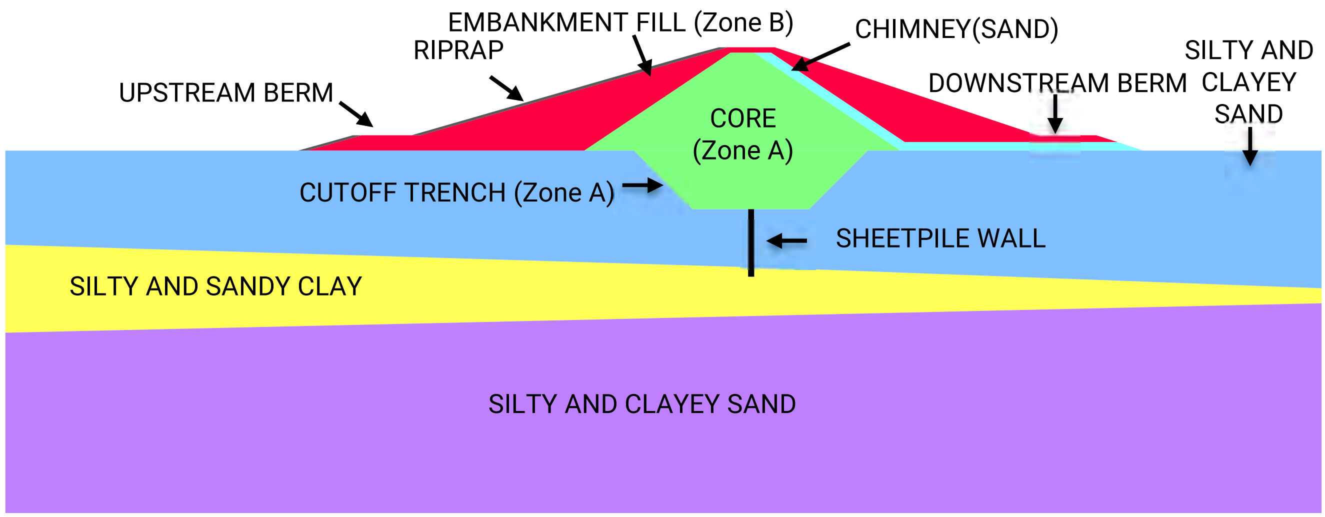

Figure 5. **Alternative 2 Embankment Geometry and Zonation— Birch**

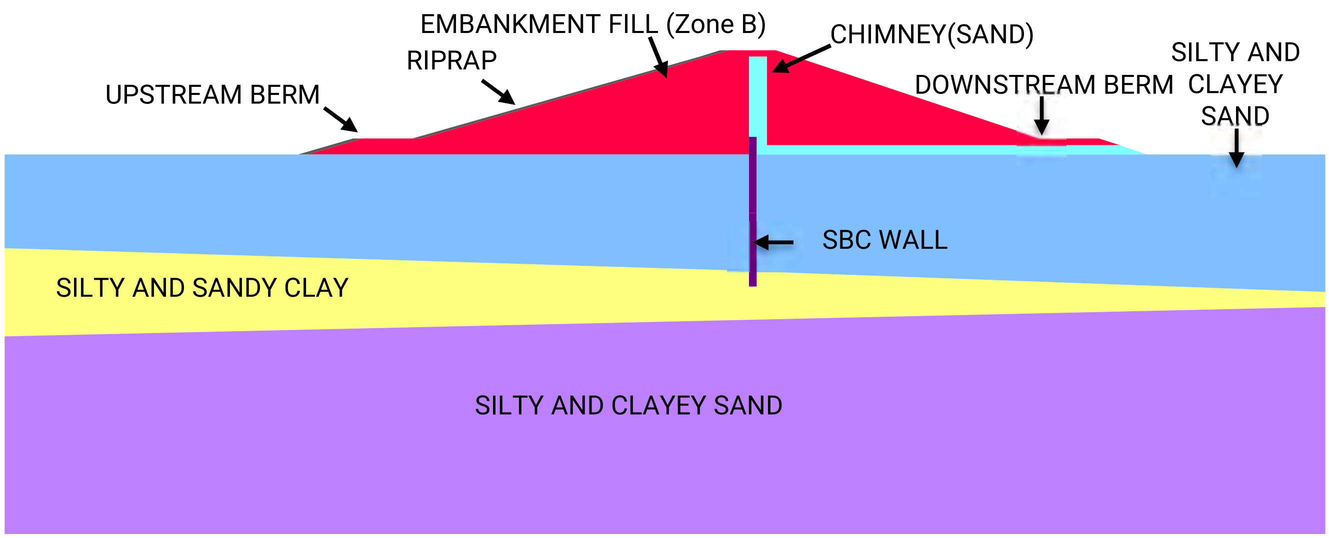

Figure 6. **Alternative 3 Embankment Geometry and Zonation— Birch**

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.

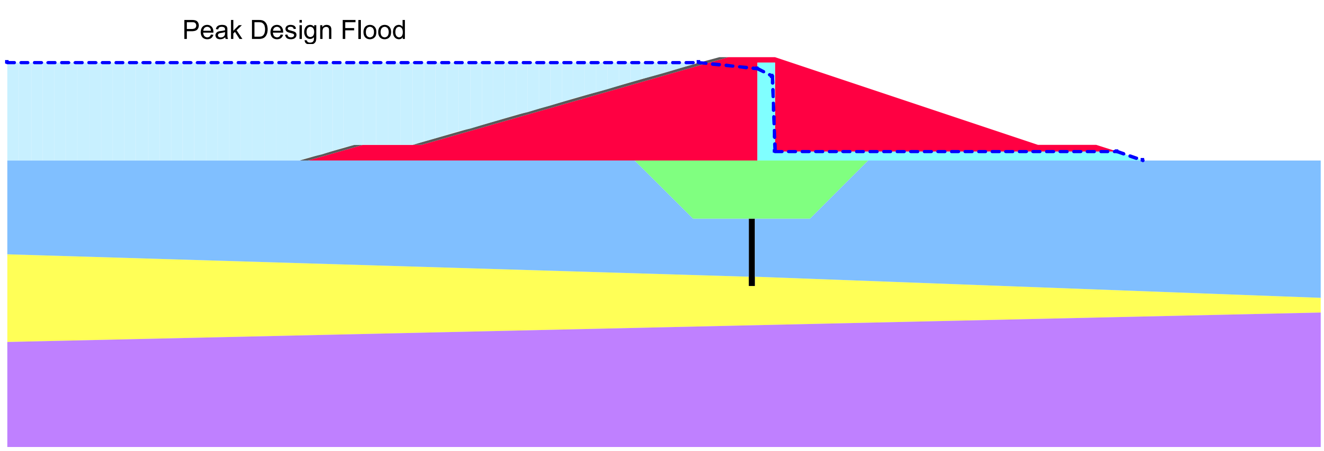

Figure 7. Alternative 1 Typical Phreatic Surface Location Adopted for this Study

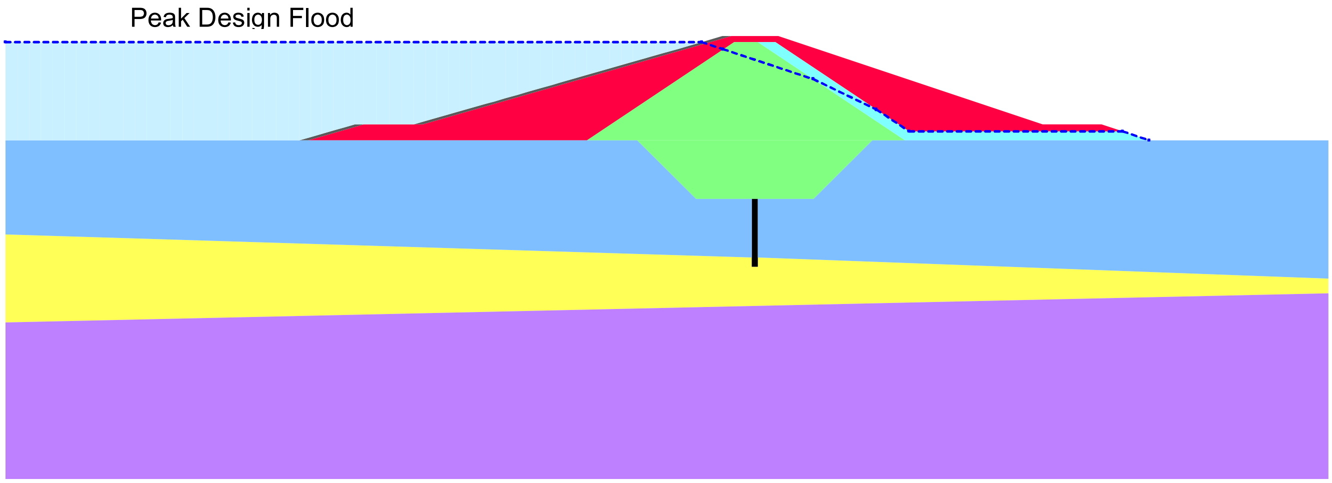

Figure 8. **Alternative 2 Typical Phreatic Surface Location Adopted for this Study**

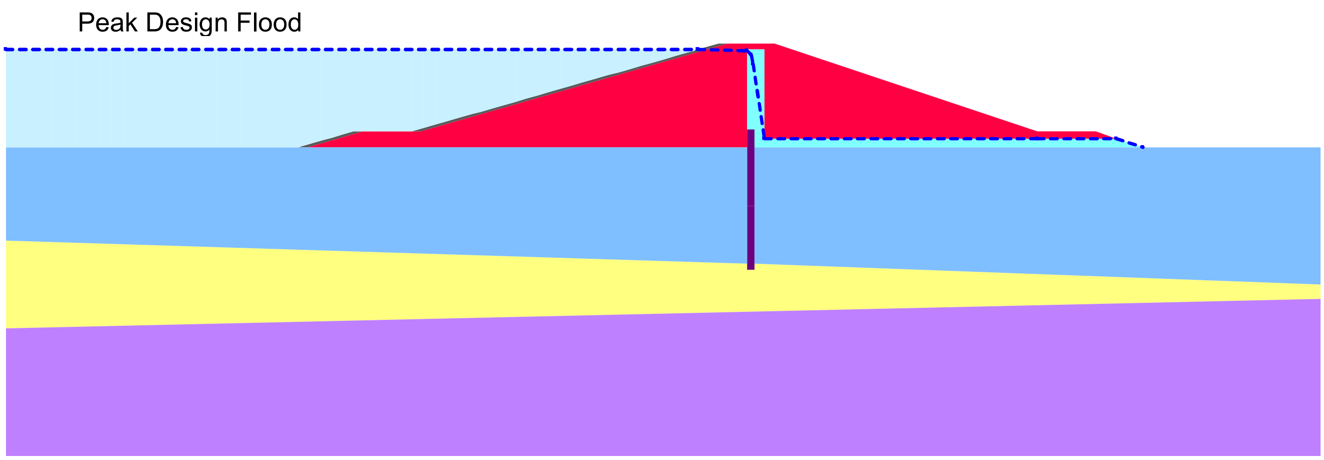

Figure 9. **Alternative 3 Typical Phreatic Surface Location Adopted for this Study**

