Walnut Springs Spillway and Bank Stabilization Repair

Project Planning Memorandum

Seguin, Texas
June 6, 2024

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1 Overview

This memorandum outlines the preliminary findings and recommendations developed by HDR Engineering Inc. (HDR) for recent failures of existing bank stabilization projects at the Walnut Springs Park and just downstream of W. Nolte St. along the east bank across from the public library in Seguin, Texas.

The height of the high eastern bank of Walnut Branch varies from approximately 20 to 30 feet above the channel bottom elevation. The bank is composed primarily of clay that is susceptible to failure due to its own overburden pressure. Failures of existing bank stabilization measures at Walnut Spring Park are likely due to a combination of erosion and flanking, existing bank instability, and slope toe degradation and failure. The east bank downstream of W. Nolte St. is similar in clay composition and has a low existing factor of safety for slope stability based on the limited data available. Only the upstream end of the engineered limestone block wall has failed likely due to flanking by overbank flows and toe scour.

At varying depths below the clay layer that makes up the banks above the channel bottom is a clayshale layer that has greater structural integrity. To raise the expected factor of safety of the banks at this location to recommended design standards, will require structural solutions that integrate with the clayshale layer.

This memorandum provides preliminary analysis of two repair alternatives at Walnut Springs Park (i.e. Site 1), and two repair alternatives south of W. Nolte St. (i.e. Site 2) For each site, there is one limited repair option, and one complete repair option that will bring the expected factor of safety to recommended design standards.

2 Project Background

The City of Seguin (City) retained HDR to provide project planning services for the repair and stabilization of the bed and banks along Walnut Branch at two sites shown in Figure 2-1 Project Limits. Site 1 is in Walnut Springs Park between the existing pedestrian bridge and W. Nolte Street. Site 2 begins at N. Nolte Street and ends near W. Washington Street directly across from the public library.

Figure 2-1 Project Limits

2.1 Historical Improvements

Two prior infrastructure projects were completed in the last 15 years within the Site 1 and Site 2 footprints. For Site 1, the City provided 2009 construction plans for Walnut Branch Linear Park (linear park or Walnut Springs Park) which documents proposed park infrastructure north of W. Nolte Street. No other engineering data or reports were available for this linear park project. For Site 2, HDR obtained documentation from the 2016 U.S. Army Corps of Engineers (USACE) Walnut Branch Ecosystem Restoration Project which in part included a doweled limestone block wall along the east bank downstream of W. Nolte St. HDR understands that during or after construction of the USACE project at Site 2, a flood damaged the east bank improvements in the Site 1 linear park. Based on visual observations, it appears the repair contractor used the same detail for the repair of the existing Site 1 east limestone block wall as was used in the original at Site 2 wall construction.

2.2 Project Scope

This planning project had two phases; the objective of the first phase of the planning services was to complete a visual assessment in the field and provide an initial qualitative opinion of potential risk for additional wall and embankment failure. The work product of the first project phase is provided in Appendix B.

The second phase was to develop feasibility engineering concepts to address observed erosion and existing embankment failures and provide opinion of probable construction costs and permitting constraint summaries. This report documents the development of phase two work products.

3 Data Collection

3.1 Record Documents and Open-Source Data

The existing project documentation for Site 1 and Site 2 and the surrounding is limited. HDR obtained limited data primarily from the City, USACE, and other consultants. A table summary of collected reports and documents are provided in the Appendix A. Files are provided digitally.

3.2 Survey

As a subconsultant to HDR, Maestas & Associates, LLC (Maestas) completed a limited topographic survey of Site 1 and 2, provide in Appendix C. Topographic survey data was collected using a static scanner. Limited spot elevations and planimetric data were collected since this planning study prioritized terrain data to support the preliminary geotechnical evaluations. Because the topographic survey was limited in scope, a composite terrain was created by supplementing it with USGS 2019 Hurricane LiDAR data (70cm resolution) obtained from TxGIO DataHub.

3.3 Geotechnical Borings

Three independent geotechnical investigations were conducted in the vicinity of the project area for three different projects and clients. The three studies were performed by Terracon Consultants, Inc. (Terracon), Rock Engineering & Testing Laboratory, Inc. (RETL), and Professional Service Industries, Inc. (PSI). The three studies are summarized herein.

Terracon (2023) Geotechnical Data Investigation

As a subconsultant to HDR, Terracon completed a preliminary field investigation and a laboratory testing program (Terracon Project No. 90235129R – Geotechnical Data Report (GDR) dated December 7, 2023). The field investigation included two borings on the west side of Walnut Branch Creek. The borings were completed on August 31st and September 1st, 2023. Boring B-1 was drilled near Site 1 and boring B-2 was drilled near Site 2and extended to depths of 40 ft and 60 ft below ground surface, respectively. Both borings were drilled at the top of the banks due to access limitations. The locations of the Terracon borings in relation to Walnut Branch Creek are shown by Figure A-2 in the (GDR) provided by Terracon, attached to this memo in Appendix D.

RETL (2022) Geotechnical Study

RETL completed a subsurface exploration, laboratory testing program, and foundation and pavement recommendations for Guadalupe County Tax Auditor’s Office for the proposed tax office drive through addition at 307 West Court Street (RETL Project No.

G222281 – Report Dated April 28, 2022). The study included a single boring that was drilled on March 30, 2022 to a depth of 25 ft below ground surface. The location of the Rock boring in relation to Walnut Branch Creek is shown by the *Boring Location Plan* in the Appendix of the report provided by RETL. The RETL report is attached to this memo in Appendix A.

Intertek-PSI (2014) Geotechnical Engineering Study

PSI is currently known as Intertek-PSI. PSI completed a Geotechnical Engineering Study for the City of Seguin in 2014 to support the design and construction of a public library, which has since been built (PSI Project No. 0312-896 – Geotechnical Engineering Study dated May 28, 2014). Nine borings were drilled for the Seguin Public Library immediately south of West Nolte Street and immediate west of Walnut Branch Creek Six of the nine borings were drilled to depths between 50 ft and 60 ft below grade the other three borings were drilled to 10 ft below grade. The locations of the PSI borings in relation to Walnut Branch Creek are shown on the *Boring Location Plan* in the Appendix of the report provided by PSI. The PSI report is attached to this memo in Appendix A.

3.4 Field Reconnaissance

On October 13, HDR staff completed a field visit to observe visible erosion damage along Walnut Branch at Sites 1 and 2. Photographs from this field visit are provided in Appendix B. At Site 1, minor erosion was observed on the west bank (photographs 1-6) and most of the erosion damage and bank failures were observed on the east bank. Damage to a limestone block retaining wall was observed along the bank shown in photographs 7 through 9. It appears a block footer is missing in this section and backfill is being scoured away from overbank flows. Just downstream, shown in photograph 10 and 11, a stairwell leads to a dead-end path with no railing and a leaning retaining wall. Photograph 12 shows non-structural terraces that do have visible signs of significant damage or erosion. On the inside of the creek bend, sedimentation is collecting behind the low head dam up to its crest elevation (Photograph 13). The flanking of the damaged low head dam is shown in photographs 14 and 15 where a sycamore tree failed along the bank creating a bank scallop at the displaced root ball. Additional damage to a block wall was observed just downstream of the low head dam; however, vegetation prevented more detailed observation of the extent of damage or potential causes of failure.

Site 1 – Representative Photographs by HDR, October 13, 2023

1) East Bank – Bank erosion behind elevated manhole

2) West Bank – Flanking by erosion

3) West Bank – Erosion behind toe wall

4) East Bank – Sedimentation and existing utility

5) West Bank – Concrete toe repair at existing utility

6) West Bank – Minor erosion along unarmored bank

Site 1 – Representative Photographs by HDR, October 13, 2023 – continued

7) East Bank – Missing footer and major scour behind wall

8) Looking downstream – channel downcutting

9) East Bank – Loss of backfill behind wall

10) East Bank – overbank scour

11) East Bank – leaning wall and overbank scour

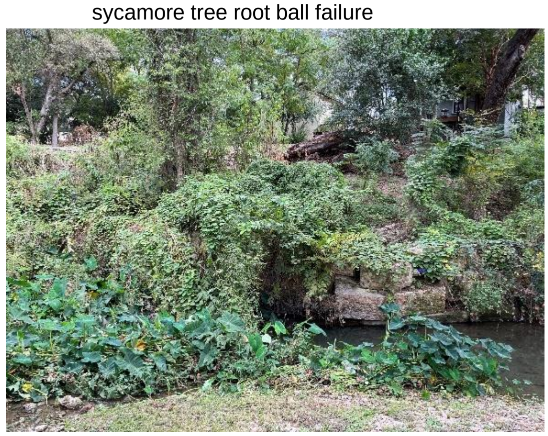
12) East Bank – no observed erosion, stacked stone terraces (non-structural)

Site 1 – Representative Photographs by HDR, October 13, 2023 – continued

13) Looking downstream (low head dam) – sedimentation

14) East Bank – Flanking of low head dam, sycamore tree root ball failure

15) Damage to low head dam



16) East Bank – Block wall failure

17) East Bank – top of bank above sycamore tree root ball failure

18) East Bank – Looking upstream at block wall failure near low head dam

For Site 2, bank erosion and bank stabilization wall failures were only observed on the east bank. The most upstream segment of the east bank is characterized by dry stack limestone chop stone and cobbles in good condition with little to no observed erosion. These small dry stack cobble stone features are located mostly on private property and do not appear to be structurally engineered. Just downstream of the private stone and cobble landscape features is an engineered limestone block wall constructed as part of the 2016 U.S. Army Corps of Engineers (USACE) Walnut Branch Ecosystem Restoration Project. The upstream end of this block wall has partially failed as shown in photographs 20 and 21. The failure appears to be from flanking by overbank flows. The remaining 145 linear feet of this wall appears to be in good condition with no visible signs of leaning or compromised backfill. Erosion was observed behind the downstream terminal blocks, but the wall remains intact (photographs 24 and 25). A small seep was observed at the downstream end of this wall as evidenced by a change in vegetation and moisture. HDR also staff documented drainage patterns behind the USACE wall from street and yard flows estimate that the drainage areas are less than 1.5 acre and do not appear to have formed significant rills down the bank. No creek bank stabilization measures were observed on the west bank.

Site 2 – Representative Photographs by HDR, October 13, 2023

19) East Bank – dry stack limestone chop stone

20) East Bank – Upstream failure of 2015 USACE block wall

21) East Bank – Looking upstream along intact wall

22) East Bank – intact wall

Site 2 – Representative Photographs, October 13, 2023 – continued

23) East Bank – Looking upstream at downstream terminus

24) East Bank – downstream terminus

25) East Bank – downstream terminus, minor erosion

26) Looking upstream from downstream of east bank wall

4 Preliminary Geotechnical Evaluation

4.1 Soil Stratigraphy and Design Parameters

Soil stratigraphy and design parameters were primarily developed by using the geotechnical investigation results provided by Terracon. Information obtained from Rock and PSI were used to supplement and generally confirm results from Terracon. The location of the borings from the three exploration programs in relation to the project site are shown on Figure 4-1 to Figure 4-3.

Figure 4-1 PSI-Intertek borings for Seguin Public Library

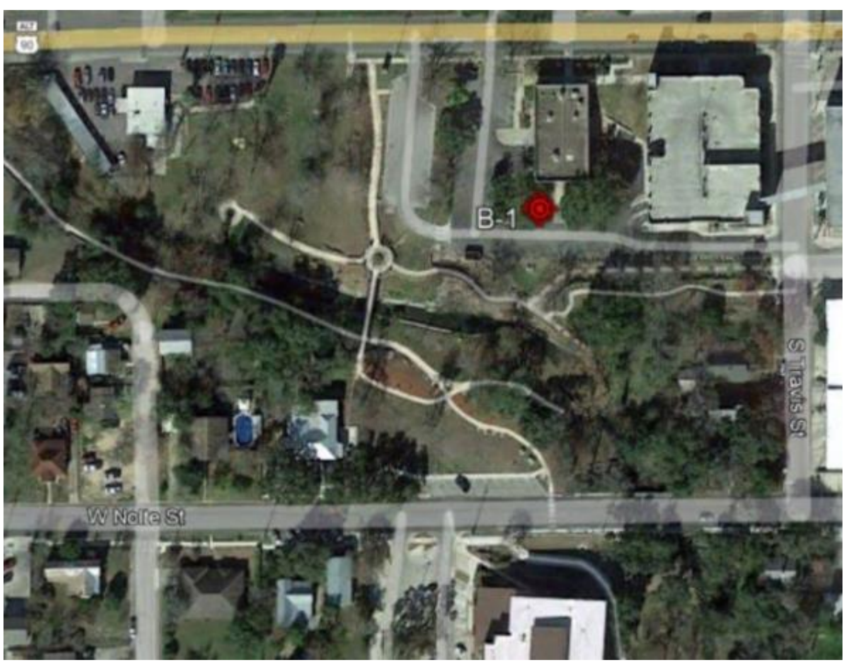


Figure 4-2 Rock boring immediately south of Guadalupe County Finance Center

Figure 4-3 Terracon borings north and south of W Nolte St, east of Walnut Branch Creek

The Terracon borings and the Rock boring are on the east side of Walnut Branch Creek and show generally the same geologic profile. The Rock boring, which was terminated at 25 ft below grade, serves only to confirm the general stratigraphy in the near surface north of Walnut Branch Creek. Assessment of strata deeper than the Rock boring is made from analysis of the Terracon and PSI borings.

The Terracon and rock investigation programs encountered a predominantly coarse-grained stratum near the surface. The coarse-grained strata consisted primarily of gravel with a variable amount of sand, silt, and clay. This strata was encountered between the ground surface and 2 ft below ground surface (2 ft of gravelly lean clay was encountered at the surface of Rock's boring), to a depth of between 8 ft and 19 ft. SPT blow counts in this stratum ranged from 15 bpf to 50 blows for 3 inches of penetration. The coarse-grained stratum was denser at Terracon Boring B-2 than at Terracon Boring B-1 and Rock Boring B-1.

Below the gravelly stratum, each boring from Terracon and Rock encountered fat clay. The Rock boring was terminated at 25 ft in fat clay, while the Terracon borings show the bottom of the fat clay stratum between 27 ft and 35 ft below ground surface. This fat clay stratum is described as *"stiff to very stiff", "tan and gray",* and *"light brown and light gray"* on the boring logs, with hand penetrometer values of 3.5 tsf or greater.

The Terracon borings, B-1 and B-2, encountered clayshale at depths of 27 ft and 35 ft below ground surface, corresponding to elevations 477.2 ft and elevation 481.8 ft, respectively. The clayshale is described as *"dark gray, hard"*, with an average shear strength from unconsolidated-undrained triaxial tests of approximately 3.8 tsf. Samples from the clayshale were obtained via Shelby Tube rather than via rock coring, an indication that the material Terracon described as clayshale may be more soil-like than rock-like.

The deeper borings drilled for the public library by PSI-Intertek, which are closest to Walnut Branch Creek, revealed a variable thickness deposit of clayey gravel and gravelly clay, with a maximum depth of 18 feet. Below the clayey gravel and gravelly clay stratum, PSI-Intertek identified stiff, fat clay rather than clayshale as identified in Terracon Borings B-1 and B-2. The difference in material identification across the two exploration programs indicates that competent clayshale is deeper on the west side of Walnut Branch Creek than it is on the east side of the creek.

Boring B-1 of the Terracon borings encountered ground water table level around El. 500 ft at Site 1, which was drilled near a spring. However, Boring B-2 did not encounter any ground water table level. PSI-Intertek encountered ground water between approximate elevations 486 ft and 498 ft in the borings drilled for the library. Ground water was not encountered in Rock Boring B-1.

Terracon's investigation included field and laboratory testing of select samples including classification and strength tests. Geotechnical design parameters developed for use in stability analyses are summarized in Table 1.

Table 1. Geotechnical design parameters

Material	Unit Weight (pcf)	Undrained (Total) Strength Parameters (UU Strength)		Undrained (Total) Strength Parameters (CU Strength)		Drained (Effective) Strength Parameters (CD Strength)	
		s_u (psf)	Φ' (deg)	c_R (psf)	Φ_R (deg)	c' (psf)	Φ' (deg)
CH Clay	120	2,000		100	15	50	20
SC/GC	120	--	32/34	0	32/34	0	32/34
Clayshale	110	4,000		800	16	500	25

4.2 Preliminary Stability Calculations

4.2.1 Modeling Approach for Global Stability Analysis of the Banks

The global stability of existing and proposed bank slopes and walls were modeled using the SLOPE/W module of GeoStudio (version 2023.1.2). The subsurface stratigraphy was developed primarily from the boring logs provided by Terracon (2023). The undrained strength parameters were selected to evaluate short-term stability and the drained strength parameters were selected to evaluate long-term stability. Both undrained and drained strength parameters are used for the staged rapid drawdown analysis.

The global slope stability analyses focused on the slope assuming the wall itself is stable. The stabilities of the existing and proposed banks were analyzed using steady-state seepage conditions, including the flood water level loading condition at El. 493 ft and normal water level loading condition at El. 487 ft for Site 1 and El. 485 ft for Site 2 analysis assuming 1 ft of water at the toe wall. Additionally, a rapid drawdown condition was analyzed for water levels changing from flood loading to normal loading condition. The water levels were modeled by drawing the phreatic surface level in SLOPE/W. The slip surface search was defined by the entry/exit method using optimization option for searching non-circular failure surfaces. The factor of safety (FS) of each slip surface was calculated using the limit equilibrium approach with Spencer's Method (1967), which satisfies both force and moment equilibrium. The existing and proposed wall elements were modeled using a high strength material to eliminate any slip surfaces that can go through the wall itself. Stability under rapid drawdown conditions was estimated using Duncan's staged rapid drawdown analysis (Duncan et al., 1990). Although the sudden drop of water level from flood to normal level is not expected for this site, rapid drawdown analysis was performed for completeness. No surcharge load is included in the analysis. The results of the slope stability analysis are presented in terms of factor of safety values in the following sections.

The recommended factor of safety values were selected based on the guidance from U.S. Army Corps of Engineers (USACE) Slope Stability Engineering and Design Manual, EM 1110-2-1902 (2003). For short-term (end of construction) and long-term loading conditions, a Factor of Safety of 1.3 and 1.5 are recommended, respectively, while a factor of safety between 1.1 and 1.3 is acceptable for rapid drawdown loading conditions.

Wall external stability calculations of overturning, sliding, and bearing capacity focusing on the toe wall stability are explained in Section 4.2.2.

4.2.2 Preliminary Wall Stability Calculations

Additional analyses were performed to evaluate existing wall stability against sliding, overturning, and bearing capacity failures. The recommended factor of safety values were selected based on the guidance from USACE Retaining and Flood Walls Engineering and Design Manual, EM 1110-2-2502 (1989), Slope Stability Engineering and Design Manual, EM 1110-2-1902 (2003), and on the FHWA Rockery Design and Construction Guidelines, Publication No. FHWA-CFL/TD-06-006. For sliding stability, a factor of safety of 1.5 is recommended for normal loading and 1.3 is recommended for flood loading. A factor of safety of 2.0 is recommended for overturning stability. Recommended bearing capacity factors of safety are 3.0 for normal loading and 2.0 for flood loading.

4.3 Preliminary Geotechnical Evaluation of Existing Conditions

4.3.1 Site 1 – North of W. Nolte Street

Soil stratigraphy used for analysis of Site 1 was developed based on boring B-1 of Terracon's investigation assuming linear stratigraphy for the site. Slope stability of the existing bank was analyzed for a section at Sta. 5+50 including the 7-ft tall retaining wall at the toe. A total of five analyses were performed to assess slope stability at Site 1. Stability under drained and undrained strength conditions, with Walnut Branch Creek both at normal water level and under flood conditions, was assessed, as was stability after rapid drawdown of flood water to normal water level. Calculated factor of safety values for the critical slip surface of Site 1 existing banks are summarized in Table 2. Shown by Plates E-1 through E-5 in Appendix E to this report show slope stability models for existing conditions at Site 1.

Table 2 Preliminary Results of Existing Wall Stability, Site 1

Slope Stability Plate #	Soil Strength Assumption	Water Level	Calculated Factor of Safety	Recommended Min. Factor of Safety
E-1	Drained	Normal	1.0	1.5
E-2	Drained	Flood	1.3	1.5
E-3	Undrained	Normal	3.2	1.3
E-4	Undrained	Flood	4.2	1.5
E-5	3-Stage Analysis	RDD from Flood to Normal	0.8	1.1-1.3

According to Terracon Boring B-1, clayshale at Site 1 is encountered around Elev. 477 ft, above which is fat clay. The clayshale is approximately 8 feet deeper than the bottom of the lowest limestone block of a typical Site 1 cross section. Model results show that the critical slip surface forms above the clayshale through the fat clay layer. While the calculated factor of safety values exceeds the recommended factor of safety values for undrained loading conditions, those calculated for the long-term drained loading and rapid drawdown loading are lower than the recommended values.

Additionally, the stability of the existing wall against sliding, overturning, and bearing capacity failure were calculated. Resulting factor of safety values are summarized in Table 3. Results show that calculated factor of safety values do not meet the recommended values for some failure modes and loading conditions, but do meet the required factor of safety for other conditions, see Table 3. It is also important to note that all analyses assume the creek bed remains at its current elevation and do account for additional bed or toe scour.

Table 3. Global Wall Stability, Preliminary Factor of Safety (FS) Values for Site 1 Existing Conditions

Failure Mode	Normal Loading	Flood Loading	Recommended Min. FS	
			Normal	Flood
Sliding	1.1	1.1	1.5	1.3
Overturning	2.6	2.8	2.0	2.0
Bearing Capacity	1.7	2.3	3.0	2.0

4.3.2 Site 2- South of W. Nolte Street

Soil stratigraphy used for analysis of Site 2 was developed based on boring B-2 of Terracon's investigation assuming linear stratigraphy for the site. Stability of the existing bank was analyzed for a section at Sta. 5+50 including the 7-ft tall retaining wall at the toe of the slope. Calculated factor of safety values for the critical slip surface of Site 2 existing banks are summarized in Table 4. Results from Site 2 existing conditions slope stability models are shown by Plates E-6 through E-10 in Appendix E. Terracon Boring B-2 indicates, clayshale is one to two feet deeper than the bottom of the lowest block of a typical wall section at approximate Elev. 482 feet. Similar to Site 1, slope stability models of Site 2 show that the critical slip surface forms above the clayshale through the fat clay layer. While the calculated factors of safety exceed the recommended factor of safety values for undrained loading conditions, those calculated for the long-term drained loading and rapid drawdown loading are lower than the recommended values.

Table 4. Preliminary Results of Existing Wall Stability, Site 2

Slope Stability Plate #	Soil Strength Assumption	Water Level	Calculated Factor of Safety	Recommended Min. Factor of Safety
E-6	Drained	Normal	1.0	1.5
E-7	Drained	Flood	1.0	1.5
E-8	Undrained	Normal	3.2	1.3
E-9	Undrained	Flood	3.5	1.5
E-10	3-Stage Analysis	RDD from Flood to Normal	0.8	1.1-1.3

Additionally, the stability of the existing wall against sliding, overturning, and bearing capacity failure were calculated. Resulting factor of safety values are summarized in Table 5. Results show that calculated factor of safety values meet the desired values for overturning and bearing capacity stability, though do not meet the desired sliding stability factor of safety. It is also important to note that all analysis assume the creek bed remains at its current elevation and do account for additional bed or toe scour.

Table 5. Global Wall Stability, Preliminary Factor of Safety (FS) Values for Site 2 Existing Conditions

Failure Mode	Normal Loading	Flood Loading	Recommended Min. FS	
			Normal	Flood
Sliding	1.0	1.1	1.5	1.5
Overturning	2.5	2.7	2.0	2.0
Bearing Capacity	>3.0	>3.0	3.0	2.0

4.4 Preliminary Geotechnical Evaluation – Preliminary Concepts

Preliminary concepts developed for each site with their respective potential improvements are summarized below. Retrofit concepts primarily focus on improving the global stability of banks to factor of safety values between 1.3 to 1.5 depending on the loading case, with a minimum factor of safety of 1.1 to 1.3 under rapid drawdown conditions.

4.4.1 Site 1 – North of W. Nolte Street

The preliminary concept developed for increasing the bank stability at Site 1 includes combination of a deep foundation and toe wall. This concept is defined as the retrofit option which targets increasing the slope stability factor of safety to 1.3 (short-term) to 1.5 (long-term conditions) at Site 1. As the clayshale encountered at Boring B-1 is considerably deeper than the base of existing wall elevation, a deep foundation system that extends into the clayshale soils is recommended in this area for improving the bank stability. A cantilever sheet pile only system is sensitive to pile drivability conditions and the top elevation of clayshale in that area; therefore, an alternative solution is a combi wall system. A combi wall system includes a combination of king piles connected with sheet piles. This system can be constructed by predrilling at the location of king piles to extend the king piles deeper into clayshale. Sheet piles connecting the king piles can be driven to refusal and without needing to extend as deep as the king piles. While both these methods provide slope stabilization by providing a connection between upper softer layers to the deeper stiff clayshale layer, a toe wall similar to the existing wall will continue to serve as the facing. CWALSHT analyses were performed to analyze the stability of a sheet pile wall and a combi wall following EM 1110-2-2504 (1994). Results show that if the clayshale layer is at El. 477 ft (as shown in B-1), a combi wall will be needed to achieve penetration depths necessary for stability. Preliminary analysis showed a combi wall including 30-in diameter 28-ft long PAZ30 Grade 50 steel pipe king piles embedded 10-ft into clayshale, and 18-ft long AZ14-770 sheet piles sitting on top of the clayshale layer provides the required stability in this area. Alternatively, stability results showed a 28-ft long NZ38, Grade 50 steel cantilevered sheet pile can be used as an alternative to the combi wall if the clayshale elevation is at El. 467 ft (10 ft deeper than the clayshale contact shown on B-1) or deeper. These preliminary analyses were performed in support of opinion of cost development. However, a more refined analysis is recommended for the design phase.

The stability of a representative cross section (approximate Station 5+50) was analyzed including a combi wall installed at the toe of the existing wall embedded 1-ft into clayshale (tip at El. 476 ft). Factor of safety values calculated by the slope stability analyses are summarized in Table 6. Plates E-11 to E-15 show a steel section at the toe of the wall extending 1 ft into clayshale. The steel section is a generalized structural element assumed to be internally stable and used to force the slope failure surface through the clayshale, demonstrating the gain in factor of safety in doing so.

