ALA WAI CANAL FLOOD RISK MANAGEMENT STUDY
O'AHU, HAWAI'I

FINAL FEASIBILITY STUDY REPORT WITH INTEGRATED ENVIRONMENTAL IMPACT STATEMENT

APPENDIX A
HYDROLOGY AND HYDRAULIC ENGINEERING

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Ala Wai Canal Project
Feasibility Study
Honolulu, Hawaii

Existing Without-Project Hydrologic Appendix

Appendix A1

U.S. ARMY CORPS OF ENGINEERS
HONOLULU DISTRICT
FORT SHAFTER, HAWAII

February 2017

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Existing Without-Project Hydrologic Appendix

Ala Wai Canal Project

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Abbreviations and Acronyms

A	Ala Wai (designating sub-watershed)
BWS	Board of Water Supply
City	City and County of Honolulu
cfs	cubic feet per second
CN	curve number
USACE	United States Army Corps of Engineers
DLNR	State of Hawaiʻi, Department of Land and Natural Resources
FEMA	Federal Emergency Management Agency
GIS	geographical information system
HEC-HMS	Hydrologic Engineering Center-Hydrologic Modeling System
HEC-SSP	Hydrologic Engineering Center-Statistical Software Package
HEC-GeoHMS	Hydrologic Engineering Center Geospatial Hydrologic Modeling Extension
HEC-GeoRAS	Hydrologic Engineering Center Geospatial River Analysis System Extension
HSG	hydrologic soil group
IDF	Intensity-duration-frequency
ID	Identification number
J	junction
K	Makiki (designating sub-watershed)
LiDAR	light detection and ranging
M	Mānoa (designating sub-watershed)
MP	Mānoa-Pālolo (designating sub-watershed)
MSL	Mean sea level
MWP	Mānoa Watershed Project
NOAA	National Oceanit and Atmospheric Agency
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
P	Pālolo (designating sub-watershed)
SCS	Soil Conservation Service
T_c	time of concentration
TR-55	Technical Release 55
UHM	University of Hawaiʻi at Mānoa
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
W	Waikīkī (designating sub-watershed)

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Executive Summary

Introduction

In 2001, the United States Army Corps of Engineers (USACE) recommended flood mitigation and ecosystem restoration measures for the Ala Wai Watershed, located on the southeast sector of the island of Oʻahu, Hawaiʻi. As part of this larger goal, USACE contracted Oceanit to develop a Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) for a range of potential storms in the Ala Wai Watershed. HEC-HMS is the USACE hydrologic model. The purpose of this study was to estimate peak flow discharges at particular drainage junctions in the Ala Wai Watershed corresponding to the following storm return periods: 2-, 5-, 10-, 20-, 50-, 100-, 200-, and 500-year. These storm return periods correlate to storm chance exceedance probabilities of 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent, respectively.

Purpose

Whereas this study focuses on the HEC-HMS model, this study uses a total of five different methods to estimate peak flow discharges throughout the Ala Wai Watershed for potential storms ranging in duration and intensity. Estimated peak flow discharges are based on the existing conditions of the Ala Wai Watershed's sub-watersheds of Makiki, Mānoa, and Pālolo valleys; Mānoa- Pālolo and Ala Wai Canals; and Waikīkī. Discharge at junctions of interest throughout these sub-watersheds was studied. Oceanit modeled storms using both rainfall-runoff and peak flow frequency methods for a range of storm scenarios, as follows. The study (1) researched and collected relevant hydrologic data; (2) constructed and calibrated both rainfall-runoff and peak flow frequency hydrologic models; and (3) weighted and compared the results from these models to arrive at estimated peak flow discharges.

Study Area

The Ala Wai Watershed encompasses a drainage area of 10,400 acres (16.2 square miles) of area that are economically significant and densely populated. The existing conditions throughout the Ala Wai Watershed are relevant to its hydrologic analysis, including the character of the watershed's overall climate, topography, geology, vegetation, land use and cover, and water resources. Hawaiʻi's high moisture, orographic rainfall, and northeasterly trade winds create wet conditions in the upper Ala Wai Watershed. The topography of the upper Ala Wai watershed is relatively steep and stony that, in combination with heavy rainfall, provides conditions prone to flash flooding. The lower Ala Wai watershed has finer well-drained soil, but much of it is urbanized, meaning its terrain surfaces are impervious. In terms of streams, the Makiki, Mānoa, and Pālolo streams drain their respective sub-watersheds. Mānoa and Pālolo streams combine to form the Mānoa-Pālolo Canal that empties into the Ala Wai Canal. Runoff and drainage from Waikīkī empties into the Ala Wai Canal as well.

Data Collection Procedure

Data collection for hydrologic analysis included rainfall gage data, stream flow gage data, records of historical storms, maps of storm drainage systems, geospatial data, and field surveys observations. Storms that occurred on December 17–18, 1967; October 30, 2004; and March 31, 2006 were used to calibrate the HEC-HMS model. The City and County of Honolulu drainage maps and University of Hawai'i's utility maps were used to determine the existing storm drainage system. Geospatial information, including LiDAR data and aerial maps established terrain roughness characteristics and stream channel cross sections. Rainfall data was extrapolated to be converted into intensity-duration-frequency (IDF) curves, illustrating rainfall intensities according to their duration.

Hydrologic Analysis Procedure

Hydrologic analysis of sub-watersheds of the Ala Wai Watershed predicted from the application of five hydrologic modeling methods: the HEC-HMS model, USGS regression method, City and County of Honolulu drainage standards Plate 6, Federal Emergency Management Agency Flood Insurance Study, and the HEC Statistics Software Package (SSP). The HEC-HMS model of the Ala Wai Watershed was the focus of this report, and the results from this model were relied on more than other methods.

SCS curve number Loss Method was applied and Clark Unit Hydrograph transform method was applied for non-urbanized areas, and the Kinematic Wave Transform Method was used for urbanized areas. The Ala Wai Canal was assumed to be a reservoir for the purposes of this study because of backwater effects that are possible in the mouth of Ala Wai Canal. Also, according to the TR-55 method, the water flow path was separated into three portions: sheet flow, shallow concentrated flow, and channel flow, which are summed to calculated time of concentration. Manning's *n* values were selected for the land surface characteristics for the Ala Wai Watershed. Curve number calculations were established according to the hydrologic soil group.

Results

Final "best" peak flow discharges were determined by comparing the various derived discharge-frequency curves graphically and by the accuracy or uncertainty of each method. Table ES-1 shows the results of peak flows discharges at the mouth of the Ala Wai Canal.

Peak Flow Discharges at Mouth of Ala Wai Canal								
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
Methodology	**Peak flow discharge (cubic feet per second)**							
HEC-HMS (original Dec 2008)	6,000	10,100	13,390	15,190	16,740	17,670	18,690	20,480
Plate 6						22,500		
FEMA			13,700		23,000	28,200		36,200
HEC-HMS (updated 2016)	8,080	12,000	14,100	16,000	17,800	19,100	20,700	22,200
Final Used (2016)	**8,000**	**11,500**	**13,500**	**16,000**	**18,000**	**19,500**	**20,500**	**22,000**

Table ES-1. Peak Flow Discharges at Mouth of Ala Wai Canal (Updated March 2016)

1 Introduction

1.1 Background

In 2001, the United States Army Corps of Engineers (USACE) recommended flood mitigation and ecosystem restoration measures for the Ala Wai Watershed, located on the southeast sector of the island of Oʻahu, Hawaiʻi. These measures constitute the Ala Wai Watershed Project that encompasses a drainage area of approximately 10,400 acres of the valleys of Makiki, Mānoa, and Pālolo, and low-lying areas of Mōʻiliʻili, McCully, and Waikīkī. These areas are economically significant and densely populated, and many have high potential for flooding. Historically, floods have occurred in the Mānoa, Makiki, and Mōʻiliʻili areas due to quick concentration of storm waters that overwhelms the drainage system capacities. Depending on a storm's intensity and duration, the steep slopes of the upper Ala Wai Watershed can create flood conditions due to its steep slopes and impervious surfaces from urbanization. In the past, such as during the severe storm of October 30, 2004, flash flood waters with accumulated debris have caused significant property damage to residential, commercial, and public land (Belt Collins 1998).

Storm runoff in these areas flows through drainage systems that ultimately empty into the Ala Wai Canal. In turn, the Ala Wai Canal flows into the Pacific Ocean. The Ala Wai Canal was constructed in the 1920s, and has experienced heavy sedimentation and economic degradations since its inception (Belt Collins 1998). The proposed flood mitigation measures for the Ala Wai Watershed Project must be based on the best hydrologic and hydraulic data available.

USACE contracted Oceanit to conduct hydrologic analysis for a range of potential storms in the Ala Wai Watershed. This hydrologic study uses five different methods to estimate peak flow discharges throughout the Ala Wai Watershed for potential storms ranging in duration and intensity. Best available predictions are based on the existing conditions of the Ala Wai Watershed's sub-watersheds of Makiki, Mānoa, and Pālolo valleys, Ala Wai Canal, and Waikīkī. Also, the existing conditions of junctions along the Mānoa-Pālolo Canal were considered because of the canal's crucial position as a drainage channel between Mānoa-Pālolo and the Ala Wai Canal, where it empties. Oceanit was directed to model storms using both rainfall-runoff and peak flow frequency methods for a range of storm scenarios, as follows.

1.2 Purpose and Scope

The purpose of this study was to estimate peak flow discharges at particular drainage junctions in the Ala Wai Watershed corresponding to the following storm return periods: 2-, 5-, 10-, 20-, 50-, 100-, 200-, and 500-year. These storm return periods correlate to storm chance exceedance probabilities of 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent, respectively. The study's scope is solely hydrologic and encompasses the Ala Wai Watershed's sub-watersheds of Makiki, Mānoa, and Pālolo valleys, Ala Wai Canal, and Waikīkī. The study also examines the junctions along the Mānoa-Pālolo Canal.

1.3 Methodology

This hydrologic study provides estimated peak flow discharges for a range of storms for particular junctions throughout the Ala Wai Watershed by applying five hydrologic methods as appropriate and necessary. The following were completed in this study: (1) relevant hydrologic data was researched and collected; (2) rainfall-runoff models were constructed and calibrated; (3) peak flow discharges based on rainfall intensity-frequency-duration curves were modeled; and (4) these peak flow discharges were weighted and compared to arrive at final results that represent the *best* estimated peak flow discharges.

First, research on the overall existing conditions in the Ala Wai Watershed study area was conducted. Section 2.1 describes these overall existing conditions, and then Sections 2.2 through 2.4 detail the existing conditions in each sub-watershed. Conditions that were necessarily evaluated for hydrologic modeling included the slope, character, elevation, vegetative coverage, acreage, and use of the sub-watershed lands. Many of these conditions were evaluated from review of existing literature, gathering of geospatial data, and inspection during field visits. This data collection is documented in Section 3.5. Sub-basins within each sub-watershed were delineated using the geospatial data (see Section 3.6). Also, Manning's *n* values, which describe land cover and roughness, were selected (see Section 4.1.5). The existing conditions of drainage systems in the study area were primarily collected from the City and County of Honolulu's Storm Drainage System Maps (Section 3.4), and were confirmed during field visits. Primarily, drainage junctions of interest in the Ala Wai Watershed were determined from evaluating the existing drainage facilities.

Second, potential storm rainfall amount determinations were extrapolated from historic rainfall data. The storm rainfall amounts that were the input for the hydrologic model are considered the meteorological model. The rainfall and stream flow data were collected from rain gage and stream flow gage records as available for the study area (see Sections 3.1 through 3.2). Records from three severe storms were collected and later used to calibrate the hydrologic model (see Section 3.3). Rainfall amounts that constitute the frequency storms in the meteorological model were gathered from a study entitled "Rainfall Frequency Study for Oahu" (Giambelluca 1984) known commonly as Report R-73. Rainfall amounts were gathered from Report R-73 for the storm chance exceedance probabilities of 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent. Intensity-Duration-Frequency curves were established for input into the model.

Third, five methods were used to model the Ala Wai Watershed's hydrology. The rainfall-runoff method used was USACE's Hydrologic Engineering Center–Hydrologic Modeling System (HEC-HMS). The peak flow frequency methods used were the United States Geological Survey (USGS) regression equations, the City and County of Honolulu (the City) Plate 6 storm drainage standards, the Federal Emergency Management Agency (FEMA) Flood Insurance Study for the City and County of Honolulu (2004), and Hydrologic Engineering Center–Statistical Software Package (HEC-SSP). The fourth step in this study was, depending on the data available, applying these methods for each sub-watershed or junctions if available for the range of potential storms: chance exceedance probabilities of 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent. The methods used for each junction (by sub-watershed) are shown in Table 1-1 and designated by a checkmark.

Junction	Drainage area (mi^2)	HEC-HMS	USGS Regression Equations	FEMA-FIS	C&C-Plate 6	HEC-SSP
MAKIKI						
JK1	2.33	√	√		√	
JK2	2.49	√	√	√	√	
JK3	2.89	√			√	
MANOA						
JM8	5.97	√	√	√	√	
PALOLO						
JP1	1.15	√	√		√	√
JP2	2.94	√	√		√	
JP3	3.62	√	√	√	√	√
JP4	4.07	√	√		√	
MANOA-PALOLO						
JMP1	10.04	√	√		√	
JMP2	10.34	√	√	√	√	√
JMP3	10.68	√			√	
ALAWAI						
Mouth of Ala Wai Canal	16.22	√		√	√	

Table 1-1. Methods Used by Sub-Watershed Junction

J = junction; K = Makiki; M = Mānoa; P = Pālolo; MP = Mānoa-Pālolo; and mi = miles. A checkmark indicates a method that was used for a particular junction or outlet.

1.3.1 HEC-HMS Analysis

The HEC-HMS model was the primary method of this study. The HEC-HMS method is a precipitation-runoff process model that requires three components including a basin model, a meteorological model, and a control model. The basin model layout was created according to sub-basin delineation and junctions of interest. For the purposes of this study, sub-watershed refers to the larger areas of Makiki, Mānoa, Pālolo, Ala Wai Canal, and Waikīkī; the term "sub-basin" refers to the smaller sub-watersheds within these sub-watersheds to avoid confusion. Also the term "sub-basin" is commonly accepted for the HEC-HMS model delineation of small drainage areas.

1. **Basin Model:** Under the basin model, Ala Wai Watershed was divided into 38 sub-basins. The SCS loss method and Clark Unit Hydrograph transform methods were applied for upper Makiki, Mānoa, and Pālolo valleys because these areas are considered non-urban. The Kinematic Wave Transform Method was applied for the lower Makiki ,Ala Wai Canal, and Waikīkī areas because these areas are considered urban. Selected stream flow routing methods included the Muskingum-Cunge method to account for the peak flow attenuation and the Modified Puls method to account for the backwater effects for reaches collected in the Ala Wai Canal. Ala Wai Canal was modeled as a reservoir. Several basin models were created based on the calibration and determination purposes.

2. **Meteorological Model:** A meteorological model was used to specify how precipitation would be generated for each sub-watershed in the selected basin model. For calibration purposes, hyetographs were used based on the gage weights. For predictive purposes, the frequency storms were used to produce synthetic flood events, according to exceedance probabilities.

3. **Control Model:** A control model was used to set the computation parameters. This study used a five-minute time interval for all computations.

1.3.2 Peak Flow Discharge Results

Ultimately, all five of these accepted hydrologic methods offer the *best* estimated peak flow discharges at particular junctions through Ala Wai Watershed for a range of potential storms. Available results were first weighted by accuracy or uncertainty of method, and then plotted on log-probability graph paper. Selection was completed for a best fit curve function for the peak flow discharge frequency curve at each junction of interest. Final peak flow discharges are presented in Section 5.

1.4 Acknowledgements

The USACE project managers for this study were Mr. Derek Chow and Ms. Cindy Barger. The hydrologic analysis technical team consisted of Mr. Michael Wong with the USACE and Mr. Simon Li of Oceanit. Oceanit's project manager was Dr. Dayananda Vithanage with assistance from Mr. Jay Stone and Ms. Joanne Hiramatsu. Oceanit's technical staff included Ms. Frances Ajo, Mr. Robert Bourke, Mr. Kevin Gooding, Mr. Jonathan Levy, and Mr. Manabu Tagomori. Thank you to the National Climatic Data Center, Board of Water Supply (BWS), and USGS for providing rain and stream gage data pertinent to this study.

2 Study Area Description

The Ala Wai Watershed contains five sub-watersheds that are addressed in this study: Makiki, Mānoa, Pālolo, Ala Wai Canal, and Waikīkī. The Mānoa-Pālolo Canal is also addressed in terms of its drainage junctions. Section 2.1 describes the existing conditions throughout the Ala Wai Watershed, including the overall climate, topography, geology, vegetation, land use, and water resources. These conditions are similar in each of the Ala Wai sub-watersheds that are described in Sections 2.2 through 2.6.

2.1 Ala Wai Watershed

The subject of this hydrology study is the Ala Wai Watershed, which is located on the southeastern sector of the island of O'ahu, Hawai'i as shown in Figure 2-1. The watershed encompasses 10,378 acres, or 16.215 square miles. The Ala Wai Watershed stretches from the Ko'olau Mountains at Pu'u Kōnāhuanui's peak (3,105 feet) down through the three urban valleys of Makiki, Mānoa, and Pālolo, to the low-lying areas of McCully, Mō'ili'ili, and Waikīkī. Storm runoff in the watershed flows through numerous drainage systems in these areas and ultimately empties into the Ala Wai Canal. The three major sub-watersheds that constitute the Ala Wai Watershed are Makiki, Mānoa, and Pālolo; all three of these sub-watersheds are valley systems of economic significance and dense population. The Makiki, Mānoa, and Pālolo Streams receive flows from each of these valley systems, respectively (see Figure 2-2). Another Ala Wai sub-watershed is at the confluence of the Mānoa and Pālolo Streams, referred to as the Mānoa-Pālolo Canal, which empties storm water runoff into the Ala Wai Canal between the Ala Wai Golf Course and 'Iolani School. The area surrounding the Ala Wai Canal and the adjacent tourist area of Waikīkī comprise another sub-watershed. These major sub-watersheds are shown in Figure 2-2. (According to the existing conditions, sub-basins are delineated within each sub-watershed, and these sub-basin delineations are presented in Section 3, and shown in Figure 3-4.)

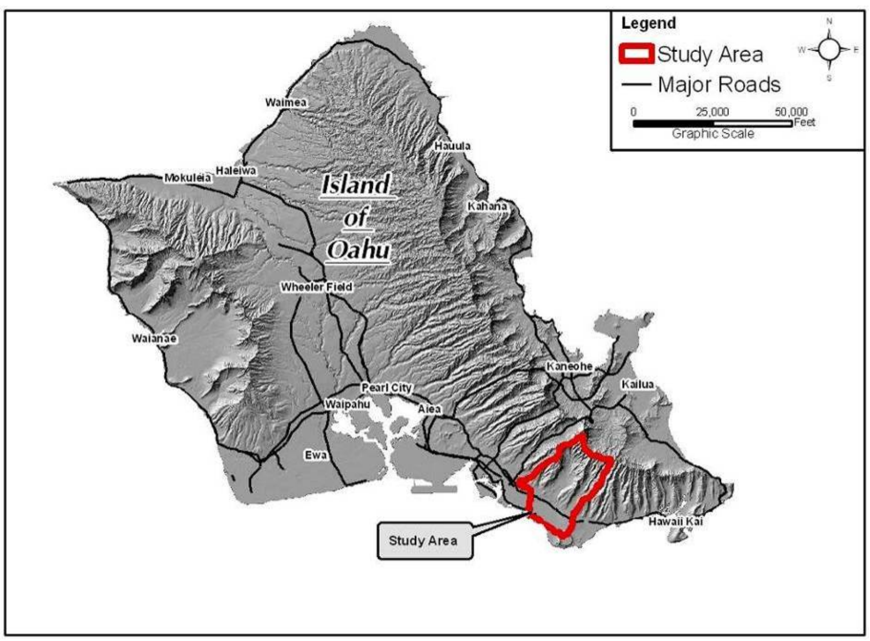

Figure 2-1. Ala Wai Watershed Location Map

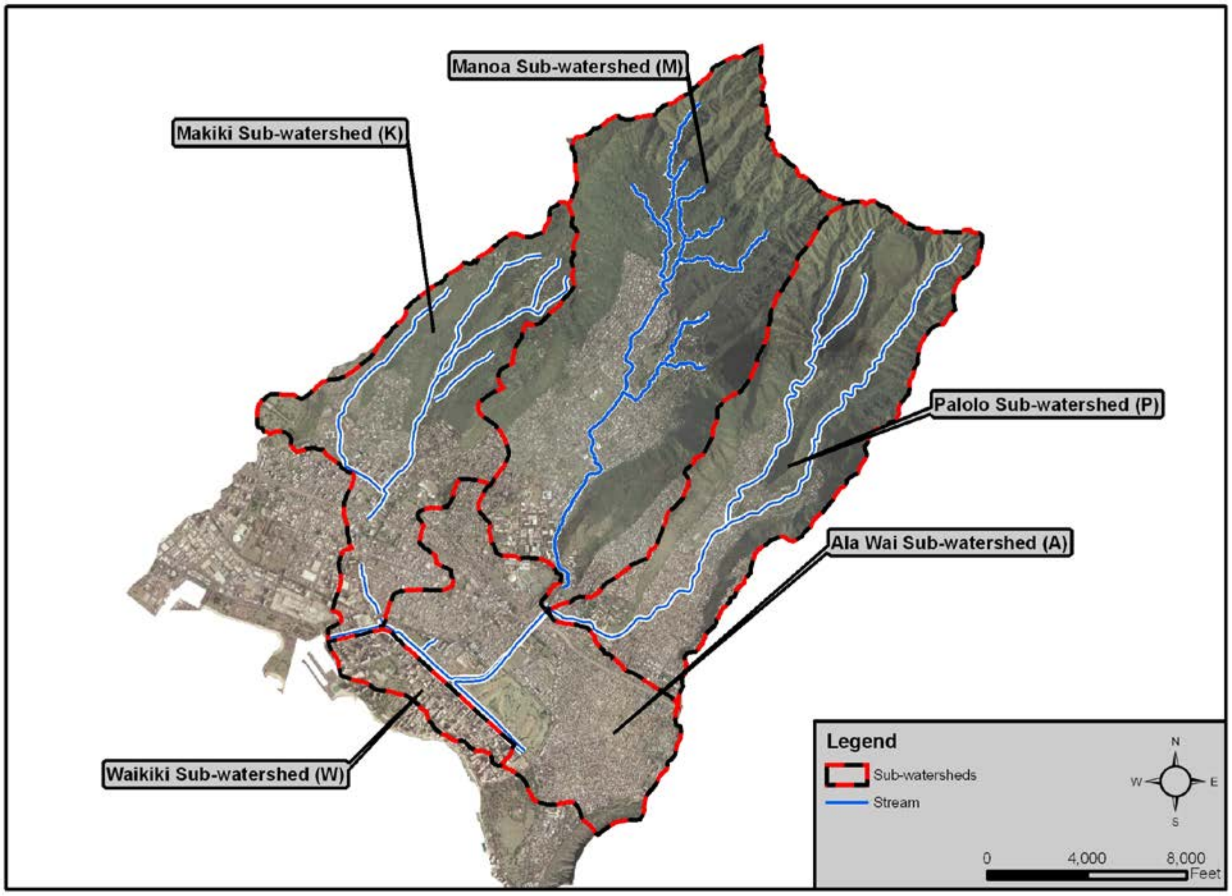

Figure 2-2. Major Streams and Sub-watersheds of Ala Wai Watershed

2.1.1 Climate and Flood Hydrology

Hawai'i's subtropical climate is governed by northeasterly trade winds that regulate weather patterns. The trade winds rise over the Ko'olau Mountain ridges, creating high moisture and orographic rainfall in the mountainous regions. These regions, such as the valley systems of Makiki and Mānoa typically receive more than 160 inches of annual rainfall, whereas the Pālolo valley system receives less annual rainfall (Giambelluca 1984). Generally, rainfall amount decreases as one moves down the valley systems to the southern coast of O'ahu, and so the low-lying areas of the Ala Wai Canal and Waikīkī receive about 30 inches of annual rainfall. The wet winter season occurs from October to April, and the dry summer season occurs from May to September. It should be noted that the three severe storms described for this study occurred in October, December, and March, during the wet winter season. Temperatures on O'ahu fluctuate according to the season, with the winter temperature averaging a high of 77 degrees Fahrenheit (°F) and a low of 64°F. In the summer, temperatures average a high of 81°F and a low of 70°F (National Weather Service, 2008).

Floods on Oahu, other than those generated by high ocean waves, are caused by high intensity rainfall. Most major rainstorms that bring flood-producing rainfall are caused by the non-trade wind or Kona wind conditions which occurred during the wet winter season. Rainstorms can bring intense local showers affecting a small area or can blanket the entire island with rain. High-intensity rainfall, small drainage-basin size, steep basin and stream slopes, and little channel storage, produce floods that are flashy (Wong, 1994). Most drainage basins have rapid response to rainfall characterized by steep triangular hydrographs. Time to peak is usually less than 1 hour and even for large intense storms, the rise and recession of the flood hydrograph usually occurs with 6 hours.

2.1.2 Geology and Soils

The valleys and gulches forming the Ala Wai Watershed are incised into the Koʻolau Volcano. The Koolau lavas are divided into the Koʻolau Basalt and the Honolulu Volcanics. Both of these formations play an important role in the Ala Wai Watershed. The Koʻolau Basalt primarily consists of Pliocene aged shield stage tholeiitic basalt. The Honolulu Volcanics are composed of Pleistocene aged alkalic basalt, basanite, and nephelinite (Lagenheim and Clague, 1987). Holocene and Pleistocene sedimentary caprock is found at the seaward end of the watershed.

The rocks of the Koʻolau Basalt can be divided into three groups, lava flows (aʻa and pahoehoe), pyroclastic deposits, and dikes. The lava flows of the Koʻolau basalt are usually thin bedded with an average thickness of about ten feet (Wentworth and MacDonald, 1953). These beds are composed of aʻa and pahoehoe flows and pyroclastic deposits. Aʻa contains a solid central core between two gravely clinker layers. Pahoehoe flows are usually characterized by a smooth ropy texture. Pyroclastic deposits originate from explosive volcanism. They are composed of friable sand-like ash and indurated tuff deposits. Dikes are thin near vertical sheets of rock that intruded or squeezed into existing lava flows or pyroclastic deposits.

The Honolulu Volcanics erupted much later than the Koʻolau Basalt and overlay the deeply eroded Koʻolau Volcano and its associated alluvial deposits. In Ala Wai they are composed of lava flows and ash and tuff. The lava flows have flow structures similar to the Koʻolau Basalt. The pyroclastic deposits are characterized by easily erodable, sand-like ash and relatively soft and easily erodable tuff. The Sugar Loaf flow which outcrops in cliffs in the UH Quarry poured down from Sugar Loaf on the northwest side of Mānoa Valley and pushed the lower section of Mānoa Stream to the southeast.

The caprock is composed of a wedge of terrestrial and marine sediments. It forms a coastal plain about 8000 feet wide in the Ala Wai area. The caprock is over 1000 feet thick in the seaward areas of the watershed (Wentworth, 1951). Near the ocean, much of the caprock has been covered with artificial fill.

Mānoa and Pālolo valleys are deeply eroded amphitheater shaped valleys that was later backfilled with alluvium and Honolulu Volcanic deposits. The original valleys were probably "V" shaped but the alluvial and volcanic fill material has formed a broad, flat-bottomed valley. The valley fill material is weathered at the surface but despite the heavy rainfall is probably fresh and unweathered in the subsurface. The ridges and valley walls of Mānoa and Pālolo Valleys are generally composed of Koʻolau Basalt (In some areas Honolulu pyroclastics drape the walls). The layered flows of Koʻolau Basalt have eroded into steep weathered cliffs which facilitate rapid runoff. Dikes in the back of the valleys impound groundwater at high elevations which contributes to perennial streamflow.

The altitude within the watershed ranges from mean sea level along the coastal areas, to 40 feet near the confluence of Mānoa and Pālolo Streams, and approximately 2,400 feet in the mountains. Several soil groups are found in the Ala Wai Watershed. The Lualualei-fill land-Ewa association is a well-drained soil that may be found in the lower elevations. These soils have fine textured or moderately fine-textured subsoil or underlying material. The upper watershed is comprised of rock land-stony steep land association. These soils are generally found on steep to precipitous lands and are well-drained to excessively drained (MacDonald et al. 1970).

2.2 Makiki Sub-Watershed

The Makiki sub-watershed is the westernmost of the Ala Wai Canal drainage sub-watersheds, and drains 1,850 acres or 2.89 square miles of land. Makiki Stream, which is approximately 3.5 miles long, drains the sub-watershed. The stream's tributaries include Kanahā Stream, the main tributary that connects to Makiki Stream via Kanahā Ditch (a long lateral channel of about 6,400 feet), Kānealole Stream, Moleka Stream, and Maunalaha Stream (Townscape 2003). The upper segment of the sub-watershed is in the Koʻolau Mountains and is bordered to the west by the Punchbowl Crater.

Whereas the upper sub-watershed is largely forested and undeveloped, the sub-watershed becomes more urbanized as one moves seaward. The upper Makiki sub-watershed has preservation land uses and is considered non-urbanized in this study. The lower Makiki sub-watershed includes the populated Makiki areas of Wilder Avenue, Mānoa Road, and McCully Street. The urbanized portion of the sub-watershed has residential and commercial land uses. Makiki Stream runoff from urban areas and minor streams ultimately discharges into the Ala Wai Canal between McCully Street and Kalākaua Avenue bridges.

2.3 Mānoa Sub-Watershed

Mānoa sub-watershed is located between the Makiki and Pālolo drainage sub-watersheds and drains 3,822 acres (5.97 square miles) of land from the Koʻolau Mountains to the confluence of Mānoa and Pālolo Streams. The upper sub-watershed has preservation land uses and is considered non-urban. In the upper sub-watershed area, several smaller tributaries feed into the Waihī and Waiakeakua Streams and flow into the Mānoa Stream. Mānoa Stream drains the sub-watershed. The Mānoa Stream passes by Noelani Elementary School, the University of Hawaiʻi at Mānoa (UHM) upper campus, and Kānewai Field, and finally meets the Pālolo Stream to form the Mānoa-Pālolo Canal.

Most of the ground surface in the upper sub-watershed is covered with primarily non-native forest, and the middle segment of the sub-watershed is highly urbanized. The natural path and the characteristics of the Mānoa Stream have been altered significantly. Urban culverts discharge storm runoff into the Mānoa Stream throughout the developed area.

2.4 Pālolo Sub-Watershed

The Pālolo drainage sub-watershed is the easternmost of the Ala Wai Canal drainage sub-watersheds, and drains 2,601 acres (4.07 square miles) of land. The Mānoa sub-watershed borders it to the west, and the Mauʻumae Ridge borders the sub-watershed to the east. The Pālolo sub-watershed drains the Koʻolau Mountains and extends down Pālolo Valley to Waiʻalae Avenue. For the purposes of this study, the upper Pālolo sub-watershed is considered non-urban because it has preservation land use. Pūkele Stream and Waiʻōmaʻo Stream are the sub-watershed's two tributary streams. These streams flow into the Pālolo Stream that drains mostly the urbanized portion of the sub-watershed. The land uses in this area are commercial and residential. The Pālolo Stream meets the Mānoa Stream as the Mānoa-Pālolo Canal. As the Pālolo Stream passes through the urban Pālolo area, the stream is a concrete-lined channel that was part of a flood control project constructed by the City and County of Honolulu.

2.5 Mānoa-Pālolo Canal Junctions

The Mānoa and Pālolo Streams meet as the Mānoa-Pālolo Canal downstream of Kānewai Field and immediately north of Waiʻalae Avenue. The Mānoa-Pālolo Canal discharges into the Ala Wai Canal downstream of the Ala Wai Golf Course. Even though Mānoa-Pālolo Canal drains a segment of the Ala Wai Canal sub-watershed, it does so through large storm drainage outfalls that empty directly into the canal. Thus, only junctions (not areas of the sub-watershed) of the Mānoa-Pālolo Canal were examined for this study, and the large outfalls that enter the canal drain 20,285 acres of land.

2.6 Ala Wai Canal Sub-Watershed

The Ala Wai Canal sub-watershed drainage system is 1805 acres (2.82 square miles) including the Mānoa-Pālolo Canal. Historically, the lower portion of Ala Wai Watershed consisted of wetlands and provided ample storage for heavy runoff from the watershed. Ala Wai Canal was designed to drain the wetlands formed by the streams and create dry land for Waikīkī resort development, and the canal was constructed in the 1920s. At the time of the Ala Wai Canal project, the urban development in the watershed was limited, but today the Waikīkī area is heavily urbanized. Runoff from Makiki, Mānoa, and Pālolo sub-watersheds contains suspended materials from the natural reaches of these watersheds, and, as a result, Ala Wai Canal has experienced significant sedimentation over the years.

For the purpose of this study, the Ala Wai Canal was modeled as a reservoir using USACE's HEC-HMS. Considering that the canal may be subject to backflow and meets the ocean at mean sea level, a reservoir model is appropriate due to the low elevation and likelihood of water storage. This assumption significantly affected the modeling of the Ala Wai Canal.

2.7 Waikīkī Sub-Watershed

The Waikīkī drainage sub-watershed is the southern-most and coastal area of the Ala Wai Canal drainage sub-watersheds, and drains 298 acres (0.47 square miles) of coastal land. The Waikīkī area is heavily urbanized and not only a vital center of the tourism industry on O'ahu but also a popular residential, shopping, and nightlife area. Historically, the Waikīkī area was swamp land, and thus the sub-watershed is low-lying. The sub-watershed is characterized by impervious surfaces, and storm drainage runoff either flows as overland flow, flows directly into the ocean, or flows through the City drainage system directly into the Ala Wai Canal. The canal is at a similar elevation as the Waikīkī sub-watershed itself.

3 Data Gathered

The character of the land, the historical rainfall data, and historical stream flow data are relevant to the hydrological analysis of the Ala Wai Watershed. Data used for HEC-HMS model calibration included rain gage data, stream flow gage data, stage gage data, and tide gage data records of historical storms, and field surveys. These data were used to create rainfall intensity-duration-frequency curves. Rainfall data were the input for the HEC-HMS rainfall-runoff model calibration.

3.1 Rain Gages

Data sets from thirteen rain gages were used for the Ala Wai Watershed hydrologic analysis. Four of these rain gages are operated by the National Weather Service (NWS), four rain gages are operated by the BWS, three rain gages are operated by USGS, one rain gage is operated by the UHM, and one rain gage is privately operated. The characteristics of each gage are listed in Table 3-1. Figure 3-1 maps these rain gages in or nearby the study area, labeled by their name and identification number (ID). As shown, rain gages are located in a diversity of elevations and locations throughout the greater Ala Wai Watershed.

Typically, rainfall in upper elevations of the sub-watersheds is greater than that of the lower elevations. For the Makiki sub-watershed, the rain gage at the highest elevation is the Tantalus Peak gage at 1,665 feet above mean sea level (MSL). The Mānoa Tunnel rain gage at 650 feet above MSL is the highest for the Mānoa sub-watershed, and the Pālolo Tunnel rain gage is located at 995 feet above MSL. The lowest rain gage for the entire Ala Wai watershed is the Waikīkī Zoo gage at about 5 feet above MSL. It should be noted that three rain gages were located outside the study area. The Waikīkī Zoo rain gage (717.2) was used to represent the Ala Wai Canal and Waikīkī sub-watersheds. The Wihelmina Rise rain gage (721) was used to represent the middle Pālolo sub-watershed, and the Punchbowl Crater rain gage (709) was used to represent the lower Makiki sub-watershed. Figure 3-2 shows the annual rainfall distribution in the Ala Wai Watershed by major sub-watersheds.

Rain gage data sets vary according to whether records are taken in real time (typically 15-minute intervals) or daily. Records were used to extrapolate the rainfall hyetographs for all the sub-watersheds in the calibration basin models. Also, rain gage records provided essential data for three storms that were used to calibrate the HEC-HMS model. Those storms occurred on December 17–18, 1967; October 30, 2004; and March 31, 2006.

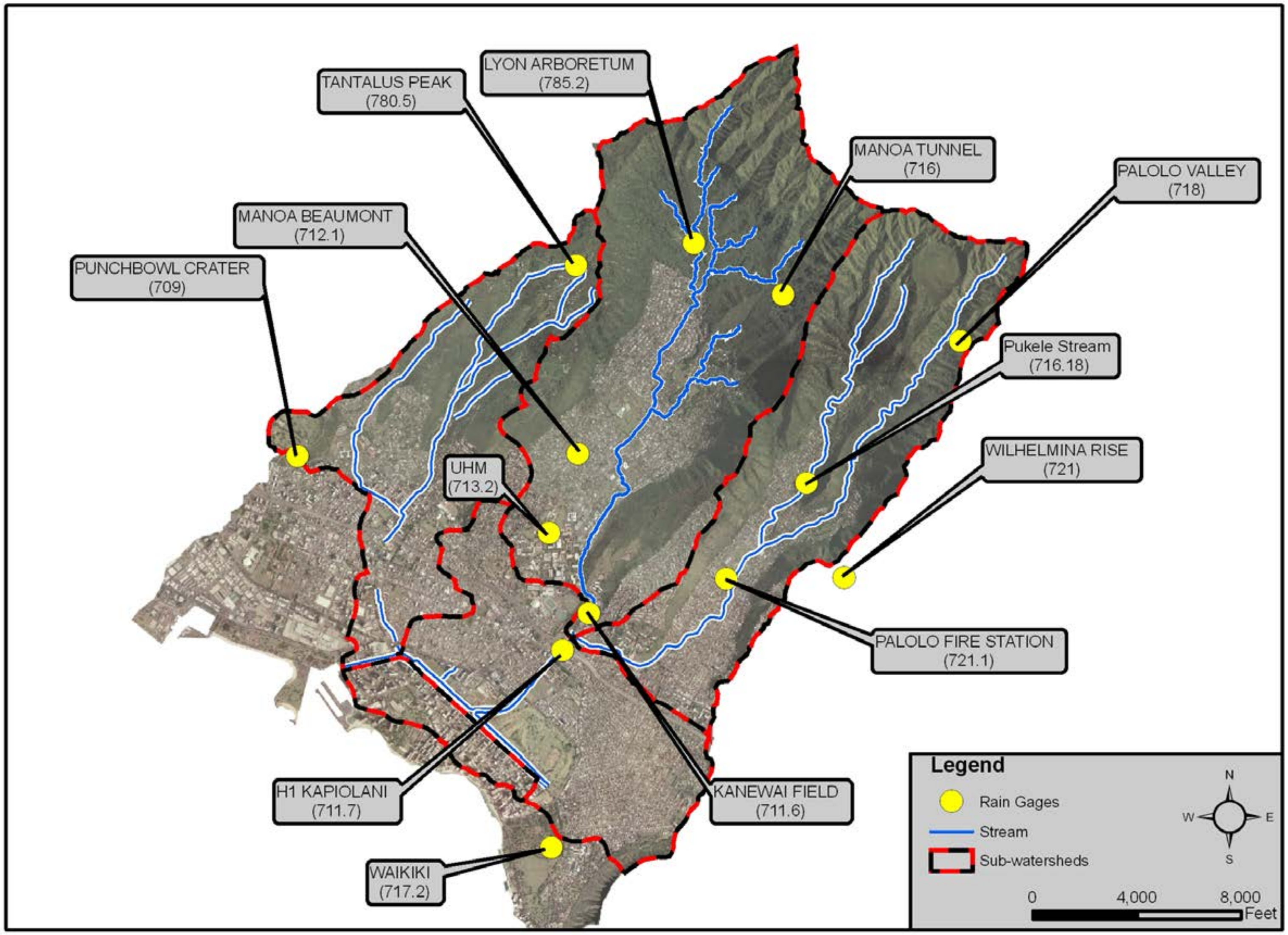

Figure 3-1. Ala Wai Watershed Rain Gages Used by Identification Number

Characteristics of Rain Gages Used

Name	ID	Latitude	Longitude	Elevation (ft)	Records	Real-time recording†	Daily recording*	Operator
Lyon Arboretum	785.2	21°20'08"	157°48'12"	500	1975– Present	√		NWS
Mānoa Tunnel	716	21°19'48"	157°47'36"	650	1927– Present		√	BWS
Kānewai Field	711.6	21°17'47"	157°48'56"	38	1999– Present	√		USGS
Mānoa Beaumont	712.1	21°18' 48"	157°49'00"	200	1947– Present		√	Private
UHM	713.2	21°18'18"	157°49'12"	120	1952– Present		√	UH
Pālolo Fire Stn.	721.1	21°18'00"	157°48'00"	190	1950– Present	√		NWS
Pālolo Tunnel	718	21°20'00"	157°49'00"	995	1926– Present	√		BWS
H-1 Kapiolani	711.7	21°17'22"	157°48'56"	20	2005– Present	√		USGS
Punchbowl Crater	709	21°18'48"	157°50'54"	355	1950– Present		√	NWS
Waikīkī Zoo	717.2	21°16'00"	157°49'00"	5	1957– Present	√		NWS
Wihelmina Rise	721	21°18' 00"	157°47'12"	1100	1927– Present		√	BWS
Pūkele Stream	716.18	21°18'36"	157°47'27"	345	1927– 2005	√		USGS
Tantalus Peak	780.5	21°20'00"	157°49'00"	1665	1927– Present		√	BWS

Table 3-1. Characteristics of Rain Gages Used.

† Real-time recording is by time intervals of 15 minutes. *Daily recording is 24-hour period

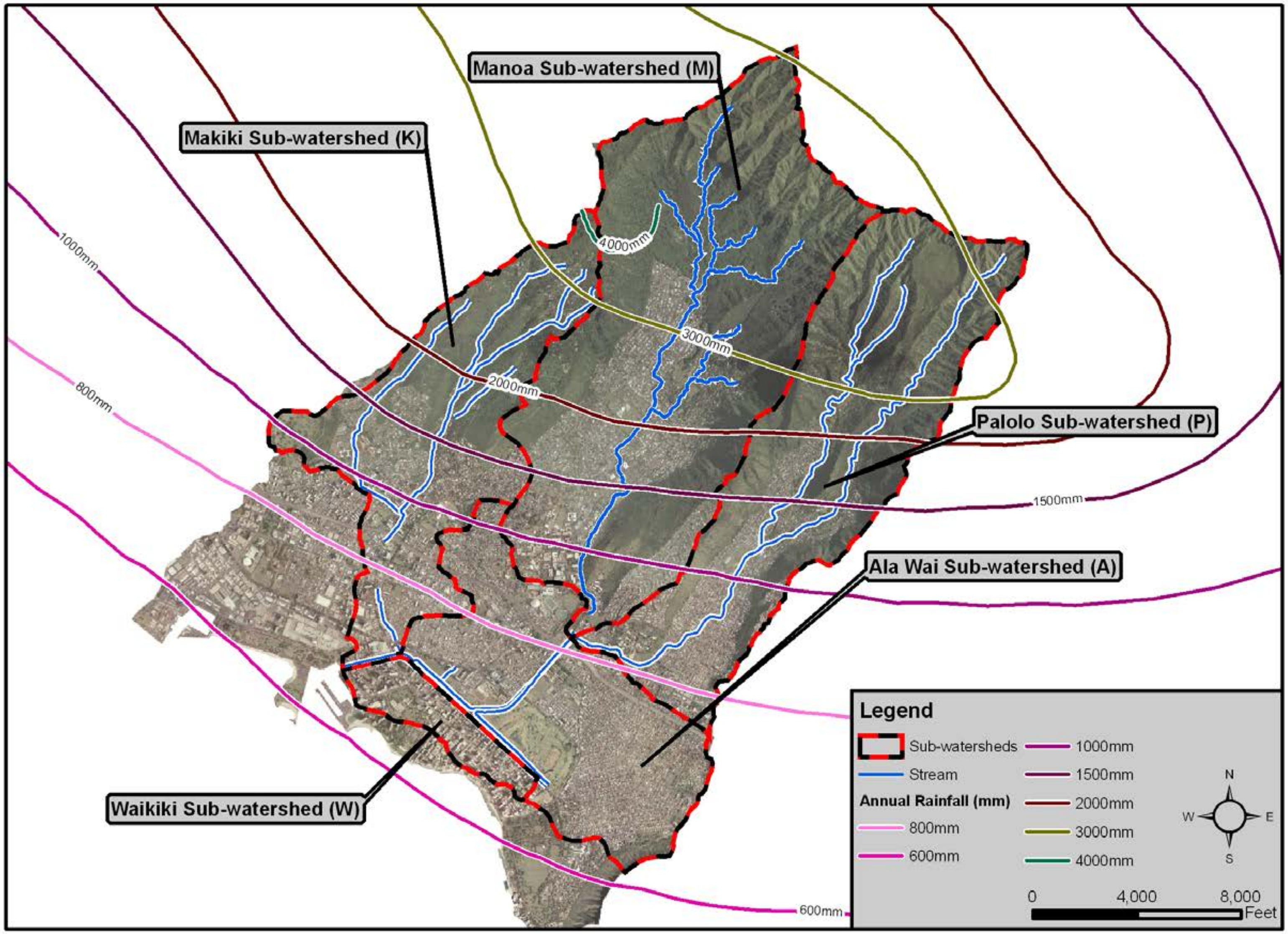

Figure 3-2. Annual Rainfall Distribution for Ala Wai Watershed by Major Sub-watershed

3.2 Stream Flow Gages

Historic stream gage records were used to develop the sub-basin analyses for the HEC-HMS model. Data sets came from nine stream gages throughout the Ala Wai Watershed, and these gages are shown in Figure 3-3 labeled with their USGS identification number. Stream gage data for three storms were essential for calibrating the HEC-HMS model (see calibration discussion in Section 3.8). These three storms occurred in 1967, 2004, and 2006 and are discussed in Section 3.8. Stream gage data for these events are limited depending on whether the gages' record continuously, such as by 15-minute intervals, or whether they simply record peak flow values. The characteristics of the stream gages are given in Table 3-2, and the stream flow gages are shown in Figure 3-3.

3.3 Stage Gages

The Waikīkī and Ala Wai Canal sub-watersheds are located on low-lying coastal land, and data from two stage gages were used in these areas, as shown in Figure 3-4. Stage gage data was essential for calibrating the Ala Wai Canal sub-watershed model detailed in Section 4.6. The nearest stage gage in the ocean was the National Oceanic and Atmospheric Agency's (NOAA's) tide level station 1612340 at Honolulu Harbor, which was used to calibrate the model. The other gage used was USGS 16247130 at Ala Wai Elementary School. These stage gages are located west of the study area as shown in Figure 3-2. Although there are no public published stage records, the local USGS office provided Oceanit with continuous stage data for the October 30, 2004, storm for calibration purposes (see Section 4).

Characteristics of Stream Gages Used									
Gage Location	**Waihī**	**Waiakeakua**	**Lowrey**	**Kānewai**	**Pūkele**	**Wai'ōma'o**	**Pālolo**	**Makiki**	**Mānoa-Pālolo**
Gage Number	16238500	16240500	16241500	16242500	1624400	16246000	16247000	16238000	16247100
Gage Location, Latitude	21°19'55"	21°19'52"	21°18'53"	21°17'47"	21°18'36"	21°18'34"	21°17'35"	21°17'02"	21°17'24"
Gage Location, Longitude	157°48'12"	157°48'08"	157°48'41"	157°48'56"	157°47'27"	157°47'11"	157°48'25"	157°50'22"	157°49'17"
Gage Elevation (ft)	289.84	294.5	294.5	38	344.78	373.66	95	10	5
Drainage Area (USGS, mi^2)	1.14	1.06	4.02	5.05	1.18	1.04	3.63	2.23	10.6
Drainage Area (mi^2)	1.19	1.07	4.22	5.643	1.146	1.036	3.62	2.49	10.34
Period of Continuous Record	1913–1983	1913–Present	---	1999–Present	1927–2004	1927–1971	1953–Present	---	1967–Present
Peak Flow Record Only	---	---	2003-2004	---	---	---	---	2003-2004	---
Number of Annual Peaks Available for Analysis	63	88	3	6	59	39	32	2	40

Table 3-2. Characteristics of Stream Gages Used

Characteristics of Stage Gages Used		
Gage Location	**Honolulu Harbor**	**Ala Wai Elementary School**
Gage Number	1612340	16247130
Gage Location, Latitude	21° 18.4'	21°17'16"
Gage Location, Longitude	157° 52.0'	157°49'51"
Gage Elevation (ft)	B.M. ELV. 8.06 Feet	5
Period of Continuous Record	1905-present	2003-2004

Table 3-3. Characteristics of Stage Gages Used

Note: B.M. ELV.= Bench Mark Elevation

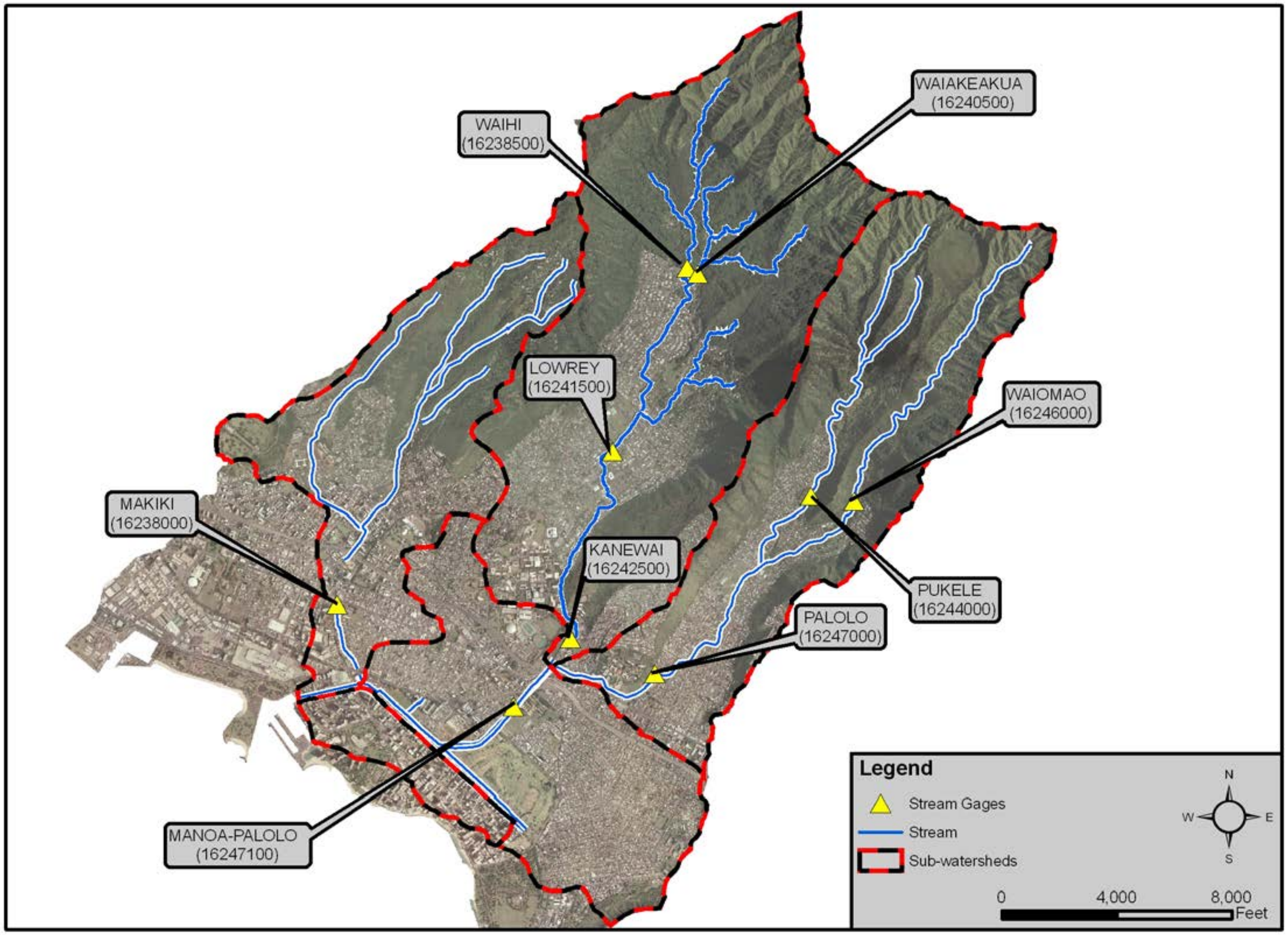

Figure 3-3. Ala Wai Watershed Stream Gages Used by ID Number

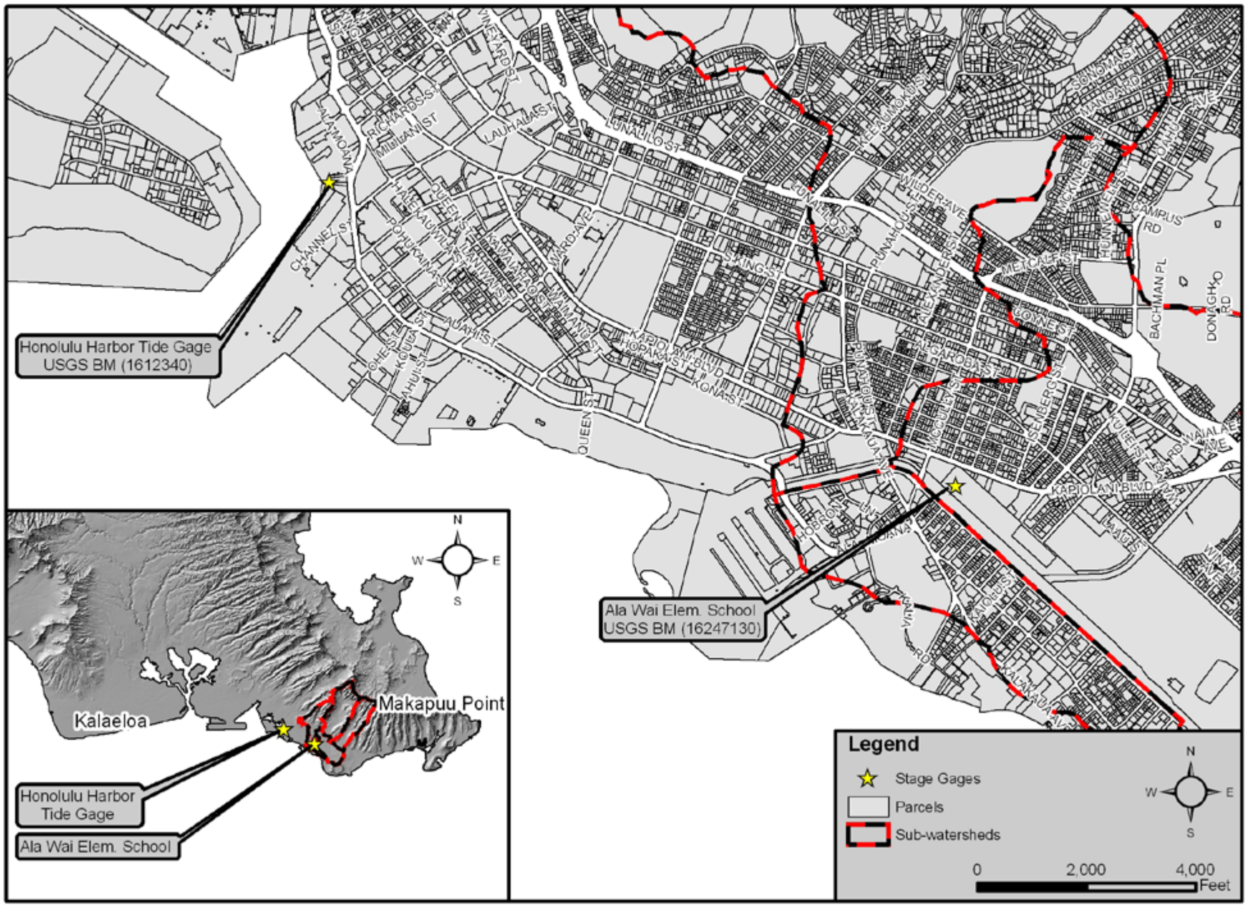

Figure 3-4. Ala Wai Watershed Stage Gages Used (with ID Number)

3.4 Drainage Systems

The City's municipal storm drainage system drains the sub-watersheds of the study area. Runoff from storms flows into the streams or drainage systems throughout the study area. The City's drainage maps were used to identify the locations of the existing storm drainage system. These maps provided information about the characteristics of drainage system segments, including whether the segments are natural or channelized and the size of outlets throughout the system. The drainage systems evaluation results were used in determining the sub-basins boundaries. For example, the boundaries of sub-basin K4 were mainly determined from drainage evaluation.

University of Hawai'i at Mānoa provided utility maps showing the drainage systems through the campus area. Existing conditions of the UHM's storm drainage system, such as the size of relevant culverts, were gathered from these maps. Detailed drainage systems information can be found in the *Final Drainage Evaluation Report Ala Wai Watershed Project* (Oceanit 2008a). The drainage systems information within the UHM upper campus was used to determine the boundaries of sub-basin M12. Based on this information, the boundaries of sub-basins M12 were changed slightly. As a result, this sub-watershed's drainage area was different from the Manoa Watershed Study—it changed from 0.672 to 0.749 square miles.

3.5 Geospatial Data

Geospatial information [1] and field survey observations were used to determine hydrologic conditions, such as terrain roughness characteristics and stream channel cross sections. Information collected included LiDAR data and aerial maps. Numerous field visits to the various sub-watersheds of the study area were made over the course of January 2008 until September 2008 to confirm and/or describe any relevant existing condition of a drainage system facility or the existing conditions in a sub-basin.

LiDAR data were inputted into ArcView GIS 3.3 with the HEC-GeoHMS 1.1 extension to create a geospatial model of the Ala Wai Watershed. The HEC-GeoHMS (USACE 2003) model was used to delineate the initial sub-watershed boundaries, calculate sub-watershed areas, and determine flow path lengths and slopes. However, the sub-watersheds within the study area were not completely delineated by the HEC-GeoHMS model alone. The existing drainage infrastructure and the locations of potential conceptual design measures were important factors for sub-watershed delineation. The final sub-watershed delineation was the result of a combination of the HEC-GeoHMS model, an evaluation of the existing storm drainage system, and the potential locations of the conceptual design measures. LiDAR data were used to approximate the boundaries of sub-basins and sub-watersheds. In addition, ArcView GIS 3.3 and drainage maps were used to determine the boundaries of urbanized areas of the sub-watersheds' drainage areas because better resolution was available for evaluation.

3.6 Sub-Basin Delineation

For the purposes of this study, sub-watershed refers to the larger watershed areas of Makiki, Mānoa, Pālolo, and Waikīkī, and the term "sub-basin" refers to the smaller sub-watersheds within these sub-watersheds. These terms are used to avoid confusion. Also the term "sub-basin" is commonly accepted for the HEC-HMS model delineation of small drainage areas. Sub-basins provide clear boundaries for hydrologic study, and sub-basins were delineated according to a couple of assumptions. The key assumption is that the City's drainage systems, the underground storm sewers, does not cross a topographic sub-basin boundary for all return periods from 2-year through 500-year storms.

This assumption takes into account all the storm runoff for storms, but not all storm runoff necessarily flows through storm drainage systems. According to the United States Department of Agriculture (1990; Module 206A), "Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by street, lawns, and so on to the outlet." This suggests that

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[1] The aerial images that were used for the hydrologic analysis are from the National Geospatial-Intelligence Agency (NGA) supplied by the USGS. The specifications for these images are 0.3 meter pixel size, rectified natural color image orthoimage. The working image was re-sampled to 1-meter pixel size.

The digital elevation LiDAR data used in this hydrologic analysis were obtained from AIRBORNE 1, with an accuracy of 4 elevation points per square meter. The original data were reprojected to North American Datum (NAD) 83 HARN 1993 Universal Transverse Mercator (UTM) Zone 4 meters. The grid size was 2 meters by 2 meters.

storm runoff flows along the natural geographic flow path and not necessarily through the storm drainage system. Based on the City's storm drainage standards, the drainage capacities with catchment areas greater than 100 acres should meet 100-year storm drainage standards; the drainage capacities with catchment areas equal to or less than 100 acres should meet 10-year storm drainage standards. Consequently, at junctions with contributing drainage systems, peak discharges may be lower than predicted. Similarly, at junctions where drainage system catchment areas are not considered, actual peak discharges may be higher than predicted.

Some delineations of sub-basins and assumptions about sub-basins were necessary for the low-lying areas of Mānoa-Pālolo Canal, Ala Wai Canal sub-watershed, and Waikīkī sub-watershed. Because Mānoa-Pālolo Canal receives drainage from other sub-watersheds with relatively large drainage systems, only the junctions in the Mānoa-Pālolo Canal were examined and there were no sub-basins delineated around the canal itself. Also, delineation for the Waikīkī sub-watershed was particularly problematic because some of its sub-basins drain directly into the ocean with a relatively small flow directed through the outfalls designated on the drainage maps.

It should be noted that all the hydrologic analysis results in this study for Mānoa sub-watershed were exactly the same as performed in the *Mānoa Watershed Project Final Hydrology Report* (Oceanit 2008b) to keep consistency with the previous Mānoa Watershed Project hydrologic study. Another assumption was made about the UHM area in the Mānoa sub-watershed. The drainage area of sub-watershed M12 (UHM upper campus) was changed from the previous 0.672 square miles (Oceanit 2008) to 0.747 square miles. This drainage area determination accounts for the contribution of a 96-inch culvert storm drainage system at Dole Street Bridge. The characteristics of the storm sewer network were collected from the UHM Utility Map (2008).

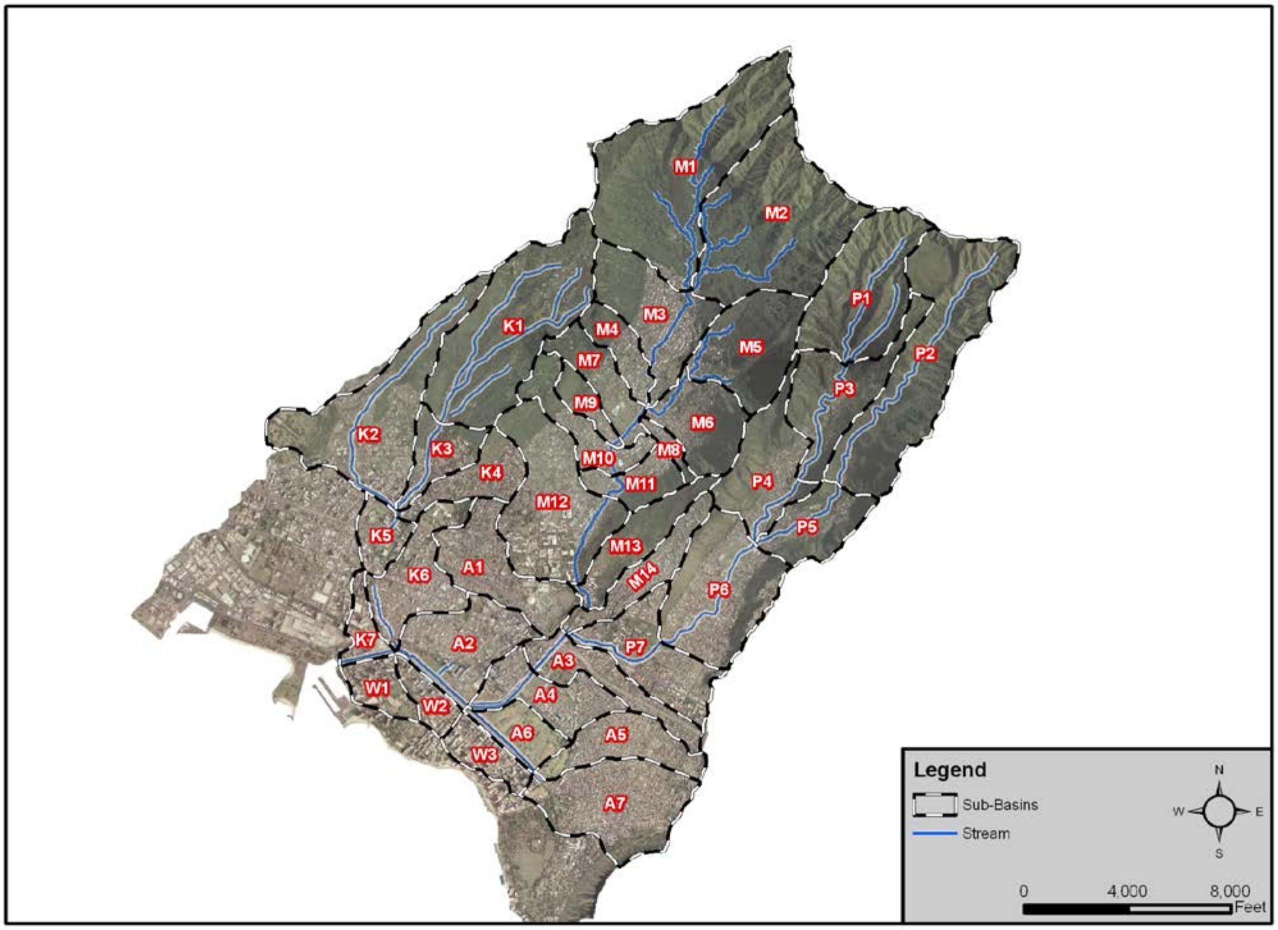

Figure 3-5. Ala Wai Watershed Sub-Basin Delineation

Ala Wai Watershed delineation of sub-basins was based on the junctions that are confluences of study area streams. The following table of sub-basin delineations designates the respective sub-watershed by the following.

- 'J' for junctions, or stream confluences, throughout the watershed
- 'K' for sub-basins in the Makiki sub-watershed
- 'M' for sub-basins in the Mānoa sub-watershed
- 'P' for sub-basins in the Pālolo sub-watershed
- Note that Mānoa-Pālolo Canal sub-watershed has junctions only and not sub-basins because other sub-basins empty into this canal but it does not drain its surrounding area
- 'A' for sub-basins in the Ala Wai sub-watershed; assumed to be a reservoir for the purposes of this study (see earlier discussion in Section 3.6)
- 'W' for sub-basins in the Waikīkī sub-watershed

Ala Wai Watershed Sub-Basin Delineation		
Sub-Basin/Junction	Sub-Basin or Junction Name	Drainage Area (mi^2)
MAKIKI		
KI	Upper Makiki Stream	1.00
K2	Kanahā Stream	0.85
K3	Middle Makiki Stream	0.22
K4	East Mānoa Road	0.25
JK1	Confluence of Makiki and Kanahā Streams	2.33
K5	Lower Makiki Stream	0.16
JK2	USGS Stream Gage near King St. 16238000	2.49
K6	Washington Middle School	0.40
JK3	Confluence of Makiki Stream and Ala Wai Canal	2.89
MĀNOA		
M1	Waihī	1.20
M2	Waiakeakua	1.07
JM1	Confluence of Waihī and Waiakeakua Streams	2.27
M3	Pawaina	0.51
M4	Poelua	0.18
M5	Woodlawn_Ditch 1	0.50
M6	Woodlawn_Ditch 2	0.35
JM2	Confluence of Mānoa Stream & Woodlawn Ditch	3.81
M7	Park	0.25
M8	Kahaloa	0.06
M9	Lowrey	0.11
JM3	Lowrey Ave. Bridge	4.22
M10	Woodlawn	0.26
JM4	Woodlawn Dr. Bridge	4.48
M11	Noelani	0.19
JM5	Mānoa Stream near Noelani Elementary School	4.67
M12	Dole (UHM campus)	0.75
JM6	Dole Street Bridge	5.42
M13	Kānewai	0.30
JM7	Kānewai Field Gage	5.72
M14	Saint Louis Heights	0.25
JM8	Just Upstream of the Confluence of Mānoa & Pālolo Streams	5.97

Table 3-4. Ala Wai Watershed Sub-Basin Delineation

Ala Wai Watershed Sub-Basin Delineation (Continued)		
Sub Basin or Junction Number	Sub Basin or Junction Name	Drainage area (mi^2)
PĀLOLO		
P1	Upper Pūkele Stream	0.67
P3	Middle Pūkele Stream	0.48
JP1	USGS Pūkele Gage 16244000	1.15
P2	Upper Waiʻōmaʻo Stream	1.04
P4	Lower Pūkele Stream	0.45
P5	Lower Waiʻōmaʻo Stream	0.31
JP2	Confluence of Pūkele and Waiʻōmaʻo Streams	2.94
P6	Pālolo Stream	0.68
JP3	USGS Pālolo Gage 16247000	3.62
P7	Waialae Avenue	0.45
JP4	Just Upstream of the Confluence of Mānoa & Pālolo Streams	4.07
MĀNOA-PĀLOLO		
JMP1	Confluence of Mānoa and Pālolo Streams	10.04
A3	H1 Freeway	0.30
JMP2	USGS Stream Gage 16247100	10.34
A4	Date Street	0.34
JMP3	Confluence of Mānoa-Pālolo and Ala Wai Canals	10.68
ALA WAI & WAIKĪKĪ		
A5	Kaimukī	0.32
A7	Diamond Head Drainage System	0.62
A6	Ala Wai Golf Course	0.20
W3	Kuhio	0.18
A1	UHM lower campus and Punahou School	0.45
A2	Mōʻiliʻili	0.47
W2	Kālakaua	0.13
A8	Hawaii Convention Center	0.12
W1	Ala Moana Blvd.	0.16
OUTLET	Mouth of Ala Wai Canal	16.21

Table 3-4 (Continued). Ala Wai Watershed Sub-Basin Delineation

Drainage systems collect the majority of runoff in Waikīkī, and thus, information about these systems was used to delineate the Waikīkī sub-watersheds. Most of the runoff flows through the City's drainage systems and discharges into the Ala Wai Canal. However, a small portion of runoff flows directly into the ocean. This small portion is overland flow or is emptied directly into the ocean by drainage pipes.

3.7 Storm Records Used for Calibration

Calibration of the HEC-HMS model relied on sub-basin analysis that used available records of three storms in December 17-18, 1967; October 30, 2004; and March 31, 2006. However, partial stream flow data were available for some gages and junctions had different recording equipment. Below is a list of the records available by location and storm. The locations refer to the HEC-HMS model layout.

- A partial data set from JM3 (Lowrey Ave. Bridge) from the 2004 storm was used for calibration
- At M2 (Waiakeakua sub-basin), peak flow data were used for the 1967 storm, and real-time data were used for the 2004 and 2006 storms
- At JMP2 (USGS stream gage 17247100 at Kaimukī High School), peak flow data were used for the 1967 storm, and real-time data were used for the 2004 and 2006 storms.
- At JP1 (USGS Pūkele Stream gage), peak flow data from the 1967 storm were used, and real-time data from the 2004 storm were used
- At JP3 (USGS stream gage 17247000 at Pālolo Stream), peak flow data from all three storms were used, but some of these data were discarded because they were clearly inaccurate—comparison to other gage readings downstream during the same storm showed clear inconsistencies

3.7.1 December 1967 Storm

On December 16, 1967, a surface weather front appeared to be stationary west of Hawaiʻi (DLNR 1968). Torrential rains started falling on Oʻahu around the middle of the night on December 17. Many rainfall stations reported excessive rainfall during the storm. Pālolo Valley, Waiʻalae-Kāhala, Niu Valley, and Waimānalo suffered extensive flood damage. Rainfall amounts registered in the windward area had a rainfall frequency of about a 25-year storm (DLNR, 1968). The Tantalus Peak rain gage registered 5 inches of rainfall for a 3-hour period ending at 3:00 AM. The Pālolo Tunnel rain gage, maintained by the BWS, recorded 10.06 inches between the middle of the night and 8:00 AM hours, with 2.4 inches from 4:00 AM to 5:00 AM. The rainfall intensity was almost uniformly distributed from the coastal area to the Koʻolau Mountains. The USGS stream gage 16247000 at the Pālolo Stream recorded a record high peak discharge of 4,270 cubic feet per second (cfs); the USGS stream gage 16247100 at the Mānoa-Pālolo Drainage Canal recorded its highest estimated discharge at 10,100 cfs.

3.7.2 October 2004 Storm

A storm on October 30, 2004, that caused flooding in the Mānoa Valley was characterized as about a 20-year storm (NWS 2005). This return period corresponds to a 5% probability of occurrence. The persistent and heavy rainfall created swift and high stream flows that were recorded throughout the Mānoa Stream by various rain and stream gages. The heaviest rainfall happened around 7:30 PM, at which time the Lyon Arboretum rain gage recorded 1.29 inches in 15 minutes. The gage records for the October 2004 storm were used to calibrate the HEC-HMS model.

3.7.3 March 2006 Storm

On March 31, 2006, a strong storm caused the NWS to issue flash flood warnings for Oʻahu because rain fell on already saturated ground. The storm moved over the windward (eastern) half of Oʻahu during the late morning, and rainfall of 1 to 2 inches were recorded within one-hour periods by several NWS gages (NWS 2006). The NWS Waimānalo rain gage recorded over 3 inches of rainfall within a two-hour period. During the six weeks prior to this storm, Oʻahu had experienced heavy rains that saturated lands on the windward side of the island. The March 31 rainfall, coupled with the saturated character of the land, produced flash floods throughout the island (NWS 2006). The Moanalua, Makiki, and Mānoa Streams overtopped their banks, and residents of Mānoa valley were alerted of flash flooding in the area. Various intersections and flooding forced the partial closure of the area's major highway, H-1 Freeway, and downtown streets were clogged with traffic (Pacific Business News 2006).

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4 Hydrologic Analysis Procedure

Hydrologic analysis of sub-watersheds of the Ala Wai Watershed utilized up to five hydrologic modeling methods. Given the HEC-HMS model layout for the Ala Wai Watershed, the hydrologic analyses for sub-watersheds were completed on the basis of the existing conditions—particularly whether or not sub-watersheds are urbanized. For the sub-watersheds without much urbanized area, hydrologic models were calibrated using the storm records outlined in Section 3.8. The hydrologic model, as shown in Figure 4-1 was based on the sub-watersheds delineated. These sub-watersheds include the upper Makiki, upper Mānoa, and upper Pālolo. Thus, Sections 4.2 through 4.5 outline the necessary parameters that were calculated: rainfall amount, time of concentration, and curve numbers. As mentioned earlier, the Clark Unit Hydrograph was used as the transform method for these areas that are not urbanized.

For the sub-watersheds with more urbanized area, the hydrologic models used the Kinematic Wave Transform Method. Section 4.7 provides the Kinematic Wave Transform Method analyses of the urbanized areas of the Ala Wai Canal and Waikīkī sub-watersheds, alongside the Mānoa-Pālolo Canal junctions considered.

4.1 Hydrologic Model Layout

Stream junctions of interest that are listed in Table 3-3 are illustrated as the final hydrologic model layout as shown below in Figure 4-1.

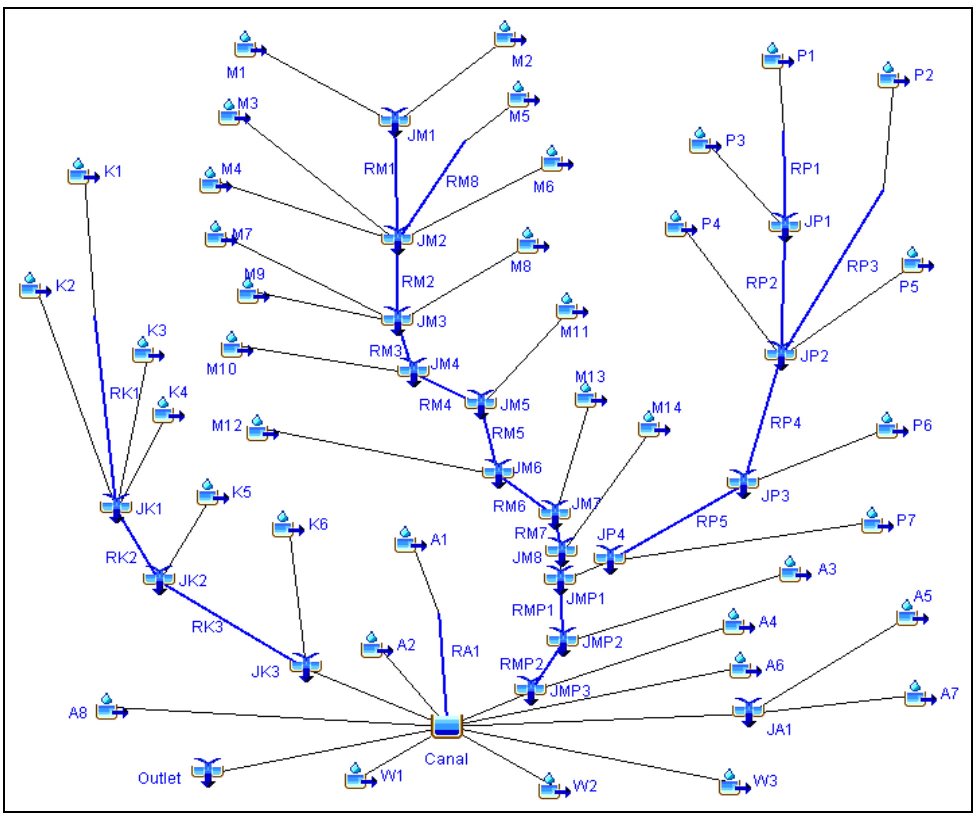

Figure 4-1. Ala Wai Watershed HEC-HMS Model Layout

4.2 Meteorological Model

The storm rainfall amounts that were the input for the hydrologic model are considered the meteorological model. The rainfall and stream flow data were collected from rain gage and stream flow gage records as available for the study area (see Sections 3.1 through 3.2).

4.2.1 Rainfall Amount Determination

Rainfall amount determination was necessary for 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent chance exceedance storms. These amounts were interpolated and/or extrapolated from "Rainfall Frequency Study for O'ahu", Report R-73, by Giambelluca, Lau, Fok and Schroeder (1984). For the 1-, 6-, and 24-hour rainfall amounts for the recurrence periods of 50, 10, 2, and 1 percent chance exceedance, values (shown in Table 4-1) were obtained directly from R-73 (Giambelluca 1984). The rainfall depths from R-73 were plotted, and the resulting smooth curve-function was used to estimate the rainfall depths that were not directly shown in R-73. Thus, for the percent chance exceedance storms less than the 1 percent storm, the rainfall amounts for various durations between 1 hour and 24 hours were determined from the duration nomographs presented in R-73. These curves are shown in Figure 4-3. The 0.5 and 0.2 percent chance exceedance storms' rainfall amounts were estimated by extrapolation using the rainfall depths relationships above the 1 percent chance exceedance storm. Rainfall values less than 1-hour were computed using 1-hour value. According to R-73, the 30-, 15-, and 5-minute rainfall values were determined by multiplying the 1-hour value by 0.714, 0.539, and 0.264, respectively.

Flow in the upper sub-watersheds may be underestimated due to sudden rainfall events that concentrate quickly as runoff because of high amounts of rainfall. Conversely, low rainfall is apparent in the lower sub-watersheds, and the relatively flat topography lends to underestimates of peak flows because runoff along the coastal areas may flow directly into the ocean. Thus, rainfall presented here is an average, based on the center point of the sub-basin and interpolated and extrapolated from the rainfall data available. The center point of each sub-basin was determined using the geospatial data discussed in Section 3.5. It should be noted that the 2001 Ala Wai Flood Study (USACE 2001) used a different approach for determining one rainfall value by averaging rainfall in the upper watershed and lower watershed rather than by averaging by the entire watershed.

Rainfall Intensity Duration Values for the Ala Wai Watershed

Percent Chance Exceedance	Recurrence Interval Year	Duration								
		5-min	15-min	30-min	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
50%	2	0.40	0.81	1.07	1.50	2.20	2.65	3.50	4.40	5.30
20%	5	0.49	1.00	1.32	1.85	2.80	3.40	4.45	5.70	7.15
10%	10	0.63	1.28	1.70	2.38	3.35	4.10	5.50	7.00	8.60
5%	20	0.70	1.43	1.89	2.65	3.80	4.65	6.25	8.05	10.05
2%	50	0.83	1.70	2.25	3.15	4.35	5.35	7.20	9.45	11.80
1%	100	0.91	1.86	2.46	3.45	4.85	6.00	8.25	10.90	13.65
0.5%	200	1.02	2.08	2.75	3.85	5.35	6.55	9.15	12.10	15.20
0.2%	500	1.16	2.37	3.14	4.40	6.10	7.55	10.40	13.65	17.00

Reference: Giambelluca and others, 1984, DLNR Report R-73

Table 4-1. Determined Rainfall Intensity Duration Values in inches for Ala Wai Watershed, Oahu, Hawaii. Note: rainfall intensity frequency data determined from maps and nomographs in Giambelluca and others, 1984, DLNR Report R-73.

4.2.2 Rainfall Intensity-Duration-Frequency Curves

The rainfall-depth duration curves graph in Figure 4-2 shows the rainfall data as determined in average amounts for the percent chance exceedance storms. The rainfall amounts are for a 24-hour period, and were converted to intensity-duration-frequency (IDF) curves to offer rainfall intensities according to the range of storms examined (see Figure 4-3). The IDF curve is a crucial input into the HEC-HMS model analysis.

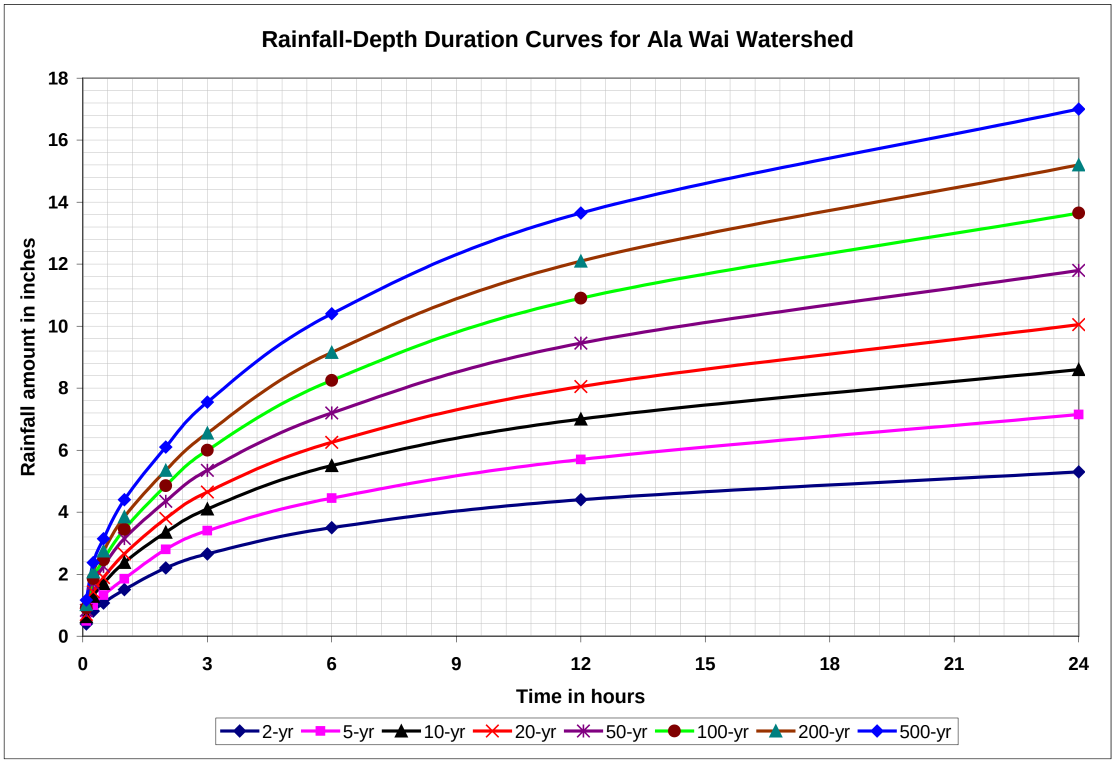

Figure 4-2. Rainfall-Depth Duration Curves for Ala Wai Watershed

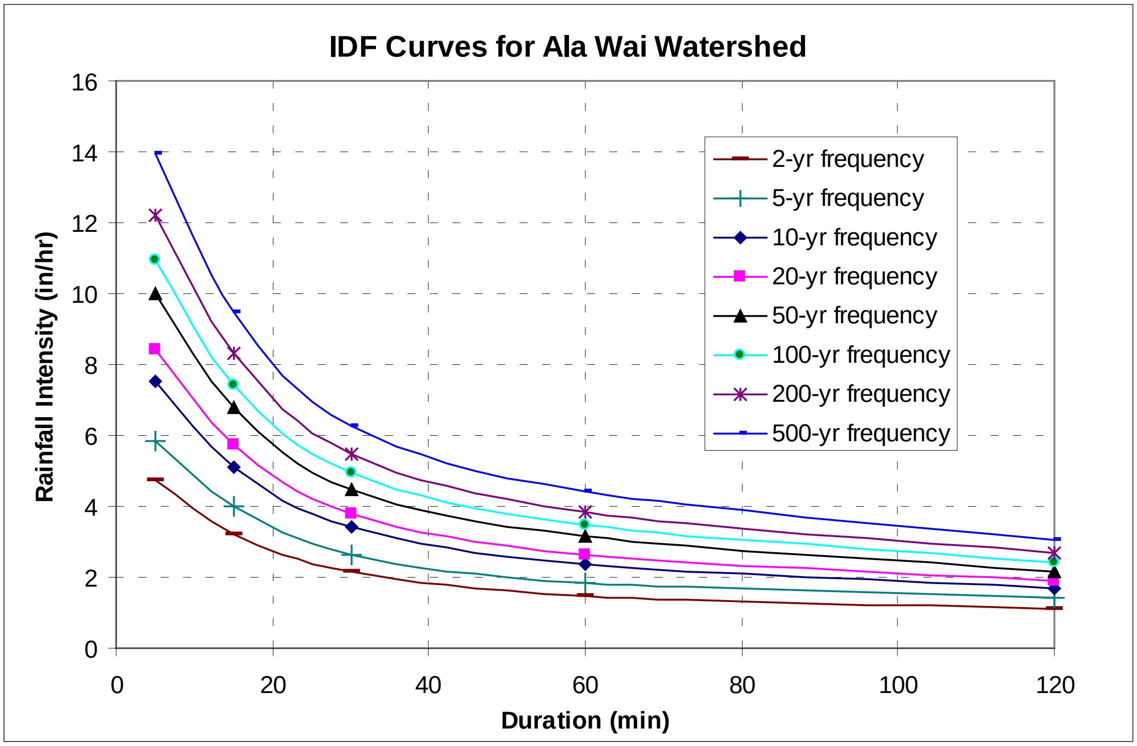

Figure 4-3. IDF curves for Ala Wai Watershed

4.2.3 Time of Concentration Calculation

The Clark Unit Hydrograph requires the parameter of the time of concentration (T_c) for each sub-basin. According to the TR-55 method, three types of flow path constitute the water flow: sheet flow, shallow concentrated flow, and channel flow; these three flows were added together to calculate time of concentration. According to the NRCS's Technical Report 55 (1986), "Time of concentration is the time for runoff to travel from the hydraulically most distant point of watershed to a point of interest within the watershed." The majority of the flow path may be channel flow as appropriate. Calculation of time of concentration is necessary for preparing the transform method for a unit hydrograph. The TR-55 velocity approach method was used to calculate time of concentration; that means the traveling time is a function of watercourse length and the velocity. The average velocity is a function of watercourse, slope, and type of channel.

A certain number of assumptions were made regarding sheet flow. The sheet flow segment describes the time period from raindrop impact until overland flow accumulates to a depth of about 0.1 foot, and one assumption made for time of concentration calculations was that the flow length for the stream reaches analyzed were not longer than 100 feet. The sheet flow segment T_c is calculated using Manning's kinematic solution, dependent on Manning's roughness coefficient *n*, the flow length, the rainfall amount, and the land slope. According to the SCS training material module 206A, "in most watersheds the overland [sheet] flow length is probably about 50 ft." (USDA, 1990) A maximum length of 100 feet is allowed in WinTR-55, and SCS suggests that a visit to the watershed is the best manner of determining the appropriate sheet flow length. Because this study lacked the appropriate observations for sheet flow during site visits, and considering previous studies and engineering judgement, a sheet flow length of 80 feet was set for all sub-watersheds in the Ala Wai Watershed for the calculation of time of concentration.

Overall, the flow length was determined from the City drainage maps and the known characteristics of the stream reach. Also, estimated flow length and land slope data were gathered from the geospatial data collected (see Section 3.5) using ArcView GIS 3.3. LiDAR topographic data and 5-foot elevation contours were used to calculate the slope of each sub-watershed.

4.2.4 Manning's *n* Roughness Coefficients

The surface Manning's roughness coefficients, based on the ground surface conditions, were determined as either 0.4 (woods with light underbrush) or 0.24 (dense grasses) using Table 3-1 from TR-55 (NRCS 1986). Where storm drainage systems are present in the sub-watershed, the appropriate flow path was used to estimate the time of concentration. Drainage pipe flow not under a pressure condition is treated as a portion of channel flow. The wetted perimeter condition assumes the full-flow condition for the drainage system pipes and the natural channel of the streambed. Altogether, the Manning's roughness coefficient for storm drainage facilities was selected as 0.015.

TR-55 Method Time of Concentration Parameters

Sheet Flow Characteristics					Shallow Concentrated Flow			Channel Flow Characteristics					Time of Concentration
Sub-Basin	**Manning's *n***	**Flow Length (ft)**	**Two-Year 24-hour Rainfall (in)**	**Land Slope**	**Surface Description**	**Flow Length (ft)**	**Slope**	**Flow Length (ft)**	**Cross-Section Area (ft^2)**	**Wetted Perimeter(ft)**	**Channel Slope**	**Manning's *n***	**T_C (hr)**
K1	0.4	80	5.3	0.450	Unpaved	1200	0.218	7200	30	19	0.174	0.035	**0.202**
K2	0.4	80	5.3	0.375	Unpaved	1150	0.278	9900	20	18	0.090	0.035	**0.311**
K3	0.24	80	5.3	0.313	Paved	1850	0.305	3100	30	19	0.042	0.035	**0.170**
K4	0.24	80	5.3	0.405	Paved	1450	0.365	5200	7.07	9.42	0.042	0.015	**0.165**
M14	0.24	50	5.3	0.250	Paved	1200	0.150	4150 1200	4.91 160	7.85 48	0.128 0.017	0.015 0.035	**0.152**
P1	0.4	80	5.3	0.260	Unpaved	1600	0.450	4850	40	24	0.159	0.040	**0.189**
P2	0.4	80	5.3	0.200	Unpaved	1850	0.172	9200	40	24	0.090	0.035	**0.313**
P3	0.24	80	5.3	0.306	Unpaved	2300	0.321	5500	48	20	0.061	0.035	**0.203**
P4	0.24	80	5.3	0.280	Unpaved	2800	0.285	1950 2400	3.14 48	6.28 20	0.115 0.0375	0.015 0.035	**0.215**
P5	0.24	80	5.3	0.260	Paved	800	0.285	700 4050	1.77 48	4.7 20	0.236 0.0395	0.015 0.035	**0.163**
P6	0.24	80	5.3	0.270	Paved	1150	0.550	800 5600	4.9 120	7.85 48	0.0625 0.0187	0.015 0.018	**0.168**
P7	0.24	80	5.3	0.180	Paved	700	0.040	3100 3500	4.9 160	7.85 48	0.03 0.02	0.015 0.018	**0.218**

Table 4-2. TR-55 Method Time of Concentration Parameters

Table 4-2 shows the values for sheet flow, shallow concentrated flow, and channel flow that were used to calculate the times of concentration. The times of concentration range from 0.152 hours in the Mānoa 14 sub-basin to 0.313 hours in the Pālolo 2 sub-basin, as shown in Table 4-3.

4.3 Curve Numbers Calculation

Runoff curve numbers, according to the TR-55 method (NRCS 1986), were used to determine the loss method of the HEC-HMS. Soil types in the study area were identified, and assigned to their appropriate hydrologic soil group (HSG in Table 4-3) classification. Geospatial data collected were used to determine land cover appropriate to each sub-basin, and for the various sub-watersheds. The different types of land cover and associated curve numbers are shown in Table 4-3. For the specific sub-basins, curve numbers were multiplied by the areas of the soil types by sub-watershed. For each sub-watershed, the product of these calculations was averaged over the total sub-watershed area to arrive at a composite curve number.

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Calculation of Composite Curve Numbers for Ala Wai Watershed

Sum of WS_Acre		New HydroGrp HSG (All D and blank to C)				Curve Number			Area x CN			Composite
SUB-BASIN	LAND USE	A	B	C	Total	A	B	C	A	B	C	CN
A1	Bare Land	0.0	3.1	1.0	5.0	72	82	87	0.0	253.0	89.8	
	Evergreen Forest	0.5	0.2	0.2	0.8	30	55	70	13.6	11.2	11.2	
	Grassland	3.2	4.7	4.4	12.3	39	61	74	126.4	287.4	323.2	
	High Intensity Developed	13.9	73.8	34.6	122.3	89	92	94	1240.5	6791.8	3249.2	
	Low Intensity Developed	62.6	55.4	22.6	140.7	77	85	90	4821.2	4710.6	2036.1	
	Scrub/Shrub	2.1	3.2	3.4	8.7	30	48	65	63.4	152.5	222.4	
A1 Total		83.3	140.4	66.2	289.9	A1 Composite CN						**84**
A2	Bare Land		1.6	6.1	7.7	72	82	87	0.0	132.6	527.6	
	Cultivated Land			0.2	0.2	77	86	91	0.0	0.0	20.2	
	Evergreen Forest		0.1	0.4	0.6	30	55	70	0.0	6.9	30.8	
	Grassland		4.1	12.5	16.6	39	61	74	0.0	250.9	926.5	
	High Intensity Developed		106.5	109.9	216.4	89	92	94	0.0	9799.5	10327.7	
	Low Intensity Developed		13.0	19.8	32.8	77	85	90	0.0	1105.8	1781.7	
	Scrub/Shrub		2.1	7.2	9.2	30	48	65	0.0	99.5	465.2	
	Water		0.1	15.1	15.1	98	98	98	0.0	8.0	1476.0	
A2 Total			127.5	171.1	298.7	A2 Composite CN						**90**
A3	Bare Land		1.9	0.0	1.9	72	82	87	0.0	157.9	0.0	
	Evergreen Forest		0.2	0.0	0.2	30	55	70	0.0	12.2	0.0	
	Grassland		5.6	0.0	5.6	39	61	74	0.0	342.0	0.0	
	High Intensity Developed		143.8	0.0	143.8	89	92	94	0.0	13229.4	0.0	
	Low Intensity Developed		38.5	0.0	38.5	77	85	90	0.0	3276.0	0.0	
	Scrub/Shrub		3.8	0.0	3.8	30	48	65	0.0	183.9	0.0	
A3 Total			193.9	0.0	193.9	A3 Composite CN						**89**
A4	Bare Land		3.0	0.9	3.9	72	82	87	0.0	244.7	79.4	
	Evergreen Forest		0.2	0.1	0.2	30	55	70	0.0	8.8	4.4	
	Grassland		23.6	6.0	29.6	39	61	74	0.0	1440.7	441.8	
	High Intensity Developed		122.7	5.0	127.8	89	92	94	0.0	11290.6	472.8	
	Low Intensity Developed		29.1	7.6	36.8	77	85	90	0.0	2477.6	685.8	
	Scrub/Shrub		7.9	6.5	14.3	30	48	65	0.0	378.5	419.3	
	Water		0.2	3.6	3.8	98	98	98	0.0	18.0	349.9	
A4 Total			186.7	29.6	216.3	A4 Composite CN						**85**
A5	Bare Land		1.2	0.2	1.4	72	82	87	0.0	98.4	19.3	
	Evergreen Forest		0.2	0.0	0.2	30	55	70	0.0	12.2	0.0	
	Grassland		12.2	2.2	14.5	39	61	74	0.0	745.9	165.1	
	High Intensity Developed		131.9	0.8	132.7	89	92	94	0.0	12138.5	72.6	
	Low Intensity Developed		48.7	4.7	53.4	77	85	90	0.0	4140.4	422.9	
	Scrub/Shrub		0.9	0.6	1.5	30	48	65	0.0	43.8	38.8	
	Water			0.4	0.4	98	98	98	0.0	0.0	38.4	
A5 Total			195.2	8.9	204.1	A5 Composite CN						**88**
A6	Bare Land		2.7	5.0	7.7	72	82	87	0.0	223.0	437.4	
	Evergreen Forest			0.5	0.5	30	55	70	0.0	0.0	32.1	
	Grassland		52.8	18.6	71.3	39	61	74	0.0	3219.5	1372.7	
	High Intensity Developed			5.1	5.1	89	92	94	0.0	0.0	478.9	
	Low Intensity Developed		0.1	3.1	3.1	77	85	90	0.0	5.9	275.9	
	Scrub/Shrub		4.5	15.0	19.5	30	48	65	0.0	215.9	977.5	
	Water		1.2	17.5	18.7	98	98	98	0.0	116.1	1714.1	
A6 Total			61.2	64.7	126.0	A6 Composite CN						**72**
A7	Bare Land		1.5	2.5	3.9	72	82	87	0.0	120.8	214.5	
	Evergreen Forest		2.0	0.1	2.1	30	55	70	0.0	111.6	4.2	
	Grassland	0.2	12.8	1.1	14.1	39	61	74	7.7	783.5	79.1	
	High Intensity Developed	0.0	238.4	3.8	242.3	89	92	94	3.5	21935.1	361.6	
	Low Intensity Developed	0.3	67.2	10.3	77.9	77	85	90	25.4	5715.0	928.2	
	Scrub/Shrub	0.8	15.5	38.1	54.4	30	48	65	24.3	744.6	2478.1	
	Water		0.0	2.2	2.2	98	98	98	0.0	0.3	211.5	
A7 Total		1.4	337.5	58.0	396.9	A7 Composite CN						**85**
A8	Bare Land			0.1	0.1	72	82	87	0.0	0.0	12.4	
	Grassland		0.1	0.4	0.6	39	61	74	0.0	8.7	32.9	
	High Intensity Developed		28.8	37.8	66.6	89	92	94	0.0	2647.5	3556.8	
	Low Intensity Developed		1.4	3.0	4.4	77	85	90	0.0	120.1	271.4	
	Scrub/Shrub		0.7	2.4	3.1	30	48	65	0.0	32.0	159.0	
	Water			4.3	4.3	98	98	98	0.0	0.0	424.2	
A8 Total			31.0	48.2	79.2	A8 Composite CN						**92**
K1	Evergreen Forest	268.5	1.4	66.9	336.8	30	55	70	8056.1	75.6	4681.3	
	Grassland	3.3		1.4	4.8	39	61	74	129.7	0.0	106.4	
	Low Intensity Developed	30.9	0.0	3.9	34.9	77	85	90	2382.0	0.7	354.0	
	Scrub/Shrub	216.6		48.9	265.5	30	48	65	6499.0	0.0	3178.2	
K1 Total		519.4	1.4	121.1	642.0	K1 Composite CN						**40**
K2	Bare Land	0.7		0.4	1.1	72	82	87	48.0	0.0	38.7	
	Evergreen Forest	97.2		51.4	148.6	30	55	70	2917.0	0.0	3595.5	
	Grassland	69.3	0.5	19.5	89.3	39	61	74	2702.9	27.6	1443.9	
	High Intensity Developed	34.4	29.8	14.5	78.7	89	92	94	3062.2	2742.4	1363.0	
	Low Intensity Developed	103.2	10.2	43.3	156.7	77	85	90	7949.2	863.6	3901.4	
	Scrub/Shrub	47.3	0.0	22.5	69.8	30	48	65	1419.1	1.1	1461.9	
K2 Total		352.2	40.4	151.7	544.3	K2 Composite CN						**62**

Table 4-3. Calculation of Composite Curve Numbers for Ala Wai Watershed

Calculation of Composite Curve Numbers for Ala Wai Watershed (Continued)

Sum of WS_Acre		New HydroGrp HSG (All D and blank to C)				Curve Number			Area x CN			Composite
SUB-BASIN	LAND USE	A	B	C	Total	A	B	C	A	B	C	CN
K3	Evergreen Forest	17.1	7.3	4.4	28.9	30	55	70	513.0	404.0	309.3	
	Grassland	5.7	2.2	0.7	8.6	39	61	74	221.6	131.5	52.9	
	High Intensity Developed	2.3	11.1	0.0	13.4	89	92	94	203.6	1024.5	0.0	
	Low Intensity Developed	55.4	18.0	2.9	76.3	77	85	90	4267.4	1533.9	258.1	
	Scrub/Shrub	7.7	1.2	7.1	16.0	30	48	65	230.3	58.4	462.2	
K3 Total		88.2	39.9	15.1	143.2	K3 Composite CN						68
K4	Bare Land	1.1		0.0	1.1	72	82	87	77.3	0.0	3.3	
	Evergreen Forest	3.1	0.3	0.0	3.4	30	55	70	93.4	15.9	0.0	
	Grassland	13.7	1.0	0.3	15.0	39	61	74	532.7	61.0	25.7	
	High Intensity Developed	4.8	6.2	0.0	11.0	89	92	94	426.9	573.0	2.0	
	Low Intensity Developed	100.1	9.9	0.0	110.0	77	85	90	7708.3	837.6	0.0	
	Scrub/Shrub	18.9	1.1	0.0	20.0	30	48	65	566.2	52.9	0.0	
K4 Total		141.6	18.5	0.4	160.5	K4 Composite CN						68
K5	Bare Land		0.4	0.0	0.4	72	82	87	0.0	36.5	0.0	
	Evergreen Forest		0.4	0.0	0.4	30	55	70	0.0	20.8	0.0	
	Grassland		4.4	0.0	4.4	39	61	74	0.0	266.0	0.0	
	High Intensity Developed		73.3	0.0	73.3	89	92	94	0.0	6748.0	0.0	
	Low Intensity Developed		21.6	0.0	21.6	77	85	90	0.0	1839.8	0.0	
	Scrub/Shrub		3.2	0.0	3.2	30	48	65	0.0	154.1	0.0	
K5 Total			103.4	0.0	103.4	K5 Composite CN						88
K6	Bare Land	0.3	0.7	0.2	1.2	72	82	87	21.0	54.7	19.3	
	Evergreen Forest	2.6		0.0	2.6	30	55	70	77.6	0.0	0.0	
	Grassland	0.3	7.6	3.2	11.2	39	61	74	12.2	466.6	237.3	
	High Intensity Developed	3.1	141.2	53.6	197.9	89	92	94	271.8	12991.1	5041.6	
	Low Intensity Developed	7.6	22.3	3.9	33.8	77	85	90	581.9	1898.8	349.7	
	Scrub/Shrub	4.6	5.8	0.3	10.6	30	48	65	137.4	276.9	18.8	
	Water			0.0	0.0	98	98	98	0.0	0.0	1.4	
K6 Total		18.4	177.6	61.3	257.3	K6 Composite CN						87
M1	Evergreen Forest	124.6	48.4	80.4	253.4	30	55	70	3738.1	2661.3	5628.2	
	Grassland	3.2	6.2	4.2	13.7	39	61	74	125.3	379.7	313.8	
	High Intensity Developed		0.7	0.9	1.5	89	92	94	0.0	63.5	80.5	
	Low Intensity Developed	0.3	7.6	5.4	13.4	77	85	90	26.2	643.0	490.4	
	Scrub/Shrub	51.6	40.2	393.3	485.1	30	48	65	1547.7	1929.2	25565.2	
M1 Total		179.7	103.1	484.3	767.1	M1 Composite CN						56
M10	Bare Land	0.6		1.1	1.7	72	82	87	44.8	0.0	96.3	
	Evergreen Forest	19.6		0.5	20.1	30	55	70	588.4	0.0	32.0	
	Grassland	5.2	0.1	3.0	8.3	39	61	74	201.0	7.9	222.3	
	High Intensity Developed	4.5	3.5	17.5	25.5	89	92	94	400.8	323.4	1641.2	
	Low Intensity Developed	40.3	4.3	33.4	77.9	77	85	90	3103.4	362.7	3003.1	
	Scrub/Shrub	24.9		9.2	34.1	30	48	65	746.9	0.0	600.2	
M10 Total		95.1	7.9	64.6	167.6	M10 Composite CN						68
M11	Bare Land			0.4	0.4	72	82	87	0.0	0.0	32.6	
	Evergreen Forest		1.0	11.4	12.3	30	55	70	0.0	52.6	795.4	
	Grassland	0.0	4.8	0.8	5.6	39	61	74	0.6	292.0	56.8	
	High Intensity Developed		1.2	4.7	5.9	89	92	94	0.0	108.6	445.2	
	Low Intensity Developed	5.1	17.4	23.8	46.2	77	85	90	389.1	1480.0	2139.9	
	Scrub/Shrub		1.2	49.9	51.1	30	48	65	0.0	57.4	3246.6	
M11 Total		5.1	25.5	91.0	121.6	M11 Composite CN						75
M12	Bare Land	2.9	0.5	0.6	4.0	72	82	87	208.8	37.5	54.7	
	Evergreen Forest	12.9	2.3	4.9	20.1	30	55	70	387.8	125.0	344.2	
	Grassland	20.5	9.6	6.8	36.9	39	61	74	799.3	588.6	500.5	
	High Intensity Developed	12.9	59.2	5.0	77.1	89	92	94	1150.7	5446.9	469.0	
	Low Intensity Developed	151.6	61.9	9.5	222.9	77	85	90	11674.1	5257.5	851.8	
	Scrub/Shrub	49.0	15.9	53.1	118.0	30	48	65	1471.2	761.7	3452.9	
M12 Total		249.9	149.3	79.9	479.1	M12 Composite CN						70
M13	Bare Land		1.0	0.2	1.2	72	82	87	0.0	78.0	19.4	
	Evergreen Forest		31.1	46.5	77.6	30	55	70	0.0	1712.6	3254.3	
	Grassland		1.0	3.1	4.0	39	61	74	0.0	60.4	226.4	
	High Intensity Developed		7.4	3.1	10.5	89	92	94	0.0	684.6	288.0	
	Low Intensity Developed		14.4	11.7	26.2	77	85	90	0.0	1226.4	1056.9	
	Scrub/Shrub		17.9	51.3	69.2	30	48	65	0.0	858.6	3334.6	
M13 Total			72.8	115.9	188.7	M13 Composite CN						68

Table 4-3 (Continued). Calculation of Composite Curve Numbers for Ala Wai Watershed

Calculation of Composite Curve Numbers for Ala Wai Watershed (Continued)

Sum of WS_Acre		New HydroGrp HSG (All D and blank to C)				Curve Number			Area x CN			Composite
SUB-BASIN	LAND USE	A	B	C	Total	A	B	C	A	B	C	CN
M14	Bare Land		1.2	0.7	1.9	72	82	87	0.0	99.5	63.2	
	Evergreen Forest		0.3	0.2	0.4	30	55	70	0.0	14.0	13.3	
	Grassland		6.3	1.7	8.1	39	61	74	0.0	387.3	126.8	
	High Intensity Developed		47.7	4.1	51.8	89	92	94	0.0	4389.8	387.0	
	Low Intensity Developed		76.8	12.4	89.2	77	85	90	0.0	6526.7	1118.4	
	Scrub/Shrub		7.6	3.5	11.2	30	48	65	0.0	366.4	229.7	
M14 Total			139.9	22.7	162.7	M14 Composite CN						84
M2	Evergreen Forest		92.1	91.6	183.7	30	55	70	0.0	5063.4	6414.8	
	Grassland		0.4	12.3	12.7	39	61	74	0.0	26.3	910.0	
	Low Intensity Developed		0.2	2.5	2.7	77	85	90	0.0	15.3	227.4	
	Scrub/Shrub		29.0	458.6	487.6	30	48	65	0.0	1392.0	29806.5	
M2 Total			121.7	565.0	686.7	M2 Composite CN						64
M3	Bare Land		0.2	0.0	0.2	72	82	87	0.0	18.2	0.0	
	Evergreen Forest	6.6	14.7	12.0	33.4	30	55	70	199.3	810.7	841.2	
	Grassland	7.9	20.1	3.3	31.3	39	61	74	309.9	1227.1	242.7	
	High Intensity Developed	13.4	20.7	5.2	39.3	89	92	94	1191.5	1907.7	488.2	
	Low Intensity Developed	20.5	54.6	14.0	89.1	77	85	90	1578.9	4637.3	1262.7	
	Scrub/Shrub	36.6	37.7	57.1	131.3	30	48	65	1096.7	1809.9	3709.0	
M3 Total		85.0	148.1	91.6	324.7	M3 Composite CN						66
M4	Evergreen Forest	2.1	0.3	1.7	4.1	30	55	70	63.1	15.2	117.1	
	Grassland	2.9	0.1	3.5	6.6	39	61	74	113.1	8.9	261.2	
	High Intensity Developed	14.7	0.1	7.0	21.7	89	92	94	1306.2	6.4	656.1	
	Low Intensity Developed	20.1	0.2	11.3	31.6	77	85	90	1547.9	16.3	1015.4	
	Scrub/Shrub	25.1	0.0	25.5	50.6	30	48	65	752.3	0.6	1655.4	
M4 Total		64.9	0.7	48.9	114.5	M4 Composite CN						66
M5	Bare Land		0.2	0.0	0.2	72	82	87	0.0	18.2	0.0	
	Evergreen Forest		56.1	27.3	83.3	30	55	70	0.0	3083.8	1909.6	
	Grassland		4.2	0.0	4.2	39	61	74	0.0	254.6	0.0	
	High Intensity Developed		1.2	0.0	1.2	89	92	94	0.0	113.0	0.0	
	Low Intensity Developed		27.5	0.0	27.5	77	85	90	0.0	2336.1	0.0	
	Scrub/Shrub		40.4	163.1	203.6	30	48	65	0.0	1940.8	10603.9	
M5 Total			129.6	190.4	320.0	M5 Composite CN						63
M6	Bare Land		0.2	0.0	0.2	72	82	87	0.0	18.2	0.0	
	Evergreen Forest		18.2	7.8	25.9	30	55	70	0.0	999.4	543.3	
	Grassland		9.1	0.6	9.7	39	61	74	0.0	554.6	42.6	
	High Intensity Developed		2.9	1.5	4.5	89	92	94	0.0	269.8	144.3	
	Low Intensity Developed		67.7	5.2	72.9	77	85	90	0.0	5754.8	471.5	
	Scrub/Shrub		40.0	72.8	112.8	30	48	65	0.0	1919.1	4730.1	
M6 Total			138.1	87.9	226.0	M6 Composite CN						68
M7	Bare Land	0.4		1.1	1.5	72	82	87	32.0	0.0	95.3	
	Evergreen Forest	13.7		1.8	15.5	30	55	70	411.9	0.0	123.5	
	Grassland	2.8		24.1	26.9	39	61	74	110.2	0.0	1780.6	
	High Intensity Developed	9.5		5.2	14.7	89	92	94	843.2	0.0	489.6	
	Low Intensity Developed	13.9		22.8	36.7	77	85	90	1072.0	0.0	2049.7	
	Scrub/Shrub	25.7		36.4	62.1	30	48	65	771.4	0.0	2366.4	
M7 Total		66.1		91.3	157.4	M7 Composite CN						64
M8	Bare Land		0.4	0.0	0.4	72	82	87	0.0	29.9	1.9	
	Evergreen Forest			1.7	1.7	30	55	70	0.0	0.0	117.6	
	Grassland		0.7	1.3	2.1	39	61	74	0.0	44.1	98.4	
	High Intensity Developed		0.5	3.2	3.7	89	92	94	0.0	44.3	300.0	
	Low Intensity Developed		10.4	7.8	18.2	77	85	90	0.0	881.5	703.6	
	Scrub/Shrub		1.0	8.0	9.0	30	48	65	0.0	47.0	523.2	
M8 Total			12.9	22.1	35.0	M8 Composite CN						80
M9	Bare Land	0.4		0.5	0.8	72	82	87	25.7	0.0	39.4	
	Evergreen Forest	2.3		1.2	3.5	30	55	70	68.7	0.0	87.4	
	Grassland	2.7		2.4	5.1	39	61	74	106.2	0.0	176.6	
	High Intensity Developed	0.5		5.3	5.8	89	92	94	40.7	0.0	497.7	
	Low Intensity Developed	5.0		21.2	26.1	77	85	90	382.1	0.0	1903.9	
	Scrub/Shrub	6.2		23.9	30.0	30	48	65	184.9	0.0	1551.0	
M9 Total		17.0		54.4	71.4	M9 Composite CN						71
P1	Evergreen Forest		27.6	8.6	36.1	30	55	70	0.0	1517.2	598.6	
	Grassland		0.2	12.0	12.2	39	61	74	0.0	12.0	891.1	
	Scrub/Shrub		11.9	365.5	377.5	30	48	65	0.0	573.4	23759.2	
P1 Total			39.7	386.1	425.8	P1 Composite CN						64

Table 4-3 (Continued). Calculation of Composite Curve Numbers for Ala Wai Watershed

Calculation of Composite Curve Numbers for Ala Wai Watershed (Continued)

Sum of WS_Acre		New HydroGrp HSG (All D and blank to C)				Curve Number			Area x CN			Composite
SUB-BASIN	LAND USE	A	B	C	Total	A	B	C	A	B	C	CN
P2	Bare Land		0.4	0.0	0.4	72	82	87	0.0	29.9	0.0	
	Evergreen Forest		37.9	51.3	89.2	30	55	70	0.0	2084.9	3590.3	
	Grassland		0.9	19.3	20.1	39	61	74	0.0	52.6	1425.9	
	High Intensity Developed		1.5	0.0	1.5	89	92	94	0.0	133.5	0.0	
	Low Intensity Developed		9.1	0.0	9.1	77	85	90	0.0	769.4	1.4	
	Scrub/Shrub		29.8	513.0	542.8	30	48	65	0.0	1432.6	33343.	
P2 Total			79.5	583.6	663.0	P2 Composite CN						65
P3	Bare Land		0.1	0.0	0.1	72	82	87	0.0	11.7	0.0	
	Evergreen Forest		65.4	36.3	101.7	30	55	70	0.0	3599.5	2538.8	
	Grassland		6.7	3.5	10.2	39	61	74	0.0	407.5	258.1	
	High Intensity Developed		3.8	0.0	3.8	89	92	94	0.0	351.0	0.0	
	Low Intensity Developed		9.9	0.6	10.6	77	85	90	0.0	841.7	58.5	
	Scrub/Shrub		43.1	138.1	181.2	30	48	65	0.0	2070.0	8973.6	
P3 Total			129.1	178.5	307.6	P3 Composite CN						62
P4	Bare Land		1.2	1.7	2.9	72	82	87	0.0	95.5	150.2	
	Evergreen Forest		5.2	31.6	36.8	30	55	70	0.0	284.5	2215.1	
	Grassland		6.4	9.7	16.1	39	61	74	0.0	390.0	714.8	
	High Intensity Developed		17.8	12.2	30.1	89	92	94	0.0	1642.0	1147.2	
	Low Intensity Developed		26.6	25.1	51.8	77	85	90	0.0	2262.4	2263.2	
	Scrub/Shrub		12.4	138.0	150.3	30	48	65	0.0	593.0	8969.7	
P4 Total			69.5	218.4	287.9	P4 Composite CN						72
P5	Bare Land		0.2	0.1	0.4	72	82	87	0.0	18.2	12.4	
	Evergreen Forest		6.9	6.9	13.9	30	55	70	0.0	381.6	486.4	
	Grassland		1.8	4.2	6.0	39	61	74	0.0	107.7	310.4	
	High Intensity Developed		3.9	5.7	9.6	89	92	94	0.0	362.7	534.8	
	Low Intensity Developed		19.6	41.4	61.0	77	85	90	0.0	1667.5	3721.7	
	Scrub/Shrub		10.7	94.2	104.9	30	48	65	0.0	514.0	6121.2	
P5 Total			43.2	152.5	195.7	P5 Composite CN						73
P6	Bare Land		1.3	5.3	6.6	72	82	87	0.0	106.2	462.5	
	Evergreen Forest			1.0	1.0	30	55	70	0.0	0.0	73.1	
	Grassland		3.4	28.5	31.9	39	61	74	0.0	206.4	2109.5	
	High Intensity Developed		35.3	172.3	207.6	89	92	94	0.0	3248.2	16199.	
	Low Intensity Developed		19.2	79.2	98.4	77	85	90	0.0	1631.9	7131.8	
	Scrub/Shrub		1.3	89.4	90.7	30	48	65	0.0	60.4	5813.3	
P6 Total			60.4	375.9	436.3	P6 Composite CN						85
P7	Bare Land		1.0	0.0	1.0	72	82	87	0.0	84.6	0.0	
	Cultivated Land			0.2	0.2	77	86	91	0.0	0.0	20.2	
	Evergreen Forest		0.2	0.2	0.4	30	55	70	0.0	12.2	15.6	
	Grassland		12.1	1.3	13.4	39	61	74	0.0	738.4	93.7	
	High Intensity Developed		145.3	50.3	195.6	89	92	94	0.0	13369.	4724.6	
	Low Intensity Developed		47.7	13.6	61.2	77	85	90	0.0	4052.5	1221.0	
	Scrub/Shrub		12.2	0.6	12.8	30	48	65	0.0	585.6	41.8	
P7 Total			218.6	66.2	284.7	P7 Composite CN						88
W1	Evergreen Forest	0.6		5.4	6.0	30	55	70	18.7	0.0	377.5	
	Grassland	0.2		8.8	9.0	39	61	74	8.7	0.0	652.2	
	High Intensity Developed	9.4		60.2	69.6	89	92	94	833.4	0.0	5661.4	
	Low Intensity Developed	3.5		15.2	18.8	77	85	90	271.3	0.0	1371.0	
	Water			0.0	0.0	98	98	98		0.0	2.9	
W1 Total		13.7		89.7	103.4	W1 Composite CN						89
W2	Evergreen Forest	0.1		1.0	1.1	30	55	70	4.3	0.0	69.0	
	Grassland			0.8	0.8	39	61	74	0.0	0.0	56.5	
	High Intensity Developed	8.4		63.0	71.4	89	92	94	749.3	0.0	5918.8	
	Low Intensity Developed	1.0		8.5	9.4	77	85	90	74.7	0.0	762.1	
	Scrub/Shrub			0.0	0.0	30	48	65	0.0	0.0	0.1	
	Water			0.0	0.0	98	98	98		0.0	2.2	
W2 Total		9.5		73.2	82.7	W2 Composite CN						92
W3	Bare Land	0.3		0.0	0.3	72	82	87	20.7	0.0	0.0	
	Evergreen Forest	0.9		0.9	1.7	30	55	70	26.4	0.0	60.3	
	Grassland	0.1	0.0	0.0	0.1	39	61	74	3.8	0.7	0.0	
	High Intensity Developed	57.7	0.1	41.6	99.5	89	92	94	5137.	12.1	3910.8	
	Low Intensity Developed	4.3		5.5	9.9	77	85	90	334.2	0.0	497.4	
	Water			0.6	0.6	98	98	98		0.0	54.4	
W3 Total		63.3	0.1	48.5	112.0	W3 Composite CN						90
Grand Total		2053.8	3345	4979	10377.3	Ala Wai Watershed Composite CN						70

Table 4-3 (Continued). Calculation of Composite Curve Numbers for Ala Wai Watershed

4.4 Mānoa-Pālolo Model Calibration

The final HEC-HMS model for the Ala Wai Watershed consisted of 38 sub-basins. The model used the SCS runoff curve number method as the loss method to be consistent with the previous Mānoa Watershed Project hydrologic study. The model for the Ala Wai Watershed used the Clark Unit Hydrograph as the transform method for the sub-basins that are not fully urbanized. The Clark Unit Hydrograph was used as the transform method for the sub-basins in Makiki Valley (K1-K4), Mānoa Valley (M1 to M14), and Pālolo Valley (P1 to P7). The urbanized sub-basins of lower Makiki, Ala Wai Canal, and Waikīkī applied the Kinematic Wave Transform Method. Because there are insufficient rainfall and stream flow data in the low-lying areas of the Ala Wai Watershed, it was difficult to calibrate the sub-basin parameters within in the Ala Wai Canal and Waikīkī sub-watersheds. Most of the parameters of the Kinematic Wave Transform Method were based on physical measurements; it is assumed that the peak discharges of the urbanized sub-basins are correct. The actual calibration models are those of Mānoa and Pālolo valleys (Section 4.4), a pilot calibration model for Makiki valley (Section 4.5), and a reservoir calibration model for Ala Wai Canal (Section 4.7). This last model represents the calibration for the entire watershed. Figure 4-4 shows the calibration model layout for the Manoa-Palolo valleys.

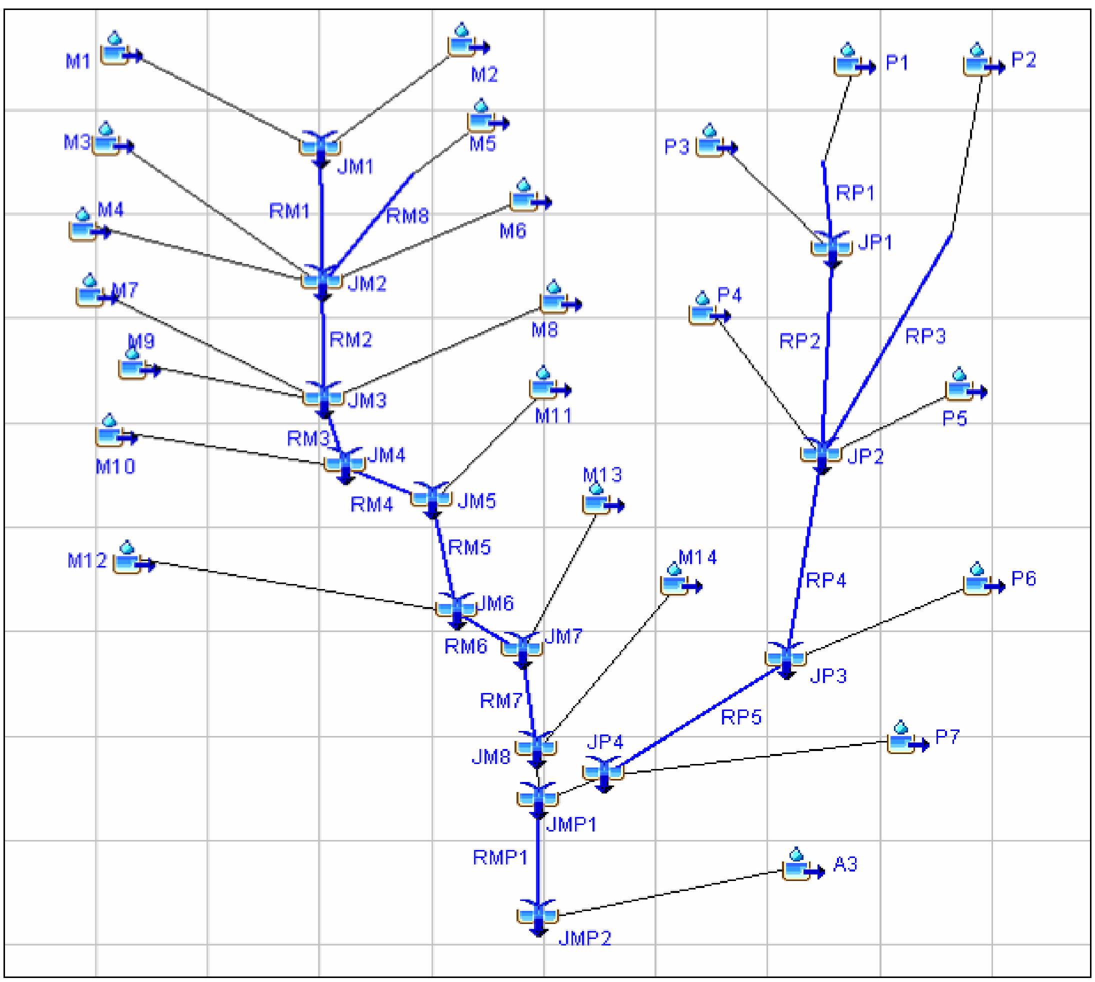

Figure 4-4. HEC-HMS Mānoa-Pālolo Calibration Model Layout

4.4.1 October 2004 Storm Calibration for Mānoa-Pālolo Area

The calibration for the storm of October 30, 2004, was based on the method used in the previous Mānoa Watershed Project hydrologic study. The calibration parameters used for the Mānoa sub-watershed in the Mānoa Watershed Project hydrologic study were used for the HEC-HMS model in the Ala Wai Watershed hydrologic study. The gage weights for sub-basins in Mānoa valley were the same as those used in the Mānoa Watershed Project described earlier. The main task of the calibration for the Ala Wai Watershed hydrologic study focused on the Pālolo sub-watershed and the area downstream of Kānewai Field gage, to the USGS stream gage 16247100. This stream gage is located on Kaimukī High School and had full stream flow records for the event. Gage weights were used for calibration purposes. The Thiessen polygon method was initially applied to determine the gage weight for each sub-basin. Figure 4-5 shows the Thiessen polygons for the October 2004 storm for the Ala Wai Watershed. The Thiessen polygon method does not account for orthographic rainfall effect in mountain areas. After taking into consideration the rainfall pattern, data quality, and storm movement and distribution, the final gage weights and relevant 24-hour rainfall of the October 2004 storm for each sub-basin were determined as shown in Table 4-4. (Note: 'MP' is used to abbreviate the Mānoa-Pālolo area.)

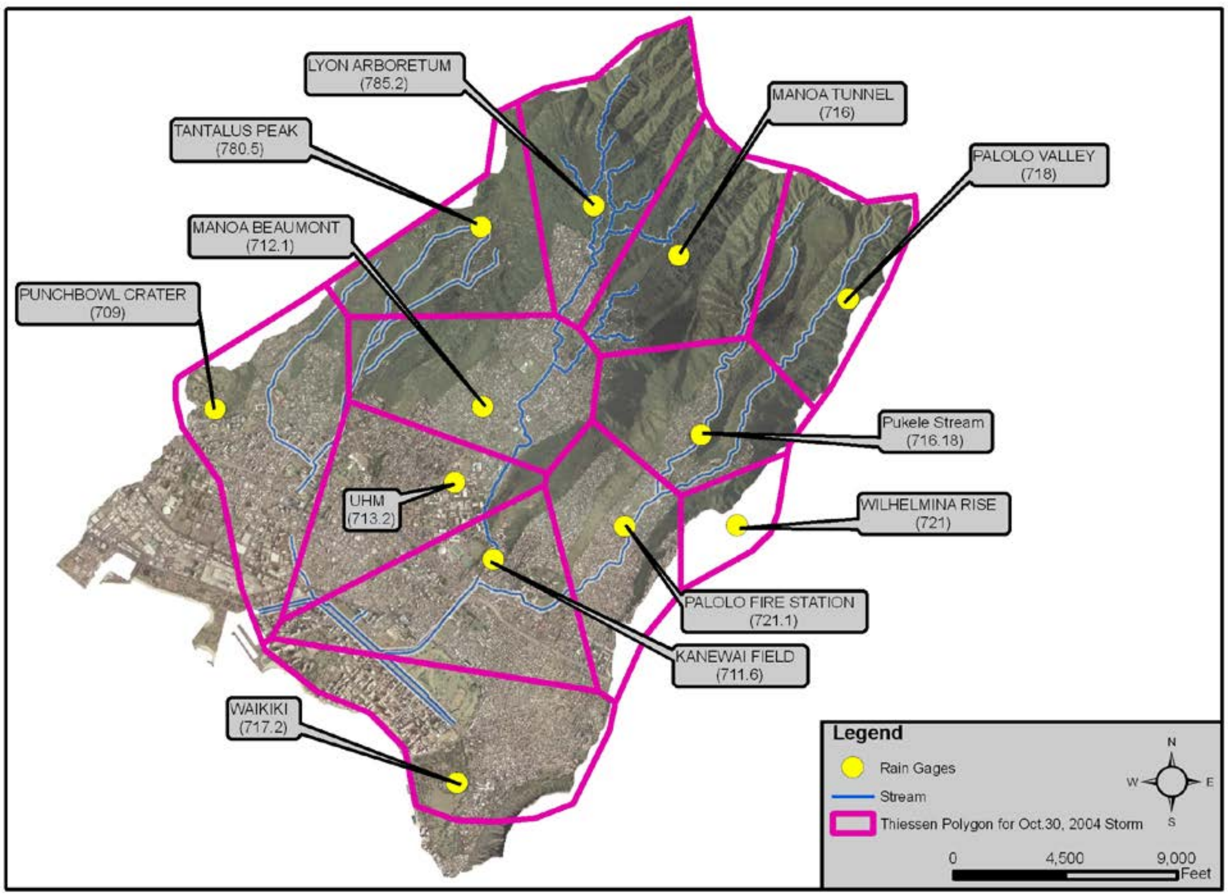

Figure 4-5. Rain Gages and Thiessen Polygons for the October 30, 2004

Meteorological Model: Gage Weights for October 30, 2004, Storm for MP

Gage weights	Thiessen Polygon (Gages in red are real time recording)											
Sub-basin	**Lyon Arboretum**	**Manoa Tunnel**	**Kanewai**	**Manoa Beaumont**	**UHM**	**Palolo Fire Sta**	**Palolo Valley**	**Pūkele**	**Tantalus Peak**	**Waikiki**	**Wilhemina Rise**	**24hr Rain (in)**
ID	785.2	716	711.6	712.1	713.2	721.1	718	Pukele	780.5	717.2	721	
Total Rainfall (in)	10.08	11.14	1.67	4.62	2.4	2.13	6.21	4.07	7.8	0.05	1.64	(in)
A3			0.7			0.2				0.1		1.60
M1	0.8	0.2										10.29
M2	0.3	0.5					0.2					9.84
M3	0.6	0.2							0.2			9.84
M4	0.5	0.2							0.3			9.61
M5	0.3	0.4					0.3					9.34
M6	0.4	0.3		0.1				0.2				8.65
M7	0.4			0.3					0.3			7.76
M8	0.3			0.4				0.3				6.09
M9	0.3			0.4					0.3			7.21
M10	0.3			0.4					0.3			7.21
M11	0.3			0.4				0.3				6.09
M12			0.1	0.5	0.4							3.44
M13			0.3	0.2	0.2	0.3						2.57
M14			0.5			0.5						1.9
P1		0.3					0.5	0.2				7.26
P2		0.1					0.6	0.3				6.06
P3		0.1					0.3	0.6				5.42
P4		0.2				0.1		0.7				5.29
P5						0.1		0.5			0.4	2.9
P6						0.9					0.1	2.08
P7			0.5			0.5						1.9

Table 4-4. Meteorological Model: Gage Weights for October 30, 2004, Storm for MP

The HEC-HMS meteorological model's parameters were calibrated using the October 30, 2004, storm data. Table 4-5 lists the final parameters for the HEC-HMS model in the Mānoa and Pālolo sub-watersheds. The parameters of the calibrated times of concentration are close to those calculated using the TR-55 method. The meteorological model used storm hydrographs for calibration and frequency based rainfall to compute the synthetic flood events.

	Loss Method --- SCS Curve Number		Transform--Clark Unit Hydrograph	
Sub-basin	Initial Abstraction (inch)	Curve Number	Time of Concentration (hour)	Storage Coefficient (hour)
A3 (Plane 1)	0.75	83	Kinematic Wave Transform	
A3 (Plane 2)	0.10	98		
M1	0.60	62	0.24	0.42
M10	0.60	76	0.26	0.60
M11	0.60	75	0.50	0.30
M12	0.30	73	0.25	0.65
M13	0.60	68	0.27	0.40
M14	1.00	84	0.15	0.30
M2	0.60	64	0.23	1.10
M3	0.60	69	0.25	0.70
M4	0.60	73	0.23	0.80
M5	0.60	63	0.31	0.90
M6	0.60	68	0.25	0.85
M7	0.60	71	0.19	1.50
M8	0.60	80	0.16	1.80
M9	0.60	75	0.17	1.50
P1	2.20	64	0.10	0.40
P2	1.20	65	0.30	0.55
P3	3.20	62	0.10	0.68
P4	1.20	72	0.10	0.30
P5	1.20	73	0.16	0.30
P6	1.20	85	0.10	0.25
P7	1.20	88	0.18	0.30

Table 4-5. Calibrated Model Parameters for October 2004 Storm for MP

At junctions JP1 and JMP2, the observed rainfall from the October 2004 storm and the modeled stream flows are shown in Figures 4-6 and 4-7. The modeled peak flows occur slightly after the observed peak flows; and the peak flow for junction JP1, Pūkele Stream gage, was modeled at a higher amount than the observed peak flow in 2004. The time of concentration values may be too high in this case. For the October 2004 storm, real-time data from M2, partial data from JM3, partial real-time data from JM7, real-time data from JP1, peak flow data from JP3, and continuous data from JMP2 were used. Because the HEC-HMS model was calibrated using the October 2004 storm data in the Mānoa Watershed Project hydrologic study (Oceanit 2008), the parameters for all Mānoa sub-basins except M14 in the Ala Wai Watershed hydrologic study were kept the same as they were in the Mānoa Watershed Project study.

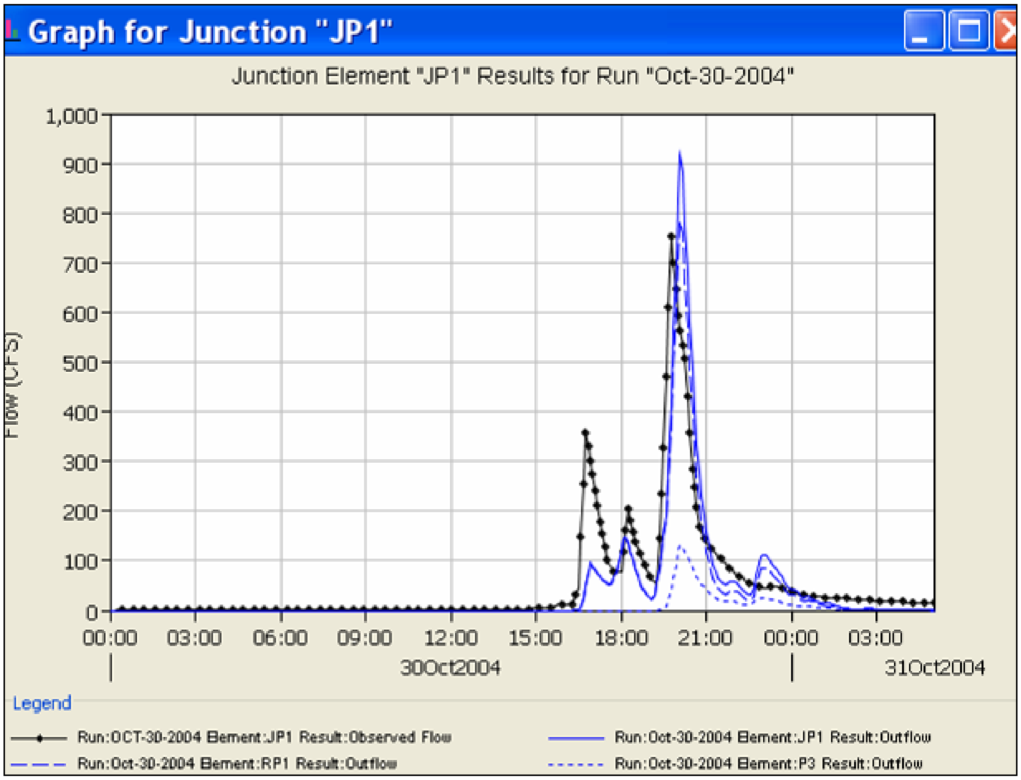

Figure 4-6. Observed and Modeled Stream Flows at Junction JP1 (Pūkele Gage [2440]) October 2004 Storm [calibration parameters: Simulated peak flow=775 cfs; observed peak flow=753 cfs; percent difference of peak discharge=2.9; percent difference of runoff volume=-15.4; peak-weighted root mean square error=96.6 cfs; Nash-Sutcliffe efficiency coefficient=0.256]

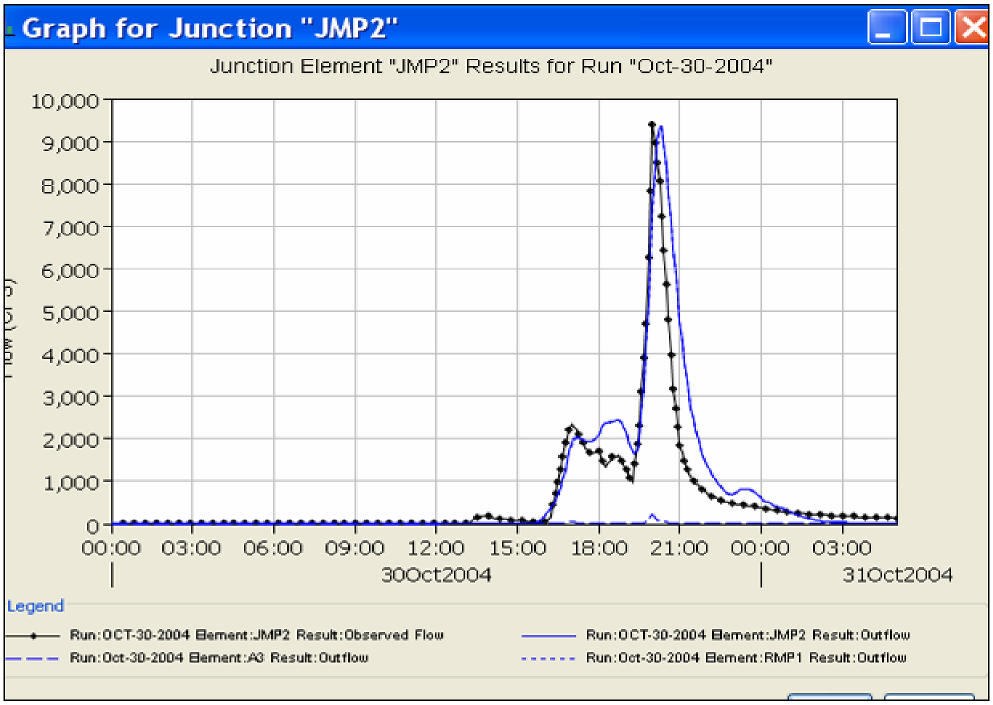

Figure 4-7. Observed and Modeled Flows at Junction JMP2 (Mānoa-Pālolo Gage [2471]) October 2004 Storm [calibration parameters: Simulated peak discharge=9,670 cfs; observed peak discharge=9,380 cfs; percent difference of peak discharge=3.1; percent difference of runoff volume=-24.6; peak-weighted root mean square error=571.2 cfs; Nash-Sutcliffe efficiency coefficient=0.826]

4.4.2 December 1967 Storm Calibration for Mānoa-Pālolo Area

The calibration for the storm of December 17–18, 1967, was based on the method used in the previous Mānoa Watershed Project hydrologic study. The calibration parameters in the Mānoa Watershed Project study for the Mānoa sub-watershed were not changed. The gage weights for sub-basins in the Mānoa sub-watershed were the same as that in the Mānoa Watershed Project study. The Thiessen polygon method was initially applied to determine the gage weight for each sub-basin. Figure 4-8 shows the Thiessen polygons for the December 1967 storm for the Ala Wai Watershed. After taking into consideration the rainfall pattern, data quality, and storm movement and distribution, the final gage weights and relevant 24-hour rainfall of the December 1967 storm for each sub-basin were determined as shown in Table 4-6. (Note: 'MP' is used to abbreviate the Mānoa-Pālolo area.)

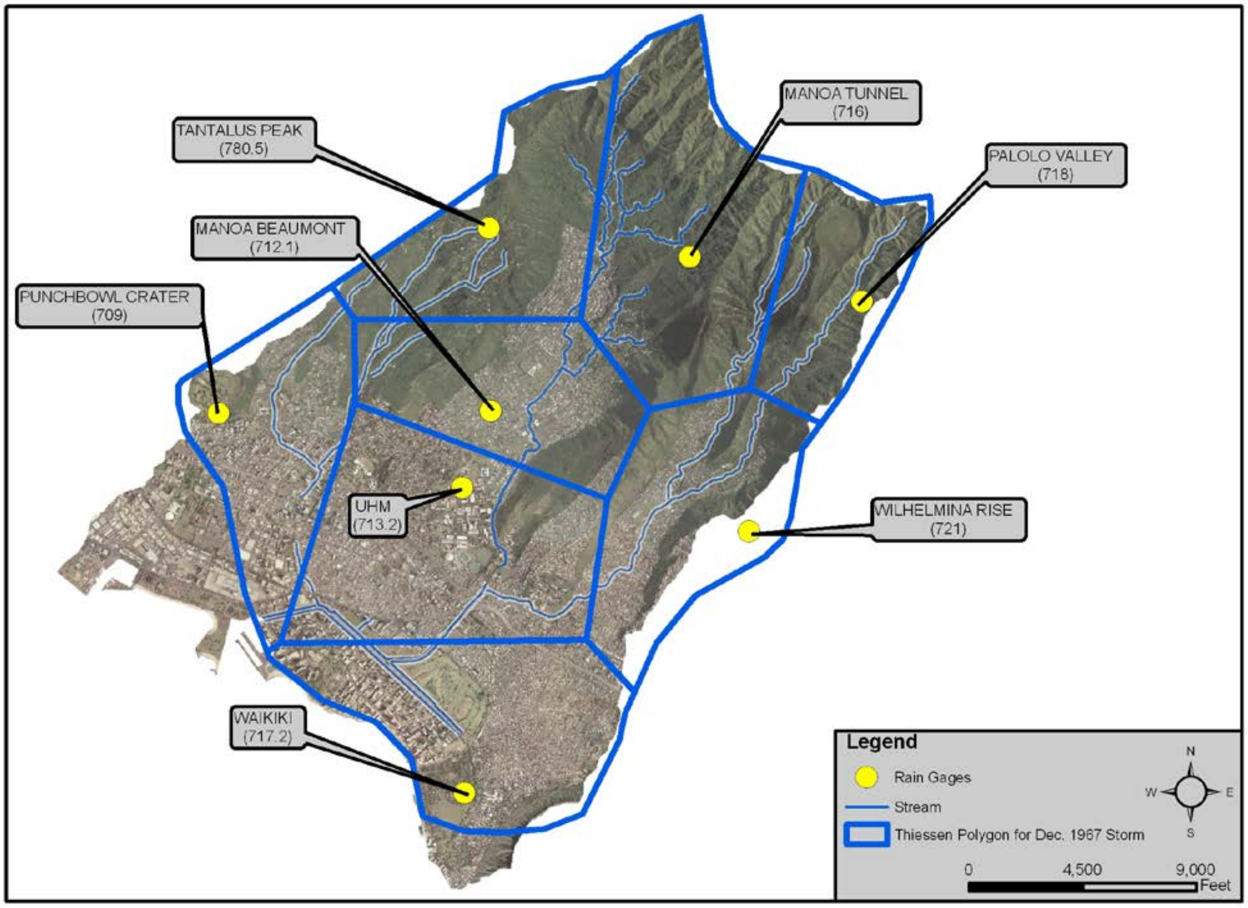

Figure 4-8. Rain Gages and Thiessen Polygons for December 1967 Storm for MP

The gage weights for the December 1967 storm, shown in Table 4-6, were calculated by considering the Thiessen polygons shown in Figure 4-8, the rainfall pattern, and the storm movement and distribution.

