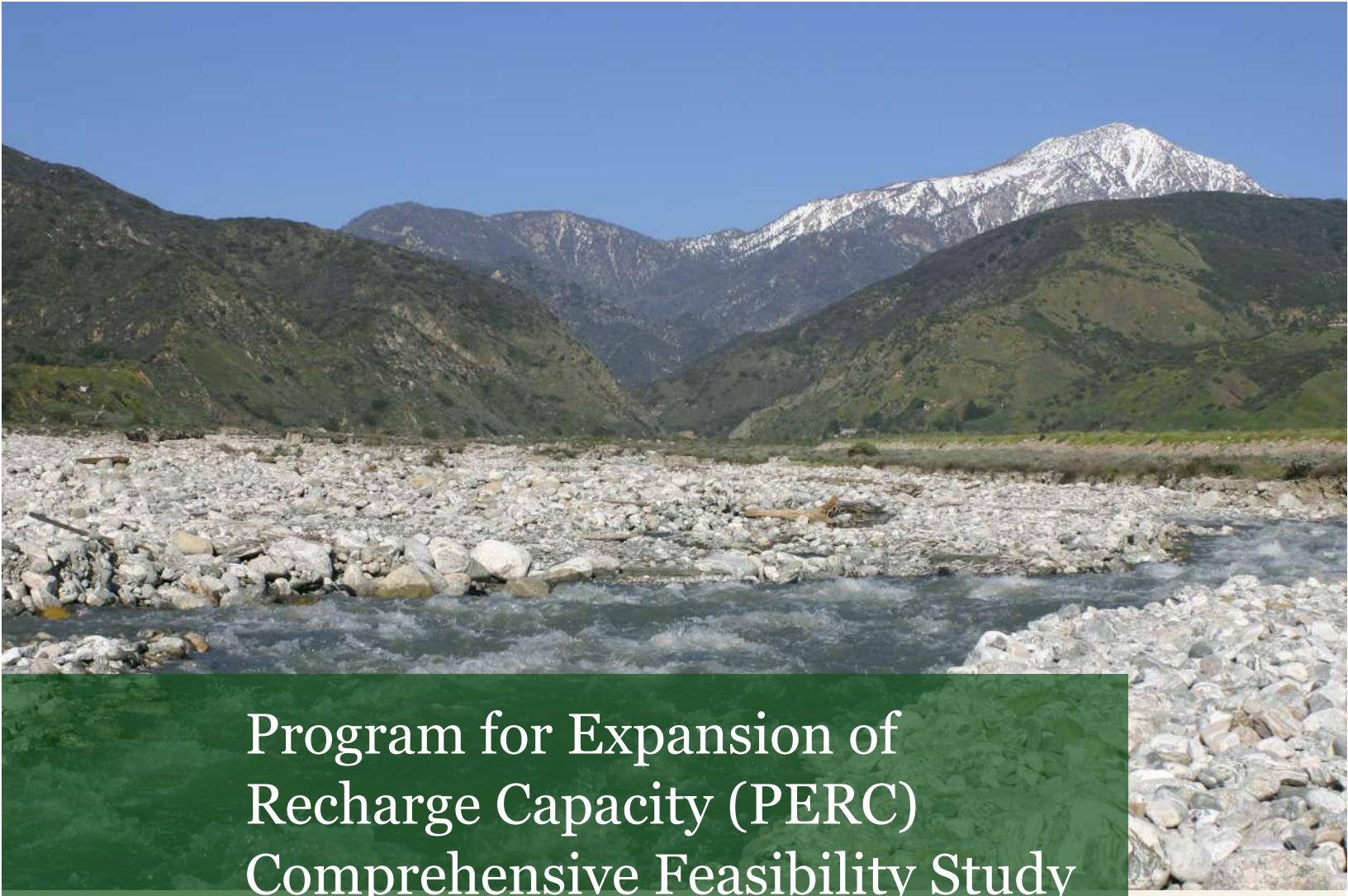


Program for Expansion of Recharge Capacity (PERC) Comprehensive Feasibility Study

Volume 4

Detailed Feasibility Analysis of PERC Project Alternatives in the Mill Creek Watershed

November 2025

San Bernardino Valley
Water Conservation District
Helping Nature Store Our Water

Acknowledgements

The PERC Comprehensive Feasibility Study was made possible through the cooperation and technical contributions of multiple agencies, consultants, and interested parties dedicated to enhancing groundwater recharge and water supply resiliency within the Upper Santa Ana River Watershed. This report reflects the collective efforts of the following:

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Introduction to Volume 4

Detailed Feasibility Analysis of Program for Expansion of Recharge Capacity (PERC) Project Alternatives in the Mill Creek Watershed

Volume 4 of the *PERC Comprehensive Feasibility Study* includes Detailed Feasibility Analyses for the following as PERC Project Alternatives in the Mill Creek Watershed:

- Mill Creek Recharge Basins Alternative 1
- Mill Creek Recharge Basins Alternative 2
- Mill Creek Recharge Basins Alternative 3

Table 1 lists all ten PERC Project Alternatives with the corresponding Study Volume, geographic location, and site for each.

Table 1. PERC Comprehensive Feasibility Study Alternatives

Volume/ Location	Site	PERC Alternative
Volume 2 East Twin Creek Watershed	Waterman Recharge Basins	Waterman Grading Alternative
	Twin Creek Recharge Basins (TCRB)	TCRB Small Grading Alternative
		TCRB Large Grading Alternative
	Lynwood Recharge Basins	Lynwood Operational Alternative
Volume 3 Plunge Creek Watershed	Oak Creek Recharge Basins (OCRB)	OCRB Construction Alternative
	Plunge Creek Recharge Basins (PCRB)	PCRB Interim Construction Alternative
		PCRB Ultimate Construction Alternative
Volume 4 Mill Creek	Mill Creek Recharge Basins (MCRB)	MC Grading Alternative 1
		MC Grading Alternative 2
		MC Grading Alternative 3

Technical details supporting Volume 4, such as the Preliminary Design Report, can be found in the Appendices. Additional documentation is provided in the Citations at the end of this report.

About Mill Creek Watershed

Mill Creek, in San Bernardino County, is a 17.8-mile-long stream that is a major tributary of the Santa Ana River. It originates in the San Bernardino Mountains and flows towards its confluence with the Santa Ana River, which is located just downstream of the upper Santa Ana Canyon mouth and SBVWCD's Mill Creek Recharge Basins (also known as Mill Creek Spreading Grounds).

Mill Creek Recharge Basins

1 Executive Summary

There are three PERC Project Alternatives for the Mill Creek Recharge Basins (MCRB); known as Grading Alternatives 1, 2, and 3. All the Alternatives propose improvements to the existing basins, addition of new basins, and construction of a bypass channel for maintenance.

- **Alternative 1** includes improvements to **29 basins** and addition of **3 new basins**.
- **Alternative 2** includes improvements to **31 basins** and addition of **6 new basins**.
- **Alternative 3** includes improvements to 31 basins plus addition of **10 new basins**.

A summary of the Alternatives' scores and ranks is provided in the table below.

MCRB Alternative 3 ranks highest of the Mill Creek Alternatives, at 3rd place among all 10 PERC Project Alternatives. It has an estimated annual groundwater recharge benefit of **3,751 AFY** and estimated infiltration-rate capacity of 91.4 cfs. It generates a **firm yield of 155,664 AF** of recharge over the 50-year planning horizon – the highest of any Alternative.

The projected capital cost is $84 million with an annual O&M cost, in 2025 value, estimated at $224,815.

MCRB Grading Alternative 3 ranks very high according to the priorities established by the PERC partners with the **3rd highest score** and warrants consideration for implementation.

MCRB Grading Alternatives Scores and Ranks

Consideration (MAA Account)	**Alternative 1** Score	**Alternative 2** Score	**Alternative 3** Score
Groundwater Recharge Benefits	1.96	2.19	2.33
Cost Aspects	0.79	0.57	0.57
Other Technical Aspects	0.44	0.47	0.51
Social Aspects	0.23	0.25	0.26
Legal Aspects	0.65	0.65	0.62
Total Score	**3.95**	**4.14**	**4.29**
Rank (of 10)	**7th**	**5th**	**3rd**

2 Introduction

There are three potential PERC Project Alternatives for the MCRB, known as **Grading Alternatives 1, 2, and 3**. In this document, the Alternatives are evaluated based on the five aspects identified by SBVWCD and project partners as important and relevant to ranking PERC Project Alternatives, which are:

- Groundwater Recharge Benefits
- Cost Aspects
- Other Technical Aspects
- Social Aspects
- Legal Aspects

A complete description of these aspects, their respective components, and the ranking methodology for PERC Project Alternatives is provided in Volume 1.

Sections 3 through 5 below provide information specific to the MCRB Grading Alternatives, including the following:

- **Section 3 – Site and Features**: Descriptions of the site location, existing facilities, geologic setting, and surrounding watershed.
- **Section 4 – Design and Analysis**: Descriptions of water sources, the three proposed Alternative designs, and the characteristics of each as they align with the five aspects (listed above).
- **Section 5 – Summary Results**: Tables presenting the scores and ranks of Alternatives 1, 2, and 3.

3 Site and Features

The Mill Creek recharge site in Mentone, California, plays a crucial role in SBVWCD's overall groundwater management strategy, contributing to the replenishment of the Bunker Hill Groundwater Basin aquifer which is a vital water source for the San Bernardino area.

The generally undeveloped eastern section of the property is bounded by Mill Creek on the north, Valalla Lane and Mill Creek Road on the south, Garnet Street on the west, and San Bernardino Valley Municipal Water District's Tate Pump Station on the east. The larger western portion of the site is bound by Mill Creek on the north, Garnet Street on the east, and the Horace Hinkley Treatment Plant and an orchard on the west. The southern boundary of the property is formed by Madeira Avenue from Garnet Street to approximately 1,000 feet west of Sapphire Avenue. The boundary turns north at this location to about San Bernardino Avenue, which then serves as the southern boundary extending westward to the treatment plant.

The intake and eastern-most basin (Basin 1) are located near Garnet Street and follow Mill Creek downstream to the western-most basin near the confluence with the Santa Ana River.

The Site location is shown in wide-scale aerial view in **Figure 1**.

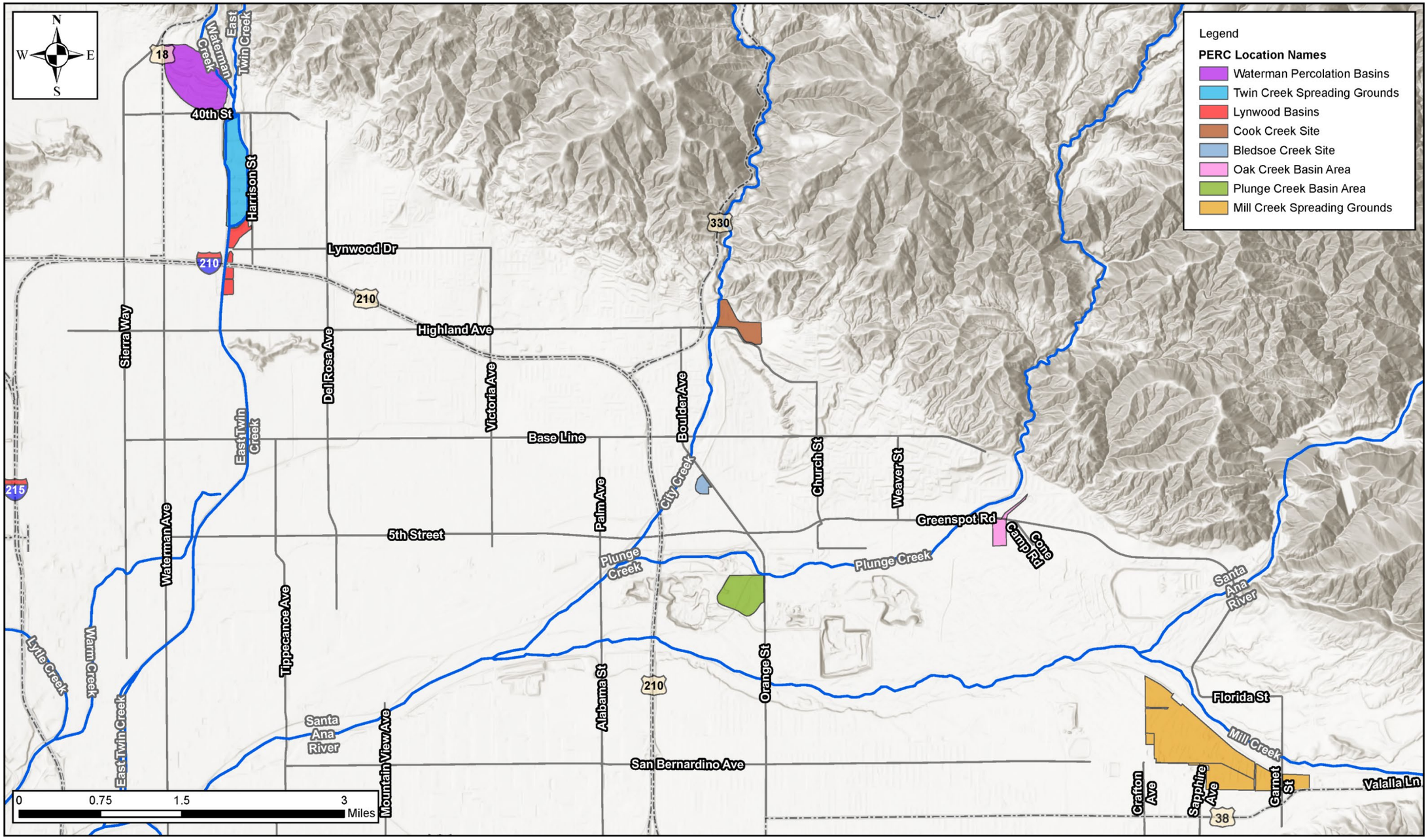


Figure 1. PERC Project Locations

3.1 Existing Facilities

The existing Mill Creek recharge facilities include 60 retention basins (**Figure 2**). Additional locations also retain water during large overflow events but are not considered part of the typical operations. The current system has three main flow inlet locations: South Intake (Mill Creek), North Intake (Mill Creek), and the Redlands Aqueduct.

The main Mill Creek diversion occurs at the South Intake, which consists of a reinforced concrete control structure that diverts most flows to the eastern portion of the basins, and some flows west into the North Intake. Flows from Mill Creek are controlled at the South Intake. Flows that reach the intake structure are further controlled upstream within the creek via "soft plugs". Soft plugs are natural berms of bed material graded by the District to divert low flows towards the South Intake system. The soft plugs vary in size from 2 to 4 feet in height and have an estimated capacity of up to 400 cfs. During large flow events within the creek, these earthen structures overtop and allow creek flows to continue downstream. After large events, District staff reestablish the "soft plugs".

The Redlands Aqueduct diverts Santa Ana River water to the western basins through a controlled structure. The Redlands Aqueduct diverts flows from the Santa Ana River at a rate of up to 25 cfs. Over the past two years, flows through this structure have averaged 13 cfs. discharging directly into Basin #45. Future allocations suggest that 25 cfs can be potentially delivered during wet years.

The recharge basins currently operate through a series of weirs, channels and culverts. The weirs allow for raising, lowering and diverting flows from one basin to another to allow for optimization and/or taking basins offline for maintenance. Some of the existing diversion structures are natural features set in place by District staff to divert flows.

The MCRB are configured to have desilting basins immediately downstream of the intake structures. Basins 1 and 2 are considered desilting basins for the South Intake (see Figure 2) for existing basin configurations. Basin 12 is the desilting basin for the North Intake. The basins downstream from the Redlands Aqueduct are not considered detention basins, as the flows from the aqueduct are relatively clean.

Currently, the basins operate using a series of weirs and man-made earthen diversions that allow access for maintenance. As part of the maintenance, SBVWCD removes sediment, primarily from the desilting basins, and stores the material adjacent to the basins. Basin inverts are scarified to help promote good characteristics of percolation.



Figure 2. Existing Mill Creek Recharge Basins

3.2 Site Assessment

Site assessment included a full site aerial and ground survey to identify all basin inlet and outlet structures. In April of 2022, Guida Surveying Inc. conducted an aerial survey to obtain Light Detection and Ranging (LiDAR) survey to produce 1-foot contours and surface digital terrain model (DTM). Supplemental ground surveys were also performed at the internal structures within the basins and the main intake structures. This information was the basis for hydraulic modeling. A DTM was created and imported as a 3-D surface into HEC-RAS for the base model.

3.3 Geotechnical Assessment and Analysis

Geotechnical subsurface investigations were performed to identify percolation rates and to provide geotechnical design recommendations. Due to environmental constraints, most of the field test locations had to be altered from the original plan. Final testing locations are provided as **Figure 3**. The original goal of testing soils at the existing basin inverts had to be adjusted in many cases to areas around the perimeter of the basins. Preliminary Design Report (PDR) Attachment II, *Mill Creek Geotechnical Investigation Report*, includes the geotechnical design recommendations and percolation rates developed by Kleinfelder (Kleinfelder, 2025). These rates were used in the HEC-RAS models, referred to as the "Kleinfelder Infiltration" models.

For design purposes, SBVWCD provided percolation rates based on findings from adjacent basin geotechnical investigations. At these facilities, basin invert sampling was more available and thus more relevant. Consequently, a second hydraulic model was prepared, using "SBVWCD Infiltration" values.

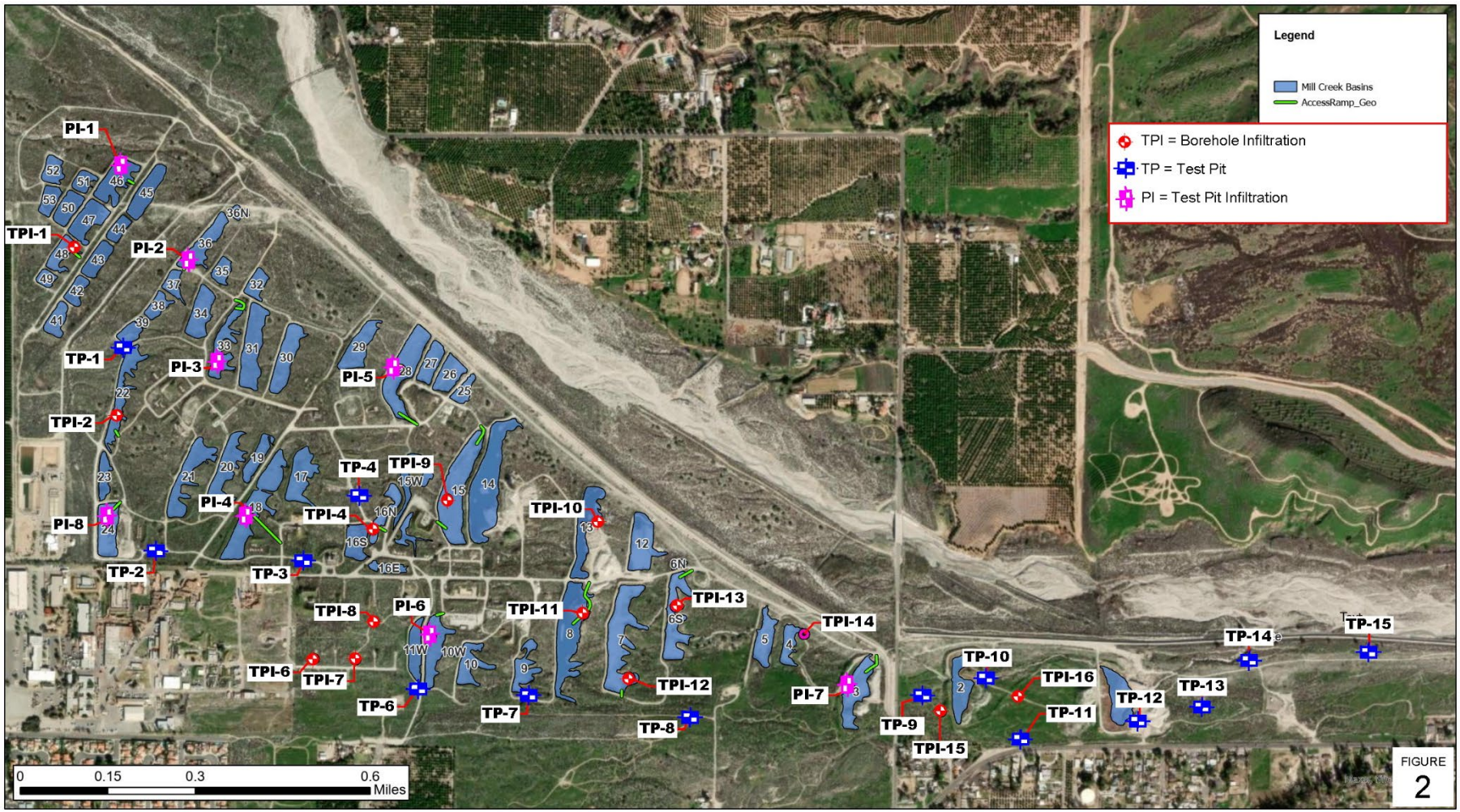


Figure 3. Geotechnical Investigation Site Locations

Design guidelines were established from the investigation that helped identify slope stability for proposed basin grading. Per the requirements of the study, a maximum slope of 3:1 was recommended for all basin slopes, upstream and downstream. In some cases where existing basins contain slopes less than this, with maximum water surface elevations above existing grade, the consultant team will likely propose to improve these slopes as part of the next phase of design.

Detailed discussions of the geotechnical subsurface investigation and analysis results are included as part of PDR Attachment II, *Mill Creek Geotechnical Investigation Report*.

3.4 Watershed Hydrology

A hydrology study, *Mill Creek Regional Flood Frequency Analysis* (PDR Attachment 3) was conducted as part of this project in the form of a regional flood frequency analysis using available data from the Mill Creek stream gages as well as surrounding regional stream gages in the area. This study helped to understand the frequencies of various flows that could potentially be diverted into the MCRB project so that the modeling team could run simulations for existing conditions and MCRB Grading Alternatives 1, 2, and 3. **Figure 4** depicts the Mill Creek Watershed and its geography relative to the MCRB facilities.

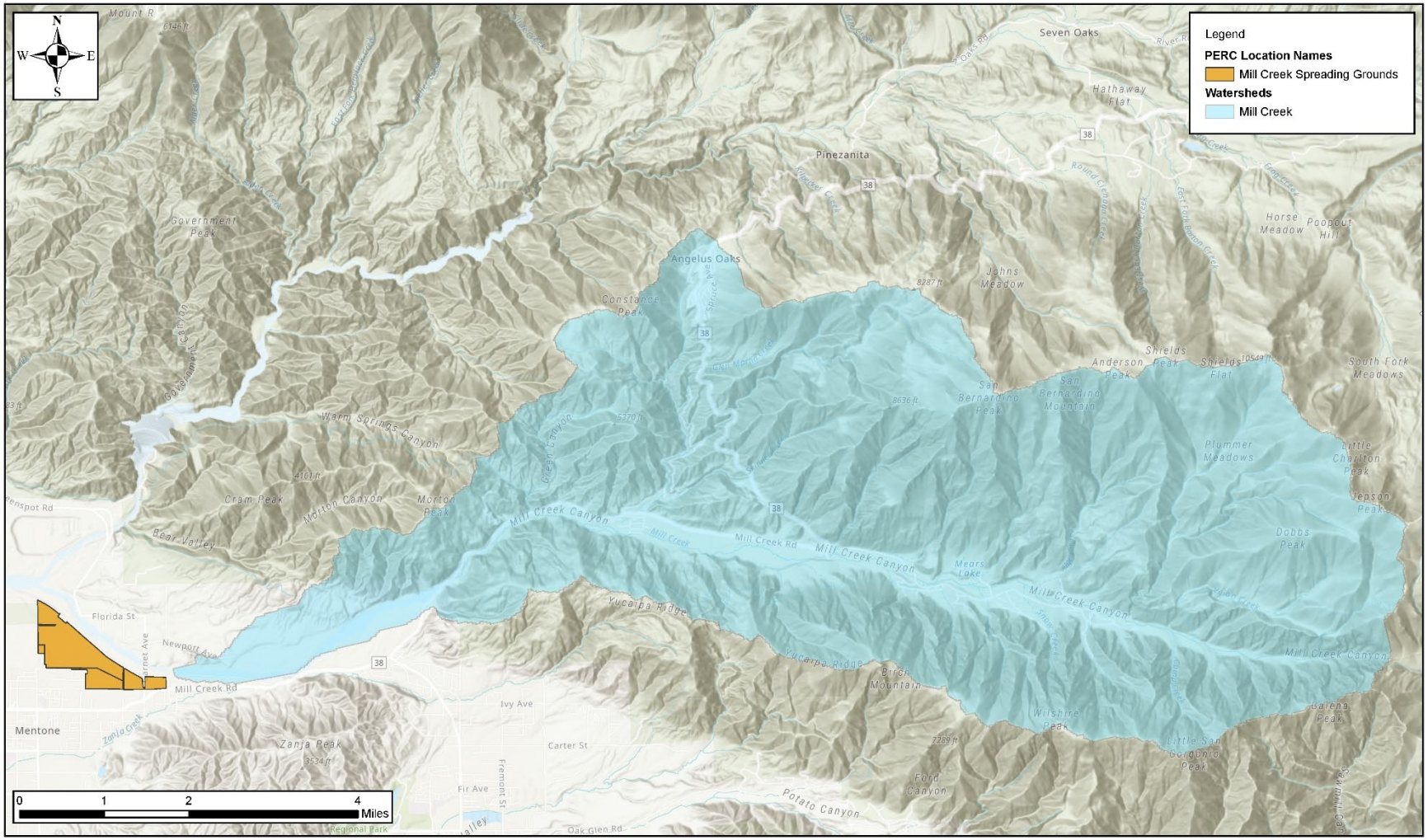


Figure 4. Mill Creek Watershed and MCRB

3.5 Water Source Hydrology

Two operational flowrates were used to account for typical operational flows and potential maximum flowrates. Estimates for recharge yield were based on the typical operational flows, where the larger flowrates provided a "stress" test of the facilities and their hydraulic structures.

- **Operational Flows**: The typical, or expected daily, operational flowrate was selected to be **46 cfs at the diversion from Mill Creek** to the recharge basins just upstream from the North Intake. This, in conjunction with the Forecast-Informed Reservoir Operations (FIRO) flows, constituted the expected daily flow scenario models.
- **Stress Flows**: Stress flow scenarios were based on the maximum potential flows diverted from Mill Creek, based on the hydraulic capacity of the diversion structure. This flow, in conjunction with flows from two FIRO flow sources, constituted the stress flow scenario models.

Detailed analysis can be found in PDR Attachment III (Flood Frequency Analysis).

3.5.1 FIRO Flows

Forecast-Informed Reservoir Operations (FIRO) is a process led by USACE Research and Development Center (ERDC) and the U.C. San Diego Scripps Institute of Oceanography (Scripps) Center for Western Weather and Water Extremes in cooperation with San Bernardino Valley Municipal Water District (SBVMWD), with the goal to identify strategies to protect and increase the reliability of the local water supply.

As a result of this process, specific flows have been identified at the Mill Creek basin site with the reliance of independent flows from multiple sources throughout the year in addition to flows captured from the Mill Creek diversion.

Table 2 shows the various sources of expected future FIRO flows for the Mill Creek basins. For purposes of analysis in the study, a total of 131 cfs was used for the FIRO flow evaluations.

During the latter part of the final hydraulic calculations, the FIRO flows were revised per SBVWCD discussion with other agencies. The revised flows were reduced by 25 cfs from what was used in this preliminary analysis. These changes will be made during 65-percent design. Nevertheless, the initial FIRO flows can still be used for comparative purposes as the reduction would be consistent in all cases. The original FIRO flows used in the calculations are listed in **Table 2** below.

Table 2. Expected Daily FIRO Flows Modeled at Mill Creek Basins

Source	Inlet Location	Flow (cfs)
ZT Unger (FIRO)	South Intake	40
Greenspot (FIRO, Valley District)	South Intake	20
South Intake (Mill Creek)	South Intake	16
North Intake (Mill Creek)	North Intake	30
Redlands Aqueduct (FIRO)	Redlands Aqueduct	25
	TOTAL FLOW	**131 cfs**

The FIRO flowrate approximates an expected daily flow for a wet year. Statistically, flows within Mill Creek (46 cfs) have a return period of 5.5 years. This 46 cfs is divided between the South and North Intakes. The FIRO flow scenario also includes 25 cfs at the Redlands Aqueduct, 40 cfs from the ZT Unger site, and 20 cfs from the Greenspot site.

3.5.2 Stress Flows

Another flowrate used to model different scenarios within the Alternatives is referred to as the "Stress" flows. Stress flows include the ***maximum potential diversion flow*** that the upper, newly modified, diversion structure can convey. This flow was calculated to be a maximum of 162 cfs. The flow is then divided between the South and North Intakes. An additional 13 cfs is added from the Redlands Aqueduct. For most of the stress scenarios, the flowrate used at the South Intake was 122 cfs, with 40 cfs diverted to the North Intake, and 13 cfs at the Redlands Aqueduct for a total peak flowrate of 175 cfs (**Table 3**).

Table 3. Expected Stress Flows Modeled at Mill Creek Basins

Source	Inlet Location	Flow (cfs)
South Intake (Mill Creek)	South Intake	122
North Intake (Mill Creek)	North Intake	40
Redlands Aqueduct	Redlands Aqueduct	13
	TOTAL FLOW	**175 cfs**

4 Design and Analysis

All three **MCRB Grading Alternatives** would improve multiple basins, add new basins, and construct a bypass channel (South Canal) to redirect flows as needed for maintenance operations.

Recent modification to the South Intake structure has allowed for a slight increase in flow capacity to enter the basins (at Basin 1). Improvements to that structure also provide protection against large boulders and debris from accumulating within the main intake chute.

One major improvement conceptualized for the MCRB Grading Alternatives is expansion of existing desilting Basins 1 and 2. All proposed improvements show these two basins being expanded to five new basins (Basin 1, 1A, 1B, 1C, and 2). The proposed south canal runs along the southern edge of all five basins and provides the ability to take Basins 1, 1A, 1B, and 1C offline for maintenance, utilizing Basin 2 (and possibly Basin 3) for desilting.

Figure 5 shows the proposed improvements of the desilting basins (existing basins in red). The proposed diversion canal can be seen traversing along the south edge of the proposed new basins. Again, desilting basin expansion and addition of the South Canal are included with all three MCRB Grading Alternatives.

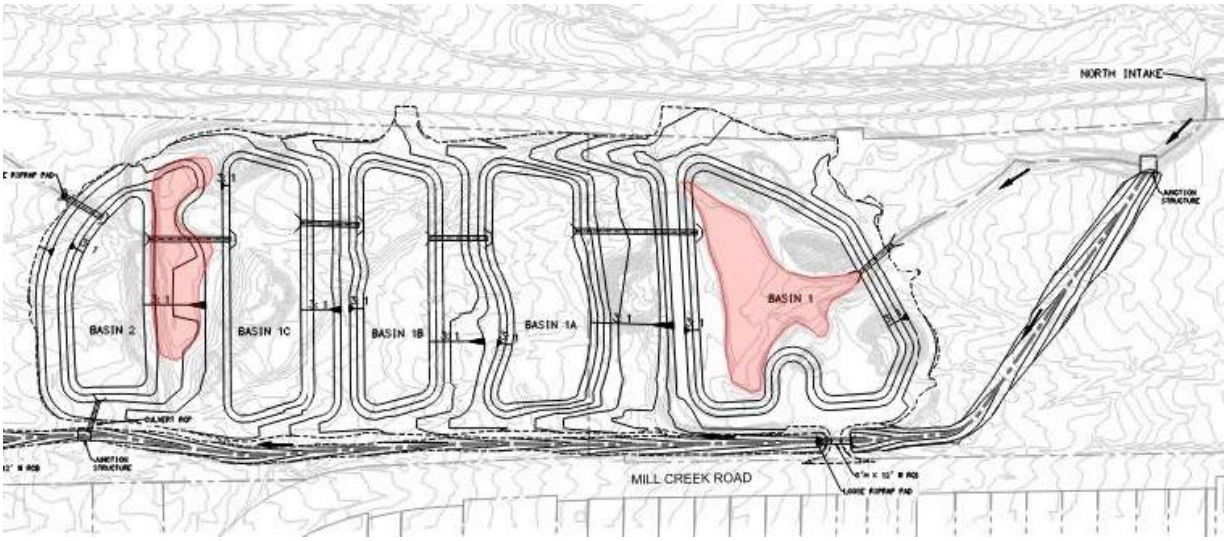


Figure 5. Proposed Desilting Basins and New South Canal Below South Intake

4.1 Proposed Design

All basin improvements utilize a maximum graded slope, both upstream and downstream, of 3:1 for slope stability. Current conditions at some basins reveal downstream slopes of 1.5:1 or steeper. These basin slopes were not considered at this conceptual phase but will be modified in the 65-percent plans. In most cases, they will be increased to 3:1 if the maximum basin water surface elevations (WSE) extend higher than the existing downstream grade.

Based on findings from the Geotechnical Investigation Report (PDR Attachment II), some proposed basins may require modifications to the maximum allowable WSE to ensure slope stability. These modifications will be provided in the 65-percent design and will not greatly affect the overall alternative recharge yields.

The main outlet for each alternative is Basin 52-53, which is a proposed improvement to existing Basin 53. This differs from the existing basin system, which has two outlets, Basin 53 and Basin 52.

Each basin inlet and outlet structure will be improved to accommodate additional flows and provide better operation control within the basin system. The main control is within the outlet structures. A "Monk-Type" structure (**Figure 6**) is proposed to be used that allows for weir boards to be manually placed within the structure to control the WSE.

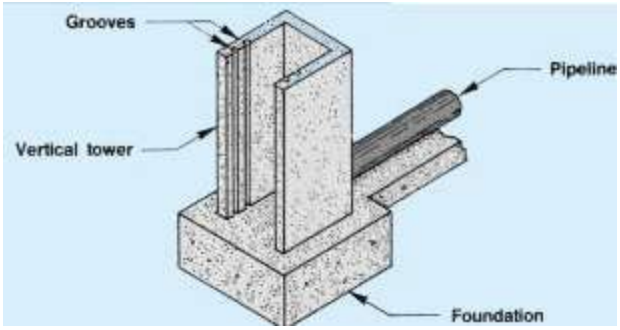


Figure 6. Typical Outlet ("Monk" Structure)

Some outlet weir structures will be constructed with a reinforced concrete pipe and some with a rectangular channel.

Vertical rising sluice gates may also be used within the south canal that operate either manually or automatically. Exact locations may be identified during the 65-percent design.

Alternatives 1, 2, and 3 each contain a proposed six diversion structures. These structures are in-line diversions meant to control flow patterns within the system of basins for the purpose of optimizing flows within the system and for maintenance operations. These types of structures are proposed to be sluice gate-type structures that can be automated. Exact types of diversions will be identified in the 65-percent design plans.

4.1.1 MCRB Grading Alternative 1

MCRB Grading Alternative 1 includes improvements to 29 existing basins and the addition of 3 new basins. Many of the existing smaller basins are proposed to be combined and regraded into larger basins. A summary of these modifications is provided in **Table 4** below.

Table 4. MCRB Grading Alternative 1 Modification Summary

Modification	Basin Numbers
Increase Volume	1, 2, 3, 4, 6s, 7, 8, 9, 10, 11, 13, 14w, 15w, 16n, 16s, 18, 22, 23, 24, 29, 34, 35
Combine and Increase Volume	36-37, 38-40, 42-43, 44-45, 46-47, 50-51, 52-53
New Basins	1a, 1b, 1c

Figure 7 shows the proposed improvements for Alternative 1. This figure also shows the locations for diversion structures, the South Canal and its proposed connections to various basins, and improved basin outlet structures. It is anticipated that 45 new inlet structures, 8 sluice gates, and 6 diversion gates will be constructed as part of Alternative 1.

4.1.2 MCRB Grading Alternative 2

Alternative 2 includes all the improvements of Alternative 1 (Table 4) in addition to three new basins (24e, 41, 41s), and two additional modifications to existing basins (29, 31).

Figure 8 shows the proposed improvements for Alternative 2. Improvements that are not included in Alternative 1 are highlighted in orange. It is anticipated that 49 new inlet structures, 8 sluice gates, and 6 diversion gates will be constructed as part of Alternative 2.

4.1.3 MCRB Grading Alternative 3

Alternative 3 includes all the improvements of Alternative 2 in addition to four new basins (SW1, SW2, SW3, SW4). **Figure 9** shows the proposed improvements for Alternative 3. Improvements that were not included in Alternatives 1 or 2 are highlighted in orange. It is anticipated that 56 new inlet structures, 8 sluice gates, and 6 diversion gates will be constructed as part of Alternative 3.

Figures 7, 8, and 9 are simplified overviews of the conceptual-level Alternatives. More detailed plans are included in the PDR (Volume 4, Appendix A).

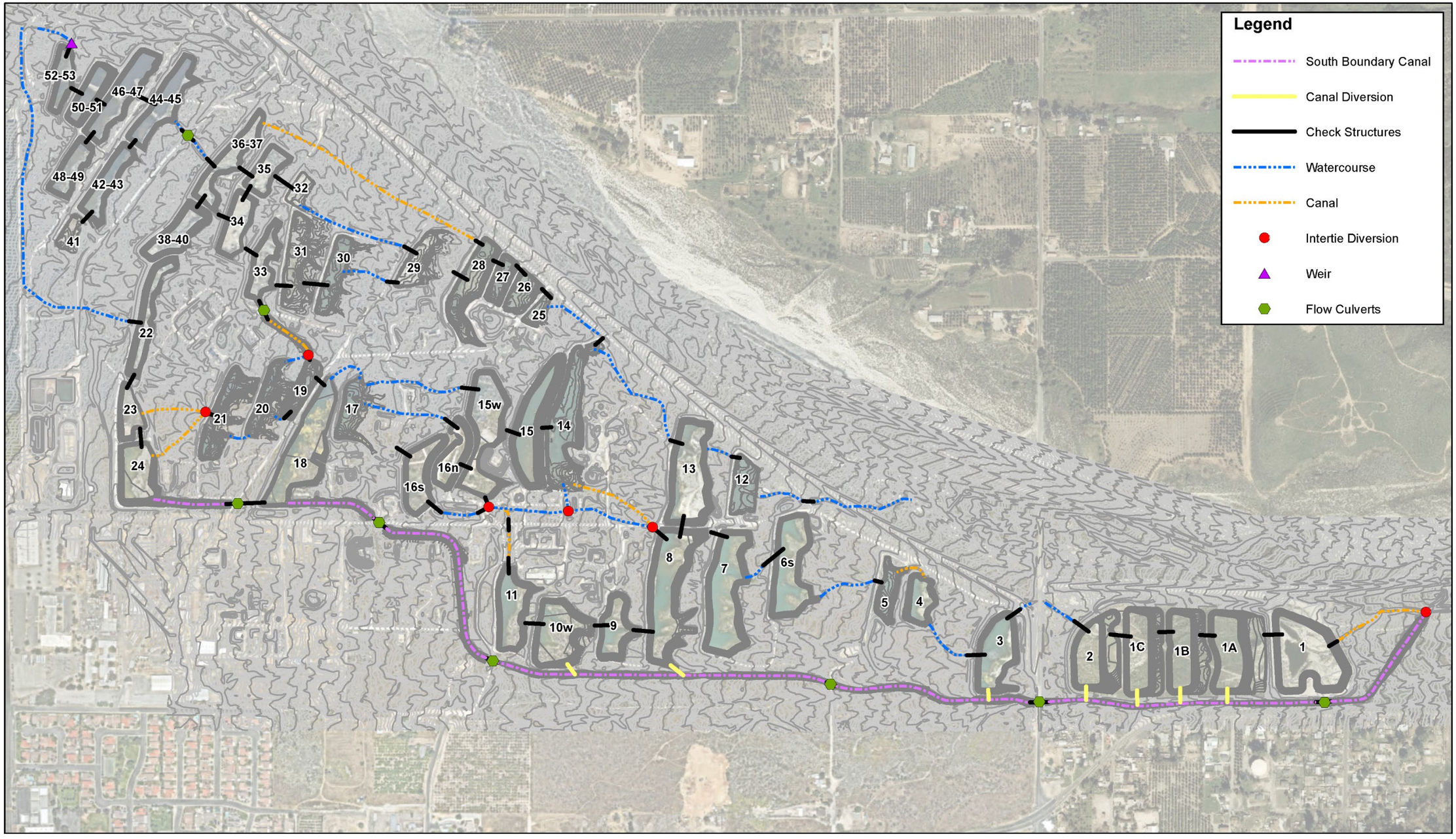


Figure 7. MCRB Grading Alternative 1 Overview

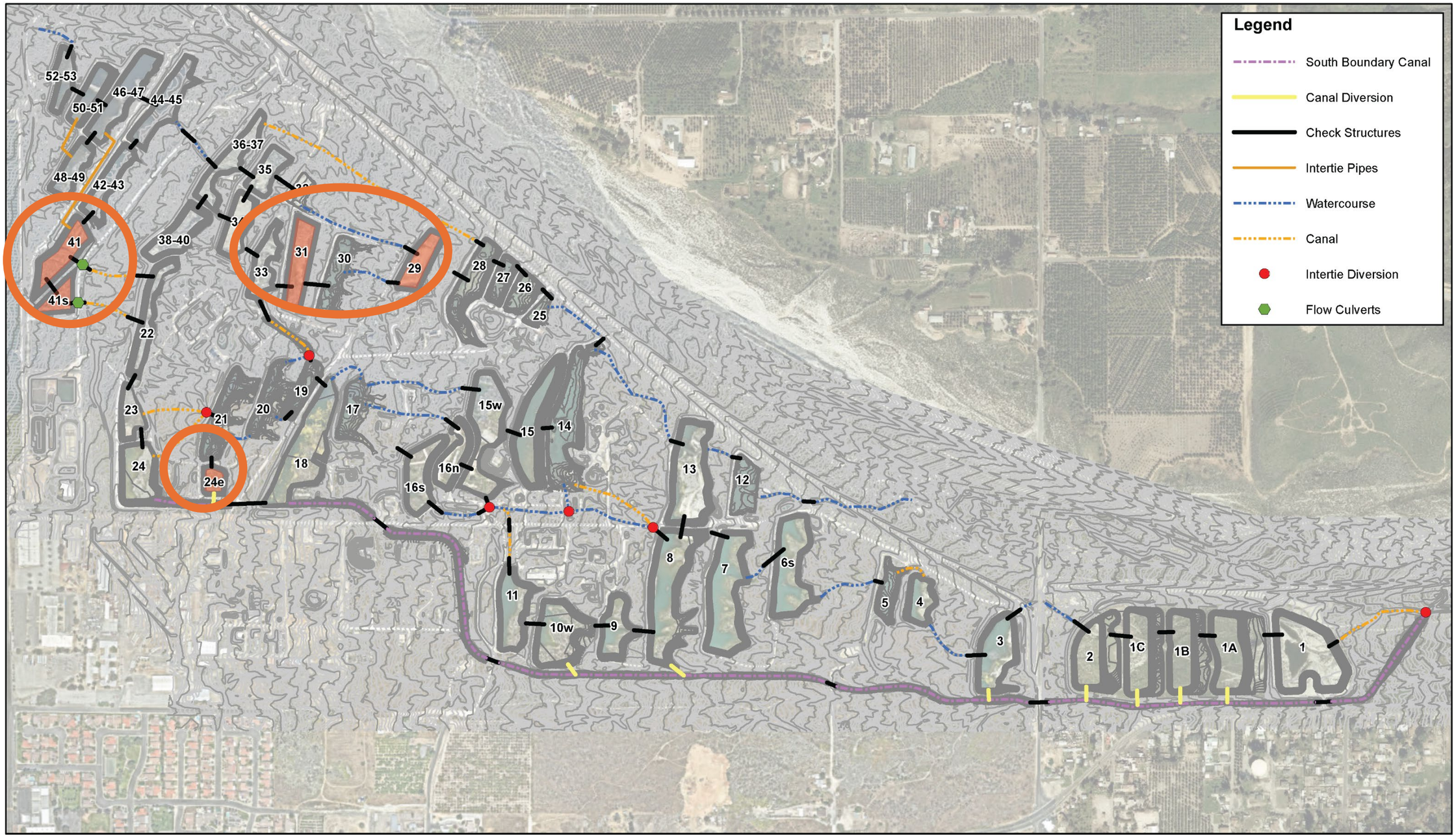


Figure 8. MCRB Grading Alternative 2 Overview

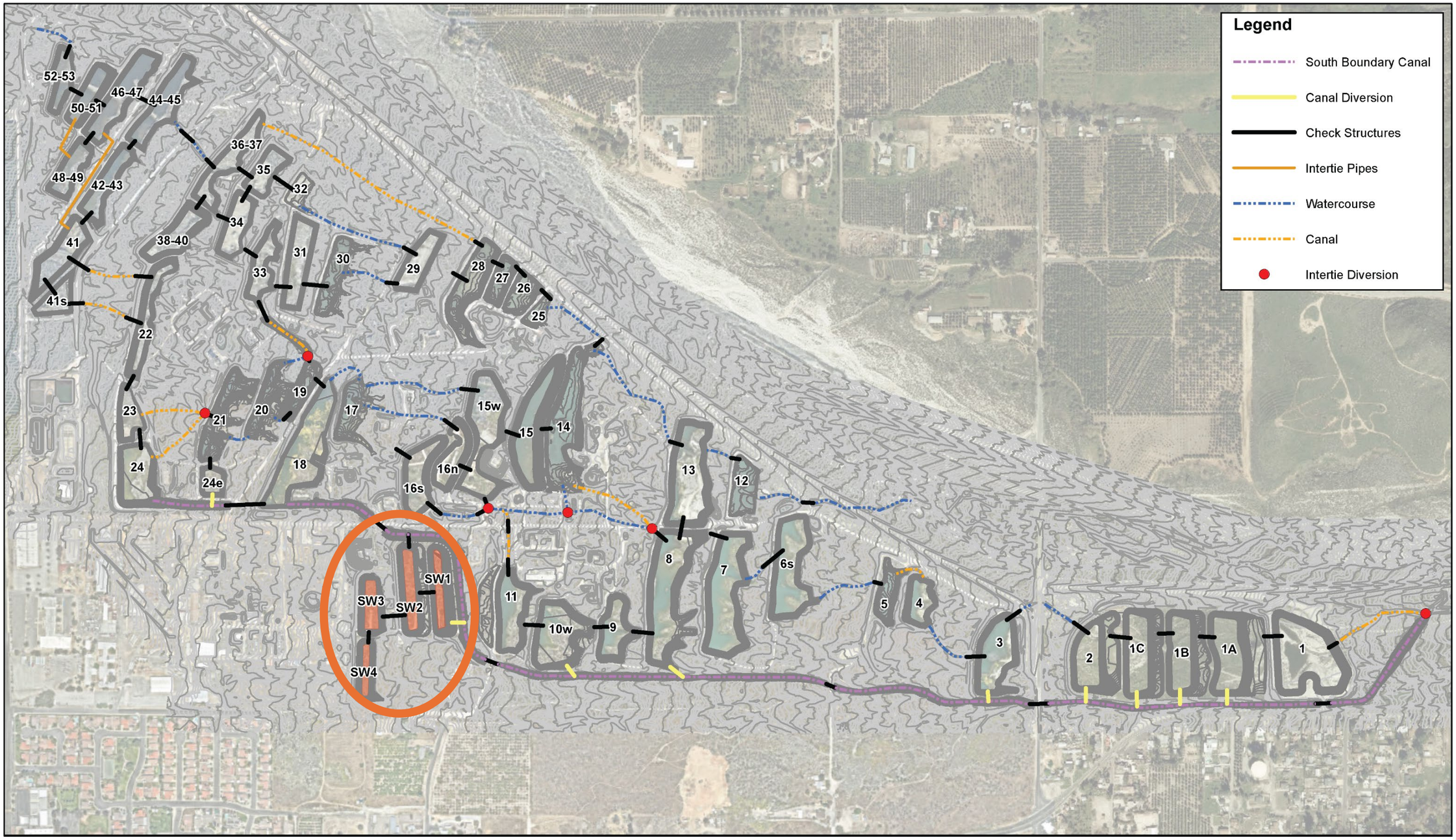


Figure 9. MCRB Grading Alternative 3 Overview

4.1.4 Geotechnical Considerations

Preliminary geotechnical recommendations for design and construction where modifications are proposed for existing detention basins and outlet structures; for new basins, diversion structures, and inlet and outlet structures; and for a new conveyance channel as shown on the preliminary design plans are provided in the *Geotechnical Design Recommendations Memorandum* (PDR Attachment II).

These recommendations by Kleinfelder were based on testing and evaluation they conducted of anticipated seepage and stability of the berms associated with the proposed improvements in conjunction with their previously prepared *Field Investigation and Infiltration Testing Memorandum* for the Mill Creek recharge facility dated May 23, 2024.

4.1.5 Project Operations

MCRB operations will primarily consist of diverting Mill Creek water off the main channel and into the intake structures where flows can be diverted to the north or south cannels. Once flows are in the cannels, water is diverted into different basins where water can flow from one basin to the next downstream basin. All gates are manually adjusted to maintain a desirable water level. Daily observations will be made at the intake structures and recharge basins to maintain safe and desirable recharge activities.

Project operations for **FIRO water** rely upon releases to the MCRB from the Redlands Aqueduct or from the Greenspot Pipeline. FIRO releases may be limited to non-storm periods or the dry season depending on available capacity of the MCRB.

4.1.6 Timeline

As described in Volume 1 in the Comprehensive PERC Feasibility Study Report, the timeline for project alternatives is calculated by estimating the time to design, permit, fund, and construct a project. Volume 1 summarizes these estimates alongside those for the other PERC Project Alternatives. Each timeline factor for the MCRB Grading Alternatives is described below. Timing for all 3 Alternatives is the same.

- **Design Time**: Design for any of the MCRB Grading Alternatives to the 65+% level required for permitting consideration is estimated to take **18 months**.
- **Permit Time**: The permit time forecast for MCRB Grading Alternatives is **18 months**. This is based on SBVWCD and SBVMWD experience and programmatic expertise.
- **Funding Time**: The time forecast spanning from project decision-making to having a capital funding plan in place is estimated to be **72 months**. The funding timeline is thought to be initiated after the Design and Permitting Time. The timing of project

funding then is **July 1, 2034**. This is the timeline used to generate a net present value factor for capital funding needs.

- **Construction Time**: The construction period includes the time needed to take project designs into biddable specifications and contract documents, conduct a public bidding process for contractor selection, the project construction period, and project works commissioning. For the MCRB Grading Alternatives, this construction period is estimated to require **18 months**.

The project operational period is estimated to be **January 1, 2036, to December 31, 2075**. The project O&M costs are estimated over this period. Estimated groundwater recharge yield is predicated on a 40-year operating period.

4.2 Groundwater Recharge Benefits

Hydraulic analysis was conducted to evaluate current and potential recharge yield for each MCRB Grading Alternative. Full results are available in the *Mill Creek Basin Hydraulic Technical Memorandum* (PDR Attachment I).

The model used to evaluate the basins was the USACE HEC-RAS 6.5 software. Flows were added to the model at three primary locations: South Intake; North Intake; and Redlands Aqueduct intake. The flows were based on the two previously discussed **FIRO** and **Stress** flows (Section 3.5). Each of these flows were modeled as constant inflows over the duration of the model run. In most cases, the inflow durations lasted for up to 5 days.

- **Modeling Existing Conditions**: The existing condition model was evaluated to provide a sense of calibration, or correlation, between the model results and reality. The results were reviewed by SBVWCD engineers and field crews to verify that the results were realistic. The existing conditions model showed that the capacity of the current basins *cannot adequately recharge* future predicted FIRO flows.

- **Modeling Future Conditions with Alternatives**: Alternatives 1, 2, and 3 model runs were evaluated to understand the probable benefit of increasing the size and recharge yield of basins. Model runs were performed for multiple flow and maintenance scenarios for each Alternative. The full list of maintenance scenarios is provided in PDR Attachment I.

The model results reflected the performance of the basin interconnections, including weir structures and intertie channel systems to function under different scenario conditions. The results were provided for hydrogeologic investigation to understand the groundwater dynamics for the given proposed basins, and to identify if “mounding” would occur under their current proposed grading footprints. The groundwater models were run for a 52-day duration. The three representative models included the following:

- Existing Condition Basins (FIRO Flows)
- Alternative 1, Scenario 7 (FIRO Flows)
- Alternative 3, Scenario 1 (Stress Flows)

FIRO flows are *preliminary estimates* and subject to change, as the FIRO program is still in development. **Table 5** shows the total inflow volumes for each of the model conditions used. It should be noted that these values assume models have reached a steady state condition.

Table 5. Groundwater Model Daily Inflows and Outflows (District Infiltration Rates)

Modeled Condition	Total Inflow (cfs)	Total Outflow (Discharge at Basin 53) (cfs)	Total Flow Captured (cfs)
Existing Conditions (FIRO Flows)	131	50.3	80.7
Alternative 1, Scenario 7 (FIRO Flows)	131	9.9	121.1
Alternative 3, Scenario 1 (Stress Flows)	175	5.7	169.3

Alternative 3, Scenario 2 (FIRO) was not evaluated in HEC-RAS since the Alternative 1, Scenario 7 model showed several "dry" basins. Since Alternative 3 has several *more* proposed basins than Alternative 1, the FIRO flows would have left more basins "dry" with a similar "total flow captured" as Alternative 1, Scenario 7.

Alternative 3, Scenario 1 was tested using the "Stress" flows to understand how the system would react to the expected maximum flow diversion. It can be deduced that both Alternative 2 and Alternative 1 would predictively "discharge" larger flows out of Basin 53.

Multiple iterations of each Alternative and Scenario were performed to try and fill as many basins as possible. This method requires substantial time, as each iteration of each structure requires recalculation, which can take up to five days each. Consequently, for models showing excess system outflow (out of basin 53), a ratio (based on infiltration rate) was used to approximate how "dry" basins, if filled, could reduce the system outflow by way of capture and percolation. Although these numbers were not completely optimized (via HEC-RAS), they provide a guide as to the probable groundwater recharge rate.

4.2.1 Annual Average Recharge

The estimated annual groundwater recharge benefits for the MCRB Grading Alternatives are presented in **Table 6**.

Table 6. MCRB Grading Alternatives Annual Average Recharge

Annual Groundwater Recharge	Alternative 1	Alternative 2	Alternative 3
Groundwater Recharge Benefit (net new AFY)	131 AFY	50.3 AFY	80.7 AFY

4.2.2 Infiltration-Rate Capacity

A geotechnical investigation was performed to determine the existing basin infiltration parameters. As part of the analysis, a range of percolation values were presented for each of the basins. Some hurdles of this testing involved the environmental permitting. This resulted in providing estimates for areas not tested. The detailed *Mill Creek Geotechnical Investigation Report* is provided as PDR Attachment II. Model scenarios run with these infiltration rates are referred to as "Kleinfelder infiltration" rates.

SBVWCD developed a range of infiltration parameters for "clogged" conditions based on experience with comparable facilities. The clogged condition assumes a non-maintained condition that reduces the potential infiltration of each basin. The clogged condition was the basis for running proposed Alternatives through the model to identify a worst-case scenario with respect to groundwater recharge yield. A full list of each basin's percolation rate for the clogged scenario can be found in PDR Attachment II. Model scenarios run with these infiltration rates are referred to as "District infiltration" rates.

The rating curves for each basin were modeled in HEC-RAS using outflow rating curves. Calculations were made for each basin based on depth, surface area, and percolation rate. The resulting infiltration outflow hydrographs consisted of a depth vs outflow in cubic feet per second. These rating curves can be seen in PDR Attachment II.

4.2.3 Firm Yield Over Planning Horizon

The MCRB Grading Alternatives have a 40-year period of operation within the PERC 50-year planning horizon (2025-2075). Based on the average annual yield, the firm yields of increased engineered recharge for the Mill Creek Grading Alternatives range from 93,000 to 155,000 AF over the 50-year planning horizon (**Table 7**).

Table 7. MCRB Grading Alternatives Firm Yield

Groundwater Recharge Benefit	Alternative 1	Alternative 2	Alternative 3
Firm Yield over 50-Year Horizon (net new AF)	93,286 AF	128,105 AF	155,664 AF

4.2.4 Groundwater Recharge Benefits Summary

Table 8 presents the Groundwater Recharge Benefits values associated with this PERC Project Alternative that were input to the MAA Model for final scoring.

Table 8. MCRB Grading Alternatives Groundwater Recharge Benefits Summary

MCRB Groundwater Recharge Benefit Summary			
Alternative	Annual Average New Recharge (AFY)	Infiltration Rate Capacity Increase (cfs)	Firm Yield over 50-Year Planning Horizon (AF)
MCRB Alternative 1	2,248	70.3	93,286
MCRB Alternative 2	3,087	86.6	128,105
MCRB Alternative 3	3,751	91.4	155,664

4.3 Cost Aspects

4.3.1 Capital Costs

A preliminary construction cost estimate was developed for each alternative. This estimate is part of a larger study to evaluate current and potential recharge yield for various improvements and to identify a cost-benefit relationship to support SBVWCD with information to garner a recommendation.

A detailed preliminary cost analysis was conducted and included in a technical memorandum (PDR Attachment V). Environmental mitigation cost estimates were also included based on coordination with SBVWCD. A total construction contingency of 30-percent was added to each project.

Below are the preliminary cost estimates for each of the Alternatives. **Table 6** shows the total construction costs, including contingencies, which range from $74M-$84M on a 2025 basis. Although some details are still being developed, the estimates should provide a reasonable insight to project costs. For a detailed breakdown of costs, refer to PDR Attachment V.

Table 9. Preliminary Capital Cost Estimates

Alternative	Estimated Capital Cost (USD)
MCRB Grading Alternative 1	$73,800,000
MCRB Grading Alternative 2	$78,200,000

Alternative	Estimated Capital Cost (USD)
MCRB Grading Alternative 3	$ 84,000,000

The timing of the capital needed is **July 1, 2034 to fund construction.** The discount factor used for capital is 4% with an expectation that the cost of money is 7% and inflation is 3%. The projected construction employment in FTE equates to **170** (Alternative 1), **180** (Alternative 2), or **190** (Alternative 3) full-time jobs for 18 months.

4.3.2 Operations & Maintenance

MCRB maintenance would principally be for siltation management and basin clogging. Based upon the type and quality of water being recharged, staff will observe the infiltration rate. When infiltration at a basin reaches an undesirable level, that basin will be taken offline to allow the basin to dry and have the sediment removed. Any basin repairs needed will be made at that time.

Diversion structures at the main channel will be maintained on a daily basis. The gates are operated manually and require exercising daily. Any debris build up will be removed. Cannels will be maintained as needed throughout the life of the project. Diversion structures will be monitored and maintained as needed.

Comparison of operating costs are made in present day dollars. The average annual costs of basin maintenance are estimated at **$170,000-$225,000 per year**. The O&M costs are not discounted for the sake of comparison since they will be incurred in the year that they occur. Therefore, projecting a borrowing cost for capital is not pertinent for operations. Similarly, inflationary factors are not material to comparing costs that would be similarly inflated over a similar period. The total employment forecast for O&M of the MCRB Grading Alternatives is 0.99 FTE (Alternative 1), 1.15 FTE (Alternative 2), or 1.28 FTE (Alternative 3) on a perpetual basis.

4.3.3 Cost Aspects Summary

Table 10 presents the Cost Aspects values associated with this PERC Project Alternative that were input to the MAA Model for final scoring.

Table 10. MCRB Grading Alternative Cost Aspects Summary

MCRB Cost Aspects Summary		
Alternative	**Capital Costs (2025$)**	**Annual O&M Costs (2025$)**
MCRB Grading Alternative 1	$73,800,000	$170,843
MCRB Grading Alternative 2	$78,200,000	$205,068
MCRB Grading Alternative 3	$ 84,000,000	$224,815

4.4 Other Technical Aspects

Other aspects of the MCRB Grading Alternatives are summarized here for use in comparison among other PERC Alternatives in the MAA, as presented in Volume 1.

4.4.1 Groundwater Quality Improvements

The addition of recharge into the Bunker Hill Basin Zone B is rated on its prospective annual average recharge for the equivalent MGD. That quantification is in keeping with the discussions and evaluations of Salt & Nutrient Management Planning. MCRB estimated average daily recharge rate based on a historic probability-based calculation of how often and how much water is materially available in the annual average groundwater recharge potential equates to the following for each Alternative:

- MCRB Grading Alternative 1: **6.2 MGD**
- MCRB Grading Alternative 2: **7.0 MGD**
- MCRB Grading Alternative 3: **7.6 MGD**

4.4.2 Conservation and Habitat Benefits

There are no anticipated species impacts from the MCRB Grading Alternatives, as the basins are existing features. Nor is there any habitat restoration.

The MCRB Grading Alternatives are a part of the Upper Santa Ana River Habitat Conservation Plan (ICF, 2020). The expected benefit to flows in the Santa Ana River is **1.2 cfs** as a chronic year-round rate from any of the three Alternatives.

4.4.3 Hydraulic Stress and Pressure Zone Influence

The value of relief to hydraulic stress is described in Volume 1. It equates to the distance from the PERC Project Alternative to the center of the Pressure Zone near West 5th Street and Waterman Avenue in the City of San Bernardino. The MCRB Grading Alternative features are measured as **17,100 yds**.

4.4.4 Access to State Water Project Supply

The MCRB Grading Alternatives have **direct access** to State Water Project supply.

4.4.5 Groundwater Mounding

The result of the HEC-RAS models were processed and delivered to the team hydrogeologists for further analysis. Results were prepared as GIS shapefiles for basin surface areas and infiltration results. This data is added to the groundwater models to identify impact on the Bunker Hill Basin, as well as to identify proposed basin grading characteristics in the way of "mounding". The hydrogeology models provide guidance as to basin proximity. If basins are too close together, exfiltration from one basin may inadvertently provide a source of inflow into another basin.

Analysis indicated that the peak mounding from the MCRB Grading Alternatives would occur around 50 days after the start of spreading in both clogged and unclogged conditions (Geoscience, 2025). Recharge would also result in an increase in the area of shallow groundwater (groundwater less than fifty feet below ground surface). It is recommended to install additional monitoring wells in locations further east from the current monitoring wells to gather water level data that can be used to improve the groundwater model's calibration. The construction of these wells would be done in the next phase of the project development. Results of the current groundwater model are summarized in **Table 11**.

Table 11. Groundwater Mounding Results Summary (PCRB Interim Alternative)

Scenario	**Maximum Change in Groundwater Elevation** (ft)	**Time of Maximum Mounding** (days after start of recharge)	**Baseline Area of Shallow Groundwater** (acres)	**Recharge Scenario of Shallow Groundwater** (acres)
Clogged	10	52	1,324	1,379
Unclogged	20	51	1,322	1,418

4.4.6 Other Technical Aspects Summary

Table 12 presents the Other Technical Aspects values associated with this PERC Project Alternative that were input to the MAA Model for final scoring.

Table 12. MCRB Grading Alternative Other Technical Aspects Summary

MCRB Other Technical Aspects Summary					
Alternative	Average Annual Recharge Rate (MGD)	Santa Ana River Contribution (cfs)	Proximity to Groundwater Pressure Zone (yds)	Access to SWP Supply	Groundwater Mounding
MCRB Alt. 1	6.2	1.2	17,100	Direct	Yes
MCRB Alt. 2	7.0	1.2	17,100	Direct	Yes
MCRB Alt. 3	7.6	1.2	17,100	Direct	Yes

4.5 Social Aspects

4.5.1 Job Creation

The MCRB Grading Alternatives projected construction employment in FTE is equal to **250** (Alternative 1), **260** (Alternative 2), and **280** (Alternative 3) **full-time jobs for 18 months**. The total employment forecast for O&M of the MCRB Grading Alternatives is 0.99 FTE (Alternative 1), 1.15 FTE (Alternative 2), or 1.28 FTE (Alternative 3) on a perpetual basis.

4.5.2 Ponded Water Depth

The MCRB Grading Alternatives ponded water depths is **12 feet**. One social aspect that will be taken into consideration is prevention of drowning. Experience at SBVWCD basins demonstrates that water basins will attract unhoused individuals or passersby in hot weather to enter the basin. Access control will be a key consideration in design. Additionally, graded slopes for easy ingress and egress may reduce drowning and other safety hazards from sloped ground.

4.5.3 Social Aspects Summary

Table 13 presents the Social Aspects values associated with this PERC Project Alternative that were input to the MAA Model for final scoring.

Table 13. MCRB Grading Alternative Social Aspects Summary

MCRB Social Aspects Summary			
Alternative	Construction Jobs (FTE/time period)	O&M Jobs (FTE)	Ponded Water Depth (ft)
MCRB Alt. 1	250 /18 months	0.99	12
MCRB Alt. 2	260 /18 months	1.15	12
MCRB Alt. 3	280 /18 months	1.28	12

4.6 Legal Aspects

4.6.1 CEQA and Permitting Analysis

A brief overview of the potential permit needs and issues for any of the MCRB Grading Alternatives is provided in **Table 11** below (Alternatives 1, 2, and 3 would all have the same requirements).

Table 14. Potential Permit Requirements for the MCRB Grading Alternatives (1, 2, and 3)

Potential Permit	Preliminary Review Determination
CEQA	Based on preliminary assessment, an Initial Study will be required, which is expected to result in a Mitigated Negative Declaration (IS/MND1 for Alternatives 1 and 2; IS/MND2 for Alternative 3).
USFWS	Anticipated Safe Harbor Agreement.
CDFW CESA 2081	Expected to be typical.
CDFW LSA /1600	Expected to be typical.
USACE 401/ RWQCB	None required.
USACE 404	None required.
USACE 408	Expected to be typical.
CA DOSD Jurisdictional Dam	None required.
SBCFCD	None required.

The MCRB footprint is inside the Federal Aviation Administration (FAA) secondary buffer zone of 10,000 feet, Perimeter B, from Redlands Municipal Airport. The FAA has specific design and permitting review for water holds that could provide nesting habitat for waterfowl; a flight

safety hazard. If the MCRB Construction Alternatives moves forward, FFA should be consulted to ensure all requirements are met.

4.6.2 Land Availability

This land is currently owned by SBVWCD.

4.6.3 Support for Claimed Water Rights

The support for new Water Rights not yet perfected is equated to the maximum diversion rate to groundwater recharge, estimated for MCRB Grading Alternative 1 at 70.3 cfs, Alternative 2 at 86.6 cfs, and Alternative 3 at 91.4 cfs.

4.6.4 Legal Aspects Summary

Below are the Legal Aspects values associated with this PERC Project Alternative that were input to the MAA Model for final scoring.

Table 15. MCRB Grading Alternatives Legal Aspects Summary

MCRB Legal Aspects Summary		
CEQA/Permits Required	Land Availability	Support for Claimed Water Rights (cfs)
See Table 9	Yes	Alternative 1: 70.3 cfs Alternative 2: 86.6 cfs Alternative 3: 91.4 cfs

5 Summary Results

The PERC Comprehensive Feasibility Study looked at each Project Alternative's characteristics and viability over the following key considerations: Technical, Economic, Operational, and Legal Feasibility. In some respects, this amounts to a discovery process to identify whether the project has any fatal flaws that prevent its execution. The MCRB Grading Alternatives **do not have any apparent fatal flaws**. More information can and would be developed for the MCRB Grading Alternatives if one or more is taken forward as a PERC project for permitting, design, and construction.

5.1.1 MAA Indicator Values

PERC and its regional partners executed an integrated multiple accounts analysis (MAA) by assessing their key considerations and weighting them. The MAA developed for PERC as a comprehensive analytical tool and the weighted scoring method are described in Volume 1. **Table 16** shows the MAA Indicators and their respective MCRB Grading Alternative Values.

Table 16. PCRB Interim Construction Alternative MAA Indicator Values

Account	Subaccounts	Indicators	MCRB Grading Alternatives Values		
			Alternative 1	Alternative 2	Alternative 3
Groundwater Recharge Benefits	Groundwater Recharge-Benefit	AFY	2,248	3,087	3,751
	Infiltration-rate-capacity increase	Net Increase in Infiltration Rate Capacity (cfs)	70.3	86.6	91.4
		Ease of Maintenance (FTE)	0.59	0.66	0.73
		Timeline to Implement (years)	8.5	8.5	8.5
	Recharge benefit on a "firm yield" 50-year horizon	AF	93,286	128,105	155,664
Cost Aspects	Capital Costs	Capital Costs NPV	$73,800,000	$78,200,000	$84,000,000
		Grant Sources	20%	20%	20%
	O&M	O&M in 2025 $	$170,843	$205,068	$224,815
Other Technical Aspects	Groundwater Quality Improvements	Volume of Annual Recharge (MGD)	6.2	7.0	7.6

Account	Subaccounts	Indicators	MCRB Grading Alternatives Values		
			Alternative 1	Alternative 2	Alternative 3
	Conservation and Habitat Benefits	Santa Ana River Contribution (cfs)	1.2	1.2	1.2
		Habitat Restoration (acres)	7	14	29
		Species/ Habitat Impacts (acres)	-35.3	-38.3	-42.8
	Proximity to Groundwater Pressure Zone	Distance to Pressure Zone (yds)	17,100	17,100	17,100
	Access to State Water Project Supply	Direct, Indirect or No Access	Direct	Direct	Direct
	Groundwater Mounding	Rate Limiting Mounding *(Liquefaction Issue)*	Yes	Yes	Yes
Social Aspects	Project Job Creation	Number of Jobs Forecast for Construction	170	180	190
		Number of Jobs Forecast from Operational Needs	0.99	1.15	1.28
	Public Safety Hazards of Project	Ponded Water Depth (ft)	12	12	12
Legal Aspects	CEQA	CEQA Filing Type (e.g. NOE)	IS-MND1	IS-MND1	IS-MND2
	Permits Required/ Requirements	USFWS Permit Requirements	Safe Harbor	Safe Harbor	Safe Harbor
		CDFW CESA (2081) Permit	Typical	Typical	Typical
		CDFW LSA /1600 Permit	Typical	Typical	Typical
		USACE/RWQCB 401 Permit	None Required	None Required	None Required
		USACE 404 Permit	None Required	None Required	None Required
		USACE 408 Permit Required	Typical	Typical	Typical

Account	Subaccounts	Indicators	MCRB Grading Alternatives Values		
			Alternative 1	Alternative 2	Alternative 3
		CA DOSD Jurisdictional Dam	None Required	None Required	None Required
		City of San Bernardino Consent Decree Area Permit	None Required	None Required	None Required
		SBCFCD	None Required	None Required	None Required
	Land Availability	Property Ownership by Partner Agencies (Y/N)	Yes	Yes	Yes
	Water Rights	Support for utilization of water rights not yet Licensed (New cfs)	70.3	86.6	91.4

5.1.2 Score and Rank

The MAA Summary Scoring prioritizes **MCRB Grading Alternative 3** as very high with 3rd priority ranking (out of 10) when considering the stakeholder-developed characteristics, criteria, and their weighted importance in meeting regional goals. It is a feasible and viable project that **warrants near-term prioritization** for applying human resources, financial resources, and political capital. Alternatives 1 and 2 rank at 7th and 5th place, respectively.

The summary scoring for the MCRB Grading Alternatives is shown in **Table 17**.

Table 17. MCRB Grading Alternatives MAA Scoring Summary

Account	MCRB Grading Alternatives Summary Scores		
	Alternative 1	**Alternative 2**	**Alternative 3**
Groundwater Recharge Benefits	1.96	2.19	2.33
Cost Aspects	0.79	0.57	0.57
Other Technical Aspects	0.44	0.47	0.51
Social Aspects	0.23	0.25	0.26
Legal Aspects	0.65	0.65	0.62
Total	**4.07**	**4.14**	**4.29**
Priority Rank	**7th**	**5th**	**3rd**

Citations

FAA, 2020, Advisory Circular No. 150/5200-33C re: Hazardous Wildlife Attractants on or near Airports, 47 p. including Appendices.

Geoscience, 2025. "Groundwater Modeling for Mill Creek Diversion and Basins Project Feasibility Study." Prepared for Q3 Consulting and San Bernardino Valley Water Conservation District, dated August 1, 2025.

ICF, 2025 (In Preparation), "Upper Santa Ana River Habitat Conservation Plan" Stakeholder Review Draft, Prepared for San Bernardino Valley Municipal Water District as Lead Program Agency, Update to 2020 stakeholder review draft.

Kleinfelder, 2025, "Draft Geotechnical Design Recommendations ARTP Mill Creek Diversion and Basins Preliminary Design/Feasibility Study Memorandum." Prepared for Q3 Consulting and San Bernardino Valley Water Conservation District, dated August 1, 2025.

Kleinfelder, 2025, "Field Investigation and Infiltration Testing Memorandum, Mill Creek Recharge Facility." Prepared for San Bernardino Valley Water Conservation District, dated May 23, 2024.

Q3 Consulting, 2025, "Mill Creek Basin Cost Technical Memorandum" prepared for San Bernardino Valley Water Conservation District, July 2025.

Q3 Consulting, 2025, "Mill Creek Basin Hydraulic Technical Memorandum" prepared for San Bernardino Valley Water Conservation District, June 2025.

Q3 Consulting, 2025, "Mill Creek Preliminary Design Report" prepared for San Bernardino Valley Water Conservation District, July 2025 DRAFT, 246 p. including Appendices.

Q3 Consulting, 2025, "Mill Creek Regional Flood Frequency Analysis" prepared for San Bernardino Valley Water Conservation District, March 2025 FINAL DRAFT.

Appendix A

Mill Creek Preliminary Design Report

San Bernardino Valley Water Conservation District
Program for Expansion of Recharge Capacity (PERC) Comprehensive Feasibility Study

Volume 4

Detailed Feasibility Analysis of PERC Project Alternatives for Mill Creek Watershed

San Bernardino Valley Water Conservation District

Mill Creek Preliminary Design Report



San Bernardino County, California

July 2025 *(Updated 9-26-2025)*

DRAFT

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Attachment IV Mill Creek Basin Hydrogeology Technical Memorandum (Waiting)
Attachment V Mill Creek Cost Technical Memorandum

1 INTRODUCTION

The San Bernardino Valley Water Conservation District (District) provides Groundwater Recharge for the Bunker Hill Basin. The District entered into an agreement with Q3 Consulting to provide engineering services to develop preliminary design plans and provide technical assistance in evaluating preliminary conceptual plans prepared by the District.

1.1 Purpose and Goals

The purpose of this project is to evaluate current and potential recharge yield for various improvements within the Mill Creek recharge basins facility and to identify a cost-benefit relationship to support the District with information to garner a recommendation for improvements. This report summarizes the results of detailed site analyses focusing on proposed alternative improvements for the Active Recharge Transfer Project (ARTP) Mill Creek basins. Both initial and future conditions were evaluated to understand the potential recharge yield gain and cost to construct various improvements to help support the District's Feasibility Study

1.2 Description of Existing Facilities

The existing Mill Creek Basin location includes 60 retention basins. Additional locations also retain water during large overflow events but are not considered part of the typical operations. The current system has three main flow inlet locations: South Intake (Mill Creek), North Intake (Mill Creek), and the Redlands Aqueduct.

The main Mill Creek diversion occurs at the South Intake, which consists of a reinforced concrete control structure that diverts most flows to the eastern portion of the basins, and some flows west into the North Intake. The Redlands Aqueduct diverts Santa Ana River water to the western basins through a controlled structure.

The basins currently operate through a series of weirs, channels and culverts. The weirs allow for raising, lowering and diverting flows from one basin to another to allow for optimization and/or taking basins offline for maintenance. Some of the existing diversion structures are natural features set in place by District staff to divert flows. Figure 1-1 shows the existing Mill Creek basin system.

1.2.1 Site Survey

The initial project task included a full site aerial and ground survey to identify all basin inlet and outlet structures. In April of 2022, Guida Surveying Inc. conducted an aerial survey to obtain Light Detection and Ranging (LiDAR) survey to produce 1' contours and surface digital terrain model (DTM). Supplemental ground surveys was also performed at the internal structures within the basins and the main intake structures. The horizontal datum for the survey is the California Coordinate System of 1988 (CCS83), Zone 5, North American Datum of 1983 (NAD83), in U.S. Survey Feet, while the vertical datum is North American Vertical Datum of 1988 (NAVD88) in U.S. Survey feet.

This information was the basis for hydraulic modeling. A DTM was created and imported as a 3-D surface into HEC-RAS for our base model.

1.2.2 Mill Creek Basins Source Flows

Flows from Mill Creek are controlled at the South Intake. Flows that reach the intake structure are further controlled upstream within the creek via "soft plugs". Soft plugs are natural berms of bed material graded by the District to divert low flows towards the South Intake system. The soft plugs vary in size from 2 to 4 feet in height and have an estimated capacity of up to 400 cfs. During large flow events within the

creek, these earthen structures overtop and allow creek flows to continue downstream. After large events, District staff reestablish the "soft plugs".

The Redlands Aqueduct diverts flows from the Santa Ana River at a rate of up to 25 cfs. Over the past two years, flows through this structure have averaged 13 cfs. discharging directly into Basin #45. Future allocations suggest that 25 cfs can be potentially delivered during wet years.

1.2.3 Existing Basin System Characteristics

The Mill Creek basins are configured to have desilting basins immediately downstream of the intake structures. Basin 1 and Basin 2 are considered desilting basins for the South Intake (See Figure 1) for existing basin configurations. Basin 12 is the desilting basin for the North Intake. The basins downstream from the Redlands Aqueduct are not considered detention basins, as the flows from the aqueduct are relatively clean.

Currently the system of basins operate using a series of weirs and man-made earthen diversions that allow the District the ability to maintain certain basins. As part of the maintenance, the District removes sediment, primarily from the desilting basins, and stores the material adjacent to the basins. Basin inverts are scarified to help promote good characteristics of percolation.

Figure 1-1. Mill Creek Existing Recharge Basins



2 BASIS OF DESIGN

2.1 Development of Design

The District prepared a first-cut concept design for the proposed Mill Creek basin improvements. The District concept was considered as one of several potential alternatives. After iterations of various inflow parameters, Q3 developed three alternatives for further development. Each of these alternatives considered "maintenance scenarios", which included shutting down certain basins or groups of basins to understand how the system would perform hydraulically. A new "south canal" was proposed by the District's original design to act as a means to bypass specific basins in case of maintenance shut down.

2.2 Concept Alternatives

Three main concepts were developed and included varying levels of basin improvement. Each of these alternatives included modifications to the original District concept design. For this project, the following concepts were developed.

- Alternative 1 – 2 new basins, 30 Improved Basins, and addition of south canal
- Alternative 2 – Same as Alternative 1 with 3 additional new basins and 2 improved basins
- Alternative 3 – Same as Alternative 2 with 4 new basins

Each of these alternatives were evaluated for varying levels of "operation" or maintenance scenarios to refine the internal basin structure sizing to ensure adequate conveyance capacity. The selected alternative will be refined further as part of the 65-percent design. Refer to the figures in Section 3 for a schematic view of each alternative. Exhibits 1-3 show the proposed concept plans for each of the alternatives.

2.3 Geotechnical Analysis

Geotechnical subsurface investigations were performed to identify percolation rates and to provide geotechnical design recommendations. Due to environmental constraints, most of the field test locations had to be altered from the original plan. The original goal of testing soils at the existing basin inverts had to be adjusted in many cases to areas around the perimeter of the basins. Addendum IIa (Kleinfelder, 2024) contains the geotechnical technical memorandum for the study and the resulting percolation rates. These rates were used in the HEC-RAS models, referred to as the Kleinfelder Infiltration models. Design guidelines were also established from the investigation that helped identify slope stability for proposed basin grading. Per the requirements of the study, a maximum slope of 3:1 was recommended for all basin slopes, upstream and downstream. In some cases where existing basins contain slopes less than this, with maximum water surface elevations above existing grade, Q3 will propose to improve these slopes as part of the next phase of design.

For design purposes, the District also provided percolation rates based on findings from adjacent project site basin geotechnical investigations. At these facilities, basin invert sampling was more available and thus more relevant. Consequently, two sets of hydraulic models were prepared using Kleinfelder Infiltration values and District Infiltration values.

Detailed discussions of the geotechnical subsurface investigation and analysis results are included as part of the geotechnical technical memorandum Attachment IIb (Kleinfelder, 2025).

2.4 Hydrologic Analysis

A hydrology study, *Mill Creek Regional Flood Frequency Analysis* (Addendum III, Q3, 2024) was conducted as part of this project in the form of a regional flood frequency analysis using available data from the Mill Creek stream gages as well as surrounding regional stream gages in the area. This study

helped to understand the frequencies of various flows that could potentially be diverted into basins to support the cost-benefit for the various proposed alternatives.

Two operational flowrates were investigated as part of this study to account for typical operational flows and potential maximum flowrates. Estimates for recharge yield were based on the typical operational flows, where the larger flowrates provided a "stress" test of the facilities and their hydraulic structures. The typical, or expected daily, operational flowrate was selected to be 46 cfs at the diversion from Mill Creek to the recharge basis just upstream from the North Intake. This in conjunction with the Forecast-Informed Reservoir Operations (FIRO) flows constituted the expected daily flow scenario models.

Stress flow scenarios were based on the maximum potential flows diverted from Mill Creek, based on the hydraulic capacity of the diversion structure. This flow in conjunction with flows from two FIRO flow sources constituted the stress flow scenario models.

A detailed analysis of hydrology consisting of a regional flood frequency analysis can be found in Attachment II.

2.4.1 FIRO Flows

Forecast-Informed Reservoir Operations (FIRO) is a process led by the U.S Army Engineer Research and Development Center (ERDC) and the Center for Western Weather and Water Extremes at UC San Diego Scripps Institute of Oceanography (Scripps) and in cooperation with San Bernardino Valley Water District, with the goal to identify strategies to protect and increase the reliability of the local water supply.

As a result of this process, specific flows have been identified at the Mill Creek basin site with the reliance of independent flows from multiple sources throughout the year in addition to flows captured from the Mill Creek diversion.

Table 1 shows the various sources of expected future FIRO flows for the Mill Creek basins. For purposes of analysis in the study, a total of 131 cfs was used for the FIRO flow evaluations.

During the ladder part of the final hydraulic calculations, the FIRO flows were revised per District discussion with other agencies. The revised flows were reduced by 25 cfs from what was used in this preliminary analysis. These changes will be made during 65-percent design. Nevertheless, the initial FIRO flows can still be used for comparative purposes as the reduction would be consistent for all cases. The original FIRO flows used in the calculations are listed in Table 2-1 below. FIRO flow sources for the Mill Creek basins include ZT Unger, Greenspot, and the Redlands Aqueduct.

Table 2-1. Expected Daily FIRO flows Modeled at Mill Creek Basins

Source	Inlet Location	Flow (cfs)
ZT Unger (FIRO)	South Intake	40
Greenspot (Valley District-FIRO)	South Intake	20
North Intake (Mill Creek)	South Intake	16
South Intake (Mill Creek)	North Intake	30
Redlands Aqueduct (FIRO)	Redland Aqueduct	25
		Total: 131 cfs

FIRO flows were used as part of the evaluation of potential basin improvement alternatives. This flowrate has been used to resemble an expected daily flow for a wet year. Statistically, flows within Mill Creek (46 cfs) has a return period of 5.5 years. This 46 cfs is divided between the South and North Intakes.

2.4.2 Stress Flows

Another flowrate used to model different scenarios within alternatives is referred to as the "Stress" flows. Stress flows include the maximum potential diversion flow that the upper, newly modified, diversion structure can convey. This flow was calculated to be a maximum of 162 cfs. The flow is then divided between the South and Nouth Intakes. An additional 13 cfs is added from the Redlands Aqueduct. For most of the stress scenarios, the flowrate used at the South Intake was 122 cfs, with 40 cfs diverted to the North Intake, and 13 cfs at the Redlands Aqueduct for a total peak flowrate of 175 cfs.

2.5 Hydraulic Analyses

The model used to evaluate the basins was the U.S. Army Corps of Engineer's (USACE) HEC-RAS 6.5 software. Models were set up using flow LiDAR data to help create a 3-dimentional (3D) surface. The District requested a 2-dimentional (2D), unsteady analysis of the basins. Weir and culvert structures were added to the model as 1D structures based off of field survey data.

For scenarios including maintenance of basins, the proposed South Canal provides source flows for both the middle and west models. The hydraulic analysis will confirm and verify the following design requirements:

- The proposed conditions for Alternatives 1, 2, and 3 can adequately route maximum flows without overtopping, while increasing groundwater recharge.
- Provide a numeric value of groundwater recharge versus cost of construction for the District's Feasibility Study.

The hydraulic analysis of the project alternatives are presented in the hydraulic technical memorandum Attachment III (Q3, 2025).

2.5.1 Inflow Hydrographs

Flows were added to the model at three primary locations: South Intake; North Intake; and at the Redlands Aqueduct. These flows were based on the two previously discussed FIRO and Stress flows. Each of these flows were modeled as constant inflows over the duration of the model run. In most cases, the inflow durations lasted for up to 5 days. For example, the FIRO flowrate at the South Intake included a portion of Mill Creek diverted flow (16 cfs), the ZT Unger flow (40 cfs), and the Greenspot flow (20 cfs). The Nouth Intake flow consisted of flows redirected from the South Intake diversion (30 cfs). The Redlands Aqueduct flow was 25 cfs. Each of these flows were entered into the 2D HEC-RAS model as constant flows over several days. Iterations of models were run until the basins reached a constant depth, which occurred when the basins inflow matched the infiltration capacity. This is apparent when the basin all basin depths reach a constant and the outlet flows reach a constant.

2.5.2 Diversion and Outlet Structures

The internal basin structures were modeled as either culverts, open channels (rectangular channels), or combination of weirs and outlet channels. HEC-RAS 2D does not currently calculate reversal of flows accurately. Fortunately, the Mill Creek basins primarily operate as a gravity system. In some cases, such as diversions, downstream tailwater conditions do impact the hydraulics of a diversion when using pipes or closed conduit structures. In these cases, open channel diversions were used to accurately account for potential surcharge or backwater flows. This was the case for some of the diversions from the proposed south canal to the basins. If the basin water surface elevation rose above the water surface in the canal, flows would begin reversing back into the canal.

HEC-RAS models the culvert structures using the FHWA equations, changing the unsteady flows at each time step and calculating the conveyance given the headwater and tailwater conditions. Weirs are calculated in HEC-RAS using the general weir equation, assuming zero velocity at the upstream section. These structures were manipulated for various flow and alternative scenarios to move targeted flows through the system of basins to maximize attenuation of volume within each basin.

In some locations, in-channel diversions are proposed to allow District maintenance staff to direct flows to alternative drainage paths for the purpose of maintenance. To model these facilities, an 'open channel' structure was used to allow the model to calculate reversal of flow if necessary. In these conditions, an iterative approach was used to sized each structure until the desired result was achieved.



Typical Basin Inlet & Outlet Structures

2.5.3 Infiltration Parameters

A geotechnical investigation was performed to determine the existing basin infiltration parameters. As part of the analysis, a range of percolation values were presented for each of the basins. Some hurdles of this testing involved the environmental permitting. This resulted in providing estimates for areas not tested. For a detailed geotechnical technical memorandum, see Addendum 2. Model scenarios run with these infiltration rates are referred to as "Kleinfelder" infiltration rates.

The District, working with other ARPT projects has developed a range of infiltration parameters for "plugged" conditions. This condition assumed a non-maintained condition that reduces the potential infiltration of each basin. This "clogged" condition was the basis for running proposed alternatives to identify a "worst case scenario" with respect to groundwater recharge yield for the two given inflow parameters. A full list of each basin's percolation rate for the "clogged" scenario can be found in Addendum 2. Model scenarios run with these infiltration rates are referred to as "District" infiltration rates.

The rating curves for each basin were modeled in HEC-RAS using outflow rating curves. Calculations were made for each basin based on depth, surface area, and percolation rate. The resulting infiltration outflow hydrographs consisted of a depth vs outflow in cubic feet per second. These rating curves can be seen in Addendum II.

2.6 Groundwater Modeling Analysis

For any recharge site, increasing the volume of recharge raises the level of groundwater, at a minimum, in the areas surrounding the individual (and system) basins. Mounding is the local or regional raising of the groundwater table and can cause exfiltration within adjacent basins that reside within a close proximity.

Project team members Geoscience, Inc. prepared groundwater models based on select scenarios to evaluate the proposed basin grading plan under varying flow conditions. The results of the surface models (HEC-RAS 2D) were processed and delivered to the team hydrogeologists as input for their groundwater models. Analysis. HEC-RAS results were processed and prepared in GIS shapefiles and Excel spreadsheets for basin surface areas and infiltration results. This data is added to the groundwater models to identify impact on the Bunker Hill Basin, as well as to identify proposed basin grading characteristics in the way of mounding. The hydrogeology models can provide guidance as to basin proximity. If basins are too close together, exfiltration from one basin may inadvertently provide a source of inflow into another basin.

Three scenarios were given to Geoscience for modeling purposes: Existing Conditions (FIRO flows), Alternative 1 (FIRO flows), and Alternative 3 (Stress flows). The existing conditions model was used as a basis for comparing the two other alternatives. Since Alternative 1 (FIRO) contained some 'dry' basins in the hydraulic model results. As a result, no further FIRO conditions were reviewed, since both Alternative 2 and 3 contained additional basins that would produce similar recharge results as Alternative 1 (FIRO). Alternative 3 (Stress) was modeled only as it provided the most recharge yield and would be used as a guide for Alternative 1 and 2 (Stress) conditions. The results of the Hydrogeology Study for these basins can be found in Attachment IV (Geoscience, 2025).

The initial model results showed that groundwater reaches the surface for a large portion of the existing basins under FIRO flows, which does not support actual operating conditions. Available well data exists in the western portion of the Mill Creek basins, near basins 45 and 53, but not within the central or upper reaches of the basins. Assumptions for existing groundwater and depth to bedrock revealed conditions that may not be accurate for the site. Although this study provides valuable information for the lower reaches for the basins (between basins 36-53), additional field data is needed for the basins upstream to validate the model.

Conclusion: It is recommended that additional wells be constructed to monitor groundwater elevations through at least one wet season. This new data can be correlated with the existing well data to calibrate the groundwater models and provide a more accurate depiction of the potential mounding. As such, current groundwater models are deemed inconclusive.

2.7 Alternative Cost Estimates

A preliminary construction cost estimate was developed for each alternative. This estimate is part of a larger study to evaluate current and potential recharge yield for various improvements and to identify a cost-benefit relationship to support the San Bernardino Valley Water Conservation District (District) with information to garner a recommendation.

A detailed preliminary cost analysis was conducted and included in a technical memorandum (Addendum V, Q3, 2025). Table 2-2 show the preliminary Unit Pricing and contingencies used for the study. Environmental mitigation cost estimates were also included and based on each project per coordination with the District. A total construction contingency of 65-percent was added to each project.

Table 2-2. Preliminary Unit Prices

ITEM	UNIT	UNIT COST
MOBILIZATION	LS	$2,000,000.00-$2,250,000
DEMOLITION	LS	$0.00
CLEAR AND GRUB	AC	$1,000.00
EXCAVATION (BASINS & CANAL)	CY	$5.00
BACKFILL (BASINS & CANAL) Non-Compacted	CY	$12.00
STRUCTURE EXCAVATION	CY	$55.00
EXPORT	CY	$15.00
SPILLWAY (Basins 1, 2A-2D only)	EA	$50,000.00
MILL CREEK DIVERSION STRUCTURE	EA	$2,000,000.00
12' x 6' RCB (incl wingwalls and street repair)	CY	$4,000.00
OULET STRUCTURE	EA	$50,000.00
INLET STRUCTURE (Inc. Weirs)	EA	$50,000.00
FLOW METER (INCL STRUCTURE)	EA	$100,000.00
¼ TON RIPRAP (NON-CONCRETED)	CY	$200.00
DIVERSION GATES WITHIN INNERTIES	EA	$50,000.00
EROSION CONTROL / SWPPP	LS	$50,000.00
UTILITIES	LS	$500,000.00
SOUTH CANAL DIVERSION STRUCTURES	EA	$50,000.00
SCADA	LS	$500,000.00

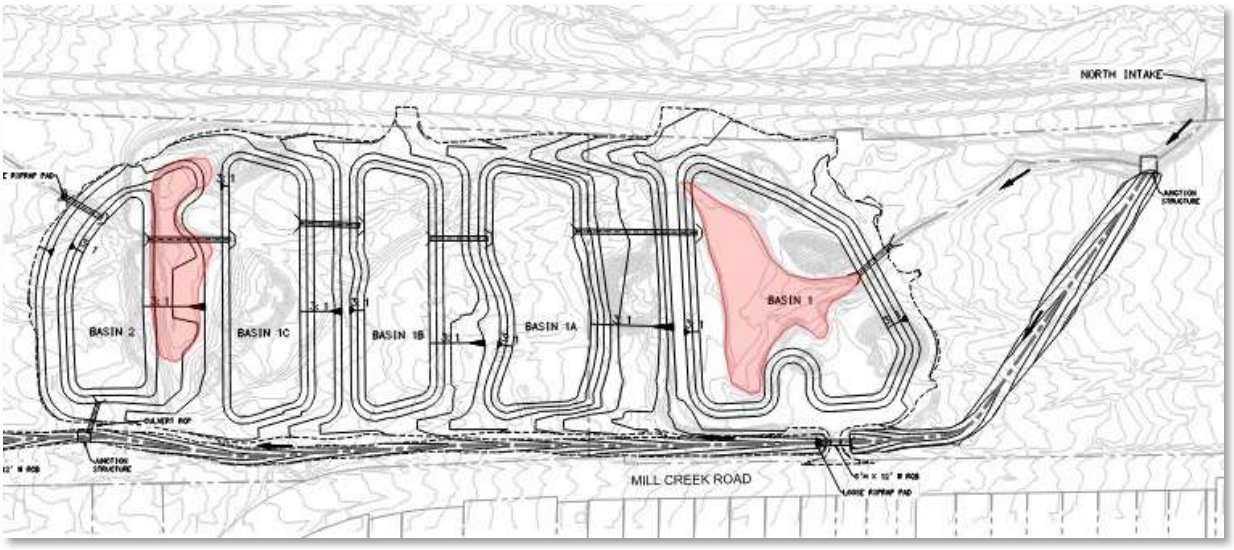
3 DESIGN ALTERNATIVES

3.1 Proposed Improvements

Each alternative includes proposed improvement to multiple basins and the addition of a bypass channel (South Canal) to allow for flows to be redirected if needed during maintenance operations. Figure 3-1 shows the proposed improvements to the existing system. The "red" shaded areas show the existing Basins 1 and 2 and the proposed diversion canal can be seen traversing along the south edge of the proposed new basins.

Recent modification to the South Intake structure has allowed for a slight increase in flow capacity to enter the basins (Basin 1). Improvements to the structural also provide protection against large boulders and debris from accumulating within the main intake chute.

One major improvement with these alternatives, is an expansion of the existing desilting basins 1 and 2. Proposed improvements show these two basins being expanded to five new basins (Basin 1, 1A, 1B, 1C, and 2). The proposed south canal runs along the southern edge of all five basins and provide the ability to take Basins 1, 1A, 1B and 1C offline for maintenance, utilizing Basin 2 and possibly Basin 3 for desilting. The graphic below shows the proposed improvements of the desilting basins (existing basins in red).



Proposed Desilting Basins Below South Intake

All basin improvements utilize a maximum graded slope, both upstream and downstream, of 3:1 for slope stability. Current conditions at some basins reveal downstream slopes of 1.5:1 or steeper. These basin slopes were not considered in these concept drawing but will be modified in the 65-percent plans. In most cases, they will be increased to 3:1 if the maximum basin water surface elevations (WSE) extend higher than the existing downstream grade.

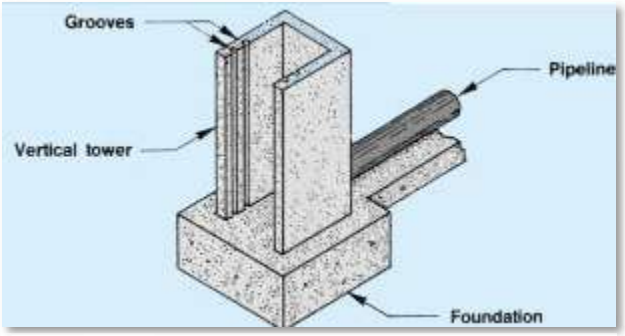
Based on findings from the Geotechnical technical memorandum (Attachment I), some proposed basins may require modifications to the maximum allowable WSE to ensure slope stability. These modifications will be provided in the 65-percent design and will not greatly affect the overall alternative recharge yields.

The main outlet for each alternative is Basin 52-53, which is a proposed improvement to existing Basin 53. This differs from the existing basin system, which has two outlets, Basin 53 and Basin 22.

Internal Structures

Each basin inlet and outlet structure will be improved to account for the potential of additional flows and provide better operation control within the basin system. The main control is within the outlet structures. A "Monk-Type" structure is proposed to be used that allows for weir boards to be manually placed within the structure to control the basin water surface elevation.

Some outlet weir structures will be constructed with a reinforced concrete (RC) pipe and some with a rectangular channel.



Typical Outlet ("Monk" Structure)

Vertical rising sluice gates may also be used within the south canal that operate with either manually or automatically. Exact locations may be identified during the 65-percent design.

Diversion Structures

For Alternatives 1, 2, and 3, each contain a proposed six diversion structures. These structures are in-line diversions meant to control flow patterns within the system of basins for the purpose of optimizing flows within the system and for maintenance operations. These types of structures are proposed to be sluice gate-type structures that can be automated. Exact types of diversions will be identified in the 65-percent design plans.

3.2 Alternative 1

Alternative 1 includes improvements to 29 existing basins and the addition of three new basins. Many of the existing smaller basins are proposed to be combined and regraded into larger basins. Figure 3-2 shows the proposed improvements for Alternative 1. This figure also shows the locations for diversion structures, the South Canal and its proposed connections to various basins, and improved basin outlet structures. It is anticipated that 45 new inlet structures, eight sluice gates, and six diversion gates will be constructed as part of this project.

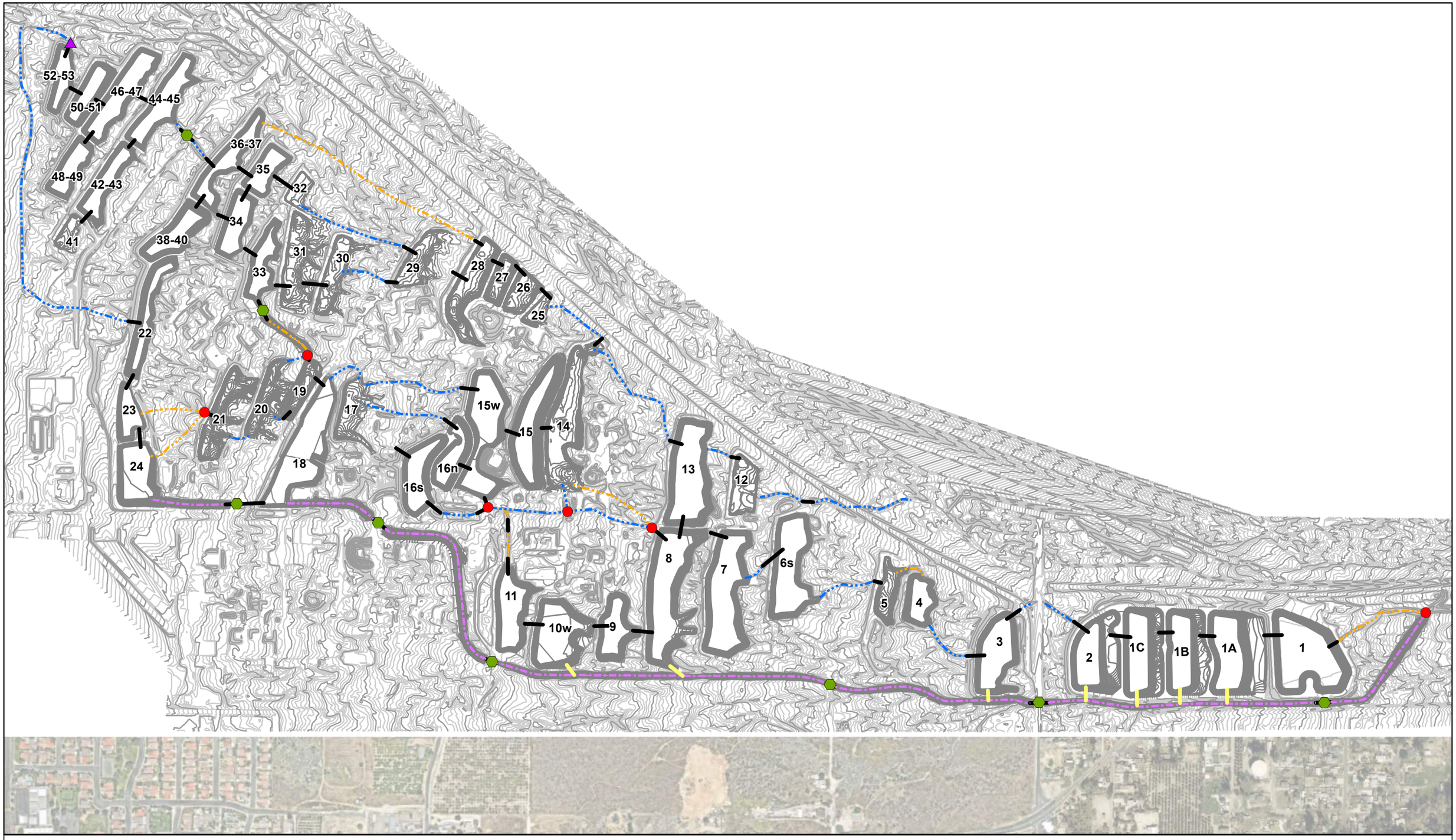
3.2.1 Alternative 1 Cost Estimate

A rough order magnitude preliminary engineering construction cost estimate was developed for Alternative 1. For specific cost details refer to the cost technical memorandum (Attachment 5).

Table 3-1. Preliminary Project Construction Cost for Alternative 1

Item	Cost
Project Construction	$47,300,000
Contingency (55%)	$26,000,000
Habitat Mitigation Measures*	$500,000
Total Project Cost	**$73,800,000**

The habitat mitigation measures cost was coordinated with the District. During the final design process, this number is subject to change.



Alternative 1

Figure 3-1

3.3 Alternative 2

Alternative 2 includes all the improvements of Alternative 1 in addition to three new basins, and two additional modifications to existing basins.

Figure 3-3 shows the proposed improvements for Alternative 2. This figure also shows the locations for diversion structures, the South Canal and its proposed connections to various basins, and improved basin outlet structures. It is anticipated that 49 new inlet structures, eight sluice gates, and six diversion gates will be constructed as part of this project.

3.3.1 Alternative 2 Cost Estimate

A rough order magnitude preliminary engineering construction cost estimate was developed for Alternative 2. For specific cost details refer to the cost technical memorandum (Attachment 5).

Table 3-2. Preliminary Project Construction Cost for Alternative 2

Item	Cost
Project Construction	$50,100,000
Contingency (55%)	$27,600,000
Habitat Mitigation Measures*	$500,000
Total Project Cost	**$78,200,000**

The habitat mitigation measures cost was coordinated with the District. During the final design process, this number is subject to change.

3.4 Alternative 3

Alternative 3 includes all the improvements of Alternative 2 in addition to four new basins. Figure 3-4 shows the proposed improvements for Alternative 3. This figure also shows the locations for diversion structures, the South Canal and its proposed connections to various basins, and improved basin outlet structures. It is anticipated that 56 new inlet structures, eight sluice gates, and six diversion gates will be constructed as part of this project.

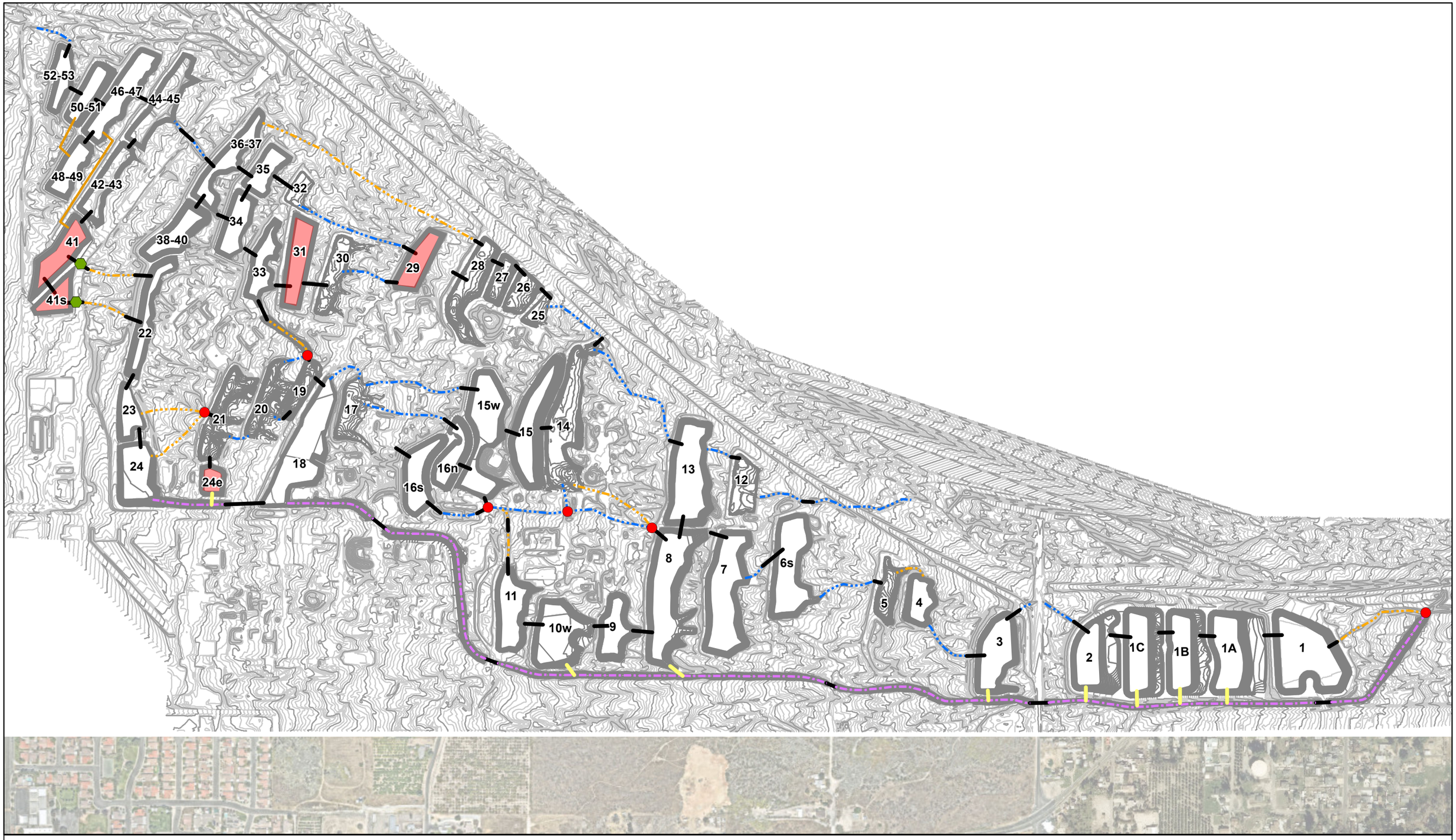
3.4.1 Alternative 3 Cost Estimate

A rough order magnitude preliminary engineering construction cost estimate was developed for Alternative 3. For specific cost details refer to the cost technical memorandum (Attachment 5).

Table 3-2. Preliminary Project Construction Cost for Alternative 3

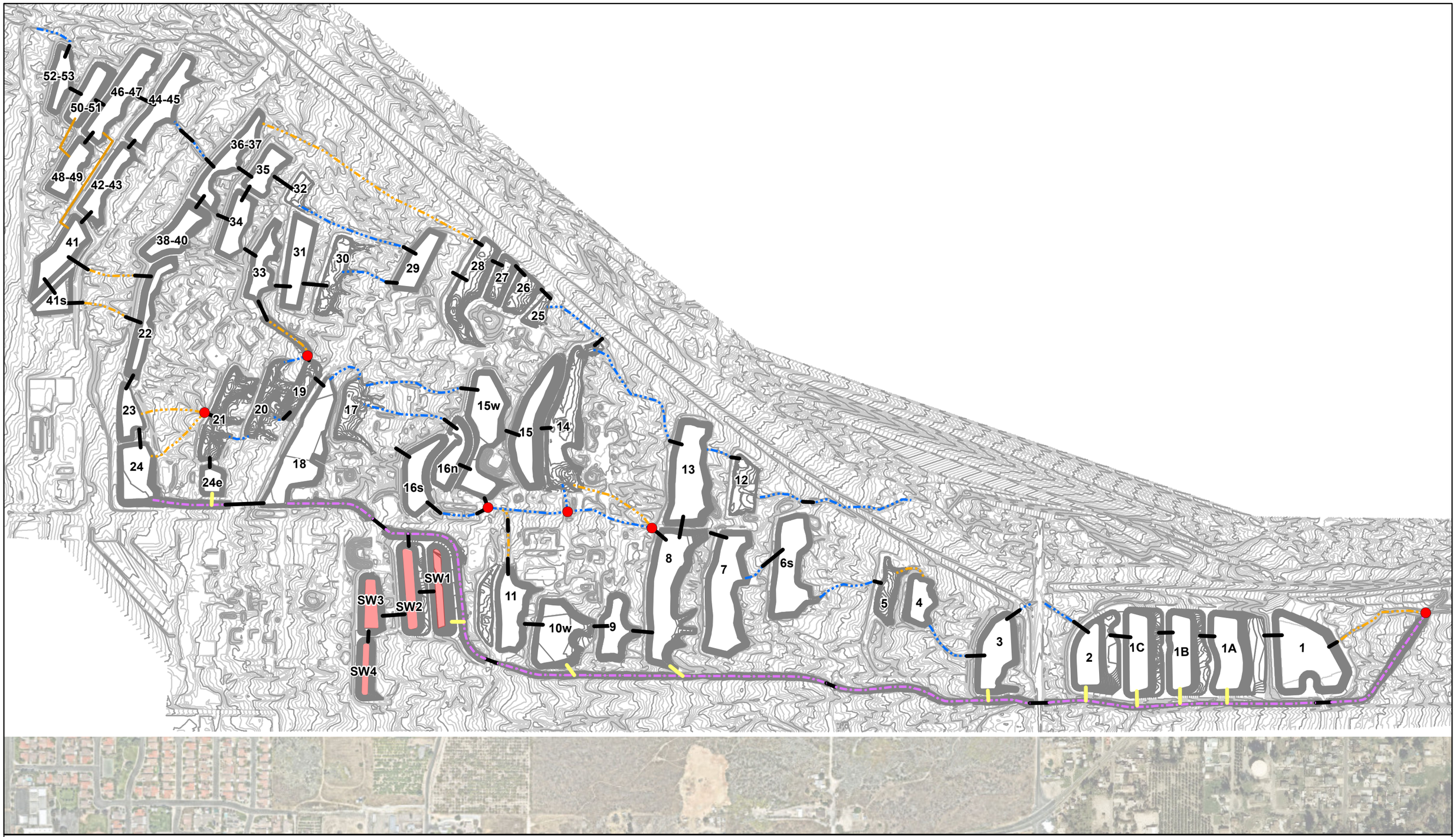
Item	Cost
Project Construction	$53,900,000
Contingency (55%)	$29,600,000
Habitat Mitigation Measures*	$500,000
Total Project Cost	**$84,000,000**

The habitat mitigation measures cost was coordinated with the District. During the final design process, this number is subject to change.



