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Evaluation of Flood Inundation in Crystal Springs Creek, Portland, Oregon

Open-File Report 2016–1079

U.S. Department of the Interior
U.S. Geological Survey

Cover: Photograph of Crystal Springs Creek, Portland, Oregon, looking downstream from the Umatilla Street Bridge. Photograph by Adam J. Stonewall, U.S. Geological Survey, June 25, 2013.

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SALLY JEWELL, Secretary

U.S. Geological Survey
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Conversion Factors

Inch/Pound to International System of Units

Multiply	By	To obtain
	Length	
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
	Flow rate	
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)

Datums

Vertical coordinate information is referenced to the North American Vertical Datum of 1988 (NAVD 88).
Horizontal coordinate information is referenced to North American Datum of 1983 (NAD 83).
Elevation, as used in this report, refers to distance above the vertical datum.

Abbreviations

AEP	annual exceedance probability
BES	Bureau of Environmental Services
FIS	flood insurance study
GIS	geographic information system
HEC-RAS	Hydrologic Engineering Center-River Analysis System
USGS	U.S. Geological Survey

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Abstract

Efforts to improve fish passage have resulted in the replacement of six culverts in Crystal Springs Creek in Portland, Oregon. Two more culverts are scheduled to be replaced at Glenwood Street and Bybee Boulevard (Glenwood/Bybee project) in 2016. Recently acquired data have allowed for a more comprehensive understanding of the hydrology of the creek and the topography of the watershed. To evaluate the impact of the culvert replacements and recent hydrologic data, a Hydrologic Engineering Center-River Analysis System hydraulic model was developed to estimate water-surface elevations during high-flow events. Longitudinal surface-water profiles were modeled to evaluate current conditions and future conditions using the design plans for the culverts to be installed in 2016. Additional profiles were created to compare with the results from the most recent flood model approved by the Federal Emergency Management Agency for Crystal Springs Creek and to evaluate model sensitivity.

Model simulation results show that water-surface elevations during high-flow events will be lower than estimates from previous models, primarily due to lower estimates of streamflow associated with the 0.01 and 0.002 annual exceedance probability (AEP) events. Additionally, recent culvert replacements have resulted in less ponding behind crossings. Similarly, model simulation results show that the proposed replacement culverts at Glenwood Street and Bybee Boulevard will result in lower water-surface elevations during high-flow events upstream of the proposed project. Wider culverts will allow more water to pass through crossings, resulting in slightly higher water-surface elevations downstream of the project during high-flows than water-surface elevations that would occur under current conditions. For the 0.01 AEP event, the water-surface elevations downstream of the Glenwood/Bybee project will be an average of 0.05 ft and a maximum of 0.07 ft higher than current conditions. Similarly, for the 0.002 AEP event, the water-surface elevations will be an average of 0.04 ft and a maximum of 0.19 ft higher than current conditions.

Introduction

Crystal Springs Creek is a 2-mi-long urban stream in the Johnson Creek watershed in southeast Portland, Oregon (figs. 1 and 2). It issues from a large spring near Reed College and flows through a developed area of residential, commercial, industrial, and urban park land. Crystal Springs Creek is above ground for most of its course and is crossed by 13 roadways, one railway, and numerous foot bridges. Base flow is relatively constant and higher than in most other parts of the Johnson Creek watershed due to the spring-fed nature of the creek (Lee and Snyder, 2009). Crystal Springs Creek is designated as critical habitat for at least three salmonid species[1] under the Endangered Species Act by the National Marine Fisheries Service (National Oceanographic and Atmospheric Administration, 2005; City of Portland, 2014a).

The City of Portland Bureau of Environmental Services (BES) is replacing or removing nine culverts that impeded fish passage along Crystal Springs Creek as part of the City of Portland's Grey to Green Initiative (City of Portland, 2014b). As of November 2015, six culverts had been replaced with stream crossings that more closely resembled natural streambeds and provided easier passage for fish, and one culvert had been removed and abandoned. The final two culverts at Bybee Boulevard and at Glenwood Street were scheduled to be replaced in 2016.

[1] Lower Columbia River Chinook salmon (*Oncorhynchus tshawytscha*), lower Columbia River steelhead (*Oncorhynchus mykiss*), and coho salmon (*Oncorhynchus kisutch*).

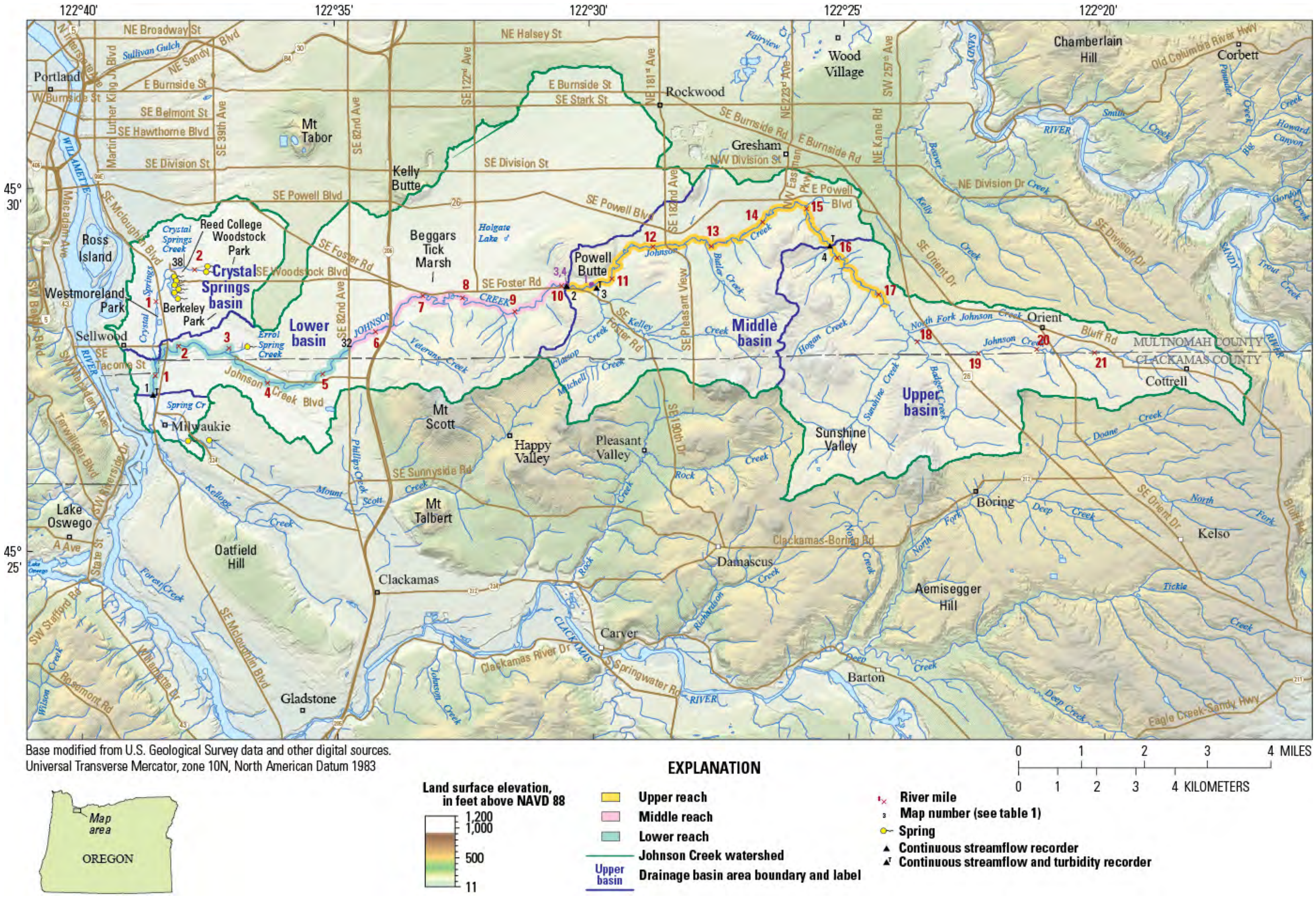

Figure 1. Map showing location of the Crystal Springs Creek Watershed within the Johnson Creek Watershed, Portland, Oregon.