Meteorological Model: Gage Weights for December 17–18, 1967, Storm for MP								
Gage weights	**Thiessen Polygon**							
Sub-Basin	**Mānoa Tunnel**	**Mānoa Beaumont**	**UHM**	**Pālolo Valley**	**Tantalus Peak**	**Waikīkī**	**Wilhemina Rise**	**24-hr Rain (in)**
ID	716	712.1	713.2	718	780.5	717.2	721	
Total Rainfall (in)	10.42	9.43	9.5	10.88	8.1	8.21	9.56	
A3			0.6		0.1	0.3		8.97
M1	0.4				0.6			9.03
M2	0.6			0.2	0.2			10.05
M3	0.2	0.2			0.6			8.83
M4	0.2	0.2			0.6			8.83
M5	0.5			0.3	0.2			10.09
M6	0.3	0.5			0.2			9.46
M7		0.6			0.4			8.90
M8		0.8			0.2			9.16
M9		0.7			0.3			9.03
M10		0.8			0.2			9.16
M11		0.6	0.2		0.2			9.18
M12		0.4	0.5		0.1			9.33
M13		0.2	0.7		0.1			9.35
M14		0.2	0.6		0.1		0.1	9.35
P1	0.5			0.4	0.1			10.37
P2				0.7	0.1		0.2	10.34
P3	0.3			0.4	0.1		0.2	10.2
P4	0.3				0.1		0.6	9.68
P5					0.1		0.9	9.41
P6			0.2		0.1		0.7	9.4
P7			0.45		0.1	0.15	0.3	9.18

Table 4-6. Meteorological Model: Gage Weights for December 17–18, 1967, Storm for MP

The meteorological model used storm hydrographs for calibration and frequency based rainfall to compute the synthetic flood events. For creating the peak discharges for various return periods, the frequency storm with an intensity position at 50% was used in computing the peaks and hydrographs. Table 4-7 lists the final parameters for the HEC-HMS model in the Mānoa and Pālolo sub-watersheds. The parameters of the calibrated time of concentrations are close to those calculated using the TR-55 method. At junctions JP1, JP3, and JMP2, the modeled stream flows for the December 1967 storm show a series of stream flow peaks as shown in Figures 4-9 to 4-11. For the December 1967 storm, peak flow data from M2, data from JP1, peak flow data from JP3, and continuous data from JMP2 were used.

	Loss Method --- SCS Curve Number		Transform--Clark Unit Hydrograph	
Sub-basin	**Initial Abstraction (inches)**	**Curve Number**	**Time of Concentration (hour)**	**Storage Coefficient (hour)**
A3 (Plane 1)	1.50	83	Kinematic Wave Transform	
A3 (Plane 2)	0.15	98		
M1	0.70	62	0.22	0.30
M10	0.70	76	0.26	0.25
M11	0.70	75	0.19	0.25
M12	0.70	73	0.26	0.22
M13	0.70	68	0.26	0.30
M14	1.80	84	0.10	0.68
M2	0.50	64	0.22	0.22
M3	0.70	69	0.22	0.30
M4	0.70	73	0.22	0.30
M5	0.70	63	0.23	0.30
M6	0.70	68	0.22	0.30
M7	0.70	71	0.18	0.30
M8	0.70	80	0.15	0.30
M9	0.70	75	0.17	0.30
P1	1.20	64	0.21	0.30
P2	1.80	65	0.30	0.20
P3	1.20	62	0.16	0.25
P4	0.72	72	0.25	0.23
P5	0.65	73	0.30	0.34
P6	0.73	85	0.24	0.31
P7	1.80	88	0.10	0.80

Table 4-7. Calibrated Model Parameters for December 1967 Storm for MP

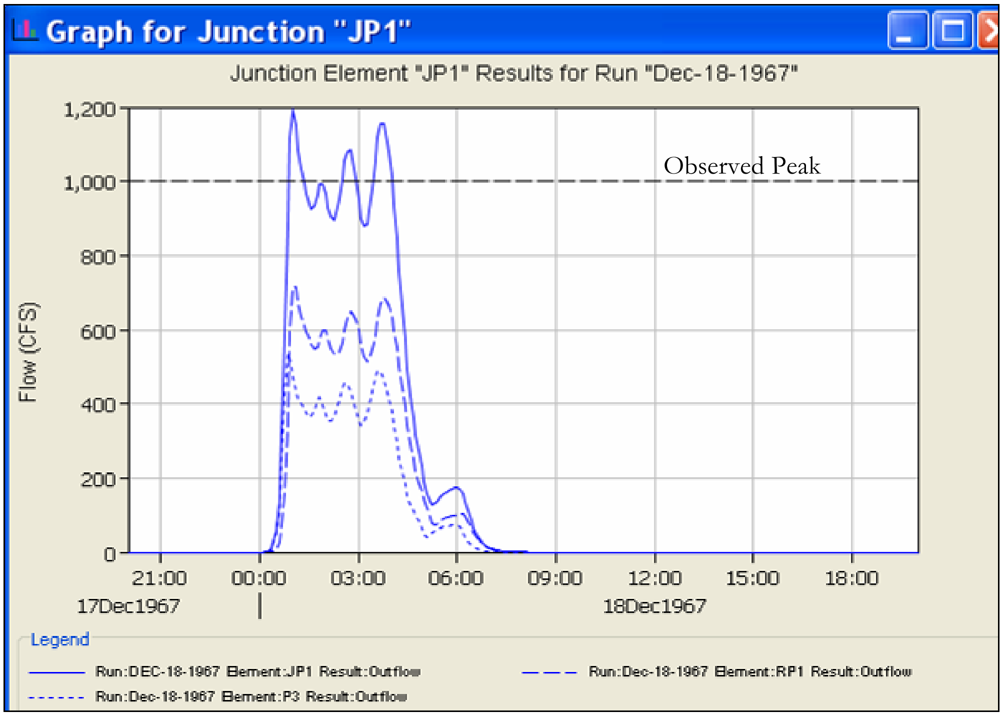

Figure 4-9. Modeled Stream Flows at Junction JP1 (Pūkele Gage [2440]) December 1967 Storm [Simulated peak discharge =1,190 cfs; observed peak discharge=1,000 cfs]

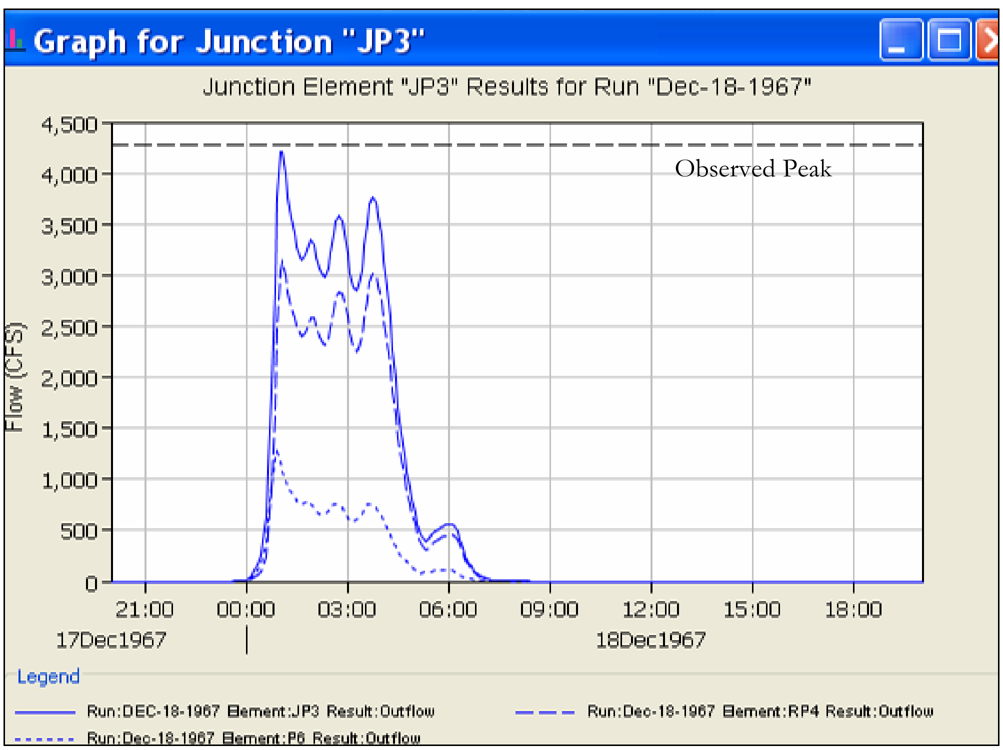

Figure 4-10. Modeled Stream Flows at JP3 (USGS Pālolo Gage [16247000]) December 1967 Storm [Simulated peak discharge =4,220 cfs; observed peak discharge=4,270 cfs]

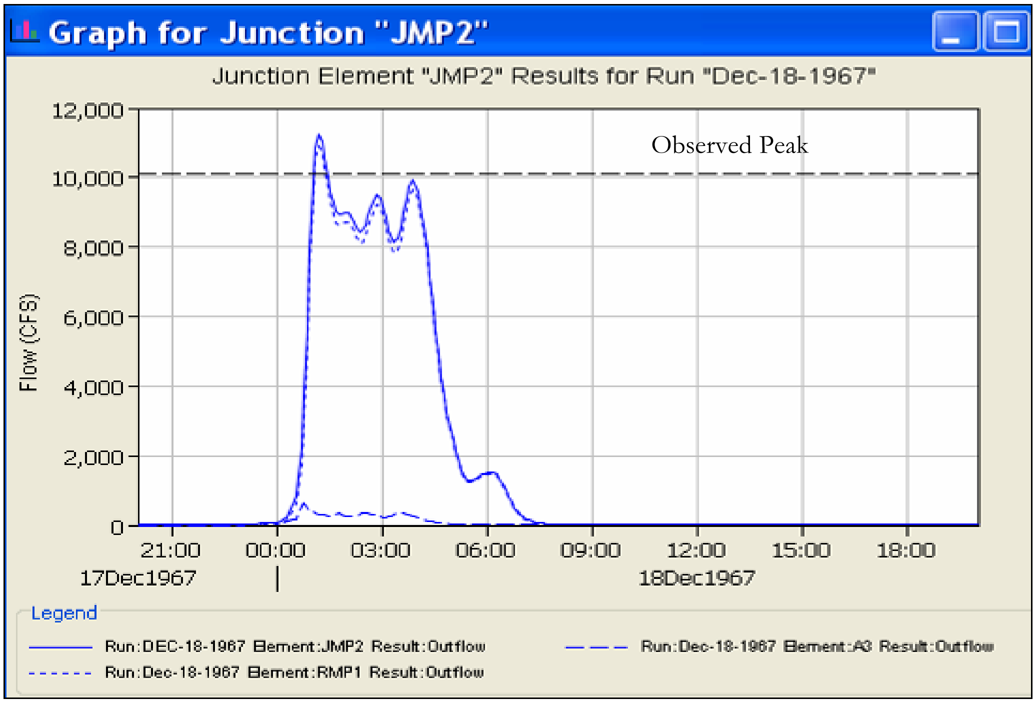

Figure 4-11. Modeled Stream Flows at JMP2 (USGS Stream Gage [16247100]) December 1967 Storm [Simulated peak discharge =11,200 cfs; observed peak discharge=10,100 cfs]

4.4.3 March 2006 Storm Calibration for Mānoa-Pālolo Area

The Thiessen polygon method was initially applied to determine the gage weight. Figure 4-12 shows the Thiessen polygons for the March 31, 2006, storm for the Ala Wai Watershed. After taking into consideration the rainfall pattern, data quality, and storm movement and distribution, the final gage weights and relevant 24-hour rainfall of the March 2006 storm for each sub-basin were determined as shown in Table 4-8. The March 31, 2006, storm is a significant example because the storm produced a small amount of rain that generated a large amount of runoff because the soils in the study area were already saturated from six weeks of heavy rains. (Note: 'MP' is used to abbreviate the Mānoa-Pālolo area.)

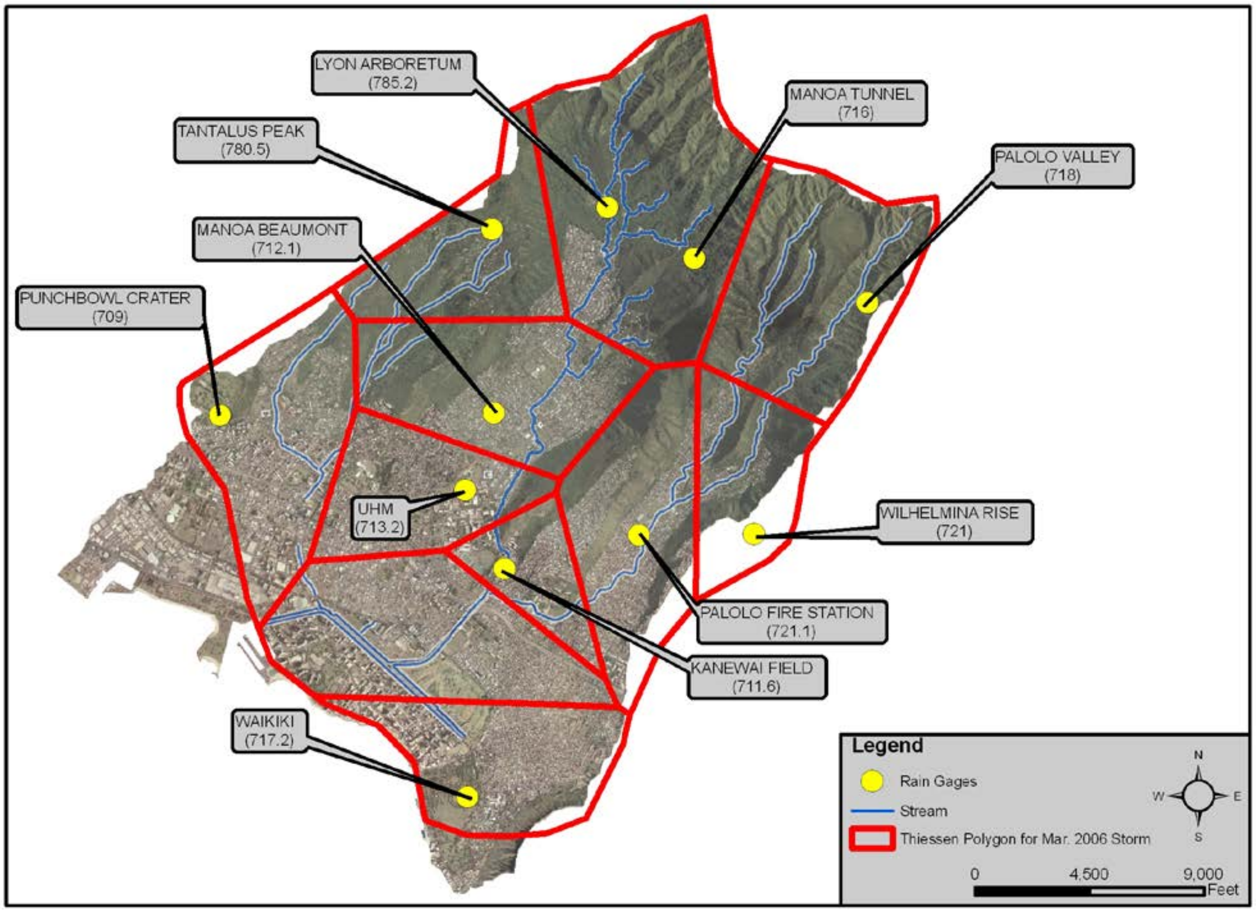

Figure 4-12. Rain Gages and Thiessen Polygons for March 2006 Storm for MP

Meteorological Model: Gage Weights for March 31, 2006 Storm for MP

Gage weights	Thiessen Polygon									
Sub-Basin	**Lyon Arboretum**	**Kānewai**	**Mānoa Beaumont**	**UHM**	**Pālolo Fire Stn.**	**Pālolo Valley**	**Tantalus Peak**	**Wilhemina Rise**	**H-1 at Kapiolani**	**24hr Rain (in)**
ID	785.2	711.6	712.1	713.2	721.1	718	780.5	721	711.7	
Total Rainfall (in)	3.35	3.49	3.25	4.75	3.00	2.84	2.60	3.49	3.53	
A3		0.1			0.1				0.8	3.47
M1	0.9						0.1			3.27
M2	0.9					0.1				3.30
M3	0.6		0.3				0.1			3.25
M4	0.1		0.6				0.3			3.07
M5	0.9				0.1					3.31
M6	0.1		0.7		0.2					3.21
M7	0.1		0.6				0.3			3.07
M8			0.8		0.2					3.20
M9	0.1		0.8	0.1						3.41
M10		0.1	0.8				0.1			3.21
M11			0.7	0.1	0.2					3.35
M12		0.2	0.3	0.5						4.05
M13		0.5		0.2	0.3					3.60
M14		0.5			0.4				0.1	3.30
P1	0.2					0.8				2.94
P2	0.1					0.7		0.2		3.02
P3	0.1				0.1	0.5		0.3		3.10
P4					0.6	0.1		0.3		3.13
P5					0.2			0.8		3.39
P6		0.1			0.8			0.1		3.10
P7		0.6			0.3				0.1	3.35

Table 4-8. Meteorological Model: Gage Weights for March 2006 Storm for MP

The HEC-HMS meteorological model's parameters were calibrated using the March 31, 2006, storm data. Table 4-9 lists the final parameters for the HEC-HMS model in the Mānoa and Pālolo sub-watersheds. The parameters of the calibrated time of concentrations are close to those calculated using the TR-55 method. The meteorological model used storm hydrographs for calibration and frequency based rainfall to compute the synthetic flood events.

	Loss Method --- SCS Curve Number		Transform Method --- Clark Unit Hydrograph	
Sub-basin	**Initial Abstraction (inches)**	**Curve Number**	**Time of Concentration (hour)**	**Storage Coefficient (hour)**
A3 (Plane 1)	0	92	Kinematic Wave Transform	
A3 (Plane 2)	0	98		
M1	0	88	0.20	0.10
M10	0	92	0.18	0.10
M11	0	92	0.10	0.10
M12	0	92	0.20	0.10
M13	0	90	0.10	0.10
M14	0	90	0.12	0.10
M2	0	70	0.32	0.12
M3	0	92	0.20	0.10
M4	0	92	0.10	0.10
M5	0	72	0.20	0.10
M6	0	75	0.15	0.10
M7	0	80	0.15	0.10
M8	0	92	0.10	0.10
M9	0	92	0.10	0.10
P1	0	64	0.10	0.10
P2	0	65	0.10	0.10
P3	0	62	0.10	0.10
P4	0	72	0.10	0.10
P5	0	73	0.10	0.10
P6	0	85	0.10	0.11
P7	0	90	0.10	0.10

Table 4-9. Calibrated Model Parameters for March 2006 Storm

The modeled stream flow for the March 2006 storm in M2 and at JMP2 show a small flow peak flow followed by a higher peak flow, as shown in Figures 4-13 and 4-14. The modeled peak flows are higher and earlier than the observed flows; however, the highest peaks match well. Due to the extremely saturated soil within the study area during this storm, the sub-basins' curve numbers were allowed to change to match the peak at JMP2 for calibration.

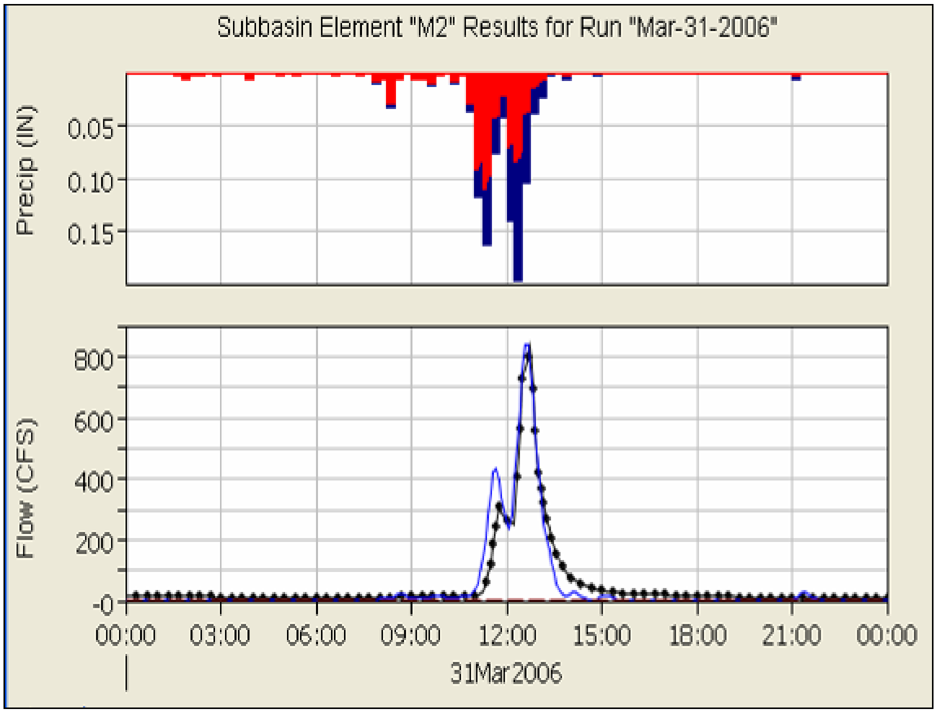

Figure 4-13. Observed and Modeled Stream Flows at Waiakeakua Stream (Sub-basin M2), March 2006 Storm [calibration parameters: Simulated peak discharge=837 cfs; observed peak discharge=832 cfs; percent difference of peak discharge=0.6; percent difference of runoff volume=-8.7; peak-weighted root mean square error=36.2 cfs; Nash-Sutcliffe efficiency coefficient=0.914]

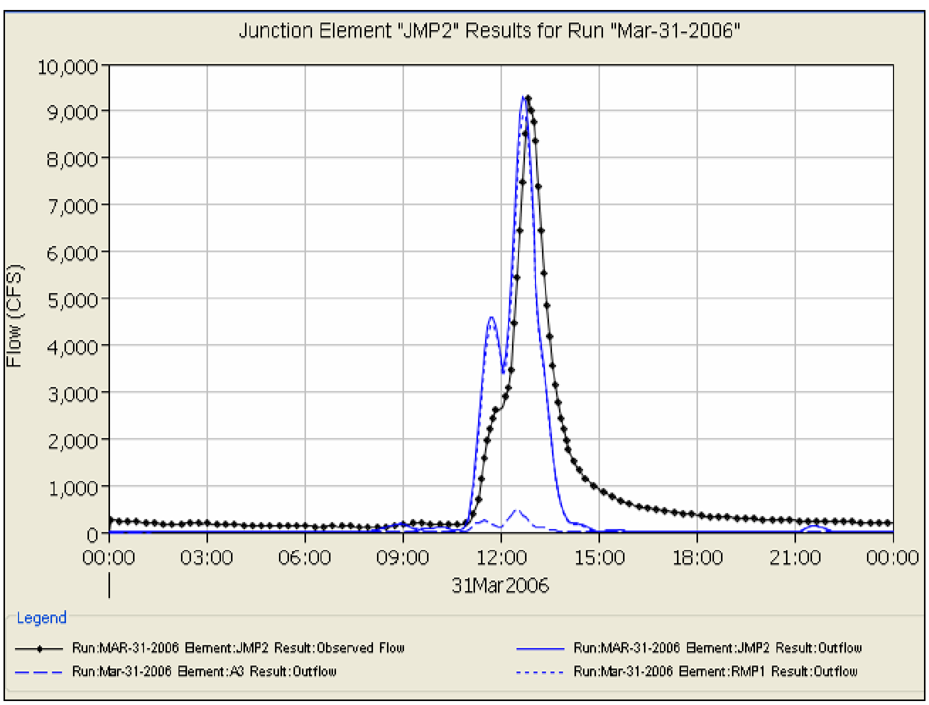

Figure 4-14. Observed and Modeled Stream Flows at JMP2 (USGS Stream Gage [16247100]), March 2006 Storm [calibration parameters: Simulated peak discharge=9,200 cfs; observed peak discharge=9,260cfs; percent difference of peak discharge=-0.7; percent difference of runoff volume=-28.6; peak-weighted root mean square error=734.2 cfs; Nash-Sutcliffe efficiency coefficient=0.776]

4.4.4 Final Loss and Transform Parameters for Mānoa-Pālolo Area

The loss method was determined by using the NRCS runoff CN method to take advantage of the results from the Mānoa Watershed Project hydrologic study. The parameters of initial abstraction were optimized for the Waiakeakua sub-basin and then were assigned to all the other sub-basins. Impervious parameters were set to zero because the percentage of the sub-basin that is impervious is specified in the CN. The optimization was used in each individual calibration (see Section 4.2). The final model parameters were the weighted average ones.

There is more confidence with the storms of October 30, 2004, and December 17–18, 1967, and less confidence with the storm of March 31, 2006. More weighting values were given to the calibrated parameters of the storm events of October 2004 and December 1967. The calibrated parameters of the October 2004 and December 1967 storm events were assigned twice the weight of the calibrated parameters for the March 31, 2006, storm. The finalized calibrated parameters of the HEC-HMS model were weighted as (2*2004 + 2*1967 + 1* 2006)/5. The weighted averaged loss method and transform method parameters for the Mānoa-Pālolo area are listed in Tables 4-10 and 4-11.

Curve Number Loss Method Calibration: Manoa-Palolo basin model								
	October-30-2004		December-18-1967		March-31-2006		Weighted Average	
Sub-basin	Initial Abstraction (inch)	Curve Number	Initial Abstraction (inch)	Curve Number	Initial Abstraction (inch)	Curve Number	Initial Abstraction (inch)	Curve Number
A3 (Plane 1)	0.75	83	1.50	83	0	92	**0.90**	**85**
A3 (Plane 2)	0.10	98	0.15	98	0	98	**0.10**	**98**
M1	0.60	62	0.70	62	0	88	**0.52**	**67**
M10	0.60	76	0.70	76	0	92	**0.52**	**79**
M11	0.60	75	0.70	75	0	92	**0.52**	**78**
M12	0.30	73	0.70	73	0	92	**0.40**	**77**
M13	0.60	68	0.70	68	0	90	**0.52**	**72**
M14	1.00	84	1.80	84	0	90	**1.12**	**85**
M2	0.60	64	0.50	64	0	70	**0.44**	**65**
M3	0.60	69	0.70	69	0	92	**0.52**	**74**
M4	0.60	73	0.70	73	0	92	**0.52**	**77**
M5	0.60	63	0.70	63	0	72	**0.52**	**65**
M6	0.60	68	0.70	68	0	75	**0.52**	**69**
M7	0.60	71	0.70	71	0	80	**0.52**	**73**
M8	0.60	80	0.70	80	0	92	**0.52**	**82**
M9	0.60	75	0.70	75	0	92	**0.52**	**78**
P1	2.20	64	1.20	64	0	64	**1.36**	**64**
P2	1.20	65	1.80	65	0	65	**1.20**	**65**
P3	3.20	62	1.20	62	0	62	**1.76**	**62**
P4	1.20	72	0.72	72	0	72	**0.77**	**72**
P5	1.20	73	0.65	73	0	73	**0.74**	**73**
P6	1.20	85	0.73	85	0	85	**0.77**	**85**
P7	1.20	88	1.80	88	0	90	**1.20**	**88**

Table 4-10. Final HEC-HMS Model Loss Method Parameters

Clark Unit Hydrograph Transform Method Calibration: Manoa-Palolo basin model								
	October-30-2004		December-18-1967		March-31-2006		Average Values	
Sub-basin	T_c (Hour)	S_c (Hour)	T_c (Hour)	S_c (Hour)	T_c (Hour)	S_c (Hour)	T_c (Hour)	S_c (Hour)
M1	0.24	0.42	0.22	0.30	0.20	0.10	**0.22**	**0.31**
M2	0.23	1.10	0.22	0.22	0.32	0.12	**0.24**	**0.55**
M3	0.25	0.70	0.22	0.30	0.20	0.10	**0.23**	**0.42**
M4	0.23	0.80	0.22	0.30	0.10	0.10	**0.20**	**0.46**
M5	0.31	0.90	0.23	0.30	0.20	0.10	**0.26**	**0.50**
M6	0.25	0.85	0.22	0.30	0.15	0.10	**0.22**	**0.48**
M7	0.19	1.50	0.18	0.30	0.15	0.10	**0.18**	**0.74**
M8	0.16	1.80	0.15	0.30	0.10	0.10	**0.14**	**0.86**
M9	0.17	1.50	0.17	0.30	0.10	0.10	**0.16**	**0.74**
M10	0.26	0.60	0.26	0.25	0.18	0.10	**0.24**	**0.36**
M11	0.50	0.30	0.19	0.25	0.10	0.10	**0.30**	**0.24**
M12	0.25	0.65	0.26	0.22	0.20	0.10	**0.24**	**0.37**
M13	0.27	0.40	0.26	0.30	0.10	0.10	**0.23**	**0.30**
M14	0.15	0.30	0.10	0.68	0.12	0.10	**0.12**	**0.41**
P1	0.10	0.40	0.21	0.30	0.10	0.10	**0.14**	**0.30**
P2	0.30	0.55	0.30	0.20	0.10	0.10	**0.26**	**0.32**
P3	0.10	0.68	0.16	0.25	0.10	0.10	**0.12**	**0.39**
P4	0.10	0.30	0.25	0.23	0.10	0.10	**0.16**	**0.23**
P5	0.16	0.30	0.30	0.34	0.10	0.10	**0.20**	**0.28**
P6	0.10	0.25	0.24	0.31	0.10	0.11	**0.16**	**0.25**
P7	0.18	0.30	0.10	0.80	0.10	0.10	**0.13**	**0.46**

Table 4-11. Final HEC-HMS Transform Method Parameters
Note: T_c is the time of concentration, S_c is the storage coefficient

4.5 Makiki Model Calibration

Data from the USGS Makiki Stream Gage (16238000) at King Street Bridge was used to calibrate the Makiki HEC-HMS model. This gage measured two peaks in 2004. One peak was 487 cfs recorded on February 28, 2004, and the other peak was 1,000 cfs recorded on October 30, 2004. There were no sufficient rainfall data for February 28, 2004, so the 1,000 cfs peak on October 30, 2004, was used to calibrate the Makiki HEC-HMS model. The Thiessen polygons for October 30, 2004, in the Makiki sub-watershed can be seen in Figure 4-15, and they are the same as those for the Mānoa-Pālolo calibration of the October 30, 2004 storm. Because there was no timing rainfall gage within the Makiki sub-watershed, the Lyon Arboretum rainfall gage (785.2) was selected as the time weight gage for all sub-basins in the sub-watershed (see Table 3.1 for rainfall gage information). (Note: 'K' is used to abbreviate for the Makiki sub-watershed.)

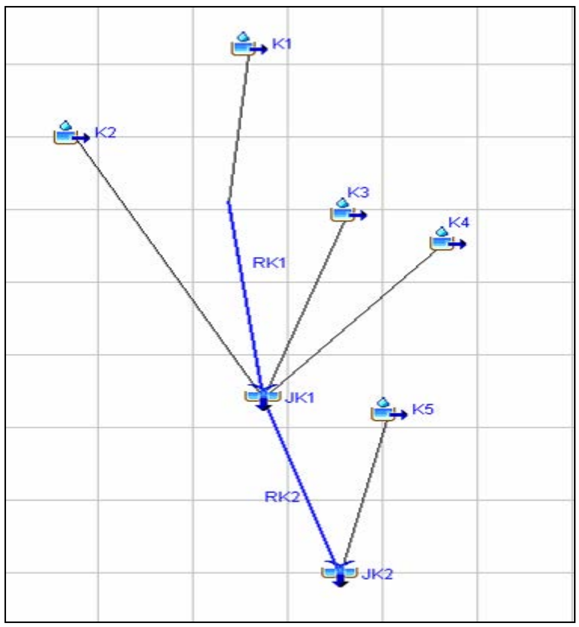

Figure 4-15. HEC-HMS Makiki Sub-Watershed Calibration Model Layout

4.5.1 October 2004 Storm Calibration for the Makiki Sub-Watershed

Due to the limited data available for the Makiki sub-watershed, the October 30, 2004, storm data were the only storm data used to calibrate the Makiki meteorological model. The calibration was based on the peak discharge of 1,000 cfs at King Street Bridge (USGS stream gage 16238000). Figure 4-5 shows the Thiessen polygons for the October 2004 storm for the Ala Wai Watershed. The Thiessen polygon method does not account for orthographic rainfall effect in mountain areas. The rainfall pattern, data quality, and storm movement and distribution were taken into consideration for the final gage weights. For the Makiki sub-watershed, the final gage weights for the 24-hour rainfall of the October 2004 storm were calculated and are given in Table 4-12.

Gage Weights for Makiki Sub-Watershed October 30, 2004, Storm						
Gage Weights	Thiessen Polygons (Gages in red recorded)					
Sub-Basin ID	Lyon Arboretum	Mānoa Beaumont	UHM	Tantalus Peak	Punchbowl Crater	24-hr Rain (inch)
	785.2	712.1	713.2	780.5	709	
Total Rainfall (in)	**10.08**	**4.62**	**2.4**	**7.8**	**0.05**	
K1	0.1	0.3		0.5	0.1	**6.39**
K2	0.1			0.2	0.7	**3.23**
K3	0.1	0.4	0.2		0.3	**3.62**
K4	0.1	0.4	0.4		0.1	**3.91**
K5	0.1		0.3		0.6	**2.30**

Table 4-12. Gage Weights for October 2004 Storm Makiki Sub-Watershed

The HEC-HMS meteorological model's parameters were calibrated using the October 30, 2004, storm data for the Makiki sub-watershed. Table 4-13 lists the calibrated parameters for the HEC-HMS model in the Makiki sub-watershed. The parameters of the calibrated times of concentration are close to those calculated using the TR-55 method. The meteorological model used storm hydrographs for calibration and frequency-based rainfall to compute the synthetic flood events. The final model parameters for the Makiki sub-watershed are given in Table 4-14.

Sub-basin	Initial Loss (inch)	Curve Number	Time of Concentration (hour)	Storage Coefficient (hour)
K1	1	42	0.18	0.45
K2	1	62	0.23	0.25
K3	1	68	0.12	0.2
K4	1	68	0.12	0.25
K5 (Plane 1)	0.7	85		
K5 (Plane 2)	0.1	98		

Table 4-13. Calibrated Parameters of Makiki Sub-Watershed

Sub-basin	Initial Loss (inch)	Curve Number	Time of Concentration (hour)	Storage Coefficient (hour)
K1	1	42	0.18	0.45
K2	1	62	0.23	0.35
K3	1	68	0.12	0.32
K4	1	68	0.12	0.35
K5 (Plane 1)	0.7	85		
K5 (Plane 2)	0.1	98		

Table 4-14. Finalized Parameters in HEC-HMS Model Makiki Sub-Watershed

The calibrated and final parameters of the HEC-HMS Model for the Makiki sub-watershed only differ by a few storage coefficients. These differences are due to the differing use and character of land in upper versus lower Makiki. The land use of the upper Makiki sub-watershed, a natural area with preservation land use, is similar to those of the upper Mānoa and Pālolo sub-watersheds. This similarity is reflected in the storage coefficient calculated. That is, the calibrated Clark Unit Hydrograph storage coefficient of the K1 sub-basin is 0.45 hour (hr), as shown in Tables 4-13 and 4-14, and the calibrated Clark Unit Hydrograph storage coefficients of sub-basins M5, P2, and M2, are 0.50, 0.32, and 0.57 hr respectively. In contrast, for lower Makiki sub-basins of K2, K3, and K4, the storage coefficients were increased slightly to match the calibrated storage coefficients in the Mānoa and Pālolo sub-watersheds. Figure 4-16 shows the modeled stream flows for JK2.

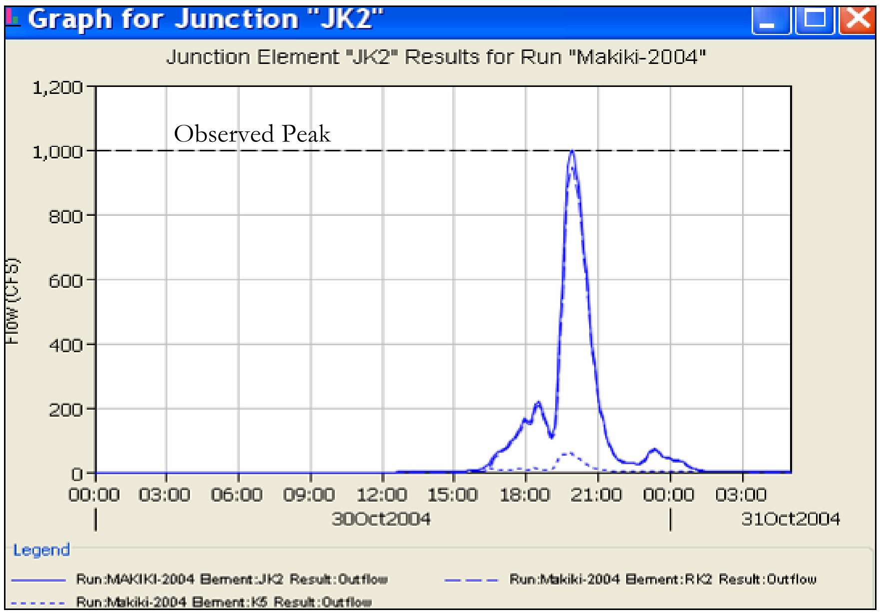

Figure 4-16. Modeled Stream Flows at JK2 (King Street Bridge, USGS stream gage 16238000) [Simulated peak discharge =1,000 cfs; observed peak discharge=1,000 cfs]

4.6 Kinematic Wave Transform Method Parameters

The Kinematic Wave Transform Method was used for the urbanized sub-basins. The Kinematic Wave technique is widely accepted for use in urbanized runoff modeling (USACE, 2001) because the parameters for various elements constituting the model are directly related to measurable, physical basin features. Parameters such as storm drain catchment length, drainage area, roughness, slope, and channel geometry are used to define the flow of water over basin surfaces into the stream channel. For the urbanized sub-basins, two overland flow plane elements were used to represent pervious land areas such as lawns and gardens and impervious areas such as streets and roofs. In this study, a sub-basin was modeled by combining two overland planes, a collector channel, and a main channel. The lengths, slopes, and roughness coefficients of the overland flow planes were based on the average of several values within the sub-watershed. Table 4-15 lists the values of the flow planes. Urbanized watersheds typically have various storm drainage systems, man-made channels, and natural channels. To model complex urban systems in a manageable fashion, the concept of typical collector channels was employed. The collector system was formulated from average parameters, in the sub-watershed. Tables 4-16 and 4-17 summarize the values of the collector channels and main channels.

In order to use the composite runoff curve number in a kinematic wave model, the sub-watershed must be divided into its pervious and impervious components. A curve number of 98 was used for the impervious areas (USACE 1973). The following equation can be applied to calculate the adjusted pervious curve number. The adjusted pervious curve number was used as the loss rate for the pervious areas.

$$X = \frac{CNc - 98 \times f}{1 - f}$$

$$\begin{aligned} \textit{Where} \quad X &= \textit{Adjusted pervious curve number} \\ CNc &= \textit{Composite curve number} \\ f &= \textit{total percent impervious}, 0 \le f \le 1 \end{aligned}$$

Kinematic Wave Transform Flow Planes for Urbanized Sub-Basins					
Sub-basin	Initial Abstraction (inch)	CN	Area (%)	Composite CN	Adjusted Pervious CN
A1 (Plane 1)	1.00	78	70	84	78
A1 (Plane 2)	0.10	98	30		
A2 (Plane 1)	0.75	86	65	90	86
A2 (Plane 2)	0.05	98	35		
A3 (Plane 1)	0.90	83	60	89	83
A3 (Plane 2)	0.10	98	40		
A4 (Plane 1)	0.75	76	60	85	76
A4 (Plane 2)	0.10	98	40		
A5 (Plane 1)	0.75	81	60	88	81
A5 (Plane 2)	0.10	98	40		
A6 (Plane 1)	1.00	69	90	72	69
A6 (Plane 2)	0.10	98	10		
A7 (Plane 1)	1.00	76	60	85	76
A7 (Plane 2)	0.10	98	40		
A8 (Plane 1)	0.75	86	50	92	86
A8 (Plane 2)	0.10	98	50		
K5 (Plane 1)	1.00	85	75	88	85
K5 (Plane 2)	0.10	98	25		
K6 (Plane 1)	1.20	80	60	87	80
K6 (Plane 2)	0.10	98	40		
W1 (Plane 1)	0.80	83	60	89	83
W1 (Plane 2)	0.10	98	40		
W2 (Plane 1)	1.00	86	50	92	86
W2 (Plane 2)	0.10	98	50		
W3 (Plane 1)	0.95	82	50	90	82
W3 (Plane 2)	0.10	98	50		

Table 4-15. Kinematic Wave Transform Flow Planes for Urbanized Sub-Basins

Kinematic Wave Collector Channels								
Sub-basin	**Length (ft)**	**Slope (ft/ft)**	**Manning's n**	**Area (mi^2)**	**Shape**	**Diameter (ft)**	**Width (ft)**	**Side Slope (xH:1V)**
A1 (Sub-Collector)								
A1 (Collector)	1200	0.015	0.016	0.0207	Circle	3		
A2 (Sub-Collector)								
A2 (Collector)	2500	0.01	0.015	0.03	Circle	4		
A3 (Sub-Collector)								
A3 (Collector)	2800	0.06	0.018	0.03	Circle	3		
A4 (Sub-Collector)								
A4 (Collector)	2200	0.004	0.014	0.03	Circle	4		
A5 (Sub-Collector)								
A5 (Collector)	1200	0.035	0.018	0.03	Circle	2.5		
A6 (Sub-Collector)								
A6 (Collector)	750	0.006	0.06	0.01	Trapezoid		2	10
A7 (Sub-Collector)								
A7 (Collector)	1200	0.035	0.018	0.03	Circle	1.5		
A8 (Sub-Collector)								
A8 (Collector)	2400	0.003	0.015	0.03	Circle	4		
K5 (Sub-Collector)								
K5 (Collector)	1000	0.005	0.016	0.02	Circle	2		
K6 (Sub-Collector)								
K6 (Collector)	2600	0.005	0.018	0.035	Circle	3		
W1 (Sub-Collector)								
W1 (Collector)	1200	0.0015	0.015	0.025	Circle	1.5		
W2 (Sub-Collector)								
W2 (Collector)	800	0.0025	0.015	0.015	Circle	3		
W3 (Sub-Collector)								
W3 (Collector)	900	0.002	0.015	0.015	Circle	3		

Table 4-16. Kinematic Wave Collector Channels

Kinematic Wave Main Channels								
Sub-basin	**Route Upstream**	**Length (ft)**	**Slope (ft/ft)**	**Shape**	**Manning's n**	**Diameter (ft)**	**Width (ft)**	**Slope (xH:1V)**
A1	No	1200	0.067	Circle	0.016	3		
A2	Yes	3600	0.001	Trapezoid	0.015		255	0
A3	Yes	800	0.0075	Trapezoid	0.03		50	5
A4	Yes	3100	0.001	Trapezoid	0.035		50	5
A5	No	5800	0.021	Circle	0.015	4		
A6	No	3650	0.001	Trapezoid	0.022		255	0
A7	No	6200	0.0267	Circle	0.015	4		
A8	Yes	2200	0.0015	Trapezoid	0.015		155	0
K5	Yes	700	0.056	Trapezoid	0.035		20	0
K6	Yes	3050	0.049	Trapezoid	0.035		20	0
W1	No	2800	0.0015	Circle	0.016	2		
W2	No	1500	0.0028	Circle	0.014	3		
W3	No	2100	0.0028	Circle	0.015	3		

Table 4-17. Kinematic Wave Main Channels

4.7 Reservoir and Reach Modeling

A number of assumptions were made during hydrologic modeling using the HEC-HMS method. These assumptions were made regarding the reservoir, reach, and junction modeling for the Ala Wai Watershed study area. Building upon the other sub-watershed model calibration, this final model represents the calibration of the entire watershed.

4.7.1 Ala Wai Canal as Reservoir

In order to consider backwater effect caused by the ocean tides, the Ala Wai Canal was modeled as a reservoir by assuming there is an imaginary boundary between the mouth of Canal and the ocean. "A reservoir is an element with one or more inflow and one computed outflow and is modeled by the assumption that water surface in the reservoir is level" (USACE 2008). The routing method was selected as the outflow structure. The size and type of imaginary outlet structure were mainly selected based on the cross section at the mouth of Ala Wai Canal. Noda and Associates (1994) study showed that the channel is a rectangular shape with a dimension of 152 feet x 14 feet near Ala Moana Bridge. The GeoRAS model also created similar cross sections at the mouth of the canal. The inlet elevation for this outlet structure was selected as -6.2 feet which was obtained from the October 30, 2004 storm calibration; then the rise of structure should be about 8 ft. The span of the structure was selected as 152 ft to match the field measurement. Figure 4-17 lists the reservoir model settings and Figure 4-18 shows its related outflow structure. There is no tide gage at the Ala Wai Canal mouth, the tide gage in Honolulu Harbor (NOAA tide level station 1612340) was used to represent the tail water effect. Consequently, the specified stage method was used to represent the main tail water. The elevation-storage function for the reservoir (Ala Wai Canal) was estimated by applying the bathymetric survey data for Ala Wai Canal conducted by Oceanit (2008c) and the LiDAR data for surrounding areas, as show in Table 4-18 and Figure 4-18.

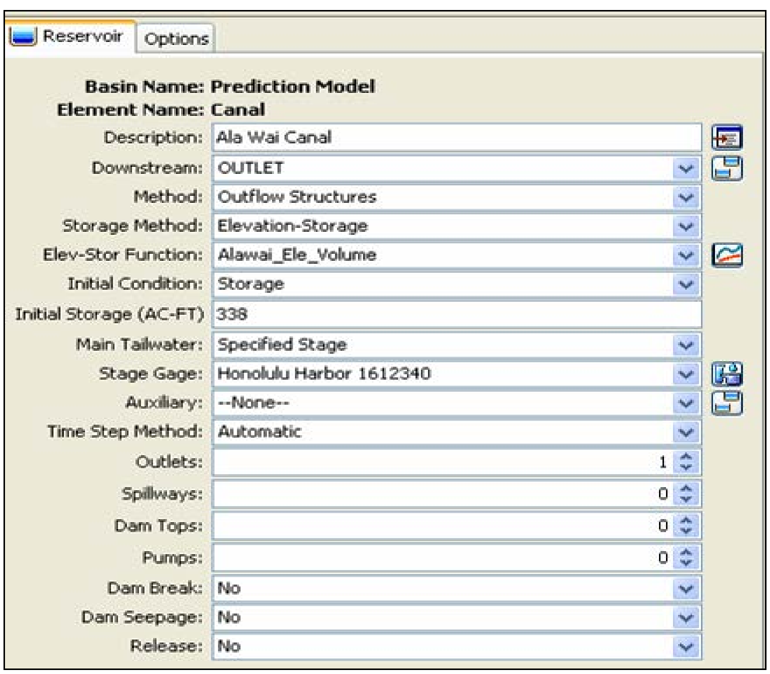

Figure 4-17. Model Settings for Reservoir (Ala Wai Canal)

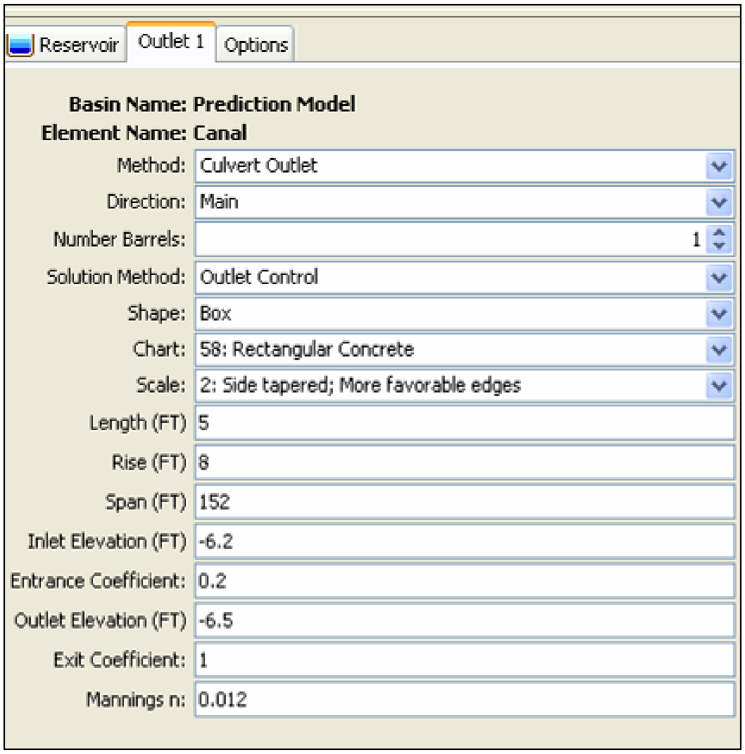

Figure 4-18 Model Settings for the Outflow Structure of Ala Wai Reservoir

Elevation-Storage Function Data

Elevation (ft)	Storage (acre-ft)	Elevation (ft)	Storage (acre-ft)	Elevation (ft)	Storage (acre-ft)
-15.5	0	-7.5	46.21	1	374.51
-15	0.01	-7	58.81	1.5	399.14
-14.5	0.03	-6.5	72.87	2	424.53
-14	0.09	-6	88.32	2.5	451.72
-13.5	0.22	-5.5	105.22	3	481.35
-13	0.43	-5	123.38	3.5	516.12
-12.5	0.74	-4.5	142.62	4	565.43
-12	1.2	-4	162.89	4.5	649.36
-11.5	1.89	-3.5	183.98	5	790.16
-11	2.99	-3	205.54	5.5	994.63
-10.5	4.89	-2.5	227.5	6	1260.41
-10	8.01	-2	249.61	6.5	1576.57
-9.5	12.49	-1.5	271.72	7	1930.93
-9	18.44	-1	293.83	7.5	2313.63
-8.5	25.97	-0.5	315.94	8	2718.57
-8	35.25	0	338.05		
-7.5	46.21	0.5	350.41		

Table 4-18. Elevation-Storage Curve Function Data

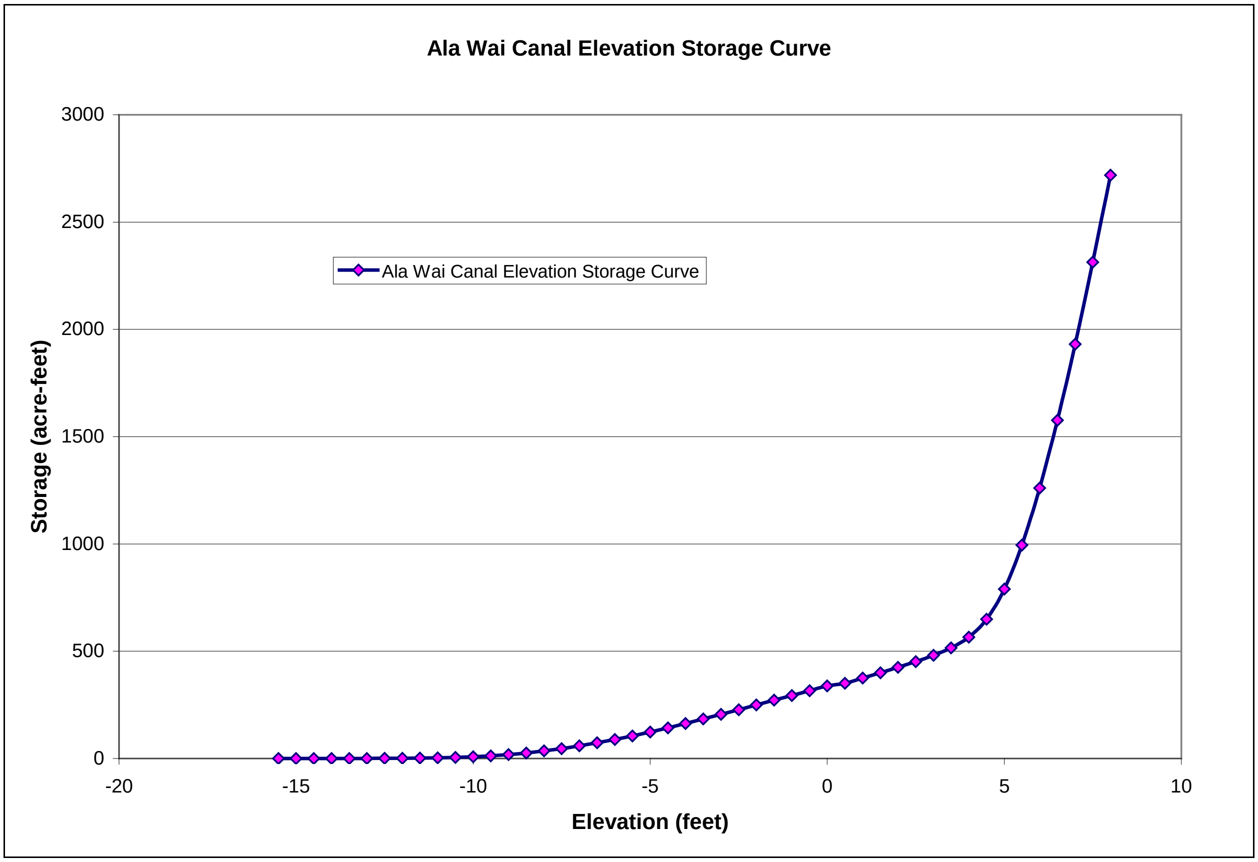

Figure 4-19. Elevation Storage Curve for Ala Wai Canal

The reservoir model was calibrated using the observed stage in the Ala Wai Canal from the October 2004 and December 1967 storm events. For modeling of the 2004 storm, the recorded stream flow hydrograph at USGS stream gage 16247100 was used to represent the inflow from upstream of Manoa and Palolo Streams; and Makiki calibrated model hydrograph at JK2 (USGS stream gage 16238000) was used to represent the inflow from Makiki area. Figure 4-20 illustrates the HEC-HMS model layout for October 30, 2004 storm calibration. Figure 4-21 shows the modeled and observed stages in Ala Wai Canal. The stage peak time matched very well at about 20:15pm with only 0.1 foot difference between the observed stage and the modeled stage.

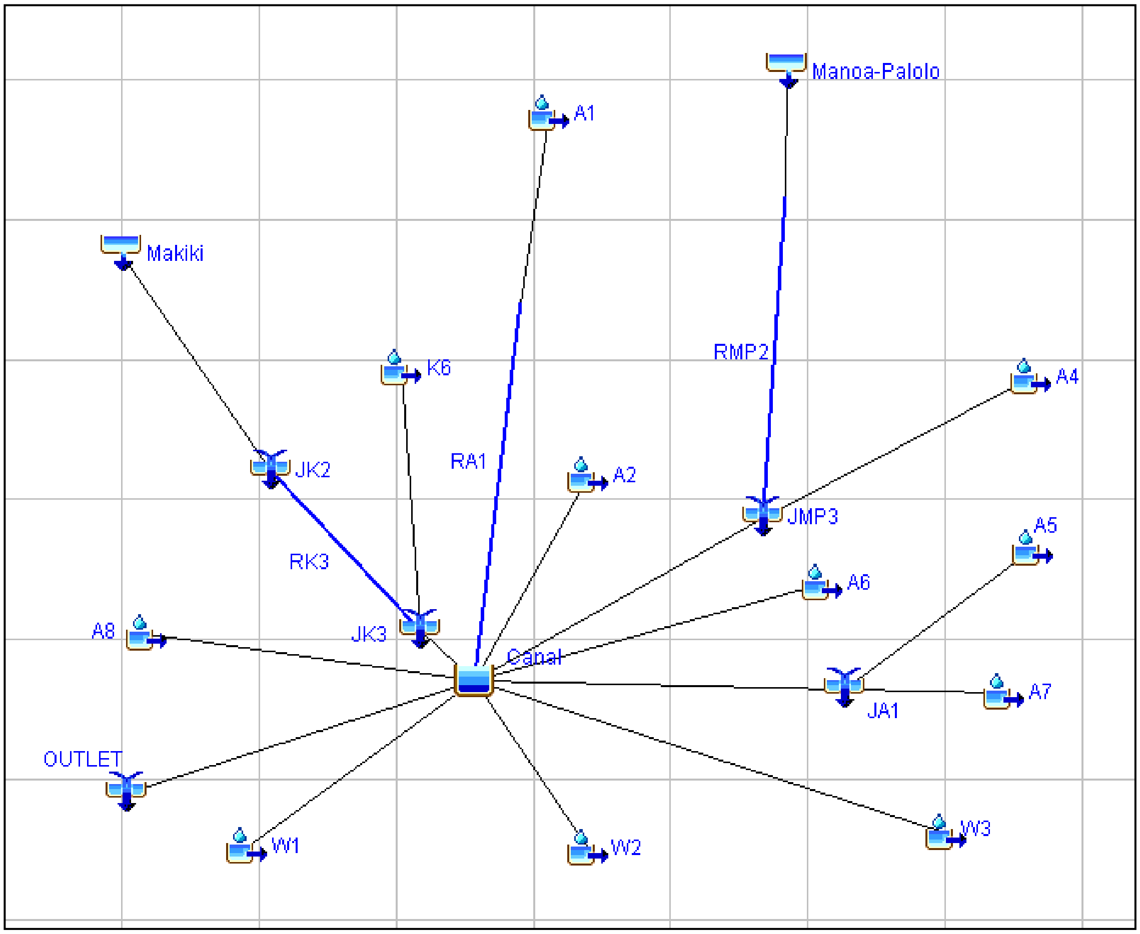

Figure 4-20. Model Layout for 2004

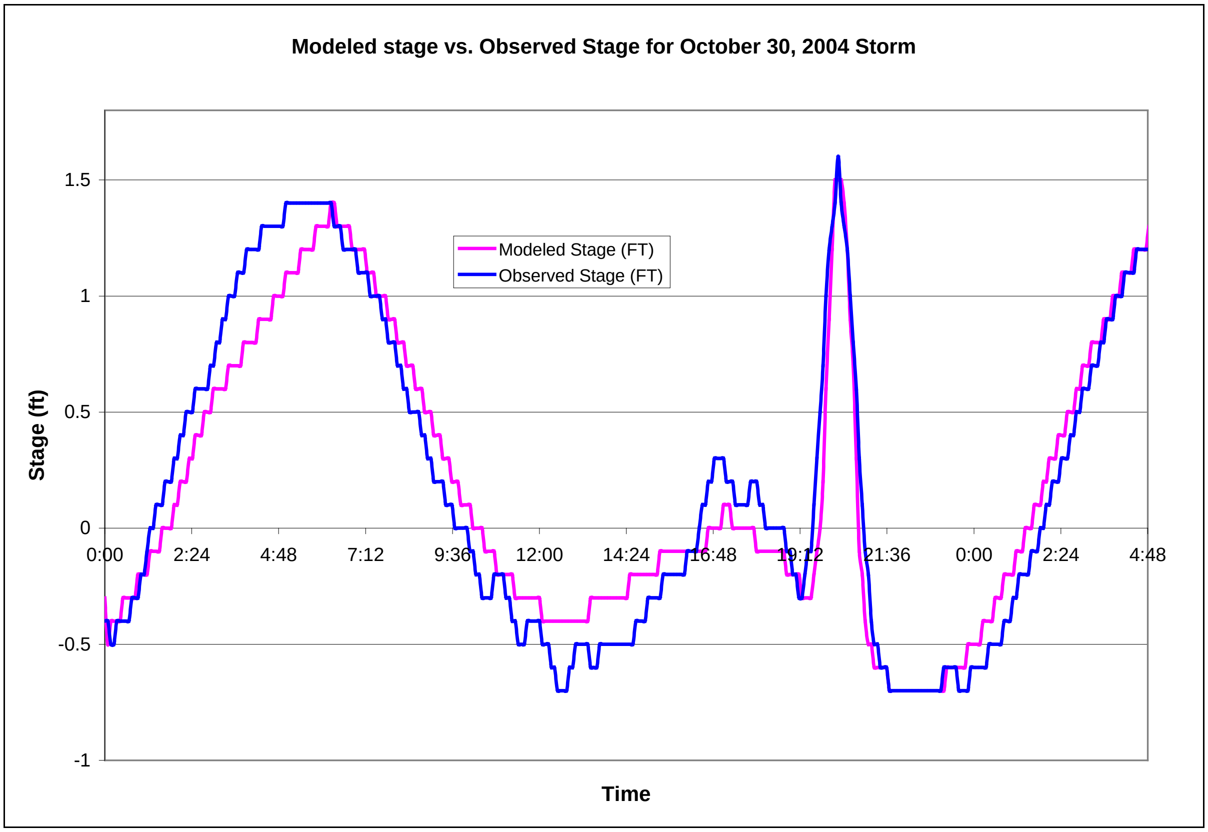

Figure 4-21. Calibrated Water Elevation vs. Observed Stage for Ala Wai Canal on October 30, 2004 Storm

For modeling of the December 17—18, 1967 storm, the calibrated Mānoa-Pālolo model hydrograph at USGS stream gage 16247100 was used to represent the upstream inflow. The finalized Makiki model described in Section 4.5 was used to represent the Makiki sub-watershed. Figure 4-22 shows the HEC-HMS model layout for calibrating this storm. The DLNR post flood report (1968) noted that Ala Wai Canal in Waikiki overflowed at the confluence with Mānoa-Pālolo Drainage Canal. Ala Wai Boulevard and adjacent streets near the confluence were flooded with water up to two feet deep (DLNR, 1968). The modeled peak stage was about 4.4 feet, or about 2.2 feet above Ala Wai Boulevard. Figure 4-23 shows the modeled stage in feet.

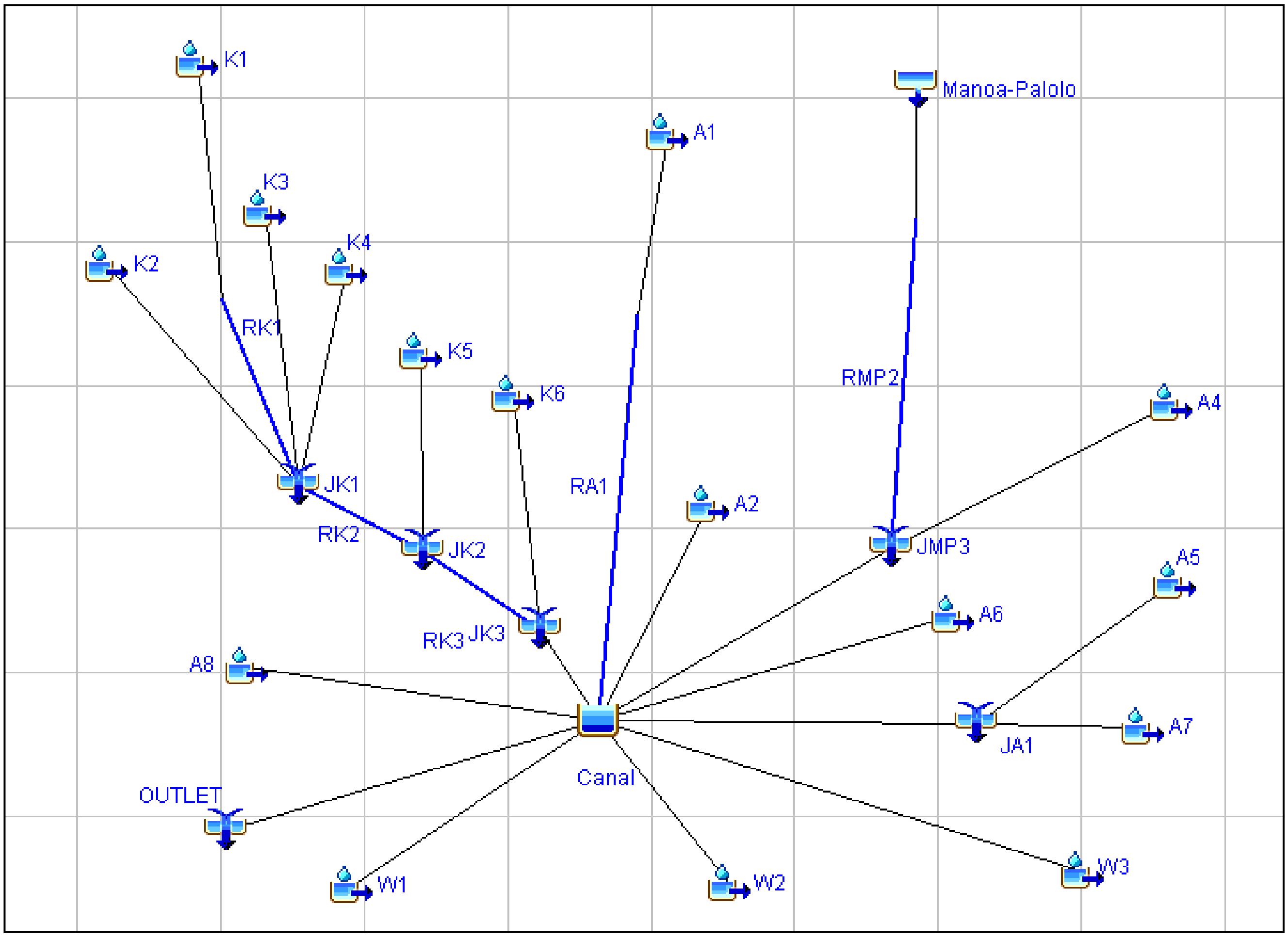

Figure 4-22. Model Layout for 1967

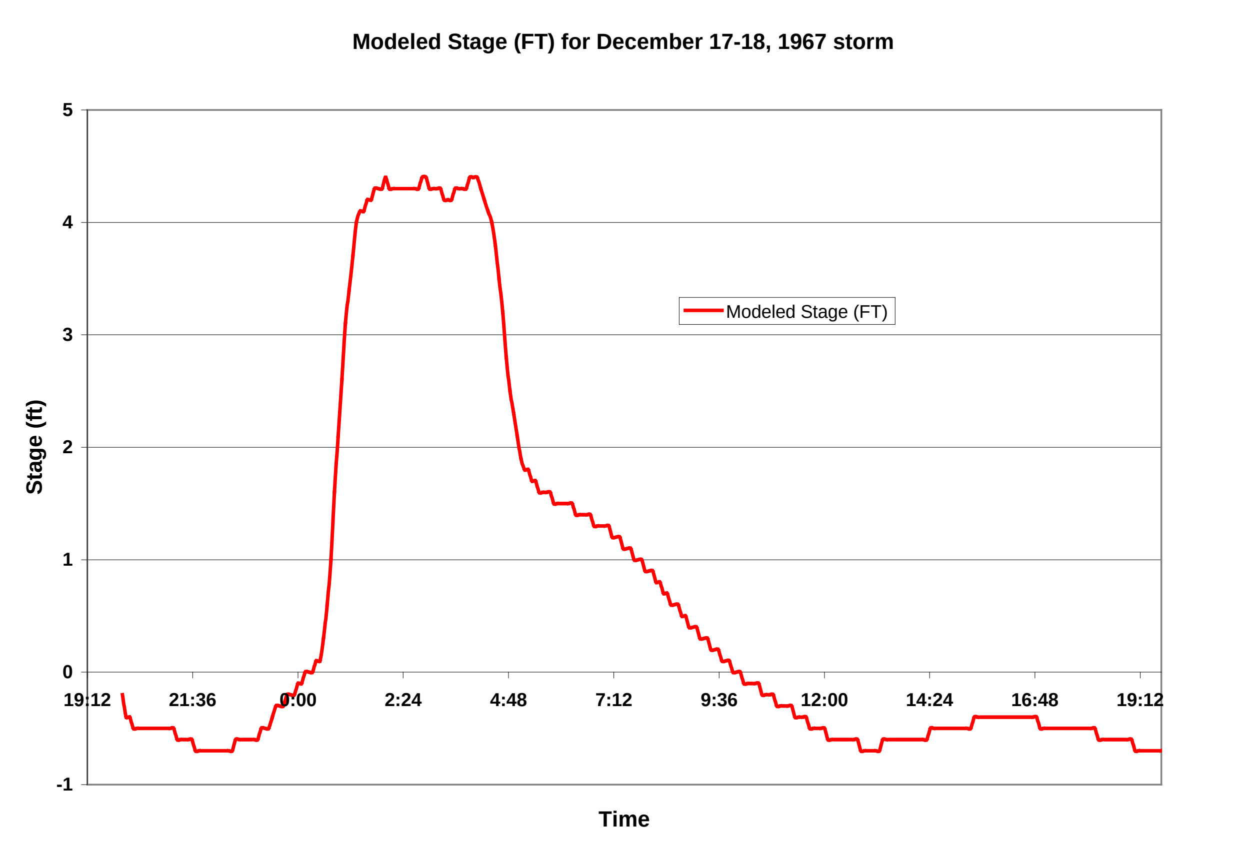

Figure 4-23. Calibrated Water Elevation at Ala Wai Canal for December 17-18, 1967 Storm

4.7.2 Reaches: Muskingum-Cunge and Modified Puls Channel Routing

The Muskingum-Cunge channel routing parameters were used and included the Manning's *n* values, length, slope, and cross-sections. The Manning's *n* values for the stream channel and its banks were determined using Chow's (1959) guidelines and channel conditions. The length of each reach was determined using GIS Arcview 3.3 maps; the slopes were estimated using contours generated from LiDAR data; and the widths were determined from field measurements and the cross-sectional data obtained from GeoRAS. The channel routing parameters are shown in Table 4-19.

Muskingum-Cunge Channel Routing for HEC-HMS Model

Reach	Length (ft)	Slope (ft)	Manning's n	Shape	Width (ft)	Side Slope (xH:V)
RK1	4350	0.0415	0.05	Trapezoid	10	2
RK2	2650	0.0101	0.03	Trapezoid	20	2
RM7	1180	0.008	0.035	Trapezoid	50	2
RMP1	1900	0.0053	0.04	Trapezoid	50	2
RP1	5900	0.056	0.046	Trapezoid	15	2
RP2	3300	0.015	0.04	Trapezoid	15	2
RP3	4350	0.04	0.04	Trapezoid	12	2
RP4	5950	0.0185	0.0162	Rectangle	30	----
RP5	4300	0.0186	0.0162	Rectangle	30	----

Table 4-19. Muskingum-Cunge Channel Routing for HEC-HMS Model

The Modified Puls Routing Method was used for the Ala Wai Canal modeling to take backwater effects into consideration. The Modified Puls Routing Method is also called storage routing or level pool routing and is most often applied to reservoir routing. Because the Ala Wai Canal was modeled as a reservoir, the stream reaches that discharge into the reservoir were modeled using the Modified Puls Routing Method. The storage-discharge functions for reaches RMP2 (Mānoa-Pālolo Canal) and RK3 (Makiki Stream) were defined based on the elevation-discharge measurements of stream gages 16247100 at the Mānoa-Pālolo Canal and 16238000 at King Street bridge. The storage-discharge function for reach RA1 (Alanaio Stream) was defined by using Manning's equation. Figure 4-24 shows the locations of these three reaches. Figures 4-25, 4-26, and 4-27 Show the storage-discharge curves for these three reaches, respectively.

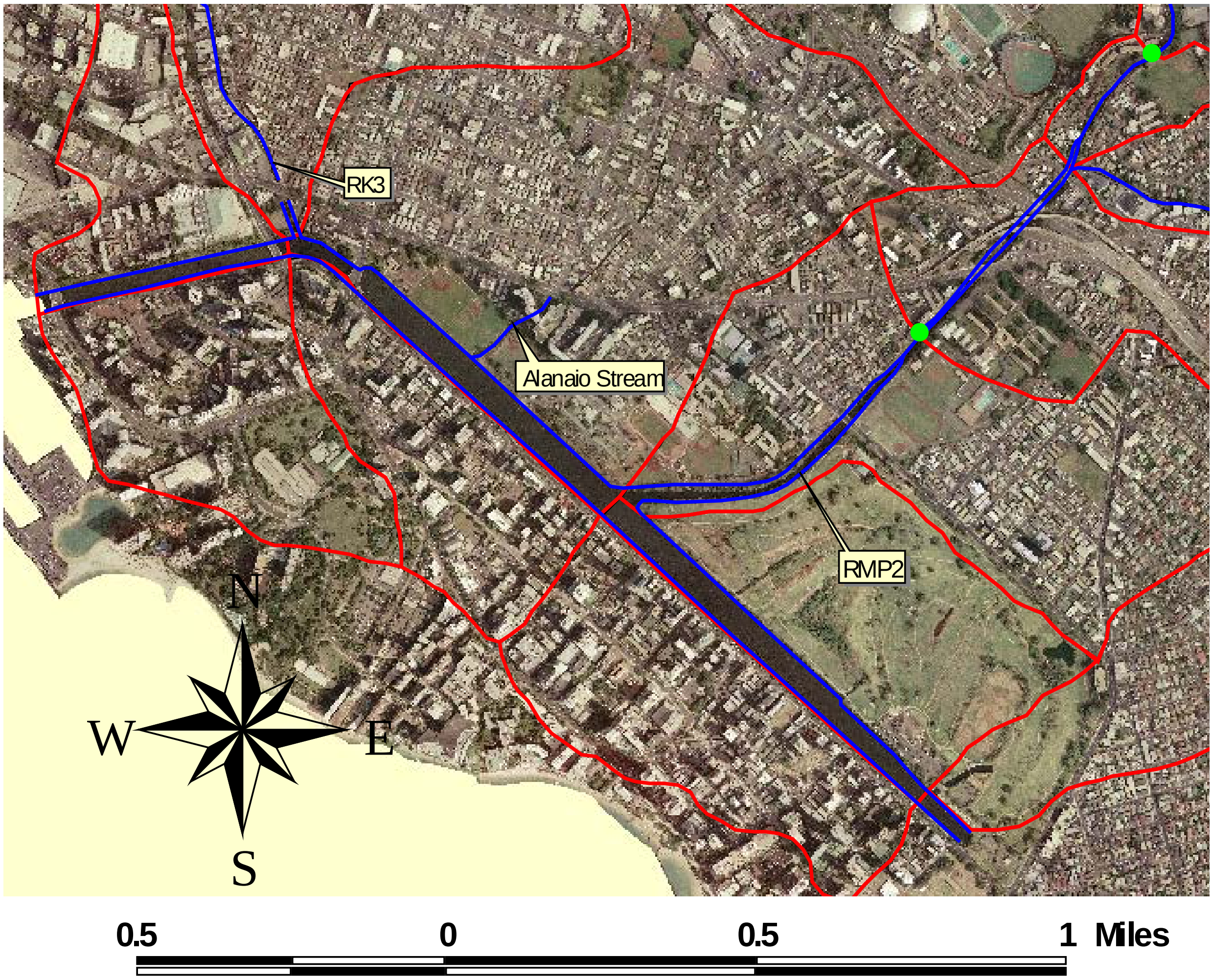

Figure 4-24. Reach locations for Modified Puls Routing Method

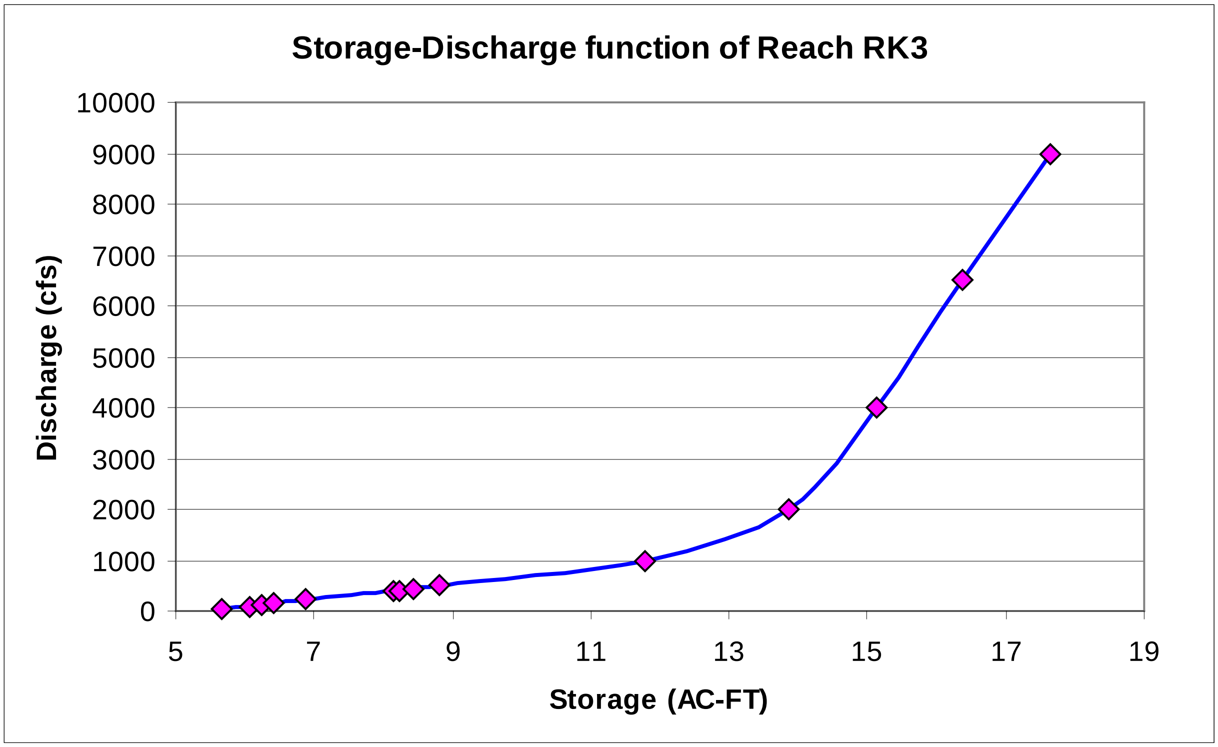

Figure 4-25. Storage-Discharge Curve for Reach RK3

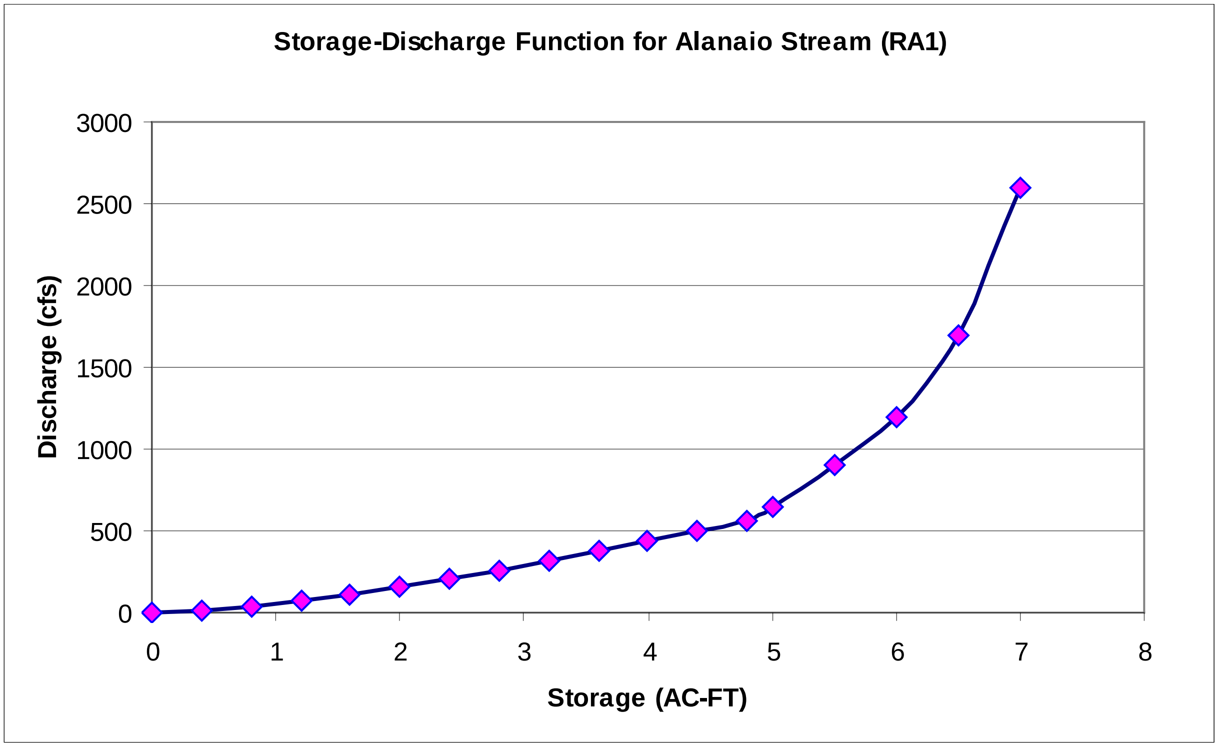

Figure 4-26. Storage-Discharge for Reach RA1

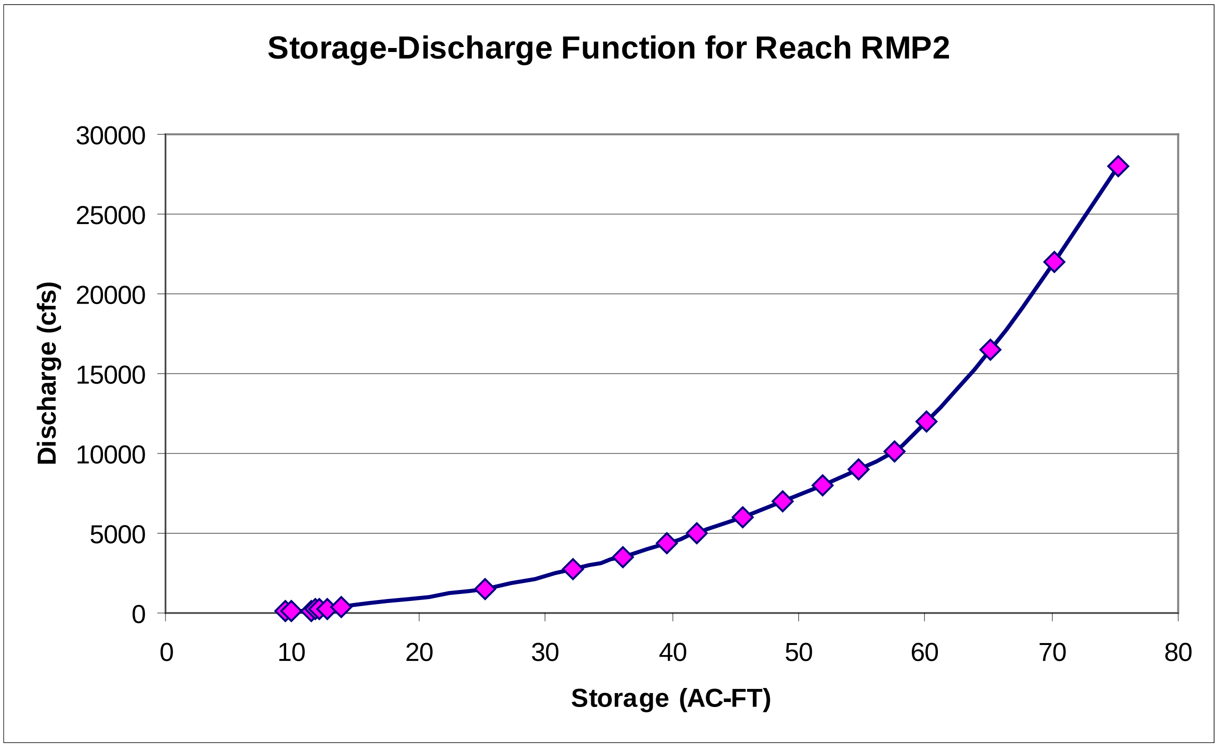

Figure 4-27. Storage-Discharge for Reach RMP2

4.8 Inflow Hydrographs at Kānewai Gage

For consistency with the previous Mānoa Watershed Project hydrologic study, the final results from that study were used to represent the whole Mānoa sub-watershed at the Kānewai Field stream gage. Inflow hydrographs were obtained from the HEC-HMS model of the Mānoa Watershed Project study for the storm chance exceedances of 50 through 0.2 percent. Table 4-20 lists the peak discharges at the Kānewai Field stream gage (USGS 16242500). Figures 4-28 and 4-29 provide the modeled stream flow at Kānewai Field, based on the results from the Mānoa Watershed Project hydrologic study (Oceanit 2008).

Peak Discharges at Kānewai Field Stream Gage from Mānoa Watershed Project								
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceeded	50%	20%	10%	5%	2%	1%	0.5%	0.2%
Peak Discharges (cfs)	2,500	4,300	6,000	7,600	9,500	10,700	12,000	14,000

Table 4-20. Peak discharges at Kānewai Field Stream Gage from Mānoa Watershed Project Hydrologic Study

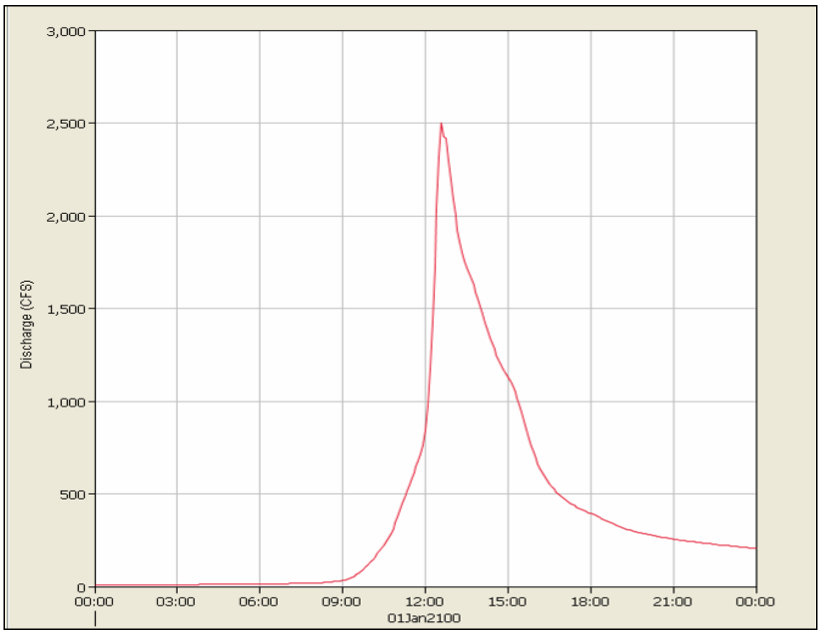

Figure 4-28. Inflow Hydrograph for the 50-percent Chance Flood Used to Represent the Manoa Sub-Watershed in the Ala Wai Watershed HEC-HMS Model (at Kānewai Field)

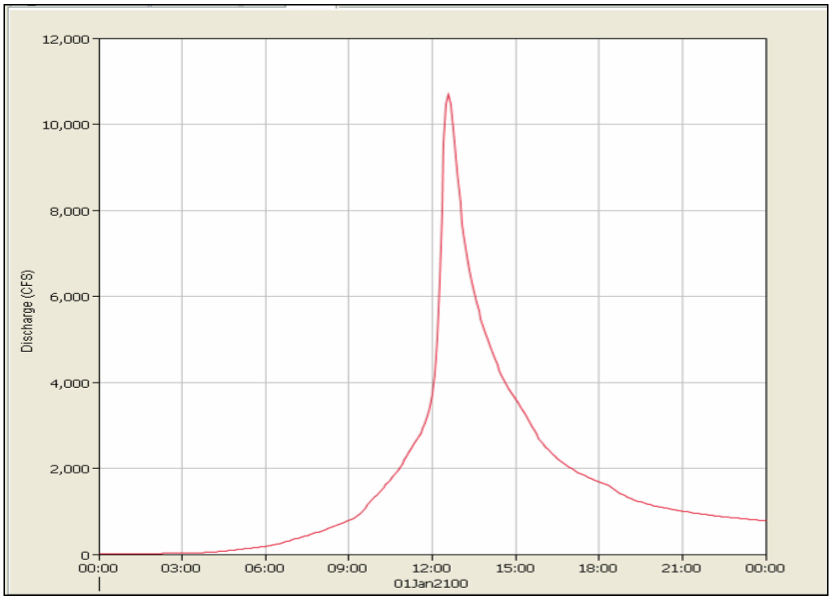

Figure 4-29. Inflow Hydrograph for the 1-percent Chance Flood Used to Represent the Manoa Sub-Watershed in the Ala Wai Watershed HEC-HMS Model (at Kānewai Field)

4.9 Peak Flow Results

For predicting the peak discharges for various return periods, the frequency storm with an intensity position at 50 percent was used in computing the peaks and hydrographs. The HEC-HMS model predicted peak discharges at various junctions in the Ala Wai Watershed are listed in Table 4-21. The final HEC-HMS model layout is shown below in Figure 4-30.

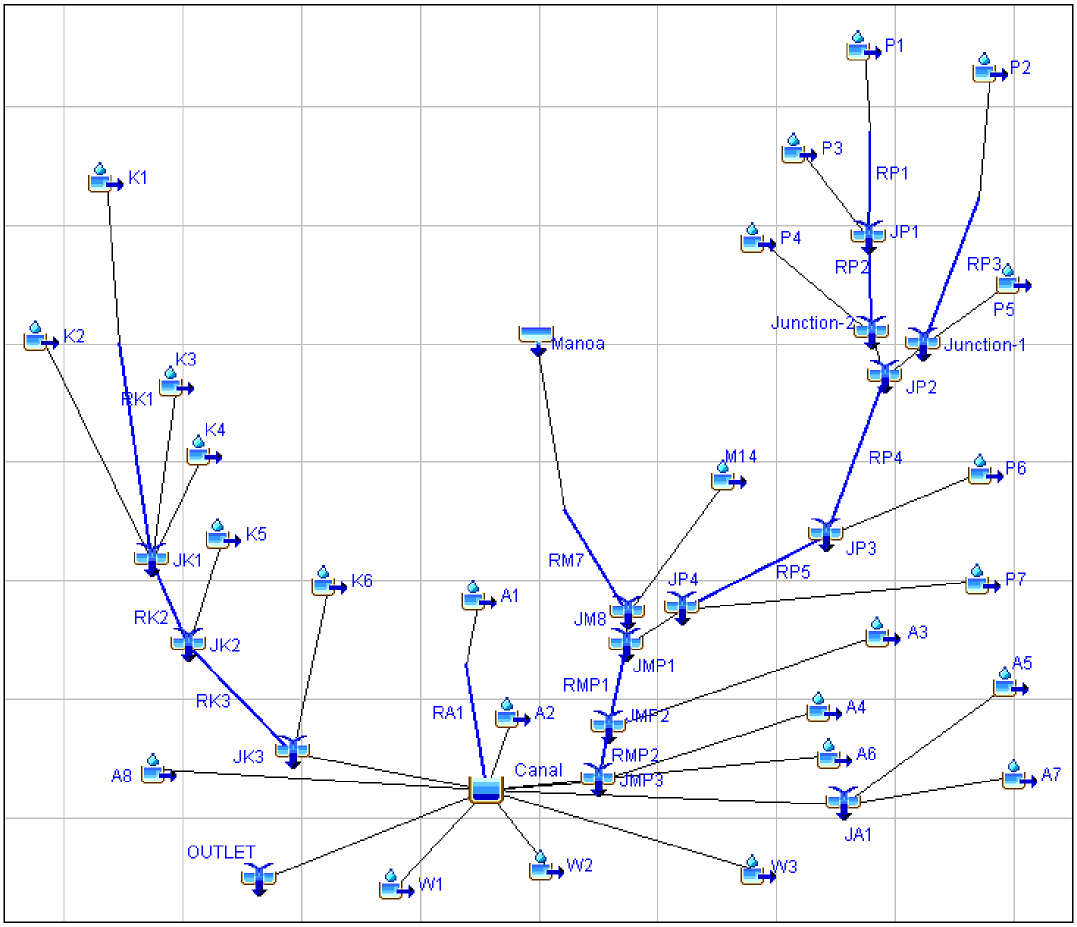

Figure 4-30. Ala Wai Watershed HEC-HMS Model

HEC-HMS Model Results—Peak Flow Discharges at Junctions

Table 4-21 HEC-HMS Model Peak Flow Discharges at Junctions								
	Peak flow discharge (cfs)							
Return Period (yr)	2	5	10	20	50	100	200	500
Percent Chance Exceedance	50%	20%	10%	5%	2%	1%	0.5%	0.2%
JK1	570	1,200	1,890	2,400	3,150	3,740	4,380	5,240
JK2	660	1,360	2,110	2,650	3,440	4,060	4,730	5,630
JK3	890	1,770	2,690	3,340	4,280	5,000	5,790	6,850
JM8	2,560	4,450	6,210	7,860	9,810	11,100	12,400	14,500
JP1	320	730	1,150	1,460	1,900	2,220	2,590	3,110
JP2	940	2,030	3,190	4,010	5,180	6,040	6,980	8,320
JP3	1,330	2,710	4,170	5,180	6,620	7,670	8,850	10,500
JP4	1,550	3,120	4,720	5,810	7,400	8,550	9,860	11,600
JMP1	4,020	7,170	10,300	12,900	16,100	18,500	20,900	24,400
JMP2	4,090	7,340	10,500	13,000	16,300	18,700	21,100	24,700
JMP3	4,220	7,450	10,700	13,300	16,600	18,900	21,400	24,900
Ala Wai Canal	6,000	10,100	13,400	15,200	16,700	17,700	18,700	20,500

Table 4-21. HEC-HMS Model Predicted Peak Discharges at Junctions

4.10 USGS Regression Equations and City and County’s Plate 6

Regional regression equations developed by the USGS (Wong, 1994) for estimating peak discharges for the 50-, 20- 10-, 4-, 2-, and 1-percent chance exceedance probabilities at gaged and ungaged sites were used to calculate peak flows in the sub-watersheds. The equations for Leeward O‘ahu were used for the sub-watersheds in this study. The drainage area (DA) and median annual rainfall (P) in these equations are independent parameters. These regression equations are valid for ungaged sites when (1) the drainage areas are between 0.03 and 45.7 square miles; (2) where less than 36 percent of the area is urbanized; and (3) the median rainfall is between 29 and 239 inches. The median annual rainfall for each sub-watershed was determined from DLNR (1982). The median annual rainfall amounts for the junctions were calculated by the weighting mean method with respect to the sub-watershed areas. The equations used bias-correction factors along with the accuracy of the estimates in equivalent years of record (Wong, 1994). The peak discharges calculated using these regression equations and Plate 6 of the City’s drainage standards (2000) for each junction are presented in Table 4-22. The accuracy of these results is 16 years for the 1 percent chance exceedance event and 15 years for the other storm events.

The City storm drainage standards (2000) specify the use of the rational method for drainage areas of 100 acres or less and Plate 6 for drainage areas greater than 100 acres, and this method was used for some of the sub-basins in the Ala Wai Watershed study area. Plate 6 is an envelope curve developed from maximum known peaks and regression analysis of 100-year peak flows. This curve is assumed to represent a 100-year peak flow but actually has a slightly higher return period (Wong 1994). The accuracy of this curve is based not on the average years of recorded data but by the standard error of regression. The accuracy of data used for peak determination of the 100-year envelope is unknown. In the absence of accurate data, an equivalent years of record of 10 years is assigned (Interagency Advisory Committee on Water Data 1982).

Plate 6 was applied to calculate the 100-year peak discharges in all sub-basins because the corresponding drainage areas exceed 100 acres. Plate 6 provides three curves relating to the peak discharge of the 100-year return period storm (1 percent chance exceedance probability). Curve B from Plate 6 was used for the Mānoa sub-watershed.

USGS Regression Equations and Plate 6 Calculation in cubic feet per second (cfs) for junctions, Ala Wai Watershed, Oahu, Hawaii.

Percent Chance Flood	Equation with BCF	JK1	JK2	JM8	JP1	JP2	JP3	JP4	JMP1	JMP2	Accuracy in Years of Record
50	Q_2=3.635 $(DA)^{0.634}$ $P^{1.08}$	660	670	1,660	650	1,035	1,040	1,040	2,120	2,110	**4.2**
20	Q_5=27.58 $(DA)^{0.642}$ $P^{0.773}$	1,340	1,370	3,100	1,160	1,930	2,020	2,060	4,060	4,080	**5.8**
10	Q_{10}=77.32 $(DA)^{0.646}$ $P^{0.621}$	1,960	2,000	4,330	1,580	2,700	2,870	2,970	5,760	5,800	**8.2**
4	Q_{25}=225.7 $(DA)^{0.646}$ $P^{0.464}$	2,900	2,980	6,120	2,200	3,830	4,150	4,330	8,240	8,320	**11.4**
2	Q_{50}=440.7 $(DA)^{0.645}$ $P^{0.368}$	3,840	3,960	7,870	2,810	4,940	5,410	5,690	10,680	10,810	**13.7**
1	Q_{100}=788.3 $(DA)^{0.643}$ $P^{0.286}$	4,680	4,840	9,330	3,320	5,880	6,500	6,860	12,740	12,910	**15.8**
	Plate 6 (100-yr)	5,300	5,600	11,000	3,200	6,500	7,700	8,100	15,500	16,000	**10**

Table 4-22. USGS Regression Equations and Plate 6 Calculation

4.11 FEMA Flood Insurance Study

The hydrologic and hydraulic analysis for the original Flood Insurance Study (FIS) for the City and County of Honolulu was performed by R.M. Towill Corporation in 1976. FEMA revised the previous FIS for the City and published the most updated FIS in 1979.

For Makiki Stream, USGS regression equations were used to obtain peak flow discharges for the 10-, 50-, and 100-year flooding events (FEMA, 2004). The 500-year flood was determined by a regression equation utilizing the same basic data and regression techniques as applied by USGS. These regression equations applied the ratio of the drainage area covered by forests and vegetation to total drainage area in percent instead of the median rainfall that current USGS regression equations applied to determine the peaks. Figure 18 in FIS (FEMA, 2004) was the results that only applied to one place with the drainage area as about 2.49 square miles. This drainage area is equal to the drainage area of junction JK2; in other words, only junction JK2 is available to have FEMA flood insurance analysis peak flow discharges.

For Palolo Stream, peak discharges were based on a statistical analysis results by using the 25-year recording annual peaks at USGS Gaging Station 16247000. The analysis followed the standard log-Pearson type III method procedures as outlined by the Water Resources Council. So the FEMA FIS analysis for Palolo Stream is only applied to junction JP3 that USGS gage 16247000 located.

For Manoa-Palolo and Ala Wai Canals, the peak discharges were determined by using SCS hydrograph method. Probably because of the higher proportion of urbanized areas, the SCS method resulted in slightly higher peak discharges.

For JM8, which is part of Manoa sub-watershed, same analysis was used as previous Manoa watershed study conducted by Oceanit (2008b).

Table 4-23 shows the FEMA flood insurance study analysis for Makiki, Palolo, Manoa-Palolo Canal and Ala Wai Canal.

Peak Flow Discharges in cfs Calculated by FEMA				
Return Period (yr)	**10**	**50**	**100**	**500**
Percent Chance Exceedance	**10%**	**2%**	**1%**	**0.2%**
JK2	1,850	3,250	3,950	5,950
JM8	7,600	11,500	13,600	17,000
JP3	2,790	4,510	5,340	7,530
JMP2	12,000	19,200	23,000	28,500
Ala Wai Canal	13,700	23,000	28,200	36,200

Table 4-23. Peak Flow Discharges Calculated by FEMA

4.12 Flow Frequency Analysis

HEC-SSP version 1.0 Beta was used to perform the flow frequency analysis. This software is limited to performing flood flow frequency analysis based on Bulletin 17B, "Guidelines for Determining Flood Flow Frequency" (Interagency Advisory Committee on Water Data 1982). Three USGS stream gages that have sufficient data to perform the flow frequency analysis are within the study area. The USGS stream gage 16247100 at the Mānoa-Pālolo Canal (Junction JMP2), adjacent to the Kaimukī High School, has a drainage area of 10.34 square miles. Thirty-eight effective annual peaks were used in the HEC-SSP model to predict the peaks for the various return periods at this junction. The USGS Pālolo Stream gage 16247000 (Junction JP3) has a drainage area of 3.62 square miles. Thirty-two effective annual peaks were used at this junction. The USGS Pūkele Stream (tributary of Pālolo Stream) gage 16244000 (Junction JP1) has a drainage area of 1.15 square miles. Fifty-nine effective annual peaks were used at this junction. The following figures and tables show the flow frequency results from HEC-SSP model (Figures 4-31-33 and Tables 4-24 through 4-26).

At USGS Gaging Station 16247000, there are 32 effective annual peaks available to perform the statistical frequency analysis. The continuous recorded annual peaks are from 1953 to 1979 and from 2003 to 2007, but no data is available between 1980 and 2002. The recorded annual peaks from 2003 to 2007 seem incorrect for the following two reasons.

(1) On October 30, 2004, the recorded peak at this gage was 776 cfs. The tributary stream gage upstream (Pukele) recorded a 753 cfs peak, and another tributary (Waiomao Stream) received the same rain as Pukele Stream received. At USGS gage 16247100 downstream, the recorded peak was 9,380 cfs and the Manoa Stream at Kanewai gage recorded a peak at 5,860 cfs. Thus, the peak flow at the Palolo gage should be in a range of 1,500 to 3,000 cfs rather than the 776 recorded because it received similar rainfall as Manoa.

(2) The peak for March 31, 2006 storm at Palolo Stream Gage was 1,390 cfs, at downstream gage USGS 16247100, the recorded peak was 9,320 cfs, the rainfall was uniformly distributed into the study area, the Palolo valley should have generated a range 2,000 to 3,000 cfs peak flow. Since there was possible channel conditions changed during the last 50 years, the data in this gage may be lower than actual stream flows, as a result, the HEC-SSP and FEMA analysis (used 25-year annual peaks) got lower peak discharges.

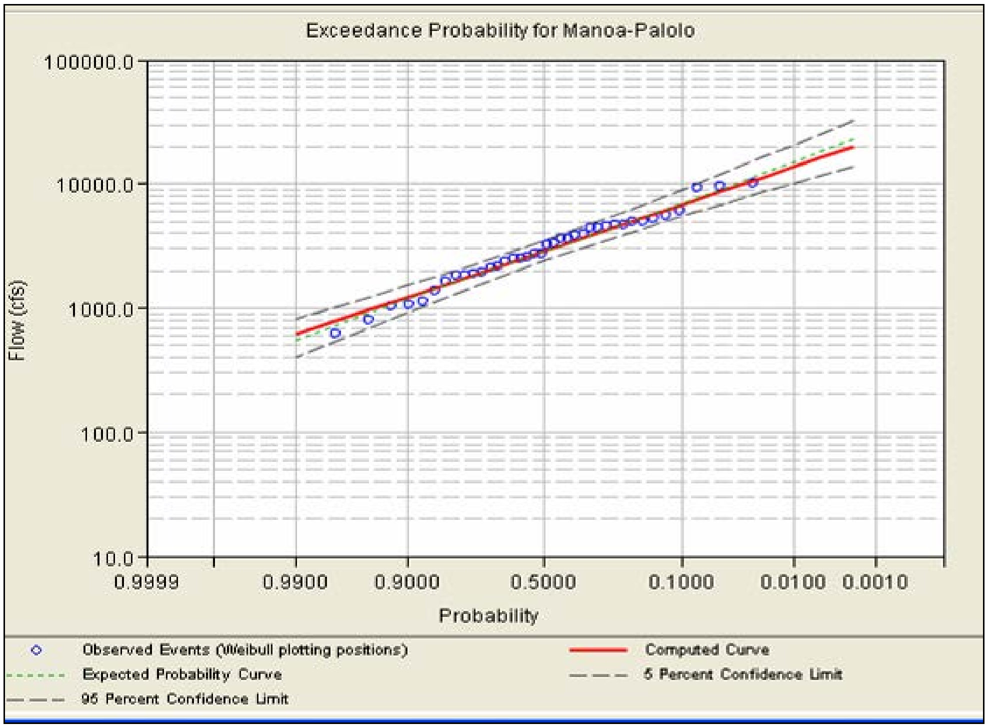

Figure 4-31. Exceedance Probability for Mānoa-Pālolo Canal Stream Gage JMP2 (USGS Stream Gage[16247100])

Percent Chance Exceedance	Return Period (year)	Computed Flow (cfs)	Confidence Limits Flow (cfs)	
			0.05	0.95
0.2	500	19,800	32,538	13,949
0.5	200	16,200	25,443	11,719
1	100	13,700	20,783	10,143
2	50	11,400	16,677	8,654
5	20	8,670	12,017	6,804
10	10	6,800	9,013	5,475
20	5	5,070	6,407	4,179
50	2	2,880	3,459	2,404

Table 4-24. Flood Flow Frequency Results for Mānoa-Pālolo Canal Stream Gage JMP2 (USGS Stream Gage [16247100])

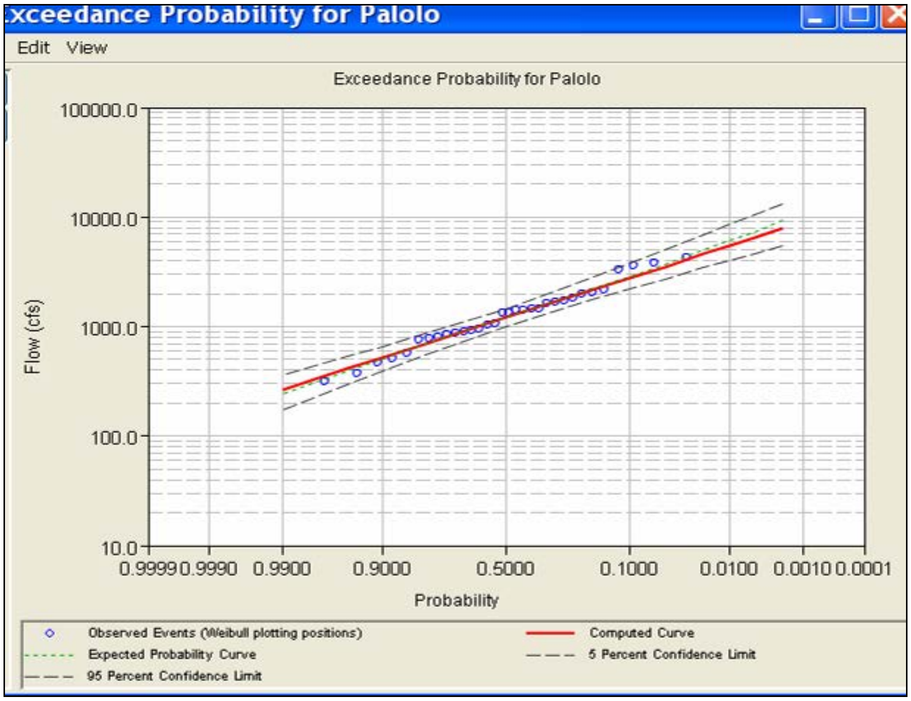

Figure 4-32. Exceedance Probability for Pālolo Stream Gage JP3 (USGS Pālolo Gage [16247000])

Percent Chance Exceedance	Return Period (year)	Computed Flow (cfs)	Confidence Limits Flow (cfs)	
			0.05	0.95
0.2	500	7,820	13,366	5,422
0.5	200	6,430	10,478	4,589
1	100	5,470	8,578	3,996
2	50	4,580	6,900	3,433
5	20	3,510	4,991	2,725
10	10	2,780	3,757	2,212
20	5	2,090	2,683	1,705
50	2	1,210	1,466	997

Table 4-25. Flood Flow Frequency Results for Pālolo Stream Gage JP3 (USGS Pālolo Gage [16247000])

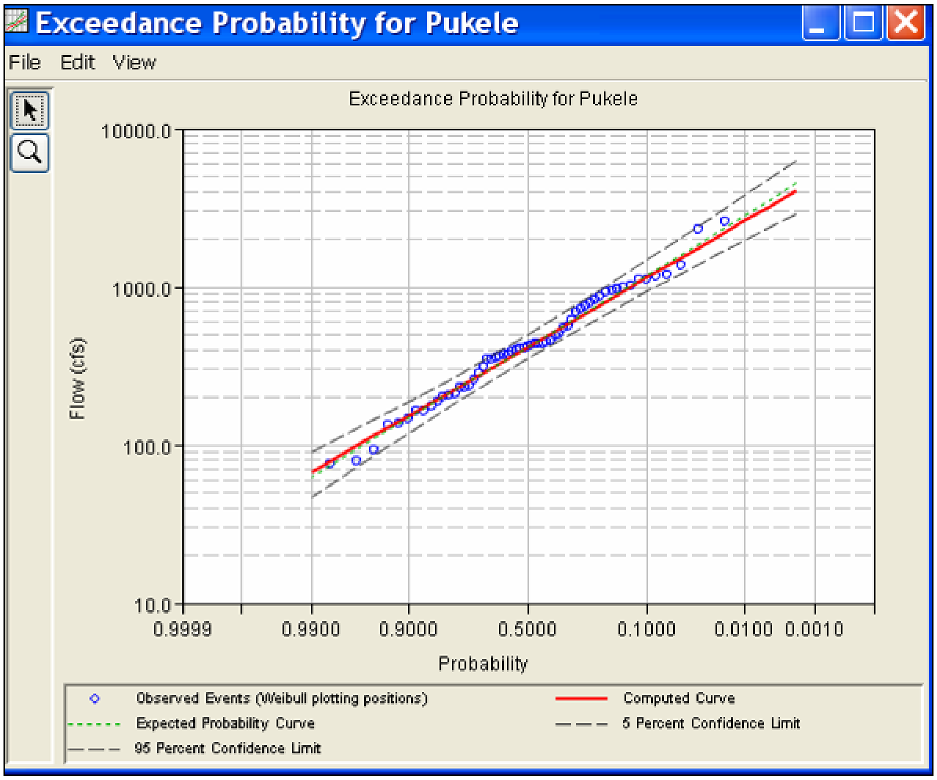

Figure 4-33. Exceedance Probability for Pūkele Stream Gage JP1 (USGS Pūkele Gage [16244000])

Percent Chance Exceedance	Return Period (year)	Computed Flow (cfs)	Confidence Limits Flow (cfs)	
			0.05	0.95
0.2	500	4,050	6,330	2,880
0.5	200	3,190	4,800	2,330
1	100	2,620	3,820	1,960
2	50	2,110	2,980	1,620
5	20	1,530	2,060	1,210
10	10	1,150	1,490	930
20	5	810	1,010	680
50	2	420	500	350

Table 4-26. Flood Flow Frequency Results for Pūkele Stream JP1 (USGS Pūkele Gage [16244000])

5 Results of Hydrologic Model

All of the hydrologic analysis methodologies estimate peak flow discharges (cfs) for return periods (percent chance exceedance storms) by junction; the methodologies include the HEC-HMS modeling, the USGS regression method, City Plate 6, FEMA Flood Insurance Study, and the HEC-SSP model. Each of these methodologies provides a predictive measure for peak discharges, and used together they offer a clear and accurate depiction of where peak flows will occur during 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent chance exceedance storms.

5.1 Determination of Final Peak Flow Discharges

The USACE Engineer Manual (EM) 1110 – 2 -1619 (1996, Table 4-5, page 4-5) provides guidelines to assign accuracies to flood frequency estimates determined by various methods in term of equivalent years of record. Estimates assigned higher equivalent years of record are considered more reliable than those with lower equivalent years of record. Equivalent years of record is an assigned unit of accuracy. In comparing methodologies, those with higher equivalent years of record were considered more accurate and given greater weight. Based on engineering judgment following the guidelines, the HEC-SSP model is the most reliable with equivalent years of record 59, 32, and 38 for junctions JP1, JP3, and JMP2, respectively. The HEC-HMS model was calibrated to three historical storms for Manoa and Palolo sub-watersheds, two historical storms for Ala Wai Canal reservoir model, and one historical storm event for Makiki sub-watershed. Although there was no calibration to the urbanized sub-basins, the parameters physical measurable Kinematic Wave transform method was applied. An equivalent record length of 20 years was assigned to the results generated by HEC-HMS model based on guidelines provided in EM 110-2-1619 (USACE, 1996) and the confidence in the calibration data sets.

FEMA flood insurance study within Ala Wai watershed area applied various methods to determine the peak discharges, based on the analysis done with equivalent record lengths of 15 years and were assigned to FEMA results in junctions JK2, JM8, JMP2, and Ala Wai Canal. An equivalent record length of 25 years was assigned to FEMA results in junction JP3 in response to its statistic analysis using 25-year recorded annual peaks. The weighting factors for the HEC-HMS modeling, the USGS regression, City Plate 6, FEMA Flood Insurance Study, and the HEC-SSP methodologies are shown in Table 5-1.

Weighting Factors for Peak Discharges Development								
Methodology	Accuracy in Equivalent Years of Record							
Percent Chance Exceedance	50%	20%	10%	5%	2%	1%	0.5%	0.2%
HEC-HMS	20	20	20	20	20	20	20	20
Regression	15	15	15	15	15	16		
Plate 6						10		
FEMA			15 25(JP3)		15 25(JP3)	15 25(JP3)		15 25(JP3)
HEC-SSP	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)

Table 5-1. Weighting Factors Used To Develop Final Peak Flow Values

Determination of the final peak flow discharges at junctions of interest for the sub-watersheds studied was conducted in three steps: (1) the peak flow discharge values produced by each method were weighted; (2) all the available peak flow discharge values were plotted on log probabilistic graph paper by percent chance exceedence; and (3) the best fit curve of the peak flow discharges was graphed assuming watershed linearity, that is, that the peak flow discharge-frequency curves should be defined by a single function (illustrated as a smooth curve) for each sub-watershed. Engineering judgment was used in the selection of the best fit curve to account for the relative accuracies of each method and for the up and downstream relationships to determine the resulting final peak flow discharge values.

The determination of final peak flow discharges assumes that the sub-watersheds examined in this study exhibit linearity, meaning that a single function may describe the runoff from a sub-watershed. Sub-watershed linearity is based on the concept that peak flow discharge frequency curves serve their descriptive purpose as continuous, smooth curves. Thus, even after peak flow discharges were weighted and plotted on log-probabilistic graph paper, the best curve fit for these discharge values was determined using engineering judgment. The best fit curve was the final step in determining peak flow discharge values at the junctions of interest.

5.2 Makiki Peak Flow Discharges

Peak flow discharges at junctions of interest in the Makiki sub-watershed were weighted according to the process detailed in Section 5.1, plotted on log-probabilistic graph paper, and a best fit curve was analyzed. Table 5-2 provides peak flow discharge results for the Makiki sub-watershed at junctions of interest by methodology, the weighted values, and the 'FINAL' best fit values.

Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
JK1 (Confluence of Makiki and Kanaha Streams, A=2.328 mi^2)								
HEC-HMS	570	1,200	1,890	2,400	3,150	3,740	4,380	5,240
Regression	660	1,350	1,960	2,900	3,840	4,680		
Plate 6						5,300		
Weighted	610	1,260	1,920	2,620	3,450	4,410	4,380	5,240
FINAL	**650**	**1,300**	**1,900**	**2,550**	**3,400**	**4,100**	**4,800**	**5,700**
JK2 (USGS Stream Gage at King St. 16238000, A= 2.49 mi^2)								
HEC-HMS	660	1,360	2,110	2,650	3,440	4,060	4,730	5,630
Regression	670	1,370	2,000	2,980	3,960	4,850		
Plate 6						5,600		
FEMA			1,850		3,250	3,950		5,950
Weighted	660	1,360	2,000	2,790	3,540	4,490	4,730	5,770
FINAL	**660**	**1,330**	**1,960**	**2,580**	**3,500**	**4,250**	**4,950**	**5,900**
JK3 (Confluence of Makiki Stream and Ala Wai Canal, A=2.892 mi^2)								
HEC-HMS	890	1,770	2,690	3,340	4,280	5,000	5,790	6,850
Plate 6						6,100		
Weighted	890	1,770	2,690	3,340	4,280	5,370	5,790	6,850
FINAL	**760**	**1,600**	**2,400**	**3,200**	**4,300**	**5,250**	**6,100**	**7,200**

Table 5-2. Peak Flow Discharges at Makiki Junctions by Methodology

The junction near the confluence of the Makiki Stream and Ala Wai Canal (JK3) received the highest amount of peak flow discharge in the Makiki sub-watershed. This was expected, because JK3 represents the flow exiting the entire Makiki sub-watershed. The peak discharge values attained by the Plate 6 and Regression methods appear higher than the peak discharge values attained through HEC-HMS modeling, as seen in Figures 5-1 through 5-3. As mentioned in Section 5.1, the peak discharge values were not only weighted, but also the final values were determined by the best fit curve shown in Figures 5-1 through 5-3. This best fit curve takes into account all of the methods used. In short, the final best fit curve was used to calculate the final peak discharges. Figures 5-1 through 5-3 graph the peak flow discharge by methodology over the percent chance exceedance for Makiki junctions of interest.

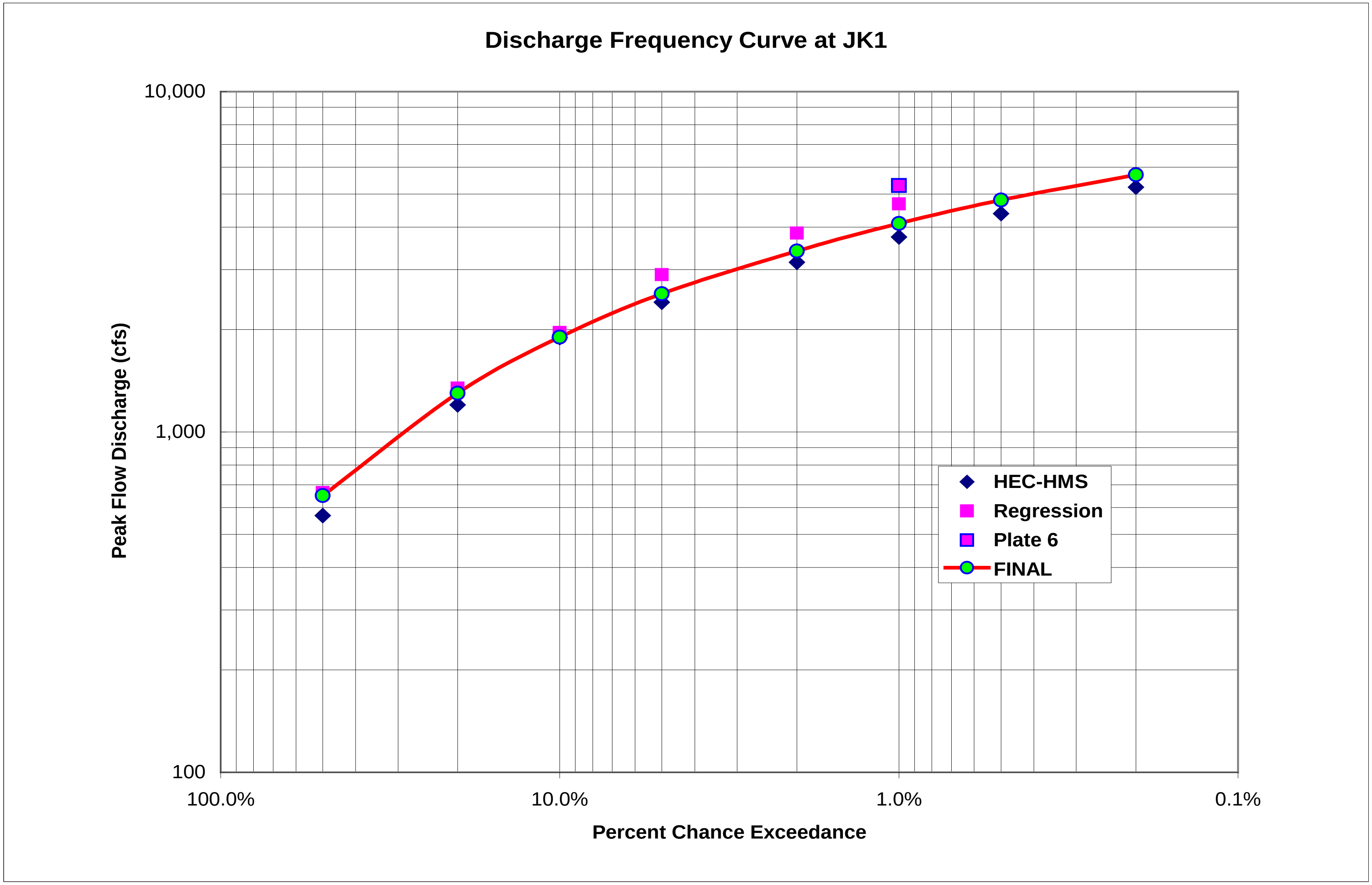

Figure 5-1. Final Discharge Frequency Curve at JK1 (Confluence of Makiki and Kanahā Streams)

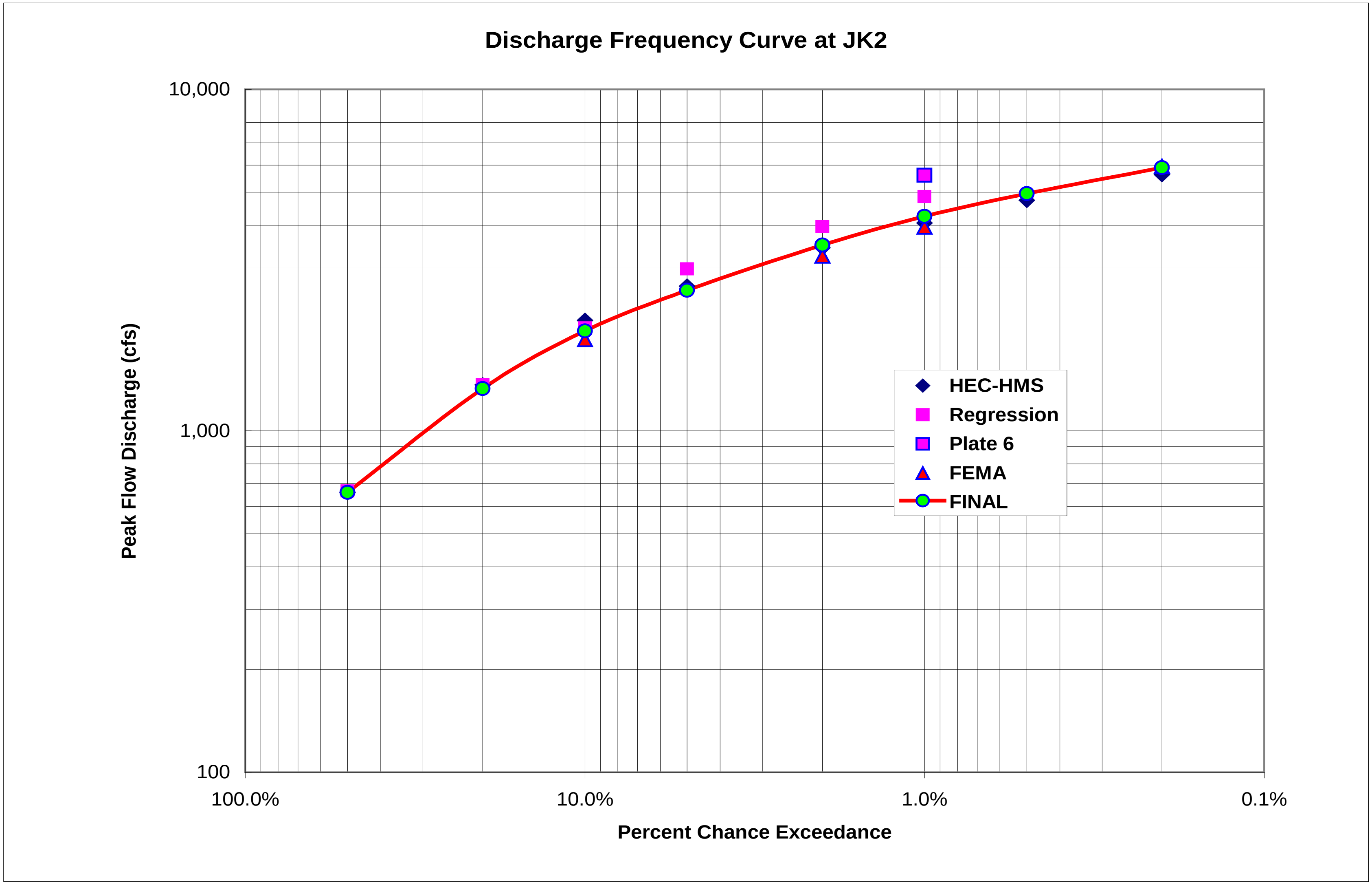

Figure 5-2. Final Discharge Frequency Curve at JK2 (USGS Stream Gage [16238000])

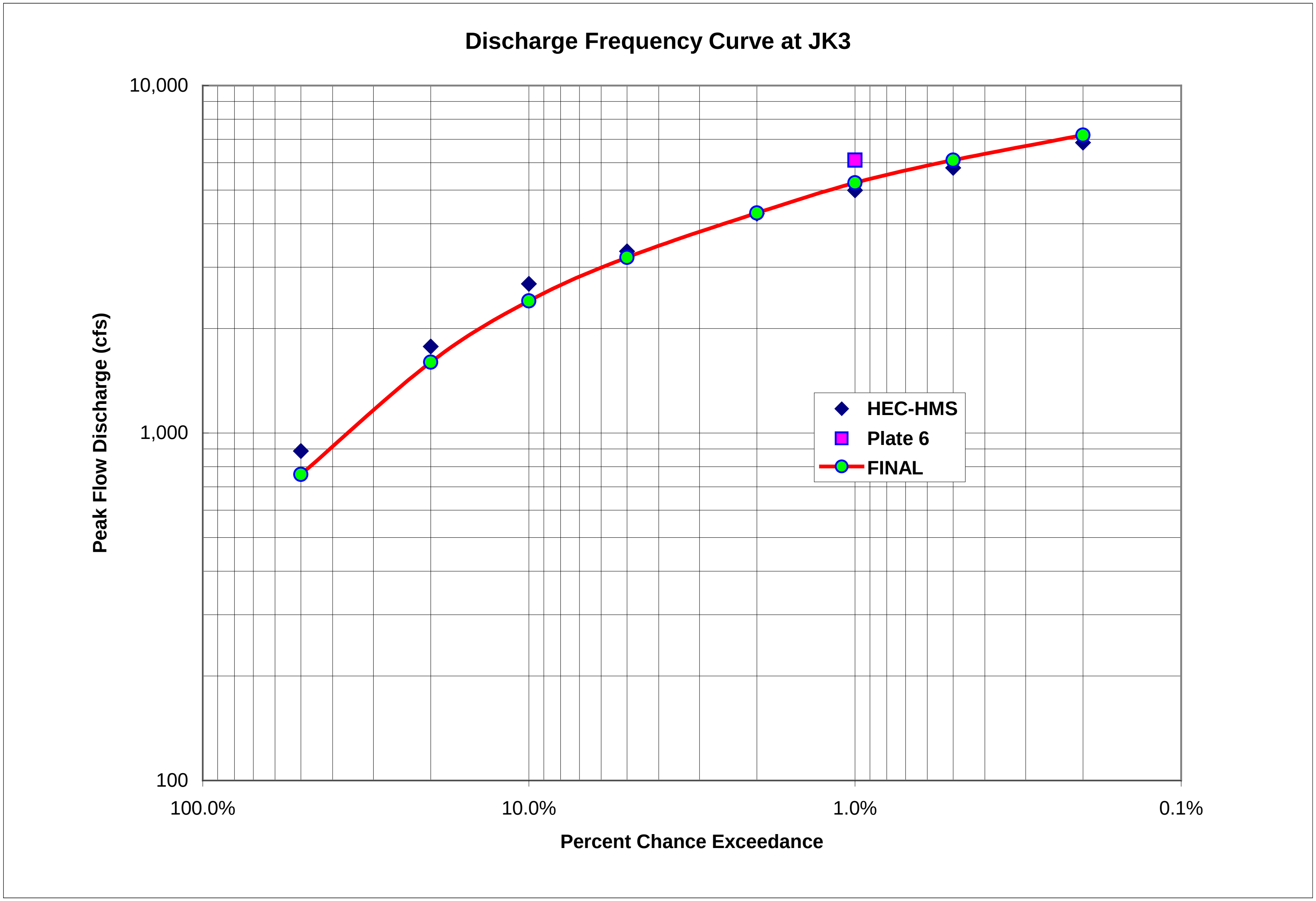

Figure 5-3. Final Discharge Frequency Curve at JK3 (Confluence of Makiki Stream and Ala Wai Canal)

5.3 Mānoa Peak Flow Discharges

Peak flow discharges for the Mānoa sub-watershed were determined in a previous study, and these values were used for the current study. The HEC-HMS peak flow discharges calculated in the Mānoa Watershed Project hydrology report (Oceanit 2008b) at the junction just upstream of the confluence of the Mānoa and Pālolo Streams (JM8) were used. This junction, JM8, is where flow exits the Mānoa sub-watershed, and thus this peak discharge value accounts for all the runoff exiting the Mānoa sub-watershed. Table 5-3 provides the peak flow discharge results by methodology and the 'FINAL' values. The final peak flow discharges from this study are plotted in Figure 5-4.

Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
JM8 (Right above Confluence of Manoa and Palolo Streams, A=5.972 mi^2)								
HEC-HMS	2,560	4,450	6,210	7,860	9,810	11,100	12,400	14,500
Regression	1,660	3,100	4,330	6,120	7,870	9,330		
Plate 6						11,000		
FEMA			7,600		11,500	13,600		17,000
Weighted	2,180	3,870	6,060	7,110	9,730	11,200	12,400	15,600
FINAL	**2,600**	**4,450**	**6,150**	**7,800**	**9,700**	**11,000**	**12,400**	**14,400**

Table 5-3. Peak Flow Discharges at Mānoa Junctions by Methodology

The junction that is just upstream of the confluence of the Mānoa and Pālolo streams (JM8) receives the highest amount of peak flow discharge in the Mānoa sub-watershed. Figure 5-4 illustrates the peak flow discharge results at the Mānoa junctions of interest.

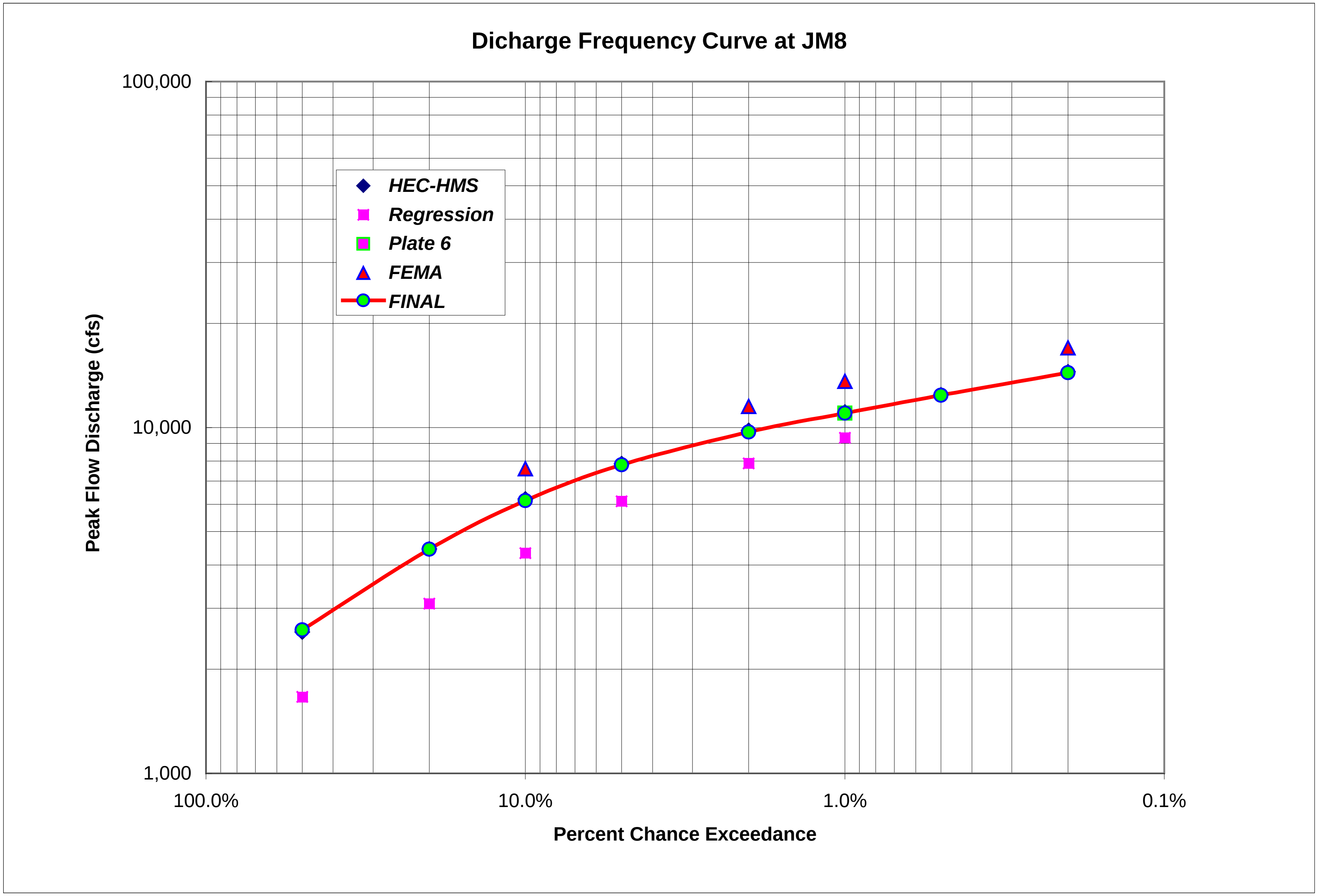

Figure 5-4. Final Discharge Frequency Curve at Junction JM8 (Upstream of the Confluence of Mānoa & Pālolo Streams)

5.4 Pālolo Peak Flow Discharges

Pālolo peak flow discharges at junctions of interest were determined through the process described in Section 5.1. Table 5-4 provides peak flow discharge results for the sub-watershed by methodology and weighted followed by 'FINAL' values.

Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
JP1 (Pukele Stream Gage 16244000, A= 1.146 mi^2)								
HEC-HMS	320	730	1,150	1,460	1,900	2,220	2,590	3,110
Regression	650	1,160	1,580	2,200	2,810	3,320		
Plate 6						3,400		
HEC-SSP	420	810	1,150	1,530	2,110	2,620	3,190	4,050
Weighted	440	850	1,220	1,620	2,180	2,720	3,040	3,810
FINAL	**400**	**800**	**1,150**	**1,550**	**2,100**	**2,500**	**2,900**	**3,400**
JP2 (Confluence of Pukele and Waiomao Streams, A=2.938 mi^2)								
HEC-HMS	940	2,030	3,190	4,010	5,180	6,040	6,980	8,320
Regression	1,035	1,930	2,700	3,828	4,940	5,880		
Plate 6						6,200		
Weighted	980	1,990	2,980	3,930	5,080	6,020	6,980	8,320
FINAL	**950**	**1,850**	**2,700**	**3,650**	**4,900**	**5,900**	**6,900**	**8,000**
JP3 (Palolo Stream Gage 16247000, A=3.62 mi^2)								
HEC-HMS	1,330	2,710	4,170	5,180	6,620	7,670	8,850	10,500
Regression	1,040	2,020	2,870	4,150	5,410	6,500		
Plate 6						7,700		
FEMA			2,790		4,510	5,340		7,530
HEC-SSP	1,210	2,090	2,780	3,510	4,580	5,470	6,430	7,820
Weighted	1,210	2,260	3,100	4,150	5,140	6,240	7,360	8,410
FINAL	**1,200**	**2,100**	**3,000**	**4,000**	**5,500**	**6,500**	**7,500**	**8,600**
JP4 (Right above the confluence of Manoa and Palolo Streams, A= 4.065 mi^2)								
HEC-HMS	1,550	3,120	4,720	5,810	7,400	8,550	9,860	11,600
Regression	1,040	2,060	2,970	4,330	5,690	6,860		
Plate 6						8,100		
Weighted	1,330	2,660	3,970	5,180	6,660	7,870	9,860	11,600
FINAL	**1,250**	**2,200**	**3,100**	**4,200**	**5,700**	**6,900**	**7,900**	**9,100**

Table 5-4. Peak Flow Discharges at Pālolo Junctions by Methodology

The junction that is just upstream the confluence of the Mānoa and Pālolo streams (JP4) receives the highest amount of peak flow discharge in the Pālolo sub-watershed, as it is situated at the downstream (*makai*) end of the watershed and drainage system. In the Pālolo sub-watershed, at the Pūkele Stream gage junction (JP1), the regression method calculates higher flow discharge values than other methods, and the HEC-HMS model seems to underestimate the peak flow discharges for many of the storms under study; the discharge frequency curve fit closely mirrors the findings of the

HEC-SSP analysis which applied 59 historical annual peaks. However, at the next junction downstream, the confluence of the Pūkele and Wai‘ōma‘o Streams (JP2), all the methodologies used provide similar peak flow discharge values. The HEC-SSP analysis was not used for this junction. The discharge frequency curve for the junction at the Pālolo Stream gage (JP3) seems to be higher than HEC-SSP findings at lower exceedance probabilities, this is probably due to the shorter historical annual peak records and the incontinuous and incorrect records. Downstream at the junction just upstream of the confluence of the Mānoa and Pālolo Stream (JP4), the frequency curve fit is close to the low regression equation values. All of these results are illustrated by junction for the Pālolo sub-watershed in Figures 5-5 through 5-8. These figures graph the peak flow discharge by method over the percent chance exceedance storm.