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			**Computed By**	P. Turkson
Project	SCW Flood Control Dams	**Unit**		**Date**	12/9/2024
Project No.	411500	**File No.**		**Approved By**	
Title	Slope Stability Analysis			**Date**	
				Page	12

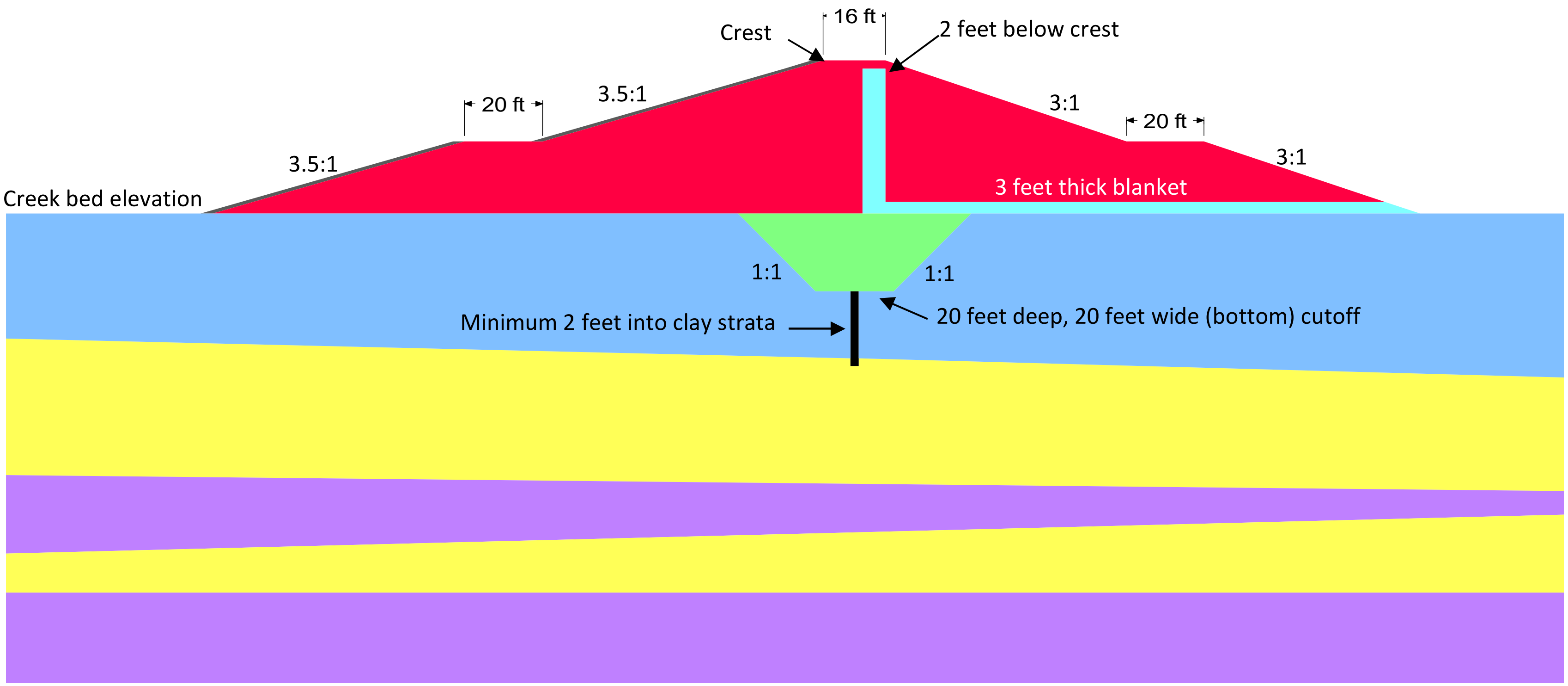

Figure 10 **Alternative 1 Typical Section Used in Slope Stability Analysis**

Client	SJRA			**Computed By**	P. Turkson
Project	SCW Flood Control Dams	**Unit**		**Date**	12/9/2024
Project No.	411500	**File No.**		**Approved By**	
Title	Slope Stability Analysis			**Date**	
				Page	13

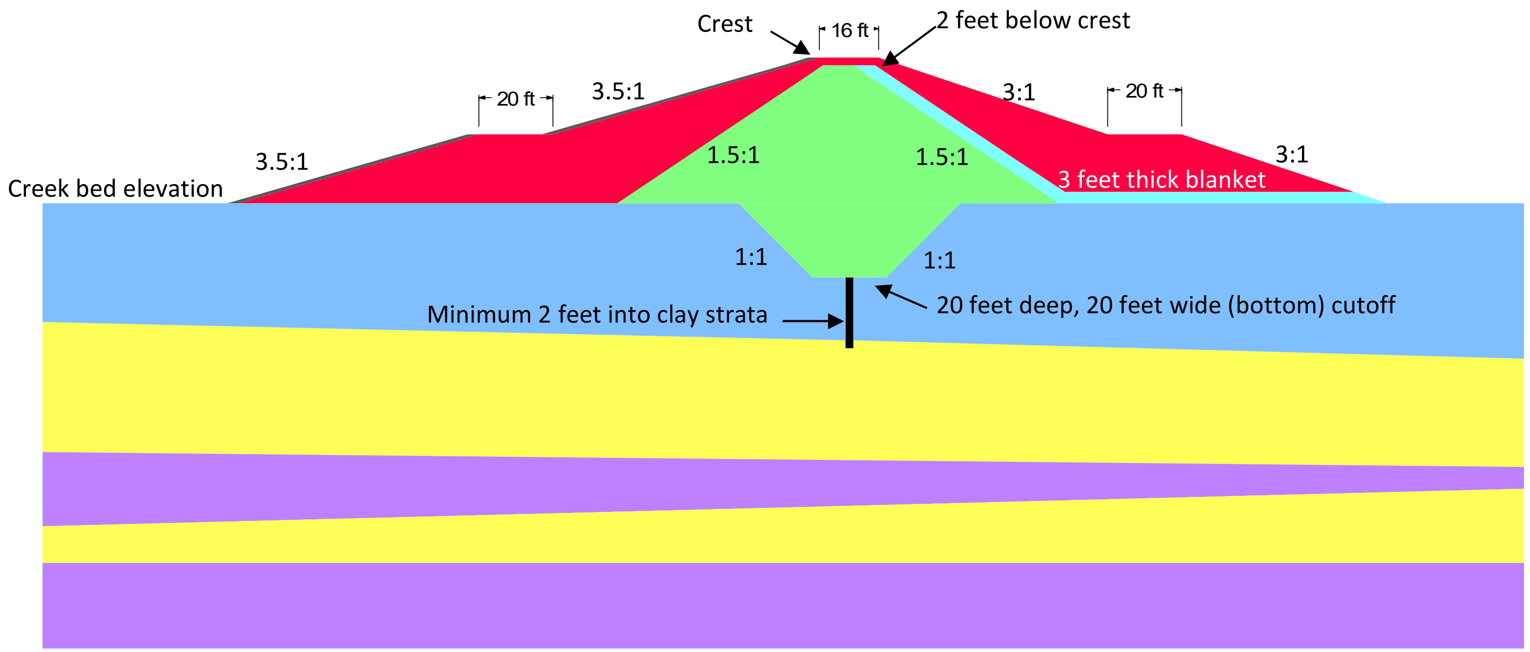

Figure 11 **Alternative 2 Typical Section Used in Slope Stability Analysis**

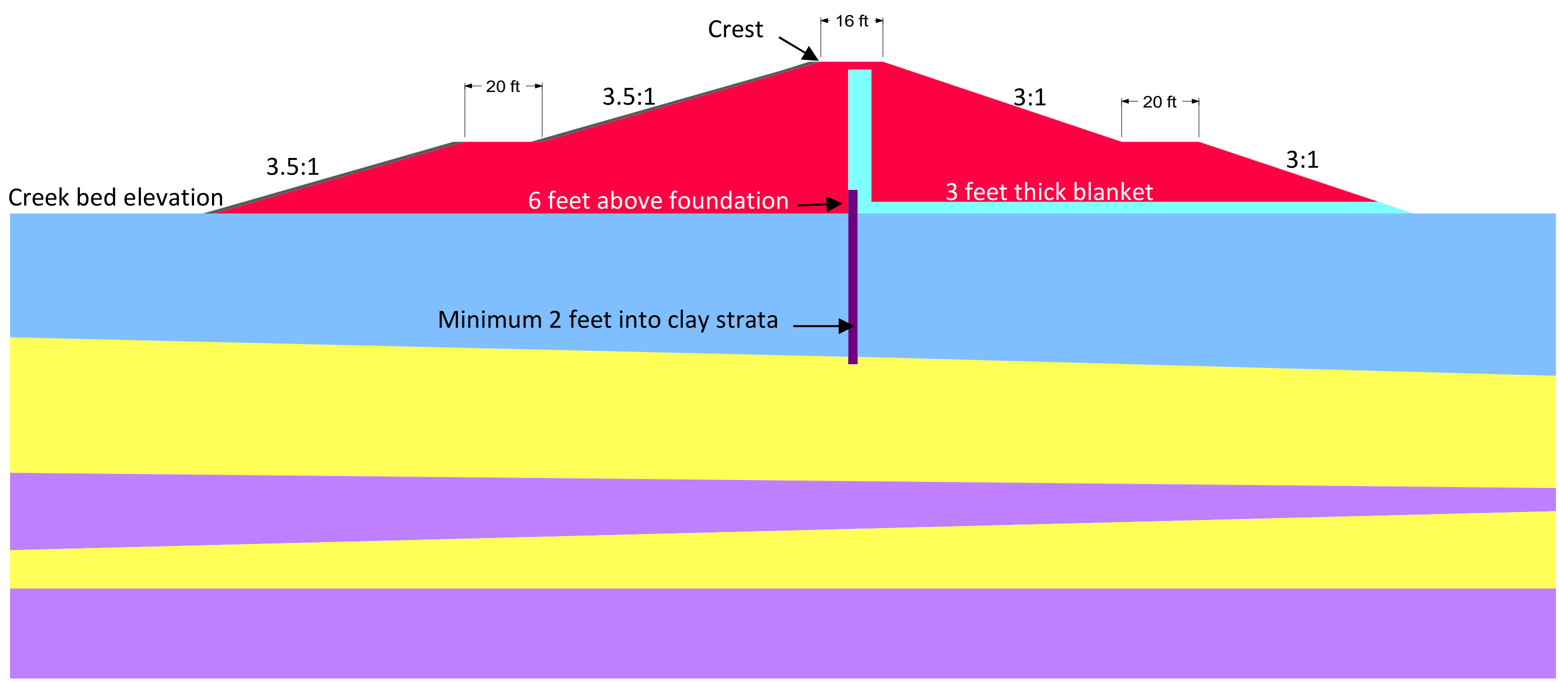

Figure 12. Alternative 3 Typical Section Used in Slope Stability Analysis

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**.

