US Army Corps of Engineers®
Honolulu District

Appendix A
Hydrology and Hydraulics

Tafuna, Tutuila, American Samoa

Draft Integrated Feasibility Report and Environmental Assessment

December 2021

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Appendix A

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1 Introduction

1.1 Study Area Description

American Samoa is the southernmost territory of the United States. It is comprised of five main islands and two coral atolls. The total land area is approximately 76 square miles. Tutuila is the largest of the volcanic islands, with a total land area of 53 square miles. It is approximately 18 miles long and varies in width from one to six miles. The mountainous terrain is characterized by steep slopes dipping to the sea, narrow valleys, and coastal plains. The mountains are of volcanic origin and have been eroded by rainfall and stream flow. The highest elevation is at Matafao Peak, which is 2,142 feet above mean sea level (MSL). A broad coral reef surrounds the island in several areas. A map of Tutuila is provided in Figure 1-1.

Tutuila's population of over 54,000 resides along the coastal areas and on the Tafuna-Leone Plain. The most populated coastal area borders Pago Pago Harbor, which is a central location for business activities, government facilities, and employment opportunities. Pago Pago Harbor, a natural embayment, nearly bisects the island and extends almost three miles inland. Pago Pago is the capital of American Samoa and is located in the northwest corner of Pago Pago Harbor. The study area for this analysis is specifically focused on the Tafuna plain, refer to Figure 1-1, which is comprised of many drainageways and streams. The Leaveave Drainageway is the focus area of this study. Refer to Figure 1-2 for the location map.

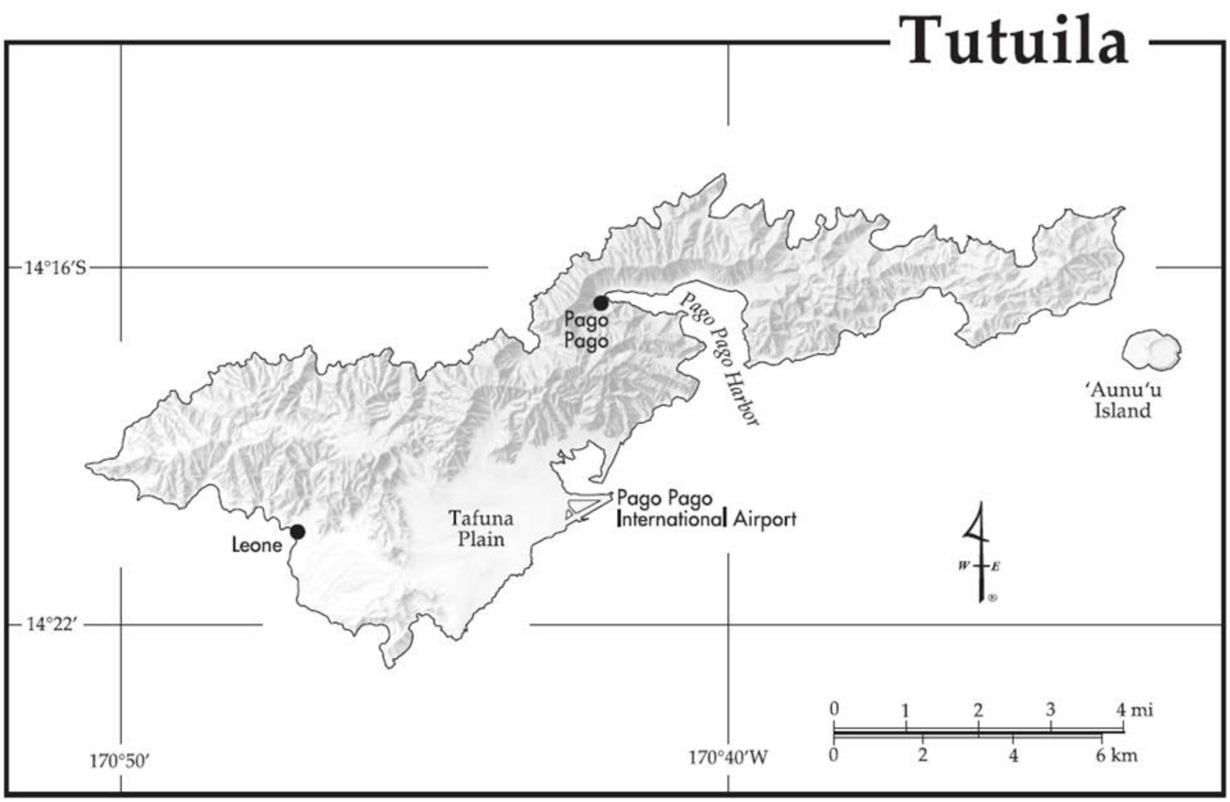

Figure 1-1. Study Area

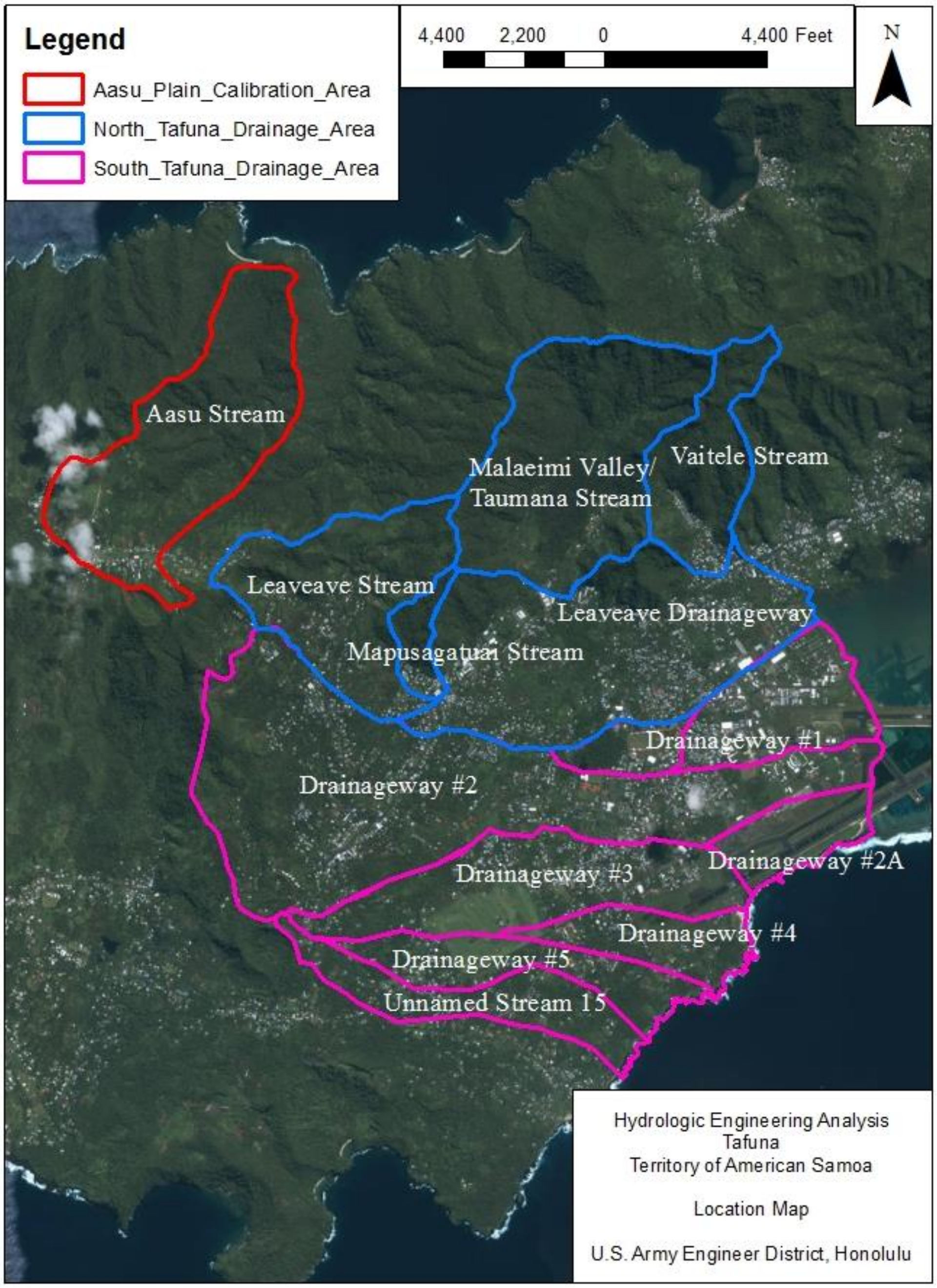

Figure 1-2. Location Map

Based on a review of previous reports and data, specifically those areas that meet the 800 cfs requirement, as outlined in ER-1165-2-21, will be analyzed as part of this study. Refer to Figure 1-3 below for the approximated 800 cfs area. Taumata Stream, Leaveave Stream and Vaitele Stream were the reaches that meet the 800 cfs requirement and the hydrologic and hydraulic models were updated for those reaches.

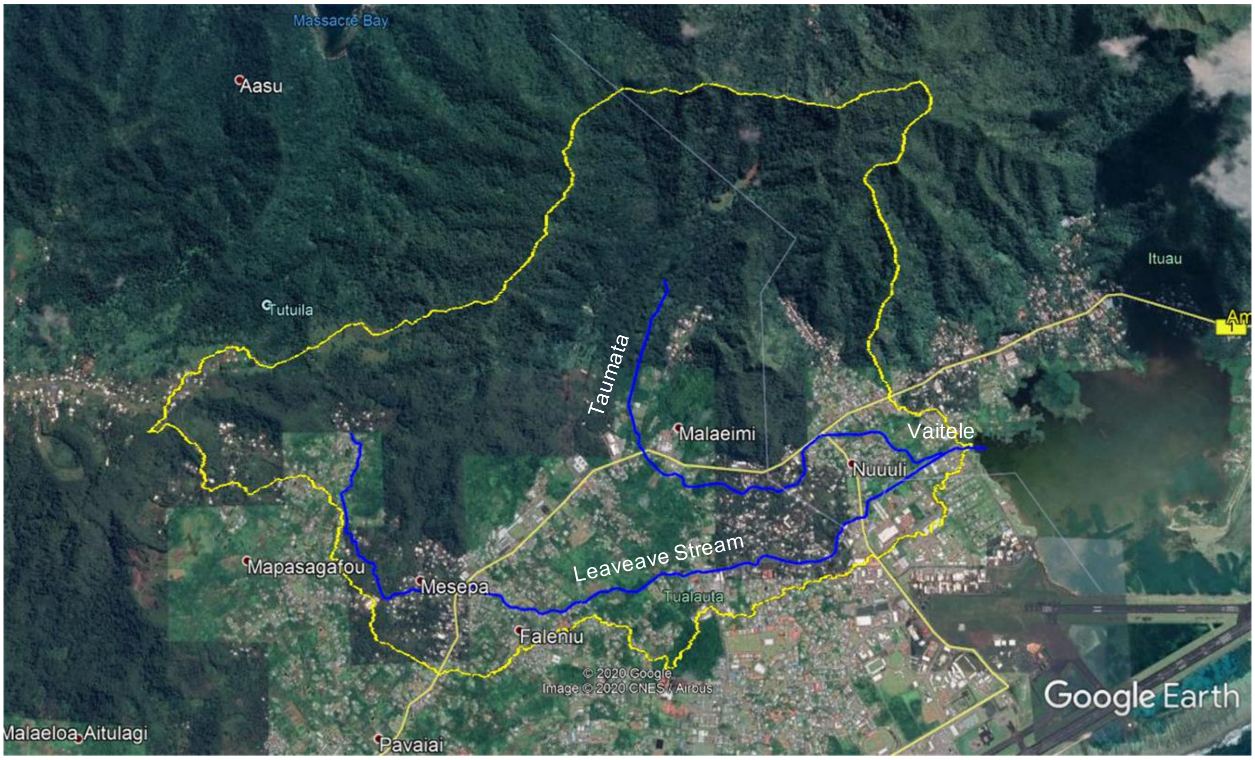

Figure 1-3. Approximated 800 CFS Area

1.2 Previous Reports

The existing hydrologic and hydraulic (H&H) conditions for the Tafuna floodplain were studied as part of the previously completed reports and studies.

Tafuna Plain Drainage Study, October 1994. U.S. Army Corps of Engineers, Honolulu District. This report was prepared under the Section 22 of the Water Resources Development Act of 1974. It was prepared in response to the American Samoa Government request for assistance in preparation of a drainage study for Tafuna Plain.

Hydrologic and Hydraulic Engineering Analysis Tafuna Study Area, 19 September 2016. U.S. Army Corps of Engineers, Honolulu District. This report summarized the methodology and results of the hydrologic and hydraulic analysis of Leaveave Stream and Drainageway 2 in Tutuila, American Samoa. The results were compared to the 2006 FIRM.

Hydrologic and Hydraulic Engineering Analysis Tafuna Study Area, 6 August 2019. U.S. Army Corps of Engineers, Honolulu District. This report summarized the methodology and results of the hydrologic and hydraulic analysis of Drainageway 4, 5 and Unnamed Stream 15 in Tutuila, American Samoa. The results were compared to the 2006 FIRM.

1.3 GIS Terrain Data and Layers

Several terrain models and data layers were used to perform the hydrologic and hydraulic analysis of the study area. The Geographical Information Systems (GIS) data, sources, and description are summarized in Table 1-1.

Table 1-1. Summary of Geospatial Data

Data	Description and Source
LIDAR	Light Detection and Ranging (LiDAR) data was collected by Photo Science, Inc. in June to July 2012. It includes the entire island of Tutuila. The project design of the LiDAR data acquisition was developed to support a nominal post spacing of 1.0 meter or better. The dataset was developed based on a horizontal projection/datum of UTM, Zone 2, NAD83 (PACP00), meters. The vertical datum used for the island of Tutila was NAVD88 (ASVD02), meters. The Root-Mean-Square Error vertical values were 0.067 meters.
TIN	The LiDAR data was converted to a Triangulated Irregular Network (TIN) by the USACE, Honolulu District. This data was used to create the cross-sections for the HEC-RAS hydraulic models. Unlike the LiDAR data which was projected in meters, the TIN was projected to UTM, Zone 2S, NAD83 (PACP00) feet. The z-factor is 3.280848.
DEM	A Digital Elevation Model (DEM) was created by converting the TIN to a raster. The cell size was 4 x 4 feet. This DEM was used to delineate the sub-basins and to prepare a number of hydrologic inputs using HEC-RAS Mapper.
Imagery	IKONOS imagery, purchased by NOAA between 2000 and 2003, was used for background mapping of the study area and floodplain. World Imagery, provided by ESRI, was used for larger scale background mapping, such as when it was necessary to show the entire island of Tutuila.
Soils	Polygon soils data were collected from the NRCS Soil Survey Geographic (SSURGO) database and used for determining the curve number for each sub-basin in the watershed.
Land Use	Spatial data pertaining to land use classification was provided by the U.S. National Oceanic and Atmospheric Administration (NOAA) Coastal Change Analysis Program. This was used to determine the curve number (CN) loss parameter for each subbasin and manning's roughness parameter values in the HEC-RAS 2D flow model.
Villages	A shapefile was used to identify which village a particular location was associated with. This shapefile was created and provided by the U.S. Census Bureau (2011).