Alternative 2

Figure 3-2



Alternative 3

Figure 3-3

EXHIBITS

Concept Plans

EXHIBIT 1

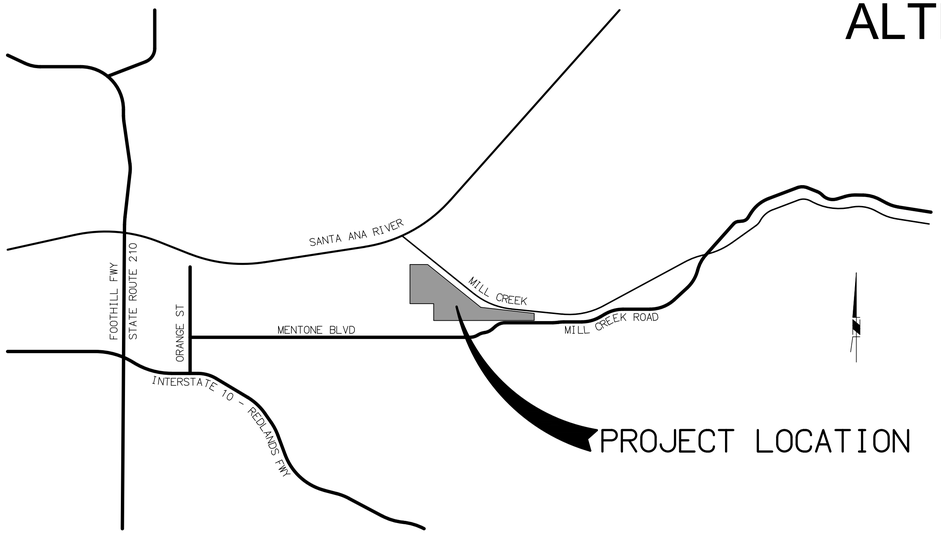
Alternative 1: Concept Plans

SAN BERNARDINO VALLEY WATER CONSERVATION DISTRICT

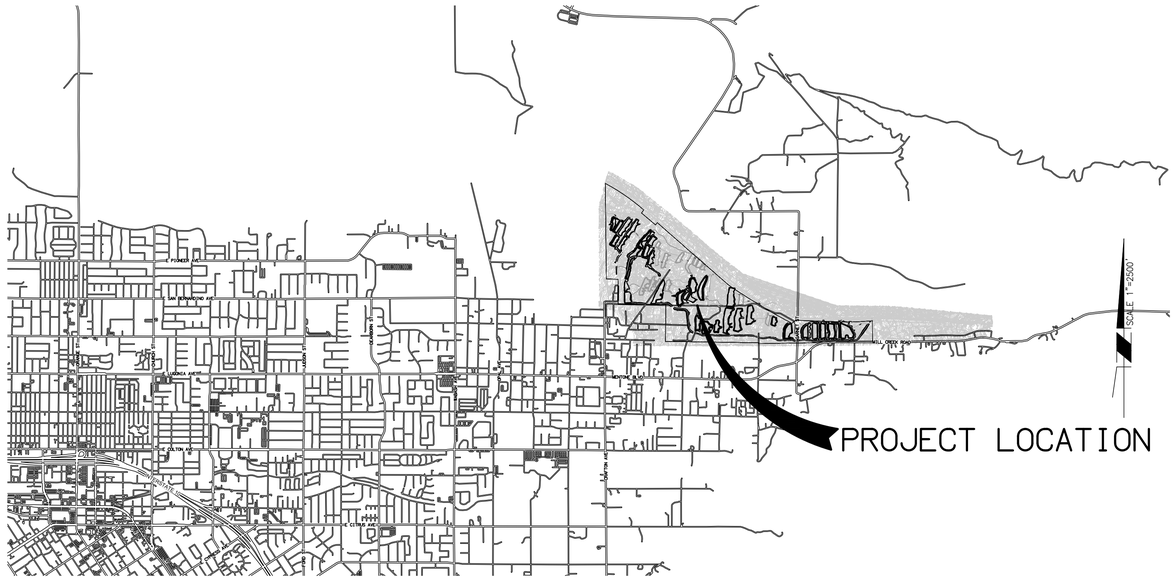
ACTIVE RECHARGE RECHARGE PROJECT

MILL CREEK

ALTERNATIVE 1



VICINITY MAP
NTS



LOCATON MAP

INDEX

SHEET NO.	SHEET TITLE
01	TITLE SHEET
02	INDEX MAP
03	BASIN GRADING 41 TO 53
04	BASING GRADING 25 TO 40
05	BASIN GRADING 14 TO 24E
06	BASING GRADING 6S TO 13 AND SW1 TO SW4
07	BASIN GRADING 3 TO 5
08	BASIN GRADING 1 TO 2
09	STRUCTURAL DETAILS
10	MISCELLANEOUS DETAILS

811 Know what's below. Call before you dig. DIGALERT Underground Service Alert

Section 4216/4216.9 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert 811 two working days before you dig

BENCHMARK:

NO.	DATE	REVISIONS	APPROVED	DATE
1				
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4				
5				
6				
7				

Prepared under the supervisor of: _____

DRAWN BY: _____

DESIGNED BY: _____

CHECKED BY: _____

San Bernardino Valley Water Conservation District
Helping Nature Store Our Water

SAN BERNARDINO VALLEY WATER CONVERSATION DISTRICT

PROJECT NO. SBVWCD - 0000

ATRP MILL CREEK
ALTERNATIVE 1 PRELIMINARY DESIGN
TITLE SHEET

SHEET 01
OF 08

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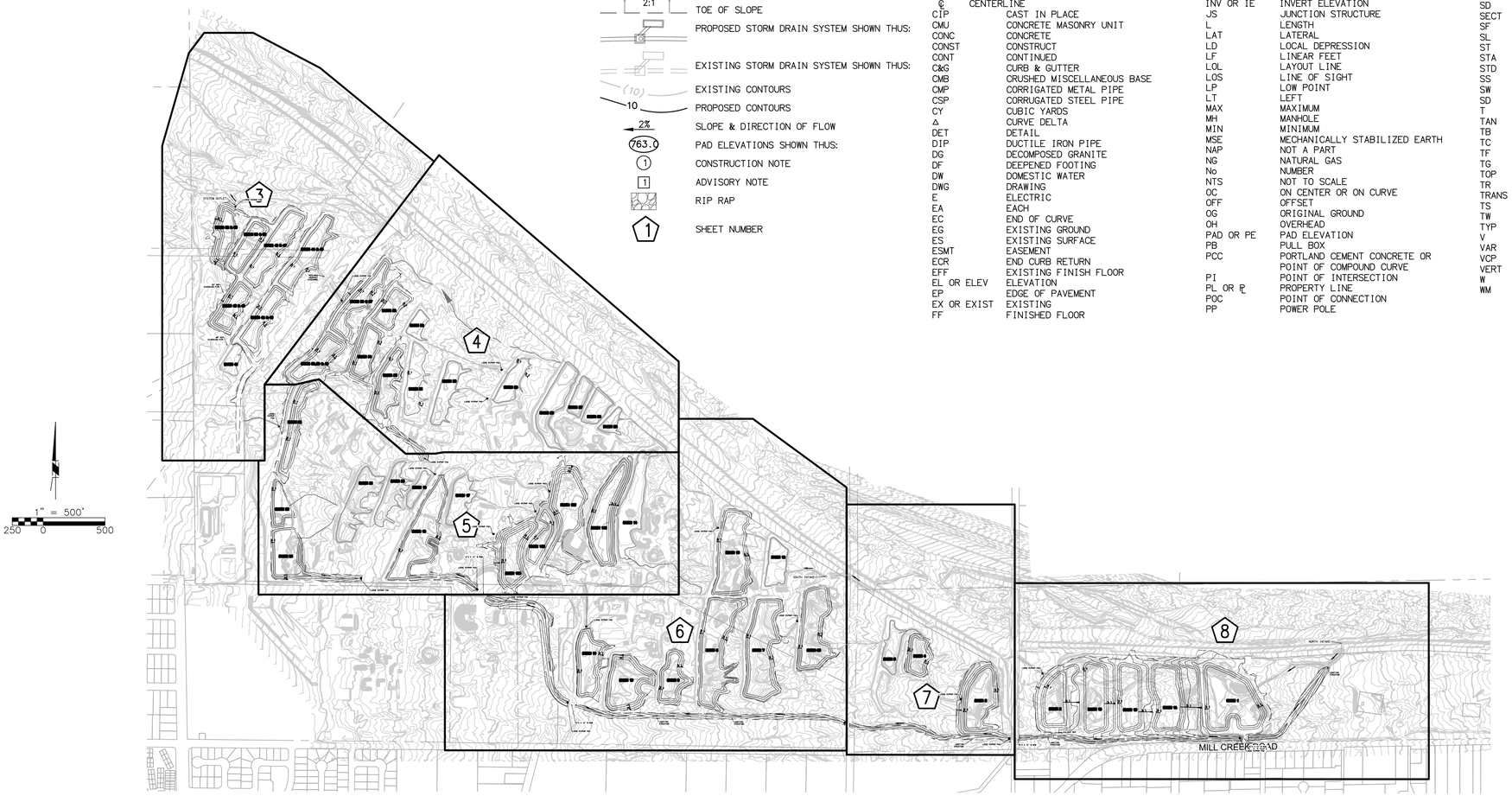
LEGEND

- TRACT BOUNDARY
- RIGHT-OF-WAY LINE
- CENTERLINE
- CMU RETAINING WALL
- EASEMENT
- DAYLIGHT LINE
- CUT FILL LINE
- TOP OF SLOPE
- TOE OF SLOPE (2:1)
- PROPOSED STORM DRAIN SYSTEM SHOWN THUS:
- EXISTING STORM DRAIN SYSTEM SHOWN THUS:
- EXISTING CONTOURS (10)
- PROPOSED CONTOURS 10
- SLOPE & DIRECTION OF FLOW 2%
- PAD ELEVATIONS SHOWN THUS: 763.0
- CONSTRUCTION NOTE (1)
- ADVISORY NOTE [1]
- RIP RAP
- SHEET NUMBER (1)

AB	AGGREGATE BASE
AC	ASPHALTIC CONCRETE
ACP	ASBESTOS CEMENT PIPE
∆	ANGLE POINT
BC	BEGIN CURVE
BCR	BEGIN CURB RETURN
BDRY	BOUNDARY
BOP	BOTTOM OF PIPE
BVC	BEGIN VERTICAL CURVE
BV	BUTTERFLY VALVE
BW	BACK OF WALK, BOTTOM OF WALL
CAB	CRUSHED AGGREGATE BASE
CB	CATCH BASIN
℄	CENTERLINE
CIP	CAST IN PLACE
CMU	CONCRETE MASONRY UNIT
CONC	CONCRETE
CONST	CONSTRUCT
CONT	CONTINUED
C&G	CURB & GUTTER
CMB	CRUSHED MISCELLANEOUS BASE
CMP	CORRIGATED METAL PIPE
CSP	CORRUGATED STEEL PIPE
CY	CUBIC YARDS
∆	CURVE DELTA
DET	DETAIL
DIP	DUCTILE IRON PIPE
DG	DECOMPOSED GRANITE
DF	DEEPENED FOOTING
DW	DOMESTIC WATER
DWG	DRAWING
E	ELECTRIC
EA	EACH
EC	END OF CURVE
EG	EXISTING GROUND
ES	EXISTING SURFACE
ESMT	EASEMENT
ECR	END CURB RETURN
EFF	EXISTING FINISH FLOOR
EL OR ELEV	ELEVATION
EP	EDGE OF PAVEMENT
EX OR EXIST	EXISTING
FF	FINISHED FLOOR

ABBREVIATIONS

EVC	END VERTICAL CURVE
FG	FINISHED GRADE
FG	FIRE HYDRANT
FL	FLOWLINE ELEVATION
FS	FINISH SURFACE
G	GAS
GB	GRADE BREAK
GF	GARAGE FLOOR
GV	GATE VALVE
HGL	HYDRAULIC GRADE LINE
HORIZ	HORIZONTAL
HP	HIGH POINT
INT	INTERSECTION
INV OR IE	INVERT ELEVATION
JS	JUNCTION STRUCTURE
L	LENGTH
LAT	LATERAL
LD	LOCAL DEPRESSION
LF	LINEAR FEET
LOL	LAYOUT LINE
LOS	LINE OF SIGHT
LP	LOW POINT
LT	LEFT
MAX	MAXIMUM
MH	MANHOLE
MIN	MINIMUM
MSE	MECHANICALLY STABILIZED EARTH
NAP	NOT A PART
NG	NATURAL GAS
No	NUMBER
NTS	NOT TO SCALE
OC	ON CENTER OR ON CURVE
OFF	OFFSET
OG	ORIGINAL GROUND
OH	OVERHEAD
PAD OR PE	PAD ELEVATION
PB	PULL BOX
PCC	PORTLAND CEMENT CONCRETE OR POINT OF COMPOUND CURVE
PI	POINT OF INTERSECTION
PL OR ℙ	PROPERTY LINE
POC	POINT OF CONNECTION
PP	POWER POLE
PRC	POINT OF REVERSE CURVE
PROP OR PR	PROPOSED
PROF	PROFILE
PT	POINT OF TANGENT OR POINT
PVC	POLYVINYL CLORIDE
PVI	POINT OF VERTICAL INTERSECTION
R	RADIUS
RCP	REINFORCED CONCRETE PIPE
RET	RETAINING
R/W OR ROW	RIGHT OF WAY
RT	RIGHT
RW	RECLAIMED OR RECYCLED WATER
SCE	SOUTHERN CALIFORNIA EDISON
SD	STORM DRAIN
SECT	SECTION
SF	SQUARE FEET
SL	STREET LIGHT
ST	STREET
STA	STATION
STD	STANDARD
SS	SANITARY SEWER
SW	SIDEWALK OR STEM WALL
SD	STORM DRAIN
T	TELEPHONE
TAN	TANGENT LENGTH
TB	TOP OF BERM
TC	TOP OF CURB
TF	TOP OF FOOTING
TG	TOP OF GRATE
TOP	TOP OF PIPE
TR	TOP OF RETAINING
TRANS	TRANSITION
TS	TOP OF SLOPE OR TRAFFIC SIGNAL
TW	TOP OF WALL
TYP	TYPICAL
V	VALVE
VAR	VARIES
VCP	VITRIFIED CLAY PIPE
VERT	VERTICAL
W	WATER OR WITH
WM	WATER METER



SHEET INDEX MAP

1" = 100'
50 0 100

811 Know what's below. Call before you dig. DIG ALERT Underground Service Alert

Section 4216/4216.9 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert 811 two working days before you dig

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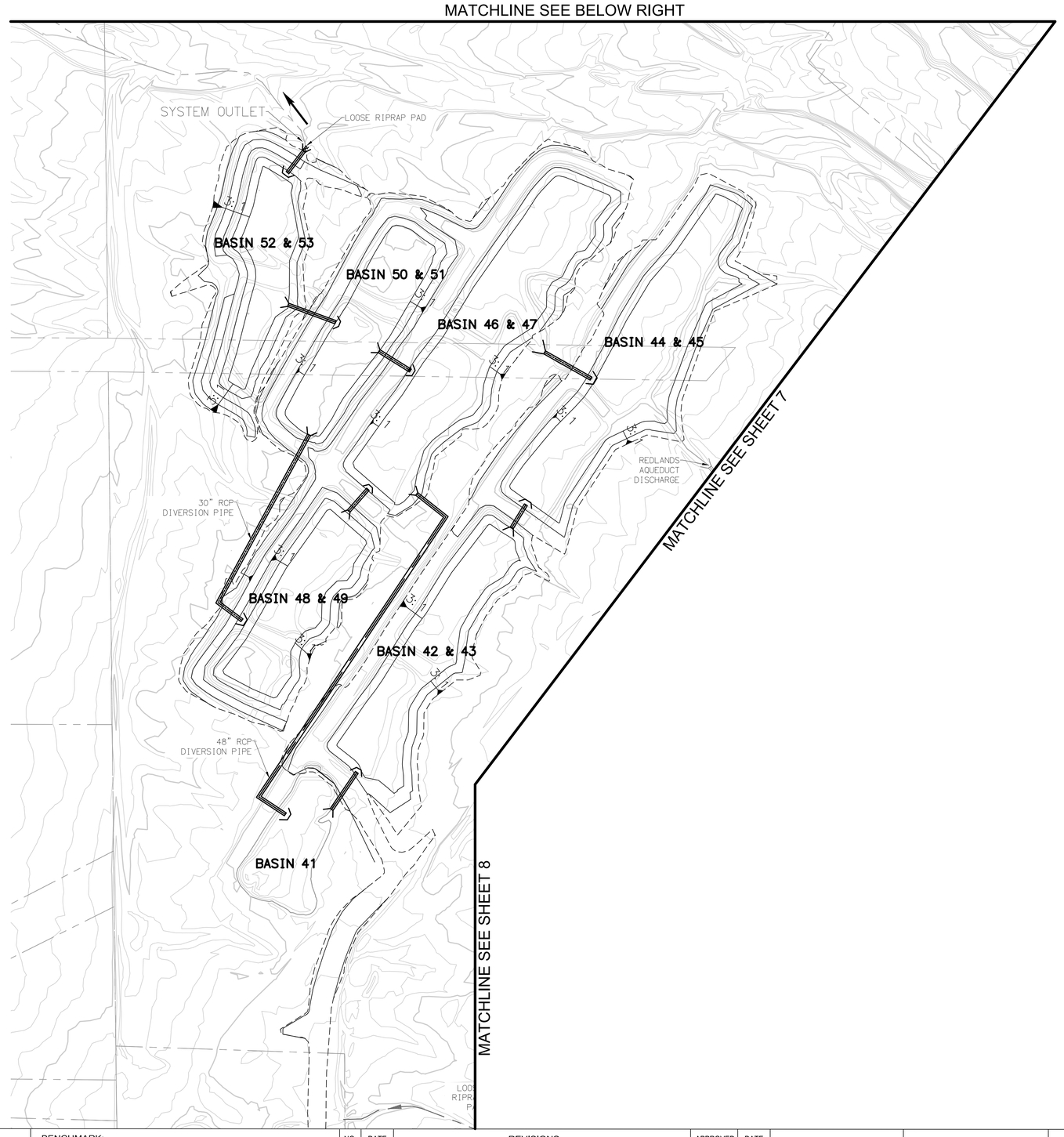
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PROJECT NO. SBVWCD - 0000

ATRP MILL CREEK
ALTERNATIVE 1 PRELIMINARY DESIGN
INDEX MAP

SHEET 02
OF 08

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ATRP MILL CREEK
ALTERNATIVE 1 PRELIMINARY DESIGN
BASIN GRADING 41 TO 53

PROJECT NO.
SBVWCD -
0000

SHEET 03
OF 08

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MATCHLINE SEE BELOW LEFT

OUTLET STRUCTURE WITH WEIR

OUTLET STRUCTURE

EXISTING INTER-BASIN UNLINED WATERCOURSE

PROPOSED INTER-BASIN UNLINED WATERCOURSE

PROPOSED SOUTH CANAL

MAJOR EXISTING FLOW LINE

1" = 100'
50 0 100

MATCHLINE SEE SHEET 6

LOOSE RIPRAP PAD

BASIN 36 & 37

BASIN 35

BASIN 32

BASIN 34

BASIN 38,39 & 40

BASIN 33

BASIN 31

BASIN 30

LOOSE RIPRAP PAD

BASIN 29

LOOSE RIPRAP PAD

BASIN 28

BASIN 27

BASIN 26

BASIN 25

MATCHLINE SEE SHEET 8

MATCHLINE SEE SHEET 9

MATCHLINE SEE SHEET 8

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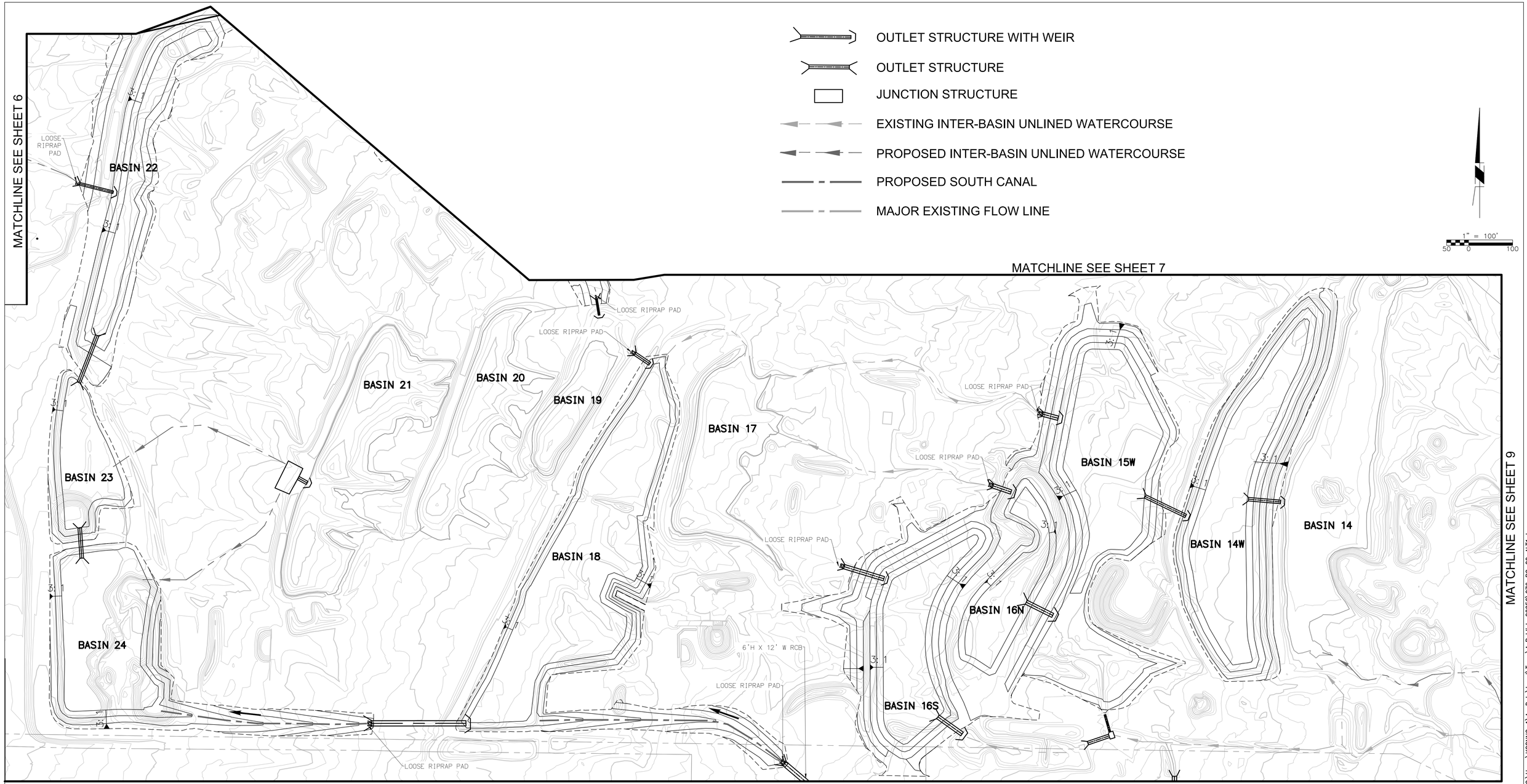
PROJECT NO. SBVWCD - 0000

ATRP MILL CREEK
ALTERNATIVE 1 PRELIMINARY DESIGN
BASIN GRADING 25 TO 40

SHEET 04

OF 08

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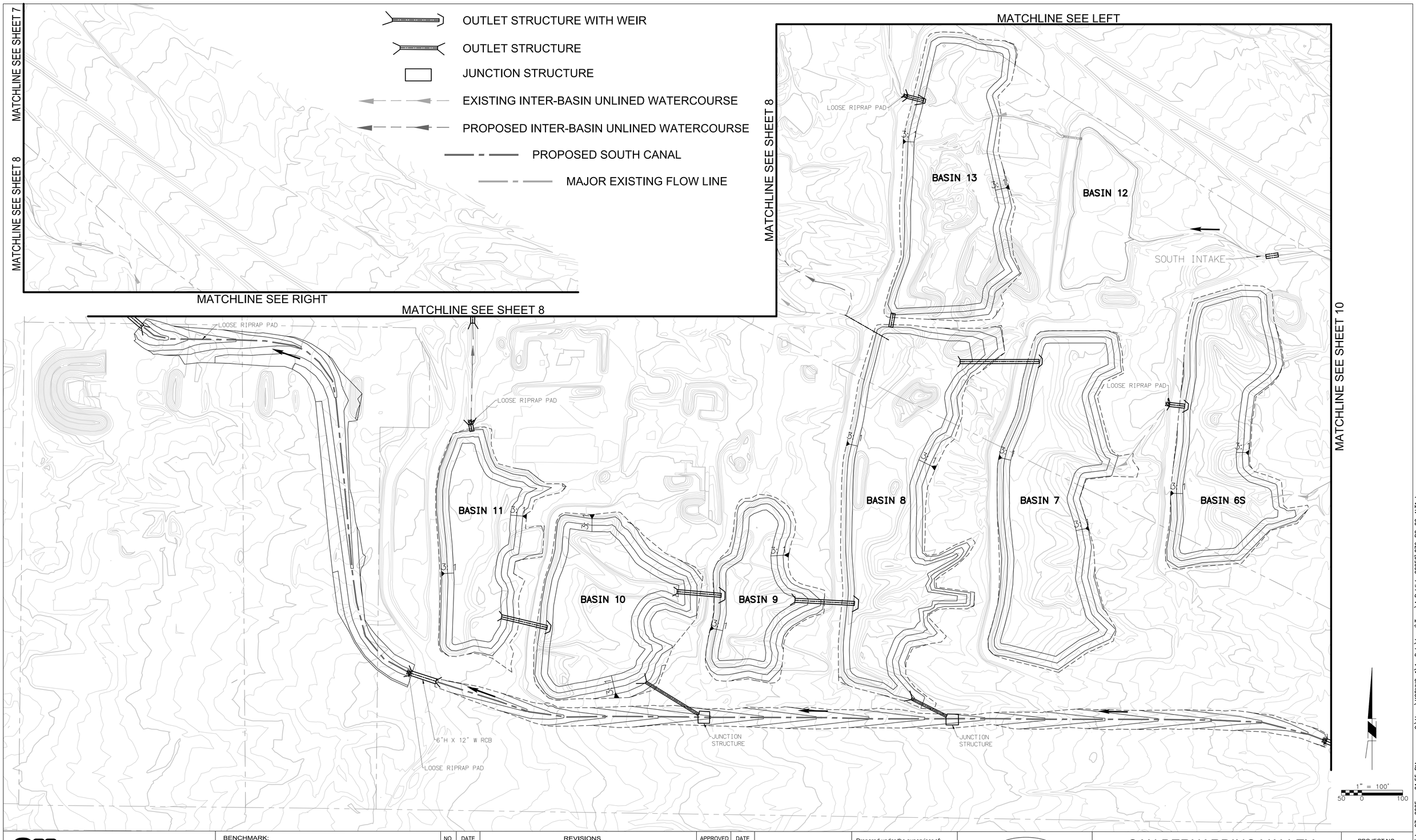
ATRP MILL CREEK
ALTERNATIVE 1 PRELIMINARY DESIGN
BASIN GRADING 14 TO 24E

PROJECT NO. SBVWCD - 0000

SHEET 05

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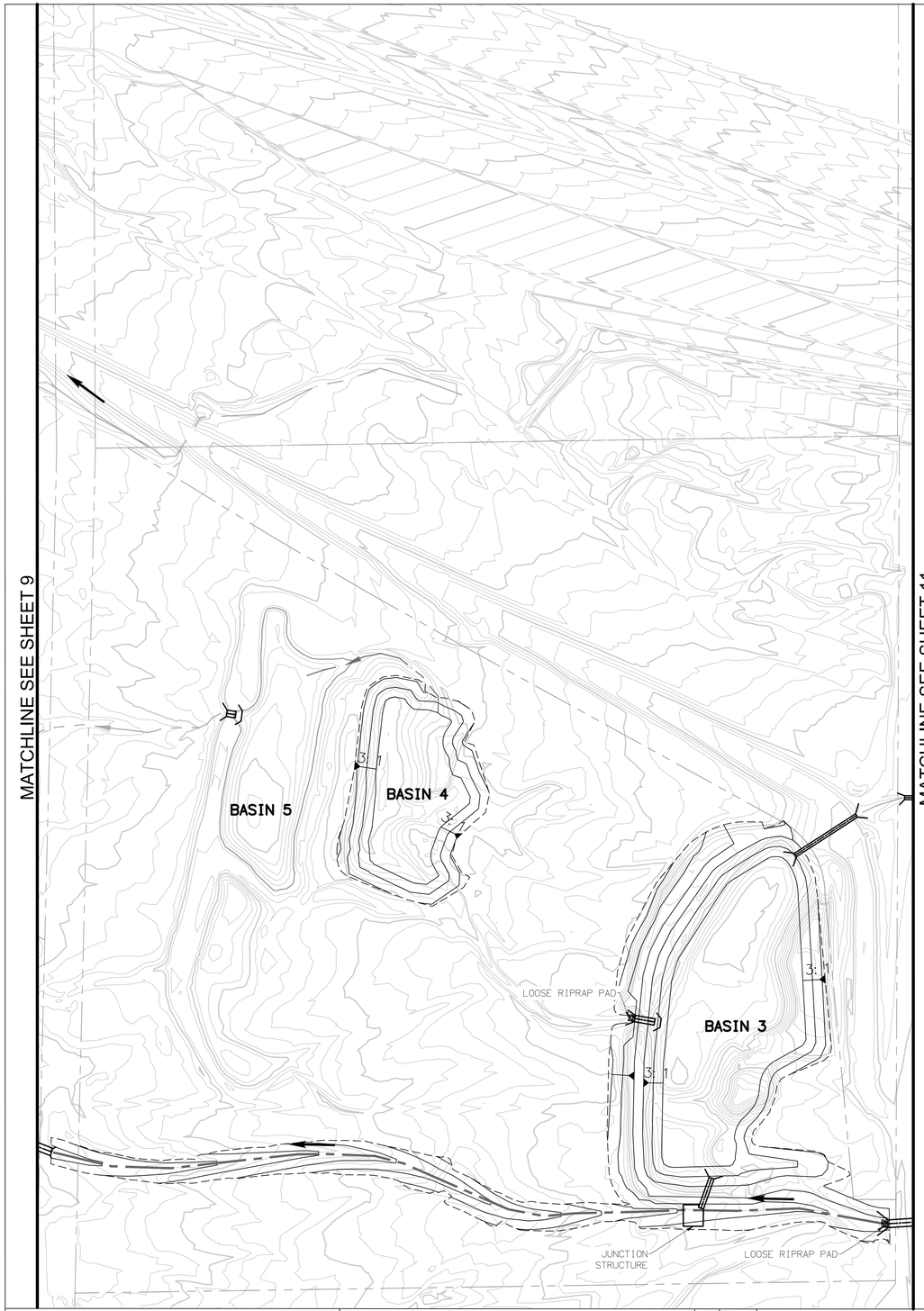
ATRP MILL CREEK
ALTERNATIVE 1 PRELIMINARY DESIGN
BASIN GRADING 6S TO 14 AND SW1 TO SW4

PROJECT NO. SBVWCD - 0000

SHEET 06

OF 08

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OUTLET STRUCTURE WITH WEIR

OUTLET STRUCTURE

JUNCTION STRUCTURE

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PROPOSED INTER-BASIN UNLINED WATERCOURSE

PROPOSED SOUTH CANAL

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ATRP MILL CREEK
ALTERNATIVE 1 PRELIMINARY DESIGN
BASIN GRADING 3 TO 5

SHEET 07
OF 08

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MATCHLINE SEE SHEET 10

MATCHLINE BELOW 12



OUTLET STRUCTURE WITH WEIR

OUTLET STRUCTURE

JUNCTION STRUCTURE

EXISTING INTER-BASIN UNLINED WATERCOURSE

PROPOSED INTER-BASIN UNLINED WATERCOURSE

PROPOSED SOUTH CANAL

MAJOR EXISTING FLOW LINE

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ATRP MILL CREEK
ALTERNATIVE 1 PRELIMINARY DESIGN
BASIN GRADING 1 TO 2

SHEET 08

OF 08

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EXHIBIT 2

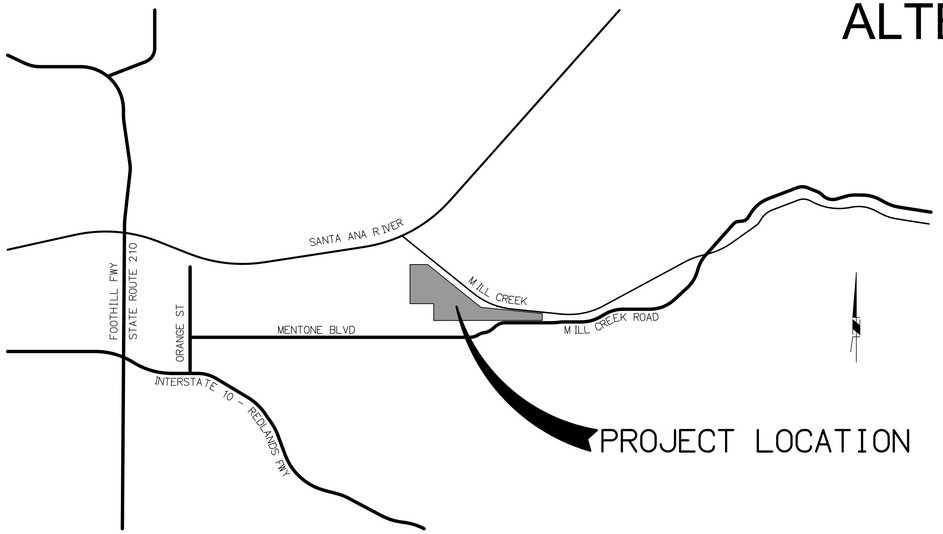
Alternative 2: Concept Plans

SAN BERNARDINO VALLEY WATER CONSERVATION DISTRICT

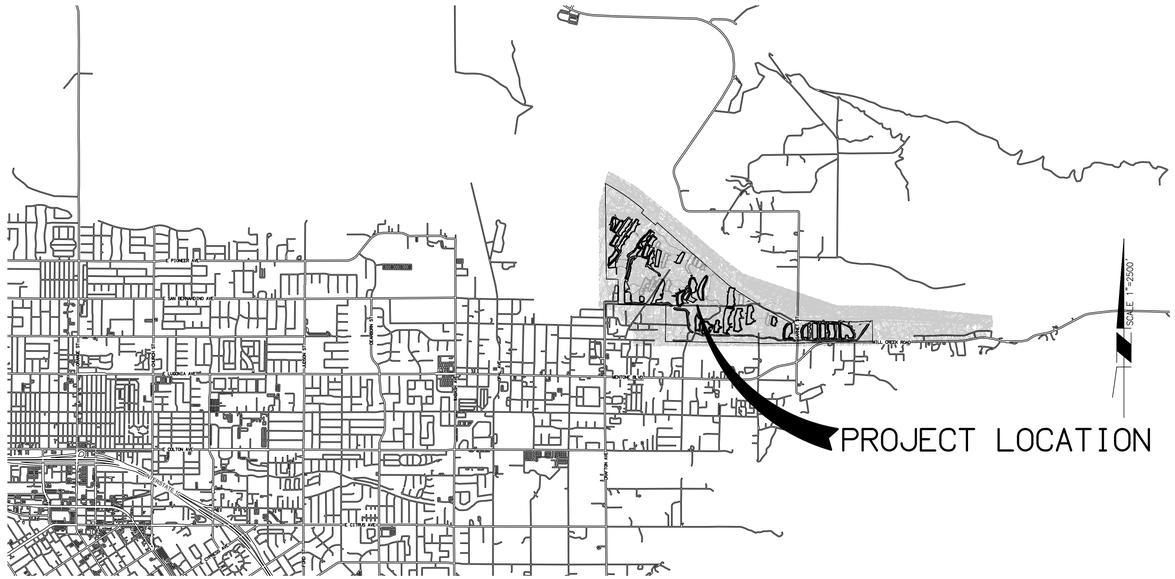
ACTIVE RECHARGE RECHARGE PROJECT

MILL CREEK

ALTERNATIVE 2



VICINITY MAP
NTS



LOCATON MAP

INDEX

SHEET NO.	SHEET TITLE
01	TITLE SHEET
02	INDEX MAP
03	BASIN GRADING 41 TO 53
04	BASING GRADING 25 TO 40
05	BASIN GRADING 14 TO 24E
06	BASING GRADING 6S TO 13 AND SW1 TO SW4
07	BASIN GRADING 3 TO 5
08	BASIN GRADING 1 TO 2
09	STRUCTURAL DETAILS
10	MISCELLANEOUS DETAILS

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SAN BERNARDINO VALLEY WATER CONVERSATION DISTRICT

ATRP MILL CREEK
ALTERNATIVE 2 PRELIMINARY DESIGN
TITLE SHEET

PROJECT NO. SBVWCD - 0000

SHEET 01

OF 08

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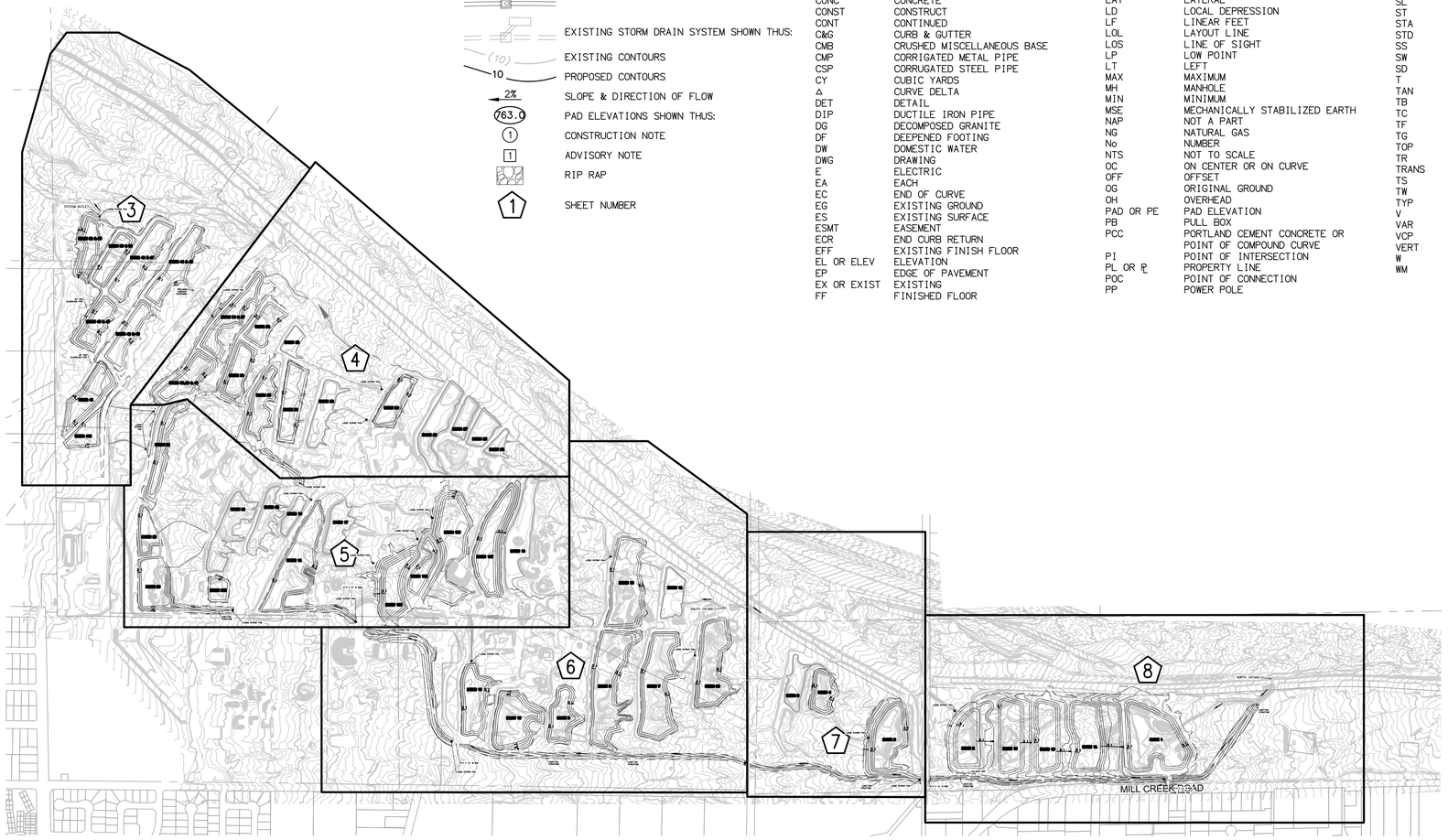
LEGEND

- TRACT BOUNDARY
- RIGHT-OF-WAY LINE
- CENTERLINE
- CMU RETAINING WALL
- EASEMENT
- DAYLIGHT LINE
- CUT FILL LINE (C / F)
- TOP OF SLOPE
- TOE OF SLOPE (2:1)
- PROPOSED STORM DRAIN SYSTEM SHOWN THUS:
- EXISTING STORM DRAIN SYSTEM SHOWN THUS:
- EXISTING CONTOURS (10)
- PROPOSED CONTOURS 10
- SLOPE & DIRECTION OF FLOW 2%
- PAD ELEVATIONS SHOWN THUS: 763.0
- CONSTRUCTION NOTE (1)
- ADVISORY NOTE [1]
- RIP RAP
- SHEET NUMBER (1)

AB	AGGREGATE BASE
AC	ASPHALTIC CONCRETE
ACP	ASBESTOS CEMENT PIPE
∠	ANGLE POINT
BC	BEGIN CURVE
BCR	BEGIN CURB RETURN
BDRY	BOUNDARY
BOP	BOTTOM OF PIPE
BVC	BEGIN VERTICAL CURVE
BV	BUTTERFLY VALVE
BW	BACK OF WALK, BOTTOM OF WALL
CAB	CRUSHED AGGREGATE BASE
CB	CATCH BASIN
℄	CENTERLINE
CIP	CAST IN PLACE
CMU	CONCRETE MASONRY UNIT
CONC	CONCRETE
CONST	CONSTRUCT
CONT	CONTINUED
C&G	CURB & GUTTER
CMB	CRUSHED MISCELLANEOUS BASE
CMP	CORRIGATED METAL PIPE
CSP	CORRUGATED STEEL PIPE
CY	CUBIC YARDS
Δ	CURVE DELTA
DET	DETAIL
DIP	DUCTILE IRON PIPE
DG	DECOMPOSED GRANITE
DF	DEEPENED FOOTING
DW	DOMESTIC WATER
DWG	DRAWING
E	ELECTRIC
EA	EACH
EC	END OF CURVE
EG	EXISTING GROUND
ES	EXISTING SURFACE
ESMT	EASEMENT
ECR	END CURB RETURN
EFF	EXISTING FINISH FLOOR
EL OR ELEV	ELEVATION
EP	EDGE OF PAVEMENT
EX OR EXIST	EXISTING
FF	FINISHED FLOOR

ABBREVIATIONS

EVC	END VERTICAL CURVE
FG	FINISHED GRADE
FG	FIRE HYDRANT
FL	FLOWLINE ELEVATION
FS	FINISH SURFACE
G	GAS
GB	GRADE BREAK
GF	GARAGE FLOOR
GV	GATE VALVE
HGL	HYDRAULIC GRADE LINE
HORIZ	HORIZONTAL
HP	HIGH POINT
INT	INTERSECTION
INV OR IE	INVERT ELEVATION
JS	JUNCTION STRUCTURE
L	LENGTH
LAT	LATERAL
LD	LOCAL DEPRESSION
LF	LINEAR FEET
LOL	LAYOUT LINE
LOS	LINE OF SIGHT
LP	LOW POINT
LT	LEFT
MAX	MAXIMUM
MH	MANHOLE
MIN	MINIMUM
MSE	MECHANICALLY STABILIZED EARTH
NAP	NOT A PART
NG	NATURAL GAS
No	NUMBER
NTS	NOT TO SCALE
OC	ON CENTER OR ON CURVE
OFF	OFFSET
OG	ORIGINAL GROUND
OH	OVERHEAD
PAD OR PE	PAD ELEVATION
PB	PULL BOX
PCC	PORTLAND CEMENT CONCRETE OR POINT OF COMPOUND CURVE
PI	POINT OF INTERSECTION
PL OR ℄	PROPERTY LINE
POC	POINT OF CONNECTION
PP	POWER POLE
PRC	POINT OF REVERSE CURVE
PROP OR PR	PROPOSED
PROF	PROFILE
PT	POINT OF TANGENT OR POINT
PVC	POLYVINYL CLORIDE
PVI	POINT OF VERTICAL INTERSECTION
R	RADIUS
RCP	REINFORCED CONCRETE PIPE
RET	RETAINING
R/W OR ROW	RIGHT OF WAY
RT	RIGHT
RW	RECLAIMED OR RECYCLED WATER
SCE	SOUTHERN CALIFORNIA EDISON
SD	STORM DRAIN
SECT	SECTION
SF	SQUARE FEET
SL	STREET LIGHT
ST	STREET
STA	STATION
STD	STANDARD
SS	SANITARY SEWER
SW	SIDEWALK OR STEM WALL
SD	STORM DRAIN
T	TELEPHONE
TAN	TANGENT LENGTH
TB	TOP OF BERM
TC	TOP OF CURB
TF	TOP OF FOOTING
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Helping Nature Store Our Water

SAN BERNARDINO VALLEY WATER CONVERSATION DISTRICT

ATRP MILL CREEK
ALTERNATIVE 2 PRELIMINARY DESIGN
INDEX MAP

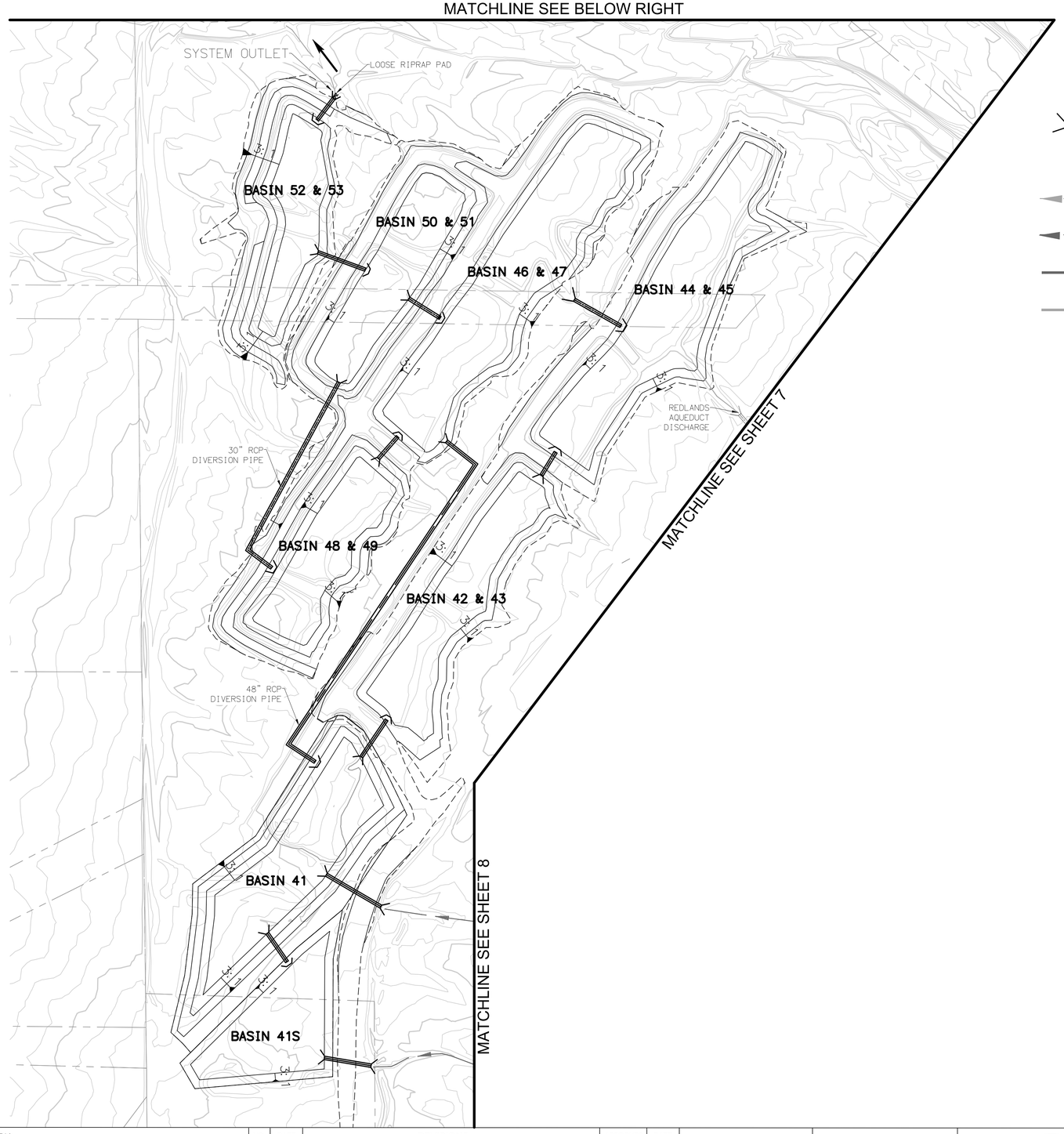
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SHEET 02

OF 08

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MATCHLINE SEE BELOW RIGHT



OUTLET STRUCTURE WITH WEIR

OUTLET STRUCTURE

EXISTING INTER-BASIN UNLINED WATERCOURSE

PROPOSED INTER-BASIN UNLINED WATERCOURSE

PROPOSED SOUTH CANAL

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ATRP MILL CREEK
ALTERNATIVE 2 PRELIMINARY DESIGN
BASIN GRADING 41 TO 53

PROJECT NO. SBVWCD - 0000

SHEET 03

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MATCHLINE SEE BELOW LEFT

OUTLET STRUCTURE WITH WEIR

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PROPOSED INTER-BASIN UNLINED WATERCOURSE

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BASIN 30

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BASIN 29

LOOSE RIPRAP PAD

BASIN 28

BASIN 27

BASIN 26

BASIN 25

MATCHLINE SEE SHEET 6

MATCHLINE SEE SHEET 8

MATCHLINE SEE SHEET 9

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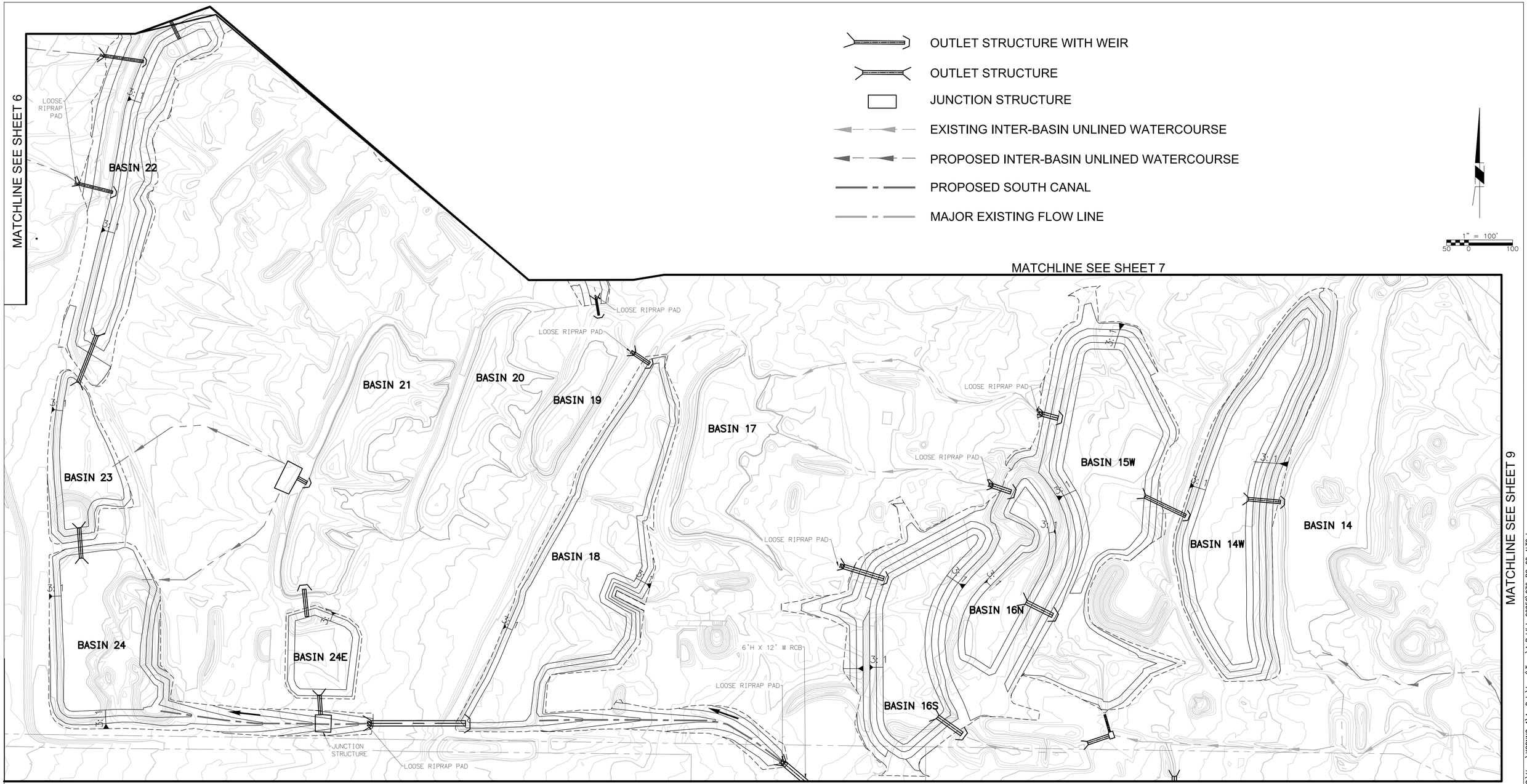
ATRP MILL CREEK
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SHEET 04

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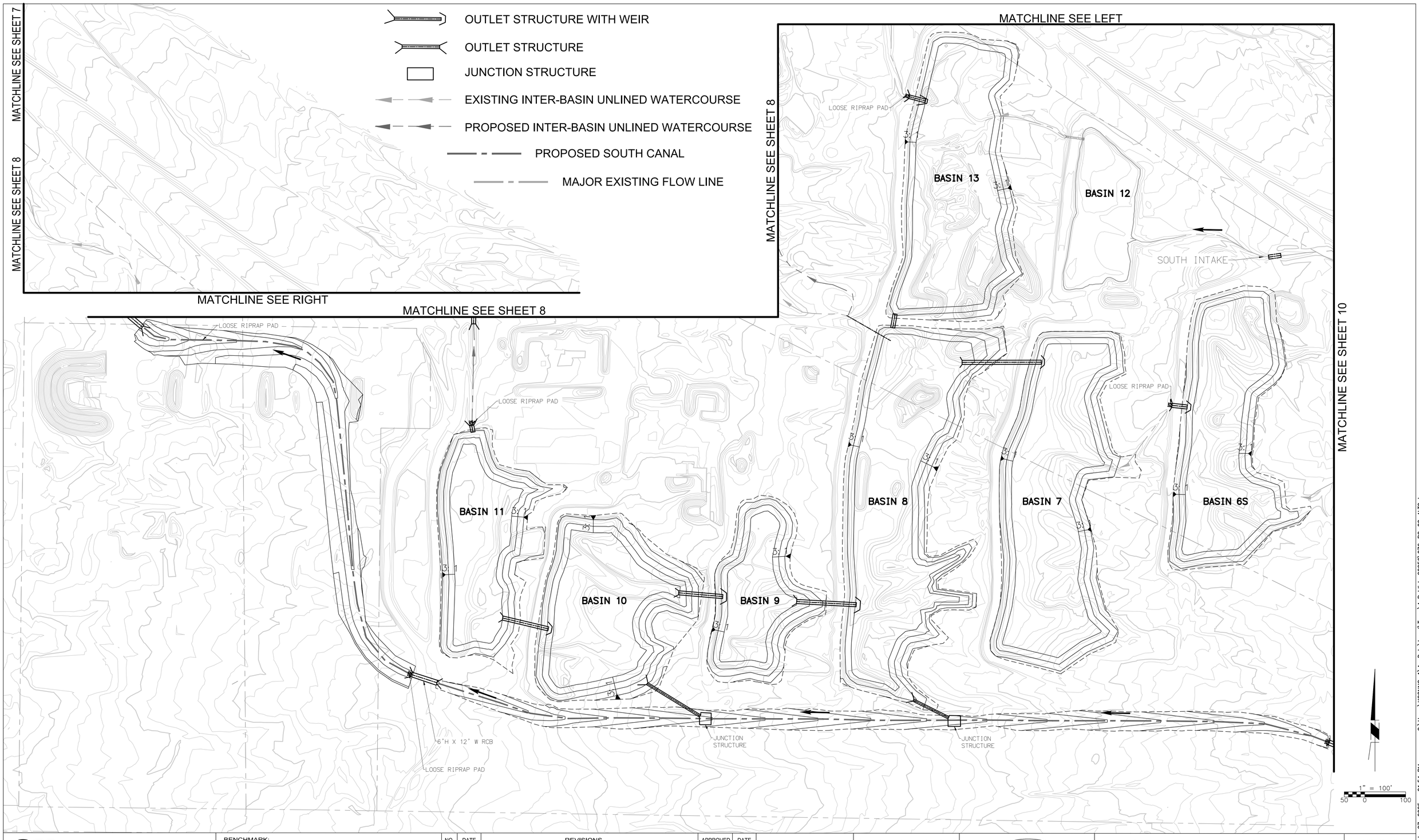
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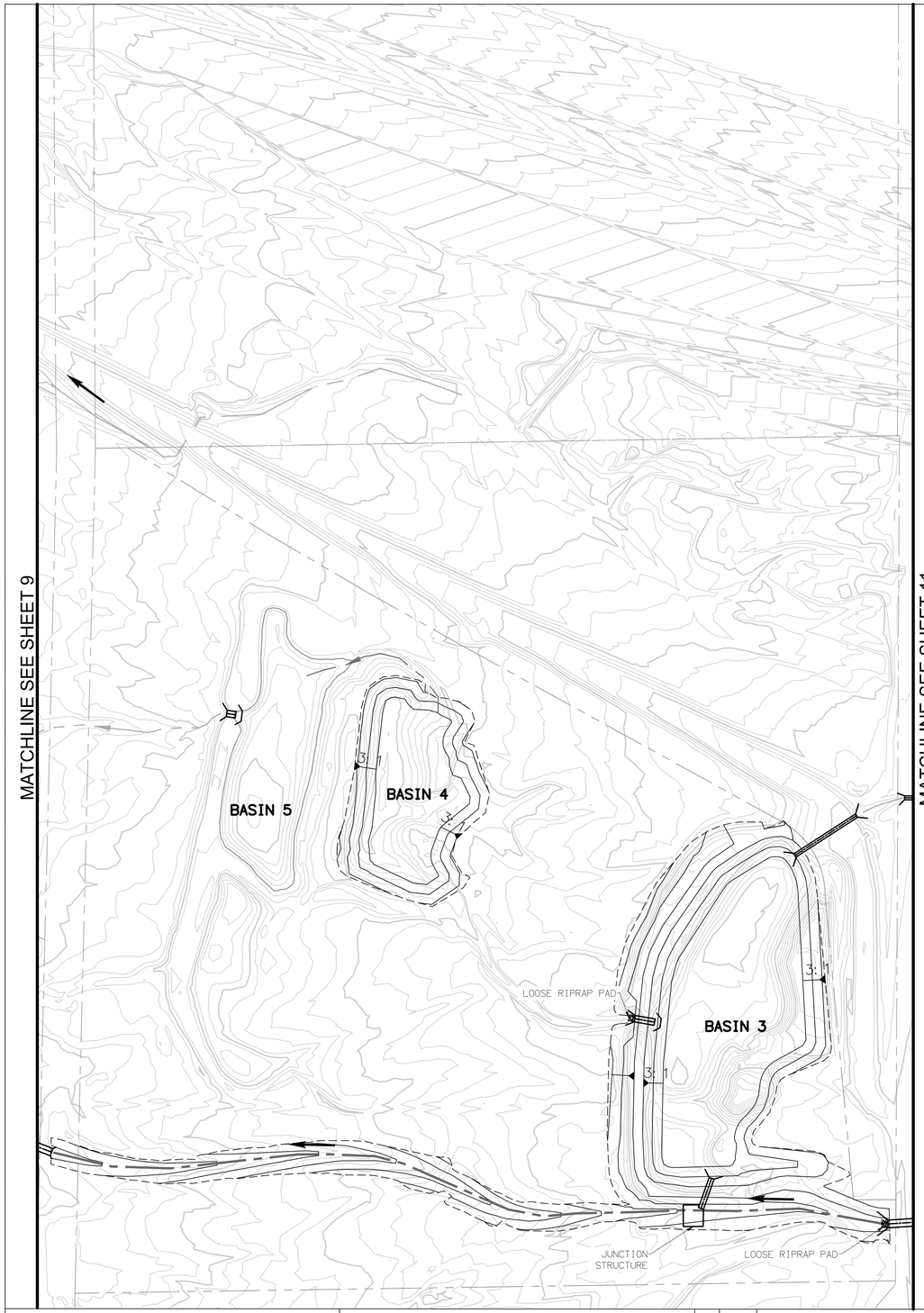
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SHEET 06

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OUTLET STRUCTURE WITH WEIR

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ATRP MILL CREEK
ALTERNATIVE 2 PRELIMINARY DESIGN
BASIN GRADING 3 TO 5

SHEET 07

OF 08

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MATCHLINE BELOW 12



OUTLET STRUCTURE WITH WEIR

OUTLET STRUCTURE

JUNCTION STRUCTURE

EXISTING INTER-BASIN UNLINED WATERCOURSE

PROPOSED INTER-BASIN UNLINED WATERCOURSE

PROPOSED SOUTH CANAL

MAJOR EXISTING FLOW LINE

1" = 100'

50 0 100

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ATRP MILL CREEK
ALTERNATIVE 2 PRELIMINARY DESIGN
BASIN GRADING 1 TO 2

SHEET 08

OF 08

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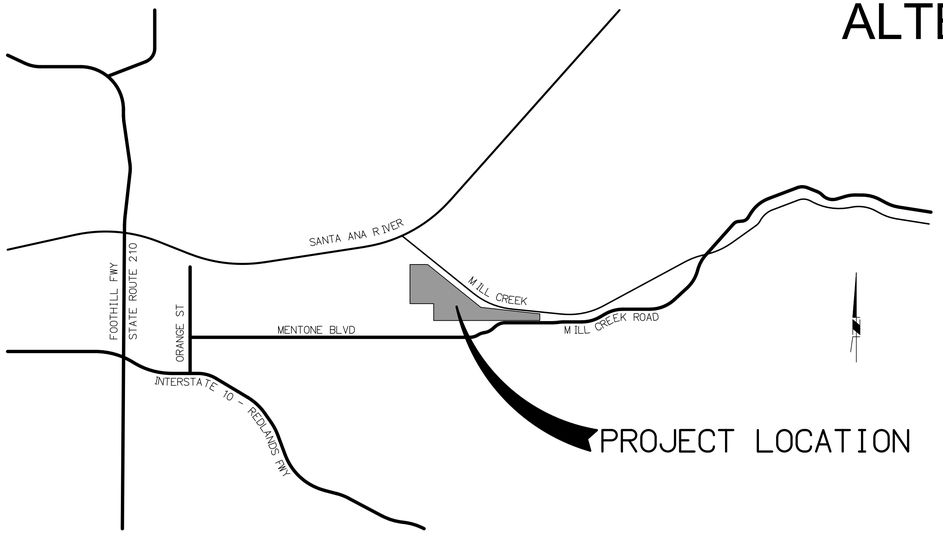
Alternative 3: Concept Plans

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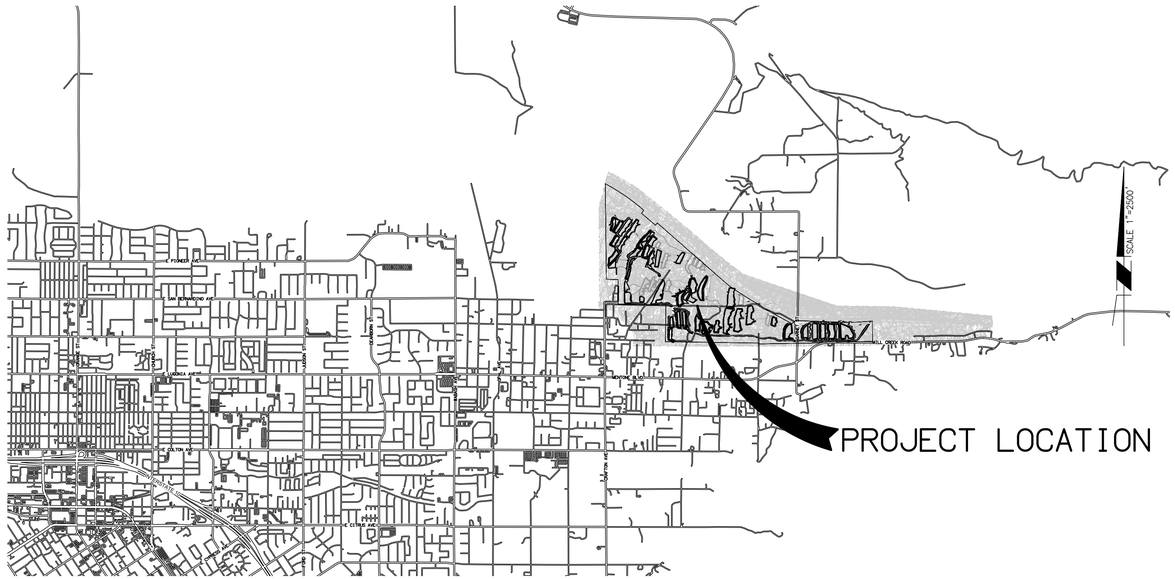
ACTIVE RECHARGE RECHARGE PROJECT

MILL CREEK

ALTERNATIVE 3



VICINITY MAP
NTS



LOCATON MAP

INDEX

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01	TITLE SHEET
02	INDEX MAP
03	BASIN GRADING 41 TO 53
04	BASING GRADING 25 TO 40
05	BASIN GRADING 14 TO 24E
06	BASING GRADING 6S TO 13 AND SW1 TO SW4
07	BASIN GRADING 3 TO 5
08	BASIN GRADING 1 TO 2
09	STRUCTURAL DETAILS
10	MISCELLANEOUS DETAILS

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ATRP MILL CREEK
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TITLE SHEET

SHEET 01
OF 08

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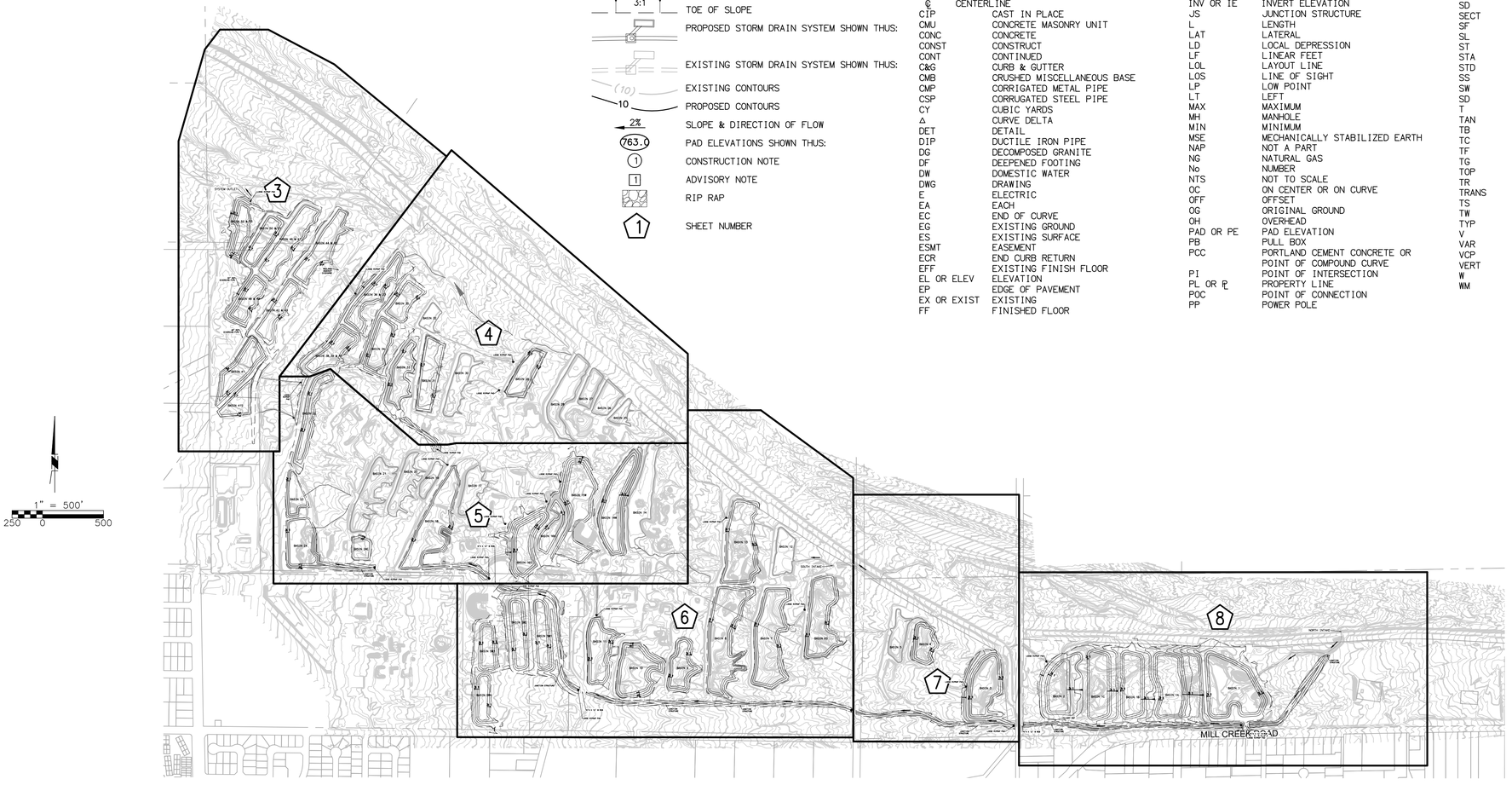
LEGEND

- TRACT BOUNDARY
- RIGHT-OF-WAY LINE
- CENTERLINE
- CMU RETAINING WALL
- EASEMENT
- DAYLIGHT LINE
- CUT FILL LINE (C / F)
- TOP OF SLOPE
- TOE OF SLOPE (3:1)
- PROPOSED STORM DRAIN SYSTEM SHOWN THUS:
- EXISTING STORM DRAIN SYSTEM SHOWN THUS:
- EXISTING CONTOURS (10)
- PROPOSED CONTOURS 10
- SLOPE & DIRECTION OF FLOW 2%
- PAD ELEVATIONS SHOWN THUS: 763.0
- CONSTRUCTION NOTE (1)
- ADVISORY NOTE [1]
- RIP RAP
- SHEET NUMBER (1)

AB	AGGREGATE BASE
AC	ASPHALTIC CONCRETE
ACP	ASBESTOS CEMENT PIPE
∠	ANGLE POINT
BC	BEGIN CURVE
BCR	BEGIN CURB RETURN
BDRY	BOUNDARY
BOP	BOTTOM OF PIPE
BVC	BEGIN VERTICAL CURVE
BV	BUTTERFLY VALVE
BW	BACK OF WALK, BOTTOM OF WALL
CAB	CRUSHED AGGREGATE BASE
CB	CATCH BASIN
℄	CENTERLINE
CIP	CAST IN PLACE
CMU	CONCRETE MASONRY UNIT
CONC	CONCRETE
CONST	CONSTRUCT
CONT	CONTINUED
C&G	CURB & GUTTER
CMB	CRUSHED MISCELLANEOUS BASE
CMP	CORRIGATED METAL PIPE
CSP	CORRUGATED STEEL PIPE
CY	CUBIC YARDS
Δ	CURVE DELTA
DET	DETAIL
DIP	DUCTILE IRON PIPE
DG	DECOMPOSED GRANITE
DF	DEEPENED FOOTING
DW	DOMESTIC WATER
DWG	DRAWING
E	ELECTRIC
EA	EACH
EC	END OF CURVE
EG	EXISTING GROUND
ES	EXISTING SURFACE
ESMT	EASEMENT
ECR	END CURB RETURN
EFF	EXISTING FINISH FLOOR
EL OR ELEV	ELEVATION
EP	EDGE OF PAVEMENT
EX OR EXIST	EXISTING
FF	FINISHED FLOOR

ABBREVIATIONS

EVC	END VERTICAL CURVE
FG	FINISHED GRADE
FG	FIRE HYDRANT
FL	FLOWLINE ELEVATION
FS	FINISH SURFACE
G	GAS
GB	GRADE BREAK
GF	GARAGE FLOOR
GV	GATE VALVE
HGL	HYDRAULIC GRADE LINE
HORIZ	HORIZONTAL
HP	HIGH POINT
INT	INTERSECTION
INV OR IE	INVERT ELEVATION
JS	JUNCTION STRUCTURE
L	LENGTH
LAT	LATERAL
LD	LOCAL DEPRESSION
LF	LINEAR FEET
LOL	LAYOUT LINE
LOS	LINE OF SIGHT
LP	LOW POINT
LT	LEFT
MAX	MAXIMUM
MH	MANHOLE
MIN	MINIMUM
MSE	MECHANICALLY STABILIZED EARTH
NAP	NOT A PART
NG	NATURAL GAS
No	NUMBER
NTS	NOT TO SCALE
OC	ON CENTER OR ON CURVE
OFF	OFFSET
OG	ORIGINAL GROUND
OH	OVERHEAD
PAD OR PE	PAD ELEVATION
PB	PULL BOX
PCC	PORTLAND CEMENT CONCRETE OR POINT OF COMPOUND CURVE
PI	POINT OF INTERSECTION
PL OR ℙ	PROPERTY LINE
POC	POINT OF CONNECTION
PP	POWER POLE
PRC	POINT OF REVERSE CURVE
PROP OR PR	PROPOSED
PROF	PROFILE
PT	POINT OF TANGENT OR POINT
PVC	POLYVINYL CLORIDE
PVI	POINT OF VERTICAL INTERSECTION
R	RADIUS
RCP	REINFORCED CONCRETE PIPE
RET	RETAINING
R/W OR ROW	RIGHT OF WAY
RT	RIGHT
RW	RECLAIMED OR RECYCLED WATER
SCE	SOUTHERN CALIFORNIA EDISON
SD	STORM DRAIN
SECT	SECTION
SF	SQUARE FEET
SL	STREET LIGHT
ST	STREET
STA	STATION
STD	STANDARD
SS	SANITARY SEWER
SW	SIDEWALK OR STEM WALL
SD	STORM DRAIN
T	TELEPHONE
TAN	TANGENT LENGTH
TB	TOP OF BERM
TC	TOP OF CURB
TF	TOP OF FOOTING
TG	TOP OF GRATE
TOP	TOP OF PIPE
TR	TOP OF RETAINING
TRANS	TRANSITION
TS	TOP OF SLOPE OR TRAFFIC SIGNAL
TW	TOP OF WALL
TYP	TYPICAL
V	VALVE
VAR	VARIES
VCP	VITRIFIED CLAY PIPE
VERT	VERTICAL
W	WATER OR WITH
WM	WATER METER



SHEET INDEX MAP

1" = 100'
50 0 100

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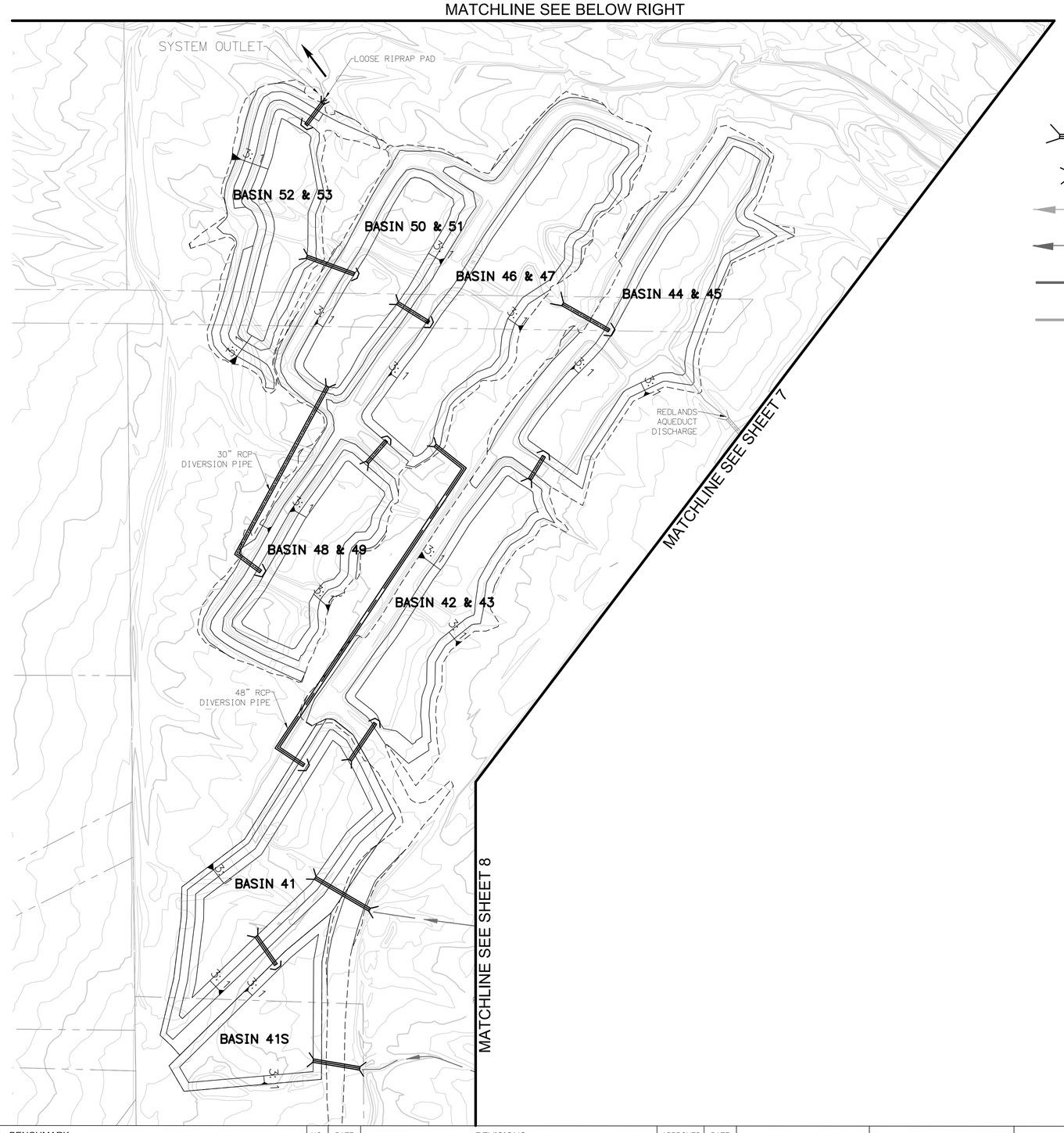
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ATRP MILL CREEK
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INDEX MAP

SHEET 02
OF 08

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OUTLET STRUCTURE WITH WEIR

OUTLET STRUCTURE

EXISTING INTER-BASIN UNLINED WATERCOURSE

PROPOSED INTER-BASIN UNLINED WATERCOURSE

PROPOSED SOUTH CANAL

MAJOR EXISTING FLOW LINE

1" = 100'

50 0 100

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ATRP MILL CREEK
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BASIN GRADING 41 TO 53

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SHEET 03

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MATCHLINE SEE BELOW LEFT

OUTLET STRUCTURE WITH WEIR

OUTLET STRUCTURE

EXISTING INTER-BASIN UNLINED WATERCOURSE

PROPOSED INTER-BASIN UNLINED WATERCOURSE

PROPOSED SOUTH CANAL

MAJOR EXISTING FLOW LINE

LOOSE RIPRAP PAD

BASIN 36 & 37

BASIN 35

BASIN 32

BASIN 34

BASIN 38,39 & 40

BASIN 33

BASIN 31

BASIN 30

LOOSE RIPRAP PAD

BASIN 29

LOOSE RIPRAP PAD

BASIN 28

BASIN 27

BASIN 26

BASIN 25

MATCHLINE SEE SHEET 6

MATCHLINE SEE SHEET 8

MATCHLINE SEE SHEET 9

MATCHLINE SEE SHEET 8

1" = 100'

50 0 100

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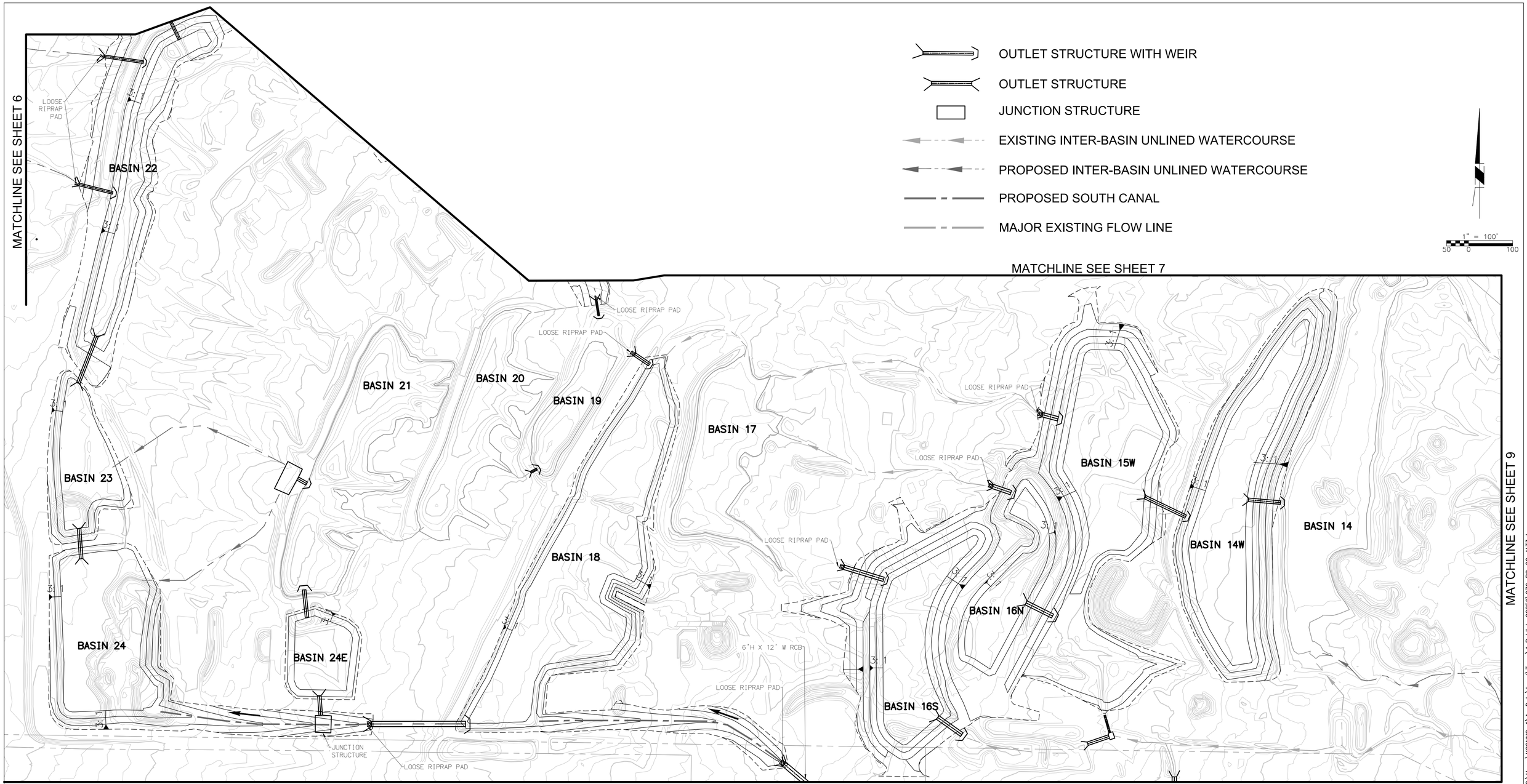
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ATRP MILL CREEK
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BASIN GRADING 25 TO 40

SHEET 04

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BASIN GRADING 14 TO 24E

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OUTLET STRUCTURE WITH WEIR

OUTLET STRUCTURE

JUNCTION STRUCTURE

EXISTING INTER-BASIN UNLINED WATERCOURSE

PROPOSED INTER-BASIN UNLINED WATERCOURSE

PROPOSED SOUTH CANAL

MAJOR EXISTING FLOW LINE

MATCHLINE SEE SHEET 7

MATCHLINE SEE SHEET 8

MATCHLINE SEE RIGHT

MATCHLINE SEE SHEET 8

MATCHLINE SEE LEFT

MATCHLINE SEE SHEET 8

MATCHLINE SEE SHEET 10

LOOSE RIPRAP PAD

BASIN SW2

BASIN SW1

BASIN SW3

BASIN SW4

BASIN 11

BASIN 10

BASIN 9

BASIN 8

BASIN 7

BASIN 6S

BASIN 13

BASIN 12

SOUTH INTAKE

JUNCTION STRUCTURE

6'H X 12' W RCB

3:1

1" = 100'

50 0 100

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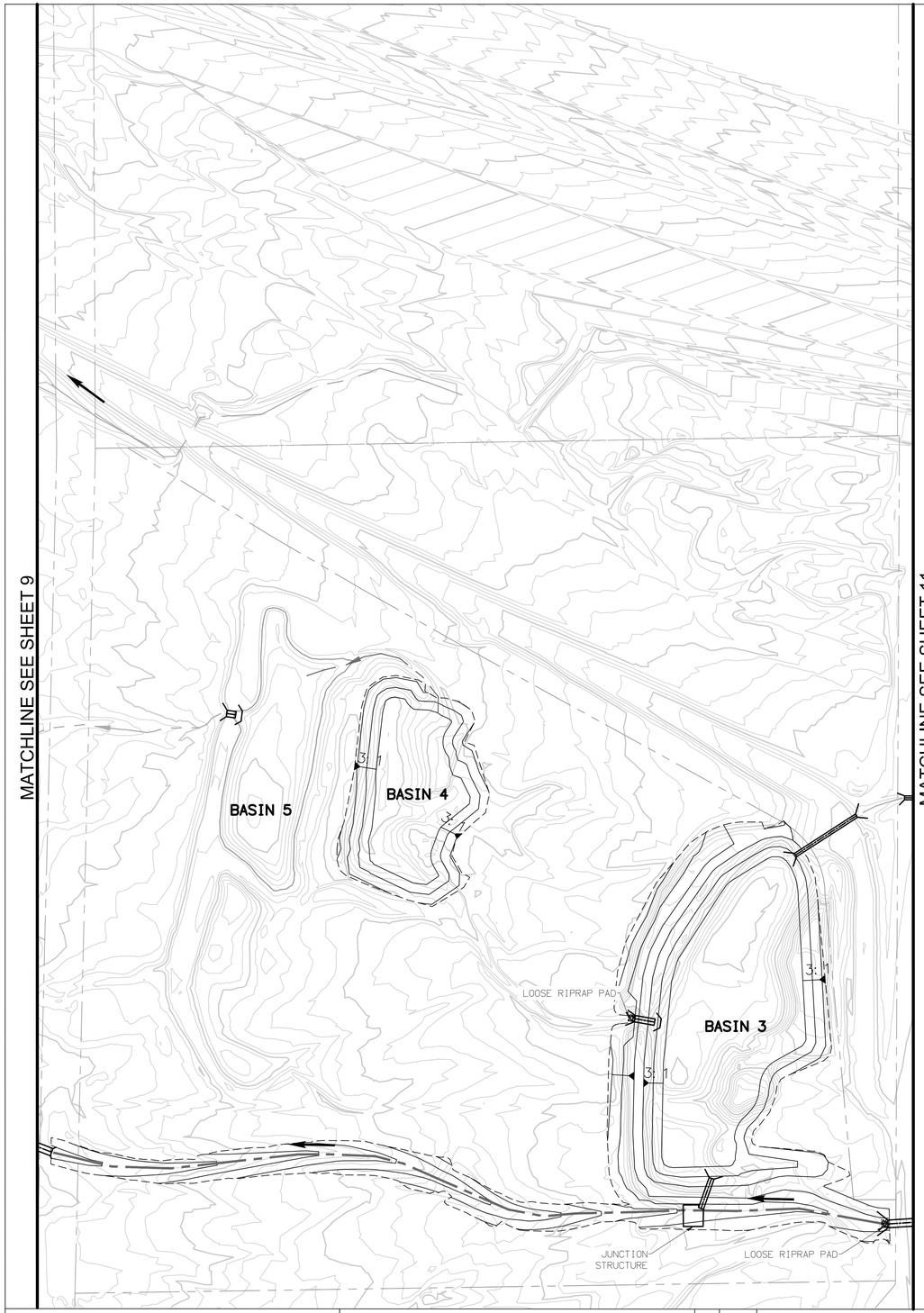
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BASIN GRADING 6S TO 13 AND SW1 TO SW4

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JUNCTION STRUCTURE

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PROPOSED INTER-BASIN UNLINED WATERCOURSE

PROPOSED SOUTH CANAL

MAJOR EXISTING FLOW LINE

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ALTERNATIVE 3 PRELIMINARY DESIGN
BASIN GRADING 3 TO 5

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MATCHLINE BELOW 12



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JUNCTION STRUCTURE

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PROPOSED INTER-BASIN UNLINED WATERCOURSE

PROPOSED SOUTH CANAL

MAJOR EXISTING FLOW LINE

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ATRP MILL CREEK
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BASIN GRADING 1 TO 2

SHEET 08

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ATTACHMENTS

ATTACHMENT I

Mill Creek Basin Hydraulic Technical Memorandum

San Bernardino Valley Water Conservation District

Mill Creek Basin Hydraulic Technical Memorandum



San Bernardino County, California

June 2025

Prepared for:

San Bernardino Valley
Water Conservation District
39611 Oak Glen Road, Bldg. #12
Oak Glen, CA 92399

Prepared by:

Q3 Consulting
27042 Towne Centre Drive, Suite 110
Foothill Ranch, CA 92610
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1 INTRODUCTION

This report summarizes the results of a detailed hydraulic analysis of the existing and alternative proposed improvements for the Active Recharge Transfer Project (ARTP) Mill Creek basins. The goal of this project is to evaluate current and potential recharge yield for various improvements and to identify a cost-benefit relationship to support the San Bernardino Valley Water Conservation District (District) with information to garner a recommendation.

1.1 Existing Mill Creek Basins Operations

The existing Mill Creek Basin location includes 60 retention basins. Additional locations also retain water during large overflow events but are not considered part of the typical operations. The current system has three main flow inlet locations: South Intake (Mill Creek), North Intake (Mill Creek), and the Redlands Aqueduct.

The main Mill Creek diversion occurs at the South Intake, which consists of a reinforced concrete control structure that diverts most flows to the eastern portion of the basins, and some flows west into the North Intake. The Redlands Aqueduct diverts Santa Ana River water to the western basins through a controlled structure.

The basins currently operate through a series of weirs, channels and culverts. The weirs allow for raising, lowering and diversion of flows from one basin to another to allow for optimization and/or taking basins offline for maintenance. Some of the existing diversion structures are natural features set in place by District staff to divert flows.

1.1.1 Mill Creek Basins Source Flows

Flows from Mill Creek are controlled at the South Intake. Flows that reach the intake structure are further controlled upstream within the creek via “soft plugs”. Soft plugs are natural berms of bed material graded by the District to divert low flows towards the South Intake system. The soft plugs vary in size from 2 to 4 feet and have an estimated capacity of up to 400 cfs. During large flow events within the creek, these earthen structures overtop and allow creek flows to continue downstream. After large events, District staff reestablish the “soft plugs”.

The Redlands Aqueduct diverts flow from the Santa Ana River at a rate of up to 25 cfs. Over the past two years, flows through this structure have averaged 13 cfs. discharging directly into Basin #45. Future allocations suggest that 25 cfs can be delivered during wet years.

1.1.2 Existing Basin System Characteristics

The Mill Creek basins are configured to have desilting basins immediately downstream of the intake structures. Basin 1 and Basin 2 are considered desilting basins for the South Intake (See Figure 2-1) for existing basin configurations. Basin 12 is the desilting basin for the North Intake. The basins downstream from the Redlands Aqueduct are not considered detention basins, as the flows from the aqueduct are relatively clean.

Currently the system of basins operate by using a series of weirs and man-made earthen diversions that allow the District the ability to maintain certain basins. As part of the maintenance, the District removes sediment, primarily from the desilting basins, and stockpile the material adjacent to the basins. Basin inverts are scarified to help promote good characteristics of percolation.

1.2 Hydrology

A hydrology study (Addendum 1) was conducted as part of this project in the form of a regional flood frequency analysis using available data from the Mill Creek stream gages as well as surrounding regional stream gages in the area. This study helped to understand the frequencies of various flows that could potentially be diverted into the basins to support the cost-benefit for the various proposed alternatives.

1.2.1 FIRO Flows

Forecast-Informed Reservoir Operations (FIRO) is a process led by the U.S Army Engineer Research and Development Center (ERDC) and the Center for Western Weather and Water Extremes at UC San Diego Scripps Institute of Oceanography (Scripps) and in cooperation with San Bernardino Valley Water District, with the goal to identify strategies to protect and increase the reliability of the local water supply.

As a result of this process, specific flows were identified at the Mill Creek basin site with the reliance of independent flows from multiple sources throughout the year in addition to flows captured from the Mill Creek diversion. These flows are preliminary and could be subject to change.

Table 1 shows the various sources of expected future FIRO flows for the Mill Creek basins. For purposes of analysis in the study, a total of 131 cfs was used for the FIRO flow evaluations.

Table 1-1. Expected Daily FIRO flows at Mill Creek Basins

Source	Location	Flow (cfs)
ZT Unger	South Intake	40
Greenspot (Valley District)	South Intake	20
South Intake (Mill Creek)	South Intake	16
North Intake (Mill Creek)	North Intake	30
Redlands Aqueduct	Redland Aqueduct	25
		Total: 131 cfs

FIRO flows were used as part of the evaluation of potential basin improvement alternatives. This flowrate has been used to resemble an expected daily flow for a wet year. Statistically, flows within Mill Creek (46 cfs) has a return period of 5.5 years. This 46 cfs is divided between the South and North Intakes. The FIRO flow scenario also includes 25 cfs at the Redlands Aqueduct, 40 cfs form the ZT Unger site, and 20 cfs form the Greenspot site.

1.2.2 Stress Flows

Another flowrate used to model different scenarios within alternatives is referred to as the “Stress” flows. Stress flows include the maximum potential diversion flow that the upper diversion structure can convey. This flow was calculated to be 122 cfs. This flow is then divided between the South and North Intakes. Stress flows also include 40 cfs from the ZT Unger site and 13 cfs from the Redlands Aqueduct.

For a full evaluation of the *Mill Creek Regional Flood Frequency Analysis*, see Addendum A.

1.3 Basin Modeling Analyses

The model used to evaluate the basins was the U.S. Army Corps of Engineer’s (USACE) HEC-RAS 6.5 software. Models were set up using flow LiDAR data to help create a 3-dimentional (3D) surface. The District requested a 2-dimentional (2D), unsteady analysis of the basins. Weir and culvert structures were added to the model as 1D structures based off of field survey data.

1.3.1 Model Assumptions and Setup

Some of the assumptions used in the HEC-RAS model include the following:

- Hydrostatic pressure does not impact infiltration rates.
- Inflow hydrographs assume to be constant.
- Interties do not contain transmission losses (only South Canal)
- Onsite rainfall does not impact results of average daily runs.
- HEC-RAS computation time step – 1 second.
- HEC-RAS output intervals – 6 minutes
- Expansion Coefficient – 0.30
- Contraction Coefficient – 0.10
- Grid Cell Size – 10'x10'
- Equations Used – SWE-ELM (Shallow Water Equation)
- Manning's Coefficient – 0.05

The grid size used was 10'x10', but for low flow interties, a 5' x 5' grid size (or less) was used to better define the flow paths. Some surface corrections were made in cases where gaps occurred in the data. A series of locations were presented to the District's field staff to correct these locations using "ridges" where appropriate.

3D Surface Development: A 3-D surface was developed based on topographic data acquired as part of the project, which included aerial LiDAR and ground survey. The basins model extends over 11,000 linear feet and covers an area of approximately 400 acres.

Boundary Conditions: The overall basin analysis is divided into three separate models due to the size. Dividing into three models allows for shorter computational times. These models identified as East Model, Middle Model and West Model are linked together via intertie flows. The outflows from the East Model are added to the inflow to the Middle Model, and the outflow of the Middle Model are added to the inflow to the West Model.

East Model – The east mode includes Basins 1 through 7. Inflow into Basin 1 is from the South Intake. Flows added to this location included the Greenspot and ZT Unger flows for the FIRO scenarios.

Middle Model – The middle model includes Basins 8 through 28. Inflows include the North Intake and outflow from Basin 7 (East Model).

West Model – The west model includes basin 29 through 52. Inflows include outflows from Basin 16s, 16s, 15w, 28 and the Redlands Aqueduct.

For scenarios including maintenance of basins, the proposed South Canal provides source flows for both the middle and west models.

1.3.2 Inflow Hydrographs

Flows were added to the model at three primary locations: South Intake; North Intake; and at the Redlands Aqueduct. These flows were based on the two previously discussed FIRO and Stress flows. Each of these flows were modeled as constant inflows over the duration of the model run. In most cases, the inflow durations lasted for up to 5 days. For example, the FIRO flowrate at the South Intake included a portion of Mill Creek diverted flow (16 cfs), the ZT Unger flow (40 cfs), and the Greenspot flow (20 cfs). The North Intake flow consisted of flows redirected from the South Intake diversion (30 cfs). The Redlands Aqueduct flow was 25 cfs. Each of these flows were entered into the 2D HEC-RAS model as constant flows over several days. Iterations of models were run until the basins reached a constant depth, which occurred when the basins inflow matched the infiltration capacity. This is apparent when the basin all basin depths reach a constant and the outlet flows reach a constant.

1.3.3 Diversion and Outlet Structures

The internal basin structures were modeled as either culverts, open channels (rectangular channels), or combination of weirs and outlet channels. HEC-RAS 2D does not currently calculate reversal of flows well. Fortunately, the Mill Creek basins primarily operate as a gravity system. In some cases, such as diversions, downstream tailwater conditions do impact the hydraulics of a diversion when using pipes or closed conduit structures. Consequently, in some cases, open channel diversions were used to accurately account for potential surcharge or backwater flows. This was the case for some of the diversions from the South Canal to the basins. If the basin water surface elevation rose above the water surface in the canal, flows would begin reversing back into the canal.

HEC-RAS models culvert structures using the FHWA equations, changing the unsteady flows at each time step and calculating the conveyance given the headwater and tailwater conditions. Weirs are calculated in HEC-RAS using the general weir equation. These structures were manipulated for various flow and alternative scenarios to move targeted flows through the system of basins to maximize the amount of flows to fill basins.

In some locations, in-channel diversions are proposed to allow District maintenance staff to direct flows to alternative drainage paths for the purpose of maintenance.

1.3.4 Infiltration Parameters

A geotechnical investigation was performed to determine the existing basin infiltration parameters. As part of the analysis, a range of percolation values were presented for each of the basins. Some hurdles of this testing involved the environmental permitting. This resulted in providing estimates for areas not tested. For a detailed technical memorandum, see Addendum 2. Model scenarios run with these infiltration rates are referred to as “Kleinfelder” infiltration rates.

The District, working with other ARPT projects has developed a range of infiltration parameters for “plugged” conditions. This condition assumed a non-maintained condition that reduces the potential infiltration of each basin. This “clogged” condition was the basis for running proposed alternatives to identify a “worst case scenario” with respect to groundwater recharge yield for the two given inflow parameters. A full list of each basin’s percolation rate for the “clogged” scenario can be found in Addendum 3. Model scenarios run with these infiltration rates are referred to as “District” infiltration rates.

The rating curves for each basin were entered into HEC-RAS as outflow rating curves. Calculations were made for each basin based on depth, surface area, and percolation rate. The resulting infiltration outflow hydrographs consisted of a depth vs outflow in cubic feet per second. These rating curves can be seen in Addendum 3.

1.3.5 Hydrogeology

The result of the HEC-RAS models were processed and delivered to the team hydrogeologists for further analysis. Results were prepared as GIS shapefiles for basin surface areas and infiltration results. This data is added to the groundwater models to identify impact on the Bunker Hill Basin, as well as to identify proposed basin grading characteristics in the way of “mounding”. The hydrogeology models will provide guidance as to basin proximity. If basins are too close together, exfiltration from one basin may inadvertently provide a source of inflow into another basin.

At the time of this DRAFT, the hydrogeology analysis is underway and results will be provided in Addendum 3 in the Final Design Report.

2 ALTERNATIVE ANALYSES

Four model configurations were assessed in this project,

1) Existing Conditions,
2) Alternative 1,
3) Alternative 2,
4) Alternative 3.

Each of these configurations were evaluated using varying inflow parameters. A 'current' flow parameter was used initially to assess the existing basins and structures. This model assumed 80 cfs in the South Intake, 40 cfs in the North Intake, and 13 cfs in the Redlands Aqueduct based on current observed maximum operational flows. Subsequent runs were performed using the FIRO and Stress flows.

2.1 Existing Condition Model

The existing condition model was evaluated to provide a sense of calibration, or correlation, between the model results and reality. Initial models were prepared using District provided daily maximum flows of 80 cfs (South Intake), 40 cfs (North Intake), and 13 cfs (Redlands Aqueduct) in addition to FIRO flows. These results were reviewed by District engineers and field crews to correlate the results with reality. Some model corrections were made to the surface data to "correct" areas not conforming to actual operation. Once corrected.

Another important aspect of modeling the existing conditions is to provide a baseline for the current operational recharge yield to compare proposed improvements with. A cost-benefit evaluation will be prepared as part of the future feasibility study, comparing recharge yield and cost of construction and operation.

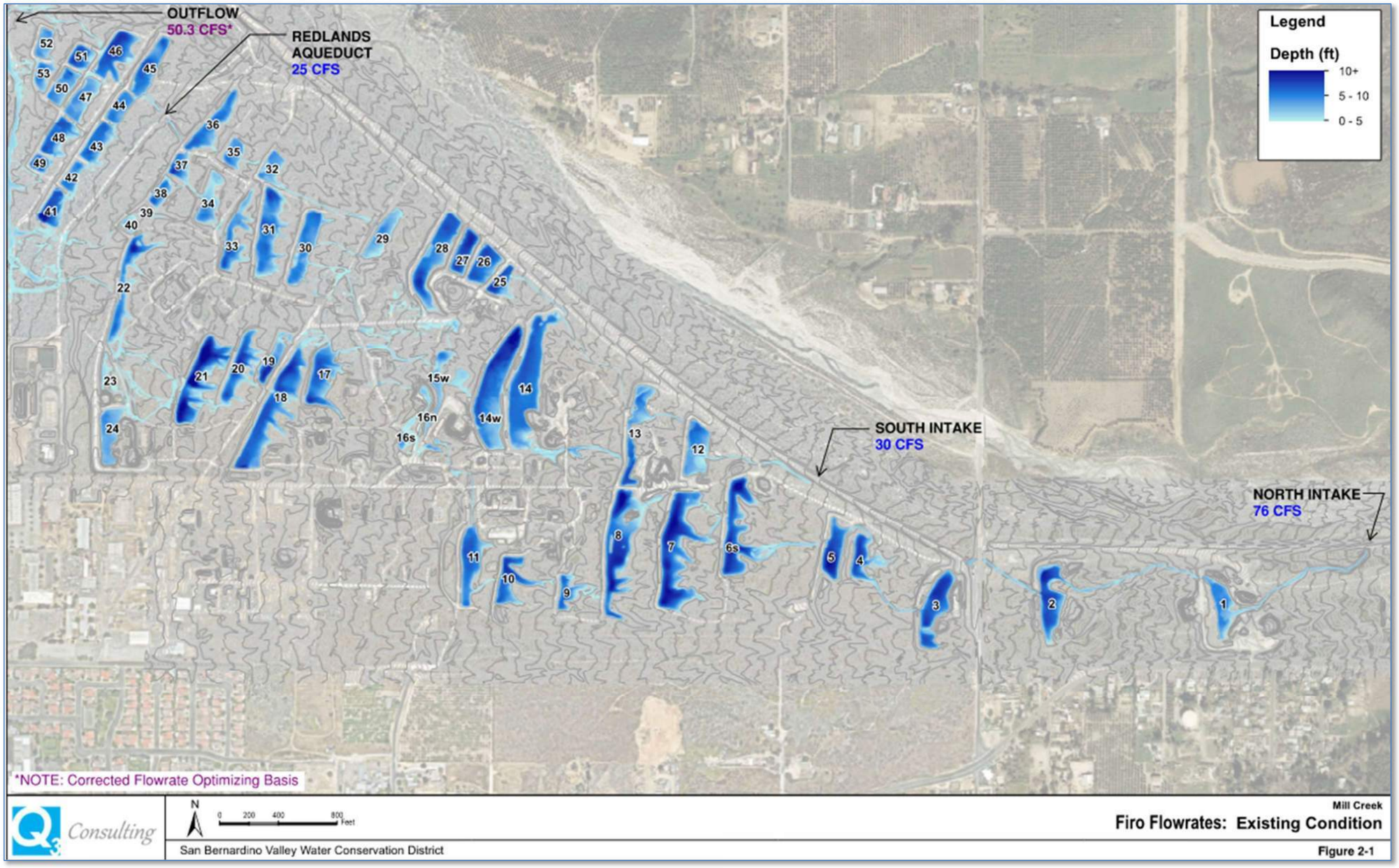
Figure 2-1 shows the model configurations of the Existing Conditions model using FIRO flows. With a total inflow of 131 cfs, the existing system bypasses flow from both Basin 52, 41, and 22. A total of 50.3 cfs flow is released downstream. As can be seen in this image some of the basins have potential to "hold" more flows via manipulating the internal basin outlet structure weirs. This unattenuated basin capacity was further evaluated to understand the theoretical maximum capture and recharge capacity of the existing system. It was found, under the "clogged" infiltration rate conditions that and additional 2.4 cfs could be percolated. This results in a total outflow of 47.8 cfs (50.3cfs – 2.4cfs) of flow leaving the basin site. Under the "clogged" scenario, the maximum potential infiltration capacity is approximately 83.2 cfs (131cfs – 47.8 cfs). This value is based on the water depths calculated in HEC-RAS under the individual basin outlet weir heights designed to keep flow moving while attenuating a particular amount. These structures (weirs) could be adjusted slightly to further slightly increase maximum recharge. For purposes of this baseline analysis, these results are adequate for comparison of future proposed alternatives.

The FIRO flow, clogged infiltration rate condition results are shown in Figure 2-1 for comparative reasons.

Various models were performed using different ratios of flow splits at South and North intakes. This configuration of 76 cfs at the South Intake and 30 cfs at the North Intake provided the best results with respect to maximizing the basin attenuation/percolation.

It should be noted that the Stress flow scenario was not run for the Existing Conditions as it would have overburdened the existing basins causing substantial overtopping.

Figure 2-1. Existing Condition FIRO Flow HEC-RAS Maximum Depth Graphic



2.2 Alternative 1

Alternative 1 includes proposed improvement to multiple basins and the addition of a bypass channel (South Canal) to allow for flows to be redirected if needed during maintenance operations. A list of basins to be improved can be seen in Table 2-1.

Table 2-1. Alternative 1 Proposed Basin Modifications

Existing Basin Number	New Basin Number	Modifications
1	1	Increase Volume
None	1A	New Basin
None	1B	New Basin
None	1C	New Basin
2	2	Increase Volume
3	3	Increase Volume
4	4	Increase Volume
6S	6S	Increase Volume
7	7	Increase Volume
8	8	Increase Volume
9	9	Increase Volume
10	10	Increase Volume
11	11	Increase Volume
13	13	Increase Volume
14	14w	Increase Volume
15w	15w	Increase Volume
16N	16N	Increase Volume
16S	16S	Increase Volume
18	18	Increase Volume
24	24	Increase Volume
23	23	Increase Volume
22	22	Increase Volume
38-40	38-40	Combine and Increase Volume
36-37	36-37	Combine and Increase Volume
34	34	Increase Volume
35	35	Increase Volume
29	29	Increase Volume
42-43	42-43	Combine and Increase Volume
44-45	44-45	Combine and Increase Volume
46-47	46-47	Combine and Increase Volume
50-51	50-51	Combine and Increase Volume
52-53	52-53	Combine and Increase Volume

As seen in the existing conditions model, the capacity of the current basins cannot adequately recharge FIRO flows. Alternative 1 contains the improvements in Table 2-1 in addition to construction of a South Canal to help route flows to various locations within the basins and the ability to greatly reduce the number of basins that can be pulled off-line during maintenance.

Figure 2-2 shows the proposed improvements for Alternative 1. This figure also shows the locations for diversion structures, the South Canal and its proposed connections to various basins, and improved basin outlet structures.

Alternative 1 was run of several scenarios. A list of scenarios can be seen in Table 2-2. Scenarios include variations of inflows from FIRO to Stress flows, and different maintenance actions. Using the South Canal, multiple scenarios were run for different basin maintenance operations, allowing flows to route around specified basins. Maintenance scenarios were run to ensure internal structures were large enough to pass flows through the system.

Table 2-2. Alternative 1 Scenario List

Alternative	Scenario	Flows (S, N, Redlands) cfs	Infiltration	Maintenance (Off-line)
1	1	Stress (122, 40, 13)	Kleinfelder	None
	2	Stress (122, 40, 13)	Kleinfelder	1, 1A, 1B, 1C, 2, 12, 13
	3	Stress (122, 40, 13)	Kleinfelder	3, 4, 5, 6, 19, 20, 21, 22, 22E, 23, 24
	4	Stress (122, 40, 13)	Kleinfelder	25, 26, 27, 28, 29, 30, 31, 32
	5	Stress (122, 40, 13)	Kleinfelder	42-43, 44-45, 46-47, 48-49, 50-51, 52-53
	6	Stress (122, 40, 13)	District	
	7	FIRO (76, 30, 25)	District	

From this list of scenarios, Scenario 7 was used to compare to the Existing Conditions FIRO run to identify the ability to capture and recharge more runoff. As can be seen in Figure 2-2, the outflow from Basin 52-53 is 9.9 cfs. This compared to the existing condition outflow of 50.3 cfs., for a difference of 40.4 cfs.

Figure 2-2 shows the results of the HEC-RAS model, maximum depth conditions. In this case, some of the basins do not show flow. By maximizing the potential infiltration parameters of a few basins, the outflow numbers were reduced to 9.9 cfs. Other iterations of this run could be performed until more flow is diverted to empty basins in an iterative modeling approach to get the outflow number down to around 5 to 6 cfs. It has been identified by the District that a riparian restoration project is to be established near the outlet of the basin 52-53. Consequently, flow will need to be discharged to sustain this project (5 to 6 cfs).

In summary, Alternative 1, Scenario 7 shows that during FIRO flow conditions, using District infiltration rates, that a much-improved capture and recharge is available. It should be noted that each alternative used a different set of "District" infiltration rates. For example, large difference can be seen between infiltration rates (Kleinfelder versus District) for each alternative (See Section 2.3).

Figure 2-2. Alternative 1 – Proposed Improvements

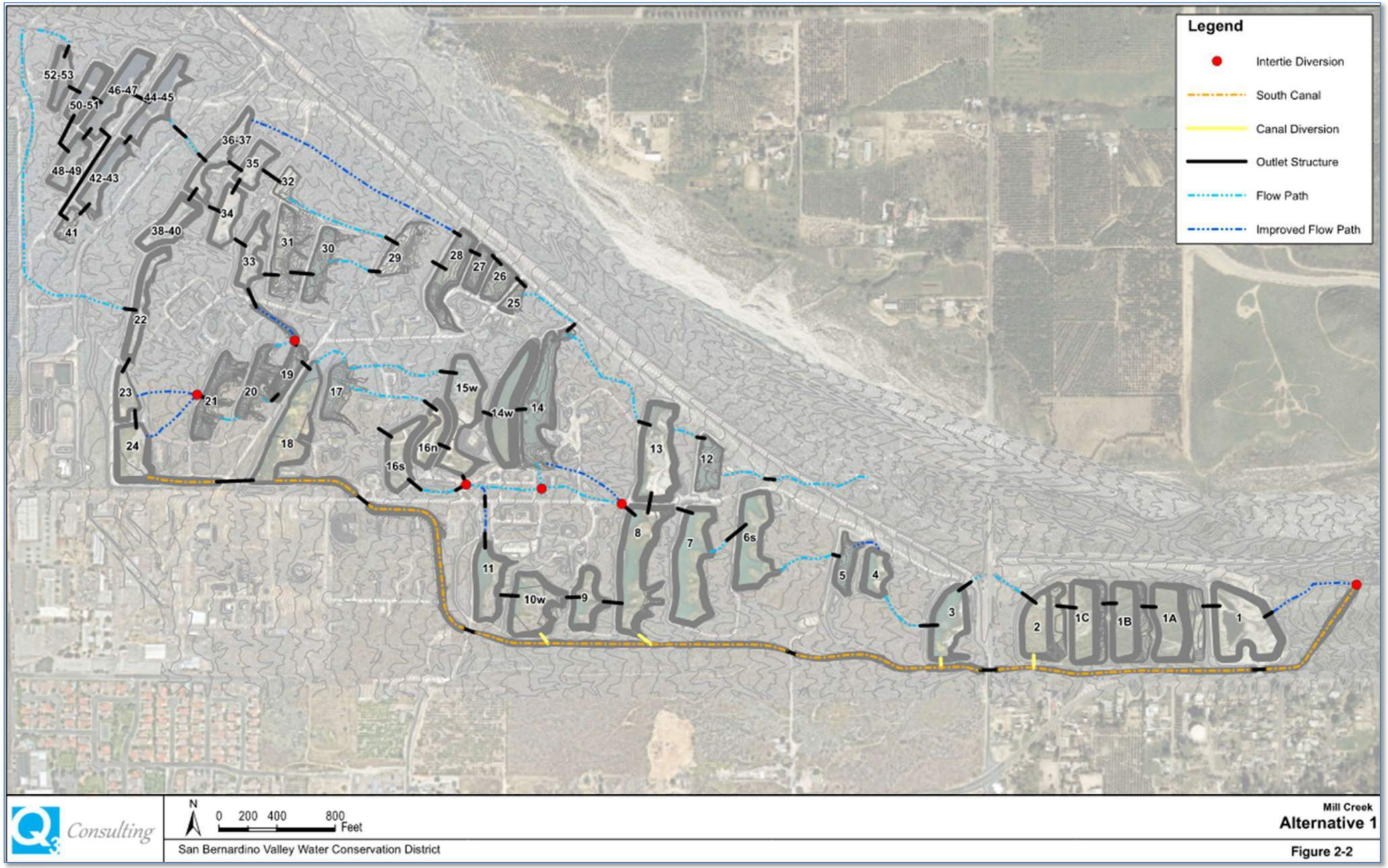


Figure 2-3. Alternative 1 – FIRO Flow Results HEC-RAS Maximum Depth Graphic

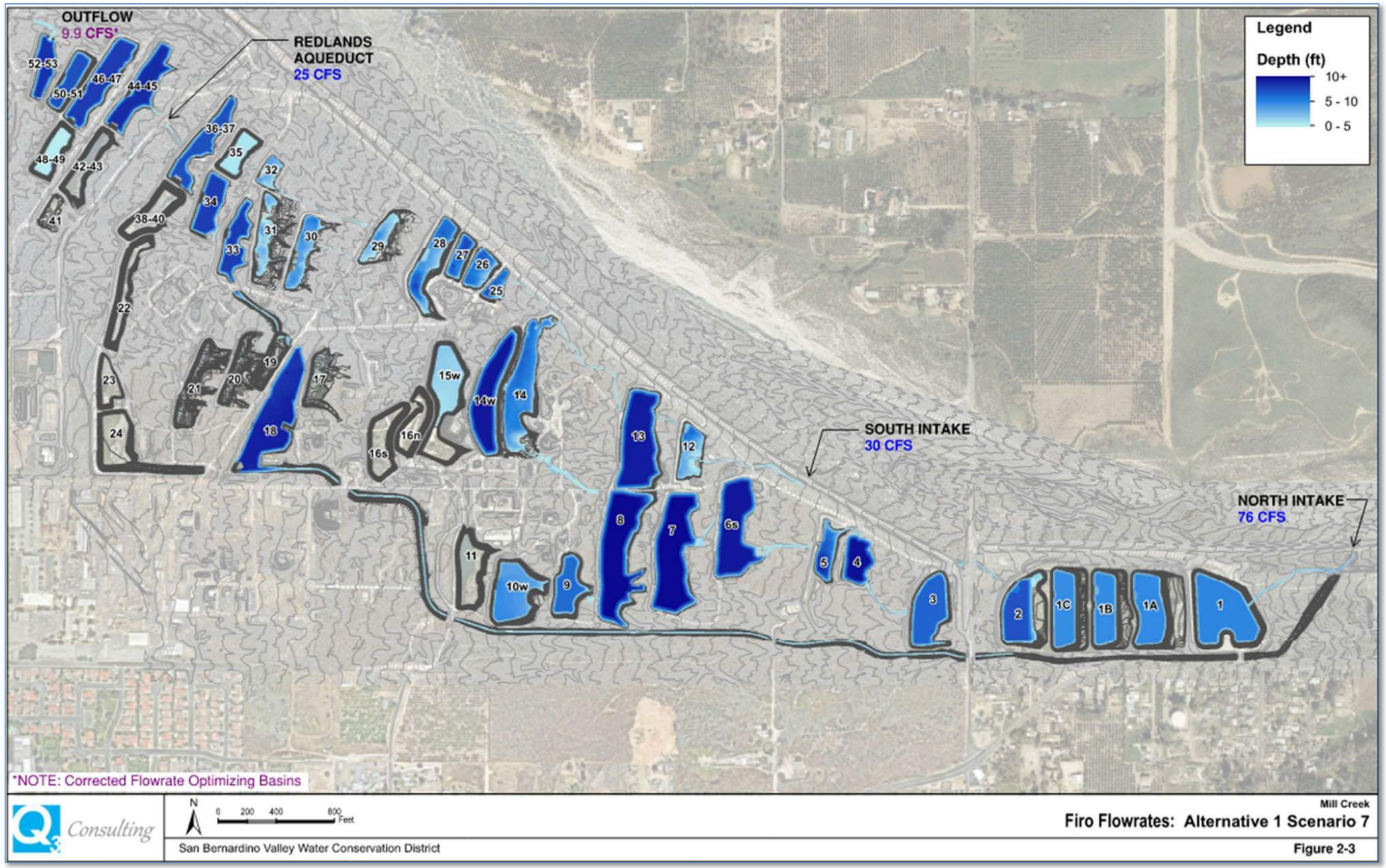
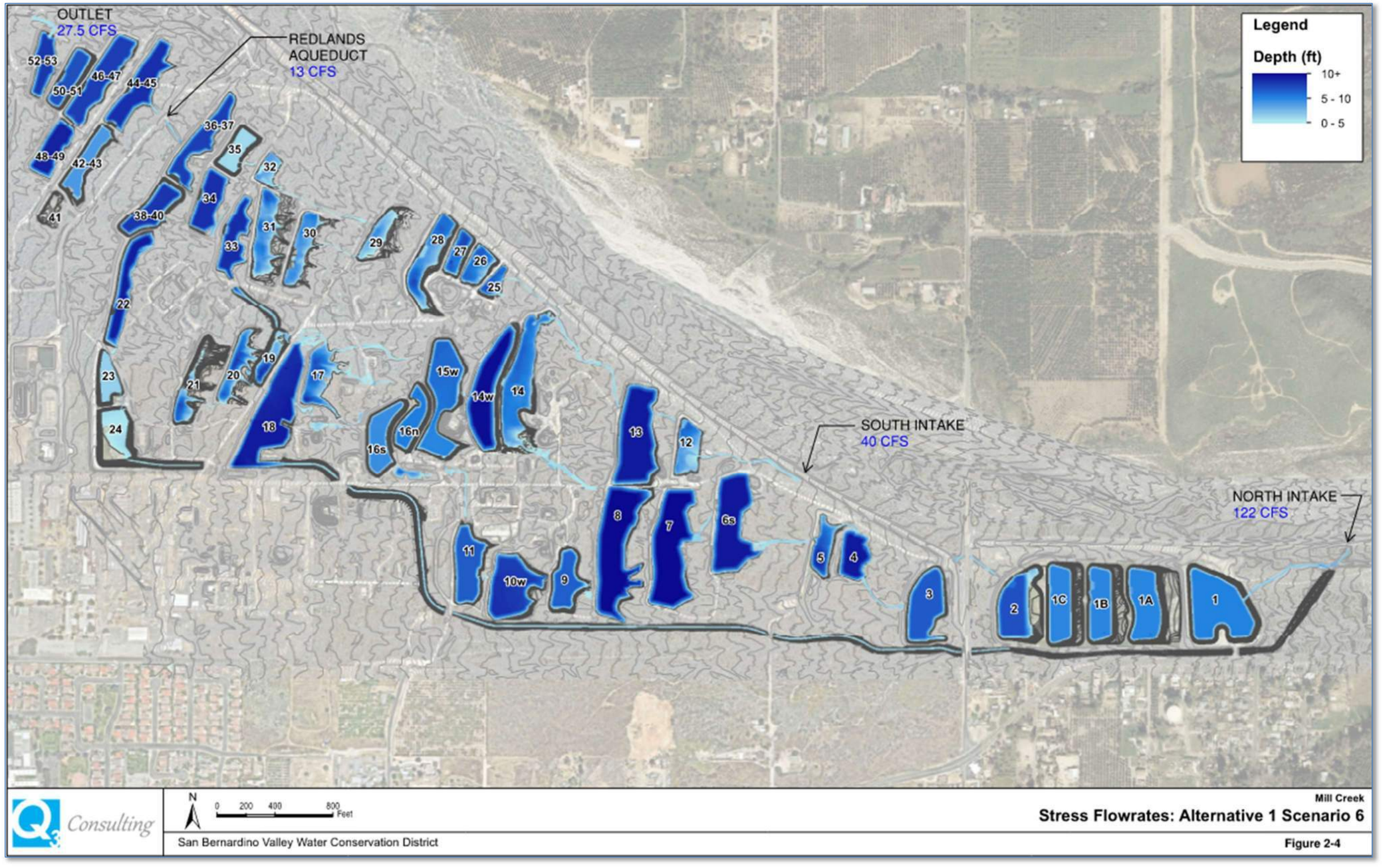


Figure 2-4. Alternative 1 – Stress Flow Results HEC-RAS Maximum Depth Graphic



2.3 Alternative 2

Alternative 2 includes all of the proposed improvements for Alternative 1, including three (3) new basins and improvements to two others. A list of improvements beyond those identified in Alternative 2 can be seen in Table 2-3.

Table 2-3. Alternative 2 Proposed Basin Modifications

Existing Basin Number	New Basin Number	Modifications
None	24E	New Basin
None	41	New Basin
None	41S	New Basin
29	29	Increase Volume
31	31	Increase Volume

Alternative 2 contains the improvements in Table 2-3 in addition to construction of a South Canal to help route flows to various locations within the basins and the ability to greatly reduce the number of basins that can be pulled off-line during maintenance.

Figure 2-5 shows the proposed improvements for Alternative 2. This figure also shows the locations for diversion structures, the South Canal and its proposed connections to various basins, and improved basin outlet structures.

Alternative 2 was run of several scenarios. A list of scenarios can be seen in Table 2-4. Scenarios include variations of inflows from FIRO to Stress flows, and different maintenance actions.

Table 2-4. Alternative 2 Scenario List

Alternative	Scenario	Flows (S, N, Redlands) cfs	Infiltration
2	1	Stress (122, 40, 13)	Kleinfelder
	2	Stress (122, 40, 13)	District
	3	FIRO (76, 30, 25)	District

Figures 2-7 and 2-8 show the comparison of the two different infiltration rates used on the basins under the Stress flow conditions. It should be noted that the Kleinfelder values are much more conservative and show the basins filling more than the results of the District values.