Table 6. Preliminary Results of Proposed Wall Stability, Site 1

Slope Stability Plate #	Soil Strength Assumption	Water Level	Calculated Factor of Safety	Recommended Min. Factor of Safety
E-11	Drained	Normal	1.5	1.5
E-12	Drained	Flood	1.6	1.5
E-13	Undrained	Normal	4.3	1.3
E-14	Undrained	Flood	5.7	1.5
E-15	3-Stage Analysis	RDD from Flood to Normal	1.3	1.1-1.3

4.4.2 Site 2- South of W. Nolte Street

The preliminary concept developed for stabilizing the bank at Site 2 includes a limestone block wall with a footing that is founded on clayshale. As the clayshale encountered at Boring B-2 is only one to two feet deeper than the base of existing wall elevation, a deep foundation is not necessary at this site to improve the factor of safety. The conceptual wall as modeled includes a footing approximately 6 feet deep, as measured from top to bottom of footing, and 4 feet long, as measured from front to back of footing, founded at least two feet into competent clayshale. The limestone block wall above the footing would be designed to be internally stable with adequate free draining backfill material. The wall would key into banks at 45-degree from the wall alignment to protect backfill soils from erosion by flanking, and be doweled into competent clayshale. Calculated factor of safety values for the proposed wall with larger footing are summarized in Table 7.

Table 7. Preliminary Results of Proposed Wall Stability, Site 2

Slope Stability Plate #	Soil Strength Assumption	Water Level	Calculated Factor of Safety	Recommended Min. Factor of Safety
E-16	Drained	Normal	1.4	1.5
E-17	Drained	Flood	1.4	1.5
E-18	Undrained	Normal	4.5	1.5
E-19	Undrained	Flood	5.0	1.5
E-20	3-Stage Analysis	RDD from Flood to Normal	1.4	1.1-1.3

The stability of the proposed wall against sliding, overturning, and bearing capacity failure were estimated. Resulting factor of safety values are summarized in Table 8. The preliminary results show that calculated factor of safety values meet the desired values for sliding, overturning, and bearing capacity stability. These analyses assume the creek bed remains at its current elevation and do not account for additional bed or toe scour.

Table 8. Global Wall Stability, Preliminary FS Values for Site 2 Proposed Conditions

Failure Mode	Normal Loading	Flood Loading	Recommended Min. FS	
			Normal	Normal
Sliding	2.2	2.4	1.5	1.5
Overturning	2.8	2.6	2.0	2.0
Bearing Capacity	>3.0	>3.0	3.0	2.0

4.5 Existing Concrete Spillway (Low Head Dam)

The top crest of the existing concrete spillway (i.e. low head dam) has failed and sits on the bottom of the channel downstream of the structure (Photograph.15). The eastern edge of the spillway is also flanked and damaged. The recommended alternative consists of a new reinforcement concrete cap that is doweled into the existing concrete spillway (Figure 4-4). It is assumed that the existing concrete is sound enough to dowel into for the repair. There may be some minor spillway concrete repair as well, depending on the condition while doweling into it. The spillway repair also would include extending crest into the proposed east bank wall to eliminate the flanking. An 8” diameter PVC pipe was observed at the east end of spillway, but the purpose was unknown and should be explored in more detail in final design.

The original linear park plans called for water calming rocks and rock riprap (D50 = 18”) upstream and downstream of the spillway. Rocks of that size were not widely observed in the field, and it is unknown if that size was used during construction or have washed away. To protect the downstream toe of the spillway, it is recommended that rock riprap

(D50=24”) is used across the bottom of the downstream channel and along the west overbank to prevent toe scour along the low head dam.

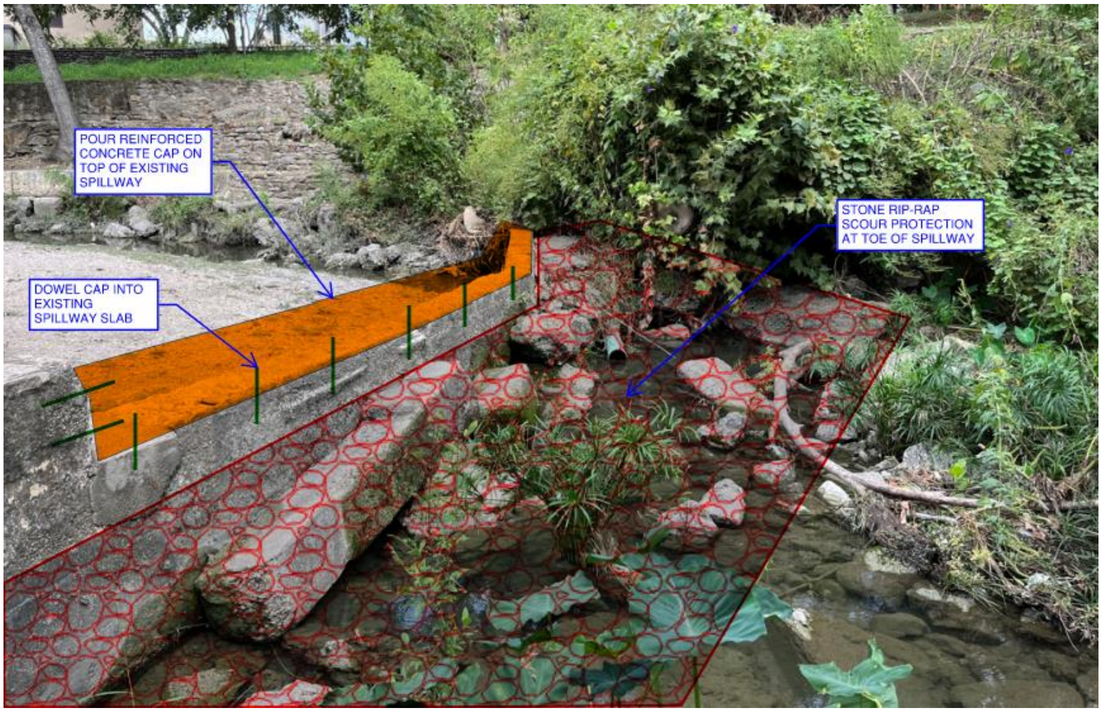


Figure 4-4. Conceptual Spillway Crest Repair Alterative

The sedimentation observed behind the spillway is expected to continue. Sediment will collect to the elevation of the restored crest. The upstream source of the sediment is unknown but could be from urban runoff in the watershed or from erosion of bed and banks upstream. If sediment sources cannot be addressed, one strategy for managing sediment is to have a dedicated sediment sink or trap where sediment will collect and access allows for its continual removal. The existing spillway currently serves as a sediment trap and has reasonable equipment access. The spillway also serves as grade control for the upstream reach. A higher, restored crest may offer more protection against downcutting of the upstream stream bed. Since the sediment load is from upstream sources and the low head dam is proposed to remain, sedimentation upstream of the structure will continue to be a maintenance activity. There is not a crest configuration that would significantly impact the sedimentation rate.

The low head dam does not impound a significant volume of water behind it. Given its low height and volume of impoundment, it is not considered a dam by the Texas Commission on Environmental Quality Dam Safety Program and does not pose the high risks associated with typical dam safety issues.

5 Discussion of Conceptual Solutions

5.1 Site 1 Alternatives

Two conceptual alternatives are considered for Site 1 and are illustrated in Sheets 1 and 2 in Appendix F. Alternative A is a repair only option with no adverse impact to global stability. It focusses on repairing the existing park hardscapes and visible erosion only but does not increase the slope stability factor of safety to the recommended minimum standard between 1.3 and 1.5. Upstream of the low head dam, approximately 16 LF of the existing limestone block toe wall would be removed and rebuilt using a typical detail that is similar to the existing wall structure. The wall at the base of the existing staircase (Photo 11) above the limestone block toe wall would be replaced with a new concrete footer wall. The backslope above the lower limestone block wall would be protected with large diameter (D50=18” to 24”) dry stone rock riprap. The 2009 plans for the linear park project called for an approximate D50=16” along the backslope, but it is unknown if this size of rock was used or if it contributed to the previous wall failure. When the wall failed and was repaired sometime after 2014, it appears 6” diameter partially grouted rock was used at least at the upstream end where portions of the installation remain (Photos 7 and 9). A majority of the backslope currently has no visible protection from erosion (Photo 10). The City has stated their preference for a low maintenance solution along the backslope so vegetated turf reinforcement which would require mowing was not considered for this upstream wall segment since it is within existing linear park hardscape features.

For the east bank downstream of the low head concrete spillway, a full replacement of the existing failed limestone block wall is recommended because the upstream end at the low head dam is flanked and unprotected, and additional wall failures were observed downstream. The failed wall could be replaced with a 160 LF combi wall system which includes a combination of king piles connected with sheet piles (See Section 4.3.1). The exposed face of the combi wall would be the steel sections. A concrete cap is proposed for the top of the combi wall system; therefore, the proposed combi wall would have a similar aesthetic to the concrete capped sheet pile wall in Max Starcke Park along River Drive West and the Guadalupe River (Figure 5-1).

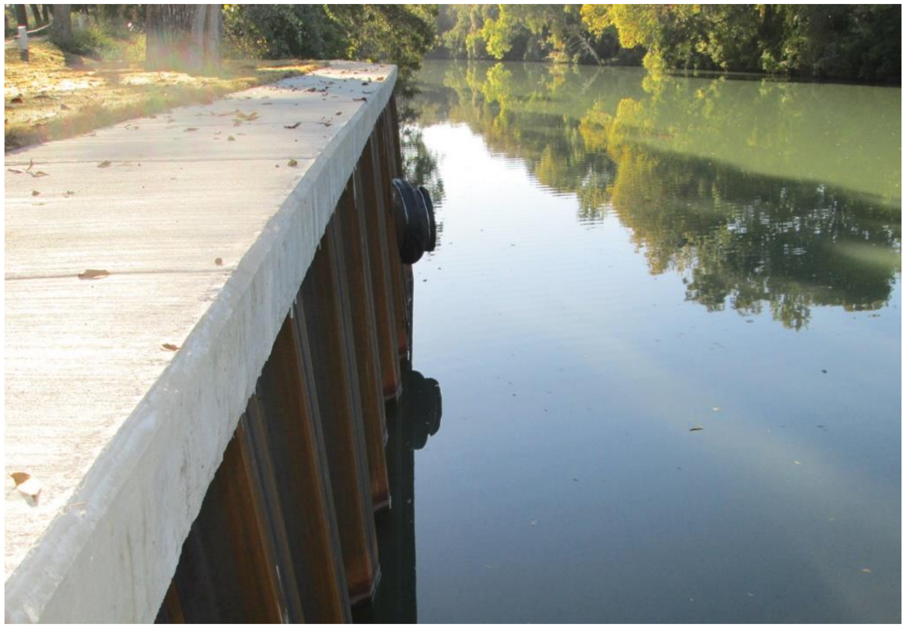


Figure 5-1. Concrete Capped Sheet Pile Wall - Max Starcke Park

The combi wall extends upstream of the low head dam and terminates into the bank at a 30-to-45-degree angle to resist flanking. The combi wall construction avoids the existing limestone block walls built as part of the linear park project, leaving a short section of existing stacked stone terraces which do not appear to be structural retaining walls but are visibly intact (Photo 12). The downstream terminus is at the W. Nolte Street wingwall. The height of the wall is an important final design consideration to balance cost, aesthetics, and the potential for overtopping flows causing scour. For this downstream wall segment, vegetated turf reinforcement along the backslope may be an option since vegetation is not currently maintained along this bank segment. Conservatively rock riprap is included in the cost assumption for the lower backfill protection and but maintenance preferences, height of combi wall, and hydraulic analysis should be considered in final design.

Alternative B is a conceptual alternative that would address the visible erosion along the east bank but would also increase the slope stability factor of safety to at least 1.3. This alternative is similar to Alternative A but the full 240 LF of toe wall would be replaced with a concrete capped combi wall. The upstream terminus is at the stepped spring feature and the downstream terminus is the W. Nolte Street wingwall. Like Alternative A, repairs to the low head spillway and retaining wall at the base of the stairway are included as well as scour countermeasures along the back slope of the wall. Both Alternative A and B also include the spillway crest restoration option.

5.2 Site 2 Alternatives

Two conceptual alternatives are considered for Site 2 and shown in Sheets 3 and 4 in Appendix F. Alternative C focusses on addressing the visible erosion only which was mainly observed at the upstream terminus and to a lesser extent at the downstream terminus. At the upstream end, approximately 12 LF of existing wall would be removed and reconstructed as a doweled limestone block wall (height varies with grade but 8’ max) founded at least 2ft into the clayshale layer (See Section 4.4.2). An additional 10LF wall extension would be constructed to key into the bank at a 45 degree angle to prevent flanking which is assumed to be the cause of the current wall failure. This key in would also be protected with large diameter stone riprap. Only minor erosion was observed at the downstream terminus where backfill has washed away at this 4' tall wall (Photo 25). To limit additional loss of backfill, this alternative proposes to reconstruct approximately 10 LF of wall as a 45 degree key that terminates no further downstream than the current wall so as to avoid the seep in the bank. The newly constructed wall segments would achieve a minimum slope stability factor of safety of 1.4 but the existing wall in between would remain in its original configuration with an approximate slope stability factor of safety of 1.0 and sliding factor of safety between 1.0 and 1.1.

Alternative D represents a conceptual solution for the full replacement of the existing limestone block wall with a new, 112 LF doweled limestone block wall that is founded at least 2ft into the clayshale layer. The footprint is nearly identical to the existing wall however the upstream and downstream ends would be keyed into the back a minimum of 10ft at a 45-degree angle and protected with large diameter riprap. This concept was shown to achieve a slope stability factor of safety of 1.4. In final design, calculation refinement and/or additional geotechnical data could help achieve a higher calculated factor of safety.

5.3 Opinion of Probable Construction Costs

5.3.1 Cost Opinion Discussion

An Associate for the Advancement of Cost Engineering (AACE) Class 4 opinion of probable cost (OPCC) has been estimated for each of the four alternatives.

- Level of Project Definition: Between 1 and 15 percent complete
- End Usage: Concept study, feasibility analysis
- Expected Accuracy Range: Low = -20 to -30 percent; High = +20 to +40 percent
    - Definition of Estimate: Class 4 estimates are generally prepared based on limited information, and subsequently have wide accuracy ranges. They are typically used for alternatives or concept screening, determination of feasibility, concept evaluation, and preliminary budget approval.
    - Estimating Methods: Class 4 estimates are frequently a mix of forced deterministic, stochastic estimating methods such as, gross unit costs/ratios. TxDOT average unit prices, City of Austin bid tabs, bid tabs from similar projects performed by HDR, preliminary unit price quotes from specialty contractors were considered as well.
- OPPCs are represented in 2024 dollars and include a 35% contingency to reflect the undefined work associated with the current level of project definition. A yearly escalation is recommended to estimate future costs.
- Exclusions: Engineering and professional services are excluded from the costs. OPCCs are based on information available to the engineer at the time of the writing of this report and the engineer's experience and qualifications. Since the engineer has no control over the cost of labor, materials, equipment, or services

furnished by others, or over the contractor's methods of determining prices, or over competitive bidding or market conditions, the engineer does not guarantee that proposals, bids, or actual project or construction costs will not vary from the opinions of probable construction cost the engineer prepares.

5.3.2 Opinion of Probable Cost Tables

A comparison of OPCCs for each of the four alternatives considered is provided in Table 9. A more detailed breakdown of estimated construction costs are provided in Table 10 through Table 13.

Table 9. Comparison of Class 4 OPCCs for each alternative

Alternative	Class 4 OPCC	Low End Range (-20%)	High End Range (+40%)
SITE 1			
A	$776,000	$621,000	$1,086,000
B	$1,049,000	$839,000	$1,468,000
SITE 2			
C	$122,000	$98,000	$171,000
D	$343,000	$275,000	$480,000
Note: Opinion of Cost Assumes May 2024 Dollars. A minimum yearly escalation is required for estimates of future costs.			

Table 10. Alternative A - Site 1

Quantity	Unit	Item Description	Unit Price	Amount
1	LS	PREPARING RIGHT OF WAY	$15,500.00	$15,500
10	SY	REMOVE P.C. CONCRETE WALL	$200.00	$2,000
1175	SF	DEMO DRY STACK WALL AND FOOTING	$35.00	$41,125
150	CY	CHANNEL EXCAVATION, PLAN QUANTITY	$65.00	$9,750
7	CY	STRUCTURAL EXCAVATION AND BACKFILL	$160.00	$1,120
6	CY	CLASS S CONCRETE FOR RETAINING WALL	$950.00	$5,700
210	CY	DRY ROCK RIPRAP (D50 = 18IN)	$185.00	$38,850
45	CY	DRY ROCK RIPRAP (D50 = 24IN)	$215.00	$9,675
1	LS	TOTAL MOBILIZATION PAYMENT (10%)	$52,000.00	$52,000
1	LS	BARRICADES, SIGNS, & TRAFFIC HANDLING	$10,000.00	$10,000
2	CY	CLASS C CONCRETE (SPILLWAY)	$ 875.00	$1,750
18	LF	WALL FOOTING	$130.00	$2,340
18	CY	CONCRETE CAP	$1,800.00	$32,400
160	LF	COMBI-WALL	$1,900.00	$304,000
72	SF	DRYSTACK BLOCK WALL SYSTEMS	$120.00	$8,640
1	LS	EROSION PROTECTION AND REVEGETATION	$19,230.00	$19,230
1	LS	CARE OF SURFACE WATER	$20,000.00	$20,000
			BASE PROJECT SUBTOTAL	**$574,080**
			CONTINGENCY (35%)	$200,928
			TOTAL OPINION OF PROBABLE CONSTRUCTION COST	**$776,000**
		Class 4 OPCC Accuracy of Estimate	**RANGE**	
		LOW END (for Class 4 Estimate)	-20%	$621,000
		HIGH END (for Class 4 Estimate)	40%	$1,086,000
Note: Opinion of Cost Assumes May 2024 Dollars. A minimum yearly escalation is required for estimates of future costs. Costs include spillway crest repairs.				

Table 11. Alternative B - Site 1

Quantity	Unit	Item Description	Unit Price	Amount
1	LS	PREPARING RIGHT OF WAY	$20,000.00	$20,000
10	SY	REMOVE P.C. CONCRETE WALL	$200.00	$2,000
1490	SF	DEMO DRY STACK WALL AND FOOTING	$35.00	$52,150
130	CY	CHANNEL EXCAVATION, PLAN QUANTITY	$55.00	$7,150
7	CY	STRUCTURAL EXCAVATION AND BACKFILL	$160.00	$1,120
6	CY	CLASS S CONCRETE FOR RETAINING WALL	$950.00	$5,700
220	CY	DRY ROCK RIPRAP (D50 = 18IN)	$185.00	$40,700
45	CY	DRY ROCK RIPRAP (D50 = 24IN)	$215.00	$9,675
1	LS	TOTAL MOBILIZATION PAYMENT (10%)	$71,000.00	$71,000
1	LS	BARRICADES, SIGNS, & TRAFFIC HANDLING	$10,000.00	$10,000
2	CY	CLASS C CONCRETE (SPILLWAY)	$ 875.00	$1,750
28	CY	CONCRETE CAP	$1,800.00	$50,400
245	LF	COMBI-WALL	$1,900.00	$465,500
1	LS	EROSION PROTECTION AND REVEGETATION	$19,230.00	$19,230
1	LS	CARE OF SURFACE WATER	$20,000.00	$20,000
			BASE PROJECT SUBTOTAL	**$776,375**
			CONTINGENCY (35%)	**$271,732**
			TOTAL OPINION OF PROBABLE CONSTRUCTION COST	**$1,049,000**
		Class 4 OPCC Accuracy of Estimate	**RANGE**	
		LOW END (for Class 4 Estimate)	-20%	$839,000
		HIGH END (for Class 4 Estimate)	40%	$1,468,000

Note: Opinion of Cost Assumes May 2024 Dollars. A minimum yearly escalation is required for estimates of future costs. Costs include spillway crest repairs.

Table 12. Alternative C - Site 2

Quantity	Unit	Item Description	Unit Price	Amount
1	LS	PREPARING RIGHT OF WAY	$7,000.00	$7,000
180	SF	DEMO DRY STACK WALL AND FOOTING	$35.00	$6,300
30	CY	CHANNEL EXCAVATION, PLAN QUANTITY	$65.00	$1,950
25	CY	DRY ROCK RIPRAP (D50 = 18IN)	$185.00	$4,625
1	LS	TOTAL MOBILIZATION PAYMENT (5%)	$4,000.00	$4,000
1	LS	BARRICADES, SIGNS, & TRAFFIC HANDLING	$7,500.00	$7,500
32	LF	WALL FOOTING	$130.00	$4,160
220	SF	DRYSTACK BLOCK WALL SYSTEMS	$120.00	$26,400
1	LS	EROSION PROTECTION AND REVEGETATION	$13,130.00	$13,130
1	LS	CARE OF SURFACE WATER	$15,000.00	$15,000
			BASE PROJECT SUBTOTAL	**$90,065**
			CONTINGENCY (35%)	**$31,523**
			TOTAL OPINION OF PROBABLE CONSTRUCTION COST	**$122,000**
		Class 4 OPCC Accuracy of Estimate	**RANGE**	
		LOW END (for Class 4 Estimate)	-20%	$98,000
		HIGH END (for Class 4 Estimate)	40%	$171,000
Note: Opinion of Cost Assumes May 2024 Dollars. A minimum yearly escalation is required for estimates of future costs.				

Table 13. Alternative D - Site 2

Quantity	Unit	Item Description	Unit Price	Amount
1	LS	PREPARING RIGHT OF WAY	$7,000.00	$7,000
1160	SF	DEMO DRY STACK WALL AND FOOTING	$35.00	$40,600
150	CY	CHANNEL EXCAVATION, PLAN QUANTITY	$65.00	$9,750
25	CY	DRY ROCK RIPRAP (D50 = 18IN)	$185.00	$4,625
1	LS	TOTAL MOBILIZATION PAYMENT (5%)	$12,000.00	$12,000
1	LS	BARRICADES, SIGNS, & TRAFFIC HANDLING	$7,500.00	$7,500
165	LF	WALL FOOTING	$130.00	$21,450
1015	SF	DRYSTACK BLOCK WALL SYSTEMS	$120.00	$121,800
1	LS	EROSION PROTECTION AND REVEGETATION	$14,180.00	$14,180
1	LS	CARE OF SURFACE WATER	$15,000.00	$15,000
			BASE PROJECT SUBTOTAL	**$253,905**
			CONTINGENCY (35%)	**$88,867**
			TOTAL OPINION OF PROBABLE CONSTRUCTION COST	**$343,000**
		Class 4 OPCC Accuracy of Estimate	**RANGE**	
		LOW END (for Class 4 Estimate)	-20%	$275,000
		HIGH END (for Class 4 Estimate)	40%	$480,000
Note: Opinion of Cost Assumes May 2024 Dollars. A minimum yearly escalation is required for estimates of future costs.				

5.4 Permitting Considerations

Based on review of available information from previous permitting and coordination as well as the evaluation of repair options discussed herein, HDR offers the following observations.

- Walnut Branch is a water of the U.S. under current guidelines and previous delineation appears consistent.

- For Section 404 permitting, the previous Nationwide Permit (NWP) 3, SWF-2021-00460, appears valid for repairs that match the layout submitted by TRC March 1, 2022, as documented in the information received from USACE on January 9, 2024. The information provided by USACE contains emails with Section 106 / Texas Historical Commission (THC) concurrence; therefore, THC concurrence is documented for what was previously authorized and would occur in the same footprint.
- For Site 2 downstream of Nolte and Section 408 permission from the USACE, the City would likely be using maintenance responsibility for repairs that return to previous design. The City should review the archived files including their operation and maintenance responsibility and documentation from the USACE project.
- If the project repairs fall within the same footprint as previous NWP 3 authorization and are “maintenance” to the previous USACE project downstream of Nolte, there would likely be minimal to no additional federal permitting and agency coordination required. Furthermore, if the proposed repairs maintain the same footprint that previously had THC approvals and the previous THC concurrence that indicates no historic properties are present or effected, no additional cultural resources effort and coordination is anticipated for the same footprint. However, the permitting requirements should be confirmed as design progresses and the engineering design and construction footprint are determined. The City may want to confirm with agencies, especially the USACE, that the proposed project is covered by previous permitting and the maintenance provisions of the USACE project downstream of Nolte. If necessary, based on the design footprint or agency feedback, additional resources review effort and permitting coordination may be necessary.
- Walnut Branch is within a Zone AE with floodway special flood hazard area as shown in FEMA Map 48187C0280G, effective 3/27/24. Any floodplain modifications must demonstrate a no rise criteria. Since the recommended alternatives are within the footprint of existing improvements, it is anticipated that a no rise criteria can be achieved but will need to addressed in final design as final wall heights and grading is determined.

6 Future Considerations for Final Design

The objective of this planning report is to identify engineering concepts to address observed erosion and failures of existing bank stabilization projects and provide opinion of probable construction costs and permitting constraint summaries. The project is based in part on visual assessments and a limited number of observations and data. Visual assessments are not able to detect hidden, covered, inaccessible, or internal structural or material defects. Assumptions were made about the existing constructed features and construction plans provided by the City. As-built drawings or construction inspection data were not provided and original construction has been repaired at several locations; therefore actual construction may have differed from the design plans which may impact HDR’s opinions and recommendations.

Since a limited geotechnical subsurface investigation was performed and soil conditions may vary between or beyond the points explored or observed, it is recommended that additional borings are collected at Site 1 and 2 on the west side close to the planned work. An additional boring at Site 1 would help refine the tip elevation and wall heights for a deep foundation system. At Site 2, founding the doweled limestone block wall into the clayshale is critical, and based on the one boring collected for this project and the borings from the library, the clayshale layer appears to vertically dip towards the west.

The topographic graphic survey completed for this project was limited in its scope with a focus on collecting topographic information from a static scanner and available online geospatial data. The collected planimetric features and spot elevations were limited, and additional survey will be needed to support a final design phase.

Based on the prior permitting correspondence by others, it appears minimal to no additional federal permitting and agency coordination are required especially if the chosen repairs maintain the existing footprint. However, the permitting requirements should be confirmed as design progresses and the engineering design and construction footprint are determined. The City may want to confirm with agencies, especially the USACE, that the proposed project is covered by previous permitting and the maintenance provisions of the USACE project downstream of Nolte. If necessary, based on the design footprint or agency feedback, additional resources review effort and permitting coordination may be necessary.

