Letter of Map Revision (LOMR) Submittal
to the Willamette River FIS
UP Railroad Bridge to
Middle & Coast Fork Confluence
City of Springfield OR, Community Number – 415592
& Lane County OR, Community Number – 415591

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1.0 INTRODUCTION

1.1 Purpose of Study

The purpose of this study is to improve the accuracy of the FEMA Flood Insurance Study for a reach of the Willamette River in Springfield, Oregon. The reach is illustrated in Figure 1 and extends from the Union Pacific (UP) railroad bridge (formerly Southern Pacific) near River Mile 185.5 upstream to River Mile 187 at the confluence of the Middle and Coast Forks. The City of Springfield recently acquired new topographic data for the area which revealed that the flood data shown on the effective FEMA FIRM maps does not provide an accurate picture of the flood risk and therefore needs to be revised.

1.2 Authority and Acknowledgments

The hydrologic and hydraulic analyses for this study were performed by Northwest Hydraulic Consultants Inc. (**nhc**) under contract to the Wildish Land Company of Eugene, Oregon. The City of Springfield, Oregon was an active participant and proponent of the map update. This work was completed in May 2008.

1.3 Coordination

The following meetings were held with the City of Springfield and the Wildish Land Company. A representative from Lane County participated in the March 18, 2008 meeting.

Date	Location	Meeting Topic
March 18, 2008	City of Springfield Office	Review problems with the effective maps. Discuss preliminary study results and discuss the FEMA LOMR process.
May 1, 2008	City of Springfield Office	Review of the final study results.

The government agencies and consulting firms contacted for information relevant to this study include:

Agency / Consulting Firm	Information Provided
City of Springfield	New topographic mapping data
Minister & Glaeser Surveying	New channel surveys (bathymetry)

2.0 AREA STUDIED

2.1 Study Reach

This report describes an investigation of riverine flooding on the Willamette River within the City of Springfield, Oregon and unincorporated Lane County. The detailed study reach extends from the UP railroad bridge upstream 1.5 miles to the confluence of the Middle and Coast Forks of the Willamette River (see Figure 1). The lower 1 mile of the Middle Fork was also updated with new topography and bathymetry, to a location upstream of the Dorris Ranch Park site where the results match the effective FIS (Figure 1). Along the lower Coast Fork, original HEC-2 cross-sections were used to match results to the effective FIS.

2.2 Principal Flood Problems

As a result of upstream regulation, there have been very few large flow events in the last 40+ years. The largest flows during that time period occurred in 1996, first in January-February, then in the November-December timeframe. These events caused no significant damage along the study reach.

2.3 Flood Protection Measures

As stated above, the upstream watershed – both on the Middle and on the Coast Fork Willamette Rivers – is highly regulated. Furthermore, varying flow releases from the upstream reservoirs typically results in different timing of flood peaks on the Middle and Coast Fork which acts to further attenuate the flood peaks on the mainstem Willamette below the confluence.

There are no structural berms, levees, or other flood protection structures within the study reach.

3.0 ENGINEERING METHODS

3.1 Hydrologic Analysis

The flow quantiles used herein are the same as the effective study, and are given in Table 1. These include flows along the mainstem Willamette below the confluence as well as flows for the lower Middle Fork which is also modeled. Because this is a LOMR, rather than a detailed restudy, it was unnecessary to undertake a present hydrologic analysis. However, **nhc** did complete in 1999 a comprehensive river engineering study for the Willamette and McKenzie Rivers confluence, approximately 10 miles downstream, which included a detailed evaluation of basin hydrology. This study was undertaken after the large events in 1996 and evaluated stream gauge records, operating procedures of the various upstream dams, and data from the Corps of Engineers and others. The 100-yr discharge on the Willamette River (mainstem) at Springfield resulting from this study was 74,000 cfs. The 100-yr discharge at Springfield in the effective FIS is 71,000 cfs, which is within 5 percent of the updated value. The difference is small enough that it is appropriate to use the effective discharge for this LOMR update. The hydrology section of **nhc**'s 1999 study is attached to this report as Appendix A.

3.2 Hydraulic Analysis

An HEC-RAS model was created to define the hydraulic characteristics of the study reach. The model was used to compute water surface profiles for the 10-, 50-, 100-, and 500- year floods, floodplain inundation limits for the 100- and 500-year events, and floodway boundaries for the 100-year flood. Development of the model is described in the following paragraphs.

