St. John's Church Project Final EIR
Appendix B – Scour Analysis Subject

St. John's Church Scour Analysis at Temescal Creek, dated May 1, 2012, hereby supersedes the version of the same analysis dated March 30, 2012, and included as Appendix B of the St. John's Church Project Final EIR. The March 30, 2012 version was included in the Final EIR in error.

The May 1, 2012 version includes text that provides clarification regarding the timeframe in which the analysis was evaluated. The text clarifications are limited to pages 1 and 4, and do not affect the model results or determinations included in the analysis.

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APPENDIX B

SCOUR ANALYSIS

Kamman Hydrology
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Memorandum

Date: May 1, 2012

To: Kyle Simpson, DCE

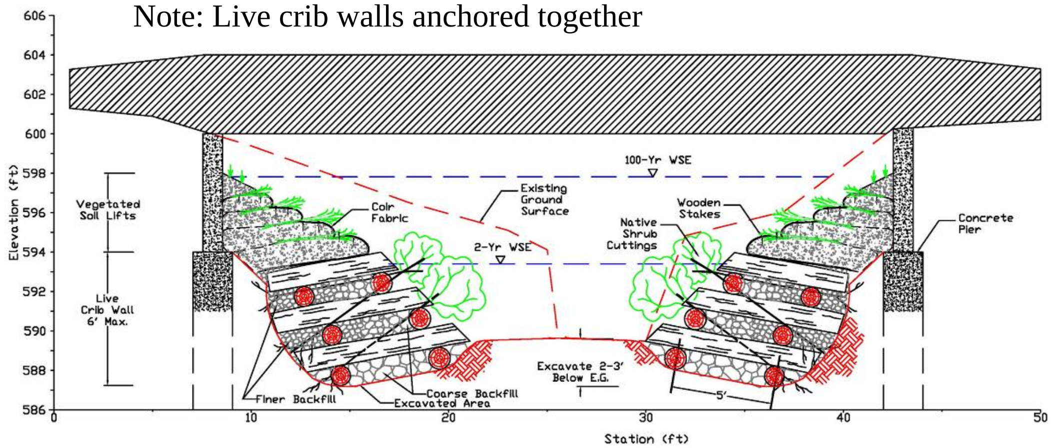
From: Stephanie Lapine, PE and Rachel Kamman, PE

Subject: St. John's Church Scour Analysis at Temescal Creek

Purpose:
This document summarizes the procedures and results of KHE's hydraulic analysis of scour risks associated with the proposed bridge construction and bank modifications at the St. John's project site on Temescal Creek. The bridge deck is sited above the 100 year floodplain (100 yr WSE), and the deck and associated footings do not impinge on creek hydraulics over the foreseeable range of design flows. (See Figure 1). Our study examined Q2 and Q100 design flows conservatively associated with future "full watershed build-out" conditions. (KHE, May 2010).

The proposed channel modifications associated with the bridge design increase the channel cross sectional width under both low flow and high flow conditions. However, the earthen banks under the bridge will be replaced with a bioengineered design encompassing live (vegetated) crib wall and vegetated soil lifts. The scour analysis is undertaken to determine the necessary depth of footing for the bridge to preclude local scour. In addition, KHE examined the impacts of the proposed channel modification on the predicted channel velocities to determine if the project poses an increase in potential bed mobilization risk. A numerical model is used to predict the changes in flow velocities at and in the vicinity of the bridge associated with proposed channel modifications. An increase in velocity would indicate an increase in potential scour risk relative to existing conditions. KHE's analysis examines changes flow velocity both upstream and downstream of the proposed bridge structure.

Figure 1: Cross Section at Proposed Bridge Location (Not to scale)
Note: Live crib walls anchored together



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Summary and Conclusions:

The scour analysis of the proposed bridge at St. John's Church evaluated the potential for regional scour, local scour due to change in channel cross section, and abutment scour associated with flow around the proposed crib wall structure. The following conclusions are drawn from the scour analysis:

- Regional Bed Scour is not a significant risk in the reach because the bed elevation is constrained by culverts at the upstream and downstream limits of the reach (See Figure 2). Between these controls, the channel appears at stable grade and composed of a mixture of medium to fine gravel and medium to large concrete block cobble. The channel modifications proposed with bridge construction will create a 35% increase in flow area, reducing local flow velocities. The proposed project will reduce overall scour risks in the reach.

- Contraction Scour associated with the local change in channel cross section was evaluated at the proposed bridge site and immediately downstream for existing (EC) and proposed conditions (PC) at 2-yr and 100-yr flow rates. *To provide a conservative estimate of scour risk, the analysis assumed the bed was composed solely of gravel observed in the reach; concrete cobble block was ignored.* The results of this analysis indicated that 0.9 to 1.4 feet of scour could be anticipated in the existing conditions (EC) reach in the absence of the concrete cobble. Under proposed conditions (PC), which encompasses a larger cross section, the predicted equilibrium scour depths were reduced to 0.15 ft and 0 ft respectively. Downstream of the proposed bridge site, 0.49 to 0.81 feet of scour is predicted in the absence of the concrete cobble under both existing (EC) and proposed conditions (PC). The presence of the proposed bridge will not exacerbate or reduce scour potential downstream of the site.

- The prediction of minimal scour (in the absence of course cobble) suggests that 1) the proposed geometry approximates an equilibrium cross section for the anticipated range of flows; and 2) the proposed design could be successfully implemented without local replacement of the concrete cobble.

- Abutment Scour created by localized deflection of flows adjacent to the bridge foundation will be precluded by construction the live crib wall and soil bank. However, scour pressure on the bank structure is likely on the upstream left bank due to the oblique angle of upstream culvert discharge. KHE estimated scour depths of between 1.1 and 1.5 feet at this location. The proposed design specifies construction of the crib wall start 2 - 3 ft below existing grade, and as such is expected to provide adequate protection against local scour.

- Total Scour the sum of the regional, contraction and abutment scour estimates, is presented in Table 1 and reflects the likely maximum scour depth for the bridge. The total scour at 2-yr and 100-yr flows is estimated as 1.26 ft to 1.46 feet respectively. These estimates did

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not consider the presence of concrete rubble in the bed. The proposed design places footing of the live crib wall 2 - 3 feet below the existing grade, and therefore is expected to withstand anticipated scour.