The conceptual alternatives identified are intended to help prioritize future investments. Since most of the observed erosion and damage was on the east side where vertical hardscapes and taller hillsides exist, the alternatives prioritize these higher risk areas within the footprint of existing projects. In final design, it may be economically efficient to add toe erosion repair along the west side since these areas are likely to be disturbed during construction and a contractor would already be mobilized. If the footprint of the existing west bank features is preserved, the permitting constraints likely would not be impacted.

For the Site 1 deep foundation solutions, a specialty contractor should be consulted to verify site access, constructability, and vibratory impacts to existing park features. The recommended factor of safeties for Sites 1 and 2 are in part a function of uncertainty and consequences of failure. In final design, calculation refinement and/or additional geotechnical data could help achieve a higher calculated factor of safety and adjustment of the minimum design recommendations.

7 References

Duncan, J.M., Wright, S.G. and Wong, K.S. (1990) "Slope Stability during Rapid Drawdown". Proceedings of the H. Bolton Seed Memorial Symposium. Vol. 2

Spencer, E. 1967. "A Method of Analysis of Embankments assuming Parallel Inter-column Forces", Geotechnique, Vol 17 (1), pp. 11-26

USACE Design of Sheet Pile Walls, EM 1110-2-2504 (1994).

USACE Slope Stability Engineering and Design Manual, EM 1110-2-1902 (2003).

USACE Retaining and Flood Walls Engineering and Design Manual, EM 1110-2-2502 (1989)

Appendix A. Record Documents

This is a digital appendix.

Folder	Document: Description
CovenantStreetBridge	Plan sheets and bore logs for project located downstream of Site 1 and 2
GeotechData	Geotech Report (Rock - G222281): 2022 boring at location B1, North of West Nolte St. Depth 25'
	Intertek PSI - City of Seguin Library - 0312-0896: Historical boring information from the library construction
	Seguin Walnut Branch Pedestrian Bridge Repair #TF-2022-17 DWGS
LinearPark_DesignPlans	Walnut Branch Linear Park Phase 1 (2009 Plan Set)
	Seguin Walnut Branch Pedestrian Bridge Repair #TF-2022-17 DWGS – 2022 repairs at pedestrian bridge
USACE 205 Channel - Walnut Branch	CHANNEL_IMPROVEMENT_ALL_reduced.pdf – Local Flood Protection, Walnut Branch August 1989, 89-B-0260
	MAINTENANCE PERMIT (404 & 408) Subfolder – Operation and maintenance manual and maintenance permit document
	USACE 205 Subfolder – Appraisal reports along Walnut Branch
	Walnut Creek - NBC Bank Lot 2 Block 5 Folder - Deeds and plats along Walnut Branch
	206 Reforestation PDF DGN folder - CAD and PDF of reforestation project
USACE 206 Re-forestation	2016 12 05 4th Quarter Financial Report (Federal)
	2018 07 25 USACE Close out Letter to Seguin - Completion letter and O&M manual
	2022 9 15 Draft PEA - Programmatic Environmental Assessment Email
	Maintenance Permit (404 & 408)
	USACE 205 Channel - Walnut Branch Folder - Channel improvements plans and O&M manual
	USACE 206 Folder - Jacobs's project folder for reforestation project. Includes plans, minutes, memos, bid and construction phase documents, etc.
	USACE 206 _ 02-26-21 folder: Correspondence and misc. attachments
	USACE Permits – Misc. permit documentation.
	USACE 404 - Walnut Springs 2021-00460: Misc. NWP 42 and 13 application documentation
Base Folder	2022020 SWF-2021-00460 NWP 3 Letter: Nationwide permit 3 approval for Low head dam removal
	Aerial_Exhibit_Portrait: Proposed Stream Restoration/ Repair Project map by TRC
	Geotech Report (Rock - G222281). 2022 boring at location B1, North of West Nolte St. Depth 25'

	Memo regarding project status report for the repair of the Walnut Springs Park dam and bank stabilization
	Seguin Walnut Branch Spillway NWP 13 PCN Application_Revised_3-1-2022
	Walnut Branch Dam - Spillway Environmental Permitting Status Report from TRC Engineers
	Walnut Branch Fact Sheet
	Walnut Branch PlanningDesignReportandEnvironmentalAssessment2003

Appendix B. Preliminary Site Assessment, October 13, 2023 and Digital Photos

This is a hybrid appendix, containing a letter and digital photos.

November 17, 2023

Pablo Martinez, P.E., CFM
Project Engineer
Capital Projects and Engineering
108 E. Mountain Street,
Seguin, TX 78155

RE: Preliminary Visual Qualitative Assessment of Bed and Bank Conditions along Walnut Branch

Dear Mr. Martinez,

The City of Seguin (City) retained HDR Engineering Inc. (HDR) to provide project planning services for the repair and stabilization of the bed and banks along Walnut Branch at two sites (Figure 1). Site 1 is in Walnut Springs Park between the existing pedestrian bridge and W. Nolte Street. Site 2 begins at N. Nolte Street and ends near W. Washington Street directly across from the library. The objective of the first phase of the planning services is to provide a preliminary qualitative opinion of potential risk for additional wall and embankment failure. The second phase is to develop feasibility engineering concepts to address erosion and embankment failures and provide opinion of probable construction costs and permitting constraint summaries. This letter memorandum summarizes the Phase 1 efforts. Phase 2 is underway and will yield more detailed documentation and evaluation.

Figure 1. Project Limits - Site Observations 10/13/23

4401 West Gate Blvd., Suite 400, Austin, TX 78745
T 512.912.5100 **F** 512.912.5158

On October 13, HDR staff completed a field visit to observe visible erosion damage along Walnut Branch at Sites 1 and 2. At Site 1, minor erosion was observed on the west bank and most of the erosion damage and bank failures were observed on the east bank. Damage to a limestone block retaining wall was observed along the bank shown in photographs 7 through 9. It appears a block footer is missing in this section and backfill is being scoured away from overbank flows. Just downstream, shown in photograph 10 and 11, a stairwell leads to a dead-end path with no railing and a leaning retaining wall. The flanking of the damaged low head dam is shown in photographs 14 and 15 where a sycamore tree failed along the bank creating a bank scallop at the displaced root ball. Additional damage to a block wall was observed just downstream of the low head dam; however, vegetation prevented more detailed observation of the extent of damage or potential causes of failure.

Site 1 – Representative Photographs, October 13, 2023

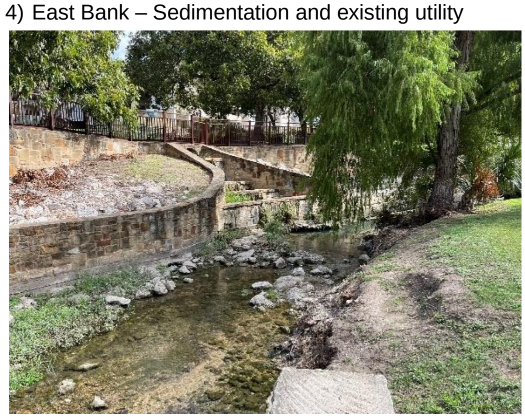

1) East Bank – Bank erosion behind elevated manhole


2) West Bank – Flanking by erosion


3) West Bank – Erosion behind toe wall


4) East Bank – Sedimentation and existing utility


5) West Bank – Concrete toe repair at existing utility


6) West Bank – Minor erosion along unarmored bank

Site 1 – Representative Photographs, October 13, 2023 - continued

7) East Bank – Missing footer and major scour behind wall

8) Looking downstream – channel downcutting

9) East Bank – Loss of backfill behind wall

10) East Bank – overbank scour

11) East Bank – leaning wall and overbank scour

12) East Bank – no observed erosion

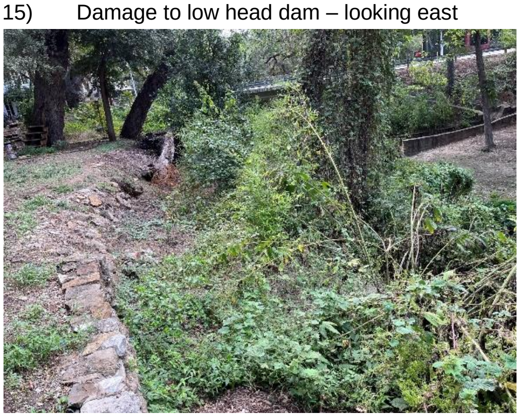
Site 1 – Representative Photographs, October 13, 2023 - continued

13) Looking downstream (low head dam) - sedimentation

14) East Bank – Flanking of low head dam, sycamore tree root ball failure

15) Damage to low head dam – looking east

16) East Bank – Block wall failure



17) East Bank – top of bank above sycamore tree root ball failure

18) East Bank – Looking upstream at block wall failure

For Site 2, bank erosion and bank stabilization wall failures were only observed on the east bank. The most upstream segment of the east bank is characterized by dry stack limestone chop stone and cobbles. The engineered limestone block wall construction as part of the U.S. Army Corps of Engineers (USACE) 2016 project has failed at the upstream end as shown in photographs 20 and 21. The failure appears to be from flanking by overbank flows. The remaining 145 linear feet of this wall appears to be in relatively good condition with no visible signs of leaning or compromised backfill. Erosion was observed behind the downstream terminal blocks, but the wall remains intact (photographs 24 and 25). HDR staff documented drainage patterns behind this wall from street and yard flows for documentation and evaluation in the Phase 2 preliminary engineering report. No retaining walls or creek bank stabilization measures were observed on the west bank. However, significant damage was noted to the 1930's era historical stone walls along the pedestrian path next to the library. While the walls along the trail are outside of the study area, it is noted that what little remains intact of these historical walls appears to be very unstable.

Site 2 – Representative Photographs, October 13, 2023

19) East Bank – dry stack limestone chop stone walls

20) East Bank – Upstream failure of 2016 USACE block wall

21) East Bank - Looking upstream along intact wall

22) East Bank – intact wall

Site 2 – Representative Photographs, October 13, 2023 - continued

23) East Bank - Looking upstream at downstream terminus of 2016 wall

24) East Bank – downstream terminus of 2016 wall

25) East Bank – downstream terminus of 2016 wall

26) Looking upstream from downstream of east bank wall

27) West Bank – failed historic stone walls along trail

28) West Bank – failed historic stone walls along trail

Additional investigation and evaluations will be performed in Phase 2 of the study, but based on preliminary visual observations, additional wall and embankment failure is likely to occur. While the severity of risk and likelihood of additional failure cannot be determined based on a visual inspection alone, Figure 2 illustrates two areas where the most damage and erosion were observed. The City has placed signs throughout Sites 1 and 2 warning the public of potential safety concerns due to unstable materials, erosion, and slip, trip and fall risks.

For Site 1, the bank scallop at the low head dam (photograph 16) and failed block wall is the largest observed damage area and a potential hazard to the public. The level of temporary protection provided by the existing bank vegetation and root ball is unknown. The scour hole behind the existing wall at the spring (photograph 9) is another potential hazard to the public.

For Site 2, the upstream end of the block wall has been flanked (photograph 20) and the materials are unstable, but additional failure along this wall was not observed. While the risk of additional failure may exist because of uncontrollable factors, the consequence of failure at each site is different and should be considered by the City in Phase 2.

Figure 2. Areas of greatest creek bank/wall damage based on visual observation only

The field assessment in Phase 1 was limited to visual assessments only and HDR's opinions are based on a limited number of observations and data. Visual inspections are not able to detect hidden, covered, inaccessible, or internal structural or material defects. Engineering and computational evaluations have not been completed. It is possible that conditions could vary between or beyond the data and observations evaluated. HDR makes no other representation, guarantee, express or implied, regarding the services or communication (oral or written). HDR will continue to work with the City on the Phase 2 evaluations to help prioritize future investments in the erosion and stabilization repairs at Sites 1 and 2.

Sincerely,
HDR Engineering, Inc.

Eric J. Stewart, P.E., CFM
Project Manager

Cc: Melissa Reynolds (City of Seguin)

Appendix C. Survey

This is a digital appendix.

Appendix D. Geotechnical Data Report (Terracon 2023)

Revised Geotechnical Data Report

Walnut Branch Creek Walls

Seguin, Texas

December 7, 2023

Terracon Project No. 90235129R

Prepared for:

HDR Engineering, Inc.
San Antonio, Texas

Prepared by:

Terracon Consultants, Inc.
San Antonio, Texas

Terracon
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Nationwide
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- Facilities
- Environmental
- Geotechnical
- Materials

December 7, 2023

HDR Engineering, Inc.
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San Antonio, Texas 78216-5550

Attn: Mr. Thomas Wesling, P.E.
C: (210) 841-2800
E: thomas.wesling@hdrinc.com

Re: Revised Geotechnical Data Report
Walnut Branch Creek Walls
313 West Nolte Street
Seguin, Texas
Terracon Project Number: 90235129R

Dear Mr. Wesling:

Terracon Consultants, Inc. (Terracon) has completed the geotechnical services for the above referenced project. This report presents the findings of the subsurface exploration for the proposed project. We appreciate the opportunity to work with you on this project and look forward to contributing to the ongoing success of this project by providing Materials Testing services during construction. Should there be any questions, please do not hesitate to contact our office.

Sincerely,
Terracon Consultants, Inc.
(Firm Registration: TX F3272)

Carlos Cotilla
Staff Engineer

Gregory P. Stieben, P.E.
Senior Consultant

TABLE OF CONTENTS

REVISED GEOTECHNICAL DATA REPORT
WALNUT BRANCH CREEK WALLS
SEGUIN, TEXAS
Terracon Project No. 90235129R
DECEMBER 7, 2023

1.0 INTRODUCTION

Terracon Consultants, Inc. (Terracon) is pleased to submit our Revised Geotechnical Data Report for the proposed Walnut Branch Creek Walls project located near 313 West Nolte Street in Seguin, Texas. The project scope was performed in general accordance with Terracon Proposal No. P90235129R, dated May 31, 2023.

2.0 PROJECT INFORMATION

2.1 Project Description

Item	Description
Site layout	Refer to Appendix A; Exhibit A-1: Site Location Plan and Exhibit A-2: Boring Location Plan.
Evaluation	The proposed project includes the evaluation of the subsurface soil and depth to water conditions near the existing retaining walls within Walnut Springs Park and the Seguin Public Library site.

2.2 Site Location and Description

Item	Description
Location	The project is located near 313 West Nolte Street in Seguin, Texas. The boring locations were selected by the Client. Boring locations are within the Walnut Springs Park and near the Seguin Public Library.
Current ground cover	Bare soil, and grass.
Existing topography	Unknown.

3.0 SUBSURFACE CONDITIONS

3.1 Typical Profile

Conditions encountered at the boring locations are indicated on the individual boring logs. Stratification boundaries on the boring logs represent the approximate location of changes in soil types; in situ, the transition between materials may be gradual. Details for the borings can be found on the boring logs in Appendix A of this report.

3.2 Groundwater Conditions

Borings B-1 and B-2 were advanced using dry drilling techniques to the boring termination depths of approximately 40 feet and 60 feet, respectively, in an effort to evaluate groundwater conditions at the time of our field program. Upon reaching groundwater, groundwater levels were recorded during drilling and after boring completion. Information regarding groundwater measurements is summarized below and can be found on the boring logs.

Boring No.	Approximate Boring Depth[1] (feet)	Approximate Groundwater Depth (feet)[1]	
		Initial / During Dry Drilling	After Boring Completion
B-1	40	6	4
B-2	60	---	---

1. Below existing grade at the time of our field program.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the proposed improvements may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project and should be evaluated prior to construction.

APPENDIX A

A-1 SITE LOCATION PLAN

Walnut Branch Creek Walls ■ Seguin, TX
December 7, 2023 ■ Terracon Project No. 90235129R

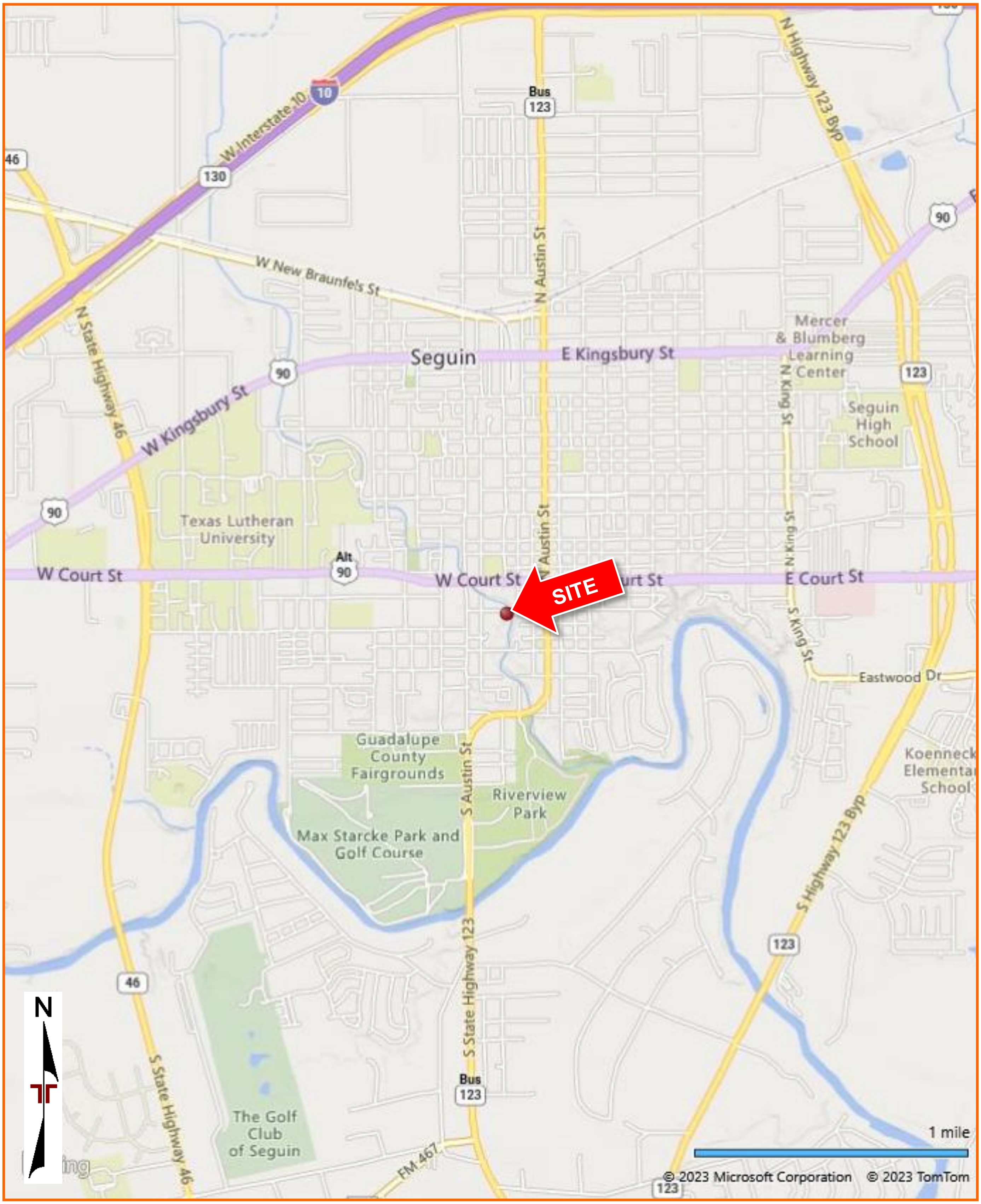


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

TOPOGRAPHIC MAP IMAGE COURTESY OF THE U.S. GEOLOGICAL SURVEY QUADRANGLES INCLUDE: SEGUIN, TX (1/1/1994).

A-2 BORING LOCATION PLAN

Walnut Branch Creek Walls ■ Seguin, TX
December 7, 2023 ■ Terracon Project No. 90235129R

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

AERIAL PHOTOGRAPHY PROVIDED BY MICROSOFT BING MAPS

Field Exploration Description

The boring locations were staked by Terracon personnel with a handheld GPS device. The location of the borings should be considered accurate only to the degree implied by the means and methods used to define them.

A truck-mounted, rotary drill rig equipped with continuous flight augers was used to advance the borehole. Soil sampling was performed using thin-wall tube and/or split-barrel sampling procedures. In the thin-walled tube sampling procedure, a thin-walled, seamless steel tube with a sharp cutting edge was pushed hydraulically into the soil to obtain a relatively undisturbed sample. In the split-barrel sampling procedure, a standard 2-inch O.D. split-barrel sampling spoon is driven into the ground with a 140-pound hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the standard penetration resistance value. These values are indicated on the boring logs at the depths of occurrence. If the sampler was driven less than the final 12 inches, the N value is recorded on the log as the number of blows and amount of penetration.

The samples were tagged for identification, sealed to reduce moisture loss, and taken to our laboratory for further examination, testing, and classification. Information provided on the boring logs attached to this report includes soil descriptions, consistency evaluations, boring depths, sampling intervals, and groundwater conditions. The borings were backfilled with cement bentonite grout after completion of drilling and patched with asphalt accordingly.

Our field representative prepared the field logs as part of the drilling operations. The field logs included visual classifications of the materials encountered during drilling and our field representative interpretation of the subsurface conditions between samples. The final boring logs included with this report represent the engineer's/geologist's interpretation of the field logs and include modifications based on visual observations, laboratory observations and testing of the samples in the laboratory.

The scope of services for our geotechnical engineering services does not include addressing any environmental issues pertinent to the site.

Terracon
6911 Blanco Rd
San Antonio, TX

Boring Log No. B-1

Location: See Exploration Plan

Latitude: 29.5681° Longitude: -97.9673°

Depth (Ft.) Elevation.: 504.22 (Ft.)

Depth (Ft.)	Material Description	Elevation (Ft.)	Sample Depth (Ft.)	Water Level Observations	Field Test Results	Test Type	Compressive Strength (tsf)	Strain (%)	Confining Pressure (psi)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
1.0	**FAT CLAY (CH)**, dark brown, hard	503.22			4.5+ (HP)								
	CLAYEY GRAVEL WITH SAND (GC), tan, medium dense to dense				16-17-15 N=32								
	- percent passing no. 4 sieve = 56%			▽ 6 ft	5-7-8 N=15					12.0		47-17-30	33
				▽ 4 ft	12-12-16 N=28								
					11-12-15 N=27								
	- percent passing no. 4 sieve = 52%		10		11-11-10 N=21					10.5		64-19-45	19
12.0		492.22			6-7-11 N=18								
	FAT CLAY (CH), tan and gray, stiff to very stiff				7-8-13 N=21								
	- percent passing no. 4 sieve = 100%				3-5-7 N=12					35.6		82-22-60	99
					8-9-9 N=18								
	- hard below 18 feet				4.25 (HP)					34.4		73-26-47	100
			20										
					13-14-16 N=30								
27.0		477.22											
	CLAY-SHALE, dark gray, hard				4.5+ (HP)	UU	7.78	10	13	22.4	99		
					4.5+ (HP)								
			30										
	- percent passing no. 4 sieve = 100%				12-20-25 N=45					27.0		80-24-56	100
					10-22-31 N=53								
40.0		464.22	40										

Boring Terminated at 40 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
▽ 6 feet while drilling
▽ 4 feet at completion of drilling

Drill Rig
CME 55

Hammer Type
Automatic

Driller
Bobby

Notes
Elevations were provided by the Client

Advancement Method
Flight Auger: 0-16 feet
Hollow Stem: 0-40 feet

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
AC

Boring Started
08-31-2023

Boring Completed
08-31-2023

Exhibit A-4

Terracon
6911 Blanco Rd
San Antonio, TX

Boring Log No. B-2

Location: See Exploration Plan

Latitude: 29.5667° Longitude: -97.9663°

Elevation.: 516.78 (Ft.)

Depth (Ft.)	Material Description	Elevation
0.1	**Asphalt about ¾ inches thick**	516.68
0.8	**Base Material about 8 inches thick**	515.98
9.0	**CLAYEY SAND WITH GRAVEL (SC)**, tan, very dense - percent passing no. 4 sieve = 75%	507.78
19.0	**SILTY CLAYEY GRAVEL WITH SAND (GC-GM)**, tan, very dense - percent passing no. 4 sieve = 58% - percent passing no. 4 sieve = 68%	497.78
35.0	**FAT CLAY (CH)**, tan and gray, very stiff to hard - percent passing no. 4 sieve = 100%	481.78
60.0	**CLAY-SHALE**, dark gray, hard - percent passing no. 4 sieve = 100%	456.78
	Boring Terminated at 60 Feet	

Field Test Results	Test Type	Compressive Strength (tsf)	Strain (%)	Confining Pressure (psi)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
40-50/6"								
26-31-37 N=68					4.0		24-14-10	32
30-32-29 N=61								
22-28-28 N=56								
23-27-50/4"					3.1		18-11-7	18
50/3"								
27-12-50/4"								
50/4"								
23-35-39 N=74					9.1		20-14-6	38
4.5+ (HP)	UU	3.52	5	13	24.8	98		
4.0 (HP)					26.6		74-23-51	100
3.5 (HP)	UU	3.76	2.9	15	31.3	95		
3.5 (HP)								
3.5 (HP)	UU	2.78	2.8	18	5.5	114		
4.5+ (HP)								
4.25 (HP)							69-25-44	98
4.5+ (HP)								
4.5+ (HP)	UU	7.54	3.6	23	40.2	89		
4.5+ (HP)								
4.5+ (HP)					23.5		65-25-40	100
4.5+ (HP)								
4.5+ (HP)	UU	7.38	4.3	27	21.7	102		
4.5+ (HP)								
4.5+ (HP)								
25-32-50/5"								

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes

Elevations were provided by the Client

Water Level Observations
No free water observed

Advancement Method
Flight Auger: 0-60 feet

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Drill Rig
CME 55

Hammer Type
Automatic

Driller
Bobby

Logged by
AC

Boring Started
09-01-2023

Boring Completed
09-01-2023

Exhibit A-5

APPENDIX B

Laboratory Testing

Samples retrieved during the field exploration were taken to the laboratory for further observation by the project geotechnical engineer and were classified in accordance with the Unified Soil Classification System (USCS) described in this Appendix. At that time, the field descriptions were confirmed or modified as necessary and an applicable laboratory testing program was formulated to determine engineering properties of the subsurface materials.