Figure 2-5. Alternative 2 – Proposed Improvements

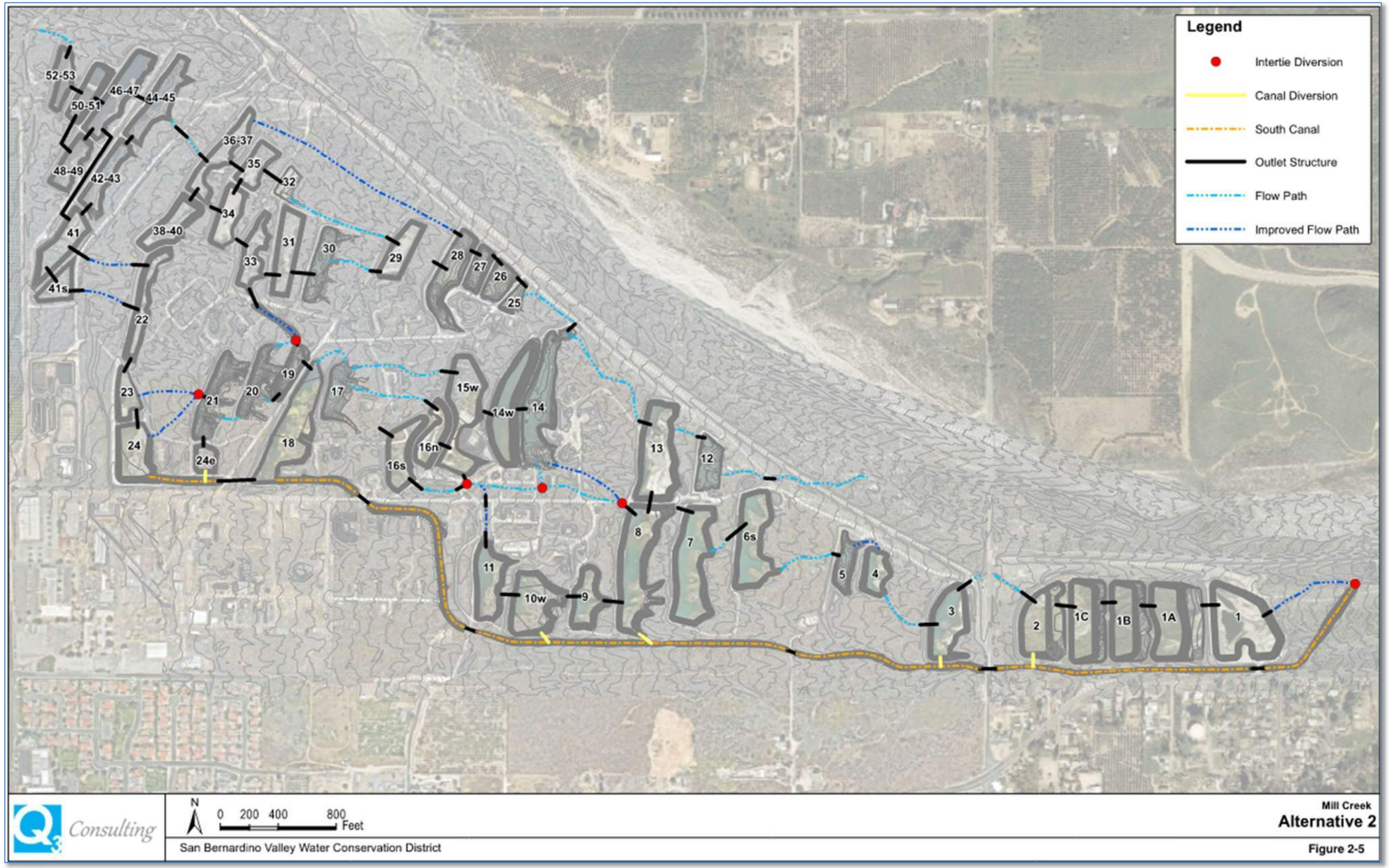


Figure 2-6. Alternative 2– FIRO Flow Results HEC-RAS Maximum Depth Graphic

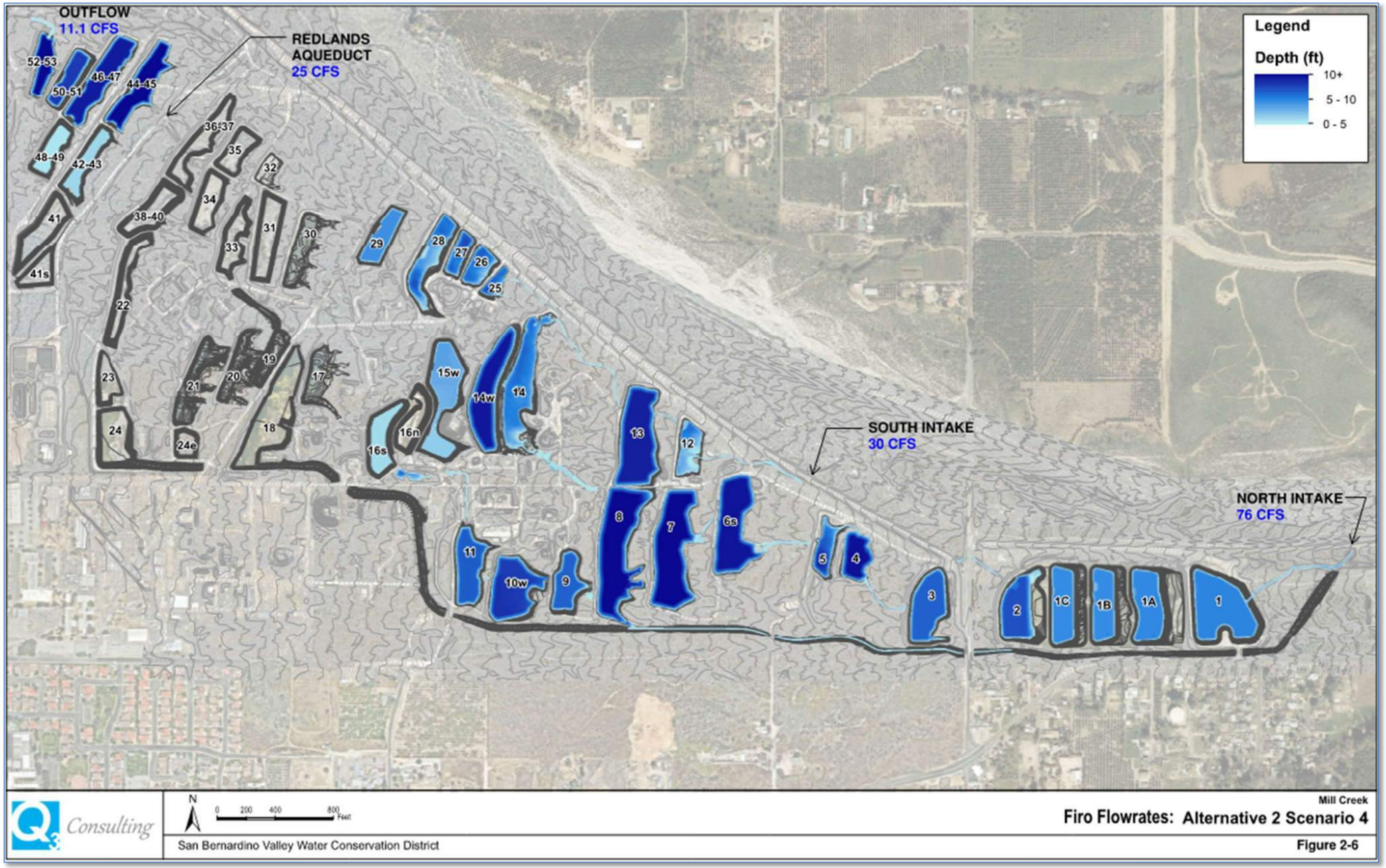


Figure 2-7. Alternative 2– STRESS Flow Results HEC-RAS Maximum Depth Graphic (District Infiltration Rates)

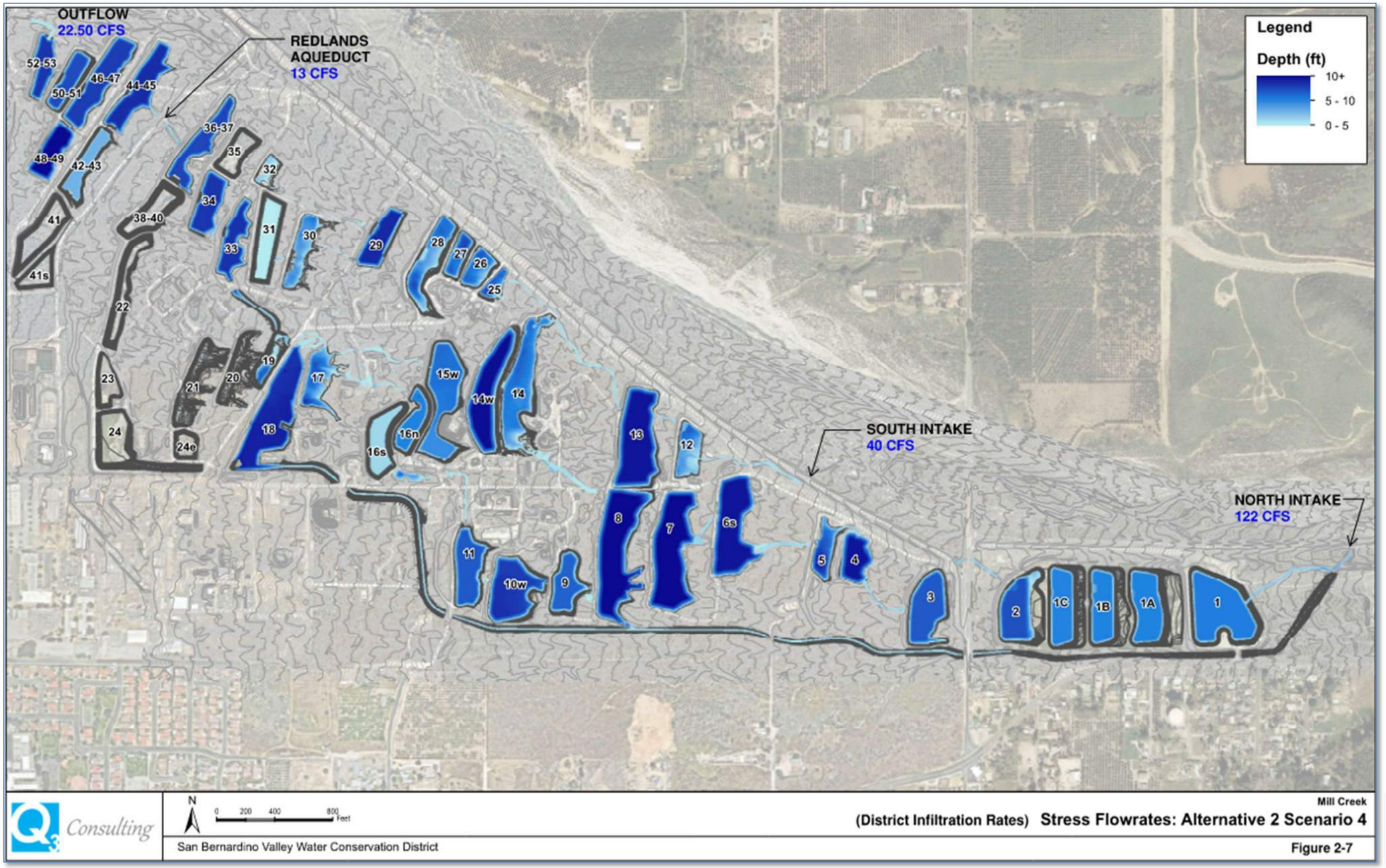
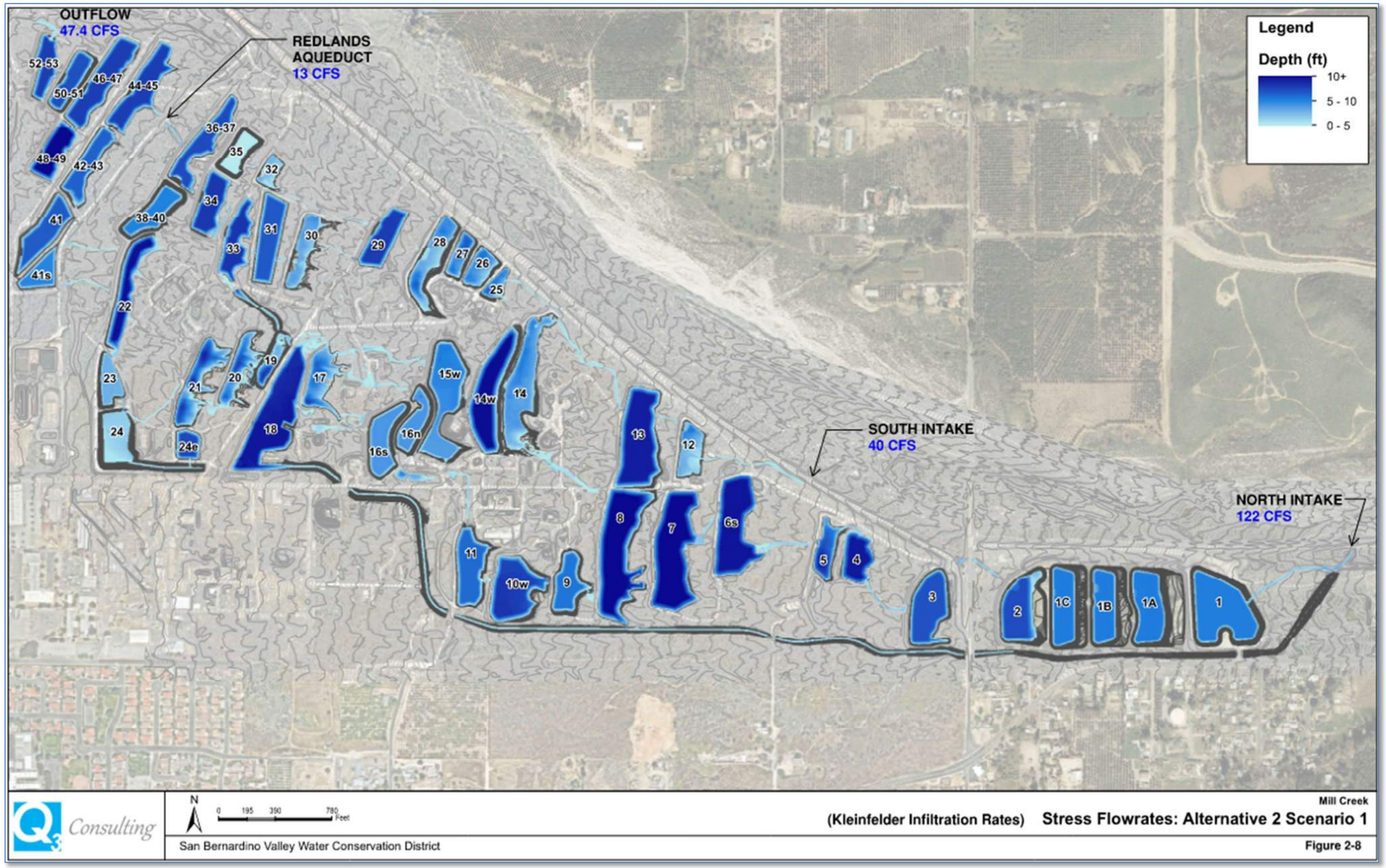


Figure 2-8. Alternative 2– STRESS Flow Results HEC-RAS Maximum Depth Graphic (Kleinfelder Infiltration Rates)



2.4 Alternative 3

Alternative 3 includes all of the proposed improvements for Alternative 2, including four (4) new basins. A list of improvements beyond those identified in Alternative 3 can be seen in Table 2-3.

Table 2-5. Alternative 2 Proposed Basin Modifications

Existing Basin Number	New Basin Number	Modifications
None	SW1	New Basin
None	SW2	New Basin
None	SW3	New Basin
None	SW4	New Basin

Alternative 3 contains the improvements in Table 2-5 in addition to construction of a South Canal to help route flows to various locations within the basins and the ability to greatly reduce the number of basins that can be pulled off-line during maintenance.

Figure 2-9 shows the proposed improvements for Alternative 3. This figure also shows the locations for diversion structures, the South Canal and its proposed connections to various basins, and improved basin outlet structures.

Alternative 3 was run for one scenario (District Infiltration) due to the results of the Alternative 2 FIRO flow model. That model suggested that the basins capacity to percolate was much greater than the inflow. Consequently, Alternative 3, FIRO would show even less basins filling. Alternative 3 was run for the Stress flow conditions using the District's infiltration parameters. Figure 2-10 shows the results of this run. As can be seen, most of the basins fill except for basin 30, 31, and 35. With only 5.7 cfs discharging from Basin 52-53, the capacity of the basins is close to optimized. Kleinfelder infiltration rates would show more flow conveying through system, since the percolation rates as a whole are less than the District rates.

Figure 2-9. Alternative 3 Proposed Improvements

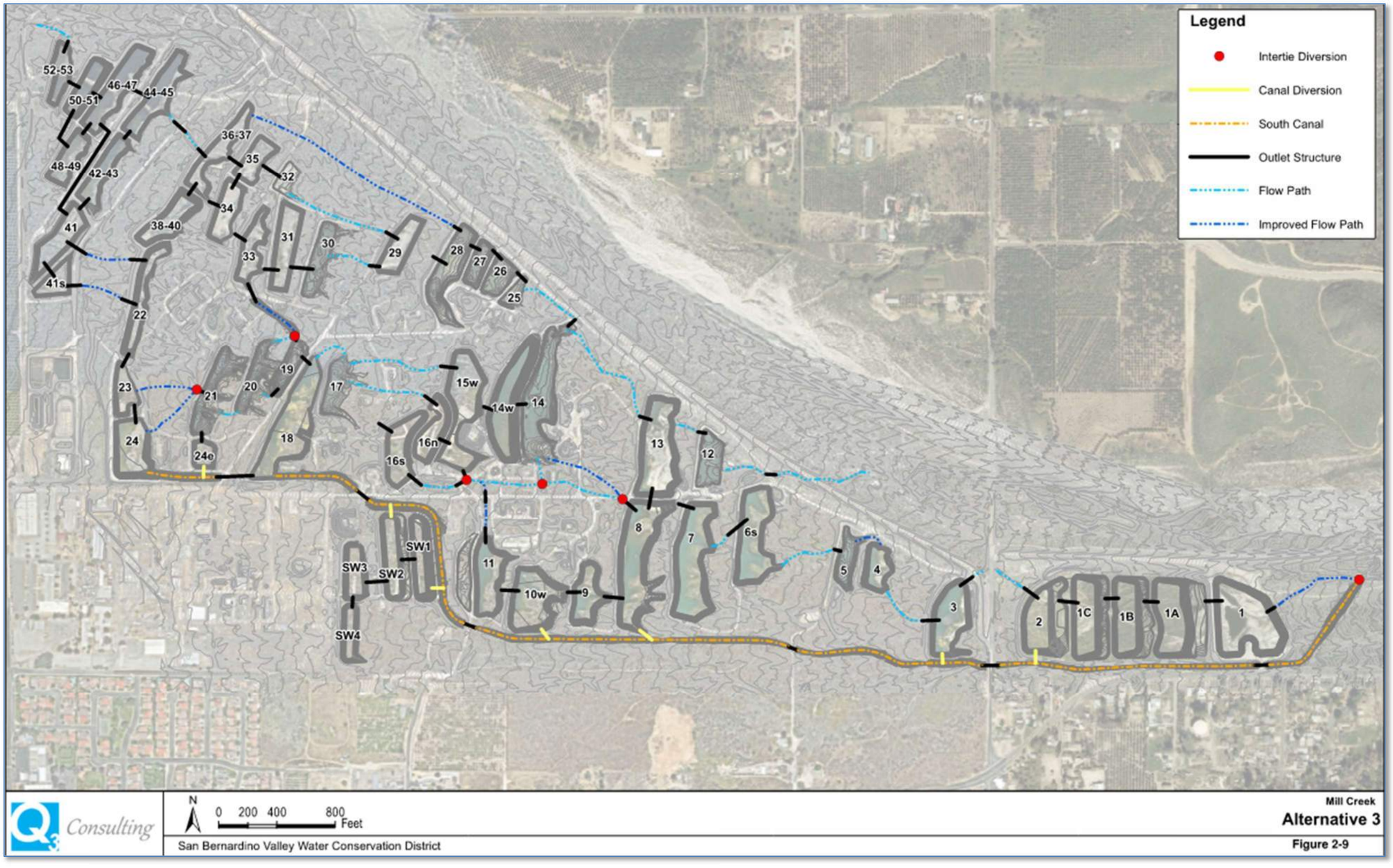
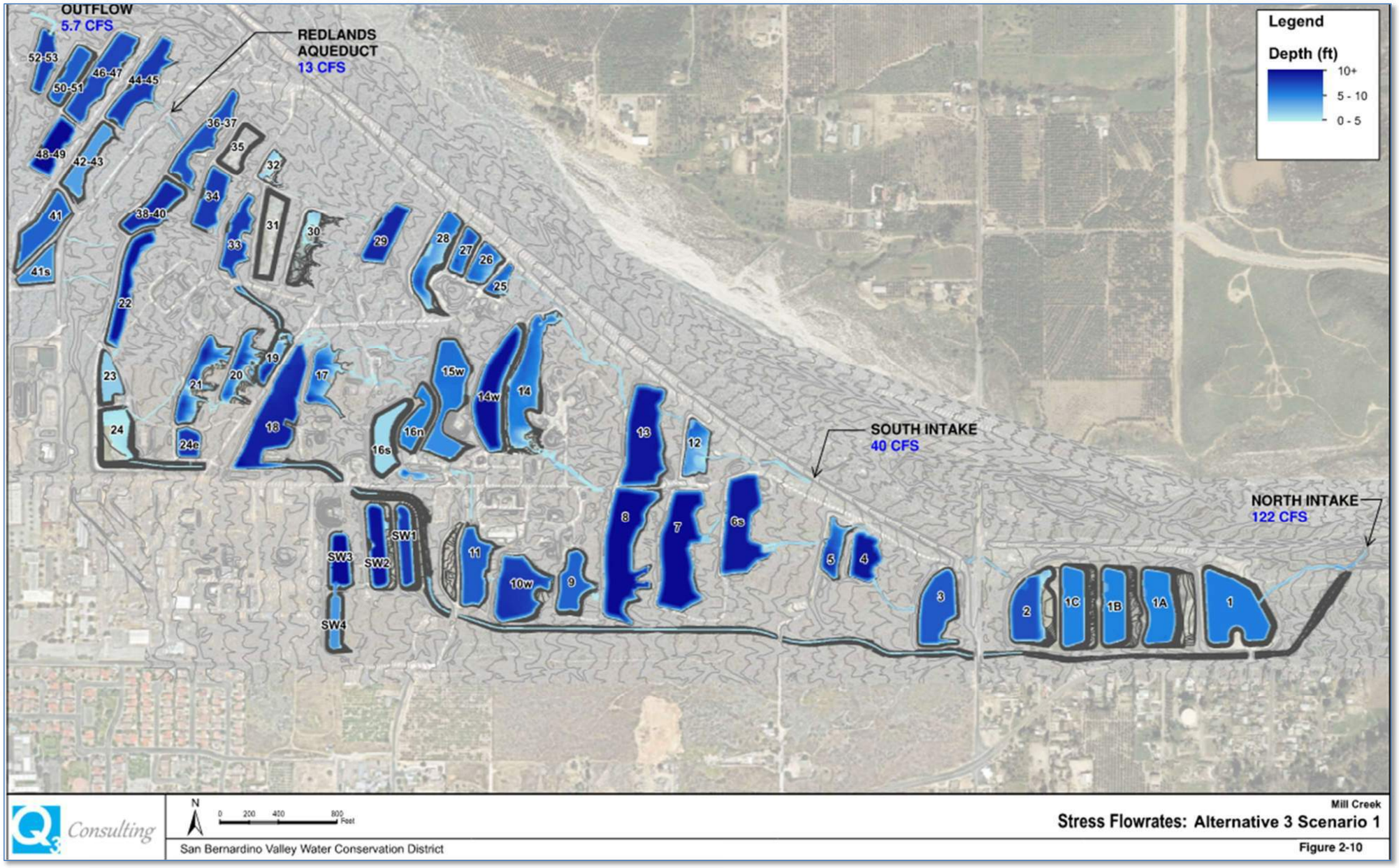


Figure 2-10. Alternative 3, STRESS Flow Results HEC-RAS Maximum Depth Graphic (District Infiltration Rates)



3 SUMMARY

Three alternative basin configurations were evaluated to understand the probable benefit of increasing the size and recharge yield of the Mill Creek basins. HEC-RAS models were performed for each of the Alternative basin alignments, for multiple flow and maintenance scenarios.

For this TM, two primary flow scenarios were tested; 1) FIRO flows; and 2) Stress flows. As described previously, these two flows are intended to measure expected flow (FIRO) and maximum flow (Stress) events. The purpose of the HEC-RAS models was to evaluate the performance of the basins interconnections, including weir structures and intertie channel systems to function under different scenario conditions.

The results of three models were provided for hydrogeologic investigation to understand the groundwater dynamics for the given proposed basins, and to identify if "mounding" would occur under their current proposed grading footprints. The groundwater models are currently being run for a 52-day duration. The three models included:

1) Existing Condition Basins (FIRO Flows);
2) Alternative 1, Scenario 7 (FIRO Flows); and
3) Alternative 3, Scenario 1 (Stress Flows).

The FIRO flows provided by the District in this study are preliminary estimates and subject to potential change. The volumetric inflow for the two flow scenarios can be extrapolated assuming a constant flowrate at each of the three intake locations (South, North, and Redlands Aqueduct). Under Stress flow conditions, the South Intake includes both the Greenspot and ZT Unger expected flows.

Table 4-1 shows the total inflow volumes for each of the model conditions used for the initial groundwater investigation. It should be noted that these values assume models have reached a steady state condition.

Table 3-1. Groundwater Model Daily Inflows and Outflows (District Infiltration Rates)

MODELED CONDITION	Total Inflow (CFS)	Total Discharge at Basin 53 (CFS)	Total Flow Captured (CFS)
Existing Conditions (FIRO Flows)	131	50.3	**80.7**
Alternative 1, Scenario 7 (FIRO Flows)	131	9.9	**121.1**
Alternative 3, Scenario 1 (Stress Flows)	175	5.7	**169.3**

Alternative 3, Scenario 2 (FIRO) was not evaluated in HEC-RAS since the Alternative 1, Scenario 7 model showed several "dry" basins. Since Alternative 3 has several more proposed basins than Alternative 1, the FIRO flows would have left more basin "dry" with a similar "total flow captures" as Alternative 1, Scenario 7.

Alternative 3, Scenario 1 was tested using the "Stress" flows to understand how the system would react to the expected maximum flow diversion. It can be deducted that both Alternative 2 and Alternative 1 would predictively "discharge" larger flows out of Basin 53.

Basin Optimization Estimations

Original HEC-RAS models were manipulated to ensure basins allowed flows to move through the system without overtopping using HEC-RAS. Multiple iterations of each Alternative and Scenario were performed to try and fill as many basins as possible. This method requires substantial time, as each iteration of each structure requires a recalculation that take up to five days each. Consequently, for models showing excess system outflow (out of basin 53), a ratio based on infiltration rate in Excel was

used to approximate how "dry" basins, if filled could reduce the system outflow by way of capture and percolation.

Although these numbers were not completely optimized (via HEC-RAS), they provide a guide as to the probable groundwater recharge rate. For example, Alternative 3, Scenario 1 was optimized based on estimated additional capacity of basins, not filled, in the HEC-RAS models. Basins 31 and 35 appear "dry" in the HEC-RAS results. Instead of remodeling using an iterative approach to manipulate adjacent basin weirs to control inflows to these basins, it was estimated based on the given infiltration rates how much potential flow could be captured and infiltrated in these basins. This percolation rate was subtracted from the system outflow as can be seen in the "optimized figures" (for Alternative 3, Scenario 1, see Figure 2-10).

If the values in Table 4-1 were extrapolated over a single day period (24-hours), volumes of potential recharge can be identified. Table 4-2 shows the approximate volumes for selected Alternatives and Scenarios. This table also shows estimated optimization values for comparison. It should be noted that this is assuming all basins are running at a steady condition. In reality, a lag of up to six (6) days is necessary to fill the basins.

Table 3-2. Estimated Captured Volumes Based on HEC-RAS (District Infiltration Rates)

MODELED CONDITION	Total Inflow Volume (Ac-Ft)	Total Volume Captured (Ac-Ft)
Existing Conditions (FIRO Flows)	259.7	**159.9**
Alternative 1, Scenario 7 (FIRO Flows)	259.7	**240.0**
Alternative 1, Scenario 6 (Stress Flows)*	346.9	**304.0**
Alternative 2, Scenario 2 (Stress Flows)*	346.9	**336.9**
Alternative 3, Scenario 1 (Stress Flows)*	346.9	**346.4**

**Note: Flow Optimization Estimated*

For purposes of comparison, the Existing Conditions model and the Alternative 1 model use FIRO flows and District Infiltration rates. Scenario 3 uses Stress flows, which may not appear to be a comparable sample to Existing Conditions but rather shows the ability to handle larger flows. Stress flows have been seen to blow out the Existing Conditions model, overtopping existing basins, and intertie channels. Alternative 1 and Alternative 2 can accept the Stress flows but would also release more flows out Basin 53.

Figures 4-1, 4-2, and 4-3 show the optimized results for Alternatives 1, 2, and 3 respectively. These represent an approximation of the maximum potential operation of each alternative.

Next Steps

The results of this TM will be included as part of the project Preliminary Design Report. These results in conjunction with the Estimated Cost Estimates technical memorandum will also be used in the District's Feasibility Study to identify the economic feasibility of these potential alternatives. The selected alternative will be developed to 65% Design Plans.

Figure 3-1. Alternative 1, Optimized STRESS Flow Results Maximum Depth Graphic (District Infiltration Rates)

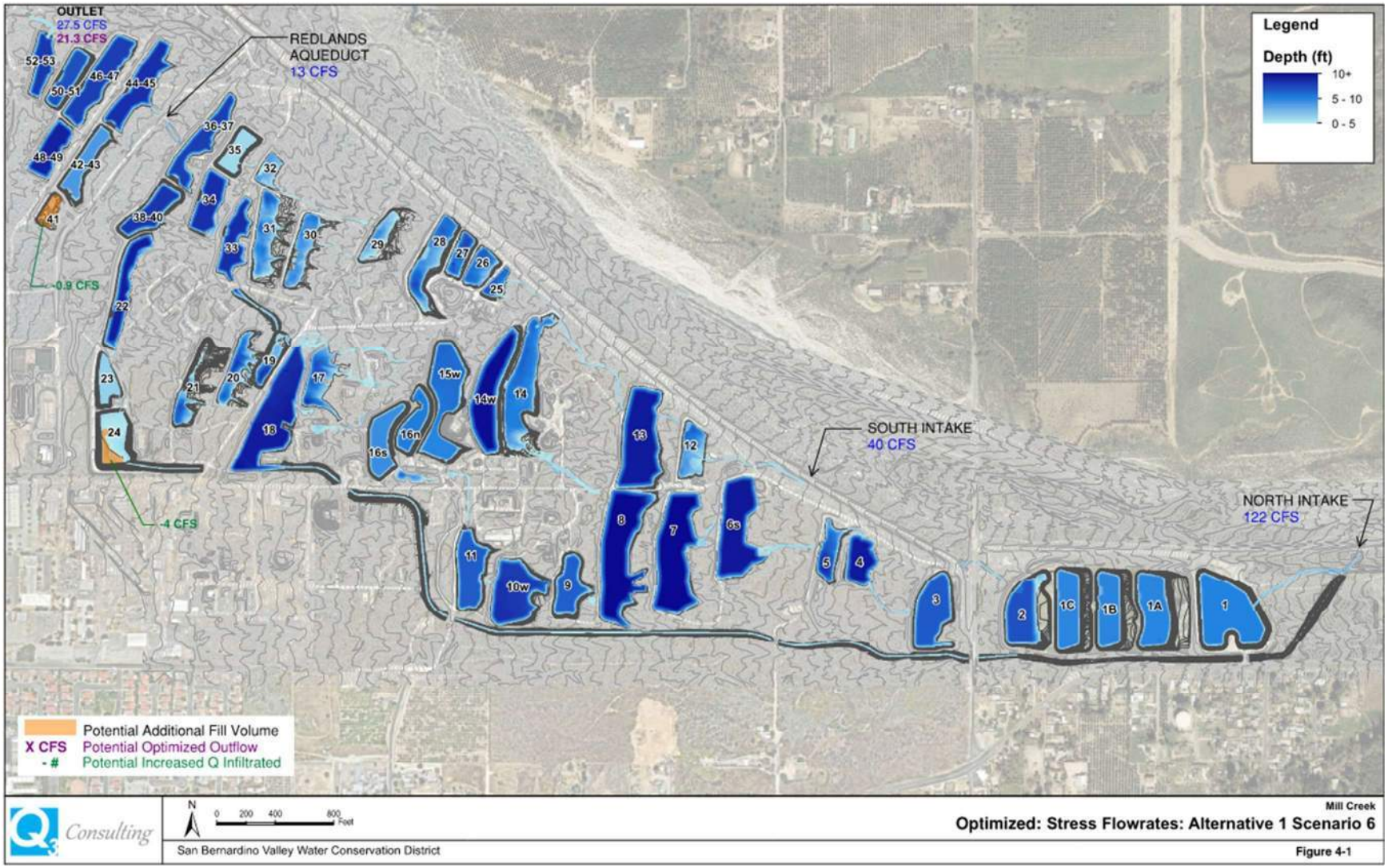


Figure 3-2. Alternative 2, Optimized STRESS Flow Results HEC-RAS Maximum Depth Graphic (District Infiltration Rates)

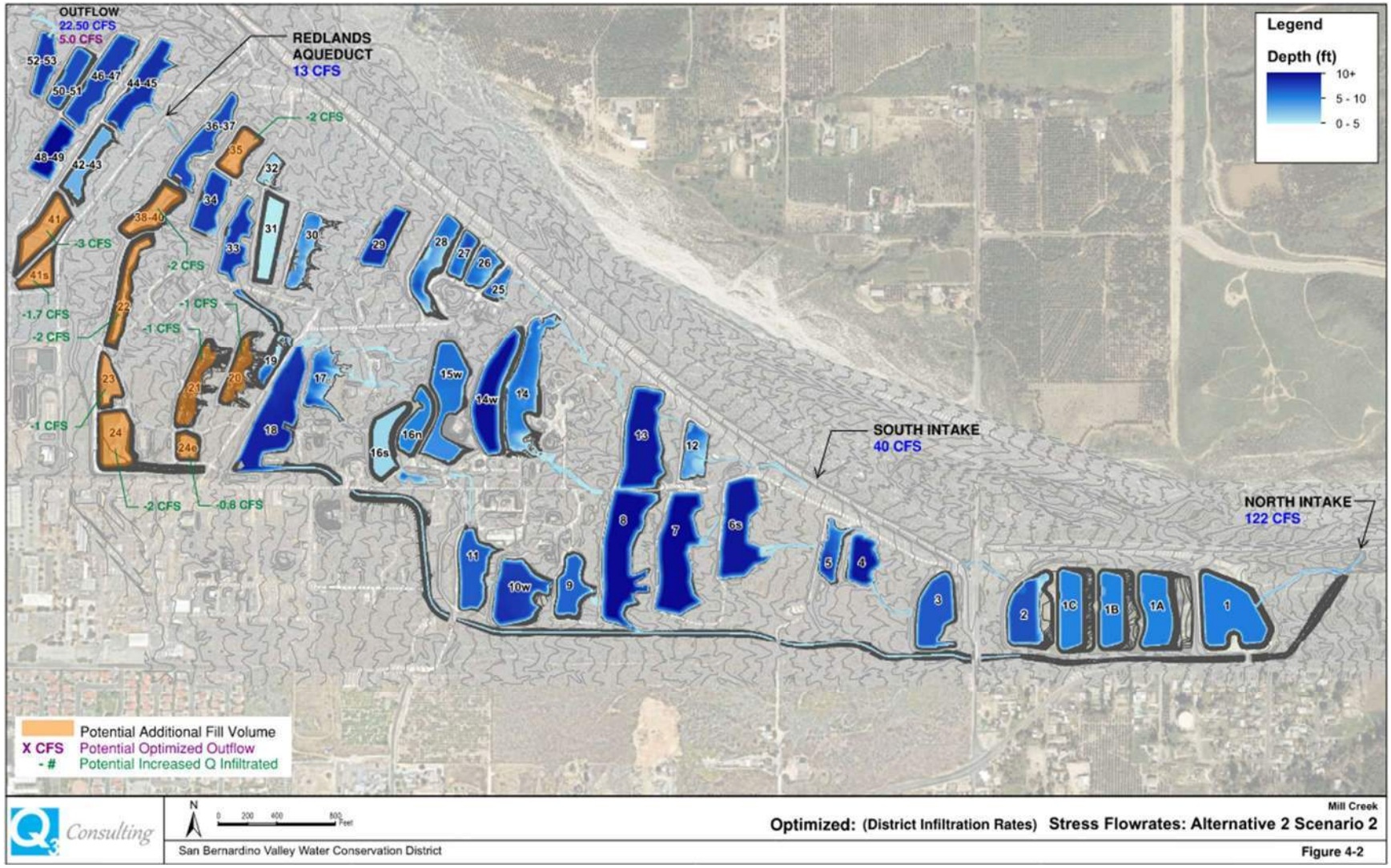
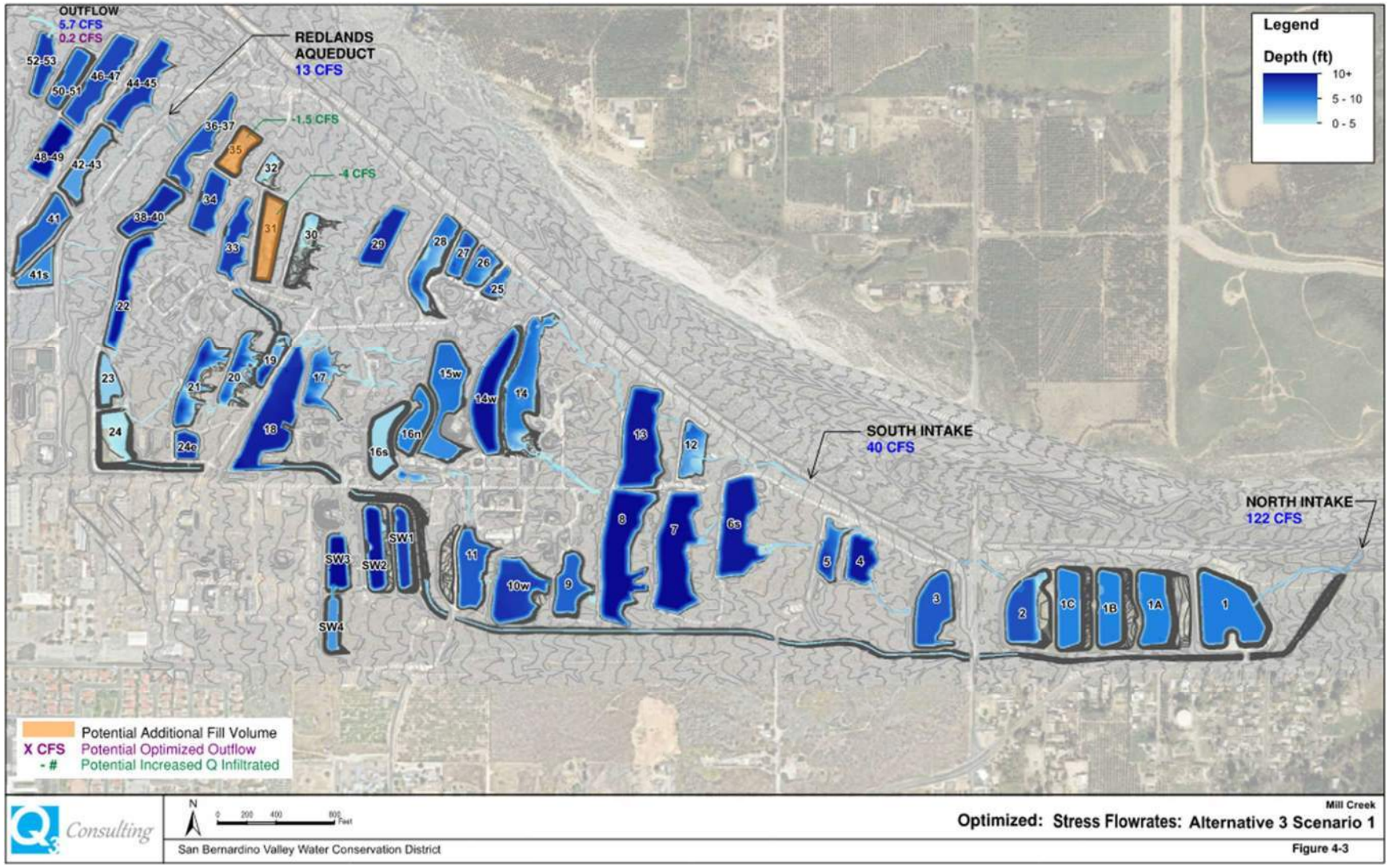


Figure 3-3. Alternative 3, Optimized STRESS Flow Results HEC-RAS Maximum Depth Graphic (District Infiltration Rates)

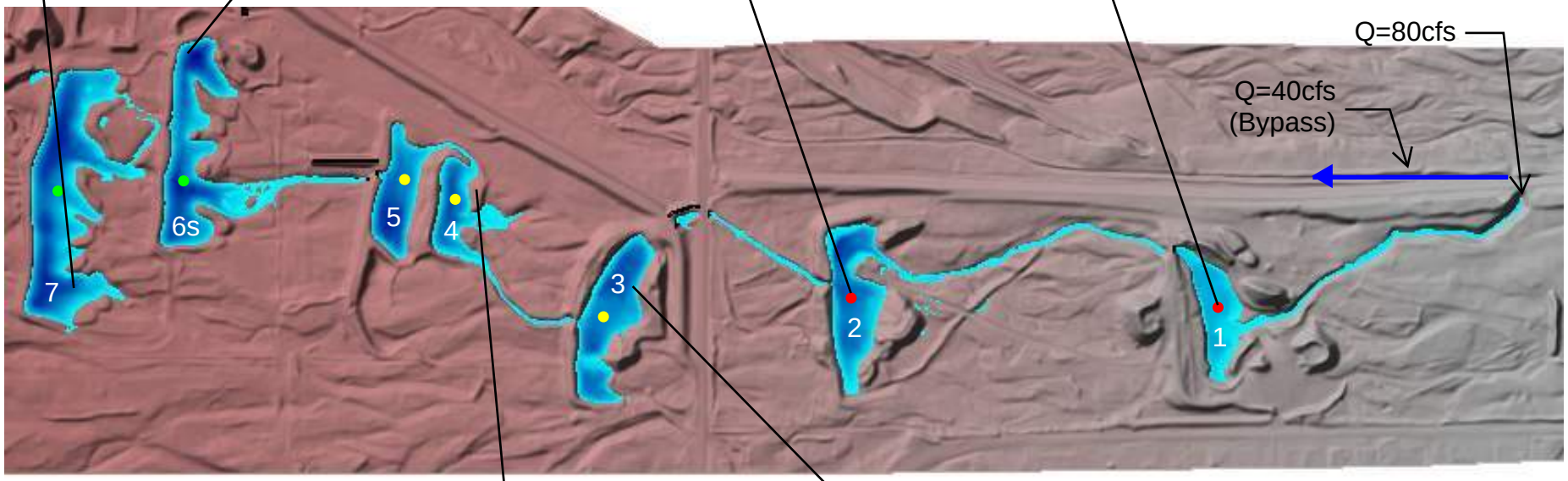


EXHIBITS

Geotechnical Infiltration Rate Schematics

Kleinfelder Infiltration Rates

EAST MODEL WITH INFILTRATION
(CLEAN CONDITIONS)



LEGEND

● = 0'/Day	0"/Hr
● = 1'/Day	0.5"/Hr
● = 2'/Day	1"/Hr
● = 3'/Day	1.5"/Hr
● = 5'/Day	2.5"/Hr
● = 10'/Day	5"/Hr

Testing Site/Method: (RF 6, RF 7, SF 8)

RF = Reduction Factor

MIDDLE MODEL WITH INFILTRATION

PI-5 (2.9, 2.5, 2.2)
Orig. Model: 1.0"/hr
Proposed Rate: 2.5"/hr
(Basins 27, 28)

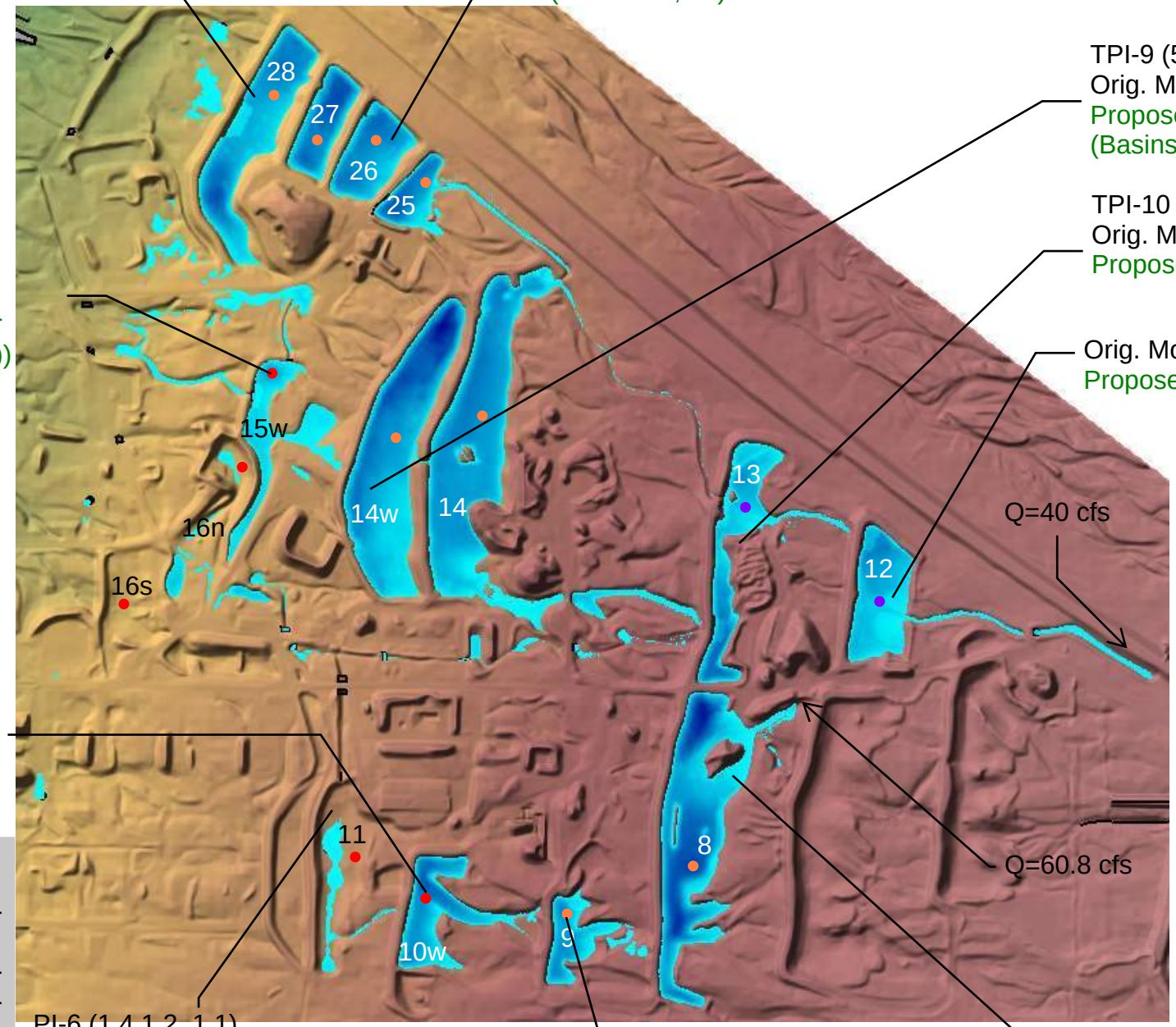
Orig. Model: 0.5"/hr
Proposed Rate: 1.75"/hr
(Basins 25, 26)

TPI-9 (5.0, 4.2, 3.7)
Orig. Model: 0.5"/hr
Proposed Rate: 1.5"/hr
(Basins 14 and 14w)

TPI-10 (3.5, 3.0, 2.6)
Orig. Model: 0"/hr
Proposed Rate: 1.0"/hr

TPI-4 (5.0, 4.2, 3.7)
Orig. Model: 1.0"/hr
Proposed Rate: 2.0"/hr
(Basins 16s, 16n, 15w))

Orig. Model: 0"/hr
Proposed Rate: 0.1"/hr



Q=40 cfs

Orig. Model: 1.0"/hr
Proposed Rate: 1.4"/hr

Q=60.8 cfs

LEGEND

● = 0'/Day	0"/Hr
● = 1'/Day	0.5"/Hr
● = 2'/Day	1"/Hr
● = 3'/Day	1.5"/Hr
● = 5'/Day	2.5"/Hr
● = 10'/Day	5"/Hr

PI-6 (1.4,1.2, 1.1)
Orig. Model: 1"/hr
Proposed Rate: 1.4"/hr

Orig. Model: 0.5"/hr
Proposed Rate: 1.4"/hr

TPI-11 (3.5, 3,2.6)
Orig. Model: 0.5"/hr
Proposed Rate: 1.4"/hr

WEST MODEL WITH INFILTRATION

PI-1 (0.6, 0.6, 0.5)
Orig. Model: 5"/hr
Proposed Rate: 1.0"/hr
(Basins 46, 47)

Orig. Model: 5"/hr
Proposed Rate: 2.5"/hr
(Basins 45, 44, 51)

TPI-1 (6.9, 5.9, 5.2)
Orig. Model: 5"/hr
Proposed Rate: 2.5"/hr
(Basins 41, 42, 43, 50, 53, 52)

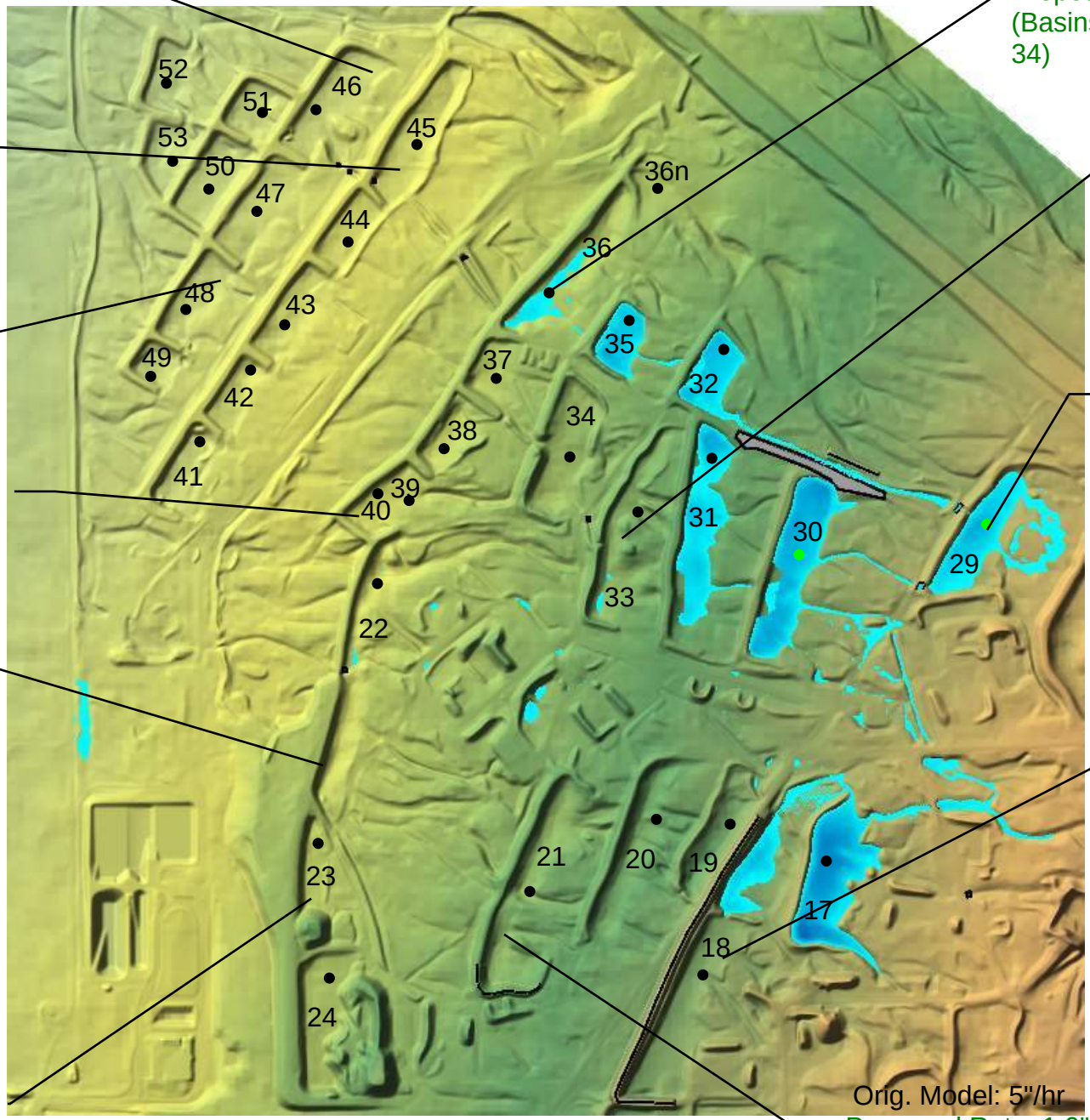
Orig. Model: 5"/hr
Proposed Rate: 2.0"/hr
(Basins 38, 39, 40)

TPI-2 (1.5, 1.3, 1.5)
Orig. Model: 5"/hr
Proposed Rate: 2.0"/hr
(Basin 22)

LEGEND

= 0'/Day	0"/Hr
= 1'/Day	0.5"/Hr
= 2'/Day	1"/Hr
= 3'/Day	1.5"/Hr
= 5'/Day	2.5"/Hr
= 10'/Day	5"/Hr

PI-8 (2.0, 1.3, 1.5)
Orig. Model: 5"/hr
Proposed Rate: 2.0"/hr
(Basins 23, 24)



PI-2 (1.7, 1.5, 1.3)
Orig. Model: 5"/hr
Proposed Rate: 1.7"/hr
(Basins 36, 36n, 37, 34)

PI-11 (1.6, 1.3, 1.2)
Orig. Model: 5"/hr
Proposed Rate: 1.6"/hr
(Basins 30, 31, 32)

Orig. Model: 2.5"/hr
Proposed Rate: 1.5"/hr
(Basins 29, 30)

PI-4 (0.8, 0.6, 0.6)
Orig. Model: 5"/hr
Proposed Rate: 0.8"/hr
(Basins 17, 18)

Orig. Model: 5"/hr
Proposed Rate: 1.0"/hr
(Basins 21, 20, 19)

ATTACHMENT II

Mill Creek Geotechnical Investigation Report

MEMORANDUM

To:	Mr. Tom Ryan, PE Q3 Consulting
From:	Jeffrey S. Walker, PE, GE – Kleinfelder Eric C. Pond, PE – Kleinfelder
Date:	August 1, 2025
Subject:	**DRAFT** Geotechnical Design Recommendations ARTP Mill Creek Diversion and Basins Preliminary Design/Feasibility Study
Figures:	Figure 1 – Vicinity Map Figure 2 – Exploration Location Map Figure 3 – Benching Details
Appendices:	Appendix A – Preliminary Design Drawings Appendix B – Laboratory Test Results Appendix C – Calculations

Kleinfelder previously prepared a Field Investigation and Infiltration Testing Memorandum dated May 23, 2024, for the San Bernardino Valley Water Conservation District's (District's) existing Mill Creek recharge facility located in Mentone, CA (see Figure 1). The generally undeveloped eastern section of the property is bounded by Mill Creek on the north, Valalla Lane and Mill Creek Road on the south, Garnet Street on the west, and San Bernardino Valley Municipal Water District's Tate Pump Station on the east. The larger western portion of the site is bound by Mill Creek on the north, Garnet Street on the east, and the Horace Hinkley Treatment Plant (HHTP) and an orchard on the west. The southern boundary of the property is formed by Madeira Avenue from Garnet Street to approximately 1,000 feet (ft) west of Sapphire Avenue. The boundary turns north at this location to about San Bernardino Avenue, which then serves as the southern boundary extending westward to the HHTP.

Kleinfelder's previous work included a field exploration program to advance thirty-six test pits, with fourteen test pits excavated to evaluate subsurface conditions and twenty-two test pits excavated to evaluate subsurface conditions and to perform infiltration testing. Locations of the test pits are shown on Figure 2. Laboratory grain size distribution testing was performed on samples following removal of particles greater that approximately 3 inches to assist with classification of the soil materials and to evaluate the hydraulic conductivity based on correlations with particle size for comparison with the field infiltration testing. Logs of the test pit excavations, the laboratory testing data, and results of the infiltration testing are provided in the May 23, 2024, memorandum.

Since our initial investigation and testing, we have evaluated the anticipated seepage and stability of the berms associated with the proposed improvements and have developed geotechnical parameters for design of drainage structures associated with the basin infrastructure. Kleinfelder has prepared this memorandum at the request of Q3 and the District to present our preliminary geotechnical recommendations for design and construction where modifications are proposed for existing detention basins and outlet structures; for new basins, diversion structures, and inlet and outlet structures; and for a new conveyance channel as shown on the preliminary design plans presented in Appendix A (consistent with Alternative 3). Kleinfelder's services were performed in general accordance with our approved scope of work.

A summary of our analyses, conclusions and recommendations for design are provided in the following sections of the memorandum.

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1 SEEPAGE AND SLOPE STABILITY ANALYSES

1.1 Basis for Analysis/Design Criteria

1.1.1 Seepage

The maximum allowable exit gradient is 0.5 per U.S. Army Corps of Engineers (USACE) EM 1110-2-1913 Page B-12 (USACE 2000).

1.1.2 Slope Stability Analysis

The minimum required factors of safety for our slope stability analyses are based on the following:

- Static (Downstream): 1.5 per Los Angeles County Public Works (LACPW) Manual for Preparation of Geotechnical Reports Section 3.5.1.2.
- Seismic (Downstream): 1.1 per Los Angeles County Public Works (LACPW) Manual for Preparation of Geotechnical Reports Section 3.5.1.2.
- Rapid Drawdown (Upstream): 1.5 per Los Angeles County Public Works (LACPW) Manual for Preparation of Geotechnical Reports Section 3.3.9 (LACPW, 2013).

The soils encountered at the site are granular with the presence of cobbles and boulders. Drainage from the berms is anticipated to occur as rapidly as any drop in water level within the basin. Risk associated with a delayed dissipation of pore water pressure during a rapid drawdown event is considered negligible. It is our opinion that a rapid drawdown condition is not critical and therefore has not been considered.

1.1.3 Design Ground Water Elevation

The design ground water elevation (DGWE) assumed in our analysis is based on the following:

- Static: The assumed pool elevation is based on 3 feet of freeboard at the upstream face of the berm, and the DGWE is assumed to be at the ground surface below the toe of the slope on the downstream side.
- Seismic: USACE EM 1110-2-1913 Section 6-5.d does not require that the maximum design water surface elevation be assumed for evaluation of the stability during the design earthquake. However, the initial minimum freeboard of 3 feet at the upstream face has been conservatively assumed for our analysis. Where the safety factor under seismic conditions does not meet the minimum criteria based on this initial assumption, an increased freeboard has been assumed and evaluated as discussed herein.

1.1.4 Liquefaction

Liquefaction has not been analyzed for the project site at the time this memorandum was prepared. Additional subsurface investigation (instrumented Becker Penetration Testing) is recommended should liquefaction potential be identified as a concern.

1.2 Soil Properties

The subsurface conditions at the site consist predominantly of poorly graded gravel and cobbles with sand with lesser amounts of well-graded gravel and cobbles with sand, poorly graded sand and gravel, and periodic well graded sands with silt. Cobbles and boulders are present throughout the site (Kleinfelder 2024). Samples collected during the original field investigation were utilized to perform additional laboratory tests to support development of strength parameters for our engineering analyses. The results of these laboratory tests are presented in Appendix B.

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Soil properties used in our analyses are summarized in Table 1. Detailed parameter development is documented in Appendix C.1.

Table 1: Summary of Recommended Design Parameters

Unit Wt. γ	Peak Shear Strength		Ultimate Shear Strength		Permeability
	ϕ'	c'	ϕ_u	c_u	k_v, k_h
(pcf)	(Deg.)	(psf)	(Deg.)	(psf)	cm/s
125	37	200	33	100	0.01

1.3 Seismic Coefficient (k_{eh})

The seismic site class is estimated to be either C or D. The peak ground acceleration (PGA) due to a 2,475-year return period event for site class C and D is 1.13g and 1.04g, respectively. See Appendix C.2 (Attachment 2) for details. Considering one third of the PGA as the horizontal coefficient for evaluation of seismic stability, and assuming deformation of up to 6 inches is tolerable, the horizontal seismic coefficient (k_{eh}) is 0.38g and 0.35g for site class C and D conditions, respectively. Therefore, we have used 0.38g in our analysis.

1.4 Analysis Methodology

Preliminary engineering plans provided by Q3 showing new basins and proposed improvements to existing basins were evaluated to select critical cross-sections for seepage and slope stability analysis. Maps showing the locations of these cross-sections are presented in Appendix C.2 (Attachment 1) along with the cross-sections. Several of the critical cross-sections have similar slope heights and inclinations and have been grouped together, resulting in five critical sections for analysis (Sections A through E). Table 2 summarizes pertinent details regarding each critical cross-section with an asterisk denoting which cross-section was analyzed.

Table 2: Analysis Section Details

Cross-Section Location (Basin Nos.)	Section Name	Slope Height (feet)	Upstream Slope	Downstream Slope	DGWE (feet)	
					Upstream	Downstream
1A-2C	E	22	3H:1V	3H:1V	2057	2038
1B*	B	13	3H:1V	2H:1V	2082	2072
2A	B	12	3H:1V	3H:1V	2029	2020
12*,**	C	12	1.1H:1V	1.25H:1V	1939	1930
14-15W*	A	23	3H:1V	3H:1V	1879	1859
14W	B	13	3H:1V	3H:1V	1851	1841
24*	D	17	2H:1V	3H:1V	1795	1781
35-36*	E	22	3H:1V	3H:1V & 1.75H:1	1799	1780

Note:
*: Cross section analyzed.
**: Existing condition.

The computer program GeoStudio was used to evaluate the seepage and slope stability of the basins. The Spencer and Morgenstern-Price (M-P) methods of analysis were used, as they satisfy both force and moment equilibrium. A circular failure surface search was used for both static and seismic conditions. Peak shear strength was used under static conditions and under seismic conditions during shaking (with seismic coefficient k_{eh}), while the ultimate shear strength was used to model seismic conditions following shaking (without k_{eh}).

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1.5 Analysis Results

Table 3 summarizes the maximum exit gradient at the downstream face and the minimum FS for each section analyzed using the program GeoStudio. The maximum exit gradient at the downstream face is less than 0.5 for all sections analyzed and meets the project design criteria. The minimum FS values meet or exceed the project requirements except under seismic conditions at Sections A and E (shown in red in Table 3). A sensitivity analysis was performed for Sections A and E to evaluate the effectiveness of lowering the freeboard on the FS under seismic shaking conditions. As shown in Table 4, the minimum freeboard required to satisfy the FS criteria for the seismic condition at Sections A and E is 6 feet and 5 feet, respectively. The GeoStudio results are included in Appendix C.2 (Attachment 3).

Table 3: Summary of GeoStudio Results

Analyzed Section	Cross Section	Slope Ht	Downstream Exit Gradient	Minimum Calculated FS					
				Static		Seismic			
						with k_{eh}		without k_{eh}	
		(feet)		M-P	Spencer	M-P	Spencer	M-P	Spencer
A	14-15W	23	0.32	2.47	2.47	**1.03**	**1.03**	1.89	1.89
B	1B	13	0.35	2.61	2.61	1.23	1.23	1.89	1.89
C	12	12	0.47	2.24	2.24	1.26	1.26	1.56	1.56
D	24	17	0.26	2.85	2.85	1.15	1.15	2.15	2.15
E	35-36	22	0.49	2.66	2.66	**1.06**	**1.06**	1.69	1.69

Note: Text in **red** indicates location where the minimum design criteria is not met under assumed design conditions (3 feet of freeboard).

Table 4: Summary of Minimum Required Freeboard at Sections A and E

Analyzed Section	Cross Section	Minimum Required Freeboard at Upstream Slope to Meet FS Under Seismic Conditions	
		Static	Seismic
		(feet)	(feet)
A	14-15W	3	6(1)
E	35-36	3	5(1)

Notes:
1. Based on slope stability analysis for design berm elevations

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2 CONCLUSIONS AND RECOMMENDATIONS

2.1 Detention Basins

Based on evaluation of the preliminary proposed basin modification design as shown in Appendix A, the required FS and exit gradient criteria meet the project requirements detailed in Sections 1.1.1 and 1.1.2 for Sections B through D with 3 feet of freeboard. For Sections A and E, the freeboard must be increased to 6 feet and 5 feet, respectively, to meet the required FS under seismic conditions.

For existing basins, where infiltration will continue but grading is not currently proposed, consideration should be given to flattening the existing slope configurations. Where such basins are fully entrenched and the maximum pool elevation is near or below the downstream toe of the berm, there is little risk for failure. However, for general site safety and overall slope stability, it is recommended that remedial grading be performed to maintain a maximum 2:1 slope configuration at all such locations. Where necessary, outlet structures should be extended to accommodate a maximum 2:1 slope configuration.

2.2 Shallow Foundations

Calculations for bearing capacity and lateral resistance to support shallow and mat foundation and retaining wall design are presented in Appendix C.3.

2.2.1 Bearing Capacity

For design of diversion structures, culverts, headwalls, weirs, and other drainage elements, spread/continuous footings and mat/slab foundations may be designed for net allowable soil bearing pressures of 3,000 and 1,500 pounds per square foot (psf), respectively, for dead plus sustained live loads. The ultimate bearing pressure may be calculated by multiplying the allowable bearing pressure by factor of 3. A one-third increase in the above bearing pressures can be used for short term load conditions for wind or seismic loads.

Foundations should be embedded at least 18 inches below the lowest adjacent grade or as deep as needed to protect against scour. Scour potential should be evaluated by the civil engineer and footing depth established accordingly. The minimum recommended spread footing size is 3 feet by 3 feet while the minimum recommended continuous footing size is 2 feet.

We estimate the total static settlement for spread footings and mat/slab foundations designed and constructed in accordance with the recommendations presented above to be 1 inch or less.

2.2.2 Lateral Resistance

Lateral load resistance may be derived from passive resistance along the vertical sides of foundations, friction acting at the base of foundations, or a combination of the two. An allowable passive resistance of 300 psf per foot of depth may be used for design. Allowable passive resistance values should not exceed 3,000 psf. An allowable coefficient of friction value of 0.4 between the base of foundations and subgrade soils can be used for sliding resistance using the dead load forces. Friction and passive resistance may be combined without reduction. We recommend that the first foot of soil cover be neglected in the passive resistance calculations if the ground surface is not protected from erosion or disturbance by a slab, pavement, or in a similar manner. A one-third increase in the above lateral resistance can be used for short term load conditions for wind or seismic loads.

2.2.3 Subgrade Reaction Modulus of Mat/Slab Foundation

A subgrade reaction modulus, k_1, of 150 pounds per cubic inch (pci) may be used for design of a mat foundation supported on engineered fill or native soil. This recommended modulus is for a

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1-foot-square plate and should be adjusted for the actual size of the foundation element using appropriate formulas as provided below, or an acceptable equivalent structural analysis method or software application. The modulus of vertical subgrade reaction (k_v) may be estimated as follows:

$$k_v = k_1 \left(\frac{m + 0.5}{1.5m}\right)\left(\frac{B + 1}{2B}\right)^2$$

where, B is the width of the foundation in feet and, m is the foundation length (L) to width (B) ratio, m = L/B.

2.3 Retaining Structures/Below Grade Walls

Design earth pressures for retaining structures, below grade walls, and temporary shoring depend primarily on the allowable wall movement, wall inclination, type of backfill materials, backfill slopes, surcharges, and drainage. Earth pressures provided assume that granular (sandy) soils will be used as backfill. If a drainage system is not installed, the wall should be designed to resist hydrostatic pressure in addition to the earth pressure. Determination of whether the active or at-rest condition is appropriate for design will depend on the flexibility of the walls/shoring. Walls/shoring that are free to rotate at least 0.002 radians (deflection at the top of the wall of at least 0.002 x H, where H is the unbalanced wall height) may be designed for the active condition. Walls/shoring that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The recommended active and at-rest earth pressures and passive resistance values are provided in Table 5.

Table 5: Lateral Earth Pressures for Below Grade Walls/Shoring (Granular Sandy Soils)

Wall Movement	Backfill Condition	Equivalent Fluid Pressure (pcf)	Seismic Increment (pcf)
Free to Deflect (active condition)	Level	31	34
Restrained (at-rest condition)		50	N/A

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The above lateral earth pressures do not include the effects of surcharges (e.g., traffic, footings), compaction, or truck-induced wall pressures. Any surcharge (live load, including traffic, or dead load) located within a 1:1 (horizontal to vertical) plane drawn upward from the base of the excavation should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind walls (or shoring) with a level backfill (or ground) condition may be calculated by multiplying the surcharge by 0.25 for cantilevered walls/shoring under active conditions and 0.40 for restrained walls/shoring under at-rest conditions. Walls/shoring adjacent to areas subject to vehicular traffic including maintenance vehicles should be designed for a 2-foot equivalent vertical soil surcharge (250 psf). Lateral load contributions from other surcharges located behind walls/shoring should also be included where present.

Lateral load resistance may be derived from passive resistance along the vertical sides of foundations, friction acting at the base of foundations, or a combination of the two. An allowable passive resistance of 300 psf per foot of depth may be used for design of walls and shoring. Allowable passive resistance values should not exceed 3,000 psf. An allowable coefficient of friction value of 0.5 between the base of foundations and subgrade soils can be used for sliding resistance using the dead load forces. Friction and passive resistance may be combined without reduction. We recommend that the first foot of soil cover be neglected in the passive resistance calculations if the ground surface is not protected from erosion or disturbance by a slab,

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pavement, or in a similar manner. A one-third increase in the above lateral resistances can be used where it is required to design for short term load conditions for wind or seismic loads.

Walls should be properly drained or designed to resist hydrostatic pressures. Adequate drainage is essential to provide a free-draining backfill condition so that there is no hydrostatic buildup behind the wall. Drainage may consist of a manufactured drain panel. This system generally consists of a prefabricated drain panel lined with filter fabric. At the wall base, we recommend that a gravel drain be installed to collect and discharge drainage to a suitable outlet. The drain should consist of a 4-inch-diameter perforated PVC pipe placed, with the perforations down, at the bottom of approximately 2 cubic feet of clean gravel per foot of wall length. The gravel drain should be wrapped in filter fabric (Mirafi 160N or equivalent). The pipe should be sloped to drain to a suitable outlet or a low spot and be allowed to percolate naturally. If drainage behind the wall is omitted, the wall should be designed for full hydrostatic pressure. The design of any drain panel system should be submitted to Kleinfelder for review to check that our recommendations have been properly incorporated into the design. Installation of the drainage system should be reviewed and documented by a Kleinfelder representative.

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3 CONSTRUCTION CONSIDERATIONS

3.1 Grading and Site Preparation

3.1.1 General

Site preparation and earthwork operations should be performed in accordance with applicable codes, safety regulations, other local, state, or federal specifications, and the recommendations included in this report. References to maximum unit weights are established in accordance with the latest version of ASTM D1557.

3.1.2 Site Preparation

Abandoned utilities and other existing improvements, if any, within the proposed project area should be completely removed and resultant excavation(s) backfilled with compacted fill placed as recommended below. Debris produced by demolition operations should be separated and disposed of off-site. Existing pipelines or conduits that extend beyond the limits of the proposed construction, and are to be abandoned in place, should be plugged with non-shrink cement grout to prevent migration of soil and/or water through the abandoned structures. Demolition, disposal, and grading operations should be observed and tested by a representative of the geotechnical engineer-of-record.

Areas within grading limits (cut and/or fill) should be cleared and grubbed to remove all vegetation, root masses, and large roots to ¼-inch diameter. Following clearing and grubbing, all dry, loose, or soft earth materials, undocumented fill, previously tilled or disturbed soils, organics, caliche nodules, or deleterious materials should be excavated to the satisfaction of the geotechnical engineer-of-record.

Prior to grading and subgrade preparation, all vegetation should be cut and removed from the site. Roots and vegetative matter more than one inch should be removed by screening or raking. It should be noted that significant amounts of oversized material (greater than 3 inches) was observed during the test pit investigations (Kleinfelder 2024). Oversize material should be stockpiled and removed from the work area or processed for reuse as engineered fill. After grubbing, the surface soils may be moisture-conditioned and recompacted as engineered fill.

3.1.3 Subgrade Preparation:

Prior to placement of fill within berm or overexcavation areas, the base of the approved subgrade should be scarified, moisture conditioned to near optimum, and recompacted to at least 90 percent relative compaction in accordance with ASTM D1557.

3.1.4 Embankment/Berm Construction:

Where fills will be constructed to extend existing embankments, the new fill should be keyed into the existing prepared subgrade soils at the toe of the proposed slope. The width of the keyway should be equivalent to the existing slope height (minimum width of eight feet and maximum width of fifteen feet) and have a depth of at least three feet at the downslope side. The key should be inclined at least 2 percent into the slope. The embankment should then be placed and compacted in uniform 8-inch lifts benched into the face of the existing embankment slope, with minimum bench widths of four feet and maximum bench heights of four feet or as needed to maintain stability of the existing slope. A schematic of the benching requirements is shown on Figure 3. All fill materials should be placed and compacted in accordance with the recommendations in Section 3.3.

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3.1.5 Foundation Excavation

It is anticipated that shallow foundations will be used for the diversion structures, RCB culverts, and any other structures required. Following excavation to the foundation subgrade elevations, the exposed subgrade should be observed by a representative of the geotechnical engineer to evaluate the presence of satisfactory materials at design elevations. If unsatisfactory material, such as soft or disturbed soil, debris, or otherwise unsuitable soil is present at the base of footing excavations, it should be overexcavated to the depth determined by the geotechnical engineer and replaced with structural concrete, 2-sack sand-cement slurry, or engineered fill.

At a minimum, the upper 8 inches of soil below the bottom of the foundation should be scarified, moisture-conditioned to within 2 percent of the optimum moisture content and recompacted to at least 90% of the maximum dry density. Oversized material greater than 3 inches in size should be removed from the scarified zone prior to compaction.

3.2 Excavation Characteristics and Oversized Material

The on-site soils contain a large portion of oversized materials, including cobbles and boulders at the surface and throughout the subsurface profile. Excavations for foundations, utility trenches, and general site grading are expected to encounter oversized materials, which will likely require the use of heavy-duty earthwork equipment. The contractor should anticipate difficult excavations due to the oversized materials throughout the project site.

3.3 Fill Material and Compaction Requirements

The on-site soil, minus debris, organic matter, oversize cobbles and boulders, or other deleterious materials may be used as engineered fill. Cobbles and boulders that cannot be crushed or screened to 3 inches in size or less should be removed from the work area.

3.3.1 Berm and Structure Backfill

Onsite soil can be used as engineered fill for the berms and backfill behind concrete structures it meets the gradation requirements indicated above. Fill soils should be free of debris, organic matter, or other deleterious materials. Berm fill should be placed in lifts not to exceed eight inches in loose thickness or the maximum thickness that the compaction tool can effectively compact, whichever is less, mixed with a disk or harrow to blend the fill and break up the surface of the previously compacted lift, moisture conditioned to within approximately 2% of optimum, and compacted to at least 90 percent of the maximum dry density. Each loose lift of fill should be well-mixed such that layers of different soil types are not continuous. If lamination of the fill lifts is observed during quality control testing of the fill, the loose lift will need additional disking to improve the bond between layers.

Import soils, though not expected to be needed, should have no particles greater than 3 inches in maximum dimension. The contractor should provide documentation that all imported soil is free of hazardous materials, including petroleum or petroleum byproducts, chemicals, and harmful minerals. Test results with the geotechnical and analytical properties of the proposed import material should be provided to the Geotechnical Engineer-of-Record for review and approval prior to transportation and use on site.

3.3.2 Top of Berm Access Roads

The berm crest may serve as an access road for light trucks and occasional maintenance vehicles. We recommend that the upper 12 inches of the berm fill on the crest be compacted to a minimum 95 percent relative compaction. A minimum four-inch-thick layer of Aggregate Base (in accordance with Caltrans Standard Specification or Greenbook Specification) can also be placed as a wearing surface on the prepared berm subgrade (compacted to 95 percent relative

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compaction). If more frequent heavy traffic is anticipated, an asphalt concrete pavement section can also be designed.

3.3.3 Pipe Bedding

Bedding material for pipelines is dependent on the pipe material and the subgrade conditions. Subject to the approval of the Civil Engineer, bedding in areas that will not become saturated should consist of onsite granular soil or crushed rock (maximum particle size of 1 inch) wrapped in a filter fabric to reduce to potential for migration of fines into the trench bottom and sidewalls. In areas where pipe bedding can become saturated, such as the laterals extending into or between basins, we recommend that controlled low-strength material (CLSM) bedding be utilized. In lieu of CLSM, a sand-cement slurry meeting the compressive strength requirements for CLSM may be used. Granular pipe bedding should be moisture conditioned to within approximately 2% of optimum and compacted to at least 90 percent of the maximum dry density.

3.3.4 Pipe Zone Backfill

Pipe zone backfill, defined as backfill above pipe bedding to a point one foot above the crown of the pipe, will depend on the type of pipe being installed and is subject to the same restrictions as pipe bedding with respect to trench saturation. For reinforced concrete pipe, onsite soils (maximum particle size of 1 inch) may be used as pipe zone backfill, subject to the approval of the Civil Engineer. Granular pipe zone backfill, if utilized, should be placed on both sides of the pipe at the same time to avoid shifting or deforming the pipe. Pipe zone backfill should be placed no higher than the spring line prior to compaction of the backfill. Pipe zone backfill should be placed in lifts not to exceed eight inches in thickness, moisture conditioned to within approximately 2% of optimum, and compacted to at least 90 percent of the maximum dry density. Care should be taken to ensure that the pipe haunches are well compacted. CLSM should be used as pipe zone backfill for pipes extending into or between basins and should be extended vertically to at least a foot above the maximum recharge pool elevation.

3.3.5 CLSM

CLSM for use as bedding and pipe zone backfill should be in accordance with the Greenbook Specification Section 201-6, with a 28-day unconfined compressive strength between 50 and 200 psi. Onsite material (screened to remove oversize particles per the Greenbook Specification) is expected to be suitable for use in CLSM.

3.3.6 Trench Zone Backfill

Trench zone backfill can be defined as backfill above the pipe zone to the subgrade surface. Trench zone backfill should meet the specifications for berm fill. Trench zone backfill should be placed and compacted to at least 90 percent of the maximum dry density within approximately 2% of optimum moisture content. Jetting of trench zone backfill should not be permitted. As noted in Section 3.3.4, CLSM should be used as trench zone backfill for pipes extending into or between basins to at least a foot above the maximum recharge pool elevation.

3.3.7 Drainage Aggregate

If needed, drainage aggregate should consist of hard, durable, clean sand, gravel, ¾- to 1-inch crushed stone, and shall be free from organic material, clay balls, or other deleterious substances and conform to Caltrans Standard Specification for Class 1 or 2 permeable material.

3.3.8 Filter Fabric

Filter fabric for separation of fill and oversized material such as drainage aggregate should consist of non-woven geotextile such as Mirafi 160N or approved equal.

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3.4 Temporary Slopes and Excavations

All temporary excavations must comply with applicable local, state, and federal safety regulations, including California Occupational Safety and Health Act (OSHA) requirements. The responsibility for excavation safety and stability of temporary construction slopes lies solely with the contractor. Depending on the excavation depths, temporary shoring may be required in certain areas to protect workers' safety, existing facilities, adjacent roads, to minimize the size of excavations, and other reasons. The contractor should retain a geotechnical engineer to provide any necessary analysis or consultation related to the excavations and shoring. Kleinfelder is providing this information below solely as a service to our client. Under no circumstances should the information provided be interpreted to mean that owner or designer is assuming responsibility for engineering of excavations or shoring, construction site safety, or the contractors' activities; such responsibility is not being implied and should not be inferred.

For planning purposes and field observation, excavations in the granular on-site soils should be sloped at 1½H (horizontal):1V (vertical) or flatter for excavations less than 20 feet deep. In temporary excavations, minor sloughing and/or raveling should be anticipated as the soils dry out. At the first signs of slope instability, the contractor's geotechnical engineer and the owner's representative should be contacted. The contractor's geotechnical engineer should observe the excavations so that modifications can be made to the excavations, as necessary, based on variations in the encountered soil conditions. All applicable excavation safety requirements and regulations, including OSHA requirements, should be met.

Where sloped excavations or shoring are used, tops of the slopes should be barricaded so that vehicles, stockpiled materials, and storage loads do not impact the stability of the excavation, as determined by the contractor's geotechnical engineer. Surcharge loads should not be permitted within five feet of the top of excavation or within a horizontal distance equal to the excavation depth, whichever is greater, unless the excavation is properly shored. Greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. The contractor's geotechnical engineer should be advised of such heavy vehicle loadings so that specific setback requirements can be established. The contractor should implement measures along the tops of the slopes to reduce runoff that may enter the excavation and erode the slope faces.

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It is important to note that the soil and groundwater conditions may vary significantly at the structure locations. The contractor should verify that similar conditions exist throughout the proposed area of excavation. If different subsurface conditions are encountered at the time of construction, Kleinfelder should be notified immediately to evaluate the unanticipated conditions.

The project specifications should not allow excavations within a 1½ H:1V plane extending outward and downward from a horizontal distance of 2 feet beyond the bottom outer edge of existing improvements to be attempted without bracing and/or underpinning the improvements.

Vertical or steeply sided trench excavations should not be attempted without proper shoring or bracing. All trench excavations should be braced and shored in accordance with good construction practice and all applicable safety ordinances and codes. Trench shields should not be utilized as shoring. Stockpiled (excavated) materials should be placed no closer to the edge of an excavation than a distance equal to 1½ times the depth of the excavation, but no closer than 4 feet.

3.5 Dewatering

Based on the material encountered at the site, dewatering is not expected to be an issue during construction.

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3.6 Construction Monitoring

We recommend that a qualified Geotechnical Engineer or an experienced technician under the direction of the Geotechnical Engineer be present during excavation and grading to confirm that the Contractor complies with the intent of these recommendations. Detailed recommendations for additional services are presented in Section 4.

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4 ADDITIONAL SERVICES

4.1 Final Design and Plan and Specification Review

If this project is advanced to final design by Q3, Kleinfelder should be retained to review any changes to the preliminary design to verify that the recommendations contained in this memorandum remain applicable to the project. It is recommended that Kleinfelder perform a general review of the project plans and specifications before they are finalized for bid and construction to verify that our geotechnical recommendations have been properly interpreted and implemented during design. If Kleinfelder is not retained to support final design, the new geotechnical consultant will need to take over as Geotechnical Engineer-of-Record, and Kleinfelder will assume no responsibility for use of the recommendations contained in this report.

4.2 Construction Observation and Testing

The construction process is an integral design component with respect to the geotechnical aspects of a project. Because geotechnical engineering is an inexact science due to the variability of natural processes, and because sampling is only performed on a limited portion of the soils affecting the performance of the proposed improvements, unanticipated or changed conditions can be encountered during grading. Proper geotechnical observation and testing during construction are imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, it is recommended that Kleinfelder be retained during the construction of the proposed improvements to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes if subsurface conditions or methods of construction differ from those assumed while completing this study.

Our services are typically needed at the following stages of construction:

- after clearing and grubbing and during initial overexcavation.
- during excavation of key trenches and overexcavation and slope benches.
- during grading activities, selective mixing and blending of the berm fill material.
- during fill placement and compaction of the berm fill material.
- after trench excavation to check trench subgrade.
- during utility trench backfill.
- during base placement and compaction.
- after excavation for foundations

If Kleinfelder is not retained to provide services during construction, we will not be responsible for misinterpretation of our recommendations.

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5 LIMITATIONS

Kleinfelder's work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data known to date. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

If another consultant advances this project to final design, the new consultant should retain a geotechnical consultant to review the geotechnical aspects of the final design and perform additional investigation, testing, and engineering analysis as necessary to take over as Geotechnical Engineer-of-Record for the project. Kleinfelder will assume no responsibility for use of the recommendations contained in this report if we are not retained to review the project final design.

It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Land use, site conditions, or other factors may change over time, and additional work may be required with the passage of time. Any party, other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use and the nature of the new project, Kleinfelder may require additional work be performed and an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized person.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site. This report may be used only by the client and the registered design professional in responsible charge and only for the purposes stated, within a reasonable time from its issuance, but in no event later than two years from the date of the report.

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REFERENCES

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Kleinfelder (KLF), (2024). Summary of Field Investigation and Infiltration Testing ARTP Mill Creek Diversion and Basins Preliminary Design and Feasibility Study, May 23.

Los Angeles County Public Works (LACPW), Geotechnical and Materials Engineering Division (2013). *Manual for Preparation of Geotechnical Reports.*

U.S. Army Corps of Engineers (USACE), (2000). *Design and Construction of Levees (EM 1110-2-1913).*

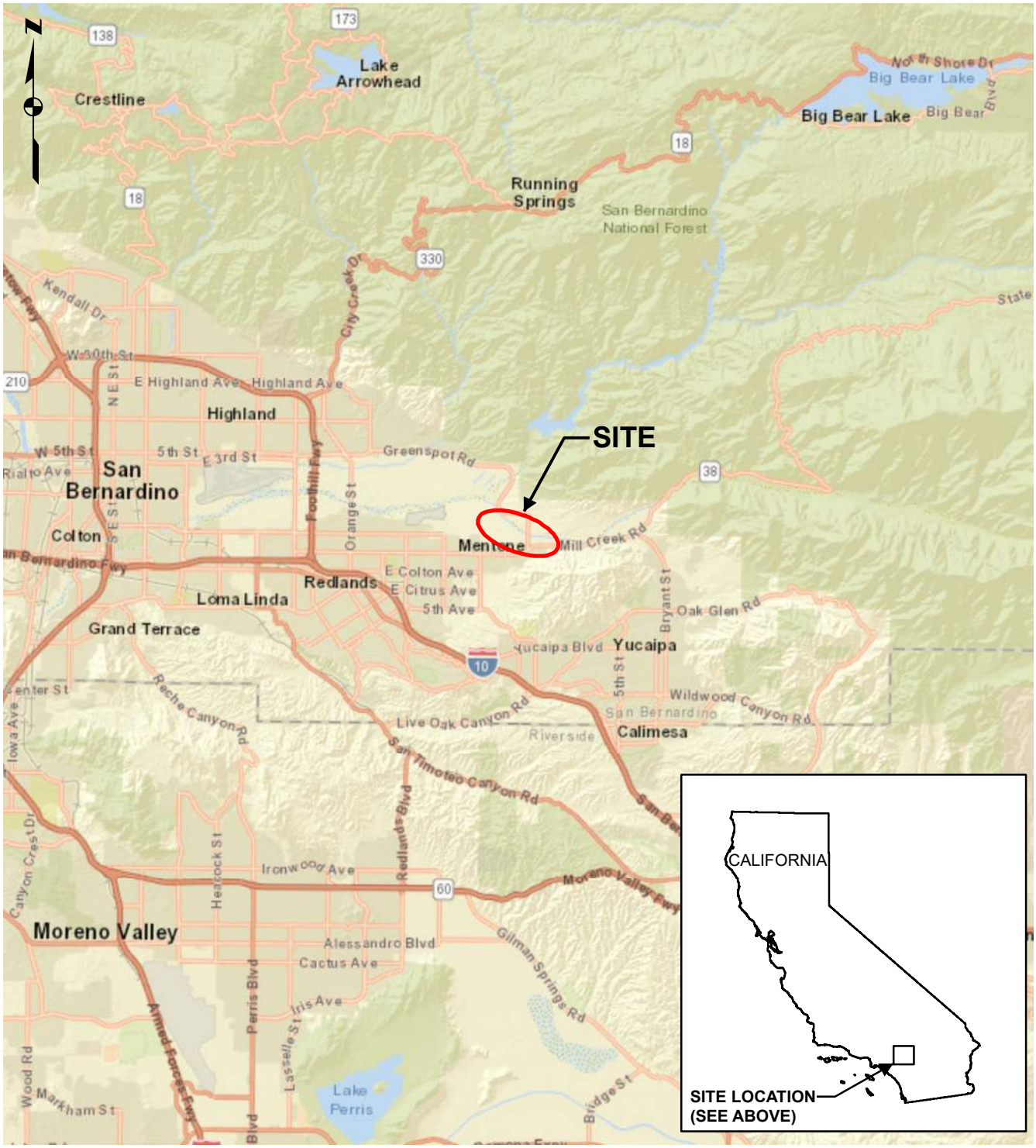
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FIGURES

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DRAWN BY:	JP
CHECKED BY:	JZ
FILE NAME:	20230500_Vic.mxd

SITE VICINITY MAP

ARTP MILL CREEK DIVERSION AND BASINS
PRELIMINARY DESIGN AND FEASIBILITY STUDY
MENTONE, CALIFORNIA

FIGURE
1

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Mill Creek PERC
Geotechnical Testing Locations

Coordinate System:
NAD 1983 StatePlane California V FIPS 0405 Feet
Projection: Lambert Conformal Conic
Datum: North American 1983
Source: SBVWCD. CASIL, SBVMWD
GIS Contact: Laura Torres
M:\Active Recharge\MC ARTP (PERC)\Maps\Map for Geotech Work
August 08, 2023

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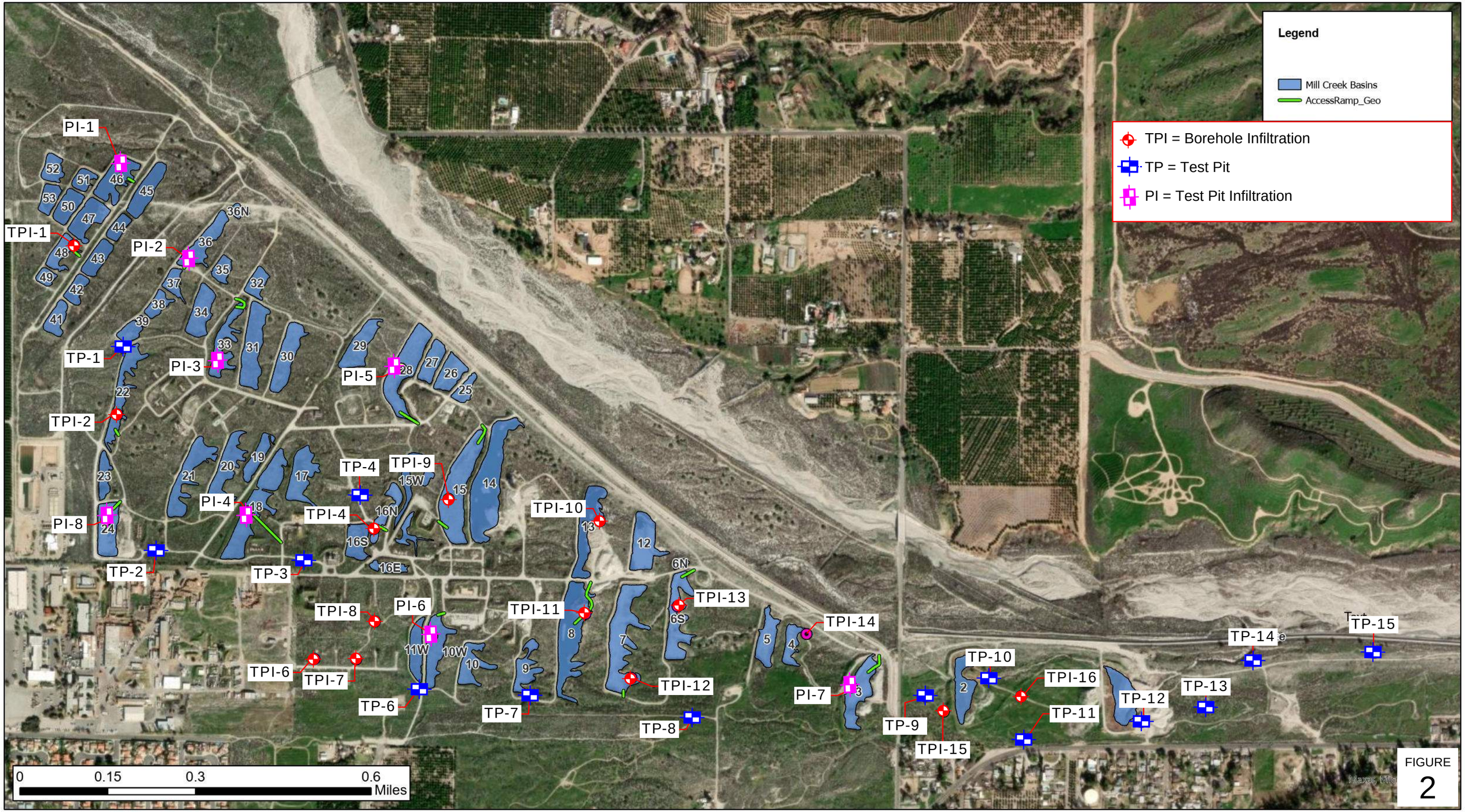
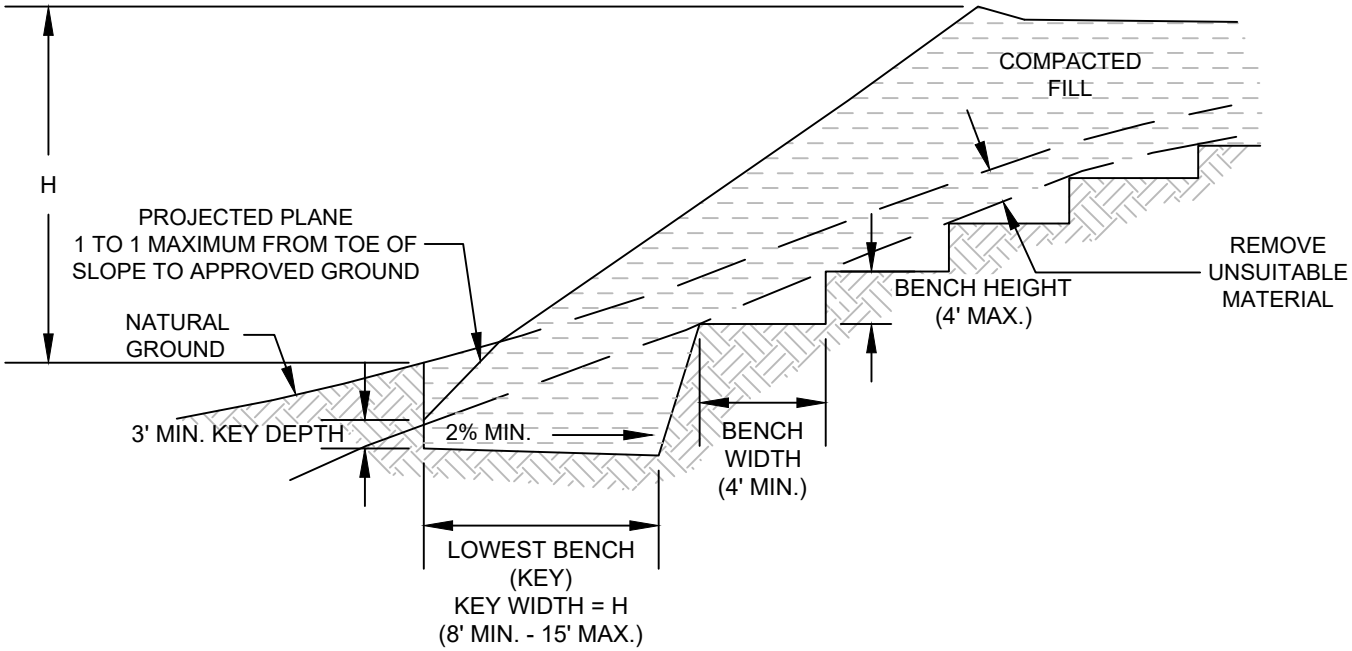


FIGURE
2

PLOTTED: 7/31/2025 6:54 PM BY: dean fahrney

BENCHING DETAILS



NOTE:

1) LOWEST BENCH: DEPTH AND WIDTH SUBJECT TO FIELD CHANGE BASED ON CONSULTANT'S INSPECTION.

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CHECKED BY:	JW
DATE:	07/2025
REVISED:	-

BENCHING DETAILS

ARTP MILL CREEK DIVERSION AND BASINS
PRELIMINARY DESIGN AND FEASIBILITY STUDY
MENTONE, CALIFORNIA

FIGURE

3

APPENDIX A

Preliminary Design Drawings

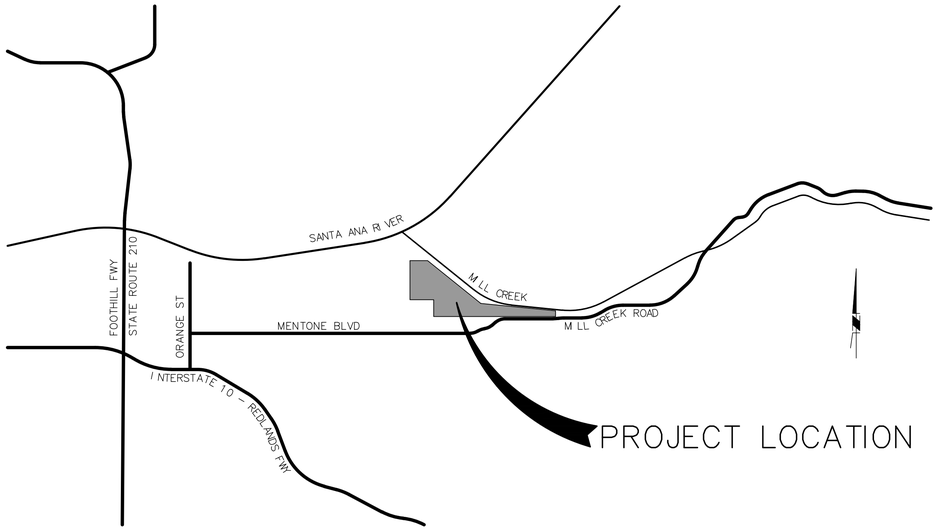
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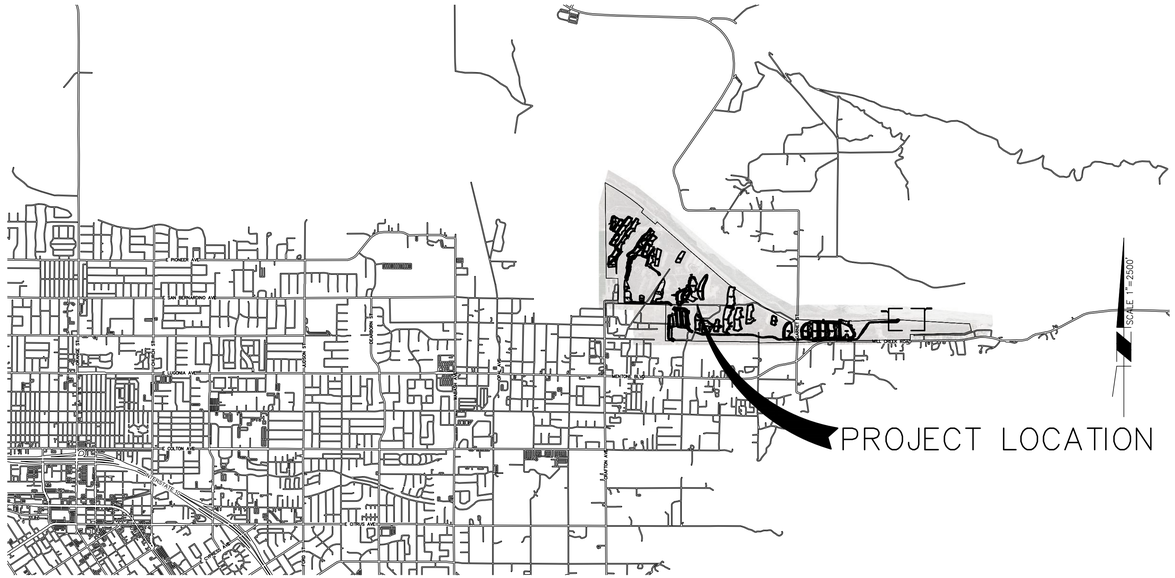
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ACTIVE RECHARGE RECHARGE PROJECT

MILL CREEK



VICINITY MAP
NTS



LOCATON MAP

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Section 4216/4216.9 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert 811 two working days before you dig

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DESIGNED BY:

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ATRP MILL CREEK
PRELIMINARY DESIGN
076-RG-00 - 01

PROJECT NO.
SBVWCD -
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SHEET 01
OF 34

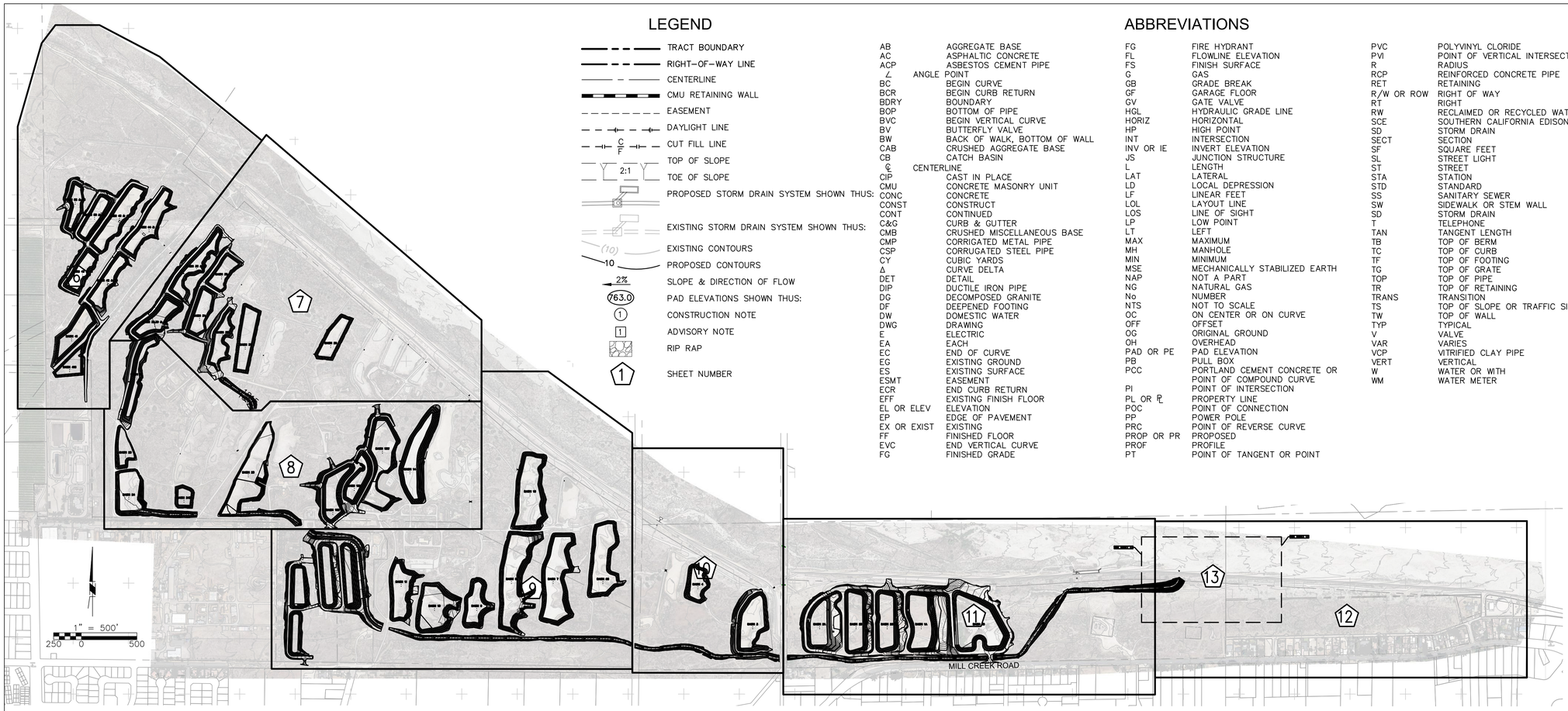
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LEGEND

- TRACT BOUNDARY
- RIGHT-OF-WAY LINE
- CENTERLINE
- CMU RETAINING WALL
- EASEMENT
- DAYLIGHT LINE
- CUT FILL LINE (C / F)
- TOP OF SLOPE
- TOE OF SLOPE (2:1)
- PROPOSED STORM DRAIN SYSTEM SHOWN THUS:
- EXISTING STORM DRAIN SYSTEM SHOWN THUS:
- EXISTING CONTOURS (10)
- PROPOSED CONTOURS 10
- SLOPE & DIRECTION OF FLOW 2%
- PAD ELEVATIONS SHOWN THUS: 763.0
- CONSTRUCTION NOTE (1)
- ADVISORY NOTE [1]
- RIP RAP
- SHEET NUMBER (1)

ABBREVIATIONS

AB	AGGREGATE BASE
AC	ASPHALTIC CONCRETE
ACP	ASBESTOS CEMENT PIPE
∠	ANGLE POINT
BC	BEGIN CURVE
BCR	BEGIN CURB RETURN
BDRY	BOUNDARY
BOP	BOTTOM OF PIPE
BVC	BEGIN VERTICAL CURVE
BV	BUTTERFLY VALVE
BW	BACK OF WALK, BOTTOM OF WALL
CAB	CRUSHED AGGREGATE BASE
CB	CATCH BASIN
℄	CENTERLINE
CIP	CAST IN PLACE
CMU	CONCRETE MASONRY UNIT
CONC	CONCRETE
CONST	CONSTRUCT
CONT	CONTINUED
C&G	CURB & GUTTER
CMB	CRUSHED MISCELLANEOUS BASE
CMP	CORRIGATED METAL PIPE
CSP	CORRUGATED STEEL PIPE
CY	CUBIC YARDS
Δ	CURVE DELTA
DET	DETAIL
DIP	DUCTILE IRON PIPE
DG	DECOMPOSED GRANITE
DF	DEEPENED FOOTING
DW	DOMESTIC WATER
DWG	DRAWING
E	ELECTRIC
EA	EACH
EC	END OF CURVE
EG	EXISTING GROUND
ES	EXISTING SURFACE
ESMT	EASEMENT
ECR	END CURB RETURN
EFF	EXISTING FINISH FLOOR
EL OR ELEV	ELEVATION
EP	EDGE OF PAVEMENT
EX OR EXIST	EXISTING
FF	FINISHED FLOOR
EVC	END VERTICAL CURVE
FG	FINISHED GRADE
FG	FIRE HYDRANT
FL	FLOWLINE ELEVATION
FS	FINISH SURFACE
G	GAS
GB	GRADE BREAK
GF	GARAGE FLOOR
GV	GATE VALVE
HGL	HYDRAULIC GRADE LINE
HORIZ	HORIZONTAL
HP	HIGH POINT
INT	INTERSECTION
INV OR IE	INVERT ELEVATION
JS	JUNCTION STRUCTURE
L	LENGTH
LAT	LATERAL
LD	LOCAL DEPRESSION
LF	LINEAR FEET
LOL	LAYOUT LINE
LOS	LINE OF SIGHT
LP	LOW POINT
LT	LEFT
MAX	MAXIMUM
MH	MANHOLE
MIN	MINIMUM
MSE	MECHANICALLY STABILIZED EARTH
NAP	NOT A PART
NG	NATURAL GAS
No	NUMBER
NTS	NOT TO SCALE
OC	ON CENTER OR ON CURVE
OFF	OFFSET
OG	ORIGINAL GROUND
OH	OVERHEAD
PAD OR PE	PAD ELEVATION
PB	PULL BOX
PCC	PORTLAND CEMENT CONCRETE OR POINT OF COMPOUND CURVE
PI	POINT OF INTERSECTION
PL OR ℄	PROPERTY LINE
POC	POINT OF CONNECTION
PP	POWER POLE
PRC	POINT OF REVERSE CURVE
PROP OR PR	PROPOSED
PROF	PROFILE
PT	POINT OF TANGENT OR POINT
PVC	POLYVINYL CLORIDE
PVI	POINT OF VERTICAL INTERSECTION
R	RADIUS
RCP	REINFORCED CONCRETE PIPE
RET	RETAINING
R/W OR ROW	RIGHT OF WAY
RT	RIGHT
RW	RECLAIMED OR RECYCLED WATER
SCE	SOUTHERN CALIFORNIA EDISON
SD	STORM DRAIN
SECT	SECTION
SF	SQUARE FEET
SL	STREET LIGHT
ST	STREET
STA	STATION
STD	STANDARD
SS	SANITARY SEWER
SW	SIDEWALK OR STEM WALL
SD	STORM DRAIN
T	TELEPHONE
TAN	TANGENT LENGTH
TB	TOP OF BERM
TC	TOP OF CURB
TF	TOP OF FOOTING
TG	TOP OF GRATE
TOP	TOP OF PIPE
TR	TOP OF RETAINING
TRANS	TRANSITION
TS	TOP OF SLOPE OR TRAFFIC SIGNAL
TW	TOP OF WALL
TYP	TYPICAL
V	VALVE
VAR	VARIES
VCP	VITRIFIED CLAY PIPE
VERT	VERTICAL
W	WATER OR WITH
WM	WATER METER



SHEET INDEX MAP

1" = 100'
50 0 100

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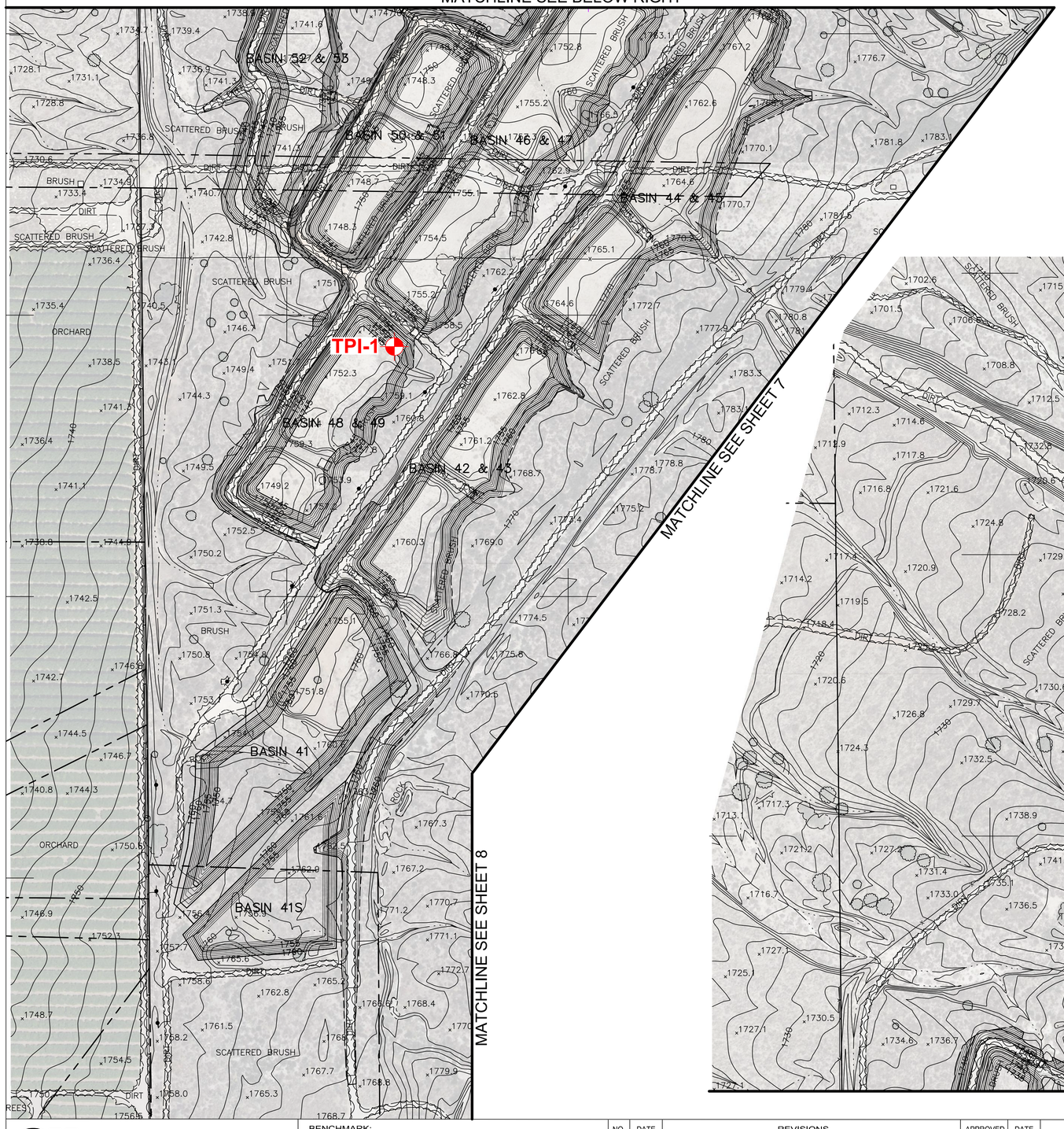
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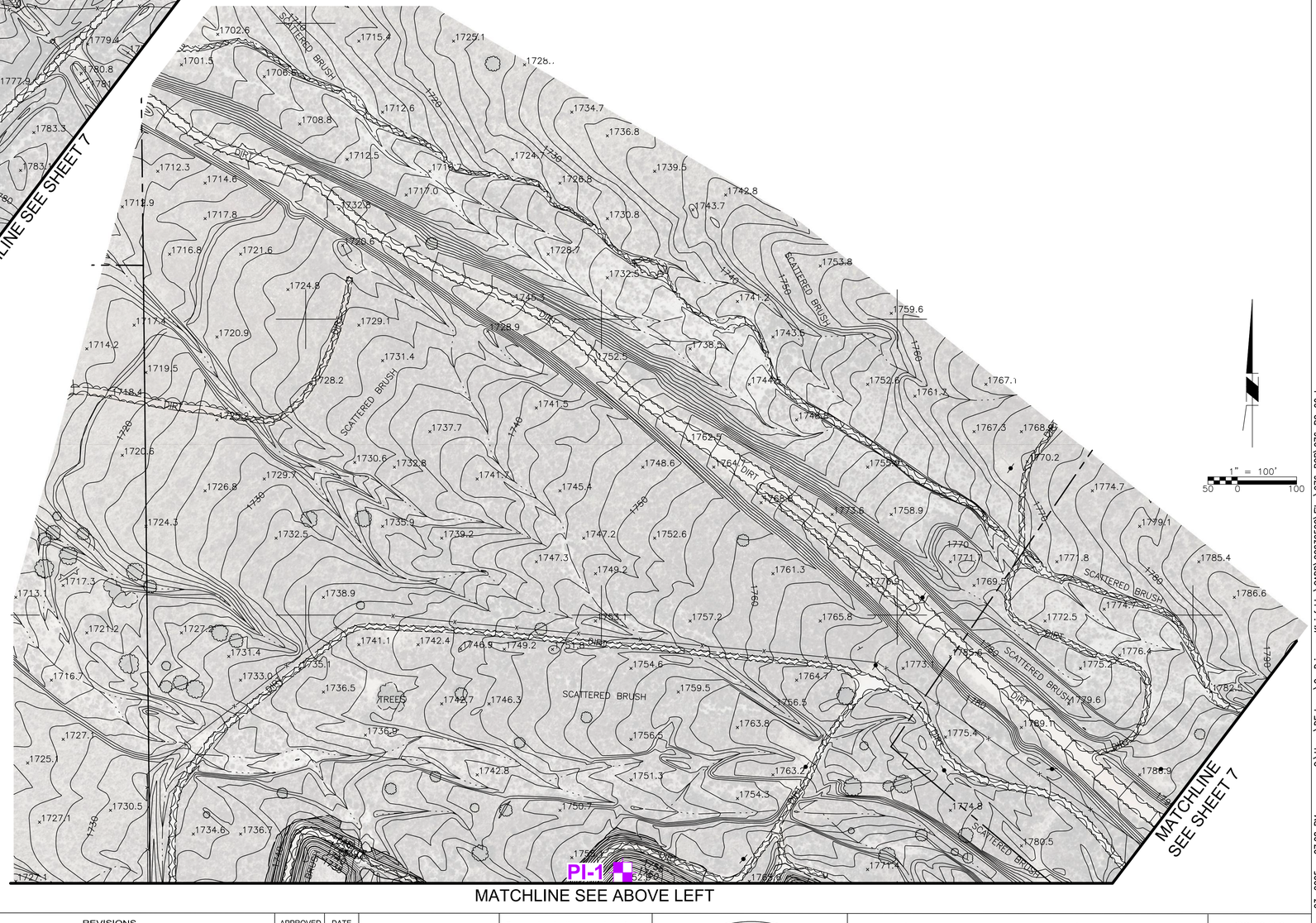
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MATCHLINE SEE BELOW RIGHT





MATCHLINE SEE ABOVE LEFT

EXPLANATION

TPI-16 APPROXIMATE BOREHOLE INFILTRATION LOCATION

TP-16 APPROXIMATE TEST PIT LOCATION

PI-7 APPROXIMATE TEST PIT INFILTRATION LOCATION

1" = 100'

