King County

Memorandum

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To: Project File

From: Chris Brummer
Project No./Name: FL9001 / 1112049 Countyline Levee Setback Project
Subject: 8th Street East Bridge Scour Analysis

Introduction

This memorandum describes the methods and results of a scour analysis performed for the 8th Street East (Stewart Road SE) Bridge located on the Lower White River at river mile (RM) 5.0 in the City of Sumner. This scour analysis was performed to evaluate potential increases in the scour depth at the bridge piers resulting from the construction of the Countyline Levee Setback project proposed upstream of the bridge on the left bank of the Lower White River. The scour analysis was also performed to refine estimates of the channel bathymetry used in the hydraulic modeling performed in support of the Countyline project design and to evaluate whether the anticipated scour at the bridge piers influences the water-surface elevation predicted by the hydraulic model.

Background

The 8th Street East Bridge was constructed in 1952 by Pierce County and is currently owned and maintained by the City of Sumner. The bridge is 232 feet long and includes two in-water piers, each supported by a pile group and concrete pile cap. A profile of the bridge extracted from the 1952 construction drawings is shown in Figure 1.

The elevations shown in Figure 1 are in feet relative to the National Geodetic Vertical Datum of 1929 (NGVD 29) and must be increased by 3.5 feet to convert to the North American Vertical Datum of 1988 (NAVD 88), which is the datum used throughout this memorandum. The “anticipated high water” elevation of 62.0 feet NGVD 29 (65.5 feet NAVD 88) shown in Figure 1 corresponds to a design flow of 19,000 cubic feet per second (cfs) based on the stage-discharge relationship developed at the bridge location for channel conditions as existed in 1949. Since 1952 and since the cessation of river channel dredging operations in the late 1980s, the average channel-bed elevation at the bridge crossing has risen approximately 6 feet. Incidentally, the elevation of the top of the existing gravel bar at the bridge crossing (as shown in Figure 2) is within 1 foot of the 1949 “anticipated high water” elevation. The results of recent, two-dimensional hydraulic modeling of the modern 100-year recurrence interval flow of 15,532 cfs indicate a water-surface elevation of 71.5 feet at the bridge crossing (Herrera 2013).

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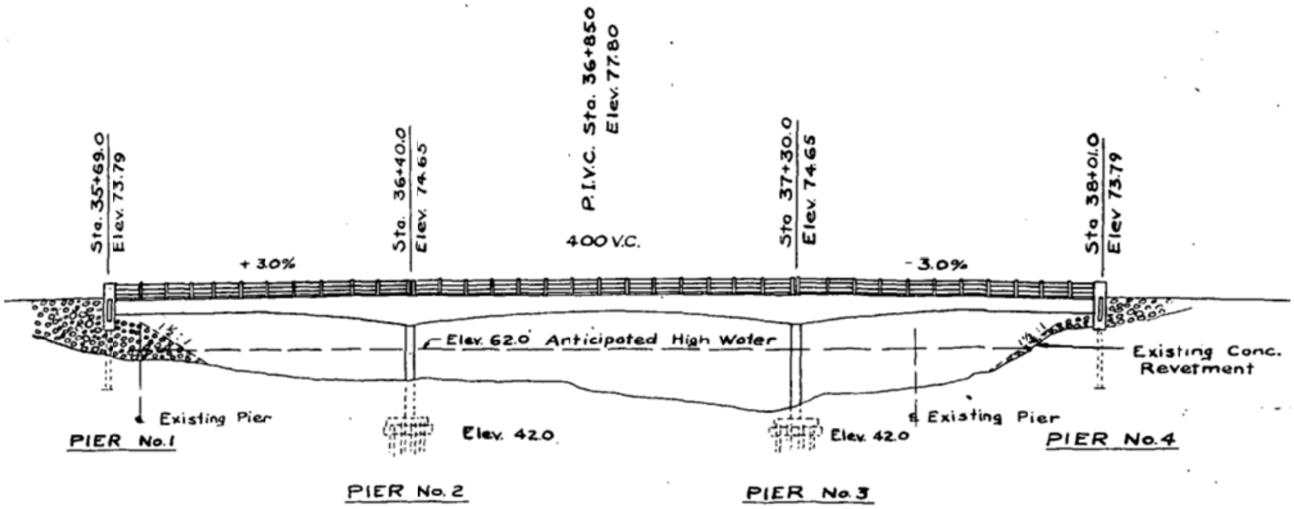

Figure 1. Bridge profile (looking upstream) extracted from the 1952 construction drawings. Elevations are relative to NGVD 29; see text below for additional explanation.

