RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT A – BACKGROUND HYDROLOGIC INFORMATION

Report for the US Army Corps of Engineers, and the cities of Fargo, ND & Moorhead, MN

By: BARR ENGINEERING

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HYDRAULIC STRUCTURES

EXHIBIT A – BACKGROUND HYDROLOGIC INFORMATION

F-A1.0 INTRODUCTION

The background hydrology presented in this exhibit summarizes the existing hydrology (without project impacts) for the Red River of the North and the five major tributaries in North Dakota within the Project area (Wild Rice River, Sheyenne River, Maple River, Lower Rush River, and Rush River). The hydrology is summarized for the 10-, 50-, 100-, and 500-year events at the locations where the proposed Diversion Channel would intersect the rivers, unless otherwise noted. The additional hydrology information presented here includes water surface elevations, average channel velocities and measured velocity distributions. More details about background hydrology can be found in Appendix A.

The scenarios summarizing the “coincidental tributary hydrology” for the North Dakota tributaries takes into account the hydrology on the Red River of the North and can be thought of as the expected flow or water surface elevation in a tributary when a flood of a certain event occurs on the Red River of the North. All hydrology presented in this exhibit is Phase 4 hydrology unless noted otherwise, and a definition of this hydrology can be found in Appendix A.

The scenarios summarizing the “local tributary hydrology” for the North Dakota tributaries are based on when the peak flows and water surface elevations occur on the Maple River. The local tributary hydrology has been analyzed for both existing and with-project conditions. Although the analysis of the local peak tributary flows was only extended as far as Pearly downstream of the Project for this phase.

F-A2.0 BACKGROUND HYDROLOGY OF THE RED RIVER OF THE NORTH

F-A2.1 BACKGROUND HYDROLOGY OF THE RED RIVER OF THE NORTH AT THE FCP CONTROL STRUCTURE

The following information regarding the FCP was previously presented as part of Appendix F Exhibit A in the Phase 3 report submitted on August 6th, 2010, and is included here for completeness. All of the following FCP information is based on Phase 3 hydrology and the HEC-RAS steady flow model developed during Phase 3.

Existing flows, water surface elevations, flow areas, and average velocities for the Red River of the North at the location of the proposed FCP Control Structure (Station 467.4) for the 2-, 5-, 10-, 20-, 50-, 100-, 200-, and 500-year events are presented for the Year 0, 25 and 50 hydrology in Tables FA-1, FA-2, and FA-3, respectively. Existing water surface elevations in the Red River of the North at the FCP Control Structure (Station 467.4) for the flood events listed above, plotted against the cross section data, are presented for the Year 0, 25 and 50 hydrology in Figures FA-1, FA-2, and FA-3.

Flows and water surface elevations decrease from Year 0 to Year 25 to Year 50 hydrology. The decrease in flows from Year 0 to Year 50 hydrology at the FCP Control Structure is more significant for the more frequent events (37 percent for 2-year events) than for less frequent events (12 percent for 500-year events).

Table F-A1 Background Existing Year 0 Hydrology of the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	Median*	360	880.02	1,041	0.34
Phase3.1	2	5,600	892.77	3,868	1.84
Phase3.1	5	12,150	900.72	9,031	2.44
Phase3.1	10	17,000	904.66	14,955	2.78
Phase3.1	20	22,000	907.71	24,913	2.91
Phase3.1	50	29,300	910.68	45,927	2.84
Phase3.1	100	34,700	911.10	69,644	2.72
Phase3.1	200	46,200	912.65	89,961	2.85
Phase3.1	500	61,700	914.42	113,230	2.97

*Note: Value extrapolated from existing data. Estimate of median flow is available for Year 0 Hydrology only.

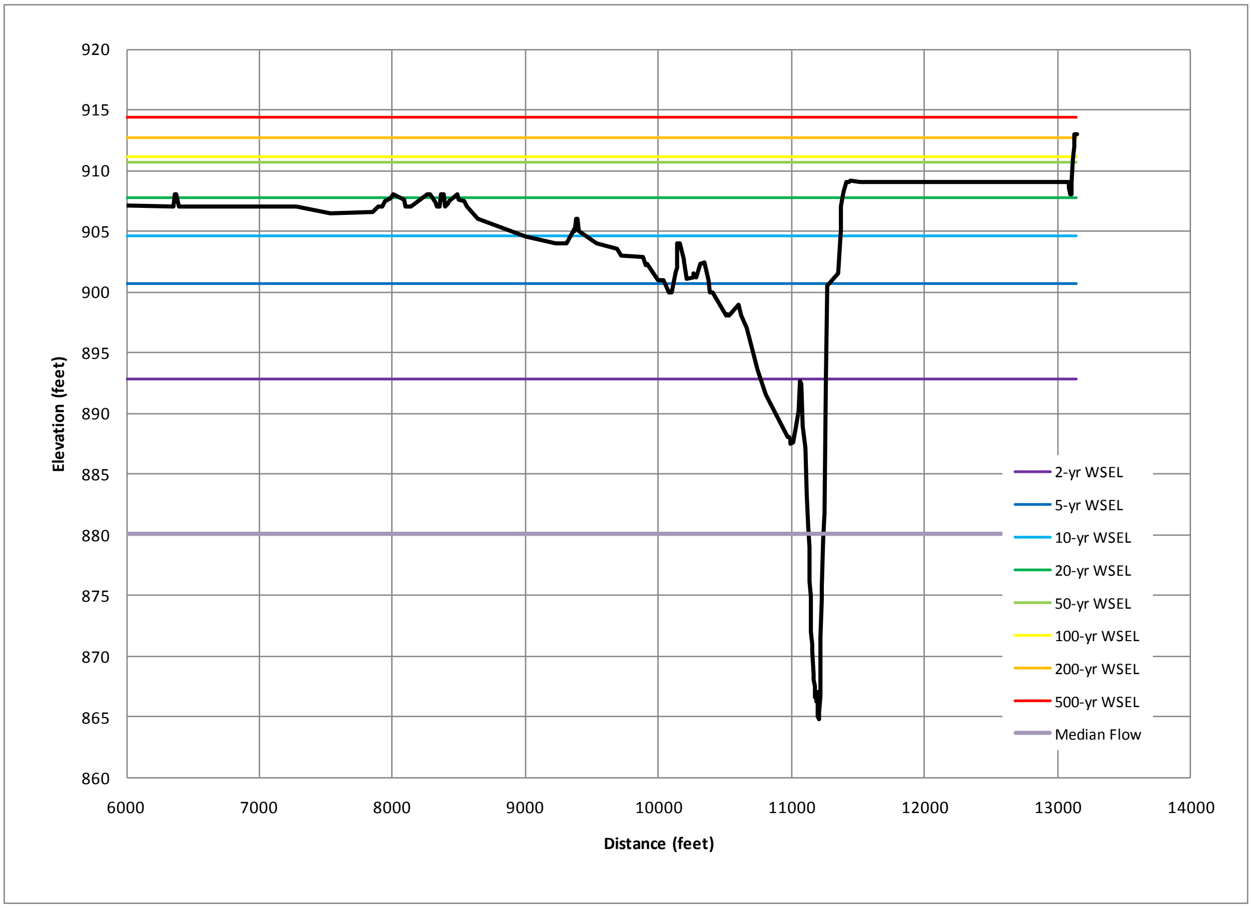


Figure F-A1 Background Existing Year 0 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

Table F-A2 Background Existing Year 25 Hydrology of the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	2	4,352	890.64	2,928	1.68
Phase3.1	5	10,608	899.22	7,704	2.33
Phase3.1	10	15,394	903.48	12,683	2.64
Phase3.1	20	20,345	906.81	20,712	2.91
Phase3.1	50	27,441	910.08	41,191	2.86
Phase3.1	100	32,921	911.03	68,760	2.61
Phase3.1	200	42,242	912.16	83,548	2.80
Phase3.1	500	57,641	914.00	107,699	2.93

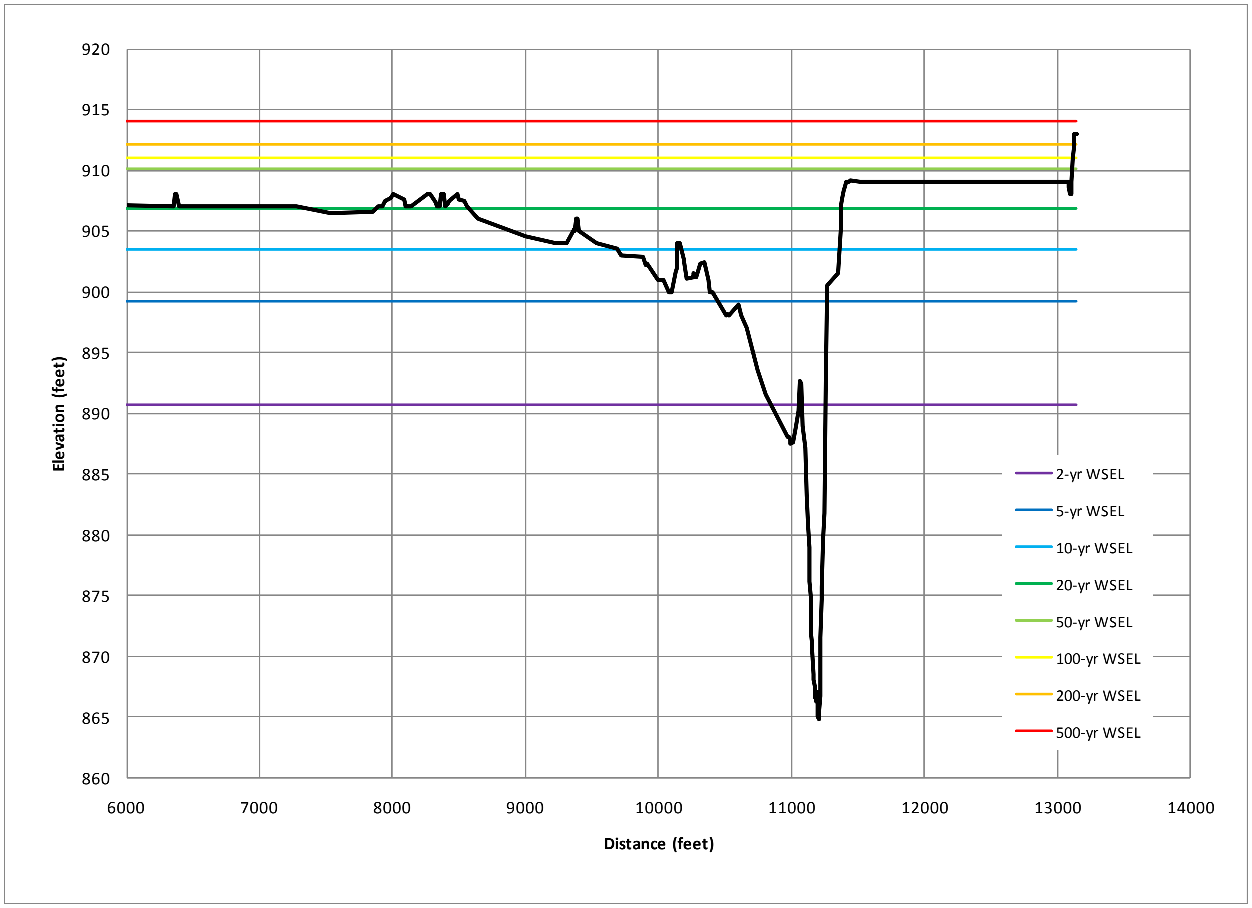


Figure F-A2 Background Existing Year 25 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

Table F-A3 Background Existing Year 50 Hydrology of the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	2	3,506	888.95	2,372	1.53
Phase3.1	5	9,161	897.66	6,572	2.18
Phase3.1	10	13,965	902.36	11,003	2.54
Phase3.1	20	18,855	905.90	18,066	2.82
Phase3.1	50	25,764	909.46	36,310	2.90
Phase3.1	100	31,304	910.80	46,858	2.99
Phase3.1	200	38,787	911.67	77,099	2.78
Phase3.1	500	54,034	913.60	102,503	2.90

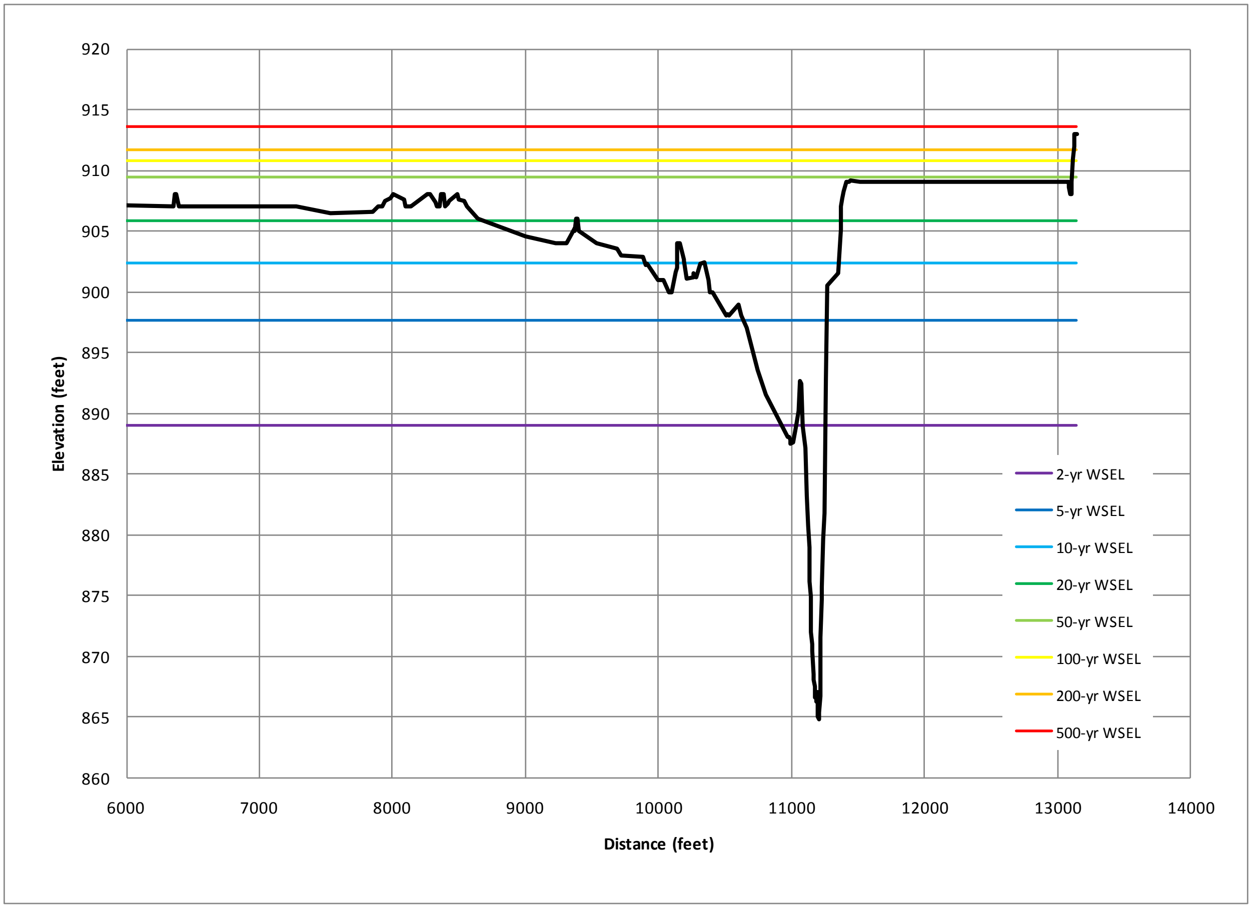


Figure F-A3 Background Existing Year 50 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

F-A2.2 BACKGROUND HYDROLOGY OF THE RED RIVER OF THE NORTH AT THE FCP OUTLET

The following information regarding the FCP was previously presented as part of Appendix F Exhibit A in the Phase 3 report submitted on August 6th, 2010, and is included here for completeness. All of the following FCP information is based on Phase 3 hydrology and the HEC-RAS steady flow model developed during Phase 3.

Existing flows, water surface elevations, flow areas, and average velocities for the Red River of the North at the proposed location of the FCP Diversion Channel Outlet (Station 299) for the 2-, 5-, 10-, 20-, 50-, 100-, 200-, and 500-year events are presented for the Year 0, 25 and 50 hydrology in Tables FA-4, FA-5, and FA-6, respectively. Existing water surface elevations in the Red River of the North at the proposed FCP Diversion Channel Outlet (Station 299) for the flood events listed above, plotted against the cross section data, are presented for the Year 0, 25 and 50 hydrology in Figures FA-4, FA-5, and FA-6.

Table F-A4 Background Existing Year 0 Hydrology of the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	median	876	858.07	657	1.32
Phase3.1	2	8,328	874.63	4,404	2.56
Phase3.1	5	16,039	882.84	10,452	3.26
Phase3.1	10	22,069	885.93	29,360	3.36
Phase3.1	20	28,491	887.53	44,094	3.45
Phase3.1	50	37,826	889.13	58,887	3.66
Phase3.1	100	45,160	889.67	63,892	4.07
Phase3.1	200	56,895	890.05	149,180	3.03
Phase3.1	500	72,930	890.98	171,235	3.29

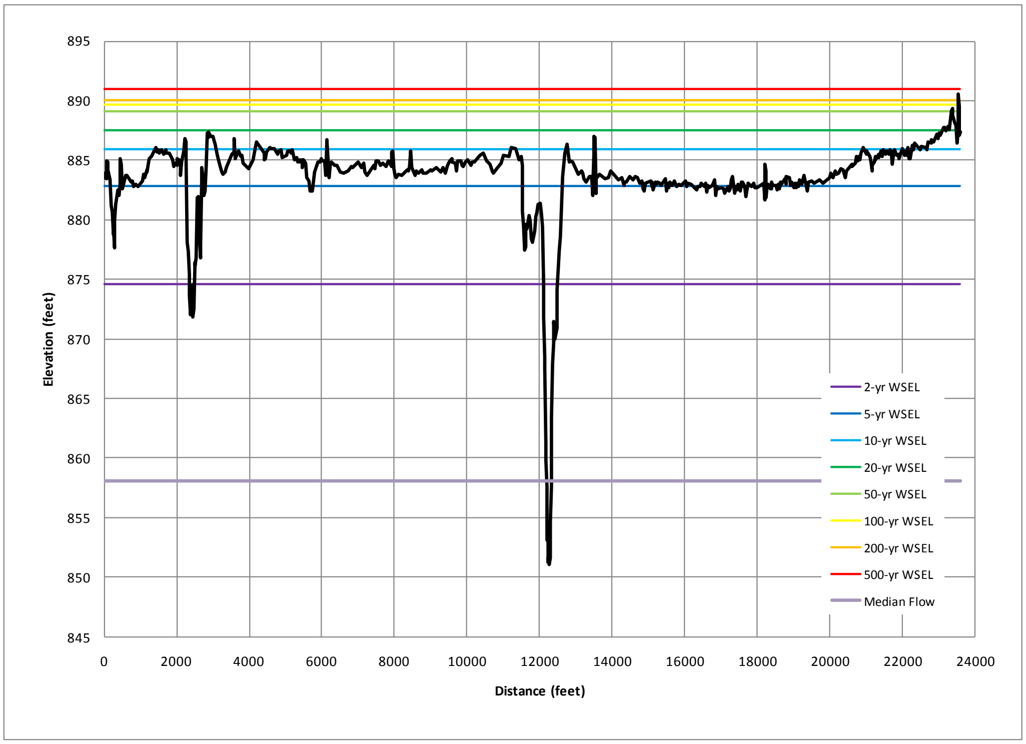


Figure F-A4 Background Existing Year 0 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Table F-A5 Background Existing Year 25 Hydrology of the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	2	7,124	872.92	3,752	2.42
Phase3.1	5	14,681	881.76	8,959	3.16
Phase3.1	10	20,667	885.43	24,889	3.35
Phase3.1	20	27,055	887.20	41,079	3.43
Phase3.1	50	36,261	888.90	56,774	3.62
Phase3.1	100	43,687	889.62	63,410	3.96
Phase3.1	200	54,114	890.07	149,695	2.87
Phase3.1	500	70,317	890.83	167,563	3.26

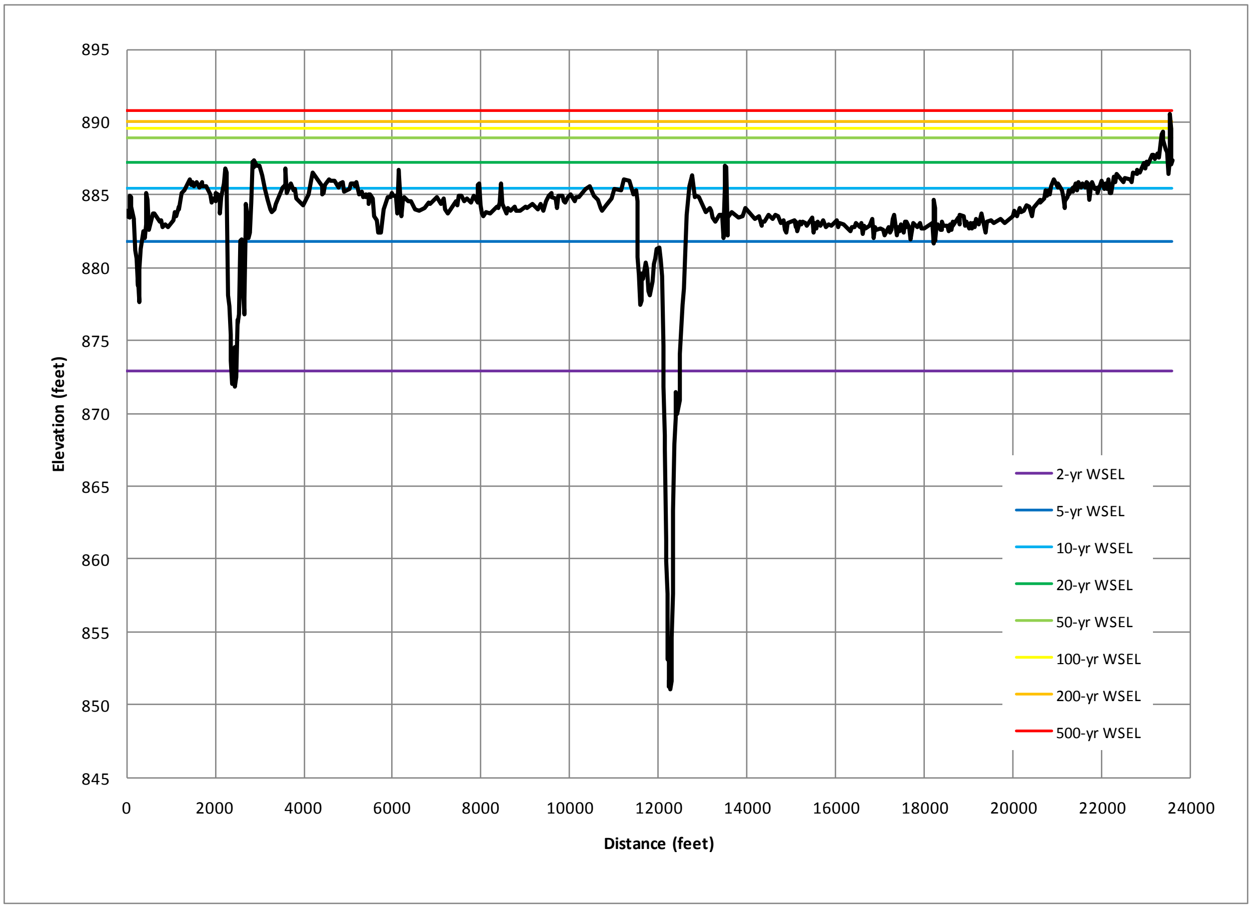


Figure F-A5 Background Existing Year 25 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Table F-A6 Background Existing Year 50 Hydrology of the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	2	6,247	871.54	3,253	2.31
Phase3.1	5	13,447	880.72	7,896	3.03
Phase3.1	10	19,446	884.94	20,728	3.33
Phase3.1	20	25,786	886.92	38,506	3.41
Phase3.1	50	34,892	888.68	54,729	3.59
Phase3.1	100	42,390	889.78	64,865	3.76
Phase3.1	200	51,664	890.02	148,529	2.76
Phase3.1	500	67,999	890.68	164,052	3.23

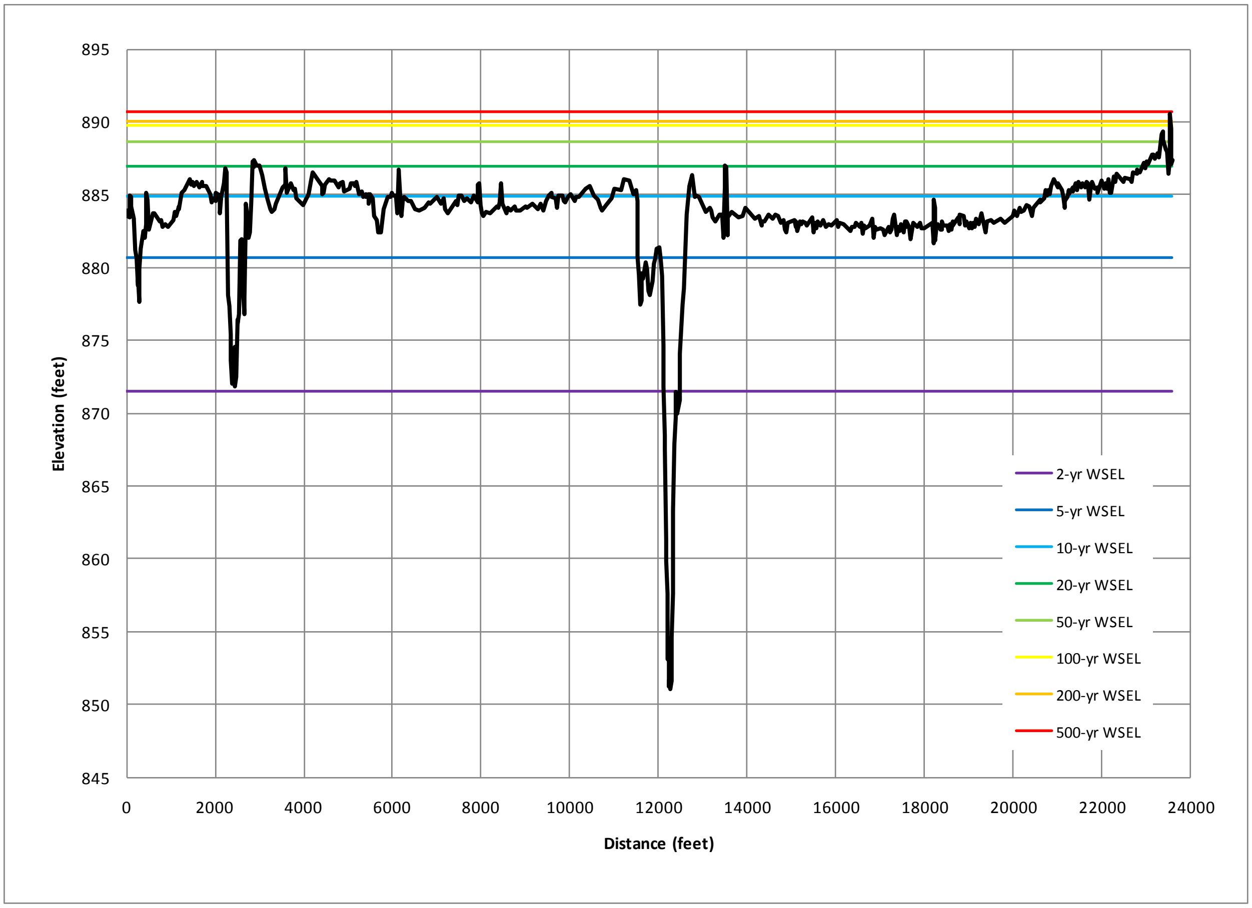


Figure F-A6 Background Existing Year 50 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Flows and water surface elevations decrease from Year 0 to Year 25 to Year 50 hydrology. The decrease in flows from Year 0 to Year 50 hydrology at the FCP Diversion Channel Outlet is more significant for the more frequent events (25 percent for 2-year events) than for less frequent events (seven percent for 500-year events).

F-A2.3 BACKGROUND HYDROLOGY OF THE RED RIVER OF THE NORTH AT THE LPP CONTROL STRUCTURE

Existing flows, water surface elevations, and average velocities for the Red River of the North at the approximate location of the LPP Control Structure (Phase 3 Station 478.8, Phase 4 Station 2531315) for the 10-, 50-, 100-, and 500-year events are presented in Table FA-7. The LPP Control Structure will be constructed in a proposed channel off of the existing river channel. Phase 3 Station 478.8 and Phase 4 Station 2531315 are at approximately the same location in the existing river channel as the LPP Control Structure will be in the proposed channel. Existing water surface elevations in the Red River of the North at the approximate location of the LPP Control Structure (Phase 3 Station 478.8, Phase 4 Station 2531315) for the flood events listed above, plotted against the cross section data, are presented in Figure FA-7.

Table F-A7 Background Existing Hydrology of the Red River of the North at the Approximate Location of the LPP Control Structure (Phase 3 Station 478.8, Phase 4 Station 2531315)

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
	Phase 3			Phase 4		
2	4,000	896.91	1.57	-	-	-
5	7,000	904.9	1.77	-	-	-
10	10,500	908.42	2.2	10,271	908.06	1.12
50	21,000	914.1	3.05	18,207	913.76	0.89
100	25,000	914.94	3.33	21,458	914.65	0.90
500	32,000	917.7	2.46	28,623	915.94	0.98

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.

Note: Phase 3 values based on steady flow analysis

Note: Phase 4 values based on unsteady flow analysis

*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

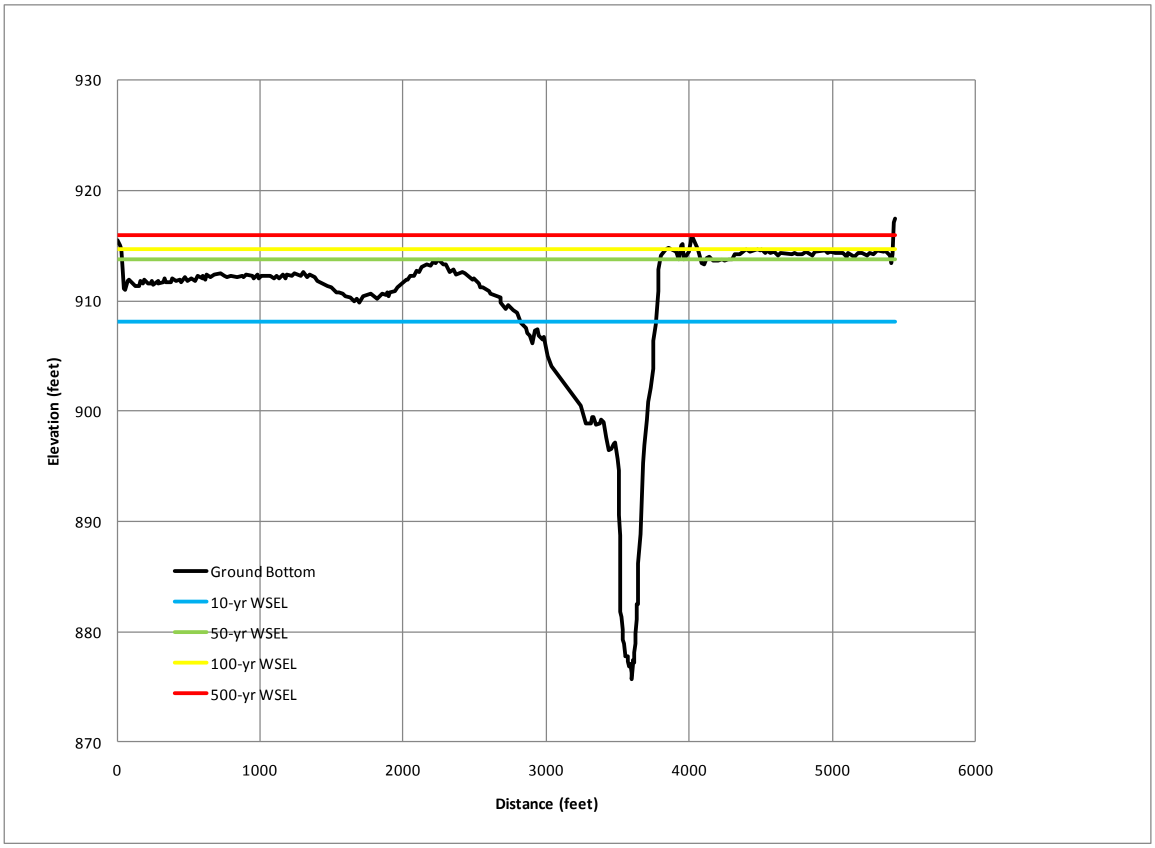


Figure F-A7 Background Existing Water Surface Elevations in the Red River of the North at the Approximate Location of the LPP Control Structure (Station 2531315)

Flows and water surface elevations decrease from Phase 3 to Phase 4 due to changes from the Phase 3 hydrology discussed in Appendix A, and a change in methodology for computing water surface elevations for Phase 4. Phase 4 water surface elevations were calculated using a HEC-RAS unsteady flow model that accounts for flow through the floodplain and off channel storage areas, compared to Phase 3 that used a combination of a HEC-RAS steady flow model for the Red River and rating curves for the tributaries. The Phase 4 model includes storage areas and storage area connections to provide an accurate depiction of how flows enter and exit the main channel and accounts for conveyance of flow through the floodplain.

F-A2.4 BACKGROUND HYDROLOGY OF THE RED RIVER OF THE NORTH AT THE LPP OUTLET

Existing flows, water surface elevations, and average velocities for the Red River of the North at the proposed location of the LPP Diversion Channel Outlet (Phase 3 Station 292, Phase 4 Station 2208555) for the 10-, 50-, 100-, and 500-year events are presented in Table FA-8. Existing water surface elevations in the Red River of the North at the proposed LPP Diversion Channel Outlet (Phase 3 Station 292, Phase 4 Station 2208555) for the flood events list above, plotted against the cross section data, are presented in Figure FA-8.

Table F-A8 Background Existing Hydrology of the Red River of the North at the Approximate Location of the LPP Diversion Channel Outlet (Phase 3 Station 292, Phase 4 Station 2208555)

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
	Phase 3			Phase 4		
2	8,328	870.05	2.57	-	-	-
5	16,039	878.37	3.35	-	-	-
10	22,069	881.72	3.3	23,069	881.12	0.87
50	37,826	884.38	3.75	40,743	883.33	0.54
100	45,160	885.20	3.18	47,522	883.92	0.56
500	72,930	886.86	3.92	62,085	885.03	0.61

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

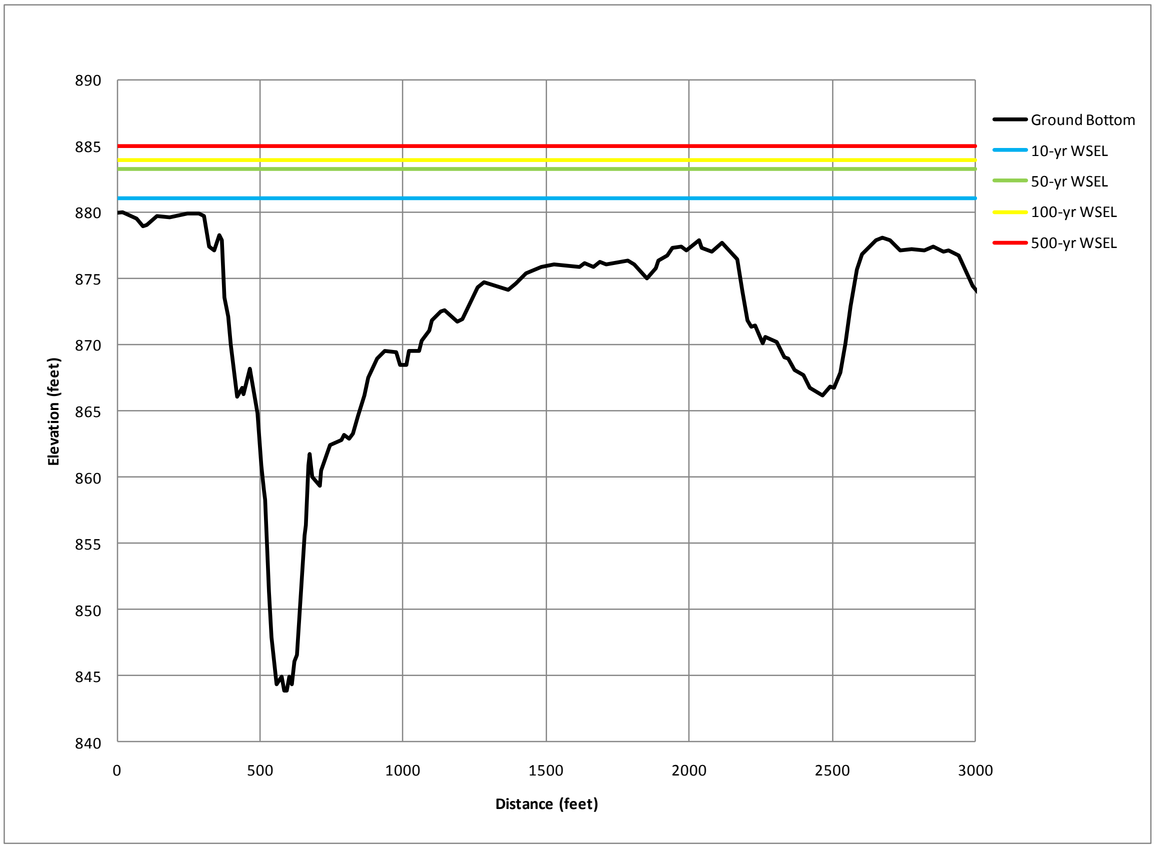


Figure F-A8 Background Existing Water Surface Elevations in the Red River of the North at the Approximate Location of the LPP Diversion Channel Outlet (Station 2208555)

Flows and water surface elevations in Phase 3 and Phase 4 can be different. Phase 4 water surface elevations were calculated using a HEC-RAS unsteady flow model that accounts for flow through the floodplain and off channel storage areas, compared to Phase 3 that used a combination of a HEC-RAS steady flow model for the Red River and rating curves for the tributaries. The Phase 4 model includes storage areas and storage area connections to provide an accurate depiction of how flows enter and exit the main channel and accounts for conveyance of flow through the floodplain.

F-A2.5 MEASURED VELOCITIES IN THE RED RIVER OF THE NORTH

Actual velocities in the Red River of the North have been measured at two US Geological Survey (USGS) gaging stations: Fargo (USGS Gage 05054000) and Hickson (USGS Gage 05051522). Figure F-A9 presents maximum channel velocities in the Red River of the North at Fargo plotted against flow. Figure F-A10 presents velocity distributions in the Red River of the North at Fargo measured on July 7, 2007 and March 28, 2009. Figures F-A11 and F-A12 present average channel velocities plotted against flow in the Red River of the North at Fargo and Hickson, respectively.

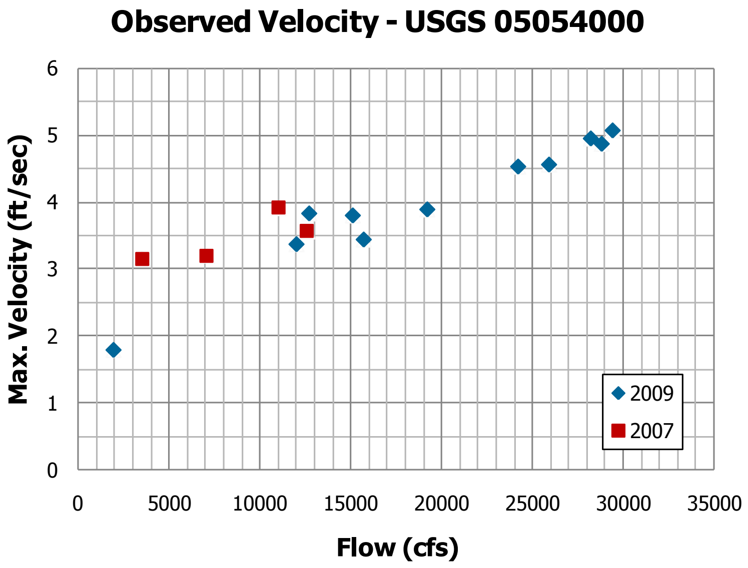


Figure F-A9 Maximum channel velocities in the Red River measured at USGS gage 05054000 (Fargo, ND)

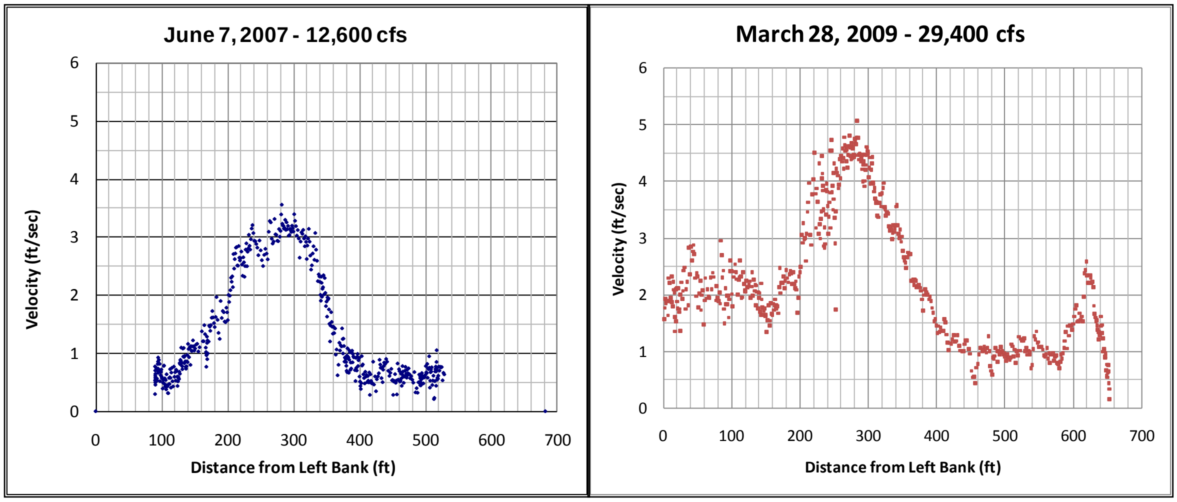


Figure F-A10 Velocity distributions measured in the Red River at USGS gage 05054000 (Fargo, ND)

12,600 cfs corresponds to approximately a 5-year event based on Phase 3 hydrology (low flow events were not updated during Phase 4), and 29,400 cfs between a 10-year and 50-year event based on Phase 4 hydrology.

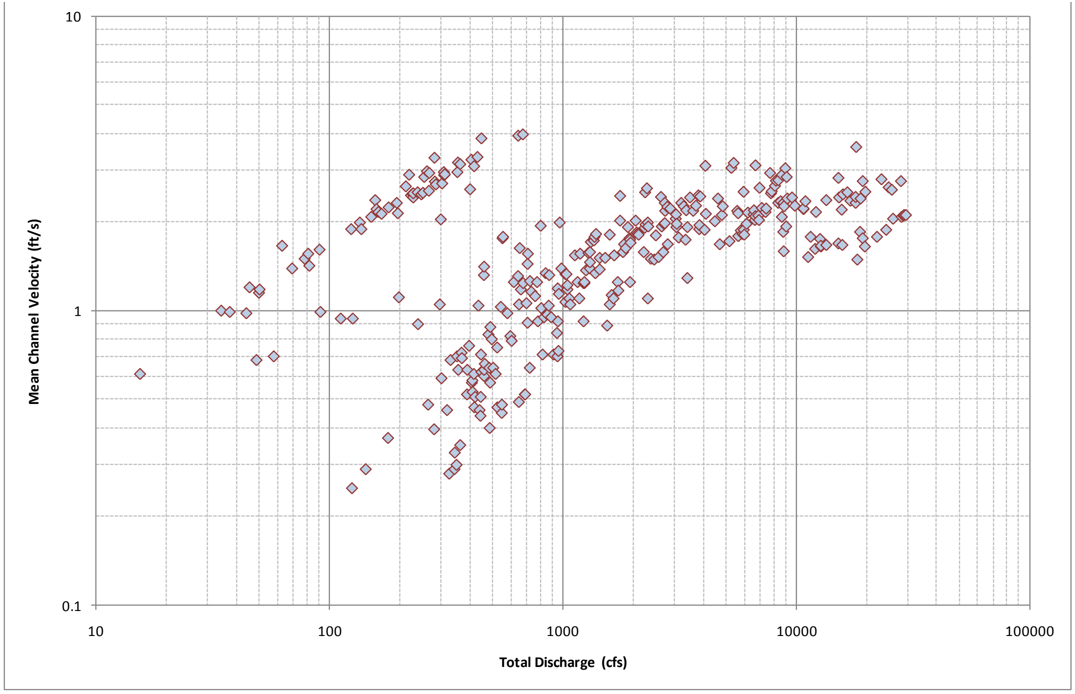


Figure F-A11 Average channel velocities versus total discharge in the Red River measured at USGS gage 05054000 (Fargo, ND)

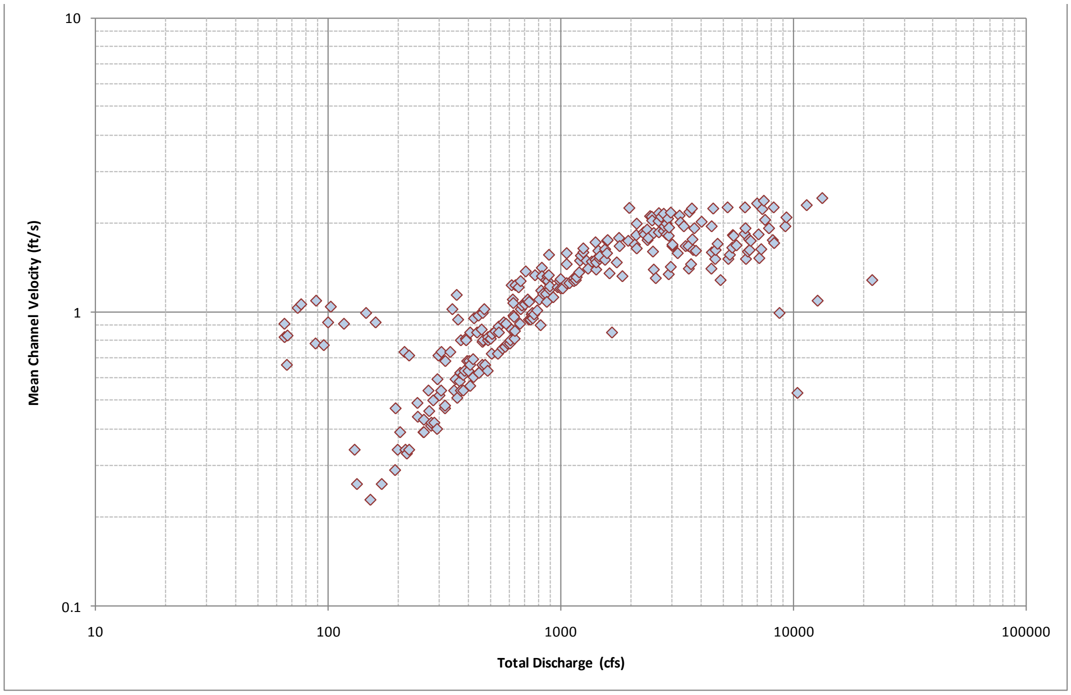


Figure F-A12 Average channel velocities versus total discharge in the Red River measured at USGS gage 05051522 (Hickson, ND)

F-A3.0 BACKGROUND HYDROLOGY OF THE WILD RICE RIVER

Flows, water surface elevations, and average velocities for the Wild Rice River for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-9. Water surface elevations in the Wild Rice River for the flood events listed above, plotted against the cross section data, and are presented for local tributary hydrology in Figure FA-13.

Table F-A9 Background Existing Local Hydrology of the Wild Rice River at the Approximate Location of the LPP Diversion Channel

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3			**Phase 4**		
2	1,388	902.22	1.16	-	-	-
5	3,996	908.94	1.36	-	-	-
10	6,593	912.69	1.47	5,444	910.59	1.76
50	14,639	916.97	0.62	8,824	915.15	0.58
100	19,016	917.92	0.66	8,688	915.84	0.45
500	31,107	920.22	0.76	9,565	916.99	0.35

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

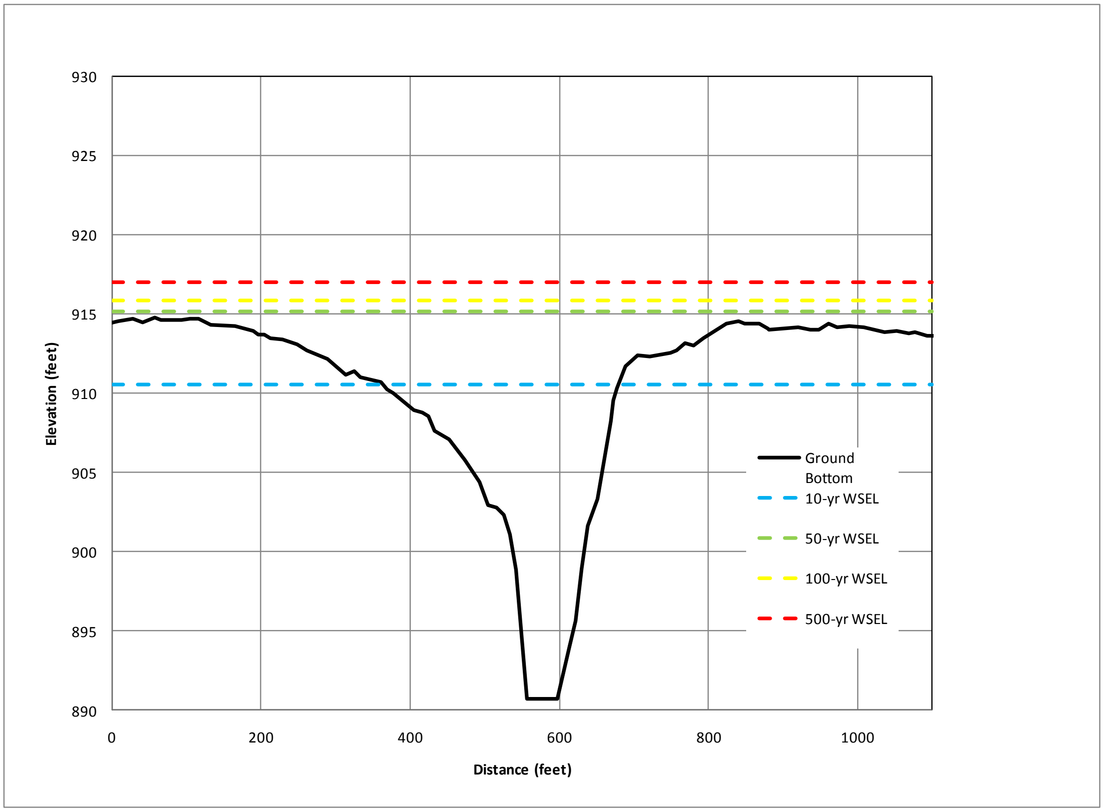


Figure F-A13 Background Existing Local Hydrology Water Surface Elevations in the Wild Rice River at the Approximate Location of the LPP Diversion Channel (Station 69855)

Flows, water surface elevations, flow areas, and average velocities for the Wild Rice River for the 10-, 50-, 100-, and 500-year events are presented coincidental tributary hydrology in Table FA-10. Water surface elevations in the Wild Rice River for the flood events listed above, plotted against the cross section data, and are presented coincidental tributary hydrology in Figure FA-14.

Table F-A10 Background Existing Coincidental Hydrology of the Wild Rice River at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3			Phase 4		
2	1,419	902.31	1.16	-	-	-
5	3,021	906.76	1.35	-	-	-
10	6,185	912.18	1.47	6,393	912.13	1.60
50	11,655	916.10	0.61	8,641	915.82	0.45
100	13,780	916.74	0.61	8,503	916.31	0.39
500	18,342	917.77	0.66	8,767	916.88	0.35

*Event recurrence interval refers to the Red River flow record.

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.

Note: Phase 3 values based on steady flow analysis

Note: Phase 4 values based on unsteady flow analysis

**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

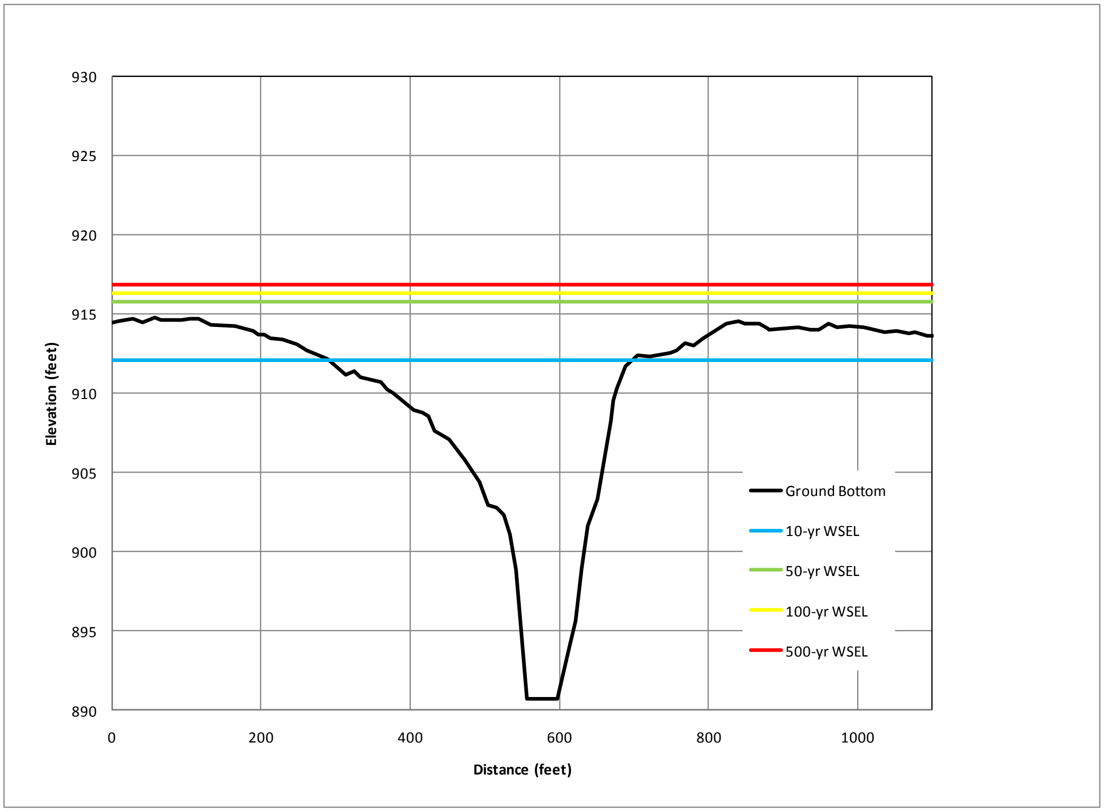


Figure F-A14 Background Existing Coincidental Hydrology Water Surface Elevations in the Wild Rice River at the Approximate Location of the LPP Diversion Channel (Station 69855)

Flows and water surface elevations in Phase 3 and Phase 4 can be different. Phase 4 water surface elevations were calculated using a HEC-RAS unsteady flow model that accounts for flow through the floodplain and off channel storage areas, compared to Phase 3 that used a combination of a HEC-RAS steady flow model for the Red River and rating curves for the tributaries. The Phase 4 model includes storage areas and storage area connections to provide an accurate depiction of how flows enter and exit the main channel and accounts for conveyance of flow through the floodplain.

Actual velocities in the Wild Rice River have been measured at the USGS Gage 05053000 at Abercrombie (approximately 10 miles upstream of the Diversion Channel). Figure F-A15 present average channel velocities plotted against flow in the Wild Rice River. Figure F-A16 presents velocity distributions in the Wild Rice River measured on March 24, 2009 and May 27, 2009.