50 0 100

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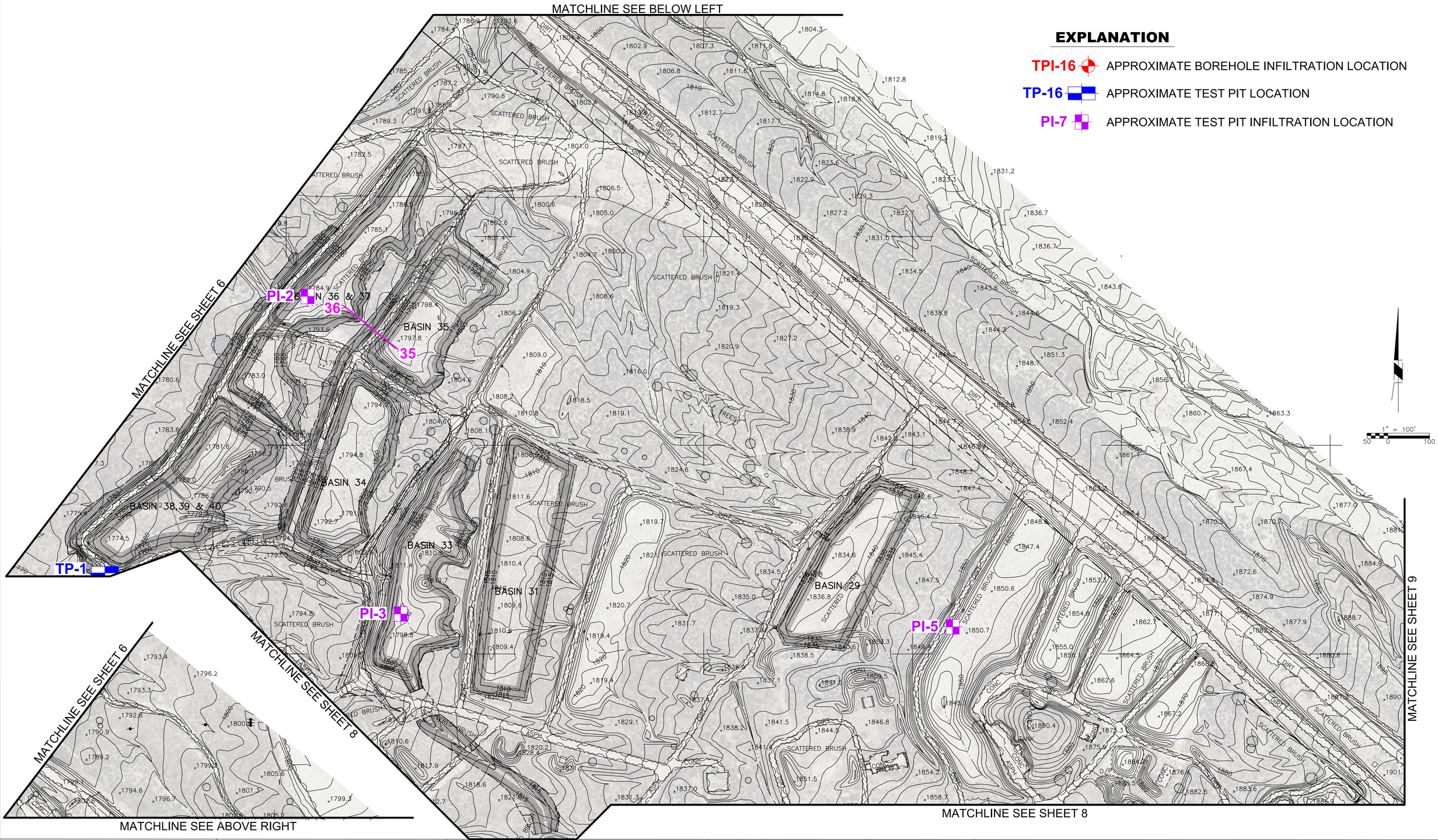
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SHEET 07
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EXPLANATION

TPI-16 APPROXIMATE BOREHOLE INFILTRATION LOCATION

TP-16 APPROXIMATE TEST PIT LOCATION

PI-7 APPROXIMATE TEST PIT INFILTRATION LOCATION

1" = 100'

50 0 100

MATCHLINE SEE SHEET 6

MATCHLINE SEE SHEET 7

MATCHLINE SEE SHEET 9

BASIN 22

BASIN 23

BASIN 24

BASIN 24E

BASIN 20

BASIN 15W

BASIN 14

BASIN 16M

BASIN 16S

TPI-2

TP-2

PI-8

24'

PI-4

TP-4

TPI-4

TP-3

TPI-9

SCATTERED BRUSH

ASPH

DIRT

CONC

811 Know what's below. Call before you dig. DIGALERT Underground Service Alert

Section 4216/4216.9 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert 811 two working days before you dig

BENCHMARK:

NO.	DATE	REVISIONS	APPROVED	DATE
1				
2				
3				
4				
5				
6				
7				

Prepared under the supervisor of:

DRAWN BY:

DESIGNED BY:

CHECKED BY:

San Bernardino Valley Water Conservation District
Helping Nature Store Our Water

SAN BERNARDINO VALLEY WATER CONVERSATION DISTRICT

ATRP MILL CREEK
PRELIMINARY DESIGN
076-RG-00 - 08

PROJECT NO. SBVWCD - 0000

SHEET 08

OF 34

Plotted by: DFehrmey Mar. 23, 2025 07:14 PM C:\Users\DFehrmey\OneDrive - Kleinfelder\CADD\20230500\Files\076-RGP\076-RG-00.dwg

EXPLANATION

TPI-16 APPROXIMATE BOREHOLE INFILTRATION LOCATION

TP-16 APPROXIMATE TEST PIT LOCATION

PI-7 APPROXIMATE TEST PIT INFILTRATION LOCATION

MATCHLINE SEE SHEET 7

MATCHLINE SEE SHEET 8

MATCHLINE SEE RIGHT

MATCHLINE SEE LEFT

MATCHLINE SEE SHEET 8

MATCHLINE SEE SHEET 10

TPI-16

TPI-11

TPI-13

TPI-12

TPI-8

TPI-6

TPI-7

PI-6

TP-6

TP-7

TP-8

12'

12

14W'

14W

BASIN 11

BASIN 10

BASIN 9

BASIN 8

BASIN 7

BASIN 6S

1" = 100'

50 0 100

811 Know what's below. Call before you dig. DIGALERT Underground Service Alert

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San Bernardino Valley Water Conservation District
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SAN BERNARDINO VALLEY WATER CONVERSATION DISTRICT

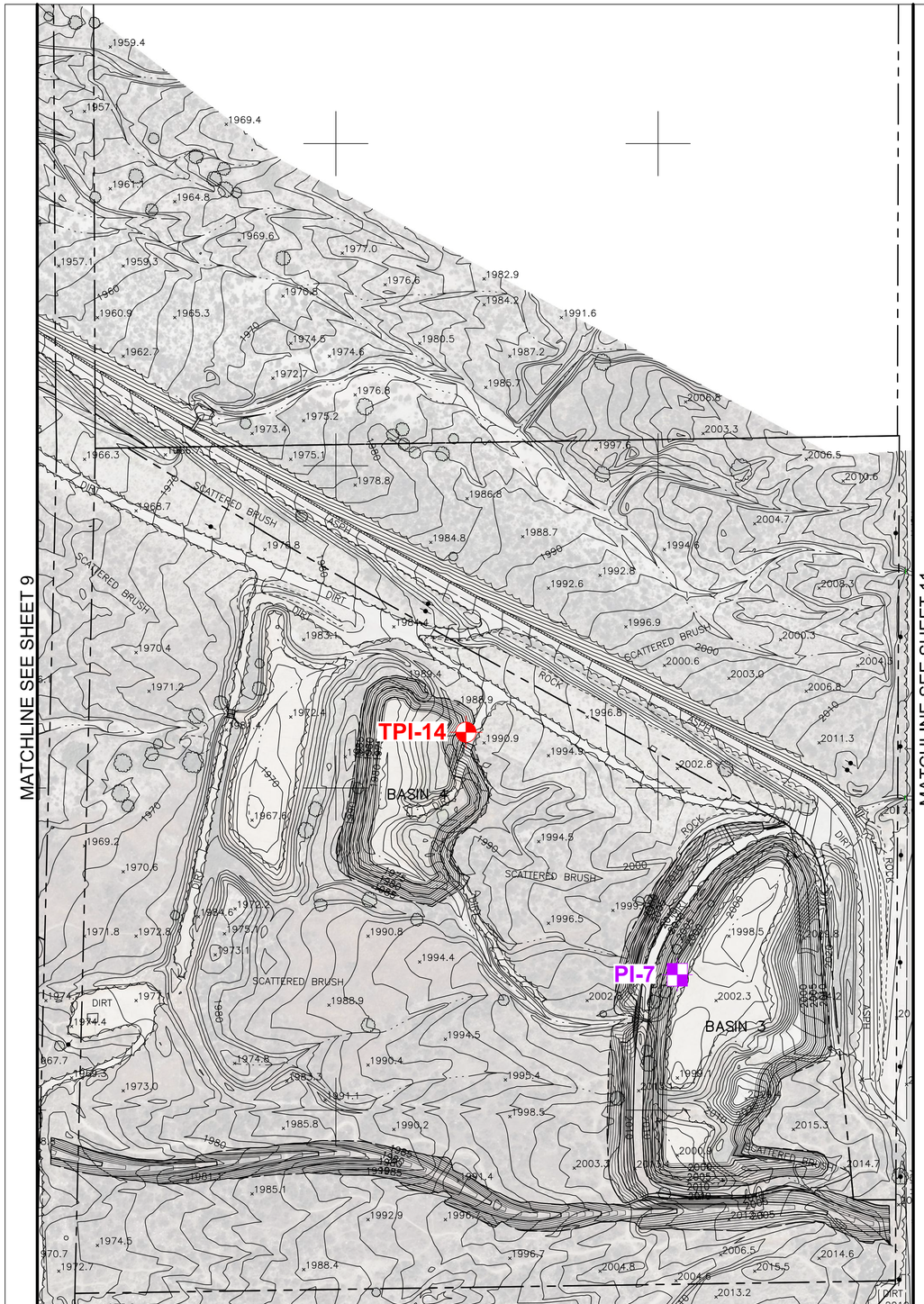
ATRP MILL CREEK
PRELIMINARY DESIGN
076-RG-00 - 09

PROJECT NO. SBVWCD - 0000

SHEET 09

OF 34

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EXPLANATION

TPI-16 APPROXIMATE BOREHOLE INFILTRATION LOCATION

TP-16 APPROXIMATE TEST PIT LOCATION

PI-7 APPROXIMATE TEST PIT INFILTRATION LOCATION

1" = 100'
50 0 100

811 Know what's below. Call before you dig. DIG ALERT Underground Service Alert

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Prepared under the supervisor of:

DRAWN BY:

DESIGNED BY:

CHECKED BY:

San Bernardino Valley Water Conservation District
Helping Nature Store Our Water

SAN BERNARDINO VALLEY WATER CONVERSATION DISTRICT

PROJECT NO. SBVWCD - 0000

ATRP MILL CREEK
PRELIMINARY DESIGN
076-RG-00 - 10

SHEET 10
OF 34

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MATCHLINE SEE SHEET 10

MATCHLINE BELOW 12

SEE SHEET 13

MILL CREEK ROAD

TP-9 TP-10 TP-11 TP-12 TP-13 TP-14

TPI-15 TPI-16

2A' 2A 2C 1A 1B 1B'

BASIN 2A BASIN 2B BASIN 2C BASIN 1A BASIN 1B

EXPLANATION

TPI-16 APPROXIMATE BOREHOLE INFILTRATION LOCATION

TP-16 APPROXIMATE TEST PIT LOCATION

PI-7 APPROXIMATE TEST PIT INFILTRATION LOCATION

1" = 100'

50 0 100

811 Know what's below. Call before you dig. DIG ALERT Underground Service Alert

Section 4216/4216.9 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert 811 two working days before you dig

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3				
4				
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6				
7				

Prepared under the supervisor of:

DRAWN BY:

DESIGNED BY:

CHECKED BY:

San Bernardino Valley Water Conservation District
Helping Nature Store Our Water

SAN BERNARDINO VALLEY WATER CONVERSATION DISTRICT

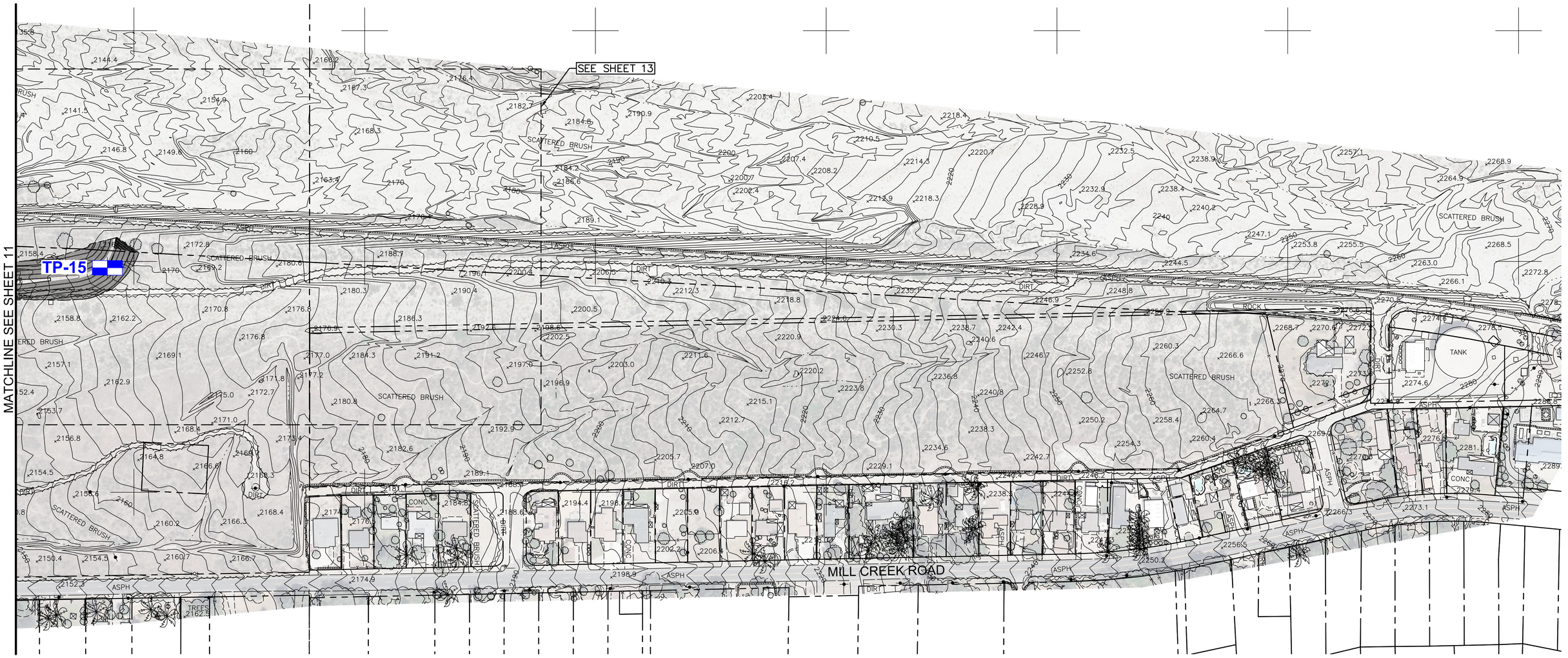
ATRP MILL CREEK
PRELIMINARY DESIGN
076-RG-00 - 11

PROJECT NO. SBVWCD - 0000

SHEET 11

OF 34

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EXPLANATION

TPI-16 APPROXIMATE BOREHOLE INFILTRATION LOCATION

TP-16 APPROXIMATE TEST PIT LOCATION

PI-7 APPROXIMATE TEST PIT INFILTRATION LOCATION

1" = 100'

50 0 100

811 Know what's below. Call before you dig. DIG ALERT Underground Service Alert

Section 4216/4216.9 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert 811 two working days before you dig

BENCHMARK:

NO.	DATE	REVISIONS	APPROVED	DATE
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3				
4				
5				
6				
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Prepared under the supervisor of:

DRAWN BY:

DESIGNED BY:

CHECKED BY:

San Bernardino Valley Water Conservation District
Helping Nature Store Our Water

SAN BERNARDINO VALLEY WATER CONVERSATION DISTRICT

ATRP MILL CREEK
PRELIMINARY DESIGN
076-RG-00 - 12

PROJECT NO.
SBVWCD -
0000

SHEET 12

OF 34

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APPENDIX B

Laboratory Test Results

DRAFT

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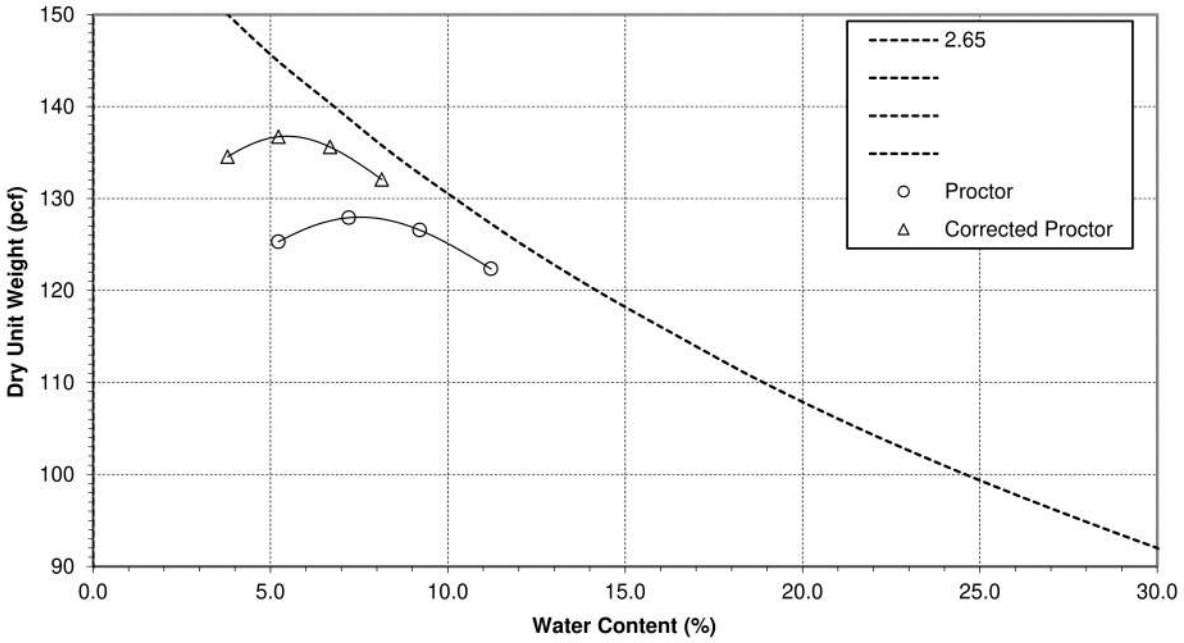
Laboratory Test Report

Client:	**Q3 Consulting**	Report No.:	**25-ONT-00490 Rev. 1**	Issued:	**4/1/2025**
Project:	**20230500.001A**				
	ARTP Mill Bledsoe Cook Creek Prelim Eng	Sampled by:	**Geo-Team**	Date:	**3/18/2024**
	07-072L - Laboratory Testing	Submitted by:	**Teo Albers**	Date:	**3/11/2025**

Tested on **3/18/2025** by **N. Brown**

Material Description: **Brown Silty Sand with gravel**

Location: **SW-1**



Test Method: ASTM D1557 C	Uncorrected	Corrected
Maximum Dry Unit Weight (pcf)	**128.0**	**136.8**
Optimum Water Content (%)	**7.5**	**5.4**
Oversize Fraction, retained on 3/4 (%)		**27**
Bulk Specific Gravity of Oversize Fraction		**2.68**

Correction for Oversize Fraction, ASTM D4718

Rammer Type: Mechanical

Specimen Preparation: Moist

Reviewed on 4/1/2025 by Teo Albers,
Staff Professional II

Limitations: Pursuant to applicable building codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided. This report may not be reproduced, except in full, without written approval of Kleinfelder.

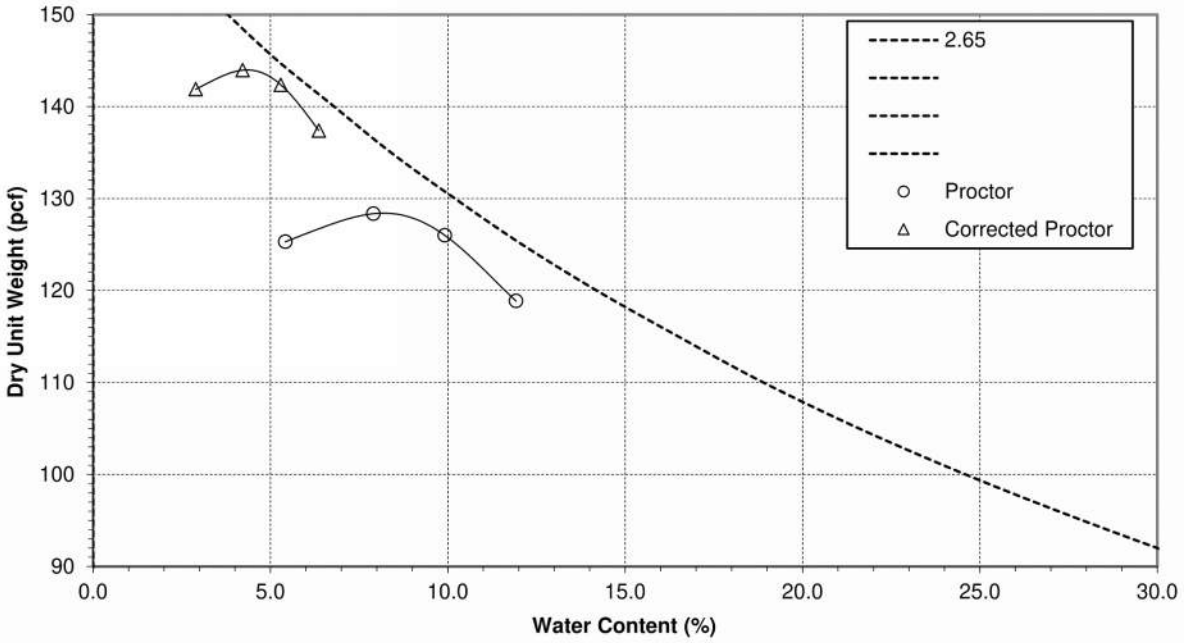
Laboratory Test Report

Client:	**Q3 Consulting**	Report No.:	**25-ONT-00490 Rev. 1**	Issued:	**4/1/2025**
Project:	**20230500.001A**				
	ARTP Mill Bledsoe Cook Creek Prelim Eng	Sampled by:	**Geo-Team**	Date:	**3/18/2024**
	07-072L - Laboratory Testing	Submitted by:	**Teo Albers**	Date:	**3/11/2025**

Tested on **3/18/2025** by **N. Brown**

Material Description: **Not Indicated**

Location: **NE-1**



Test Method: ASTM D1557 C	Uncorrected	Corrected
Maximum Dry Unit Weight (pcf)	**128.4**	**144.0**
Optimum Water Content (%)	**8.2**	**4.3**
Oversize Fraction, retained on 3/4 (%)		**47**
Bulk Specific Gravity of Oversize Fraction		**2.68**

Correction for Oversize Fraction, ASTM D4718

Rammer Type: Mechanical

Specimen Preparation: Moist

Reviewed on 4/1/2025 by Teo Albers,
Staff Professional II

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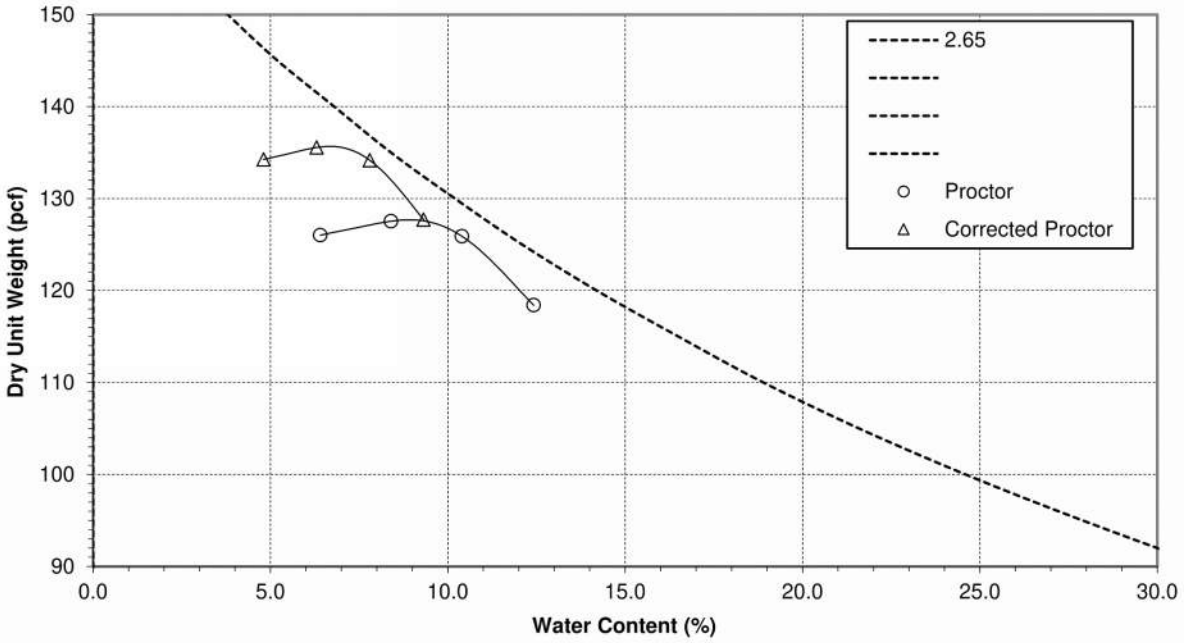
Laboratory Test Report

Client:	**Q3 Consulting**	Report No.:	**25-ONT-00490 Rev. 1**	Issued:	**4/1/2025**
Project:	**20230500.001A**				
	ARTP Mill Bledsoe Cook Creek Prelim Eng	Sampled by:	**Geo-Team**	Date:	**3/18/2024**
	07-072L - Laboratory Testing	Submitted by:	**Teo Albers**	Date:	**3/11/2025**

Tested on by **N. Brown**

Material Description: **Brown Silty Sand with Gravel**

Location: **CTR-1**



Test Method: ASTM D1557 C	Uncorrected	Corrected
Maximum Dry Unit Weight (pcf)	**127.7**	**135.7**
Optimum Water Content (%)	**8.9**	**6.7**
Oversize Fraction, retained on 3/4 (%)		**25**
Bulk Specific Gravity of Oversize Fraction		**2.68**

Correction for Oversize Fraction, ASTM D4718

Rammer Type: Mechanical

Specimen Preparation: Moist

Reviewed on 4/1/2025 by Teo Albers,
Staff Professional II

Limitations: Pursuant to applicable building codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided. This report may not be reproduced, except in full, without written approval of Kleinfelder.

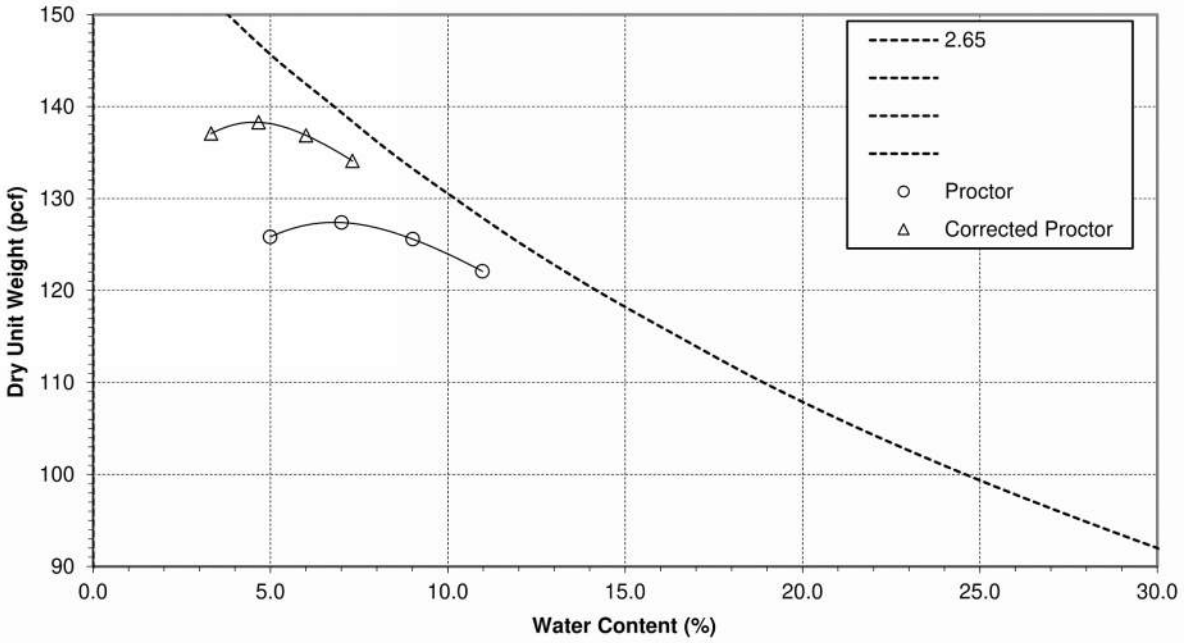
Laboratory Test Report

Client:	**Q3 Consulting**	Report No.:	**25-ONT-00490 Rev. 1**	Issued:	**4/1/2025**
Project:	**20230500.001A**				
	ARTP Mill Bledsoe Cook Creek Prelim Eng	Sampled by:	**Geo-Team**	Date:	**3/18/2024**
	07-072L - Laboratory Testing	Submitted by:	**Teo Albers**	Date:	**3/11/2025**

Tested on **3/19/2025** by **N. Brown**

Material Description: **Brown Silty Sand with Gravel**

Location: **SE-1**



Test Method: ASTM D1557 C	Uncorrected	Corrected
Maximum Dry Unit Weight (pcf)	**127.4**	**138.3**
Optimum Water Content (%)	**6.8**	**4.5**
Oversize Fraction, retained on 3/4 (%)		**33**
Bulk Specific Gravity of Oversize Fraction		**2.67**

Correction for Oversize Fraction, ASTM D4718

Rammer Type: Mechanical

Specimen Preparation: Moist

Reviewed on 4/1/2025 by Teo Albers,
Staff Professional II

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Project:	**20230500.001A**				
	ARTP Mill Bledsoe Cook Creek Prelim Eng	Sampled by:	**Geo-Team**	Date:	**3/18/2024**
	07-072L - Laboratory Testing	Submitted by:	**Teo Albers**	Date:	**3/11/2025**

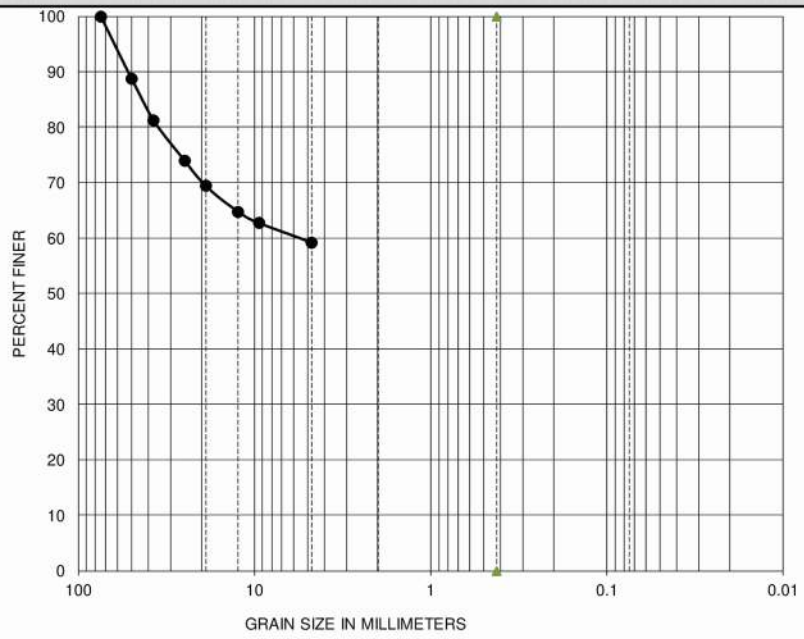
Tested on **3/20/2025** by **Nathan Brown**

Material Description: **Brown Silty Sand with Gravel**

Location: **SW-1**

Test Method: **ASTM C136**

U.S. Standard Sieve Size		% Passing
3 Inch	75-mm	**100**
2 Inch	50-mm	**89**
1 1/2 Inch	37.5-mm	**81**
1 Inch	25-mm	**74**
3/4 Inch	19-mm	**69**
1/2 Inch	12.5-mm	**65**
3/8 Inch	9.5-mm	**63**
No. 4	4.75-mm	**59**



Reviewed on 4/1/2025 by Teo Albers,
Staff Professional II

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Project:	**20230500.001A**				
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	07-072L - Laboratory Testing	Submitted by:	**Teo Albers**	Date:	**3/11/2025**

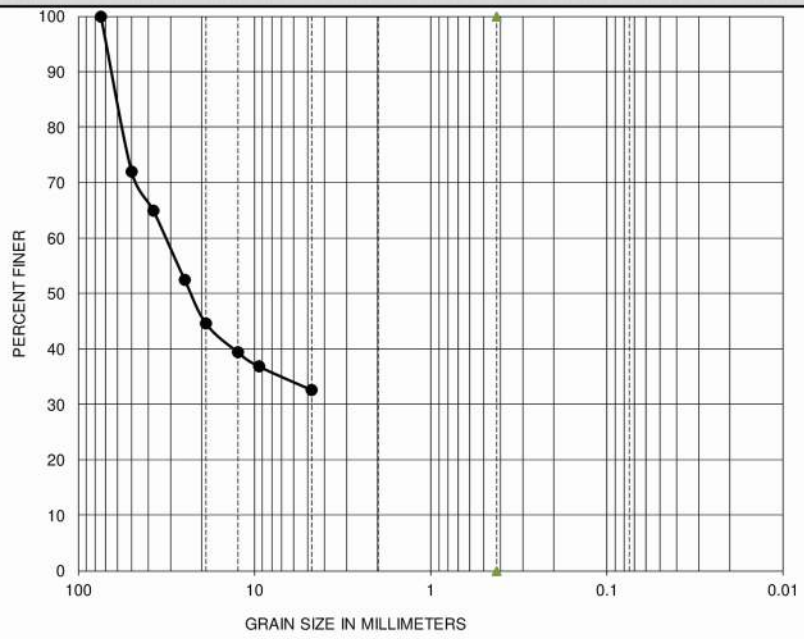
Tested on **3/20/2025** by **N. Brown**

Material Description: **Brown Silty Sand with Gravel**

Location: **NE-1**

Test Method: **ASTM C136**

U.S. Standard Sieve Size		% Passing
3 Inch	75-mm	**100**
2 Inch	50-mm	**72**
1 1/2 Inch	37.5-mm	**65**
1 Inch	25-mm	**53**
3/4 Inch	19-mm	**45**
1/2 Inch	12.5-mm	**39**
3/8 Inch	9.5-mm	**37**
No. 4	4.75-mm	**33**



Reviewed on 4/1/2025 by Teo Albers,
Staff Professional II

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Project:	**20230500.001A**				
	ARTP Mill Bledsoe Cook Creek Prelim Eng	Sampled by:	**Geo-Team**	Date:	**3/18/2024**
	07-072L - Laboratory Testing	Submitted by:	**Teo Albers**	Date:	**3/11/2025**

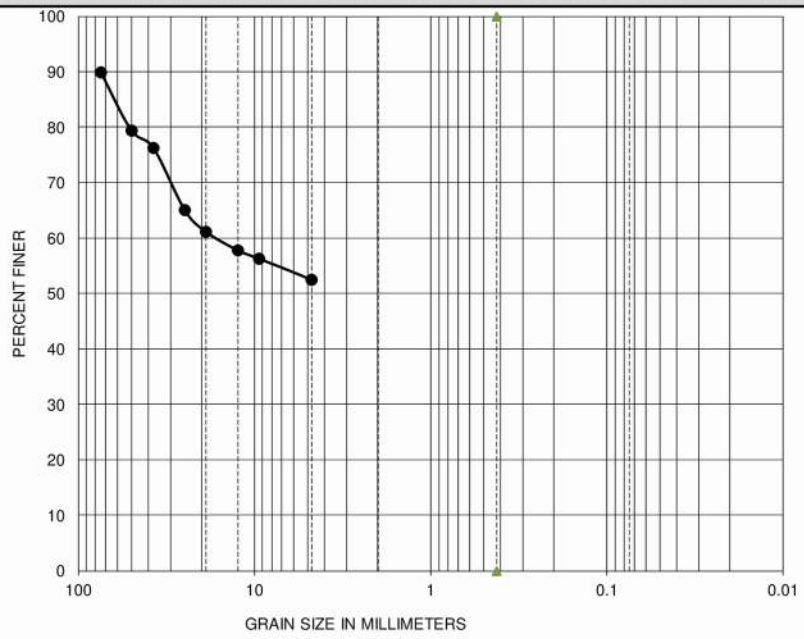
Tested on **3/20/2025** by **N. Brown**

Material Description: **Brown Silty Sand with Gravel**

Location: **CTR-1**

Test Method: **ASTM C136**

U.S. Standard Sieve Size		% Passing
3 Inch	75-mm	**90**
2 Inch	50-mm	**79**
1 1/2 Inch	37.5-mm	**76**
1 Inch	25-mm	**65**
3/4 Inch	19-mm	**61**
1/2 Inch	12.5-mm	**58**
3/8 Inch	9.5-mm	**56**
No. 4	4.75-mm	**53**



Reviewed on 4/1/2025 by Teo Albers,
Staff Professional II

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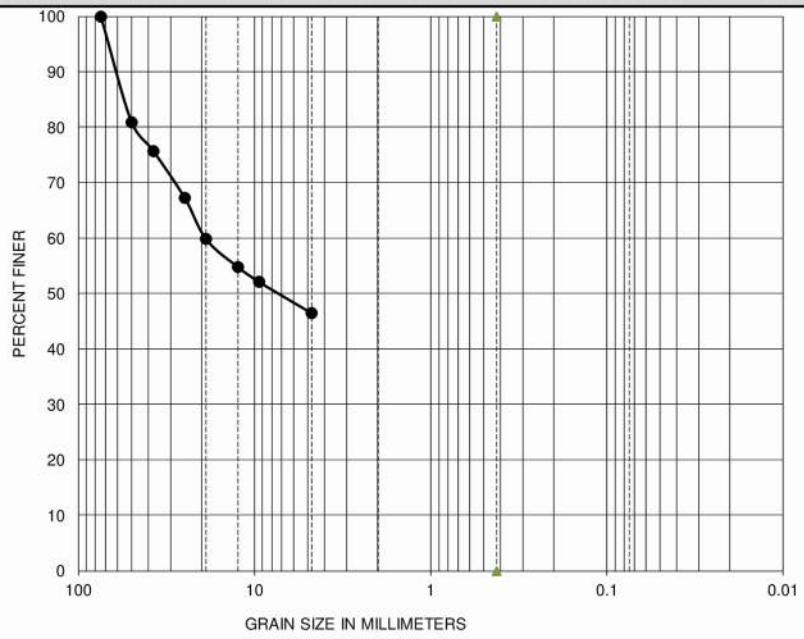
Tested on **3/20/2025** by **N. Brown**

Material Description: **Brown Silty Sand with Gravel**

Location: **SE-1**

Test Method: **ASTM C136**

U.S. Standard Sieve Size		% Passing
3 Inch	75-mm	**100**
2 Inch	50-mm	**81**
1 1/2 Inch	37.5-mm	**76**
1 Inch	25-mm	**67**
3/4 Inch	19-mm	**60**
1/2 Inch	12.5-mm	**55**
3/8 Inch	9.5-mm	**52**
No. 4	4.75-mm	**46**



Reviewed on 4/1/2025 by Teo Albers,
Staff Professional II

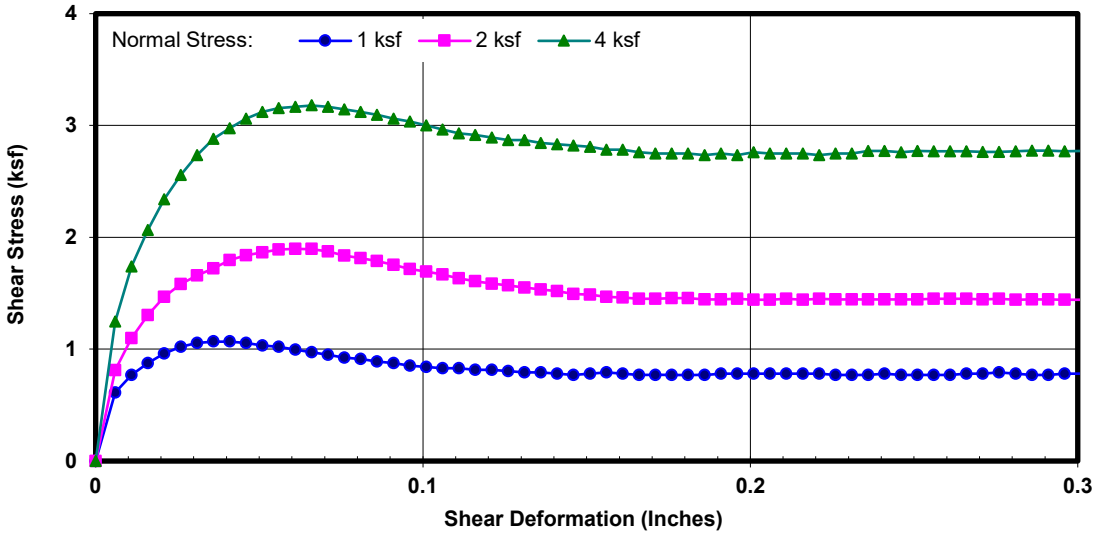
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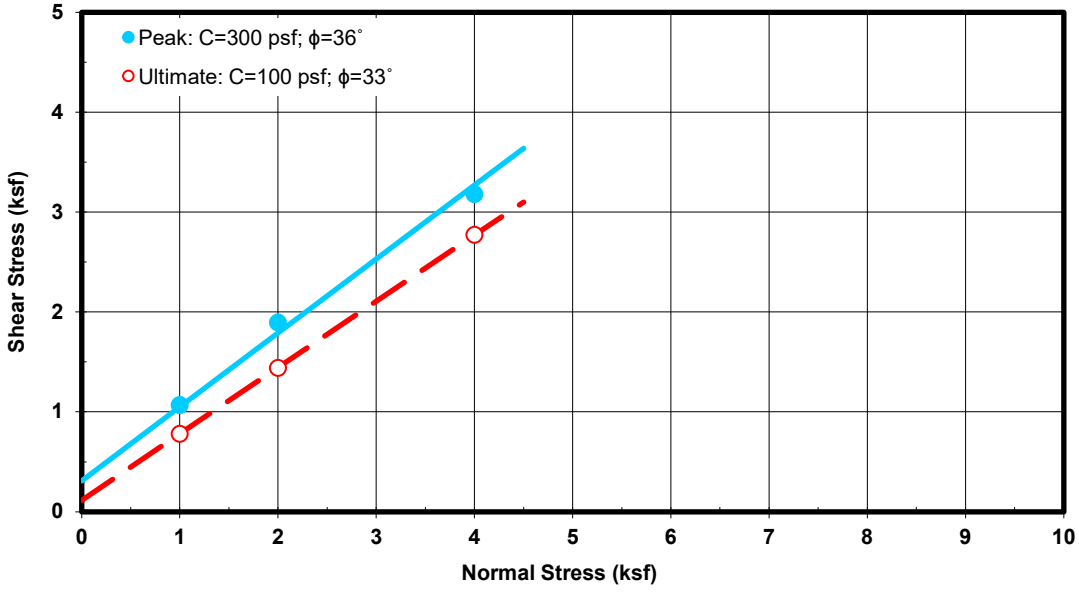
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	ARTP Mill Creek		**Tested By:**	ST	**Date:** 04/09/25
Project No.:	20230500 Task 07-072L		**Computed By:**	JG	**Date:** 04/10/25
Boring No.:	SW-1		**Checked by:**	AP	**Date:** 04/10/25
Sample Type:	Composite	**Depth (ft):** -			
Remold Cond.:	Remolded to 90% of 128 pcf at 7.5% MC				
Soil Description:	Silty Sand w/gravel				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
123.7	115.2	7.4	15.9	43	93	1	1.068	0.780
						2	1.896	1.440
						4	3.180	2.772





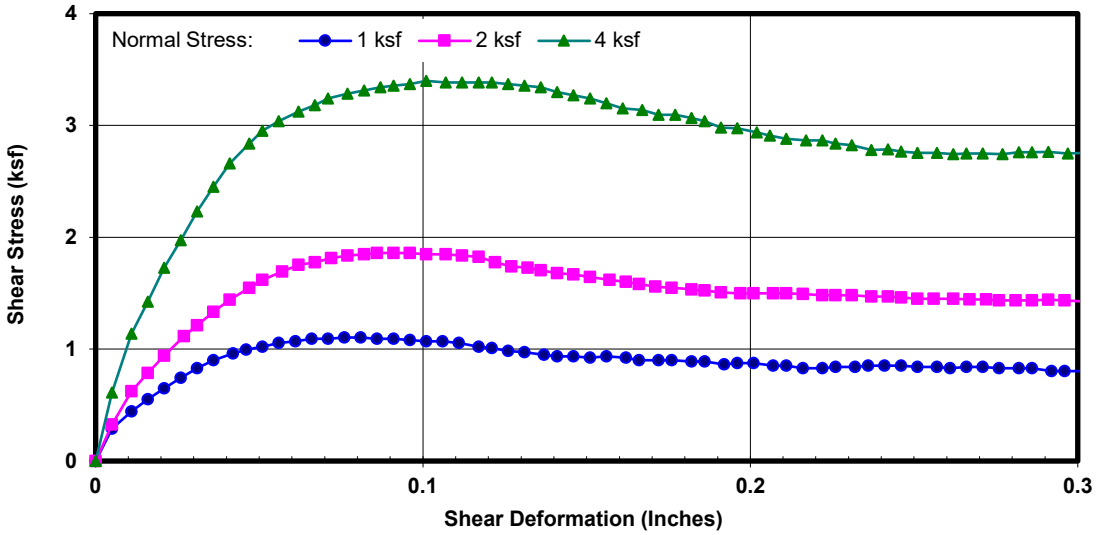
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

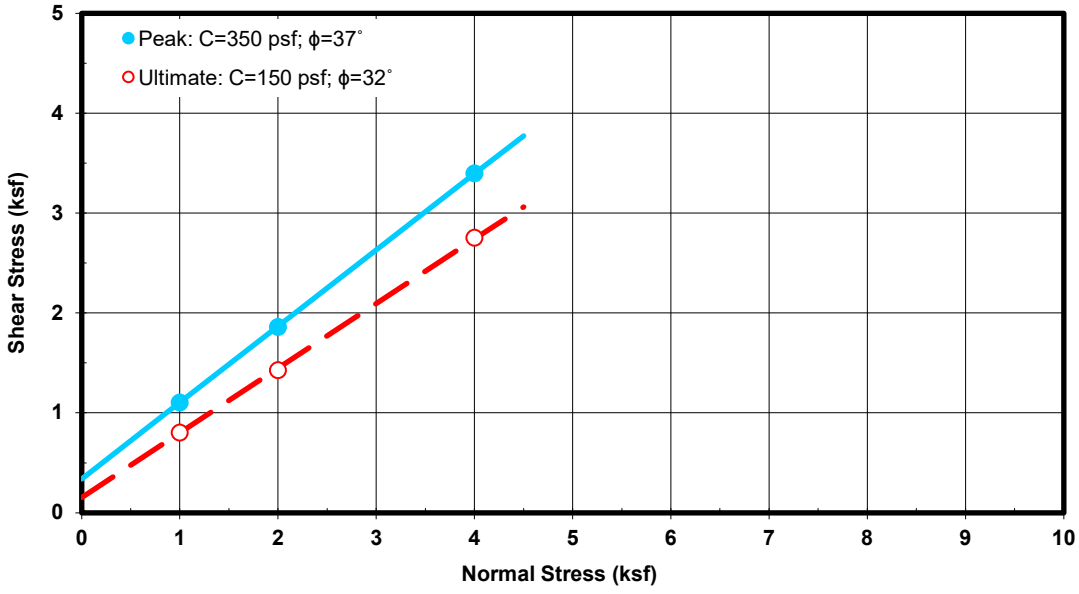
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: ARTP Mill Creek
Project No.: 20230500 Task 07-072L
Boring No.: NE-1
Sample Type: Composite **Depth (ft):** -
Remold Cond.: Remolded to 90% of 128.4 pcf at 8.2% MC
Soil Description: Silty Sand w/gravel
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 04/08/25
Computed By: JG **Date:** 04/09/25
Checked by: AP **Date:** 04/10/25

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
124.9	115.6	8.1	15.1	48	89	1	1.104	0.804
						2	1.860	1.428
						4	3.398	2.754





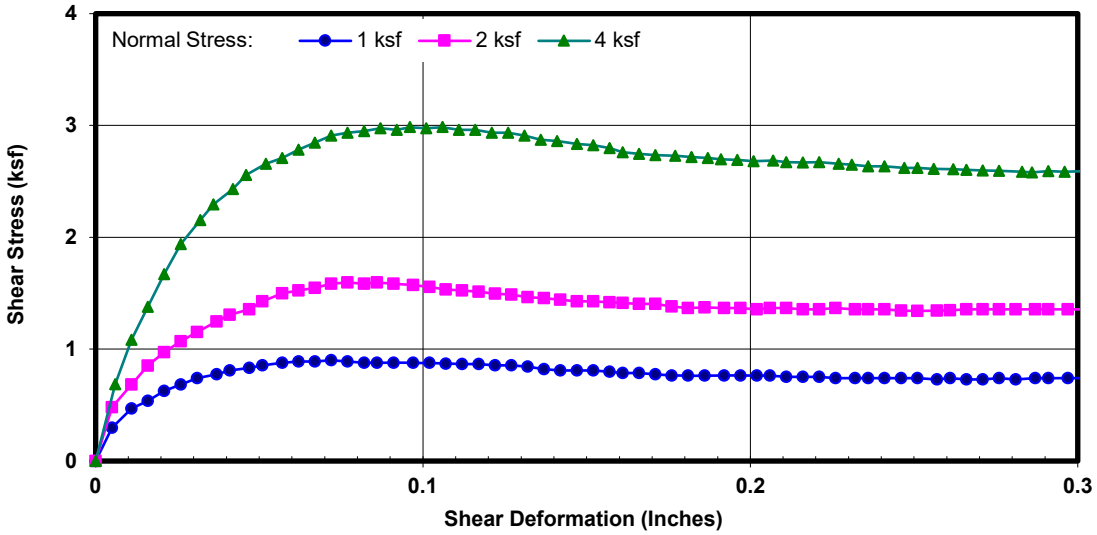
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

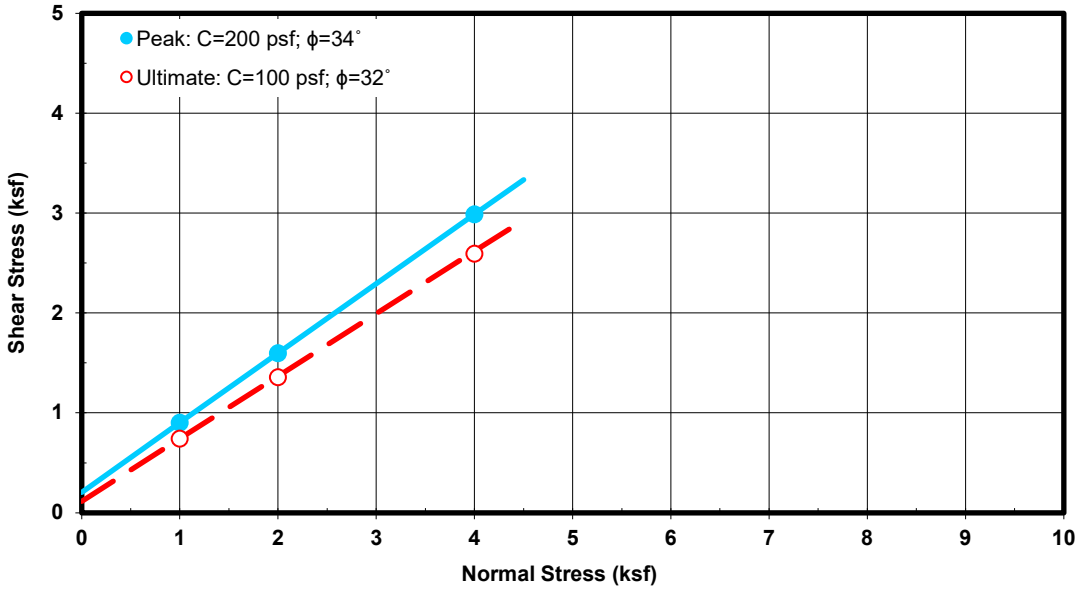
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: ARTP Mill Creek
Project No.: 20230500 Task 07-072L
Boring No.: CTR-1
Sample Type: Composite **Depth (ft):** -
Remold Cond.: Remolded to 90% of 127.7 pcf at 8.9% MC
Soil Description: Silty Sand w/gravel
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 04/09/25
Computed By: JG **Date:** 04/10/25
Checked by: AP **Date:** 04/10/25

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
124.9	114.9	8.7	15.8	51	91	1	0.901	0.741
						2	1.596	1.356
						4	2.986	2.592





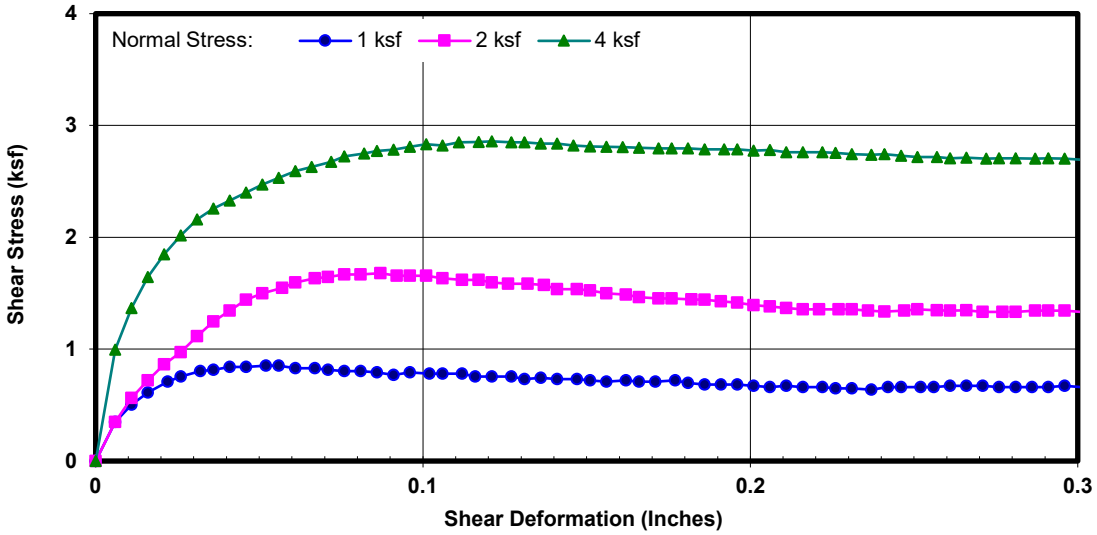
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

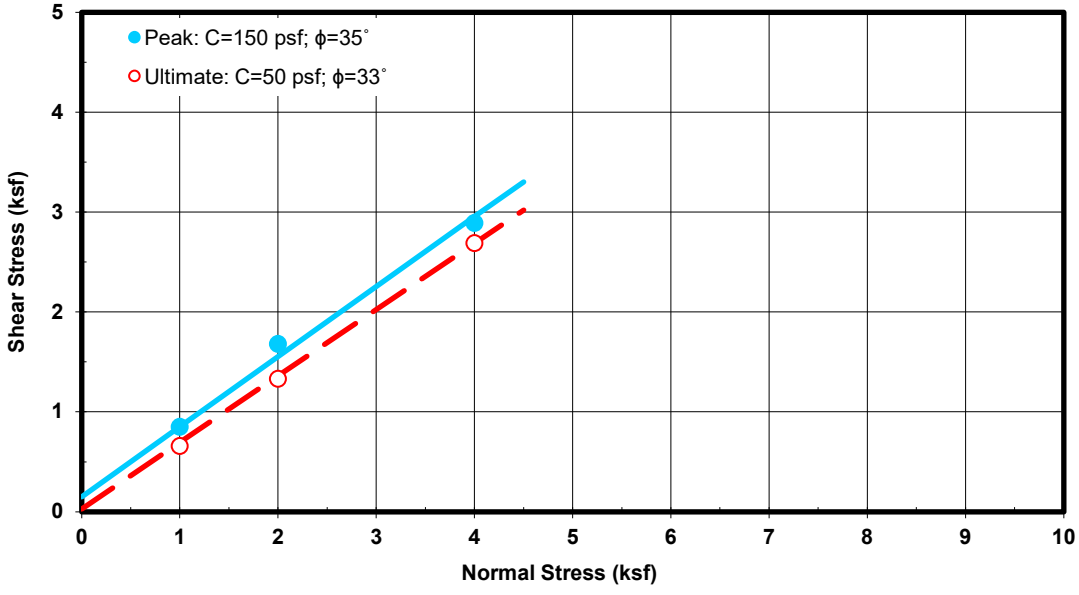
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: ARTP Mill Creek
Project No.: 20230500 Task 07-072L
Boring No.: SE-1
Sample Type: Composite **Depth (ft):** -
Remold Cond.: Remolded to 90% of 127.4 pcf at 6.8% MC
Soil Description: Silty Sand w/gravel
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 04/08/25
Computed By: JG **Date:** 04/09/25
Checked by: AP **Date:** 04/10/25

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
121.9	114.6	6.4	15.6	37	89	1	0.852	0.660
						2	1.680	1.332
						4	2.892	2.691





APPENDIX C
Calculations

DRAFT

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APPENDIX C.1

Parameter Development

DRAFT

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PROJECT	Mill Creek Basin	PROJECT NO.	20230500.001A		
SUBJECT	Parameter Development	BY	Qiqi Huang	DATE	04/15/2025
		REVIEWED BY	ECPond/JSW	DATE	07/31/2025

1.0 Purpose

Develop design parameters for seepage and stability analyses.

2.0 Given

- Appendix B (herein), Laboratory Test Results
- KLF (2024), Memorandum – Summary of Field Investigation and Infiltration Testing, ARTP Mill Creek Diversion and Basins Preliminary Design and Feasibility Study

3.0 Assumptions/ Design Considerations

Not applicable.

4.0 Analysis

4.1 Strength Parameters

Peak Shear Strength

The results of direct shear laboratory tests performed for this project are plotted in Appendix C.1 (Attachment 1). The recommended design peak strength envelope selected is represented by a friction angle of 37 degrees and cohesion of 200 psf.

We have also performed a back-analysis of existing slopes 12 feet in height at approximately 1.25H:1V and 10 feet in height at approximately 1.1H:1V. The results of our back analysis using the 2-D limit equilibrium software package GeoStudio, are presented in Appendix C.1 (Attachment 1). This back analysis was performed using Spencer's method with a circular search scheme. The results show that the recommended design strength parameters result in a safety factor exceeding 1.0 for the existing condition.

Ultimate Shear Strength

The results of direct shear laboratory tests performed for this project are plotted in Appendix C.1 (Attachment 1). The recommended design ultimate strength envelope selected is represented by a friction angle of 33 degrees and cohesion of 100 psf.

Unit Weight

The laboratory test results for maximum relative density/optimum moisture content are summarized in Attachment 1. Based on these results we recommend a design moist unit weight of 125 pcf

4.2 Hydraulic Conductivity (Permeability)

The vertical coefficient of permeability (k_v) is based on the KLF field investigation and infiltration testing memorandum Table 4 (KLF 2024) with a reduction factor of 2+2+3=7, as shown in Table 1. The average vertical permeability is 0.009 cm/s with a maximum of 0.015 cm/s and minimum of 0.003 cm/s. Generally, the onsite material is predominantly sandy gravel with boulders and

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PROJECT	Mill Creek Basin	PROJECT NO.	20230500.001A		
SUBJECT	Parameter Development	BY	Qiqi Huang	DATE	04/15/2025
		REVIEWED BY	ECPond/JSW	DATE	07/31/2025

cobbles. Therefore, a kv of 0.01 cm/s is used in the analysis, which corresponds to typical values for sandy gravel per EPRI EL-6800 (EPRI, 1990) Table 7-1.

Table 1: Summary of Vertical Permeability

Test ID Number	Vertical Infiltration Rate		Vertical Permeability
			k_v (1)
	(in/hr)	(cm/s)	(cm/s)
TPI-1	152.5	0.108	0.015
TPI-2	32.7	0.023	0.003
TPI-4	108.9	0.077	0.011
TPI-6	130.7	0.092	0.013
TPI-7	59.9	0.042	0.006
TPI-8	38.1	0.027	0.004
TPI-9	108.9	0.077	0.011
TPI-10	76.3	0.054	0.008
TPI-11	76.3	0.054	0.008
TPI-12	103.5	0.073	0.010
TPI-13	32.7	0.023	0.003
TPI-14	114.4	0.081	0.012
TPI-15	76.3	0.054	0.008
TPI-16	87.1	0.061	0.009
		Max	0.015
		Min	0.003
		Avg.	0.009
		Std. Dev.	0.004

Note:

1. k_v = Vertical Infiltration Rate / Reduction Factor

Based on the nature of the site soils we have assumed a k_h:k_v ratio of 1.0 is representative of the conditions for seepage and stability analyses. This is based on the high percentage of gravel, cobbles, and boulders in the site soils, with low fines content.

5.0 Results

Recommended project design parameters are summarized in Table 2

Table 2: Summary of Recommended Design Parameters

Unit Wt. γ	Peak Shear Strength		Ultimate Shear Strength		Permeability
	ϕ'	c'	ϕ_u	c_u	k_v, k_h
(pcf)	(Deg.)	(psf)	(Deg.)	(psf)	cm/s
125	37	200	33	100	0.01

PROJECT	Mill Creek Basin	PROJECT NO.	20230500.001A		
SUBJECT	Parameter Development	BY	Qiqi Huang	DATE	04/15/2025
		REVIEWED BY	ECPond/JSW	DATE	07/31/2025

6.0 Attachments

1. Strength Parameter Calculations

7.0 Reference

- Bentley Systems Inc. (2024), GeoStudio 2024.1.0.1406.
- EPRI (1990), Manual on Estimating Soil Properties for Foundation Design. (EL-6800)
- Appendix B (herein), Laboratory Test Results
- Kleinfelder (2024), Memorandum – Summary of Field Investigation and Infiltration Testing ARTP Mill Creek Diversion and Basins Preliminary Design and Feasibility Study. May 23, 2024.

PROJECT Mill Creek Basin PROJECT NO. 20230500.001A
SUBJECT Parameter Development BY Qiqi Huang DATE 04/15/2025
REVIEWED BY ECPond/JSW DATE 07/31/2025

8.0 Record of Revision

Revision No.	Reason for Revision
0	Original calculation package

ORIGINAL AND REVISED CALCULATIONS/ANALYSIS APPROVAL

	Rev. 0				
Originated By:	Qiqi Huang 04/15/2025				
Checked By:	ECPond/JSW 07/31/2025				

APPENDIX C.1

ATTACHMENT 1

Strength Parameter Calculations

DRAFT

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Project No.: 20230500.001A
Subject: Mill Creek Basin
Strength Parameter

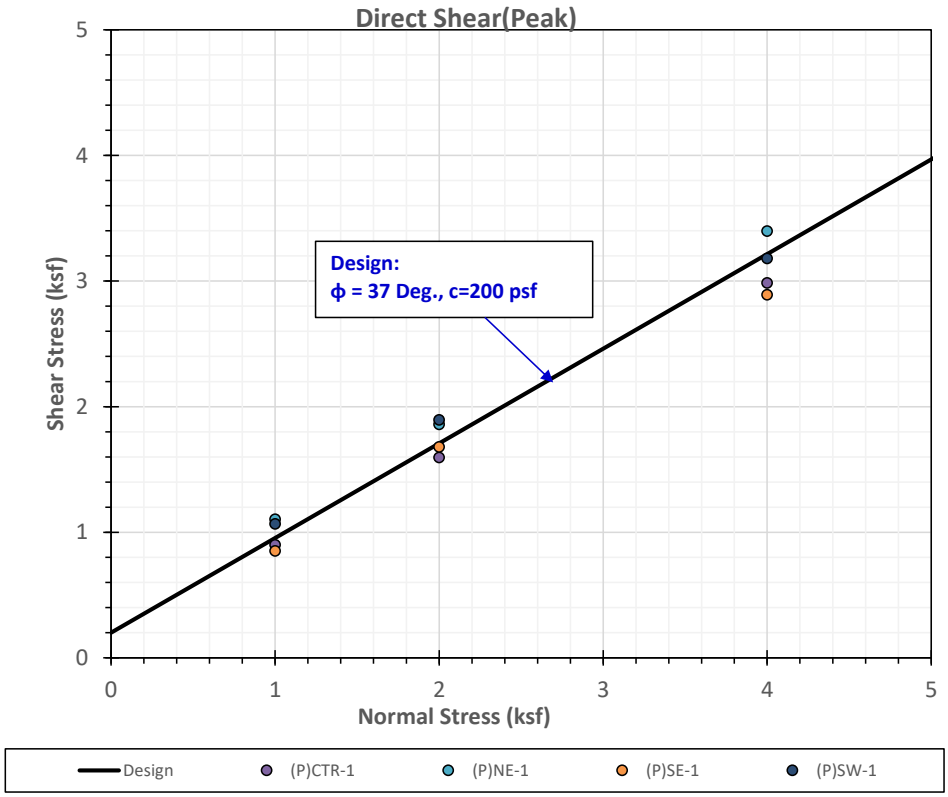
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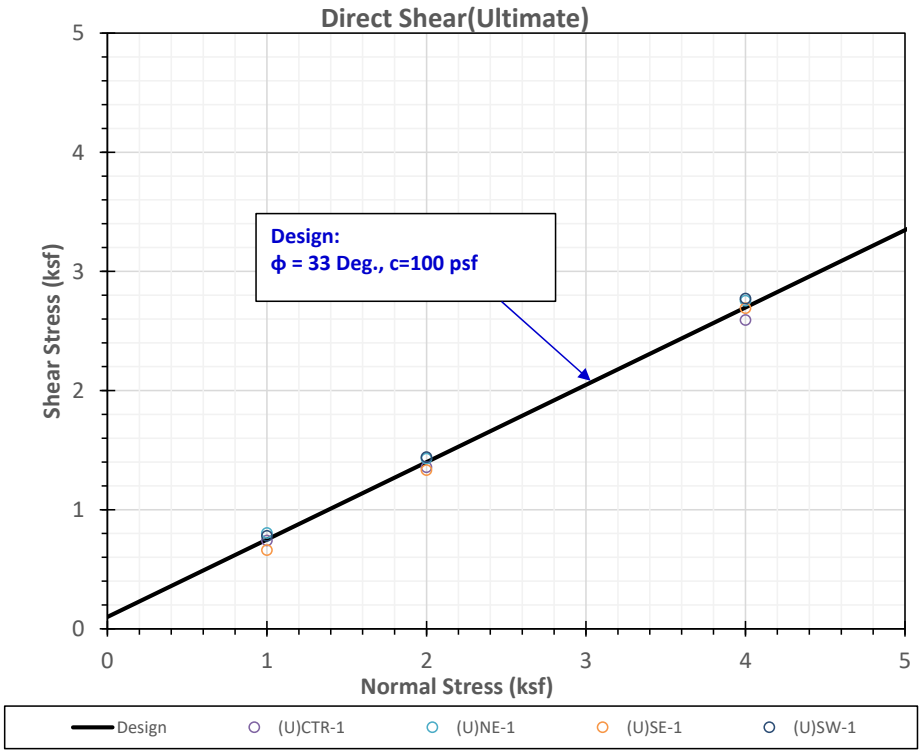
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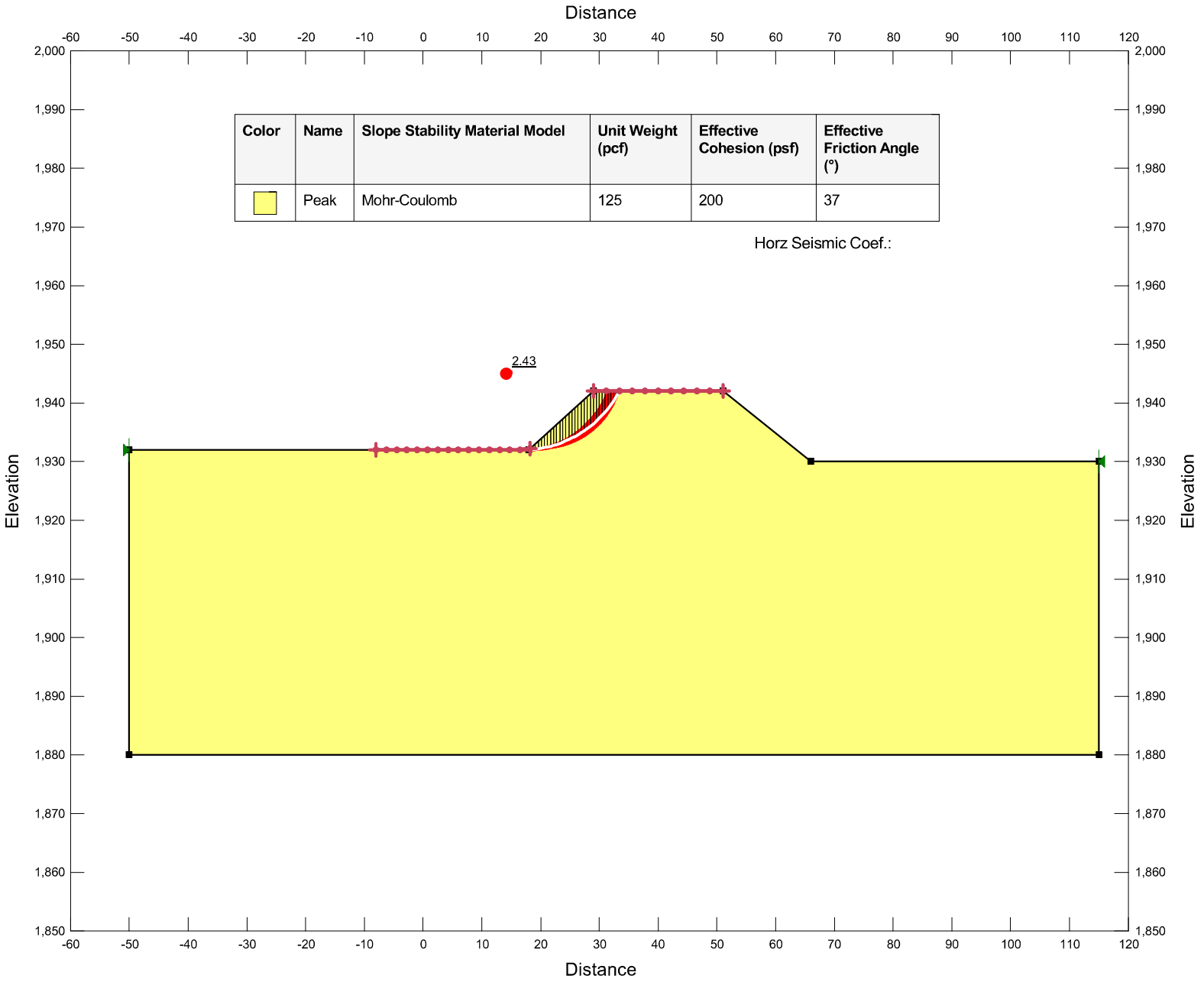
Boring ID	USCS	Unit Wt.	Normal Stress	Peak Shear Strength	Ultimate Shear Strength
		(pcf)	(ksf)	(ksf)	(ksf)
CTR-1	SM	124.9	1.0	0.901	0.741
			2.0	1.596	1.356
			4.0	2.986	2.592
NE-1	SM	124.9	1.0	1.104	0.804
			2.0	1.86	1.428
			4.0	3.398	2.754
SE-1	SM	121.9	1.0	0.852	0.66
			2.0	1.68	1.332
			4.0	2.892	2.691
SW-1	SM	123.7	1.0	1.068	0.78
			2.0	1.896	1.44
			4.0	3.18	2.772

Project No.: 20230500.001A
Subject: Mill Creek Basin
Strength Parameter

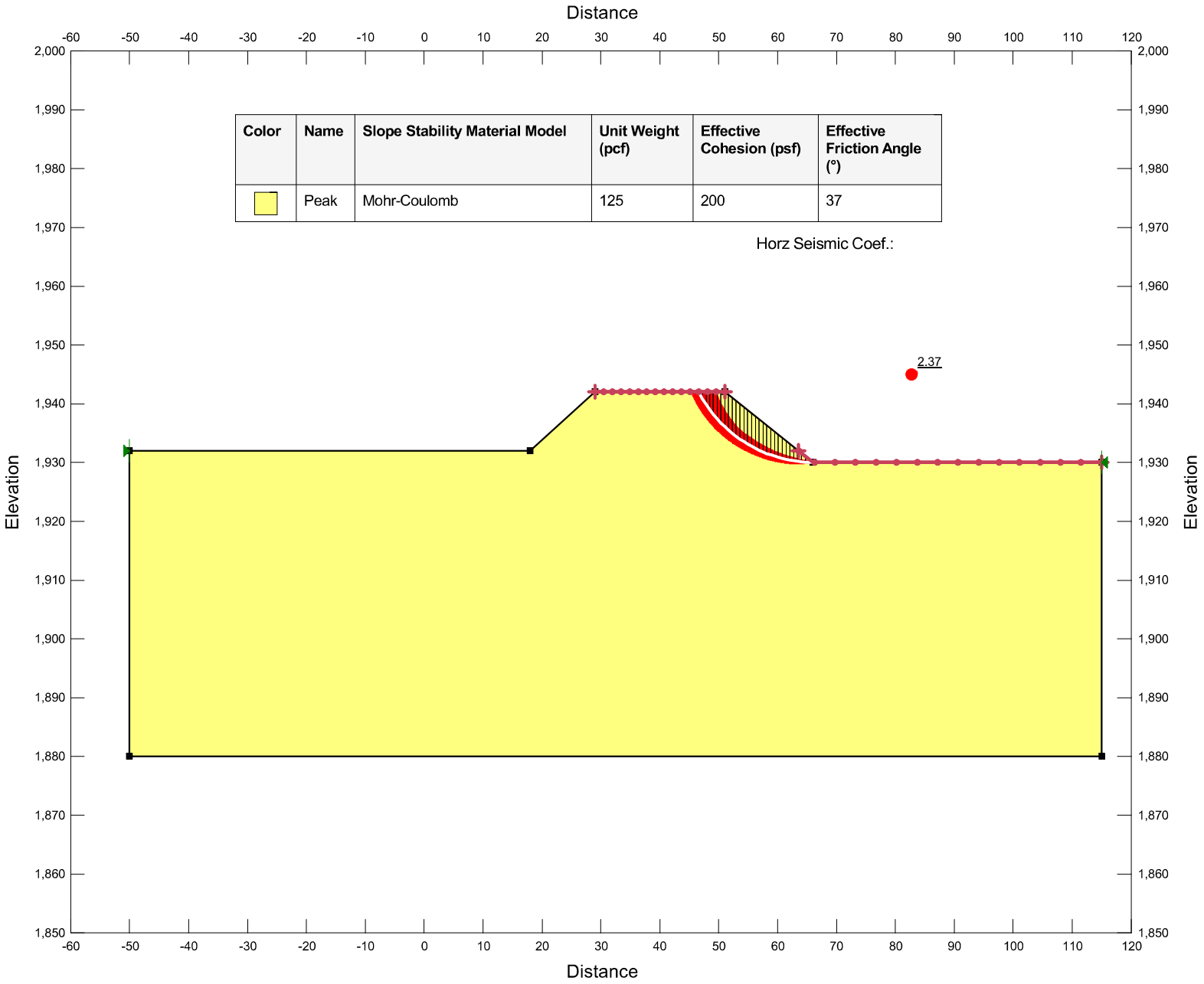
Computed by: QH Date: 4/15/2025 Chk'd by: ECP Date: 4/30/2025







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File Name: MCB_Sec_12.gsz
Created By: Qiqi Huang
Date:04/15/2025



Title: Mill Creek Basin. Name: 1.25H:1V
File Name: MCB_Sec_12.gsz
Created By: Qiqi Huang
Date:04/15/2025

APPENDIX C.2

Seepage and Slope Stability Analyses

DRAFT

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PROJECT	Mill Creek Basin	PROJECT NO.	20230500.001A		
SUBJECT	Seepage and Slope Stability	BY	Q. Huang	DATE	07/15/2025
	Analysis	REVIEWED BY	ECPond/JSW	DATE	07/31/20025

1.0 Purpose

- Perform seepage and slope stability evaluation for Mill Creek Basins.

2.0 Given

- Basin cross sections shown in Appendix C.2 (Attachment 1).
- Design soil properties shown in Table 1 from parameter development (Appendix C.1).

Table 1: Geotechnical Design Parameters

Unit Wt.	Peak Shear Strength		Ultimate Shear Strength		Vertical Permeability
γ	ϕ'	c'	ϕ_u	c_u	k_v
(pcf)	(Deg)	(psf)	(Deg)	(psf)	(cm/s)
125	37	200	33	100	0.01

3.0 Assumptions/ Design Considerations

- Maximum allowable exit gradient at downstream face is 0.5.
- The following minimum factor of safety (FS) values were considered for slope stability:
  - 1.5 for static loading of slopes
  - 1.1 for seismic loading (pseudo-static)
- Horizontal permeability (k_h) is assumed to be the same as vertical permeability (k_v) based on the high gravel, cobbles, and boulders, with low fines content in the site soils.
- Design ground water elevation (DGWE) is based on the following:
  - Static: 3 feet of freeboard on the upstream face and the DGWE is assumed to be at the ground surface bellow the toe of the slope on the downstream side.
  - Seismic: the earthquake doesn't coincide with high water (static condition) per USACE EM 1110-2-1913 Section 6-5.d, however, the DGWE at the upstream face under the static case is conservatively assumed in the initial analysis and adjusted if the FS does not meet the requirement.
- The site class is estimated to be either C or D. The peak ground acceleration (PGA) due to a 2475-year return period event for site class C and D is 1.13g and 1.04g, respectively. See Appendix C.2 (Attachment 2) for calculations. Considering one third of the PGA as the horizontal coefficient for evaluation of seismic stability, assuming deformation of up to 6 inches is tolerable, the horizontal seismic coefficient (k_{eh}) is 0.38g and 0.35g for site class C and D conditions, respectively. Therefore, use 0.38g in the analysis.
- Liquefaction is not a concern.

4.0 Analysis

The computer program, GeoStudio, is used to evaluate the seepage and slope stability of the basins. Spencer and Morgenstern-Price (M-P) method are used as they satisfy both force and moment equilibrium. A circular failure surface search is used for static and seismic conditions. Table 2 summarizes the analysis sections.

PROJECT	Mill Creek Basin	PROJECT NO.	20230500.001A		
SUBJECT	Seepage and Slope Stability	BY	Q. Huang	DATE	07/15/2025
	Analysis	REVIEWED BY	ECPond/JSW	DATE	07/31/20025

Table 2: Analysis Sections

Cross Section	Analyzed Section	Slope Ht (feet)	Upstream Slope	Downstream Slope	DGWE (feet)	
					Upstream	Downstream
1A-2C	E	22	3H:1V	3H:1V	2057	2038
1B*	B	13	3H:1V	2H:1V	2082	2072
2A	B	12	3H:1V	3H:1V	2029	2020
12*	C	12	1.1H:1V	1.25H:1V	1939	1930
14-15W*	A	23	3H:1V	3H:1V	1879	1859
14W	B	13	3H:1V	3H:1V	1851	1841
24*	D	17	2H:1V	3H:1V	1795	1781
35-36*	E	22	3H:1V	3H:1V & 1.75H:1	1799	1780

Note:
*: Cross section analyzed.

Peak shear strength is used under static conditions and under seismic conditions during shaking (with seismic coefficient k_{eh}) while the ultimate shear strength is used to model seismic conditions following shaking (without k_{eh}).

5.0 Results

Table 3 summarizes the maximum exit gradient at the downstream face and the minimum FS for each section analyzed using GeoStudio. The maximum exit gradient at the downstream face is less than 0.5. The minimum FS values meet or exceed the requirements except Sections A and E under seismic conditions. The minimum freeboard to satisfy the FS requirements of sections A and E under seismic conditions is 6 feet and 5 feet, respectively. Table 4 summarizes the minimum freeboard at the upstream face for the section analyzed under static and seismic conditions, respectively. The GeoStudio results are included in Appendix C.2 (Attachment 3).

We have also evaluated two generic slope configurations with 3:1 upstream slopes and either a 3:1 or 2:1 downstream slope face. For these cases the maximum pool elevation in the basin must be maintained at or below 16 feet or 15 feet, respectively, above the downstream toe elevation to satisfy the minimum safety factor under seismic loading. The GeoStudio results are included in Appendix C.2 (Attachment 3).

Table 3: Summary of GeoStudio Results

Analyzed Section	Cross Section	Slope Ht (feet)	Downstream Exit Gradient	Minimum FS					
				Static		Seismic			
						W k_{eh}		W/O k_{eh}	
				M-P	Spencer	M-P	Spencer	M-P	Spencer
A	14-15W	23	0.32	2.47	2.47	1.03	1.03	1.89	1.89
B	1B	13	0.35	2.61	2.61	1.23	1.23	1.89	1.89
C	12	12	0.47	2.24	2.24	1.26	1.26	1.56	1.56
D	24	17	0.26	2.85	2.85	1.15	1.15	2.15	2.15
E	35-36	22	0.49	2.66	2.66	1.06	1.06	1.69	1.69

PROJECT	Mill Creek Basin	PROJECT NO.	20230500.001A		
SUBJECT	Seepage and Slope Stability	BY	Q. Huang	DATE	07/15/2025
	Analysis	REVIEWED BY	ECPond/JSW	DATE	07/31/20025

Table 4: Summary of Minimum Freeboard at Upstream Side

Analyzed Section	Cross Section	Minimum Required Freeboard at Upstream Face	
		Static	Seismic
		(feet)	(feet)
A	14-15W	3	6(1)
B	1B	3	3
C	12	3	3
D	24	3	3
E	35-36	3	5(1)

Notes:
1. Based on slope stability analysis

6.0 Attachments

1. Cross Sections
2. Peak Ground Accelerations
3. GeoStudio Results

7.0 Reference

AASHTO (2020), LRFD Bridge Design Specifications, 9th Edition.
Bentley Systems Inc. (2024), GeoStudio 2024.1.0.1406.
USACE (2000), Design and Construction of Levees (EM 1110-2-1913)

PROJECT	Mill Creek Basin	PROJECT NO.	20230500.001A		
SUBJECT	Seepage and Slope Stability	BY	Q. Huang	DATE	07/15/2025
	Analysis	REVIEWED BY	ECPond/JSW	DATE	07/31/20025

8.0 Record of Revision

Revision No.	Reason for Revision
0	Original calculation package

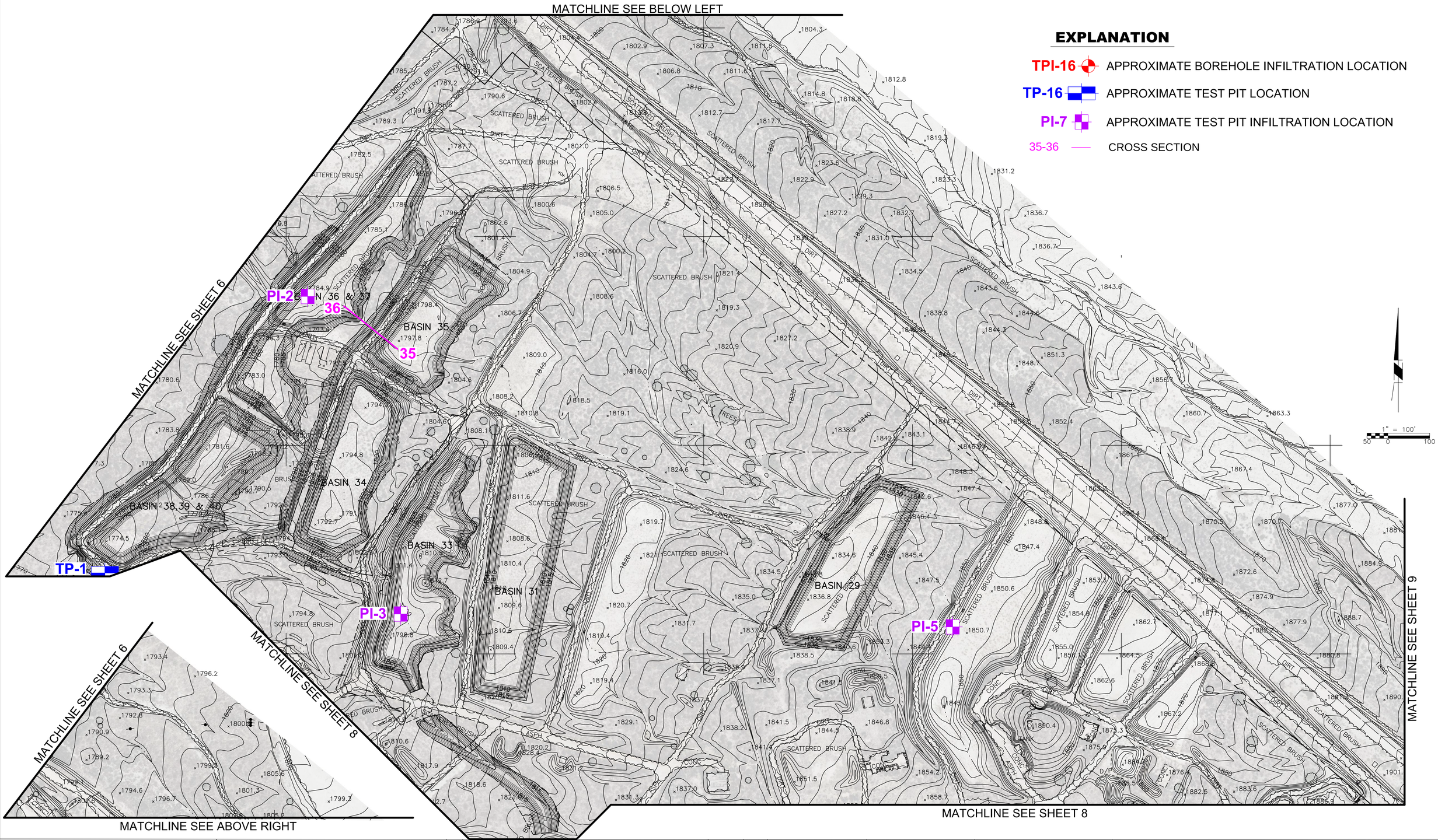
ORIGINAL AND REVISED CALCULATIONS/ANALYSIS APPROVAL

	Rev. __0__				
Originated By:	Qiqi Huang 07/15/2025				
Checked By:	ECPond/JSW 07/31/2025				

APPENDIX C.2
ATTACHMENT 1
Cross-Sections

DRAFT

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Section 4216/4216.9 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert 811 two working days before you dig

BENCHMARK:

NO.	DATE	REVISIONS	APPROVED	DATE
1				
2				
3				
4				
5				
6				
7				

Prepared under the supervisor of:

DRAWN BY:

DESIGNED BY:

CHECKED BY:

San Bernardino Valley Water Conservation District
Helping Nature Store Our Water

SAN BERNARDINO VALLEY WATER CONVERSATION DISTRICT

PROJECT NO. SBVWCD - 0000

ATRP MILL CREEK
PRELIMINARY DESIGN
076-RG-00 - 07

SHEET 07
OF 34

Plotted by: DFahrney Mar. 23, 2025 07:11 PM C:\Users\DFahrney\OneDrive - Kleinfelder\CADD\20230500\Files\076-RGP\076-RG-00.dwg

EXPLANATION

TPI-16 — APPROXIMATE BOREHOLE INFILTRATION LOCATION

TP-16 — APPROXIMATE TEST PIT LOCATION

PI-7 — APPROXIMATE TEST PIT INFILTRATION LOCATION

35-36 — CROSS SECTION

1" = 100'

50 0 100

MATCHLINE SEE SHEET 6

MATCHLINE SEE SHEET 7

MATCHLINE SEE SHEET 9

BASIN 22

BASIN 23

BASIN 24

BASIN 24E

BASIN 20

BASIN 16S

BASIN 16M

BASIN 15W

BASIN 14

TPI-2

TP-2

PI-8

24'

24

PI-4

TP-3

TP-4

TPI-4

15W

TPI-9

14

SCATTERED BRUSH

ASPH

DIRT

CONC

ROCK

D/P

BRUSH

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BENCHMARK:

NO.	DATE	REVISIONS	APPROVED	DATE
1				
2				
3				
4				
5				
6				
7				

Prepared under the supervisor of:

DRAWN BY:

DESIGNED BY:

CHECKED BY:

San Bernardino Valley Water Conservation District
Helping Nature Store Our Water

SAN BERNARDINO VALLEY WATER CONVERSATION DISTRICT

ATRP MILL CREEK
PRELIMINARY DESIGN
076-RG-00 - 08

PROJECT NO. SBVWCD - 0000

SHEET 08

OF 34

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EXPLANATION

- TPI-16 — APPROXIMATE BOREHOLE INFILTRATION LOCATION
- TP-16 — APPROXIMATE TEST PIT LOCATION
- PI-7 — APPROXIMATE TEST PIT INFILTRATION LOCATION
- 35-36 — CROSS SECTION

MATCHLINE SEE SHEET 7

MATCHLINE SEE SHEET 8

MATCHLINE SEE RIGHT

MATCHLINE SEE LEFT

MATCHLINE SEE SHEET 8

MATCHLINE SEE SHEET 10

TPI-16 TPI-8 TPI-6 TPI-7 TPI-11 TPI-13 TPI-12

PI-6

TP-6 TP-7 TP-8

12' 12 14W' 14W

BASIN 11 BASIN 10 BASIN 9 BASIN 8 BASIN 7 BASIN 6S

SCATTERED BRUSH BRUSH DIRT ASPH CONC ROCK

1" = 100'

50 0 100

811 Know what's below. Call before you dig. DIGALERT Underground Service Alert

Section 4216/4216.9 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert 811 two working days before you dig

BENCHMARK:

NO.	DATE	REVISIONS	APPROVED	DATE
1				
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San Bernardino Valley Water Conservation District
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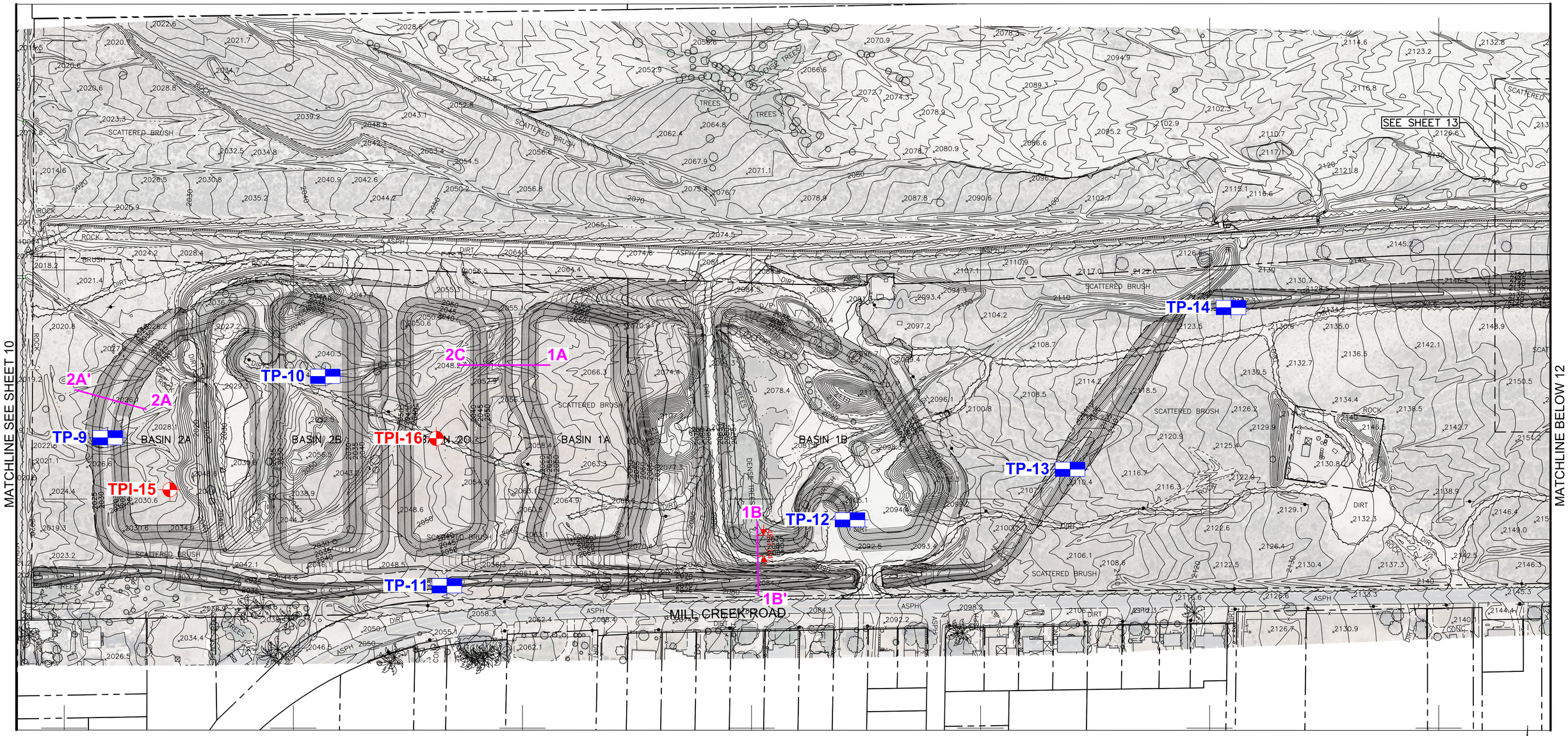
ATRP MILL CREEK
PRELIMINARY DESIGN
076-RG-00 - 09

PROJECT NO. SBVWCD - 0000

SHEET 09

OF 34

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EXPLANATION

- TPI-16 — APPROXIMATE BOREHOLE INFILTRATION LOCATION
- TP-16 — APPROXIMATE TEST PIT LOCATION
- PI-7 — APPROXIMATE TEST PIT INFILTRATION LOCATION
- 35-36 — CROSS SECTION

1" = 100'
50 0 100

811 Know what's below. Call before you dig. DIG ALERT Underground Service Alert

Section 4216/4216.9 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert 811 two working days before you dig

BENCHMARK:

NO.	DATE	REVISIONS	APPROVED	DATE
1				
2				
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Prepared under the supervisor of:

DRAWN BY:

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CHECKED BY:

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SAN BERNARDINO VALLEY WATER CONVERSATION DISTRICT

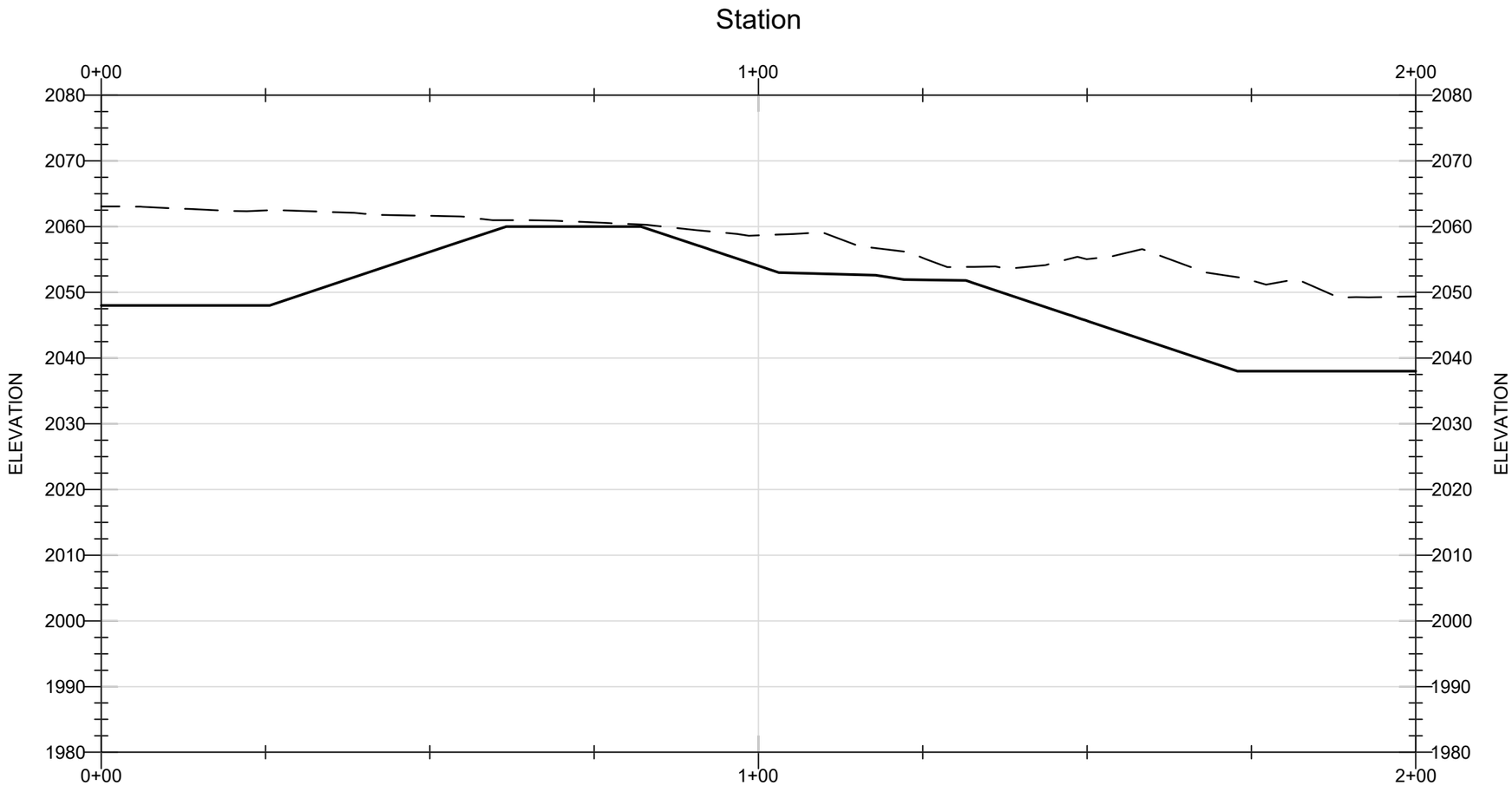
ATRP MILL CREEK
PRELIMINARY DESIGN
076-RG-00 - 11

PROJECT NO.
SBVWCD -
0000

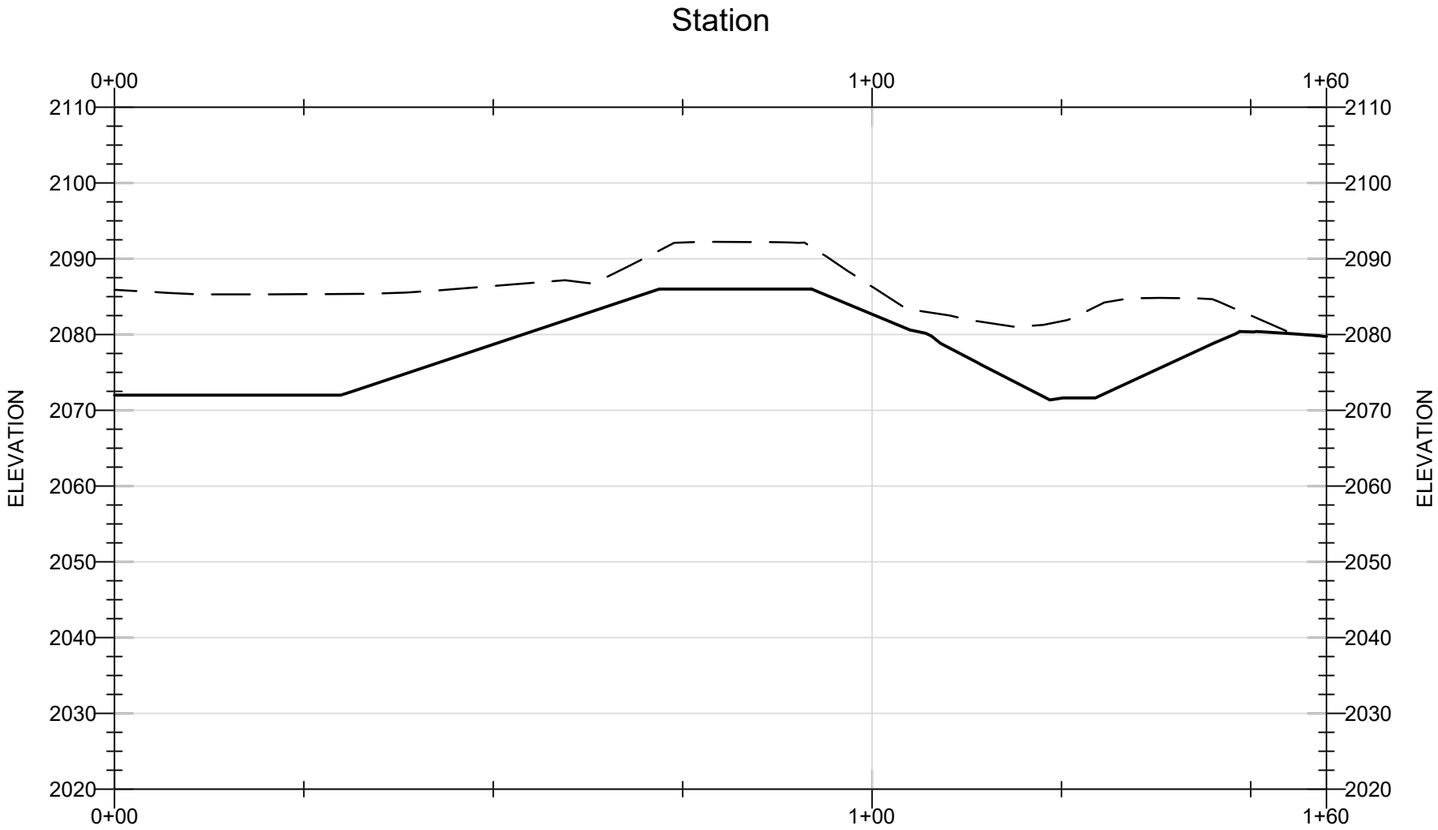
SHEET 11
OF 34

Plotted by: DFahrney Mar. 23, 2025 07:19 PM C:\Users\DFahrney\OneDrive - Kleinfelder\CADD\20230500\Files\076-RGP\076-RG-00.dwg

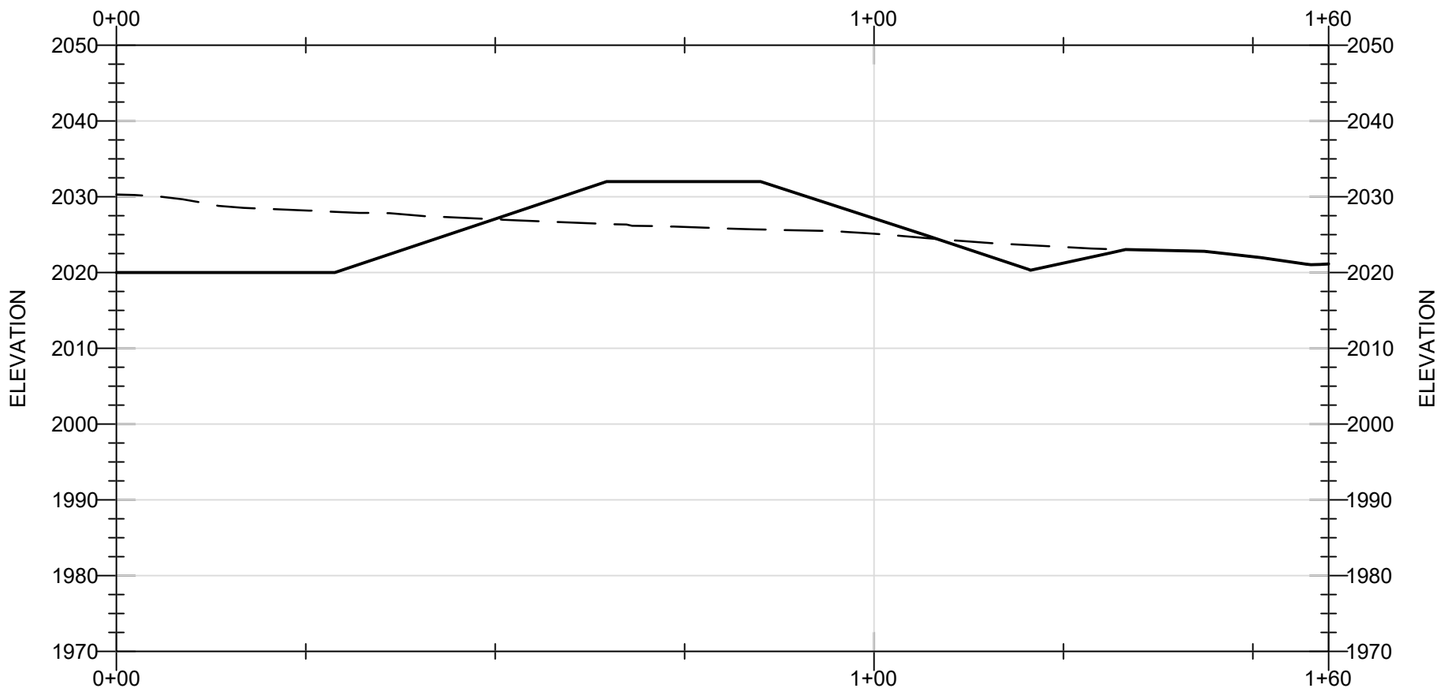
1A-2C PROFILE



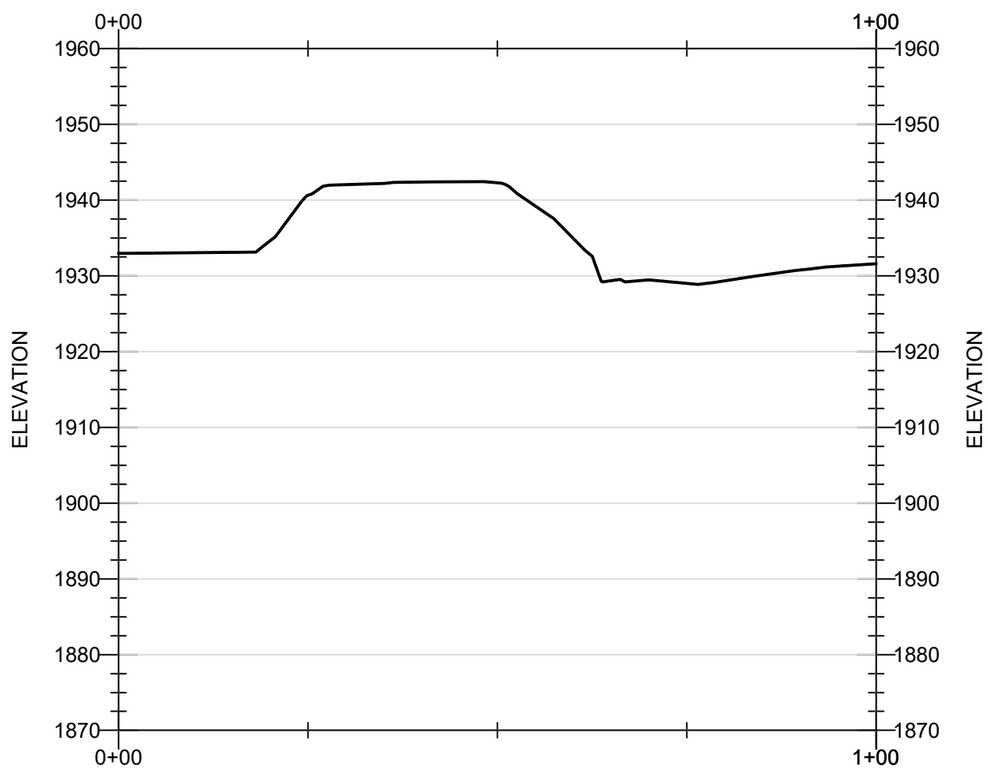
1B PROFILE


Station
0+00
1+00
1+60
2110
2100
2090
2080
2070
2060
2050
2040
2030
2020
ELEVATION
ELEVATION

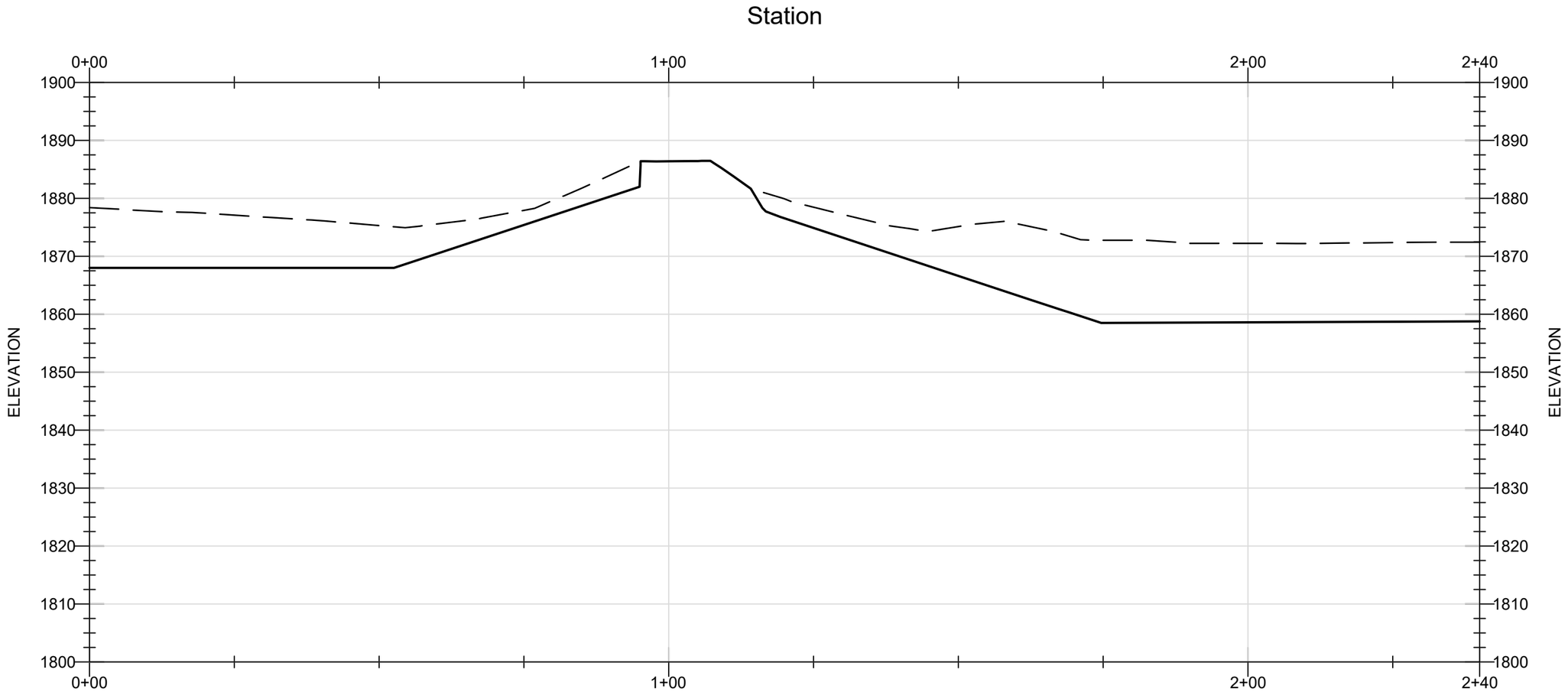
2A PROFILE


Station
0+00
1+00
1+60
2050
2040
2030
2020
2010
2000
1990
1980
1970
ELEVATION

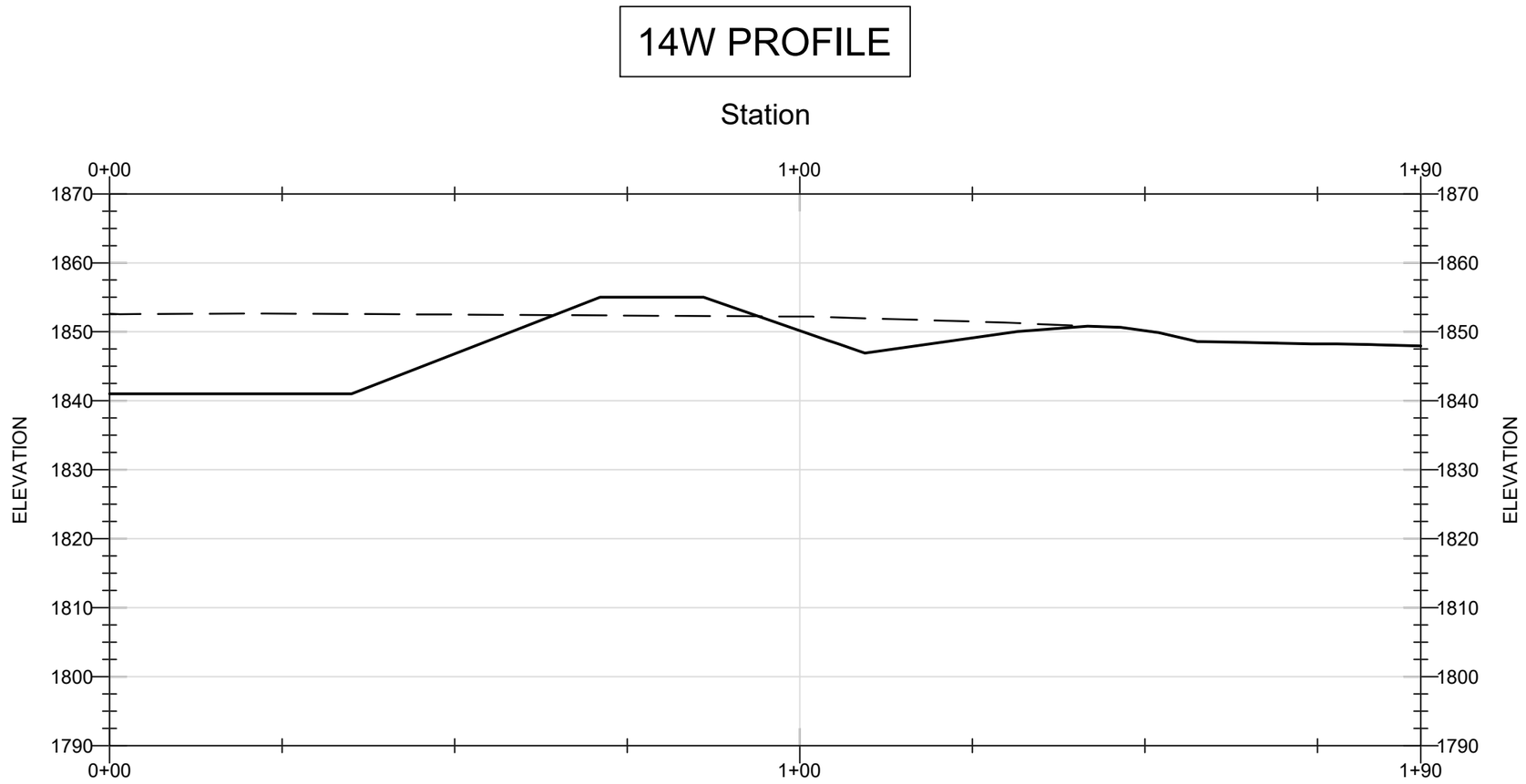
12 PROFILE


Station
0+00
1+00
1960
1950
1940
1930
1920
1910
1900
1890
1880
1870
ELEVATION

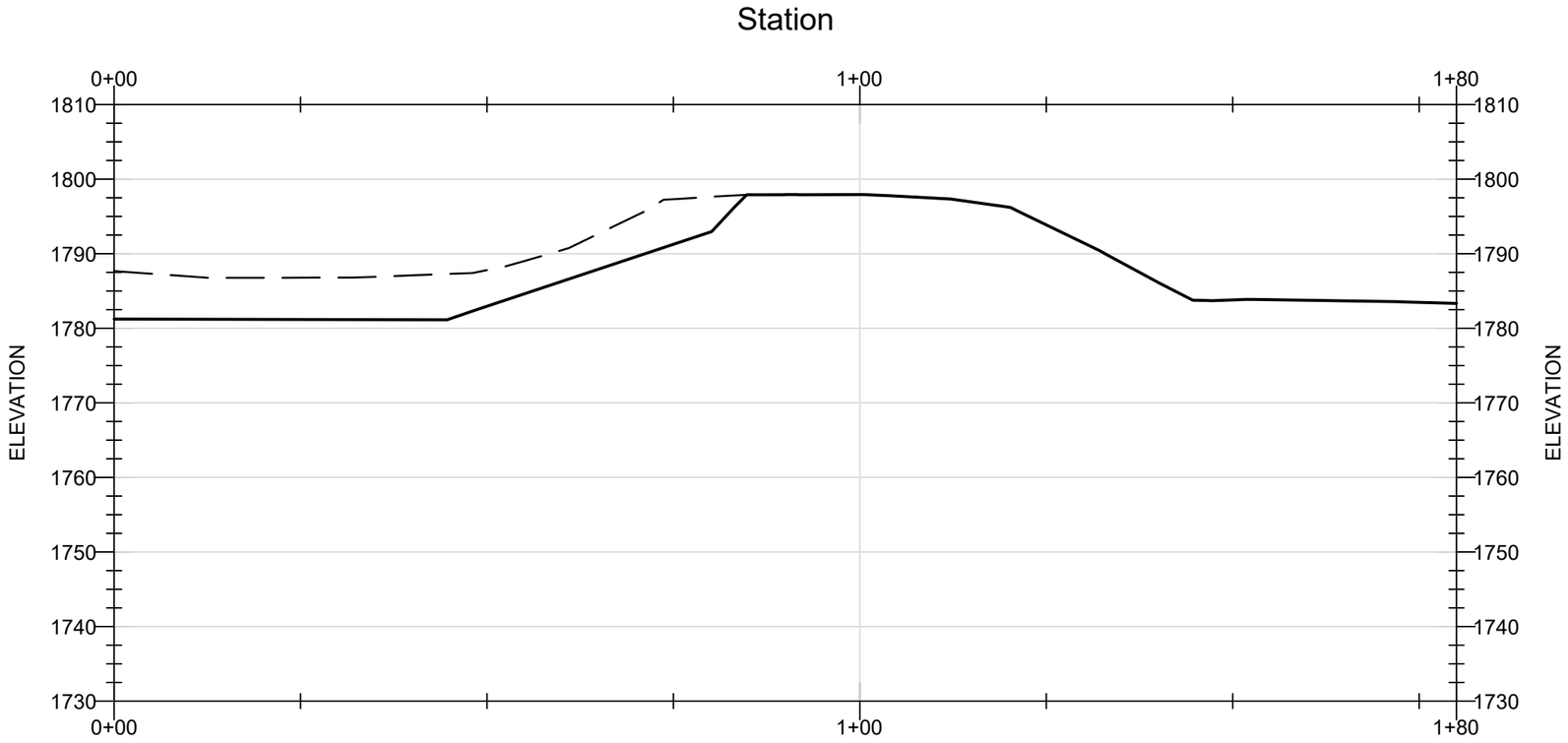
14-15W PROFILE



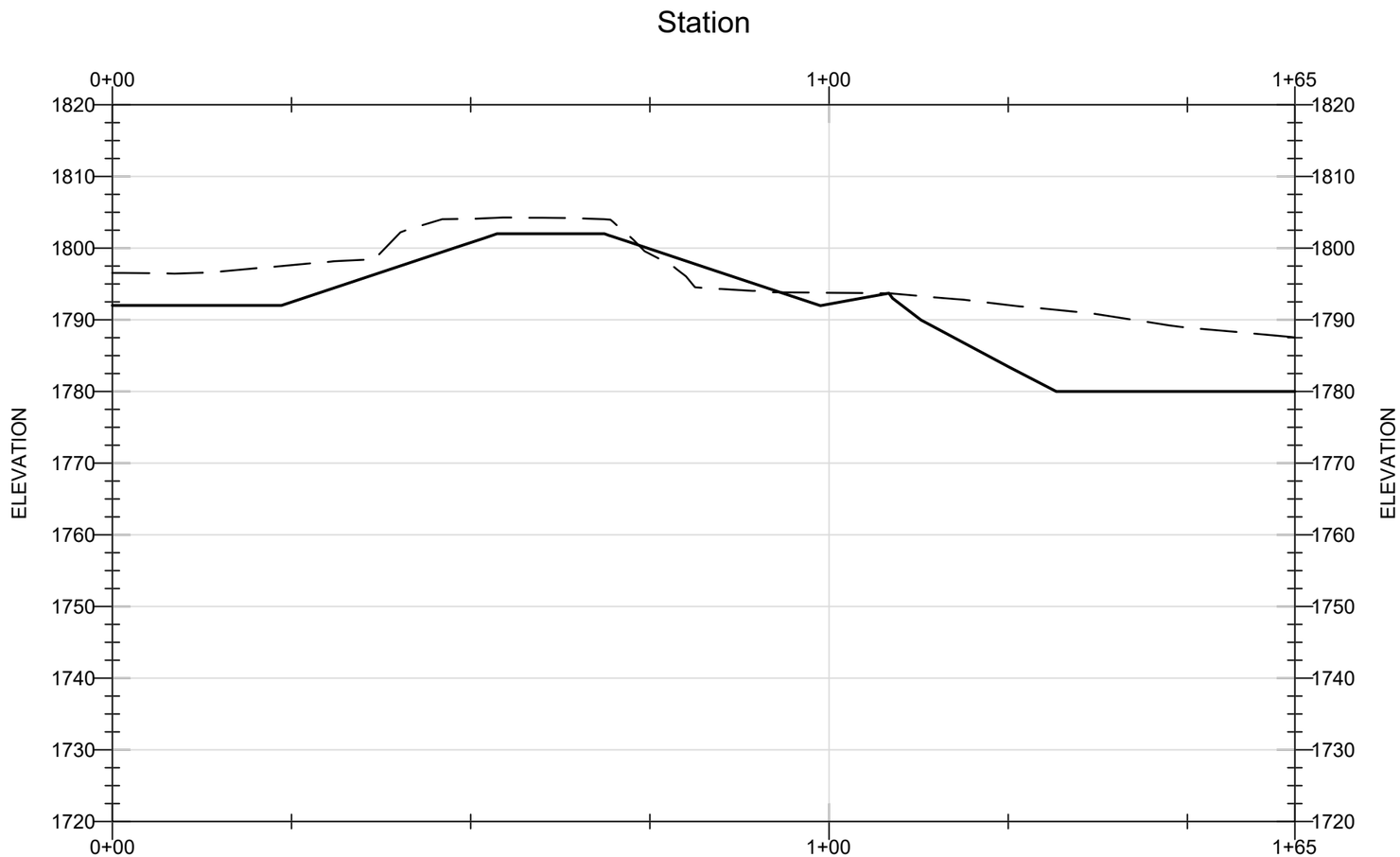
14W PROFILE


Station
0+00
1+00
1+90
1870
1860
1850
1840
1830
1820
1810
1800
1790
ELEVATION
ELEVATION