Laboratory tests were conducted on selected soil samples and the test results are presented in this appendix. The laboratory test results were used for the geotechnical engineering analyses, and the development of foundation and earthwork recommendations. Laboratory tests were performed in general accordance with the applicable ASTM, local or other accepted standards.

Selected soil samples obtained from the site were tested for the following engineering properties:

- Water contents - ASTM D2216,
- Atterberg limits - ASTM D4318,
- Sieve analyses - ASTM D6913,
- Wash minus 200 sieves - ASTM D1140,
- Hydrometer analyses - ASTM D7928.
- UU triaxial compression tests - ASTM D2850,
- CU triaxial compression tests - ASTM D4767,
- Corrosion potential testing (pH, soluble sulfates, soluble chlorides, electrical resistivity, and Redox) – ASTM G51, D512, D516, G57, and G200.

Sample Disposal

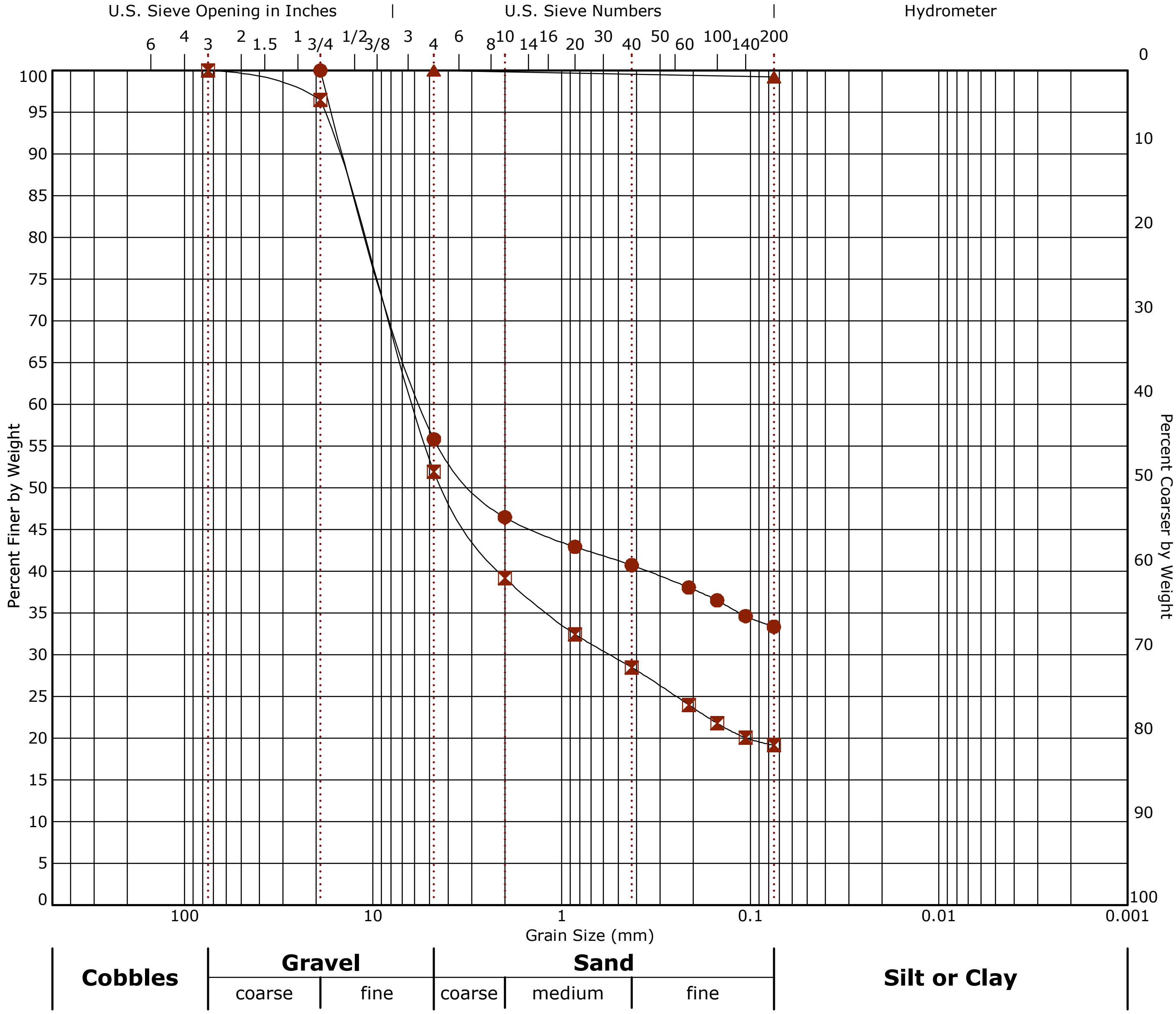
All samples were returned to our laboratory. The samples not tested in the laboratory will be stored for a period of 30 days subsequent to submittal of this report and will be discarded after this period, unless other arrangements are made prior to the disposal period.

Terracon
6911 Blanco Rd
San Antonio, TX

Exhibit B-2

Grain Size Distribution

ASTM D422 / ASTM C136



	Boring ID	Depth	% Cobbles	% Gravel	% Sand	% Fines	% Silt	% Clay	USCS
●	**B-1**	3 - 4.5	0.0	44.2	22.5	33.4			GC
⊠	**B-1**	8.5 - 10	0.0	48.0	32.8	19.1			GC
▲	**B-1**	14 - 15.5	0.0	0.0	0.8	99.2			CH

	Description
●	CLAYEY GRAVEL with SAND
⊠	CLAYEY GRAVEL with SAND
▲	FAT CLAY

	Remarks
●	
⊠	
▲	

● Sieve	● % Finer	⊠ Sieve	⊠ % Finer	▲ Sieve	▲ % Finer
3/4"	100.0	3"	100.0	#4	100.0
#4	55.81	3/4"	96.47	#200	99.21
#10	46.48	#4	51.95		
#20	42.93	#10	39.18		
#40	40.72	#20	32.46		
#70	38.05	#40	28.47		
#100	36.51	#70	24.0		
#140	34.63	#100	21.8		
#200	33.36	#140	20.06		
		#200	19.15		

Grain Size

	●	⊠	▲
D_{60}	5.417	6.102	
D_{30}		0.554	
D_{10}			

Coefficients

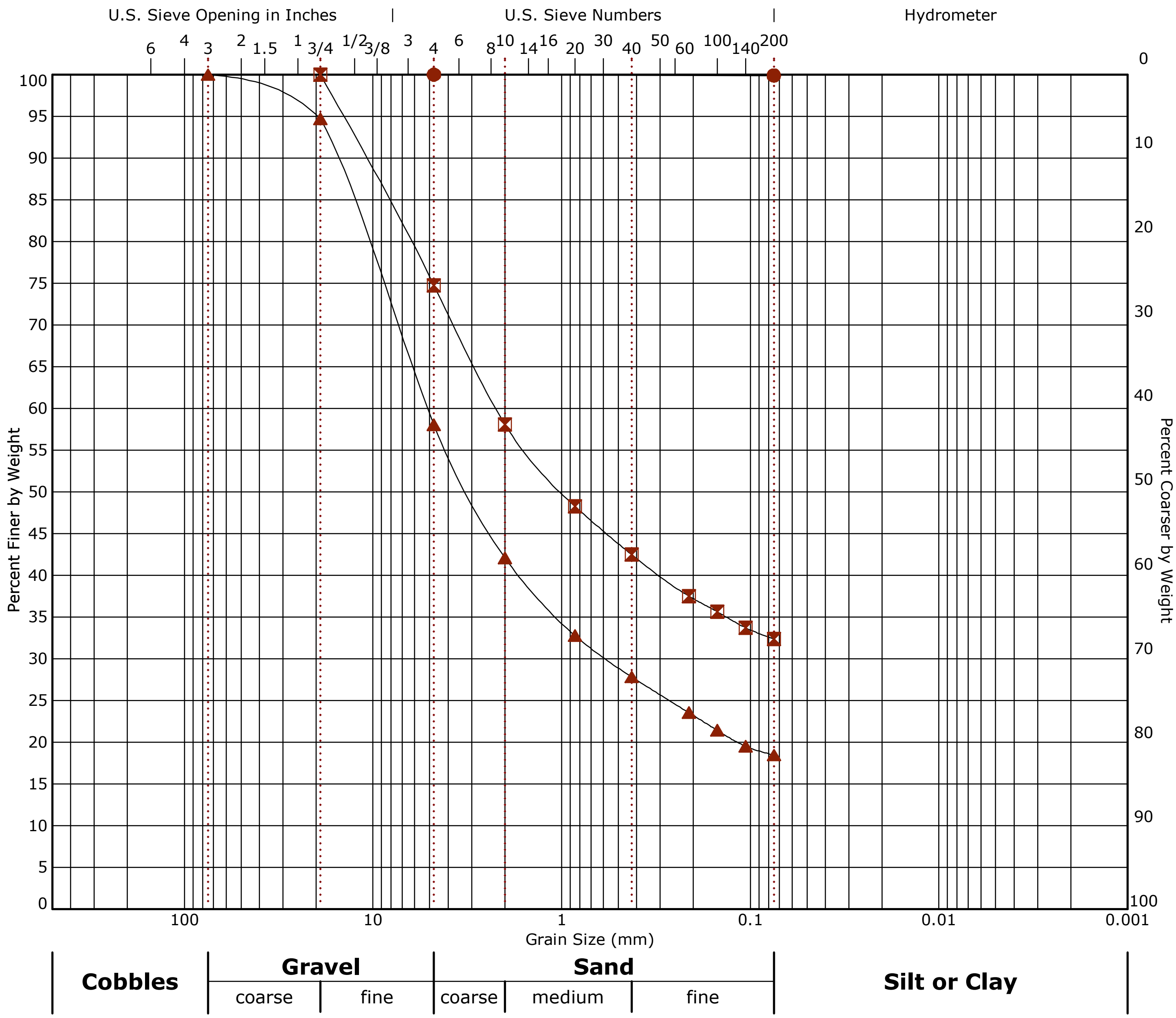
	●	⊠	▲
C_c			
C_u			

Terracon
6911 Blanco Rd
San Antonio, TX

Exhibit B-3

Grain Size Distribution

ASTM D422 / ASTM C136



	Boring ID	Depth	% Cobbles	% Gravel	% Sand	% Fines	% Silt	% Clay	USCS
●	B-1	33.5 - 35	0.0	0.0	0.1	99.9			CH
⊠	B-2	3 - 4.5	0.0	25.2	42.4	32.4			SC
▲	B-2	9 - 10.3	0.0	41.9	39.6	18.5			GC-GM

	Description
●	FAT CLAY
⊠	CLAYEY SAND with GRAVEL
▲	SILTY, CLAYEY GRAVEL with SAND

	Remarks
●	
⊠	
▲	

● Sieve	● % Finer	⊠ Sieve	⊠ % Finer	▲ Sieve	▲ % Finer
#4	100.0	3/4"	100.0	3"	100.0
#200	99.9	#4	74.77	3/4"	94.72
		#10	58.09	#4	58.05
		#20	48.26	#10	42.1
		#40	42.48	#20	32.78
		#70	37.52	#40	27.81
		#100	35.65	#70	23.56
		#140	33.73	#100	21.43
		#200	32.4	#140	19.53
				#200	18.47

Grain Size

	●	⊠	▲
D_{60}		2.208	5.113
D_{30}			0.577
D_{10}			

Coefficients

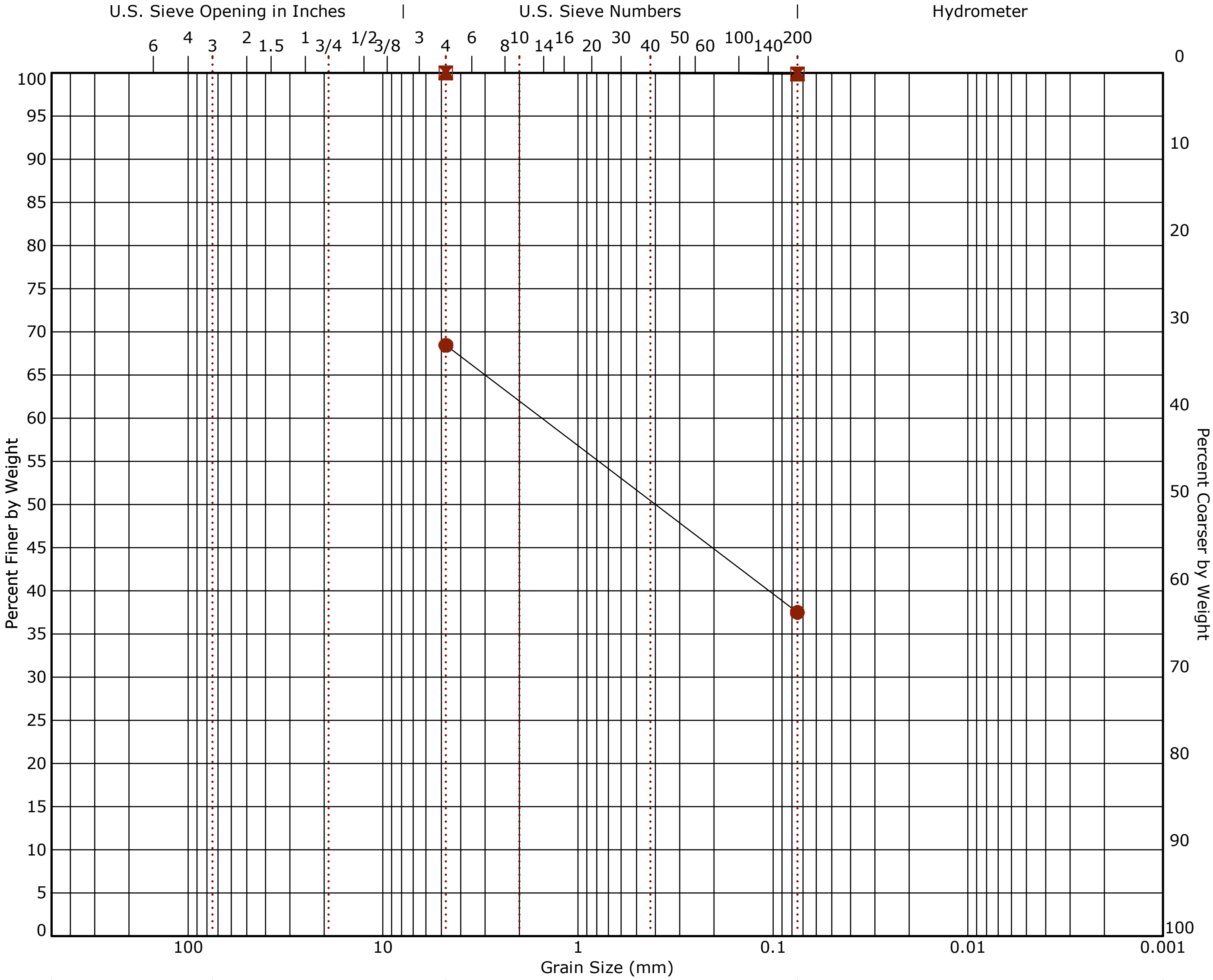
	●	⊠	▲
C_c			
C_u			

Terracon
6911 Blanco Rd
San Antonio, TX

Exhibit B-4

Grain Size Distribution

ASTM D422 / ASTM C136



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

	Boring ID	Depth	% Cobbles	% Gravel	% Sand	% Fines	% Silt	% Clay	USCS
●	B-2	17 - 18.5			30.9	37.5			GC-GM
⊠	B-2	23 - 24	0.0	0.0	0.1	99.9			CH
▲	B-2	44 - 45	0.0	0.0	0.1	99.9			CH

	Description
●	SILTY, CLAYEY GRAVEL with SAND
⊠	FAT CLAY
▲	FAT CLAY

	Remarks
●	
⊠	
▲	

● Sieve	● % Finer	⊠ Sieve	⊠ % Finer	▲ Sieve	▲ % Finer
#4	68.44	#4	100.0	#4	100.0
#200	37.51	#200	99.88	#200	99.86

Grain Size

	●	⊠	▲
D_{60}	1.532		
D_{30}			
D_{10}			

Coefficients

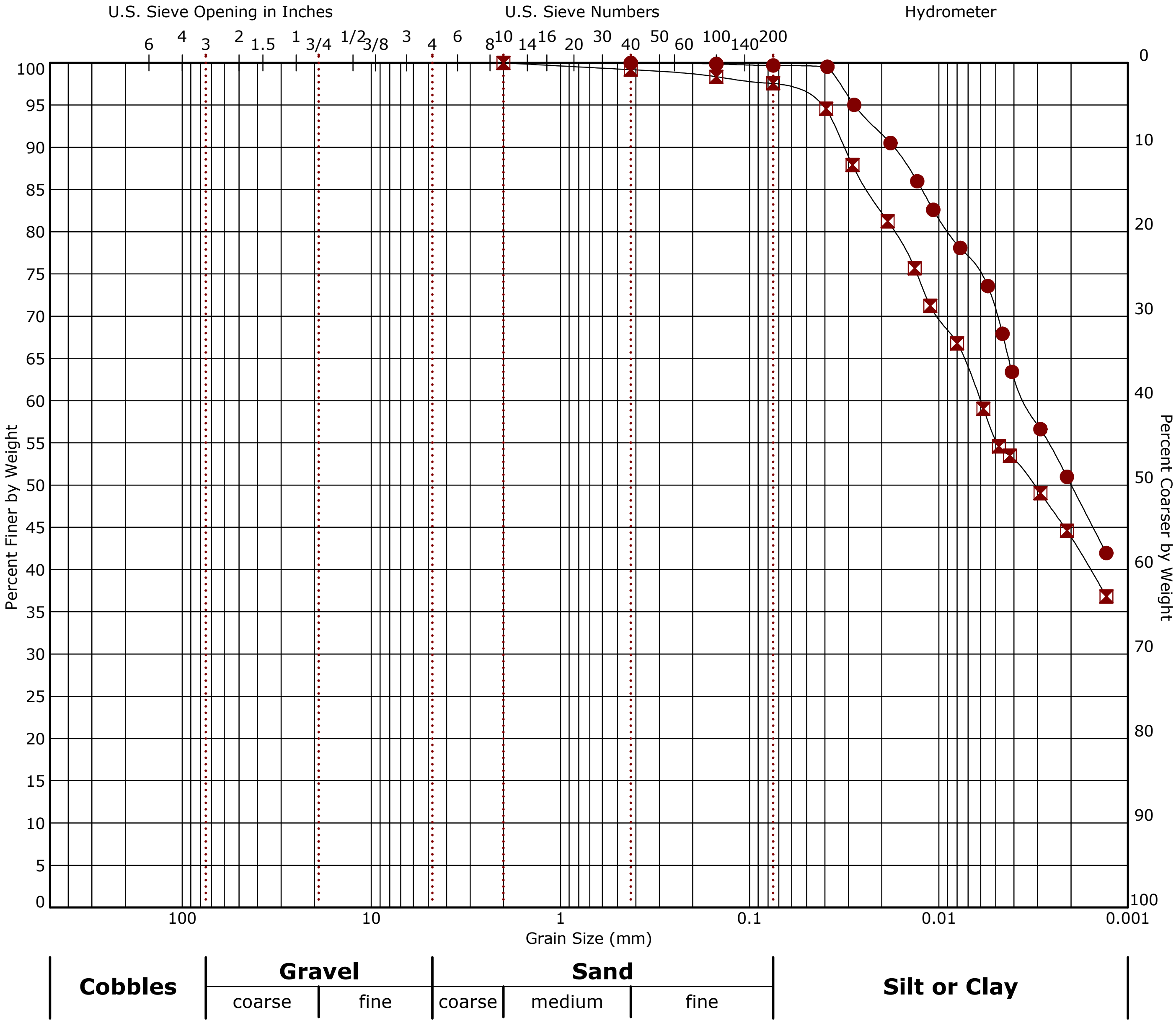
	●	⊠	▲
C_c			
C_u			

Terracon
6911 Blanco Rd
San Antonio, TX

Exhibit B-5

Grain Size Distribution

ASTM D422 / ASTM C136



	Boring ID	Depth (Ft)	USCS Classification	USCS	AASHTO	LL	PL	PI	Cc	Cu
●	B-1	18 - 20	FAT CLAY	CH	A-7-6 (55)	73	26	47		
⊠	B-2	34 - 35	FAT CLAY	CH	A-7-6 (50)	69	25	44		

	Boring ID	Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	B-1	18 - 20	0.425	0.003			0.0	0.0	0.3		29.1	70.6
⊠	B-2	34 - 35	2	0.006			0.0	0.0	2.4		42.1	55.5

CHEMICAL LABORATORY TEST REPORT

Project Number: 90235129
Service Date: 09/26/23
Report Date: 09/29/23

Terracon
10400 State Highway 191
Midland, Texas 79707
432-684-9600

Client

HDR Engineering, Inc.
613 Northwest Loop 410 Suite 700
San Antonio, TX 78216-5550

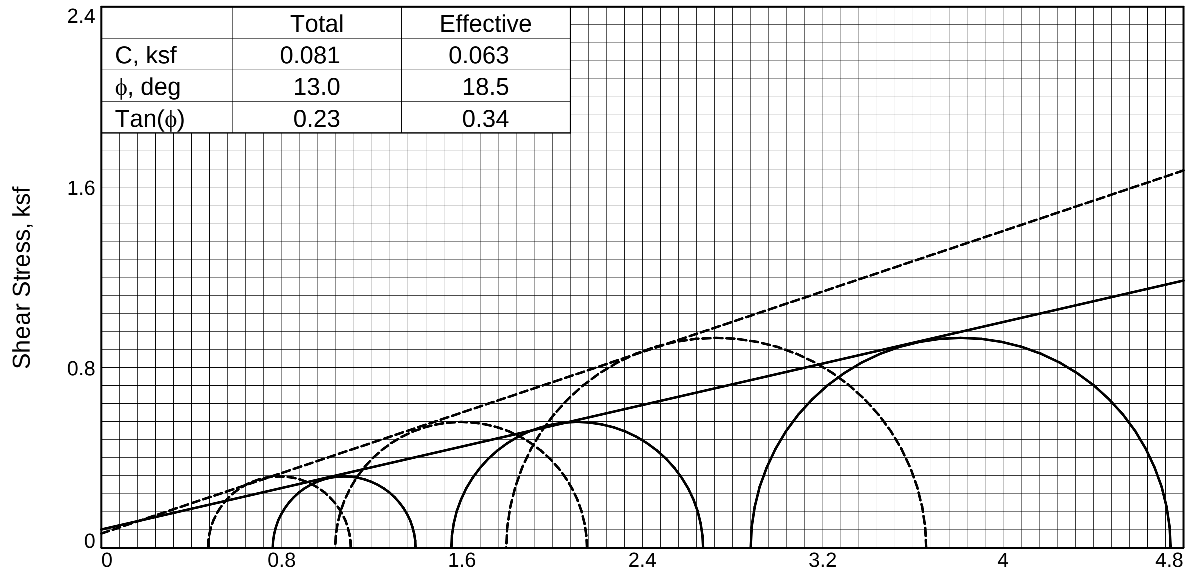
Project

Walnut Branch Creek Walls
313 West Nolte Street
Seguin, TX

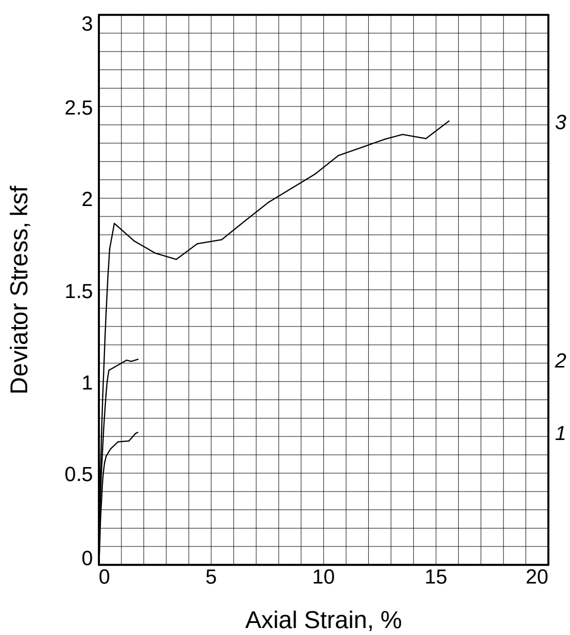
Exhibit B-6

Sample Location	B-1	B-2
Sample Depth (ft.)	5-6.5	28-29
pH Analysis, ASTM - G51-18	9.0	8.2
Water Soluble Sulfate (SO4), ASTM C 1580 (mg/kg)	138	380
Sulfides, ASTM - D4658-15, (mg/kg)	nil	nil
Chlorides, ASTM D 512 , (mg/kg)	63	81
RedOx, ASTM D-1498, (mV)	+513	+507
Total Salts, ASTM D1125-14, (mg/kg)	570	1,570
Resistivity, ASTM G187, (ohm-cm)	2,685	1,033

Analyzed By: Amata Cisse
Amata Cisse
Engineering Assistant



	Total	Effective
C, ksf	0.081	0.063
ϕ, deg	13.0	18.5
Tan(ϕ)	0.23	0.34



Sample No.		1	2	3
Initial	Water Content, %	32.1	32.1	32.1
	Dry Density, pcf	90.9	90.9	90.9
	Saturation, %	100.2	100.2	100.2
	Void Ratio	0.8746	0.8746	0.8746
	Diameter, in.	2.750	2.750	2.750
	Height, in.	6.050	6.050	6.050
At Test	Water Content, %	32.0	31.9	31.7
	Dry Density, pcf	91.0	91.1	91.4
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.8730	0.8701	0.8645
	Diameter, in.	2.749	2.772	2.794
	Height, in.	6.048	5.941	5.831
Strain rate, %/min.		0.0040	0.0040	0.0040
Back Pressure, psi		50.000	50.000	50.000
Cell Pressure, psi		55.280	60.780	70.000
Fail. Stress, ksf		0.633	1.116	1.863
Excess Pore Pr., ksf		0.287	0.515	1.084
Ult. Stress, ksf		0.633	1.116	1.863
Excess Pore Pr., ksf		0.287	0.515	1.084
$\bar{\sigma}_1$ Failure, ksf		1.106	2.154	3.658
$\bar{\sigma}_3$ Failure, ksf		0.473	1.037	1.796

Type of Test:
CU with Pore Pressures

Sample Type: Undisturbed

Description: Dark gray and tan Fat Clay

LL= 73 **PL=** 26 **PI=** 47

Assumed Specific Gravity= 2.73

Remarks: ASTM D4767 CU w/pore pressure

Exhibit B-7

Client:

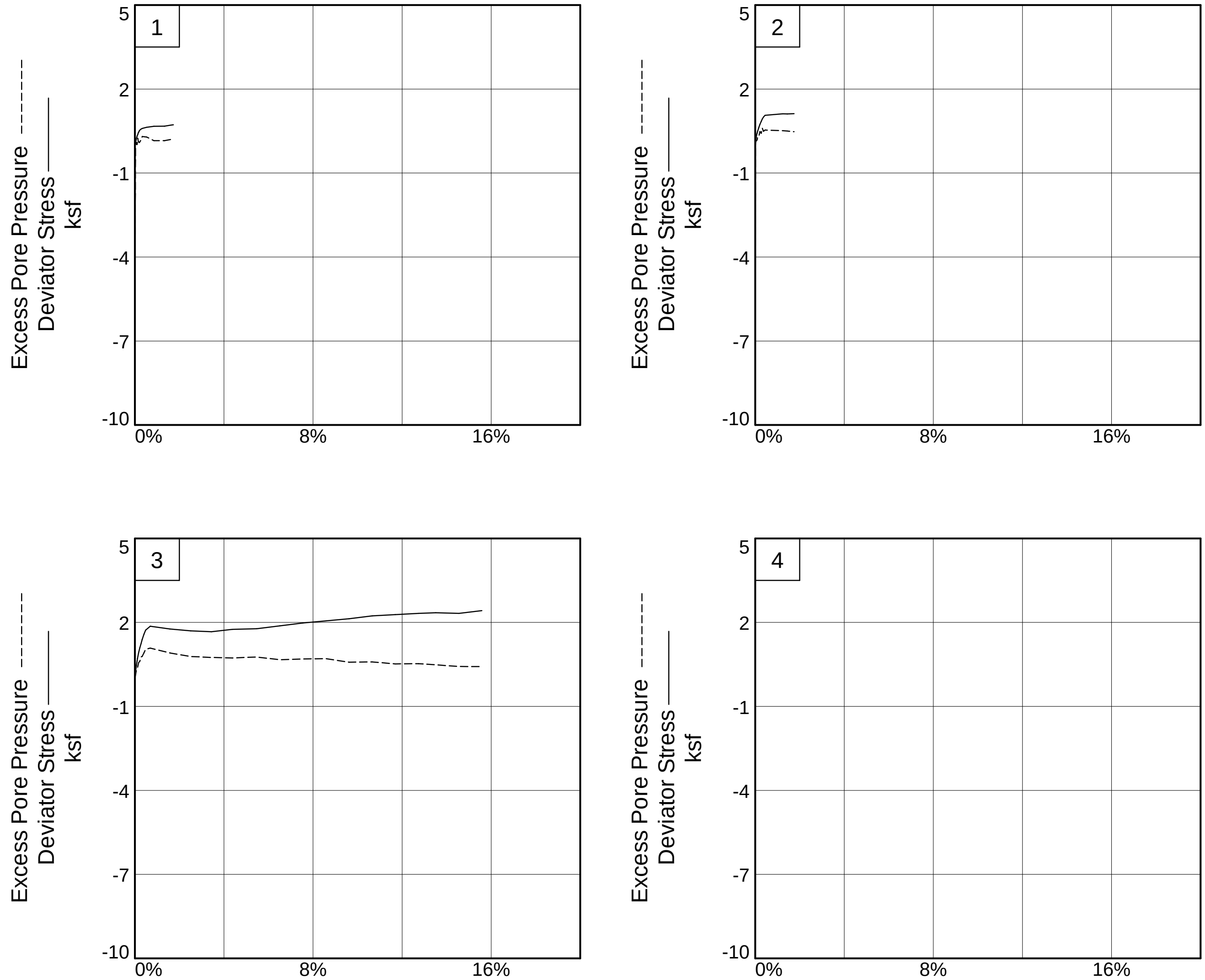
Project: Walnut Branch Creek Walls

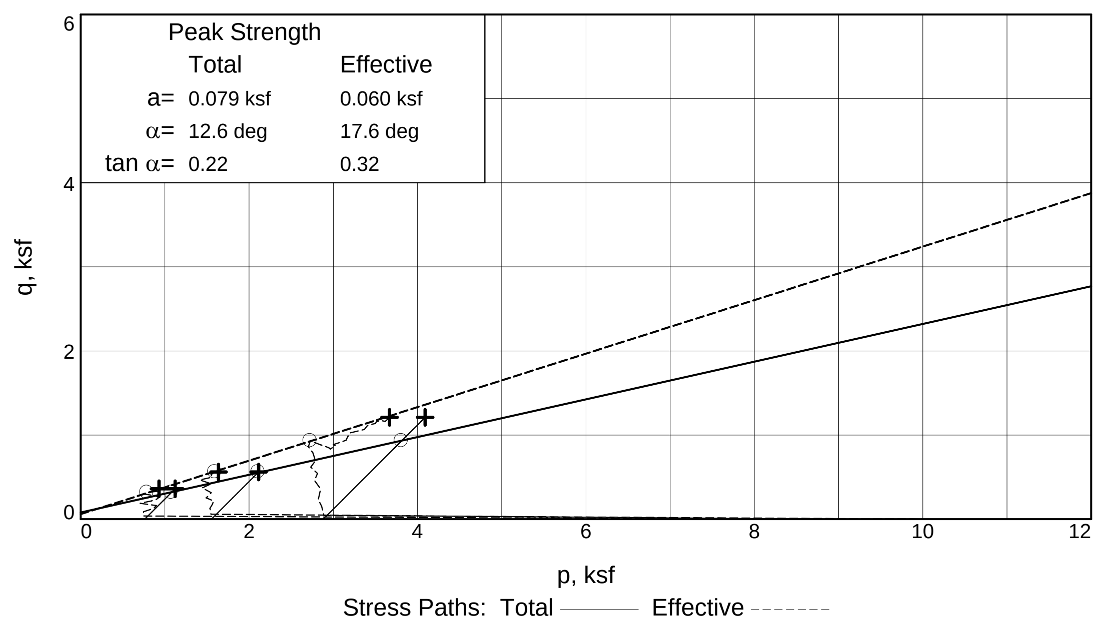
Location: B-1

Depth: 18-20 ft.

Proj. No.: 90235129 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Houston, TX





Client:

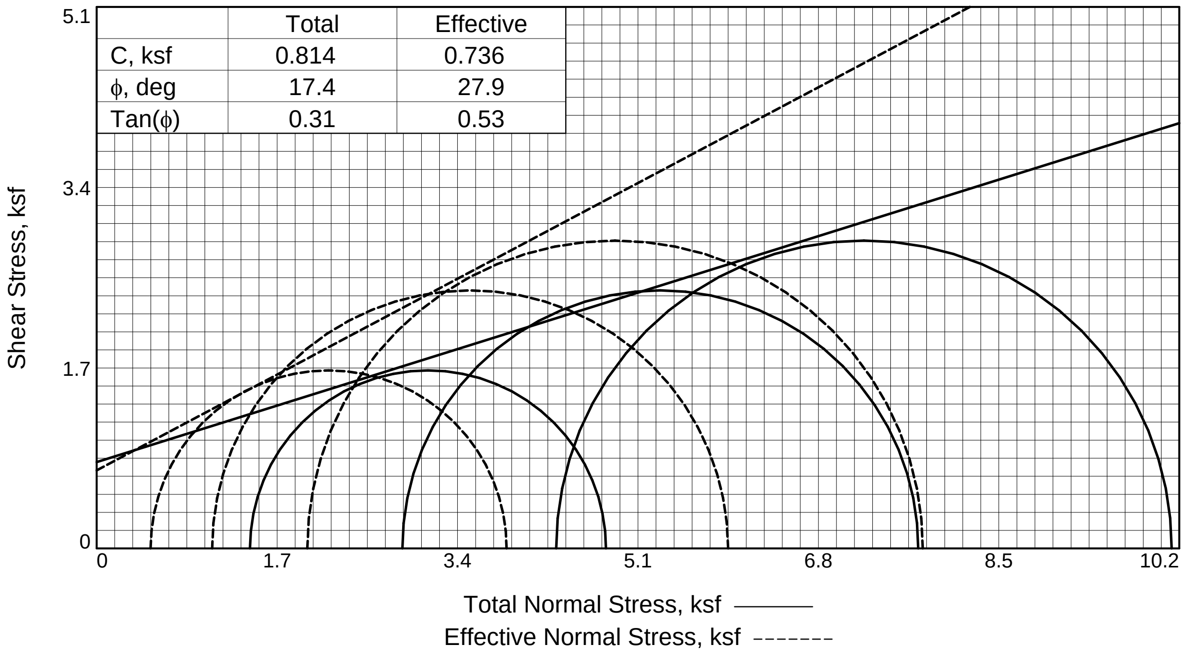
Project: Walnut Branch Creek Walls

Location: B-1 **Depth:** 18-20 ft.

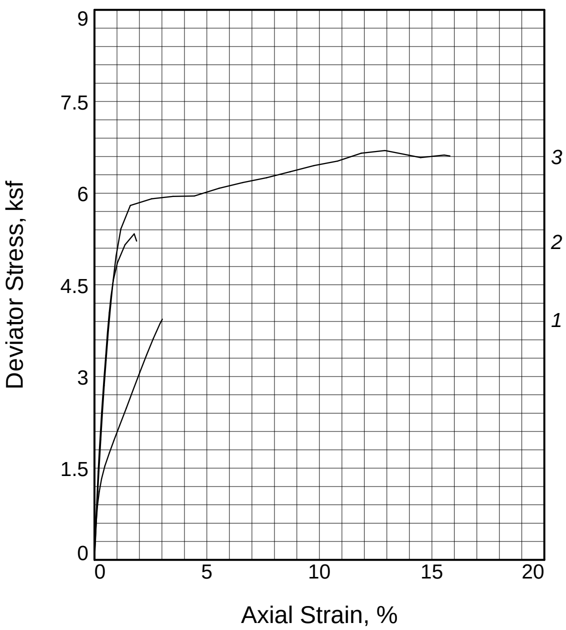
Project No.: 90235129

Exhibit B-8

Terracon Consultants, Inc.



	Total	Effective
C, ksf	0.814	0.736
ϕ, deg	17.4	27.9
Tan(ϕ)	0.31	0.53



Sample No.		1	2	3
Initial	Water Content, %	27.6	27.6	27.6
	Dry Density, pcf	95.8	95.8	95.8
	Saturation, %	96.7	96.7	96.7
	Void Ratio	0.7789	0.7789	0.7789
	Diameter, in.	2.763	2.763	2.763
	Height, in.	6.050	6.050	6.050
At Test	Water Content, %	28.5	28.4	28.2
	Dry Density, pcf	95.9	96.0	96.3
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.7775	0.7746	0.7704
	Diameter, in.	2.762	2.803	2.828
	Height, in.	6.048	5.863	5.748
Strain rate, %/min.		0.0040	0.0040	0.0040
Back Pressure, psi		60.000	60.000	60.000
Cell Pressure, psi		70.030	80.000	90.070
Fail. Stress, ksf		3.354	4.861	5.799
Excess Pore Pr., ksf		0.936	1.792	2.345
Ult. Stress, ksf		3.354	4.861	5.799
Excess Pore Pr., ksf		0.936	1.792	2.345
$\bar{\sigma}_1$ Failure, ksf		3.862	5.949	7.784
$\bar{\sigma}_3$ Failure, ksf		0.508	1.088	1.985

Type of Test:
CU with Pore Pressures

Sample Type: Undisturbed

Description: Dark gray Fat Clay w/gypsum

LL= 69 **PL=** 25 **PI=** 44

Assumed Specific Gravity= 2.73

Remarks: ASTM D4767 CU w/pore pressure

Exhibit B-9

Client:

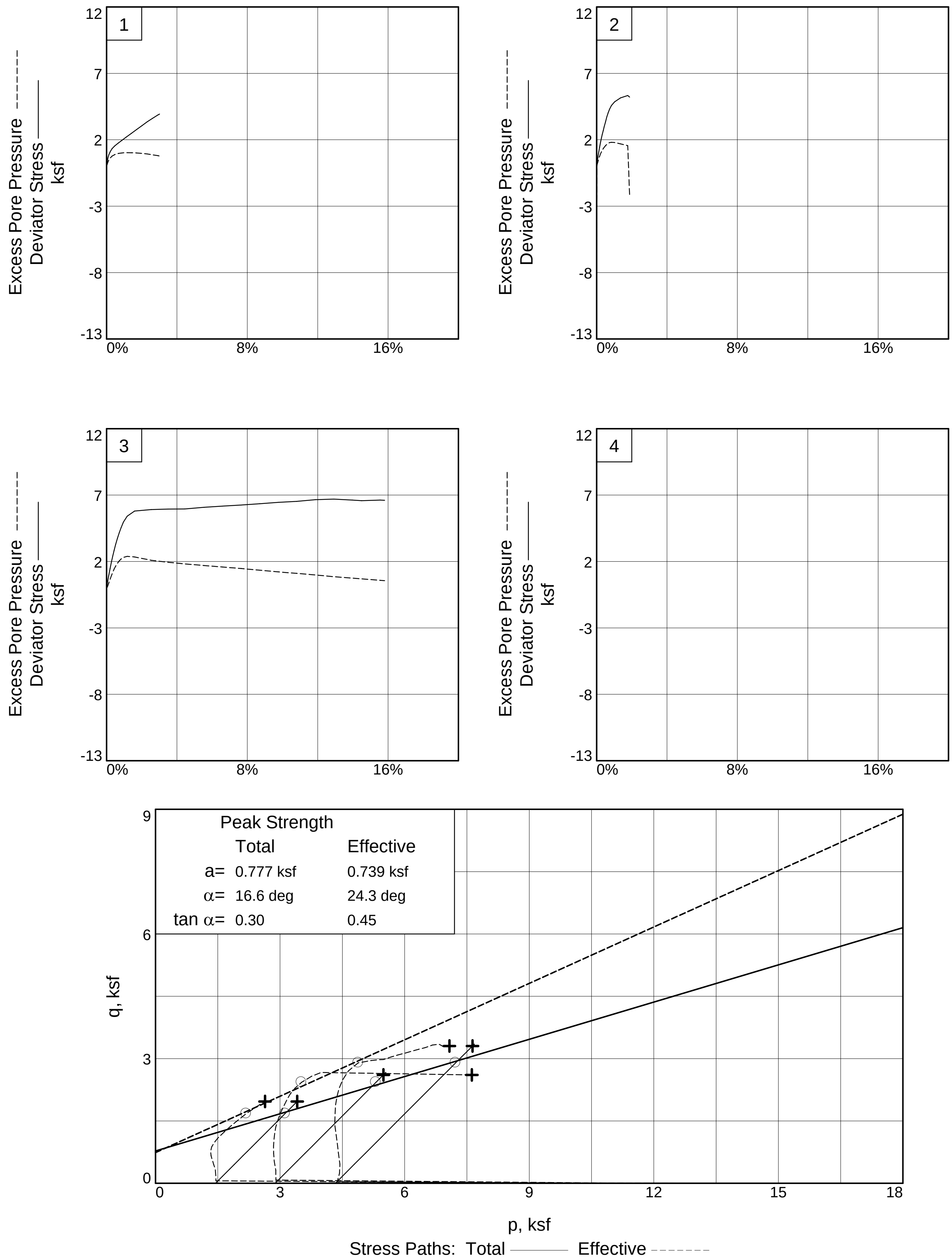
Project: Walnut Branch Creek Walls

Location: B-2

Depth: 34-35 ft.

Proj. No.: 90235129 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Houston, TX



Client:

Project: Walnut Branch Creek Walls

Location: B-2 **Depth:** 34-35 ft.

Project No.: 90235129

Exhibit B-10

Terracon Consultants, Inc.

APPENDIX C

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

SAMPLING

Auger	Split Spoon
Shelby Tube	Macro Core
Ring Sampler	Rock Core
Grab Sample	No Recovery

WATER LEVEL

- Water Initially Encountered
- Water Level After a Specified Period of Time
- Water Level After a Specified Period of Time

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

FIELD TESTS

(HP)	Hand Penetrometer
(T)	Torvane
(b/f)	Standard Penetration Test (blows per foot)
(PID)	Photo-Ionization Detector
(OVA)	Organic Vapor Analyzer

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance Includes gravels, sands and silts.			CONSISTENCY OF FINE-GRAINED SOILS (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance			
Descriptive Term (Density)	**Standard Penetration or N-Value Blows/Ft.**	**Ring Sampler Blows/Ft.**	**Descriptive Term (Consistency)**	**Unconfined Compressive Strength, Qu, tsf**	**Standard Penetration or N-Value Blows/Ft.**	**Ring Sampler Blows/Ft.**
Very Loose	0 - 3	0 - 6	Very Soft	less than 0.25	0 - 1	< 3
Loose	4 - 9	7 - 18	Soft	0.25 to 0.50	2 - 4	3 - 4
Medium Dense	10 - 29	19 - 58	Medium-Stiff	0.50 to 1.00	4 - 8	5 - 9
Dense	30 - 50	59 - 98	Stiff	1.00 to 2.00	8 - 15	10 - 18
Very Dense	> 50	$\geq$ 99	Very Stiff	2.00 to 4.00	15 - 30	19 - 42
			Hard	> 4.00	> 30	> 42

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 - 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300 mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm
Silt or Clay	Passing #200 sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 - 12
Modifier	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1 - 10
Medium	11 - 30
High	> 30

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	**Group Name** [B]
Coarse Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [E]	GW	Well-graded gravel [F]
			$Cu < 4$ and/or $1 > Cc > 3$ [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel [F,G,H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [E]	SW	Well-graded sand [I]
			$Cu < 6$ and/or $1 > Cc > 3$ [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G,H,I]
			Fines classify as CL or CH	SC	Clayey sand [G,H,I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	$PI > 7$ and plots on or above "A" line [J]	CL	Lean clay [K,L,M]
			$PI < 4$ or plots below "A" line [J]	ML	Silt [K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay [K,L,M,N]
					Organic silt [K,L,M,O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K,L,M]
			PI plots below "A" line	MH	Elastic Silt [K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay [K,L,M,P]
					Organic silt [K,L,M,Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains $\geq$ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains $\geq$ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains $\geq$ 30% plus No. 200 predominantly sand, add "sandy" to group name.

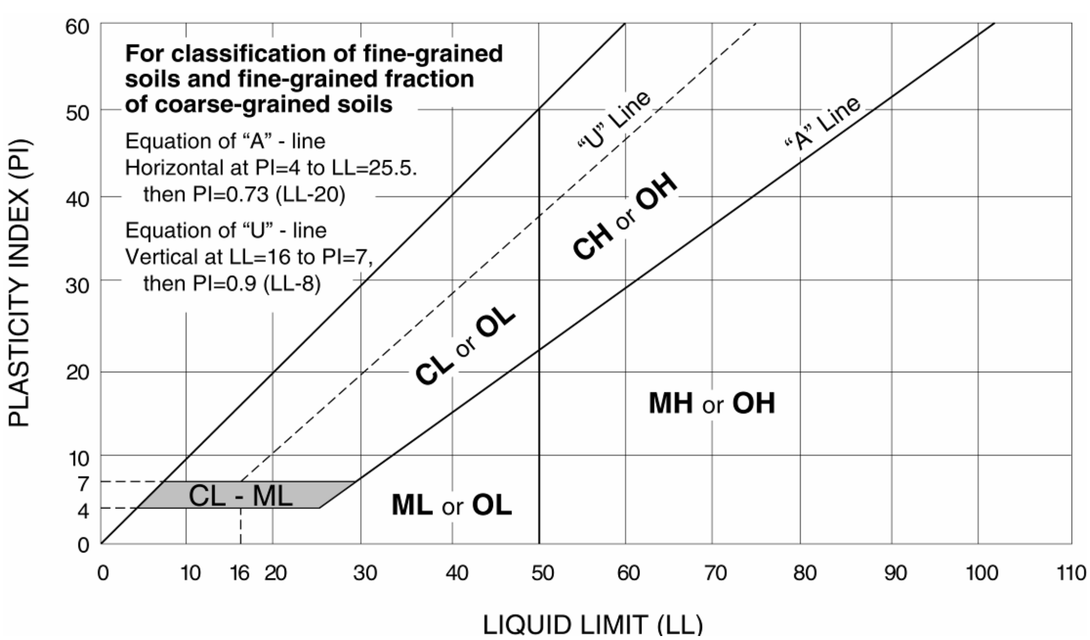
[M] If soil contains $\geq$ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] $PI \geq 4$ and plots on or above "A" line.

[O] $PI < 4$ or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.

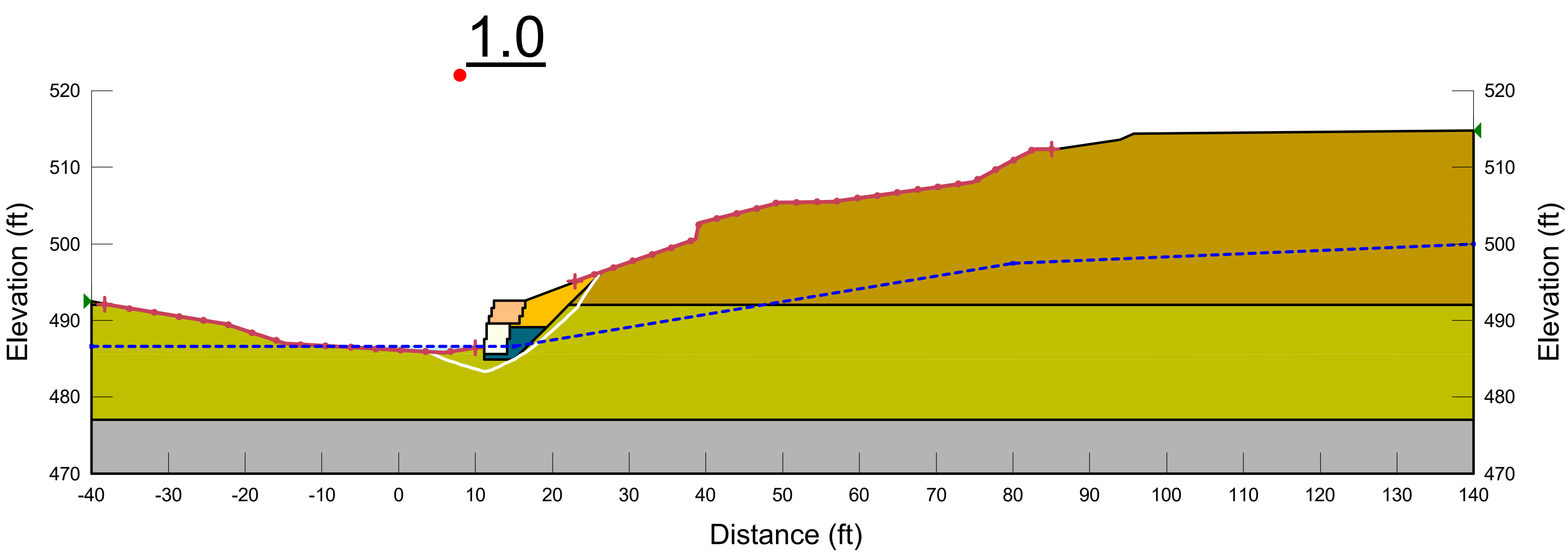


Terracon

Exhibit C-2

Appendix E. Preliminary Geotechnical Evaluations

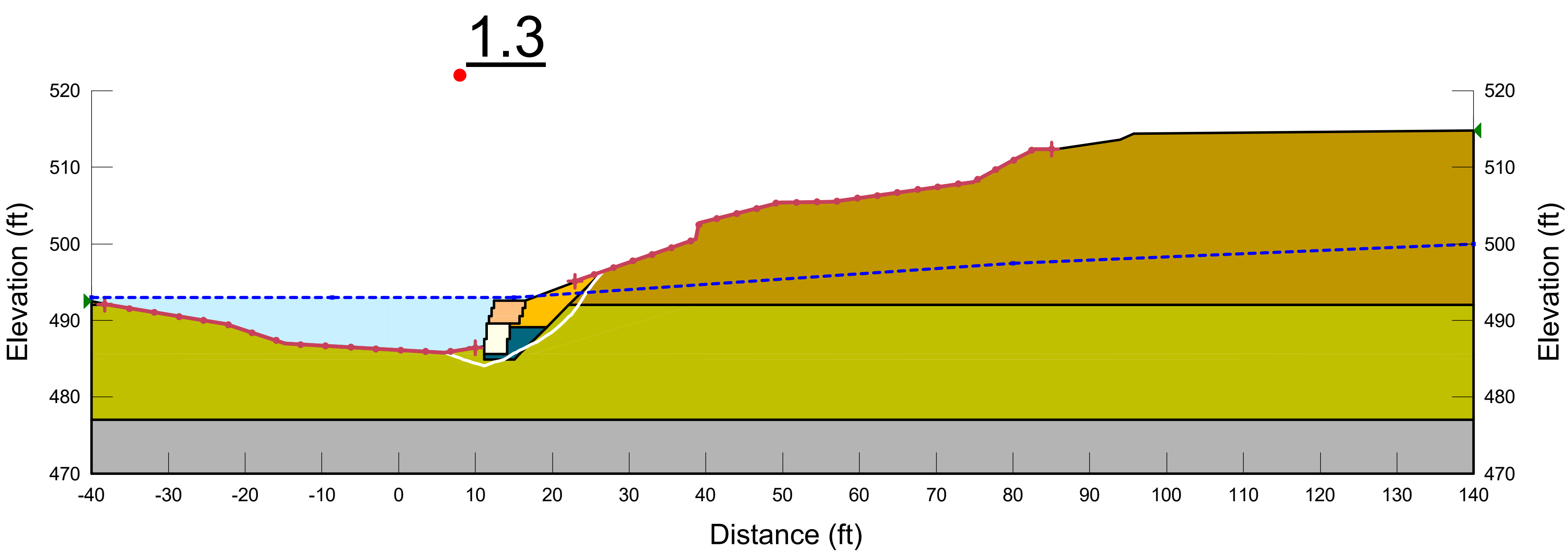
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Ballast Rock	Mohr-Coulomb	125	0	36	0	1
	CH Drained	Mohr-Coulomb	120	50	20	0	1
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 1	Mohr-Coulomb	120	0	32	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1
	Wrapped Soil Blocks	High Strength	120				1



Site 1. Existing wall, slope stability under drained conditions at normal water level.