3.2.1 Channel and Floodplain Topography

Thirteen new cross-sections were obtained and used within the defined study reach. Seven of these locations correspond to the river location of the original FEMA HEC-2 cross-sections, with some adjustments made to improve overbank alignment and extent at a few locations. The remaining six cross-sections reside between the existing sections and improve the accuracy of the model by reducing the reach lengths and increasing geometric detail. Figure 2 compares the new cross-section locations to the original (effective) study. Where coincident, the new cross-sections retain the same numeric label in the model input; however, the alphabetical names on the work maps have been shifted to accommodate the new cross-sections. Reach lengths were recomputed due to the insertion of new sections and realignment of others.

The modified reach begins upstream of HEC-2 cross-section 301, at the upstream face of the UP railroad bridge (cross-section AO on the published FIRM), and extends to Middle Fork HEC-2 cross-section 70 (cross-section C on the published FIRM). Table 2 cross-references and clarifies the cross-section labeling with the river stationing in the models.

Floodplain topography for the cross-sections was provided by the City of Springfield in the form of TIN data and 2-ft contours from year 2000. These were merged together by **nhc** into a new TIN (also shown on Figure 2) with HEC-geoRAS used to cut each of the thirteen new cross-sections from the updated TIN. Minister & Glaeser Surveying was then retained in April 2008 to collect new bathymetric surveys of the thirteen cross-sections. These were merged directly into the cross-sections which were cut from the TIN to create a completely updated set of cross-sections within the study reach.

Upstream of the confluence along the Coast Fork Willamette, and its distributary Berkshire Slough, an HEC-2 model was created from hardcopies (PDFs) of the original model input. This model was imported into HEC-RAS and used to re-simulate the lower end of the Coast Fork to a point where the results match the effective study.

The final HEC-RAS model created for the LOMR actually begins 1.15 miles downstream from the UP railroad bridge. The cross-sections in this 1.15 mile reach are exactly the same as those used in the effective HEC-2 model. Results within this lower reach match the effective study, confirming the original HEC-2 results and providing appropriate tailwater conditions for the detailed study reach.

3.2.2 Hydraulic Structures

In addition to the UP railroad bridge, a pair of highway bridges is located about 1 mile downstream from the railroad bridge. Each is shown in Figure 1. Within the study reach itself there are no hydraulic structures.

3.2.3 Starting Water Surface Elevation

The tailwater reach of the HEC-RAS model begins at HEC-2 cross-section 265, AF on the effective FIRM and FIS. Known water surface elevations were used as the boundary condition at this location from the effective HEC-2 results. From there upstream to cross-section 301 (the UP railroad bridge), the results closely match the existing effective study. The detailed HEC-RAS study reach then begins upstream from cross-section 301.

Starting water surface elevations for the Coast Fork and Berkshire Slough HEC-2 model were taken from the mainstem HEC-RAS results at cross-section 340, which is located at the confluence and is also duplicated in the original HEC-2 model.

3.2.4 Model Calibration

No high water marks are available to re-calibrate the model.

Channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were taken directly from the original HEC-2 model. Based upon field observation and our engineering judgment, the roughness factors appear reasonable. The channel "n" values are 0.040 for all of the study reach except the upstream Middle Fork cross-section, section 370 in the updated HEC-RAS model (section 70 in the HEC-2 model), which is 0.045. Overbank "n" values range from 0.050 to 0.080.

3.2.5 Floodplain Discussion

The topographic floodplain data provided by the City of Springfield reveal a ridge of high ground along the right (north) bank of the lower Middle Fork, near the south boundary of Dorris Ranch Park, extending downstream from cross-section 360 (equivalent to HEC-2 section 60 on the Middle fork) to the confluence. The ridge then turns north and continues to about cross-section 335 on the mainstem Willamette (see Figures 1 and 2). During the 100-yr simulation there are a few low spots shown along part of this ridge, close to the confluence; however, the high ground will nonetheless prevent any significant conveyance of floodwaters landward of the ridge. The effective HEC-2 study assumed the entire width of the cross-sections in this area (330 and 340 on the mainstem and 50 on the Middle Fork) conveyed discharge. While much of this area within Dorris Ranch Park would indeed flood, it would primarily be due to backwater from downstream near cross-section 320. For the updated HEC-RAS model, appropriate ineffective areas were added to the right overbank floodplain from cross-section 330 through 350 (Middle Fork 50). This is a large portion of floodplain removed from active conveyance, resulting in up to a 2-ft increase in the Base Flood Elevations (BFEs) in the confluence area.