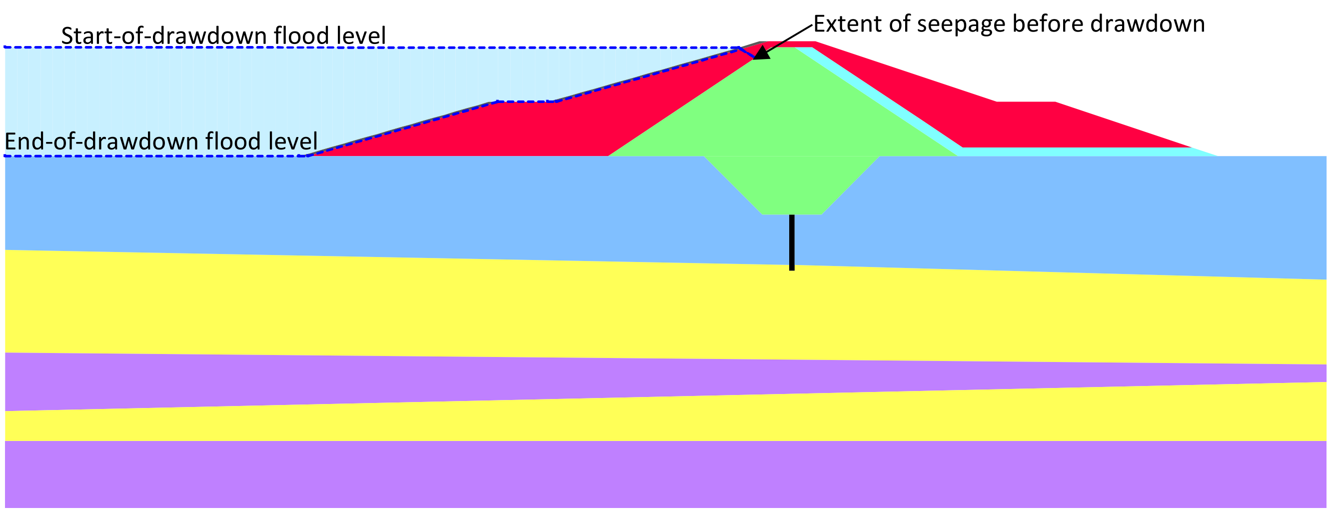

Figure 13. Illustration of Truncated Phreatic Surface for Rapid Drawdown

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

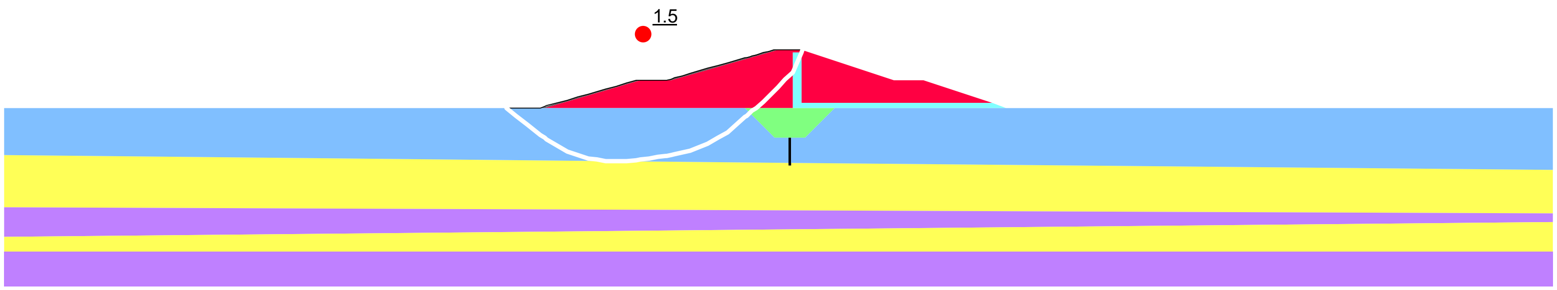

Figure 14 **Alternative 1 End of Construction Upstream Factor of Safety— Walnut**

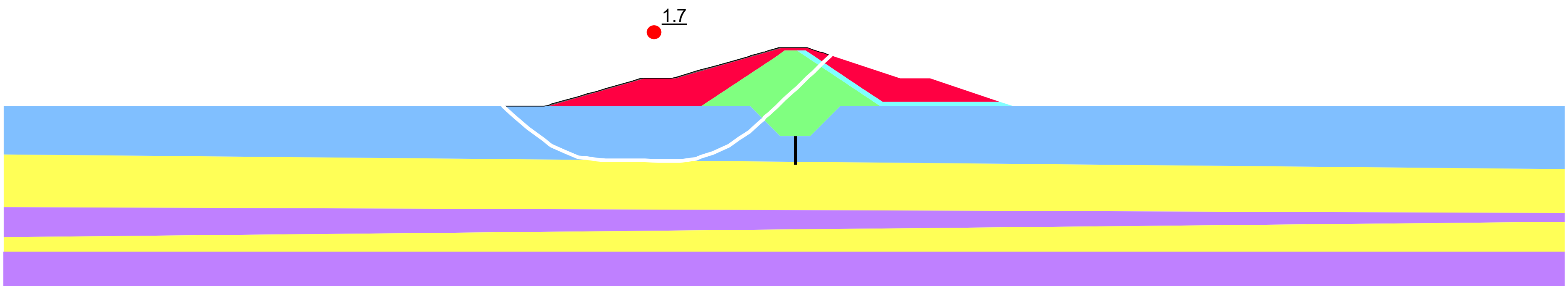

Figure 15 **Alternative 2 End of Construction Upstream Factor of Safety— Walnut**

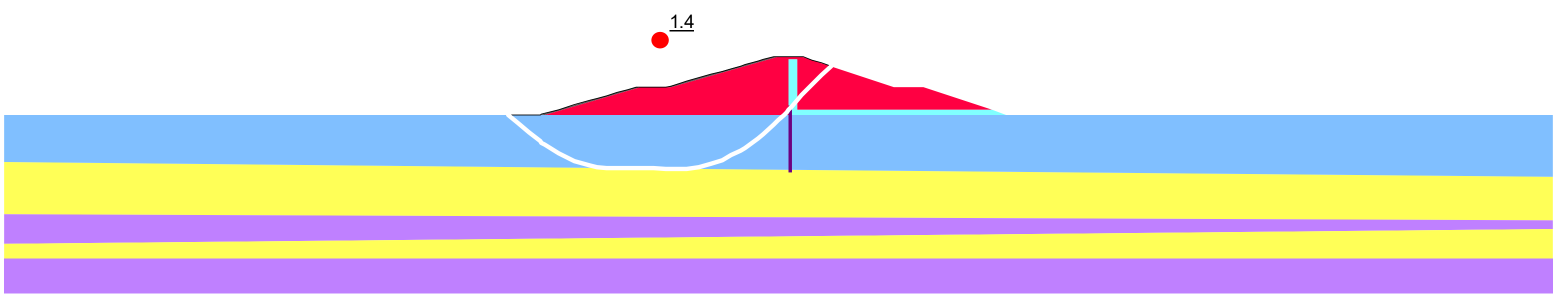

Figure 16 **Alternative 3 End of Construction Upstream Factor of Safety— Walnut**

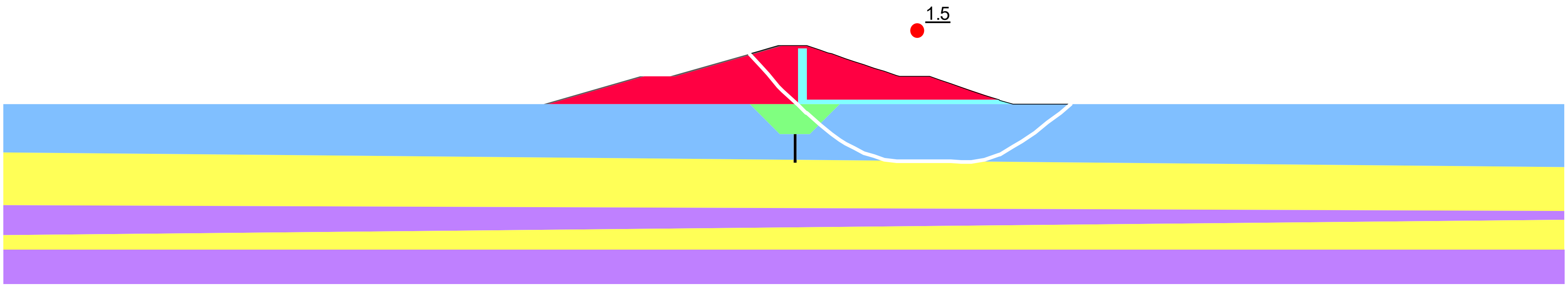

Figure 17 **Alternative 1 End of Construction Downstream Factor of Safety— Walnut**