Plate E-1

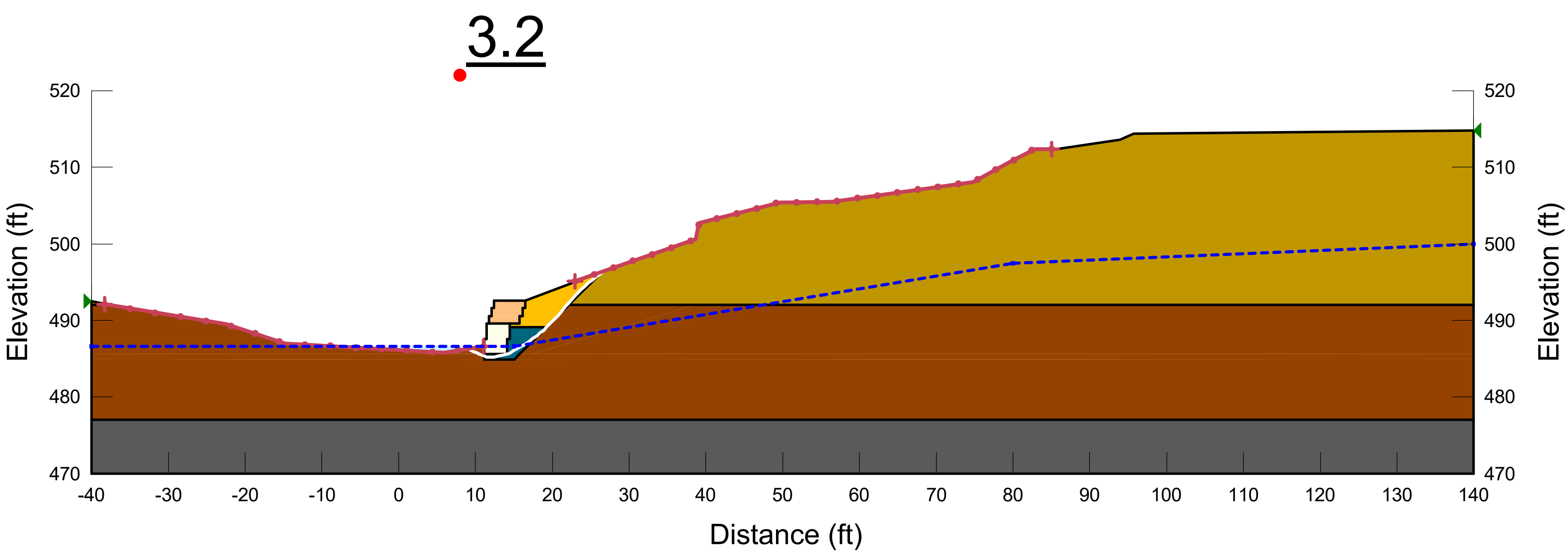
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Ballast Rock	Mohr-Coulomb	125	0	36	0	1
	CH Drained	Mohr-Coulomb	120	50	20	0	1
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 1	Mohr-Coulomb	120	0	32	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1
	Wrapped Soil Blocks	High Strength	120				1



Site 1. Existing wall, slope stability under drained conditions at flood water level.

Plate E-2

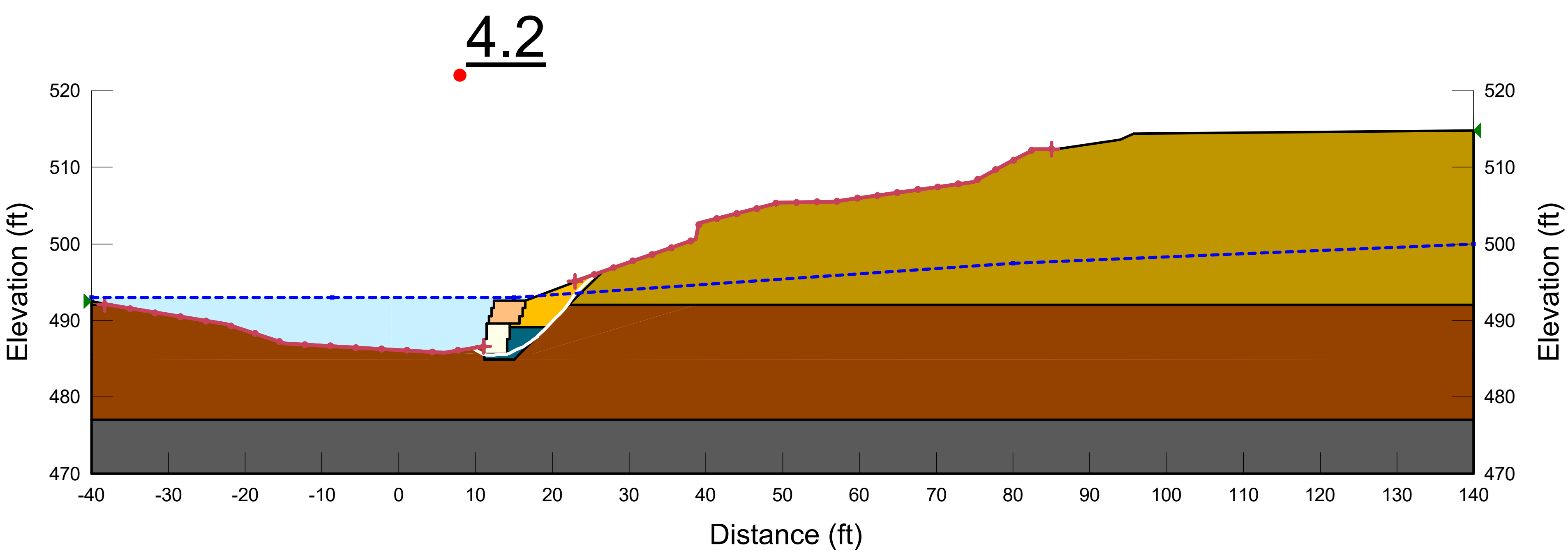
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Total Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Ballast Rock	Mohr-Coulomb	125	0	36	0	1
	CH Undrained	Mohr-Coulomb	120	2,000	0	0	1
	Clayshale Undrained	Mohr-Coulomb	130	4,000	0	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 1	Mohr-Coulomb	120	0	32	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1
	Wrapped Soil Blocks	High Strength	120				1



Site 1. Existing wall, slope stability under undrained conditions at normal water level.

Plate E-3

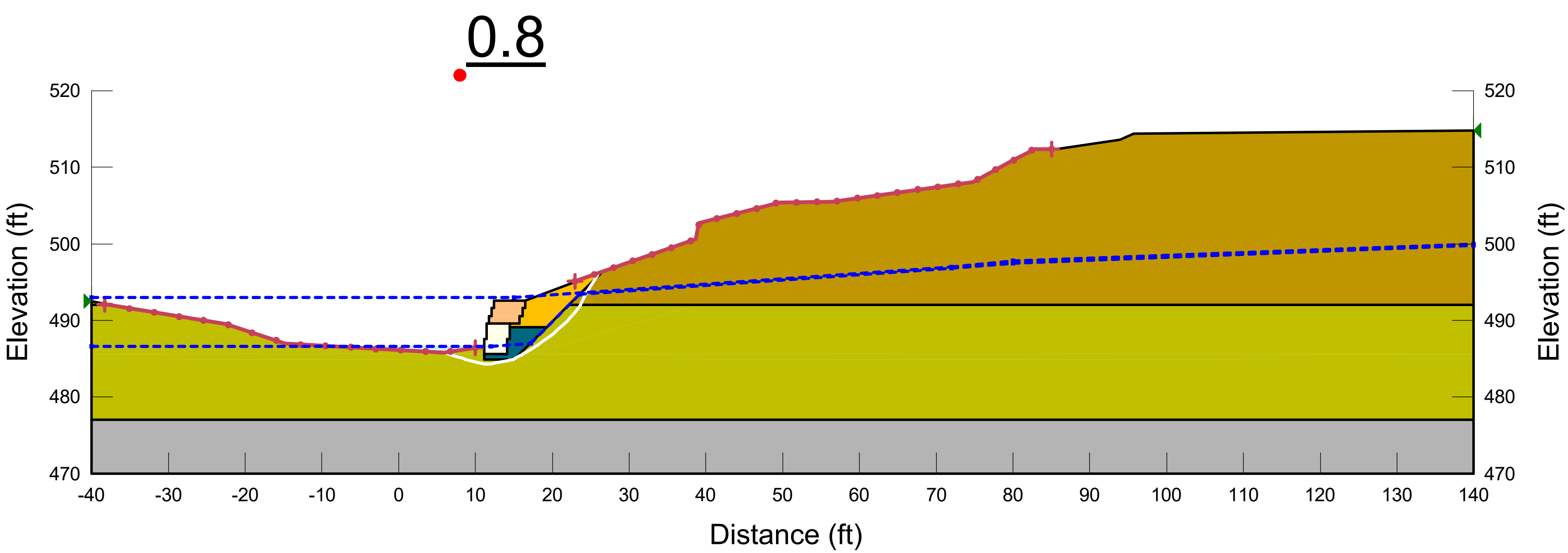
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Total Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Ballast Rock	Mohr-Coulomb	125	0	36	0	1
	CH Undrained	Mohr-Coulomb	120	2,000	0	0	1
	Clayshale Undrained	Mohr-Coulomb	130	4,000	0	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 1	Mohr-Coulomb	120	0	32	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1
	Wrapped Soil Blocks	High Strength	120				1



Site 1. Existing wall, slope stability under undrained conditions at flood water level.

Plate E-4

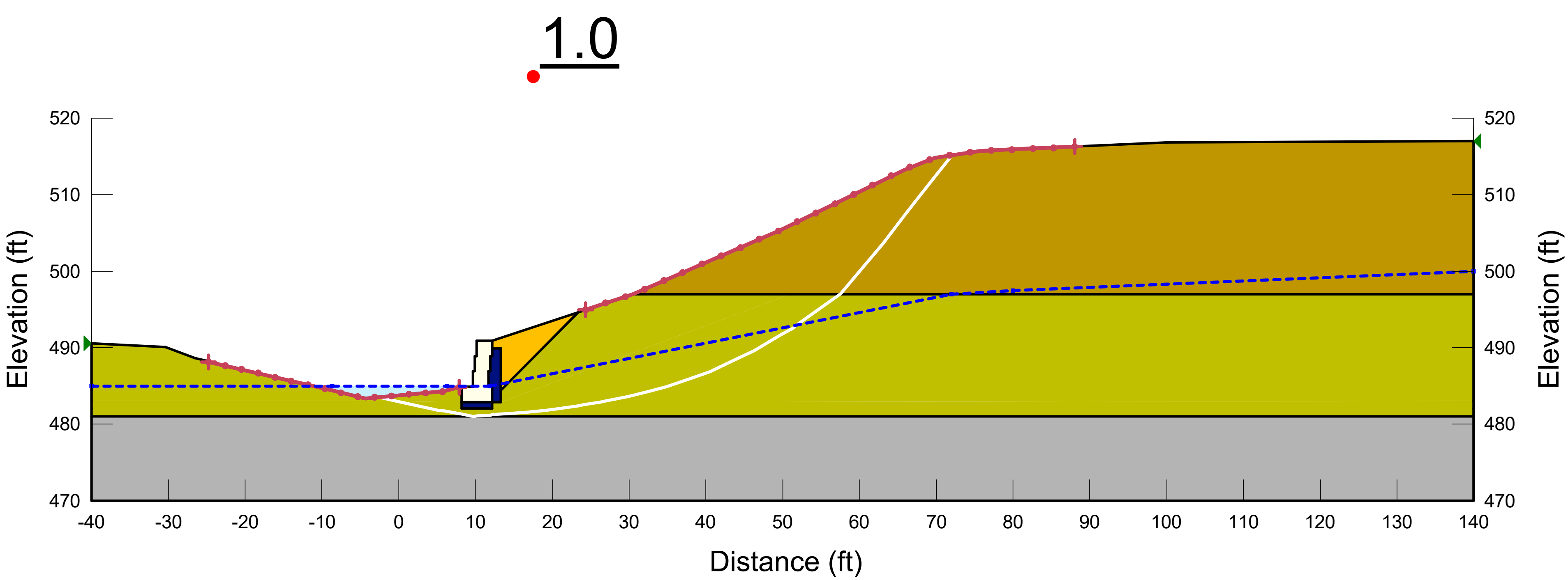
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)	Piezometric Surface	Piezometric Surface After Drawdown
	Ballast Rock	Mohr-Coulomb	125	0	36	0	10	35	1	2
	CH Drained	Mohr-Coulomb	120	50	20	0	100	15	1	2
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	800	16	1	2
	Limestone Blocks	High Strength	150						1	2
	SC/GC Site 1	Mohr-Coulomb	120	0	32	0	0	0	1	2
	Structural Backfill	Mohr-Coulomb	120	0	32	0	0	0	1	2
	Wrapped Soil Blocks	High Strength	120						1	2



Site 1. Existing wall, slope stability under rapid drawdown conditions

Plate E-5

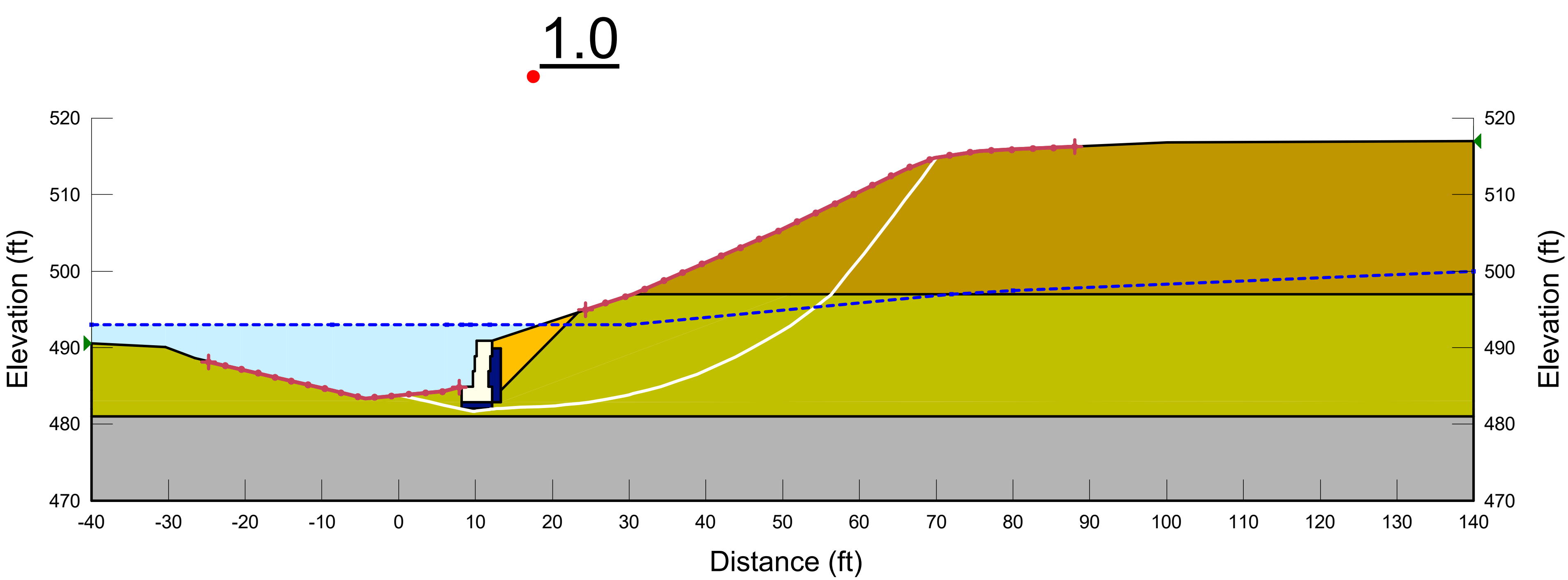
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Drained	Mohr-Coulomb	120	50	20	0	1
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	1
	Crushed Stone	Mohr-Coulomb	125	0	34	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 2	Mohr-Coulomb	120	0	34	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 2. Existing wall, slope stability under drained conditions at normal water level.

Plate E-6

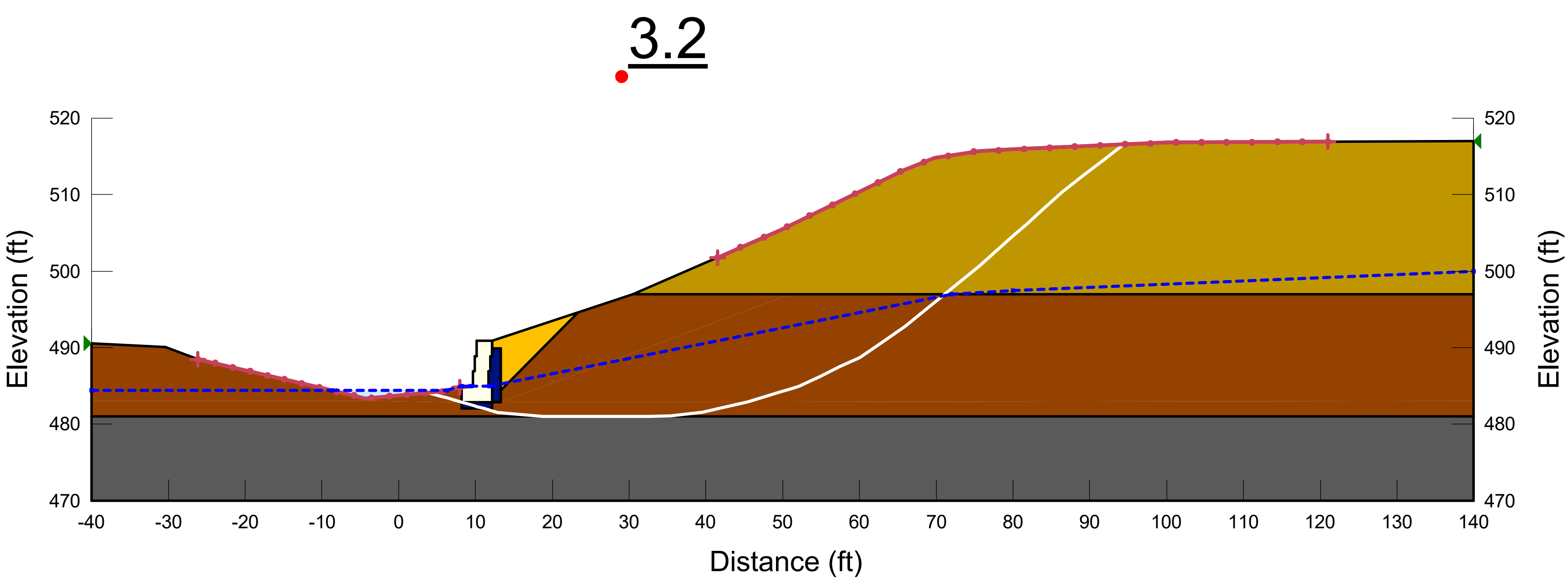
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Drained	Mohr-Coulomb	120	50	20	0	1
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	1
	Crushed Stone	Mohr-Coulomb	125	0	34	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 2	Mohr-Coulomb	120	0	34	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 2. Existing wall, slope stability under drained conditions at flood water level.

Plate E-7

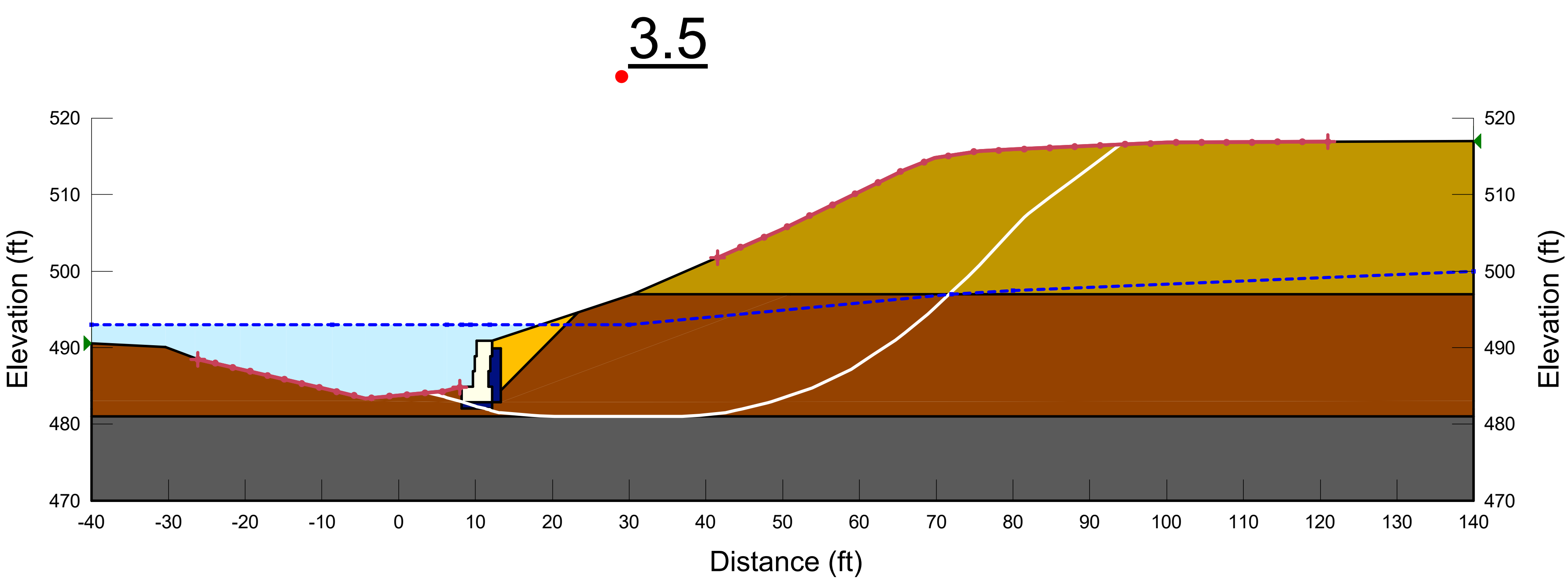
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Total Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Undrained	Mohr-Coulomb	120	2,000	0	0	1
	Clayshale Undrained	Mohr-Coulomb	130	4,000	0	0	1
	Crushed Stone	Mohr-Coulomb	125	0	34	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 2	Mohr-Coulomb	120	0	34	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 2. Existing wall, slope stability under undrained conditions at normal water level.

Plate E-8

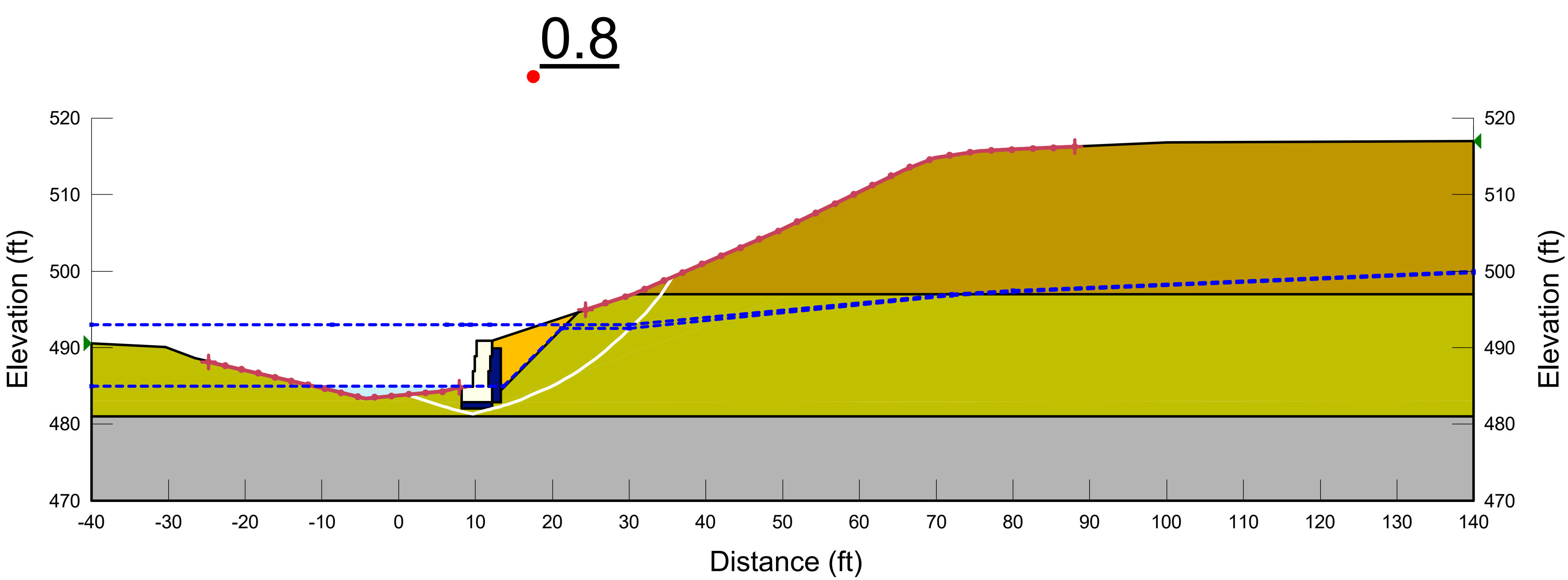
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Total Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Undrained	Mohr-Coulomb	120	2,000	0	0	1
	Clayshale Undrained	Mohr-Coulomb	130	4,000	0	0	1
	Crushed Stone	Mohr-Coulomb	125	0	34	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 2	Mohr-Coulomb	120	0	34	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 2. Existing wall, slope stability under undrained conditions at flood water level.

