TECHNICAL MEMORANDUM

DATE:	November 1, 2019
PREPARED FOR:	Rodriguez Transportation Group, Inc. and the Texas Department of Transportation
PREPARED BY:	Eric R. Friedrich, P.E. (TX PE# 64818) Teague Nall and Perkins, Inc. (Firm PE #230)
SUBJECT:	Atlas 14 Rainfall Updates
PROJECT:	Oak Hill Parkway (US 290 / SH 71 Interchange)

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PERMIT PURPOSES

1. Introduction

In November 2018, TNP (formerly H&H Resources) submitted a Hydrology and Hydraulics Study report for the Oak Hill Parkway (OHP) project, the planned reconstruction of the US290/SH71 interchange in southwest Austin. The modeling in that study was based on the Effective Federal Emergency Management Agency (FEMA) models for Williamson Creek. These models were released in 2008 and reflect older rain data for Travis County. In September 2018, the National Oceanic and Atmospheric Administration (NOAA) released a study showing significantly higher rainfall frequency values in parts of Texas. The study, published as “NOAA Atlas 14, Volume 11, Precipitation-Frequency Atlas of the United States, Texas” (Atlas 14), recommended increased values across the state, particularly in larger cities such as Austin and Houston. Implementation of Atlas 14 data has resulted in changes to rainfall rates and amounts that define the full spectrum of storm events, including the 100-year precipitation. This Technical Memorandum (TM) is intended to update the November 2018 report recommendations to accommodate Williamson Creek flows based on more recent Atlas 14 rainfall.

TNP was tasked with incorporating the Atlas 14 rainfall data for Travis County into the hydrologic and hydraulic models for the OHP. The previous OHP report outlined preliminary infrastructure recommendations for the US 290 / SH 71 interchange schematic design in order to mitigate water surface elevation (WSEL) increases in Williamson Creek due to increased impervious area and infrastructure additions.

This TM describes the modeling updates due to Atlas 14 and outlines additional recommendations to the OHP schematic design to mitigate Williamson Creek flow and WSEL increases emanating from the increased rainfall frequency values.

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In addition, some modifications are recommended to the proposed HWY71 and Old Bee Cave (OBC) detention sites to address Atlas 14 and to fully comply with City of Austin (COA) and Texas Commission on Environmental Quality (TCEQ) standards for dam construction.

2. Rainfall Frequency Values – Atlas 14, September 2018

The new rainfall data is available on-line at:
https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=tx

The website requires a specific location for which rainfall data will then be extracted. For the Oak Hill project, the specific location was the centroid of the watershed upstream and including the Oak Hill project area. The centroid is located at latitude (30.244508) and longitude (-97.890536) (decimal degrees). Figure 1 below illustrates the centroid location from which September 2018 Atlas 14 rainfall was extracted.

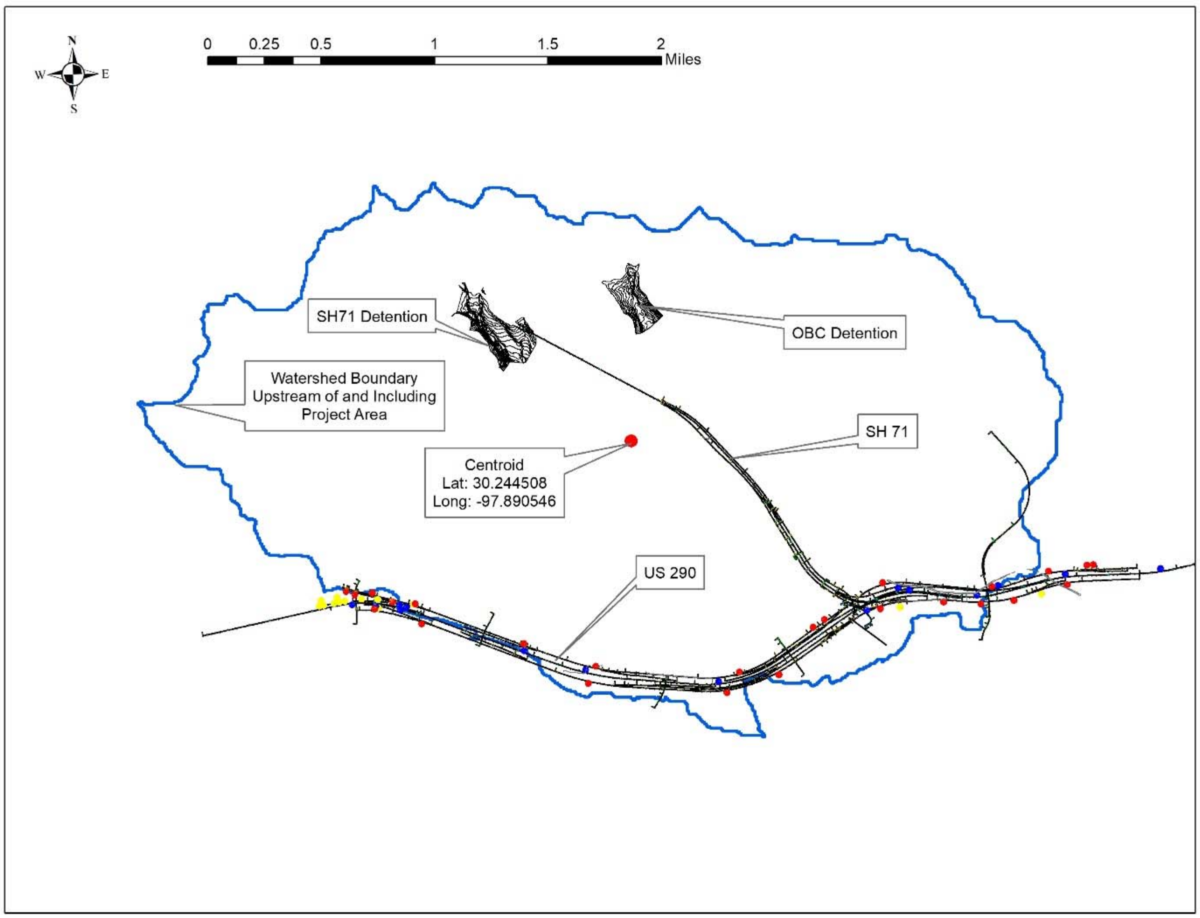

Figure 1 - Centroid Location

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Table 1 compares the previous 10-year, 25-year, 50-year, and 100-year rainfall depth amounts with the Atlas 14 update.

Table 1 - Previous Rainfall and Atlas 14 Rainfall Depth Comparison

	Previous Rainfall	Atlas 14	Previous Rainfall	Atlas 14	Previous Rainfall	Atlas 14	Previous Rainfall	Atlas 14
Duration	**10-Year (inches)**		**25-Year (inches)**		**50-Year (inches)**		**100-Year (inches)**	
5 min.	0.67	0.808	0.81	0.995	0.93	1.14	1.05	1.30
15 min.	1.47	1.62	1.76	1.98	2.01	2.27	2.29	2.59
1 hour	2.68	3.01	3.28	3.71	3.79	4.25	4.37	4.85
2 hours	3.42	3.84	4.20	4.86	4.88	5.71	5.66	6.69
3 hours	3.71	4.35	4.55	5.61	5.28	6.70	6.11	7.97
6 hours	4.21	5.21	5.14	6.83	5.94	8.27	6.85	9.97
12 hours	4.81	6.01	5.90	7.88	6.86	9.55	7.96	11.50
1 day	6.10	6.80	7.64	8.88	8.87	10.70	10.20	12.90

3. Compliance with COA and TCEQ Standards

Two proposed detention facilities, HWY71 and OBC, were reconfigured to comply with several COA and TCEQ standards. The following modifications were made:

1) The emergency spillway inverts were set at the maximum WSEL of the 25-Year event. This is a TCEQ standard that serves to limit excessive operation of the emergency spillway. The TCEQ guidelines state, "Most emergency spillways are built to prevent passage of flows for less than about the 50- or 100-year flood." Most emergency spillways are earthen and therefore unable to successfully maintain structural integrity through continuous use. A moderate amount of damage is expected during extreme events, but the standard assumes these are infrequent, and can be repaired relatively quickly before another extreme event occurs. The 25-year event was used to define the emergency spillway invert.

2) Two feet of freeboard has been configured between the top of dam elevation and the maximum WSEL of the 100-year event. This is a COA standard. TCEQ has freeboard requirements, but for the two proposed detention facilities, TCEQ procedures will probably produce less required freeboard. However, the TCEQ guidelines caveat their standards with the following statement: "Design-flood criteria established by other public agencies, if shown to be more conservative, will generally be acceptable."

3) Additionally, COA requires that all dams safely pass 75 percent of the probable maximum flood (PMF). PMF events have not been routed through the two proposed detention facilities in the current modeling. However, design and construction logistics, and ROW space constraints, will certainly dictate that the downstream slope of each dam

be armored, in the same way the existing Oak Hill and Lantana regional detention facilities are armored.

This creates the possibility, however, for a variance from the first two standards discussed, limiting the use of the emergency spillway and a minimum two feet of freeboard between the top of dam elevation and the 100-year event maximum WSEL. If the downstream slope is armored, then overtopping of the dam is not as much of a concern. The armoring would protect the embankment from erosion, thereby making available discharge conveyance over the dam under a wide range of storm events.

This configuration of proposed regional detention facilities is summarized in the following Table 2 and Table 3. Additional detention scenarios and analysis are discussed in sections 6 and 7.

Table 2 - HWY71 Regional Detention Configuration

Primary Spillway	
Description	3-4'x4' Concrete Box Culvert
Length	500 Feet
Inlet Invert Elevation	915.0 Feet
Outlet Invert Elevation	909.0 Feet
Emergency Spillway	
Description	Broad-Crested Weir
Length	705 Feet
Invert Elevation	933.3 Feet
Top of Dam	
Maximum Crest Elevation	936.0 Feet

Table 3 - OBC Regional Detention Facility

Primary Spillway	
Description	One 24” Reinforced Concrete Pipe
Length	160 Feet
Inlet Invert Elevation	908.0 Feet
Outlet Invert Elevation	906.0 Feet
Emergency Spillway	
Description	Broad-Crested Weir
Length	150 Feet
Invert Elevation	924.9 Feet
Top of Dam	
Maximum Crest Elevation	928.0 Feet

4. Hydrologic Impact – Atlas 14, September 2018

The Atlas 14 rainfall values in Table 1 were incorporated into the revised existing and proposed HEC-HMS models. Table 4 summarizes the discharge results and compares them to previous results. Figure 2-5 graphically illustrate the differences for each probability storm event.

The Atlas PROPOSED condition only includes the changes described previously for the COA and TCEQ dam compliance. All other infrastructure is unchanged from previous work. Infrastructure changes under proposed conditions to mitigate WSEL increases from the Atlas 14 update are described later in this TM.

Table 4 - Discharge Comparison, Previous vs. Atlas 14 Update

HEC RAS XS	HMS Node	Previous Revised Existing				Previous Proposed				Atlas 14 Update Revised Existing				Atlas 14 Update Proposed			
		10-yr	25-yr	50-yr	100-yr	10-yr	25-yr	50-yr	100-yr	10-yr	25-yr	50-yr	100-yr	10-yr	25-yr	50-yr	100-yr
90177	OAKHILL	950	1105	1196	1290	951	1106	1197	1290	1098	1508	1811	2091	1061	1477	1813	2091
89063	JWCR50***	1734	2356	2797	3157	1330	1916	2054	2980	2821	1089	1749	2821	950	1089	1744	2821
84632	JWCR60**	2625	3825	4717	5463	1864	2516	3313	3974	4564	2615	3084	4564	1707	1947	2920	4664
79948	**JWCR80**	**4015**	**5985**	**7519**	**8739**	**3405**	**5027**	**6491**	**7225**	**6229**	**4429**	**5406**	**6229**	**2374**	**3147**	**3686**	**5632**
75171.9	**JWCR1050***	**4835**	**7429**	**9424**	**11159**	**4414**	**6665**	**8791**	**10114**	**5457**	**8930**	**11164**	**13049**	**5457**	**8340**	**10392**	**12121**
75017	**JWCR1050**	**4835**	**7429**	**9424**	**11159**	**4414**	**6665**	**8791**	**10114**	**5457**	**8930**	**11164**	**13049**	**5457**	**8340**	**10392**	**12121**
67082	JWCR1040	4968	7582	9677	11587	4592	6912	9031	10448	13049	8930	11164	13049	5457	8340	10392	12121
59867	JWCR100	5145	7875	10055	11950	4808	7260	9447	10929	13679	9185	11635	13679	5659	8585	10765	12760
55940	JWCR160	10593	15374	19083	22356	10477	15053	18742	21678	14388	9619	12123	14388	5961	9047	11314	13449
50574	JWCR130	11262	16391	20732	24439	11201	16009	20448	23857	27311	18581	23036	27311	12586	18223	22463	26601
49429	JWCR120	11437	16636	21061	24849	11388	16263	20791	24299	30304	20383	25475	30304	13456	20087	25007	29671
46107	JWCR1170	11467	16547	20975	24781	11419	16210	20691	24191	30909	20743	25951	30909	13685	20467	25509	30294
43122	JWCR1000	11843	16984	21556	25521	11814	16674	21290	24964	30977	20736	26009	30977	13710	20427	25543	30351
37465	JWCR103	11901	16992	21460	25435	11863	16689	21171	24844	32429	21548	27181	32429	14235	21259	26742	31883
30000	JWCR360	12031	17072	21532	25542	11984	16772	21239	24922	32648	21555	27247	32648	14317	21245	26789	32117
23527	JWCR880	12141	17172	21704	25736	12083	16870	21404	25121	33283	21820	27706	33283	14532	21511	27246	32752
17814	JWCR370A	12181	17212	21743	25753	12117	16908	21434	25155	33882	22137	28168	33882	14724	21834	27713	33360
13810	JWCR370	12208	17245	21781	25796	12143	16942	21475	25203	34029	22232	28293	34029	14792	21930	27834	33499
7301	JWCR3900	12263	17329	21856	25926	12180	17029	21514	25344	34115	22291	28368	34115	14834	21986	27909	33587
4393	J400W	12284	17342	21819	25709	12194	17042	21468	25229	34480	22475	28634	34480	14938	22146	28169	33954
2454	JWCR400	12419	17511	22030	25956	12331	17216	21681	25480	34528	22464	28344	34528	14974	22126	27880	34000
618	outlet	12408	17494	21991	25831	12317	17198	21641	25388	34951	22736	28675	34951	15170	22403	28217	34436