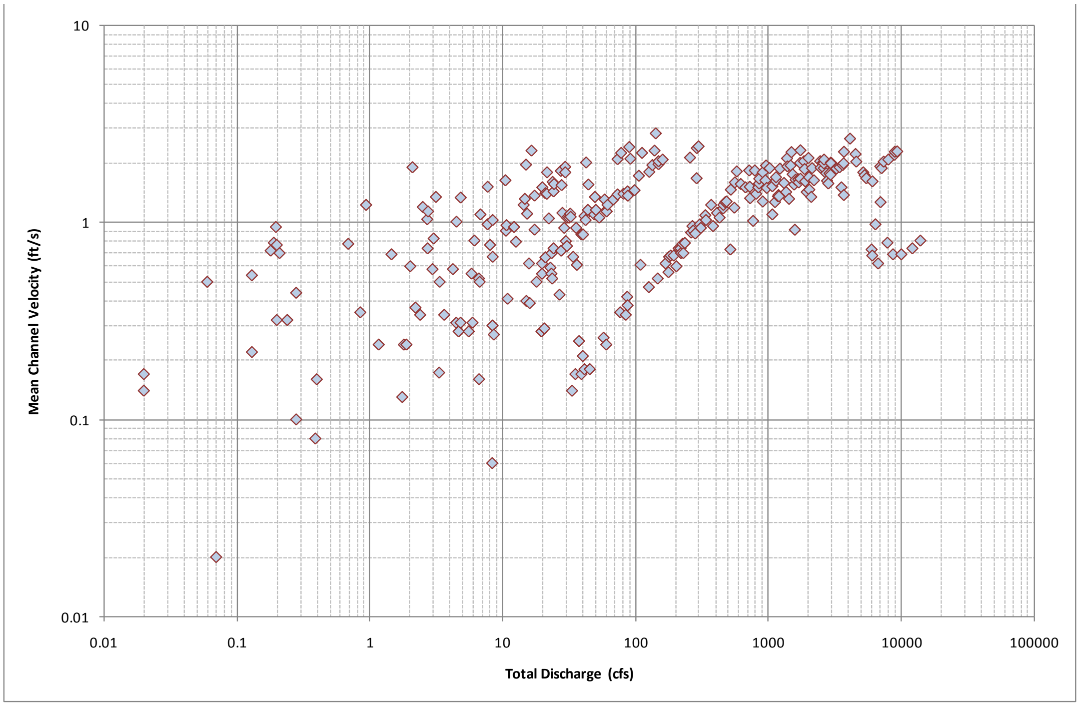
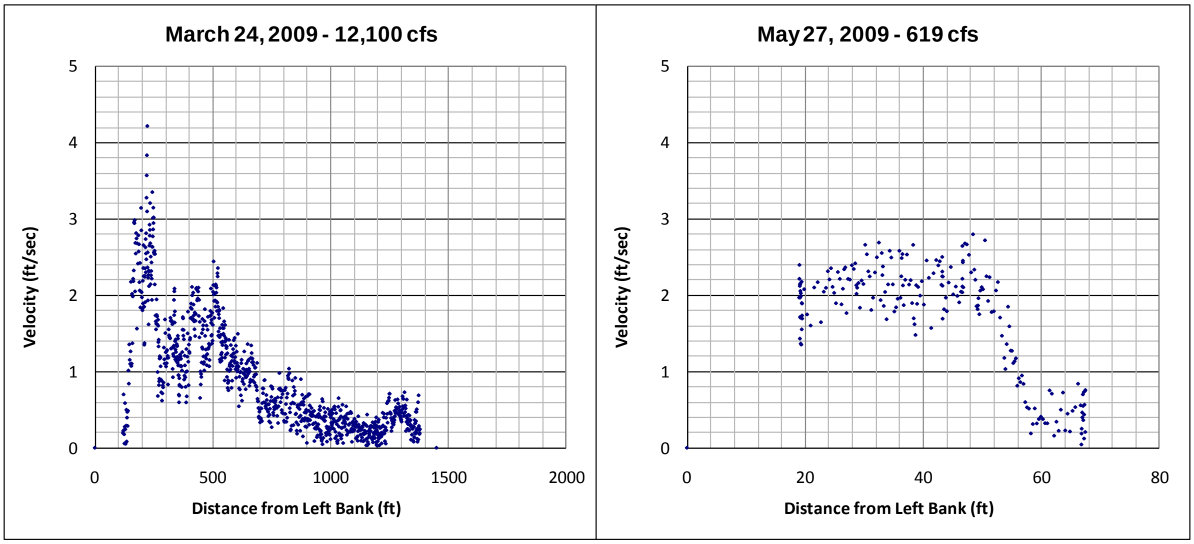


Figure F-A15 Average channel velocities versus total discharge in the Wild Rice River measured at USGS gage 05053000 (Abercrombie, ND)


March 24, 2009 - 12,100 cfs

May 27, 2009 - 619 cfs

Figure F-A16 Velocity distributions measured in the Wild Rice River at USGS gage 05053000 (Abercrombie, ND)

12,100 cfs corresponds to approximately a 50-year local event (For Phase 4, Table F-A9 only includes conveyed by the main channel of the Wild Rice River and does not include flows in the overbanks.) and 619 cfs corresponds to less than a 2-year local event in the Wild Rice River based on Phase 3 hydrology (hydrology for low flow events was not updated as part of the Phase 4 analysis).

F-A4.0 BACKGROUND HYDROLOGY OF THE SHEYENNE RIVER

Flows, water surface elevations, and average velocities for the Sheyenne River for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-11. Water surface elevations in the Sheyenne River for the flood events listed above, plotted against the cross section data, and are presented for the local hydrology in Figure FA-17. During Phase 3 it was assumed that the Sheyenne River had a maximum capacity of approximately 4,600 cfs and flows beyond this capacity were assumed to breakout to the Drain 14 and 21C systems. For Phase 4 the HEC-RAS unsteady flow model accounts for these breakout flows to the Drain 14 and 21C systems within the model, and as a result the conveyance capacity of the Sheyenne River is approximately 4,000 cfs.

Table F-A11 Background Existing Local Hydrology of the Sheyenne River at the Approximate Location of the LPP Diversion Channel

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
	Phase 3			Phase 4		
2	1,200	912.71	1.85	-	-	-
5	2,400	916.02	2.47	-	-	-
10	3,400	917.69	2.93	3,843	918.78	2.71
50	4,500	919.35	3.07	4,021	919.08	2.77
100	4,600	919.50	3.06	4,040	919.11	2.77
500	4,600	919.50	3.06	4,059	919.14	2.78

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

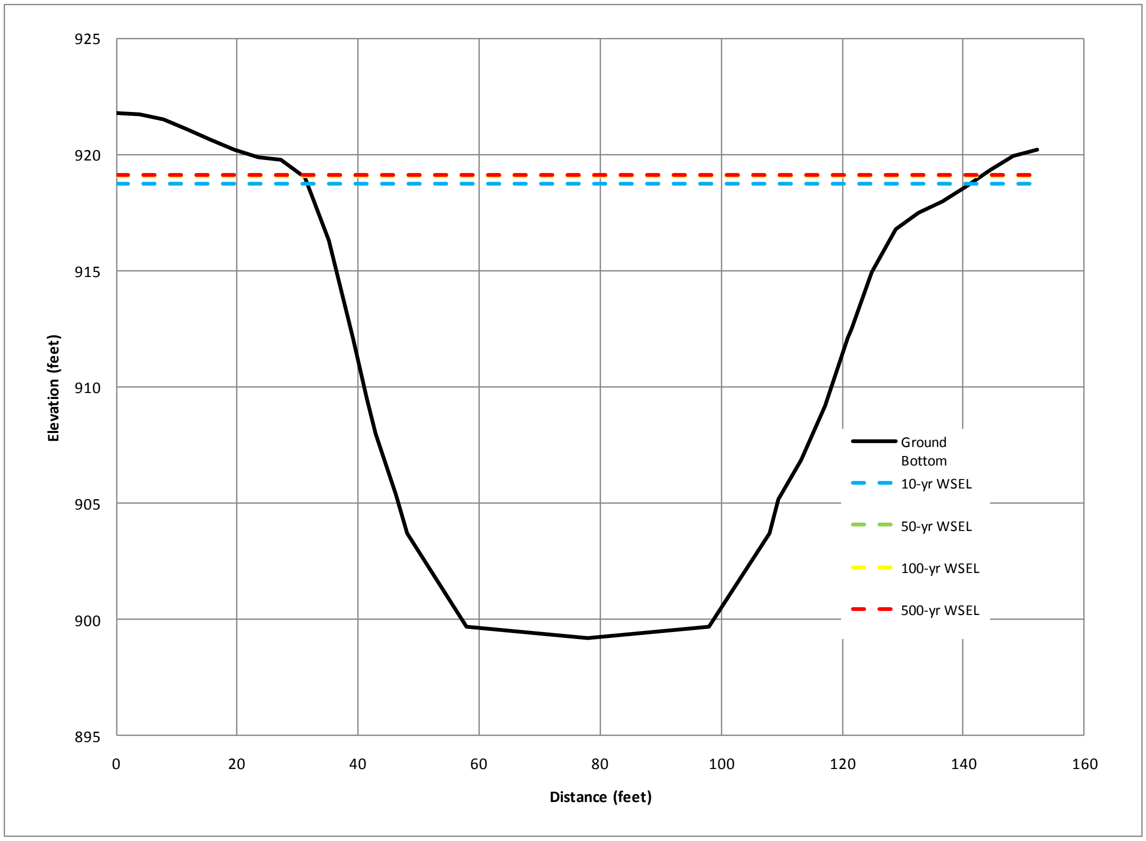


Figure F-A17 Background Existing Local Hydrology Water Surface Elevations in the Sheyenne River at the Approximate Location of the LPP Diversion Channel

Flows, water surface elevations, and average velocities for the Sheyenne River for the 10-, 50-, 100-, and 500-year events are presented for coincidental tributary hydrology in Table FA-12. Water surface elevations in the Sheyenne River for the flood events listed above, plotted against the cross section data, and are presented for coincidental tributary hydrology in Figure FA-18.

Table F-A12 Background Existing Coincidental Hydrology of the Sheyenne River at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3			**Phase 4**		
2	1,325	913.27	1.89	-	-	-
5	1,935	915.15	2.19	-	-	-
10	2,565	916.31	2.56	1,787	914.50	1.77
50	4,510	919.37	3.07	3,627	918.42	2.62
100	4,600	919.50	3.06	3,921	918.96	2.72
500	4,600	919.50	3.06	4,011	919.10	2.75

* Event recurrence interval refers to the Red River flow record.

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.

Note: Phase 3 values based on steady flow analysis

Note: Phase 4 values based on unsteady flow analysis

**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

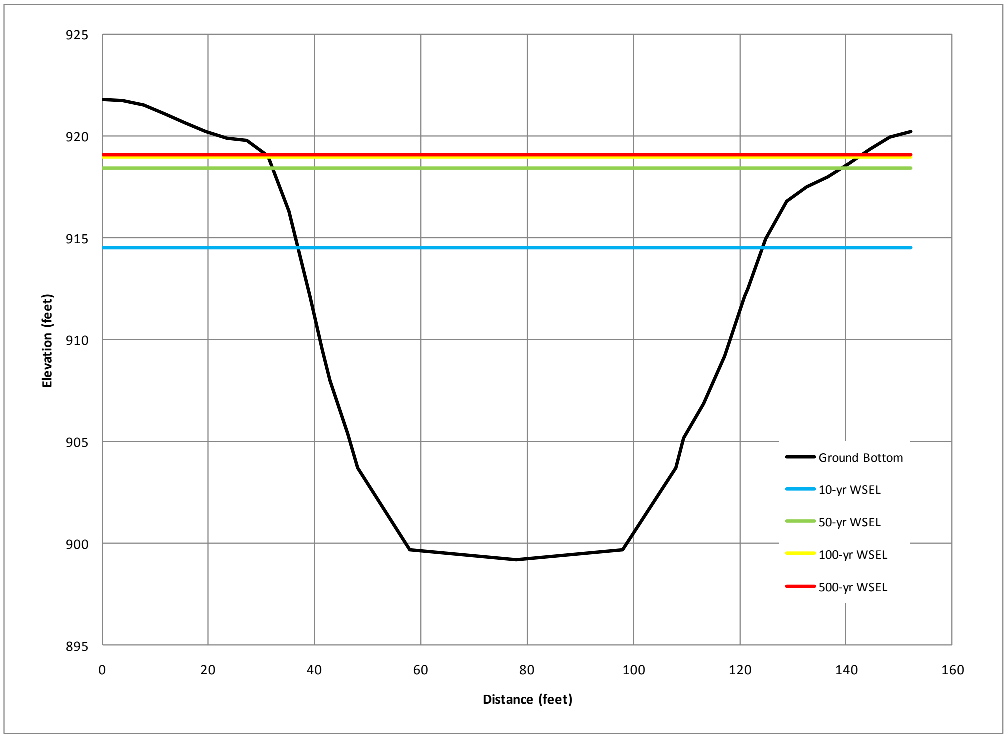


Figure F-A18 Background Existing Coincidental Hydrology Water Surface Elevations in the Sheyenne River at the Approximate Location of the LPP Diversion Channel

Flows and water surface elevations in Phase 3 and Phase 4 can be different. Phase 4 water surface elevations were calculated using a HEC-RAS unsteady flow model that accounts for flow through the floodplain and off channel storage areas and accounts for available storage in the floodplain when computing water surface elevations. The Phase 4 model includes storage areas and storage area connections to provide an accurate depiction of how flows enter and exit the main channel and accounts for conveyance of flow through the floodplain. As compared to Phase 3 that used a rating curve that closely matched the Phase 4 HEC-RAS model for flows contained within the tributary channel, but did not accurately characterize the stage-flow relationship for higher stages when flows leave the main tributary channel and are conveyed through the floodplain.

Actual velocities in the Sheyenne River have been measured at the USGS Gage 05059300, located above the Sheyenne River Diversion near Horace, North Dakota (approximately 1 mile downstream of the Diversion Channel as the crow flies). Figure F-A19 presents average channel velocities plotted against flow in the Sheyenne River near Horace, ND. Figure F-A20 presents velocity distributions in the Sheyenne River measured on April 30, 2009 and June 30, 2009.

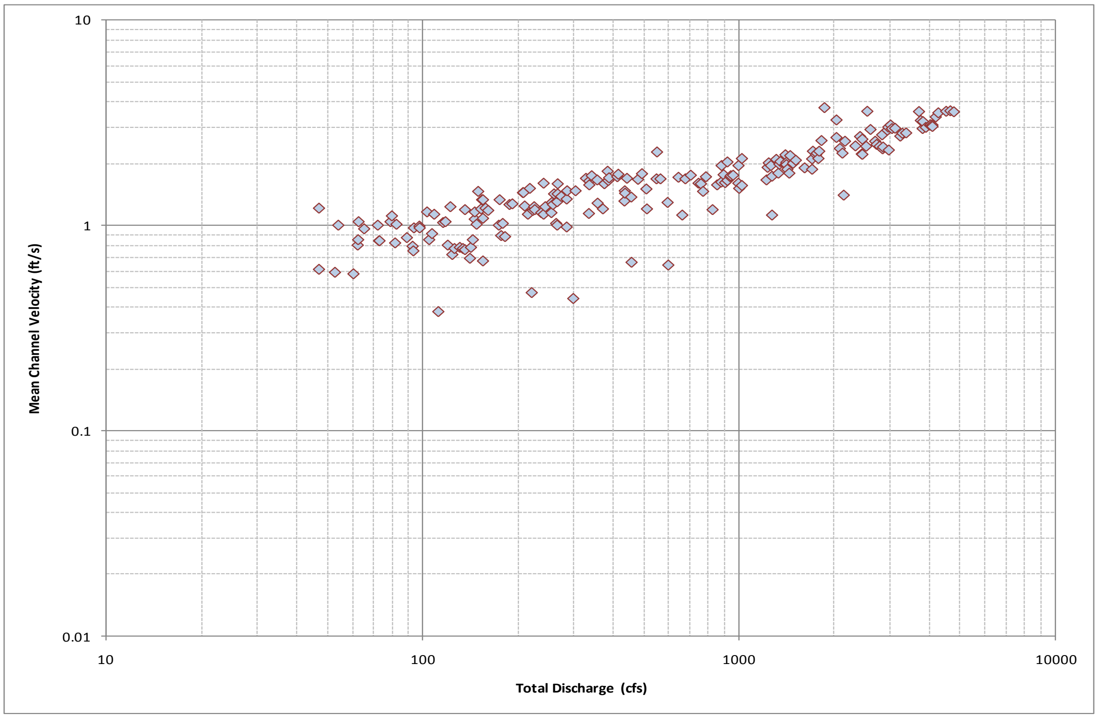


Figure F-A19 Average channel velocities versus total discharge in the Sheyenne River measured at USGS gage 05059300 (Horace, ND)

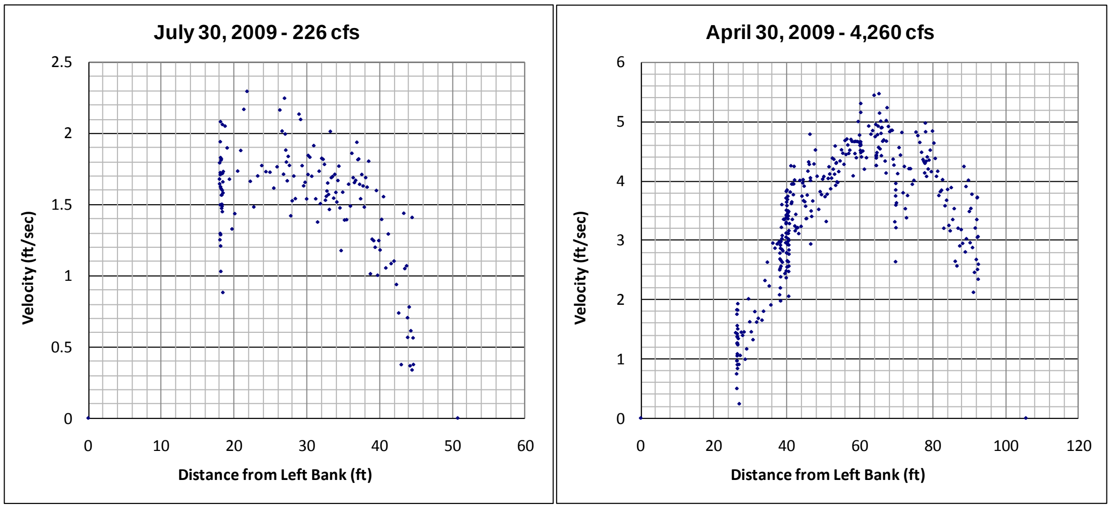


Figure F-A20 Velocity distributions measured in the Sheyenne River at USGS gage 05059300 (Horace, ND)

226 cfs corresponds to the mean annual flow and 4260 cfs corresponds to between a 20-year and 50-year local event in the Sheyenne River based on Phase 3 hydrology. Although approximately 4,000 cfs is the maximum capacity of the Sheyenne River before flows begin to breakout to the Drain 14 and 21C systems to the west.

F-A5.0 BACKGROUND HYDROLOGY OF THE MAPLE RIVER

Flows, water surface elevations, and average velocity for the Maple River for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-13. Water surface elevations in the Maple River for the flood events listed above, plotted against the cross section data, and are presented for the local tributary hydrology in Figure FA-21.

Table F-A13 Background Existing Local Hydrology of the Maple River at the Approximate Location of the LPP Diversion Channel

Event (yr)	Total Flow (cfs) *	River Flow (cfs)	WSEL (ft)	Velocity (ft/s)	River Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3				**Phase 4**		
2	970	970	890.73	2.26	-	-	-
5	2,010	2,010	894.18	2.76	-	-	-
10	3,550	3,550	896.82	3.10	4,804	899.07	2.89
50	6,430	5,180	898.77	3.12	5,366	899.94	2.51
100	8,270	5,240	898.83	3.11	5,179	900.11	2.31
500	9,590	5,290	899.14	2.88	4,840	900.36	2.02

* For large flows on the Maple River, the total flow is split between the overland flow, Maple River flow and Drain 14 flows.

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.

Note: Phase 3 values based on steady flow analysis

Note: Phase 4 values based on unsteady flow analysis

**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

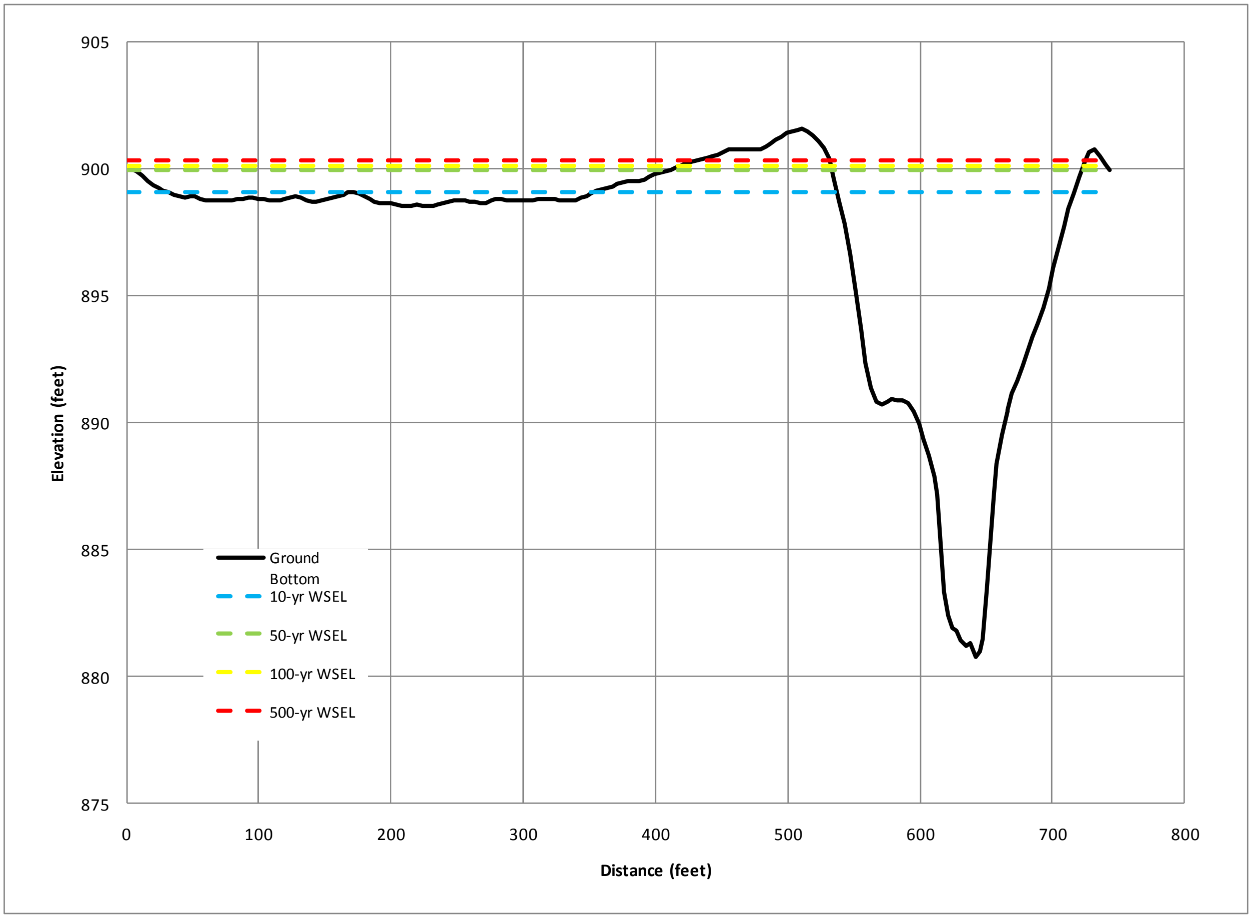


Figure F-A21 Background Existing Local Hydrology Water Surface Elevations in the Maple River at the Approximate Location of the LPP Diversion Channel

Flows, water surface elevations, and average velocity for the Maple River for the 10, 50-, 100-, and 500-year events are presented for the coincidental tributary hydrology in Table FA-14. Water surface elevations in the Maple River for the flood events listed above, plotted against the cross section data, and are presented for the coincidental tributary hydrology in Figure FA-22.

Table F-A14 Background Existing Coincidental Hydrology of the Maple River at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3			Phase 4		
2	1,370	891.17	2.99	-	-	-
5	2,000	893.52	3.05	-	-	-
10	2,650	895.36	2.95	5,004	899.08	3.00
50	4,400	898.83	2.61	5,368	899.95	2.50
100	4,925	899.96	2.16	5,228	900.08	2.35
500	5,115	900.34	2.06	5,059	900.21	2.20

*Event recurrence interval refers to the Red River flow record.

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.

Note: Phase 3 values based on steady flow analysis

Note: Phase 4 values based on unsteady flow analysis

**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

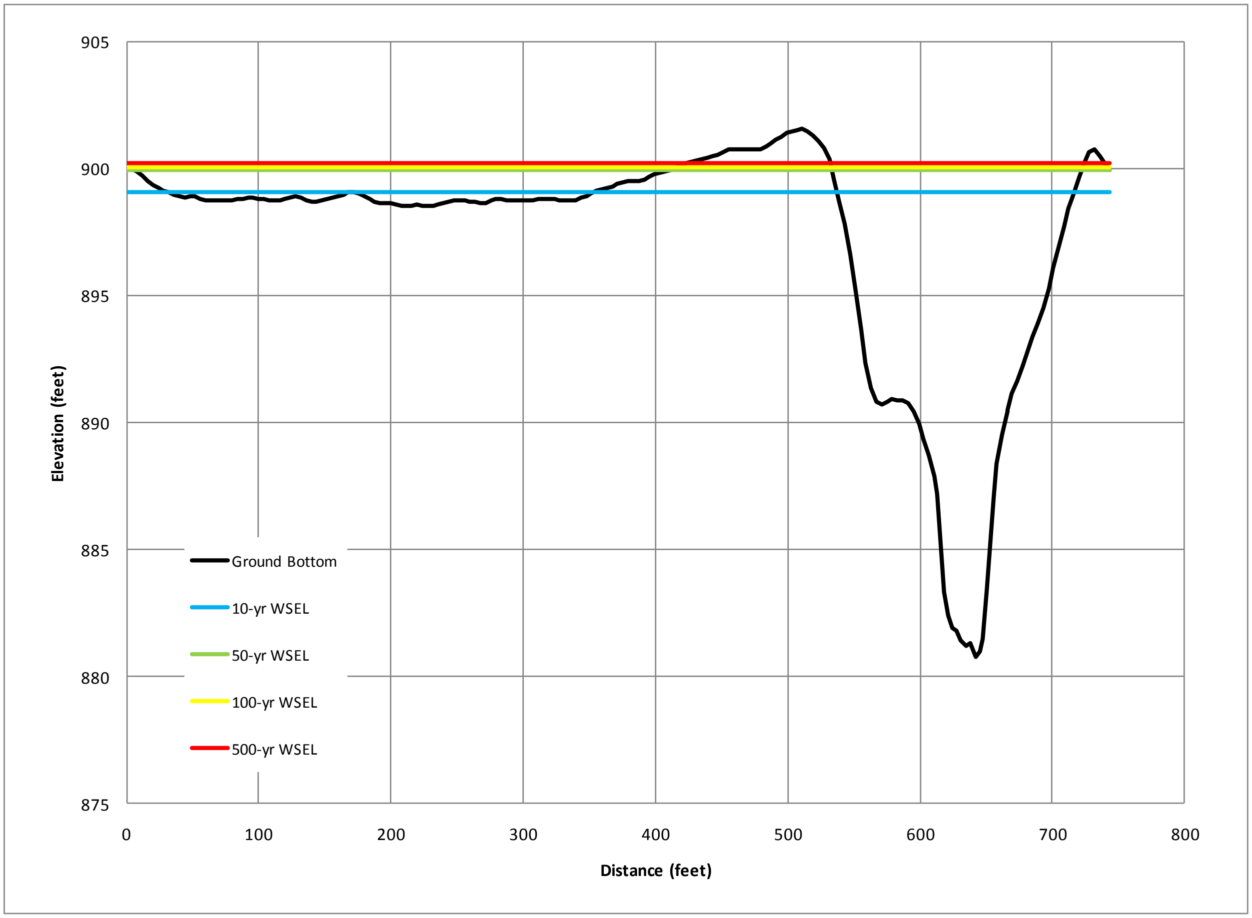


Figure F-A22 Background Existing Coincidental Hydrology Water Surface Elevations in the Maple River at the Approximate Location of the LPP Diversion Channel

Flows and water surface elevations in Phase 3 and Phase 4 can be different. Phase 4 water surface elevations were calculated using a HEC-RAS unsteady flow model that accounts for flow through the floodplain and off channel storage areas and accounts for available storage in the floodplain when computing water surface elevations. The Phase 4 model includes storage areas and storage area connections to provide an accurate depiction of how flows enter and exit the main channel and accounts for conveyance of flow through the floodplain. As compared to Phase 3 that used a rating curve that closely matched the Phase 4 HEC-RAS model for flows contained within the tributary channel, but did not accurately characterize the stage-flow relationship for higher stages when flows leave the main tributary channel and are conveyed through the floodplain.

Actual velocities in the Maple River have been measured at the USGS Gage 05060100 located below Mapleton, North Dakota (approximately five miles upstream of the Diversion Channel as the crow flies). Figure F-A23 presents the average channel velocities plotted against flow in the Maple River. Figure F-A24 presents velocity distributions in the Maple River measured on April 1, 2009 and April 12, 2009.

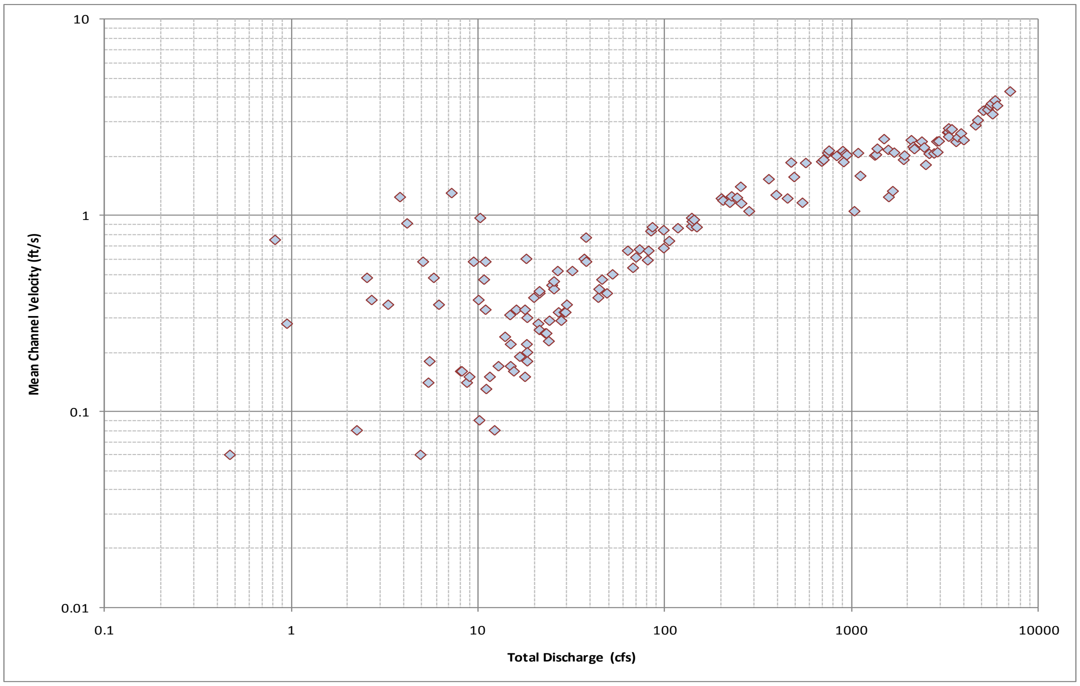


Figure F-A23 Average channel velocities versus total discharge in the Maple River measured at USGS gage 05060100 (Mapleton, ND)

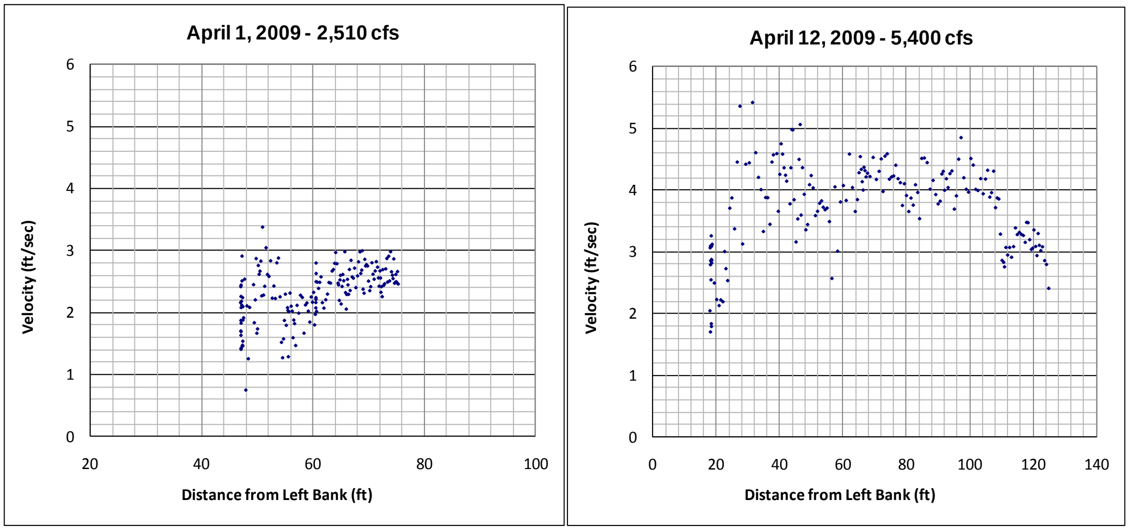


Figure F-A24 Velocity distributions measured in the Maple River at USGS gage 05060100 (Mapleton, ND)

2,510 cfs corresponds to between a 5- and 10-year event and 5,400 cfs corresponds to approximately a 50-year local event in the Maple River.

F-A6.0 BACKGROUND HYDROLOGY OF THE LOWER RUSH RIVER

Flows, water surface elevations, and average velocities for the Lower Rush River for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-15. Water surface elevations in the Lower Rush River for the flood events listed above, plotted against the cross section data, and are presented for the local tributary hydrology in Figure FA-25.

Table F-A15 Background Existing Local Hydrology of the Lower Rush River at the Approximate Location of the LPP Diversion Channel

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)**
	Phase 3			**Phase 4**		
2	302	889.07	1.20	-	-	-
5	774	890.97	1.49	-	-	-
10	1,200	891.47	0.98	315	892.93	6.32
50	2,369	892.08	0.72	841	894.67	5.75
100	2,937	892.29	0.70	921	894.85	5.51
500	4,365	892.80	0.68	983	894.98	4.36

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.
** Phase 4 velocity values are associated with the peak water surface elevation and may not represent the peak velocity. Velocity values taken from the culverts in the Lower Rush River at ShySC131B

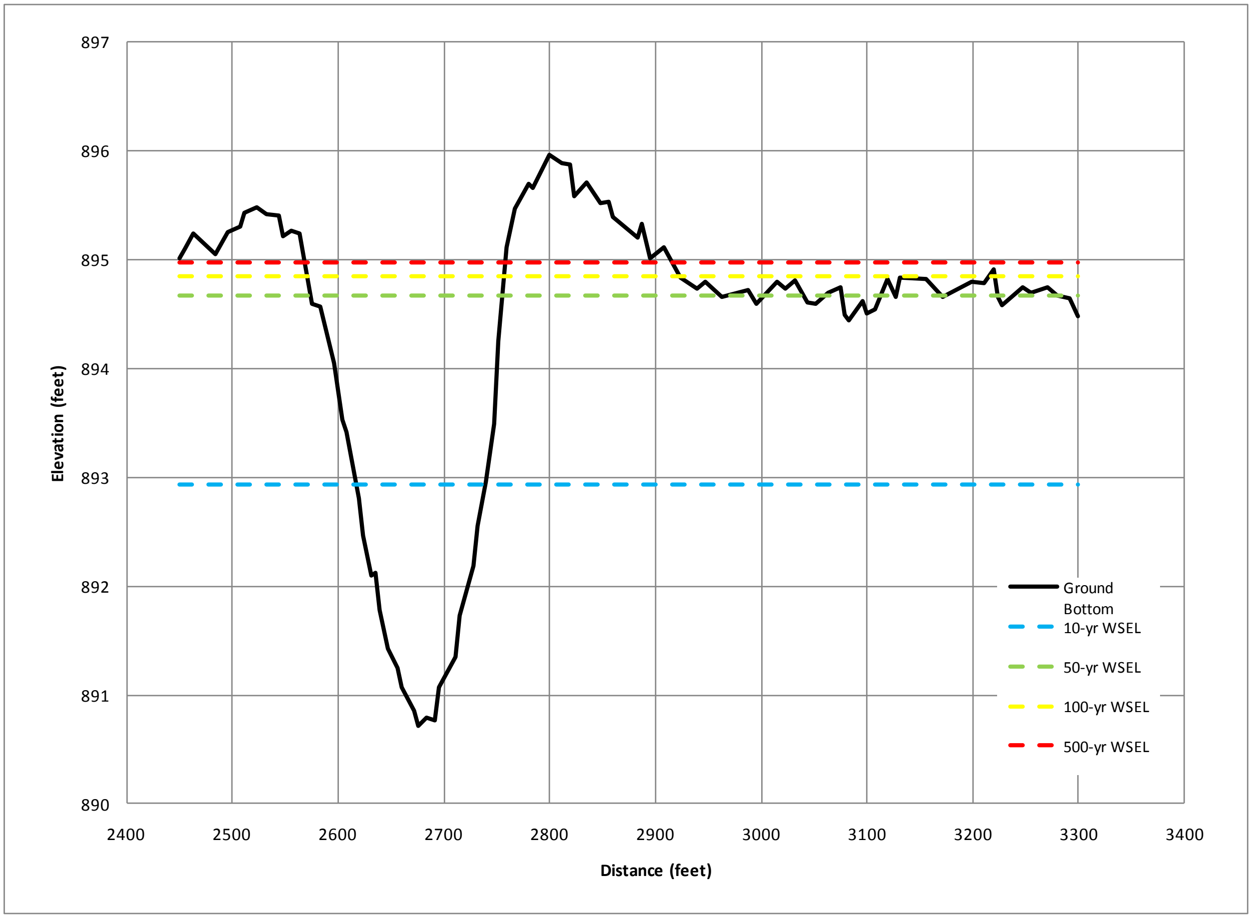


Figure F-A25 Background Existing Local Hydrology Water Surface Elevations in the Lower Rush River at the Approximate Location of the LPP Diversion Channel

Flows and water surface elevations in Phase 3 and Phase 4 can be different. In Phase 3 an assumed flow distribution was used to determine how flows were distributed between the Rush and Lower Rush Rivers. During Phase 4, additional analysis was completed, as discussed in Appendix B, to determine how flow is distributed between the Rush and Lower Rush Rivers.

Flows, water surface elevations, and average velocities for the Lower Rush River for the 10-, 50-, 100-, and 500-year events are presented for the coincidental tributary hydrology in Table FA-16. Water surface elevations in the Lower Rush River for the flood events listed above, plotted against the cross section data, and are presented for the coincidental tributary hydrology in Figure FA-26. The water surface elevations for coincidental hydrology are higher than for the local hydrology, even though the flows for coincidental hydrology are much lower than for the local hydrology. This is due to backwater effects from the Red River of the North.

Table F-A16 Background Existing Coincidental Hydrology of the Lower Rush River at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)***
	Phase 3			**Phase 4**		
2	85	886.45	1.19	-	-	-
5	123	887.05	1.13	-	-	-
10	163	889.73	0.53	326	893.02	6.56
50	285	893.89	0.03	797	894.64	5.74
100	355	895.18	0.02	880	894.83	5.24
500	380	895.9	0.02	847	894.74	5.79

*Event recurrence interval refers to the Red River flow record.

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.

Note: Phase 3 values based on steady flow analysis

Note: Phase 4 values based on unsteady flow analysis

**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

*** Phase 4 velocity values are associated with the peak water surface elevation and may not represent the peak velocity. Velocity values taken from the culverts in the Lower Rush River at ShySC131B

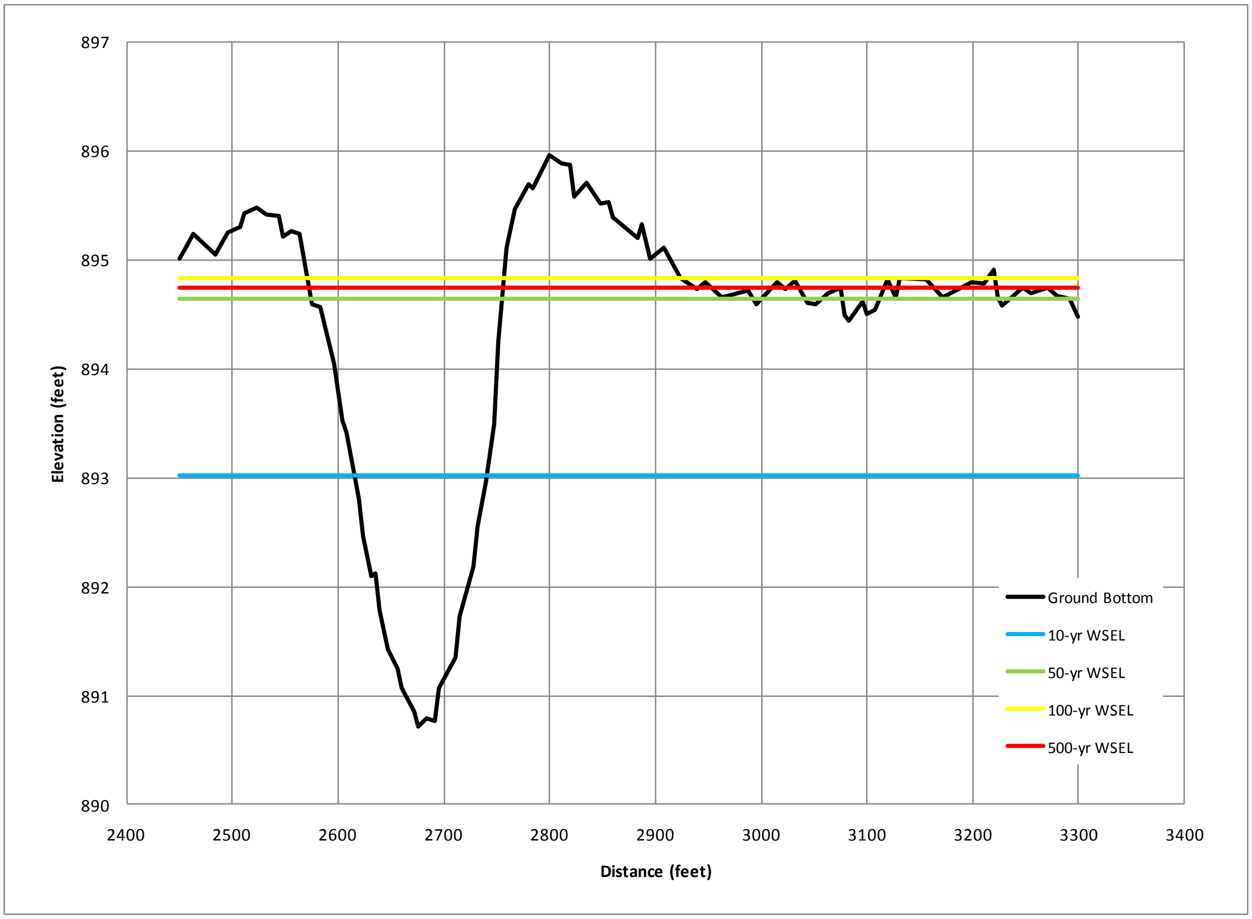


Figure F-A26 Background Existing Coincidental Hydrology Water Surface Elevations in the Lower Rush River at the Approximate Location of the LPP Diversion Channel

The implementation of an improved hydrologic model has decreased the flows and water surface elevations in Phase 4. In Phase 3, the discharges in the Lower Rush River were calculated from a rating curve. The Phase 4 hydrologic model incorporated storage areas and storage area connections outside of the typical river cross sections. The storage areas and their connections provided a more accurate description of the water movement into the Lower Rush River from the surrounding landscape.

F-A7.0 BACKGROUND HYDROLOGY OF THE RUSH RIVER

Flows, water surface elevations, and average velocities for the Rush River for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-17. Water surface elevations in the Rush River for the flood events listed above, plotted against the cross section data, and are presented for the local tributary hydrology in Figure FA-27.

Table F-A17 Background Existing Local Hydrology of the Rush River at the Approximate Location of the LPP Diversion Channel

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
	Phase 3			Phase 4		
2	415	886.50	1.04	-	-	-
5	1,065	889.20	1.13	-	-	-
10	1,650	889.79	0.81	616	899.48	0.28
50	3,258	890.52	0.61	1,222	891.08	0.78
100	4,040	890.76	0.62	1,536	891.36	0.89
500	6,008	891.30	0.65	2,666	891.87	0.99

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

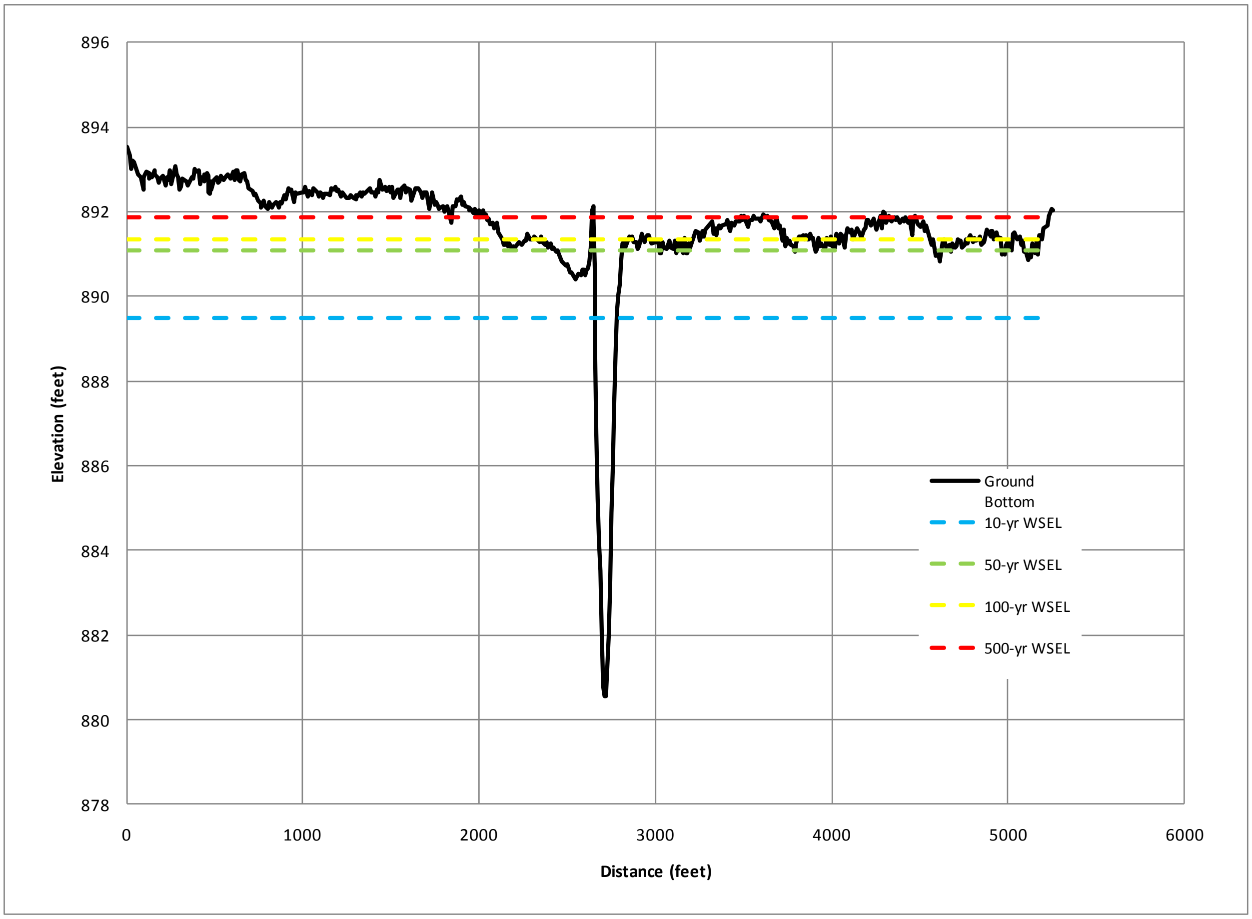


Figure F-A27 Background Existing Local Hydrology Water Surface Elevations in the Rush River at the Approximate Location of the LPP Diversion Channel

Flows, water surface elevations, flow area, and average velocity for the Rush River for the 10-, 50-, 100-, and 500-year events are presented for coincidental tributary hydrology in Table FA-18. Water surface elevations in the Rush River for the flood events listed above, plotted against the cross section data, and are presented for coincidental tributary hydrology in Figure FA-28. The water surface elevations for coincidental hydrology are higher than for the local hydrology, even though the flows for coincidental hydrology are much lower than for the local hydrology. This is due to backwater effects from the Red River of the North.

Table F-A18 Background Existing Coincidental Hydrology of the Rush River at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3			Phase 4		
2	170	883.34	1.20	-	-	-
5	246	886.44	0.63	-	-	-
10	326	889.12	0.37	1,433	889.73	1.10
50	570	893.08	0.03	2,887	891.67	1.43
100	710	894.21	0.03	3,137	891.78	1.27
500	760	894.98	0.03	3,892	891.92	1.37

*Note: Event recurrence interval refers to the Red River flow record.

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.

Note: Phase 3 values based on steady flow analysis

Note: Phase 4 values based on unsteady flow analysis

**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

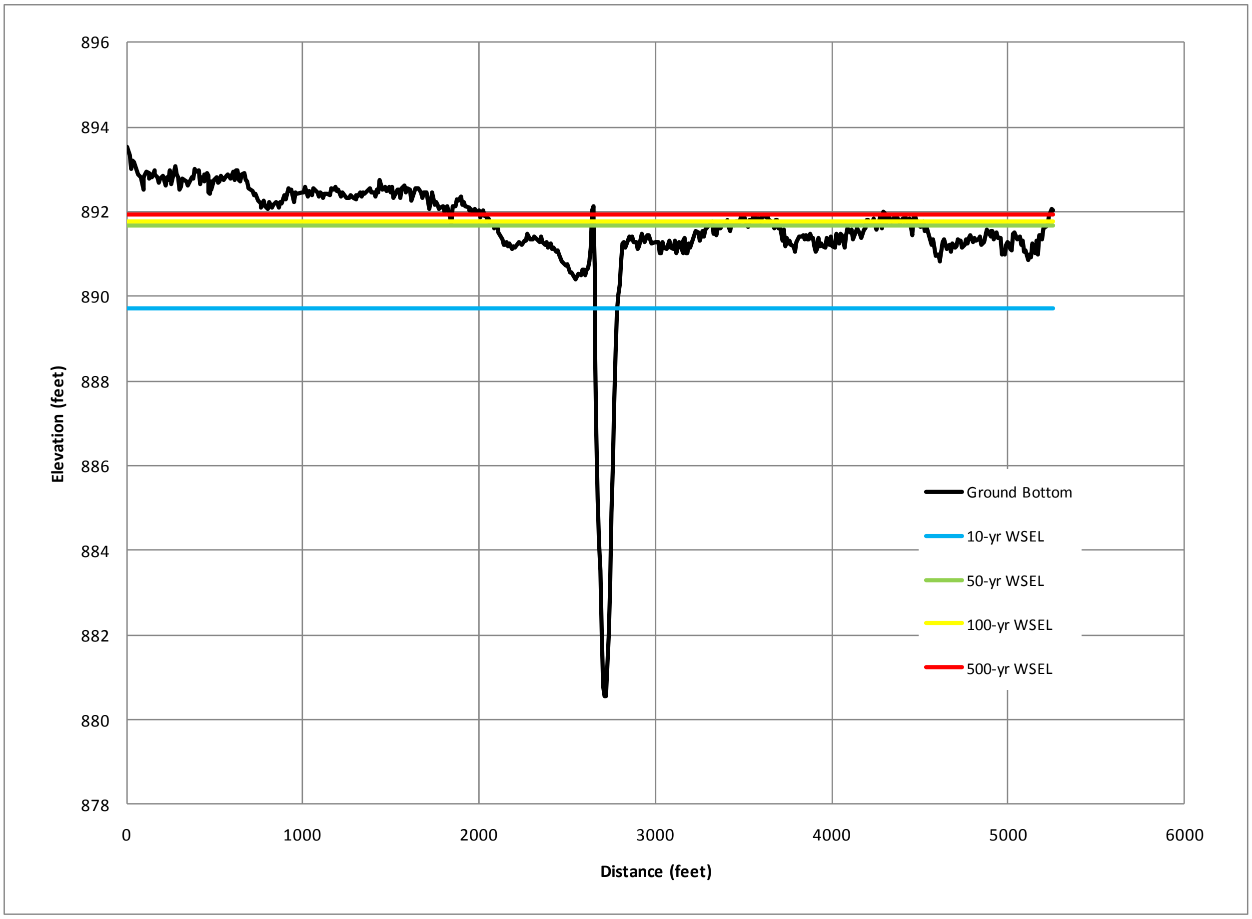


Figure F-A28 Background Existing Coincidental Hydrology Water Surface Elevations in the Rush River at the Approximate Location of the LPP Diversion Channel

The implementation of an improved hydrologic model has decreased the flows and water surface elevations in Phase 4. In Phase 3, the discharges in the Rush River were calculated from a rating curve. The Phase 4 hydrologic model incorporated storage areas and storage area connections outside of the typical river cross sections. The storage areas and their connections provided a more accurate description of the water movement into the Rush River from the surrounding landscape.

F-A8.0 BACKGROUND HYDROLOGY OF WOLVERTON CREEK

Flows, water surface elevations, and average velocities for Wolverton Creek for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-19. Water surface elevations in Wolverton Creek for the flood events listed above, plotted against the cross section data, and are presented for the local tributary hydrology in Figure FA-29.

Table F-A19 Background Existing Local Hydrology of Wolverton Creek at the Approximate Location of Wolverton Creek Control Structure

Event (yr)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
	Phase 4		
10	100	903.77	0.39
50	92	909.87	0.03
100	41	911.77	0.01
500	96	914.12	0.01

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.

Note: Phase 4 values based on unsteady flow analysis

*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow. During large flood events the maximum water surface elevation is typically controlled by the water surface elevation on the Red River. Therefore the flow and velocity that correspond to the maximum water surface elevation are typically lower than the peak local flow or velocity on Wolverton Creek that occur before the peak water surface elevation.

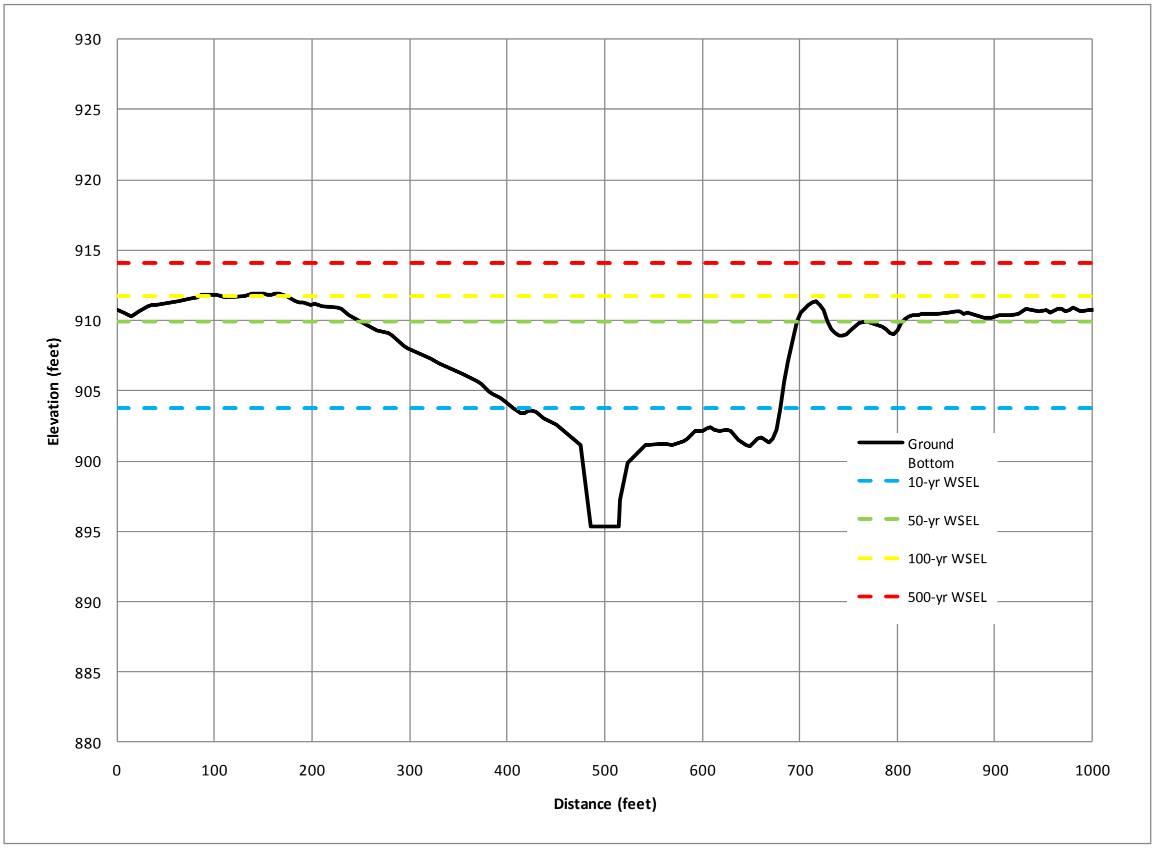


Figure F-A29 Background Existing Local Hydrology Water Surface Elevations in Wolverton Creek at the Approximate Location of the Wolverton Creek control structure (Station 9173)

Flows, water surface elevations, flow area, and average velocity for Wolverton Creek for the 10-, 50-, 100-, and 500-year events are presented for coincidental tributary hydrology in Table FA-20. Water surface elevations in Wolverton Creek for the flood events listed above, plotted against the cross section data, and are presented for coincidental tributary hydrology in Figure FA-30.

Table F-A20 Background Existing Coincidental Hydrology of Wolverton Creek at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 4		
5	-	-	-
10	325	907.42	0.89
50	846	912.98	0.14
100	997	913.79	0.14
500	952	914.88	0.11

*Note: Event recurrence interval refers to the Red River flow record.

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.

**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow. During large flood events the maximum water surface elevation is typically controlled by the water surface elevation on the Red River. Therefore the flow and velocity that correspond to the maximum water surface elevation are typically lower than the peak local flow or velocity on Wolverton Creek that occur before the peak water surface elevation.