Figure 2: Site Plan with Model Geometry and Sediment Sampling Locations

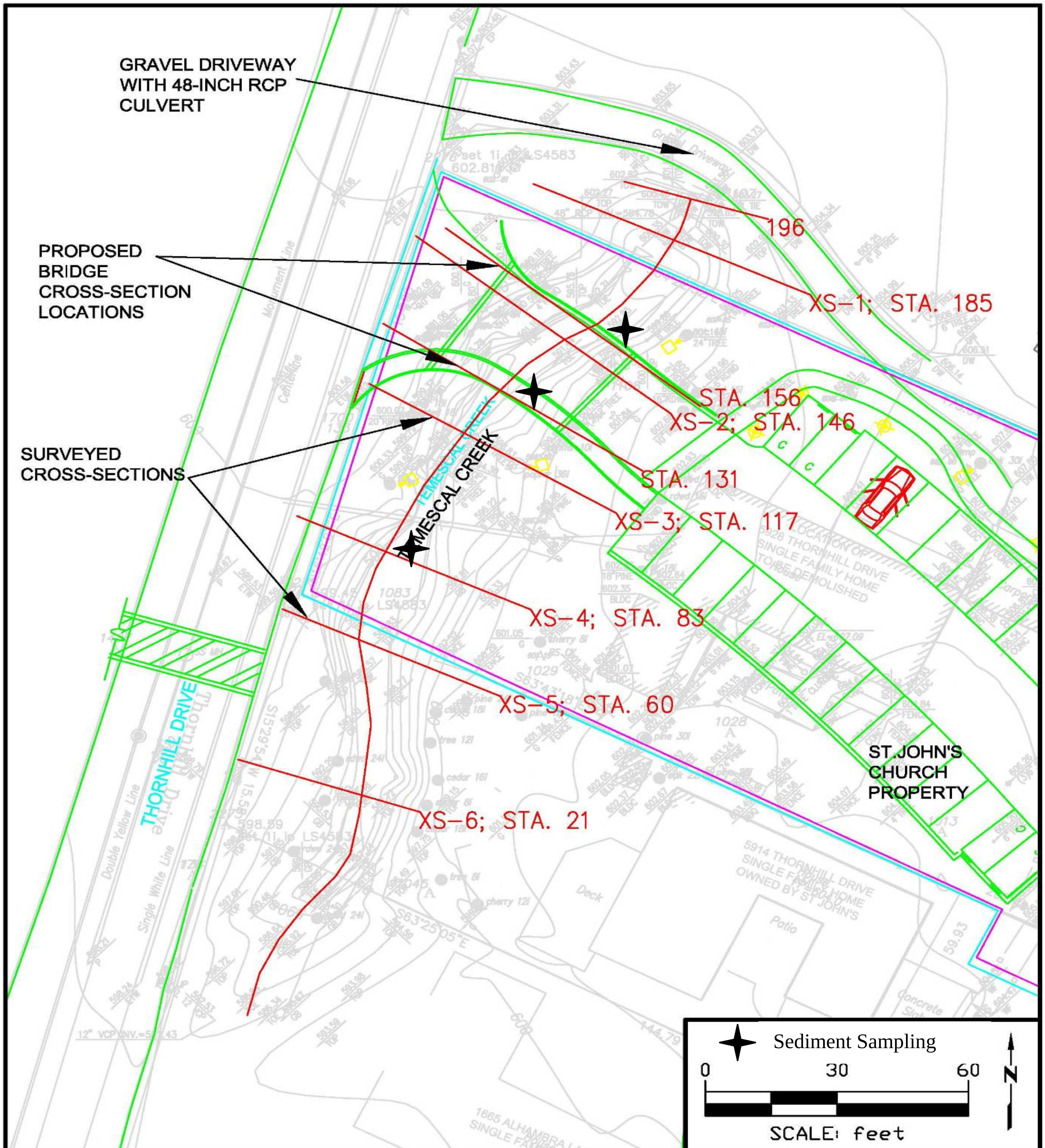

Table 1: Predicted Scour Depths below Channel Cross Section at Proposed Bridge Location

Return Period	Scenario	Type of Scour	Contraction Scour Depth Ys, (ft)	Abutment Scour Depth Ys, (ft)	Total Potential Scour Depth Ys, (ft)
2-Yr	EC	Clear Water	0.90	n/a	0.90
100-Yr	EC	Live Bed	1.05	n/a	1.05
100-Yr	EC	Clear Water	1.38	n/a	1.38
2-Yr	PC	Clear Water	0.15	1.11	1.26
100-Yr	PC	Live Bed	0	1.46	1.46

Hydraulic Analysis Approach:
KHE's bridge scour analysis utilized the HEC-RAS model results and channel configurations prepared during prior analysis and engineering design. The site plan and model geometry are presented in Figure 2 (previously cited as Figure 4 in KHE's May 2010 report).

Design flows for 2-yr and 100-yr storm events were defined as 161 cfs and 569 cfs respectively. These flow rates, determined in KHE's Hydrology Report (May 2010), conservatively reflect design flows associated with future "full watershed build-out" conditions. "Full build-out" hydrology assumes conversion of 96 acres of currently undeveloped upstream parcels to residential development, as determined from the City of Oakland's Zoning and Parcel Maps, and the Alameda County Assessor's Use Codes. Flows do not reflect a future time horizon, but considered future conditions to be those in which all currently zoned development to has taken place. From a runoff impact stand point, this represents the most conservative assumption about future conditions. The proposed conditions scenario includes modifications to the HEC-RAS model cross sections associated with project implementation. Flows through the reach are conserved and there are no other known water sources or sinks between the channel cross section upstream (XS 185) and the project cross section (XS 156).

The area downstream of the culvert and upstream of the proposed bridge site comprises XS 196 through XS 156 and is considered the "upstream" area. The "project" area lies between XS 156 and XS 131 and encompasses the proposed bridge site. The "downstream" area lies below cross-section Sta. 131.

For the scour analysis summarized below, KHE followed procedures described in the Federal Highway Administration's (FHWA) 2001 Hydraulic Engineering Circular No.18 (HEC-18) *Evaluating Scour at Bridges* (Fourth Edition). In order to frame the analysis in terms of the change in scour potential, KHE evaluated the difference in scour potential between Existing Conditions (EC) scenarios and Proposed Conditions (PC) scenarios. The HEC-18 model code requires specification of a structure in order to run the scour algorithm. Therefore a "fake" bridge deck located above and out of the channel was added to the existing conditions (EC) scenario to enable computation of scour depths at the bridge location under existing conditions. To evaluate scour depth and impacts downstream of the proposed bridge location, a "fake" bridge deck was located above and out of the channel for both proposed conditions (PC) and existing conditions (EC) downstream of the proposed bridge site (XS 117).

Scour Analysis

The FHWA defines total potential scour for a reach as the composite of long term channel elevation change (aggradation or degradation), general scour which is frequently driven by a change in cross section (typically a contraction), and local scour which occurs adjacent to piers or abutments in contact with the flow field.

