Baker Creek Hydrologic Analysis

McMinneville, OR

Prepared for:

Friends of Baker Creek
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May 2019
PBS Project 71440.000

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CERTIFICATE OF ENGINEER

Baker Creek Hydrologic Analysis

Final Report

The technical information and data contained in this report was prepared under the direction and supervision of the undersigned, whose seal, as a professional engineer licensed to practice as such, is affixed below.

REGISTERED PROFESSIONAL ENGINEER
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OREGON
JULY 10, 2018
JUSTIN R MAYNARD
EXPIRES: **12/31/2020**

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1 EXECUTIVE SUMMARY

Friends of Baker Creek has retained PBS Engineering and Environmental, Inc. (PBS) to perform a hydrologic analysis of Baker Creek and evaluate the potential floodplain impacts of recent and future development. The Baker Creek watershed is situated above the northern limits of the City of McMinnville. Previous studies commissioned by FEMA and/or the local communities of McMinnville and Yamhill County established flood hazard areas, published originally in 1983 and modernized without re-study in 2010. In recent years, residents have observed flood waters encroaching into FEMA-defined 0.2-percent annual chance, Zone X flood hazard areas on a relatively frequent basis.

The purpose of this study is to investigate factors affecting the extents of the floodplain around Baker Creek and to examine their potential impact on effective FEMA SFHA delineations, which were developed in the early 1980s. A variety of such factors exist, including:

- Data availability
- Changes in Baker Creek watershed land use
- Geomorphological changes to the Creek
- Grading activities and development

Effective floodplain delineations were created based on the limited data available at the time of the detailed study and contemporary land use and elevation data. This study is founded on recent gaging statistics and watershed conditions and shows that a number of factors may be the cause of recently observed encroachment of flood waters into the mapped 500-year flood zones.

The analysis herein supports the development of calibrated hydrologic and hydraulic models. The models have been verified anecdotally using photos provided by residents, along with nearby rainfall gage data to establish the proximity of the historical events to a 2-year, 24-hour rainfall event. It's apparent based on the extent of such frequent inundation illustrated by modern hydraulic models that the effective flood insurance rate maps are in need of revision, based on current data and methodology.

2 INTRODUCTION

2.1 Setting and History

Baker Creek is part of a system of tributaries of the nearby Yamhill River. The vast majority of its approximately 26 square mile watershed is undeveloped and forested land, but the lower reach of Baker Creek flows through some farm land in Yamhill County, then along the northern incorporated limits of McMinnville, adjacent to residential developments.

FEMA's existing mapping for the area were updated in 2010 with modern background imagery and flood hazard designations. The upstream reach of the Creek is delineated as Zone A, with no specific study to establish base flood elevations (BFEs). The area adjacent to urban development was studied in detail and mapped as Zone AE with BFEs and regulatory floodway defined within the Creek's banks. Flood Insurance Rate Map (FIRM) information is provided in Table 1.

Table 1: Effective FEMA FIRM Panel Information

FIRM Panel	Community	Effective Date	Flood Zone	BFEs
41071C 0402D	Yamhill County City of McMinnville	3/2/2010	Zone AE	Yes
41071C 0401D	Yamhill County City of McMinnville	3/2/2010	Zone A/AE	No

The 2010 FIRM update simply modernized existing special flood hazard areas (SFHAs) and BFEs based on updated elevation data and aerial imagery. The detailed study that established the BFEs shown on the maps was actually performed prior to or concurrently with the 1983 effective mapping, as noted in the historical Flood Insurance Study (FIS). A vicinity map with effective SFHAs for a 100-year event is provided in Figure 1.

Residents have indicated that two storm events in the Winter of 2018-2019 caused flood waters in the creek to rise beyond 100-year levels shown on the FIRM panels. These storms were not characterized by exceptional rainfall depths. This study is intended to identify potential reasons for the relatively frequent encroachment of flood waters into the FEMA-defined Zone X SFHA (mapped as 0.2% Chance Flood Areas).

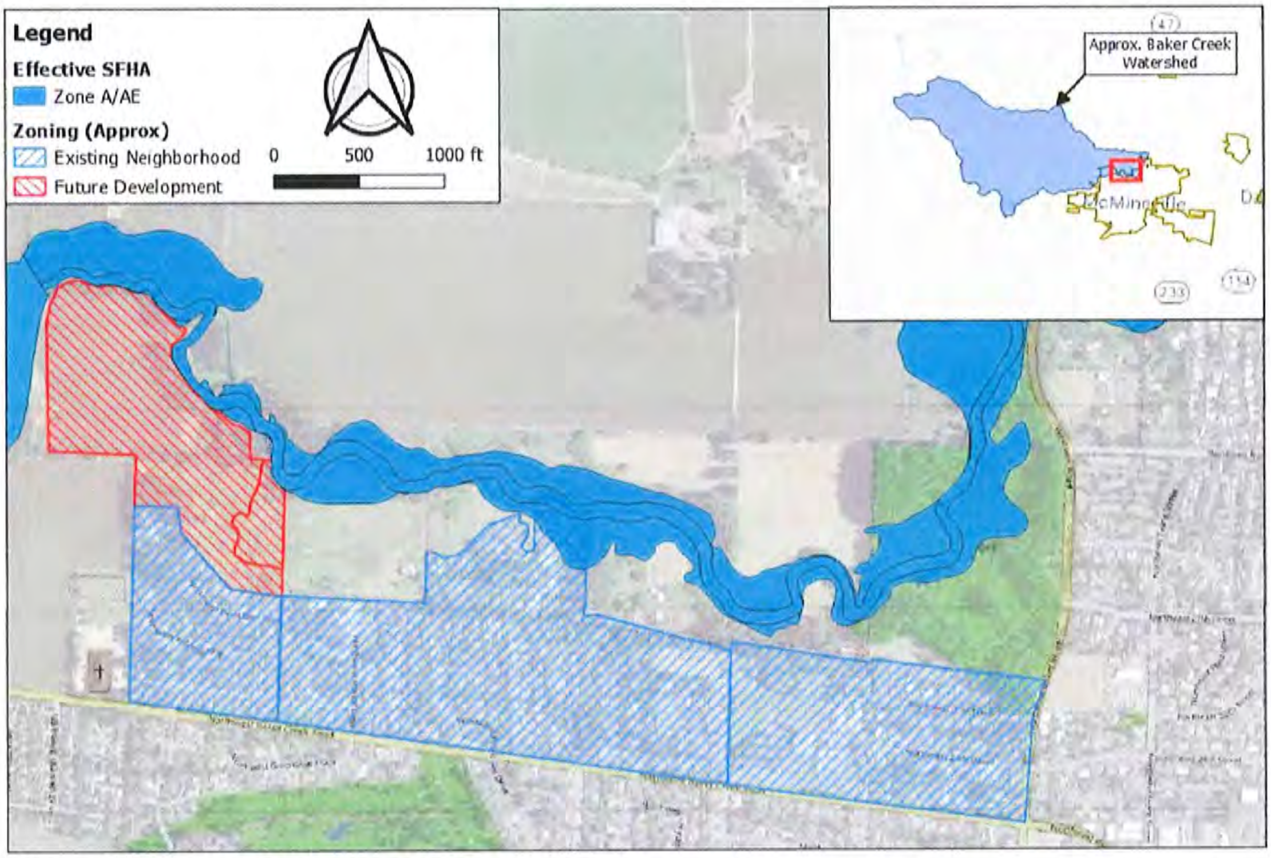

Figure 1: Project Vicinity Map and Watershed Area.

2.2 Local Floodplain Regulation

The City of McMinnville, like many other communities, restricts development within any 100-year floodplain. The City regulates floodplain development activities by way of Title 17 ("Zoning") of the City of McMinnville Municipal Code (CMMC). Specifically, Section 17.48 ("F-P Flood Area Zone"), states that the purpose of limiting activities within designated flood zones is to "protect the community from financial burdens through flood damage losses." Areas regulated by the City's code include all property within the City lying within FEMA-designated SFHAs.

The City's code further states that the permitted uses within an "F-P" zone are limited to farming, public parks/recreation (without structures), and sewage pump stations. Other conditional uses are allowed, subject to CMMC 17.48.040 and 17.48.045. In general, encroachment of structures or fill into the floodplain will not be allowable that cause an increase in flood height or that induce hazardous velocities.

3 HYDROLOGY

This analysis utilizes an empirical Curve Number (CN) approach to estimating unsteady runoff hydrographs. This methodology was developed by the Soil Conservation Service (SCS), now known as the United States Department of Agriculture (USDA). The empirical CN values are utilized to quantify the direct runoff rate caused by a rainfall event based on soil types and land cover in the crossing's tributary watershed area. HEC-HMS hydrologic modeling software was used to develop runoff hydrographs based on regional rainfall statistics and storm distributions described herein.

3.1 Watershed Delineation

Hydrologic analysis is based on the use of LiDAR-derived, gridded topographic data, obtained from the Oregon Department of Geology and Mineral Industries (DOGAMI). This elevation dataset has been utilized to ensure that Baker Creek's tributary drainage area is accurately considered by the hydrologic modeling software.

USGS StreamStats was initially utilized to develop a watershed area delineation (shown in Figure 1), encompassing the land area contributing runoff to the Creek to a point downstream of the neighborhoods. Based on observations of the topography, satellite imagery, and engineering judgement, the tributary watershed was broken into multiple sub-watersheds to create inputs for the hydraulic models (Section 4). The result of sub-watershed delineation is shown in Figure 2.

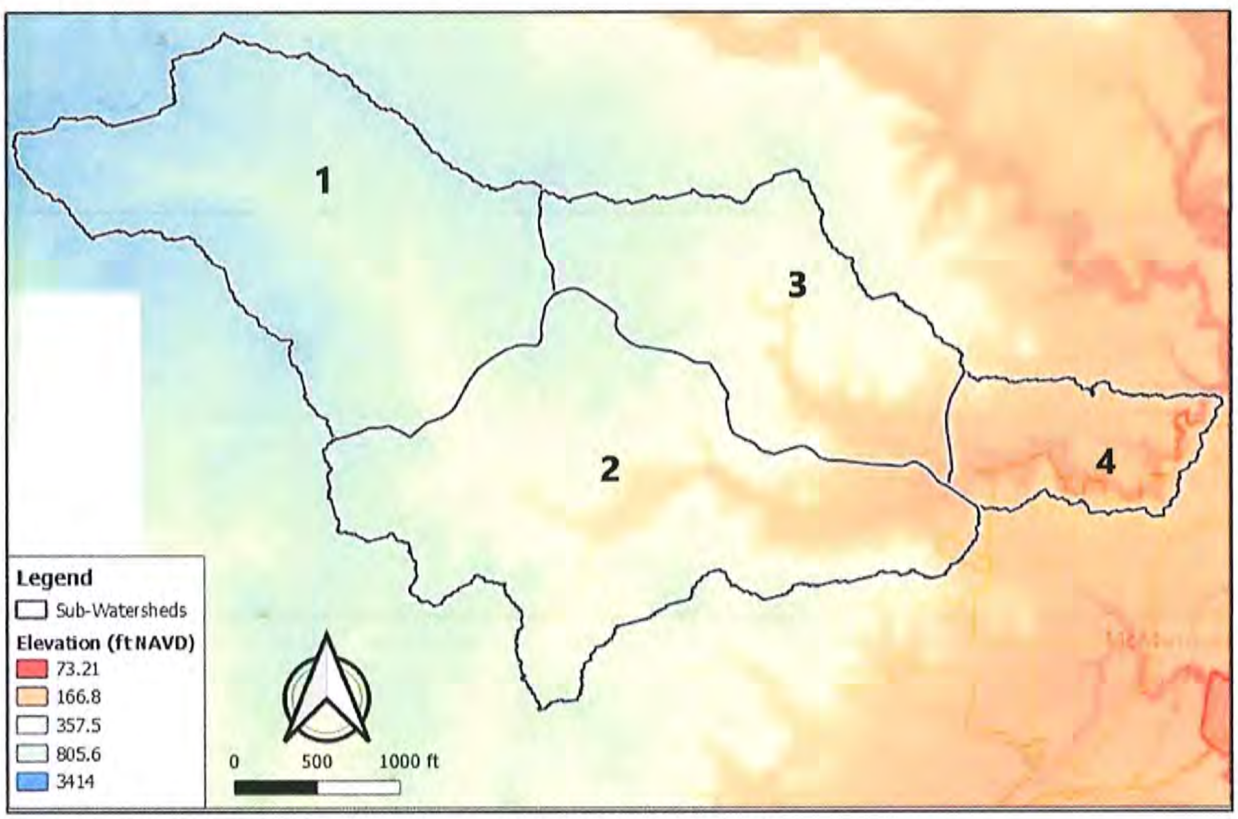

Figure 2: Sub-Watershed Delineation and LiDAR Elevation Grid.

The hydrologic model is intended to capture the full tributary drainage area of the creek affecting the neighborhoods with the highest chance of being impacted by changing floodplains. The full watershed area is approximately 25.8 square miles in size, with variable land cover conditions through. General characteristics of Sub-watershed areas are described in Table 2.

Table 2: Sub-Watershed Summary

Sub-Watershed ID	Description
1	Middle Reach (South Branch), Primarily Forested
2	Middle Reach (North Branch), Forested with Rural Residential
3	Lower Reach, Farm Land and Some Urban Residential
4	Upper Reach, Forested with Higher Conductivity Soils

3.2 Soil and Land Cover Data

Baker Creek lies within the Yamhill Hydrologic Unit. Soil data was acquired from the Soil Survey Geographic Database (SSURGO), maintained by the US Department of Agriculture (USDA). According to the SSURGO database, soils within the Baker Creek watershed are identified primarily as clay loam and silty clay loam. Hydrologic Soil Groups (HSG) within the watershed are highly variable, ranging in infiltration capacity from very limited infiltration capacity (Type D) to moderate infiltration capacity (Type B) for purposes of hydrologic and hydraulic modeling, as shown in Figure 3.