***Downstream of HWY71 Regional Detention

**Downstream of both HWY71 and OBC Regional Detention

*Upstream of Main Lanes and Frontage Crossing

Bold Text indicates OHP project stream reach

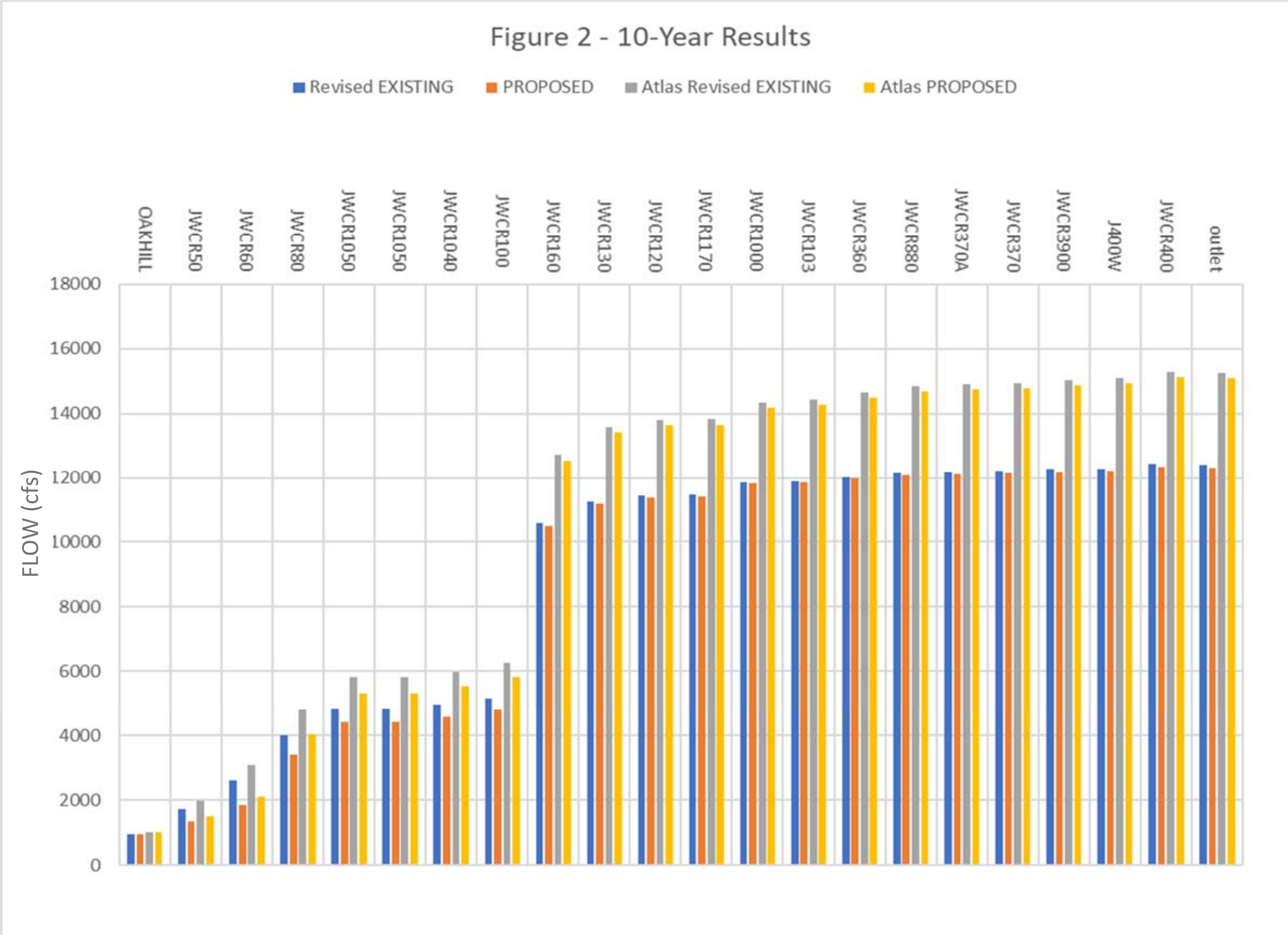
Figure 2 - 10-Year Results
Revised EXISTING
PROPOSED
Atlas Revised EXISTING
Atlas PROPOSED
FLOW (cfs)
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
OAKHILL
JWCR50
JWCR60
JWCR80
JWCR1050
JWCR1050
JWCR1040
JWCR100
JWCR160
JWCR130
JWCR120
JWCR1170
JWCR1000
JWCR103
JWCR360
JWCR880
JWCR370A
JWCR370
JWCR3900
J400W
JWCR400
outlet

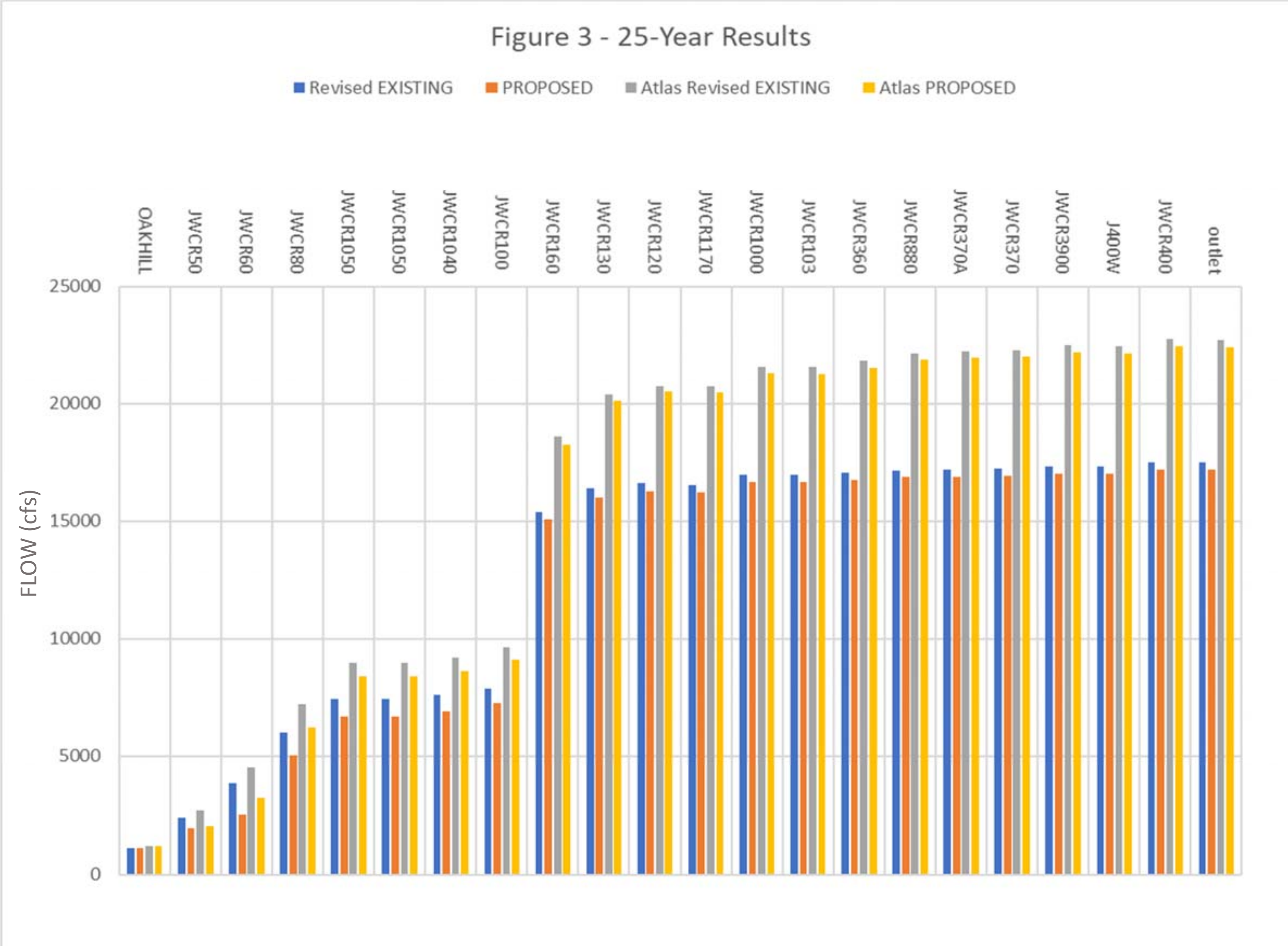
Figure 3 - 25-Year Results
Revised EXISTING
PROPOSED
Atlas Revised EXISTING
Atlas PROPOSED
FLOW (cfs)
0
5000
10000
15000
20000
25000
OAKHILL
JWCR50
JWCR60
JWCR80
JWCR1050
JWCR1050
JWCR1040
JWCR100
JWCR160
JWCR130
JWCR120
JWCR1170
JWCR1000
JWCR103
JWCR360
JWCR880
JWCR370A
JWCR370
JWCR3900
J400W
JWCR400
outlet

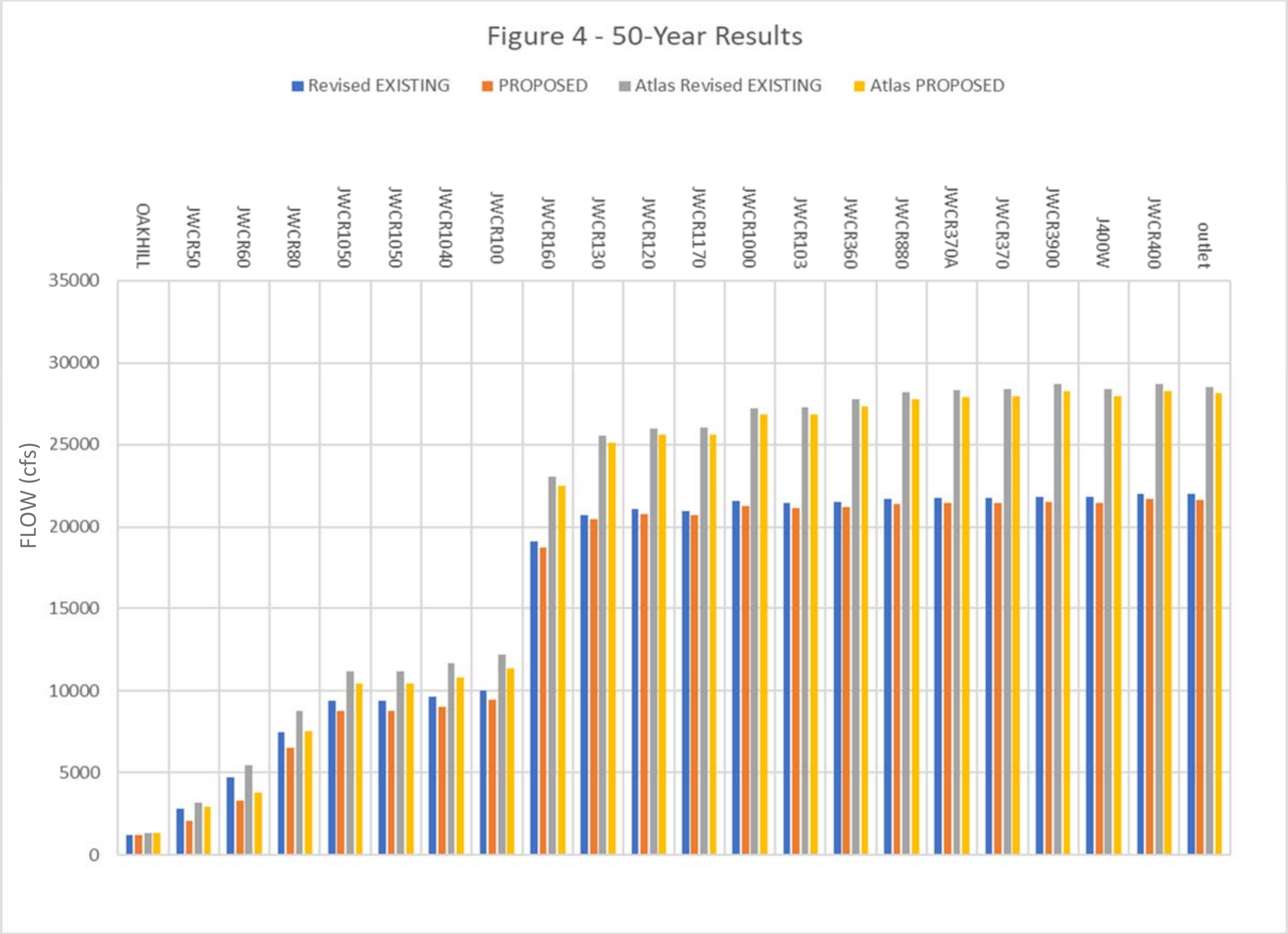
Figure 4 - 50-Year Results
Revised EXISTING
PROPOSED
Atlas Revised EXISTING
Atlas PROPOSED
FLOW (cfs)
35000
30000
25000
20000
15000
10000
5000
0
OAKHILL
JWCR50
JWCR60
JWCR80
JWCR1050
JWCR1050
JWCR1040
JWCR100
JWCR160
JWCR130
JWCR120
JWCR1170
JWCR1000
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JWCR360
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JWCR370A
JWCR370
JWCR3900
J400W
JWCR400
outlet