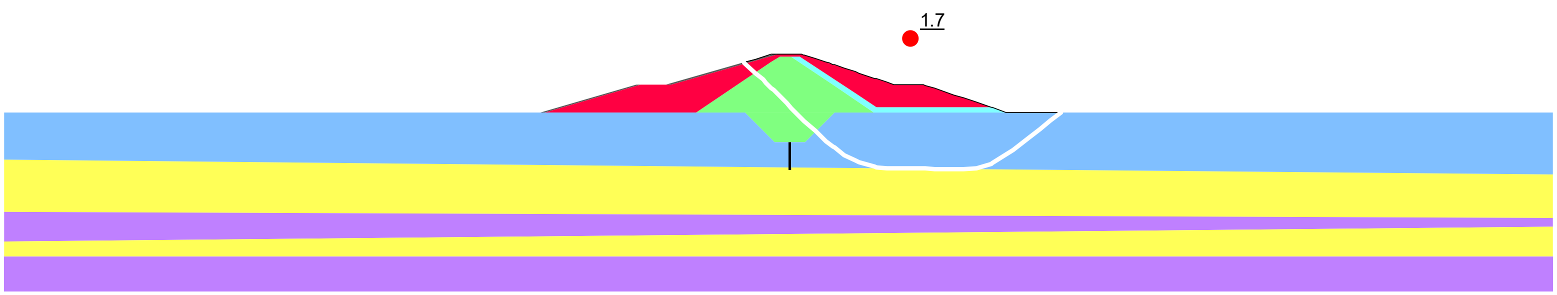

Figure 18 **Alternative 2 End of Construction Downstream Factor of Safety— Walnut**

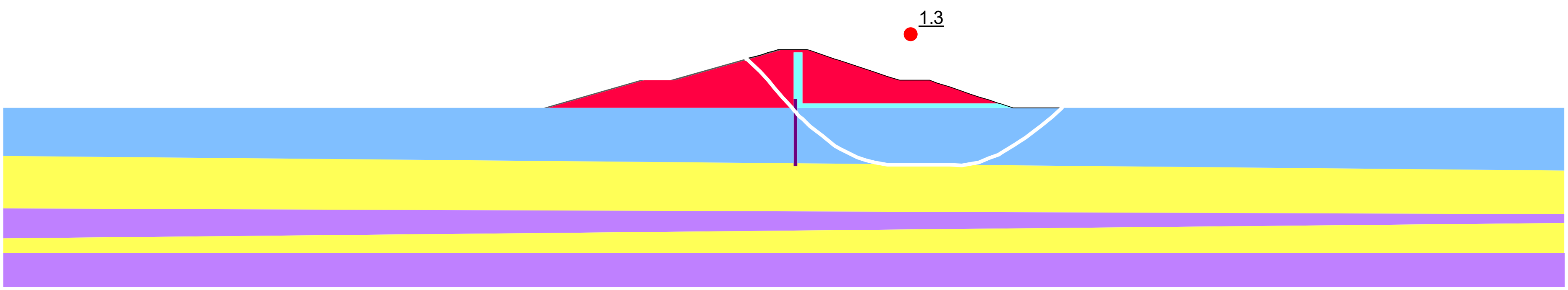

Figure 19 **Alternative 3 End of Construction Downstream Factor of Safety— Walnut**

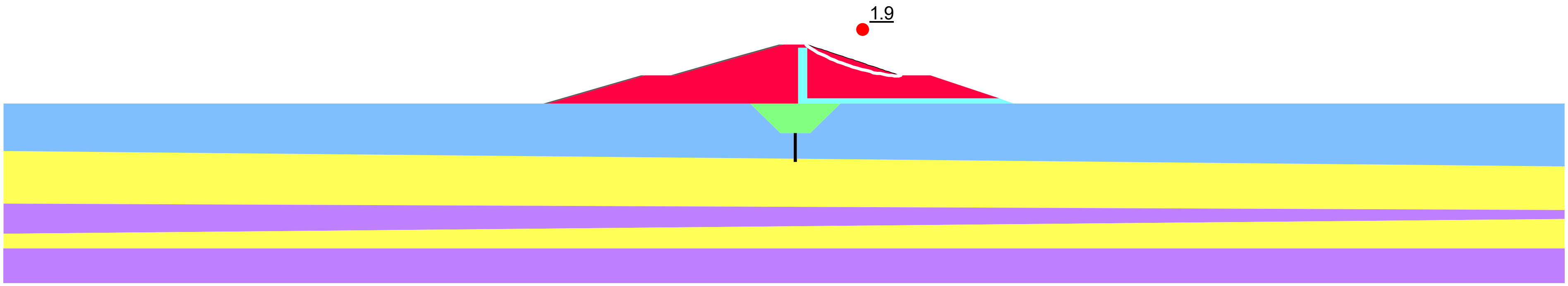

Figure 20 **Alternative 1 100-Year Peak Flood Factor of Safety— Walnut**

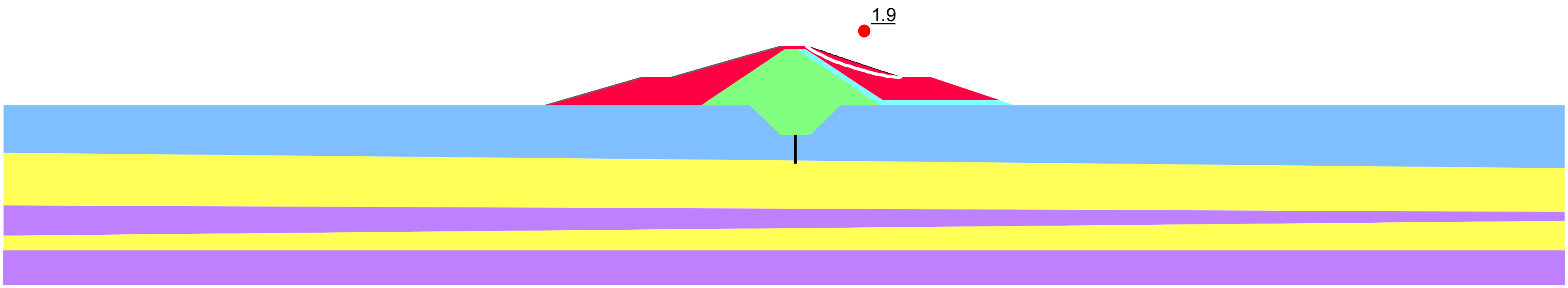

Figure 21 **Alternative 2 100-Year Peak Flood Factor of Safety— Walnut**

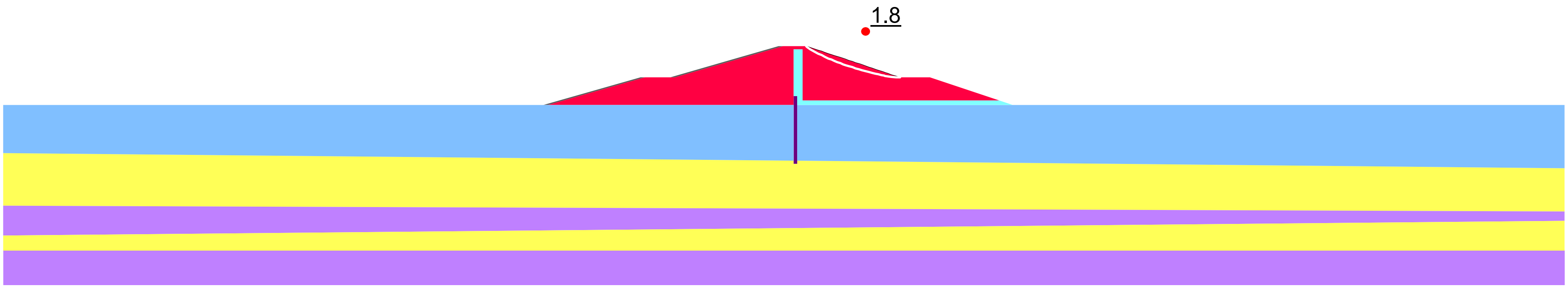

Figure 22 **Alternative 3 100-Year Peak Flood Factor of Safety— Walnut**

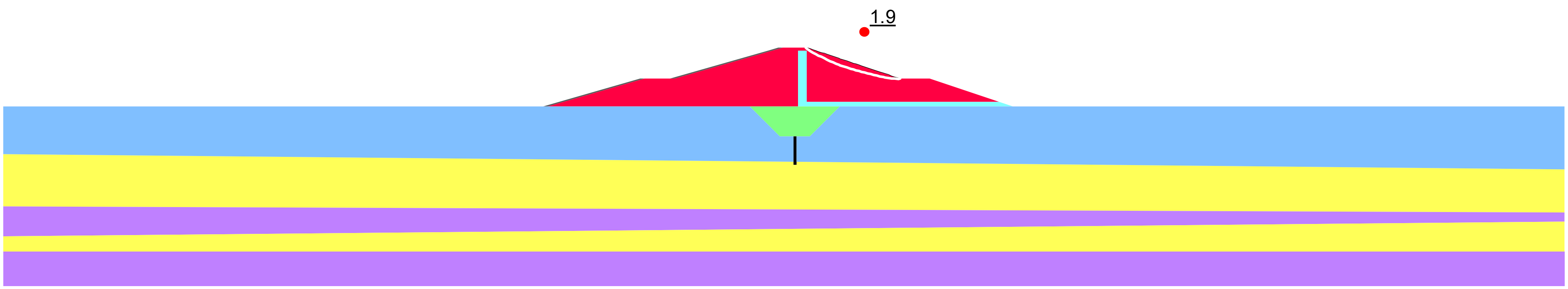

Figure 23 **Alternative 1 Peak Design Flood Factor of Safety— Walnut**

Client	SJRA			**Computed By**	P. Turkson
Project	SCW Flood Control Dams	**Unit**		**Date**	12/9/2024
Project No.	411500	**File No.**		**Approved By**	David Bentler
Title	Slope Stability Analysis			**Date**	12/9/2024
				Page	23