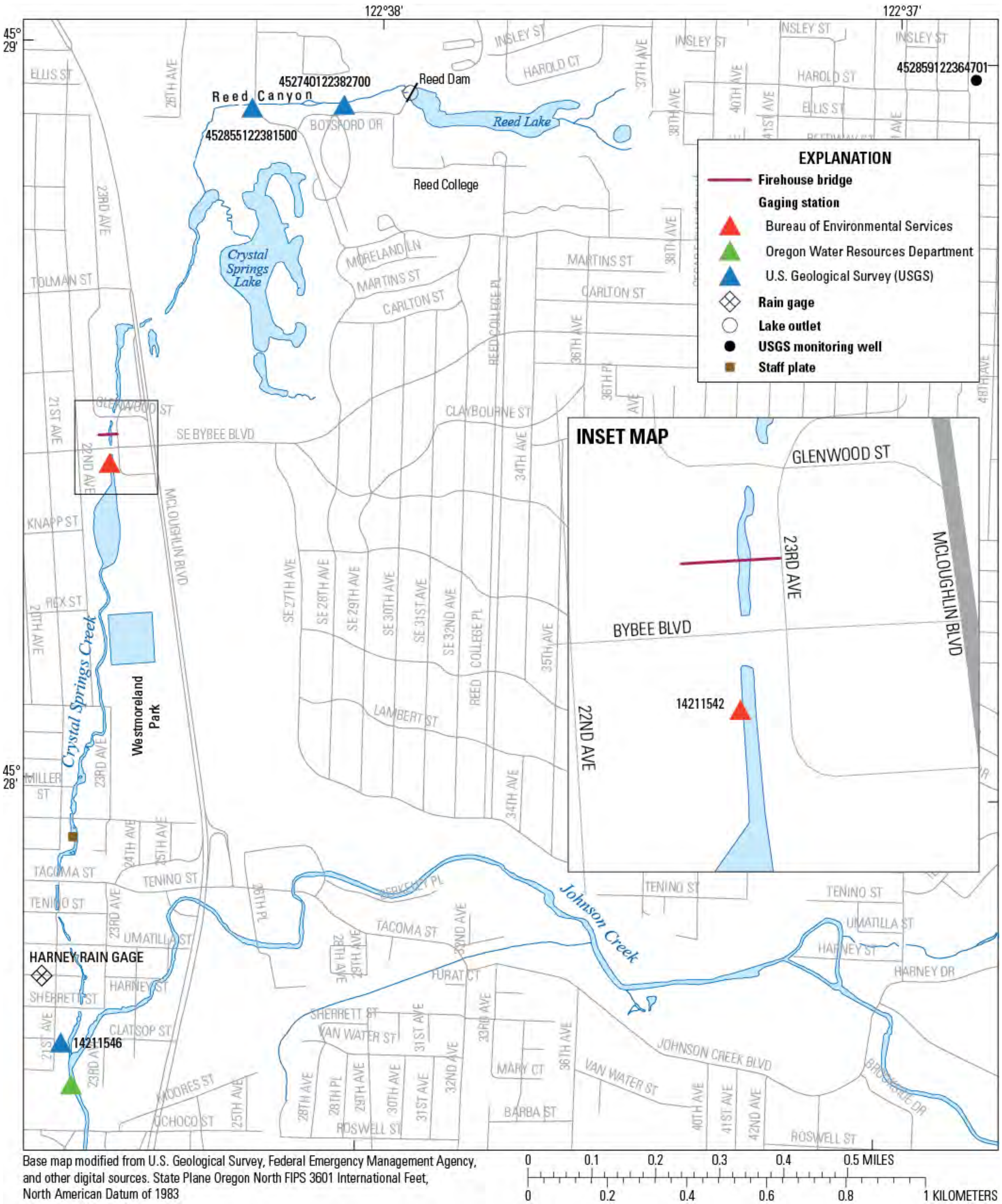

Figure 2. Map showing locations of Crystal Springs Creek roadway crossings, Portland, Oregon.

Purpose and Scope

This report describes the development of a Hydrologic Engineering Center-River Analysis System (HEC-RAS) model used to estimate longitudinal water-surface elevations associated with various quantities of streamflow for the reach of Crystal Springs Creek extending from slightly downstream of Reed Dam (river station 10,596 ft, or approximately river mile 2.0) to the mouth. The model is a refinement of the steady HEC-RAS model detailed in Stonewall (2014), which was created to model water-surface elevations in the area of the Glenwood Street and Bybee Boulevard crossings (approximately river mile 1.0).

Simulated current conditions are compared to a version of the model updated with the design geometry of two culverts scheduled for replacement (at Bybee Boulevard and at Glenwood Street, fig. 2). Sensitivity analyses were conducted to evaluate the following:

1. The influence of the timing of lateral inflows;
2. The differences between the steady and unsteady versions of the HEC-RAS model;
3. The differences between streamflow values calculated in Stonewall (2014) and the existing flood insurance study (FIS); and
4. The influence of the shape of hydrograph that acts as the upstream boundary condition.

Previous Studies

In the most recent flood insurance study for Crystal Springs Creek (Federal Emergency Management Agency, 2010), a U.S. Army Corps of Engineers rainfall-runoff model (HEC-HMS; U.S. Army Corps of Engineers, 1998) was used to estimate annual exceedance probabilities (AEP) streamflows. Streamflow inputs were then routed through a previous version of a HEC-RAS model (U.S. Army Corps of Engineers, 2010). The HEC-HMS model used the Muskingum-Cunge channel flow routing method (U.S. Army Corps of Engineers, 1991). The HEC-HMS model defined subbasins and diversions to simulate runoff intercepted by combined sewers and other stormwater infrastructure, including sumps (dry wells). This and other studies in and around Crystal Springs Creek are described in some detail in Stonewall (2014).

Model Development

The HEC-RAS (version 4.1.0, U.S. Army Corps of Engineers, 2010) computer program was used to simulate the 0.01 and 0.002 AEP water-surface elevations in Crystal Springs Creek from immediately downstream of Reed Dam to the mouth. HEC-RAS is a one-dimensional hydraulic model and commonly is used to simulate estimated flood inundation (Christiansen and Eash, 2008). HEC-RAS can be set up to run in either steady or unsteady mode. Steady mode is a simpler model and does not model attenuation or storage, whereas unsteady mode accounts for both. The Crystal Springs Creek model was run in unsteady mode to evaluate the potential effect of the timing of lateral inflows throughout the watershed and to account for potential areas of storage. The model was run in steady mode to evaluate the floodway and to compare against the existing FIS, which was also run in steady mode. Table 1 lists the names and conditions of all the model plans developed for this study.

Table 1. Hydrologic Engineering Center-River Analysis System (HEC-RAS) model plans used in this study.

[Current geometry data reflect stream crossings as of September 2014. Proposed geometry data incorporate the geometry data for the planned culvert replacements at Bybee Boulevard and Glenwood Street. Current-short geometry data are from just above Bybee Boulevard to the confluence of Johnson Creek. Updated hydrographs have been calibrated to match annual exeedance probabilities of streamflow calculated using the hydrologic model developed in Stonewall (2014). Maximum Q hydrograph is designed for each lateral hydrograph to peak at the same time as flood wave in the main channel at the location of the input. FIS, flood insurance study]

Model plan	HR21	HR22	HR23	HR24	HR25	HR26	HR27	HR29	HR31	HR32	HR33
Annual exceedance probability	0.01	0.01	0.002	0.002	0.01	0.01	0.01	0.01	0.10	>0.10	n/a
Geometry data	Current	Proposed	Current	Proposed	Current	Current	Current	Current	Current	Current	Current-short
Boundary condition hydrograph	Updated	Updated	Updated	Updated	Updated	Existing FIS	Updated	Updated	Updated	Updated	January 19, 2012 streamflow
Lateral inflow hydrographs	Updated	Updated	Updated	Updated	n/a	n/a	Maximum Q	n/a	Updated	Updated	Updated
Steady/unsteady	Unsteady	Unsteady	Unsteady	Unsteady	Steady	Steady	Unsteady	Steady	Unsteady	Unsteady	Unsteady

Model Geometry

Channel cross sections were developed from surveys conducted upstream and downstream of each stream crossing and at other important locations (such as sections of stream that have been reconfigured since the last light detection and ranging (lidar) coverage was created) and areas that act as a hydraulic control within the creek. Structural geometry data (culvert diameters, bridge widths, etc.) also were collected on and around bridges and culverts and compared to data reported in the most recent FIS. Additional spatial data were collected in areas where the channel recently had been reconfigured, in particular in Westmoreland Park and just upstream of SE 28th Avenue. The surveyed cross-sectional data were compared against the lidar data for accuracy and were also compared against points with established elevations (Stonewall, 2014, table 5). The root-mean-squared error for all established benchmarks elevations was 0.10 ft, which is within acceptable limits (Rydlund and Densmore, 2012).