3.2.6 Flood Profiles

The flood profiles for the 10-, 50-, 100-, and 500-year events for the study reach generated using the HEC-RAS model are illustrated in Exhibit 1, Plates 01P – 03P. Flood profiles are also presented beyond the study reach along the Coast Fork Willamette, Plates 04P – 05P. The published (effective) profiles for Berkshire Slough are located upstream of the modified area and will not change. The downstream end of the mainstem profiles shown in Plate 01P merge seamlessly to the effective profiles in the downstream Willamette reach. Likewise, the upstream ends of the flood profiles as shown in Plates 03P and 05P merge seamlessly to the effective profiles at the respective locations on the Middle Fork, Coast Fork, and Berkshire Slough.

3.3 Vertical Datum

The effective maps and profiles use the NGVD 1929 datum; therefore, all results presented herein are also with reference to NGVD 1929. To convert elevations to NAVD 1988, the preferred datum for new studies by FEMA, add 3.7 ft (NGS, 1994).

4.0 FLOODPLAIN MANAGEMENT APPLICATIONS

4.1 Floodplain Boundaries

The revised floodplain boundaries for the 100- and 500-year events are shown on Plates 1 and 2 at scales matching the original FIRM panels, 1 inch equals 500 feet. Floodplain and floodway regions are shaded and cross-hatched in a manner consistent with typical FIRM maps. The base map and all FIS features have also been provided in digital form. Furthermore, Figure 3 directly compares the revised 100-yr floodplain boundaries and BFEs to the effective study, providing a visual picture of the proposed changes resulting from the updated study.

4.2 Floodways

The 100-year floodway boundaries developed in this study were determined with the HEC-RAS model, with the assumption of equal conveyance reduction from each side of the floodplain where the topography allows (HEC-RAS method 4). At some locations, namely from the confluence downstream about 0.5 mile, and the lower 0.5 mile of the study reach (above UP bridge) there is no appreciable left or right floodplain, respectively. Throughout the study reach, it was possible to encroach the floodway completely to the top of the channel banks without exceeding, and in many cases even reaching, a 1 foot rise. As required by FEMA, the floodway cannot encroach into the active channel. Near the confluence and along the right bank high ridge, this was largely enabled by the appropriate placement of ineffective flow areas landward of the ridge. By doing so, the majority of the discharge remains within and close to the river channel. Furthermore, there is an observable lowering of the channel bed using the new surveys (Figure 4) which also acts to keep more of the discharge in the channel. Note also that the regulated 100-yr discharge is on the order of 71,000 to 74,000 cfs, compared to the estimated unregulated discharge of 200,000 cfs (Appendix page A-2). This means the channel is oversized, which further explains the ability to encroach fully to the channel and also explains the lack of significant recent flooding within this reach.

Floodway widths were computed at each cross-section. Between sections, the floodway boundaries were interpolated and manually fitted. The results of the floodway analysis are tabulated for each cross-section in Table 3. The floodway boundary is also shown on the work maps, Plates 1 and 2, as well as Figure 3 where it’s compared to the wider effective floodway. Along the Coast Fork and Berkshire Slough, the floodway boundaries remain the same. In locations where the floodway and the 100-year floodplain boundary coincide, only the floodway boundary is shown.

5.0 BIBLIOGRAPHY AND REFERENCES

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FEMA (Federal Emergency Management Agency), 2002. Guidelines and Specifications for Flood Hazard Mapping Partners.