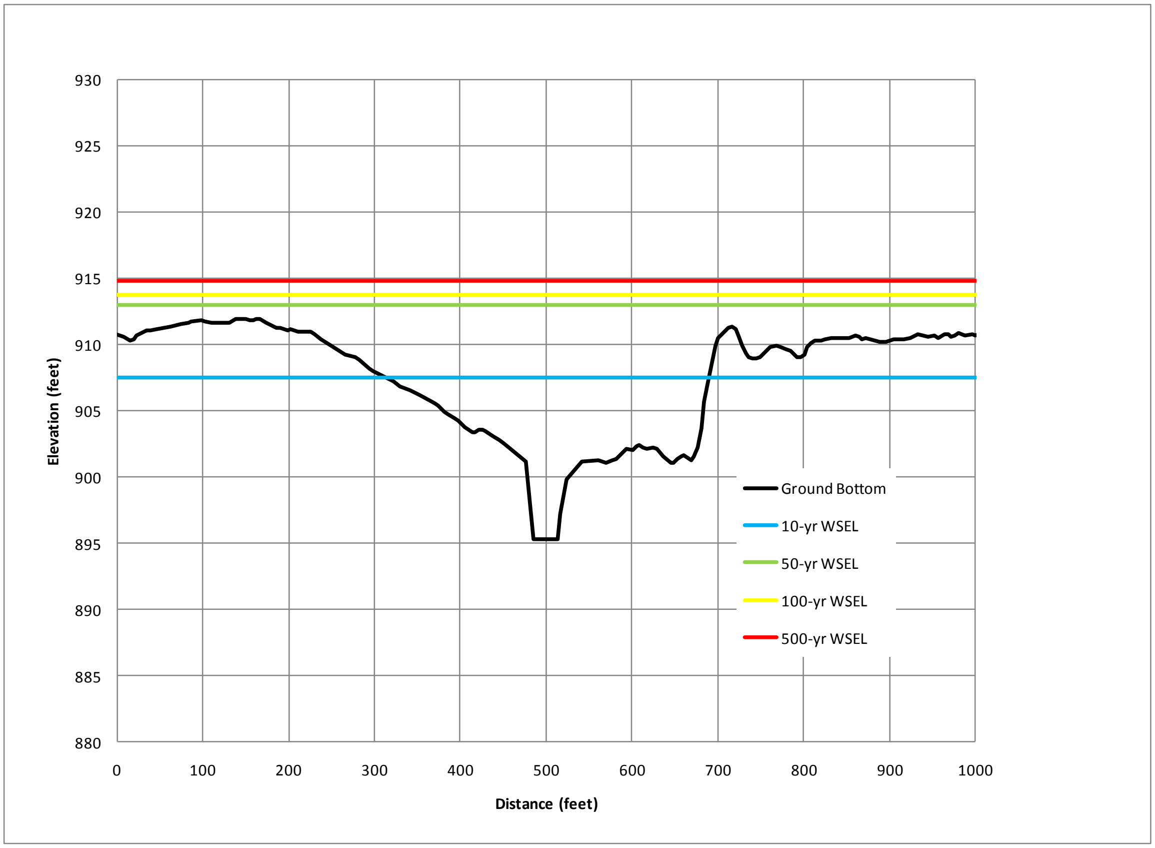


Figure F-A30 Background Existing Coincidental Hydrology Water Surface Elevations in Wolverton Creek at the Approximate Location of the Wolverton Creek control structure (Station 9173)

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT B – ALTERNATIVES MATRIX FOR CONTROL STRUCTURE ON RED RIVER OF THE NORTH AND INLET WEIR OF DIVERSION CHANNEL

Report for the US Army Corps of Engineers, and the cities of Fargo, ND & Moorhead, MN

By: Barr Engineering Co.

FINAL – Version February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT B – ALTERNATIVES MATRIX FOR CONTROL STRUCTURE ON RED RIVER OF THE NORTH AND INLET WEIR OF DIVERSION CHANNEL

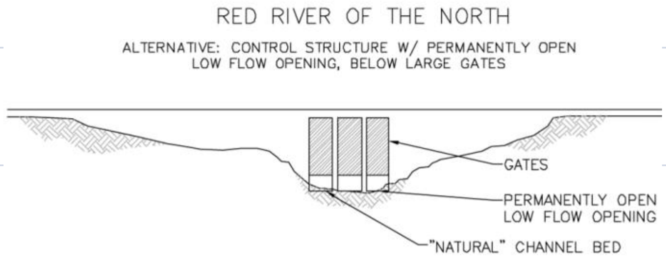
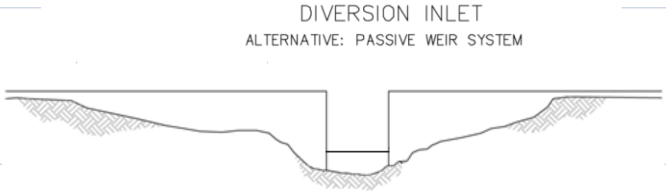
F-B1.0 ALTERNATIVES MATRIX FOR CONTROL STRUCTURES AND INLET WEIRS

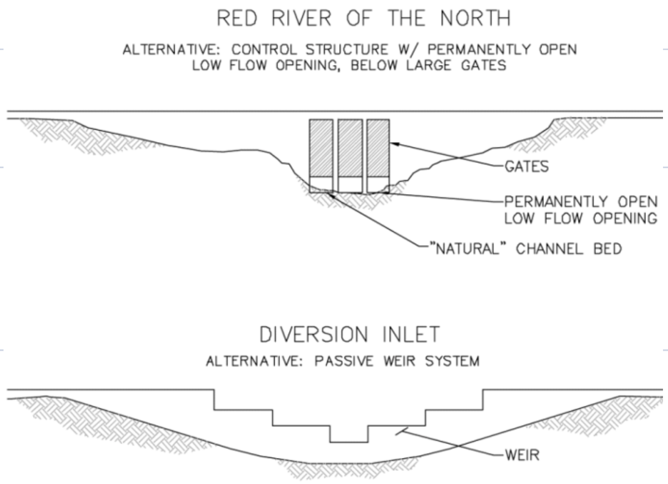
Several design concepts for the Red River Control Structure and Diversion Inlet Structure were considered in this feasibility analysis. The advantages and disadvantages of eight (8) combinations of Red River Control Structure and Diversion Inlet Structure are presented in the following table, referred to as an “Alternatives Matrix”. The advantages and disadvantages described in the alternatives matrix include hydraulic performance, flood flows, low flows, potential environmental impacts, permitting, and operation and maintenance. The combinations of structures included in the alternatives matrix are listed below:

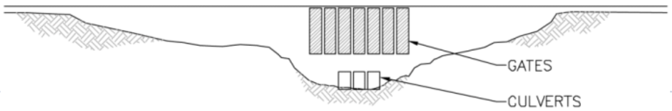
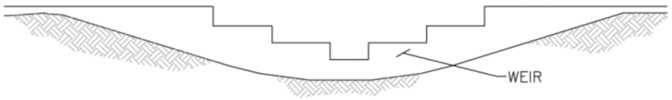
1.) Control structure on the Red River with gated openings and operable gates from above, passive weir inlet to diversion; this is the design recommended as of 02-28-2011
2.) Control structure on the Red River with gated openings and operable gates from above, passive stepped weir inlet to diversion
3.) Control structure on the Red River with ungated low box culverts and operable gates from above, passive weir inlet to diversion
4.) Control structure on the Red River with open area and natural channel bottom, operable gates above, and passive weir inlet to diversion
5.) Weir Control Structure on Diversion, with no structure on Red River
6.) Weir Control Structure on Diversion, with no structure on Red River (longer weir)
7.) Weir Control Structure on Diversion with gates below 5-year crest elevation, with no structure on Red River

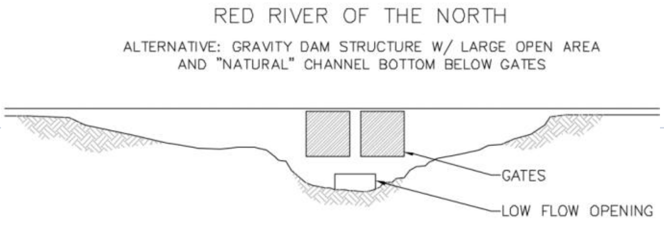
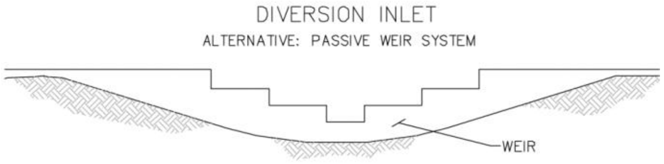
8.) Channel constriction (ungated) on the Red River and operable Diversion Inlet (gated)

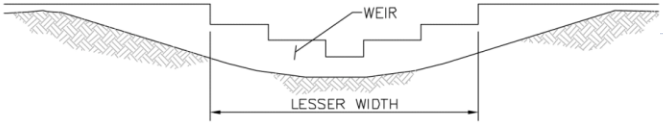
Table F-B1 Red River Control Structure and Diversion Inlet Alternatives Matrix

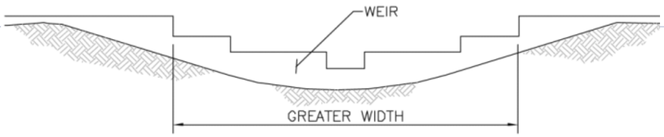
Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
1	Diversion Inlet	Control structure on the Red RIver with gated openings and operable gates from above, passive weir inlet to diversion. **This is the design recommended as of 02-28-2011.**	**Hydraulic Performance:**	High level of control of upstream Red River water surface profile; ability to stage water upstream of control structures		
sketch below:   			**Flood Flows:**	Provides floodplain access of Red River channel upstream of structure; limits flood flows into protected area		Risk of gate malfunction and flooding of protected area; Failure mode still diverts water around the protected area. Bulkhead slots provide backup
			Low Flows:	Passive transmission of low flows into protected area through low openings		
			Potential Environmental Impacts:		Periodic high velocities could pose environmental impact issues	
			Permitting:		Building a structure on the Red River poses regional and international permitting issues	
			Operation and Maintenance:		Ongoing operation and maintenance costs for superstructure adding to life-cycle cost	Ice/gate interaction, debris fouling, ice fouling

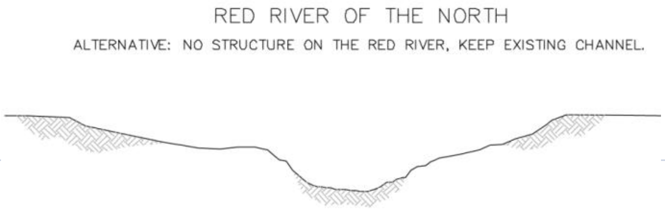
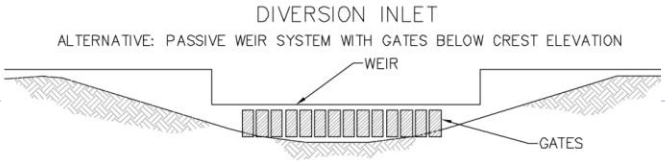
Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
2	Diversion Inlet	Control structure on the Red RIver with ungated openings and operable gates from above, stepped passive weir inlet to diversion. **This is the design recommended as of 12-31-2009.**	**Hydraulic Performance:**	High level of control of upstream Red River water surface profile; ability to match existing rating curve on the Red River	Unable to stage sufficient water upstream of control structure	
sketch below: 			**Flood Flows:**	Provides floodplain access of Red River channel upstream of structure; limits flood flows into protected area		Risk of gate malfunction and flooding of protected area; Failure mode still diverts water around the protected area. Bulkhead slots provide backup
			Low Flows:	Passive transmission of low flows into protected area through low openings		
			Potential Environmental Impacts:		Periodic high velocities could pose environmental impact issues	
			Permitting:		Building a structure on the Red River poses regional and international permitting issues	
			Operation and Maintenance:		Ongoing operation and maintenance costs for superstructure adding to life-cycle cost	Ice/gate interaction, debris fouling, ice fouling

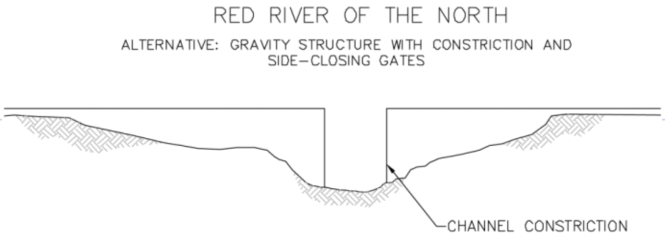
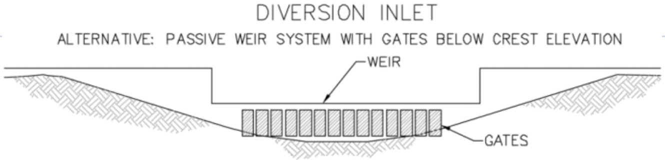
Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
3	Diversion Inlet	Control structure on the Red River with ungated low box culverts and operable gates above, passive weir inlet to diversion.	**Hydraulic Performance:**	High level of control of upstream Red River water surface profile; ability to match existing rating curve on the Red River	Unable to stage sufficient water upstream of control structure	
sketch below: RED RIVER OF THE NORTH ALTERNATIVE: GRAVITY DAM STRUCTURE W/ OPEN CULVERTS BELOW NUMEROUS SMALL GATES  DIVERSION INLET ALTERNATIVE: PASSIVE WEIR SYSTEM  			**Flood Flows:**	Provides floodplain access of Red River channel upstream of structure; limits flood flows into protected area		Risk of gate malfunction and flooding of protected area; Failure mode still diverts water around the protected area
			Low Flows:	Passive transmission of low flows into protected area through culverts		
			Potential Environmental Impacts:		Box culverts replacing channel bed could pose environmental impact issues; periodic high velocities	
			Permitting:		Building a structure on the Red River poses regional and international permitting issues	
			Operation and Maintenance:		Ongoing operation and maintenance costs for superstructure adding to life-cycle cost	Ice/gate interaction, debris fouling, ice fouling

Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
4	Diversion Inlet	Control structure on the Red River with open area and natural channel bottom, operable gates above, and passive weir inlet to diversion	**Hydraulic Performance:**	High level of control of upstream Red River water surface profile; ability to match existing rating curve on the Red River	High velocities through low opening could result in erosion impacting structural integrity	
sketch below:			**Flood Flows:**	Provides floodplain access of Red River channel upstream of structure; limits flood flows into protected area		Risk of gate malfunction and flooding of protected area; Failure mode still diverts water around the protected area
			**Low Flows:**	Passive transmission of low flows into protected area through low opening		
			Potential Environmental Impacts:	Potentially less impact to channel than option 1 during lower flow velocities	Periodic high velocities could pose environmental impact issues and may affect channel bed	
			**Permitting:**		Building a structure on the Red River poses regional and international permitting issues	
			Operation and Maintenance:		Ongoing operation and maintenance costs for superstructure adding to life-cycle cost	Ice/gate interaction, debris fouling, ice fouling

Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
5	Diversion Inlet	Weir Control Structure on Diversion, with no structure on Red River	**Hydraulic Performance:**		No good control of upstream flows passing into protected side.	
sketch below: RED RIVER OF THE NORTH ALTERNATIVE: NO STRUCTURE ON THE RED RIVER, KEEP EXISTING CHANNEL.			**Flood Flows:**		Impact to water surface profiles in the Red River and the channel's access to the floodplain upstream	Benefits significantly reduced; B/C ratio much less than 1 (i.e. no project) due to limitations of flow sent to diversion channel resulting in higher flows in protected area
			**Low Flows:**	Passive transmission of low flows into protected area through existing channel		
DIVERSION INLET ALTERNATIVE: PASSIVE WEIR SYSTEM WITH LESSER WIDTH			**Potential Environmental Impacts:**	Less immediate impact to the existing Red River channel bed	Possible environmental impact to upstream floodplain due to impacts on water surface profile	
			**Permitting:**	Not building a structure on the Red River poses fewer regional and international permitting issues		
			Operation and Maintenance:	Less costly to operate and maintain than structures that span Red River		Ice fouling, debris fouling at diversion inlet

Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
6	Diversion Inlet	Weir Control Structure on Diversion, with no structure on Red River (longer weir)	**Hydraulic Performance:**		No good control of upstream flows passing into protected side.	
sketch below: RED RIVER OF THE NORTH ALTERNATIVE: NO STRUCTURE ON THE RED RIVER, KEEP EXISTING CHANNEL.			**Flood Flows:**		Impact to water surface profiles in the Red River and the channel's access to the floodplain upstream	Benefits significantly reduced; risk of B/C ratio much less than 1 (i.e. no project) due to limitations of flow sent to diversion channel resulting in higher flows in protected area
			**Low Flows:**	Passive transmission of low flows into protected area through existing channel	Possible environmental impact to upstream floodplain due to impacts on water surface profile	
DIVERSION INLET ALTERNATIVE: PASSIVE WEIR SYSTEM WITH GREATER WIDTH			**Potential Environmental Impacts:**	Less immediate impact to the existing Red River channel bed		
			**Permitting:**	Not building a structure on the Red River poses fewer regional and international permitting issues		
			Operation and Maintenance:	Less costly to operate and maintain than structures that span Red River		Ice fouling, debris fouling at diversion inlet

Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
7	Diversion Inlet	Weir Control Structure on Diversion with gates below 5-year crest elevation, with no structure on Red River	**Hydraulic Performance:**	More control of upstream water surface profile than options 5 or 6.	Low level of control of upstream Red River water surface profile. No good control of flows passing into protected side.	
sketch below:  			**Flood Flows:**		Potential Impact to water surface profiles in the Red River and the channel's access to the floodplain upstream	Benefits significantly reduced; risk of B/C ratio less than 1 (i.e. no project) due to limitations of flow sent to diversion channel resulting in higher flows in protected area. Failure of diversion structure results in catastrophic flooding
			Low Flows:	Passive transmission of low flows into protected area through existing channel		
 			**Potential Environmental Impacts:**	Less immediate impact to the existing Red River channel bed	Possible environmental impact to upstream floodplain due to impacts on water surface profile	
			Permitting:	Not building a structure on the Red River poses fewer regional and international permitting issues		
			Operation and Maintenance:		More costly than passive weir system on diversion inlet	Ice/gate interaction, debris fouling, ice fouling at diversion inlet

Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
8	Diversion Inlet	Channel Constriction (ungated) on the Red River and operable Diversion Inlet (gated)	**Hydraulic Performance:**	More control of upstream water surface profile than options 5 or 6.	Low level of control of upstream Red River water surface profile. No good control of flows passing into protected side.	
sketch below:  			**Flood Flows:**	Provides some floodplain access of Red River channel upstream of structure		Benefits significantly reduced; risk of B/C ratio less than 1 (i.e. no project). Risk of gate malfunction and flooding of protected area.; failure mode increases risk of flood flows into the protected area
			Low Flows:	Passive transmission of low flows into protected area through open area		
 			**Potential Environmental Impacts:**	Less immediate impact to the existing Red River channel bed	Possible environmental impact to upstream floodplain due to impacts on water surface profile	
			Permitting:	Leaving an unrestricted portion of the Red River may reduce perceptions it is a dam	Building a structure on the Red River poses regional and international permitting issues	
			Operation and Maintenance:	Perhaps less costly to operate and maintain than structures that span the entire Red River	Costly to operate and maintain superstructure adding to life-cycle cost	Ice/gate interaction, debris fouling, ice fouling at diversion inlet

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES

EXHIBIT C – ALTERNATIVES MATRIX FOR DIVERSION STRUCTURES AT NORTH DAKOTA TRIBUTARIES

Report for the US Army Corps of Engineers, and the Cities of Fargo, ND and Moorhead, MN

By: Barr Engineering Co.

FINAL - February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT C– ALTERNATIVES MATRIX FOR DIVERSION STRUCTURES AT NORTH DAKOTA TRIBUTARIES

This exhibit presents 1) a parametric study of the Maple and Sheyenne River Diversion Channel Crossing Structures and 2) Alternatives matrix for Diversion Structures at the North Dakota tributaries. The parametric study was completed in May 2010, it is being published for the first time in this Phase 3 report, and it discusses the effect of changing a variety of structural dimensions on the hydraulics of the Sheyenne and Maple River structures. The alternatives matrix was completed in December 2009 and it was printed as Exhibit E of Appendix F of the Phase 2 report submitted on January 6th, 2010.

F-C1.0 PARAMETRIC STUDY OF THE MAPLE AND SHEYENNE RIVER DIVERSION CHANNEL CROSSING STRUCTURES

The following information was previously presented as Exhibit C of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

The parametric study presented here was used in May 2010 to determine the effect of changing a variety of structural dimensions of the Maple and Sheyenne aqueduct structures on the hydraulics of the LPP Diversion Channel. The goal of this parametric study was to assist the coordination between the structural design and hydraulic design teams.

F-C1.1 INPUTS

The parametric study was completed using the May 24, 2010 HEC-RAS model of the North Dakota East 35k alignment - Phase 3.0 Year 0 Hydrology model. The 5-, 10-, 20-, 50-, 100-, 200-, and 500-year events in the Red River of the North were modeled.

A variety of structural dimensions were changed in the model:

1. The width of the Diversion Channel at the aqueduct varied between 250 feet, 275 feet, 300 feet and 350 feet.

2. The thickness of the piers in the Diversion Channel section at the aqueduct crossing varied between 1 foot, 2 feet, 3 feet, and 4 feet.

3. The spacing of the piers (centerline to centerline) in the Diversion Channel section at the aqueduct crossing varied between 25 feet, 30 feet, 30.5 feet, 37.5 feet, and 43.75 feet.

4. The length of the Diversion Channel underneath the aqueduct varied between 65 feet and 80 feet.

5. The thickness of the aqueduct (bottom) slab carrying the Maple or Sheyenne River over the Diversion Channel varied between 1.5 feet, 2 feet, 3 feet, 3.5 feet, and 4 feet.

6. The elevation of the top of the Maple River aqueduct structure varied between 892.7, 893.7, 894.7, and 898.7 feet.

The Diversion Channel transitions into and out of the aqueduct crossing (i.e., the radius of the vertical wing wall, distance between cross sections, location of levees, etc.) were changed in accordance to the modification with the structural dimensions listed above.

It should be noted that the May 24, 2010 model is not the final Phase 3 HEC-RAS model used for assessing impacts. The May 24, 2010 model included a variety of parameters that were changed for the Phase 3 model. These outdated parameters included a 125 foot bottom width upstream of the Sheyenne River, and the levees along the Connecting Channel were located at the bank station. The three gates on the Red River Control Structure were only 40 feet wide and were optimized during the work in April 2010. The HEC-RAS model was updated two days later on May 26, 2010 to have a 100 foot bottom width upstream of the Sheyenne River and a 50 foot levee setback from the bank station along the Connecting Channel. The gates were subsequently changed to 50 feet wide on May 27, 2010 and gate operations were optimized on June 1, 2010. The parametric study was completed before the May 26, 2010 model was available. Additional changes to the hydrology and some cross sections were made in the June 9, 2010 model. Therefore, the results described in this parametric study are not final results. However, these results allow to weight the relative changes in hydraulics based on changes in structural dimensions.

F-C1.2 RESULTS

The results of 22 different model runs are shown in Table F-C1 for the Sheyenne River crossing and Table FC-2 for the Maple River crossing. The main conclusions from this parametric study can be summarized as:

1. Widening the Diversion Channel from 250 feet to 350 feet while holding all other variables the same, results in decreasing head loss (by approximately 55 to 60%) and decreasing velocities (by approximately 30 to 35%) in the Diversion Channel at the tributary crossing. (See model runs 1, 4 and 13).

2. Increasing the thickness of the piers in the Diversion Channel from 1 foot to 3 feet while holding all other variables the same, results in increasing head loss (by approximately 15 to 20%) and increasing velocities (by less than 10%) in the Diversion Channel at the tributary crossing. (See model runs 1 and 3).

3. Increasing the spacing of the piers in the Diversion Channel from 25 feet to 30.5 feet while holding all other variables the same, results in decreasing head loss (by less than 10%) and decreasing velocities (by less than 10%) in the Diversion Channel at the tributary crossing. (See model runs 18, 19, 20. and 21).

4. Increasing the length of the Diversion Channel underneath the aqueduct from 65 feet and 80 feet has no effect on head loss and velocities in the Diversion Channel at the tributary crossing. (See model runs 4, 5, 15 and 16).

5. Increasing the thickness of the aqueduct (bottom) slab carrying the Maple or Sheyenne River over the Diversion Channel from 1.5 feet to 3.5 feet while holding all other variables the same, results in increasing head loss (by approximately 25 to 40%) and increasing velocities (by approximately 10 to 15%) in the Diversion Channel at the tributary crossing. (See model runs 1, 2, 4 and 11).

6. Increasing the top of the Maple River aqueduct by 1 foot to an elevation of 893.7 would prevent the 50-year event in the Diversion Channel from overtopping the aqueduct (i.e., the 100-year event would overtop the structure) but would increase the head loss (by approximately 15 to 20%) and increase the velocities (by less than 10%) in the Diversion Channel at the Maple River crossing. (See model runs 6, 7, 15 and 16).

7. Increasing the top of the Maple River aqueduct by 6 feet to an elevation of 898.7 would prevent the 500-year event from overtopping the Maple River aqueduct but would significantly increase the head loss (by approximately 75%) and the velocities (by approximately 25 to 30%) in the Diversion Channel at the Maple River crossing. (See model runs 4 and 9).

8. Increasing the elevation of the top of the Maple River aqueduct has little effect on the velocities and head loss at the Sheyenne River crossing, although the water surface elevations at the Sheyenne River crossing in the Diversion Channel may increase slightly. (See model runs 8, 9, 14, 15, 16 and 17).

Table FC-1 Summary of Parametric Study Results for the Diversion Channel Crossing of the Sheyenne River

Run	Diversion Channel Width (ft)	Pier Width (ft)	Pier Spacing (ft)	Length of Diversion Channel (ft)	Thickness of Slab (ft)	Elevation of Top of Maple Structure (ft)	500-year WSEL Upstream of Structure (ft)	500-year WSEL Down-stream of Structure (ft)	500-year Head Loss Across the Structure (ft)	500-year Velocity at Structure (ft/s)
1	250	1	25	65	1.5	892.7	911.75	909.95	1.80	9.37
2	250	1	25	65	3.5	892.7	912.50	910.12	2.38	10.74
3	250	3	25	65	1.5	892.7	912.12	909.97	2.15	10.11
4	300	1	25	65	1.5	892.7	911.19	909.94	1.25	7.83
5	300	1	25	80	1.5	892.7	911.19	909.94	1.25	7.83
6	300	4	37.5	65	4	893.7	912.20	910.05	2.15	9.90
7	300	2	25	65	2	893.7	911.42	909.95	1.47	8.38
8	300	1	25	65	1.5	894.7	911.22	909.98	1.24	7.83
9	300	1	25	65	1.5	898.7	911.28	910.03	1.25	7.83
10	300	3	25	65	1.5	892.7	911.44	909.94	2.50	8.47
11	300	1	25	65	3.5	892.7	911.69	909.97	1.72	8.99
12	300	3	25	65	3.5	892.7	912.02	909.96	2.06	9.72
13	350	1	25	65	1.5	892.7	910.84	909.93	0.91	6.72
14	350	4	43.75	65	4	892.7	911.50	909.97	1.53	8.39
15	350	4	43.75	65	4	893.7	911.49	909.97	1.52	8.36
16	350	4	43.75	80	4	893.7	911.49	909.97	1.52	8.36
17	350	4	43.75	65	4	894.7	911.51	909.99	1.52	8.36
18	275	2	25	65	2	893.7	911.74	909.99	1.75	9.15
19	275	2	30.5	65	2	893.7	911.68	909.98	1.70	9.01
20	275	3	25	65	3	893.7	912.32	910.02	2.30	10.2
21	275	3	30.5	65	3	893.7	912.15	910.02	2.13	9.96
22	300	3	30	65	3	893.7	911.80	910.00	1.80	9.17

Table FC-2 Summary of Parametric Study Results for the Diversion Channel Crossing of the Maple River

Run	Diversion Channel Width (ft)	Pier Width (ft)	Pier Spacing (ft)	Length of Diversion Channel (ft)	Thickness of Slab (ft)	Elevation of Top of Maple Structure (ft)	500-year WSEL Upstream of Structure (ft)	500-year WSEL Down-stream of Structure (ft)	500-year Head Loss Across the Structure (ft)	500-year Velocity at Structure (ft/s)	Event that Overtops the Maple Structure (year)
1	250	1	25	65	1.5	892.7	897.58	895.83	1.75	9.32	50
2	250	1	25	65	3.5	892.7	898.40	896.17	2.23	10.28	50
3	250	3	25	65	1.5	892.7	897.84	895.82	2.02	9.87	50
4	300	1	25	65	1.5	892.7	897.10	895.89	1.21	7.78	50
5	300	1	25	80	1.5	892.7	897.10	895.89	1.21	7.78	50
6	300	4	37.5	65	4	893.7	898.38	896.22	2.16	9.95	100
7	300	2	25	65	2	893.7	897.50	895.86	1.64	8.80	100
8	300	1	25	65	1.5	894.7	897.49	895.88	1.61	8.76	100
9	300	1	25	65	1.5	898.7	898.00	895.88	2.12	9.85	>500
10	300	3	25	65	1.5	892.7	897.09	895.88	1.21	7.78	50
11	300	1	25	65	3.5	892.7	897.52	895.87	1.65	8.83	50
12	300	3	25	65	3.5	892.7	897.51	895.87	1.64	8.83	50
13	350	1	25	65	1.5	892.7	896.79	895.91	0.88	6.63	100
14	350	4	43.75	65	4	892.7	897.32	895.91	1.41	8.11	50
15	350	4	43.75	65	4	893.7	897.55	895.88	1.67	8.73	100
16	350	4	43.75	80	4	893.7	897.55	895.88	1.67	8.73	100
17	350	4	43.75	65	4	894.7	897.80	895.88	1.92	9.31	100
18	275	2	25	65	2	893.7	897.78	895.85	1.93	9.55	100
19	275	2	30.5	65	2	893.7	897.73	895.86	1.87	9.44	100
20	275	3	25	65	3	893.7	898.20	895.84	2.36	10.45	100
21	275	3	30.5	65	3	893.7	898.12	895.85	2.27	10.27	100
22	300	3	30	65	3	893.7	897.82	895.87	1.95	9.51	100

F-C2.0 ALTERNATIVES MATRIX FOR DIVERSION STRUCTURES AT NORTH DAKOTA TRIBUTARIES

The following information was previously presented as Exhibit C of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

Several design concepts for the tributary crossing structures were investigated during Phase II (November and December of 2009 with the submittal on December 31, 2009) to determine if they were feasible. The advantages and disadvantages of eight (8) alternatives and the original Phase I recommended design of the tributary crossing structures for the Sheyenne, Maple, Lower Rush and Rush Rivers are presented in the following text. Alternative deigns analyzed during Phase II were intended to:

- Reduce the construction costs
- Allow fish passage
- Address winter freezing
- Address ice flow in the Diversion Channel during flood events

During Phase I, which was submitted on August 31, 2009, concept level designs of hydraulic structures for tributaries of the Red River of the North that cross the proposed Diversion Channel were developed. Crossings were developed based on the primary design constraint that the head loss across each crossing should not be greater than 0.5-ft. As a result, a closed box culvert aqueduct-type structure was recommended at each tributary crossing. However, during Phase II the primary design constraint has changed such that an increase of the water surface elevation at the inlet to the Diversion Channel in the Red River is not allowed. This shift in design criteria allows for additional alternate crossing designs that were not possible during Phase I of the study.

Additionally, during Phase I it was assumed that the Diversion Channel would be dry (i.e., the Diversion Channel would only convey flow during flood events). However, for Phase II the US Army Corps of Engineers (USACE) and the City of Fargo have concluded that the Diversion Channel will not necessarily be dry, and will almost always have some water in it. The water will most likely be due to inflows from drain tiles of surrounding fields. This allows consideration of passing less flow into the protected area. During Phase I, a minimum flow corresponding to the 5-year flood event in the tributary would be conveyed into the protected area. During the Phase II analysis in November and December of 2009, it was agreed upon with the USACE that a minimum of the 2-year event in the tributary would be conveyed into the protected area. By reducing the tributary flow conveyed into the protected area from the 5-year flow to the 2-year flow, additional flow will be directed into the Diversion Channel. This reduction in flow to the protected area will result in smaller crossing structures and a larger spillway which will reduce the overall cost of the crossing structure. Similar to the Phase I design, it was assumed that no flow from the ND Diversion Channel into the protected area would be allowed.

Following is a description of the design developed during Phase I of the study, and alternatives that were analyzed in Phase II of the study. These alternatives and combinations of these alternatives were modeled in HEC-RAS. The optimum Phase II designs are described in the text of Appendix F.

F-C2.1 PHASE I PROPOSED DESIGN – AQUEDUCT IN TRIBUTARY AND TRANSITION IN DIVERSION CHANNEL – ALL TRIBUTARIES.

The Phase I design consisted of an aqueduct represented by enclosed box culverts to pass the 5-year flow from the tributaries into the protected area over the Diversion Channel, and a weir spillway to divert waters from the tributary into the ND Diversion Channel. The total height of the box culvert is 6.25 feet with an effective flow depth of 3.5 feet. This translates to very high velocities in the tributary box culvert and a higher risk of ice jams in the tributary. This design was proposed for all five major tributary crossings: the Wild Rice River, Sheyenne River, Maple River, Lower Rush River and Rush River. At the crossing, the Diversion Channel will be concrete lined to reduce channel roughness and increase velocities to compensate for the reduction in flow area without resulting in greater water depths. The transition from the excavated Diversion Channel to the concrete lined section would have contraction and expansion zones of 2.5:1 (or 400 feet) and 4:1 (or 600 feet) respectively, with a central rectangular cross section that is wide enough to allow placement of the enclosed box culverts to pass flows from the tributary into the protected area. The central rectangular cross section in the Diversion Channel was 200 feet wide for all tributaries. The length of the central rectangular cross section in the Diversion Channel was 400 feet for the Wild Rice River and Maple River and 200 feet for the remaining three tributaries, which translates to a total length of concrete lined section in the Diversion Channel of 1400 feet or 1600 feet. Additionally, the crossing is completed with a siphon to pass winter flows in the tributary to the protected area when there is the possibility of damage to the box culverts due to freeze-thaw cycles. Figure F-C1 shows a schematic of the design submitted for Phase I of the study.

F-C2.2 ALTERNATIVE 1: PHASE I DESIGN WITH OPEN AQUEDUCT AND GATES

Alternative 1 would be similar to the Phase I design with the exception of removing the top of the box culverts that function as the aqueduct. Removing the top of the box culverts would allow mixing of flow in some of the tributaries with the diversion channel during large flow events. Flow from the aqueduct will be able to overtop into the diversion channel, and similarly flow from the diversion channel will be able to be directed into the protected area. In order to control the amount of flow that is passed into the protected area an additional set of gates will be placed on the downstream side of the tributary crossing. Figure F-C2 illustrates these modifications to the Phase I design.

During low flows this design will perform similar to the Phase I design, flow in the tributary and diversion channel will remain separate. However, during high flow events water will be able to overtop the aqueduct into the Diversion Channel. By removing the top of the aqueduct it reduces the possibility of freeze-thaw damage to the aqueduct during spring and autumn flows when the water is contained within the enclosed box culvert. By reducing this freeze-thaw risk it is possible that a siphon to convey winter flows will not be necessary. Removing the top of the aqueduct also allows for natural light into the crossing which may help encourage fish passage through the structure. Finally, it is possible that with this design a separate spillway from the tributary into the Diversion Channel will not be necessary. During high flow events on the tributary the aqueduct will overtop directly into the Diversion Channel.

However, by allowing mixing of flow some flow will be diverted from the diversion channel into the protected area. By adding gates at the downstream end of each crossing the manual labor involved during a flood event to control the flow into the protected area increases. Further analysis needs to be completed to understand how this alternative will impact winter freezing in the crossing structure and how the proposed modifications will impact ice flow in both the tributary and Diversion Channel.

F-C2.3 ALTERNATIVE 2: PHASE I DESIGN AND PASS ALL DIVERSION CHANNEL FLOW BELOW AQUEDUCT

The Alternative 2 design would be similar to the Phase I design with the exception that all of the flow in the Diversion Channel will be passed below the aqueduct which can be accomplished in one of the three ways described below.

First, the width of the Diversion Channel at the tributary crossing (L) can be increased. Increasing the width of the Diversion Channel increases the cross sectional area available to convey flow in the Diversion Channel which will theoretically reduce the water surface elevation. These modifications to the Phase I design are shown in Figure F-C3. However, due to tailwater effects from the Red River for large events, this first option is not feasible as it is not possible to significantly lower the water surface elevation in the Diversion Channel by widening it.

Second, inverts of the aqueduct can be increased to an elevation above the water surface elevation in the diversion channel. These modifications to the Phase I design are shown in Figure F-C3A.

Third, the invert of the Diversion Channel can be lowered at the tributary crossings. This could be completed by lowering the entire cross section to match the invert of a low flow channel.

Theoretically, by removing the aqueduct from intersecting the flow in the Diversion Channel, no head losses will occur at the crossing because there will not be any restrictions in the diversion channel. Additionally, this would reduce the stress applied on the aqueduct structure during high flow events by flow in the diversion channel which may reduce the cost to construct the structure. However, widening the diversion channel would require additional excavation which would increase the cost.

Increasing the aqueduct invert elevations will impact upstream flood elevations on the tributaries and possibly create ponded areas. Raising the invert of the aqueduct without a sufficient transition to the regular invert elevation of the tributary could lead to conditions that mimic a low-head dam. Additionally, increasing the inverts of the aqueduct could divert additional water from the tributaries into the Diversion Channel and result in impacting water surface elevations on the tributary upstream of the Diversion Channel. Finally, the impacts on ice flow when passing all the Diversion Channel flow below the tributary crossing is unclear and requires further analysis.

F-C2.4 ALTERNATIVE 3: ELIMINATE TRANSITION TO CONCRETE LINED SECTION

The Alternative 3 design would eliminate the transition to a concrete lined section all together. Box culverts would be placed along the bottom of the Diversion Channel to convey flow. The bottom width of the diversion channel would be increased so that all of the flow passes below the tributary crossing. Tributary flow would be conveyed over the top of the box culverts in an open natural channel. A schematic of this crossing is shown in Figure F-C4. However, due to tailwater effects from the Red River for large events, this alternative is not feasible as it is not possible to significantly lower the water surface elevation in the Diversion Channel by widening it.

If this option were feasible, the following items would have been taken into account when deciding on a Phase II design: (a) replacing the aqueduct with an open channel reduces the possibility of freeze-thaw damage during spring and autumn flows when the water is contained within the enclosed box culvert, (b) reducing the risk of freeze-thaw allows for the possibility that a siphon to convey winter flows will not be necessary, (c) removing the concrete lined channel and siphon may result in reduced construction costs, (d) providing a tributary crossing with a natural bottom (i.e. earth bottom rather than concrete lined) may encourage fish passage, and (e) providing an open top may also better handle ice flows than the current design.

Replacing the concrete lined channel with an earth berm and box culverts would also likely result in an increase in head loss across the crossing. It is unclear how wide the diversion channel would need to be to avoid impacts to the water surface elevation at the inlet to the Diversion Channel. Finally, similar to Alternative 2 it is unclear how well this crossing will handle ice flows in the Diversion Channel because all of the flow is conveyed below the tributary crossing.

F-C2.5 ALTERNATIVE 4: PHASE I DESIGN & REPLACE LONG CONCRETE TRANSITION WITH VERTICAL WALLS

The Alternative 4 design would be similar to the Phase I design with the exception of removing the concrete lined transitions upstream and downstream of the tributary crossing and replacing them with radial vertical walls or with tapered vertical wing walls. Figure F-C5 illustrates the radial vertical walls on the Phase I design. The primary benefit of removing the concrete lined transitions include a reduction of the length of concrete lined channel at each crossing which should ultimately result in reduced construction costs. However removing the transitions will also reduce the hydraulic efficiency of the crossing.

Overall, if the tributary is kept in a box culvert closed aqueduct, similar to the Phase I design, it is anticipated that it will handle ice flows and winter freezing similar to the Phase I design. Additionally if the aqueduct is not modified from the Phase I design it is anticipated that this type of crossing will have similar benefits and restrictions to fish passage as the Phase I design.

F-C2.6 ALTERNATIVE 5: PHASE I DESIGN WITH TALLER BOX CULVERTS

The Alternative 5 design would be similar to the Phase I design with the exception of increasing the height of the box culverts used for the aqueduct. Increasing the height of the boxes will increase the hydraulic efficiency of the crossing. Figure F-C6 illustrates these modifications to the Phase I design. With a taller box culvert the required width of the tributary crossing (X2) can be reduced. Consequently, reducing the width of the tributary crossing will correspond to an overall reduction to the required length of concrete lined channel at each crossing. Additionally, it is anticipated that the larger opening will encourage better fish passage through the structure when compared to the Phase I design, however the flow is still constrained in a closed box culvert and during high flows high velocities are still anticipated. Finally, it is also anticipated that the larger openings will facilitate ice flow in the tributaries during winter months.

Consequently, the additional box culvert height will reduce the cross sectional area of the diversion channel increasing the head loss at each crossing. However, for Phase II design, the design constraint at each crossing is to not increase the water surface elevation in the Red River at the inlet to the diversion (rather than the Phase I constraint of only allowing 0.5-ft of head loss at each crossing), and it is possible that increasing the height of the aqueduct will result in increasing the head loss across an individual crossing while not impacting the water surface elevation at the inlet to the diversion channel.

F-C2.7 ALTERNATIVE 6: DIVERT RUSH AND LOWER RUSH DIRECTLY INTO DIVERSION CHANNEL

The Rush River and Lower Rush River will be diverted directly into the Diversion Channel. The portions of these Rivers downstream of the Diversion Channel are primarily straight drainage channels and do not display many characteristics typically associated with natural streams. By diverting these two channels directly into the Diversion Channel, two crossing structures are eliminated from the overall design thereby reducing the overall costs. Additionally, removing the crossing structures will likely facilitate the handling of ice flows in both the Diversion Channel and tributaries. Fish passage structures could also be designed for these tributaries. Downstream of these Rivers, habitat enhancements and low flow channel meandering could be implemented, increasing the quality and quantity of habitat in these Rivers, when compared to existing conditions.

F-C2.8 ALTERNATIVE 7: REDUCE FLOWS INTO THE PROTECTED AREA

By reducing the tributary flow conveyed into the protected area from the 5-year flow to the 2-year flow additional flow will be directed into the Diversion Channel. This reduction in flow will result in smaller crossing structures and a larger spillway which will reduce overall cost of the crossing structure.

F-C2.9 ALTERNATIVE 8: PHASE I WITH SIDE CHANNELS AT MAPLE CROSSINGS (“BITTNER ALTERNATIVE”)

This alternative was originally proposed by Mark Bittner from the City of Fargo. The “Bittner Alternative” as understood by Barr, is described below and shown in Figures F-C7A and F-C7B.

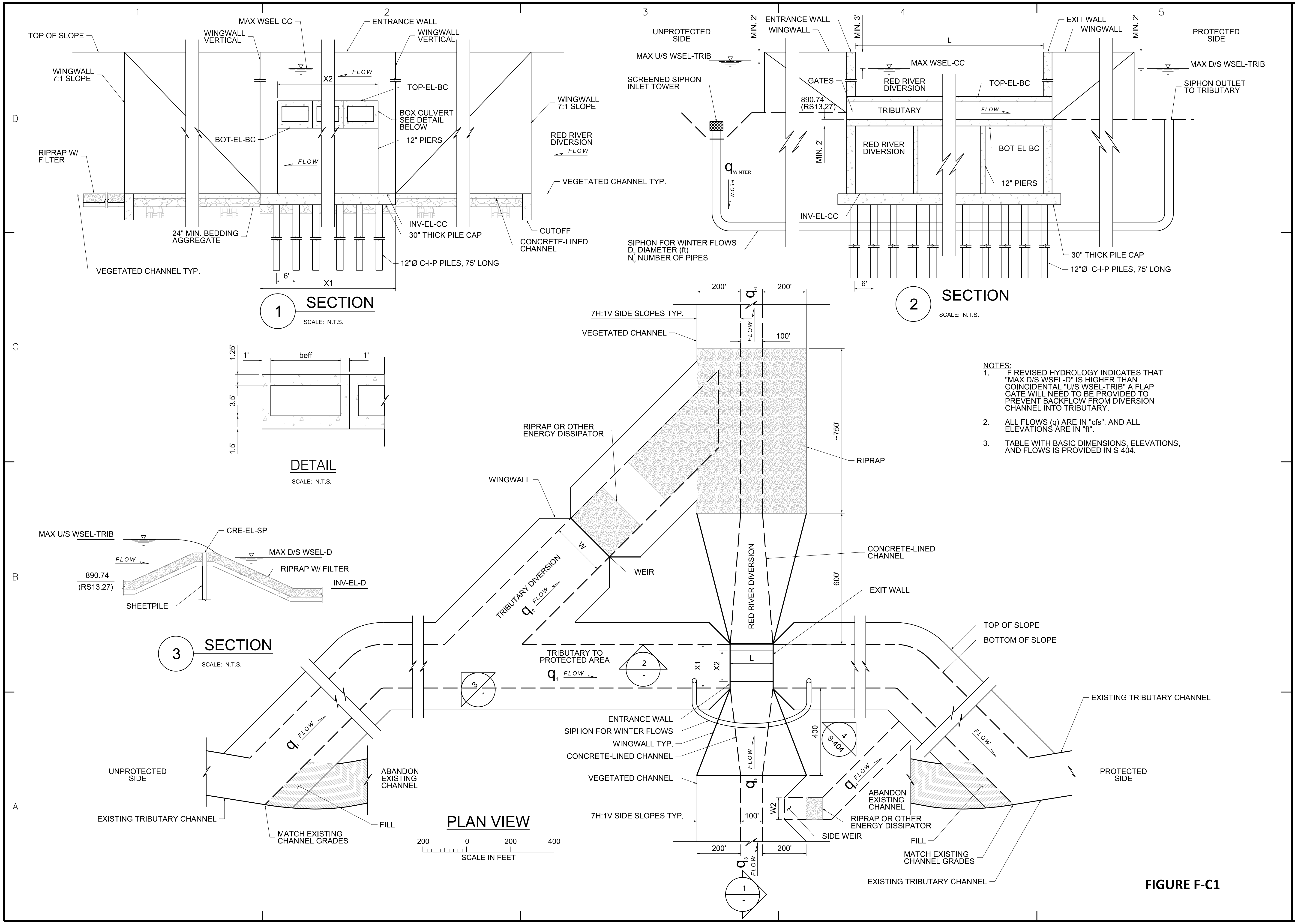
The invert elevation of the Maple River crossing of the ND Diversion Channel would be identical to the natural channel bottom on either side of the ND Diversion Channel (i.e. the bottom would not be raised as had been done in Alternative 2 in which the tributary crossings were modified to pass all flow below the aqueduct). The Maple River crossing would be a rectangular channel with a bottom width of 105 feet. The sides of the channel would be approximately 20 feet tall to prevent the Maple River 2-year flow overflowing into the ND Diversion Channel and to prevent the Diversion Channel from overtopping the Maple River Crossing during high flow events in the ND Diversion Channel. In order to route the high flows in the ND Diversion Channel, an additional channel would be constructed to the east side (protected area side) of the ND Diversion. The second channel would have the same bed slope as the ND Diversion Channel but its invert elevation would be the same as the Maple River at the Maple River Crossing. Three gates would be installed to control the flow of water in the second channel. One gate would be installed in the Maple River to control the amount of flow passing into the protected area. The other two gates would be installed in the second channel on either side of the Maple River to keep the 2-year flow within the Maple River Channel. The gates could then be raised for larger events (such as the 100-year event or greater) which require additional capacity from the second channel. Therefore, for this Alternative 8, no weir spillway located on the west side (unprotected side) of the ND Diversion to divert water from the tributary to the Diversion Channel is needed.

A summary of the efforts were presented to Mr. Bittner. His response was that he was thinking of a slightly different alignment, shown in Figures F-C8A through F-C8D. This configuration would include low flow culverts under the tributary crossings for the Maple and Sheyenne Rivers. The tributaries would remain an open aqueduct. The Diversion Channel would not narrow as it approaches the tributary to allow for a longer weir length as high flows in the Diversion Channel overtop the aqueduct.

The Bittner alternative as initially understood by Barr would be effective in conveying the Diversion Channel flow pass the Maple River without raising the water surface in the RRN beyond the acceptable amount; however the configuration of three gates was too costly. The reconfigured Bittner alternative raised water surface elevations in the RRN by too significantly to be a feasible alternative.

F-C2.10 CONCLUSIONS

In conclusion, the modified Phase II design constraints allowed for further analysis of the selected crossing in Phase I as well as consideration of additional crossing alternatives. Alternative designs discussed above were intended to reduce project cost, allow fish passage, and address winter freezing and ice flow. However, not every alternative design discussed provided an improvement over the Phase I design for all of the design considerations. During the modeling process, all alternatives were modeled in HEC-RAS. The optimum Phase II designs for the Sheyenne, Maple, Lower Rush and Rush Rivers crossing with the ND Diversion Channel are described in Appendix F.



NOTES:

1. IF REVISED HYDROLOGY INDICATES THAT "MAX D/S WSEL-D" IS HIGHER THAN COINCIDENTAL "U/S WSEL-TRIB" A FLAP GATE WILL NEED TO BE PROVIDED TO PREVENT BACKFLOW FROM DIVERSION CHANNEL INTO TRIBUTARY.
2. ALL FLOWS (q) ARE IN "cfs", AND ALL ELEVATIONS ARE IN "ft".
3. TABLE WITH BASIC DIMENSIONS, ELEVATIONS, AND FLOWS IS PROVIDED IN S-404.

FIGURE F-C1

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FLOOD CONTROL-HYDRAULIC
STRUCTRUE ON WILD RICE RIVER (ND)
PLAN VIEW AND SECTIONS

Sheet
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number:
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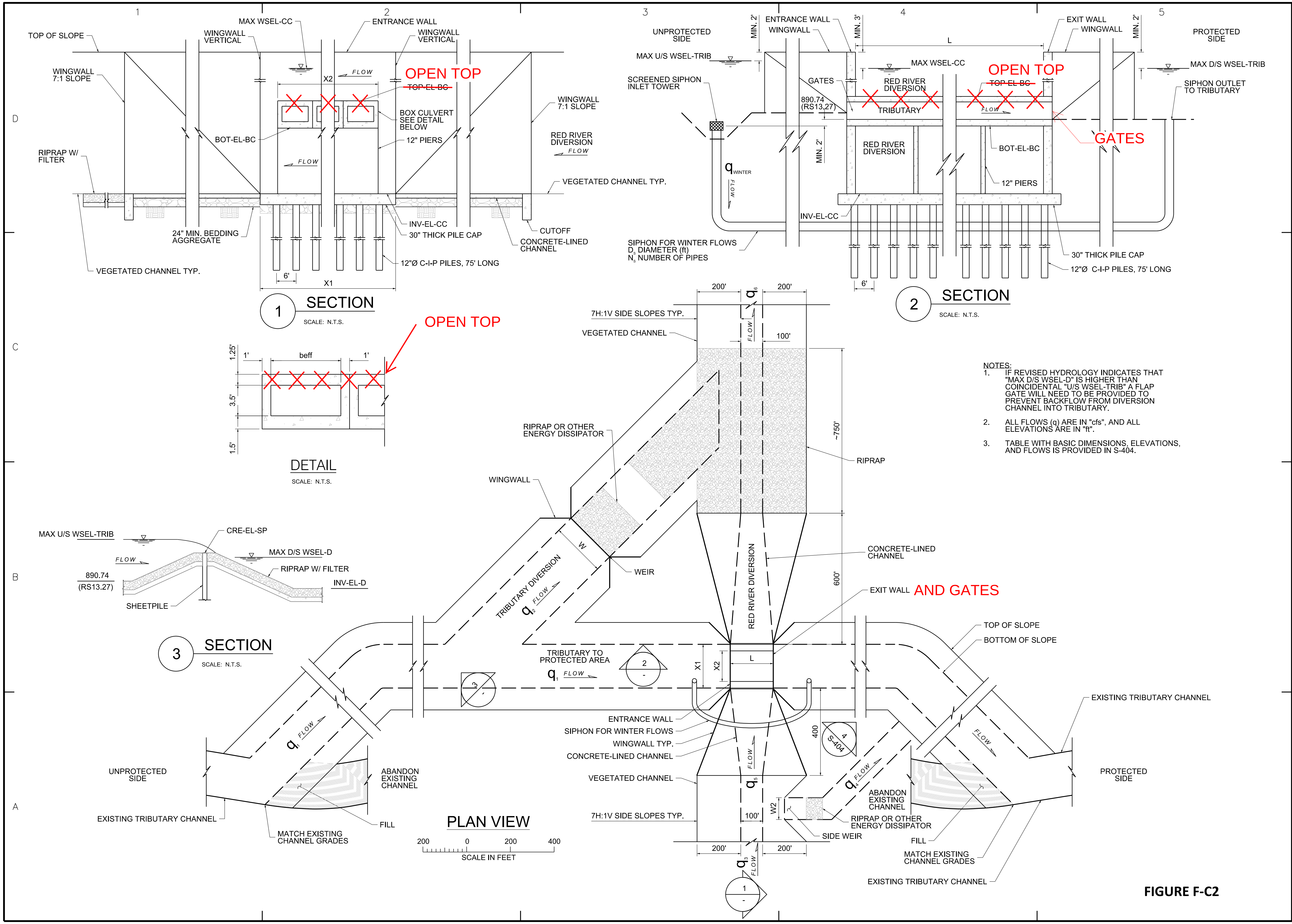

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FIGURE F-C2

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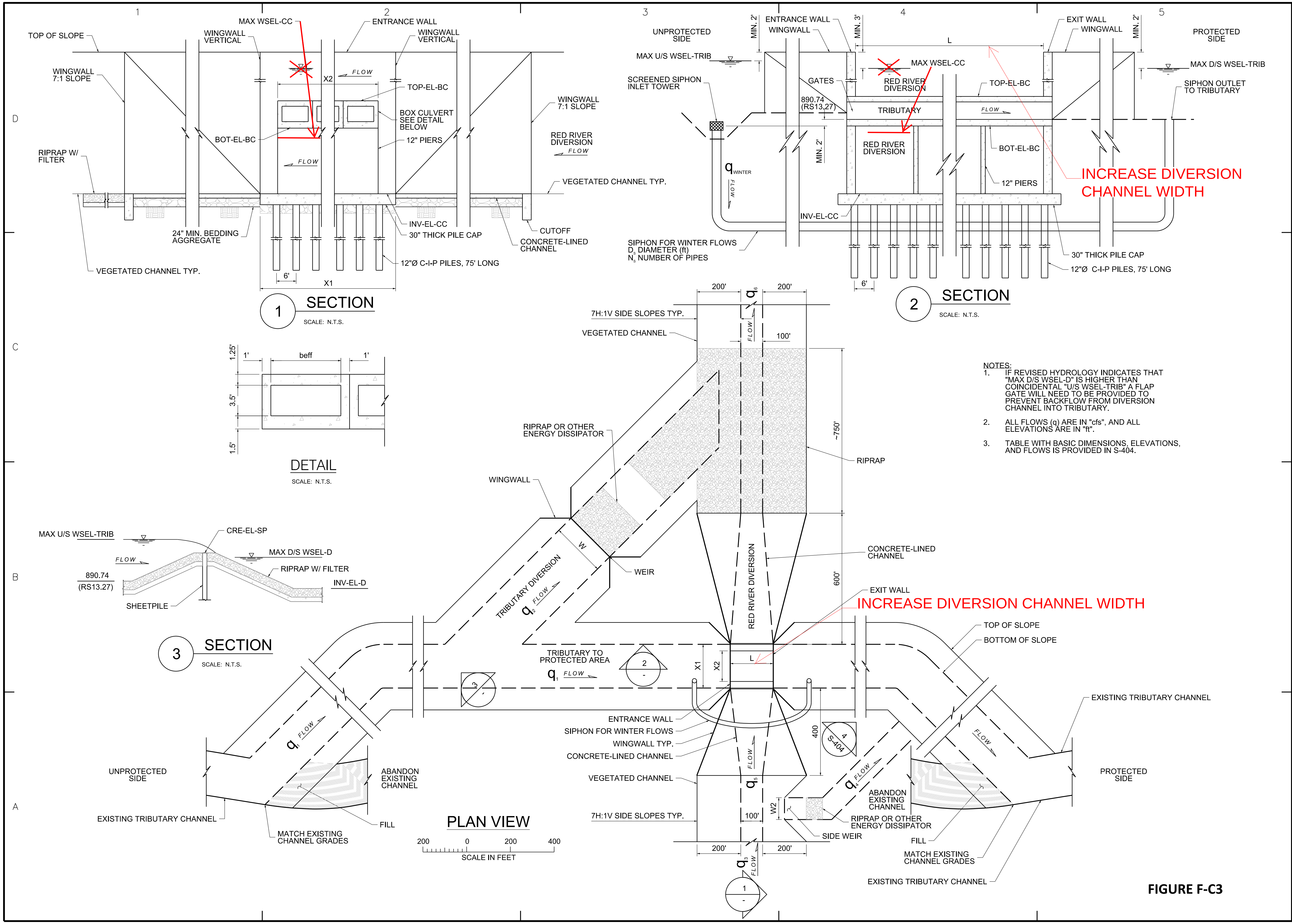