Regional Scour:

In this reach, regional channel incision is constrained by the invert elevations of culverts located both upstream and downstream of the project site. The upstream and downstream culverts locations are shown on Figure 2, and are approximately 30 ft above and 300 ft below the proposed bridge location respectively. Between the culverts, bed scour can be induced locally, if flows are sufficient to mobilize the bed material. However, regional bed scour below the elevation of the downstream culvert invert is not likely. Site inspection and Figure 2 contours show the creek to be at a stable grade with both upstream and downstream culvert inverts.

To address concerns regarding potential scour risks around the proposed structure, KHE applied FHWA methodologies to evaluate the potential scour risks associated with: 1) the change in channel cross section (Contraction Scour), and 2) the scour adjacent to the live crib wall installation (Abutment Scour).

Contraction Scour:

Contraction scour occurs when the flow area of a stream is altered. From the continuity equation, a change in flow area creates an inverse change in flow velocity which directly affects the bed shear stress through the changed section. Typically, we would use this analysis to address a contraction, which decreases channel cross section and increases local flow velocity and bed shear stress. Bed shear stress is a measure of erosive force and in turn the potential for scour at the site of channel geometry change. As scour increases, the flow area increases until an equilibrium condition is reached which balances flow area with erosive shear forces. The equilibrium is a function of flow area, velocity through the reach and sediment size. The HEC-18 code determines this equilibrium condition for a single steady flow rate.

At the St John's project site, the channel cross section is increased with bridge construction. Our analysis compares predicted equilibrium bed elevation (expressed as a change in channel depth) driven by the change in cross section from the upstream reach (HEC XS 185) to the bridge cross section HEC XS-156) under existing (EC) and proposed conditions (PC). A parallel analysis compares the predicted equilibrium bed elevation downstream at XS 117 under existing (EC) and proposed conditions (PC). The analysis described below was conducted using the existing HEC-RAS model and FHWA's HEC-18 scour assessment model.

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Prior to computing the equilibrium scour depth for the bridge section, the model determines if the reach conditions will support Clear Water or Live Bed scour. These conditions correspond to conditions assuming that bed is immobile or mobilized respectively. Calculations are made per HEC-18 Equation 5.1 to determine critical velocity (V_c) as a function of grain size and flow depth as follows:

$$V_c = K_u y^{1/6} D^{1/3} \tag{5.1}$$

where:

V_c	=	Critical velocity above which bed material of size D and smaller will be transported, m/s (ft/s)
y	=	Average depth of flow upstream of the bridge, m (ft)
D	=	Particle size for V_c, m (ft)
D_{50}	=	Particle size in a mixture of which 50 percent are smaller, m (ft)
K_u	=	6.19 SI units
K_u	=	11.17 English units

Application of Equation 5.1 (above) requires that KHE define the D_{50} particle size. KHE characterized bed substrate composition and representative median grain size (D_{50}) based on site inspection and analysis of grain size distribution using pebble counts. KHE staff conducted pebble counts at three locations in the reach identified on Figure 2. Samples were collected utilizing a standard geomorphic pebble count method as described by Wolman[1]. A grain size frequency distribution is defined for each sampling point to describe the sediment size characteristics at a given location (Figure 4). This assessment yielded a D_{50} is 0.027 feet (8.17 mm). **The D_{50} does not include large concrete chunks, which were intentionally excluded from the pebble count to generate a conservative assessment of bed mobility and potential scour depth in the absence of the concrete rubble presently armoring the bed.**

Concrete rubble is found throughout the reach, and appears to provide significant armoring of the bed. KHE determined that typical rubble sizes ranged from 8 to 16 inches, and conservatively estimated that 25% of the bed surface could be considered rubble. (See Figure 3).

[1] Wolman, M.G., 1954. A Method for Sampling Coarse River-Bed Material. Transactions of the American Geophysical Union, volume 35, number 6.

Figure 3: Temescal Creek Channel Looking Upstream Toward Culvert from Bridge Site

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Figure 4: Grain Size Distributions in Temescal Creek near St John's Church