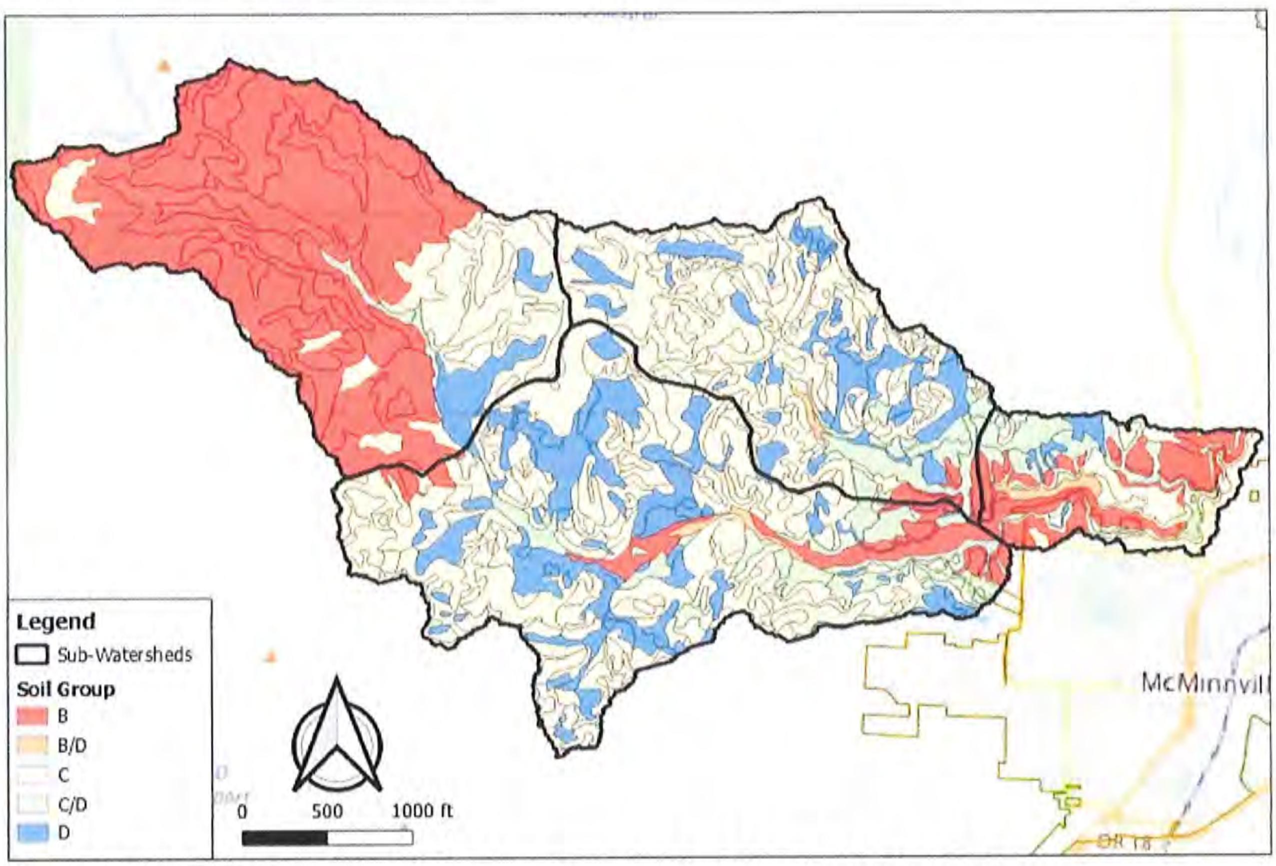

Figure 3: SSURGO Database Soil Units in the Finnegan Road Crossing Drainage Area.

For the purposes of establishing Curve Number parameters, Type "B/D" soils will be considered Type C. An areal summary of HSGs present in the watershed is provided in

Table 3: Hydrologic Soil Group Summary.

HSG	Area (Acre)	Percent of Total
B	5,048	30.5%
C	7,906	47.8%
C/D	1,358	8.2%
D	2,218	13.4%
Grand Total	**16,529**	**100%**

The National Landcover Database (NLCD), based on Landsat imagery and published by the Multi-Resolution Land Characteristics Consortium, indicates that the watershed is dominated by Grassland and Forested cover types with urban development in the City of McMinnville situated in the watershed's lowest reaches. NLCD information is shown in Figure 4.

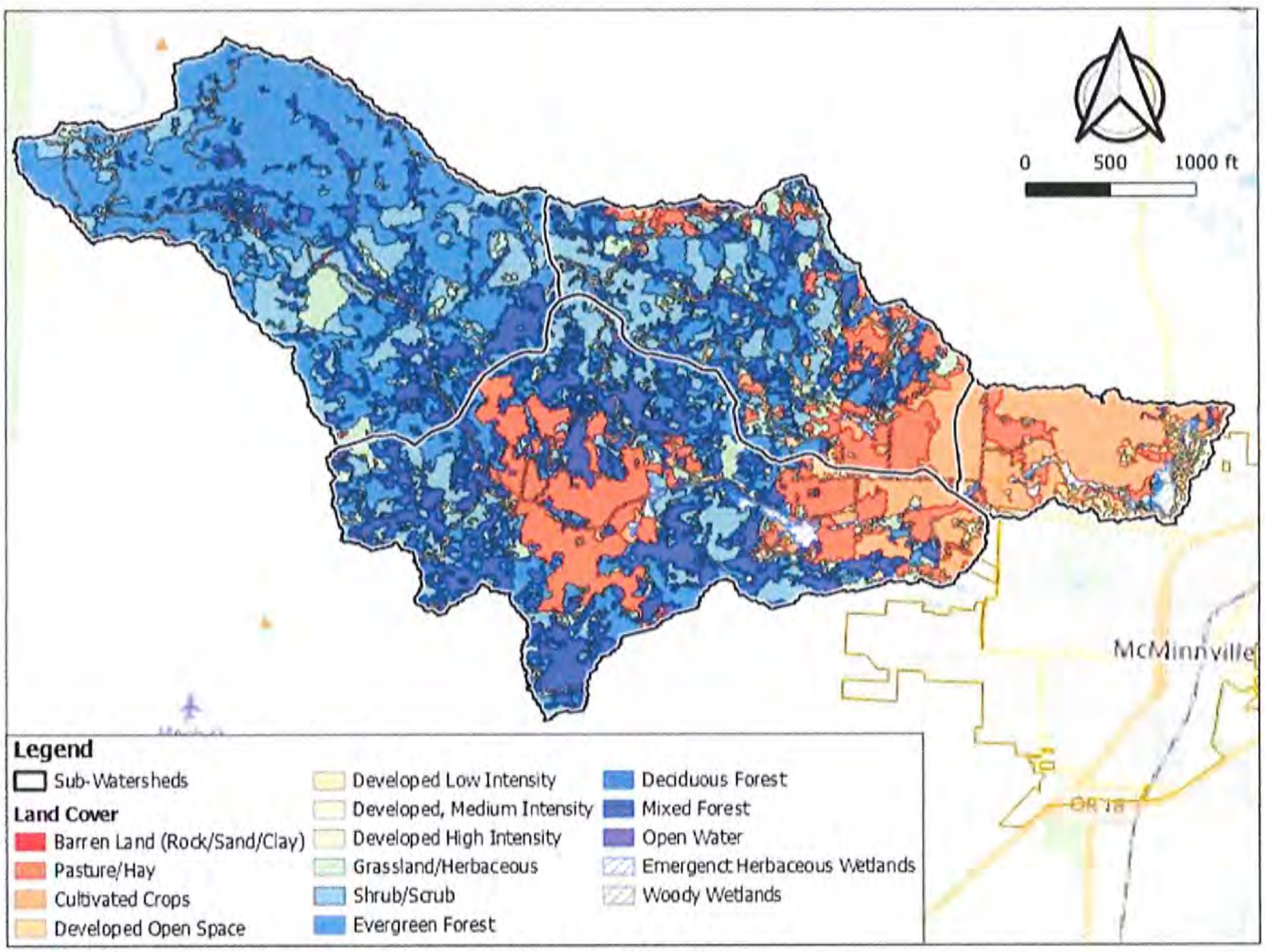

Figure 4: Landsat Data Within the Drainage Area Boundary

A breakdown of watershed land cover areas is provided in Table 4. The upper reach of the watershed (Sub-Watershed 1) is mostly forested with some open grassland, while the middle reaches (Sub-Watersheds 2 and 3) also include some pasture cover. The lowest reaches are nearly entirely characterized by open space, agriculture, and urban development.

Table 4: Watershed Land Cover Summary

Land Cover	Area (Acres)	Percent of Total
Barren Land (Rock/Sand/Clay)	9	0.05%
Pasture/Hay	1,962	11.87%
Cultivated Crops	1,357	8.21%
Developed Open Space	627	3.79%
Developed Low Intensity	202	1.22%
Developed Medium Intensity	60	0.36%
Developed High Intensity	7	0.04%
Grassland/Herbaceous	890	5.38%
Shrub/Scrub	2,768	16.75%
Deciduous Forest	341	2.06%
Evergreen Forest	4,880	29.52%
Mixed Forest	3,207	19.40%
Open Water	2	0.01%
Emergent Herbaceous Wetlands	18	0.11%
Woody Wetlands	201	1.21%
Grand Total	**16,529**	**100%**

3.3 Curve Number Estimates

Initial Curve Number estimates were based on published tables, depending upon soil and land cover characteristics within each sub-watershed. Based on land use types present in the watershed, the Curve Number Values shown in Table 5 were applied.

Table 5: Published Curve Number Values from TR-55 Applied to Drainage Areas

Land Cover Type	Hydrologic Soil Group				
	A	B	C	C/D	D
Barren Land (Rock/Sand/Clay)	59	74	82	84	86
Cultivated Crops	49	69	79	81.5	84
Deciduous Forest	43	65	76	79	82
Developed High Intensity	77	85	90	91	92
Developed Low Intensity	54	70	80	82.5	85
Developed Open Space	39	61	74	77	80
Developed, Medium Intensity	60	74	82	84	86
Emergent Herbaceous Wetlands	100	100	100	100	100
Evergreen Forest	43	65	76	79	82
Grassland/Herbaceous	30	58	71	74.5	78
Mixed Forest	43	65	76	79	82
Open Water	100	100	100	100	100
Pasture/Hay	39	61	74	77	80
Shrub/Scrub	35	56	70	73.5	77
Woody Wetlands	100	100	100	100	100

3.4 Time of Concentration

Curve number methodology requires that a time of concentration be estimated for each watershed in order to apply the unit hydrograph and calculate runoff. The time of concentration is defined as the time required for runoff from the most remote point in the watershed to reach the watershed's point of concentration, or its most downstream point. The SCS method prescribes a watershed lag method for calculating time of concentration as follows:

$$T_c = \frac{L^{0.8}(S+1)^{0.7}}{1{,}900 * Y^{0.5}}$$

Where: T_c = Time of Concentration (hours)
L = Longest Flow Path (LFP) length (feet)
$S = \frac{1{,}000}{CN} - 10 =$ Maximum potential retention (inches)
Y = Average watershed land slope (%)

Calculation of the maximum potential retention parameter requires the CN value for the watershed, estimated as described in Section 0. This equation has been developed to represent the time of concentration for watersheds of varying type and size.

3.5 Hydrologic Model

The parameters estimated for each watershed, including area, time of concentration, and curve number were used as inputs to the HEC-HMS computer model, which includes the SCS Unit Hydrograph method for developing runoff hydrographs. Sub-watershed parameters are provided in Table 6. A schematic of the model is shown in Figure 5.

Table 6: Sub-Watershed Hydrologic Model Input Properties

Sub-Watershed	Area (Square Mile)	Curve Number	Longest Flow Path (ft)	Ave Slope (%)	T_c (hour)	Downstream Routing Length (ft)	Downstream Channel Slope (%)
1	8.60	66	27,690	17.05	1.61	31,550	0.92
2	9.47	76	36,100	12.93	1.77	4,225	0.21
3	5.52	75	28,870	11.51	1.59		
4	2.25	77	7,500	3.77	0.90	N/A – to Hydraulic Model	

Figure 5: Schematic of the Hydrologic Model with Routing Through Urban Residential.

The model has been developed to analyze the 100-year, 24-hour rainfall event, upon which Zone A/AE SFHAs are generally defined, and the 2-year, 24-hour rainfall event. The rainfall depths for these events are based on ODOT Precipitation Frequency data. The rainfall depths are provided in Table 7

Table 7: Rainfall Event Depths Based on ODOT Precipitation GIS Data

Sub-Watershed	2-year, 24-hour	100-year, 24-hour
1	4.17	7.66
2	3.07	5.77
3	3.00	5.65
4	2.50	4.81

The HEC-HMS model does not provide a comprehensive output report; however, the model is available for review upon request.

3.6 Calibration

To ensure that basin data incorporated into the hydrology model produce reasonable results, a calibration can be performed. The most straightforward approach to calibrating runoff parameters is to compare peak flow results to stream gage statistics. Because Baker Creek is an un-gaged watershed, it is difficult to provide a direct calibration. Some nearby gages resting at the point of concentration of somewhat similar watersheds have been identified that can aid in calibrating the model. The USGS has also published regression equations intended to calculate discharge for rural, unregulated streams in Western Oregon (*Estimation of Peak Discharges for Rural, Unregulated Streams in Western Oregon*, 2005). The publication constitutes a comprehensive analysis of 418 gaged watersheds in Western Oregon and a vast improvement in methodology over the studies upon which the effective flow rates were based.