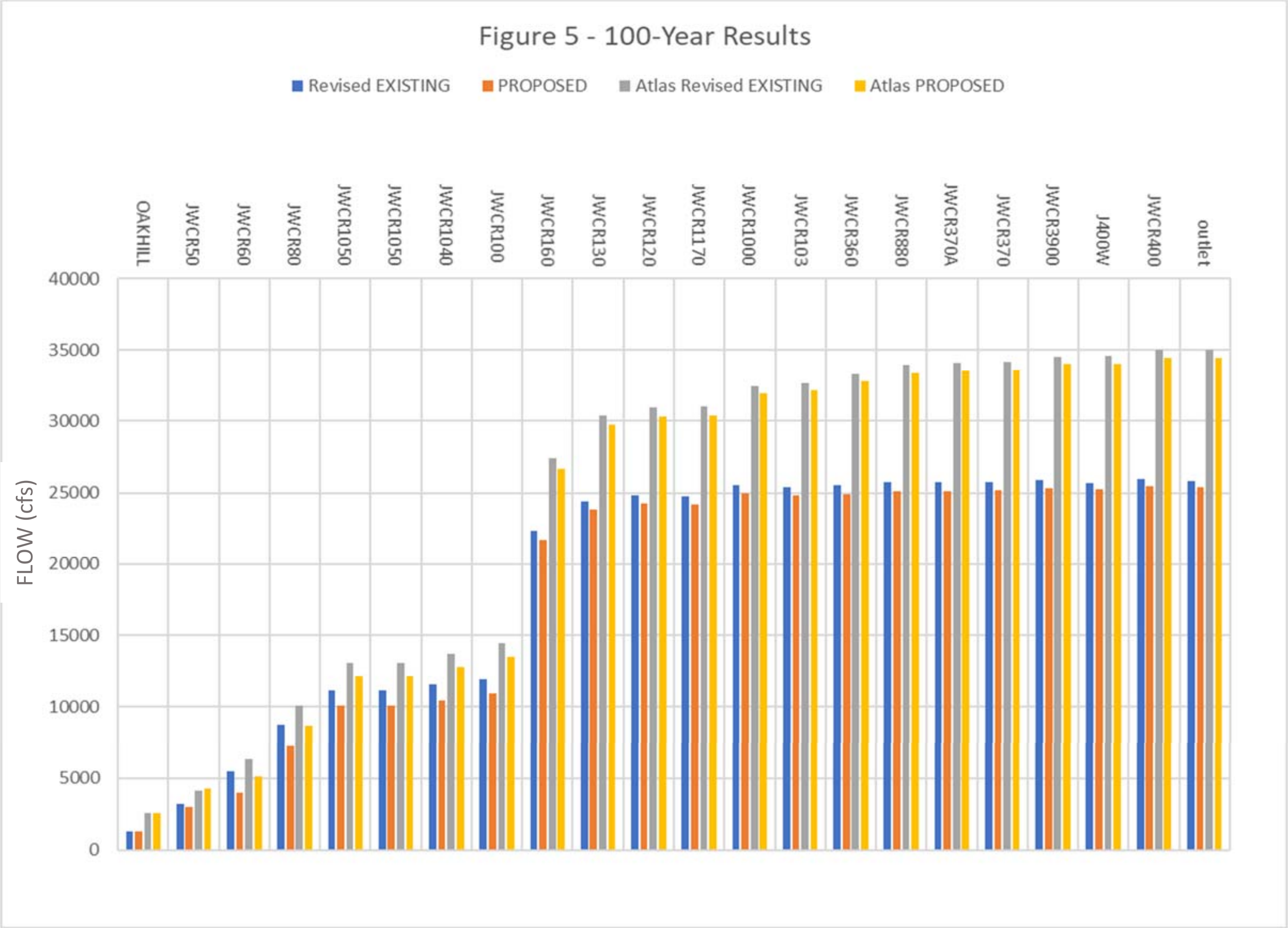
Figure 5 - 100-Year Results
Revised EXISTING
PROPOSED
Atlas Revised EXISTING
Atlas PROPOSED
FLOW (cfs)
40000
35000
30000
25000
20000
15000
10000
5000
0
OAKHILL
JWCR50
JWCR60
JWCR80
JWCR1050
JWCR1050
JWCR1040
JWCR100
JWCR160
JWCR130
JWCR120
JWCR1170
JWCR1000
JWCR103
JWCR360
JWCR880
JWCR370A
JWCR370
JWCR3900
J400W
JWCR400
outlet

5. Hydraulic Impact – Atlas 14 Update, September 2018

The 100-year discharges calculated in HEC-HMS from the updated Atlas 14 rainfall have been incorporated to the project HEC-RAS model. The results are illustrated in the following tables and graphs.

The Atlas 14 PROPOSED condition only includes the changes described previously for the COA and TCEQ dam compliance. All other infrastructure is unchanged from previous work. Infrastructure changes under proposed conditions to mitigate WSEL increases from the Atlas 14 update are described later in this technical memorandum.

The following Table 5 summarizes the mitigated results prior to and after the Atlas 14 updates. Prior to the updates, only one river station location, 75491, still indicated a slight rise in 100-year elevations from existing to proposed. After the Atlas 14 updates are applied the locations along Williamson Creek showing WSEL impacts have significantly expanded.

In particular, there are sustained impacts downstream of the HWY71 proposed regional detention and upstream of the State Highway 71 bridge. These are due primarily to a 100-year discharge increase from the Oak Hill regional detention facility. The Oak Hill facility is located in Williamson Creek immediately upstream from its confluence with the HWY71 detention tributary. The Oak Hill increase is illustrated in Figure 6 and Figure 7, showing the HEC-HMS results output.

The Oak Hill facility is owned and operated by the COA and is a critical structure in the analysis. With the application of updated Atlas 14 flow data, there is a one-foot increase in the peak 100-year elevation in the pond. The previous peak discharge was 1,290 cfs, and after the Atlas 14 updates, the peak discharge increases to 2,558 cfs. Despite a near two-fold increase in outflow from the Oak Hill facility, there is only a moderate inflow increase from 4,000 cfs to 4,673 cfs. This indicates that most of the increase can be attributed to the performance capacity of the existing structure rather than the rainfall increases.

Table 5 - Previous Mitigated 100-year Results Prior to and After Atlas 14 Updates

HEC-RAS River Station	Location	100-Year Prior to Atlas 14			100-Year After Atlas 14		
		Existing W.S. Elev (ft)	Proposed W.S. Elev (ft)	Change	Existing W.S. Elev (ft)	Proposed W.S. Elev (ft)	Change
89063	Downstream of HWY71 Detention	912.48	912.29	-0.19	913.19	913.25	0.06
86254		887.43	887.19	-0.24	888.66	888.77	0.11
85611		883.63	883.36	-0.27	884.79	884.92	0.13
85045		879.44	879.43	-0.01	881.18	881.63	0.45
84982	Upstream of State Hwy 71 Bridge	879.4	879.39	-0.01	881.19	881.77	0.58
84745		878.54	877	-1.54	879.11	877.88	-1.23
84632	Downstream of Both Proposed Ponds	877.66	876.44	-1.22	878.29	877.42	-0.87
83997		874.68	873.42	-1.26	875.32	874.42	-0.9
79948		845.88	845.14	-0.74	846.46	845.83	-0.63
79547		842.98	842.83	-0.15	843.34	843.13	-0.21
79004	Main Lanes Berm	840.09	838.4	-1.69	840.57	840.04	-0.53
78807	Upstream of Old Bee Cave	837.08	836.99	-0.09	837.35	839.67	2.32
78502	Upstream of WestBound Flyover	834.6	834.21	-0.39	835.12	834.94	-0.18
77960		831.34	830.57	-0.77	831.84	831.33	-0.51
77525		827.84	827.1	-0.74	828.45	827.53	-0.92
76871		823.3	822.78	-0.52	823.64	824.59	0.95
76786	Upstream of William Cannon Bridge	822.8	822.49	-0.31	823.4	824.33	0.93
76285		818.34	817.17	-1.17	818.61	817.58	-1.03
75854		815.45	815.22	-0.23	815.87	816.2	0.33
75491		814.34	814.38	0.04	814.82	815.72	0.9
75171.9		813.21	813.06	-0.15	813.65	815.26	1.61
75017	Upstream of US 290 Crossings	813.16	812.17	-0.99	813.6	815.24	1.64
74437		808.41	807.64	-0.77	808.86	808.17	-0.69
74163		805.59	805.38	-0.21	805.9	805.75	-0.15
74022	Upstream of Joe Tanner	805.18	804.98	-0.2	805.47	805.32	-0.15
73960		804.9	804.71	-0.19	805.14	805.01	-0.13

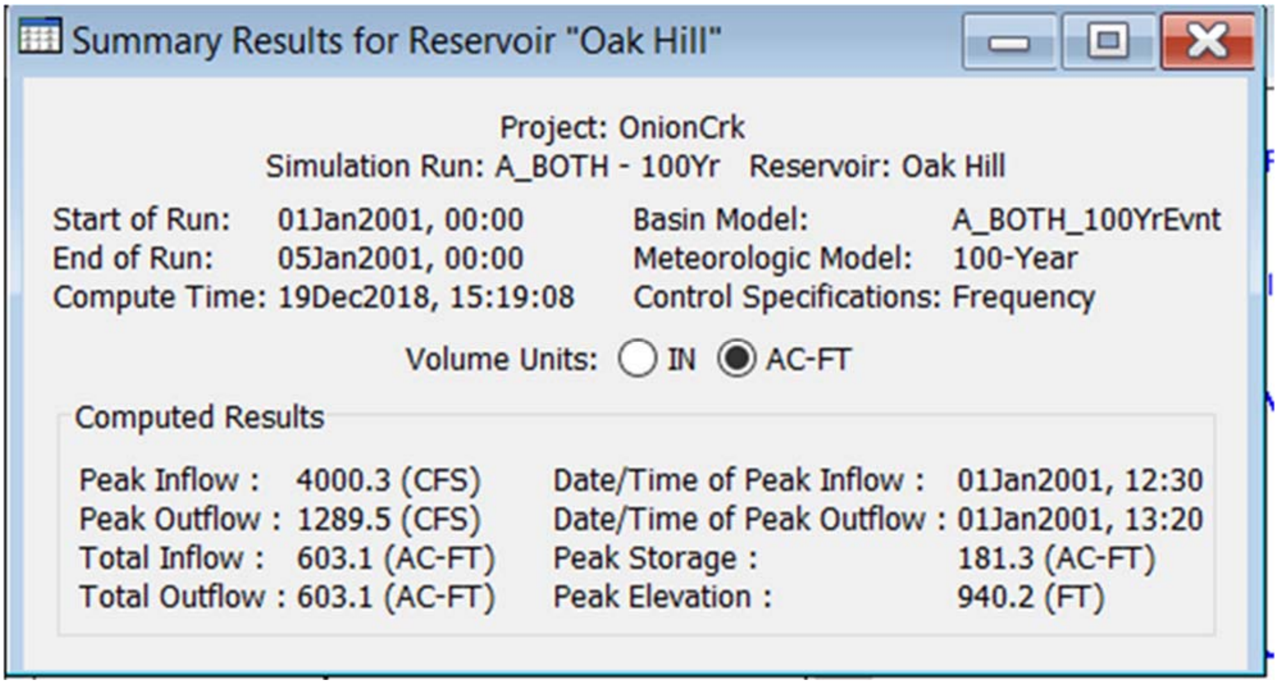

Figure 6 - Oak Hill Detention Facility Prior to Atlas 14 Update

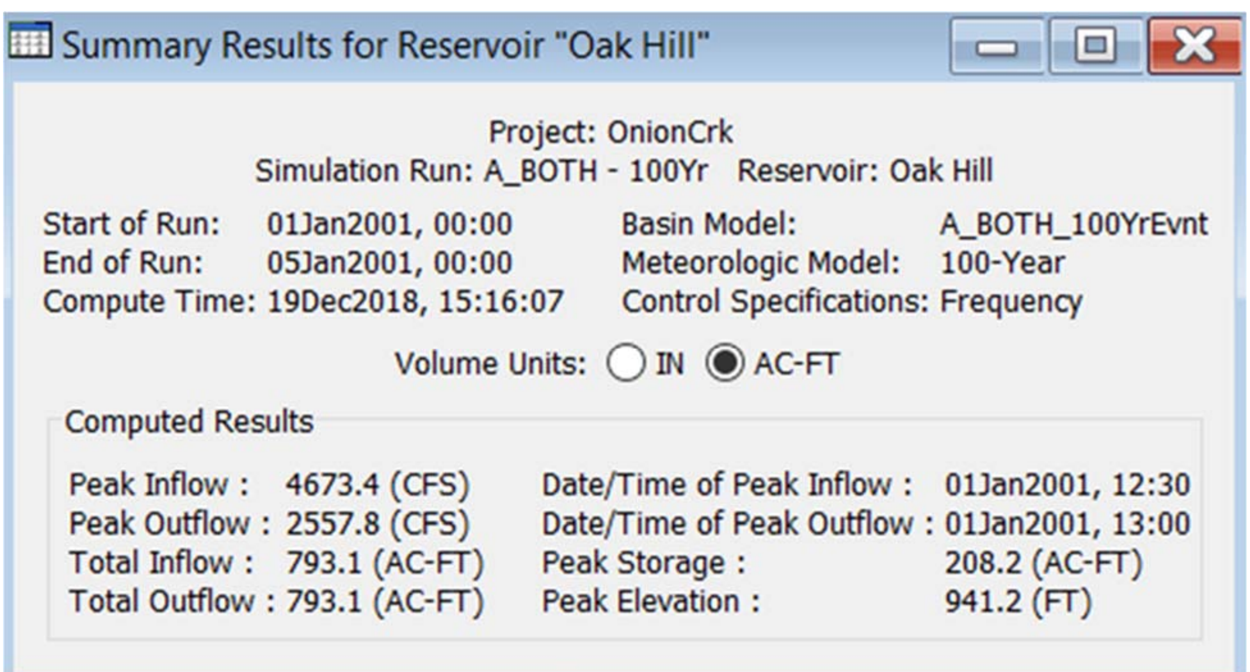

Figure 7 - Oak Hill Detention Facility After Atlas 14 Update

6. Assessment of Additional Storage in Proposed Regional Detention

The HEC-HMS modeling, with updated Atlas 14 data, shows a reduction in the 100-year peak flows throughout the reach, except for downstream of the HWY 71 detention facility, as shown in Table 4 HMS Node "JWCR60". This increase is due to a combination of the increased Atlas 14 rainfall and the reduced mitigative effect of the existing Oak Hill storage facility. Additional model runs were conducted to assess the impact of additional storage in the proposed regional detention facilities. The additional storage would consist of natural ground excavation to increase the available storage volume within the footprints of the two reservoirs. Where feasible, 20% additional volume was used to assess whether greater impact reduction could be realized.

In the following Table 6, there are four proposed conditions that are compared to the existing condition subsequent to the Atlas 14 update. Prop1 is the previous configuration of the proposed regional facilities examined with the Atlas 14 updates. Prop2 is the COA/TCEQ compliance configuration for the two proposed regional detention facilities. Prop3 is the COA/TCEQ compliance configuration along with an additional 20% storage volume within detention footprint. Prop4 is a review of the result if no regional detention facilities are constructed.