24 PROFILE


Station
0+00
1+00
1+80
ELEVATION
1810
1800
1790
1780
1770
1760
1750
1740
1730

35-36 PROFILE



APPENDIX C.2

ATTACHMENT 2

Peak Ground Accelerations

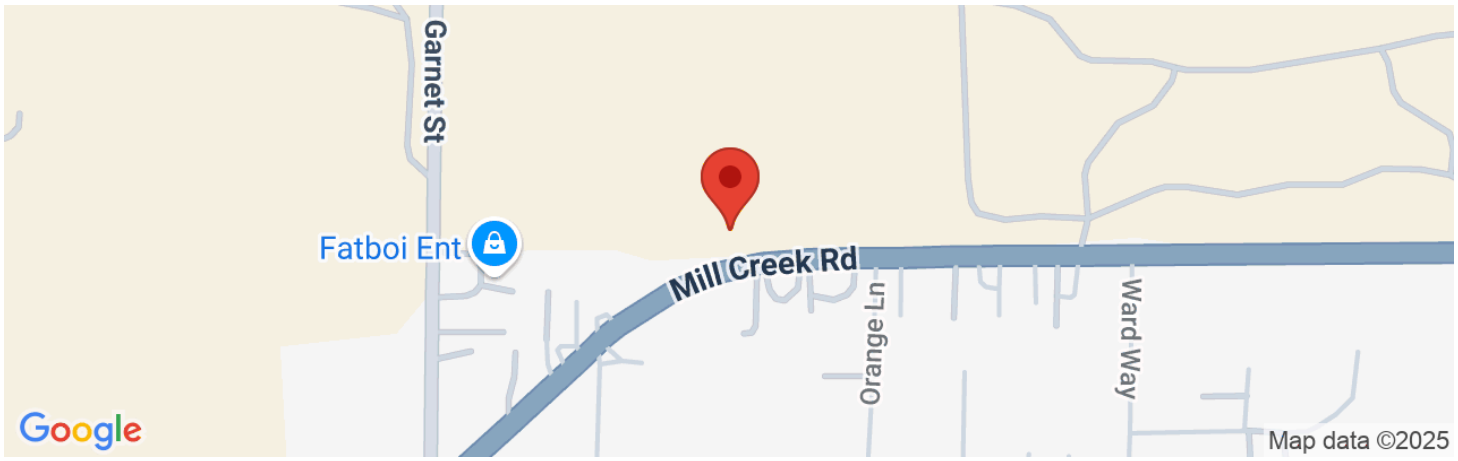
DRAFT

© 2025 Kleinfelder

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

Mill Creek Basin

Latitude, Longitude: 34.07342, -117.09661



Date	4/14/2025, 5:42:46 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	2.262	MCE_R ground motion. (for 0.2 second period)
S_1	0.923	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.714	Site-modified spectral acceleration value
S_{M1}	1.293	Site-modified spectral acceleration value
S_{DS}	1.809	Numeric seismic design value at 0.2 second SA
S_{D1}	0.862	Numeric seismic design value at 1.0 second SA

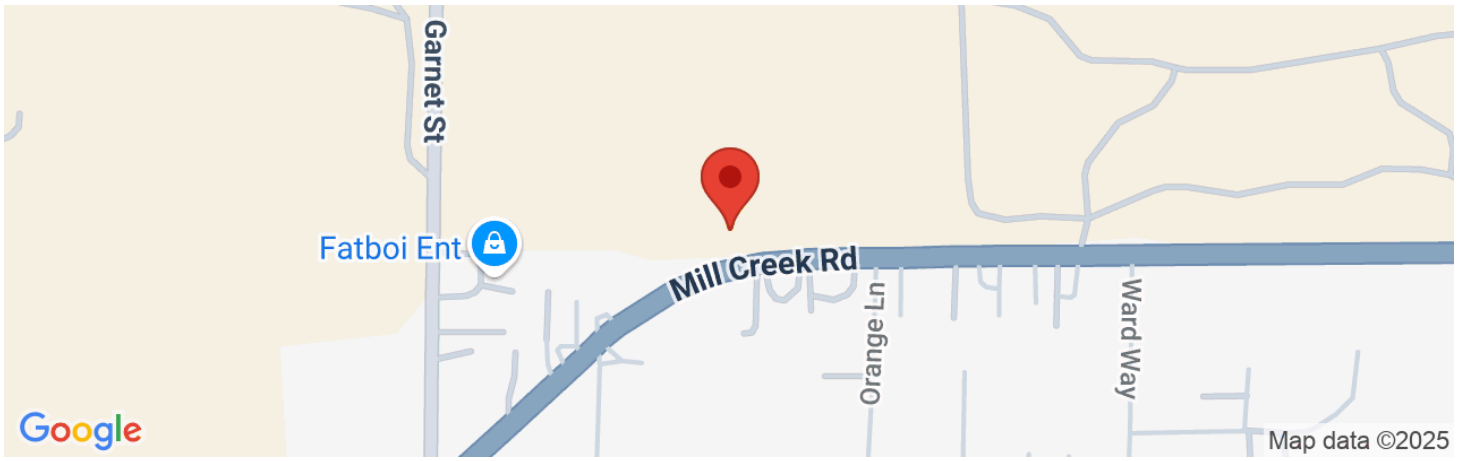
Type	Value	Description
SDC	E	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.4	Site amplification factor at 1.0 second
PGA	0.942	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.13	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.808	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	3.105	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.262	Factored deterministic acceleration value. (0.2 second)
S1RT	1.113	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.256	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.923	Factored deterministic acceleration value. (1.0 second)
PGAd	0.942	Factored deterministic acceleration value. (Peak Ground Acceleration)

Type	Value	Description
PGA_{UH}	1.218	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.905	Mapped value of the risk coefficient at short periods
C_{R1}	0.886	Mapped value of the risk coefficient at a period of 1 s
C_V	1.3	Vertical coefficient

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

Mill Creek Basin

Latitude, Longitude: 34.07342, -117.09661



Date	4/14/2025, 5:45:01 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.262	MCE_R ground motion. (for 0.2 second period)
S_1	0.923	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.262	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.508	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.942	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	1.036	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.808	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	3.105	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.262	Factored deterministic acceleration value. (0.2 second)
S1RT	1.113	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.256	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.923	Factored deterministic acceleration value. (1.0 second)
PGAd	0.942	Factored deterministic acceleration value. (Peak Ground Acceleration)

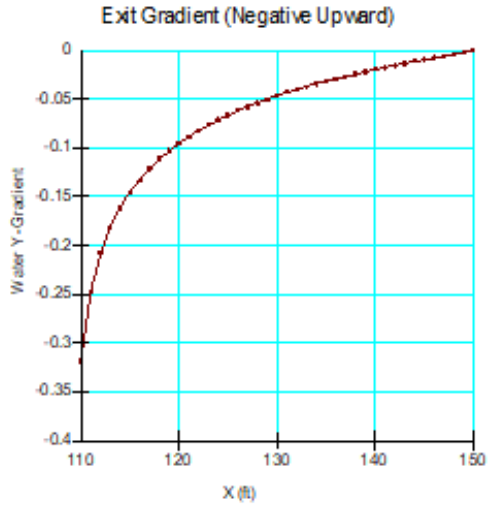
Type	Value	Description
PGA_{UH}	1.218	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.905	Mapped value of the risk coefficient at short periods
C_{R1}	0.886	Mapped value of the risk coefficient at a period of 1 s
C_V	1.5	Vertical coefficient

APPENDIX C.2
ATTACHMENT 3
GeoStudio Plots

DRAFT

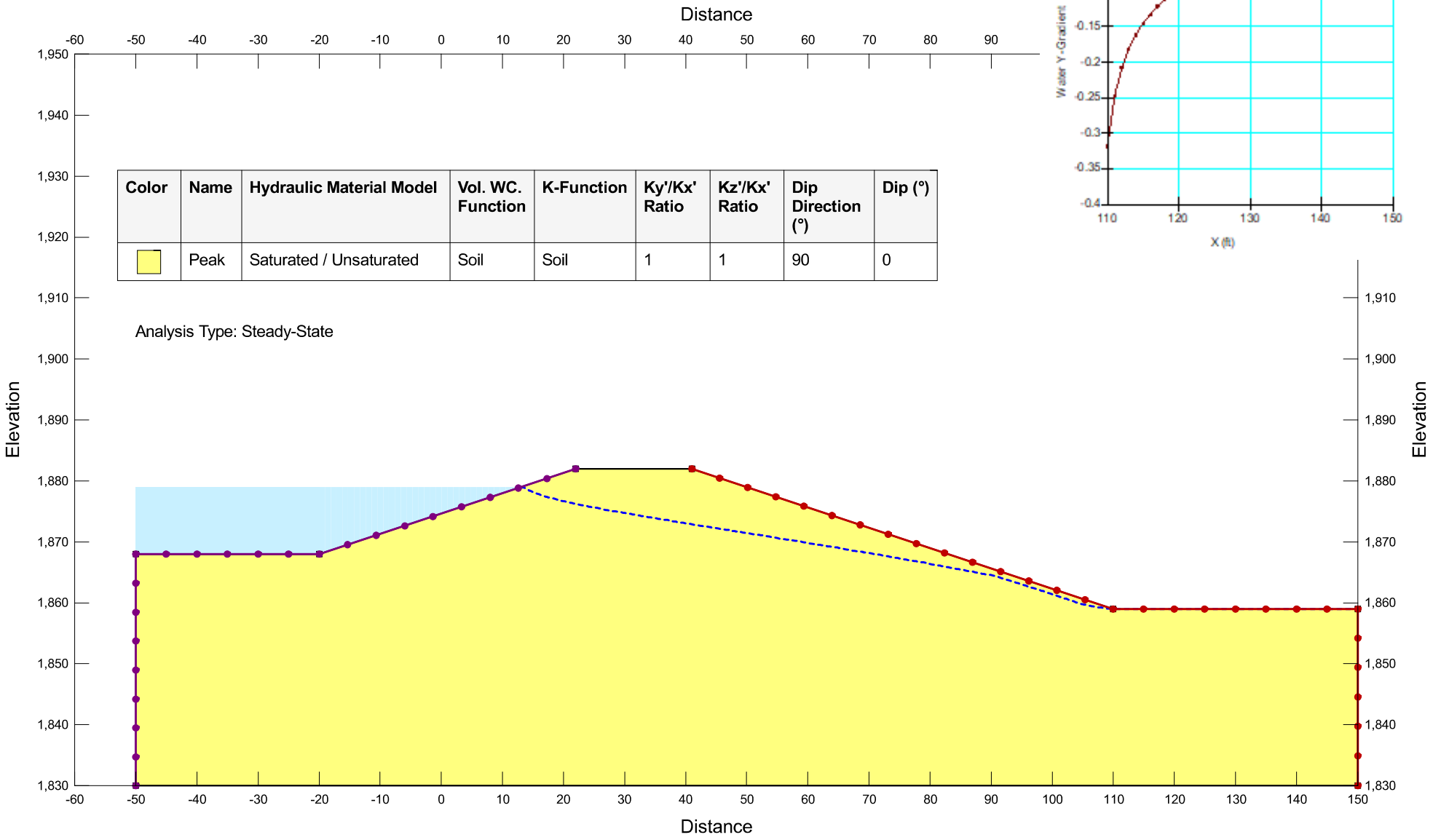
© 2025 Kleinfelder

Section A (14-15W)
Seepage



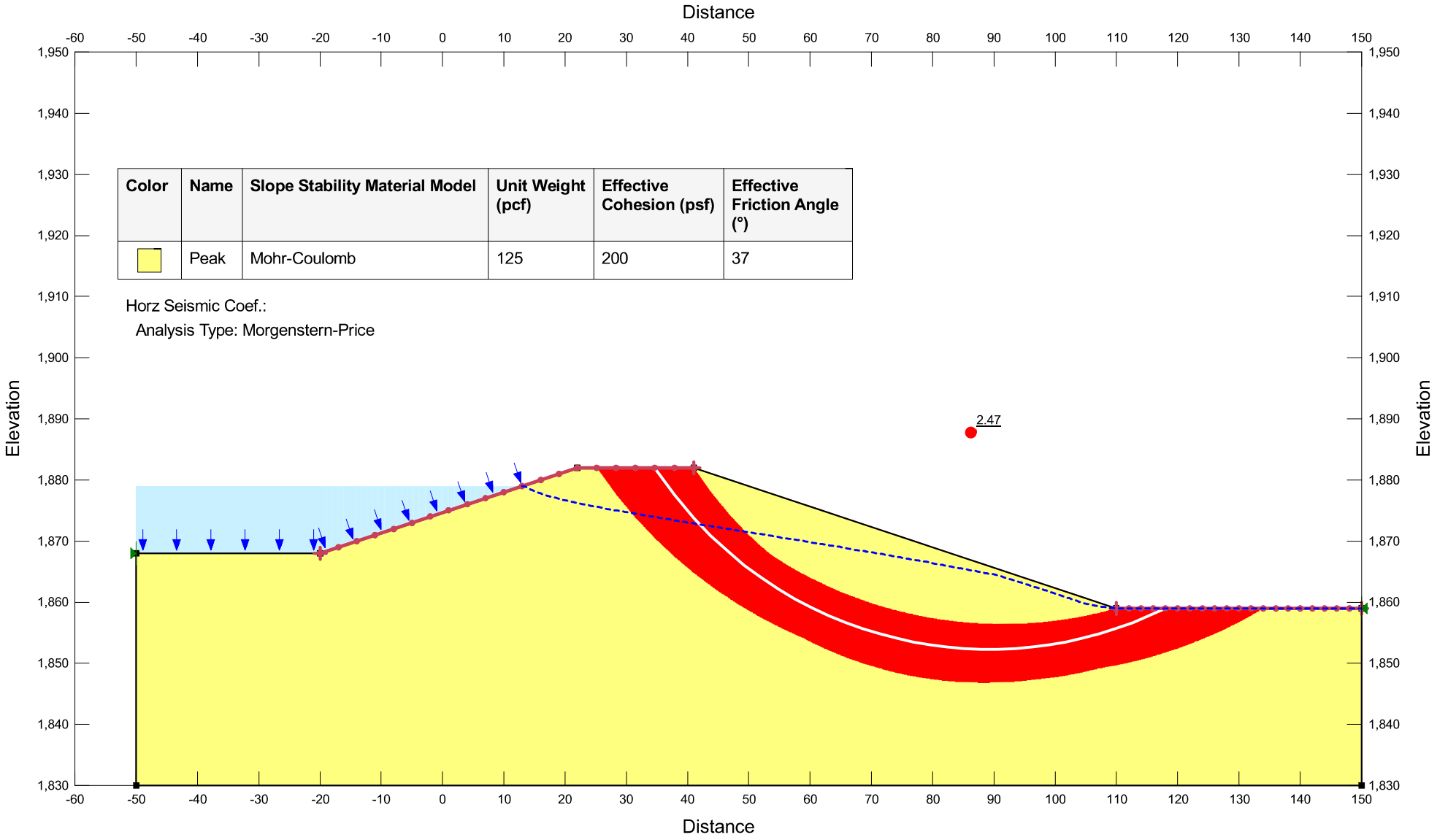
Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Kz'/Kx' Ratio	Dip Direction (°)	Dip (°)
	Peak	Saturated / Unsaturated	Soil	Soil	1	1	90	0

Analysis Type: Steady-State



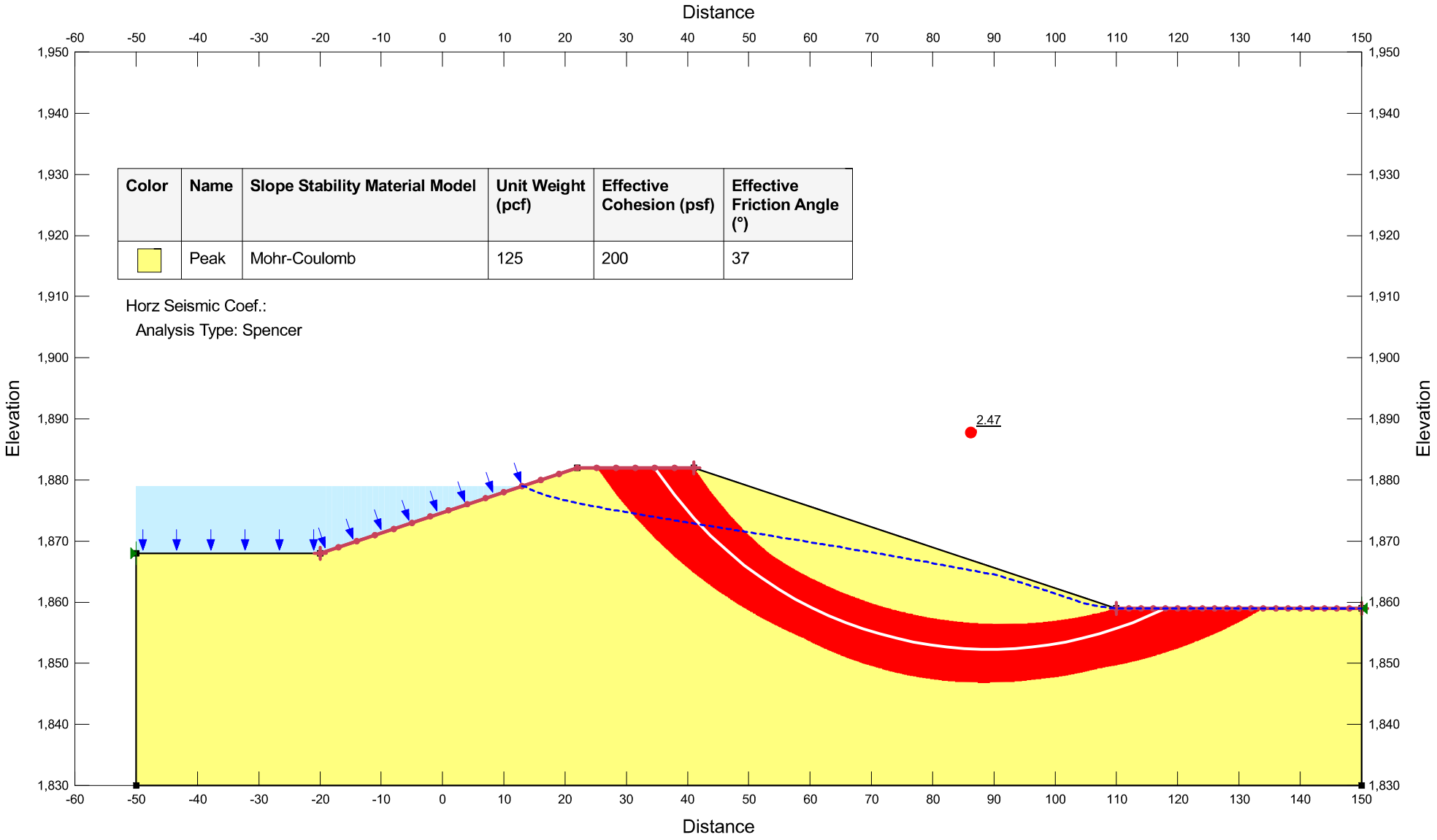
Title: Mill Creek Basin. Name: Seepage
File Name: MCB_Sec_14-15W_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section A (14-15W)
Static (M-P)



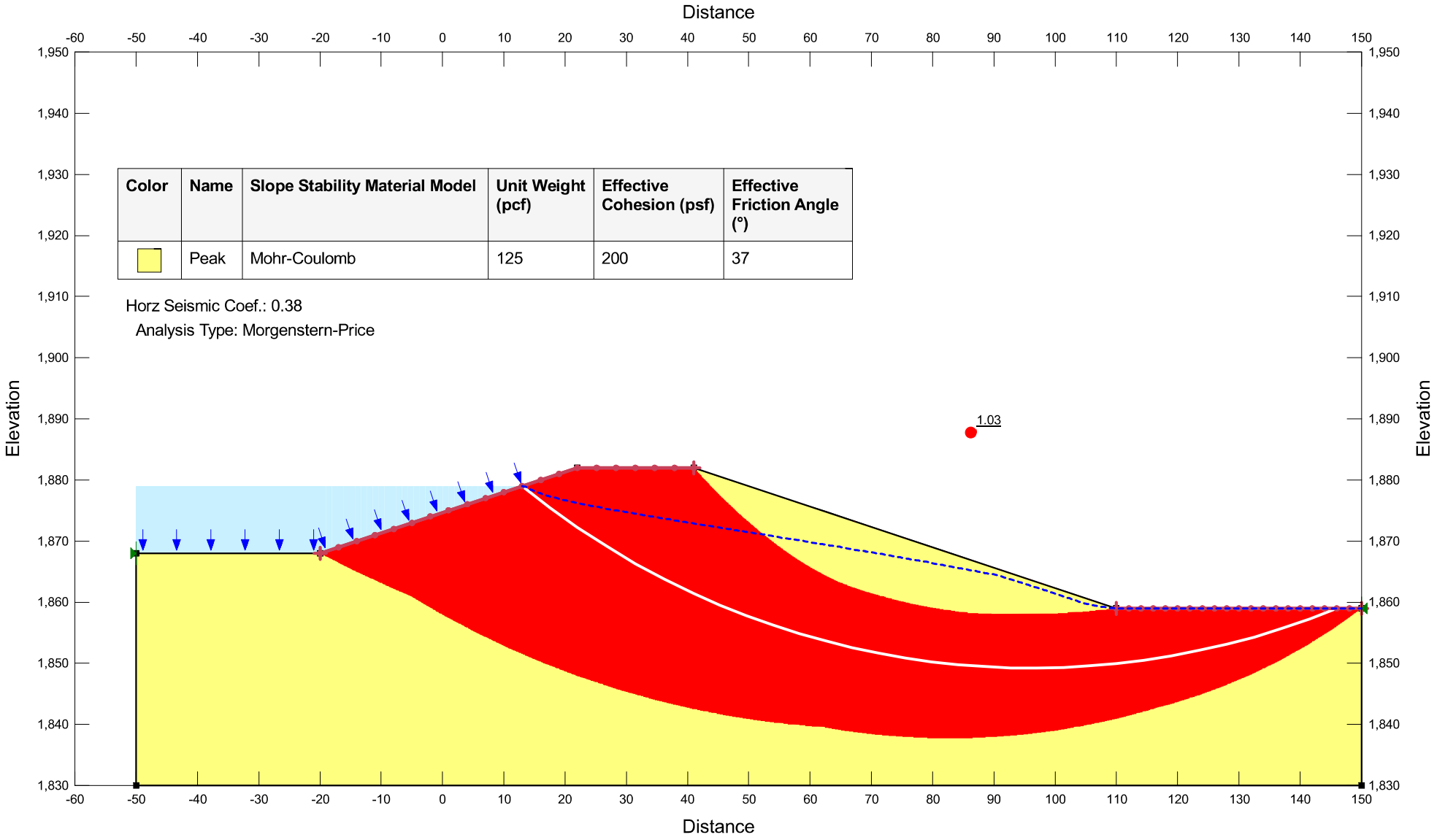
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Created By: Qiqi Huang
Date:04/15/2025

Section A (14-15W)
Static (Spencer)



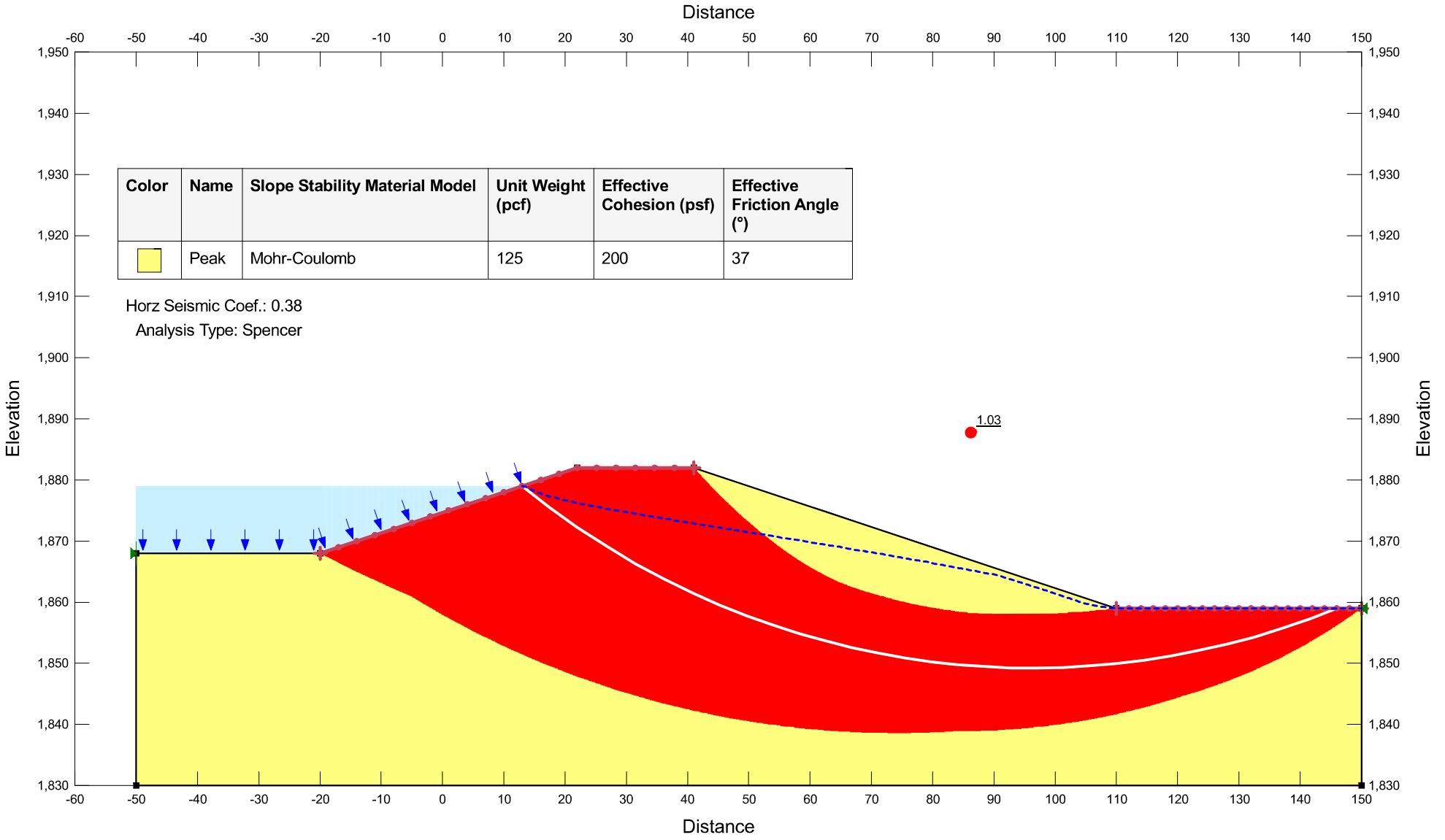
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File Name: MCB_Sec_14-15W_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section A (14-15W)
Seismic W K_{eh} (M-P)



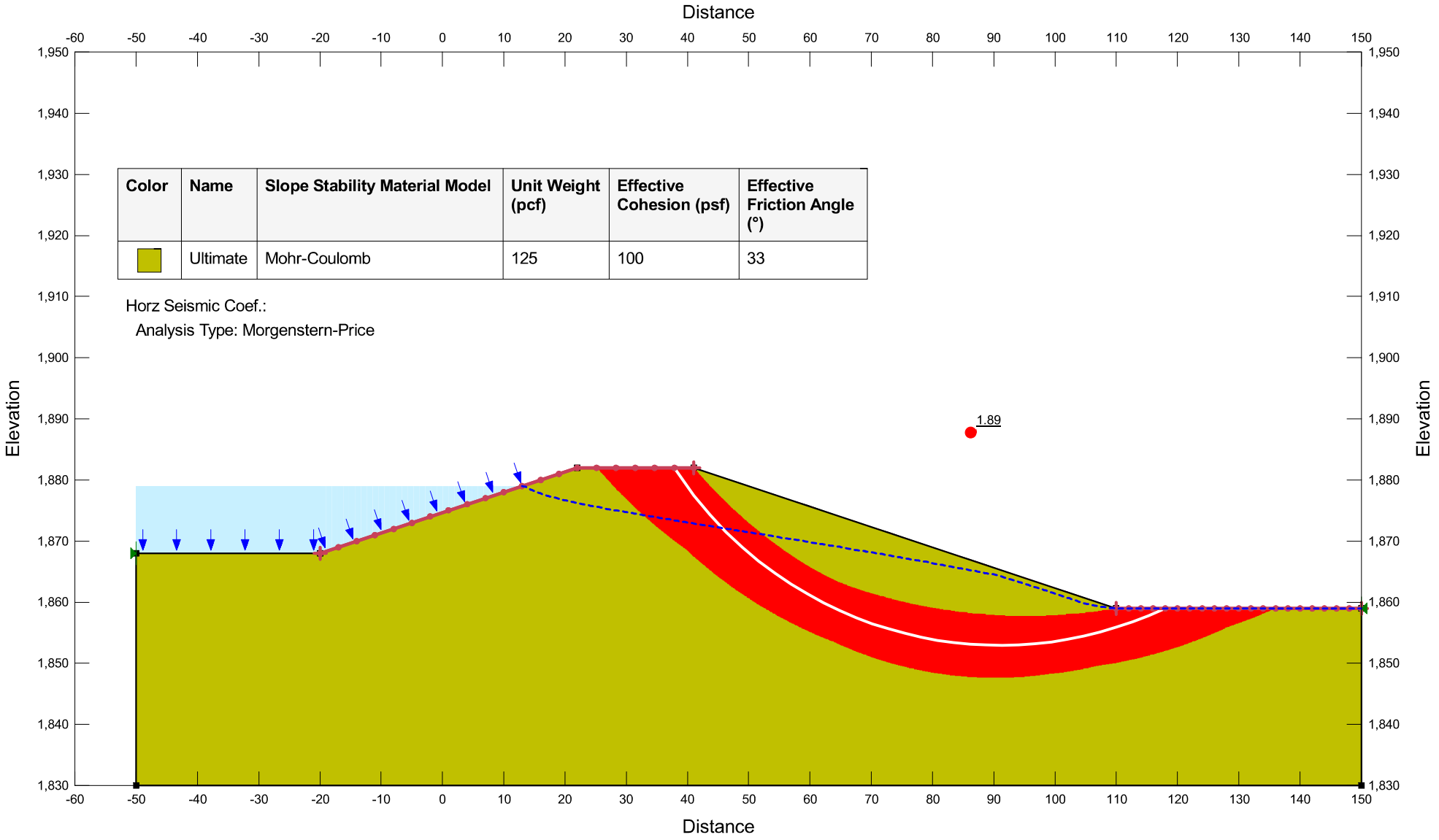
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File Name: MCB_Sec_14-15W_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section A (14-15W)
Seismic W K_{eh} (Spencer)



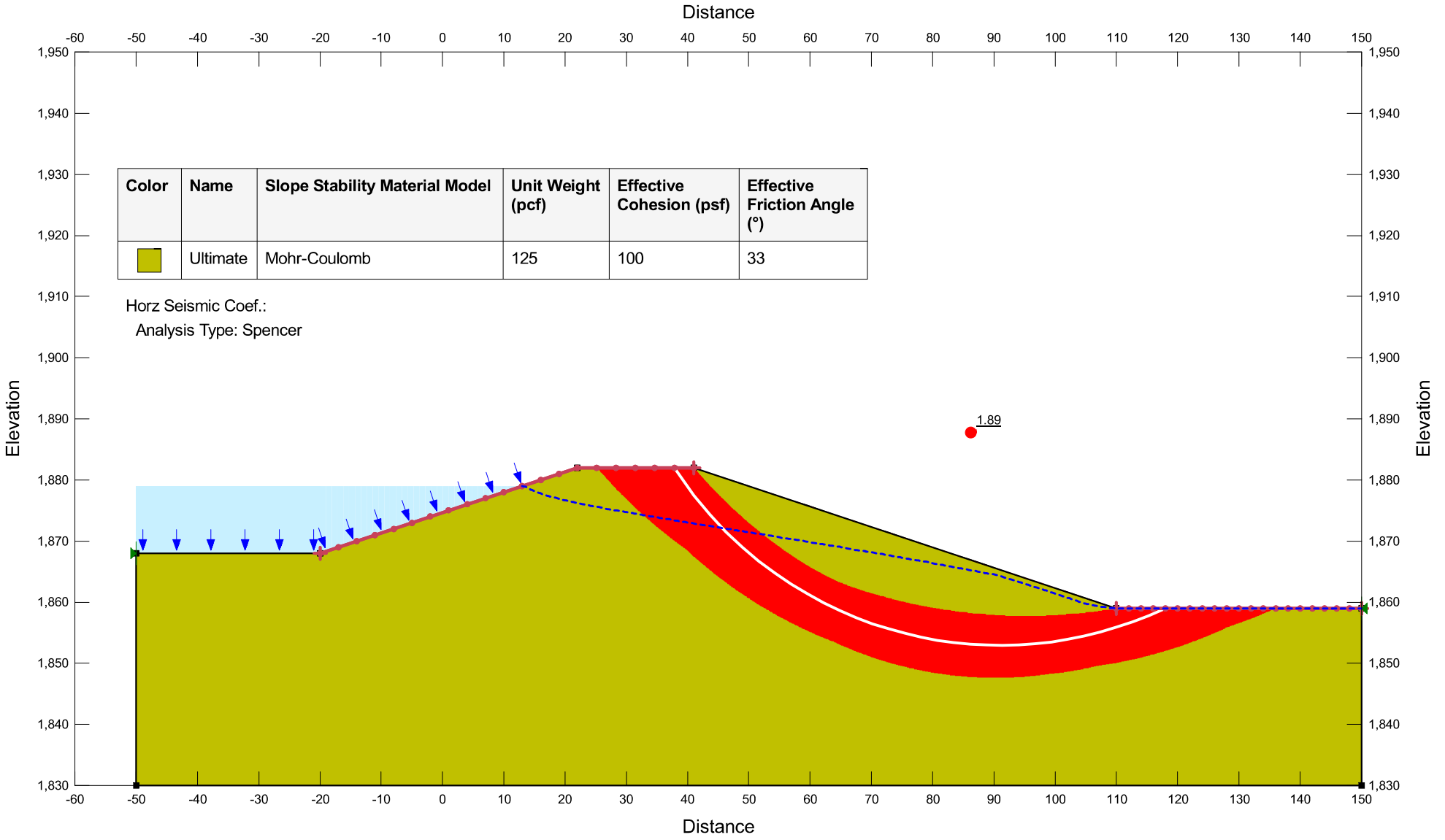
Title Mill Creek Basin Name Seismic_Peak_Spence
File Name: MCB_Sec_14-15W_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section A (14-15W)
Seismic W/O K_{eh} (M-P)



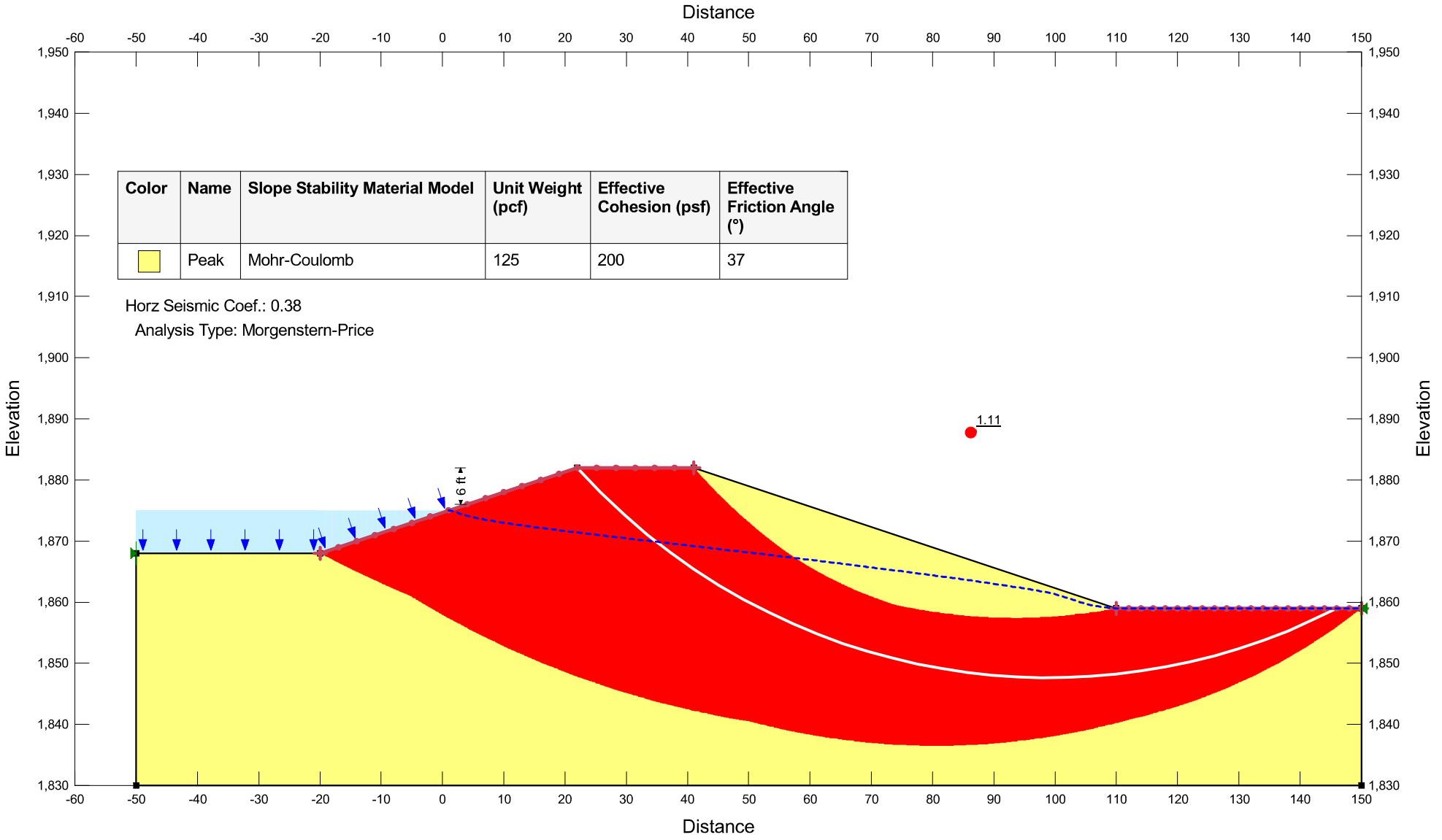
Title: Mill Creek Basin. Name: Seismic_Ultimate_M-P
File Name: MCB_Sec_14-15W_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section A (14-15W)
Seismic W/O K_{eh} (Spencer)



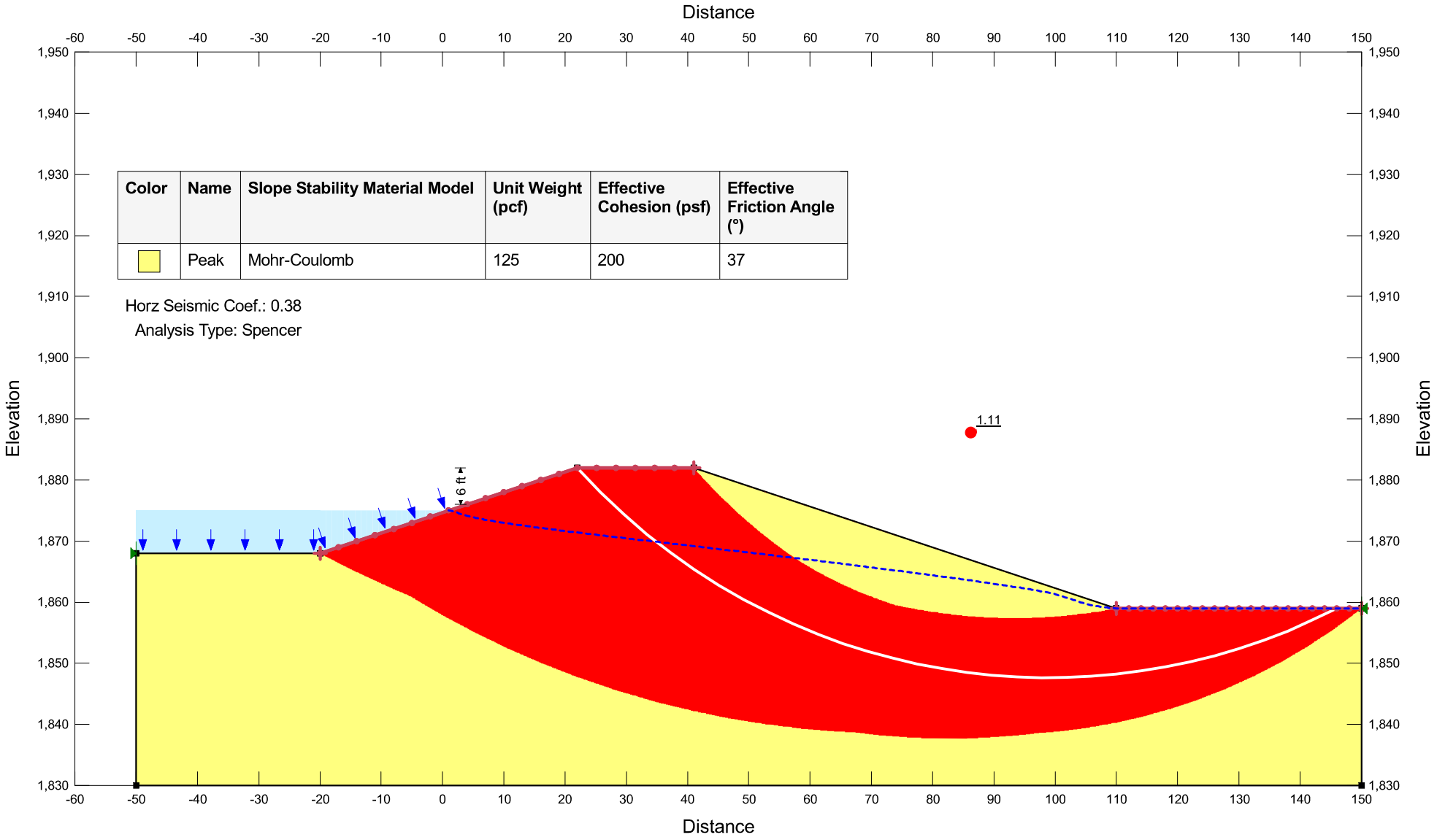
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File Name: MCB_Sec_14-15W_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section A (14-15W)
Seismic W K_{eh} (M-P) - FS=1.10



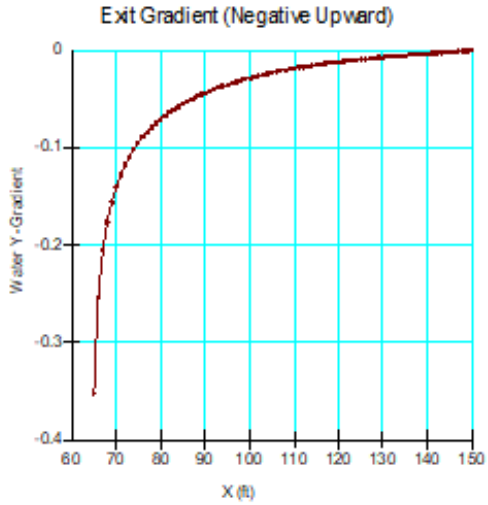
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Created By: Qiqi Huang
Date:04/15/2025

Section A (14-15W)
Seismic W K_{eh} (Spencer) - FS=1.10

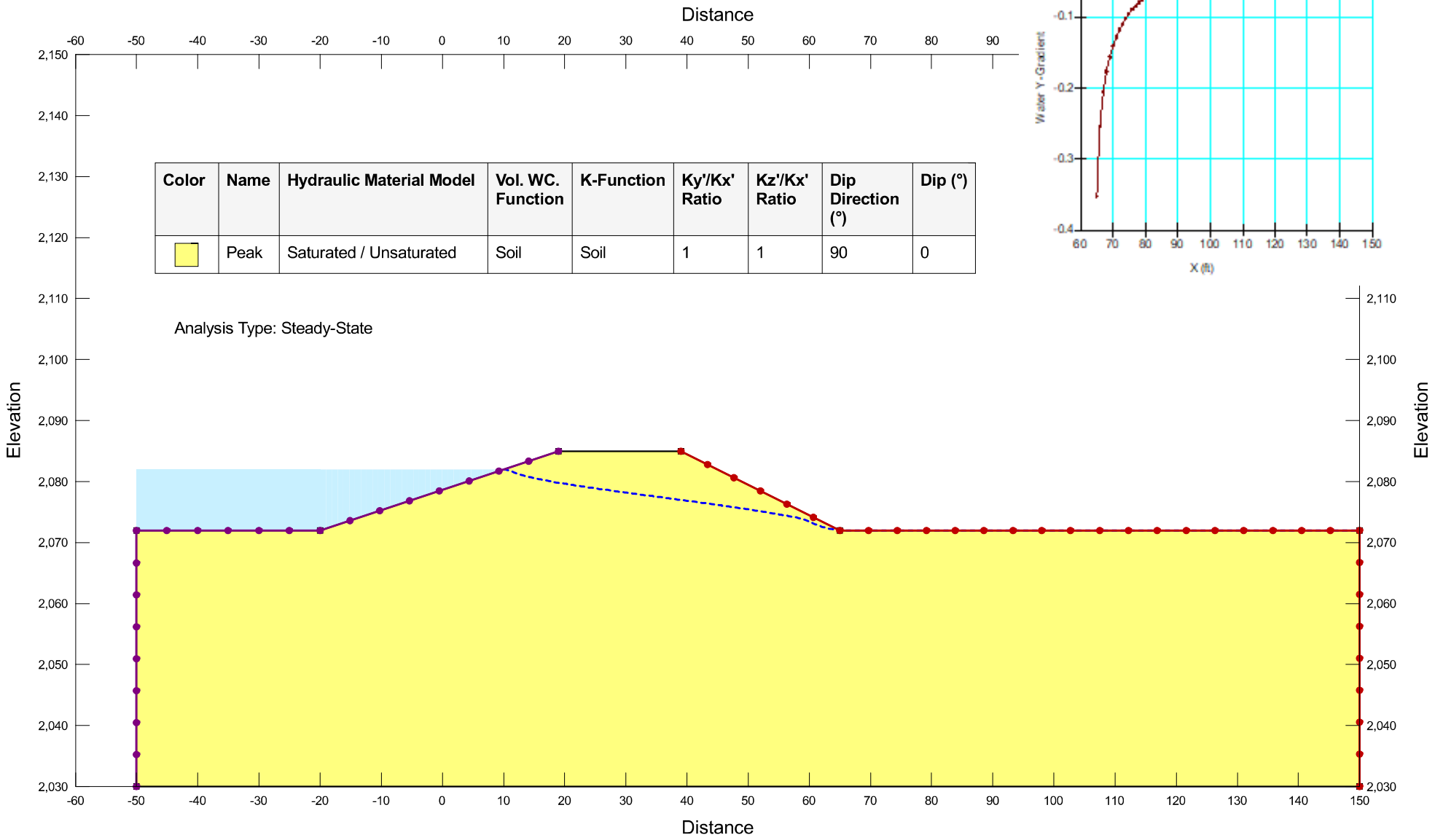


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File Name: MCB_Sec_14-15W_EQ1.1.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section B (1B)
Seepage

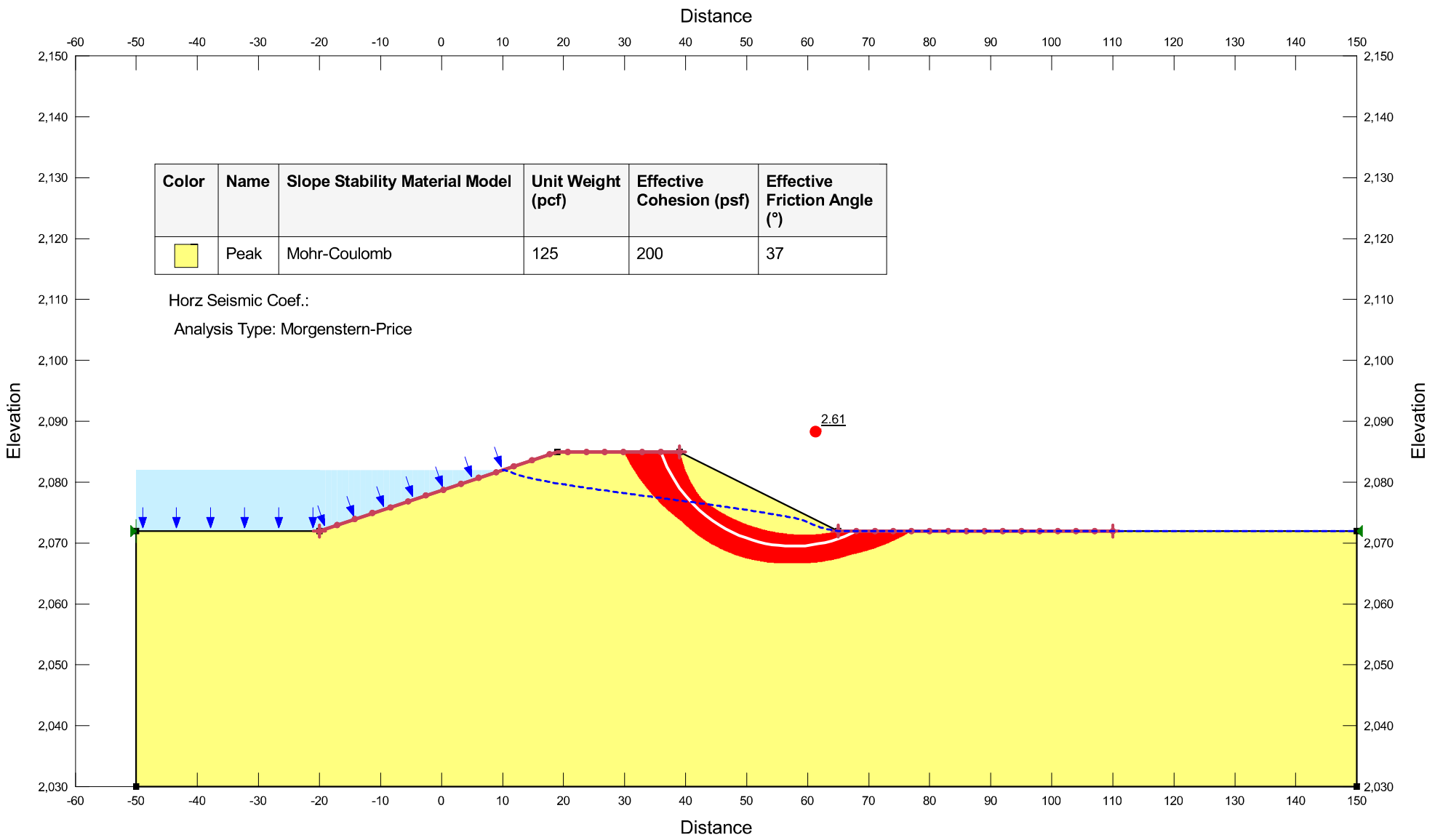


Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Kz'/Kx' Ratio	Dip Direction (°)	Dip (°)
	Peak	Saturated / Unsaturated	Soil	Soil	1	1	90	0



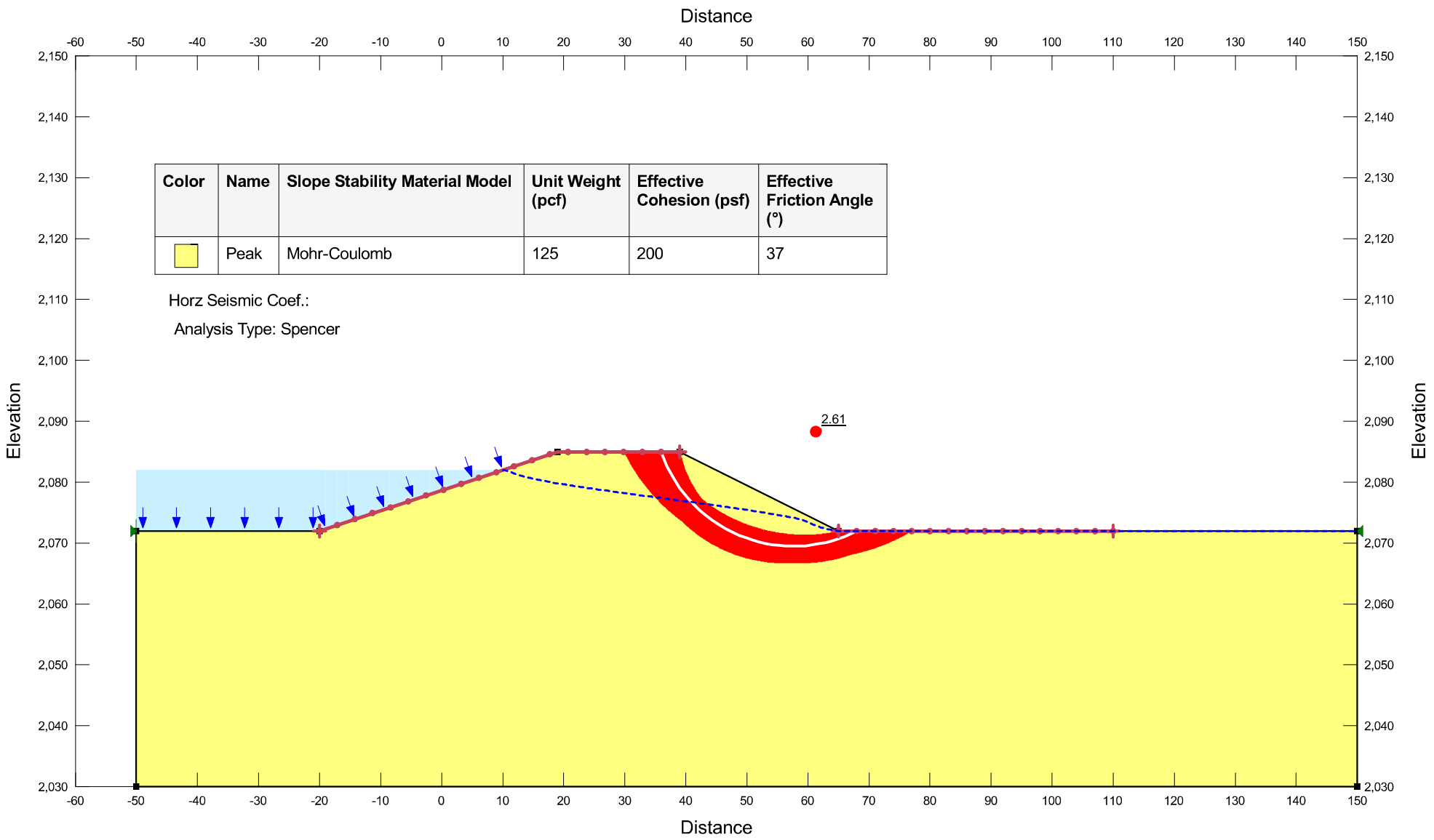
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File Name: MCB_Sec_1B_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section B (1B)
Static (M-P)



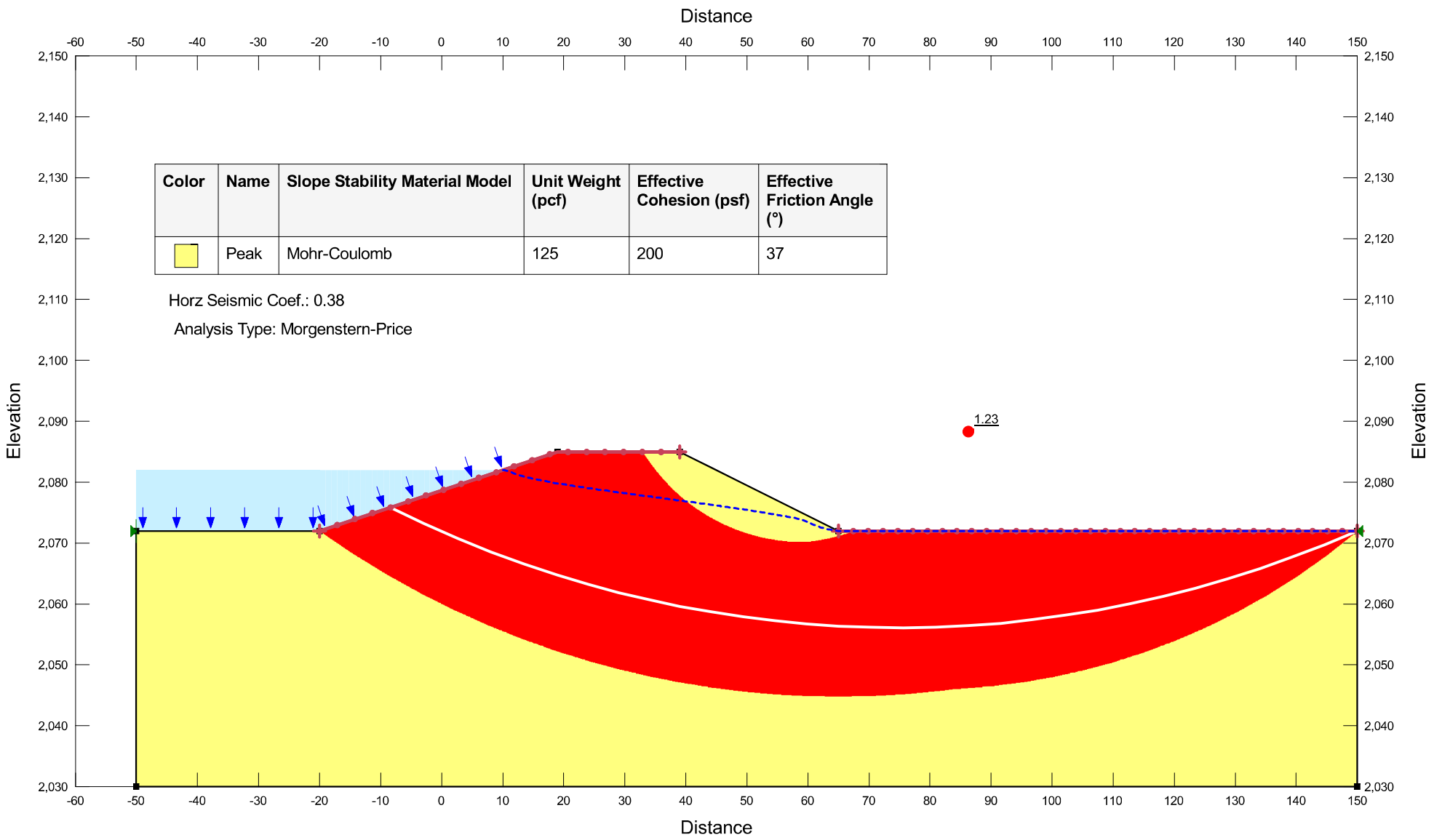
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Created By: Qiqi Huang
Date:04/15/2025

Section B (1B)
Static (Spencer)



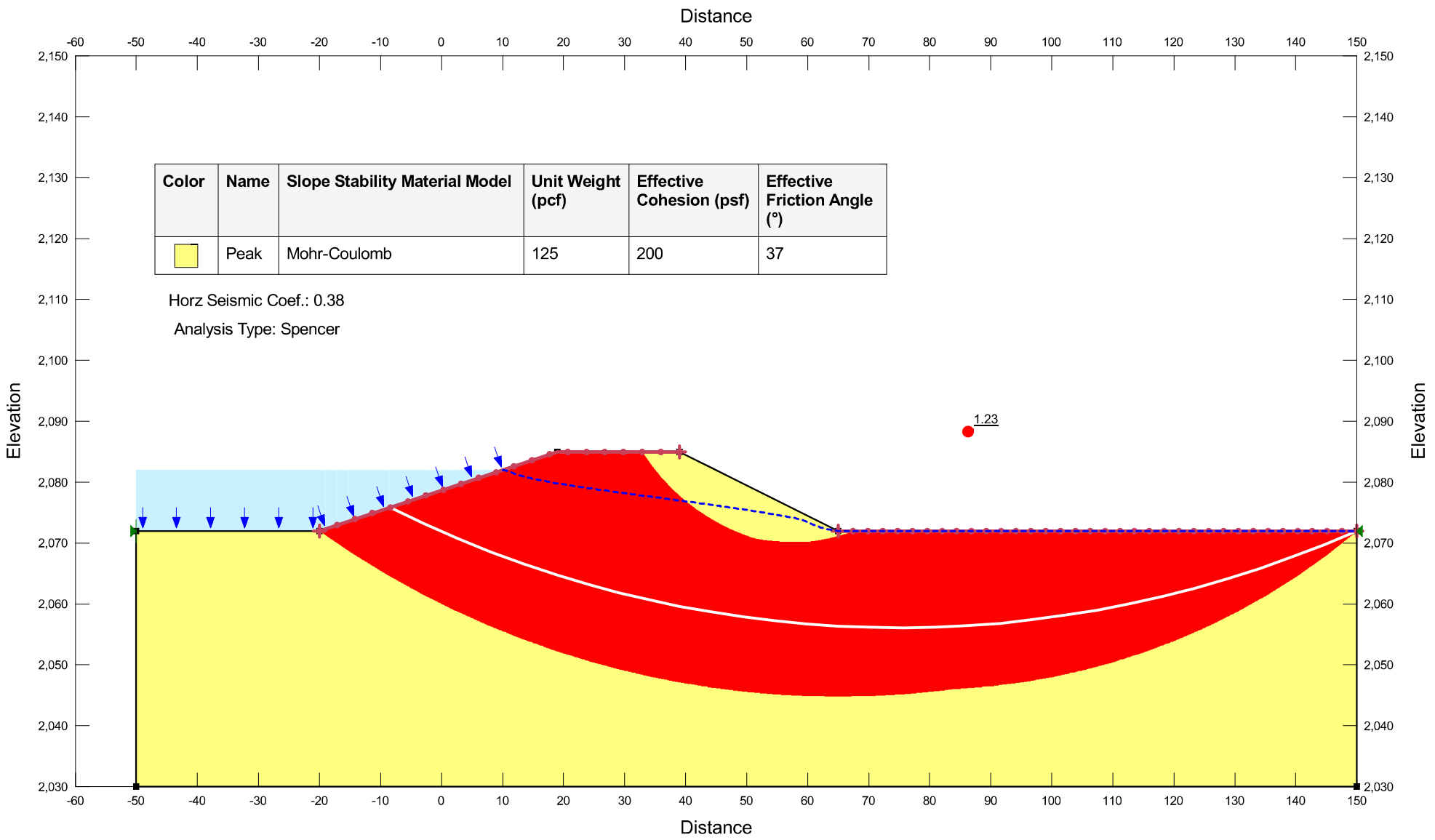
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Created By: Qiqi Huang
Date:04/15/2025

Section B (1B)
Seismic W K_{eh} (M-P)



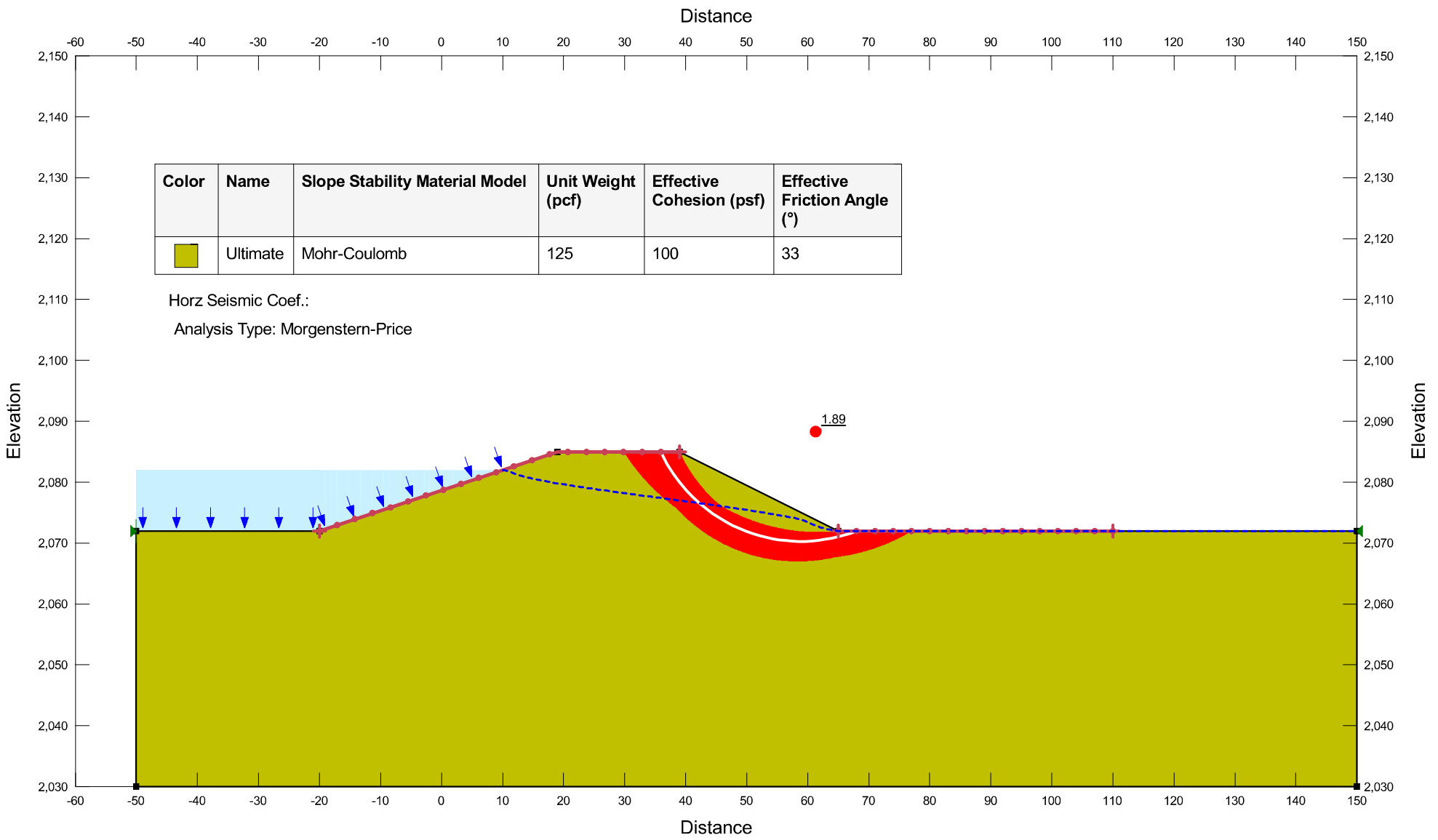
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File Name: MCB_Sec_1B_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section B (1B)
Seismic W K_{eh} (Spencer)



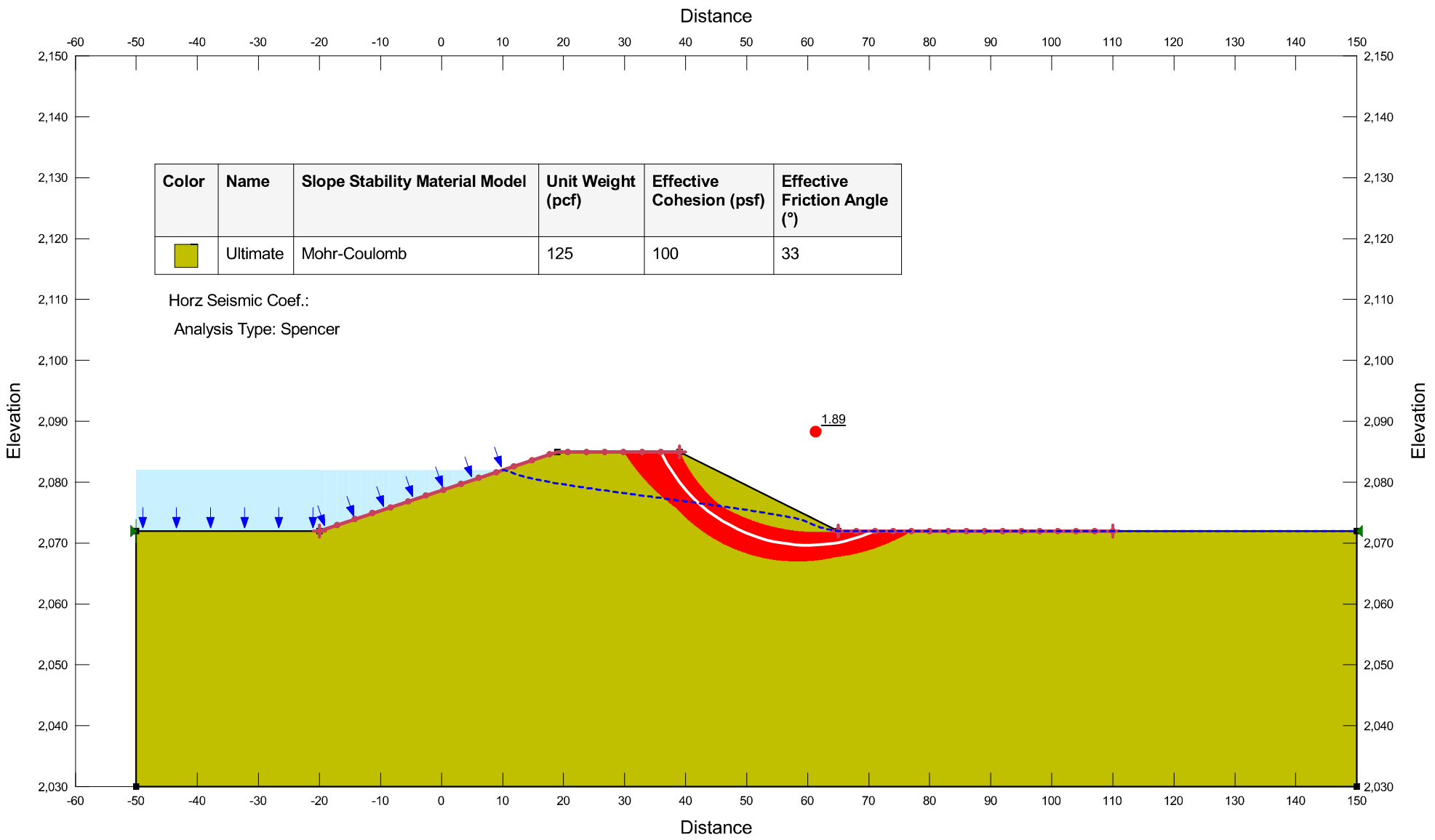
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Created By: Qiqi Huang
Date:04/15/2025

Section B (1B)
Seismic W/O K_{eh} (M-P)



Title: Mill Creek Basin. Name: Seismic_Ultimate_M-P
File Name: MCB_Sec_1B_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section B (1B)
Seismic W/O K_{eh} (Spencer)



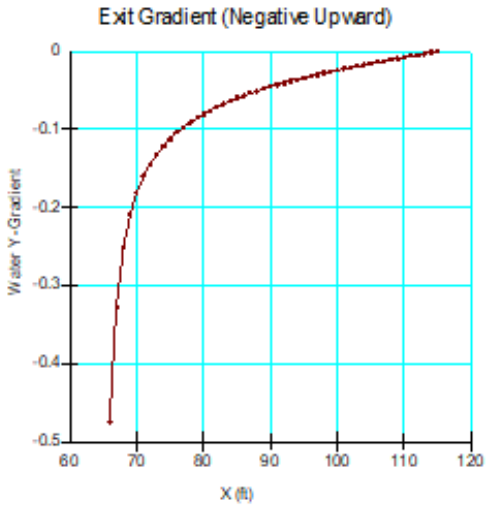
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Ultimate	Mohr-Coulomb	125	100	33

Horz Seismic Coef.:

Analysis Type: Spencer

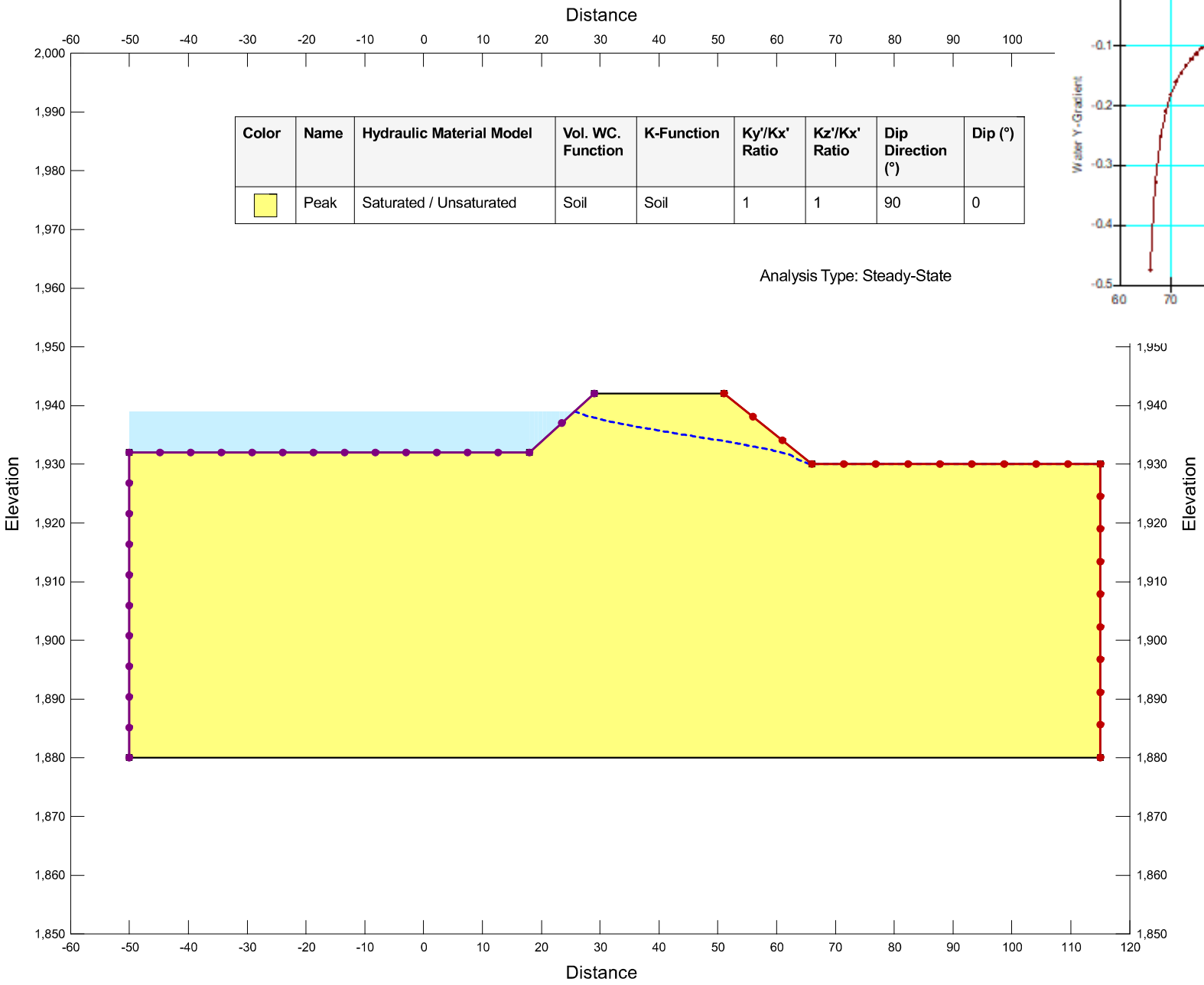
Title: Mill Creek Basin. Name: Seismic_Ultimate_Spencer
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Created By: Qiqi Huang
Date:04/15/2025

Section C (12)
Seepage



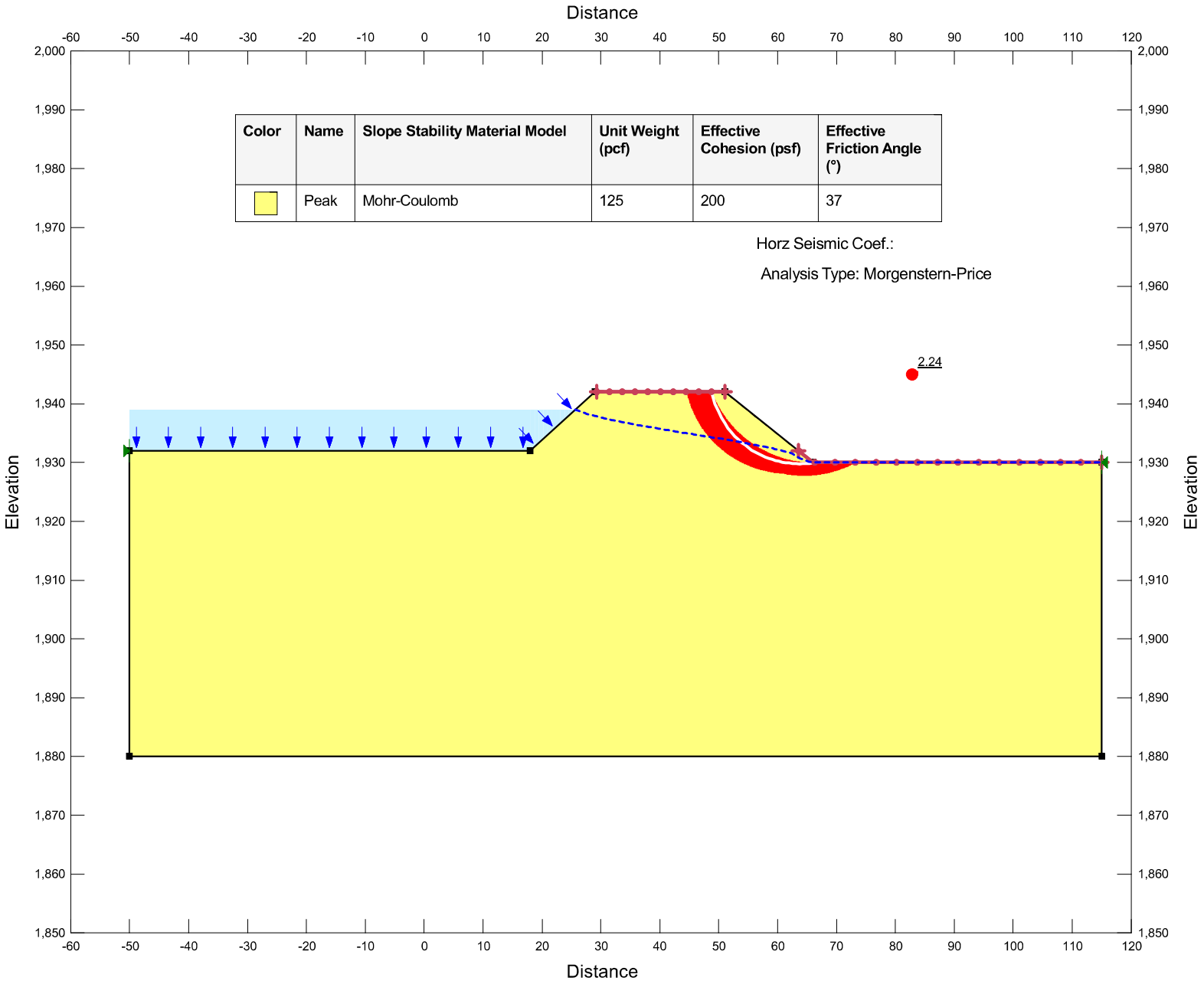
Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Kz'/Kx' Ratio	Dip Direction (°)	Dip (°)
	Peak	Saturated / Unsaturated	Soil	Soil	1	1	90	0

Analysis Type: Steady-State



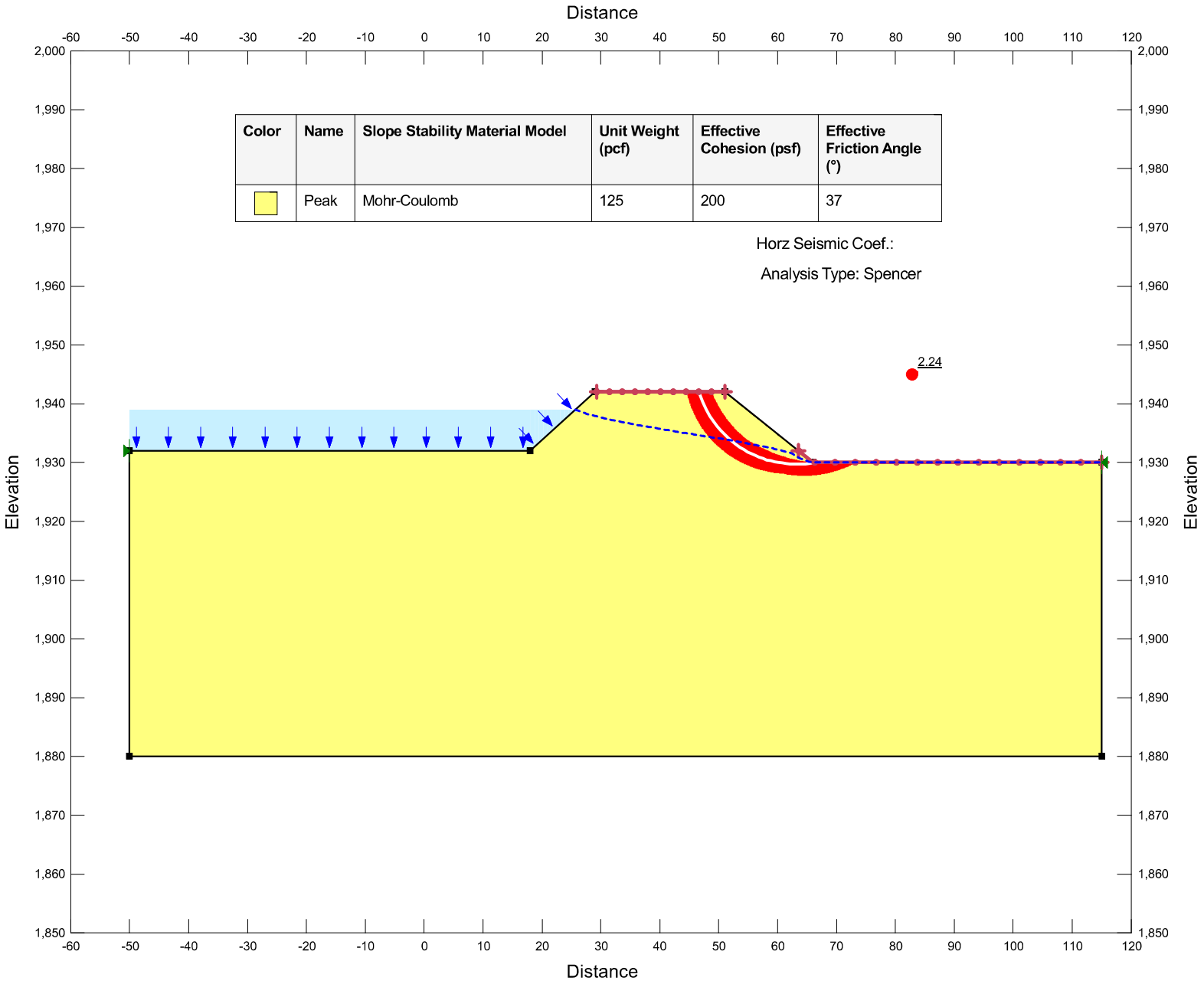
Title: Mill Creek Basin. Name: Seepage
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Created By: Qiqi Huang
Date:04/15/2025

Section C (12)
Static (M-P)



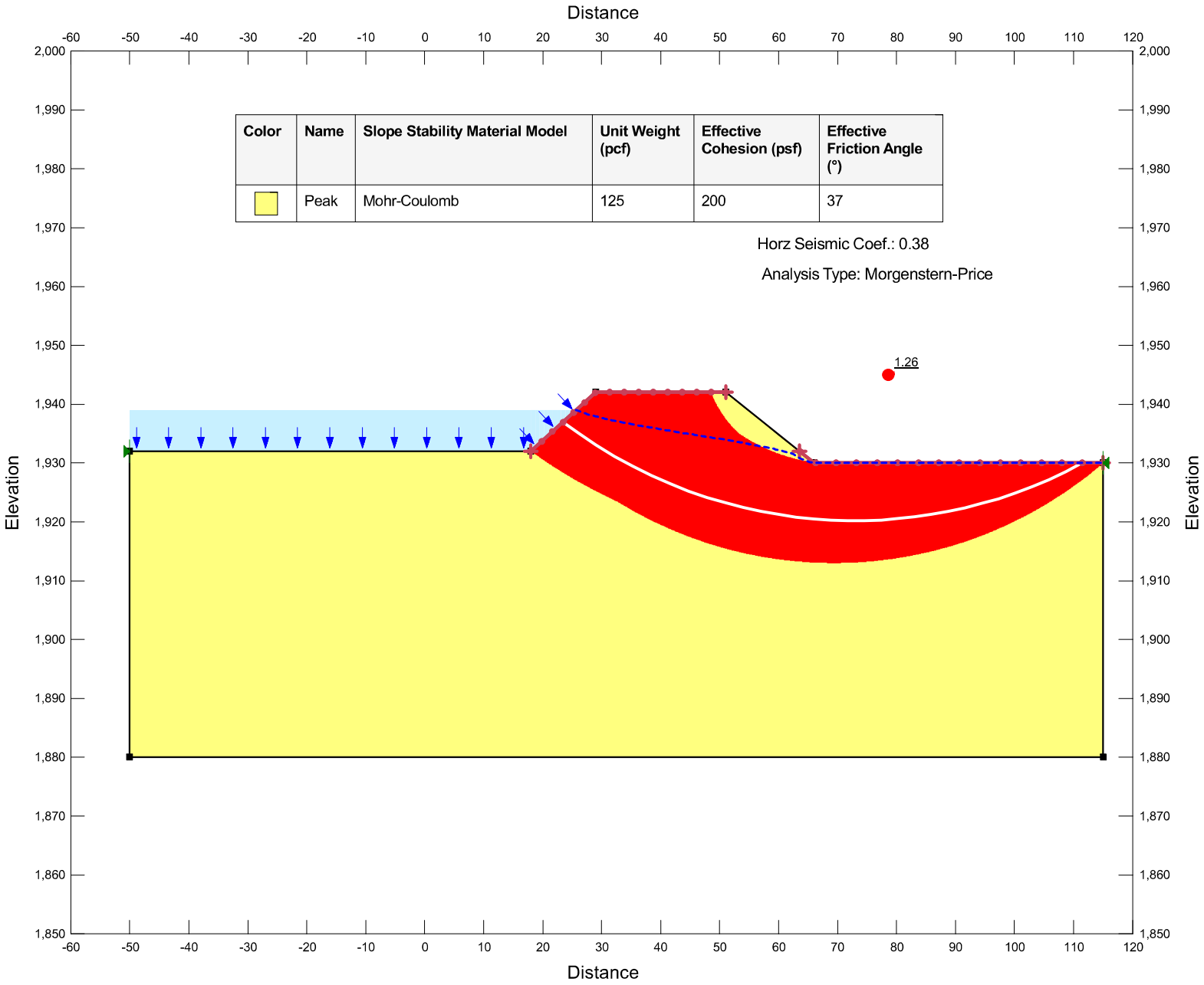
Title: Mill Creek Basin. Name: Static_M-P
File Name: MCB_Sec_12_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section C (12)
Static (Spencer)



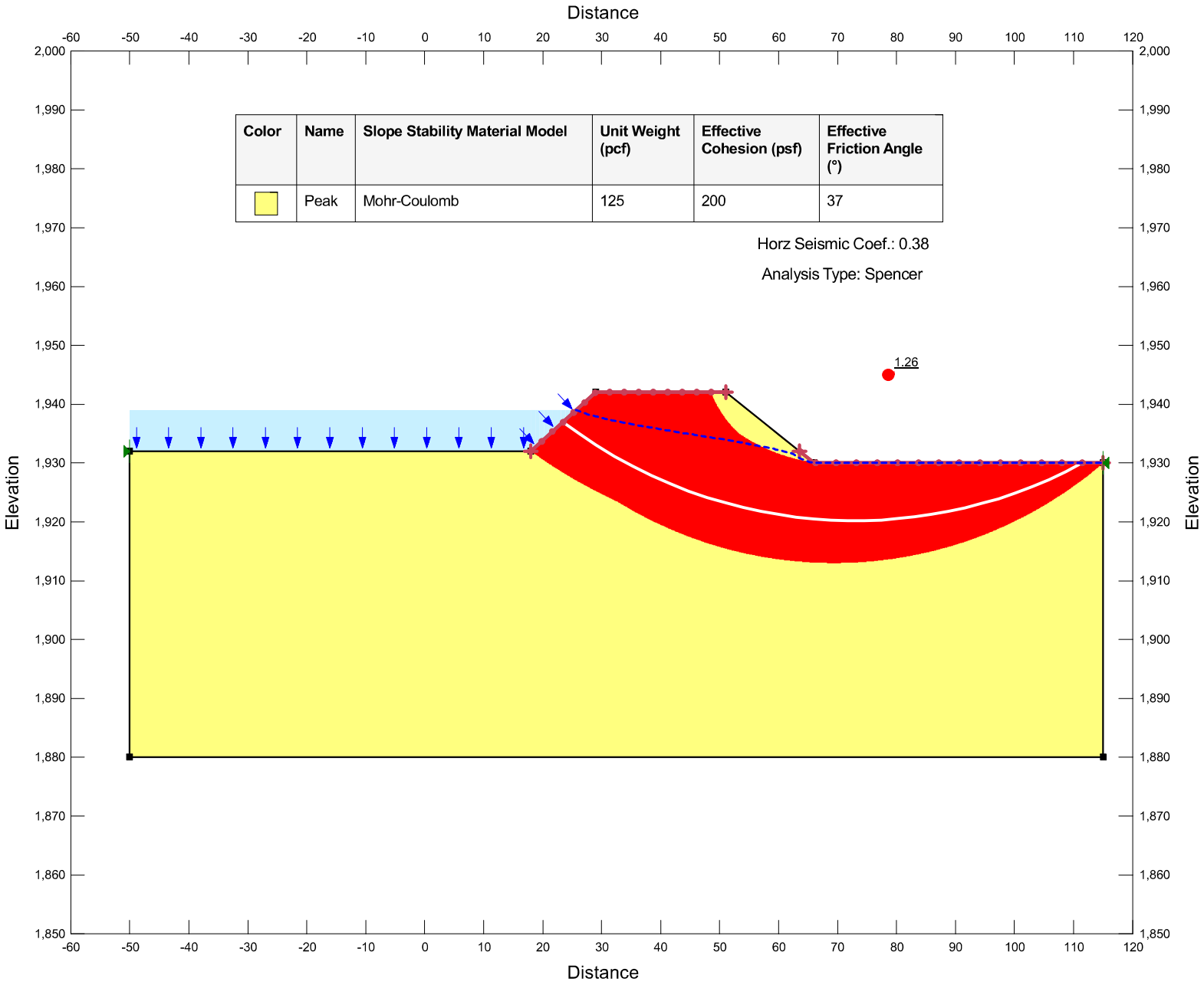
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Created By: Qiqi Huang
Date:04/15/2025

Section C (12)
Seismic W K_{eh} (M-P)



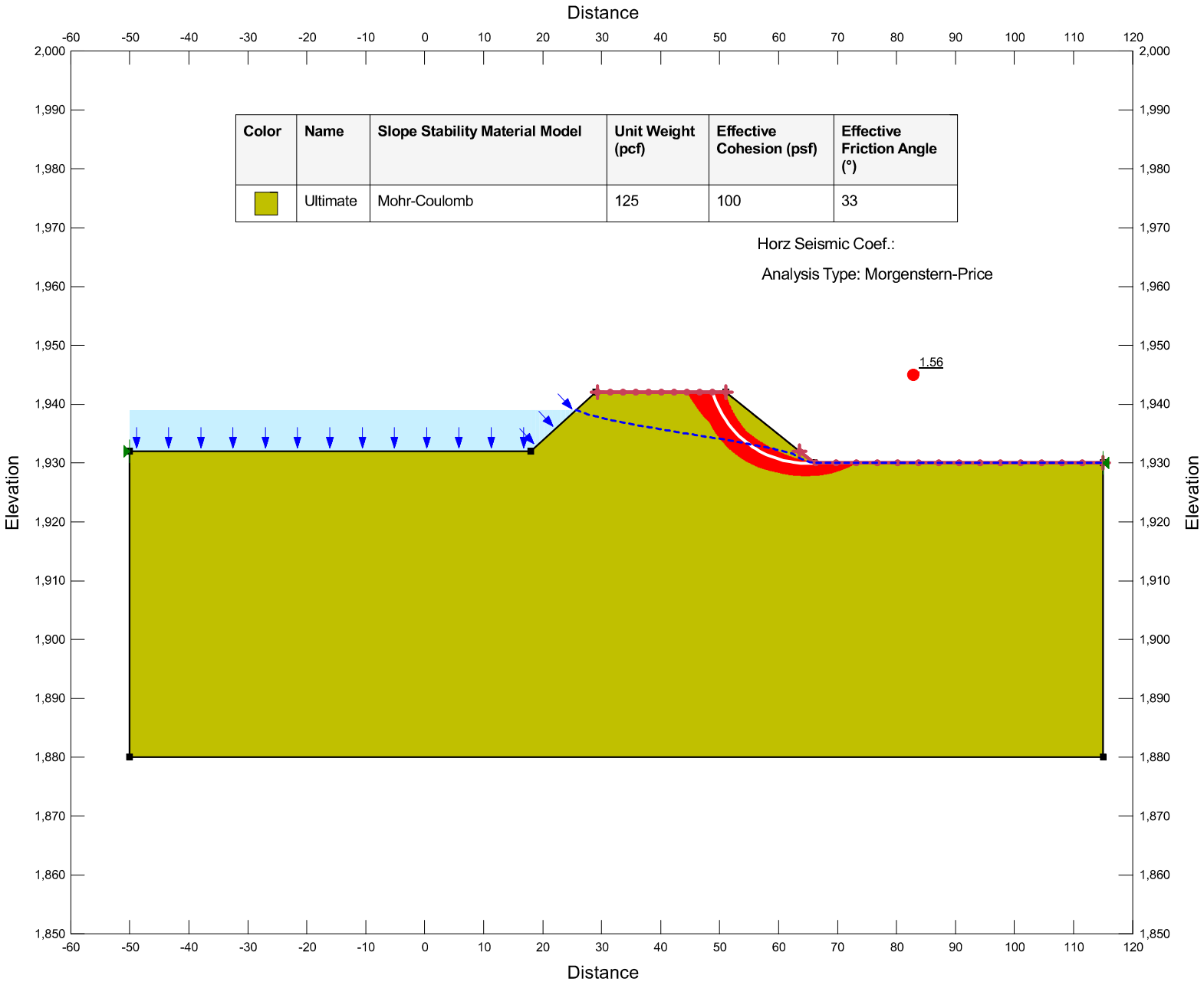
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Created By: Qiqi Huang
Date:04/15/2025

Section C (12)
Seismic W K_{eh} (Spencer)



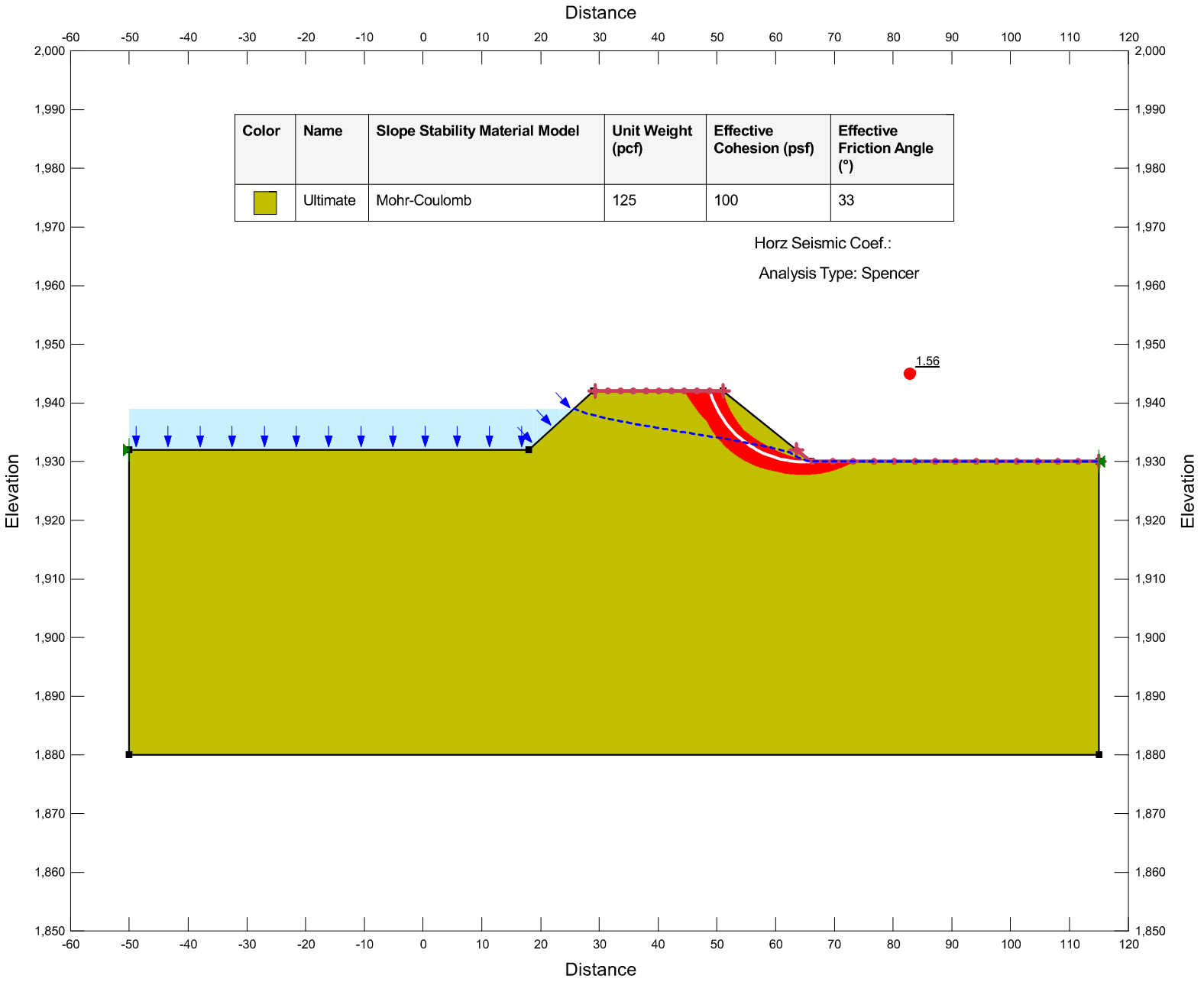
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Section C (12)
Seismic W/O K_{eh} (M-P)



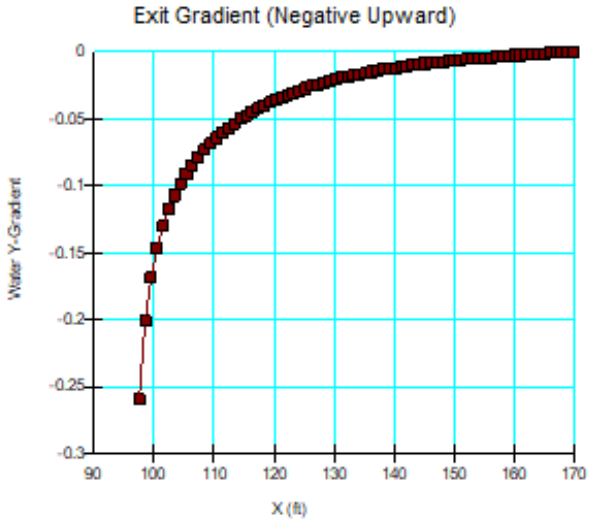
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Date:04/15/2025

Section C (12)
Seismic W/O K_{eh} (Spencer)



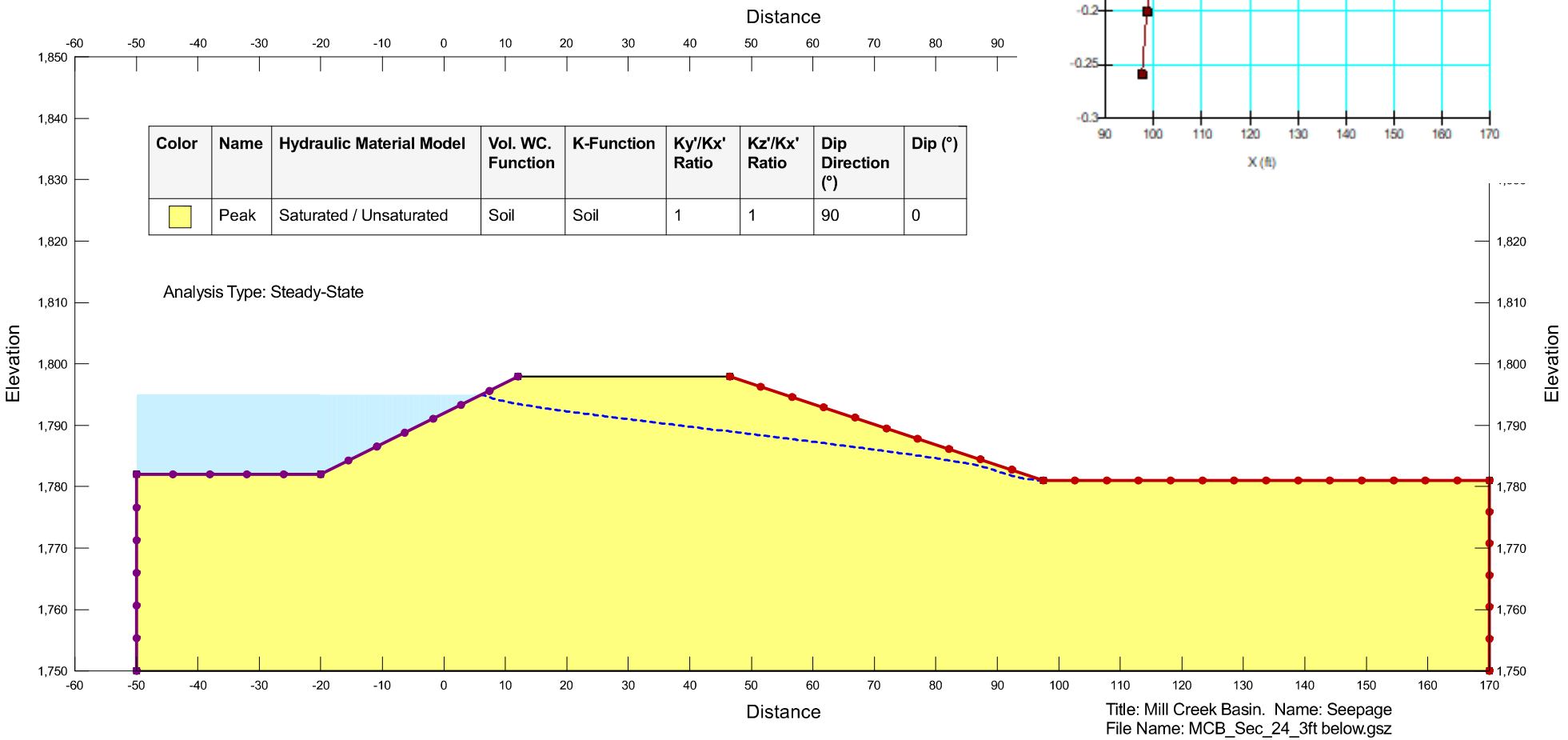
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Date:04/15/2025

Section D (24)
Seepage



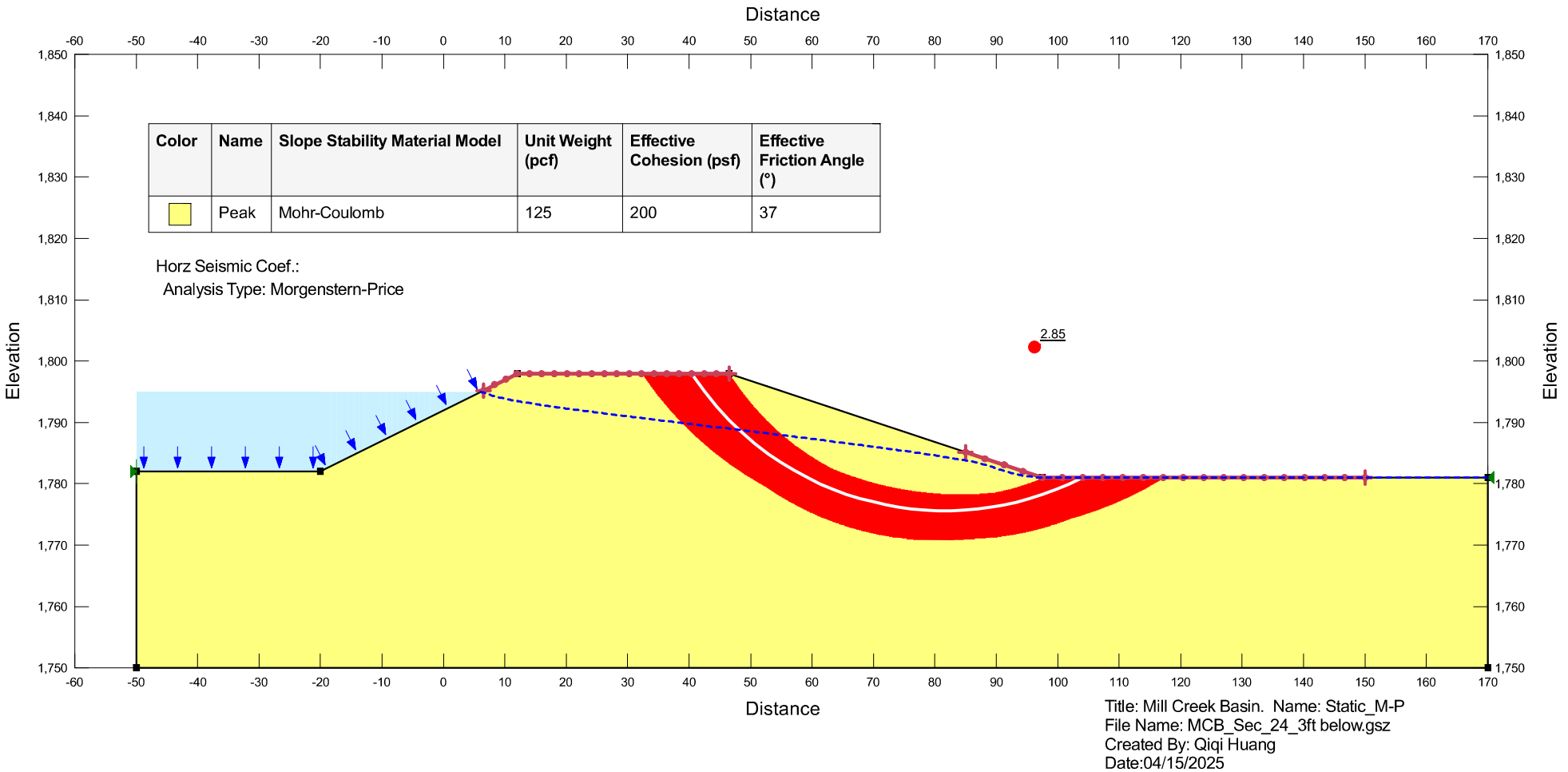
Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Ky'/Kx' Ratio	Kz'/Kx' Ratio	Dip Direction (°)	Dip (°)
	Peak	Saturated / Unsaturated	Soil	Soil	1	1	90	0

Analysis Type: Steady-State

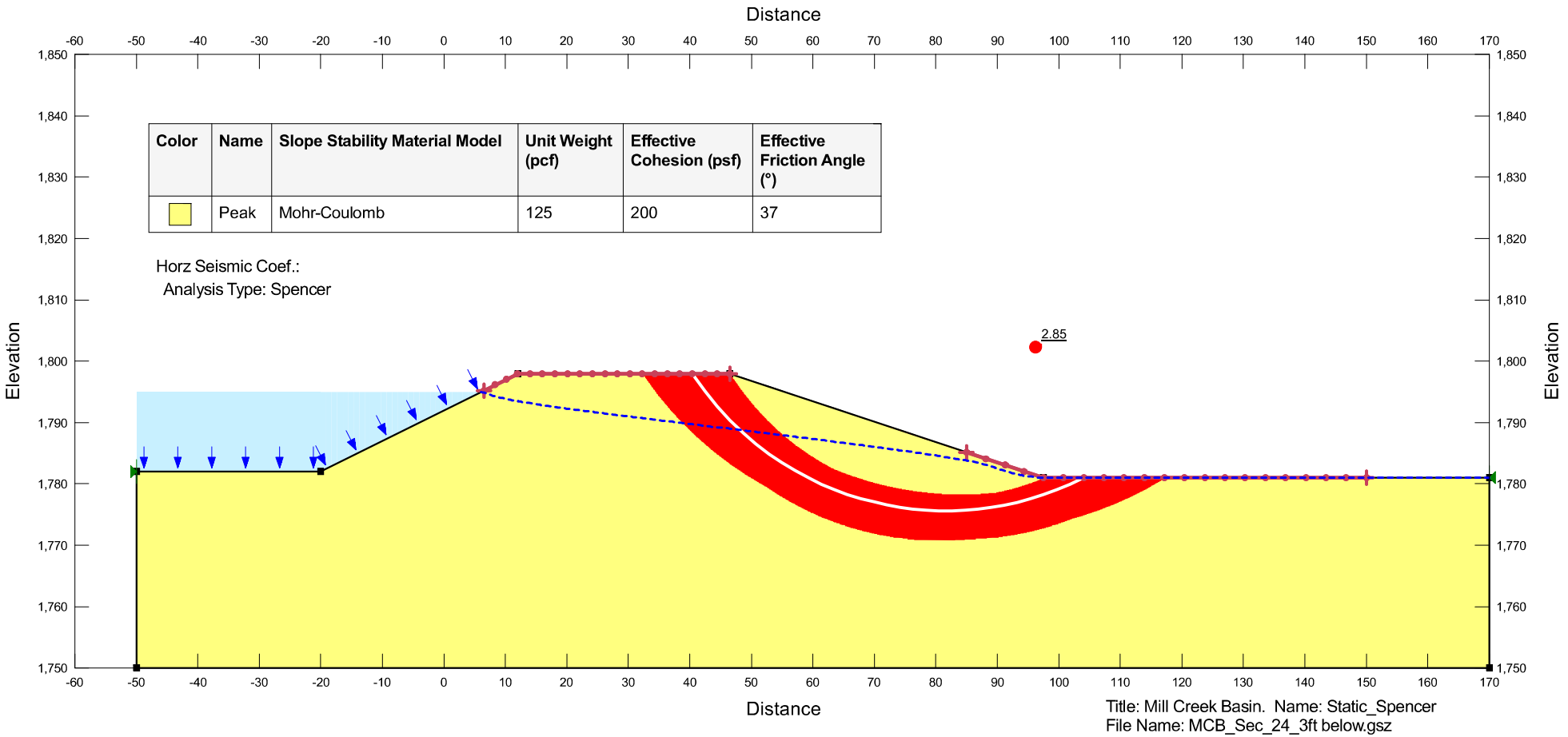


Title: Mill Creek Basin. Name: Seepage
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Created By: Qiqi Huang
Date:04/15/2025

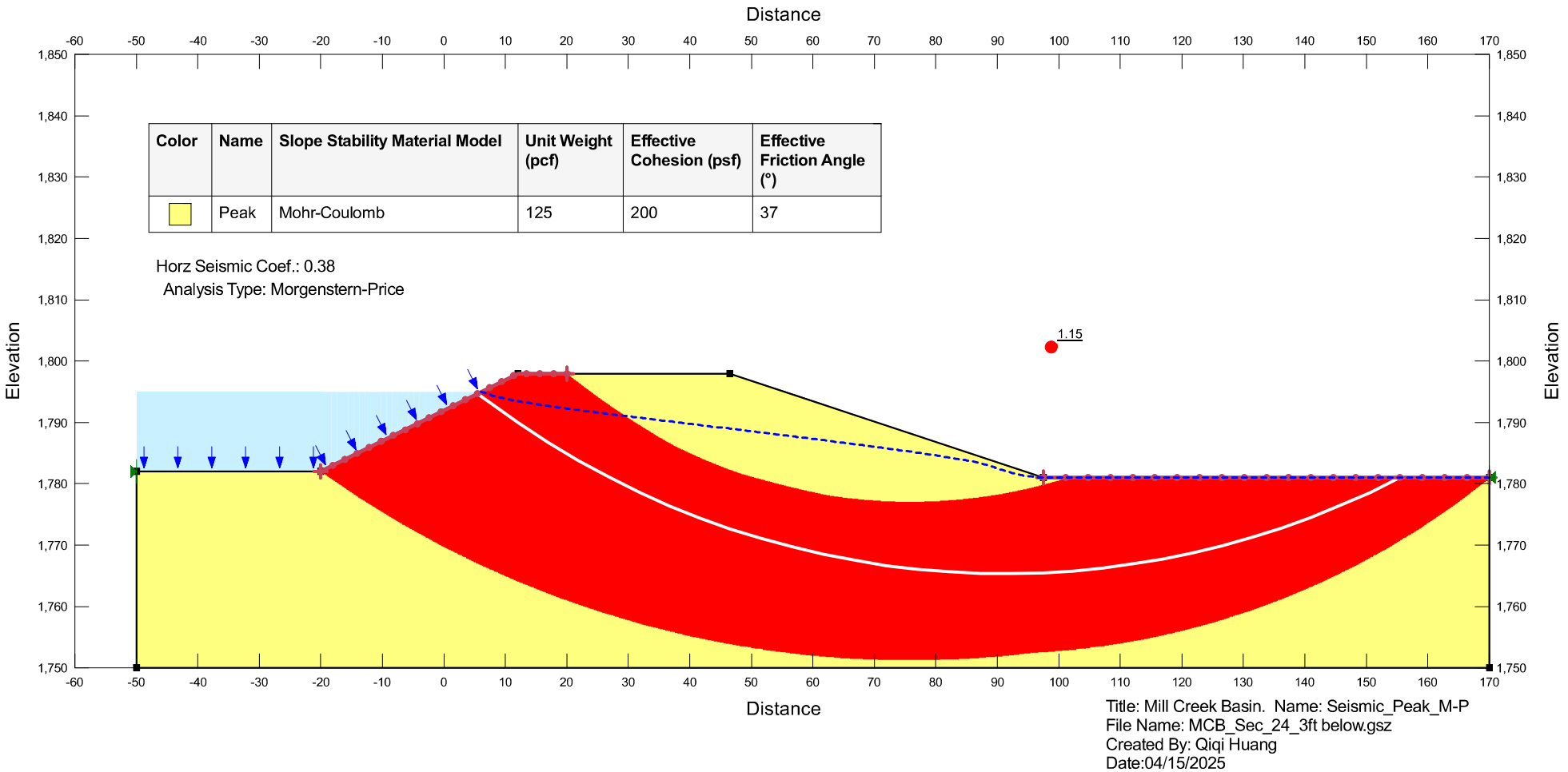
Section D (24)
Static (M-P)