2 Hydrology

The 2016 and 2019 hydrologic analysis from the previous reports listed in Section 1.2 was used as a starting point for the analysis. The watershed was reviewed for any changes in land use or other parameters and the analysis updated accordingly. The discharge-frequency relationships at key points in the study area were determined by developing rainfall-runoff models using the Hydrologic Engineering Center's Hydrologic Modeling System software (HEC-HMS).

HEC-HMS was used to perform the rainfall-runoff computations for the 50%, 20%, 10%, 5%, 2%, 1%, 0.5%, and 0.2% AEP (2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year) flood frequency events. Precipitation data was taken from NOAA's Atlas 14 Precipitation-Frequency Data Server. The loss method used was the SCS curve number, which was determined by taking the weighted average of each sub-basin based on its soil and cover conditions. The transform method was Clark's unit hydrograph method. Muskingum-Cunge routing was selected for the routing reach method. This method lends itself to circumstances where limited observed data is available and can be used in reaches that have a small slope.

2.1 Previous Modeling Results

The 2016 hydrology was determined using Arc Hydro Tools and the Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) software. It was used to delineate the watershed and sub-basins, define major streams within the study area, and determine the longest flow path within each sub-basin.

There are three main components of an HEC-HMS model: basin model, meteorologic model, and control specifications. The basin model, shown in **Figure 2-1. HEC-HMS Basin Model for Leaveave Draingeway**Figure 2-1, contains the physical description of the watershed. Hydrologic elements (sub-basins, reaches, sources, sinks, reservoirs, and junctions) are connected to one another to define the physical representation of the real world watershed. The hydrologic elements also require parameter information, such as impervious area, in order for the program to compute the rainfall-runoff response in the watershed. The meteorologic model calculates the precipitation input needed by sub-basin elements in the basin model. The control specification defines the time period and time step required for simulations.

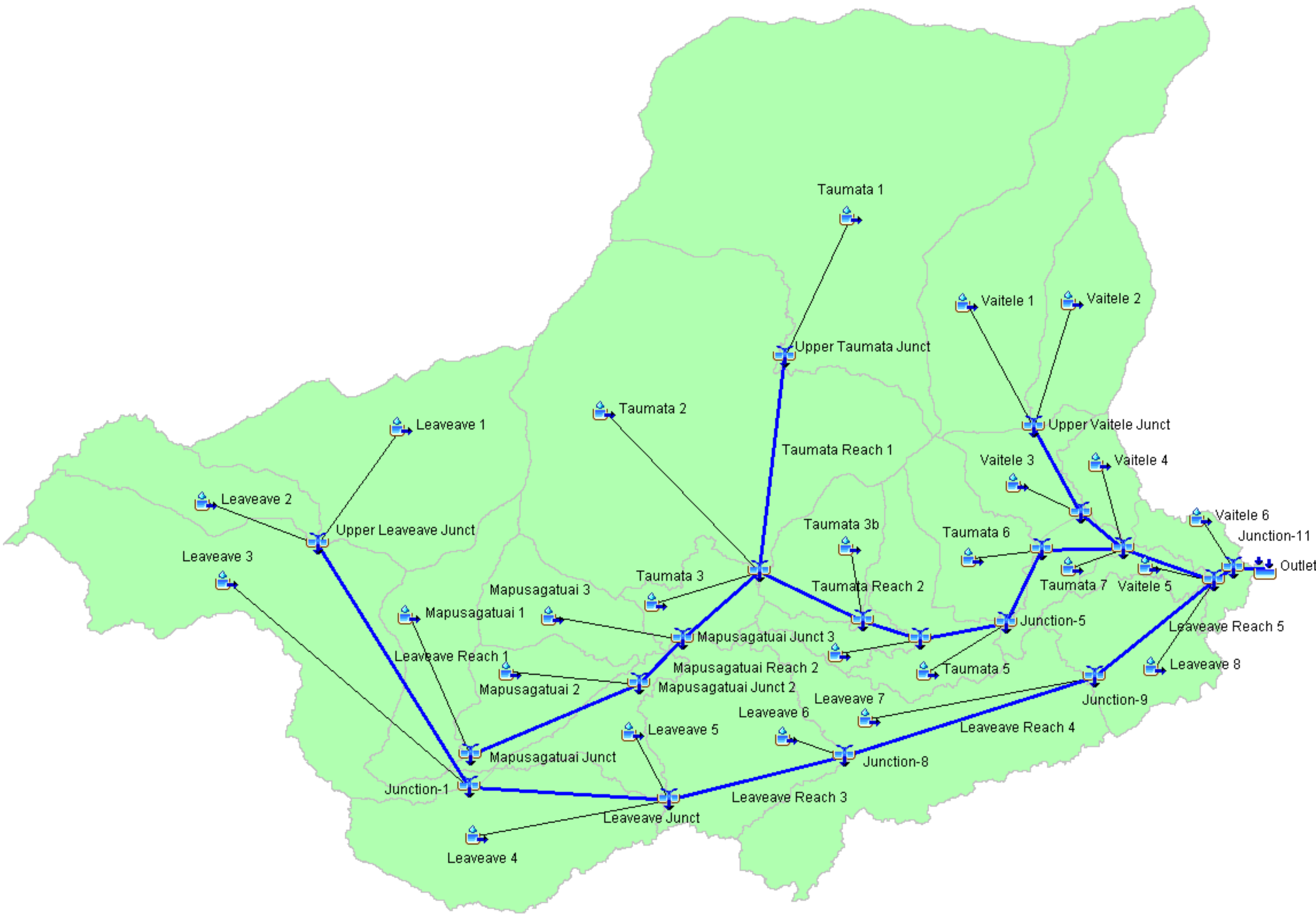

Figure 2-1. HEC-HMS Basin Model for Leaveave Draingeway

HEC-HMS contains many methods for simulating the rainfall-runoff response in a watershed. Modeling methods were chosen based on data availability and appropriateness for the project area. Table 2-1 contains the modeling methods chosen and a list of the required parameters. These methods were chosen based on data availability and the appropriateness for the project area.

Table 2-1. HEC-HMS Modeling Methods and Required Parameters

Modeling Method	Parameter	Description
SCS Curve Number Loss Method	Curve Number	A composite number based on the area's hydrologic soil group, land use, treatment and hydrologic condition.
	Impervious (%)	Impervious area directly connected to the channel network
Clark Unit Hydrograph Transform Method	Time of Concentration	Travel time from the most hydrological remote point in the sub-basin to the watershed outlet
	Storage Coefficient (hr)	Accounts for storage in the watershed
Muskingum-Cunge Routing Method	Length (ft)	Total length of the channel reach
	Slope (ft/ft)	Average slope of the channel reach
	Manning's n	Representative channel roughness
	Invert	An optional parameter used as a reference to compute the stage elevation.
	Shape	Five options are provided for specifying the cross-section shape; the channel diameter, bottom width, and side slopes may also be required depending on the shape chosen.

The following list provides an overview of the steps followed for determining the peak flow hydrographs at key points in the study area for various frequency events. These steps are discussed in more detail in the subsequent sections.

1. Sub-Basin Delineation
2. Sub-Basin Loss and Transform Parameter Estimation
3. Reach Routing and Loss Parameter Estimation
4. Development of the Meteorologic Model
5. Simulation of the Frequency Storm Events in HEC-HMS

The HEC-HMS model was used to simulate various storm events. The resulting peak discharges at each sub-basin within the Leaveave Drainageway are presented in Table 2-2.

Table 2-2. 2016 Computed Flow Discharges at Sub-Basins in the Leaveave Drainageway

Sub-Basin Element	Peak Flow Discharges (cfs)							
	50% ACE	20% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
Leaveave 1	209	329	426	568	682	801	924	1,100
Leaveave 2	30.0	57.7	82.3	119	149	181	214	261
Leaveave 3	178	290	379	508	612	724	840	1,000
Leaveave 4	100	148	187	242	286	332	379	445
Leaveave 5	50.8	73.1	90.6	115	135	155	176	205
Leaveave 6	27.7	51.8	73.0	105	132	161	191	235
Leaveave 7	53.2	88.4	118	163	198	237	277	334
Leaveave 8	106	153	190	244	286	329	374	435
Mapusagatuai 1	107	162	205	266	314	366	420	494
Mapusagatuai 2	102	146	180	227	264	303	342	396
Mapusagatuai 3	63.5	100	129	172	206	243	280	332
Taumata 1	296	523	709	981	1,210	1,450	1,700	2,050
Taumata 2	191	356	497	709	883	1,070	1,260	1,540
Taumata 3	77.1	105	127	157	181	205	230	264
Taumata 3b	48.0	72.3	91.8	120	143	166	190	224
Taumata 4	8.8	16.4	23.0	32.9	41.0	49.7	58.8	71.6
Taumata 5	52.6	83.9	109	145	174	205	237	282
Taumata 6	78.6	122	156	205	244	284	326	385
Taumata 7	37.7	53.1	65.3	82.1	95.5	110	124	144
Vaitele 1	200	317	410	544	650	762	879	1,040
Vaitele 2	90.5	143	185	245	293	345	398	473
Vaitele 3	40.5	65.9	86.2	115	139	163	188	224
Vaitele 4	36.2	52.4	65.5	83.8	98.3	113	129	150
Vaitele 5	10.7	16.9	22.0	29.2	34.9	40.9	47.2	55.9
Vaitele 6	33.7	47.2	58.1	73.2	85.0	97.3	110	127

2.2 Basin Characteristics

GIS data was used to delineate the basins, subbasins, and rivers. Each basin was divided into subbasins based on key locations in the watershed (e.g. the location of a stream flow gage, junction, or existing detention basin). Drainage areas and centroid locations of each subbasin is provided in Table 2-3.

Table 2-3. Subbasin Identification and Information

Subbasin Name	Drainage area (mi²)	Centroid location	
		Latitude	Longitude
Leaveave 1	0.26	-14.3136	-170.7486
Leaveave 2	0.09	-14.3145	-170.7563
Leaveave 3	0.32	-14.3192	-170.7539
Leaveave 4	0.19	-14.3271	-170.7446
Leaveave 5	0.06	-14.3235	-170.7407
Leaveave 6	0.16	-14.3237	-170.7364
Leaveave 7	0.26	-14.3239	-170.7296
Leaveave 8	0.11	-14.3204	-170.7215
Mapusagatuai 1	0.12	-14.3206	-170.7471
Mapusagatuai 2	0.08	-14.3223	-170.7439
Mapusagatuai 3	0.08	-14.3191	-170.7427
Taumata 1	0.63	-14.3033	-170.7324
Taumata 2	0.57	-14.3123	-170.7380
Taumata 3	0.05	-14.3188	-170.7374
Taumata 3b	0.08	-14.3171	-170.7328
Taumata 4	0.03	-14.3200	-170.7324
Taumata 5	0.09	-14.3192	-170.7294
Taumata 6	0.09	-14.3166	-170.7277
Taumata 7	0.04	-14.3178	-170.7243
Vaitele 1	0.27	-14.3053	-170.7251
Vaitele 2	0.12	-14.3083	-170.7231
Vaitele 3	0.06	-14.3142	-170.7256
Vaitele 4	0.04	-14.3144	-170.7227
Vaitele 5	0.02	-14.3177	-170.7211
Vaitele 6	0.06	-14.3172	-170.7192

In some instances, it may be necessary to make manual adjustments to the boundaries and stream to reflect the terrain conditions more appropriately. The Leaveave Drainageway, as identified in previous studies, was divided into 25 sub-basins, refer to Figure 2-1. The previous model will be revised to focus on those areas that meet the 800 cfs rule.

2.3 **Model Parameters**

2.3.1 Initial Estimation for Loss Parameters

The SCS Curve Number method was used in the model to account for precipitation loss due to infiltration. This method uses three parameters: initial abstraction, the Curve Number (CN) and percent impervious area. The initial abstraction, which defines the amount of precipitation that must fall before surface runoff occurs, was left blank to allow HEC-HMS to automatically calculate these values using the default ratio of 0.2.

The curve number was found by using information from Table 2-2 of Urban Hydrology for Small Watersheds (USDA 1986), which is replicated in Figure 2-2 and Figure 2-3. The percentage of each category within a basin was estimated with GIS, while the hydrologic group determinations were made using the United States Department of Agriculture's Web Soil Survey [website: http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx].

A weighted average CN was determined for each sub-basin, based on its soil and cover conditions. Soil data was obtained from the Natural Resources and Conservation Service (NRCS)'s Web Soil Survey (WSS), an online tool that provides basic soil information in an area of interest. A map was created showing the different hydrologic soil groups (HSG) within the watershed and the percent areal weighted distribution for each sub-basin. This was used in unison with the areal weighted distribution for the representative land cover to determine the overall weighted CN for each sub-basin, which is presented in Table 2-4.

Table 2-2a Runoff curve numbers for urban areas 1/

Cover description: Cover type and hydrologic condition	Average percent impervious area 2/	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) 3/:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) 4/		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) 5/		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.
2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Figure 2-2. TR-55 Runoff Curve Numbers for Urban Areas

Table 2-2b Runoff curve numbers for cultivated agricultural lands [1]

Cover description			Curve numbers for hydrologic soil group			
Cover type	Treatment [2]	Hydrologic condition [3]	A	B	C	D
Fallow	Bare soil	—	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T+ CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T+ CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

[1] Average runoff condition, and I_a=0.2S
[2] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
[3] Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Figure 2-3. TR-55 Runoff Curve Numbers for Cultivated Agricultural Lands

Table 2-4. Final Computed HEC-HMS Model Basin Parameters

Subbasin Name	Drainage area (mi^2)	Curve Number
Leaveave 1	0.26	56
Leaveave 2	0.09	47
Leaveave 3	0.32	55
Leaveave 4	0.19	63
Leaveave 5	0.06	65
Leaveave 6	0.16	48
Leaveave 7	0.26	55
Leaveave 8	0.11	64
Mapusagatuai 1	0.12	60
Mapusagatuai 2	0.08	66
Mapusagatuai 3	0.08	56
Taumata 1	0.63	51
Taumata 2	0.57	47
Taumata 3	0.05	71
Taumata 3b	0.08	60
Taumata 4	0.03	47
Taumata 5	0.09	56
Taumata 6	0.09	57
Taumata 7	0.04	67
Vaitele 1	0.27	56
Vaitele 2	0.12	56
Vaitele 3	0.06	55
Vaitele 4	0.04	64
Vaitele 5	0.02	57

The distribution of land use classification is shown in Figure 2-4. As the impervious areas were already included in the curve number calculations, the second parameter, “percent impervious area,” will be set to zero. This is recommended in the HEC-HMS Technical Reference Manual and was consistent with the previous studies in the area.