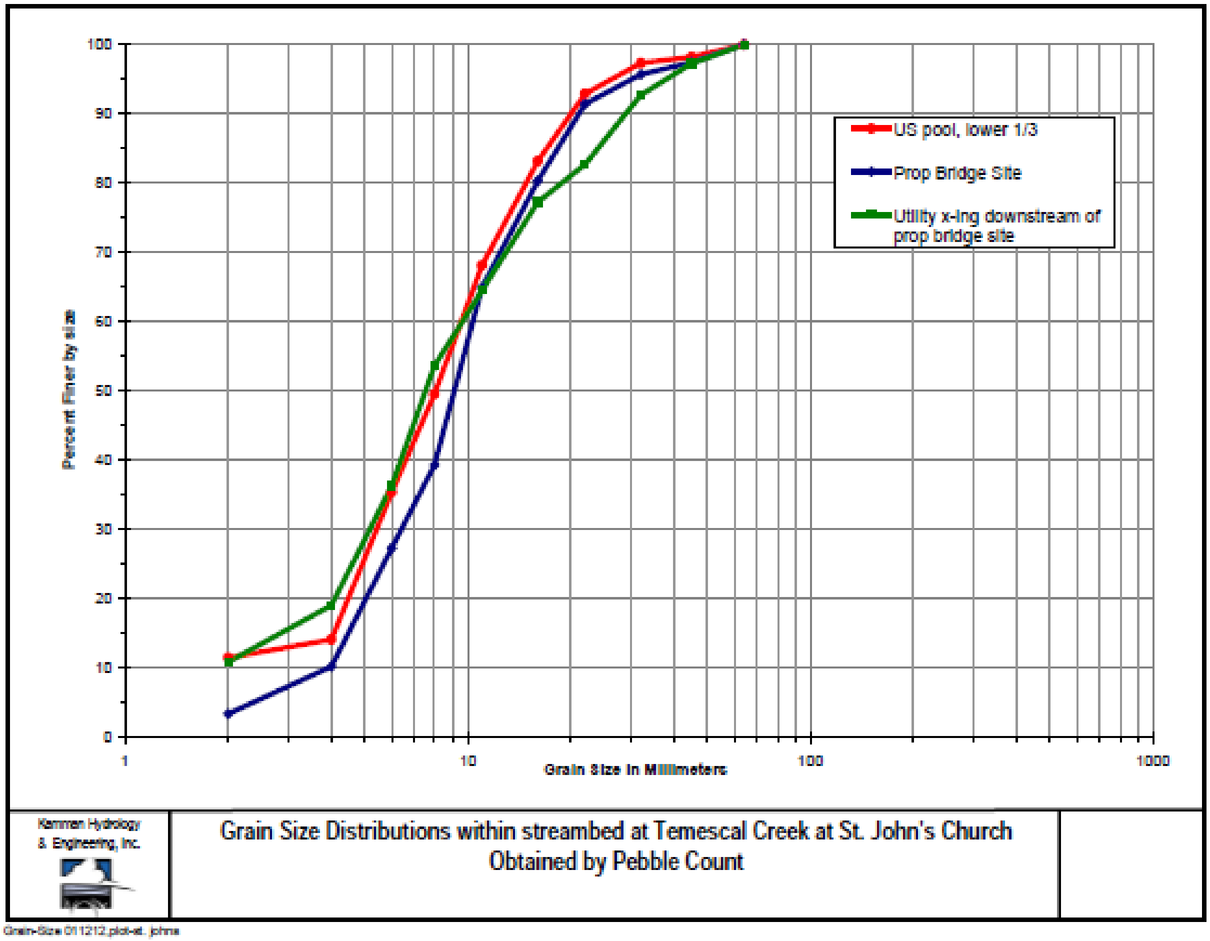

Table 2 summarizes simulation results using the HEC-RAS model to predict channel velocities, and the HEC-18 model to predict critical velocities for bed transport. More detailed simulation output tables are provided in Attachment A.

Table 2: Predicted Flow Velocity and Scour Velocity Thresholds above St John's Bridge

Return Period	Scenario	Critical Vel. (Vc, Ft/sec)	Reach Vel. (V, ft,sec)	Selection Criterion	Type of Scour*
2-Yr	EC	4.05	3.11	Vc > V	Clear Water
2-Yr	PC	4.02	3.28	Vc > V	Clear Water
100-Yr	EC	4.47	4.46	Vc= V	Clear Water or Live Bed
100-Yr	PC	4.46	4.51	Vc < V	Live Bed

* Definitions for clear water and Live Bed scour.

Table 2 indicates that the 100-yr flows require a live bed scour calculation, and 2-year flows require a clear water solution. The Live Bed and Clear Water scour equations used in HEC-18 are summarized as:

FHWA's Live-Bed Contraction Scour Equation

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \quad (5.2)$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)} \quad (5.3)$$

where:

y_1 = Average depth in the upstream main channel, m (ft)
y_2 = Average depth in the contracted section, m (ft)
y_o = Existing depth in the contracted section before scour, m (ft) (see Note 7)
Q_1 = Flow in the upstream channel transporting sediment, m^3/s (ft^3/s)
Q_2 = Flow in the contracted channel, m^3/s (ft^3/s)

W_1 = Bottom width of the upstream main channel that is transporting bed material, m (ft)
W_2 = Bottom width of the main channel in the contracted section less pier width(s), m (ft)
k_1 = Exponent determined below

FHWA's Clear-Water Contraction Scour Equation

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2}\right]^{3/7} \quad (5.4)$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)} \quad (5.5)$$

where:

y_2 = Average equilibrium depth in the contracted section after contraction scour, m (ft)
Q = Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W, m^3/s (ft^3/s)
D_m = Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section, m (ft)
D_{50} = Median diameter of bed material, m (ft)
W = Bottom width of the contracted section less pier widths, m (ft)
y_o = Average existing depth in the contracted section, m (ft)
K_u = 0.025 SI units
K_u = 0.0077 English units

The predicted contraction scour under existing (EC) and proposed (PC) conditions at XS 156 at the proposed bridge site and at XS 117 downstream of the proposed bridge site are summarized in Tables 3 and 4 and presented graphically in Figures 5 and 6. The EC 100-year flow scenario for the proposed bridge site was evaluated for both Clear Water and Live Bed Scour scenarios because the approach velocity was determined to be equal to the defined velocity threshold.

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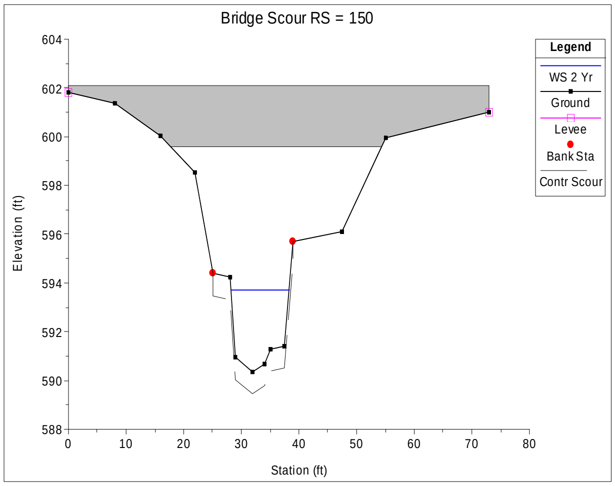
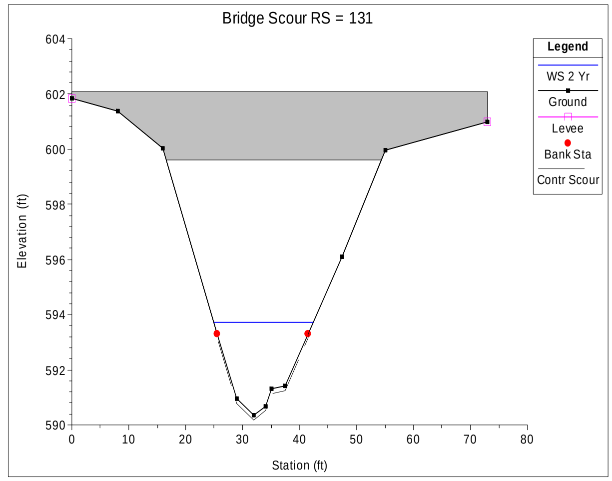
Figure 5: HEC-18 Contraction Scour Results at the Proposed Bridge Location

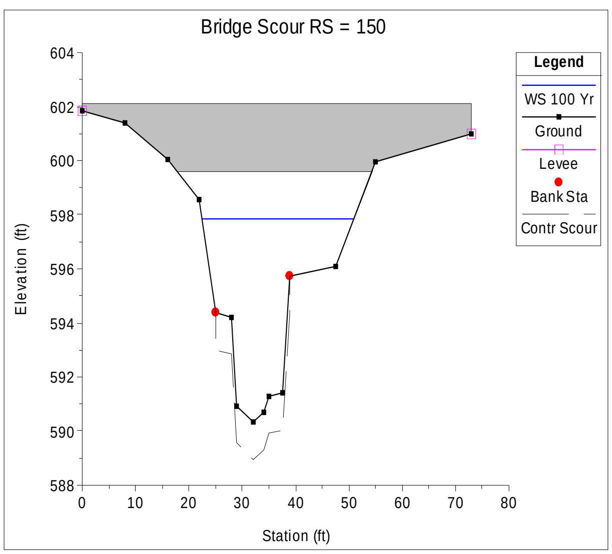
<u>Existing Conditions</u>: Q2 Simulation
Clear Water Analysis