Similar nearby watersheds are summarized in Table 8. Runoff curve numbers and lag times were estimated for these three watersheds for input into the HEC-HMS model. Results are summarized in Table 9. The peak 100-year flows estimated by the hydrologic model for the two most similar basins (Tualatin River and Scoggins Creek) appear to be lower than their gage statistics. This suggests that the generalized curve numbers from TR-55 are not necessarily representative of actual watershed conditions.

Table 8: Nearby Watersheds Used for Calibration.

Watershed/Gage	100-yr, 24-hr Rainfall (in)	Mean Elevation (ft NAVD)	Area (sq mi)	Estimated CN	Lag (Hr)	Gage 100-year Peak Flow (cfs)*	Regression Peak Flow (cfs)
Butte Creek at Monitor, OR	6.34	1,760	59.1	71	2.39	9,090	8,800
Tualatin River near Gaston, OR	6.48	1,335	48.5	69	3.22	6,830	6,800
Scoggins Creek near Gaston, OR	6.90	1,560	16.1	67	2.16	3,720	2,700
Baker Creek	6.29	780	25.8	74	1.16	N/A	3,150

*Based on Log-Pearson Type III statistical distribution, applied by the USGS (*Estimation of Peak Discharges for Rural, Unregulated Streams in Western Oregon*, 2005)

Table 9: Western Oregon Peak Flow Estimates Based on Regression Equations.

Watershed	Modeled Peak Flow (cfs)	Model Diff. From Gage	Modeled Diff. From Regression
Butte Creek at Monitor, OR	7,076	-22%	-19%
Tualatin River near Gaston, OR	6,350	-7%	-7%
Scoggins Creek near Gaston, OR	2,730	-36%	1%
Baker Creek	4,720	N/A	50%

In order to better match peak flow statistics for each watershed, the curve number values were adjusted, and lag times recalculated until the modeled peak flows were more representative of gage statistics. Of the modeled calibration watersheds, Butte Creek and Tualatin Creek watersheds are most similar in composition to the Baker Creek drainage area.

A comparison of soil types and land use composition of each watershed is provided in Table 10 and Table 11.

Table 10: Hydrologic Soil Group Composition of Each Modeled Watershed

HSG	Baker	Butte	Tualatin	Scoggins
A	0%	6%	0%	0%
B	31%	44%	69%	86%
C	48%	31%	29%	14%
C/D	8%	3%	1%	0%
D	13%	16%	0%	0%

Table 11: Aggregated Land Cover Composition of Each Modeled Watershed

Land Cover Description	Baker	Butte	Tualatin	Scoggins
Barren Land (Rock/Sand/Clay)	0%	0%	1%	2%
Cultivated Crops	8%	5%	2%	0%
Developed Covers	5%	1%	4%	2%
Wetland (Emergent/Woody)	1%	1%	1%	0%
Forested Covers	51%	60%	60%	40%
Grassland/Pasture/Shrub	34%	33%	33%	57%
Open Water	0%	0%	0%	0%

Ultimately, curve numbers were adjusted to the values shown in Table 12.

Table 12: Curve Number Adjustment to Match Gage Statistics.

Watershed	Calibrated CN	CN Adjustment	Calibrated Model Peak Flow (cfs)	Diff From Gage Statistics
Butte Creek at Monitor, OR	76	7.0%	8,840	-3.0%
Tualatin River near Gaston, OR	71	2.9%	7,000	2.5%
Scoggins Creek near Gaston, OR	73	8.9%	3,740	0.5%
Baker Creek	78	**5.0%**	**6,170**	**N/A**

While USGS regression equations provide a useful benchmark to ensure flow estimates are reasonable, they do not take complex conditions such as land use, soil composition, and watershed shape into consideration. In the Willamette Valley region, prediction equations are based on watershed size, slope, and 2-year precipitation intensity. This analysis indicates that the regression equations may not be a good fit for a watershed with such high concentrations of Type C and D soils and the extent of cultivated crops and development in the lower portion of the watershed.

The Flood Insurance Study (FIS) indicates the 1% annual chance peak discharge for the 26 square mile watershed is approximately 2,030 cfs. Flows estimated by both the hydrologic model and by USGS regression equations are significantly higher than those presented in the effective FIS. See the Conclusion section for further discussion of flow rate discrepancies between current modeling methods and the 1983 FIS.

Anecdotal evidence of frequent flooding has also been supplied by residents near the Creek. Namely, November 2015 storm events caused flood waters to approach residents and encroached on back yards. This anecdotal evidence, along with Portland area rainfall gage records, can provide an empirical model verification. This is further discussed in Section 3.2.2.

4 HYDRAULIC ANALYSIS

Runoff hydrographs produced by the hydrologic analysis detailed in Section 3 are used as inputs to HEC-RAS 5.0.6 hydraulic modeling software. A simple, 1-dimensional model of the reach in the vicinity of the neighborhoods was developed to analyze water surface elevations (WSEL) and approximate floodplain extents for existing and future conditions. This model is used to estimate the impact of higher flows on special flood hazard area boundaries, compared with the original FIS

4.1 Model Development

Aerial imagery and LiDAR topography have been used to define the extents of a 1-dimensional modeling area that will encompass the full extents of flow in Baker Creek. The model uses a river centerline defined along the low flow channel alignment, with cross sections defined generally perpendicular to the flow path. The model geometry is shown in Figure 6.

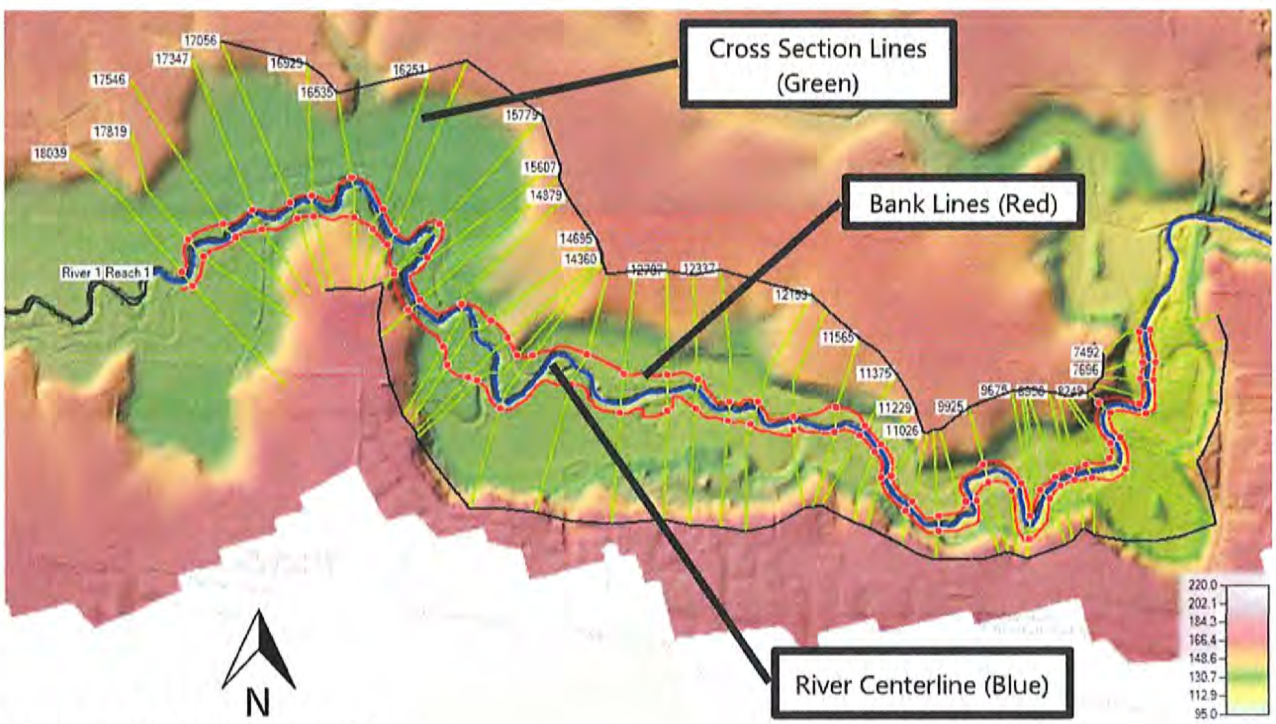

Figure 6: 1-D Model Geometry

A 1-dimensional approach has been selected due to the relatively defined incision of the channel. Because of the channel definition, there is no loss of flow from the 1-dimensional model sections that would warrant a more complex modeling approach with 2-dimensional overbank areas.

4.1.1 *Manning's 'n' Roughness*

In general, the modeled channel reach is characterized by relatively heavy vegetation in the immediate vicinity of the channel and field in the overbanks. Some specific areas were identified that differ from the dominant land cover, including more heavily vegetated overbanks. Published Manning's 'n' roughness values from Chow (1959) have been assigned to the 1-D cross sections based on the predominant ground cover through each reach.

Ground cover and 'n' values utilized by the model are provided in Table 13. Manning's n refinement

Table 13: Manning's 'n' Roughness Values for Modeled Channel Reach

Land Cover	Manning's 'n' Value
Channel (Clean)	0.035
Channel (Heavy Vegetation)	0.05
Overbank Fields	0.025

4.1.2 *Boundary Conditions*

Boundary conditions must be defined for inflows and outflows to the 1-dimensional channel model. Inflows are defined by steady flow for the effective FIS flow rate, while unsteady hydrographs taken from the HEC-HMS model act as inflow boundaries to the existing condition model. The unsteady model includes two hydrographs: one at the upstream end, representing inflows from the 23.6 square mile upstream watershed and a second distributed uniformly throughout the modeled reach, representing local inflow to the reach from a 1.4 square mile area. Unsteady inflow hydrographs from HEC-HMS are shown in Figure 7.

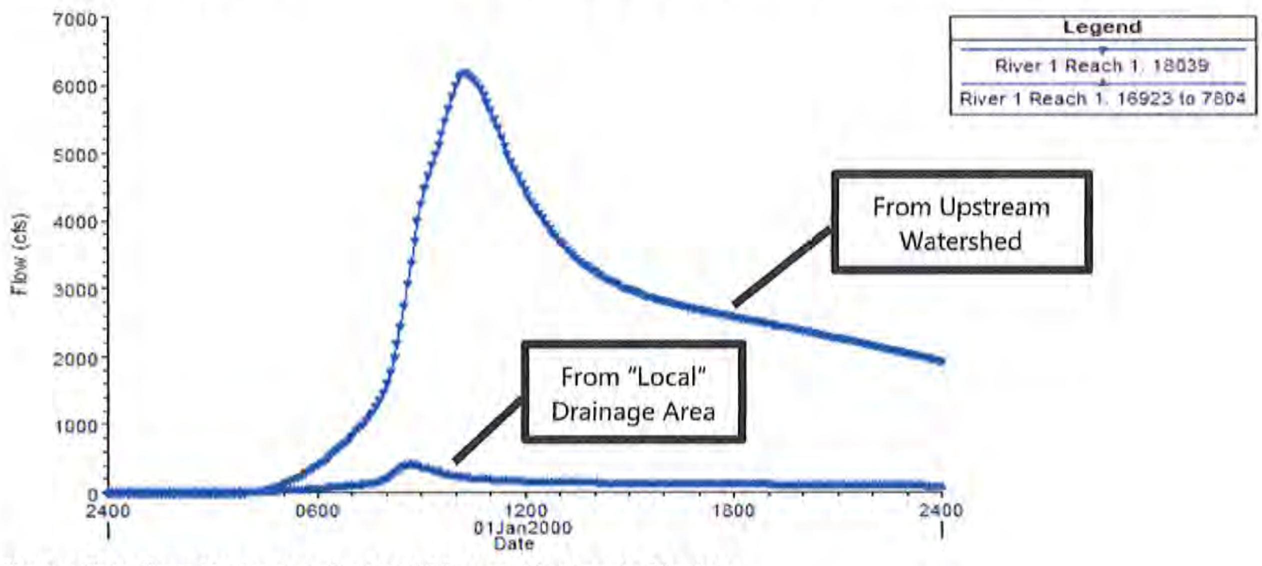

Figure 7: Inflow Hydrographs for 100-year Unsteady Models.

For the steady state effective model, a normal depth condition is based on upstream and downstream main channel friction slopes of 0.5% and 0.1%, respectively. These slopes were estimated based on a profile extending approximately 500 feet beyond the upstream and downstream ends of the model. Outflows from the unsteady existing condition model are defined by a normal depth flow condition based on the downstream channel slope of 0.1%. It's important to note that this study does not consider any tailwater impact due to the Yamhill River, downstream. Flood Insurance Studies indicate that backwater effects of the Yamhill River would not extend upstream into the reach analyzed by this hydraulic model.

4.1.3 Structures

The modeled reach included two water crossing structures in the effective water surface profiles. These structures are completely inundated on the effective 100-year flood mappings and are assumed not to have a significant impact on depths for the purpose of this analysis. A bridge also exists downstream of the model at NW Westside Road. If a letter of map change is submitted, these structures should be added to the model to ensure full compliance with FEMA's modeling requirements concerning hydraulic structures.

4.2 Model Results

4.2.1 Effective Flow Results

The effective flow rate of 2,030 cfs was input into the HEC-RAS model in order to compare the floodplain to the effective SFHAs. Model results extracted from HEC-RAS for this steady, 1-dimensional analysis consist of continuous, gridded results for the peak flood stage across the inundated areas. Figure 8 illustrates the gridded maximum depth result around the neighborhood, compared with existing SFHA boundaries. Figure 9 provides an overlay of modeled effective flow depths compared with the boundary of proposed development northwest of the neighborhoods.