There are similar results for Prop1, Prop2, and Prop3. Prop1 resulted in marginally better results within the project footprint, but marginally worse downstream of the HWY71 detention. Prop3 with the additional storage is only slightly improved over the Prop2 results, and this is not enough benefit to justify the cost of storage excavation within the proposed detention footprints. The Prop4 results indicate a potential need for additional regional detention to assist with mitigating 100-year elevation increases.

Table 6 - Additional Proposed Scenarios for the 100-year Event

HEC-RAS River Station	Location	Existing W.S. Elev - ft	Prop1 W.S. Elev - ft	Change	Prop2 W.S. Elev - ft	Change	Prop3 W.S. Elev - ft	Change	Prop4 W.S. Elev - ft	Change
89063	DS of HWY71 Detention	913.19	913.32	0.13	913.25	0.06	913.29	0.10	913.19	0.00
86254		888.66	888.87	0.21	888.77	0.11	888.86	0.20	888.66	0.00
85611		884.79	885.01	0.22	884.92	0.13	885.01	0.22	884.83	0.04
85045		881.18	881.72	0.54	881.63	0.45	881.72	0.54	881.75	0.57
84982	US of State Hwy 71 Bridge	881.19	881.88	0.69	881.77	0.58	881.88	0.69	881.89	0.70
84745		879.11	878.00	-1.11	877.88	-1.23	877.91	-1.20	878.84	-0.27
84632	DS of Both Proposed Ponds	878.29	877.53	-0.76	877.42	-0.87	877.45	-0.84	878.29	0.00
83997		875.32	874.55	-0.77	874.42	-0.90	874.46	-0.86	875.32	0.00
79948		846.46	845.72	-0.74	845.83	-0.63	845.83	-0.63	846.50	0.04
79547		843.34	843.06	-0.28	843.13	-0.21	843.13	-0.21	843.22	-0.12
79004	Main Lanes Berm	840.57	839.79	-0.78	840.04	-0.53	840.03	-0.54	840.96	0.39
78807	US of Old Bee Cave	837.35	839.34	1.99	839.67	2.32	839.66	2.31	840.59	3.24
78502	US of WestBound Flyover	835.12	834.82	-0.30	834.94	-0.18	834.94	-0.18	835.56	0.44
77960		831.84	831.20	-0.64	831.33	-0.51	831.33	-0.51	831.98	0.14
77525		828.45	827.70	-0.75	827.53	-0.92	827.53	-0.92	828.00	-0.45
76871		823.64	823.51	-0.13	824.59	0.95	824.58	0.94	826.50	2.86
76786	US of W. Cannon Bridge	823.40	823.19	-0.21	824.33	0.93	824.31	0.91	826.56	3.16
76285		818.61	817.52	-1.09	817.58	-1.03	817.57	-1.04	817.99	-0.62
75854		815.87	815.81	-0.06	816.20	0.33	816.20	0.33	816.91	1.04
75491		814.82	815.16	0.34	815.72	0.90	815.71	0.89	816.51	1.69
75171.9		813.65	814.37	0.72	815.26	1.61	815.25	1.60	816.20	2.55
75017	US Main Lanes and Frontage	813.60	813.46	-0.14	815.24	1.64	815.24	1.64	816.18	2.58
74437		808.86	808.11	-0.75	808.17	-0.69	808.17	-0.69	808.42	-0.44
74163		805.90	805.73	-0.17	805.75	-0.15	805.75	-0.15	805.93	0.03
74022	US of Joe Tanner	805.47	805.32	-0.15	805.32	-0.15	805.32	-0.15	805.50	0.03
73960		805.14	805.03	-0.11	805.01	-0.13	805.00	-0.14	805.16	0.02

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7. Further Analysis of Existing Oak Hill and Proposed HWY 71 Detention Facilities

The application of updated Atlas 14 flows resulted in several areas of increased impact in the HEC-RAS modeling. While the modifications to proposed infrastructure downstream are discussed in detail later in this memorandum, a particular range of impact, occurring just downstream of the proposed HWY 71 detention facility, appear to be the direct result of the combined performance of the existing Oak Hill detention facility and the proposed HWY 71 facility. The previous proposed design with updated flows results in a sustained increase in both flow and WSEL starting at the confluence of the tributaries downstream of the two facilities and extending a mile downstream to the HWY 71 bridge crossing. This extended area of impact, through a well-developed area of the Williamson Creek watershed, necessitates further analysis into how that combined performance might be optimized to mitigate these impacts.

The increased flows appear to be a result of both a significantly increased discharge from the existing Oak Hill facility and a shifted hydrograph peak from the HWY 71 facility. As discussed previously, the updated Atlas 14 flows have increased the volume of 100-year discharge over the top of the Oak Hill dam, resulting in a major increase in peak flow.

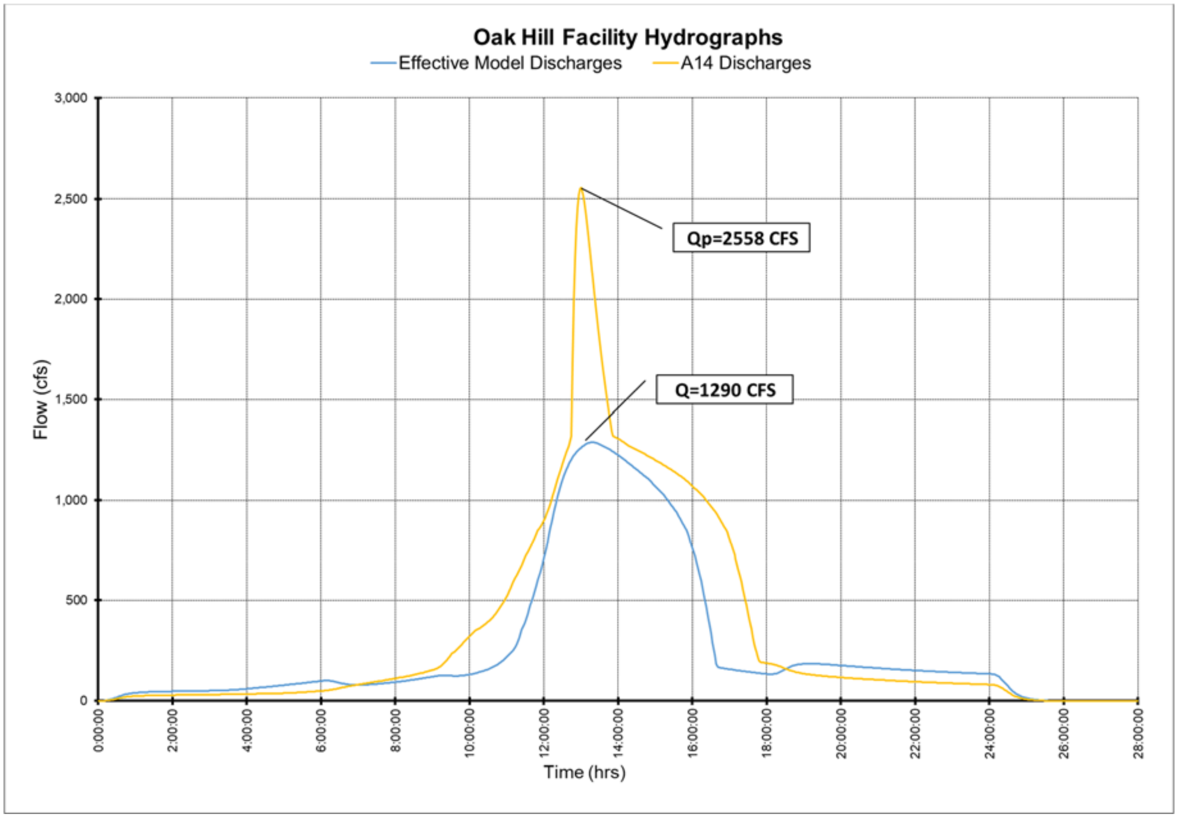

Figure 8 - Oak Hill Facility Hydrographs

The proposed HWY 71 facility peak discharge is decreased and the time to peak is delayed by approximately 6 minutes.

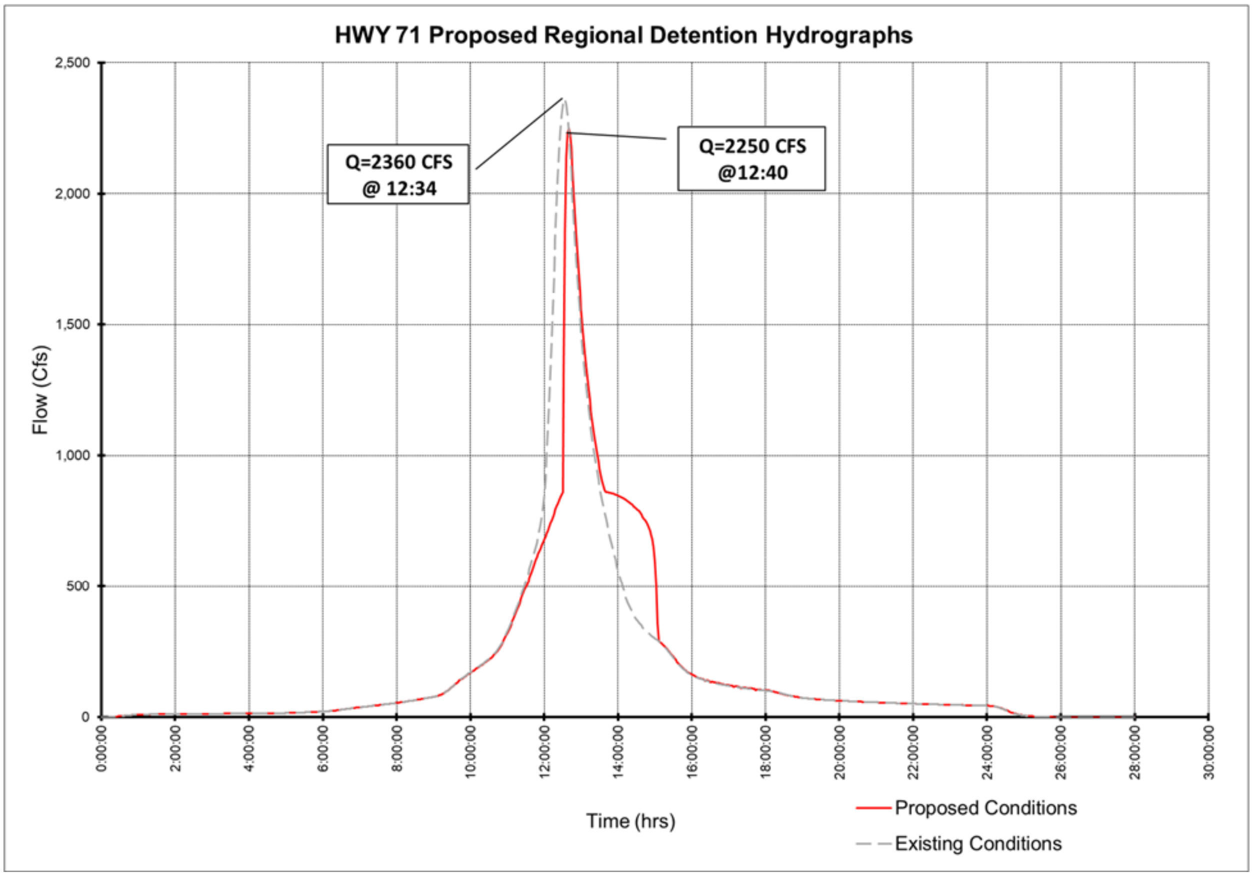

Figure 9 - HWY 71 Proposed Regional Detention Hydrographs

This subtle shift in timing moves the peak from the HWY 71 sub-basin closer to the peak at the confluence between the two detention facility tributaries, resulting in an increased peak discharge at that confluence. While this shift was also present in the previous design, an overall peak reduction was still achieved throughout the modeling. It was only with the addition of the substantial discharge increase from the existing Oak Hill facility that a downstream impact materialized.