<u>Proposed Conditions</u>: Q2 Simulation
Clear Water Analysis



<u>Existing Conditions</u>: Q100 Simulation
Clear Water Analysis



<u>Proposed Conditions</u>: Q100 Simulation
Live Bed Analysis

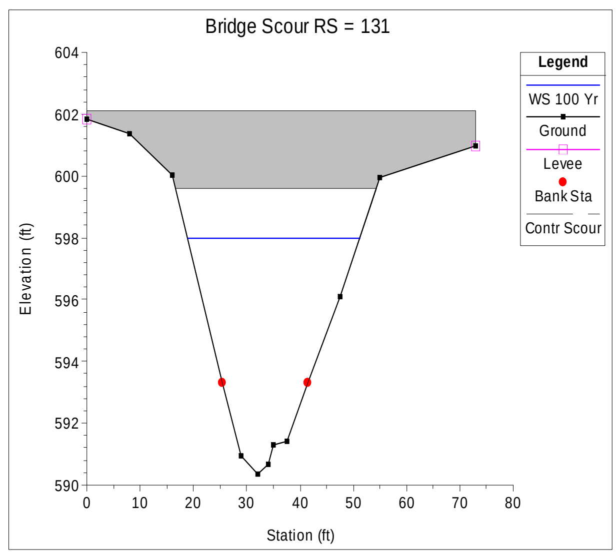


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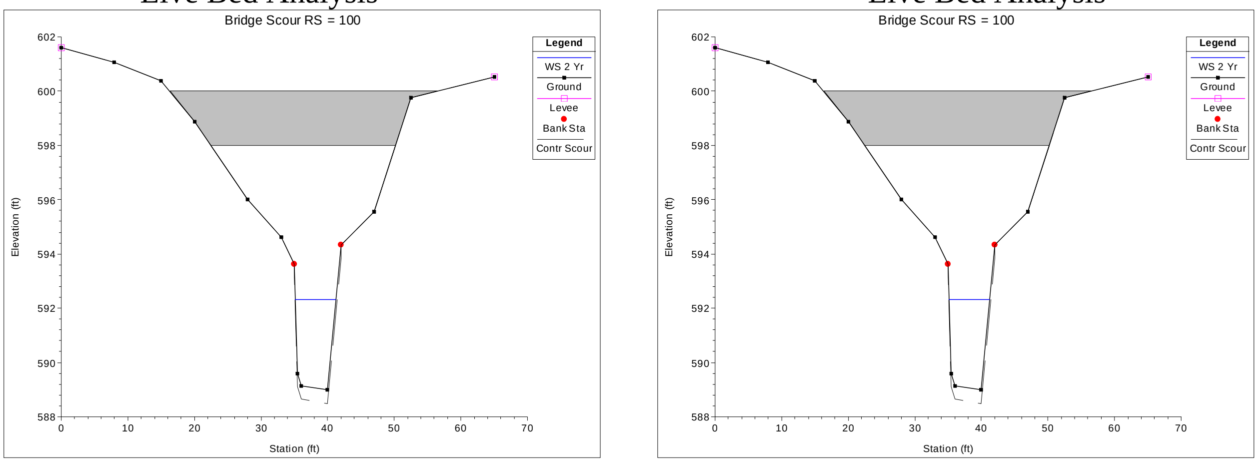
Figure 6: HEC-18 Contraction Scour Results Downstream of the Proposed Bridge Location

<u>Existing Conditions</u>: Q2 Simulation
Live Bed Analysis

<u>Proposed Conditions</u>: Q2 Simulation
Live Bed Analysis



Existing Conditions: Q100 Simulation
Live Bed Analysis

Proposed Conditions: Q100 Simulation
Live Bed Analysis

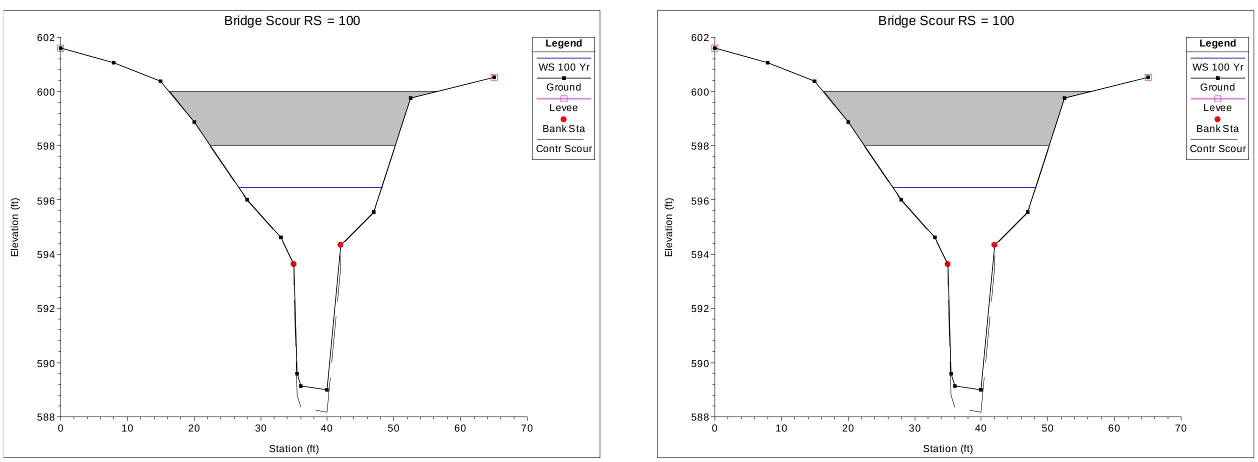


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Table 3: Predicted Scour Depth below Channel Cross Section at Proposed Bridge Location

Return Period	Scenario	Scour Depth Ys, (ft)	Type of Scour*
2-Yr	EC	0.90	Clear Water
100-Yr	EC	1.05	Live Bed
100-Yr	EC	1.38	Clear Water
2-Yr	PC	0.15	Clear Water
100-Yr	PC	0	Live Bed

* Definitions of Clear and Live Bed Scour

Table 4: Predicted Scour Depth below Channel Cross Section Downstream of Proposed Bridge Location

Return Period	Scenario	Scour Depth Ys, (ft)	Type of Scour*
2-Yr	EC	0.49	Live Bed
100-Yr	EC	0.81	Live Bed
2-Yr	PC	0.49	Live Bed
100-Yr	PC	0.81	Live Bed

* Definitions of Clear and Live Bed Scour

The analysis indicates that at the proposed bridge location under existing conditions (EC), equilibrium bed elevations are 0.9 to 1.38 ft lower than the existing bed elevation for the 2-yr and 100-yr flow scenarios respectively. We hypothesize that the higher-than-predicted elevation of the existing bed is due to the concrete rubble which is present in the bed but was not considered in the analysis.[2] We hypothesize that under current conditions, the concrete plays an active role in preventing bed scour, and that additional bed incision would likely result if the rubble were to be removed.