Plate E-9

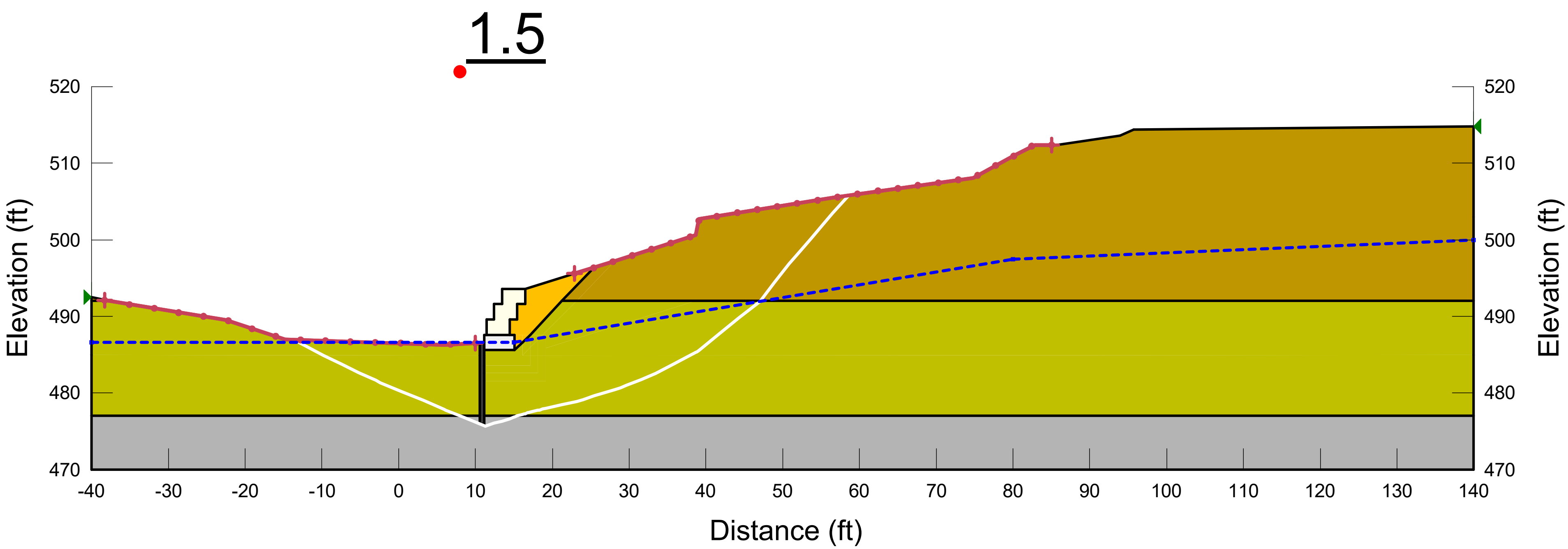
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)	Piezometric Surface	Piezometric Surface After Drawdown
	CH Drained	Mohr-Coulomb	120	50	20	0	100	15	1	2
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	800	16	1	2
	Crushed Stone	Mohr-Coulomb	125	0	34	0	20	33	1	2
	Limestone Blocks	High Strength	150						1	2
	SC/GC Site 2	Mohr-Coulomb	120	0	34	0	0	0	1	2
	Structural Backfill	Mohr-Coulomb	120	0	32	0	0	0	1	2



Site 2. Existing wall, slope stability under rapid drawdown conditions.

Plate E-10

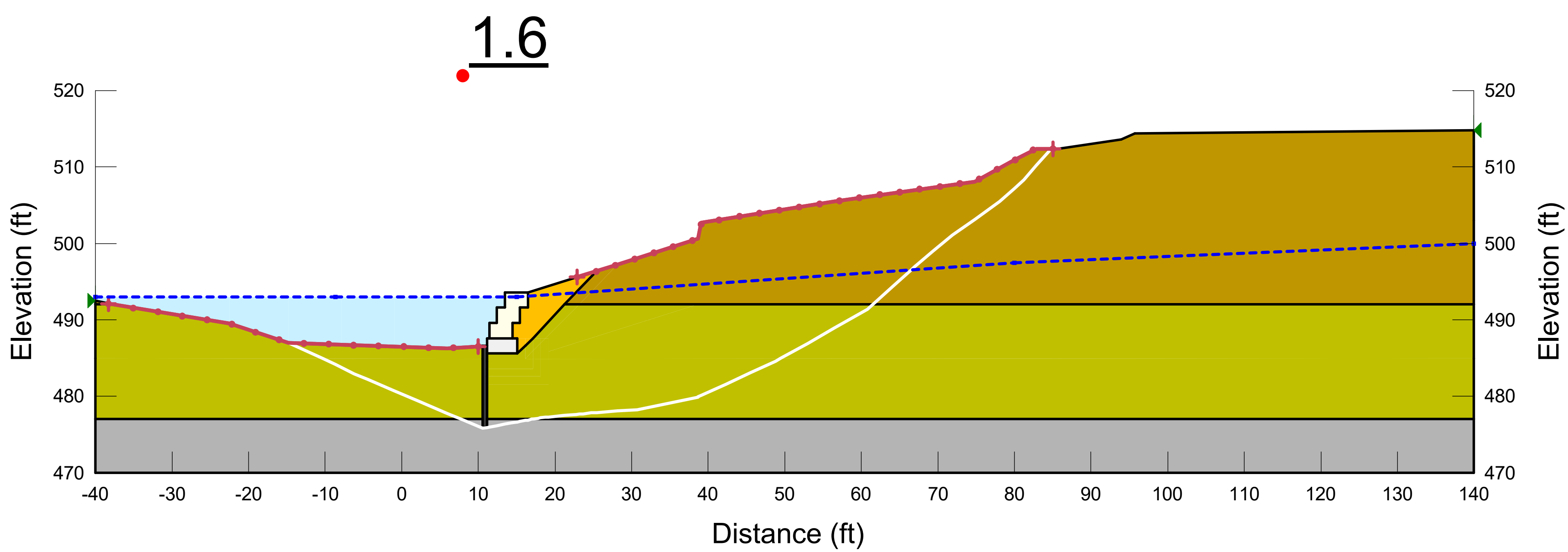
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Drained	Mohr-Coulomb	120	50	20	0	1
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	1
	Concrete	High Strength	150				1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 1	Mohr-Coulomb	120	0	32	0	1
	Steel	High Strength	200				1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 1. Proposed wall, slope stability under drained conditions at normal water level.

Plate E-11

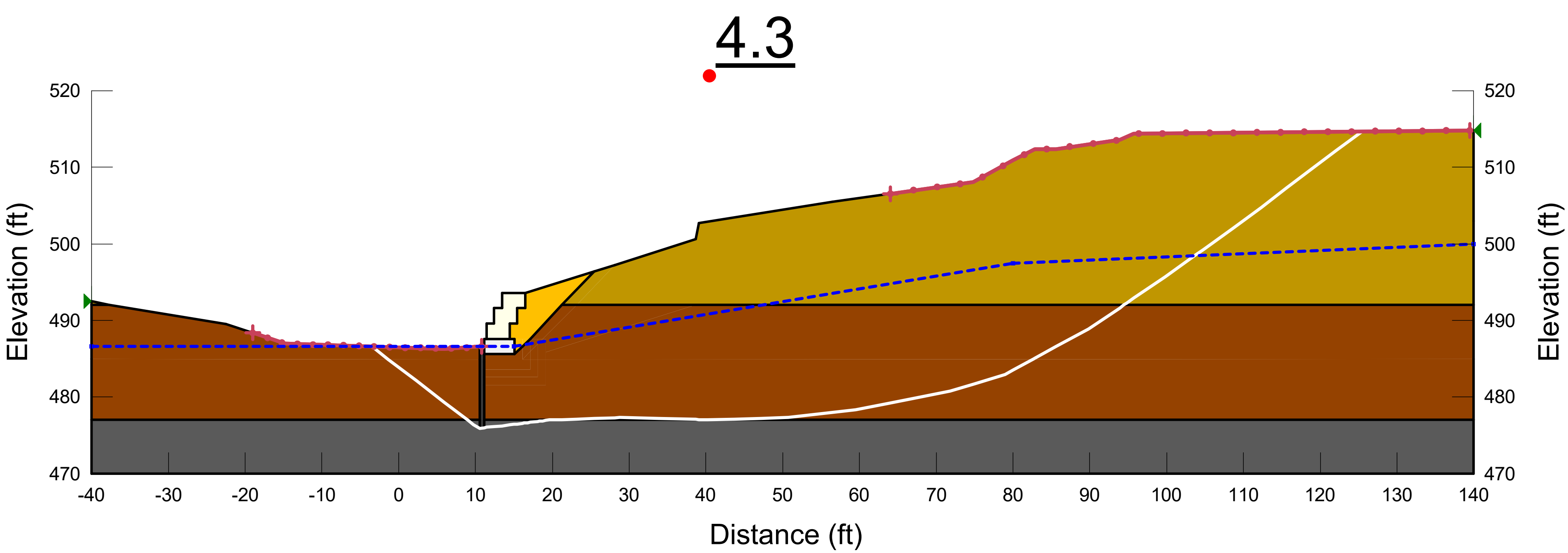
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Drained	Mohr-Coulomb	120	50	20	0	1
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	1
	Concrete	High Strength	150				1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 1	Mohr-Coulomb	120	0	32	0	1
	Steel	High Strength	200				1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 1. Proposed wall, slope stability under drained conditions at flood water level.

Plate E-12

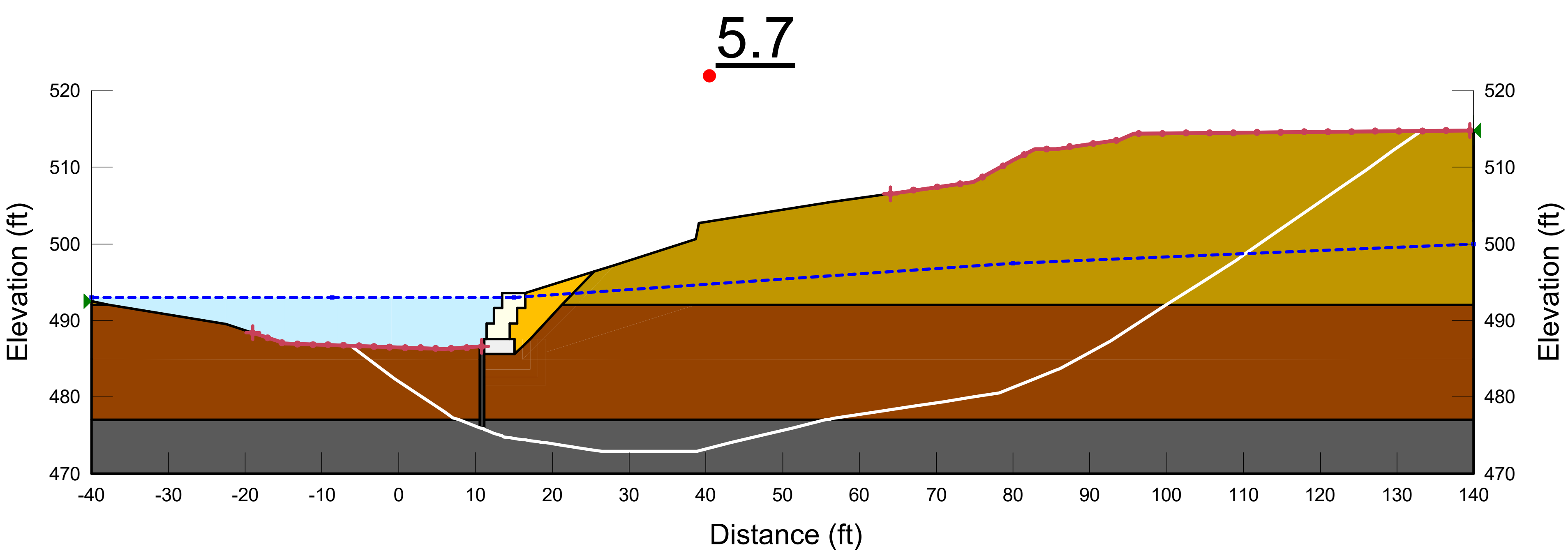
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Total Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Undrained	Mohr-Coulomb	120	2,000	0	0	1
	Clayshale Undrained	Mohr-Coulomb	130	4,000	0	0	1
	Concrete	High Strength	150				1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 1	Mohr-Coulomb	120	0	32	0	1
	Steel	High Strength	200				1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 1. Proposed wall, slope stability under undrained conditions at normal water level.

Plate E-13

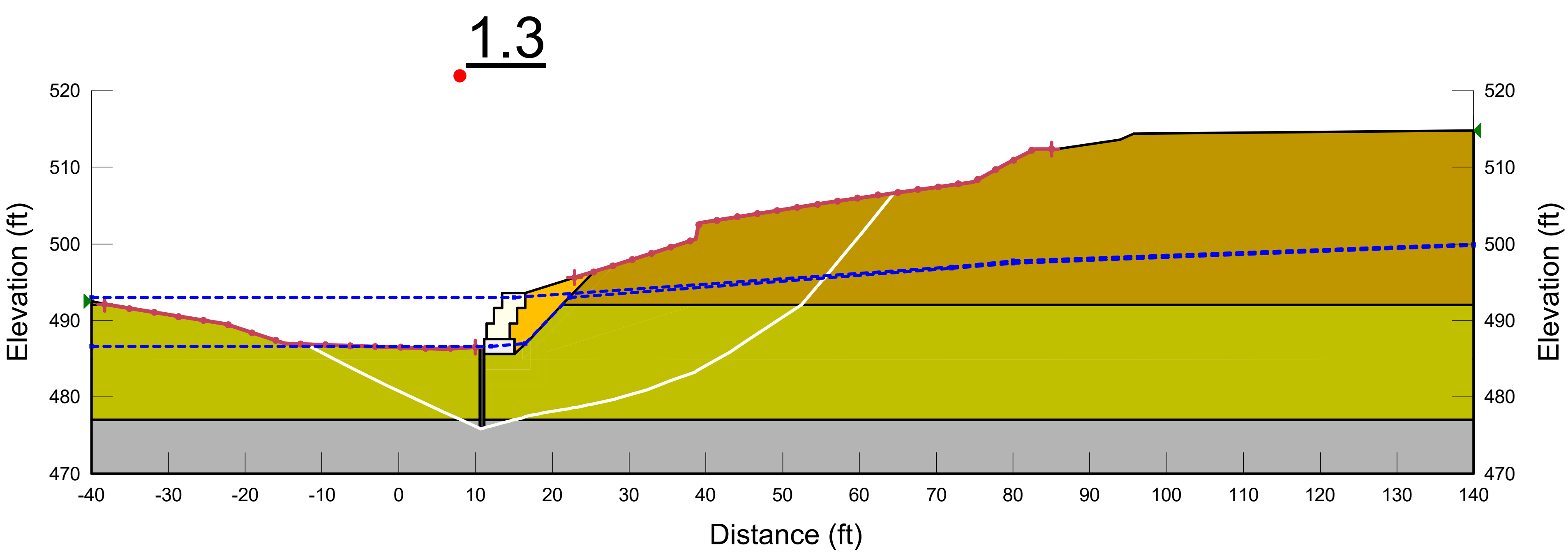
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Total Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Undrained	Mohr-Coulomb	120	2,000	0	0	1
	Clayshale Undrained	Mohr-Coulomb	130	4,000	0	0	1
	Concrete	High Strength	150				1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 1	Mohr-Coulomb	120	0	32	0	1
	Steel	High Strength	200				1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 1. Proposed wall, slope stability under undrained conditions at flood water level.

Plate E-14

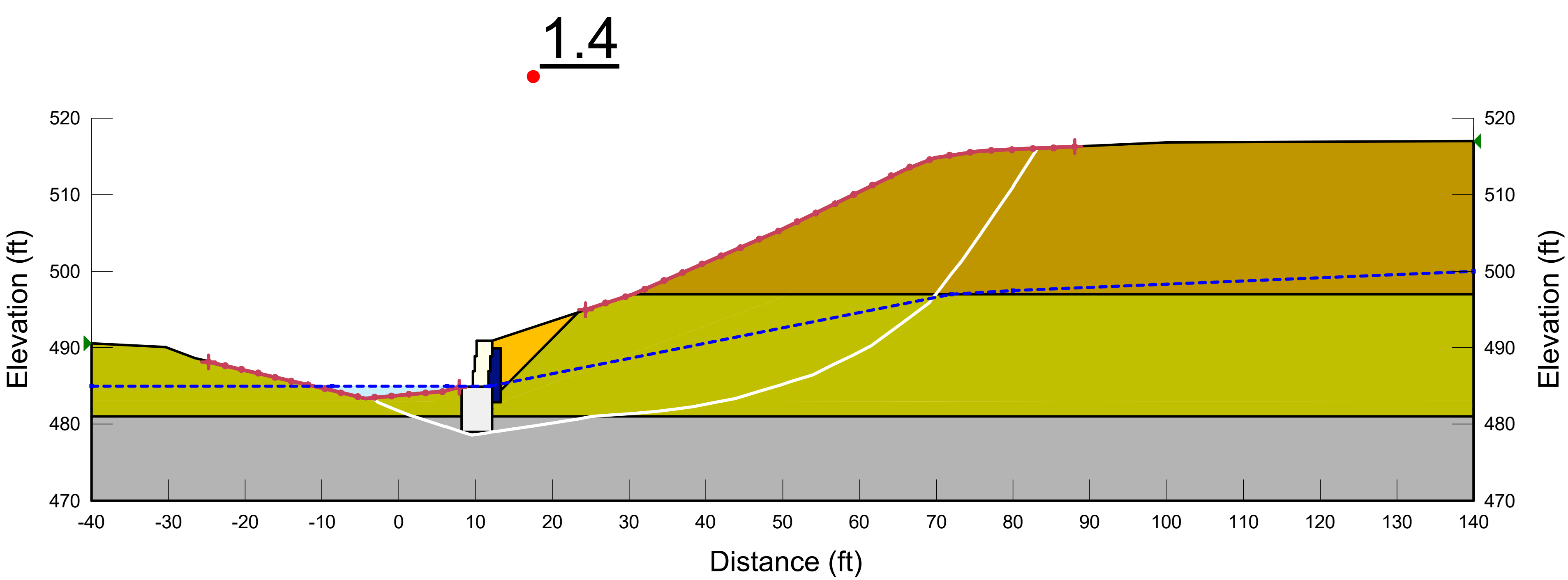
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)	Piezometric Surface	Piezometric Surface After Drawdown
	CH Drained	Mohr-Coulomb	120	50	20	0	100	15	1	2
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	800	16	1	2
	Concrete	High Strength	150						1	2
	Limestone Blocks	High Strength	150						1	2
	SC/GC Site 1	Mohr-Coulomb	120	0	32	0	0	0	1	2
	Steel	High Strength	200						1	2
	Structural Backfill	Mohr-Coulomb	120	0	32	0	0	0	1	2



Site 1. Proposed wall, slope stability under rapid drawdown conditions.

Plate E-15

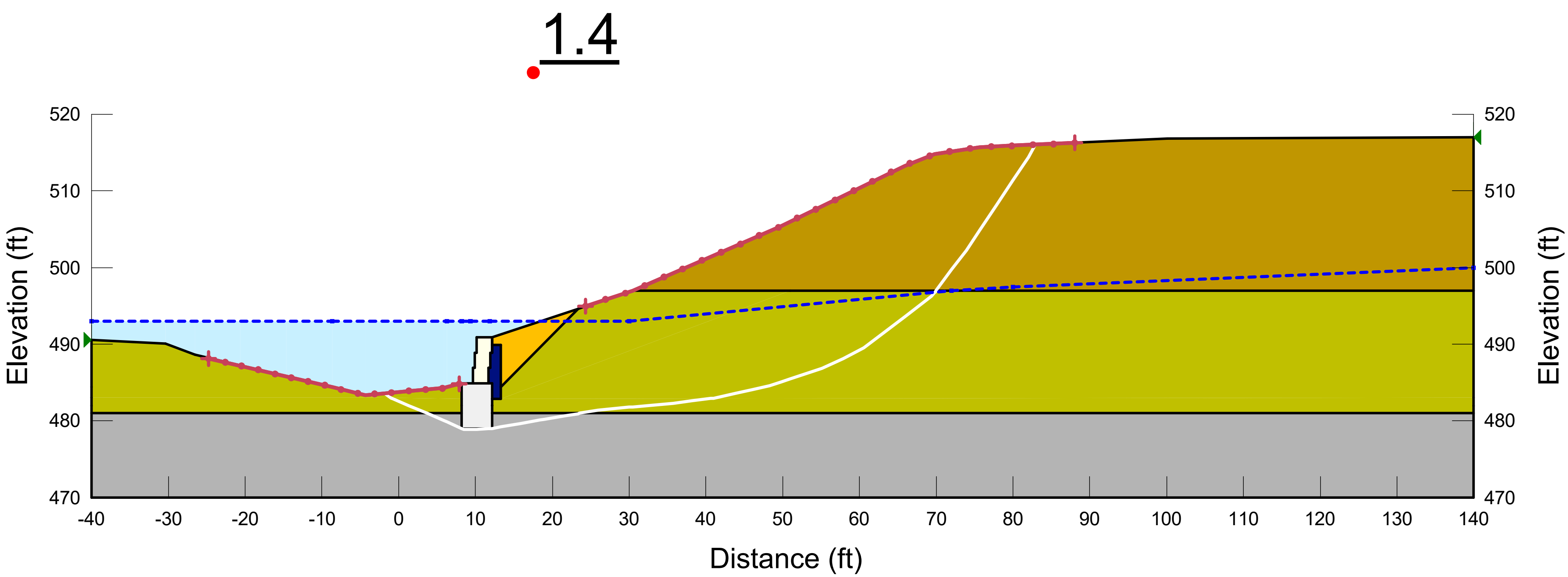
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Drained	Mohr-Coulomb	120	50	20	0	1
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	1
	Concrete	High Strength	150				1
	Crushed Stone	Mohr-Coulomb	125	0	34	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 2	Mohr-Coulomb	120	0	34	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 2. Proposed wall, slope stability under drained conditions at normal water level.

Plate E-16

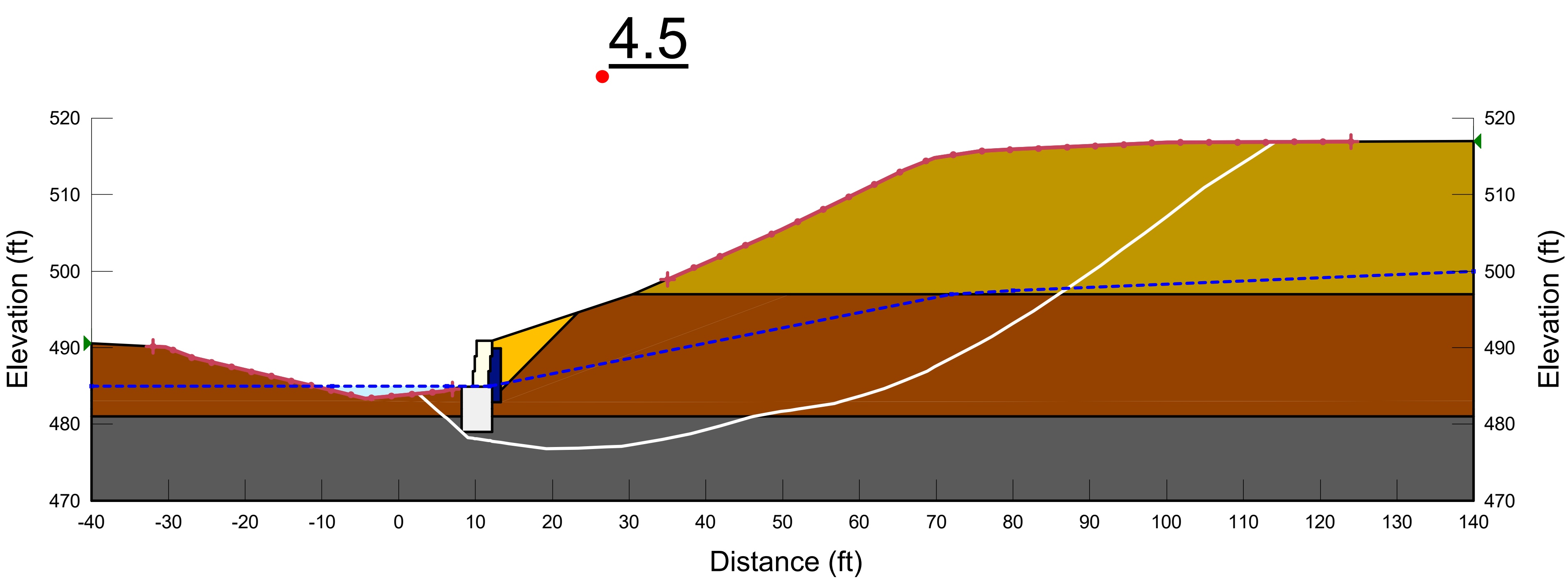
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Drained	Mohr-Coulomb	120	50	20	0	1
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	1
	Concrete	High Strength	150				1
	Crushed Stone	Mohr-Coulomb	125	0	34	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 2	Mohr-Coulomb	120	0	34	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 2. Proposed wall, slope stability under drained conditions at flood water level.

Plate E-17

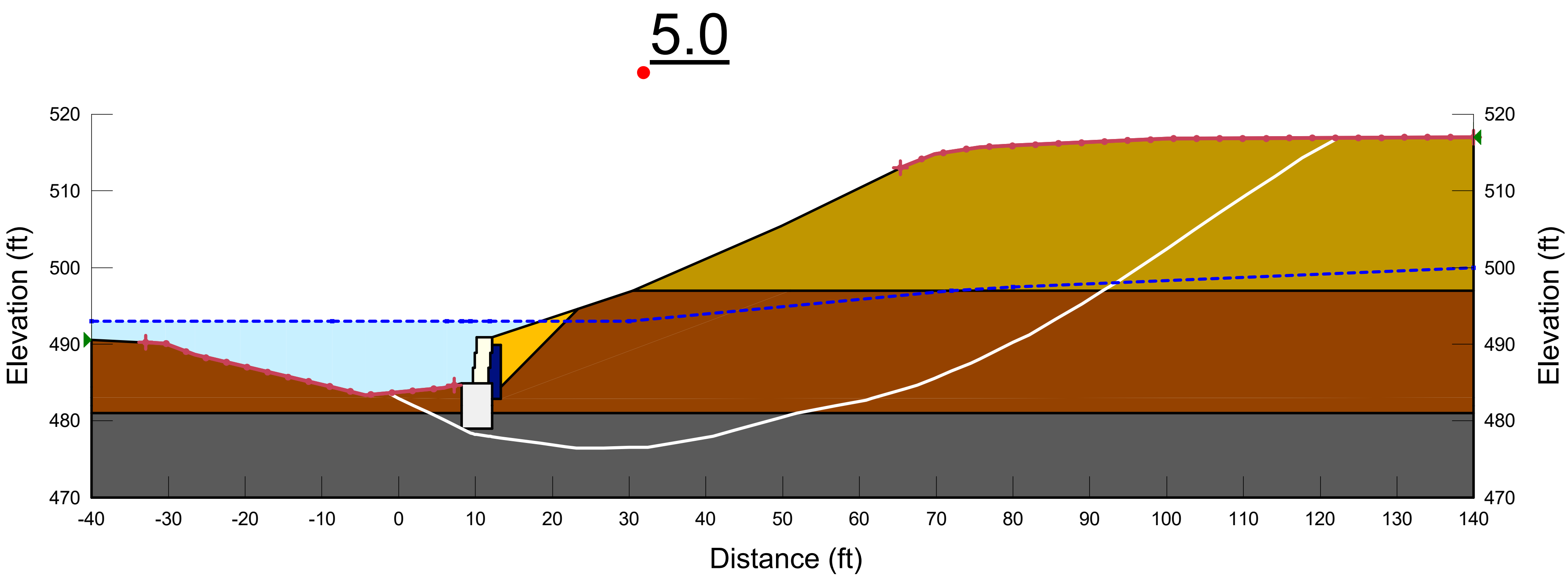
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Total Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Undrained	Mohr-Coulomb	120	2,000	0	0	1
	Clayshale Undrained	Mohr-Coulomb	130	4,000	0	0	1
	Concrete	High Strength	150				1
	Crushed Stone	Mohr-Coulomb	125	0	34	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 2	Mohr-Coulomb	120	0	34	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 2. Proposed wall, slope stability under undrained conditions at normal water level.