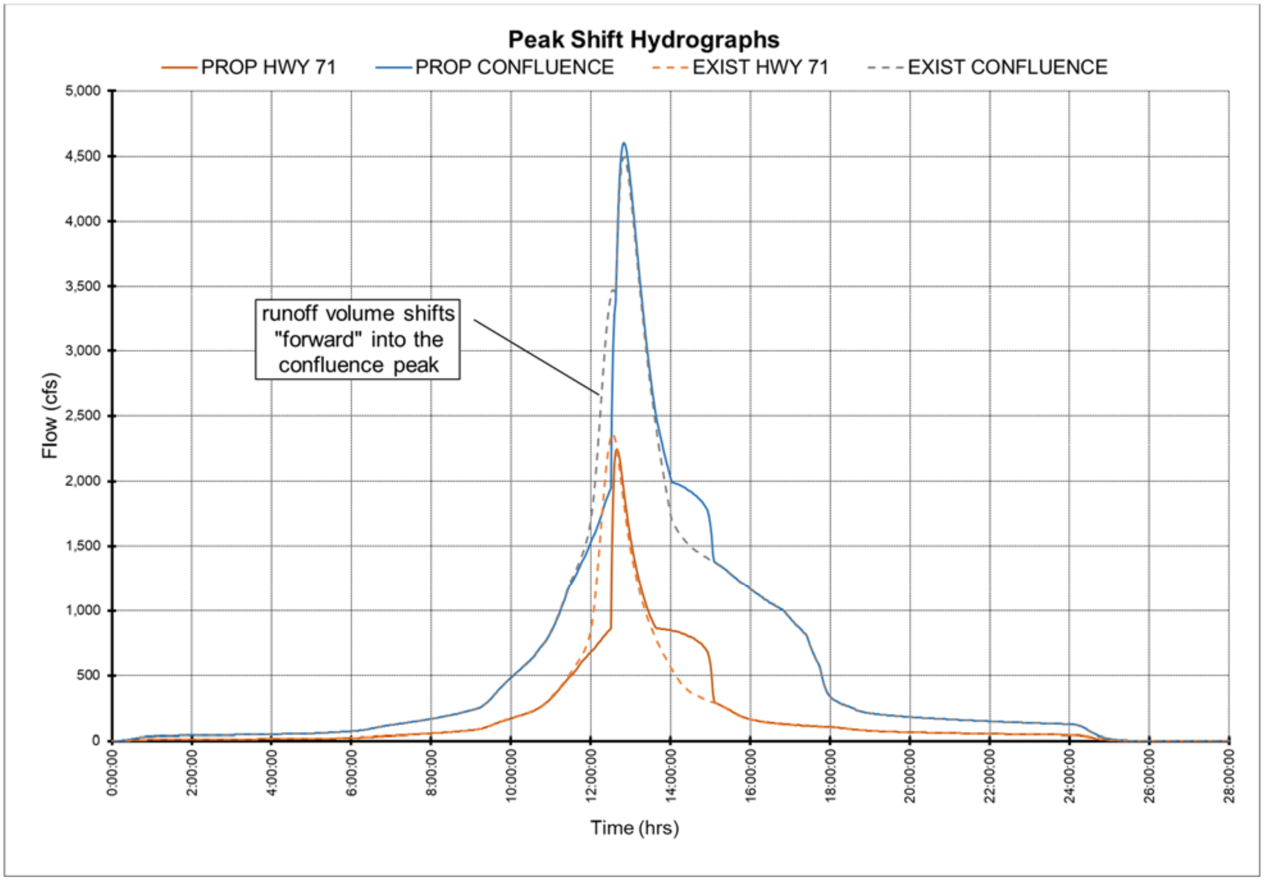

Figure 10 - Peak Shift Hydrograph

Due to the significant influence of the existing Oak Hill facility on the downstream flows, it was determined that an analysis of potential modifications to this facility should be pursued. The effective HMS model, provided by the COA, represents the Oak Hill storage facility via a series of rating curves. The model notes indicate that these curves were developed by Espey Consultants, Inc based on the COA 2003 LIDAR data. It is important to note that reservoir modeling in HEC-HMS (v 3.5) falls into one of two major categories, direct input from outside data, such as ratings curves or gage data, or geometric input which is the defining of individual components of a reservoir from which the software will then generate a rating curve. Therefore, in order to evaluate possible alternate dam configurations for this facility, such as invert changes, outlet redesign, or spillway modifications, it is necessary to switch from the direct input method used in the effective model to a geometric input. Subsequently, the COA provided TNP with data collected from a 2007 Freese & Nichols, Inc (FNI) Dam Safety Study. This data included a stage-discharge rating curve, as well as geometric data, such as a top of dam profile, and basic information on the outlet works including culvert size, length and invert elevations. Since the updated data did not include any storage information for the facility, TNP developed an updated stage-storage curve using the 2012 COA LIDAR. Using the FNI dam data, a new geometric reservoir was modeled in HMS with the dam modeled "as-is" creating a baseline for comparison.

Using this geometric baseline model, several modifications to the existing Oak Hill facility were analyzed, including adjusting the size of the outlet culvert, creating a secondary spillway, and adjusting the dam profile. The only analysis modification found that mitigates downstream impacts requires raising the elevation of the dam. This modification is not recommended due to the risk of affecting flood elevations on adjacent properties. Since no modifications to the existing Oak Hill facility were deemed viable and the rating curve provided by FNI produced higher discharges from the facility than the geometric alternative, all subsequent analyses proceeded using the FNI rating curve.

TNP also analyzed several alternative configurations of the proposed HWY71 facility to determine if the timing of the discharge hydrograph could be shortened or lengthened. TNP determined there is not enough storage in HWY71 facility to lengthen the discharge hydrograph timing. Alternately, releasing discharge more quickly from the HYW71 facility does lessen downstream impacts by shortening the hydrograph timing.

TNP then examined the results of removing the HWY71 facility, while retaining the OBC facility, and compared them to the previous proposed scenario that included both the HWY71 and OBC facilities. These results are found in Table 7. Removing the HWY71 facility produces a peak discharge reduction downstream of the Oak Hill facility as opposed to the increase that occurs if the HWY71 facility is retained. Further downstream in the project area, the peak discharge reductions are not as great after removal of HWY71 than when including HWY71, although there are still substantial peak discharge reductions in the project area. The discharges from this design scenario in which the HWY71 facility is removed, have also been applied and analyzed in the hydraulic modeling. These results are discussed in Section 8 of this Technical Memorandum.

Table 7 - Comparison of Peak Flows (cfs) With and Without HWY 71 Detention

HEC-RAS XS	HMS Node	Existing Atlas 14 Update	Proposed 100 Atlas 14 Update with both HWY71 and OBC Facilities	Difference	Proposed 100 Atlas 14 Update without HWY71 Facility and with OBC Facility	Difference
4031	JWCR90A	2091	2091	0	2091	0
90177	OAK HILL	2821	2821	0	2821	0
89063	JWCR50	4564	4664	100	4564	0
84632	JWCR60	6229	5632	-597	5550	-679
79948	JWCR80	10053	8635	-1419	9224	-829
75171.9	JWCR1050	13049	12121	-928	12516	-533
75017	JWCR1050	13049	12121	-928	12516	-533
67082	JWCR1040	13679	12760	-919	13174	-505
59867	JWCR100	14388	13449	-939	13892	-496
55940	JWCR160	27311	26601	-710	26912	-399
50574	JWCR130	30304	29671	-633	29950	-354
49429	JWCR120	30909	30294	-615	30559	-350
46107	JWCR1170	30977	30351	-626	30628	-349
43122	JWCR1000	32429	31883	-546	32113	-316
37465	JWCR103	32648	32117	-530	32344	-304
30000	JWCR360	33283	32752	-530	32983	-300
23527	JWCR880	33882	33360	-522	33588	-294
17814	JWCR370A	34029	33499	-530	33729	-300
13810	JWCR370	34115	33587	-528	33817	-297
7301	JWCR3900	34480	33954	-526	34181	-300
4393	J400W	34528	34000	-527	34229	-299
2454	JWCR400	34951	34436	-515	34658	-293
618	outlet	34949	34433	-516	34655	-294

8. Mitigation of Hydraulic Impact – Atlas 14 Update, September 2018

Due to the increased existing and proposed flows based on the updated Atlas 14 precipitation data and the removal of the HWY 71 storage facility, a review of the previous hydraulic analysis (Nov-2018 TNP -formerly HHR) determined that the previously proposed crossing designs were insufficient to mitigate the 100-year WSEL increases.

The updated discharges were first applied to the revised existing conditions model to establish the basis for comparison. The flows were then applied to the proposed conditions model, which reflects the project's approved schematic design, or 'Concept A' as described in the November 2018 report. The incorporation of updated discharges resulted in the inundation of most proposed bridge crossings in the project area by the 100-yr event flows and of the roadway profile at the design flood in multiple locations. These inundations resulted in several areas showing a rise in WSELs compared to existing conditions. Further revisions to the bridge designs and overbank mitigations are necessary to reduce these impacts to proposed conditions.

8.1. Hydraulic Modeling

8.1.1. Revised Existing Conditions

Applying the updated Atlas 14 flows to the revised existing conditions model results in an overall increase in WSELs throughout the stream reach under consideration, ranging from 0.06' to 2.18' higher than in previous existing conditions modeling. In the previous modeling, the US 290 bridge crossing was inundated by the 100-year event; with the updated discharges, the inundation depth increases by 0.41' for the 100-year event. In previous modeling the William Cannon existing bridge was also inundated by the 100-year event; the inundation depth increases by 0.53' with the updated discharges. The existing low water crossing at Old Bee Cave Road had an inundation depth of 9.01', and with increased flows, the inundation is now increased to 9.28'. The WSEL above the Highway 71 crossing has also increased by 1.76'. This results in inundation of the low chord for this crossing, whereas the previous modeling had 0.58' of freeboard during the 100-year event.

8.1.2. Proposed Conditions Revisions

In the previous hydraulic analysis (Nov-2018 TNP – formerly HHR), preliminary bridge openings were designed to meet the service levels for proposed improvements by establishing low chords, span lengths and abutment designs necessary to meet the design criteria. The original design criteria established a 25-year event service level for all frontage roads and a 100-year event service level for all main lane roadways. In this latest reassessment the design requirements were further refined by the TxDOT Austin District. The previous design service level of 25 years for all frontage roads was retained while the main lanes now require a 50-yr service level with one-foot minimum freeboard below bridge low chords. An exception applies at Old Bee Cave bridge, where TxDOT requested one-foot minimum freeboard above the 100-yr WSEL, due to a commitment to the City of Austin. Mitigation of potential impacts due to increases in WSEL are still based upon the 100-yr discharge.

Applying, the increased Atlas 14 flows necessitated several design adjustments along Williamson Creek. Various bridge configurations and profile changes were applied to the frontage road and main lane alignments; proposed overbank grading and channel benching were altered, added, or expanded to increase conveyance. While most modifications to the bridge crossing profiles were required in order to meet minimum hydraulic design criteria, other considerations beyond hydraulic performance, such as intersection connections, clearances, and relocation of the shared use the path, resulted in roadway/bridge profile adjustments. TNP used schematic profile data and typical sections where available, but due to the preliminary nature of the design, some assumptions were necessary.

US 290 Main Lane and Frontage Road Grouped Crossings

The US 290 crossing at Williamson Creek, which consists of the eastbound frontage road bridge, the west bound main lane bridge, the eastbound main lane bridge, and the westbound frontage road bridge, required several design changes to produce a mitigated condition for 100-year flows based on Atlas 14 data. The increased flows result in a submerged low chord at both the eastbound and westbound frontage road bridges. These pressure flow transitions adversely affect the proposed water surface profile contributing to significant upstream rise.

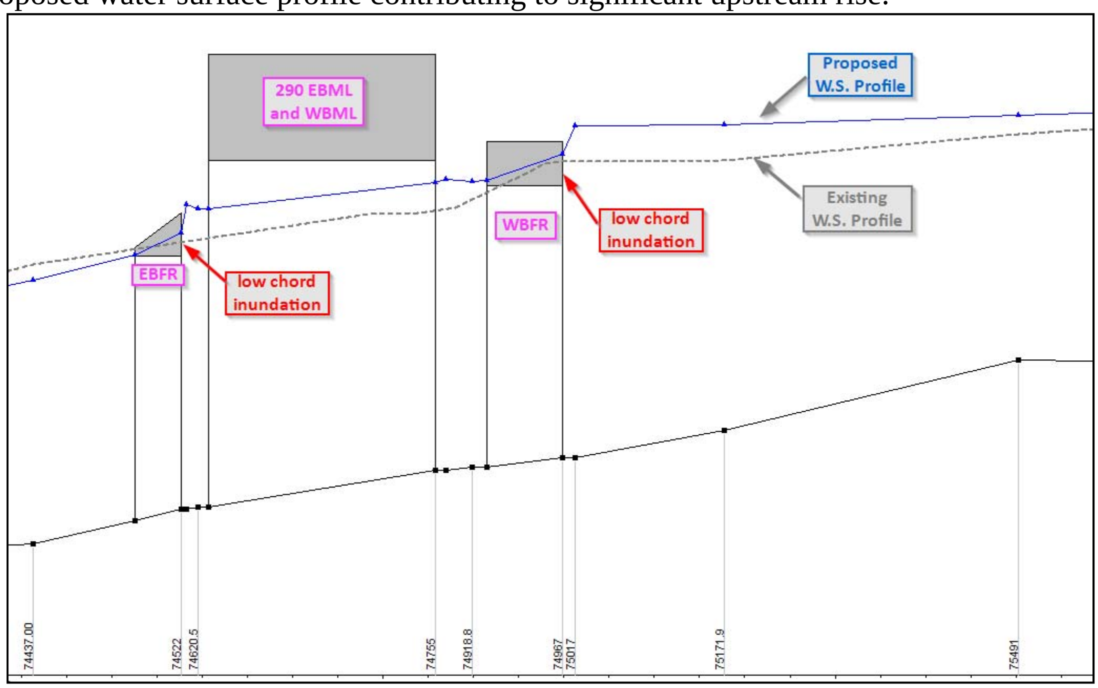

Figure 11 - Backwater Rise with Low Chord Inundation

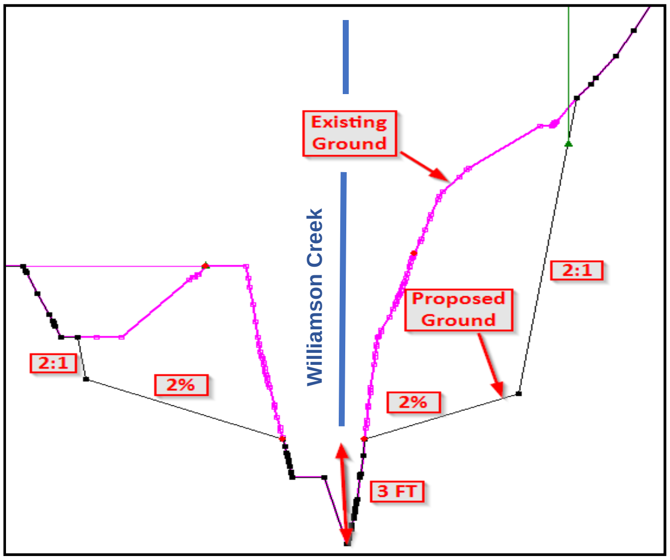

Figure 12 - Overbank Grading Details

Initial grading modifications were applied in the vicinity of the US 290 bridges. Originally, the overbank mitigation grading began at the bank point located approximately 5 feet above the channel bottom, and the overbank grading extended at a slope of 2% or greater to maintain proper drainage to the channel. In order to increase conveyance further this grading was dropped to 3 feet above the creek flowline. These graded areas used catch slopes ranging from 3:1 to 2:1 depending on location needs.

Figure 12 - Overbank Grading Details

In addition to these changes, right-of-way and easement limits necessitated alterations to the overbank grading boundaries in areas up and downstream of the 290 crossings, resulting in several areas being reduced.