A geographic information system (GIS) grid of lidar-derived elevations in the North American Vertical Datum of 1988 of the Crystal Springs Creek area was provided by the Oregon Department of Geology and Mineral Industries (Jed Roberts, Oregon Department of Geology and Mineral Industries, written commun., June 4, 2015). Cross sections were extracted from the grid using HEC-GeoRAS software (U.S. Army Corps of Engineers, 2011). The modeled stream elevations were projected onto the land surface using the lidar data and HEC-GeoRAS, which is a set of procedures, tools, and utilities to process HEC-RAS output in ArcGIS™ (Esri®, 2014). The program can be used to prepare geometric data for import into HEC-RAS and to process streamflow simulation results exported from HEC-RAS. When necessary, the maps generated by HEC-GeoRAS were modified to provide a smooth and logical transition in the ground surface between surveyed and lidar data.

Of the 200 current-condition cross sections developed for the study, about two-thirds were generated from contours, breaklines, and spot elevations, as opposed to full cross-sectional surveys. In-channel data for these “synthetic” cross sections were estimated by interpolating between surveyed cross sections. Synthetic cross sections are used to increase model stability in areas where the channel cross section changes rapidly. Most were added in the upstream reaches of the creek (between SE 28th Avenue and Reed Lake [fig. 2]) where the terrain is steepest, the water levels are most shallow, and the model is most unstable. A relatively high density of synthetic cross sections were also added in Westmoreland Park (bounded between Bybee Boulevard to the north and Lambert Street to the south), where occasional overbank flows resulted in inconsistent channel widths, which also contributed to model instability.

Two storage areas were added to the model geometry file. The “Golf Course” storage area is located just upstream of the railroad tracks, off the left bank of the creek. The “Median” storage area is located between the railroad tracks and Highway 99, off the right bank of the creek. Both storage areas are connected to the channel using lateral weirs. The geometry file representing current conditions contained 200 cross sections and 21 crossings (plate 1). The geometry file representing future conditions, including the proposed work for the replacement of the Glenwood and Bybee culverts (hereafter referred to as ‘proposed’ conditions), contained 230 cross sections and 21 crossings. Based on the design plans for channel modification associated with the culvert replacements, 13 cross sections were removed from, and 43 cross sections were added to, the current-conditions geometry file to create the proposed conditions geometry file. All cross sections removed or added were within the project boundaries.

Boundary Conditions

A streamflow hydrograph of Crystal Springs Creek just downstream of Reed Dam served as the upstream boundary condition for the HEC-RAS model of the system. The downstream boundary condition for the HEC-RAS model was a normal depth using a slope of 0.003. This slope was calculated using an average slope of the downstream sections of the creek (river stations 1,024–462 ft). Lateral inflows to the system included tributary drainages and water from the City of Portland stormwater management system. Most of the water entering the stormwater system is not discharged into Crystal Spring Creek. However, during especially large flow events some overflow into the creek is possible (Gregory Savage, Bureau of Environmental Services, oral commun., 2015). For mapping purposes, the simulated water-surface elevations were tied into the existing water-surface elevations from the Johnson Creek FIS downstream of river station 345.381 (fig. 3).

The simulated upstream boundary condition hydrograph and lateral inflow hydrographs from the HEC-HMS model (Federal Emergency Management Agency, 2010) were obtained from BES. Lateral inflows were input to the HEC-RAS model based on the physical description of their confluence with the creek contained within the HEC-HMS model. These locations were then checked against GIS layers of the storm system map provided by BES. All of the simulated streamflow from the lateral inflow hydrographs enters downstream of SE 28th Avenue (U.S. Geological Survey [USGS] site number 452855122381500; fig. 2). Consequently, simulated peak streamflows tended to be much higher downstream of SE 28th Avenue than upstream.

Six of the lateral inflows discharge to Crystal Springs Creek downstream of the streamgage operated by BES at SE Bybee Boulevard (USGS site number 14211542, hereafter referred to as the “Bybee streamgage” for this report). These inflows were assumed to be negligible in the HEC-RAS model developed by Stonewall (2014), which assumed a constant streamflow between the Bybee streamgage and the mouth of Crystal Springs Creek (USGS site number 14211546). The constant streamflow assumption was based on the comparability of streamflow measurements made on the same day at the Bybee streamgage and at the mouth of Crystal Springs Creek. However, none of the measurements were made during extreme events. Therefore, the constant streamflow assumption is dropped, and those six lateral inflows are included in the model.

Model Calibration

The HEC-RAS model was calibrated by adjusting the upstream and lateral inflow hydrographs and the Manning’s roughness coefficient to obtain a satisfactory match of the streamflow and water surface elevation at the Bybee streamgage. The 0.01 and 0.002 AEP streamflow values reported by Stonewall (2014) were used as target calibration values at the Bybee streamgage (45.0 and 54.3 ft^3/s, respectively). There are no other streamgages in the watershed with continuous records of streamflow and stage data, leaving the Bybee streamgage as the only location where flood frequency analysis could be done with some degree of confidence.

After the initial model runs, results were assessed for validity, accuracy, and appropriate engineering practices. Areas of concern were addressed, including critical water-surface calculations, water-surface elevations between adjacent cross sections, and use of ineffective flow areas. The model was calibrated primarily by changing Manning’s n values. Because no recorded data approached the 0.01 AEP, additional model runs were added for calibration (table 2) and compared against recorded water-surface elevations at the Bybee streamgage. The largest of these events (January 19, 2012) was comparable to the 0.10 AEP and was modeled in plan HR31 (table 1). All smaller events were modeled in plan HR32. During calibration, HR31 was given more weight in the calibration than the other events because it is closer in size to the 0.01 and 0.002 events being evaluated. Simulated water-surface elevations at the Bybee streamgage were within 0.15 ft of measured elevations for all events. The average difference between simulated and recorded event at the streamgage was 0.03 ft.

Inflow Hydrographs

The calibration of the simulated lateral inflows and the upstream boundary condition hydrographs were divided into two elements. The first element was a constant, relatively large groundwater-supplied base flow based on the minimum values from the HEC-HMS model. The second element was a streamflow peak derived from the simulated precipitation event and is represented in the hydrograph as the area above the base flow (fig. 3). The base flow-driven elements of the lateral inflow hydrographs were left unchanged from the HEC-HMS model. The precipitation-driven elements of the hydrographs were scaled (factor *f* in fig. 4*B*) to cumulatively match the total streamflow estimated from the Bybee streamgage minus the base flow element. A theoretical example of such scaling is shown in figure 4. Precipitation-driven segments of the upper boundary condition and lateral inflow hydrographs were adjusted until an exact match with the target AEP streamflow was achieved at the Bybee streamgage in the HEC-RAS model.

Table 2. Comparison of water-surface elevations during the largest measured streamflows with the simulated water-surface elevation for plan HR32 at the Bybee streamgage, Crystal Springs Creek, Portland, Oregon.

[WS, water-surface elevation; ft, foot, ft^3/s, cubic foot per second]

Date	Streamflow (ft^3/s)	Measured WS (ft)	Modeled WS (ft)	Difference (ft)
01-19-12	33.7	51.97	51.95	-0.02
11-19-12	29.43	51.92	51.82	-0.10
11-22-11	26.6	51.70	51.74	0.04
01-18-12	25.54	51.62	51.70	0.08
09-28-13	25.2	51.55	51.70	0.15