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FIGURE F-C3

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PLAN VIEW AND SECTIONS

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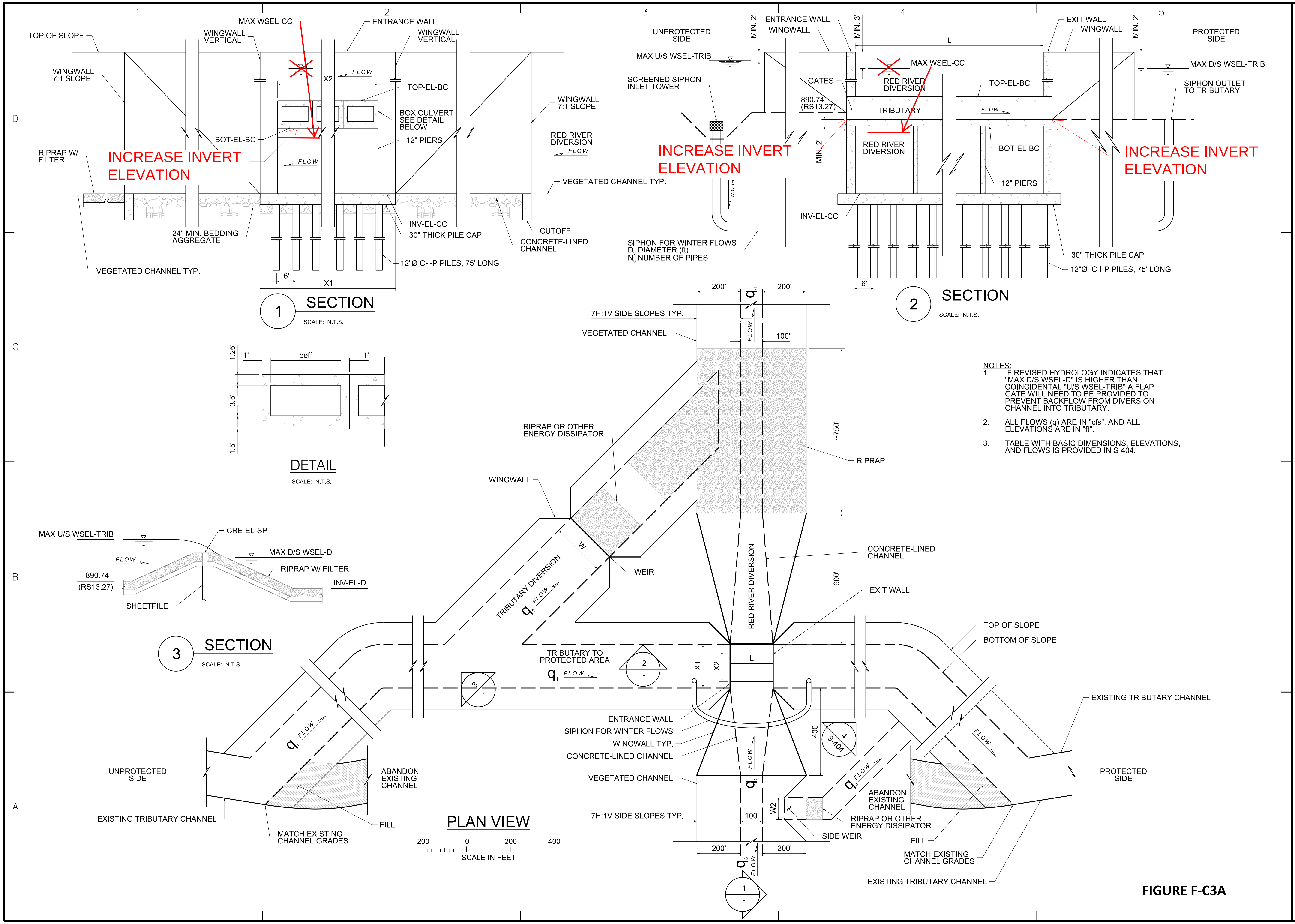

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FIGURE F-C3A

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STRUCTRUE ON WILD RICE RIVER (ND)
PLAN VIEW AND SECTIONS

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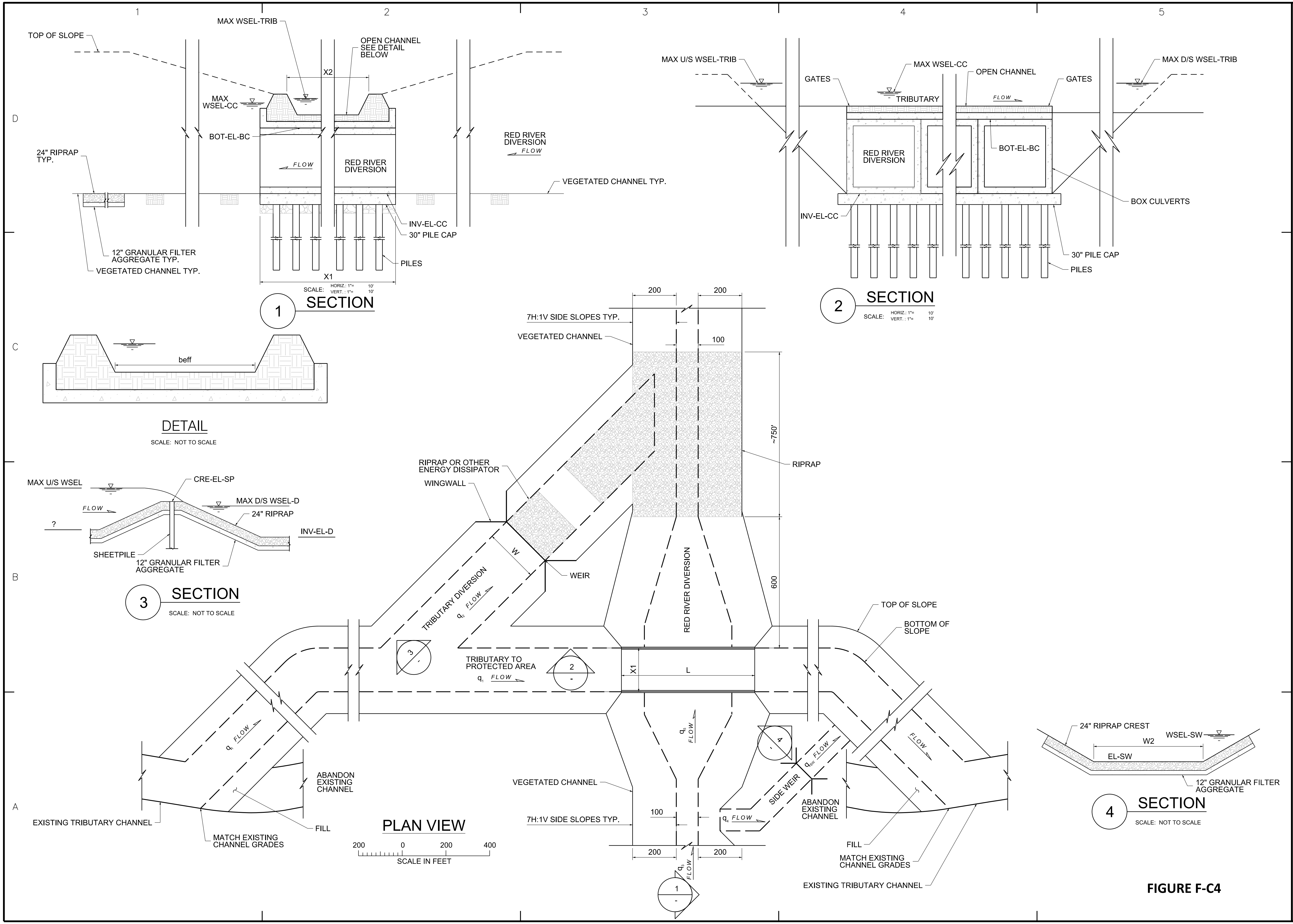

FIGURE F-C4

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DOCUMENT TYPE
PROJECT NAME (& STAGE)
RIVER BASIN - LOCATION (OPTIONAL)
LOCATION (CITY/STATE)

FLOOD CONTROL
TRIBUTARY CROSSING
PLAN VIEW AND SECTIONS

Sheet
reference
number:
X-000
Sheet x of x

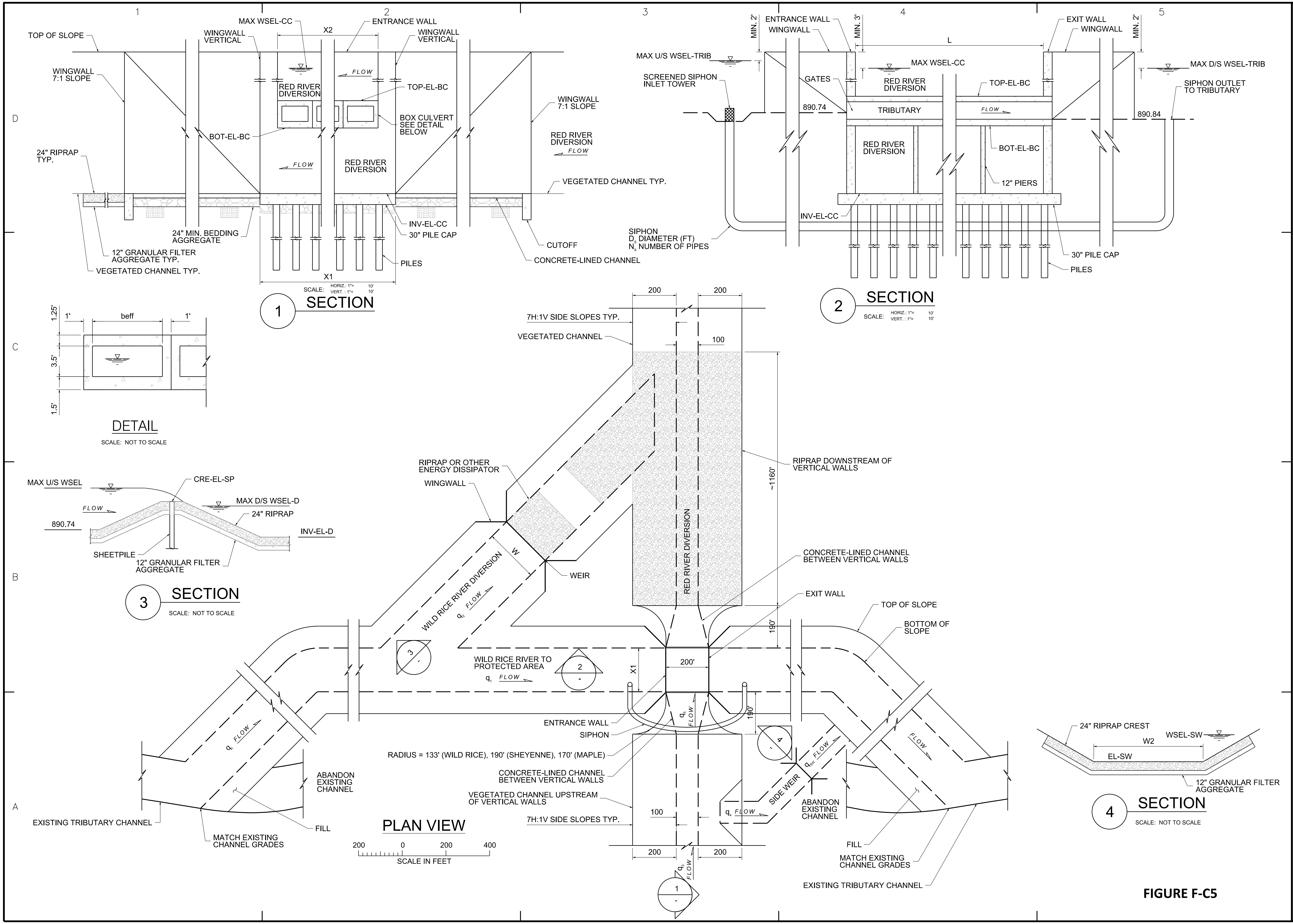

FIGURE F-C5

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Date: DY MON YR
Rev. XX
Solicitation No.: DACW37-YR-B-0000
Drawing Number: R-PR-P-CD/000
Plot Date:

DOCUMENT TYPE
PROJECT NAME (& STAGE)
RIVER BASIN - LOCATION (OPTIONAL)
LOCATION (CITY/STATE)

FLOOD CONTROL
WILD RICE RIVER CROSSING
PLAN VIEW AND SECTIONS

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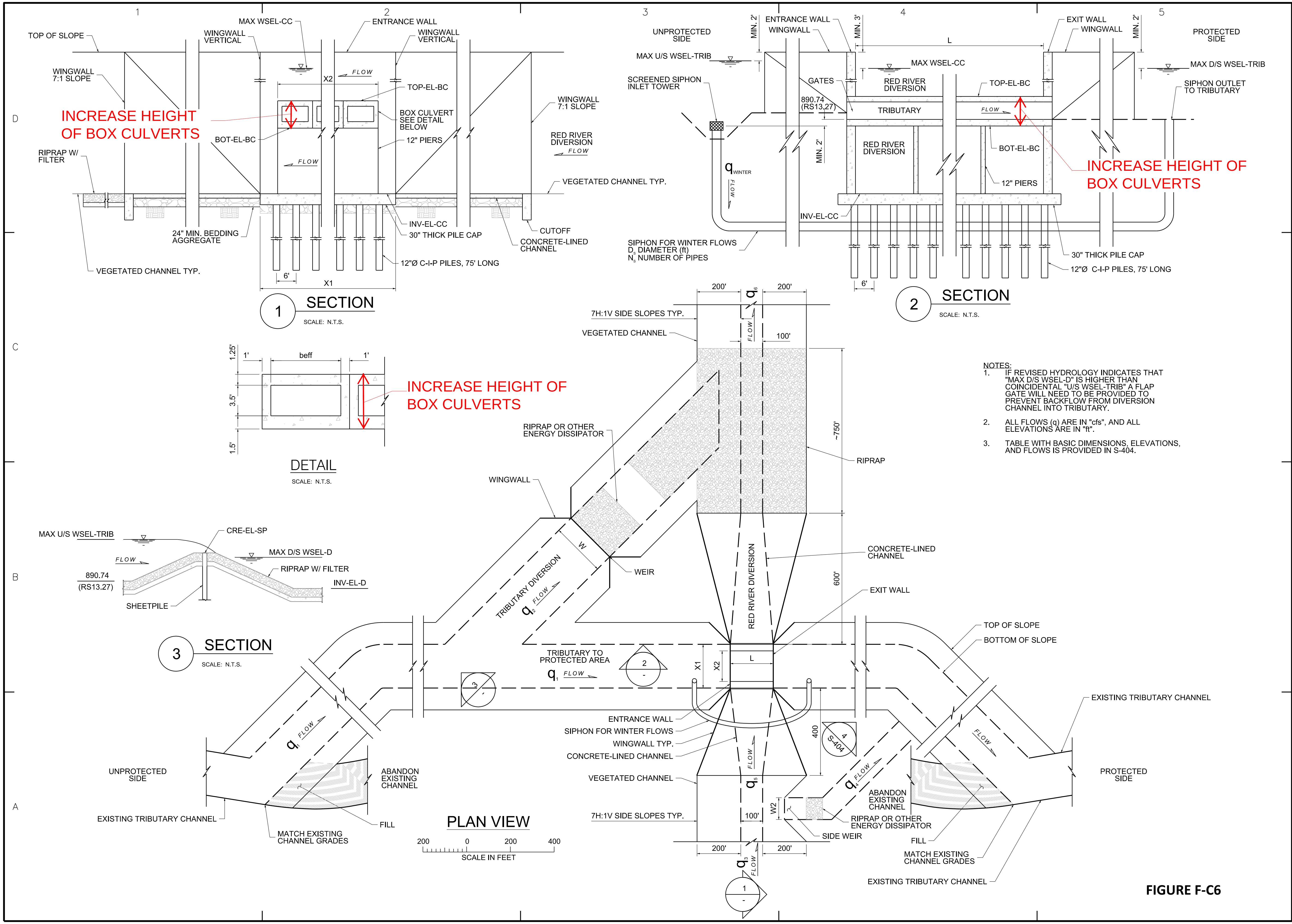

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FIGURE F-C6

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CONCEPT DRAWING
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BARR

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DULUTH, MINNESOTA
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

DATE 12/17/09

SHEET NO. ①

PROJECT NAME Fargo By-pass

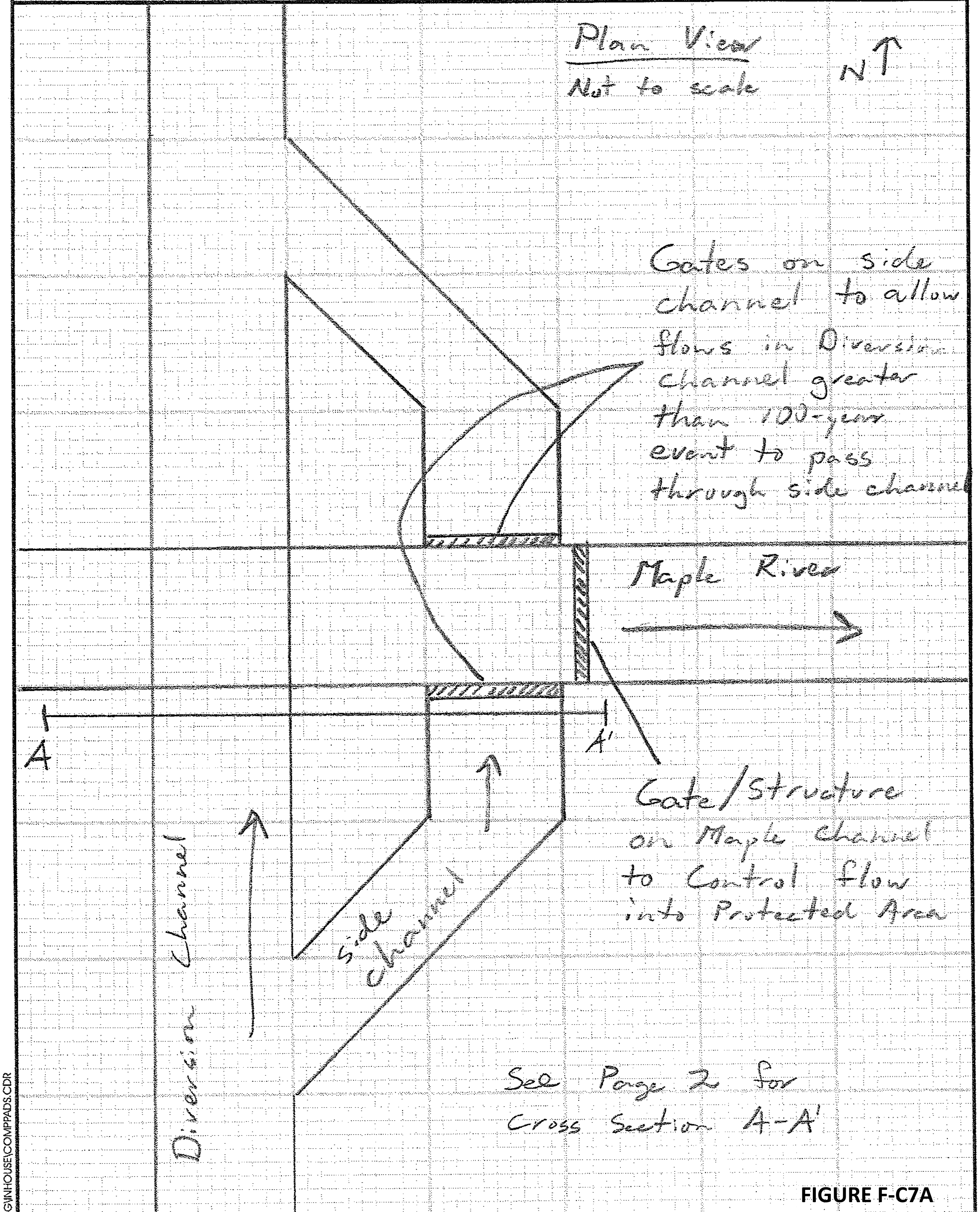
PROJECT NUMBER 34-09-1004

COMPUTED	CHECKED	SUBMITTED
BY JDW	BY	TO
DATE	DATE	DATE

SUBJECT Maple crossing of Diversion Channel

Plan View

Not to scale



Gates on side channel to allow flows in Diversion channel greater than 100-year event to pass through side channel

Gate/Structure on Maple channel to Control flow into Protected Area

See Page 2 for Cross Section A-A'

FIGURE F-C7A

H:\RLG\INHOUSE\COMPPADS.CDR

10 Squares per Inch

REV.12/1/98

BARR

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DULUTH, MINNESOTA
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

DATE 12/17/09

SHEET NO. 2

PROJECT NAME Fargo By-pass

PROJECT NUMBER 34-09-1004

SUBJECT Maple River Crossing

COMPUTED	CHECKED	SUBMITTED
BY JDW	BY	TO
DATE	DATE	DATE

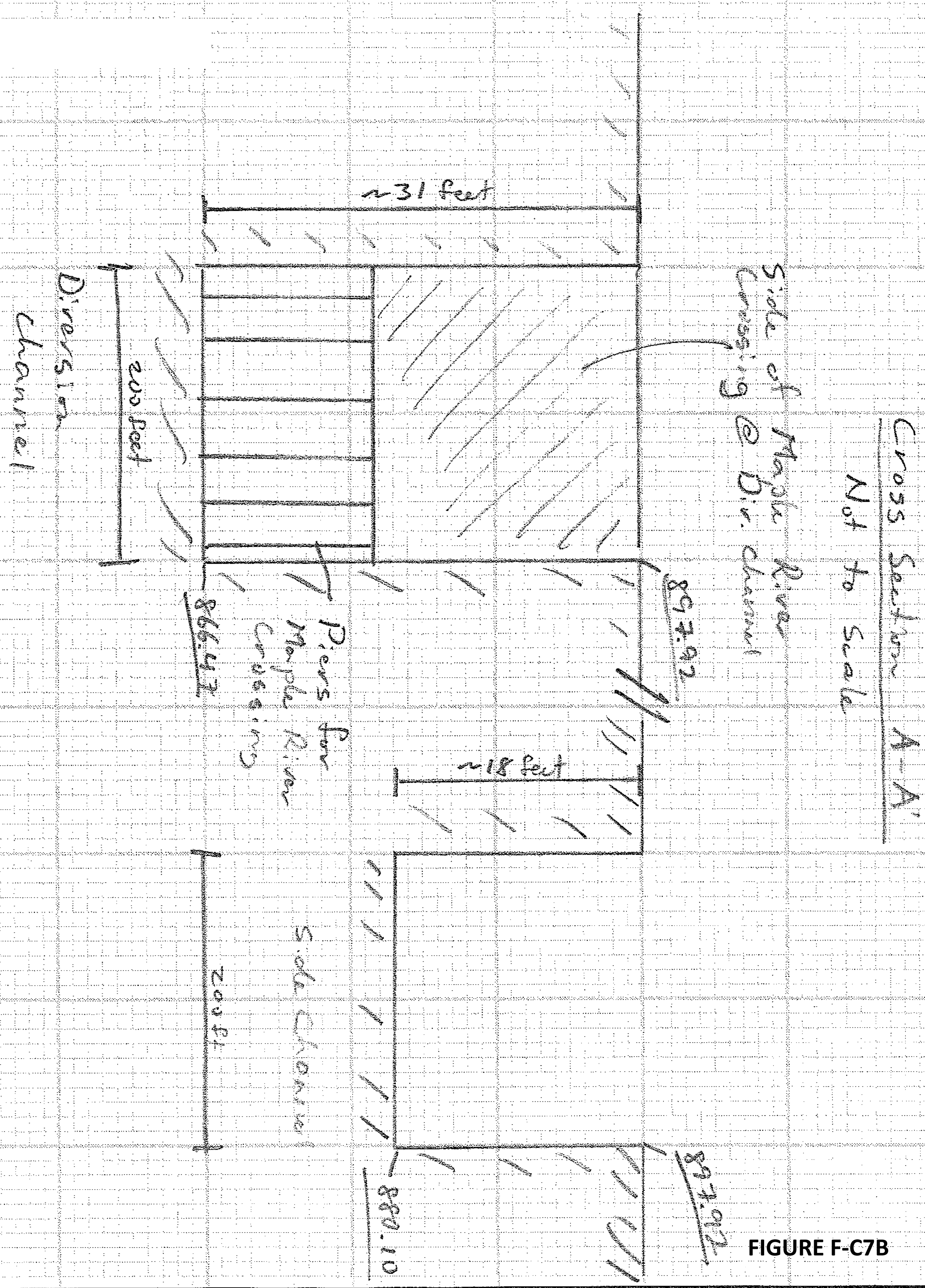


FIGURE F-C7B

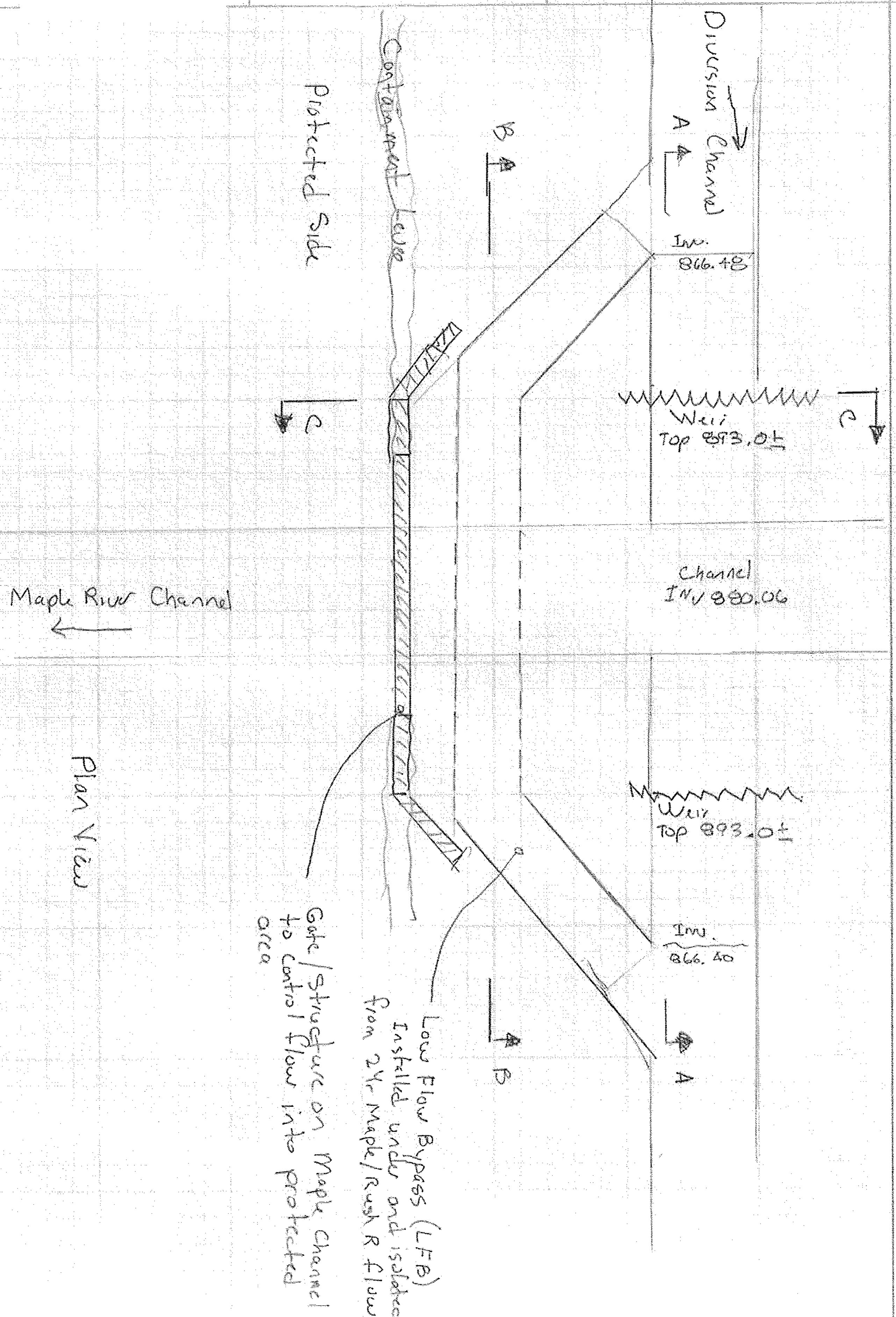
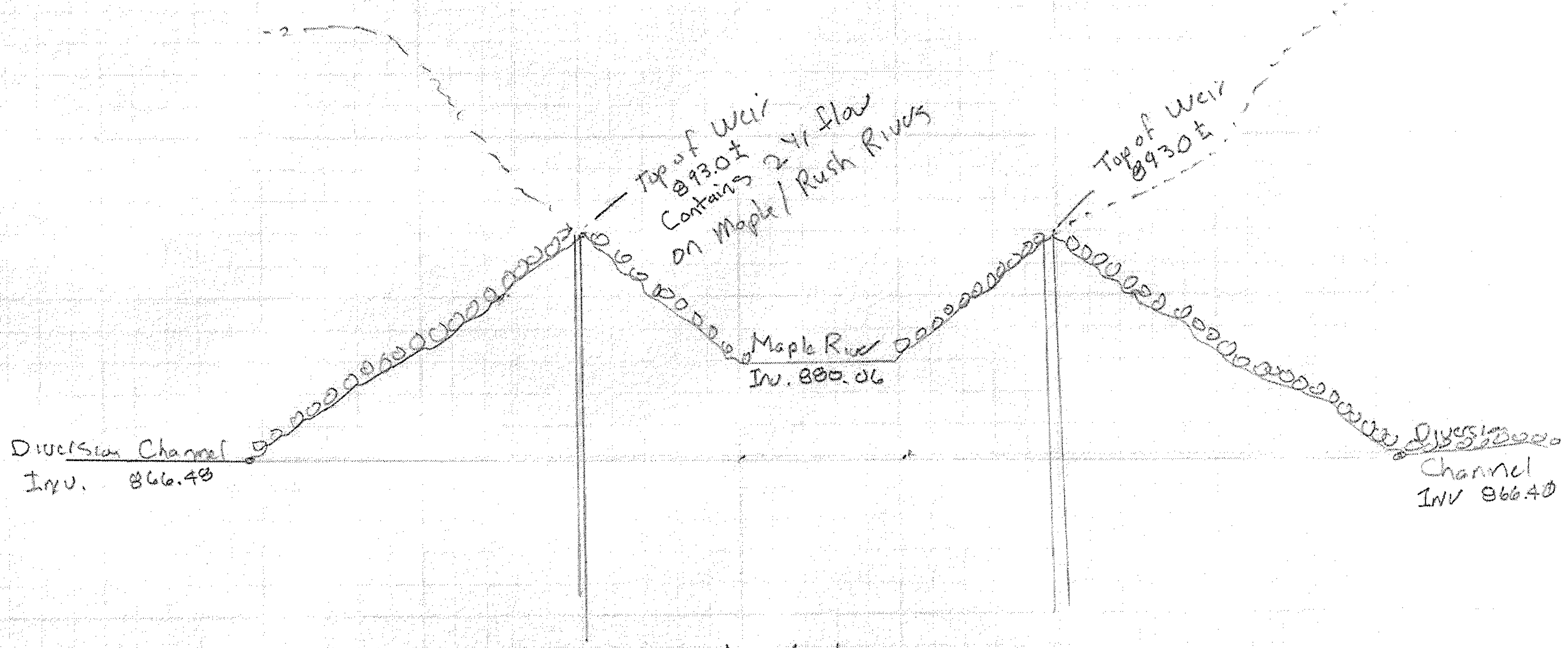
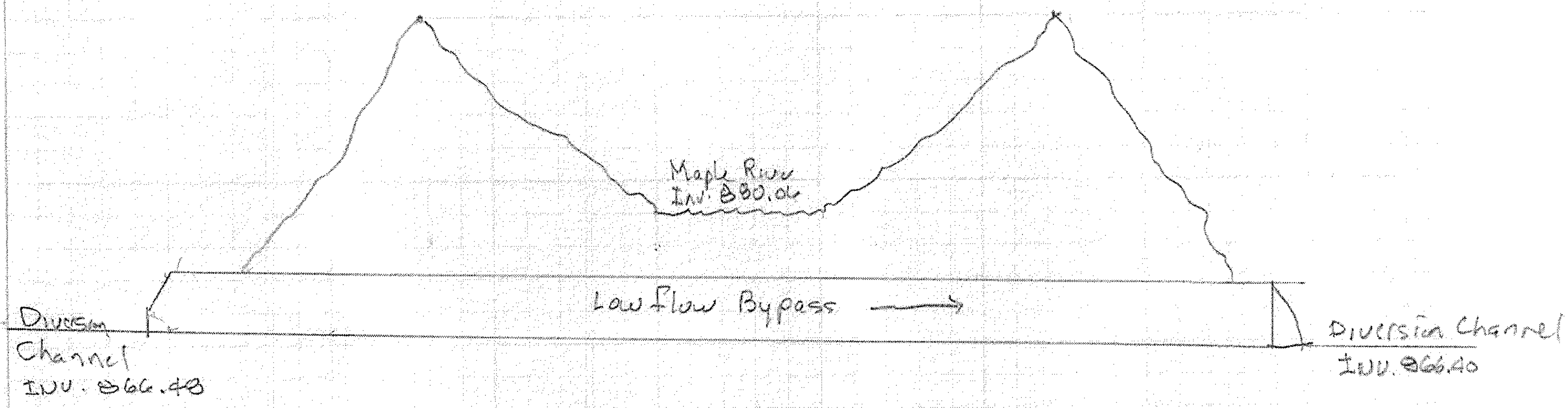

Diversion Channel
A
A
B
B
C
C
Containment Levee
Protected Side
Inv. 866.48
Weir Top 893.0±
Channel INV 880.06
Maple River Channel
Weir Top 893.0±
Inv. 866.40
Plan View
Low Flow Bypass (LFB) Installed under and isolated from 2Yr Maple/Rush R flows
Gate/Structure on Maple Channel to control flow into protected area

FIGURE F-C8A



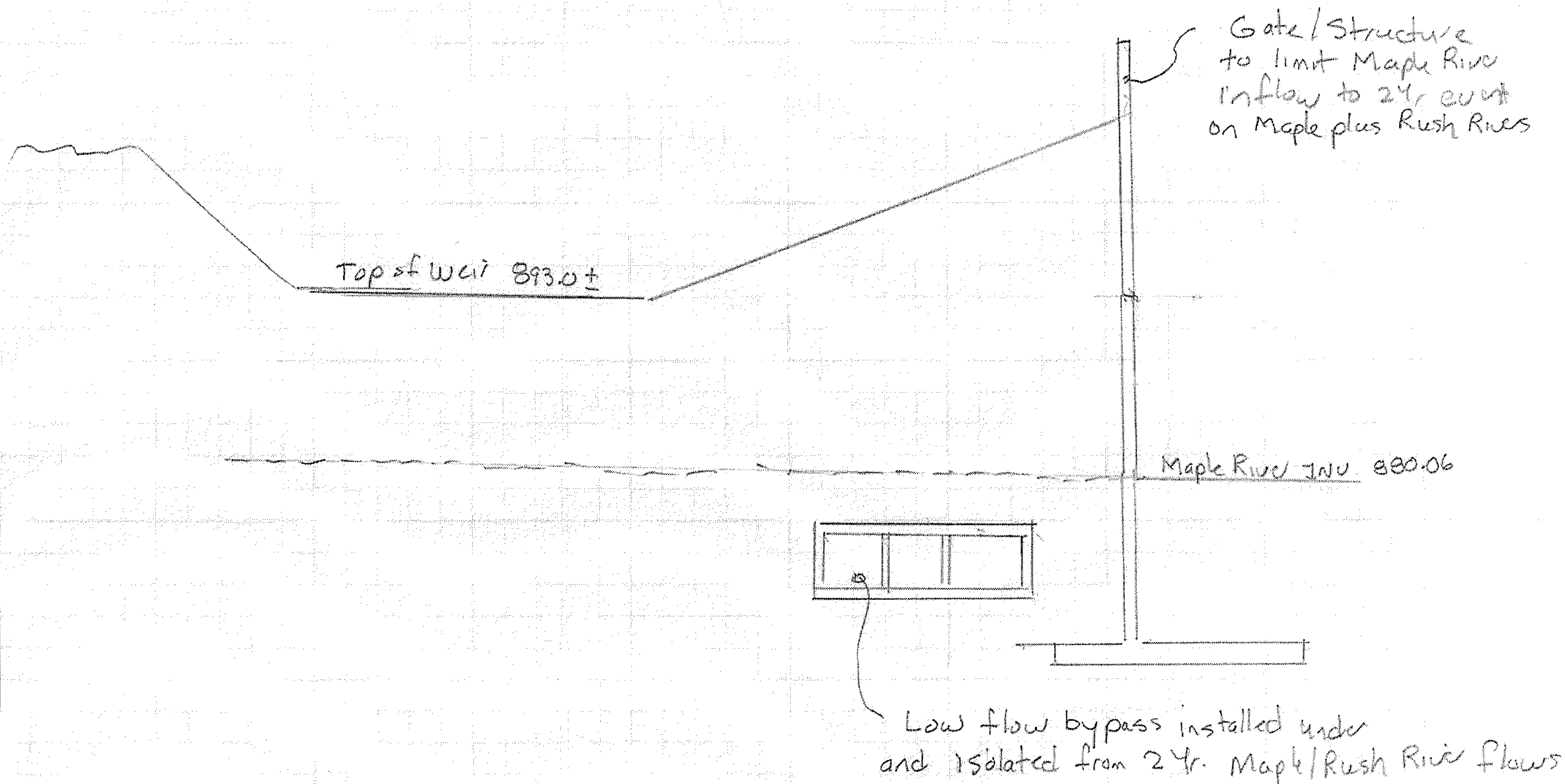
Section A-A

FIGURE F-C8B



Section B-B

FIGURE F-C8C



Section C-C

FIGURE F-C8D

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT D– HYDRAULIC DESIGN COMPUTATIONS

Report for the US Army Corps of Engineers, and the Cities of Fargo, ND and Moorhead, MN

By: Barr Engineering Co.

FINAL –February 28, 2011

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HYDRAULIC STRUCTURES

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F-D1.0 FEDERALLY COMPARABLE PLAN (FCP)

The following information regarding the FCP was previously presented as part of Appendix F Exhibit D in the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

F-D1.1 CONTROL STRUCTURE ON RED RIVER OF THE NORTH

A description of the FCP control structure on the Red River of the North can be found in Appendix F. Appendix F, Exhibit B discusses the variety of structure types that were investigated for this control structure during the Phase 2 design. This portion of Exhibit D presents design concepts and calculations for the Red River of the North control structure, detailed HEC-RAS modeling outputs at the control structure, and comparisons of water surface elevations at the control structure using Phase 3 hydrology.

F-D1.1.1 Flow Partitioning

The partitioning of the flow between the protected area downstream of the Red River Control Structure and the Diversion Channel was performed by Moore Engineering in HEC-RAS using preliminary designs of the Red River Control Structure and Diversion Inlet Structure. The resulting flows are presented in Table F-D1, Table F-D2, and Table F-D3 for the Year 0, Year 25, and Year 50 hydrology, respectively. A graphical representation of these results, along with a tabular summary, is presented in Appendix F. The flows presented in Appendix F include the 2-year, 5-year, 10-year, 20-year, 50-year, 100-year, 200-year, and 500-year events on the Red River of the North for the Year 0, Year 25, and Year 50 hydrology.

F-D1.1.2 Gate Number and Sizing – Concepts

The number and sizing of the gates in the Red River Control Structure is driven by the ability of the design to achieve the desired flow through the gates as determined by Moore Engineering for each given flood event without increasing or decreasing the upstream water surface elevation. Initially, three concepts which included operable gates above permanently open areas were considered and presented to the U.S. Army Corps of Engineers (USACE) during a meeting on December 2, 2009. Those concepts are shown in Exhibit B of Appendix F and include a design with three identical gates, a design with five identical gates, and a design with two large identical gates combined with two smaller gates.

Based on discussion with the USACE and preliminary modeling of these concepts during Phase 2, the concept with 3 identical gates was selected for further evaluation. This selection was made because it allowed a wider gate width (for ice and boat passage) and redundancy of gates for operation and maintenance issues.

F-D1.1.3 Gate Number and Sizing – Optimization

After the three identical gate concept was selected for further analysis, optimization of the gate number and sizing was performed using the orifice equation during Phase 2 (Exhibit C of Appendix F in the January 6, 2010 submittal). For Phase 3, a HEC-RAS model developed by Moore Engineering was used to further refine the optimization of gate number and sizing. During Phase 3, following the recommendation from USACE – Environmental, a width of 50 feet was selected for each of the three gates. Preference was given for multiple, identically sized gates for operation and maintenance purposes. Also, it is desirable to have as little impact on the smaller, higher frequency, flood events (i.e. no change in WSEL or gate operation required); thus, a minimum open width is necessary to avoid flow constriction during smaller flood events.

The 500-year event (during which the flow through the structure is the greatest) results in the greatest open area and therefore can be used to determine the minimum number of gates required. For all smaller flood events, the gates may be partially or fully closed to achieve the desired flow into the protected area. The gate height is a user-defined variable in this process. In optimizing the design of the Red River Control Structure for the FCP alignment, 49-feet tall openings were used for the 35,000 cfs diversion. The number of gates necessary for the FCP (as determined using the above process) is three 50-feet wide gates. The height of the gates is 47 feet. The height of the ungated, open area below the gates is 2 feet. Thus, the maximum open height is 49 feet when the gates are completely open. A concept drawing for the control structure is presented in Figure F-D1.

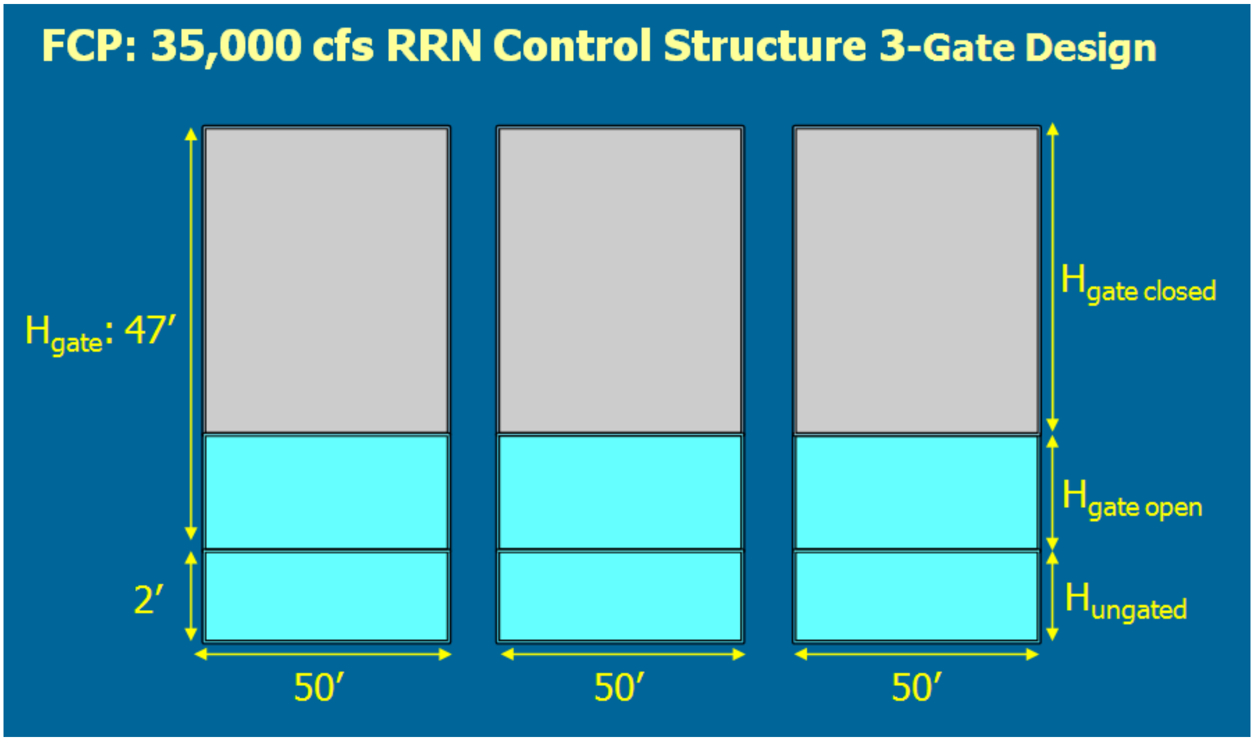


Figure F-D1 FCP Red River of the North Control Structure 3-Gate Design

F-D1.1.4 Gate Operation Optimization and HEC-RAS Results

After the gate openings were estimated as described above, the HEC-RAS model developed by Moore Engineering was used to fine-tune the gate openings for the different flow events. Flows, water surface elevations, head difference, gate opening height and average velocities for varying flood events at the FCP Red River of the North control structure are presented for Year 0, Year 25 and Year 50 hydrology in Tables F-D1, F-D2 and F-D3, respectively. These variables were obtained from the HEC-RAS models with the exception of average velocity through the gates. HEC-RAS does not provide this variable so it was calculated by dividing the flow by the flow area. The location of the control structure in the FCP HEC-RAS model is inline structure station 467. Figures F-D2, F-D3, and F-D4 show the comparison of the headwater and tailwater elevations for various flood events at the FCP Control Structure for the Year 0, Year 25, and Year 50 hydrology, respectively.

Table F-D1 HEC-RAS Outputs for the Proposed FCP Control Structure at the Red River of the North for Year 0 Hydrology

Event (yr)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Red River* (ft)	Proposed Tailwater Elevation in Red River* (ft)	Head Diff-erence (ft)	Gate Open Height (ft)	Average Velocity through Gates (ft/s)
99.99%	50	50	878.94	878.93	0.01	49	0.02
99%	251	251	879.61	879.6	0.01	49	0.11
Winter Mean	255	255	879.62	879.61	0.01	49	0.12
Median	360	360	879.97	879.96	0.01	49	0.16
September Mean	370	370	880.00	879.99	0.01	49	0.16
95%	550	550	880.59	880.58	0.01	49	0.24
90%	829	829	881.51	881.5	0.01	49	0.33
80%	1,360	1,360	883.17	883.16	0.01	49	0.50
April & May Mean	1,656	1,360	884.11	884.09	0.02	49	0.58
2	5600	5,600	892.51	892.48	0.03	49	1.36
5	12,150	9,640	901.18	897.97	3.21	5.55	11.8
10	17,000	9,654	904.53	898.28	6.25	4	16.4
20	22,000	9,759	906.86	898.6	8.26	3.52	18.7
50	29,300	9,803	909.5	898.88	10.62	2.7	24.6
100	34,700	10,929	910.82	899.97	10.85	3	24.5
200	46,200	16,367	912.53	904.01	8.52	5.8	17.6
500	61,700	26,938	913.61	909.5	4.11	13.5	13.1

* Headwater at Station 467.1 and Tailwater at Station 466.9

Table F-D2 HEC-RAS Outputs for the Proposed FCP Control Structure at the Red River of the North for Year 25 Hydrology

Event (yr)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Red River* (ft)	Proposed Tailwater Elevation in Red River* (ft)	Head Diff-erence (ft)	Gate Open Height (ft)	Average Velocity through Gates (ft/s)
2	4,352	4,352	890.37	890.35	0.02	49	1.14
5	10,608	9,640	899.64	897.9	1.74	7.5	8.6
10	15,394	9,647	903.49	898.21	5.28	4.35	14.8
20	20,345	9,741	906.18	898.54	7.64	3.65	17.8
50	27,441	9,845	909.04	898.88	10.16	2.85	23.0
100	32,921	9,918	910.61	899.07	11.54	2.5	26.4
200	42,242	14,209	912.06	902.53	9.53	4.7	20.2
500	57,641	22,686	913.75	907.72	6.03	9.5	15.9

* Headwater at Station 467.1 and Tailwater at Station 466.9

Table F-D3 HEC-RAS Outputs for the Proposed FCP Control Structure at the Red River of the North for Year 50 Hydrology

Event (yr)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Red River* (ft)	Proposed Tailwater Elevation in Red River* (ft)	Head Diff-erence (ft)	Gate Open Height (ft)	Average Velocity through Gates (ft/s)
2	3,506	3,506	888.7	888.68	0.02	49	0.99
5	9,161	9,161	897.39	897.33	0.06	44.1	1.4
10	13,965	9,546	902.66	898.06	4.6	4.6	13.8
20	18,855	9,588	905.56	898.35	7.21	3.7	17.3
50	25,764	9,649	908.31	898.67	9.64	2.95	21.8
100	31,304	10,176	910.03	899.32	10.71	2.8	24.2
200	38,787	12,071	911.66	900.92	10.74	3.4	23.7
500	54,034	19,605	913.32	906.04	7.28	7.5	17.4

* Headwater at Station 467.1 and Tailwater at Station 466.9

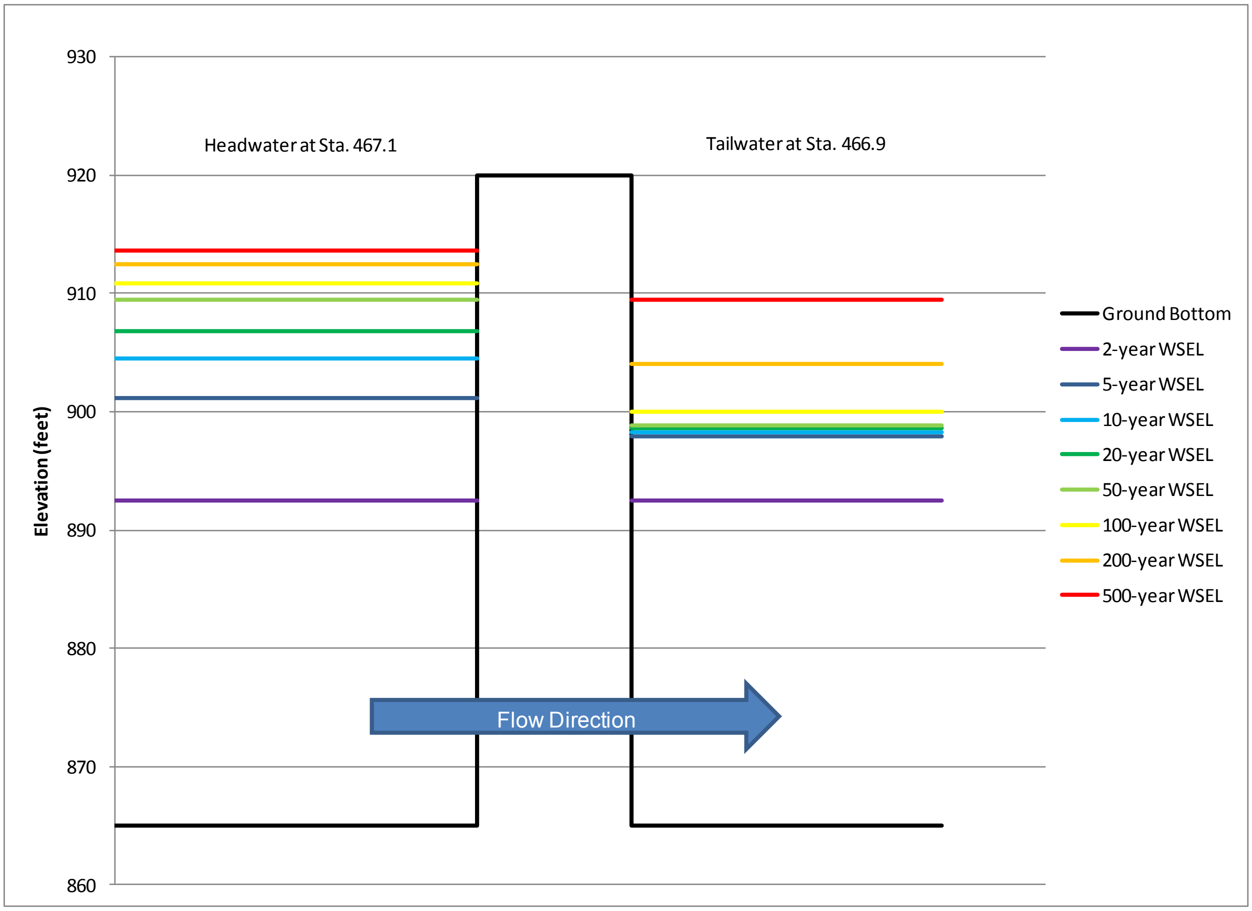


Figure F-D2 **Comparison of the Headwater and Tailwater Elevations at the FCP Control Structure for Year 0 Hydrology**

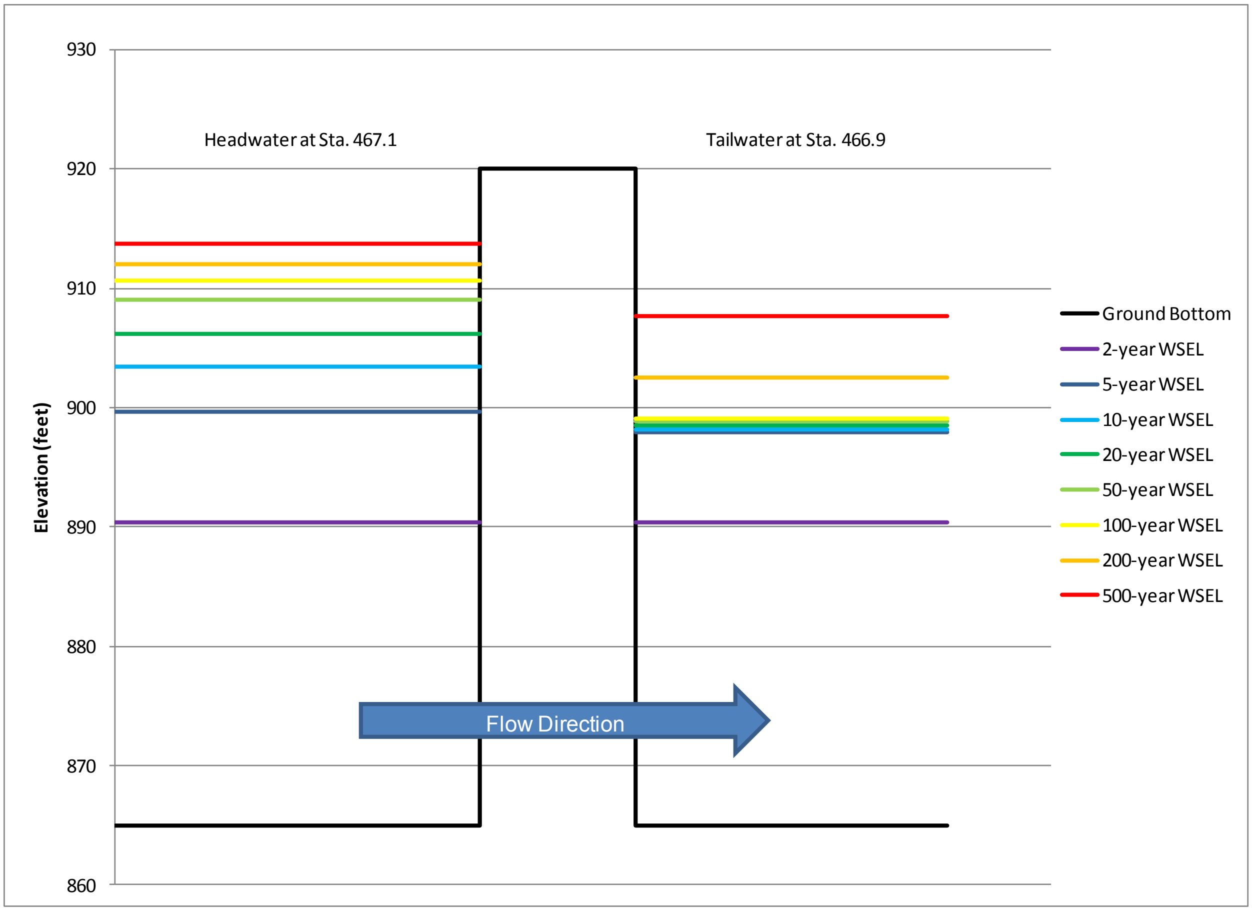


Figure F-D3 Comparison of the Headwater and Tailwater Elevations at the FCP Control Structure for Year 25 Hydrology

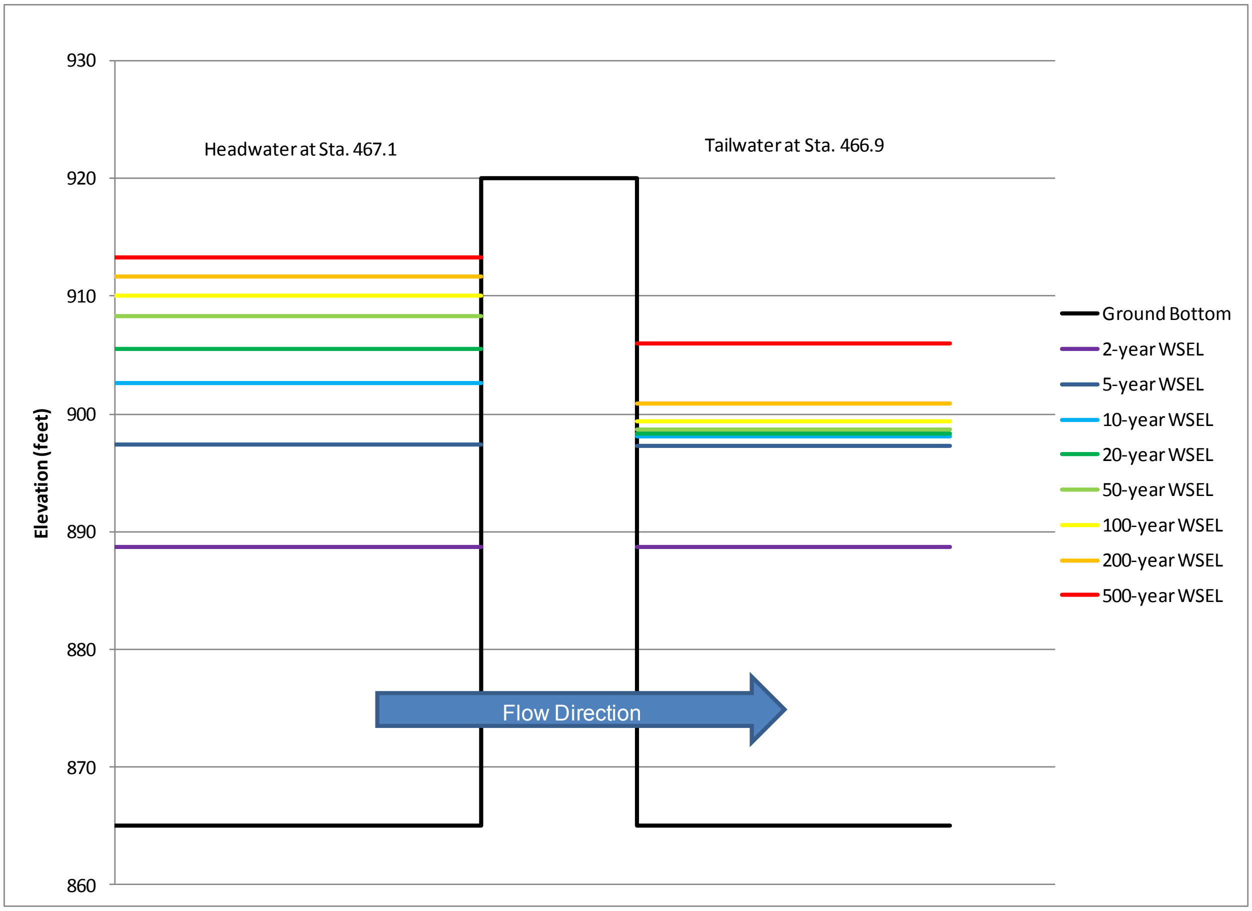


Figure F-D4 **Comparison of the Headwater and Tailwater Elevations at the FCP Control Structure for Year 50 Hydrology**

F-D1.1.5 Transition from the Natural Channel to the Control Structure

The transition from the natural channel into the control structure is accomplished with 7:1 side slopes, as shown in Figure F-D5.