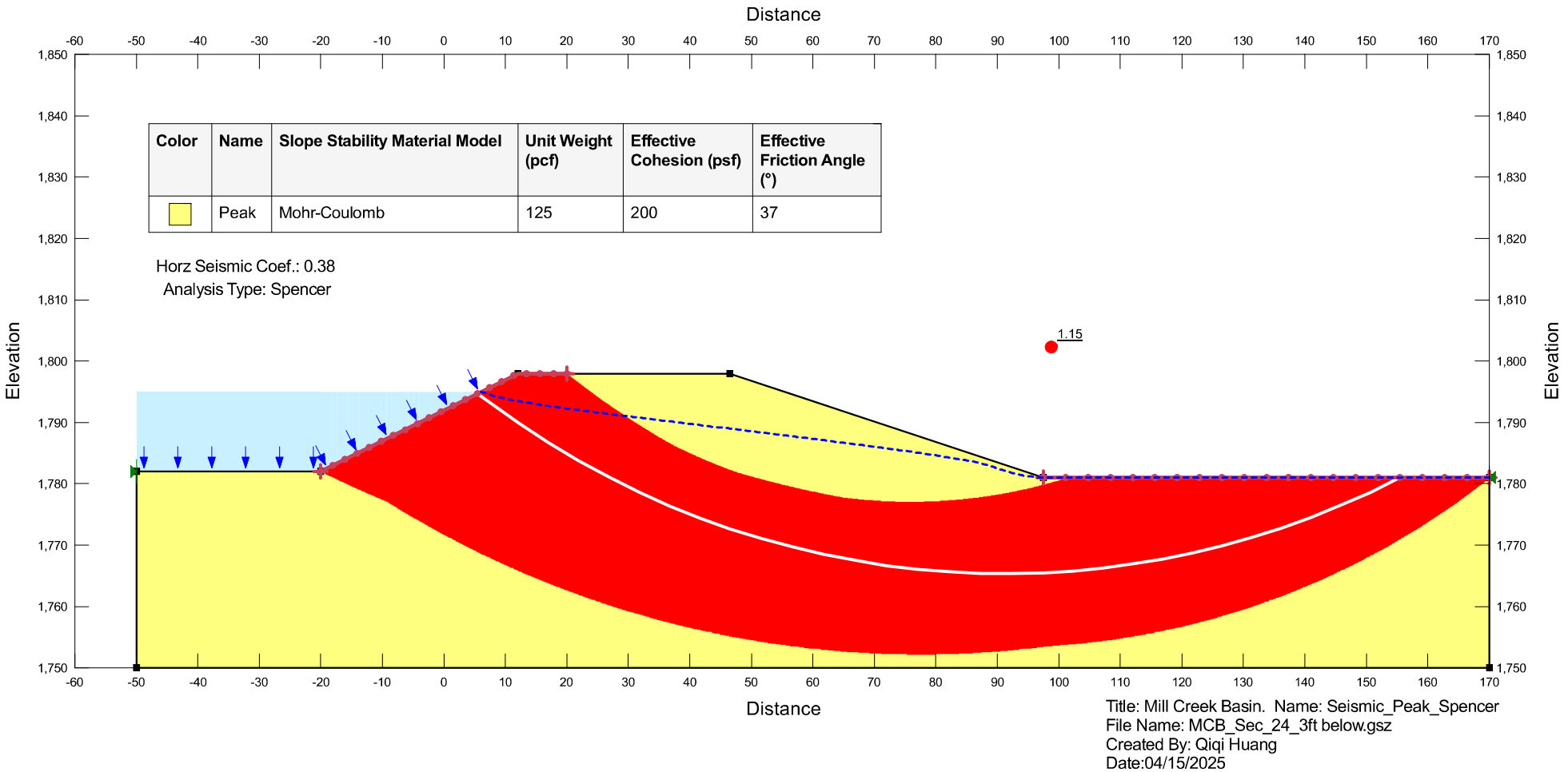
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Static (Spencer)



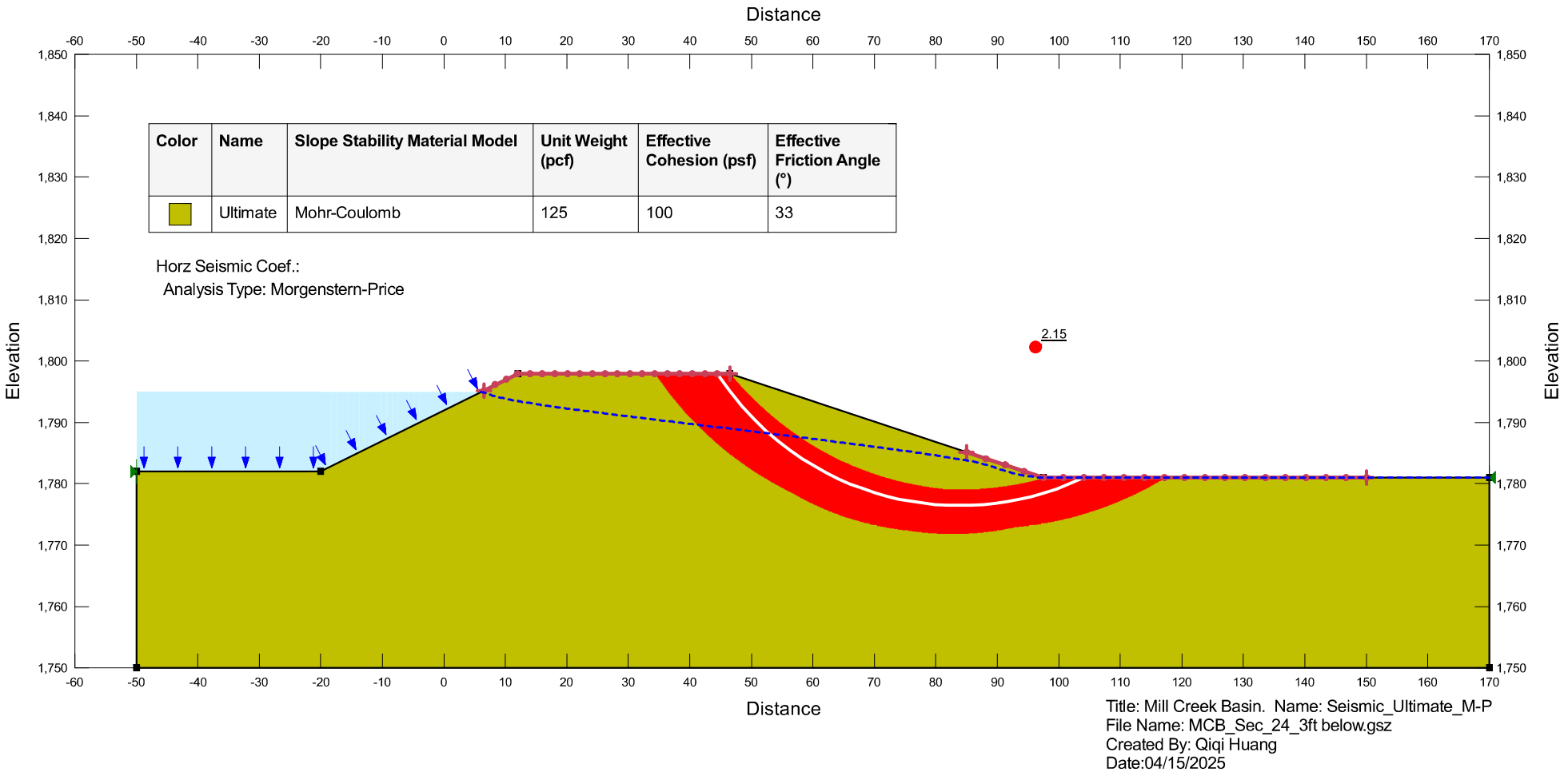
Section D (24)
Seismic W K_{eh} (M-P)



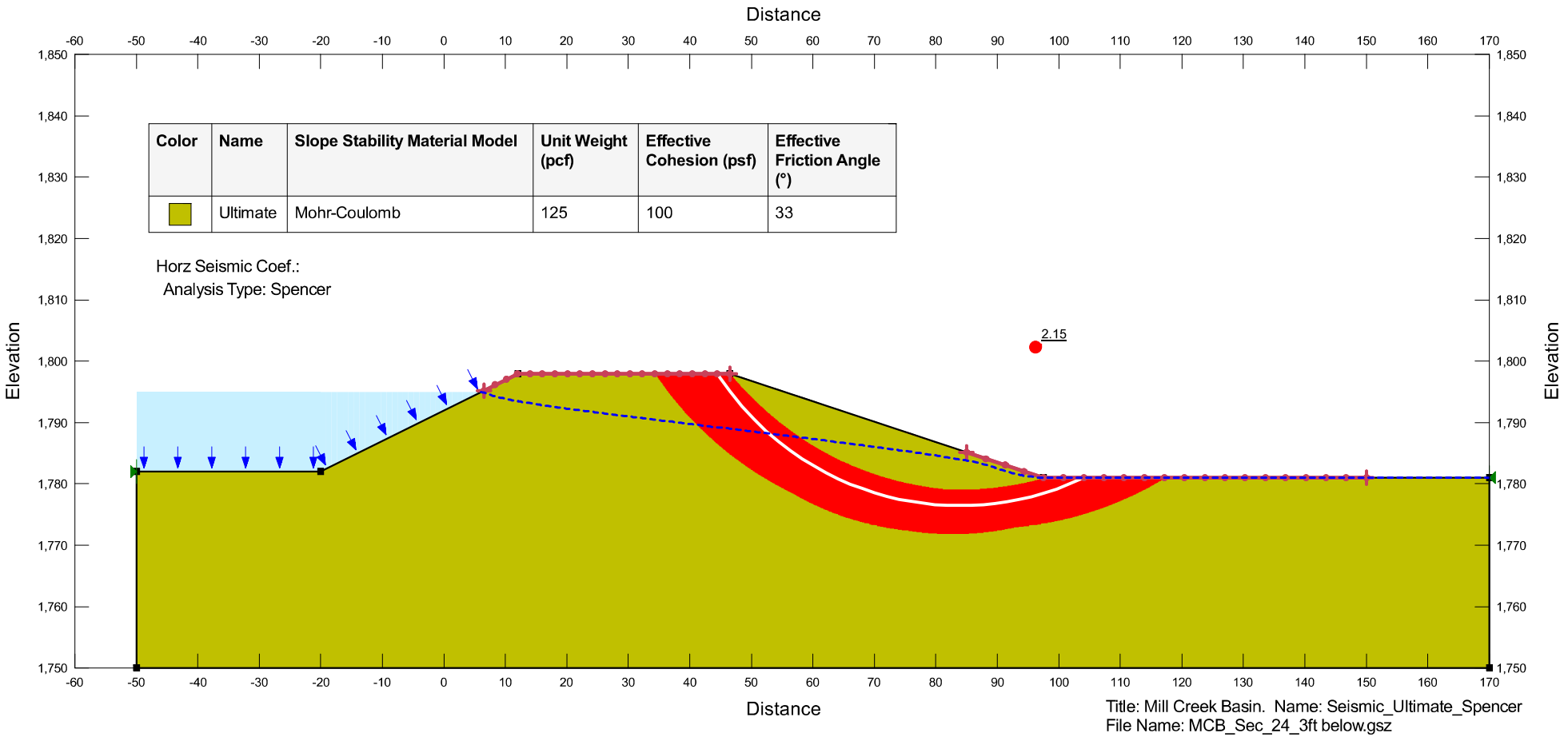
Section D (24)
Seismic W K_{eh} (Spencer)



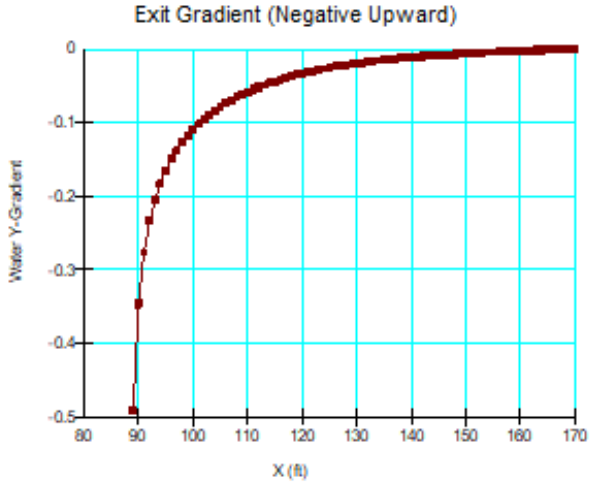
Section D (24)
Seismic W/O K_{eh} (M-P)



Section D (24)
Seismic W/O K_{eh} (Spencer)

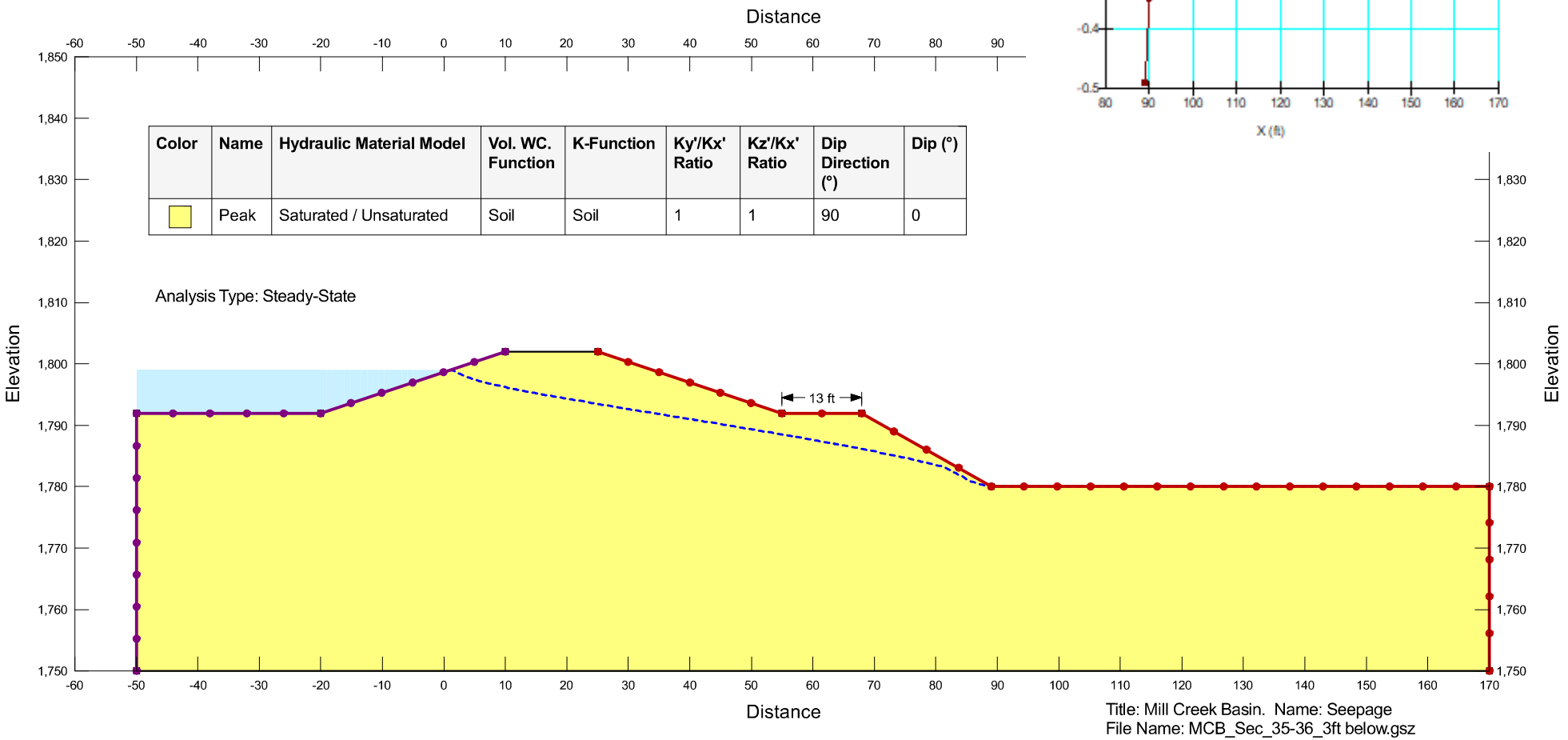


Section E (35-36)
Seepage



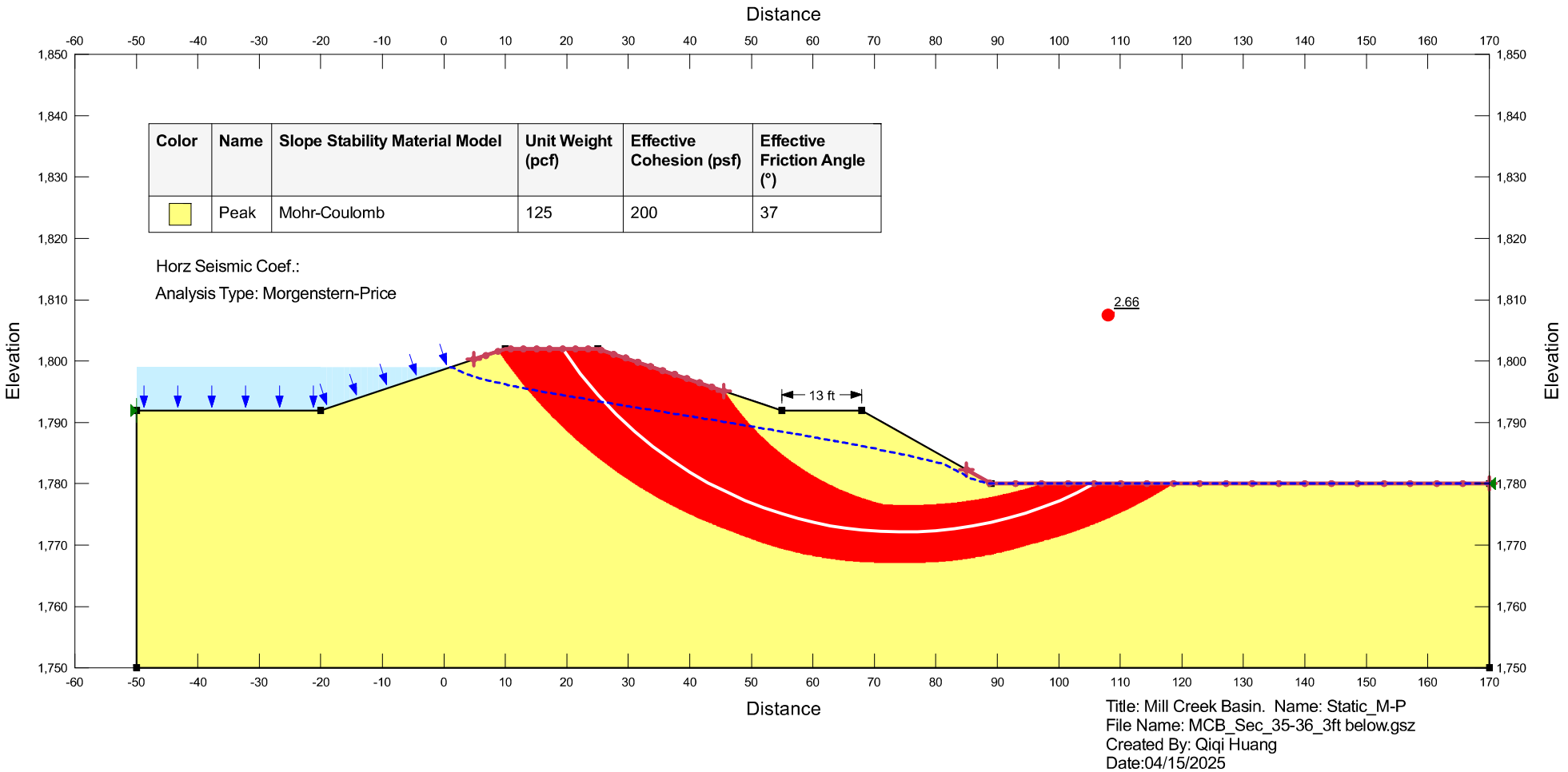
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	Peak	Saturated / Unsaturated	Soil	Soil	1	1	90	0

Analysis Type: Steady-State

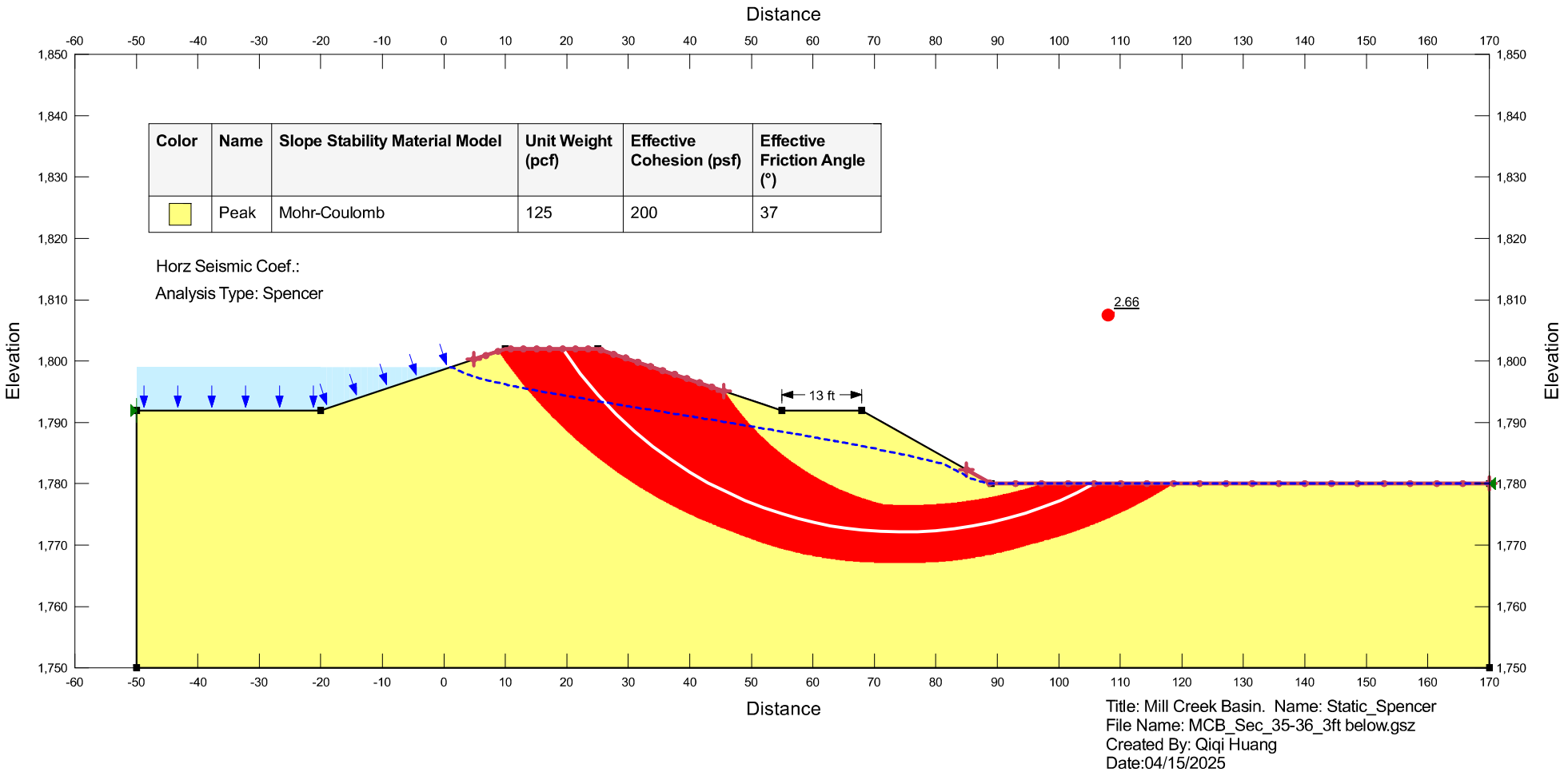


Title: Mill Creek Basin. Name: Seepage
File Name: MCB_Sec_35-36_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

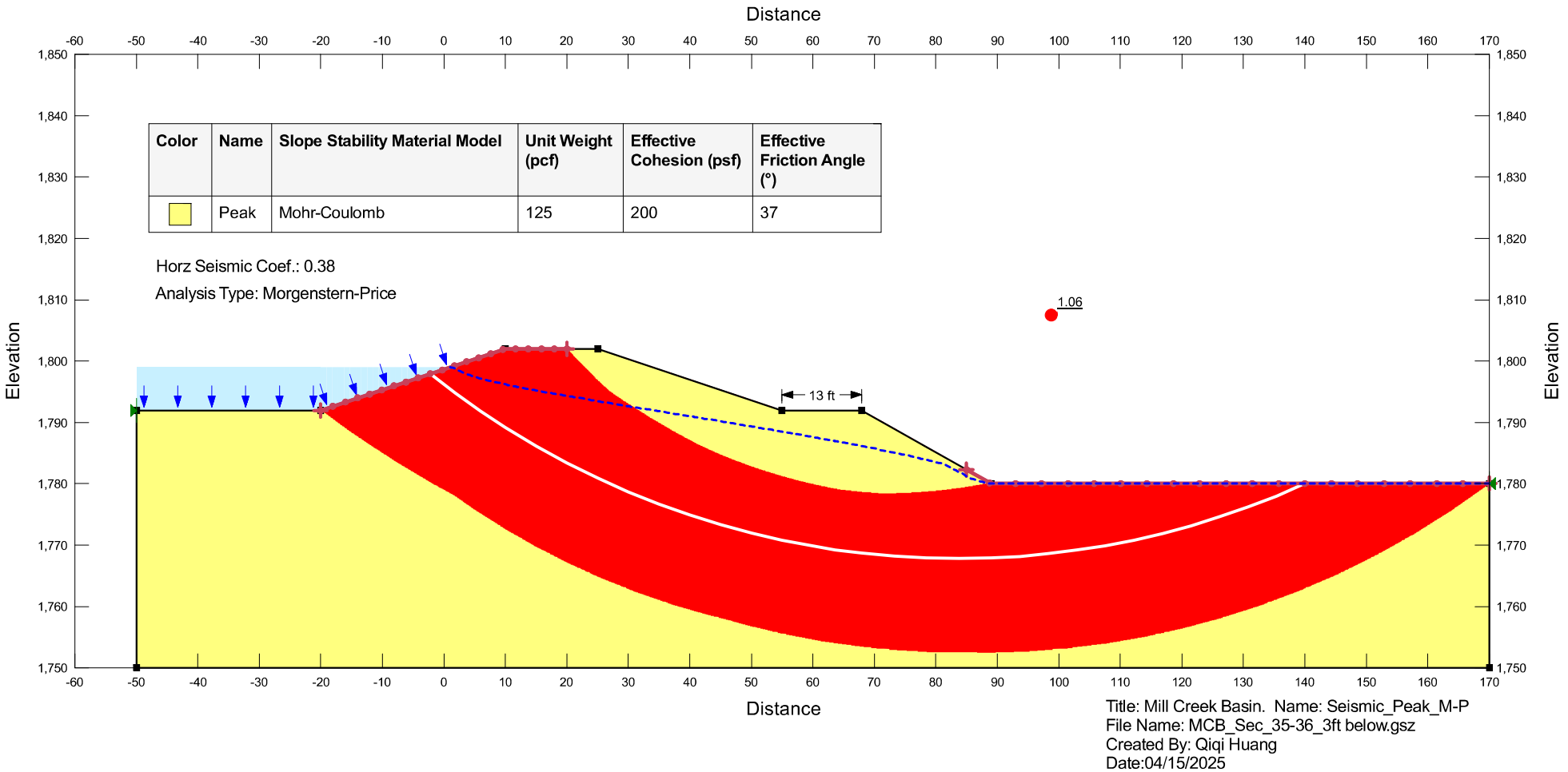
Section E (35-36)
Static (M-P)



Section E (35-36)
Static (Spencer)

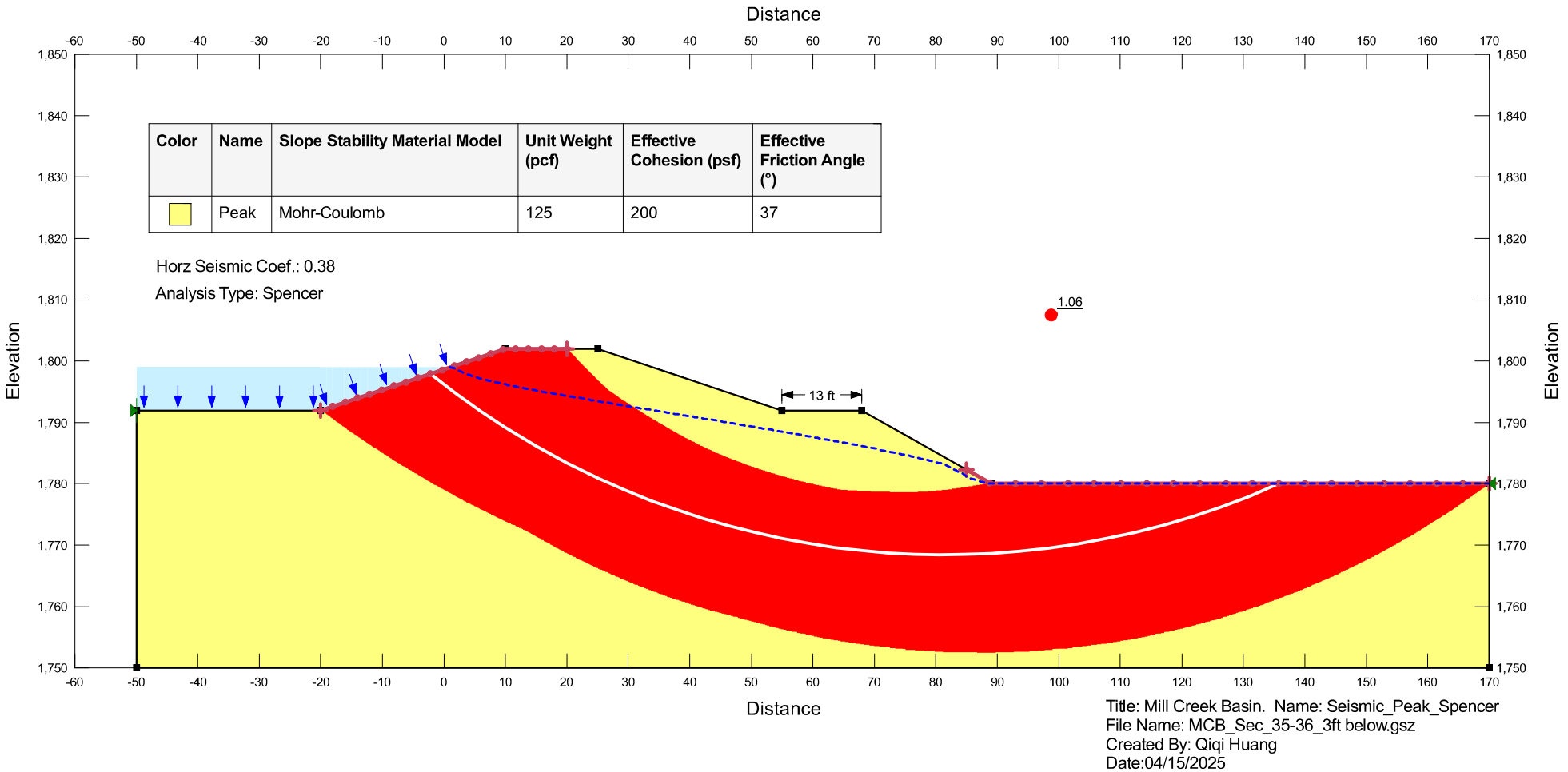


Section E (35-36)
Seismic W K_{eh} (M-P)



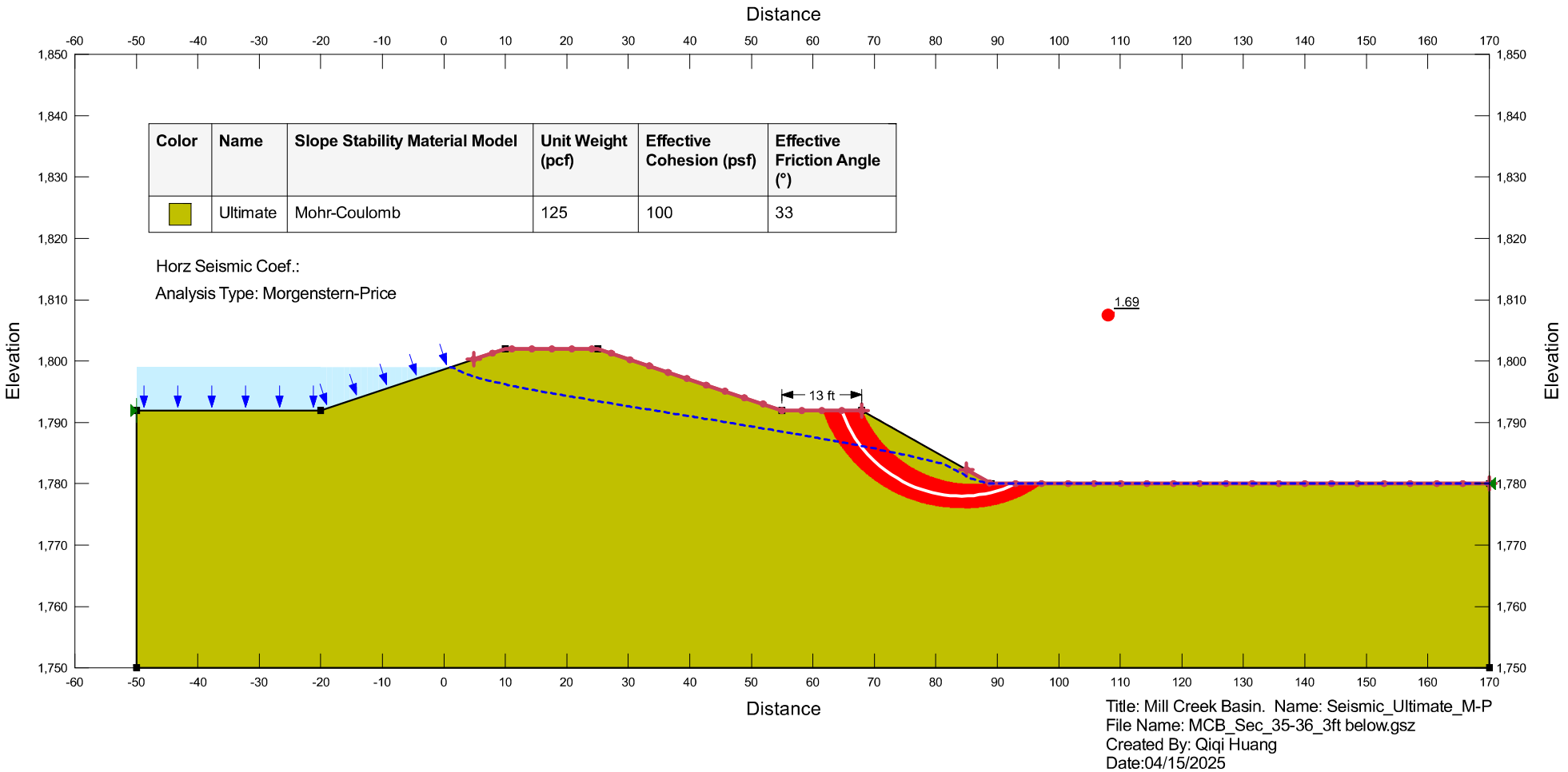
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File Name: MCB_Sec_35-36_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section E (35-36)
Seismic W K_{eh} (Spencer)



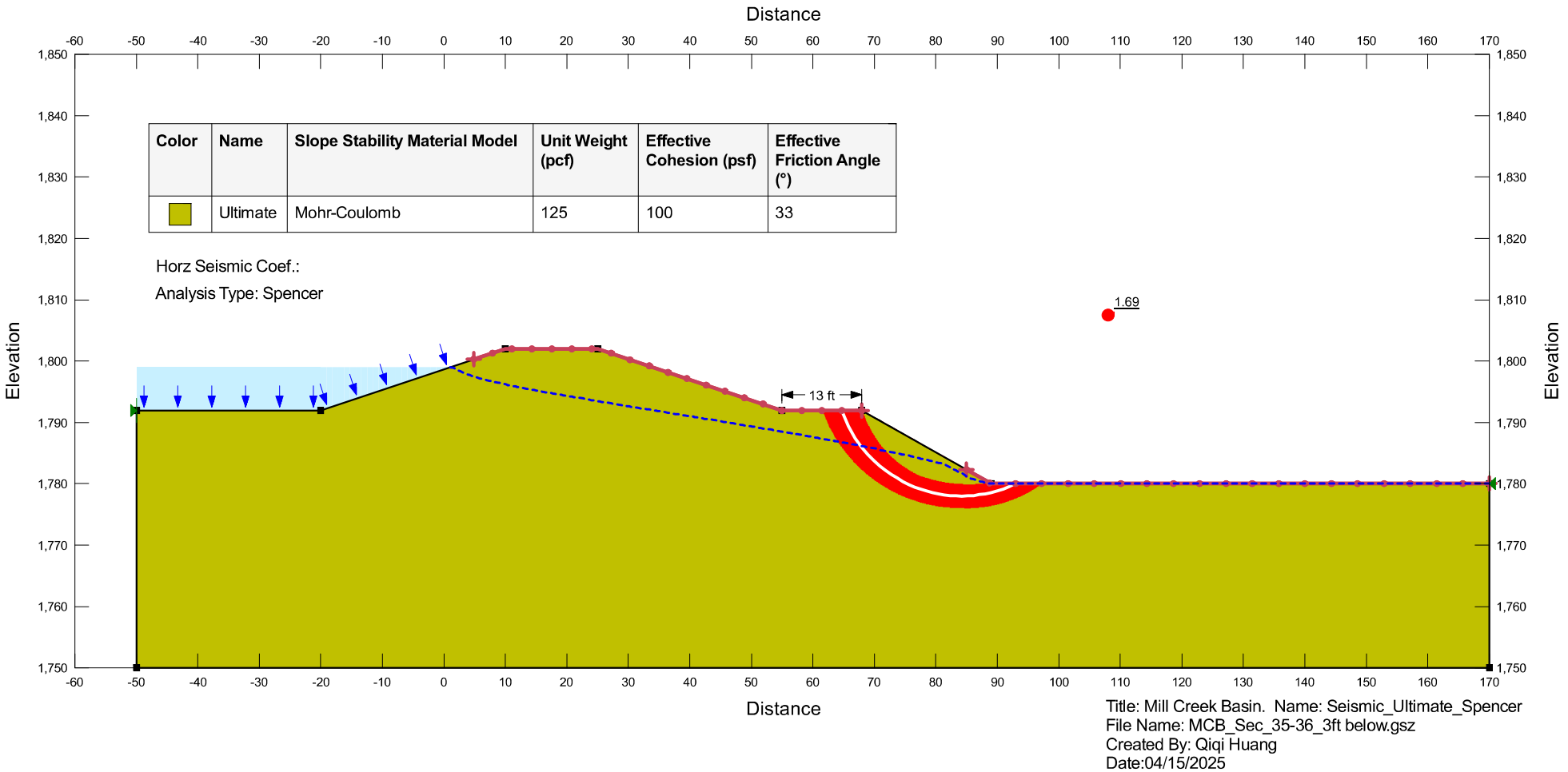
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Created By: Qiqi Huang
Date:04/15/2025

Section E (35-36)
Seismic W/O K_{eh} (M-P)



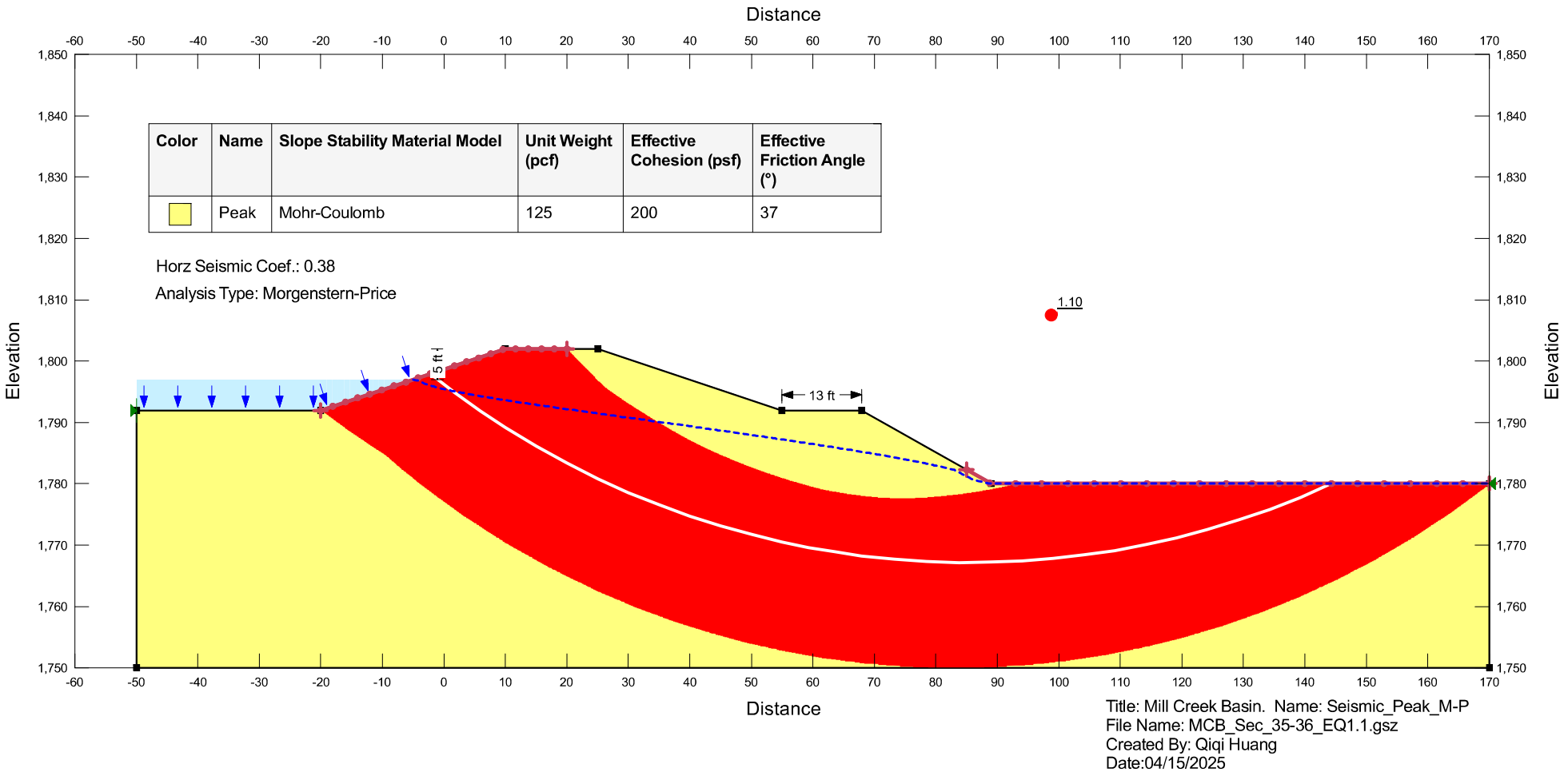
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Created By: Qiqi Huang
Date:04/15/2025

Section E (35-36)
Seismic W/O K_{eh} (Spencer)



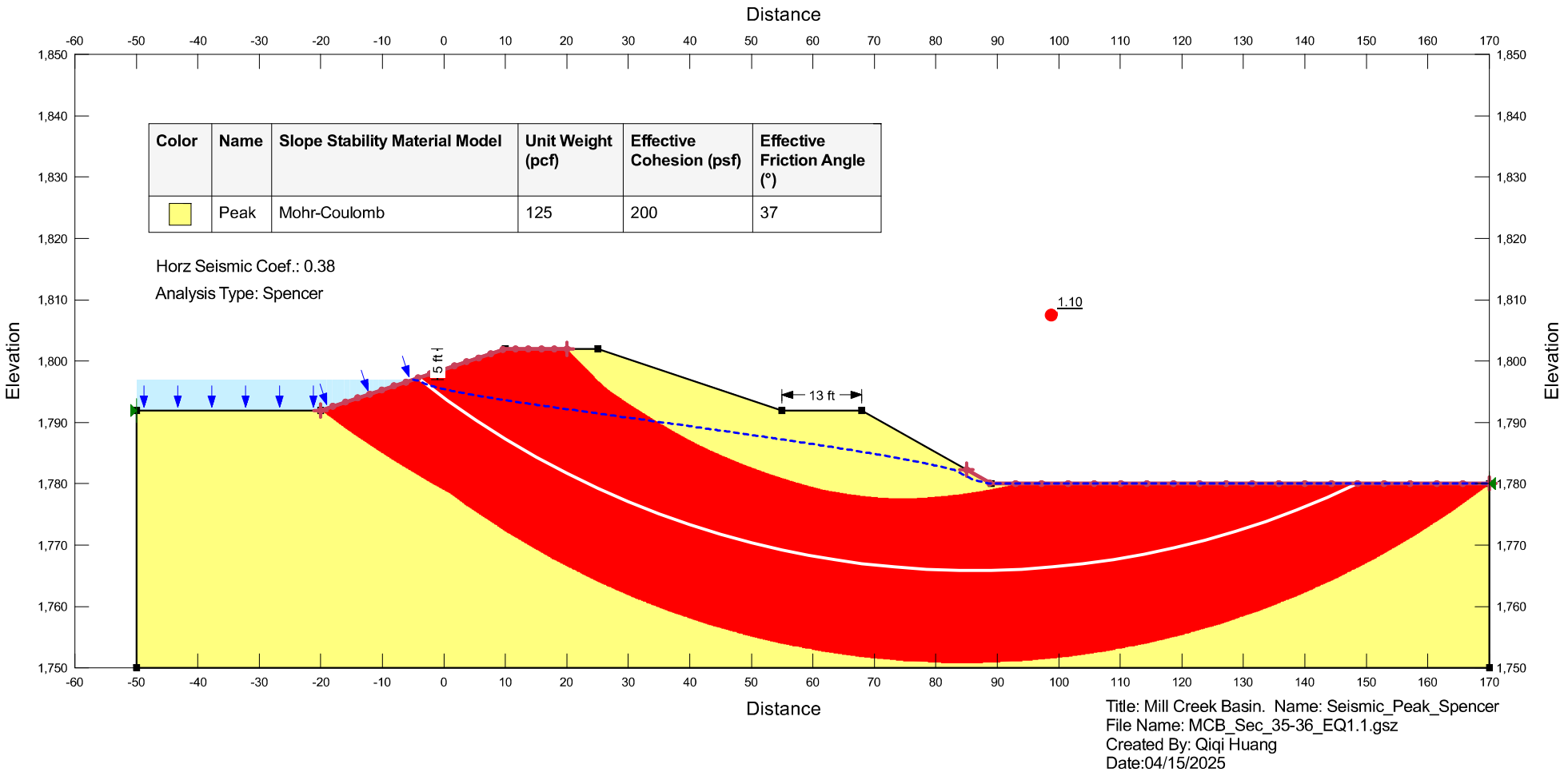
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File Name: MCB_Sec_35-36_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section E (35-36)
Seismic W K_{eh} (M-P) - FS=1.10



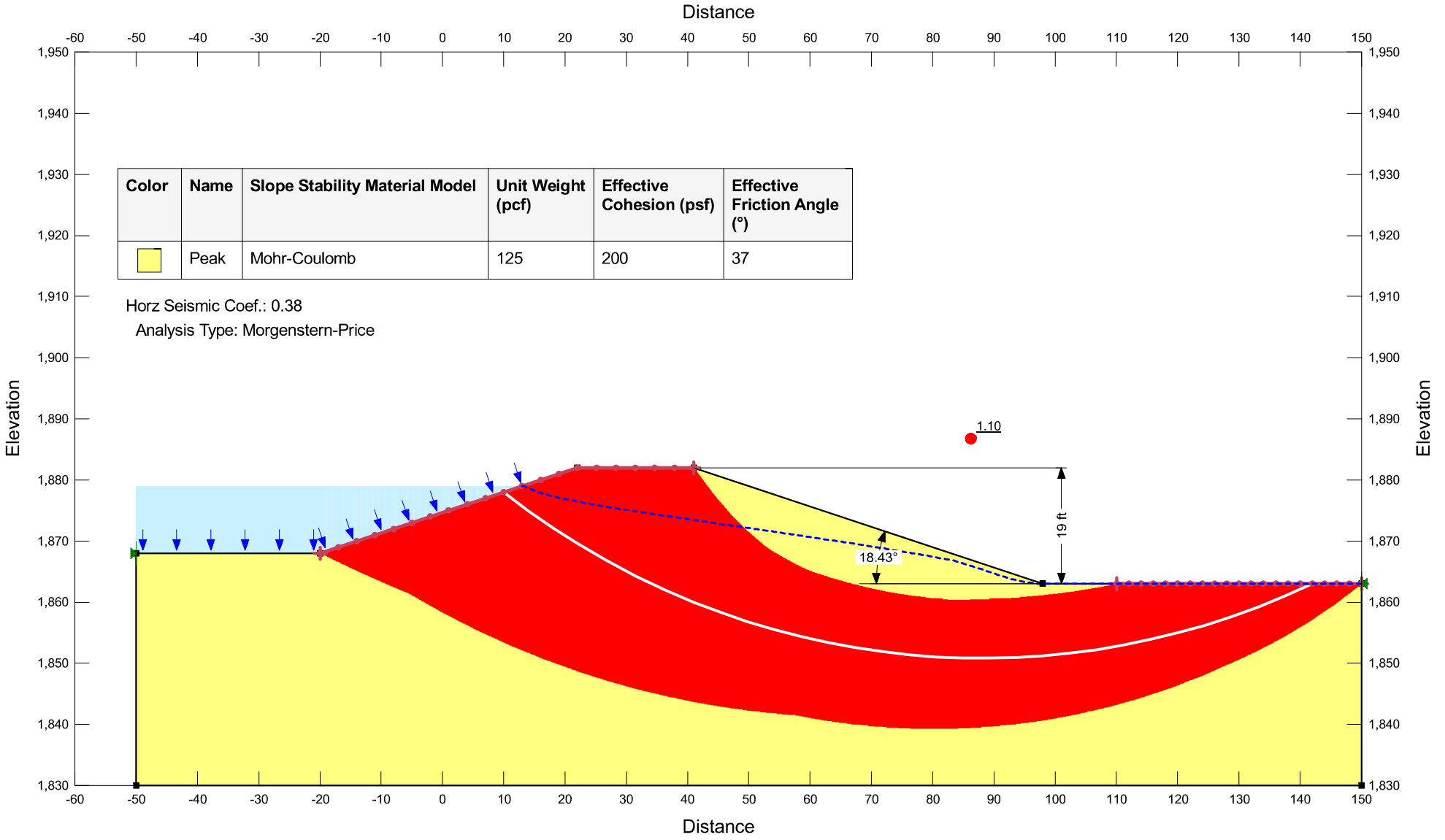
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File Name: MCB_Sec_35-36_EQ1.1.gsz
Created By: Qiqi Huang
Date:04/15/2025

Section E (35-36)
Seismic W K_{eh} (Spencer) - FS=1.10



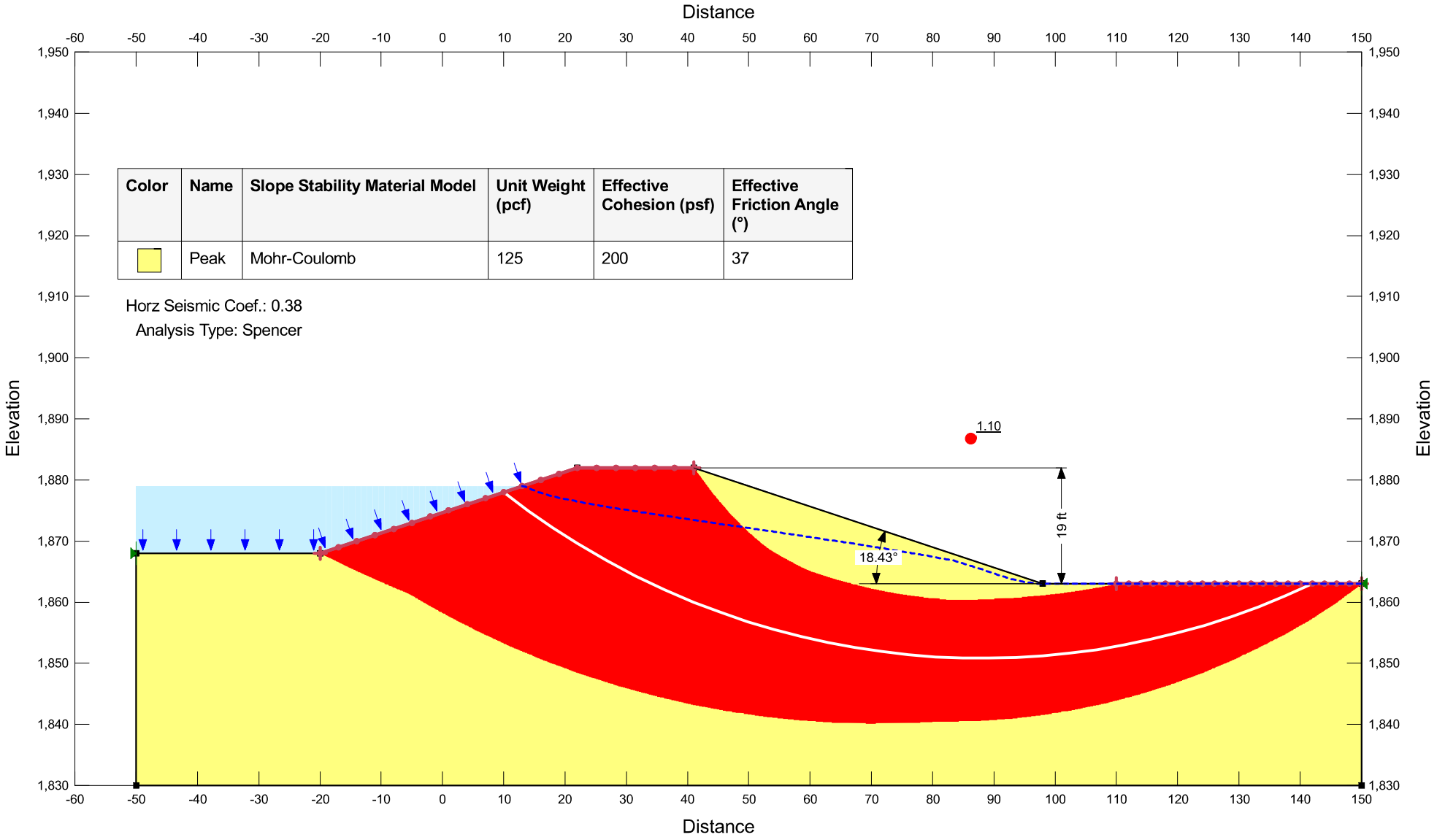
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File Name: MCB_Sec_35-36_EQ1.1.gsz
Created By: Qiqi Huang
Date:04/15/2025

Maximum Water Height at 3H:1V
Seismic W K_{eh} (Spencer) - FS=1.10



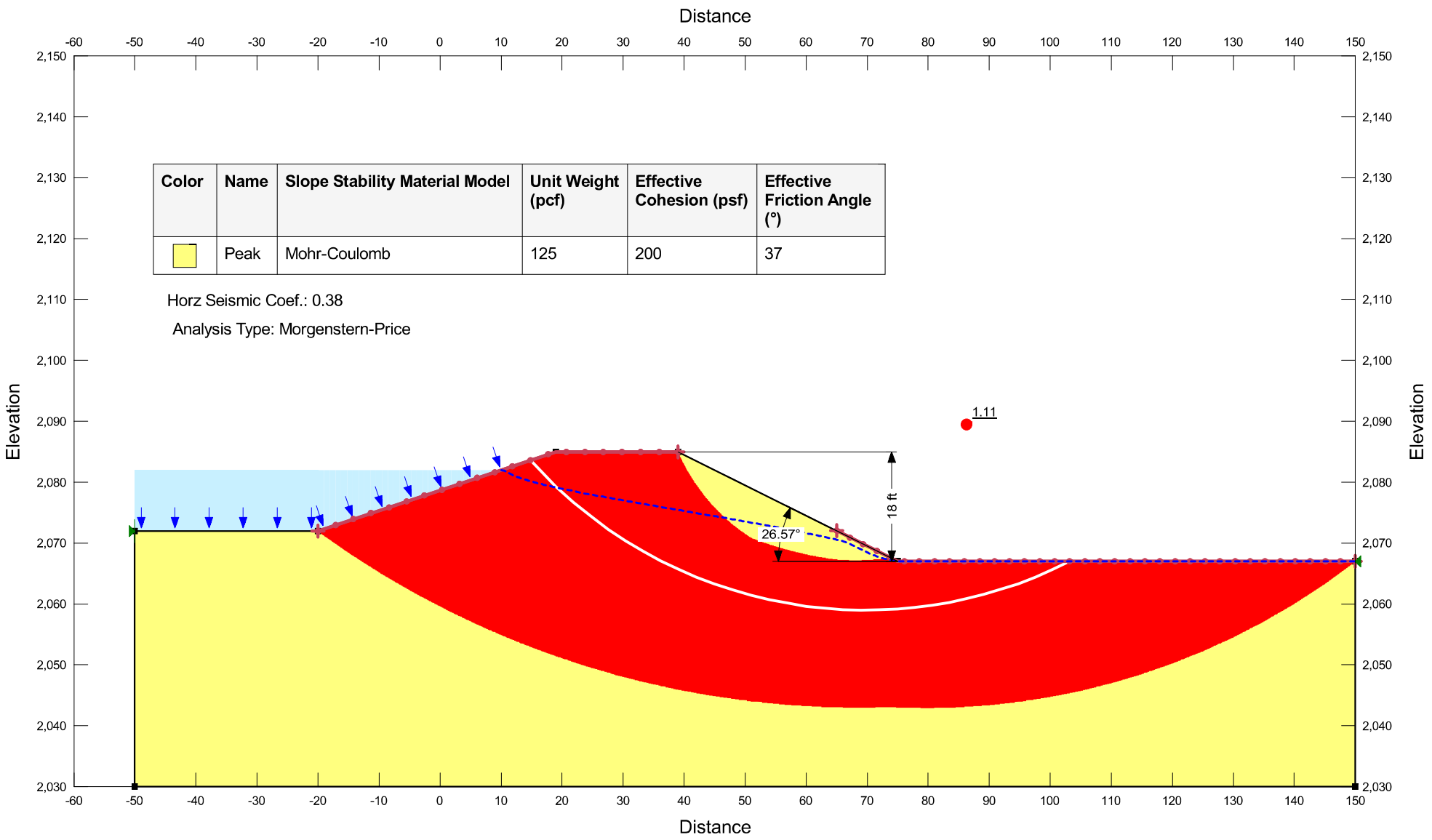
Title: Mill Creek Basin. Name: Seismic_Peak_M-P
File Name: MCB_H19_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Maximum Water Height at 3H:1V
Seismic W K_{eh} (M-P) - FS=1.10



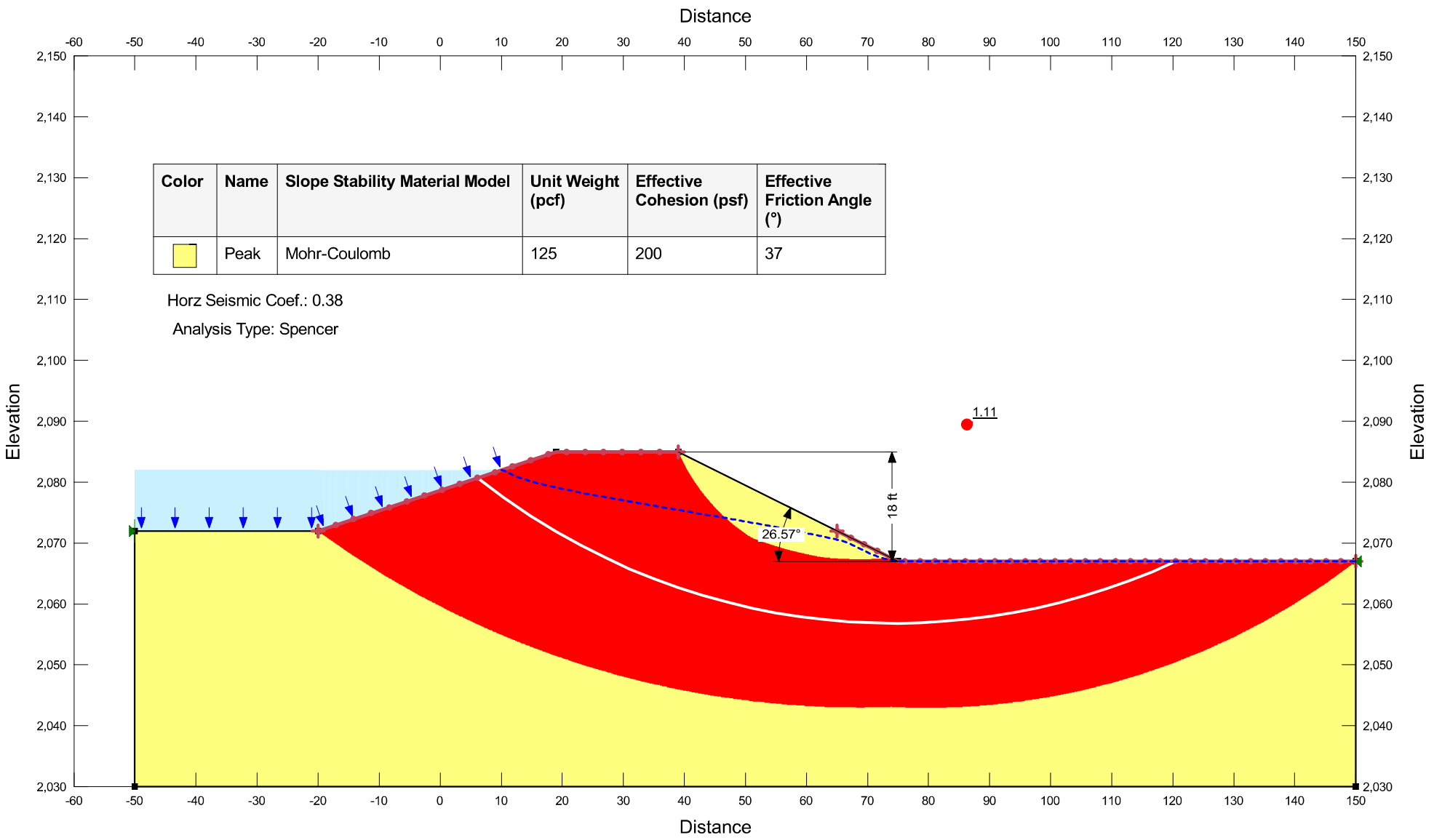
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File Name: MCB_H19_3ft below.gsz
Created By: Qiqi Huang
Date:04/15/2025

Maximum Water Height at 2H:1V
Seismic W K_{eh} (Spencer) - FS=1.10



Title: Mill Creek Basin. Name: Seismic_Peak_M-P
File Name: MCB_Sec_1B_3ft below_EQ 1.1.gsz
Created By: Qiqi Huang
Date:04/15/2025

Maximum Water Height at 2H:1V
Seismic W K_{eh} (M-P) - FS=1.10



Title: Mill Creek Basin. Name: Seismic_Peak_Spencer
File Name: MCB_Sec_1B_3ft below_EQ 1.1.gsz
Created By: Qiqi Huang
Date:04/15/2025

APPENDIX C.3

Bearing Capacity and Lateral Earth Pressure Calculations

DRAFT

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ANALYSIS OF SOIL BEARING CAPACITY OF FOOTINGS

(EZ_FTGCAP v2.0, 2006, Modified By: Dr. EZ)

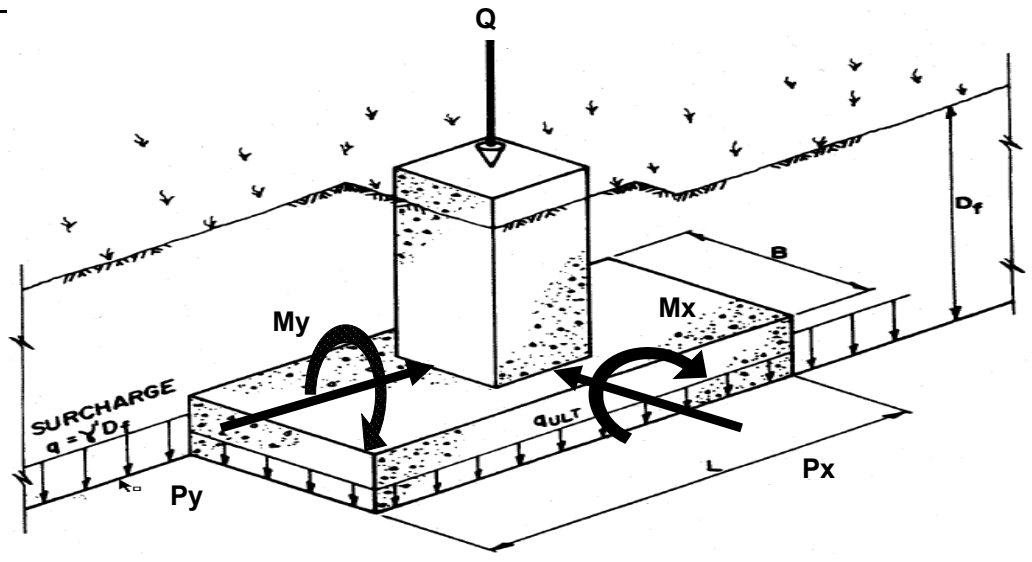
PROJECT INFORMATION		
Project Name		
Project No.		
Location		

FOOTING GEOMETRY		
Footing Location		
Footing Description	L/B=1	
Footing Width, B	3.00	ft
Footing Length, L	3.00	ft
Min. Footing Embedment Depth, D_f	1.50	ft
Bottom of Footing Elevation	0.00	ft
Design GWT Elevation	0.00	ft
Ground Slope Angle, β	0.00	degrees
Footing Base Angle, α	0.00	degrees

SOIL PARAMETERS		
Total Soil Unit Weight, γ_t	125.00	pcf
Saturated Soil Unit Weight, γ_{sat}	125.00	pcf
Soil Cohesion, c	1.00	psf
Soil Friction Angle, ϕ	37.00	degrees

FOOTING LOADS		
Design Vertical Loads, Q		kips
Shear Load (along X-axis), P_x		kips
Shear Load (along Y-axis), P_y		kips
Moment (about X-axis), M_x		kip-ft
Moment (about Y-axis), M_y		kip-ft

FACTORS OF SAFETY		
Factor of Safety for Sliding Falure, $FS_{sliding}$	1.50	
Factor of Safety for Bearing Failure, $FS_{bearing}$	3.00	
Bearing Capacity Failure Mode	General Shear	



ANALYSIS OF SOIL BEARING CAPACITY OF FOOTINGS

(EZ_FTGCAP v2.0, 2006, Modified By: Dr. EZ)

Performed By: _____ Date: _____

Reviewed By: _____ Date: _____

PROJECT INFORMATION

Project Name	
Project No.	
Location	

FOOTING GEOMETRY

Footing Location	
Footing Description	L/B=1
Footing Width, B	3.00 ft
Footing Length, L	3.00 ft
Min. Footing Embedment Depth, D_f	1.50 ft
Bottom of Footing Elevation	0.00 ft
Design GWT Elevation	0.00 ft
Ground Slope Angle, β	0.00 degrees
Footing Base Angle, α	0.00 degrees

INPUT SOIL PARAMETERS

Total Soil Unit Weight, γ_t	125.00 pcf
Saturated Soil Unit Weight, γ_{sat}	125.00 pcf
Soil Cohesion, c	1.00 psf
Soil Friction Angle, ϕ	37.00 degrees

FOOTING LOADS

Design Vertical Loads, Q	0.00 kips
Shear Load (along X-axis), P_x	0.00 kips
Shear Load (along Y-axis), P_y	0.00 kips
Moment (about X-axis), M_x	0.00 kip-ft
Moment (about Y-axis), M_y	0.00 kip-ft

FACTORS OF SAFETY

Factor of Safety for Sliding Falure, $FS_{sliding}$	1.50
Factor of Safety for Bearing Falure, $FS_{bearing}$	3.00
Bearing Capacity Failure Mode	General Shear

COMPUTED PARAMETERS

Min. Ground Surface Elevation	1.50 ft
Depth to Design GWT, Z_w	1.50 ft
Load Eccentricity (parallel to X-axis), $e_x = M_y/Q$	0.00 ft
Load Eccentricity (parallel to Y-axis), $e_y = M_x/Q$	0.00 ft
Resultant Shear Load, $P = (P_x^2 + P_y^2)^{1/2}$	0.00 kips
Effective Footing Width, $B' = B - 2\ e_x$	3.00 ft
Effective Footing Length, $L' = L - 2\ e_y$	3.00 ft
Effective Footing Area, $A_f = B'\ L'$	9.00 ft^2
Applied Gross Bearing Pressure, $q_{app} = Q/(BL)$	0.00 ksf
Effective Cohesion, c_{eff}	1.00 psf
Effective Friction Angle, ϕ_{eff}	37.00 degrees
Footing Base Adhesion, $C_a = 0.55\ c_{eff}$	0.55 psf
Depth of Wedge Zone, $D = 0.5B \tan(45°+\phi_{eff}/2)$	3.01 ft
Effective Soil Unit Weight at Footing Base, γ_{eff}	62.60 pcf
Effective Soil Pressure at Footing Base, $q_b = \gamma_{eff} D_f$	93.90 psf
Angle of Friction Between Footing and Soil, $\delta = 2/3\ \phi_{eff}$	24.67 degrees
Angle of Load Eccentricity, $\theta = \tan^{-1}(e_x/e_y)$	0.00 degrees
Passive Earth Pressure Coefficient, $K_p = \tan^2(45°+\phi_{eff}/2)$	4.02
Depth Factor Parameter, k	0.50
Exponent, n	1.50

COMPUTED BEARING CAPACITY FACTORS	ANALYSIS METHOD: Terzaghi (1943)	Meyerhof (1963)	Hansen (1970)	Vesic (1973)
Bearing Capacity Factors, N_c	70.07	55.63	55.63	55.63
N_q	53.80	42.92	42.92	42.92
N_γ	68.14	53.27	47.38	66.19
Shape Factors, s_c	1.30	1.80	1.77	1.77
s_q	N/A	1.40	1.60	1.75
s_γ	0.80	1.40	0.60	0.60
Depth Factors, d_c	N/A	1.20	1.20	1.20
d_q	N/A	1.10	1.12	1.12
d_γ	N/A	1.10	1.00	1.00
Inclination Factors, i_c	N/A	N/A	1.00	1.00
i_q	N/A	N/A	1.00	1.00
i_γ	N/A	N/A	1.00	1.00
Ground Factors (base on slope), g_c	N/A	N/A	1.00	1.00
g_q	N/A	N/A	1.00	1.00
g_γ	N/A	N/A	1.00	1.00
Base Factors (tilted base), b_c	N/A	N/A	1.00	1.00
b_q	N/A	N/A	1.00	1.00
b_γ	N/A	N/A	1.00	1.00

SUMMARY OF RESULTS	ANALYSIS METHOD: Terzaghi (1943)	Meyerhof (1963)	Hansen (1970)	Vesic (1973)
Ultimate Net Bearing Capacity, c_{ult} (ksf):				
General: $q_{ult} = cN_c s_c d_c i_c g_c b_c + q_{b(}N_q s_q d_q i_q g_q b_q - 1) + 0.5\gamma_{eff} N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$	10.2	14.0	9.9	11.7
When $\phi = 0$: $q_{ult} = 5.14\ S_u\ (1 + s_c + d_c - i_c - b_c - g_c)$				

Recommended Ultimate Net Bearing Capacity, $_{ult}$	9.9 ksf
Allowable Net Bearing Capacity, $c_{all} = q_{ult}/FS_{bearing}$	3.3 ksf *
Ultimate Shear Resistance: $F_{ult} = Q \tan\delta + C_a B L$	5.0 kips
Check Sliding Failure: $(F_{ult}/P) \geq FS_{sliding}$?	N/A
Check Load Eccentricity (along X-axis): $e \leq B/6$	N/A
Check Load Eccentricity (along Y-axis): $_y \leq L/6$	N/A
Check Bearing Failure: $(c_{ult}/q_{app}) \geq FS_{bearing}$?	N/A

Notes:

* The footing should be checked for tolerable settlement. Allowable gross bearing capacity may be limited to 10 ksf due to settlement.

REFERENCES:

1. Bowles, J. (1988), "Bearing Capacity of Foundations," Foundation Analysis and Design - 4th Edition, McGraw-Hill, Inc., pp. 179-208.
2. Caltrans Bridge Design Specifications (2003), Section 4 - Foundations, pp. 4-1 through 4-17, dated November 2003.
3. Hansen, J.B. (1970), "A Revised and Extended Formula for Bearing Capacity," Danish Geotech. Institute, Bulletin No. 28, Copenhagen, 21 pp.
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5. Terzaghi, K. (1943), Theoretical Soil Mechanics, John Wiley & Sons, 510 pp.
6. Vesic, A.S. (1973), "Analysis of Ultimate Loads of Shallow Foundations," JSMFD, ASCE, Vol. 99, SM 1, Jan., pp. 45-73.

ANALYSIS OF SOIL BEARING CAPACITY OF FOOTINGS

(EZ_FTGCAP v2.0, 2006, Modified By: Dr. EZ)

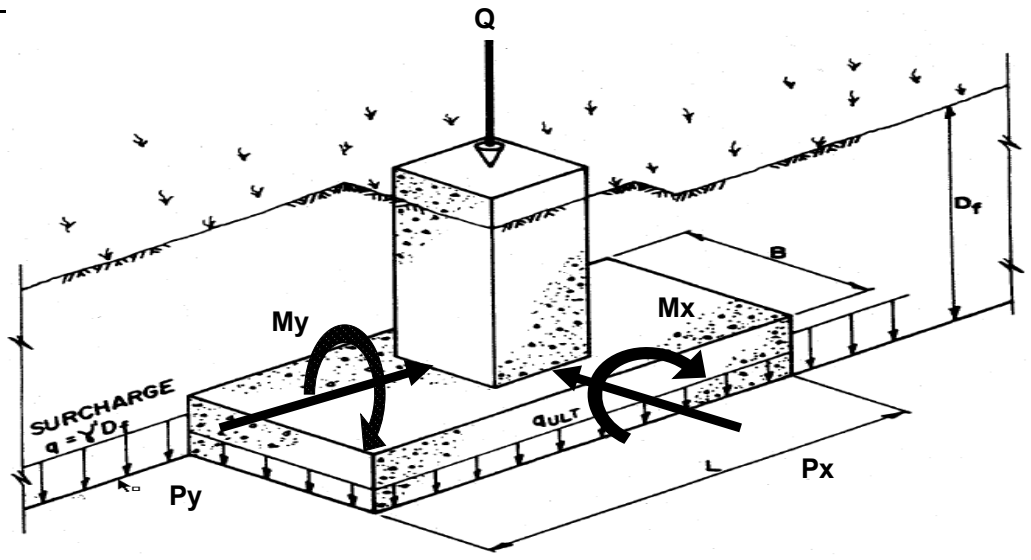
PROJECT INFORMATION		
Project Name		
Project No.		
Location		

FOOTING GEOMETRY		
Footing Location		
Footing Description	L/B=10	
Footing Width, B	2.00	ft
Footing Length, L	20.00	ft
Min. Footing Embedment Depth, D_f	1.50	ft
Bottom of Footing Elevation	0.00	ft
Design GWT Elevation	0.00	ft
Ground Slope Angle, β	0.00	degrees
Footing Base Angle, α	0.00	degrees

SOIL PARAMETERS		
Total Soil Unit Weight, γ_t	125.00	pcf
Saturated Soil Unit Weight, γ_{sat}	125.00	pcf
Soil Cohesion, c	1.00	psf
Soil Friction Angle, ϕ	37.00	degrees

FOOTING LOADS		
Design Vertical Loads, Q		kips
Shear Load (along X-axis), P_x		kips
Shear Load (along Y-axis), P_y		kips
Moment (about X-axis), M_x		kip-ft
Moment (about Y-axis), M_y		kip-ft

FACTORS OF SAFETY		
Factor of Safety for Sliding Falure, $FS_{sliding}$	1.50	
Factor of Safety for Bearing Failure, $FS_{bearing}$	3.00	
Bearing Capacity Failure Mode	General Shear	



ANALYSIS OF SOIL BEARING CAPACITY OF FOOTINGS

(EZ_FTGCAP v2.0, 2006, Modified By: Dr. EZ)

Performed By: _____ Date: _____

Reviewed By: _____ Date: _____

PROJECT INFORMATION

Project Name	
Project No.	
Location	

FOOTING GEOMETRY

Footing Location	
Footing Description	L/B=10
Footing Width, B	2.00 ft
Footing Length, L	20.00 ft
Min. Footing Embedment Depth, D_f	1.50 ft
Bottom of Footing Elevation	0.00 ft
Design GWT Elevation	0.00 ft
Ground Slope Angle, β	0.00 degrees
Footing Base Angle, α	0.00 degrees

INPUT SOIL PARAMETERS

Total Soil Unit Weight, γ_t	125.00 pcf
Saturated Soil Unit Weight, γ_{sat}	125.00 pcf
Soil Cohesion, c	1.00 psf
Soil Friction Angle, ϕ	37.00 degrees

FOOTING LOADS

Design Vertical Loads, Q	0.00 kips
Shear Load (along X-axis), P_x	0.00 kips
Shear Load (along Y-axis), P_y	0.00 kips
Moment (about X-axis), M_x	0.00 kip-ft
Moment (about Y-axis), M_y	0.00 kip-ft

FACTORS OF SAFETY

Factor of Safety for Sliding Falure, $FS_{sliding}$	1.50
Factor of Safety for Bearing Falure, $FS_{bearing}$	3.00
Bearing Capacity Failure Mode	General Shear

COMPUTED PARAMETERS

Min. Ground Surface Elevation	1.50 ft
Depth to Design GWT, Z_w	1.50 ft
Load Eccentricity (parallel to X-axis), $e_x = M_y/Q$	0.00 ft
Load Eccentricity (parallel to Y-axis), $e_y = M_x/Q$	0.00 ft
Resultant Shear Load, $P = (P_x^2 + P_y^2)^{1/2}$	0.00 kips
Effective Footing Width, $B' = B - 2\,e_x$	2.00 ft
Effective Footing Length, $L' = L - 2\,e_y$	20.00 ft
Effective Footing Area, $A_f = B'\,L'$	40.00 ft^2
Applied Gross Bearing Pressure, $q_{app} = Q/(BL)$	0.00 ksf
Effective Cohesion, c_{eff}	1.00 psf
Effective Friction Angle, ϕ_{eff}	37.00 degrees
Footing Base Adhesion, $C_a = 0.55\,c_{eff}$	0.55 psf
Depth of Wedge Zone, $D = 0.5B \tan(45°+\phi_{eff}/2)$	2.01 ft
Effective Soil Unit Weight at Footing Base, γ_{eff}	62.60 pcf
Effective Soil Pressure at Footing Base, $q_b = \gamma_{eff} D_f$	93.90 psf
Angle of Friction Between Footing and Soil, $\delta = 2/3\,\phi_{eff}$	24.67 degrees
Angle of Load Eccentricity, $\theta = \tan^{-1}(e_x/e_y)$	0.00 degrees
Passive Earth Pressure Coefficient, $K_p = \tan^2(45°+\phi_{eff}/2)$	4.02
Depth Factor Parameter, k	0.75
Exponent, n	1.09

COMPUTED BEARING CAPACITY FACTORS	ANALYSIS METHOD: Terzaghi (1943)	Meyerhof (1963)	Hansen (1970)	Vesic (1973)
Bearing Capacity Factors, N_c	70.07	55.63	55.63	55.63
N_q	53.80	42.92	42.92	42.92
N_γ	68.14	53.27	47.38	66.19
Shape Factors, s_c	1.03	1.08	1.08	1.08
s_q	N/A	1.04	1.06	1.08
s_γ	0.80	1.04	0.96	0.96
Depth Factors, d_c	N/A	1.30	1.30	1.30
d_q	N/A	1.15	1.18	1.18
d_γ	N/A	1.15	1.00	1.00
Inclination Factors, i_c	N/A	N/A	1.00	1.00
i_q	N/A	N/A	1.00	1.00
i_γ	N/A	N/A	1.00	1.00
Ground Factors (base on slope), g_c	N/A	N/A	1.00	1.00
g_q	N/A	N/A	1.00	1.00
g_γ	N/A	N/A	1.00	1.00
Base Factors (tilted base), b_c	N/A	N/A	1.00	1.00
b_q	N/A	N/A	1.00	1.00
b_γ	N/A	N/A	1.00	1.00

SUMMARY OF RESULTS	ANALYSIS METHOD: Terzaghi (1943)	Meyerhof (1963)	Hansen (1970)	Vesic (1973)
Ultimate Net Bearing Capacity, q_{ult} (ksf): General: $q_{ult} = cN_c s_c d_c i_c g_c b_c + q_{b(}N_q s_q d_q i_q g_q b_q - 1) + 0.5\gamma_{eff} N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$ When $\phi = 0$: $q_{ult} = 5.14\,S_u\,(1 + s_c + d_c - i_c - b_c - g_c)$	8.4	8.8	7.9	9.1

Recommended Ultimate Net Bearing Capacity, q_{ult}	7.9 ksf
Allowable Net Bearing Capacity, $q_{all} = q_{ult}/FS_{bearing}$	2.6 ksf *
Ultimate Shear Resistance: $P_{ult} = Q \tan\delta + C_a B L$	22.0 kips
Check Sliding Failure: $(P_{ult}/P) \geq FS_{sliding}$?	N/A
Check Load Eccentricity (along X-axis): $e_x \leq B/6$	N/A
Check Load Eccentricity (along Y-axis): $e_y \leq L/6$	N/A
Check Bearing Failure: $(q_{ult}/q_{app}) \geq FS_{bearing}$?	N/A

Notes:

* The footing should be checked for tolerable settlement. Allowable gross bearing capacity may be limited to 10 ksf due to settlement.

REFERENCES:

1. Bowles, J. (1988), "Bearing Capacity of Foundations," Foundation Analysis and Design - 4th Edition, McGraw-Hill, Inc., pp. 179-208.
2. Caltrans Bridge Design Specifications (2003), Section 4 - Foundations, pp. 4-1 through 4-17, dated November 2003.
3. Hansen, J.B. (1970), "A Revised and Extended Formula for Bearing Capacity," Danish Geotech. Institute, Bulletin No. 28, Copenhagen, 21 pp.
4. Meyerhof, G.G. (1963), "Some Recent Research on Bearing Capacity of Foundations," Canadian Geotech. Journal, Vol. 1, No. 1, pp. 16-26.
5. Terzaghi, K. (1943), Theoretical Soil Mechanics, John Wiley & Sons, 510 pp.
6. Vesic, A.S. (1973), "Analysis of Ultimate Loads of Shallow Foundations," JSMFD, ASCE, Vol. 99, SM 1, Jan., pp. 45-73.

EVALUATION OF STATIC & SEISMIC WALL PRESSURES

EZ_SeismicEarthPressures v1.0

INPUT								
Soil Unit Weight	γ	=	125	pcf	=	19.6	kN/m^3	
Friction Angle	ϕ	=	37	Degrees	=	0.6457718	radians	
Wall Friction Angle	δ	=	0.0	Degrees	=	0	radians	
Wall Inclination	β	=	0	Degrees	=	0	radians	
Slope Inclination	i	=	0.0	Degrees	=	0	radians	
Horizontal Seismic Coefficient	K_h	=	0.380					
Vertical Seismic Coefficient	K_v	=	0					
At-Rest Earth Pressure Coefficient	K_0	=	0.4					
Wall Depth for Pressure Diagrams	H	=	10	feet				

OUTPUT

COULOMB STATIC				SEISMIC INCREMENT			
K_A	EFP		i_A	K_{AE}-K_A	EFP		i_{AE}
	pcf	kN/m3	deg		pcf	kN/m3	deg
0.249	31	4.9	63.5	0.272	34.0	5.3	44.30
RANKINE STATIC				max K_h=	0.75355405		
0.25	31	4.9	63.5	max β=	16.19320899		deg

COULOMB CALCULATION

STATIC ACTIVE EARTH PRESSURE							STATIC CRITICAL INCLINATION		
	A	B	C	D	E	K_A	C1	C2	i_A
degrees	37.0	0.0	37.0	37.0	0.0				63.50
radians	0.646	0.000	0.646	0.646	0.000	0.249	1.252	1.000	1.108284075

SEISMIC ACTIVE EARTH PRESSURE							SEISMIC CRITICAL INCLINATION		
	θ	A'	B'	C'	D'	E'	K_{AE}	C1E	C2E	i_{AE}
degrees	20.8	16.2	20.8	37.0	16.2	0				44.30
radians	0.363	0.283	0.363	0.646	0.283	0.000	0.521	1.582	2.419	0.7731234

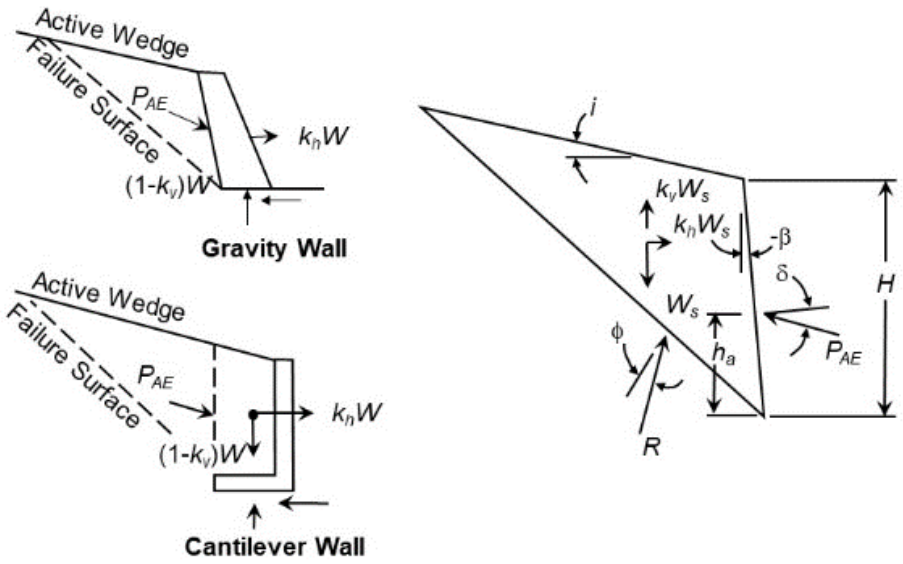


Figure A11.3.1-1—Mononobe–Okabe Method Force Diagrams

$$K_{AE} = \frac{\cos^2(\phi - \theta_{MO} - \beta)}{\cos\theta_{MO}\cos^2\beta\cos(\delta + \beta + \theta_{MO})} \times \left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta_{MO} - i)}{\cos(\delta + \beta + \theta_{MO})\cos(i - \beta)}}\right]^{-2} \qquad \text{(A11.3.1-1)}$$

Project No.: 20230500.001A
Subject: Mill Creek Basion
Passive Pressure and Coefficient of Friction Calculation

Computed by: QH Date: 7/28/2025 Chk'd by: ECP Date: 7/28/2025

Problem Statement:

Calculate passive pressure and coefficient of friction for the subgrade

Calculations:

Unit Weight	γ_{tot} =	125		pcf	
Cohesion	c =	0		psf	
Effective Friction Angle	ϕ =	37		Deg.	
Active Earth Pressure Coef.	K_a =	0.25			= [TAN (45 - ϕ/2)]^2
Passive Earth Pressure Coef.	K_p =	4.02			= [TAN (45 + ϕ/2)]^2 Assume zero wall friction
Passive Earth Pressure at Cut Line	σ_{p0} =	0		psf	=2c*(K_p)$^{0.5}$
	K_p * γ_{tot} =	503		pcf	
Ultimate Passive Earth Pressure at Depth H below Cut Line	P_p =	**0 + 503 * H**		psf	= σ_{p0} + K_p * γ_{tot} * H
Factor of Safety for Permanent Wall	FS_{perm}=	1.50			
Allowable Passive Pressure for Permanent Wall	$P_{p\text{-}perm}$ =	0 + 335 * H		psf	= P_p / Fs_{perm}
Use	$P_{p\text{-}perm}$ =	**300 * H**		psf	
Ultimate Coefficient of Friction	μ_1 =	0.75			= TAN (ϕ) AASHTO C10.6.3.4
Factor of Safety	FS=	1.50			
Allowable Coefficient of Friction	μ_{a1} =	**0.50**			= μ_1 / FS
Use	μ_{a1} =	**0.4**			

ATTACHMENT III

Mill Creek Regional Flood Frequency Analysis

San Bernardino Valley Water Conservation District

Mill Creek Regional Flood Frequency Analysis



San Bernardino County, California

March 2025

FINAL DRAFT

Prepared for:

San Bernardino Valley
Water Conservation District
39611 Oak Glen Road, Bldg. #12
Oak Glen, CA 92399

Prepared by:

Q3 Consulting
27051 Towne Centre Drive, Suite 270
Foothill Ranch, CA 92610
JN 40.076.000

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1 REGIONAL FLOOD FREQUENCY ANALYSIS

A regional flood frequency analysis was conducted herein to serve as the basis to (1) fill data gaps in Mill Creek streamflow record, (2) extend the Mill Creek streamflow record, and (3) synthesize Mill Creek predictive streamflow records, which capture the range and average translated regional streamflow records as well as the minimum and maximum envelope of the predictive range to account for the spatial and temporal variability of the hydrologic region..

This synthesized predictive streamflow record provides the basis for defining hydrologic regimes to use in the analysis of long-term hydraulics, sedimentation, and scour in Mill Creek adjacent to the San Bernardino Valley Water Conservation District recharge facilities along the south bank, and ultimately, contribute to the planning and design of water conservation improvements.

1.1 Goals and objectives

The objectives focused on the determination of the following for the portion of the San Bernardino mountains and upland valleys that are relevant to the drainages selected for potential water conservation activities along Mill Creek:

- *Regional flood frequency curves*. Develop a regional flood frequency analysis focused on Mill Creek based on available streamflow data to determine flood frequency statistics of annual maximum peak and daily mean streamflow for the hydrologic region
- *Empirical streamflow relationships*. Derive empirical streamflow relationships between Mill Creek and the remaining gauged watersheds, which form the hydrologic region to serve as the basis for computing Mill Creek streamflow equivalents
- *Synthesis of predictive streamflow records.* Synthesize a range of Mill Creek predictive streamflow records, which cover the minimum, average, and maximum translation from the region.

1.2 General approach and assumptions

The following general approach and assumptions were employed herein:

- Conduct data screening, including the evaluation of discordancy, heterogeneity, goodness-of-fit, and intersite dependencies to support the adoption of an acceptable hydrologic region centered on the Mill Creek watershed
- Perform a regional flood frequency analysis on the adopted hydrologic region based on the method of *L*-moments (Hosking and Wallis, 1997)
- Compute 50-, 20-, 10-, 2-, 1-, 0.5-, 0.2-, and 0.1-percent annual chance (0.5, 0.8, 0.9, 0.95, 0.98, 0.99, 0.995, 0.998, 0.999 nonexceedance) event quantiles
- Derive empirical streamflow relationships based on event quantiles for annual maximum peak and daily mean streamflow, pairing Mill Creek with each of the remaining gauged watersheds, which form the adopted hydrologic region
- Synthesize Mill Creek predictive streamflow records based on streamflow equivalents computed from the empirical streamflow relationships derived above

1.3 Data screening

There are a combined 28 available active and inactive streamflow gauges (sites) located in either the Santa Ana River or Salton Sea Basins, as depicted in Figure 1-1, that were initially considered for regional flood frequency analysis. Site information is listed in Table 1-1, which includes drainage area, elevation, location (latitude and longitude), length of record, and range of water years covered where a water year is considered to span from October 1st to September 30th of the following calendar year.

These available 28 sites were screened for their potential use in regional flood frequency analysis. Characteristics of the streamflow record for each of these sites were inventoried as shown in Table 1-2, and subsequently used to narrow the list of potential sites suitable for regional flood frequency analysis. Of the available 28 sites, one was excluded due to climatic dissimilarities, five (5) were excluded due to extensive urbanization, and seven (7) were excluded due to limitations in the type, quantity, and/or quality of observed data, resulting in 15 sites ***highlighted*** in Table 1-2 that remain available for further evaluation.

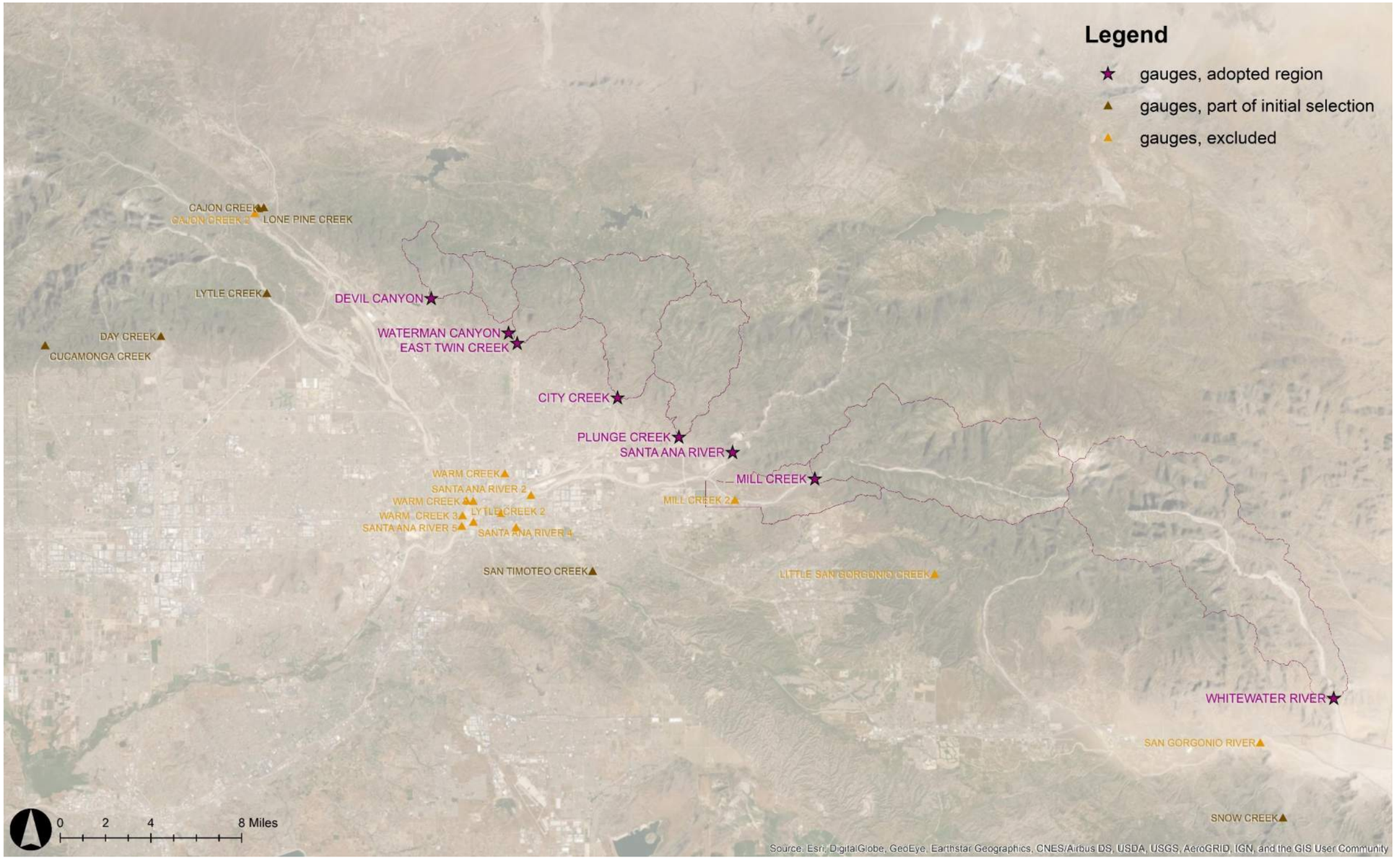
The tributary drainage areas associated with the remaining 15 sites range from less than five (5) square miles to 210 square miles. The water years covered among active records span from 1897 through 2021.

Data records of the remaining 15 sites were evaluated further and adjusted, as necessary to resolve issues related to the following:

- *Missing data*. There are 108 data points missing from a total of 1,221 years of recorded observations resulting in an effective dataset of 1,113 years of recorded observations
- *Historic peaks*. The Whitewater River record identifies a historic peak of 42,000 *cfs* occurring on March 2, 1938; and the Santa Ana River record identifies a historic peak of 53,700 *cfs* occurring on February 2, 1891; these historical observations precede the operational periods of their respective records and represent the only historical peaks on record for the 15 sites considered; these historical peaks were added to the record because of their significance despite their unknown basis of determination
- *Maximum daily averages*. There are 36 data points annotated as maximum daily averages; these observations were generally assumed to reflect the maximum flow conditions for the affected records and water years; and thus, were retained in the dataset
- *Zero values*. There are a combined 11 zero or blank data points; Zero values may occur, but not very often, which is not unreasonable in some years for the climatic conditions experienced in the Santa Ana and Salton regions; and accordingly, these data points were included in the dataset
- *Estimated values*. There are 47 data points distributed among 13 of the 15 records that are annotated as discharge estimates; there is no apparent reason to suspect any of these data points are erroneous and thus, were retained in the dataset
- *Affects related to regulation or diversion*. The following ten (10) sites were identified as potentially being affected by either regulation or diversion: (1) Santa Ana River, (2) Mill Creek, (3) Plunge Creek, (4) City Creek, (5) San Timoteo Creek, (6) Lytle Creek, (7) Devil Canyon, (8) Day Creek, (9) Whitewater River, and (10) Snow Creek; and it has been demonstrated that *L*-moments are negligibly affected by regulation (Azquith, 2002); and thus, the affected data points for these ten (10) sites were retained in the dataset
- *Unknown dates*. The record for Devil Canyon has two (2) data points identified as having an unknown month or day, but otherwise, there is no apparent reason to suspect these data points are erroneous; and thus, were retained in the dataset
- *Affects related to urbanization, mining, agricultural activities, and/or channelization*. There are no data points identified as being potentially influenced by land management or development practices
- *Change in base discharge*. There are no data points affected by a change in the base discharge

The resolved 15-site record dataset contains a total of 1113 streamflow observations, with effective sample record lengths varying from 31 to 103 water years. The at-site sample *L*-CV ranges from 0.61 to 0.85, the at-site sample *L*-skewness ranges from 0.53 to 0.81, and the L-Kurtosis ranges from 0.32 to 0.63. The unbiased at-site sample L-moments for the selected group of 15 sites are presented in Table 1-3 (annual maximum peak streamflow) and Table 1-4 (annual maximum daily mean streamflow).

Figure 1-1. USGS streamflow gauge location map



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Table 1-1. Selected streamflow gauge sites located within the Santa Ana and Salton Basins

USGS ID	description	hydrologic unit	NAD83		drainage area, in *sq mi*	NGVD29 elevation, in *feet*	record length, in *years*	recorded water years
			latitude	longitude				
11051500	Santa Ana River near Mentone	Santa Ana	34.1083	-117.1006	210	1590	102+1	1891; 1897-1998
11054000	Mill Creek near Yucaipa	Santa Ana	34.0908	-117.0375	42.4	1580	67	1919-86
11055000	Mill Creek near Mentone	Santa Ana	34.0778	-117.0992	49.1	1280	27	1939-65
11055500	Plunge Creek near East Highlands	Santa Ana	34.1183	-117.1417	16.9	1010	103	1919-2021
11055800	City Creek near Highland	Santa Ana	34.1439	-117.1887	19.6	1590	102	1920-2021
11056000	Santa Ana River near San Bernardino	Santa Ana	34.0820	-117.2559	329	1840	32	1929-60
11056200	Santa Ana River at Waterman Ave at San Bernardino	Santa Ana	34.0706	-117.2789	359		24	1961-78
11056500	Little San Gorgonio Creek near Beaumont	Santa Ana	34.0292	-116.9461	1.74		36	1950-85
11057000	San Timoteo Creek near Redlands	Santa Ana	34.0331	-117.2089	118		53	1927-79
11057500	San Timoteo Creek near Loma Linda	Santa Ana	34.0614	-117.2675	125		67	1955-2021
11058500	East Twin Creek near Arrowhead Springs	Santa Ana	34.1792	-117.2656	8.8		102	1919-2021
11058600	Waterman Canyon Creek near Arrowhead Springs	Santa Ana	34.1858	-117.2722	4.82		110	1912-2021
11059000	Warm Creek Floodway at San Bernardino	Santa Ana	34.0958	-117.2759	47.8		20	1961-80
11059300	Santa Ana River at E Street near San Bernardino	Santa Ana	34.0650	-117.3003	541		83	1939-2021
11060400	Warm Creek near San Bernardino	Santa Ana	34.0783	-117.3003	11		58	1964-2021
11062000	Lytle Creek near Fontana	Santa Ana	34.2122	-117.4581	46.6		102	1920-2021
11063000	Cajon Creek near Keenbrook	Santa Ana	34.2670	-117.4600	40.6		64	1920-83
11063500	Lone Pine Creek near Keenbrook	Santa Ana	34.2664	-117.4639	15.1		89	1920-2008
11063510	Cajon Creek below Lone Pine Creek near Keenbrook	Santa Ana	34.2633	-117.4670	56.5	1610	50	1972-2021
11063680	Devil Canyon Creek near San Bernardino	Santa Ana	34.2083	-117.3314	5.49	2000	102	1920-2021
11065000	Lytle Creek at Colton	Santa Ana	34.0789	-117.3056	186	1320	64	1958-2021
11065800	Warm Creek near Colton	Santa Ana	34.0692	-117.3087	192	2000	42	1921-62
11066050	Santa Ana River at Colton	Santa Ana	34.0625	-117.3092	721		5	1962-66
11067000	Day Creek near Etiwanda	Santa Ana	34.1850	-117.5398	4.56	2870	45	1928-72
11073470	Cucamonga Creek near Upland	Santa Ana	34.1795	-117.6289	9.68	2600	48	1928-75
10256000	Whitewater River at Whitewater	Salton	33.9467	-116.6408	57.5	1950	31+1	1938; 1949-79
10256400	San Gorgonio River near Whitewater	Salton	33.9189	-116.6978	154	2916	14	1966-79
10256500	Snow Creek near Whitewater	Salton	33.8706	-116.6811	10.9	2010	99	1923-2021

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Table 1-2. Initial screening of gauged sites considered for regional flood frequency analysis

USGS ID	description	hydrologic unit	record length, in *years*	missing records	zero records	USGS qualification code occurrences									exclusion code		
						1	2	5	6	7	Bm	C	D	E	1	2	3
11051500	Santa Ana River*	Santa Ana	102	0	1	20	5	82	0	1	0	0	0	0			
11054000	Mill Creek	Santa Ana	67	11	0	1	11	56	0	0	0	0	0	0			
11055000	Mill Creek 2	Santa Ana	27	0	1	0	0	27	0	0	1	0	0	0			✔
11055500	Plunge Creek	Santa Ana	103	0	0	0	1	102	0	0	0	0	0	0			
11055800	City Creek	Santa Ana	102	0	0	0	0	102	0	0	0	0	0	0			
11056000	Santa Ana River 2	Santa Ana	32	17	1	0	2	0	0	0	0	0	0	0			✔
11056200	Santa Ana River 3	Santa Ana	24	8	0	0	3	0	0	0	0	0	0	0			✔
11056500	Little San Gorgonio Creek	Santa Ana	36	0	1	0	7	0	0	0	1	0	0	0			✔
11057000	San Timoteo Creek	Santa Ana	53	2	0	0	8	51	0	0	0	0	0	0			
11057500	San Timoteo Creek 2	Santa Ana	67	9	0	0	0	0	0	0	0	41	0	0		✔	
11058500	East Twin Creek	Santa Ana	102	2	3	0	3	0	0	0	0	0	0	0			
11058600	Waterman Canyon	Santa Ana	110	28	0	5	1	0	0	0	0	0	0	0			
11059000	Warm Creek	Santa Ana	20	0	0	0	1	0	0	0	0	0	0	0			✔
11059300	Santa Ana River 4	Santa Ana	83	14	1	4	2	0	68	0	0	46	0	0		✔	
11060400	Warm Creek 2	Santa Ana	58	3	0	1	1	0	0	0	0	46	0	0		✔	
11062000	Lytle Creek	Santa Ana	102	16	1	0	5	85	0	0	0	0	0	0			
11063000	Cajon Creek	Santa Ana	64	6	0	1	2	0	0	0	0	0	0	0			
11063500	Lone Pine Creek	Santa Ana	89	11	0	0	1	0	0	0	0	0	0	0			
11063510	Cajon Creek 2	Santa Ana	50	6	0	1	0	0	0	0	0	0	0	0			
11063680	Devil Canyon	Santa Ana	102	2	4	0	0	98	0	0	2	0	0	0			
11065000	Lytle Creek 2	Santa Ana	64	1	0	0	0	63	0	0	0	63	0	0		✔	
11065800	Warm Creek 3	Santa Ana	42	0	0	0	2	42	0	0	0	0	0	0	✔		
11066050	Santa Ana River 5	Santa Ana	5	0	0	0	1	5	0	0	0	0	0	0			✔
11067000	Day Creek	Santa Ana	45	0	1	0	3	44	0	0	0	0	0	0			
11073470	Cucamonga Creek	Santa Ana	48	1	1	0	3	0	0	0	0	0	0	0			
10256000	Whitewater	Salton	31	0	0	0	1	30	0	1	0	0	0	0			
10256400	San Gorgonio River	Salton	14	0	0	0	0	0	0	0	0	0	0	0			✔
10256500	Snow Creek	Salton	99	29	0	9	3	0	70	0	0	0	0	0			

qualification code key:
- 1 - discharge is a maximum daily average
- 2 - discharge is an estimate
- 5 - discharge affected to unknown degree by regulation or diversion
- 6 - discharge affected by regulation or diversion
- 7 - discharge is a historic peak
- B - month or day of occurrence is unknown or not exact
- C - all or part of the record affected by urbanization, mining, agricultural changes, channelization, or other
- D - base discharge changed during this year
- E - only annual maximum peak available for this year

exclusion code key:
- 1 - governing climate zone considered too dissimilar
- 2 - significantly influenced by urbanization
- 3 - limited by useable record years

potential sites selected for regional analysis

*limited to record prior to the completion of 7 Oaks Dam

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Table 1-3. Maximum annual peak streamflow unbiased L-moments, 15 sites

site i	USGS ID	description	n_i	l_1	l_2	$t^{(i)}$	$t_3^{(i)}$	$t_4^{(i)}$
1	10256000	Whitewater River	31	3403.19	2890.28	0.8493	0.8063	0.6302
2	11054000	Mill Creek	56	1737.66	1384.76	0.7969	0.7389	0.5312
3	11051500	Santa Ana River	102	3403.03	2548.68	0.7489	0.7030	0.5409
4	11055500	Plunge Creek	102	777.97	489.07	0.6287	0.5327	0.3219
5	11055800	City Creek	102	1019.89	683.08	0.6698	0.5932	0.3717
6	11058500	East Twin Creek	97	528.34	354.53	0.6710	0.5726	0.3458
7	11058600	Waterman Canyon	82	210.04	128.19	0.6103	0.5603	0.3740
8	11063680	Devil Canyon	96	234.85	168.84	0.7189	0.7103	0.5517
9	10256500	Snow Creek	70	638.89	461.15	0.7218	0.6441	0.4555
10	11062000	Lytle Creek	85	1674.55	1198.37	0.7156	0.6532	0.4419
11	11063000	Cajon Creek	58	1956.48	1346.05	0.6880	0.5981	0.3659
12	11063500	Lone Pine Creek	78	387.50	248.45	0.6412	0.5657	0.4236
13	11067000	Day Creek	44	342.25	236.99	0.6925	0.6647	0.5179
14	11073470	Cucamonga Creek	46	668.57	487.47	0.7291	0.7123	0.5796
15	11057000	San Timoteo Creek	51	1253.88	870.20	0.6940	0.6389	0.3486

n_i - effective record length of site i

sample L-moments:

l_1 - mean

l_2 - L-scale

L-moment ratios:

t - coefficient of L-variation

t_3 - L-skewness

t_4 - L-kurtosis

Table 1-4. Maximum annual daily mean streamflow unbiased L-moments, 15 sites

site i	USGS ID	description	n_i	l_1	l_2	$t^{(i)}$	$t_3^{(i)}$	$t_4^{(i)}$
1	10256000	Whitewater River	31	409.65	322.21	0.7865	0.8067	0.6656
2	11054000	Mill Creek	58	486.03	385.54	0.7932	0.7346	0.5322
3	11051500	Santa Ana River	102	1109.20	724.85	0.6535	0.5595	0.3694
4	11055500	Plunge Creek	104	235.09	139.75	0.5945	0.4842	0.2871
5	11055800	City Creek	103	313.60	198.80	0.6339	0.5605	0.3444
6	11058500	East Twin Creek	103	130.99	76.67	0.5853	0.4602	0.2335
7	11058600	Waterman Canyon	84	54.65	31.96	0.5849	0.5656	0.4149
8	11063680	Devil Canyon	101	61.06	37.10	0.6076	0.5735	0.3989
9	10256500	Snow Creek	73	213.04	150.95	0.7086	0.6534	0.4494
10	11062000	Lytle Creek	97	585.77	430.89	0.7356	0.6804	0.5032
11	11063000	Cajon Creek	57	541.54	381.46	0.7044	0.5979	0.3343
12	11063500	Lone Pine Creek	77	82.32	57.86	0.7028	0.6377	0.4758
13	11067000	Day Creek	44	186.52	152.34	0.8168	0.8222	0.7255
14	11073470	Cucamonga Creek	47	255.36	191.06	0.7482	0.7242	0.5902
15	11057000	San Timoteo Creek	48	214.19	154.70	0.7223	0.6541	0.4474

n_i - effective record length of site i

sample L-moments:

l_1 - mean

l_2 - L-scale

L-moment ratios:

t - coefficient of L-variation

t_3 - L-skewness

t_4 - L-kurtosis

1.4 Intersite dependence

Intersite dependence between the selected sites was evaluated to account for redundancies in the regional dataset.

The form of dependence structure used arises from assuming that if each site's frequency distribution were transformed to the Normal distribution-call this transformation T – then the joint distribution for all N sites would be multivariate Normal. This is considered a realistic dependence structure for at least some kinds of environmental data (Hosking and Wallis, 1987). Data generation then involves the following steps: generate the matrix **R** of intersite correlations; generate a random vector **y** having multivariate Normal distribution with covariance matrix **R**; and apply the inverse of transformation T to attain data with the required marginal distribution.

The correlation matrix **R** used in simulation of correlated data should be chosen to be consistent with the correlation patterns in the data. When there is not a particular pattern of correlation among the sites in the region, it is reasonable to take the sites to be equally correlated, that is, the correlation between sites i and j is $\rho_{ij} = \rho$ for i not equal to j and the matrix **R** has the form

$$\mathbf{R} = \begin{bmatrix} 1 & \rho & \rho & \Lambda & \rho \\ \rho & 1 & \rho & \Lambda & \rho \\ \rho & \rho & 1 & \Lambda & \rho \\ \text{M} & \text{M} & & \text{O} & \\ \rho & \rho & & & 1 \end{bmatrix}$$

The quantity ρ is estimated by the average cross-correlation of the data for all pairs of sites. Let Q_{ik} be the data value for site i at time point k. The sample correlation between sites i and j is given by

$$r_{ij} = \frac{\sum_k \left(Q_{ik} - \overset{\prime}{Q}_i\right)\left(Q_{jk} - \overset{\prime}{Q}_j\right)}{\left\{\sum_k \left(Q_{ik} - \overset{\prime}{Q}_i\right)^2 \sum_k \left(Q_{jk} - \overset{\prime}{Q}_j\right)^2\right\}^{0.5}},$$

where

$$\overset{\prime}{Q}_i = n_{ij}^{-1} \sum_k Q_{ik}$$

and the sums over k extend over all time points for which sites i and j both have data, and n_{ij} is the number of such time points. The average intersite correlation is given by

$$\bar{r} = \left\{0.5N\left(N-1\right)\right\}^{-1} \sum_{1 \le i < j \le N} \sum r_{ij}$$

The initial 15-site region produces an intersite dependence coefficient of 0.49, which is quite high. To limit the degree of cross correlation within the dataset, several iterations were conducted to identify a potential region, which minimizes intersite dependence. Ultimately, an 8-site region was selected, which produces an intersite dependence of 0.19 (annual maximum peak streamflow) and 0.21 (annual maximum

daily mean streamflow). This 8-site region at-site L-moments are presented in Table 1-5 (annual maximum peak streamflow) and Table 1-6 (annual maximum daily mean streamflow).

The annual maximums are graphically displayed in Figure 1-2 (annual maximum peak streamflow for Whitewater, Mill, Santa Ana, and Plunge), Figure 1-3 (annual maximum peak streamflow for City, East Twin, Waterman, and Devil), Figure 1-4 (annual maximum daily mean streamflow for Whitewater, Mill, Santa Ana, and Plunge), and Figure 1-5 (annual maximum daily mean streamflow for City, East Twin, Waterman, and Devil).