Under proposed conditions (PC) equilibrium bed elevations are predicted to be 0.15 ft and 0.0 ft lower than the proposed design grade. This indicates the proposed design cross section would be relatively stable under expected flow conditions, even if no concrete rubble were present in the bed material. The proposed design provides a 35% wider flow area under both 2-yr and 100-yr flow conditions, which is largely responsible for reducing channel velocities and in turn, local scour risks. The design as currently proposed would key the live crib wall into the bed 2 – 3 ft below existing grade. As such, the design can be considered robust in the context of both existing and proposed channel cross sections.

[2] This excess stream power is likely responsible for the localized bank erosion observed in the reach. Bank erosion occurs to dissipate excess stream energy in the context of a bed which is armored or subject to grade control.

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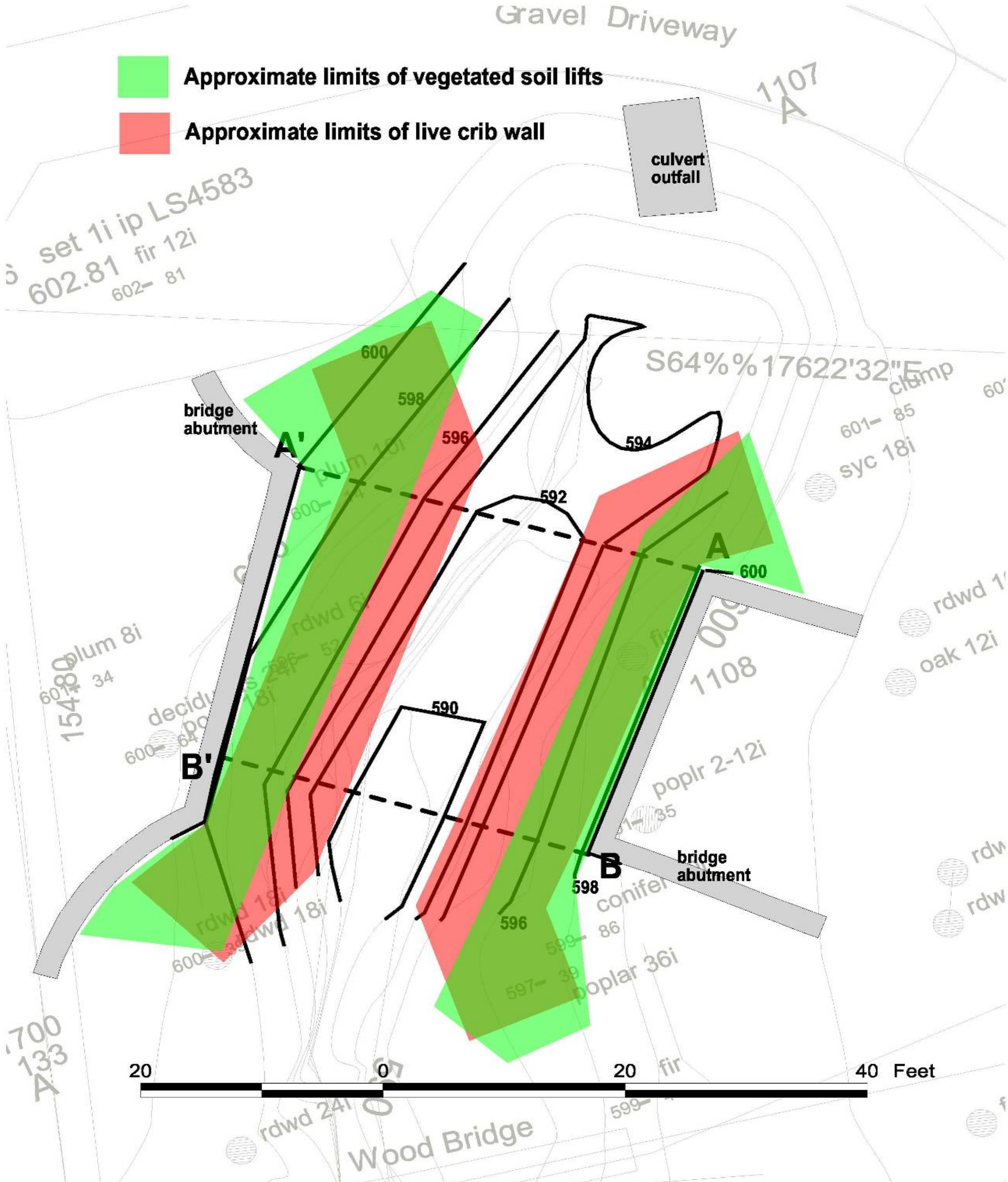
Downstream of the proposed bridge location, under both existing (EC) and proposed conditions (PC), equilibrium bed elevations are 0.49 to 0.81 ft lower than the existing bed elevation for the 2-yr and 100-yr flow scenarios respectively. Again, we hypothesize that the higher-than-predicted elevation of the existing bed is due to the concrete rubble which is present in the bed but was not considered in the analysis. The presence (PC) or absence (EC) of an upstream bridge does not influence the scour potential of unmodified area downstream of the bridge.

Abutment Scour:

Abutment scour occurs when the bridge abutments block approaching flow and are subject to the erosive forces at the contact between the structure and the flow field. A plan view of the proposed design (Figure 7) shows the abutments set back 10 feet behind the live crib wall and engineered soil bank. These soft bank features are designed to support vegetation, which once colonized will create a second soft buffer between the flow and the structure. The proposed soft bank wraps abound the bridge abutments and ties smoothly into the contours of the existing bank. The design is consistent with FHWA guidelines which recommend protecting abutments from local scour using riprap and/or guide banks. (FHWA, 2001 pg.7.7).

While the engineered bank protects the bridge structure, the bank itself is subject to scour by the obliquely passing water. The most “at risk” location in the structure is the left upstream bank which is set at approximately a 30 deg. angle from the channel flow line. KHE utilized FHWA’s recommended procedures to evaluate abutment scour at this location in the proposed structure. The Froehlich abutment scour equation is recommended to evaluate both live bed and clear water scour at sites like the St. John’s Bridge where the ratio of flow depth to abutment length is less than 25.