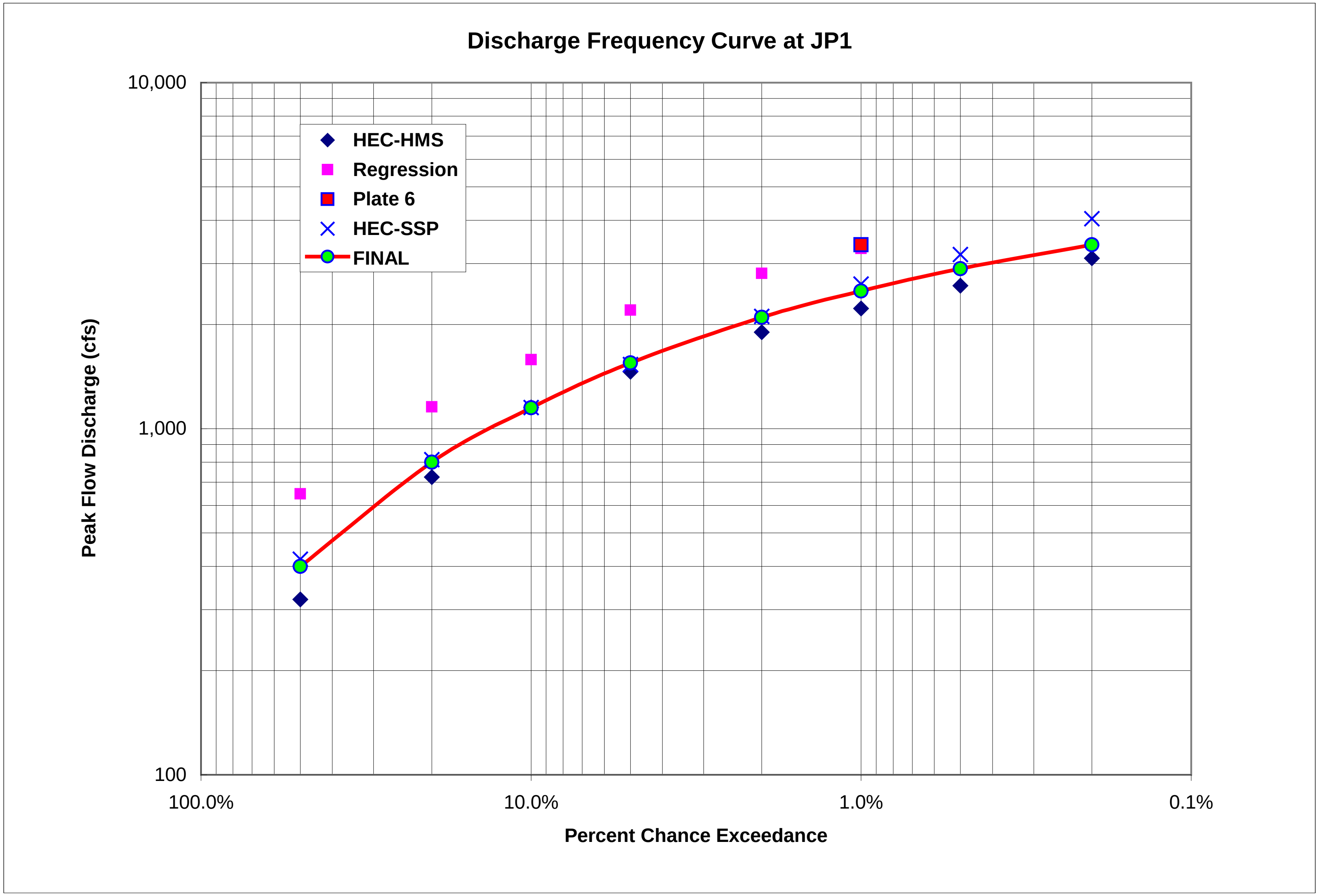

Figure 5-5. Final Discharge Frequency Curve at JP1 (USGS Pūkele Gage [16244000])

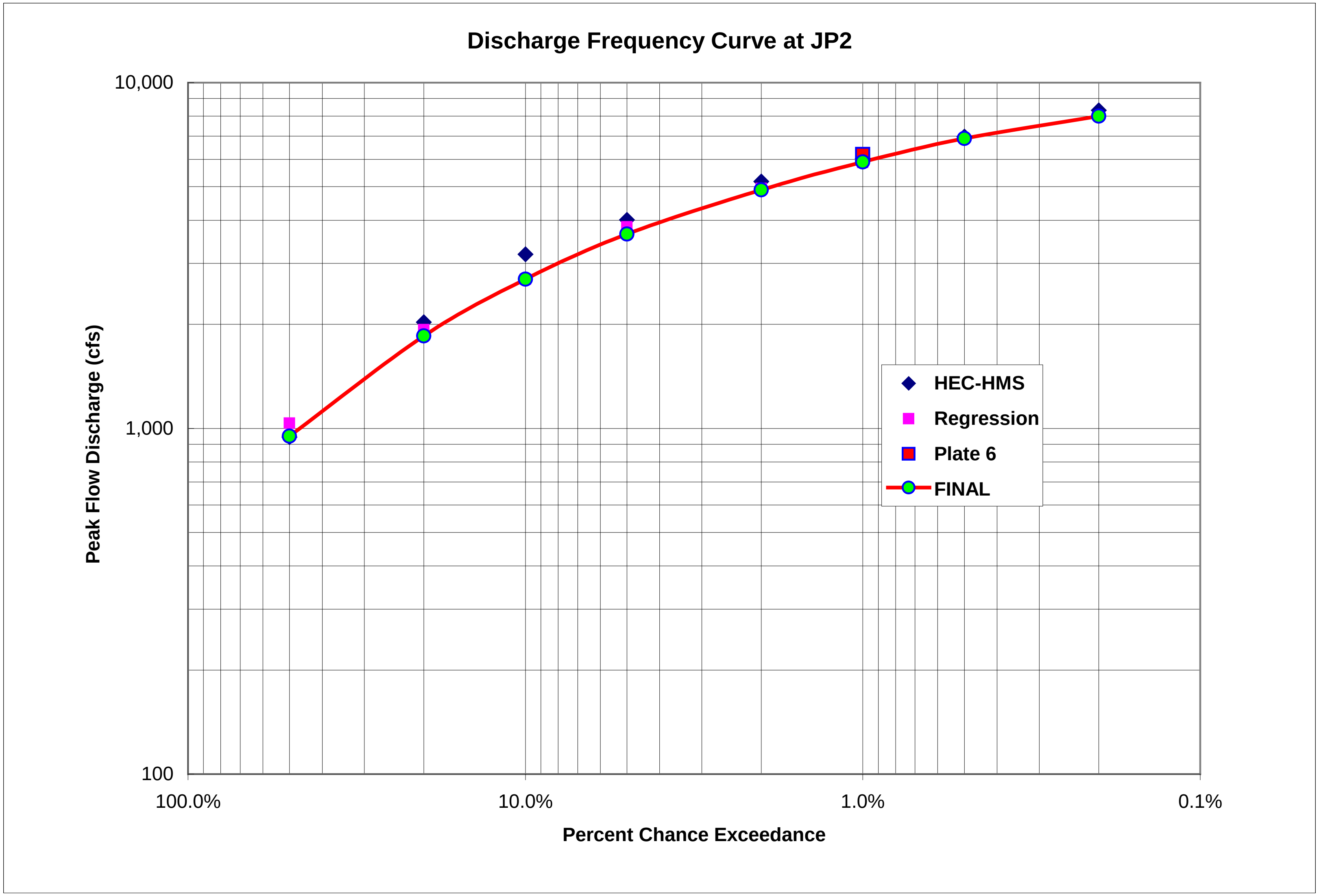

Figure 5-6. Final Discharge Frequency Curve at JP2 (Confluence of Pūkele and Wai'ōma'o Streams)

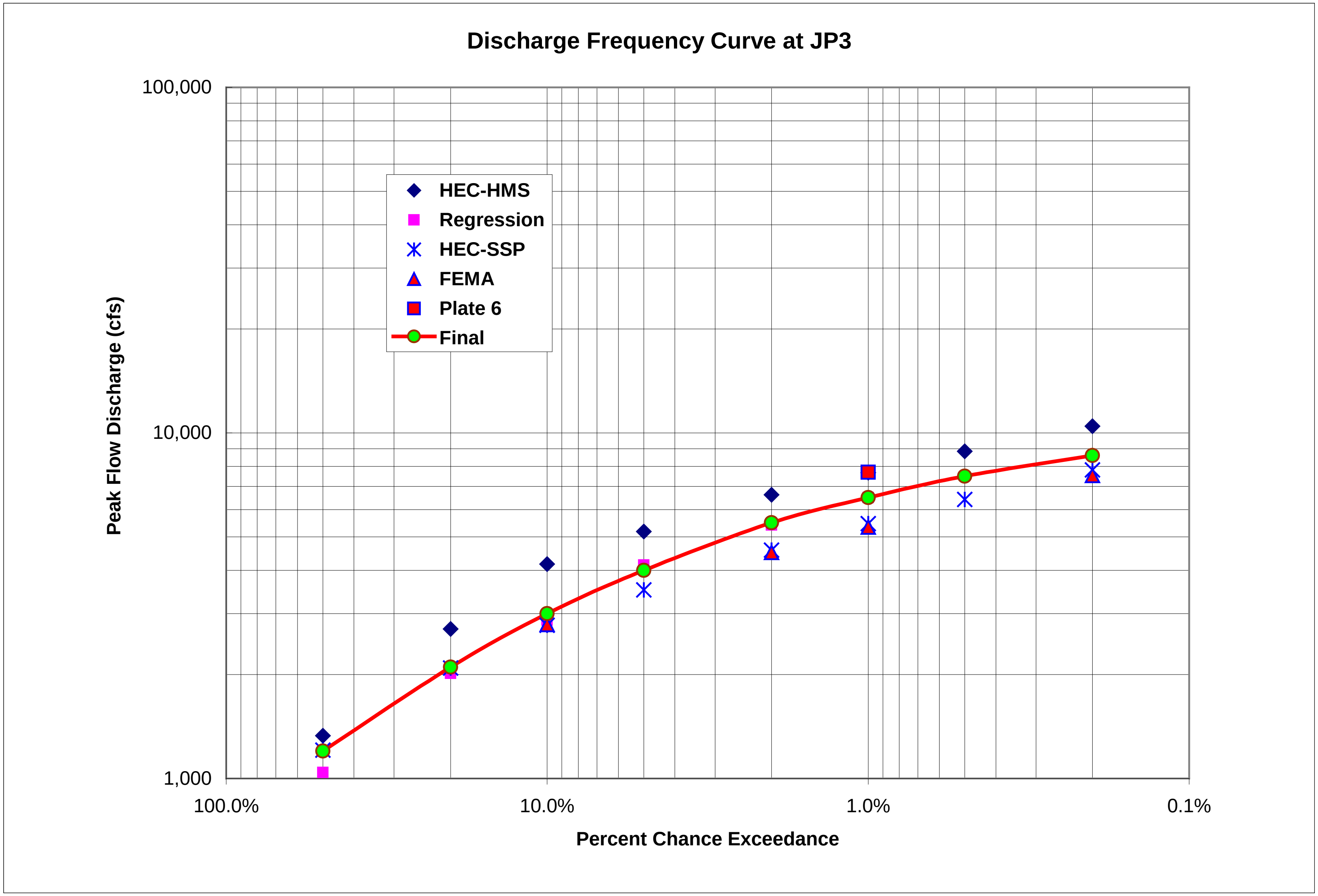

Figure 5-7. Final Discharge Frequency Curve at JP3 (USGS Pālolo Gage [16247000])

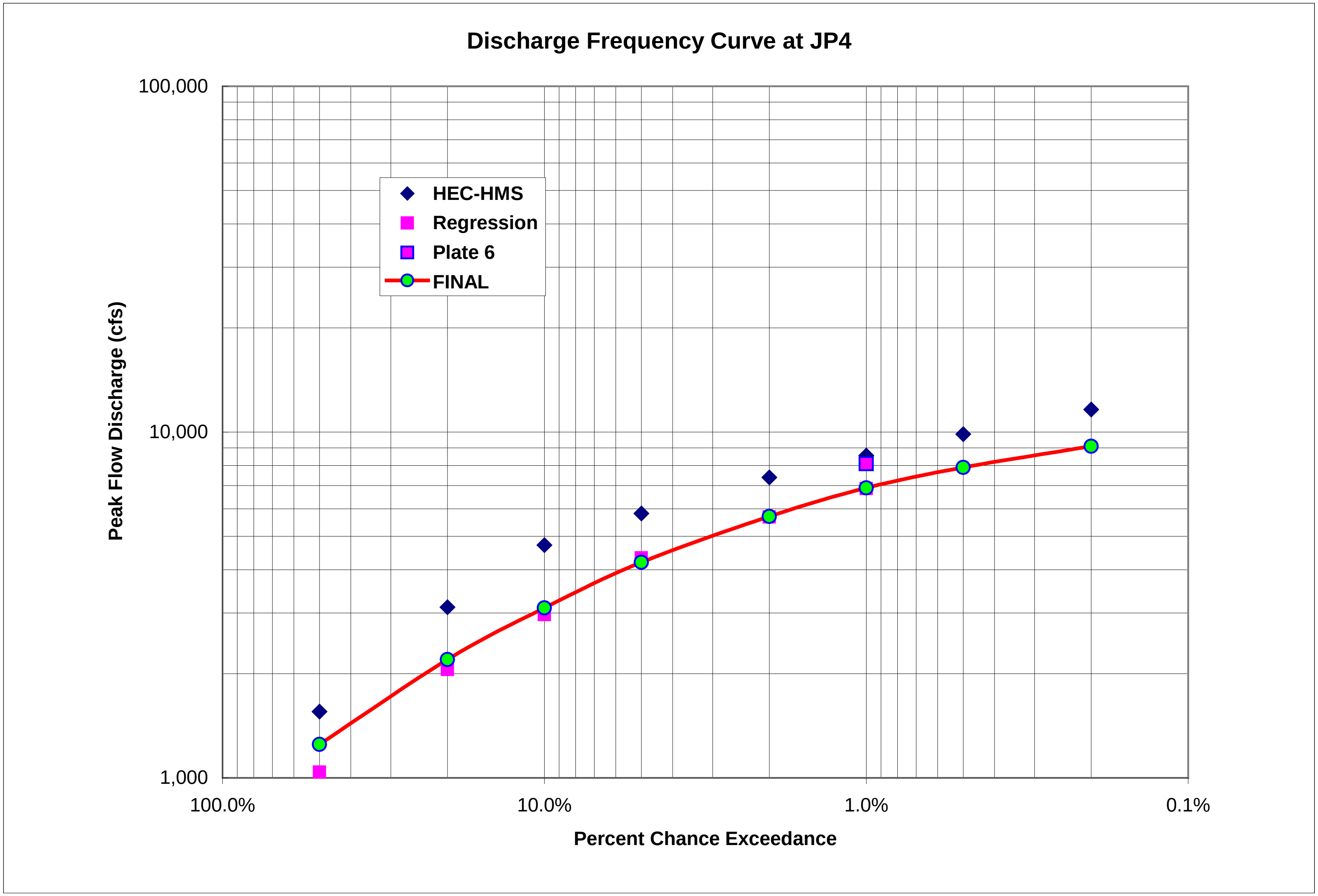

Figure 5-8. Final Discharge Frequency Curve at JP4 (Upstream of the Confluence of Mānoa & Pālolo Streams)

5.5 Mānoa-Pālolo Peak Flow Discharges

Weighting of methodologies were used where peak flow discharges for multiple methodologies were available. Table 5-5 provides peak flow discharge results for the Mānoa-Pālolo Canal by methodology and then as 'FINAL' values through the weighting process described.

Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
JMP1 (Confluence of Manoa and Palolo Streams, A= 10.037 mi²)								
HEC-HMS	4,020	7,170	10,300	12,900	16,100	18,500	20,900	24,400
Regression	2,120	4,060	5,760	8,240	10,700	12,700		
Plate 6						15,500		
Weighted	3,210	5,840	8,360	10,900	13,800	15,800	20,9000	24,400
FINAL	**3,350**	**6,000**	**8,400**	**10,900**	**14,100**	**16,500**	**18,700**	**21,800**
JMP2 (Manoa-Palolo Stream Gage 16247100, A= 10.34 mi²)								
HEC-HMS	4,090	7,340	10,500	13,000	16,300	18,700	21,100	24,700
Regression	2,110	4,080	5,800	8,320	10,800	12,900		
Plate 6						16,000		
FEMA			12,000		19,200	23,000		28,500
HEC-SSP	2,883	5,065	6,800	8,670	11,400	13,700	16,200	19,800
Weighted	3,070	5,520	8,470	9,890	13,900	16,400	18,100	23,200
FINAL	**3,400**	**6,100**	**8,500**	**11,150**	**14,400**	**16,800**	**19,000**	**22,100**
JMP3 (Right above the confluence of Manoa-Palolo and Ala Wai Canals, A=10.678 mi²)								
HEC-HMS	4,220	7,450	10,700	13,300	16,600	18,900	21,400	24,900
Plate 6						16,500		
Weighted	4,220	7,450	10,660	13,260	16,560	18,100	21,400	24,900
FINAL	**3,450**	**6,200**	**8,700**	**11,400**	**14,700**	**17,100**	**19,300**	**22,400**

Table 5-5. Peak Flow Discharges at Mānoa-Pālolo Junctions by Methodology

The junction directly upstream of the confluence of the Mānoa-Pālolo and Ala Wai Canals (JMP3) receives the highest amount of peak flow discharge in the Mānoa-Pālolo Canal sub-watershed. This junction is located at the downstream (*makai*) end of the watershed and drainage system, and so it is not surprising that peak flow discharge would occur at the 'bottom' of the sub-watershed as the water flows down toward sea level. For the Mānoa-Pālolo Canal junctions studied (JMP1 and JMP3), the HEC-HMS modeling results provide higher peak flow discharges than the other methodologies used, particularly the Regression method and HEC-SSP calculation. At junction JMP2 (USGS gage 16247100), the final *best* estimates are lower than HEC-SSP findings but parallel to those values. Noda and Associates (1994) used 24 historical annual peaks to determine the peak flow discharges; their result for 100 year was at 12,429 cfs, whereas in this study, HEC-SSP provided 13,700 cfs. These results are illustrated in the final discharge frequency curves Figures 5-9 through 5-11. These figures show the peak flow discharge by method and junction, and dependent on the percent chance exceedance storm.

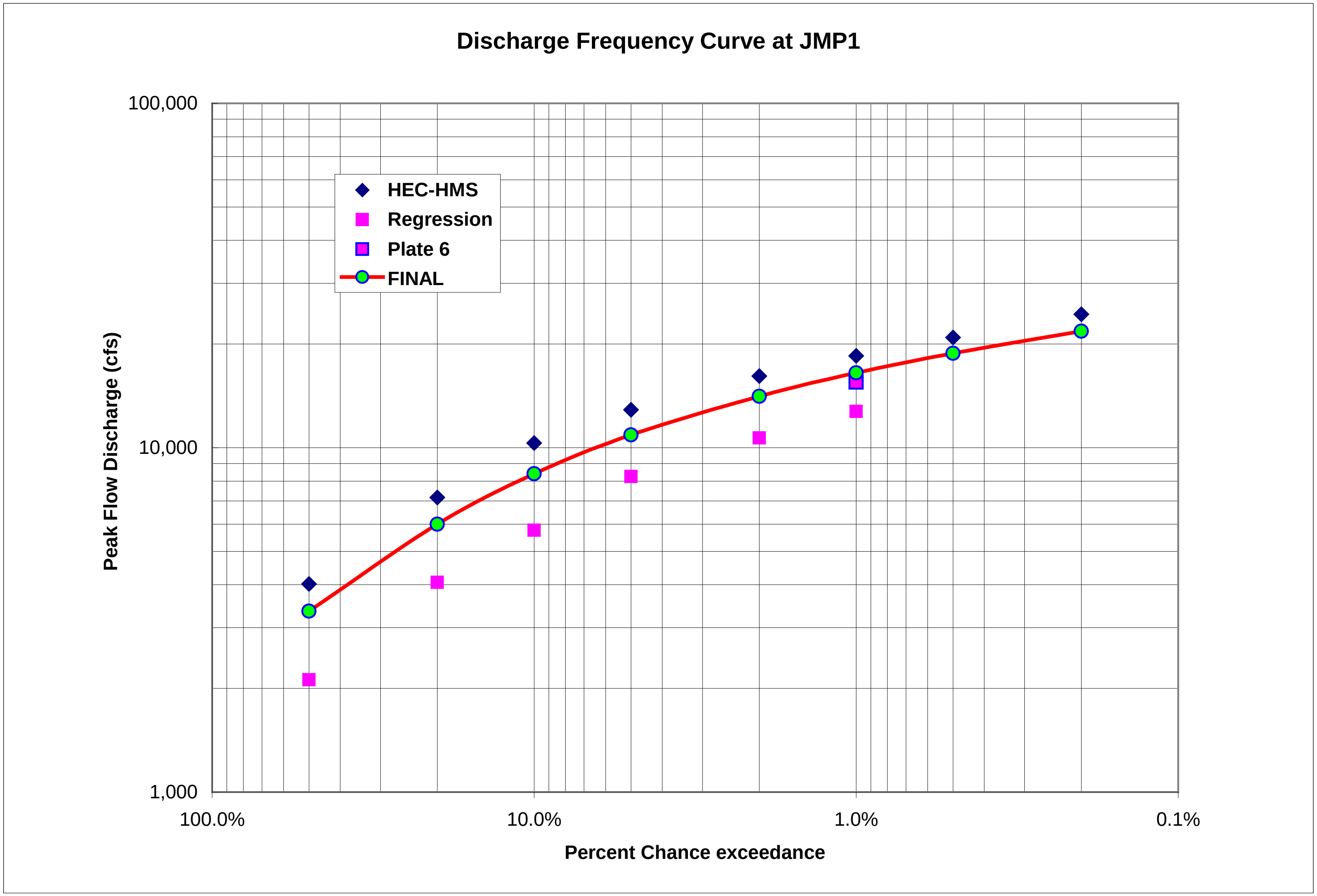

Figure 5-9. Final Discharge Frequency Curve at JMP1 (Confluence of Mānoa & Pālolo Streams)

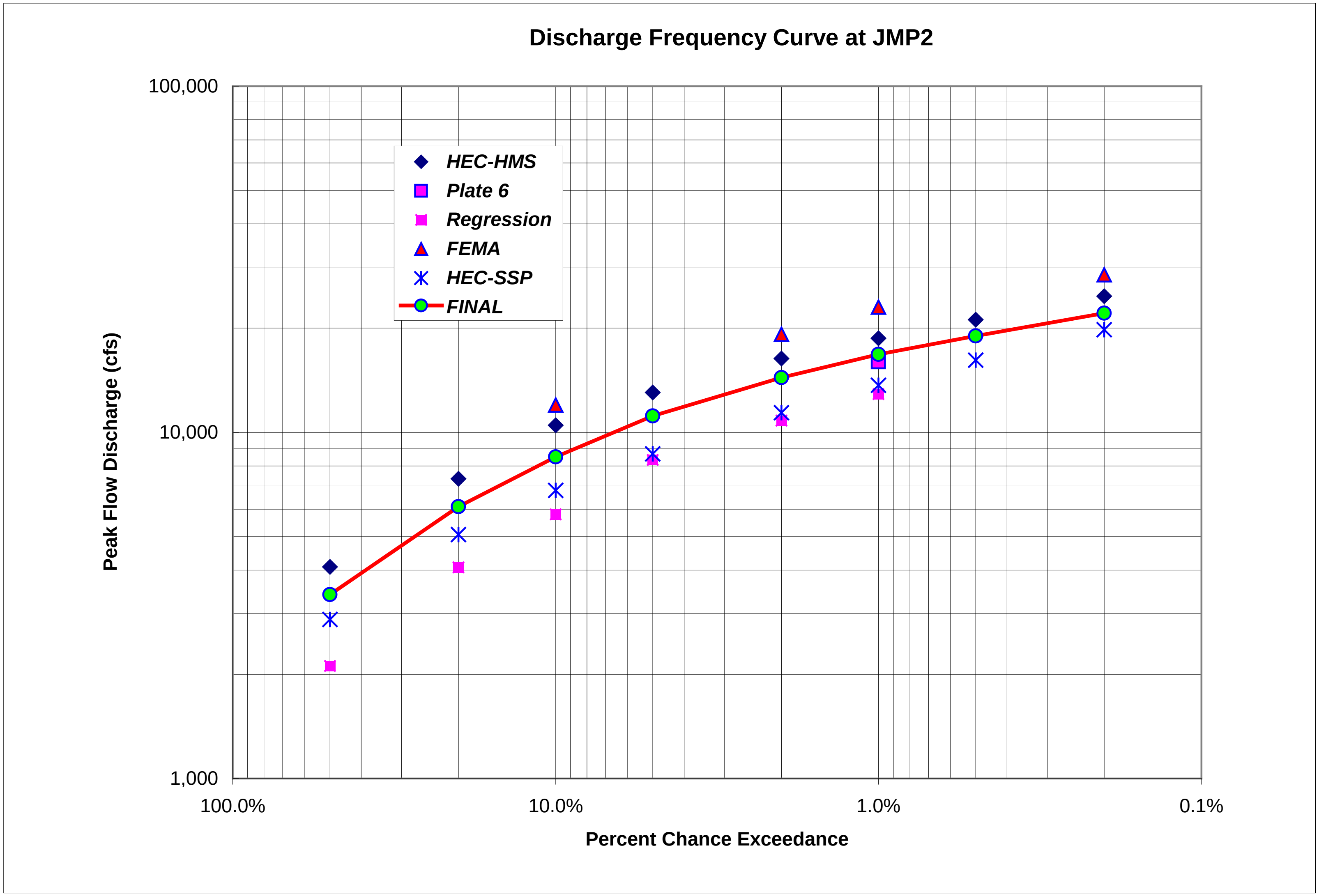

Figure 5-10. Final Discharge Frequency Curve at JMP2 (USGS Stream Gage [16247100])

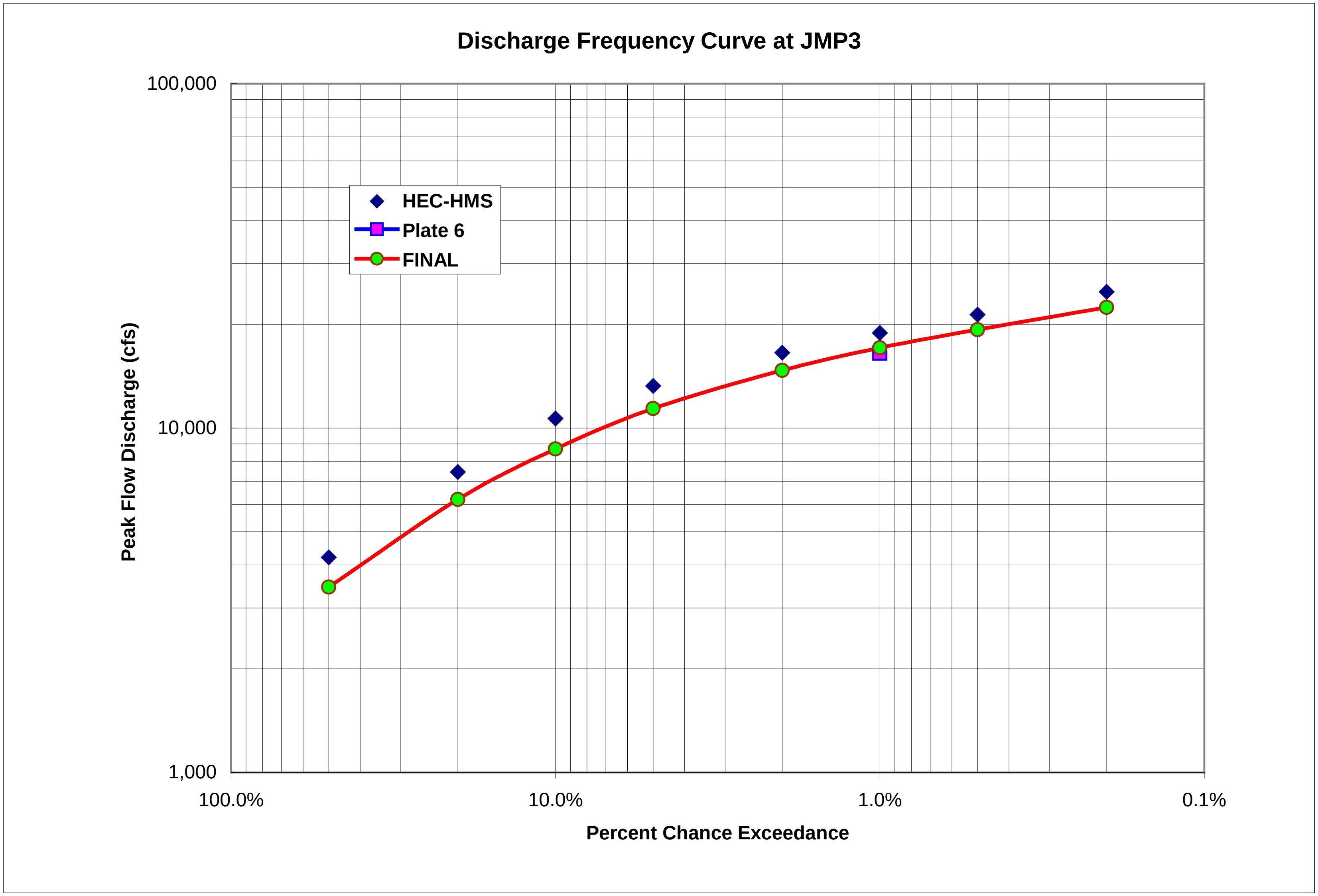

Figure 5-11. Final Discharge Frequency Curve at JMP3 (Confluence of Mānoa -Pālolo and Ala Wai Canals)

5.6 Ala Wai Canal Peak Flow Discharges

As mentioned earlier, Ala Wai Canal was modeled as a reservoir, considering backwater effects caused by the tides due to the sub-watershed location near mean sea level. The reservoir model treated Ala Wai Canal and the adjacent lower area as a detention basin. As the modeled flood wave passes through the reservoir, storage occurs that can greatly reduce the peak flow. The magnitude of this reduction depends on the boundary setting of the modeled reservoir. The storage-elevation function for the Ala Wai Canal reservoir model was determined using bathymetric survey data for the channel and LiDAR data for the surrounding area (Section 4.6.1). No other method accounted for analysis of the surrounding storage area; consequently, the flow peaks determined by other methods are much higher than those determined by the reservoir model. In conclusion, the HEC-HMS results that modeled Ala Wai Canal as a reservoir are considered the most accurate.

Table 5-6 provides peak flow discharge results for Ala Wai Canal sub-watersheds by methodology and then weighted followed by 'FINAL' values through the best fit curve process.

Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
Ala Wai Canal (Mouth of Ala Wai Canal, A=16.215 mi^2)								
HEC-HMS	6,000	10,100	13,400	15,200	16,700	17,700	18,700	20,500
Plate 6						22,500		
FEMA			13,700		23,000	28,200		36,200
Weighted	6,000	10,100	13,500	15,200	19,400	22,300	18,700	27,200
FINAL	**6,000**	**9,500**	**12,500**	**15,200**	**17,500**	**18,500**	**19,500**	**20,500**

Table 5-6. Peak Flow Discharges at the Ala Wai Canal Mouth by Methodology

The inflows to Ala Wai Canal increased, whereas the outflow did not increase significantly. For example, at the 50-year frequency storm, inflow was estimated as 24,850 cfs from HEC-HMS model, and the outflow from the Ala Wai Canal was estimated as 16,700 cfs with a peak elevation of 5.4 feet. At the 100-year frequency storm, HMS model shows that inflow was 28,200 cfs, and outflow was 17,700 cfs at a peak elevation at 5.8 feet. The canal will be overtopped at this storm condition and the water will be stored in the adjacent areas. Figure 5-12 shows the peak flow discharge over the percent chance exceedance by methodology at the mouth of the Ala Wai Canal.

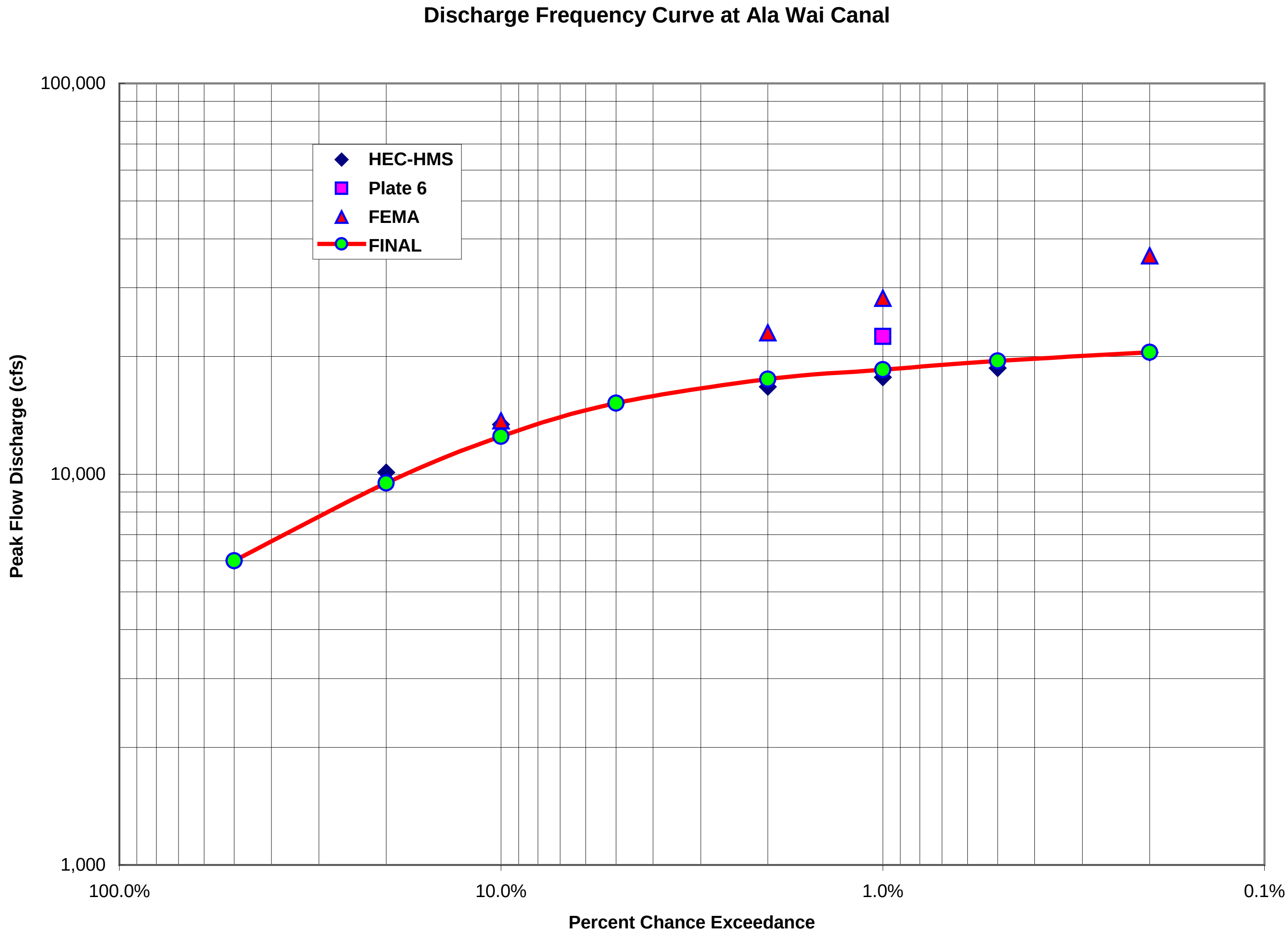

Figure 5-12. Final Discharge Frequency Curve at the Mouth of the Ala Wai Canal

5.7 Peak Flow Discharge Update (March 2016)

As discussed in Hydraulic Appendix, peak flow values were updated and adjusted based on new rainfall-frequency-intensity data and regional regression equations. When the hydrologic studies for Manoa and Ala Wai Watersheds were conducted, the 1984 rainfall frequency data for Oahu was used in the rainfall-runoff modeling (Giambelluca and others, 1984). In March 2009, the updated rainfall frequency data for the State of Hawaii was released as the Precipitation Frequency Data Server (PFDS) which is part of National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Volume 4, Version 2.0, Hawaiian Islands, released March 30, 2009. Atlas 14 is official documentation of precipitation frequency estimates for the United States. Documentation can be found at: http://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume4.pdf, last accessed September 28, 2009 while the actual; server is located at: http://hdsc.nws.noaa.gov/hdsc/pfds/hi/hi_pfds.html. This tool computes the rainfall frequency and intensity with 90 percent confidence limits for the 1- to 100-year storms for durations from 5 minutes to 60 days. The updated rainfall frequency values are presented in Table 5-7. A comparison between the previous (Table 4-1) and newer NOAA rainfall frequency duration values, indicated that the newer intensity values were higher than the older data by an average of 4 to 13 percent depending on the rainfall recurrence interval and duration. Only the lower rainfall intensity durations at the 50- and 10-percent chance storms showed the Report R-73 values to be consistently higher. In general the 10- and 1-percent (10- and 100-year) rainfall intensities, all durations (5 minutes to 24-hours) were on average higher by 0.04 inches and 0.44 inches. The NOAA values also tended to be higher in general compared to the Report R-73 for the Manoa watershed as well. Manoa watershed values were higher on average by 0.10 inches and 0.62 inches for the 10- and 1-percent chance storms. In general, the shorter frequency time periods had a larger change then the longer rainfall time periods.

The NOAA rainfall frequency duration values were used in the same HEC-HMS model as the Report R-73 rainfall frequency duration values to generate updated peak flow discharges for all frequency storm events. The average percent difference between the newer HEC-HMS peak flow values compared with the older (2008) range varied from minus 7 percent for the Manoa Stream 10-percent chance flood to plus 36 percent for the non-Manoa Stream 50-percent chance floods. In general, the Manoa watershed peak flow values did not show significant changes as the Report R-73 rainfall frequency duration values tended to be higher than the NOAA rainfall frequency duration values for the 20 to 2 percent annual chance exceedence flood events. For the non-Manoa watersheds, the NOAA rainfall frequency duration values generated peak flow values from 7 to 36 percent larger than the Report R-73 rainfall frequency duration values depending on recurrence interval. Results of the updated HEC-HMS model with the prior peak flow computations by junction are presented in Table 5-8. Table 5-8 presents all the prior data in Table 28 from the Manoa Watershed Report (Oceanit, 2008b) and Tables 5-2, 5-3, 5-4, 5-5, and 5-6 of this report. Two methods used in the Manoa Watershed study (Oceanit, 2008b) but not used in this study for the Ala Wai Watershed were the TR-55 method and FLO-2D model.

Also after the hydrologic studies for the Manoa and Ala Wai watersheds were completed, the USGS released newer regional regression equations for the State of Hawaii in 2010 (Oki and others, 2010). The newer regional regression equations for Leeward O'ahu, used the same independent variables, drainage area and mean annual precipitation, as the older 1994 Leeward O'ahu equations (Table 4-

22). Changes were in the independent variable coefficients (Oki and others, 2010, Table 13). The standard error of prediction in percent and coefficient of determination, R^2, were similar among the two sets of equations. The 1994 equations had an average standard error of prediction of 40-percent and R^2 of 0.72. The 2010 equations had an average standard error of predication of 42-percent and an R^2 of 0.75. The 2010 equations, as presented in table 5-9, provided an equation for the 0.2-percent chance flood which the 1994 equations did not. The same independent variables used with the 1994 equators were used in the 2010 equations and these results by junction are presented in Table 5-8. Note that at junctions JK3 and JMP3, the 2008 analysis did not compute regression equation results so these were added to the update in Table 5-8. Also note that the regional regression equations are provided for the 4-percent chance (25-year) flood but presented here in the various tables as the 5-perecnt chance (20-year) flood. For use of these regression equations, the two recurrence intervals were considered sufficiently close for comparison purposes in this report. Peak flow discharge values from using the 2010 regression equations resulted in values all lower than values derived from the 1994 regression equations.

Percent Chance Exceedance	Return Period (years)	Depth (inches) for Specified Duration								
		5-min	15-min	30-min	1-hour	2-hours	3-hours	6-hours	12-hours	24-hours
99	1	0.38	0.66	0.97	1.40	1.87	2.12	2.74	3.35	3.92
50	2	0.47	0.80	1.19	1.72	2.33	2.71	3.49	4.29	5.18
20	5	0.61	1.04	1.54	2.22	3.04	3.54	4.58	5.68	6.96
10	10	0.72	1.24	1.83	2.64	3.61	4.21	5.46	6.80	8.39
5	20	0.81	1.49	2.11	3.05	4.15	4.94	6.28	8.00	9.95
4	25	0.89	1.52	2.25	3.24	4.42	5.16	6.69	8.36	10.42
2	50	1.02	1.75	2.59	3.74	5.09	5.94	7.69	9.61	12.05
1	100	1.16	1.99	2.95	4.25	5.78	6.74	8.74	10.92	13.77
0.5	200	1.31	2.25	3.34	4.82	6.53	7.61	9.86	12.30	15.60
0.2	500	1.53	2.62	3.88	5.61	7.57	8.82	11.42	14.23	18.18

Rainfall Intensity Frequency data determined from NOAA Atlas 14 Precipitation Frequency Data Server using watershed centroid of 21.3092 N, 157.8071 W. Values for the 5-percent chance storm are interpolated.

Revision of data in Table 4-1

Table 5-7. Updated Rainfall Intensity Frequency Data for the Ala Wai Watershed, Oahu, Hawaii

As part of the update, the same weighting equations and factors (Table 5-1) were used with both the HEC-HMS computations with the NOAA rainfall frequency duration values and the USGS 2010 equation values along with prior Plate 6, FEMA, and HEC-SSP results were applicable, to provided new weighted results (Table 5-8). These newer results along with the prior computed peak flow estimates were used to determine new peak flow estimates. In many cases, for Manoa and Palolo Streams, only the rarer 0.5-percent and 0.2-perecent chance flood were revised by increasing the peak flow values. In other cases at Makiki Stream and the Ala Wai Canal, all flood frequency events were increased from the prior 2008 computed values (Table 5-8).

The updated adjusted peak discharge values by junction are listed in Table 5-8 with all computed values and more succinctly with just the final determined values in Table 5-10. As discussed prior in Section 5.1, the final determination of the peak flow discharges was based upon engineering judgment incorporating both the weighted equation values and the graphical adjustments. These values were then adjusted by location, as described in the Hydraulic Appendix for use in the HEC-RAS model. The uncertainty of the peak flow discharge values, as discussed in Section 5.1, is based on the equivalent years of record. The final equivalent years of record (EYOR) used in the risk and uncertainty HEC-FDA model is based on stream reach and is presented in Table 5-11. The Makiki Watershed with the least amount of available data was given the lowest EYOR of 18 years, while the remaining sub-watersheds were assigned values from 25 to 30 years. The highest values were from sub-basins where the peak flow discharges were almost entirely based on gaged data; Pukele and Waiomao Streams. The assigned EYOR is based on the overall confidence in the reliability or accuracy of the peak flow discharge estimates and as applied in HEC-FDA constrains the confidence limits of the sampling of peak flow discharge estimates. The HEC-FDA analytical frequency curve data is initial values and may be changed as part of the hydraulic and economic modeling efforts as described in the hydraulic and economic appendices. The analytical curve frequency values in Table 5-11 were determined from the final output discharges computed by HEC-RAS for the intermediate sea-level rise scenario (Appendix A2 and Appendix A3) and methodology in Bulletin 17B, Appendix 5 (Interagency Advisory Committee on Water Data, 1982) to match HEC-RAS input data into HEC-FDA. Actual use of analytical frequency curve values versus graphical curve values is discussed in the Economic Appendix.

Table 5-8: Peak Flow Discharge Values Update								
Methodology	**Peak Flow Discharge (cfs)**							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.50%**	**0.20%**
JM1 (confluence of Waihī and Waiakeakua Streams, A=2.27 mi^2)								
TR-55	1,270	2,040	2,580	3,170	3,820	4,270	4,800	5,590
HEC-HMS (R-73)	*1,120*	*2,140*	*2,870*	*3,570*	*4,340*	*5,010*	*5,610*	*6,600*
HEC-HMS (Atlas 14)	*1,270*	*2,000*	*2,630*	*3,340*	*4,340*	*5,160*	*6,060*	*7,330*
FLO-2D	1,260	2,920	3,800	4,740	6,010	6,950	7,670	8,800
Regression (1994)	1,010	1,800	2,470	3,430	4,380	5,160	---	---
Regression (2010)	*780*	*1,460*	*2,080*	*2,900*	*3,640*	*4,460*	---	*6,820*
Plate 6	---	---	---	---	---	5,400	---	---
FEMA	---	---	3,750	---	5,600	6,500	---	8,100
Weighted (2008)	1,160	2,240	3,130	3,780	4,900	5,630	6,180	7,430
Weighted (2016)	*1,130*	*2,110*	*3,000*	*3,570*	*4,740*	*5,520*	*6,350*	*7,450*
FINAL (2008)	1,200	2,000	2,800	3,600	4,600	5,400	6,100	6,900
FINAL (2016)	**1,200**	**2,000**	**2,800**	**3,600**	**4,600**	**5,500**	**6,200**	**7,400**
JM2 (confluence of Mānoa Stream and Woodlawn Ditch, A=3.81 mi^2)								
TR-55	2,210	3,500	4,410	5,410	6,500	7,260	8,140	9,470
HEC-HMS (R-73)	1,870	3,590	4,800	5,960	7,210	8,320	9,280	10,900
HEC-HMS (Atlas 14)	*2,140*	*3,350*	*4,400*	*5,540*	*7,200*	*8,530*	*9,980*	*12,100*
FLO-2D	1,730	3,670	5,040	6,330	7,920	8,710	9,100	9,800
Regression (1994)	1,490	2,630	3,580	4,930	6,250	7,320	---	---
Regression (2010)	*1,190*	*2,170*	*3,050*	*4,210*	*5,230*	*6,360*	---	*9700*
Plate 6	---	---	---	---	---	7,800	---	---
FEMA	---	---	5,500	---	8,400	9,800	---	12,100
Weighted (2008)	1,790	3,330	4,680	5,680	7,310	8,270	8,930	10,670
Weighted (2016)	*1,780*	*3,140*	*4,490*	*5,370*	*7,090*	*8,120*	*9,190*	*10,700*
FINAL (2008)	1,700	3,200	4,600	5,700	7,150	8,150	9,000	10,300
FINAL	**1,700**	**3,200**	**4,600**	**5,700**	**7,150**	**8,200**	**9,000**	**10,500**
JM3 (Lowrey Avenue Bridge, A=4.22 mi^2)								
TR-55	2,500	3,930	4,940	6,040	7,260	8,100	9,070	10,500
HEC-HMS (R-73)	2,110	3,990	5,330	6,620	8,010	9,230	10,300	12,100
HEC-HMS (Atlas 14)	*2,310*	*3,640*	*4,790*	*6,040*	*7,840*	*9,290*	*10,900*	*13,100*
FLO-2D	1,820	3,610	4,760	5,930	7,190	7,330	7,540	10,200
Regression (1994)	1,560	2,770	3,780	5,220	6,630	7,780	---	---
Regression (2010)	*1,250*	*2,290*	*3,200*	*4,460*	*5,550*	*6,750*	---	*10400*
Plate 6	---	---	---	---	---	8,400	---	---
FEMA	---	---	5,700	---	9,000	10,500	---	13,000
Weighted (2008)	1,950	3,540	4,900	5,940	7,640	8,580	8,960	11,540
Weighted (2016)	*1,920*	*3,320*	*4,660*	*5,580*	*7,380*	*8,390*	*9,180*	*11,500*
FINAL (2008)	1,900	3,500	4,800	6,000	7,350	8,350	9,300	10,800
FINAL	**1,900**	**3,500**	**4,800**	**6,000**	**7,400**	**8,350**	**9,300**	**11,000**
JM4 (Woodlawn Drive Bridge, A=4.49 mi^2)								
TR-55	2,690	4,220	5,290	6,470	7,750	8,640	9,680	11,200
HEC-HMS (R-73)	2,270	4,270	5,700	7,020	8,410	9,790	10,900	12,800
HEC-HMS (Atlas 14)	*2,500*	*3,880*	*5,090*	*6,380*	*8,320*	*9,780*	*11,500*	*13,900*
FLO-2D	2,000	4,060	4,760	6,730	7,900	8,020	8,550	11,300
Regression (1994)	1,580	2,830	3,880	5,380	6,850	8,040	---	---
Regression (2010)	*1,280*	*2,350*	*3,310*	*4,590*	*5,720*	*6,980*	---	*10,700*
Plate 6	---	---	---	---	---	8,900	---	---
FEMA	---	---	6,000	---	9,400	11,000	---	13,700
Weighted (2008)	2,080	3,810	5,110	6,390	8,080	9,090	9,710	12,350
Weighted (2016)	*2,070*	*3,570*	*4,860*	*6,000*	*782*	*8,880*	*9,940*	*12,200*
FINAL (2008)	2,000	3,700	5,000	6,300	7,700	8,700	9,800	11,600
FINAL	**2,000**	**3,700**	**5,000**	**6,300**	**7,700**	**8,800**	**9,900**	**11,700**

Table 5-8: Peak Flow Discharge Values Update -- Continued

Methodology	Peak Flow Discharge (cfs)							
Return Period (yr)	2	5	10	20	50	100	200	500
Percent Chance Exceedance	50%	20%	10%	5%	2%	1%	0.50%	0.20%
JM5 (Noelani Bend, A=4.68 mi^2)								
TR-55	2,830	4,420	5,540	6,760	8,110	9,030	10,100	11,700
HEC-HMS (R-73)	2,370	4,440	5,940	7,310	8,800	10,200	11,300	13,400
HEC-HMS (Atlas 14)	*2,620*	*4,050*	*5,320*	*6,680*	*8,710*	*10,210*	*12,000*	*14,500*
FLO-2D	2,060	4,190	5,850	7,480	8,900	9,500	9,890	14,400
Regression (1994)	1,600	2,880	3,950	5,490	6,990	8,220	---	---
Regression (2010)	*1,290*	*2,380*	*3,370*	*4,680*	*5,840*	*7,130*	---	*11,000*
Plate 6	---	---	---	---	---	9,100	---	---
FEMA	---	---	6,200	---	9,700	11,400	---	14,200
Weighted (2008)	2,160	3,940	5,490	6,760	8,530	9,620	10,470	13,580
Weighted (2016)	*2,140*	*3,700*	*5,240*	*6,370*	*8,260*	*9,410*	*10,700*	*13,300*
FINAL (2008)	2,100	3,800	5,200	6,700	8,200	9,350	10,500	12,000
FINAL	**2,100**	**3,800**	**5,200**	**6,700**	**8,200**	**9,400**	**10,600**	**12,400**
JM6 (Dole Street Bridge, A=5.35 mi^2)								
TR-55	3,280	5,110	6,390	7,790	9,320	10,400	11,600	13,500
HEC-HMS (R-73)	2,780	5,140	6,860	8,390	10,200	11,700	13,100	15,400
HEC-HMS (Atlas 14)	*3,070*	*4,710*	*6,160*	*7,660*	*10,000*	*11,760*	*13,800*	*16,700*
FLO-2D	2,060	4,260	5,990	7,860	9,000	9,520	9,970	14,500
Regression (1994)	1,640	3,000	4,160	5,830	8,870	8,820	---	---
Regression (2010)	*1,330*	*2,490*	*3,550*	*5,000*	*6,230*	*7,630*	---	*11,900*
Plate 6	---	---	---	---	---	10,000	---	---
FEMA	---	---	6,700	---	10,500	12,500	---	15,600
Weighted (2008)	2,430	4,270	5,930	7,260	9,600	10,610	12,090	14,510
Weighted (2016)	*2,360*	*4,060*	*5,710*	*7,010*	*8,990*	*10,300*	*11,800*	*14,500*
FINAL (2008)	2,300	4,100	5,700	7,200	9,000	10,100	11,400	13,200
FINAL	**2,300**	**4,100**	**5,700**	**7,200**	**9,000**	**10,200**	**11,800**	**13,600**
JM7 (Kānewai Field Gage, A=5.64 mi^2)								
TR-55	3,450	5,380	6,730	8,210	9,830	10,900	12,200	14,200
HEC-HMS (R-73)	2,920	5,410	7,190	8,820	10,700	12,300	13,700	16,200
HEC-HMS (Atlas 14)	*3,220*	*4,930*	*6,470*	*8,040*	*10,500*	*12,300*	*14,400*	*17,500*
FLO-2D	2,120	4,380	6,090	7,900	9,000	9,610	10,100	14,900
Regression (1994)	1,660	3,050	4,250	5,980	7,670	9,070	---	---
Regression (2010)	*1,340*	*2,530*	*3,620*	*5,090*	*6,390*	*7,840*	---	*12,300*
Plate 6	---	---	---	---	---	10,200	---	---
FEMA	---	---	6,900	---	10,900	12,900	---	16,100
Weighted (2008)	2,450	4,480	6,200	7,680	9,610	10,840	11,980	15,450
Weighted (2016)	*2,500*	*4,210*	*5,910*	*7,230*	*9,300*	*10,600*	*12,200*	*15,100*
FINAL (2008)	2,500	4,300	6,000	7,600	9,500	10,700	12,000	14,000
FINAL	**2,500**	**4,300**	**6,000**	**7,600**	**9,500**	**10,700**	**12,100**	**14,500**
JM8 (Right above Confluence of Manoa and Palolo Streams, A=5.97 mi^2)								
HEC-HMS (R-73)	2,560	4,450	6,210	7,860	9,810	11,100	12,400	14,500
HEC-HMS (Atlas 14)	*3,360*	*5,170*	*6,760*	*8,400*	*10,900*	*12,800*	*15,000*	*18,200*
Regression (1994)	1,660	3,100	4,330	6,120	7,870	9,330	---	---
Regression (2010)	*1,350*	*2,570*	*3,690*	*5,200*	*6,550*	*8,050*	---	*12,700*
Plate 6	---	---	---	---	---	11,000	---	---
FEMA	---	---	7,600	---	11,500	13,600	---	17,000
Weighted (2008)	2,180	3,870	6,060	7,110	9,730	11,200	12,400	15,600
Weighted (2016)	*2,440*	*4,110*	*6,100*	*7,240*	*9,660*	*11,200*	*13,300*	*15,400*
FINAL (2008)	2,600	4,450	6,150	7,800	9,700	11,000	12,400	14,400
FINAL	**2,600**	**4,450**	**6,150**	**7,800**	**9,700**	**11,200**	**12,500**	**15,300**

Table 5-8: Peak Flow Discharge Values Update -- Continued								
Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.50%**	**0.20%**
JK1 (Confluence of Makiki and Kanaha Streams, A=2.33 mi^2)								
HEC-HMS (R-73)	570	1,200	1,890	2,400	3,150	3,740	4,380	5,240
HEC-HMS (Atlas 14)	*800*	*1,430*	*1,990*	*2,630*	*3,570*	*4,360*	*5,240*	*6,480*
Regression (1994)	660	1,350	1,960	2,900	3,840	4,680	---	---
Regression (2010)	*510*	*1,070*	*1,610*	*2,400*	*3,120*	*3,960*	---	*6,500*
Plate 6	---	---	---	---	---	5,300	---	---
Weighted (2008)	610	1,260	1,920	2,620	3,450	4,410	4,380	5,240
Weighted (2016)	*680*	*1,280*	*1,830*	*2,530*	*3,380*	*4,190*	---	*6,490*
FINAL (2008)	650	1,300	1,900	2,550	3,400	4,100	4,800	5,700
FINAL	**650**	**1,300**	**1,900**	**2,550**	**3,400**	**4,200**	**4,800**	**6,400**
JK2 (USGS Stream Gage at King St. 16238000, A= 2.49 mi^2)								
HEC-HMS (R-73)	*660*	*1,360*	*2,110*	*2,650*	*3,440*	*4,060*	*4,730*	*5,630*
HEC-HMS (Atlas 14)	*930*	*1,610*	*2,220*	*2,900*	*3,900*	*4,730*	*5,660*	*6,960*
Regression (1994)	*670*	*1,370*	*2,000*	*2,980*	*3,960*	*4,850*	---	---
Regression (2010)	*520*	*1,090*	*1,650*	*2,460*	*3,220*	*4,090*	---	*6,750*
Plate 6	---	---	---	---	---	5,600	---	---
FEMA	---	---	1,850	---	3,250	3,950	---	5,950
Weighted (2008)	660	1,360	2,000	2,790	3,540	4,490	4,730	5,770
Weighted (2016)	750	1,390	1,940	2,710	3,500	4,520	---	6,870
FINAL (2008)	660	1,330	1,960	2,580	3,500	4,250	4,950	5,900
FINAL	**700**	**1,400**	**2,000**	**2,700**	**3,500**	**4,500**	**5,600**	**6,700**
JK3 (Confluence of Makiki Stream and Ala Wai Canal, A=2.89 mi^2)								
HEC-HMS (R-73)	*890*	*1,770*	*2,690*	*3,340*	*4,280*	*5,000*	*5,790*	*6,850*
HEC-HMS (Atlas 14)	*1,220*	*2,110*	*2,930*	*3,710*	*4,890*	*5,870*	*6,910*	*8,430*
Regression (2010)	*540*	*1,150*	*1,750*	*2,640*	*3,470*	*4,420*	---	*7,390*
Plate 6	---	---	---	---	---	6,100	---	---
Weighted (2008)	890	1,770	2,690	3,340	4,280	5,370	5,790	6,850
Weighted (2016)	930	1,700	2,420	3,250	4,280	5,440	---	7,980
FINAL (2008)	760	1,600	2,400	3,200	4,300	5,250	6,100	7,200
FINAL	**900**	**1,700**	**2,600**	**3,300**	**4,500**	**5,700**	**6,800**	**8,000**

Table 5-8: Peak Flow Discharge Values Update -- Continued								
Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.50%**	**0.20%**
JP1 (Pukele Stream Gage 16244000, A= 1.15 mi^2)								
HEC-HMS (R-73)	320	730	1,150	1,460	1,900	2,220	2,590	3,110
HEC-HMS (Atlas 14)	*380*	*770*	*1,130*	*1,520*	*2,080*	*2,530*	*3,060*	*3,780*
Regression (1994)	650	1,160	1,580	2,200	2,810	3,320	---	---
Regression (2010)	*490*	*920*	*1,310*	*1,850*	*2,320*	*2,860*	---	*4,330*
Plate 6	---	---	---	---	---	3,400	---	---
HEC-SSP	420	810	1,150	1,530	2,110	2,620	3,190	4,050
Weighted (2008)	440	850	1,220	1,620	2,180	2,720	3,040	3,810
Weighted (2016)	*420*	*820*	*1,170*	*1,580*	*2,140*	*2,710*	*3,160*	*4,040*
FINAL (2008)	400	800	1,150	1,550	2,100	2,500	2,900	3,400
FINAL	**440**	**820**	**1,200**	**1,600**	**2,100**	**2,700**	**3,000**	**3,900**
JP2 (Confluence of Pukele and Waiomao Streams, A=2.94 mi^2)								
HEC-HMS (R-73)	940	2,030	3,190	4,010	5,180	6,040	6,980	8,320
HEC-HMS (Atlas 14)	*1,300*	*2,350*	*3,300*	*4,350*	*5,800*	*7,000*	*8,320*	*10,100*
Regression (1994)	1,035	1,930	2,700	3,828	4,940	5,880	---	---
Regression (2010)	*810*	*1,570*	*2,270*	*3,220*	*4,090*	*5,050*	---	*7,920*
Plate 6	---	---	---	---	---	6,200	---	---
Weighted (2008)	980	1,990	2,980	3,930	5,080	6,020	6,980	8,320
Weighted (2016)	*1,090*	*2,020*	*2,860*	*3,870*	*5,070*	*6,170*	*8,320*	*9,170*
FINAL (2008)	950	1,850	2,700	3,650	4,900	5,900	6,900	8,000
FINAL	**1000**	**2,000**	**2,900**	**3,900**	**5,100**	**6,100**	**7,600**	**9,200**
JP3 (Palolo Stream Gage 16247000, A=3.62 mi^2)								
HEC-HMS (R-73)	1,330	2,710	4,170	5,180	6,620	7,670	8,850	10,500
HEC-HMS (Atlas 14)	*1,860*	*3,200*	*4,360*	*5,660*	*7,460*	*8,940*	*10,600*	*12,800*
Regression (1994)	1,040	2,020	2,870	4,150	5,410	6,500	---	---
Regression (2010)	*820*	*1,640*	*2,410*	*3,480*	*4,460*	*5,560*	---	*8,930*
Plate 6	---	---	---	---	---	7,700	---	---
FEMA	---	---	2,790	---	4,510	5,340	---	7,530
HEC-SSP	1,210	2,090	2,780	3,510	4,580	5,470	6,430	7,820
Weighted (2008)	1,210	2,260	3,100	4,150	5,140	6,240	7,360	8,410
Weighted (2016)	*1,320*	*2,320*	*3,070*	*4,150*	*5,170*	*6,350*	*8,030*	*10,800*
FINAL (2008)	1,200	2,100	3,000	4,000	5,500	6,500	7,500	8,600
FINAL	**1,200**	**2,200**	**3,100**	**4,100**	**5,500**	**6,600**	**8,000**	**10,000**
JP4 (Right above the confluence of Manoa and Palolo Streams, A= 4.07 mi^2)								
HEC-HMS (R-73)	1,550	3,120	4,720	5,810	7,400	8,550	9,860	11,600
HEC-HMS (Atlas 14)	*2,170*	*3,700*	*4,970*	*6,350*	*8,350*	*9,980*	*11,800*	*14,200*
Regression (1994)	1,040	2,060	2,970	4,330	5,690	6,860	---	---
Regression (2010)	*820*	*1,670*	*2,480*	*3,620*	*4,670*	*5,850*	---	*9,540*
Plate 6	---	---	---	---	---	8,100	---	---
Weighted (2008)	1,330	2,660	3,970	5,180	6,660	7,870	9,860	11,600
Weighted (2016)	*1,590*	*2,830*	*3,900*	*5,180*	*6,770*	*8,190*	*11,800*	*12,200*
FINAL (2008)	1,250	2,200	3,100	4,200	5,700	6,900	7,900	9,100
FINAL	**1,300**	**2,600**	**3,500**	**4,800**	**6,300**	**7,600**	**9,400**	**12,000**

Table 5-8: Peak Flow Discharge Values Update -- Continued								
Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.50%**	**0.20%**
JMP1 (Confluence of Manoa and Palolo Streams, A= 10.04 mi^2)								
HEC-HMS (R-73)	4,020	7,170	10,300	12,900	16,100	18,500	20,900	24,400
HEC-HMS (Atlas 14)	*5,400*	*8,620*	*11,400*	*14,300*	*18,600*	*21,900*	*25,700*	*31,200*
Regression (1994)	2,120	4,060	5,760	8,240	10,700	12,700	---	---
Regression (2010)	*1,760*	*3,400*	*4,930*	*7,020*	*8,890*	*11,000*	---	*17,700*
Plate 6	---	---	---	---	---	15,500	---	---
Weighted (2008)	3,210	5,840	8,360	10,900	13,800	15,800	20,900	24,400
Weighted (2016)	*3,840*	*6,380*	*8,630*	*11,200*	*14,400*	*16,800*	*25,700*	*25,400*
FINAL (2008)	3,350	6,000	8,400	10,900	14,100	16,500	18,700	21,800
FINAL	**3,400**	**6,200**	**8,400**	**11,000**	**14,200**	**16,700**	**20,500**	**22,900**
JMP2 (Manoa-Palolo Stream Gage 16247100, A= 10.34 mi^2)								
HEC-HMS (R-73)	4,090	7,340	10,500	13,000	16,300	18,700	21,100	24,700
HEC-HMS (Atlas 14)	*5,560*	*8,800*	*11,600*	*14,600*	*19,000*	*22,400*	*26,300*	*31,700*
Regression (1994)	2,110	4,080	5,800	8,320	10,800	12,900	---	---
Regression (2010)	*1,760*	*3,420*	*5,000*	*7,080*	*9,000*	*11,100*	---	*18,000*
Plate 6	---	---	---	---	---	16,000	---	---
FEMA	---	---	12,000	---	19,200	23,000	---	28,500
HEC-SSP	2,883	5,065	6,800	8,670	11,400	13,700	16,200	19,800
Weighted (2008)	3,070	5,520	8,470	9,890	13,900	16,400	18,100	23,200
Weighted (2016)	*3,390*	*5,750*	*8,470*	*9,970*	*13,800*	*16,700*	*19,700*	*22,690*
FINAL (2008)	3,400	6,100	8,500	11,150	14,400	16,800	19,000	22,100
FINAL	**3,600**	**6,400**	**8,600**	**11,200**	**14,500**	**17,000**	**20,900**	**23,200**
JMP3 (Right above the confluence of Manoa-Palolo and Ala Wai Canals, A=10.68 mi^2)								
HEC-HMS (R-73)	4,220	7,450	10,700	13,300	16,600	18,900	21,400	24,900
HEC-HMS (Atlas 14)	*5,760*	*9,130*	*12,000*	*15,100*	*19,700*	*23,100*	*27,100*	*32,700*
Regression (2010)	*1,780*	*3,470*	*5,050*	*7,200*	*9,150*	*11,300*	---	*18,400*
Plate 6	---	---	---	---	---	16,500	---	---
Weighted (2008)	4,220	7,450	10,660	13,260	16,560	18,100	21,400	24,900
Weighted (2016)	*4,050*	*6,700*	*9,020*	*11,700*	*15,200*	*17,700*	*27,100*	*26,600*
FINAL (2008)	3,450	6,200	8,700	11,400	14,700	17,100	19,300	22,400
FINAL	**4,000**	**6,700**	**9,000**	**11,700**	**15,000**	**17,500**	**21,500**	**24,000**
Ala Wai Canal (Mouth of Ala Wai Canal, A=16.22 mi^2)								
HEC-HMS (R-73)	6,000	10,100	13,400	15,200	16,700	17,700	18,700	20,500
HEC-HMS (Atlas 14)	*8,080*	*12,000*	*14,400*	*16,000*	*17,800*	*19,100*	*20,700*	*22,200*
Plate 6	---	---	---	---	---	22,500	---	---
FEMA	---	---	13,700	---	23,000	28,200	---	36,200
Weighted (2008)	6,000	10,100	13,500	15,200	19,400	22,300	18,700	27,200
Weighted (2016)	*8,080*	*12,000*	*14,100*	*16,000*	*20,000*	*22,900*	*20,700*	*28,200*
FINAL (2008)	6,000	9,500	12,500	15,200	17,500	18,500	19,500	20,500
FINAL	**8,000**	**11,500**	**13,500**	**16,000**	**18,000**	**19,500**	**20,500**	**22,000**

Table 5-8: Peak Flow Discharge Values Update -- Continued								
Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.50%**	**0.20%**
JMP1 (Confluence of Manoa and Palolo Streams, A= 10.04 mi^2)								
HEC-HMS (R-73)	4,020	7,170	10,300	12,900	16,100	18,500	20,900	24,400
HEC-HMS (Atlas 14)	*5,400*	*8,620*	*11,400*	*14,300*	*18,600*	*21,900*	*25,700*	*31,200*
Regression (1994)	2,120	4,060	5,760	8,240	10,700	12,700	---	---
Regression (2010)	*1,760*	*3,400*	*4,930*	*7,020*	*8,890*	*11,000*	---	*17,700*
Plate 6	---	---	---	---	---	15,500	---	---
Weighted (2008)	3,210	5,840	8,360	10,900	13,800	15,800	20,900	24,400
Weighted (2016)	*3,840*	*6,380*	*8,630*	*11,200*	*14,400*	*16,800*	*25,700*	*25,400*
FINAL (2008)	3,350	6,000	8,400	10,900	14,100	16,500	18,700	21,800
FINAL	**3,400**	**6,200**	**8,400**	**11,000**	**14,200**	**16,700**	**20,500**	**22,900**
JMP2 (Manoa-Palolo Stream Gage 16247100, A= 10.34 mi^2)								
HEC-HMS (R-73)	4,090	7,340	10,500	13,000	16,300	18,700	21,100	24,700
HEC-HMS (Atlas 14)	*5,560*	*8,800*	*11,600*	*14,600*	*19,000*	*22,400*	*26,300*	*31,700*
Regression (1994)	2,110	4,080	5,800	8,320	10,800	12,900	---	---
Regression (2010)	*1,760*	*3,420*	*5,000*	*7,080*	*9,000*	*11,100*	---	*18,000*
Plate 6	---	---	---	---	---	16,000	---	---
FEMA	---	---	12,000	---	19,200	23,000	---	28,500
HEC-SSP	2,883	5,065	6,800	8,670	11,400	13,700	16,200	19,800
Weighted (2008)	3,070	5,520	8,470	9,890	13,900	16,400	18,100	23,200
Weighted (2016)	*3,390*	*5,750*	*8,470*	*9,970*	*13,800*	*16,700*	*19,700*	*22,690*
FINAL (2008)	3,400	6,100	8,500	11,150	14,400	16,800	19,000	22,100
FINAL	**3,600**	**6,400**	**8,600**	**11,200**	**14,500**	**17,000**	**20,900**	**23,200**
JMP3 (Right above the confluence of Manoa-Palolo and Ala Wai Canals, A=10.68 mi^2)								
HEC-HMS (R-73)	4,220	7,450	10,700	13,300	16,600	18,900	21,400	24,900
HEC-HMS (Atlas 14)	*5,760*	*9,130*	*12,000*	*15,100*	*19,700*	*23,100*	*27,100*	*32,700*
Regression (2010)	*1,780*	*3,470*	*5,050*	*7,200*	*9,150*	*11,300*	---	*18,400*
Plate 6	---	---	---	---	---	16,500	---	---
Weighted (2008)	4,220	7,450	10,660	13,260	16,560	18,100	21,400	24,900
Weighted (2016)	*4,050*	*6,700*	*9,020*	*11,700*	*15,200*	*17,700*	*27,100*	*26,600*
FINAL (2008)	3,450	6,200	8,700	11,400	14,700	17,100	19,300	22,400
FINAL	**4,000**	**6,700**	**9,000**	**11,700**	**15,000**	**17,500**	**21,500**	**24,000**
Ala Wai Canal (Mouth of Ala Wai Canal, A=16.22 mi^2)								
HEC-HMS (R-73)	6,000	10,100	13,400	15,200	16,700	17,700	18,700	20,500
HEC-HMS (Atlas 14)	*8,080*	*12,000*	*14,400*	*16,000*	*17,800*	*19,100*	*20,700*	*22,200*
Plate 6	---	---	---	---	---	22,500	---	---
FEMA	---	---	13,700	---	23,000	28,200	---	36,200
Weighted (2008)	6,000	10,100	13,500	15,200	19,400	22,300	18,700	27,200
Weighted (2016)	*8,080*	*12,000*	*14,100*	*16,000*	*20,000*	*22,900*	*20,700*	*28,200*
FINAL (2008)	6,000	9,500	12,500	15,200	17,500	18,500	19,500	20,500
FINAL	**8,000**	**11,500**	**13,500**	**16,000**	**18,000**	**19,500**	**20,500**	**22,000**

Percent Chance Flood	USGS 2010 Equation	Coefficient of Determination R^2	Standard Error of Prediction in percent	Standard Model Error in percent
50	$Q_2 = 2.339\ (DA)^{0.679}\ P^{1.113}$	0.74	51	48
20	$Q_5 = 17.58\ (DA)^{0.668}\ P^{0.820}$	0.76	42	40
10	$Q_{10} = 49.09\ (DA)^{0.664}\ P^{0.674}$	0.77	40	38
4	$Q_{25} = 145.2\ (DA)^{0.657}\ P^{0.520}$	0.76	40	37
2	$Q_{50} = 290.4\ (DA)^{0.652}\ P^{0.422}$	0.76	40	37
1	$Q_{100} = 539.5\ (DA)^{0.646}\ P^{0.335}$	0.75	41	38
0.2	$Q_{500} = 1841\ (DA)^{0.633}\ P^{0.162}$	0.73	44	40

Table 5-9. Updated USGS 2010 Leeward O'ahu Regression Equations

HEC-HMS Model Junction		Peak Flow Discharge (cfs)							
		2	5	10	20	50	100	200	500
		50%	20%	10%	5%	2%	1%	0.50%	0.20%
Manoa Watershed Model	JM1	1,200	2,000	2,800	3,600	4,600	5,500	6,200	7,400
	JM2	1,700	3,200	4,600	5,700	7,150	8,200	9,000	10,500
	JM3	1,900	3,500	4,800	6,000	7,400	8,350	9,300	11,000
	JM4	2,000	3,700	5,000	6,300	7,700	8,800	9,900	11,700
	JM5	2,100	3,800	5,200	6,700	8,200	9,400	10,600	12,400
	JM6	2,300	4,100	5,700	7,200	9,000	10,200	11,800	13,600
	JM7	2,500	4,300	6,000	7,600	9,500	10,700	12,100	14,500
	JM8	2,600	4,450	6,150	7,800	9,700	11,200	12,500	15,300
Ala Wai Watershed Model	JK1	650	1,300	1,900	2,550	3,400	4,200	4,800	6,400
	JK2	700	1,400	2,000	2,700	3,500	4,500	5,600	6,700
	JK3	900	1,700	2,600	3,300	4,500	5,700	6,800	8,000
	JP1	440	820	1,200	1,600	2,100	2,700	3,000	3,900
	JP2	1,000	2,000	2,900	3,900	5,100	6,100	7,600	9,200
	JP3	1,200	2,200	3,100	4,100	5,500	6,600	8,000	10,000
	JP4	1,300	2,600	3,500	4,800	6,300	7,600	9,400	12,000
	JMP1	3,400	6,200	8,400	11,000	14,200	16,700	20,500	22,900
	JMP2	3,600	6,400	8,600	11,200	14,500	17,000	20,900	23,200
	JMP3	4,000	6,700	9,000	11,700	15,000	17,500	21,500	24,000
	Ala Wai	8,000	11,500	13,500	16,000	18,000	19,500	20,500	22,000

Table 5-10. Updated Peak Flow Discharges for the Ala Wai Watershed by HEC-HMS Model Junction

Table 5-11. Peak Flow Discharge Analytical Frequency Data for Without Project Intermediate Future based on HEC-RAS output discharges and Uncertainty in Equivalent Years of Record for use in HEC-FDA, Ala Wai Watershed, Oahu, Hawaii

Stream or Sub-Watershed	HEC-HMS Model Sub-Basin or Junction	HEC-RAS Reach Name	HEC-FDA Reach Name	HEC-FDA Analytical Frequency Curve Data (Log Units)			EYOR
				Mean	Std. Dev.	Skew	
Ala Wai, Waikiki	Ala Wai	Ala Wai Lower	ALA 1	3.8294	0.2534	-1.8690	30
		Ala Wai Middle	ALA 2	3.5409	0.2649	-0.3489	
		Ala Wai Upper	ALA 3	2.9837	0.1965	-0.5220	
Makiki	K2	Kanaha Ditch	KAH 1 KAH 2	2.4314	0.3201	-0.0040	18
	----	Kanaha Split	KAO 1	----	----	----	
	JK3	Makiki Lower	MAK 1	2.8525	0.2267	0.5912	
	JK2		MAK 2	2.7785	0.2470	0.4437	
	JK1	Makiki Upper	MAK 3	2.3610	0.4448	-0.4781	
	K1, K3		MAK 4	2.1310	0.4945	-0.5551	
Manoa	JM7, JM 8	Manoa Stream Main Reach	MAN 1	3.3937	0.3197	-0.3843	25
	JM 6		MAN 2	3.2937	0.3434	-0.5271	
	JM 4, JM 5		MAN 3 MAN 4	3.2428	0.3610	-0.6361	
	JM 3		MAN 5	3.1836	0.4043	-0.6881	
	JM 1, JM 2		MAN 6 MAN 7	3.0792	0.2842	-0.0140	
	----	UH_Split	UNI 1 UNI 2	----	----	----	18
Manoa-Palolo Canal	JMP 1 to JMP 3	Palolo Lower	MPC 1 MPC 2	3.5232	0.3127	-0.1657	30
Palolo	JP 4	Palolo Main	PAL 1	3.1106	0.3353	-0.0573	27
	JP 4	Palolo Main	PAL 2	3.0792	0.3183	-0.0158	27
	JP 3	Palolo Main	PAL 3 PAL 4	2.9809	0.3823	-0.3210	27
	Junction 2	Pukele Tributary	PUK 1	2.6725	0.3980	-0.6359	44
	Junction 1	Waiomao Ditch	WAI 1	2.7077	0.3936	-0.4793	35

EYOR = Equivalent Years of Record; ----, not a separate sub-basin in HEC-HMS model

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Ala Wai Canal Project
Feasibility Study
Honolulu, Hawaii

Existing Without-Project Hydraulic and With-Project Hydrologic and Hydraulic Appendix

Appendix A2

U.S. ARMY CORPS OF ENGINEERS
HONOLULU DISTRICT
FORT SHAFTER, HAWAII
March 2017

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1 INTRODUCTION

Through a cooperative effort undertaken by the State of Hawaii Department of Land and Natural Resources (DLNR) Engineering Division and the U.S. Army Corps of Engineers (USACE) as part of the Ala Wai Canal Project Feasibility Study, a hydrologic and hydraulic study of the Ala Wai Canal Watershed was initiated in 2001 and was amended in 2006. A large portion of this watershed is highly susceptible to flooding. The purpose of this study is to determine the feasibility of flood damage reduction alternatives for the Ala Wai Canal Watershed. This report presents a description of the analytical approach, analyses performed, and the results obtained for a detailed without-project hydraulic study and a with-project hydrologic and hydraulic study of the approximately 19 square miles of the Ala Wai Canal Watershed. Results of this study include water surface profiles for the 50%, 20%, 10%, 5%, 2%, 1%, 0.5%, and 0.2% Annual Chance Exceedance (ACE) storm events for the existing without-project conditions, future without-project conditions, and for several respective with-project alternatives.

2 GENERAL

2.1 Scope of Work

An analysis of the watershed and stream hydrology and hydraulics was performed using the U.S. Army Corps of Engineers' (HEC-HMS) Hydrologic Engineering Center's-Hydrologic Modeling System in conjunction with the (HEC-RAS) Hydrologic Engineering Center's-River Analysis System. The results of this modeling effort were used to develop depth-duration-frequency rating curves for each portion of the study. The watershed was first analyzed under current development conditions assuming no implementation of any flood damage reduction alternatives. These scenarios were then modified to include an initial array of five project alternatives aimed at reducing flood damages at different areas in the watershed. Three alternatives are described in this Appendix, Alternatives 1A, 2A, and 3A. 10% level of designs were created for Alternatives 2A and 3A.

The study area extends from the ridge of the Ko'olau Mountains to the nearshore waters of Mamala Bay and includes Makiki, Manoa, and Palolo streams. These streams all drain to the Ala Wai Canal, a 2-mile-long, man-made waterway constructed during the 1920s to drain extensive coastal wetlands, thus allowing development of the Waikiki district.

2.2 Previous Hydrologic and Hydraulic Models and Studies

Varieties of studies have been previously conducted in the Ala Wai Canal Watershed and were reviewed as part of this project. These studies include the following:

- **Federal Emergency Management Agency's National Flood Insurance Program Study (*FEMA, 1979*).** The Federal Emergency Management Agency (FEMA) contracted the U.S. Army Corps of Engineers to determine flood hazards for the McCully and Moiliili areas that encompass the Ala Wai Canal and the

Manoa-Palolo Drainage Canal. This study delineated the 1-percent ACE (100-year) floodplains and was completed in February 1979. The discharge of 28,300 cfs (cubic feet per second) at the canal mouth was used to delineate the 1-percent ACE floodplains.

- **Ala Wai Canal Improvement Project, Storm Water Capacity Study (*Edward K. Noda and Associates, 1994).*** The State of Hawaii contracted Edward K. Noda and Associates to conduct this study to determine the hydraulic effects associated with dredging the Ala Wai Canal. This study concluded that by lowering the canal invert elevation to -12.0 and -10.0 feet mean lower low water at strategic locations, the maximum 1-percent ACE (100-year) flood elevation would be at approximately 5.0 feet mean sea level near the top of the ocean side canal bank. The 1-percent ACE flow used in this study was 22,389 cfs at the mouth of the canal.
- **Ala Wai Flood Study (*USACE, 2001*).** Conducted under the Planning Assistance to States Program (Section 22, WRDA of 1974), this study investigated and recommended appropriate solutions to resolve flooding from the Ala Wai Canal. The Land Division of DLNR was the non-Federal sponsor of the study. The analysis demonstrated that there are possible structural measures that could be implemented to mitigate flooding by increasing the flood carrying capacity of the Canal. Specific measures included dredging, levees and floodwalls, and detention/sedimentation basins. The study indicated that dredging would increase the flood capacity of the channel, but would not provide full protection against a 1-percent ACE (100-year) flood.
- **Ala Wai Watershed Analysis (*Townscape Inc. and Dashiell, 2003*).** The purpose of this effort was to review existing literature and evaluate existing data to identify the water resource problems, studies, and recommended actions to improve watershed health, as related to water supply, flood control, and ecosystem restoration. This document was prepared as a component of the USACE/DLNR Ala Wai Watershed Feasibility Study.
- **Hydrology and Hydraulics Study, Flood of October 30, 2004, Mānoa Stream (*USACE, 2006*).** A storm on October 30, 2004, caused significant flooding in Mānoa Valley, especially in areas adjacent to Mānoa District Park and Woodlawn Drive Bridge. A post-flood analysis was conducted using rainfall-runoff and the 1977 stream hydraulic computer modeling (older HEC-2 model), the results of which were used to assess the feasibility of several short-term flood mitigation measures. Alternatives analyzed included construction of levees or floodwalls along sections of the channel between Mānoa District Park and Woodlawn Drive and installation of an artificial channel between East Mānoa Road and Woodlawn Drive. Of these alternatives, the channel drop structure at Woodlawn Drive Bridge was determined to have the best potential for increasing the capacity with the least amount of maintenance, aesthetic, bridge structure, and drainage impacts to mitigate. DLNR was the non-Federal sponsor of the study.
- **Final Hydrology Report, Manoa Watershed Project (*Oceanit, 2008a*).** The Natural Resources Conservation Service (NRCS) contracted Oceanit through the USACE to develop conceptual designs and prepare a feasibility report, watershed plan, and a preliminary draft environmental impact statement

(PDEIS) for alternate flood hazard reduction schemes aimed at preventing similar flooding in the future. Several rainfall runoff models and frequency-based methods were used to estimate the peak discharges at various junctions in the watershed. Rainfall runoff models included Technical Release 55 (TR-55) from the NRCS, Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) from the USACE, and FLO-2D, a distributed model. Frequency-based methods included the Plate 6 of the City and County of Honolulu drainage standards, USGS regression equations, and FEMA peak discharge-frequency drainage area curves. Peak discharges calculated using the above methods were compared, and best estimates of the peak discharges for the following return periods were determined: 50-, 20-, 10-, 5-, 2-, 1-, 0.5-, and 0.2-percent ACE.

- **Final Hydraulic Analysis Report, Manoa Watershed Project (*Oceanit, 2008b*).** The NRCS contracted Oceanit through the USACE to explore alternatives for flood reduction along the Manoa Stream corridor. In order to qualify the effects of each proposed alternative, the existing extent of flood inundation must be known. The Hydrologic Engineering Center-River Analysis System (HEC-RAS) was used to analyze the extent of the inundated areas from Manoa Stream for these eight storm events: 50-, 20-, 10-, 5-, 2-, 1-, 0.5-, and 0.2-percent ACE (2-, 5-, 10-, 20-, 50-, 100-, 200-, and 500-year). The 50-, 5-, 1-, and 0.2-percent ACE storms were mapped. Peak flow data at critical junctions along the stream were supplied by the Final Hydrology Report, Manoa Watershed Project (*Oceanit, 2008a*). The hydraulic analysis was conducted without debris blockages at bridge openings.
- **Technical Summary Report, Mānoa Watershed Project (*Oceanit, 2008c*).** Following the October 30, 2004, flooding event in Mānoa Valley new LiDAR data was obtained. NRCS initiated the Mānoa Watershed Project, which included development of hydrologic and hydraulic models using HEC-HMS and HEC-RAS, and design of conceptual flood reduction measures, based on the work completed as part of the Mānoa Stream Hydrology and Hydraulics Study (*USACE, 2006*). The intent of the project was to prepare a Watershed Plan and EIS under the NRCS Watershed Program (PL83-566). However, the funds needed to complete the EIS were not received, and thus the scope of the project was reduced to technical reports and conceptual measures to mitigate flooding. The results of this effort were eventually incorporated, with expansion of the hydrology and hydraulics modeling, into the Ala Wai Watershed Project.
- **Final Hydrology Report, Ala Wai Watershed Project (*Oceanit, 2008d).*** This study estimated peak flow discharges at particular drainage junctions in the Ala Wai Watershed corresponding to the following storm return periods: 50-, 20-, 10-, 5-, 2-, 1-, 0.5-, and 0.2-percent ACE (2-, 5-, 10-, 20-, 50-, 100-, 200-, and 500-year). Updates were done in November 2010 and March 2016 (Appendix A1).
- **Final Drainage Evaluation Report, Ala Wai Canal Watershed Project (*Oceanit, 2008e*).** This study evaluates the existing Ala Wai Watershed drainage facilities to determine the existing capacity of the drainage system. The existing discharge capacity is compared with the 2000 City and County of Honolulu's

Storm Drainage Standards to determine whether or not each drainage facility (mostly culverts can pass a 10-, 50-, or 100-year storm, depending on the drainage area serviced by the outlet.

- **Conceptual Engineering Report, Ala Wai Canal Flushing System & Ala Wai Golf Course Detention System (*Mitsunaga & Associates, Inc., 2014*).** The Department of Land and Natural Resources of the State of Hawaii proposes to improve the water quality of the Ala Wai Canal to standards acceptable for water recreational activities including canoeing, kayaking, fishing and minimal power boating. The specific objectives of the proposed project are to: decrease sources of pollution through detention ponds and/or filters on tributaries to the canal and improve watershed management, increase water flow and circulation in the canal while addressing environmental concerns, and define and implement maintenance management practices for the canal. This Conceptual Engineering Report developed alternatives to address the diversion of the off-site storm water through Ala Wai Golf Course and a flushing system or the Ala Wai Canal.

3 WITHOUT PROJECT HYDRAULIC MODELING

3.1 Overview

The HEC-RAS (Hydrologic Engineering Center - River Analysis System) computer program, version 4.1.0 was used for hydraulic modeling (*U.S. Army Corps of Engineers, 2010*). This HEC-RAS model was created by joining separate HEC-RAS models of Makiki, Manoa, and Palolo Streams and Manoa-Palolo Drainage and Ala Wai Canals together. The HEC-RAS model of Manoa Stream is documented in Oceanit (*2008b*) and the separate models for the Makiki and Palolo Streams and the Manoa-Palolo Drainage and Ala Wai Canals were originally created by Oceanit and West Consultants by July 2009 and then corrected and merged together by the U.S. Army Corps of Engineers by November 2009. In 2013 the merged model was then updated again to be more accurate. This model consisted of 8 rivers, 13 reaches, 1,287 cross sections, of which 476 are interpolated, 49 bridges (this includes culverts), 2 inline weirs, and 16 lateral weirs. The model was developed using data that was considered the best available at the time.

3.1.1 Study Reach Descriptions

The Makiki Stream portion of the HEC-RAS model starts from the confluence with the Ala Wai Canal to a point approximately 2.0 miles upstream and includes the Kanaha Ditch Tributary from its confluence with Makiki Stream to a point approximately 0.8 miles upstream. Due to the dominate effect of the Ala Wai Canal during high flows, the downstream reach of Makiki Stream downstream of Fern Street was not modeled in detail; both the Kapiolani Boulevard and pedestrian walkway bridges were ignored in the model. The stream channels of both Makiki and Kanaha Streams have been highly modified with concrete and confined from the point when they enter the urbanized area and include sections when the stream channel is confined to sections entirely underground.

The Manoa HEC-RAS model was modified from the Oceanit (*2008b*) by adding the potential for split flow to leave Manoa Stream near Woodlawn Drive Bridge and enter the University of Hawaii at Manoa Campus. Also the Dole Street Bridge was added to the model for completeness, although its effect on flow is very minimal as the low chord of the bridge is still about 10 ft higher than the 0.2-percent ACE flood water-surface elevation. The Manoa Stream reach extends from the confluence with Manoa-Palolo Drainage Canal upstream about 3.1 miles to the point where the Waihi and Waiakeakua Stream tributaries meet. The Manoa Stream channel is mostly natural with some segments modified by concrete channel or stream bank hardening.

The Palolo Stream Main reach extends upstream approximately 1.9 miles from the confluence with the Manoa-Palolo Drainage Canal. Upstream of Palolo Stream, the Pukele Stream and Waiomao Stream tributaries were modeled, extending 1.1 and 0.9 miles upstream. The Palolo Stream Main reach Channel is mostly confined to one large concrete channel.

The Manoa-Palolo Drainage Canal is called the Palolo Stream Lower reach in the model and extends about 0.9 miles from the Ala Wai Canal to the confluence of the Manoa and Palolo Stream. This canal consists of a modified channel with segments of natural bed and banks, but mostly hardened stream banks.

The Ala Wai Canal section is modeled using three reaches from the mouth of the to the Makiki Stream confluence (Lower), from Makiki Stream to the Manoa-Palolo Drainage Canal confluence (Middle), and from the Manoa-Palolo Drainage Canal confluence to the upstream end (Upper). The reach lengths are approximately 2490, 3365, and 4260 ft respectively. The Ala Wai Canal channel has a natural bottom with hardened banks and has tidal influence. Bathymetric data was collected by Oceanit in 2008 and used to compute the cross-section data for the canal.

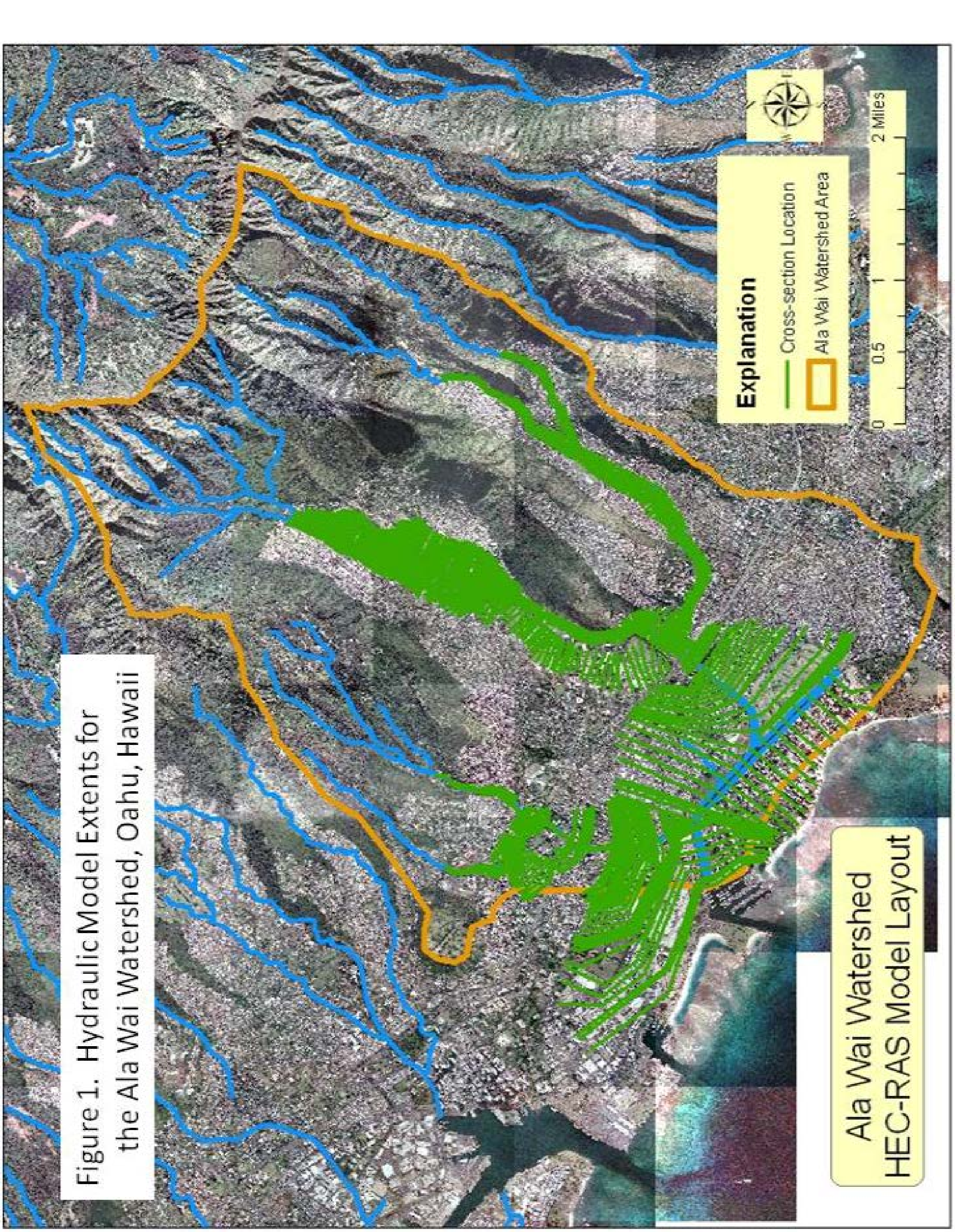

Figure 1: Hydraulic Model Extents for the Ala Wai Canal Watershed, Oahu, Hawaii

3.1.2 Terrain Data

Topographic data for the hydraulic model is primarily based on airborne light detection and ranging (LIDAR) data. The LIDAR data was collected, processed, and verified by Oceanit and their sub consultants in late 2006 and early 2007. The LIDAR data has an accuracy of 45 cm (1.5 ft) horizontal, 37 cm (1.2 ft) vertical and was processed with 1.4 m (5 ft) horizontal point spacing. Datum of data is NAD 1983 HARN projected into Stateplane_Hawaii_3_FIPS_5103_Feet horizontal and mean sea level, local tidal datum, vertical. The current approved National Geodetic Survey (NGS) National Spatial Reference System (NSRS) vertical datum for Oahu is Local Tidal Datum, mean sea level, tidal epoch 1983-2001, geoid 2012A. Horizontal datum is NAD 83 (PA 11). The bare ground LIDAR data was then converted into a Triangular Irregular Network (TIN) format using Geographical Information Systems (GIS) software. The TIN format is needed for using HEC-GeoRAS version 10.1 for ArcGIS version 10.1 to extract the necessary spatial and elevation data for the hydraulic model (*U.S. Army Corps of Engineers, 2011*). The HEC-RAS model schematic layout is illustrated in **Figure 2**.

Elevations used in the hydraulic model were extracted from the TIN. Further refinement to the extracted topographic data especially along and near the stream channels and in other critical areas with large amounts of vegetation overgrowth was done within the HEC-RAS program. Therefore, the Manning’s n-values were adjusted and refined where needed. At selected stream cross-section locations conventional land surveys or site investigations and field measurements were done to confirm channel inverts, top of bank locations, bridge dimensions, and other elements relevant to hydraulic modeling. This information was collected and originally entered into the HEC-RAS model by Oceanit and their sub-consultants. Most of the refinements to the cross-sections in the model were based on channel and bridge plans, especially in the Makiki and Palolo areas where the majority of the streams have been channelized. For the upper Manoa Stream area, where much of the stream channel is privately owned, cross-section adjustments were done based on field observations or measurements and not surveyed cross-sections. Cross-section data in the Ala Wai Canal were based on the bathymetric survey data collected in 2008.

3.1.3 Cross-Section Modeling

Cross-section locations for the HEC-RAS model were determined by the channel slope, channel shape, and location of structures. In general cross-sections were spaced about 100 to 500 feet apart for the Ala Wai and Manoa-Palolo Drainage Canals, about 25 feet apart for the Lower Makiki Stream and Palolo Stream Main reaches, and about 50 to 100 ft apart for all the other stream reaches. Near hydraulic structures, such as bridges and culverts, cross-sections were located closer together. Cross-sections in the area of stream confluences or junctions had to be modified to prevent the cross-section lines from crossing each other. These cross-sections were bent, or “doglegged” to insure the overbank areas were not double-counted or were purposely ignored in the case of Makiki Stream near the Ala Wai Canal. Makiki Stream at the mouth of the stream is heavily influenced from the water surface elevation of the Ala Wai Canal. This backwater effect completely overcomes the downstream cross-sections of Makiki

Stream and can give erroneous results if the cross-section length is cut short to avoid crossing any adjacent section lines. For this reason, it was decided to start the Makiki reach just upstream of Kapiolani Blvd. This provided a balance between the influence of the canal water surface elevation and the necessity to properly model more frequent flow events.

Ineffective flow areas were defined at cross-sections to separate areas of active conveyance from adjacent low lying areas that do not contribute to downstream conveyance due to either the presence of high ground along the reach or the expansion/contraction from another control upstream or downstream, such as a bridge or culvert opening. Levee stations were also used to confine flow to channels for lower flow rates especially where the ground elevations were lower than the top of channel. The contraction coefficients for majority of the cross-sections were 0.1, while 0.3 was used near bridges and culverts. The expansion coefficient was set to 0.3 for most cross-sections except near bridges, culverts or lateral structures where a value of 0.5 was used to account for the potential for greater losses.

An important component of hydraulic modeling is the selection of Manning's n-values (roughness coefficients). Manning's n-values were determined from previous studies and several site surveys conducted by Oceanit to characterize the channel roughness. Reasonable values are usually determined from site visits and the use of guides such as Barnes (*1967*) or Arcement (*1989*). Manning's n-values were further refined based on model calibration. Previous modeling on Manoa Stream (*Oceanit, 2008b*) was calibrated to the 2004 flood event on Manoa Stream. Previous modeling of the Ala Wai Canal (*U.S. Army Corps of Engineers, 2001 and 2005*) was calibrated to the limited data of the 1967 event on Palolo Stream and Manoa-Palolo Drainage Canal. Makiki and Kanaha Streams do not have any calibration or gaged data to aid in calibration or model comparison, but are mostly concrete lined channels so the n-values should be stable. Calibrated n-values are assumed to also account for any sediment or debris "bulking" during those storm events. Manning's n-values for the Ala Wai Canal Watershed HEC-RAS model are presented by modeled River and Reach in **Table 1**.

The range of Manning's n-values used can be roughly characterized by channel description. Natural stream channels with minimal vegetation in the channel, steep banks, trees and brush on banks, and bottoms consisting of gravels, cobbles, and few large boulders were given values from 0.03 to 0.04. Natural channel sections that were uniform and contained smooth graveled beds were given a value of 0.025. The Ala Wai Canal was given an n-value of 0.03. Lined or concrete channels were given a value of 0.018 and smooth overbank areas in parks or the golf course were given values of 0.06. The majority of overbank areas in mixed urban areas or overland flow (split flow) areas were given values of 0.1 to 0.125. Previous channel n-values used were 0.04 for the natural channels with 0.06 for all overbank areas (*M&E Pacific, 1977*) to 0.027 to 0.04 for the Ala Wai Canal and 0.25 for the urban overbank areas (*U.S. Army Corps of Engineers, 2005*).

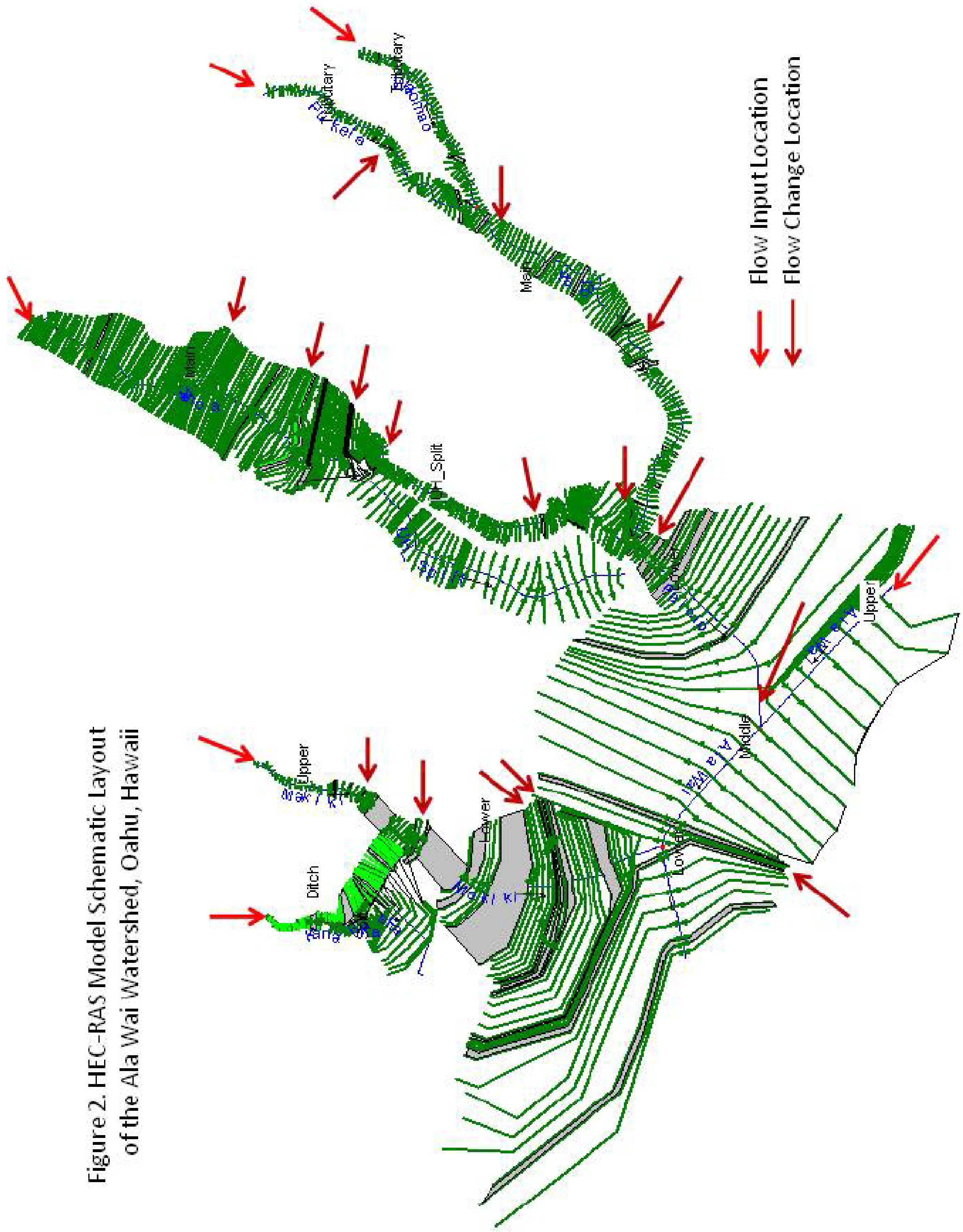

Figure 2: HEC-RAS Model Schematic Layout of the Ala Wai Canal Watershed, Oahu, Hawaii

Table 1. Manning's n-values for roughness used in the Ala Wai Canal Watershed HEC-RAS Model, Honolulu, Hawaii

Table 1. **Manning's n-values for roughness used in the Ala Wai Watershed HEC-RAS Model, Honolulu, Hawaii**						
Model Stream Names		**Model Cross-section Location Range**		**Channel Section**		
				Left Overbank	**Main Channel**	**Right Overbank**
River	**Reach**	**From**	**To**			
Ala Wai	Upper	6370	9724	0.125	0.03	0.125
Ala Wai	Middle	2580	5825	0.125	0.03	0.06 to 0.125
Ala Wai	Lower	33	2324	0.125	0.03	0.125
Kahana	Ditch	3238	4372	0.125	0.04	0.125
Kahana	Ditch	3	3084	0.125	0.025	0.125
Kahana	Split	809	3508	0.125	0.1	0.125
Makiki	Upper	6319	10768	0.125	0.025	0.125
Makiki	Lower	5952	6286	0.125	0.025	0.125
Makiki	Lower	2632	4725	0.125	0.025 to 0.06	0.125
Makiki	Lower	639	2560	0.125	0.025	0.125
Manoa	Main	10592	16506	0.125	0.04	0.125
Manoa	Main	9792	10566	0.125	0.025	0.125
Manoa	Main	8775	9691	0.125	0.018	0.125
Manoa	Main	7734	8699	0.125	0.025	0.125
Manoa	Main	7190	7653	0.125	0.03	0.125
Manoa	Main	6129	7149	0.125	0.035	0.125
Manoa	Main	84	6054	0.125	0.04	0.125
Palolo	Main	8951	15326	0.125	0.018	0.125
Palolo	Main	7102	8838	0.125	0.03	0.125
Palolo	Main	5303	7000	0.125	0.018	0.125
Palolo	Lower	1813	5198	0.125	0.025	0.125
Palolo	Lower	859	1356	0.125	0.025	0.06 to 0.125
Pukele	Tributary	2004	5958	0.125	0.04	0.125
Pukele	Tributary	146	1893	0.125	0.018	0.125
UH_Split	UH_Split	132	6929	0.125	0.125	0.125
Waiomao	Tributary	514	4900	0.125	0.04	0.125
Waiomao	Tributary	110	414	0.125	0.018	0.125

3.1.4 Bridge and Culvert Modeling

Geometric data (culvert diameters and dimensions, culvert length, bridge span, etc.) for all bridges and culverts was obtained by Oceanit and entered into the HEC-RAS model. In many cases, like along Palolo Stream, bridges over concrete channels do not constrict the flow until the flow overtops the banks and thus, have minimal impact to most of the smaller flow frequency results. In other cases, like Woodlawn Drive Bridge in Manoa, a significant constriction occurs at the smaller flow frequency results. The HEC-RAS model results take these factors into account. A list of bridges and culverts are presented in **Table 2**.

For the Ala Wai Canal Watershed HEC-RAS model all bridges and culverts were modeled using the energy method for low flow if no bridge piers were present. Where bridges had piers, the energy and momentum method was selected and the highest energy answer was then used in the HEC-RAS model for the resulting computations. For the momentum method, the coefficient of drag for the piers was 1.20 at all piers except 1.60 was used at East Manoa Road and Waialae Avenue Bridges, 2.0 was used at the double box culvert at 10th Avenue. For situations where the water surface elevation reaches the low chord of the bridge, the pressure and weir method was selected at all bridges except for those bridges crossing the Ala Wai Canal, Manoa-Palolo Drainage Canal, and Dole Street Bridge on Manoa Stream which used the energy only method due to the flat slopes of these reaches or in the case of Dole Street, where the low chord was 10 feet higher than the 0.2-percent ACE flood elevation.

For determining blocked bridge potential from debris, both from large boulders or floating vegetation, the type of bridge, bridge location, and historical performance was used to determine the percent blockage which was used for all flow modeling. In general, concrete lined channels with supercritical flow tend to wash debris downstream quickly and maintain a “self-cleaning” condition. Most urban debris and trash is small in size with bicycles being the largest observed. Such sized debris has a low potential to create a blockage. In small steep channels, large vegetative debris also has low potential for downstream movement as such debris gets trapped or lodged across the channel and only after being broken up by the force of water will smaller pieces begin to be transported downstream.

The first bridge or culvert in the HEC-RAS model below the forest reserve or undeveloped areas was given a 25% reduction in open area blockage to represent the potential for sediment or debris to constrict these openings. Other percent reductions used were 15% and 45% (see **Table 2**). Two bridges in Manoa, East Manoa Road and Woodlawn Drive have had serious debris problems during the 2004 flood event so were given blocked areas equivalent to those determined from that event. All blockages were modeled in the HEC-RAS model by creating obstructions to a height from the channel bed to an elevation that represents the percent reduction in area. This was done by culvert blockage routines for culverts or cross-section obstructions for bridges in the HEC-RAS model.

Table 2. Bridge and Culvert Location and Information used in the Ala Wai Canal Watershed HEC-RAS Model, Honolulu, Hawaii

Table 2. **Bridge and Culvert Location and Information used in the Ala Wai Watershed HEC-RAS Model, Honolulu, Hawaii**						
Model Stream Names		**Model Cross-section Location**	**Name of Crossing**	**Type of Structure**	**Bed Material under Crossing**	**Percent Blockage used for Debris**
River	**Reach**					
Ala Wai	Middle	3020	McCully St	Bridge w/piers	Natural	----
Ala Wai	Lower	2280	Kalakaua Ave	Bridge w/piers	Natural	----
Ala Wai	Lower	370	Ala Moana Blvd	Bridge w/piers	Natural	----
Kanaha	Ditch	3168	Nehoa St	Box Culvert	Concrete	25
Kanaha	Ditch	2346	Lewalani Dr	Box Culvert	Concrete	----
Kanaha	Ditch	1713	Liholiho St	Bridge	Concrete	----
Kanaha	Ditch	1533	Private Dr upstream of Kewalo	Bridge	Concrete	----
Kanaha	Ditch	1386	Kewalo St	Bridge	Concrete	----
Kanaha	Ditch	798	Keeamoku St	Bridge	Concrete	----
Kanaha	Ditch	237	Makiki St	Arch Culvert	Concrete	----
Kanaha	Ditch	102	Underground section just upstream of Makiki Stream confluence	Bridge	Concrete	----
Makiki	Upper	8575	Makiki St	Bridge	Concrete	25
Makiki	Upper	8290	Oneele Pl	Bridge	Natural	----
Makiki	Upper	7974	Private Dr just downstream of Oneele Pl	Bridge	Natural	----
Makiki	Upper	7330	Nehoa St and underground section	Box Culvert	Concrete	----
Makiki	Lower	6272	Anapuni St	Box Culvert	Concrete	----
Makiki	Lower	6029	Wilder St	Bridge	Concrete	----
Makiki	Lower	5337	Underground Section just downstream of Wilder St	Box Culvert	Concrete	----
Makiki	Lower	4544	Wooden Deck Bridge downstream of underground section	Bridge	Natural	----
Makiki	Lower	3639	Underground Section at S. Beretania St	Box Culvert	Concrete	10
Makiki	Lower	2547	Private Drive just upstream of Phillip St	Bridge	Natural	----
Makiki	Lower	2435	Phillip St	Bridge	Concrete	----

Table 2. **Bridge and Culvert Location and Information used in the Ala Wai Watershed HEC-RAS Model, Honolulu, Hawaii ---- Continued**

Model Stream Names		Model Cross-section Location	Name of Crossing	Type of Structure	Bed Material under Crossing	Percent Blockage used for Debris
River	Reach					
Makiki	Lower	2228	Covered section just downstream of Phillip St	Bridge	Concrete	----
Makiki	Lower	1311	Fern St	Bridge	Concrete	----
Manoa	Main	14686	Pawaina St	Bridge	Natural	----
Manoa	Main	10461	Kahaloa Dr	Bridge	Concrete	----
Manoa	Main	9273	Lowrey Ave	Bridge	Concrete	----
Manoa	Main	8749	East Manoa Rd	Bridge w/piers	Concrete	25
Manoa	Main	7756	Woodlawn Dr	Bridge	Natural	45
Manoa	Main	2520	Dole St	Bridge w/piers	Natural	----
Palolo	Main	15248	Kiwila Rd	Box Culvert	Concrete	----
Palolo	Main	13489	Kahlua Rd	Bridge	Concrete	----
Palolo	Main	12621	Paalea Ave	Box Culvert	Concrete	----
Palolo	Main	11053	Private Rd	Bridge	Concrete	----
Palolo	Main	10863	Private Rd	Bridge	Concrete	----
Palolo	Main	9873	Palolo Ave	Arch Culvert	Concrete	----
Palolo	Main	7515	3rd Ave	Arch Culvert	Concrete	----
Palolo	Main	6578	St Louis Dr	Arch Bridge	Concrete	----
Palolo	Main	6059	Footbridge	Bridge	Concrete	----
Palolo	Main	5343	Koali Rd	Bridge	Concrete	----
Palolo	Lower	5110	Waialae Ave	Bridge w/piers	Natural	----
Palolo	Lower	4732	H-1 Highway	Bridge w/piers	Natural	----
Palolo	Lower	4501	South King St	Bridge w/piers	Natural	----
Palolo	Lower	4031	Kapiolani Blvd	Bridge w/piers	Natural	----
Palolo	Lower	2317	Date St	Bridge	Natural	----
Pukele	Tributary	3172	Palolo Ave/10th Ave	Box Culvert	Natural	25
Pukele	Tributary	355	Ahe St	Box Culvert	Concrete	----
Waiomao	Tributary	1924	10th Pl	Box Culvert	Concrete	25
Waiomao	Tributary	1268	10th Ave	Double Box Culverts	Concrete	10

3.1.5 Peak Flow Data

Peak flow data from the Manoa and Ala Wai hydrologic studies (*Oceanit, 2008a and 2008d*) were used after adjusting these peak flow values with updated rainfall-frequency data at the hydrologic model junctions, see Appendix A1. The peak flow frequency data was then adjusted from the hydrologic model junctions to corresponding cross-section locations in the HEC-RAS model where the flow would enter the stream channel in order to capture the change in flow that would occur during each of the frequency based events. There were areas where flow left the main stream channel and followed a new path downhill. To account for these split flow areas, lateral weirs were set to enter the split flow reach at specific stream stations. The HEC-RAS program calculates the weir flow leaving the stream and into the split flow "stream" channel. In the flow file the initial flow for the split flow reaches were set to 1 cubic foot per second (cfs) and the split flow optimization algorithm was used to balance the amount of flow being diverted depending on the value of the peak flow frequency. The peak-flow values used and the input model locations are presented in **Table 3**. The 1-percent ACE peak flow for the Ala Wai Canal in this model was 19,500 cfs. Previous estimates of the 1-percent ACE peak flow value at the mouth of the Ala Wai Canal have ranged from 22,900 cfs (*Edward K. Noda and Associates, Inc., 1994*) to 28,200 cfs (*Federal Emergency Management Agency, 2004*).

3.1.6 Split Flow Assumptions

During the development of the Ala Wai Canal Watershed HEC-RAS model, potential locations where peak-flow discharges leave the defined stream channels and do not return were identified by model results, previous models (U.S. Army Corps of Engineers, 2005) or from knowledge of previous flood events. These locations were modeled in HEC-RAS through the use of lateral weirs (structures) and the split-flow optimization routine in steady flow models. In sections of the model where overtopping flow would move parallel to the stream channel or downhill, no lateral weirs were used. Two locations, along Kanaha and Manoa Streams, were modeled as "stream reaches" to account for the floodplain impacts of these flows and at Waikiki the flow was allowed to leave the model to go into the ocean. The use of split-flow optimization reduces the flow in the stream channel downstream of the lateral weir location if peak-flows leave the channel.

Kanaha Ditch is a cross slope man-made drainage channel that carries runoff to Makiki Stream. Between Nehoa Street and Lewalani Drive flood waters in the model overtop the right bank (looking downstream) and would tend to flow away from the ditch to the south (Figure 3).

A number of lateral weirs were created in the HEC-RAS model along the right bank of the Kanaha Ditch Reach. Those weirs were set at the top of the right bank elevations of the ditch and the overflow was assigned to specific cross-sections in the split-flow reach where it was presumed to flow. Weir flow coefficients were set to 1.0. For all lateral weirs in the model, the weir flow coefficient was determined depending on the round roughness conditions near those areas which would represent the most likely overflow conditions. The Kanaha Split reach extends about 2,600 ft down slope past Wilder Avenue to the H-1 Freeway area. Elevation data provided indicates that the flood extent would not cross the freeway. The model of this split flow reach does not account for the collection of the overtopping flows to be collected by the local storm drain system.

Table 3. Input Peak Flow Discharges in cubic feet per second for Ala Wai Canal Watershed HEC-RAS for Existing Conditions, Honolulu, Hawaii

Model Stream		Model Input Cross-section Location	Percent ACE Flood							
River	Reach		50%	20%	10%	5%	2%	1%	0.5%	0.2%
Ala Wai	Upper	9724	1,000	1,400	1,800	2,300	3,040	3600	4320	5300
Ala Wai	Middle	5825	3,600	6,400	8,600	11,200	14,500	17,000	20,900	23,200
Ala Wai	Lower	2324	8,000	11,500	13,500	16,000	18,000	19,500	20,500	22,000
Kanaha	Ditch	4372	270	500	700	930	1,240	1,500	1,800	2,200
Kanaha	Split	3508	1.01	1.02	1.03	1.04	1.05	1.06	1.07	1.08
Makiki	Upper	10768	250	500	800	1,100	1,400	1,700	2,000	2,400
Makiki	Upper	7674	240	490	770	1,040	1,290	1,510	1,720	1,960
Makiki	Lower	6286	650	1,300	1,900	2,550	3,400	4,200	4,800	6,400
Makiki	Lower	3189	700	1,400	2,000	2,700	3,500	4,500	5,600	6,700
Makiki	Lower	1465	900	1,700	2,600	3,300	4,500	5,700	6,800	8,000
Manoa	Main	16506	1,200	2,000	2,800	3,600	4,600	5,500	6,200	7,400
Manoa	Main	10968	1,700	3,200	4,600	5,700	7,150	8,200	9,000	10,500
Manoa	Main	9274	1,900	3,500	4,800	6,000	7,400	8,350	9,300	11,000
Manoa	Main	7839	2,000	3,700	5,000	6,300	7,700	8,800	9,900	11,700
Manoa	Main	6175	2,100	3,800	5,200	6,700	8,200	9,400	10,600	12,400
Manoa	Main	2477	2,300	4,100	5,700	7,200	9,000	10,200	11,800	13,600
Manoa	Main	1807	2,500	4,300	6,000	7,600	9,500	10,700	12,100	14,500
Manoa	Main	1230	2,600	4,450	6,150	7,800	9,700	11,200	12,500	15,300
Palolo	Main	15526	1,000	2,000	2,900	3,900	5,100	6,100	7,600	9,200
Palolo	Main	9520	1,200	2,200	3,100	4,100	5,500	6,600	8,000	10,000
Palolo	Main	7552	1,300	2,600	3,500	4,800	6,300	7,600	9,400	12,000
Palolo	Lower	5198	3,400	6,200	8,400	11,000	14,200	16,700	20,500	22,900
Pukele	Tributary	5958	440	820	1,200	1600	2,100	2,700	3,000	3,900
Pukele	Tributary	3629	560	1,130	1,710	2,220	2,940	3,500	4,100	5,400
UH_Split	UH_Split	6929	1.01	1.02	1.03	1.04	1.05	1.06	1.07	1.08
Waioma	Tributary	4900	550	1,050	1,540	1,960	2,550	3,000	3,500	4,700

The Manoa Stream split flow reach is called the UH Split reach in the model. This reach was added along Manoa Stream up- and downstream of Woodlawn Drive Bridge. Lateral weirs were added along the right bank of Manoa Stream to account for the overtopping flows which inundated the University of Hawaii campus in 2004. Weir flow coefficients were set at 2.0. The UH Split reach extends for about 6,900 ft from Woodlawn Drive to the lower campus quarry area where it is assumed that the flow would pond and not flow back into Manoa Stream or to the Manoa-Palolo Drainage Canal. Again, in the model this split-flow reach does not account for the collection of the overtopping flows in the reach to be collected by the local storm drain systems.

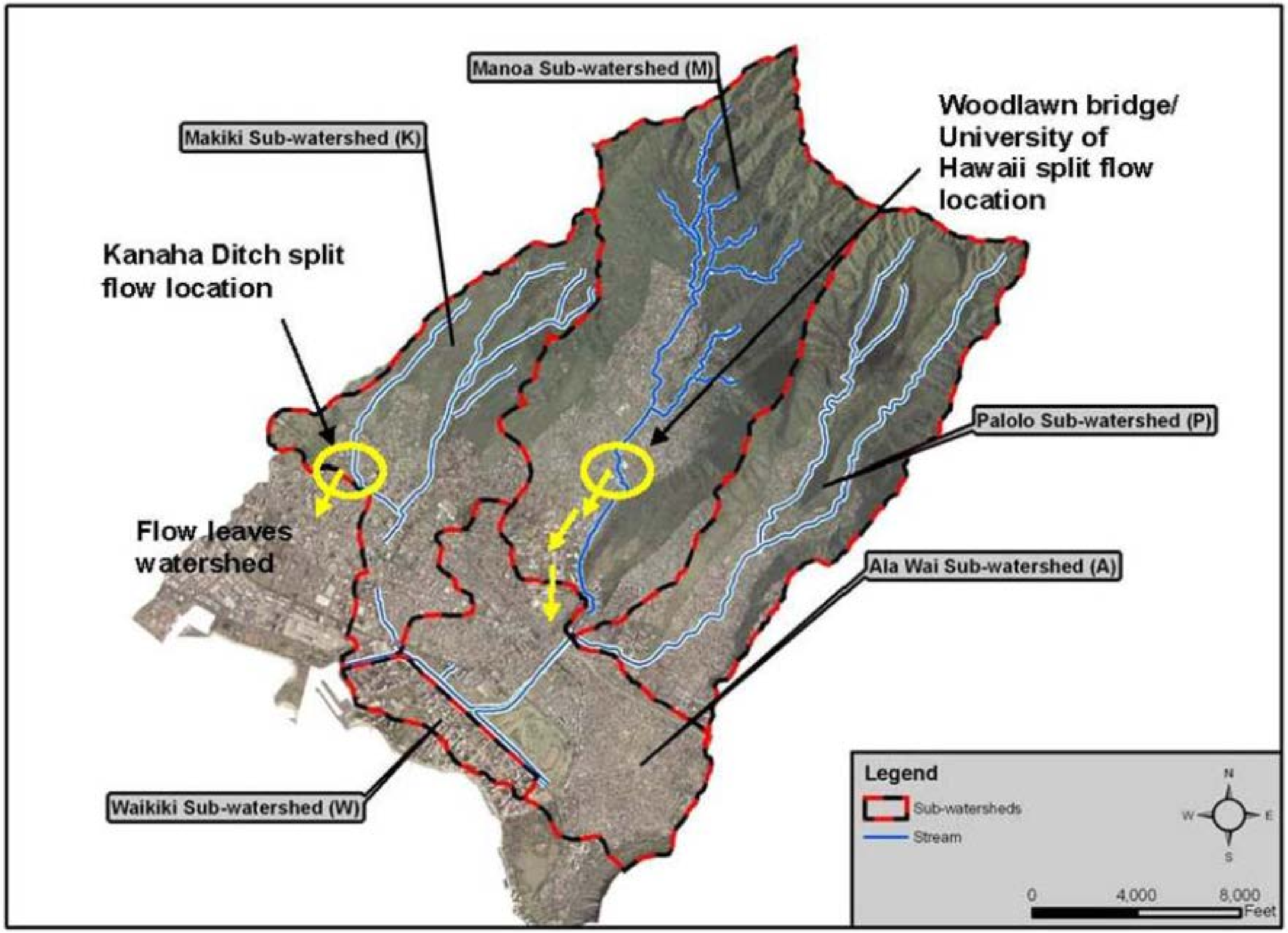

Figure 3: Location of the Split Flow Reaches in the Ala Wai Canal Watershed, Oahu, Hawaii

Along the left overbank of the Ala Wai Canal, from its upstream end down to about McCully Street, flow overtops the “ridge-line” along Waikiki and leaves the model by entering the ocean. This overtopping of the natural topography begins to occur at approximately elevation 6 feet. The flow leaving the system effectively reduces the discharge in the lower end of the canal. To account for the overtopping, lateral weirs were inserted into the model along the Middle and Upper Ala Wai Canal Reaches. Since these weirs are located in an urban environment, weir flow is influenced by the proximity of buildings, automobiles, etc. A weir coefficient of 1.0 was used here. The lateral weirs were set to allow the flow to leave the system (model).

The table below shows all of the lateral structures, the 10-, 2-, 1-, and 0.2-percent (10-, 50-, 100-, and 500-Year) ACE events are shown. The flow upstream (Q US), total flow leaving the structure (Q Leaving Total), and the downstream flow (Q DS) are shown in this table.

Table 4. Lateral Structure Output Table in cubic feet per second for Without-Project Ala Wai Canal Watershed HEC-RAS Model, Honolulu, Hawaii

River	**Reach**	**River Sta**	**Profile**	**Q US**	**Q Leaving Total**	**Q DS**
				(cfs)	**(cfs)**	**(cfs)**
Pukele	Tributary	236	10 YR	1710	0	1710
Pukele	Tributary	236	50 YR	2940	0	2940
Pukele	Tributary	236	100 YR	3500	30	3470
Pukele	Tributary	236	500 YR	5400	310	5090
Manoa	Main	7946	10 YR	4800	0	4800
Manoa	Main	7946	50 YR	7400	10	7690
Manoa	Main	7946	100 YR	8350	26	8773
Manoa	Main	7946	500 YR	11000	174	11526
Manoa	Main	7706	10 YR	5000	0	5000
Manoa	Main	7706	50 YR	7690	74	7616
Manoa	Main	7706	100 YR	8773	146	8627
Manoa	Main	7706	500 YR	11526	400	11126
Manoa	Main	1821	10 YR	5700	0	6150
Manoa	Main	1821	50 YR	8916	0	9616
Manoa	Main	1821	100 YR	10027	0	11027
Manoa	Main	1821	500 YR	13025	31	14694

Table 4. Lateral Structure Output Table in cubic feet per second for Without-Project Ala Wai Canal Watershed HEC-RAS Model, Honolulu, Hawaii (cont.)						
River	Reach	River Sta	Profile	Q US	Q Leaving Total	Q DS
				(cfs)	(cfs)	(cfs)
Kanaha	Ditch	3000	10 YR	700	177	521
Kanaha	Ditch	3000	50 YR	1240	537	703
Kanaha	Ditch	3000	100 YR	1500	729	771
Kanaha	Ditch	3000	500 YR	2200	1258	942
Kanaha	Ditch	2770	10 YR	521	121	399
Kanaha	Ditch	2770	50 YR	703	320	382
Kanaha	Ditch	2770	100 YR	771	423	348
Kanaha	Ditch	2770	500 YR	942	768	174
Kanaha	Ditch	2150	10 YR	399	0	399
Kanaha	Ditch	2150	50 YR	382	0	382
Kanaha	Ditch	2150	100 YR	348	45	303
Kanaha	Ditch	2150	500 YR	174	1709	2
Kanaha	Ditch	1660	10 YR	399	13	386
Kanaha	Ditch	1660	50 YR	382	92	291
Kanaha	Ditch	1660	100 YR	303	251	52
Kanaha	Ditch	1660	500 YR	2	907	2
Kanaha	Ditch	1500	10 YR	386	61	237
Kanaha	Ditch	1500	50 YR	291	303	1
Kanaha	Ditch	1500	100 YR	52	616	2
Kanaha	Ditch	1500	500 YR	2	1371	2
Kanaha	Ditch	1300	10 YR	327	40	286
Kanaha	Ditch	1300	50 YR	1	759	1
Kanaha	Ditch	1300	100 YR	2	1365	2
Kanaha	Ditch	1300	500 YR	2	2791	2
Kanaha	Ditch	1170	10 YR	286	87	206
Kanaha	Ditch	1170	50 YR	1	1076	1
Kanaha	Ditch	1170	100 YR	2	1889	2
Kanaha	Ditch	1170	500 YR	2	3783	2

Table 4. Lateral Structure Output Table in cubic feet per second for Without-Project Ala Wai Canal Watershed HEC-RAS Model, Honolulu, Hawaii (cont.)						
River	Reach	River Sta	Profile	Q US	Q Leaving Total	Q DS
				(cfs)	(cfs)	(cfs)
Kanaha	Ditch	990	10 YR	206	69	142
Kanaha	Ditch	990	50 YR	1	743	1
Kanaha	Ditch	990	100 YR	2	1291	2
Kanaha	Ditch	990	500 YR	2	2564	2
Kanaha	Ditch	700	10 YR	142	16	129
Kanaha	Ditch	700	50 YR	1	946	1
Kanaha	Ditch	700	100 YR	2	1802	2
Kanaha	Ditch	700	500 YR	2	3853	2
Kanaha	Ditch	500	10 YR	129	0	129
Kanaha	Ditch	500	50 YR	1	402	1
Kanaha	Ditch	500	100 YR	2	1056	2
Kanaha	Ditch	500	500 YR	2	2776	2
Ala Wai	Upper	9720	10 YR	1800	517	1284
Ala Wai	Upper	9720	50 YR	3040	2072	968
Ala Wai	Upper	9720	100 YR	3600	2798	802
Ala Wai	Upper	9720	500 YR	5300	5219	81
Ala Wai	Middle	5800	10 YR	8084	636	7450
Ala Wai	Middle	5800	50 YR	12344	1964	10380
Ala Wai	Middle	5800	100 YR	14029	2354	11675
Ala Wai	Middle	5800	500 YR	17375	3608	13767

3.1.7 Boundary Conditions

Since the Ala Wai Canal Watershed HEC-RAS model is a steady state hydraulic model, only peak flow data and boundary conditions for each event to be modeled were required. In a steady flow model, peak-flow hydrographs traveling from one stream to the next are assumed to occur peak to peak. Boundary conditions for upstream junctions or tributary streams are based on the model results from the downstream reach. The downstream boundary condition for the entire model is the mouth of the Ala Wai Canal where the starting water-surface elevation of 1.08 feet which is the Mean Higher High Water (MHHW) level at the Honolulu Harbor tide gage. The starting water surface elevation at the canal mouth was changed to incorporate sea-level rise in the future and the inter-annual variability of the tidal data of 0.4 feet. The computed sea-level rise values in Appendix A3 were added to the MHHW water-surface elevation for the modeling of the sea-level rise scenarios (Table 5). The starting water-surface elevation for the Kanaha Split reach was set to normal depth (slope=0.0171). The starting water-surface elevation for the UH Split reach was set to normal depth (slope=0.0001). The starting water-surface elevations for the remaining streams were determined as a result of the hydraulic calculations of the stream junctions. All junctions were set to compute water-surface elevations using the Energy Method.

Table 5. Starting Backwater Values in Feet at the Ala Wai Canal Mouth in the HEC-RAS Model for the Ala Wai Canal Watershed, Oahu, Hawaii

Year	**Condition**	**Low Sea Level Rise**	**Intermediate Sea Level Rise**	**High Sea Level Rise**
2025	Base	1.64	1.74	2.05
2075	Future/Design	1.89	2.50	4.44
2125	Future	2.14	3.71	8.69

4 WITHOUT PROJECT MODEL RESULTS AND FLOOD INUNDATION MAPPING

The HEC-RAS model was used to determine the water-surface elevations and floodplain extents for the 50-, 20-, 10-, 5-, 2-, 1-, 0.5-, and 0.2-percent ACE floods (2-5-, 10-, 20-, 50-, 100-, 200-, and 500-year recurrence intervals). Floodplain extents for the 10-, 50-, 100-, and 500-year recurrence intervals are shown on **Figures 4** through **7**. The water-surface elevation data for the 8 flood events are used in the HEC-FDA (Flood Damage Analysis) program to determine flood damages for the economic analyses. The HEC-FDA program uses the water-surface elevations in determining flood damages and not the flood maps, so any irregularities in the presented maps has no impact on the damage calculations.

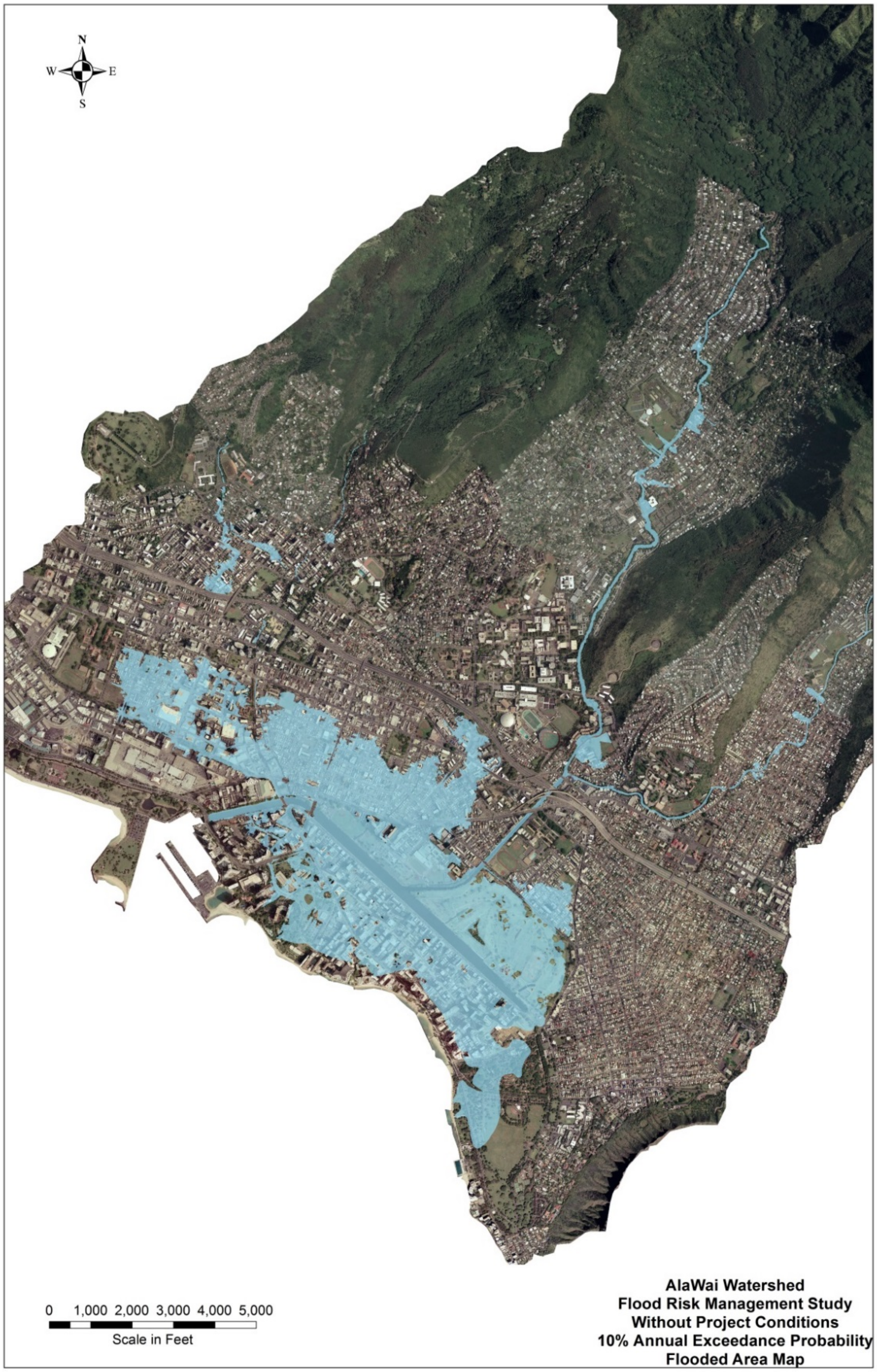

Figure 4: Floodplain Outlines for the 10-Percent ACE (10-year) Flood, Ala Wai Canal Watershed, Oahu, Hawaii.

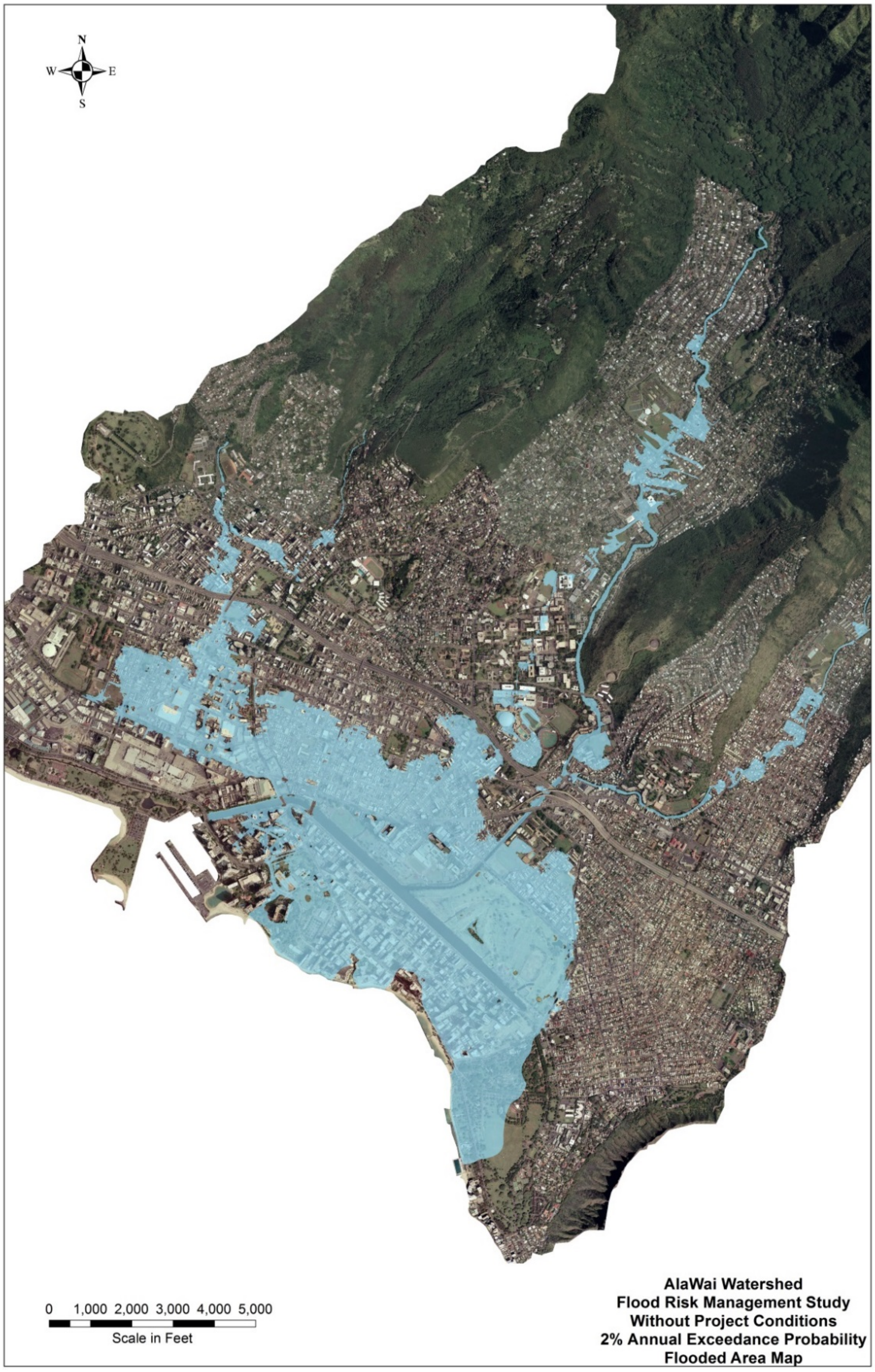

Figure 5: Floodplain Outlines for the 2-Percent ACE (50-year) Flood, Ala Wai Canal Watershed, Oahu, Hawaii.

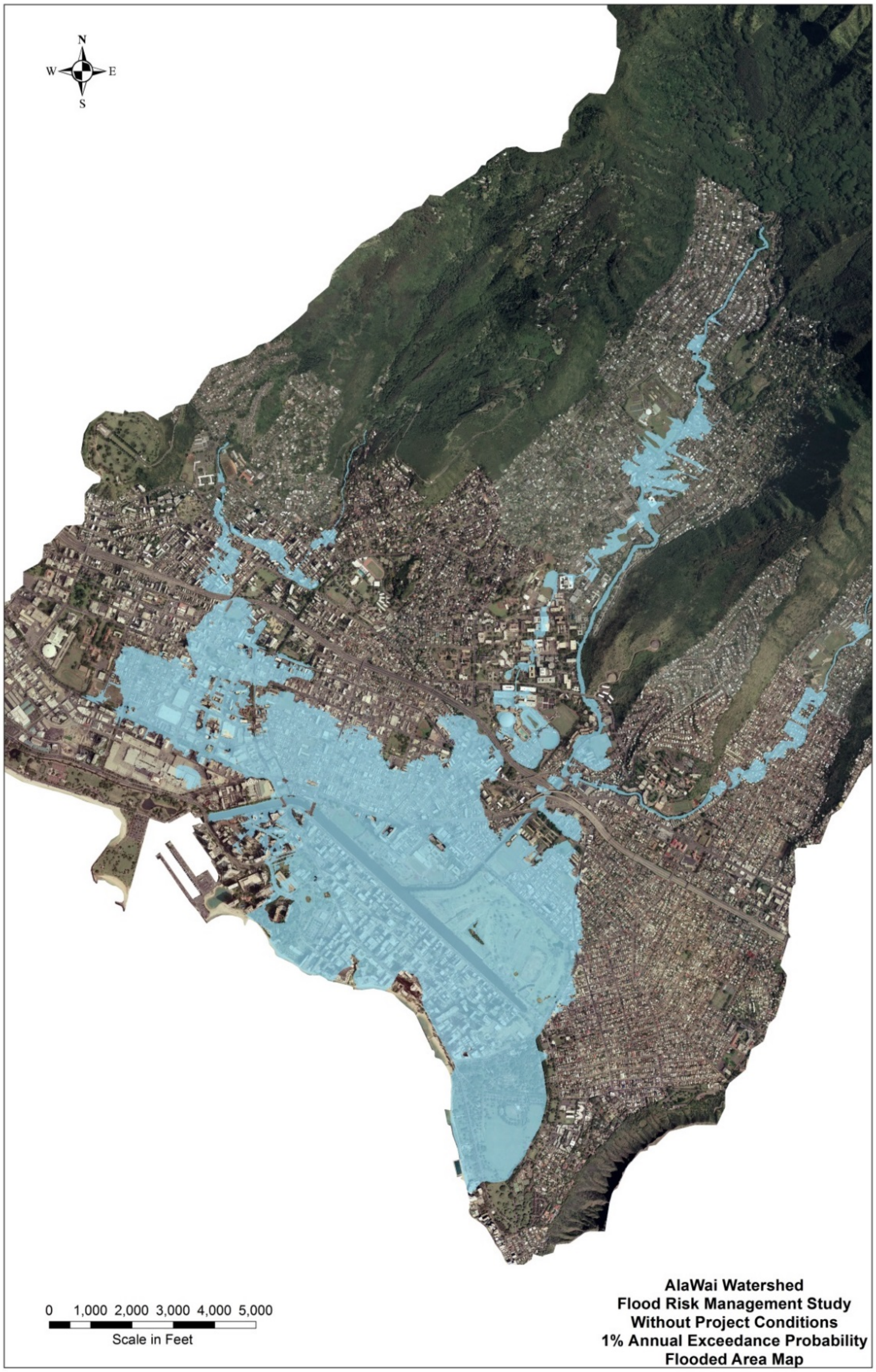

Figure 6: Floodplain Outlines for the 1-Percent ACE (100-year) Flood, Ala Wai Canal Watershed, Oahu, Hawaii.

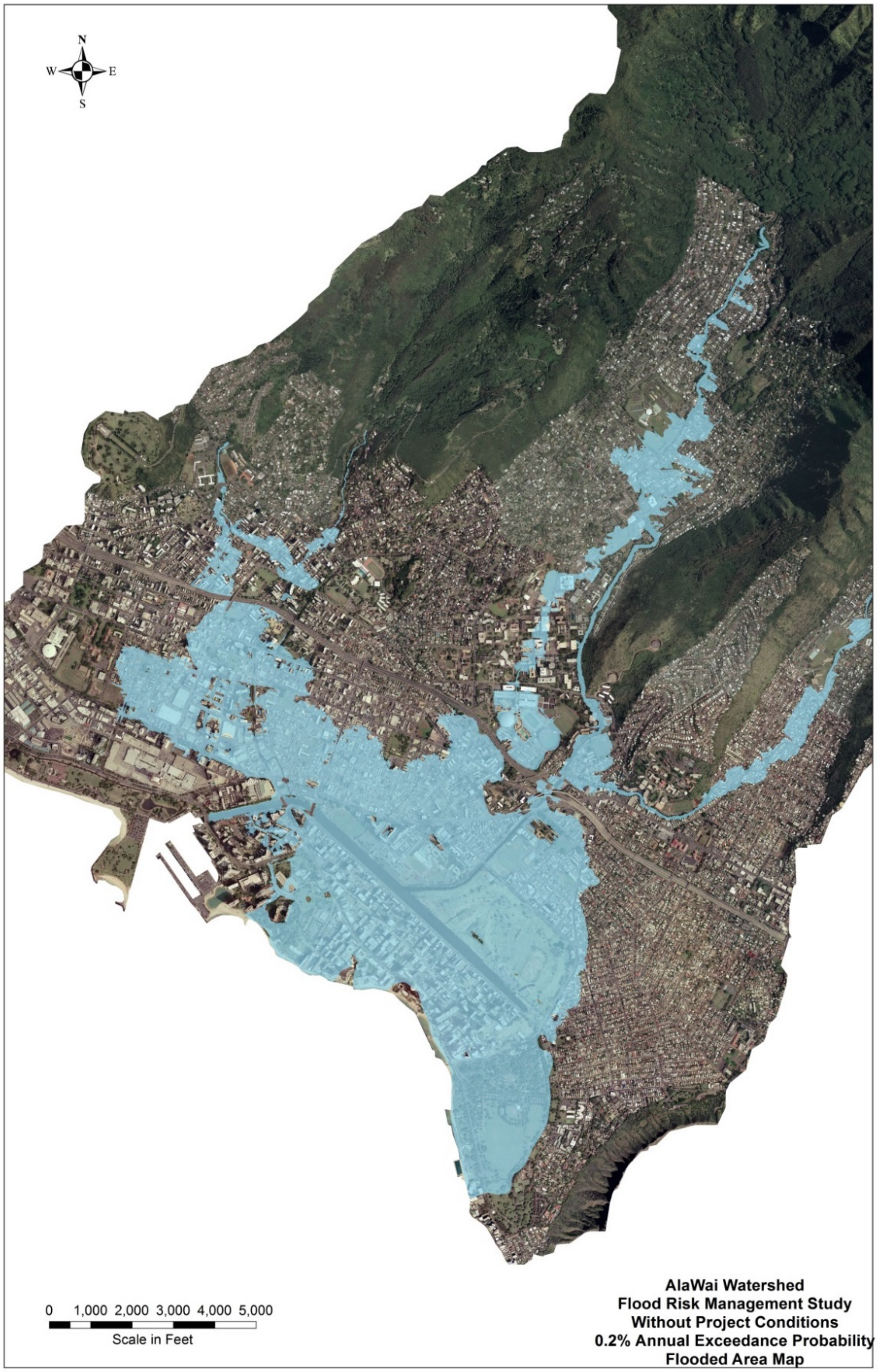

Figure 7: Floodplain Outlines for the 0.2-Percent ACE (500-year) Flood, Ala Wai Canal Watershed, Oahu, Hawaii.

