Innovative approaches
Practical results
Outstanding service

MITCHELL LAKE DAM IMPROVEMENTS
PRELIMINARY ENGINEERING REPORT

Prepared for:

San Antonio Water System

Under contract with Alan Plummer Associates, Inc.

NOVEMBER 2020

FREESE AND NICHOLS

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STATE OF TEXAS
GRADY HILLHOUSE
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LICENSED
PROFESSIONAL ENGINEER

11/19/2020

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Table of Contents

Table of Figures

Table of Tables

APPENDICES:

1.0 BACKGROUND

San Antonio Water System (SAWS) contracted with Alan Plummer Associates, Inc. (APAI), with Freese and Nichols, Inc. (FNI) as a subcontractor, to perform engineering services as a part of the Mitchell Lake Wetlands Quality Treatment Initiatives project (Contract No. P-17-004-GC). Phase 1 of the project involves the evaluation of potential treatment options for discharges from Mitchell Lake, including options that would include modifications to Mitchell Lake Dam. This report summarizes the preliminary analysis and design for modifications to Mitchell Lake Dam.

1.1 DAM DESCRIPTION

Mitchell Lake Dam (TX01453) is located in south San Antonio, Texas in Bexar County. The dam was originally constructed in 1901 and consists of an earthen embankment and a spillway at the left abutment. The embankment is approximately 3,200 feet long with a maximum height of approximately 20 feet. The crest width is approximately 15 feet with side slopes ranging from approximately 2:1 (H:V) to 3:1. The crest elevation varies between approximately 526.0 feet and 528.5 feet, although the majority of the embankment has a crest elevation of around 528.0 feet. Figure 1.1 shows an aerial photo of the dam.

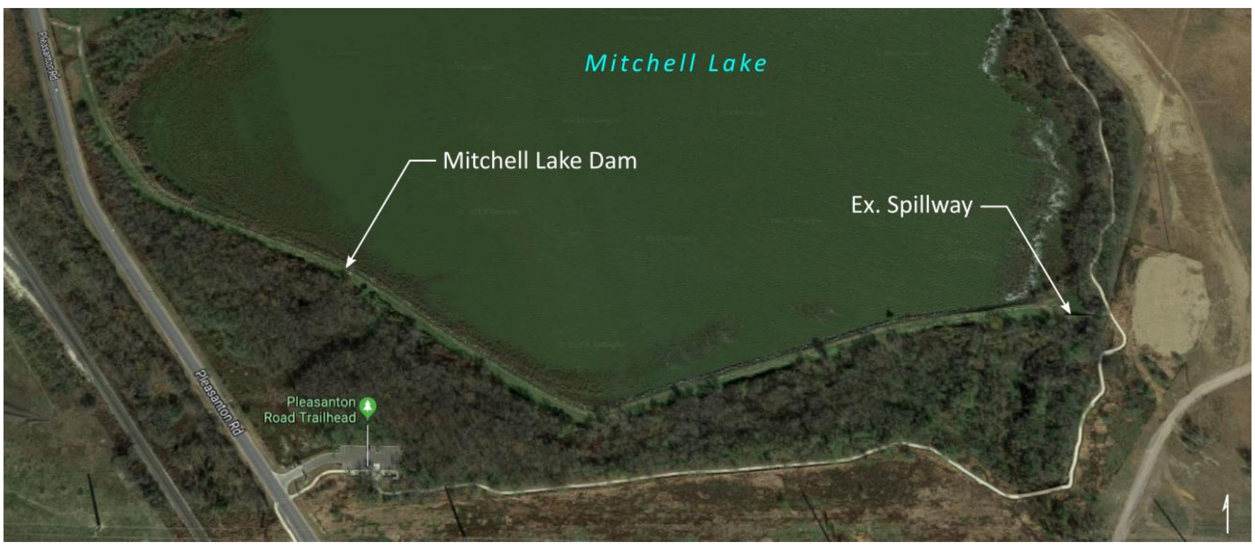

Figure 1.1: Overview of Mitchell Lake Dam

The existing service spillway consists of a 55-foot long concrete gravity structure with eight 36-inch diameter gate valves. A ninth gate valve discharges to a 36” RCP which leads to an irrigation canal away from Cottonmouth Creek. Eight gate valves in the service spillway have invert elevations ranging from 520.71 to 520.78 feet, with an average invert elevation of 520.76 feet. Water passing through the gate valves passes through a stone and mortar chute approximately 225 feet in length, terminating in an eroded plunge pool on Cottonmouth Creek, a tributary to the Medina River. Figure 1.2 shows the existing spillway.

Figure 1.2: Mitchell Lake Dam Spillway (Looking Right along Downstream Side of Gate Valves)

Mitchell Lake is a nationally significant water body as a refuge for migratory shorebirds and waterfowl. In 2004, SAWS entered into an operating agreement with the National Audubon Society establishing the Mitchell Lake Audubon Center.

1.2 PREVIOUS WORK

Considerable work has been previously performed to evaluate Mitchell Lake Dam and develop proposed improvements for its maintenance, repair, and rehabilitation. FNI reviewed all previous documentation related to Mitchell Lake Dam provided by SAWS to identify any trends related to the safety of the dam and understand historical planning and concept development. This section is intended to provide an overview of the conclusions and recommendations from previous reports and inspections.

1.2.1 *1970-1981 State Inspections*

Between 1970 and 1981, representatives from the Texas Water Rights Commission (TWRC) and the Texas Department of Water Resources (TDWR), which were predecessor agencies of the Texas Commission on Environmental Quality (TCEQ) performed inspections of Mitchell Lake Dam as a part of the State Dam Safety Program.

On January 13, 1970, engineers from the TWRC inspected Mitchell Lake Dam. Items noted during the inspection included ponding on the downstream toe of the embankment due to poor drainage, erosion on the upstream face of the embankment due to wave action, and overgrown vegetation along the embankment. It was recommended that erosion protection be installed on the upstream slope of the embankment and that a maintenance program be initiated to manage the vegetation along the dam [18].

On February 26, 1975, engineers from the TWRC inspected the dam again. The inspection team noted that the embankment crest had been rehabilitated and that concrete rubble riprap had been installed along the upstream slope of the embankment. Deficiencies noted during the inspection included failed training walls on the spillway exit channel and the possibility that the dam was not hydraulically adequate (not able to safely pass the design storm without overtopping the earthen embankment). Representatives from TWRC returned to the dam in September 1975 to perform a survey of the dam for use in a hydrologic analysis. At this time, the spillway exit channel walls had been repaired and regrading of the area downstream of the dam to improve drainage was planned for the near future [19].

On March 14, 1977, engineers from the TDWR inspected Mitchell Lake Dam. Items noted during the inspection included a cavity behind the left wall of the stilling basin, poor drainage downstream of the embankment toe, and minor erosion on the upstream face of the embankment. The inspection report mentions that TDWR's hydrologic study was complete and concluded that Mitchell Lake Dam was not hydraulically adequate. A meeting in March 1976 was held between the City of San Antonio and TDWR to discuss improvements to the dam. The City had initiated a study to investigate potential improvements, but the study was not complete by the time of the inspection [17].

On August 6, 1981, engineers from TDWR inspected Mitchell Lake Dam again. Items noted during the inspection included overgrown vegetation along the embankment and spillway, an erosion gully on the downstream slope of the embankment, and poor drainage downstream of the embankment toe. The inspection report mentions another meeting in January 1978 between the City and TDWR to discuss the challenges of meeting water quality, water rights, and dam safety requirements [16].

1.2.2 *1991 Mitchell Lake Wetlands Enhancement Project*

In 1991, the City of San Antonio initiated a series of studies to investigate options for the restoration of Mitchell Lake, including cleanup of the polder/decant basins, odor reduction, wildlife habitat improvements, and meeting water quality standards for the Medina River. An executive summary of the program was prepared by Raba-Kistner [11]. The summary states that the City of San Antonio was preparing designs to replace the existing Mitchell Lake Dam. No other information about the dam improvements is included in the report.

1.2.3 *2000 Mitchell Lake Master Implementation Plan*

In 1996, SAWS established a set of eight goals for Mitchell Lake related to improvement of the environmental and community benefits of the amenity. In 1999, SAWS initiated an effort to develop a Master Implementation Plan for Mitchell Lake through engineering analysis, data collection, and an extensive public input program. The Master Plan included recommendations for on-site improvements including water quality improvement measures, wildlife habitat improvements, a park, and an education center. The plan included a review of the condition of the dam by a professional engineer. The review provided recommendations for improvements including regular maintenance items and repairs recommended in previous inspection reports. $3.5M was included in the master implementation plan cost estimate for dam improvements.

1.2.4 *2008 TCEQ Inspection*

TCEQ attempted to perform an inspection of Mitchell Lake Dam in October 2005 but were unable due to overgrown vegetation along the dam. In a letter, they requested that the vegetation be managed to allow full access to the dam. On May 28, 2008, engineers from TCEQ inspected Mitchell Lake Dam after clearing of the vegetation by SAWS [15]. The dam was observed to be in overall poor condition. Major items noted during the inspection included lack of adequate grass cover, animal rutting, overgrown vegetation, steep embankment slopes, inadequate upstream erosion protection, and deterioration of concrete/masonry exit channel. Based on a review of aerial photography downstream of the dam, TCEQ stated that the dam appeared to be a low hazard structure, but that additional investigation of the downstream conditions was warranted. Major recommendations from the inspection included the following:

- Establish a vegetation maintenance program that would include control of vegetation along the structure and establishment of erosion control grass on the embankment.
- Repair irrigation and drainage structures as necessary.
- Repair upstream erosion control measures for protection against wave action.
- Perform an updated hydrologic and hydraulic study to investigate the dam's hazard classification and hydraulic adequacy.

A written corrective action plan was requested in the inspection report. SAWS responded to the inspection report in August 2008 with a corrective action plan for the dam.

1.2.5 *2014 Mitchell Lake Hydrologic and Hydraulic Analysis*

In response to TCEQ's recommendation, SAWS initiated a hydrologic and hydraulic analysis of Mitchell Lake Dam. The study, completed by Arcadis [2], included detailed hydrologic and hydraulic modeling to determine the hydraulic adequacy of the dam. The study concluded that Mitchell Lake Dam is not hydraulically adequate under existing conditions because the 28% Probable Maximum Flood (PMF) would overtop a low area along the crest of the earthen embankment.

1.2.6 *2015 Mitchell Lake Dam Conceptual Design*

In 2015, SAWS engaged Merrick & Company to develop a conceptual design for improvements that would reduce the quantity and duration of discharge events at Mitchell Lake [10]. The proposed improvements included raising the embankment crest to elevation 528 feet, flattening the slopes of the earthen embankment, and installation of a principal and auxiliary spillway to manage discharges from Mitchell Lake. Three alternatives for seepage control in the embankment were evaluated. The improvements were based on design criteria including a maximum normal pool elevation of 517.5 feet, a dam safety design storm equal to 28% of the PMF, and a peak water surface elevation during the design storm of 527.0 feet. The total project construction cost was estimated to be between $10.9M and $23.0M depending on the selected seepage control measures.

The Merrick study included recommendations for further analysis of the dam, including the following:

- Conduct a geotechnical investigation and testing program to refine assumptions about material strengths and permeability and identify potential borrow sources for embankment construction.
- Perform geotechnical and slope stability analysis to refine embankment section design and concrete spillway foundation assumptions.
- Perform breach analysis to confirm the dam's hazard classification and associated hydraulic design criteria.

In 2016, Merrick provided a supplemental memorandum documenting additional analysis of Mitchell Lake Dam [9]. Additional hydrologic analysis was conducted to determine a configuration that would contain the full 100-year event under ultimate basin conditions without engaging the auxiliary spillway. The auxiliary spillway crest was set at elevation 525.8 feet. At this crest elevation, it was estimated that a labyrinth weir spillway with a footprint length of 240 feet could safely pass a design storm equal to the 34% PMF while keeping Mitchell Lake below a peak elevation of 528.0 feet. The supplemental memo also discussed the need and potential costs for a cofferdam during construction of the improvements to the embankment of Mitchell Lake Dam.

1.2.7 *Constructed Wetlands Preliminary Analysis*

Starting in 2016, SAWS engaged the APAI team to evaluate the potential for installing constructed wetlands to manage normal discharges from the lake and reduce the number and volume of uncontrolled releases into the Medina River. These efforts have included extensive coordination with state and federal regulatory agencies, pilot-scale testing of wetland efficacy, and detailed characterization of Mitchell Lake's hydrology through single-event and monthly water balance modeling. Much of this preliminary work has served as the basis for the proposed improvements to Mitchell Lake Dam as described herein. The efforts are currently ongoing and will be documented in various reports provided to SAWS after the current project phase.

2.0 EXISTING CONDITIONS

2.1 SITE VISIT

FNI engineers performed a site visit to Mitchell Lake Dam on November 13, 2017. The purpose of the visit was to document the existing condition of the embankment and spillway. A photo log of the site visit is presented in Appendix A. The following observations were noted during the site visit:

- In general, it was clear that SAWS had performed extensive vegetation control since TCEQ's previous inspection in 2008. Vegetation along the embankment crest and side slopes were not overgrown except in localized areas, mainly on the upstream slope of the embankment around the rubble erosion protection. Also, no major bare areas or surface erosion were noted along the dam.
- The vegetation downstream of the embankment toe is very thick with both large trees and dense woody undergrowth. Inspection of the immediate downstream vicinity of the embankment was not possible due to the vegetation. Previous inspections have noted poor drainage and ponding water in this area, but this could not be confirmed.
- The crest of the embankment was in good condition with consistent width and no major rutting from vehicular traffic.
- The downstream slope was in good condition. The slope varies in steepness between around 2:1 (H:V) and 3:1 and has large rocks scattered along the surface. These conditions may make mowing of the slope difficult. One small animal burrow was noted on the downstream slope.
- The upstream slope is sparsely covered with concrete rubble as erosion protection, with coverage increasing closer to the spillway. Erosion from wave action was noted in several locations along the upstream slope. In general, the existing concrete rubble is not adequate to provide long-term protection against erosion due to wave action. The upstream slope also had localized areas of dense vegetation.
- Abandoned irrigation features were noted in several locations. In most cases, these channels and conduits do not pose any issue to the dam. However, there is at least one location on the embankment where an abandoned pipe penetrates the embankment with a masonry headwall. Any penetration is a potential location for seepage through the embankment.
- The spillway gates and concrete bulkhead were in fair condition. Some of the gates have been welded in the open position. The steel catwalk over the spillway appeared to be in fair condition with no major corrosion or damaged members.
- The spillway exit channel was in poor condition. Deterioration of the channel lining was noted in several locations. The downstream end of the exit channel is eroded and undermined at the plunge pool. At least one section of the channel lining has failed at the downstream end as evidenced by the exposed steel reinforcement. The channel beyond the plunge pool is significantly eroded and incised.

2.2 SURVEY

A topographic and bathymetric survey was performed by Vickrey and Associates, Inc. in January 2018. The survey shows that the elevation of the lowest upstream toe of the embankment is approximately 517 feet. The survey included limited points on the crest of the dam, but the points collected varied between 526 and 528.5 feet in elevation. From lidar topography data, the lowest downstream toe elevation for the embankment is approximately 508 feet. Note that all elevations in this report reference the North American Vertical Datum of 1988 (NAVD88) and references to left or right assume the viewer is facing downstream.

3.0 PROPOSED IMPROVEMENTS

3.1 DESIGN CRITERIA

3.1.1 *Pool Elevations*

SAWS selected key elevations for design of the dam modifications based on existing water rights and project goals related to reducing the volume and frequency of uncontrolled releases to Cottonmouth Creek and the Medina River. Hydrologic modeling used to establish these design criteria is documented separately. The Maximum Normal Operating Pool (MNOP) was set at 518.5 feet.

A flood pool is established above the MNOP to allow for metered discharges of reservoir water to proposed wetlands downstream of the dam. The preliminary design includes outlet works to allow for separate discharges via gravity flow to the two wetland trains on each side of Cottonmouth Creek. The outlet works will allow for a variable discharge rate that increases with increasing lake level up to a maximum combined flow of 7 MGD. The flow rate to each of the two wetland subsystems will need to be regulated in proportion to the downstream wetland area. Based on the conceptual layout, approximately 67% of the flow will need to be routed to the west wetland area and 33% to the east. To accommodate these requirements, specialized weirs or motorized actuators may need to be considered during the design phase of the project. Instrumentation will also be needed to provide continuous flow measurement into each of the downstream wetland subsystems.

The preliminary auxiliary spillway crest elevation was set at elevation 521.5 feet. When the reservoir rises above this elevation, discharges will pass through the auxiliary spillway into Cottonmouth Creek. This elevation was selected to impound no more than 2,513 acre-feet below the top of flood pool in Mitchell Lake. When combined with water in the proposed wetland (127 acre-feet), the reservoir will store no more than the authorized impoundment volume of 2,640 acre-feet per Water Rights Permit 19-2153C.

3.1.2 *Dam Crest Elevation*

FNI provided a draft cost sensitivity analysis memorandum and participated in discussions with the SAWS project team to select the proposed crest elevation for Mitchell Lake Dam. The team selected elevation 528.0 feet for preliminary design of the dam improvements. This elevation is the approximate average crest elevation of the existing embankment. Raising the embankment above this elevation would allow for a smaller auxiliary spillway but may increase cost and complexity of the project due to potential property and easement acquisition around the reservoir rim.

3.1.3 *Design Storm*

TCEQ regulates the safety of dams in Texas. State design criteria for dams are found in Chapter 299 of Title 30, Texas Administrative Code and Hydrologic and Hydraulic Guidelines for Dams in Texas [14]. With a maximum height of approximately 20 feet and a maximum storage of 6,516 acre-feet, Mitchell Lake Dam is classified as an intermediate-sized structure. Dams are also classified by the potential for loss of human life and/or property damage within the area downstream of the dam. The hazard classification of a dam can be low, significant, or high depending on the potential downstream impacts that could result from failure. FNI prepared a Probable Maximum Flood (PMF) study and breach analysis of the existing structure in 2019 [8]. This study indicated that potential breach impacts of Mitchell Lake were consistent with a significant hazard classification, including appreciable economic loss in a rural area, isolated damage to homes and secondary highways, and interruption of service of public utilities.

A dam's hazard classification is not a static determination, and several factors can influence this classification over time. Increased development within the watershed can increase the peak inflow to the lake during the design storm, leading to a higher peak lake level and greater inundation during a potential breach. Development can also occur within the potential breach inundation area, increasing the number of lives at risk or the potential economic damage that could result from a breach. SAWS requested FNI assume a high hazard classification in the design of improvements to the dam to ensure that it remains in compliance with state regulations over the design life of the structure. As an intermediate size, high hazard structure, the modified dam would be required to safely pass a design storm of 78% of the PMF. Note that TCEQ will allow a reduction in the design storm for an existing dam to 75% of the PMF provided the following requirements are met.

- The dam has an Emergency Action Plan (EAP)
- The dam has an Operation and Maintenance (O&M) Plan, including a detailed inspection program
- The owner submits an annual report to TCEQ documenting compliance with the above.

This study uses a design storm of 80% of the PMF to allow for conservatism in the design. The final design of the spillway may consider a 5% reduction in the PMF.

3.1.4 *Geotechnical*

The slope stability criteria are based on TCEQ requirements, supplemented with USACE requirements as necessary when not specified by TCEQ. Table 3.1 presents the required factor of safety for slope stability for embankments listed by TCEQ Design and Construction Guidelines for Dams in Texas [13] and USACE EM 1110-2-1902 Slope Stability [22] for three loading conditions.

Table 3.1: Slope Stability Minimum Safety Factors

Loading Condition	Minimum Factor of Safety for Slope Stability per TCEQ (2009)	Minimum Factor of Safety for Slope Stability per USACE EM 1110-2-1902 (2003)
Long Term, Steady State Seepage Conditions	1.5	1.5
Rapid Draw Down	1.2	1.1-1.3
End of Construction including During Construction	1.25	1.3

Seepage analysis was performed according to USACE EM 1110-2-1901, Seepage Analysis and Control for Dams [23], USBR Design Standard 13, Chapter 8 [24], and supplemented with Duncan et al. [6] as necessary. For uplift and blowout, a minimum safety factor of 1.5 is considered reasonable for existing dams and a minimum safety factor of 2.0 is considered reasonable for new dams.

3.1.5 *Other Design Criteria*

Subsequent phases of design will consider detailed structural and geotechnical design criteria for the dam and spillway improvements. The preliminary design avoids significant disturbance of the existing spillway and exit channel to the extent possible as historic preservation of this structure may be required. Work is ongoing to identify exact requirements associated with preservation of historical structures at the dam.

3.2 PRELIMINARY PROJECT LAYOUT

FNI prepared a preliminary project layout for the dam modification project to serve as the basis for the OPCC in accordance with the design criteria described above. The preliminary layout consists of the following major components:

- Regrading and raising the Mitchell Lake Dam embankment to have a consistent crest elevation and stable slopes
- Installation of rock riprap on the upstream embankment slope for wave action protection
- Installation of a new labyrinth weir auxiliary spillway
- Installation of two principal spillways and conduits to divert flood pool storage to the proposed constructed wetlands

Detailed drawings of the preliminary project layout are included with this report in Appendix B.

3.2.1 *Embankment*

The proposed modification includes raising and regrading the existing embankment to have a crest width of 15 feet, side slopes of 3H:1V, a crest elevation of 528 feet above NAVD88 with upstream riprap protection for wave action, and a flexible base crest road. Note that the majority of the Mitchell Lake Dam embankment sits above the right channel bank beyond the right abutment and above the proposed normal reservoir elevation of 518.5. The proposed typical cross-section was selected to place most of the fill on the downstream side of the existing embankment to minimize fills in within the reservoir and to reduce the need to excavate sediment from the reservoir bed which may contain contaminants. Construction of the proposed embankment will include regrading along the toe for positive drainage away from the dam.

3.2.2 *Auxiliary Spillway*

The spillway layout is based on hydrologic calculations to allow safe passage of 80% of the PMF, meeting state design criteria for an intermediate size, high hazard dam. The spillway consists of an 8-cycle labyrinth weir with a footprint length of 160 feet and a width of 55 feet, a concrete chute, stilling basin, and training walls. Chute, training wall, and stilling basin dimensions are roughly sized based on capacity-factored examples of similar projects and engineering judgment.

Gated inlets are included on each side of the spillway to serve as principal spillways and deliver metered discharges to the proposed wetlands. The inlets consist of concrete vaults equipped with trash racks, stoplog slots, and downward-opening, 24-inch wide weir gates for metering releases from the flood pool. Reinforced concrete pressure pipes serve as conduits to each wetland train. The OPCC includes the cost of these conduits to the ends of the spillway wingwalls. The proposed wetland OPCC will include the conduits beyond this point. The spillway will also be equipped with level-measurement instrumentation for monitoring. Construction of the spillway will include excavation of an entrance channel to allow free flow from the reservoir to the weir and an exit channel to connect the spillway to Cottonmouth Creek downstream. The spillway is located to avoid disturbance of the existing spillway which may require preservation as a historic structure.

3.2.3 *Other Improvements*

In addition to embankment and spillway improvements, FNI understands that the dam modification project is part of a larger improvement program at Mitchell Lake. The Medina River Greenway hike and bike trail runs along the east side of Mitchell Lake and crosses Cottonmouth Creek just downstream of the dam. It is probable that the proposed project at Mitchell Lake Dam will include improvements for public access and recreational amenities. Although the scope of these facilities has not yet been identified, FNI has incorporated allowances for miscellaneous improvements in the preliminary OPCC that may be required in the final project, including the following:

- Aesthetic upgrades to concrete spillway (e.g. form-lined faces and integral color)
- Site improvements for drainage, access, and security around the dam, potentially including a vehicle bridge or pedestrian catwalk over the proposed auxiliary spillway
- Historic preservation of the existing spillway structure (i.e. decommissioning by blocking upstream end with earthen embankment and stabilization of downstream plunge pool in coordination with new concrete spillway training wall).
- Relocation of a portion of the hike and bike trail
- Landscaping, interpretive signage, and trails to facilitate public access to some or all of the project

4.0 HYDROLOGY AND HYDRAULICS

A hydrologic model was used to estimate the inflow hydrograph for the regulatory design storm and route the storm through the reservoir and dam. The hydrologic model was originally developed by Arcadis as a part of their 2015 Hydrologic and Hydraulic Analysis report [2]. The model was reviewed and updated using HEC-HMS version 4.3 and under TCEQ guidelines for estimation of the PMF. The model used the NRCS Curve Number method for estimating precipitation losses and unit hydrograph transformation to determine runoff hydrographs from individual subbasins. The Muskingum-Cunge method was used for reach routing. The model also included routing through three reservoirs upstream of Mitchell Lake: Ballasetal Lake, Timberlodge Lake, and Canvasback Lake. The total contributing drainage area to Mitchell Lake is 9.8 square miles. The following revisions were made to the Arcadis hydrologic model for use in this study, in addition to minor cleanup and fixes:

- The proposed auxiliary spillway was added to Mitchell Lake Dam and the existing spillway removed. Discharge through the proposed principal spillways is conservatively neglected for design storm routing.
- The stage-storage curve for Mitchell Lake was updated using newer bathymetric and topographic data.
- Meteorological models were updated with PMP depths using the TCEQ PMP GIS Tool.
- The model time step was revised from 30 minutes to 1 minute. Lag times for small basins were revised to have a minimum value of 5 minutes.
- The curve numbers assigned to each basin were revised to reflect future land use conditions and Antecedent Runoff Condition (ARC) III per TCEQ guidelines.
- Discharge for PMP runs was ratioed to 80% to represent the design storm for this study.

The following sections provide further detail on hydrologic modeling, hydraulic design of the spillway, and model results.

4.1 PRECIPITATION

Probable Maximum Precipitation (PMP) was estimated for the contributing basin using the TCEQ PMP geoprocessing service. PMP depths are summarized by storm type and duration in Table 4.1. The maximum depth for each duration was used to develop the probable maximum flood (PMF).

Table 4.1: Probable Maximum Precipitation Summary

Storm Type	1-hour	2-hour	3-hour	6-hour	12-hour	24-hour	48-hour	72-hour
Local	11.6	18.3	21.6	26.8	35.5	42.7	45.8	45.8
General	6.6	9.7	14.0	21.2	25.3	28.1	32.1	33.9
Tropical	13.6	18.0	20.3	24.1	31.5	37.8	45.8	45.8
Maximum	**13.6**	**18.3**	**21.6**	**26.8**	**35.5**	**42.7**	**45.8**	**45.8**

The total precipitation for each PMP storm duration was temporally distributed per TCEQ guidelines [14] to obtain the hyetograph for each storm event (Figure 4.1).

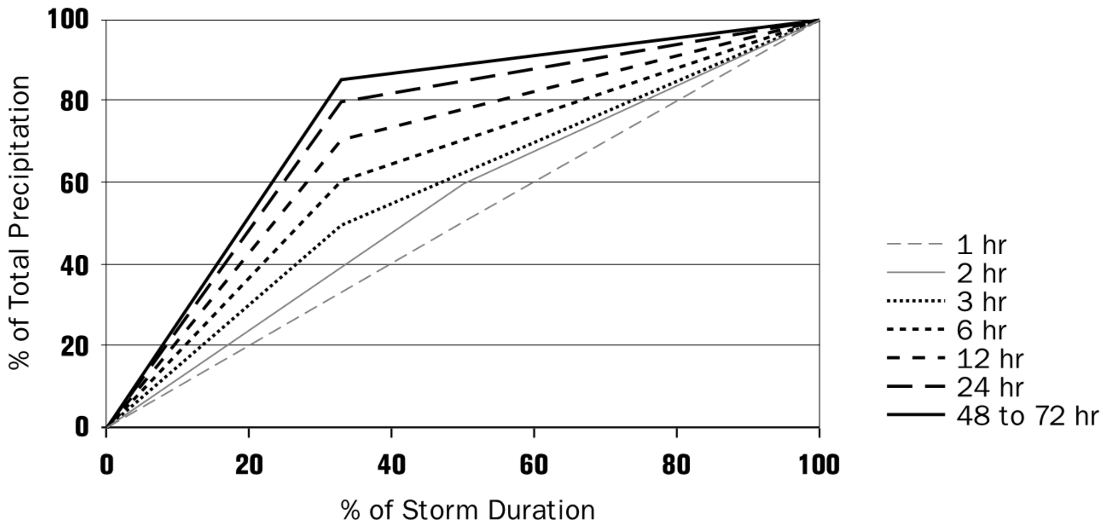

Figure 4.1: PMP Temporal Distribution (Source: TCEQ)

4.2 ELEVATION-CAPACITY CURVE

Table 4.2 presents the elevation-capacity curve for Mitchell Lake. Storage for elevations below 518 is based on bathymetric data collected by Vickrey and Associates Inc. under contract with APAI in January 2018. Storage for elevations between elevations 519 and 531 feet was estimated from Central Texas 2017 Lidar data.