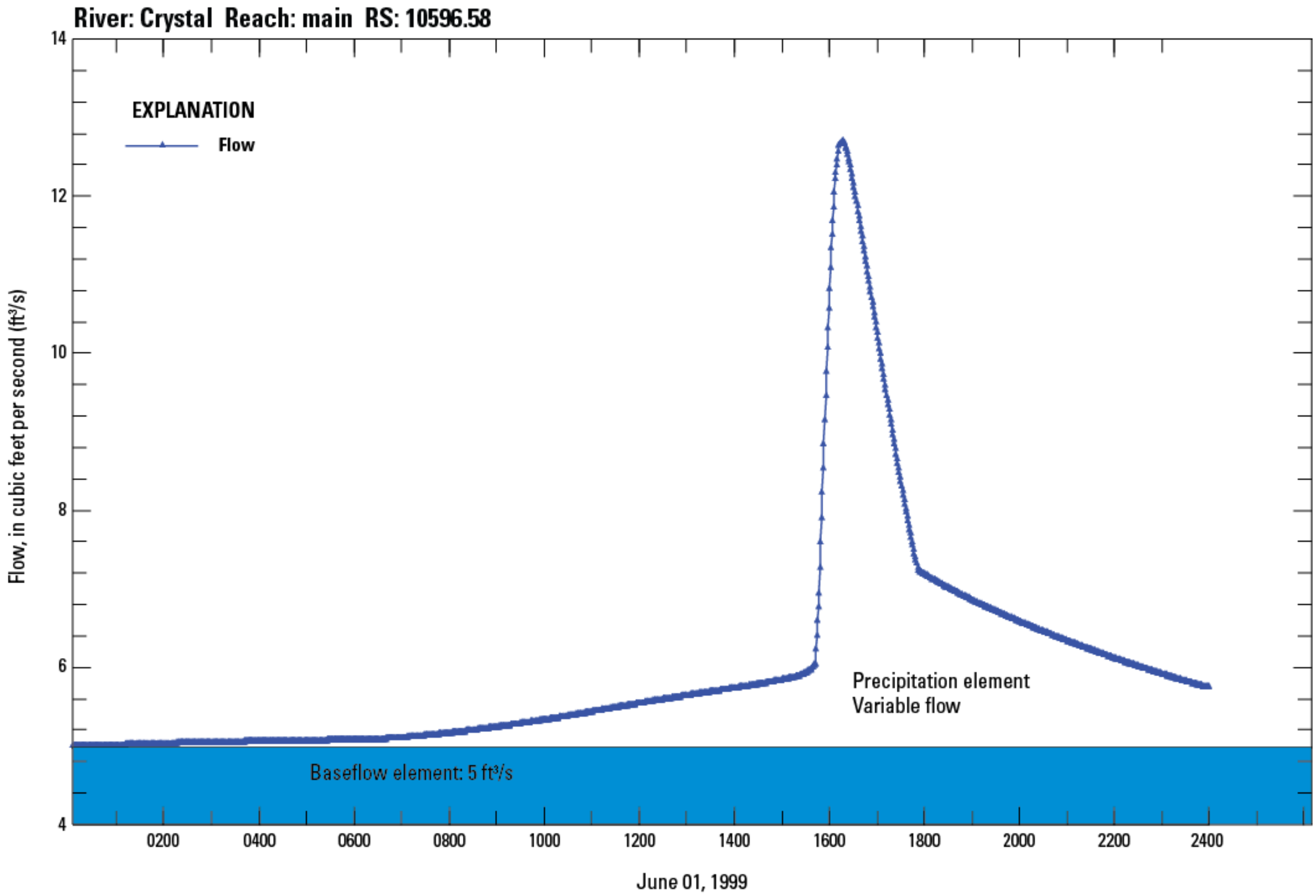

Figure 3. Graph showing example of base flow and precipitation elements of the upstream boundary condition (river station 105096.58), Crystal Springs Creek, Portland, Oregon.

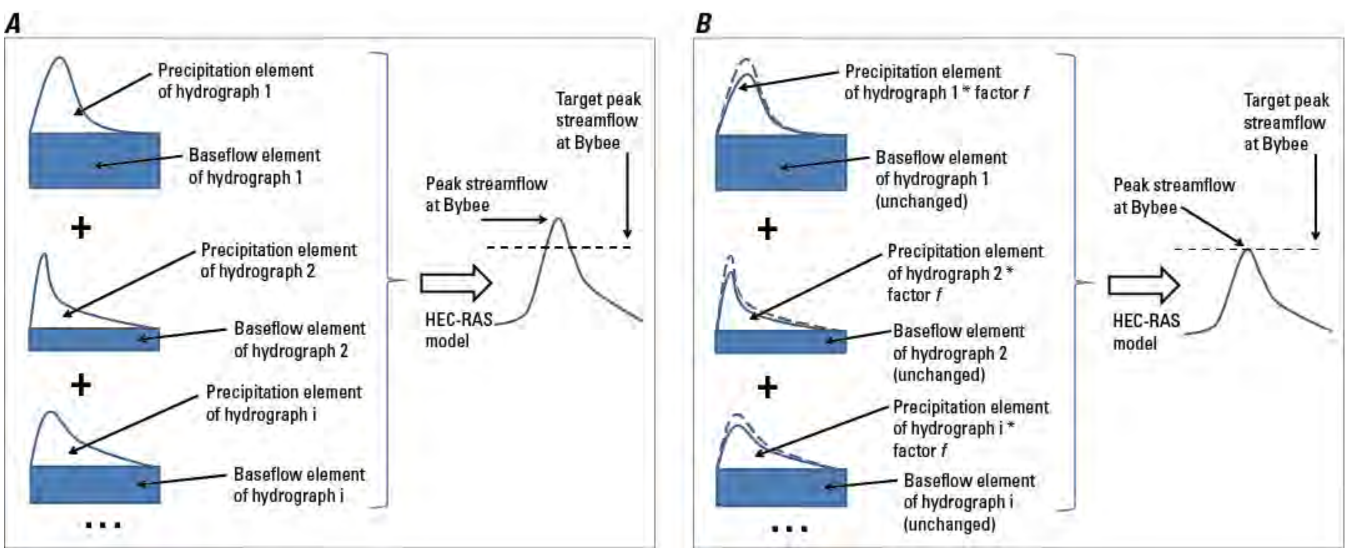

Figure 4. Diagrams showing theoretical examples of the calibration of the precipitation elements of input hydrographs for the Bybee streamgage, Crystal Springs Creek, Portland, Oregon.

Hydrographs from the calibrated profiles for the 0.01 and 0.002 AEP events (plans HR21 and HR23, respectively) at the Bybee streamgage were compared to the three highest measured hydrographs recorded at the Bybee streamgage (fig. 5) that were not the result of construction work or other direct human effects. Based on the frequency analysis by Stonewall (2014), these three peaks would represent AEPs of about 0.10, 0.26, and 0.52. Although the simulated hydrographs have a general shape and duration somewhat similar to measured data, the simulated hydrographs are steeper. The steepness for each 4-hour period was determined for the simulated and measured hydrographs. The measured hydrographs ranged in maximum steepness from 2.3 to 2.8 (ft^3/s)/hr. The simulated 0.01 and 0.002 AEP hydrographs had maximum steepness values of 6.3 and 8.2 (ft^3/s)/hr, respectively. Although it is presumed that larger streamflow events will result in steeper hydrographs, it is difficult to quantify reasonable levels of steepness for a specific amount of streamflow based on measured levels of steepness at another amount of streamflow. Additionally, precipitation distributions of individual storm events can differ greatly. For example, an intense, short-duration thunderstorm is likely to produce a steeper hydrograph than a 5-day-long frontal system that produces the same quantity of streamflow.