Figure 2. Photograph of the 8th Street East Bridge under existing conditions (July 25, 2011), looking downstream (compare to Figure 1) and showing accumulations of large wood on the upstream faces of the bridge piers.

The concrete pier caps for piers 2 and 3 were constructed at an elevation of 45.5 feet, which is 6 feet below the historical thalweg elevation of 51.5 feet shown on the 1952 construction drawings and 8.5 feet below the existing (2012) thalweg elevation of 54 feet. Scour below this depth could reduce the friction length of the piles and increase the risk of pile buckling, pier settling, and tilting (Lagasse et al. 2010).

The two in-water bridge piers are prone to the accumulation of woody debris, as shown in Figure 2. Documentation provided by Pierce County (the prior bridge owner) indicates that wood was removed from the bridge crossing on at least two occasions (e.g., 1992 and 1999). Woody debris increases the effective pier width and can increase the calculated scour depth (Lagasse et al. 2010).

Two major utilities are suspended beneath the bridge. They include an 18-inch diameter water main on the downstream-facing side of the upstream concrete arch girders (with the pipe and support brackets suspended below portions of the arch girders) and a 4-inch diameter natural gas main on the downstream-facing side of the downstream concrete arch girders.

The Countyline Levee Setback project is located upstream of the bridge crossing and will involve the removal of significant portions of the existing left bank levee and the construction of a new setback levee along the eastern margin of a 77-acre wetland complex in the historical floodplain that will have a southern tie-in point at the eastern approach to the bridge. Under existing conditions, only two-thirds of the 100-year recurrence interval flow passes under the 8th Street East Bridge. The remaining one-third of flow overtops the southeastern wetland boundary, flows south along 142nd Avenue, and overtops the 8th Street East (Stewart Road SE) roadway east of the bridge (Herrera 2013). The new setback levee will eliminate this split flow and direct the entire 100-year flow under the 8th Street East Bridge.

Methods

The maximum probable scour depth was calculated at both in-water piers for four flow conditions or simulations defined by previous geomorphic response scenarios and hydraulic modeling completed by Herrera (2013) for the Countyline Levee Setback project. They include:

1. existing (2011) conditions (designated simulation S4a),
2. conditions immediately following construction (S1a),
3. future, no-action conditions after 15 years of continued sediment aggradation (S4c), and
4. future conditions 15 years after construction (S1c).

Hydraulic data (flow depth and velocity) for all simulations were extracted from the two-dimensional hydraulic models developed for the Countyline project. All calculations assumed flow conditions for the modern 100-year recurrence interval event (peak flow of 15,532 cfs).

The maximum probable scour depth at the bridge piers was calculated using the methodology presented in the Hydraulic Engineering Circular No. 18 (HEC-18) published by the U.S. Federal Highway Administration (Richardson and Davis. 2001). The HEC-18 method used in this analysis is based on the Colorado State University (CSU) Equation for live-bed scour with a correction factor (K_4) for armoring:

$$\frac{y_s}{a} = 2.0\, K_1\, K_2\, K_3\, K_4 \left(\frac{y_1}{a}\right)^{0.35} Fr_1^{0.43}$$

where

y_s = scour depth (feet)

a = pier width (2 feet)

y_1 = flow depth upstream of pier (based on 2-D hydraulic model, in feet)

K_1 = correction factor for pier nose shape (1.0 for rounded)

K_2 = correction factor for angle of attack of flow (2.74 for pier length of 29 feet and angle of attack of 15 degrees)

K_3 = correction factor for bed condition (1.1 for plane bed)