4.1 Model Limitations

The HEC-RAS model was designed as a steady flow model using a mixed flow regime solution. As such, a number of warning messages appeared in several reaches indicating an unbalanced solution at cross sections resulting in critical depth or non-convergence. In steep reaches this message typically indicates the model is defaulting to critical depth because a super-critical flow answer is possible. The subcritical regime provides a conservative estimate of water-surface elevations for evaluating flooding but a mixed flow solution is more accurate. A mixed flow regime uses both subcritical and supercritical regimes. In less steep reaches, the primary reason for warning messages is probably due to the sharp contrast in Manning's n values between the channel at 0.018 to 0.04 and the overbanks at 0.125. The convergence problem seems to be most pronounced at bridges and culverts where the bridge deck and presence of weirs complicate the model solution process. Slight adjustments to ineffective flow limits helped reduce the non-convergences, but there are some locations where the messages could not be avoided for all storm events. One such location is the Makiki Stream between Anapuni Street and Wilder Street (cross sections 6316 to 5952).

Other warning messages such as conveyance ratios exceeding the 0.7 to 1.4 guidelines, velocity head differences exceeding 0.5 feet (ft) and energy losses greater than 1.0 ft between cross sections are due to the cross section geometry of the study area and cannot be avoided. Cross-section spacing can help with the steady flow solution. Average cross-section spacing for the various reaches in the Ala Wai Canal Watershed HEC-RAS model is presented in **Table 6**. In general, cross-section spacing from 50 to 100 ft in steep reaches and from 250 to 750 ft in flat reaches is adequate. In concrete channels such as Makiki Lower and Palolo Main reaches cross-sections were spaced at least 25 ft apart, this is to gain accuracy.

The use of split flow reaches to model areas where there is no flow under normal conditions are difficult to model accurately. The HEC-RAS model as with all hydraulic models, require a flow value to be used for each reach. Zero flow is not an allowable input value. For the Kanaha and UH Split reaches a flow value of 1 cfs was used as the initial input. Even with just a 1 cfs flow there will be a water-surface elevation, when in some cases there should be no water at all. For the 50- to 10-percent ACE floods, when flow is not leaving the main channel, the model results would indicate flood depths, although very low, when none would be present. This is a model artifact which needs to be accounted for when making flood inundation maps or using the data in HEC-FDA. For HEC-FDA, the water-surface elevation input data was changed to ground elevation at certain locations and flood events based on logic.

Split flow optimization was used to allow HEC-RAS to calculate lower flows in the main channel in locations where lateral weirs allow the flow to leave the main channel. Because there are multiple lateral weirs in the model, and in some locations multiple connection, such as the golf course reach, split flow optimization failed to converge. If split flow optimization is not used, results can be more conservative.

Table 6. Cross-section Spacing in the Ala Wai Canal Watershed HEC-RAS Model, Oahu, Hawaii

HEC-RAS River, and Reach Names	Average Cross-section Spacing in Feet
Ala Wai Canal, Upper, Middle, and Lower	286
Kanaha, Ditch (includes interpolated)	47
Kanaha, Split	87
Makiki Stream, Upper	107
Makiki Stream, Lower	31
Manoa Stream, Main	61
Palolo Stream, Main	96
Palolo Stream, Lower (includes Manoa-Palolo Drainage Canal)	180
Pukele Stream, Tributary	97
UH_Split, UH_Split	160
Waiomao Stream, Tributary	90

4.2 Flood Inundation Mapping

The flood inundation maps in **Figures 4** to **7** were generated by the HEC-GeoRAS software using TIN (Triangulated Irregular Network) elevation data and the maximum water-surface elevation profiles computed by HEC-RAS. A raster cell size of 5 feet was used to create the inundation outlines. HEC-GeoRAS converts the TIN elevation data and the maximum water-surface elevation data to raster layers with a 5 foot by 5 foot grid size before comparing them to one another. HEC-GeoRAS evaluates whether the water-surface elevation grid has a higher elevation than the ground-surface elevation grid. If the water-surface elevation was higher than the ground-surface elevation, the cell was considered inundated. The results were in raster datasets of the inundation depths. The inundation depth grid was then converted to a floodplain polygon coverage showing the maximum extents of flooding. The automated delineation process creates areas of no inundation, as the example figure, **Figure 8** shows areas or polygons where inundation is included or not included. As you can see in the figure there are some locations where inundation is not connected to the main inundation extents.

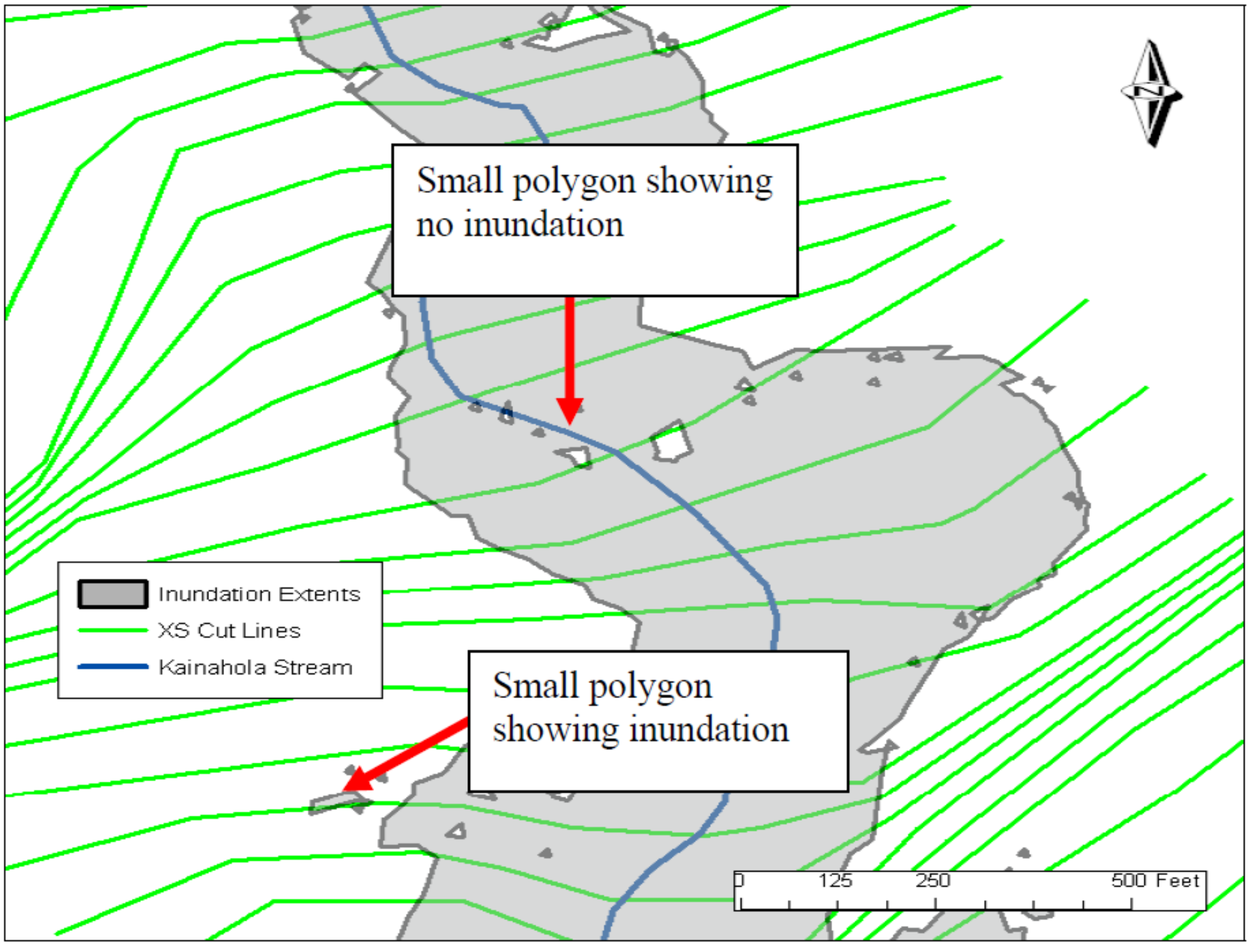

Figure 8: Example of inundation extents coverage created by HEC-GeoRAS.

Some non-connected areas of inundation, as described above, are an initial limitation of the mapping process because the computed water-surface is limited to the extents of the cross-sections; however, final mapping results should involve engineering judgment to modify the floodplain boundaries based on modeling assumptions and topographic data. If floodplain mapping is needed from the model results, then manual edits were done to either fill-in or remove areas where flooding is or is not likely to occur. An example of **Figure 8** with manual edits is shown on **Figure 9**.

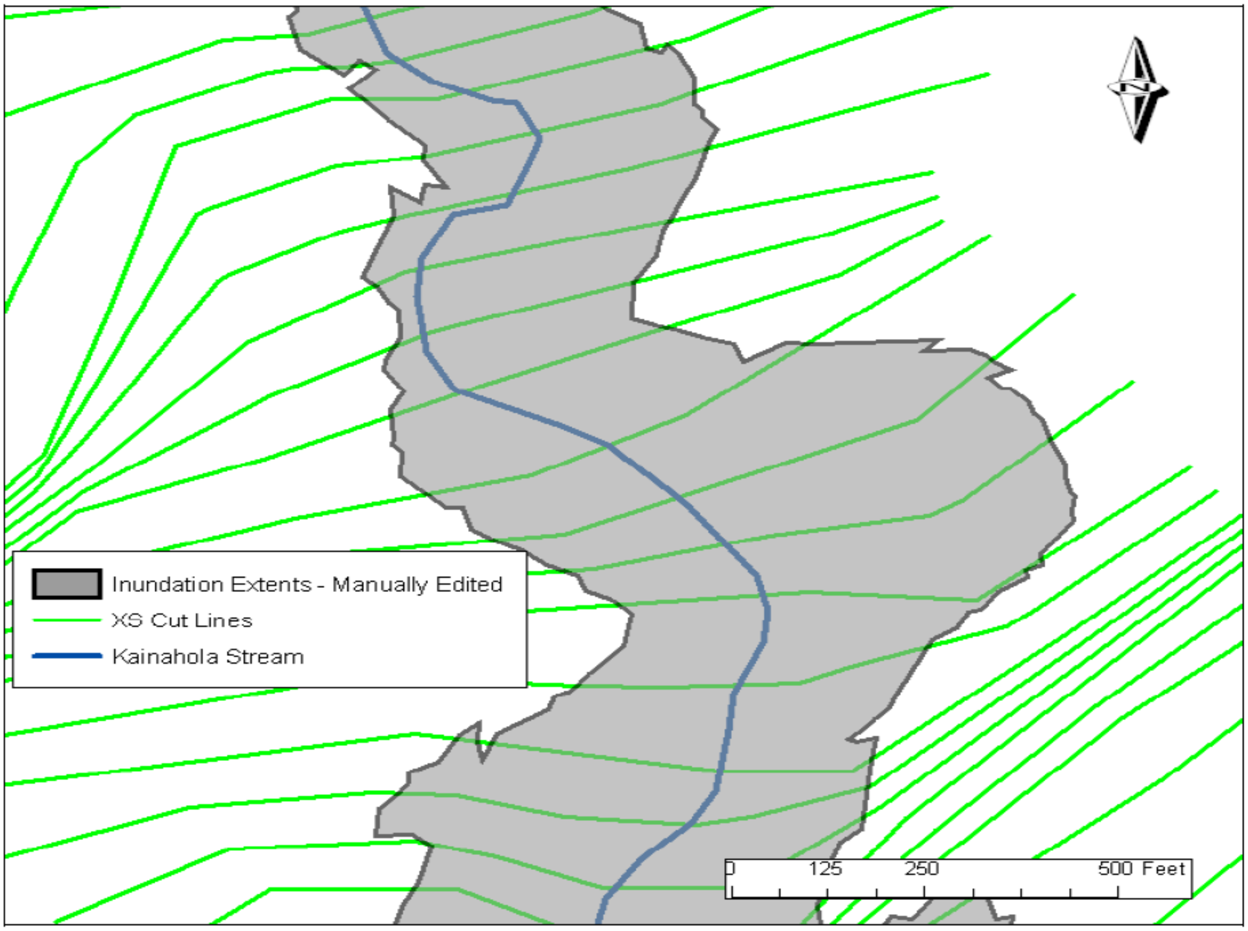

Figure 9: Example of inundation extents coverage manually edited.

4.3 Model Results

The inclusion of the lateral weirs along the Waikiki area of the Ala Wai Canal reaches resulted in a reduction in peak flow downstream of McCully Street at the 1-percent ACE (100-year) flood event; a total of approximately 5,150 cfs leaves the system and flows into the ocean. With consideration of the effects of floodplain storage and backwater along Makiki Stream, the peak flow at the mouth of the canal is reduced to about 12,675 cfs from its upstream peak of 19,500 cfs. This results in a greatly reduced flood inundation area between Kalakaua Ave. and Ala Moana Blvd. Based on the peak flow values computed for this study the Ala Wai Canal has about a 20- to 10-percent ACE (5- to 10-year) flood event capacity before overtopping. This is less than the 10-percent ACE (10-year) flood event capacity documented in Edward K. Noda and Associates, Inc. (*1994*) even with the dredging done in 2008. One reason for a reduced capacity may be due to the use of MHHW as a downstream boundary condition for all flood events and the use of a steady flow HEC-RAS model which tends to be more conservative than the in-house model used by Edward K. Noda and Associates, Inc. (*1994*). The Kalakaua Ave. Bridge was the main reason for high water-surface elevations in the upstream sections of the canal.

The flooding along lower Makiki Stream is mainly due to the high water-surface elevations of the Ala Wai Canal being such that the Makiki Stream cannot drain. Water backs up into Makiki and overtops the channel and floods the surrounding area. The flooding occurs between King St. and Kapiolani Blvd. The elevations along Kapiolani Blvd. are slightly higher than the surrounding ground, and as a result, act as a berm or weir preventing most of the flow from flooding the downstream area. A small area in the vicinity of Kaheka St. allows flow to flow over Kapiolani Blvd. and eventually into the lower area of the parking facility of the Ala Moana Center. Channel capacities for the model reaches are listed in **Table 7**. The split reaches don't have any capacities because they are not actual streams they are just the natural ground. The flooding inundation extents for lower Makiki Stream on **Figures 4** to **7** are somewhat overestimated due to the small existing channel sizes and large floodplain areas modeled. The flood mapping routines will fill in the entire cross-section width even though water may not actually flow into those areas for other factors such as walls and buildings which are not always representative in the model.

Table 7. Approximate Average Bankfull Channel Capacities and Beginning Level of Damages by Annual Probability for Stream Reaches in the HEC-RAS Model for the Ala Wai Canal Watershed, Oahu, Hawaii

River	Reach	Average Bankfull Peak Discharge Capacity (cfs)	Percent Chance Flood	Recurrence Interval (years)
Ala Wai	Lower	12,200	20	5
Ala Wai	Middle	6,900	20	5
Ala Wai	Upper	1,300	20	5
Kanaha	Ditch	350	20	5
Kanaha	Split	N/A	N/A	N/A
Makiki	Upper	1,200	5	20
Manoa	Main	3,500 to 7,600	20 to 2	5 to 50
Palolo	Main	3,400 to 6,000	10 to 2	10 to 50
Palolo	Lower	15,400	2	50
Pukele	Tributary	2,700	2	50
UH_Split	UH_Split	N/A	N/A	N/A
Waiomao	Tributary	2,600	2	50

The flooding along Kanaha Ditch is similar to the flooding along Makiki Stream. The water-surface elevation at the confluence of the ditch causes a backwater effect in the ditch and does not allow it to drain. Water overtops the channel and flows down slope. Water flows across Wilder St. and floods the area approximately bounded by Kewalo St. and Keeamoku St. Water is stopped by the H1 Freeway where it will pond and presumably make its way into the stormwater drainage systems.

Another issue with the flood modeling in the Makiki area is the discharge contained in the stream channel where floodwalls exist. These floodwalls are built upon the stream channel walls (**Photo 1**) and may or may not prove suitable to contain large flood events. It was assumed in the HEC-RAS model that none of these types of levee-like structures would fail during any of the model runs. **Photo 2** shows that some locations may not prove adequate and the resulting flood inundation areas may be larger than shown on **Figures 4** to **7**.

As can be seen for the flood inundation maps (**Figures 4** to **7**), the floodplain boundaries for the lower Makiki Stream, the Manoa-Palolo Drainage Canal, and the Ala Wai Canal are shown in some areas as reaching the watershed boundaries. This result could be overly conservative if the hydrograph volume is insufficient to produce such flooding. Also, the flood inundation limits reach the model boundary such as at the upper end of the Ala Wai Canal. Flooding at these locations will more than likely flow into Kapiolani Park and then into the ocean. A manually edited map should account for this possibility.

Results of the detailed modeling of Palolo Stream indicate a channel capacity capable of holding a 2-percent ACE (50-year) flood event between Palolo Ave. Bridge and Kahlua Rd. Bridge. The capacity downstream is between the 5- to 2-percent ACE (20- to 50-year) flood events.

As previously documented (*U.S. Army Corps of Engineers, 2005; Oceanit, 2008b*) the Manoa Stream has channel capacity limitations between Kahaloa Drive Bridge and Woodlawn Drive Bridge which creates a flooding hazard for the nearby residences and the University of Hawaii campus.

Flood depths for the 1-percent ACE flood event around the Ala Wai Canal, Manoa-Palolo Drainage Canal, and lower Makiki Streams (**Figure 6**) are about 1.5 to 3 feet deep on average for the out of channel floodplain. Flood depths are about 2 to 3 feet deep on average for the split flow reaches of Kanaha Split and the UH_Split overland flooding. In the upper Makiki, Manoa and Palolo Streams, flood depths can get up to 5 feet depending on the location.

As a check to existing conditions HEC-RAS model computations, a comparison was made at results near USGS gaging stations located in the study area. One difficulty is that the gages are operated using gage datum which does not correspond to a known datum such as mean sea level, and thus, to make this comparison, an approximation of gage datum converted to HEC-RAS model datum, mean sea level, was made based on information supplied by the USGS at their gage locations and the elevation data of the nearest RAS model cross-section. It is expected that this conversion has a 1 foot error and that an additional difference of +/- 0.7 foot would result from model error and USGS gage rating curve error. **Table 8** shows this comparison. The discharge value comparison is based on a common discharge values from the station rating curves and the HEC-RAS model results at that location.

Photo 1. Makiki Stream looking upstream of Fern Street, November 2009

Photo 2. Makiki Stream looking upstream of Fern Street, April 2006 Flood damage to CMU wall built on channel wall from March 31, 2006 storm. Photo by Oceanit.

Table 8. Comparison of Existing Conditions HEC-RAS Model Results with Data from USGS Stream Gaging Stations, Ala Wai Canal Watershed, Oahu, Hawaii.

USGS Gage Station Number	HEC-RAS Model Reach	HEC-RAS cross-section Location	Comparison Discharge Value in ft^3/s	Gage Water Surface Elevation converted to Model Elevation (ft)	HEC-RAS Model Water-Surface Elevation (ft)
16242500	Manoa Main	1230	4,450	36.4 or 38.8	38.07
16247000	Palolo Main	9520	1,200	96.2	95.72
16247100	Palolo Lower	3201	10,910	11.9	12.93

As can be seen from comparison, water surface elevation results fall within the expected error of +/- 1.7 feet. At station 16242500, two water station elevations were computed for the conversion as the data points for the conversion was more vague that at the other two locations.

4.4 Sensitivity and Uncertainty

To test the sensitivity of the model to variation in Manning's "n" values, model runs were made varying the "n" values by +/- 20%. Results of the model runs show that for the upper reaches of Makiki, Manoa, Palolo, Pukele, Waiamao, Kanaha, there was virtually no difference in water surface elevation. This is due to the fact that these reaches are flowing at critical depth or in the supercritical flow regime. The lower reaches of of Makiki and the Manoa-Palolo canal, the difference in water surface was about 0.25 ft. In the UH_split reach, the difference in water surface elevation varied from +/- 0.2 ft. to a maximum of +/- 0.4 ft. On the alaWai canal, the difference ranged from +/- 0.2 feet in the upper reach to +/- 0.4 ft. in the lower reach.

These values were much lower than the minimum values specified in Engineer Manual 1110-2-1619, <u>Risk-Based Analysis for Flood Damage Reduction Studies</u>, (EM 1110-2-1619) provides guidance in determining risk and uncertainty. Table 5-2 of EM 1110-2-1619 provides minimum values for the standard deviation of error in stage. This table is reproduced below as **Table 9**.

Table 9. Minimum Standard Deviation of Error in Stage

Manning's n Value Reliability[1]	Cross Section Based on Field Survey Or Aerial Spot Elevation	Cross Section Based on Topographic Map with 2-5' Contours
Good	0.3	0.6
Fair	0.7	0.9
Poor	1.3	1.5

1- Where good reliability of Manning's n value equates to excellent to very good model adjustment/validation to a stream gage, a set of high water marks in the project effective size range, and other data. Fair reliability relates to fair to good model adjustment/validation for which some, but limited, high water mark data is available. Poor reliability equates to poor model adjustment/validation or essentially no data exists for model adjustment/validation.

As stated in Section 3.1.2, elevation data used in the hydraulic model was obtained from LIDAR data collected in late 2006 and early 2007. The data has an accuracy of 1.5 feet in the horizontal plane and 1.2 feet in the vertical plane. The elevations were processed with a 5 foot point spacing. Spot elevations were developed to a 0.1 foot accuracy.

Manning's "n" values were estimated from field observations and engineering judgement. These values were refined based on model calibration as described in Section 4.3. Due to the limited data available for calibration, the reliability of the Manning's "n" values used can be considered fair.

Using **Table 9** as a guide, the standard deviation was set to 0.7 feet. This factor was applied to all water surface elevations calculated during the model runs.

5 WITH PROJECT HYDROLOGIC MODELING

5.1 Detention Analysis

In order to determine the effectiveness of detention basins throughout the watershed, the HEC-HMS model, the Technical Summary Report, Manoa Watershed Project (*Oceanit, 2008c*), and other hydrologic analysis were used to study different detention basin scenarios.

5.2 Preliminary Analysis

Initially, 12 different sites were selected throughout the watershed. **Figure 10** shows the locations of the proposed detention basins. Each detention basin was designed to maximize effectiveness while remaining within reasonable vertical and horizontal limitations. Each basin was examined to determine the potential flow reduction. Basins were analyzed for all eight storm events.

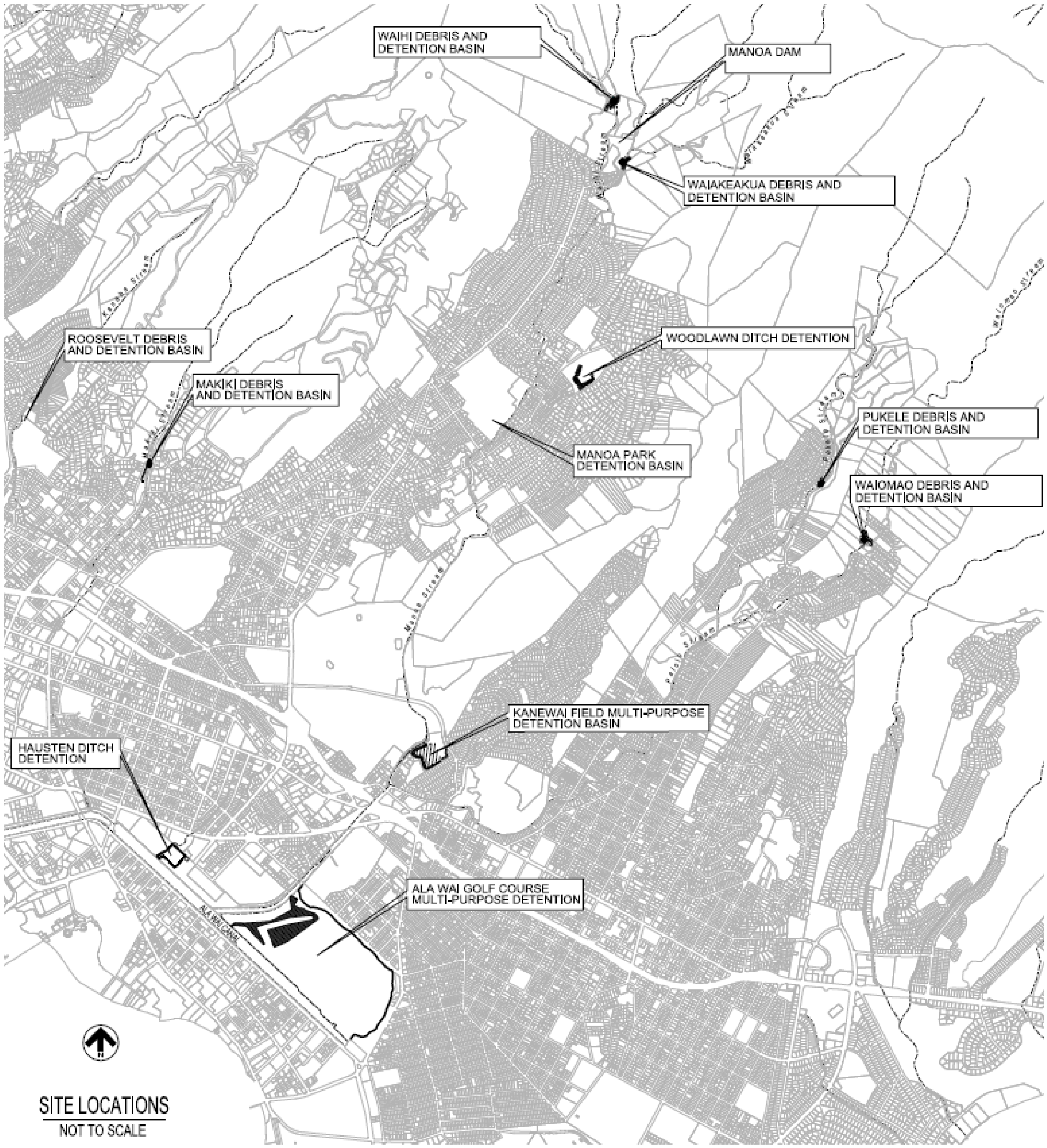

Figure 10: Preliminary Detention Basin Locations

Roosevelt Debris and Detention Basin

This basin is designed as both a way to temporarily contain water and stop the flow of debris into Kanaha Stream. The design assumes a downstream capacity of 1,254 cfs and 20 foot high berms with an emergency spillway. The basin is not designed to permanently contain water, but to detain large volumes of water to slow the rate into

Kanaha Stream. In addition, the basin is designed to use existing open space (i.e. no residential houses). There is an arch culvert under the berm; it is 12' long and 4'-1" high. The outlet allows about 946 cfs to pass through into Kanaha Stream.

Makiki Debris and Detention Basin

This basin is designed as both a way to temporarily contain water and stop the flow of debris into Makiki Stream. The design assumes a downstream capacity of 450 cfs and 24 foot high berms with an emergency spillway. The basin is not designed to permanently contain water, but to detain large volumes of water to slow the rate into Makiki Stream. In addition, the basin is designed to use existing open space (i.e. no residential houses). There is an arch culvert under the berm; it is 12' long and 4'-1" high. The outlet allows about 390 cfs, the 20-percent ACE (5-year) storm, to pass through into Makiki Stream.

Hausten Ditch Detention

This basin is designed to temporarily contain water while the slide gates at Hausten Ditch Bridge are up. The slide gates will be down during a flood event to prevent water from Ala Wai Canal flowing back into Hausten Ditch. In addition, the basin is designed to use existing open space (i.e. no residential houses).

Ala Wai Golf Course Multi-Purpose Detention

This multi-purpose detention basin is designed to contain water on the golf course and prevent flood waters from leaving. There will be earthen berms constructed on the north and east sides of the golf course, the berms will basically follow the existing golf cart road. The berms have an average height of 3 feet.

A sediment basin would be located in the vicinity surrounding holes 12 to 18 of the Ala Wai Golf Course. Flows from the Manoa-Palolo Drainage Canal would be diverted into the sediment basin, which would help reduce the amount of sediment deposited into the Ala Wai Canal. Flows would reenter into the Canal at two locations, a new outlet connected to the sediment basin and an existing outlet.

Manoa Dam

This dam is designed to contain all of the predicted upstream storm water. Through modeling and rain data, it was determined that the dam needed to retain about 17,000,000 cf of water behind the residential neighborhoods of Manoa. This large quantity of water severely restricted the dam location and forced a maximum height of 50 feet. Two spillways capable of handling 3,450 cfs each located above the river bed. The spillways were sized to allow overflow for a 0.2-percent ACE (500-year) storm, with a peak flow of 6,900 cfs. There are two outlets, one into Waihi Stream and one at the junction of Luaalea and Waiakeakua Stream. The Waihi Stream 5x7 foot culvert has a capacity of 1,770 cfs and the Waiakeakua Stream 5x6 foot culvert has a capacity of 1,890 cfs, representing flows from a 20-percent ACE (5-year) storm. Flows greater than this will cause water to back up and be retained behind the dam (*Oceanit, 2008c*). In addition, the structure is to not interfere with existing farms and houses near the basin site.

Waihi Debris and Detention Basin

This basin is designed as both a way to temporarily contain water and stop the flow of debris into the Manoa residential area. The design assumes a maximum downstream capacity of 3,000 cfs and a maximum 24 foot height and minimum containment of 125,000 cubic feet. The basin is not designed to permanently contain water, but to detain large volumes of water to slow the rate into Manoa Stream. In addition, the structure is to not interfere with existing farms and houses near the basin site.

There is an arch culvert under the berm; it is 12' long and 4'-1" high. The outlet allows about 2,000 cfs, the 20-percent ACE (5-year) storm, to pass through into Manoa Stream. The emergency spillway would begin to overflow when the retention capacity of 125,000 cf is exceeded.

Waiakeakua Debris and Detention Basin

This basin is designed as both a way to temporarily contain water and stop the flow of debris into the Manoa residential area. The design assumes a maximum downstream capacity of 3,000 cfs and a maximum 20 foot height and minimum containment of 346,000 cubic feet. The basin is not designed to permanently contain water, but to detain large volumes of water to slow the rate into Manoa Stream. In addition, the structure is to not interfere with existing farms and houses near the basin site.

There is an arch culvert under the berm; it is 12' long and 4'-1" high. The outlet allows about 1,475 cfs, the 20-percent ACE (5-year) storm, to pass through into Manoa Stream. The emergency spillway can handle an extra 3,150 cfs should such high flows occur.

Woodlawn Ditch Detention Basin

The basin is not designed to permanently contain water, but to detain large volumes of water to slow the rate into Woodlawn Ditch will eventually flow into Manoa Stream. The design assumes a maximum downstream capacity of 2,750 cfs and a maximum 20 foot height, there is also a 3x80 foot concrete-lined emergency spillway. The design assumes that slightly less than 750 cfs will flow into Woodlawn Ditch while the remaining flow will be contained in the detention basin during a flood event.

There is an arch culvert under the berm; it is 12' long and 4'-1" high. The outlet allows about 977 cfs out into Woodlawn Ditch.

Manoa Park Detention Basin

The basin is designed to use existing open space (i.e. no residential houses, use of park land) to create a detention basin that is both useful and aesthetically pleasing to the community. The detention basin is designed to handle an inflow of 4,250 cfs;

additional flows up to 2,490 cfs will require the use of the emergency spillway. The berm has a maximum eight of 13 feet and it would border three sides of Manoa District Park. The intake pipes from Poelua Place lies underground and has a bubble-up structure located in the south east corner of the park, covered by a concrete pad to ease in clean-up procedures. The bubble-up structure intakes both 10x10 foot box culverts and has a 2 foot diameter concrete outlet pipe for slow drainage back into Manoa Stream. Any water that does not immediately drain is bubbled into the basin. The basin has graded contours to allow water to accumulate to the southeast corner of the park at the drainage. For aesthetics and for clean-up the three surrounding berms have bleachers on the inside. The new landscaped park has room to contain two baseball fields and a soccer field.

Kanewai Field Multi-Purpose Detention Basin

The basin is designed to use existing open space (park land) to temporarily contain floodwaters. The intake into Kanewai Field is intended to handle 3,960 cfs of the stream inflow and outflow. The existing drainage pipe is able to drain 62 cfs. Kanewai Field will be surrounded by a 7 feet high earthen berm, but protecting the existing structures. On the northwest end of the basin, adjacent to Manoa Stream, the berm is graded down to a 3x60 foot spillway that will allow water to flow into the basin when the river level is high and out of the basin once the high flows have passed.

Pukele Debris and Detention Basin

This basin is designed as both a way to temporarily contain water and stop the flow of debris into Pukele Stream. The design assumes a downstream capacity of 1,700 cfs and a 24 foot high berm with an emergency spillway. The basin is not designed to permanently contain water, but to detain large volumes of water to slow the rate into Makiki Stream. There is an arch culvert under the berm; it is 12' long and 4'-1" high. The outlet allows about 300 cfs, to pass through into Pukele Stream.

Waiomao Debris and Detention Basin

This basin is designed as both a way to temporarily contain water and stop the flow of debris into Waiomao Stream. The design assumes a downstream capacity of 1,540 cfs and a 24 foot high berm with an emergency spillway. The basin is not designed to permanently contain water, but to detain large volumes of water to slow the rate into Makiki Stream. There is an arch culvert under the berm; it is 12' long and 4'-1" high. The outlet allows about 400 cfs, to pass through into Pukele Stream.

6 WITH PROJECT HYDRAULIC MODELING

6.1 Detention Analysis

For each of the detention scenarios modeled (See With Project Hydrologic Modeling Section) the HEC-RAS model was modified with the revised flows.

6.2 Debris Catchment Analysis

In order to determine the effectiveness of debris catchments throughout the watershed, the HEC-RAS model, the Technical Summary Report Manoa, Watershed Project (*Oceanit, 2008c*), and other analysis were used to study different debris catchment scenarios.

6.2.1 Preliminary Analysis

Initially, 7 different sites were selected throughout the watershed. **Figure 11** shows the locations of the proposed debris catchment sites. Each debris catchment was designed to maximize effectiveness while remaining in a viable location within reasonable vertical and horizontal limitations.

Waiakeakua Debris Catchment

This structure is designed to catch large debris. It allows all flood flows to pass through the debris poles. It is assumed that the largest river flow will be below the maximum height of the poles and will span the length of the debris catchment structure. The structure consists of a 2-foot thick concrete pad that spans 140 feet across the stream and the floodplain with a width of 8 feet. Steel posts 8 inches in diameter and between 4 to 7 feet high are evenly spaced at 4 feet along the center of the concrete pad.

Waihi Debris Catchment

This structure is designed to catch large debris. It allows all flood flows to pass through the debris poles. It is assumed that the largest river flow will be below the maximum height of the poles and will span the length of the debris catchment structure. The structure consists of a 2-foot thick concrete pad that spans 140 feet across the stream and the floodplain with a width of 8 feet. Steel posts 8 inches in diameter and between 4 to 7 feet high are evenly spaced at 4 feet along the center of the concrete pad (*Oceanit, 2008c*).

Poelua Place Debris Basin

This basin is designed to provide a capture point for large debris before they reach bridges downstream. In addition, the debris basin will slow the velocity of floodwaters. The debris basin is intended to capture debris in flows larger than a 50-percent ACE (2-year) stream flow and allow passage of all flows.

The debris basin consists of two separate structures. On the east side is the actual basin. The old oxbow lot will be dug down to reclaim the bend in the stream. A small berm surrounding the basin protects the existing neighborhoods from any water that may spill out of the basin. Water carrying large debris enters the northern end of the basin, curves around, and reenters the stream at the south end. Any large debris floating atop the flood waters are caught by a debris catcher consisting of five 8 inch diameter steel posts secured by a concrete pad.

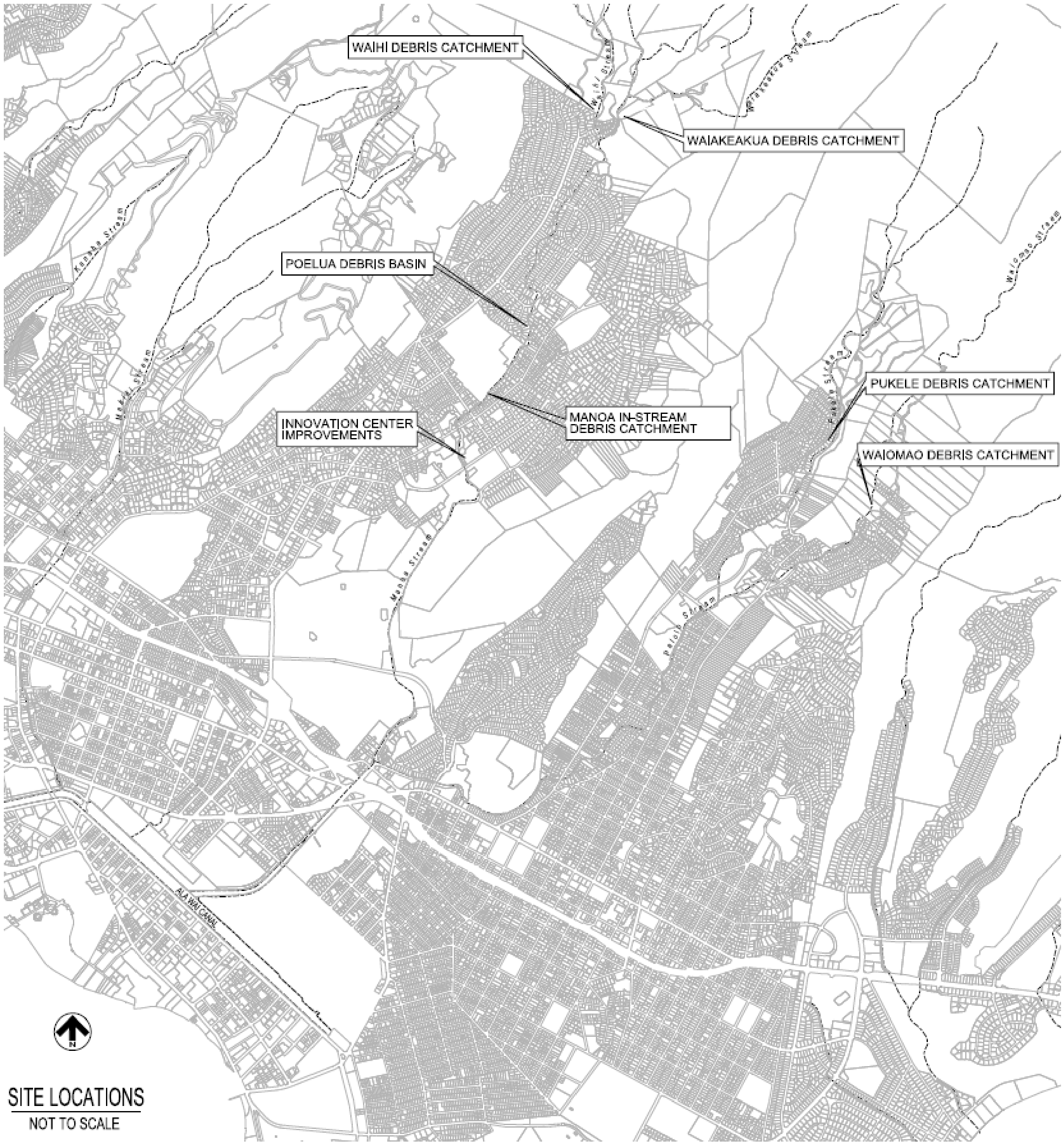

Figure 11: Preliminary Debris Catchment Locations

Manoa In-Stream Debris Catchment

This structure is designed to catch large debris. It allows all flood flows to pass through the debris poles. It is assumed that the largest river flow will be below the maximum height of the poles and will span the length of the debris catchment structure. The structure consists of a 2-foot thick concrete pad that spans 60 feet across the stream and the floodplain with a width of 8 feet. Steel posts 8 inches in diameter and between 4 to 7 feet high are evenly spaced at 4 feet along the center of the concrete pad.

Innovation Center Improvements

The basis of this design is to restore the original floodplain between East Manoa Road Bridge and Woodlawn Bridge to lower the volume of floodwaters entering downstream of Woodlawn Bridge. A new floodplain will be created. The floodplain is intended to be inundated and capture debris when flows exceed the 50-percent ACE (2-year) storm flow. Water on the floodplain will re-enter the stream after is passes through the debris catchers (*Oceanit, 2008c*). The debris catchers consists of a 1-foot thick concrete pad that spans 250 feet parallel to the stream and the floodplain with a width of 6 feet. Steel posts 8 inches in diameter and 4 feet high are evenly spaced at 6 feet along the center of the concrete pad.

Waiomao Debris Catchment

This structure is designed to catch large debris. It allows all flood flows to pass through the debris poles. It is assumed that the largest river flow will be below the maximum height of the poles and will span the length of the debris catchment structure. The structure consists of a 2-foot thick concrete pad that spans 50 feet across the stream and the floodplain with a width of 8 feet. Steel posts 8 inches in diameter and between 4 to 7 feet high are evenly spaced at 4 feet along the center of the concrete pad.

Pukele Debris Catchment

This structure is designed to catch large debris. It allows all flood flows to pass through the debris poles. It is assumed that the largest river flow will be below the maximum height of the poles and will span the length of the debris catchment structure. The structure consists of a 2-foot thick concrete pad that spans 25 feet across the stream and the floodplain with a width of 8 feet. Steel posts 8 inches in diameter and between 4 to 7 feet high are evenly spaced at 4 feet along the center of the concrete pad.

6.3 Floodwall AnalysisIn order to determine the effectiveness of floodwalls throughout the watershed, the HEC-RAS model, the Technical Summary Report, Manoa Watershed Project (*Oceanit, 2008c*), and other analysis were used to study different floodwall scenarios.

6.3.1 Preliminary Analysis

Initially, 3 different sites were selected throughout the watershed. **Figure 12** shows the locations of the proposed floodwall sites. Each floodwall system was designed to maximize effectiveness while remaining in a viable location within reasonable vertical and horizontal limitations.

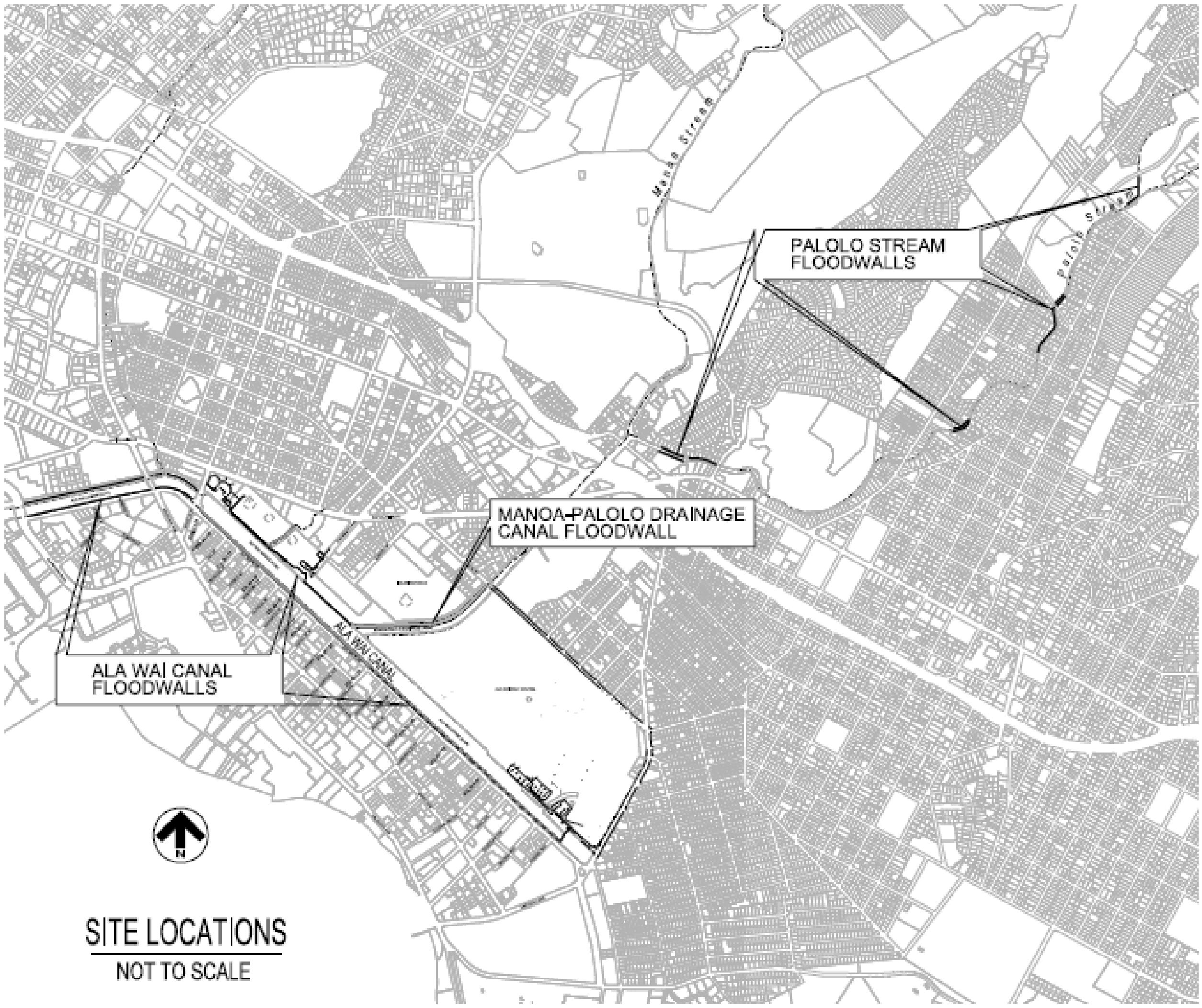

Figure 12: Preliminary Floodwall Locations

Palolo Stream Floodwalls

The Palolo Stream Floodwall system is designed to keep the 1-percent ACE (100-year) flow within the existing concrete stream channel with 90% Assurance. The floodwalls will be constructed out of reinforced concrete.

Manoa-Palolo Drainage Canal Floodwall

The Manoa-Palolo Drainage Canal Floodwall system is designed to keep the 1-percent ACE (100-year) flow from flooding the right bank with 90% Assurance. The floodwall will be constructed out of reinforced concrete and will run along the right bank from Date Street to the Ala Wai Canal.

Ala Wai Canal Floodwalls

The Ala Wai Canal Floodwall system is designed to keep the 1-percent ACE (100-year) flow within the existing channel with 90% Assurance. The entire floodwall system will be constructed out of reinforced concrete. At the upstream end the floodwall system will connect to the Ala Wai Golf Course Levee and run along the left bank of the Ala Wai Canal. There will be no floodwall on the right bank of the Canal next to the golf course. The floodwall system will continue to run along both banks all the way to the Ala Moana Blvd Bridge.

7 ALTERNATIVES

After doing analysis on the different detention basins combined with other measures such as debris catchments and floodwalls alternatives were developed. See the Feasibility Report for the full formulation of the Alternatives. This section explains which measures were used for three different alternatives.

7.1 Alternative 1A

Alternative 1A was developed using the Manoa Dam to detain as much flow as it could without using other detention measures along Manoa Stream. The emphasis of this alternative was the Manoa Dam because majority of the flows that reach the Ala Wai Canal are from Manoa Stream. The Manoa Dam would be constructed to allow debris to be caught during storm events to prevent debris from the upper watershed from entering Manoa Stream. An in-stream debris catchment was also used in this alternative to catch debris that entered the stream downstream of the dam. This measure is located right below Manoa District Park.

Waiomao Debris and Detention Basin and Pukele Debris and Detention Basins were also used in this alternative above Palolo Stream. These two measures were used to lower the peak flow in Palolo Stream, these measures also prevented debris from entering the concrete lined Palolo Stream. The two debris and detention basins did not lower the 1-percent ACE (100-year) storm enough to provide 90% Assurance so floodwalls were also added to this alternative in certain areas to attain 90% Assurance for the 1-percent ACE (100-year) event. A floodwall was also added on the right bank of the Manoa-Palolo Drainage Canal to prevent flooding in and around Iolani School.

The Roosevelt Debris and Detention Basin was used in the Makiki Watershed to lower the peak flow. This measure also catches debris to prevent it from entering Makiki Stream. Floodwalls were analyzed in Makiki but they were not feasible because of the required heights.

Even with the Manoa Dam, floodwalls along the Ala Wai Canal are still needed to prevent the 1-percent ACE (100-year) storm event from flooding Waikiki and Moiliili. The Hausten Ditch Detention was used in conjunction with the Ala Wai Canal Floodwall system to prevent interior drainage from flooding Moiliili when slide gates are closed at the Hausten Ditch Pedestrian Bridge.

7.2 Alternative 2A

The emphasis for Alternative 2A was not to detain water in the un-urbanized areas. Therefore, there were no detention basins or dams in the upstream areas of Manoa Stream but the Woodlawn Ditch Detention Basin, the Manoa Park Detention Basin, and the Kanewai Field Multi-Purpose Detention Basin were used in this alternative. Because there were no debris and detention basins in the upstream area a debris catchment will be constructed at Waiakeakua Stream and Waihi Stream. An in-stream debris catchment will also be constructed at Poelua Place; this is just upstream of the Manoa Park Detention Basin's intake so the debris catchment prevents debris from going down stream of that location including preventing debris from clogging up the intake. An in-stream debris catchment will also be constructed next to the Innovation Center just upstream of Woodlawn Bridge.

No detention measures were used in the Palolo Watershed; there just isn't any land available along Palolo Stream. The floodwall from Alternative 1 was also added on the right bank of the Manoa-Palolo Drainage Canal to prevent flooding in and around Iolani School.

The Makiki Debris and Detention Basin and the Roosevelt Debris and Detention Basins were used in the Makiki Watershed because that was the only measure that could be used to lower flows in that watershed. Same as Alternative 1, the floodwalls were analyzed in Makiki but they were not feasible because of the required heights.

Floodwalls along the Ala Wai Canal are also needed to prevent the 1-percent ACE (100-year) storm event from flooding Waikiki and Moiliili. The Hausten Ditch Detention was used in conjunction with the Ala Wai Canal Floodwall system to prevent interior drainage from flooding Moiliili when slide gates are closed at the Hausten Ditch Pedestrian Bridge. The Ala Wai Golf Course Multi-Purpose Detention was also used in conjunction with the Ala Wai Canal Floodwall system.

7.3 Alternative 3A

This alternative was created to lower as much flow as possible without using the Manoa Dam, a more effective and logical approach to flood protection. The Waiakeakua Debris and Detention Basin along with the Waihi Debris and Detention Basin and the Woodlawn Ditch Detention Basin were used along Manoa Stream. The in-stream debris catchment below Manoa District Park would also be a part of this alternative to catch debris that enters the stream downstream of Waiakeakua and Waihi Streams. The debris catchment at the Innovation Center was initially part of this alternative but after doing incremental justification the Kanewai Field Multi-Purpose Detention Basin was a better overall measure to use in this alternative instead.

The Waiomao Debris and Detention Basin and the Pukele Debris and Detention Basin were both used for the Palolo Watershed. After doing further analysis the floodwalls along Palolo Stream were too costly and infeasible. Further analysis was also done on the floodwall along the Manoa-Palolo Drainage Canal and that measure was not incrementally justified.

The Makiki Watershed initially used the Makiki Debris and Detention Basin and the Roosevelt Debris and Detention Basins but after incremental justification the Roosevelt Debris and Detention Basin was not justified. As in Alternatives 1 and 2, the floodwalls were analyzed in Makiki but they were not feasible because of the required heights.

Floodwalls along the Ala Wai Canal are also needed in this alternative to prevent the 1-percent ACE (100-year) storm event from flooding Waikiki and Moiliili. The Hausten Ditch Detention was used in conjunction with the Ala Wai Canal Floodwall system to prevent interior drainage from flooding Moiliili when slide gates are closed at the Hausten Ditch Pedestrian Bridge. The Ala Wai Golf Course Multi-Purpose Detention was also used in conjunction with the Ala Wai Canal Floodwall system. This alternative became our TSP.

Detailed hydrologic modeling of the detention basins included as part of the TSP was performed to refine the design and determine their effectiveness as part of the system-wide alternative. The detention basins were inserted into the HEC-HMS hydrologic model and simulation runs were made for the 50%, 20%, 10%, 5%, 2%, 1%, 0.5%, and 0.2% AEP events. In addition, a 0.999% AEP event simulation was run for purposes of inclusion into the HEC-FDA economic model. Figure 13 below shows the HEC-HMS basin schematic for Alternative 3A.

For each detention basin, elevation-storage relationships were developed in GIS from the elevation data. Initial outlet configurations were selected and the hydrologic model was run to determine the basin effectiveness. The initial objective of the basin design was to allow flows up to the 20% AEP event to pass with minimal storage and to reduce outflows at the 1% AEP event to be sufficiently reduced to provide flood damage reduction benefits. Through an iterative process, embankment elevations and outlet configurations were adjusted until satisfactory results were obtained. The final basin specifications and modeling results are presented in the following section.

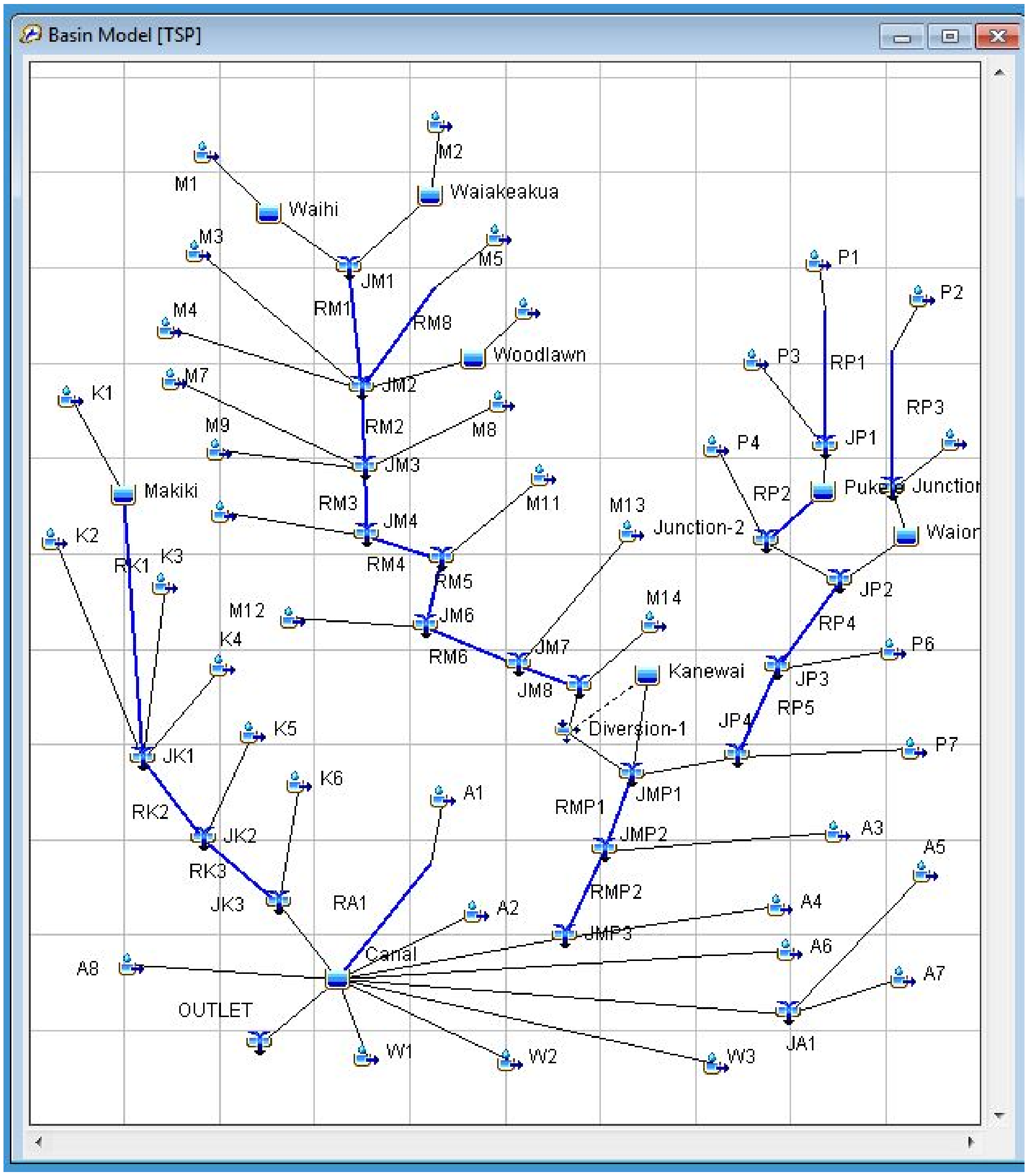

Figure 13. HEC-HMS Basin Schematic – Alternative 3A

7.3.1 Detention Analysis

As a result of the hydrologic modeling, refinements to the basin specifications were made which differ from the preliminary analysis. The refined basin specifications are described in the following paragraphs.

Waiakaekua Detention and Debris Basin

The detention basin will be formed by constructing an earthen berm approximately 34 feet in height and having a 20 foot top width. The top of the berm will be set at elevation 338.0 ft msl. A 1 foot thick concrete spillway 40 feet in length and 80 feet in width will be constructed at elevation 334.0 ft msl. The downstream face of the embankment will have a slope of 2H:1V and the upstream face will have a slope of 3H:1V. A 2 foot thick layer of grouted riprap will protect the embankment face from erosion.

The outlet of the basin will consist of a 12' x 6' corrugated metal arch. The ends of the arch will be mitered to match the embankment slopes. The estimated length of this culvert is about 200 feet. The upstream invert of the culvert is at elevation 304.0 ft msl and the downstream invert is at elevation 299.0 ft msl. At the upstream end of the outlet culvert, a series of seven (7) eight inch diameter concrete filled steel pipe will be installed to serve as a debris trap. The downstream end of the outlet culvert will require riprap protection from erosion.

This basin configuration was modeled in the hydrologic model and the following table, **Table 10**, shows the results of the modeling.

Table 10. Waiakaekua Detention Basin Modeling Results

Exceedance Prob. (%)	Recurrence Interval	HEC-HMS Output			
		Inflow (cfs)	Outflow (cfs)	Storage (ac-ft)	Elev. (ft)
0.999	1	331	331	0.3	309.2
50	2	514	512	0.9	311.2
20	5	813	758	4.2	317
10	10	1075	930	9	321.2
5	20	1365	1083	16.7	325.6
2	50	1786	1272	33.1	332.1
1	100	2128	1643	43.9	335.1
0.5	200	2503	2167	47.9	336.2
0.2	500	3034	2834	51.6	337.2

Woodlawn Ditch Detention Basin

The detention basin will be formed by constructing an earthen berm approximately 20 feet in height and having a 10 foot top width. The top of the berm will be set at elevation 220.0 ft msl. A 1 foot thick concrete spillway 25 feet in length and 80 feet in width will be constructed at elevation 217.0 ft msl. The downstream face of the embankment will have a slope of 2H:1V and the upstream face will have a slope of 3H:1V. A 2 foot thick layer of grouted riprap will protect the embankment face from erosion.

The outlet of the basin will consist of a 12' x 4.1' corrugated metal arch culvert. The ends of the arch will be mitered to match the embankment slopes. The estimated length of this culvert is about 110 feet. The upstream invert of the culvert is at elevation 200.0 ft msl and the downstream invert is at elevation 199.0 ft msl.

This basin configuration was modeled in the hydrologic model and the following T**able 11** shows the results of the modeling.

Table 11. Woodlawn Ditch Detention Basin Modeling Results

Exceedance Prob. (%)	Recurrence Interval	HEC-HMS Output			
		Inflow (cfs)	Outflow (cfs)	Storage (ac-ft)	Elev. (ft)
0.999	1	122	119	0.7	205.3
50	2	190	184	1.2	207.1
20	5	300	282	2.1	209.5
10	10	394	364	3.1	211.5
5	20	497	446	4.5	213.4
2	50	646	554	7.1	216.4
1	100	765	710	8.6	217.6
0.5	200	896	866	9.1	218.1
0.2	500	1079	1068	9.8	218.5

Kanewai Field Detention Basin

The detention basin will be formed by constructing an earthen berm approximately 9 feet in height and having a 10 foot top width. The top of the berm will be set at elevation 43.0 ft msl. Flow will enter the basin by way of a lateral weir located at the northwest corner of the basin along the left bank of Manoa Stream. The weir will consist of a 2 foot thick concrete spillway set at elevation 40.0 ft msl and having a weir length of 54 feet. The weir will have side slopes of 3H:1V until meeting the berm elevation for a total length of 60 feet. The interior face of the embankment will have a slope of 2H:1V and the exterior face will have a slope of 3H:1V. A 2 foot thick layer of grouted riprap will protect the spillway approach slopes from erosion.

The outlet of the basin consists of an existing 2' diameter drainage pipe leading back into Manoa Stream. The inlet of the drainage pipe is set at the floor of the basin at its existing elevation of 34.0 ft msl. The outlet of the pipe extends into the left bank of the stream.

This basin configuration was modeled in the hydrologic model and the following **Table 12** shows the results of the modeling.

Table 12. Kanewai Detention Basin Modeling Results

Exceedance Prob. (%)	Recurrence Interval	HEC-HMS Output			
		Inflow (cfs)	Outflow (cfs)	Storage (ac-ft)	Elev. (ft)
0.999	1	0	0	0	30
50	2	0	0	0	30
20	5	1	1	0	30.5
10	10	130	29	3.4	35.8
5	20	260	35.6	11.3	36.2
2	50	418	41.8	26.2	40.9
1	100	504	42.2	27.2	41
0.5	200	553	42.2	27.4	41.1
0.2	500	746	42.3	27.7	41.1

Waihi Detention and Debris Basin

The detention basin will be formed by constructing an earthen berm approximately 37 feet in height and having a 10 foot top width. The top of the berm will be set at elevation 404.0 ft msl. A 1 foot thick concrete spillway 20 feet in length and 100 feet in width will be constructed at elevation 400.0 ft msl. The downstream face of the embankment will have a slope of 2H:1V and the upstream face will have a slope of 3H:1V. A 2 foot thick layer of grouted riprap will protect the embankment face from erosion.

The outlet of the basin will consist of a 12' x 6' box culvert with wingwalls flared at an angle of 45 degrees and extending until they reach the natural channel side slope. The estimated length of this culvert is about 205 feet. The upstream invert of the culvert is at elevation 367.0 ft msl and the downstream invert is at elevation 357.0 ft msl. At the upstream end of the outlet culvert, a series of seven (7) eight inch diameter concrete filled steel pipe will be installed to serve as a debris trap. The downstream end of the outlet culvert will require riprap protection from erosion.

This basin configuration was modeled in the hydrologic model and the following **Table 13** shows the results of the modeling.

Table 13. Waihi Detention Basin Modeling Results

Exceedance Prob. (%)	Recurrence Interval	HEC-HMS Output			
		Inflow (cfs)	Outflow (cfs)	Storage (ac-ft)	Elev. (ft)
0.999	1	488	486	0.2	373.5
50	2	756	755	0.4	375.9
20	5	1199	1174	1.5	381.2
10	10	1581	1499	3.5	386.8
5	20	2006	1781	8	393.2
2	50	2611	2244	17.7	400.7
1	100	3102	2919	19.8	401.9
0.5	200	3640	3581	21.3	402.8
0.2	500	4400	4332	23	403.7

Waiamao Detention and Debris Basin

The detention basin will be formed by constructing an earthen berm approximately 33.5 feet in height and having a 10 foot top width. The top of the berm will be set at elevation 402.0 ft msl. A 1 foot thick concrete spillway 20 feet in length and 100 feet in width will be constructed at elevation 398.6 ft msl. The downstream face of the embankment will have a slope of 2H:1V and the upstream face will have a slope of 3H:1V. A 2 foot thick layer of grouted riprap will protect the embankment face from erosion.

In order to obtain the required storage, excavation of the channel bottom is required. This excavation will enlarge the channel to a 30 foot bottom width having near vertical side slopes (0.5:1HV). From the upstream invert of the outlet culvert the channel bottom will rise at a slope of 2% for a distance of about 200 feet to aid in draining the basin. At this point the channel will transition to the existing channel geometry over a distance of about 200 feet. This excavation will remove an estimated 3.060 CY of material.

The outlet of the basin will consist of a 20’ x 5’ box culvert with a headwall extending across the channel bottom. The estimated length of this culvert is about 178 feet. The upstream invert of the culvert is at elevation 368.6 ft msl and the downstream invert is at elevation 367.6 ft msl. At the upstream end of the outlet culvert, a series of seven (7) eight inch diameter concrete filled steel pipe will be installed to serve as a debris trap. The downstream end of the outlet culvert will require riprap protection from erosion.

This basin configuration was modeled in the hydrologic model and the following **Table 14** shows the results of the modeling.

Table 14. Waiamao Detention Basin Modeling Results

Exceedance Prob. (%)	Recurrence Interval	HEC-HMS Output			
		Inflow (cfs)	Outflow (cfs)	Storage (ac-ft)	Elev. (ft)
0.999	1	352	351	1.3	372.1
50	2	656	649	1.8	373.9
20	5	1141	1123	2.9	377.3
10	10	1579	1522	5	380.9
5	20	2049	1906	8.6	385.5
2	50	2709	2414	16.5	393.1
1	100	3257	2531	28.7	395.1
0.5	200	3873	2636	48.9	397
0.2	500	4707	3162	77.7	399.8

Pukele Detention and Debris Basin

The detention basin will be formed by constructing an earthen berm approximately 30 feet in height and having a 10 foot top width. The top of the berm will be set at elevation 441.0 ft msl. A 1 foot thick concrete spillway 20 feet in length and 80 feet in width will be constructed at elevation 437.0 ft msl. The downstream face of the embankment will have a slope of 2H:1V and the upstream face will have a slope of 3H:1V. A 2 foot thick layer of grouted riprap will protect the embankment face from erosion.

In order to obtain the required storage, excavation of the channel bottom is required. This excavation will enlarge the channel to a 30 foot bottom width having near vertical side slopes (0.5:1HV). From the upstream invert of the outlet culvert the channel bottom will rise at a slope of 1% for a distance of about 200 feet to aid in draining the basin. At this point the channel will transition to the existing channel geometry over a distance of about 300 feet. This excavation will remove an estimated 14,330 CY of material.

The outlet of the basin will consist of a 120’ x 6’ box culvert with a headwall extending across the channel bottom. The estimated length of this culvert is about 160 feet. The upstream invert of the culvert is at elevation 407.0 ft msl and the downstream invert is at elevation 405.0 ft msl. At the upstream end of the outlet culvert, a series of seven (7) eight inch diameter concrete filled steel pipe will be installed to serve as a debris trap. The downstream end of the outlet culvert will require riprap protection from erosion.

This basin configuration was modeled in the hydrologic model and the following **Table 15** shows the results of the modeling.

Table 15. Pukele Detention Basin Modeling Results

Exceedance Prob. (%)	Recurrence Interval	HEC-HMS Output			
		Inflow (cfs)	Outflow (cfs)	Storage (ac-ft)	Elev. (ft)
0.999	1	158	154	0.7	409.8
50	2	379	369	2	412.1
20	5	774	726	4.5	415.6
10	10	1126	1031	7.5	419.2
5	20	1521	1340	12	424.1
2	50	2079	1693	22.1	431.1
1	100	2529	1946	32.6	437.1
0.5	200	3055	2758	36.7	439.1
0.2	500	3778	3612	39.4	440.5

Makiki Detention and Debris Basin

The detention basin will be formed by constructing an earthen berm approximately 33.2 feet in height and having a 10 foot top width. The top of the berm will be set at elevation 184.0 ft msl. A 1 foot thick concrete spillway 20 feet in length and 60 feet in width will be constructed at elevation 180.8 ft msl. The downstream face of the embankment will have a slope of 2H:1V and the upstream face will have a slope of 3H:1V. A 2 foot thick layer of grouted riprap will protect the embankment face from erosion.

In order to obtain the required storage, excavation of the channel bottom is required. This excavation will enlarge the channel to a 30 foot bottom width having near vertical side slopes (0.5:1HV). From the upstream invert of the outlet culvert the channel bottom will rise at a slope of 2% for a distance of about 280 feet to aid in draining the basin. At this point the channel will transition to the existing channel geometry over a distance of about 200 feet. This excavation will remove an estimated 3.035 CY of material.

The outlet of the basin will consist of a 120’ x 6’ box culvert with a headwall extending across the channel bottom. The estimated length of this culvert is about 160 feet. The upstream invert of the culvert is at elevation 407.0 ft msl and the downstream invert is at elevation 405.0 ft msl. At the upstream end of the outlet culvert, a series of seven (7) eight inch diameter concrete filled steel pipe will be installed to serve as a debris trap. The downstream end of the outlet culvert will require riprap protection from erosion.

This basin configuration was modeled in the hydrologic model and the following **Table 16** shows the results of the modeling.

Table 16. Makiki Detention Basin Modeling Results

Exceedance Prob. (%)	Recurrence Interval	HEC-HMS Output			
		Inflow (cfs)	Outflow (cfs)	Storage (ac-ft)	Elev. (ft)
0.999	1	105	105	0.7	153.4
50	2	204	203	0.9	154.9
20	5	385	382	1.3	157.1
10	10	558	542	2.2	159.5
5	20	766	716	3.9	163.6
2	50	1084	954	8.9	171
1	100	1363	1083	15.8	175.9
0.5	200	1680	1192	26.7	180.5
0.2	500	2149	1218	52.8	181

7.3.2 Peak Flow Data

The results of the hydrologic modeling were tabulated and served as the input for the hydraulic model for improved conditions. Table 15 below shows the peak flows for Alternative 3A, the Tentatively Selected Plan (TSP). Flows for the Golf Course Detention basin, the Kanewai Detention basin, Kanaha Split reach, and the UH_split reach were indexed by 0.01 cfs due to the requirement in HEC-FDA that flows are constantly increasing within a cross section. **Tables 17** and **18** compares the peak flows from the existing without-project condition with the with-project Alternative 3A condition

Table 17. Input Peak Flow Discharges in cubic feet per second for Ala Wai Canal Watershed HEC-RAS for Alternative 3A Model, Honolulu, Hawaii

Model Stream Names		Model Input Cross-section Location	Percent ACE Flood							
River	Reach		50%	20%	10%	5%	2%	1%	0.5%	0.2%
Ala Wai	Upper	9724	1,000	1,400	1,800	2,300	3,040	3,600	4,320	5,300
Ala Wai	Middle	5825	3,520	5,850	7,420	9,310	12,100	14,000	17,700	20,700
Ala Wai	Lower	2324	7,920	10,950	12,320	14,110	15,600	16,500	17,300	19,500
Golf Cs	1	1444	1.01	1.02	1.03	1.04	1.05	1.06	1.07	1.08
Kanaha	Ditch	4372	270	500	700	930	1,240	1,500	1800	2,200
Kanaha	Split	3508	1.01	1.02	1.03	1.04	1.05	1.06	1.07	1.08
Kanewai	1	550	1.01	1.02	1.03	1.04	1.05	1.06	1.07	1.08
Makiki	Upper	10768	150	330	540	770	1,080	1,200	1,400	1,800
Makiki	Upper	7674	240	490	770	1,040	1,290	1,510	1,720	1,960
Makiki	Lower	6286	630	1,260	1,830	2,420	3,120	3,740	4,120	5,230
Makiki	Lower	3189	680	1,360	1,930	2,570	3,220	4,040	4,920	5,530
Makiki	Lower	1465	880	1,660	2,530	3,170	4,220	5,240	6,120	6,830
Manoa	Main	16506	1,170	1,870	2,450	2,890	3,540	4,210	4,890	6,660
Manoa	Main	10968	1,660	3,020	4,150	4,880	5,920	6,910	7,660	9,550
Manoa	Main	9274	1,860	3,320	4,350	5,180	6,170	7,060	7,960	10,050
Manoa	Main	7839	1,960	3,520	4,550	5,480	6,470	7,510	8,560	10,750
Manoa	Main	6175	2,060	3,620	4,750	5,880	6,970	8,110	9,260	11,450
Manoa	Main	2477	2,260	3,920	5,250	6,380	7,770	8,910	10,460	12,650
Manoa	Main	1807	2,460	4,120	5,550	6,780	8,270	9,410	10,760	13,550
Manoa	Main	1230	2,560	4,270	5,700	6,980	8,470	9,910	11,160	14,350
Palolo	Main	15526	950	1,170	2,390	2,930	2,990	3,150	4,640	7,240
Palolo	Main	9520	1,150	1,970	2,590	3,130	3,390	3,650	5,040	8,040
Palolo	Main	7552	1,250	2,370	2,990	3,830	4,190	4,650	6,440	10,040
Palolo	Lower	5198	3320	5,650	7,220	9,110	11,800	13,700	17,300	20,400
Pukele	Tributary	5958	440	820	1,200	1,600	2,100	2,700	3,000	3,900
Pukele	Tributary	3629	520	1,140	1,400	1,720	2,110	2,530	3,530	5,080
UH_Split	UH_Split	6929	1.01	1.02	1.03	1.04	1.05	1.06	1.07	1.08
Waiomao	Tributary	4900	550	1,050	1,540	1,960	2,550	3,000	3,500	4,700

Table 18. Input Peak Flow Discharge Comparison in cubic feet per second for Ala Wai Canal Watershed HEC-RAS for Existing Conditions and TSP Model, Honolulu, Hawaii

Model Stream Names		**Model Input Cross-section Location**	**Percent ACE Flood**							
			Existing Conditions				**TSP**			
River	**Reach**		**10%**	**2%**	**1%**	**0.2%**	**10%**	**2%**	**1%**	**0.2%**
Ala Wai	Upper	9724	1,800	3,040	3,600	5,300	1,800	3,040	3,600	5,300
Ala Wai	Middle	5825	8,600	14,500	17,000	23,200	7,420	12,100	14,000	20,700
Ala Wai	Lower	2324	13,500	18,000	19,500	22,000	12,320	15,600	16,500	19,500
Kanaha	Ditch	4372	700	1,240	1,500	2,200	700	1,240	1,500	2,200
Kanaha	Split	3508	1.03	1.05	1.06	1.08	1.03	1.05	1.06	1.08
Makiki	Upper	10768	800	1,400	1,700	2,400	540	1,080	1,200	1,800
Makiki	Upper	7674	770	1,290	1,510	1,960	770	1,290	1,510	1,960
Makiki	Lower	6286	1,900	3,400	4,200	6,400	1,830	3,120	3,740	5,230
Makiki	Lower	3189	2,000	3,500	4,500	6,700	1,930	3,220	4,040	5,530
Makiki	Lower	1465	2,600	4,500	5,700	8,000	2,530	4,220	5,240	6,830
Manoa	Main	16506	2,800	4,600	5,500	7,400	2,450	3,540	4,210	6,660
Manoa	Main	10968	4,600	7,150	8,200	10,500	4,150	5,920	6,910	9,550
Manoa	Main	9274	4,800	7,400	8,350	11,000	4,350	6,170	7,060	10,050
Manoa	Main	7839	5,000	7,700	8,800	11,700	4,550	6,470	7,510	10,750
Manoa	Main	6175	5,200	8,200	9,400	12,400	4,750	6,970	8,110	11,450
Manoa	Main	2477	5,700	9,000	10,200	13,600	5,250	7,770	8,910	12,650
Manoa	Main	1807	6,000	9,500	10,700	14,500	5,550	8,270	9,410	13,550
Manoa	Main	1230	6,150	9,700	11,200	15,300	5,700	8,470	9,910	14,350
Palolo	Main	15526	2,900	5,100	6,100	9,200	2,390	2,990	3,150	7,240
Palolo	Main	9520	3,100	5,500	6,600	10,000	2,590	3,390	3,650	8,040
Palolo	Main	7552	3,500	6,300	7,600	12,000	2,990	4,190	4,650	10,040
Palolo	Lower	5198	8,400	14,200	16,700	22,900	7,220	11,800	13,700	20,400
Pukele	Tributary	5958	1,200	2,100	2,700	3,900	1,200	2,100	2,700	3,900
Pukele	Tributary	3629	1,710	2,940	3,500	5,400	1,400	2,110	2,530	5,080
UH_Split	UH_Split	6929	1.03	1.05	1.06	1.08	1.03	1.05	1.06	1.08
Waiomao	Tributary	4900	1,540	2,550	3,000	4,700	1,540	2,550	3,000	4,700

7.3.3 Uncertainty and EYOR

The uncertainty of the peak flow discharge values is based on the equivalent years of record. The final equivalent years of record (EYOR), used in the risk and uncertainty of the HEC-FDA model is based on stream reach and is presented in Table 17. The Makiki Watershed with the least amount of available data was given the lowest EYOR of 18 years, while the remaining sub-watersheds were assigned values from 25 to 30 years. The highest values were from sub-basins where the peak flow discharges were almost entirely based on gaged data; Pukele and Waiomao Streams. The assigned EYOR is based on the overall confidence in the reliability or accuracy of the peak flow discharge estimates and as applied in HEC-FDA constrains the confidence limits of the sampling of peak flow discharge estimates. The initial values in **Table 19** were determined from the final output discharge values from HEC-RAS and methodology in Bulletin 17B, Appendix 5 (Interagency Advisory Committee on Water Data, 1982). The actual usage of these values versus the graphical curve method is discussed in the Economic Appendix.

Table 19. With-Project Peak Flow Discharge Frequency Data based on HEC-RAS Output Discharges Intermediate Future Condition and Uncertainty in Equivalent Years of Record used in HEC-FDA, Ala Wai Watershed, Oahu, Hawaii

Stream or Sub-Watershed	HEC-HMS Model Sub-Basin or Junction	HEC-RAS Reach Name	HEC-FDA Reach Name	HEC-FDA Analytical Frequency Curve Data (Log Units)			EYOR
				Mean	Std. Dev.	Skew	
Ala Wai, Waikiki	Ala Wai	Ala Wai Lower	ALA 1	3.8420	0.1601	-1.6634	30
		Ala Wai Middle	ALA 2	3.5099	0.1838	-0.1656	
		Ala Wai Upper	ALA 3	3.0325	0.1663	1.1793	
Makiki	K2	Kanaha Ditch	KAH 1 KAH 2	2.4314	0.3201	-0.0040	18
	----	Kanaha Split	KAO 1	----	----	----	
	JK3	Makiki Lower	MAK 1	2.8222	0.2392	0.1502	
	JK2		MAK 2	2.7783	0.2539	-0.0656	
	JK1	Makiki Upper	MAK 3	2.3255	0.4752	-0.6975	
	K1, K3		MAK 4	2.1257	0.5071	-0.5551	
Manoa	JM7, JM 8	Manoa Stream Main Reach	MAN 1	3.3773	0.3112	-0.6017	25
	JM 6		MAN 2	3.2876	0.3160	-0.5346	
	JM 4, JM 5		MAN 3 MAN 4	3.2565	0.3373	-0.6436	
	JM 3		MAN 5	3.1730	0.3751	-0.7639	
	JM 2		MAN 6	3.0596	0.2592	-0.2146	
	JM 1		MAN 7	2.9680	0.3013	-0.2146	
	----	UH_Split	UNI 1 UNI 2	----	----	----	18
Manoa-Palolo Canal	JMP 1 to JMP 3	Palolo Lower	MPC 1	3.5089	0.2757	-0.2717	30
			MPC 2	3.5169	0.2651	-0.0968	30
Palolo	JP 4	Palolo Main	PAL 1	3.0410	0.3774	-0.9202	27
			PAL 2	2.9826	0.3999	-1.1816	27
	JP 3	Palolo Main	PAL 3 PAL 4	2.8373	0.5523	-1.5663	27
	Junction 2	Pukele Tributary	PUK 1	2.6776	0.3858	-0.6359	44
	Junction 1	Waiomao Ditch	WAI 1	2.6303	0.4885	-1.3841	35

EYOR = Equivalent Years of Record; ----, not a separate sub-basin in HEC-HMS model

7.3.4 Scour and Erosion Protection

At each of the 5 primary in-stream detention basins, Makiki, Waihi, Waiakeakua, Pukele, and Waiamao, exit velocities ranged from 27 to 35 ft/s for the 1% ACE flood event. Natural stream velocities at these locations without any project features ranged from 18 to 24 ft/s for the 1% ACE flood event. Due to the increased velocities and concentrated flow at the culvert outlets, a dissipation and scour protection basin was designed and added to each of the primary in-stream detention basins. Given the similar hydraulic characteristics at each location, the 1% ACE flood event highest velocity of 35.4 ft/s at the Waihi Stream detention basin was used as the input design parameters to provide a constant set of output design parameters to be used at all five detention basins. Other design input parameters included 12 ft by 6 ft box culvert with 4.88% slope, manning's n of 0.015, tailwater depth of 4.4 ft and outlet depth in culvert of 4.9 ft.

Guidance in the Federal Highway Administration (FHWA) report FHWA-NHI-06-086, Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular Number 14, Third edition was used to design the rip rap dissipation and scour protection basins. This report provides design procedures based on research and field survey of practices and experience. For this particular design, the rip rap basin typical in **Figure 14** was used.

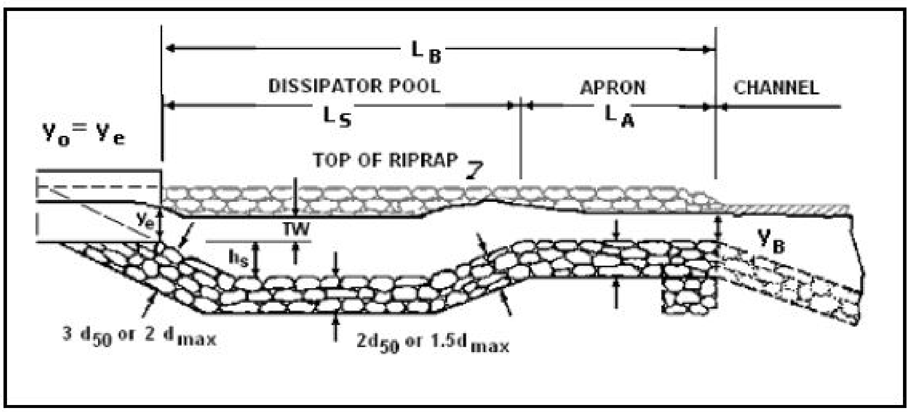

Figure 14. Typical Profile of Rip Rap Basin (Figure 10.1 in FHWA HEC-14)

Using VDOT Class Ill riprap (D_{50} = 2.2 ft) the riprap basin would have the following dimensions:

- Length of dissipation pool (Ls) = 86 ft
- Dissipator pool depth {h_S} = 8.6 ft
- Length of apron (L_A) = 43 ft
- Total basin length (L_S)= 129 ft

- Basin width at the basin exit {W_s} = 98 ft ideal, but varies by stream width at each location
- Basin side slope (z) = 2H:1V
- Riprap Basin exit velocity {V_e)= 9.5 ft/sec

The basin width at the end of the apron was 50 ft at the Makiki basin, 90 ft at the Waihi basin, 70 ft at the Waiakeakua basin, 60 ft at the Pukele basin, and 60 ft at the Waiamao basin. The design typical is illustrated in Plate 11 on sheet C-308.

7.4 WITH PROJECT HYDRAULIC MODELING

Peak flows developed from the With Project hydrologic model were used to determine water surface elevations for the TSP.

The hydraulic model geometry was modified to include the upstream detention and debris basins, floodwalls, and the Ala Wai Golf course Detention basin. In Section 3.1.4, there is a discussion on blockages of several bridge structures due to debris. Since the upstream detention basins contain mechanisms to trap debris, these blockages were removed from the model geometry.

The Ala Wai Golf Course Detention basin is an off-channel basin which accepts water which overfows the main channel. Water enters the basin from the upper reach of the Ala Wai canal and the left bank of the Manoa-Palolo canal below the Date St. Bridge. These overflows are modeled as lateral weirs. On the Ala Wai canal, the right bank serves as the weir. Overflow occurs from its confluence with the Manoa-Palolo canal up stream to its end. The weir coefficient was set to be 0.3. On the Manoa-Palolo canal, water overtops the left bank and enters the golf course detention. This weir coefficient was set to be 1.0. The golf course detention area was modeled as a separate reach in order to route flows thru the basin.

7.4.1 Model Results

On the Ala Wai canal, results of the modeling show that flow begins to enter the golf course detention from the Ala Wai side at the 50% AEP event thru a low point along the bank profile. Along the Manoa-Palolo stream, flow will begin overtopping the left bank at the 20% AEP event. At the 1% AEP event, a total of 4940 cfs is diverted to the golf course and the detention area has an elevation of about 5.3 msl.

The floodwalls along the Ala Wai canal protect Waikiki and the surrounding area from flood damage. On the left bank of the canal, the floodwalls begin at the Ala Moana Blvd. bridge and extend upstream to the end of the canal, where it joins the earthen levee that forms the golf course detention basin. On the right bank of the canal, the floodwall also begins at the Ala Moana Blvd. Bridge, but is not continuous. The floodwall breaks at the confluence with Makiki Stream. It then begins again above Makiki Stream and upstream to the confluence with the Manoa-Palolo canal. This break, at Makiki Stream, allows water to back up into the stream and prevent the stream from draining. The economic analysis will determine the effect this has on the area surrounding Makiki Stream.

Initial elevations of the floodwalls were set to be 2 feet above the 1% AEP water surface elevation. A risk analysis was performed to determine the final levee heights that will satisfy a conditional non-exceedance probability (CNP) of at least 90%. The results of the analysis show that for reach ALA1, the floodwall elevation at the index point (sta. 1477) is 6.3. For reach ALA2, the floodwall elevation at the index point (sta. 4847) is 8.75, and for reach ALA3A, at the index point (sta. 8015) is 9.3. **Table 20** shows the average elevation and height of the floodwalls by reach. Further discussion of the analysis is presented in the Economic Appendix. **Plate 2** shows cross sections plots at the index points of the modeled streams for the TSP.

Table 20. Average Elevations and Heights of Ala Wai Canal Floodwalls in Feet

REACH	**Left Bank**		**Right Bank**	
	Ave. Elev.	**Ave. Height**	**Ave. Elev.**	**Ave. Height**
ALA1	5.7	1.1	5.7	1
ALA2	8.5	3.3	8.5	3.4
ALA3	9.3	4.4	NA	NA

There is a concern that installation of the floodwalls will have an adverse impact on the three bridges crossing the Ala Wai Canal. Water surface elevations at or higher than the low chord elevations of the bridges may cause undue uplift pressure on the bridge structure and would require some measure of strengthening to mitigate. The results of the model show that the maximum water surface elevations do not impinge on the maximum low chord of the bridges. **Table 21** below show the comparison of with project water surface elevations with the bridge deck and low chord elevations.

Table 21. Comparison of Bridge Data with Water Surface Elevations for With Project Conditions

Bridge	**Top of Roadway at highest low chord (ft)**	**Highest Low Chord (ft)**	**Station on Sheet C-310**	**Left Bank Elevation at sta. (ft)**	**Top of Floodwall Elevation at sta. (ft)**	**1% AEP WSE (ft)**	**0.5% AEP WSE (ft)**	**0.2% AEP WSE (ft)**
Ala Moana Blvd	11.60	7.40	4+39	5.00	5.00	2.55	2.56	2.57
Kalakaua Ave	8.80	6.6	23+24	5.9	7.3	5.57	5.58	5.59
McCully St.	10	8.81	31+06	5.5	7.6	6.57	6.77	6.97
WSE = water surface elevation								

At the Manoa-Palolo canal, water overtops the right bank and will inundate the area around the Iolani School. An economic analysis was performed to determine whether extending the floodwall upstream to the Date St. Bridge can be incrementally justified. This analysis showed the floodwall extension to be economically infeasible.

At several locations along the streams modeled, hydraulic jumps appear. These jumps occur at the upstream side of various bridges and culverts and are mainly a result of changes in topography and channel slope as the stream bed transitions from the steep uphill areas to the flatter valley areas.

In general, implementation of the recommended plan reduces peak flows entering the Ala Wai Canal system and the floodwalls protect the surrounding area from damage due to flooding. Plate 4 shows the water surface profiles of the modeled stream for the TSP.

8 INTERIOR DRAINAGE

Along both banks of the Ala Wai canal, there are numerous drainage outlets from the storm sewer system. These outlets will require the installation of flap-gates to prevent water from backing up and inundating areas beyond the floodwalls. During storm events, these flap-gates will close and prevent water from draining into the canal. This will cause residual flood inundation to the areas protected by the floodwalls. This residual flooding is not expected to be significant due to it being shallow sheetflow and not dep ponding.

To determine impacts of storm drain or storm sewer outfalls being shut during periods of high water surface elevation in the Ala Wai Canal with the project floodwalls, stormdrains greater than 18-inch diameter pipes along the Ala Wai Canal (21 out of 43 outfalls) were analyzed for backwater impacts. Only the larger stormdrain pipe or culvert sizes were chosen for this analysis since these stormdrains had drainage areas larger than 4 acres and pipe sizes greater than 18-inches. Many of the small outfalls only drain the Ala Wai Boulevard roadway, those of single 18-inch or less diameter pipes (22 out of 43 outfalls), have very minimal drainage areas, will only have minor impacts to Ala Wai Boulevard in case of backwater, and would have only minor residual damages if any.

To evaluate the interior flooding due to backwater in gravity storm drains, the coincident frequency assumption is that the interior flooding input for these gravity outlets to the canal would use the 10% 1-hour rainfall intensity volumes and given the flashy nature of runoff in the watershed, the gates could be closed for up to 6 hours depending on the riverine flood event. Next the pipe and channel storage; i.e. volume capacity; for those stormdrain outfalls were determined based on the pipe or culvert sizes and lengths. Then using existing topographic data to determine the street elevation of stormwater inflow grates and overbank conditions, excess volumes which exceeded the storm drain capacities were mapped assuming that the shallow flooding, up to 1 foot depth, would spread following the local topology with roads serving as the primary channels of this excess backwater flow. Figure 15 illustrates the shallow flooding areas, based on 1 foot depth, for the storm drains analyzed.

As part of this interior drainage analysis, similar analyses were done at each of the pump station locations assuming the pump station did not exist or was not functioning. For these locations, the excess volume was much greater than for the other stormdrains due to larger drainage areas. Additionally for the economic justification of each pump station, multiple storm events were analyzed to determine frequency-stage curves for each impacted area. For the pump stations, the flooded area was determined using the TIN/Lidar data in ArcGIS to help determine the depth/area/duration of interior flooding which would be converted to economic damages.

For Pump Station 1, located at the east end of the Ala Wai Canal, the large drainage area of 290 acres generates an excess volume of 31 acre-feet for the 10% ACE storm event. This excess volume would first flood Kapahulu Avenue and it was assumed that the flooding would follow the streets first before spreading out. The potential inundation area would flood the Ala Wai library and Thomas Jefferson Elementary school grounds before moving south to the Kapahulu Avenue stormdrain, where it is assumed that this 10 ft by 5 ft box could help convey this water out to ocean and the remaining floodwaters would continue to move down Kapahulu Avenue towards the ocean. There are low areas towards Kapiolani Park, if floodwaters cross Kapahulu Avenue and enter the park, the park space could hold all 31 acre-ft. The area represented as Pump Station 1 and 2 Flood Area in Figure 15 represents an area of 115 acres, which is sufficient to even hold the 0.2% ACE interior flood volume of 80 acre-feet at less than 1 foot depth. This area represents the Honolulu Zoo area of Kapiolani Park and does not spread out across the park. It is assumed the drainage ditch in the park and the stormdrains along Monserrat Avenue with catch and divert the shallow flooding before it can spread to the entire park. The worst case is shown in Figure 15 as Pump Station 1 and 2 Maximum Flooding Area, which does include the entire park. That would be improbable as the 0.2% ACE interior storm events for both Pump Stations 1 and 2 generate an excess volume of 125 acre-feet, which would not be able to cover the entire 200 acres represented by that area. For Pump Station 2, the excess water would be about 3 feet deep at the base of the golf course levee and less than a foot deep over Kapahulu Avenue at Herbert Street. The potential floodwaters would run south along Kapahulu Avenue down to Ala Wai Boulevard, where it would presumably pond in the low area by the library or continue to travel down Kapahulu Avenue and also potentially enter the park area as described above, although the excess volume for this location of 19 acre-feet is much less that that from Pump Station 1.

Pump station 3 is located by University Avenue and drains an area of 125 acres. Excess volume for the 10% ACE storm event is about 12 acre-feet. Flooding would begin on Hihiwai Street and there would be 1 foot deep flooding at the Ala Wai Elementary School, community gardens, and possibly also onto the campus of Iolani School and nearby apartment and residential buildings (Figure 15). As discussed for Pump Satations1 and 2 previously, a frequency-stage curve was computed for this pump station flooded area as well for the economic justification.

Figure 15 shows the expected residual flood inundation resulting from the closing of the flap-gates and the potential power loss of the pump stations. The economic analysis will determine if the pump stations are economically justified. Other discussions on the interior flooding situation, residual risk, and floodwall superiority can be found in the main report.

9 Floodwall Resilience and Overtopping

In the main feasibility report, Section 8 presents a resiliency discussion. Specific to this appendix, resiliency refers to how well the system performs in case of capacity exceedance or overtopping on the floodwalls. Resiliency was incorporated as a structural measure into the floodwall design by constructing a scour protection apron, as a concrete sidewalk, on the protected side of the floodwall for the purpose of minimizing erosion during flood events that exceed the top of wall elevation. The design floodwall height is not expected to be exceeded until a flood event greater than 0.2% ACE (500-yr) occurs. An extrapolation of the water-surface elevation data by frequency flood event is shown in Figure 16. This figure shows that an event of about 0.13% ACE (750-year) is needed for the floodwall to be overtopped.

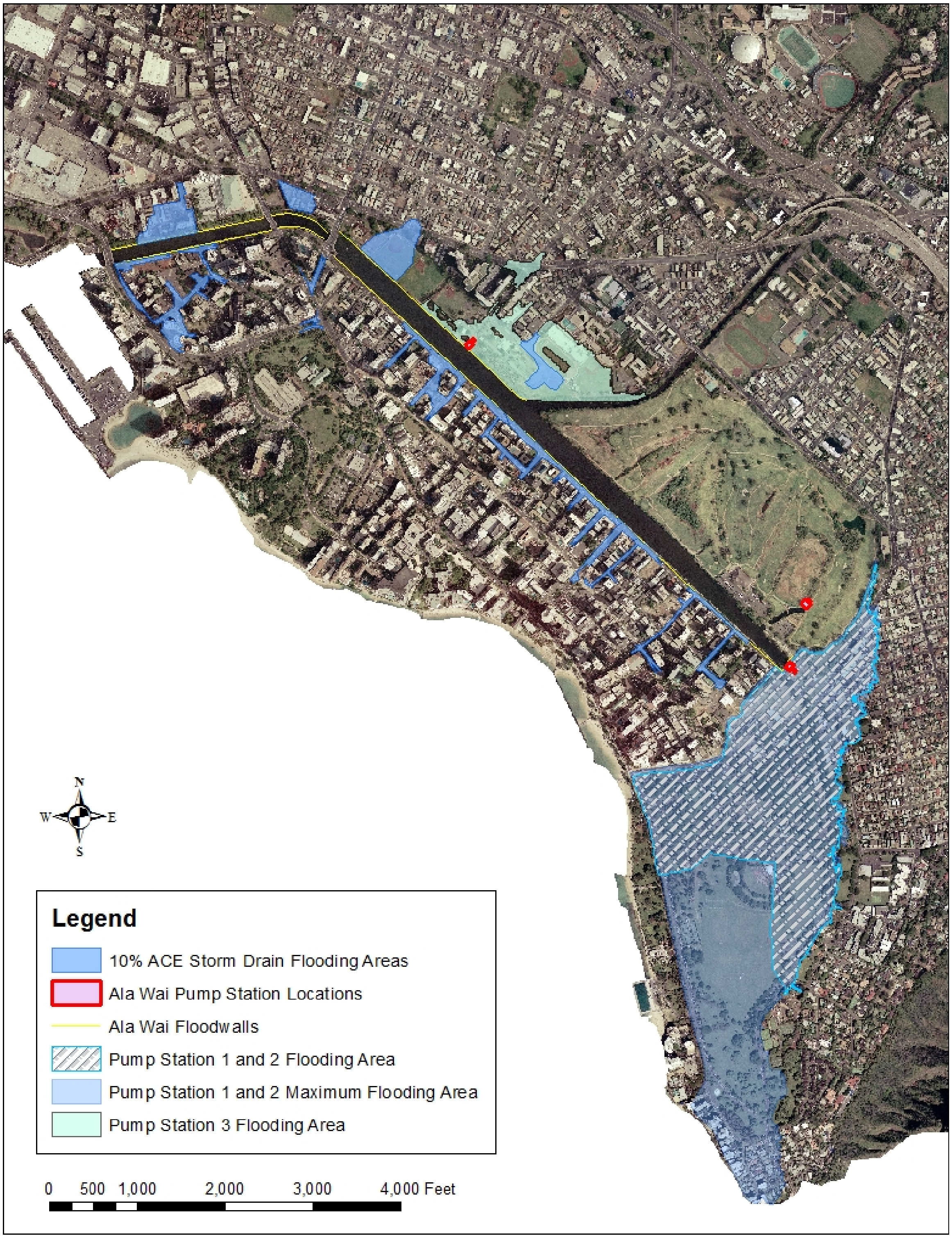

Figure 15. Residual Flood Inundation

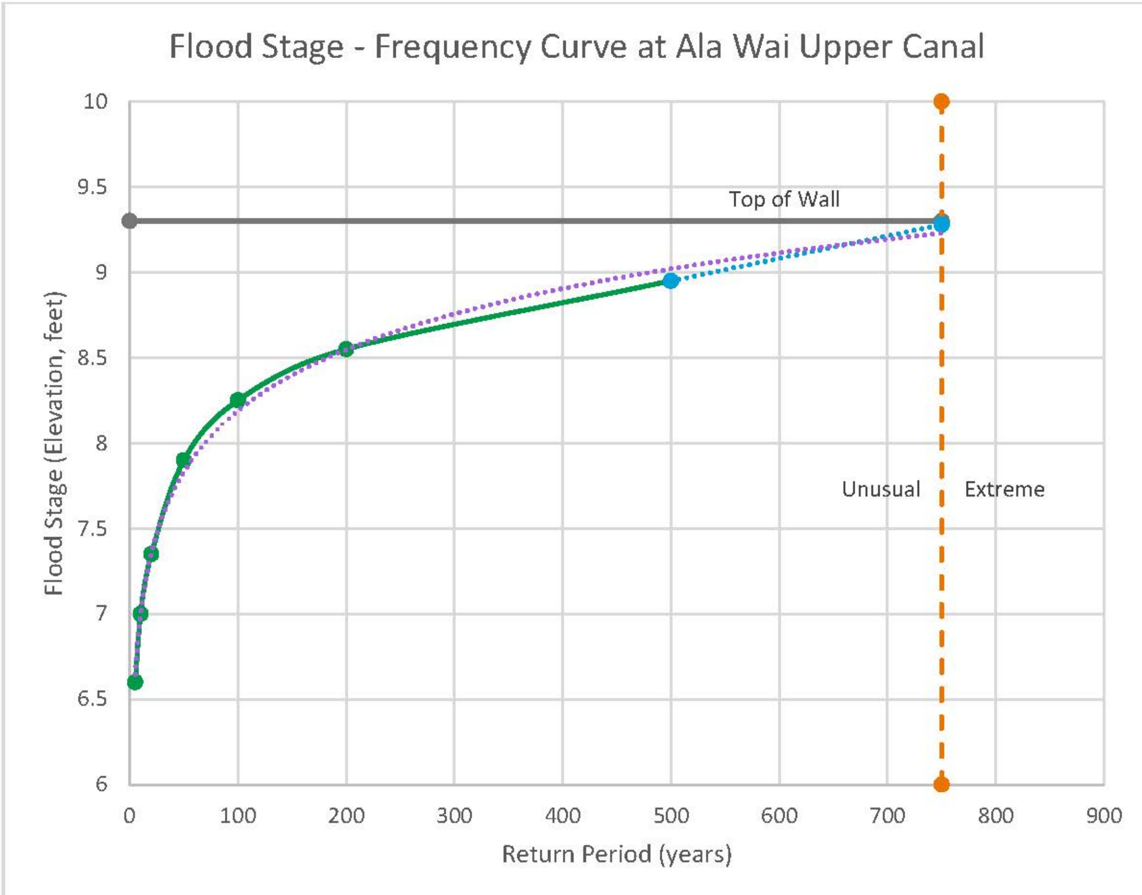

Figure 16. Extrapolated Flood Stage Frequency Curve for Upper Ala Wai Canal Floodwall Reach

10 Coastal Surge, High Sea Level Rise, and Adaptability

The floodwall height values were determined and justified by the optimization process in the National Economic Development (NED) plan development process as described in the main feasibility report and Economic Appendix. This determination was based on riverine flooding and the 2075 intermediate sea level rise backwater condition. To account for coastal surge events, flood wall heights determined for the riverine storm events were also checked versus the potential coastal surge values possible in the Ala Wai Canal regardless of a riverine storm event and also in coincident with a riverine event. This section also discusses floodwall height performance under high sea level conditions out to the year 2125.

Coastal surge or total water values were computed by Patrick O'Brien, P.E., Coastal Engineer Technical Expert with the Climate Preparedness and Resilience Community of Practice. These values, shown in **Table 22**, were computed from tidal data at the Honolulu harbor tide gage.

Table 22. Coastal Surge or Total Water Level values with Sea Level Rise in Feet, Mean Sea Datum

		Intermediate Sea Level Rise			High Sea Level Rise		
ACE (%)	Recurrence interval (year)	2025	2075	2125	2025	2075	2125
100	1.01	1.57	2.33	3.54	1.88	4.27	8.52
50	2	1.88	2.64	3.85	2.19	4.58	8.83
20	5	2.13	2.89	4.10	2.44	4.83	9.08
10	10	2.33	3.09	4.30	2.64	5.03	9.28
5	20	2.56	3.32	4.53	2.87	5.26	9.51
2	50	2.90	3.66	4.87	3.21	5.60	9.85
1	100	3.20	3.96	5.17	3.51	5.90	10.15
0.5	200	3.54	4.30	5.51	3.85	6.24	10.49
0.2	500	4.10	4.84	6.05	4.39	6.78	11.03

Note: Boundary Condition Honolulu/Waikiki Harbor TWL = MHHW + NTR (feet MSL)*
*MSL based on 1983-2001 NTE NOAA 1612340, Honolulu, HI

In comparing the 2075 intermediate and high sea level, 1% ACE total water levels to the floodwall heights determined by the NED process, it is shown in **Figure 17**, the intermediate 2075 case water level is below the floodwall heights for all the Ala Wai Canal reaches. However, the high 2075 1% ACE total water level is higher than the NED determined floodwall heights only at the lower Ala Wai Canal reach, ALA 1. Thus, a coastal event of this magnitude would still cause flooding in the lower Ala Wai Canal reach. To counter this coastal flood risk, the floodwall in the lower Ala Wai Canal reach was raised to 7.9 feet, roughly 2 feet high that the 1% ACE total water level. Note that this total water level does not account for any surge attenuation due to the nearshore coastal bathymetry or reefs. **Table 23** documents the average height change at ALA 1 compared to **Table 20**.

Table 23. Adjusted Average Elevations and Heights of Ala Wai Canal Floodwalls to account for Coastal Surge

	Left Bank		Right Bank	
REACH	Ave. Elev.	Ave. Height	Ave. Elev.	Ave. Height
ALA1	7.9	3.2	7.9	3.1
ALA2	8.5	3.3	8.5	3.4
ALA3	9.3	4.4	NA	NA

Note: Elevation in feet above MSL

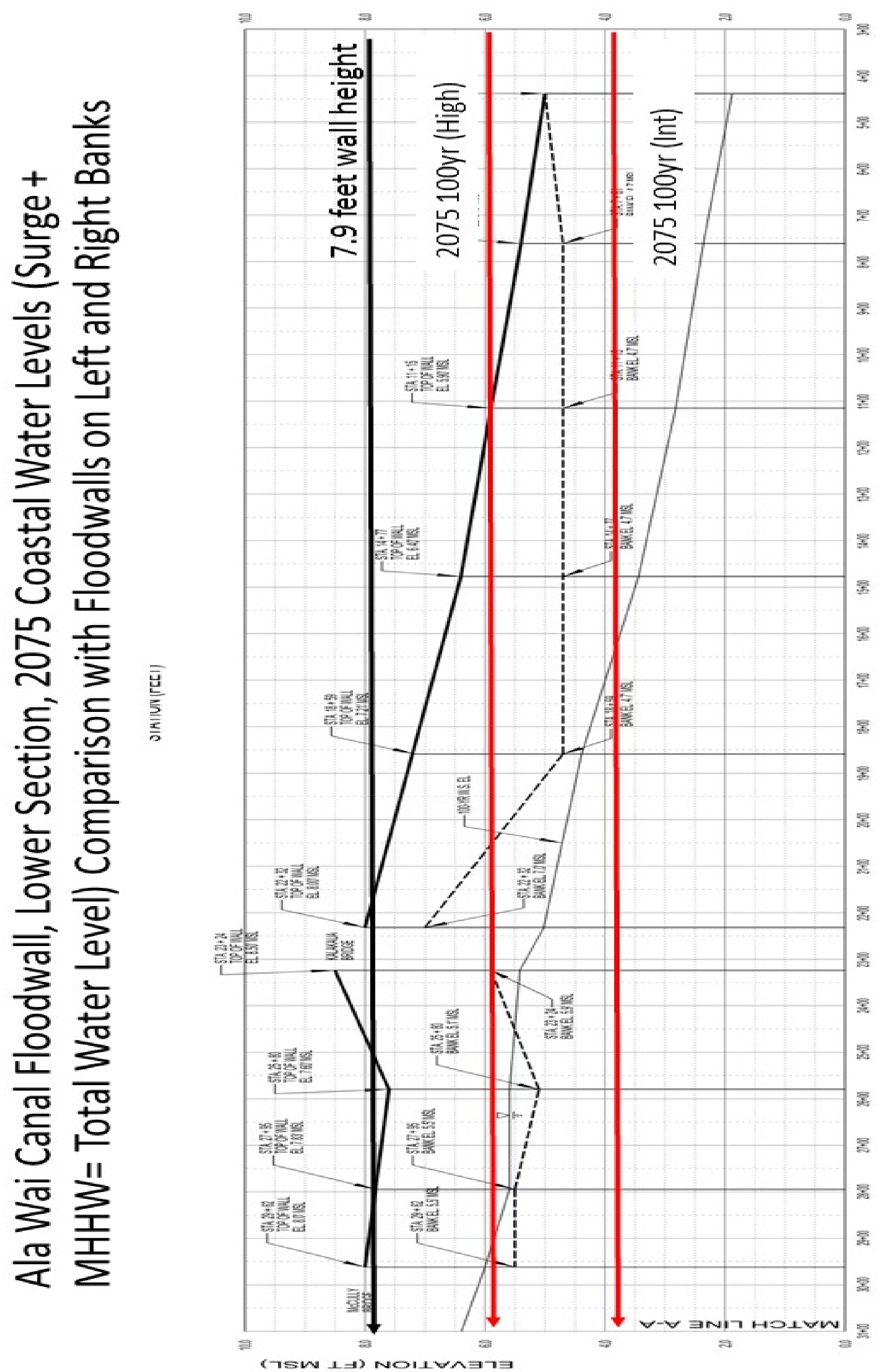

Figure 17. Comparison of 2075 Intermediate and High Coastal Surge Water Level with Floodwall Elevation at the Lower Ala Wai Canal Reach

In addition to coastal surge alone, various coincident scenarios where coastal surge events occur at the same time as riverine events were determined and resulting downstream boundary conditions were computed for these scenarios. Downstream boundary conditions are shown in **Table 24** for these coincident coastal surge and riverine flooding events. All cases have a 1% (100-year) coincident probability, which is the product of the individual probabilities. Case 2.1 is a 10-year (10% ACE) coastal surge for a 10-year (10% ACE) fluvial flow event; Case 2.2 is a 2-year (50% ACE) coastal surge for a 50-year (2% ACE) fluvial flow; Case 2.3 is a 50-year (2% ACE) coastal surge for a 2-year (50% ACE) fluvial flow.

Table 24. Downstream Boundary Conditions for Coastal Coincident Scenarios

Scenario	Recurrence intervals* (year)		Downstream Boundary Condition Elevation 2025	2075	2125
Case 2.1	10c / 10f	Intermediate	2.33	3.09	4.30
Case 2.2	2c / 50f		1.88	2.64	3.85
Case 2.3	50c / 2f		2.90	3.66	4.87
Case 2.1	10c / 10f	High	2.64	5.03	9.28
Case 2.2	2c / 50f		2.19	4.58	8.83
Case 2.3	50c / 2f		3.21	5.60	9.85

Note: Elevation in feet above MSL.
*Recurrence intervals: c = coastal surge, f = fluvial flows.
I.e. Case 2.2 = 2-year coastal surge and 50-year fluvial flows.

As discussed in Section 3.1.7 Boundary Conditions and Appendix 3, sea level rise was added to the HEC-RAS models as an increased downstream boundary condition in order to check the floodwall heights under future conditions. To check floodwall height performance, a number of HEC-RAS and HEC-FDA model runs were made using various backwater conditions to cover the wide range of SLR values and scenarios. The backwater scenarios run are shown in **Table 25** with only the high SLR with coastal surge values being added from the values of **Table 5**. Performance data for these various backwater conditions are shown in **Table 26**.

Table 25. Starting Backwater Conditions in Feet for Floodwall Performance Scenarios, Ala Wai Canal Floodwall

Scenario	Without Project		With Project			
Year	**Low SLR**	**Intermediate SLR**	**Low SLR**	**Intermediate SLR**	**High SLR**	**High SLR with Coastal Surge**
2025	1.64	1.74	1.64	1.74	2.05	----
2075	1.89	2.50	1.89	2.50	4.44	5.60
2125	----	-----	----	3.71	8.69	9.85
SLR = sea level rise; ---- = no scenario run						

Table 26. Project Performance of Ala Wai Canal Floodwalls													
Damage Reach at Index Point	Top of Floodwall Elevation (feet)	1% ACE Water Surface Elevation (feet)	Annual Exceedance Probability		Long term Risk (years)			Conditional Non-Exceedance Probability					
			Median	Expected (mean)	10	30	50	10% (10-yr)	4% (25-yr)	2% (50-yr)	1% (100-yr)	0.4% (200-yr)	0.2% (500-yr)
Intermediate Sea Level Rise 2075 Scenario, Starting Backwater 2.50 feet													
ALA 1	7.9	4.71	0.01%	0.01%	0.10%	0.30%	0.50%	99.98%	99.96%	99.96%	99.96%	99.96%	99.95%
ALA 2	8.75	5.63	0.01%	0.01%	0.15%	0.46%	0.77%	99.99%	99.97%	99.96%	99.96%	99.83%	99.54%
ALA 3	9.3	6.07	0.01%	0.02%	0.23%	0.70%	1.16%	99.97%	99.99%	99.94%	99.84%	99.69%	99.58%
Intermediate Sea Level Rise 2125 Scenario, Starting Backwater 3.71 feet													
ALA 1	7.9	5.09	0.01%	0.01%	0.14%	0.43%	0.71%	99.98%	99.98%	99.96%	99.96%	99.96%	99.95%
ALA 2	8.75	6.34	0.01%	0.05%	0.50%	1.49%	2.47%	99.90%	99.75%	99.64%	99.63%	99.18%	96.82%
ALA 3	9.3	6.52	0.01%	0.03%	0.27%	0.81%	1.35%	99.98%	99.94%	99.72%	99.53%	99.21%	98.85%
High Sea Level Rise 2075 Scenario, Starting Backwater 4.44 feet													
ALA 1	7.9	5.80	0.01%	0.03%	0.33%	0.99%	1.64%	99.93%	99.87%	99.86%	99.86%	99.85%	99.85%
ALA 2	8.75	7.23	0.01%	0.16%	1.60%	4.74%	7.77%	99.52%	98.79%	98.55%	98.53%	97.62%	96.06%
ALA 3	9.3	7.44	0.01%	0.06%	0.64%	1.91%	3.17%	99.89%	99.73%	99.36%	98.69%	97.77%	97.15%
High Sea Level Rise with Coastal Surge 2075 Scenario, Starting Backwater 5.60 feet													
ALA 1	7.9	6.29	0.01%	0.40%	3.98%	11.46%	18.37%	99.25%	98.99%	98.94%	98.90%	98.87%	98.86%
ALA 2	8.75	7.63	0.01%	0.75%	7.26%	20.23%	31.39%	97.47%	95.31%	94.61%	94.55%	93.24%	91.27%
ALA 3	9.3	7.82	0.01%	0.14%	1.43%	4.24%	6.96%	99.84%	98.90%	98.29%	97.60%	96.82%	96.33%

Table 26. Project Performance of Ala Wai Canal Floodwalls--Continued													
Damage Reach at Index Point	**Top of Floodwall Elevation (feet)**	**1% ACE Water Surface Elevation (feet)**	**Annual Exceedance Probability**		**Long term Risk (years)**			**Conditional Non-Exceedance Probability**					
			Median	**Expected (mean)**	**10**	**30**	**50**	**10% (10-yr)**	**4% (25-yr)**	**2% (50-yr)**	**1% (100-yr)**	**0.4% (200-yr)**	**0.2% (500-yr)**
High Sea Level Rise 2125 Scenario, Starting Backwater 8.69 feet													
ALA 1	7.9	8.79	99.90%	87.68%	100.0%	100.0%	100.0%	11.30%	10.88%	10.72%	10.51%	10.32%	10.18%
ALA 2	8.75	8.96	58.29%	51.31%	99.93%	100.0%	100.0%	42.85%	40.90%	39.27%	38.52%	37.63%	37.25%
ALA 3	9.3	9.10	0.01%	25.35%	94.62%	99.98%	100.0%	69.07%	65.65%	62.31%	60.11%	58.62%	57.46%
High Sea Level Rise with Coastal Surge 2125 Scenario, Starting Backwater 9.85 feet													
ALA 1	7.9	9.92	99.90%	99.70%	100.0%	100.0%	100.0%	0.14%	0.14%	0.12%	0.12%	0.12%	0.12%
ALA 2	8.75	10.01	99.90%	94.18%	100.0%	100.0%	100.0%	4.56%	4.20%	3.73%	3.57%	3.44%	3.36%
ALA 3	9.3	10.12	99.90%	80.25%	100.0%	100.0%	100.0%	17.34%	15.55%	13.24%	11.91%	11.20%	10.55%

In **Table 26**, the intermediate 2075 SLR scenario is the base scenario representing the NED plan. At these floodwall elevations, the annual exceedance probability (AEP) is close to 1 exceedance in 1000 years at all three index points. Long term risk remains below 1% except for the 50-year periods at damage reach AA 3, where there is a roughly 1.2% chance of at least one overtopping event in 50 years. The conditional non-exceedance probability (CNP) is very high. At the ALA 2 index point, for the 1% ACE flood event there is a 99.96% assurance of not over-topping the floodwall elevation or alternatively, can be stated as 99.96% assurance of containing the 1% ACE event. This assurance CNP does not decrease much for containing the 0.2% ACE event with 99.54% assurance.

Keeping with the ALA 2 index point as the point of comparison, with the intermediate 2125 SLR scenario, assurance remains high, only decreases slightly to 99.63% for the 1% ACE event and to 96.82% for the 0.2% ACE event. With the high 2075 SLR scenario, assurance decreases to 98.53% for the 1% ACE event and to 96.06% for the 0.2% ACE event. Long term risk rises to a 7.77% chance of one or more overtopping events occurring in 50 years. The 1.4 feet increase in starting backwater between the intermediate and high 2075 SLR scenarios translate to an approximate increase of 1.60 feet in the 1% ACE water-surface elevation at ALA 2 index point.

Performance begins to drop in the high SLR with coastal surge 2075 SLR scenario with a starting backwater of 5.60 feet. Assurance at ALA 2 versus the 1% ACE event is 94.55% with the 0.2% ACE event assurance dropping to 91.27%. The 1% ACE event assurance is at the 95% CNP threshold used as a minimum recommended level of assurance for levee/floodwall projects. The 5.60 feet starting backwater represents the highest backwater condition that provides a high level of assurance. At this starting backwater level, the AEP at ALA 2 rises to 0.75% which means the floodwall elevation is close to matching the 1% ACE water-surface elevation. For this scenario, the 1% ACE water-surface elevation is about 1.2 feet below the top of the floodwall. Long term risk rises to 31% chance of one or more overtopping events in 50 years.

Considering the staring backwater of 5.60 feet as the upper limit of assurance, means that for the majority of coastal coincident scenarios (**Table 24**), intermediate and high 2075 SLR and intermediate 2125 SLR scenarios, there is sufficient assurance in the floodwall heights to provide a level of protection in the event these scenarios occur. In scenarios with starting backwater elevations greater than 5.6 feet, such as the high 2125 coastal coincident (**Table 24**) and the high 2125 SLR and 2125 high SLR with coastal surge scenarios (**Table 25**), the NED floodwall heights do not provide sufficient assurance against overtopping. At the ALA 2 index point for the high 2125 SLR scenario, the computed 1% ACE event water-surface elevation is greater than the floodwall height by 0.21 feet. The resulting CNP assurance is only 38.52%, AEP is 51.31% and long-term risk becomes 100% chance of one or more overtopping events starting at 10 years.

If any of the high 2125 scenarios occur, the floodwalls would provide very little protection and would need to be raised as an adaptation measure to provide protection on the north side of the Ala Wai canal. On the south or Waikiki side of the canal, as discussed in the next paragraph, flooding in Waikiki from SLR and/or coastal surge would negate the need for adaptation of the south side floodwall.

For a discussion on adaptability, SLR conditions with project were computed and mapped. **Figures 18** to **21** show four future scenarios of years 2075 and 2125 with intermediate and high SLR values. Included on these figures are the 1% (100-year) and 0.2% (500-year) floodplains determined by the HEC-RAS model results for each scenario. Sea level rise elevations shown here follows a “low elevation” approach, where any elevations below the designated SLR elevation is shown as holding water. This is regardless of wave action (which would possibly flood more areas) or hydrologic connectivity (which would preclude areas that are not connected to the ocean). From these result maps, it can be seen that the modeled floodwall heights remain effective for retaining 1% ACE (100-year) flood events for the with-project scenario for three of the four scenarios. The highest SLR scenario, year 2125 High, has a SLR elevation of 8.69 feet, and HEC-RAS results show flooding throughout the Waikiki area. However, the majority of this flooded area will experience passive flooding due to SLR, which is not directly linked to flooding of the Ala Wai canal. Thus, in cases except the high SLR 2125 case, floodwall adaptability, in terms of floodwall raise, is not needed. In the high SLR 2125 case, a raise in floodwall would not be sufficient to protect the Waikiki area. Other adaptability measures, such as a coastal barrier, pump stations, retreat or relocation, specific building adaptations, structural fill, and other possibilities, for Waikiki would be needed for that case.

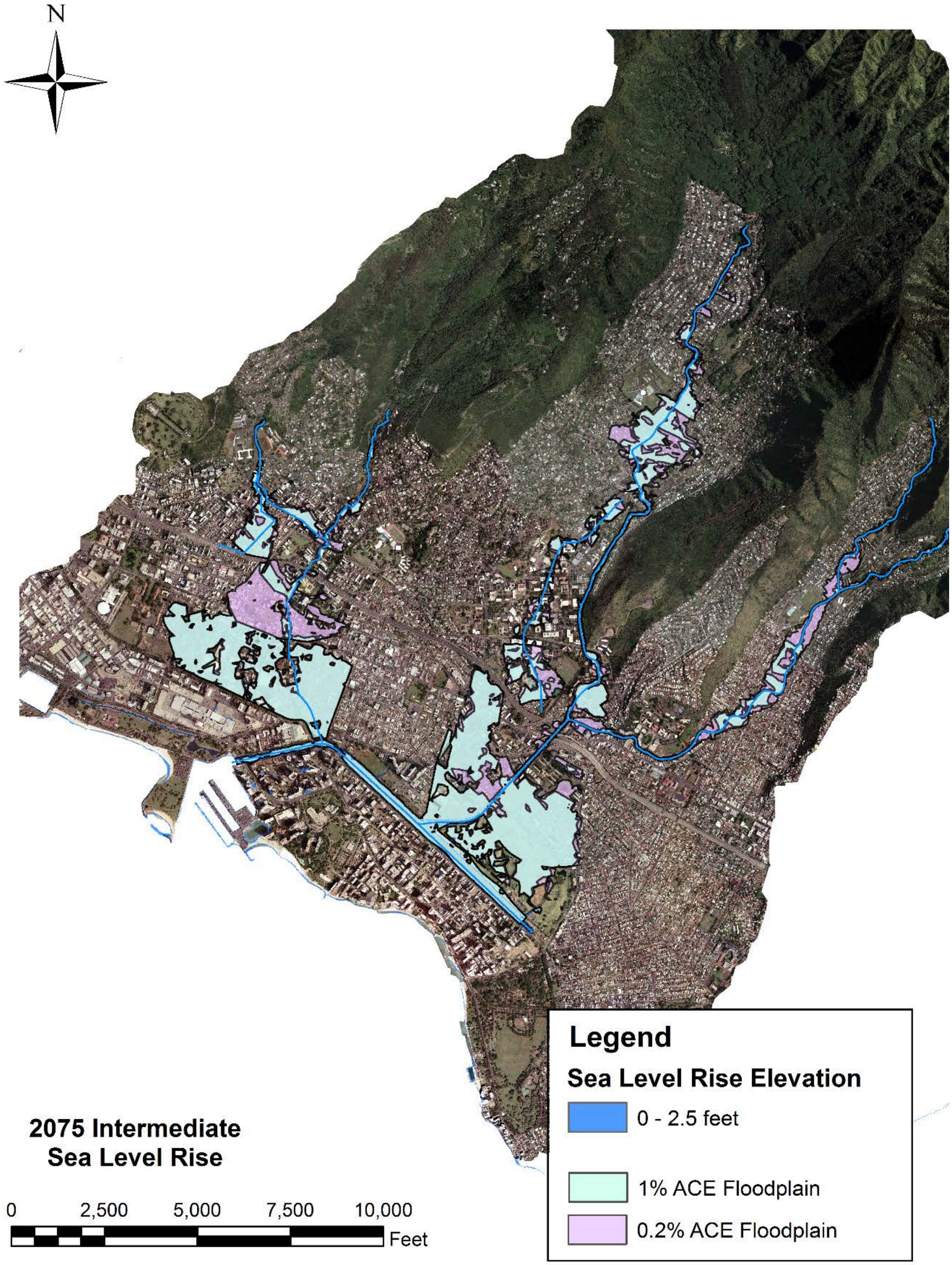

Figure 18. Floodplains for 1% ACE (100-year) and 0.2% ACE (500-year) Flood Events with 2075 Intermediate Sea Level Rise elevations.

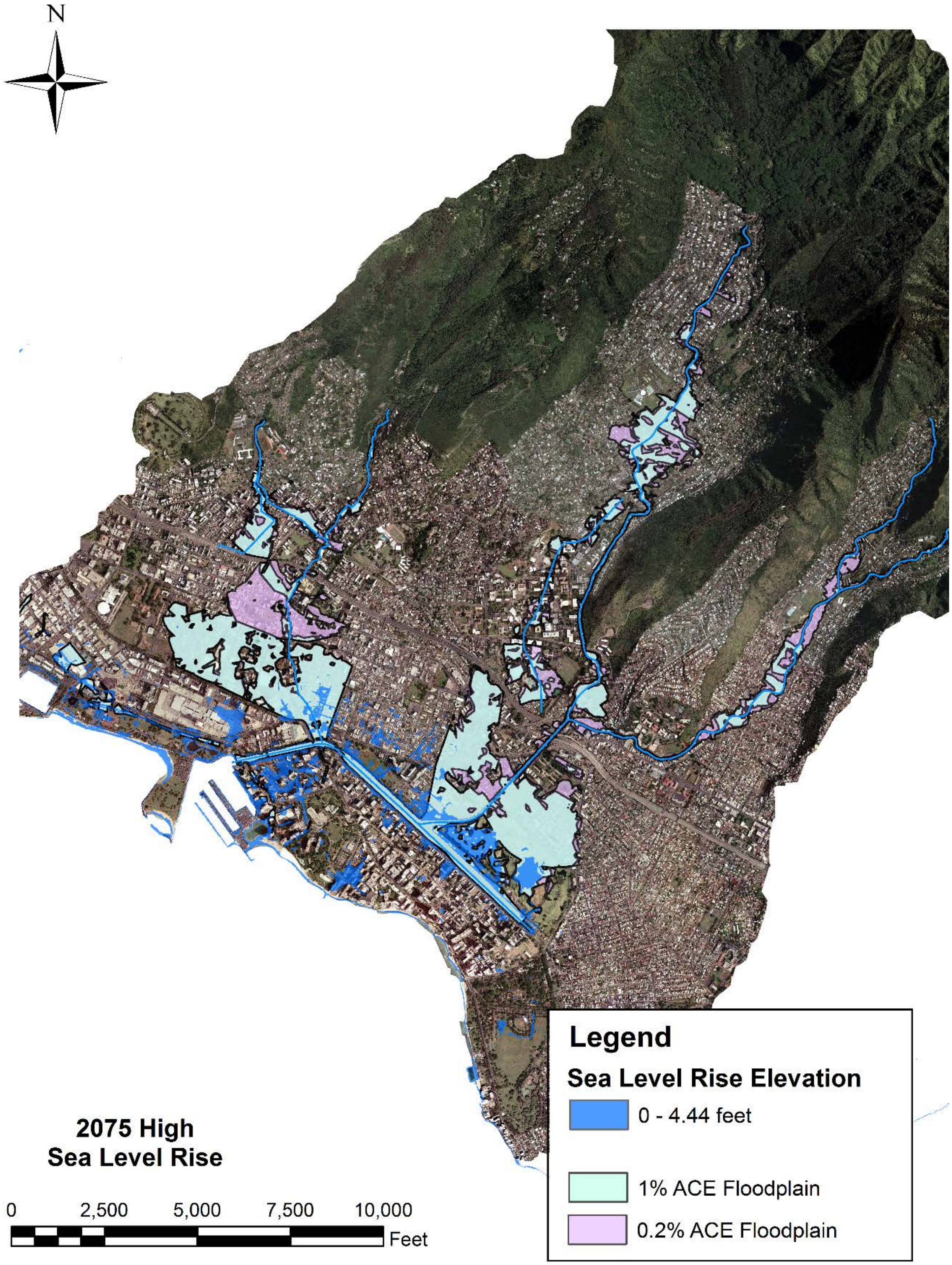

Figure 19. Floodplains for 1% ACE (100-year) and 0.2% ACE (500-year) flood events with 2075 High Sea Level Rise elevations.

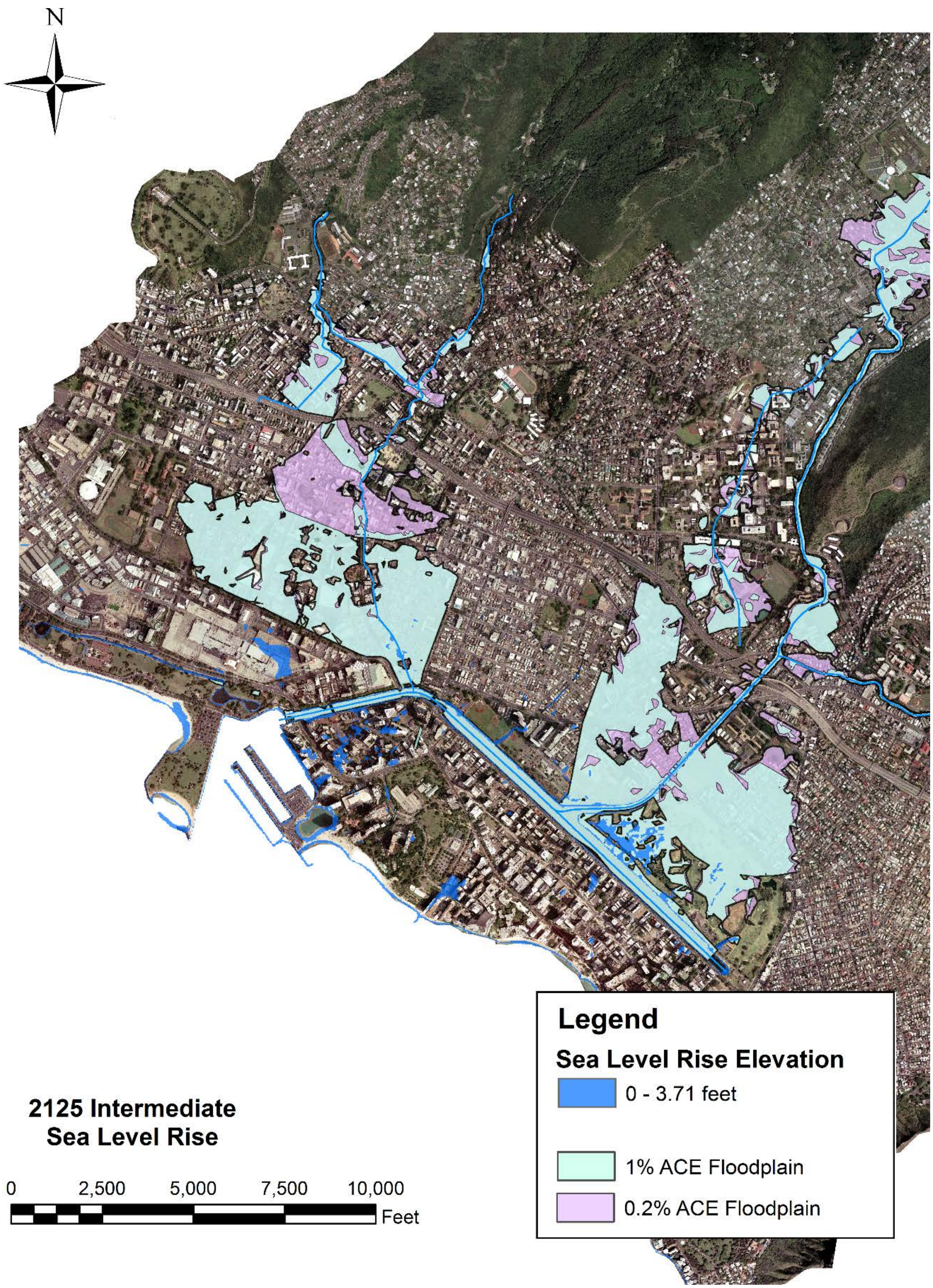

Figure 20. Floodplains for 1% ACE (100-year) and 0.2% ACE (500-year) flood events with 2125 Intermediate Sea Level Rise elevations.

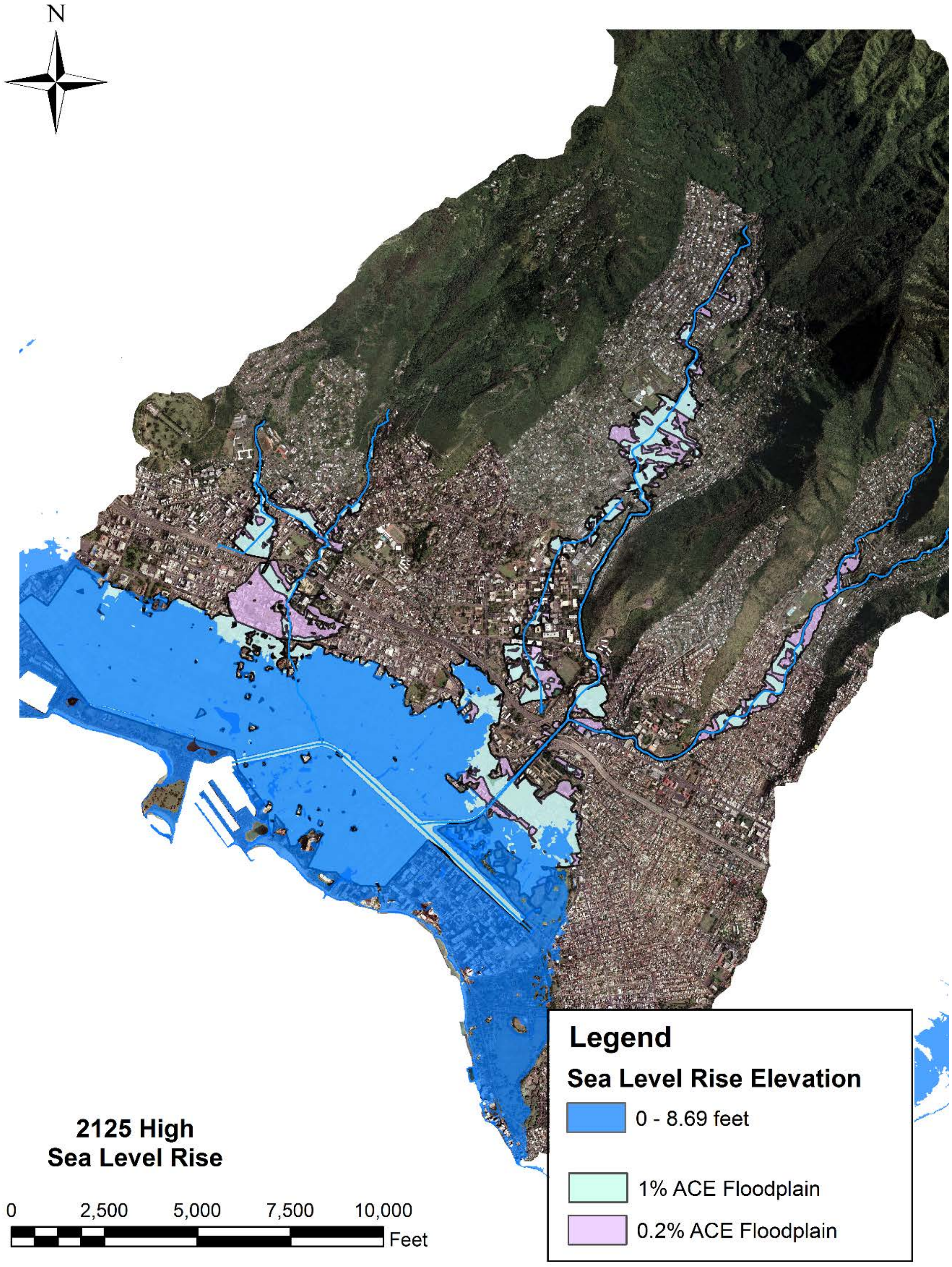

Figure 21. Floodplains for 1% ACE (100-year) and 0.2% ACE (500-year) Flood Events with 2125 High Sea Level Rise Elevations.