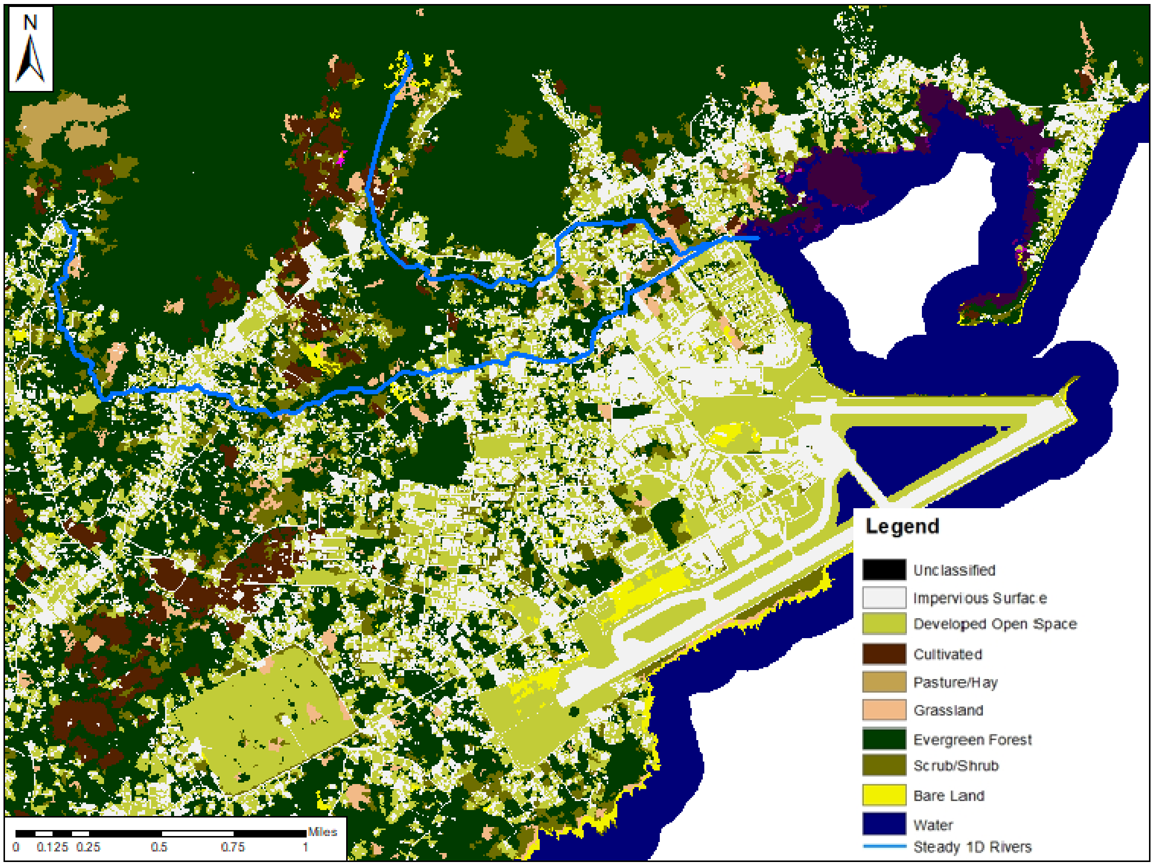

Figure 2-4. Land Cover Distribution in the Tafuna Area

2.3.2 Initial Estimation for Transform Parameters

The excess precipitation in each sub-basin was transformed into surface runoff by applying the Clark Unit Hydrograph method in the hydrologic model. This method requires two input parameters for each sub-basin: the time of concentration (t_c) and the storage coefficient (R).

The time of concentration, or the time it takes for runoff to travel from the most distant point in the watershed to the outlet, was calculated in accordance with the NRCS's Technical Release 55 (TR- 55) methodology (1986). The TR-55 method breaks the surface flow in the watershed into three flow regimes. As water travels along the longest flow path in the sub-basin, it is transformed from sheet flow (Table 2-5), to shallow concentrated flow (Table 2-6), to open channel flow (Table 2-7). Not all the flow types will be represented in each sub-basin. The time of concentration of a watershed is calculated by summing the travel time of flow through each of these flow regimes.

Table 2-5. Sheet Flow Characteristics for each Sub-Basin

Sub-Basin Name	Manning's *n* Overland	Sheet Flow Length (ft)	2-yr, 24-hr Rainfall (in)	Land Slope (ft/ft)	Tc, sheet (hrs)
Leaveave 1	0.6	250	8.22	0.29	0.22
Leaveave 2	0.6	225	8.22	0.05	0.40
Leaveave 3	0.6	189	8.22	0.60	0.13
Mapusagatuai 1	0.6	265	8.22	0.41	0.20
Mapusagatuai 2	0.6	146	8.22	0.73	0.10
Mapusagatuai 3	0.6	166	8.22	0.53	0.13
Taumata 1	0.6	272	8.22	0.71	0.17
Taumata 2	0.6	192	8.22	0.52	0.14
Taumata 3b	0.6	277	8.22	0.64	0.17
Taumata 5	0.6	215	8.22	0.73	0.14
Taumata 6	0.6	247	8.22	0.80	0.15
Vaitele 1	0.6	112	8.22	0.61	0.09
Vaitele 2	0.6	293	8.22	0.41	0.00
Vaitele 3	0.6	286	8.22	1.02	0.15
Vaitele 4	0.6	276	8.22	0.60	0.18

Table 2-6. Shallow Concentrated Flow Characteristics for each Sub-Basin

Sub-Basin Name	Surface Type	Flow Length (ft)	Watercourse Slope (ft/ft)	Average Velocity (ft/s)	Tc, shallow (hrs)
Leaveave 1	Unpaved	4910	0.11	5.43	0.25
Leaveave 2	Unpaved	4831	0.15	6.14	0.22
Leaveave 3	Unpaved	3843	0.14	6.13	0.17
Leaveave 4	Unpaved	6051	0.05	3.68	0.46
Leaveave 5	Unpaved	3219	0.04	3.05	0.29
Leaveave 6	Unpaved	5031	0.01	1.85	0.76
Leaveave 7	Unpaved	7510	0.01	1.88	1.11
Leaveave 8	Unpaved	1096	0.04	3.16	0.10
Mapusagatuai 1	Unpaved	3689	0.19	7.02	0.15
Mapusagatuai 2	Unpaved	744	0.26	8.30	0.02
Mapusagatuai 3	Unpaved	2947	0.24	7.93	0.10
Taumata 1	Unpaved	7184	0.16	6.45	0.31
Taumata 2	Unpaved	3841	0.28	8.50	0.13
Taumata 3	Unpaved	1532	0.01	1.90	0.22
Taumata 3b	Unpaved	3811	0.15	6.20	0.17
Taumata 4	Unpaved	2405	0.02	2.45	0.27
Taumata 5	Unpaved	2225	0.37	9.81	0.06
Taumata 6	Unpaved	1186	0.27	8.32	0.04
Taumata 7	Unpaved	2343	0.02	2.29	0.28
Vaitele 1	Unpaved	6801	0.20	7.24	0.26
Vaitele 2	Unpaved	3289	0.30	8.82	0.10
Vaitele 3	Unpaved	1259	0.69	13.39	0.03
Vaitele 4	Unpaved	1792	0.16	6.38	0.08
Vaitele 5	Unpaved	1207	0.02	2.35	0.14
Vaitele 6	Unpaved	2399	0.01	1.88	0.36

Table 2-7. Channel Flow Characteristics for each Sub-Basin

Sub-Basin Name	Cross Sectional Flow Area (ft^2)	Wetted Perimeter (ft)	Hydraulic Radius (ft)	Channel Slope (ft/ft)	Manning's *n* Channel	Velocity (ft/s)	Flow Length (ft)	Tc, channel (hrs)
Leaveave 2	112.0	45.0	2.5	0.05	0.08	7.22	2097	0.08
Leaveave 3	128.0	32.8	3.9	0.06	0.07	13.98	4554	0.09
Leaveave 7	23.7	13.9	1.7	0.01	0.08	2.51	626	0.07
Leaveave 8	18.5	20.5	0.9	0.01	0.08	1.91	1612	0.23
Mapusagatuai 1	42.5	18.0	2.4	0.07	0.07	9.69	1002	0.03
Taumata 1	18.6	12.3	1.5	0.03	0.06	5.75	2173	0.11
Taumata 4	473.9	87.9	5.4	0.004	0.08	3.41	2408	0.20
Taumata 5	180.0	66.0	2.7	0.01	0.06	3.41	1192	0.10
Taumata 7	266.0	52.0	5.1	0.02	0.06	11.61	1070	0.03
Vaitele 1	117.5	37.9	3.1	0.08	0.06	14.25	2195	0.04
Vaitele 2	80.0	28.4	2.8	0.09	0.06	14.00	1297	0.03
Vaitele 4	132.5	40.9	3.2	0.02	0.07	5.69	962	0.05
Vaitele 6	18.0	12.7	1.4	0.02	0.06	3.91	1273	0.09

GIS was used to determine the longest flow path, slope, and flow length of each sub-basin. Representative channel cross-sections that were primarily determined based on field observations made in 2015, field photographs included in the 1994 report (POD), and the TIN generated from LiDAR measurements taken in 2012 was used. Additional data required for the TR-55 method, such as the 2-year, 24-hour rainfall, was based on published data (NWS 2011). The computed times of concentration are presented in Table 2-8.

Table 2-8. Initial Time of Concentration for each Subbasin

Subbasin Name	Drainage area (mi^2)	Time of Concentration (hr)
Leaveave 1	0.26	0.55
Leaveave 2	0.09	0.62
Leaveave 3	0.32	0.40
Leaveave 4	0.19	0.53
Leaveave 5	0.06	0.29
Leaveave 6	0.16	0.76
Leaveave 7	0.26	1.11
Leaveave 8	0.11	0.33
Mapusagatuai 1	0.12	0.38
Mapusagatuai 2	0.08	0.23
Mapusagatuai 3	0.08	0.23
Taumata 1	0.63	0.47
Taumata 2	0.57	0.46
Taumata 3	0.05	0.22
Taumata 3b	0.08	0.35
Taumata 4	0.03	0.27
Taumata 5	0.09	0.30
Taumata 6	0.09	0.21
Taumata 7	0.04	0.28
Vaitele 1	0.27	0.39
Vaitele 2	0.12	0.13
Vaitele 3	0.06	0.22
Vaitele 4	0.04	0.26
Vaitele 5	0.02	0.23
Vaitele 6	0.04	0.2

The Clark Unit Hydrograph storage coefficient, R, accounts for storage in the watershed. This parameter was determined using a mathematical relationship between the longest flow path, drainage area, and time of concentration. An equation was adopted from the “Drainage Design Manual for Maricopa County” (Flood Control District of Maricopa County, 2013) will be used in this study. This relationship was used to make an initial estimate of the storage coefficient of each subbasin. The initial values for the storage coefficient parameters are summarized in Table 2-9.

These estimates were adjusted during the hydrologic model calibration. The equation used is as follows:

$$R = 0.37 T_C^{1.11} A^{-0.57} L^{0.80}$$

R: Storage Coefficient
Tc: Time of Concentration (hrs)
A: Drainage Area (square miles)
L: Length of flow path (miles)

Table 2-9. Initial Storage Coefficient for each Subbasin

Subbasin Name	Drainage area (mi^2)	Storage Coefficient (hr)
Leaveave 1	0.26	0.08
Leaveave 2	0.09	0.11
Leaveave 3	0.32	0.14
Leaveave 4	0.19	0.30
Leaveave 5	0.06	0.16
Leaveave 6	0.16	0.39
Leaveave 7	0.26	0.62
Leaveave 8	0.11	0.12
Mapusagatuai 1	0.12	0.09
Mapusagatuai 2	0.08	0.08
Mapusagatuai 3	0.08	0.08
Taumata 1	0.63	0.10
Taumata 2	0.57	0.13
Taumata 3	0.05	0.07
Taumata 3b	0.08	0.21
Taumata 4	0.03	0.17
Taumata 5	0.09	0.14
Taumata 6	0.09	0.08
Taumata 7	0.04	0.16
Vaitele 1	0.27	0.09
Vaitele 2	0.12	0.09
Vaitele 3	0.06	0.10
Vaitele 4	0.04	0.13
Vaitele 5	0.02	0.19
Vaitele 6	0.04	0.67

2.3.3 Reach Routing and Loss Parameterization

Muskingum-Cunge routing was selected for the routing reach method because the routing method is based on physical parameters such as channel shape, routing reach length, and surface roughness (Manning's n value). Muskingum-Cunge routing lends itself to circumstances where limited observed data is available and can be used in reaches that have a small slope. Routing reaches for each sub-basin was determined based on landform slope and channel shape. Cross sections were represented using an eight-point description of the land surface representative of the reach. The reach length, slope, and cross sections were estimated using field estimates and the terrain model constructed from the LiDAR measurements. The loss-gain method used was percolation, which was estimated to be 10 cubic feet per second (cfs) per acre. This represents the permeability of the cinder rock.

The 2D modeling in HEC-RAS is used to compliment the HEC-HMS routing data. The hydrologic basins that coincide with the HEC-RAS 2D areas were routed through the subbasin using the 2D flow area in HEC-RAS. The current terrain and topography in HEC-RAS provides a more accurate routing than HEC-HMS.

2.4 Model Calibration

Although there is no historical stream flow data within the study area, gaged data from a nearby watershed with similar characteristics was used for calibration purposes. Rainfall and stream flow data from Aasu Stream, located to the northwest of the study area, was used to calibrate the sub-basin parameters within project area. Peak and instantaneous stream flow data is available from USGS 16920500, which was in operation from 1959 to 2002. Instantaneous rainfall data is available from USGS 141842170435801. The December 26, 2001 event was used for calibration as it was the only large event with instantaneous data available from both gages. This was the fourth largest event recorded in the area by the stream flow gage. Based on a frequency analysis completed in the 2013 report completed by POH, this event is approximately equivalent to a 10% AEP (10-year) flood event.

The initial parameters for the Aasu sub-basin were computed using the same loss and transform methods used for Leaveave Drainageway and Drainageway 2. The sub-basin was also calibrated for improved accuracy. The initial and final parameters for the Aasu sub-basin are presented in Table 2-10. The calibrated hydrograph is presented as Figure 2-5.