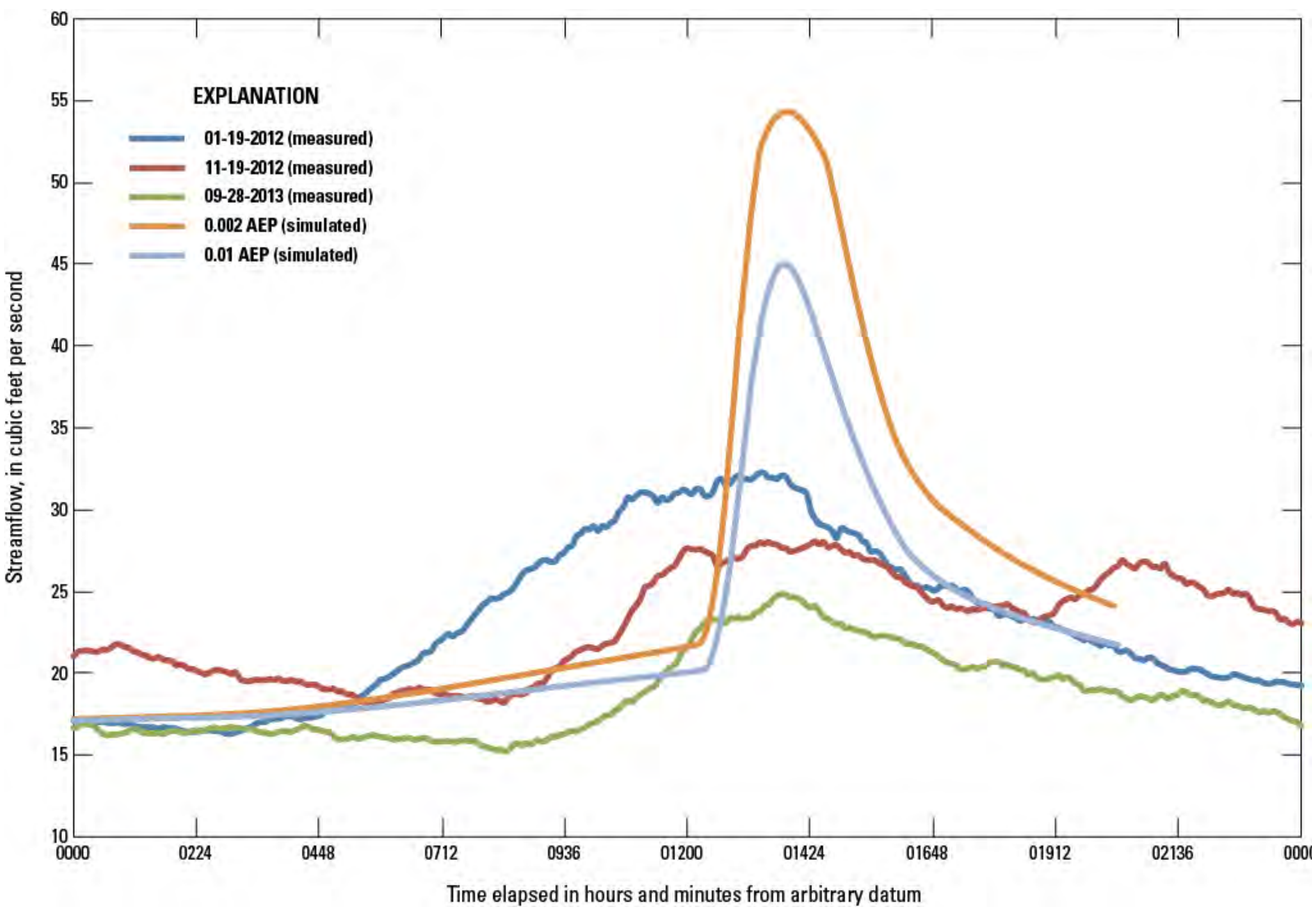

Figure 5. Hydrograph showing comparing simulated and measured streamflows at Crystal Springs Creek and Bybee Boulevard, Portland, Oregon.

To better evaluate the simulated hydrographs, the HEC-RAS model was calibrated for a 0.10 AEP event (33.2 ft^3/s; plan HR31), which is about the same streamflow as the highest measured at the Bybee streamgage that was not human-influenced (33.7 ft^3/s recorded on January 19, 2012)[2].

The modeled hydrograph for plan HR31 had a maximum steepness of 3.8 (ft^3/s)/hr, which was larger, but comparable to the maximum steepness of 2.8 (ft^3/s)/hr measured at the Bybee streamgage during January 19, 2012. The effect of the difference in hydrograph shapes is further explored in the section, “Sensitivity analysis.”

Simulated peak flows and base flows from plan HR21 were compared to the largest historical streamflow measurements at Reed College, SE 28th Avenue, SE Bybee Boulevard, and near the mouth (table 3). All simulated peak flows were greater than the highest, natural historical streamflow measurements. Measured streamflows greater than simulated peak flows associated with the 0.01 AEP streamflows would have suggested an underrepresentation of streamflow in the model, as the largest streamflow event that has occurred since the installation of the Bybee streamgage is estimated to have an AEP of 0.10 (Stonewall, 2014).

Table 3. Comparison of largest historical measured streamflows with simulated peak flows in model plan HR21, for Crystal Springs Creek, Portland, Oregon.

[HR21, HEC-RAS model plan calibrated to the 0.01 annual exceedance probability; ft^3/s, cubic foot per second]

Site	USGS station No.	HR21 base flow (ft^3/s)	HR21 peak flow	Highest measured streamflow (ft^3/s)	Date	Total number of measurements	Dates of streamflow measurements
Reed College	452856122380000	5.0	12.7	[1]7.15	01-08-99	44	1997–2010
28th Avenue	452855122381500	5.0	12.7	4.51	08-28-13	1	2013–2014
Bybee Street	14211542	17.0	45.0	20	09-30-97	16	1935–2001
Mouth	14211546	17.0	43.1	21.6	12-20-99	57	1964–2013

[1]On April 15, 1999, 11.2 ft^3/s was measured at Reed College. Twenty-one hours later, only 4.85 ft^3/s was measured at the same location. The hydrologist who made the streamflow measurements has postulated that this high streamflow measurement resulted from the removal of an upstream beaver dam (Karl Lee, U.S. Geological Survey, written commun., 2014). The Reed Canyon restoration manager has also corroborated the likelihood of a beaver dam removal at that time. The build-up of beaver dams is no longer permitted in the canyon, so similar high flushes of water such as those seem in April 1999 are no longer possible (Zac Perry, Reed College, written commun., 2014). Consequently, the April 15, 1999 streamflow measurement was not considered for this analysis. The second-highest streamflow measurement of 7.15 ft^3/s occurred on January 8, 1999, just after a large storm, and is thought to be more indicative of wet conditions.

[2] On January 19, 2012, 2.67 in. of rain were recorded at the Harney Street rain gage located within the Crystal Springs Creek watershed. This was the second-highest 1-day precipitation recording at that rain gage since it was installed in 1998. Additionally, nearly 2 in. of rain was recorded at the same gage on January 18, suggesting that the ground was saturated and water levels were elevated leading into January 19. The highest 1-day precipitation event on record at the Harney Street precipitation gage also occurred in 2012 (November 19, 3.15 in.). This precipitation event resulted in less streamflow at the Bybee streamgage (peak of 29.4 ft^3/s). This likely is because, at least in part, only 0.57 in. of rain were recorded at the rain gage on the previous day.

Manning's Roughness

Manning's roughness coefficients (Barnes, 1967) were estimated from field observations and satellite imagery. Estimates of Manning's roughness coefficients ranged from 0.04 to 0.06 for the main channel and ranged from 0.016 to 0.10 for the overbank areas. Estimates of Manning's roughness coefficients ranged from 0.04 to 0.05 for bridges and the bottom of culverts. All final Manning's roughness values were deemed within acceptable limits based on values published in Chow (1959) and Maidment (1993).

Flood Inundation Evaluation

Water-surface elevations associated with current culvert geometries and the 0.01 and 0.002 AEP events in the Crystal Springs Creek watershed generated from plans HR21 and HR23 are shown in figures 6 and 7, respectively. The simulated stream elevations the from HEC-RAS model were projected onto the land surface as represented by the lidar data using HEC-GeoRAS, a set of procedures, tools, and utilities to process HEC-RAS output in the ArcGIS™ (Esri®, 2014) geographic information system. The HEC-GeoRAS program can be used to prepare geometric data for import into HEC-RAS, and to process streamflow simulation results exported from HEC-RAS.

Both simulated inundations are largely confined in-channel, especially for the 0.01 AEP. Overbank flow during the modeled 0.01 AEP event occurs in sections of Westmoreland Park (figs. 6*B*–*C*), just upstream of SE McLaughlin Boulevard (fig. 6*A*), in the Eastmoreland Golf Course (fig. 6*A*) and in parts of Reed Canyon where the channel is not well-defined (fig. 6*A*). The water-surface elevations are also overbank due to backwater from the confluence with Johnson Creek (fig. 6*C*), as a result of the tie-in with the Johnson Creek FIS[3] (Federal Emergency Management Agency, 2010). Overbank inundation during the simulated 0.002 AEP event occurs to a greater extent in the same locations, as noted for the 0.01 AEP event and upstream of the railroad crossing (fig. 7*A*).

[3]As mentioned in section, "Boundary Conditions," all model plans were run with a normal depth downstream boundary condition. However, resulting outputs were then merged with existing FIS maps for Johnson Creek at the confluence with Crystal Springs Creek.

Figure 6. Aerial photographs showing plan views of inundation levels associated with the 0.01 annual exceedance probability (AEP) event from plan HR 21 with current culvert geometry of Crystal Springs Creek, Portland, Oregon. (*A*) Eastmoreland Golf Course area; (*B*) Westmoreland Park area; and (*C*) at the confluence with Johnson Creek.