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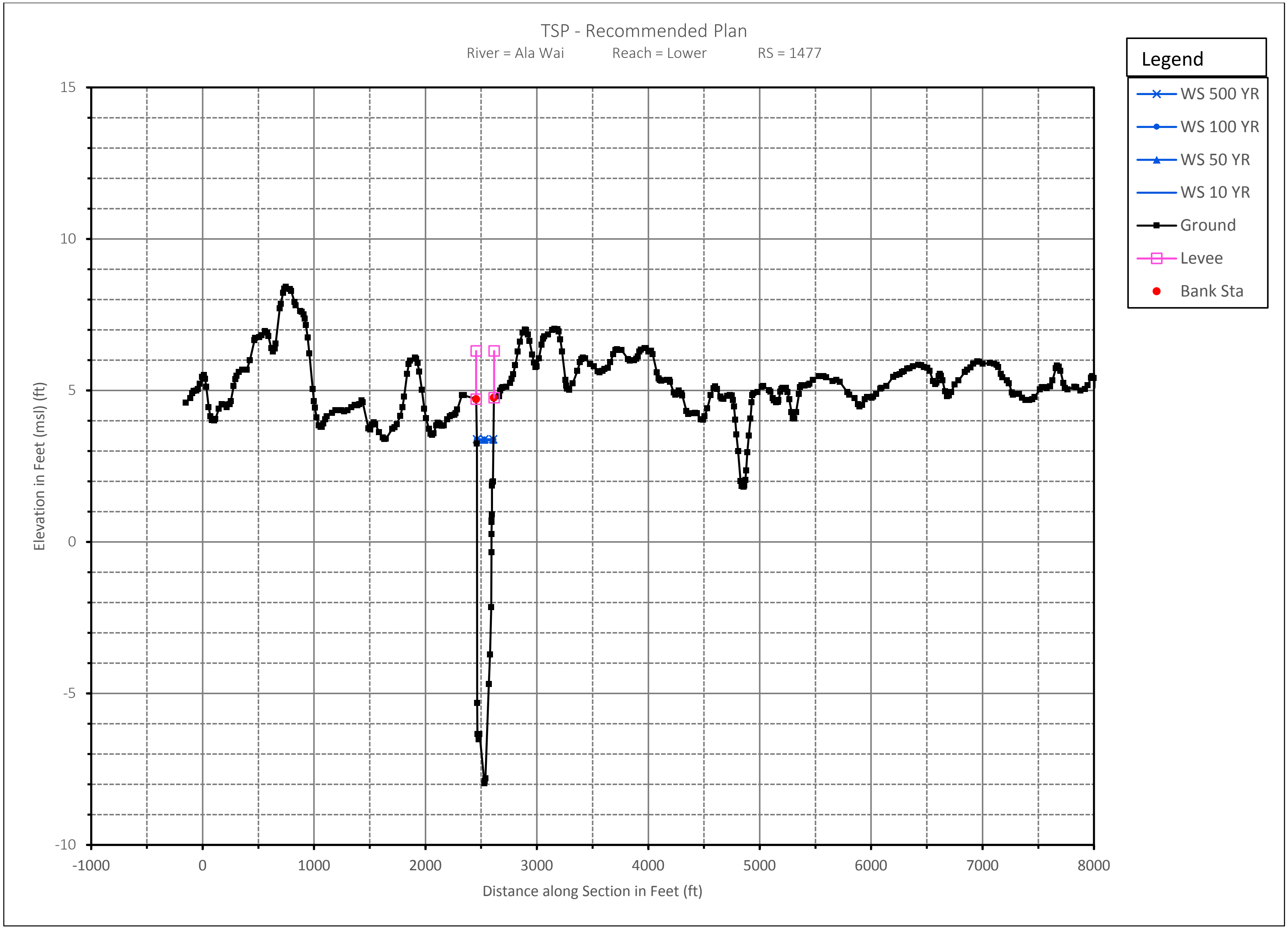
AlaWai Watershed
Flood Risk Management Study
HEC-RAS Cross Section
Locations

Plate 2 HEC-RAS Cross Section Plots for TSP at Index Station Locations



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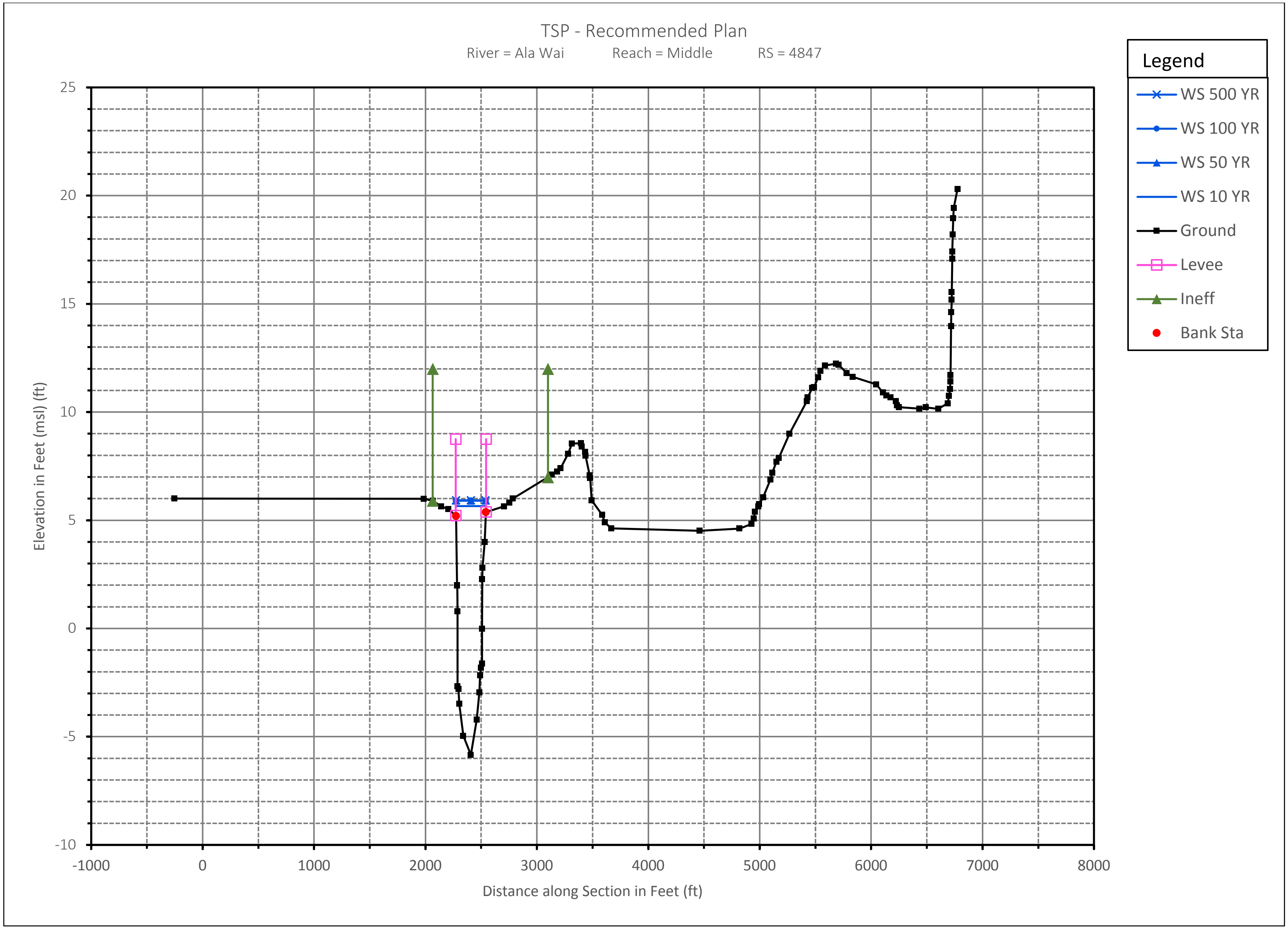

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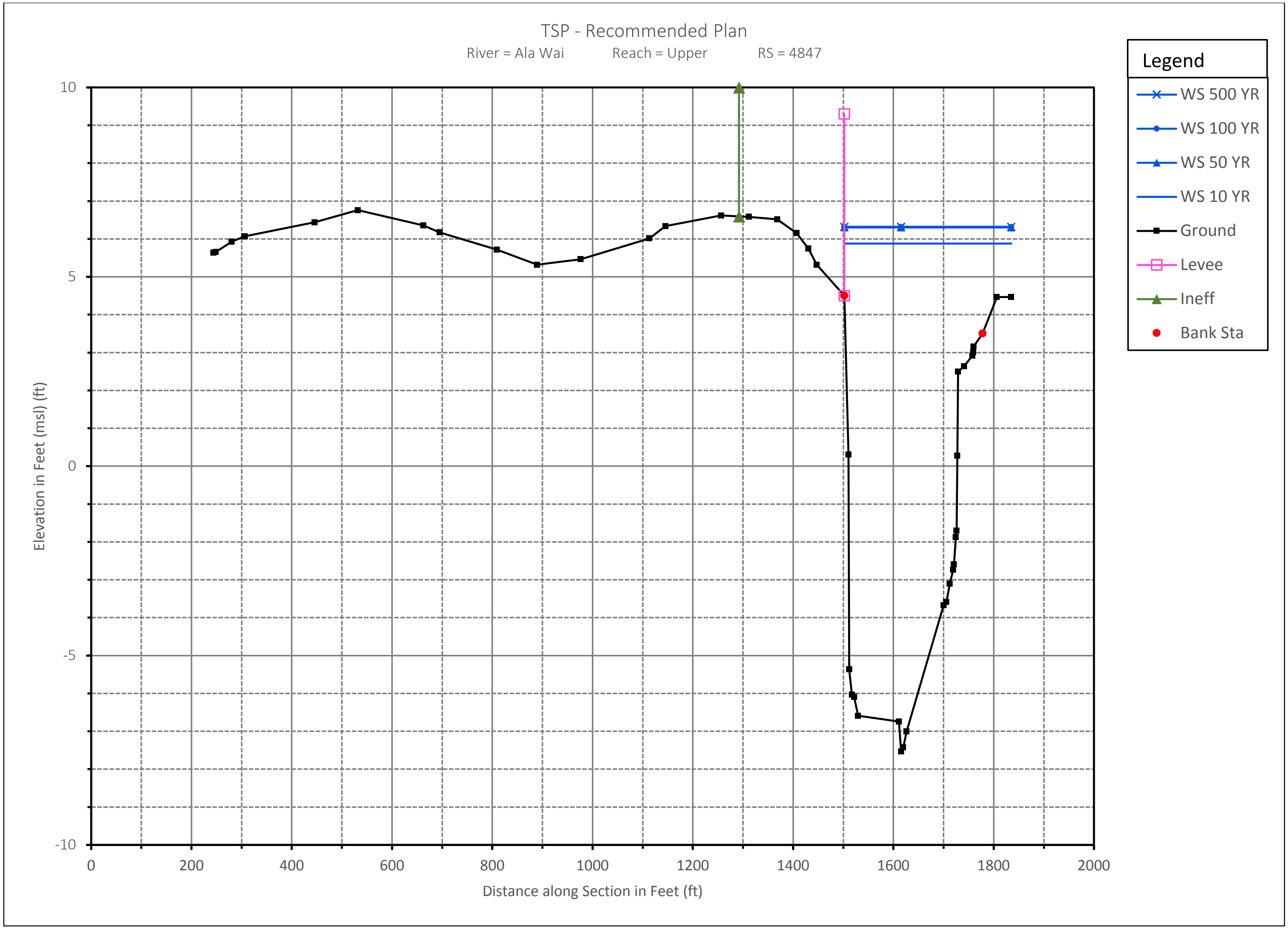

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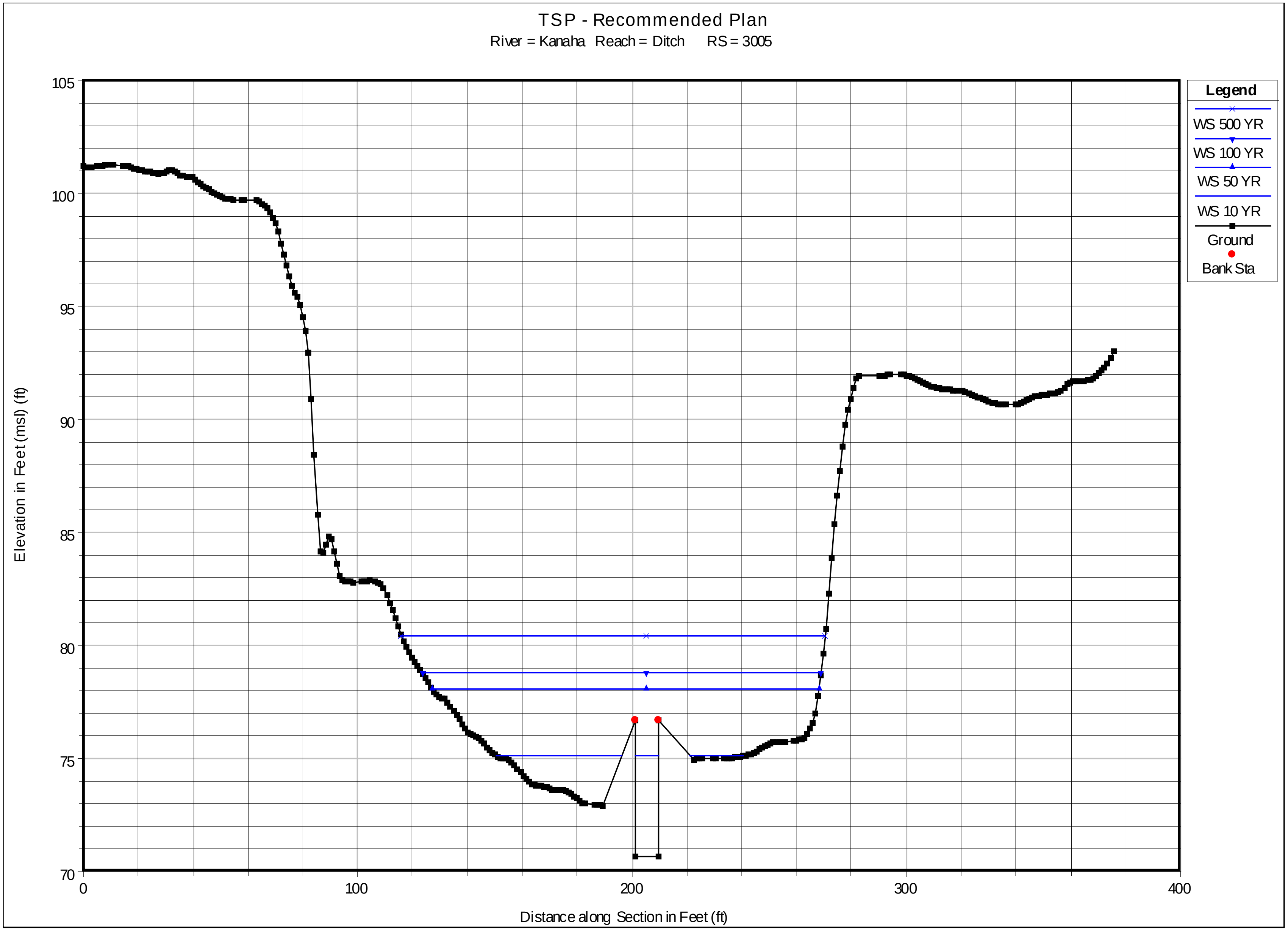

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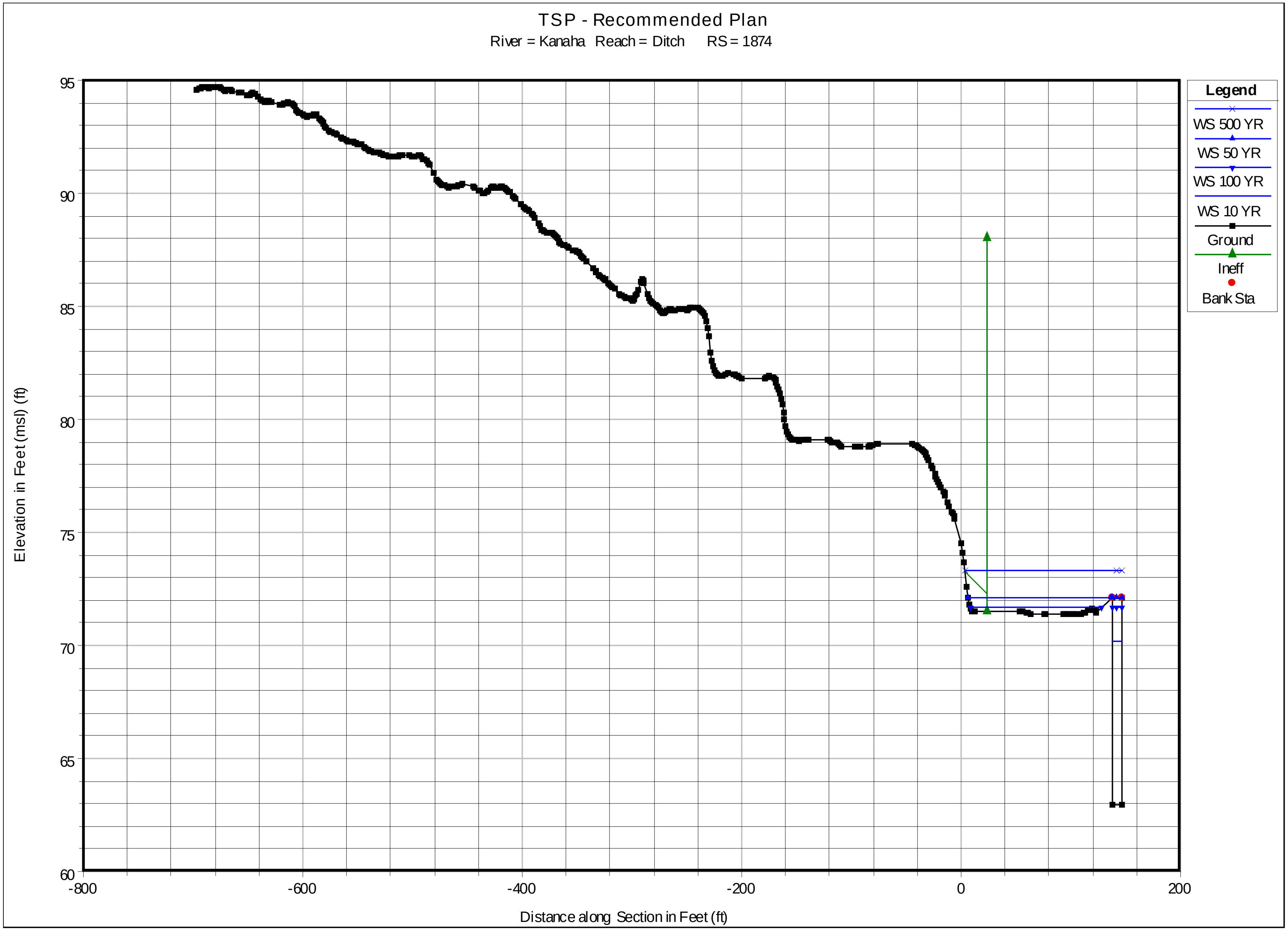

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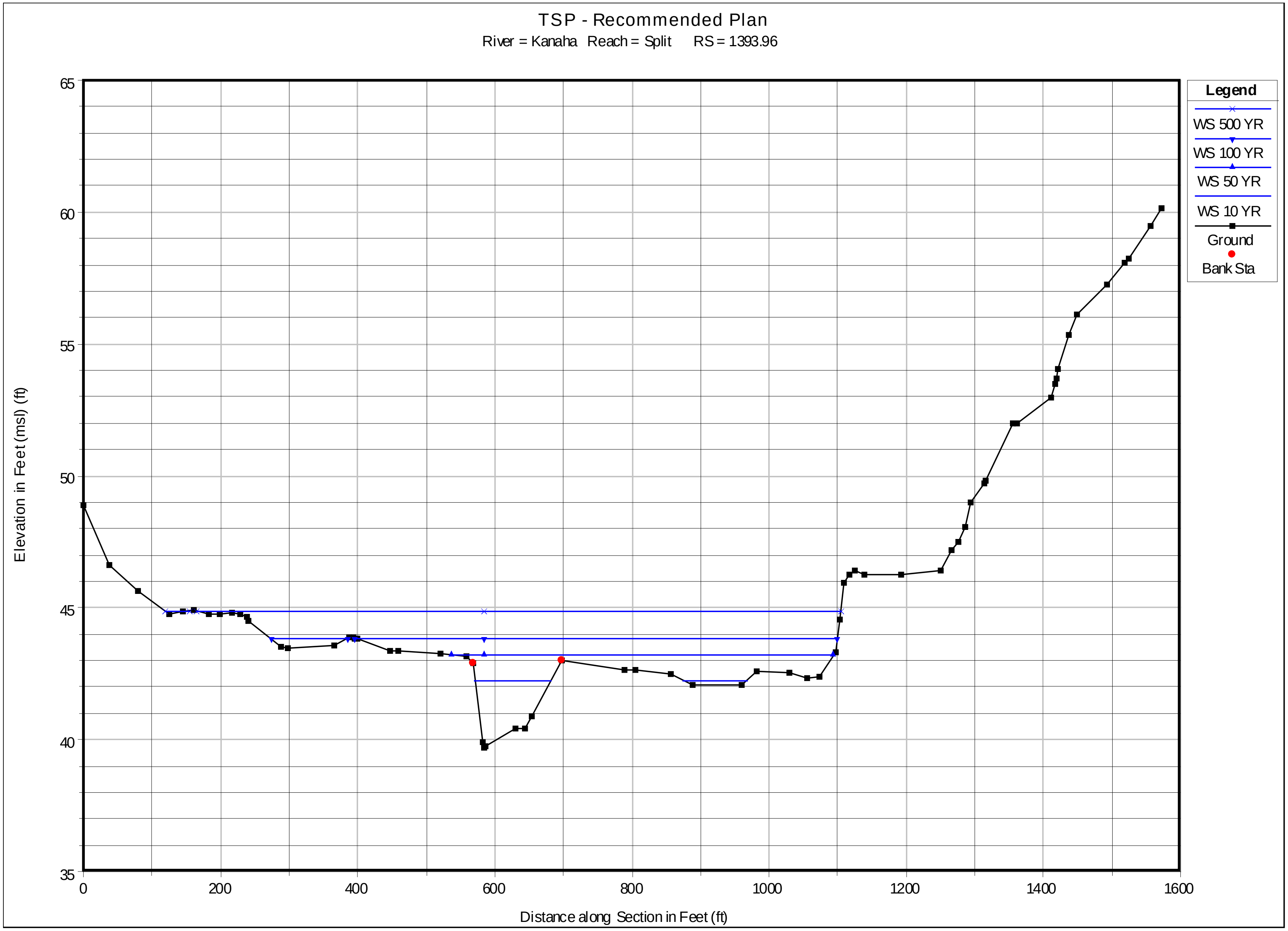

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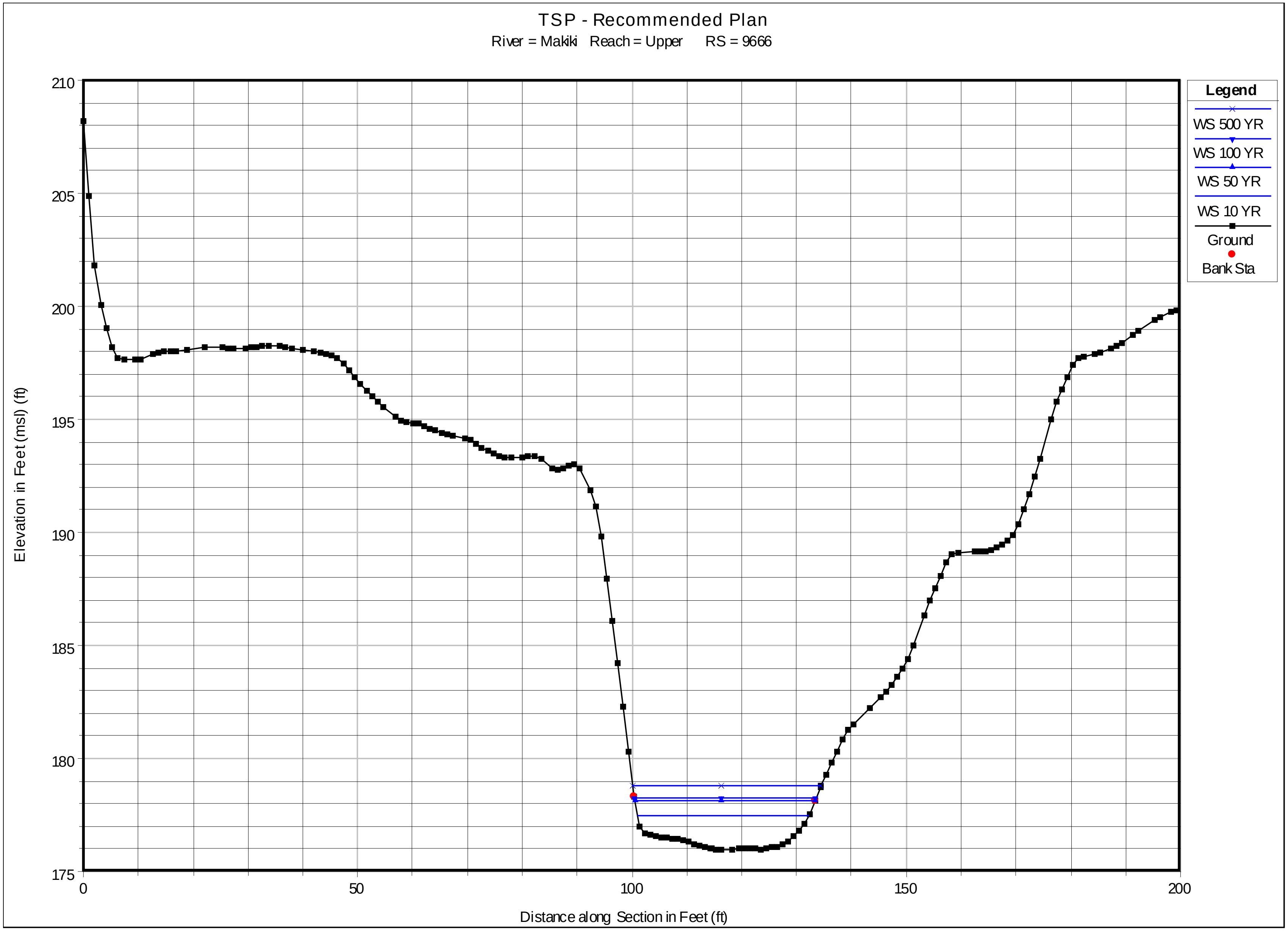

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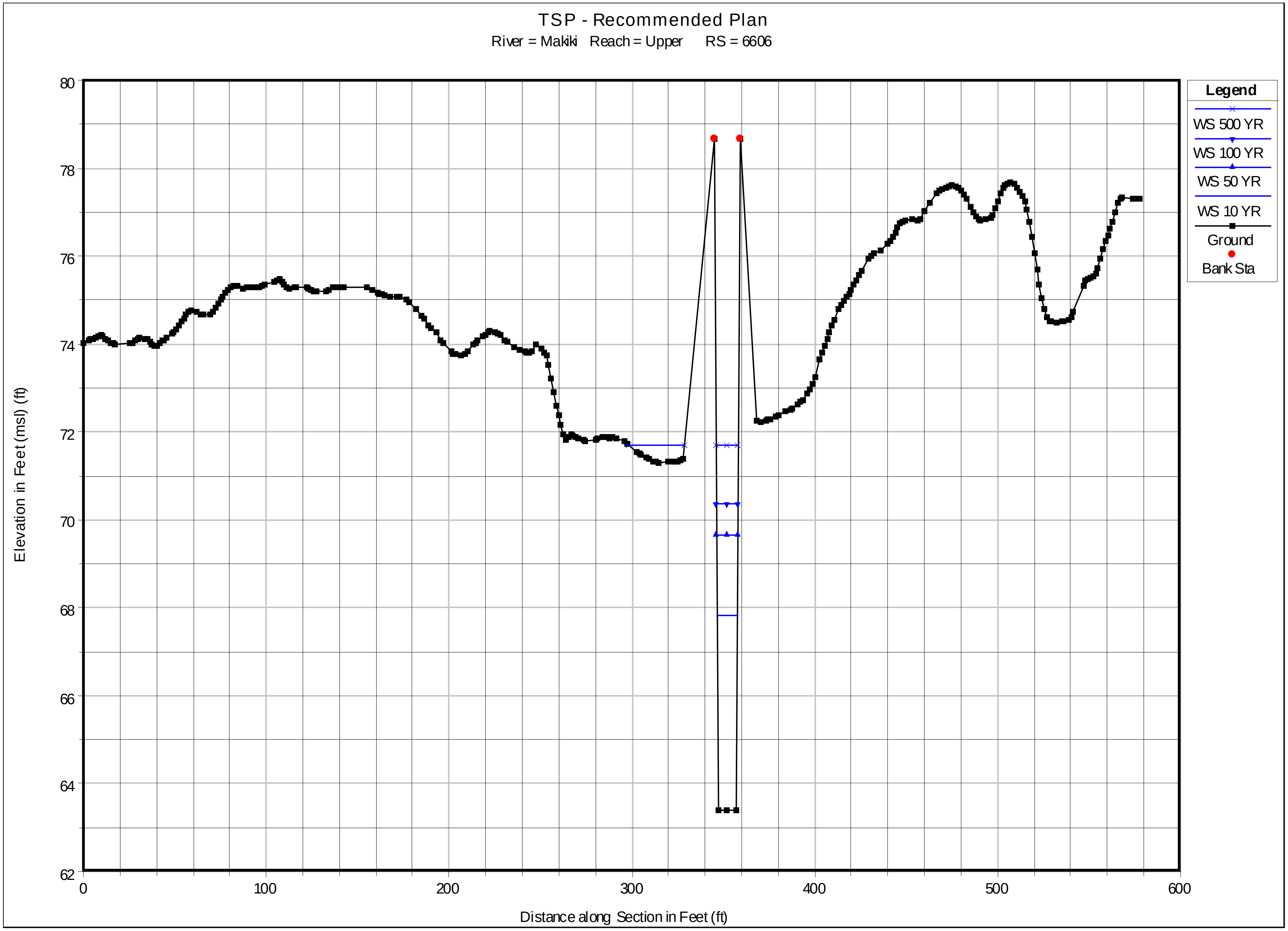

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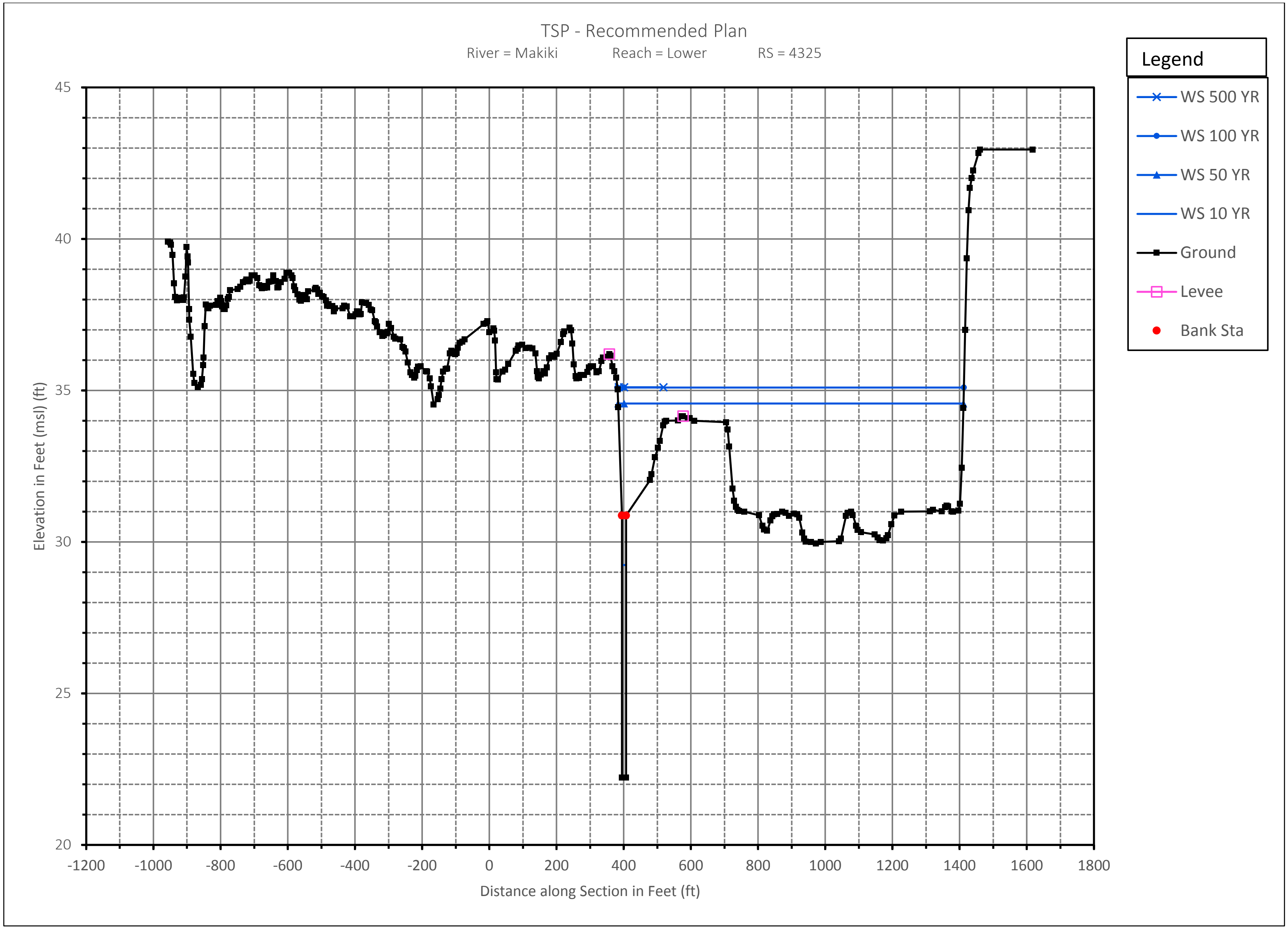

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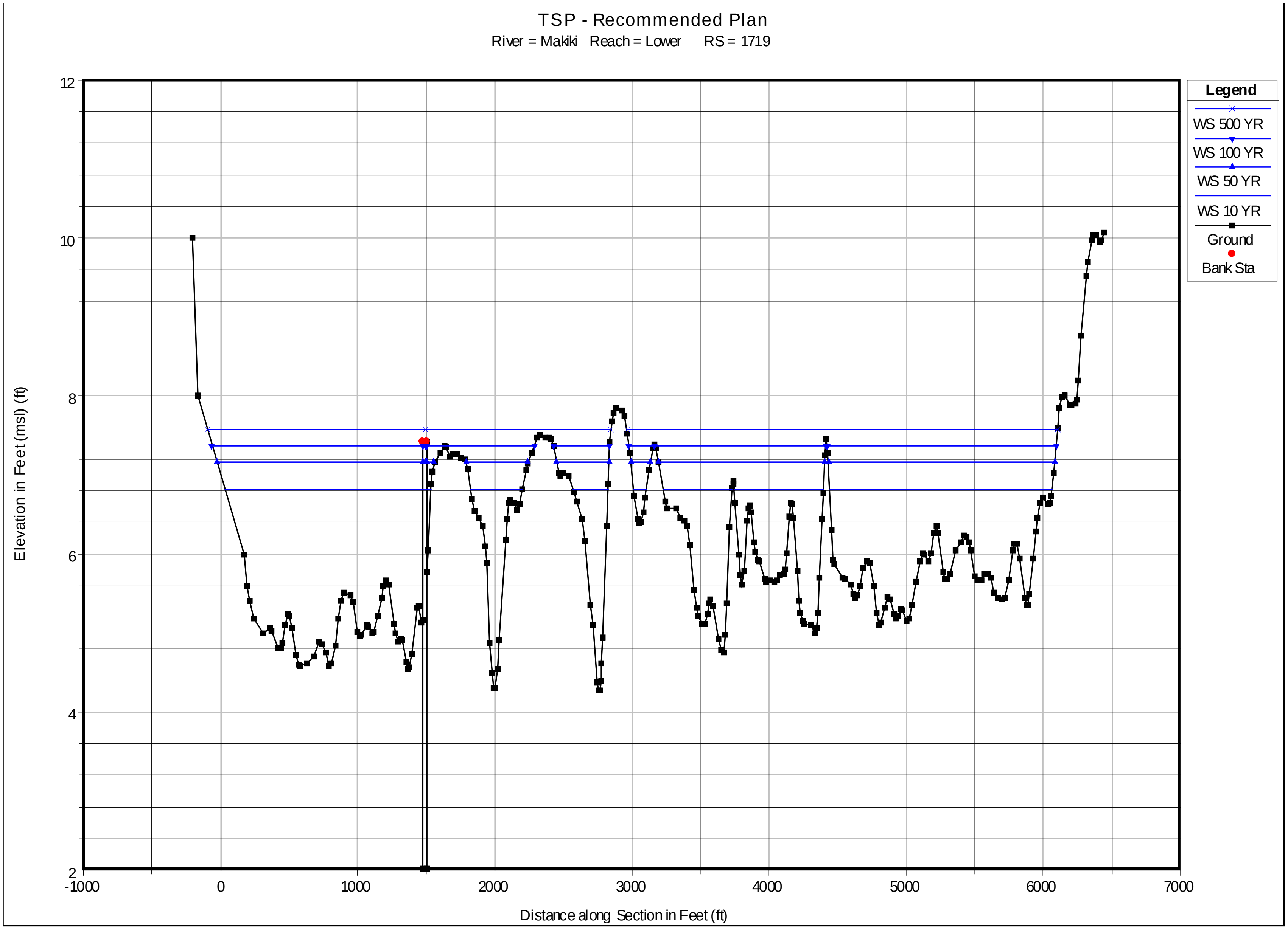

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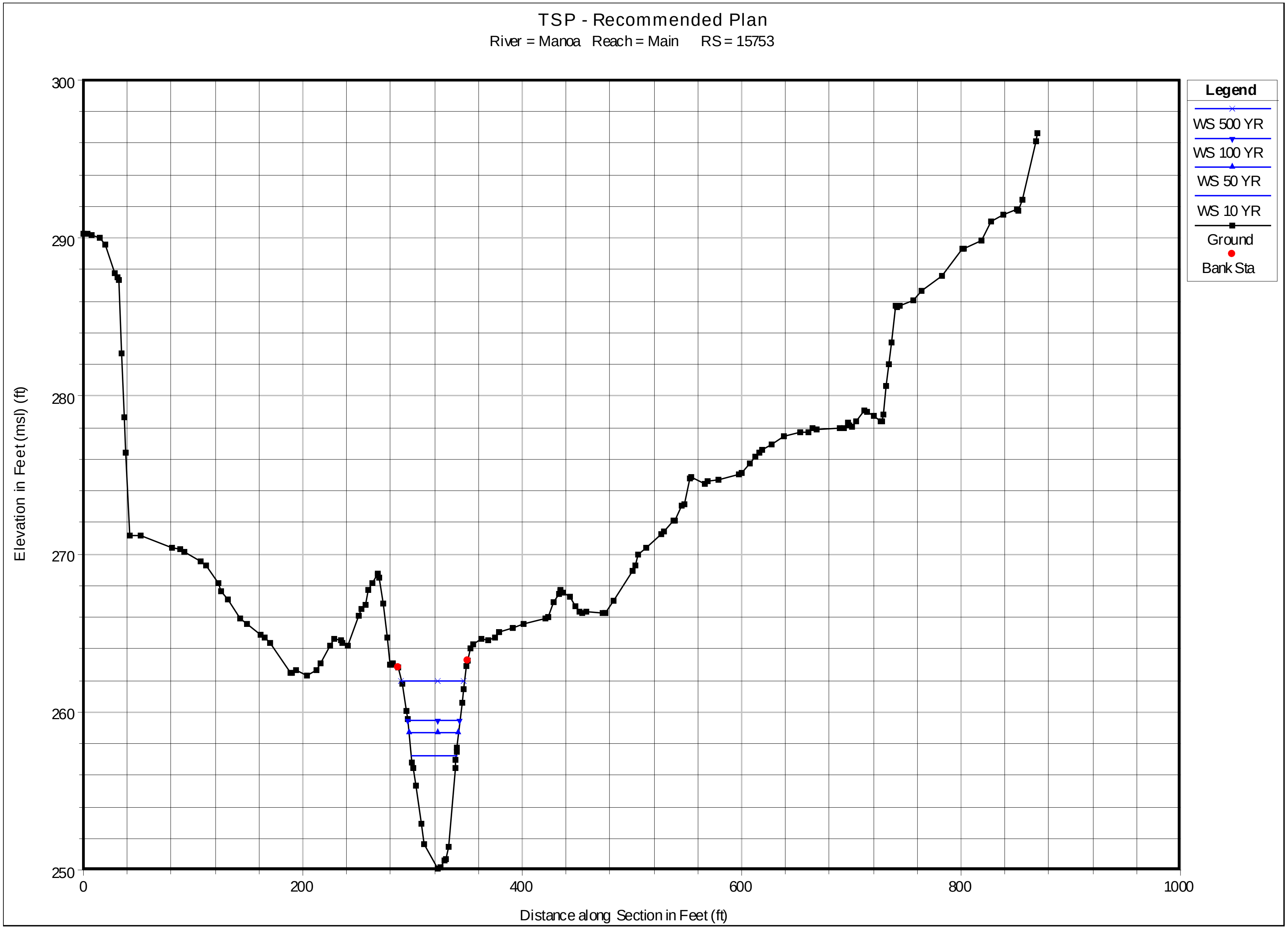

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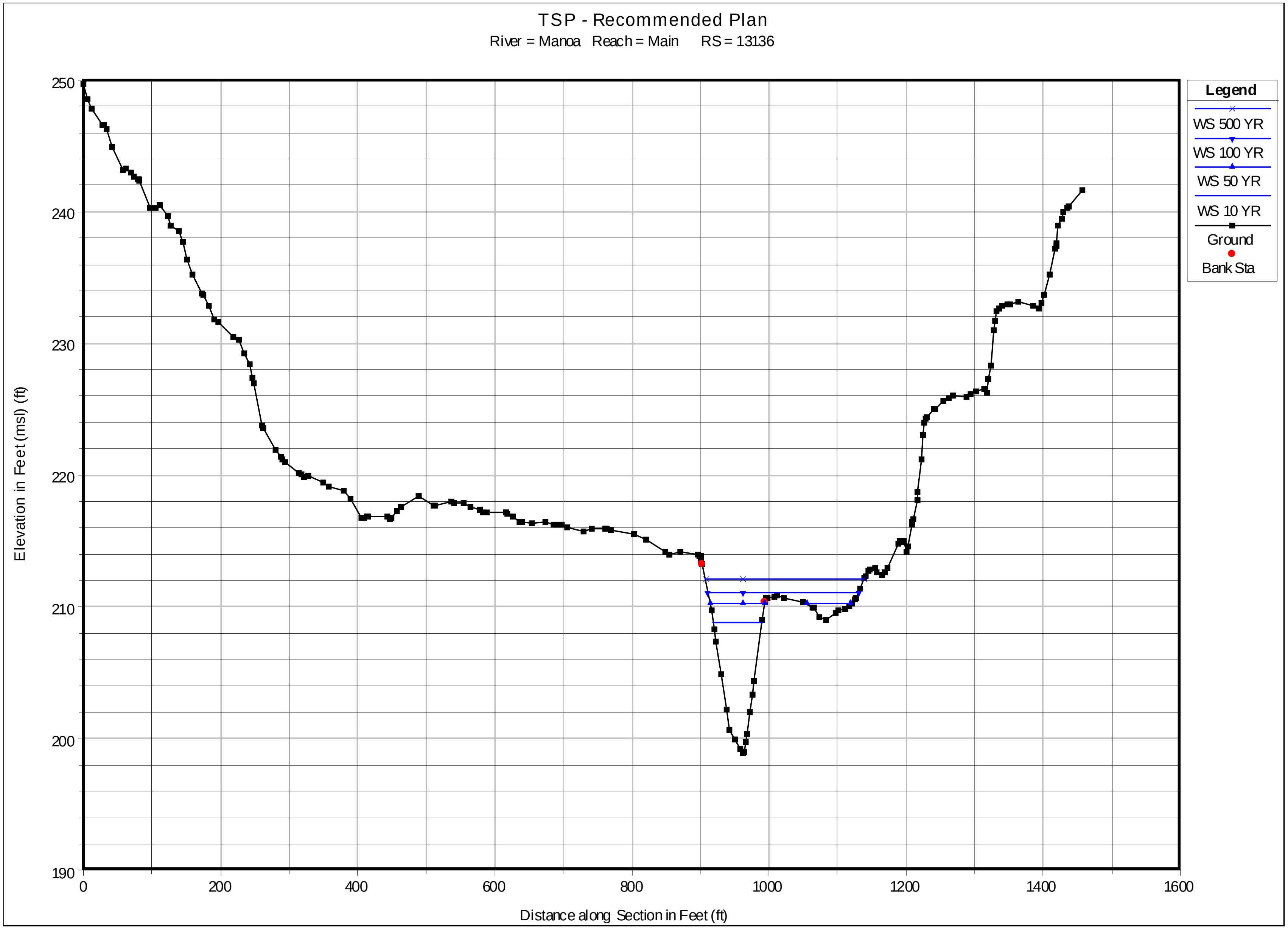

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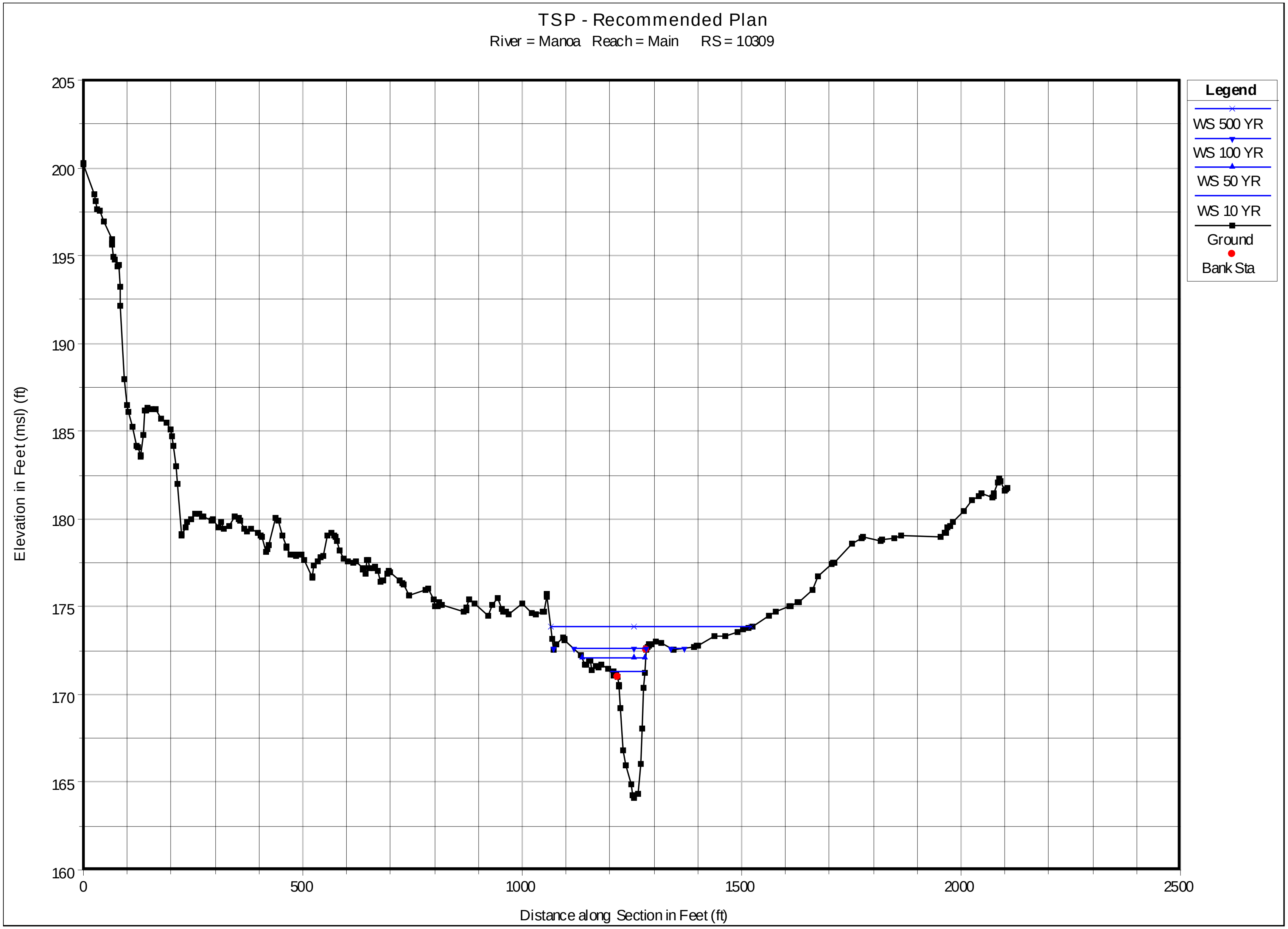

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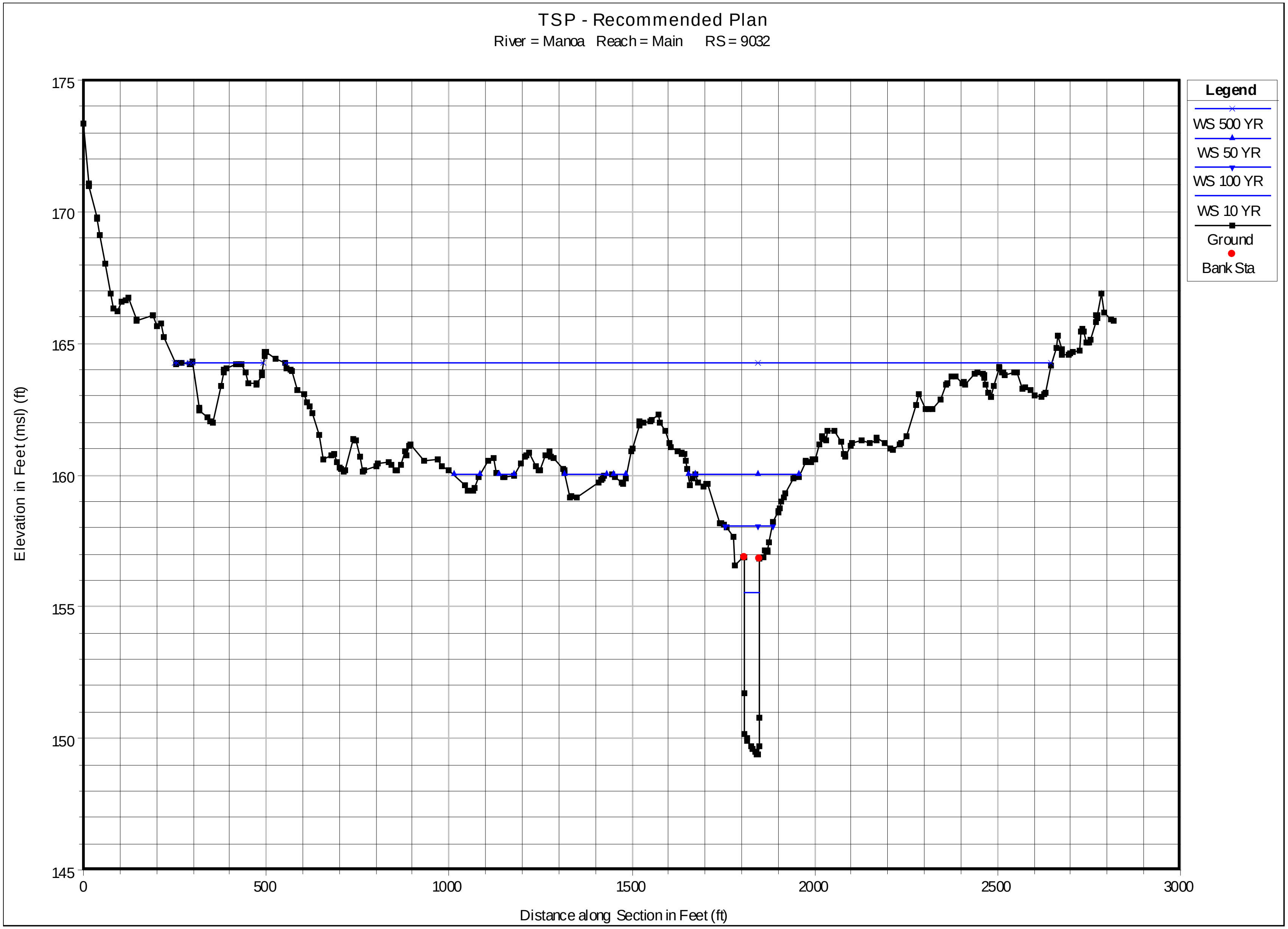

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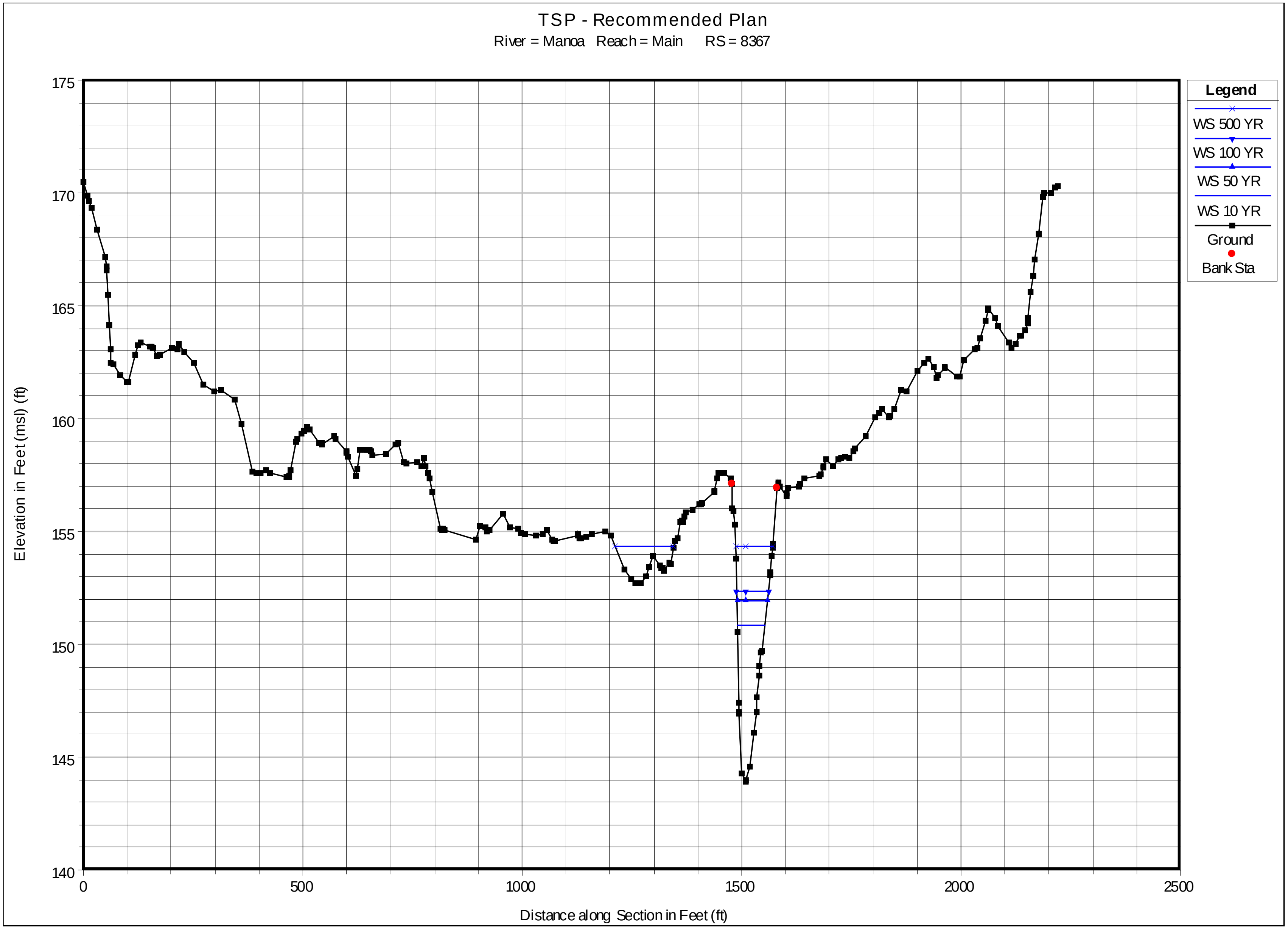

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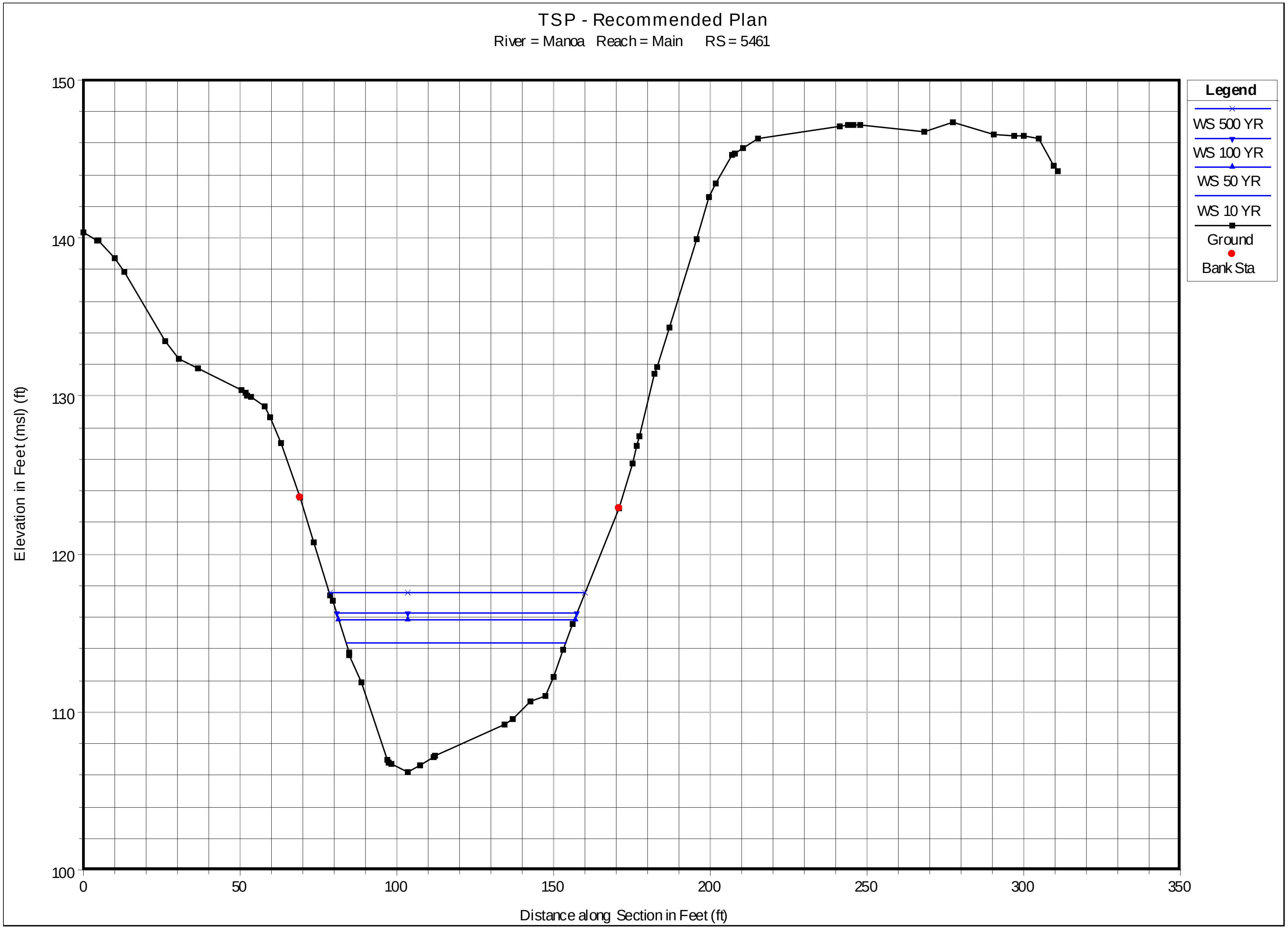

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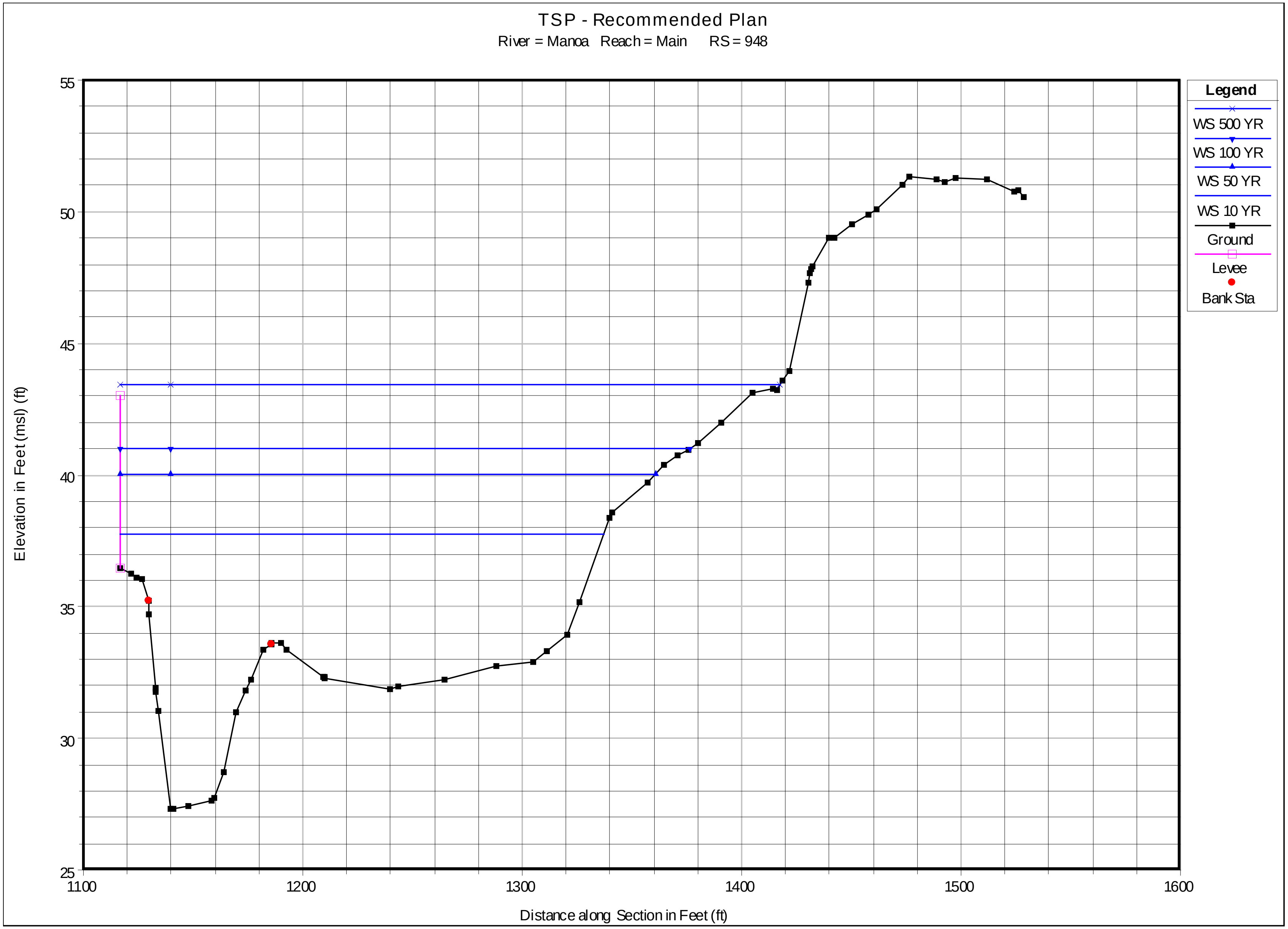

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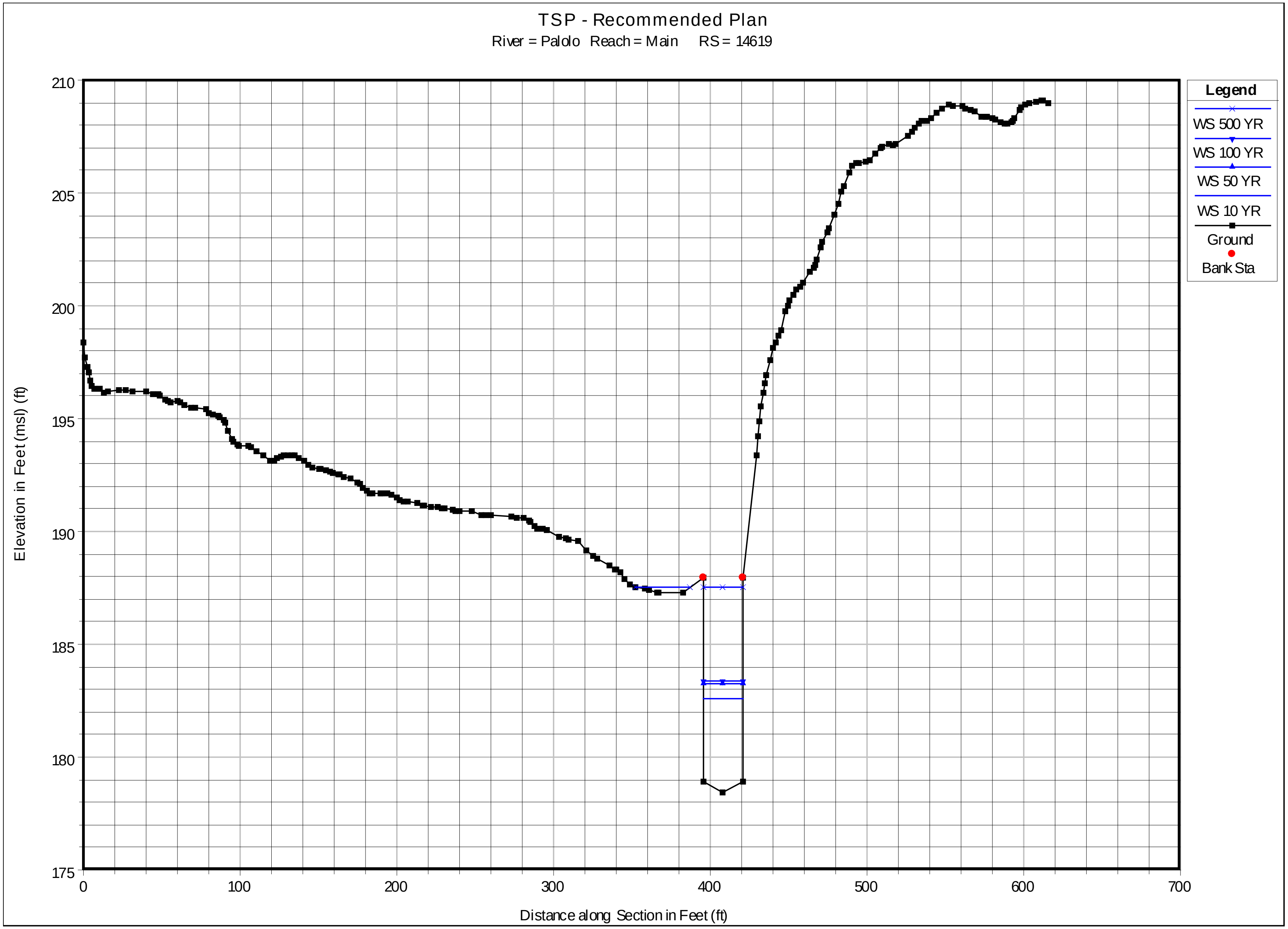

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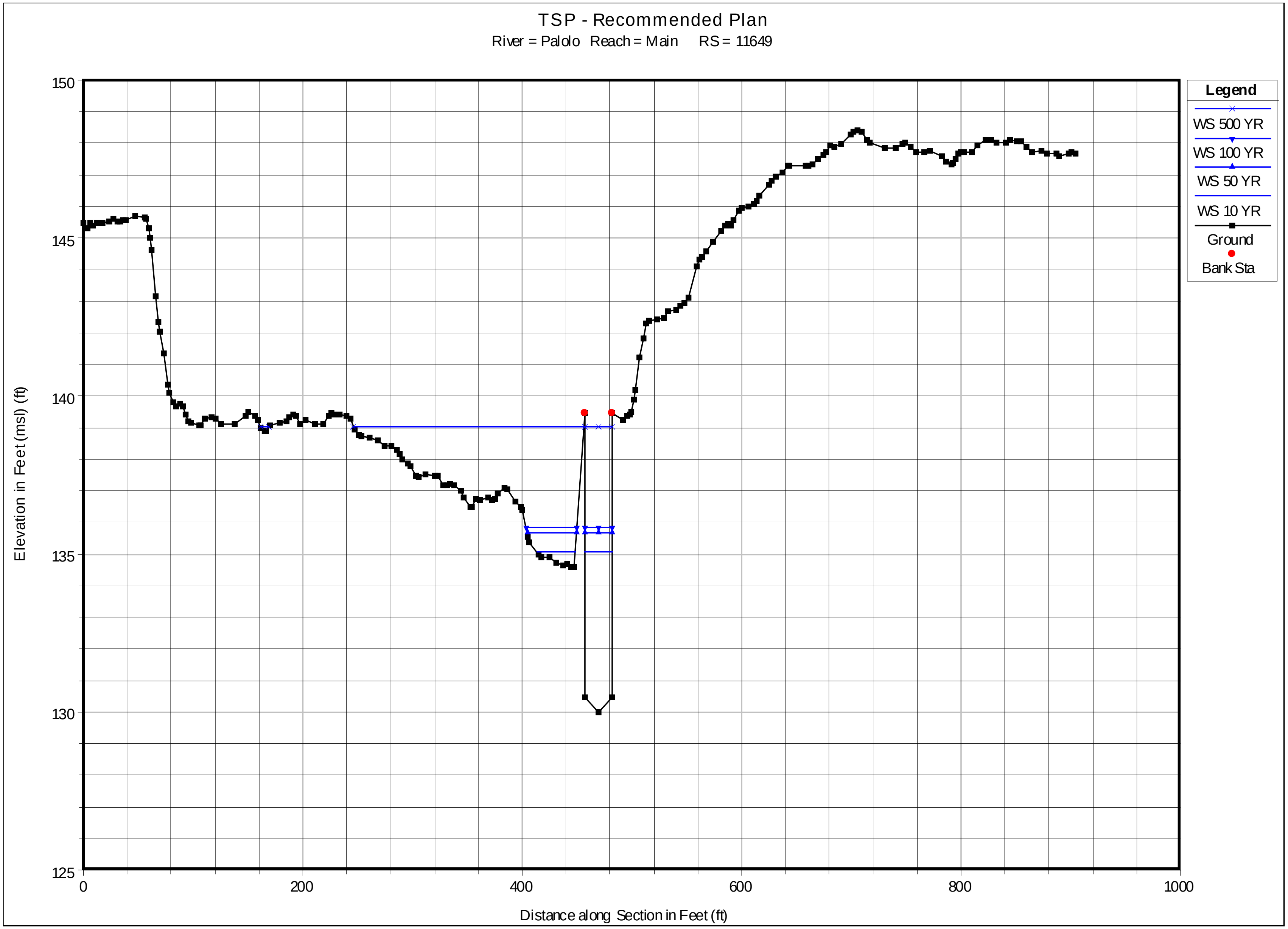

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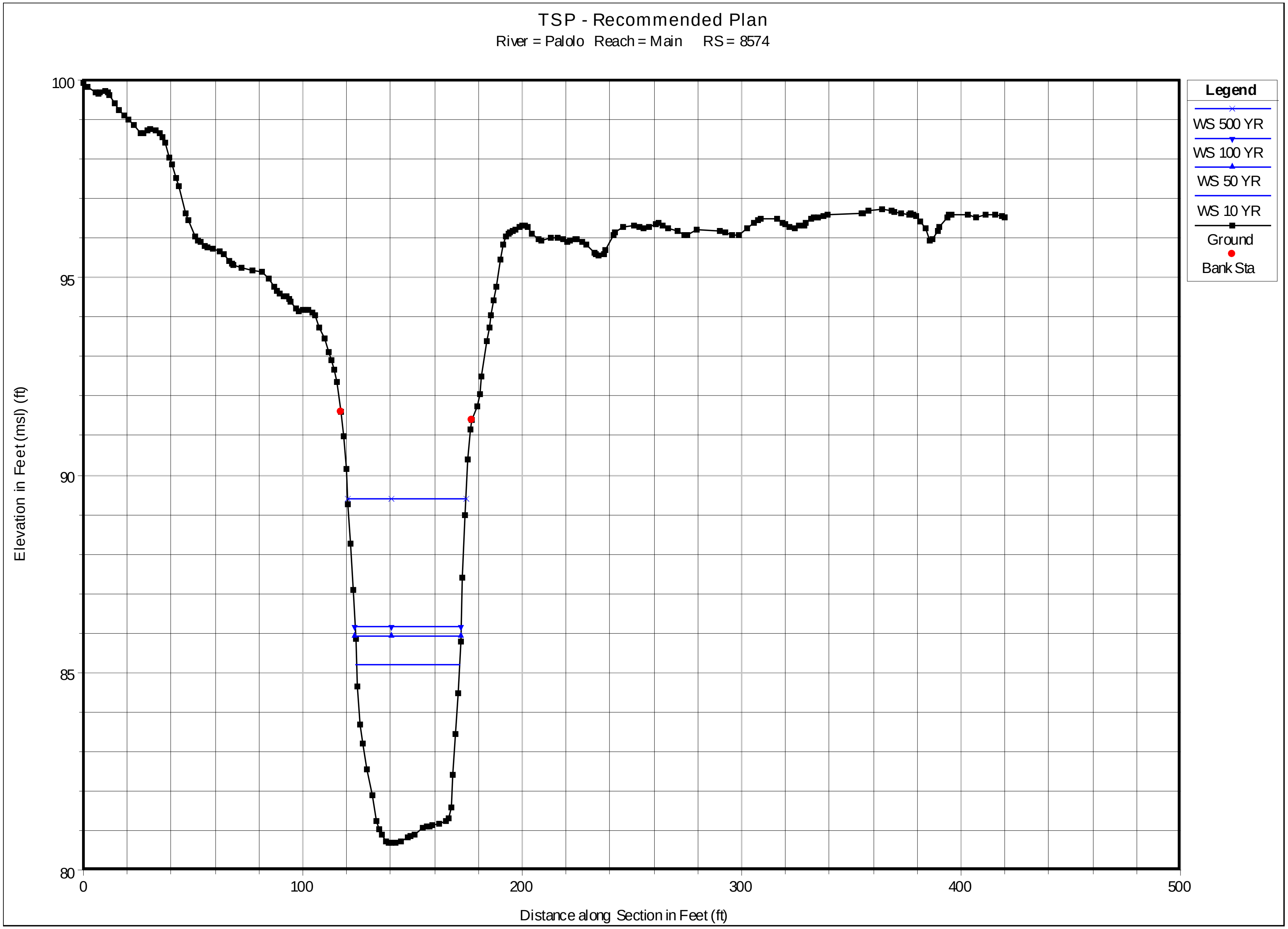

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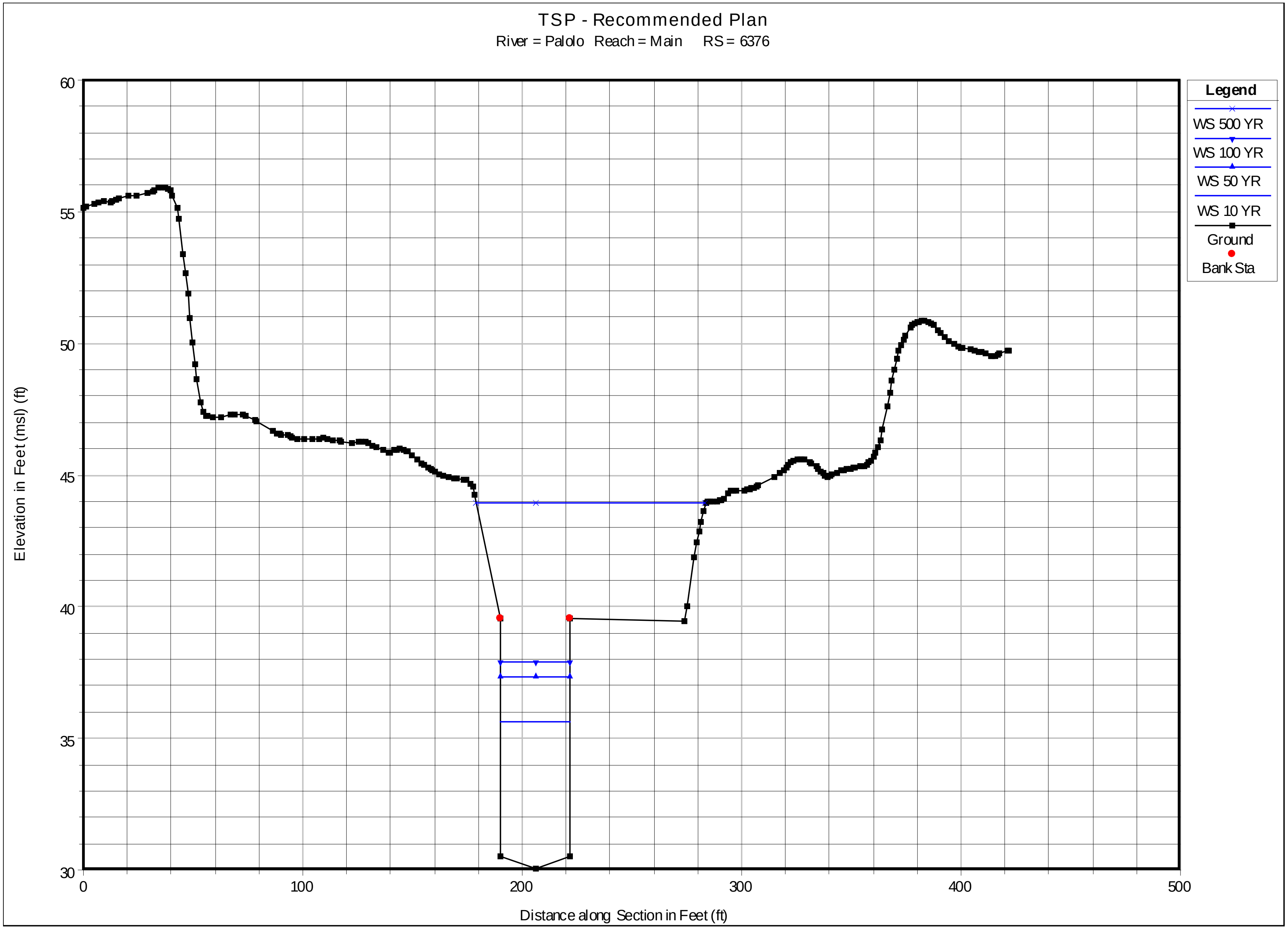

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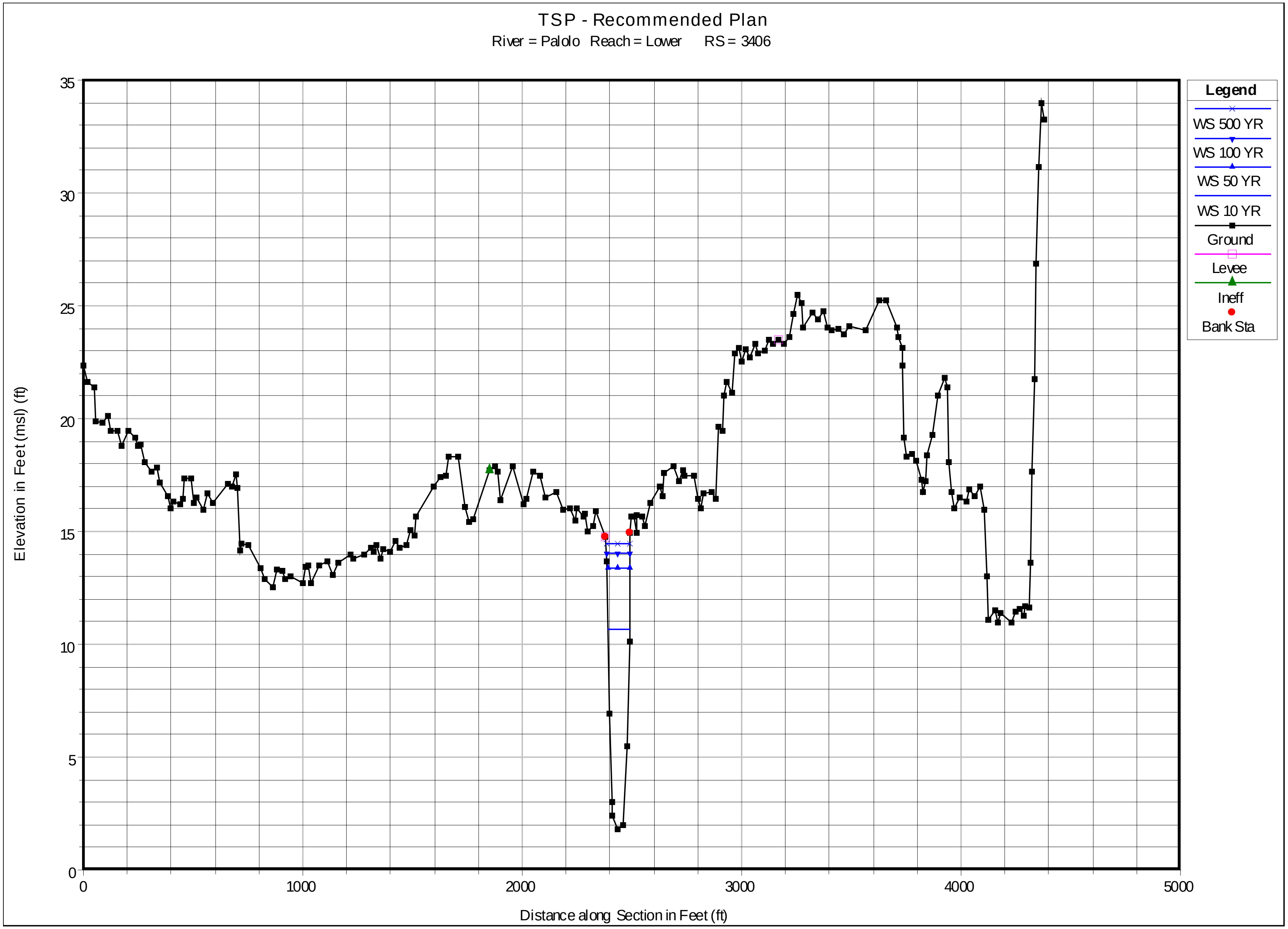

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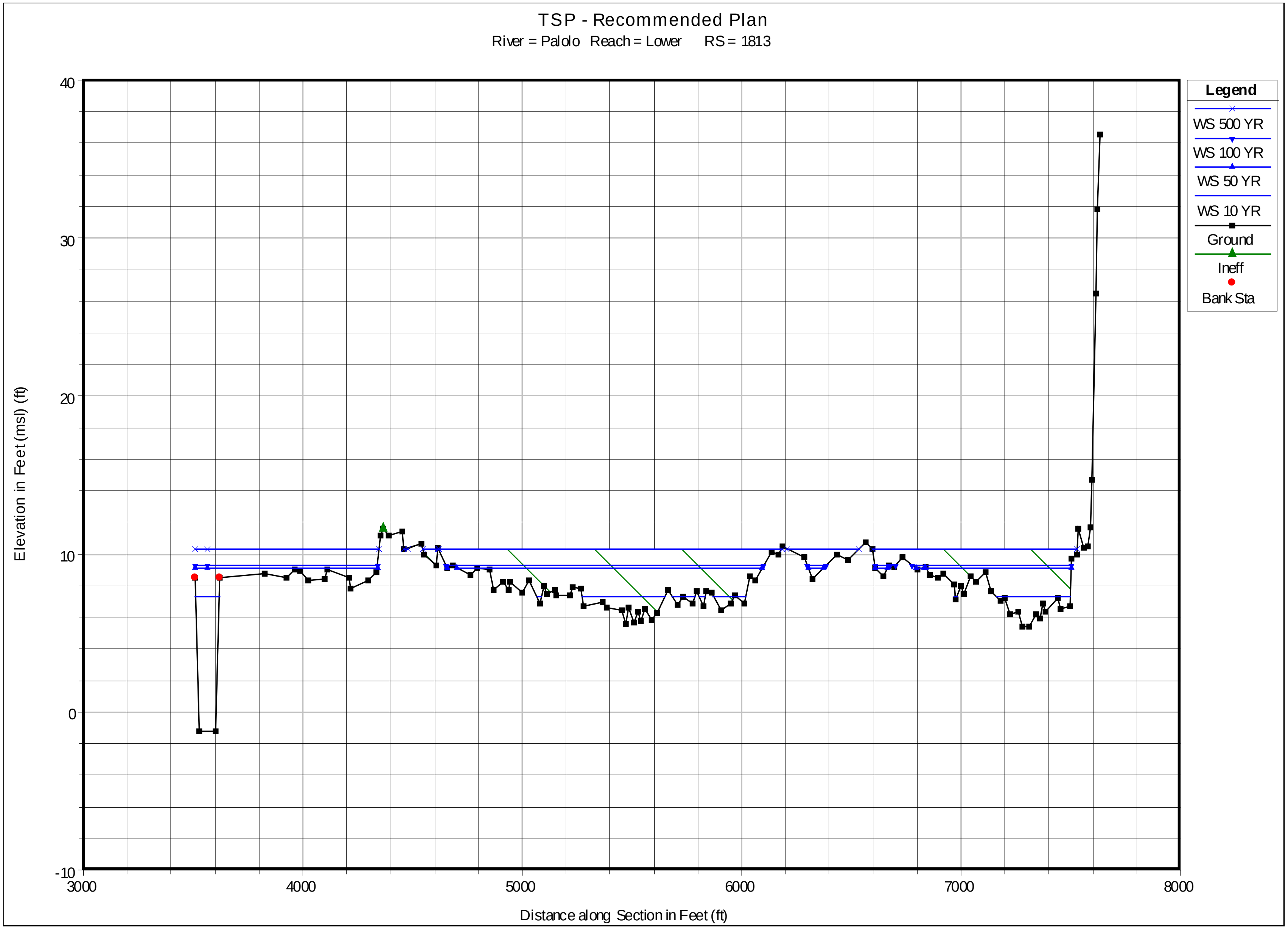

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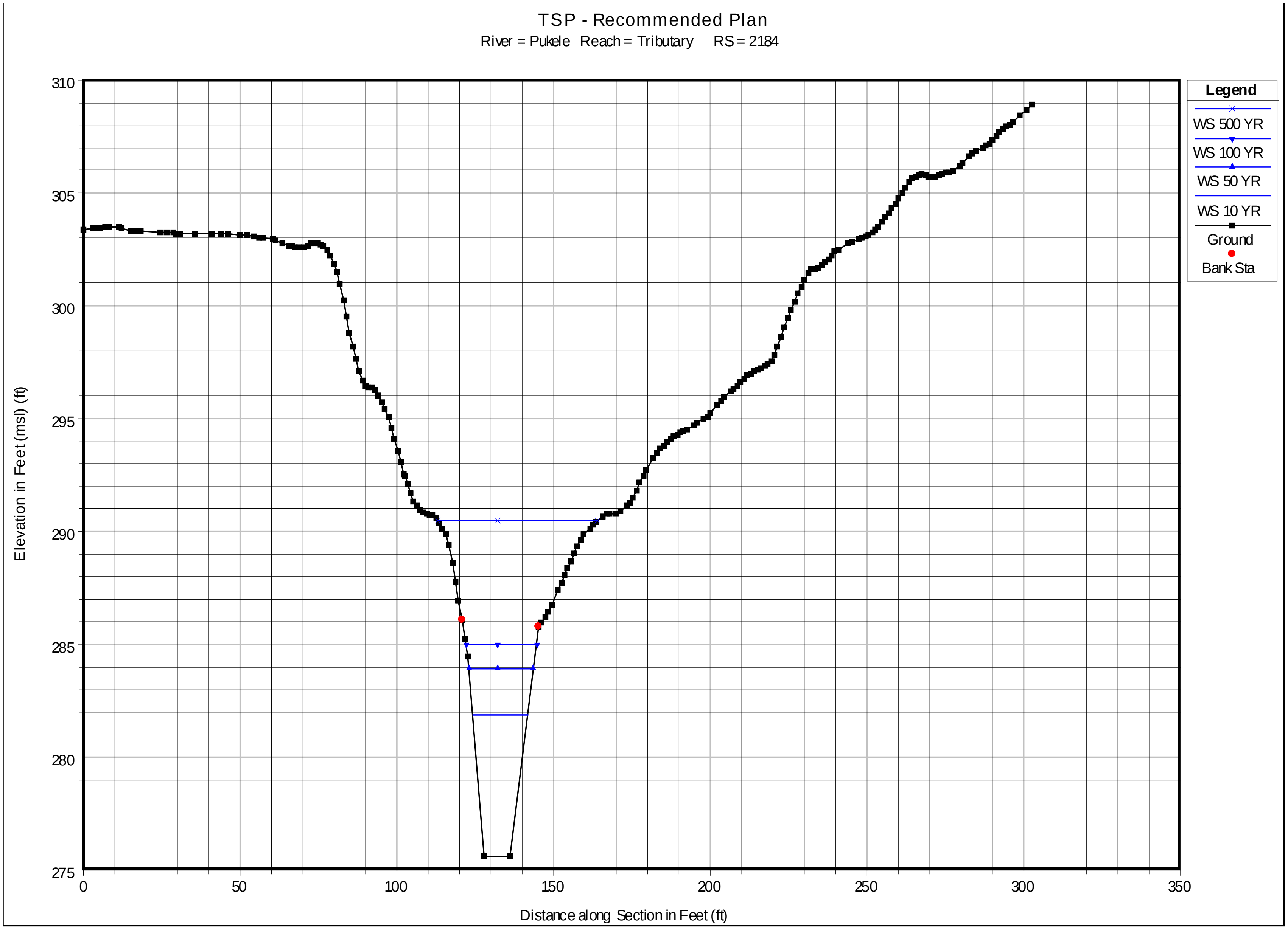

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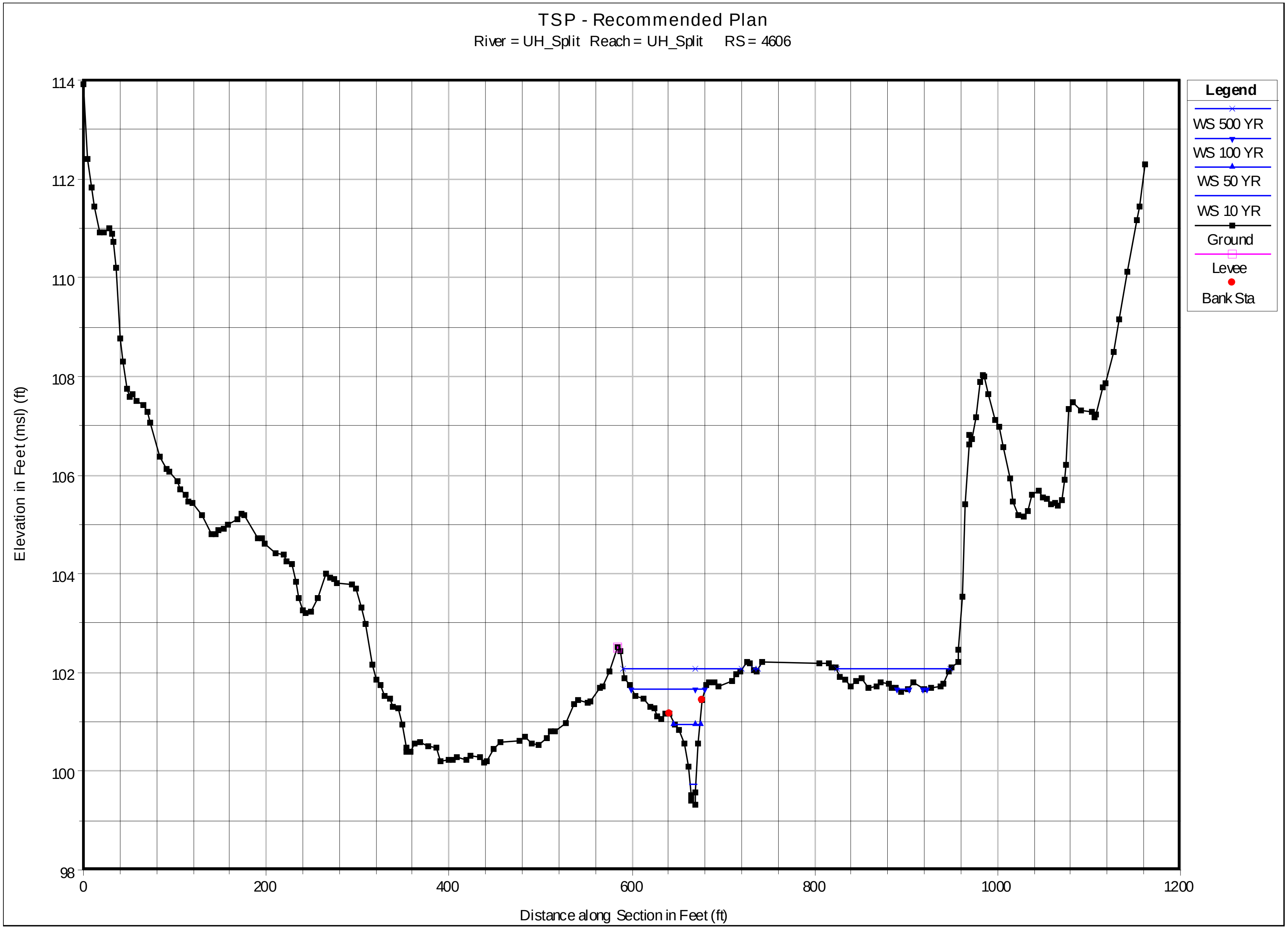

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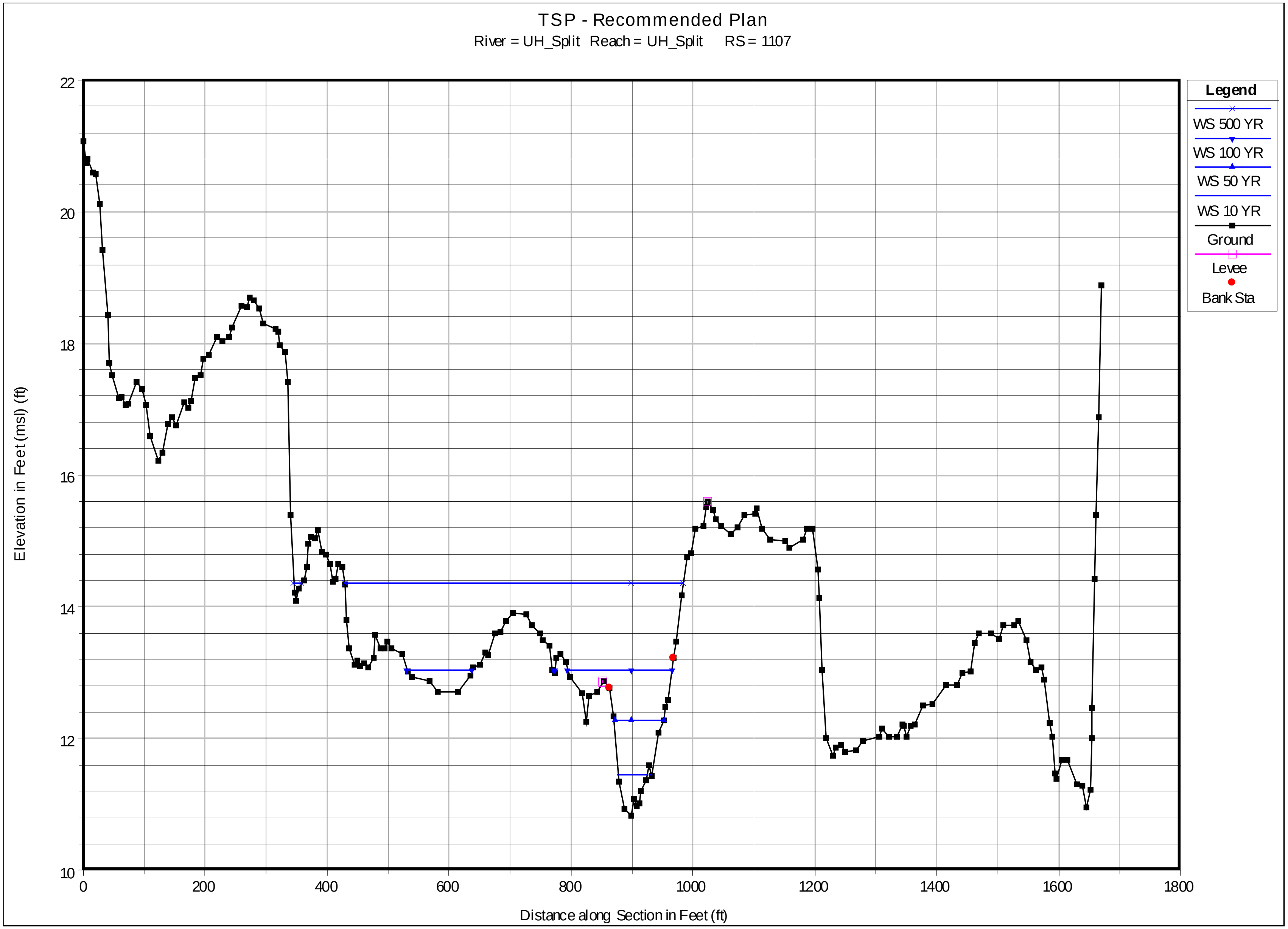

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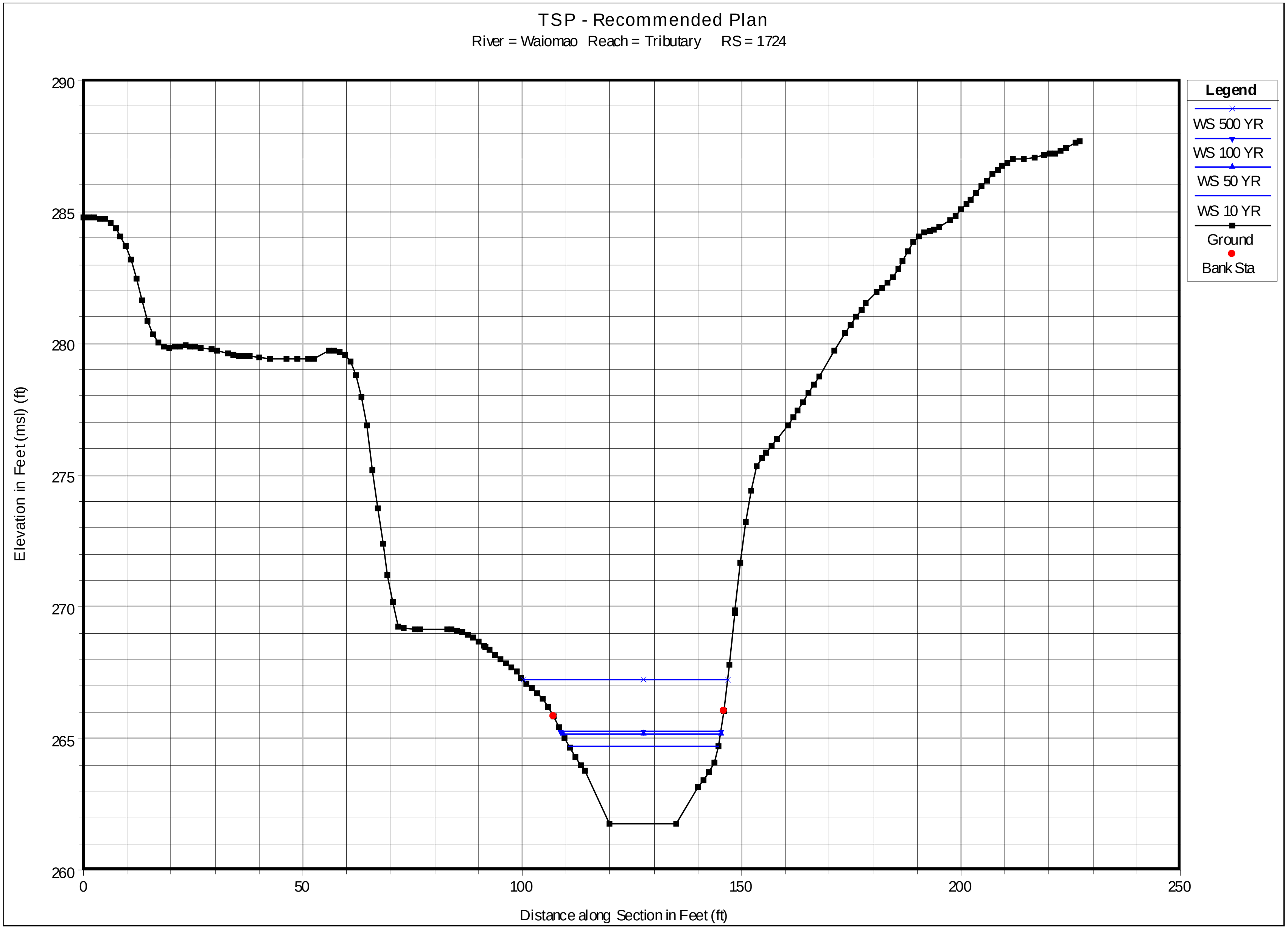
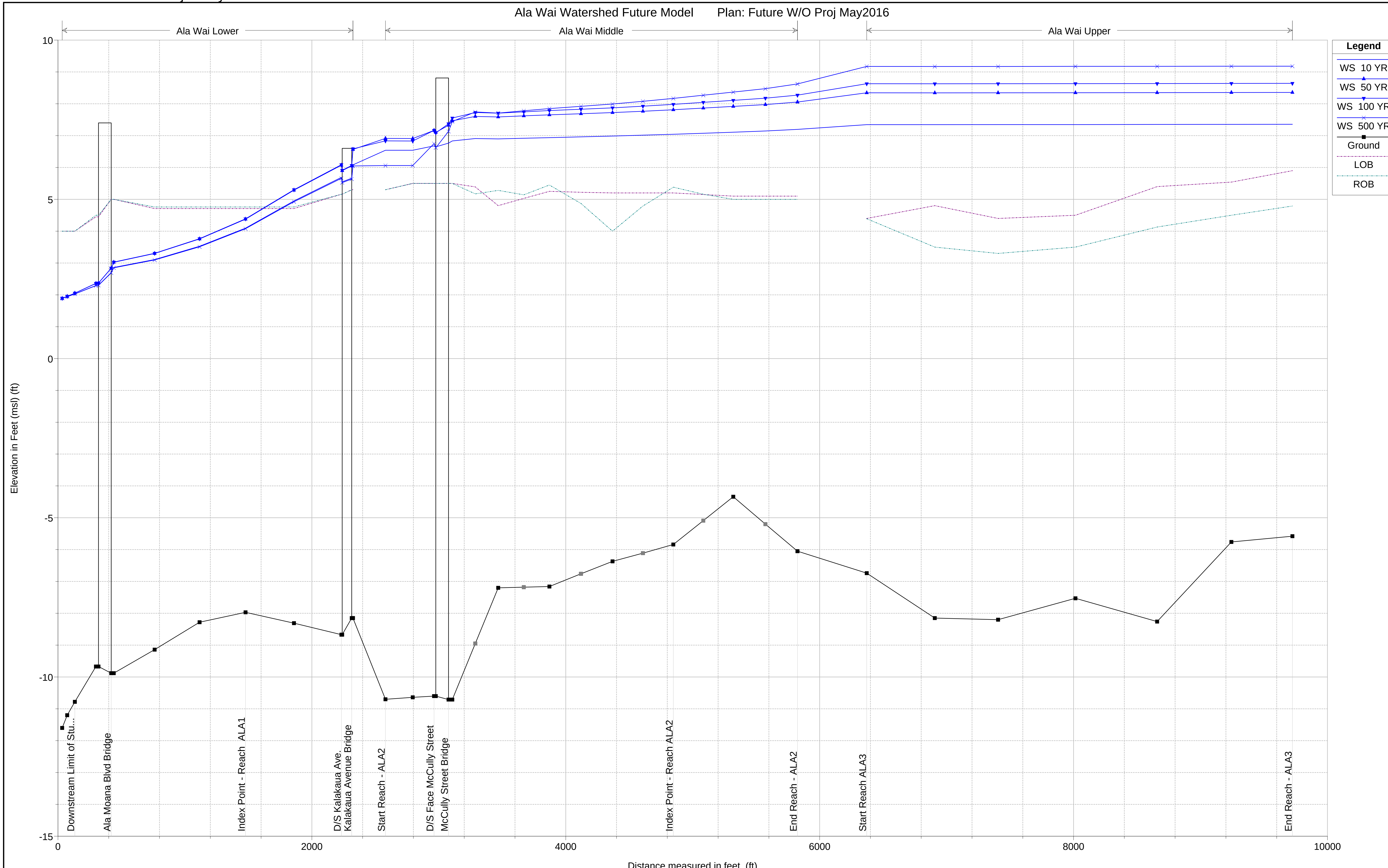


Plate 3 With-Out Project Hydraulic Profiles



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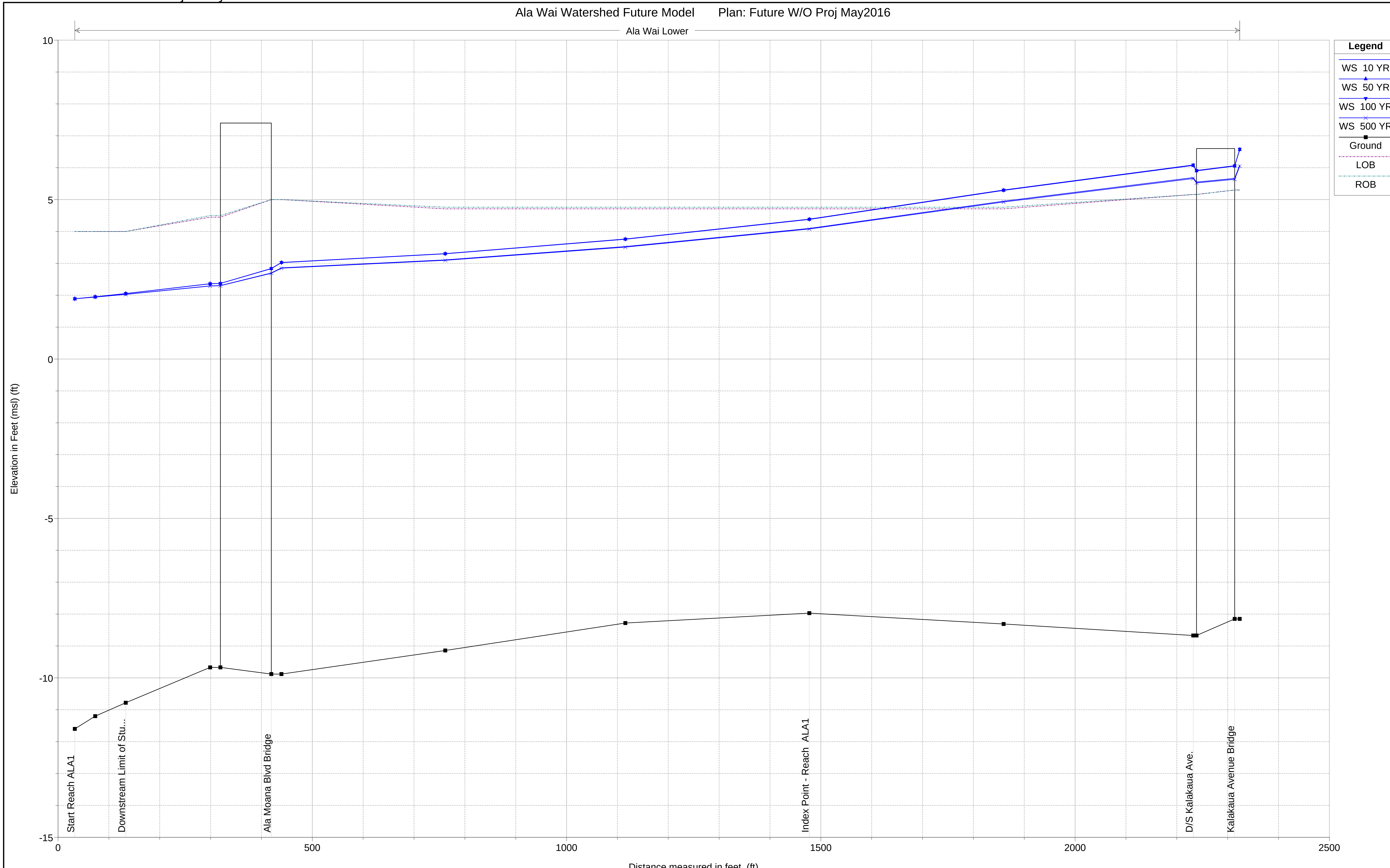

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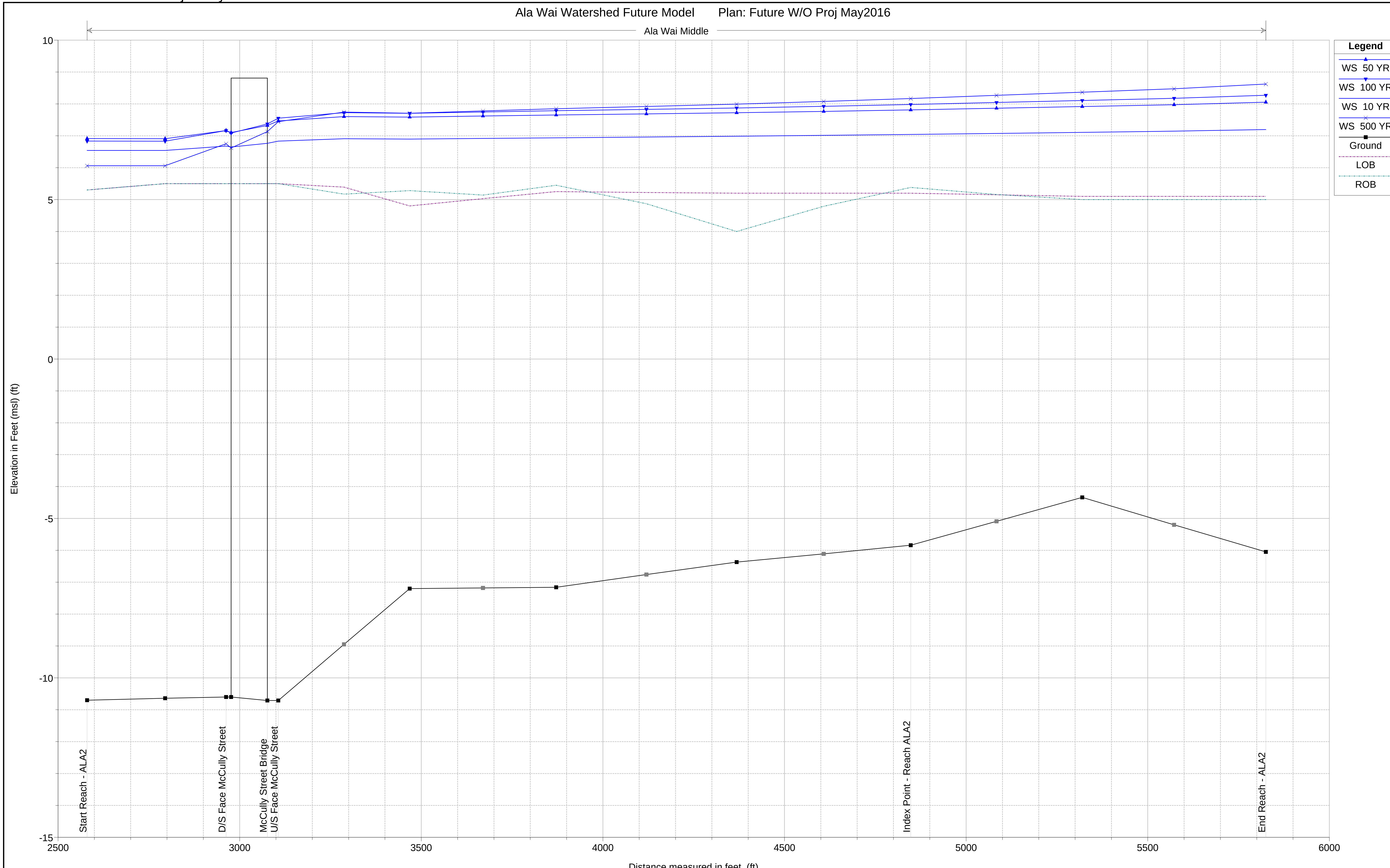

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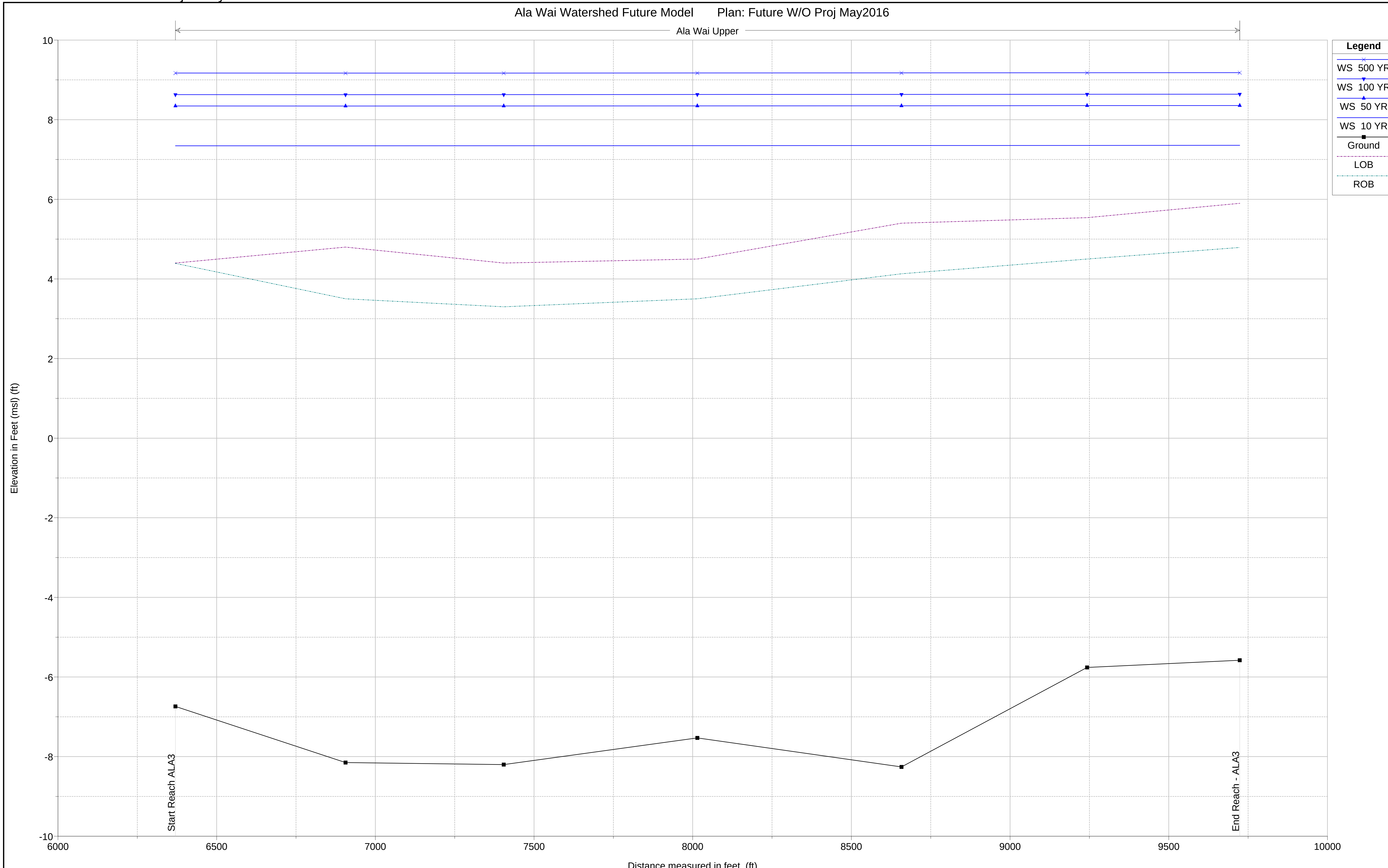

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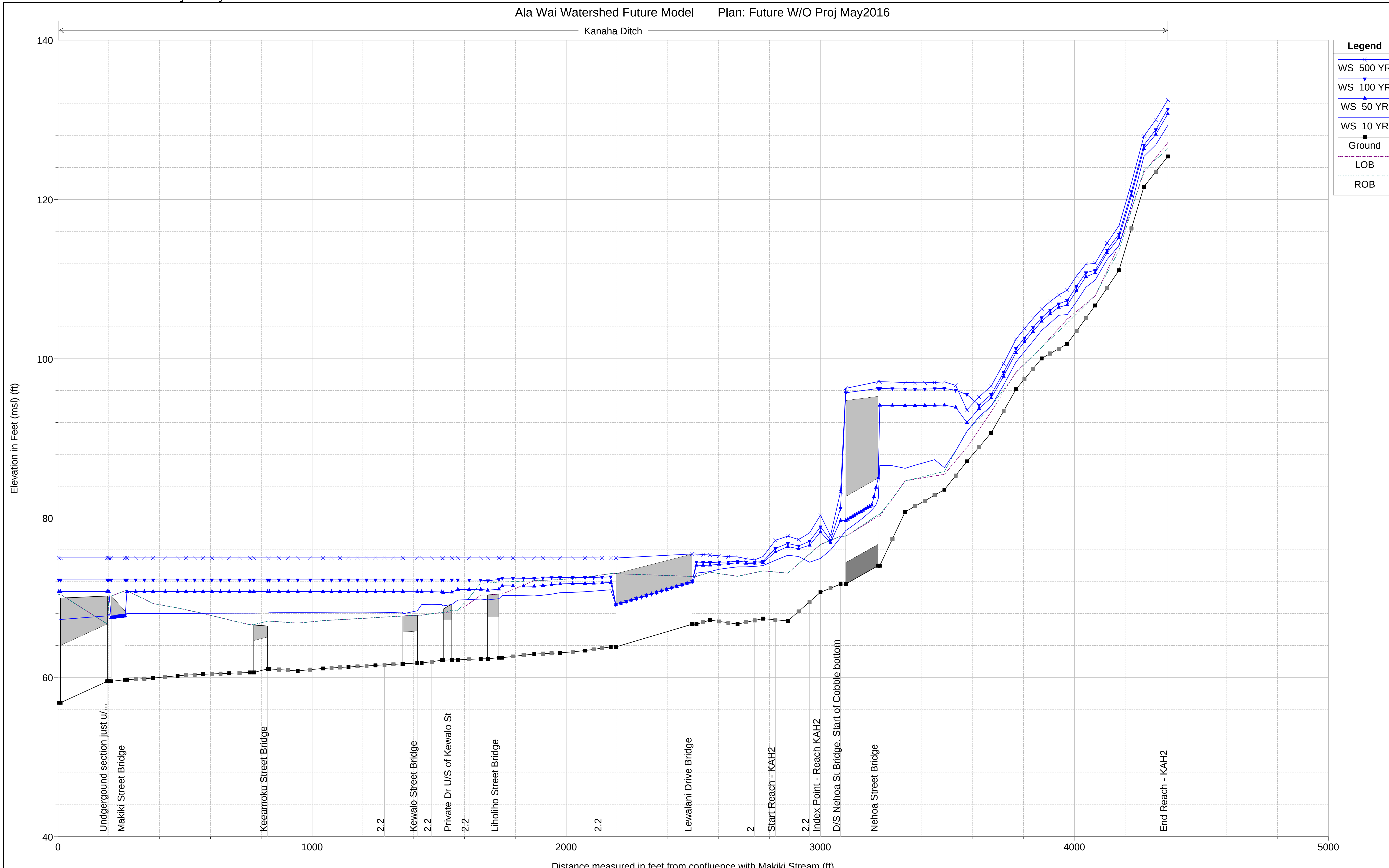

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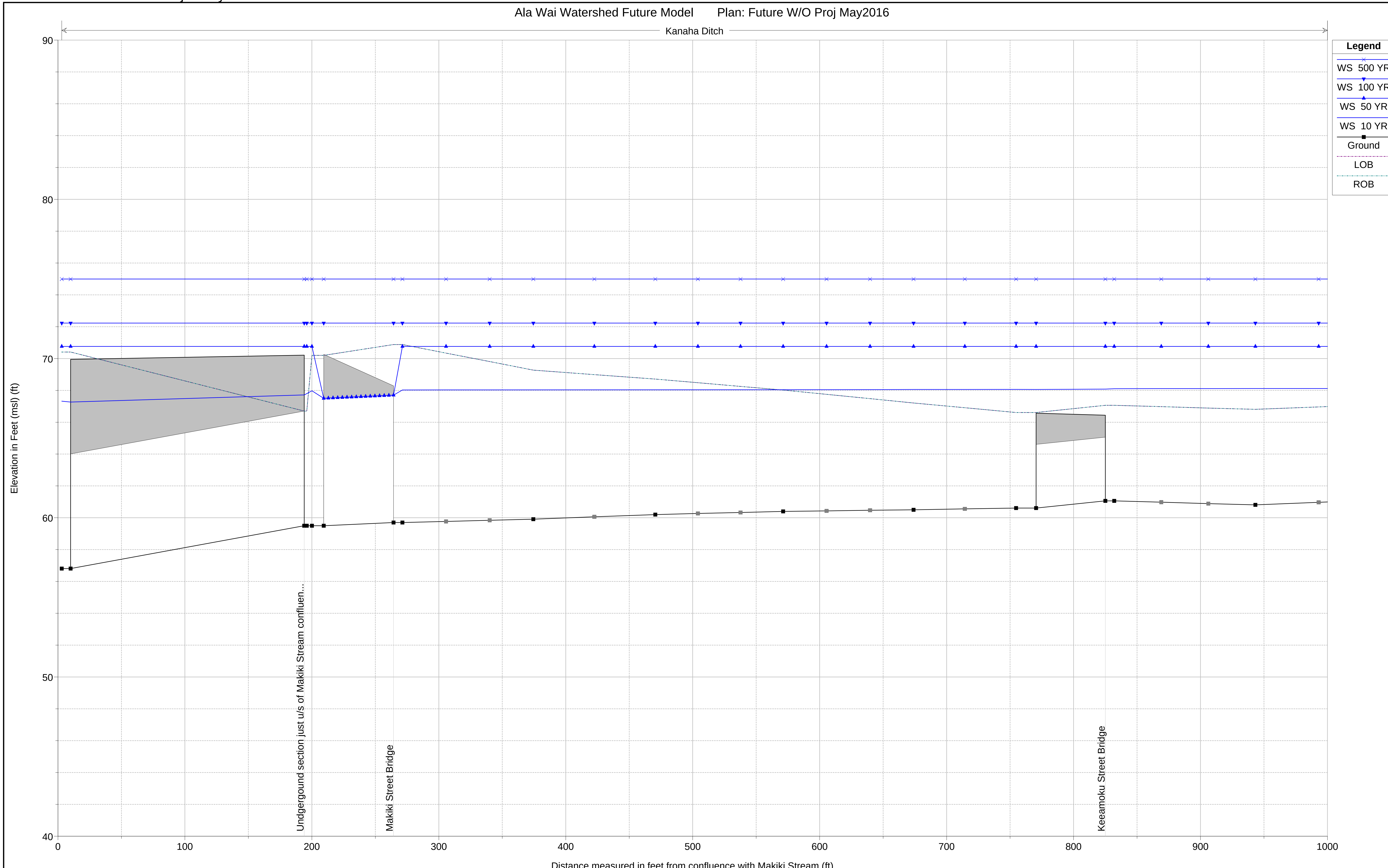

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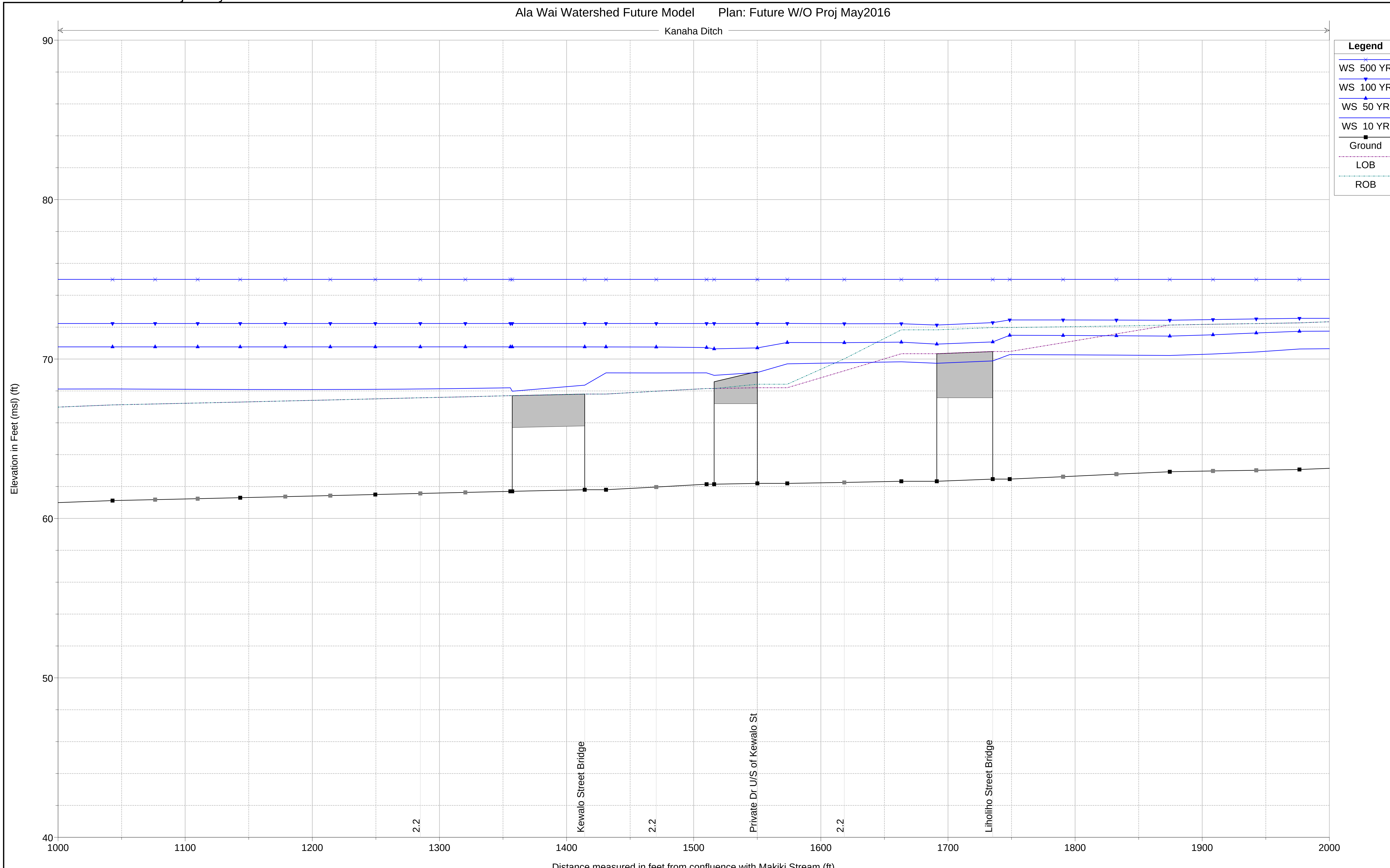

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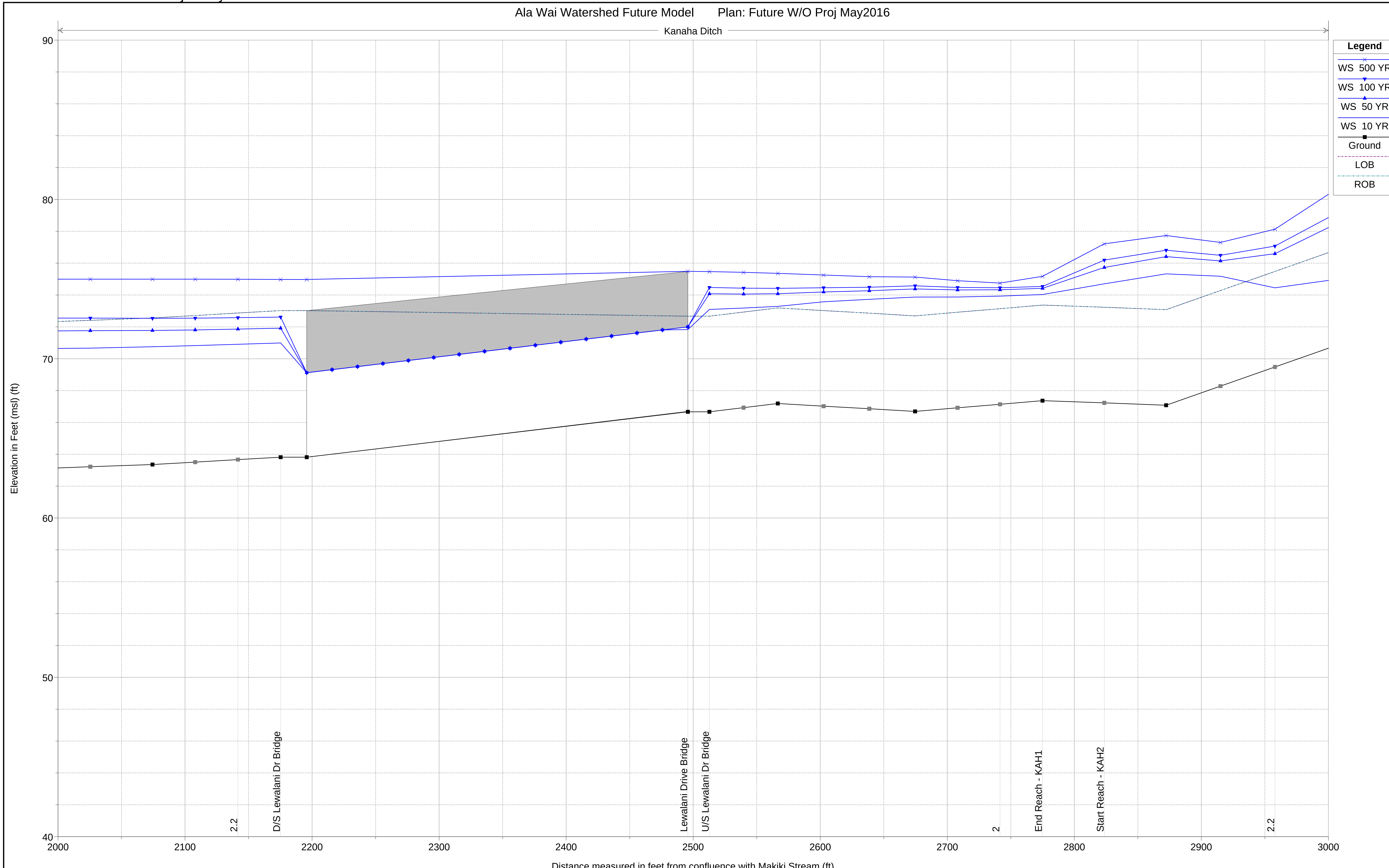

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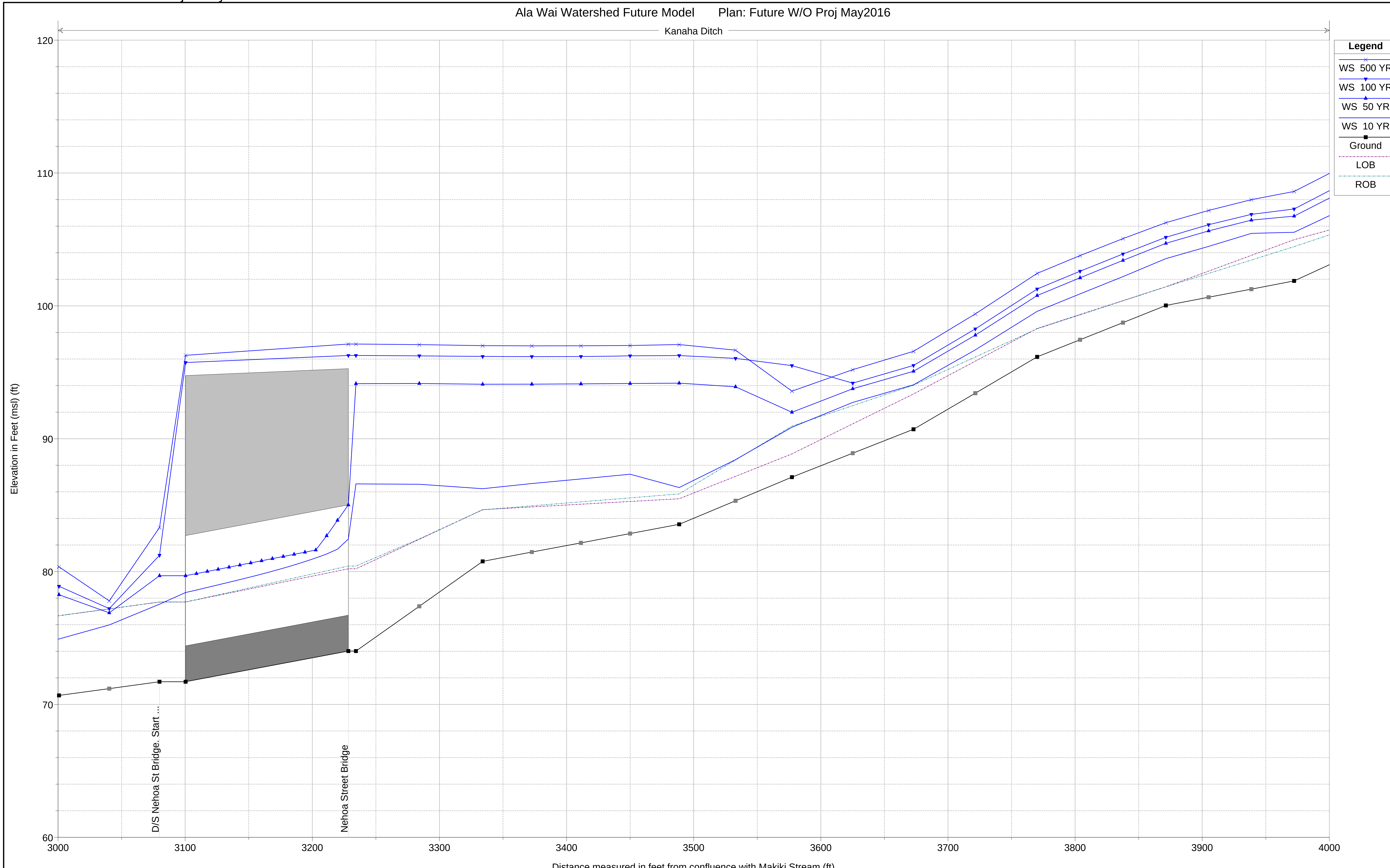
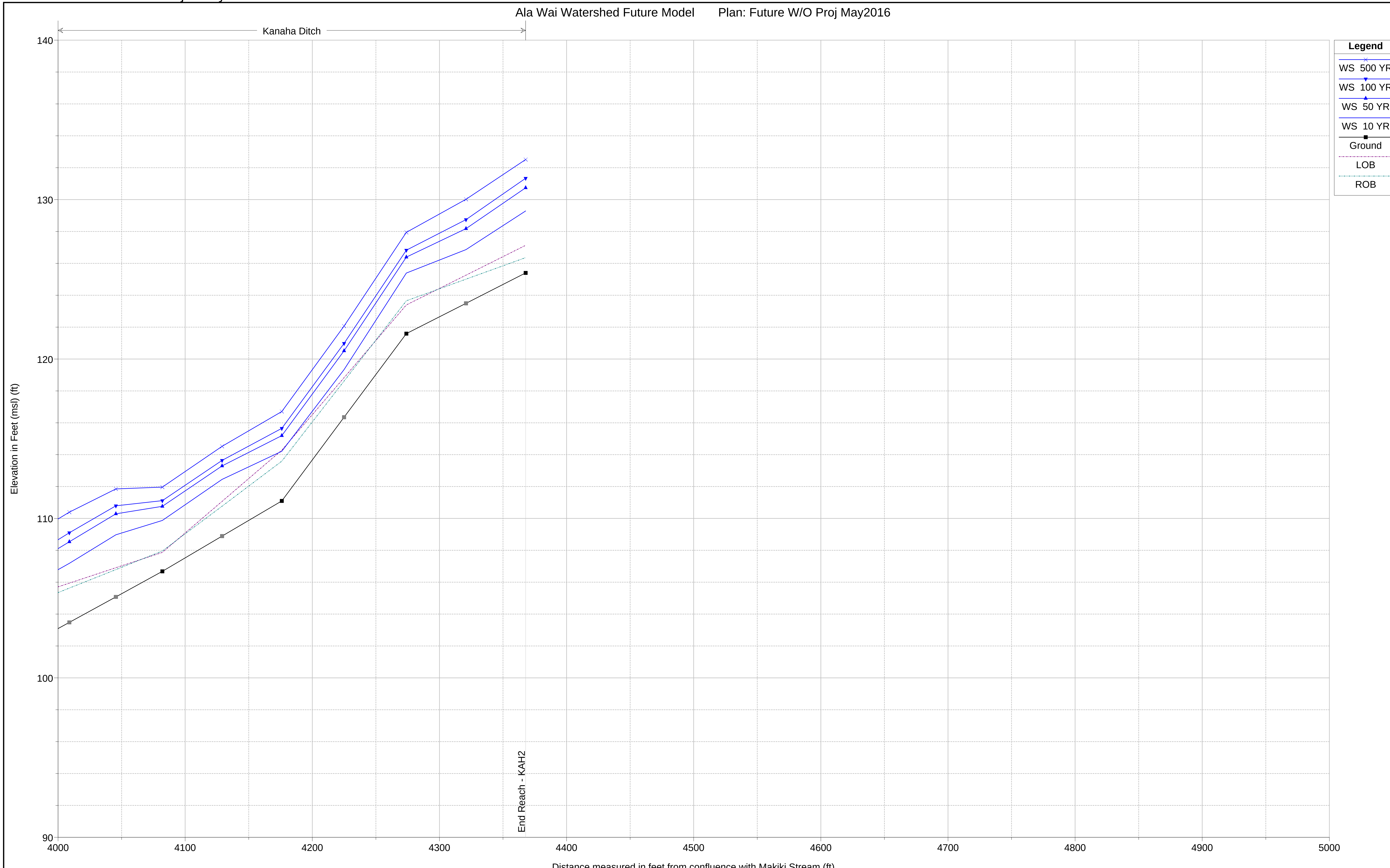


Plate 3 With-Out Project Hydraulic Profiles



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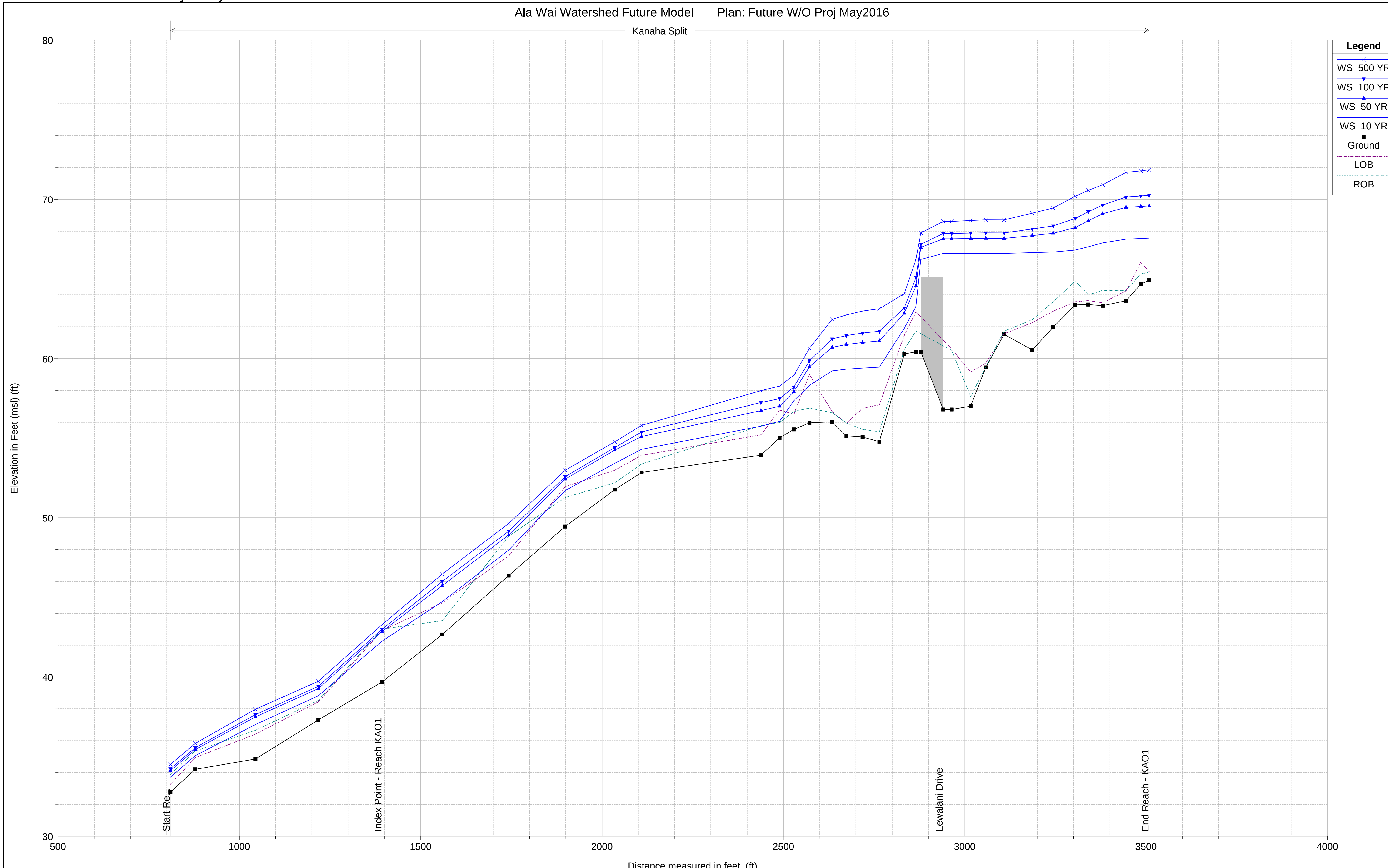

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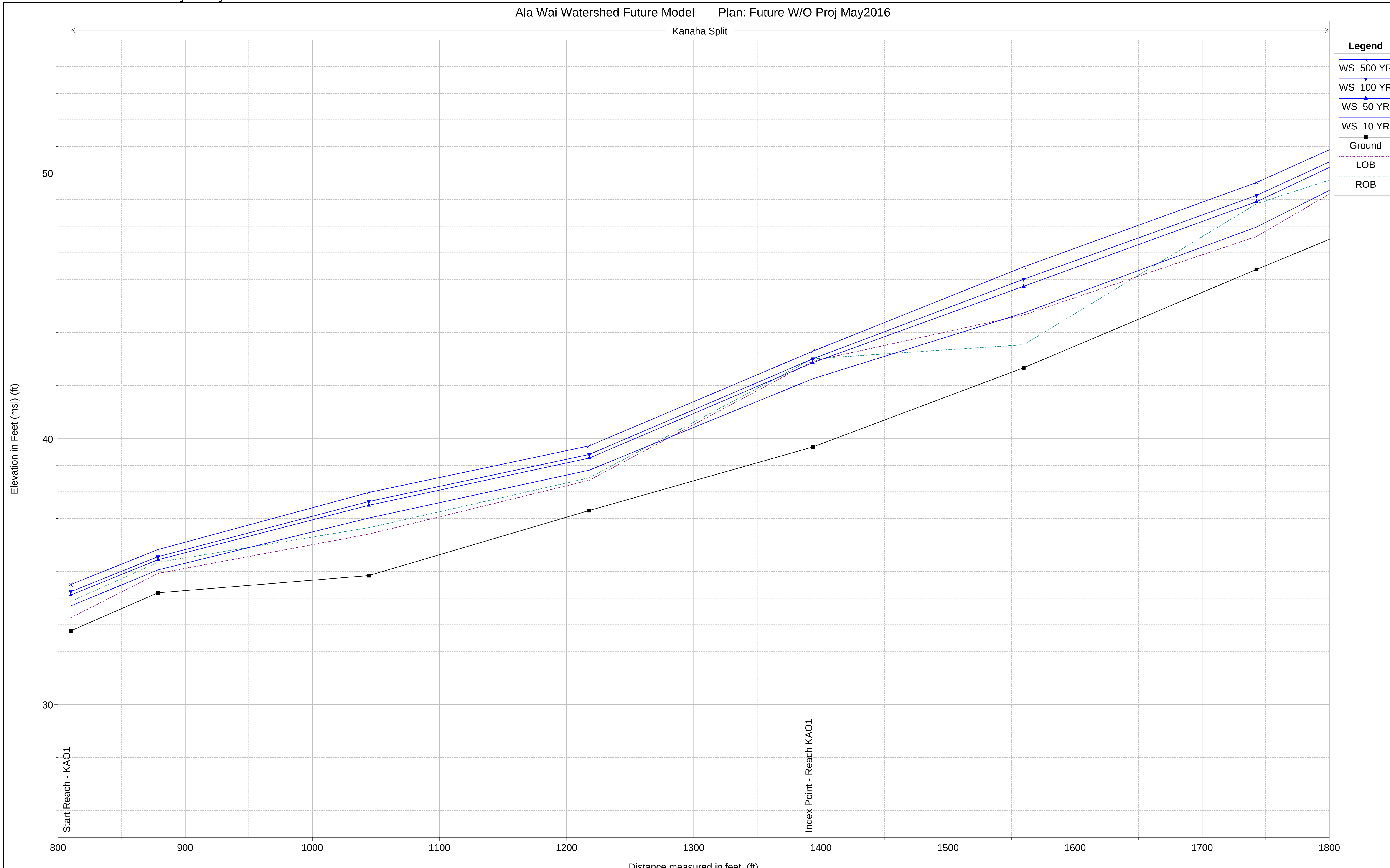