Table 2-10. Initial and Calibrated Parameters for the Aasu Sub-Basin

Calibration Status	Drainage Area (mi^2)	Curve Number	Time of Concentration (hr)	Storage Coefficient (hr)
Initial	1.05	49	0.90	0.83
Calibrated	1.05	42	0.1	0.24
% Change	0	-14.3	-88.9	-71.1

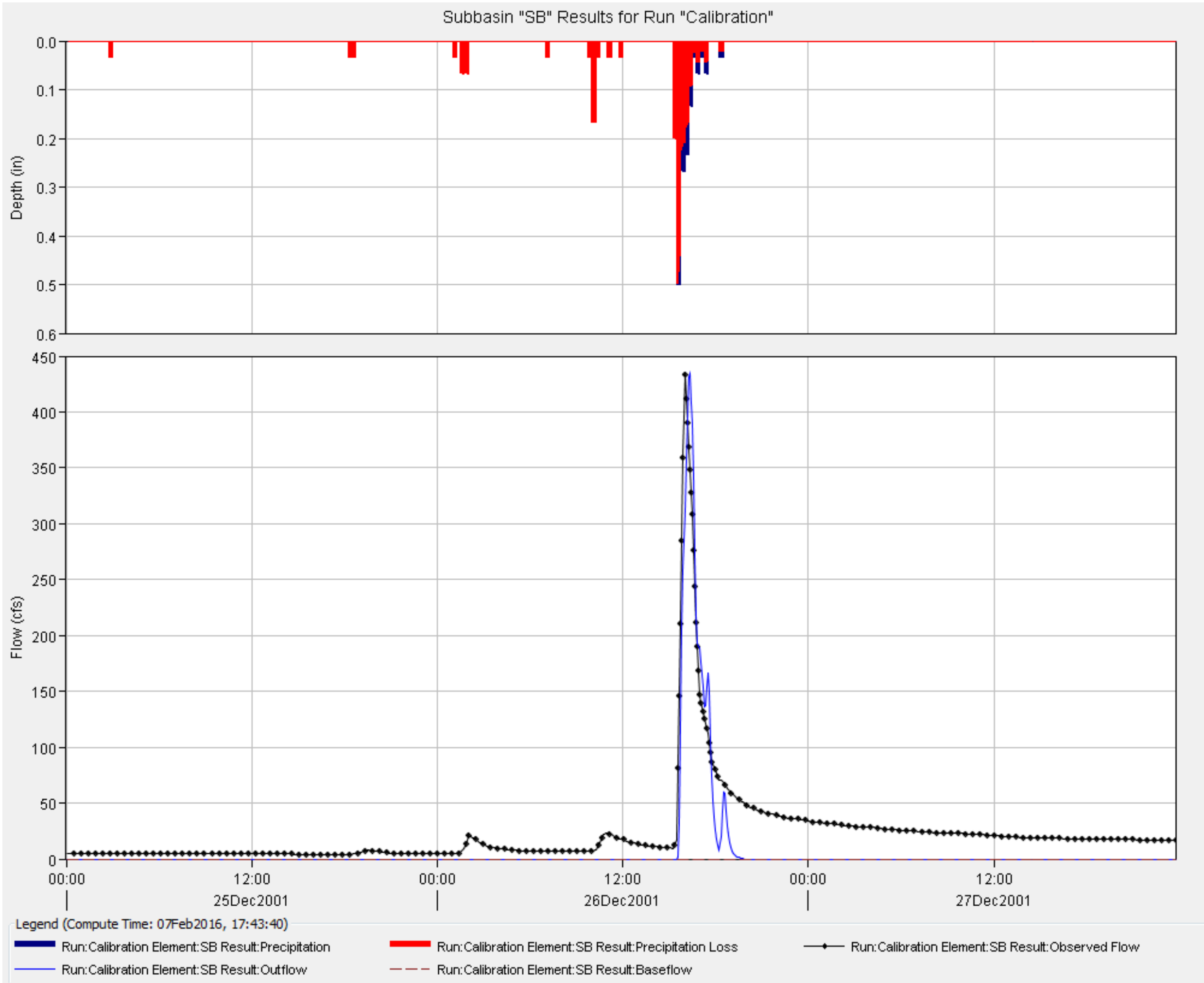

Figure 2-5. Comparison of the Calculated and Observed Hydrographs for Aasu Stream, Tutuila Island, American Samoa

Based on the resulting reduction in the time of concentration and storage coefficient at Aasu Stream, these parameters were re-evaluated for the basins within the study area. The upper sub-basins, which are similar to the steep terrain in the Aasu sub-basin, were reduced by the same percentage. The lower sub-basins, which have flatter slopes, were only reduced half as much. The final calibrated parameters are shown in Table 2-11.

Table 2-11. Final HEC-HMS Model Basin Parameters

Sub-Basin Name	Drainage Area (mi²)	Length (ft)	Slope (ft/ft)	Curve Number	Time of Concentration (hr)	Storage Coefficient (hr)
Leaveave 1	0.26	7257	0.10	48	0.10	0.26
Leaveave 2	0.09	5055	0.14	40	0.10	0.37
Leaveave 3	0.32	8586	0.11	47	0.16	0.46
Leaveave 4	0.19	6677	0.05	54	0.29	0.99
Leaveave 5	0.06	3219	0.04	56	0.16	0.54
Leaveave 6	0.16	5031	0.01	41	0.42	1.28
Leaveave 7	0.26	7510	0.01	47	0.62	2.04
Leaveave 8	0.11	2708	0.02	55	0.18	0.39
Mapusagatuai 1	0.12	4956	0.18	51	0.10	0.31
Mapusagatuai 2	0.08	3063	0.12	57	0.10	0.26
Mapusagatuai 3	0.08	3114	0.26	48	0.10	0.27
Taumata 1	0.63	7456	0.18	43	0.19	0.34
Taumata 2	0.57	6441	0.18	41	0.26	0.44
Taumata 3	0.05	2024	0.01	61	0.12	0.24
Taumata 3b	0.08	4088	0.18	52	0.19	0.69
Taumata 4	0.03	2405	0.02	41	0.15	0.57
Taumata 5	0.09	3633	0.27	48	0.16	0.48
Taumata 6	0.09	2502	0.21	49	0.12	0.25
Taumata 7	0.04	2343	0.02	58	0.16	0.52
Vaitele 1	0.27	7257	0.10	48	0.10	0.31
Vaitele 2	0.12	5055	0.14	48	0.10	0.30
Vaitele 3	0.06	8586	0.11	47	0.12	0.32
Vaitele 4	0.04	6677	0.05	55	0.14	0.44
Vaitele 5	0.02	3219	0.04	49	0.13	0.62
Vaitele 6	0.04	5031	0.01	59	0.20	0.67

2.5 Model Validation

There is no historical stream flow data within the study area to validate the model results; however, regional regression equations were developed based on peak flow data from 10 streamflow-gaging stations with 9 to 32 years of record collected between 1958 and 1990 (USGS 2000). These equations provide a second method for estimating peak flow discharges, which can be compared with the model results. The applicable range of drainage area is 0.11 to

0.78 square miles (mi^2). Therefore, these equations could only be applied to only 11 of the 25 sub-basins within the Leaveave Drainageway. The resulting peak discharges from the regression analysis are presented in Table 2-12 and the computed peak flow discharges are presented in Table 2-13 for comparison.

Table 2-12. Peak Flow Data from Regional Regression Equations

Sub-Basin Element	Drainage Area (mi^2)	Peak Flow (cfs)					
		2-yr[1] (50%)	5-yr[2] (20%)	10-yr[3] (10%)	25-yr[4] (4%)	50-yr[5] (2%)	100-yr[6] (1%)
Leaveave 1	0.26	227	348	428	534	614	695
Leaveave 3	0.32	287	437	535	660	752	845
Leaveave 4	0.19	159	246	306	388	451	518
Leaveave 6	0.16	131	204	255	325	381	440
Leaveave 7	0.26	227	348	428	534	614	695
Leaveave 8	0.11	86	135	171	222	264	309
Mapusagatuai 1	0.12	95	149	187	243	287	336
Taumata 1	0.63	617	920	1104	1317	1462	1599
Taumata 2	0.57	551	824	992	1189	1325	1455
Vaitele 1	0.27	237	362	446	555	637	720
Vaitele 2	0.12	95	149	187	243	287	336

[1] $Q_2 = 1{,}040DA^{1.13}$
[2] $Q_5 = 1{,}530DA^{1.10}$
[3] $Q_{10} = 1{,}810DA^{1.07}$
[4] $Q_{25} = 2{,}110DA^{1.02}$
[5] $Q_{50} = 2{,}300DA^{0.981}$
[6] $Q_{100} = 2{,}470DA^{0.941}$
Where DA = drainage area in square miles

Table 2-13. Computed Peak Flow Discharges in the Leaveave Drainageway SubBasins

Sub-Basin Element	Peak Flow (cfs)					
	2-yr (50%)	5-yr (20%)	10-yr (10%)	25-yr (4%)	50-yr (2%)	100-yr (1%)
Leaveave 1	209	329	426	568	682	801
Leaveave 3	178	290	379	508	612	724
Leaveave 4	100	148	187	242	286	332
Leaveave 6	27.7	51.8	73.0	105	132	161
Leaveave 7	53.2	88.4	118	163	198	237
Leaveave 8	106	153	190	244	286	329
Mapusagatuai 1	107	162	205	266	314	366
Taumata 1	296	523	709	981	1,210	1,450
Taumata 2	191	356	497	709	883	1,070
Vaitele 1	200	317	410	544	650	762
Vaitele 2	90.5	143	185	245	293	345

2.6 Precipitation Frequency Data

Point precipitation data for annual exceedance rainfall was obtained from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 Precipitation-Frequency Data Server (PFDS). This source presents rainfall frequencies from recurrence intervals of 1 to 500 years (100% to 0.2% AEP) for sites in American Samoa, (National Weather Service [NWS], 2014). Precipitation estimates for this study were taken from station 93-1691 "Malaeimi NR Mapausaga". The flood frequency events, synthetic storms were created from statistical precipitation data using the frequency storm method. Frequency Storms were used for the precipitation method. The annual chance of exceedance precipitation depth-duration data (annual maximum time series) was entered into the HEC-HMS model for each event and presented in Table 2-14. This data was applied to all sub-basins within the same drainage area uniformly.

Table 2-14. Precipitation Frequency Estimates for the Leaveave Drainageway, in inches

Duration	Annual Exceedance Probability (Recurrence Interval Year)							
	50% (2-yr)	20% (5-yr)	10% (10-yr)	4% (25-yr)	2% (50-yr)	1% (100-yr)	0.5% (200-yr)	0.2% (500-yr)
5-min	0.75	0.91	1.02	1.16	1.27	1.38	1.49	1.65
15-min	1.51	1.82	2.03	2.32	2.53	2.76	2.99	3.30
60-min	3.01	3.63	4.06	4.63	5.07	5.51	5.97	6.60
2-hr	3.45	4.40	5.06	5.92	6.59	7.27	7.98	8.94
3-hr	3.88	4.97	5.72	6.70	7.47	8.24	9.05	10.10
6-hr	4.96	6.38	7.37	8.67	9.66	10.7	11.7	13.20
12-hr	6.29	8.19	9.49	11.20	12.50	13.90	15.30	17.20
24-hr	7.55	9.92	11.60	13.70	15.40	17.10	18.80	21.20

3 Climate Change

3.1 Literature Review

The USACE is undertaking its climate change preparedness and resilience planning and implementation in consultation with internal and external experts using the best available — and actionable — climate science. As part of this effort, the USACE has developed concise reports summarizing observed and projected climate and hydrological patterns, at a hydrologic unit code (HUC2) watershed scale. The information cited in these reports comes from reputable, peer-reviewed literature and authoritative national and regional reports. Trends are characterized in terms of climate threats to USACE business lines. The reports also provide context and linkage to other agency resources for climate resilience planning, such as downscaled climate data for sub-regions, and watershed vulnerability assessment tools.

A USACE literature review report focused on the Hawaii region was finalized in September 2015 (USACE, 2015). However, the USACE report is more specific to the Hawaii region and not the other U.S. territories in the Pacific. In March 2020, the American Samoa Climate Related Vulnerability Assessment for Transportation Infrastructure Study was published. This study was completed at the request of the American Samoa Department of Public Works. The study objective was to assess the vulnerability of American Samoa's transportation assets to climate related hazards. In addition to infrastructure vulnerabilities associated with environmental factors, social characteristics that influence community resilience to climate related hazards were analyzed to inform mitigation project considerations.

The March 2020 Vulnerability Assessment looked at climate data from the National Weather Service from 1957 to 2017. The assessment noted that the average annual rainfall data shows variability, but very little increase. Similarly, the annual rainfall deviation data from the 60-year average show high variability, but no discernable trend (HHF Planners,2020). Temperature data recorded at the NWS Office located adjacent to Pago Pago International Airport was also reviewed. The average annual temperature data shows a 3-degree increase (HHF Planners,2020).

3.2 Inland Hydrology Climate Change

USACE Engineering and Construction Bulletin (ECB) 2018-14 (*Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects*), "provides guidance for incorporating climate change information in hydrologic analyses in accordance with the USACE overarching climate change adaptation policy. This policy requires consideration of climate change in all current and future studies to reduce vulnerabilities and enhance the resilience of our water resources infrastructure." The document "helps support a qualitative assessment of potential climate change threats and impacts" related to USACE analyses. The subsequent sections discuss the various tools that were developed by the USACE Climate Preparedness and Resiliency Community of Practice (CPR CoP), to meet the qualitative assessment requirements set forth in ECB 2018-14.

There are no continuous stream flow gages within the study area, nor has there been any in the past. Historic stream flow data from a nearby watershed was used for calibration purposes; those same stream gages were used in the below climate tools.

3.2.1 Nonstationary Detection Tool

Stationarity, or the assumption that the statistical characteristics of hydrologic time series data are constant through time, enables the use of well-accepted statistical methods in water resources planning and design in which the definition of future conditions relies primarily on the observed record, per USACE guidance ETL 1100-2-3. However, recent scientific evidence shows that in some locations climate change and human modifications of watersheds are undermining this fundamental assumption, resulting in nonstationarity (Milly et al., 2008, Friedman, et. al, 2016).

The same gage used for calibration was assessed using the web-based Nonstationary Detection (NSD) tool, refer to Figure 3-1. Using the full period of record for the gage there are 3 abrupt changepoints detected. However, there is also a gap in data for this gage around the time of these changepoints.

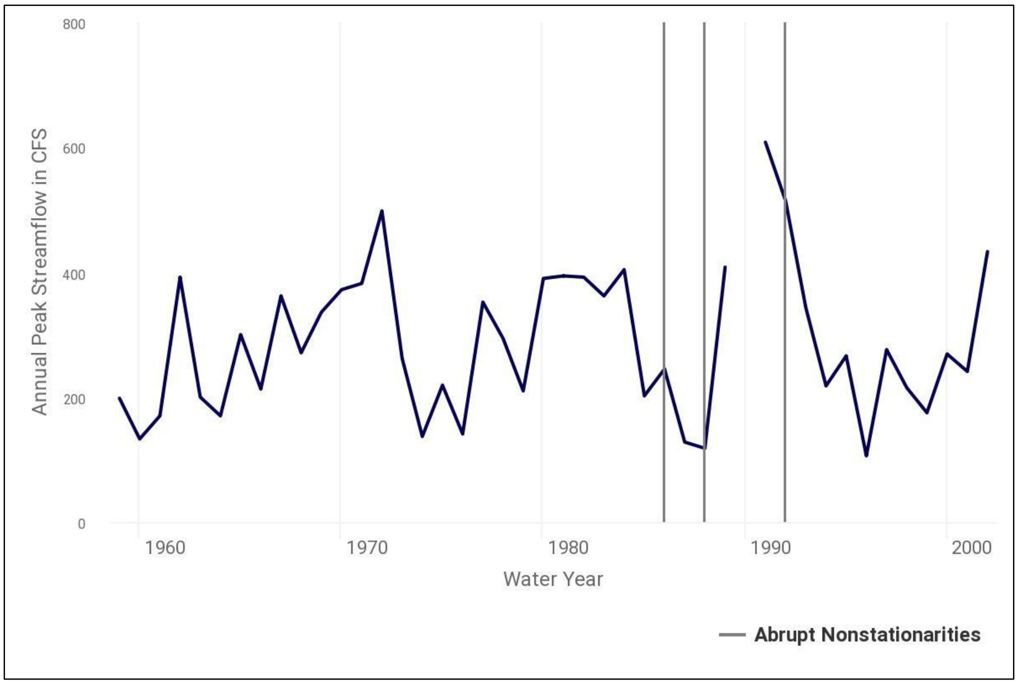

Figure 3-1. Nonstationarity Analysis: Full Period of Record; USGS 16920500 Aasu Stream at Aasu, Tutuila

Looking at the data from 1959 to 1989, just prior to the data gap, no abrupt changepoints are detected. The results from this analysis are show in **Figure 3-2**. Therefore, the abrupt changepoints detected above can be attributed to the gap in gage record.