Plate E-18

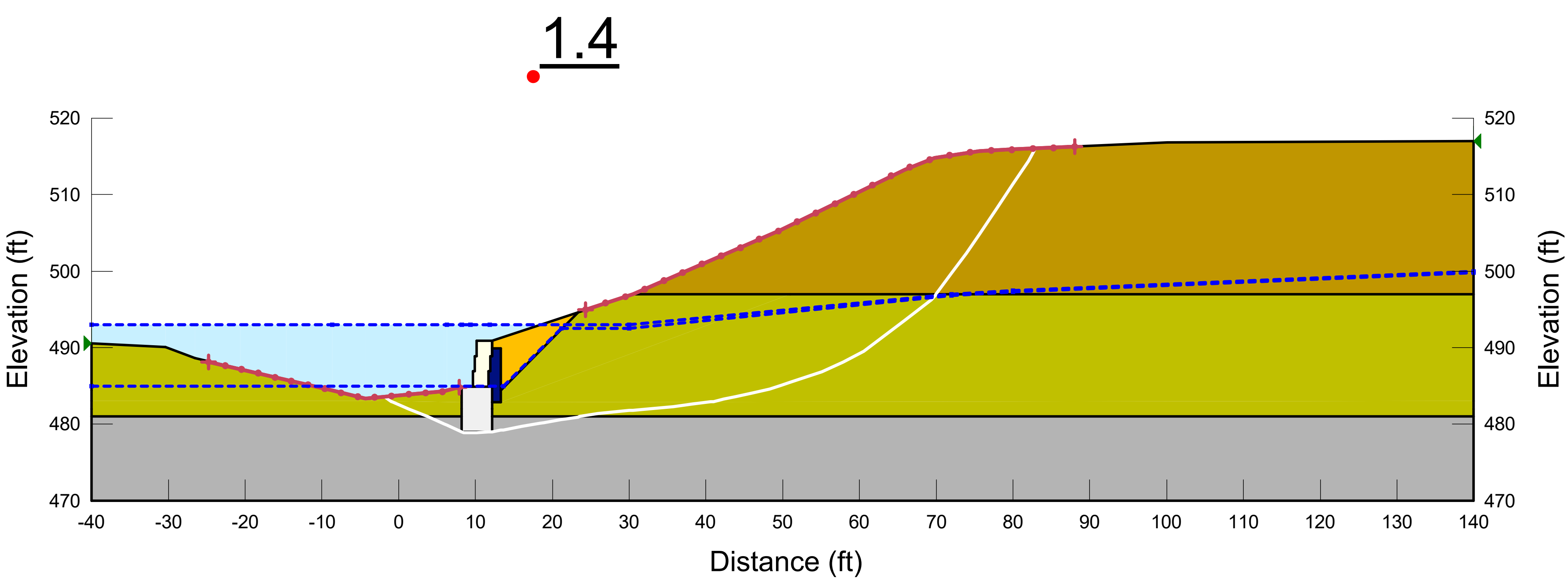
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Total Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Undrained	Mohr-Coulomb	120	2,000	0	0	1
	Clayshale Undrained	Mohr-Coulomb	130	4,000	0	0	1
	Concrete	High Strength	150				1
	Crushed Stone	Mohr-Coulomb	125	0	34	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 2	Mohr-Coulomb	120	0	34	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 2. Proposed wall, slope stability under undrained conditions at flood water level.

Plate E-19

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	CH Drained	Mohr-Coulomb	120	50	20	0	1
	Clayshale Drained	Mohr-Coulomb	130	500	25	0	1
	Concrete	High Strength	150				1
	Crushed Stone	Mohr-Coulomb	125	0	34	0	1
	Limestone Blocks	High Strength	150				1
	SC/GC Site 2	Mohr-Coulomb	120	0	34	0	1
	Structural Backfill	Mohr-Coulomb	120	0	32	0	1



Site 2. Proposed wall, slope stability under rapid drawdown conditions.

Plate E-20

Appendix F. Schematic Alternative Exhibits

2 CONCRETE TOE WALL

1 LIMESTONE BLOCK WALL

N

20 0 20 40

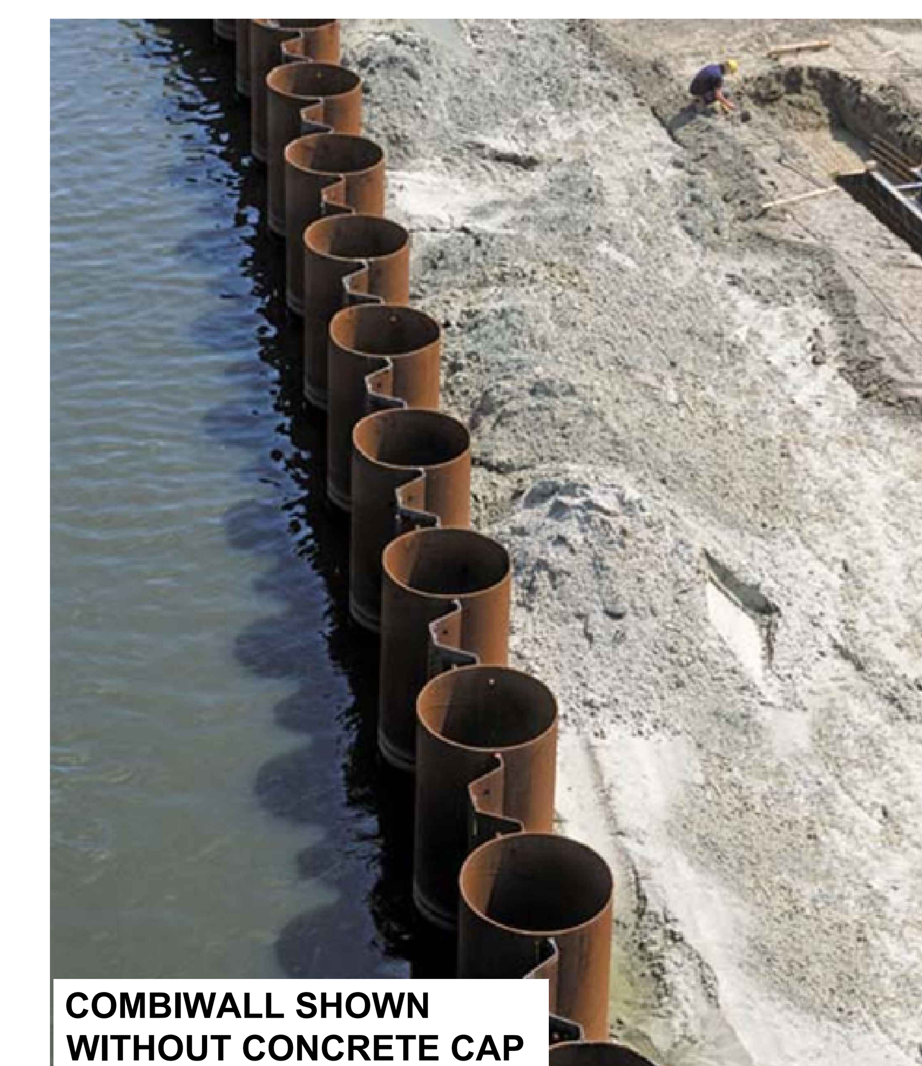
SCALE IN FEET

LEGEND

- PROPERTY LINE
- CONTOURS
- COMBINATION WALL
- PAVEMENT
- LIMESTONE BLOCK WALL
- CONCRETE WALL
- ROCK RIPRAP
- BORE HOLE

NOTES

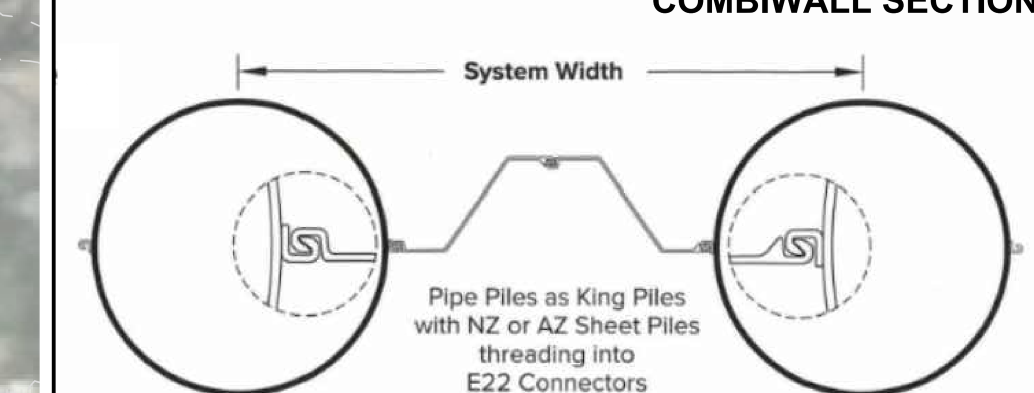
1. CONCEPTUAL LAYOUT, GEOMETRY, AND DETAILS ARE SHOWN AND ARE BASED ON LIMITED DATA AND ANALYSIS. ASSUMPTIONS AND DESIGN SUBJECT TO CHANGE IN FINAL DESIGN PHASE.
2. PROPERTY LINES ARE APPROXIMATE AND BASED ON GUADALUPE COUNTY GIS DATA.



COMBIWALL SHOWN WITHOUT CONCRETE CAP

3

COMBIWALL SECTION



HDR
TEXAS P.E. FIRM
Registration No. F-754

ISSUE	DATE	DESCRIPTION
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PROJECT MANAGER ERIC STEWART, P.E.
DESIGNED BY: PAUL SHATTUCK
DRAWN BY: PATRICK O'FLAHERTY
CHECKED BY: -
PROJECT DATE: --
PROJECT NUMBER 10381170

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HDR Engineering, Inc.
TBPELS Firm F-754

SEGUIN TEXAS

CITY OF SEGUIN

WALNUT SPRINGS BANK STABILIZATION AND SPILLWAY REPAIR PLANNING STUDY

ALTERNATIVE A - SITE 1
UPSTREAM - LIMITED REPAIR ONLY
DOWNSTREAM - INCREASE TO STANDARD F.S.

0 1" 2"

FILENAME CD01
SCALE 1" = 20'

SHEET
1 of 4

1 2 3 4 5 6 7 8

(2) CONCRETE TOE WALL

N

20 0 20 40
SCALE IN FEET

LEGEND

- PROPERTY LINE
- CONTOURS
- COMBINATION WALL
- PAVEMENT
- LIMESTONE BLOCK WALL
- CONCRETE WALL
- ROCK RIPRAP
- BORE HOLE

NOTES

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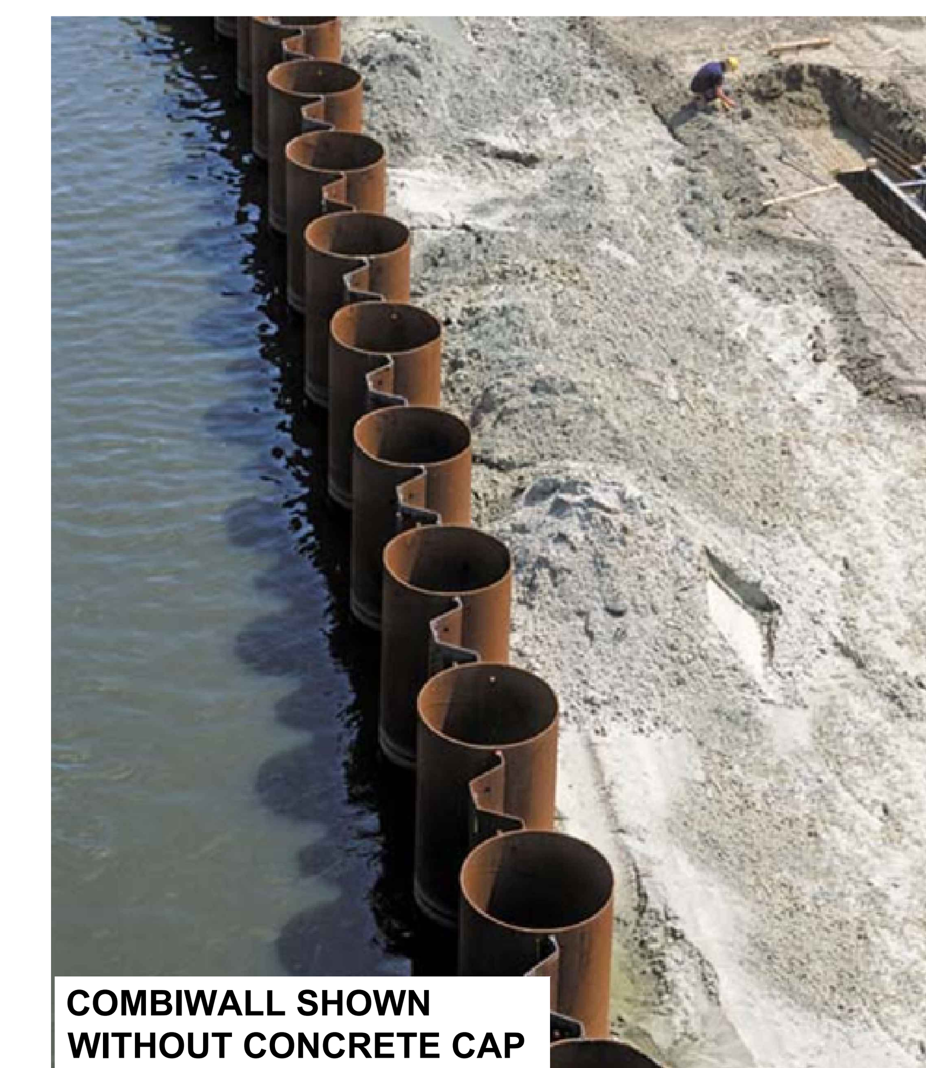
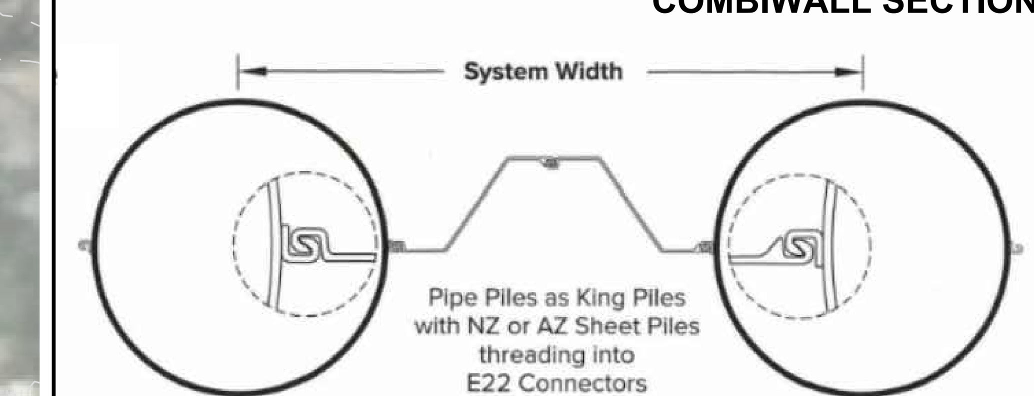
2. PROPERTY LINES ARE APPROXIMATE AND BASED ON GUADALUPE COUNTY GIS DATA.


COMBIWALL SHOWN WITHOUT CONCRETE CAP

(2) COMBIWALL SECTION



HDR
TEXAS P.E. FIRM
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PROJECT DATE:	--
PROJECT NUMBER	10381170

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SEGUIN TEXAS

CITY OF SEGUIN

WALNUT SPRINGS BANK STABILIZATION AND SPILLWAY REPAIR PLANNING STUDY

ALTERNATIVE B - SITE 1
UPSTREAM - REPAIRS & INCREASE TO STANDARD F.S.
DOWNSTREAM - INCREASE TO STANDARD F.S.

0 1" 2"

FILENAME CD02
SCALE 1" = 20'

SHEET
2 of 4

N

20 0 20 40
SCALE IN FEET

LEGEND

- PROPERTY LINE
- CONTOURS
- LIMESTONE BLOCK WALL
- ROCK RIPRAP
- BORE HOLE

NOTES

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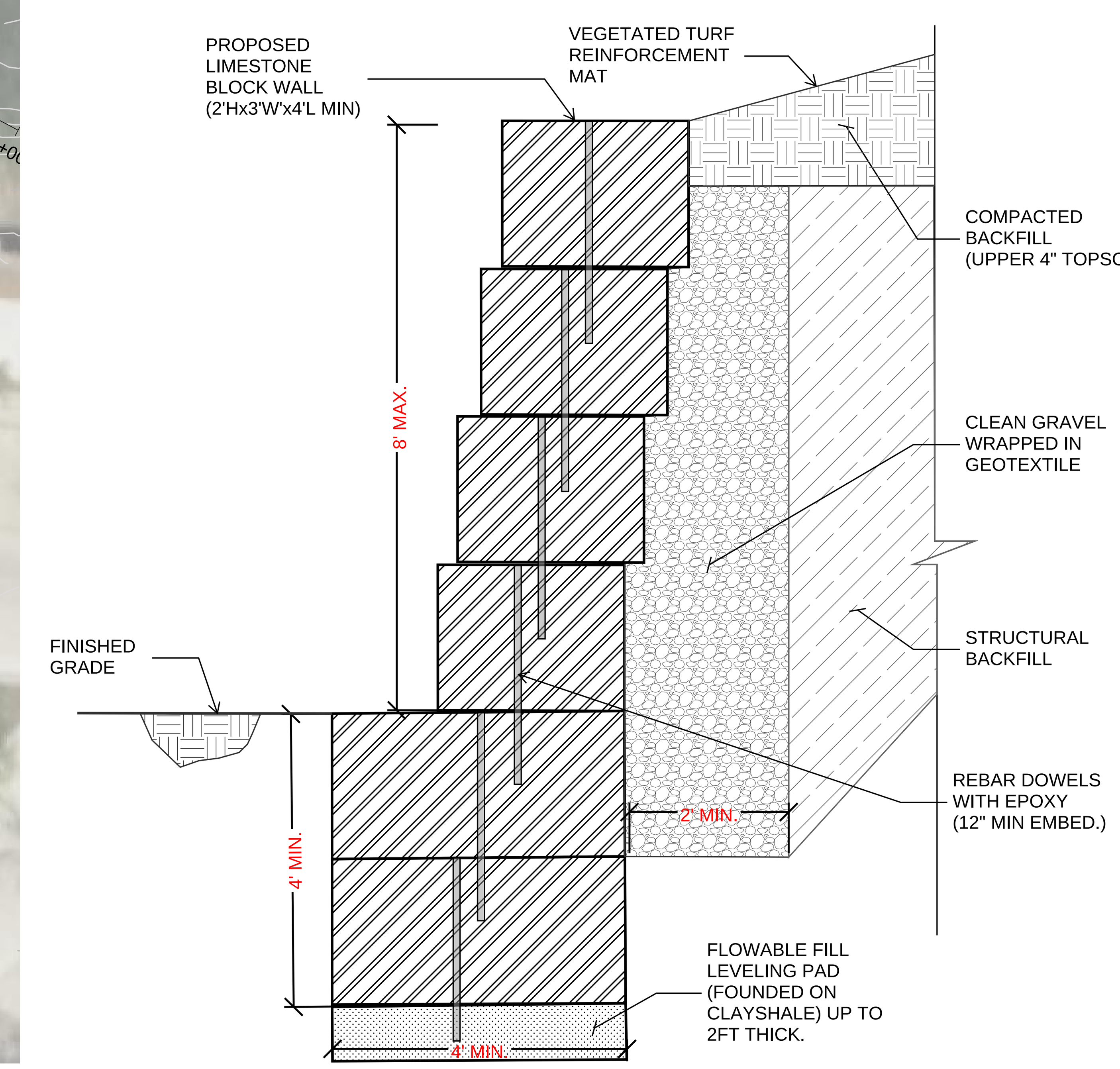
2. PROPERTY LINES ARE APPROXIMATE AND BASED ON GUADALUPE COUNTY GIS DATA.



TEXAS P.E. FIRM
Registration No. F-754

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CITY OF SEGUIN

WALNUT SPRINGS BANK STABILIZATION AND SPILLWAY REPAIR PLANNING STUDY

ALTERNATIVE C - SITE 2 REPLACE EXISTING LIMESTONE BLOCK WALL ENDS

0 1" 2"

FILENAME CD04
SCALE 1" = 20'

SHEET
3 OF 4

N

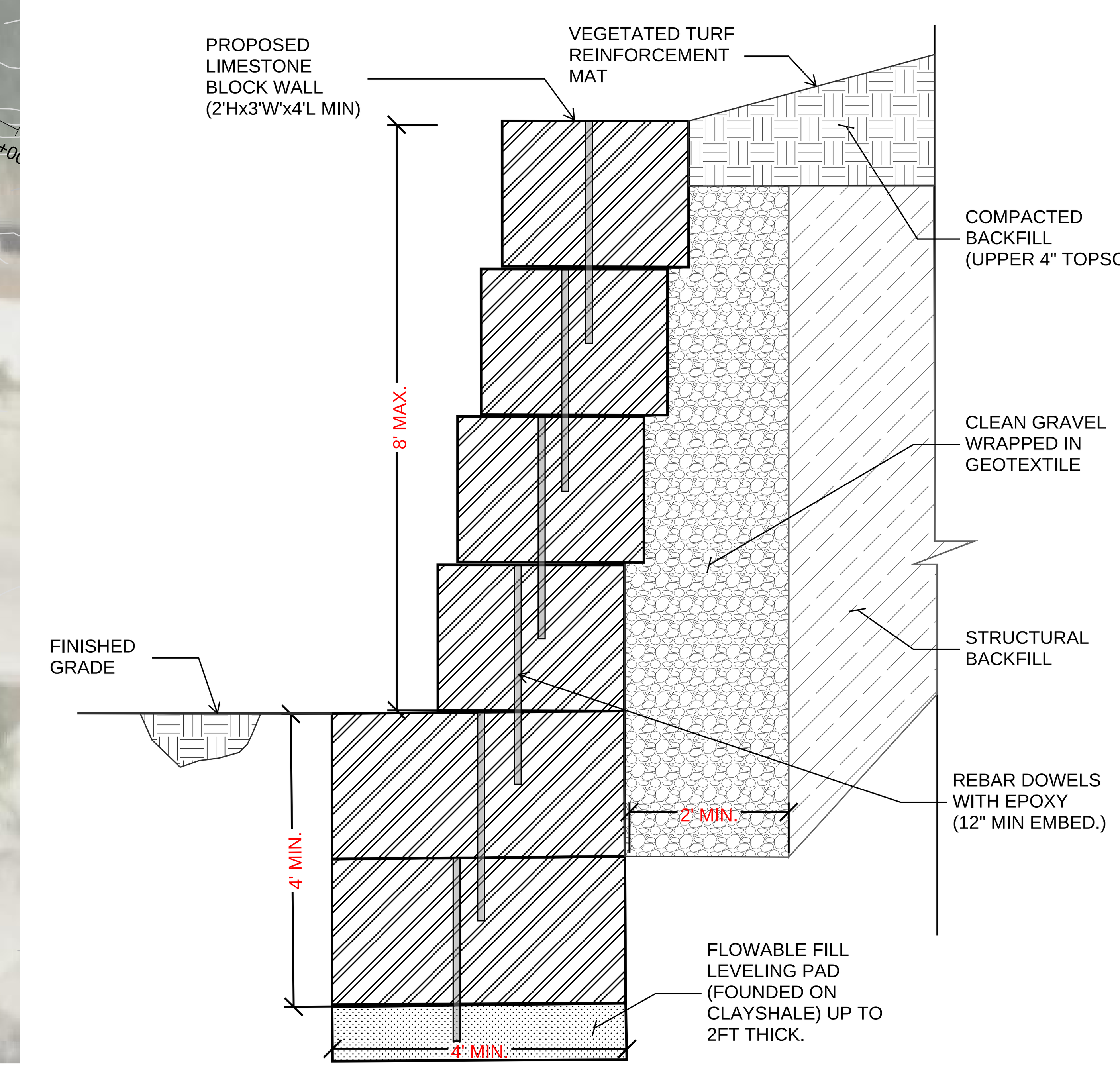
20 0 20 40
SCALE IN FEET

LEGEND

PROPERTY LINE
CONTOURS
LIMESTONE BLOCK WALL
ROCK RIPRAP
BORE HOLE

NOTES

1. CONCEPTUAL LAYOUT, GEOMETRY, AND DETAILS ARE SHOWN AND ARE BASED ON LIMITED DATA AND ANALYSIS. ASSUMPTIONS AND DESIGN SUBJECT TO CHANGE IN FINAL DESIGN PHASE.
2. PROPERTY LINES ARE APPROXIMATE AND BASED ON GUADALUPE COUNTY GIS DATA.



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SEGUIN TEXAS

CITY OF SEGUIN
WALNUT SPRINGS BANK STABILIZATION AND SPILLWAY REPAIR PLANNING STUDY

ALTERNATIVE D - SITE 2
REPLACE EXISTING LIMESTONE BLOCK WALL

0 1" 2"

FILENAME CD04
SCALE 1" = 20'