K_4 = correction factor for armoring (0.60, based on critical velocity to mobilize median-size and 99th percentile-size bed material)

Fr_1 = Froude number upstream of the pier = $V_1/(gy_1)^{1/2}$

V_1 = mean velocity of flow upstream of the pier (feet/sec)

g = acceleration of gravity (32.2 feet/sec^2)

The pier width of 21 inches shown on the 1952 construction drawings was rounded up to 2 feet for the scour analysis. The pier length (29 feet) used in the analysis to determine the K_2 correction factor is the dimension shown on the 1952 construction drawings. In addition to determining the scour depth, the lateral and streamwise extents of scour at both in-water piers were determined using the methods described by Lagasse et al. (2010). The lateral and upstream extents of scour were calculated by assuming a scour hole with 2H:1V (horizontal to vertical) slopes extending up from the sides and upstream face of each pier. The downstream scour extents assumed a scour hole with 4H:1V slopes extending downstream of the downstream end of each pier.

The accumulation of woody debris on the bridge piers was not considered because wood loading is expected to occur with or without the project and because the primary goal of this analysis is to evaluate the relative change in scour depth resulting from the change in hydraulics at the bridge due to the project. Although the project may result in an initial increase in wood flux as channels form in the reconnected forested floodplain, the project reach is expected to eventually trap more wood in the near future than it does under existing conditions, thereby reducing the potential for future wood loading on the bridge piers after construction. Additionally, wood recruitment from the floodplain and transport downstream is highly dependent on the frequency, magnitude, and duration of future flow conditions, which is uncertain.

Results

The results of the scour analysis are presented Table 1. Because the bed elevation is different for each flow simulation in Table 1 (due to anticipated sediment aggradation), the elevation of the scour hole relative to the elevation of the pile cap (45.5 feet) is more informative than is the relative scour depth. Under existing conditions (with approximately two-thirds of the flow passing under the bridge), the maximum probable scour depth for the 100-year flow event is estimated to be 6.33 to 6.59 feet, which results in a minimum separation of 10.4 feet above the pile caps. Following project construction, with the passage of all of the 100-year flow under the bridge, the maximum probable scour depth is anticipated to increase by about 2 to 3 feet to a scour depth of 8.34 to 9.34 feet, which would result in a minimum separation of 7 feet above the pile cap. The relative scour depth is expected to decrease by about 2 feet in 15 years if the project is not constructed because of sediment aggradation at the bridge and because additional flow would split from the main channel as the river bed upstream continues to fill with sediment. This outcome also assumes unobstructed flow through the current avulsion path (along 142nd Avenue SE), which has been invalidated by fill placed after the 2011 survey was collected for the hydraulic model. Fifteen years after construction, the scour depth is expected to increase slightly by 1 to 2 feet relative to existing conditions and result in a scour hole during the 100-year event that is a minimum of 8.2 feet above the pile cap. The increase in scour depth (relative to existing conditions) is due to an increase in flow velocity (as more flow is directed under the bridge) and the flow constriction caused by ongoing sediment aggradation in the vicinity of the bridge.

Conclusions

Although the Countyline project will increase the flow rate under the 8th Street East Bridge relative to existing conditions, the post-project flow rate will be less than the historical flow conveyance capacity during the period when the channel was subject to regular maintenance dredging. In fact, the greatest historical flow at the bridge crossing (15,200 cfs in 1986, when channel conveyance was considerably greater than it is today) is within 300 cfs of the current estimated 100-year flow of 15,532 cfs. Additionally, the maximum probable scour depth after construction as determined in this analysis will be within a minimum of 7 feet above the elevation of the pile caps. Thus, the project is not expected to threaten the structural integrity of the bridge piers.

References

Herrera. 2013. Final Hydraulic Modeling Results Technical Memorandum. Prepared for King County Department of Natural Resources and Parks, Water and Land Resources Division, River and Floodplain Management Section, by Herrera Environmental Consultants, Inc., Seattle, Washington. June 14, 2013 Draft.