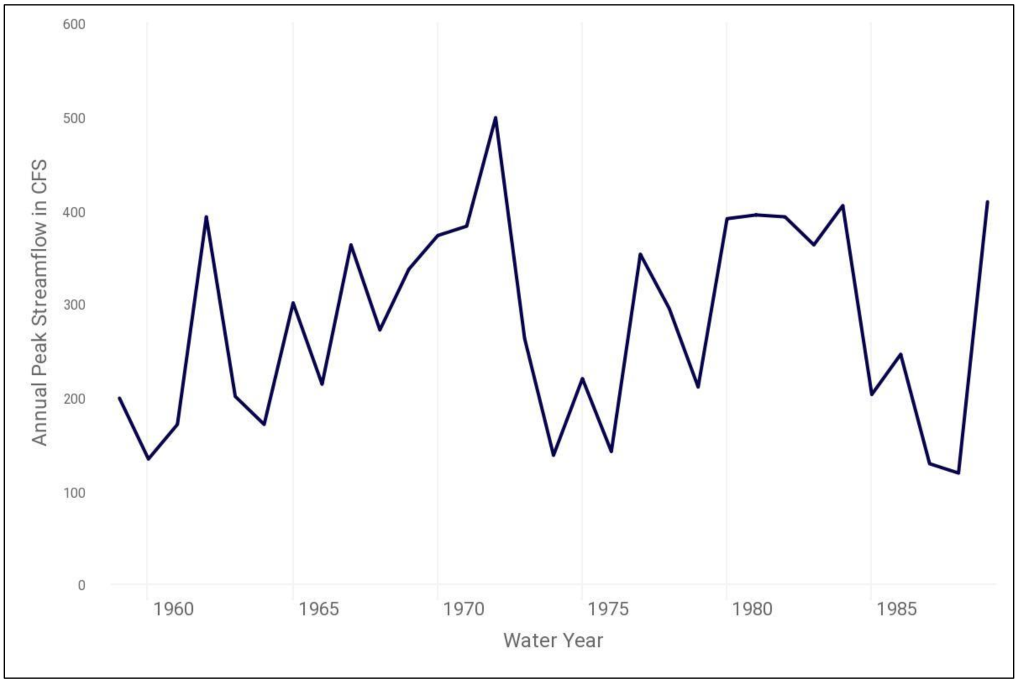

Figure 3-2. Nonstationarity Analysis: 1959-19889; USGS 16920500 Aasu Stream at Aasu, Tutuila

Using the same period, 1959 – 1989, there are no statistically significant monotonic trends detected in the peak streamflow record, refer to Figure 3-3.

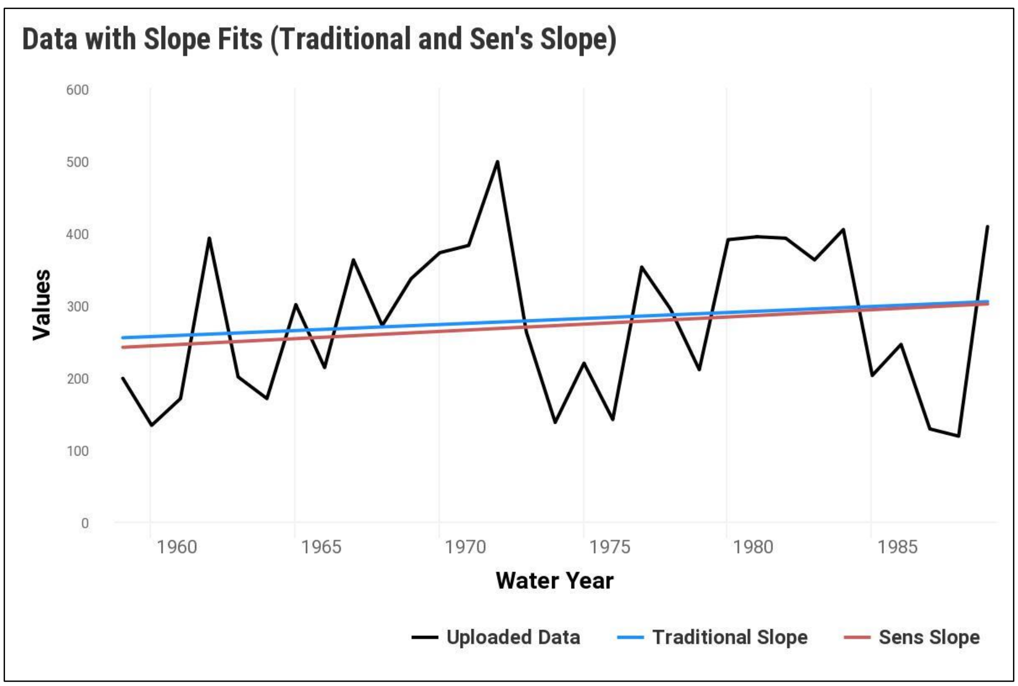

Figure 3-3. Trend Analysis for Aasu Stream at Aasu, Tutuila

3.2.2 Linear Trend Analysis

As required by ECB No. 2018-214, an investigation of the trends in the annual maximum flow gage data could not be performed using the USACE Climate Hydrology Assessment Tool, because the study is not located in one of the HUC-4 watersheds included in the tool. Therefore, the gage data used for calibration was uploaded into the time series toolbox, to determine if any statistically significant trends are identified.

(Tool down, will add figure or statement)

3.2.3 Vulnerability Assessment Tool

The USACE Vulnerability Assessment Tool was not applied, because the study is not located in the HUC-4 watersheds included in the tool.

3.3 Summary

Based on the literature review it is reasonable to conclude that temperatures are increasing in the study area and will continue to increase within the foreseeable future. The literature review does not provide definitive evidence of increasing trends in either observed or projected, precipitation and streamflow records within the region. Consequently, there is not a lot of concrete evidence that flood risk will increase due to climate change in the foreseeable future. For these reasons it is appropriate to assess the impacts of climate change on inland hydrology qualitatively throughout the plan formulation process. Table 6-1 summarizes residual risk due to climate associated with the tentatively selected plan.

4 Hydraulics

Similar to the hydrologic analysis, the 2016 and 2019 hydraulic analysis from the previous reports was used as a starting point for this study. The model utilizes both two-dimensional (2D), as well as one-dimensional (1D), unsteady flow analysis and was created for this study using the Hydrologic Engineering Center's River Analysis System (HEC-RAS) software (version 5.0.7). The 1D portion of the model was developed for the channel area generally extending from bank to bank and incorporating the bridge geometry and hydraulics. The 2D portion of the model was developed for the overbank areas to capture a more accurate extent and depth of flooding. There are many areas that have super critical flow and steep slopes which is why the model used the mixed flow regime. See Figure 4-1 for extent of hydraulic modeling in HEC-RAS.

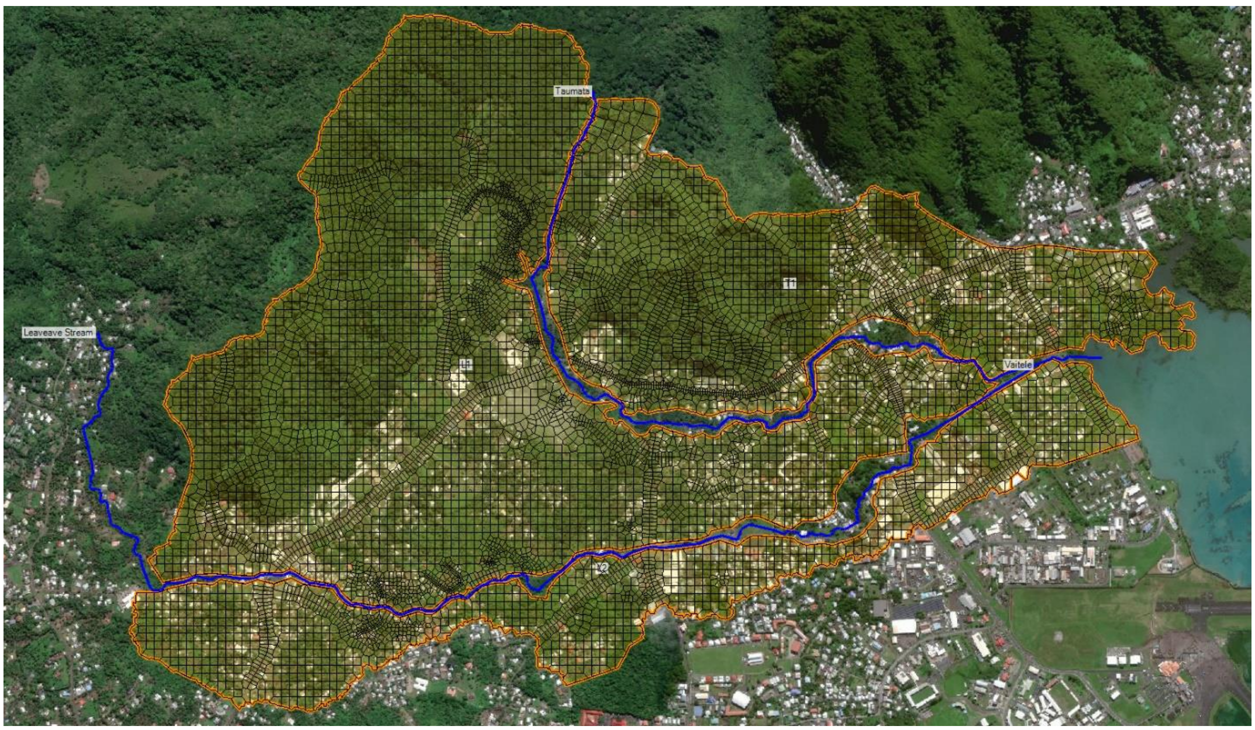

Figure 4-1. Extent of hydraulic modeling in HEC-RAS

4.1 Flow Data

Peak flow rates determined in the previous section will be used to represent the amount of water in the system for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.2% AEP events (8 profiles).

The corresponding flow data was input to the appropriate cross sections as lateral inflow or uniform lateral flow. The majority of the flow data was inputted as uniform lateral flow which is based on the terrain data. This is because the tributary areas are along the stem of the main reaches and do not enter at a specific cross section. For locations where the flow entered a specific cross section the lateral inflow hydrograph was used. See Table 4-1 for the HEC-HMS basin model elements that correspond the HEC-RAS river station cross sections.

Table 4-1. HEC-HMS Basin Model Elements and the Corresponding HEC-RAS River Stations (XS)

HEC-HMS	HEC-RAS		
Element	**River**	**Reach**	**XS**
Junction-1	Leaveave Stream	Upper	13164
Leaveave 4	Leaveave Stream	Upper	11883
Leaveave 5	Leaveave Stream	Upper	9193
Leaveave 6	Leaveave Stream	Upper	9030
Leaveave 7	Leaveave Stream	Upper	6425
Leaveave 8	Leaveave Stream	Upper	2297
Upper Taumata Junct	Taumata	Upper	11531
Taumata 2	Taumata	Upper	10807
Junction-2	Taumata	Upper	7891
Taumata 3B	Taumata	Upper	7838
Taumata 4	Taumata	Upper	7470
Taumata 5	Taumata	Upper	5193
Taumata 6	Taumata	Upper	4099
Taumata 7	Taumata	Upper	2715
Vaitele 3	Taumata	Upper	2576
Vaitele 4	Taumata	Upper	1271
Vaitele 5	Taumata	Upper	1579
Vaitele 6	Vaitele	Upper	746
N/A (Stage Hydrograph)	Vaitele	Outlet	121
Upper Vaitele Junction	Vaitele	Boundary Condition Viatele	N/A

4.1.1 Boundary Conditions

Boundary conditions are necessary to establish the starting water surface at the upstream and downstream ends of the channel system. A flow hydrograph from the HMS model was used to represent the flow entering the watershed at upstream locations.

The downstream boundary condition, entered as a stage hydrograph, was set to 2.5 ft . This considered the MHHW as recorded at the National Oceanic and Atmospheric Agency (NOAA) tide gage (Station ID: 1770000, Pago Pago, American Samoa) and for interannual variation

(IAV). This is the only NOAA tidal gauge on Tutuila and has been operated since 1989. Figure 4-2 below shows the tidal datums for the gage. This boundary condition seems appropriate when comparing it to Figure 4-3, which has a WSE of 0.84 m (2.75 ft) for the 10% AEP.

Elevations on Mean Sea Level

Station: 1770000, Pago Pago, American Samoa
T.M.: 0
Status: Accepted (Dec 7 2020)
Epoch: 0000-0000
Units: Feet
Datum: MSL
Control Station:

Datum	Value	Description
MHHW	1.43	Mean Higher-High Water
MHW	1.29	Mean High Water
MTL	0.00	Mean Tide Level
MSL	0.00	Mean Sea Level
DTL	0.04	Mean Diurnal Tide Level
MLW	-1.30	Mean Low Water
MLLW	-1.36	Mean Lower-Low Water
ASVD02		
STND	-4.68	Station Datum
GT	2.78	Great Diurnal Range
MN	2.59	Mean Range of Tide
DHQ	0.14	Mean Diurnal High Water Inequality
DLQ	0.06	Mean Diurnal Low Water Inequality
HWI	6.22	Greenwich High Water Interval (in hours)
LWI	12.38	Greenwich Low Water Interval (in hours)
Max Tide	2.68	Highest Observed Tide
Max Tide Date & Time	02/19/2019 05:42	Highest Observed Tide Date & Time
Min Tide	-2.47	Lowest Observed Tide
Min Tide Date & Time	05/06/2016 11:18	Lowest Observed Tide Date & Time
HAT	2.19	Highest Astronomical Tide
HAT Date & Time	01/20/1996 05:30	HAT Date and Time
LAT	-2.00	Lowest Astronomical Tide
LAT Date & Time	02/08/2001 12:18	LAT Date and Time

Figure 4-2. Tidal Datums: Pago Pago, America Samoa 1770000

Figure 4-3 displays the high and low annual exceedance probability levels in meters relative to the mean sea level datum. The plots show the monthly highest and lowest water levels with the 1%, 10%, 50%, and 99% annual exceedance probability levels in red, orange, green, and blue. Mean Higher High Water (MHHW), Mean High Water (MHW), Mean Lower Low Water (MLLW) and Mean Low Water (MLW) values are displayed in black. On the left are the exceedance probability levels for the mid-year of the tidal epoch currently in effect for the station (consistent with the values in Figure 4-2). On the right are projected exceedance probability levels and tidal datums for 2018 assuming continuation of the linear historic trend. The difference between the 1% AEP and 10% AEP is approximately 0.06 meters (0.20 feet). Discussion regarding future without project conditions, incorporating sea level rise are discussed further in Section 4.3.