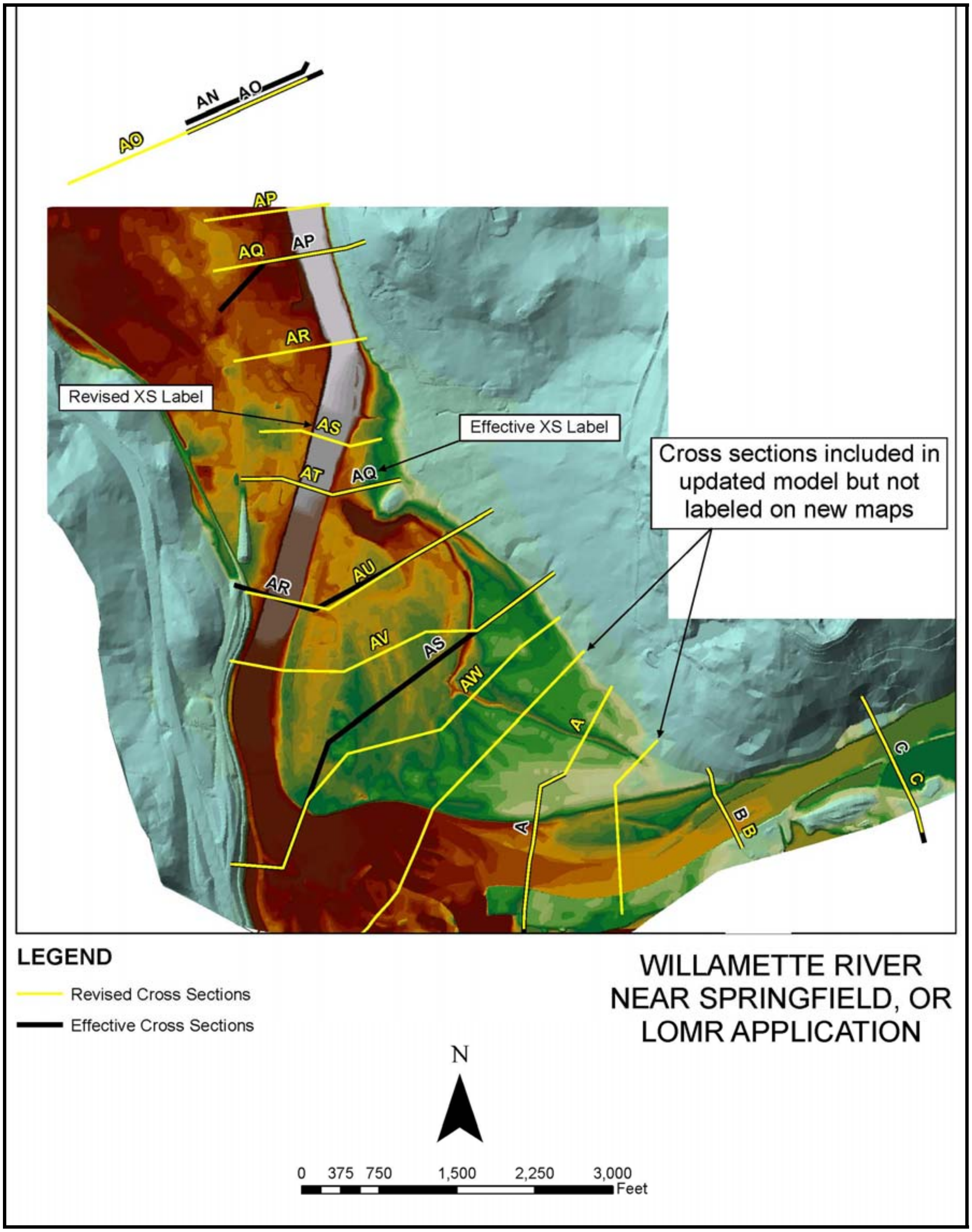
HEC (Hydrologic Engineering Center), 2001. HEC-RAS, Version 3.1.3, May 2005.

NGS (National Geodetic Survey), 1994. VERTCON, Version 2.0, May 5, 1994.

NHC, 1999. Willamette-McKenzie Rivers Flood Control and Restoration Project, Phase 1 – Draft Report, Baseline River Engineering Study, Northwest Hydraulic Consultants, Seattle WA, September 1999.