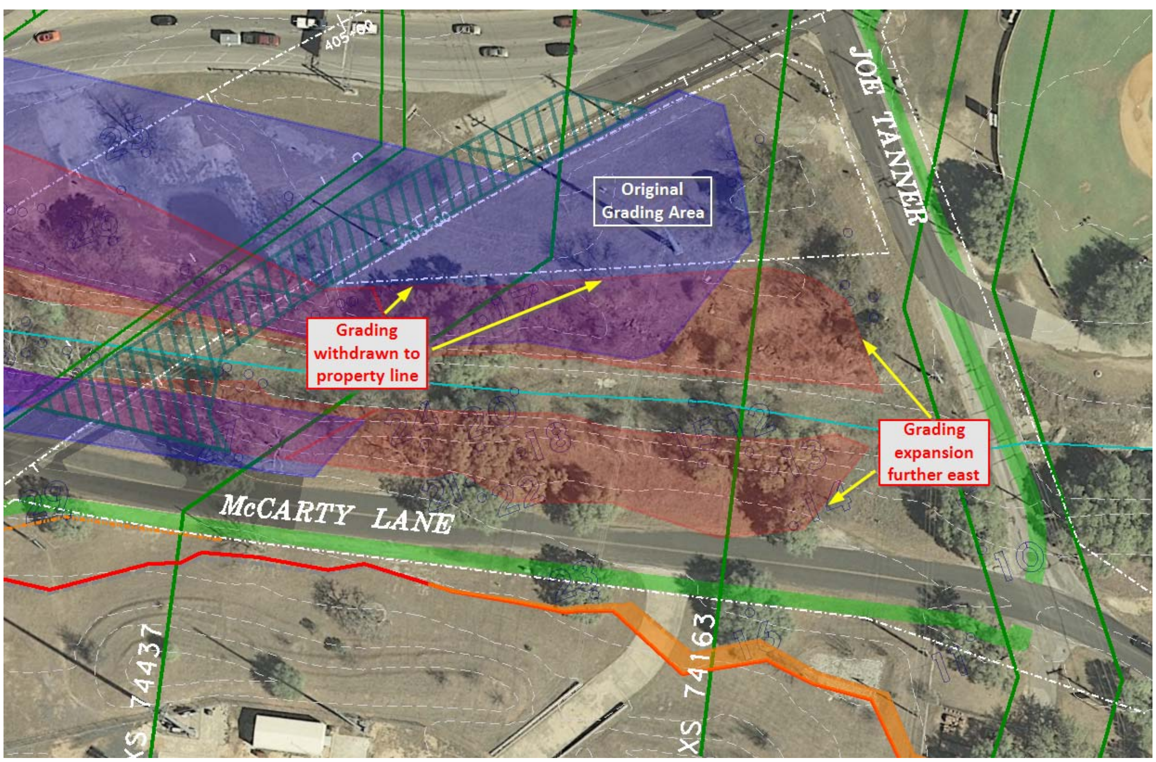

Figure 13 - Overbank Grading Changes Downstream of US 290

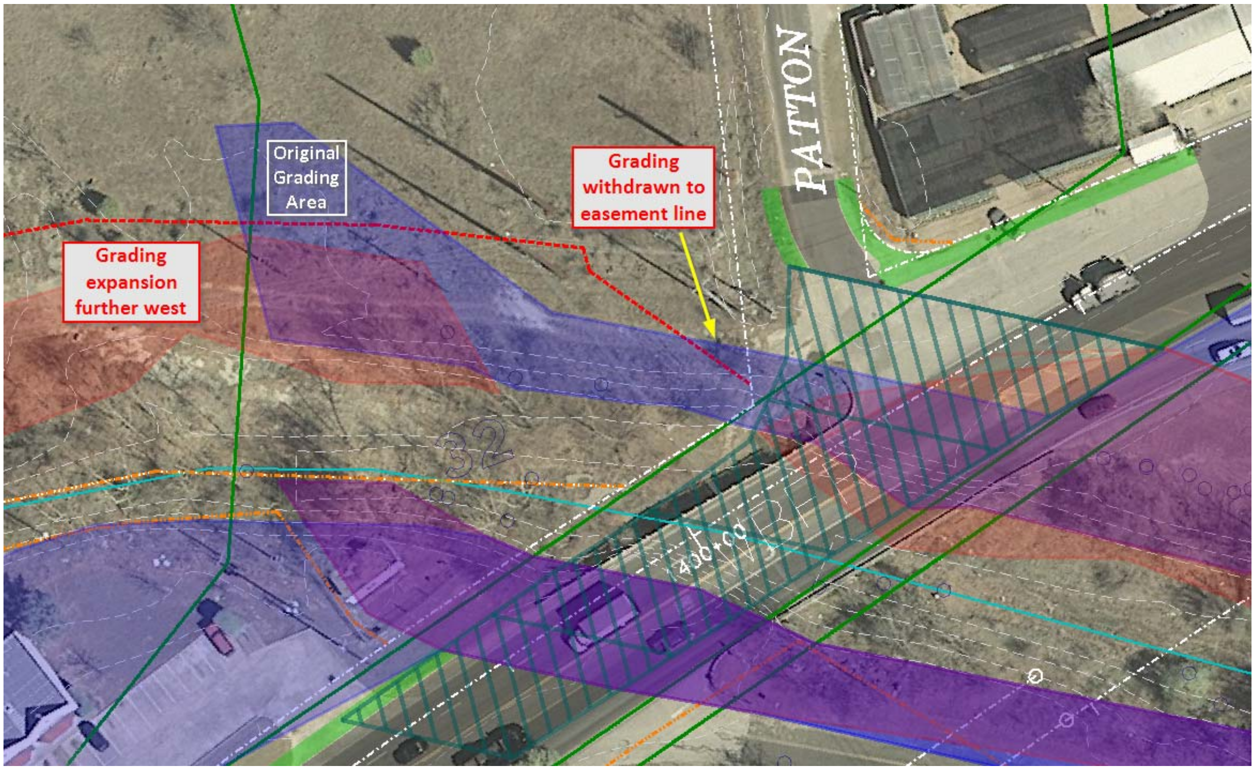

Figure 14 - Overbank Grading Changes Upstream of US 290

Even with these overbank modifications applied throughout the crossings, the increased conveyance from grading alone is not sufficient to mitigate all proposed impacts.

The following structural design changes were applied to further mitigate beyond grading. Both the eastbound and westbound frontage road bridge profiles were raised until the low chords were no longer inundated. Additionally, the main lane bridge design is significantly modified from the previous design iteration. In the previously modeled design, the main lane bridges began at approximately STA 400+00 and ended at approximately STA 404+00. In the revised schematic design, both the east and westbound main lanes crossings have been altered to serve as continuously raised lanes beginning at STA 394+00. This elevated main lane crossing allows more overbank conveyance than previously modeled under US 290. This is reflected in the HEC-RAS modeling as continuous bridge to the west with no abutment structure, an east bank abutment with a 2:1 catch slope transitioning back to roadway on infill, and 36-inch diameter columns for interior bents.

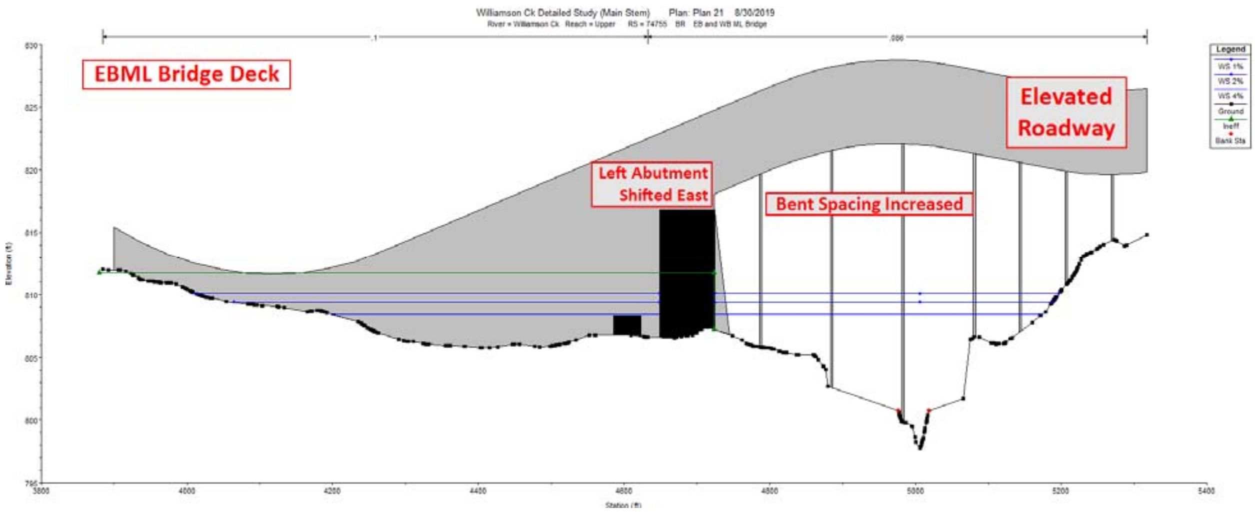

Figure 15 - Main Lane Structural Details

Despite higher roadway profiles and expanded overbank grading, persistent WSEL increases upstream of the proposed 290 crossings remain. It was ultimately determined that significant mitigation could be achieved if the westbound frontage road bridge could be extended 56 feet further to the west. The previous bridge began at WBFR station 1398+97 and ended at station 1401+81, for a total bridge length of approximately 284 feet. The extended WBFR bridge beginning station shifts to the west, starting at WBFR station 1398+41, with a new total bridge length of 340 feet.

In addition to expanding the westbound frontage road bridge, alternate span configurations were analyzed for each crossing. For the EBFR bridge the approximately 7 – 60’ spans were reconfigured in a 70’–90’–90’–90’–70’ arrangement; this reduction in number of bents and associated columns produced additional upstream impact reductions. The main lane pier spacing was also adjusted from its previous configuration using 75’ continuous spans to 90’ continuous spans from the west, with three expanded spans directly over Williamson Creek of 145’ and a final 100’ span adjacent to the eastern abutment. The expanded WBFR was altered from its original 5 – 55’ span configuration to 4 – 85’ spans as a final layout. All final recommended bridge configurations are found in Table 9-16.

These design modification in total result in the elimination of all WSEL impacts directly upstream of the grouped 290 crossings, while each structure also meets the level of service required. However, raised roadway profiles in the overbank area on the east bank downstream of the eastbound frontage road bridge have resulted in an increased water surface elevation in cross-sections 73862, 73960, 74022, and 74163. This area, just upstream and downstream of the existing Joe Tanner low water crossing, modeling indicates increased WSEL’s for the 100-year event, ranging from 0.08’ to 0.64’. With the areas immediately up and downstream of this isolated rise showing a lower proposed WSEL, it can be assumed that the resulting impact in this area would be minimal. Further evaluation of the surrounding property does not show any apparent roadway inundation or impact to any nearby structures. An approximate location of the 100-year inundation impact can be seen in Exhibit A-2. Despite expanding the overbank grading, TNP is unable to achieve the strict no rise result at these downstream cross-sections. With detailed survey of the banks and channel, this isolated rise in WSEL may be resolved in final project design.

William Cannon Drive Bridge

The previous proposed design for the William Cannon Drive bridge is no longer sufficient to mitigate impacts from the updated Atlas 14 discharge, which results in a submerged low chord at the upstream face of bridge. This condition results in a significant rise in WSELs immediately upstream of the crossing. Overbank modifications lowering the benching level from 5 feet above to 3 feet above the existing flowline were applied in the sections up and downstream of the crossing, mitigating some of the impacts. In addition, the bridge profile has been raised, the bridge length was extended by approximately 40 feet, and, due to limited right-of-way in the vicinity of this crossing, the bridge was shifted approximately 25 feet southeast. The previous abutment design slope of 2:1 on the left bank (north) remains unchanged, while a vertical abutment on the right bank (south) has been added. The previous span configuration of 5 – 40' spans was also revised to 4 – 50' spans.

Despite the elimination of the submerged low chord and the increased conveyance due to overbank grading and bridge expansions, impacts were still present in several upstream cross-sections. Additional overbank grading, using the lowered 3-foot benching design, was applied in the left overbank in an area spanning WBML station 372+00 to station 377+00, and in the right overbank area from EBML station 371+00 to station 377+00. This additional grading further mitigates the impacts due to proposed development, resulting in no increased water surface elevations directly upstream of the bridge between the William Cannon crossing and the westbound main lane flyover.

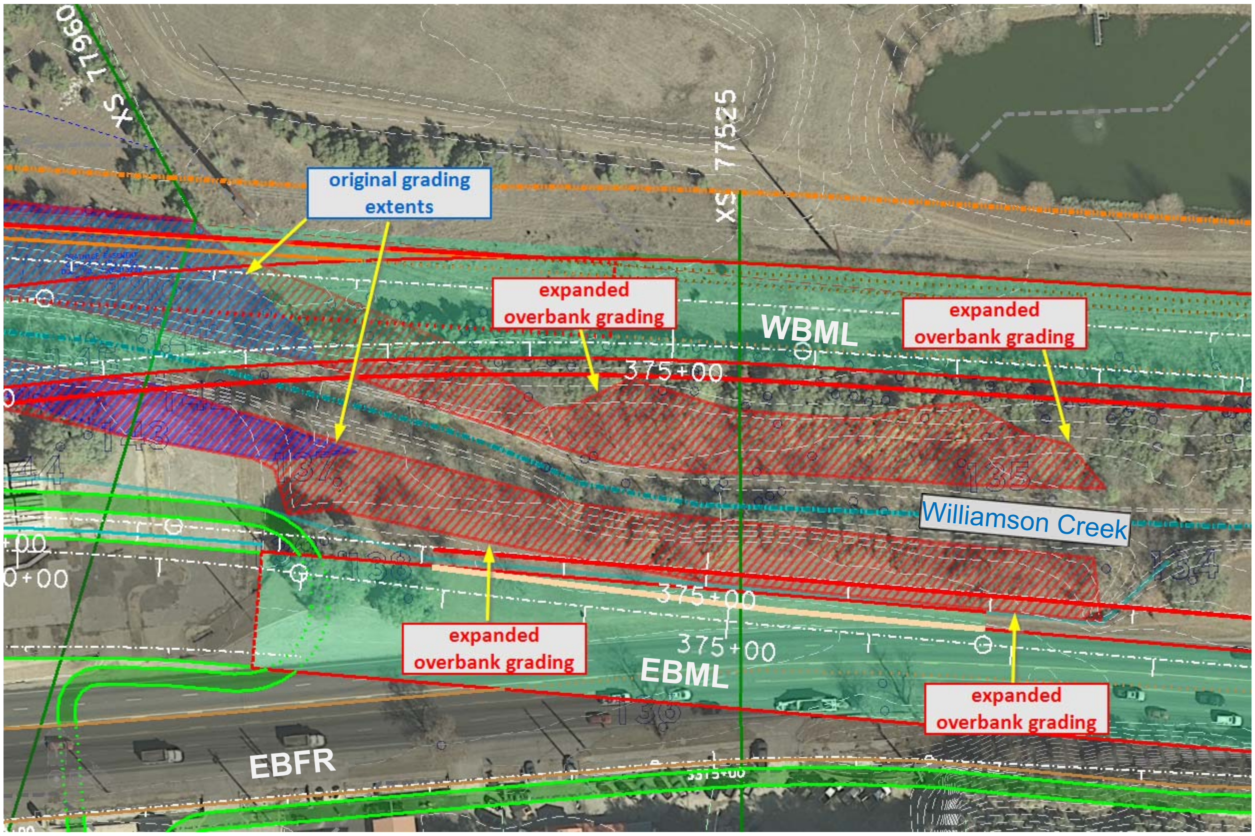

Figure 16 - Additional Overbank Grading Upstream of William Cannon Bridge

Westbound Main Lane Flyover

As discussed in detail in the previous analysis, the schematic 'Concept A' design calls for a shift of the westbound main lanes to the north bank of Williamson Creek. The resulting main lane flyover structure is proposed just east of the Old Bee Cave bridge crossing. Overbank grading modifications from the previous 5-foot design have been lowered to the 3-foot benching design and applied to cross-sections 77960 through 78502. In addition to the grading changes, a proposed extension of the flyover bridge by approximately 120' to the southwest has also been incorporated. These improvements adequately mitigate the impacts due to proposed development resulting in no increased water surface elevations between the westbound main lane flyover and the Old Bee Cave Road crossing.