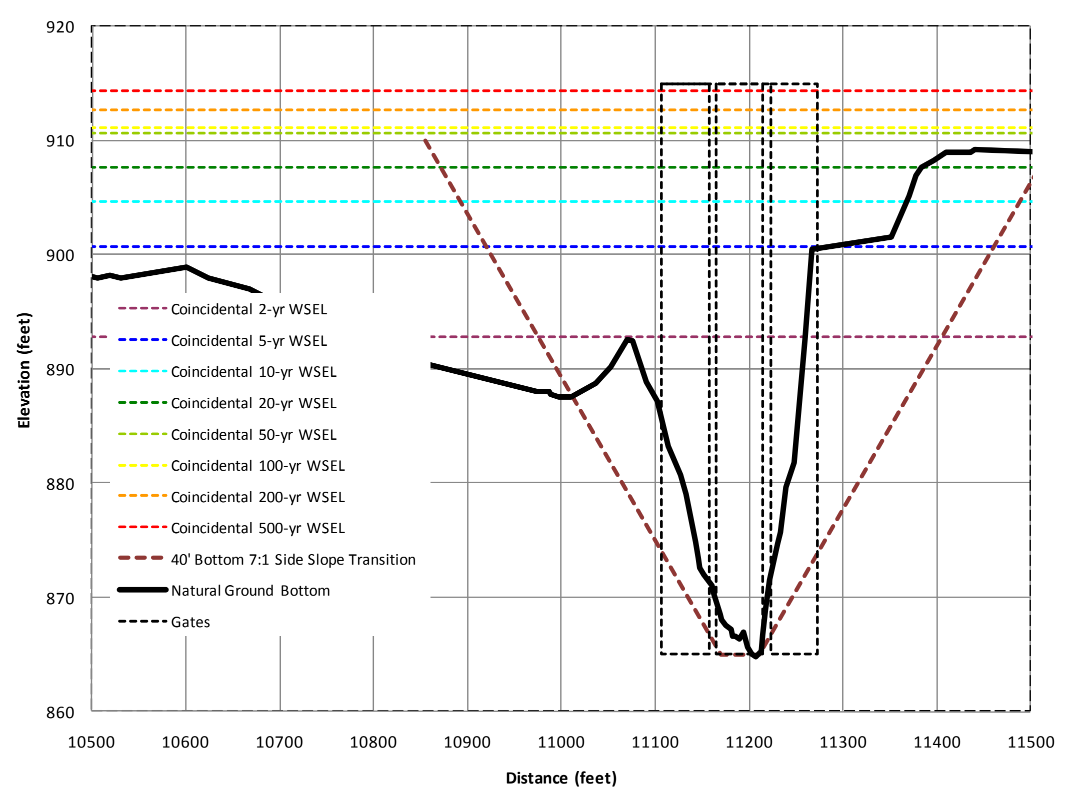


Figure F-D5 Transition from the Natural Red River Cross Section to the FCP Control Structure

F-D1.2 INLET WEIR AT RED RIVER OF THE NORTH

A description of the inlet weir to the FCP diversion can be found in Appendix F. This portion of Exhibit D presents the design dimensions of the weir as well as detailed HEC-RAS modeling results at the inlet weir location using Phase 3 hydrology.

F-D1.2.1 Design of Inlet Weir at Red River of the North

The structure designed to pass water from the Red River to the Diversion Channel for the FCP alignment is a multi-tier weir, as shown in Figure F-D6. The design was created and optimized by Moore Engineering. HEC-RAS modeling results for the inlet weir for Year 0, Year 25, and Year 50 are presented in Tables F-D4, F-D5, and F-D6, respectively.

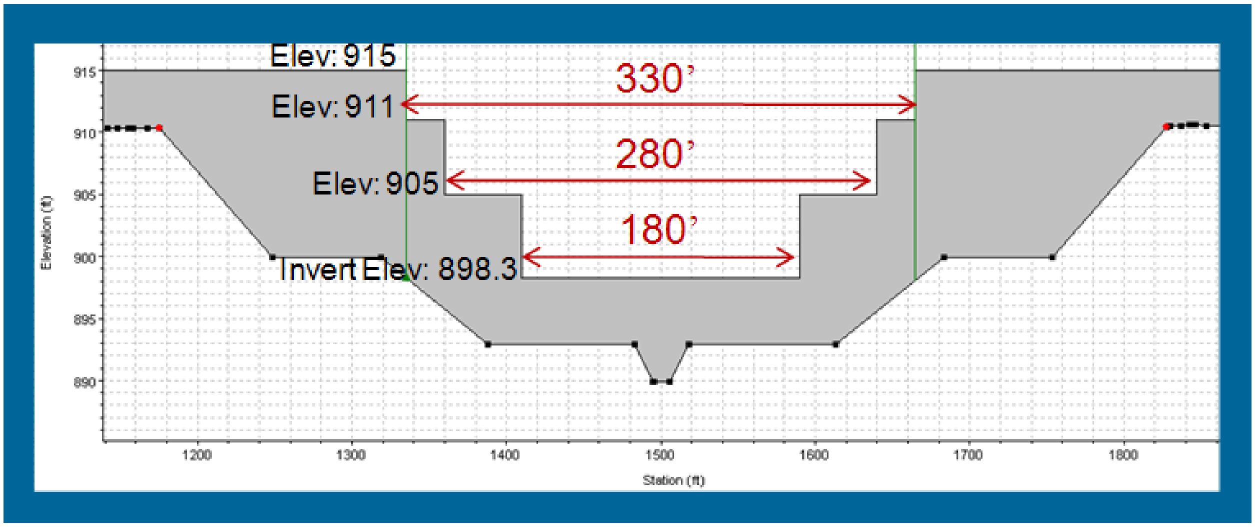


Figure F-D6 FCP Inlet Weir at Diversion Channel

Table F-D4 HEC-RAS Results for the Proposed FCP Inlet Weir at the Diversion Channel for the Red River of the North (Year 0 Hydrology)

Event in Red River (year)	Q Total (cfs)	Q Weir (cfs)	*Headwater (ft)	**Tailwater (ft)	Head Difference (ft)	Flows Area (sq ft)	Velocity over Weir (ft/s)
5	2510	2510	901.35	897.65	3.70	549	4.57
10	7346	7346	904.50	901.90	2.60	1116	6.58
20	12241	12241	906.68	904.18	2.50	1676.4	7.30
50	19497	19497	909.13	906.91	2.22	2362.4	8.25
100	23771	23771	910.41	908.32	2.09	2720.8	8.74
200	29833	29833	912.02	910.13	1.89	3222.6	9.26
500	34762	34762	913.24	911.46	1.78	3625.2	9.59

*HEC-RAS cross section 127253

**HEC-RAS cross section 127233

Table F-D5 HEC-RAS Results for the Proposed FCP Inlet Weir at the Diversion Channel for the Red River of the North (Year 25 Hydrology)

Event in Red River (year)	Q Total (cfs)	Q Weir (cfs)	*Headwater (ft)	**Tailwater (ft)	Head Difference (ft)	Flows Area (sq ft)	Velocity over Weir (ft/s)
5	968	968	899.92	895.52	4.40	291.6	3.32
10	5747	5747	903.58	901.02	2.56	950.4	6.05
20	10604	10604	906.04	903.47	2.57	1497.2	7.08
50	17596	17596	908.52	906.25	2.27	2191.6	8.03
100	23003	23003	910.19	908.07	2.12	2659.2	8.65
200	28033	28033	911.57	909.62	1.95	3074.1	9.12
500	34955	34955	913.28	911.51	1.77	3638.4	9.61

*HEC-RAS cross section 127253

**HEC-RAS cross section 127233

Table F-D6 HEC-RAS Results for the Proposed FCP Inlet Weir at the Diversion Channel for the Red River of the North (Year 50 Hydrology)

Event in Red River (year)	Q Total (cfs)	Q Weir (cfs)	*Headwater (ft)	**Tailwater (ft)	Head Difference (ft)	Flows Area (sq ft)	Velocity over Weir (ft/s)
5	0	0	898.32	889.92	8.40	3.6	0.00
10	4419	4419	902.73	900.20	2.53	797.4	5.54
20	9267	9267	905.47	902.86	2.61	1337.6	6.93
50	16115	16115	908.04	905.71	2.33	2057.2	7.83
100	21128	21128	909.63	907.46	2.17	2502.4	8.44
200	26716	26716	911.23	909.22	2.01	2961.9	9.02
500	34429	34429	913.16	911.38	1.78	3598.8	9.57

*HEC-RAS cross section 127253

**HEC-RAS cross section 127233

F-D1.3 OUTLET TO RED RIVER OF THE NORTH

The Outlet of the Diversion Channel into the Red River of the North consists of riprap over the downstream 300 feet of the Diversion Channel. Details are provided in Appendix F.

F-D2.0 LOCALLY PREFERRED PLAN (LPP)

F-D2.1 CONTROL STRUCTURE ON THE RED RIVER OF THE NORTH

A description of the LPP control structure on the Red River of the North can be found in Appendix F. Appendix F, Exhibit B discusses the variety of structure types that were investigated for this control structure during the previous design phases. This portion of Exhibit D presents design concepts and calculations for the Red River of the North control structure, detailed HEC-RAS modeling outputs, and comparisons of water surface elevations using Phase 4 hydrology and the HEC-RAS unsteady flow model.

F-D2.1.1 Flow Partitioning

The partitioning of the flow between the protected area downstream of the Red River Control Structure and through the inlet weir to the diversion channel is determined by the gate openings at the control structures located on the Red River, Wild Rice River, and Wolverton Creek. The resulting peak flows through the Red River Control Structure and the diversion channel inlet are presented in Table F-D8. The flows presented include 10-year, 50-year, 100-year, and 500-year events on the Red River of the North from Phase 4, as well as more frequent events from Phase 3 (which were not analyzed during Phase 4).

F-D2.1.2 Gate Number and Sizing – Concepts

The number and sizing of the gates in the Red River Control Structure is controlled by:

1.) Ability of the design to achieve the desired flow through the gates as determined by the target elevation at the USGS gage in Fargo, as shown in Table F-D7, while minimizing the impact to water surface elevations downstream of the Project.

2.) Provide sufficient capacity to convey low flows into the protected area and not restrict use of the river.

Table F-D7 Target Elevations at the USGS Gage in Fargo

Event	Target Elevation (NAVD88 ft)
10-Year	891.99
50-Year	892.74
100-Year	893.40
500-Year	902.66

Initially, three concepts which included operable gates above permanently open areas were considered and presented to the U.S. Army Corps of Engineers (USACE) during a meeting on December 2, 2009. Those concepts are shown in Exhibit B of Appendix F and include a design with three identical gates, a design with five identical gates, and a design with two large identical gates combined with two smaller gates.

Based on discussion with the USACE and preliminary modeling of these concepts during Phase 2, the concept with 3 identical gates was selected. This selection was made because it allowed a wider gate width (for ice and boat passage) and redundancy of gates for operation and maintenance issues.

During Phase 4 the number of gates and gate widths did not change from Phase 3. As a result of staging water on the Red River upstream of the Project the gate height increased to prevent water from overtopping the Red River Control Structure. In addition, in order to control the amount of flow into the protected area and upstream staging throughout the duration of the event, preliminary gate operations that resulted in variable gate openings over the pass of the hydrograph were developed for each flood event to control the flow through the Red River Control Structure.

F-D2.1.3 Gate Number and Sizing – Optimization

During Phase 3, following the recommendation from USACE – Environmental, a width of 50 feet was selected for each of the three gates, and this was not changed during Phase 4. Preference was given for multiple, identically sized gates for operation and maintenance purposes. Also, it is desirable to have as little impact on the smaller, more frequent, flood events (i.e. no change in WSEL or gate operation required); thus, a minimum open width is necessary to avoid flow constriction during smaller flood events.

After the three identical gate concept was selected for further analysis, preliminary gate operations were developed to control the amount of flow through the control structure. During Phase 4, the HEC-RAS unsteady flow model was used to further refine the optimization of the required gate operations throughout the duration of each flood event.

During previous design phases, the 500-year event (during which the flow through the structure is the greatest) resulted in the greatest open area and therefore could be used to determine the minimum number of gates required. For all smaller flood events, the gates may be partially or fully closed to achieve the desired flow into the protected area. During Phase 4 the gates are partially closed for each of the flood events analyzed (10-year, 50-year, 100-year, and 500-year). As a result the gate height is defined by the elevation that water is staged upstream of the Project. In optimizing the design of the Red River Control Structure for the LPP alignment, 50-feet tall gate openings were used for the LPP diversion. The number of gates necessary for the LPP (as determined during Phase 2) is three 50-feet wide gates. A concept drawing for the control structure is presented in Figure F-D7.

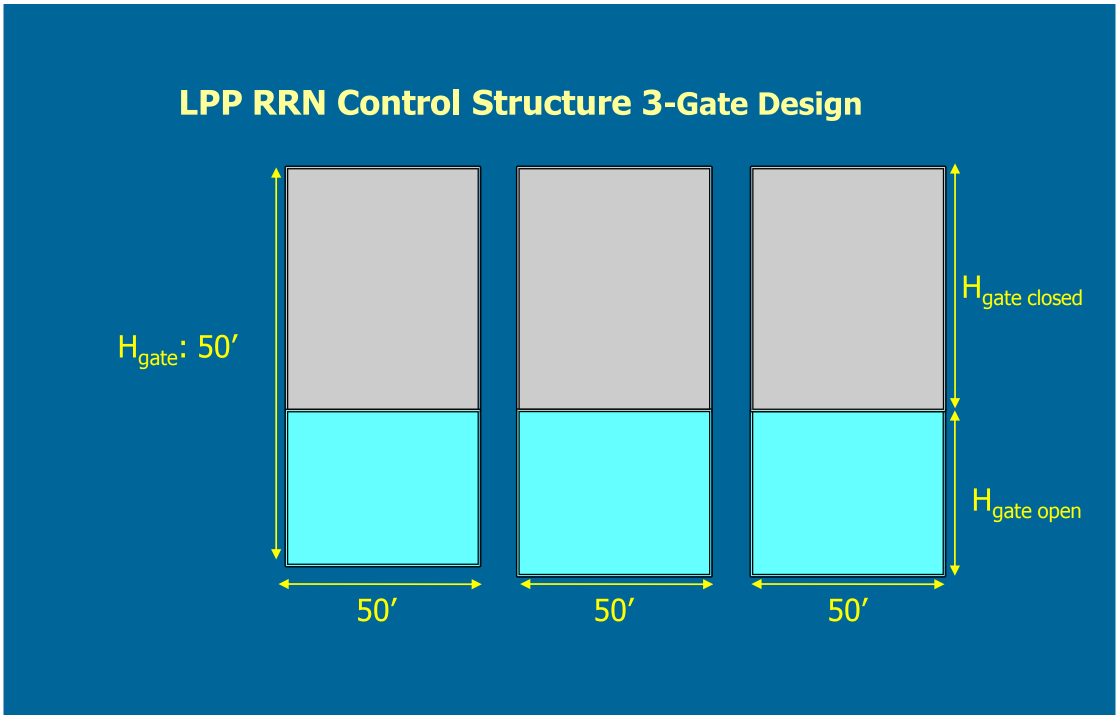


Figure F-D7 LPP Red River of the North Control Structure 3-Gate Design

F-D2.1.4 Gate Operation Optimization and HEC-RAS Results

Preliminary gate operational scheme was developed to achieve

1.) Minimize downstream impacts
2.) Maintain a target flood elevation at the USGS gage in Fargo
3.) Limit the amount of upstream staging.

In general, gate operations included lowering the gate at the beginning of the event, maintaining a constant opening through the peak of the event, and then opening the gates at the end of the event. For each flood event, there are multiple potential gate operational schemes. By simply varying how the gates are operated can result in moving from a decrease to an increase in downstream water surface elevations. The gate operational scheme presented for each of the flood events is one possible way to control how water is conveyed downstream of the Project. During final design, a complete analysis of how gate operations impact the downstream water surface elevations should be completed.

Following discussions with the USACE it was determined to utilize user defined rules to calculate the flow through the control structure, rather than the default HEC-RAS gate routines. This was done primarily because the default gate routines in HEC-RAS for tainter gates assume the gates are elevated over a sill, which does not accurately characterize the proposed gates at the Red River control structure, which are located near the bottom of the river without a sill, therefore weir flow conditions would not apply. Based on discussions with the USACE and review of data provided by the USACE including field measurements from Mississippi River Lock and Dams 6-10 and studies complete by the Bureau of Reclamation, it was found that the orifice equation could be used to characterize the flow through the gates throughout the duration of the event. The orifice equation could be used throughout the duration of the event due to the high levels of submergence at the control structure gates throughout the duration of the event (which are a result of staging and the large flood events analyzed as part of Phase 4). The orifice discharge coefficient was modified between 0.8, for submergences less than 0.67, to 1.0 when the submergence is greater than 0.8 (and linear interpolation in between), based on the level of submergence for each time step.

User defined rules were also utilized to control the gate opening throughout the duration of the event. This provided the flexibility to control the gate opening based on the water surface elevation at the USGS gage in Fargo, and to have additional control over the amount of water being let into the protected area throughout the duration of a flood event.

The operation scheme of the gates proposed in this feasibility analysis is as follows. At the beginning of each flood event the gates begin to close so that they are in the lowest position at the beginning of the event, or at the beginning of the rising limb of the hydrograph. The gates remain in the closed position during the first half of the flood event, when the majority of the flow is conveyed through the Diversion Channel. Only after the peak flow rate has passed through the downstream end of the Diversion Channel the gates on the Red River begin to open. Allowing the peak flow in the Diversion Channel to reach the Red River before the control structure gates begin to open results in decoupling the peak flow rate in the Diversion Channel from the peak flow rate on the Red River. This allows flood waters to be diverted around the Cities while minimizing downstream impacts to the floodplain.

When the gates start to open, they are opened at a rate to increase water surface elevations at the USGS gage in Fargo to the target elevation. As a result of allowing the peak flow on the Diversion Channel to pass through the system first, the control structure gates may be opened either faster, to reduce the duration that water is stored upstream of the Cities or slower to further reduce the water surface elevation at the USGS gage in Fargo. This allows the Cities to potentially achieve additional protection beyond the Phase 3 target elevation at the USGS gage in Fargo, by simply slowing the rate at which the gates are opened during the receding limb of the hydrograph. This potential additional benefit warrants further evaluation during future design phases. Based on correspondence with the USACE of February 12, 2011 it was determined that the Phase 4 models should match the target elevations at the USGS gage within 0.10-0.15 ft. Matching the target elevation to within 0.15 ft would match the project benefits calculated by the USACE within 5% from Phase 3.

In the HEC-RAS unsteady flow models, the Phase 4 gate operations only open the gates for one hour during the day. This allows the model to account for the travel time between the Red River control structure and the USGS gage at Fargo. If the gates are operated continuously then there is greater potential to allow too much water into the protected area and exceed the target elevation at the USGS gage in Fargo.

Maximum flows, water surface elevations, head difference, gate opening height and velocities during each flood event at the LPP Red River of the North control structure are presented in Table F-D8. These results were extracted from the HEC-RAS models with the exception of average velocity through the gates. HEC-RAS does not provide this result so it was calculated by dividing the discharge by the flow area. The location of the control structure in the LPP HEC-RAS model is inline structure station 2529023. Figure F-D8 show the comparison of the headwater and tailwater elevations at the LPP Control Structure.

Operating the gates at the Red River control structure restricts the flow that is conveyed into the protected area. As a result water is stored in the Red River and adjacent floodplain upstream of the control structures. Figures F-D9 – F-D12 show how operating the gates extends the duration at which flood waters are stored upstream of the control structures.

Table F-D8 HEC-RAS Outputs for the Proposed LPP Control Structure at the Red River of the North

	Phase 3							Phase 4			
Event (yr)	**Existing Flow into Protected Area (cfs)**	**Proposed Flow through Gates into Protected Area (cfs)**	**Proposed Headwater Elevation in Red River* (ft)**	**Proposed Tailwater Elevation in Red River* (ft)**	**Head Difference (ft)**	**Gate Open Height (ft)**	**Average Velocity through Gates (ft/s)**	**Existing Flow into Protected Area (cfs)**	**Proposed Max. Flow through Gates into Protected Area[3] (cfs)**	**Proposed Max. Headwater Elevation in Red River** (ft)**	**Proposed Max. Tailwater Elevation in Red River** (ft)**
2[+]	3,139	3,139	894.53	894.51	0.02	44.00	1.10	-	-	-	-
5[+]	6,160	5,497	905.18	902.21	2.97	3.30	11.1	-	-	-	-
10	10,500	7,051	908.83	902.69	6.14	2.95	15.9	10,271	9,407	916.29	904.40
50	21,000	10,918	913.38	903.77	9.61	3.00	24.3	18,207	10,178	920.85	905.15
100	25,000	12,252	914.87	904.65	10.22	3.20	25.5	21,458	11,569	922.87	906.37
500	32,000	13,077	917.84	914.1	3.74	7.00	12.5	28,623	30,569	922.33[4]	914.84

[+] Phase 3 feasibility design steady flow models

* Headwater at Station 478.77 and Tailwater at Station 478.76

** Headwater at Station 2529066 and Tailwater at Station 2528979

[3] Flow into protected area listed is maximum flow. Due to operation of the gates this does not correspond to the same time as the maximum head differential across the control structure.

[4] Maximum elevation for 500-year event is lower than the 100-year event due to gate operations required to minimize impacts to the floodplain downstream of the Project

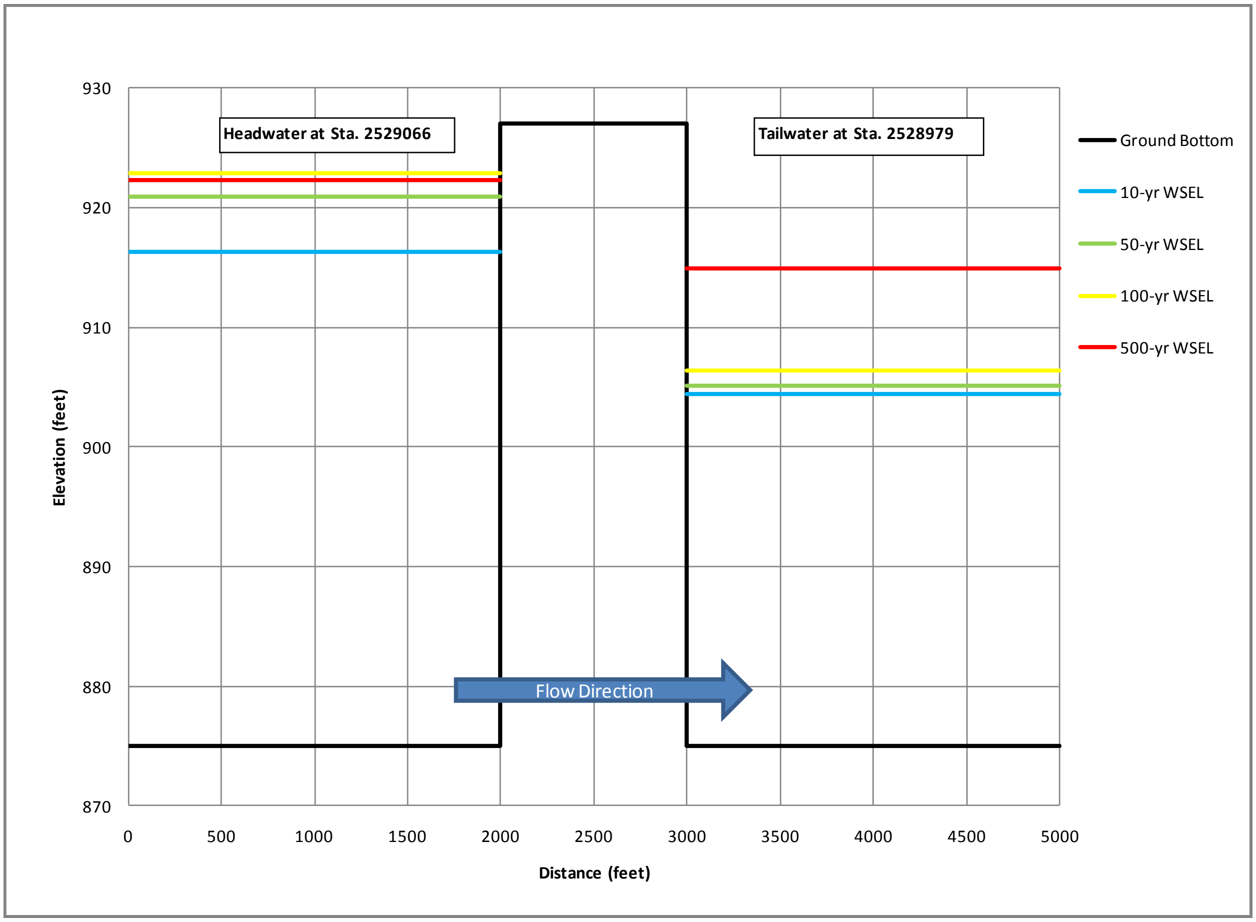


Figure F-D8 **Comparison of the Headwater and Tailwater Elevations at the LPP Control Structure**

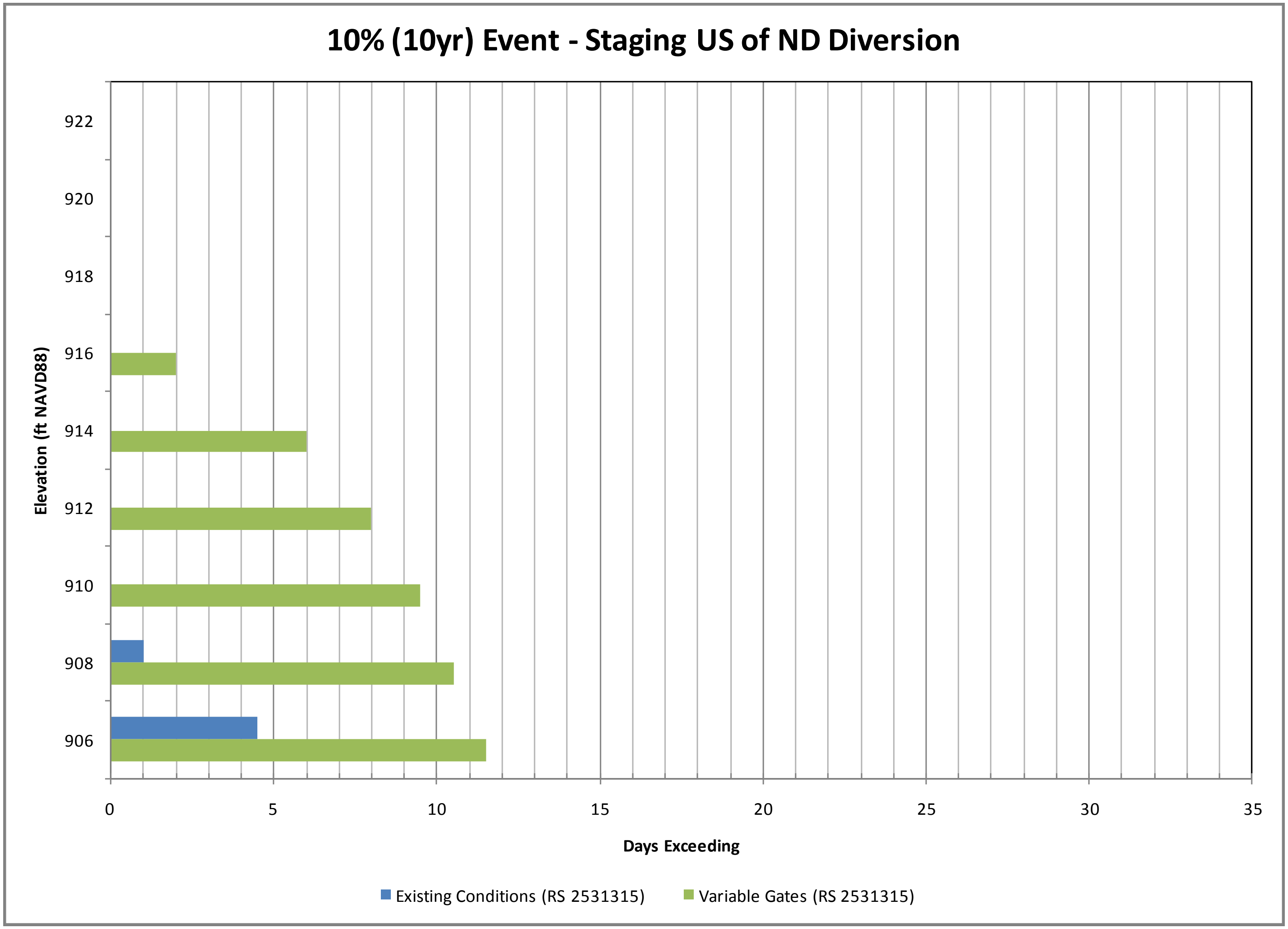


Figure F-D9 **Comparison of Increased Water Surface Elevations Upstream of the Red River Control Structure for the 10-Year Event**

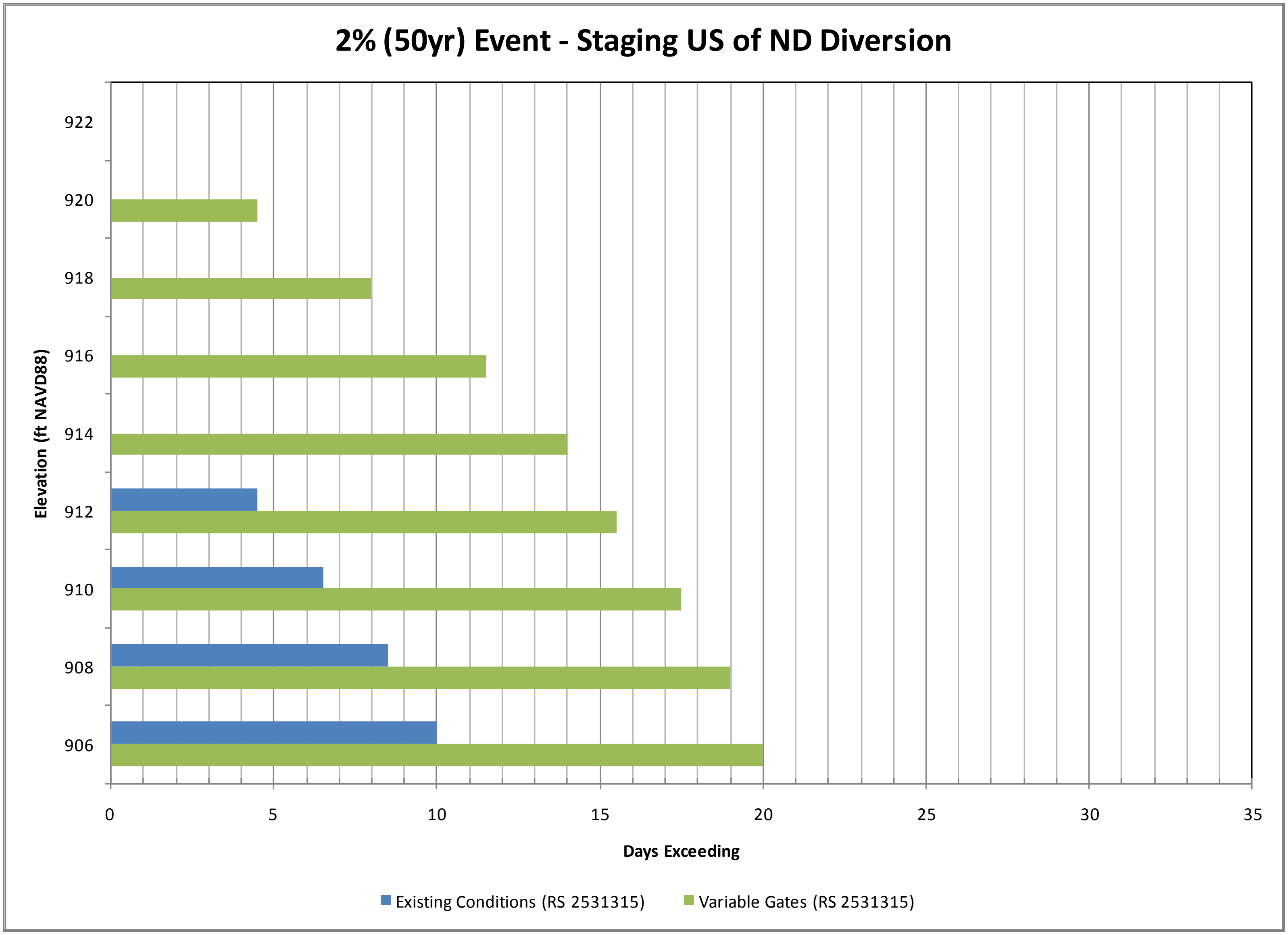


Figure F-D10 **Comparison of Increased Water Surface Elevations Upstream of the Red River Control Structure for the 50-Year Event**

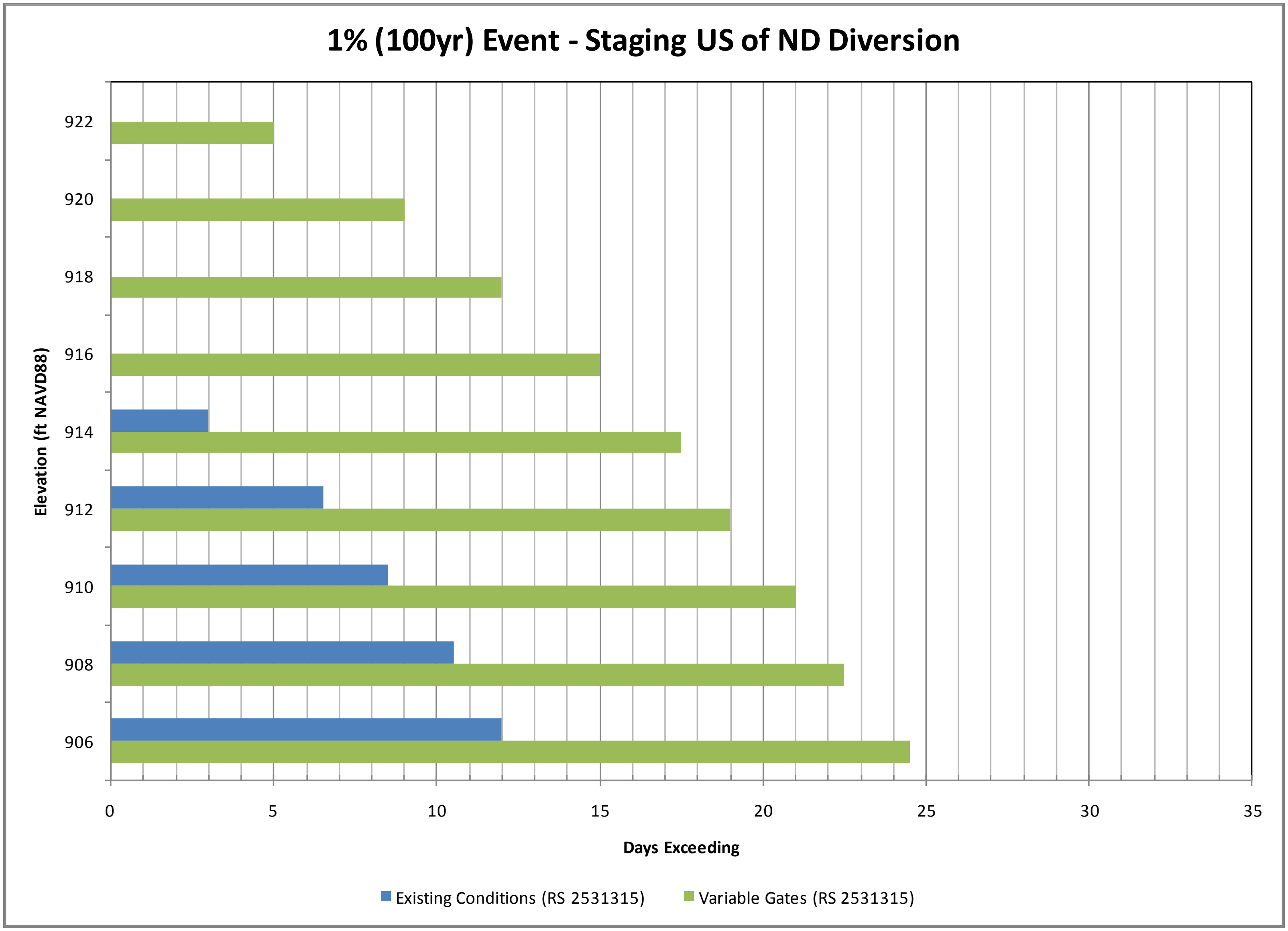


Figure F-D11 **Comparison of Increased Water Surface Elevations Upstream of the Red River Control Structure for the 100-Year Event**

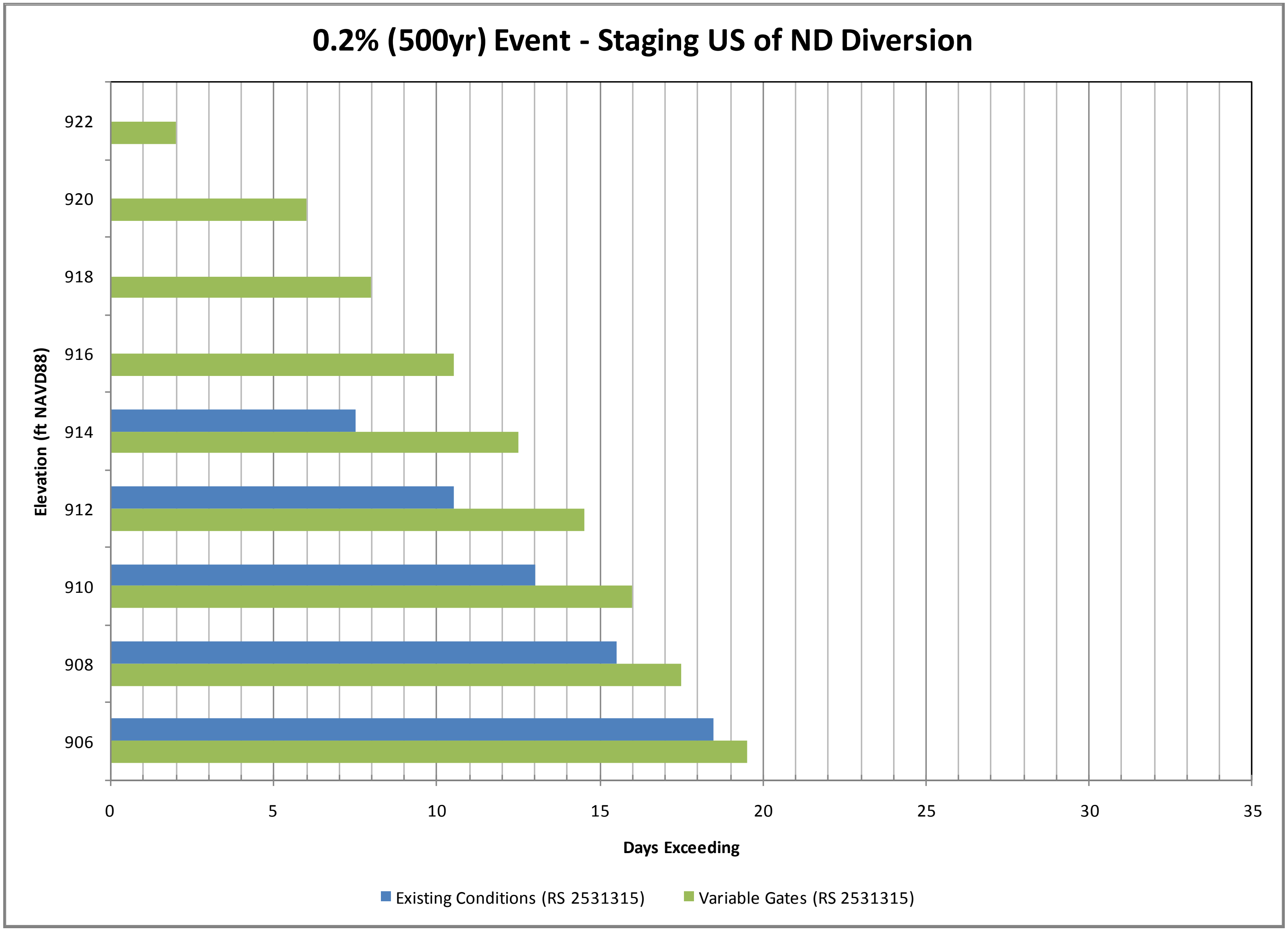


Figure F-D12 Comparison of Increased Water Surface Elevations Upstream of the Red River Control Structure for the 500-Year Event

F-D2.1.5 Initial Gate Operation

During Phase 3 feasibility design with the HEC-RAS steady flow models, it was assumed that flow was not diverted into the Diversion Channel until 9,600 cfs occurred at the USGS gage in Fargo. (However, for evaluation of downstream impacts in Phase 3, the use of HEC-RAS unsteady flow model showed that the start of gate closing needed to begin for flows smaller than 9,600 cfs when the peak of the hydrograph exceeded 9,600 cfs.) Phase 4 analysis using the HEC-RAS unsteady flow model indicates that in order to minimize impacts to the floodplain downstream of the project for events larger than 9,600 cfs, the gates at the control structures will need to be operated during the rising limb of the hydrograph, before 9,600 cfs occurs at the USGS gage in Fargo. With regards to operation of the control structure gates, the Phase 4 analysis found that:

1. The gates will not be operated when the forecasted peak flow at the USGS gage in Fargo is less than 9,600 cfs. In other words, the frequency with which the gates could be operated is determined by the likelihood of peak flows larger than 9,600 cfs. This has happened 20 times during the 108 years of record, but 11 of those 20 have happened in the past 18 years.

2. In order to achieve the goal of no downstream impacts for events larger than 9,600 cfs, the gates must operate on the rising limb of the hydrograph – and this might start before 9,600 cfs is reached in Fargo.

3. The Cities of Fargo and Moorhead may determine not to operate the system for events larger than 9,600 cfs (for instance during summer floods, when historic peak flows are max 12,000-13,000 cfs or a stage of 30 = the beginning of major flooding).

HEC-RAS unsteady flow models completed as part of the Phase 4 analysis start with the gates beginning in a position that is less than fully open (the gates are sufficiently open such that water is not staged upstream of the control structure at the very beginning of the simulation). For all events, the gates are then gradually closed over several days so that they reach the minimum opening during the rising limb of the hydrograph. This was done to keep the HEC-RAS model computationally stable and minimize the number of iterations required to converge on a solution for a given time step while lowering the gates. In reality, it may be possible to lower the gates much quicker and/or to begin with a bigger opening, without this resulting in downstream impacts. This further evaluation (optimization of the gates operation to reduce their duration of operation) should be considered in future phases.

Table F-D9 includes a summary of the flows and stages in the Phase 4 HEC-RAS models for each of the 4 design events at which the gates would begin restricting flows passed into the protected area.

Table F-D9 Maximum Flow and Stage at the Fargo Gage before Control Structure Gates Restrict Flow into the Protected Area

Design Event	**Maximum Flow in Fargo before Gate begins restricting flow**	**Maximum Stage in Fargo before Gate begins restricting flow**
10-Year	1,800	878.8
50-Year	2,400	880.5
100-Year	2,700	881.2
500-Year	5,300	885.4
USGS Gage Rating Curve	*9,600*	*~891.5*

In general, in order to minimize the impacts to the floodplain downstream of the project the gates begin to restrict flow earlier for more frequent events. This is primarily due to two reasons.

1. Downstream impacts are caused by the volume and timing along the entire rising limb of the hydrograph - not just the peak flow of the hydrograph. In order to control the timing and shape of the rising limb of the hydrograph, the gates must begin operating at lower flows/stages for smaller events.

2. For larger events there is a larger floodplain or "pool" downstream of the project. This means that the downstream impacts are relatively less sensitive to shape and timing of the rising limb of the hydrograph; that is, for larger events a small increase in discharge along the rising limb of the hydrograph may not result in measurable increases to flood levels downstream of the diversion. However, that same increase in discharge for smaller events will likely result in measurable impacts downstream of the diversion.

Although the system goes into operation before 9,600 cfs at the Fargo gage, the system only begins operation approximately 7-10 days before 9,600 cfs would occur at the Fargo gage for the 100-year synthetic design event.

Flows presented in Table F-D9 above are the ones associated to a given shape of the synthetic hydrographs (i.e., the design floods). If the hydrograph shape would be different, while maintaining the hydrograph peak flow for a given return period, the answer could be different regarding when the gates need to be operated. Finally, it is also important to acknowledge that in order to eliminate impacts downstream for relatively frequent events (like the 10-yr event), even when these impacts are so small that they do not significantly increase the risk of flooding in the downstream communities, project design considerations are "obligating" operation of the system in advance of the 9,600 cfs. Figures F-D13 – F-D16 show how the Red River control structure gates operate for each event, as well as how the peak flow rates in the diversion channel and Red River are decoupled to minimize downstream impacts.

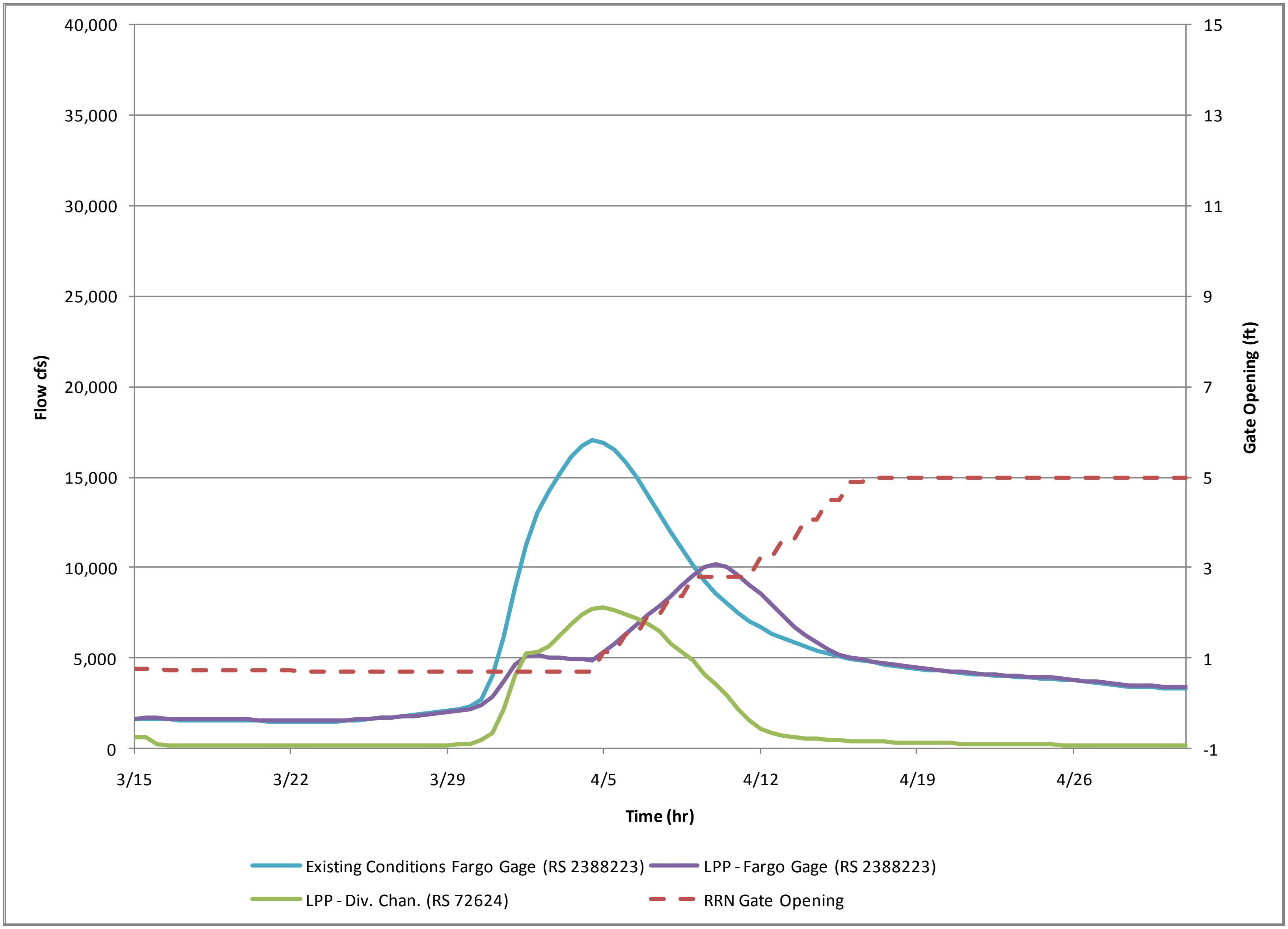


Figure F-D13 Decoupled Peak Flows in the Diversion Channel and Red River for the 10-Year Event

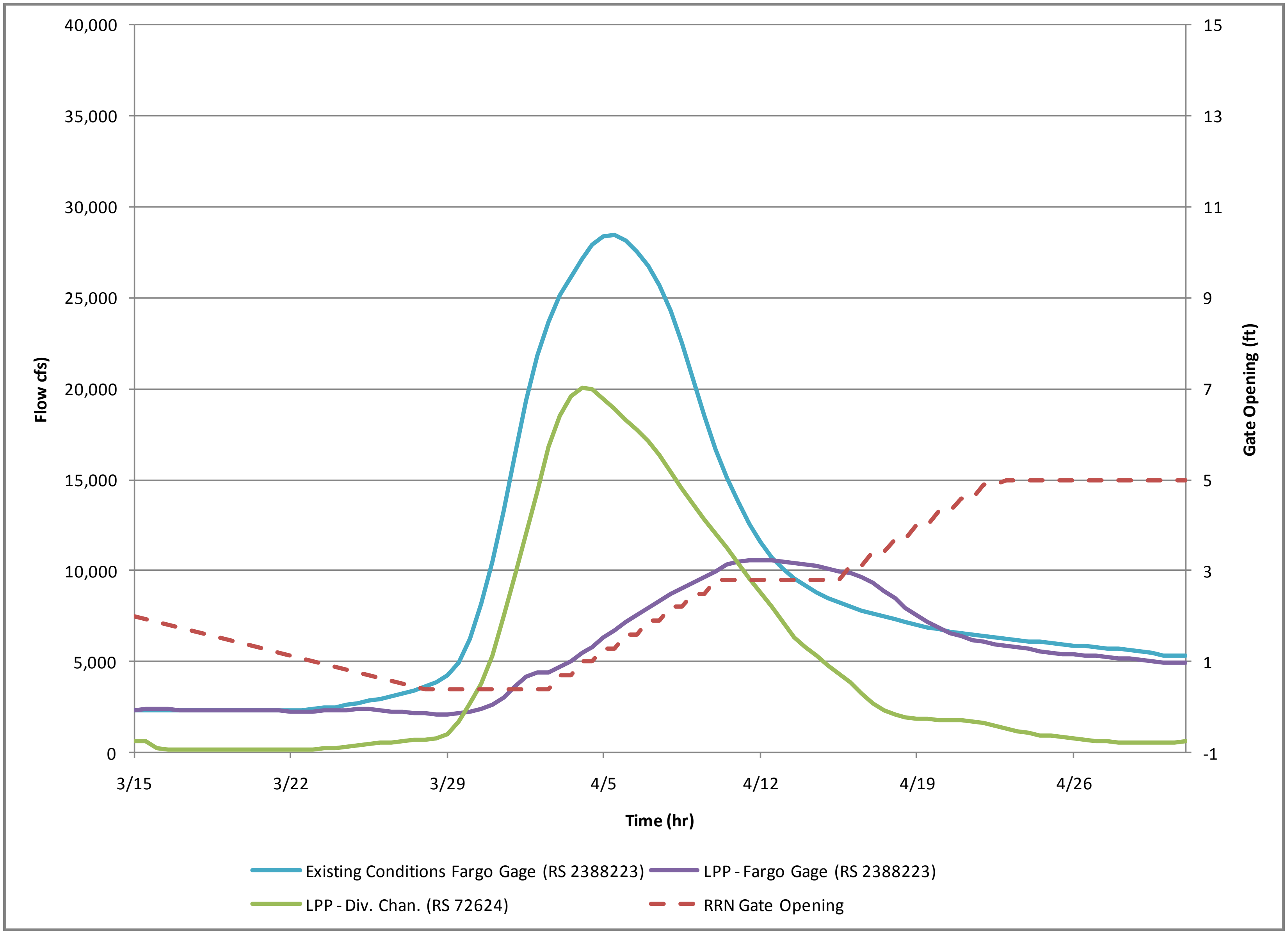


Figure F-D14 Decoupled Peak Flows in the Diversion Channel and Red River for the 50-Year Event

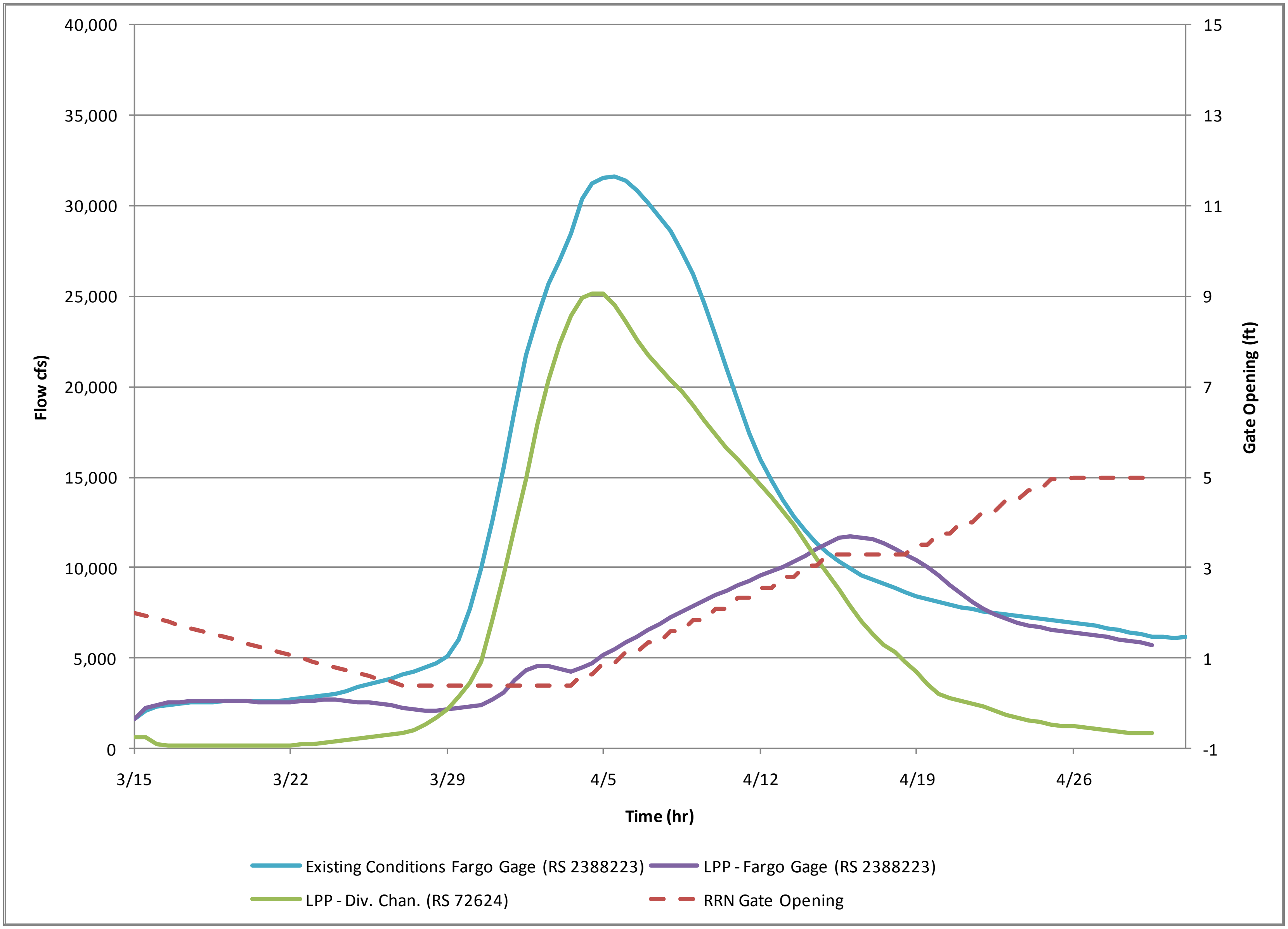


Figure F-D15 Decoupled Peak Flows in the Diversion Channel and Red River for the 100-Year Event

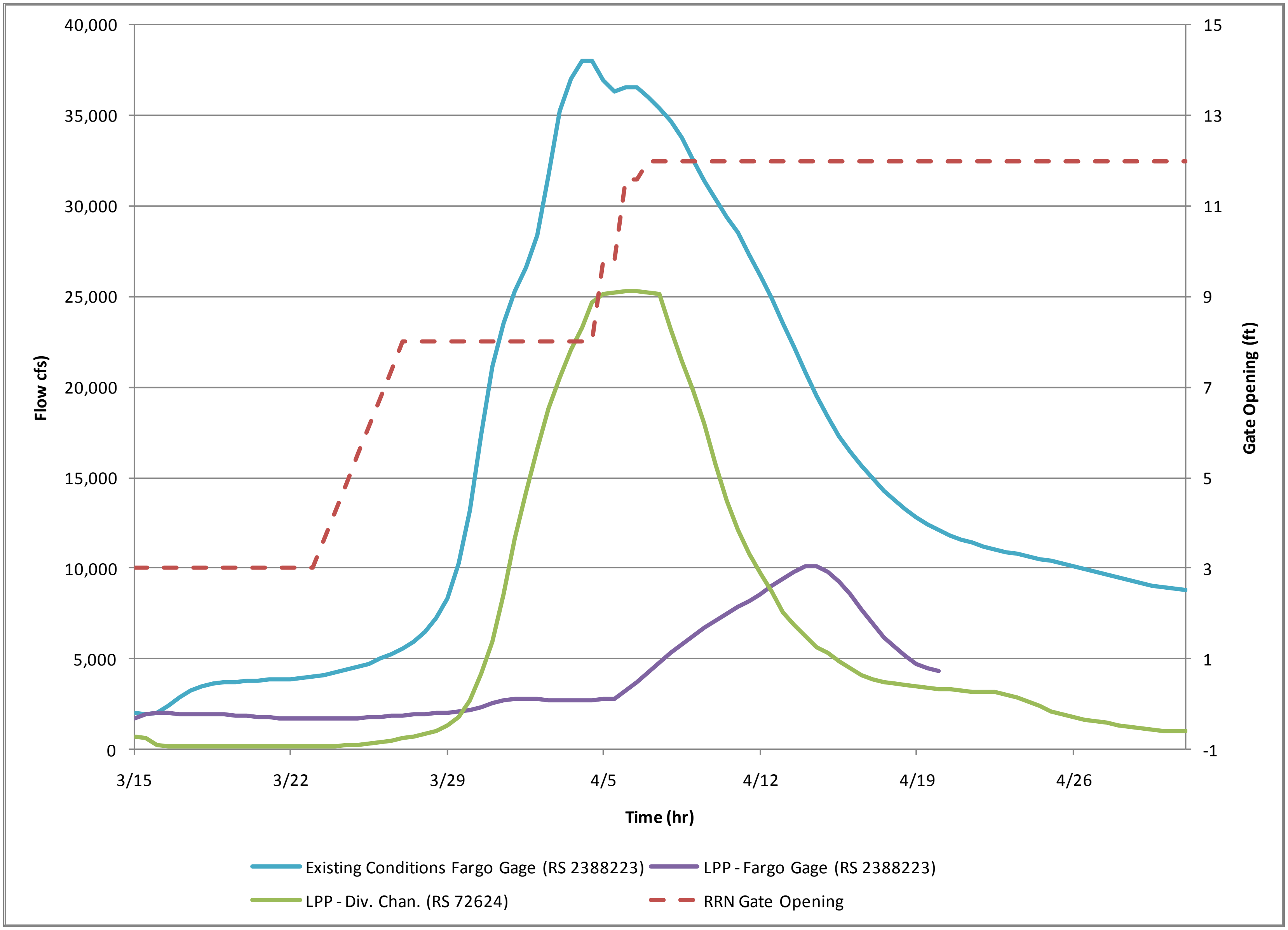


Figure F-D16 Decoupled Peak Flows in the Diversion Channel and Red River for the 500-Year Event

F-D2.1.6 <u>Transition from the Natural Channel to the Control Structure</u>
The transition from the natural channel into the control structure is accomplished with 7:1 side slopes, as shown in Figure F-D17. For more details on the geotechnical slope stability analysis of this reach, see Exhibit N of Appendix F.