Figure 6.—Continued

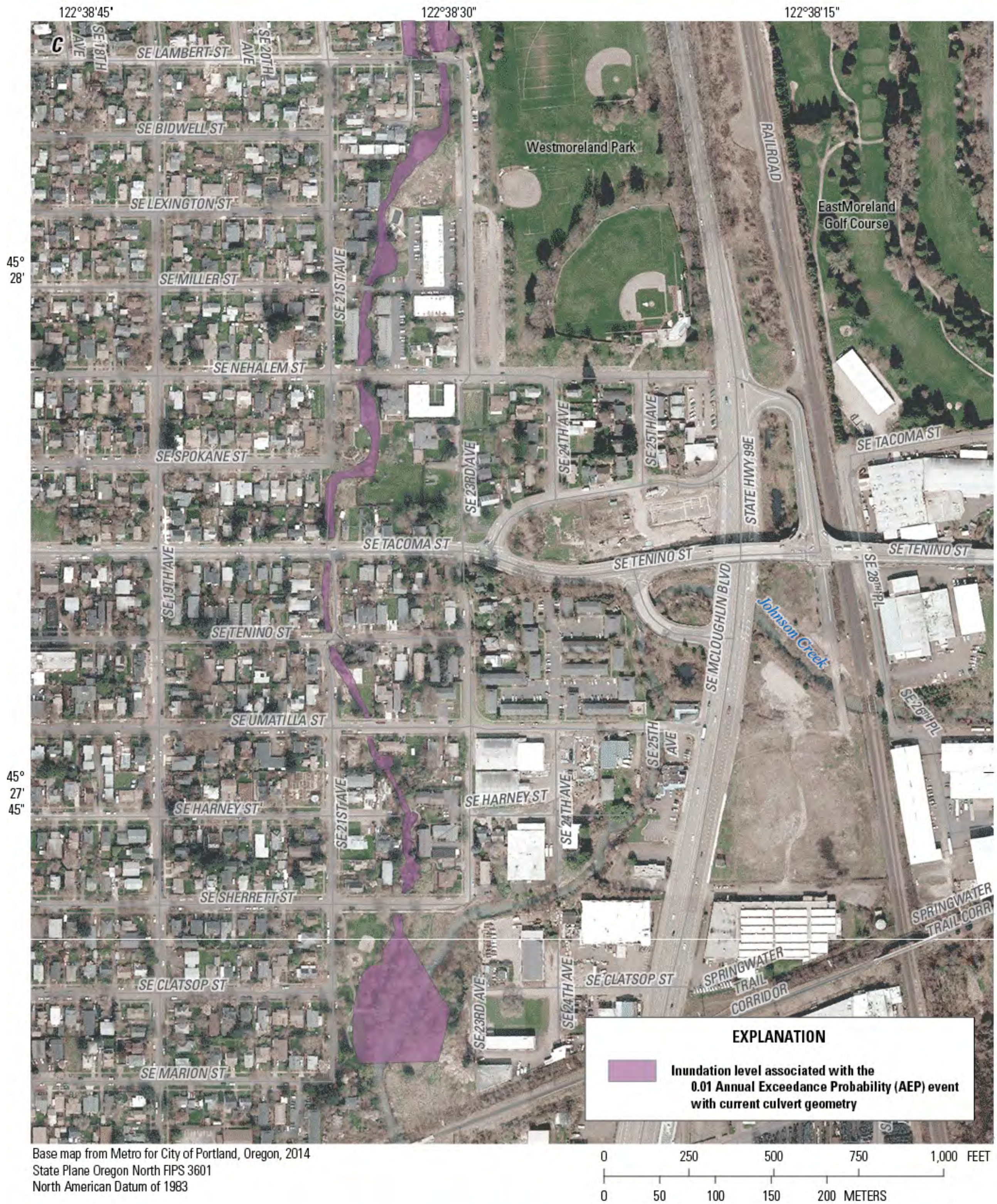

Figure 6.—Continued

Figure 7. Aerial photographs showing plan views of inundation levels associated with the 0.002 annual exceedance probability (AEP) event from plan HR23 with current culvert geometry of Crystal Springs Creek, Portland, Oregon. (*A*) Eastmoreland Golf Course area; (*B*) Westmoreland Park area; and (*C*) at the confluence with Johnson Creek.

Figure 7.—Continued

Figure 7.—Continued

Water-surface elevation profiles associated with current culvert geometries and the 0.01 and 0.002 AEP events in the Crystal Springs Creek watershed generated from plans HR21 and HR23 are shown in figure 8. As a result of the steep gradient and few streamflow inputs, streamflow upstream of SE 28th Avenue is relatively fast and shallow compared to elsewhere in the watershed. With the stream infrastructure currently in place, streamflow is most constricted at the crossings of Glenwood Street (river station 5,816 ft), Bybee Boulevard (5,552 ft), and Lambert Street (3,293 ft). Relatively minor constriction also occurs at the Miller Street crossing (2,434 ft).

Water-surface elevations associated with proposed culvert geometries and the 0.01 and 0.002 AEP events in the Crystal Springs Creek watershed generated from plans HR22 and HR24 are shown in figures 9 and 10, respectively. A profile of water-surface elevations associated with both plans is shown in figure 11. Both simulated inundation coverages closely resemble the corresponding plan that incorporated the current culvert geometry (plans HR21 and HR23, respectively). The largest discrepancies between the inundation areas of plans HR21 and HR22 were in close proximity of the Glenwood/Bybee project. Upstream of the project, plan HR21 resulted in a larger inundation footprint because of more constriction at the Bybee and Glenwood culverts. The profile (fig. 8) shows two abrupt drops in the water surface at the two crossings in the Glenwood/Bybee project and the backwater effects of these constricted openings for both AEP model runs. The abrupt drops and backwater effects as shown by the profile (fig. 11) for the proposed culvert are much less than shown for the current conditions (table 4). This difference between the two inundation footprints decreases farther upstream. Around 28th Avenue, the inundation coverages become nearly identical. Downstream of the Glenwood/Bybee project, the inundation coverage of plan HR22 is slightly larger than plan HR21. The larger inundation coverage of plan HR22 is the result of the proposed culverts having a greater cross-sectional area than the existing culverts. This larger area allows more streamflow to pass through the culvert, resulting in slightly increased flows. Conversely, the smaller cross-sectional area of the existing culverts result in more ponding upstream. This results from the proposed culverts having a greater cross-sectional area, allowing more streamflow to pass through rather than a more narrow constriction of flow which instead results in more ponding upstream of the culverts. Downstream attenuation affects plans HR21 and HR22 similarly, and by the time the stream leaves Westmoreland Park the inundation footprints of the two plans are nearly identical.

The differences between the inundation coverages of plans HR23 and HR24 are similar to those between plans HR21 and HR22. The differences are greatest near the project area, plan HR23 has a larger inundation footprint upstream of the Glenwood/Bybee project, and plan HR24 has a larger inundation footprint downstream of the project.

Table 4. Output results of selected cross sections near the Glenwood/Bybee project from the Hydrologic Engineering Center-River Analysis System (HEC-RAS), Crystal Springs Creek, Portland, Oregon.

[HEC-RAS cross sections are shown in plate 1. ft, foot; ft/s, foot per second; ft^2, square foot; ft^3/s, cubic foot per second]

HEC-RAS cross section	Plan	Streamflow (ft^3/s)	Water surface elevation (ft)	Average streamflow velocity (ft/s)	Cross-section area (ft^2)	Stream width (ft)
6061.021	HR21	44.96	53.60	0.67	68.02	43.16
6061.021	HR22	49.44	53.18	0.95	51.92	34.19
6061.021	HR23	54.41	54.06	0.62	90.09	52.29
6061.021	HR24	66.46	53.56	1.01	66.44	42.52
5759.804	HR21	44.94	53.13	0.56	79.65	32.38
5759.804	HR22	49.35	53.00	0.82	60.39	33.27
5759.804	HR23	54.37	53.49	0.60	91.52	33.63
5759.804	HR24	66.33	53.36	0.92	72.57	35.18
5344.092	HR21	41.15	52.22	0.73	58.53	32.36
5344.092	HR22	43.81	52.27	0.75	60.46	33.72
5344.092	HR23	50.11	52.47	0.78	67.64	39.24
5344.092	HR24	56.11	52.61	0.82	73.19	43.45
5057.581	HR21	40.94	52.18	0.47	86.63	81.71
5057.581	HR22	43.13	52.24	0.49	88.21	83.54
5057.581	HR23	49.73	52.44	0.52	102.85	101.10
5057.581	HR24	55.28	52.57	0.54	114.82	126.87