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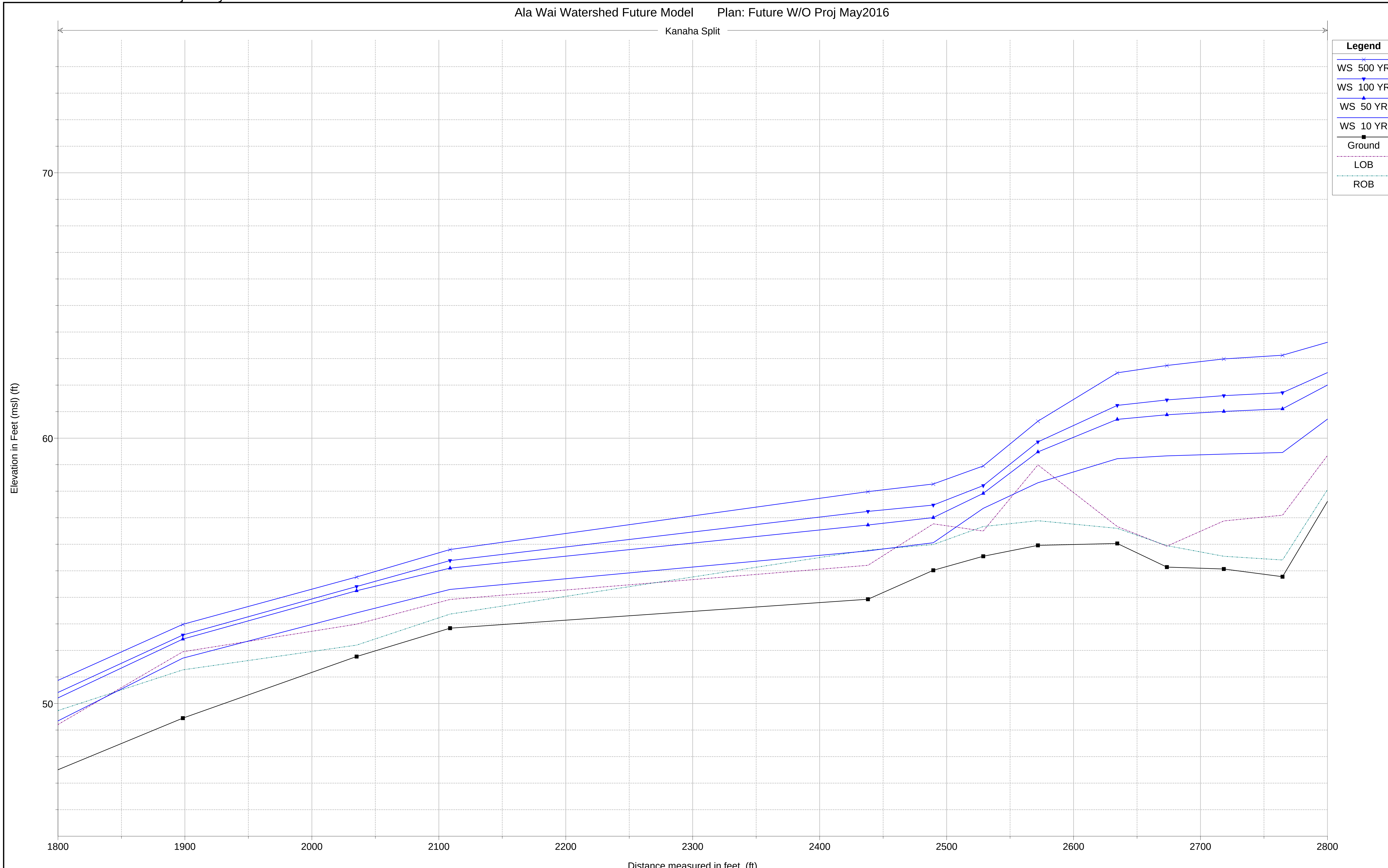

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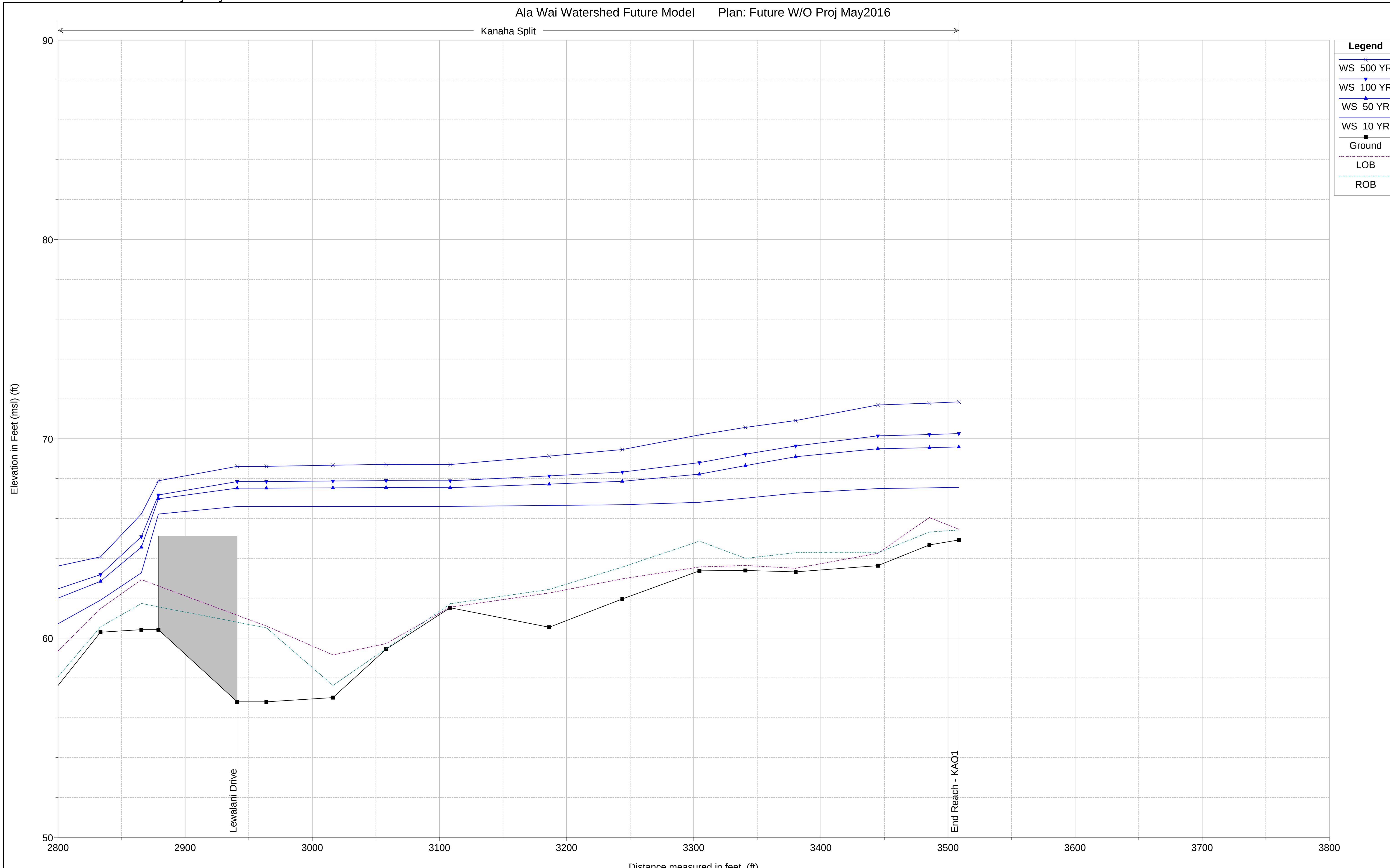

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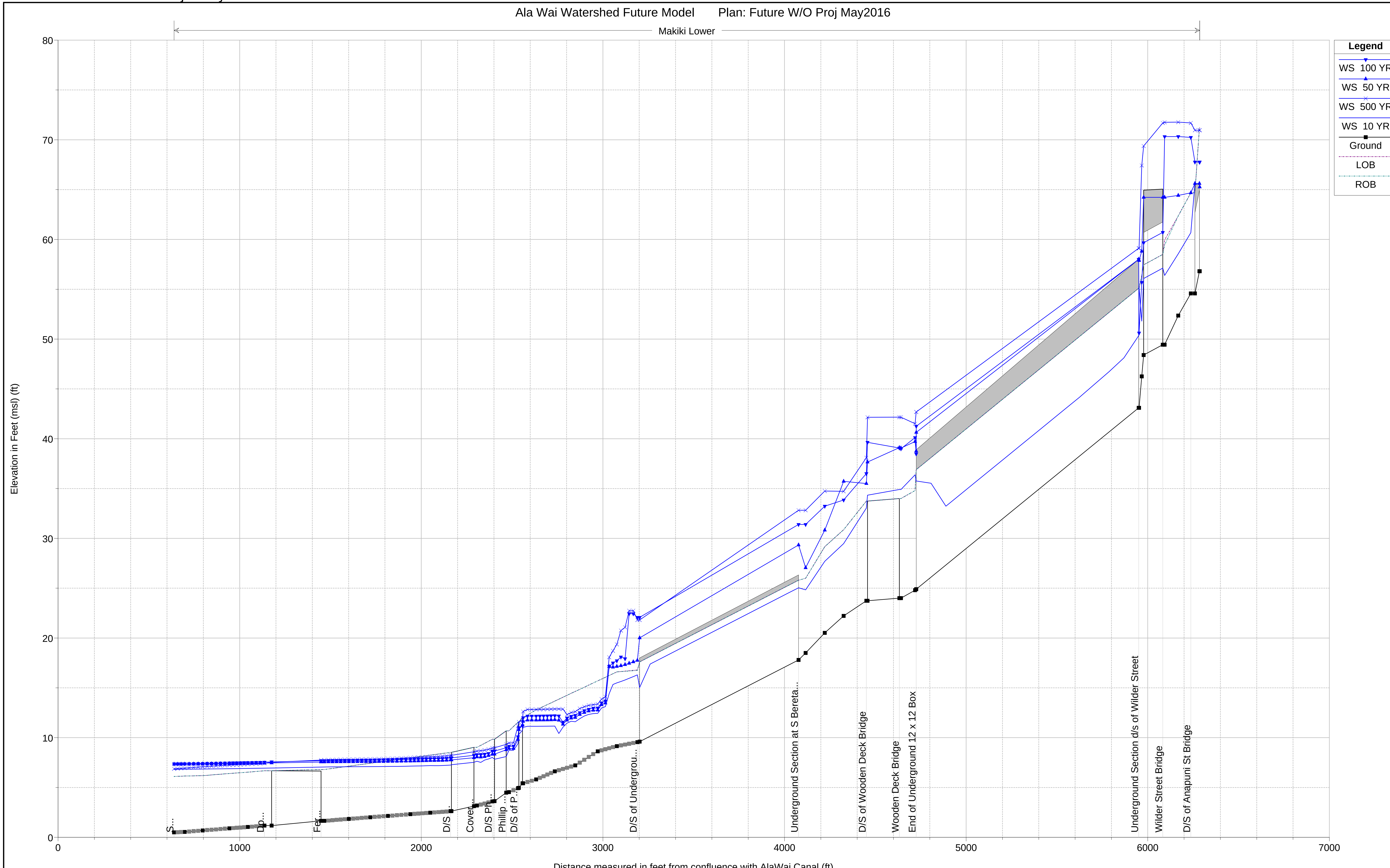

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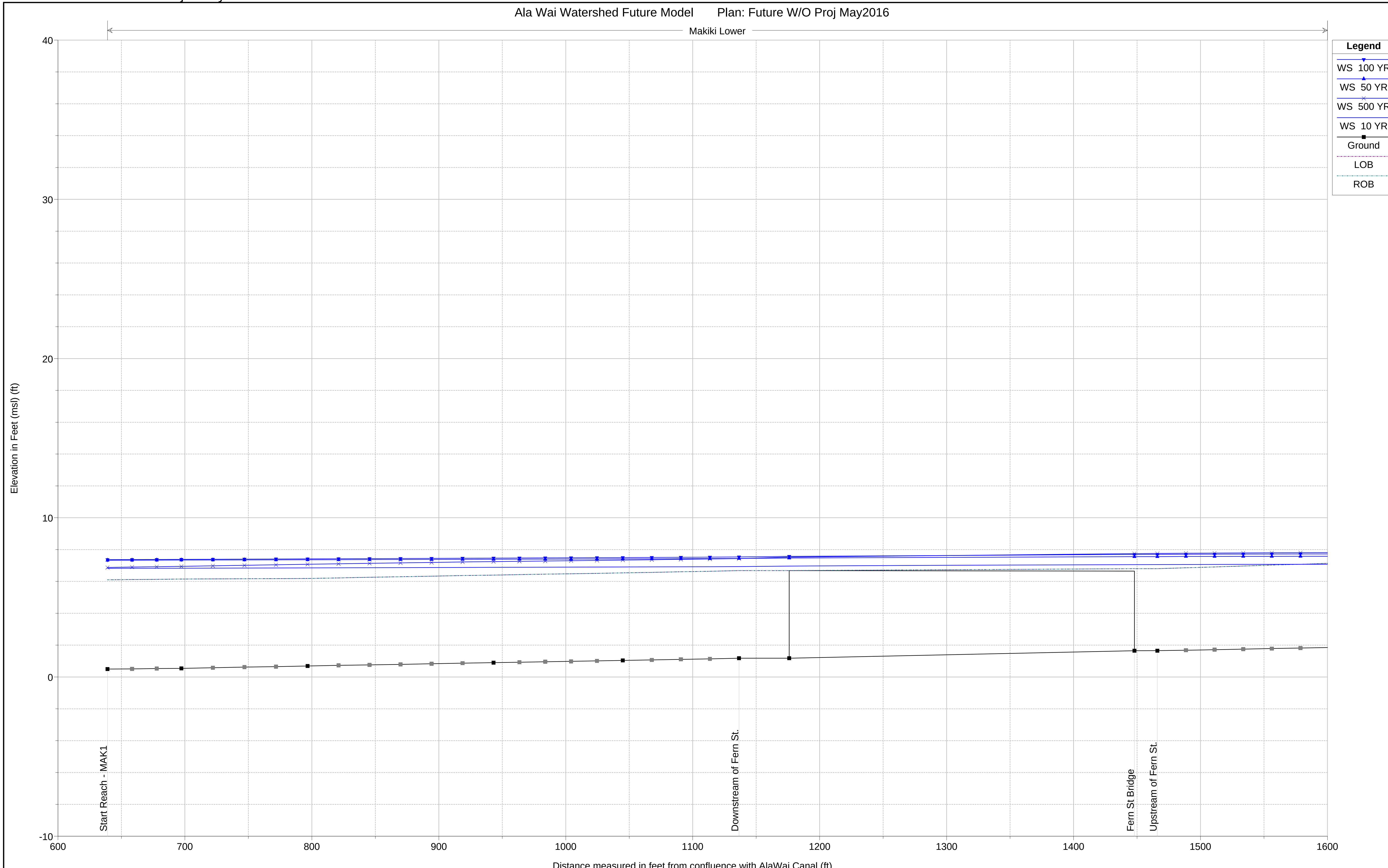

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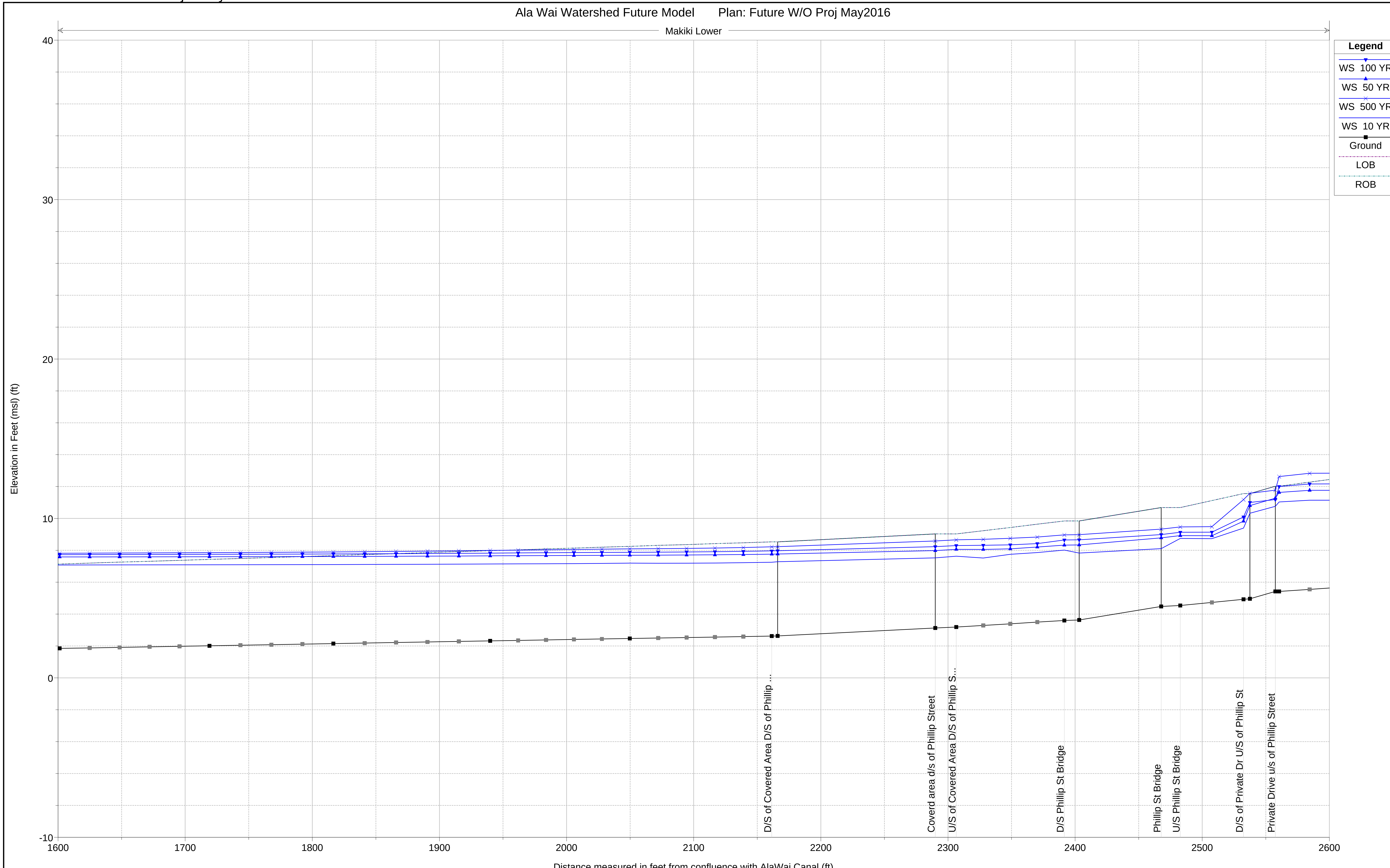

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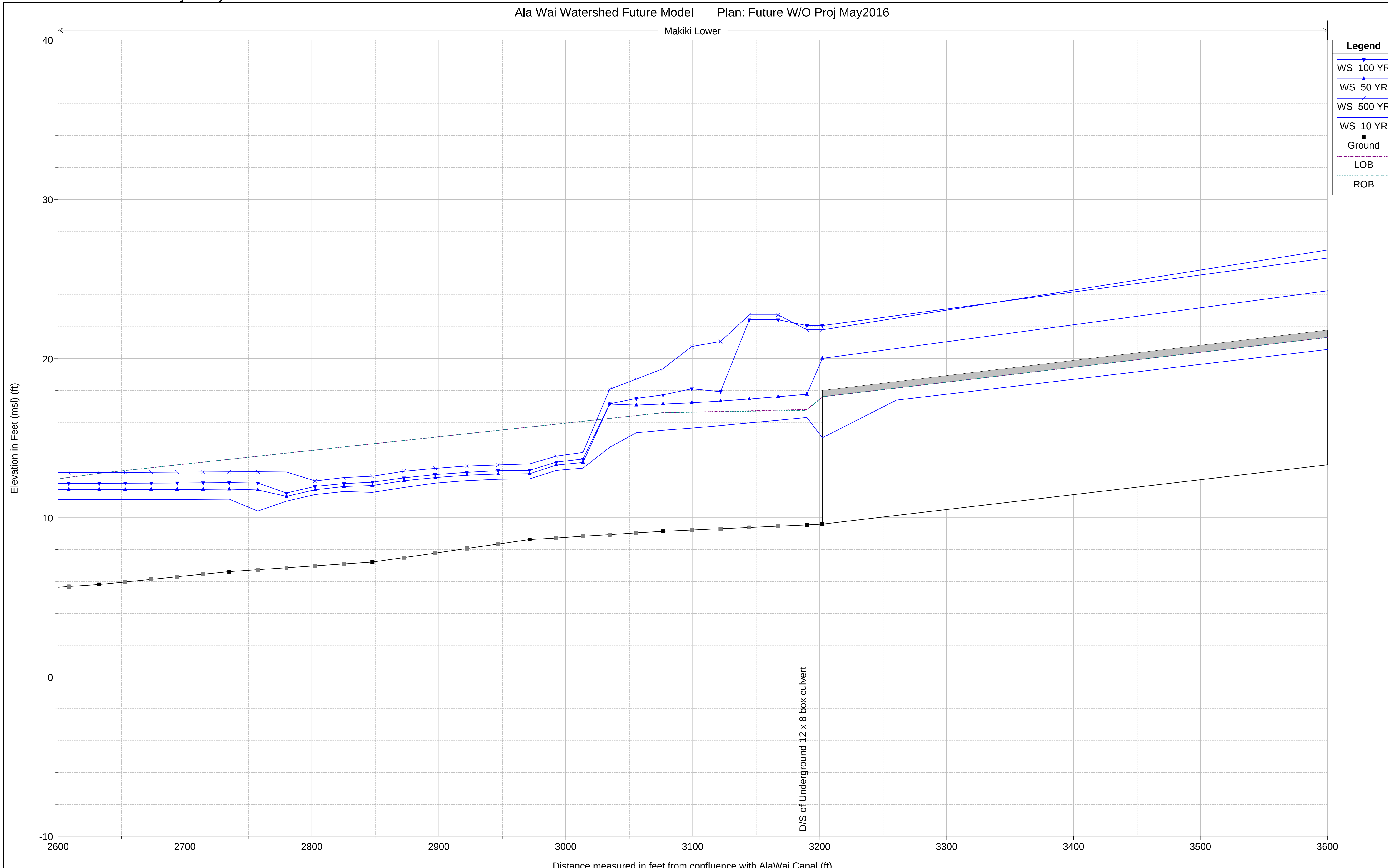

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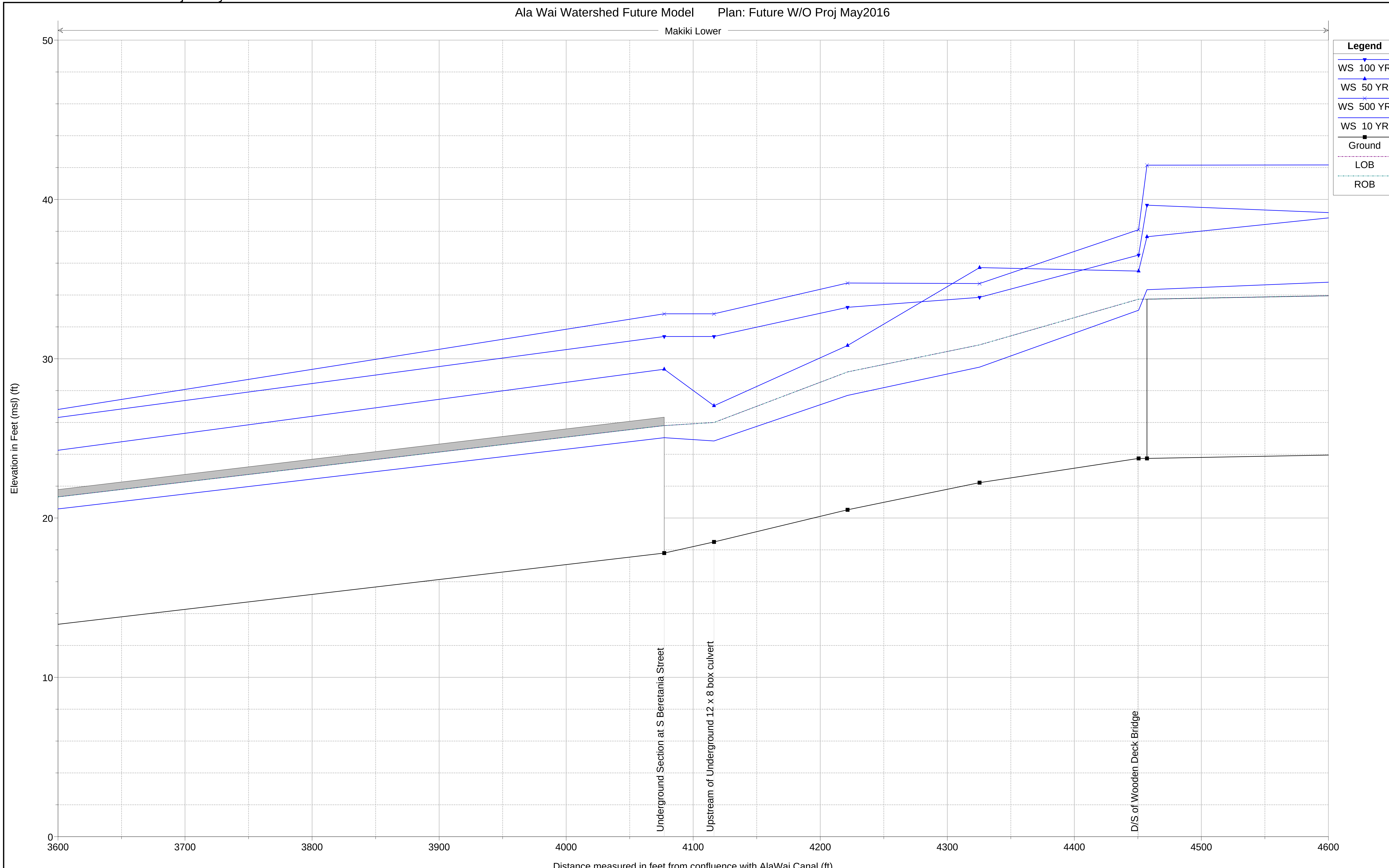

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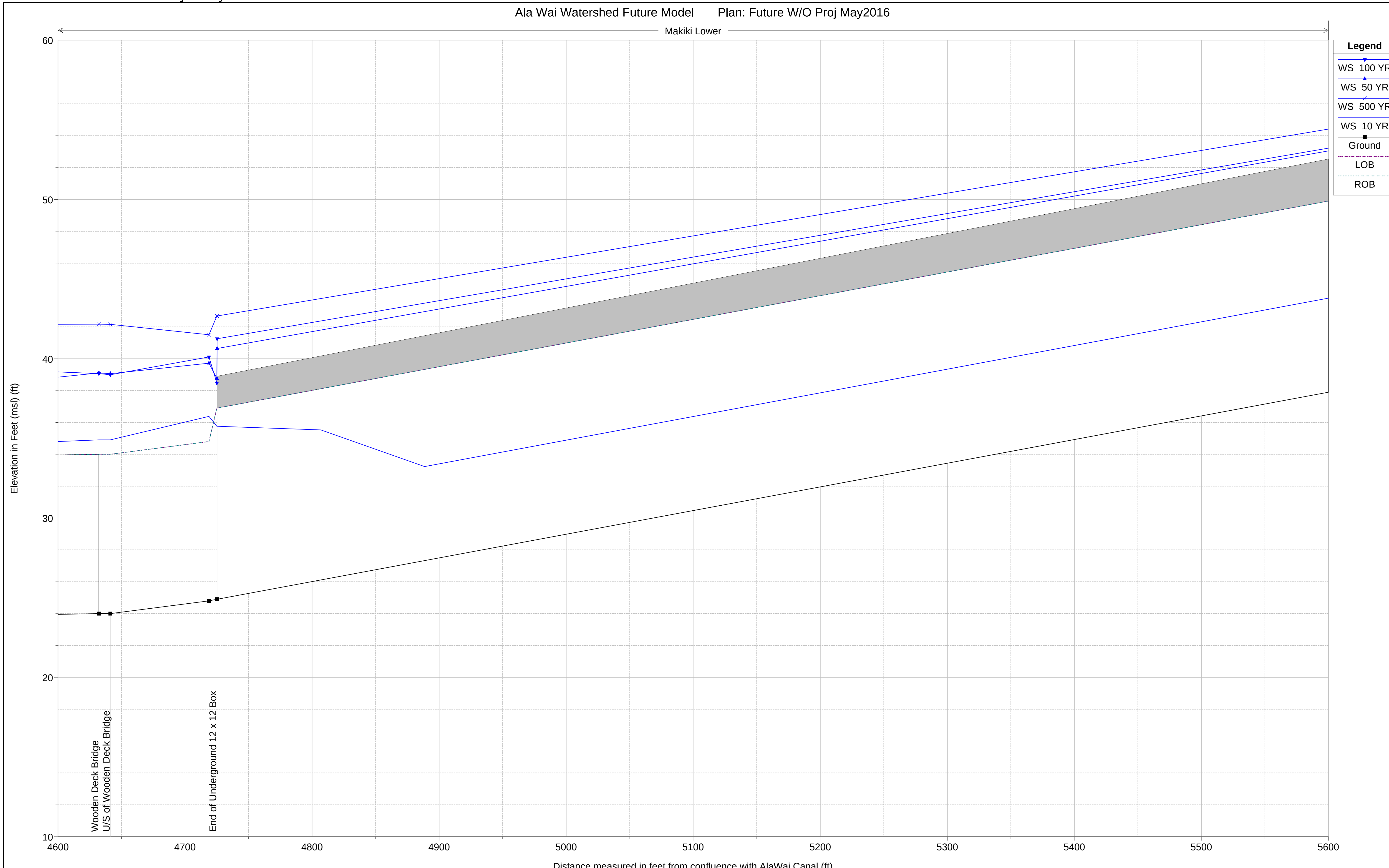

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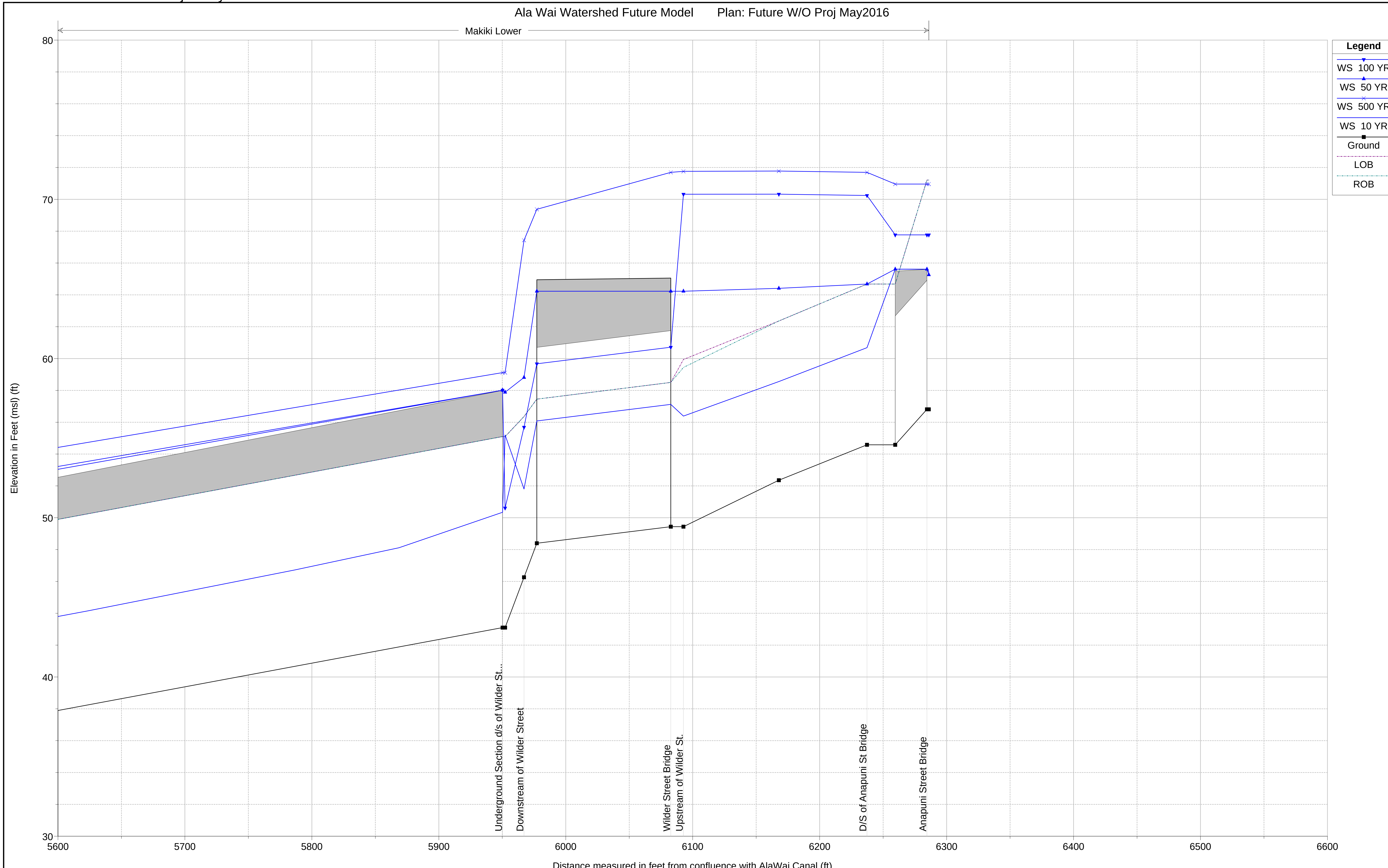

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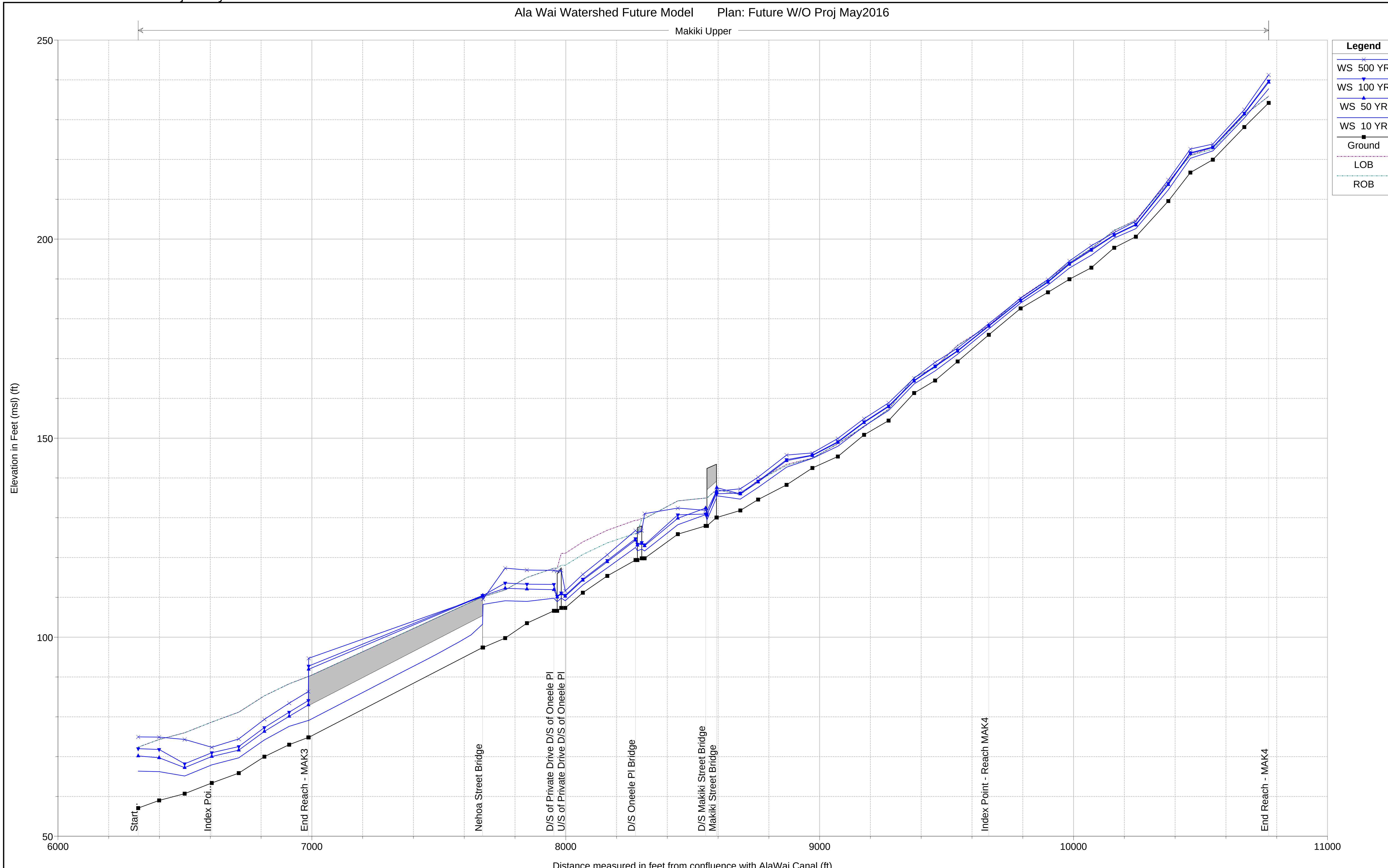

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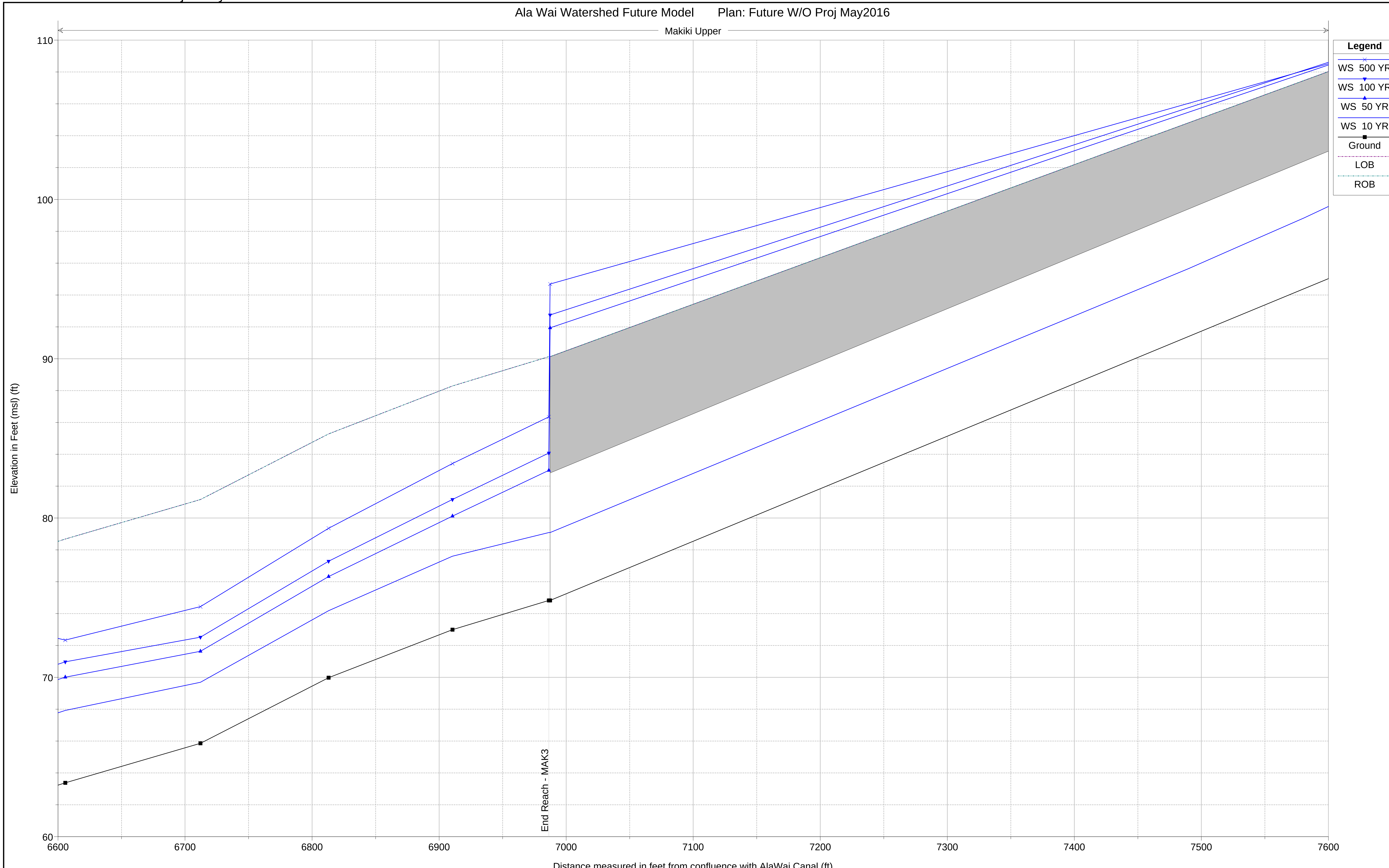

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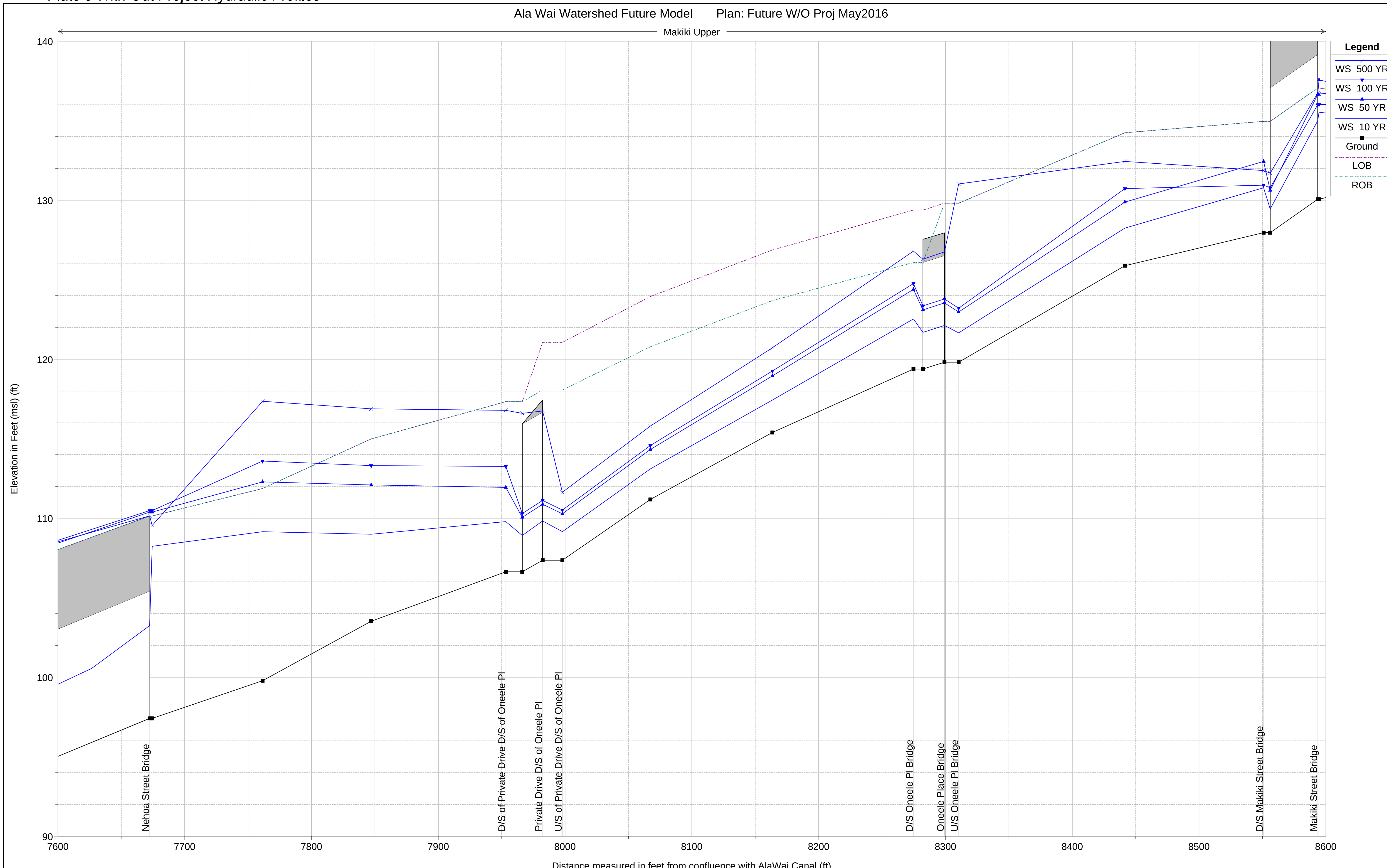

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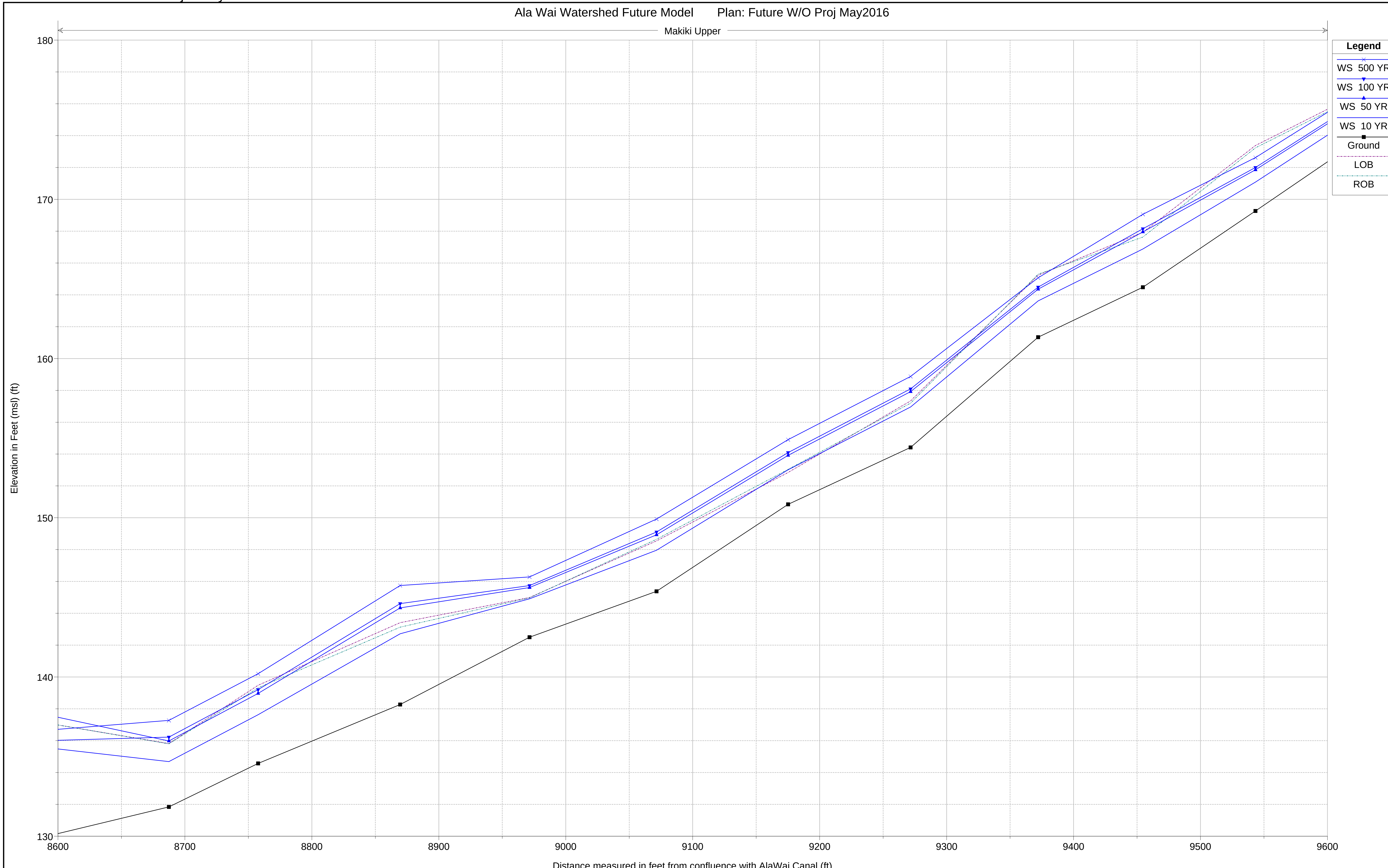

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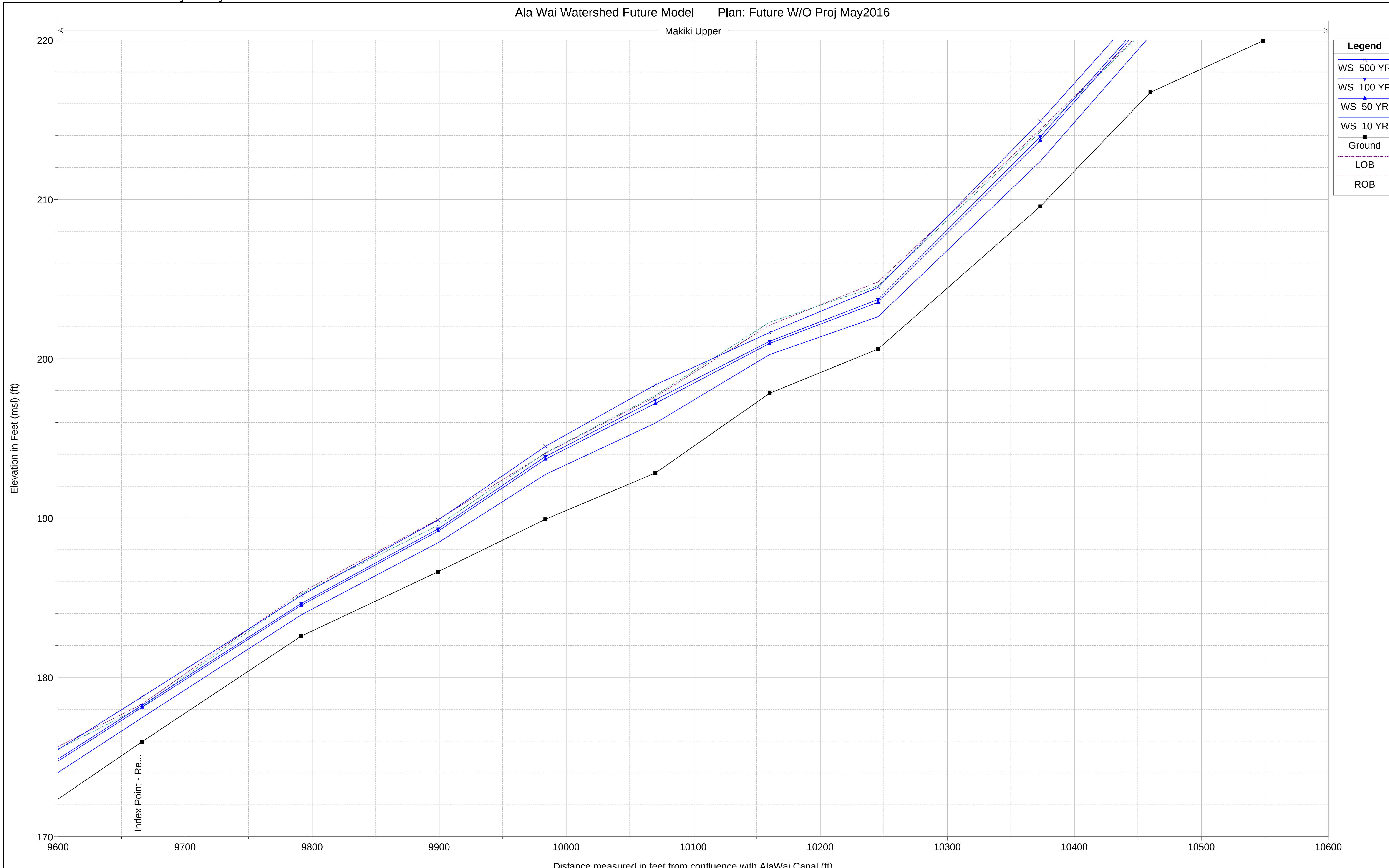

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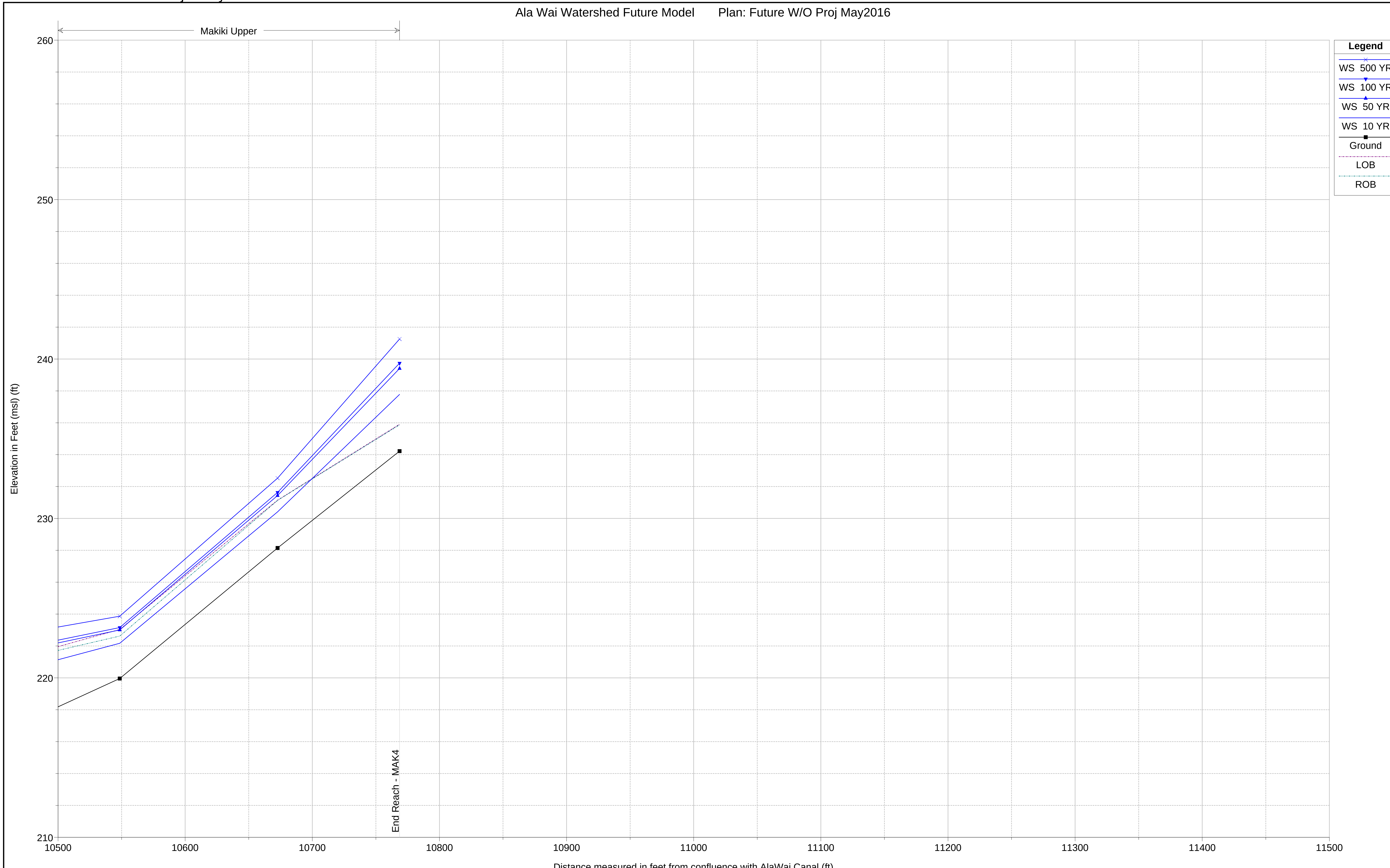

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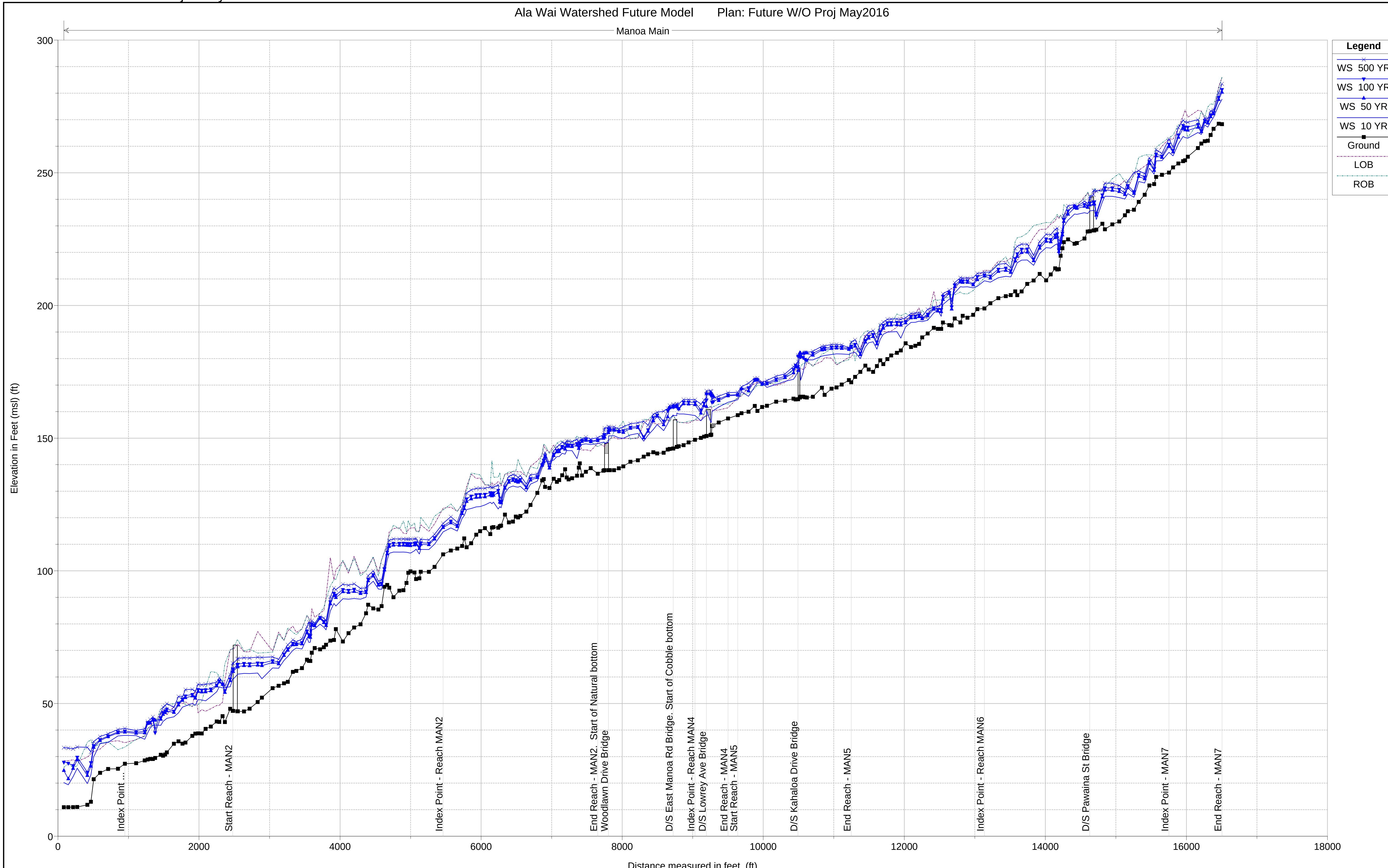

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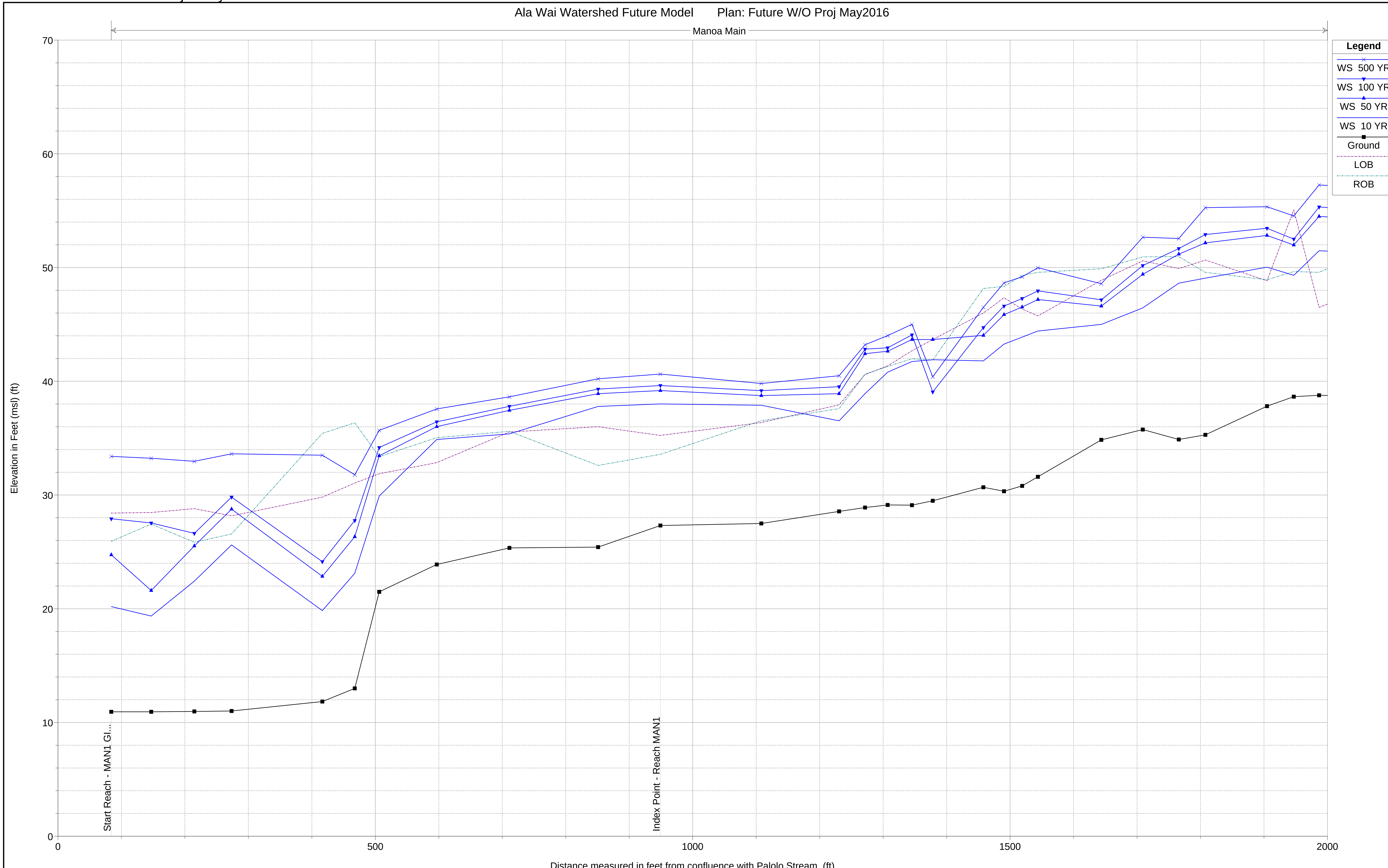

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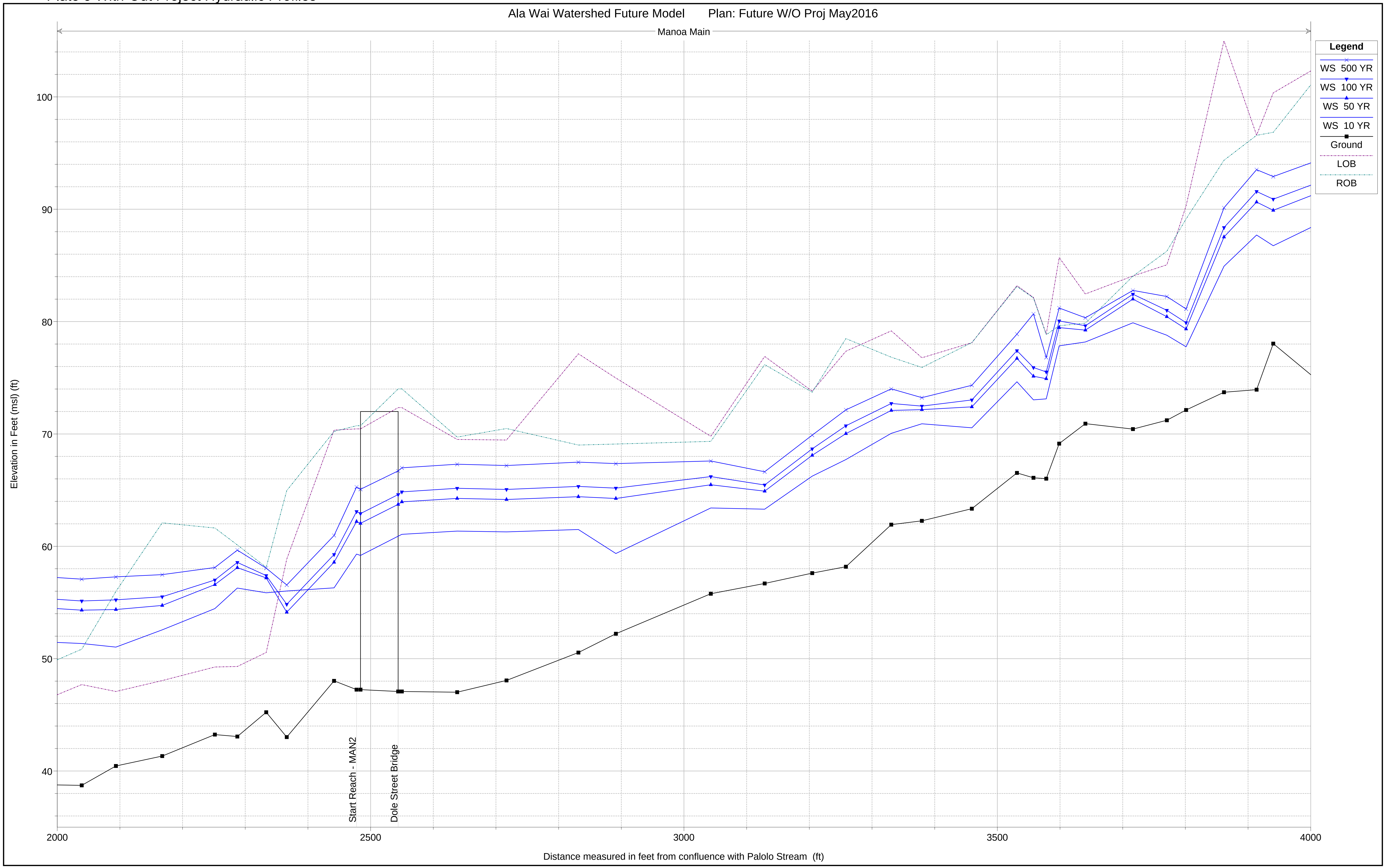

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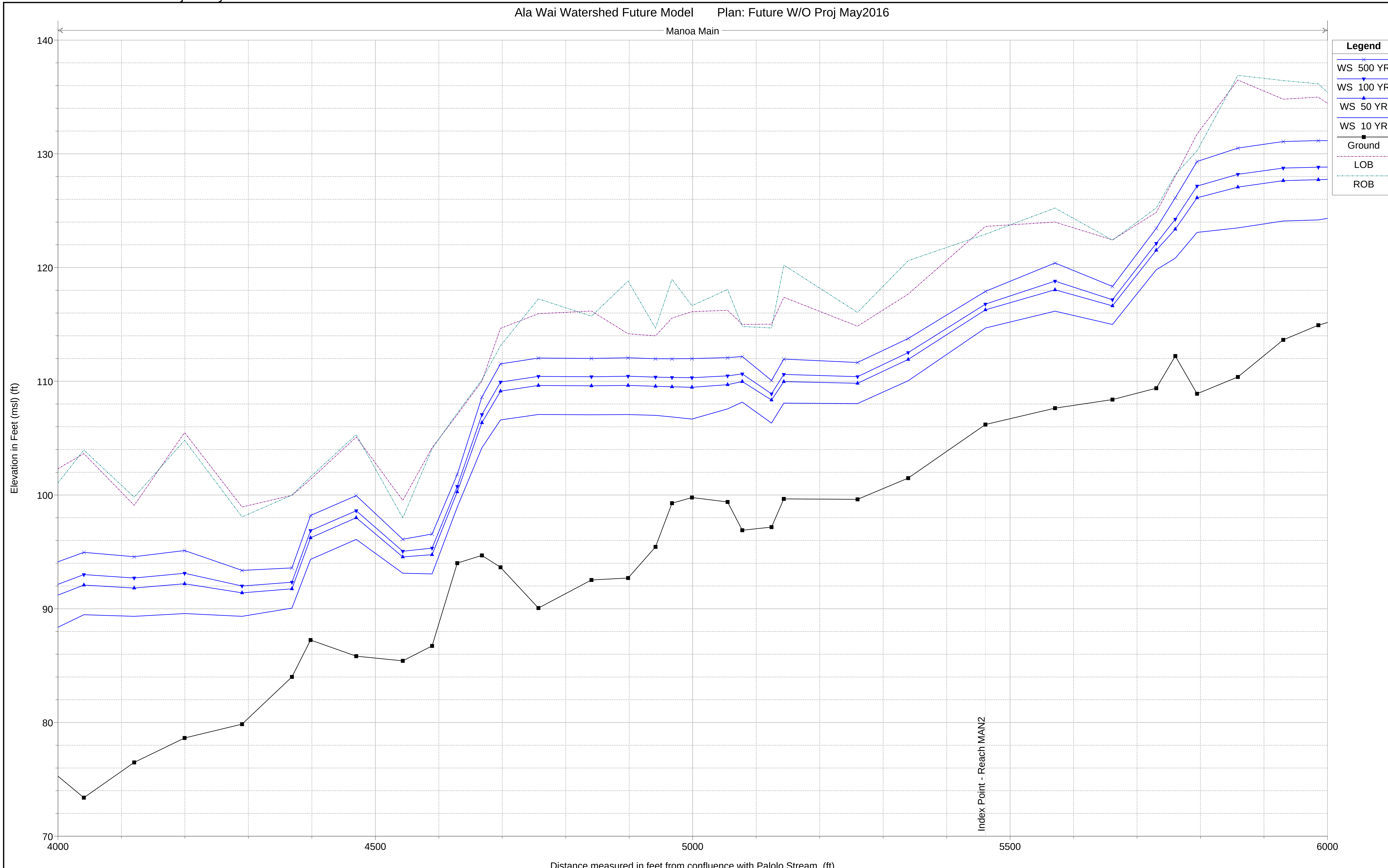

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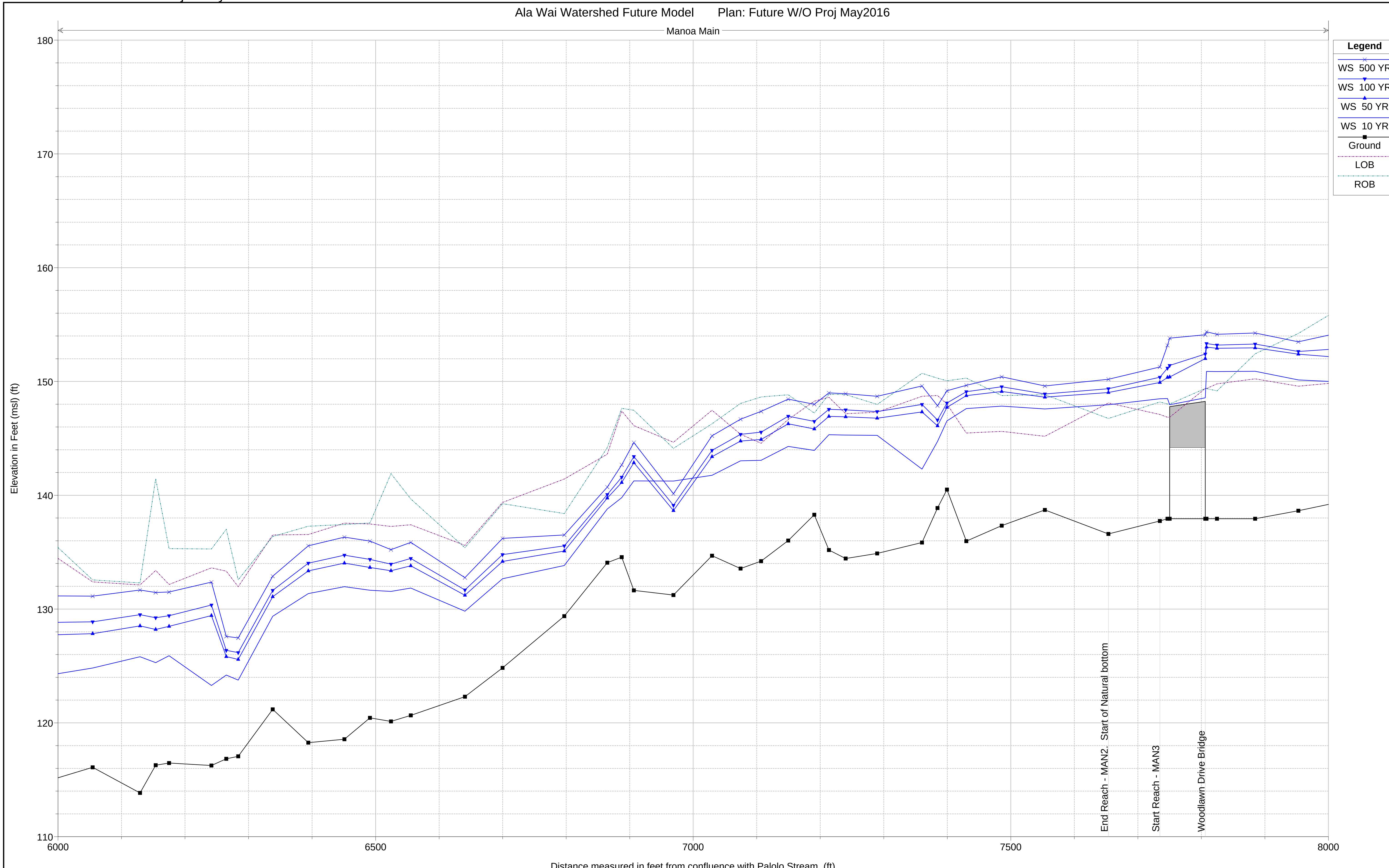

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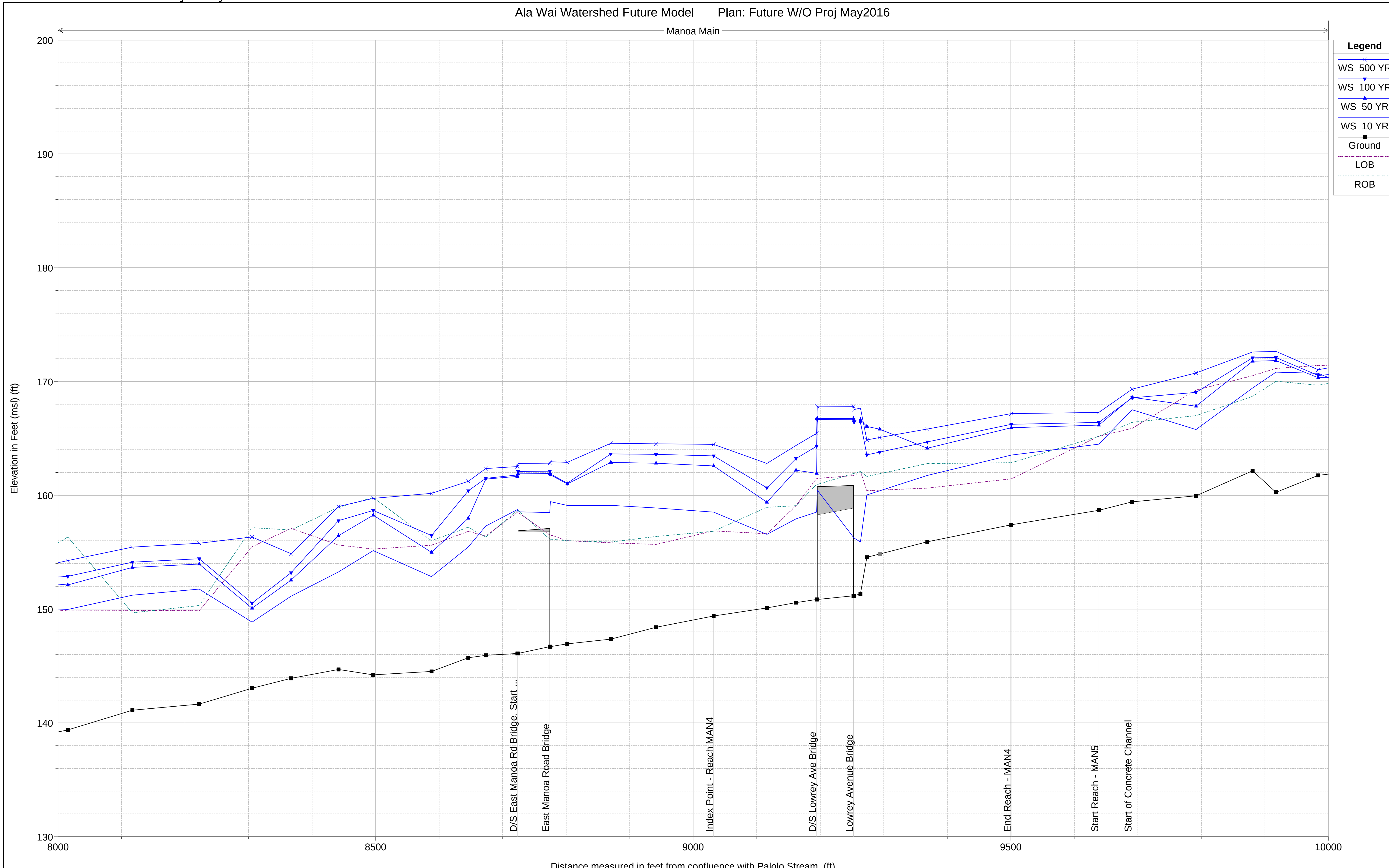

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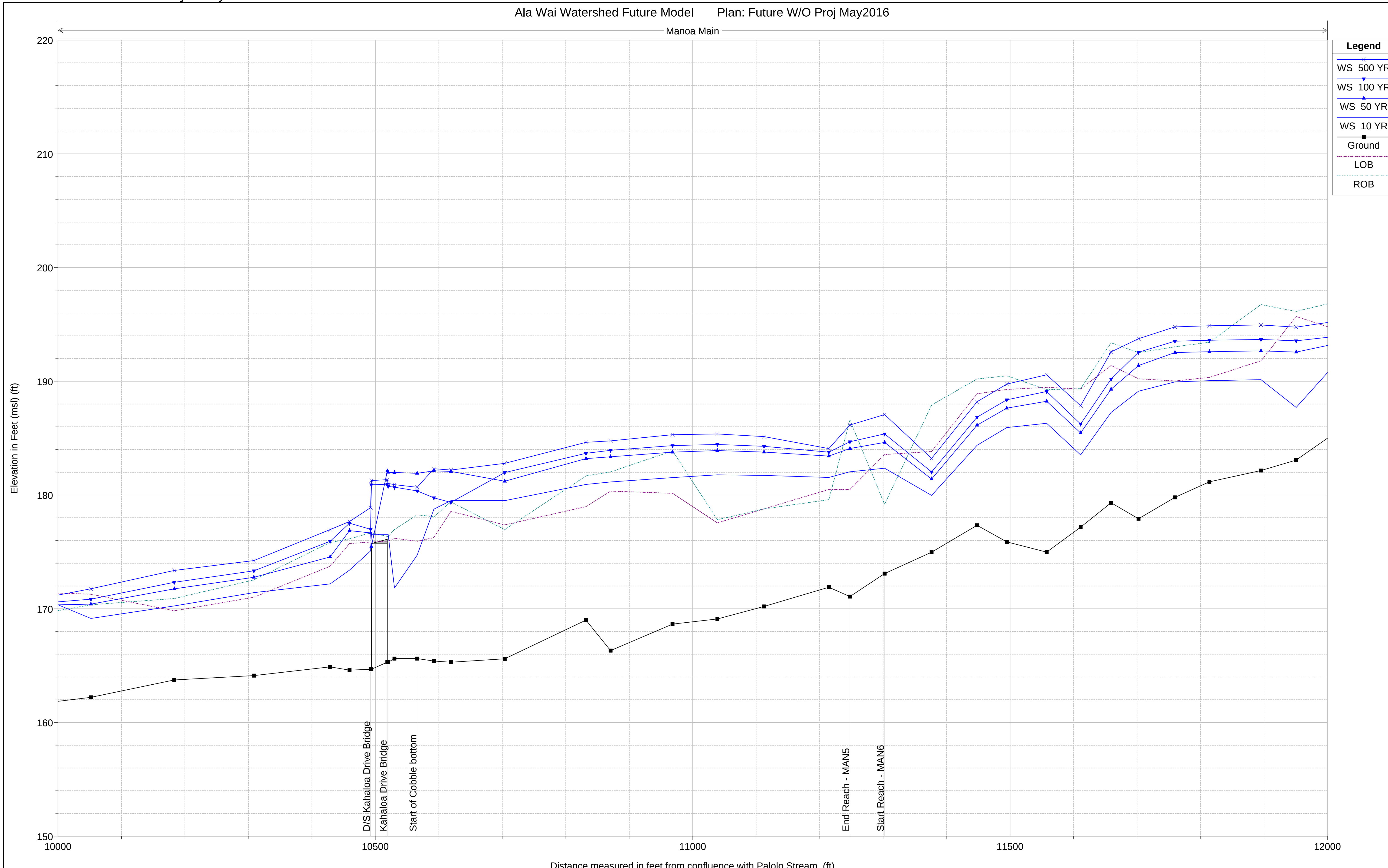

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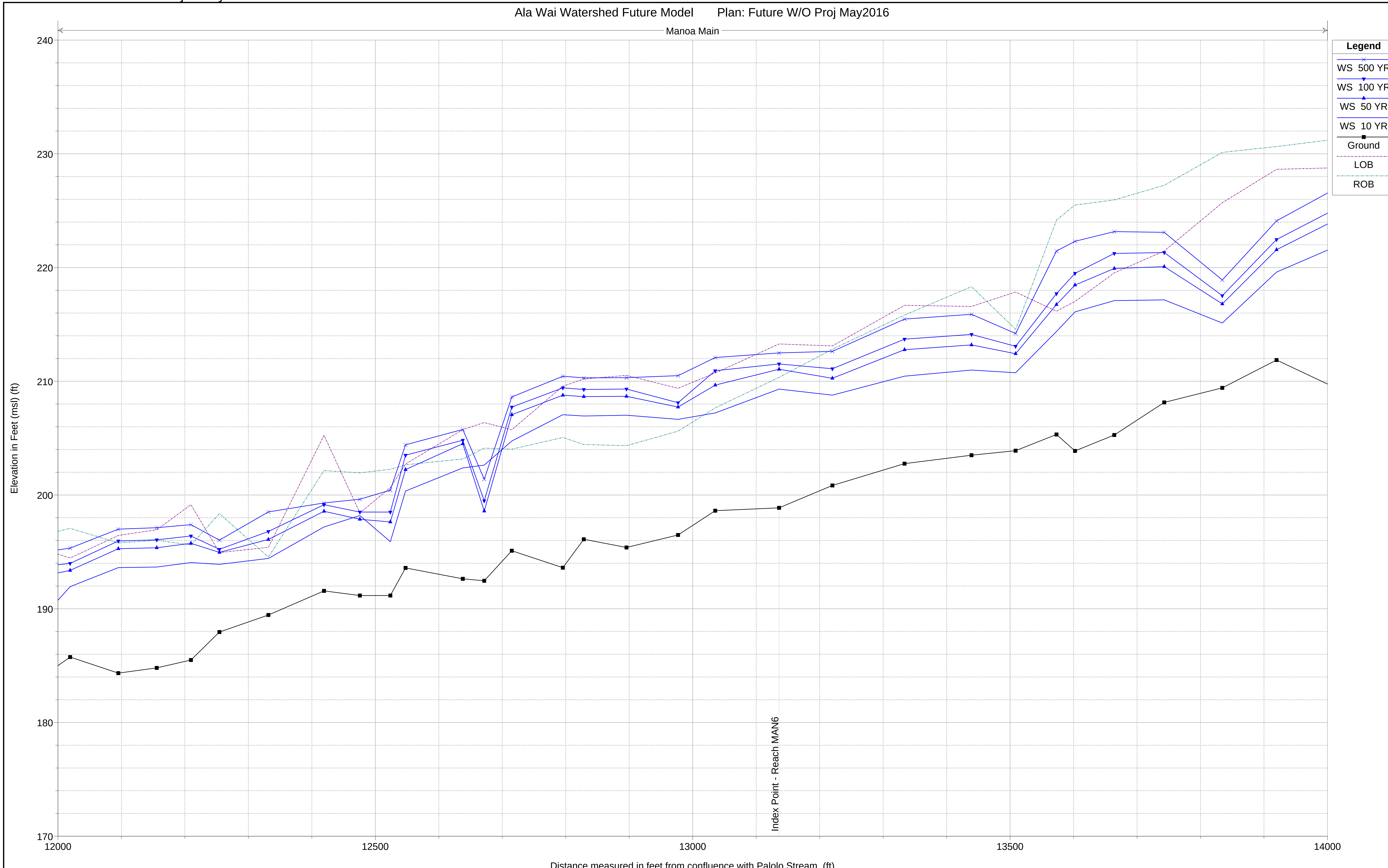

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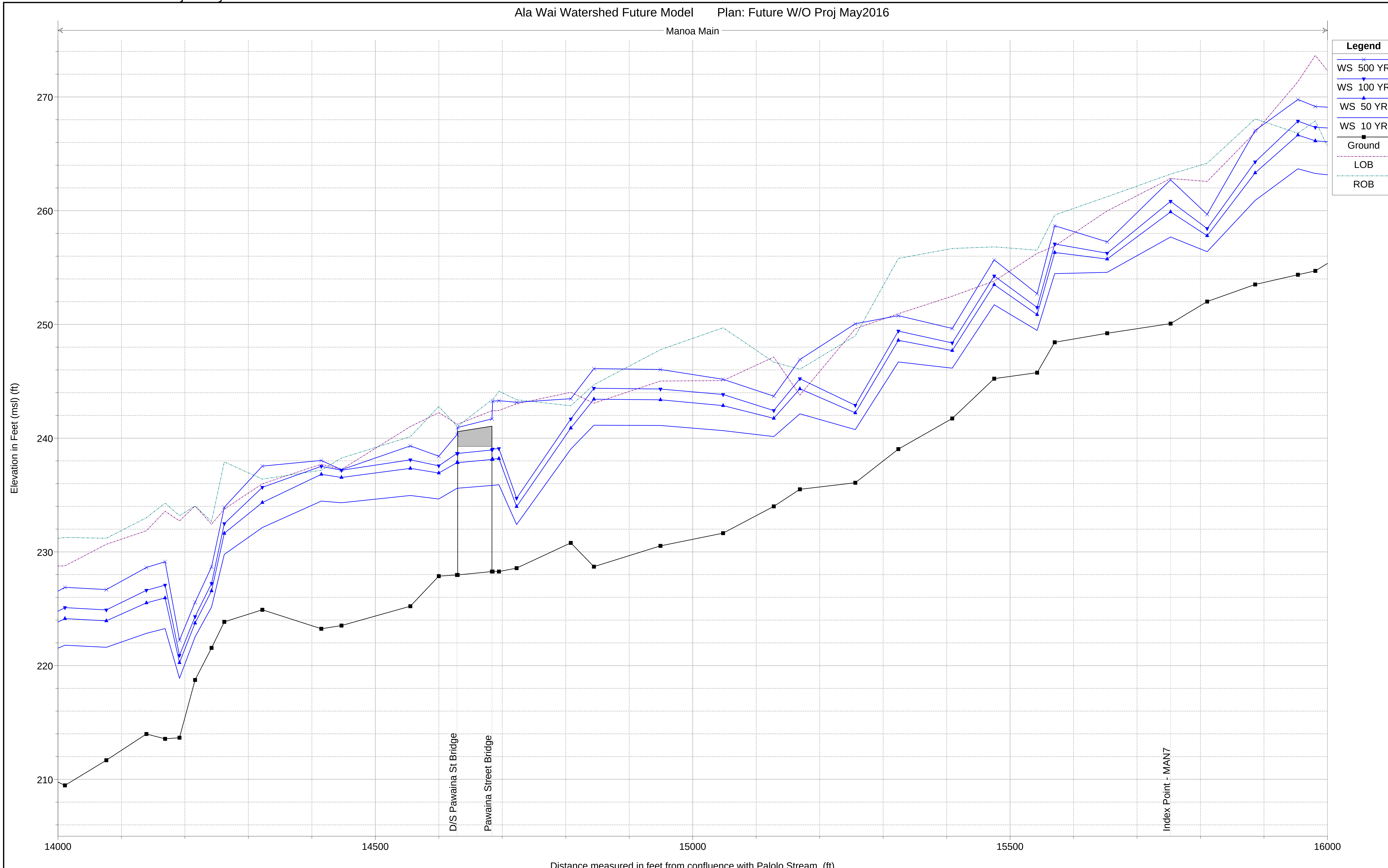

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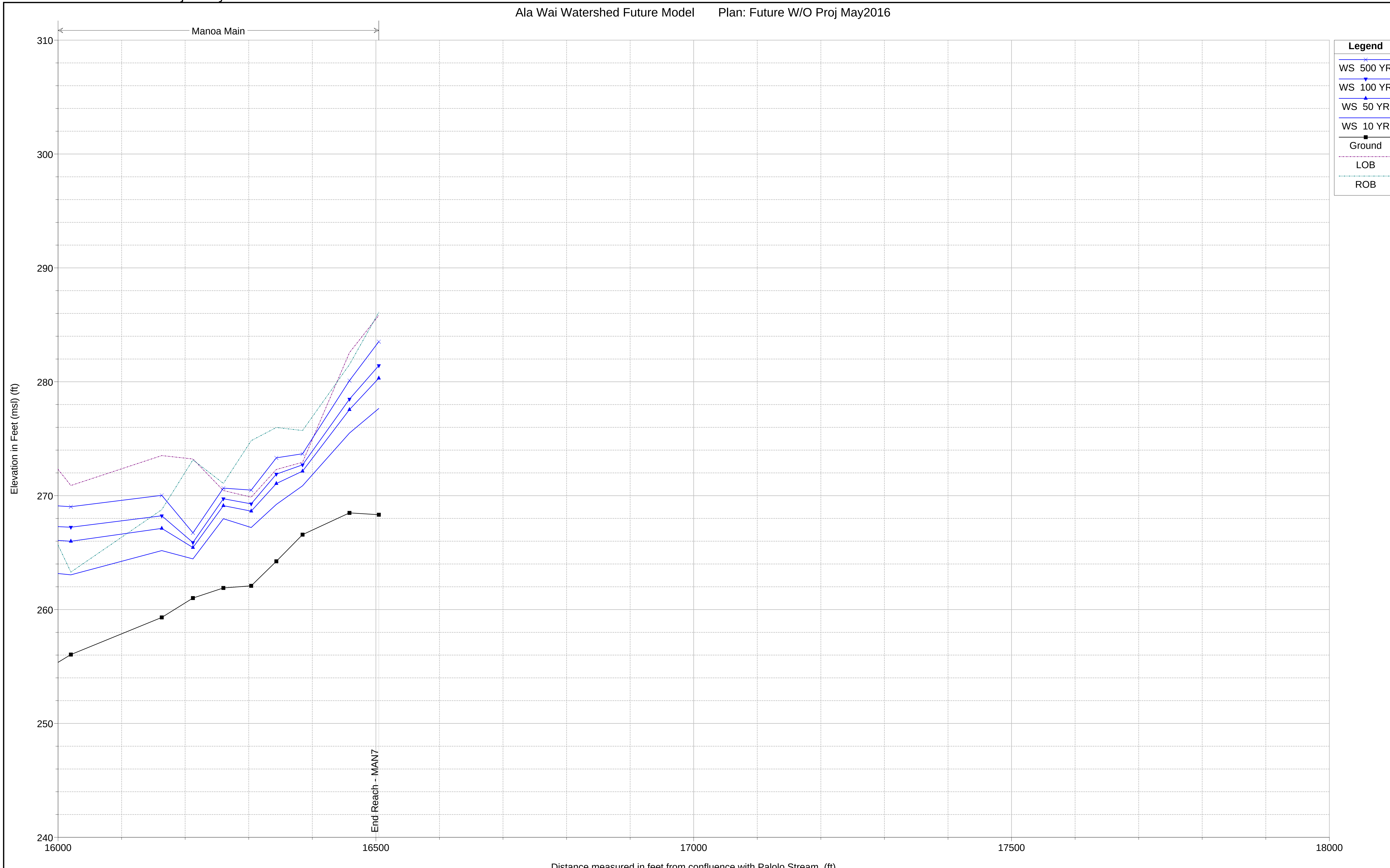

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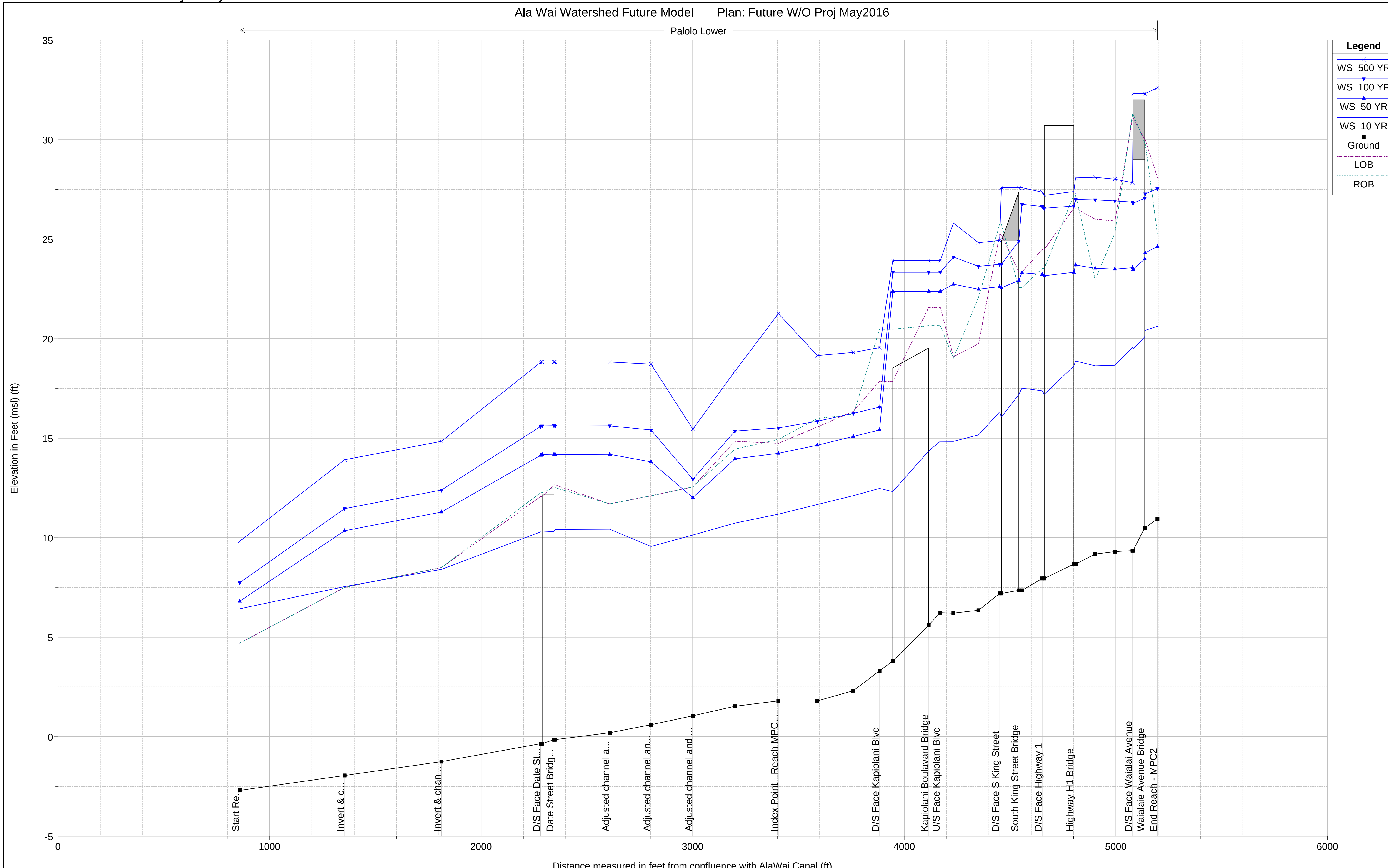

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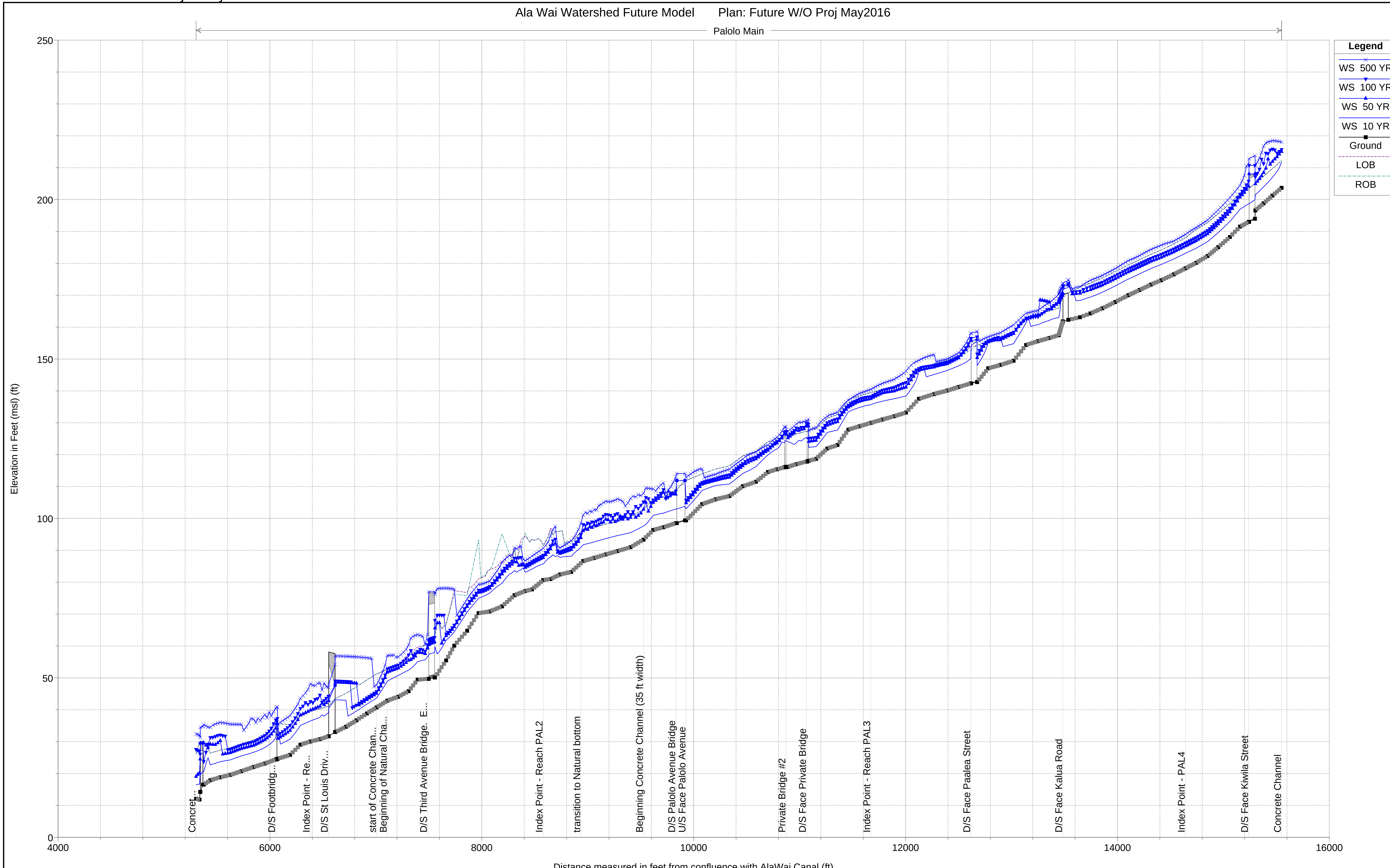

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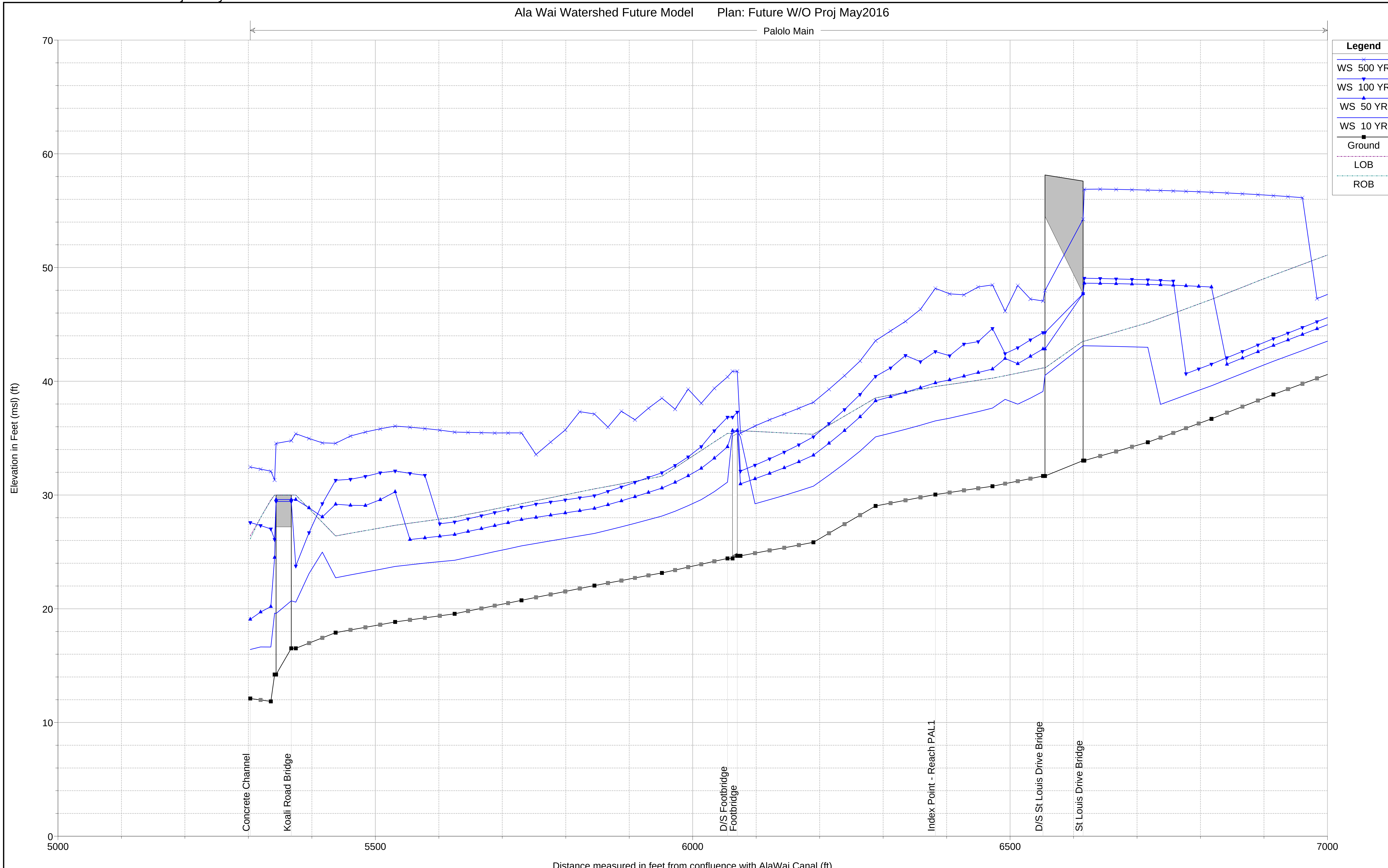

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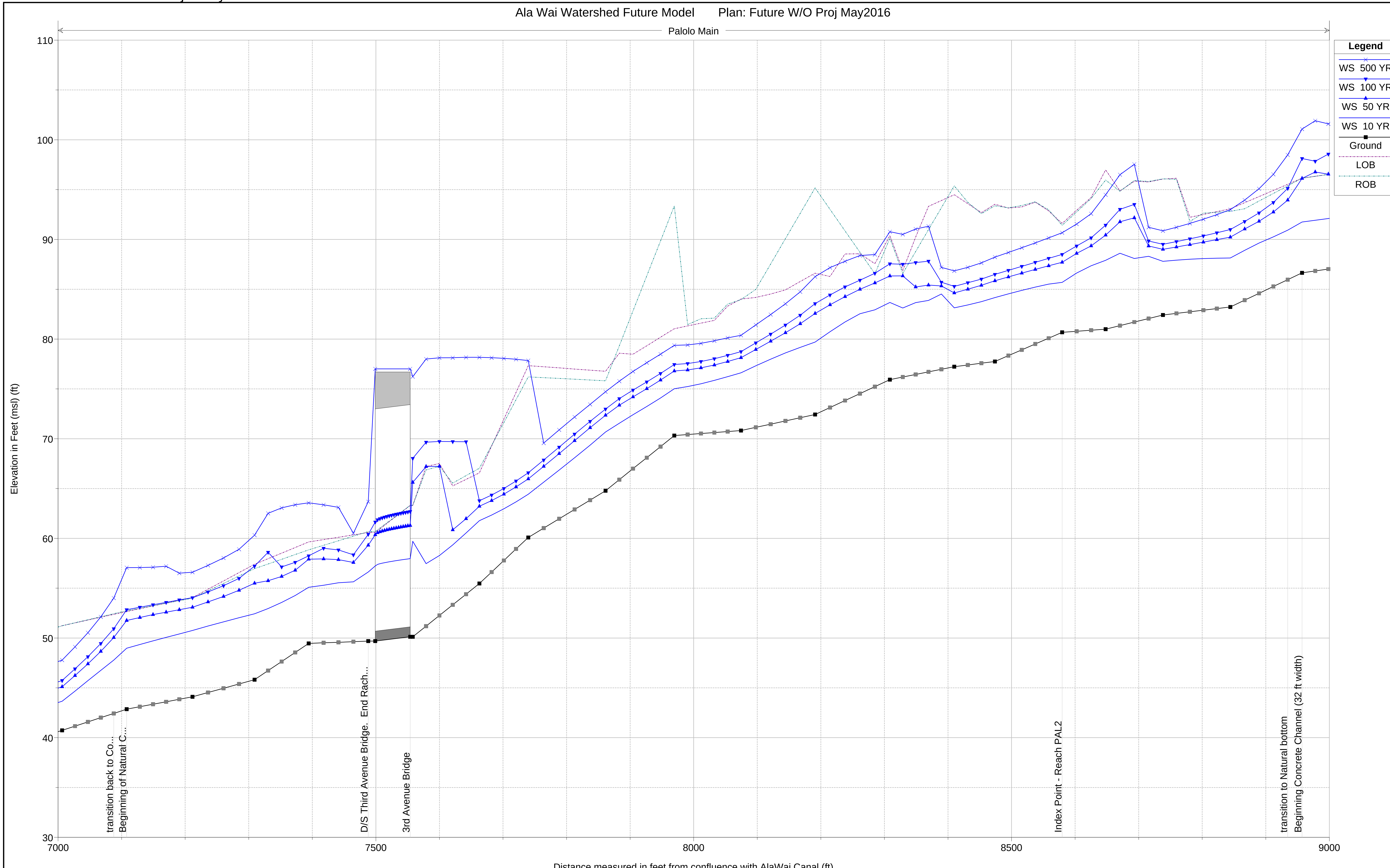

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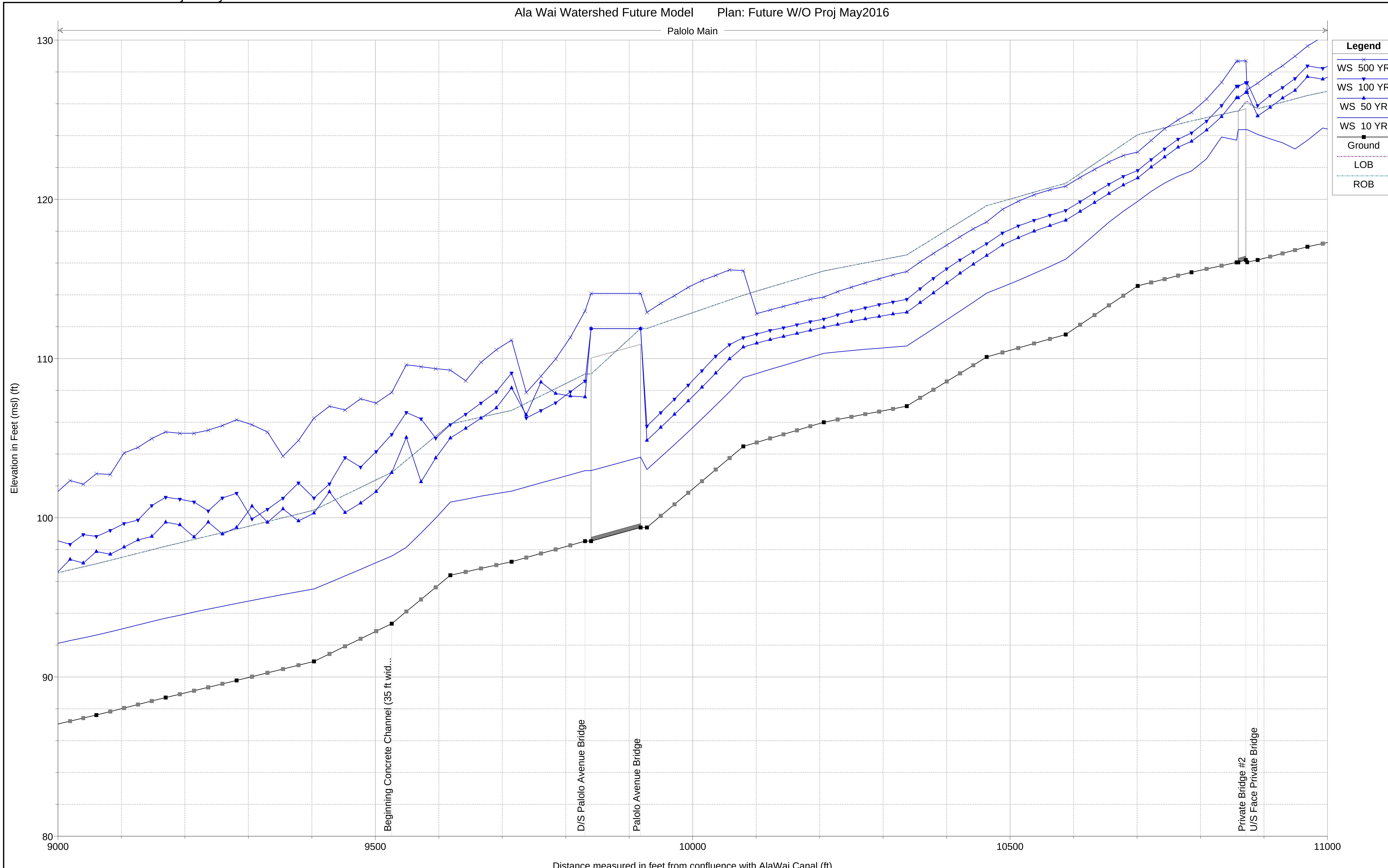

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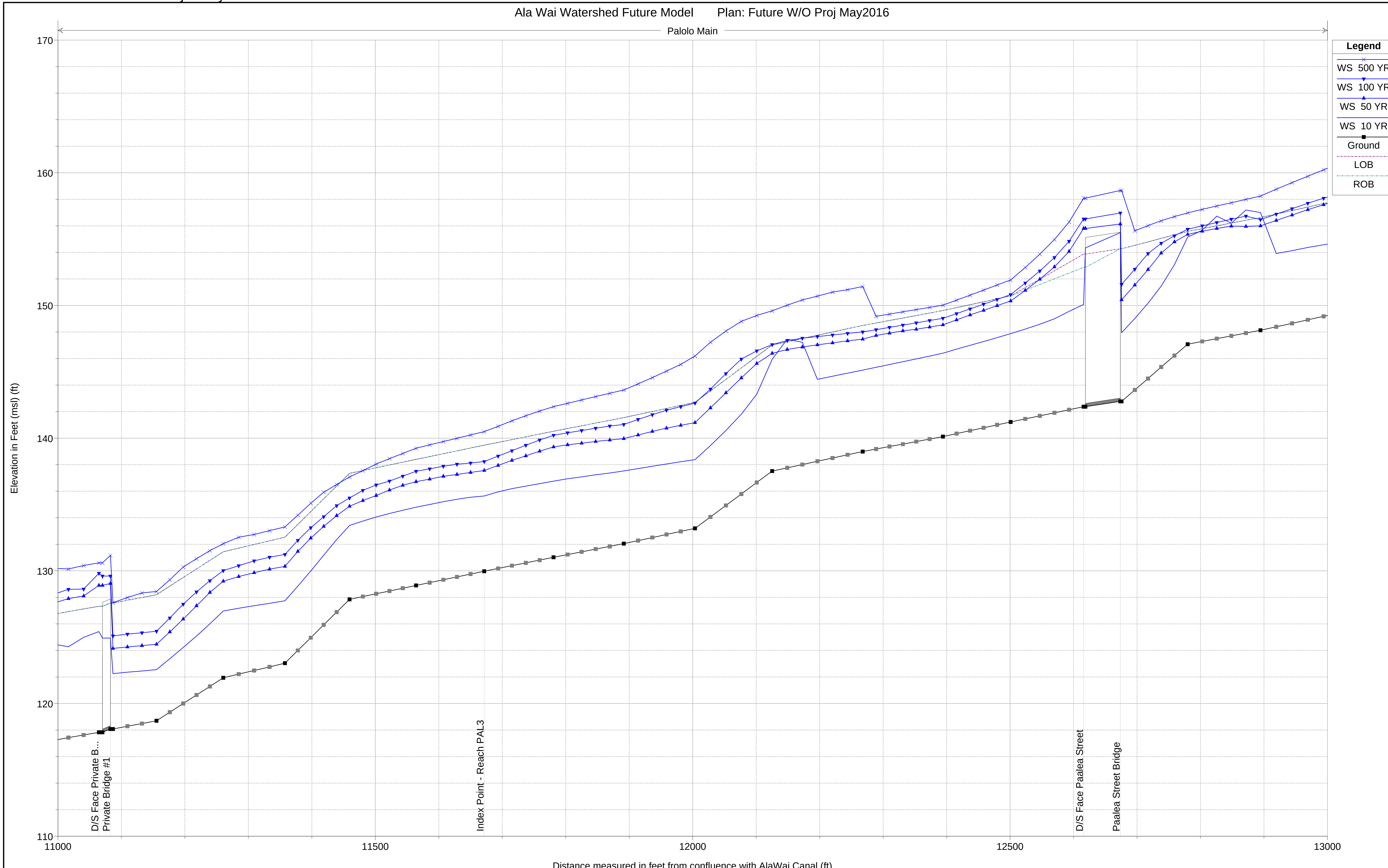

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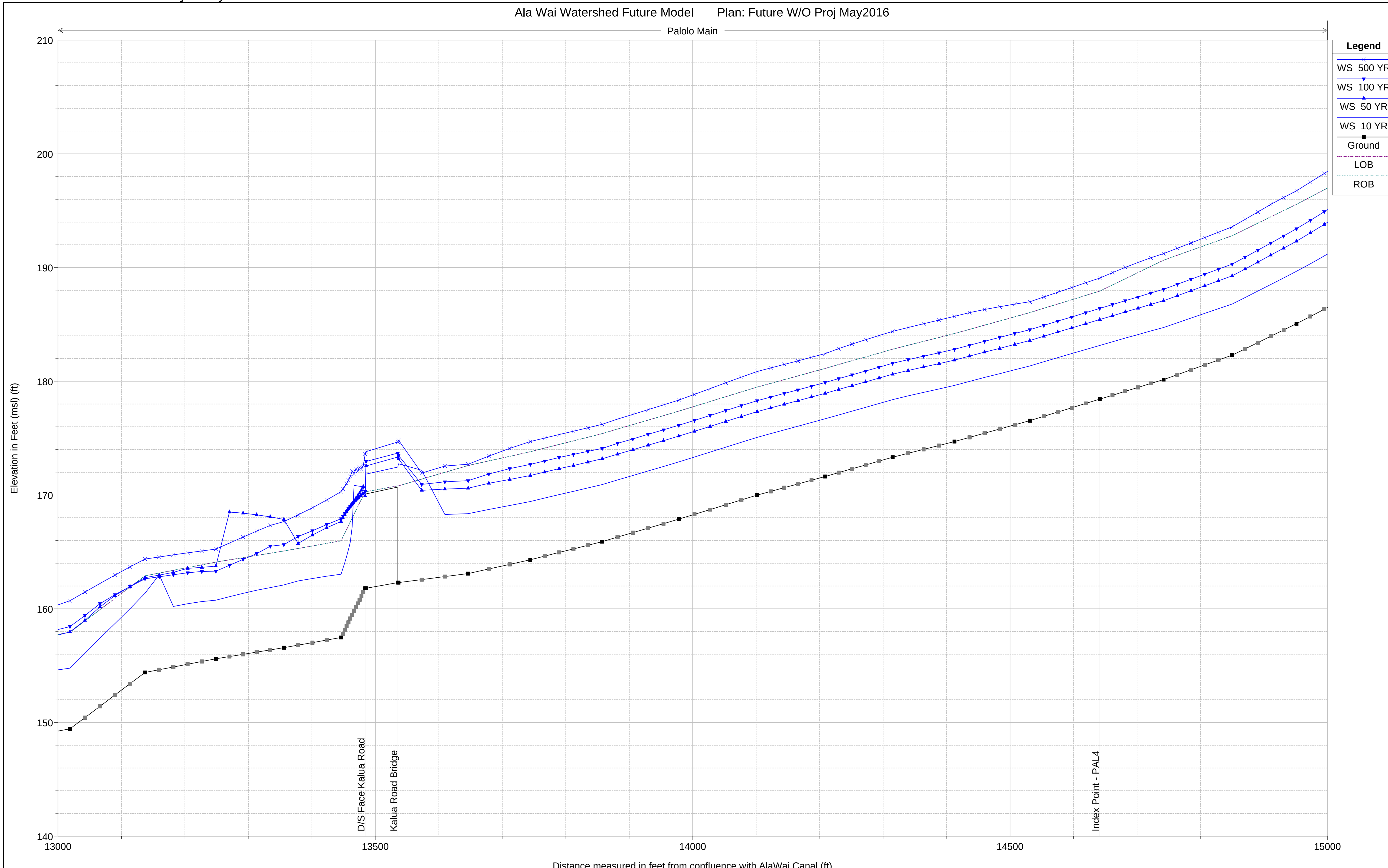

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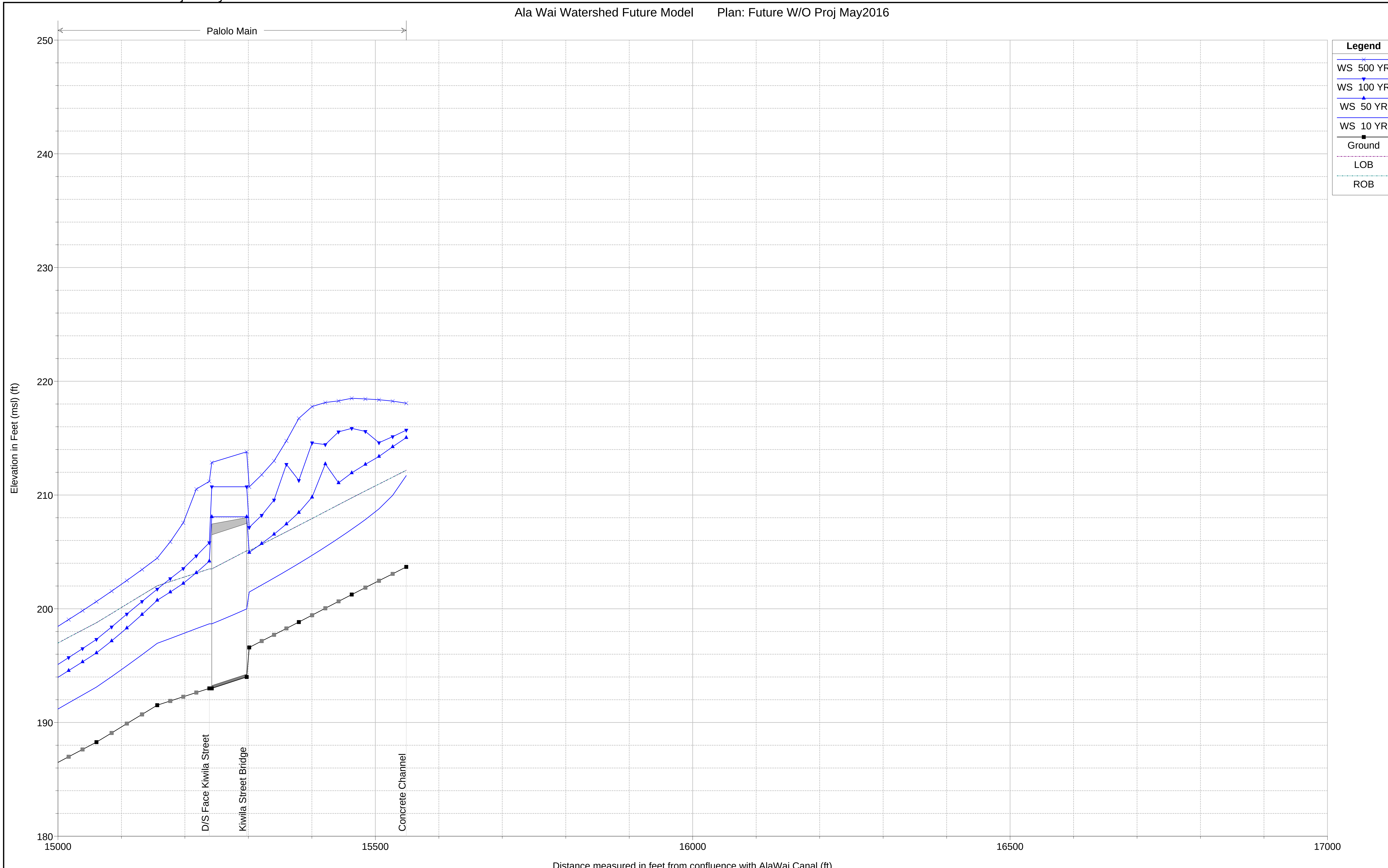

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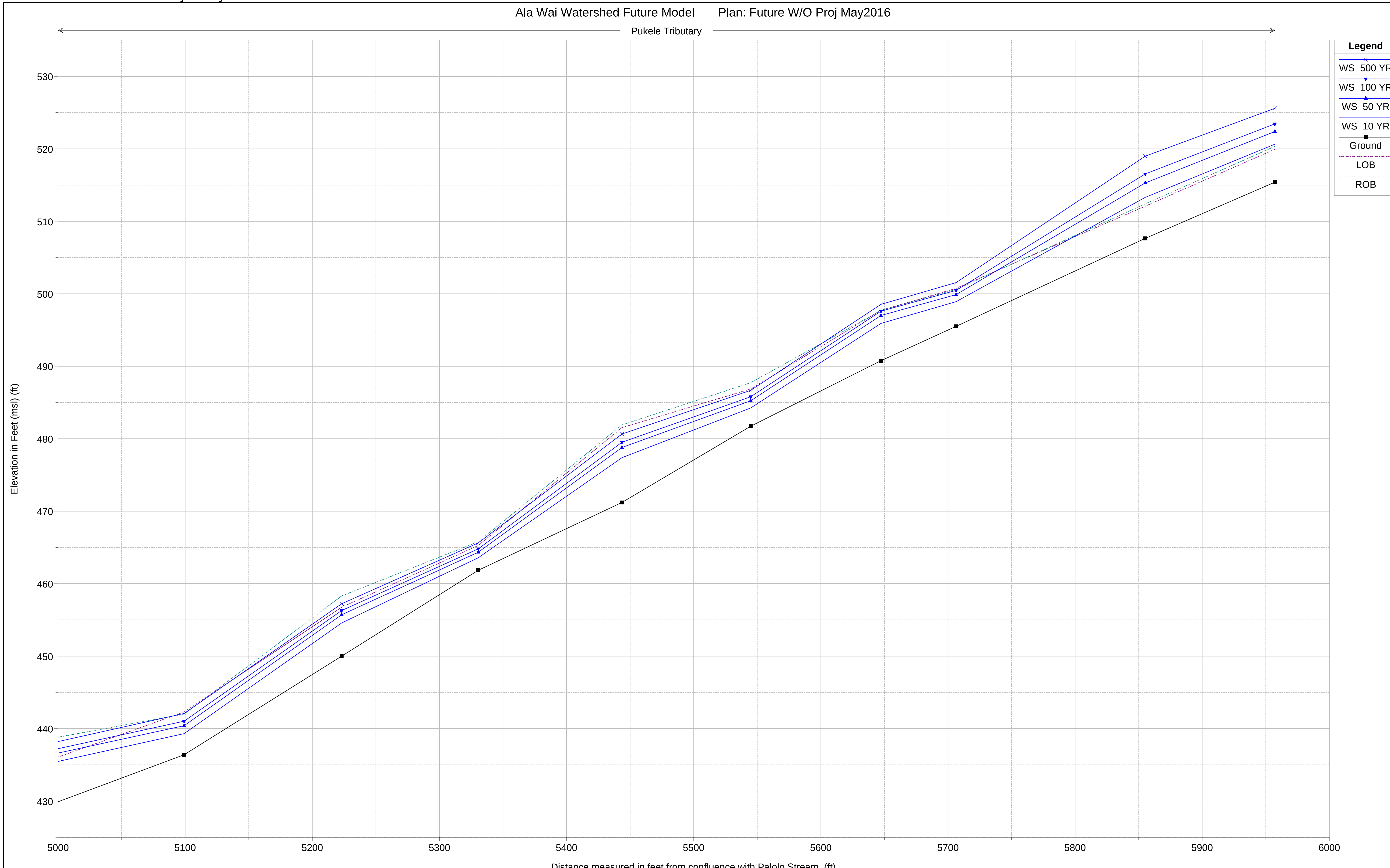

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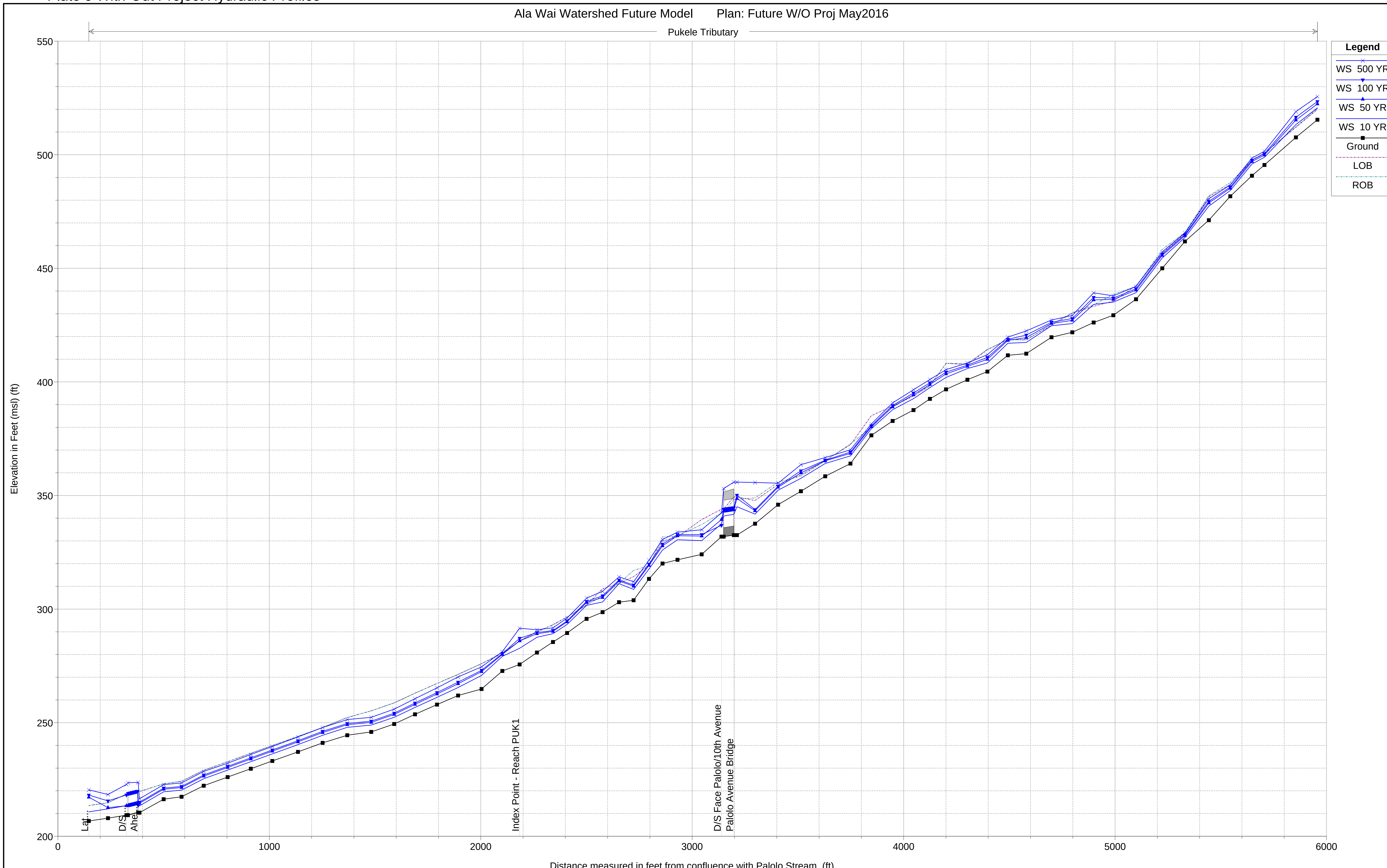

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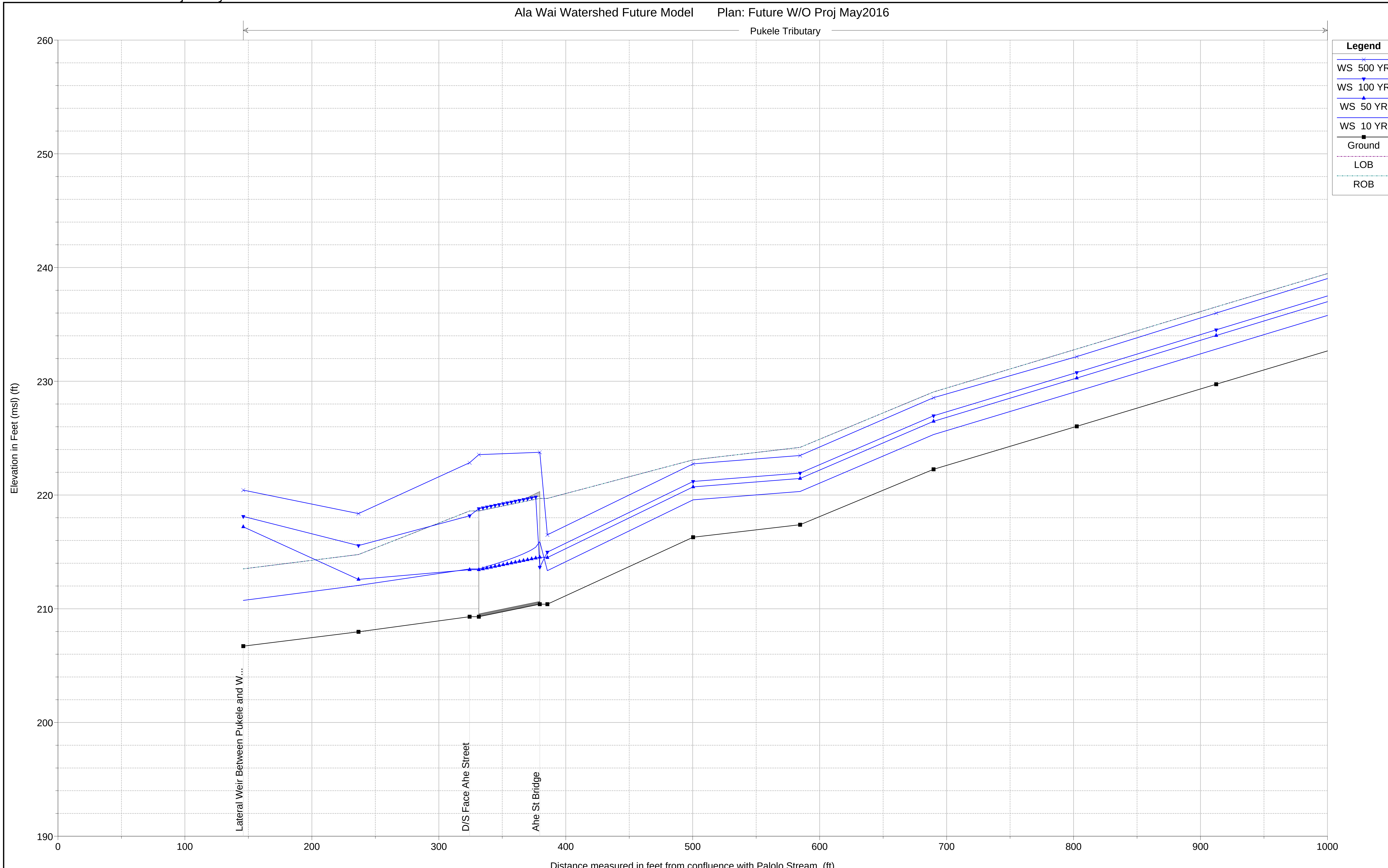

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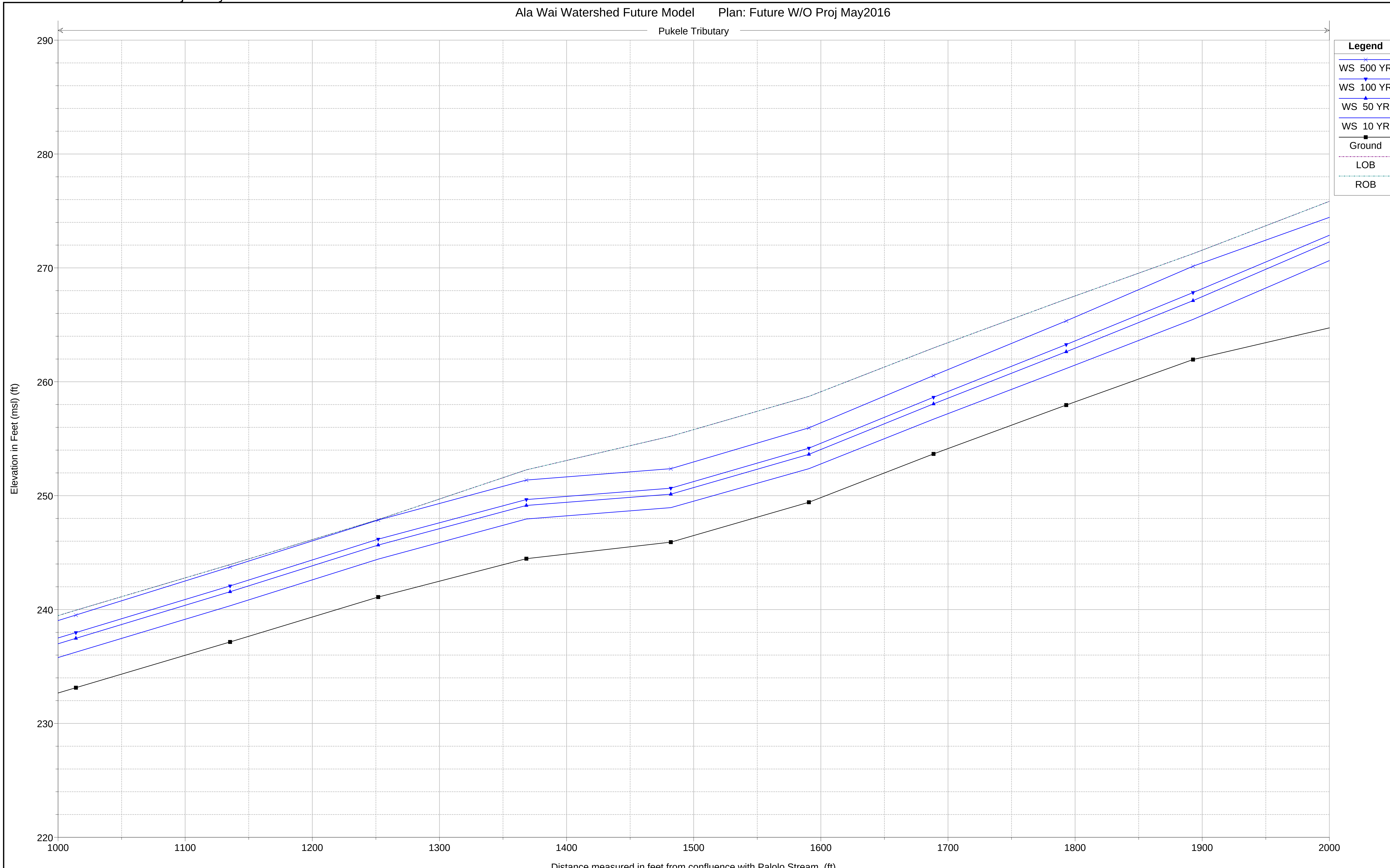

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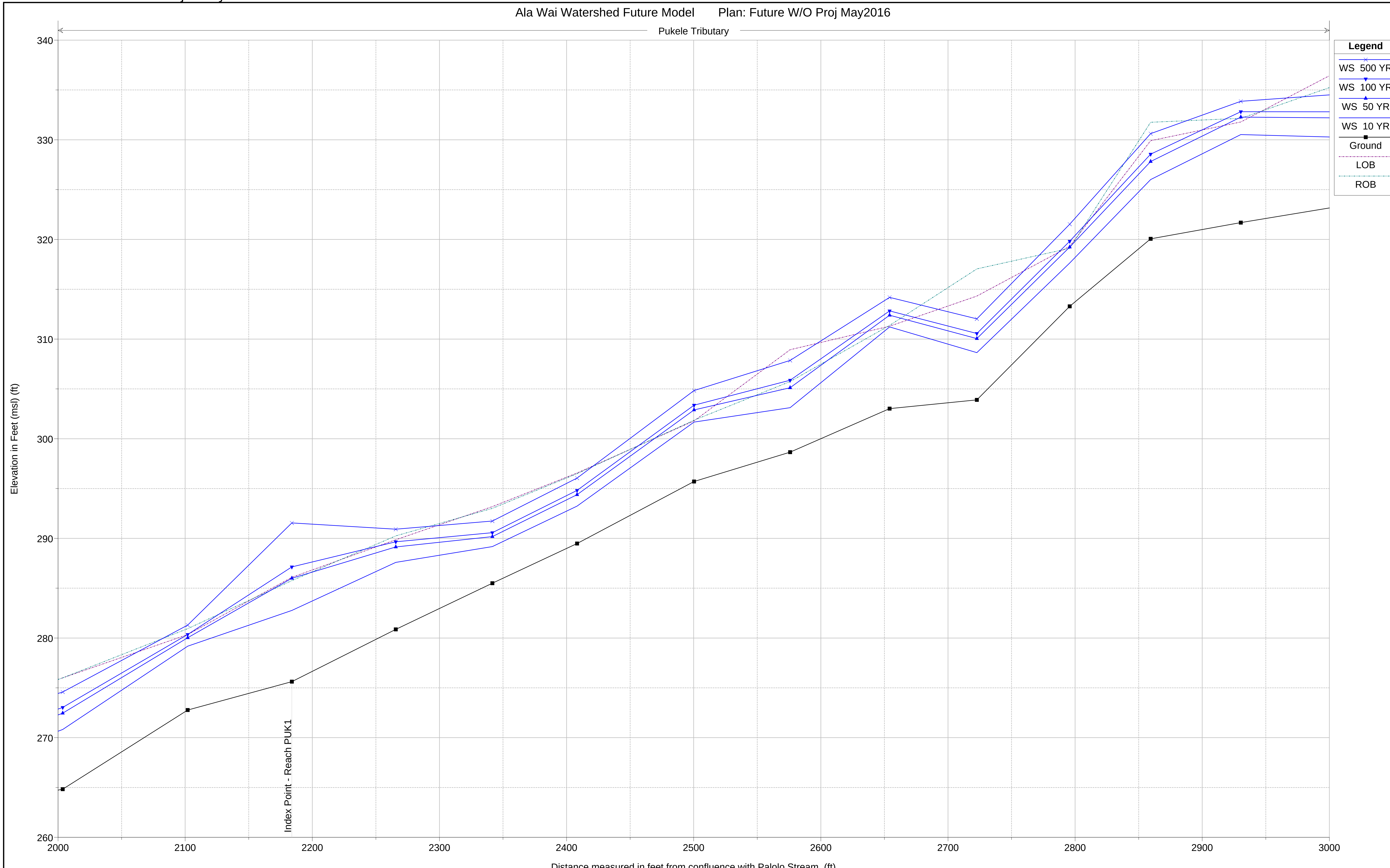

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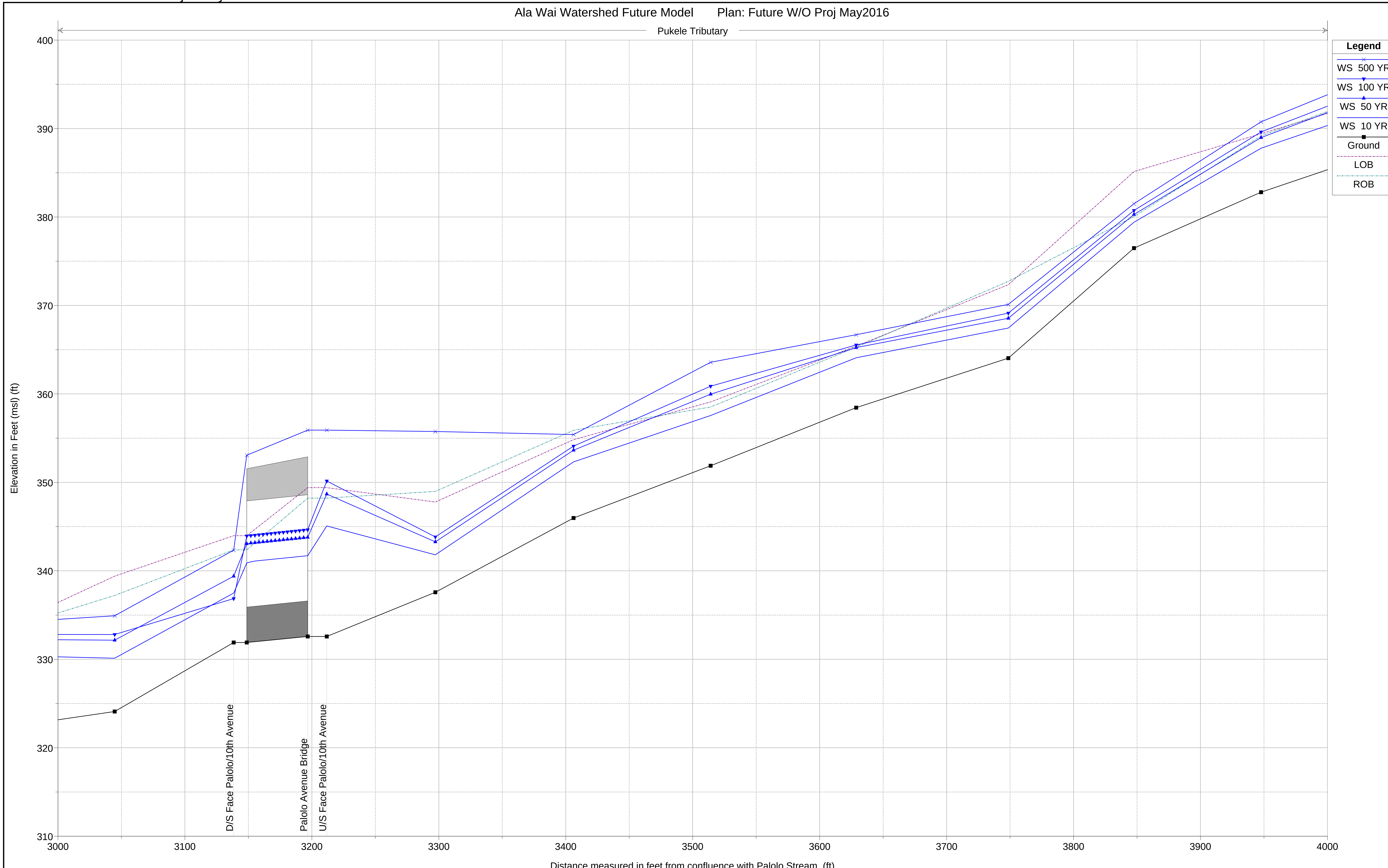