Table 4.2: Elevation-Capacity Curve

Elevation (ft-NAVD88)	Storage (acre-feet)	Elevation (ft-NAVD88)	Storage (acre-feet)
514	0	523	3,318
515	46	524	3,893
516	214	525	4,506
517	482	526	5,149
518	835	527	5,832
519	1,261	528	6,516
520	1,732	529	7,273
521	2,240	530	8,017
522	2,767	531	8,787

4.3 SPILLWAY RATING CURVE

The labyrinth weir discharge rating curve was developed using hydraulic design guidelines from Crookston and Tullis [4]. To reduce the need for significant excavation within the reservoir, the weir approach elevation was set at 516.0 feet with a crest elevation of 521.5 feet, giving the weir a height of 5.5 feet. The weir has 8 labyrinth cycles with a total footprint of 160 feet by 55 feet. The side leg angle is 8.1 degrees, giving the weir a total centerline length of approximately 906 feet. A half-round crest shape was assumed. For each headwater elevation, the weir discharge coefficient was estimated using recommended discharge coefficients. Published labyrinth weir discharge coefficients were used in the weir equation (shown below) to estimate total discharge. The calculated discharge was reduced by 5 percent to account for uncertainty in computations based on FNI's professional judgment and experience with physical scale modeling of other similar structures. The final spillway rating curve is presented in Table 4.3.

$$Q = \frac{2}{3}\sqrt{2g} \cdot C_d \cdot L_C \cdot {H_T}^{3/2}$$

Q = Discharge (cfs)
g = Gravitational Accelerations (ft/s^2)
C_d = Discharge Coefficient
L_C = Centerline Weir Length (feet)
H_T = Total Head above Weir (feet)

Table 4.3: Auxiliary Spillway Rating Curve

Headwater Elevation (feet)	H_T (feet)	C_d	Discharge (cfs)
521.5	0.0	-	0
522.5	1.0	0.57	2,600
523.0	1.5	0.48	4,000
523.5	2.0	0.41	5,300
524.0	2.5	0.36	6,500
524.5	3.0	0.32	7,600
525.0	3.5	0.30	8,900
525.5	4.0	0.28	10,100
526.0	4.5	0.26	11,500
526.5	5.0	0.25	13,000
527.0	5.5	0.25	14,700
528.0	6.5	0.24	18,200

4.4 RESULTS

The hydrologic model was run with the proposed 9-cycle, 180-foot by 45-foot labyrinth spillway for 6-, 12-, 24-, 48-, and 72-hour PMP events. The starting water surface elevation was assumed to be the crest of the principal spillways at 518.5 feet. Discharges from the principal spillways were conservatively neglected. The results of the modeling are shown in Table 4.4. The critical duration is the 24-hour event. TCEQ does not require freeboard above the peak water surface elevation during the design storm for existing dams, so the crest of the embankment was set at 528.0 feet for the preliminary design. Figure 4.2 shows the design storm hydrographs.

Table 4.4: PMP Hydrologic Model Results

	6-hour, 80% PMP	12-hour, 80% PMP	24-hour, 80% PMP	48-hour, 80% PMP	72-hour, 80% PMP
Peak Inflow (cfs)	28,895	29,431	21,008	12,005	8,004
Peak Discharge (cfs)	13,750	17,601	18,249	11,800	7,984
Peak Elevation (feet)	526.7	527.8	528.0	526.1	524.7

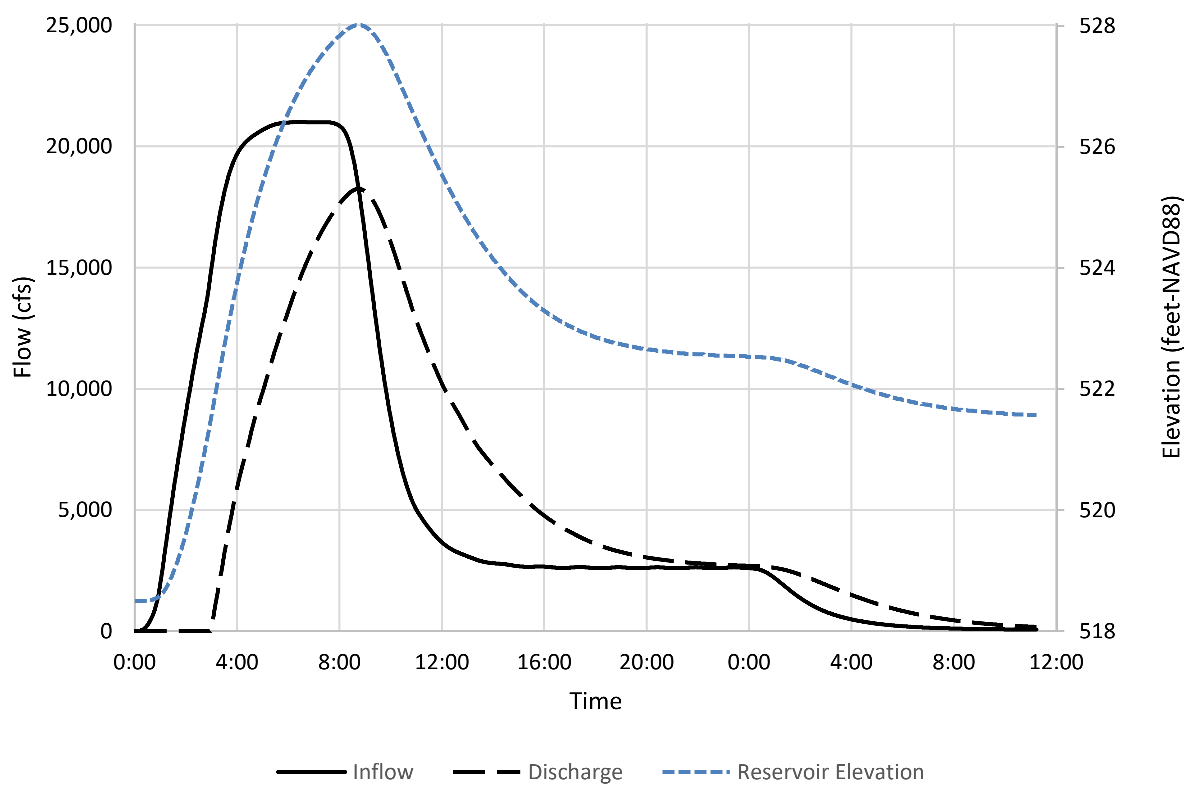

Figure 4.2: Design Storm Hydrograph

5.0 GEOLOGY AND GEOTECHNICAL

Arias Geoprofessionals performed a geotechnical investigation at this site in January 2018 [3]. The information from this investigation has been reviewed and analyzed as part of the preliminary design of improvements to Mitchell Lake Dam. The following sections document geotechnical analyses developed as the basis of the preliminary project layout.

5.1 GEOLOGY

As shown on the Geologic Map of Texas, Mitchell Lake Dam is overlaying the Fluviatile terrace deposits [25]. The Fluviatile terrace deposits typically consist of clays, sands, silts, and gravels. The Fluviatile terrace deposits encountered through the Michell Lake Dam mostly consists of dark gray to brown clay, including sand seams and gravel. The formation is located in stream bed deposits which may contain point bars, cut banks, oxbows, and abandoned channel segments common with variations in stream beds. Therefore, the soil profiles may consist of large variations within short distances.

5.2 STRATIGRAPHY

The general site stratigraphy for Mitchell Lake Dam was determined using data collected from field investigations provided within the Geotechnical Data Report by Arias Geoprofessionals [3]. The geotechnical investigation included the drilling of a total of 20 borings and 13 recovered samples. The samples were recovered using seamless push tubes in cohesive soils and a split-barrel sampler during the Standard Penetration Test (SPT). Field testing included hand penetrometer tests on cohesive soil and the Standard Penetration Test (SPT) to measure blow counts (N) in granular soil. The laboratory testing included moisture content, Atterberg limits, percentage passing no. 200 sieve, sieve analysis, dry unit weight, unconfined compressive tests, consolidated drained direct shear tests, consolidated-undrained triaxial tests with pore pressure measurements, one-dimensional consolidation test, and hydraulic conductivity tests.

The stratigraphy at the site was characterized into four soil groups—lean clay, fat clay, clayey sands, and dense clayey sands. The descriptions and properties are provided below in Table 5.1.

Table 5.1: Generalized Subsurface Conditions and Engineering Properties (Arias 2018)

Material	Depth (ft)	Material Description
Lean Clay	0 – 34 to 0 – 40	Fill Lean Clay (CL): Very Stiff to hard brown, trace gravel Lean Clay to Sandy Lean Clay (CL): Very Stiff to hard dark gray and brown with ferrous stains
Fat Clay	34 – Termination to 40 - Termination	Fat Clay (CH): Very Stiff to hard gray and brown
Clayey Sand	0 – 11	Clayey Sand (SC): Tan and brown, with calcareous nodules
Dense Clayey Sand	0 – Termination to 11 – Termination	Clay Sand (SC): Tan and brown, with calcareous nodules

5.3 BORROW MATERIAL SUITABILITY

The proposed modifications to the dam include adding compacted fill to the existing embankment to raise and level the crest and extend the slopes to 3H:1V. FNI assessed the suitability of the native soils excavated during construction of the new spillway to be used as embankment fill material. Borings B-106, B-107, and B-108 are located in the vicinity of the new spillway. Samples obtained from these borings are classified as CH, CL, and SC with CL being dominant in the area. Classification of soils from borrow area (Table 5.2) and the USCS plasticity chart (Figure 5.1) for the soil samples are presented below. CH, CL, and SC materials are generally suitable for embankment shell materials with limitation on the percent passing #200, liquid limit, and maximum particle size.

Table 5.2: Classification of Potential Borrow Materials

Soil Group	Soils from Dam Borings B-106, B-107, B-108, Average Values		
	LL	**PI**	**%Passing No. 200 Sieve**
CH	57	40	--
CL	41	26	72
SC	32	16	35

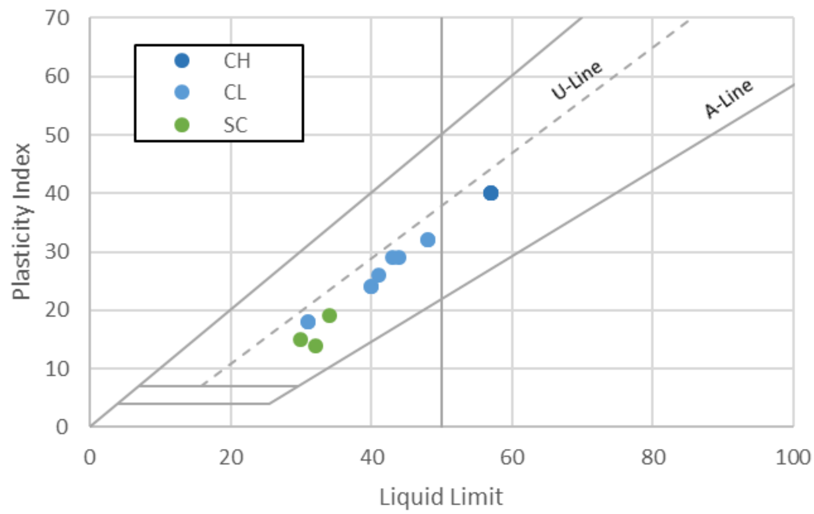

Figure 5.1: Plasticity Chart of Soils from Potential Borrow Area

For embankment stability, it is preferable to place higher plasticity materials closer to the core of the embankment while using lower plasticity materials closer to the shell surface. Therefore, cohesive random fill (CH, CL, SC) materials can be used for construction of the inner shell and low plasticity random fill (CL, SC) can be used for the embankment outer shell. Selected material specifications for the inner and outer shell are listed in Table 5.3. The outer shell shall be a minimum thickness of 3 feet vertically for adequate compaction during construction and to protect inner shell from moisture changes.

Dispersive clays are mostly classified as CL, and some are classified as SC or CH. The 2018 investigation did not include testing for dispersive characteristics of native soils. Embankments constructed with dispersive clays can experience surface erosion as clay particles suspend in rainfall-runoff. The dispersive nature of the borrow materials and embankment soils should be tested by performing a crumb test and double hydrometer test on CL samples for the final design. If dispersive soils are encountered, they should be left out of compacted fill if possible. If sufficient non-dispersive material is not available on site, dispersive materials should only used for constructing the inner shell, should be protected from surface water by placing select nondispersive fill for the outer shell, and only used where seepage is not expected.

Table 5.3: Embankment Material Specifications

Embankment Zone	Description	Specification
Outer Shell	CL, SC	Max. Particle Size: 3 in 25 % <Passing No 200 Sieve < 75 % 20 ≤LL<50
Inner Shell	CH, CL, SC	Nondispersive Max. Particle Size: 3 in 25 % <Passing No 200 Sieve LL<75

5.4 PRELIMINARY STRENGTH AND HYDRAULIC PARAMETERS

To perform slope stability analyses as part of the preliminary design, FNI developed representative material parameters based on 2018 testing data. Table 5.4 provides selected design parameters for slope stability analysis. The analysis also considers soil softening and shallow slope stability. To analyze this case, the fully softened strength (FSS) of CL clay was calculated using the power function parameters presented in Table 5.5. Detailed documentation of material parameter development is provided in Appendix C.

Table 5.4: Selected Design Parameters

Material Type	Moist Unit Weight (pcf)	Consolidated Drained		Consolidated Undrained		Total UU	Permeability	K_v/K_h Ratio
		ϕ' (deg)	c' (psf)	ϕ (deg)	c, (psf)	c, (psf)	K_v (ft/sec)	
CH	130	25	250	18	350	3,500	1.4E-09	0.25
CL	125	27	200	20	300	3,000	4.7E-08	0.33 for Embankment Fill
							1.64E -08	0.1 for Alluvial Foundation Clay
SC	120	30	--	30	--	--	3.3E-07	0.25

Table 5.5: Power Function Parameters for CL Fully Softened Strength

Soil Type	Non-dimensional		Dimensional (psf)	
	"a"	"b"	"a"	"b"
CL	0.571	0.861	1.653	0.861

5.5 SEEPAGE AND SLOPE STABILITY ANALYSIS

5.5.1 *Seepage Analysis*

5.5.1.1 Modeling Assumptions

A seepage analysis was performed to study internal erosion potential and develop pore water pressures for the slope stability analysis. The seepage behavior was modeled numerically using the SEEP/W module within GeoStudio 2020 to develop and analyze a two-dimensional finite-element model. The soil parameters used for the analyses were selected based on a review of the boring logs, laboratory results, published correlations with index properties, and engineering judgment with similar materials. Selected seepage parameters are summarized in Table 5.4. For a discussion on selecting the seepage parameters, refer to Appendix C.

Seepage analysis was performed for extreme loading conditions assuming lake level is at the peak water surface elevation of 528.0 feet and for the normal loading condition assuming lake is at the MNOP elevation of 518.5 feet.

5.5.1.2 Seepage Model Results

Geotechnical investigation shows pervious soil layers were not encountered within the foundation clays that can potentially create seepage paths. Therefore, calculations were performed five feet below the ground surface at the downstream toe to evaluate seepage severity and to determine pore pressures and flux for the proposed modifications at the typical maximum embankment cross-section without a toe drain.

Calculated total safety factors are slightly lower than recommended values. However, these recommendations were developed for dams having a confining layer (such as clay) underlain by a relatively pervious (such as sand) layer. No pervious layer was encountered at the dam borings. Additionally, the cohesion of the clayey layer is not taken into consideration for uplift calculations whereas, uplift pressures will need to overcome the strength of soil to cause a rupture. Calculated flows are less than the flow threshold of $2.2\text{x}10^{-5}$ cfs/ft of head/foot of embankment established by USACE, below which seepage erosion risk is considered negligible even for high exit gradients. Overall, analysis show seepage severity is acceptable for the proposed design and it is recommended to measure hydraulic conductivity of foundation soils for the final design.

Table 5.6: Seepage Summary of Results for Maximum Section

Loading Condition	Calculation Location	Water Level, ft-msl	Total Safety Factor	Seepage Severity, cfs/ft head/ft embankment	Seepage Severity Category
Extreme Loading Condition	5 ft Below Downstream Toe	528.0	1.24	7.5e-11	Negligible
Normal Loading Condition	5 ft Below Downstream Toe	518.5	1.43	6.1e-11	Negligible

5.5.2 *Slope Stability Analysis*

5.5.2.1 Modeling Assumptions

Slope stability analyses were performed for the maximum embankment cross-section with a 15-foot top width and 3H:1V side slopes using SLOPE/W module of GeoStudio 2020 version 10.2.0.19483. The embankment stratigraphy was developed based on boring B-107 located near the maximum embankment cross-section. The proposed embankment geometry includes raising the crest elevation to elevation 528 feet, widening the crest to 15-ft, placing riprap along the upstream embankment, and adding compacted fill material to the downstream embankment.

The scenarios were analyzed with a limit equilibrium approach to calculate force and moment equilibrium using Spencer's Method for estimating the factor of safety (FS) against rotational slope failure [12]. The results of the analyses are presented in terms of the FS, which represents the ratio of shear strength to the shear stresses along the failure surface. The entry/exit routine was used to develop circular failure surfaces and optimized, non-radial failure geometry was allowed. The deep slope failure and shallow slope failure cases for the existing conditions were analyzed by changing the entry/exit routine of the failure surfaces. The pore water pressures established by the seepage models were used directly in the slope stability models. The upstream and downstream embankments were analyzed for shallow and deep failures. The shear strength values summarized in Appendix C were used for calculating upstream and downstream embankment slope stability under multiple loading conditions. Boring B-107 shows a softer clay layer was encountered within the foundation clay. A 5-ft thick softer clay layer was included in the slope stability model by assigning reduced strength parameters to conservatively model this layer.

The analyzed loading conditions can be summarized as:

1. Steady-state seepage conditions were analyzed for both the extreme loading conditions and for the normal water loading conditions. Slope stability calculation assuming steady-state seepage (SSS) conditions can be described as:

- The upstream and downstream embankment stability were analyzed by assigning consolidated drained shear strength parameters to clay soils to analyze long-term performance. A sensitivity analysis was included by modeling a 6-ft deep tension crack full of water along the downstream slope.
- The fully softened strengths (FSS) were assigned to embankment clays and consolidated drained shear strength parameters were assigned to foundation clays. The factor of safeties for shallow slides that can occur through the fully softened layers of upstream and downstream slopes were searched by limiting the slip surface definition. Additionally, a 6-ft deep tension crack full of water was modeled at the ground surface to include the impact of tension cracks.
- The end of construction stability was calculated by assigning CU or UU strengths to fill materials. Modeled loading conditions can be described as:
    1. The consolidated undrained (CU) shear strengths were assigned to compacted clays that will be placed as a part of proposed modifications. The consolidated drained (CD) shear strength parameters were assigned to existing clay soils. This loading scenario assumes compacted fill clays have completed consolidation.
    2. The unconsolidated undrained (UU) shear strengths were assigned to compacted clays that will be placed as a part of proposed modifications, and the consolidated drained shear strength parameters were assigned to existing clay soils. This loading scenario represents loading conditions immediately after the completion of construction, before the fill materials are consolidated.

2. Rapid Drawdown (RDD) condition was analyzed assuming an instantaneous drawdown in the lake from the extreme water level to normal water level. The phreatic surfaces corresponding to peak design storm and MNOP loading conditions were calculated by the steady-state seepage analyses and drawn using the piezometric line definition option. A three-stage RDD analysis (developed by Duncan, Wright and Wong) was performed that accounts for CD and CU strength parameters, as provided for in Slope/W. This method uses effective stresses for the first stage before drawdown, and total stresses and undrained strengths for the second stage after drawdown for low-permeability soils. The lower of

the drained or undrained strengths are used during the third stage (Duncan, Wright and Brandon, 2014) [7]. The factors of safety for the upstream and the downstream slope failures were calculated.

5.5.2.2 Slope Stability Results

The results from the slope stability analyses performed are summarized in Table 5.7. Seepage and slope stability figures are provided in Appendix D. TCEQ does not provide a minimum factor of safety for fully softened strength condition but a FS of 1.25 was adapted for shallow slope stability. Results show the proposed design meets or exceeds all FS requirements for the analyzed loading conditions.

Table 5.7: Slope Stability Summary of Results

Loading Condition	Shear Strength Assignment	Slope Analyzed	Tension Crack	Calculated Factor of Safety	Min. Factor of Safety
SSS, Extreme Loading Condition	CD Strengths	Downstream	--	1.53	1.5
			Included	1.50	1.5
		Upstream	--	2.58	1.5
	FSS Embankment, CD Foundation Clays	Downstream	Included	1.29	1.25
		Upstream		2.58	1.25
	CU Strengths to Compacted Fill, CD Strengths to Existing Embankment and Foundation Clays	Downstream	--	1.60	1.25
		Upstream	--	2.58	1.25
	UU Strengths to Compacted Fill, CD Strengths to Existing Embankment and Foundation Clays	Downstream	--	1.64	1.25
SSS, Normal Loading Condition	CD Strengths	Downstream	--	1.81	1.5
			Included	1.80	1.5
		Upstream	--	2.96	1.5
	FSS Embankment, CD Foundation Clays	Downstream	Included	1.70	1.25
		Upstream		2.77	1.25
	CU Strengths to Compacted Fill, CD Strengths to Existing Embankment and Foundation Clays	Downstream	--	1.87	1.25
		Upstream	--	2.90	1.25
	UU Strengths to Compacted Fill, CD Strengths to Existing Embankment and Foundation Clays	Downstream	--	1.92	1.25
RDD	CD and CU Strengths	Downstream	--	1.79	1.25
		Upstream	--	2.70	1.25

6.0 PERMITTING AND REGULATORY REVIEW

Permitting and regulatory requirements associated with the project outside state dam safety requirements are being evaluated and coordinated separately by the SAWS project team. This section is provided as a reference to identify potential regulatory requirements that may be associated with the proposed improvements to the dam.

6.1 WATERS OF THE U.S.

Construction activities are proposed in areas that may be regulated by the U.S. Army Corps of Engineers (USACE). Any discharge of fill into jurisdictional Waters of the U.S. (WOTUS) will require a 404 permit [21]. Further coordination with USACE will be required and is being conducted separately by the project team.

6.2 CULTURAL RESOURCES

An investigation of the cultural resources at Mitchell Lake Dam was completed in early 2020 [4]. The report recommended that four resources at the site were eligible for listing on the National Register of Historic Places: the embankment dam, the floodgates, the spillway and discharge channel, and the plunge pool (termed “purge pond” in the report). A memo documenting conceptual improvements to Mitchell Lake Dam was reviewed as a part of the investigation. The study concluded that the proposed improvements would have no adverse effect on the resources recommended as eligible for the National Register. Further coordination may be required during design if proposed modifications from the dam could adversely impact the identified resources. In addition, archeological resources studies may require further consultation, and these can be initiated once the definition of design reaches a point that sets the final footprint of the project improvements.

6.3 STATE REGULATORY REQUIREMENTS

6.3.1 *Water Rights*

SAWS owns Mitchell Lake and associated water rights through Certificates of Adjudication (COA) 19-2153, 19-2153A, 19-2153B, and 19-2154. COA 19-2153 authorizes SAWS to maintain the Mitchell Lake dam and reservoir and impound therein up to 2,640 acre-feet of water. A special provision requires that SAWS maintain a suitable outlet to allow the free passage of water it is not authorized to impound. SAWS has followed the practice of leaving the spillway gates in the fully open position allowing the free passage of water through the spillway when water levels exceed the existing invert elevation of these gates at elevation 520.76 feet. At that elevation, Mitchell Lake impounds approximately 2,088 acre-feet of water.

SAWS has coordinated with TCEQ on amending their water rights to include discharge to and storage of water within the proposed constructed wetlands. The proposed spillway elevation and wetland volumes are based on the allowable impoundment within the existing water rights. SAWS has received a draft amendment from TCEQ and expects an expedited amendment to follow.

6.3.2 *General Land Office*

State-owned lands, including the bed and banks of navigable waterways, are managed by the General Land Office (GLO). A GLO easement is often required if infrastructure is placed below the "high bank" of a state-owned bed and bank. GLO easements are granted for a fee and would require a survey, which identifies the proposed location of infrastructure, high bank, and adjacent parcel boundaries. Further coordination may be required with GLO during final design.

6.3.3 *Texas Parks and Wildlife Department*

Cottonmouth Creek may include stream bed claimed by the GLO. A Marl, Sand, Gravel, Shell, or Mudshell Permit is required from TPWD if a project is anticipated to take materials from a state-claimed stream bed. The proposed modifications may involve limited removal of material from the streambed. Therefore, final design should include TPWD coordination to determine the need for this permit.

An Aquatic Resource Relocation Plan (ARRP) is needed from the TPWD for construction or maintenance projects when dewatering and the diversion of water is anticipated to occur that would strand aquatic organisms (e.g., cofferdam, open-trenching, etc.). The ARRP describes how stranding would be avoided by collecting and transporting aquatic organisms to another location. A permit to introduce fish, shellfish or plants would be required by the TPWD to relocate fish, freshwater mussels, and other organisms away from the site from which they are collected. Aquatic resource relocation can be conducted by the construction contractor, SAWS, or a third party during construction.

6.4 OTHER POTENTIAL PERMITTING AND REGULATORY REQUIREMENTS

The project is located in the City of San Antonio and is subject to the city's environmental regulations and local floodplain ordinances. The proposed modifications to the dam may have an impact on upstream and downstream regulatory floodplains. Due to the uniqueness of the project, coordination with the City floodplain manager will be required to describe the project and determine actions needed for compliance with the floodplain ordinances. Potential actions may include conducting a floodplain study to identify changes within the floodplain. The results of the study would determine whether the impact to the floodplain is significant enough to make a change to the flood risk maps. If so, the City may require SAWS to prepare and submit a Letter of Map Revision (LOMR) request to FEMA.

Additionally, the proposed project involves clearing a significant number of trees along the downstream side of the dam. The final design should include a tree survey, and a tree permit may be required for the project. Finally, FNI understands a survey for threatened and endangered species has been conducted for the dam. The final design may require addition survey for any threatened and endangered species that could be impacted by proposed improvements.

7.0 PRELIMINARY CONSTRUCTABILITY REVIEW

Certain constructability factors can significantly affect construction costs. FNI performed a preliminary constructability review of the project to identify any major cost risks and account for them in the OPCC. The constructability review is summarized in Table 7.1.