Figure 1: Location Airphoto Map

Figure 2: Cross-Section Locations with Floodplain Topography



placeholder for oversized Figure 3

Figure 4: Profile Comparison of Effective to Updated Study

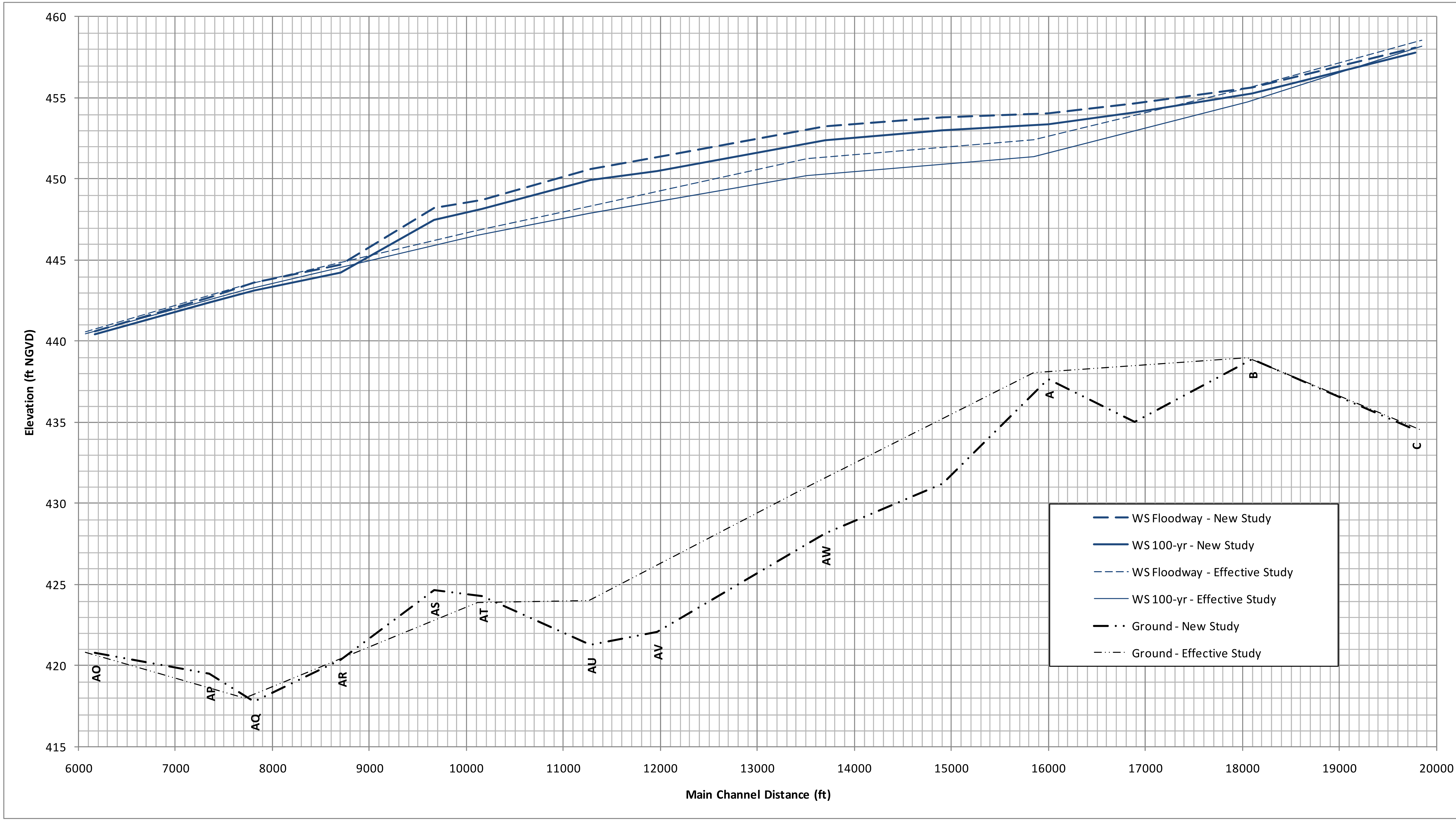


Table 1: **Flood Frequency Discharges for Study Reach**

Upstream HEC-RAS cross-section	Upstream FIRM cross-section	Description	Discharge (cfs)			
			10-yr	50-yr	100-yr	500-yr
370	C	Middle Fork Willamette inflow, upstream of Dorris Ranch Park	20,700	25,900	36,300	66,700
360	B	Adjustment to 500-yr per effective study, reflecting flow loss to Coast Fork	20,700	25,900	36,300	61,700
350	A	Adjustment to 500-yr per effective study, reflecting flow gain from Coast Fork	20,700	25,900	36,300	111,000
340	old AS new AW	Willamette (mainstem) combining Middle and Coast Forks	40,000	59,000	71,000	111,000

Table 2: Cross-Section Information

River Reach	Cross-Section Identifier: Effective FIRM	Cross-Section Identifier: Updated FIRM	Model River Station: Effective HEC-2	Model River Station: New HEC-RAS
Willamette (mainstem)	AO	AO	301	301
Willamette (mainstem)	N/A	AP	N/A	305
Willamette (mainstem)	AP	AQ	310	310
Willamette (mainstem)	N/A	AR	N/A	314
Willamette (mainstem)	N/A	AS	N/A	318
Willamette (mainstem)	AQ	AT	320	320
Willamette (mainstem)	AR	AU	330	330
Willamette (mainstem)	N/A	AV	N/A	335
Willamette (mainstem)	AS	AW	340	340
Willamette Middle Fork	N/A	N/A	N/A	345
Willamette Middle Fork	A	A	50	350
Willamette Middle Fork	N/A	N/A	N/A	355
Willamette Middle Fork	B	B	60	360
Willamette Middle Fork	C	C	70	370