Figure 7: Plan View of Bridge Abutments, Crib Wall and Soil Lifts



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Froehlich's Abutment Scour Equation (FHWA, 2001)

$$\frac{y_s}{y_a} = 2.27\,K_1\,K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1 \tag{7.1}$$

where:

K_1	=	Coefficient for abutment shape (Table 7.1)
K_2	=	Coefficient for angle of embankment to flow
K_2	=	$(\theta/90)^{0.13}$ (see Figure 7.4 for definition of θ) $\theta<90^\circ$ if embankment points downstream $\theta>90^\circ$ if embankment points upstream
L'	=	Length of active flow obstructed by the embankment, m (ft)
A_e	=	Flow area of the approach cross section obstructed by the embankment, m^2 (ft^2)
Fr	=	Froude Number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$
V_e	=	Q_e/A_e, m/s (ft/s)
Q_e	=	Flow obstructed by the abutment and approach embankment, m^3/s (ft^3/s)
y_a	=	Average depth of flow on the floodplain (A_e/L), m (ft)
L	=	Length of embankment projected normal to the flow, m (ft)
y_s	=	Scour depth, m (ft)

Applying this algorithm to the St John's site yields estimates of abutment scour for the upstream left bank of 1.46 ft and 1.11 ft for proposed conditions at 100-yr and 2-yr flows respectively. Estimation parameters are presented in Attachment 3. These results indicate that the proposed design is sufficient to withstand anticipated abutment scour because the Live Crib Wall is keyed into the bed to a depth of 2 -3 ft below the existing grade. As such, the design dimensions are sufficient to preclude abutment scour at the location most vulnerable to attack.

References:

HEC-RAS River Analysis System Hydraulic Reference Manual. 2008. US Army Corps of Engineers Hydraulic Engineering Center (HEC). Report No. CPD-69. March.

Federal Highway Administration's (FHWA) 2001. Hydraulic Engineering Circular No.18 (HEC-18) *Evaluating Scour at Bridges* (Fourth Edition).

Kamman Hydrology & Engineering, Inc. 2010. Hydrology Report. Prepared for St John's Episcopal Church, APN 48F-7390-4-9. May.