Table 7.1: Preliminary Constructability Review

Site Issues	
Property Ownership, Easements, and Right of Entry	Based on the preliminary project layout, SAWS will own or have the right to access all property involved in the project. Additional evaluation of flood frequency and property ownership around the reservoir rim should be performed for final design.
Access, Staging, and Clearance	Several options are available for staging. Traffic control or closure of the hike and bike trail may be required.
Availability of Materials	Ready-mix concrete and rock riprap are readily available in the area. The proposed layout has an excess of soil due to spillway channel excavation. The preliminary geotechnical evaluation suggests that the material should be suitable and the OPCC assumed no imported fill material.
Existing Infrastructure Impacts	There is a major utility corridor with overhead electric transmission lines and subsurface utility lines immediately downstream of the dam. Conduits to the proposed wetlands may conflict with existing lines. Subsurface utility location is recommended for final design to reduce risks. Impacts to extant irrigation features and the hiking trail will be coordinated during design.
Clearing/Grubbing	Significant clearing and grubbing of woody vegetation along the downstream toe of the embankment will be required. Coordination with the City of San Antonio may be required to comply with local tree ordinances and obtain a building permit.
Drainage	Site generally has positive drainage.
Urban vs. Rural	Site is located in an urban area. Traffic and dust control may be required. Allowable working hours for the contractor may be limited.
Dewatering	The project is located in a natural watercourse subject to flooding. Much of the embankment is founded on the abutment above the normal reservoir level, but portions of the embankment and the spillway will be subject to hydrologic risk which will likely require construction of a cofferdam.
Site Utilities	Electricity and water are not available at the site but may be available at a potential staging area on Pleasanton Road at the west end of the dam.

Construction Scheduling	
Seasonal Considerations	Construction may affect migratory bird activity on Mitchell Lake.
Weather	Average annual rainfall: 31 inches Average annual precipitation days: 81 Average annual days >90°F: 116 Average annual days <32°F: 20 Average summer highs may require consideration for concrete mix design and placement.
Long Lead Time Items	N/A
Ongoing Projects	Construction of wetlands will require coordination if performed under a separate contract.
Construction Items	
Unique, Special, or Proprietary Work	The project will require excavation and disposal of potentially contaminated soils within the reservoir to create an entrance channel to the spillway. Sediment testing will be required to determine disposal requirements.
Deep Excavations	N/A

8.0 COST OPINION

This section documents the basis of the Class 4 Opinion of Probable Construction Cost (OPCC) for the proposed project. Per AACE, a Class 4 estimate is suitable for use in assessing the feasibility of a project but should not be used for establishing project budgets. Based on AACE guidelines and experience on similar projects, FNI estimates that the true project construction cost for the proposed concept can generally be expected to fall within -30 to +50 percent of the Class 4 OPCC [1].

8.1 RISKS AND CONTINGENCY

An OPCC is necessarily an approximation and thus has an inherent level of uncertainty. Unit prices and quantities associated with each line item shown in the OPCC are subject to variability. The capacity and capabilities of contractors are highly variable. Many construction projects encounter cost requirements during final design, bidding and/or construction that could not have been reasonably identified during the preliminary design phase. Major cost risks identified for the project include the following:

- Project Scope: The proposed project layout is preliminary, and some of the engineering challenges associated with the project remain outstanding. As a project matures, the design concept naturally changes with new information and refinement. Additionally, as the owner becomes more invested in the project, additional considerations may warrant design changes.
- Care of Water: As the project is located within a major watercourse, managing and diverting normal and flood flows will represent a major construction cost. The magnitude of these measures will depend on the allowable reservoir elevation during construction and a detailed assessment of potential flood risks.
- Subsurface Conditions: Design of improvements to the dam will depend significantly on further assessment of subsurface conditions, including borrow material availability assessment, refinement of slope stability and seepage analysis, and estimates of foundation strength and settlement potential. Future design phases and on-site investigations may reveal conditions which are different from those assumed for the preliminary design.
- Existing Utilities: A major utility corridor is located adjacent to the project site, and potential conflicts with the proposed layout have not been assessed. Relocation of utilities to accommodate the proposed project could represent a major project cost.
- Permitting: The design and construction schedule can depend heavily on regulatory permitting requirements, including USACE 404 permitting and obtaining clearance from the Texas Historical Commission. Schedule delays can lead to uncertainty in project costs. Also, future design phases may consider project alternatives which reduce permitting obligations at the expense of increased construction costs.
- Economic Uncertainty: Changes to tariffs on construction commodities, labor market fluctuations, oil price variability, and natural disasters can significantly impact construction costs.

An overall contingency of 35 percent has been included in the OPCC to account for project uncertainties. This amount was selected based on the quantity and quality of available information for design, the current maturity of the project, experience on similar past projects, and engineering judgment. The contingency is the cost assigned to the average of the unknowns in the definition of the project. It is intended to account for construction costs that have not yet been identified due to the project maturity and should be expected to be fully used for construction. In addition to overall contingency, line item allowances have been included in the OPCC to account for project components that have not yet been confirmed.

Please note that many project owners or funding agencies include their own contingency to a budgetary allocation to establish the amount of funding necessary to construct the project. This additional contingency is intended to provide a ceiling so that costs are more likely to fall below the budget allocation and additional funding requests are avoided. This additional contingency has *not* been included in the OPCC.

8.2 PRICE BASE

Unless otherwise stated, all dollar values presented in this memorandum can be assumed to be nominal values with a price base of 2020. If values are to be used in a year other than 2020, they should be adjusted for factors that affect nominal prices of construction over time, as appropriate.

8.3 PROJECT COST

The Class 4 OPCC for the representative alternative of the Mitchell Lake Dam Improvements project is $11.6M. Table 8.1 presents the detailed basis of the OPCC. Potential project costs not counted below include engineering, permitting, easement or property acquisition, legal costs, public outreach, owner administration and project management, construction inspection, and ongoing operation and maintenance costs.

Table 8.1: Opinion of Probable Construction Cost

Item	Description	Qty	Unit	Unit Price	Total
1	Mobilization and Temporary Facilities	1	LS	$630,000	$630,000
2	Erosion and Sediment Controls	1	LS	$100,000	$100,000
3	Clearing and Grubbing	10	AC	$6,500	$65,000
4	Care of Water	1	LS	$400,000	$400,000
5	Demo of Existing Irrigation Features	1	LS	$50,000	$50,000
6	Demo of Existing Concrete Trail	230	SY	$50	$11,500
7	Topsoil Stripping and Stockpiling	5,100	CY	$20	$102,000
8	Required Excavation	39,000	CY	$8	$312,000
9	Disposal of Contaminated Soil	3,700	CY	$75	$277,500
10	Compacted Fill	12,700	CY	$10	$127,000
11	Rock Riprap and Bedding (Embankment)	8,500	SY	$130	$1,105,000
12	Flexible Base Crest Road	6,200	SY	$20	$124,000
13	Concrete Trail Realignment	240	SY	$50	$12,000
14	Concrete Labyrinth Weir	230	CY	$1,300	$299,000
15	Con. Labyrinth Platform, Chute, and Stilling Basin	3,000	CY	$750	$2,250,000
16	Concrete Training Walls	550	CY	$1,000	$550,000
17	Outlet Works Structures	2	EA	$250,000	$500,000
18	Concrete Drop Structure	1	LS	$50,000	$50,000
19	Spillway Internal Drainage System	1	LS	$350,000	$350,000
20	Rock Riprap (Spillway)	1,250	SY	$150	$187,500
21	24" Principal Spillway Conduits	580	LF	$275	$159,500
22	Instrumentation	1	LS	$100,000	$100,000
23	Site Restoration and Reseeding	8	AC	$5,000	$40,000
24	Allowance: Misc. improvements (security, access, etc.)	1	LS	$400,000	$400,000
25	Allowance: Historic preservation of ex. spillway structure	1	LS	$100,000	$100,000
26	Allowance: Landscaping, signage, and aesthetic upgrades	1	LS	$250,000	$250,000
			PROJECT SUBTOTAL:		$8,562,000
			CONTINGENCY:	35%	$3,038,000
			TOTAL OPINION OF PROBABLE CONSTRUCTION COST:		**$11,600,000**

9.0 CONCLUSION

Mitchell Lake is an important amenity in San Antonio. FNI performed a detailed review of previous documentation to evaluate how the condition of the dam has changed over time and understand the various plans that have been considered for its improvement. FNI also performed a site visit to the dam to observe its current condition. Generally, the embankment was in fair condition and the spillway was in poor condition. Additionally, the dam under existing conditions is not capable of safely passing the required design storm event under state dam safety regulations. Finally, as a part of the Mitchell Lake Wetlands Quality Treatment Initiatives project, changes to the operations and releases of stored water in the reservoir are required. This report documents the preliminary engineering performed for the Mitchell Lake Dam Improvements project to address the issues noted above, including the basis of design for the preliminary project layout. The proposed improvements consist of the following:

- Regrading and raising the Mitchell Lake Dam embankment to have a consistent crest elevation and stable slopes
- Installation of rock riprap on the upstream embankment slope for wave action protection
- Installation of a new labyrinth weir auxiliary spillway
- Installation of two principal spillways and conduits to divert flood pool storage to the proposed constructed wetlands

FNI performed preliminary hydrologic, hydraulic, and geotechnical analyses to support the proposed improvements to the dam. FNI also performed a review of potential permitting and regulatory requirements and potential constructability issues for the project. The opinion of probable construction cost for the proposed improvements is $11.6M. Outstanding decisions related to the dam modifications to be considered for final design include the following:

- Means of vehicle and/or pedestrian access across the proposed auxiliary spillway
- Site security, fencing, buoys, and warning signs
- Aesthetic enhancements of the project, including facilities for public overlook, landscape architecture, interpretive signage, and connection to the existing hike-and-bike trail
- Specific measures required to stabilized and preserve the existing spillway structure

10.0 REFERENCES

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Appendix A: Site Visit Photo Log

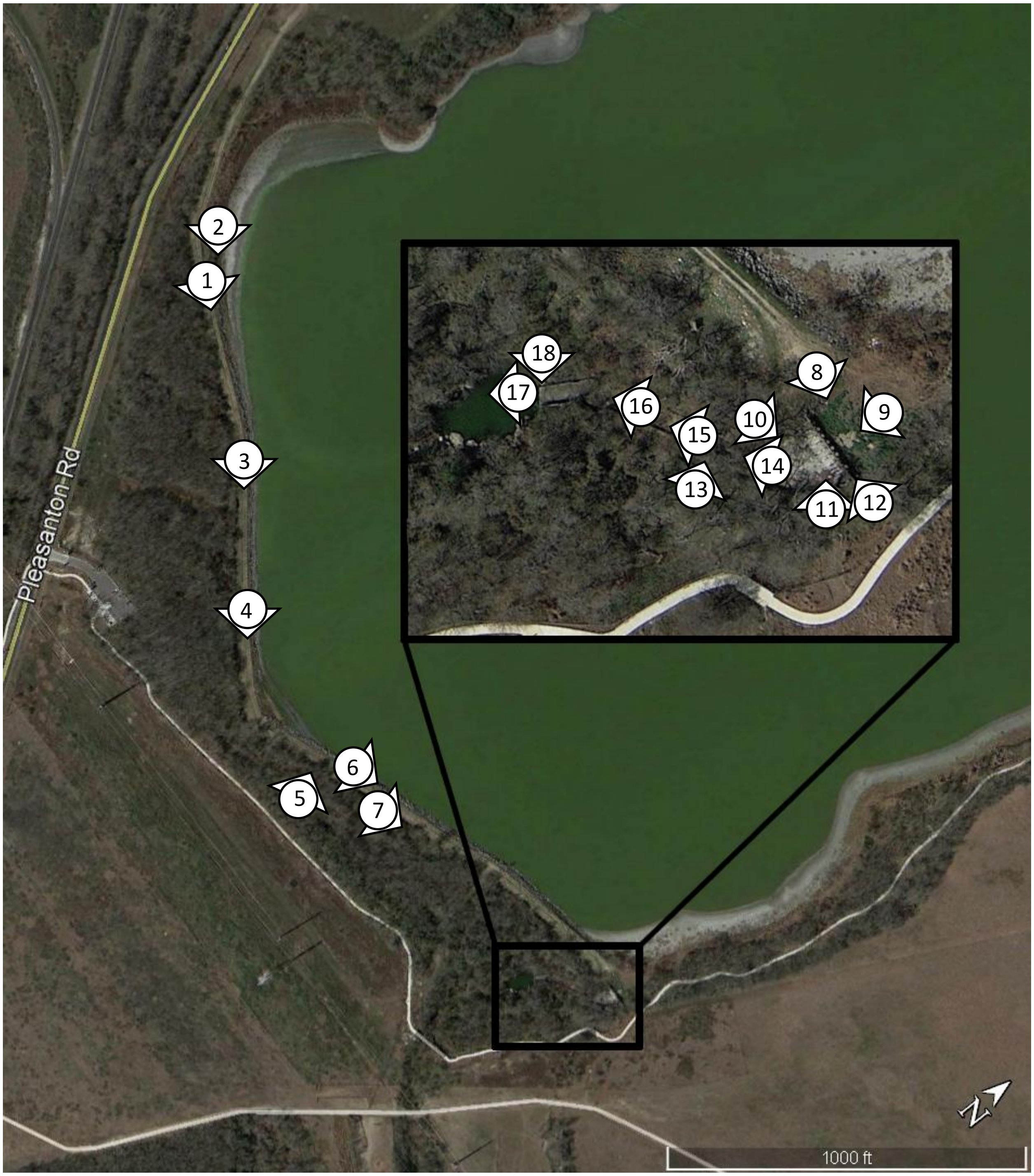


Photo Location Map

Photo 1: Embankment Upstream Face (Note steepness and sparse coverage of rubble erosion protection)

Photo 2: Dense Vegetation at Toe of Upstream Slope

Photo 3: Upstream Embankment Slope

Photo 4: Upstream Embankment Slope

Photo 5: Headwall Structure on Downstream Slope

Photo 6: Upstream Embankment Slope

Photo 7: Downstream Embankment Slope (Note steepness and dense vegetation)

Photo 8: Spillway (Looking left from upstream)

Photo 9: Spillway Upstream Face and Gates

Photo 10: Spillway Downstream Face and Channel (Looking left)

Photo 11: Spillway Downstream Face and Channel (Looking right)

Photo 12: Spillway Bulkhead Catwalk and Gate Handwheels

Photo 13: Irrigation Culvert Crossing over Spillway Channel

Photo 14: Spillway Channel Looking Downstream

Photo 15: Spillway Channel Looking Downstream

Photo 16: Spillway Channel Looking Downstream

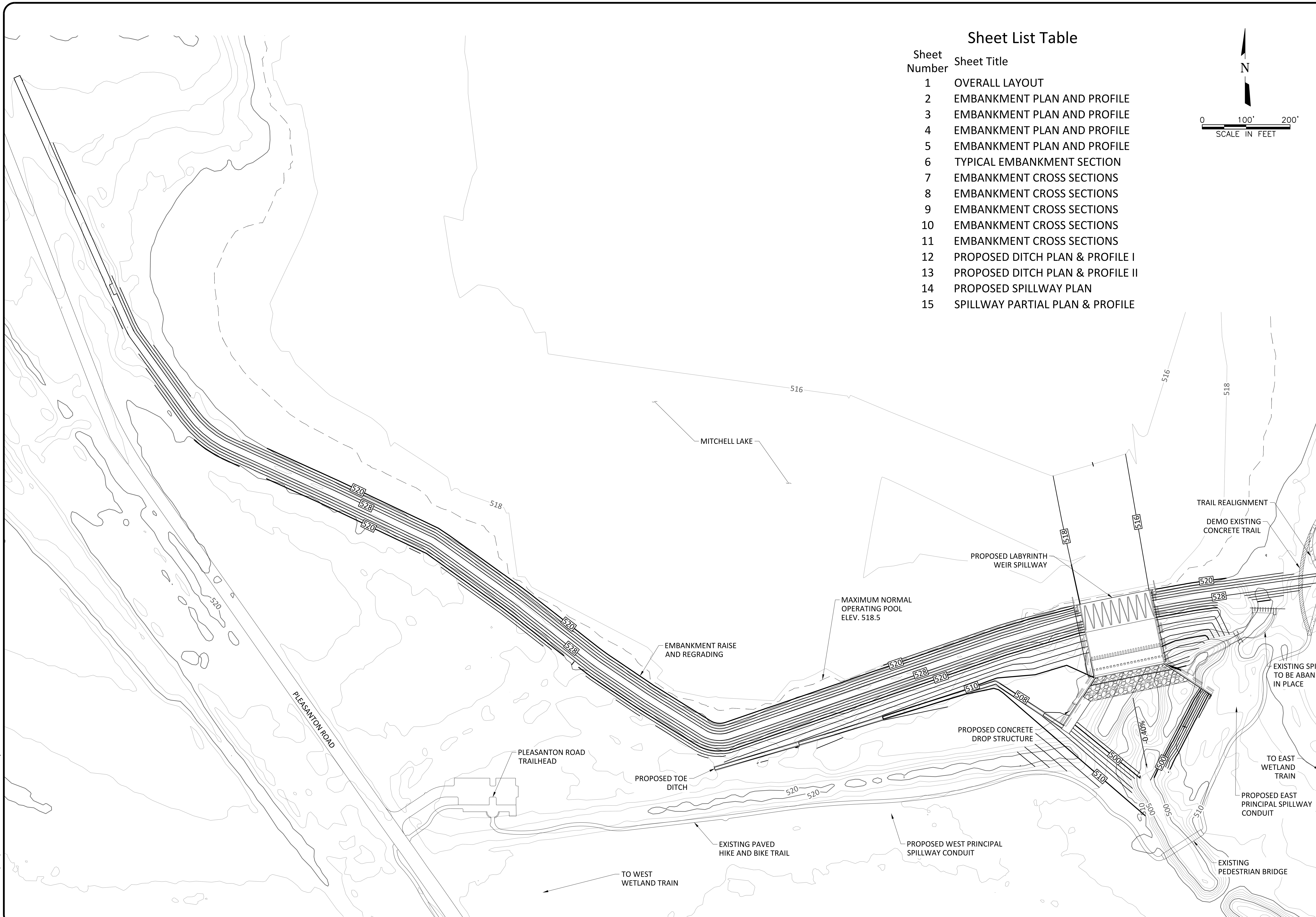
Photo 17: Plunge Pool Looking Downstream

Photo 18: Spillway Channel Wall (Note exposed reinforcing steel and sheer drop)

Appendix B: Preliminary Project Layout Drawings

Sheet List Table

Sheet Number	Sheet Title
1	OVERALL LAYOUT
2	EMBANKMENT PLAN AND PROFILE
3	EMBANKMENT PLAN AND PROFILE
4	EMBANKMENT PLAN AND PROFILE
5	EMBANKMENT PLAN AND PROFILE
6	TYPICAL EMBANKMENT SECTION
7	EMBANKMENT CROSS SECTIONS
8	EMBANKMENT CROSS SECTIONS
9	EMBANKMENT CROSS SECTIONS
10	EMBANKMENT CROSS SECTIONS
11	EMBANKMENT CROSS SECTIONS
12	PROPOSED DITCH PLAN & PROFILE I
13	PROPOSED DITCH PLAN & PROFILE II
14	PROPOSED SPILLWAY PLAN
15	SPILLWAY PARTIAL PLAN & PROFILE



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San Antonio, Texas 78216
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Web - www.freese.com

SAN ANTONIO WATER SYSTEM

MITCHELL LAKE DAM IMPROVEMENTS PRELIMINARY DESIGN

OVERALL LAYOUT

FN JOB NO.	PLU17623
DATE	04/29/2020
DESIGNED	KB
DRAWN	MG
REVISED	
CHECKED	GH
FILE NAME	CV-PLU-OVERALL-PL.dwg

NO.	ISSUE	BY	DATE

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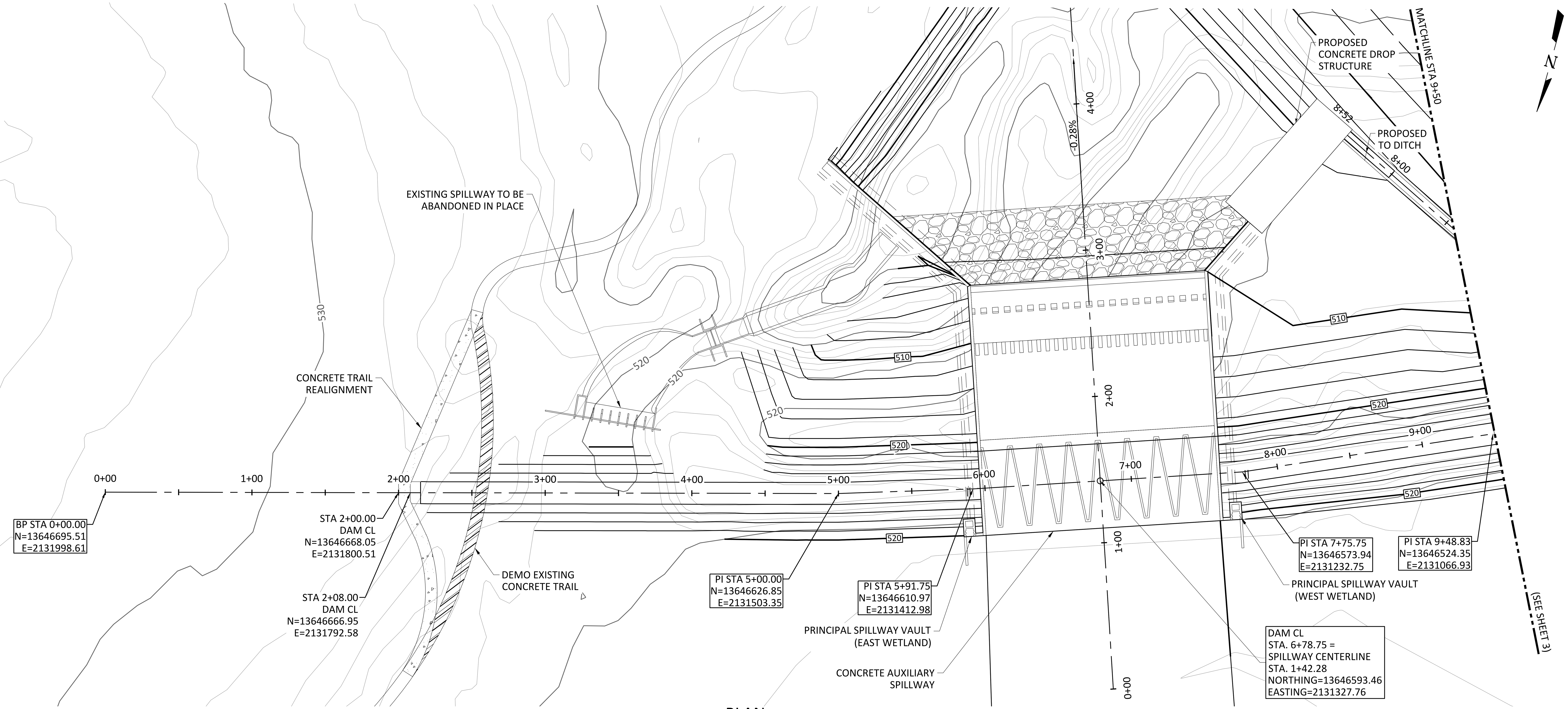

PLAN

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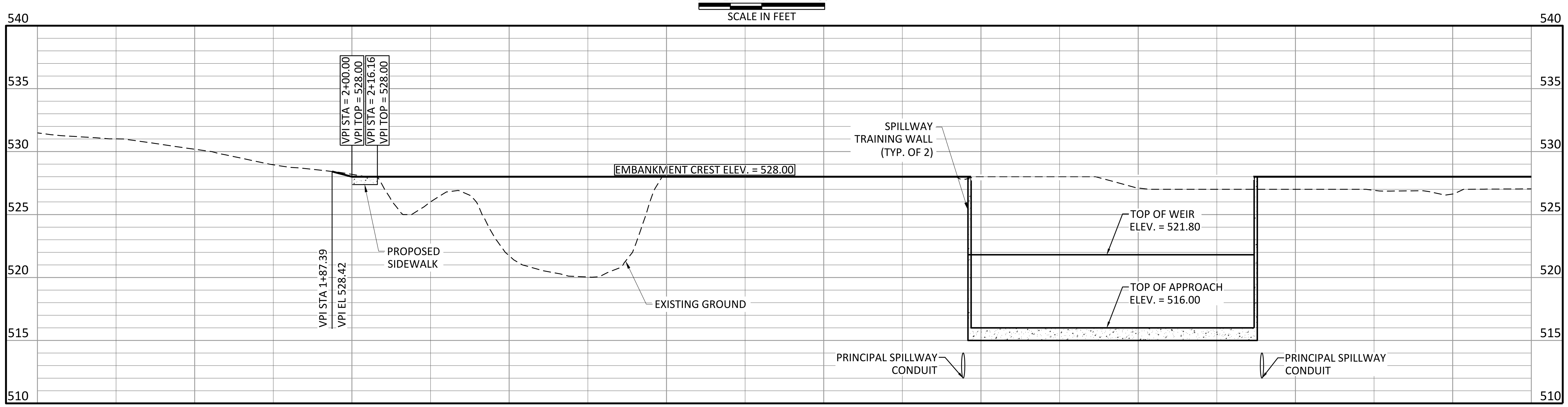

PROFILE ALONG CENTERLINE

0 40' 80'
SCALE IN FEET
HORIZONTAL

0 5' 10'
SCALE IN FEET
VERTICAL

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EMBANKMENT PLAN AND PROFILE

F&N JOB NO.	PLU17623
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FILE NAME	CV-PLU-P&P01.dwg

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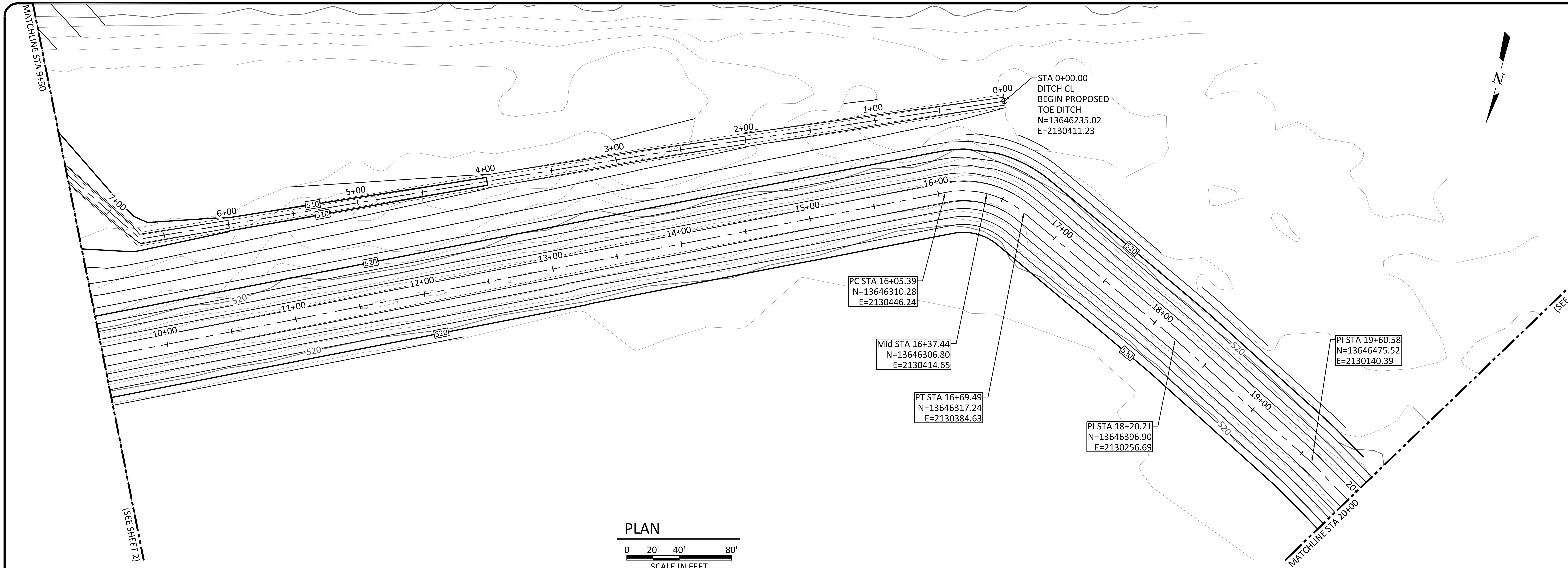