Table 3: Floodway Data Table

Flooding Source			Floodway			Base Flood Water Surface Elevation		
Cross Section (updated)	River Station HEC-RAS	Distance (ft from d/s study reach boundary)	Width (ft)	Section Area (sq ft)	Mean Velocity (ft/s)	Without Floodway (ft)	With Floodway (ft)	Increase (ft)
AO	301	0.0	427	6922	10.3	440.5	440.6	0.1
AP	305	1181.9	343	6589	10.8	442.4	442.7	0.3
AQ	310	1640.0	353	6782	10.5	443.1	443.6	0.5
AR	314	2538.8	264	5152	13.8	444.2	444.7	0.5
AS	318	3493.4	391	7575	9.4	447.5	448.2	0.7
AT	320	3995.1	337	6909	10.3	448.2	448.7	0.5
AU	330	5110.4	398	8359	8.5	449.9	450.6	0.7
AV	335	5793.7	471	8555	8.3	450.5	451.3	0.8
AW	340	7525.5	999	16186	4.6	452.4	453.2	0.8
N/A	345	8736.9	1494	20607	3.4	453.0	453.8	0.8
A	350	9827.1	663	7810	4.6	453.3	454.0	0.7
N/A	355	10715.5	900	8059	4.5	454.1	454.7	0.6
B	360	11927.7	466	5307	6.8	455.3	455.6	0.3
C	370	13613.4	302	4305	8.4	457.8	458.1	0.3

APPENDIX A – HYDROLOGY

(based on NHC, 1999)

River discharge data were required as input to hydraulic modeling and analysis of the confluence area study reaches of the Willamette and McKenzie Rivers. The hydraulic model selected for use in this study, UNET, is a hydrodynamic model which simulates the dynamic characteristics of a flood event over time (see Appendix B). Thus, a time series of flow data rather than just a single (peak) value was required as input to the model. As described in Appendix B, the hydraulic model was calibrated to observed stage data on the Willamette and McKenzie Rivers for the November 1996 flood event. The calibrated model was then run to assess flooding conditions at the confluence area during the 100-yr design event. Therefore, river discharge data was required for the Willamette and McKenzie Rivers for both the November 1996 and 100-yr floods. The development of these flow data is described in detail below.

With regard to the 100-yr event, the intent of this study was to develop data for a single flood that would reasonably estimate 100-yr conditions on each of the rivers and their combined effect at the confluence. It is important to note that near the confluence, the relative timing of flood flows on the two rivers can be as important as the individual flow peaks in determining the extent of flooding. Because it is relatively flat, flooding on the Willamette River just upstream of the confluence is particularly sensitive to the coincidence of high flows on the two rivers. The McKenzie River is steeper and thus less sensitive to coincident flows. As described below in the following sections, the 100-yr flood hydrographs developed for this study matched the estimated 100-yr peak for each of the rivers independently, and matched the relative timing of discharges observed on the two systems during the December 1964 flood.

McKenzie River

The hydrology of the McKenzie River at the confluence is affected to some degree by upstream regulation. Projects operated by the U.S. Corps of Engineers at Blue River and Cougar are the primary flood control reservoirs in this basin. These reservoirs regulate discharges from approximately 294 square miles (22%) of the 1340 square mile basin upstream of the confluence. Other reservoirs in the basin include Clear Lake, Carmen Reservoir, Smith River Reservoir, and Trail Bridge Reservoir. These combine to regulate runoff from an additional 184 square miles (14% of the total basin). Flow data is available from three USGS gages located downstream of the Blue River and Cougar reservoirs on the McKenzie River near Vida, Leaburg, and Walterville (gages 14162500, 14163500, and 14163900). Historic flow data is also available from a USGS gage on the Mohawk River (gage 14165000), which discharges to the McKenzie River near Springfield. Flow data for the McKenzie River at the confluence was estimated using these gages in addition to flood flow frequency data available from the Corps.

Flow data for the November 1996 flood event were estimated for the McKenzie River using data from the USGS gage sites on the McKenzie River near Walterville and the Mohawk River near Springfield. Data for these gages were obtained from the USGS at a 30-minute time step for all of water years 1996 and 1997 (i.e. October 1, 1995 through September 30, 1997). The McKenzie River flow hydrograph at the confluence was then estimated by summing the coincident flows at these USGS gages and adding an incremental amount for local ungaged inflow. Local inflow was estimated based on the unit area discharge (cfs/mi^2) on the Mohawk River for this event, multiplied by the area between these gages and the confluence. The resulting estimated peak flow on the McKenzie River for the November 1996 event was 45,600 cfs (approximately a 5-year event). Flow data for the February 1996 event were also estimated using a similar procedure to the November flood. This data indicated that the peak discharge on the McKenzie River during the February event was approximately 74,700 cfs, or about a 50-year event.