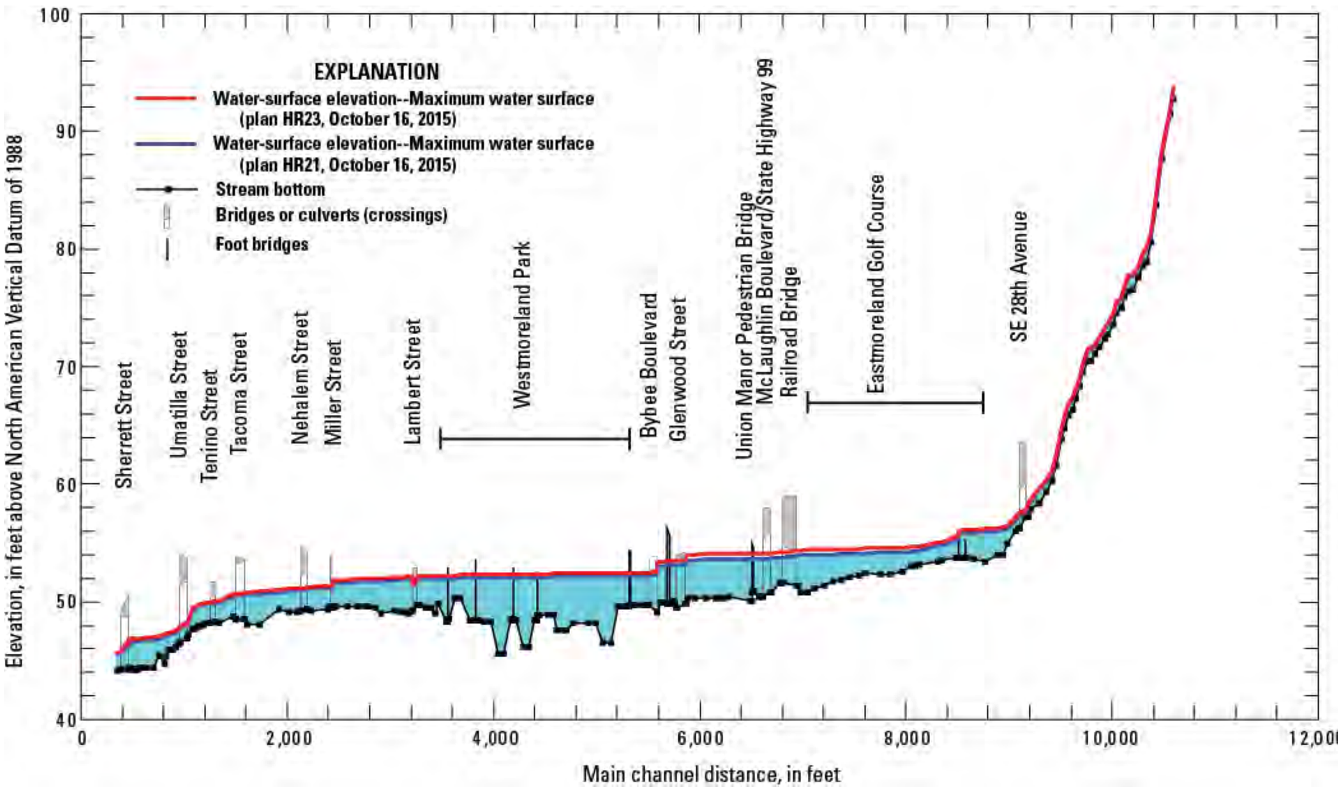

Figure 8. Profile showing water-surface elevation associated with the 0.01 (plan HR21) and 0.002 (plan HR23) annual exceedance probability (AEP) events and current culvert geometry of Crystal Springs Creek, Portland, Oregon.

Figure 9. Aerial photograph showing plan views of inundation levels associated with the 0.01 annual exceedance probability (AEP) event from plan HR22 with proposed culvert geometry of Crystal Springs Creek, Portland, Oregon. (*A*) Eastmoreland Golf Course area; (*B*) Westmoreland Park area; and (*C*) at the confluence with Johnson Creek.

Figure 9.—Continued

Figure 9.—Continued

Figure 10. Aerial photograph showing plan views of inundation levels associated with the 0.002 annual exceedance probability (AEP) event from plan HR24 with proposed culvert geometry of Crystal Springs Creek, Portland, Oregon. (*A*) Eastmoreland Golf Course area; (*B*) Westmoreland Park area; and (*C*) at the confluence with Johnson Creek.

Base map from Metro for City of Portland, Oregon, 2014
State Plane Oregon North FIPS 3601
North American Datum of 1983

Figure 10.—Continued

Figure 10.—Continued

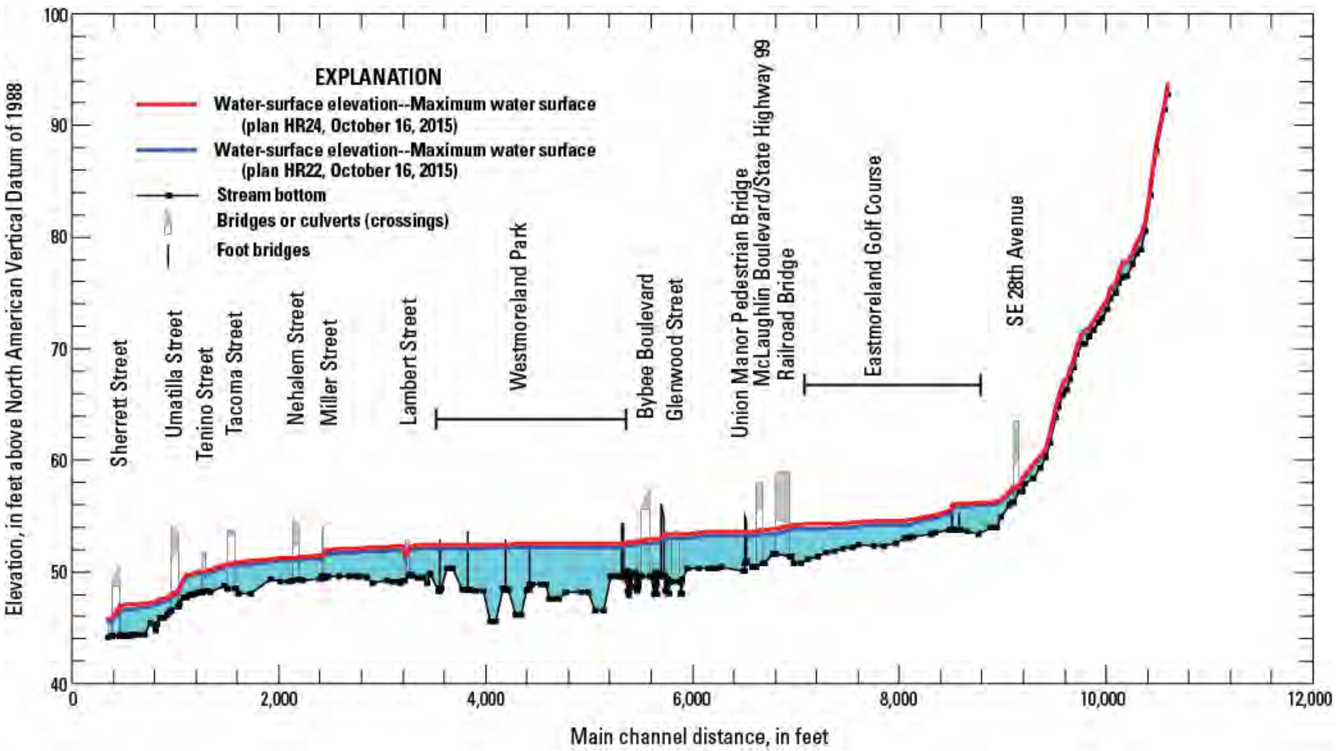

Figure 11. Profile showing water-surface elevations associated with the 0.01 (plan HR22) and 0.002 (plan HR24) annual exceedance probability (AEP) events with proposed culvert geometry of Crystal Springs Creek, Portland, Oregon.