PLAN

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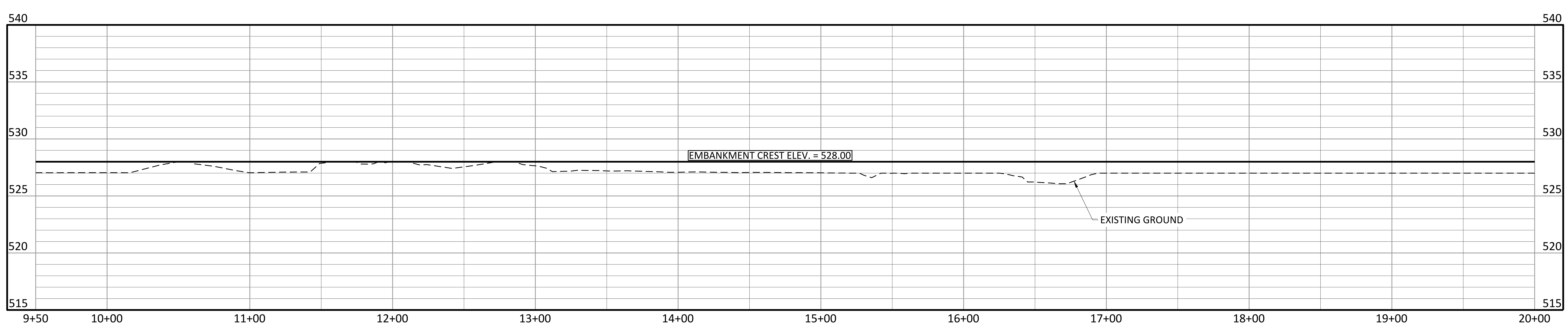

PROFILE

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SCALE IN FEET
HORIZONTAL

0 5' 10'
SCALE IN FEET
VERTICAL

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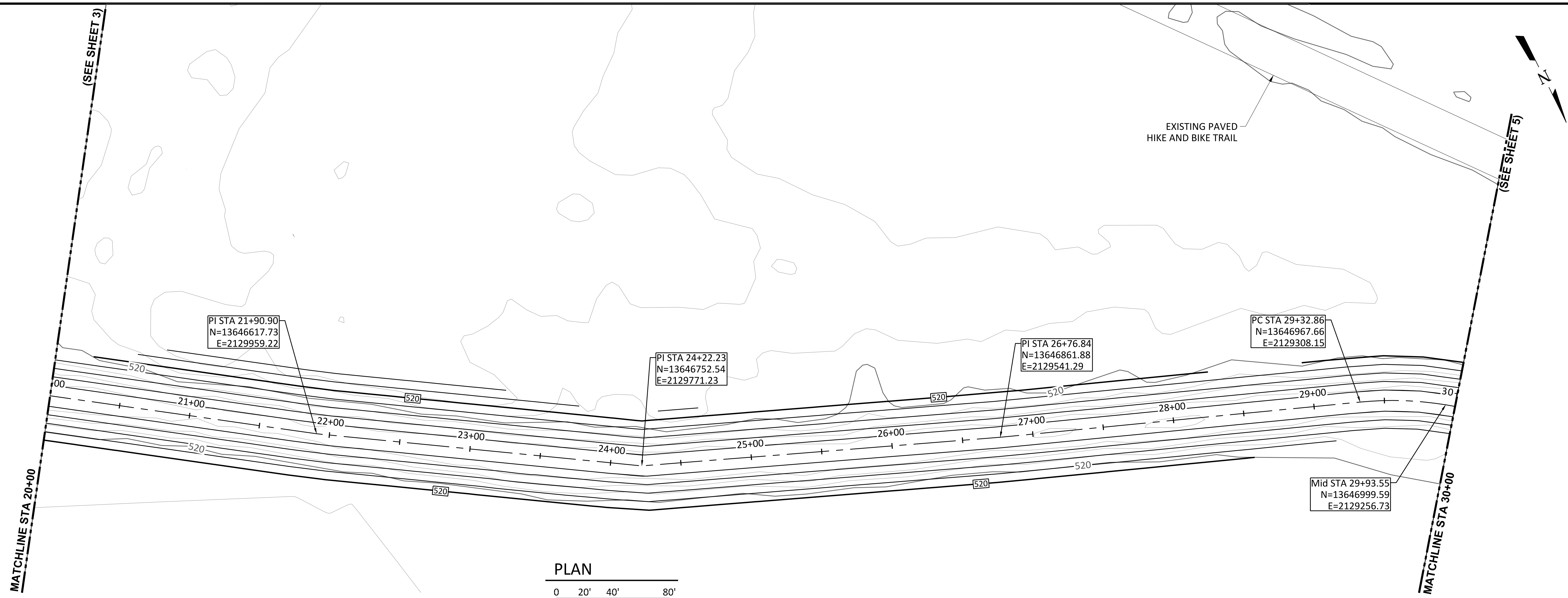

PLAN

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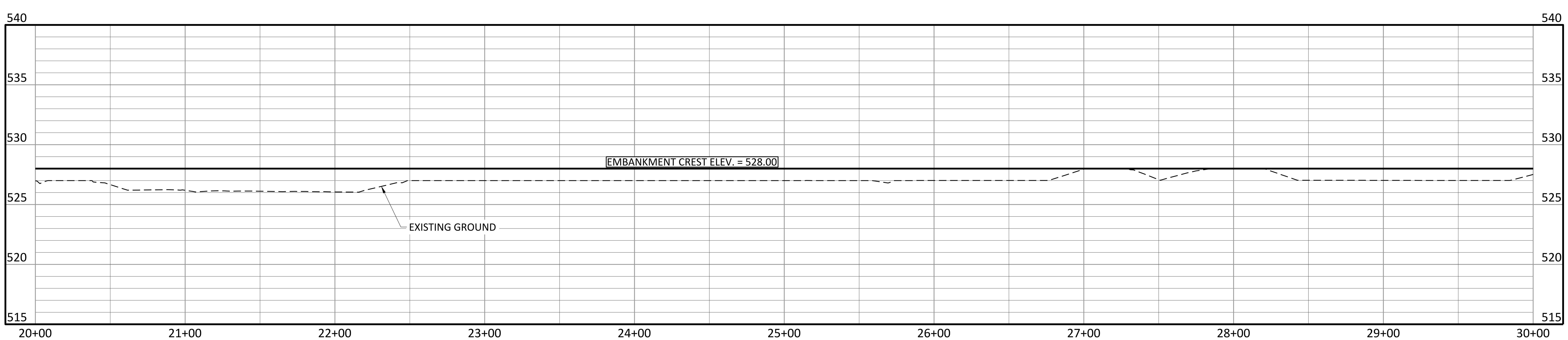

PROFILE

0 40' 80'
SCALE IN FEET
HORIZONTAL

0 5' 10'
SCALE IN FEET
VERTICAL

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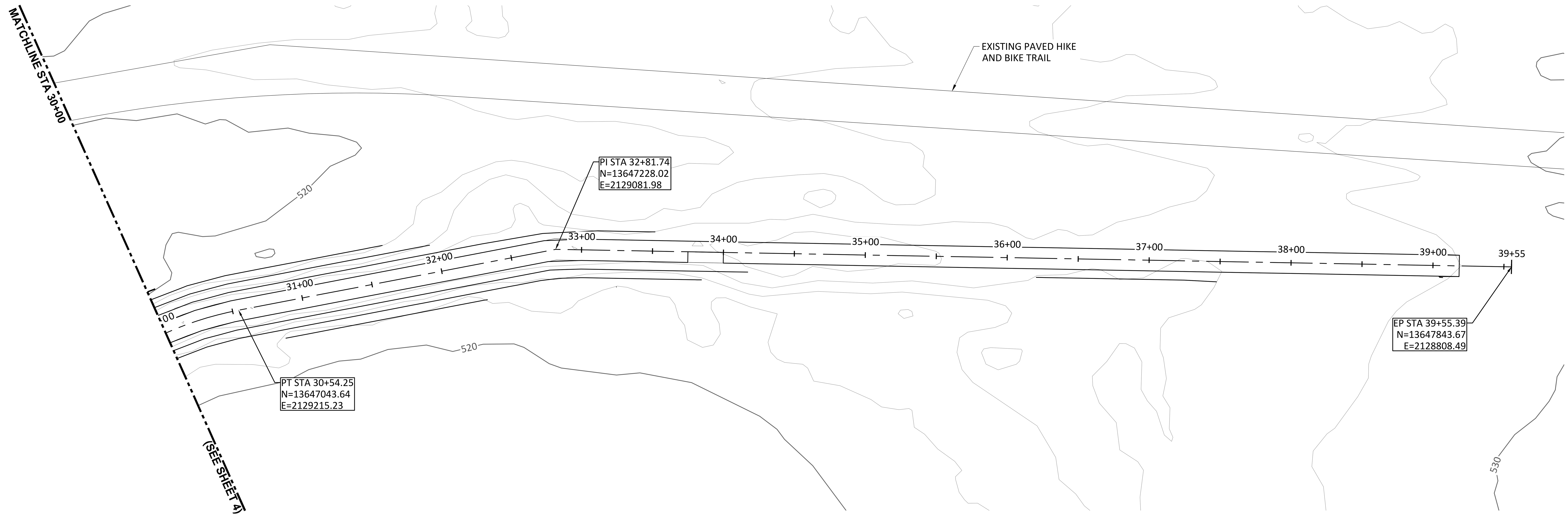

PLAN

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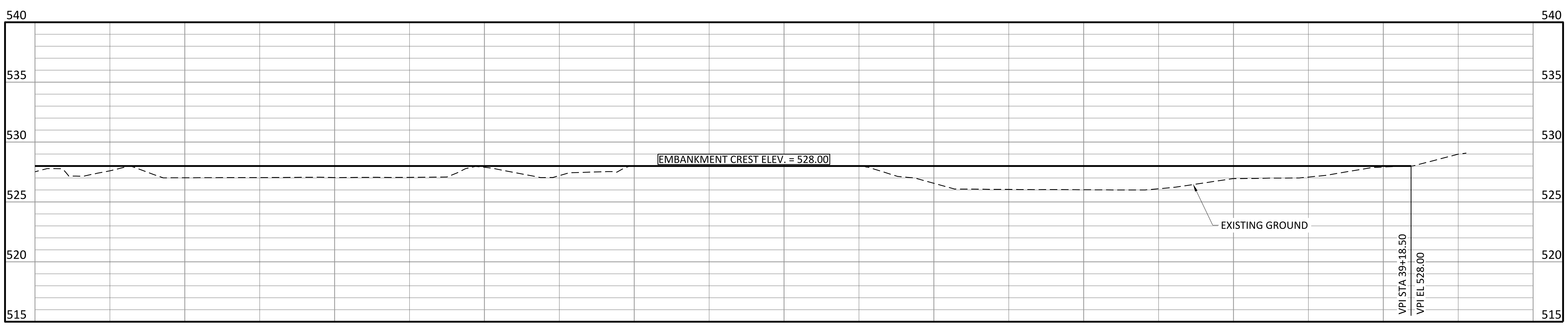

PROFILE

0 40' 80'
SCALE IN FEET
HORIZONTAL

0 5' 10'
SCALE IN FEET
VERTICAL

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EMBANKMENT PLAN AND PROFILE

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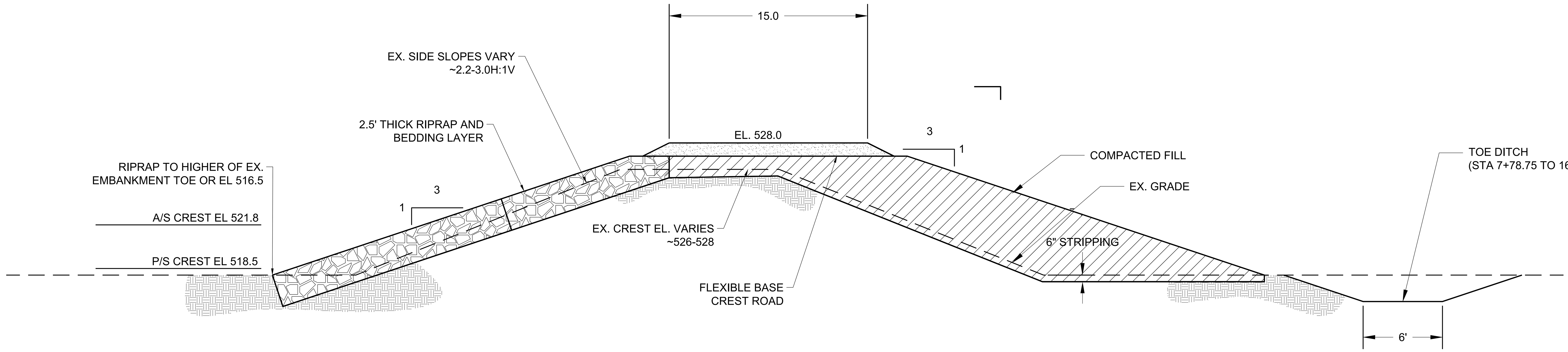
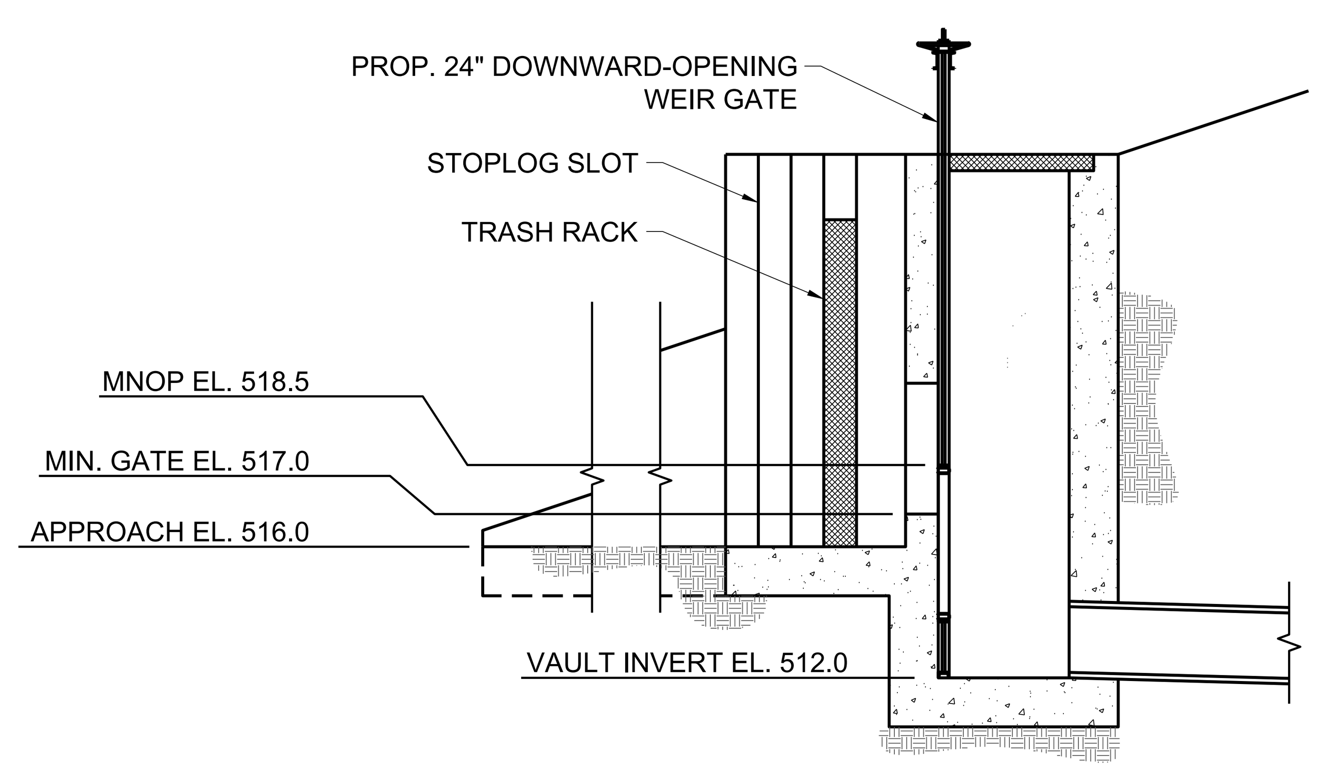


1 TYPICAL EMBANKMENT SECTION
1"=5'



2 PRINCIPAL SPILLWAY VAULT SECTION (TYP)
1"=5'

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TYPICAL EMBANKMENT SECTION

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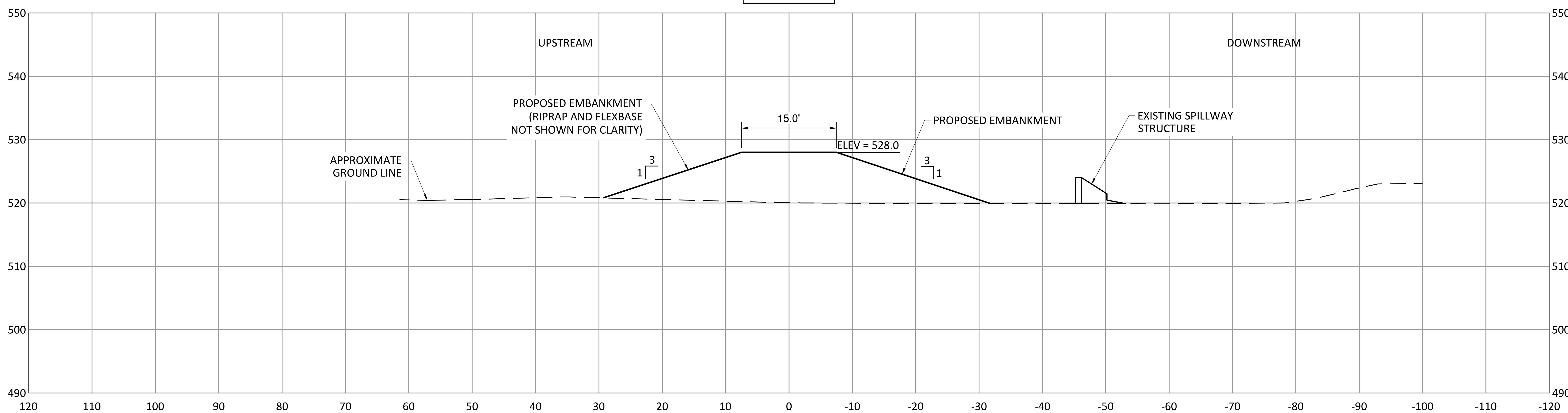

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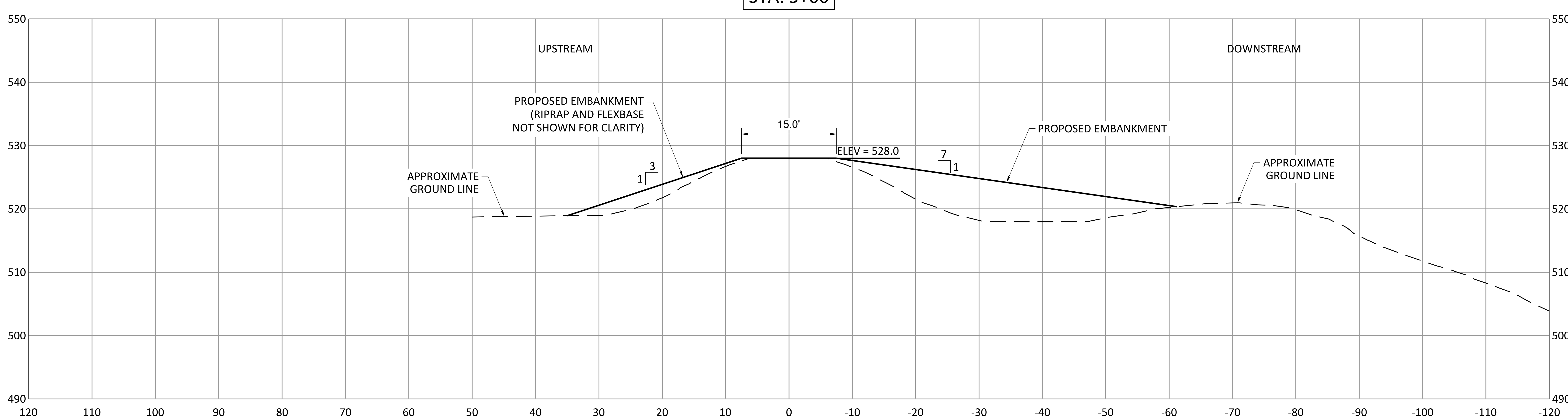
STA. 5+00



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EMBANKMENT CROSS SECTIONS

F&N JOB NO.	PLU17623
DATE	04/29/2020
DESIGNED	KB
DRAWN	MG
REVISED	
CHECKED	GH
FILE NAME	CV-PLU-XSECT.dwg

NO.	ISSUE	BY	DATE

VERIFY SCALE 0 1
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SHEET
7

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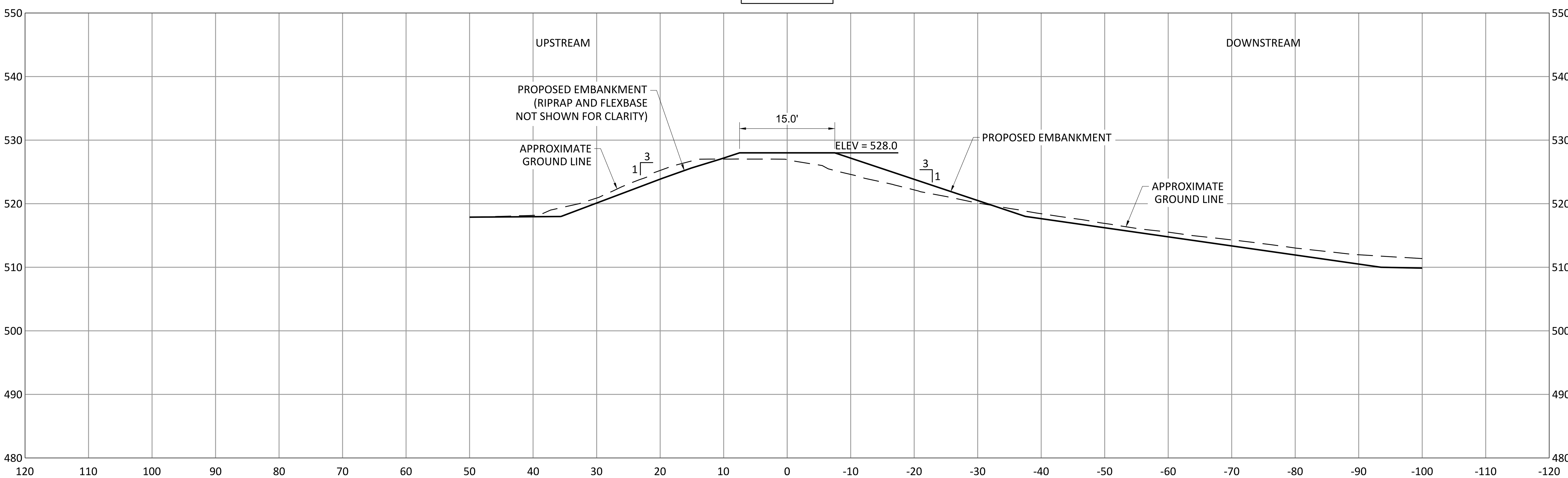

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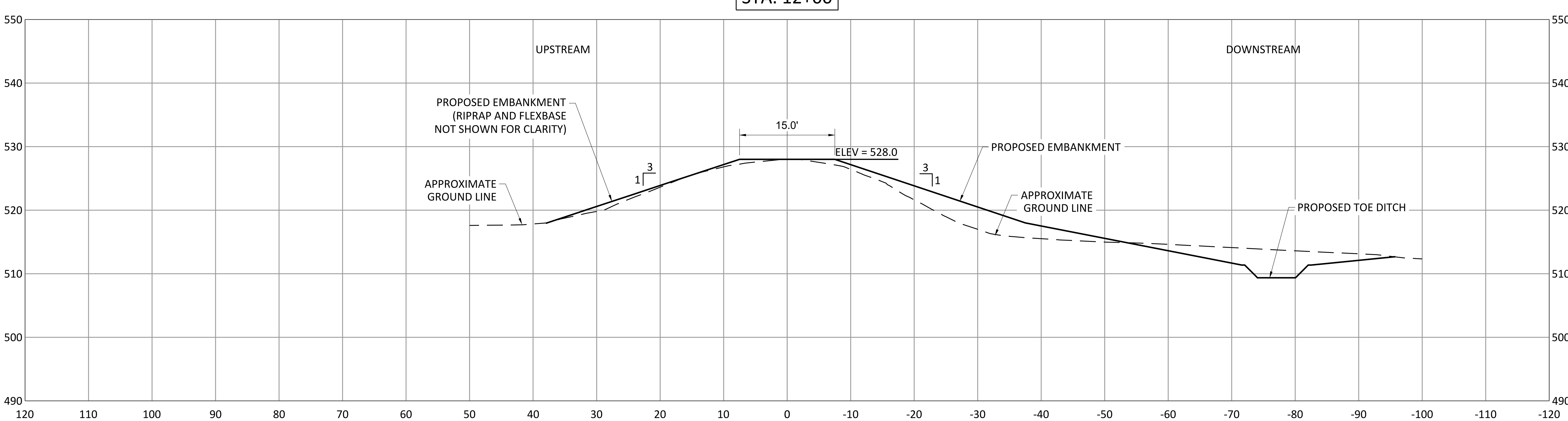
STA. 12+00



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MITCHELL LAKE DAM IMPROVEMENTS PRELIMINARY DESIGN

EMBANKMENT CROSS SECTIONS

F&N JOB NO.	PLU17623
DATE	04/29/2020
DESIGNED	KB
DRAWN	MG
REVISED	
CHECKED	GH
FILE NAME	CV-PLU-XSECT.dwg

NO.	ISSUE	BY	DATE

VERIFY SCALE 0 1
Bar is one inch on original drawing. If not one inch on this sheet, adjust scale.