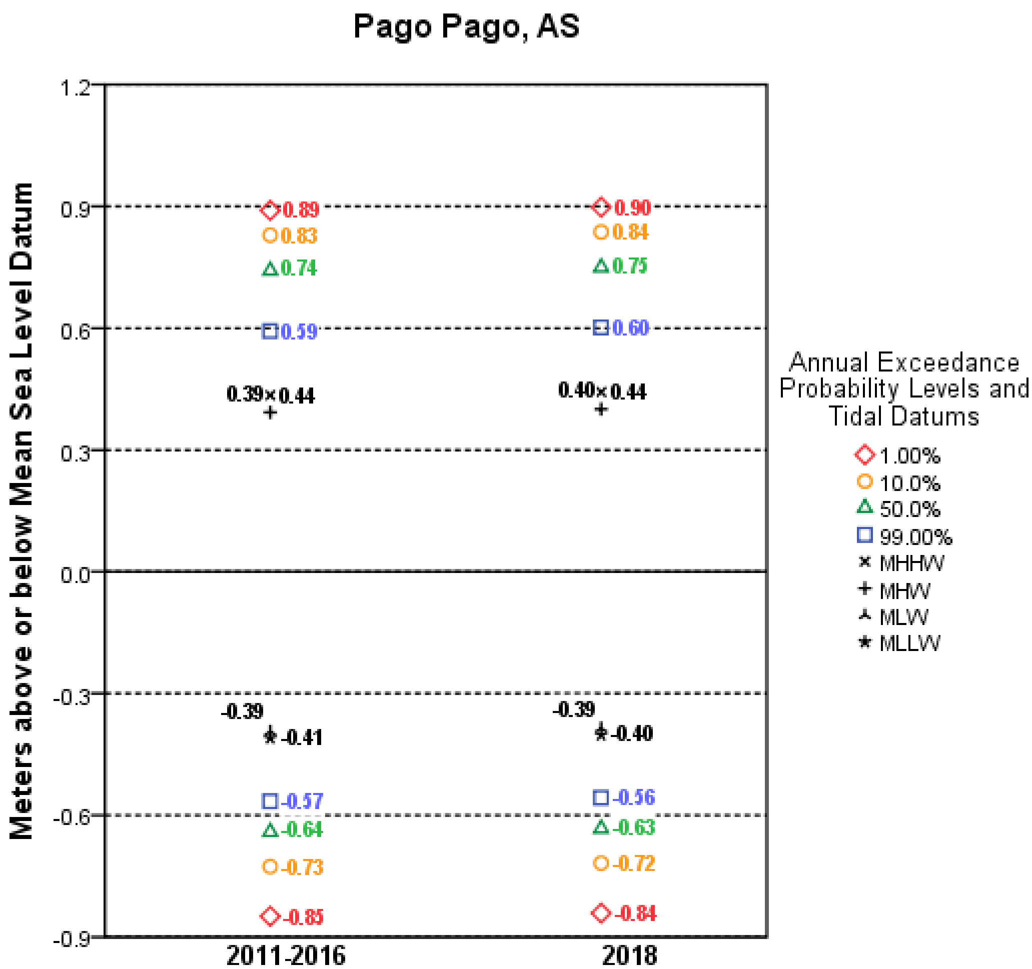

Figure 4-3. Exceedance Probability Levels and Tidal Datums: Pago Pago, America Samoa 1770000

4.2 Geometry Data

RAS Mapper, a geospatial interface in the HEC-RAS software, will be used to fully develop the geometric data required for the river hydraulics model. Elevation data will be imported to create the terrain model. The projection was set to UTM Zone 2S (Feet) with reference to the NAD 83 coordinate system. Several geometric layers required for the hydraulic model were digitized, some of which are described in Table 4-2.

Table 4-2. GIS layers created for hydraulic models

GIS layer	Description
2D Flow Areas	2D Flow Areas are created by constructing polygon areas representing the regions to be modelled.
Boundary Condition	A Boundary Condition (BC) line was added to identify the location for a specific flow condition on the boundary of a 2D Flow Area.
Breakline	Breaklines were sometimes used in 2D Flow Areas to align the computation cell faces along high ground and natural barriers that affect flow and direction (such as river banks).
SA/2D Area Connection	This internal connection feature can be used to represent embankment crests and major roads.

The terrain file from 2019 was used as an update to the model. The 1D steady flow model was converted to a 1D-2D unsteady flow model. The cross sections were trimmed to represent the stream in 1D and 2D flow areas were created to represent the overbank areas. Lateral structures were placed at the high ground and connect the 1D and 2D areas.

4.2.1 Manning's Roughness Coefficient, n

Manning's roughness coefficient, *n*, is an empirically derived coefficient that is dependent on several variables, such as vegetation, obstructions, and meandering when applied to open channels. This value was selected based on site characteristics observed in the field, aerial imagery, and land cover classifications. A land cover layer was created in HEC-RAS mapper tool using the Land Use shapefile from NOAA as shown in Figure 2-4. This shapefile was produced in the 2019 report. A document titled "*Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains"* was used to determine n values. Typical *n* values selected for this study are provided in Table 4-3 for 2D Flow Areas.

Table 4-3. Manning's n Values

Land Cover ID	Land Cover Type	Manning's *n*
1	Unclassified	0.04
2	Impervious surface	0.015
5	Open space developed	0.04
6	Cultivated	0.035
7	Pasture / hay	0.03
8	Grassland	0.035
9	Deciduous forest	0.16
10	Evergreen forest	0.16
12	Scrub / shrub	0.1
13	Palustrine forested wetland	0.1
14	Palustrine scrub shrub wetland	0.1
15	Palustrine emergent wetland	0.07
16	Estuarine forested wetland	0.1
17	Estuarine scrub shrub wetland	0.1
18	Estuarine emergent wetland	0.07
19	Unconsolidated shore	0.035
20	Bare land	0.025
21	Open water	0.04

4.2.2 Bridges

Bridges and major culverts were represented in the model as a *Bridge/Culvert*. Bridge and/or culvert geometric data required for this modeling feature were obtained during a 2015 site visit. In previous studies for low flow conditions, all the bridges and culverts were modeled using whichever computation method generated the highest energy loss. For high flow conditions, the standard step energy method was used. This was evaluated again for this study. Bridge information is provided in Table 4-4.

Table 4-4. HEC-RAS Bridge Infomation for the Leaveave Drainageway

River	Reach	XS	Bridge Name	Structure Type	Bed Material
Leaveave Stream	Upper	12763	Leaveave Culvert #1	Box Culvert	Concrete
Leaveave Stream	Upper	11923	Leaveave Route 1 Bridge	Box Culvert	Concrete
Leaveave Stream	Upper	2273	Leaveave Airport Road Bridge	Pipe Culvert	Plastic
Leaveave Stream	Upper	700	Leaveave South Bridge 1	Bridge	Concrete
Taumata	Upper	7851	Taumata Route 1 Bridge	Twin Box Culvert	Concrete
Taumata	Upper	3886	Taumata Culvert #1	Bridge	Concrete
Taumata	Upper	3785	Taumata Footbridge #1	Simple Span	Natural
Taumata	Upper	2725	Taumata Bridge	Twin Box Culvert	Concrete
Vaitele	Outlet	494	Vaitele Lower Bridge	Twin Box Culvert	Concrete

4.2.3 Lateral Weirs

Due to the limited capacity of the channel at some locations, water would overflow into other streams or floodplains. This was also the case at locations upstream of the junctions, where a stream and its tributary were within proximity of each other. To account for this interchange of flow, lateral weirs were added in the HEC-RAS geometry to make flow adjustments automatically, as computed. The lateral weirs were placed along the top of the banks or at the high ground based on the terrain for both Taumata and Leaveave Streams. The lateral weirs are used to connect the 1D stream flow to the 2D overbank flow. The weir coefficient selected is based on the HEC-RAS manual guidance.

4.2.4 Inline Weirs

Inline weirs were used to represent known fords at three locations along the “Taumata, Upper” reach. The HEC-RAS station numbers are 6655 and 4557. One inline weir was also used in the “Leaveave Upper” reach at station 1352.

4.2.5 Ineffective Flow Areas

Ineffective flow areas were used to model areas which do not actively convey flow downstream. This includes those areas in the floodplain where water will pond due to the floodplain valley expanding or contracting or due to water backing up into tributary channels, areas around bridges, and areas within a cross-section where water will only pond and not flow downstream. Ineffective flow areas located in the floodplain were defined within HEC-RAS.

Around bridges, a contraction zone of 1:1 and an expansion zone of 2:1 were used. This assumes that the flow approaching a bridge will contract 1 foot for every 1 foot of approach until the active flow area equals the opening width of the bridge. Likewise, the expansion of flow on the downstream side of the bridge opening will expand 1 foot for every 2 feet in the downstream direction.

4.2.6 Breaklines

Breaklines were inserted in the 2D flow area at the high ground to prevent water from spilling over to other cells. Due to the larger cell size adding breaklines allowed the 2D flow area to be further refined to simulate the natural flow of water instead of jumping across cells. The breaklines were inserted based on the terrain data and aerial imagery.

4.3 Future Without- and With-Project Conditions

Consistent with Engineering Regulation (ER) 1100-2-8162, sea level rise was incorporated into the downstream boundary condition. Based on the discussion in Section 4.3.1 below, a downstream stage hydrograph of 4.28 feet was used as the downstream boundary condition in all future without and with-project conditions model runs. This was determined using the low-rate estimate at the 50-yr period of analysis and taking into the account the high margin of error on the user enter rate which was more conservative than the rates built into the USACE calculator. Based on sensitivity runs, the TSP features are outside the coastal influence zone. Post TSP all proposed features will be evaluated using all three rates.

4.3.1 Sea Level Rise

Relative Sea Level Change (RSLC) is an important variable in flood risk management projects because sea level change can potentially affect the project and system performance. Therefore, projects need to consider how sensitive and adaptable engineered systems are sea level change.

ER 1100-2-8162 requires that planning studies and engineering designs over the project life cycle, for both existing and proposed conditions, consider a range of possible future rates of SLC when formulating and evaluating alternatives. This includes both structural and non-structural solutions.

This study uses current USACE guidance to assess relative sea level change. Current USACE guidance (ER 1100-2-8162 and ETL 1100-2-1) specifies the procedures for incorporating RSLC into planning studies and engineering design projects. Projects must consider alternatives that formulated and evaluated for the entire range of possible rates of RSLC for both existing and proposed projects. USACE guidance specifies evaluating alternatives using “low, “intermediate”, and “high” rates of future sea level change.

- Low: Uses the historic rate of local mean sea-level change
- Intermediate: Estimate the “intermediate” rate of local mean sea-level change using the modified NRC Curve I. It is corrected for the local rate of vertical land movement.
- High: Estimate the “high” rate of local mean sea-level change using the modified NRC Curve III. It is corrected for the local rate of vertical land movement.

USACE (ETL 1100-2-1, 2014) recommends an expansive approach to considering and incorporating RSLC into civil works projects. It is important to understand the difference between the period of analysis (POA) and planning horizon. Initially, USACE projects are justified over a period of analysis, typically 50 years. However, USACE projects can remain in service much longer than the POA. The climate for which the project was designed can change over the full lifetime of a project to the extent that stability, maintenance, and operations may be impacted, possibly with serious consequences, but also potentially with beneficial consequences. Given these factors, the project planning horizon (not to be confused with the economic period of analysis) should be 100 years, consistent with ER 1110-2-8159. Current guidance considers both short- and long-term planning horizons and helps to better quantify RSLC.

4.3.2 March 2020 American Samoa Vulnerability Assessment

The following discussion was taken from the March 2020 Vulnerability Assessment:

> Increases in earth's surface temperatures (land and sea) are causing large melt events to land-based glaciers as well as thermal expansion of ocean waters, both of which are contributing to global sea level rise. Relative sea level rise is a combination of this global change in sea level with subsidence, or sinking, of the tectonic plates. This phenomenon is occurring in American Samoa and was hastened by a powerful combination of near-simultaneous fault and thrust earthquakes that occurred in the Tonga Trench in September 2009 (Scientific American, 2010; National Science Foundation, 2010). Based on Pago Harbor tide gauge data, this event caused Tutuila to initially rise about 2 to 3 inches at the time of the earthquake event, and then sink down about 7 to 9 inches over the next 2 to 3 years due the more immediate "relaxation from the earthquake deformation."
>
> Mörner, et al., (2018) use National Aeronautics and Space Administration (NASA) Jet Propulsion Laboratory (JPL) global positioning system (GPS) data to illustrate the gradual subsidence experienced by Tutuila in the years preceding the 2009 quake, the rapid fall following the quake, and the gradually reducing rate of subsidence following a classic exponential decay curve (i.e., the rate of change becomes slower as time passes).
>
> Since then, the ongoing subsidence is estimated to be occurring at a rate of about 0.3 to 0.6 inches per year and is expected to continue in addition to anticipated climate-related sea level rise. Han, et al., (2019) note that the ocean around American Samoa, in the decades prior to the 2009 Tonga Trench earthquake event, is estimated to have risen about 0.08 to 0.12 inches per year, which is comparable to the global average (Stocker, et al., 2013). Based on these estimates, subsidence is occurring about five times faster than warming-related sea level rise, "…a total sea level rise of [12 to 16 inches, associated with viscoelastic relaxation] is predicted throughout this century, as the solid Earth adjusts to the stress change after the 2009 earthquake... it will worsen coastal flooding on the islands leading to regular occurrence of tide-induced nuisance flooding" (Han, et al. 2019).
>
> Based on the GPS record and tapering subsidence, NOAA (Chris Zervas, email communication) provided an estimate of relative sea level change accounting for subsidence at the more balanced rate from 2011 to 2018 in combination with thermal influences on sea level rise (e.g., glacial melt, thermal expansion). A relative sea level trend of 8.9 mm per year (0.35 inches) was calculated with a high margin of error (+/- 9.8 mm per year; 0.386 inches) based on uncertainty due to the strong influence of ENSO forcing in the region. The rate and extent of subsidence also contributes to uncertainty and will require monitoring over time to help inform relative sea level change estimates.

4.3.3 USACE Sea Level Calculator

Using the USACE Sea Level Change Calculator, and entering the relative SLC rate discussed above, Figure 4-4 shows that for a 50 year POA with sea level rise estimates ranging from 2.552 to 5.423 feet above relative mean sea level by the year 2080. For year 2130, the estimates range from 4.012 to 11.072 feet above relative mean sea level. Its important to keep in mind that these rates include a high margin of error (+/- 9.8 mm per year; 0.03 feet) based on uncertainty due to the strong influence of ENSO forcing in the region.