Lagasse, P.F., P.E. Clopper, L.W. Zevenbergen, W.J. Spitz, and L.G. Girard. 2010. National Cooperative Highway Research Program (NCHRP) Report 653. Effects of Debris on Bridge Pier Scour. Transportation Research Board, Washington, D.C.

Richardson, E.V. and S.R. Davis. 2001. Evaluating Scour at Bridges, 4th Edition. U.S. Department of Transportation, Federal Highway Administration, Hydraulic Engineering Circular 18, Publication FHWA NHI 01-001, 378 p.

Table 1. Results of Pier Scour Analysis at the 8th Street bridge, Countyline Levee Setback Project

Variable	Units	HEC-18 [CSU Equation (Richardson et al. 1975) with K4 correction]								Description
		S4a (No-action, year 0)		S1a (with project, year 0)		S4c (No-action, 15 years)		S1c (with project, 15 years)		
		Pier No. 2[1]	Pier No. 3	Pier No. 2	Pier No. 3	Pier No. 2	Pier No. 3	Pier No. 2	Pier No. 3	
y_1	ft	9.62	7.30	11.50	7.86	4.89	4.98	10.55	6.33	Flow depth upstream of pier
V_1	ft/s	6.30	6.53	9.43	11.04	4.31	4.42	10.41	12.20	Mean flow velocity upstream of pier
a	ft	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	Pier width
L	ft	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	Pier length
θ	degrees	15	15	15	15	15	15	15	15	Angle of attack
g	ft/s^2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	Acceleration of gravity
D_{50}	mm	16	16	16	16	16	16	16	16	Median grain size
D_{95}	mm	100	100	100	100	88	88	100	100	95th percentile grain size
K_1	dimensionless	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	Correction factor for pier nose shape
K_3	dimensionless	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	Correction factor for bed condition
K_2	dimensionless	2.74	2.74	2.74	2.74	2.74	2.74	2.74	2.74	Correction factor for angle of attack
Fr	dimensionless	0.36	0.43	0.49	0.69	0.34	0.35	0.56	0.85	Froude Number
V_{cD50}	ft/s	6.10	5.83	6.28	5.90	5.45	5.47	6.19	5.69	Critical velocity for incipient motion for D50
V_{cD95}	ft/s	8.65	8.65	8.65	8.65	8.29	8.29	8.65	8.65	Critical velocity for incipient motion for D95
V_{icD50}	ft/s	3.24	3.10	3.34	3.14	2.90	5.47	3.29	3.02	Approach velocity required to initiate scour
V_{icD95}	ft/s	5.07	5.07	5.07	5.07	4.82	4.82	5.07	5.07	Approach velocity required to initiate scour
V_R	dimensionless	2.96	4.52	5.01	9.54	2.25	2.36	6.32	14.80	Velocity-intensity ratio
K_4	dimensionless	0.47	0.50	0.51	0.56	0.45	0.45	0.53	0.60	Correction factor for "armoring"
y_s	**ft**	**6.33**	**6.59**	**8.34**	**9.34**	**4.71**	**4.80**	**8.91**	**10.11**	**Scour depth**
y_s'	ft	55.9	57.9	52.5	55.1	62.0	61.9	53.7	56.64	Elevation at max. scour depth (NAVD 88)[2, 3]
delta	ft	10.4	12.4	7.0	9.6	16.5	16.4	8.2	11.1	Height of scour hole above pile cap[3]
x_s	ft	14.7	15.2	18.7	20.7	11.4	11.6	19.8	22.2	lateral and upstream extents (2:1 slopes)[4, 5]
z_s	ft	54.3	55.4	62.4	66.4	47.8	48.2	64.6	69.4	downstream extent (4:1 slopes)[4, 5]

Notes:

Calculations based on HEC-18, CSU Equation (Richardson et al. 1975) with K4 correction

1. Pier numbering is from west to east.
2. Calculated from bed elevation minus scour depth.
3. Pile cap is at elevation 45.5 ft (NAVD 88) per 1952 construction drawings.
4. Based on Lagasse et al. 2010. NCHRP Report 653, Effects of Debris on Bridge Pier Scour.
5. lateral and downstream scour include bridge pier dimensions.