SHEET
8

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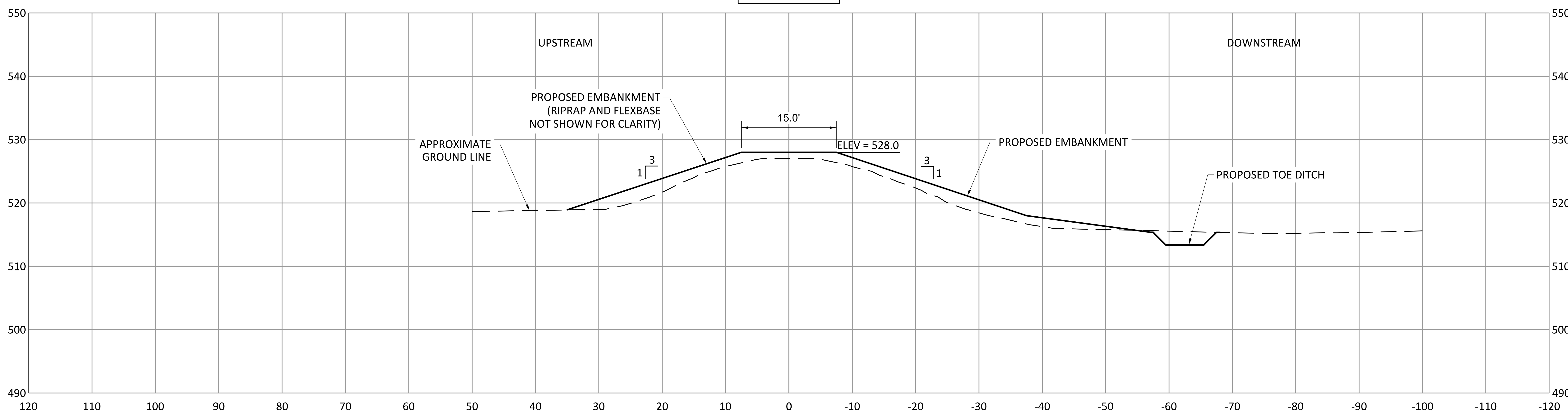

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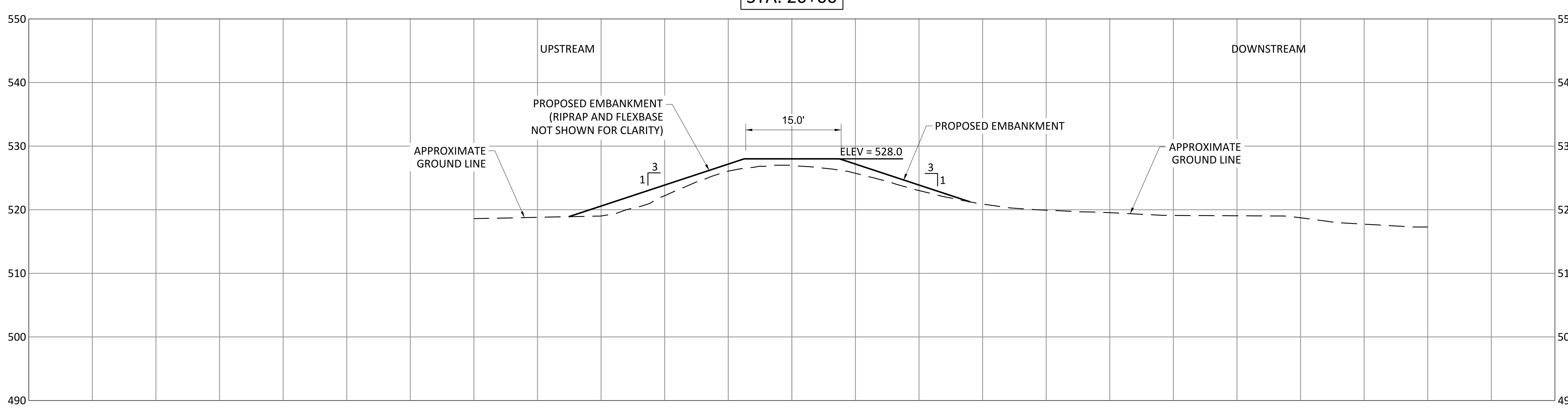
STA. 20+00



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SAN ANTONIO WATER SYSTEM
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EMBANKMENT CROSS SECTIONS

NO.	ISSUE	BY	DATE

F&N JOB NO. PLU17623
DATE 04/29/2020
DESIGNED KB
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REVISED
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FILE NAME CV-PLU-XSECT.dwg

VERIFY SCALE 0 1
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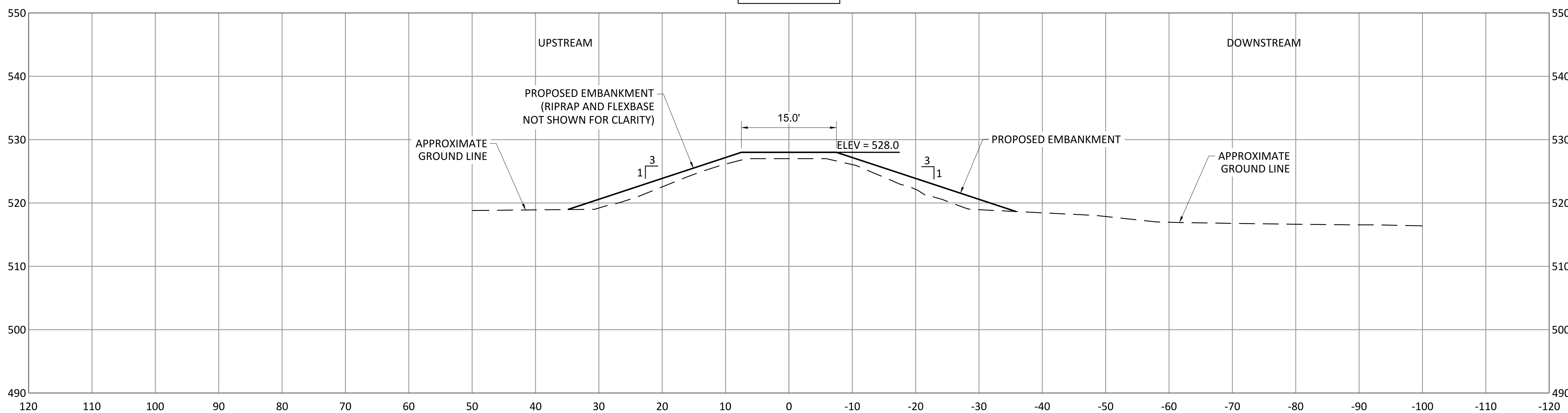

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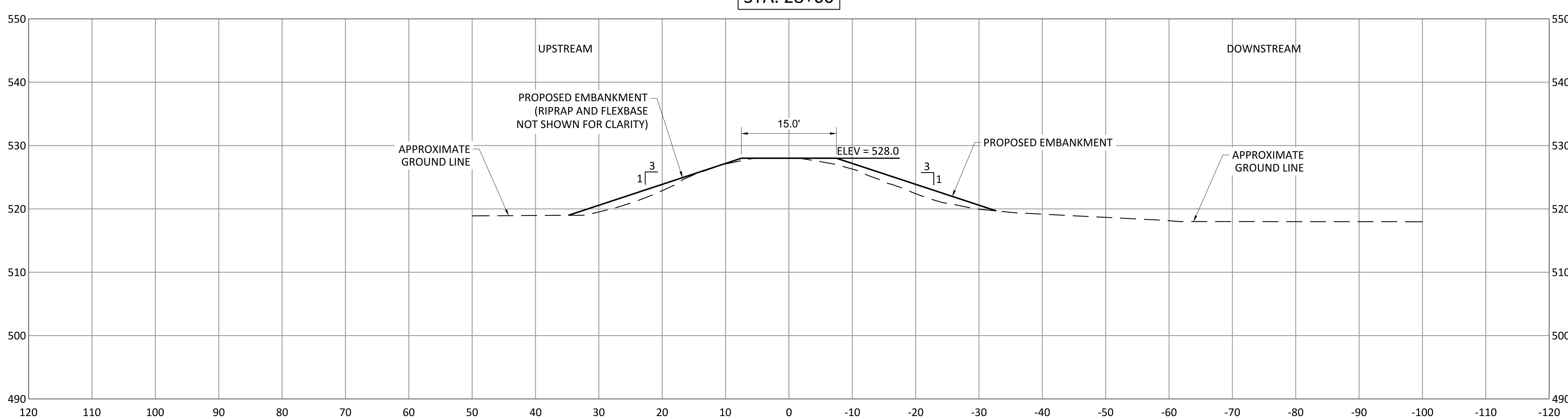
STA. 28+00



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SAN ANTONIO WATER SYSTEM
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EMBANKMENT CROSS SECTIONS

F&N JOB NO.	PLU17623
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DRAWN	MG
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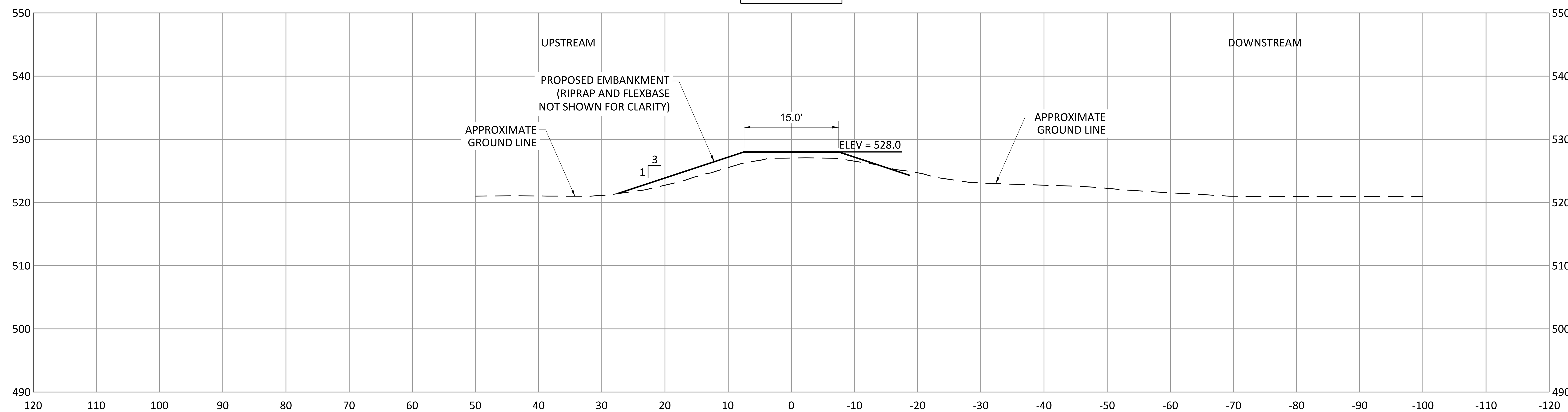
VERIFY SCALE 0 1
Bar is one inch on original drawing. If not one inch on this sheet, adjust scale.

SHEET 10

STA. 32+00



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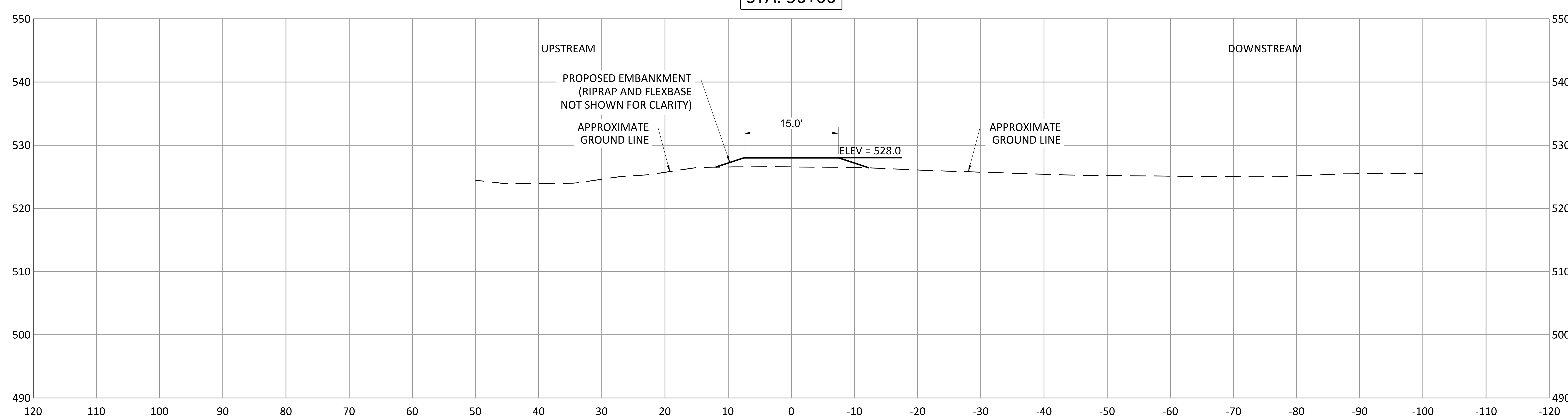
STA. 36+00



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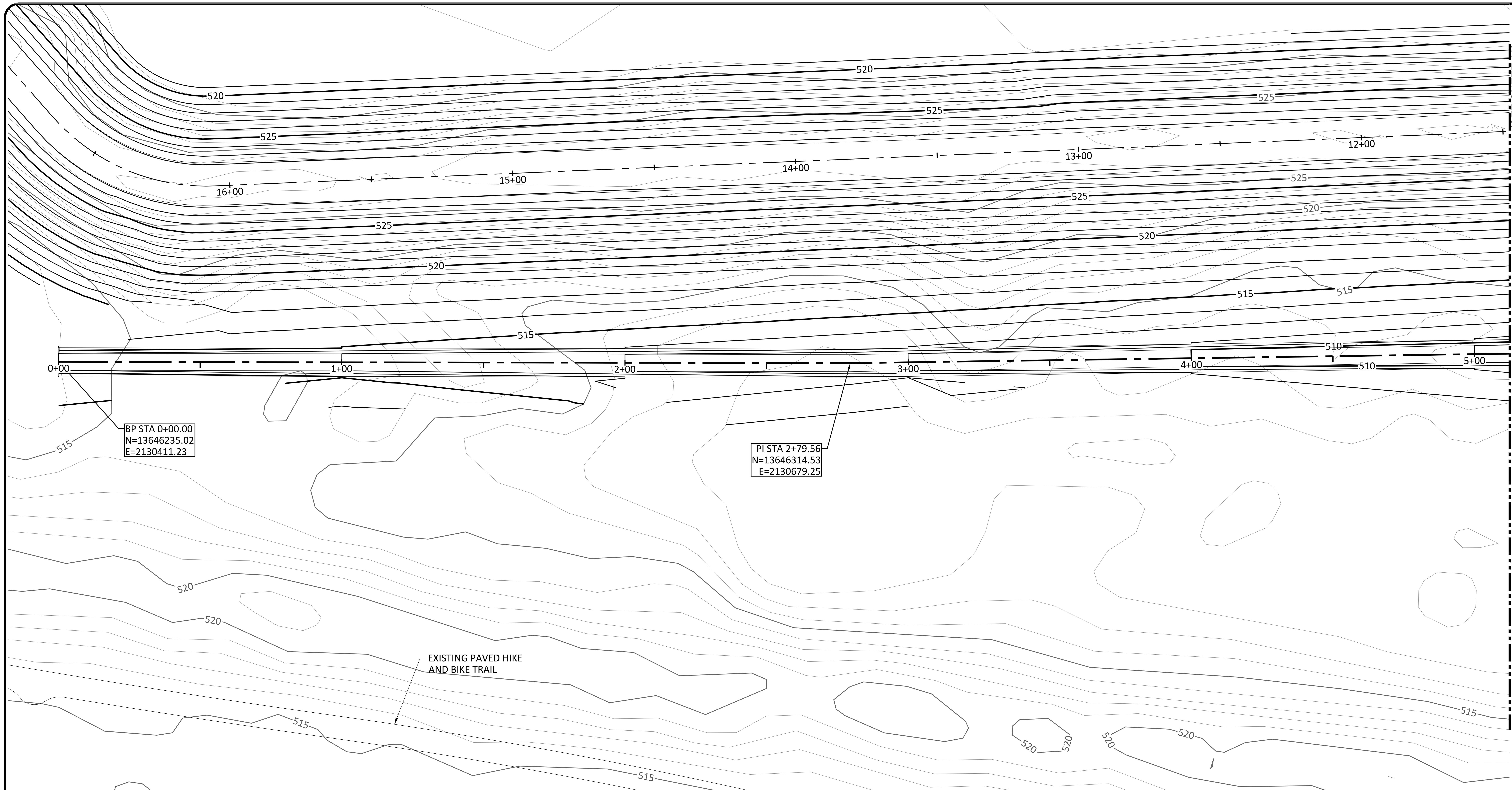

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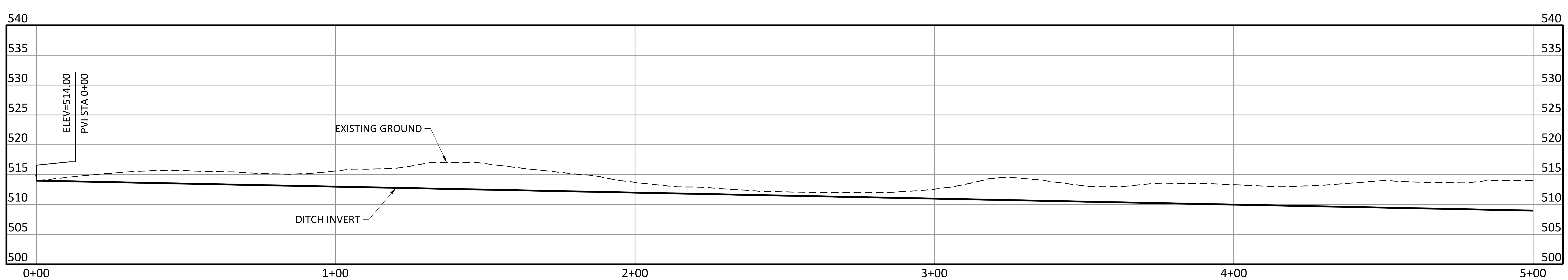

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SAN ANTONIO WATER SYSTEM
MITCHELL LAKE DAM IMPROVEMENTS PRELIMINARY DESIGN

PROPOSED DITCH PLAN & PROFILE I

FN JOB NO.	PLU17623
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DESIGNED	KB
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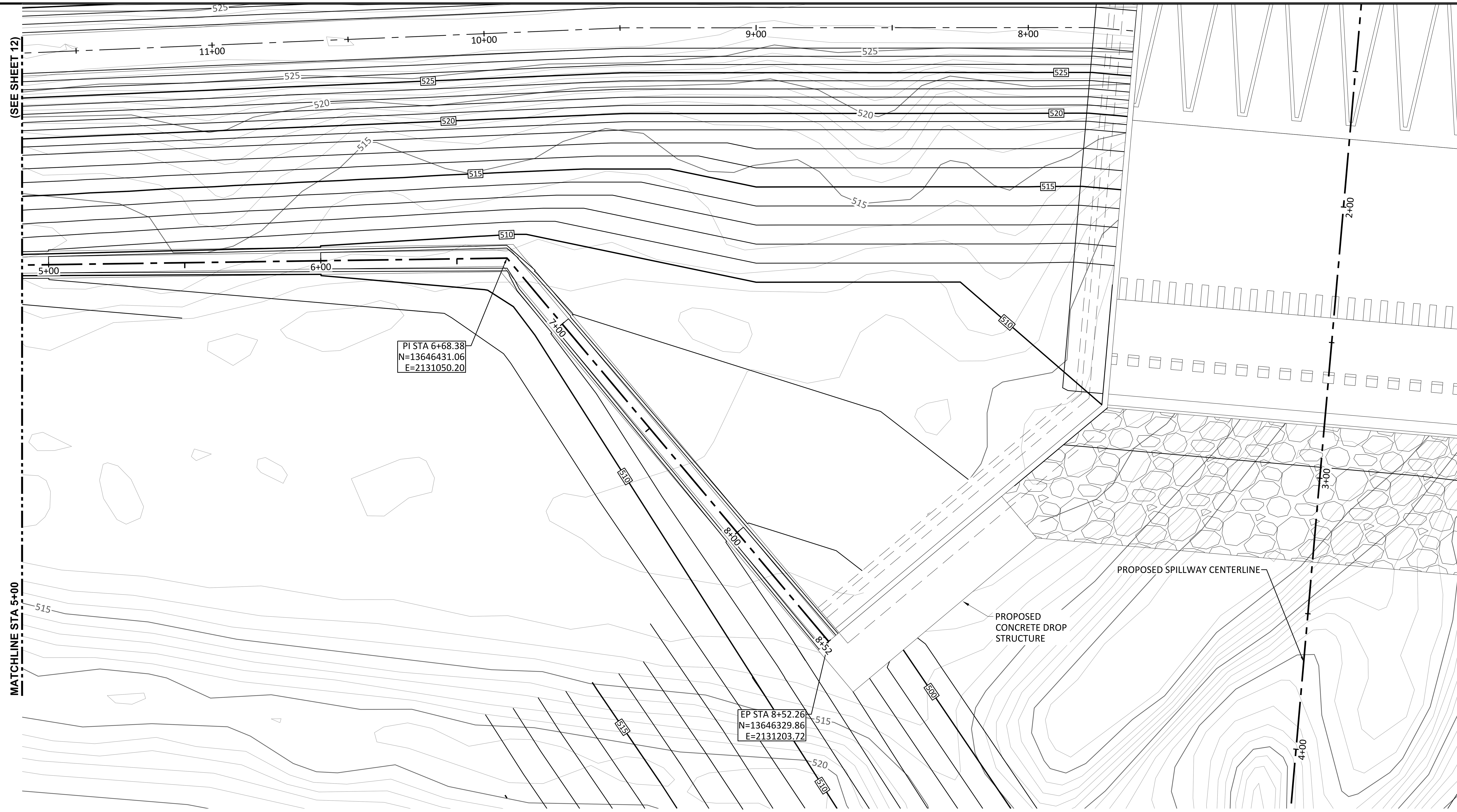

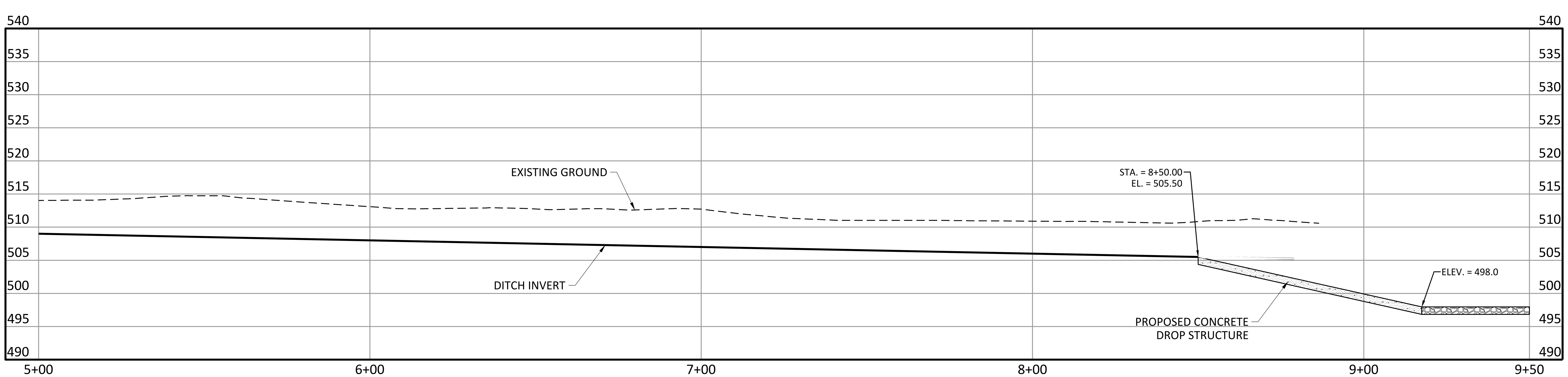

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SAN ANTONIO WATER SYSTEM

MITCHELL LAKE DAM IMPROVEMENTS PRELIMINARY DESIGN

PROPOSED DITCH PLAN & PROFILE II

F&N JOB NO.	PLU17623
DATE	04/29/2020
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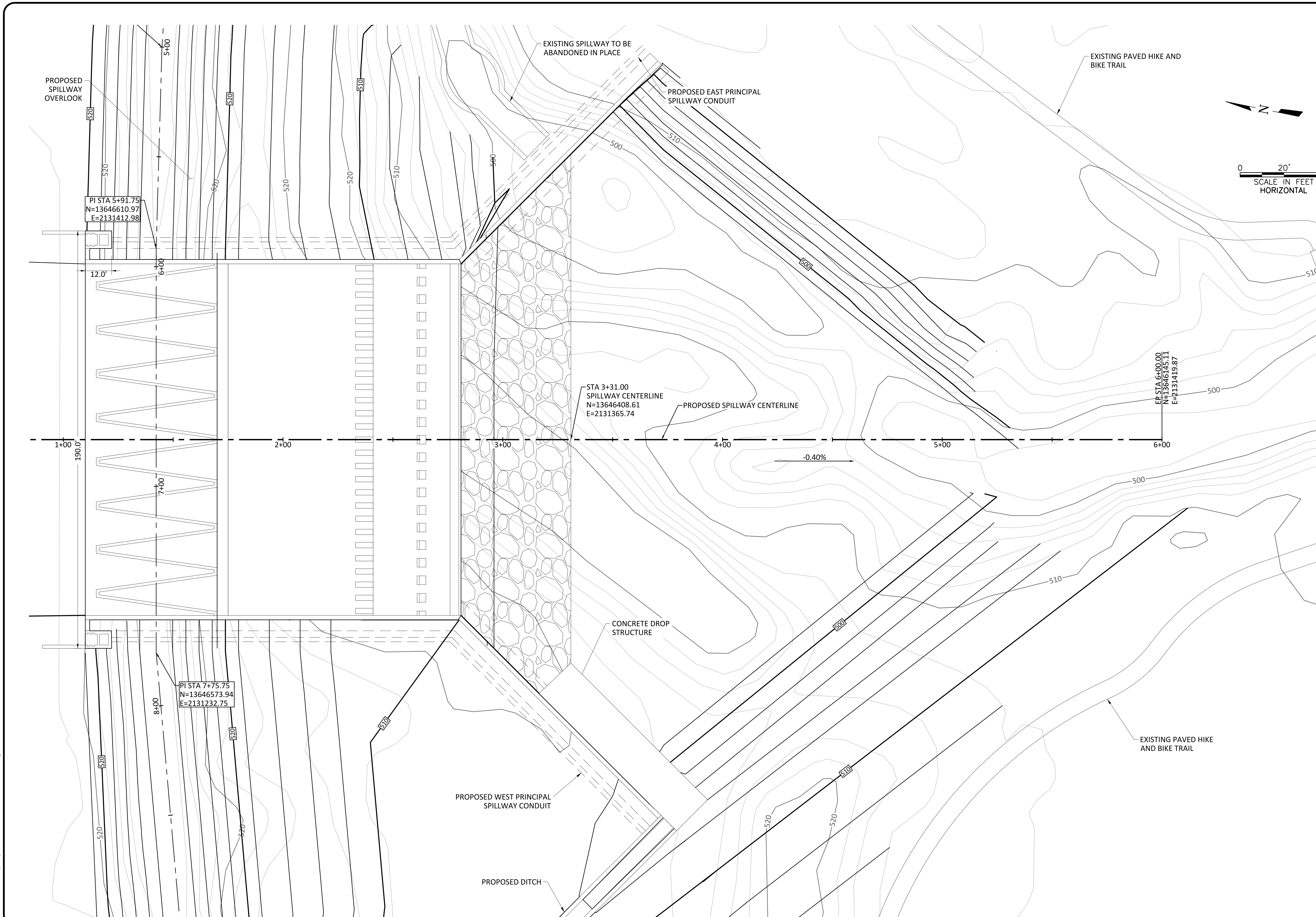

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SAN ANTONIO WATER SYSTEM
MITCHELL LAKE DAM IMPROVEMENTS PRELIMINARY DESIGN

PROPOSED SPILLWAY PLAN

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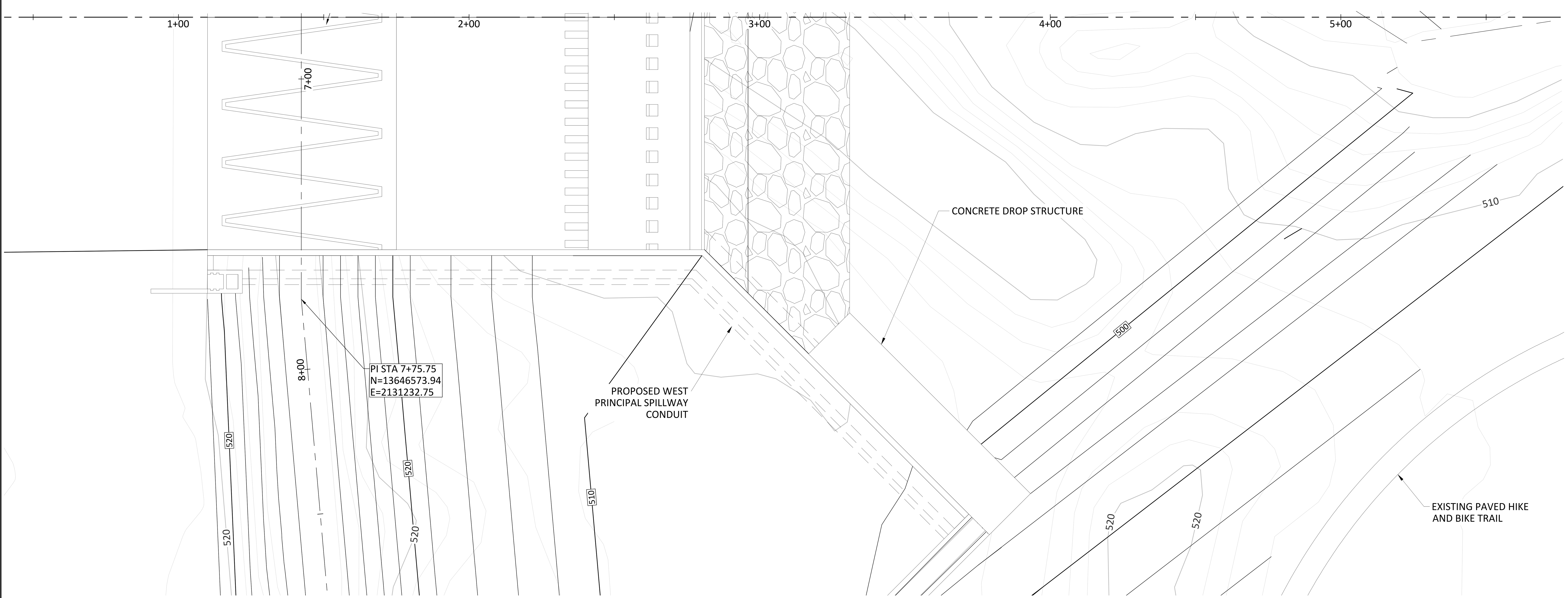