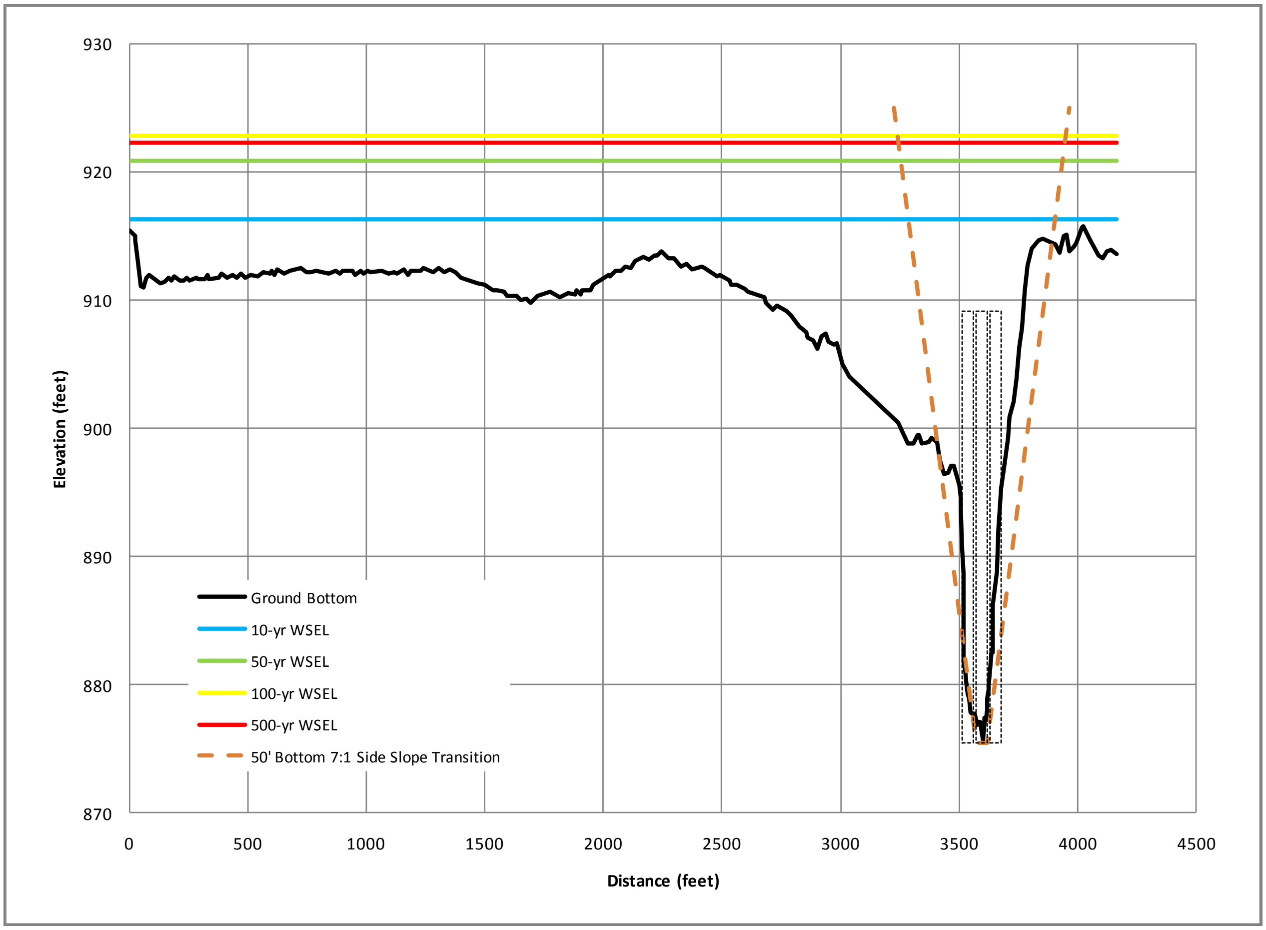


Figure F-D17 Transition from the Natural Red River Cross Section to the LPP Control Structure

F-D2.2 INLET WEIRS AT WILD RICE RIVER

A description of the passive (i.e. without moveable parts like gates) diversion inlet weir that is used to control water entering the LPP Diversion Channel from the Red River of the North and Wild Rice Rivers can be found in the main section of Appendix F. This portion of Exhibit D presents a comparison of water surface elevations at the Connection Channel and the diversion inlet weir (or Inlet Structure) and detailed HEC-RAS modeling outputs using Phase 4 hydrology.

F-D2.2.1 <u>Comparison of LPP Diversion Channel and Wild Rice River Hydrology</u>
Flows, water surface elevations (WSEL), and average velocities for the LPP Diversion Channel just downstream of the Inlet Structure to the west of the Wild Rice River (Station 152517) for the 10-, 50-, 100-, and 500-year events are presented in Table F-

D10. Water surface elevations in the LPP Diversion Channel just downstream of the primary Inlet Structure (Station 152517), in the Wild Rice River for local tributary flood events, and in the Wild Rice River for coincidental flood events, plotted against the cross section data, are presented in Figure F-D18. Tabular results of local and coincidental flood events for existing conditions in the Wild Rice River are presented in Appendix F, Exhibit A.

Table F-D10 LPP Diversion Channel Downstream of the Inlet Structure to the East of the Sheyenne River

	Phase 3[1]			Phase 4[2]		
Event (yr)	**Flow (cfs)**	**WSEL (ft)**	**Average Velocity (ft/s)**	**Flow (cfs)**	**WSEL (ft)**	**Average Velocity (ft/s)[3]**
5	2,534	896.57	2.03	-	-	-
10	7,284	901.88	2.42	7,207	896.57	2.31
50	19,387	908.70	3.09	15,737	902.44	2.76
100	24,178	910.95	3.22	19,046[4]	904.44	2.82
500	34,915	915.35	3.48	18,404	905.46	2.56

[1] Phase 3 data from station 173349 from the Phase 3 HEC-RAS steady flow model. All Phase 3 information based on Year 0 hydrology.

[2] Phase 4 data from station 152517 from the Phase 4 HEC-RAS unsteady flow mode.

[3] Maximum average velocity at time of maximum water surface elevation (which does not always occur at the same time as the peak flow)

[4] Peak flow downstream of the Inlet structure is controlled by the amount of upstream staging, which is larger for the 100-year than the 500-year

.

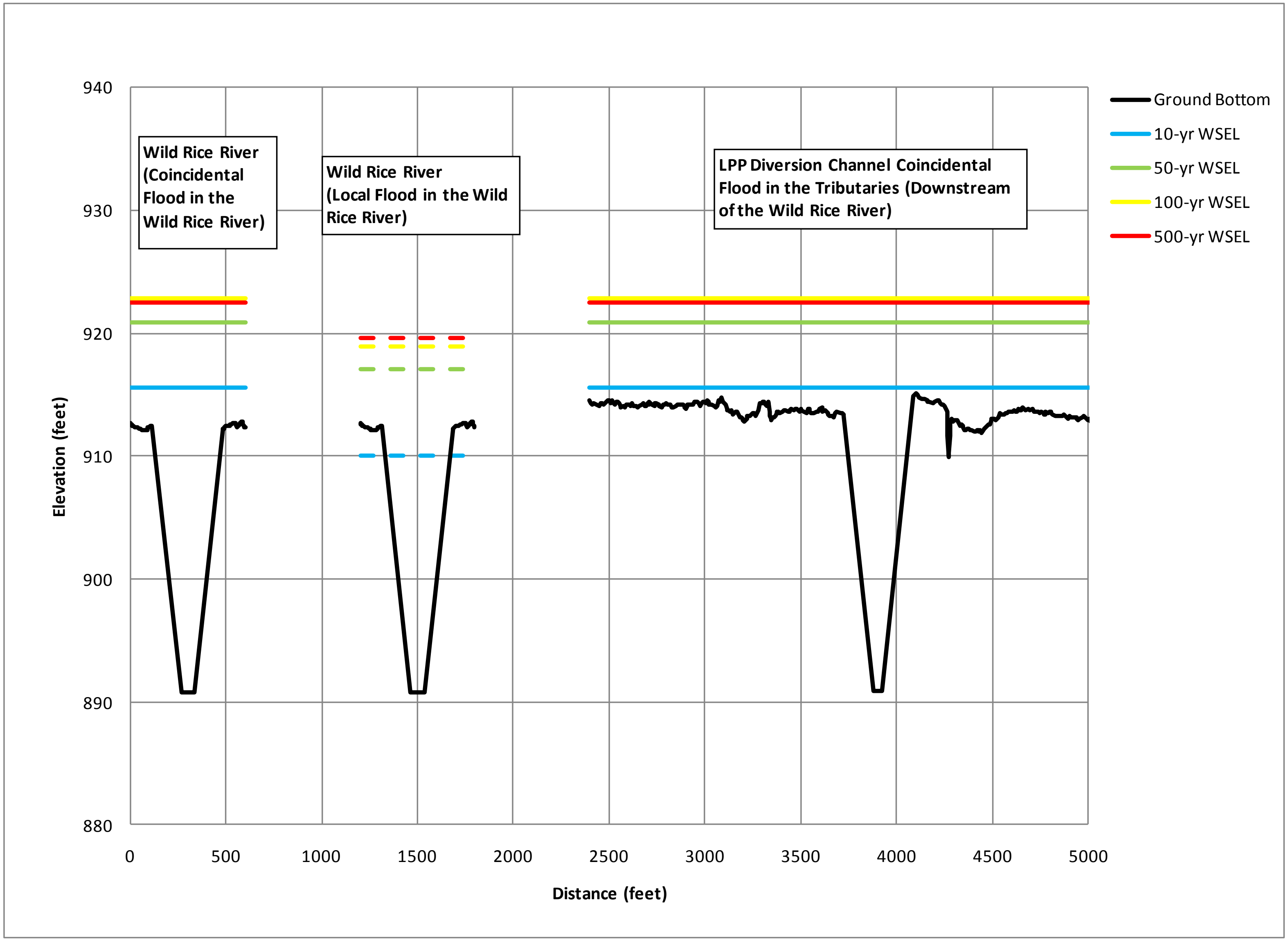


Figure F-D18 Hydrology in the LPP Diversion Channel Downstream of the Primary Inlet Structure and in the Wild Rice River

F-D2.2.2 Connecting Channel Weir (East Weir)

This hydraulic structure was included in the Phase 3 feasibility design, but it has been dropped from the Phase 4 design because it is not needed given the upstream staging.

F-D2.2.3 Primary Inlet Structure (West Weir)

As a result of modifications to the Diversion Channel there is an approximately 19-ft drop from the spill crest elevation of the inlet weir to the bottom of the Diversion Channel. In order to securely convey flow into the Diversion Channel, the broad crested type inlet weir designed for Phase 3 was modified to an ogee type spillway. To prevent ponding at the inlet weir, the Diversion Channel east of the weir is sloped back towards the Wild Rice River. Figure F-D19 and Figure F-D20 show plan and profile schematics of the proposed Inlet Structure. Additional details regarding the proposed dimensions and location of the primary Inlet Structure on the Diversion Channel to the west of the Wild Rice River are shown in Appendix F.

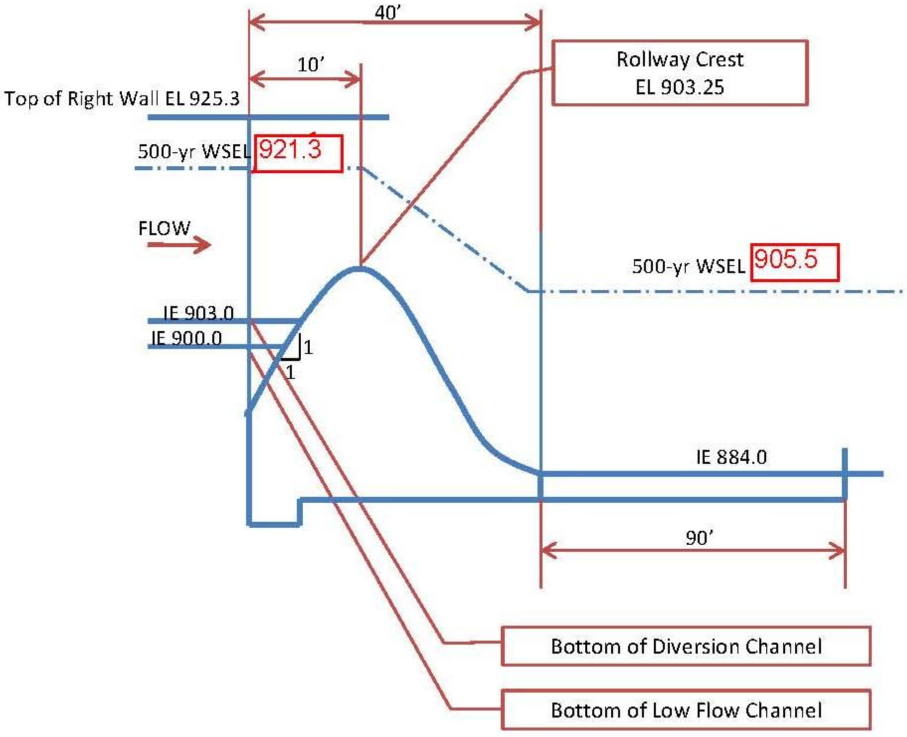


Figure F-D19 Ogee Weir Inlet to Diversion Channel Profile View Schematic

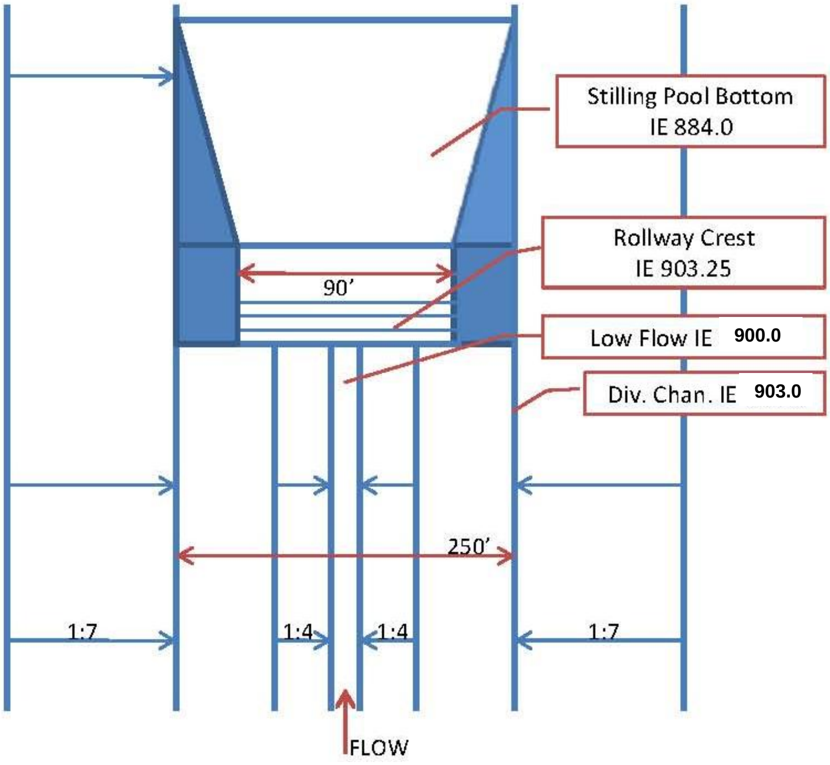


Figure F-D20 Ogee Weir Inlet to Diversion Channel Plan View Schematic

The maximum flow over the weir, headwater elevation, tailwater elevation, head difference, and velocities over the inlet weir for the 10-, 50-, 100-, and 500-year events are presented in Table F-D11. These variables were obtained from the HEC-RAS models with the exception of velocity over the weir. HEC-RAS does not provide this variable so it was calculated by dividing the flow by the flow area over the inlet weir. A comparison of headwater and tailwater elevations across the inlet structure (Sta. 152522) are presented in Figure F-D21 for peak flows on the Red River using Phase 4 hydrology.

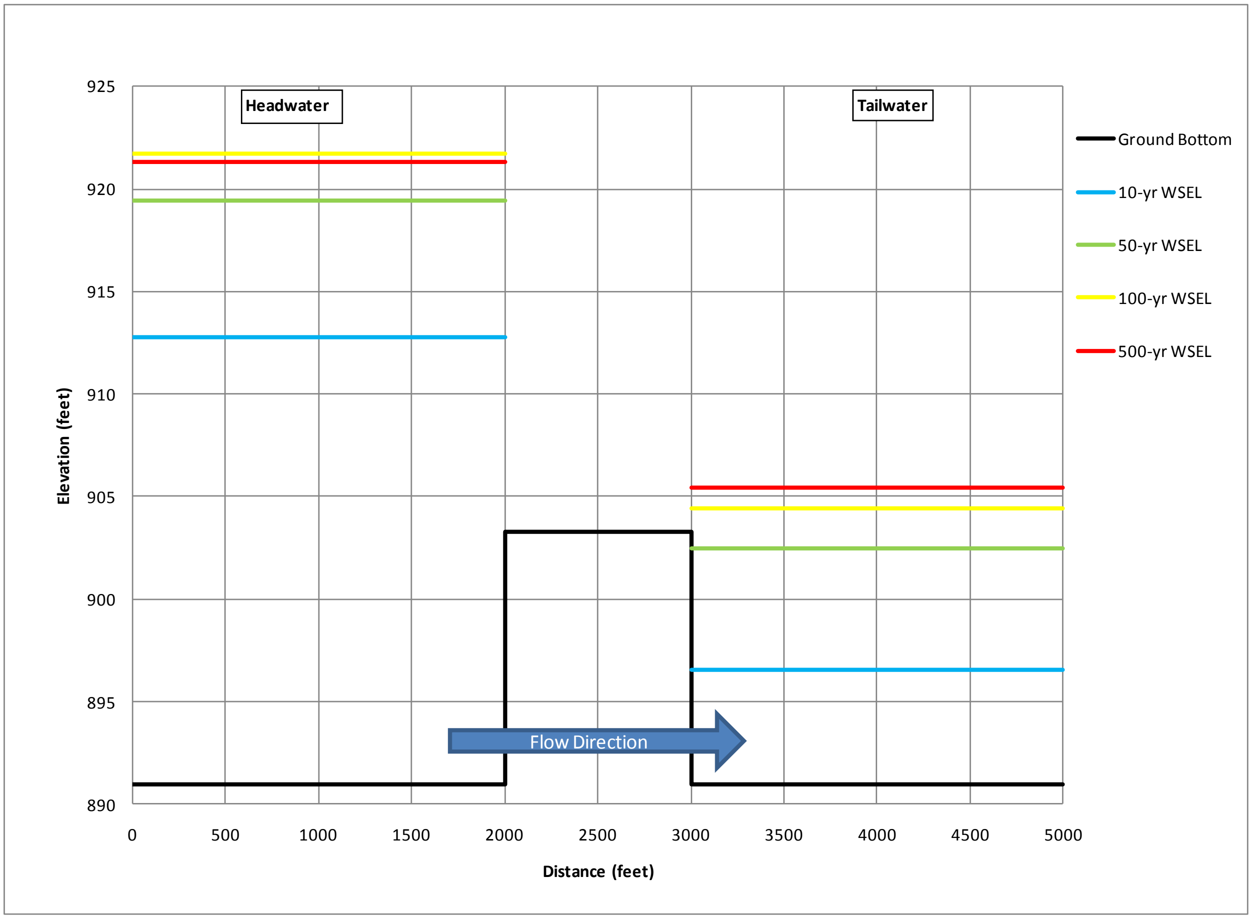


Figure F-D21 **Comparison of the Headwater and Tailwater Elevations at the LPP Diversion Channel Inlet Weir for Coincidental Hydrology**

Table F-D11 HEC-RAS Maximum Outputs for the LPP Diversion Channel Primary Inlet Weir

	Phase 3[1]					Phase 4				
Event (yr)	Flow over Weir (cfs)	Headwater Elevation (ft)	Tailwater Elevation (ft)	Head Difference (ft)	Velocity (ft/s)	Max. Flow over Weir (cfs)[2]	Max. Headwater Elevation (ft)[2]	Max. Tailwater Elevation (ft)[2]	Head Difference (ft)	Max. Velocity (ft/s)[2]
5	2,534	906.18	896.69	9.49	3.97	-	-	-	-	-
10	7,284	908.51	902.03	6.48	5.64	7,207	912.74	896.57	16.17	8.2
50	19,387	912.75	908.92	3.83	7.82	15,737	919.44	902.44	17.00	10.6
100	24,178	914.16	911.18	2.98	8.42	19,046	921.71	904.44	17.27	11.3
500	34,915	916.93	915.60	1.33	8.35	18,404	921.28	905.46	15.82	11.1

[1] Phase 3 Year 0 hydrology

[2] Maximum headwater elevation and maximum tailwater elevation do not occur at same time due to gate operations.

F-D2.3 CONTROL STRUCTURE ON THE WILD RICE RIVER

For the LPP alignment, a control structure on the Wild Rice River is necessary at the confluence of the Connecting Channel and the Wild Rice River. The combination of the Wild Rice River Control Structure and the Inlet Structure (primary inlet to the diversion channel; see Section F-D2.2) split the upstream flow in the Connecting Channel (during flood events) and the upstream flow in the Wild Rice River between the Diversion Channel and the protected area of the Wild Rice River. Based on Phase 3 hydrology, up to approximately between the 2-year and 5-year flood event in the Wild Rice River, it is expected that the Wild Rice River Control Structure will divert water into the Connecting Channel which will flow back to the Red River (this flow is the reverse of what will occur for larger flood events).

This portion of Exhibit D presents design concepts and calculations for the Wild Rice River Control Structure, detailed HEC-RAS modeling outputs, and comparisons of water surface elevations.

F-D2.3.1 Flow Partitioning

The partitioning of the flow between the protected area downstream of the Wild Rice River Control Structure and the Diversion Channel downstream of the Wild Rice River was performed using the HEC-RAS unsteady flow model. If the gates begin to close, in anticipation of a flood event that will exceed 9,600 cfs, it is possible for the inlet weir to be overtopped when a lower flow rate occurs at the USGS gage in Fargo. During the events analyzed as part of Phase 4, the gates at the Wild Rice River control structure are partially closed which results in water being stored upstream of the control structure.

F-D2.3.2 Gate Number, Sizing, and Optimization Using Orifice Equation and Modeling Results

The same general design concept (identical, wide gates) used for the Red River Control Structure was applied to the Wild Rice River Control Structure. Preference was given for multiple, identically sized gates for maintenance purposes. Also, it is desirable to have minimal impact on smaller, higher frequency, flood events (e.g. up to the 2-year event); thus, a minimum open width is necessary to avoid flow constriction during smaller flood events. The gate height was determined based on the staging elevation during the 500-year event. The gate opening during each design event was determined based on the total allowable flow into the protected area to reach the target elevation at the USGS gage in Fargo. During Phase 4, the gate openings on the Wild Rice River remained constant throughout the duration of the flood event, and flows passed into the protected area were mostly controlled by the control structure on the Red River. During subsequent design phases a detailed operations plan should be generated to control the gates at both the Wild Rice River and Red River control structures simultaneously.

Similar to the Red River control structure flow through the Wild Rice River control structure gates were calculated using the orifice equation. The discharge coefficient varied throughout the event based on the level of submergence at the structure.

Section 2.1.4 contains a summary of how flow through the control structure was calculated.

Water surface elevations downstream of the control structure were estimated using the HEC-RAS unsteady flow model. During previous design phases a gate width of 30-ft was selected based on recommendations from the USACE – Environmental. The 30-ft height of the control structure is based on the upstream WSEL during the 500-year event. A concept drawing for the control structure is presented in Figure F-D22.

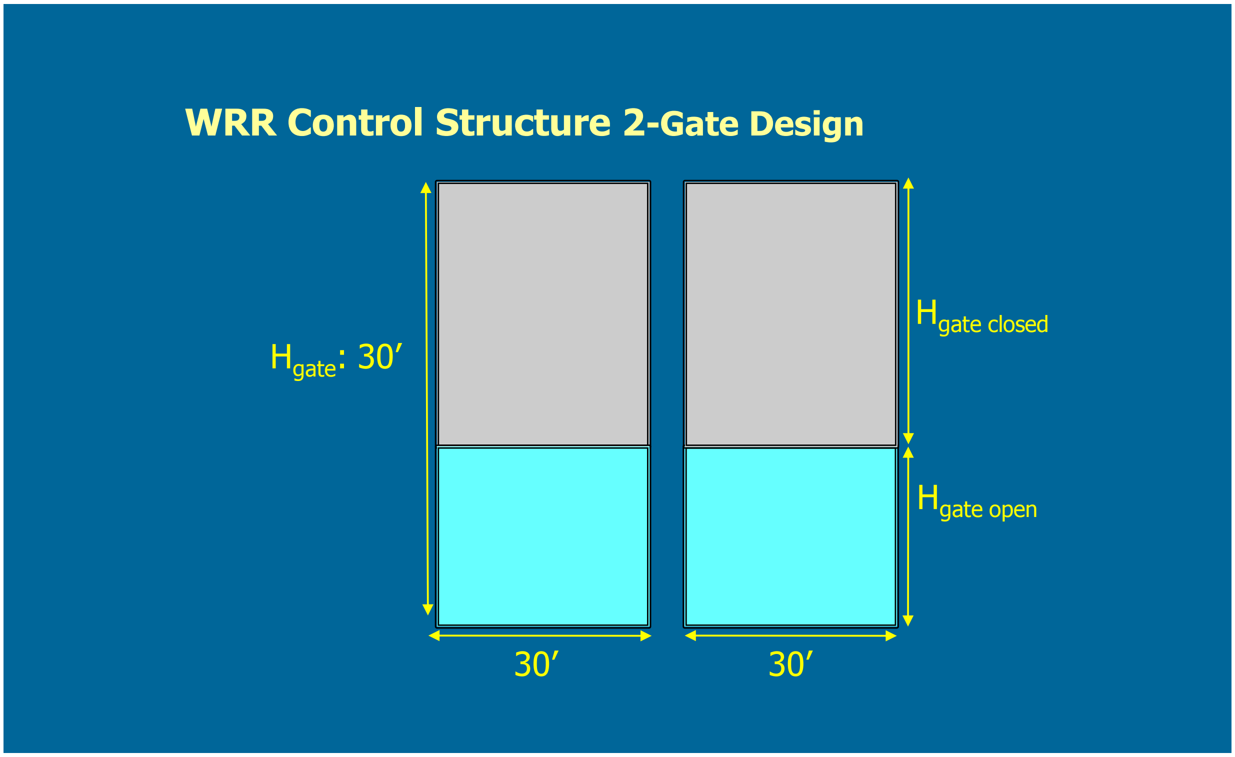


Figure F-D22   LPP Wild Rice River Control Structure 2-Gate Design

Maximum flows, water surface elevations, head difference, gate opening height and velocities for varying flood events at the LPP Wild Rice River Control Structure are presented in Tables F-D12 and F-D13. These variables were obtained from the HEC-RAS unsteady flow models, with the exception of velocity through the gates. HEC-RAS does not provide this variable so it was calculated by dividing the flow by the flow area. Figures F-D23 and F-D24 shows the comparison of the headwater and tailwater elevations for various flood events at the LPP Control Structure.

Table F-D12 Modeling Outputs for the Proposed Wild Rice River Control Structure Phase 4 Hydrology

	Phase 3							Phase 4						
Event (yr)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Wild Rice River* (ft)	Proposed Tailwater Elevation in Wild Rice River** (ft)	Head Difference (ft)	Gate Open Height (ft)	Velocity through Gates (ft/s)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Wild Rice River*** (ft)	Proposed Tailwater Elevation in Wild Rice River*** (ft)	Head Difference (ft)	Gate Open Height (ft)	Velocity through Gates (ft/s)
10	6185	2350	908.65	905.30	3.35	3.33	11.8	6,393	1,114	915.61	901.43	14.18	0.75	23.2
50	11655	2350	912.93	905.36	7.56	2.22	17.7	8,641	459	920.84	900.73	20.11	0.25	30.6
100	13780	2350	914.33	905.48	8.85	2.05	19.1	8,503	387	922.86	901.56	21.3	0.2	38.8
500	18342	2350	917.11	911.12	5.99	2.49	15.7	8,767	1,686	922.45	911.90	10.55	1.6	17.6

Coinc. indicates e.g., the 10-year coincidental event

* Headwater located at HEC-RAS Station 177958.79 of with-project model (between the east and west weirs within the diversion)

**Tailwater located at HEC-RAS Station 12.85 of Wild Rice River downstream of control structure

*** Headwater and Tailwater taken from IS 66102

Table F-D13 Modeling Outputs for the Proposed Wild Rice River Control Structure Phase 4 Local Peak Flows on the Tributaries

Phase 4							
Event (yr)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Wild Rice River* (ft)	Proposed Tailwater Elevation in Wild Rice River* (ft)	Head Difference (ft)	Gate Open Height (ft)**	Velocity through Gates (ft/s)
10	5,444	1,790	910.09	902.64	7.45	2.0	15.0
50	8,824	1,982	917.10	903.32	13.78	2.0	16.5
100	8,688	2,027	918.94	903.43	15.51	2.0	16.9
500	9,565	2,037	919.61	909.93	9.68	2.0	17.0

* Headwater and Tailwater taken from IS 66102

** Gate openings for local event were kept constant with coincidental event. During future design phases this should be reviewed and revised.

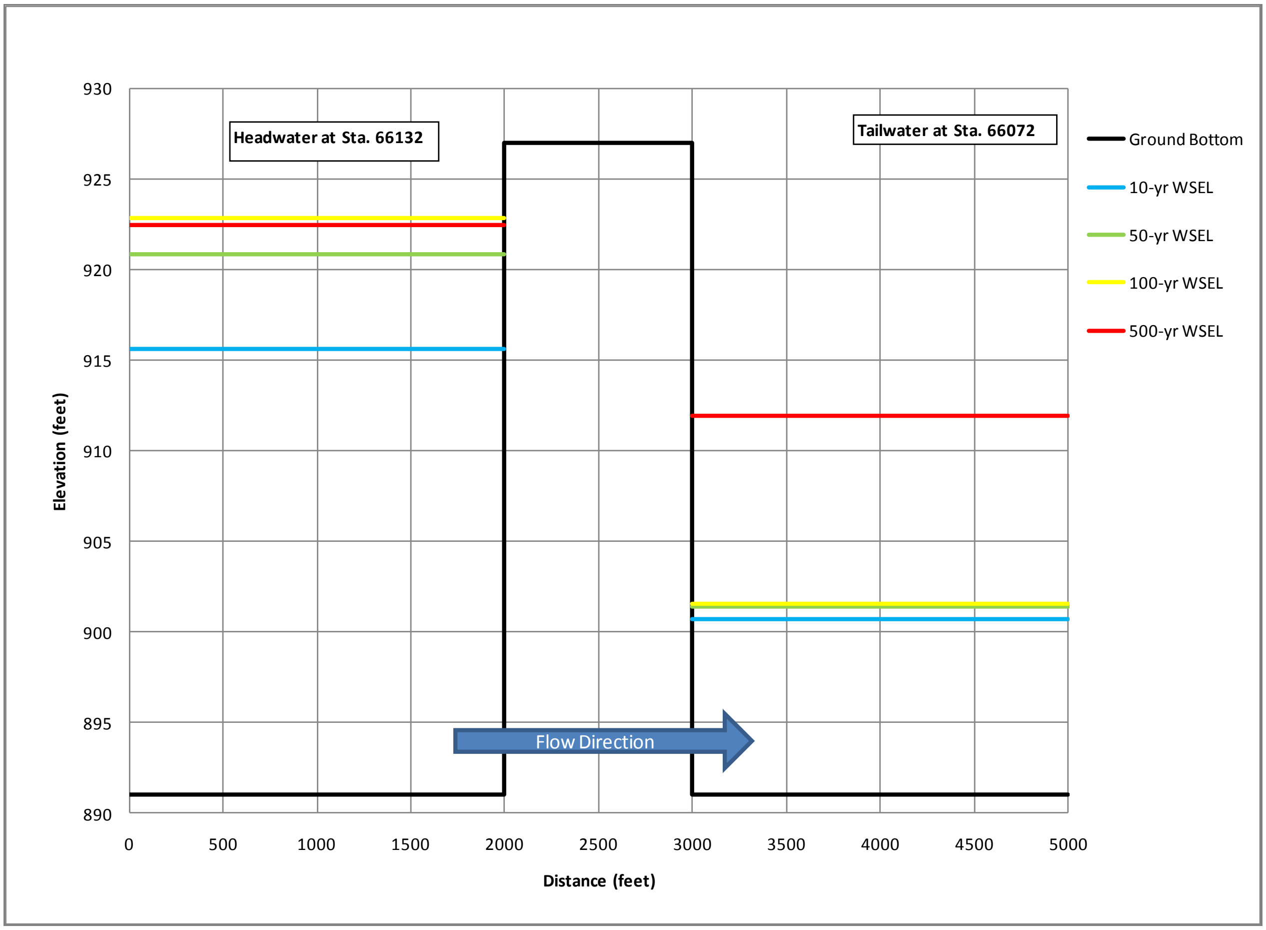


Figure F-D23 Comparison of the Headwater and Tailwater Elevations at the Wild Rice River Control Structure for Coincidental Hydrology

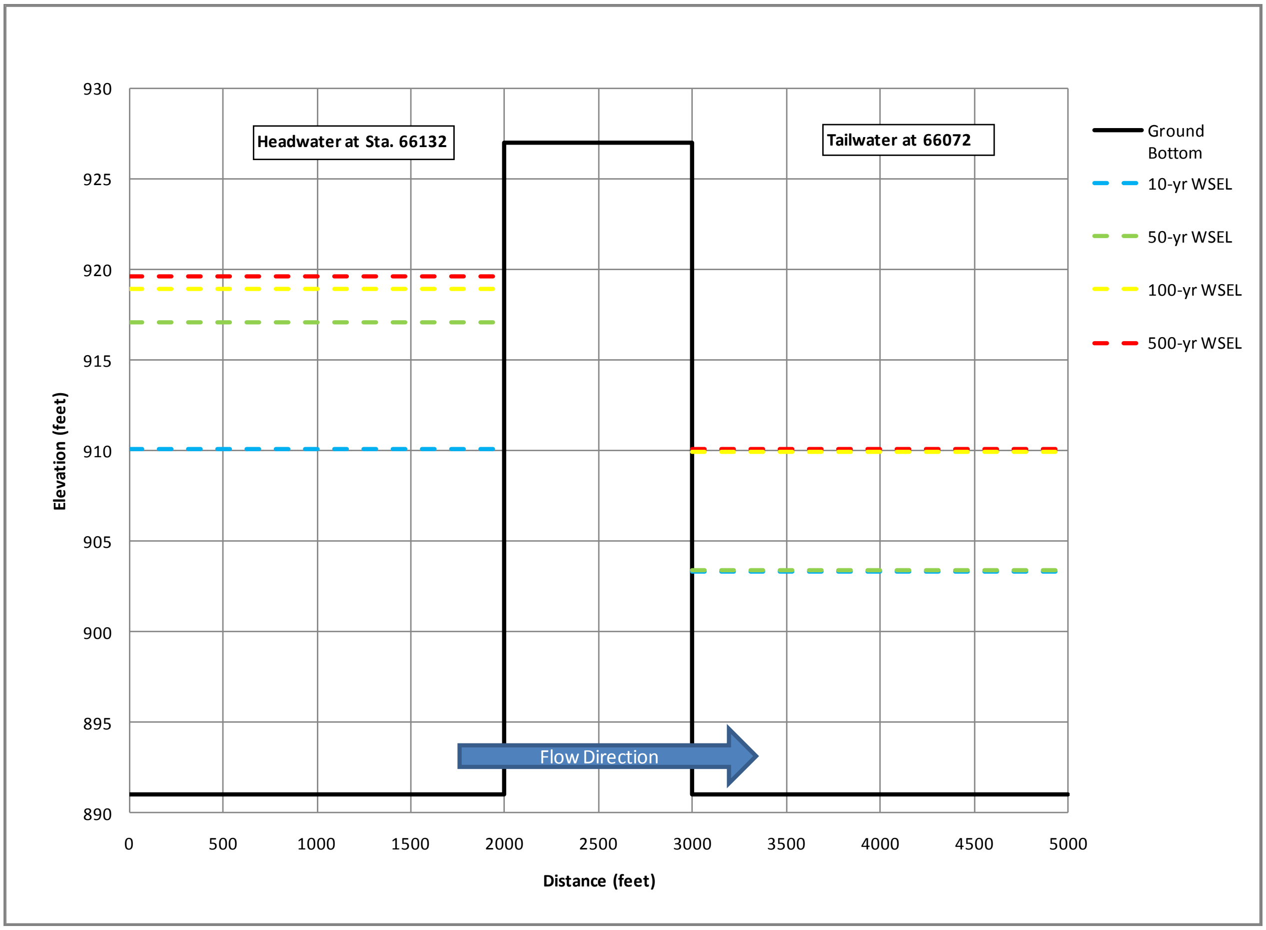


Figure F-D24 Comparison of the Headwater and Tailwater Elevations at the Wild Rice River Control Structure for Local Hydrology

F-D2.3.3 Transition from the Natural Channel to the Control Structure

The transition from the natural channel into the control structure is accomplished with 7:1 side slopes, as shown in Figure F-D25. For more details on the geotechnical slope stability analysis of this reach, see Exhibit N of Appendix F.

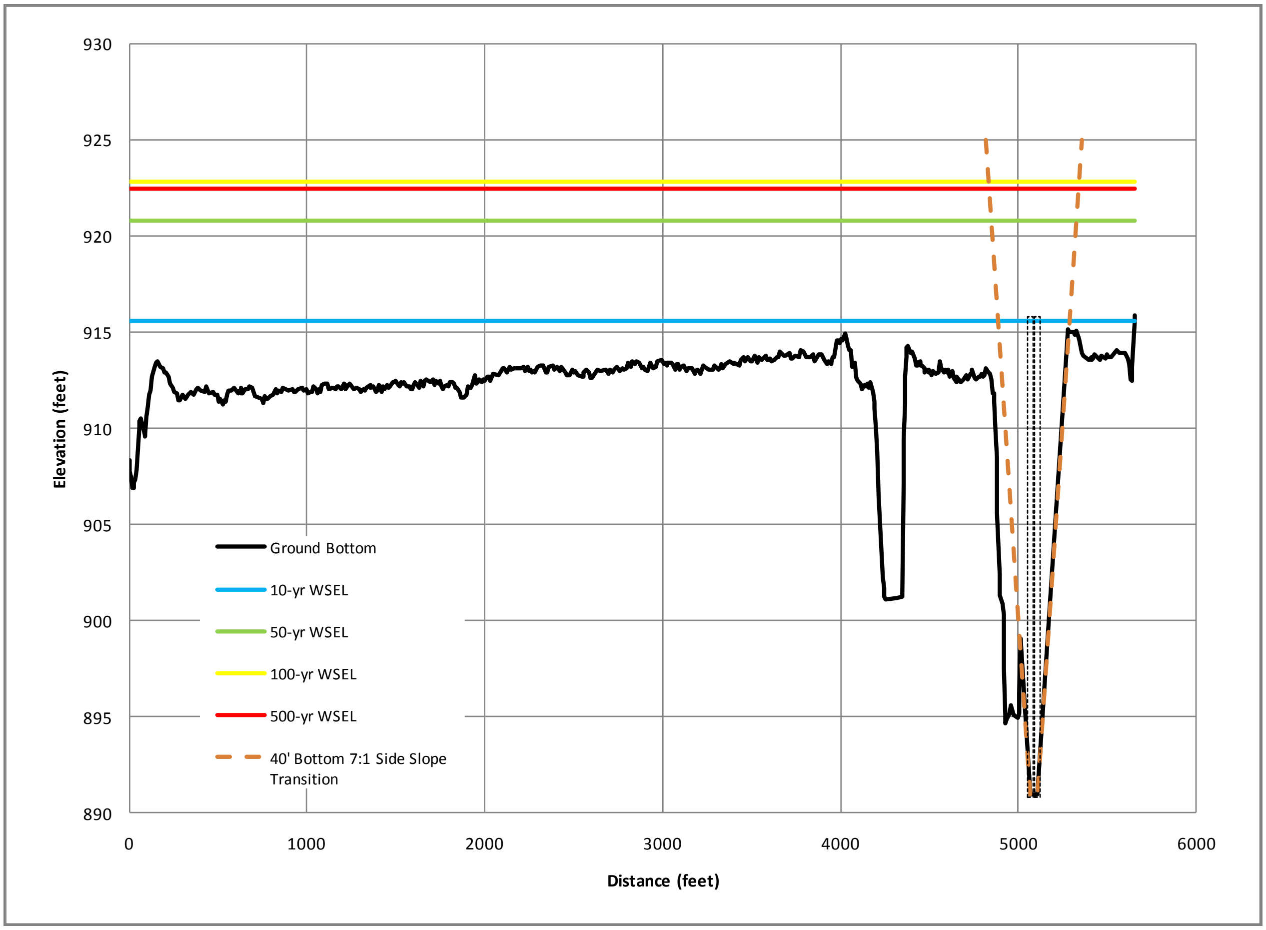


Figure F-D25 **Transition from the Natural Wild Rice River Cross Section to the Wild Rice River Control Structure**

F-D2.4 DIVERSION CHANNEL TRANSITION AND AQUEDUCT AT SHEYENNE RIVER

A description of the LPP Diversion Channel transition and aqueduct structure at the Sheyenne River can be found in the main section of Appendix F. Appendix F, Exhibit C discusses the variety of structure types that were investigated for this tributary crossing during Phase 2 design. This portion of Exhibit D presents a comparison of water surface elevations at the Sheyenne River structure, design calculations of the aqueduct crossing, and detailed HEC-RAS modeling outputs in the Diversion Channel beneath the aqueduct crossing.

F-D2.4.1 Comparison of LPP Diversion Channel and Sheyenne River Hydrology

Maximum flows, water surface elevations, and velocities for the LPP Diversion Channel downstream of the Sheyenne River crossing for the 10-, 50-, 100-, and 500-year events are presented in Table F-D14. The location shown in Table F-D14 is where flows diverted from the Sheyenne River enter the LPP Diversion Channel via the spillway weir and the Diversion Channel has returned to its typical cross section. Water surface elevations in the LPP Diversion Channel just downstream of the Sheyenne River structure, in the Sheyenne River for local tributary flood events, and in the Sheyenne River for coincidental flood events, plotted against the cross section data, are presented in Figure F-D26. Tabular results of existing conditions local and coincidental flood events in the tributaries are presented in Appendix F, Exhibit A.

Table F-D14 LPP Diversion Channel Downstream of the Sheyenne River Crossing

	Phase 3*			**Phase 4****		
Event (yr)	**Flow (cfs)**	**WSEL (ft)**	**Average Velocity (ft/s)**	**Flow (cfs)**	**WSEL (ft)**	**Velocity (ft/s)*****
10	8,406	897.67	2.53	7,353	895.8	2.23
50	22,201	904.55	3.32	17,417	901.6	2.80
100	27,071	906.63	3.45	20,832	903.5	2.85
500	37,808	910.32	3.77	21,068	904.7	2.64

* Model results from Station 148766. Phase 3 HEC-RAS steady flow model.

** Model results from Station 146691. Phase 4 HEC-RAS unsteady flow model.

*** Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

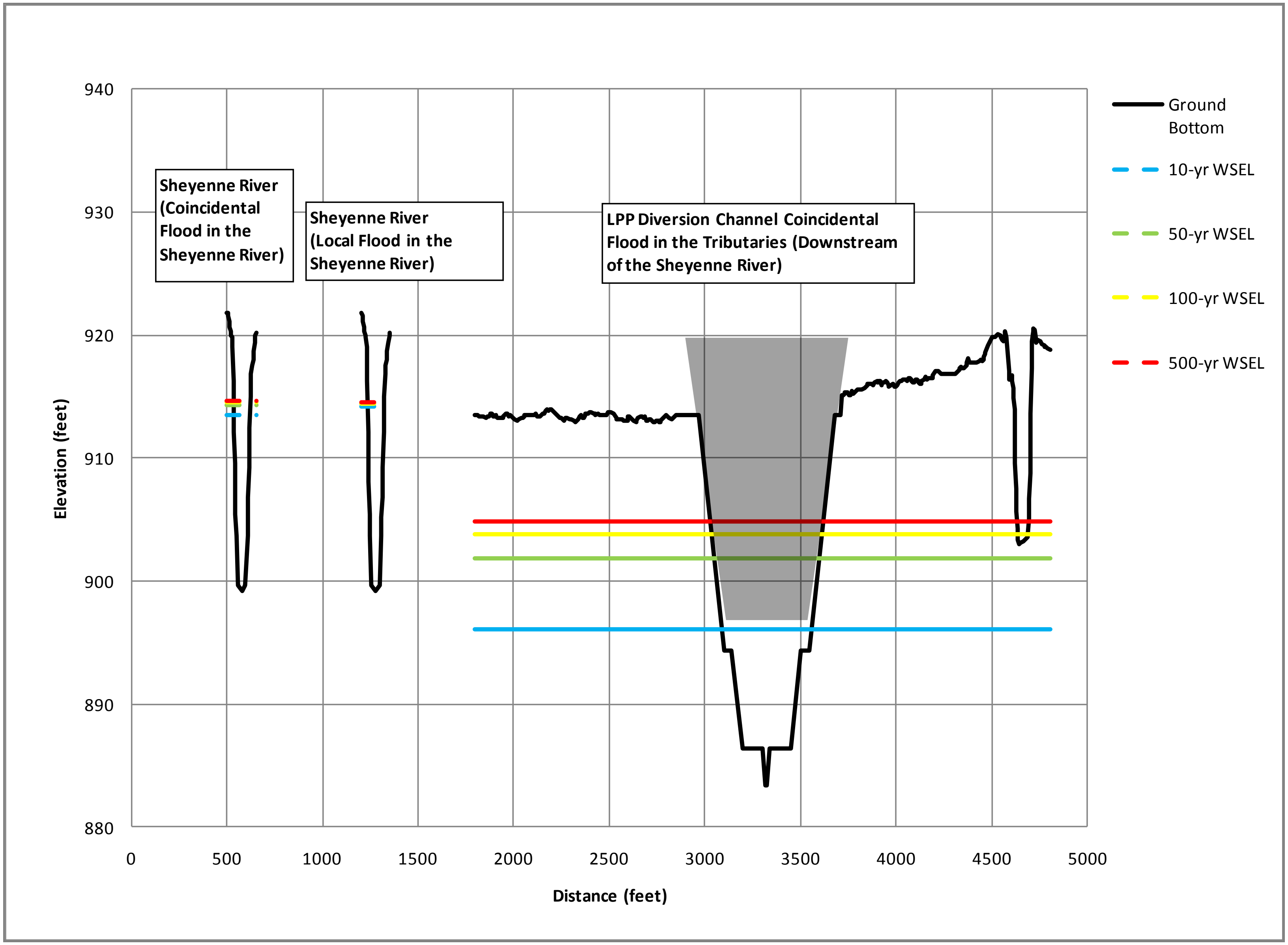
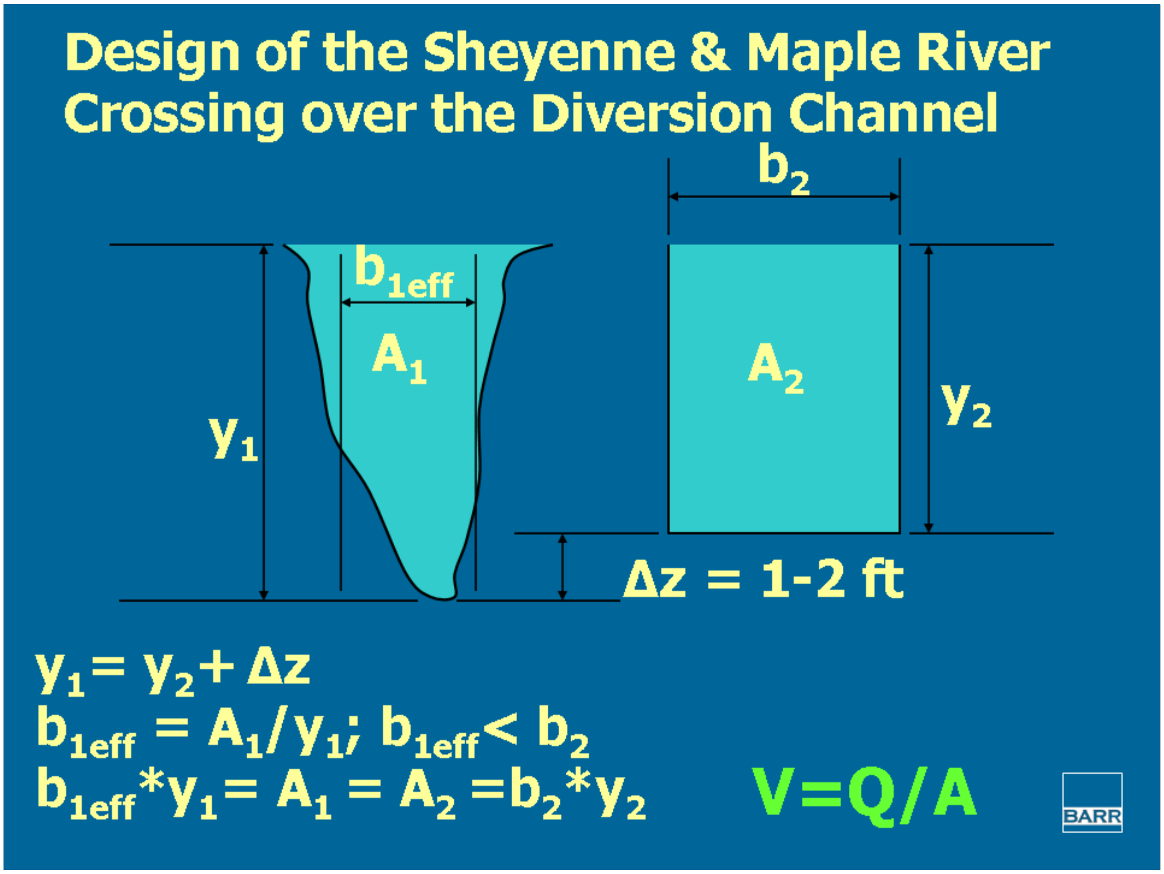


Figure F-D26 LPP Diversion Channel Downstream of the Sheyenne River Structure and in the Sheyenne River

F-D2.4.2 Design of Sheyenne River Open Aqueduct

The Sheyenne River open aqueduct was sized to pass the 2-year local flow in the Sheyenne River, or 1,200 cfs, into the protected area. The aqueduct was sized to maintain the same cross sectional area and flow velocity as the corresponding natural cross section for events smaller than the 2-year event in the tributary, as shown in Figure F-D27. The invert of the aqueduct was lowered approximately one-half foot below the existing invert, from an elevation of 899.2 to 898.7 feet. In comparison to the natural variation in the Sheyenne River's channel bottom, this change should be relatively insignificant since gradual transitions will be used on both the entrance and exit to the aqueduct. Figure F-D28 shows the Sheyenne River thalweg (deepest continuous line along the channel bottom) based on the Phase 4 HEC-RAS unsteady flow model under existing conditions, and with the Sheyenne River aqueduct lowered one-half foot. Table F-D15 present the calculations used in Phase 3 to size the Sheyenne River open aqueduct crossing over the LPP Diversion Channel. The width of the flow area for the Sheyenne River open aqueduct was found to be 46.4 feet. This was rounded to 50 feet for structural design. This design from Phase 3 was used in the Phase 4 analysis. The top of structure was set at 916 feet, one foot above the 500-year local peak event through the aqueduct.



Design of the Sheyenne & Maple River Crossing over the Diversion Channel

$y_1 = y_2 + \Delta z$

$b_{1eff} = A_1/y_1;\ b_{1eff} < b_2$

$b_{1eff} * y_1 = A_1 = A_2 = b_2 * y_2$

$V = Q/A$

Where y is water depth, z is change in bottom invert elevation, b is width, V is velocity, Q is flow and A is area.

Figure F-D27 Schematic Showing Calculations Used to Maintain Cross Section-Averaged Flow Velocity in the Sheyenne and Maple River Aqueduct Crossings.