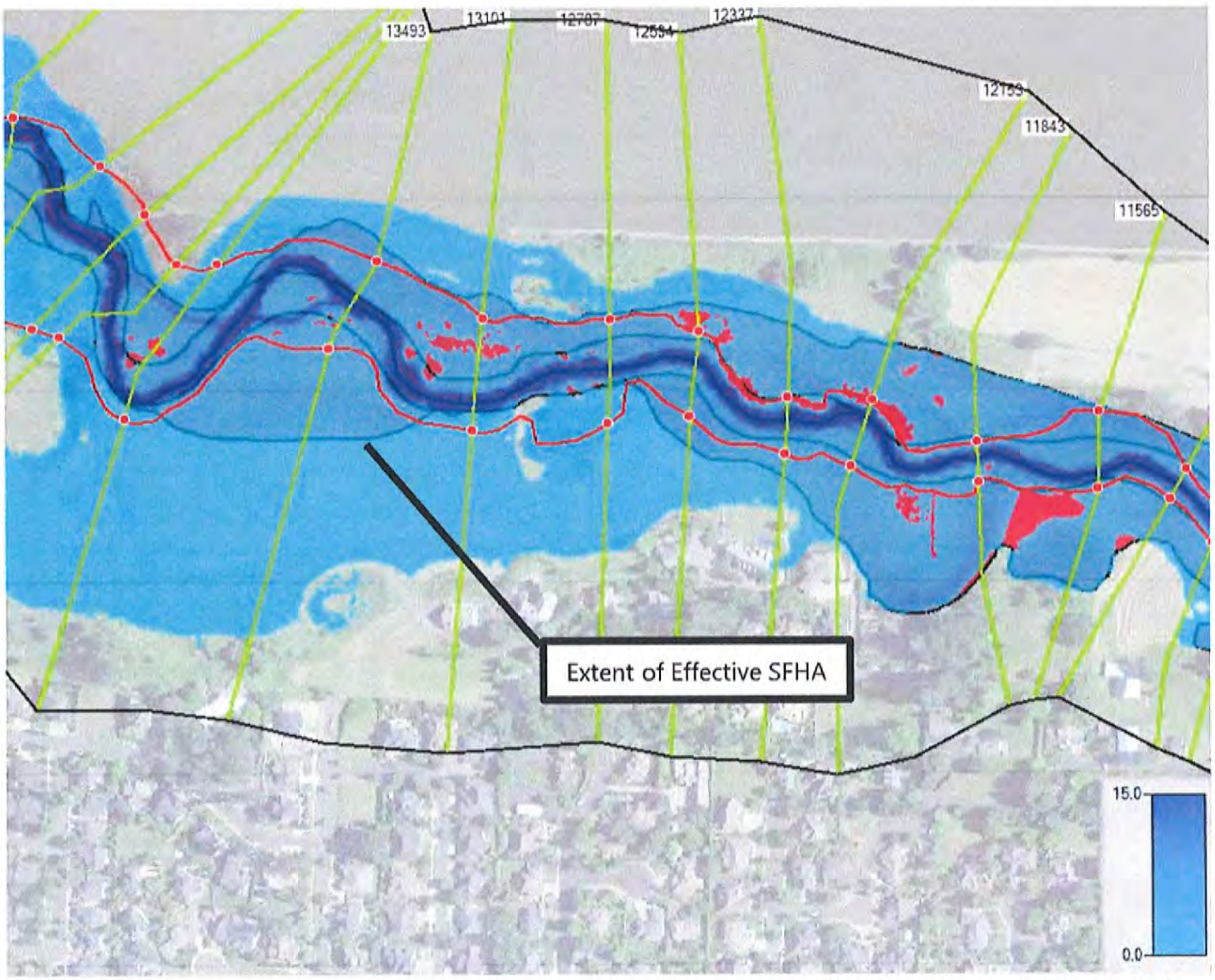

Figure 8: Effective Steady Peak Flow Depth Grid Result (in Feet) with Effective Zone AE SFHA (in Red) near Existing Residential Development.

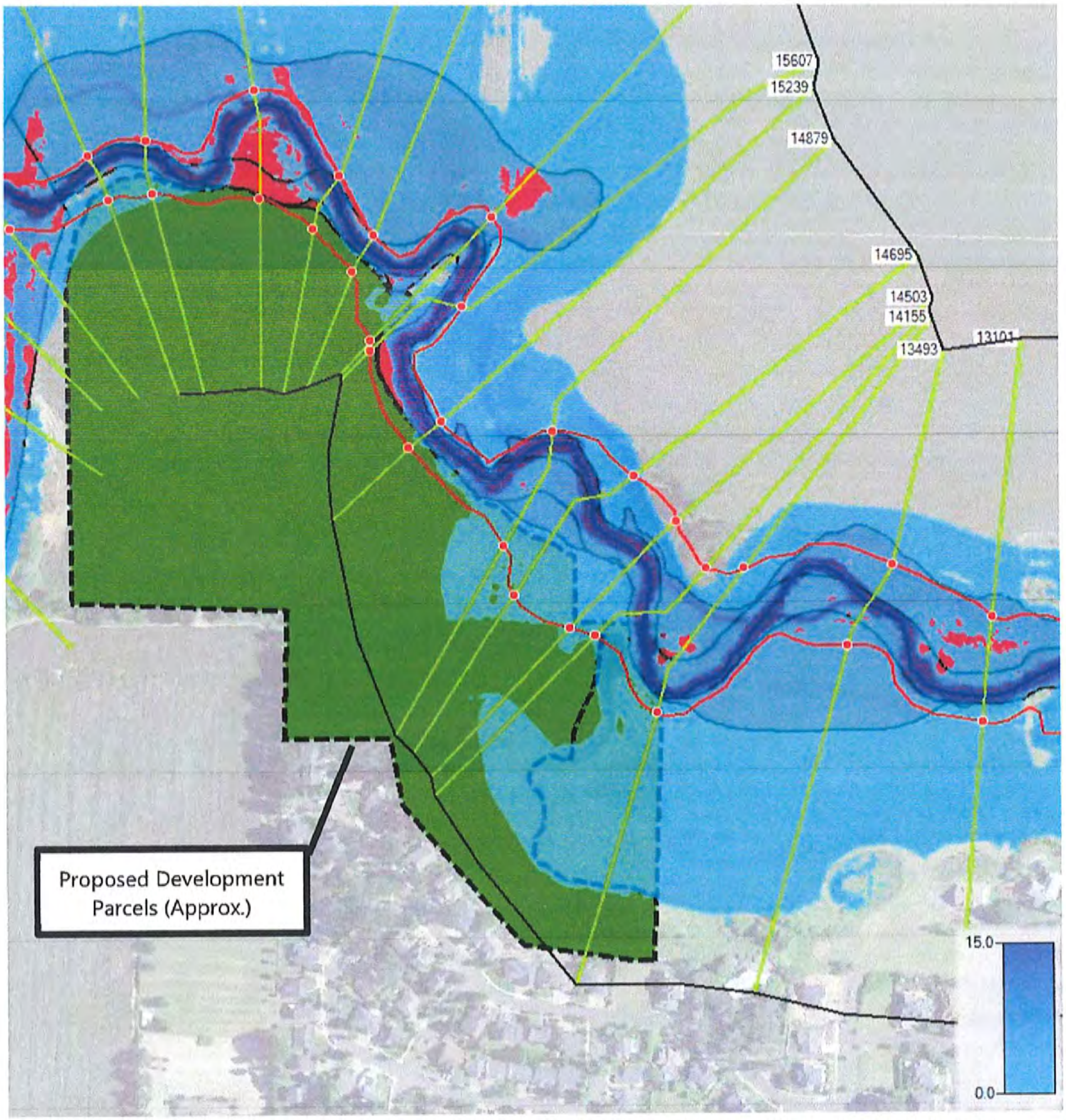

Figure 9: Effective Steady Peak Flow Depth Grid Result (in Feet) at Proposed Development Parcels.

4.2.2 Existing Condition

While a steady model was applied based only on the peak flow rate for the effective map, the hydrologic model used to analyze existing conditions produces unsteady flood hydrographs. The gridded depth results for the existing condition is shown in Figure 10 and Figure 11.

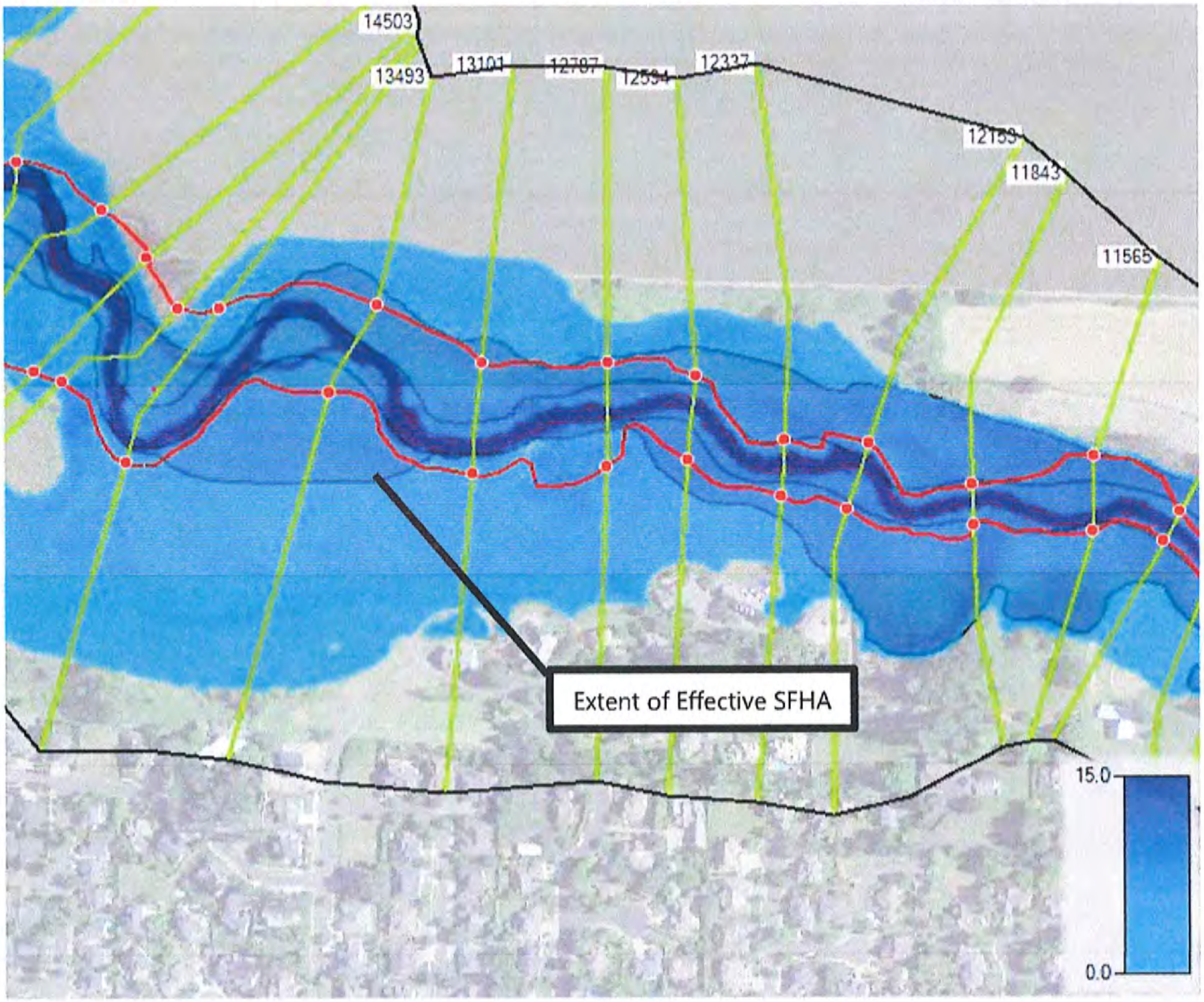

Figure 10: Existing 100-year Maximum Depth Grid (in Feet) near Existing Residential Development.