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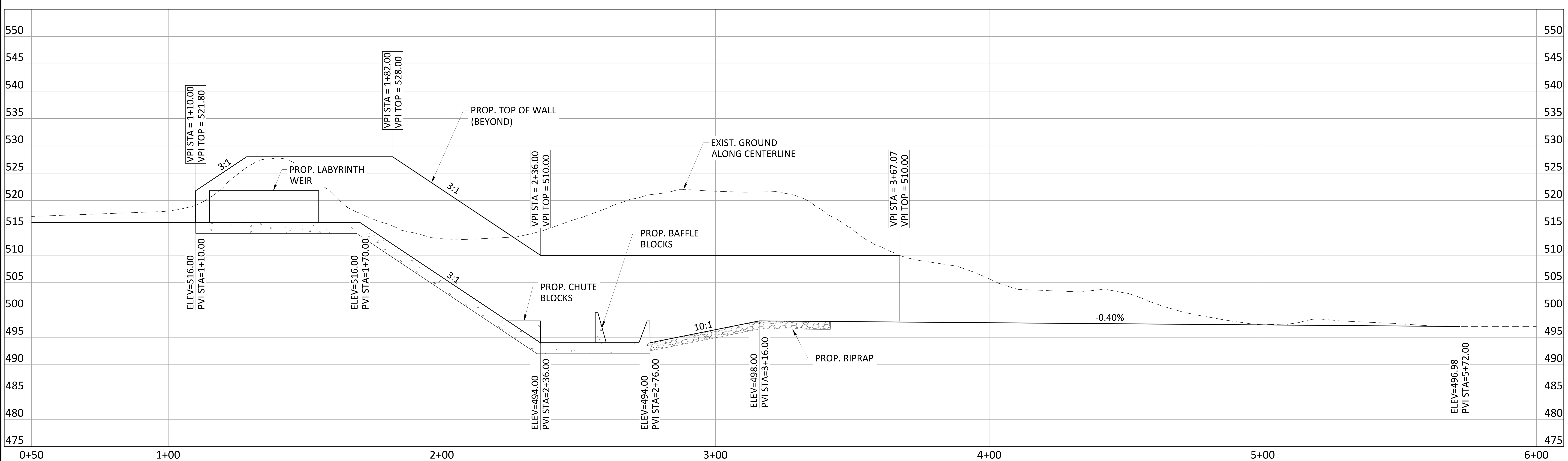

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SAN ANTONIO WATER SYSTEM
MITCHELL LAKE DAM IMPROVEMENTS PRELIMINARY DESIGN

SPILLWAY PARTIAL PLAN & PROFILE

F&N JOB NO.	PLU17623
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REVISED	
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Appendix C: Geotechnical Material Parameter Development

Rev.	Author	Author Initials	Reviewer	Reviewer Initials	Date
0	Hande Gerkus-Harris, Ph.D., P.E.	HGH	Russell G. Springer, P.E.	RGS	04/06/2020

1.0 PURPOSE AND OVERVIEW

The purpose of this worksheet is to develop shear strength material parameters for soil materials for use during the geotechnical analysis related to the Mitchell Lake Dam Improvements, Preliminary Engineering Design.

The parameter analysis is based on the geotechnical data report prepared by Arias, dated June 11, 2018. Total of 20 borings were drilled as a part of this investigation with 7 borings located at the dam, 1 boring north-west of the dam, and 12 borings located at the wetlands. Field testing included penetration tests, and laboratory testing included a variety of classification and index testing and shear strength tests.

Historical geotechnical information was summarized in the Conceptual Design Report prepared by Merrick & Company, dated December 4, 2015. This information includes the desktop study performed by PSI in 2015 and the geotechnical investigation conducted by Bryant-McClelland Consultants, Inc. in 1989. As a part of the 1989 study, 18 test borings were drilled within the dam embankment. Borings showed soils were primarily lean clays (CL) with fat clays (CH) encountered in some borings. Poorly graded sand (SP-SC) and poorly graded sand with silt (SP-SM) were mostly encountered in borings closer to the spillway, likely because of a transition in depositional materials (Hensley-Schmidt, Inc., 1990).

Geotechnical parameters developed for this study are based on the results from the field and laboratory studies performed as part of this project, historical data, published correlations, and FNI processes.

2.0 STRATIGRAPHY

According to Geotechnical Data Report by Arias (2018), the surface geology of the site consists of Fluviatile terrace deposits (Qt) consisting of alluvial deposits of clays, sands, silts and gravels. Soil borings primarily show clayey fill material (CH/CL), native clays (CH/CL) and some granular material (SC) on site. Historically available geotechnical information obtained from Proposed Dam Improvements Mitchell Lake Dam, San Antonio, Texas: Desktop Geotechnical Review and Concept Discussion dated October 14, 2015, states that this area is seismically inactive and seismic ground movements are not expected to be of concern.

When soil samples obtained from dam borings and wetland borings are compared, it can be interpreted that that clayey embankment materials are likely native materials excavated around from the site.

3.0 ANALYSIS OF UNIT WEIGHT

Most of the unit weight data were primarily obtained from strength, consolidation, and permeability test specimens. The measured moist unit weights of soils are presented as a histogram in Figure 1 for soils samples from the dam and Figure 2 for the soils sampled from wetland areas. Additionally, the unit weights of the soils were determined using laboratory test results and SPT N_{60} correlations presented in Teng (1962), developed by Terzaghi and Peck. The standard penetration test blow counts (N) are corrected for hammer energy, borehole diameter, rod length and sampler type to calculate SPT N_{60} values. Table 1 summarizes the unit weights calculated based on SPT N_{60} correlations for each soil group.

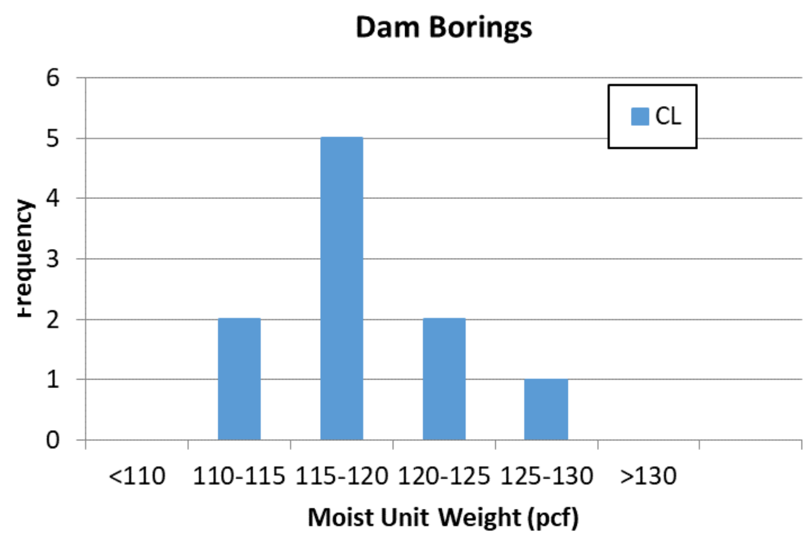


Figure 1. Moist Unit Weight Histogram, Dam Borings

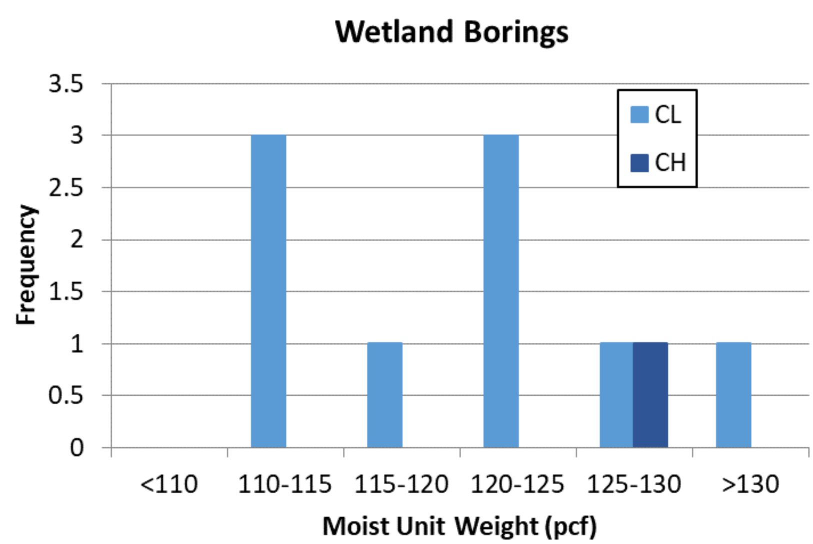


Figure 2. Moist Unit Weight Histogram, Wetland Borings

Table 1. Unit Weight Summary

Soil Group	Moist Unit Weight Based on SPT N_{60} Correlations		
	Min	Max	Average
CH	130 pcf	130 pcf	130 pcf
CL	125 pcf	130 pcf	130 pcf
SC	110 pcf	125 pcf	123 pcf

4.0 ANALYSIS OF SOIL CLASSIFICATION

The average liquid limit (LL), plasticity index (PI), and percent passing No 200 sieve for CH, CL, and SC soils are presented in Table 2. Plasticity parameters are consistent along dam and wetland borings. Figure 3 plots the data for the clays on the USCS plasticity chart.

Table 2. Classification Data Summary

Soil Group	Soils from Dam Borings, Average Values			Soils from Wetland Borings, Average Values		
	LL	PI	%Passing No. 200 Sieve	LL	PI	%Passing No. 200 Sieve
CH	57	38	96	52	35	89
CL	43	26	86	43	26	85
SC	32	16	35	32	17	42

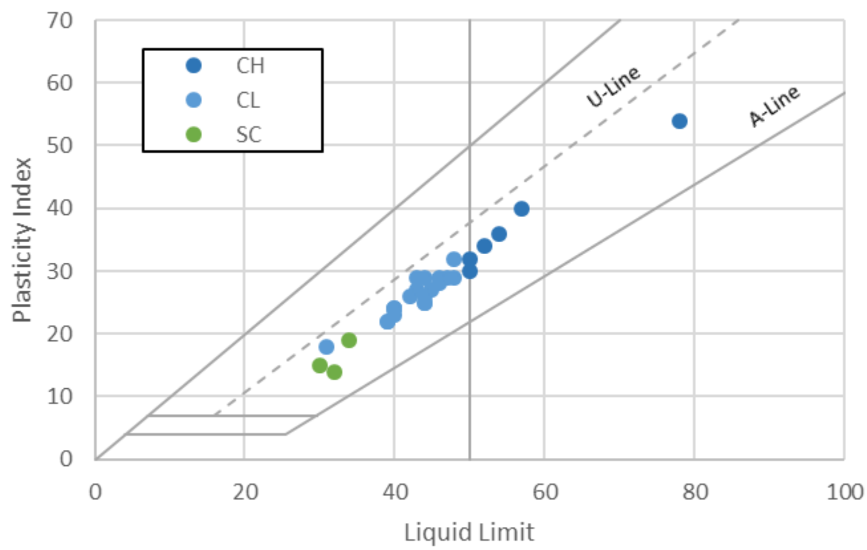


Figure 3. Plasticity Chart Plot

5.0 SHEAR STRENGTH OF SOILS

Shear strength testing was performed on CL samples. The shear strength tested consisted of:

- Direct shear testing
- Consolidated undrained (CU) triaxial testing with pore pressure measurements
- Unconfined compression strength (UCS) testing

Additional shear strength parameters were calculated based on SPT N blow counts and published correlations.

Clays, by Strength Tests

Direct Shear Tests

Effective shear strength parameters for clays were studied using consolidated-drained direct shear tests and consolidated-undrained (CU) triaxial shear tests with pore pressure measurements. Direct shear test results are presented in Table 3. Tests were performed at a constant rate of strain for increasing normal stresses. The results of the tests generally agree with the values typically encountered for CL soils. However, it is desirable to aggregate the results and study trends by soil type. The results of the consolidated-drained direct shear tests for peak stress and 0.25 inch displacement are presented on Figure 4 and Figure 5. Trendlines are provided for both a linear and a power curve regression and strength parameters are calculated for aggregated test results.

Table 3. Direct Shear Test Results

Boring Location	Boring No.	Sample Depth	USCS	At Peak Stress		At 0.25 in. Displacement	
				ϕ' (deg)	c' (psf)	ϕ' (deg)	c' (psf)
Dam Borings	B-103	19	CL	19	792	19	792
Wetland Borings	B-110	5	CL	28	173	35	0
	B-113	5	CL	18	374	25	158
	B-114	7	CL	18	374	23	216
Aggregated Results			CL	28	197	30	83

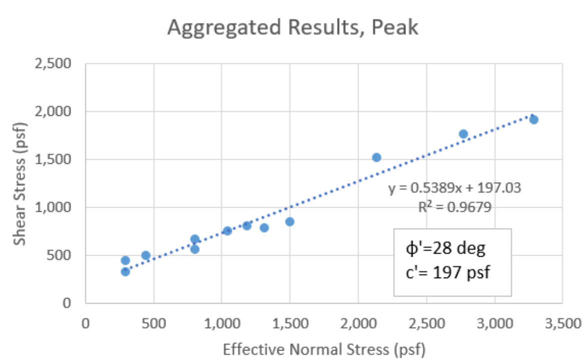


Figure 4. Aggregated Direct Shear Test Results, at Peak Shear Stress

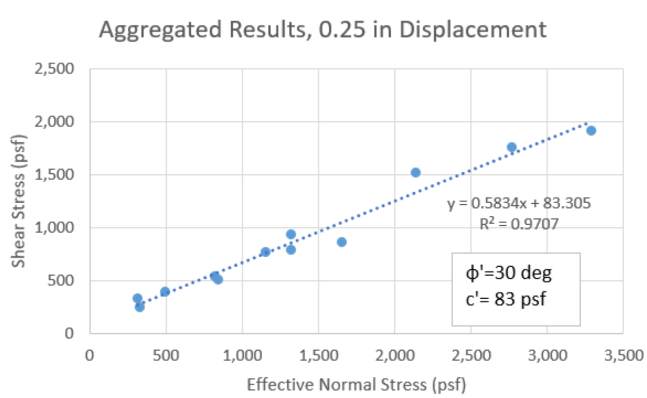


Figure 5. Aggregated Direct Shear Test Results, at 0.25-inch Displacement

CU Triaxial Test

Multi-stage consolidated-undrained (CU) triaxial shear tests were performed with pore pressure measurements. The reported effective shear strength parameters and the calculated total strength parameters are summarized in Table 4. Aggregated test results were calculated by plotting effective or total strength Mohr circle of each test as shown in Figure 6 and Figure 7.

Table 4. CU Triaxial Test Summary

Boring Location	Boring No.	Sample Depth	USCS	LL	PI	%-200	Reported		Calculated	
							ϕ' (deg)	c' (psf)	ϕ (deg)	c (psf)
Dam Borings	B-102	11	CL	44	25	94	30	418	26	295
	B-104	34	CL	--	--	--	28	432	19	387
	B-106	24	CL	43	29	89	24	720	3	2176
	Aggregated Results						27	477	19	654

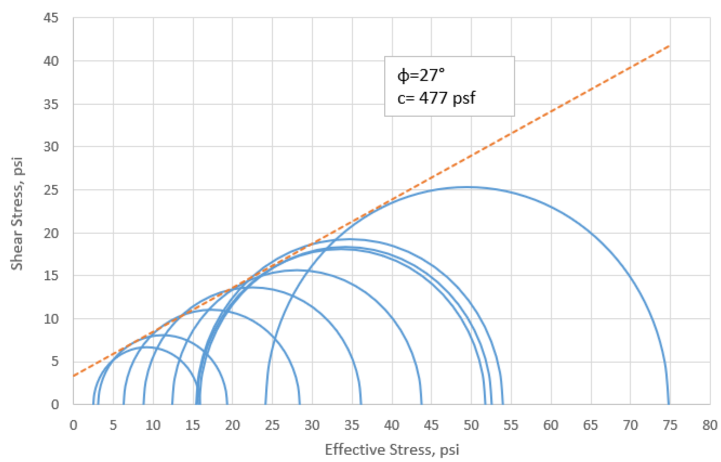


Figure 6. Effective Shear Strength from Aggregated Triaxial Shear Data, CL samples

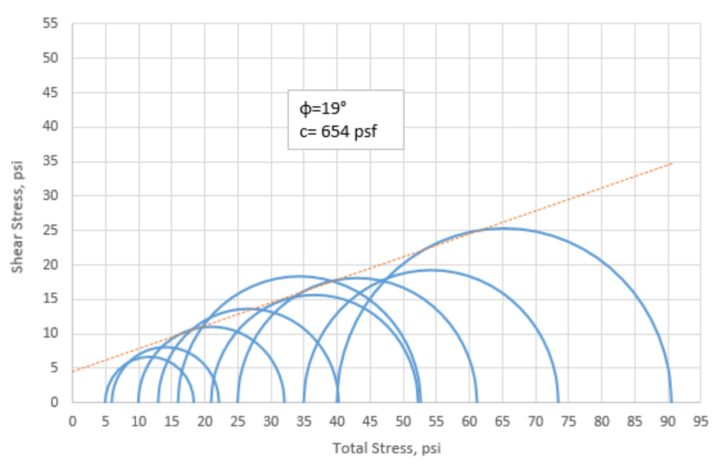


Figure 7. Total Shear Strength from Aggregated Triaxial Shear Data, CL samples

Unconfined Compressive Strength Test

The unconfined compressive strength tests were performed on seven CL samples obtained from wetland borings. Table 5 presents test results with the minimum, maximum, and average undrained shear strength values.

Table 5: Unconfined Compressive Strength Test Results

Boring No.	Sample Depth	USCS	Unconfined Compressive Strength, q_u (tsf)	Undrained Shear Strength c_u (psf)
B-110	11	CL	5.59	5,590
B-112	9	CL	9.13	9,130
B-113	11	CL	5.76	5,760
B-114	9	CL	2.35	2,350
B-115	3	CL	3.44	3,440
B-116	14	CL	4.95	4,950
B-119	3	CL	12.13	12,130
Minimum				2,350
Maximum				12,310
Average				6,193

Clays, by Correlation

Correlations based on SPT-N Blow Counts

Terzaghi and Peck (1967) published the general relationship between unconfined compressive strength (qu) with SPT-N blow count presented in Table 6. The unconfined compressive strength values calculated for CH and CL soils using this correlation and SPT N blow counts are listed in Table 7.

Table 6. N_{60} versus S_U for Cohesive Soil (Terzaghi and Peck, 1967)

Consistency	N_{60} Value, bpf	q_u (psf)	q_u (ksf)
Very Soft	< 2	< 250	< 0.25
Soft	2 – 4	250 – 500	0.25 – 0.50
Medium Stiff	4 – 8	500 – 1,000	0.50 – 1.0
Stiff	8 – 15	1,000 – 2,000	1.0 – 2.0
Very Stiff	15 – 30	2,000 – 4,000	2.0 – 4.0
Hard	> 30	> 4,000	> 4.0

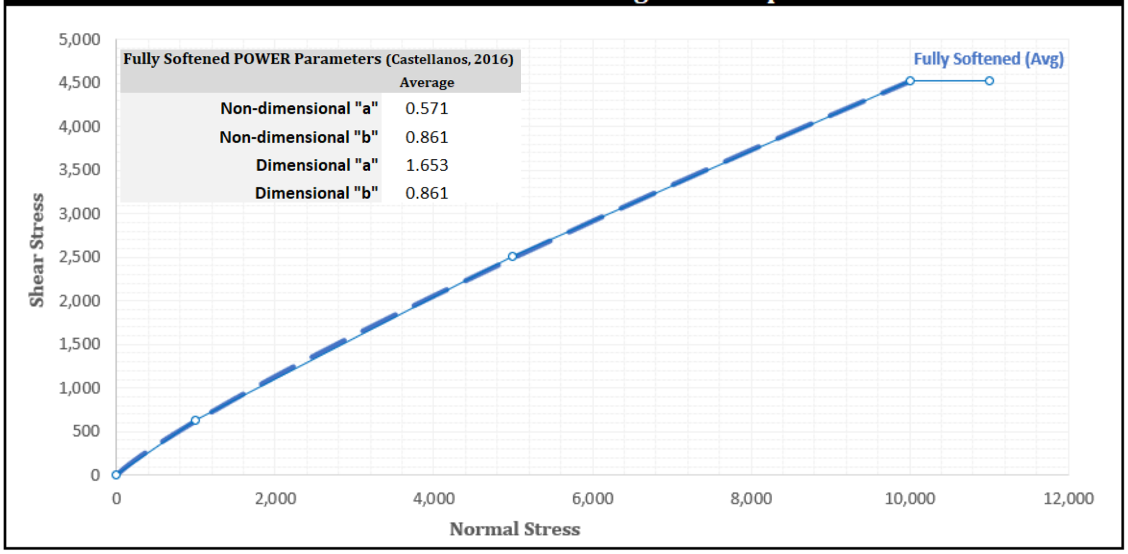
Table 7. Unconfined Compressive Strength Based on SPT N Correlations

Soil Group	Unconfined Compressive Strength (q_u)		
	Min	Max	Average
CH	3,000 psf	4,000 psf	3,833 psf
CL	1,500 psf	4,000 psf	3,375 psf

Fully Softened Strength

Published correlations were used to predict fully softened friction angles using the recent correlation published by Castellanos et al (2016). This correlation obtains power function parameters based on the plasticity index and clay fraction (parameters vary with normal stress). The predictive equation and resulting chart are provided in Figure 8.

Dimensional Shear Strength Envelopes



All Stress Units:	PSF	All Angle Units:	DEGREES

Summary of Input Data and Index Property Parameter Calculations

Material Description:	CL			**Predicted CF**	36 %
Material Use/Loc.:	Mitchell Lake Dam			**Selected CF:**	36.0 %
Liquid Limit:	43	**Plasticity Index:**	26	**Cohesion Index (CI):**	22.4
% Passing No. 200:	86	**Clay Fraction (%CF):**	0	**CF*PI:**	936

Piece-Wise Function M-C Values

Description:		FSS, Average	
Normal Stress Range (psf)		φ'	c'
0	1,000	32.8	0.0
1,000	5,000	25.4	186.2
5,000	10,000	22.5	479.8
10,000	11,000	0.0	4618.9

Figure 8. FSS Friction Angle for CL, Castellanos et al (2016)

Granular Soils, by Correlation

Granular soils of clayey sand and clayey sand with gravel were only encountered in two borings (B-108, B-109). Figure 9 provides corrected N_{60} values for the clayey sands, plotted against the sample depth. Results show in-situ sands are mostly dense to very dense. Correlations by Peck and Meyerhof (see EPRI, 1990) were used to analyze effective friction angles for sands using the blow counts, as shown graphically in Table 8. The values for these correlations are for "clean" sands and silts, and Duncan et al (1989) recommends reducing the Meyerhof values by up to 5 degrees for clayey sands. It should be noted that groundwater table level was not encountered in the borings with SC soils. Saturated SPT-N values for SC soils can be lower especially with higher fines content.

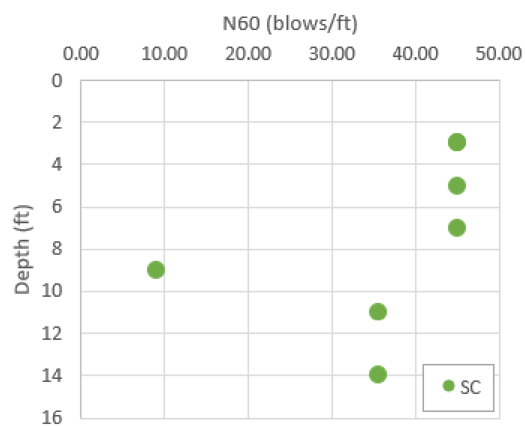


Figure 9. Corrected Blow Count vs. Depth

Table 8. N versus ϕ' for "Clean" Cohesionless Soil (from EPRI 1990)

N Value, bpf	Relative Density	Approx. ϕ', deg (Peck et al)	Approx. ϕ', deg (Meyerhof)
0 – 4	Very Loose	< 28	< 30
4 – 10	Loose	28 – 30	30 – 35
10 – 30	Medium Dense	30 – 36	35 – 40
30 – 50	Dense	36 – 41	40 – 45
> 50	Very Dense	> 41	> 45

According to these correlations, an average friction angle of 35 degrees can be calculated for dense to very dense sands, and 25 degrees for the loose to medium dense sample. These values are calculated as the upper-bound parameters for dry samples. Therefore, design friction angles were reduced 5 degrees to account for possible reduction in friction angle of SC samples with saturation.

6.0 Analysis of Seepage Parameters

Hydraulic Conductivity from Laboratory Tests

Falling head permeability tests were performed on two CL samples from dam borings. Measured average hydraulic conductivity for each sample is listed in Table 9. Measured hydraulic conductivity values are typical of CL soils for vertical permeability.

Table 9. Falling Head Hydraulic Conductivity Test Results

Boring No.	Sample Depth (ft)	USCS	Average Hydraulic Conductivity (cm/s)	Average Hydraulic Conductivity of the Sample (ft/s)	Average Hydraulic Conductivity (ft/s)
B-101	9	CL	2.8E-06	9.2E-08	4.7E-08
B-105	9	CL	4.2E-08	1.4E-09	

Hydraulic Conductivity from Correlation

Hydraulic conductivity is commonly estimated from correlations and engineering judgment. USBR Design Manual No. 13 (2014) provides typical hydraulic conductivity values for a variety of materials and provides a suitable basis for the selection of estimated values to supplement those obtained from the field and laboratory testing. The typical values are presented in Figure 10, Figure 11 and Figure 12.

Material	k_V Range (ft/yr or x10^{-6} cm/s)
Coarse sand and gravel	150,000 to 500,000
Medium to coarse sand	50,000 to 150,000
Fine to medium sand	10,000 to 50,000

Figure 10. Hydraulic Conductivity of Drain Material (USBR 2014)

Unified Soil Classification	k_V Range (ft/yr or x10^{-6} cm/s)*
GM-SM	0.0 to 10.0
GM or GC	0.0 to 10.0
SP-SM	0.0 to 10.0
SM	0.0 to 10.0
SM-SC	0.0 to 3.0
SM-ML	0.0 to 10.0
SC	0.0 to 3.0
ML	0.0 to 10.0
ML-CL	0.0 to 1.0
CL	0.0 to 1.0
MH	0.0 to 0.1

Unified Soil Classification	k_V Range (ft/yr or x10^{-6} cm/s)
GP	2,000 to 1,000,000
GW	1,000 to 100,000
GP-SP	1,000 to 50,000
GW-SW	500 to 5,000
GM	10 to 500
SP (medium to coarse)	10,000 to 20,000
SP (fine to medium)	5,000 to 10,000
SP (very fine to fine)	500 to 5,000
SW	300 to 5,000
SP-SM	10 to 1,000
SM	10 to 500

Figure 11. Typical Vertical Hydraulic Conductivity of Natural Soil (USBR 2014)

Soil	k_H Range (ft/yr or 10^{-6} cm/s)
Gravel, open-work	>2,000,000
Gravel (GP)	200,000 to 2,000,000
Gravel (GW)	10,000 to 1,000,000
Sand, coarse (SP)	10,000 to 500,000
Sand, medium (SP)	1,000 to 100,000
Sand, fine (SP)	500 to 50,000
Sand (SW)	100 to 50,000
Sand, silty (SM)	100 to 10,000
Sand, clayey (SC)	1 to 1,000
Silt (ML)	1 to 1,000
Clay (CL)	~0 to 3

Figure 12. Typical Horizontal Hydraulic Conductivity of Natural Soil (USBR 2014)

The seepage analysis will also consider anisotropic behavior. The anisotropic ratio is defined as the horizontal hydraulic conductivity divided by the vertical hydraulic conductivity (k_h/k_v). Typical anisotropic ratios are discussed in the following paragraphs.