Table 1-5. Annual maximum peak streamflow unbiased sample *L*-moments, 8 sites

site i	USGS ID	description	n_i	l_1	l_2	$t^{(i)}$	$t_3^{(i)}$	$t_4^{(i)}$
1	10256000	Whitewater River 1	31	409.65	322.21	0.7865	0.8067	0.6656
2	11054000	Mill Creek 1	58	486.03	385.54	0.7932	0.7346	0.5322
3	11051500	Santa Ana River 1	102	1109.20	724.85	0.6535	0.5595	0.3694
4	11055500	Plunge Creek	104	235.09	139.75	0.5945	0.4842	0.2871
5	11055800	City Creek	103	313.60	198.80	0.6339	0.5605	0.3444
6	11058500	East Twin Creek	103	130.99	76.67	0.5853	0.4602	0.2335
7	11058600	Waterman Canyon	84	54.65	31.96	0.5849	0.5656	0.4149
8	11063680	Devil Canyon	101	61.06	37.10	0.6076	0.5735	0.3989
		weighted means:		1.0000		0.6340	0.5621	0.3698

n_i - effective record length of site i

sample L-moments:

l_1 - mean

l_2 - L-scale

L-moment ratios:

t - coefficient of L-variation

t_3 - L-skewness

t_4 - L-kurtosis

Table 1-6. Annual maximum daily mean streamflow unbiased sample *L*-moments, 8 sites

site i	USGS ID	description	n_i	l_1	l_2	$t^{(i)}$	$t_3^{(i)}$	$t_4^{(i)}$
1	10256000	Whitewater River 1	31	409.65	322.21	0.7865	0.8067	0.6656
2	11054000	Mill Creek 1	58	486.03	385.54	0.7932	0.7346	0.5322
3	11051500	Santa Ana River 1	102	1109.20	724.85	0.6535	0.5595	0.3694
4	11055500	Plunge Creek	104	235.09	139.75	0.5945	0.4842	0.2871
5	11055800	City Creek	103	313.60	198.80	0.6339	0.5605	0.3444
6	11058500	East Twin Creek	103	130.99	76.67	0.5853	0.4602	0.2335
7	11058600	Waterman Canyon	84	54.65	31.96	0.5849	0.5656	0.4149
8	11063680	Devil Canyon	101	61.06	37.10	0.6076	0.5735	0.3989
		weighted means:		1.0000		0.6340	0.5621	0.3698

n_i - effective record length of site i

sample L-moments:

l_1 - mean

l_2 - L-scale

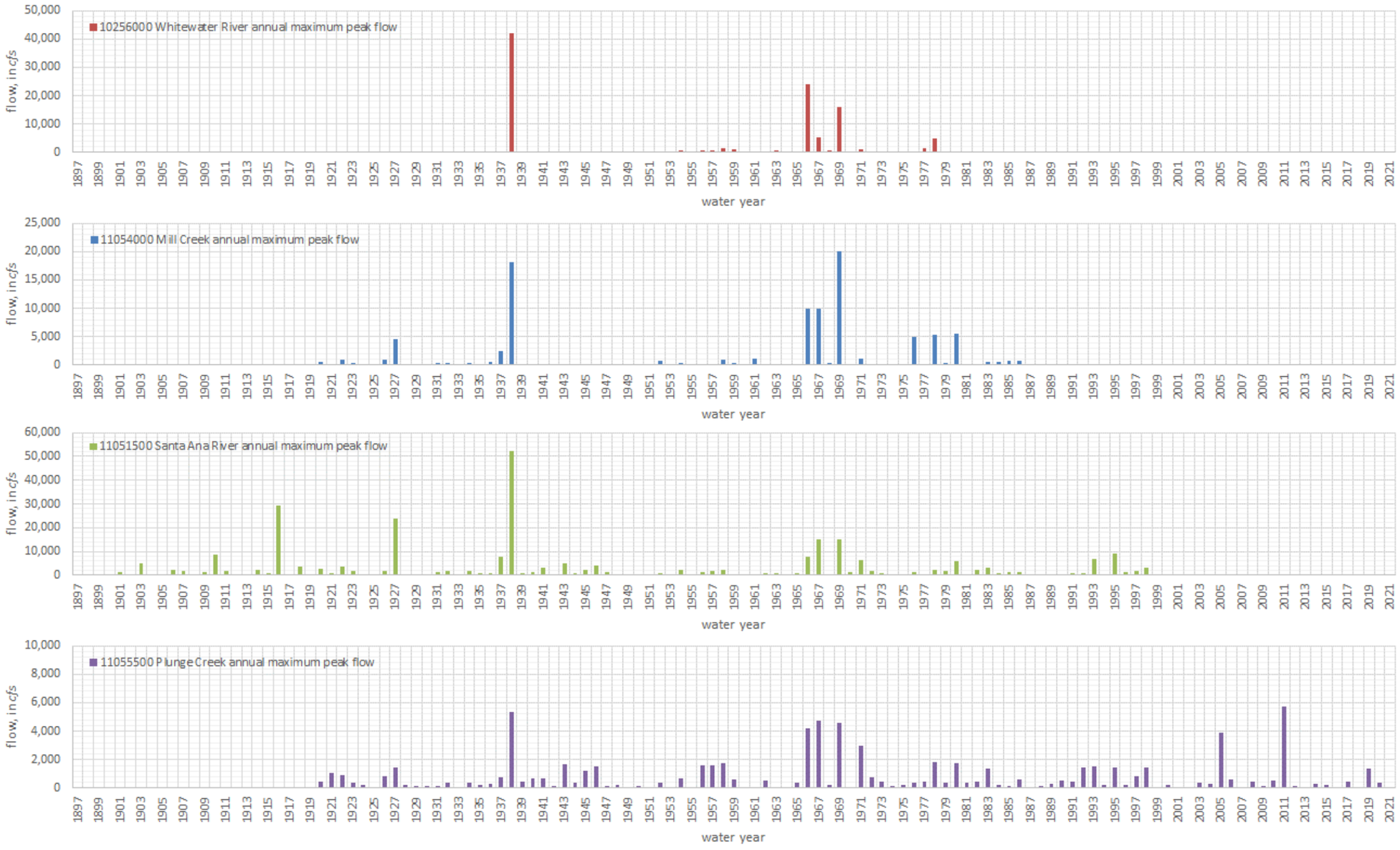
L-moment ratios:

t - coefficient of L-variation

t_3 - L-skewness

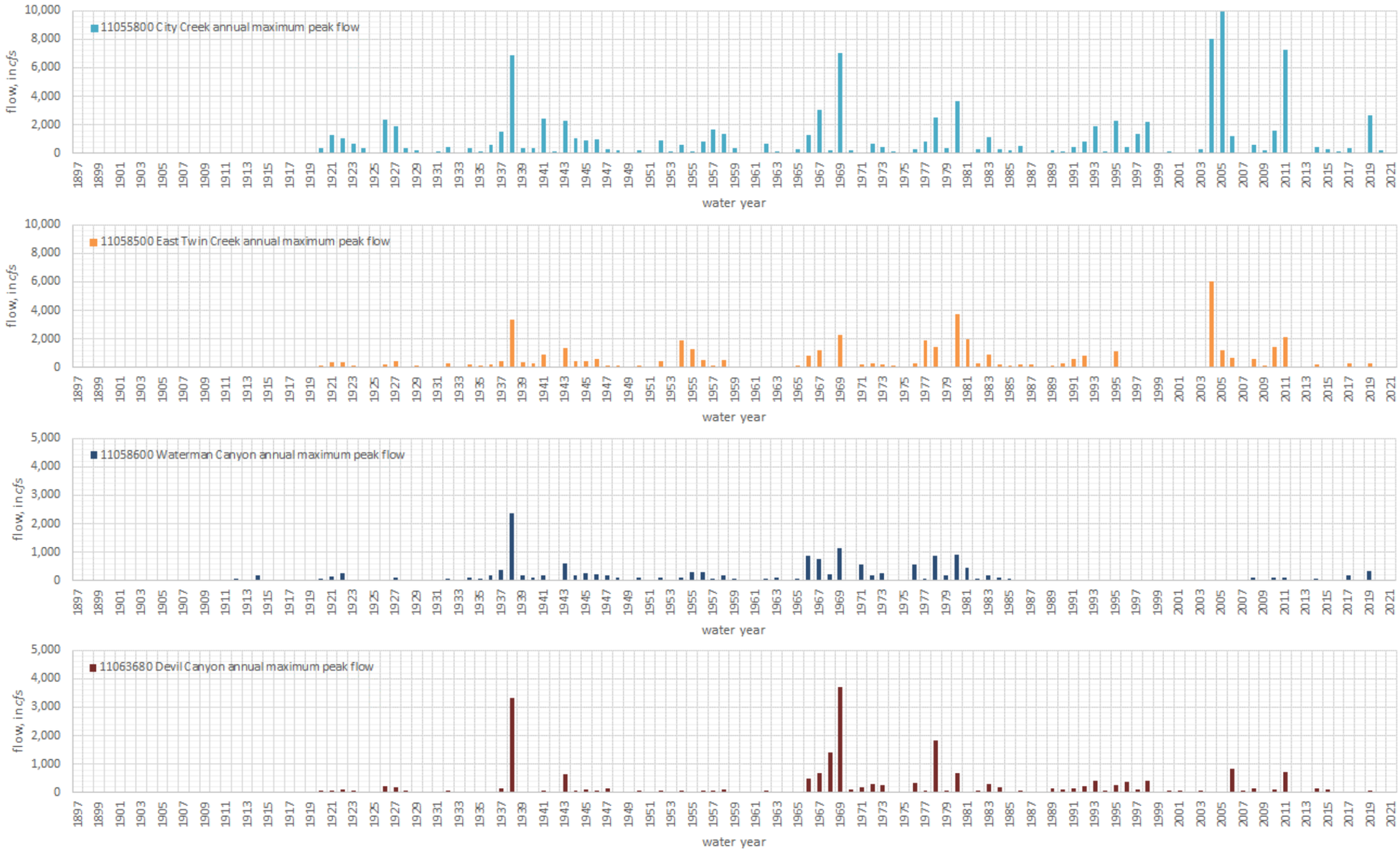
t_4 - L-kurtosis

Figure 1-2. Annual maximum peak streamflow (Whitewater, Mill, Santa Ana, and Plunge)



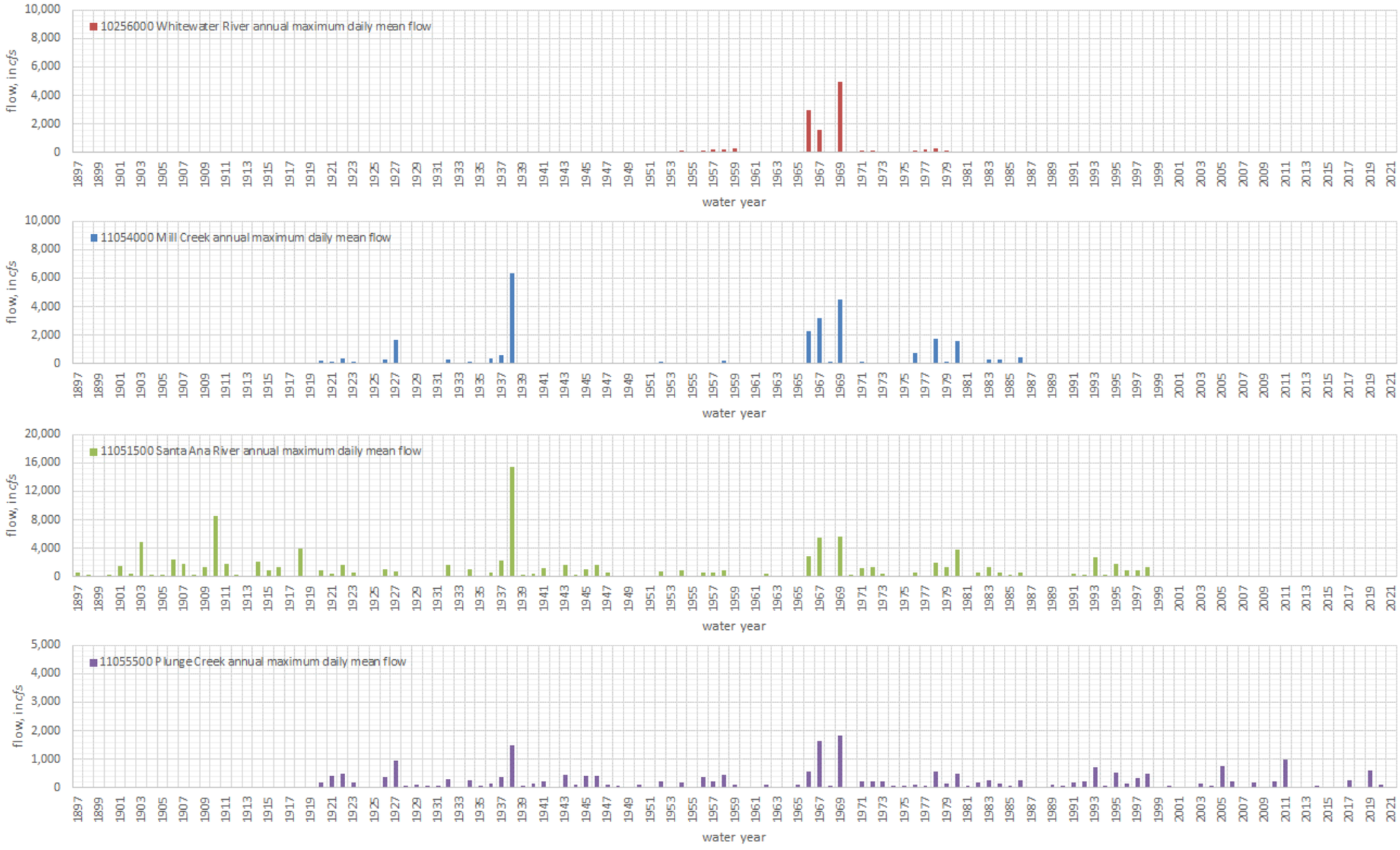
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Figure 1-3. Annual maximum peak streamflow (City, East Twin, Waterman, and Devil)



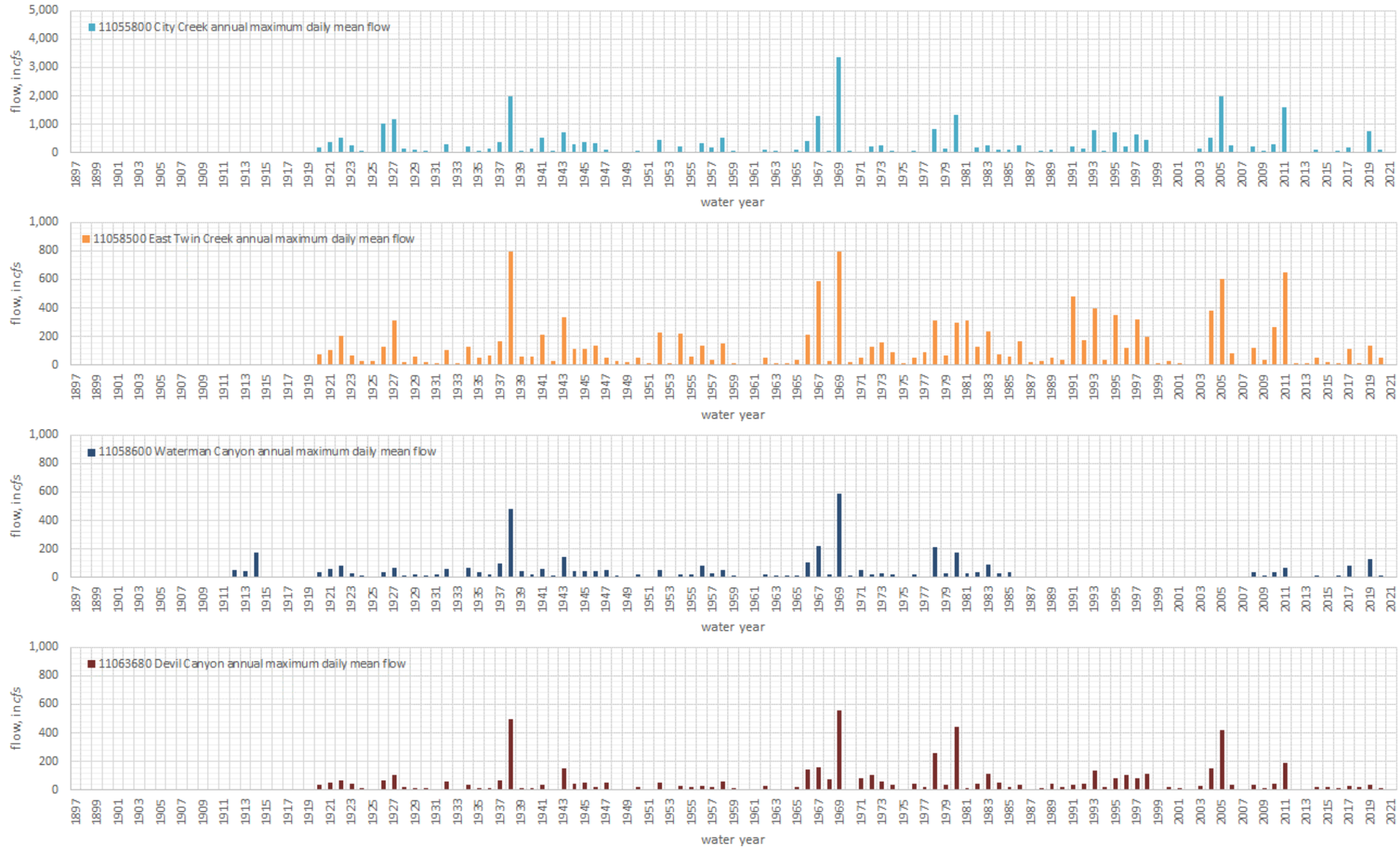
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Figure 1-4. Annual maximum daily mean streamflow (Whitewater, Mill, Santa Ana, and Plunge)



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Figure 1-5. Annual maximum daily mean streamflow (City, East Twin, Waterman, and Devil)



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Table 1-7. Annual maximum peak streamflow unbiased sample *L*-moments

site i	USGS ID	description	n_i	l_1	l_2	$t^{(i)}$	$t_3^{(i)}$	$t_4^{(i)}$
1	10256000	Whitewater River 1	31	3403.19	2890.28	0.8493	0.8063	0.6302
2	11054000	Mill Creek 1	56	1737.66	1384.76	0.7969	0.7389	0.5312
3	11051500	Santa Ana River 1	102	3403.03	2548.68	0.7489	0.7030	0.5409
4	11055500	Plunge Creek	102	777.97	489.07	0.6287	0.5327	0.3219
5	11055800	City Creek	102	1019.89	683.08	0.6698	0.5932	0.3717
6	11058500	East Twin Creek	97	528.34	354.53	0.6710	0.5726	0.3458
7	11058600	Waterman Canyon	82	210.04	128.19	0.6103	0.5603	0.3740
8	11063680	Devil Canyon	96	234.85	168.84	0.7189	0.7103	0.5517
		weighted means:		1.0000		0.6945	0.6326	0.4377

n_i - effective record length of site i

sample L-moments:

l_1 - mean

l_2 - L-scale

L-moment ratios:

t - coefficient of L-variation

t_3 - L-skewness

t_4 - L-kurtosis

Table 1-8. Annual maximum daily mean streamflow unbiased sample *L*-moments

site i	USGS ID	description	n_i	l_1	l_2	$t^{(i)}$	$t_3^{(i)}$	$t_4^{(i)}$
1	10256000	Whitewater River 1	31	409.65	322.21	0.7865	0.8067	0.6656
2	11054000	Mill Creek 1	58	486.03	385.54	0.7932	0.7346	0.5322
3	11051500	Santa Ana River 1	102	1109.20	724.85	0.6535	0.5595	0.3694
4	11055500	Plunge Creek	104	235.09	139.75	0.5945	0.4842	0.2871
5	11055800	City Creek	103	313.60	198.80	0.6339	0.5605	0.3444
6	11058500	East Twin Creek	103	130.99	76.67	0.5853	0.4602	0.2335
7	11058600	Waterman Canyon	84	54.65	31.96	0.5849	0.5656	0.4149
8	11063680	Devil Canyon	101	61.06	37.10	0.6076	0.5735	0.3989
		weighted means:		1.0000		0.6340	0.5621	0.3698

n_i - effective record length of site i

sample L-moments:

l_1 - mean

l_2 - L-scale

L-moment ratios:

t - coefficient of L-variation

t_3 - L-skewness

t_4 - L-kurtosis

1.5 Application of discordancy and heterogeneity measures

Discordancy and heterogeneity measures, as presented by Hosking and Wallis (1997), were applied to the group of sites selected to form a statistically homogeneous region:

- *Discordancy*. It is difficult choosing a discordancy measure, D_i, as a criterion for identifying discordant sites; Hosking and Wallis (1997) recommended that sites with $D_i > 3$ be regarded as discordant.
- *Heterogeneity*. A region is considered heterogeneous if the heterogeneity measure, H, is sufficiently large. Hosking and Wallis (1997) suggested that a region be regarded as "acceptably homogenous" if $H < 1$, "possibly heterogeneous" if $1 \leq H < 2$, and "definitely heterogeneous" if $H \geq 2$.

For the region evaluated herein, a heterogeneity measure less than one was targeted. Discordancy and heterogeneity measures were computed for the group of 8 sites. The discordancy results are listed in Table 1-9 (annual maximum peak streamflow) and Table 1-10 (annual maximum daily mean streamflow) and the heterogeneity statistics are presented in Table 1-11 (annual maximum peak streamflow) and Table 1-12 (annual maximum daily mean streamflow).

Table 1-9. Peak streamflow unbiased sample L-moments and discordancy

site i	USGS ID	description	n_i	$t^{(i)}$	$t_3^{(i)}$	$t_4^{(i)}$	D_i
1	10256000	Whitewater River 1	31	0.8493	0.8063	0.6302	1.13
2	11054000	Mill Creek 1	56	0.7969	0.7389	0.5312	1.03
3	11051500	Santa Ana River 1	102	0.7489	0.7030	0.5409	1.22
4	11055500	Plunge Creek	102	0.6287	0.5327	0.3219	1.00
5	11055800	City Creek	102	0.6698	0.5932	0.3717	0.68
6	11058500	East Twin Creek	97	0.6710	0.5726	0.3458	0.66
7	11058600	Waterman Canyon	82	0.6103	0.5603	0.3740	0.92
8	11063680	Devil Canyon	96	0.7189	0.7103	0.5517	1.38
		weighted means:		0.6945	0.6326	0.4377	

Table 1-10. Daily mean streamflow unbiased sample L-moments and discordancy

site i	USGS ID	description	n_i	$t^{(i)}$	$t_3^{(i)}$	$t_4^{(i)}$	D_i
1	10256000	Whitewater River 1	31	0.7865	0.8067	0.6656	1.41
2	11054000	Mill Creek 1	58	0.7932	0.7346	0.5322	1.32
3	11051500	Santa Ana River 1	102	0.6535	0.5595	0.3694	0.79
4	11055500	Plunge Creek	104	0.5945	0.4842	0.2871	0.86
5	11055800	City Creek	103	0.6339	0.5605	0.3444	1.18
6	11058500	East Twin Creek	103	0.5853	0.4602	0.2335	0.74
7	11058600	Waterman Canyon	84	0.5849	0.5656	0.4149	1.12
8	11063680	Devil Canyon	101	0.6076	0.5735	0.3989	0.58
		weighted means:		0.6340	0.5621	0.3698	

Table 1-11. Peak streamflow heterogeneity measures

H_1	observed standard deviation of group t	0.0651
	simulated mean of standard deviation of group t	0.0523
	simulated standard deviation of standard deviation of group t	0.0163
	standardized test value	0.78
H_2	observed average of t/t_3 distance	0.0968
	simulated mean of average t/t_3 distance	0.0793
	simulated standard deviation of average t/t_3 distance	0.0238
	standardized test value	0.74
H_3	observed average of t_3/t_4 distance	0.1257
	simulated mean of average t_3/t_4 distance	0.1126
	simulated standard deviation of average t_3/t_4 distance	0.0345
	standardized test value	0.38

*Regional kappa distribution parameters: ξ = -0.3713, α = 0..6143, k = -0.4686, and h = 1.8048

Table 1-12. Daily mean streamflow heterogeneity measures

H_1	observed standard deviation of group t	0.0651
	simulated mean of standard deviation of group t	0.0476
	simulated standard deviation of standard deviation of group t	0.0151
	standardized test value	1.16
H_2	observed average of t/t_3 distance	0.0765
	simulated mean of average t/t_3 distance	0.0746
	simulated standard deviation of average t/t_3 distance	0.0224
	standardized test value	0.08
H_3	observed average of t_3/t_4 distance	0.0945
	simulated mean of average t_3/t_4 distance	0.1019
	simulated standard deviation of average t_3/t_4 distance	0.0316
	standardized test value	-0.23

*Regional kappa distribution parameters: ξ = -0.1519, α = 0.6418, k = -0.3956, and h = 1.3321

1.6 L-moment Ratio Diagrams

L-moment ratio diagrams are based on the relationships between *L*-moment ratios, which can be used to identify appropriate distributions. For the region, the sample *L*-moment ratios *L*-skewness, t_3, and *L*-kurtosis, t_4, for each site as well as their regional average are plotted as depicted in Figure 1-6 (annual maximum peak streamflow) and Figure 1-7 (annual maximum daily mean streamflow. A distribution may be considered suitable if the distribution averages the scattered data and the data is spread consistently around the distribution; however, a certain degree of homogeneity must be satisfied in order to obtain a suitable regional distribution. Also, *L*-moment ratios *L*-skewness, t_3, and *L*-CV, *t*, for each site, including their regional average are plotted as shown in Figure 1-8 (annual maximum peak streamflow) and Figure 1-9 (annual maximum daily mean streamflow). The more the data is scattered, the more likely the selected region is heterogeneous. The use of such a test is subjective.

Figure 1-6. peak streamflow *L*-moment ratio diagram: *L*-skewness vs. *L*-kurtosis

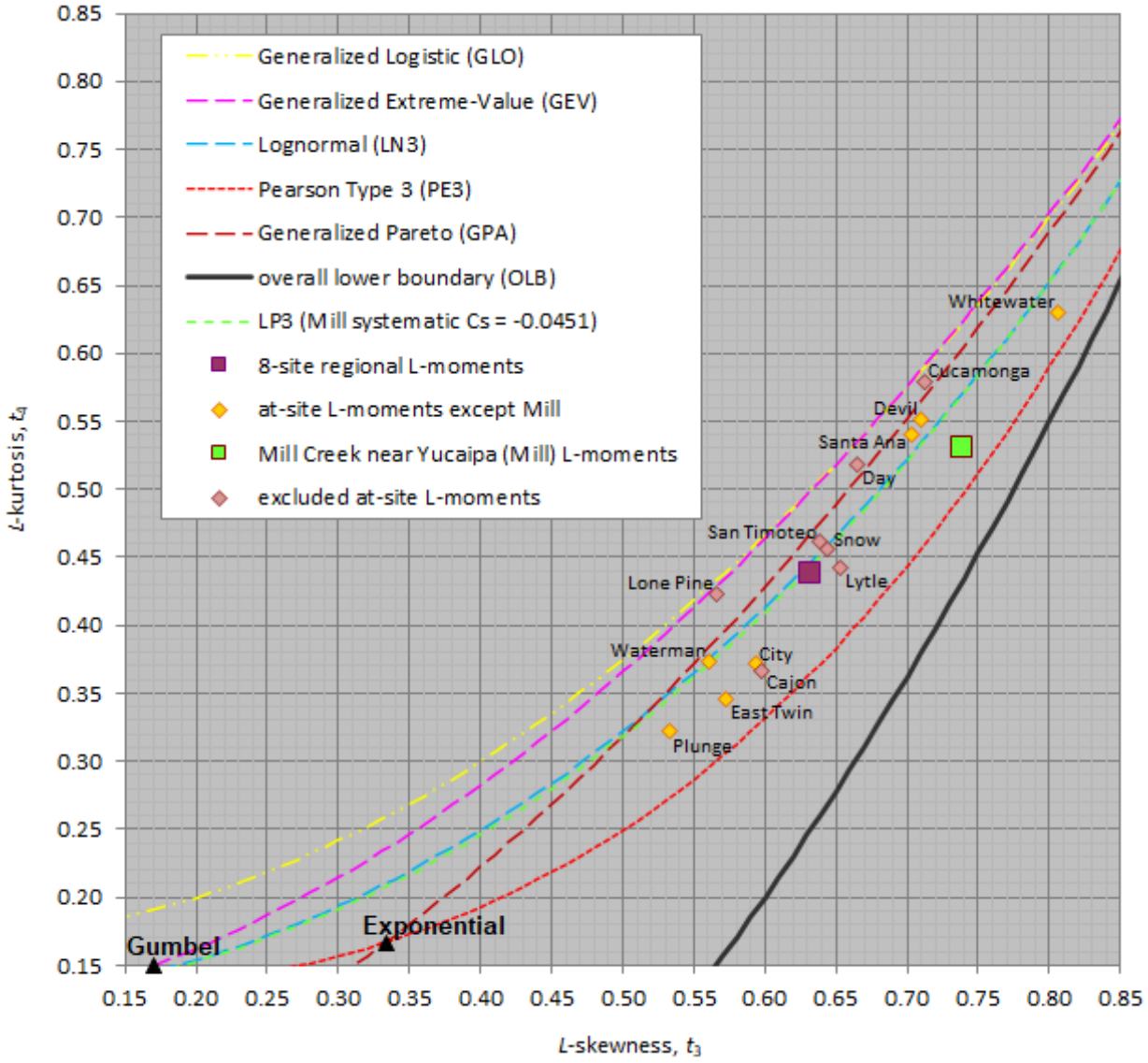


Figure 1-7. daily mean streamflow *L*-moment ratio diagram: *L*-skewness vs. *L*-kurtosis

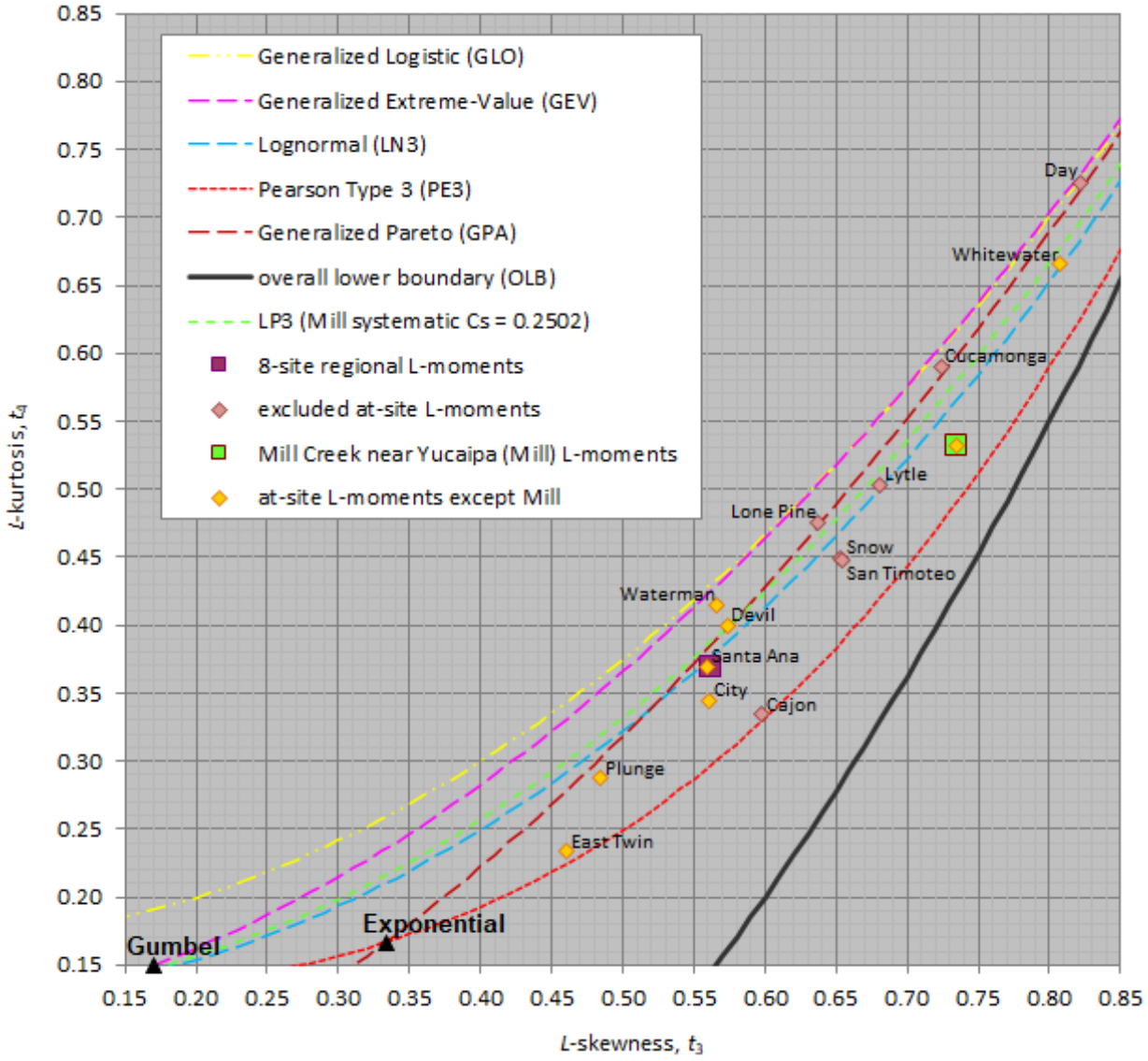


Figure 1-8. peak streamflow *L*-moment ratio diagram: *L*-skewness vs. *L*-CV

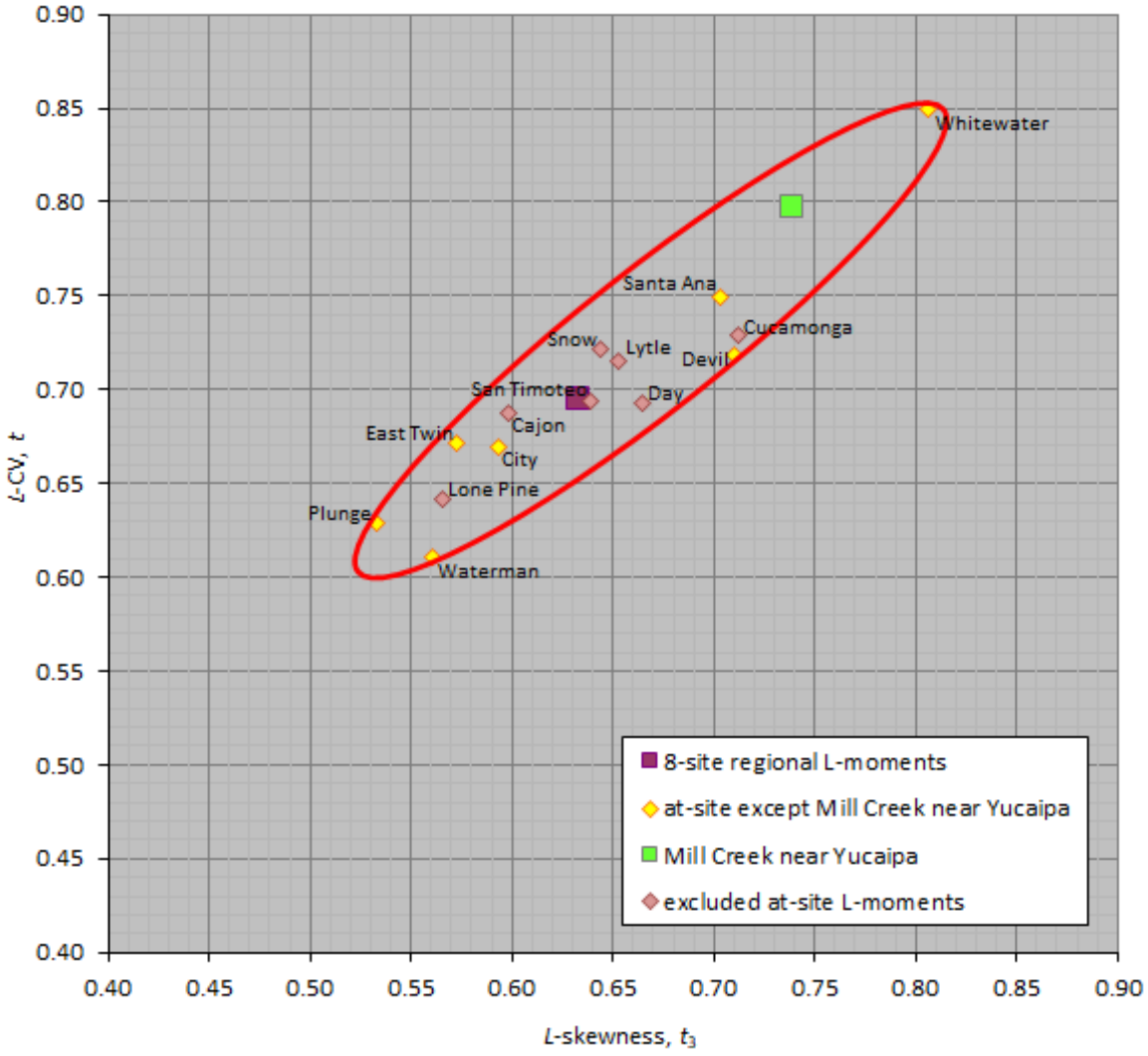
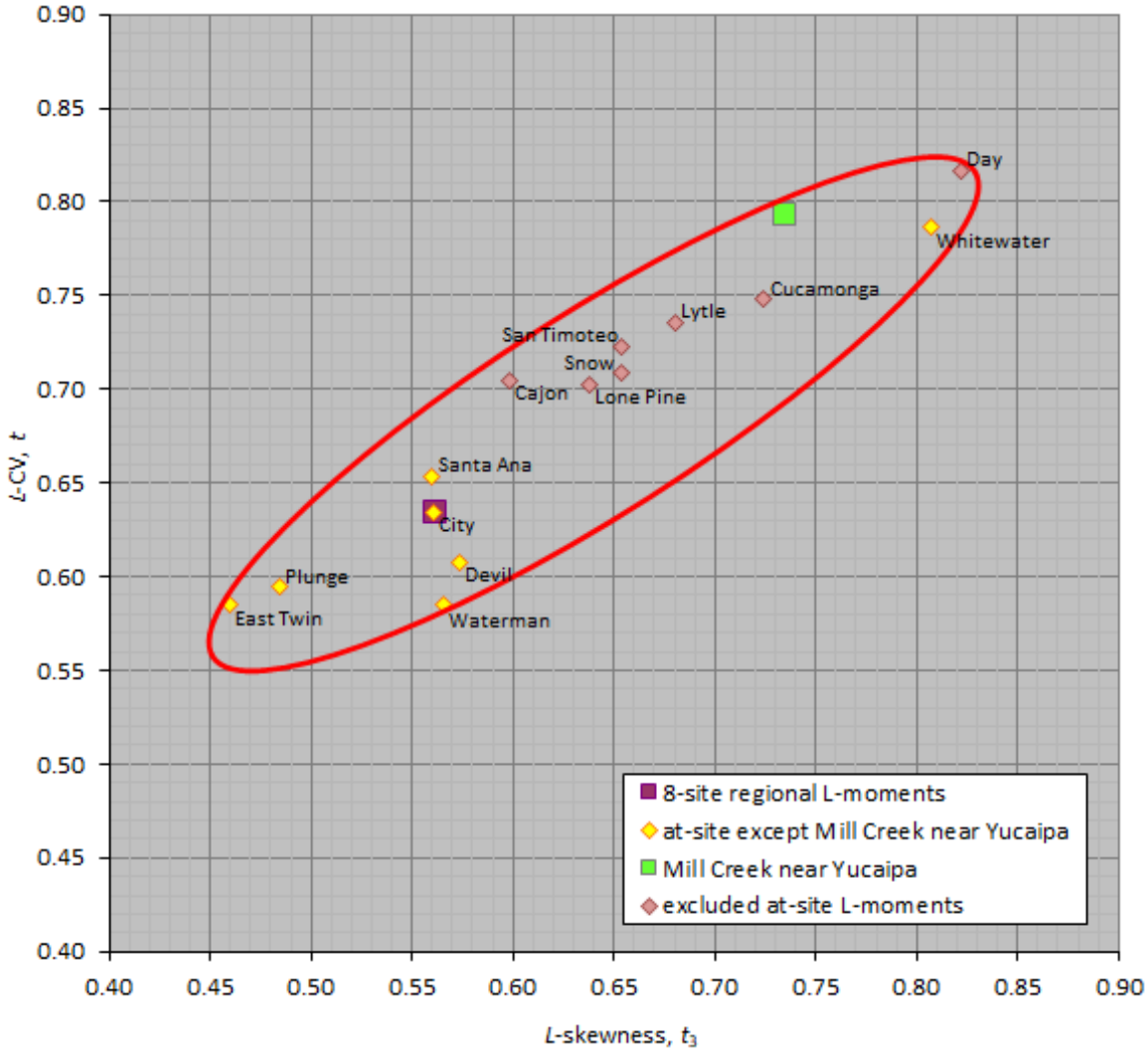


Figure 1-9. daily mean streamflow *L*-moment ratio diagram: *L*-skewness vs. *L*-CV



1.7 Application of the Z^{DIST} goodness-of-fit measure

The Z^{DIST} goodness-of-fit measure was determined by means of Monte Carlo simulations. Data were synthesized from homogeneous regions with one of four 3-parameter frequency distributions: generalized logistic (GLO), generalized extreme-value (GEV), lognormal (LN3), Pearson type 3 (PE3), and generalized Pareto (GPA); 10,000 replications of the region were simulated.

All distributions except Pearson type 3 distributions demonstrate an acceptable fit in accordance with the criteron suggested by Hosking and Wallis (1997), $|Z^{DIST}| \leq 1.64$. The Z^{DIST} results are presented in Table 1-13 (annual maximum peak streamflow) and Table 1-14 (annual maximum daily mean streamflow).

Table 1-13. Peak streamflow Z^{DIST} goodness-of-fit measure test

DIST	*L*-CK	Z^{DIST}	$\leq \lvert 1.64 \rvert$	parameter estimates for distributions accepted at the 90% level		
				location	scale	shape
GLO	0.500	0.52	✔	0.407	0.320	-0.633
GEV	0.499	0.49	✔	0.260	0.348	-0.616
GNO	0.447	-0.69	✔	0.349	0.506	-1.452
PE3	0.365	-2.55		1.000	1.869	4.183
GPA	0.467	-0.23	✔	-0.007	0.453	-0.550

Table 1-14. Daily mean streamflow Z^{DIST} goodness-of-fit measure test

DIST	*L*-CK	Z^{DIST}	$\leq \lvert 1.64 \rvert$	parameter estimates for distributions accepted at the 90% level		
				location	scale	shape
GLO	0.430	0.89	✔	0.499	0.352	-0.562
GEV	0.425	0.77	✔	0.329	0.399	-0.533
GNO	0.377	-0.46	✔	0.448	0.580	-1.254
PE3	0.297	-2.49		1.000	1.560	3.554
GPA	0.385	-0.25	✔	0.010	0.555	-0.439

According to Hosking and Wallis (1997), the criterion $|Z^{DIST}| \leq 1.64$ is somewhat arbitrary; therefore, it should only serve as a rough indicator of goodness-of-fit and is not recommended as a formal test. The *Z* statistic has the form of a significance test of goodness-of-fit and has approximately a standard normal distribution under suitable conditions. This criterion then corresponds to acceptance of the hypothesized distribution at a confidence of 90 percent; however, the assumptions necessary for *Z* to be standard normal include two that are unlikely to be exactly satisfied in practice: (1) the region is exactly homogeneous; and (2) the region has no inter-site dependence.

Furthermore, the criterion $|Z^{DIST}| \leq 1.64$ is particularly unreliable if serial correlation or cross-correlation is present in the data. Correlation tends to increase the variability of $t_4^{\mathbf{R}}$, and because there is no correlation in the simulated kappa region, the resulting estimate of σ_4 is too small; and the *Z* values are too large. Thus, a false indication of poor fit may be given. To overcome this problem, it is possible to emulate data that are correlated via Monte Carlo simulation.

The computed average intersite dependence between each pair of sites is 0.19 (annual maximum peak streamflow) and 0.21 (annual maximum daily mean streamflow) based on the selected 8 sites (Whitewater River, Mill Creek, Santa Ana River, Plunge Creek, City Creek, East Twin Creek, Waterman Canyon, and Devil Canyon).

he heterogeneity measure indicates a high degree of homogeneity while the intersite correlation among the sites is on the lower end of the spectrum. Accordingly, a goodness-of-fit measure based on the simulated population is strongly recommended to attain a more reliable measure of goodness-of-fit because of the ZDIST reliability is lessened with serial/cross-correlation.

1.8 Application of the Regional *L*-moment algorithm

The regional *L*-moment algorithm procedure shown in Table 1-15 was implemented to further assist in determine the most appropriate distribution for regional flood frequency analysis. A total of 1,000 replications of the region were simulated. The resultant goodness-of-fit measures for the simulated populations, based on the 8 selected sites, are summarized in Table 1-16 (annual maximum peak streamflow) and Table 1-17 (annual maximum daily mean flow). The Generalized Normal (GNO) distribution was ultimately chosen, taking into consideration the balance of the assortment of statistical measures evaluated herein.

Table 1-15. Algorithm for simulation of the regional L-moment algorithm

1. Specify the number of sites, N, and for each of the N sites, its record length, n_i, and the L-moments of its frequency distribution.
2. Calculate the parameters of the at-site frequency distributions given their L-moment ratios
3. For each of M repetitions of the simulation procedure, carry out the following steps
   - 3.1. Generate sample data for each site. If there is no intersite dependence, this simply requires the generation of a random sample of size n_i from the frequency distribution for site i, i = 1 to N. If intersite dependence is included in the simulations, the following steps can be used.
     - 3.1.1. Let n_0 = maximum n_i, the largest of the at-site record lengths. For each time point k = 1 to n_0, generate a realization of a random vector $\mathbf{y}_k$ with elements y_{ik}, i = 1 to N, that has a multivariate Normal distribution with mean vector zero and covariance matrix $\mathbf{R}$.
     - 3.1.2. Transform each y_{ik}, k = 1 to n_i, i = 1 to N, to the required marginal distribution, that is, calculate the data values $Q_{ik} = Q_i(\Phi(y_{ik}))$, where Q_i is the quantile function for site i and Φ is the cumulative distribution function of the standard Normal distribution.
   - 3.2. Apply the regional L-moment algorithm to the sample of regional data. This involves the following steps:
     - 3.2.1. Calculate at-site L-moment ratios and regional average L-moment ratios
     - 3.2.2. Fit the chosen distribution
     - 3.2.3. Calculate estimates of the regional growth curve and at-site quantiles
   - 3.3. Calculate the relative error of the estimated regional growth curve and at-site quantiles, and accumulate the sums needed to calculate the overall accuracy measures
4. Calculate overall measures of the accuracy of the estimated quantiles and regional growth curve

Table 1-16. Peak streamflow simulated population goodness-of-fit measures

true distribution		candidate distribution					heterogeneity measure		
		GLO	GEV	GNO	PE3	GPA	H_1	H_2	H_3
GLO	accepted	92.9%	92.1%	39.4%	0.0%	53.0%	1.72	0.95	0.44
	best fit	82.6%	15.4%	1.8%	0.0%	0.2%			
GEV	accepted	92.9%	92.1%	38.0%	0.0%	53.9%	1.65	0.87	0.36
	best fit	83.8%	14.4%	1.6%	0.0%	0.2%			
GNO	accepted	93.4%	96.3%	97.3%	0.6%	99.6%	1.60	0.62	-0.08
	best fit	14.8%	26.3%	17.4%	0.3%	41.2%			
PE3	accepted	0.0%	0.0%	3.8%	99.9%	0.0%	3.52	1.40	-0.05
	best fit	0.0%	0.0%	0.2%	99.8%	0.0%			
GPA	accepted	97.0%	97.9%	85.5%	0.2%	97.6%	1.80	0.92	0.31
	best fit	38.5%	31.4%	4.5%	0.1%	25.5%			

Table 1-17. Daily mean streamflow simulated population goodness-of-fit measures

true distribution		candidate distribution					heterogeneity measure		
		GLO	GEV	GNO	PE3	GPA	H_1	H_2	H_3
GLO	accepted	93.8%	92.5%	44.4%	0.0%	37.0%	1.28	0.74	0.36
	best fit	78.0%	19.7%	2.3%	0.0%	0.0%			
GEV	accepted	96.3%	95.4%	51.0%	0.0%	48.1%	1.22	0.64	0.25
	best fit	74.1%	22.3%	3.6%	0.0%	0.0%			
GNO	accepted	81.6%	88.4%	99.3%	1.9%	99.9%	1.23	0.43	-0.13
	best fit	7.3%	25.8%	17.8%	0.0%	49.1%			
PE3	accepted	0.0%	0.0%	2.0%	100.0%	0.3%	2.60	1.02	-0.09
	best fit	0.0%	0.0%	0.3%	99.7%	0.0%			
GPA	accepted	97.0%	97.9%	85.5%	0.2%	97.6%	1.80	0.92	0.31
	best fit	38.5%	31.4%	4.5%	0.1%	25.5%			

The accuracy of the selected distribution relative to the simulated population of the observed data was evaluated and the confidence limits of the distribution approximated. The results for these statistics are presented in Table 1-18 (annual maximum peak streamflow) and Table 1-19 (annual maximum daily mean streamflow). The estimated quantiles for the selected regional group of 8 sites based on the Generalized Normal distribution are shown in Table 1-20 (annual maximum peak streamflow) and Table 1-21 (annual maximum daily mean streamflow).

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Table 1-18. Peak streamflow simulated population accuracy measures for regional GNO frequency distribution

		nonexceedance probability, *F*								
	statistic	0.5	0.8	0.9	0.95	0.98	0.99	0.995	0.998	0.999
average for all sites {arithmetic}	abs rel bias	0.146	0.039	0.041	0.048	0.055	0.060	0.064	0.071	0.076
	relative bias	0.066	0.039	0.023	0.006	-0.014	-0.028	-0.040	-0.054	-0.064
	RMSE	0.365	0.321	0.323	0.327	0.335	0.342	0.350	0.362	0.371
	0.05 PT.	0.866	0.869	0.848	0.810	0.759	0.726	0.694	0.655	0.631
	0.95 PT.	1.292	1.239	1.235	1.242	1.273	1.285	1.310	1.334	1.352
regional growth curve {harmonic}	abs rel bias	0.151	0.048	0.041	0.047	0.054	0.060	0.065	0.073	0.079
	relative bias	0.078	0.048	0.028	0.010	-0.013	-0.028	-0.042	-0.058	-0.069
	RMSE	0.203	0.076	0.050	0.049	0.071	0.091	0.111	0.137	0.155
	0.05 PT.	0.876	0.947	0.984	0.991	0.922	0.875	0.833	0.784	0.753
	0.95 PT.	1.264	1.137	1.064	1.027	1.056	1.081	1.103	1.132	1.152

Table 1-19. Daily mean streamflow simulated population accuracy measures for regional GNO frequency distribution

		nonexceedance probability, *F*								
	statistic	0.5	0.8	0.9	0.95	0.98	0.99	0.995	0.998	0.999
average for all sites {arithmetic}	abs rel bias	0.087	0.025	0.037	0.046	0.052	0.056	0.059	0.062	0.065
	relative bias	0.032	0.023	0.013	0.002	-0.011	-0.020	-0.028	-0.038	-0.045
	RMSE	0.267	0.244	0.247	0.253	0.262	0.270	0.279	0.290	0.299
	0.05 PT.	0.850	0.867	0.856	0.830	0.793	0.764	0.737	0.703	0.679
	0.95 PT.	1.230	1.190	1.192	1.203	1.225	1.242	1.252	1.274	1.293
regional growth curve {harmonic}	abs rel bias	0.092	0.029	0.036	0.043	0.048	0.053	0.056	0.061	0.065
	relative bias	0.040	0.029	0.017	0.005	-0.009	-0.020	-0.029	-0.040	-0.047
	RMSE	0.138	0.052	0.039	0.048	0.070	0.088	0.104	0.125	0.141
	0.05 PT.	0.888	0.962	0.995	0.972	0.915	0.876	0.840	0.800	0.770
	0.95 PT.	1.196	1.093	1.038	1.034	1.066	1.087	1.107	1.131	1.148

Table 1-20. Annual maximum peak streamflow regional Generalized Normal quantiles, in *cfs*

USGS ID	desciption	n_i	nonexceedance probability, F								
			0.5	0.8	0.9	0.95	0.98	0.99	0.995	0.998	0.999
10256000	Whitewater River	31	1,189	4,026	7,620	12,910	23,368	34,710	49,857	77,323	105,195
11054000	Mill Creek, regional	56	607	2,055	3,891	6,592	11,932	17,723	25,457	39,481	53,713
	Mill Creek, systematic	56	344	1,568	3,459	6,648	13,865	22,632	35,438	61,020	89,334
11051500	Santa Ana River	102	1,189	4,025	7,620	12,909	23,367	34,709	49,854	77,319	105,190
11055500	Plunge Creek	102	272	920	1,742	2,951	5,342	7,935	11,397	17,676	24,048
11055800	City Creek	102	356	1,206	2,284	3,869	7,003	10,402	14,941	23,173	31,526
11058500	East Twin Creek	97	185	625	1,183	2,004	3,628	5,389	7,740	12,004	16,331
11058600	Waterman Canyon	82	73	248	470	797	1,442	2,142	3,077	4,772	6,492
11063680	Devil Canyon	96	82	278	526	891	1,613	2,395	3,441	5,336	7,260

Table 1-21. Annual maximum daily mean streamflow regional Generalized Normal quantiles, in *cfs*

USGS ID	desciption	n_i	nonexceedance probability, F								
			0.5	0.8	0.9	0.95	0.98	0.99	0.995	0.998	0.999
10256000	Whitewater River	31	183	538	939	1,484	2,481	3,495	4,780	6,986	9,115
11054000	Mill Creek, regional	58	218	639	1,114	1,760	2,944	4,146	5,672	8,288	10,815
	Mill Creek, systematic	58	99	445	975	1,864	3,863	6,280	9,796	16,789	24,500
11051500	Santa Ana River	102	496	1,457	2,542	4,018	6,719	9,463	12,943	18,915	24,680
11055500	Plunge Creek	104	105	309	539	852	1,424	2,006	2,743	4,009	5,231
11055800	City Creek	103	140	412	719	1,136	1,900	2,675	3,659	5,348	6,978
11058500	East Twin Creek	103	59	172	300	474	793	1,117	1,529	2,234	2,915
11058600	Waterman Canyon	84	24	72	125	198	331	466	638	932	1,216
11063680	Devil Canyon	101	27	80	140	221	370	521	713	1,041	1,359

1.9 Adopted regional flood frequency curve based on *L*-moments

The regional quantiles and corresponding confidence limits for the Mill Creek streamflow gauge are shown in Table 1-22 (annual maximum peak streamflow) and Table 1-23 (annual maximum daily mean streamflow). The accompanying regional flood frequency curve, confidence limits, and plotting positions are presented in Figure 1-10 (annual maximum peak streamflow) and Figure 1-11 (annual maximum daily mean streamflow). A plotting position is a distribution-free estimator. Calculations specified in Bulletin 17C (USGS, 2019) do not require designation of plotting positions; however, they do provide a non-probability-based graphical depiction of the sample data for a given site, which may be useful in evaluating historical events as well as the relative positioning of all data with respect to the selected distribution. A general formula for computing plotting positions is

$$p_{j:n} = (j + \gamma) / (n + \delta) \text{ for } \delta > \gamma > -1,$$

where j equals the ordered sequence of flood values with the largest assigned a value of one; and n equal to the size of the sample data set with γ and δ dependent upon the distribution. The Weibull plotting position formula was used herein assuming $\gamma = 0.0$ and $\delta = 1.0$.

The plotting positions reflect streamflow from the original record as well as flow equivalents translated from other sites in the adopted hydrologic regions using the empirical streamflow relationships derived in Section 1.3.

Table 1-22. Mill Creek annual maximum peak streamflow regional quantiles, in *cfs*

	nonexceedance probability, F								
parameter	0.5	0.8	0.9	0.95	0.98	0.99	0.995	0.998	0.999
$q(F)$	0.35	1.18	2.24	3.79	6.87	10.20	14.65	22.72	30.91
$Q(F)$	608	2,050	3,892	6,586	11,938	17,724	25,457	39,480	53,711
05%	0.866	0.869	0.848	0.810	0.759	0.726	0.694	0.655	0.631
95%	1.292	1.239	1.235	1.242	1.273	1.285	1.310	1.334	1.352
$Q(F)_{05\%}$	527	1,782	3,301	5,334	9,061	12,868	17,667	25,859	33,892
$Q(F)_{95\%}$	786	2,540	4,807	8,179	15,197	22,776	33,348	52,666	72,617

Table 1-23. Mill Creek annual maximum daily mean streamflow regional quantiles, in *cfs*

	nonexceedance probability, F								
parameter	0.5	0.8	0.9	0.95	0.98	0.99	0.995	0.998	0.999
$q(F)$	0.209	1.452	2.930	4.686	7.260	9.330	11.481	14.414	16.684
$Q(F)$	218	639	1,114	1,760	2,944	4,146	5,672	8,288	10,815
5%	0.850	0.867	0.856	0.830	0.793	0.764	0.737	0.703	0.679
95%	1.230	1.190	1.192	1.203	1.225	1.242	1.252	1.274	1.293
$Q(F)_{5\%}$	185	554	953	1,461	2,335	3,168	4,180	5,827	7,343
$Q(F)_{95\%}$	268	760	1,328	2,118	3,607	5,150	7,101	10,559	13,983

Figure 1-10. Mill Creek annual maximum peak streamflow regional flood frequency curve

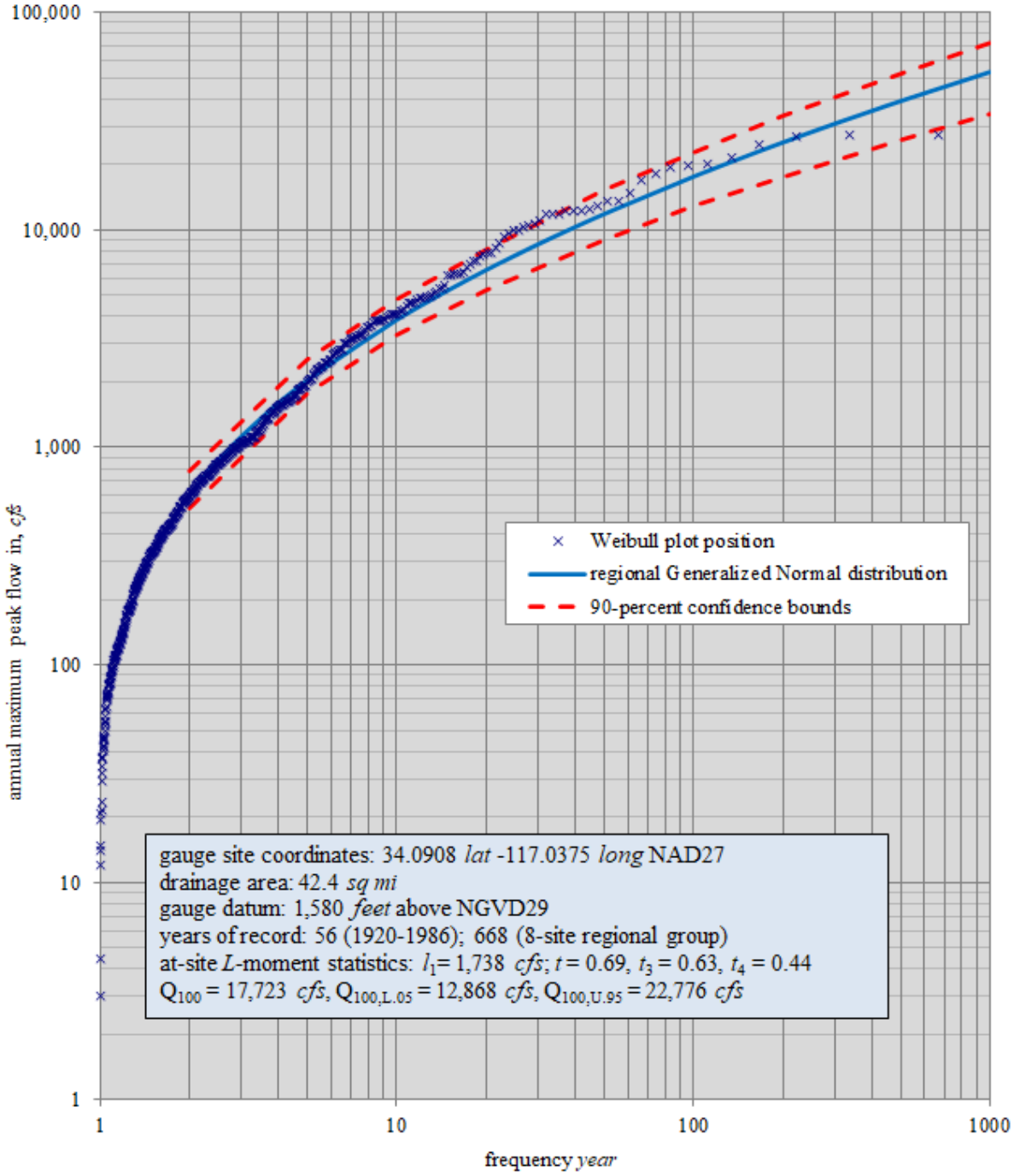
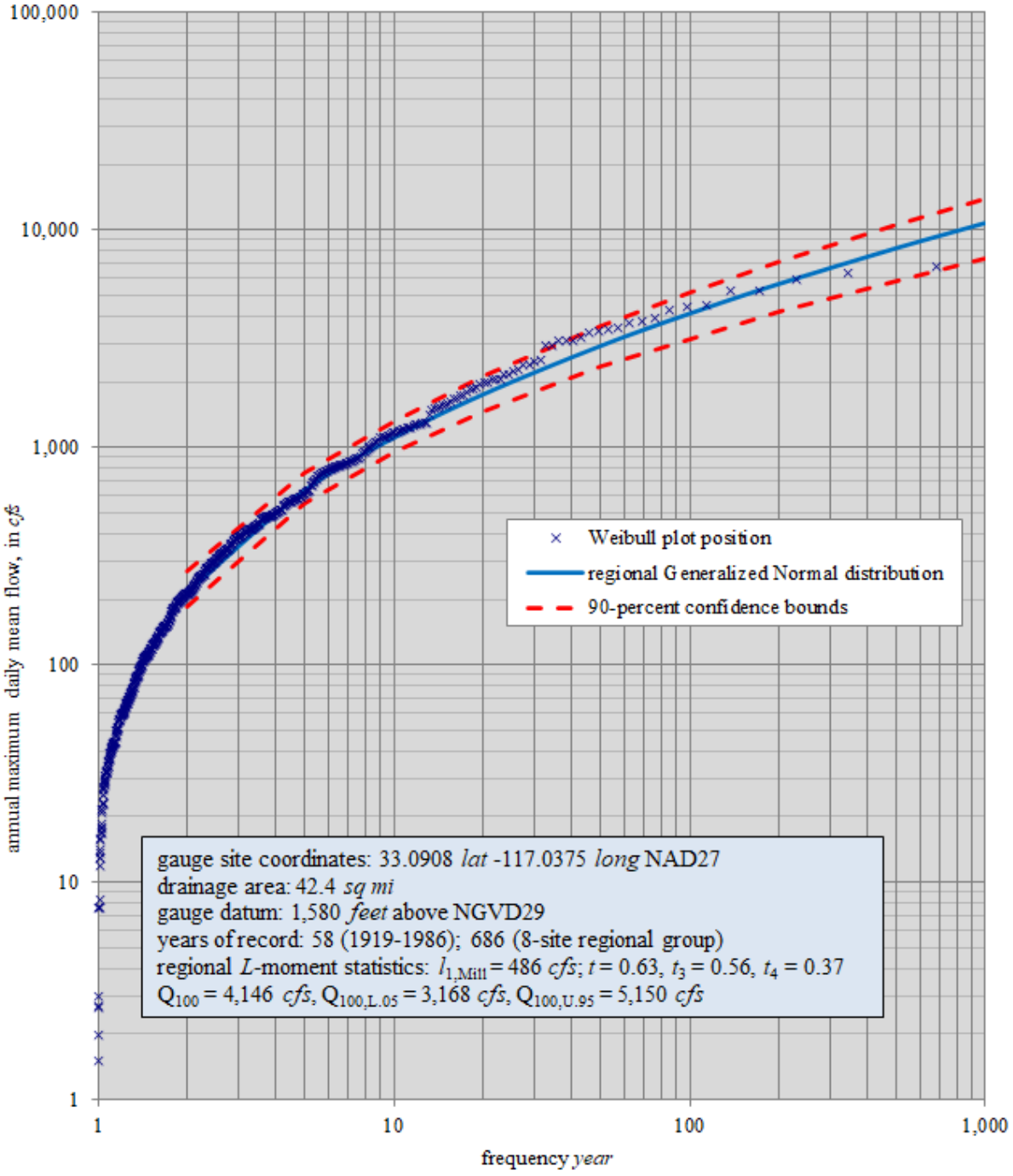


Figure 1-11. Mill Creek annual maximum daily mean streamflow regional flood frequency curve



1.10 Mill Creek predictive streamflow summary statistics

The annual maximum peak and mean daily streamflow regional quantiles were used to develop empirical streamflow relationships between Mill Creek and each of the remaining sites that are part adopted hydrologic region.

These derived streamflow relationships are presented in Figure 1-12 (Mill Creek annual maximum peak streamflow relationships) and Figure 1-13 (Mill Creek annual maximum daily mean streamflow relationships (Figure 1-13).

These paired relationships were used to translate streamflow data points from other gauged streamflow gauges located within the same adopted hydrologic region into streamflow equivalents for Mill Creek. The streamflow equivalents were used to synthesize predictive streamflow records for Mill Creek, including the average, minimum, and maximum predictive translations of the adopted hydrologic region.

The resulting statistics of this synthesis are summarized in Table 1-24 (Mill Creek maximum daily mean streamflow statistics), Table 1-25 (Mill Creek average daily mean flow), and Table 1-26 (Mill Creek average flood volume statistics).

It is recognized herein that daily mean flow is analogous to a 24-hour runoff volume. The synthesized maximum daily mean streamflow (minimum, average, and maximum) as well as the Mill Creek original and augmented records are depicted in Figure 1-14.

The augmented record utilized the synthesized average record to fill data gaps and extend the original record. Similarly, the synthesized average daily mean streamflow is presented in Figure 1-15.

1.11 Mill Creek average predictive daily mean flow average annual exceedances

The average predictive daily mean flow duration curves for selected time periods are presented in Figure 1-16, which was subsequently used to approximate the average annual exceedances for selected time periods listed in Table 1-27 (less than one occurrence on an average annual basis), Table 1-28 (1 to 20 occurrences on an average annual basis), and Table 1-29 (21 to 40 occurrence on an average annual basis). To demonstrate the use of these tables, the South Intake structure was considered, which has a maximum flow rate capacity of 162 *cfs*. For the period covered by the wet season (December through May), Table 1-29 shows an average annual exceedance of 31 occurrences per year for a flow rate of 162 *cfs*.

Figure 1-12. Mill Creek annual maximum peak streamflow regional flood frequency relationships

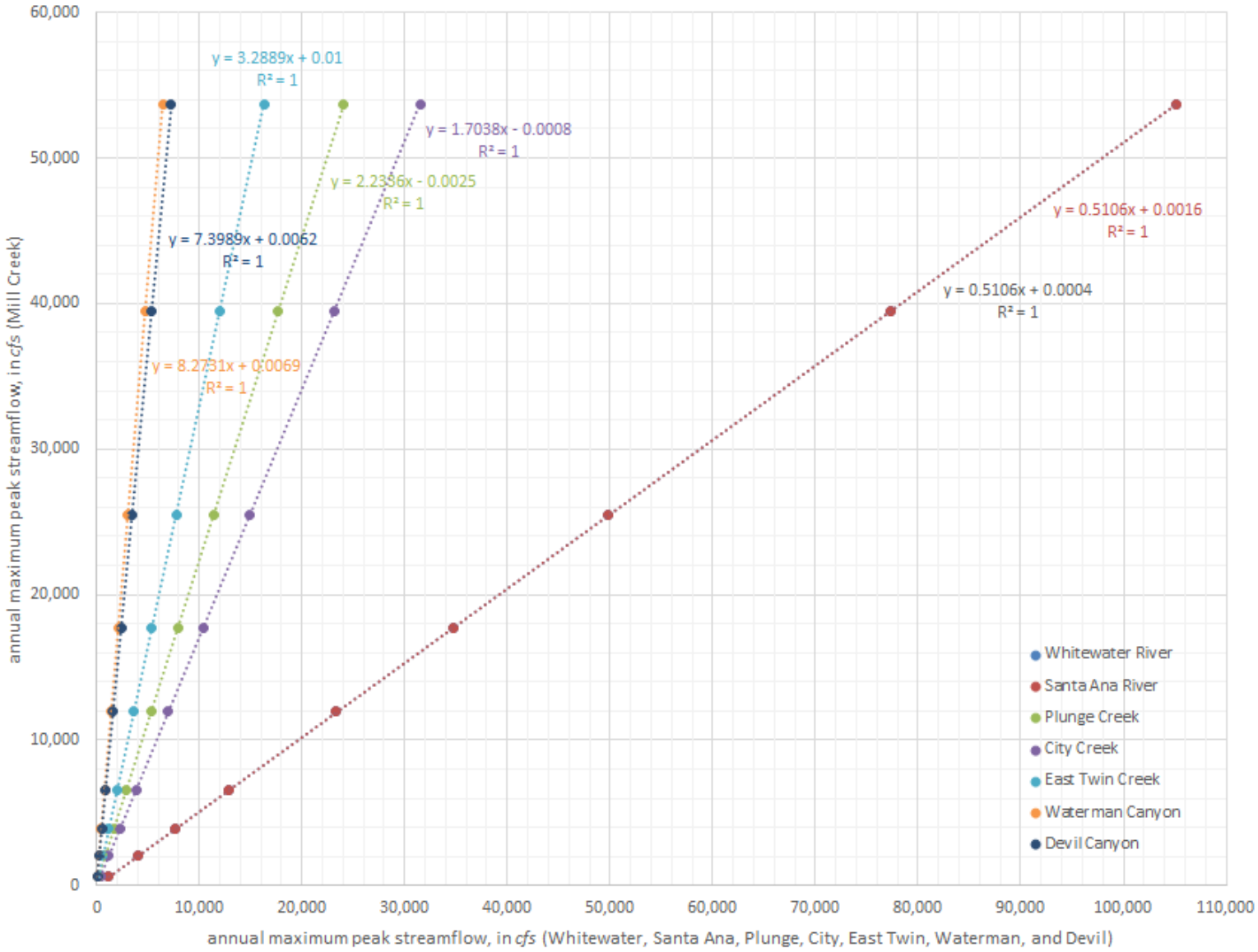


Figure 1-13. Mill Creek annual maximum daily mean streamflow regional flood frequency relationships

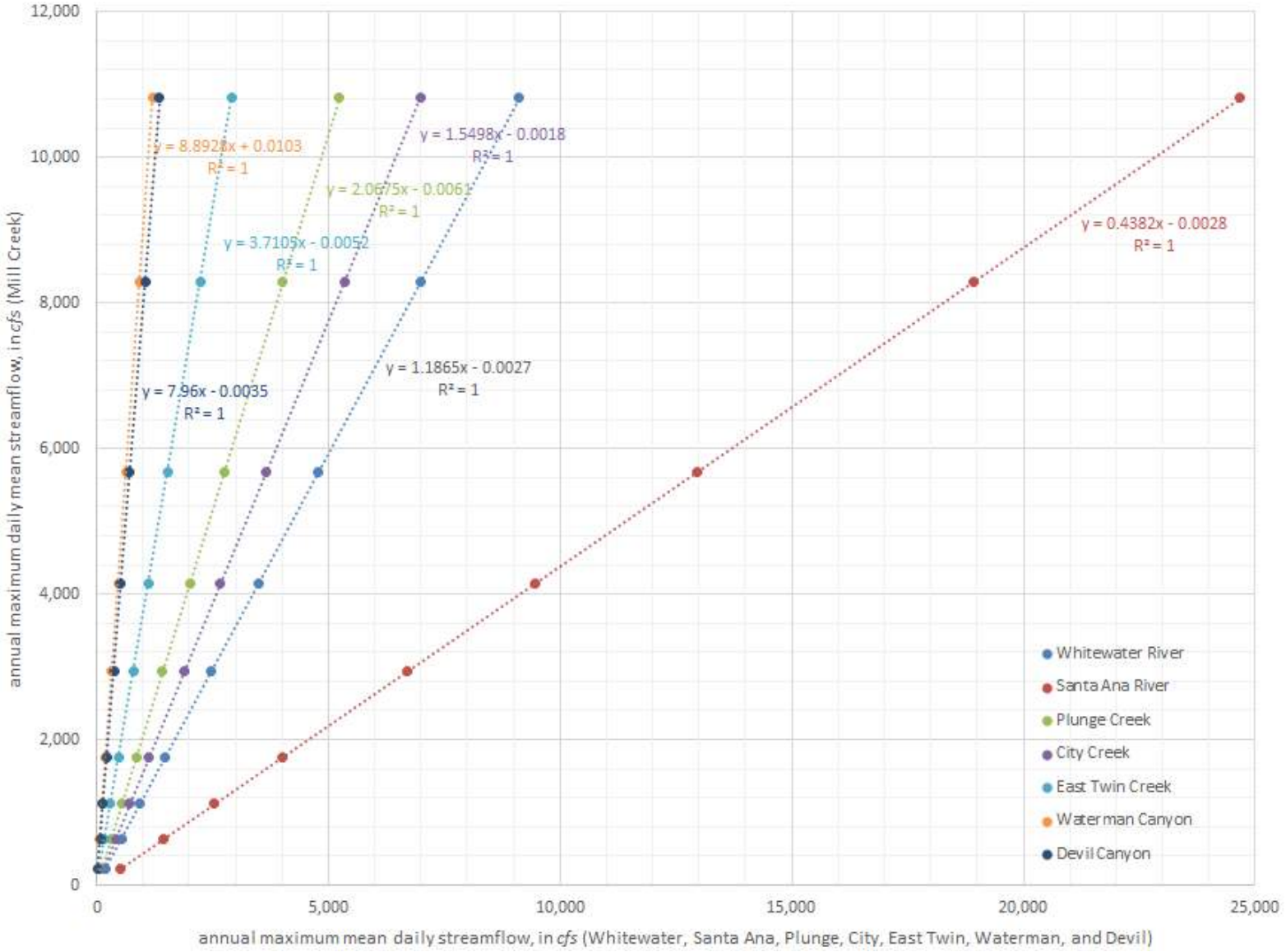


Table 1-24. Mill Creek maximum daily mean streamflow statistics

period statistc		daily mean flow, in *cfs*				
			predictive record			
		Mill Creek original record	minimum	average	average augmented	maximum
decade maximum	1897-1900	-	272	272	272	272
	1901-1910	-	3,725	3,725	3,725	3,725
	1911-1920	236	1,740	1,740	1,740	1,740
	1921-1930	1,680	375	1,214	1,680	1,995
	1931-1940	6,300	2,939	4,344	6,300	6,792
	1941-1950	50	758	1,085	1,085	1,263
	1951-1960	221	149	498	221	955
	1961-1970	4,510	2,004	3,781	4,510	5,897
	1971-1980	1,740	914	1,450	1,740	3,510
	1981-1990	421	279	584	421	1,150
	1991-2000	-	1,052	1,227	1,227	1,777
	2001-2010	-	259	1,877	1,877	3,343
	2011-2020	-	95	1,504	1,504	2,464
	2021-2021	-	15	43	43	79
monthly maximum	October	138	114	381	381	637
	November	2,290	684	1,228	2,290	3,560
	December	3,200	1,290	2,283	3,200	3,411
	January	4,510	3,725	3,781	4,510	5,897
	February	1,990	1,972	3,207	1,990	5,247
	March	6,300	2,939	4,344	6,300	6,792
	April	760	2,152	2,152	2,152	2,152
	May	480	235	283	480	500
	June	450	113	125	450	450
	July	139	49	61	139	314
	August	115	88	125	115	297
	September	793	141	287	793	793
record maximum		6,300	3,725	4,344	6,300	6,792

Table 1-25. Mill Creek average daily mean streamflow statistics

period statistc		daily mean flow, in *cfs*				
			predictive record			
		Mill Creek original record	minimum	average	average augmented	maximum
decade average annual	1897-1900	-	18	18	18	18
	1901-1910	-	27	27	27	27
	1911-1920	10	19	26	26	33
	1921-1930	12	5	15	12	32
	1931-1940	21	6	17	19	34
	1941-1950	0	4	14	13	29
	1951-1960	5	1	10	5	25
	1961-1970	23	6	21	23	48
	1971-1980	27	8	26	27	58
	1981-1990	14	4	14	10	29
	1991-2000	-	14	26	26	45
	2001-2010	-	5	18	18	35
	2011-2020	-	4	15	15	29
	2021-2021	-	2	6	6	14
average monthly	October	1	2	5	3	11
	November	3	3	7	6	16
	December	8	4	14	11	27
	January	12	11	27	23	47
	February	23	20	43	34	75
	March	37	28	51	46	82
	April	32	18	34	34	58
	May	39	10	21	30	41
	June	23	5	12	18	25
	July	8	3	7	8	15
	August	4	2	5	5	11
	September	2	1	4	3	10
record average annual		16	9	19	18	35

Table 1-26. Mill Creek average flood volume statistics

period statistc		volume, in *acre-feet*				
			predictive record			
		Mill Creek original record	minimum	average	average augmented	maximum
decade average annual	1897-1900	-	12,859	12,859	12,859	12,859
	1901-1910	-	19,749	19,749	19,749	19,749
	1911-1920	7,094	14,059	18,892	18,607	23,951
	1921-1930	9,043	3,445	11,152	9,043	23,245
	1931-1940	14,962	4,639	12,440	13,536	24,274
	1941-1950	304	3,083	10,465	9,523	21,079
	1951-1960	3,692	851	7,165	3,692	18,470
	1961-1970	16,932	4,676	15,575	16,932	34,892
	1971-1980	19,661	5,807	18,706	19,661	41,783
	1981-1990	9,924	2,546	9,920	7,304	20,839
	1991-2000	-	10,222	19,145	19,145	32,830
	2001-2010	-	3,387	12,981	12,981	25,037
	2011-2020	-	3,036	10,711	10,711	20,980
	2021-2021	-	1,607	4,399	4,399	9,789
average monthly	October	51	108	285	212	669
	November	208	159	442	343	966
	December	509	257	873	682	1,671
	January	747	678	1,686	1,412	2,898
	February	1,302	1,130	2,434	1,912	4,198
	March	2,299	1,729	3,126	2,850	5,034
	April	1,893	1,070	2,011	2,020	3,452
	May	2,373	640	1,299	1,829	2,523
	June	1,383	303	697	1,048	1,476
	July	498	177	426	503	925
	August	258	107	282	295	685
	September	133	83	230	199	574
record average annual		11,653	6,443	13,791	13,305	25,072

Figure 1-14. Synthesized annual maximum daily mean streamflow (Mill original, minimum, average, Mill augmented, and maximum)



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Figure 1-15. Synthesized annual average daily mean streamflow (Mill original, minimum, average, Mill augmented, and maximum)



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Figure 1-16. Mill Creek regionally-based average predictive daily mean flow duration curves at Garnet Street

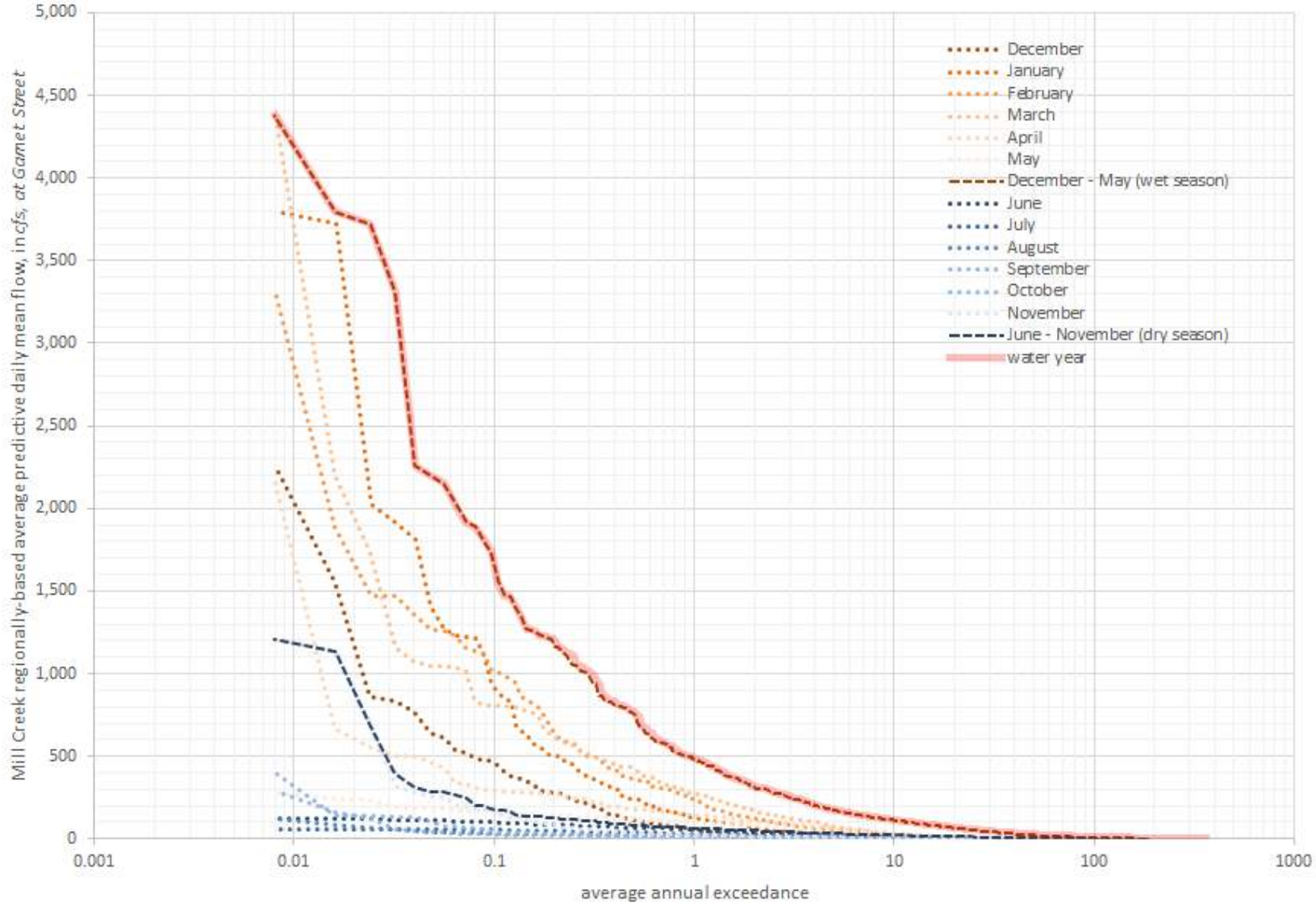


Table 1-27. Mill Creek average annual exceedance (0.01 to 0.95) daily mean flows for selected time periods at Garnet Street

average annual exceedance	Mill Creek regionally-based average predictive daily mean flow, in *cfs*, at Garnet Street														
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Dec-May	Jun-Nov	water year
0.01	333	1192	2083	3779	2962	3835	1781	270	125	61	116	258	4235	129	4236
0.02	139	910	1210	2915	1685	1970	614	239	122	60	80	142	3760	128	3760
0.05	102	259	632	1378	1263	1050	455	187	112	59	41	43	2189	124	2189
0.1	47	162	457	919	1012	805	289	165	100	55	35	27	1652	118	1652
0.2	33	83	280	502	643	613	274	142	86	50	31	24	1163	107	1210
0.25	28	66	234	437	578	562	246	137	82	44	29	24	1050	102	1103
0.3	26	57	203	371	501	512	231	133	76	44	27	23	982	97	1013
0.4	24	45	148	305	415	456	199	124	72	40	26	22	816	89	831
0.5	23	38	118	242	366	417	179	120	69	38	25	22	751	83	768
0.6	23	32	95	204	320	376	170	114	66	37	24	21	623	76	644
0.7	22	30	81	179	306	339	165	110	64	35	24	21	580	73	584
0.75	22	28	77	169	304	324	159	108	63	35	23	21	559	71	568
0.8	22	27	74	159	289	311	157	106	62	34	23	20	529	69	540
0.9	21	25	64	145	262	298	150	104	59	32	22	20	502	68	508
0.95	21	24	59	136	246	281	147	102	57	31	22	20	496	67	500
minimum	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
average	5	8	14	28	44	51	34	21	12	7	5	4	32	7	19
maximum	396	1211	2260	3796	3313	4380	2152	279	126	61	126	288	4380	1211	4380

Table 1-28. Mill Creek average annual exceedance (1 to 20 occurrences) daily mean flows for selected time periods at Garnet Street

average annual exceedance	Mill Creek regionally-based average predictive daily mean flow, in *cfs*, at Garnet Street														
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Dec-May	Jun-Nov	water year
1	21	23	57	130	238	279	143	101	55	31	22	19	483	66	489
2	15	16	28	69	125	172	108	82	42	25	17	11	315	51	318
3	11	13	19	45	82	126	91	60	34	21	13	10	251	42	257
4	9	12	16	35	63	104	75	50	28	18	11	8	205	37	208
5	8	10	14	25	50	77	64	42	23	16	9	7	173	33	176
6	7	9	13	21	43	63	56	37	19	13	8	6	154	30	157
7	6	8	12	18	35	52	46	30	15	10	6	5	140	28	141
8	5	7	11	15	30	44	38	23	13	9	5	4	129	26	131
9	5	6	10	14	26	36	31	19	11	6	4	4	120	24	122
10	4	6	10	13	21	31	27	17	9	5	4	3	113	23	114
11	4	5	9	12	18	28	23	15	8	4	3	3	106	22	108
12	3	5	8	11	16	24	19	13	7	4	3	3	101	21	103
13	3	4	8	10	14	21	17	11	6	4	3	2	94	21	96
14	3	4	7	10	13	18	15	10	5	3	2	2	89	20	91
15	2	4	6	9	12	17	14	9	4	3	2	2	85	19	87
16	2	3	6	8	11	15	13	8	4	3	2	2	81	18	83
17	2	3	5	8	10	14	12	7	4	2	2	2	76	17	79
18	2	3	5	7	10	12	11	7	3	2	2	1	73	17	75
19	2	2	4	7	9	12	10	6	3	2	1	1	69	16	72
20	2	2	4	6	8	11	9	6	3	2	1	1	66	15	69

Table 1-29. Mill Creek average annual exceedance (21 to 40 occurrences) daily mean flows for selected time periods at Garnet Street

average annual exceedance	Mill Creek regionally-based average predictive daily mean flow, in *cfs*, at Garnet Street														
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Dec-May	Jun-Nov	water year
21	1	2	3	6	7	10	8	5	3	2	1	1	63	15	66
22	1	2	3	5	7	9	8	5	2	1	1	1	60	14	64
23	1	2	3	5	6	8	7	4	2	1	1	1	58	14	61
24	1	1	2	4	5	7	6	4	2	1	1	1	56	13	59
25	1	1	2	4	4	6	5	3	2	1	1	1	53	13	57
26	1	1	2	3	3	6	4	3	1	1	1	0	52	12	55
27	1	1	2	2	2	5	4	3	1	1	0	0	49	12	53
28	0	1	1	2	1	4	3	2	1	1	0	0	47	11	51
29	0	0	1	1	0	3	2	2	0	0	0	0	46	11	50
30	0	0	1	1	0	2	0	1	0	0	0	0	44	11	48
31	0	0	0	0	0	0	0	0	0	0	0	0	43	10	46
32	0	0	0	0	0	0	0	0	0	0	0	0	41	10	45
33	0	0	0	0	0	0	0	0	0	0	0	0	40	10	44
34	0	0	0	0	0	0	0	0	0	0	0	0	38	10	43
35	0	0	0	0	0	0	0	0	0	0	0	0	37	9	41
36	0	0	0	0	0	0	0	0	0	0	0	0	36	9	40
37	0	0	0	0	0	0	0	0	0	0	0	0	35	9	39
38	0	0	0	0	0	0	0	0	0	0	0	0	33	9	38
39	0	0	0	0	0	0	0	0	0	0	0	0	32	8	37
40	0	0	0	0	0	0	0	0	0	0	0	0	31	8	36

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ATTACHMENT IV

Mill Creek Basin Hydrogeology Technical Memorandum

GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

Prepared for:
Q^3 Consulting
Engineering Feasibility Study Support Services for
San Bernardino Valley Water Conservation District

August 1, 2025

Prepared by:
GEOSCIENCE
The First Name in Groundwater

THIS REPORT IS RENDERED TO Q^3 CONSULTING IN PERFORMANCE OF ITS SERVICES FOR SAN BERNARDINO VALLEY WATER CONSERVATION DISTRICT AS OF THE DATE HEREOF, SOLELY FOR THEIR BENEFIT IN CONNECTION WITH ITS STATED PURPOSE AND MAY NOT BE RELIED ON BY ANY OTHER PERSON OR ENTITY OR BY THEM IN ANY OTHER CONTEXT. ALL CALCULATIONS WERE PERFORMED USING ACCEPTED PROFESSIONAL STANDARDS.

AS DATA IS UPDATED FROM TIME TO TIME, ANY RELIANCE ON THIS REPORT AT A FUTURE DATE SHOULD TAKE INTO ACCOUNT UPDATED DATA.



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Groundwater Modeling for Mill Creek Diversion and Basins Project Feasibility Study

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Acronyms, Abbreviations, and Initialisms

Abbrev.	Description
acre-ft/yr	acre-feet per year
CHD	constant head boundary
cfs	cubic feet per second
Conservation District	San Bernardino Valley Water Conservation District
EPA	Environmental Protection Agency
FIRO	Forecast Informed Reservoir Operations
ft	foot, or feet
Geoscience	Geoscience Support Services, Inc.
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System (by USACE)
HSPF	Hydrologic Simulation Program - Fortran
ISARM	Integrated Santa Ana River Model
PERC	Program for Enhanced Recharge Capacity
Q^3	Q^3 Consulting
RIX	San Bernardino/Colton Rapid Infiltration and Extraction Facility
RMSE	root mean square error
SAR	Santa Ana River
SBBA	San Bernardino Basin Area
SWM	Stanford Watershed Model
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey

Groundwater Modeling for Mill Creek Diversion and Basins Project Feasibility Study

1.0 Introduction

1.1 Background

San Bernardino Valley Water Conservation District (Conservation District, or SBVWCD) is investigating the feasibility of enhancing groundwater recharge through diversion and spreading of additional stormwater runoff in basins adjacent to Mill Creek in the San Bernardino Basin Area (SBBA). This project location is part of the regional Program for Enhanced Recharge Capacity (PERC) being conducted by and for the regional water suppliers and water users. Figure 1 depicts where this project is located in relation to an existing regional model described below and in relation to the focused groundwater model used in this work.

In support of this effort, the Conservation District hired Q^3 Consulting (Q^3) to provide the services requested in their Request for Proposal (RFP) and Scope of Work (SOW). Geoscience Support Services, Inc. (Geoscience), as part of Q^3's team, was tasked with evaluating potential recharge and effects on groundwater elevations based on calculations of potential stormwater inflow and basin infiltration rates. This technical memorandum presents results of groundwater modeling conducted for the Mill Creek Basins under existing conditions and different operational conditions.

1.2 Methodology

Stormwater from Mill Creek, Santa Ana River, and imported water from the State Water Project recharged at the proposed expansion of the Mill Creek Basins will create additional groundwater mounding in the vicinity of the basins. As such, potential impacts from the proposed project expansion on stability of site features and adjacent property structures must be evaluated. Groundwater modeling provides useful information for this type of evaluation and was applied to this feasibility study to understand the amount of groundwater recharge achievable at Mill Creek Basins location (i.e., accounting for rejected recharge) and to identify areas of shallow groundwater (i.e., less than 50 feet) caused by mounding.

Geoscience leveraged an existing regional groundwater flow model covering the project area, known as the Integrated Santa Ana River Model (ISARM), to construct a focused model for the Mill Creek Basins area. Model scenarios were then run under the Existing Conditions and two different proposed basin operational scenarios including FIRO Scenario (expanded basins with FIRO flowrates) and Stress Scenario (expanded and new proposed basins with optimized Stress flowrates), to evaluate realized recharge and resultant groundwater elevations. Input for the focused groundwater model scenarios incorporated estimates of Mill Creek stormflow and infiltration rates for Mill Creek Basins, as calculated by other project team members. The following sections describe the existing regional model, development of the Mill Creek Focused Groundwater Model, scenario assumptions, and scenario results.

2.0 Integrated Santa Ana River Model (ISARM)

The ISARM was developed by Geoscience in 2020 as part of a cooperative technical effort involving the San Bernardino Valley Municipal Water District (Valley District), Western Municipal Water District, Inland Empire Utilities Agency, Orange County Water District, City of Riverside Public Utilities, United States Geological Survey (USGS), United States Fish and Wildlife Service, and the California Department of Fish and Wildlife (Geoscience, 2020).

The model represents the integration and recalibration of existing, individual groundwater flow models and surface water models into a regional model that covers the Upper Santa Ana Valley Groundwater Basin. The ISARM published in 2020 simulates conditions from 1966 to 2016 using best available information at that time. The purpose of the model was to determine what factors contribute to reduced streamflow in the Santa Ana River (SAR) and to evaluate potential effects from proposed projects on streamflow and groundwater levels across the basin, including Upper SAR Habitat Conservation Plan "Covered Activities." Development and calibration of the ISARM is discussed in detail in Geoscience, 2020, and summarized briefly in the following subsections.

2.1 Conceptual Model

A conceptual model is the basis for building the structure of a groundwater model so that it best represents the hydrogeologic system. The ISARM was developed to simulate groundwater flow in the Upper Santa Ana Valley Groundwater Basin, including Yucaipa, SBBA, Rialto-Colton, Riverside-Arlington, Chino, and Temescal Groundwater Subbasins (see Figure 2). In general, the Yucaipa, SBBA, and Rialto-Colton basins share similar and overlapping depositional histories due to local tectonics associated with movement along the San Jacinto, San Andreas, and associated faults. Likewise, the Riverside-Arlington, Temescal, and Chino Basins share similar geologic histories. One of the challenges of developing an integrated regional model was identifying an approach for extending model layers – representing geologic units – across existing model boundaries.

The ISARM consists of five model layers developed using a three-dimensional lithologic model. These layers are:

- Layer 1: Shallow river, wash, and axial-channel deposits present in distinct channels, very young and young alluvial deposits, and the upper portion of old and very old alluvial deposits. Includes Upper Confining Member in the SBBA.
- Layer 2: Old and very old alluvial deposits. In the central part of the SBBA, this layer includes low permeability, interbedded silt, clay, and sand that can reach thicknesses of 300 ft (i.e., Middle Confining Member).
- Layers 3-5: Old and very old alluvial deposits.

The base of the ISARM corresponds to the bedrock surface, which is considered impermeable. The primary sources of recharge to groundwater include mountain front recharge, areal recharge from precipitation, streambed percolation, artificial recharge, and return flow from applied water. The primary discharge terms are groundwater pumping, evapotranspiration, and groundwater discharge to the SAR. The stress period (i.e., time period) used to vary model fluxes, such as pumping and recharge, occurred monthly.

2.2 ISARM Simulation Codes

The watershed model for the Upper Santa Ana Valley Groundwater Basin was developed using the Hydrologic Simulation Program - Fortran (HSPF). HSPF is a successor to the FORTRAN version of the Stanford Watershed Model (SWM). Work in 1974 resulted in the widely available codes developed for and with support of the United States Environmental Protection Agency (EPA). HSPF is a comprehensive and physically based watershed model that can simulate all water cycle components with a time step of less than one day.

The ISARM groundwater flow model was constructed using MODFLOW, a block-centered, modular finite difference groundwater flow code. Widely used and highly versatile, it was developed by the USGS (McDonald and Harbaugh, 1988) for the purpose of modeling both saturated and unsaturated groundwater flow. Specifically, the Newton formulation of the MODFLOW-2005 computer code, known as MODFLOW-NWT, was used for the ISARM. The input data for the MODFLOW-NWT packages was based on a monthly basis (i.e., monthly stress periods) from January 1966 through December 2016.

2.3 Model Grid and Boundary Conditions

The ISARM, shown on Figure 2, covers approximately 1,389 square miles (888,768 acres) with a uniform grid network consisting of 1,642 rows in the northeast to southwest direction (i-direction) and 2,243 columns in the northwest to southeast direction (j-direction). Each cell represents an area of 102.5 ft x 102.5 ft. The entire model grid is rotated 27 degrees clockwise.

Active and inactive model cells of the ISARM were assigned based on published groundwater basin boundaries and geologic mapping. Active model cells generally represent high-permeability, water-bearing basin fill materials (e.g., alluvium) while inactive, or no-flow, cells represent bedrock or low-permeability, consolidated sedimentary material. Other boundary conditions, including general-head, stream, drain, and well (specified flux), were used in the model to simulate groundwater recharge and discharge.

2.4 Recharge and Discharge

Recharge and discharge components in the ISARM include the following terms.

- Recharge:
  - Recharge from mountain front runoff,
  - Areal recharge from precipitation,
  - Streambed percolation,
  - Artificial recharge,
  - San Bernardino/Colton Rapid Infiltration and Extraction (RIX) Facility spreading,
  - Anthropogenic return flow, and
  - Underflow inflow from adjacent groundwater basin areas.
- Discharge:
  - Groundwater pumping,
  - RIX extractions,
  - Evapotranspiration, and
  - Rising groundwater discharge to streamflow.

Simulation of these terms included use of the Streamflow Routing, Recharge, Well, Drain, and Evapotranspiration packages. Details about the various recharge and discharge terms and how they were applied in the model are available in the ISARM Summary Report (Geoscience, 2020).

3.0 Development of Mill Creek Focused Groundwater Model

A focused groundwater model was developed to evaluate various operational scenarios for Mill Creek Basins, including assessment of potential recharge capacity (e.g., any rejected recharge), and identification of shallow groundwater for liquefaction potential and slope stability analyses. The focused model was constructed using data from the existing ISARM. The following subsections describe the development of this local model.

3.1 Focused Model Revisions from ISARM (2020)

The focused groundwater model was developed as a subarea model of the regional ISARM to cover the study area surrounding the existing and proposed Mill Creek Basins. The focused model covers an area of approximately 50.7 square miles (32,451 acres) with a finite-difference grid consisting of 825 rows in the i-direction (northeast to southwest) and 1,467 columns in the j-direction (northwest-southeast). The focused model cell size was refined to 34 ft by 34 ft to provide better resolution at the local scale.

The stress period used to vary groundwater model fluxes was also refined to represent a daily time period for a total model simulation period of 120 days.

Model initial heads represent the water level condition from January of 1983, representing historically high groundwater conditions during a wet hydrologic period.

Boundary conditions used in the focused model are the same as those from the ISARM except for stream cells, which were updated due to model refinement, and are shown in Figure 3. A time-varying constant head boundary (CHD package), shown as green in Figure 3, was used to simulate underflow inflow and outflow to/from the focused groundwater model domain for this modeling evaluation.

3.2 Aquifer Parameters

Aquifer parameters for the focused model, such as model layer elevations, hydraulic conductivity, specific yield, and storativity, are largely the same as those from the calibrated ISARM (Geoscience, 2020). In the vicinity of Plunge Creek, a lower hydraulic conductivity zone was modified to represent a horizontal hydraulic conductivity value of 100 ft/d. This change was based on a review of local well logs, which indicated subsurface conditions consisting largely of sands and gravels (Bouwer, 1978). Specific yield was reduced to a maximum of 0.1 in the vicinity of the project, based on data from monitoring wells near the Mill Creek Spreading Grounds. Specific storage values were updated to the value of one over thickness in Layer 1 to limit the amount of above-ground flooding simulated by the model by setting a large value for storativity in the model. Convertible layer types in MODFLOW otherwise utilize a confined storage value for storativity that is inappropriately low for unconfined aquifers with above-ground ponding. Water levels above ground result in rejected recharge and runoff. Initial groundwater elevations for the focused groundwater model runs represent January 1983 elevations were generated using the updated aquifer parameters (Figure 4). The initial head compared to the ISARM was generally slightly lower in the local model domain except for the middle basin area where flooding condition already existed in the ISARM and water levels declined due to the changes in Layer 1 specific storage.

3.3 Recharge and Discharge

Recharge and discharge (flux) estimates for the focused groundwater model were taken from the ISARM for time varying parameters such as rainfall recharge and groundwater extractions. The underflow inflow/outflow to/from the focused model area was developed from the ISARM regional head solution at the focused model boundary.

Infiltration of stormwater in the existing basin configuration and proposed Mill Creek Basin Improvements were developed from hydrologic simulations provided by Q^3. These recharge and discharge terms are discussed below. The locations of recharge and discharge terms, such as streambed percolation from Mill Creek, were also refined in the focused groundwater model due to the finer grid cell size.

3.3.1 Underflow to/from the Focused Groundwater Model

Underflow to/from the focused groundwater model occurs along the northwest, southwest, and southeast focused model boundaries and is derived from calculations made in the ISARM. It is simulated in the focused model through a time-varying head boundary, as shown in Figure 3.

3.3.2 Stormwater Infiltration in Proposed Mill Creek Basins

Infiltration of diverted stormwater in the existing basin configuration and proposed Mill Creek Basin Improvements were simulated in the focused groundwater model scenarios through the Well Package in MODFLOW. The applied infiltration was based on infiltration hydrographs provided by Q^3, incorporating results from stormwater runoff calculations and infiltration testing performed by other members of the Q^3 project team for the SBVWCD projects at Mill Creek Basins. The amount of infiltration varies based on basin operations and footprint (i.e., existing or expanded), as discussed in Section 4.0.

4.0 Model Scenario Assumptions

The focused groundwater model was used to run three scenarios to evaluate groundwater conditions with different basin expansion and operational alternatives. These scenarios included one model run under Existing Conditions and two project scenarios, as follows:

- Existing Conditions - FIRO flowrates with existing configuration of Mill Creek Basins
- FIRO Scenario – FIRO flowrates with expanded Mill Creek Basins
- Stress Scenario – Optimized Stress flowrates with

The Existing Conditions model run serves as a benchmark for comparison with Mill Creek Basins scenario runs; water balance and water level results compared to this Existing Conditions run isolate the hydrologic effects of each recharge scenario. The Mill Creek Basins recharge scenarios included one scenario run for the expanded basin footprint (see Section 4.2 below), and one scenario run for the expanded basin footprint with proposed new basins.

Each model scenario was run for a period of 120 days and model output was assessed to determine the amount of stormwater recharge achieved after accounting for rejected recharge, to identify locations of shallow groundwater where groundwater elevations are less than 50 ft below ground surface (bgs), and to evaluate water level response in selected nearby well locations. Scenario assumptions for hydrology, proposed additional basin footprints, and Mill Creek Basins recharge are outlined below.

4.1 Hydrology

The scenario runs each assume a wet hydrology with wet antecedent conditions. Long-term and annual trends in precipitation, illustrated in the cumulative departure from mean annual precipitation curve shown in Figure 5, were evaluated to select a period that matched this condition. The hydrologic period selected for the scenario runs represents conditions from February, March, April, and May in 1983.

A total of 162 cfs of Mill Creek flow was diverted to Mill Creek Basins through the North and South intakes. This streamflow assumptions for Mill Creek were developed by Q^3 team members based on maximum diversion capacity. 162 cfs of Mill Creek flow was assumed right above the North intake and 122 cfs was diverted through North intake and 40 cfs through south intake for the first 60 days of model simulation. Flow in Mill Creek was assumed to be the same as April and May 1983 as in the ISARM from day 61 to the end of model simulation.

4.2 Mill Creek Basin Footprints

The existing and two proposed Mill Creek Basin footprints are shown on Figure 6, 7 and 8. The FIRO Scenario basin footprints include the expanded basins and the addition of a southern canal for more flexibility (Figure 7). The footprints for Stress Scenario incorporate all the basin improvements from FRIO Scenario plus the addition of new proposed basins. Stormwater recharge for the various Mill Creek Basin scenarios was simulated using the Well Package, assigned to model cells representing a given basin configuration and condition.

4.3 Mill Creek Basin Recharge

Results of hydrologic modeling conducted by Q^3 using the US Army Corps of Engineers (USACE) Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS), summarized in Mill Creek Basin Hydraulic Technical Memorandum (Q^3, 2025), were utilized as infiltration input to the focused groundwater model for the first 52 days and tailed back down to zero within 60 days. Basin recharge is assumed to be zero for the remaining period of model simulation (day 61 to 120). Results from this analysis, along with infiltration data from tests conducted at the Mill Creek Basins location, guided development of a series of infiltration hydrographs under different conditions. Infiltration hydrographs for Mill Creek Basins are shown in Figures 9 through 11. Total recharge in acre feet of water estimated from the basin routing modeling by Q^3 is summarized below in Table 4-1.

Table 4-1. Recharge Assumptions for Mill Creek Basins Scenarios (acre-feet)

Month	Existing Conditions	FIRO Scenario	Stress Scenario
February	4,479	6,869	9,437
March	3,893	6,116	8,459
April	0	0	0
May	0	0	0
Total	**8,371**	**12,985**	**17,896**

Water was removed from the Mill Creek streamflow routing package to account for the simulated stormwater diversions.

5.0 Scenario Results

5.1 Water Balance

The focused groundwater model water balances for the Existing Conditions and Mill Creek Basins 120-day scenario runs are provided in Appendix A Tables 1 through 3. Total groundwater water-balance calculations are shown in Figure 12. The results indicate that the increased infiltration from FIRO and Stress Scenario only causes a minimal increase in evapotranspiration and slightly decreased streambed percolation. and 100% contributes to the increased basin storage, while Stress Scenario only slightly decreased streambed percolation and 99% of the increased infiltration contributes to the increased basin storage. The percentage of increased infiltration contributes to the increased basin storage are 100% and 99% under FIRO Scenario and Stress Scenario, respectively. However, water levels indicate the possibility of rejected recharge in some of the central and upstream basins due to shallow groundwater levels. This water may recharge further downstream in the basins or may result in rejected recharge. Where the model simulates above-ground water levels, rejected recharge may be occurring that is not captured in the water budget analysis, due to surface ponding and runoff that is not simulated in the model.

5.2 Groundwater Mounding

The estimated change in groundwater elevation between the Existing Conditions run and each Mill Creek Basins scenario run represents anticipated mounding from recharge in the proposed spreading basins. Figures 13 and 14 show anticipated mounding under basin recharge conditions for each scenario, including maximum mounding and mounding at the end of the model simulation (i.e., 120 days). Mounding conditions are summarized in the following table.

Table 5-1. Groundwater Mounding for Plunge Creek Basins

Scenario	Maximum Change in Groundwater Elevation (ft)	Time of Maximum Mounding (days after start of recharge)
FIRO Scenario	70	66
Stress Scenario	80	66

As shown, maximum mounding occurs at 66 days after the start of spreading for both FIRO and Stress Scenario. Groundwater elevations under Mill Creek Basins rise up to 70 ft under FIRO Scenario and up to 80 ft under Stress Scenario.

Changes in groundwater elevation through time are also shown in the hydrographs provided in Figures 15 and 16. Each hydrograph illustrates model-predicted groundwater elevations at a point under the Mill Creek Basins for the Existing Conditions and each scenario run.

5.3 Area of Shallow Groundwater

Soil liquefaction, which typically occurs in recent (Holocene to late Pleistocene) deposits of silt, sand, and gravel, can be a major cause of damage during earthquakes. Most liquefaction occurs where the depth to groundwater is less than 50 ft; this depth is traditionally considered adequate for most investigations of liquefaction potential (Martin and Lew, 1999). For the purpose of this study, areas with depth to groundwater of less than 50 ft bgs were evaluated for each model scenario.

Results, shown in Figures 17 and 18, indicate an area with groundwater elevations within 50 ft of ground surface in the vicinity of Mill Creek under Existing Conditions and both project scenarios during the historically wet hydrologic conditions simulated. The footprint of this area of shallow groundwater increased 0.78 to 4.73% under project scenarios, as summarized in the following table. The mound of recharged water can be seen in the shallowing of groundwater elevations near the upper Mill Creek Basins in Figure 17 and 18.

Table 5-2. Change in Area of Shallow Groundwater (<50 ft bgs)

Recharge Scenario	Area of Shallow Groundwater (acres)	Difference from Existing Conditions (acres)	Change
Existing Conditions	1,068	NA	NA
FIRO Scenario	1,077	8	0.78%
Stress Scenario	1,119	51	4.73%

Historical recharge data and measured groundwater elevation of the Mill Creek monitoring wells (MC-01, MC-02, MC-04, and MC-05) were reviewed to assess the shallow groundwater conditions in the vicinity of the Mill Creek Basins. The recharge period from April to May 2023 had a cumulative 2-month recharge of 7,925 acre-feet comparable to the Existing Conditions 60-day recharge of 8,371 acre-feet. Figure 19 presents the measured groundwater elevation from 2010 to 2024 in the Mill Creek monitoring wells. Historical shallow groundwater of less than 50 feet was observed for most wells during wet periods. When water level rises in the upper basins, rejected recharge may flow downstream and be recharged in lower basins preferentially versus what is indicated by the distribution of recharge from a surface water only model. Overall, shallow groundwater conditions exist in the measured data, particularly during periods of high recharge. The water level difference during the May 2023 period between MC-01 and MC-05 is 130 to 140 ft, indicating a rapid decline in groundwater level into the groundwater basin away from the recharge basins.

6.0 Summary

Geoscience modeled recharge in Mill Creek Basins using a focused groundwater model derived from the existing regional ISARM. The focused model was run for a period of 120 days representing the hydrologic period from February through May 1983. Simulated recharge volumes were derived from HEC-HMS modeling simulating diversion of wet March baseflows from Mill Creek and simulating the recharge of FIRO and Stress flows. Modeling results provided an indication of potential effects on groundwater elevations under different basin operational conditions, including:

- Existing Conditions - FIRO flowrates with existing configuration of Mill Creek Basins

- FIRO Scenario – FIRO flowrates with expanded Mill Creek Basins
- Stress Scenario – Optimized Stress flowrates with

Groundwater elevations under the Mill Creek Basins will rise as much as 70 and 80 ft with simulated recharge under FIRO and Stress Scenario, respectively, obtaining maximum mounding 66 days after the start of recharge. Increases in groundwater elevations are greatest under Stress Scenario. Recharge in Mill Creek Basins also causes an increase in the area of shallow groundwater (i.e., less than 50 ft from ground surface) from Existing Conditions, ranging from an increase of 0.78% under FIRO Scenario to 4.73% under Stress Scenario. Results indicate that shallow groundwater conditions may exist in the Mill Creek Basins in all scenarios, and indicate the possibility for rejected recharge during wet hydrologic conditions. Observed groundwater levels during wet periods in the historical record indicate water levels in the downgradient basins within 20 feet of ground surface.

Mounding in the lower basins and the flooding conditions in the upper basins could be a result of biased infiltration distribution. When water level rises in the upper basins, rejected recharge may flow downstream and be recharged in lower basins instead or may result in additional rejected recharge.