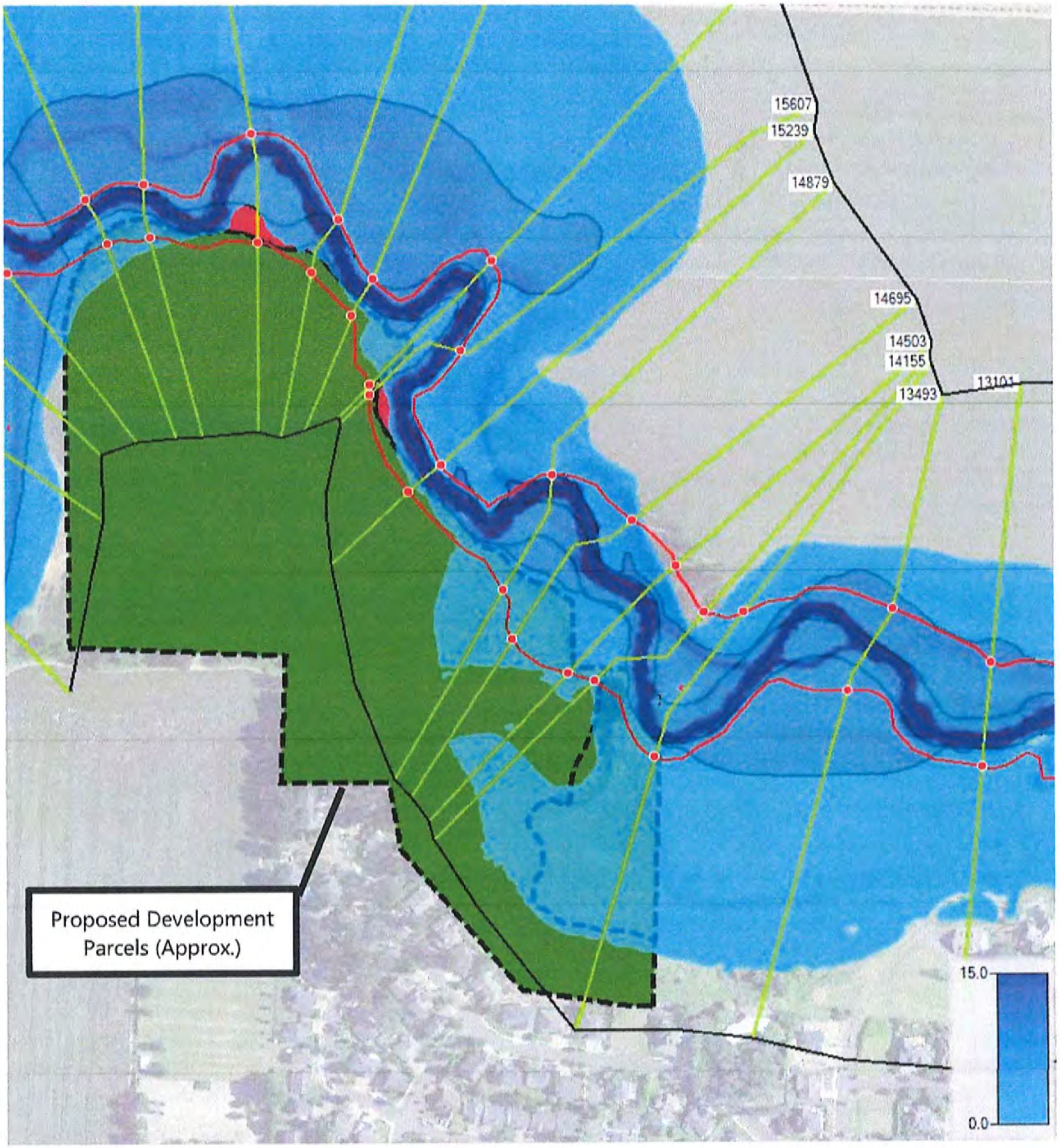

Figure 11: Existing 100-year Maximum Depth Grid (in Feet) at Proposed Development Parcels.

Cross sectional peak water surface elevation results are compared for the steady model of the effective flow rate and the unsteady model representing modern statistics and watershed conditions in Figure 12 and Figure 13.

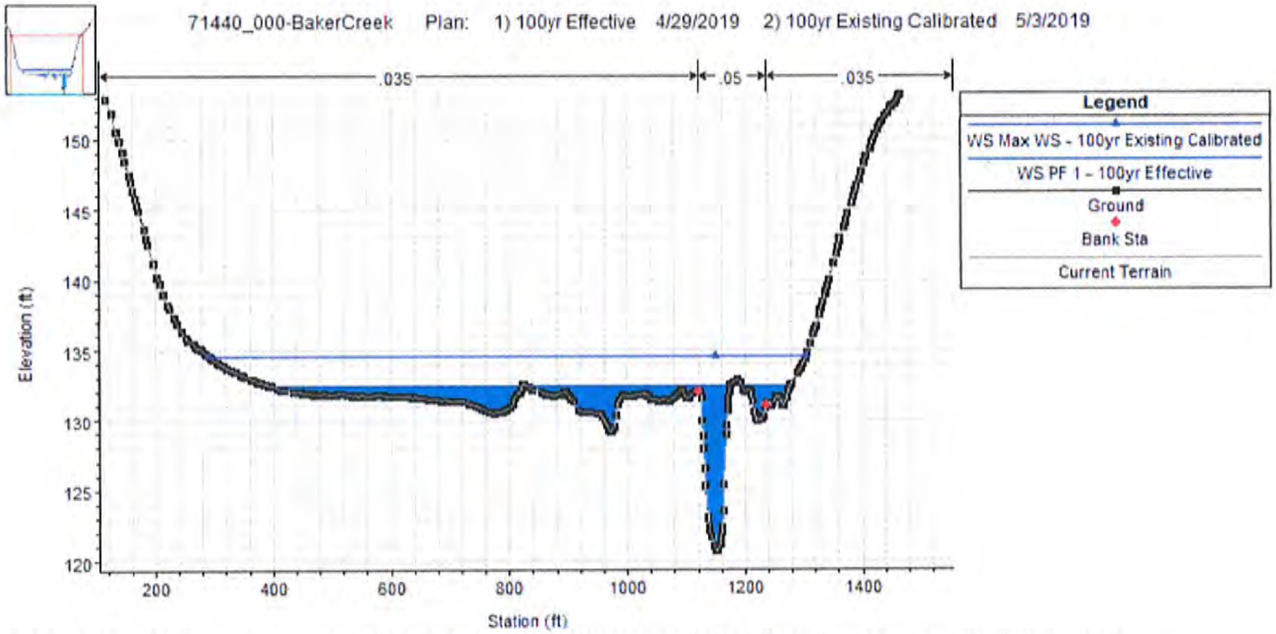

Figure 12: Comparison of Effective and Existing Water Surface at Upstream end of Proposed Development (Cross Section 17056)

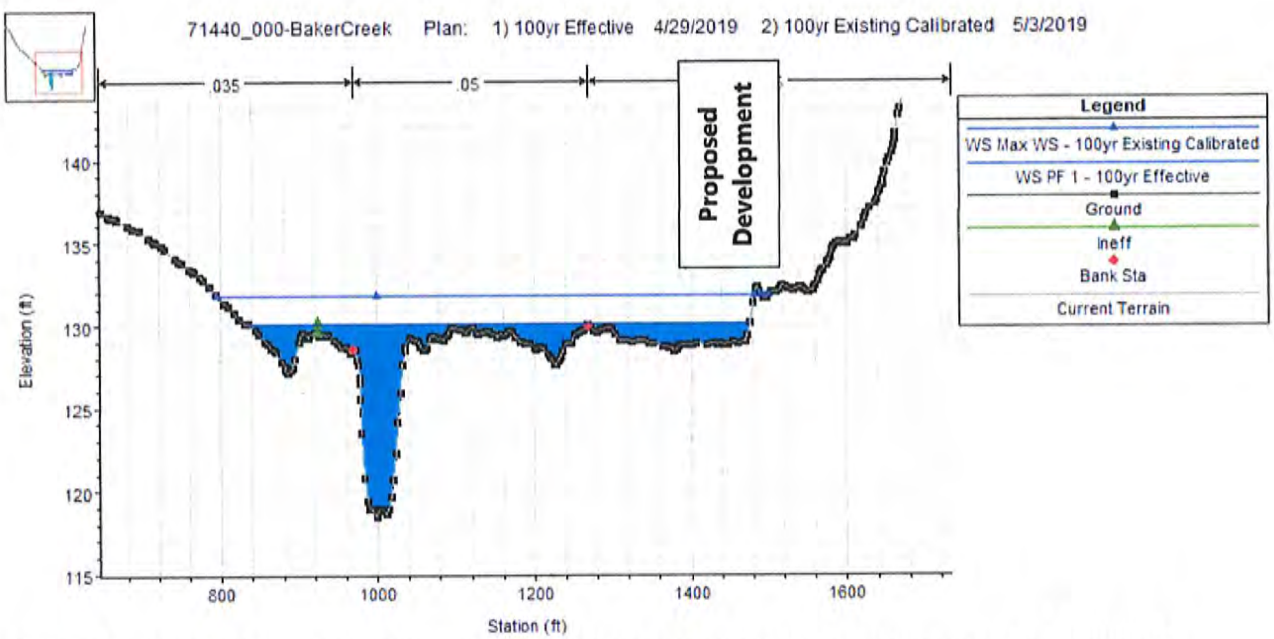

Figure 13: Comparison of Effective and Existing Water Surface at Downstream end of Proposed Development (Cross Section 14879)

A 2-year event was also modeled, in order to establish the approximate return period of flood events that have encroached on overbank 500-year floodplain areas. The 2-year event model result is shown in

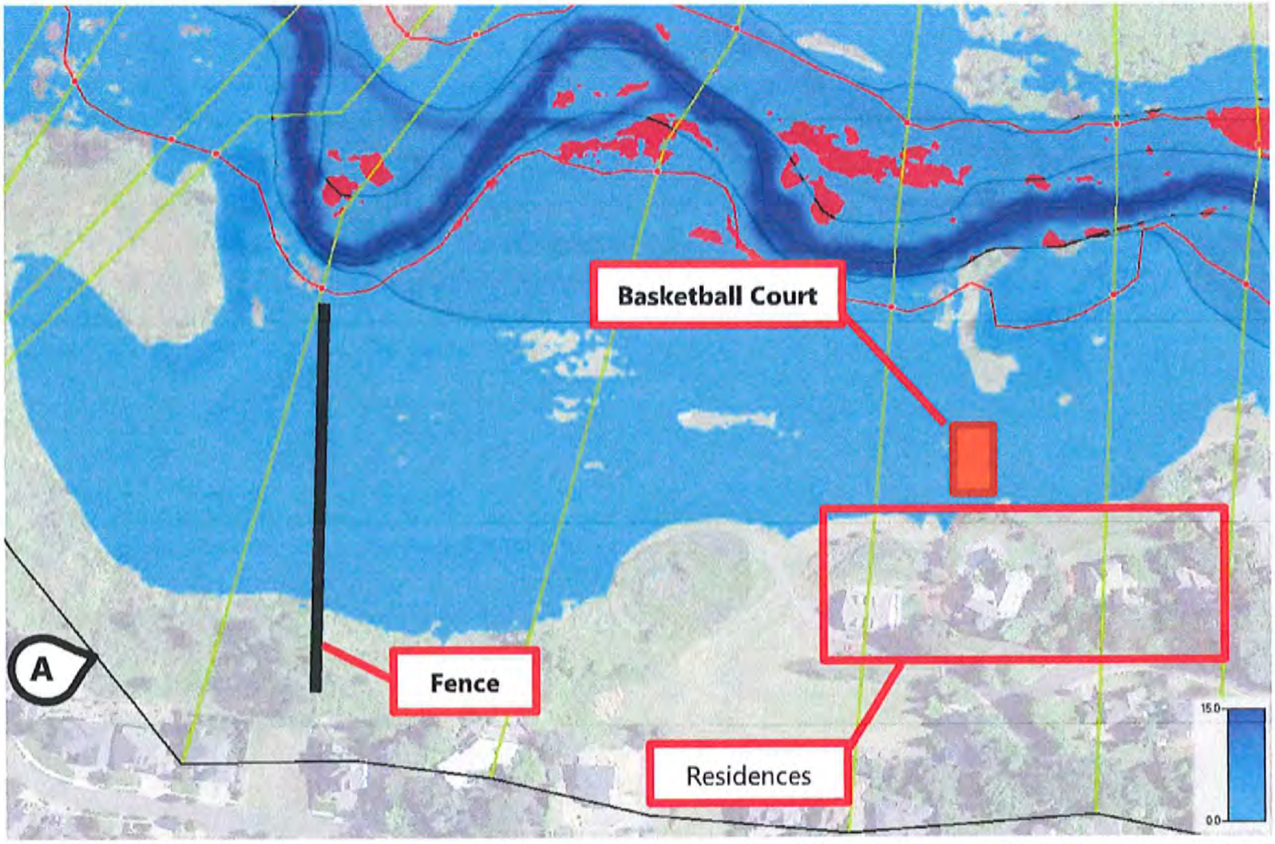

Figure 14: Annotated 2-year, 24-hour Model Maximum Depth Grid (in Feet) Near Neighborhoods with Location of Photo A.

Based on the Sylvania PCC rainfall gage near Lake Oswego, OR (part of the City of Portland HYDRA gage network), two events around November 2015 came close to a 2-year flood event. ODOT precipitation grids indicate a 2-year, 24-hour rainfall depth at the gage of about 2.4 inches. On October 31, 2015, the Sylvania gage recorded 2.02 inches of rainfall, and on December 3, 2015, the same gage recorded 2.41 inches. In October 2018, the gage recorded approximately 1.20 inches of rainfall. Photos were provided by residents and are shown in Figure 16 and Figure 17.

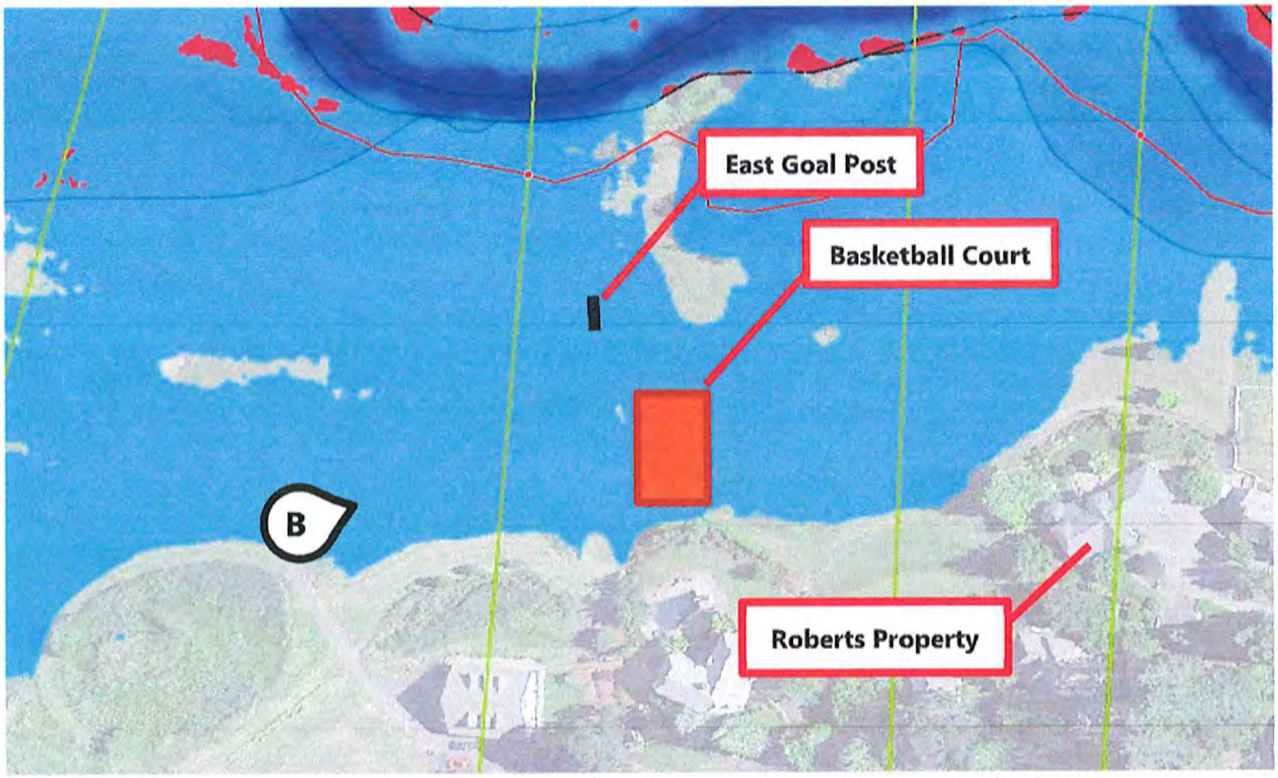

Figure 15: 2-year, 24-hour Maximum Depth Grid with Approximate Location of Photo B.