Sensitivity Analysis

Three model plans were created for a sensitivity analysis of the Crystal Springs Creek model. Model plan HR27 was created to evaluate the assumed timing of the boundary condition and lateral inflow hydrographs. Following the results of the HEC-HMS model used to create the hydrographs, all hydrographs were set to peak within a 1 hour window for plans HR21–HR24, with the exception of the small lateral inflow hydrograph located at river station 7608.24. With plan HR27, lateral inflow hydrographs were timed to coincide with the peak of the main channel. For example, in HR21, the lateral inflow hydrograph at river station 8887.253 peaked at 1647 hours, whereas the streamflow of the main channel at the same location peaked at 1625. With plan HR27, the lateral inflow was temporally adjusted to also peak at 1647. This process was then repeated iteratively for each lateral inflow moving downstream. This approach simulates a weather system that follows the streamline at the same velocity as the streamflow, and produces a maximum flow event for the same amount of total precipitation. By comparing the results of plan HR27 and HR21, it is possible to estimate the maximum possible error that can be produced by the assumptions made for lateral inflow timing in plans HR21–24.

The difference in maximum water-surface elevation between plans HR21 and HR27 is generally larger downstream than upstream. This was expected, as the magnitude of the adjustment in the timing of the peak of lateral inflows increased moving downstream. Upstream of the railroad bridge maximum water-surface elevations are 0–0.05 ft higher for plan HR27 than plan HR21. Between the railroad crossing and the Nehalem Street crossing, water-surface elevations generally were between 0.03 and 0.11 ft higher. Downstream of Nehalem Street produced the biggest differences between the two plans, as maximum water surfaces associated with HR27 were between 0.08 and 0.44 ft higher. The average difference of all cross sections between the two plans was 0.09 ft.

Model plan HR26 was created to evaluate the difference between the hydrology assumptions of the existing FIS model (Federal Emergency Management Agency, 2010) and the steady flow model of the 0.01 AEP (plan HR25). A steady version of the new HEC-RAS model was developed and used to facilitate a direct comparison with the FIS model. As discussed in Stonewall (2014), the existing FIS model assumes much greater streamflows for both the 0.01 and 0.002 AEP. Consequently, simulated water-surface elevations for plan HR26 are much higher than plan HR25. Average maximum-water-surface elevations for the entire reach were 0.71 ft higher. The most pronounced difference was between Bybee Boulevard and SE 28th Street, where many of the cross sections were about 1.4 ft higher for HR26, including the maximum difference of 1.49 ft. For the rest of the watershed, differences generally were between 0.2 and 0.8 ft.

Plan HR25 also can be used to evaluate any discrepancies between a steady-flow plan and a similar unsteady plan (HR21). Profile 3 in plan HR25 was run with no attenuation between lateral inflow inputs. In this manner, plan HR25 can be used to estimate maximum differences with plan HR21 based on attenuation and storage calculations, the main factors that separate the former steady-flow model plan with the latter unsteady version. Peak water-surface elevations derived from plan HR25 Profile 3 were generally higher, but usually within ± 0.10 ft of the peak water-surface elevations derived from plan HR21. Slightly larger differences were simulated in areas where attenuation was greatest and near the two modeled storage areas (Golf Course and Median), resulting in differences as large as 0.33 ft. Only 15 of 200 cross sections had differences greater than 0.10 ft, and the average difference between the two plans was 0.03 ft. Overall, steady flow results did not differ substantially from the unsteady results, with the possible exception of a few areas of maximum flow attenuation.

Finally, plan HR33 was developed to evaluate the sensitivity of the model to hydrograph shapes. As noted previously, the synthetic boundary condition and lateral inflow hydrographs developed from the HEC-HMS model appear to be steeper than the hydrographs recorded at the Bybee streamgage. Plan HR33 was used to simulate the event of January 19, 2012, which approximated the calculated 0.10 AEP event. For plan HR33, a new geometry file was developed that begins just upstream of the Bybee Boulevard crossing (river station 5621.827) and still extends downstream to the confluence with Johnson Creek. The streamflow record from the Bybee streamgage was entered as the upstream boundary condition. Lateral input hydrographs were left identical to those from plan HR31.

Maximum water-surface elevations for plan HR33 were slightly higher but still similar to those of plan HR31. The average difference between the two plans was 0.10 ft, with a maximum difference of 0.25 ft. The greatest differences were concentrated in the lowest part of the watershed (downstream of Umatilla Street), where plan HR33 was typically between 0.15 and 0.25 ft higher. Upstream of Nehalem Street, the difference between the two plans at all cross sections was less than 0.10 ft. These increases in maximum water surface elevations were caused by the greater overall volume of streamflow of HR33 relative to HR31.

Suggestions for Future Research

Future streamflow studies for Crystal Springs Creek can be improved through the continued collection of streamflow data at the BES Bybee Boulevard streamgage. Currently streamflow is calculated at the Bybee streamgage by recording depth and velocity at a point in the stream and using a velocity/area equation. Streamflow data at the Bybee streamgage would be improved with regular physical streamflow measurement checks against the velocity/area equation. The collection of more hydrologic data (streamflow, high water marks, etc.) throughout the Crystal Springs watershed, especially during extreme precipitation events and high base flow periods, also would improve future analyses.

Summary

Water-surface elevations associated with various streamflows of Crystal Springs Creek were estimated using results from a previously developed hydrologic model in conjunction with a Hydrologic Engineering Center-River Analysis System (HEC-RAS) model developed for this study. Twelve HEC-RAS model plans were developed to evaluate multiple annual exceedance probability (AEP) events using current and proposed channel morphologies and culvert geometries, and to evaluate model sensitivity. Model sensitivity tests included changing the timing of lateral inflow hydrographs, modifying the hydrograph shape, evaluating differences between steady and unsteady versions of the model, and evaluating differences between AEP values from different studies.

HEC-RAS models were calibrated to the Bybee streamgage, which has been operated by the City of Portland Bureau of Environmental Services since 2011. Streamflow and water-surface elevations were calibrated to the previously calculated AEP values, 0.10, 0.01, and 0.002. Input hydrographs were calibrated using scaled outputs from a previously developed HEC-HMS model. Resulting hydrographs were compared against the highest flows recorded at the Bybee Boulevard streamgage. Model simulation results show the simulated 0.01 and 0.002 AEP hydrographs to be steeper than recorded hydrographs. The simulated 0.10 AEP hydrograph has a larger, but comparable, level of maximum steepness to the highest-recorded streamflow at the streamgage; both the measured and simulated events have similar magnitudes of streamflow. Further testing showed the difference in hydrograph shapes could produce differences in maximum water surface elevations along the length of Crystal Springs Creek averaging about 0.10 ft higher at the 0.10 AEP streamflow, with differences near the confluence of Johnson Creek up to 0.25 ft.

Flood inundation levels for the 0.01 and 0.002 AEP streamflows using both current and proposed culvert and stream geometry were largely confined in-bank. Exceptions occur in stretches of Westmoreland Park, near McLoughlin Boulevard and the railroad crossing, in Eastmoreland Golf Course, and in parts of Reed Canyon.

Model results show the proposed replacement culverts at the Bybee Boulevard and Glenwood Street crossings will result in lower water-surface elevations associated with the 0.01 and 0.002 AEP streamflows upstream of Bybee Boulevard, with the biggest drops in water-surface elevation occurring near the Glenwood/Bybee project. Immediately downstream of Bybee Boulevard, the proposed replacement culverts will produce small increases in both streamflow and water-surface elevations (less than 0.20 ft).

Sensitivity analyses showed that the timing of lateral inflow hydrograph peaks could potentially have a larger effect near the confluence of Johnson Creek than farther upstream. Water-surface elevations could be as much as 0.44 ft higher near the mouth and average as much as 0.09 ft higher for the study area. Model runs made using the larger AEP streamflow values from the existing Johnson Creek flood insurance study showed substantially more inundation. Other sensitivity analyses showed that a flattened hydrograph could result in slightly higher water-surface elevations and that the differences in water-surface elevations between the steady and unsteady versions of the model were small.

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Glossary

Lateral inflow hydrograph A boundary condition hydrograph that enters the system between the upstream and downstream boundary conditions.

Normal depth The depth of flow when the slope of the water surface and channel bottom is the same and the water depth remains constant. In Hydrologic Engineering Center-River Analysis System (HEC-RAS), a frictional slope is needed from the user to use normal depth as a downstream boundary condition.

Plan In HEC-RAS, a plan is a coupling of a geometry file and a flow file (steady or unsteady).

Profile In HEC-RAS, a steady flow plan can have more than one profile. Each profile can have different values of streamflow. In unsteady plans, each time increment is represented by a profile.

River station The distance in river feet from the mouth of Crystal Springs Creek to the cross section. For example, cross section 5489 is 5,489 ft from the mouth of Crystal Springs Creek along the path of the creek, not in a straight-line distance.

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