7.0 References

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FIGURES

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EXPLANATION

- Focused Groundwater Model Boundary for Mill Creek Recharge Basins Evaluation
- Mill Creek Basins
- Integrated SAR Model (ISARM) Boundary

N

0 1 2

Miles

Aug-25

ISARM Boundary

Figure Boundary

Focused Model Boundary

Project Location within ISARM Area

PROJECT LOCATION

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GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

FIGURE 1

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SAN GABRIEL MOUNTAINS

SAN BERNARDINO MOUNTAINS

Los Angeles Co

San Bernardino Co

San Bernardino Co

Riverside Co

<---i-direction

j-direction --->

1

1

1,642

2,243

RIALTO-COLTON BASIN

SAN BERNARDINO BASIN AREA

CHINO BASIN

YUCAIPA BASIN

RIVERSIDE-ARLINGTON BASIN

TEMESCAL BASIN

Seven Oaks Dam

Prado Dam

Santa Ana River

Riverside Co

San Bernardino Co

66

15

60

83

18

215

10

91

215

EXPLANATION

- Upper Santa Ana River Watershed Boundary
- Dam Location Used in Upper Santa Ana River Watershed Model
- Integrated SAR Model (ISARM) Boundary
- Active Model Area
- General-Head Boundary
- Inactive Model Area
- Stream

Well (Specified Flux)

- Well Simulating Mountain Front Recharge
- Pumping Well
- Well Simulating Underflow Inflow

- Groundwater Flow Barrier

N

0 6 12

Miles

Aug-25

INTEGRATED SANTA ANA RIVER MODEL

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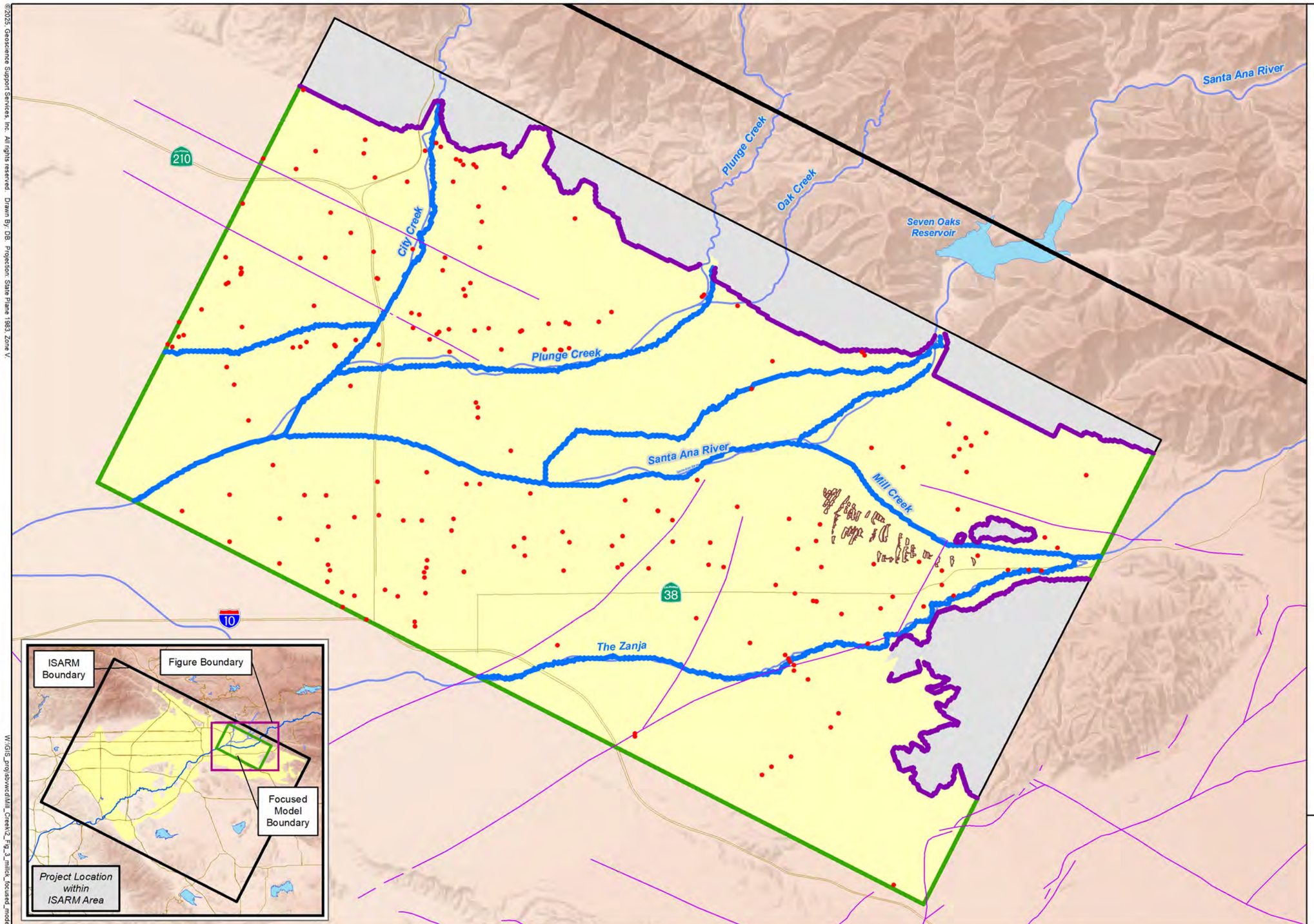
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GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

FIGURE 2

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EXPLANATION

- ISARM Boundary
- Focused Groundwater Model Boundary for Mill Creek Recharge Basins Evaluation
- Active Model Area
- Time-Varying Constant Head Boundary
- Inactive Model Area
- Stream

Well (Specified Flux)

- Well Simulating Mountain Front Recharge
- Pumping Well

- Mill Creek Basins
- Groundwater Flow Barrier

N

0 1 2

Miles

Aug-25

FOCUSED GROUNDWATER MODEL

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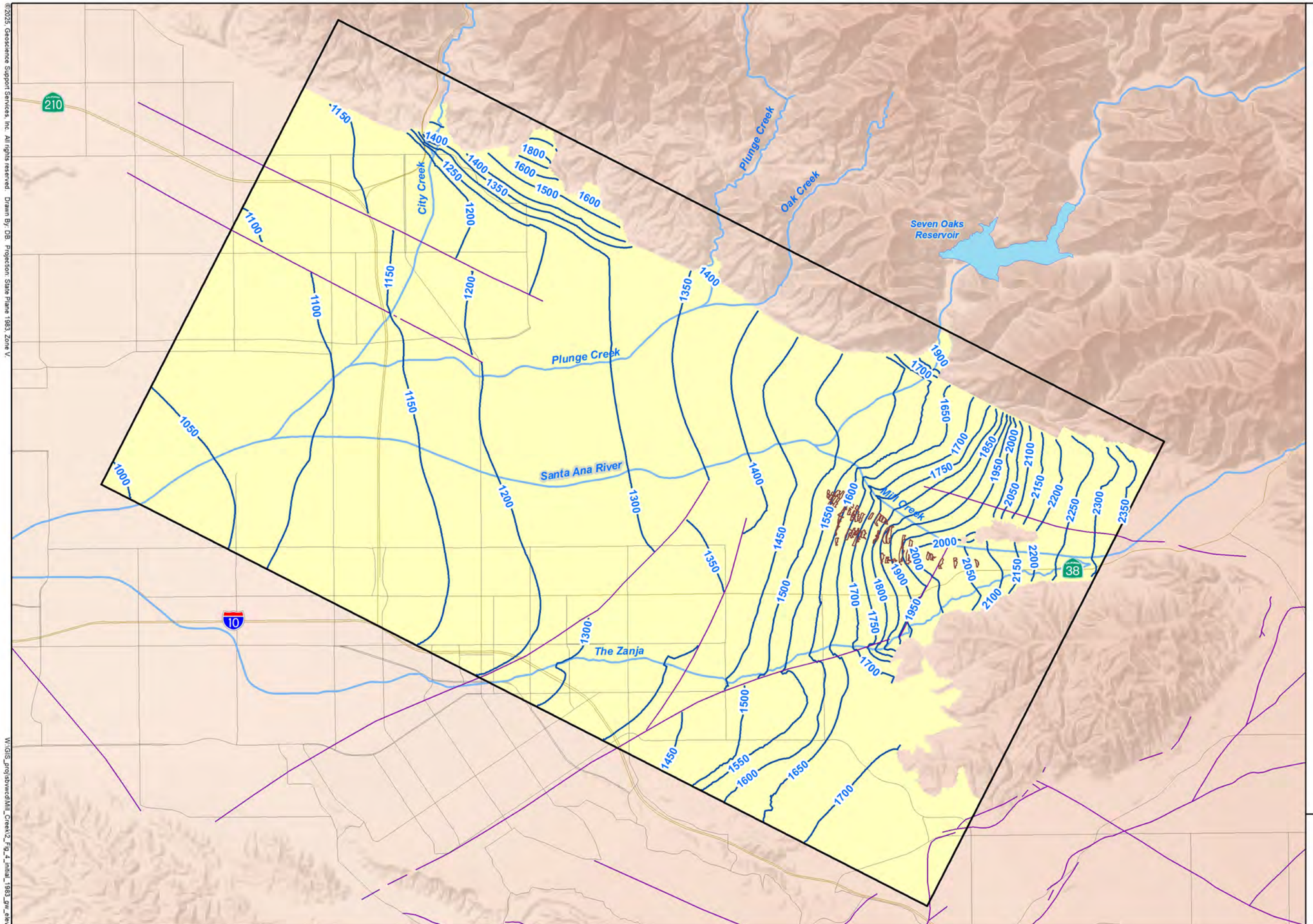
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GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

FIGURE 3

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EXPLANATION

- Focused Groundwater Model Boundary for Mill Creek Recharge Basins Evaluation
- Active Model Area
- Groundwater Flow Barrier
- Mill Creek Basins
- —1250— Groundwater Elevations (ft, msl)

N

0 1 2

Miles

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INITIAL GROUNDWATER ELEVATIONS JANUARY 1983

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GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

FIGURE 4

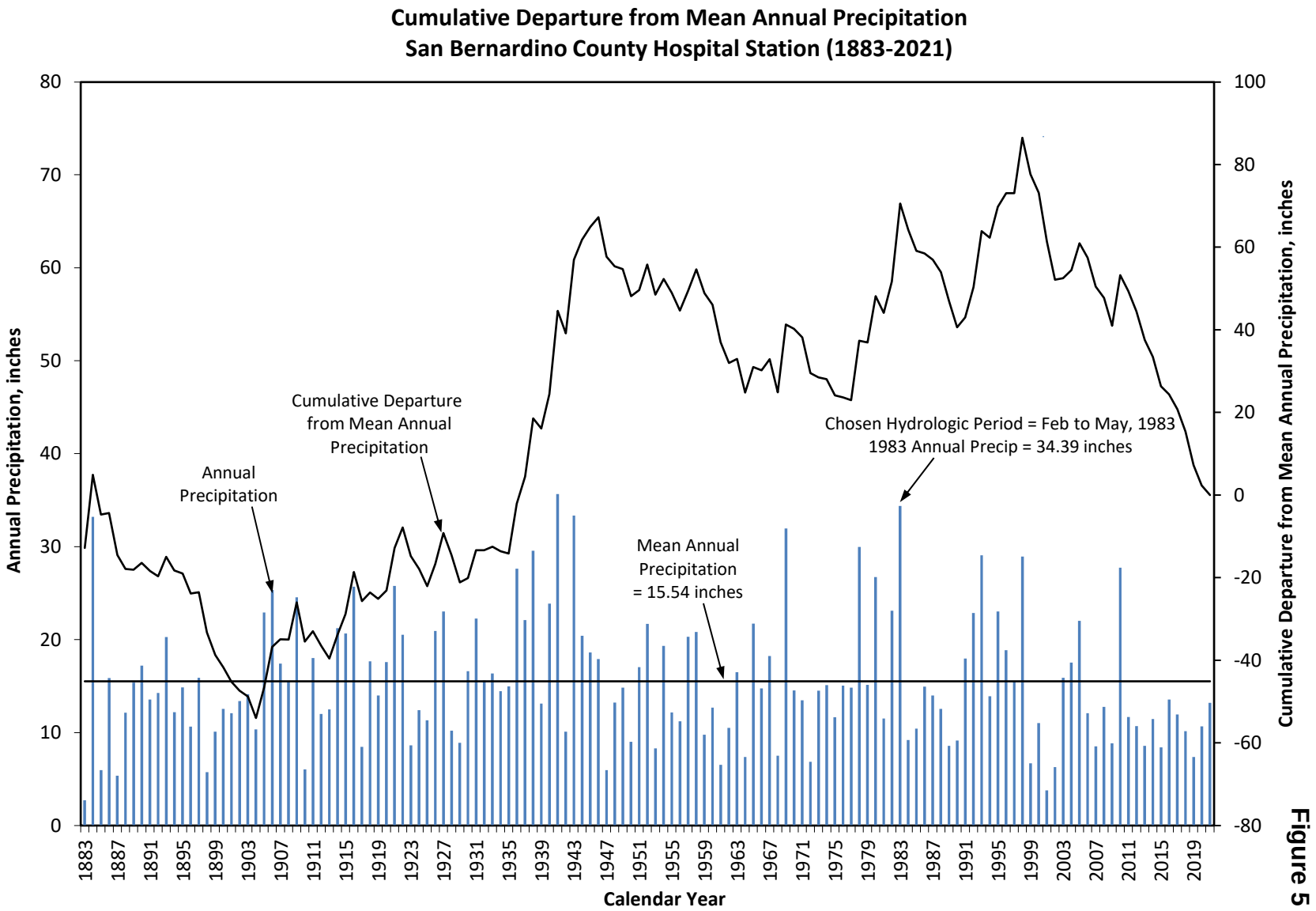
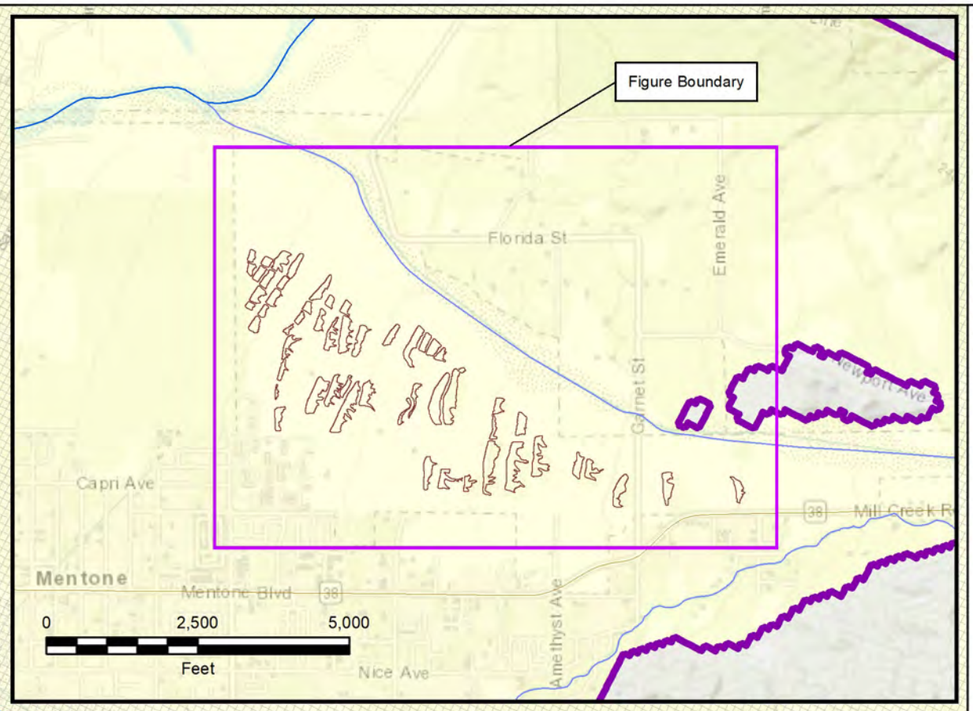

Cumulative Departure from Mean Annual Precipitation
San Bernardino County Hospital Station (1883-2021)
Annual Precipitation, inches
Cumulative Departure from Mean Annual Precipitation, inches
Calendar Year
Annual Precipitation
Cumulative Departure from Mean Annual Precipitation
Mean Annual Precipitation = 15.54 inches
Chosen Hydrologic Period = Feb to May, 1983
1983 Annual Precip = 34.39 inches

Figure 5

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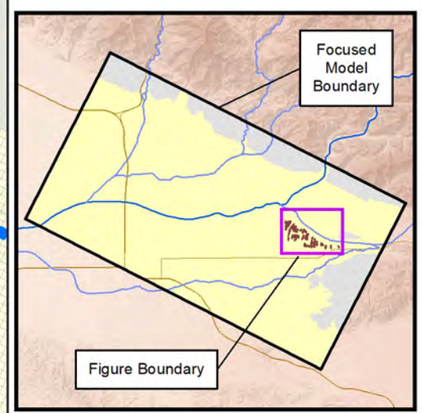
EXPLANATION

- Active Model Area
- Inactive Model Area
- Well Simulating Mountain Front Recharge
- Basin Recharge Cell
- Stream Cell
- Mill Creek Basins

N

0 500 1,000

Feet



Aug-25

MILL CREEK BASIN FOOTPRINTS EXISTING CONDITIONS

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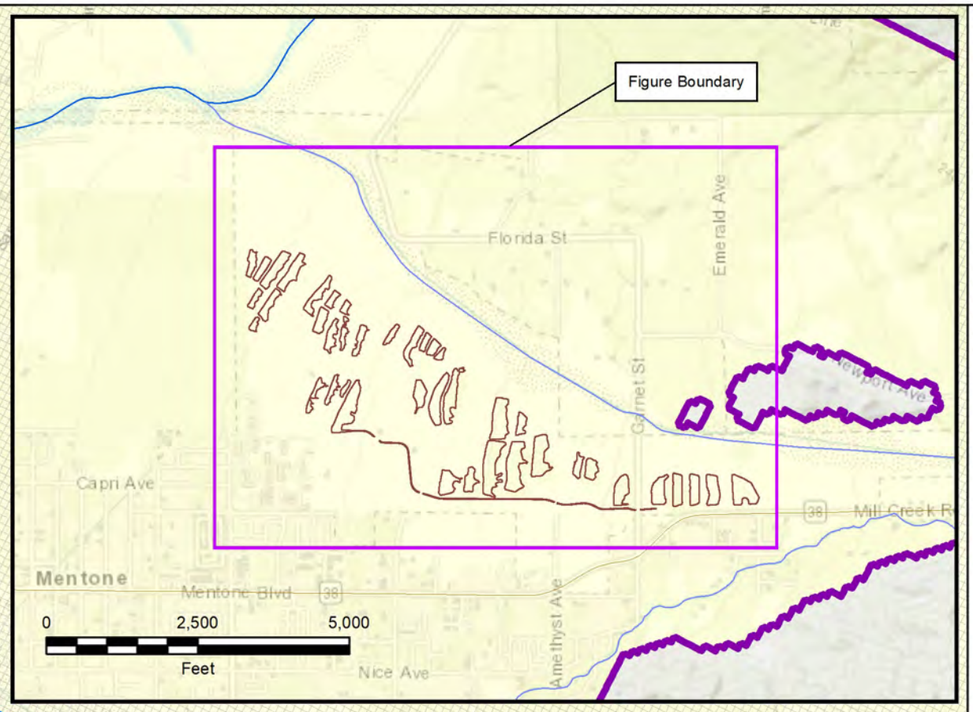
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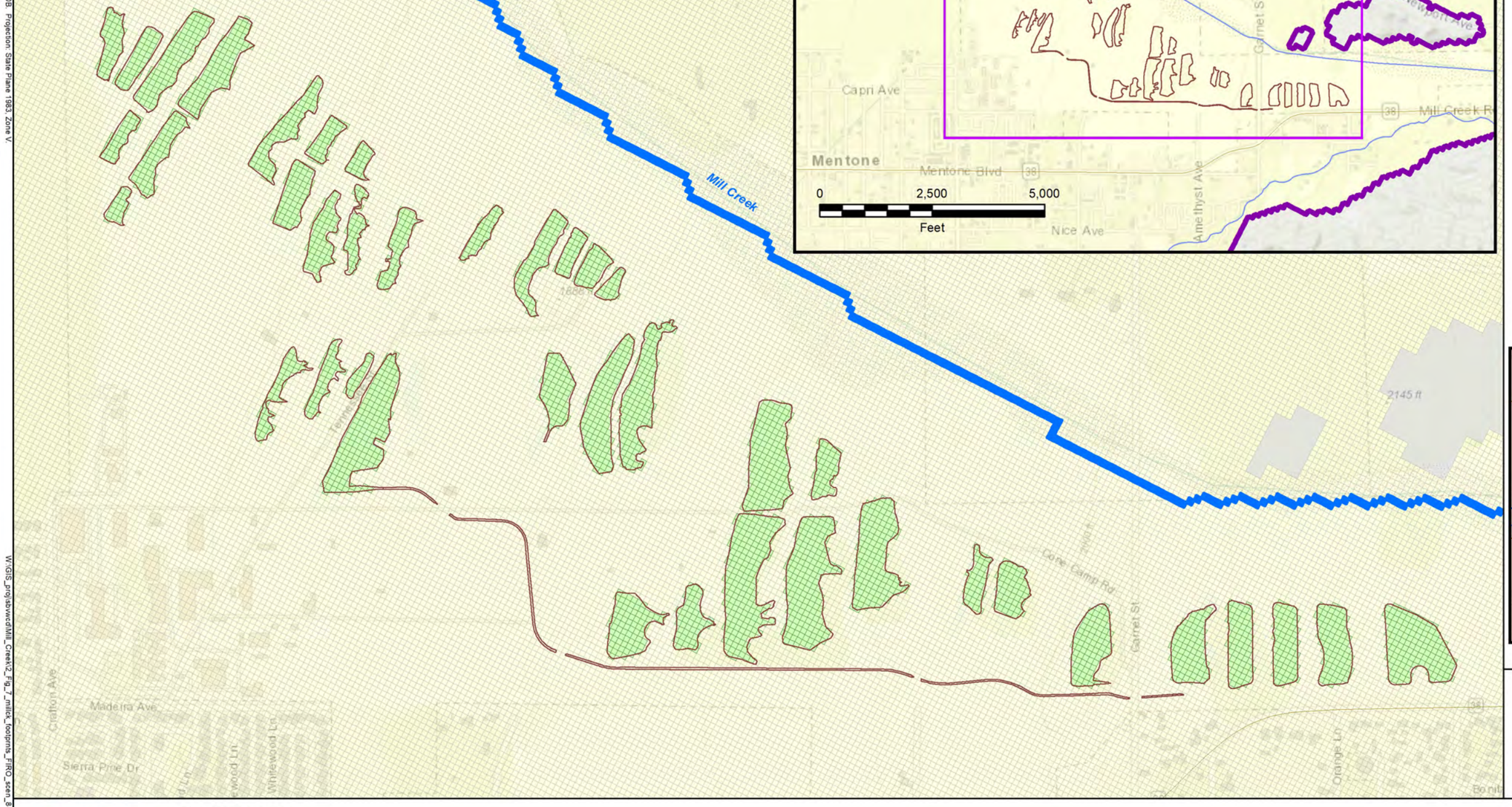
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GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

FIGURE 6

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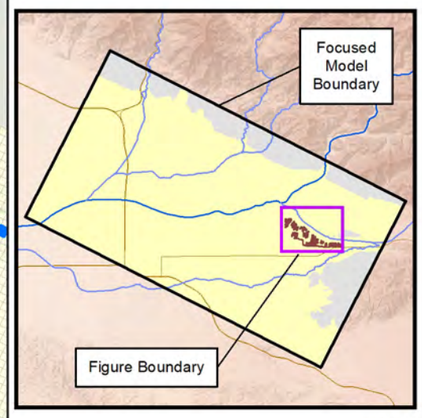
EXPLANATION

- Active Model Area
- Inactive Model Area
- Well Simulating Mountain Front Recharge
- Basin Recharge Cell
- Stream Cell
- Mill Creek Basins

N

0 500 1,000

Feet



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MILL CREEK BASIN FOOTPRINTS FIRO SCENARIO

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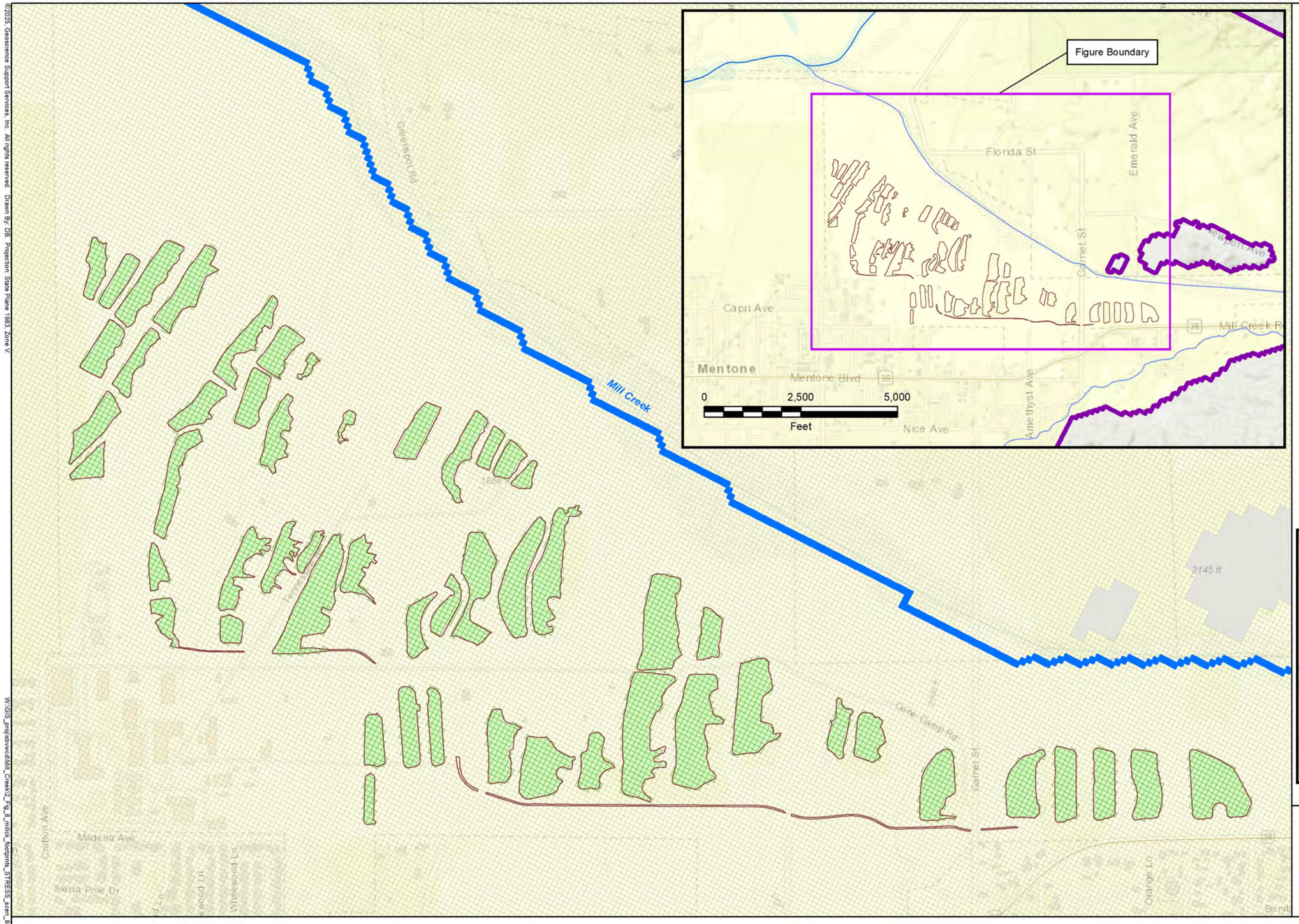
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GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

FIGURE 7

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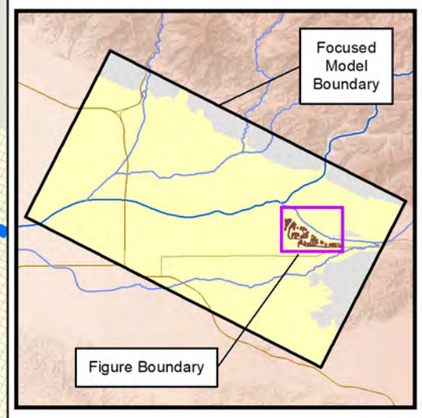
EXPLANATION

- Active Model Area
- Inactive Model Area
- Well Simulating Mountain Front Recharge
- Basin Recharge Cell
- Stream Cell
- Mill Creek Basins

N

0 500 1,000

Feet



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MILL CREEK BASIN FOOTPRINTS STRESS SCENARIO

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GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

FIGURE 8

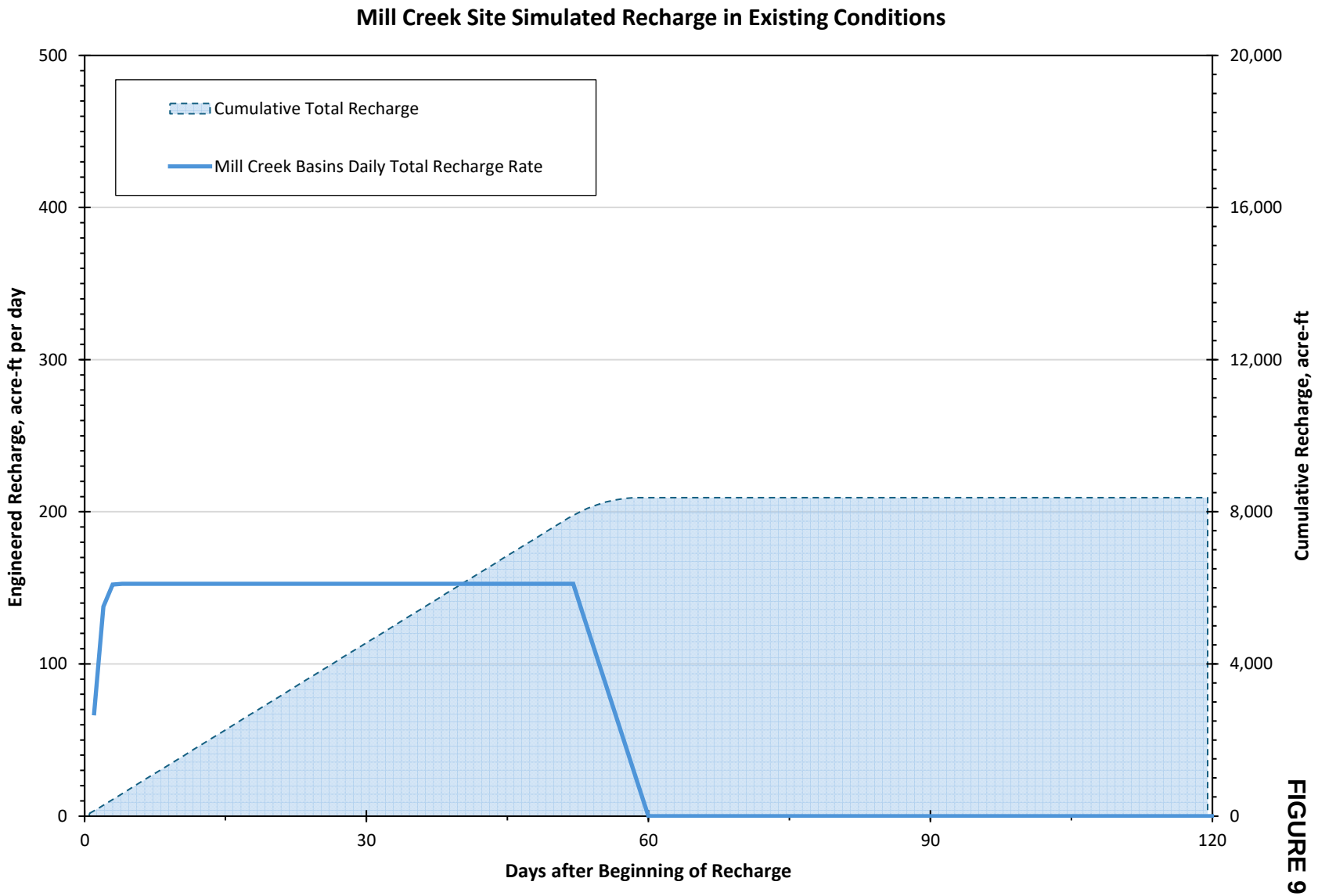

Mill Creek Site Simulated Recharge in Existing Conditions
Cumulative Total Recharge
Mill Creek Basins Daily Total Recharge Rate
Engineered Recharge, acre-ft per day
Cumulative Recharge, acre-ft
Days after Beginning of Recharge
0
100
200
300
400
500
0
4,000
8,000
12,000
16,000
20,000
0
30
60
90
120

FIGURE 9

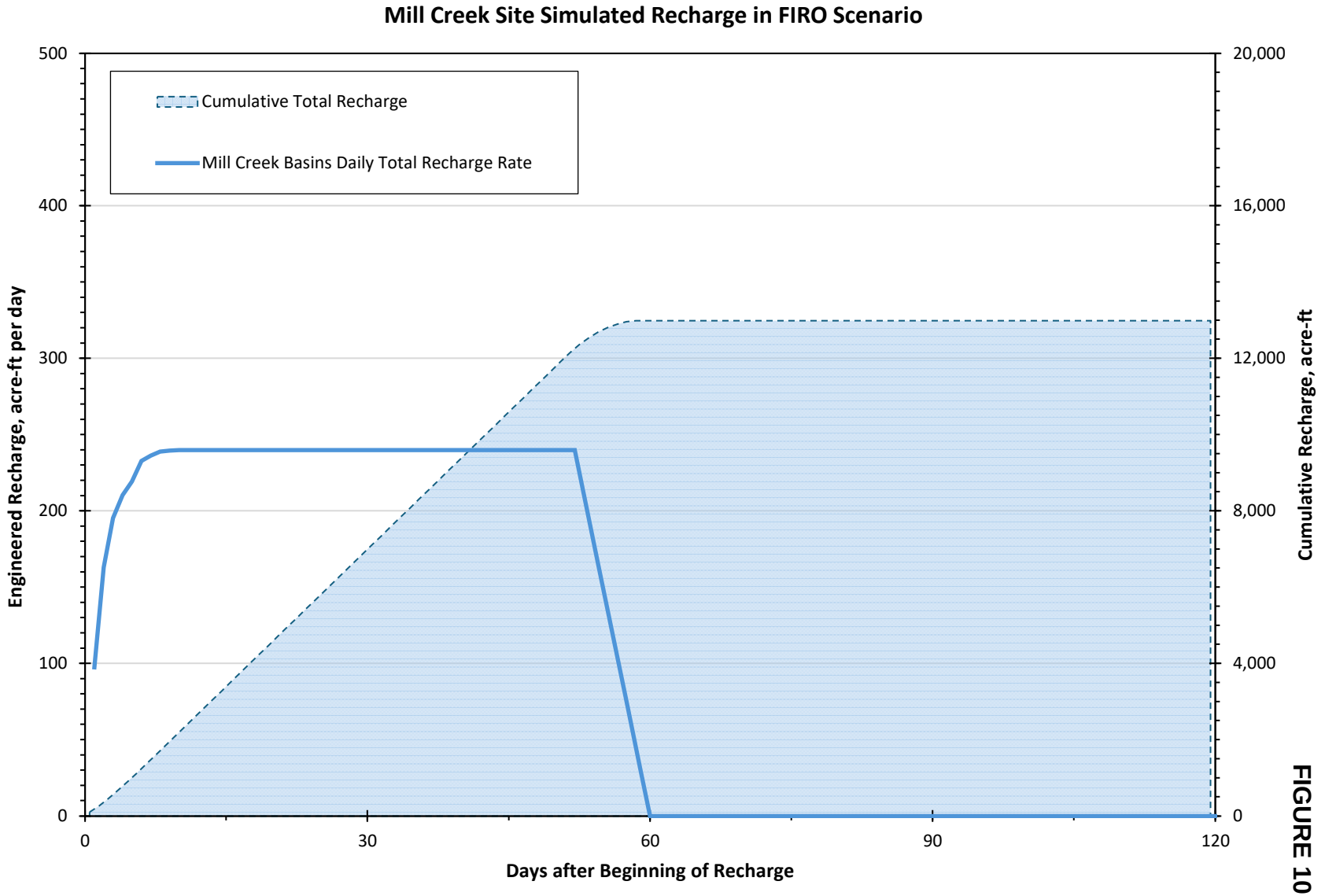

Mill Creek Site Simulated Recharge in FIRO Scenario
Cumulative Total Recharge
Mill Creek Basins Daily Total Recharge Rate
Engineered Recharge, acre-ft per day
Cumulative Recharge, acre-ft
Days after Beginning of Recharge
0
100
200
300
400
500
0
4,000
8,000
12,000
16,000
20,000
0
30
60
90
120

FIGURE 10

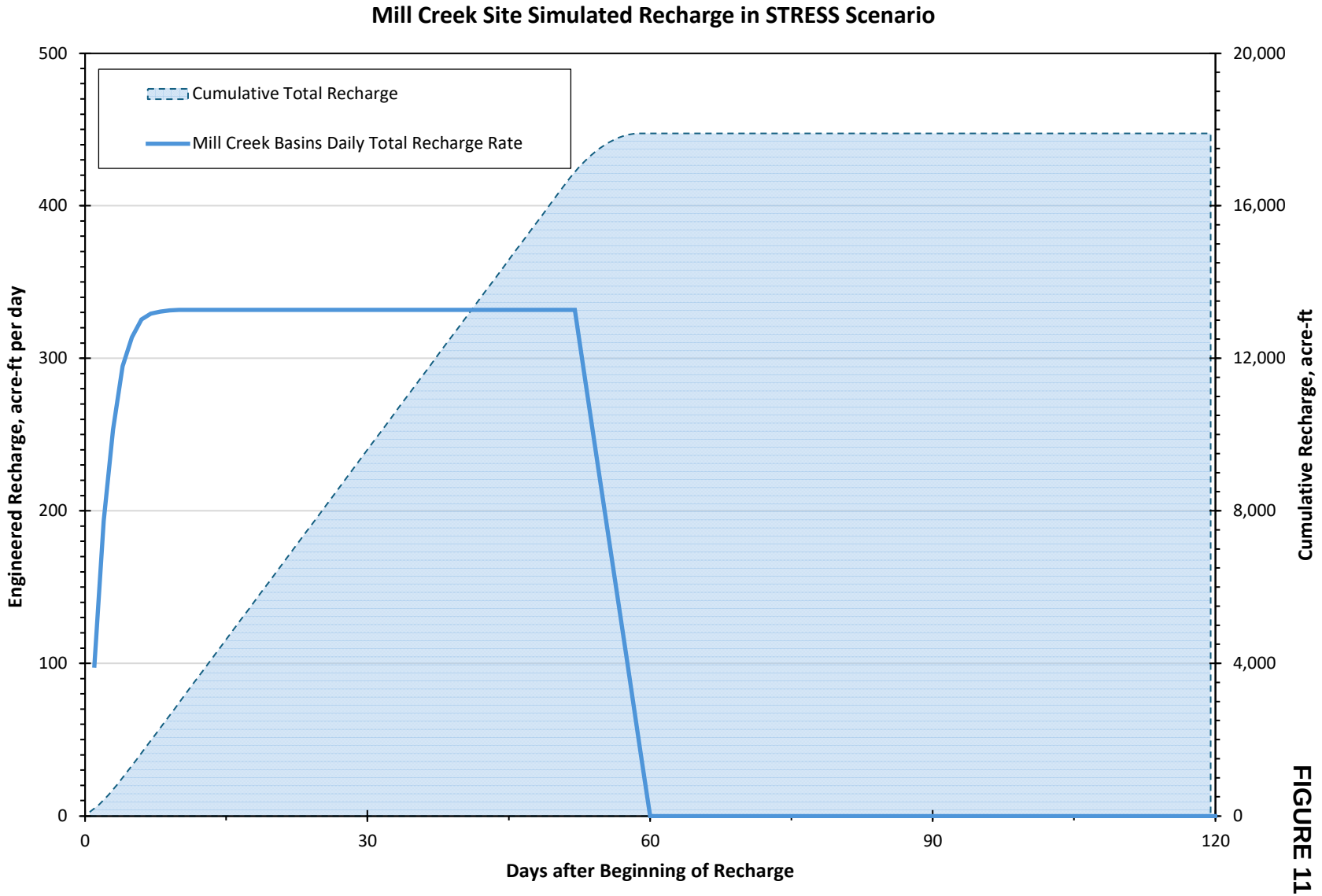
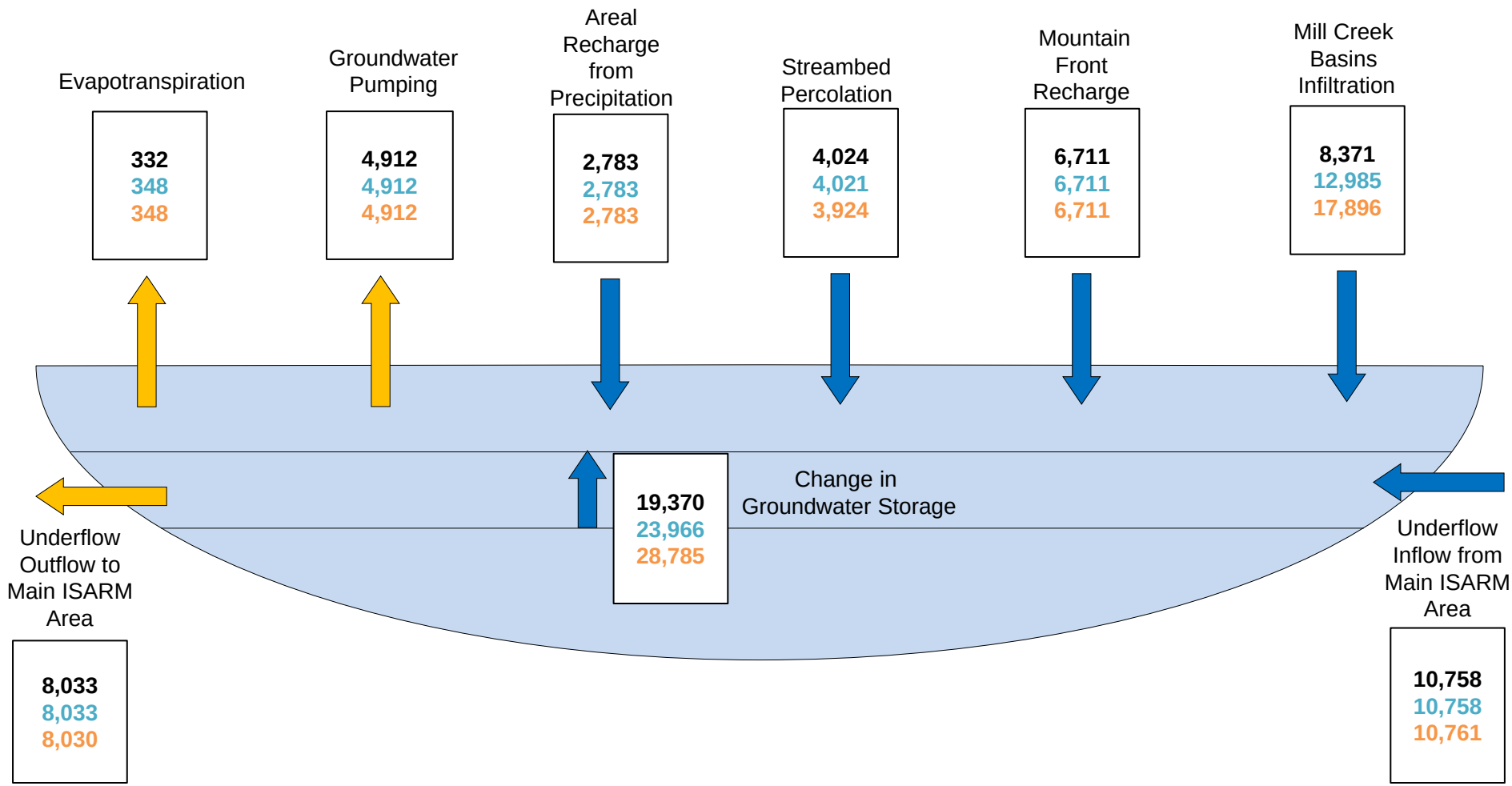

Mill Creek Site Simulated Recharge in STRESS Scenario
Cumulative Total Recharge
Mill Creek Basins Daily Total Recharge Rate
Engineered Recharge, acre-ft per day
Cumulative Recharge, acre-ft
Days after Beginning of Recharge
0
100
200
300
400
500
0
4,000
8,000
12,000
16,000
20,000
0
30
60
90
120

FIGURE 11

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Flux term from Focused Groundwater Model for Mill Creek Basins :

Mill Creek Basins Existing Conditions
Mill Creek Basins FIRO Scenario
Mill Creek Basins STRESS Scenario

All values in acre-ft

FOCUSED GROUNDWATER MODEL 120-DAY WATER BALANCE ESTIMATES

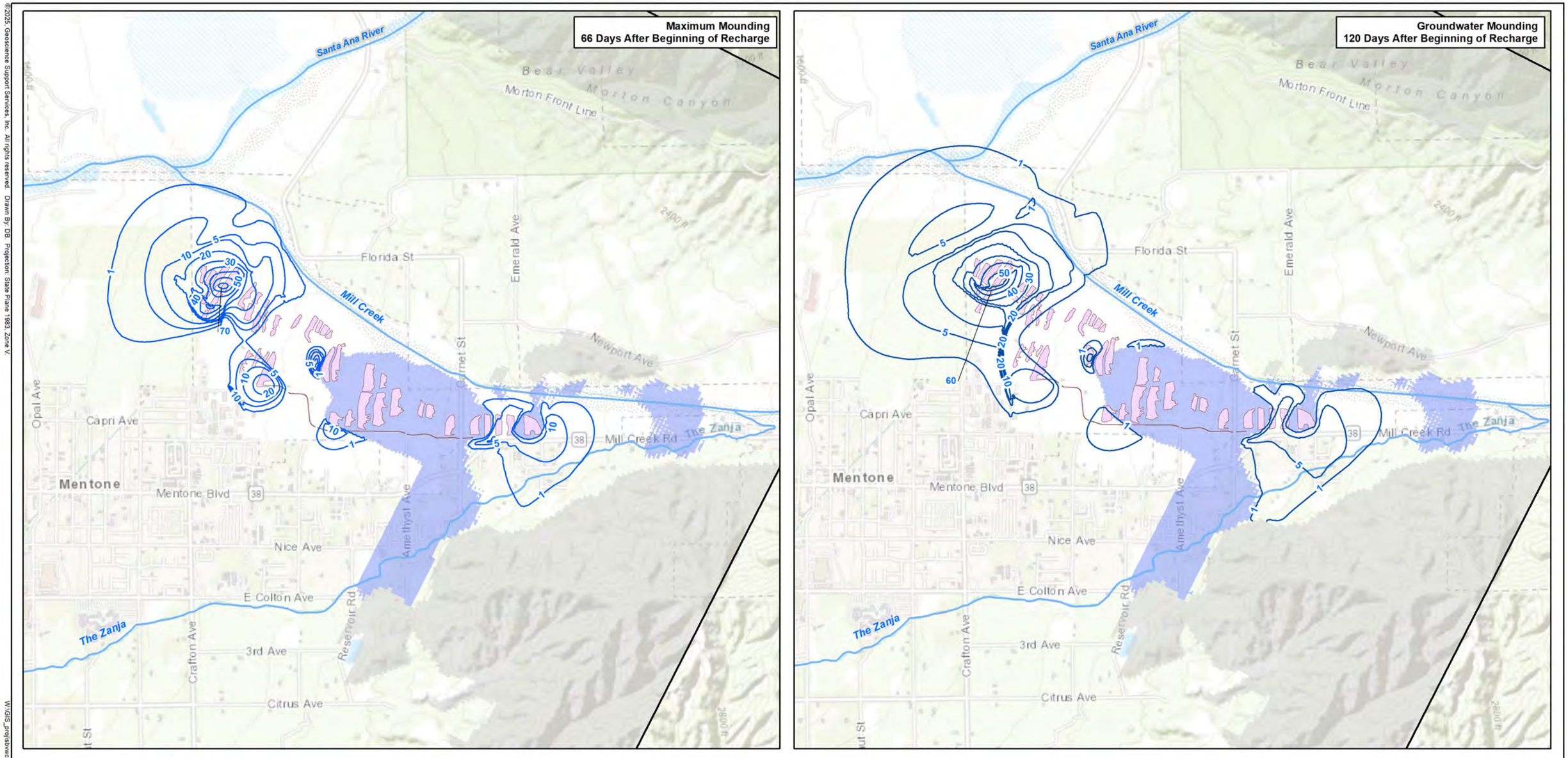
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Maximum Mounding
66 Days After Beginning of Recharge

Groundwater Mounding
120 Days After Beginning of Recharge



EXPLANATION

- Inactive Model Area
- Mill Creek Basins
- 1 — Change in Groundwater Elevation (ft)
- Flooding Under Existing Conditions

N

0 2,500 5,000

Feet

Aug-25

GROUNDWATER MOUNDING FIRO SCENARIO MINUS EXISTING CONDITIONS

GEOSCIENCE

Q³ CONSULTING

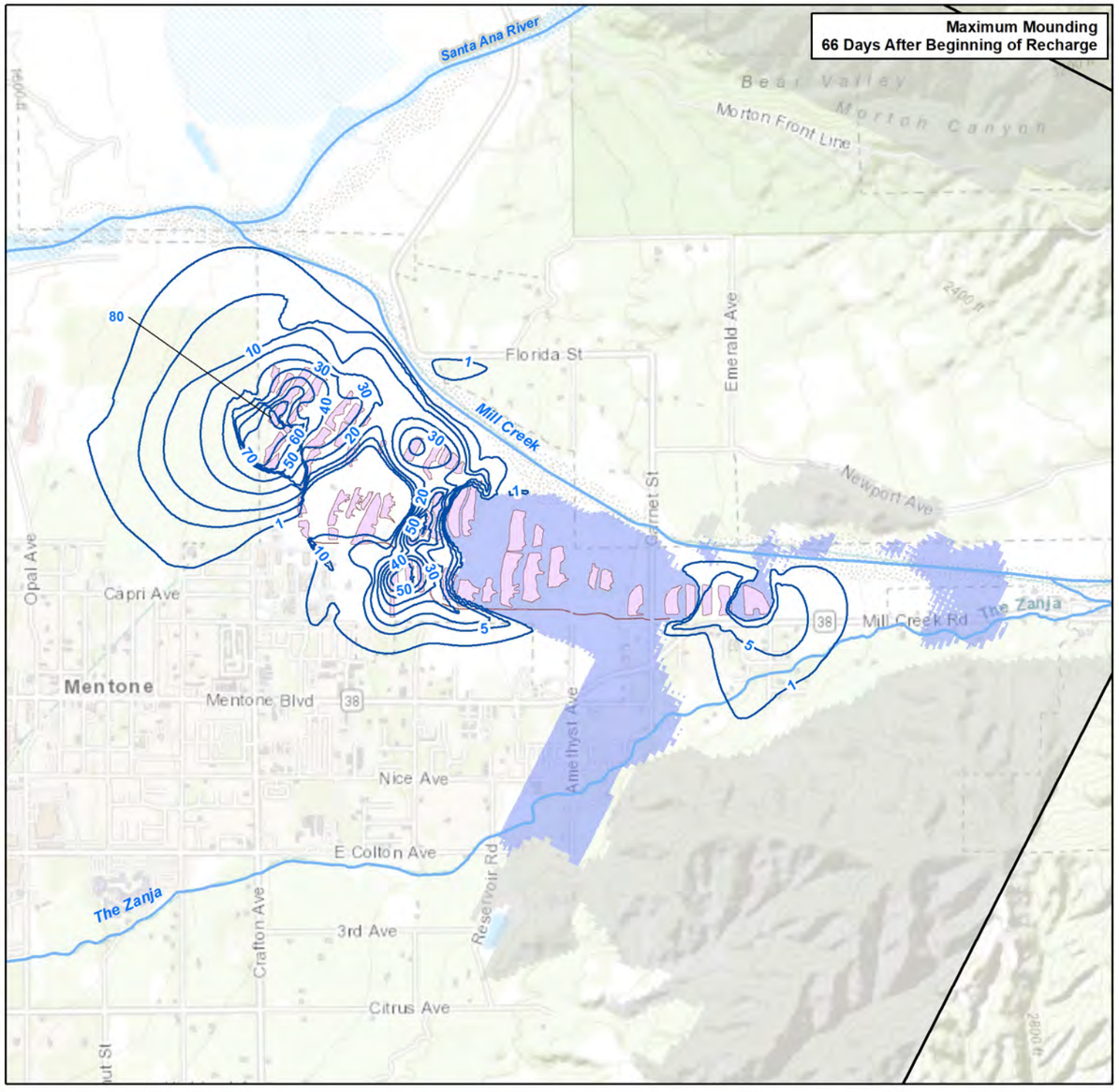
GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

FIGURE 13

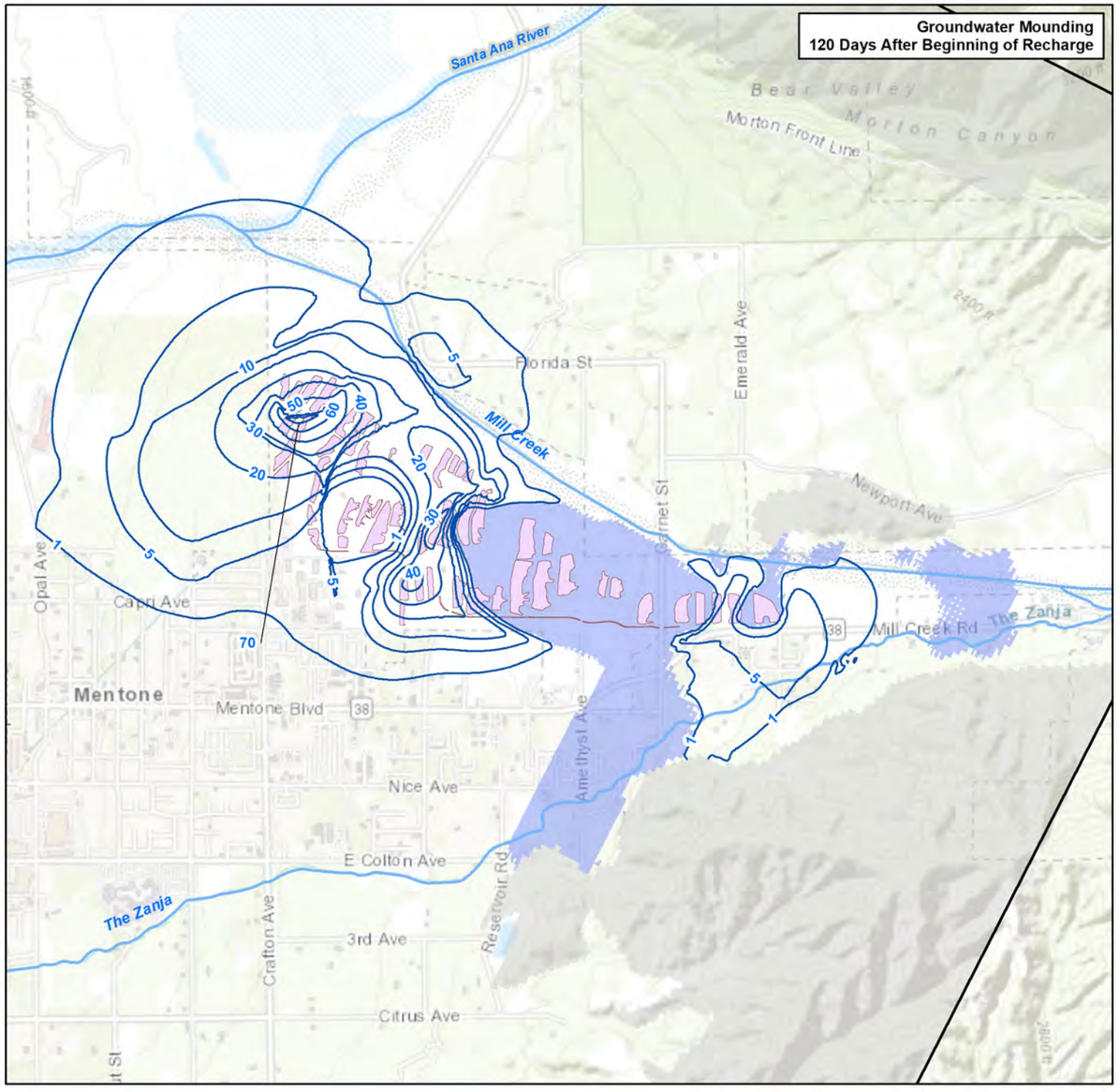
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Maximum Mounding
66 Days After Beginning of Recharge



Groundwater Mounding
120 Days After Beginning of Recharge



EXPLANATION

- Inactive Model Area
- Mill Creek Basins
- —1— Change in Groundwater Elevation (ft)
- Flooding Under Existing Conditions

N

0 2,500 5,000

Feet

Aug-25

GROUNDWATER MOUNDING STRESS SCENARIO MINUS EXISTING CONDITIONS

GEOSCIENCE

Q³ CONSULTING

GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

FIGURE 14

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DRAFT

Groundwater Elevation below Mill Creek Basins
Existing Conditions vs. FIRO Scenario

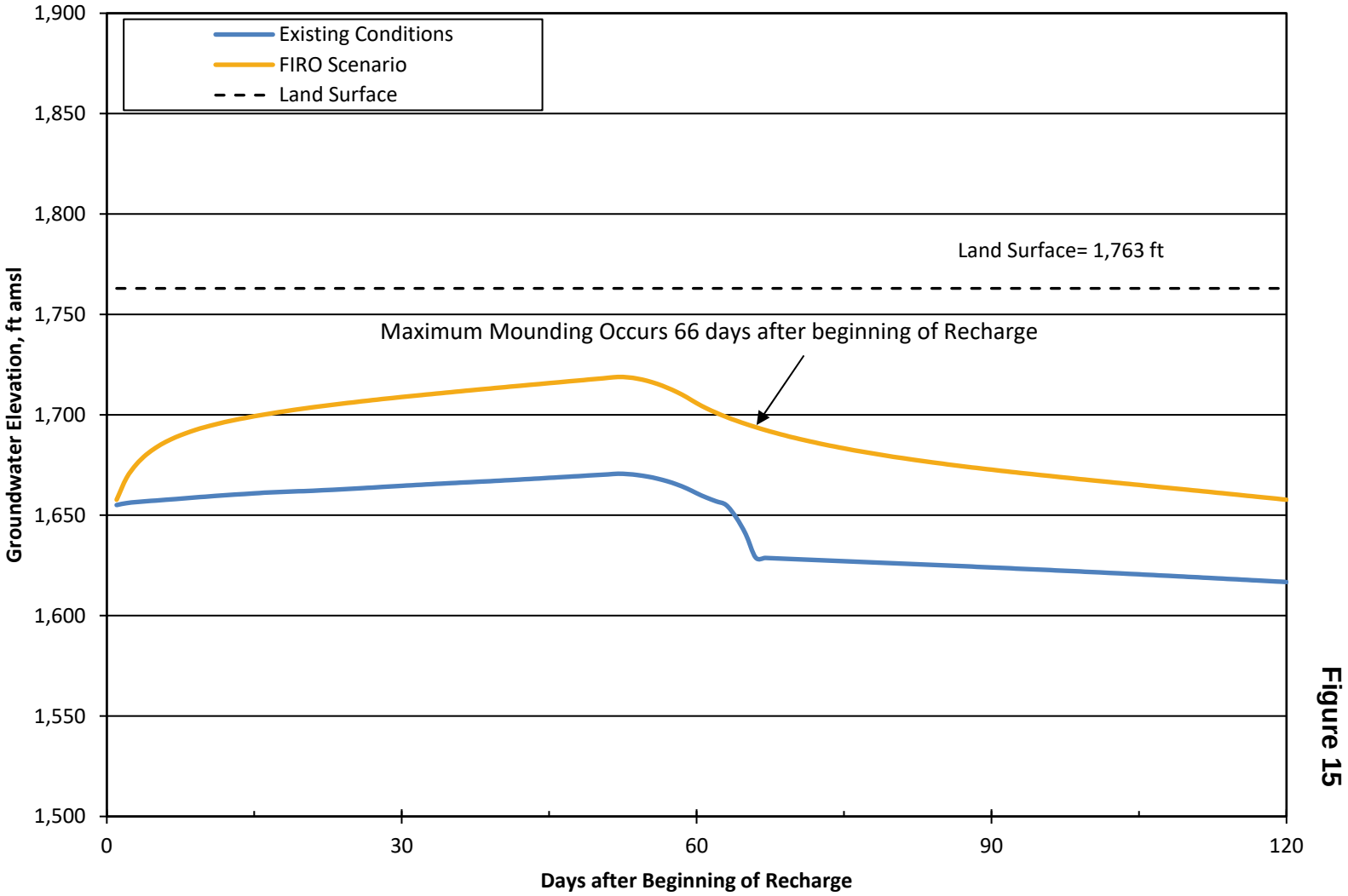


Figure 15

DRAFT

Groundwater Elevation below Mill Creek Basins
Existing Conditions vs. STRESS Scenario

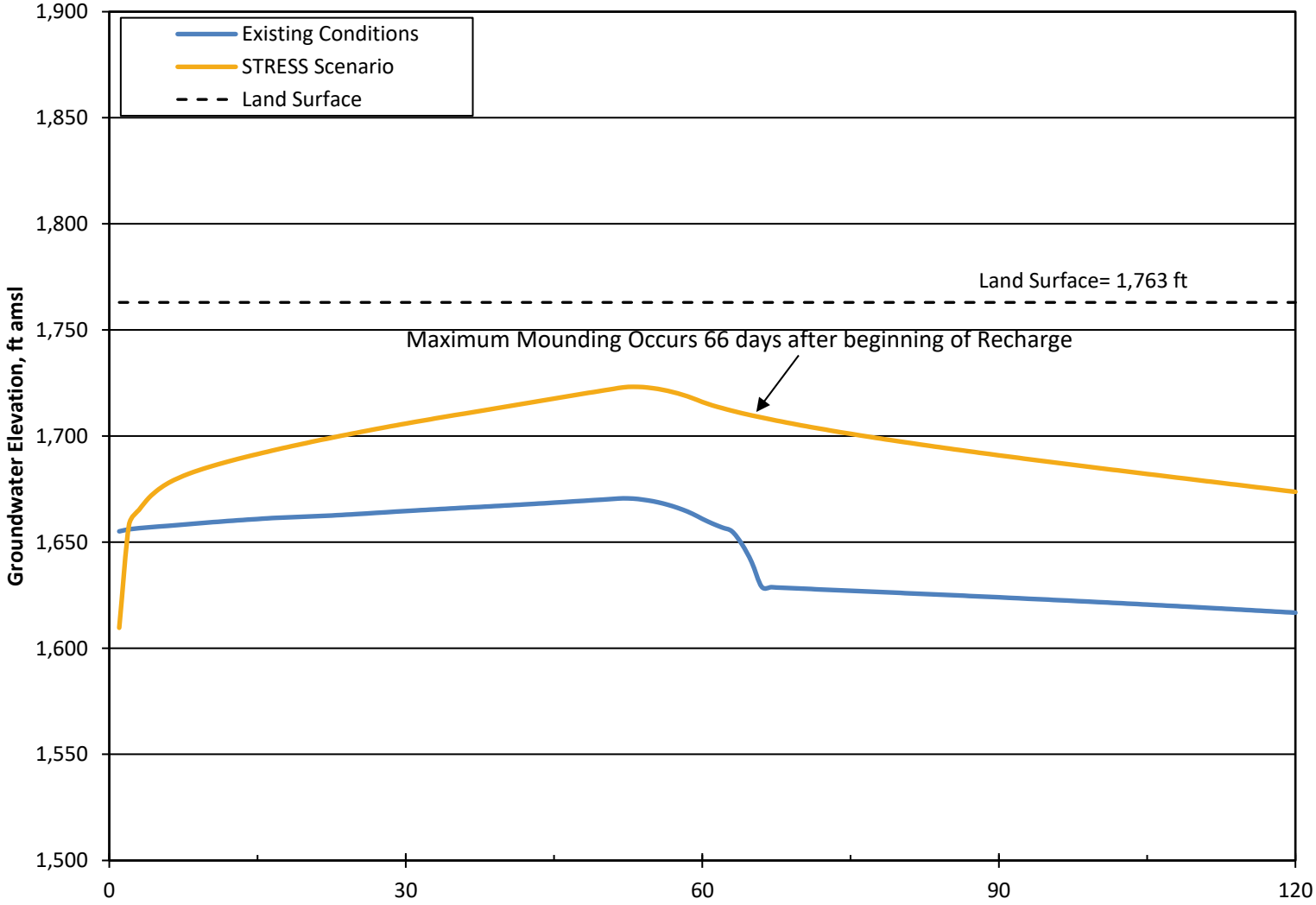


Figure 16

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Existing Conditions
66 Days After Beginning of Recharge

FIRO Scenario
66 Days After Beginning of Recharge

Santa Ana River
Bear Valley
Morton Canyon
Morton Front Line
2490 ft
Florida St
Emerald Ave
Mill Creek
Newport Ave
Capri Ave
ntone
Mentone Blvd
38
Nice Ave
The Zanja
E Colton Ave
Crafton Ave
3rd Ave
Reservoir
Zanja Peak
Citrus Ave
ut St
2800 ft

EXPLANATION

Focused Groundwater Model Boundary for Mill Creek Recharge Basins Evaluation

Inactive Model Area

Mill Creek Basins

Depth to Groundwater (<50 ft, bgs)

	Flooding		20 - 30
	0 - 10		30 - 40
	10 - 20		40 - 50

N

0 2,000 4,000

Feet

Aug-25

AREA OF SHALLOW GROUNDWATER (<50 ft bgs) EXISTING CONDITIONS VS FIRO SCENARIO

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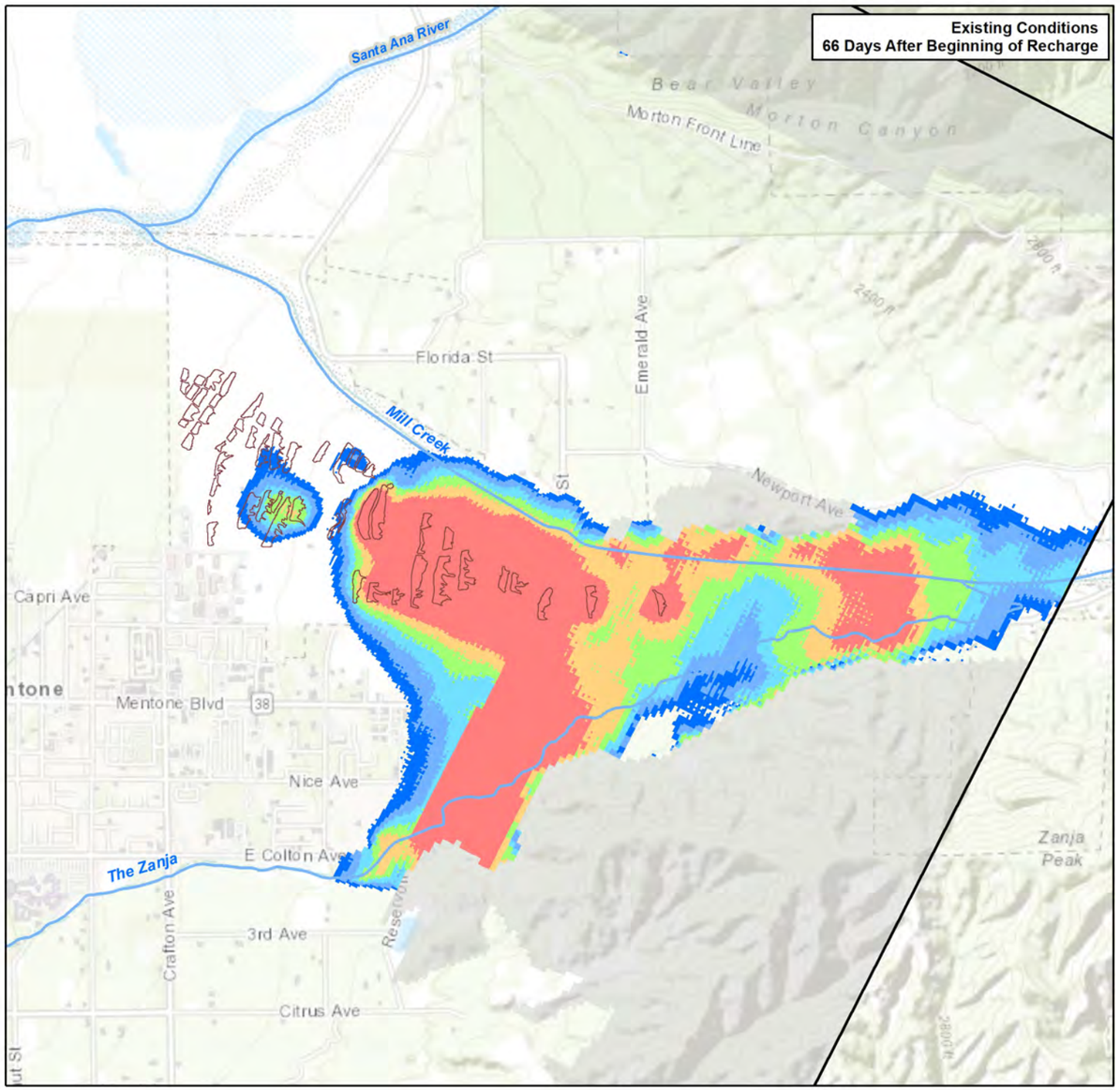
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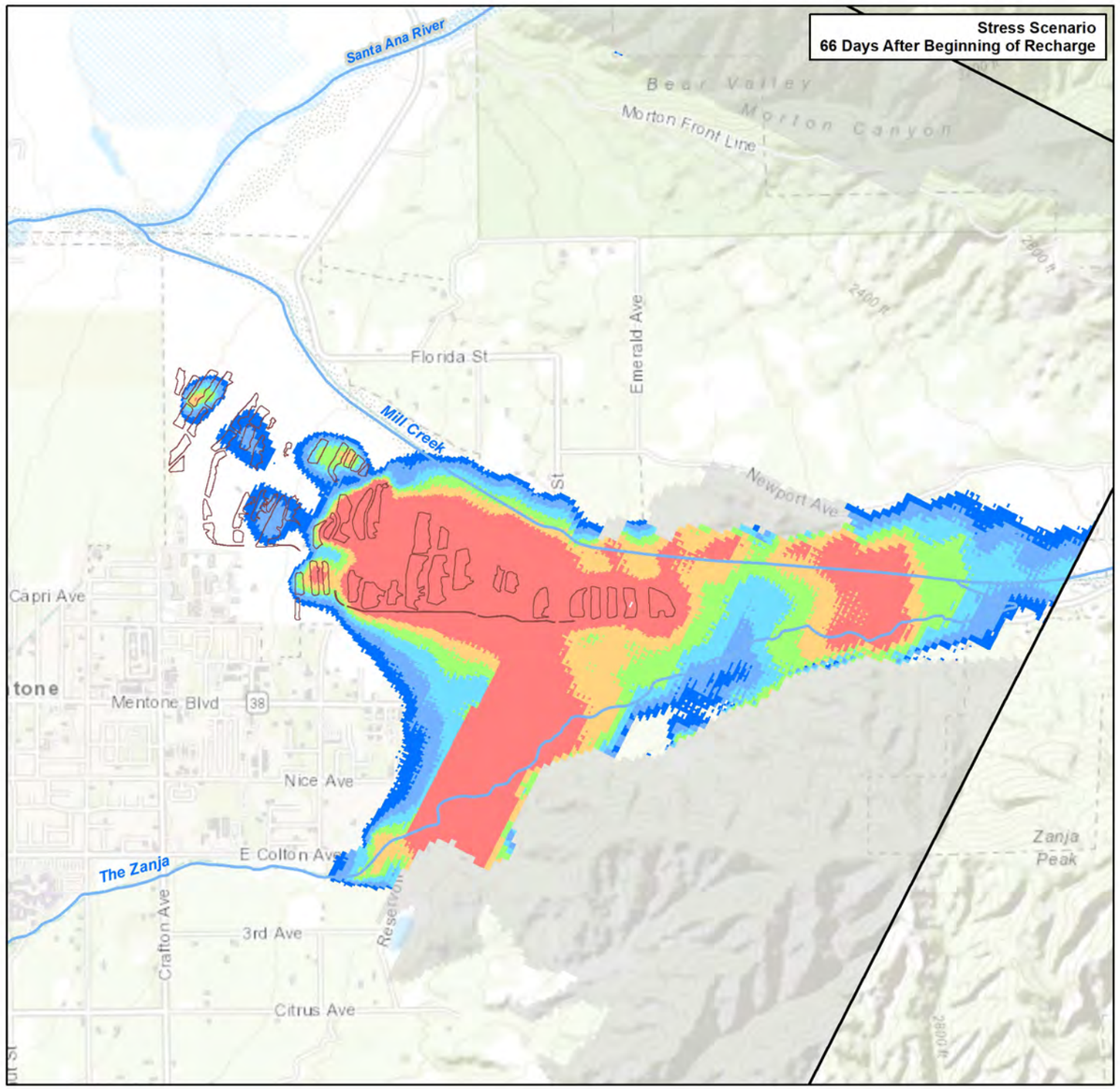
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GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

FIGURE 17

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EXPLANATION

- Focused Groundwater Model Boundary for Mill Creek Recharge Basins Evaluation
- Inactive Model Area
- Mill Creek Basins

Depth to Groundwater (<50 ft, bgs)

Flooding	20 - 30
0 - 10	30 - 40
10 - 20	40 - 50

N

0 1,800 3,600

Feet

Aug-25

AREA OF SHALLOW GROUNDWATER (<50 ft bgs) EXISTING CONDITIONS VS STRESS SCENARIO

GEOSCIENCE

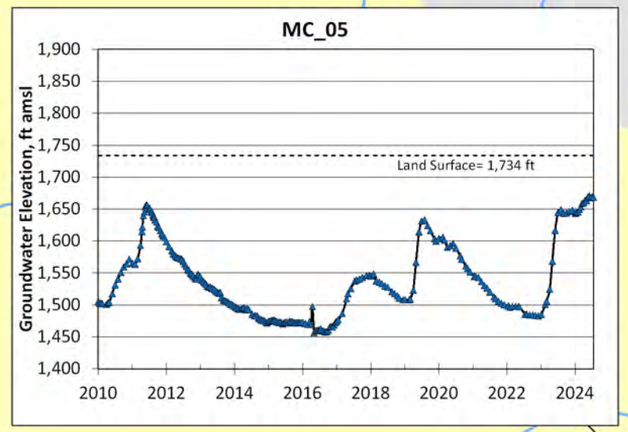
Q³ CONSULTING

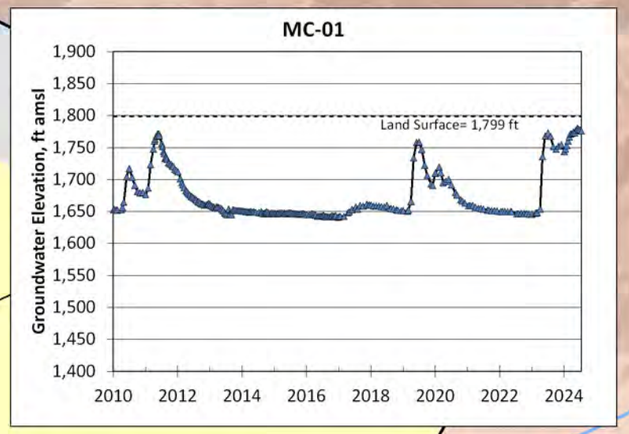
GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

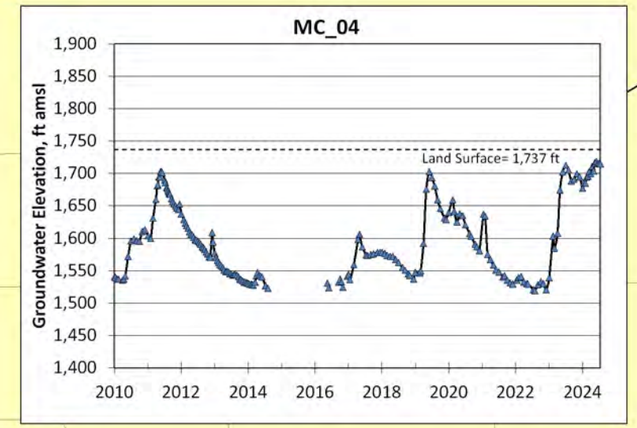
FIGURE 18

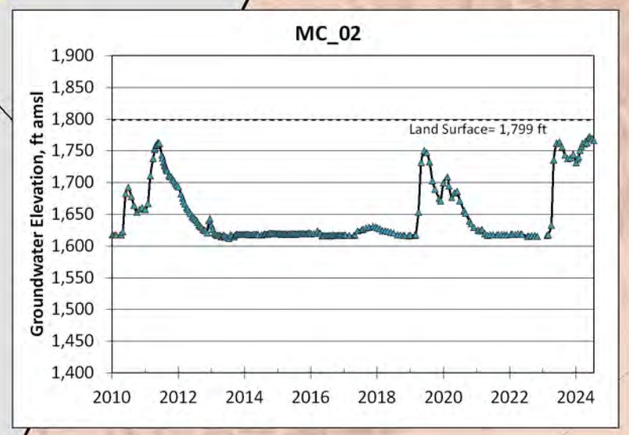
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Santa Ana River

Mill Creek

38

The Zanja

EXPLANATION

- Focused Groundwater Model Boundary for Mill Creek Recharge Basins Evaluation
- Active Model Area
- Inactive Model Area
- Mill Creek Basins
- SBVWCD Monitoring Well
- Measured Water Level

N

0 3,000 6,000

Feet

Aug-25

OBSERVED SHALLOW GROUNDWATER CONDITIONS IN MILL CREEK MONITORING WELLS

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GEOSCIENCE

Q³ CONSULTING

GROUNDWATER MODELING FOR MILL CREEK DIVERSION AND BASINS PROJECT FEASIBILITY STUDY

FIGURE 19

APPENDIX A

Tables of Model Outputs

For Three Model Runs Simulated

GEOSCIENCE

Appendix A

Table 1 - Focused Groundwater Model Water Balance for Existing Mill Creek Conditions

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
1	24	34	22	68	281	429	28	2	32	62	367	367
2	24	34	22	138	166	384	28	2	38	68	316	683
3	24	34	22	152	133	365	28	2	43	73	292	975
4	24	34	22	153	117	349	28	2	47	77	272	1,247
5	24	34	22	153	107	340	28	2	51	81	259	1,506
6	24	34	22	153	100	333	28	2	54	84	249	1,756
7	24	34	22	153	96	328	28	2	57	86	242	1,997
8	24	34	22	153	92	324	28	2	59	89	236	2,233
9	24	34	22	153	89	321	28	2	61	91	230	2,463
10	24	34	22	153	86	319	28	2	63	93	226	2,689
11	24	34	22	153	84	317	28	2	65	95	222	2,911
12	24	34	22	153	82	315	28	2	66	96	219	3,130
13	24	34	22	153	80	313	28	2	68	98	215	3,345
14	24	34	22	153	79	312	28	2	69	99	213	3,558
15	24	34	22	153	78	310	28	2	70	100	210	3,768
16	24	34	22	153	76	309	28	2	71	101	208	3,976
17	24	34	22	153	75	308	28	2	73	103	205	4,181
18	24	34	22	153	74	307	28	2	74	104	203	4,384
19	24	34	22	153	73	306	28	2	75	105	201	4,586
20	24	34	22	153	73	305	28	2	76	106	199	4,785
21	24	34	22	153	72	304	28	2	77	107	198	4,983
22	24	34	22	153	71	304	28	2	78	108	196	5,179
23	24	34	22	153	70	303	28	2	78	108	195	5,373
24	24	34	22	153	70	302	28	2	79	109	193	5,567
25	24	34	22	153	69	302	28	2	80	110	192	5,758
26	24	34	22	153	69	301	28	2	81	111	190	5,949
27	24	34	22	153	68	301	28	2	82	112	189	6,138
28	24	34	22	153	67	300	28	2	82	112	188	6,325
29	24	34	22	153	67	299	28	2	83	113	187	6,512
30	24	34	22	153	66	299	28	2	84	114	185	6,697
31	34	71	22	153	290	569	31	2	29	62	507	7,204
32	34	71	22	153	171	450	31	2	35	68	382	7,586
33	34	71	22	153	137	416	31	2	40	73	343	7,929
34	34	71	22	153	120	400	31	2	44	77	323	8,252
35	34	71	22	153	110	390	31	2	48	81	310	8,562

Appendix A

Table 1 - Focused Groundwater Model Water Balance for Existing Mill Creek Conditions

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
36	34	71	22	153	104	383	31	2	50	83	300	8,862
37	34	71	135	153	98	490	31	2	53	86	404	9,266
38	34	71	135	153	94	486	31	2	55	88	398	9,664
39	34	71	135	153	91	483	31	2	57	90	393	10,057
40	34	71	135	153	88	480	31	2	59	92	388	10,445
41	34	71	134	153	86	478	31	2	61	94	384	10,829
42	34	71	134	153	84	476	31	2	62	95	380	11,210
43	34	71	134	153	82	474	31	2	64	97	377	11,587
44	34	71	134	153	81	472	31	2	65	98	374	11,961
45	34	71	134	153	79	471	31	2	66	99	371	12,332
46	34	71	134	153	78	469	31	2	68	100	369	12,701
47	34	71	134	153	77	468	31	2	69	102	367	13,068
48	34	71	134	153	76	467	31	2	70	103	364	13,432
49	34	71	22	153	75	355	31	2	71	104	251	13,684
50	34	71	22	153	74	354	31	2	71	104	249	13,933
51	34	71	22	153	74	353	31	2	72	105	248	14,181
52	34	71	22	153	73	352	31	2	73	106	246	14,427
53	34	71	22	134	72	333	31	2	74	107	226	14,653
54	34	71	22	114	71	313	31	2	75	108	205	14,858
55	34	71	22	95	71	293	31	2	76	109	185	15,043
56	34	71	22	76	70	274	31	2	76	109	164	15,207
57	34	71	22	57	70	254	31	2	77	110	144	15,351
58	34	71	22	38	69	234	31	2	78	111	124	15,474
59	34	71	22	19	69	215	31	2	78	111	103	15,578
60	34	71	22	0	68	195	31	2	79	112	83	15,661
61	21	53	22	0	240	336	36	3	33	72	265	15,925
62	21	53	22	0	150	246	36	3	40	78	167	16,092
63	21	53	22	0	125	220	36	3	45	84	137	16,229
64	21	53	22	0	112	208	36	3	48	87	121	16,350
65	21	53	22	0	105	201	36	3	51	90	110	16,461
66	21	53	22	0	100	195	36	3	54	93	103	16,563
67	21	53	22	0	96	191	36	3	56	95	97	16,660
68	21	53	22	0	93	188	36	3	58	96	92	16,752
69	21	53	22	0	90	186	36	3	59	98	88	16,840
70	21	53	22	0	88	184	36	3	60	99	84	16,924

Appendix A

Table 1 - Focused Groundwater Model Water Balance for Existing Mill Creek Conditions

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
71	21	53	22	0	86	182	36	3	62	101	81	17,005
72	21	53	22	0	84	180	36	3	63	102	78	17,083
73	21	53	22	0	83	179	36	3	64	103	76	17,159
74	21	53	22	0	82	177	36	3	65	104	73	17,232
75	21	53	22	0	80	176	36	3	66	105	71	17,304
76	21	53	22	0	79	175	36	3	67	106	69	17,373
77	21	53	22	0	78	174	36	3	68	107	67	17,440
78	21	53	22	0	77	173	36	3	69	108	65	17,505
79	21	53	22	0	77	172	36	3	70	109	64	17,569
80	21	53	22	0	76	172	36	3	71	109	62	17,631
81	21	53	22	0	75	171	36	3	71	110	61	17,692
82	21	53	22	0	74	170	36	3	72	111	59	17,751
83	21	53	22	0	74	170	36	3	73	112	58	17,809
84	21	53	22	0	73	169	36	3	73	112	57	17,866
85	21	53	22	0	73	168	36	3	74	113	55	17,921
86	21	53	22	0	72	168	36	3	75	114	54	17,975
87	21	53	22	0	72	167	36	3	75	114	53	18,028
88	21	53	22	0	71	167	36	3	76	115	52	18,080
89	21	53	22	0	71	166	36	3	77	116	51	18,130
90	21	53	22	0	70	166	36	3	77	116	50	15,400
91	15	66	22	0	159	262	69	4	52	125	137	15,611
92	15	66	22	0	115	218	69	4	58	131	87	15,665
93	15	66	22	0	102	206	69	4	61	134	71	15,732
94	15	66	22	0	96	199	69	4	64	137	62	15,987
95	15	66	22	0	92	195	69	4	66	139	56	16,149
96	15	66	22	0	89	192	69	4	67	140	52	16,281
97	15	66	22	0	87	190	69	4	69	142	49	16,399
98	15	66	22	0	85	188	69	4	70	143	46	16,506
99	15	66	22	0	84	187	69	4	71	144	43	16,607
100	15	66	22	0	83	186	69	4	71	144	41	16,701
101	15	66	22	0	81	185	69	4	72	145	39	16,791
102	15	66	22	0	80	184	69	4	73	146	37	16,877
103	15	66	22	0	80	183	69	4	74	147	36	16,960
104	15	66	22	0	79	182	69	4	75	148	34	17,040
105	15	66	22	0	78	181	69	4	75	148	33	17,116

Table 1 - Focused Groundwater Model Water Balance for Existing Mill Creek Conditions

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
106	15	66	22	0	77	180	69	4	76	149	32	17,191
107	15	66	22	0	77	180	69	4	76	149	30	17,263
108	15	66	22	0	76	179	69	4	77	150	29	17,333
109	15	66	22	0	76	179	69	4	78	151	28	17,401
110	15	66	22	0	75	178	69	4	78	151	27	17,467
111	15	66	22	0	75	178	69	4	79	152	26	17,531
112	15	66	22	0	74	177	69	4	79	152	25	17,594
113	15	66	22	0	74	177	69	4	80	153	24	17,655
114	15	66	22	0	73	176	69	4	80	153	23	17,715
115	15	66	22	0	73	176	69	4	81	154	22	17,774
116	15	66	22	0	72	176	69	4	81	154	21	17,831
117	15	66	22	0	72	175	69	4	82	155	20	17,886
118	15	66	22	0	72	175	69	4	82	155	20	17,941
119	15	66	22	0	71	174	69	4	82	156	19	17,994
120	15	66	22	0	71	174	69	4	83	156	18	18,046
Daily Average	**23**	**56**	**34**	**70**	**90**	**272**	**41**	**3**	**67**	**111**	**161**	
120-day Total	**2,783**	**6,711**	**4,024**	**8,371**	**10,758**	**32,647**	**4,912**	**332**	**8,033**	**13,278**	**19,370**	

Table 1 - Focused Groundwater Model Water Balance for FIRO Scenario

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
1	24	34	22	97	281	459	28	2	32	62	397	397
2	24	34	22	163	166	409	28	2	38	68	341	738
3	24	34	22	195	133	408	28	2	43	73	335	1,073
4	24	34	22	210	117	407	28	2	47	77	330	1,403
5	24	34	22	219	107	406	28	2	51	81	325	1,729
6	24	34	22	233	101	413	28	2	54	84	329	2,058
7	24	34	22	236	96	412	28	2	57	87	325	2,383
8	24	34	22	239	92	411	28	2	59	89	322	2,705
9	24	34	22	239	89	408	28	2	61	91	317	3,022
10	24	34	22	240	86	406	28	2	63	93	313	3,335
11	24	34	22	240	84	404	28	2	65	95	309	3,644
12	24	34	22	240	82	402	28	2	66	96	306	3,950
13	24	34	22	240	80	400	28	2	68	98	303	4,253
14	24	34	22	240	79	399	28	2	69	99	300	4,552
15	24	34	22	240	78	397	28	2	70	100	297	4,850
16	24	34	22	240	76	396	28	2	71	101	295	5,144
17	24	34	22	240	75	395	28	2	73	103	293	5,437
18	24	34	22	240	74	394	28	2	74	104	290	5,727
19	24	34	22	240	73	393	28	2	75	105	288	6,016
20	24	34	22	240	73	392	28	2	76	106	287	6,302
21	24	34	22	240	72	392	28	2	77	107	285	6,587
22	24	34	22	240	71	391	28	2	78	108	283	6,871
23	24	34	22	240	70	390	28	2	78	108	282	7,152
24	24	34	22	240	70	390	28	2	79	109	280	7,433
25	24	34	22	240	69	389	28	2	80	110	279	7,711
26	24	34	22	240	69	388	28	2	81	111	277	7,989
27	24	34	22	240	68	388	28	2	82	112	276	8,265
28	24	34	22	240	67	387	28	2	82	112	275	8,540
29	24	34	22	240	67	387	28	2	83	113	274	8,814
30	24	34	22	240	66	386	28	2	84	114	273	9,086
31	34	71	22	240	290	657	31	2	29	62	594	9,681
32	34	71	22	240	171	537	31	2	35	68	469	10,150
33	34	71	22	240	137	504	31	2	40	73	430	10,580
34	34	71	22	240	120	487	31	2	44	77	410	10,990
35	34	71	22	240	110	477	31	2	48	81	397	11,387

Table 1 - Focused Groundwater Model Water Balance for FIRO Scenario

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
36	34	71	22	240	104	471	31	2	50	83	387	11,774
37	34	71	135	240	98	578	31	2	53	86	492	12,265
38	34	71	135	240	94	573	31	2	55	88	485	12,750
39	34	71	135	240	91	570	31	2	57	90	480	13,230
40	34	71	134	240	88	567	31	2	59	92	475	13,705
41	34	71	134	240	86	565	31	2	61	94	471	14,177
42	34	71	134	240	84	563	31	2	62	95	468	14,644
43	34	71	134	240	82	561	31	2	64	97	464	15,108
44	34	71	134	240	81	559	31	2	65	98	461	15,570
45	34	71	134	240	79	558	31	2	66	99	459	16,028
46	34	71	134	240	78	557	31	2	68	101	456	16,484
47	34	71	134	240	77	555	31	2	69	102	454	16,938
48	34	71	134	240	76	554	31	2	70	103	452	17,390
49	34	71	22	240	75	442	31	3	71	104	338	17,728
50	34	71	22	240	74	441	31	3	71	104	337	18,065
51	34	71	22	240	74	440	31	3	72	105	335	18,400
52	34	71	22	240	73	440	31	3	73	106	333	18,733
53	34	71	22	210	72	409	31	3	74	107	302	19,035
54	34	71	22	180	71	378	31	3	75	108	270	19,305
55	34	71	22	150	71	348	31	3	76	109	239	19,545
56	34	71	22	120	70	317	31	3	76	109	208	19,752
57	34	71	22	90	70	287	31	3	77	110	176	19,929
58	34	71	22	60	69	256	31	3	78	111	145	20,074
59	34	71	22	30	69	226	31	3	78	112	114	20,188
60	34	71	22	0	68	195	31	3	79	112	83	20,271
61	21	53	22	0	240	336	36	3	33	72	264	20,535
62	21	53	22	0	150	246	36	3	40	79	167	20,702
63	21	53	22	0	125	220	36	3	45	84	137	20,839
64	21	53	22	0	112	208	36	3	48	87	121	20,960
65	21	53	22	0	105	201	36	3	51	90	110	21,070
66	21	53	22	0	100	195	36	3	54	93	103	21,172
67	21	53	22	0	96	191	36	3	56	95	97	21,269
68	21	53	22	0	93	188	36	3	58	97	92	21,361
69	21	53	22	0	90	186	36	3	59	98	88	21,448
70	21	53	22	0	88	184	36	3	60	100	84	21,532

Table 1 - Focused Groundwater Model Water Balance for FIRO Scenario

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
71	21	53	22	0	86	182	36	3	62	101	81	21,613
72	21	53	22	0	84	180	36	3	63	102	78	21,691
73	21	53	22	0	83	179	36	3	64	103	76	21,767
74	21	53	22	0	82	177	36	3	65	104	73	21,840
75	21	53	22	0	80	176	36	3	66	105	71	21,911
76	21	53	22	0	79	175	36	3	67	106	69	21,980
77	21	53	22	0	78	174	36	3	68	107	67	22,047
78	21	53	22	0	78	173	36	3	69	108	65	22,112
79	21	53	22	0	77	172	36	3	70	109	64	22,176
80	21	53	22	0	76	172	36	3	70	110	62	22,238
81	21	53	22	0	75	171	36	3	71	110	61	22,299
82	21	53	22	0	74	170	36	3	72	111	59	22,358
83	21	53	22	0	74	170	36	3	73	112	58	22,416
84	21	53	22	0	73	169	36	3	73	113	56	22,472
85	21	53	22	0	73	168	36	3	74	113	55	22,527
86	21	53	22	0	72	168	36	3	75	114	54	22,581
87	21	53	22	0	72	167	36	3	75	115	53	22,634
88	21	53	22	0	71	167	36	3	76	115	52	22,685
89	21	53	22	0	71	166	36	3	77	116	50	22,736
90	21	53	22	0	70	166	36	3	77	116	49	22,785
91	15	66	22	0	159	262	69	4	52	126	136	22,921
92	15	66	22	0	115	218	69	4	58	131	87	23,008
93	15	66	22	0	102	205	69	4	61	135	71	23,079
94	15	66	22	0	96	199	69	4	64	137	62	23,141
95	15	66	22	0	92	195	69	4	66	139	56	23,197
96	15	66	22	0	89	192	69	4	67	141	52	23,249
97	15	66	22	0	87	190	69	4	69	142	48	23,298
98	15	66	22	0	85	188	69	4	70	143	46	23,343
99	15	66	22	0	84	187	69	4	71	144	43	23,386
100	15	66	22	0	83	186	69	4	71	145	41	23,427
101	15	66	22	0	81	185	69	4	72	146	39	23,466
102	15	66	22	0	80	184	69	4	73	146	37	23,503
103	15	66	22	0	80	183	69	4	74	147	36	23,539
104	15	66	22	0	79	182	69	4	75	148	34	23,573
105	15	66	22	0	78	181	69	4	75	148	33	23,606

Table 1 - Focused Groundwater Model Water Balance for FIRO Scenario

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
106	15	66	22	0	77	180	69	4	76	149	31	23,637
107	15	66	22	0	77	180	69	4	76	150	30	23,667
108	15	66	22	0	76	179	69	4	77	150	29	23,696
109	15	66	22	0	76	179	69	4	78	151	28	23,724
110	15	66	22	0	75	178	69	4	78	151	27	23,751
111	15	66	22	0	75	178	69	4	79	152	26	23,776
112	15	66	22	0	74	177	69	4	79	152	25	23,801
113	15	66	22	0	74	177	69	4	80	153	24	23,824
114	15	66	22	0	73	176	69	4	80	153	23	23,847
115	15	66	22	0	73	176	69	4	81	154	22	23,869
116	15	66	22	0	72	175	69	4	81	154	21	23,890
117	15	66	22	0	72	175	69	4	82	155	20	23,910
118	15	66	22	0	72	175	69	4	82	155	19	23,930
119	15	66	22	0	71	174	69	4	82	156	19	23,948
120	15	66	22	0	71	174	69	4	83	156	18	23,966
Daily Average	**23**	**56**	**34**	**108**	**90**	**310**	**41**	**3**	**67**	**111**	**200**	
120-day Total	**2,783**	**6,711**	**4,021**	**12,985**	**10,758**	**37,259**	**4,912**	**348**	**8,033**	**13,293**	**23,966**	

Table 1 - Focused Groundwater Model Water Balance for STRESS Scenario

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
1	24	34	22	99	281	459	28	2	32	62	397	397
2	24	34	22	193	166	439	28	2	38	68	371	768
3	24	34	22	253	133	465	28	2	43	73	392	1,160
4	24	34	22	295	117	491	28	2	47	77	414	1,574
5	24	34	22	314	107	500	28	2	51	81	419	1,993
6	24	34	22	325	101	505	28	2	54	84	421	2,415
7	24	34	22	329	96	504	28	2	57	87	418	2,832
8	24	34	22	331	92	502	28	2	59	89	413	3,245
9	24	34	22	331	89	499	28	2	61	91	408	3,653
10	24	34	22	332	86	497	28	2	63	93	404	4,058
11	24	34	22	332	84	495	28	2	65	95	400	4,458
12	24	34	22	332	82	493	28	2	66	96	397	4,855
13	24	34	22	332	80	491	28	2	68	98	394	5,249
14	24	34	22	332	79	490	28	2	69	99	391	5,640
15	24	34	22	332	78	489	28	2	70	100	388	6,028
16	24	34	22	332	76	487	28	2	71	101	386	6,414
17	24	34	22	332	75	486	28	2	73	103	384	6,797
18	24	34	22	332	74	485	28	2	74	104	382	7,179
19	24	34	22	332	73	484	28	2	75	105	380	7,559
20	24	34	22	332	73	484	28	2	76	106	378	7,936
21	24	34	22	332	72	483	28	2	77	107	376	8,312
22	24	34	22	332	71	482	28	2	78	108	374	8,687
23	24	34	22	332	70	481	28	2	78	108	373	9,060
24	24	34	22	332	70	481	28	2	79	109	371	9,431
25	24	34	22	332	69	480	28	2	80	110	370	9,801
26	24	34	22	332	69	479	28	2	81	111	369	10,170
27	24	34	22	332	68	479	28	2	82	112	367	10,537
28	24	34	22	332	67	478	28	2	82	112	366	10,903
29	24	34	22	332	67	478	28	2	83	113	365	11,268
30	24	34	22	332	66	477	28	2	84	114	364	11,631
31	34	71	22	332	290	749	31	2	29	62	686	12,318
32	34	71	22	332	171	629	31	2	35	68	561	12,879
33	34	71	22	332	137	596	31	2	40	73	522	13,401
34	34	71	22	332	120	579	31	2	44	77	502	13,903
35	34	71	22	332	110	569	31	2	48	81	489	14,391

Table 1 - Focused Groundwater Model Water Balance for STRESS Scenario

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
36	34	71	22	332	104	562	31	2	50	83	479	14,870
37	34	71	129	332	98	663	31	2	53	86	577	15,447
38	34	71	128	332	94	659	31	2	55	88	571	16,018
39	34	71	128	332	91	656	31	2	57	90	565	16,584
40	34	71	128	332	88	653	31	2	59	92	561	17,144
41	34	71	128	332	86	650	31	2	61	94	557	17,701
42	34	71	128	332	84	648	31	2	62	95	553	18,254
43	34	71	128	332	82	647	31	2	64	97	550	18,804
44	34	71	128	332	81	645	31	2	65	98	547	19,351
45	34	71	128	332	79	644	31	2	66	99	544	19,896
46	34	71	128	332	78	642	31	2	68	101	542	20,437
47	34	71	128	332	77	641	31	2	69	102	539	20,977
48	34	71	128	332	76	640	31	2	70	103	537	21,514
49	34	71	22	332	75	534	31	3	71	104	430	21,944
50	34	71	22	332	74	533	31	3	71	104	429	22,373
51	34	71	22	332	74	532	31	3	72	105	427	22,799
52	34	71	22	332	73	532	31	3	73	106	425	23,225
53	34	71	22	290	72	489	31	3	74	107	382	23,607
54	34	71	22	249	71	447	31	3	75	108	339	23,947
55	34	71	22	207	71	405	31	3	76	109	297	24,243
56	34	71	22	166	70	363	31	3	76	109	254	24,497
57	34	71	22	124	70	321	31	3	77	110	211	24,708
58	34	71	22	83	69	279	31	3	78	111	168	24,876
59	34	71	22	41	69	237	31	3	78	112	126	25,002
60	34	71	22	0	68	195	31	3	79	112	83	25,084
61	21	53	22	0	240	336	36	3	33	72	264	25,349
62	21	53	22	0	150	246	36	3	40	79	167	25,516
63	21	53	22	0	125	220	36	3	45	84	137	25,653
64	21	53	22	0	112	208	36	3	48	87	121	25,774
65	21	53	22	0	105	201	36	3	51	90	110	25,884
66	21	53	22	0	100	195	36	3	54	93	103	25,986
67	21	53	22	0	96	191	36	3	56	95	97	26,083
68	21	53	22	0	93	188	36	3	58	97	92	26,175
69	21	53	22	0	90	186	36	3	59	98	88	26,263
70	21	53	22	0	88	184	36	3	60	99	84	26,347

Table 1 - Focused Groundwater Model Water Balance for STRESS Scenario

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
71	21	53	22	0	86	182	36	3	62	101	81	26,428
72	21	53	22	0	84	180	36	3	63	102	78	26,506
73	21	53	22	0	83	179	36	3	64	103	76	26,581
74	21	53	22	0	82	177	36	3	65	104	73	26,655
75	21	53	22	0	80	176	36	3	66	105	71	26,726
76	21	53	22	0	79	175	36	3	67	106	69	26,795
77	21	53	22	0	78	174	36	3	68	107	67	26,862
78	21	53	22	0	78	173	36	3	69	108	65	26,927
79	21	53	22	0	77	172	36	3	70	109	64	26,991
80	21	53	22	0	76	172	36	3	70	110	62	27,053
81	21	53	22	0	75	171	36	3	71	110	61	27,114
82	21	53	22	0	75	170	36	3	72	111	59	27,173
83	21	53	22	0	74	170	36	3	73	112	58	27,231
84	21	53	22	0	73	169	36	3	73	112	56	27,287
85	21	53	22	0	73	168	36	3	74	113	55	27,342
86	21	53	22	0	72	168	36	3	75	114	54	27,396
87	21	53	22	0	72	167	36	3	75	114	53	27,449
88	21	53	22	0	71	167	36	3	76	115	52	27,501
89	21	53	22	0	71	166	36	3	77	116	51	27,551
90	21	53	22	0	70	166	36	3	77	116	50	27,601
91	15	66	22	0	159	262	69	4	52	126	137	27,737
92	15	66	22	0	115	218	69	4	58	131	87	27,825
93	15	66	22	0	102	206	69	4	61	135	71	27,895
94	15	66	22	0	96	199	69	4	64	137	62	27,958
95	15	66	22	0	92	195	69	4	66	139	56	28,014
96	15	66	22	0	89	192	69	4	67	140	52	28,066
97	15	66	22	0	87	190	69	4	68	142	49	28,114
98	15	66	22	0	85	188	69	4	70	143	46	28,160
99	15	66	22	0	84	187	69	4	71	144	43	28,203
100	15	66	22	0	83	186	69	4	71	145	41	28,244
101	15	66	22	0	82	185	69	4	72	146	39	28,283
102	15	66	22	0	81	184	69	4	73	146	37	28,321
103	15	66	22	0	80	183	69	4	74	147	36	28,356
104	15	66	22	0	79	182	69	4	74	148	34	28,391
105	15	66	22	0	78	181	69	4	75	148	33	28,423

Appendix A

Table 1 - Focused Groundwater Model Water Balance for STRESS Scenario

Stress Period	Areal Recharge from Precipitation and Anthropogenic Return Flow	Mountain Front Recharge	Streambed Percolation	Mill Creek Basins Infiltration	Underflow Inflow from ISARM	Total Inflow	Groundwater Pumping	Evapotranspiration	Underflow Outflow to ISARM	Total Outflow	Change in Storage	Cumulative Change in Storage
	acre-ft per simulation stress-period						acre-ft per simulation stress-period				Simulated Total acre-ft	
106	15	66	22	0	77	180	69	4	76	149	31	28,455
107	15	66	22	0	77	180	69	4	76	150	30	28,485
108	15	66	22	0	76	179	69	4	77	150	29	28,514
109	15	66	22	0	76	179	69	4	78	151	28	28,542
110	15	66	22	0	75	178	69	4	78	151	27	28,569
111	15	66	22	0	75	178	69	4	79	152	26	28,594
112	15	66	22	0	74	177	69	4	79	152	25	28,619
113	15	66	22	0	74	177	69	4	80	153	24	28,643
114	15	66	22	0	73	176	69	4	80	153	23	28,666
115	15	66	22	0	73	176	69	4	81	154	22	28,688
116	15	66	22	0	72	175	69	4	81	154	21	28,709
117	15	66	22	0	72	175	69	4	82	155	20	28,729
118	15	66	22	0	72	175	69	4	82	155	19	28,749
119	15	66	22	0	71	174	69	4	82	156	19	28,767
120	15	66	22	0	71	174	69	4	83	156	18	28,785
Daily Average	**23**	**56**	**33**	**149**	**90**	**351**	**41**	**3**	**67**	**111**	**240**	
120-day Total	**2,783**	**6,711**	**3,924**	**17,896**	**10,761**	**42,075**	**4,912**	**348**	**8,030**	**13,290**	**28,785**	

ATTACHMENT V

Mill Creek Basin Cost Technical Memorandum

San Bernardino Valley Water Conservation District

Mill Creek Basin Cost Technical Memorandum



San Bernardino County, California

July 2025

Prepared for:

San Bernardino Valley
Water Conservation District
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1 INTRODUCTION

This report summarizes the results of a preliminary cost analysis for three alternative proposed improvements for the Active Recharge Transfer Project (ARTP) Mill Creek basins. This cost estimate is part of a larger study to evaluate current and potential recharge yield for various improvements and to identify a cost-benefit relationship to support the San Bernardino Valley Water Conservation District (District) with information to garner a recommendation.

1.1 Existing Mill Creek Basins Operations

The existing Mill Creek Basin location includes 60 retention basins. Additional locations also retain water during large overflow events but are not considered part of the typical operations. The current system has three main flow inlet locations: South Intake (Mill Creek), North Intake (Mill Creek), and the Redlands Aqueduct.

The main Mill Creek diversion occurs at the South Intake, which consists of a reinforced concrete control structure that diverts most flows to the eastern portion of the basins, and some flows west into the North Intake. The Redlands Aqueduct diverts Santa Ana River water to the western basins through a controlled structure.

The basins currently operate through a series of weirs, channels and culverts. The weirs allow for raising, lowering and diversion of flows from one basin to another to allow for optimization and/or taking basins offline for maintenance. Some of the existing diversion structures are natural features set in place by District staff to divert flows.

1.1.1 Mill Creek Basins Source Flows

Flows from Mill Creek are controlled at the South Intake. Flows that reach the intake structure are further controlled upstream within the creek via "soft plugs". Soft plugs are natural berms of bed material graded by the District to divert low flows towards the South Intake system. The soft plugs vary in size from 2 to 4 feet and have an estimated capacity of up to 400 cfs. During large flow events within the creek, these earthen structures overtop and allow creek flows to continue downstream. After large events, District staff reestablish the "soft plugs".

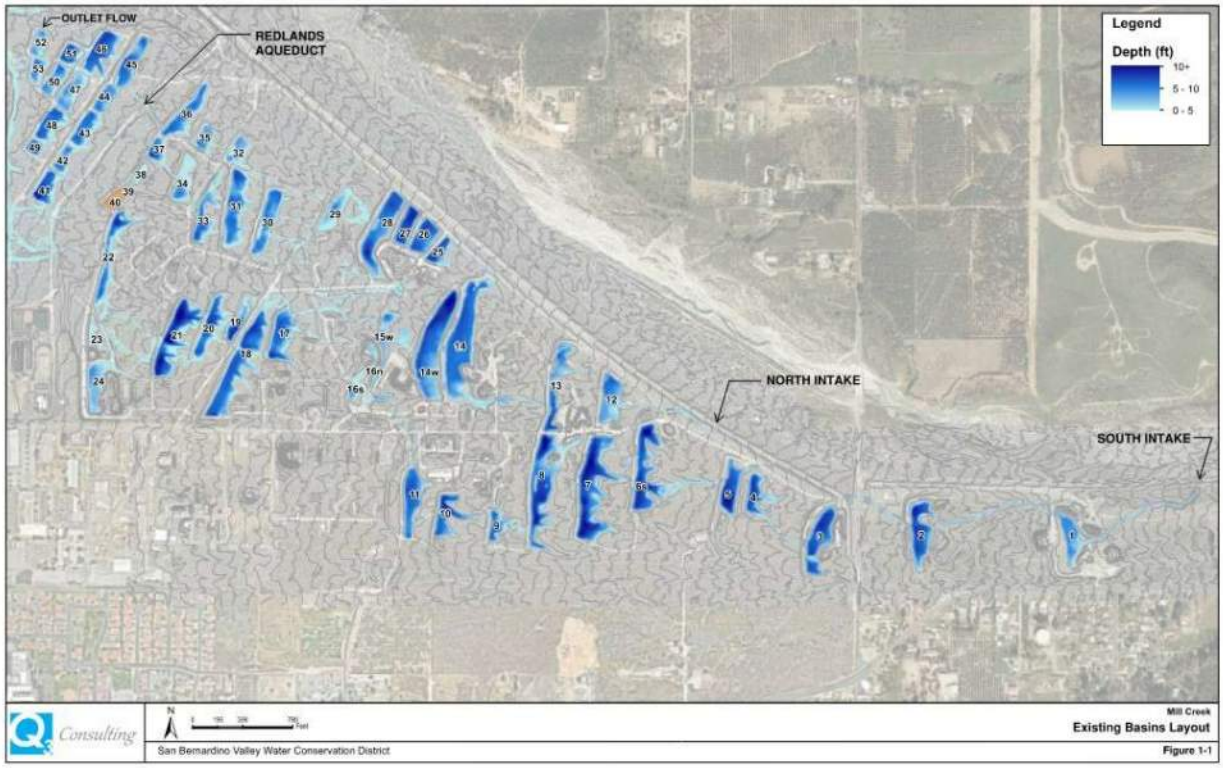
The Redlands Aqueduct diverts flow from the Santa Ana River at a rate of up to 25 cfs. Over the past two years, flows through this structure have averaged 13 cfs. discharging directly into Basin #45. Future allocations suggest that 25 cfs can be delivered during wet years.

1.1.2 Existing Basin System Characteristics

The Mill Creek basins are configured to have desilting basins immediately downstream of the intake structures. Basin 1 and Basin 2 are considered desilting basins for the South Intake (See Figure 2-1) for existing basin configurations. Basin 12 is the desilting basin for the North Intake. The basins downstream from the Redlands Aqueduct are not considered detention basins, as the flows from the aqueduct are relatively clean.

Currently the system of basins operate by using a series of weirs and man-made earthen diversions that allow the District the ability to maintain certain basins. As part of the maintenance, the District removes sediment, primarily from the desilting basins, and stockpile the material adjacent to the basins. Basin inverts are scarified to help promote good characteristics of percolation.

Figure 1-1. Existing Basins Layout



2 ALTERNATIVE EVALUATIONS

Three alternatives basin configurations were hydraulically assessed in this project,

1) Alternative 1,
2) Alternative 2, and
3) Alternative 3.

Each of these configurations were evaluated using varying inflow parameters, as discussed in the *Mill Creek Basin Hydraulic Technical Memorandum*. These alternatives were compared to the existing condition hydraulics for future evaluation in the feasibility study, currently being prepared by the District.

2.1 Alternative 1

Alternative 1 includes proposed improvement to multiple basins and the addition of a bypass channel (South Canal) to allow for flows to be redirected if needed during maintenance operations. A list of basins to be improved can be seen in Table 2-1.

Table 2-1. Alternative 1 Proposed Basin Modifications

Existing Basin Number	New Basin Number	Modifications
1	1	Increase Volume
None	1A	New Basin
None	1B	New Basin
None	1C	New Basin
2	2	Increase Volume
3	3	Increase Volume
4	4	Increase Volume
6S	6S	Increase Volume
7	7	Increase Volume
8	8	Increase Volume
9	9	Increase Volume
10	10	Increase Volume
11	11	Increase Volume
13	13	Increase Volume
14	14w	Increase Volume
15w	15w	Increase Volume
16N	16N	Increase Volume
16S	16S	Increase Volume
18	18	Increase Volume
24	24	Increase Volume
23	23	Increase Volume
22	22	Increase Volume
38-40	38-40	Combine and Increase Volume
36-37	36-37	Combine and Increase Volume
34	34	Increase Volume
35	35	Increase Volume
29	29	Increase Volume

42-43	42-43	Combine and Increase Volume
44-45	44-45	Combine and Increase Volume
46-47	46-47	Combine and Increase Volume
50-51	50-51	Combine and Increase Volume
52-53	52-53	Combine and Increase Volume

Figure 2-1 shows the proposed improvements for Alternative 1. This figure also shows the locations for diversion structures, the South Canal and its proposed connections to various basins, and improved basin outlet structures.

2.2 Alternative 2

Alternative 2 includes all of the proposed improvements for Alternative 1, including three (3) new basins and improvements to two others. A list of improvements beyond those identified in Alternative 2 can be seen in Table 2-2.

Table 2-2. Alternative 2 Proposed Basin Modifications

Existing Basin Number	New Basin Number	Modifications
None	24E	New Basin
None	41	New Basin
None	41S	New Basin
29	29	Increase Volume
31	31	Increase Volume

Alternative 2 contains these improvements, in addition to construction of a South Canal to help route flows to various locations within the basins and the ability to greatly reduce the number of basins that can be pulled off-line during maintenance.

Figure 2-2 shows the proposed improvements for Alternative 2. This figure also shows the locations for diversion structures, the South Canal and its proposed connections to various basins, and improved basin outlet structures.

2.3 Alternative 3

Alternative 3 includes all of the proposed improvements for Alternative 2, including four (4) new basins. A list of improvements beyond those identified in Alternative 3 can be seen in Table 2-3.

Table 2-3. Alternative 2 Proposed Basin Modifications

Existing Basin Number	New Basin Number	Modifications
None	SW1	New Basin
None	SW2	New Basin
None	SW3	New Basin
None	SW4	New Basin

Alternative 3 contains these improvements, in addition to construction of a South Canal to help route flows to various locations within the basins and the ability to greatly reduce the number of basins that can be pulled off-line during maintenance.

Figure 2-3 shows the proposed improvements for Alternative 3. This figure also shows the locations for diversion structures, the South Canal and its proposed connections to various basins, and improved basin outlet structures.

Figure 2-1. Alternative 1 – Proposed Improvements

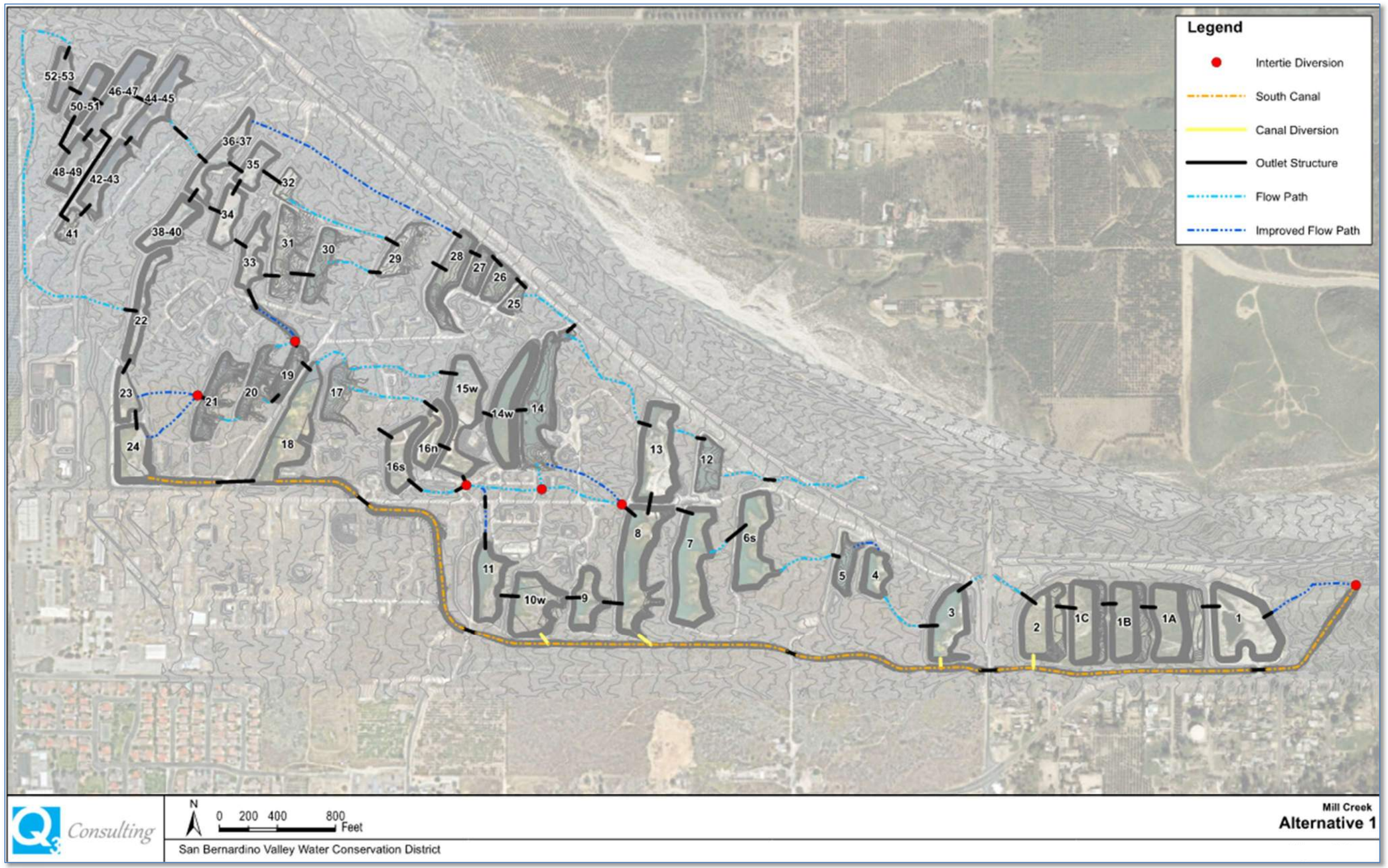


Figure 2-2. Alternative 2 – Proposed Improvements

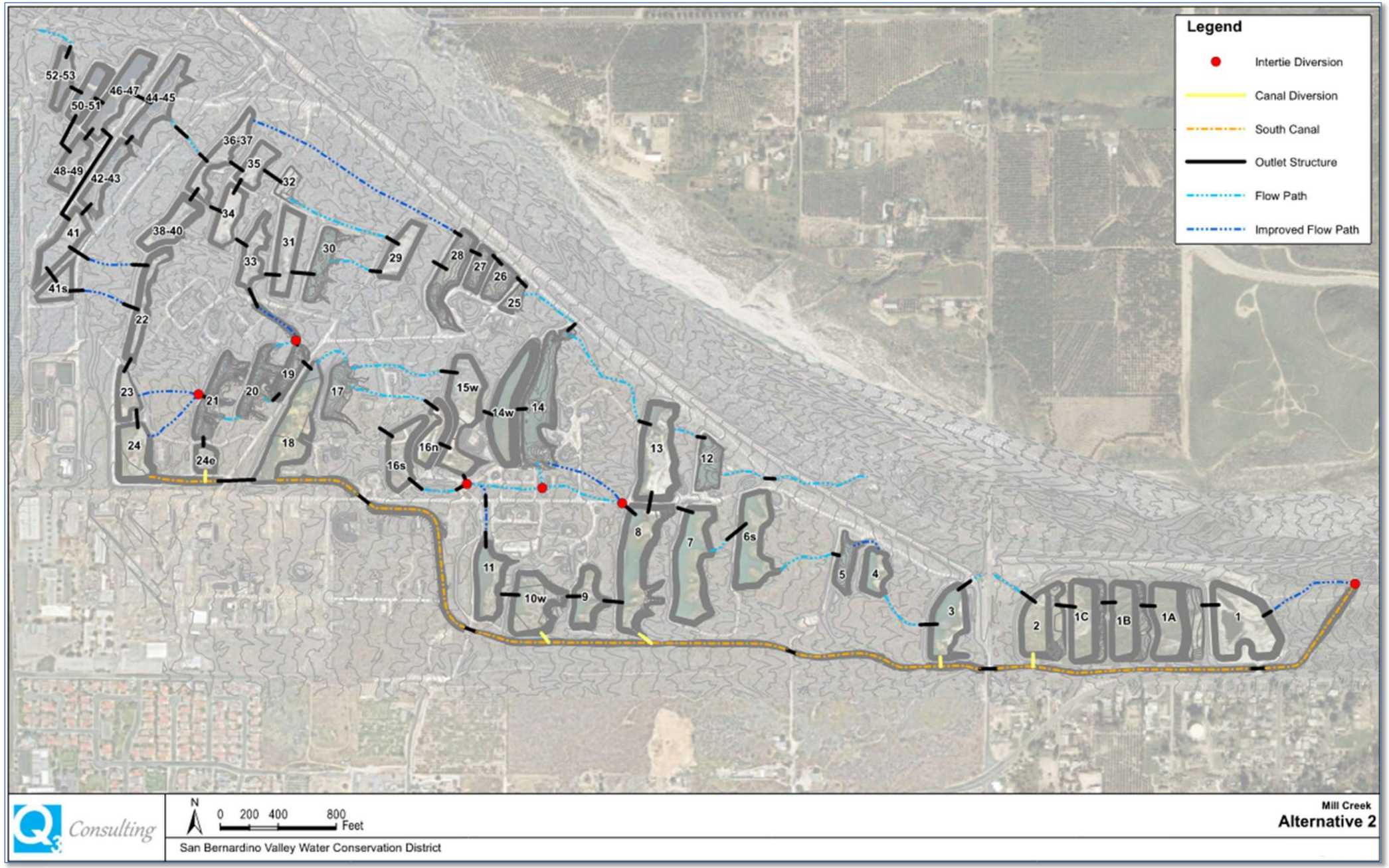
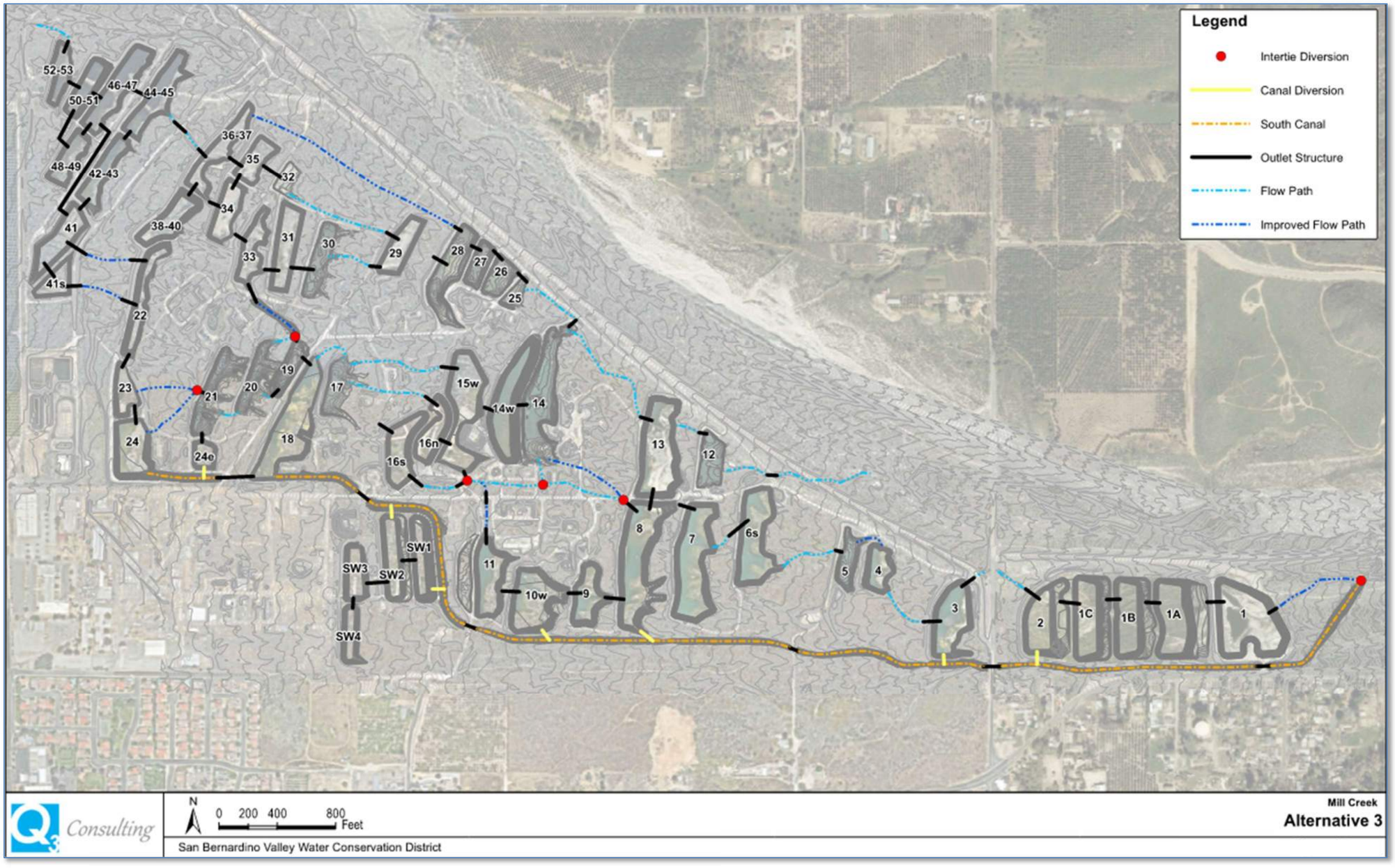


Figure 2-3. Alternative 3 - Proposed Improvements



3 PRELIMINARY COST ESTIMATES

A rough order magnitude preliminary engineering construction cost estimate was developed for each of the three alternatives. A list of unit prices are shown in Table 3-1 below based on recent District projects and other related projects within southern California. Some of the items in the table below are still being developed. For example, "Fill" is divided into 90% and 85% depending on location within the vicinity of the recharge basins. At the time of this Draft report, limits of 90-% compaction are still being developed.

Table 3-1. Preliminary Unit Prices

ITEM	UNIT	UNIT COST
MOBILIZATION	LS	$490,000-$550,000
DEMOLITION	LS	$0.00
CLEAR AND GRUB	AC	$1,000.00
EXCAVATION (BASINS & CANAL)	CY	$5.00
BACKFILL (BASINS & CANAL) Non-Compacted	CY	$12.00
STRUCTURE EXCAVATION	CY	$55.00
EXPORT	CY	$15.00
SPILLWAY (Basins 1, 2A-2D only)	EA	$50,000.00
12' x 6' RCB (incl wingwalls and street repair)	CY	$4,000.00
OULET STRUCTURE	EA	$40,000.00
INLET STRUCTURE (Inc. Weirs)	EA	$50,000.00
FLOW METER (INCL STRUCTURE)	EA	$100,000.00
SLUICE GATE	EA	$50,000
¼ TON RIPRAP (NON-CONCRETED)	CY	$200.00
DIVERSION GATES WITHIN INNERTIES	EA	$50,000.00
EROSION CONTROL / SWPPP	LS	$50,000.00
UTILITIES	LS	$500,000.00
SOUTH CANAL DIVERSION STRUCTURES	EA	$400,000.00
SCADA	LS	$500,000.00

Contingencies were calculated based on the items listed in Table 3-2 below. A total project contingency was identified as 55-percent.

Table 3-2. Preliminary Construction Contingencies

ITEM	CONTINGENCY
GENERAL CONTINGENCY	30%
DESIGN and PERMITTING	10%
HABITAT MITIGATION MEASURES (5-6%)	$3,000,000
CONSTRUCTION ADMIN	15%

Below is the list preliminary cost estimates for each of the Alternatives. Table 3-3 shows the total construction costs, including contingencies listed above. Although some details are still being developed,

the costs should provide a reasonable insight to the project costs. For a detailed breakdown of costs, refer to Addendum 4 in the report.

Table 3-3. Preliminary Alternative Project Cost Estimates

ALTERNATIVE	COST
Alternative 1	$73,800,000
Alternative 2	$78,200,000
Alternative 3	$ 84,000,000