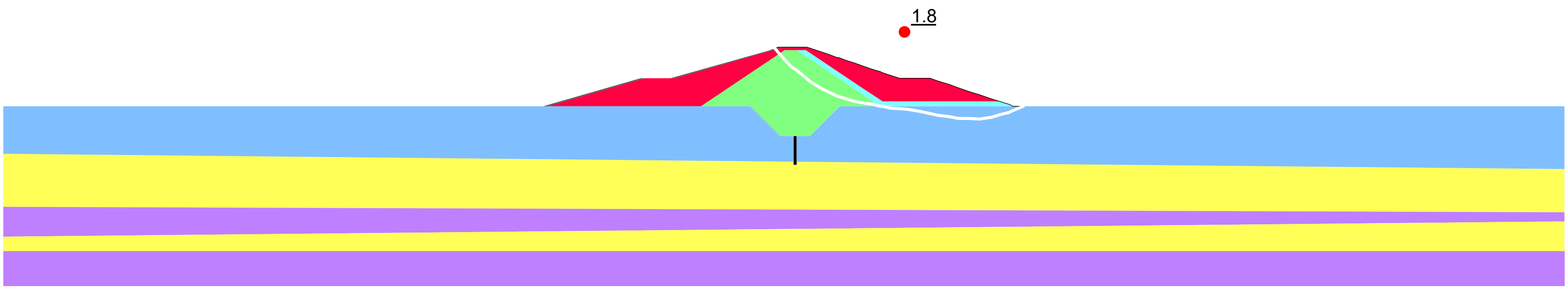

Figure 24 **Alternative 2 Peak Design Flood Factor of Safety— Walnut**

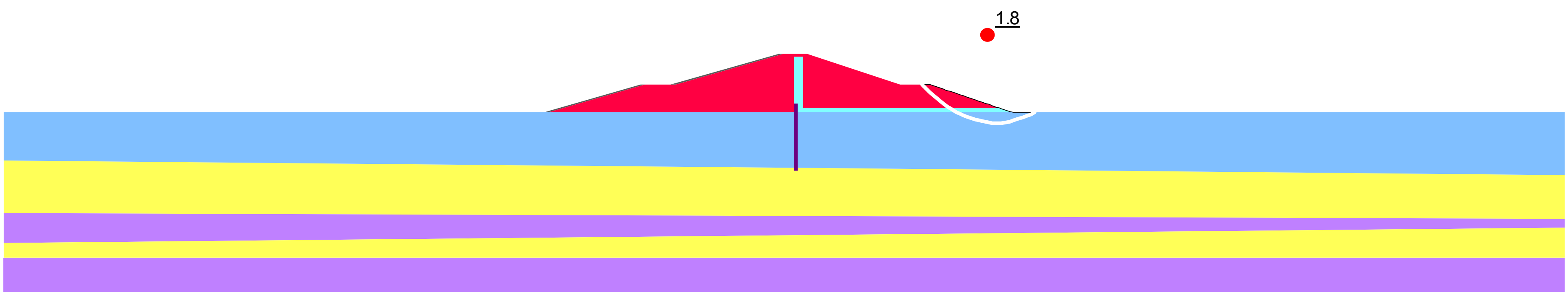

Figure 25 **Alternative 3 Peak Design Flood Factor of Safety— Walnut**

Client	SJRA			**Computed By**	P. Turkson
Project	SCW Flood Control Dams	**Unit**		**Date**	12/9/2024
Project No.	411500	**File No.**		**Approved By**	David Bentler
Title	Slope Stability Analysis			**Date**	12/9/2024
				Page	24

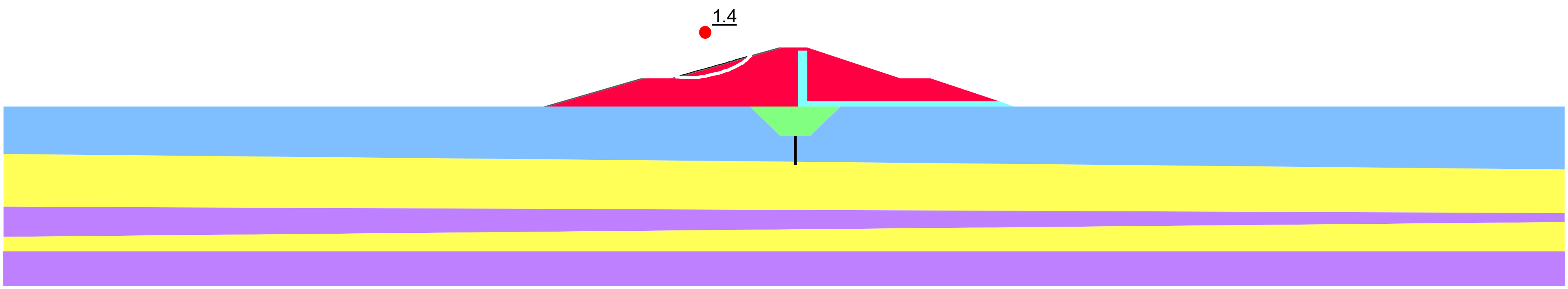

Figure 26 Alternative 1 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Walnut

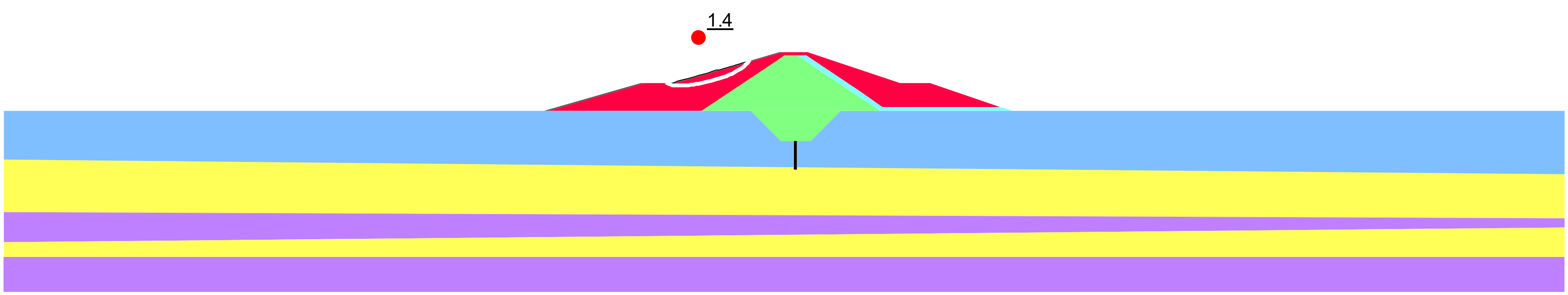

Figure 27 Alternative 2 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Walnut

Client	SJRA			**Computed By**	P. Turkson
Project	SCW Flood Control Dams	**Unit**		**Date**	12/9/2024
Project No.	411500	**File No.**		**Approved By**	David Bentler
Title	Slope Stability Analysis			**Date**	12/9/2024
				Page	25

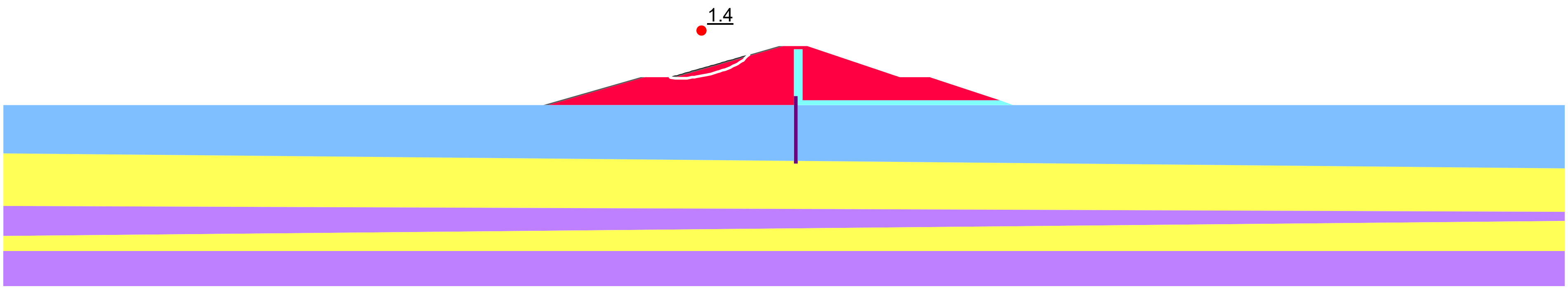

Figure 28 **Alternative 3 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Walnut**

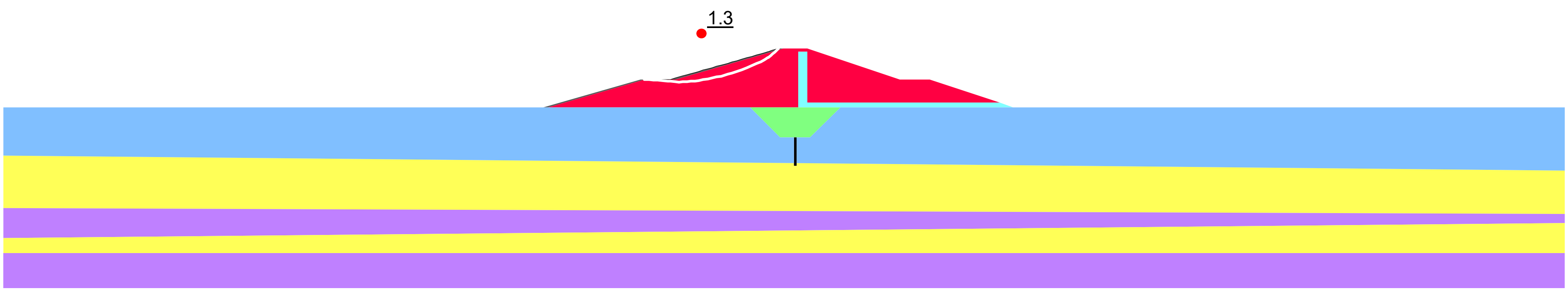

Figure 29 **Alternative 1 Rapid Drawdown from Peak Design Flood Factor of Safety— Walnut**

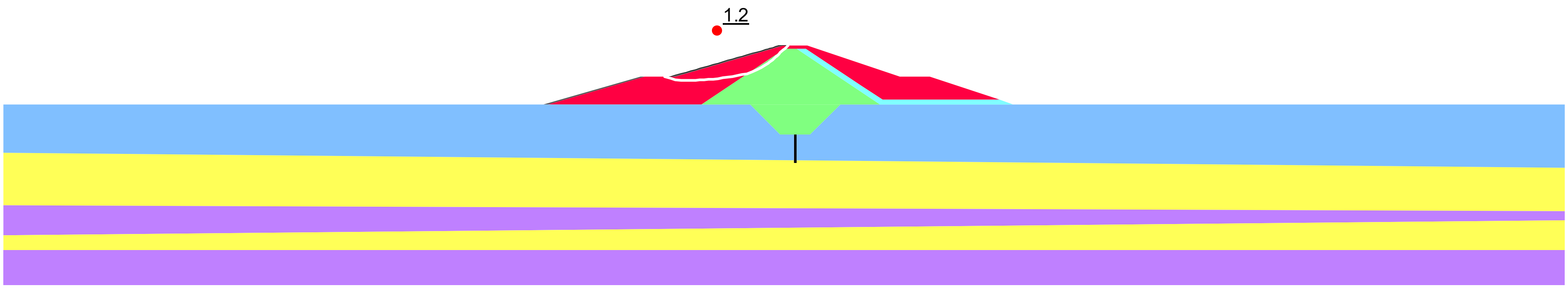

Figure 30 **Alternative 2 Rapid Drawdown from Peak Design Flood Factor of Safety— Walnut**