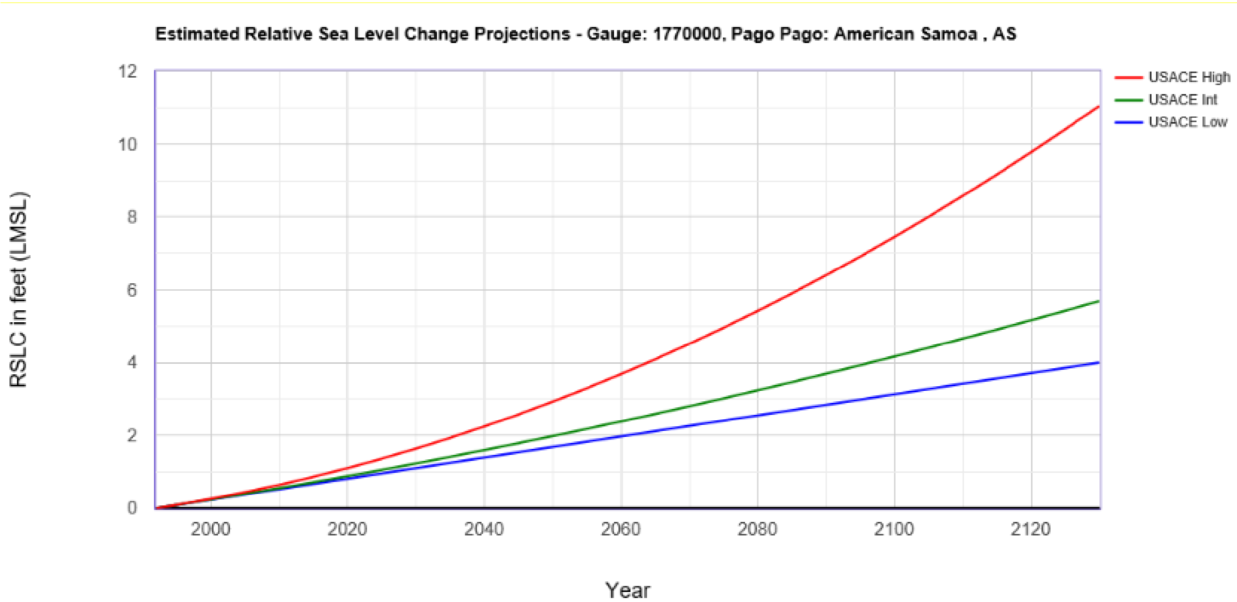

Figure 4-4: USACE Sea Level Change Curve Calculator, Pago Pago: American Samoa, AS

The RSLC for the various years are summarized in Table 4-5**.** These values will be incorporated into the future with and without project conditions to assess the impacts RSLC will have on the project area.

Table 4-5. Estimated Relative Sea Level Change Projections, Pago Pago: American Samoa

Year	Low	Intermediate	High
2030	1.112	1.24	2.256
2080	2.562	3.251	5.433
2130	4.012	5.705	11.072

The Sea Level Tracker visualizes historical, observed changes in mean sea level (MSL) as measured and reported by National Oceanic Atmospheric Administration (NOAA) tide gauges, mapped against the USACE sea level change (SLC) projections. The tool enables the comparison of actual SLC with USACE SLC projections (as described in ER 1100-2-8162), along with observed monthly water levels and trends based on historical data. Figure 4-5, provides the output of the Sea Level Tracker tool for the Pago Pago, American Samoa gage.

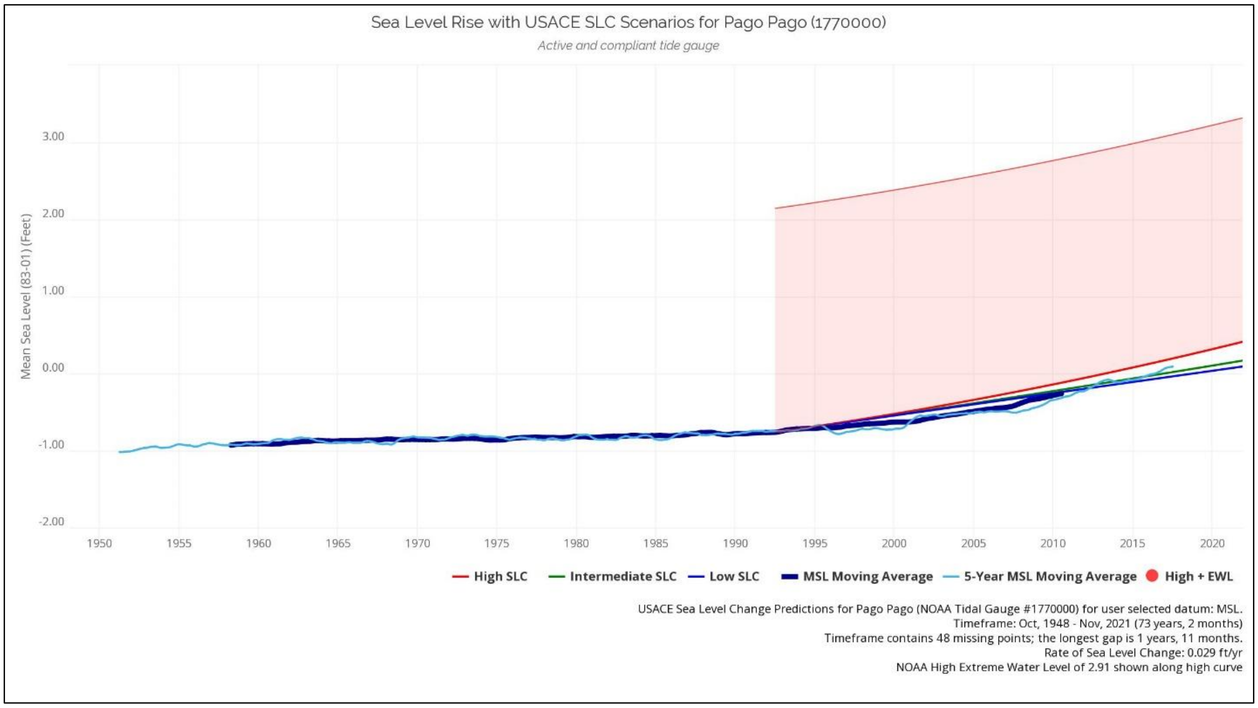

Figure 4-5: USACE Sea Level Tracker Tool, Pago Pago: American Samoa, AS

4.4 Sediment Transport

In accordance with ER 1110-2-8153, the tentatively selected plan reviewed and considered impacts due to sedimentation. The watershed drains to the Pala Lagoon, a shallow estuarine body of water and the only enclosed lagoon on Tutuila. The lagoon is thought to be an important nursery and spawning ground for fish and invertebrates found on the reef. **Table 4-6** summarizes the without project average in-channel and overbank velocities for the Taumata Stream for varying storm events. The tentatively selected plan includes a flood barrier (levee and/or floodwall) that will prevent water from frequently overtopping the stream banks. This will, therefore, decrease that amount of water flowing through the residential and commercial areas during frequent storm events. By reducing runoff from these areas, the TSP could potentially decreasing pollutant loading to the lagoon. Therefore, is not anticipated to alter sediment load conditions. As hydraulic modeling is refined post-TSP, the PDT will continue to evaluate if there will be any adverse impacts to the Pala Lagoon.

Table 4-6. Average Velocities in Study Area, Taumata Stream

	50% AEP		10% AEP		1% AEP	
	In Channel (ft/s)	Overbank (ft/s)	In Channel (ft/s)	Overbank (ft/s)	In Channel (ft/s)	Overbank (ft/s)
Without Project	5	1.7	4.4	1.3	5.2	1.6

4.5 Model Results & Limitations

There are some areas within the reaches where the flow is less than 800 cfs. This is due to water overtopping the banks and flowing out of the stream. The locations where the water is overtopping the banks is in areas where the stream is not as well defined and has low lying areas. Leaveave Drainageway has very limited data for calibration. The 1% AEP floodplain was compared to the FEMA floodplain and there is general concurrence amongst the two. Additional calibration for the HEC-RAS model was not able to be performed. The model was developed with the best available data and engineering judgement. Many model assumptions and parameters will be verified post-TSP during a site visit in early 2022.

The base and future without project conditions is depicted in Tafuna FPMS_*TF3_RevLS* HEC-RAS geometry file (.g05) and without-project plans (.p01, .p02, .p03, .p04, .p05, .p06, .p07, .p14). The base future without-project conditions is depicted in Tafuna FPMS FWOP *TF_FagaimaRd* HEC-RAS geometry file(.g03) and future without-project plans (.p01, .p02, .p03, .p04, .p05, .p06, .p07, .p14).

The without project depth grids and water surface profiles for the 10% and 1% AEP can be found on Figure 4-6 through Figure 4-9.