The 100-year peak discharge on the McKenzie River was estimated using flood flow frequency data from the Corps of Engineers for the USGS gage site near Vida and on the Mohawk River. To supplement these data, the 100-year discharge for the USGS gage on Gate Creek (a tributary to the McKenzie River downstream of Vida) was estimated by **nhc** using the Corps of Engineers HEC-FFA flood flow frequency program (USACE, 1992). The 100-year discharge on the McKenzie River at the confluence was then estimated by USACE summing the 100-year discharges at Mohawk River, Gate Creek and the McKenzie River near Vida and adding the estimated local inflow between these gage locations and the confluence. The 100-year discharges on the Mohawk River, Gate Creek and the McKenzie River near Vida were determined as 15,500 cfs, 9,040 cfs, and 43,700 cfs, respectively. The local inflow was estimated as 21,200 cfs on the basis of unit area runoff for unregulated flow at gages in the McKenzie River basin. Thus, the total 100-year discharge was estimated by summing these values to be 89,400 cfs. Because this value was approximately equal to FEMA's estimate of the 100-year discharge (89,900 cfs) it was decided to use the slightly higher, thus more conservative, FEMA value for the hydraulic modeling.

Willamette River

The hydrology of the Willamette River is controlled to a much larger degree than the McKenzie River by dams and reservoirs upstream of the confluence. The U.S. Army Corps of Engineers operates five major projects upstream of the City of Eugene. These are Dorena and Cottage Grove on the Coast Fork Willamette River and Lookout Point, Hills Creek, and Fall Creek on the Middle Fork Willamette River. Combined, these projects regulate runoff from approximately 1544 square miles (75%) of the 2052 square mile catchment upstream of the confluence with the McKenzie River (USGS, 1998). Discharges at the USGS gage at Springfield, approximately 10 miles upstream of the confluence, are reduced from a natural (unregulated) 100-year peak discharge of 200,000 cfs to a regulated peak discharge of 74,000 cfs by these projects (USACE, 1999).

Flow data for the November 1996 flood event were estimated for the Willamette River using data from several of the USGS gage sites shown in the following table.

Principal USGS Streamflow Gages Used in Confluence Study

Gage Number	Gage Name	Basin Area	Period of Record
14153500	Coast Fork Willamette River below Cottage Grove Dam	104 mi^2	Jan. 1939 - present
14155500	Row River near Cottage Grove	270 mi^2	Jan. 1939 - present
14157500	Coast Fork Willamette River near Goshen	642 mi^2	Aug. 1905 – Feb. 1912 Oct. 1950 - present
14152000	Middle Fork Willamette River at Jasper	1340 mi^2	Sept. 1905 – Mar. 1917 Oct. 1952 - present
14162500	McKenzie River near Vida	930 mi^2	Sept. 1924 - present
14163500	McKenzie River below Leaburg Dam, near Leaburg	1030 mi^2	Oct. 1989 - present
14163900	McKenzie River near Walterville	1081 mi^2	Oct. 1989 - present
14165000	Mohawk River near Springfield	177 mi^2	Sept. 1935 – Sept. 1952 Oct. 1963 – Sept. 1997
14166000	Willamette River at Harrisburg	3420 mi^2	Oct. 1944 - present

Data for the gages were obtained from the USGS at a 30-minute time step for all of water years 1996 and 1997. The Willamette River flow hydrographs at the upstream end of the study reach were then estimated by summing the coincident flows at the USGS gages at Goshen and Jasper (gages 14152000 and 14157500) and adding an incremental amount for local inflow. Local inflow was estimated based on the unit difference in flows (cfs/mi^2) between the gage at Goshen and the sum of the gages on the Coast Fork Willamette River below Cottage Grove Dam and on the Row River near Cottage Grove. The local inflow between Jasper/Goshen and the upstream end of the study reach was found by multiplying the unit area runoff for the Coast Fork by the area between these sites (46 mi^2).

The Corps of Engineers has estimated flood flow frequency quantiles for numerous locations throughout the Willamette River basin for natural (i.e. unregulated) and regulated conditions (USACE, 1999). These locations include the Coast Fork Willamette River near Goshen, the Middle Fork Willamette River near Jasper, the Willamette River at Springfield, and the Willamette River at Harrisburg. The 100-year peak discharge at the confluence was estimated as the Corps 100-year discharge on the Willamette River at Springfield (74,000) plus an incremental additional amount corresponding to the local runoff between the Springfield gage and the confluence. The incremental amount was estimated based on the unit area local runoff on the McKenzie River (cfs/mi^2) for the 100-year event multiplied by the basin area between Springfield and the confluence. The resultant estimated peak discharge for the 100-yr event at the confluence was 74,600 cfs. This is the value used in the hydraulic modeling evaluation described in Appendix B. It is higher than the 100-yr peak discharge estimate published by FEMA (71,000 cfs) (1998); however, it is believed to be more reasonable as the FEMA value is based upon relatively old hydrologic data that pre-dates 1980.