Figure 16: Flood Waters Observed During a ~~2018~~ Rainfall Event (Photo A).
2015 mc

Norma Brott Photo

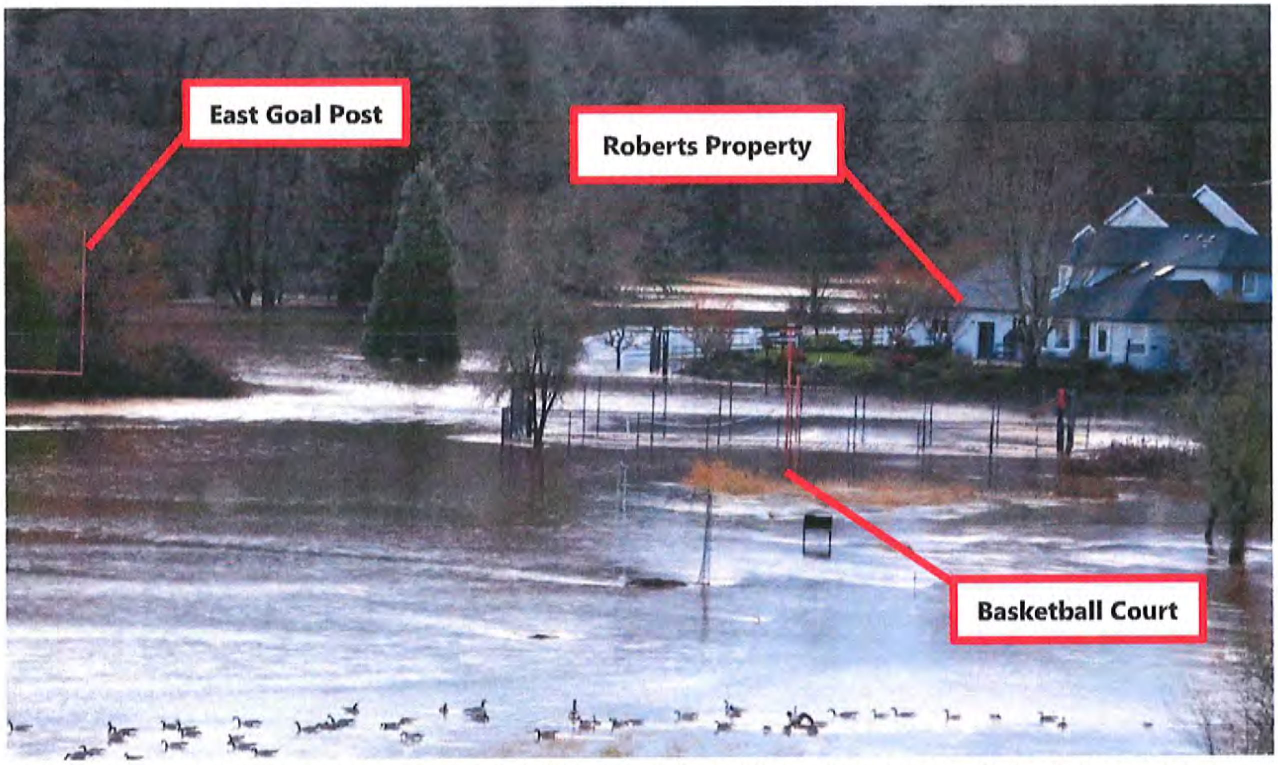

Figure 17: Flood Waters During 2015 Rainfall Events (Photo B), Looking East to Roberts Property.

4.2.3 Agricultural Activity

Residents have noted that recently, approximately 1,000 acres of filbert orchards were outfitted with "tile" drainage systems, consisting of underdrains that could allow rainfall that would otherwise be retained in surficial soils to enter perforated pipe systems and discharge to Baker Creek. These systems have been added to the Hydrology model to estimate their potential impacts. The approximate location of tiling is shown in Figure 18.

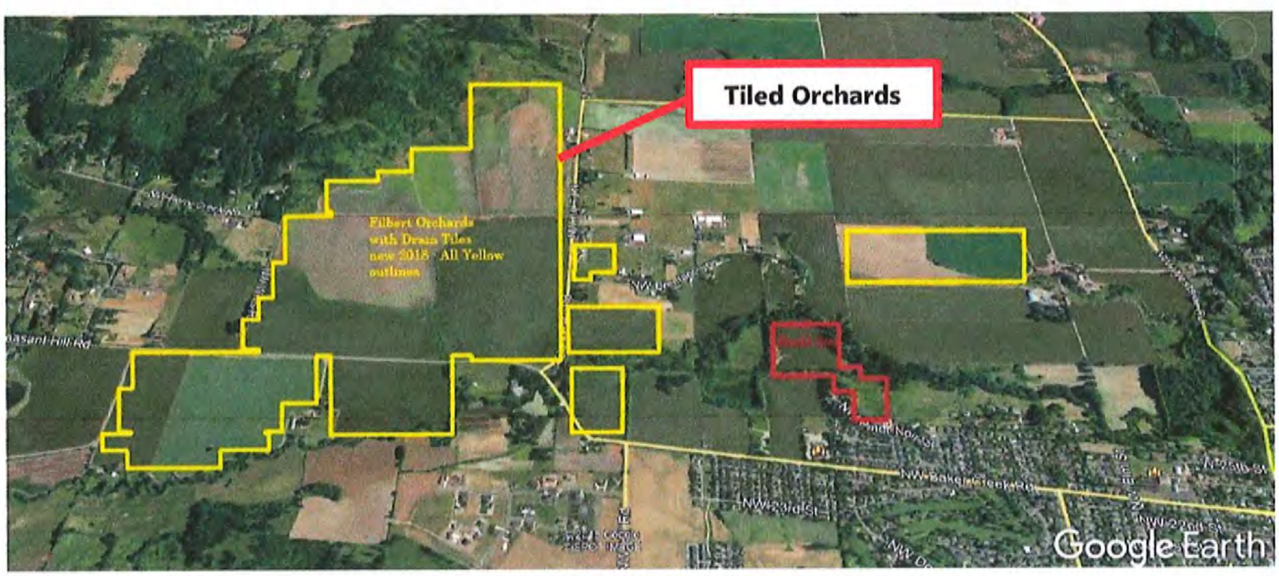

Figure 18: Approximate Boundary of Tiled Orchard Areas.

Surface runoff is assumed to remain the same. An additional sub-basin element has been added to the HEC-HMS model to represent the impact of the drain tile. The soils that characterize the agricultural areas of the watershed consist of moderately high infiltration capacity silt loams with an average conductivity of up to 2.0 inches per hour. Drain tile systems are generally installed approximately 36 inches or more below the surface.

The model assumes that rainfall that does not runoff from the surface basins as excess precipitation will be applied to the drain tile sub-basin with a curve number of 98. A high curve number is applied, assuming that the majority of infiltrated rainfall will enter the sub-drains during winter storms, when the subsurface is most saturated. Of the 4.81 inches of modeled precipitation for a 100-year event, for example, 2.91 is routed by the calibrated model as surface runoff. The remaining 1.90 inches would be applied to the drain tile subbasin.

A lag time is applied to the sub-basin based on soil properties and the depth of the sub-drain systems. Based on soil conductivity properties and an assumed drain tile depth of 36 inches, lag time for flow that infiltrates through the top layer of soil into the sub-drain system is about 18 hours. Various combinations of drain tile depth and conductivity were modeled, representing an "average" condition and a condition that would cause the greatest impact to flow in Baker Creek during a storm. Model runs are summarized in Table 14.

Table 14: Drain Tile Modeling Scenarios.

Scenario	Drain Tile Depth (in)	Conductivity (in/hr)	Lag Time (hour)
1	36	2	18
2	24	4	6

The two modeled drain tile scenarios were applied for a 100-year event and a 2-year event to estimate the approximate impact to peak flow rates in Baker Creek for a large flood event, as well as a more frequent one. The results of the analysis are provided in Table 15. Peak flows in the table are taken at NW Westside Road. The flow hydrographs from the 2-year model (with the greatest relatively peak flow difference) is shown in Figure 19.

Table 15: Comparison of Model Results with and without Drain Tile Sub-Basin.

Event	Scenario	Peak Flow Without Tile (cfs)	Peak Flow With Tile (cfs)	Peak Flow Difference
2-year	1	1,857	1,890	1.7%
	2		1,922	3.4%
100-year	1	6,394	6,405	0.2%
	2		6,425	0.5%

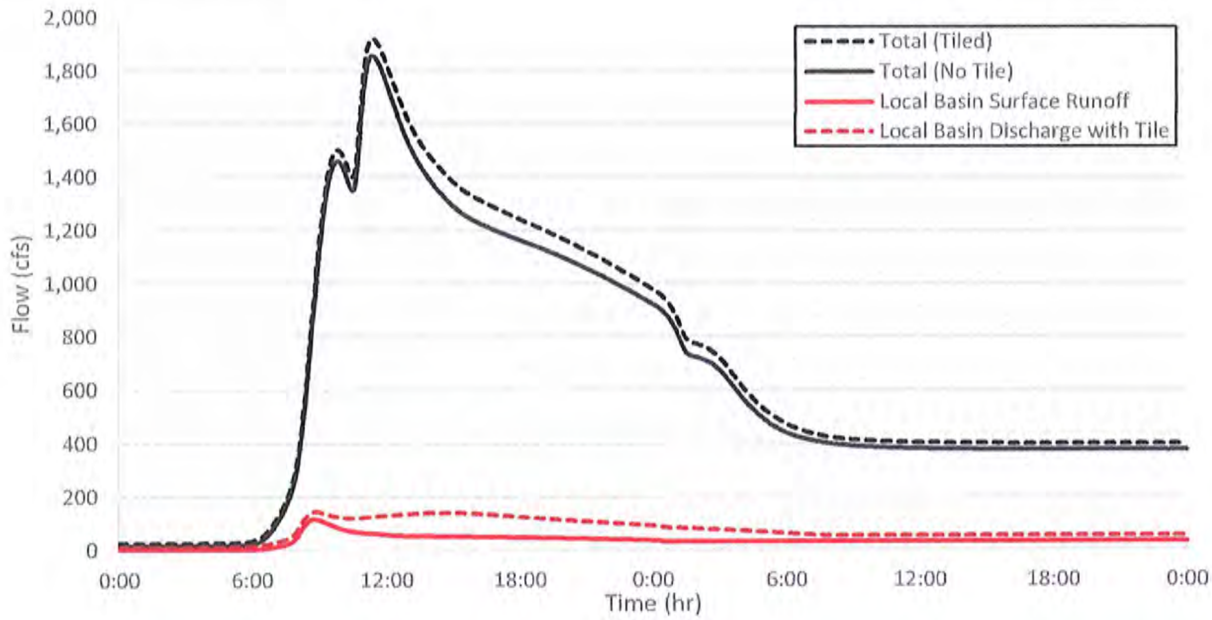

Figure 19: Baker Creek Flow Hydrographs for a 2-year Event (Drain Tile Scenario 2).

2-year, Scenario 2 drain tile hydrographs were then added to the HEC-RAS model to estimate the water surface impacts in the vicinity of the residential neighborhoods. As shown in Figure 20, an increase in water surface elevation of approximately one tenth of a foot (0.8%) from 129.77 to 129.87 is observed for the 2-year event with 24-inch deep tile sub-drains and high conductivity soils. The relative impact of the drain tile during a 100-year event (with same depth and conductivity assumptions applied) is reduced due to the magnitude of the peak from the upstream watershed (Figure 21). The WSEL difference for a 100-year event estimated from the HEC-RAS hydraulic model is about 0.02 feet (0.1%), from 131.47 to 131.49.