> <u>Protected Clays</u>: Typical k_h/k_v values for clay vary from 3 and 10. For protected clays where cracking/weathering potential is limited a k_h/k_v value of 3 to 4 (k_v/k_h =0.33 to 0.25) is recommended.
>
> <u>Weathered/Desiccated Clays</u>: Clays that are subjected to weathering/desiccation will develop cracks, and k_h/k_v values can vary from 1 to 0.1 (k_v/k_h =1 to 10).
>
> <u>Compacted Clays</u>: Clays that are compacted in lifts will develop higher horizontal conductivity due to lift interfaces. The k_h/k_v values for compacted clays vary from 3 to 10 (k_v/k_h =0.33 to 0.1).
>
> <u>Granular Soil</u>: For clayey/silty sands a k_h/k_v value of 3 to 4 (k_v/k_h =0.33 to 0.25) is recommended. A k_h/k_v value of 2 to 3 (k_v/k_h =0.5 to 0.33) is recommended for coarse granular soil that is free of fines. For imported material (concrete sand, drainage rock, etc.), a k_h/k_v value of 1 is recommended.

For the preliminary stability analysis, a range of permeability values and anisotropic ratios were selected for some materials to perform a sensitivity analysis for the transient analysis condition.

7.0 SETTLEMENT PARAMETERS

Settlement will occur in the new fill material, and also in the embankment and foundation soils beneath the new fill material. This settlement will be a result of elastic settlement, primary consolidation settlement, and possibly secondary consolidation. One dimensional consolidation tests were performed on CL samples. Results are summarized in Table 10.

Table 10: One-dimensional Consolidation Test Results

Boring No.	Sample Depth (ft)	USCS	Preconsolidation Pressure (psf)	Maximum Measured Swell Pressure (psf)	Compression Index, Cc	Recompression Index, Cr
B-101	7	CL	7,800	185	0.164	0.018
B-103	14	CL	3,000	133	0.16	0.165
B-104	29	CL	4,900	<100	0.175	0.174
B-105	24	CL	9,000	132	0.142	0.148

8.0 PARAMETER ANALYSIS SUMMARY

Based on the above analysis the shear strength and seepage parameters for the various soil materials are provided in Table 11. These parameters are used for slope stability and seepage models. Material strength parameters are slightly reduced for models where a softer layer (pocket penetrometer ≤ 0.25 tsf) was encountered within the CL layer.

Table 11. Selected Design Parameters

Material Type	Moist Unit Weight (pcf)	Consolidated Drained		Consolidated Undrained		Total UU	Permeability	K_v/K_h Ratio
		ϕ' (deg)	c' (psf)	ϕ (deg)	c, (psf)	c, (psf)	K_v (ft/sec)	
CH	130	25	250	18	350	3,500	1.4E-09	0.25
CL	125	27	200	20	300	3,000	4.7E-08	0.33 for Embankment Fill
							1.64E -08	0.1 for Alluvial Foundation Clay
SC	120	30	--	30	--	--	3.3E-07	0.25

The analysis may also include some consideration of soil softening and shallow slope instability. In these cases, the use of power functions is recommended for CL clays. A summary of the selected power function parameters is provided in Table 12.

Table 12. Power Function Parameters

Soil Type	Non-dimensional		Dimensional (psf)	
	"a"	"b"	"a"	"b"
CL	0.571	0.861	1.653	0.861

Tension forces are possible during the slope stability analysis, and tension cracks have the potential to develop in the clay soils. The depth of a potential tension crack is a function of both cohesion and soil friction. The depth of tension cracks for the CL soils is calculated as 5.2 feet using Equation 1 and assuming drained soil strength parameters (c'=200 psf, γ=125 pcf). A depth of 5.5 feet is selected for the analysis when appropriate.

$$z_{tension\ crack} = \frac{2c\prime}{\gamma\left(\tan(45-\frac{\varphi}{2})\right)} \quad \text{(Eq. 1)}$$

9.0 REFERENCES

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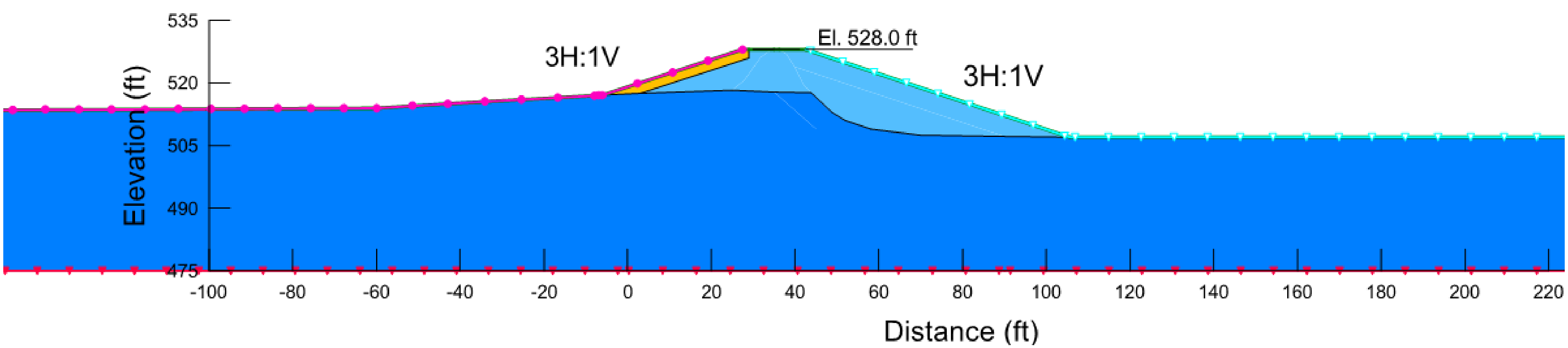
Appendix D: Seepage and Slope Stability Analysis Figures

Proposed Embankment Design

Steady State Seepage with Flood Loading Condition



Color	Name	Model	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Rotation (°)
	CL at CD Strength	Saturated / Unsaturated	Lean Clay (CL)	CL - Fredlund Ksat=4.7e-08'/s	0.33	0
	CL at CD Strength/Foundation	Saturated / Unsaturated	Lean Clay (CL)	CL - Fredlund Ksat=1.64e-08'/s	0.1	0
	Rock Riprap	Saturated / Unsaturated	Packsand	SP/SM, Fredlund Ksat=1.6e-04/s	1	0

Color	Name	Category	Kind	Parameters
	HW-Flood-EL. 528'	Hydraulic	Water Total Head	528 ft
	Potential Drainage Surface	Hydraulic	Water Flux	0 ft/sec
	Zero Flux	Hydraulic	Water Flux	0 ft/sec

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Seepage Analysis **Proposed Design**	**1**
PREPARED: HGH			

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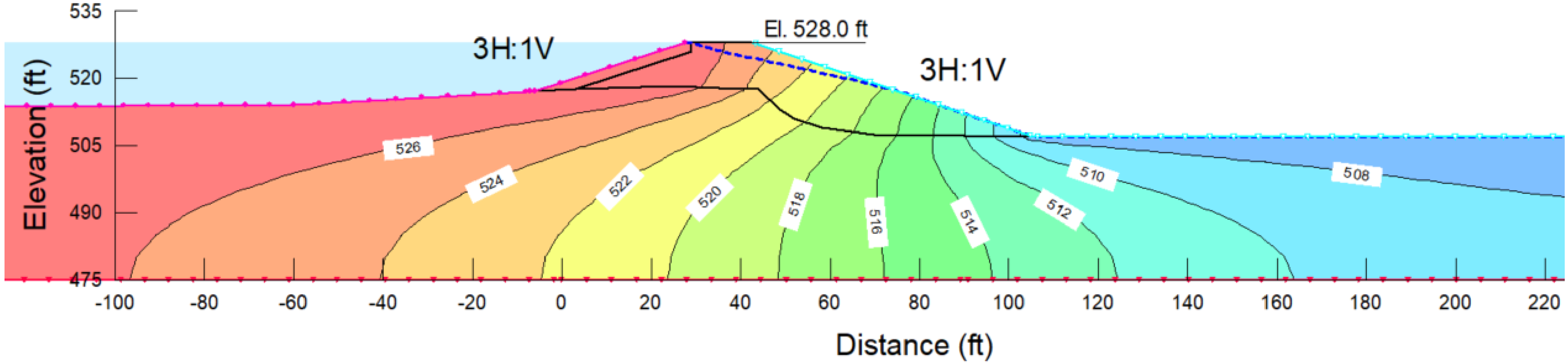
Proposed Embankment Design

Steady State Seepage with Flood Loading Condition



FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Seepage Analysis **Proposed Design**	**2**
PREPARED: HGH			

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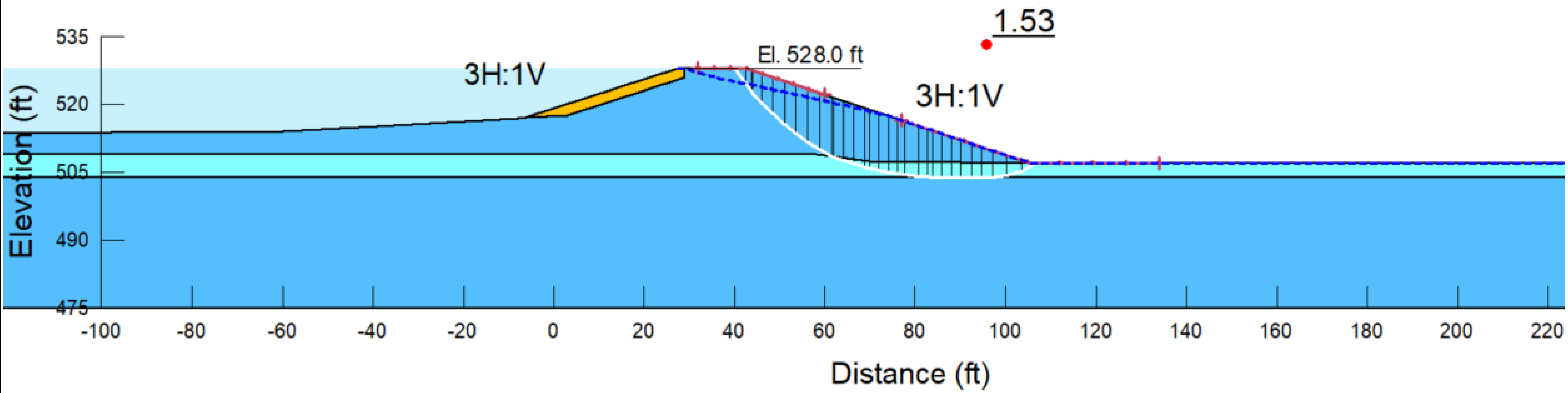
Proposed Embankment Design

Extreme Loading Condition, Downstream Slope Stability



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**3**
PREPARED: HGH			

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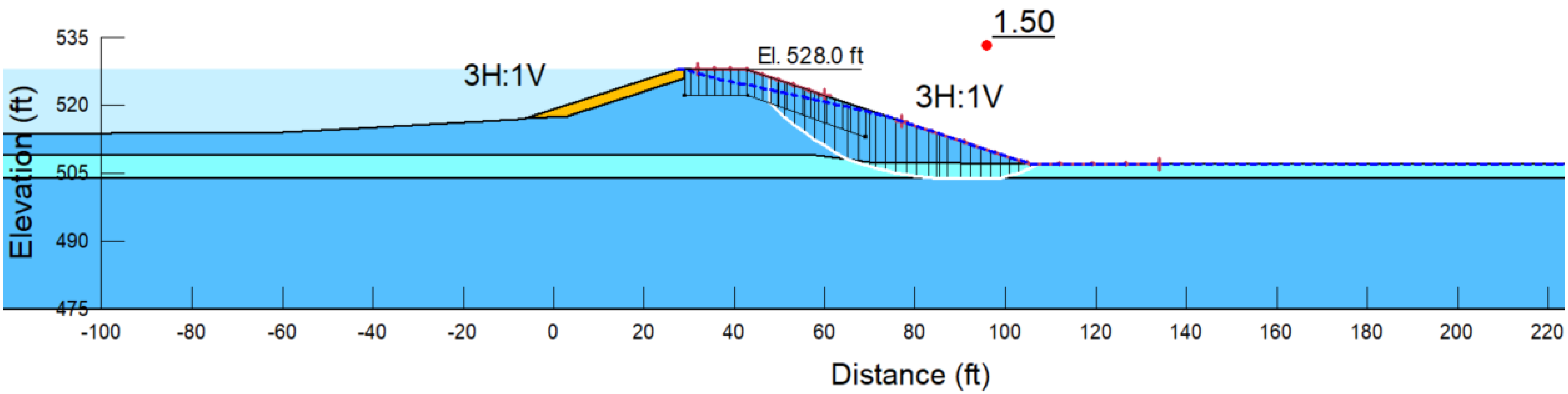
Proposed Embankment Design

Extreme Loading Condition, Downstream Slope Stability with Tension Crack



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**4**
PREPARED: HGH			

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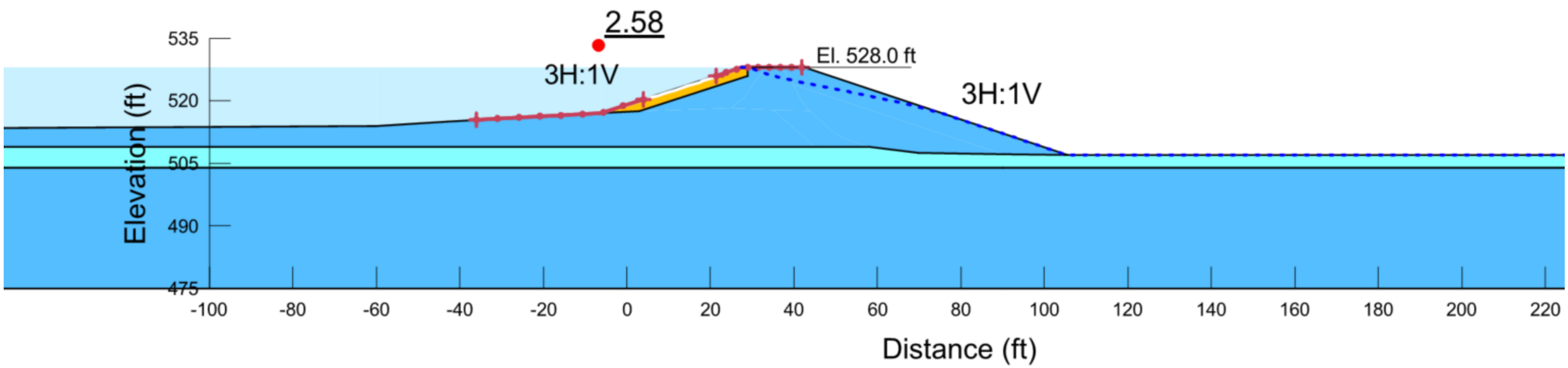
Proposed Embankment Design

Extreme Loading Condition, Upstream Slope Stability



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	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**5**
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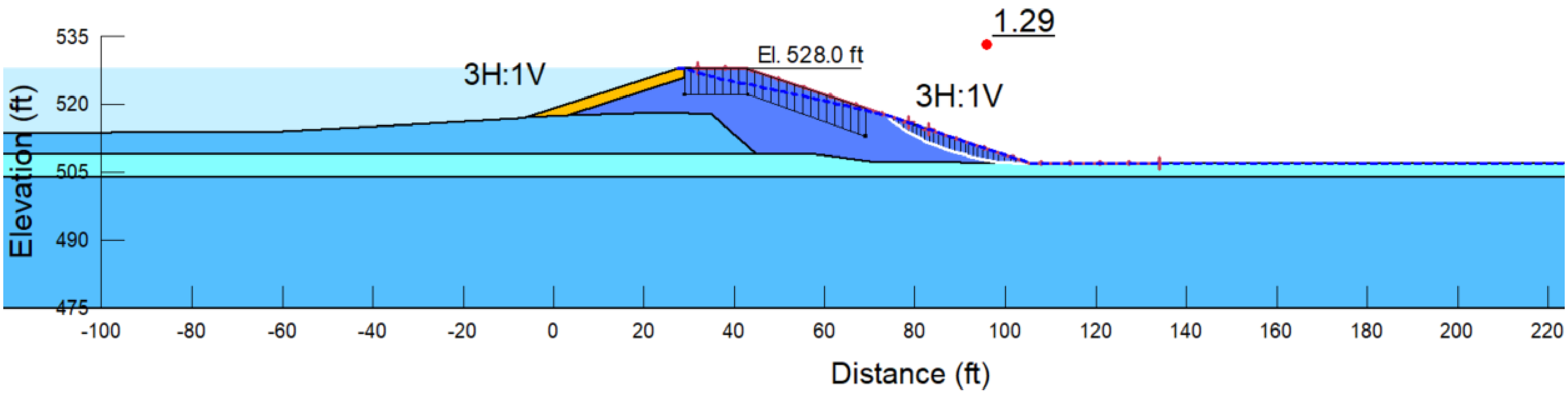
Proposed Embankment Design

Extreme Loading Condition, Downstream Slope Stability with Fully Softened Embankment

Fully Softened Strength

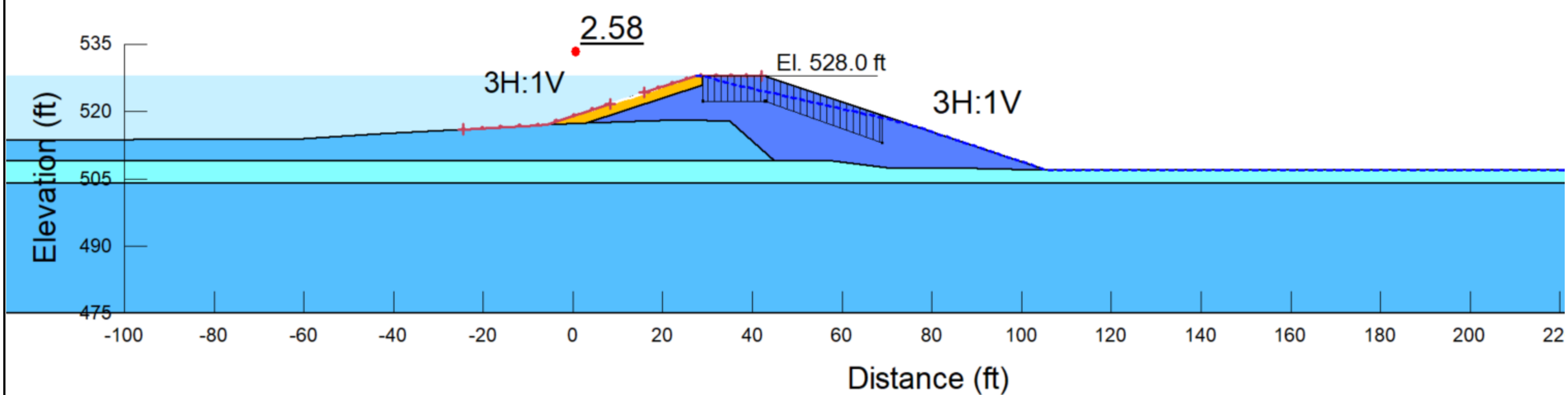


Color	Name	Model	Unit Weight (pcf)	Strength Function	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125		200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125		150	23	0
	CL-FSS	Shear/Normal Fn.	125	CL-FSS			0
	Rock Riprap	Mohr-Coulomb	135		0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**6**
PREPARED: HGH			

Proposed Embankment Design

Extreme Loading Condition, Upstream Slope Stability with Fully Softened Embankment



Color	Name	Model	Unit Weight (pcf)	Strength Function	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125		200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125		150	23	0
	CL-FSS	Shear/Normal Fn.	125	CL-FSS			0
	Rock Riprap	Mohr-Coulomb	135		0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**7**
PREPARED: HGH			

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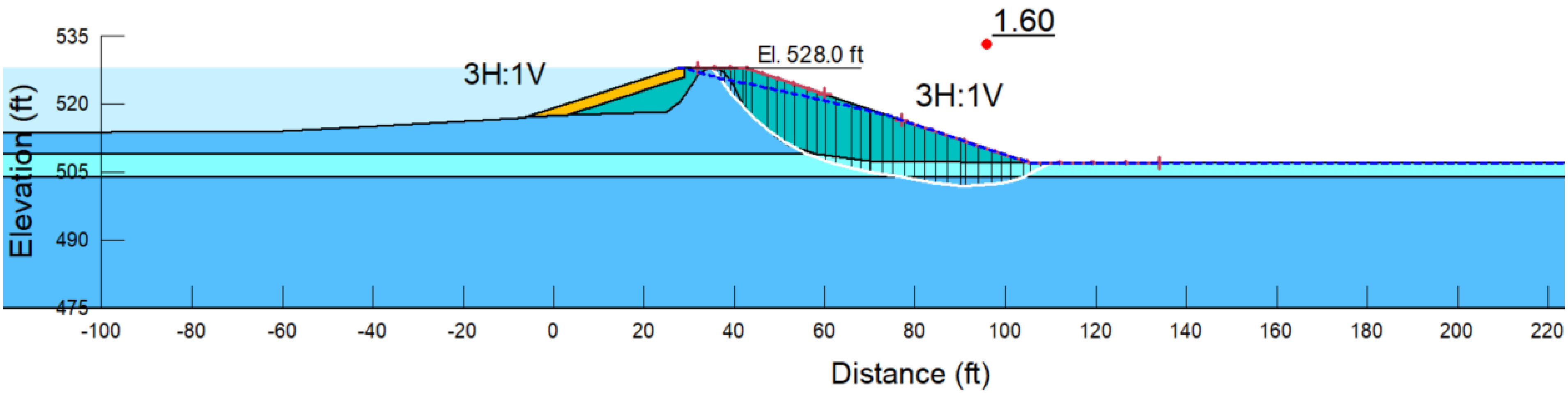
Proposed Embankment Design

Extreme Loading Condition, Downstream Slope Stability with CU Strength Clay Fill



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	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	CL-CU Strength	Mohr-Coulomb	125	300	20	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**8**
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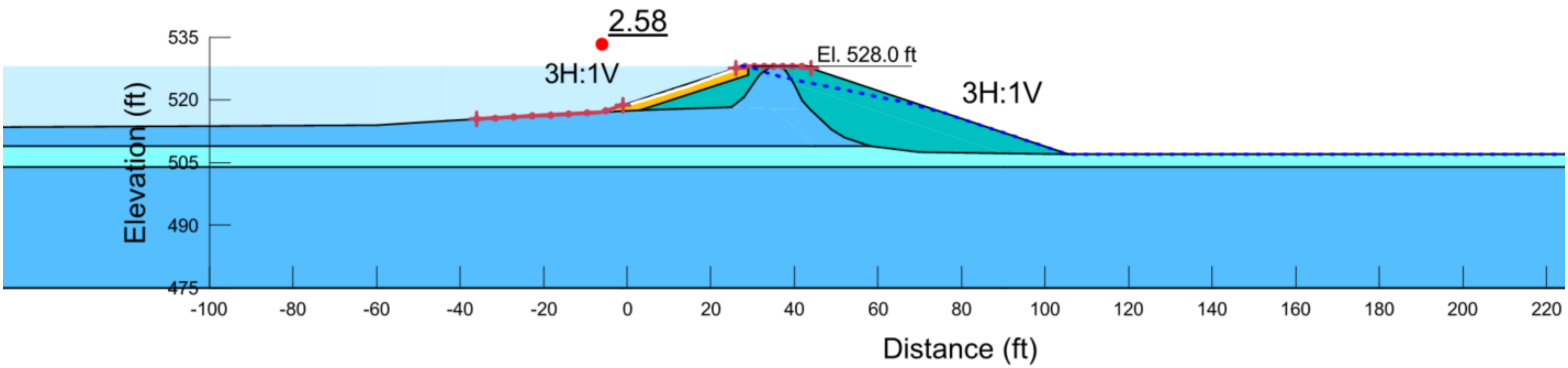
Proposed Embankment Design

Extreme Loading Condition, Upstream Slope Stability with CU Strength Clay Fill



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	CL-CU Strength	Mohr-Coulomb	125	300	20	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System	**Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis	**Proposed Design**	**9**
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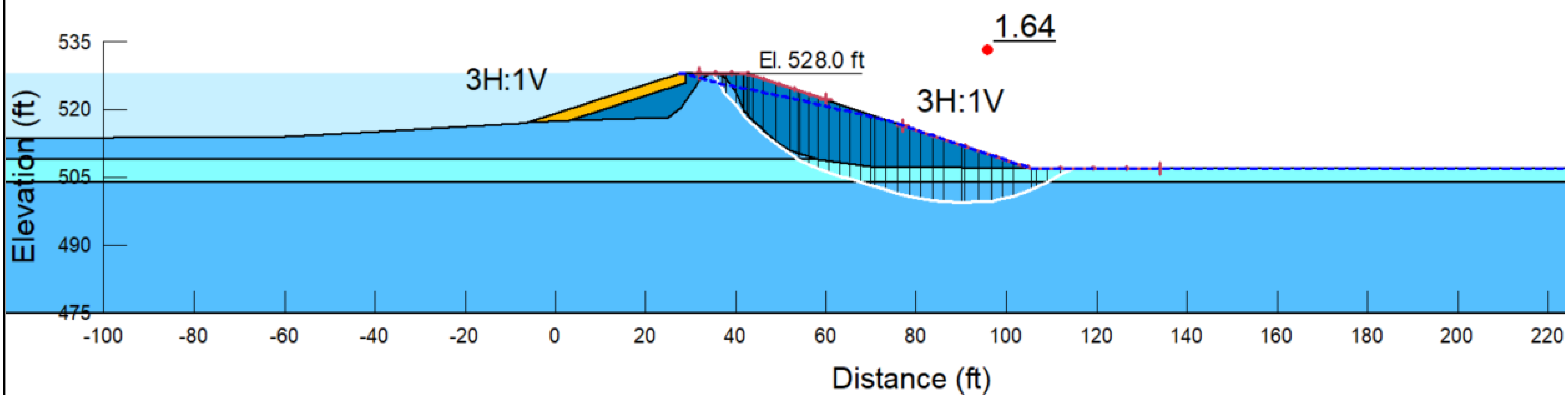
Proposed Embankment Design