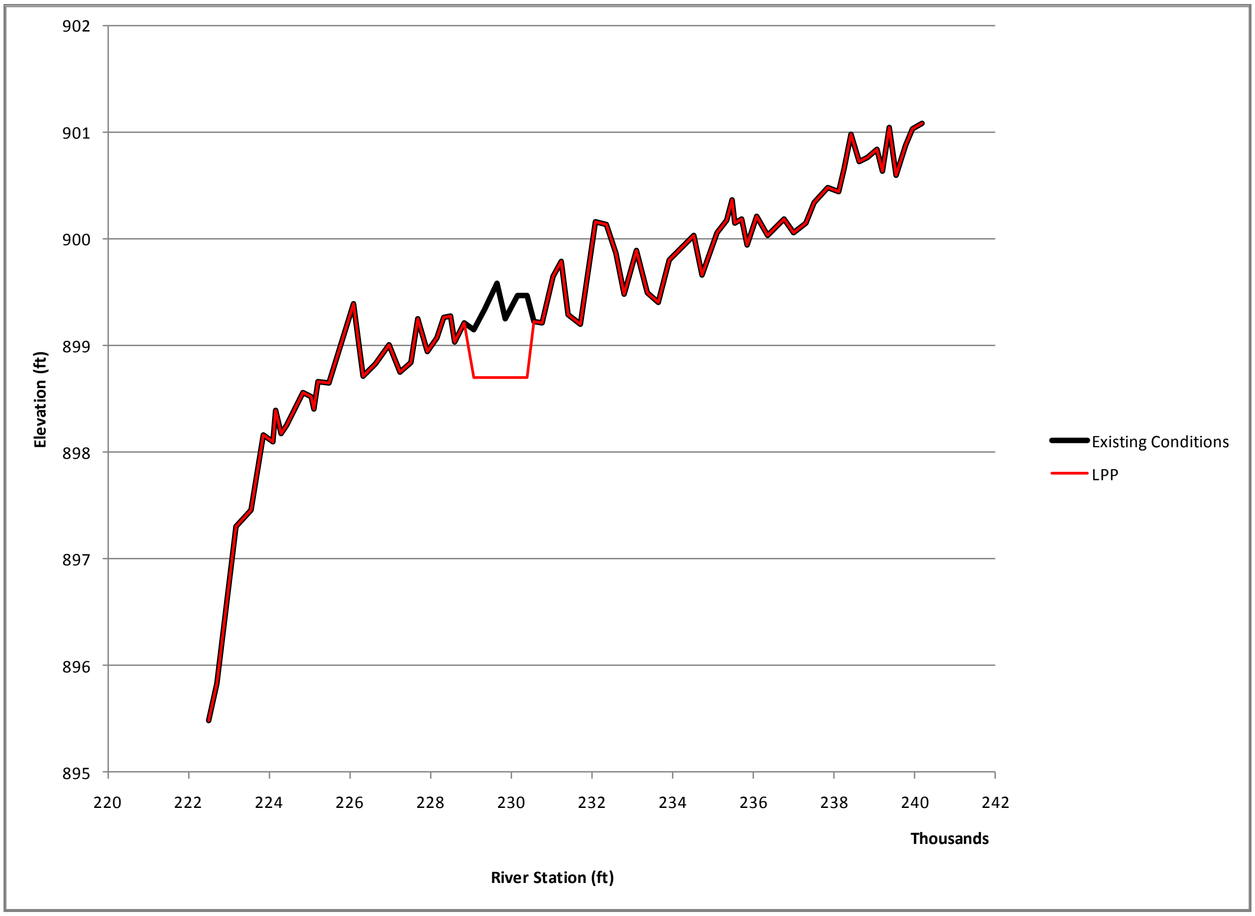


Figure F-D28 Comparison of the Sheyenne River Thalweg under Existing and Proposed Conditions

Table F-D15 Calculations Used to Size the Sheyenne River Open Aqueduct Crossing over the LPP Diversion Channel (from Phase 3)

2-Year Local Event Variables	Existing Natural Cross Section	Open Aqueduct Crossing
Water Surface Elevation (ft)	912.71	912.71
Channel Bottom Invert (ft)	897.7	898.7
Depth of Water (ft)	15.01	14.01
Flow (cfs)	1200	1200
Flow area (ft^2)	650.35	650.35
Average Velocity (ft/s)	1.85	1.85
Acceleration of Gravity (ft/s^2)	32.2	32.2
Average Channel Width (ft)	43.33	46.42
Change in Bottom Invert from Natural Conditions (ft)	0	1
Energy (ft)	15.063	15.063

Depending on the actual head in the Sheyenne River and on how much flow the Sheyenne River spillway weir diverts into the LPP Diversion Channel, somewhat more than the 2-year local event (as much as the 5-year local event) could enter the protected area. For events larger than the 2-year local event when water will be diverted over the spillway weir, the water surface elevation in the Sheyenne River open aqueduct will be less than the water surface elevation under existing conditions (Appendix F, Exhibit A - Background Hydrology). The design of the spillway weir is discussed in this exhibit in Section F-D2.5. The velocities in the Sheyenne River open aqueduct, shown in Tables F-D16 and F-D17, will never be greater than 3 feet per second and will always remain in the range of measured existing velocities as seen in Figure F-A31 Exhibit A.

Table F-D16 Elevations, Velocities and Flows in the Sheyenne River Aqueduct for Coincidental Events

	Phase 3					Phase 4				
Event Return Period (yr)	**Flow in River Upstream of Aqueduct (cfs)**	**WSEL Upstream of Aqueduct (ft)**	**Flow in River into Protected Area (cfs)**	**WSEL of River in Aqueduct (ft)**	**Average Velocity in Aqueduct (ft/s)**	**Flow in River Upstream of Aqueduct (cfs)***	**WSEL Upstream of Aqueduct (ft)**	**Flow in River into Protected Area (cfs)***	**WSEL of River in Aqueduct (ft)**	**Average Velocity in Aqueduct (ft/s)***
Mean Annual Flow	236	903.24	236	903.27	1.11	-	-	-	-	-
2 – Local	1200	912.71	1200	912.71	1.85	-	-	-	-	-
2	1325	913.27	1200	913.09	1.85	-	-	-	-	-
5	1935	915.15	1350	914.34	1.92	-	-	-	-	-
10	2,565	916.31	1,443	915.12	1.95	1,783	913.5	971	913.5	1.38
50	4,510	919.37	1,696	917.16	2.04	3,629	914.4	1,151	914.5	1.52
100	4,600	919.50	1,707	917.25	2.05	4,176	914.8	1,160	914.8	1.51
500	4,600	919.50	1,707	917.25	2.05	4,368	914.9	1,073	914.9	1.38

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

Table F-D17 Elevations, Velocities and Flows in the Sheyenne River Aqueduct for Local Events

Event Return Period (yr)	Flow in River Upstream of Aqueduct (cfs)*	WSEL Upstream of Aqueduct(ft)	Flow in River into Protected Area (cfs)*	WSEL of River in Aqueduct (ft)	Average Velocity in Aqueduct (ft/s)*
Phase 4					
10	3,996	914.4	1,816	914.3	2.44
50	4,479	914.6	1,905	914.5	2.52
100	4,526	914.7	1,918	914.5	2.54
500	4,671	914.8	1,960	914.6	2.57

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

Table F-D16 shows a significant difference in the water surface elevations between Phase 3 and Phase 4. For events greater than or equal to the 50-year event, the Phase 4 WSELs are between 4 and 5 feet lower than the Phase 3 WSELs. This is the reason why the spillway width has changed from the previous design of 55 ft wide to the current design of 300 ft wide. The head on the weir is much less in Phase 4, necessitating a significantly wider weir to maintain flows through the aqueduct less than 2,000 cfs, such that the current level of flood protection provided by the Horace to West Fargo diversion is maintained.

The transition from the natural channel into the open aqueduct is accomplished with 5:1 side slopes as shown in Figure F-D29. For more details about the slope stability analysis of this reach, see Exhibit N of Appendix F.

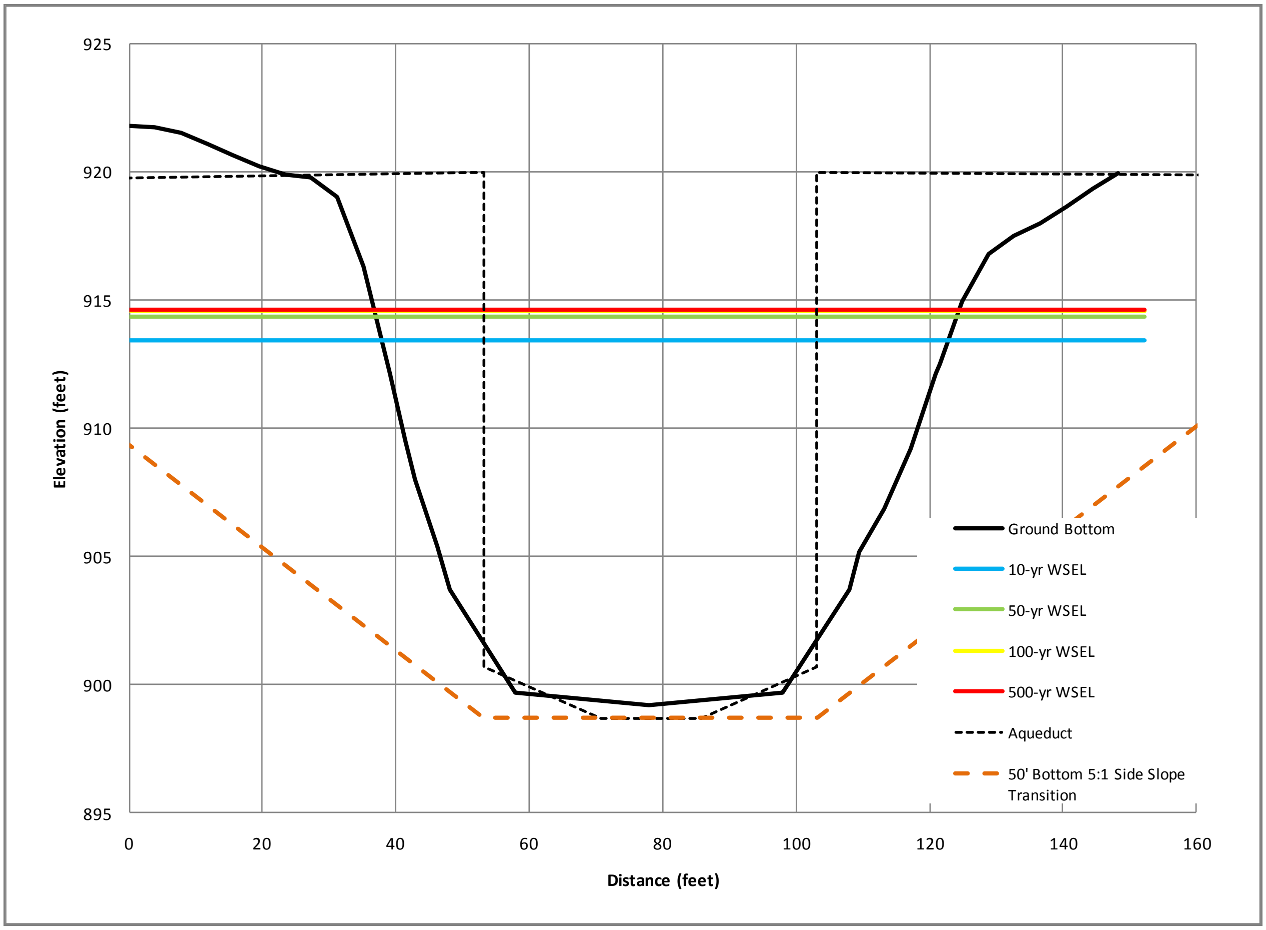


Figure F-D29 Transition from the Natural Sheyenne River Cross Section into the Aqueduct

F-D2.4.3 Design of the Low Flow Channel in the Sheyenne River Aqueduct

The Sheyenne open aqueduct will include a smaller winter channel to ensure that low flows during winter will not freeze. According to records from 1949 to 2008 at the USGS gage at Kindred, the mean annual flow from December to February is 99.5 cfs. Using Manning's equation, a low flow channel with dimensions of 10 feet by 10 feet would convey this flow across the open aqueduct. The lowest recorded flow in the winter months is 17.5 cfs. In the next phase of design, the design of the low flow channel could be further refined to look at a variety of low flow and freezing conditions that the structure may encounter and to optimize dimensions to encourage fish passage during very low flows.

F-D2.4.4 Design of LPP Diversion Channel Transition at the Sheyenne River Structure

No mixing of Sheyenne River and Diversion Channel water will occur at the open aqueduct. Flow in the Sheyenne River open aqueduct will be able to overtop the top of the wall at an elevation of 916 feet into the LPP Diversion Channel but would necessitate an event larger than the 500-year event. In Phase 4, flow through the aqueduct was completely contained within the aqueduct.

The opening under the aqueduct was maximized to minimize the head difference in the diversion channel across the structure. The bottom width of the diversion channel is 250 feet so the width of the opening was set at 250 feet. The number of piers and the size of each pier were set based on structural stability rather than flow conditions. The design was checked in the HEC-RAS unsteady flow model to ensure that the head drop across the structure was acceptable. Pressurized flow underneath the aqueduct in the Diversion Channel will occur for events in the Red River greater than or equal to the 50-year event. The proposed dimensions and location of the Sheyenne River structure are shown in Appendix F.

The total flow in the LPP Diversion Channel beneath the aqueduct, headwater elevation, tailwater elevation, head difference, and average velocities for the 10-, 50-, 100-, and 500-year events are presented in Tables F-D18 and F-D19. These variables were obtained from the HEC-RAS models with the exception of average velocity beneath the aqueduct. HEC-RAS does not provide this variable so it was calculated by dividing the flow by the flow area.

Table F-D18 Coincidental Event HEC-RAS Outputs for the LPP Diversion Channel at the Sheyenne River Aqueduct

	Phase 3					Phase 4				
Event (yr)	**Flow in Diversion Channel (cfs)**	**Headwater Elevation in Diversion Channel (ft)**	**Tailwater Elevation in Diversion Channel (ft)**	**Head Difference (ft)**	**Average Velocity (ft/s)**	**Flow in Diversion Channel (cfs)***	**Headwater Elevation in Diversion Channel (ft)**	**Tailwater Elevation in Diversion Channel (ft)**	**Head Difference (ft)**	**Average Velocity (ft/s)***
10	7,284	897.87	897.82	0.05	1.94	7,161	896.3	896.3	0.0	2.5
50	19,387	905.19	904.69	0.50	5.17	15,437	902.2	902.0	0.2	5.2
100	24,178	907.58	906.77	0.81	6.45	18,560	904.2	903.9	0.3	6.2
500	34,915	912.26	910.44	1.82	9.32	18,099	905.2	904.9	0.3	6.1

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

Table F-D19 Local Event HEC-RAS Outputs for the LPP Diversion Channel at the Sheyenne River Aqueduct

	Phase 4				
Event (yr)	**Flow in Diversion Channel (cfs)***	**Headwater Elevation in Diversion Channel (ft)**	**Tailwater Elevation in Diversion Channel (ft)**	**Head Difference (ft)**	**Average Velocity (ft/s)***
10	1,930	890.7	890.7	0.0	1.2
50	9,079	898.0	897.9	0.1	3.0
100	11,726	899.8	899.8	0.0	3.9
500	12,674	901.4	901.3	0.1	4.3

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

F-D2.5 SPILLWAY AT SHEYENNE RIVER

A weir spillway has been selected to divert water from the tributary into the LPP Diversion Channel. The design elevation of the Sheyenne River spillway in Phase 4 is fixed at 912.56 ft, which is the water surface elevation on the Sheyenne River associated with the 2-year local flood event (1,200 cfs). The water surface elevations in the LPP Diversion Channel are at least 2 feet below the weir crest elevation, so submerged flow conditions do not occur over the weir. Under these conditions, a weir coefficient of 3.1 is appropriate and was used. The width of the spillway is the only variable that was altered. Starting at the design width of 55 feet from Phase 3, the width was increased until the desired flow split was met. The local peak flow of the 500-year event on the Sheyenne River was the limiting event used to size the weir since it had higher flows and lower WSELs than the coincidental values (to peaks in the Red River of the North). The maximum flow allowed to pass through to the protected side is 2,000 cfs due to limits in the existing Horace to West Fargo diversion channel. Based on this constraint, the weir is designed to be 300 feet wide.

Table F-D20 presents the results from Phase 3 using the HEC-RAS steady flow model. Tables F-D21 and F-D22 present the results from Phase 4 using the HEC-RAS unsteady flow models.

Table F-D20 Phase 3 Design Results for the 55-Ft.-Wide Sheyenne River Spillway Using the Coincidental Flow (spillway elevation 912.71 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Flow into Protected Area (cfs)
10-year	916.3	2,565	1,122	1,443
50-year	919.4	4,510	2,814	1,696
100-year	919.5	4,600	2,893	1,707
500-year	919.5	4,600	2,893	1,707

Table F-D21 Phase 4 Design Results for the 300-Ft.-Wide Sheyenne River Spillway Using the Coincidental Flow (spillway elevation 912.56 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Aqueduct WSEL (ft)	Flow into Protected Area (cfs)
10-year	913.5	1,783	812	913.5	971
50-year	914.5	3,629	2,478	914.5	1,151
100-year	914.8	4,176	3,016	914.8	1,160
500-year	914.9	4,368	3,295	914.9	1,073

Table F-D22 Phase 4 Design Results for the 300-Ft.-Wide Sheyenne River Spillway Using the Peak Flow (spillway elevation 912.56 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Aqueduct WSEL (ft)	Flow into Protected Area (cfs)
10-year	914.4	3,996	2,180	914.3	1,816
50-year	914.6	4,479	2,574	914.5	1,905
100-year	914.7	4,526	2,609	914.5	1,918
500-year	914.8	4,671	2,712	914.6	1,960

Water surface elevations were also checked along the entire reach of the Sheyenne River to ensure that the aqueduct structure was not causing significant upstream flooding. Figures F-D30 through F-D37 compare existing conditions and LPP conditions along the entire reach of the Sheyenne River. These figures confirm that the aqueduct structure allowing the Sheyenne River to cross the LPP diversion channel is not adversely affecting the upstream portion of the reach. In fact, it is lowering water levels through and downstream of the aqueduct during the large events shown. As discussed, this structure will not affect flows less than or equal to the 2-year event due to the elevation of the spillway.

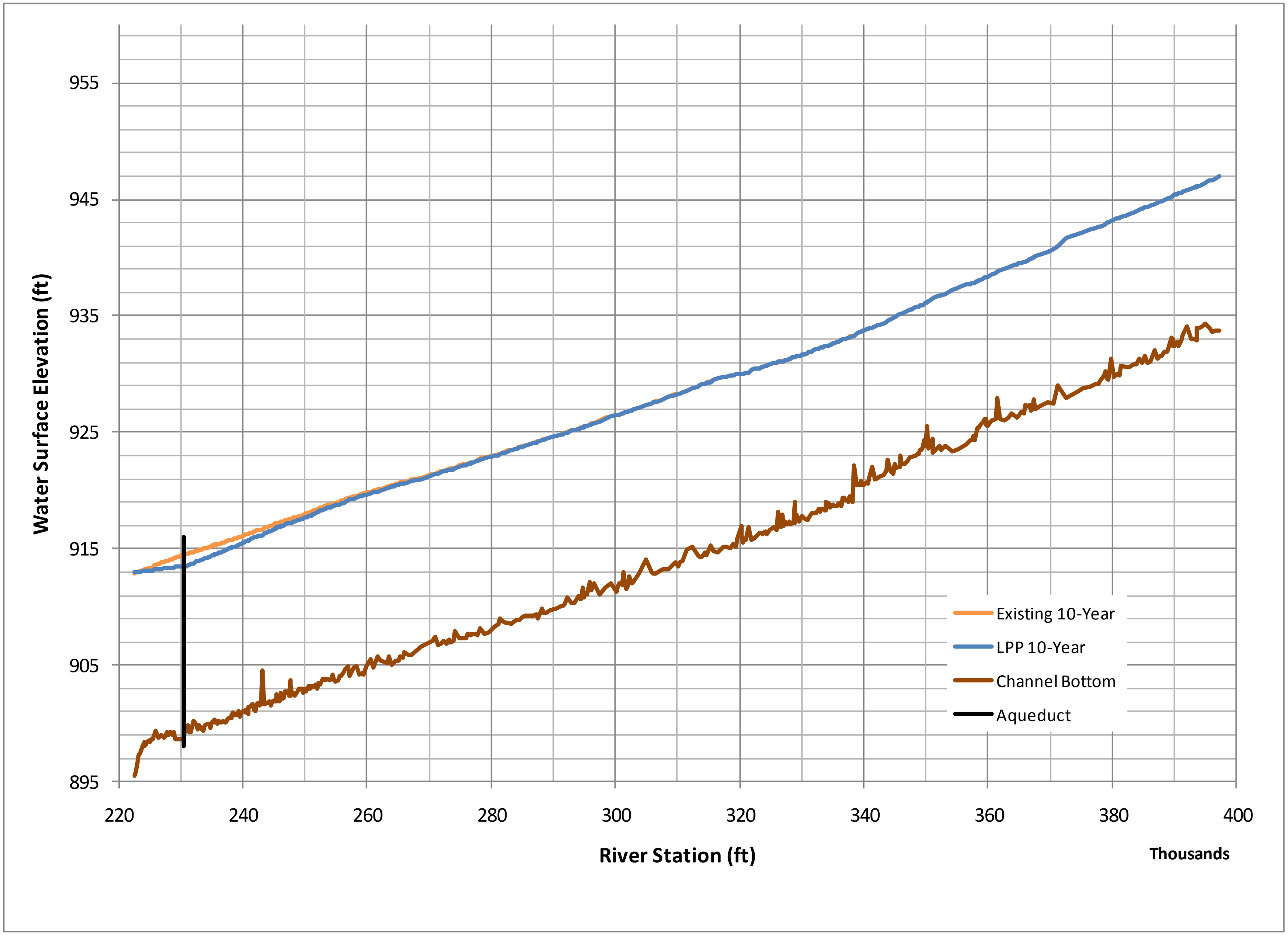


Figure F-D30 **Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Coincidental 10-Year Event**

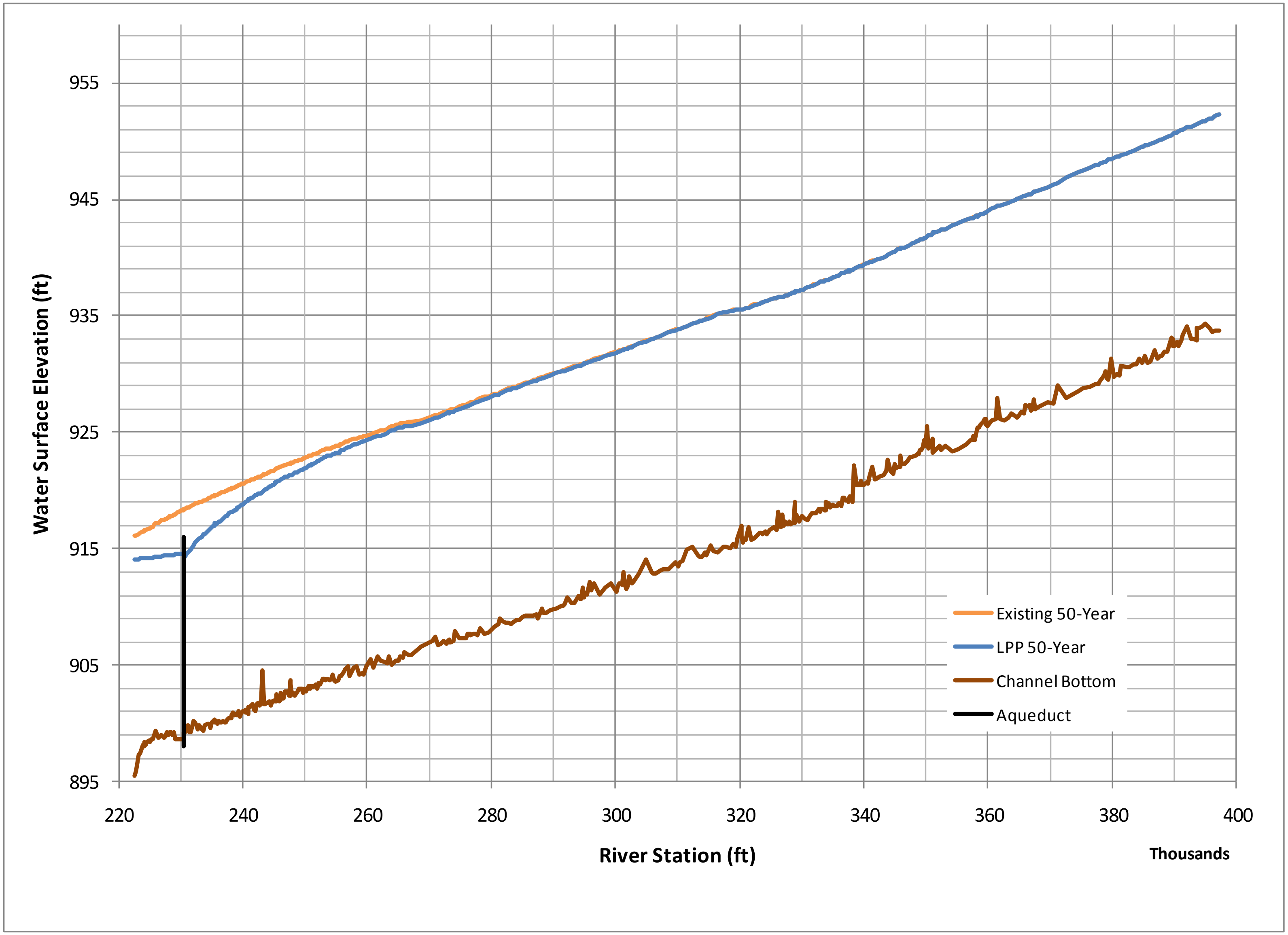


Figure F-D31 Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Coincidental 50-Year Event

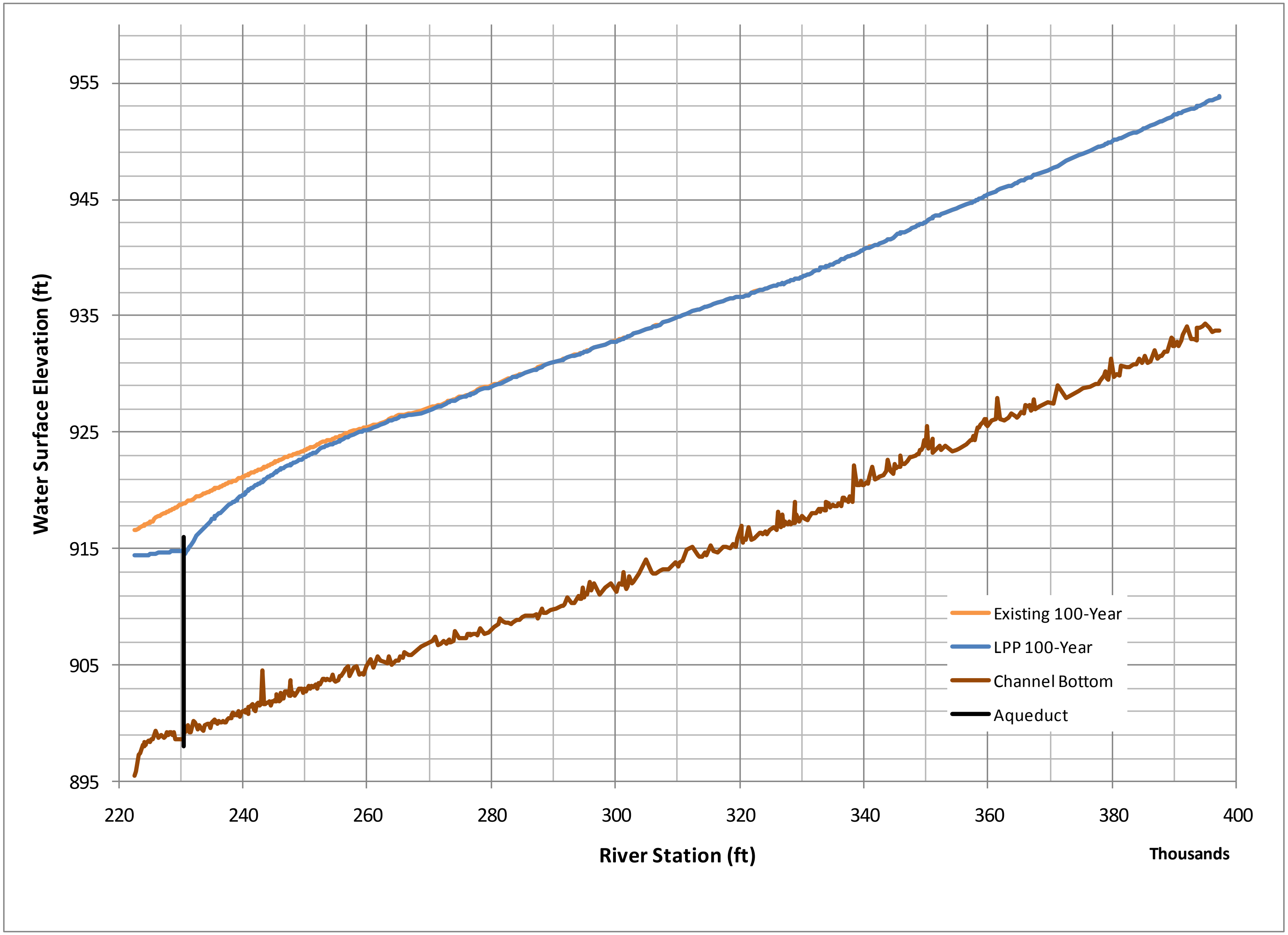


Figure F-D32 **Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Coincidental 100-Year Event**

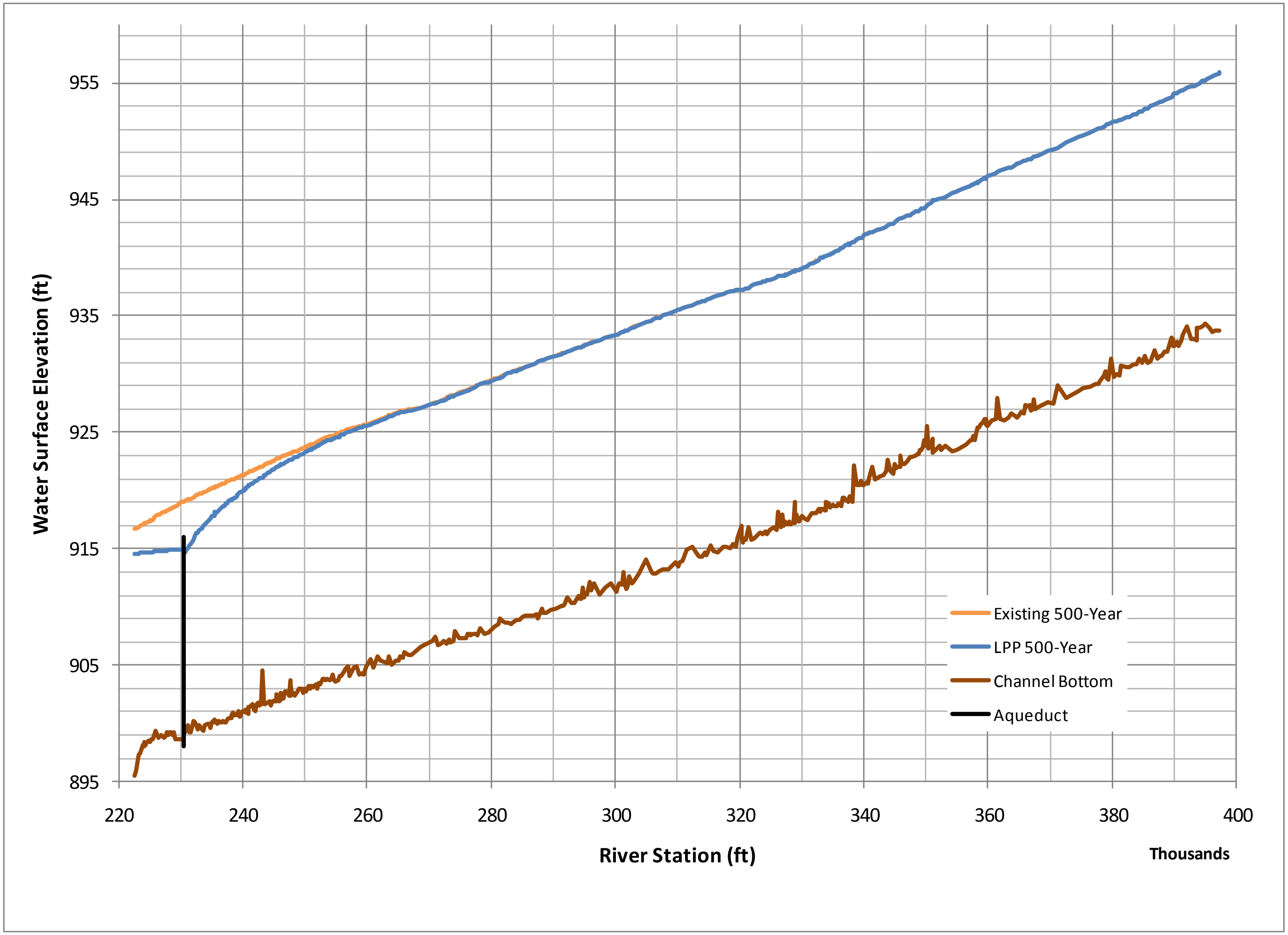


Figure F-D33 **Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Coincidental 500-Year Event**

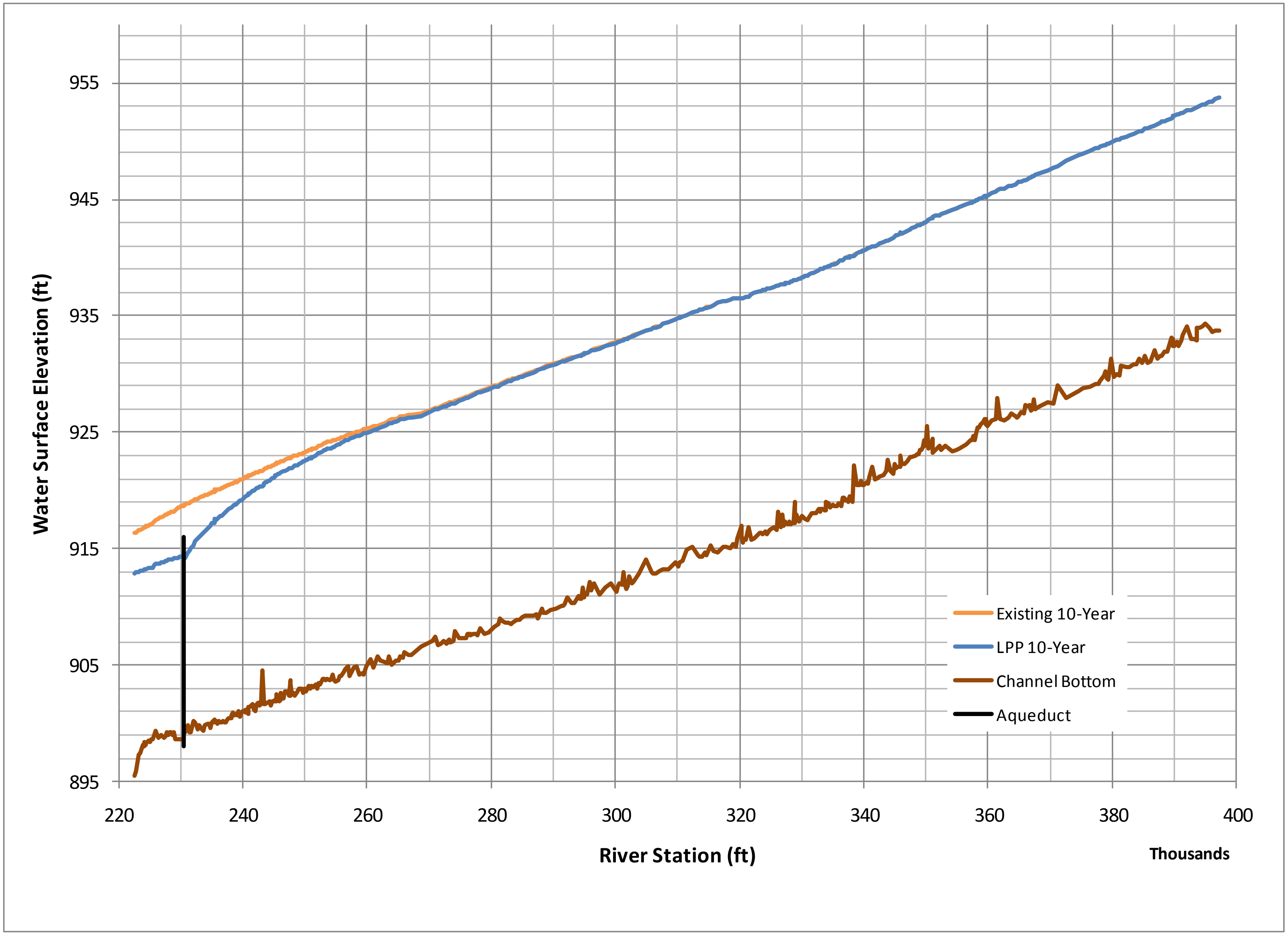


Figure F-D34 Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Local 10-Year Event

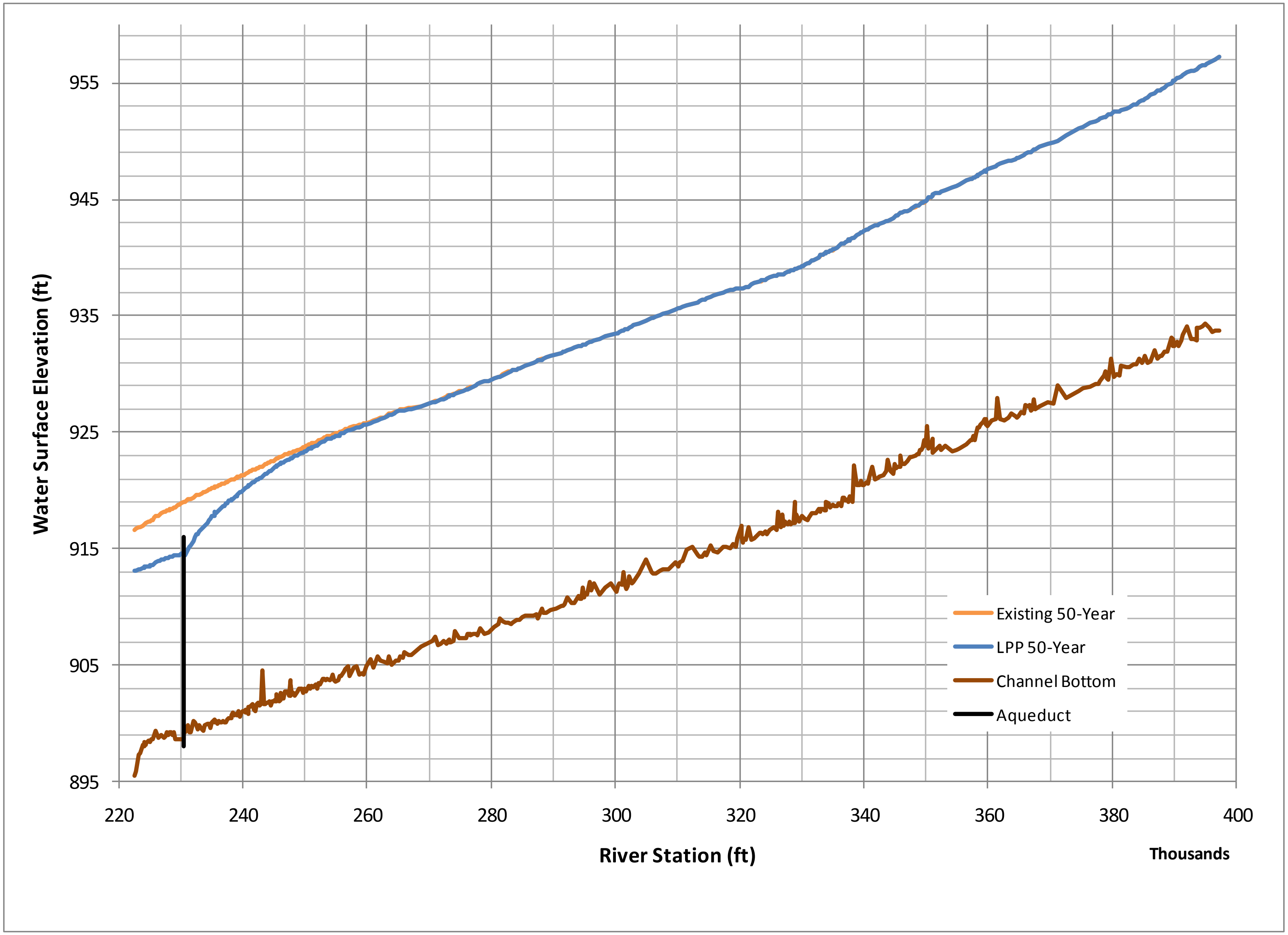


Figure F-D35 **Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Local 50-Year Event**

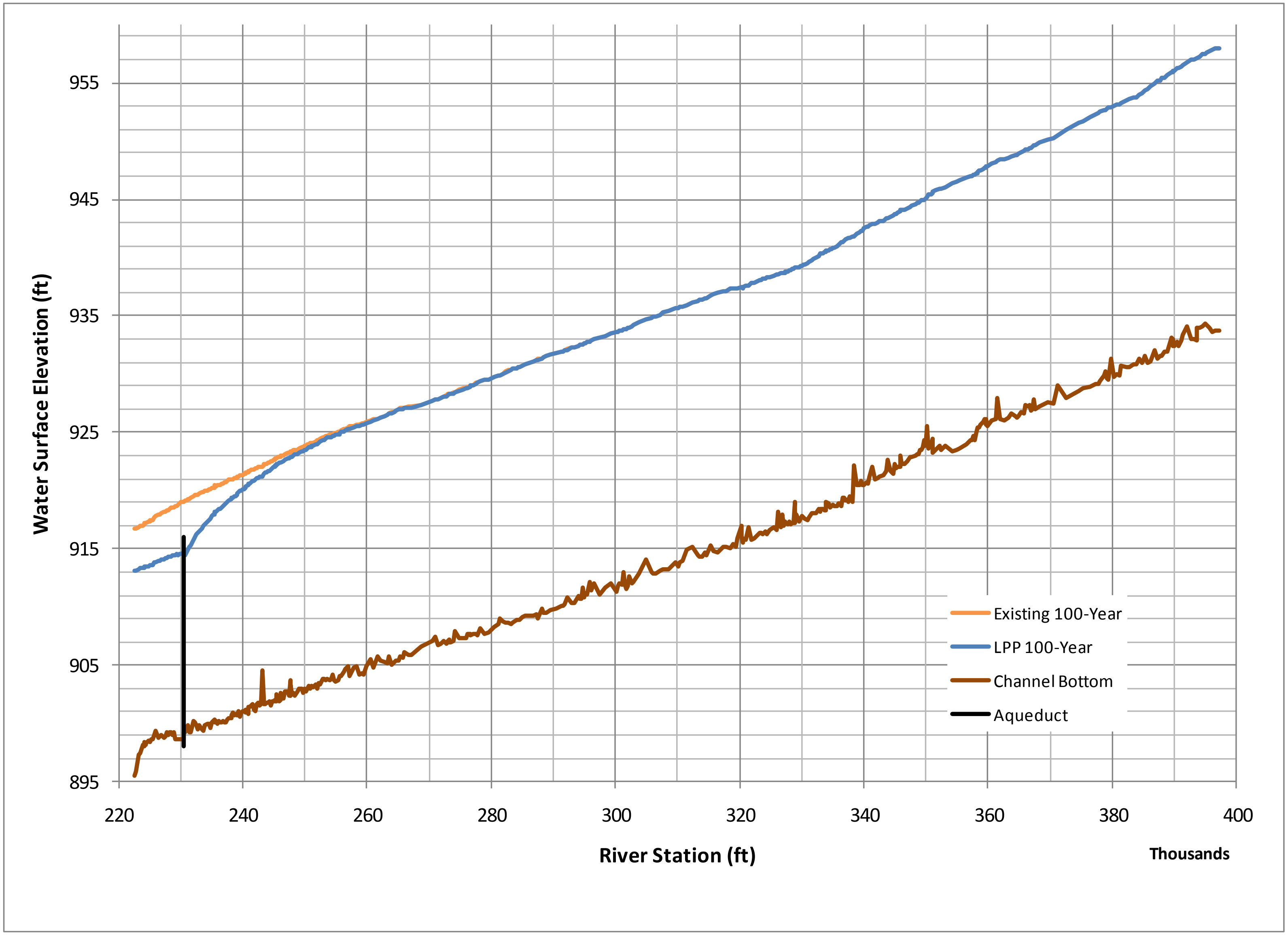


Figure F-D36 Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Local 100-Year Event

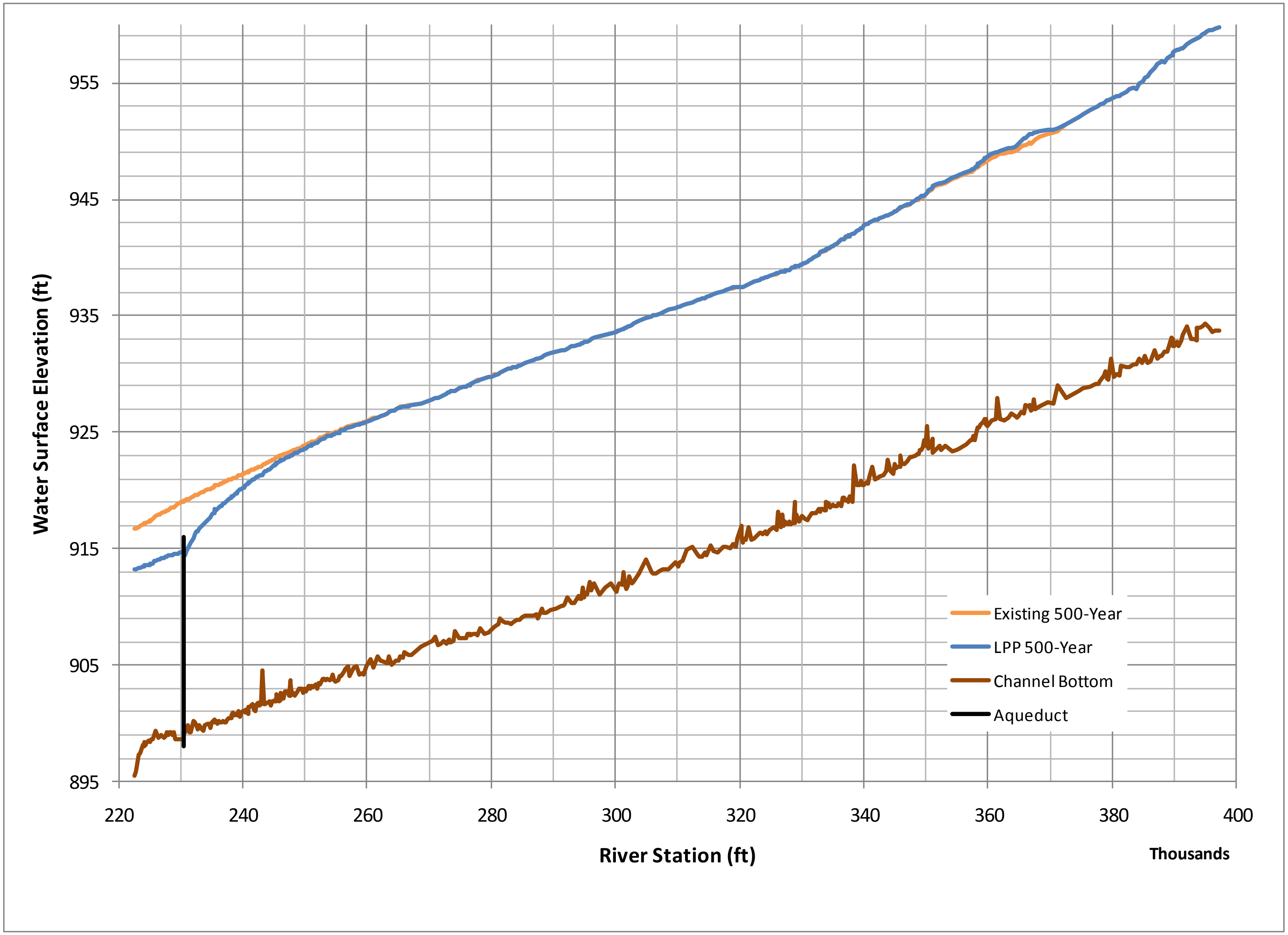


Figure F-D37 **Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Local 500-Year Event**

F-D2.6 DIVERSION CHANNEL TRANSITION AND AQUEDUCT AT MAPLE RIVER

A description of the LPP Diversion Channel transition and aqueduct structure at the Maple River can be found in the main section of Appendix F. Appendix F, Exhibit C discusses the variety of structure types that were investigated for this tributary crossing during the Phase 2 design. This portion of Exhibit D presents a comparison of water surface elevations at the Maple River structure, design calculations of the aqueduct crossing, and detailed HEC-RAS modeling outputs in the Diversion Channel beneath the aqueduct crossing.

F-D2.6.1 Comparison of LPP Diversion Channel and Maple River Hydrology

Maximum flows, water surface elevations, and velocities for the LPP Diversion Channel downstream of the Maple River crossing for the 10-, 50-, 100-, and 500-year events are presented in Table F-D23. The location shown in Table F-D23 is where flows diverted from the Maple River enter the LPP Diversion Channel via the spillway weir and the Diversion Channel has returned to its typical cross section. Water surface elevations in the LPP Diversion Channel just downstream of the Maple River structure, in the Maple River for local tributary flood events, and in the Maple River for coincidental flood events, plotted against the cross section data, are presented in Figure F-D38. Tabular results of existing conditions local and coincidental flood events in the tributaries are presented in Appendix F, Exhibit A.

Table F-D23 LPP Diversion Channel Downstream of the Maple River Crossing

	Phase 3*			Phase 4**		
Event (yr)	**Flow (cfs)**	**WSEL (ft)**	**Average Velocity (ft/s)**	**Flow (cfs)*****	**WSEL (ft)**	**Average Velocity (ft/s)*****
10	9,260	884.82	1.93	9,001	886.1	2.26
50	24,870	891.39	2.88	23,775	892.8	3.21
100	31,823	893.60	3.16	28,991	894.3	3.51
500	42,992	896.66	3.54	30,804	894.7	3.59

* Phase 3 values from Station 71959 in the Phase 3 HEC-RAS steady flow model

** Phase 4 values from Station 70018 in the Phase 4 HEC-RAS unsteady flow model

*** Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

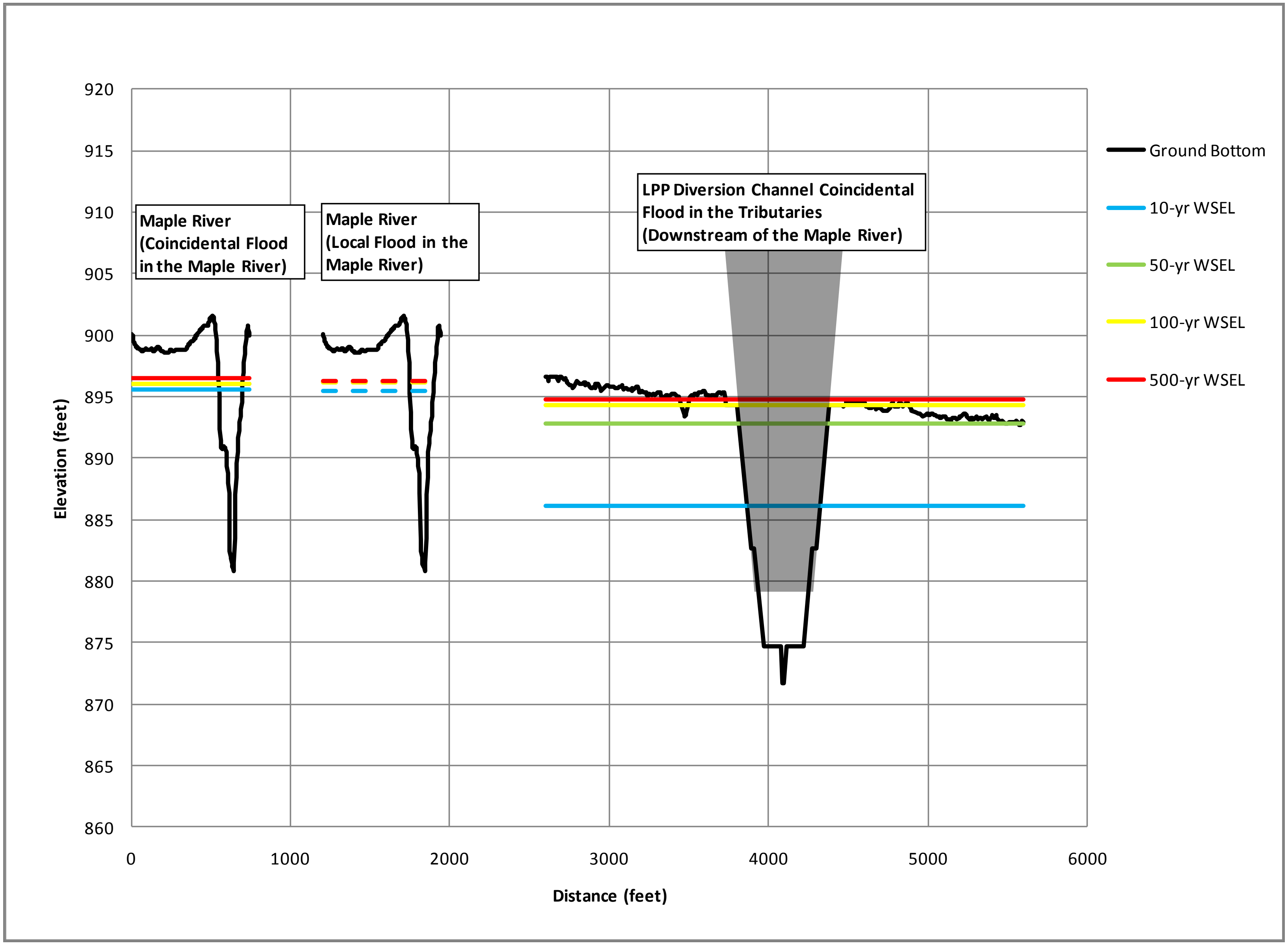


Figure F-D38 LPP Diversion Channel Downstream of the Maple River Structure and in the Maple River

F-D2.6.2 Design of Maple River Open Aqueduct

The Maple River open aqueduct was sized to pass the 2-year equivalent flow into the protected area. For Phase 3 hydrology, this is equal to the 2-year flow for the Maple River (970 cfs) plus the two year flow for the Lower Rush and Rush Rivers (302 cfs and 415 cfs, respectively), or a total of 1,687 cfs. The Lower Rush River and Rush River 2-year flows were added because these rivers will be completely diverted into the LPP Diversion Channel; this additional flow will aim to maintain bankfull flows in the downstream reach of the Sheyenne River. The aqueduct was sized to maintain the same cross sectional area and flow velocity as the corresponding natural cross section for events smaller than the equivalent 2-year event in the tributary, as shown in Figure F-D39. The invert of the aqueduct was raised approximately 1.5 feet above the existing invert, from an elevation of 879.6 to 881.06 feet, to allow for more flow area and lower velocities in the LPP Diversion Channel under the crossing. In comparison to the natural variation in the Maple River's channel bottom, this change is not significant since gradual transitions will be used on both the entrance and exit to the aqueduct. Figure F-D40 shows the Maple River thalweg (deepest continuous line along the channel bottom) under existing conditions and with the Maple River aqueduct raised 1.5 feet. Table F-D24 presents the calculations used in Phase 3 to size the Maple River open aqueduct crossing over the LPP Diversion Channel. The width of the flow area for the Maple River open aqueduct was found to be 50.3 feet. This was rounded to 50 feet for structural design. The top of structure was set at 902 feet, one foot above the 500-year peak on the Red River of the North event through the LPP diversion channel.

Table F-D24 Calculations Used to Size the Maple River Open Aqueduct Crossing over the LPP Diversion Channel (from Phase 3)

2-Year Local Event Variables	Existing Natural Cross Section	Open Aqueduct Crossing
Water Surface Elevation (ft)	893.32	893.69
Channel Bottom Invert (ft)	880.06	881.06
Depth of Water (ft)	13.26	12.63
Flow (cfs)	1687	1687
Flow Area (ft^2)	635.21	635.21
Average Velocity (ft/s)	2.66	2.66
Acceleration of Gravity (ft/s^2)	32.2	32.2
Average Channel Width (ft)	47.90	50.3
Change in Bottom Invert from Natural Conditions (ft)	0	1
Energy (ft)	13.370	13.738

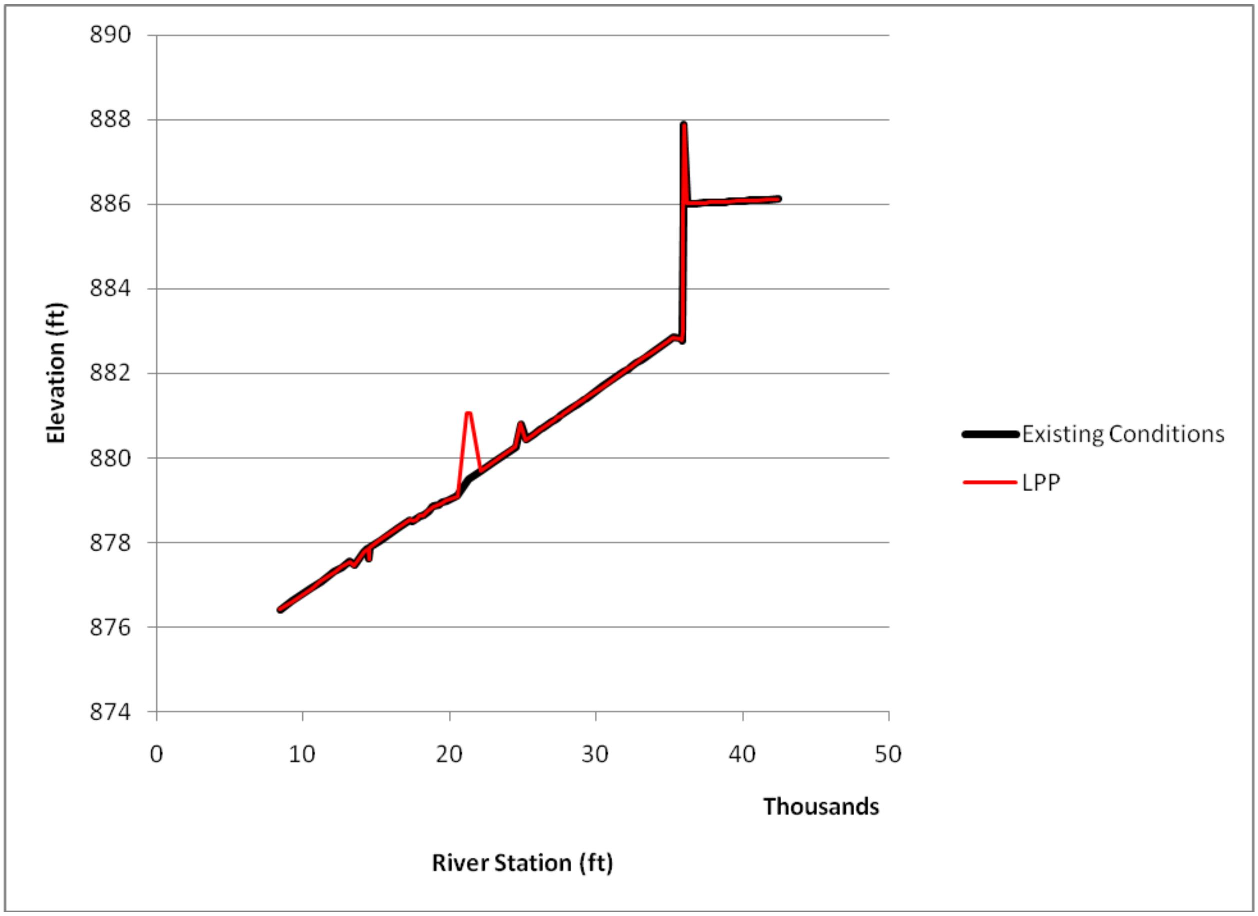


Figure F-D39 Comparison of the Maple River Thalweg under Existing and Proposed Conditions

Depending on the actual head in the Maple River for a certain event and also on how much flow the spillway weir diverts into the LPP Diversion Channel, more than the 2-year local event but less than the 10-year local event could enter the protected area. For events larger than the 2-year equivalent local event when water will be diverted over the spillway weir, the water surface elevation in the Maple River open aqueduct will be less than under existing conditions (Appendix F, Exhibit A).