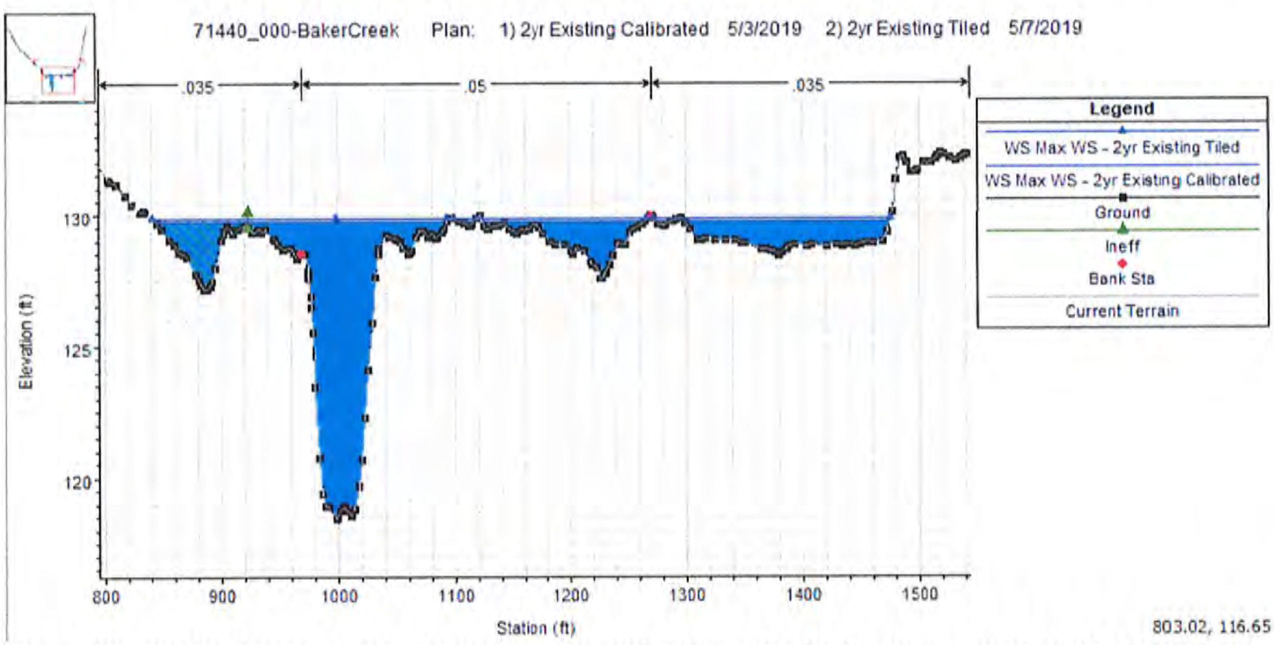

Figure 20: Existing Water Surface Elevation with and without Drain Tile for a 2-year Event (Cross Section 14879).

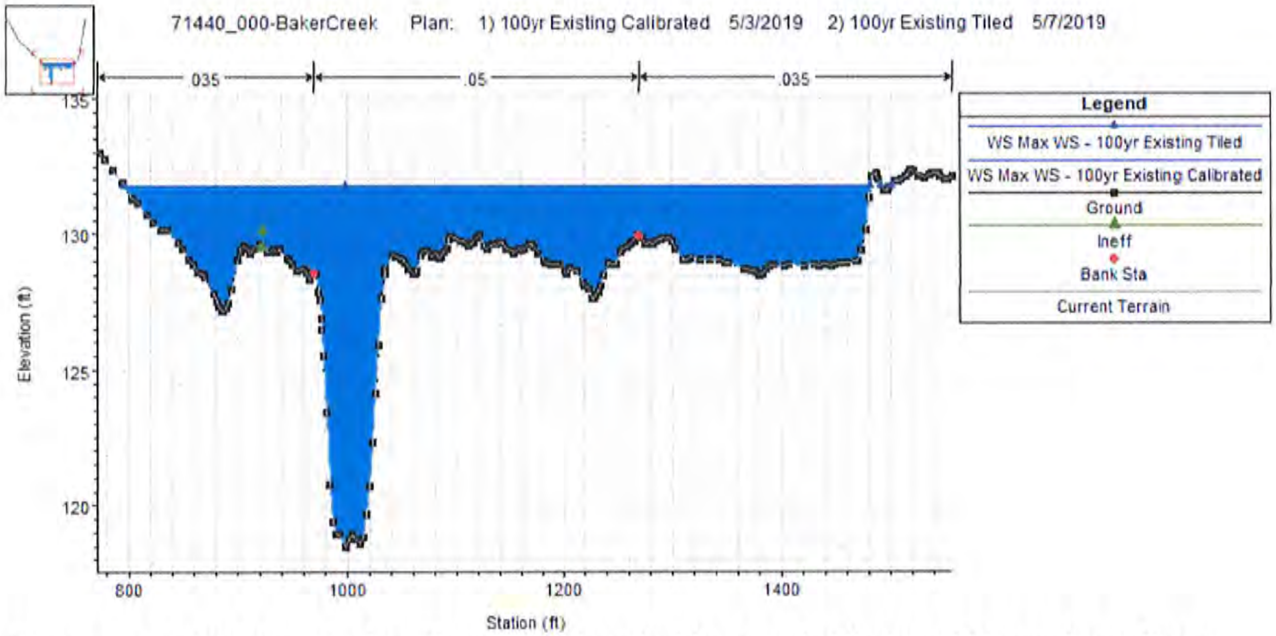

Figure 21: Existing Water Surface Elevation with and without Drain Tile for a 100-year Event (Cross Section 14879)

4.2.4 Future Condition

Based on the existing McMinnville city limits, approximately 0.12 square miles of the watershed could be developed, including the proposed Oak Ridge residential subdivision. This area has been added to the HEC-HMS model with a curve number of 83 and a lag time of 15 minutes, representing approximate conditions for medium density residential construction. This "future" model condition estimates the potential peak flow impact of a "build-out" scenario for the City of McMinnville.

The existing model result was overlain on development plans in GIS to determine whether significant floodplain blockages might be created. The result of this overlay is provided in Figure 22.

Figure 22: Proposed Condition 100-year Maximum Water Surface Elevation (ft NAVD) Grid Result from RAS Mapper.

An area of potential blockage was identified over lots 61-64. While the current development plans do not show significant grading on these lots, future residential construction and fill on the lots may occur within an area that could be mapped as 1% chance special flood hazard area. Blockage was modeled at these parcels by modifying the cross sections at their upstream and downstream side, as shown in Figure 23.

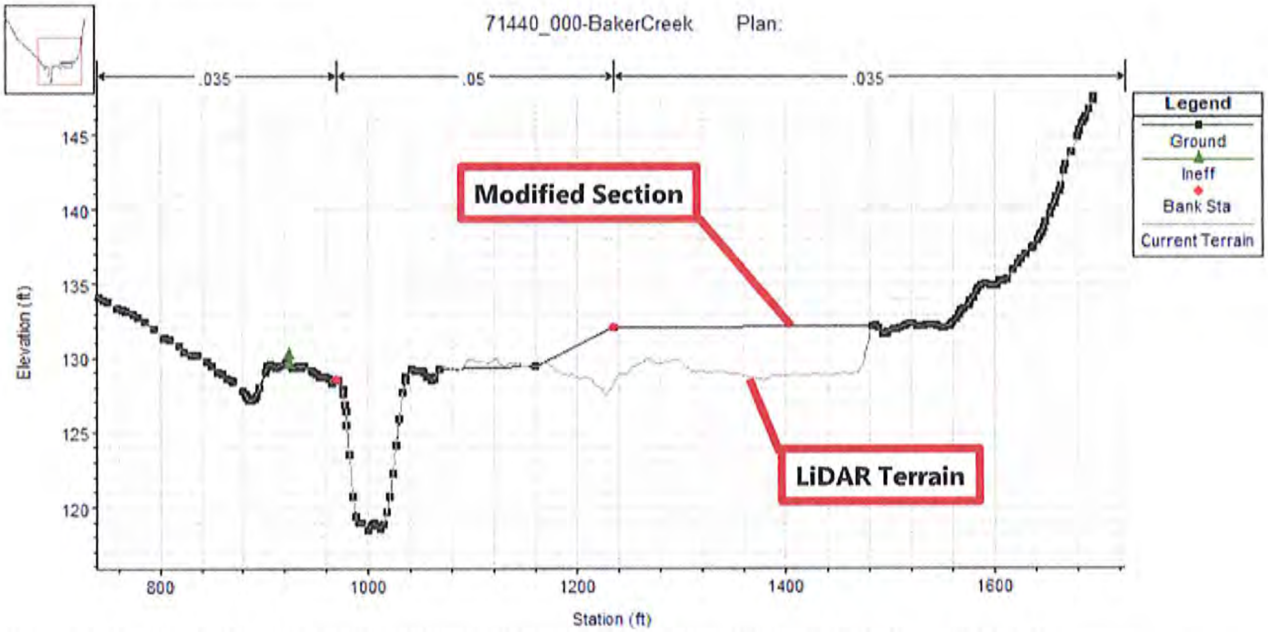

Figure 23: Floodplain Blockage Cross Section Modification (Shown at Section 14879).

The impact of this potential blockage on flood flows compared with the existing condition is shown at this same cross section in Figure 24. The existing peak water surface at the section is 131.80, while the estimated fill condition results in a peak water surface elevation of approximately 132.04.

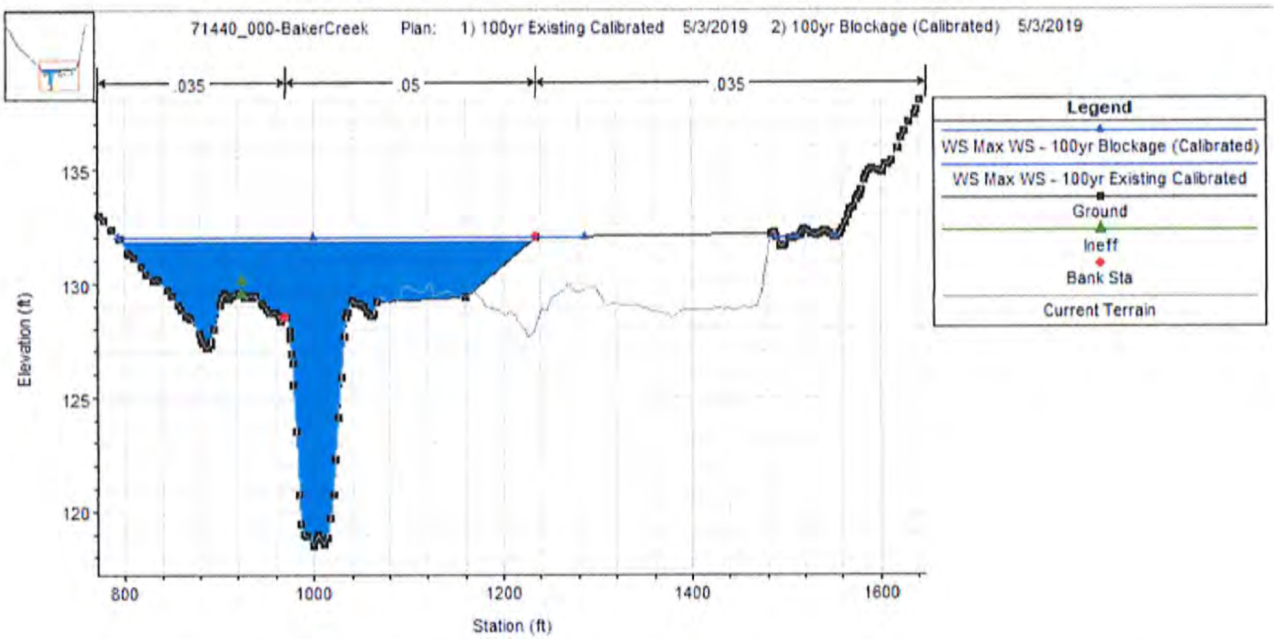

Figure 24: Comparison of Peak Water Surface for Existing and Potential Future Fill Conditions at Oak Ridge Development Lots 61-64 (Cross Section 14879).

This relatively small impact does not appear to propagate downstream, however (Figure 25), where the modeled existing and approximated future condition peak water surface elevations are 127.42 and 127.41, respectively.

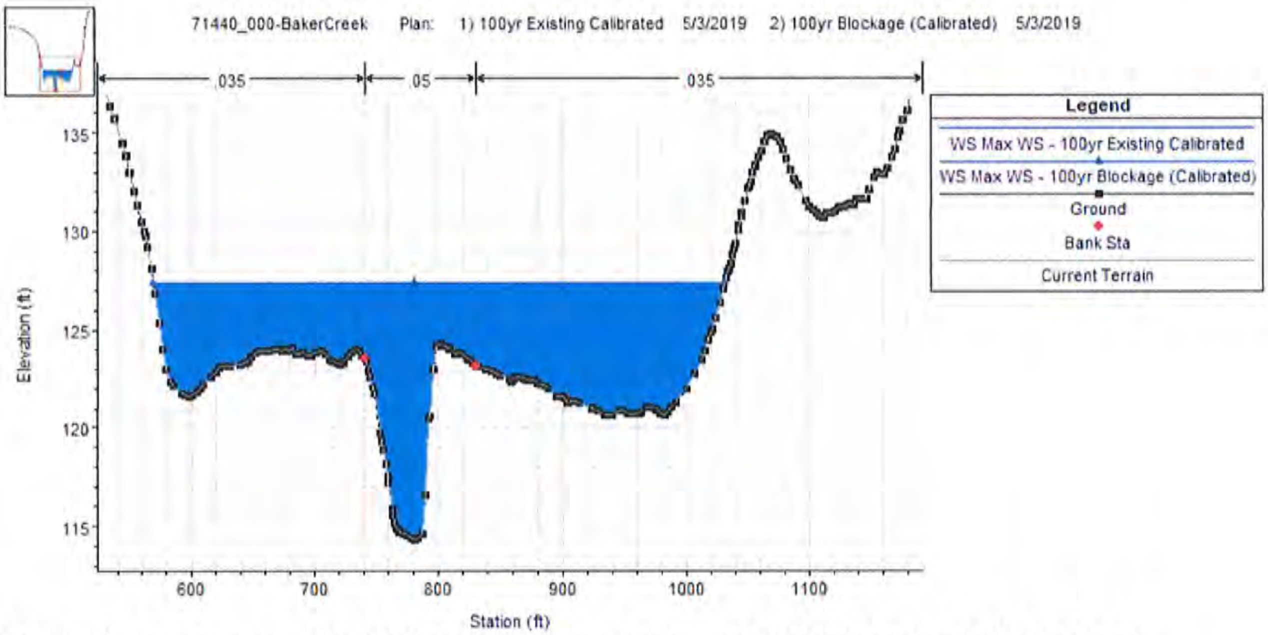

Figure 25: Comparison of Peak Water Surface for Existing and Potential Future Fill (Blockage) Conditions Downstream (Cross Section 11843).