Old Bee Cave Road Bridge

In the previous analysis, initial design assumptions used to develop the Old Bee Cave crossing were outlined in detail. These same assumptions have been applied to the Atlas 14 update analysis. In addition to updating the discharges, the overbank grading boundaries upstream of the crossing were reduced significantly due to limited right-of-way. These changes result in a submerged upstream low chord and a rise in WSELs in several upstream cross-sections. Therefore, a lower benching level of 3 feet above the flowline was applied throughout this crossing, an extension of the bridge of approximately 60 feet further west, and the bridge was shifted approximately 6 feet north to optimize conveyance and remain within the right-of-way. Even with grading and bridge expansion an increase from previous design in the upstream low chord elevation was required in order to achieve the minimum freeboard over the 100-year WSEL. The previous span configuration of 3-105' spans was altered for the newly expanded bridge to 4-95' spans. As discussed in the previous analysis, the proposed improvements still require that a more natural stream thalweg be established to maintain a stable stream channel geometry and provide for a feasible bridge structure. Various other iterations, including altered abutment designs, alternative span lengths, and further expanded bridge lengths, have also been considered but are mostly ineffective. It was determined that the most effective mitigation occurs when overbank grading is widened to the south (right) overbank just downstream of the proposed bridge.

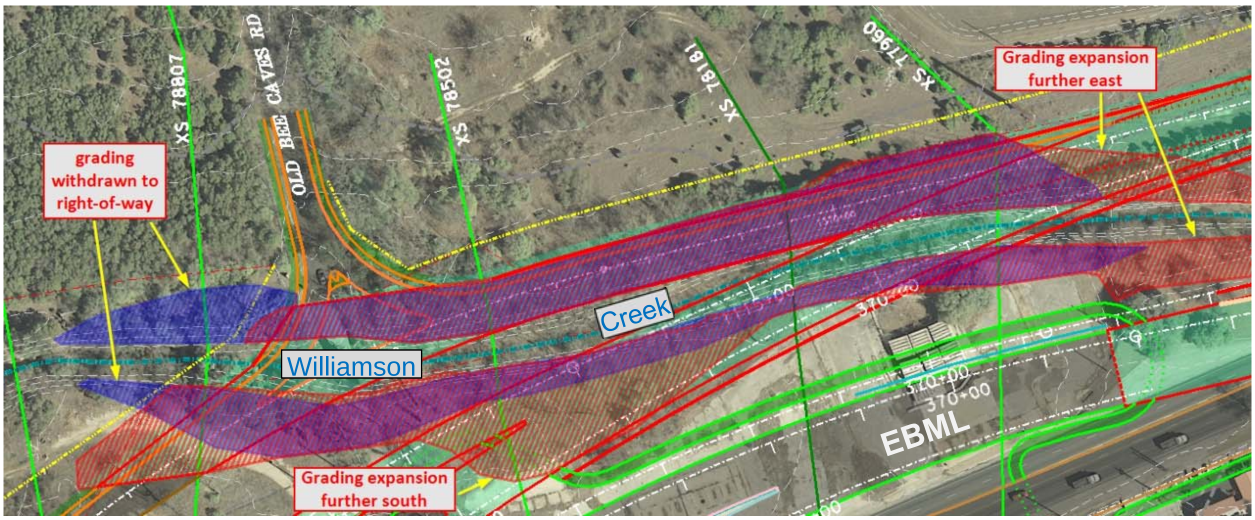

Figure 17 - Grading Modifications near Old Bee Caves Bridge and WBML Flyover

While alterations to the bridge configuration and expanded overbank grading resulted in a reduction of impacts, they did not eliminate all WSEL impacts upstream of the proposed crossing. Despite these mitigation efforts, a rise of 2.75' remains at the upstream face of the proposed Old Bee Cave Bridge, at cross-section 78807, and a 0.12' at cross-section 78807. However, a 0.34' drop in WSEL at cross-section 79948 and a 0.97' drop at cross-section 78502 downstream of the bridge indicate this rise is isolated to the area immediately upstream of the proposed bridge.

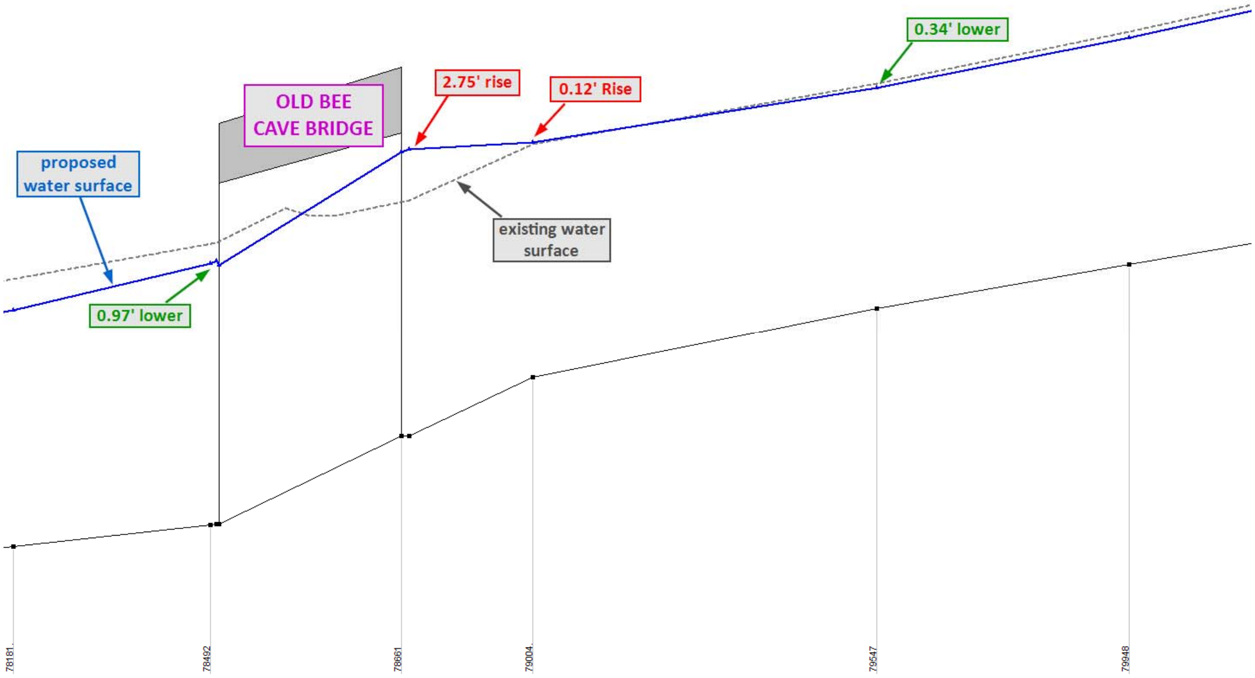

Figure 18 - Old Bee Cave Bridge Backwater Rise

The adjacent properties do not appear to have any insurable structures that would be adversely impacted by this isolated area of rise. Preliminary inundation mapping in Figure 19 show an approximate area of minor impacts due to these isolated sections of rise. As stated previously, with detailed survey of the banks and channel, these increases may be resolved in the final project design.

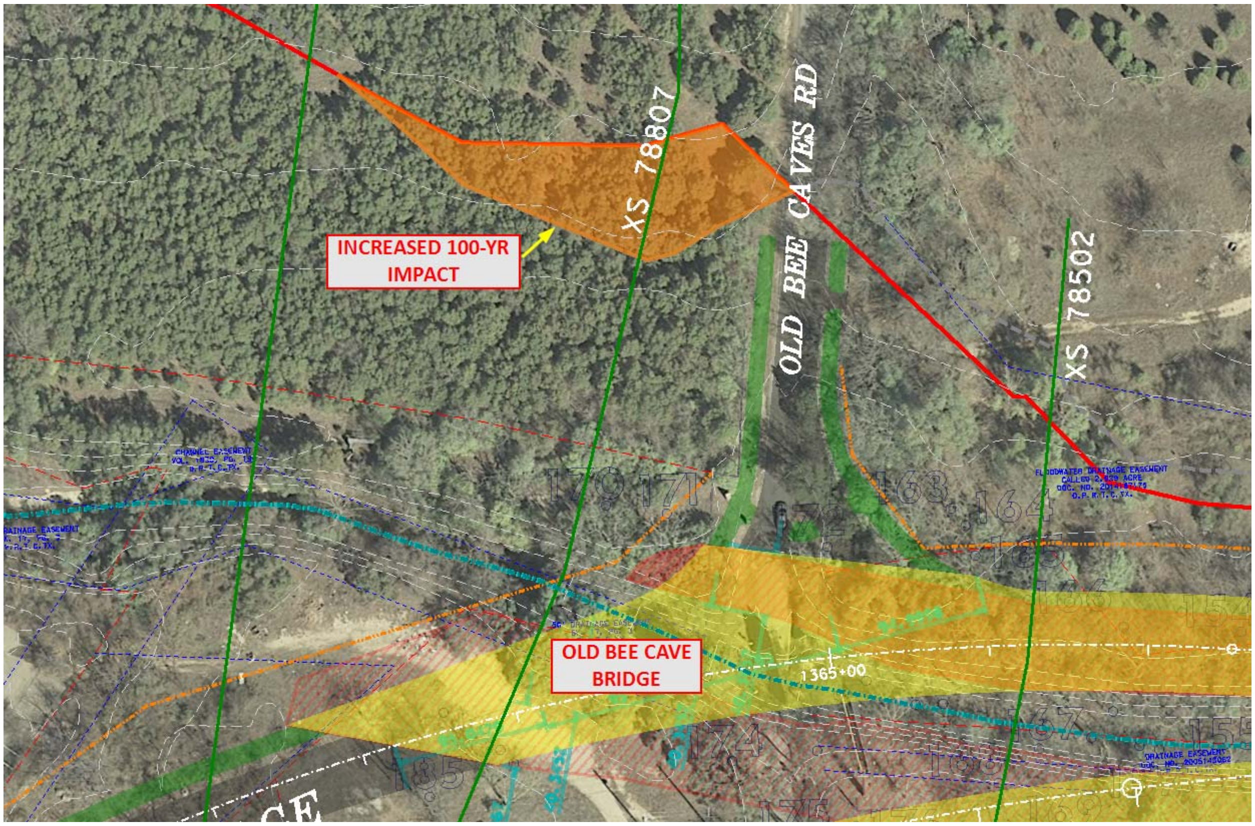

Figure 19 - Impact Areas due to Old Bee Cave Bridge Backwater Rise

State Highway 71 Bridge

Applying updated Atlas 14 flows to the proposed SH 71 crossing design results in a submerged upstream low chord. This chord inundation increases WSEL's upstream of the crossing. Due to this impact as well as tree preservation efforts along the banks of the creek, this bridge crossing modeling was revised. Originally, the bridge was modeled as one structure, expanded in width to accommodate the proposed roadway section. Modeling two parallel bridges allows for a more individualized design of abutments, piers, and overbank mitigation efforts for each bridge. Additional cross-sections and section modifications were made to both the existing and proposed models. The original proposed HWY 71 bridges were both defined as 100 feet long with four 25-foot spans. As outlined previously for other crossings, overbank benching has been lowered to 3 feet above the flowline and applied to both the EBFR and WBFR bridges. Despite this additional conveyance, the low chord of both the EBFR and WBFR remained inundated. Ultimately further profiles adjustments and lengthening of both bridges were applied to eliminate the impacts to the WSEL profile. The final WBFR bridge configuration has a total length of 120 feet with four 30-foot spans. The final EBFR bridge is modeled with a total length of 180-feet and four 45-foot spans. Because the EBFR bridge is skewed approximately 43 degrees, the effective lengths and span openings of both bridges are the same. In order to preserve the trees between the two proposed structures, overbank grading has been limited as much as possible in the right overbank area of each bridge.