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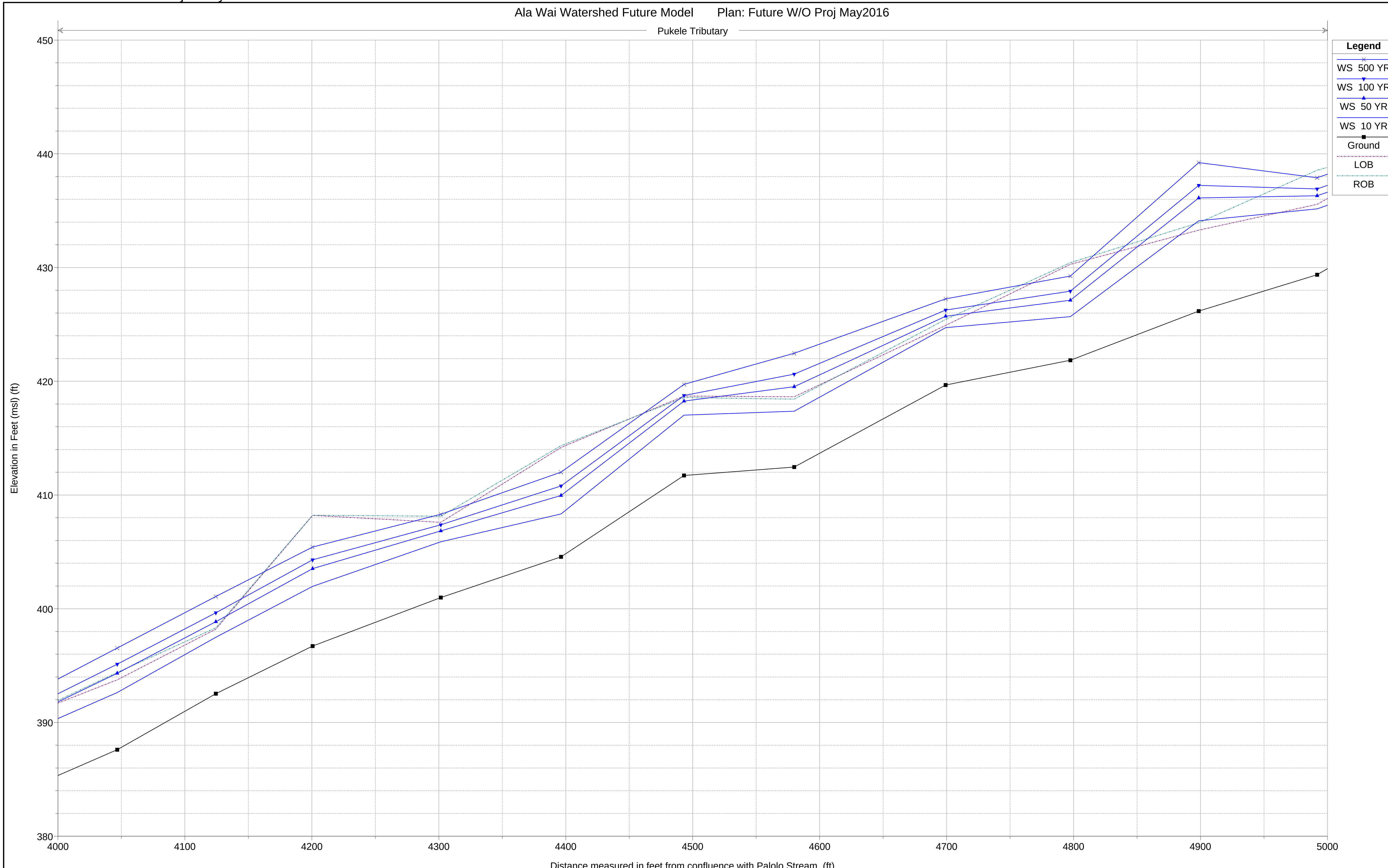

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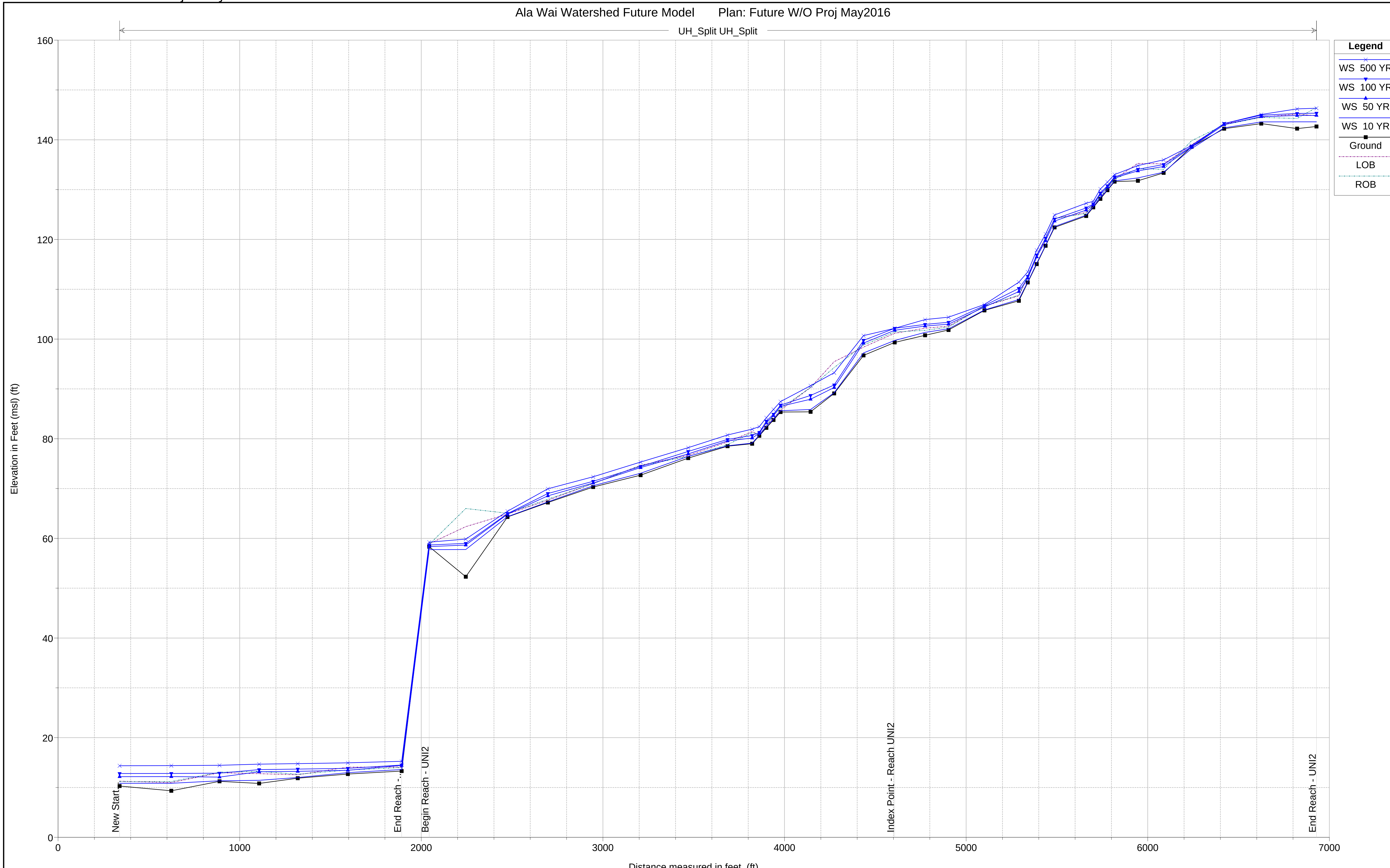

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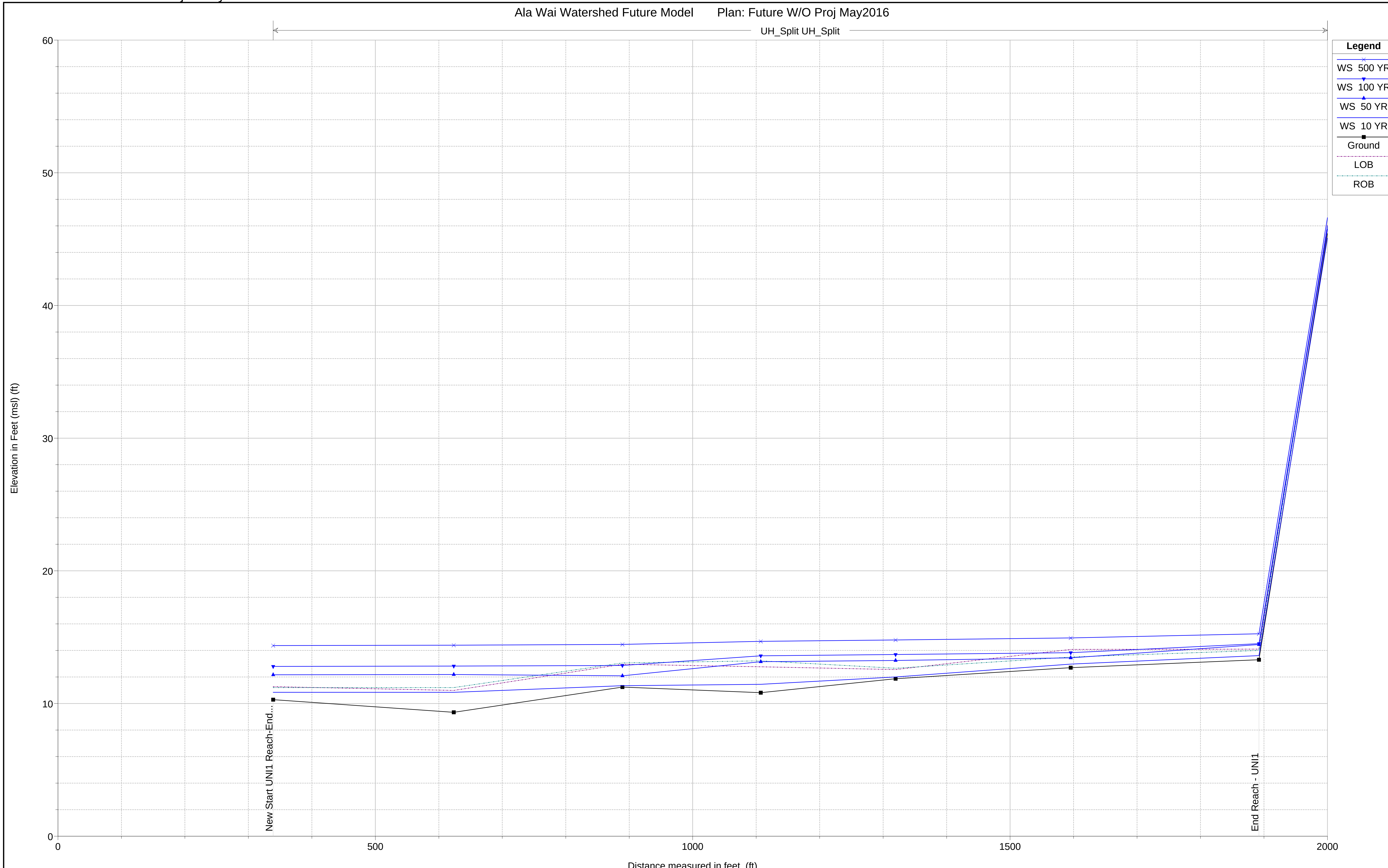

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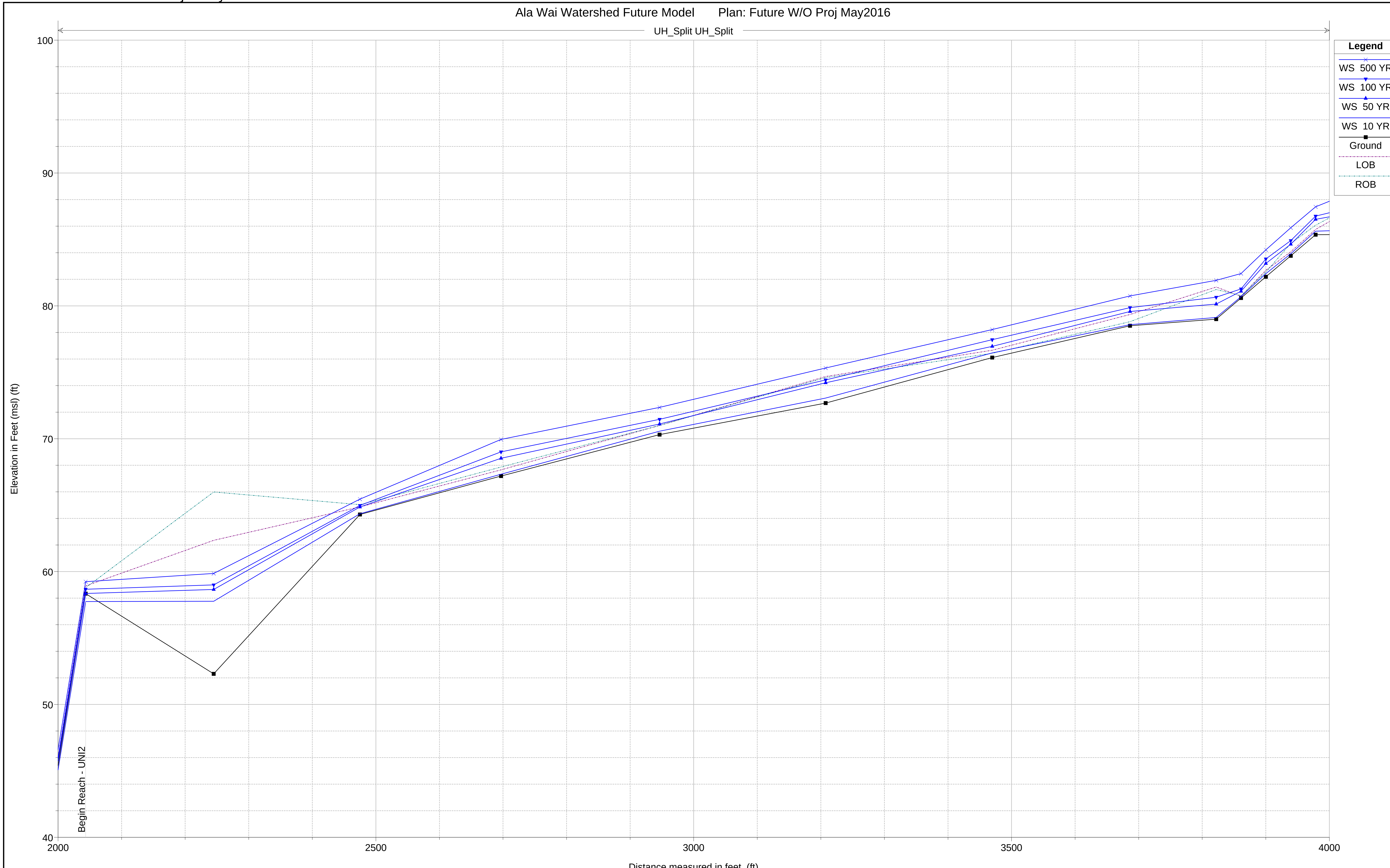

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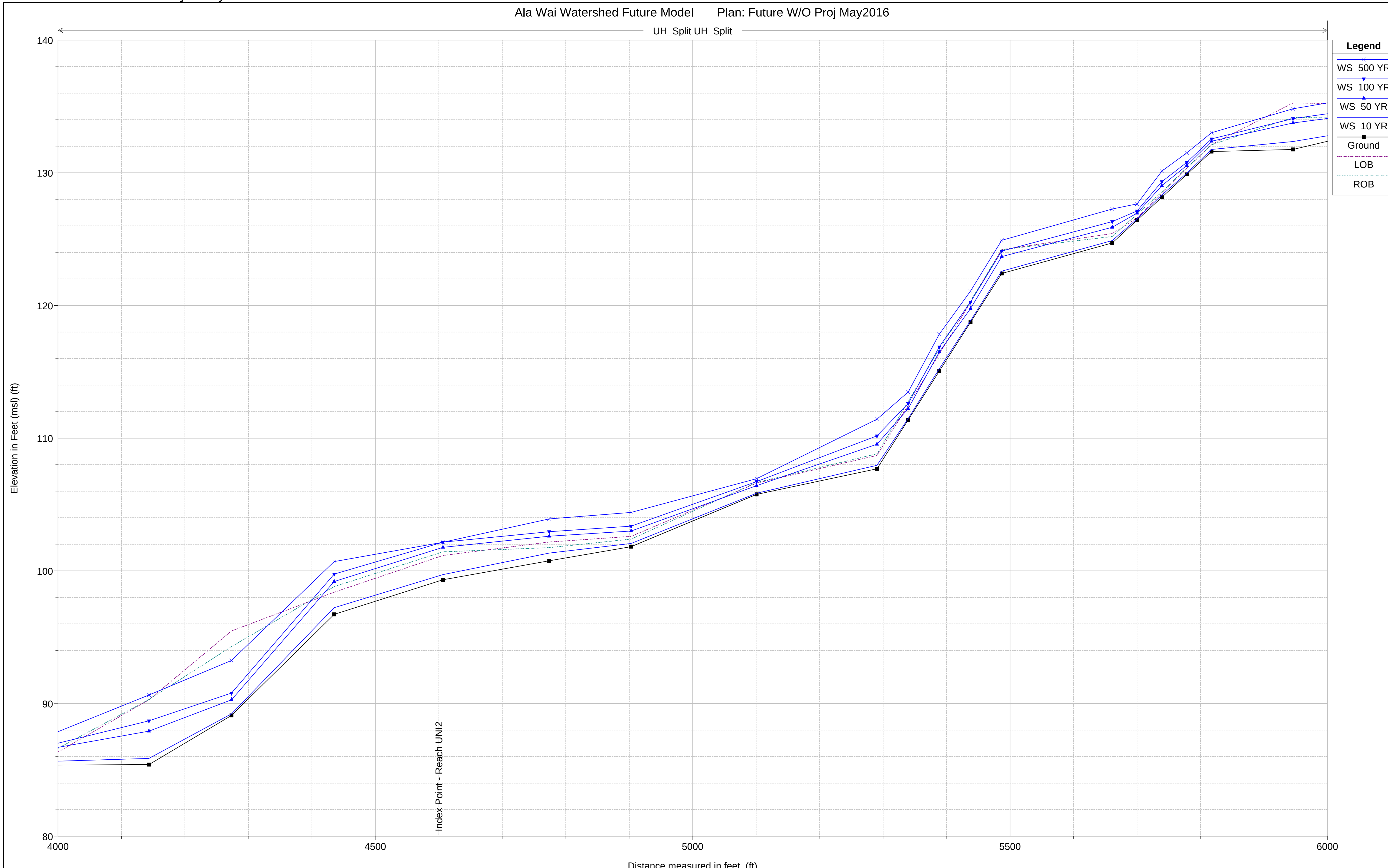

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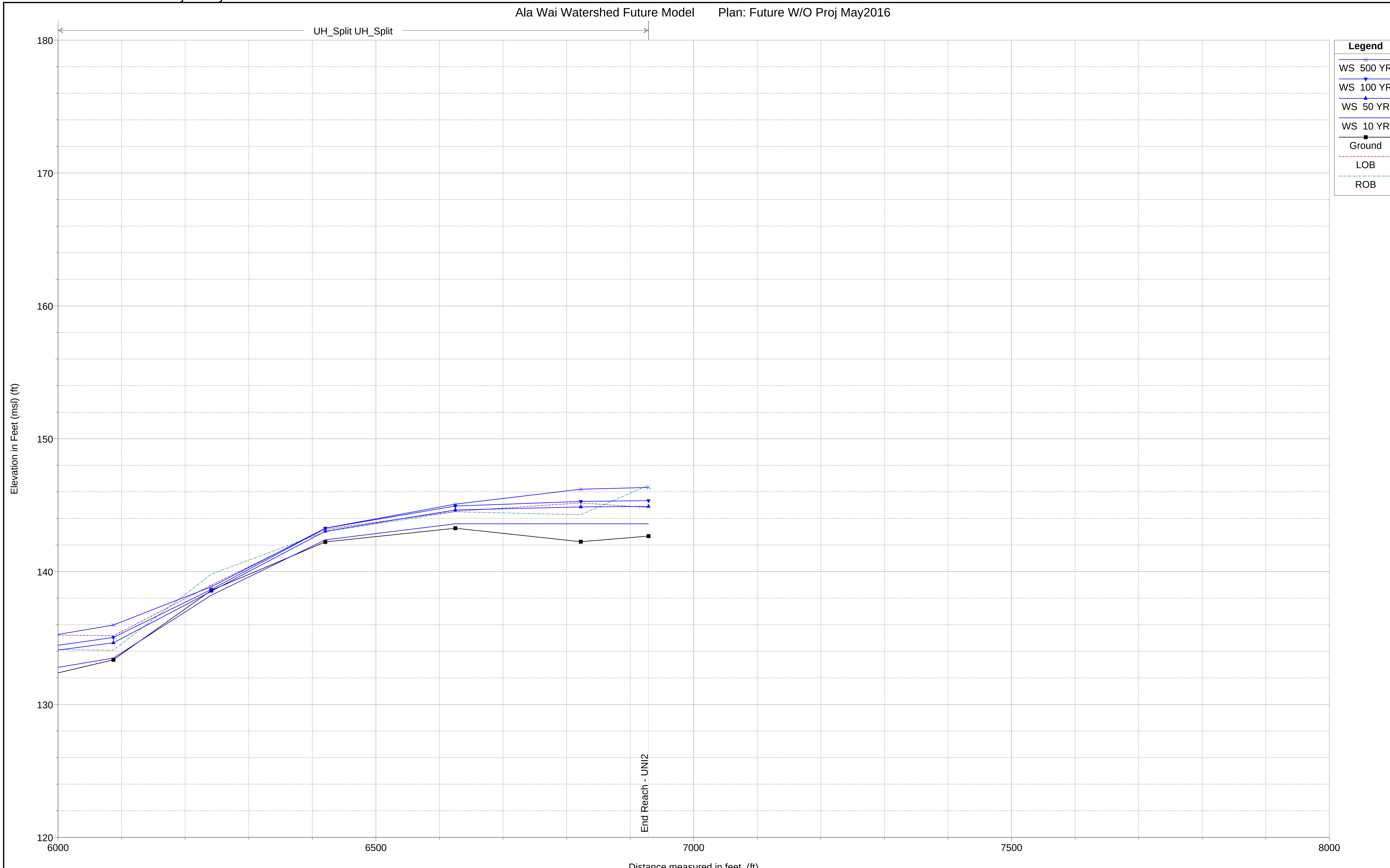

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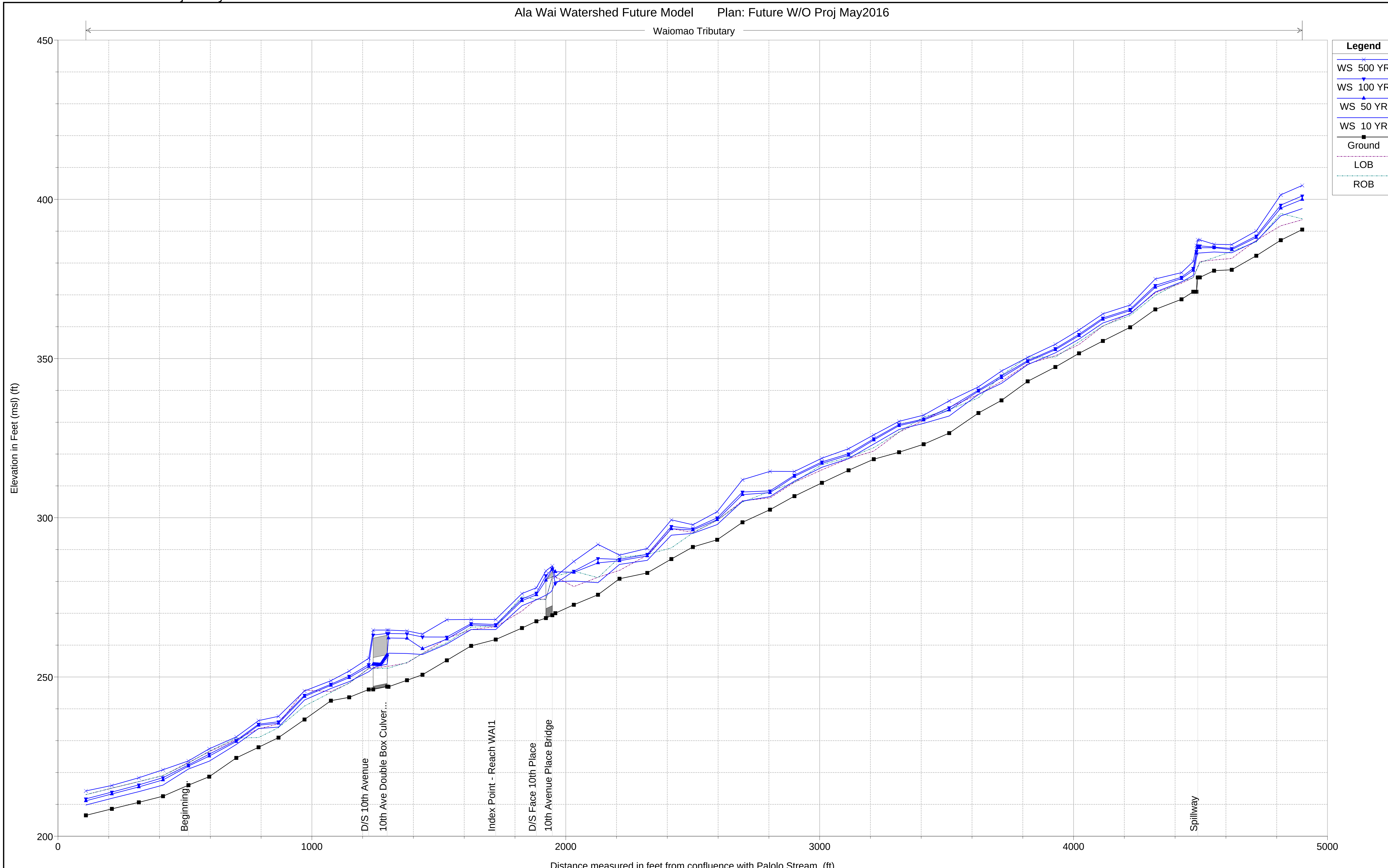

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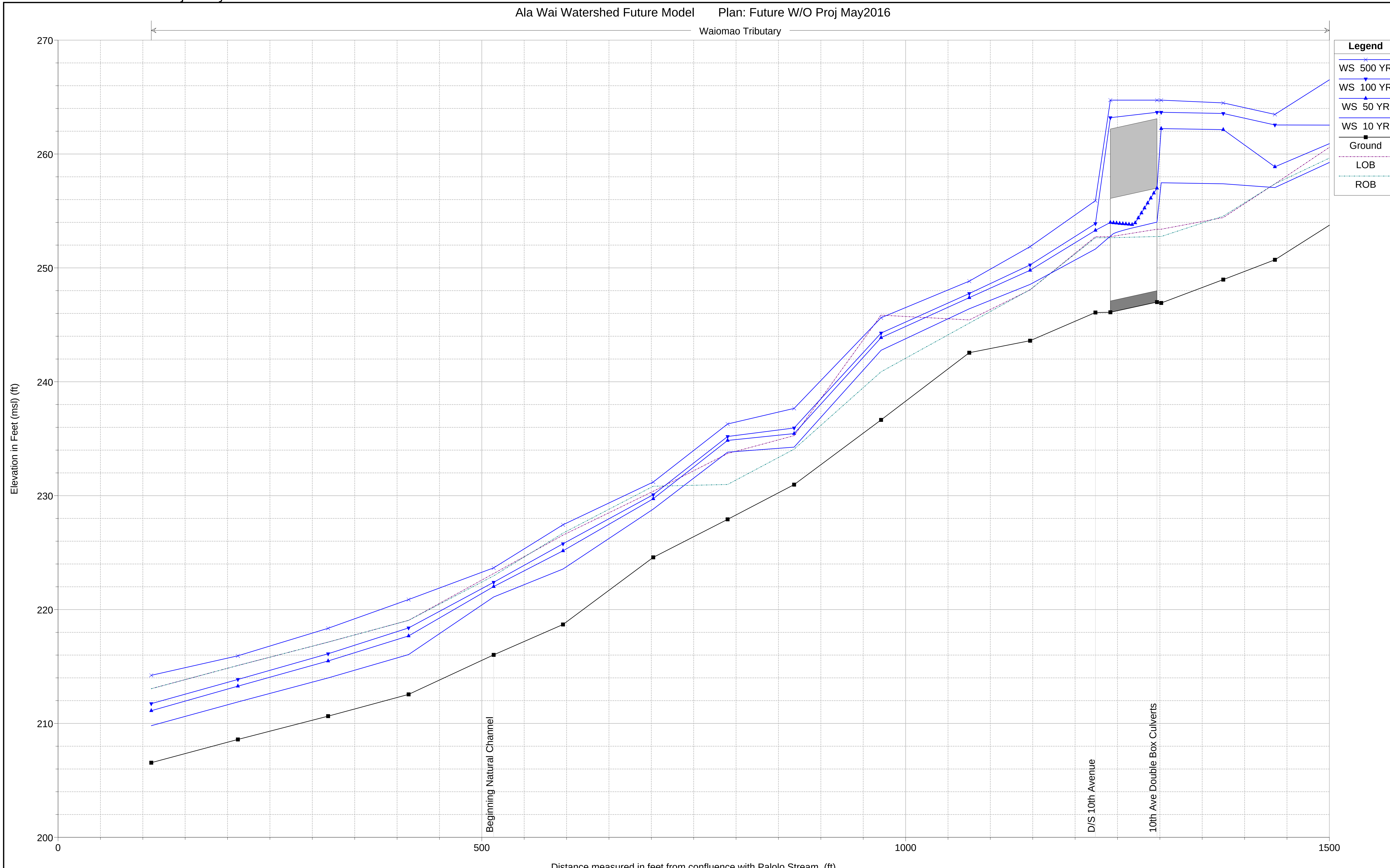

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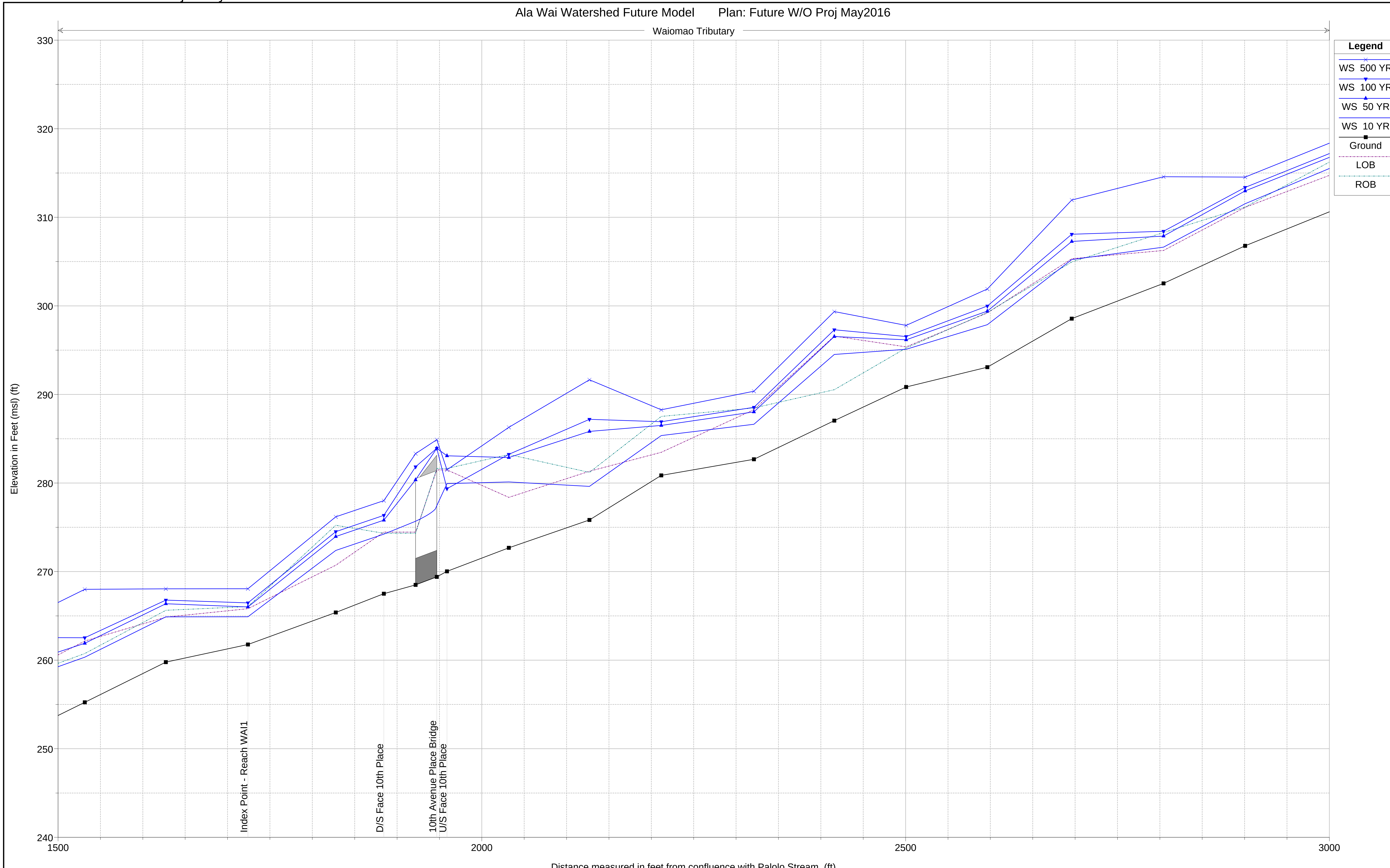

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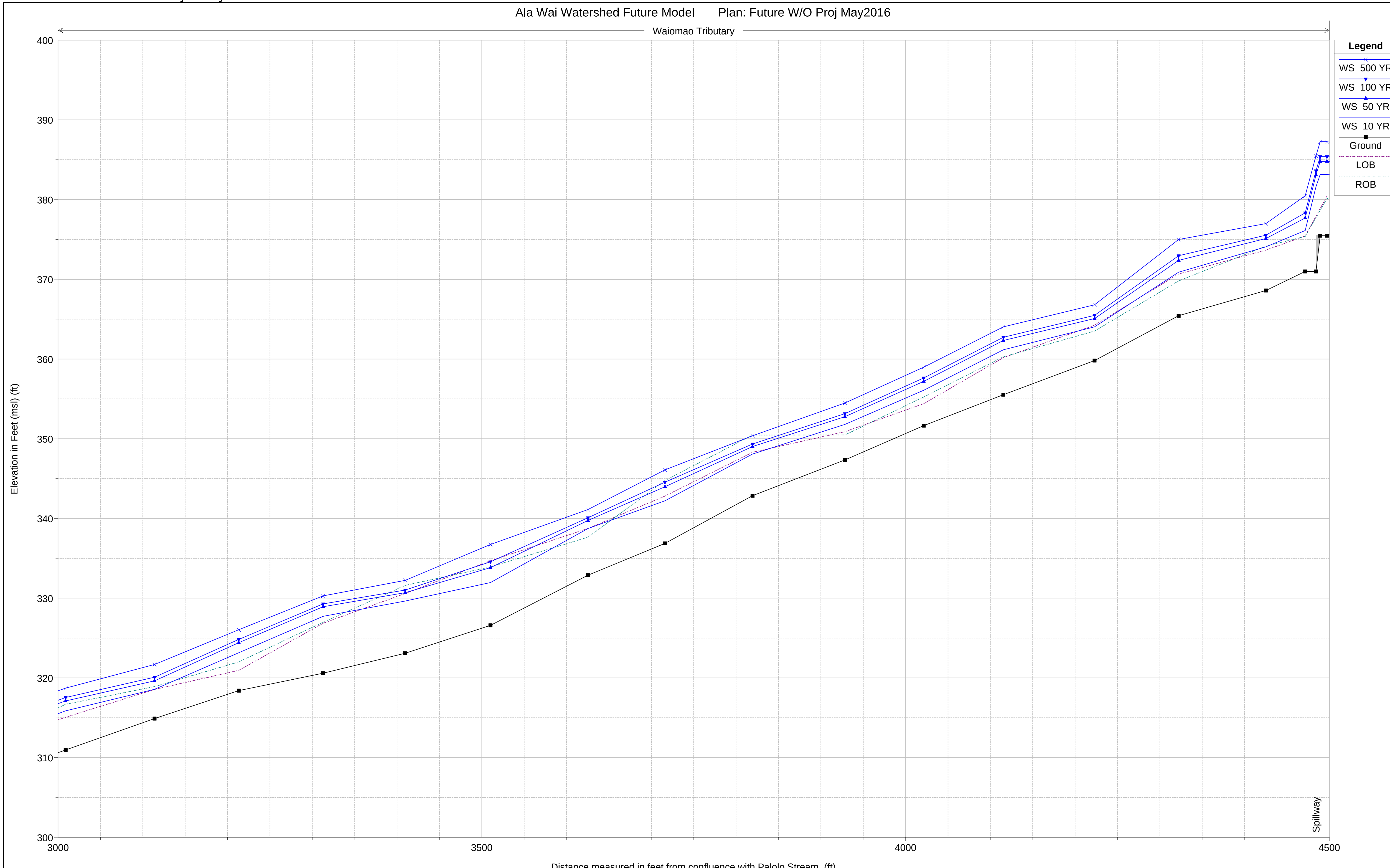

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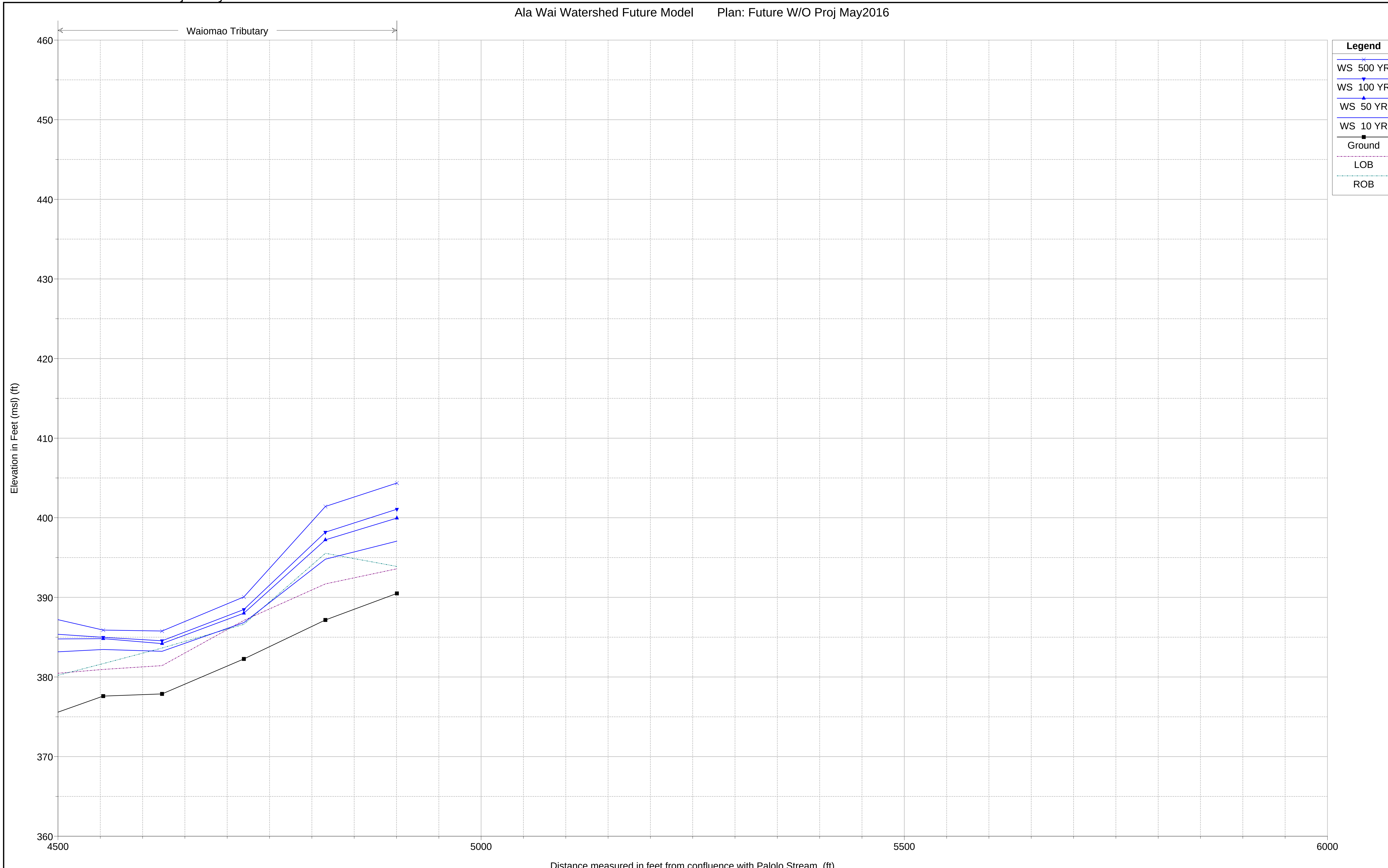

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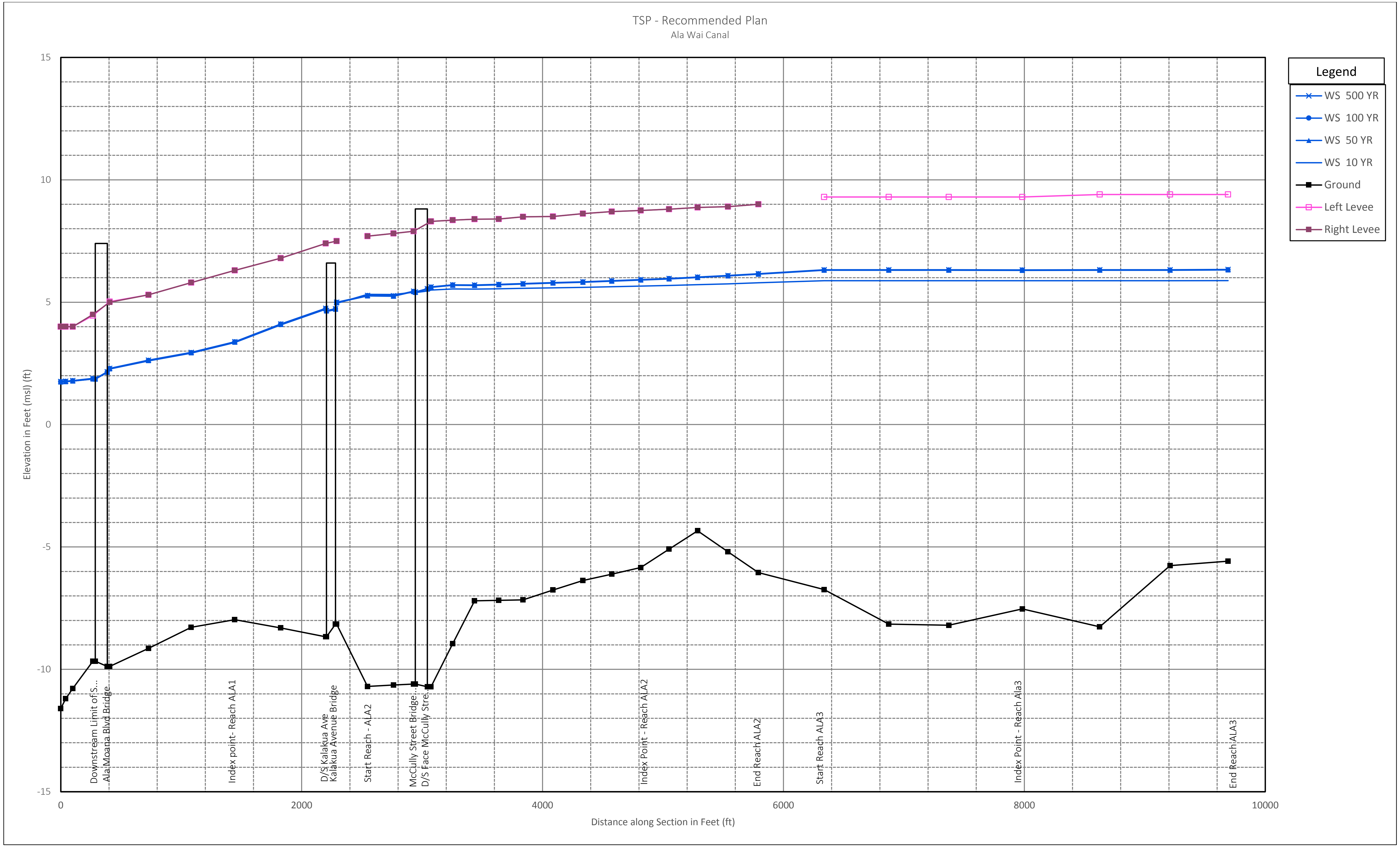

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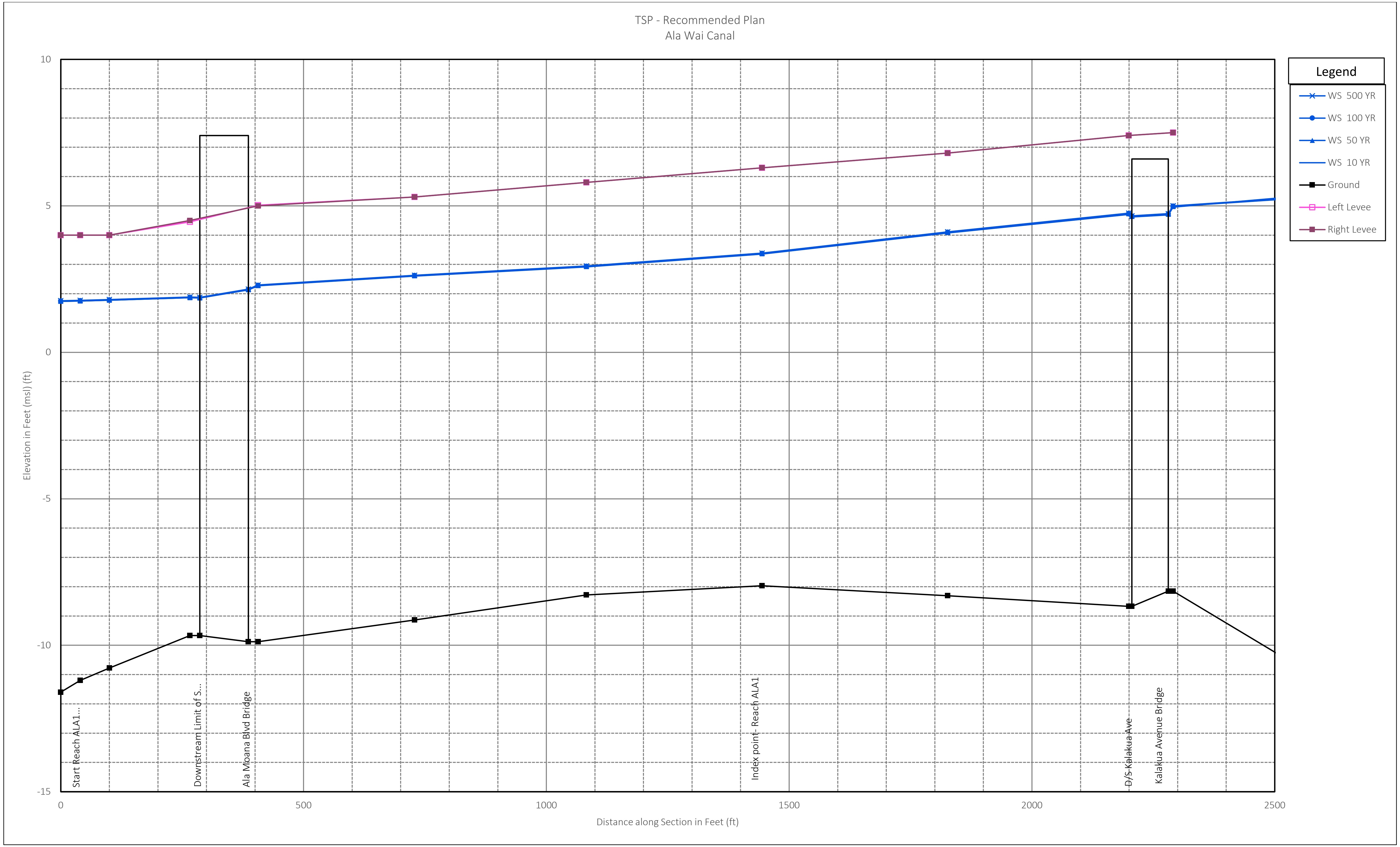

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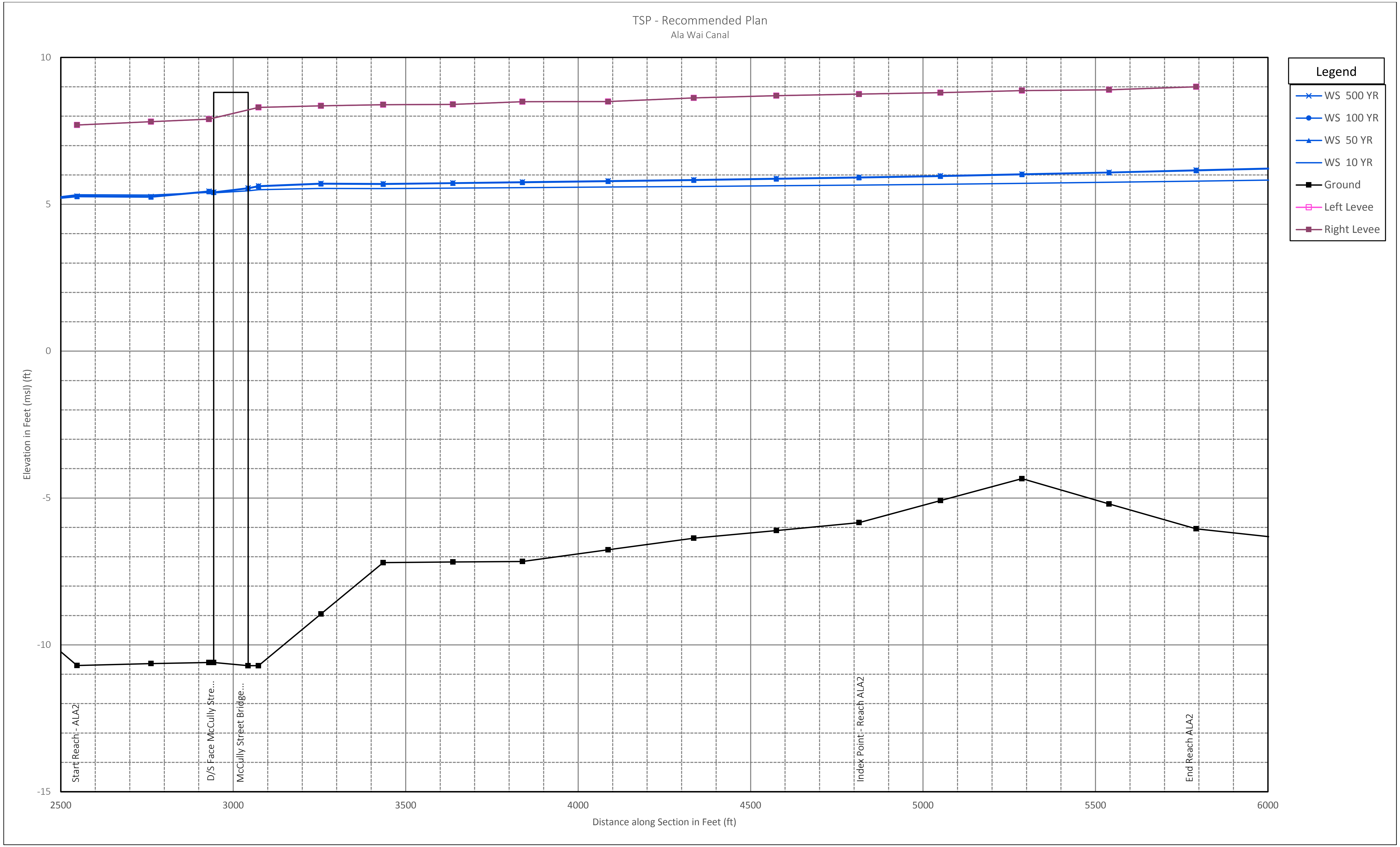

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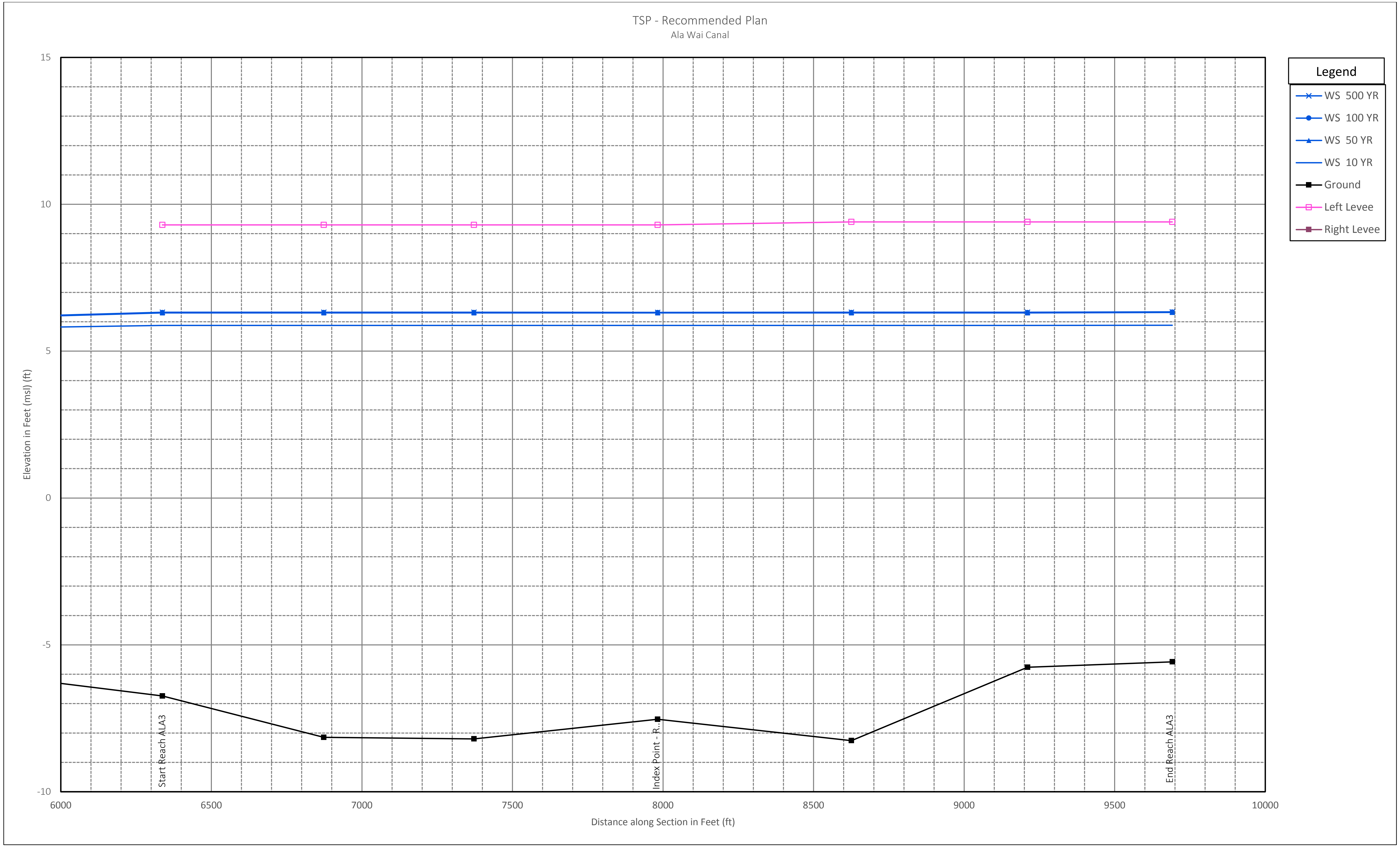

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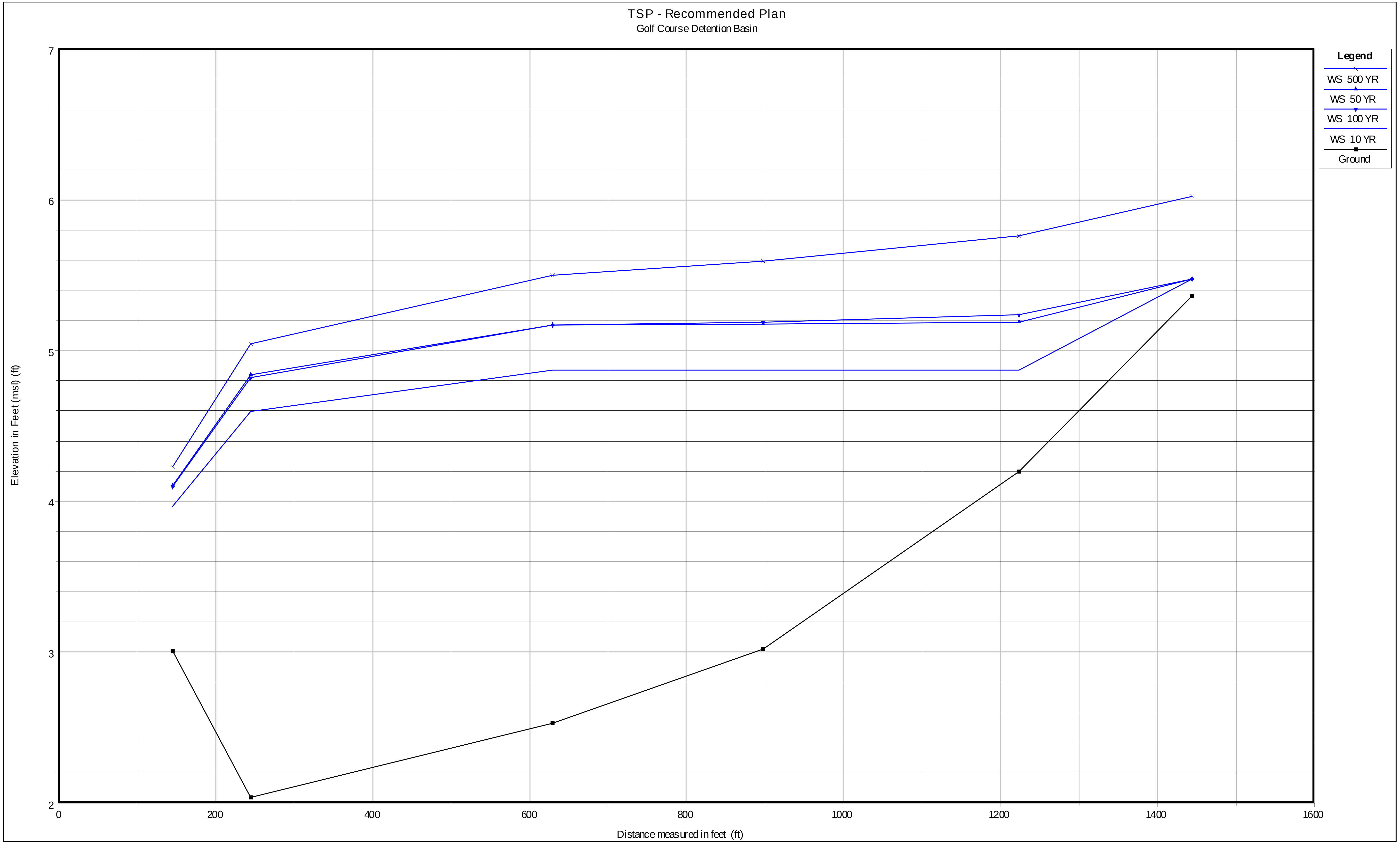

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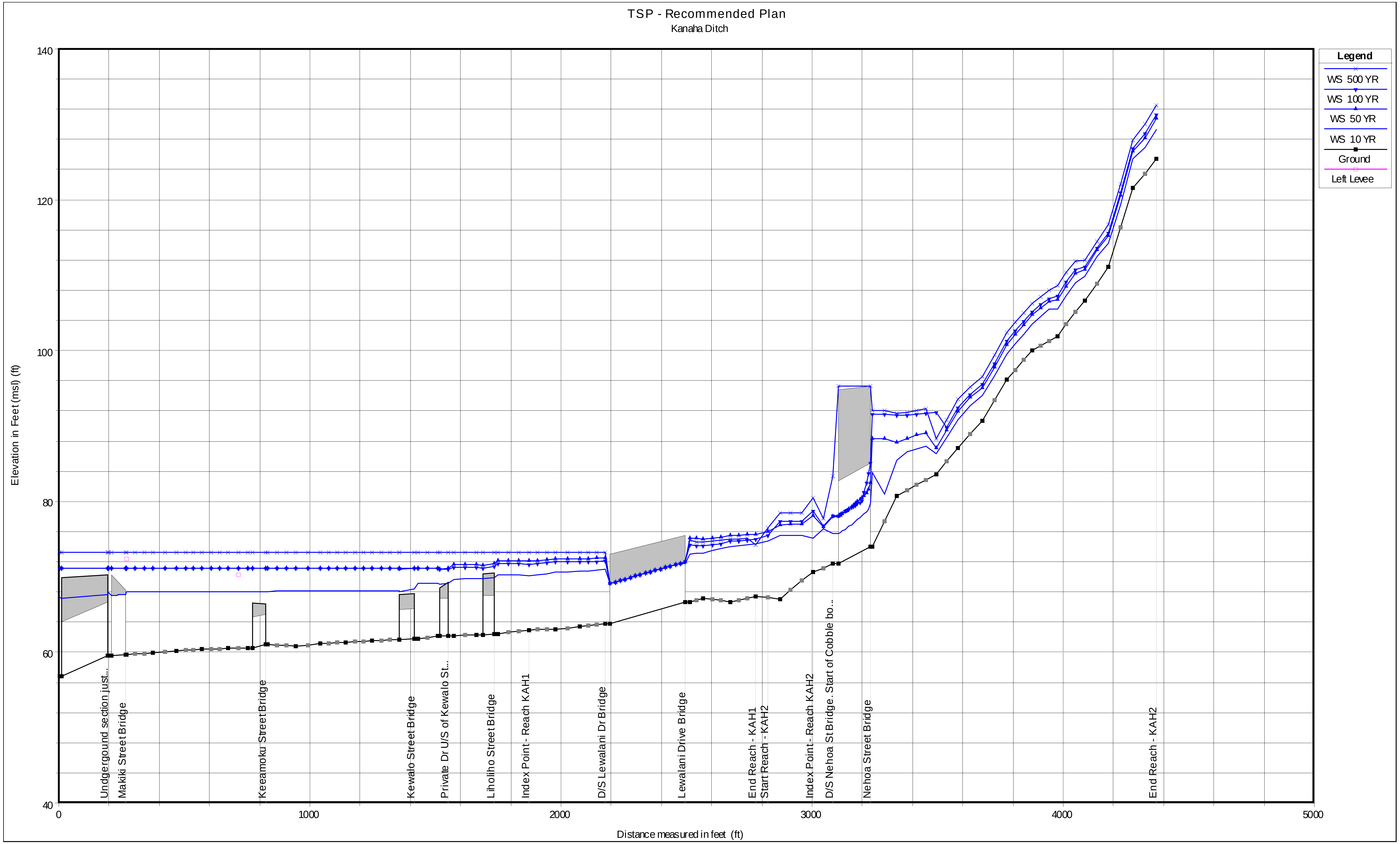

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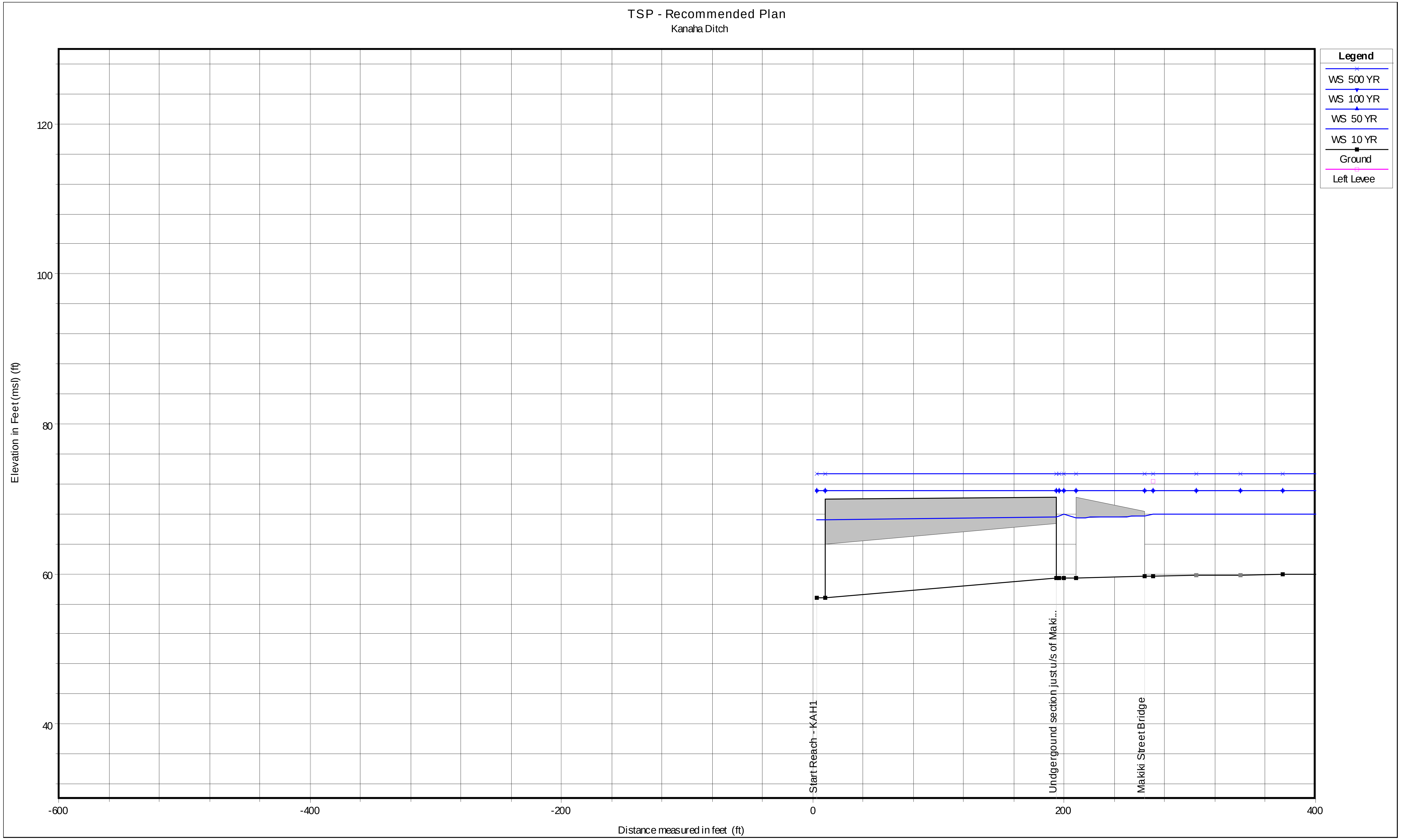

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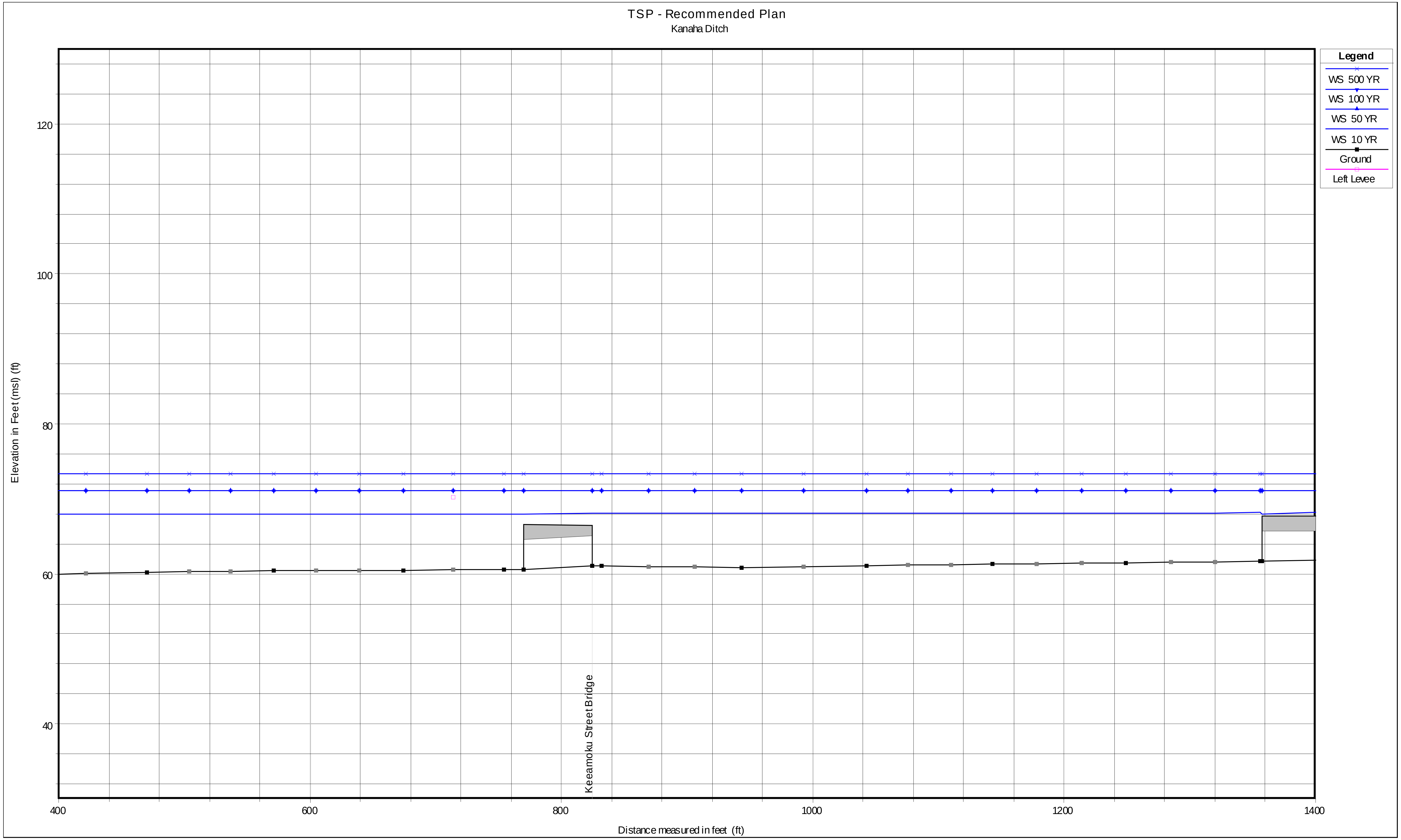

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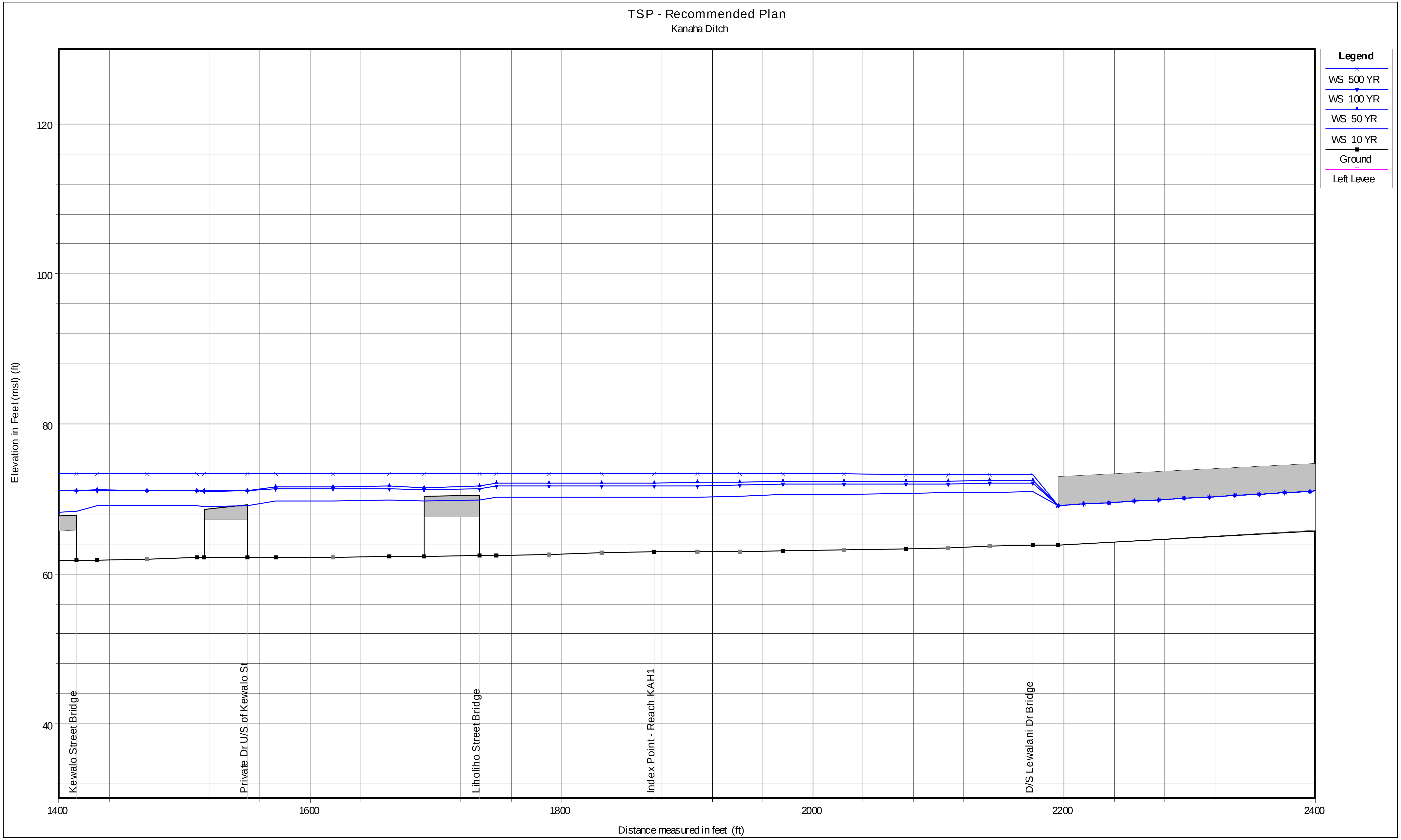

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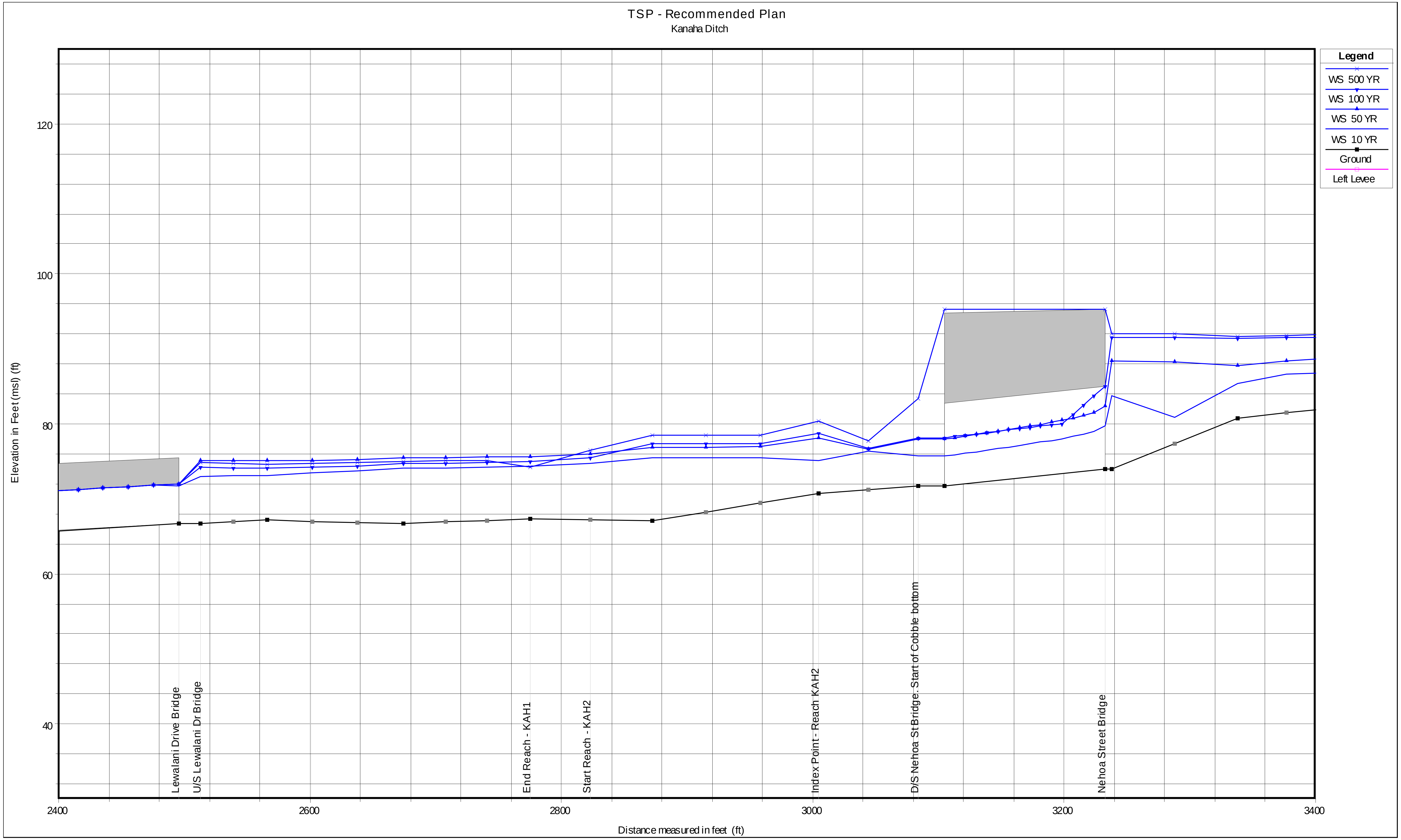

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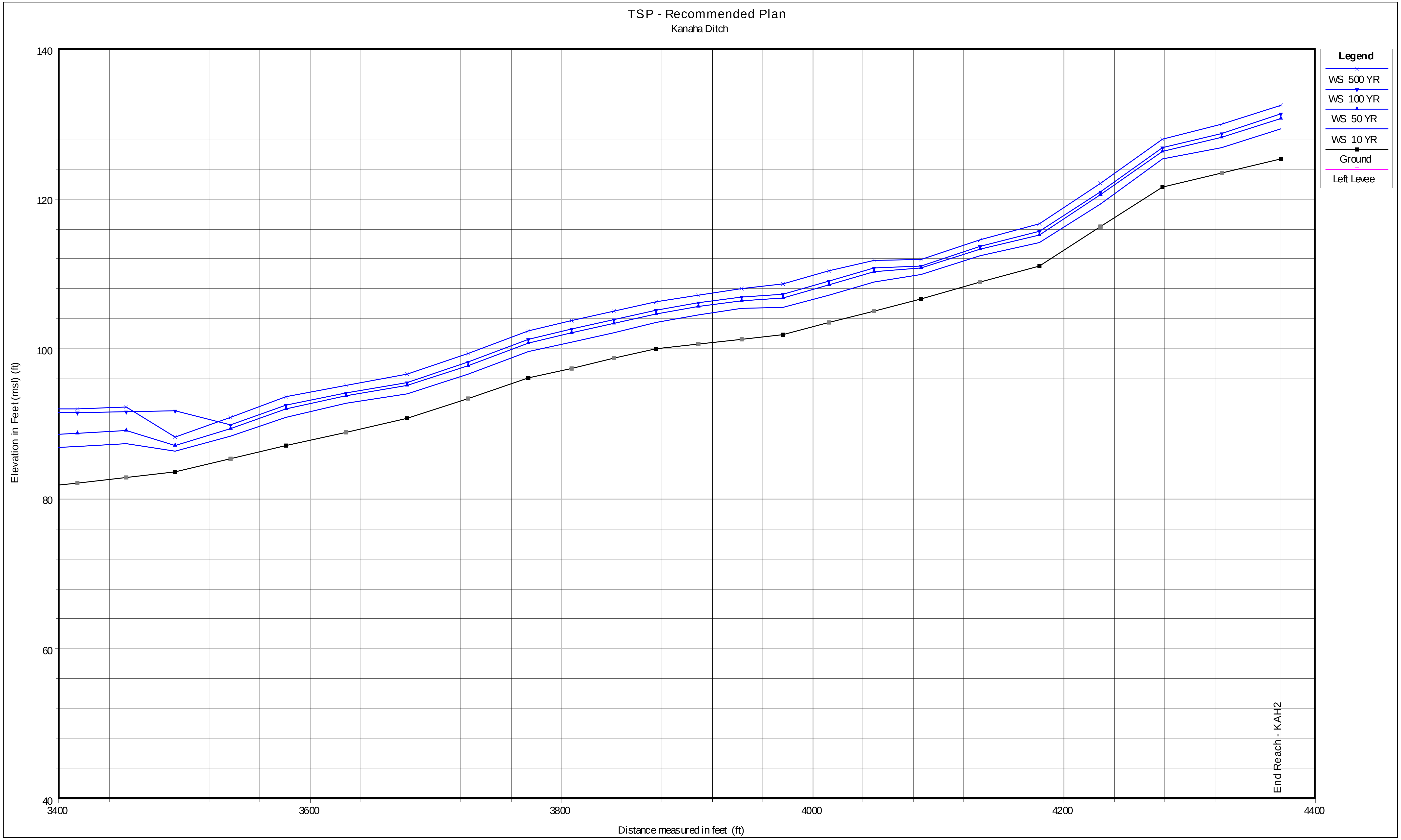

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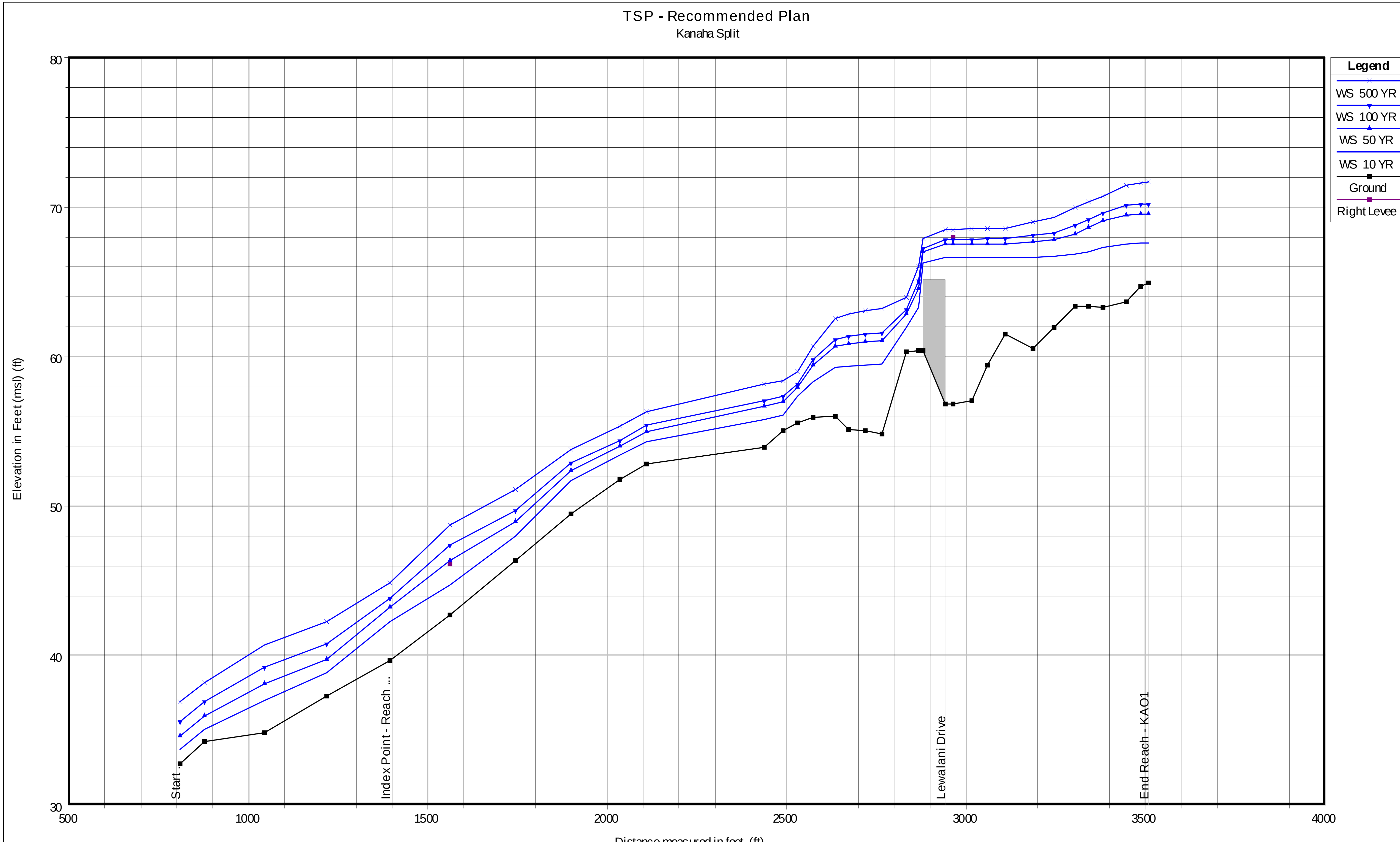

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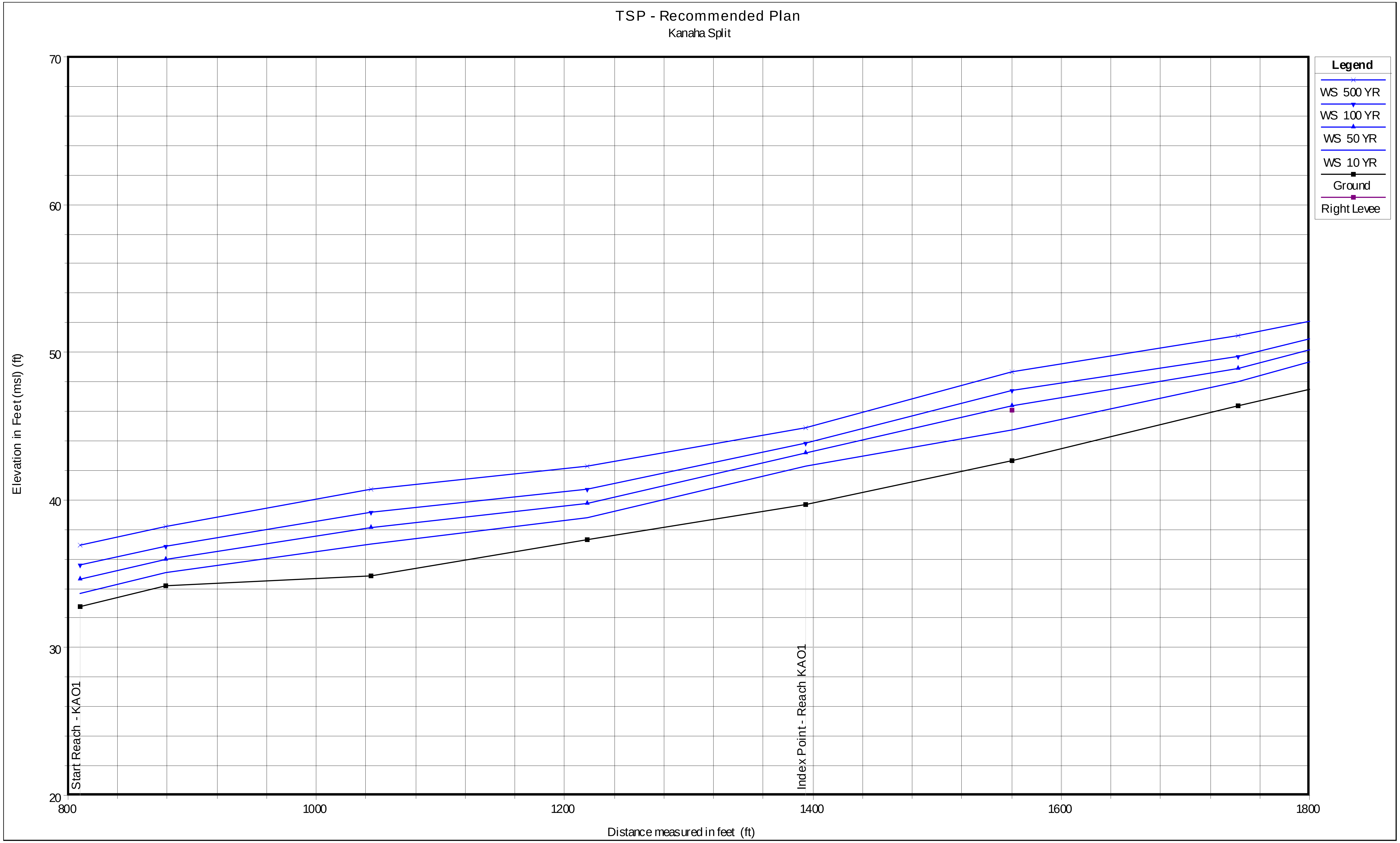

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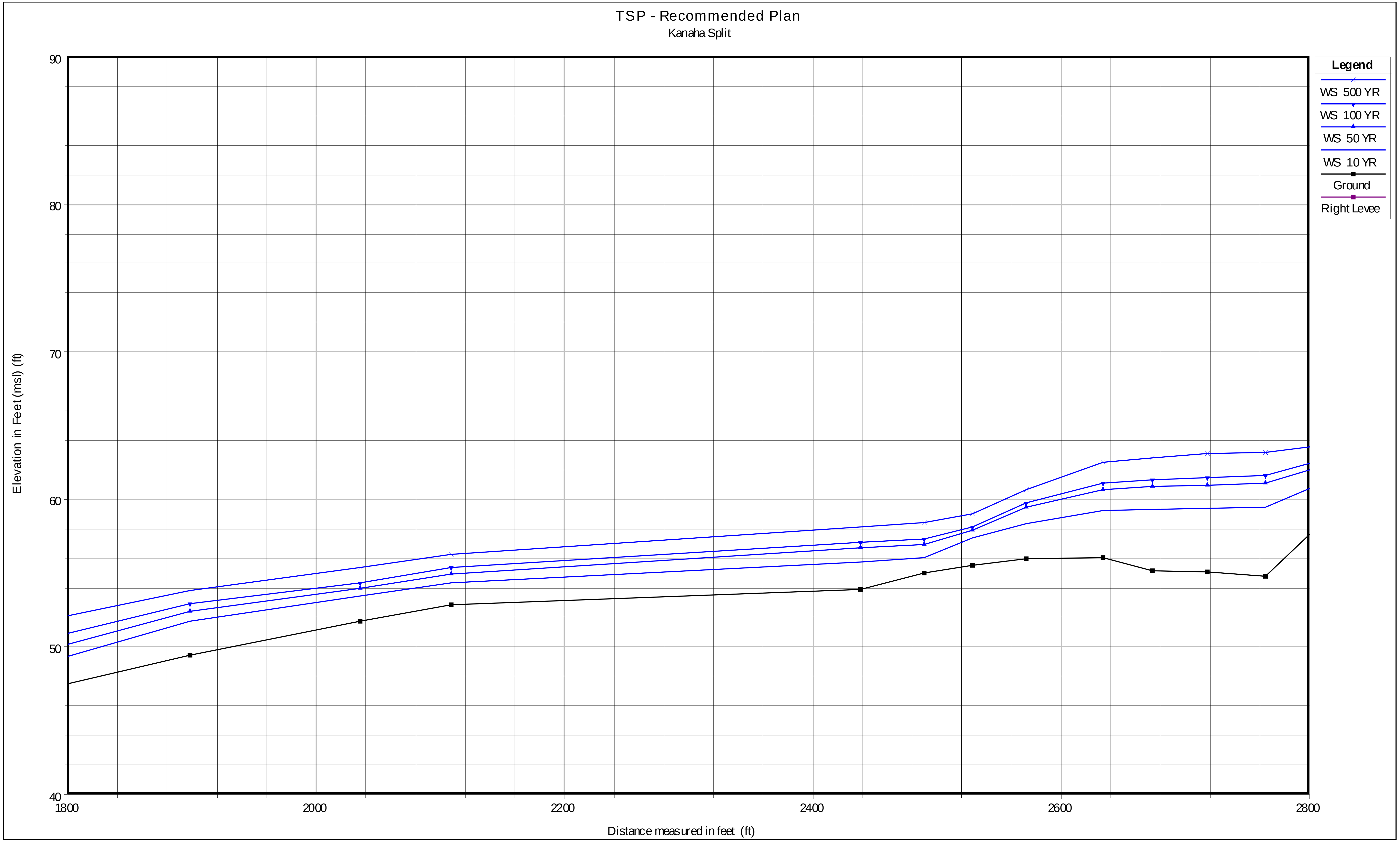

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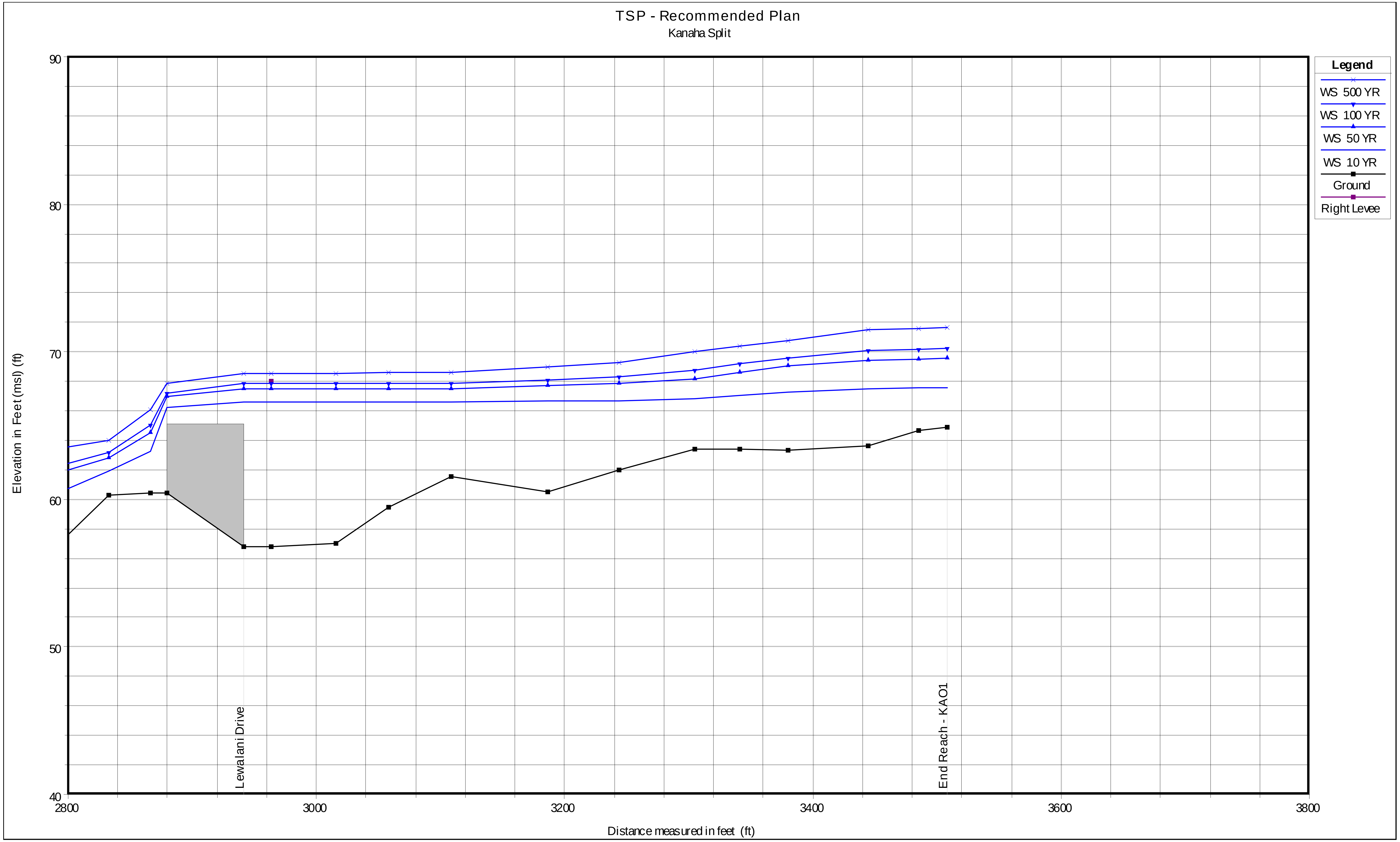
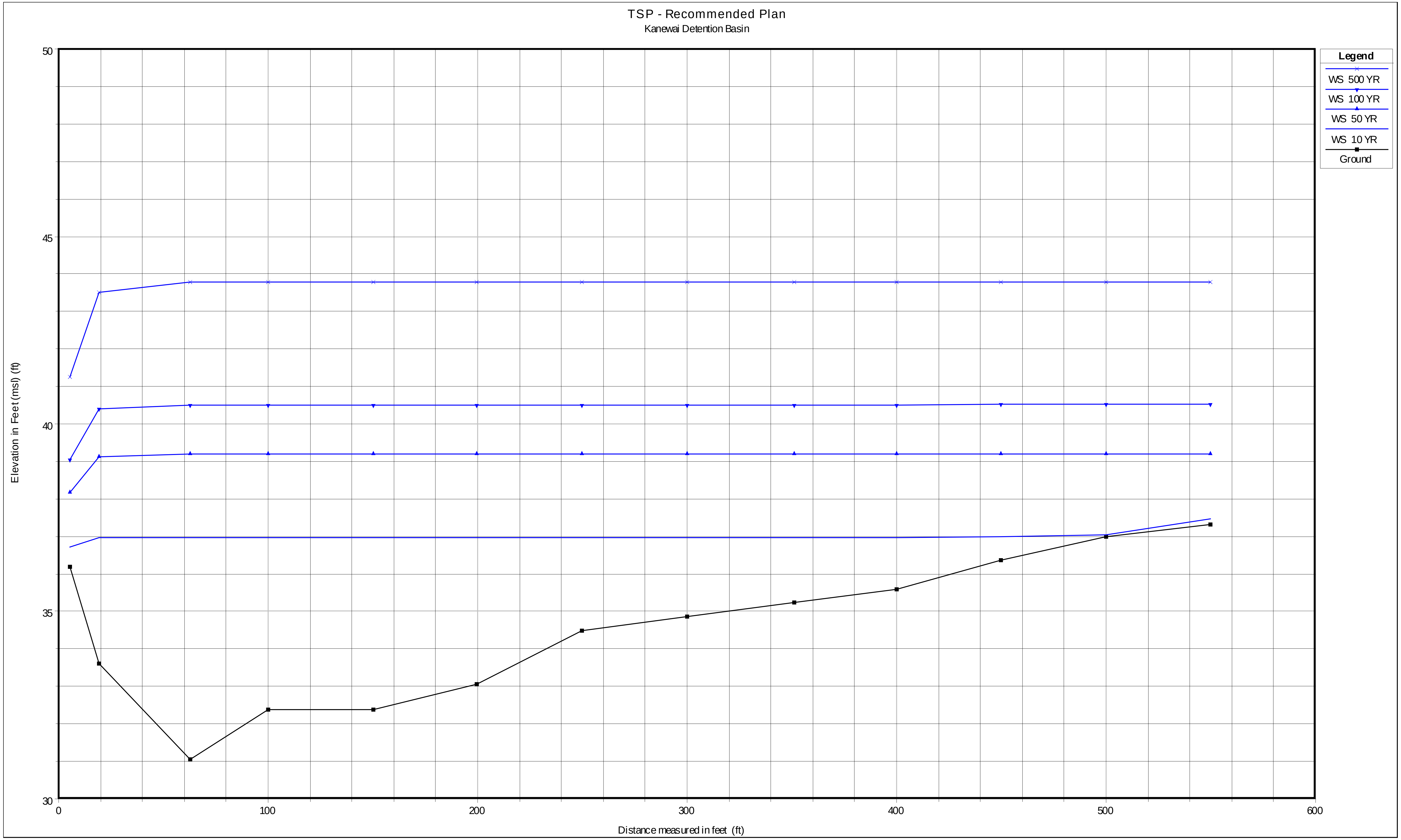


Plate 4 With-Project TSP Hydraulic Profiles



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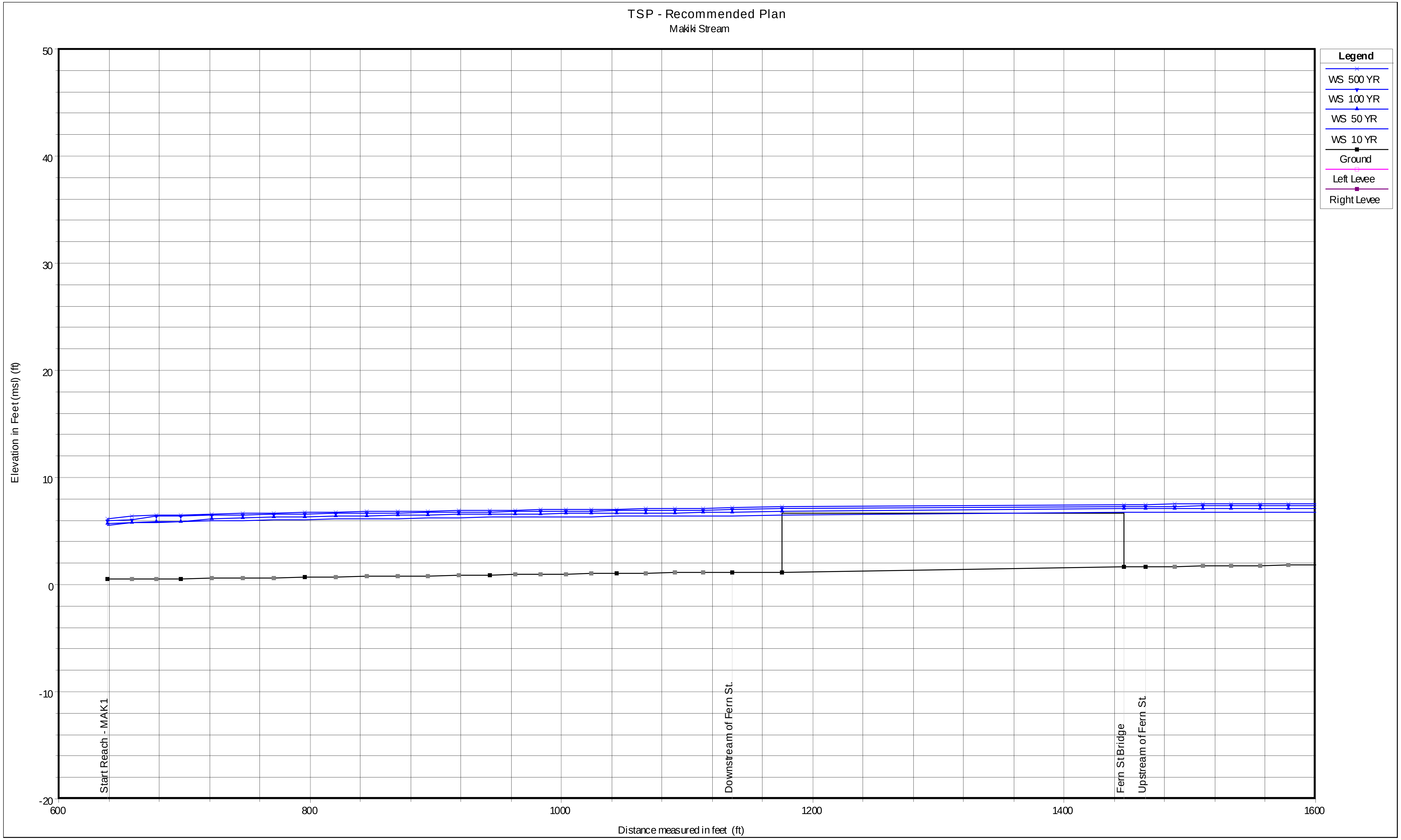

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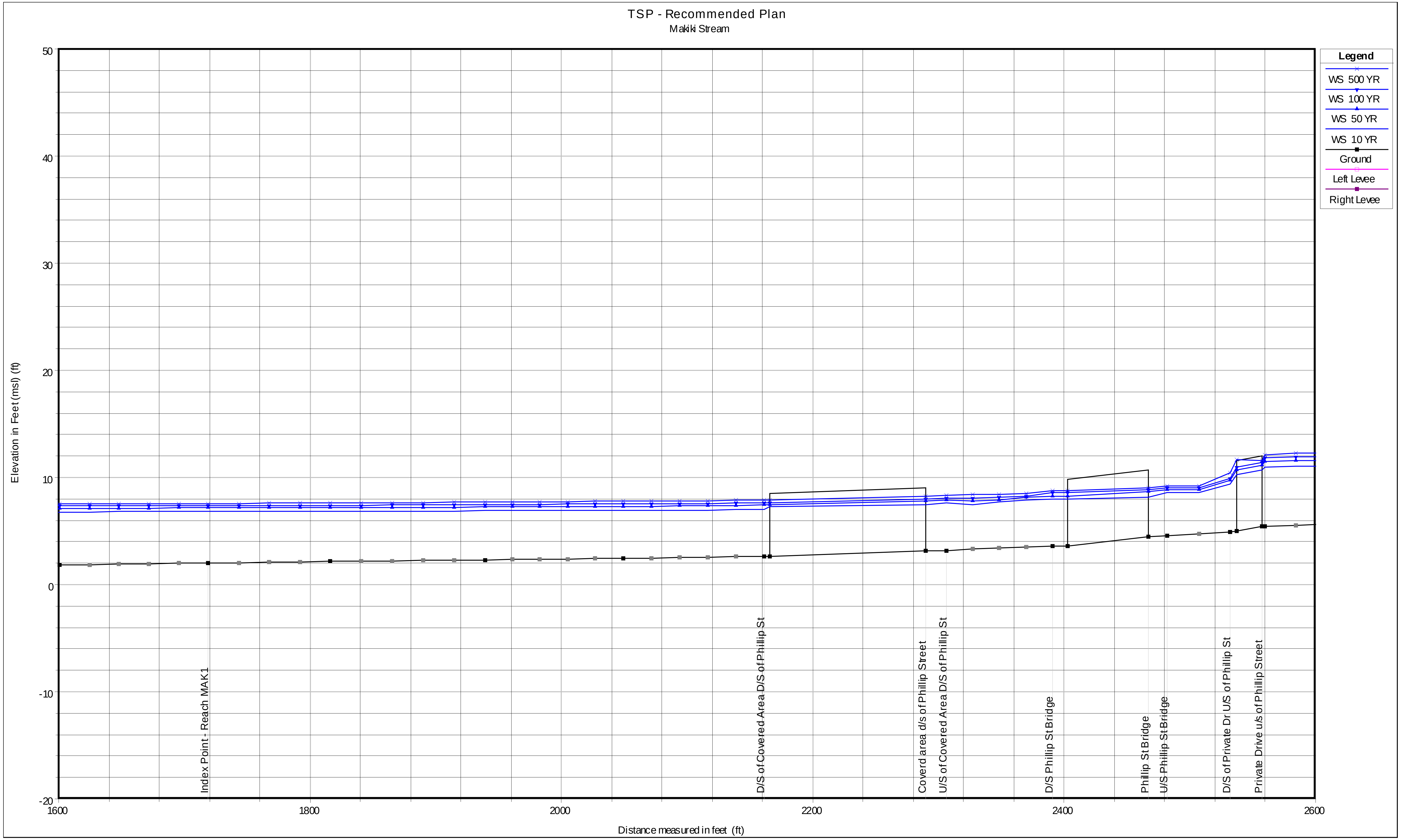

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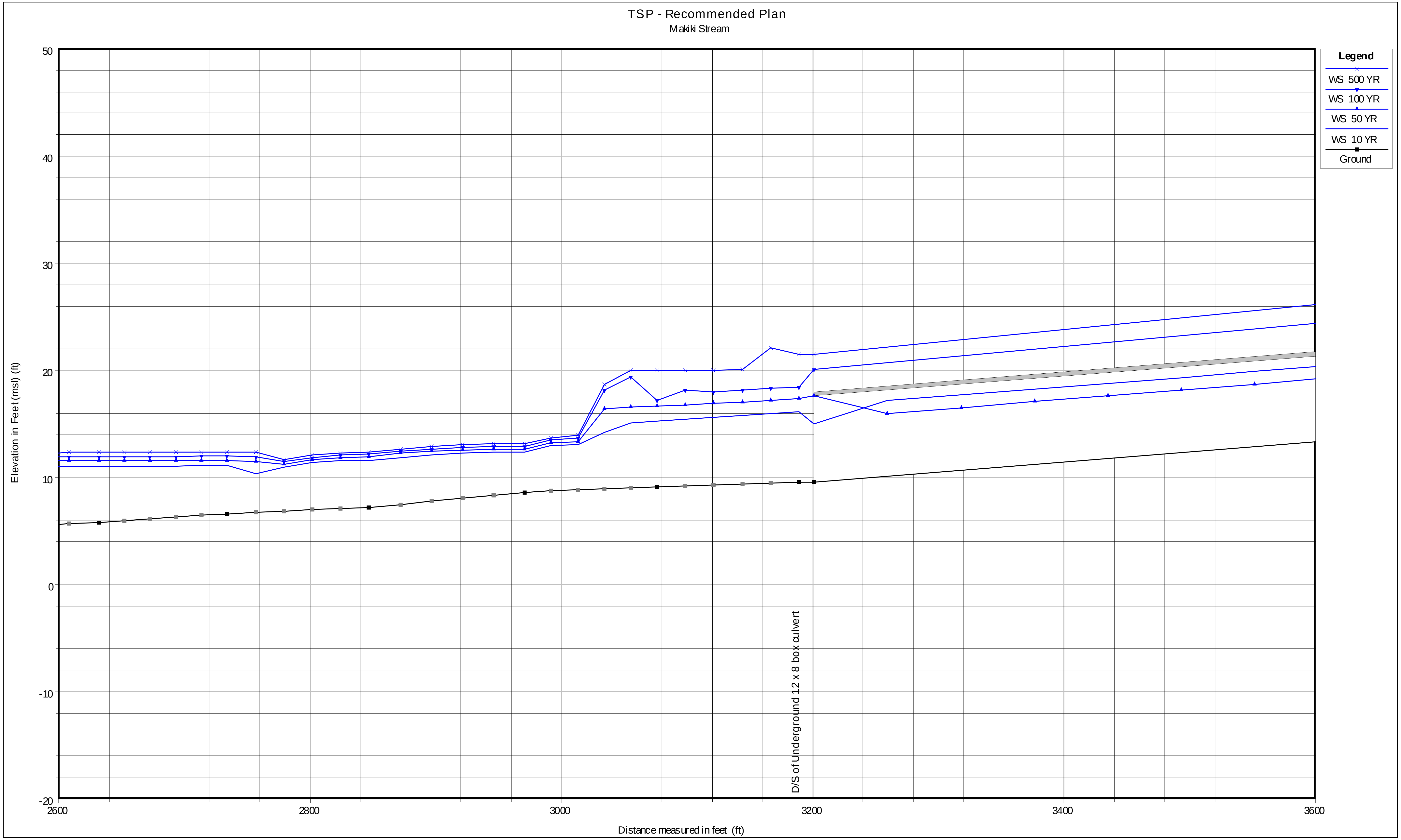

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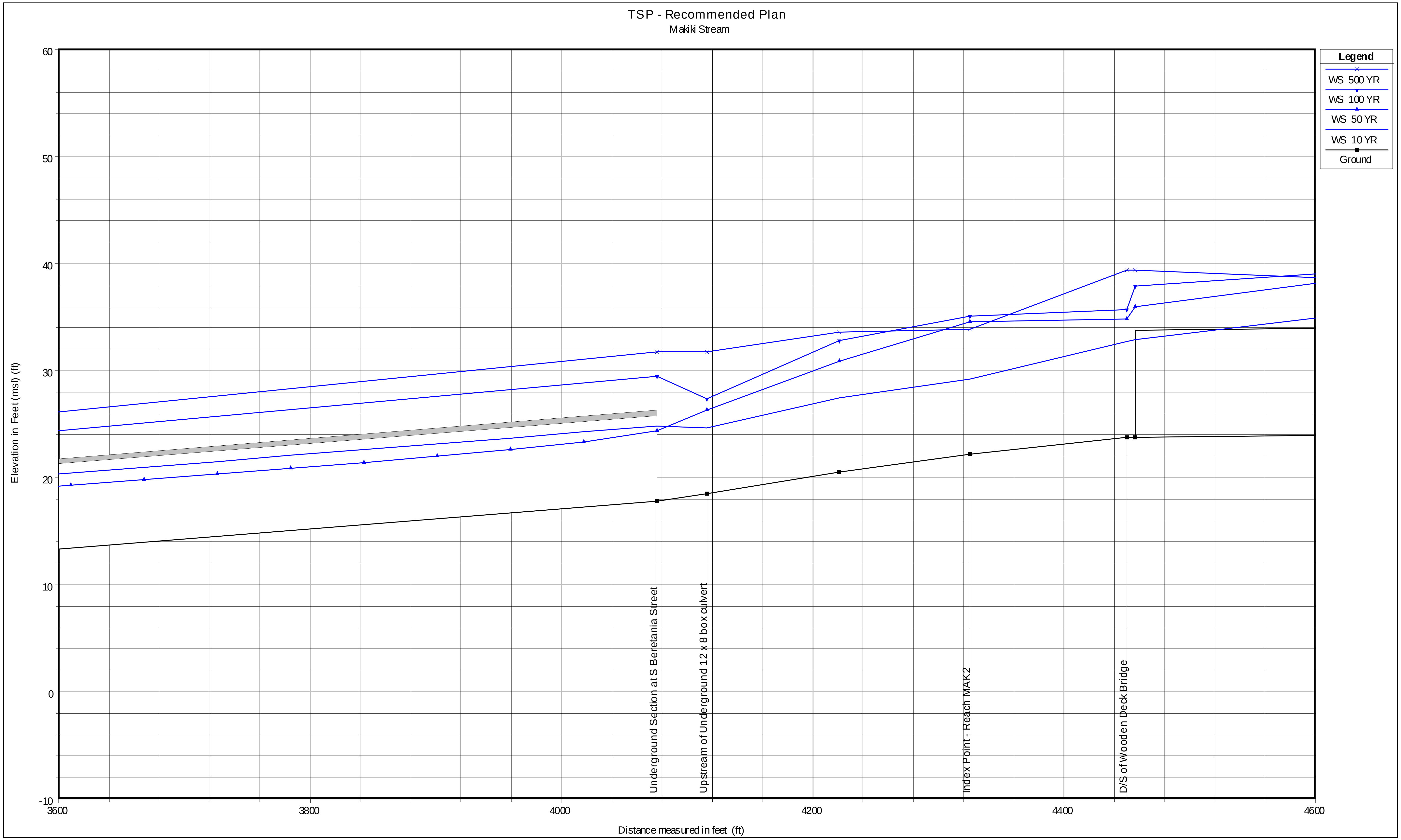

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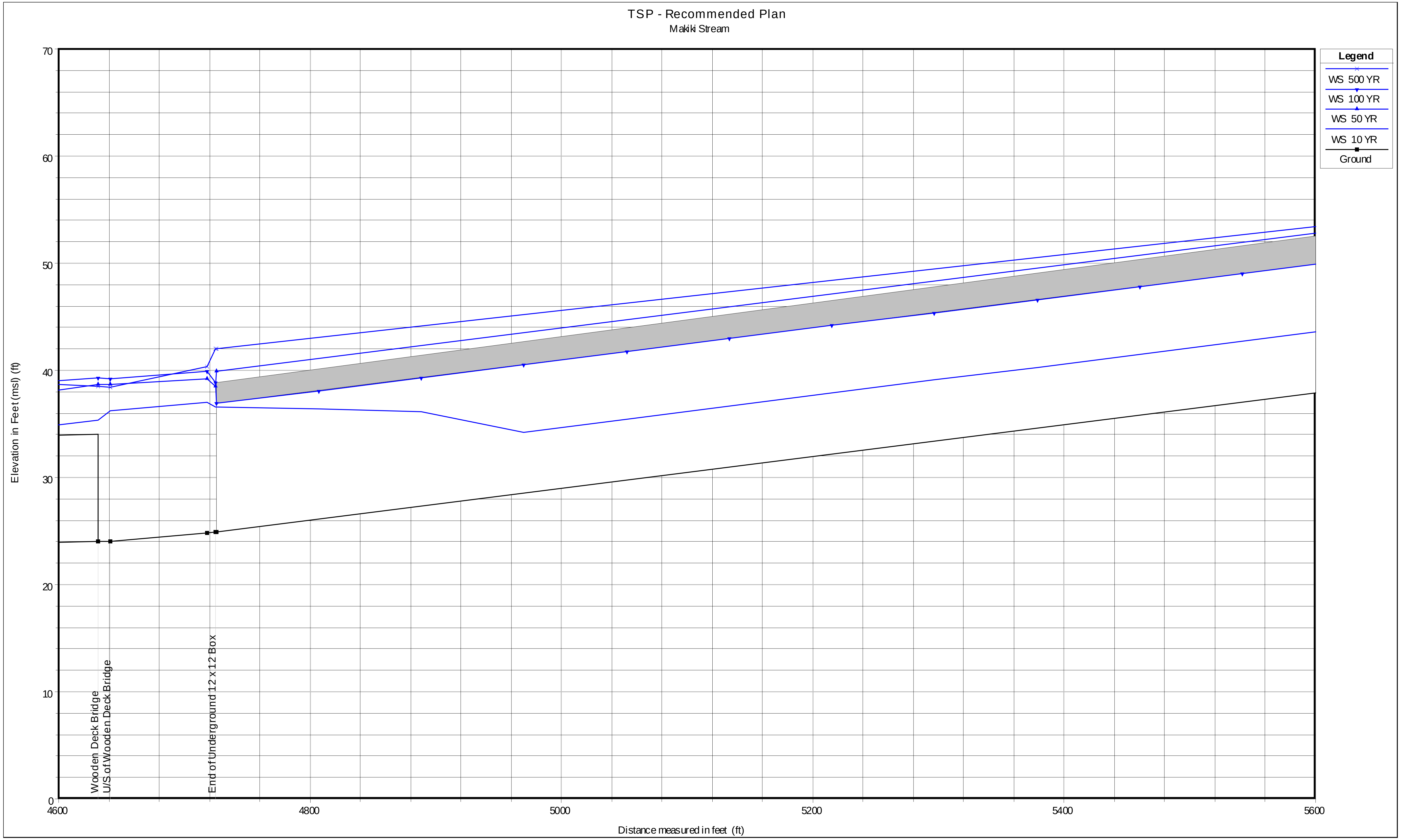

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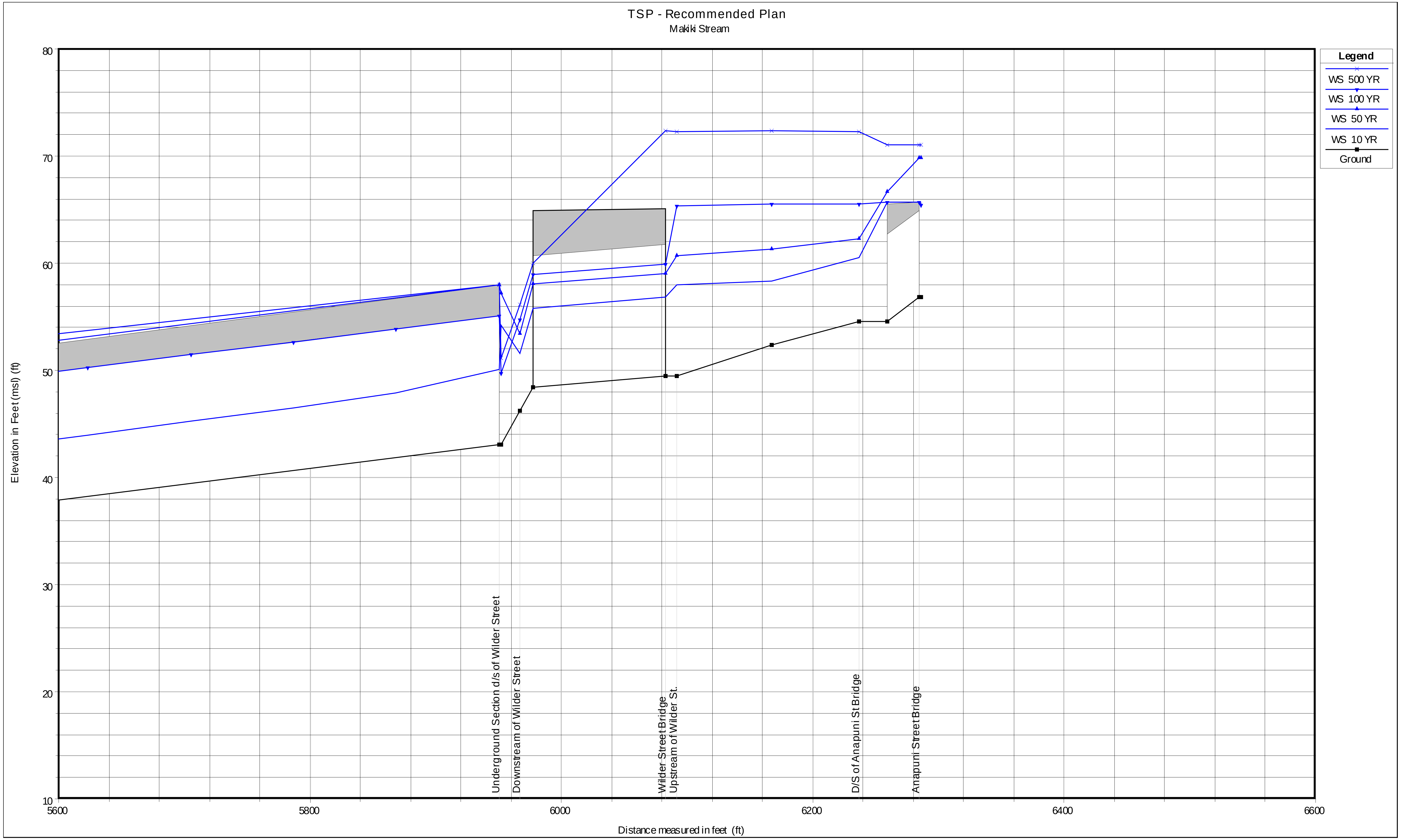

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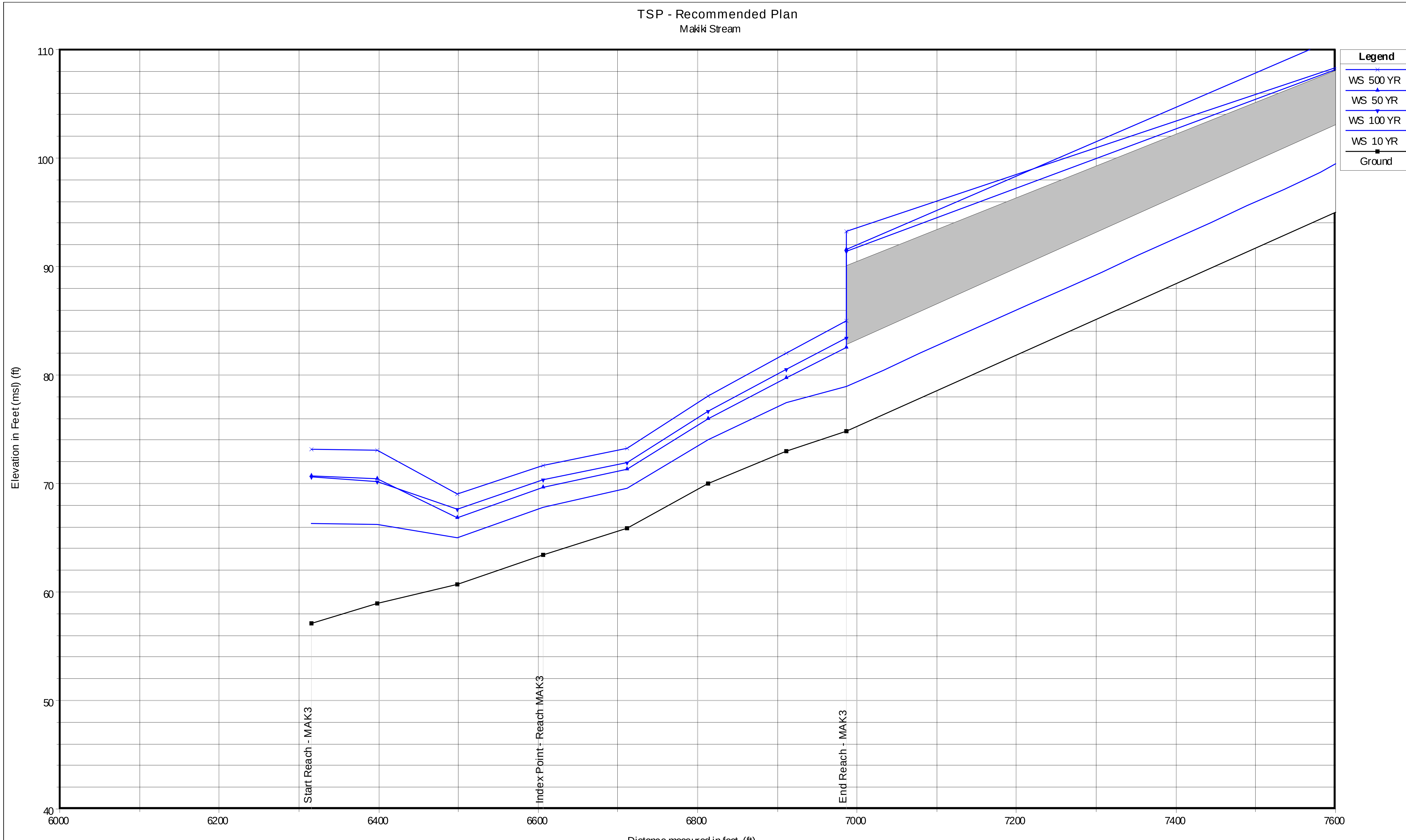

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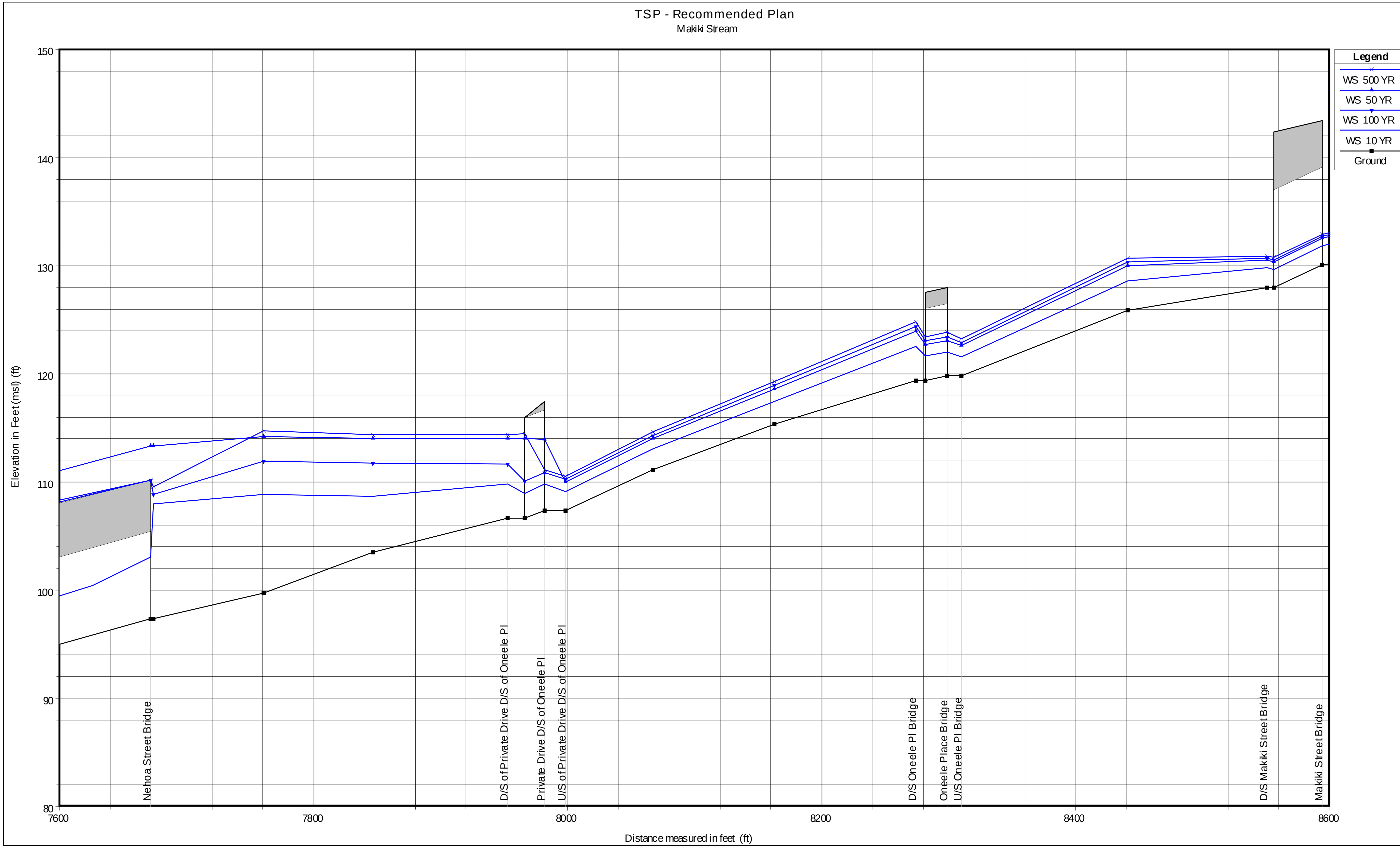

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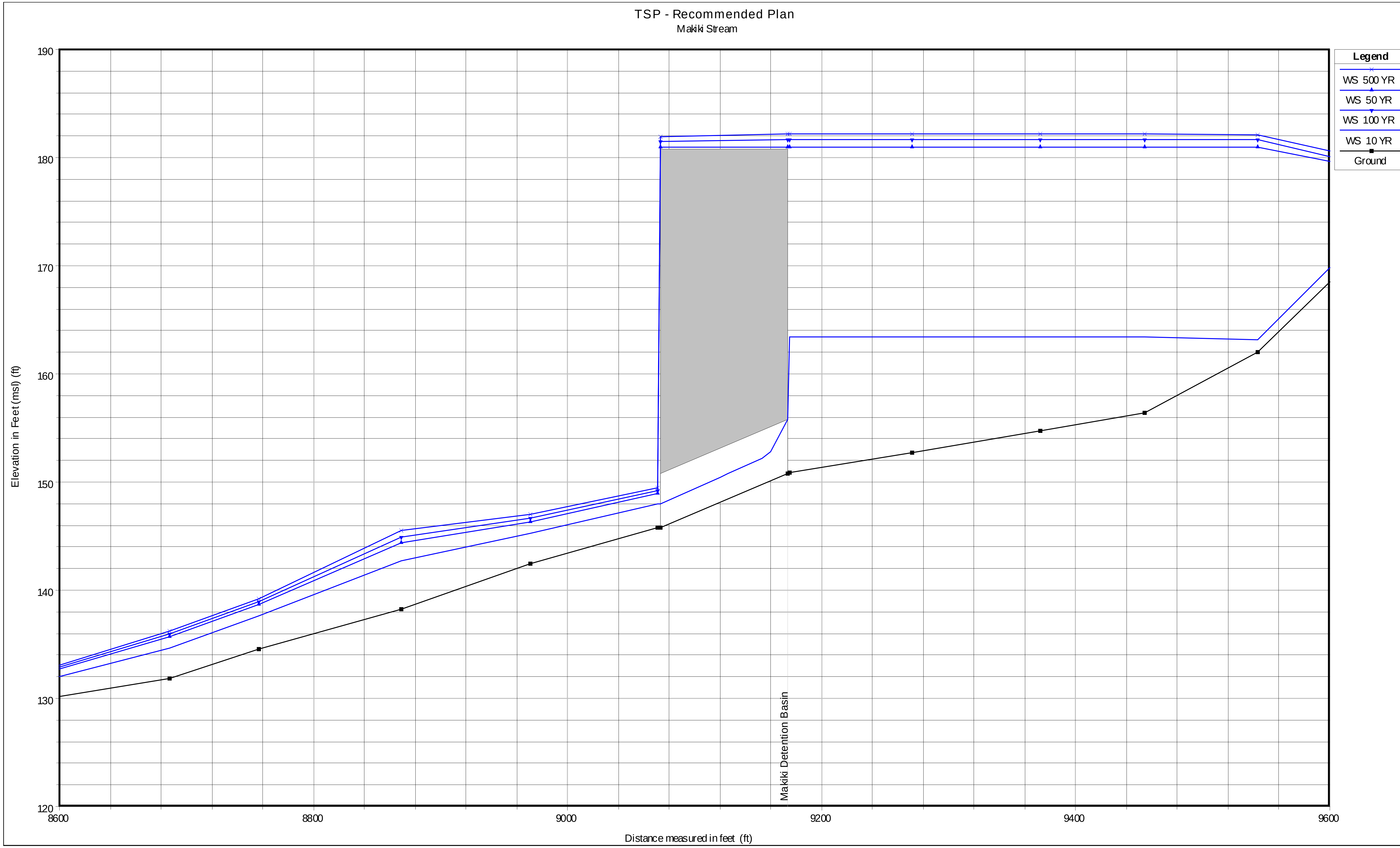

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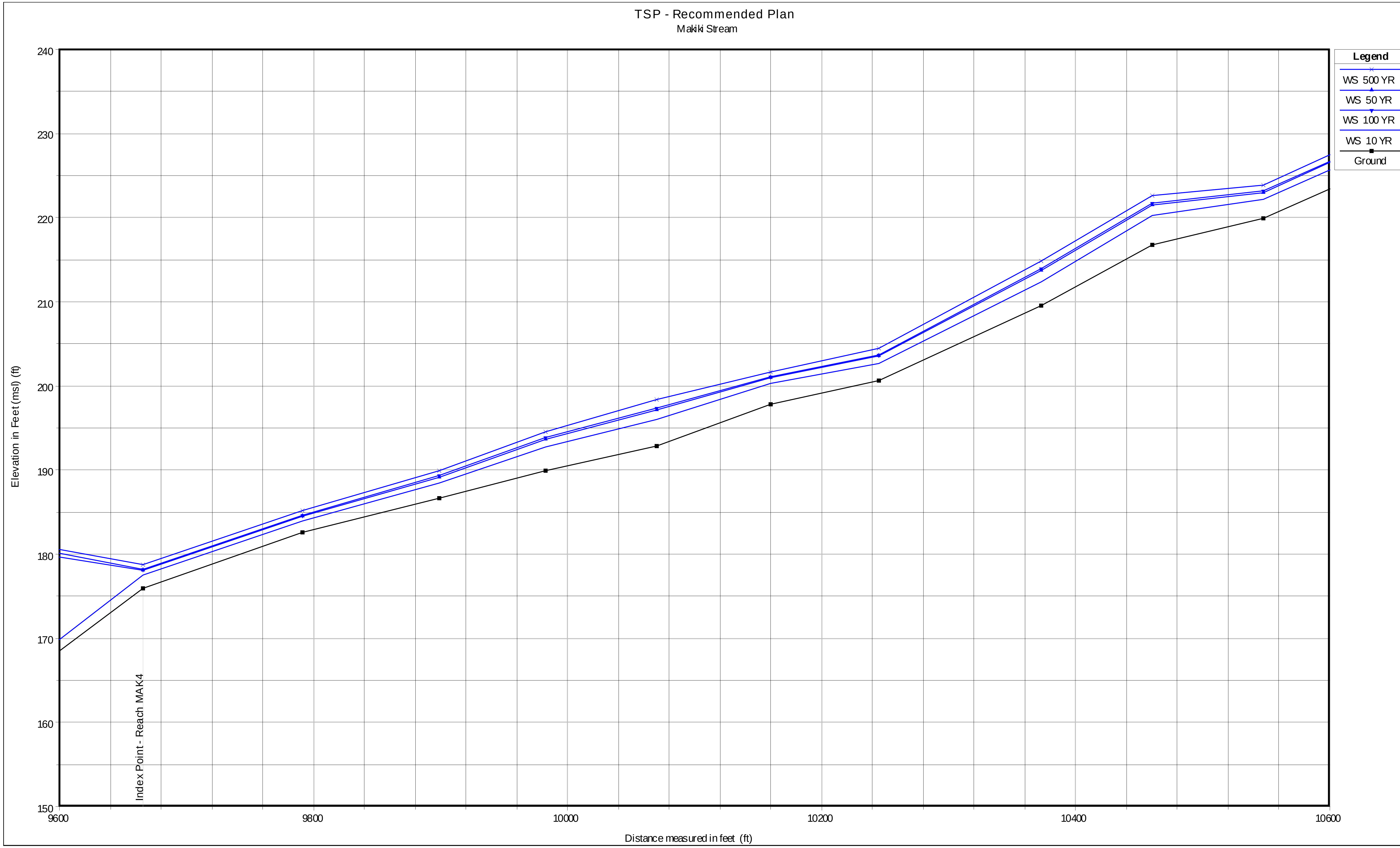

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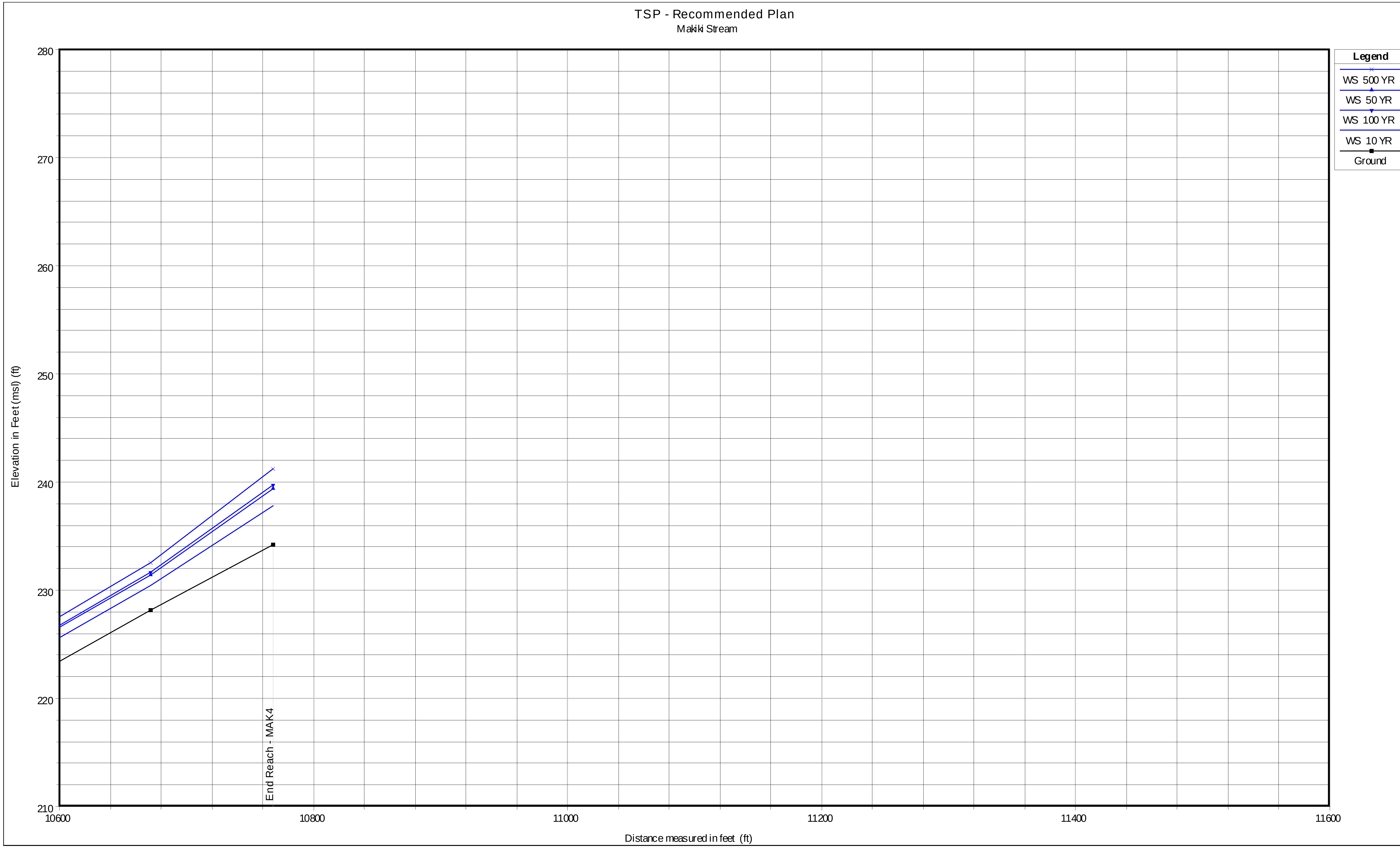