Depth and water surface elevation results are summarized for the reach adjacent to the proposed and existing developments in Table 16.

Table 16: Comparison of Depth and Water Surface Elevation at Three Locations.

Cross Section No.	Location	Max Depth (ft)			Max WSEL (ft NAVD)		
		Effective Flow	Existing Cond.	Future Cond.	Effective Flow	Existing Cond.	Future Cond.
17056	Upstream of Proposed Subdiv.	11.79	13.83	13.84	132.39	134.43	134.44
14879	Downstream End of Proposed Subdiv.	11.67	13.33	13.56	130.14	131.80	132.03
11843	Adjacent to Existing Residences	10.48	13.11	13.10	124.79	127.42	127.41

5 CONCLUSION

Based on the analysis presented in this report, significant changes in stream gage records and statistics appear to have contributed to an increase in estimated flood flows and creek stage since the effective FEMA analysis was performed. The effective FIS utilized regression equations to estimate peak flows at the time of the original study that were developed based on fewer gages with a much shorter period of record.

Higher flows may therefore be due to the use of higher resolution and more accurate land cover and soils data and due to the availability of larger quantities of data to support more recent USGS studies and the development of a higher quality model. Within the reach surrounding the existing neighborhoods in the northwest area of the City of McMinnville, peak 1% annual chance (100-year) flow values estimated by this watershed-specific analysis exceed FEMA's effective flow rates by approximately 65-75%.

With current land use and terrain data as inputs to modern hydrologic and hydraulic models, floodplains in the lower Baker Creek watershed don't appear to be well represented by the effective SFHA mapping. A comparison of Effective FEMA SFHAs and BFEs to the modeled inundation and floodplain water surface elevations is shown in Figure 26.

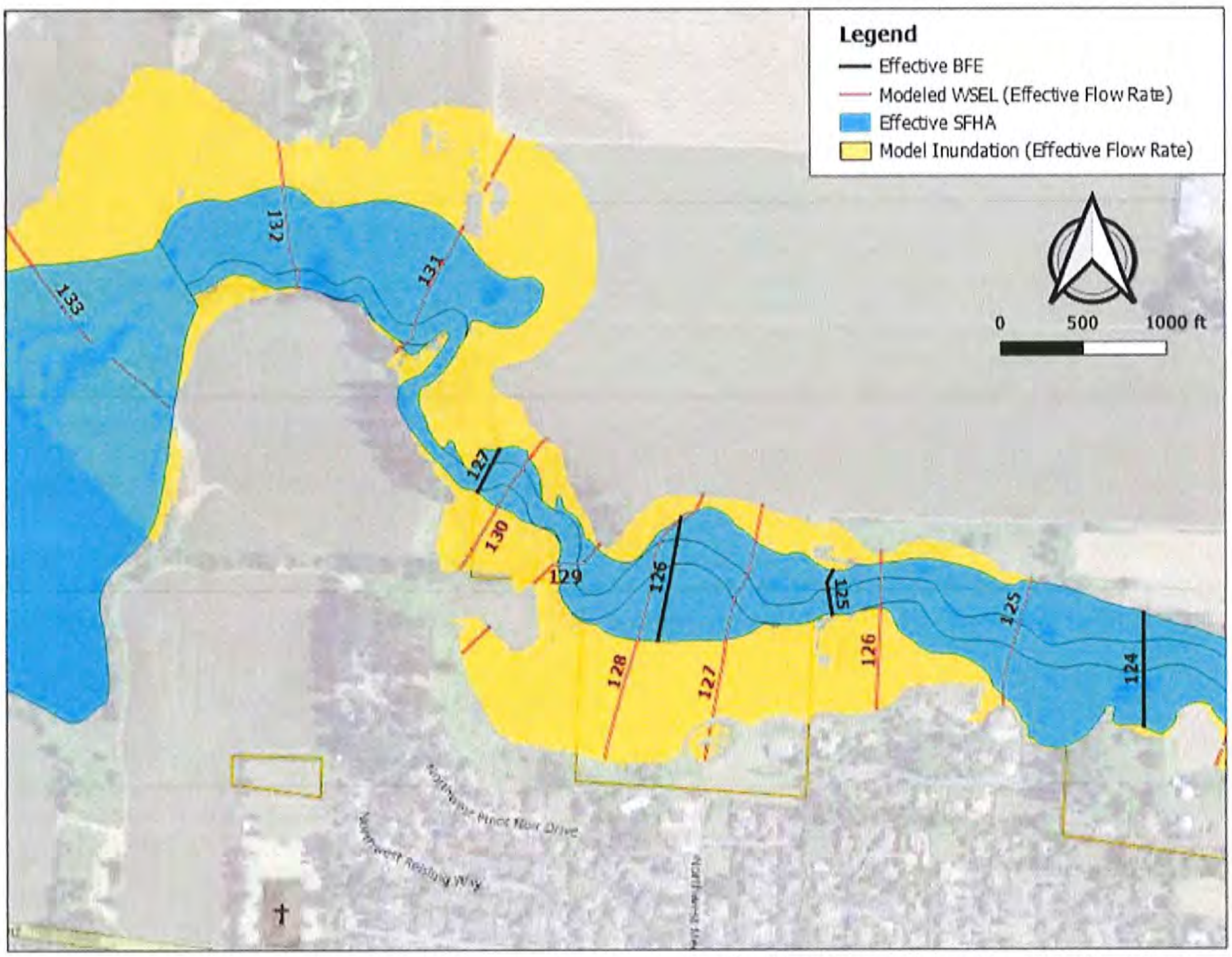

Figure 26: FEMA Effective SFHA and BFEs Overlain on Hydraulic Model Results for the Effective Flow Rate (2,030 cfs).

In actuality, the effective 100-year flow rate cited by the FIS is near a 2-year, 24-hour event, based on the calibrated hydrologic model. Photos from residents during rainfall events near the 2-year return period provide anecdotal verification of the model results.

Though the 100-year peak flow estimate has increased by a significant amount since the original determination of BFEs in the Baker Creek watershed, water surface elevation increases are relatively moderate due to the width of the floodplain surrounding Baker Creek. The change in flood depths between the steady, effective flow rate model and the existing condition model is shown in Figure 27. The figure also highlights the extent and depth of additional flooding beyond the extent of inundation from the effective 100-year peak flow rate.

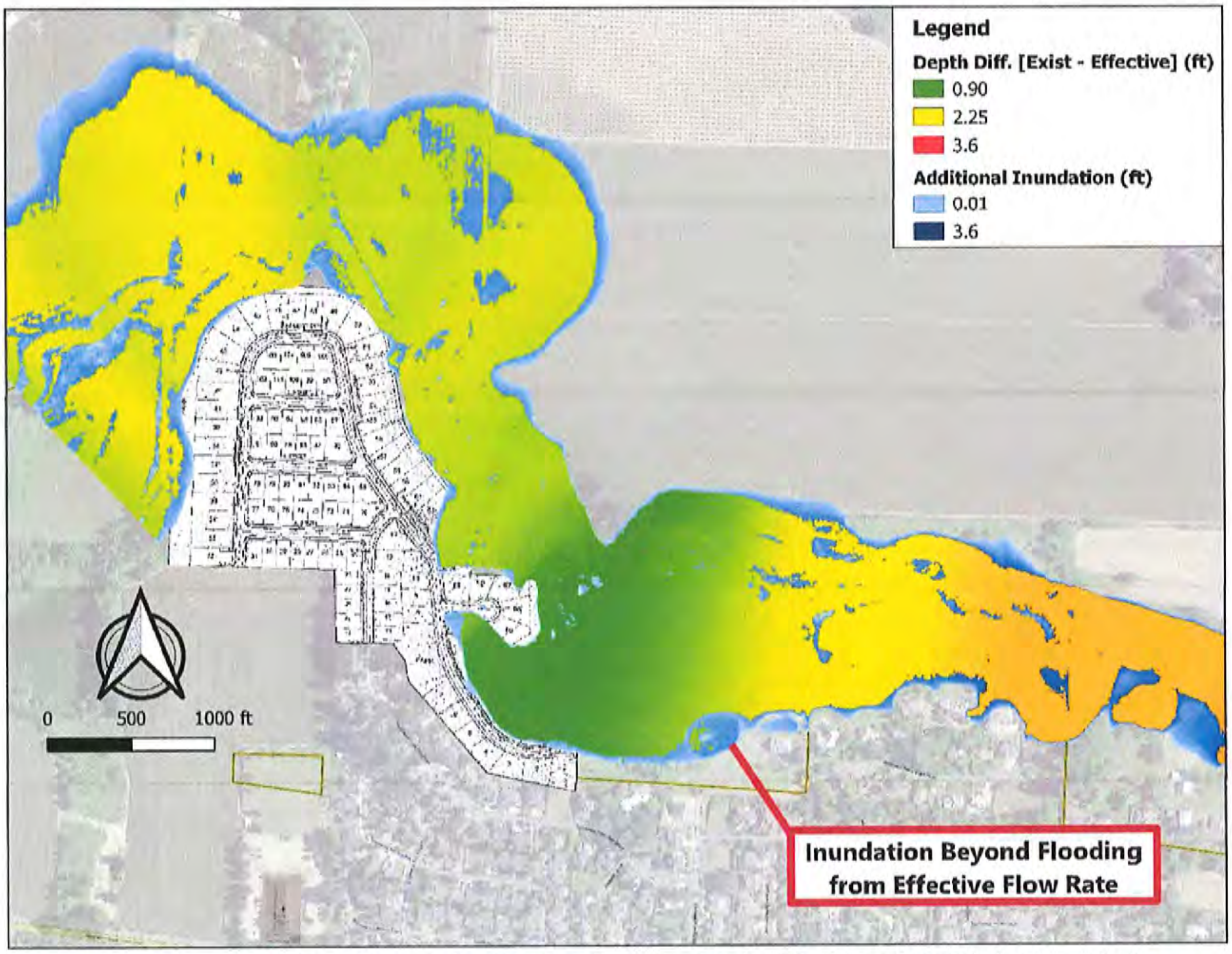

Figure 27: Difference Between Depth for Existing Condition (~6,150 cfs) and Depth for Modeled Effective Flow Rate (2,030 cfs).

This analysis indicates that recent agricultural and development activity within the lower watershed has some effect on flow rates and water surface elevations surrounding existing and future residential development. In response to the added development for a "build-out" condition, the peak flow at the downstream end of the proposed Oak Ridge subdivision (cross section 14879) for a 100-year event rises from 6,173 cfs to 6,184 cfs, a relative increase in peak flow of approximately 0.2%. The water surface elevation at the same cross section changes by approximately one hundredth of a foot. The impact likely appears small because the peak flow from the local development area occurs hours well prior to the higher magnitude peak flow from the upstream watershed. This timing difference is well illustrated by Figure 7.

Based on the modeled flow hydrographs, the potential downstream impact of blockage for the proposed development amounts to less than one hundredth of a foot of increase adjacent to existing residences, provided fill does not cross proposed property lines into the apparent floodplain.

A portion of the proposed development appears to lie within the 100-year floodplain based on modern modeling methods and statistics. These findings highlight the need for revision of the effective BFEs and flood hazard areas. True modernization of the SFHA delineation should be undertaken to ensure that future development that may be constructed within the floodplain does not incur significant increases in BFE adjacent to existing developments and carry the potential to cause hazards or property damage.

Further, recent activity, as well as proposed development is limited to the immediate vicinity of Baker Creek in the lowest reaches of the watershed and has a relatively small impact when compared with the contributions of the large, upstream drainage areas. Agricultural changes that may increase Creek flows have been generally limited to existing pasture lands, for example. Greater impacts could occur in the future, as development and agricultural activity may push further west in the watershed, replacing forested and grassland covers with much lower runoff potential. Such activity should be examined closely to ensure that a reduction in dominance of forest and rangeland doesn't induce more extreme, rapid peak flows characteristic of heavily developed watersheds.

6 REFERENCES

CH2MHill. "Storm Drainage Master Plan." *Draft Final Report*. Prepared for City of McMinnville. April 2009.

Federal Emergency Management Agency [FEMA]. "Flood Insurance Study: Yamhill County, Oregon and Incorporated Areas." Flood Insurance Study Number 41071CV000A. Effective Date March 2, 2010.

Federal Emergency Management Agency [FEMA]. "Flood Insurance Study: Yamhill County, Oregon Unincorporated Areas." Flood Insurance Study Number 41029V000. Effective Date March 30, 1983.

Oregon Department of Transportation [ODOT]. "ODOT Hydraulics Manual". April 2014.

United States Department of Agriculture - Natural Resources Conservation Service. "Urban Hydrology for Small Watersheds." Technical Release 55. Publication Number 210-VI-TR-55. Second Edition. June 1986 (Updated January 1999).

United States Geological Survey (USGS). "Estimation of Peak Discharges for Rural, Unregulated Streams in Western Oregon." Scientific Investigations Report 2005-5116. Reston, VA. 2005.

Westech Engineering, Inc. *Drawings For: Oak Ridge Meadows Phase 1*. Review Plans. Dec 2018