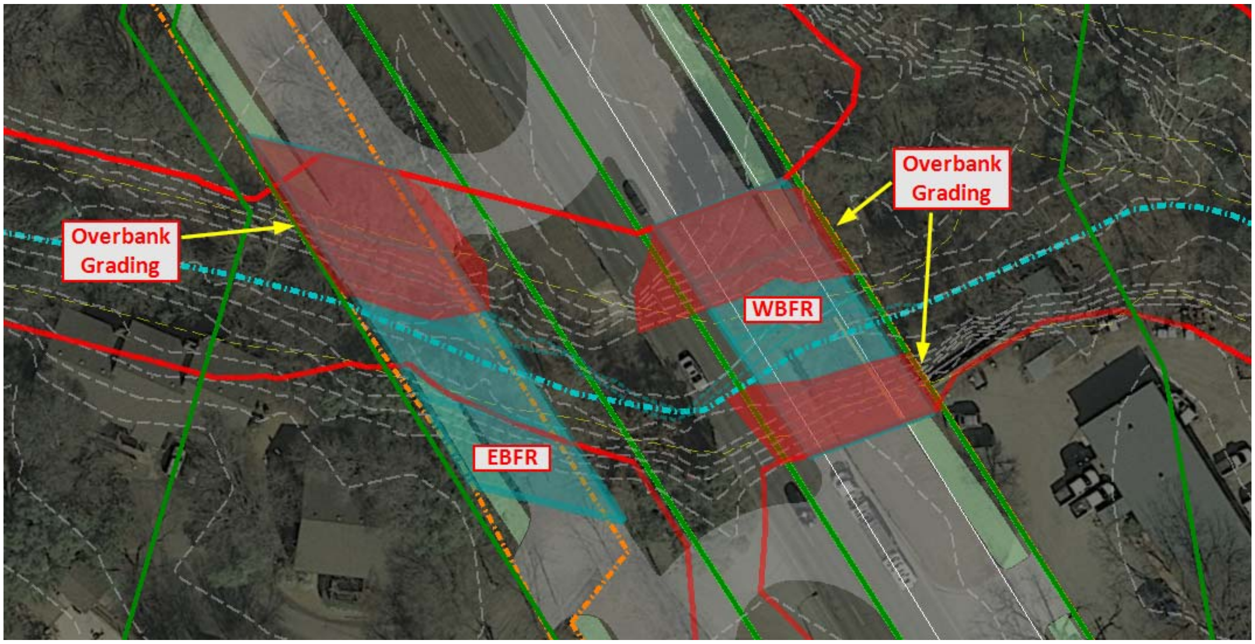

Figure 20 - Overbank Grading near HWY 71 Eastbound and Westbound Bridges

<u>Overbank Obstructions</u>

Minor modifications have been made to some overbank obstructions in this updated analysis to reflect the latest schematic profiles, the expansion of the west bound main lane flyover approximately 120 feet to the west, and the conversion of infill to elevated roadway on the west bank of Williamson Creek at the US 290 grouped crossings. These obstructions appear in the right and left overbanks of cross-sections in the area of the proposed roadway profile changes. Typically, the obstructions represent either the roadway fill associated with the new structure, or the roadway support structures that will be used to elevate main lanes in various locations along the corridor. In areas where roadway profiles were modified there were typically only minor adjustments to elevation or obstruction extent. However, there is one area where significant change from our previous design occurs in the cross-sections downstream of the grouped US290 crossings. The elevations of obstructions in the left overbank were raised significantly, to reflect roadway and bridge profile changes that were needed to maintain the required levels of service. As discussed previously in this memo, these expanded obstructions result in some minor WSEL impacts in the cross-sections immediately adjacent to the existing Joe Tanner low water crossing. These minor impacts were not ultimately mitigated but appear to have no adverse effect on existing roadways or structures. In addition to roadway structures several new obstructions were added representing the water quality ponds associated with the preliminary water quality design developed by K Friese & Associates. These pond structures were set at or above the 100-year event WSEL in order to prevent inundation during that event as required.

Main Lanes Berm

As outlined in the previous report, the proposed hydraulic design includes a berm intended to prevent inundation of a depressed section of roadway. The previous configuration extended along the WBML of US 290 for approximately 350 linear feet with a maximum height of 2.5 ft and was modeled in HEC-RAS as a levee. Due to changes in the water surface elevations in this section of the design the levee elevations were increased slightly to maintain the 100-year level of service. Table 8 shows the approximate modified levee profile.

Figure 21 - Berm Plan View

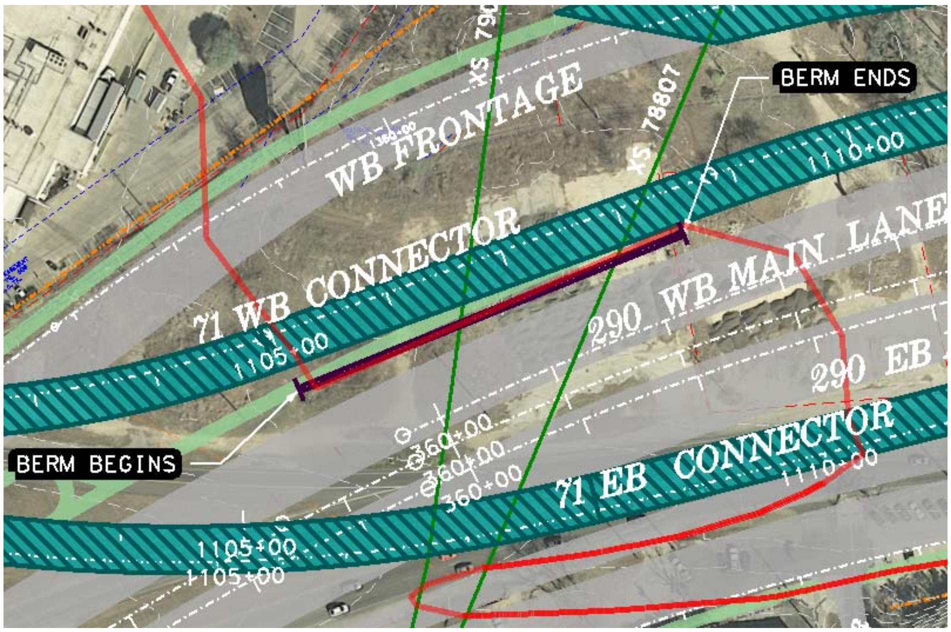

Table 8 - Levee Profile (US 290 WBML Alignment)

Levee Profile	
STA	ELEV
359+00	841
361+00	841
361+75	840
362+25	839

<u>Williamson to Barton Watershed Overflows</u>

It was determined that overflow from the Williamson Creek watershed into the Barton Creek watershed occurs above an elevation of 809, which corresponds to a low point in Patton Ranch Road. Updated Atlas 14 flows show that this overflow is currently occurring in as low as the 10-year flooding event, which has a peak WSELEV of 810.21. The recommended US 290 Crossing design has resulted in a WSELEV of 809.00 during the 10-year event. It is estimated that the proposed design will result in reduced overflow into Barton Creek watershed, which will occur during all events greater than a 10-year event.

8.2. HEC-RAS Modeling Results and Recommendations

The recommended design configuration necessary to meet conveyance criteria and to limit WSEL increases during the 100-year frequency event are outlined in the following tables.

Table 9 - US 290 EBFR Bridge Details

US 290 EB Frontage Road Bridge		
Specifications		
Beginning Station	3401+70	
End Station	3405+80	
Total Length	410'	
Pier Width	2'	
Span Configuration	70' - 90' - 90' - 90' - 70'	
Hydraulic Performance		
	DS	US
Low Chord Elev	812.63	813.19
25-yr WSEL	807.14	807.90
50-yr WSEL	807.87	808.71
100-yr WSEL	808.41	809.33

Table 10 - US 290 WBFR Bridge Details

US 290 WB Frontage Road Bridge		
Specifications		
Beginning Station	1398+41	
End Station	1401+81	
Total Length	340'	
Pier Width	2'	
Span Configuration	85' - 85' - 85' - 85'	
Hydraulic Performance		
	DS	US
Low Chord Elev	813.6	813.46
25-yr WSEL	809.60	810.47
50-yr WSEL	810.44	811.37
100-yr WSEL	811.10	812.06

Table 11 - EBML US 290 Bridge Details

US 290 EB Main Lane Bridge	
Specifications	
Beginning Station	--
End Station	405+00
Total Length	--
Pier Width	3'
Span Configuration	145' - 145' - 145' - 100'
Hydraulic Performance	
Low Chord Elev	818.08
25-yr WSEL	808.46
50-yr WSEL	809.38
100-yr WSEL	810.11

Table 12 - WBML US 290 Bridge Details

US 290 WB Main Lane Bridge	
Specifications	
Beginning Station	--
End Station	405+00
Total Length	--
Pier Width	3'
Span Configuration	145' - 145' - 145' - 100'
Hydraulic Performance	
Low Chord Elev	817.9
25-yr WSEL	809.59
50-yr WSEL	810.47
100-yr WSEL	811.18

Table 13 - William Cannon Bridge

William Cannon Bridge		
Specifications		
Beginning Station	22+23	
End Station	24+33	
Total Length	~210'	
Pier Width	2'	
Span Configuration	50' - 50' - 50' - 50'	
Hydraulic Performance		
	DS	US
Low Chord Elev	821.32	821.32
25-yr WSEL	817.15	818.73
50-yr WSEL	817.90	819.62
100-yr WSEL	818.32	820.23

Table 14 - Old Bee Cave Bride Details

Old Bee Cave Road Bridge		
Specifications		
Beginning Station	1362+21	
End Station	1366+51	
Total Length	~380'	
Pier Width	2'	
Span Configuration	95' - 95' - 95' - 95'	
Hydraulic Performance		
	DS	US
Low Chord Elev	835.29	840.83
25-yr WSEL	832.01	837.89
50-yr WSEL	833.11	839.06
100-yr WSEL	833.91	839.94

Table 15 - HWY 71 Westbound Frontage Road Bridge Details

HWY 71 WB Frontage Road Bridge		
Specifications		
Beginning Station	2076+11	
End Station	2077+31	
Total Length	120'	
Pier Width	2'	
Span Configuration	30' - 30' - 30' - 30'	
Hydraulic Performance		
	DS	US
Low Chord Elev	880.00	880.38
25-yr WSEL	877.06	877.26
50-yr WSEL	877.78	877.97
100-yr WSEL	878.45	878.78

Table 16 - HWY 71 Eastbound Frontage Road Bridge Details

HWY 71 EB Frontage Road Bridge		
Specifications		
Beginning Station	4074+51	
End Station	4076+31	
Total Length	~180	
Pier Width	2'	
Span Configuration	45' - 45' - 45' - 45'	
Hydraulic Performance		
	DS	US
Low Chord Elev	880.56	881.63
25-yr WSEL	877.59	877.89
50-yr WSEL	878.30	878.61
100-yr WSEL	879.29	879.74

The following Table 17 summarizes the HEC-RAS model results after incorporating the Atlas 14 discharge updates, and the mitigation measures described in the previous section.

Table 17 - HEC-RAS 100-year Results

HEC-RAS	EXISTING		PROPOSED		Change
RIVER	Q	WSELEV	Q	WSELEV	
STATION	(cfs)	(ft)	(cfs)	(ft)	
90177	2821	921.14	2821	921.14	0
89560	2821	915.15	2821	915.15	0
89063	4564	913.44	4564	913.44	0
88954	4564	913.4	4564	913.4	0
88894 COVERED BRIDGE D					
88832	4564	910.04	4564	910.04	0
88697	4564	909.42	4564	909.42	0
88042	4564	904.3	4564	904.3	0
87893	4564	902.95	4564	902.95	0
87863 PRIVATE DAM					
87831	4564	902.33	4564	902.33	0
87631	4564	900.62	4564	900.62	0
87444	4564	899.9	4564	899.9	0
87419 PRIVATE DAM					
87387	4564	899.74	4564	899.74	0
87324	4564	898.17	4564	898.17	0
87300 PRIVATE DRIVE					
87257	4564	898.83	4564	898.83	0
86718	4564	893.52	4564	893.52	0
86554	4564	893.63	4564	893.63	0
86512 SILVERMINE DAM					
86490	4564	893.28	4564	893.28	0
86455	4564	892.71	4564	892.67	-0.04
86417 SILVERMINE DR					
86383	4564	889.74	4564	889.73	-0.01
86254	4564	889.15	4564	889.14	-0.01
85611	4564	885.3	4564	885.01	-0.29
85045	4564	881.18	4564	880.07	-1.11
84982	4564	880.2	4564	879.92	-0.28
84851.4 STATE HWY 71					
84745	4564	878.85	4564	878.46	-0.39
84632	6229	878.23	5550	877.73	-0.5
83997	6229	875.26	5550	874.75	-0.51
83450	6229	872.55	5550	872.13	-0.42
83310	6229	872.44	5550	872.02	-0.42
83264 PRIVATE DRIVE					
83216	6229	868.45	5550	868.14	-0.31
83088	6229	867.47	5550	867.23	-0.24

HEC-RAS	EXISTING		PROPOSED		Change
RIVER	Q	WSELEV	Q	WSELEV	
STATION	(cfs)	(ft)	(cfs)	(ft)	
82500	6229	864.41	5550	864.17	-0.24
82259	6229	861.52	5550	861.28	-0.24
82227 PRIVATE DAM					
82202	6229	861.51	5550	861.27	-0.24
81951	6229	859.59	5550	859.37	-0.22
81923 PRIVATE DAM					
81903	6229	859.58	5550	859.36	-0.22
81746	6229	858.49	5550	858.06	-0.43
81703 PRIVATE DR					
81655	6229	858.72	5550	858.42	-0.3
81534	6229	857.02	5550	856.8	-0.22
80983	6229	853.3	5550	852.94	-0.36
80345	6229	849.32	5550	848.98	-0.34
79948	10053	846.38	9224	846.04	-0.34
79547	10053	843.58	9224	843.37	-0.21
79004	10053	840.35	9224	840.47	0.12
78807	10053	837.34	9224	840.09	2.75
78661 OLD BEE CAVE RD					
78502	10053	835.09	9224	834.12	-0.97
78181	10053	831.81	9224	831.48	-0.33
77960	10053	831.81	9224	830.25	-1.56
77525	10053	828.51	9224	827.49	-1.02
76871	10053	823.67	9224	821.65	-2.02
76786	10053	823.21	9224	820.8	-2.6
76587. WILLIAM CANNON D					
76285	10053	818.6	9224	818.21	-0.39
75854	10053	815.85	9224	815.65	-0.2
75491	10053	814.8	9224	814.77	-0.03
75171.9	13049	813.63	12516	813.47	-0.16
75017	13049	813.58	12516	812.97	-0.61
MAIN 290 CROSSINGS					
74437	13049	808.84	12516	808.07	-0.77
74163	13049	805.89	12516	806.53	0.64
74022	13049	805.46	12516	805.71	0.25
73988 JOE TANNER RD					
73960	13049	805.13	12516	805.24	0.11
73862	13049	804.36	12516	804.44	0.08
73413	13049	799.71	12516	799.63	-0.08

Despite testing various design iterations, two remaining modeled areas are showing impact due to the proposed Oak Hill Parkway improvements: the rise around the existing Joe Tanner low water crossing, and the backwater rise just upstream of the proposed Old Bee Cave Bridge. Due to their isolated occurrence, these areas of impact are relatively small. Approximate extents of some of these impact areas have been highlighted in Exhibits A-1 and A-2. While a strict no-rise condition could not be achieved in these locations, as stated previously, with detailed survey of the proposed detention sites, stream banks, and channels, these impacts may be resolved in final design. Additional study should be considered by the final design team, such as further analysis of the watershed spill-over occurring near Patton Ranch Road just north the US 290 grouped crossings, as the current study has conservatively assumed no loss of flow from this watershed overflow condition.