Extreme Loading Condition, Downstream Slope Stability with UU Strength Clay Fill



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	CL at UU Strength	Mohr-Coulomb	125	3,000	0	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**10**
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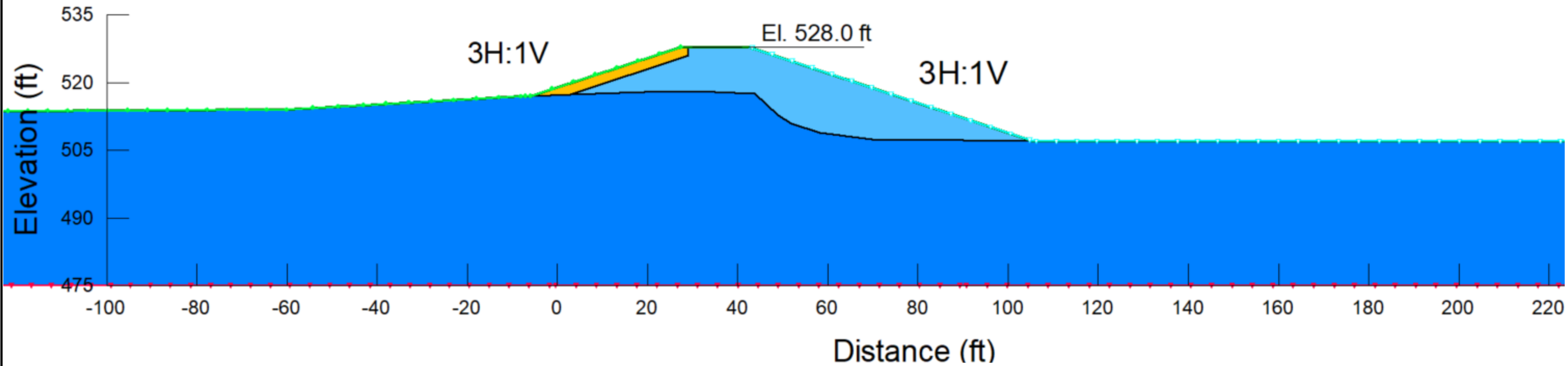
Proposed Embankment Design

Steady State Seepage with Normal Water Loading Condition



Color	Name	Model	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Rotation (°)
	CL at CD Strength	Saturated / Unsaturated	Lean Clay (CL)	CL - Fredlund Ksat=4.7e-08'/s	0.33	0
	CL at CD Strength/Foundation	Saturated / Unsaturated	Lean Clay (CL)	CL - Fredlund Ksat=1.64e-08'/s	0.1	0
	Rock Riprap	Saturated / Unsaturated	Packsand	SP/SM, Fredlund Ksat=1.6e-04'/s	1	0

Color	Name	Category	Kind	Parameters
	HW-NWS-EL. 518.5'	Hydraulic	Water Total Head	518.5 ft
	Potential Drainage Surface	Hydraulic	Water Flux	0 ft/sec
	Zero Flux	Hydraulic	Water Flux	0 ft/sec

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Seepage Analysis **Proposed Design**	**11**
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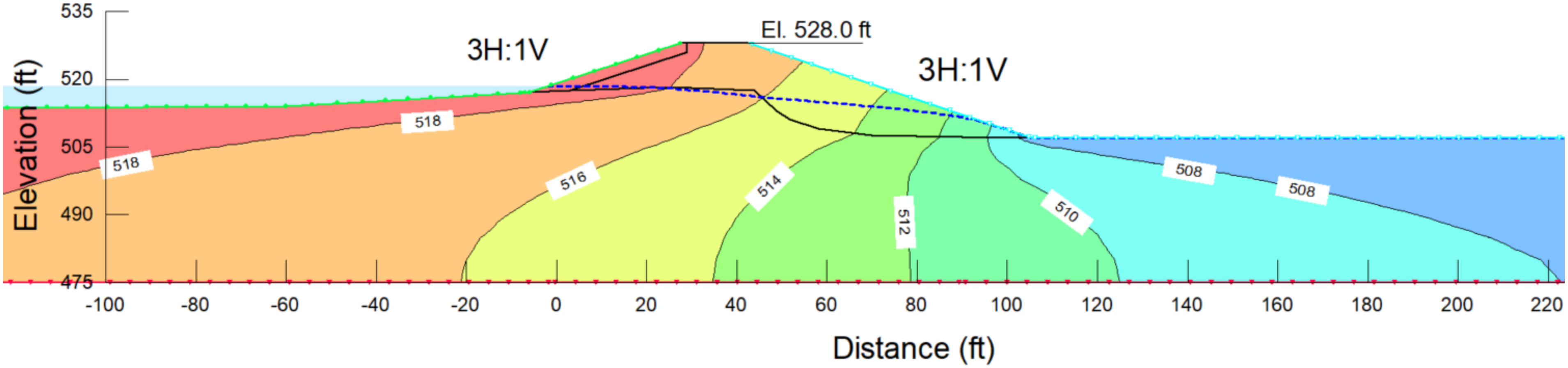
Proposed Embankment Design

Steady State Seepage with Normal Water Loading Condition



FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Seepage Analysis **Proposed Design**	**12**
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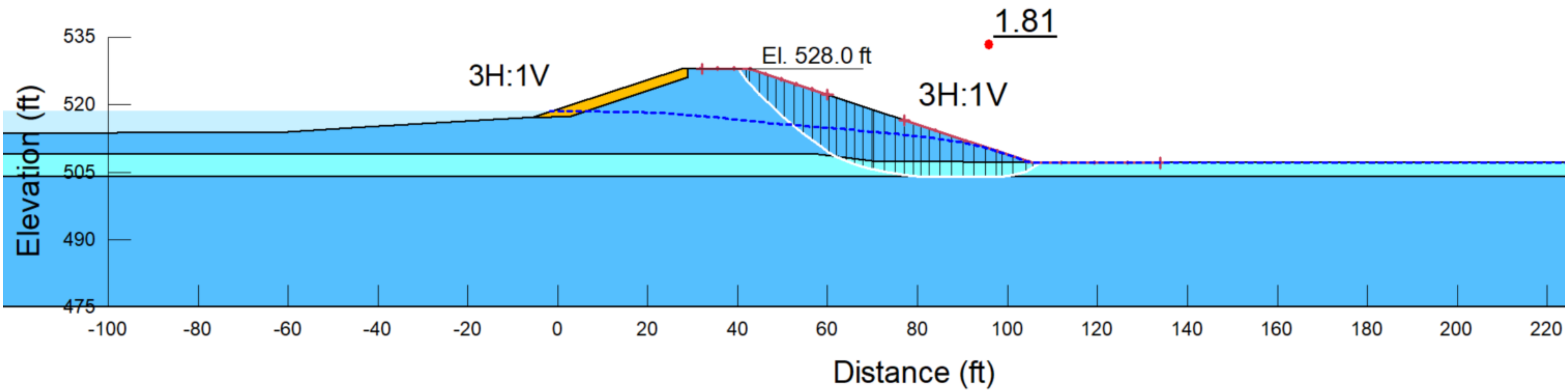
Proposed Embankment Design

Normal Water Loading Condition, Downstream Slope Stability



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

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San Antonio Water System
Mitchell Lake Dam Improvements

Slope Stability Analysis
Proposed Design

PLATE
13

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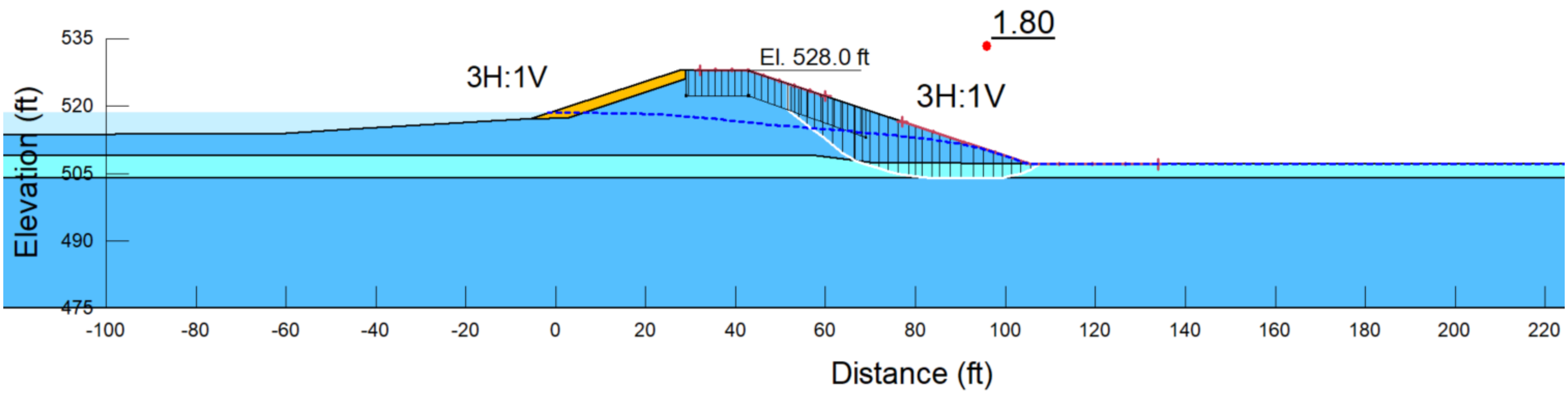
Proposed Embankment Design

Normal Water Loading Conditions, Downstream Slope Stability with Tension Crack



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**14**
PREPARED: HGH			

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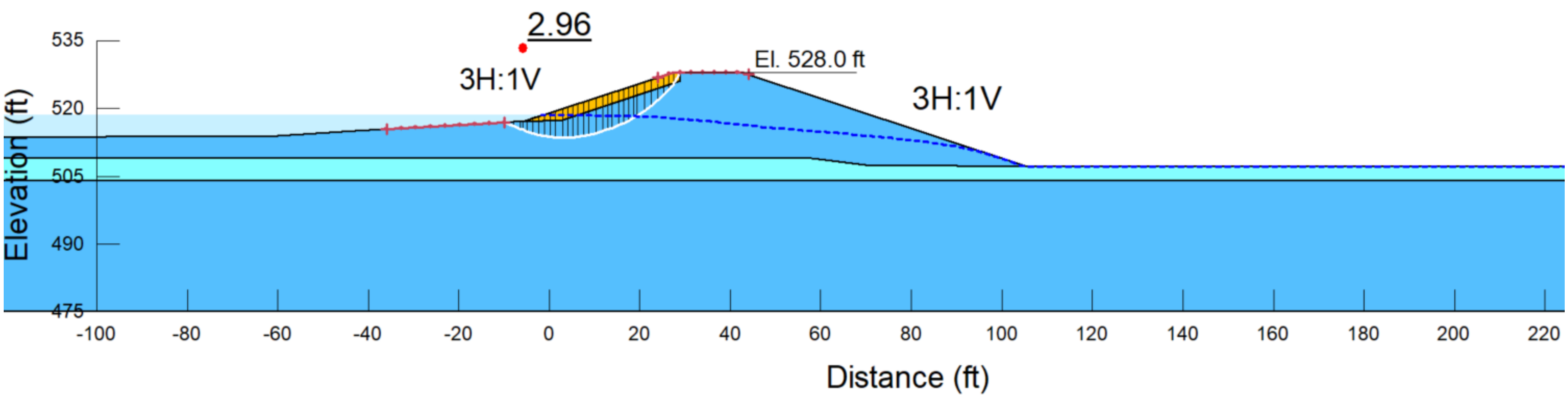
Proposed Embankment Design

Normal Water Loading Condition, Upstream Slope Stability



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

FNI PRO-	PLU17623
DATE:	April 2020
PREPARED:	HGH

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San Antonio Water System
Mitchell Lake Dam Improvements

Slope Stability Analysis
Proposed Design

PLATE
15

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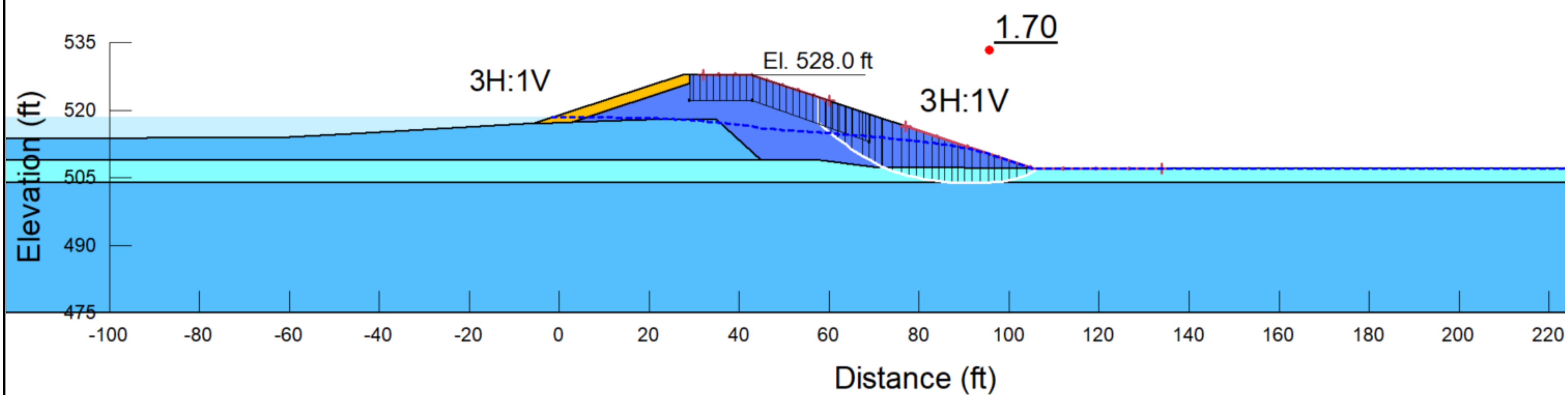
Proposed Embankment Design

Normal Water Loading Conditions, Downstream Slope Stability with

Fully Softened Embankment



Color	Name	Model	Unit Weight (pcf)	Strength Function	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125		200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125		150	23	0
	CL-FSS	Shear/Normal Fn.	125	CL-FSS			0
	Rock Riprap	Mohr-Coulomb	135		0	40	0

FNI PRO-	PLU17623
DATE:	April 2020
PREPARED:	HGH

FREESE AND NICHOLS

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San Antonio Water System

Mitchell Lake Dam Improvements

Slope Stability Analysis

Proposed Design

PLATE

16

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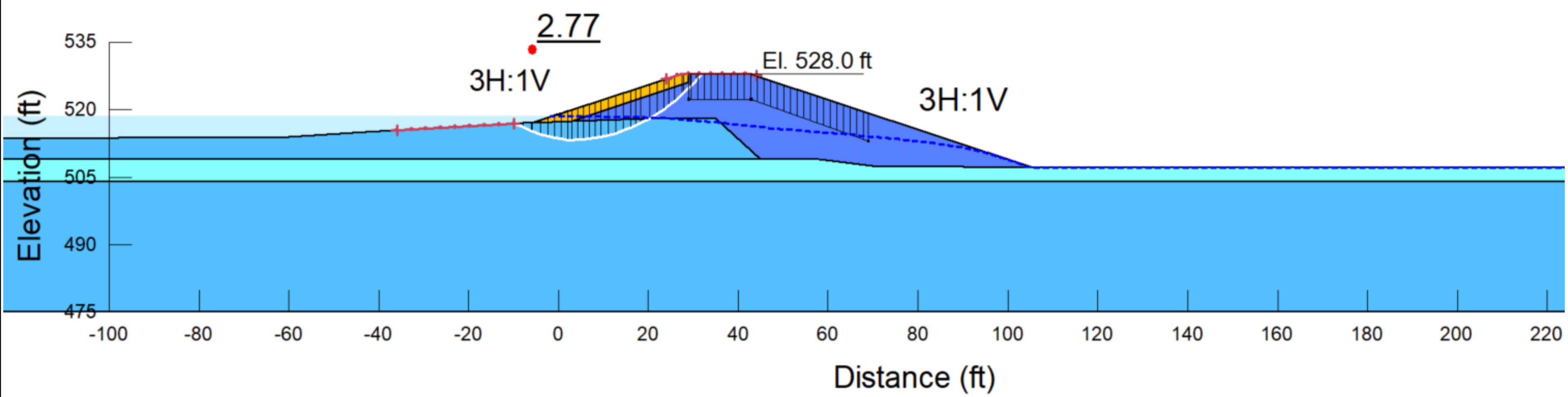
Proposed Embankment Design

Normal Water Loading Conditions, Upstream Slope Stability with Fully Softened Embankment



Color	Name	Model	Unit Weight (pcf)	Strength Function	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125		200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125		150	23	0
	CL-FSS	Shear/Normal Fn.	125	CL-FSS			0
	Rock Riprap	Mohr-Coulomb	135		0	40	0

FNI PRO-	PLU17623
DATE:	April 2020
PREPARED:	HGH

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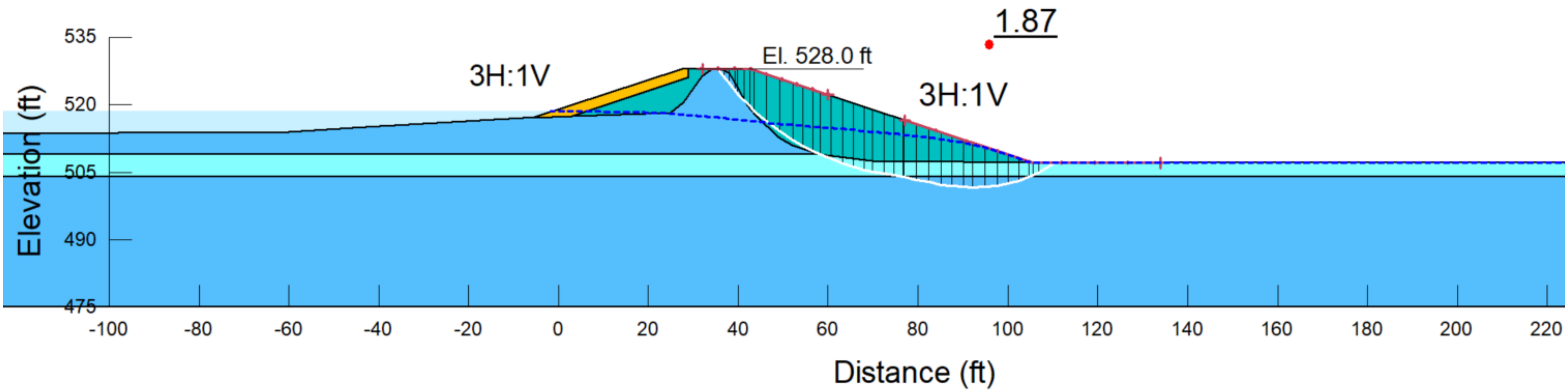
San Antonio Water System
Mitchell Lake Dam Improvements

Slope Stability Analysis
Proposed Design

PLATE
17

Proposed Embankment Design

Normal Water Loading Conditions, Downstream Slope Stability with CU Strength Clay Fill



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	CL-CU Strength	Mohr-Coulomb	125	300	20	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**18**
PREPARED: HGH			

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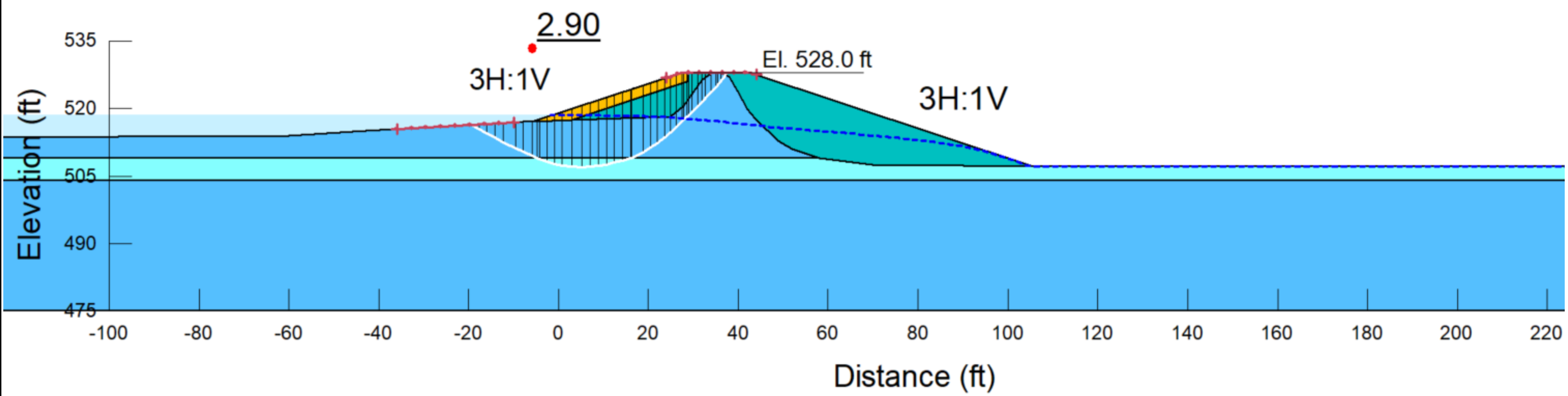
Proposed Embankment Design

Normal Water Loading Conditions, Upstream Slope Stability with CU Strength Clay Fill



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	CL-CU Strength	Mohr-Coulomb	125	300	20	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**19**
PREPARED: HGH			

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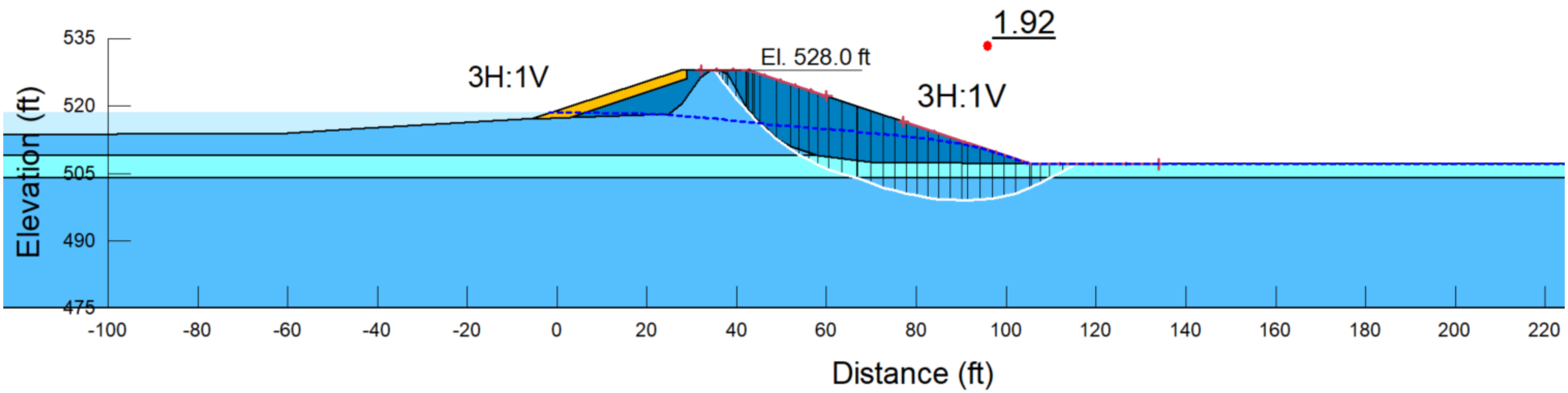
Proposed Embankment Design

Normal Water Loading Conditions, Downstream Slope Stability with UU Strength Clay Fill



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	CL at CD Strength	Mohr-Coulomb	125	200	27	0
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0
	CL at UU Strength	Mohr-Coulomb	125	3,000	0	0
	Rock Riprap	Mohr-Coulomb	135	0	40	0

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**20**
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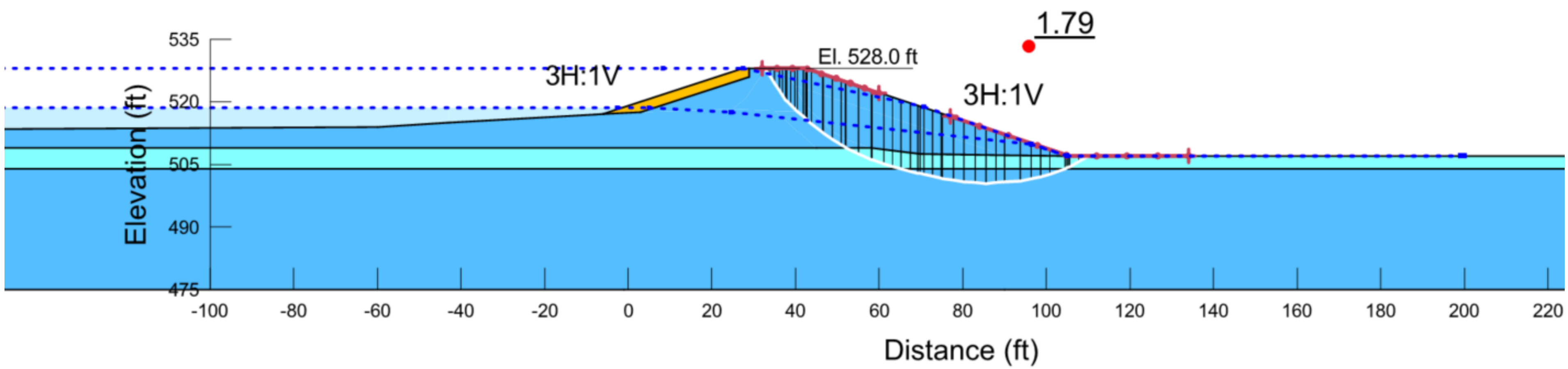
Proposed Embankment Design

Rapid Drawdown from Extreme to Normal Water, Downstream Slope



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)	Piezometric Line	Piezometric Line After Drawdown
	CL at CD Strength	Mohr-Coulomb	125	200	27	0	300	20	1	2
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0	300	20	1	2
	Rock Riprap	Mohr-Coulomb	135	0	40	0	2	39	1	2

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**21**
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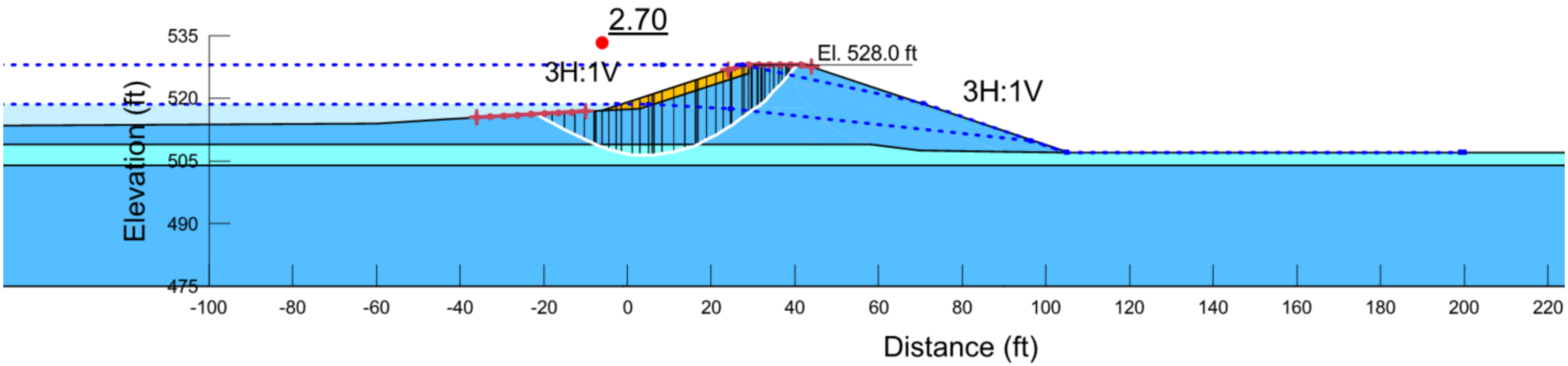
Proposed Embankment Design

Rapid Drawdown from Extreme to Normal Water, Upstream Slope



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)	Piezometric Line	Piezometric Line After Drawdown
	CL at CD Strength	Mohr-Coulomb	125	200	27	0	300	20	1	2
	CL at CD Strength/Softer	Mohr-Coulomb	125	150	23	0	300	20	1	2
	Rock Riprap	Mohr-Coulomb	135	0	40	0	2	39	1	2

FNI PRO- PLU17623	FREESE AND NICHOLS	San Antonio Water System **Mitchell Lake Dam Improvements**	**PLATE**
DATE: April 2020	9601 MCALLISTER FREEWAY, SUITE 1008 SAN ANTONIO, TX, 78216	Slope Stability Analysis **Proposed Design**	**22**
PREPARED: HGH			

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