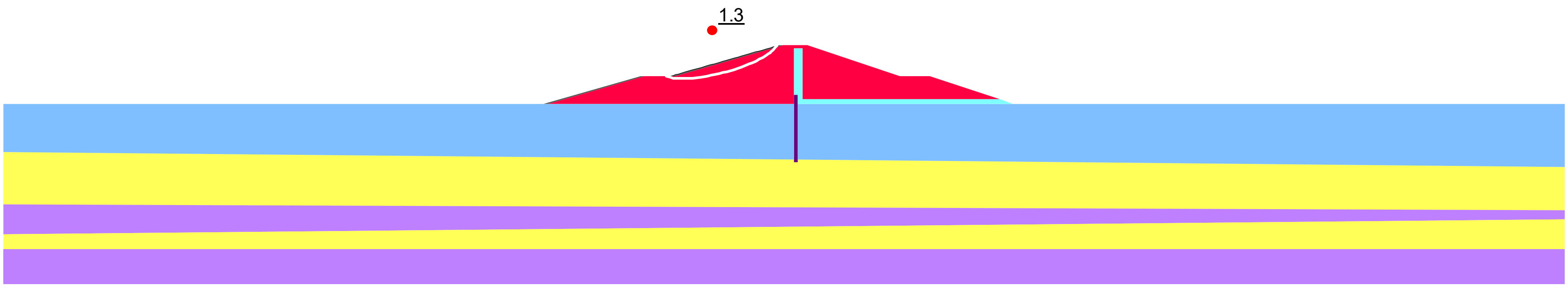

Figure 31 **Alternative 3 Rapid Drawdown from Peak Design Flood Factor of Safety— Walnut**

Attachment 2:

Critical Slip Surfaces from Static Stability Analysis— Birch

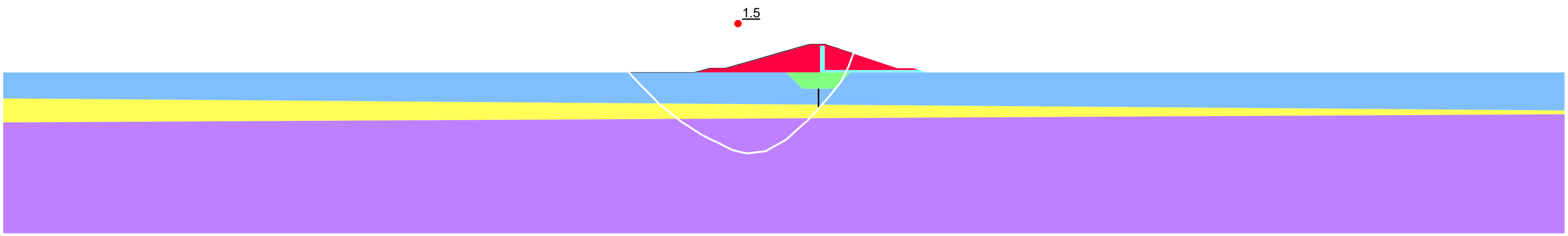

Figure 32 **Alternative 1 End of Construction Upstream Factor of Safety— Birch**

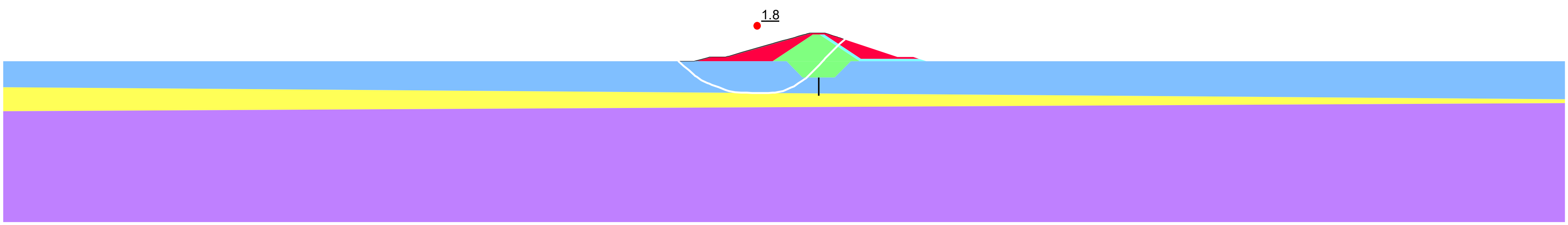

Figure 33 **Alternative 2 End of Construction Upstream Factor of Safety— Birch**

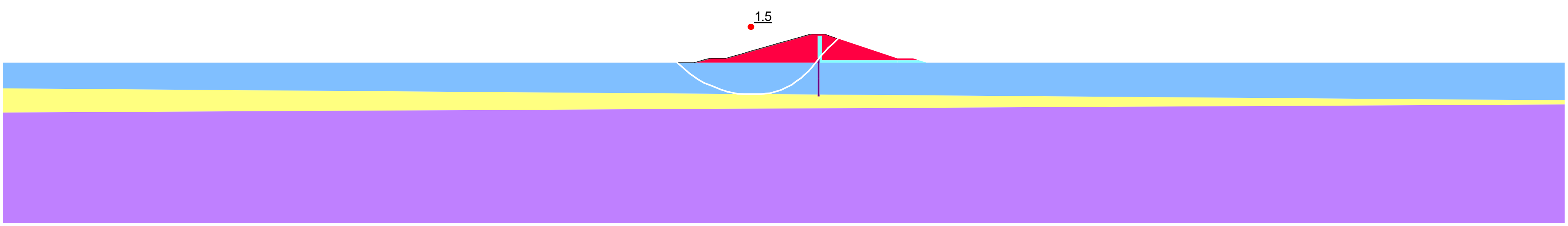

Figure 34 **Alternative 3 End of Construction Upstream Factor of Safety— Birch**

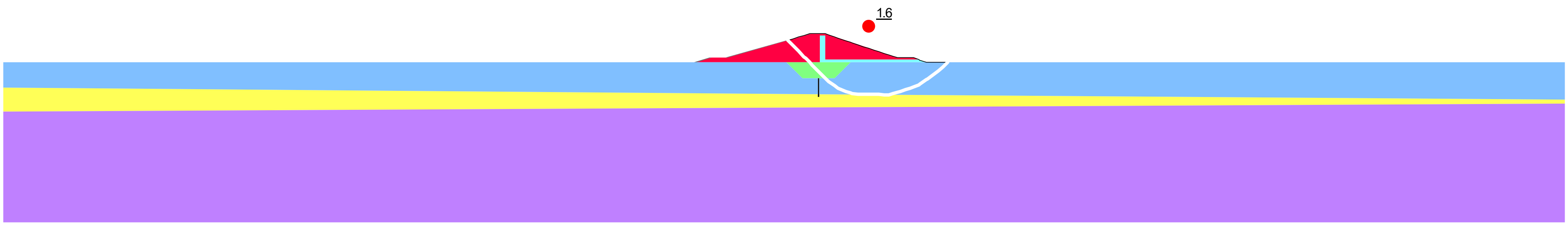

Figure 35 **Alternative 1 End of Construction Downstream Factor of Safety— Birch**

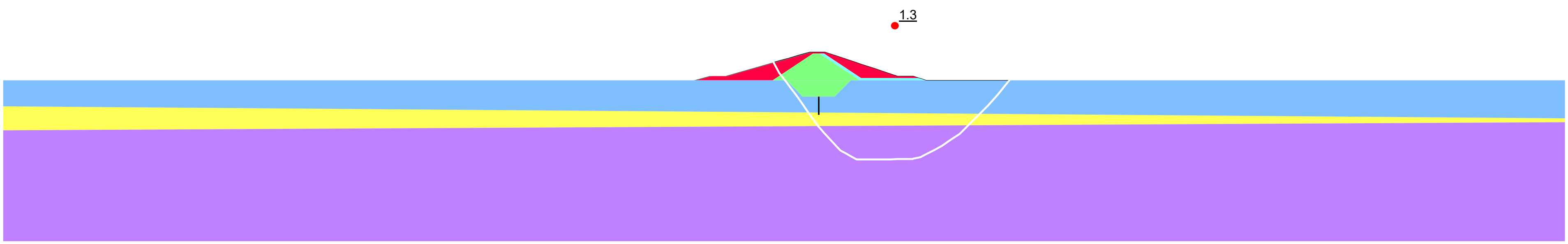

Figure 36 **Alternative 2 End of Construction Downstream Factor of Safety— Birch**

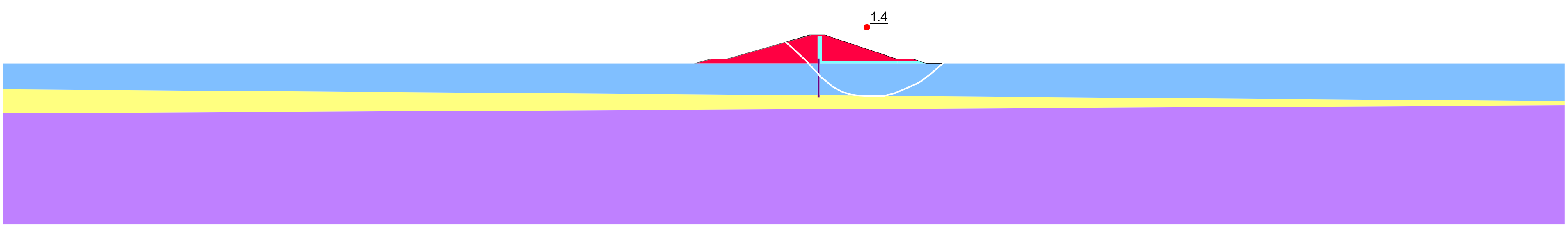

Figure 37 **Alternative 3 End of Construction Downstream Factor of Safety— Birch**