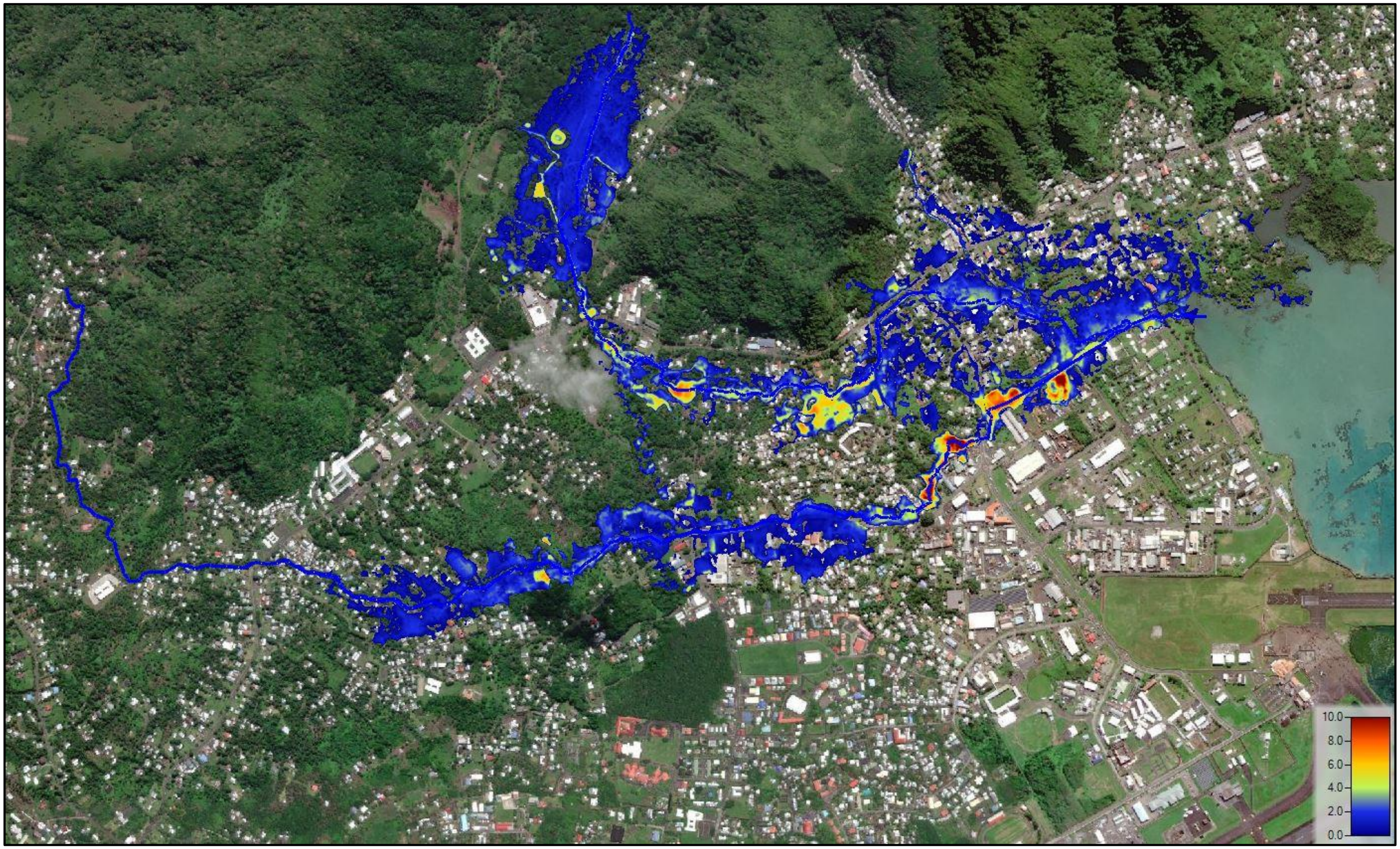

Figure 4-6. RAS Mapper depth grid for study area, 10% AEP

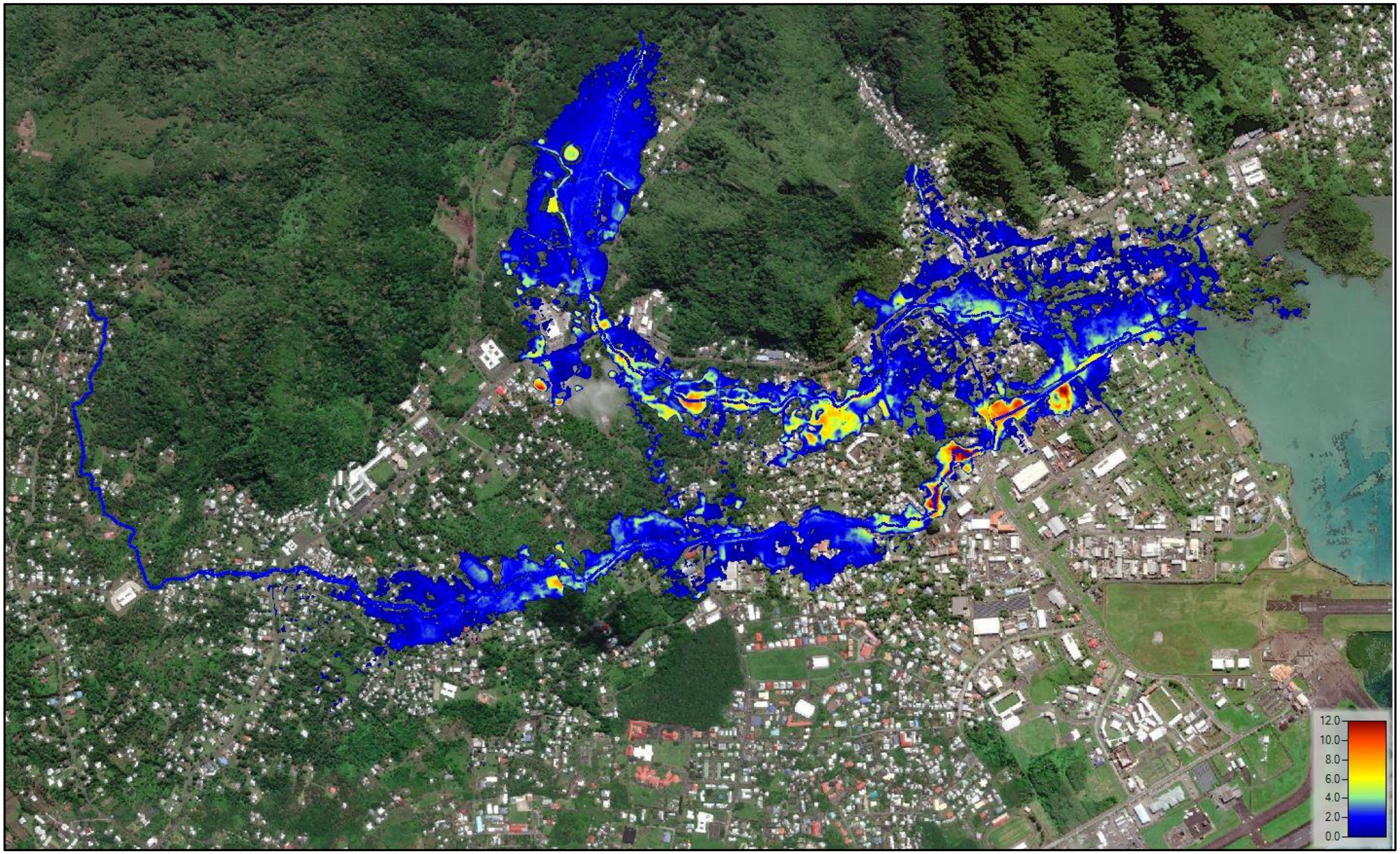

Figure 4-7. RAS Mapper depth grid for study area, 1% AEP

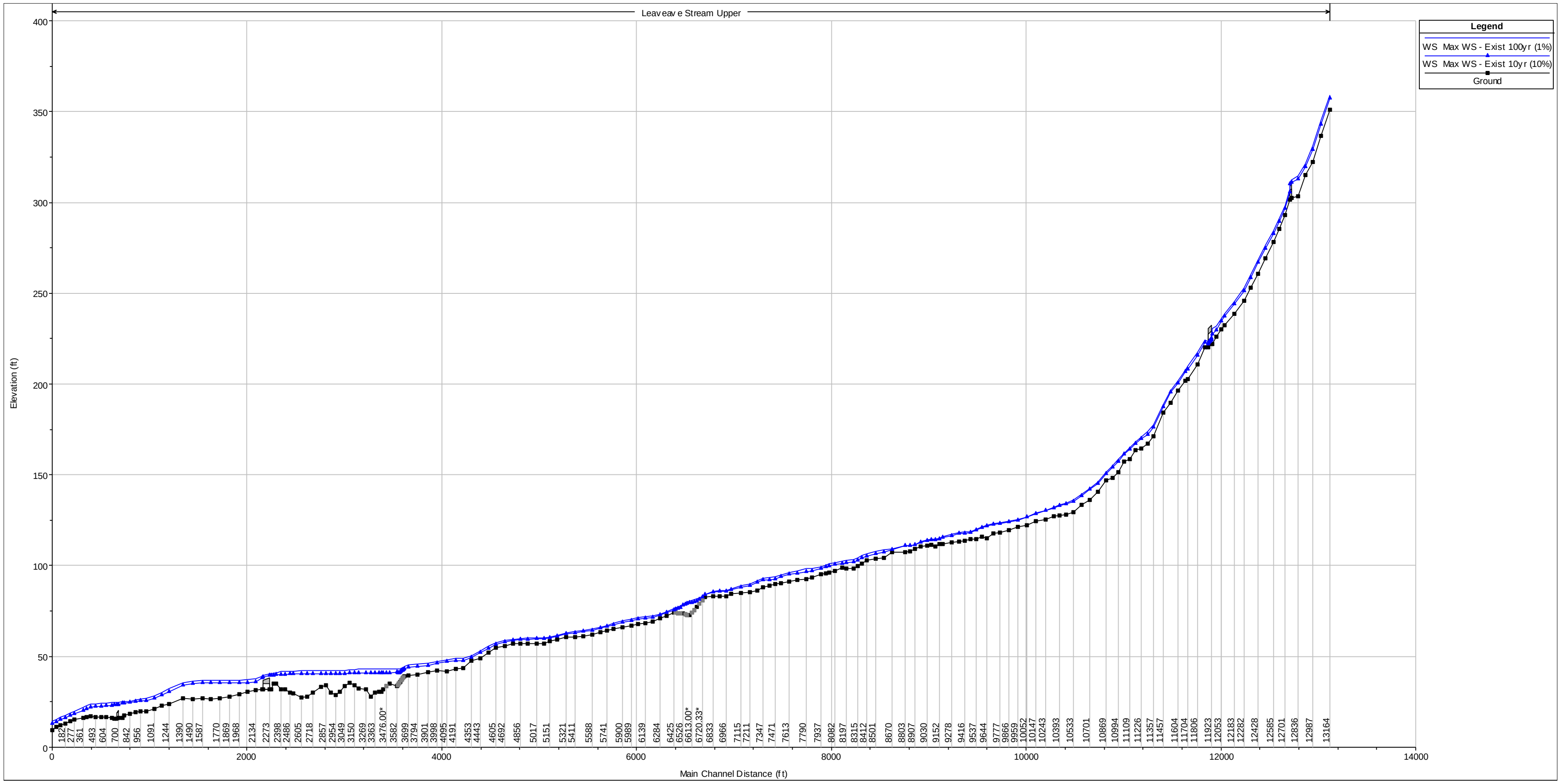

Figure 4-8. HEC-RAS water surface elevation profile plot for the 10% and 1% AEP, Leaveave Stream

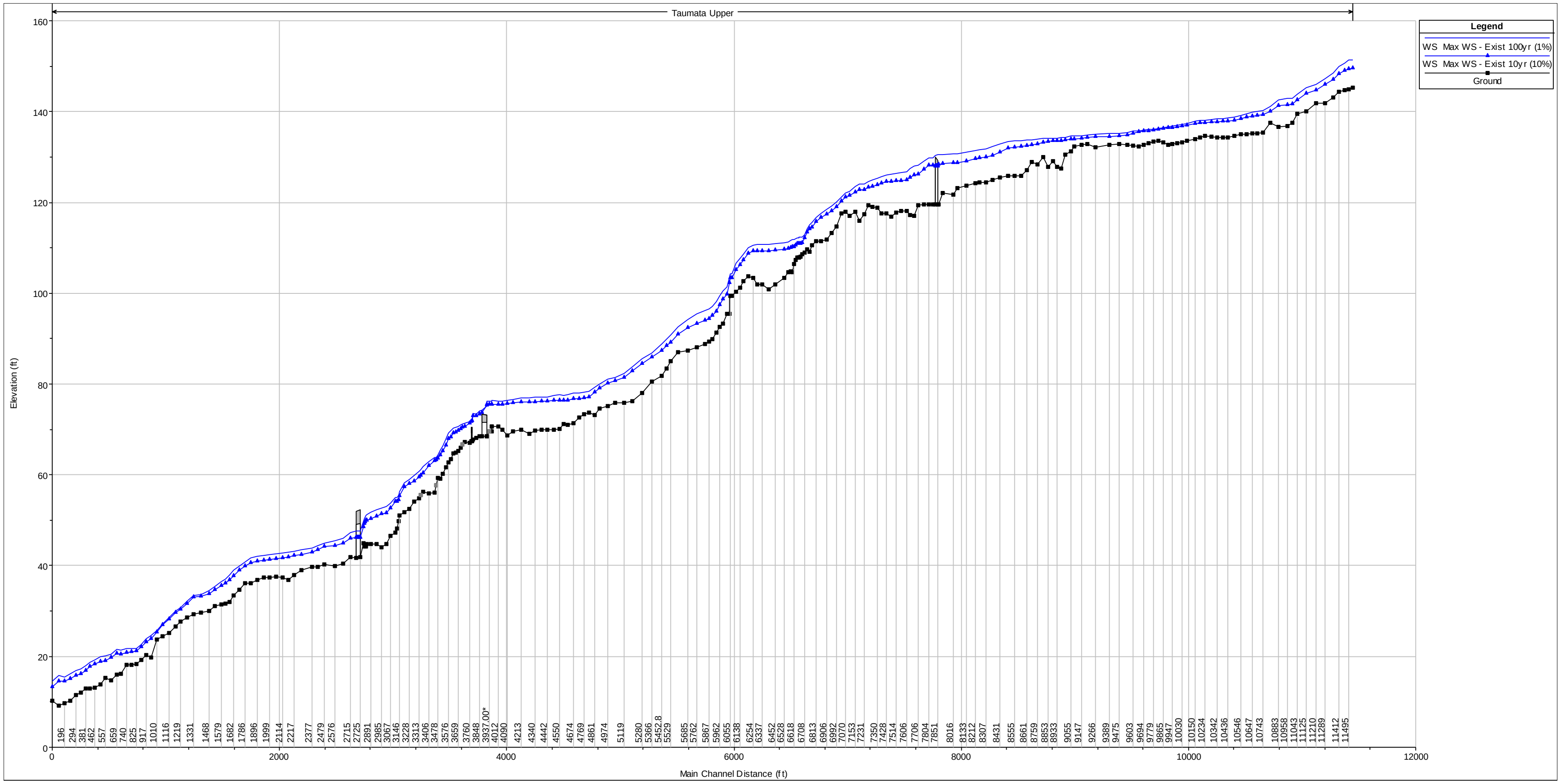

Figure 4-9. HEC-RAS water surface elevation profile plot for the 10% and 1% AEP, Taumata Stream

5 Plan Formulation

A detailed discussion of the plan formulation for this study is provided in the main report. Management measures are features or activities that can be implemented at a specific geographic location to address all or a portion of the problems. Measures can directly address the hazards, the way the hazards behave (performance), or indirectly address them through eliminating or reducing the consequences. Once the initial list of possible flood risk reduction measures was assembled, each measure was then considered in the context of the study area. From this, the initial alternatives array was developed. Three structural measures, Alternatives B, B_1, and C from the initial alternatives array were evaluated as part of the hydraulic model and economics analysis. Once the hydraulic modelling was complete, the resulting HEC-RAS water surface profiles were provided for further assessment in the economic analysis. In addition, profiles were reviewed in accordance with Executive Order (EO) 11988.

5.1 **Evaluation of Alternatives**

As stated above three structural measures, Alternatives B, B_1, and C from the initial alternatives array were evaluated in the hydraulic model and economic analysis. Based on the initial results, Alternative C was further refined to optimize benefits as part of the final array and is presented as the tentatively selected plan.

Alternative B looked at approximately 6,340 feet of channel conveyance on the Taumata Stream and 13,120 feet of channel conveyance on the Leaveave Stream. This alternative included vegetation removal and conveyance improvements such as excavation of material to create a uniform channel with a varying bottom width of 5 to 20 feet and 2:1 side slope. The extents of the channel modifications for the Taumata Stream extend from Cross Section Station 7804 to 1522 and for Leaveave Stream they extend from Cross Section Station 13164 to 46.

Alternative B_1 included the same conveyance improvements described in Alternative B above. In addition, it includes construction of a flood barrier. There is approximately 2,400 linear feet of barrier with an average height of seven feet (from ground elevation) on the Taumata Stream and approximately 3,400 linear feet of barrier with an average height of five feet (from ground elevation) on the Leaveave Stream. The extents of the flood barrier for the Taumata Stream extend from Cross Section Station 5280 to 2825, along the left bank, and for Leaveave Stream they extend from Cross Section Station 7694 to 4269, along the right bank. The flood barrier was accounted for in the model by changing the height of the lateral structures for the purpose of determining if it was a viable alternative. The exact alignment of the barrier will be confirmed during the 2022 site visit.

Based on the initial economic analysis and results, these alternatives were further refined and Alternative C was developed to optimize benefits as part of the final array and is the tentatively selected plan. This alternative is depicted in Tafuna FPMS *AltC1* HEC-RAS geometry file (.g03) and included in the Alternative C HEC-RAS with-project plans (.p01, .p02, .p03, .p04, .p05, .p06, .p07, .p14) and future with-project plans (.p01, .p02, .p03, .p04, .p05, .p06, .p07, .p14).

5.1.1 Alternative C

This alternative includes construction of an approximately 2,400-foot-long barrier only (floodwall or levee) along the Taumata Stream, as described in Alternative B_1. In addition, it includes a nonstructural component which includes dry floodproofing of 38 nonresidential buildings and elevation 242 residential structures (assuming 100% participation rate) as these structures will not receive flood protection from the flood barrier. No bridge improvements are proposed as part of the plan. Interior drainage requirements will need to be considered after TSP as the design is further developed.

Due to time constraints the PDT made the risk-informed to decision, to not explicitly model Alternative C. Rather due to the minimal benefits seen with the channel improvements, the economist took the benefits and realized from Alternative B_1 and combined it with the appropriate nonstructural plan. Further discussion regarding this is provided in the Economics Appendix. After the 2022 site visit, when conditions in the hydraulics model can be verified and confirmed, a model for the TSP will be created to confirm assumptions and assure there are no adverse impacts.

5.2 Structure Damage Analysis

Structural Damages were estimated using the Hydrologic Engineering Center Flood Damage Assessment (HEC-FDA) model. Structures within the 0.2% annual exceedance probability (500-year) floodplain of the Tafuna Watershed were included in the analysis. See Appendix B: Economics for a more detailed description of the structure inventory and survey methodology.

5.3 Risks and Uncertainty

In accordance with EM 1110-2-1619 “Risk-Based Analysis for Flood Damage Reduction Studies”, a risk analysis was performed for this study using HEC-FDA. This program uses Monte Carlo simulation to sample the interaction among the various hydrologic, hydraulic, and economic uncertainties. Uncertainties in the hydrology and hydraulics include the uncertainties associated with the discharge frequency curve and the stage-discharge curve. Both of these relationships have statistical confidence bands that define the uncertainty of the relationships at various target frequencies. The Monte Carlo simulations randomly sample within these confidence bands over a range of frequencies until target performance criteria are met. Reliability statistics are based on the results of the Monte Carlo simulations. Based on Table 4-5 in EM 1110-2-1619, equivalent record length was represented graphically using an equivalent record length of 20 years. A detailed discussion of the risk and reliability analyses can be found in Appendix B: Economics.

In accordance with Planning Bulletin 2019-04, Incorporating Life Safety in to Planning Studies, the PDT evaluated potential life safety risks during the development of the Tentatively Selected Plan using LifeSim. Post TSP, several breach scenarios of the flood barrier will be analyzed in the HEC-RAS model. Gridded data output data from HEC-RAS model (with terrain, arrival time and depth grids) were used in the LifeSim analysis. Currently, only the without project conditions have been analyzed in LifeSim

6 Summary

Results from the hydraulic model were reviewed for compliance with federal regulations. The tentatively selected plan is expected to reduce losses due to flooding however residual risks exist within the watershed. While sea level changes were considered during the plan formulation process, uncertainty with those projections exist and risk remains. Table 6-1 summarizes residual risk associated with the tentatively selected plan specifically due to the potential for a changing climate.

Table 6-1. Climate Risk Register

Feature or Measure	Trigger	Hazard	Consequence	Qualitative Likelihood
Levee/ Floodwall Heights	Increased water surface elevations in levee/floodwall areas due to sea level change	Reduced assurance on levee/ floodwalls; increased probability of overtopping	Flooding of protected area, economic damages and transportation delays	Unlikely – SLC projects increases in the future; however the TSP is outside the coastal influence zone.
Levee/ Floodwall Heights	Increased water surface elevations in levee/floodwall areas due to higher intensity rainfall	Reduced assurance on levee/ floodwalls; increased probability of overtopping	Flooding of protected area, economic damages and transportation delays	Unlikely – since there are no increasing precipitation trends.

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