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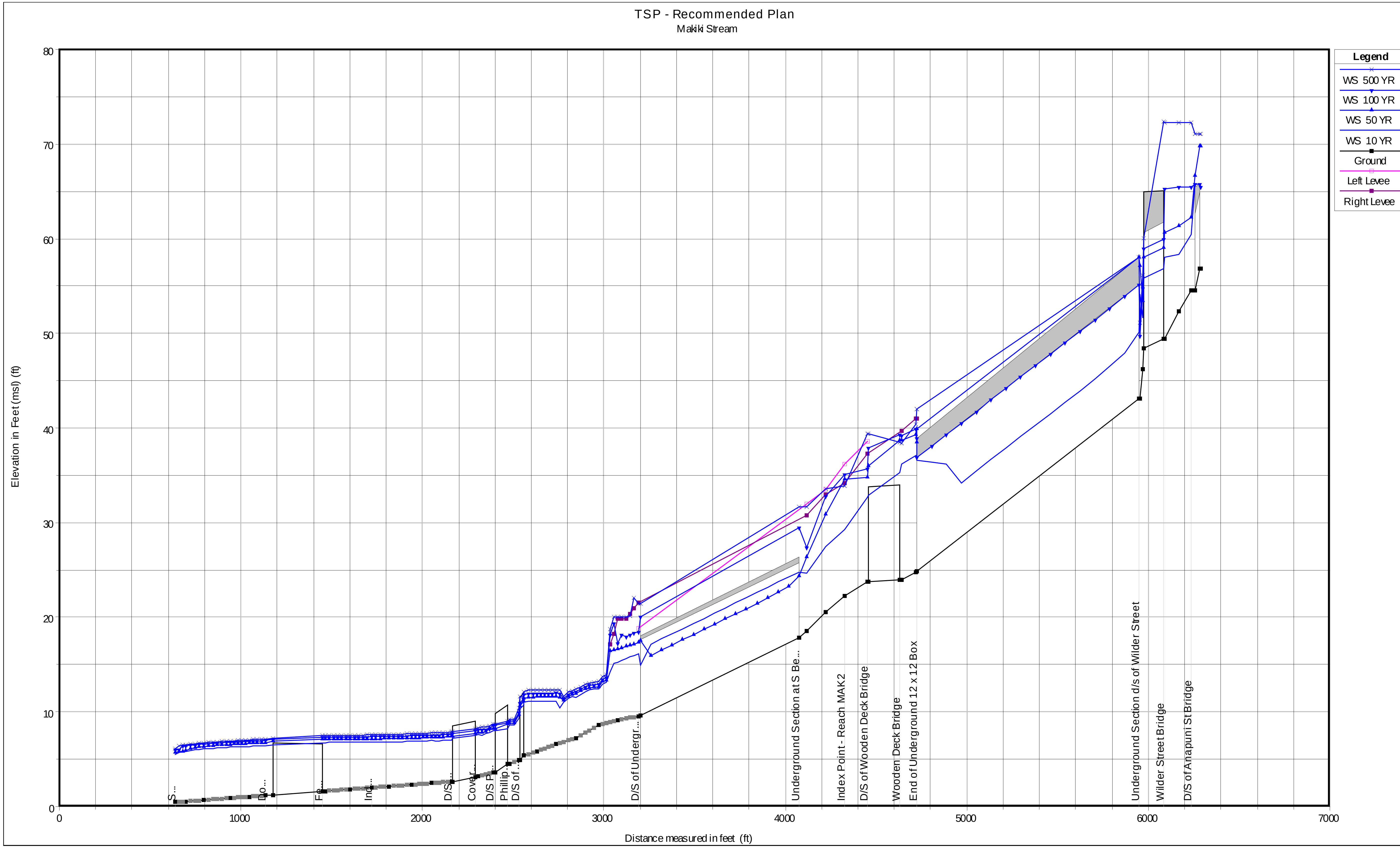

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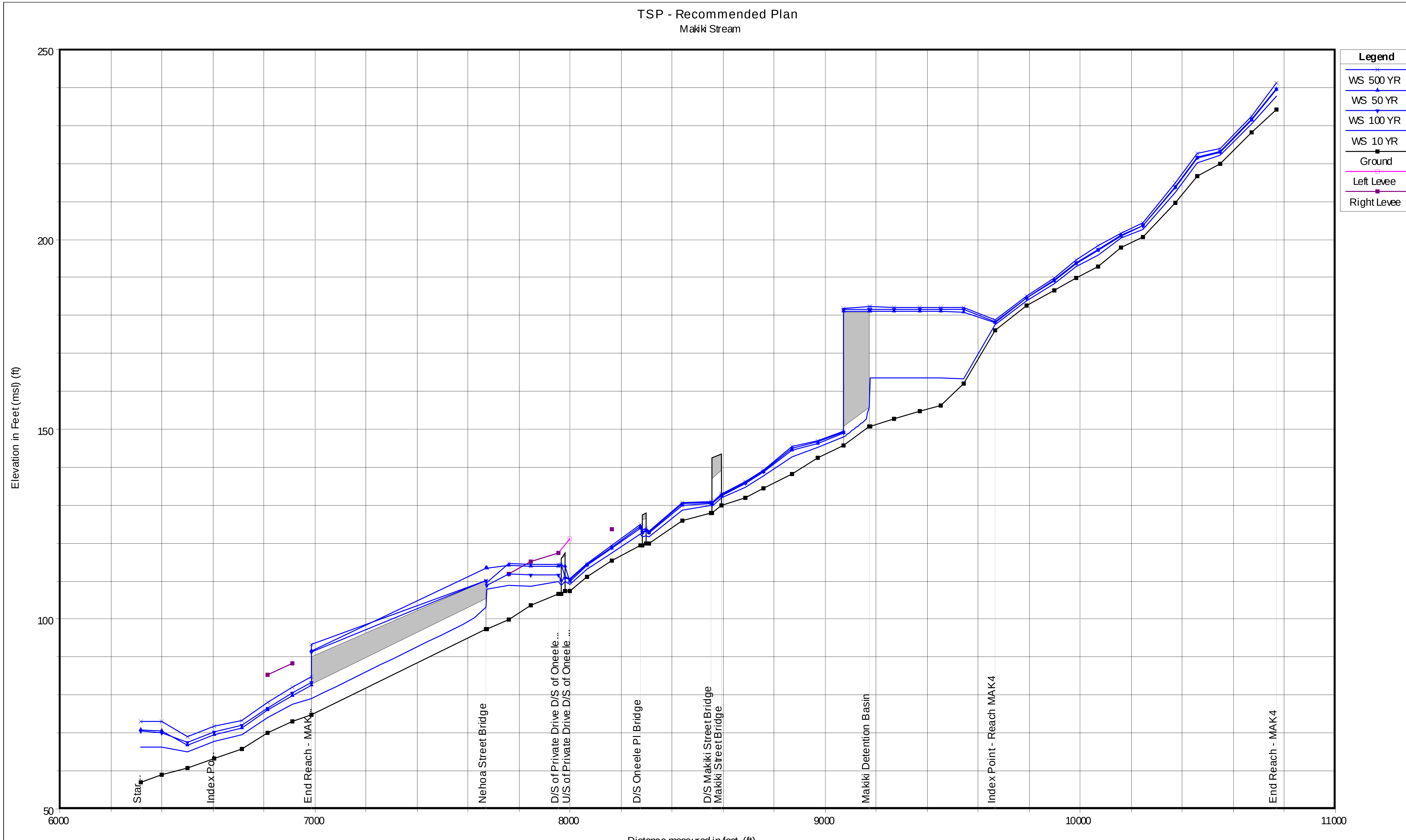

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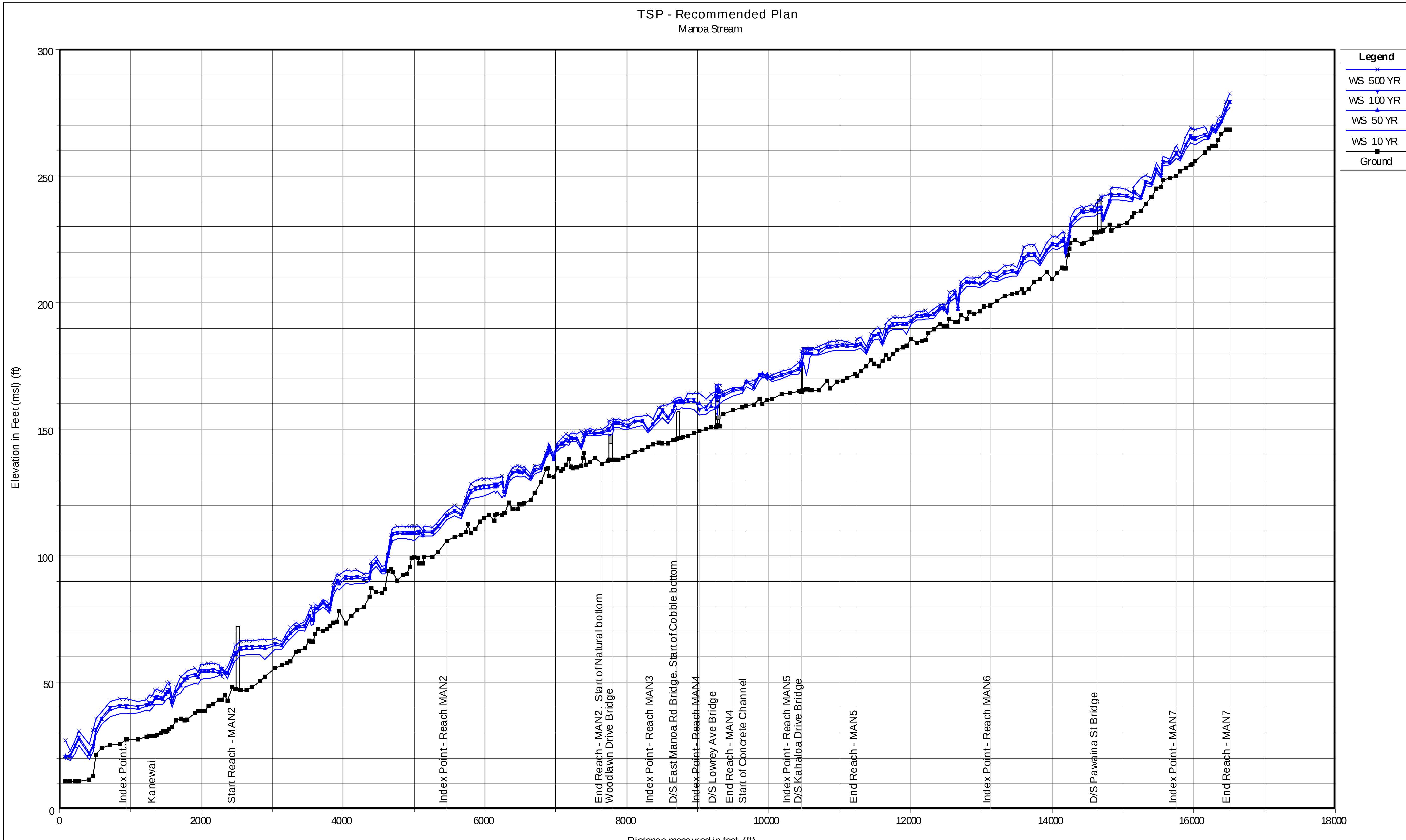

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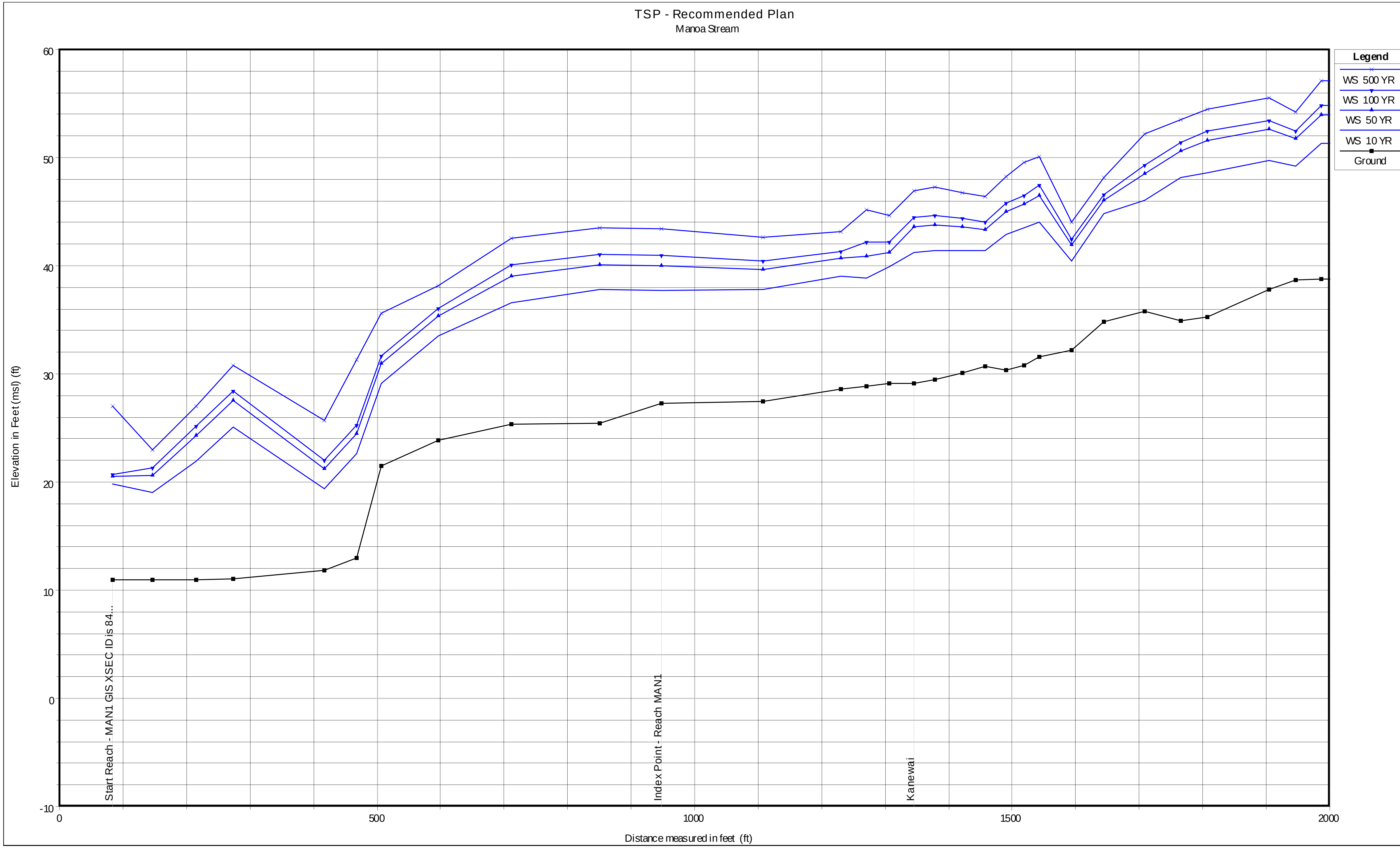

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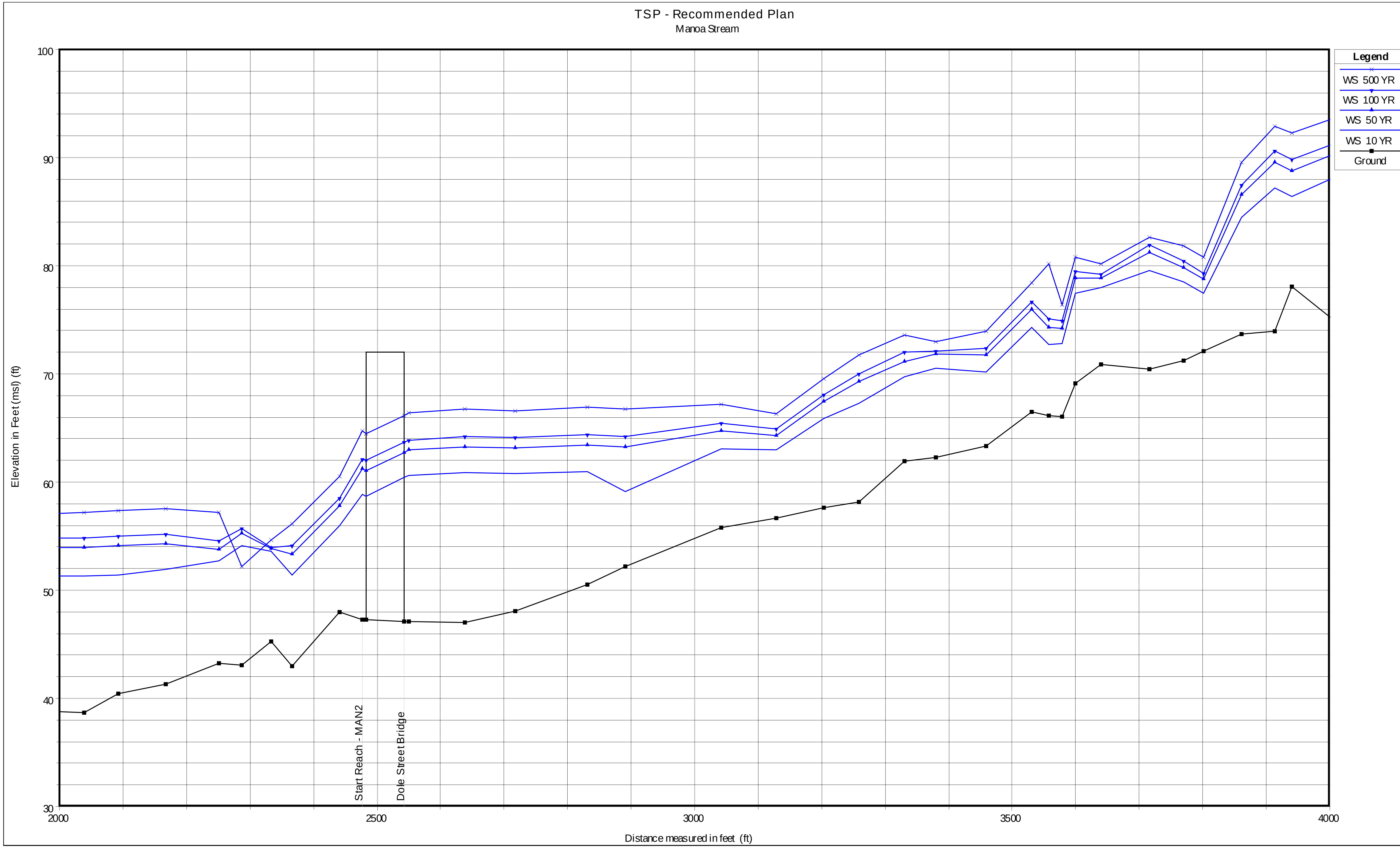

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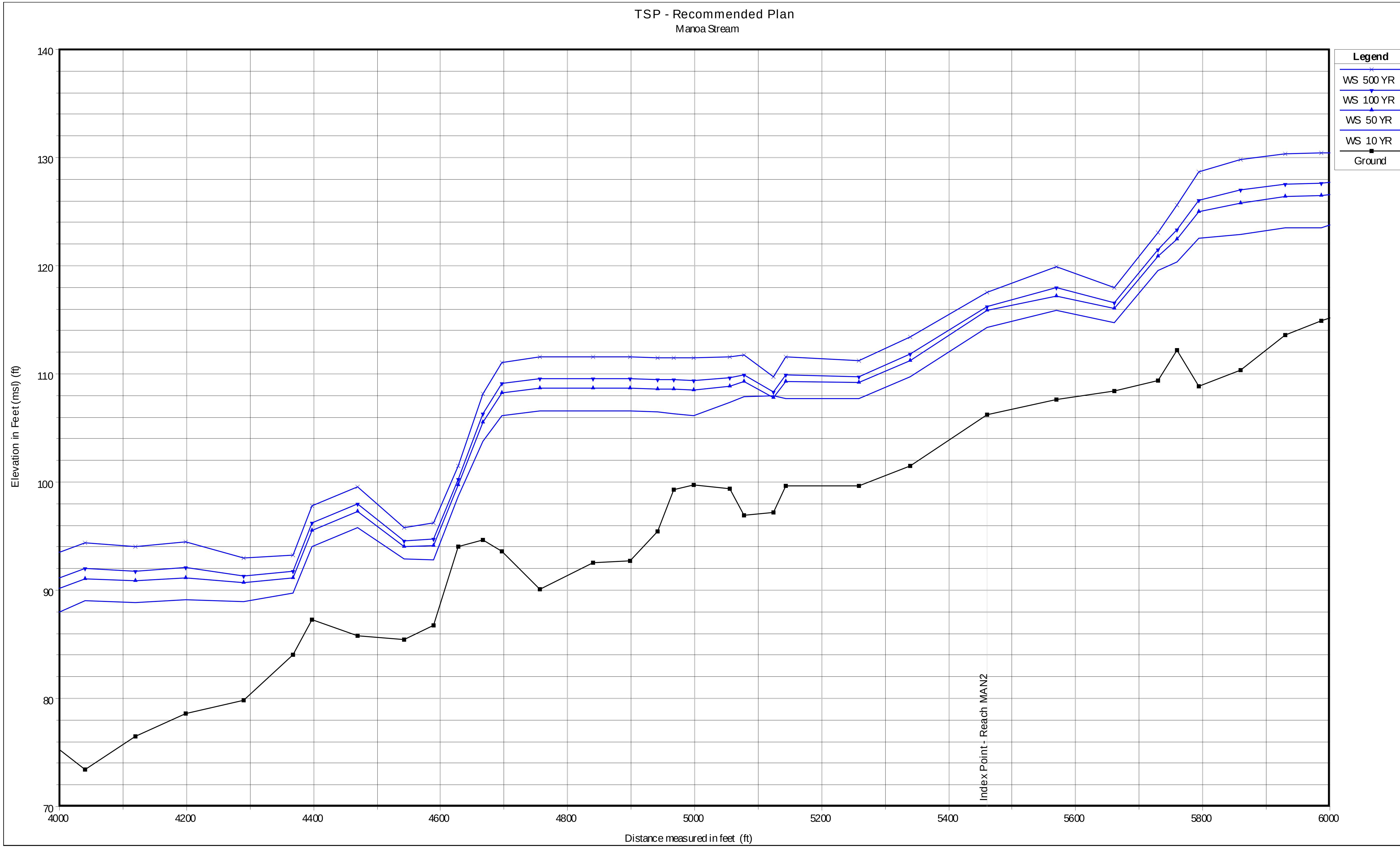

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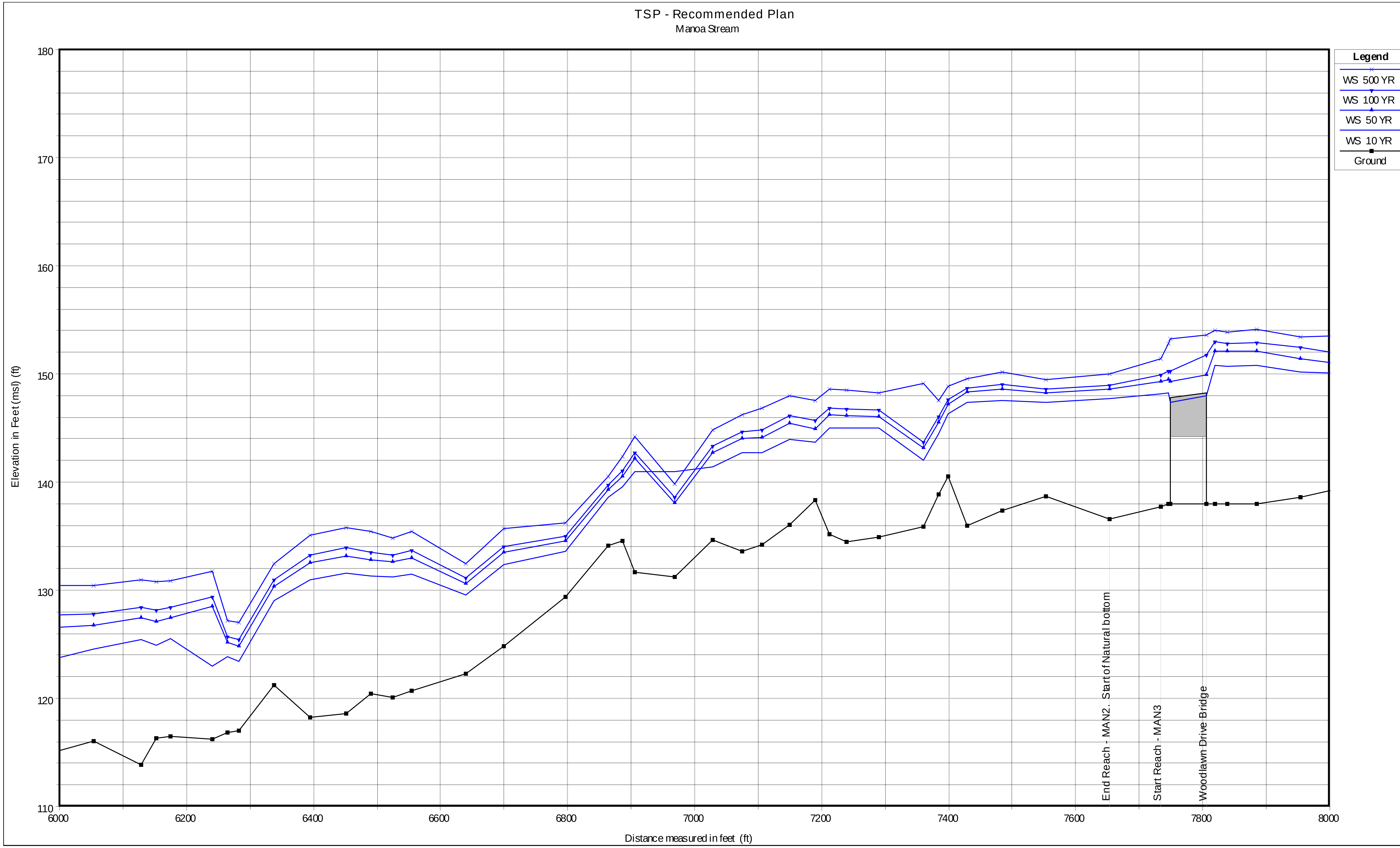

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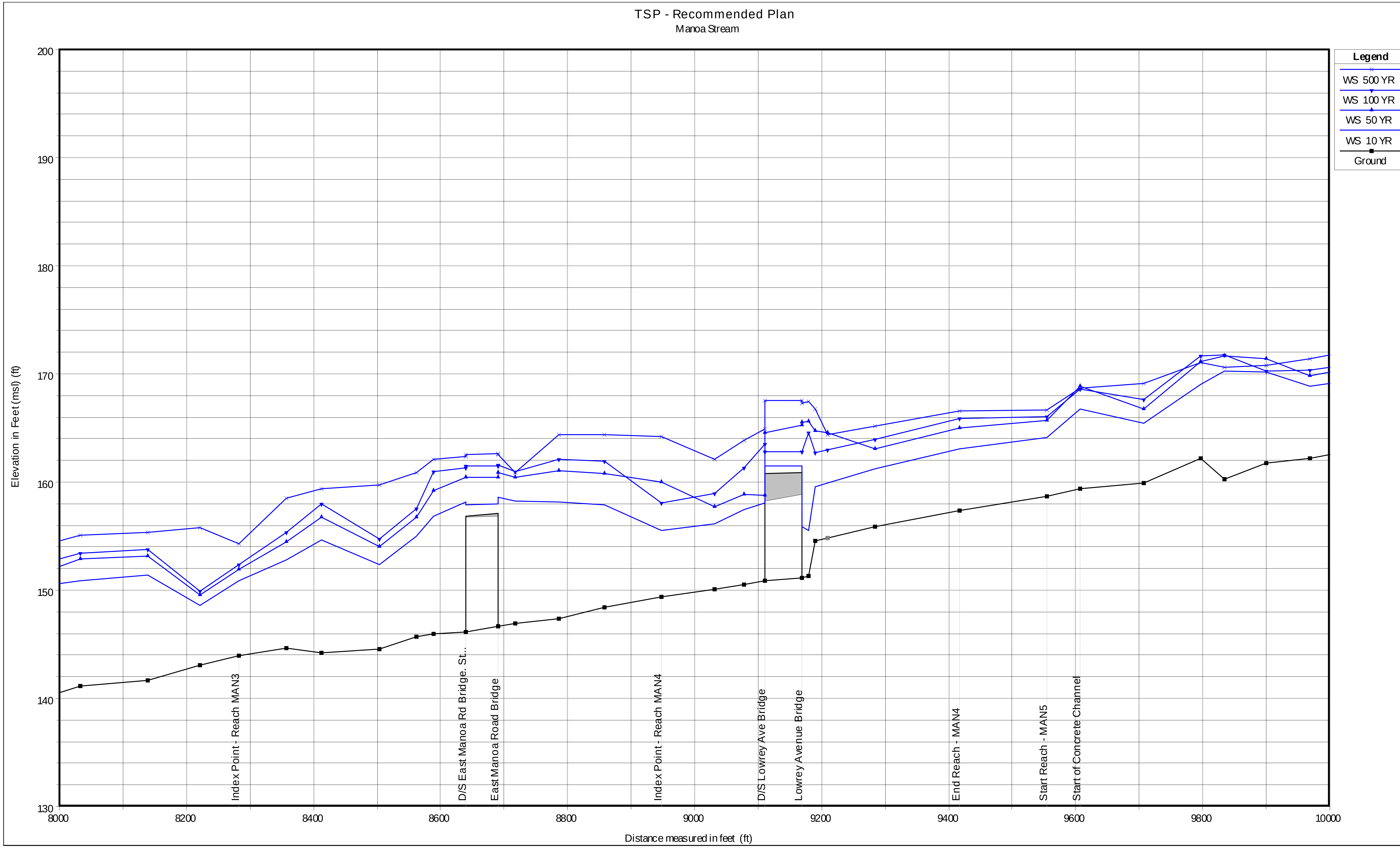

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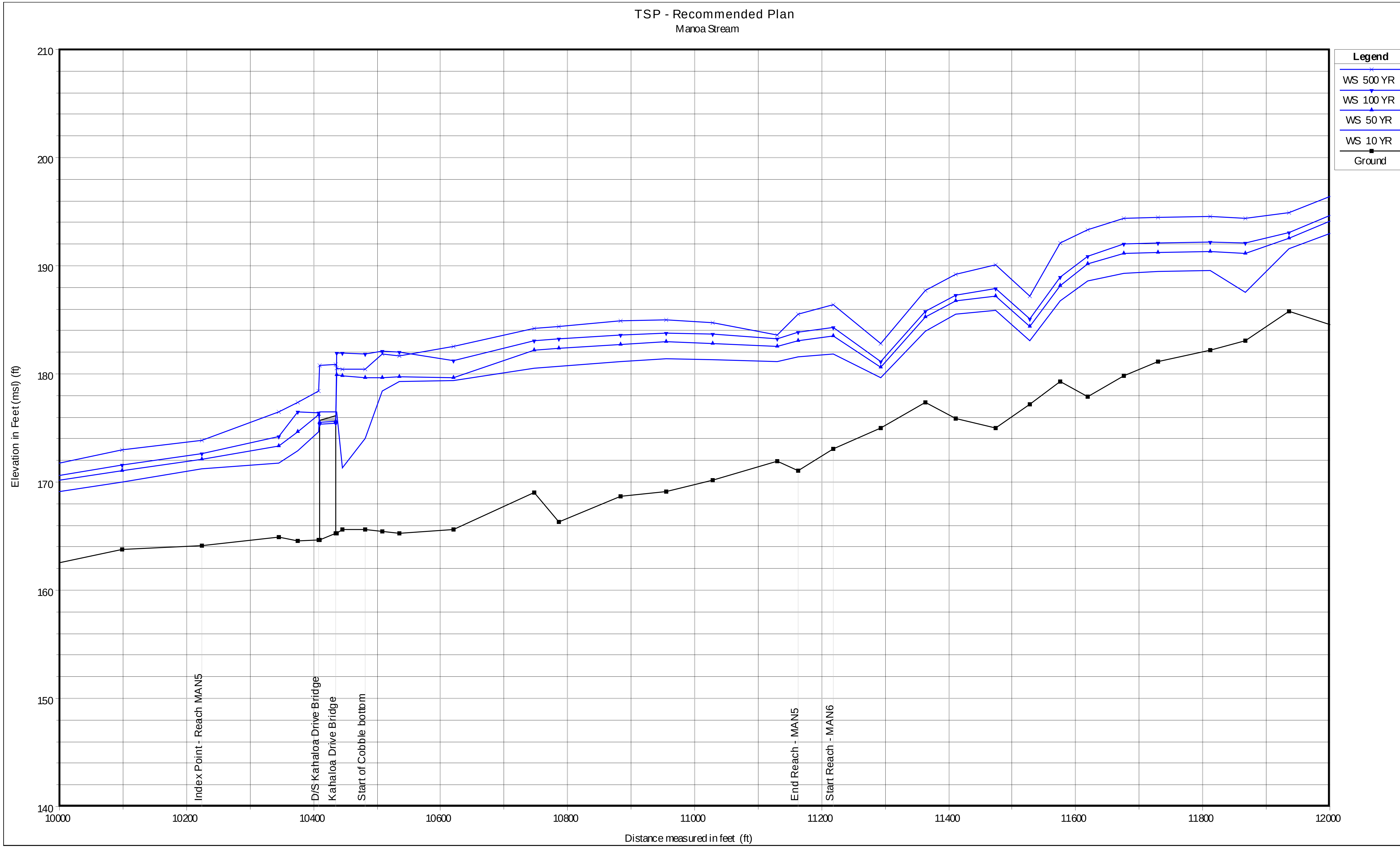

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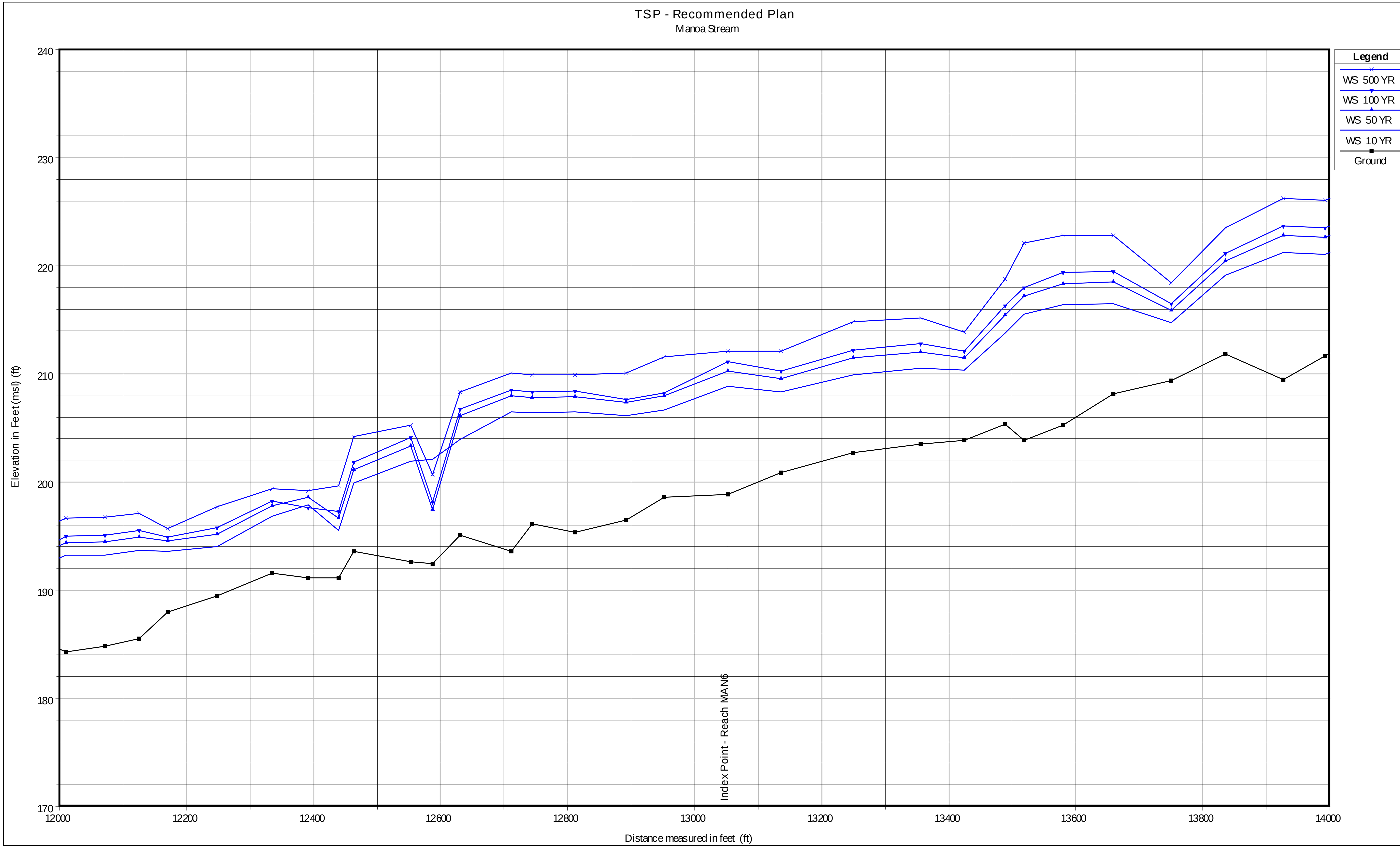

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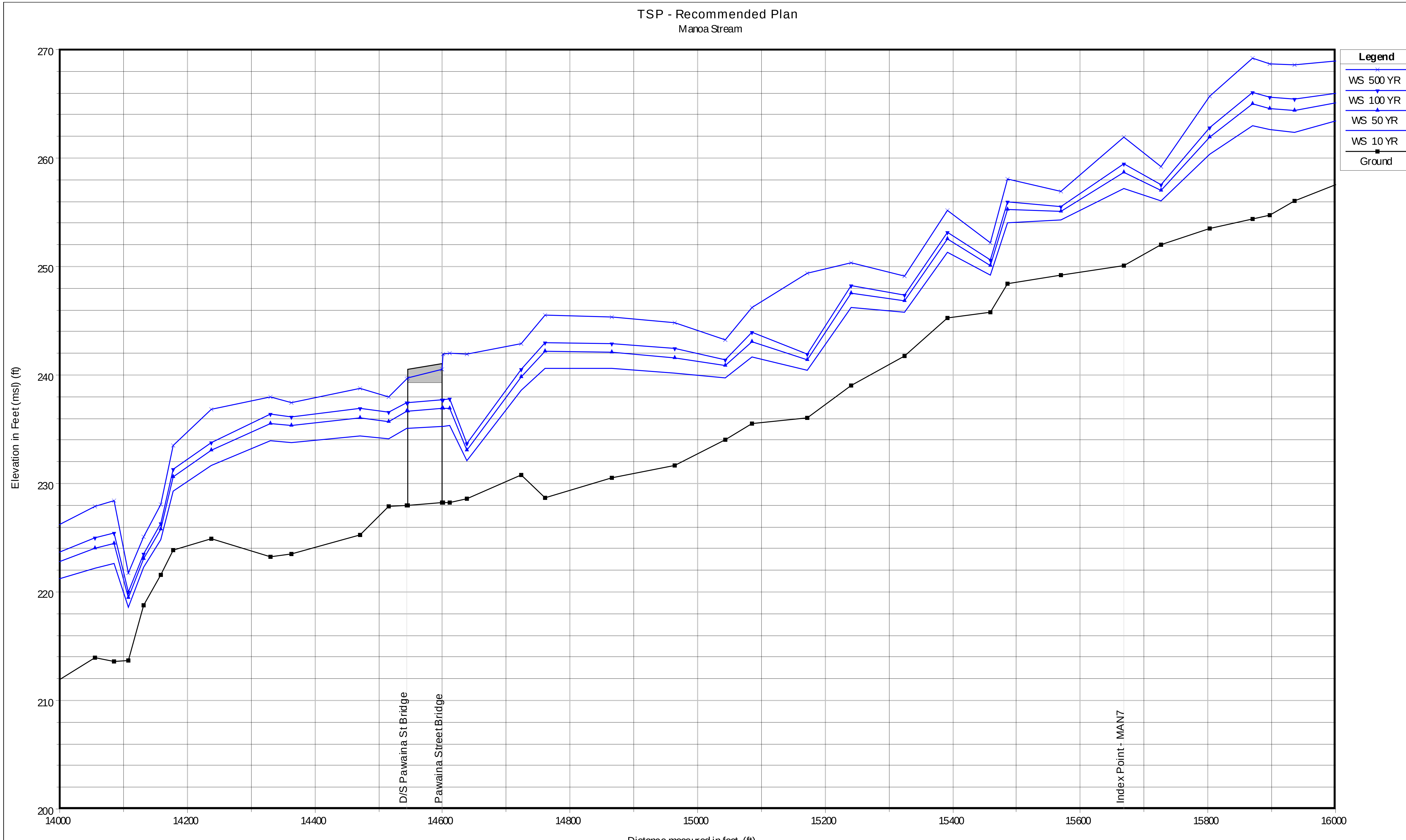

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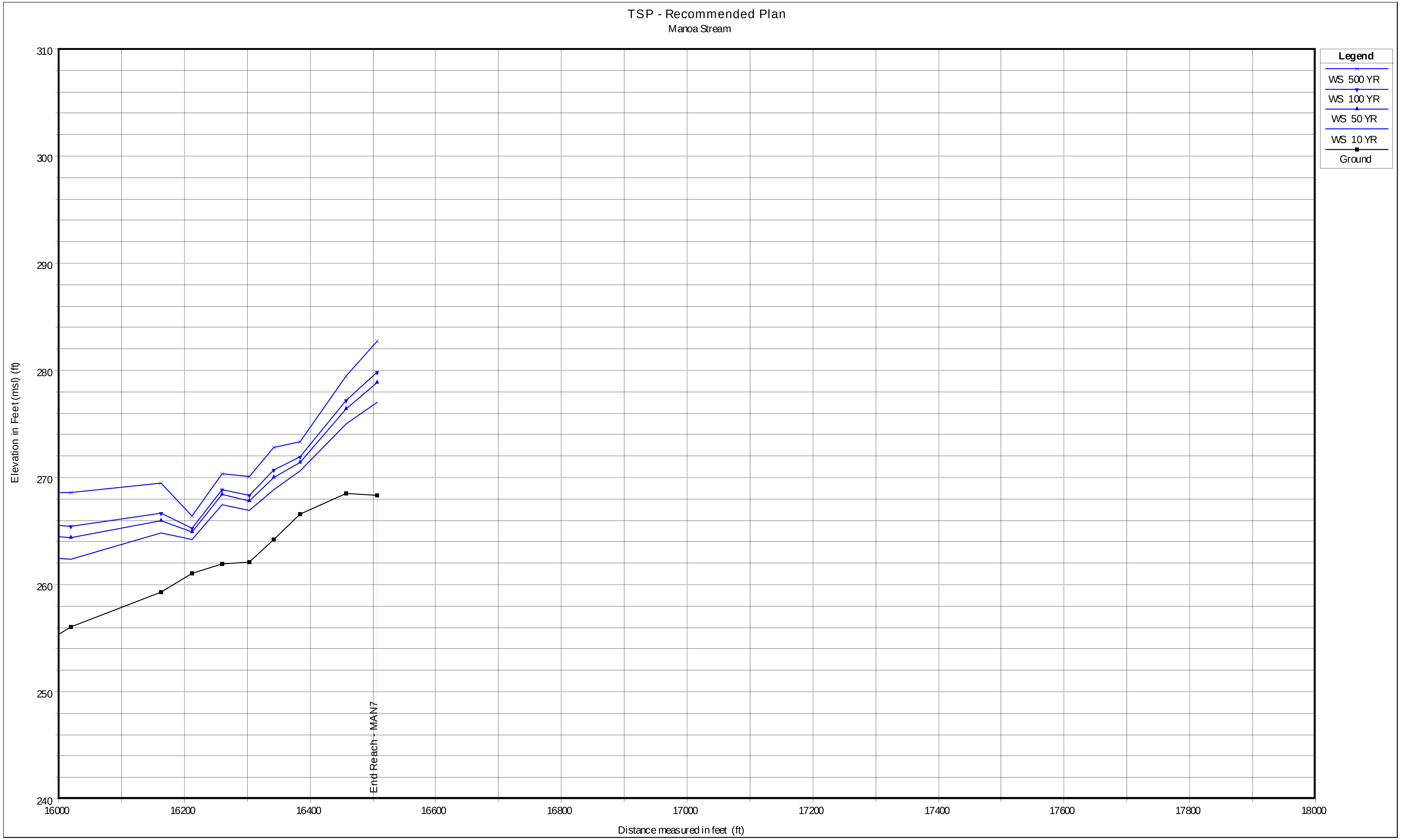

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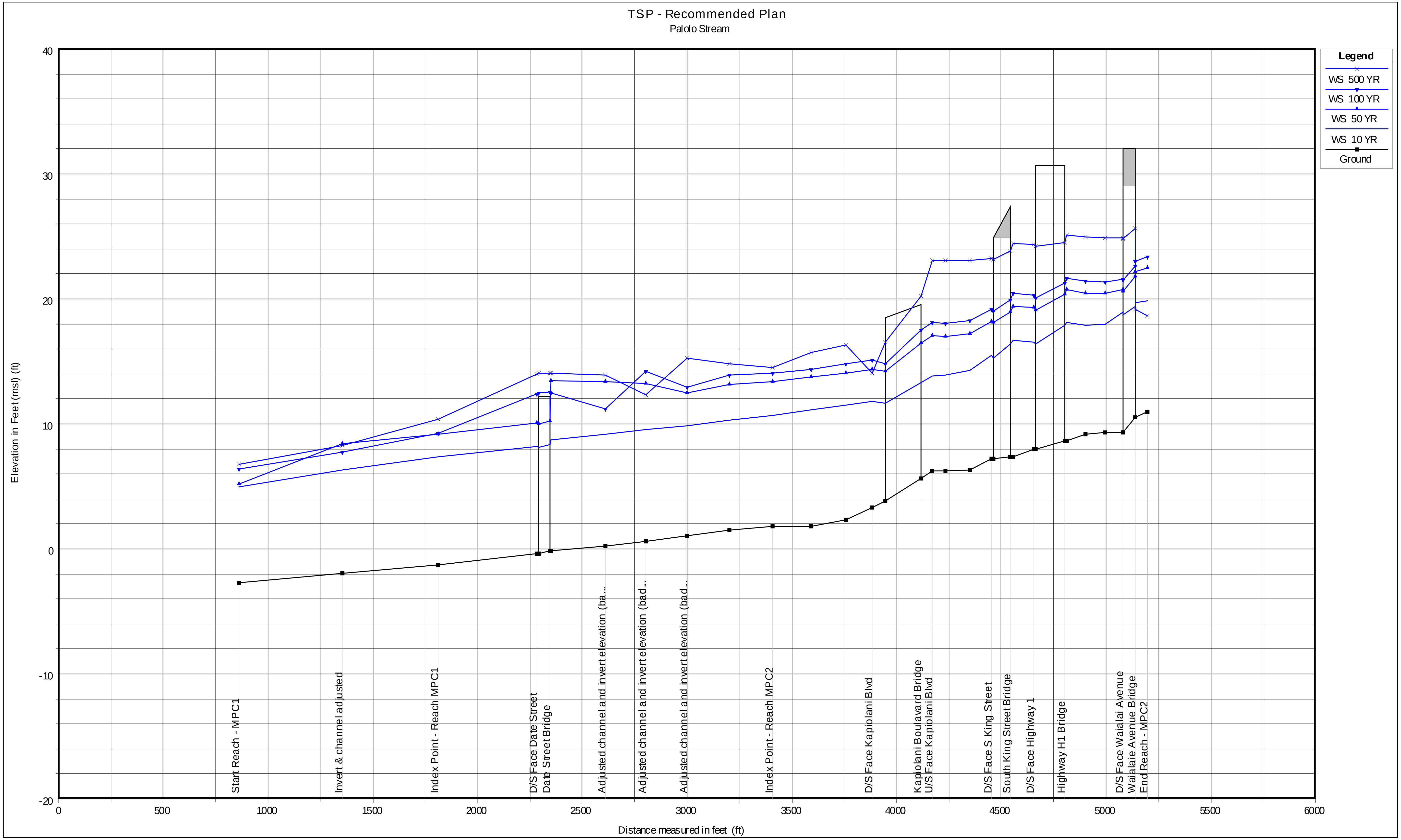

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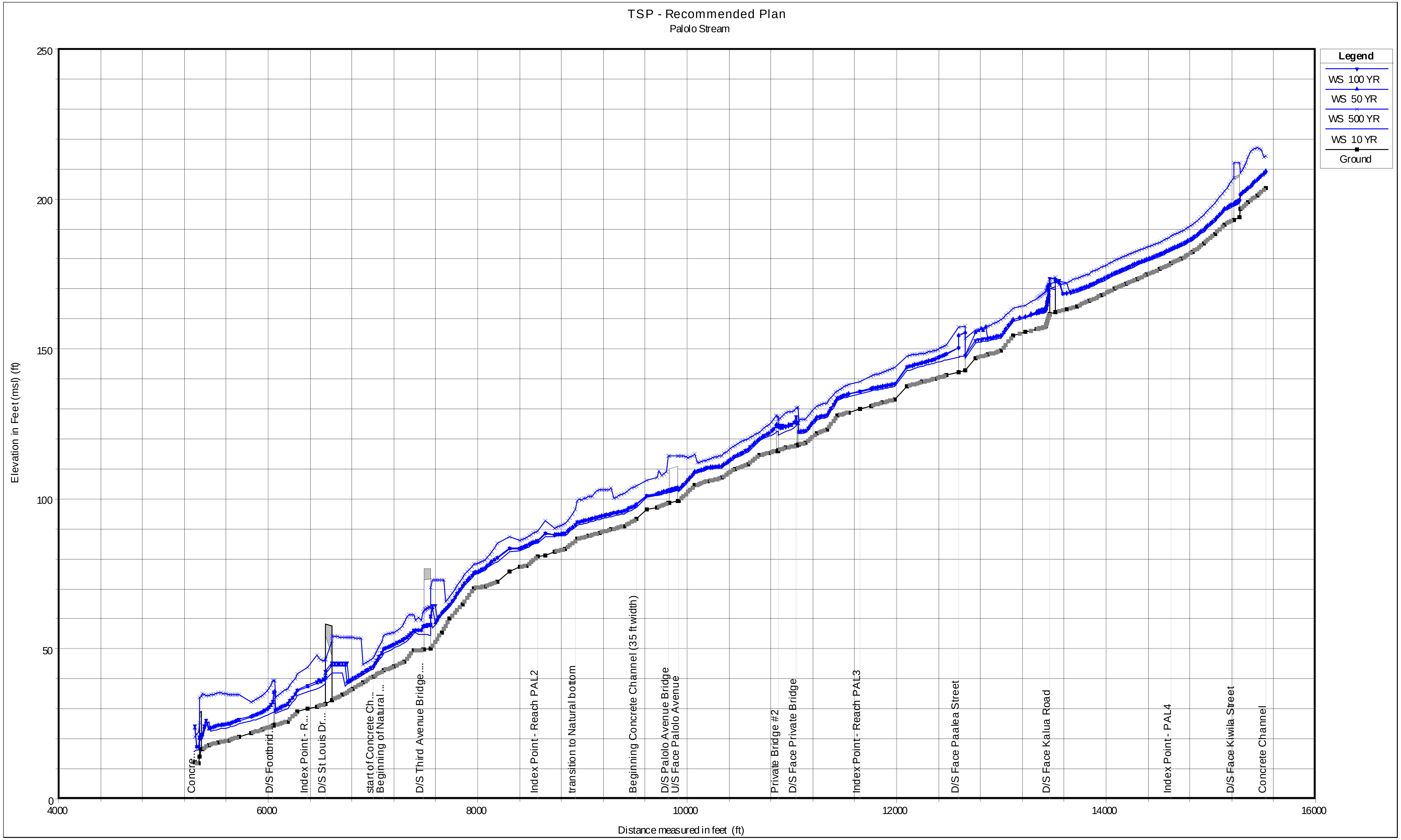

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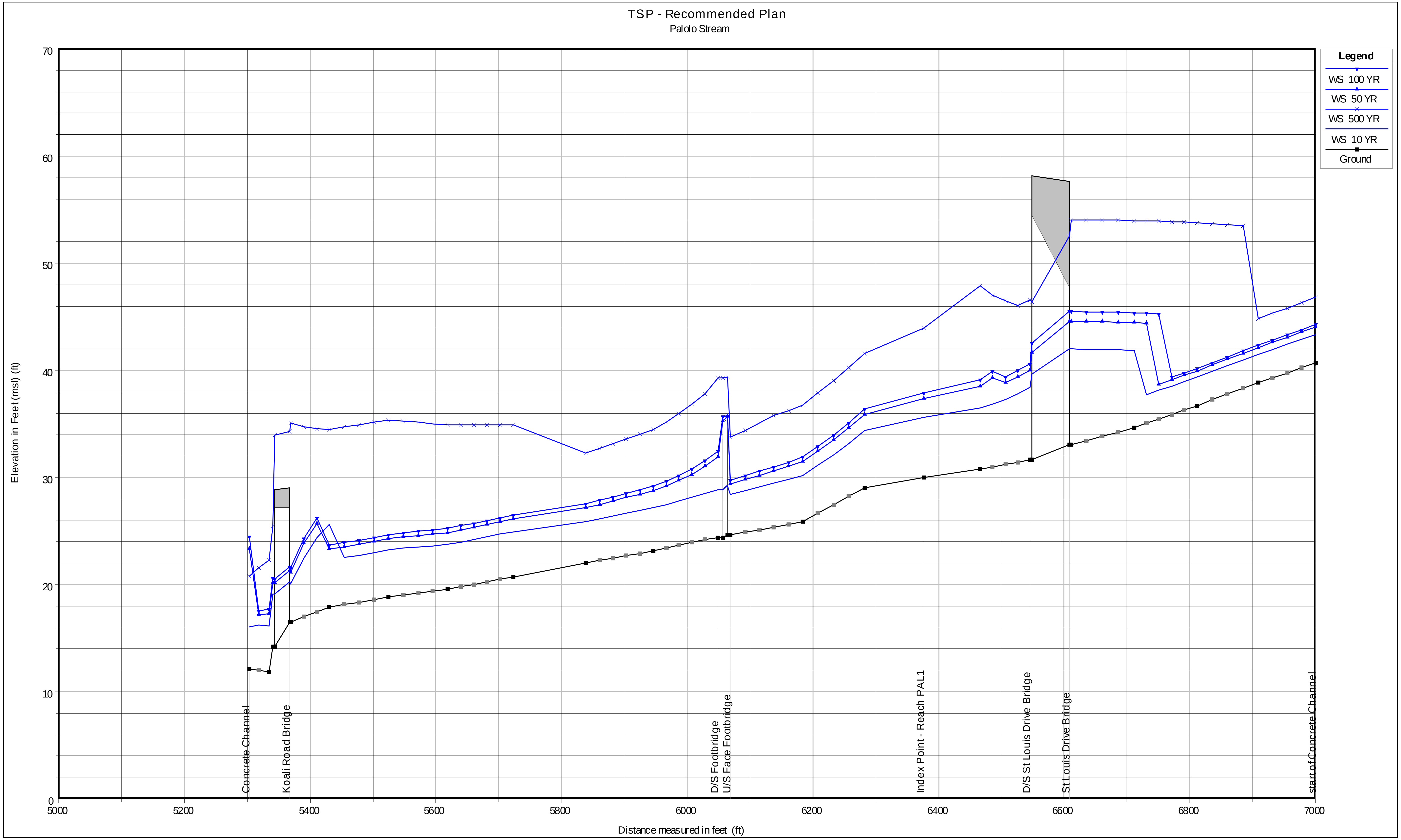

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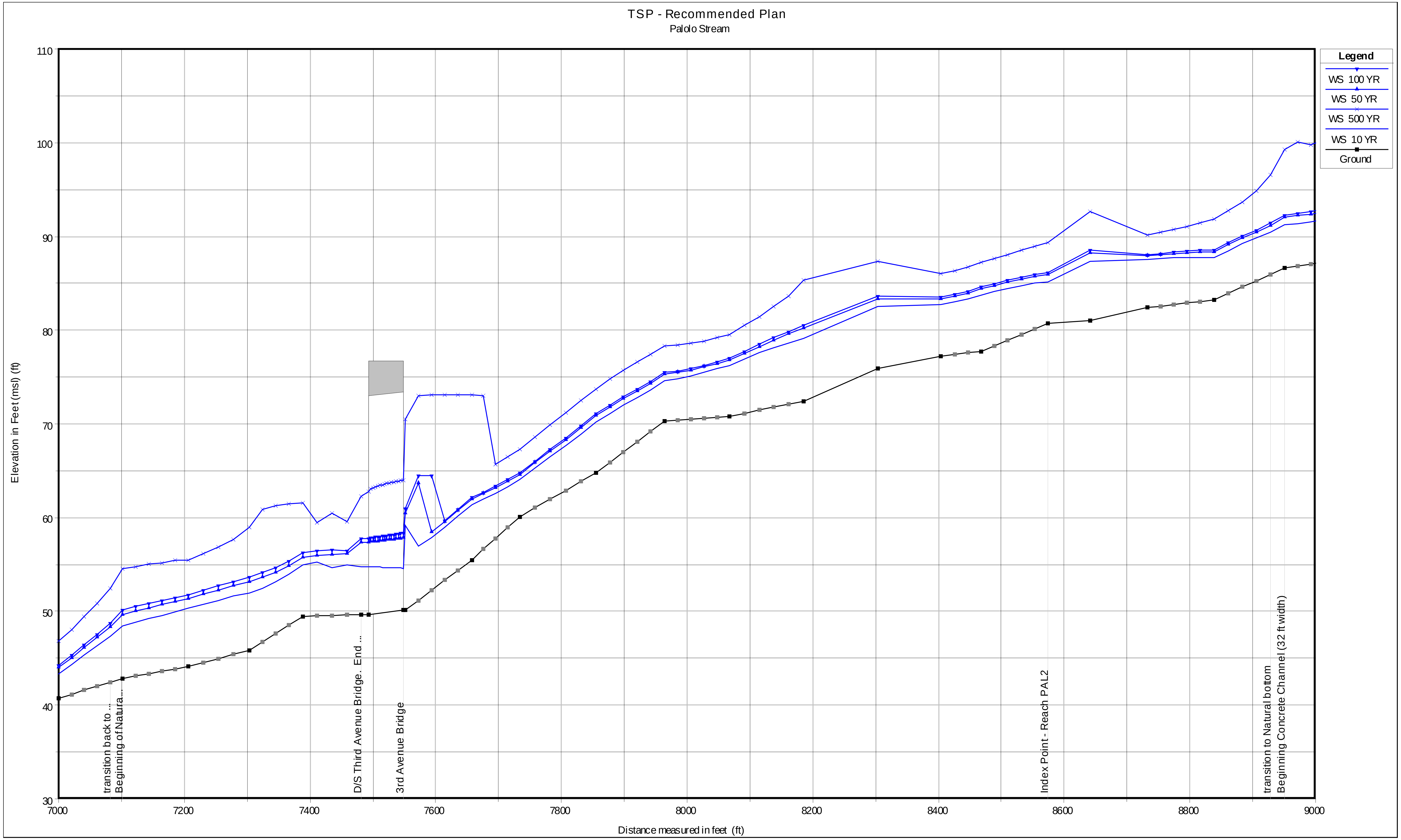

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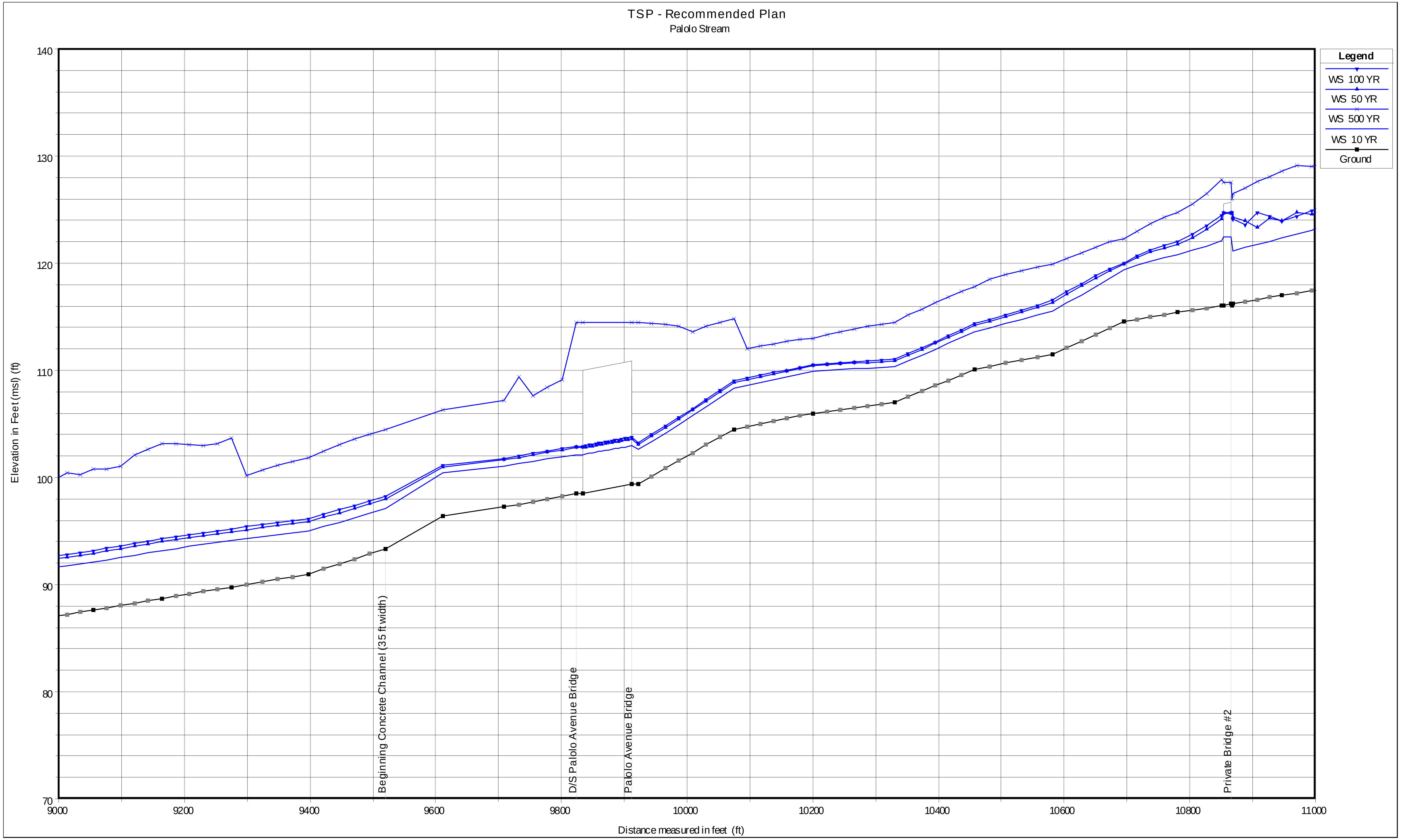

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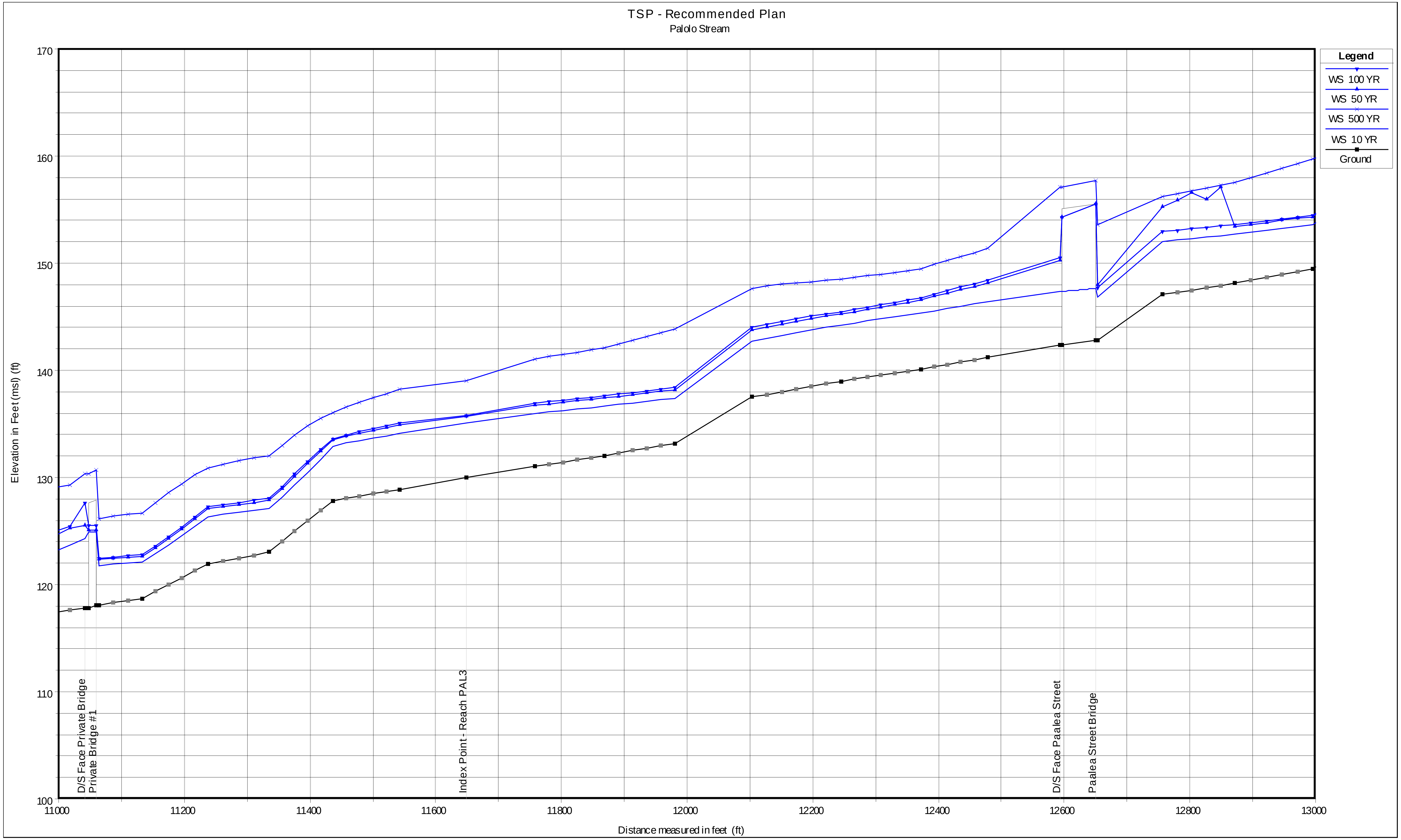

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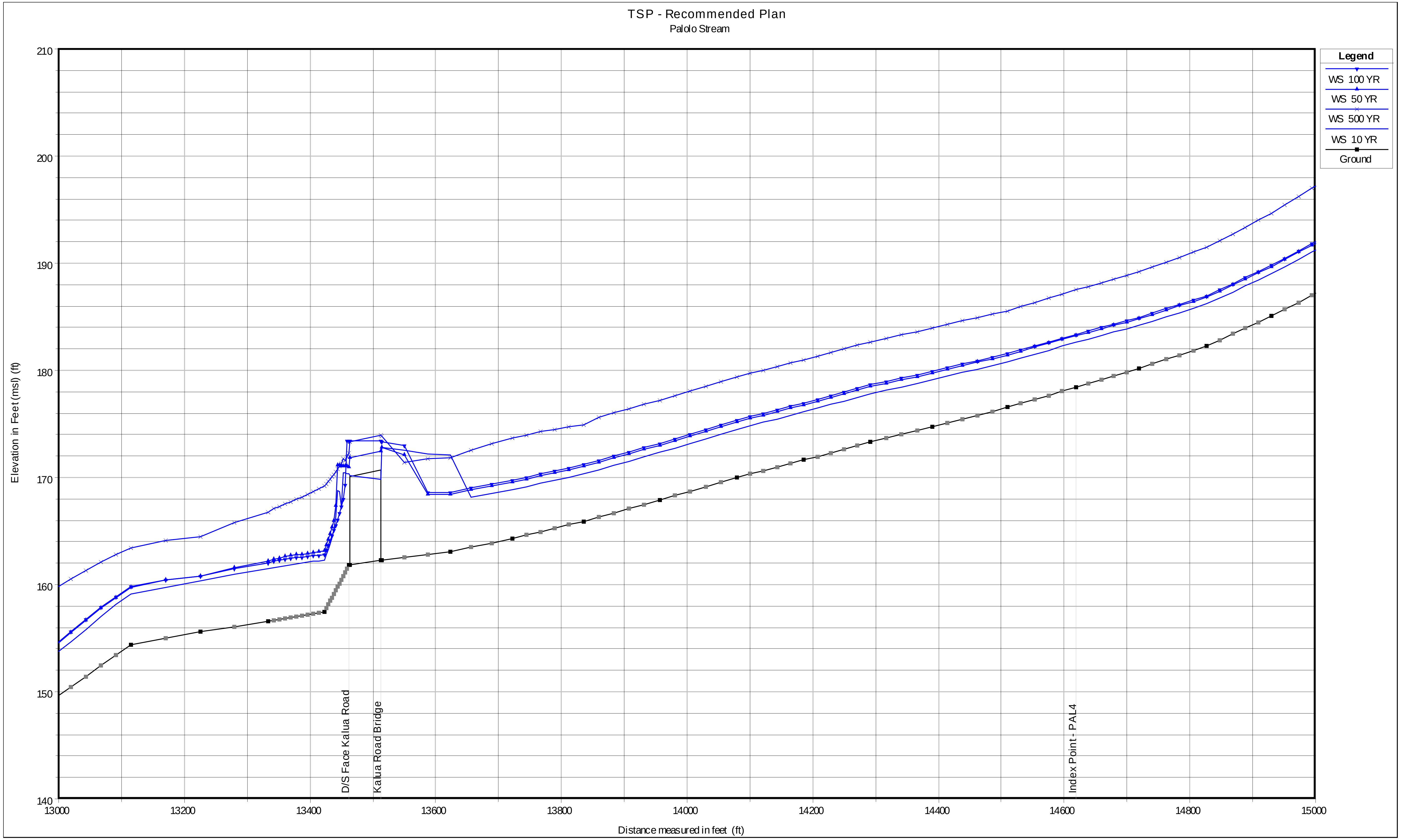

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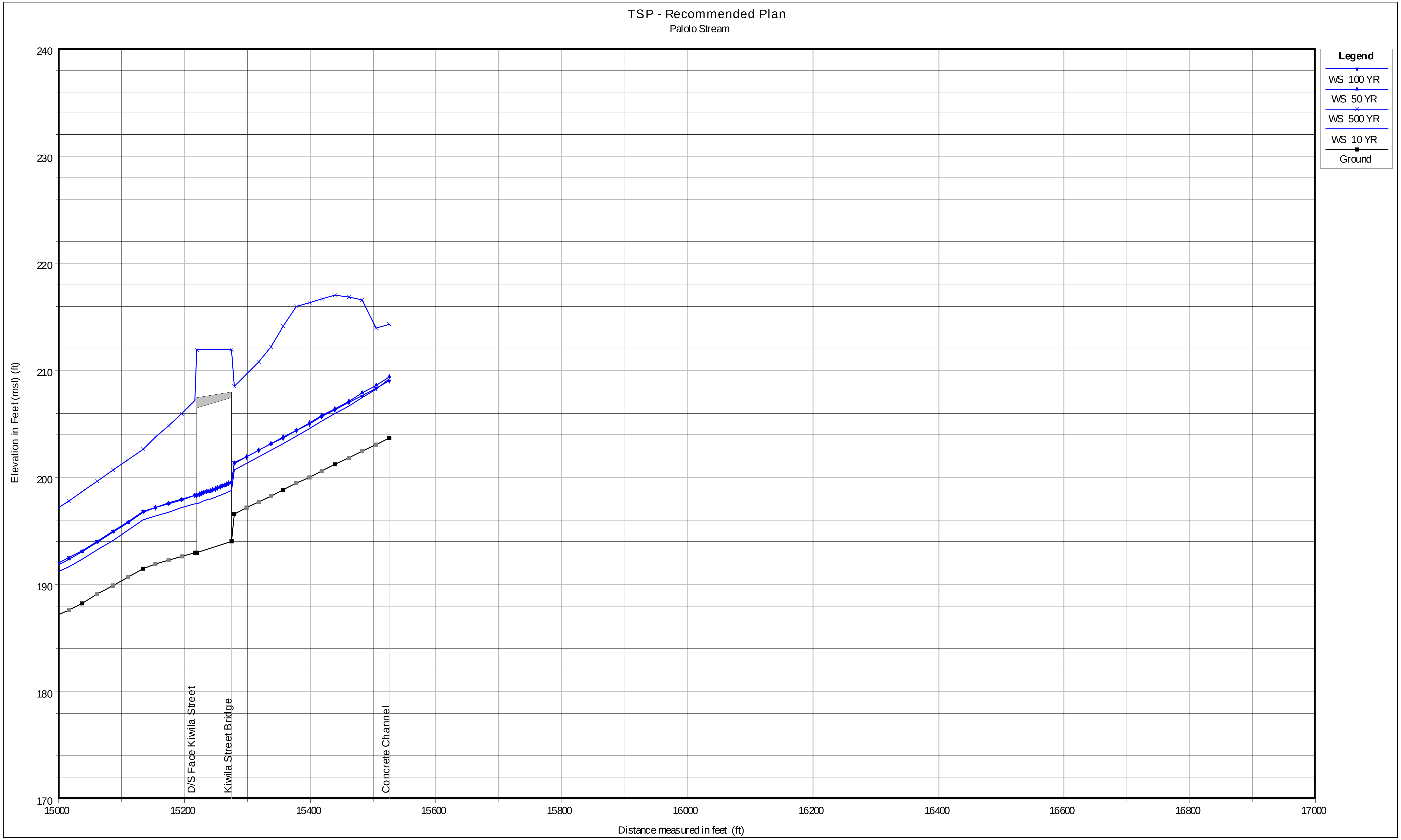

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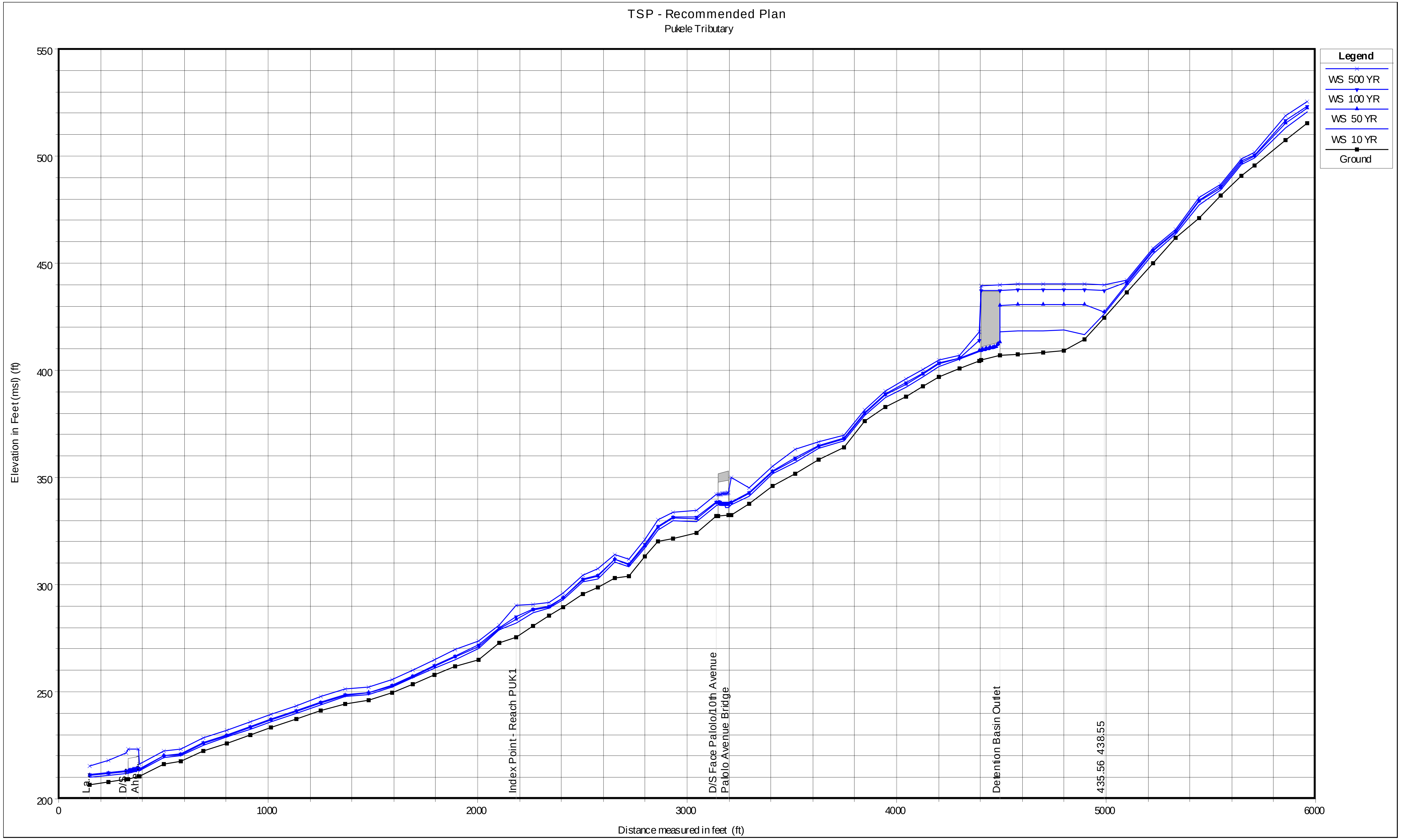

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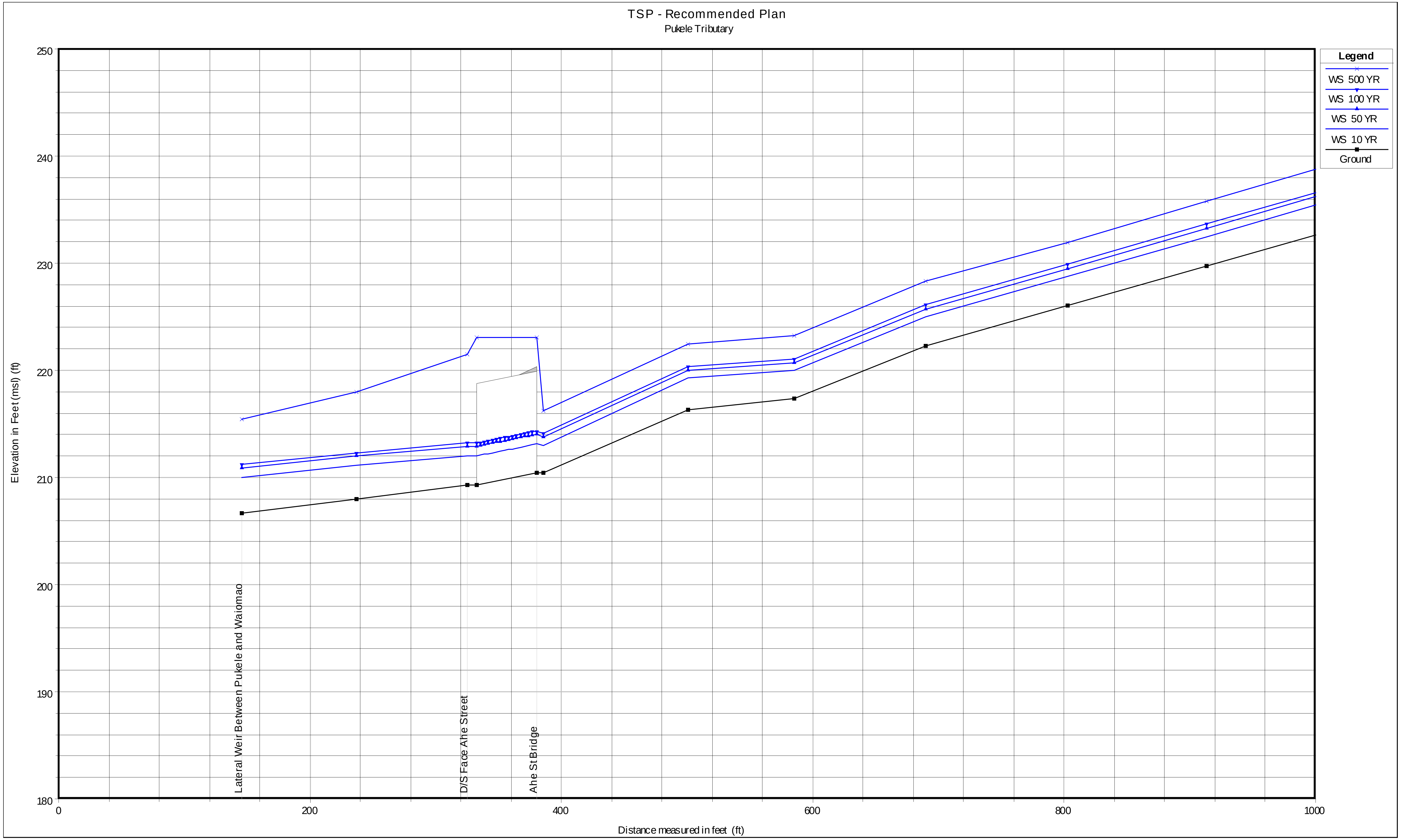

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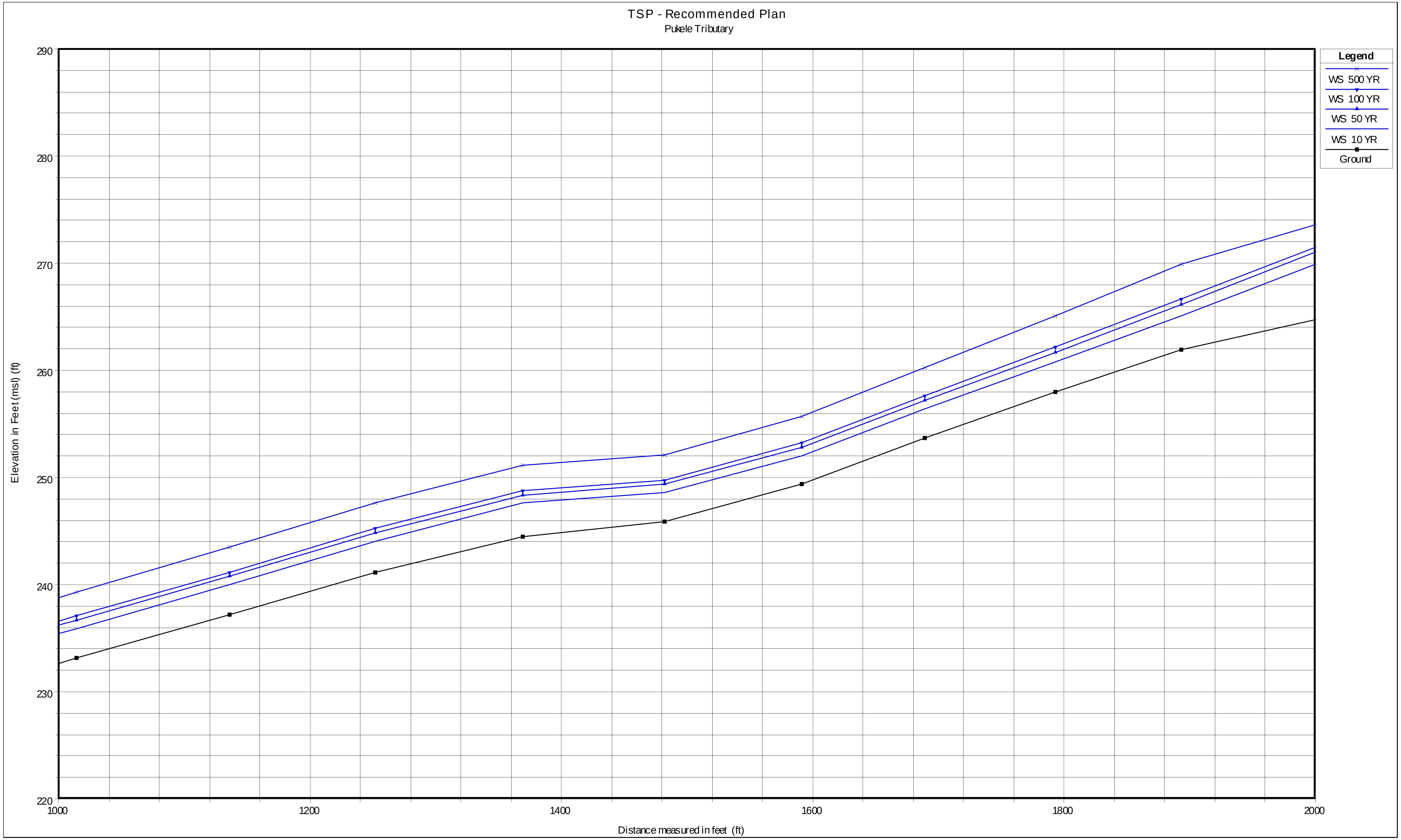

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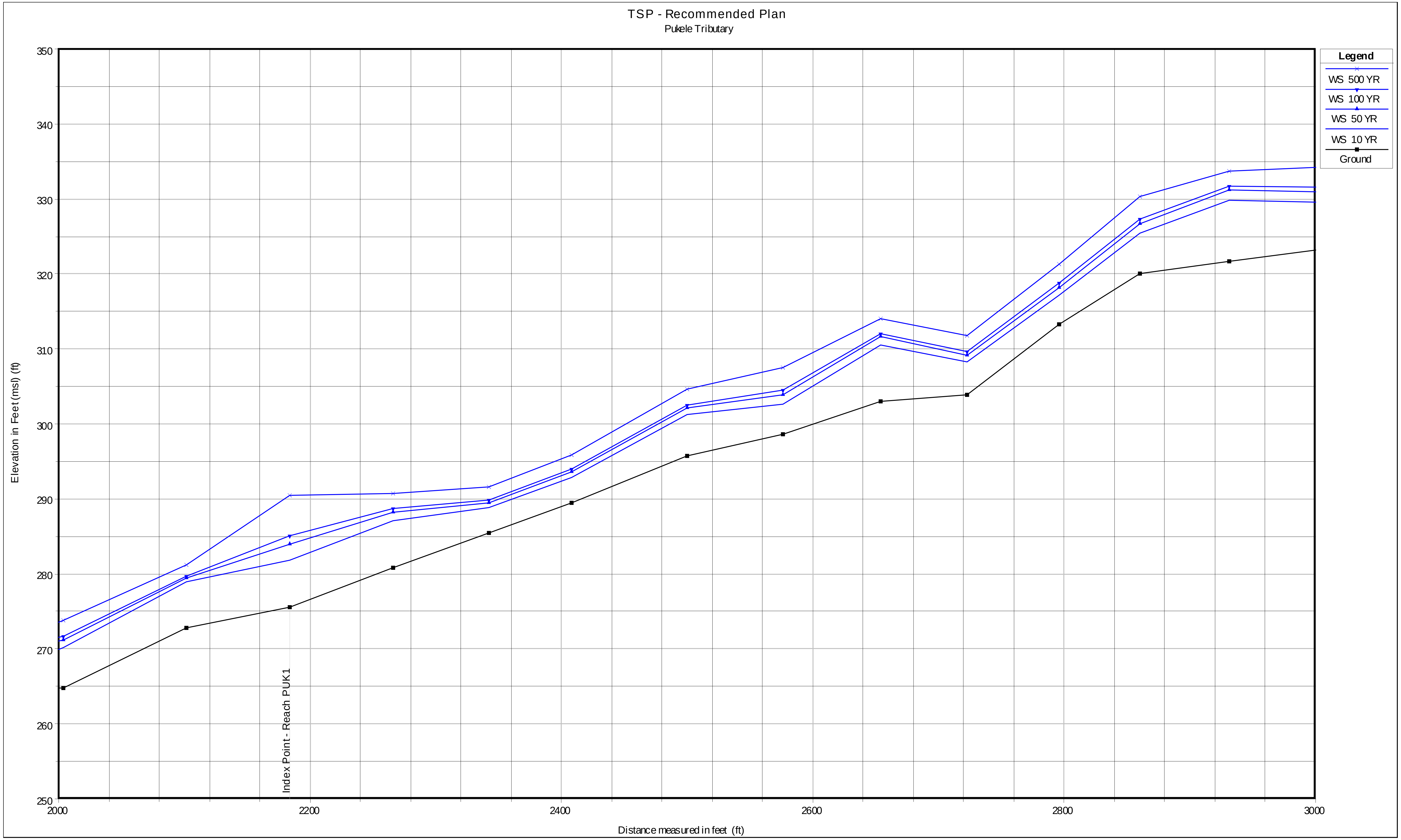

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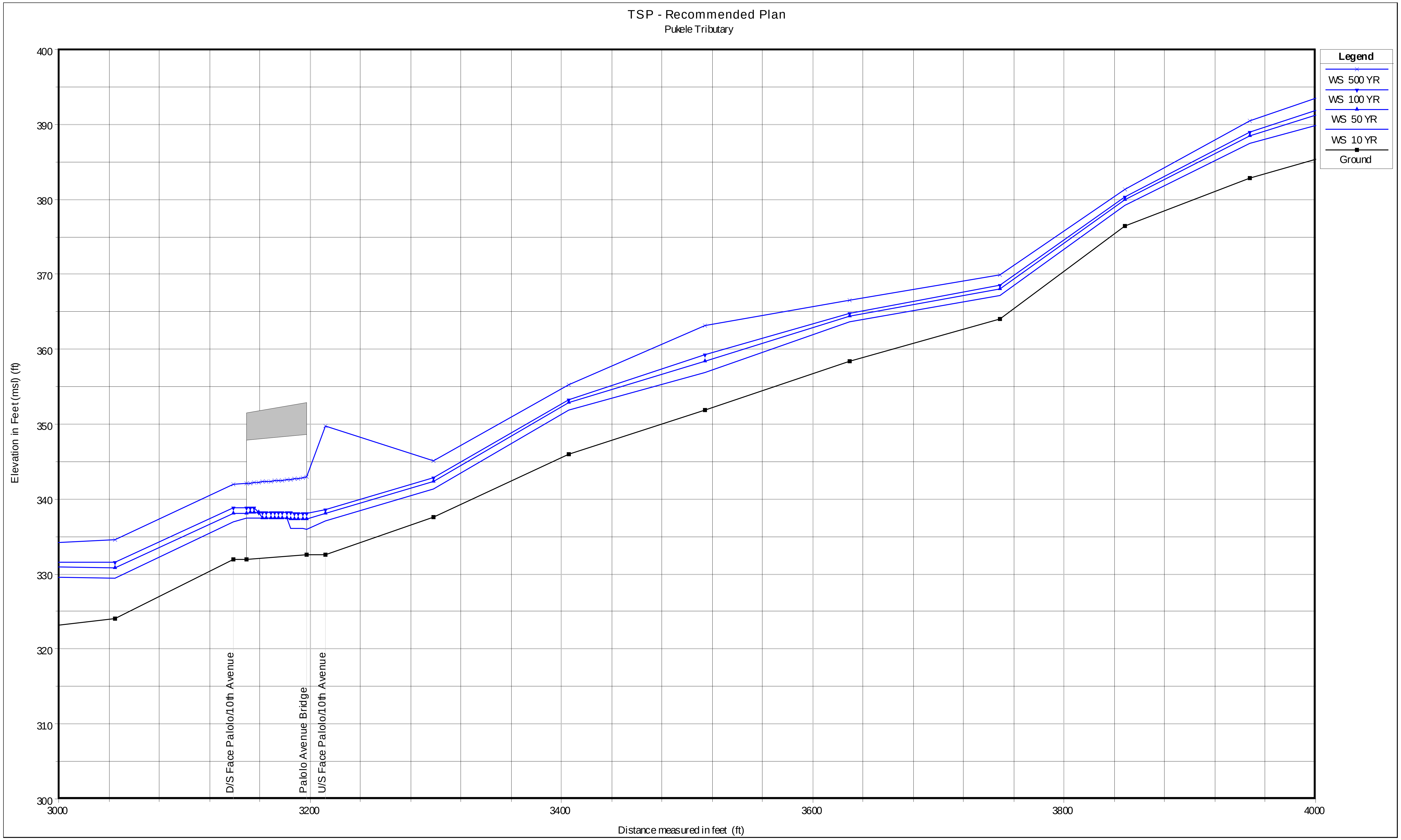

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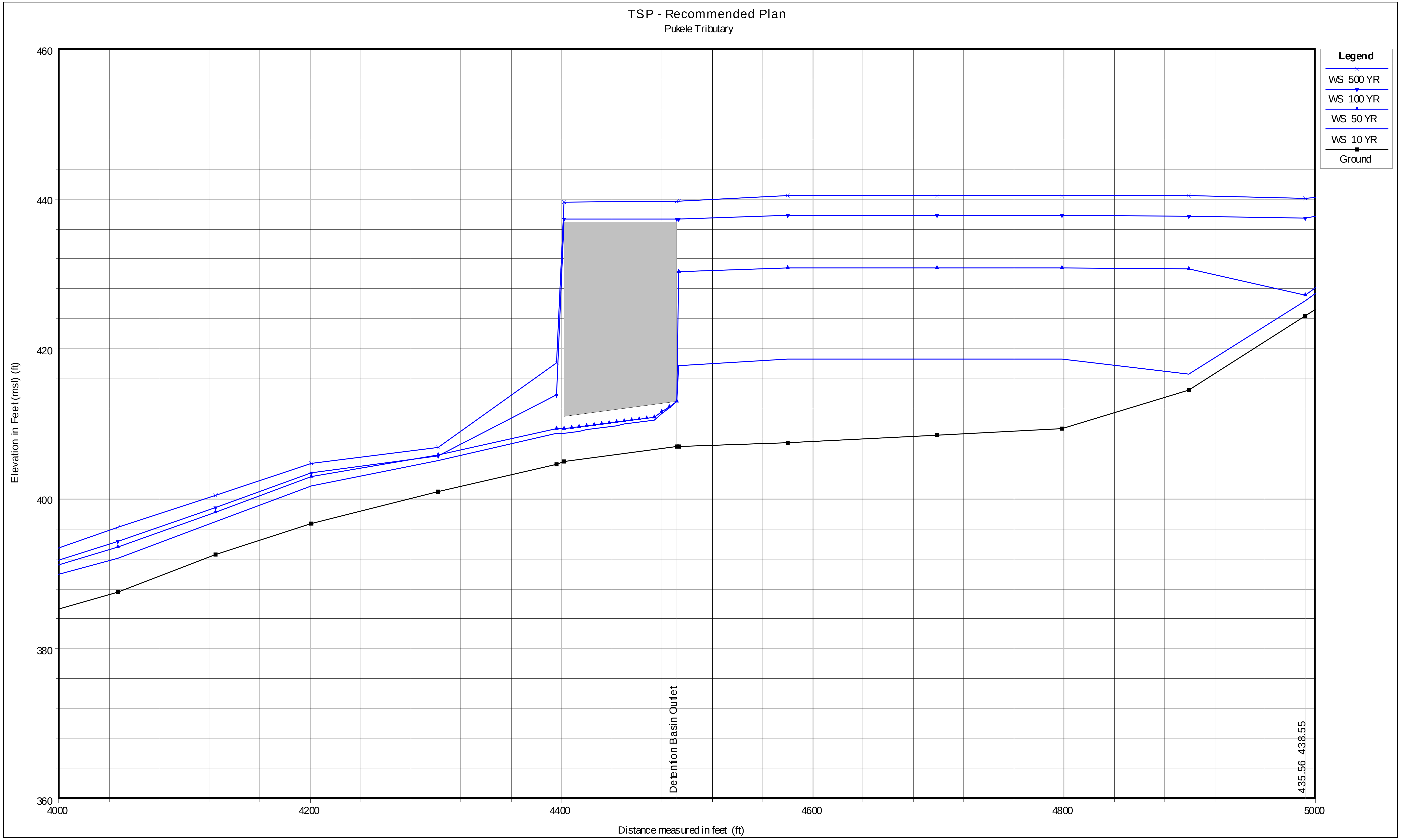

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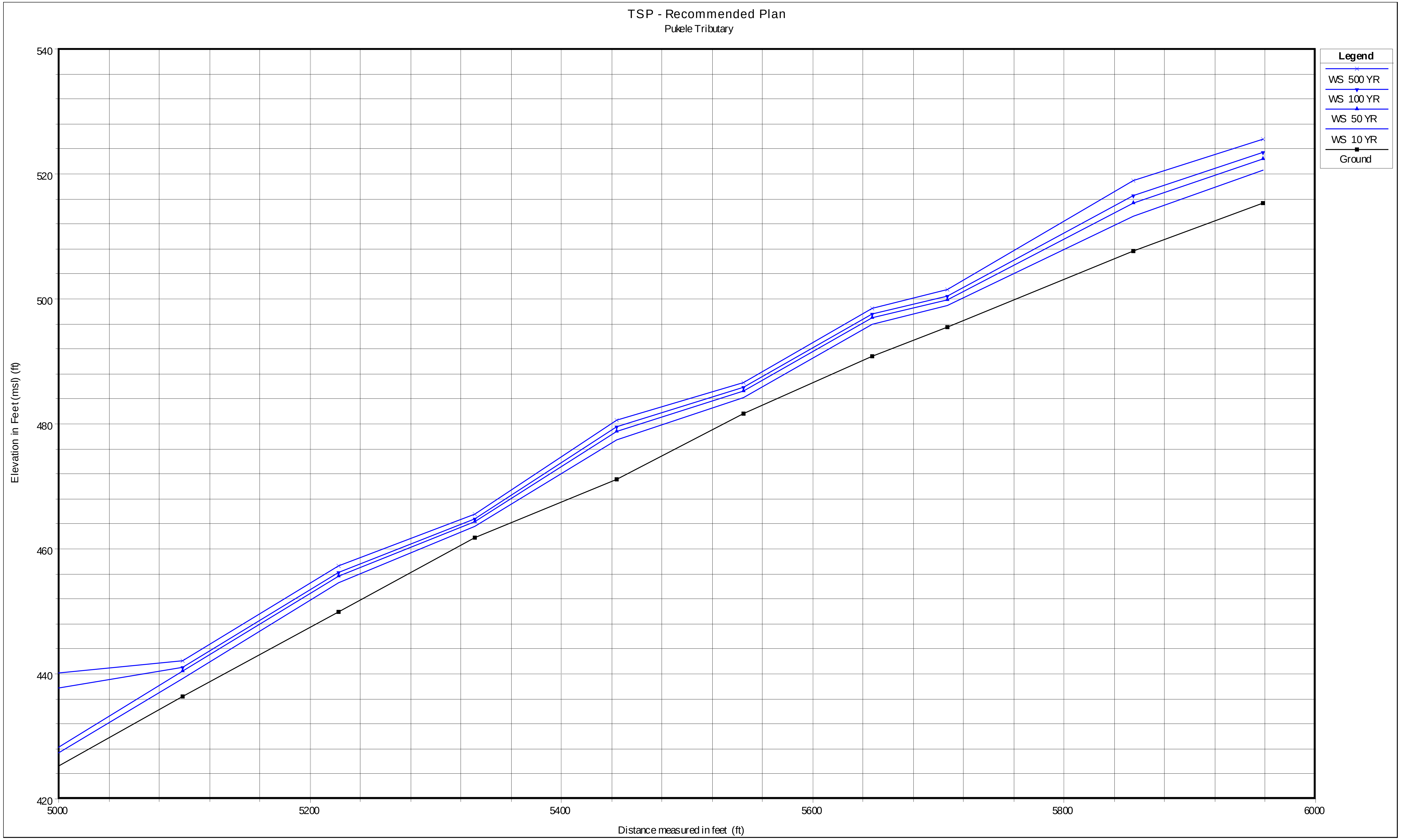

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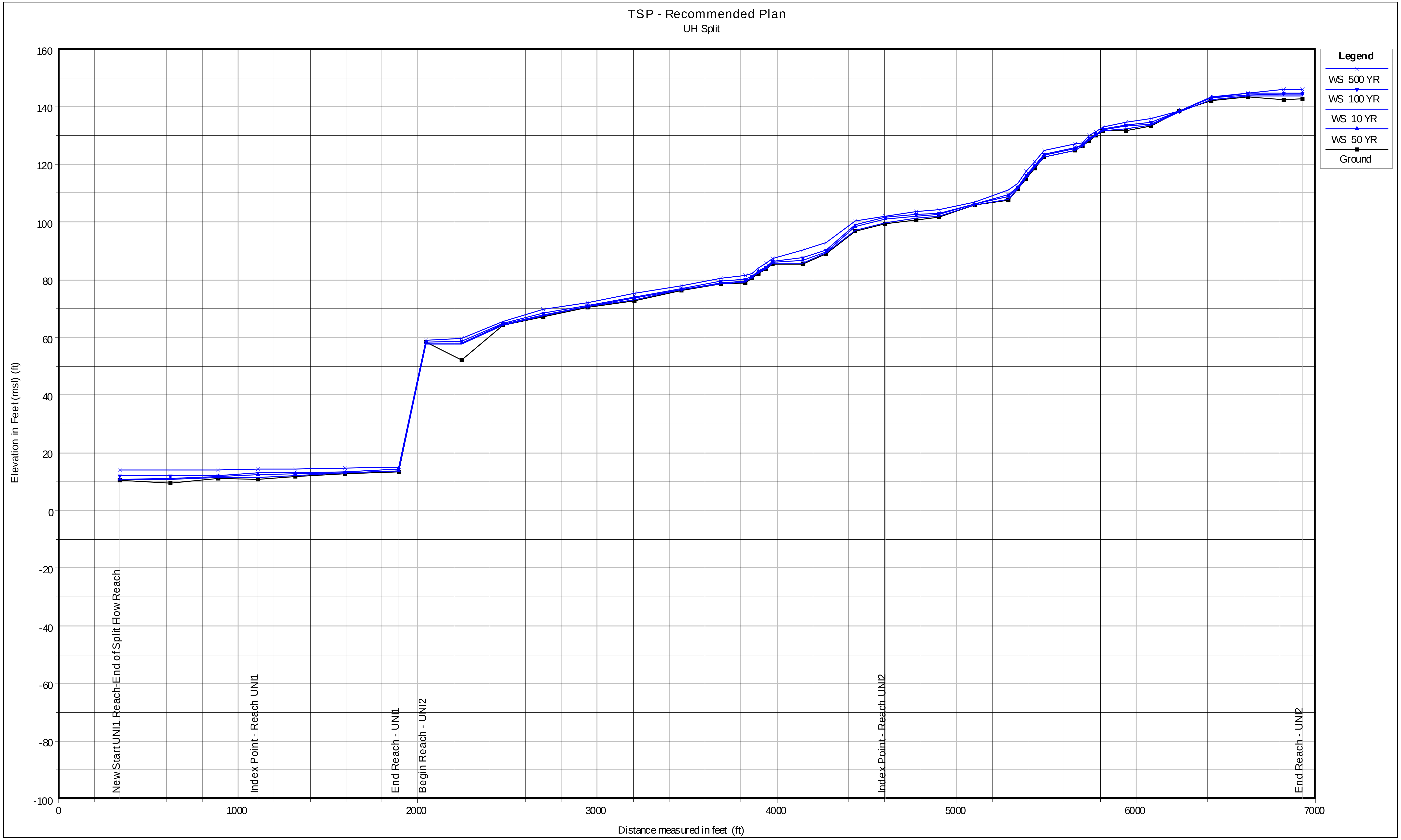

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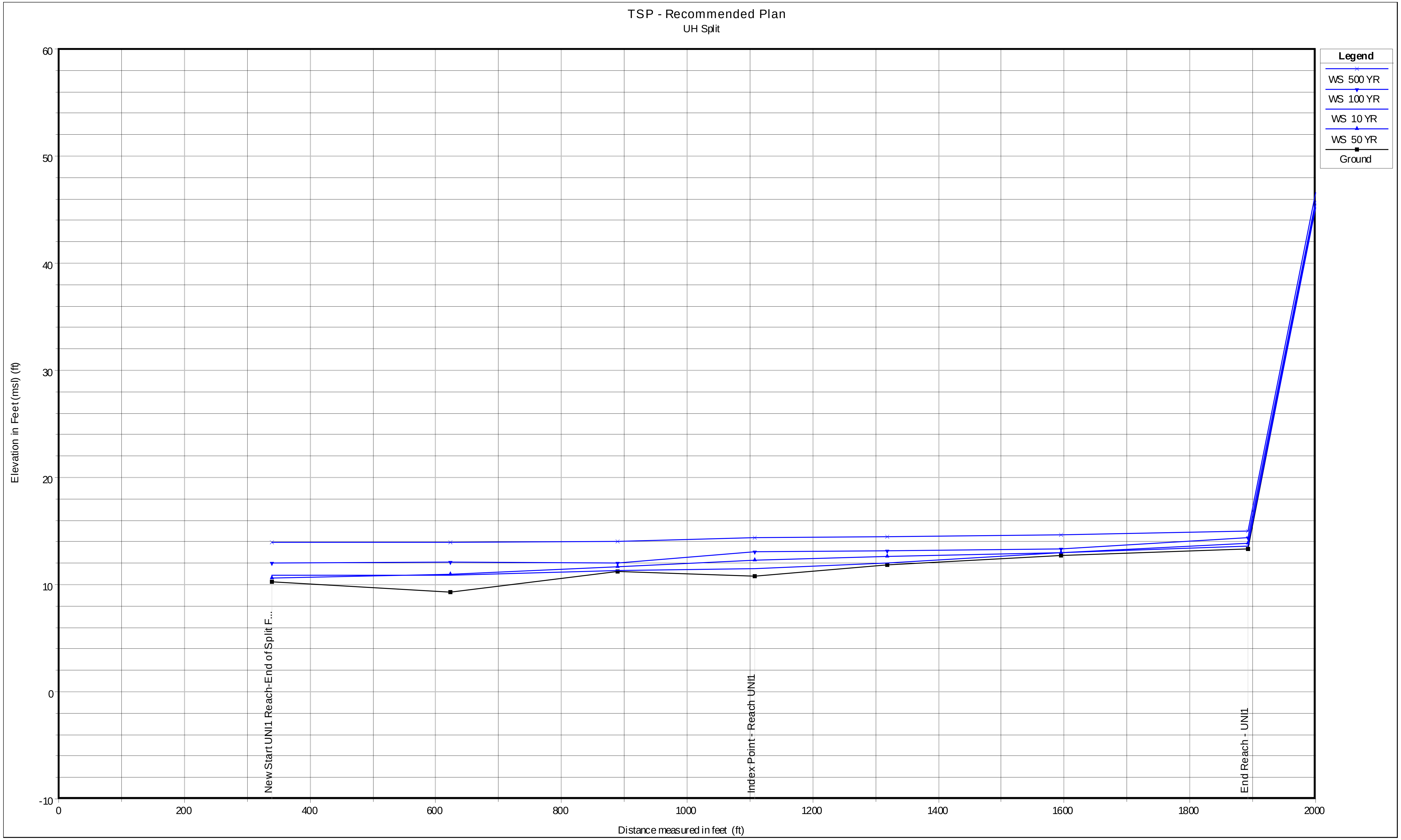

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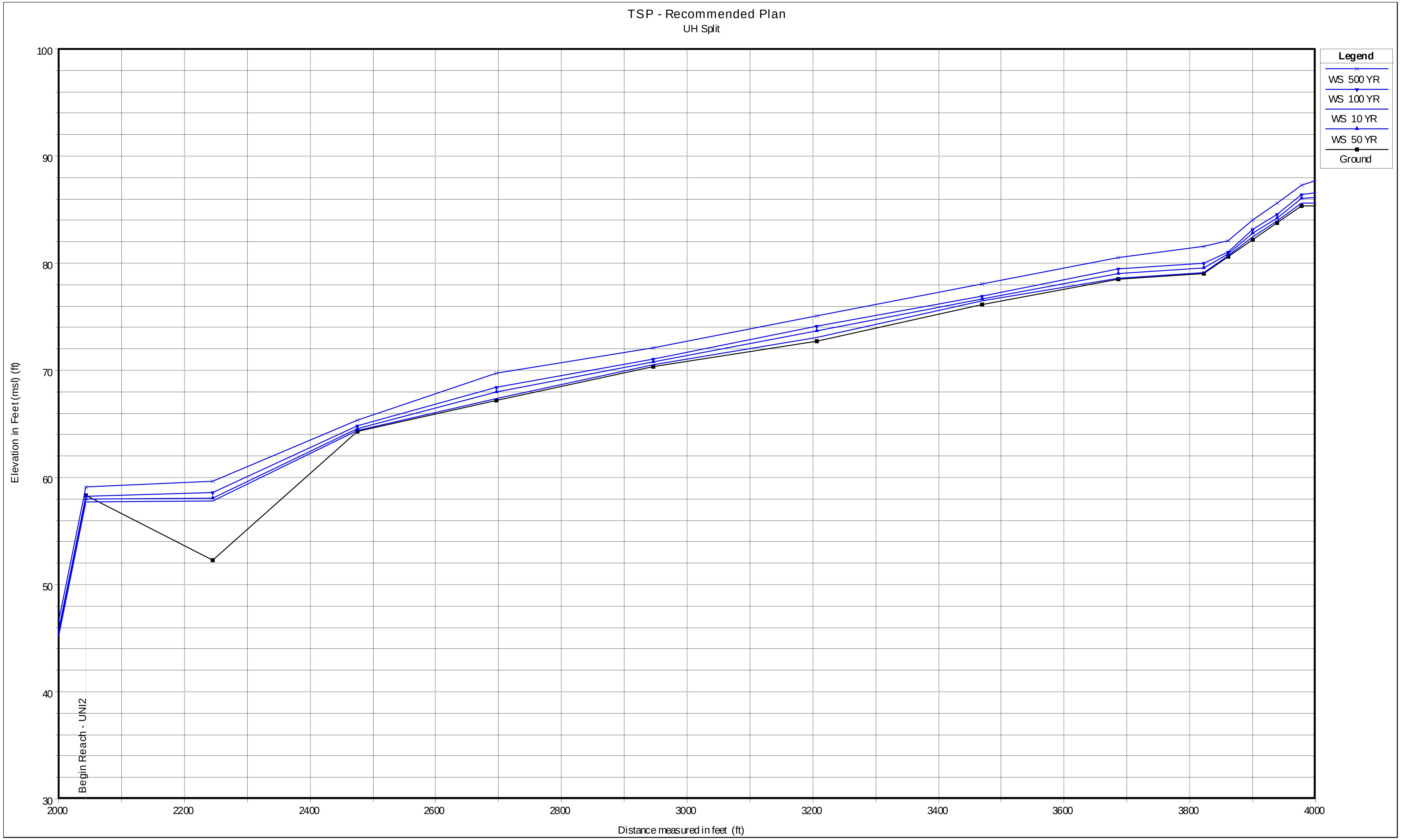

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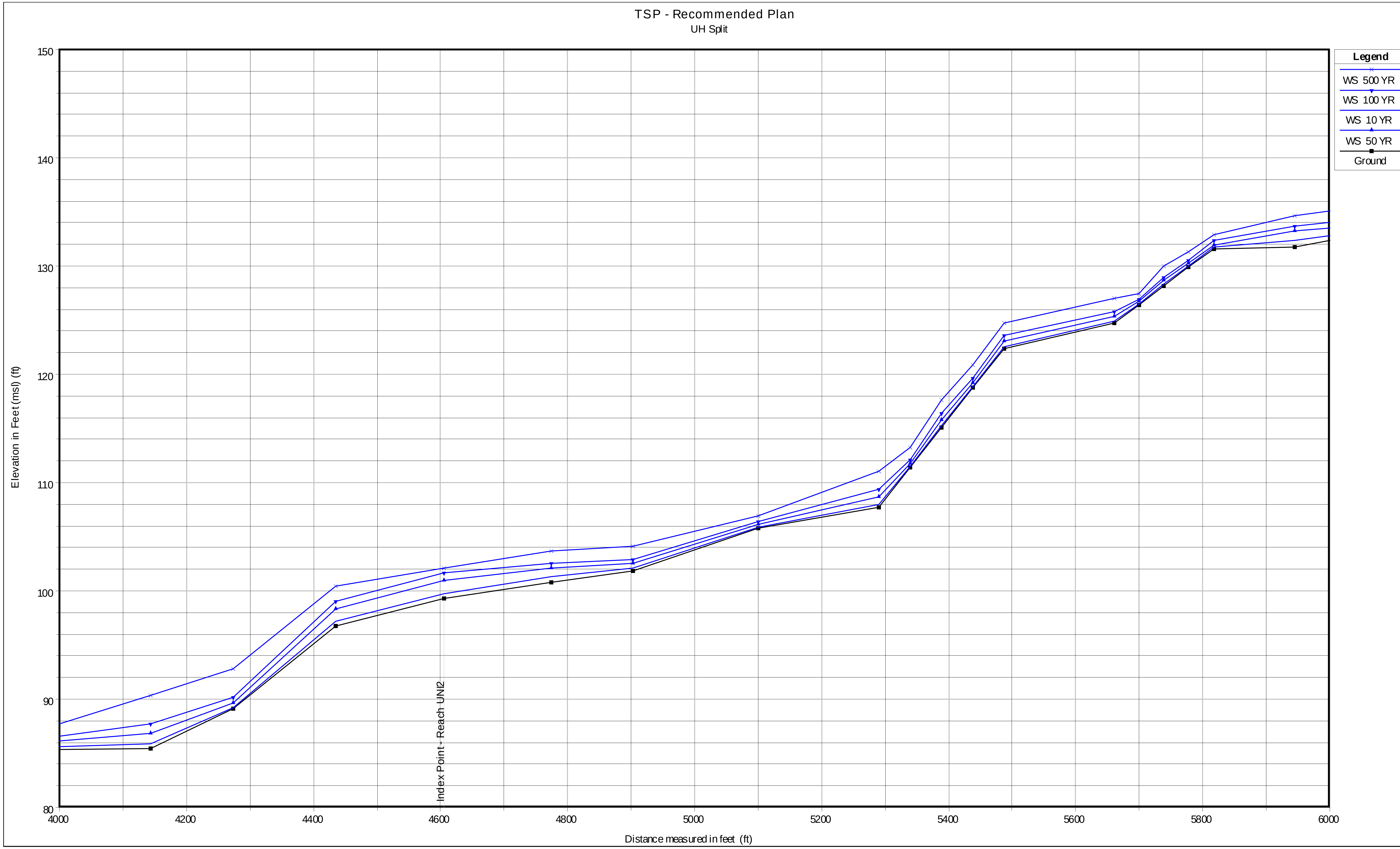

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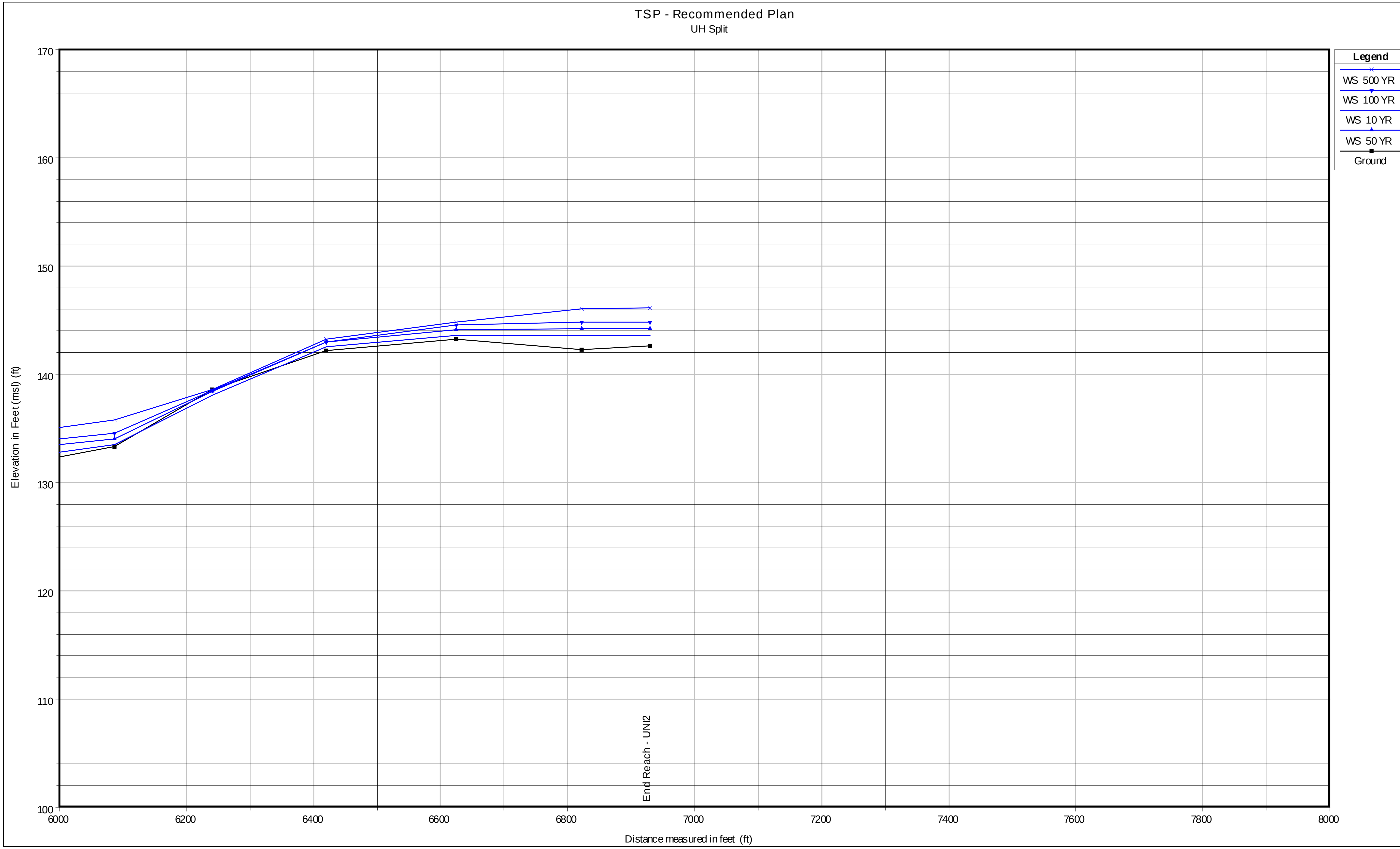

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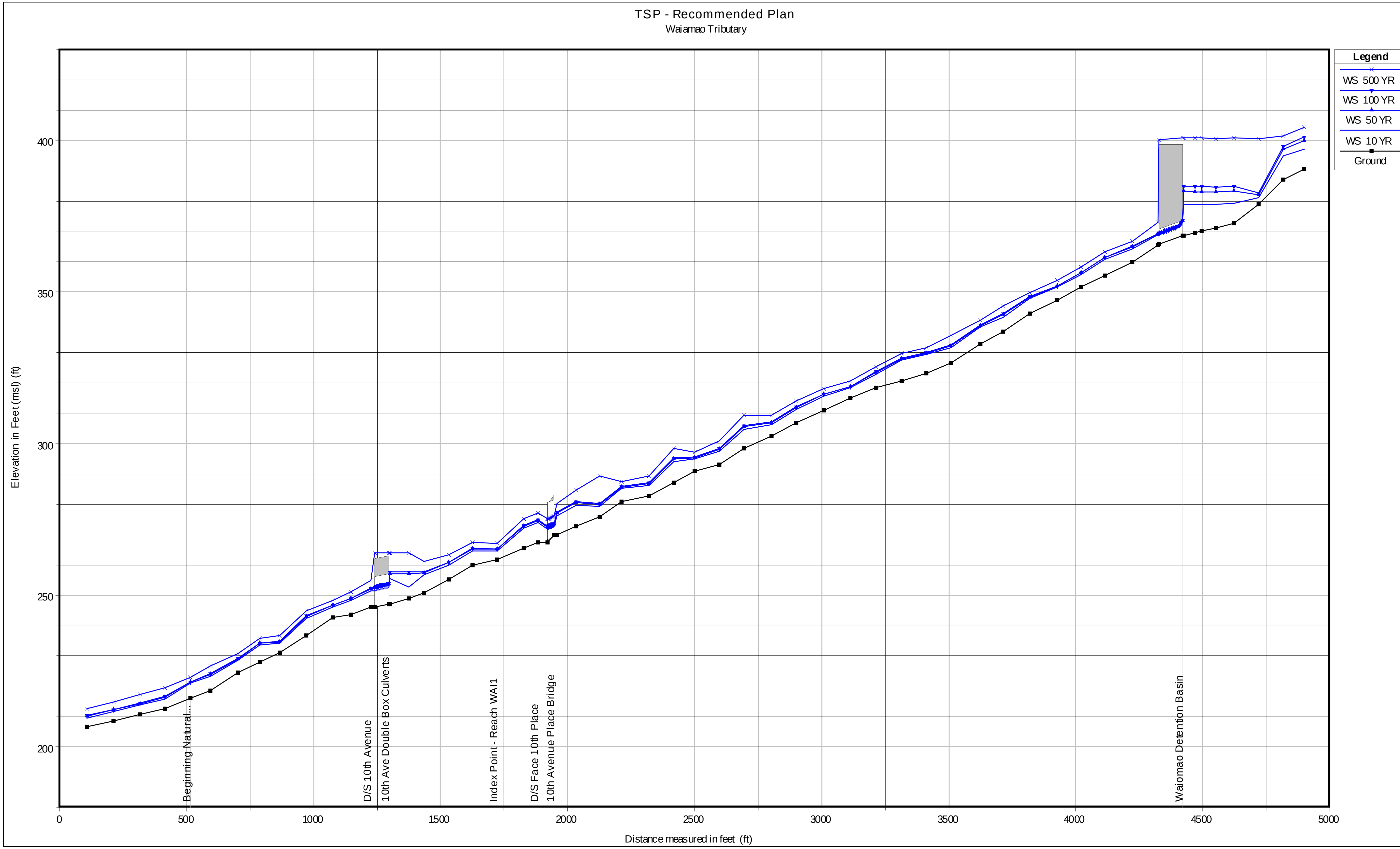

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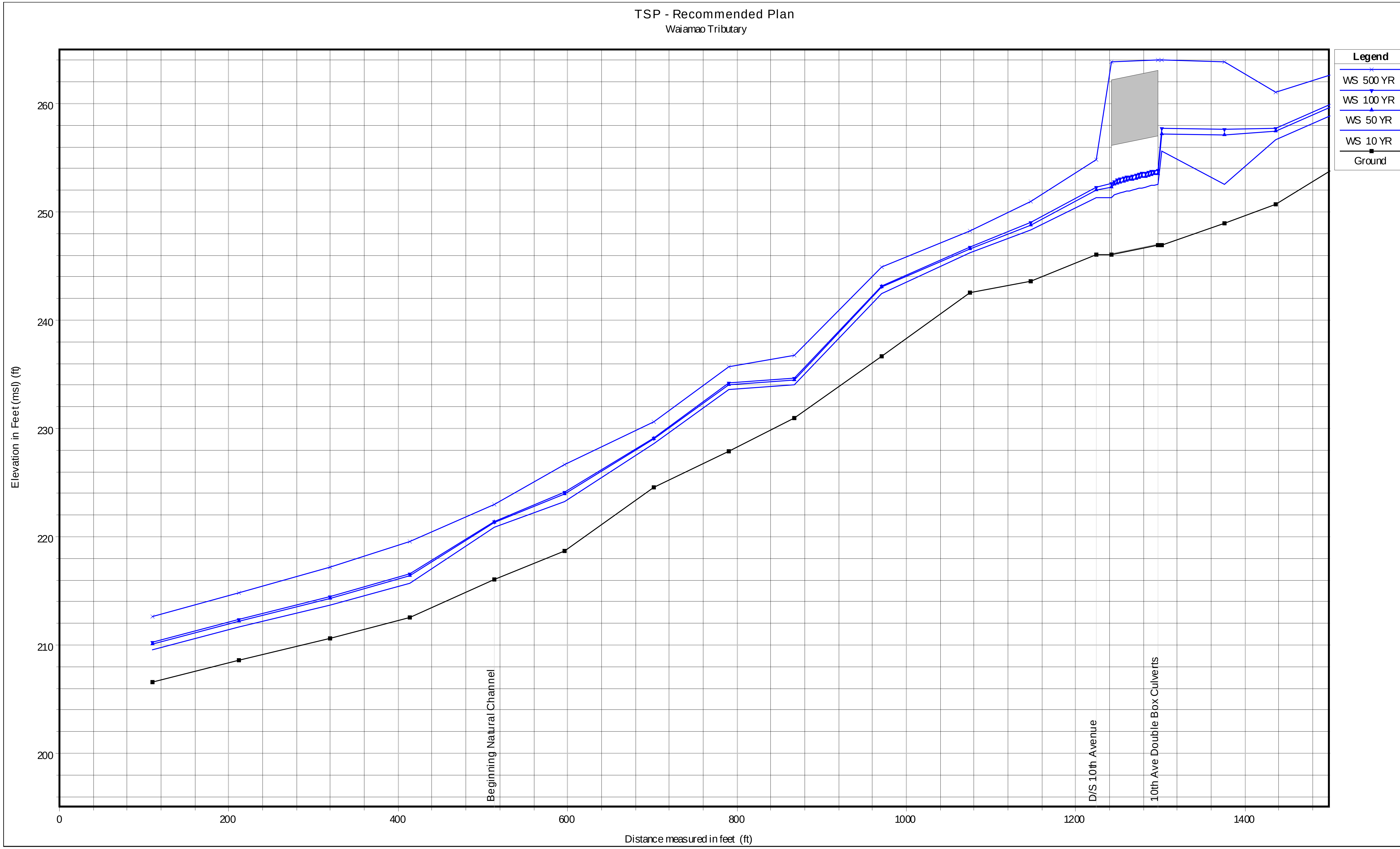

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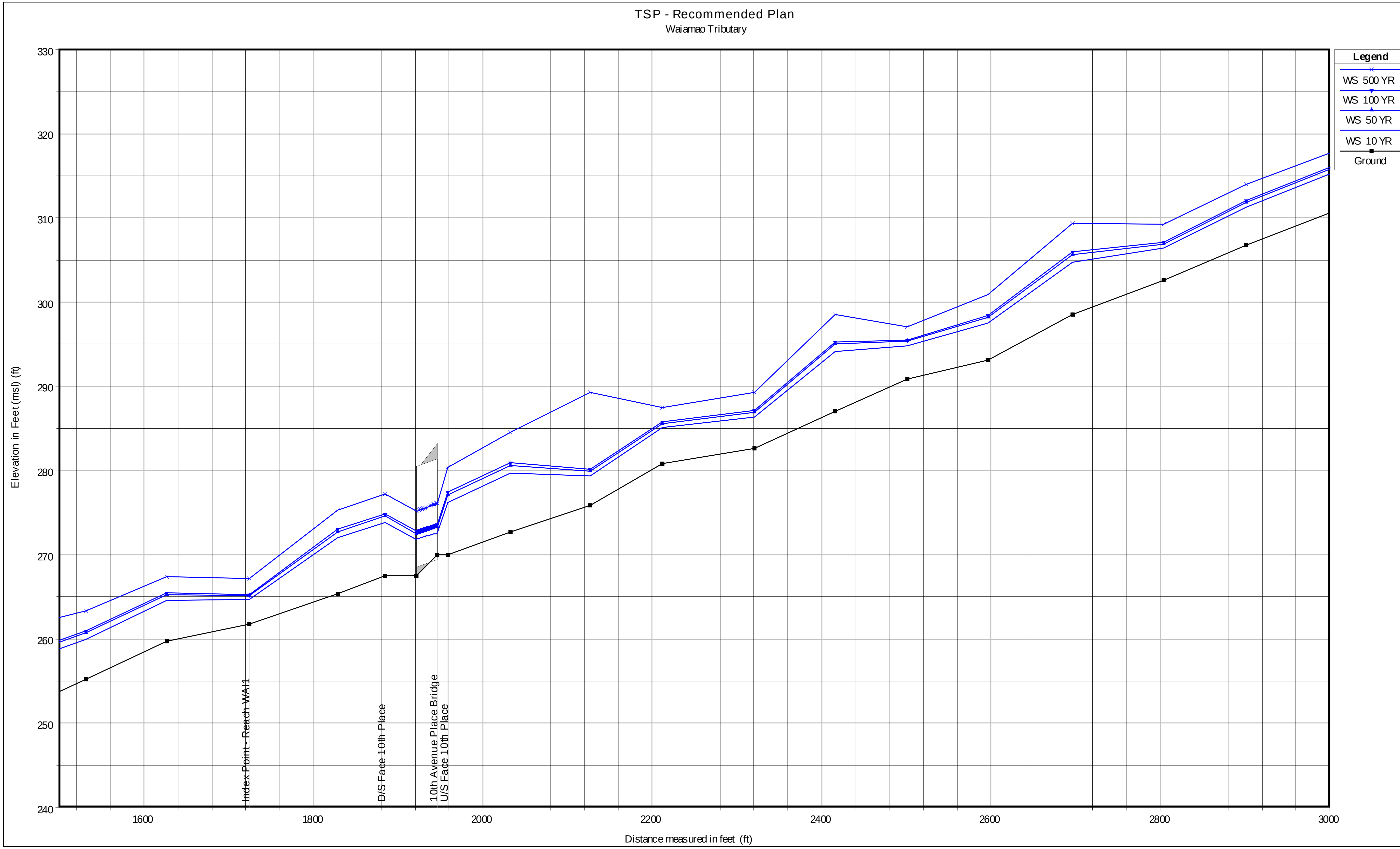

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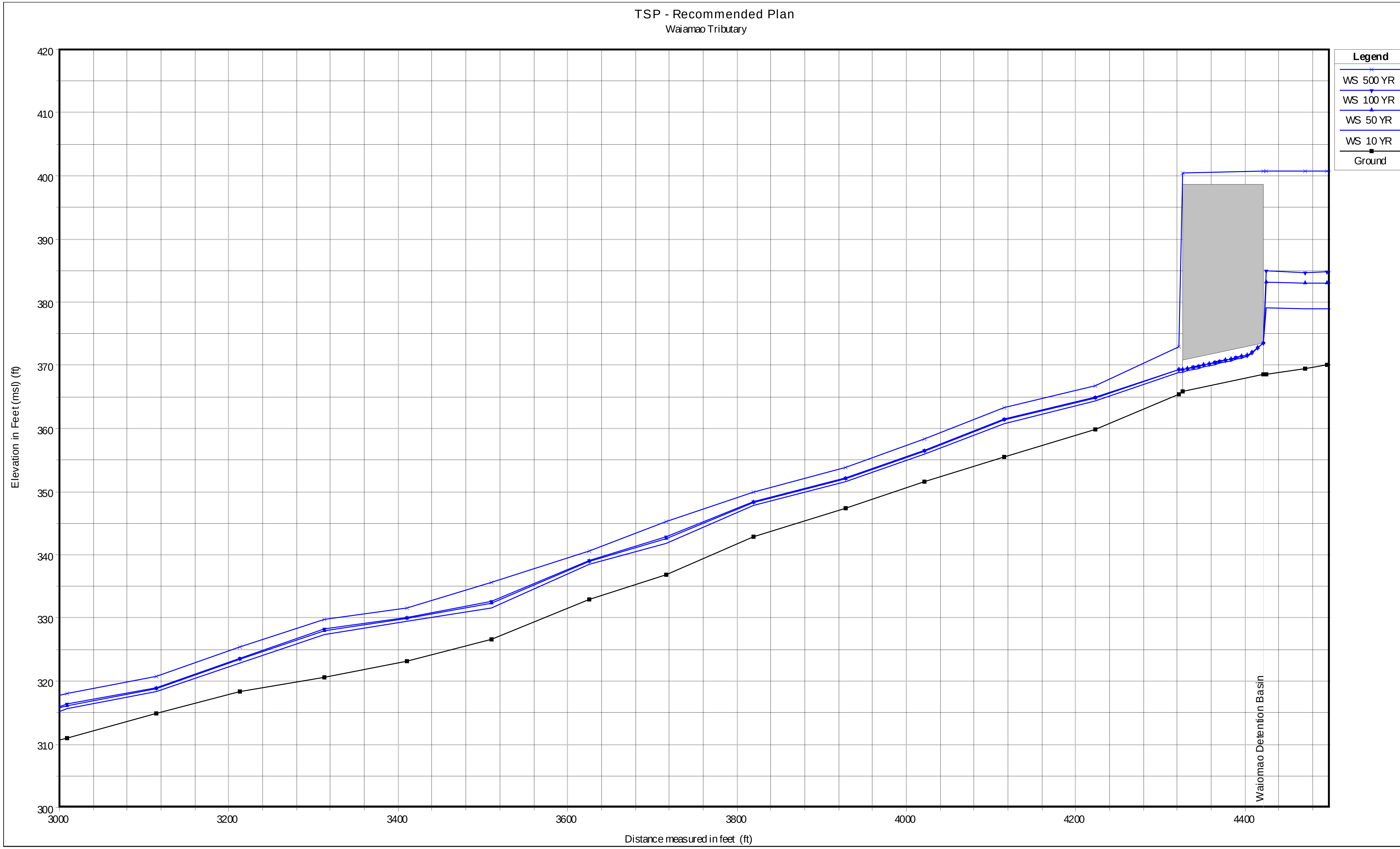

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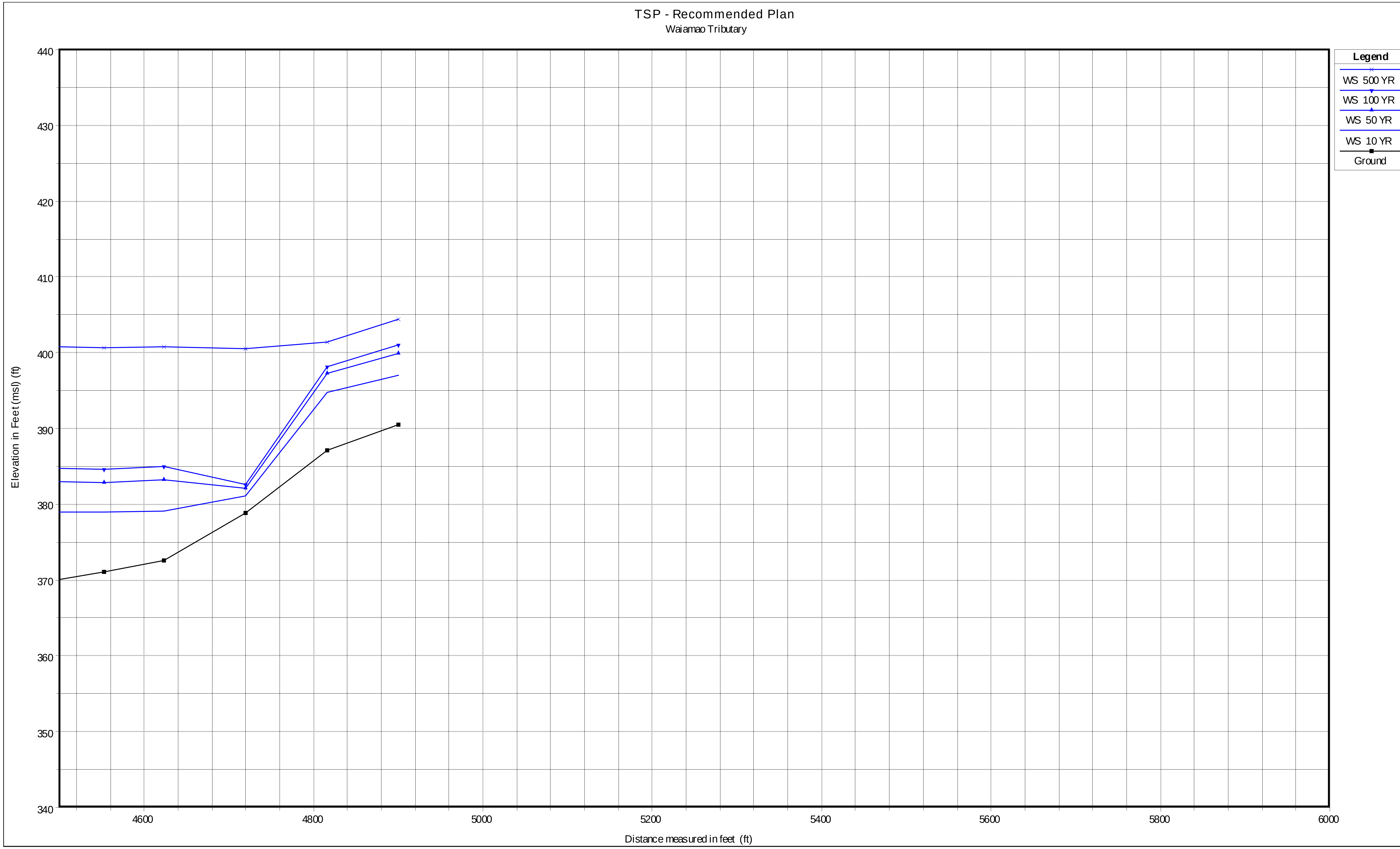

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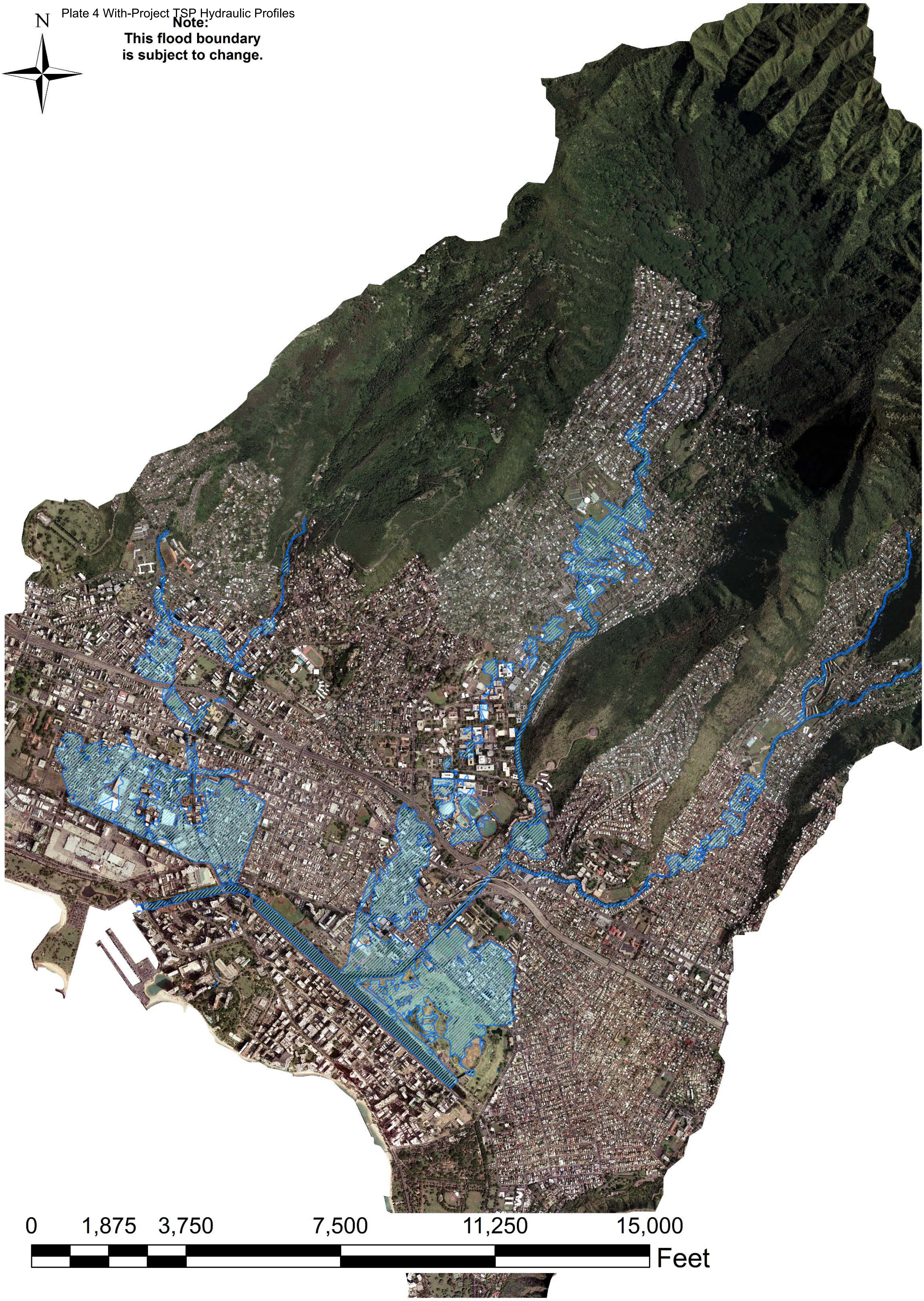

ALA WAI CANAL PROJECT - ALTERNATIVE 3A

1% ACE (100-Year) Floodplain 2075 Intermediate Scenario