Attachment 1: HEC-RAS Modeling Summary Tables

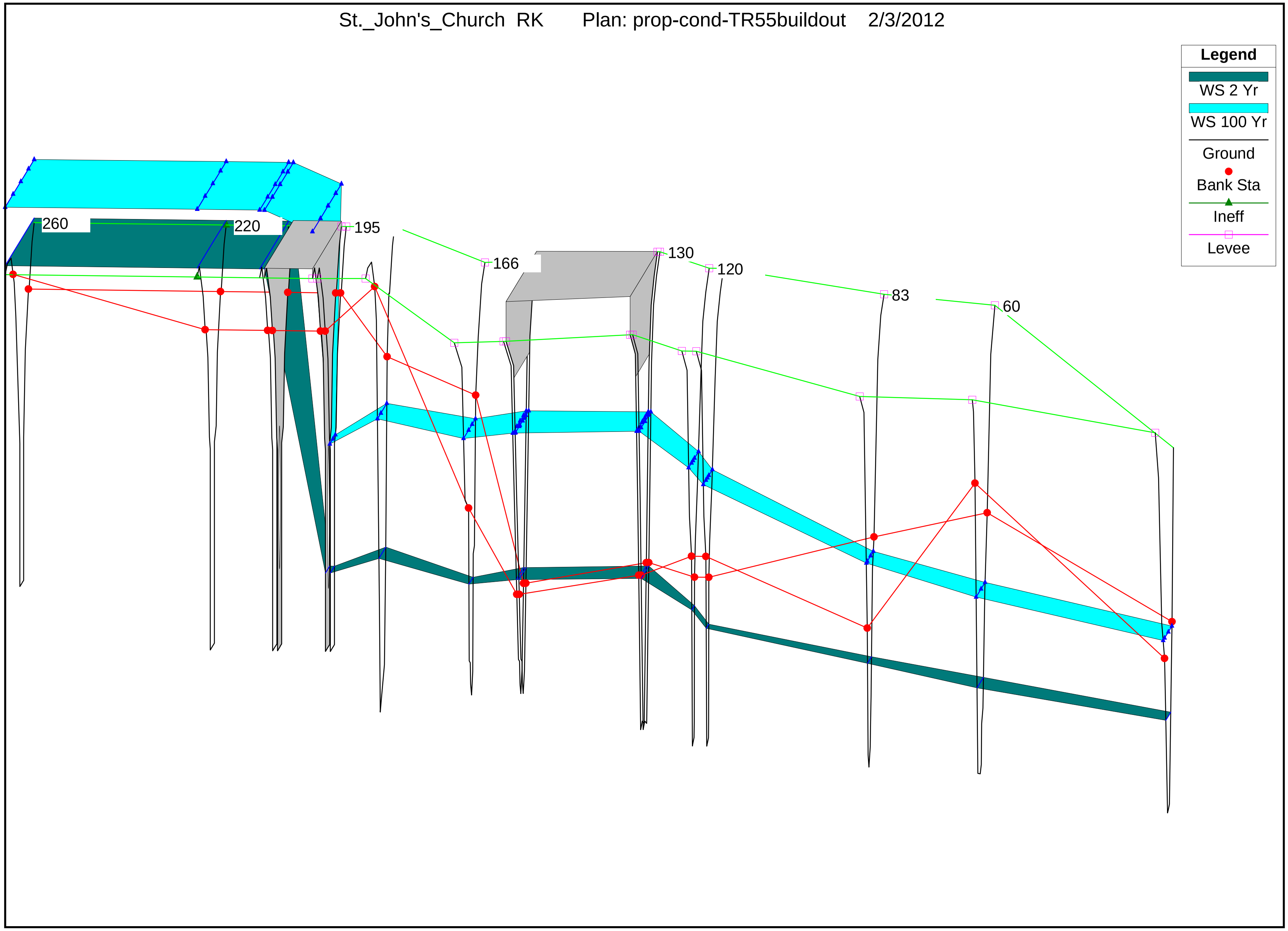
Existing Conditions Simulation

River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Hydr Depth C	W.P. Channel	W.P. Total
		(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)	(ft)
185	2 Yr	161	589.83	594.36	592.33	594.51	0.002539	3.1	51.85	16.58	0.31	3.13	20.34	20.34
185	5 Yr	264	589.83	595.78	593.03	595.97	0.002272	3.43	76.99	18.77	0.3	4.1	23.94	23.94
185	10 Yr	337	589.83	596.62	593.46	596.83	0.002185	3.61	93.31	20.06	0.3	4.65	26.07	26.07
185	25 Yr	431	589.83	597.42	593.95	597.66	0.002301	3.93	109.69	21.28	0.3	5.16	28.08	28.08
185	50 Yr	511	589.83	597.92	594.31	598.2	0.002502	4.24	120.6	22.05	0.32	5.47	29.35	29.35
185	100 Yr	569	589.83	598.24	594.56	598.55	0.00266	4.46	127.67	22.54	0.33	5.67	30.16	30.16
156	2 Yr	161	590.34	593.75	593.06	594.31	0.008681	6.03	26.7	10.17	0.66	2.62	14.19	14.19
156	5 Yr	264	590.34	595.31	593.89	595.81	0.005646	5.7	46.55	14.53	0.55	3.34	19.34	20.48
156	10 Yr	337	590.34	596.2	594.59	596.7	0.004178	5.67	62.54	24.04	0.49	4.2	19.76	30.77
156	25 Yr	431	590.34	597.04	595.08	597.54	0.003421	5.8	83.68	26.27	0.46	5.04	19.76	33.64
156	50 Yr	511	590.34	597.55	595.47	598.08	0.003299	6.07	97.35	27.62	0.45	5.54	19.76	35.37
156	100 Yr	569	590.34	597.87	595.74	598.43	0.003286	6.28	106.17	28.46	0.46	5.86	19.76	36.44
117	2 Yr	161	588.99	592.66	592.31	593.76	0.020502	8.44	19.08	6.25	0.85	3.05	11.67	11.67
117	5 Yr	264	588.99	593.75	593.48	595.32	0.023959	10.07	26.22	7.04	0.9	3.87	13.81	14.1
117	10 Yr	337	588.99	594.54	594.25	596.26	0.021711	10.56	32.64	9.69	0.87	4.53	14.44	17.37
117	25 Yr	431	588.99	595.66	595.48	597.2	0.015248	10.25	48.06	17.86	0.76	5.64	14.44	25.86
117	50 Yr	511	588.99	596.25	596.15	597.75	0.013586	10.35	59.61	20.59	0.73	6.24	14.44	28.88
117	100 Yr	569	588.99	596.56	596.47	598.1	0.013397	10.61	66.16	21.85	0.73	6.55	14.44	30.3

Proposed Conditions Simulation

River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Hydr Depth C	W.P. Channel	W.P. Total
		(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)	(ft)
185	2 Yr	161	589.83	594.2	592.33	594.36	0.002953	3.28	49.15	16.33	0.33	3.01	19.92	19.92
185	5 Yr	264	589.83	595.64	593.03	595.84	0.002505	3.55	74.31	18.55	0.31	4.01	23.58	23.58
185	10 Yr	337	589.83	596.53	593.46	596.74	0.002312	3.69	91.4	19.91	0.3	4.59	25.83	25.83
185	25 Yr	431	589.83	597.35	593.95	597.59	0.002386	3.98	108.24	21.17	0.31	5.11	27.91	27.91
185	50 Yr	511	589.83	597.85	594.31	598.14	0.002588	4.29	119.11	21.94	0.32	5.43	29.18	29.18
185	100 Yr	569	589.83	598.17	594.56	598.48	0.002748	4.51	126.13	22.43	0.34	5.62	29.98	29.98
156	2 Yr	161	590.34	593.74	592.92	594.05	0.004331	4.51	36.02	17.56	0.53	2.23	17.42	19.21
156	5 Yr	264	590.34	595.35	593.55	595.61	0.001775	4.15	69.04	23.31	0.37	3.85	17.42	25.84
156	10 Yr	337	590.34	596.3	593.93	596.55	0.001306	4.12	92.78	26.65	0.33	4.8	17.42	29.69
156	25 Yr	431	590.34	597.16	594.39	597.43	0.001161	4.34	116.97	29.53	0.32	5.66	17.42	33.06
156	50 Yr	511	590.34	597.68	594.73	597.98	0.001178	4.63	132.61	31.25	0.33	6.17	17.42	35.07
156	100 Yr	569	590.34	598	594.96	598.33	0.001207	4.85	142.81	32.33	0.34	6.49	17.42	36.32
117	2 Yr	161	588.99	592.66	592.31	593.76	0.020502	8.44	19.08	6.25	0.85	3.05	11.67	11.67
117	5 Yr	264	588.99	593.75	593.48	595.32	0.023959	10.07	26.22	7.04	0.9	3.87	13.81	14.1
117	10 Yr	337	588.99	594.54	594.25	596.26	0.021711	10.56	32.64	9.69	0.87	4.53	14.44	17.37
117	25 Yr	431	588.99	595.66	595.48	597.2	0.015248	10.25	48.06	17.86	0.76	5.64	14.44	25.86
117	50 Yr	511	588.99	596.25	596.15	597.75	0.013586	10.35	59.61	20.59	0.73	6.24	14.44	28.88
117	100 Yr	569	588.99	596.56	596.47	598.1	0.013397	10.61	66.16	21.85	0.73	6.55	14.44	30.3

Attachment 2: HEC-RAS Water Surface Profiles



Abutment Scour at the Upstream Left Bank of the St John's Church Bridge

Parameter	Description	100-yr Flows	2-yr Flows	units
	Units: English or Metric	E	E	
	Min Channel Elevation at XS 156	590.3	590.3	ft
	WSE at XS 156	598.0	593.7	ft
y1	Depth of flow at abutment on the overbank or in the main channel	6.5	2.2	ft
L	Length of embankment projected normal to flow	5	5	ft
	ratio Length to Depth	0.8	2.3	
	If ratio>25, HIRE Eq.; If ratio<25, Froehlich Eq.	Froehlich	Froehlich	
K1	Coefficient for abutment shape (Table 7.1, HEC-18)	0.55	0.55	
K2	Coefficient for angle of embankment to flow	0.866910448	0.86691045	
L'	Length of active flow obstructed by the embankment	5	10	ft
Ae	Flow area of the approach cross section obstructed by the embankment.	16	6	ft^2
Fr	Froude Number of Approach flow	0.34	0.33	
Qe	Flow obstructed by the abutment and embankment (25% of Q)	142.25	40.25	cfs
Ve	Qe/Ae (Ft/s)	8.890625	6.70833333	ft/s
Ya	Average depth of flow on floodplain (Ae/L)	1.6	0.6	ft
L	Length of embankment projected normal to the flow (ft)	10	10	ft
Ys	**Abutment Scour Depth**	**1.4637618**	**1.10714**	

Calculations per FHWA, 2001 Chapter 7.