100-year Event Hydrographs

As described above, 100-year peak discharges were estimated for the McKenzie and Willamette Rivers at their confluence. However, additional data is required as input to the hydraulic model used in this study. The hydrodynamic model (UNET) requires a complete hydrograph of flows (i.e. a time series of flows through the entire flood event). To develop these hydrographs requires assumptions about the shape of the individual hydrographs as well as the joint probability of flooding and the concurrence between peak discharges on the two rivers. A comprehensive evaluation of these issues was beyond the scope of this study. For purposes of the hydraulic modeling the following steps were taken to develop flood hydrographs:

McKenzie River

1. The 100-year discharge on the McKenzie River was estimated as 89,900 cfs as described above.
2. The flow hydrograph for the February 1996 flood was recreated from available USGS gage data as described above, resulting in a peak discharge of 74,700 cfs.
3. This recreated McKenzie River flow hydrograph was scaled up by a factor of 1.20 (89,900 cfs / 74,700 cfs) to match the estimated 100-year peak discharge. This results in a flood hydrograph with a peak discharge of 89,900 cfs and volume and recession characteristics that match the February 1996 flood.

Willamette River

4. The December 1964 storm was selected as a pattern hydrograph for the 100-year flood on the Willamette River. Review of the 1964 Willamette River flood hydrograph showed that it was a double peaked event with an initial (smaller) peak occurring early in the event, approximately coincident with the peak flow on the McKenzie River, and a second (larger) peak occurring several days later. This hydrograph was felt to be representative of flows that would be expected from large flood events on this system, with the first peak being caused by local runoff from the area below the flood control projects and the second peak resulting from releases (including spill) from the projects themselves. The flow hydrograph for the combined Willamette River flow at Jasper and Goshen for the December 1964 flood event was estimated from figures provided in the Corps Postflood report (USACE, 1966). The combined peak discharge at these gages was estimated as 65,000 cfs.
5. The flow hydrograph for flows on the Willamette River at the Springfield gage was estimated by scaling the 1964 hydrograph up to match the estimated 100-year discharge. The scaling factor used in this estimate was 1.14 (i.e. 74,000 cfs / 65,000 cfs).
6. Local inflows on the Willamette River between the Springfield gage and the confluence were estimated using unit area local inflows on the McKenzie River.

Combined Flows

7. The relative timing of the 100-year flow hydrographs on the two rivers was estimated by assuming that the first peak on the Willamette River essentially corresponded to the peak discharge on the McKenzie River (as seen during the December 1964 event). Thus the second (larger) peak on the Willamette River occurred about 4 days later than the peak on the McKenzie River. Thus, hydrographs for the two rivers at their confluence were estimated for about a 15 day period centered on the McKenzie River flood peak. Using this procedure the combined

discharge at the confluence (i.e. the coincident total flow) at the time of the McKenzie River peak was 155,500 cfs. The combined discharge at the time of the Willamette River peak was 102,500 cfs. Individual 100-yr inflow hydrographs were estimated for the McKenzie River at the confluence, for the Willamette River at Springfield and the local inflows between Springfield and the confluence. The flow hydrographs were exported in digital format for input to the hydraulic modeling.

In order to estimate existing levels of protection against various flood levels less than (occurring more frequent than) the 100-yr event, discharge hydrographs for the 2-, 10-, and 50-yr floods were estimated in a very approximate manner by applying scaling factors to the 100-yr flood hydrographs. Instantaneous peak discharges for gages on the Willamette River at Springfield, the McKenzie River near Vida, and the Mohawk River near Springfield (McKenzie tributary), taken from frequency analysis of the 2-, 10-, 50-, and 100-yr floods, were first summed together. The summed flows for each of these events provided a reasonable estimate of the total peak flow in the system (Willamette/McKenzie combined) for scaling purposes. For each lesser event, the 100-yr flood hydrographs on both rivers were then multiplied by the appropriate scaling factor from these estimated peak flows to derive approximated discharge hydrographs. For example, the 10-yr hydrographs were estimated by multiplying the developed 100-yr hydrographs by (10-yr peak/100-yr peak), where the 10- and 100-yr peaks represent the values summed together from the gage locations. For a given event, the same scaling factor was used on both the Willamette and the McKenzie Rivers.

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