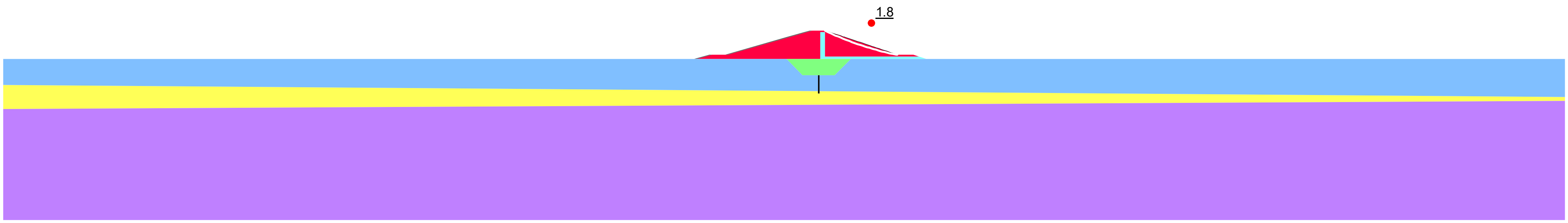

Figure 38 Alternative 1 100-Year Peak Flood Factor of Safety— Birch

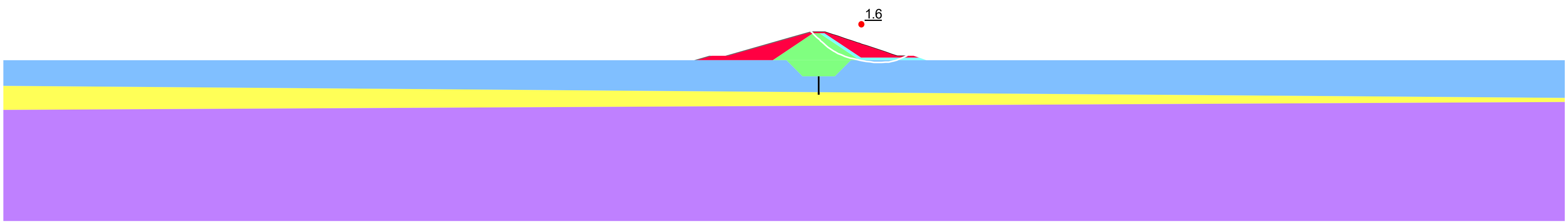

Figure 39 Alternative 2 100-Year Peak Flood Factor of Safety— Birch

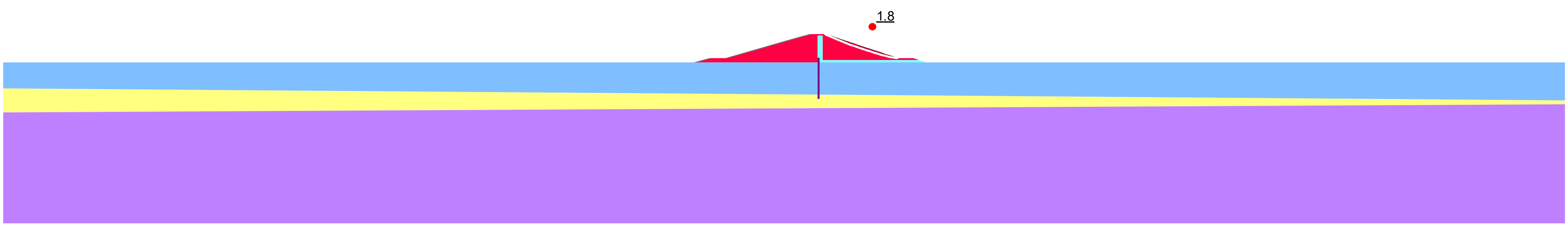

Figure 40 Alternative 3 100-Year Peak Flood Factor of Safety— Birch

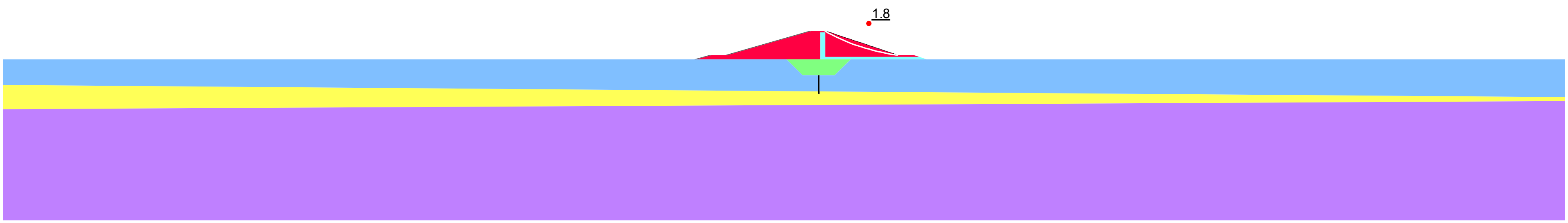

Figure 41 **Alternative 1 Peak Design Flood Factor of Safety— Birch**

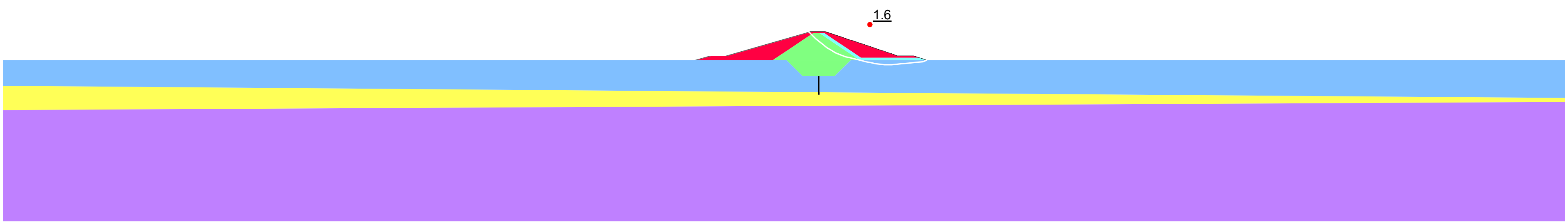

Figure 42 **Alternative 2 Peak Design Flood Factor of Safety— Birch**

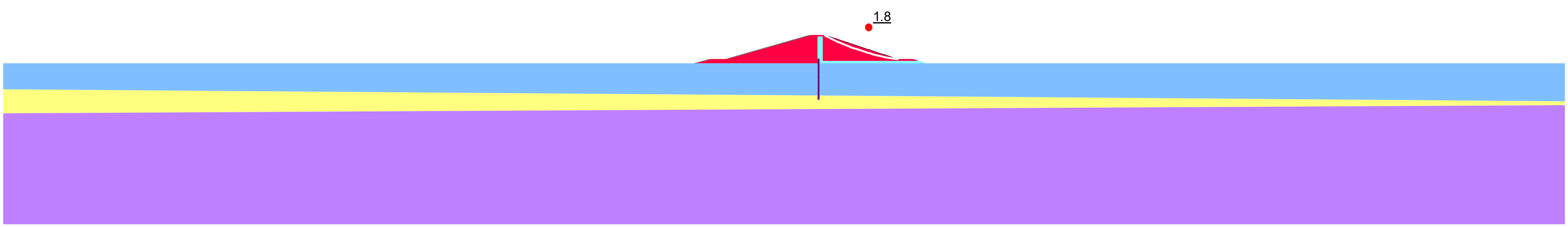

Figure 43 **Alternative 3 Peak Design Flood Factor of Safety— Birch**

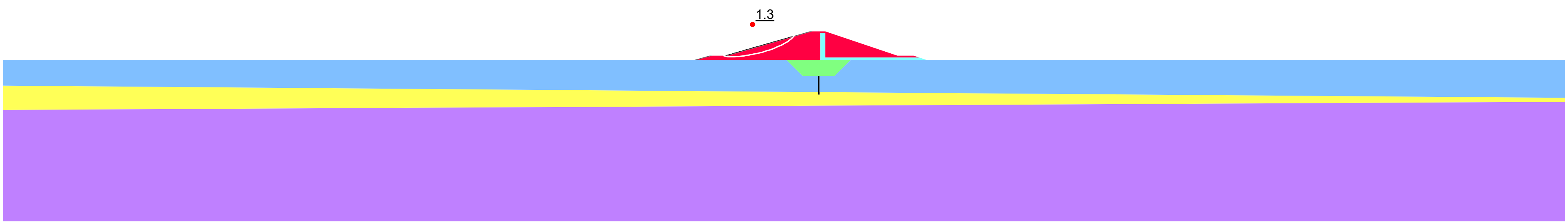

Figure 44 Alternative 1 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Birch

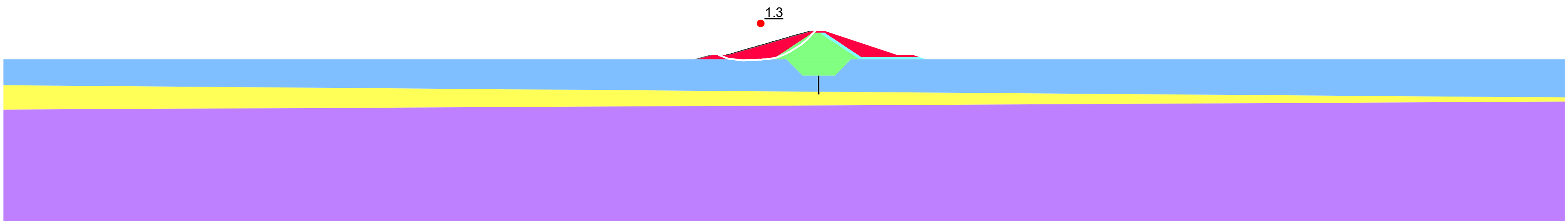

Figure 45 Alternative 2 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Birch

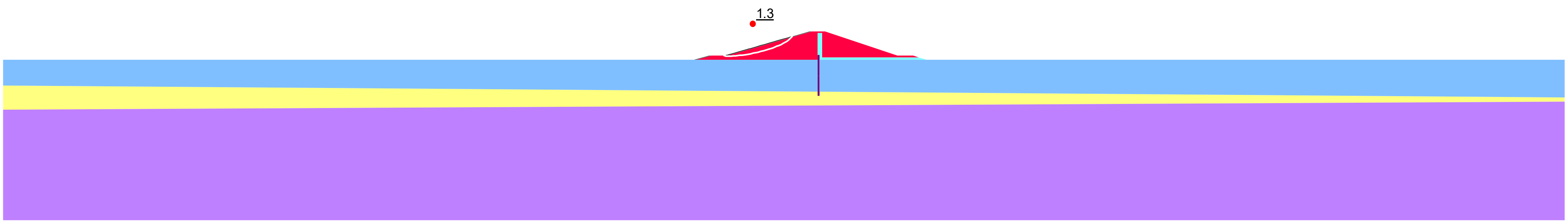

Figure 46 Alternative 3 Rapid Drawdown from 100-Year Peak Flood Factor of Safety— Birch

Client	SJRA			**Computed By**	P. Turkson
Project	SCW Flood Control Dams	**Unit**		**Date**	12/9/2024
Project No.	411500	**File No.**		**Approved By**	David Bentler
Title	Slope Stability Analysis			**Date**	12/9/2024
				Page	33

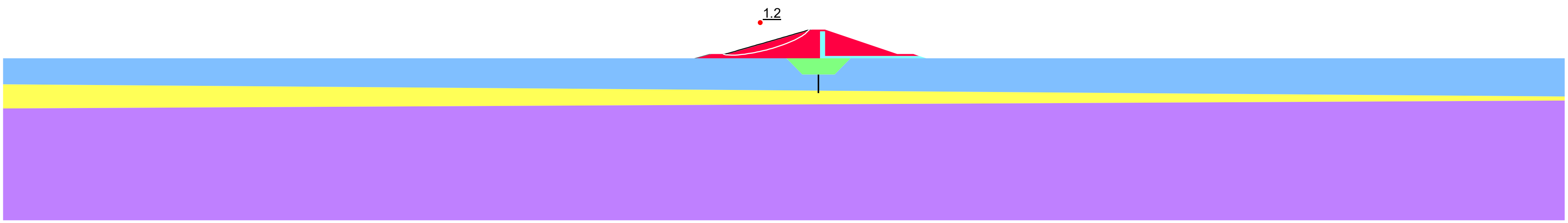

Figure 47 Alternative 1 Rapid Drawdown from Peak Design Flood Factor of Safety— Birch

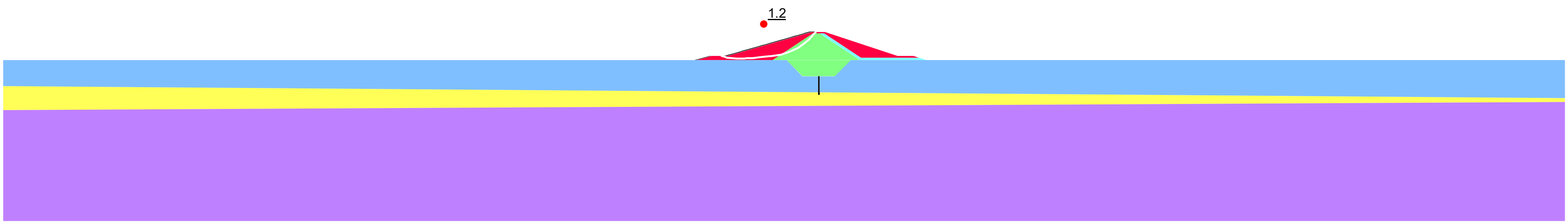

Figure 48 Alternative 2 Rapid Drawdown from Peak Design Flood Factor of Safety— Birch

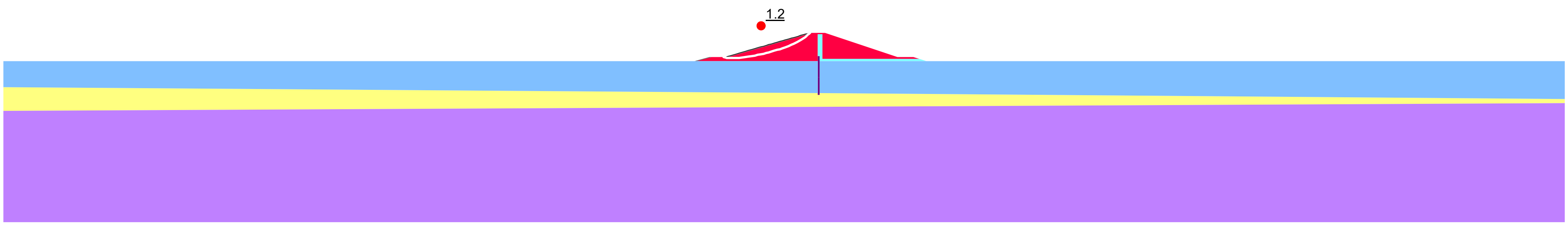

Figure 49 Alternative 3 Rapid Drawdown from Peak Design Flood Factor of Safety— Birch

Appendix B-6 Plans and profiles

Appendix B-7 Foundation treatment modification

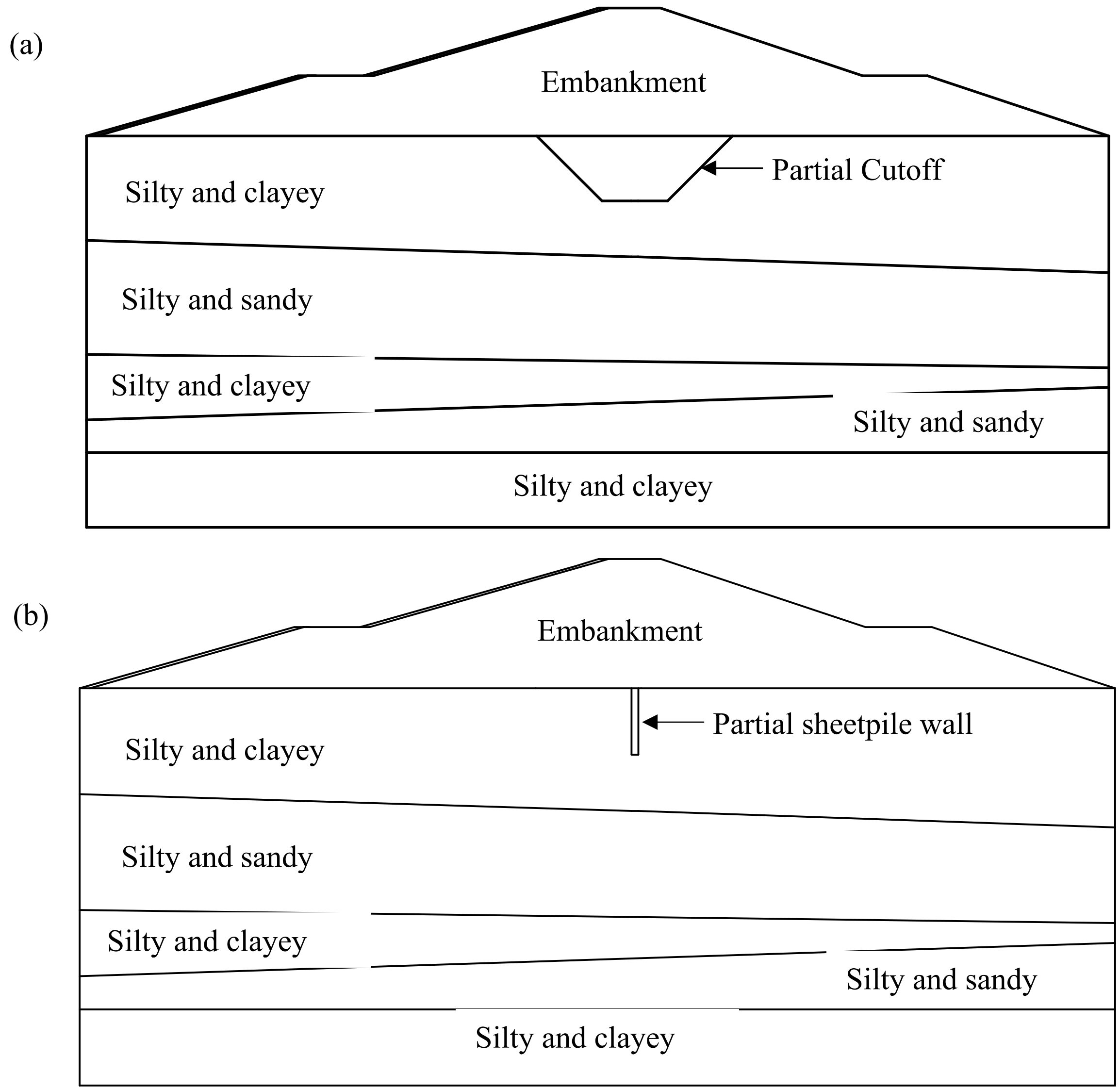

Figure G-1 Cutoff Trench with Sheet Pile Wall Foundation Modification: (a) Partial Cutoff Trench and (b) Partial Sheetpile Wall

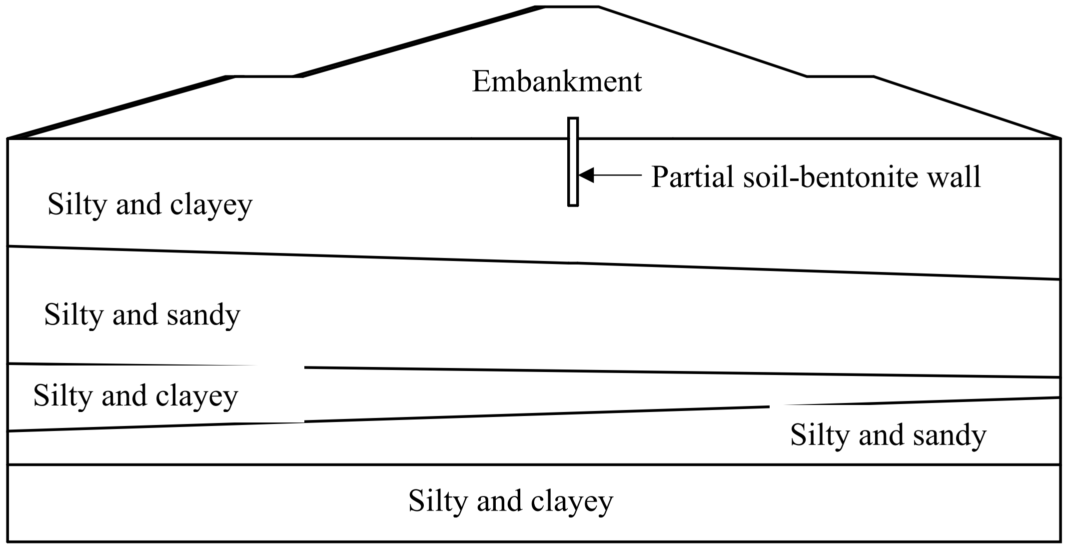

Figure G-2 Soil-Bentonite Cutoff Wall Foundation Modification

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