Table F-D25 Elevations, Velocities and Flows in the Maple River Aqueduct for Coincidental Events

	Phase 3					**Phase 4**				
Event Return Period (yr)*	**Flow in River Upstream of Aqueduct (cfs)**	**WSEL Upstream of Aqueduct(ft)**	**Flow in River into Protected Area (cfs)**	**WSEL of River in Aqueduct (ft)**	**Average Velocity in Aqueduct (ft/s)**	**Flow in River Upstream of Aqueduct (cfs)***	**WSEL Upstream of Aqueduct(ft)**	**Flow in River into Protected Area (cfs)***	**WSEL of River in Aqueduct (ft)**	**Average Velocity in Aqueduct (ft/s)***
10	2,650	895.36	1,796	894.32	2.71	5,478	896.0	2,732	894.9	4.18
50	4,400	898.83	1,992	895.32	2.79	6,994	896.6	3,007	895.6	4.35
100	4,925	899.96	2,043	895.57	2.82	7,079	896.6	2,991	895.7	4.29
500	5,115	900.34	2,416	897.41	2.96	9,119	897.3	3,581	896.4	4.90

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

Table F-D26 Elevations, Velocities and Flows in the Maple River Aqueduct for Local Events

	Phase 4				
Event Return Period (yr)*	**Flow in River Upstream of Aqueduct (cfs)***	**WSEL Upstream of Aqueduct (ft)**	**Flow in River into Protected Area (cfs)***	**WSEL of River in Aqueduct (ft)**	**Average Velocity in Aqueduct (ft/s)***
10	5,206	895.9	2,561	894.7	3.95
50	7,407	896.7	3,084	895.6	4.45
100	7,595	896.8	3,122	895.7	4.48
500	7,736	896.9	2,840	896.2	3.95

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

The transition from the natural channel into the open aqueduct is accomplished with 5:1 side slopes as shown in Figure F-D40. For more details about the geotechnical slope stability analysis of this reach, see Exhibit N of Appendix F.

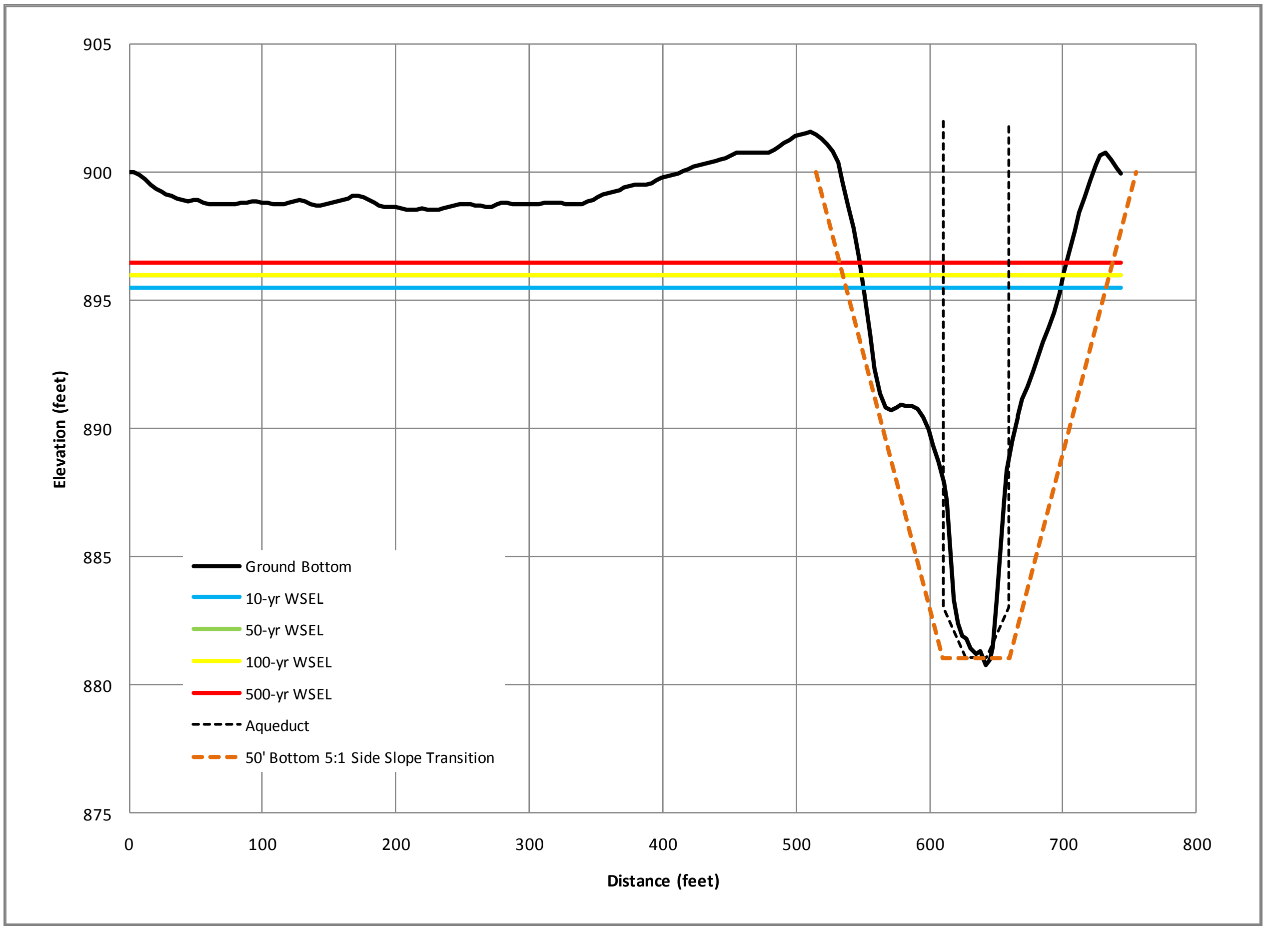


Figure F-D40 Transition from the Natural Maple River Cross Section into the Aqueduct

F-D2.6.3 <u>Design of the Low Flow Channel in the Maple River Aqueduct</u>
The Maple open aqueduct will include a smaller winter channel to ensure that low flows during winter will not freeze. According to records from 1949 to 2008 at the USGS gage at Mapleton, the mean annual flow from December to February is 10.9 cfs. Using Manning’s equation, a low flow channel with dimensions of 4 feet by 4 feet would convey this flow across the open aqueduct. The lowest recorded flow in the winter months is 0 cfs. In the next phase of design, the design of the low flow channel could be further refined to look at a variety of low flow and freezing conditions that the structure may encounter and to optimize dimensions to encourage fish passage under a variety of flows.

F-D2.6.4 <u>Design of LPP Diversion Channel Transition at the Maple River Structure</u>
No mixing of Maple River and Diversion Channel water will occur at the open aqueduct. Flow in the Maple River open aqueduct will be able to overtop the top of the wall at an

elevation of 902 feet into the LPP Diversion Channel but would necessitate an event larger than the 500-year event. In Phase 4, flow through the aqueduct was completely contained within the aqueduct.

The opening under the aqueduct was maximized to minimize the head difference in the diversion channel across the structure. The bottom width of the diversion channel is 250 feet so the width of the opening was set at 250 feet. The number of piers and the size of each pier were set based on structural stability rather than flow conditions. The design was checked in the HEC-RAS unsteady flow model to ensure that the head drop across the structure was acceptable. Pressurized flow underneath the aqueduct in the Diversion Channel will occur for events in the Red River greater than or equal to the 10-year event. The proposed dimensions and location of the Maple River structure are shown in Appendix F.

The total flow in the LPP Diversion Channel beneath the aqueduct, headwater elevation, tailwater elevation, head difference, and average velocities for the 10-, 50-, 100-, and 500-year events are presented in Tables F-D27 and F-D28. These variables were obtained from the HEC-RAS models with the exception of average velocity beneath the aqueduct. HEC-RAS does not provide this variable so it was calculated by dividing the flow by the flow area.

Table F-D27 Coincidental Event HEC-RAS Outputs for the LPP Diversion Channel at the Maple River Aqueduct

	Phase 3					Phase 4				
Event (yr)	Flow in Diversion Channel (cfs)	Headwater Elevation in Diversion Channel (ft)	Tailwater Elevation in Diversion Channel (ft)	Head Difference (ft)	Average Velocity (ft/s)	Flow in Diversion Channel (cfs)*	Headwater Elevation in Diversion Channel (ft)	Tailwater Elevation in Diversion Channel (ft)	Head Difference (ft)	Average Velocity (ft/s)*
10	8,406	885.01	884.88	0.13	2.65	6,895	886.5	886.3	0.2	4.3
50	22,462	892.53	891.48	1.05	7.07	20,078	895.1	893.0	2.1	12.5
100	28,941	895.46	893.69	1.77	8.99	25,122	897.9	894.5	3.4	15.7
500	40,293	899.22	896.73	2.49	9.54	25,293	900.8	895.0	5.8	15.8

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

Table F-D28 Local Event HEC-RAS Outputs for the LPP Diversion Channel at the Maple River Aqueduct

	Phase 4				
Event (yr)	Flow in Diversion Channel (cfs)*	Headwater Elevation in Diversion Channel (ft)	Tailwater Elevation in Diversion Channel (ft)	Head Difference (ft)	Average Velocity (ft/s)*
10	3,623	883.2	883.2	0.0	2.3
50	14,424	890.9	890.2	0.7	9.0
100	19,738	893.5	892.2	1.3	12.3
500	25,224	896.3	894.1	2.2	15.7

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

F-D2.7 SPILLWAY AT MAPLE RIVER

A weir spillway has been selected to divert water from the tributary into the LPP Diversion Channel. The design elevation of the Maple River spillway in Phase 4 is fixed at 893.63 ft, which is the water surface elevation on the Maple River associated with the 2-year equivalent local flood event (1,687 cfs; sum of 2-year events on the Maple River, Lower Rush River, and Rush River). The spillway is not submerged under normal conditions (the tailwater is less than two-thirds the critical depth over the weir crest elevation). However, for events greater than or equal to the 100-year event, the weir becomes submerged. A weir coefficient of 3.1 was used, but in the next stage of design a lower coefficient (2.6) would be recommended.

The width of the spillway is the only variable that was altered. Starting at the design width of 150 feet from Phase 3, the width was increased until the desired flow split was met. The constraint for this particular design was to match closely the results from Phase 3, with 3,000 cfs being a rough maximum for flow through the aqueduct to the protected side. As the weir width increased, the benefits from the increase became marginal. Therefore, the weir is designed to be 300 feet wide because it is a reasonably large size which limits the flow through the aqueduct. Table F-D29 presents the results from Phase 3 using the HEC-RAS steady flow model. Tables F-D30 and F-D31 present the results from Phase 4 using the HEC-RAS unsteady flow models.

Table F-D29 Phase 3 Design Results for the 150-Ft.-Wide Maple River Spillway Using the Coincidental Flow (spillway elevation 893.32 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Flow into Protected Area (cfs)
10-year	895.36	2,650	854	1,796
50-year	898.83	4,400	2,408	1,992
100-year	899.96	4,925	2,882	2,043
500-year	900.34	5,115	2,699	2,416

Table F-D30 Phase 4 Design Results for the 300-Ft.-Wide Maple River Spillway Using the Coincidental Flow (spillway elevation 893.63 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Aqueduct WSEL (ft)	Flow into Protected Area (cfs)
10-year	896.0	5,478	2,747	894.9	2,732
50-year	896.6	6,994	3,988	895.6	3,007
100-year	896.6	7,079	4,088	895.7	2,991
500-year	897.3	9,119	5,538	896.4	3,581

Table F-D31 Phase 4 Design Results for the 300-Ft.-Wide Maple River Spillway Using the Peak Flow (spillway elevation 893.63 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Aqueduct WSEL (ft)	Flow into Protected Area (cfs)
10-year	895.9	5,206	2,645	894.7	2,561
50-year	896.7	7,407	4,323	895.6	3,084
100-year	896.8	7,595	4,473	895.7	3,122
500-year	896.9	7,736	4,896	896.2	2,840

Water surface elevations were also checked along the entire reach of the Maple River to ensure that the aqueduct structure was not causing significant upstream flooding. Figures F-D43 through F-D50 compare existing conditions and LPP conditions along the entire reach of the Maple River. These figures show there are no significant adverse impacts upstream or downstream of the aqueduct structure. There is a small difference in the existing conditions model compared to the LPP model between river stations 27,000 and 40,000. This difference is due to minor geometry differences in the two models shown in Figures F-D41 and F-D42.

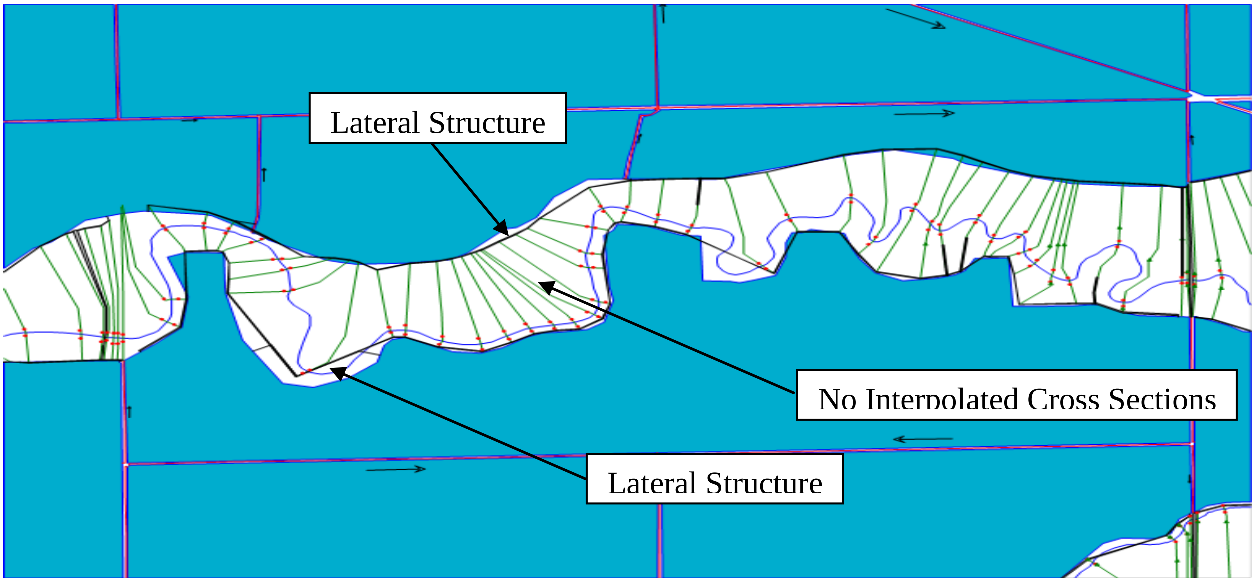


Figure F-D41 Geometry Near the Maple River Structure in the Existing Conditions Model

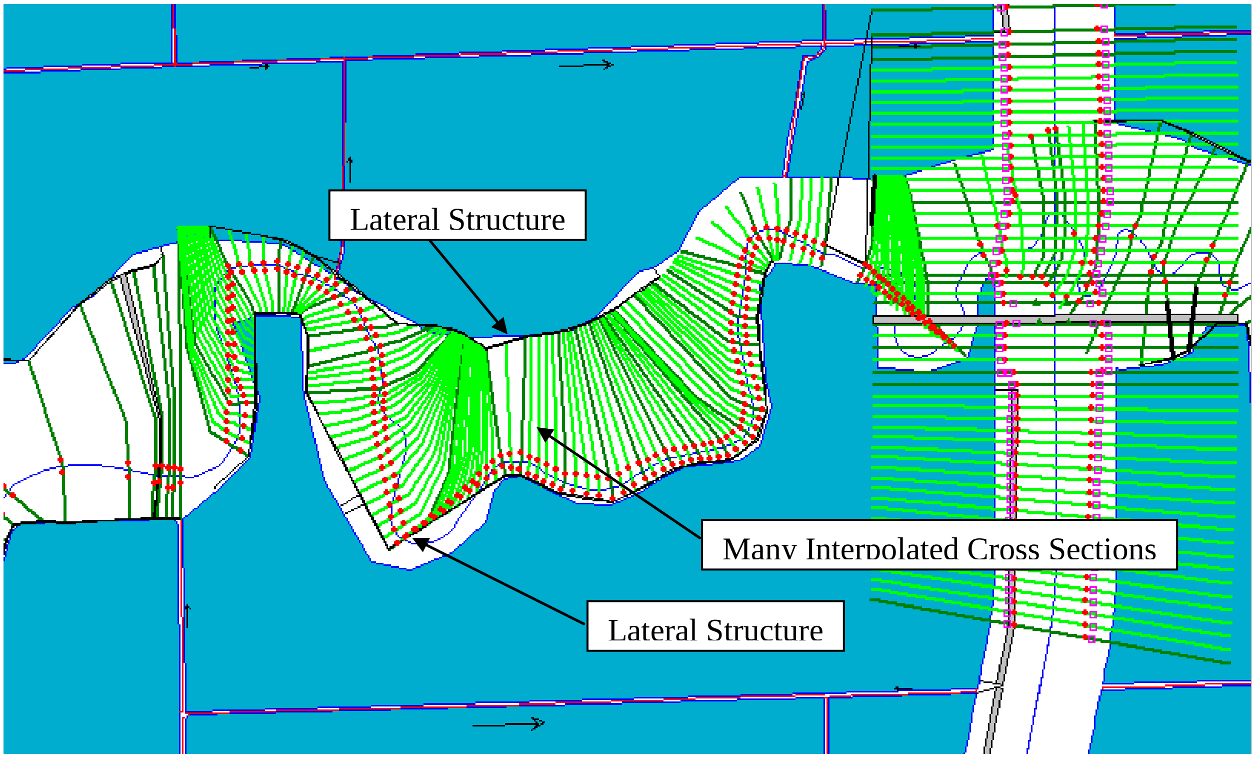


Figure F-D42 Geometry Near the Maple River Structure in the LPP Model

In Figure F-D41, the geometry shows existing lateral structures along the length of the Maple River. These structures allow high flows (greater than or equal to the 50-year event) to spill over into the adjacent storage areas (floodplain). In the LPP model, the lateral structures were initially removed to resolve stability issues. However, removing these structures prevented flow from leaving the main channel. Because of this, flows through the channel in the LPP model were higher than they ought to be, increasing the water levels. Lateral structures were added to the LPP model, but due to the addition of interpolated cross sections, the lateral structures are not exactly the same as in the existing conditions model. This is the reason for the small difference in WSEL in this area.

It is clear that the structure does not raise water levels for the smaller 10-year event (Figure F-D43). This is because in the existing conditions, the smaller event does not have water elevations high enough to spill into the floodplain. In this case, the lateral structures play no part. For higher events, the difference in this particular area of the reach is shown. However, there is no difference in WSEL upstream of this area when comparing existing conditions to the LPP conditions. This reinforces the fact that the small geometry difference in this particular area is the cause of the WSEL difference, not the Maple River structure itself.

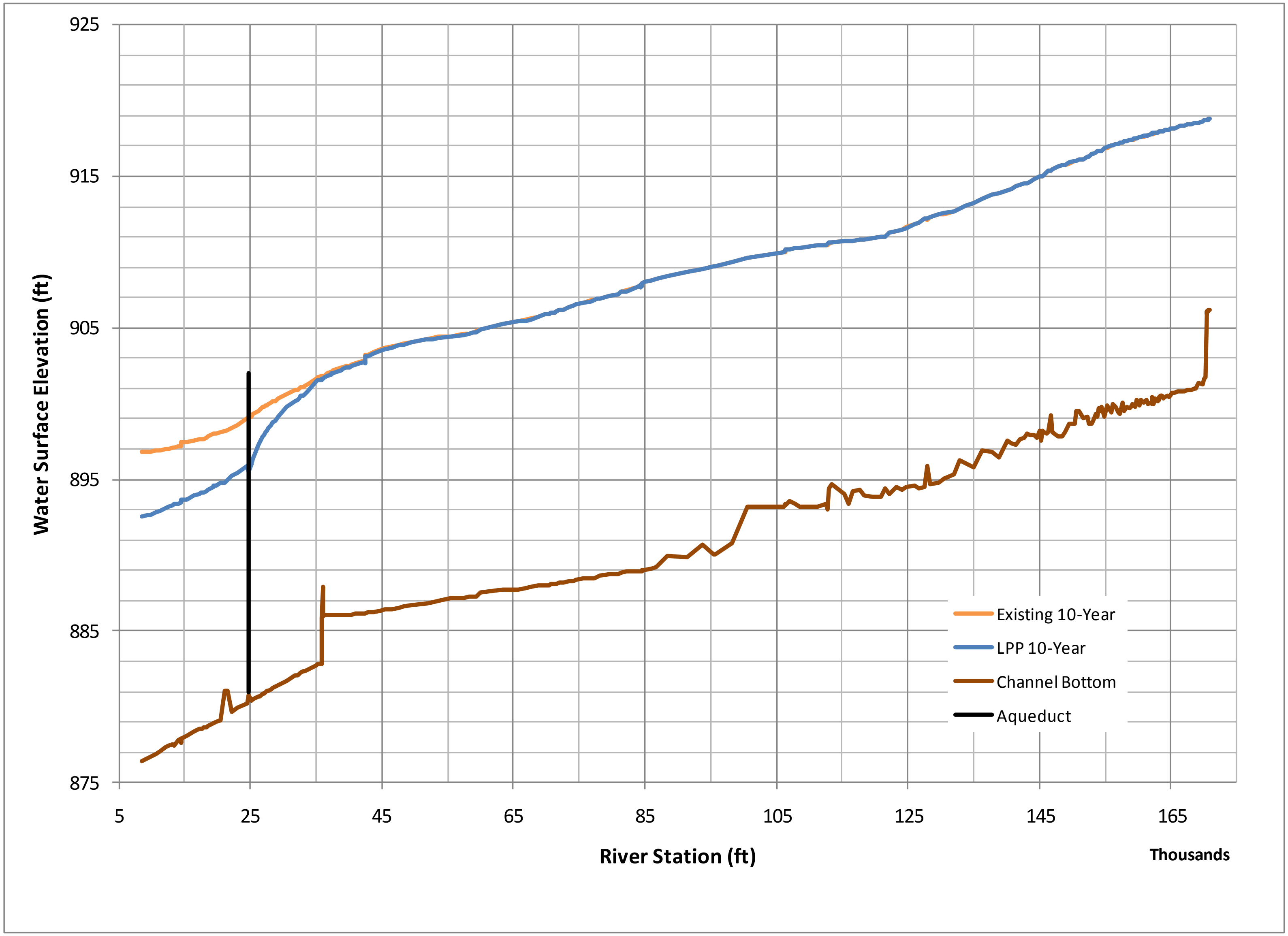


Figure F-D43 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Coincidental 10-Year Event

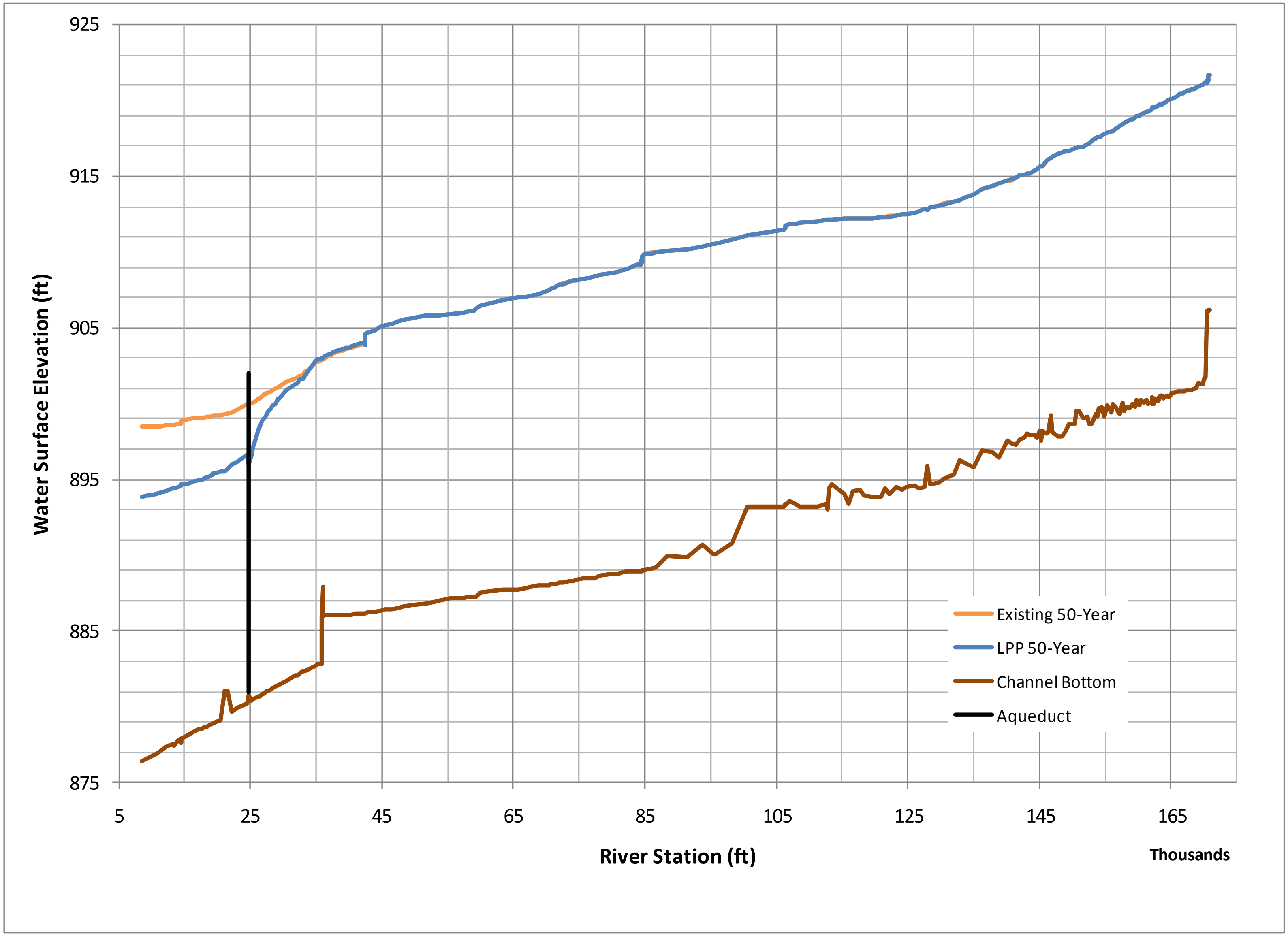


Figure F-D44 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Coincidental 50-Year Event

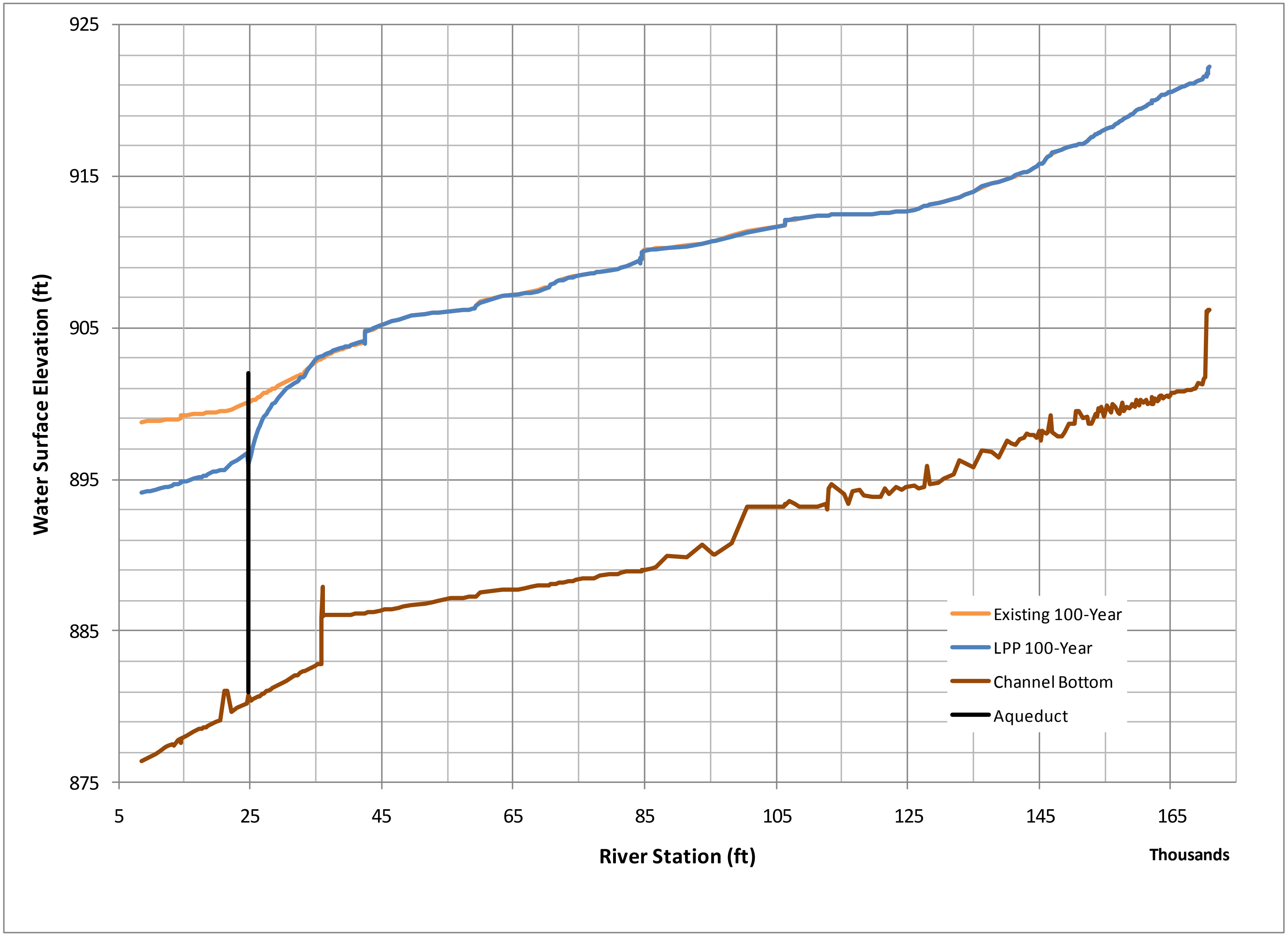


Figure F-D45 **Comparing Existing Conditions to LPP Conditions Along the Maple River for the Coincidental 100-Year Event**

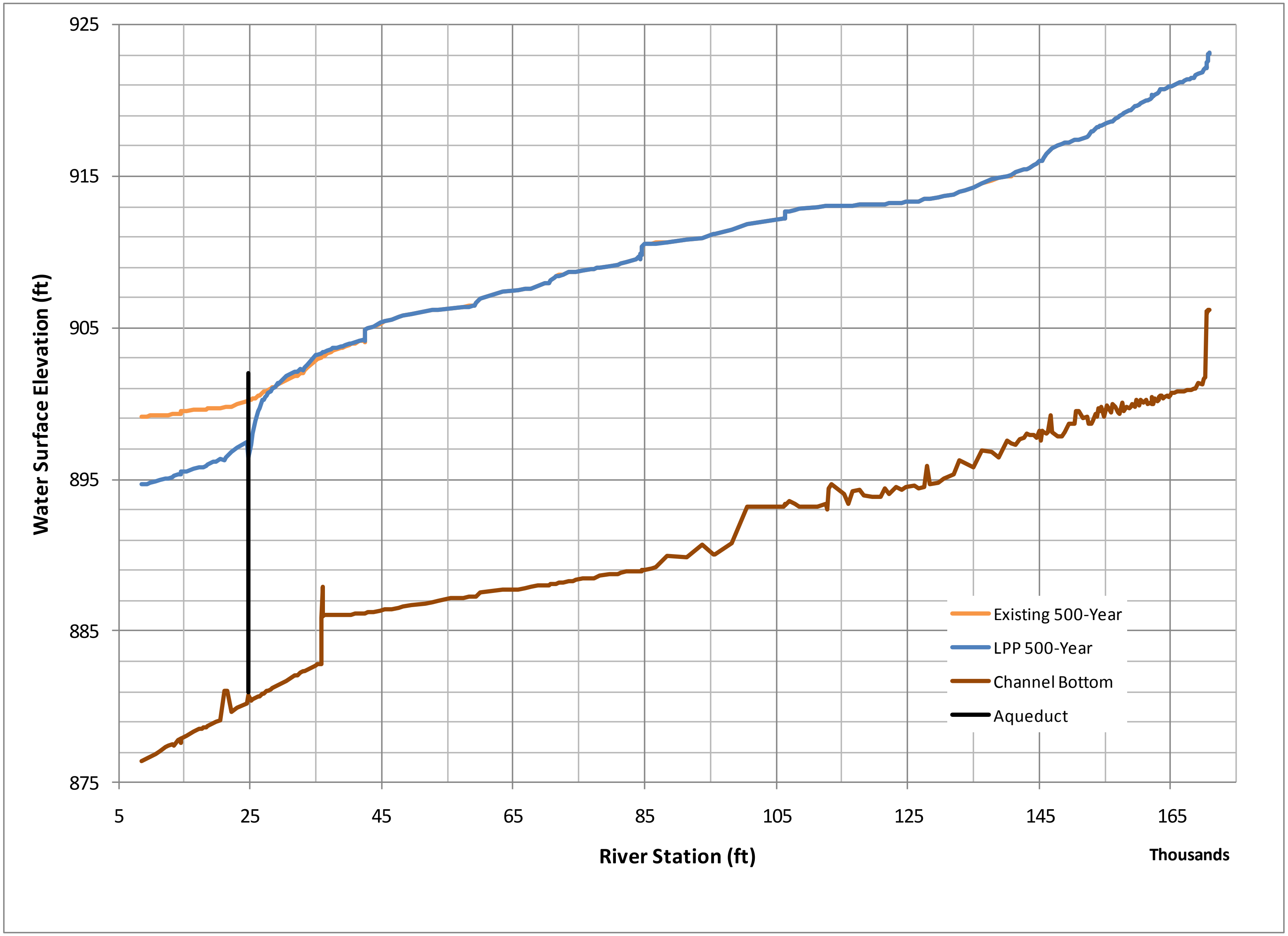


Figure F-D46 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Coincidental 500-Year Event

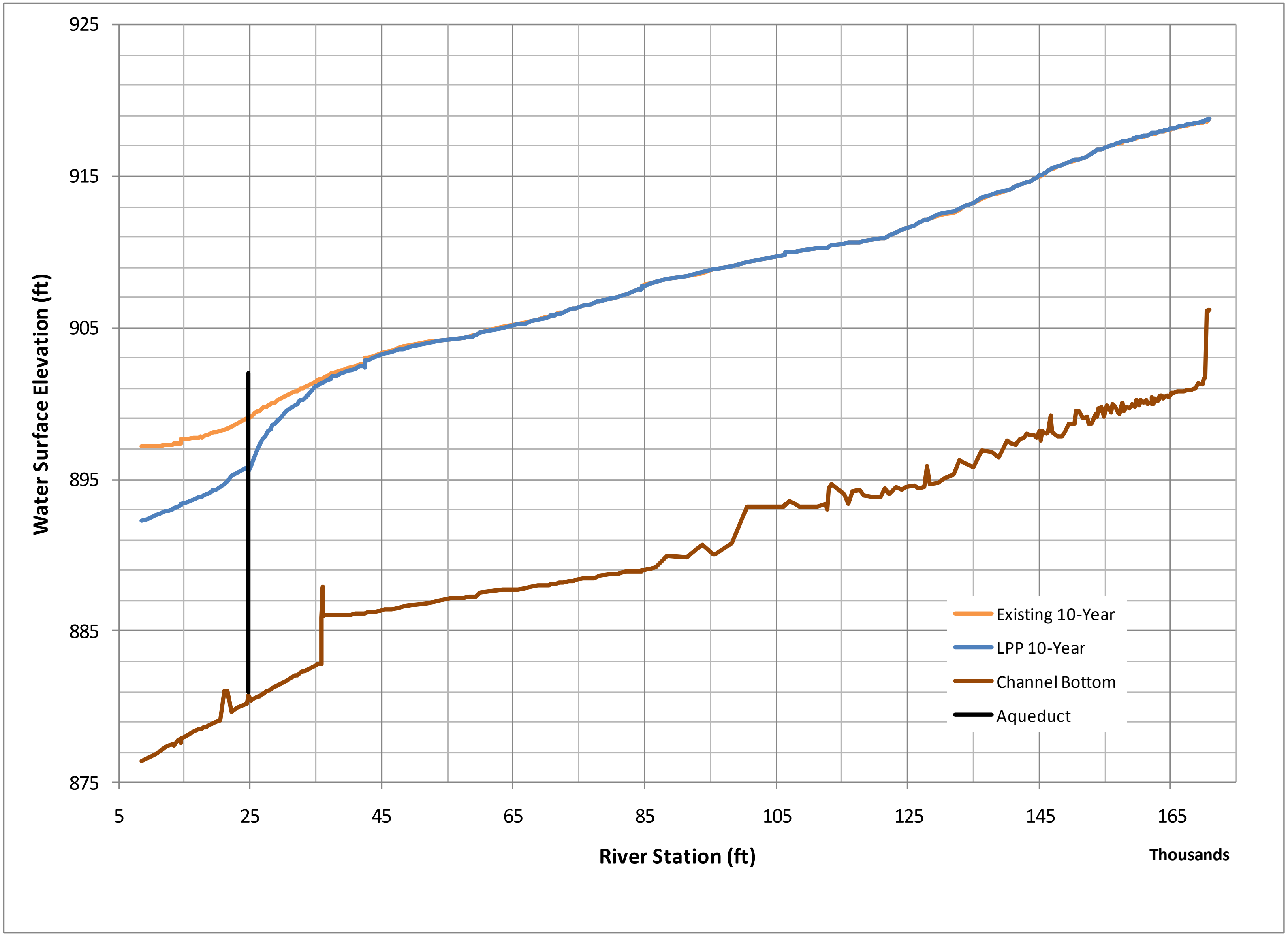


Figure F-D47 **Comparing Existing Conditions to LPP Conditions Along the Maple River for the Local 10-Year Event**

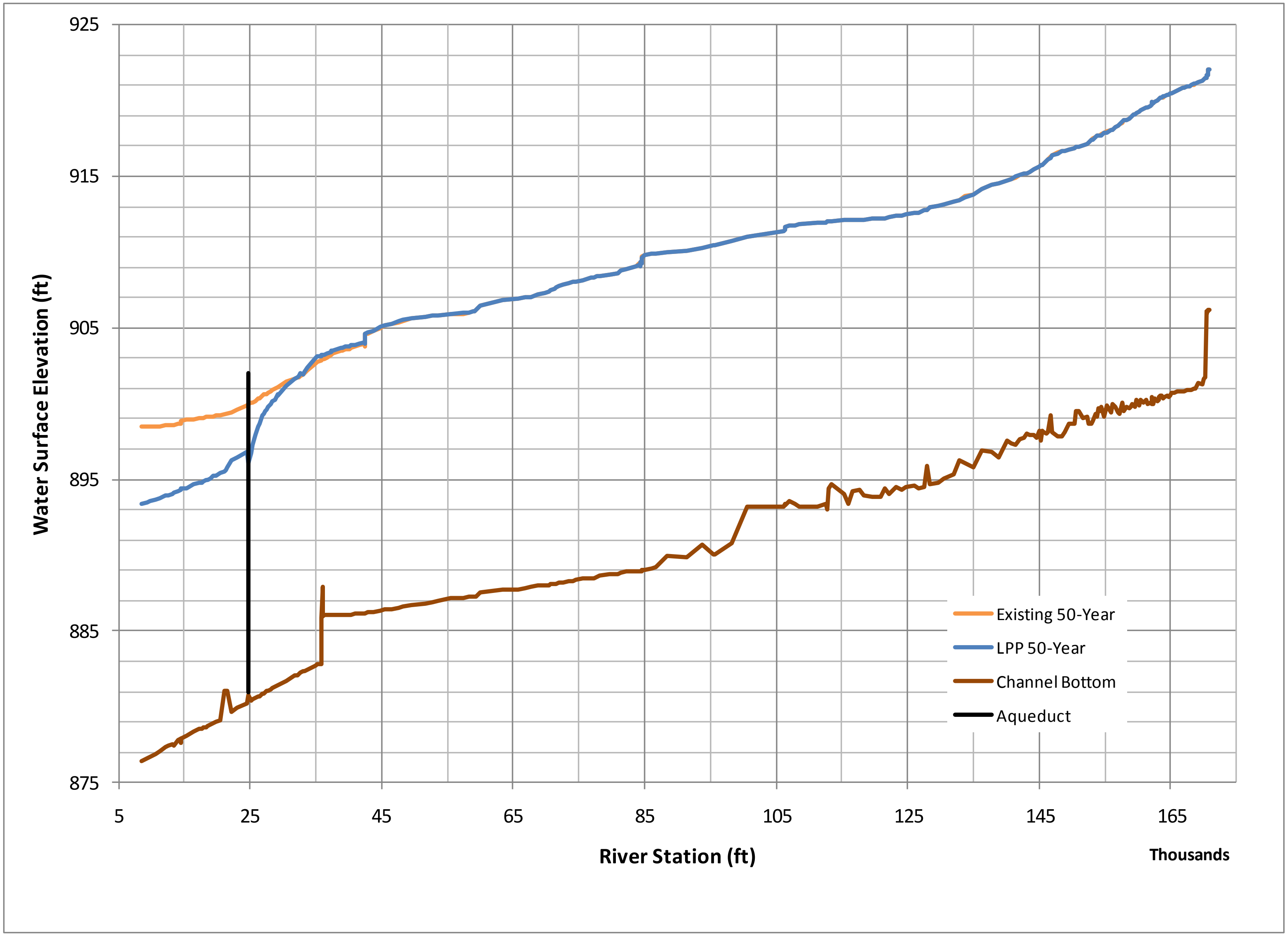


Figure F-D48 **Comparing Existing Conditions to LPP Conditions Along the Maple River for the Local 50-Year Event**

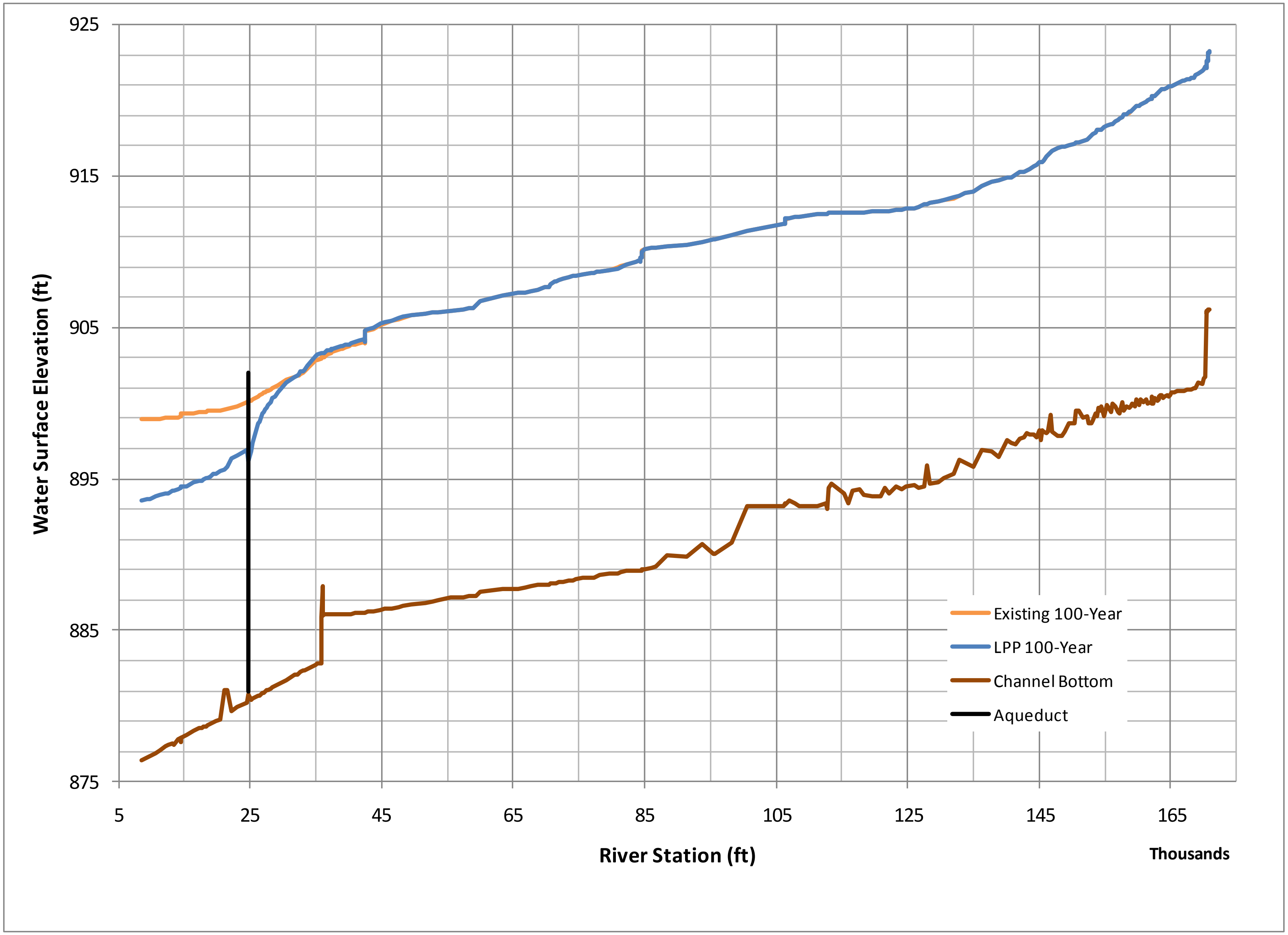


Figure F-D49 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Local 100-Year Event

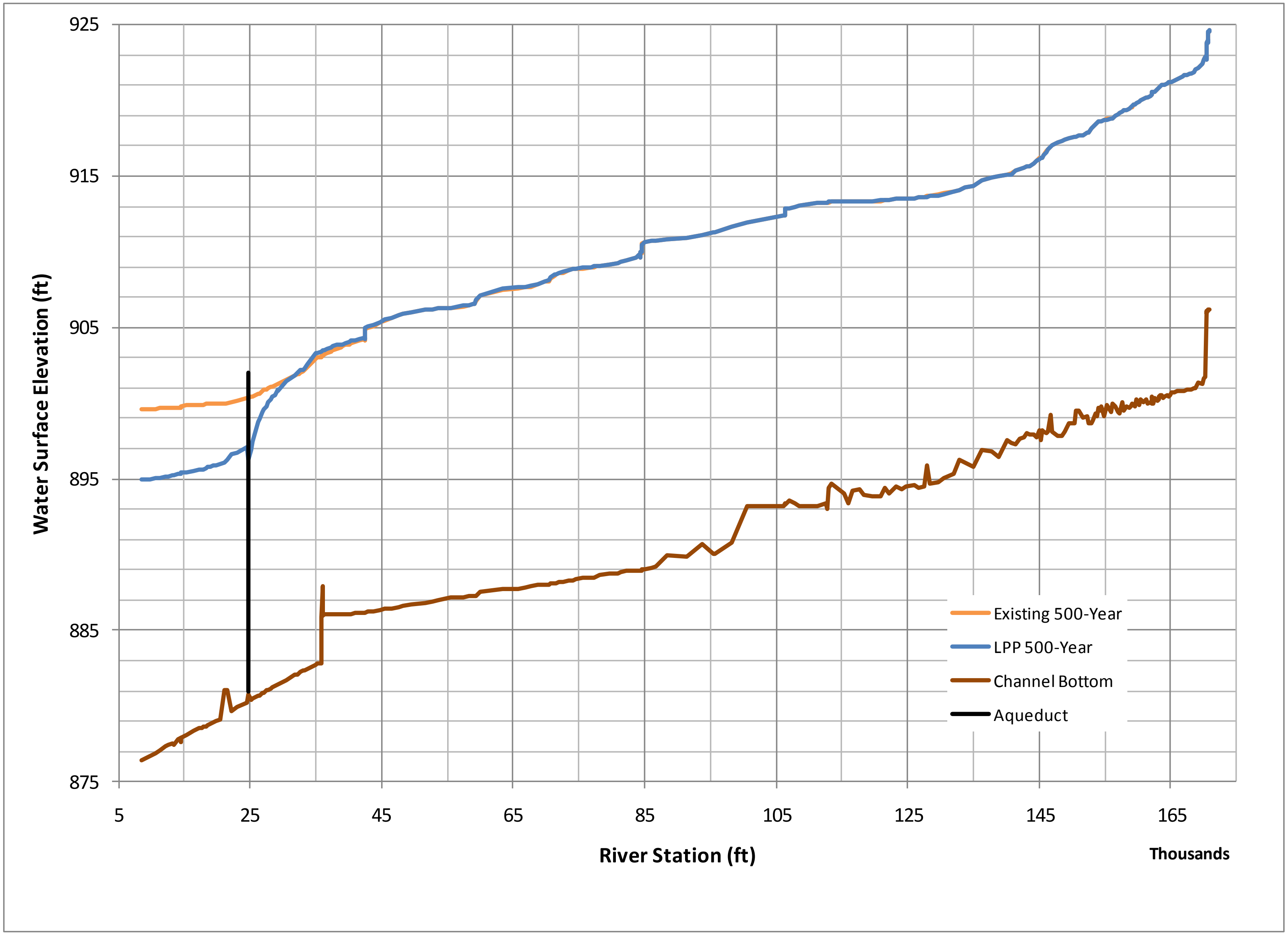


Figure F-D50 **Comparing Existing Conditions to LPP Conditions Along the Maple River for the Local 500-Year Event**

F-D2.8 DROP STRUCTURE AT LOWER RUSH RIVER

A description of the drop structure at the Lower Rush River can be found in the main section of Appendix F. This portion of Exhibit D presents a comparison of water surface elevations at the Lower Rush River structure and design calculations of the drop structure.

F-D2.8.1 Comparison of LPP Diversion Channel and Lower Rush River Hydrology

Flows, water surface elevations, flow areas, and average velocities for the Diversion Channel at the Lower Rush River drop structure for the 10-, 50-, 100-, and 500-year events are presented in Table F-D32 and F-D33. The locations shown in the tables are Station 57083 for Phase 3 and 59662 for Phase 4 which is where flows diverted from the Lower Rush River enter the Diversion Channel. Water surface elevations in the Diversion Channel at the Lower Rush River structure, in the Lower Rush River for local tributary flood events, and in the Lower Rush River for coincidental flood events, plotted against the cross section data, are presented in Figure F-D51. Tabular results of existing conditions local and coincidental flood events in the tributaries are presented in Appendix F, Exhibit A.

Table F-D32 Phase 3 Hydrology in the Diversion Channel Downstream of the Lower Rush River Structure (Station 57083)

Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
5	3,543	879.38	3,340	1.06
10	9,423	883.54	5,300	1.78
20	15,782	886.13	6,642	2.38
50	25,155	889.02	8,254	3.05
100	32,178	890.89	9,357	3.44
200	37,906	892.29	10,296	3.71
500	43,372	893.67	11,276	3.91

Table F-D33 Phase 4 Hydrology in the Diversion Channel Downstream of the Lower Rush River Structure (Station 59662)

Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
10	7,561	883.93	3,672	2.06
50	20,703	890.28	6,693	3.09
100	24,941	891.63	7,625	3.27
500	28,350	892.50	8,302	3.41

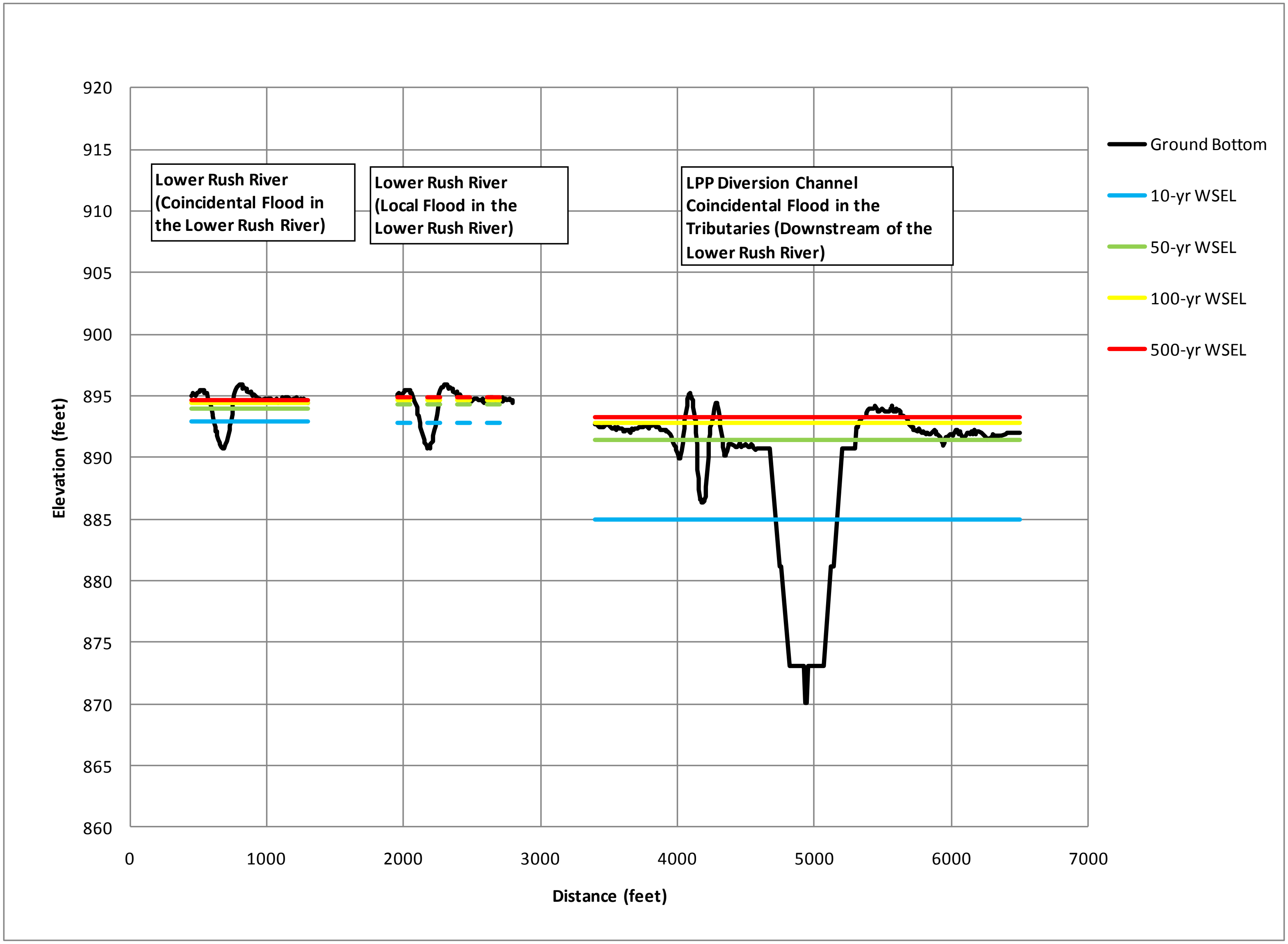


Figure F-D51 Phase 4 Hydrology in the Diversion Channel at the Lower Rush River Structure and in the Lower Rush River

F-D2.8.2 Design of Lower Rush River Drop Structure

The stepped concrete drop spillway was designed to pass the 500-year local event. For average and low flows, the majority of the flow passes into the Diversion Channel via the fish passage. The fish passage channel is discussed in Appendix F, Exhibit G and the low flow channel in the Diversion Channel extending from the Lower Rush River to the Red River of the North is discussed in Appendix F, Exhibit K. The width of the Lower Rush River stepped drop structure is 60 feet. The drop structure is designed to handle the 500-yr local event with a 50 foot stilling basin. The stilling basin length is dependent on the length of the hydraulic jump. Dimensions of the Lower Rush River stepped spillway are outlined in Table F-D34. The calculations used to determine the 60 foot width of the stepped drop structure are presented in Table F-D35.

Table F-D34 Lower Rush River Stepped Spillway Parameters

Tributary bed invert elevation	885.4	ft
Diversion channel bed elevation	872.17	ft
Diversion channel 500-yr flood elevation (local on the Tributaries)	891.97	ft
Diversion channel 10-yr flood elevation	883.87	ft
10-yr flood elevation on the Lower Rush	891.55	ft
WSEL difference between the 10-yr local event on the Lower Rush and the Diversion Channel bed invert	19.4	ft
Crest of steps	886.76	ft
Number of steps	16	
Height of each step	0.9	ft
Length of each step	1.5	ft
Total height of steps	14.6	ft
Total length of steps	24.0	ft
Spillway width	60	ft

Table F-D35 Flow Regime Over the Steps and Stilling Basin Parameters of the Lower Rush River Stepped Spillway with a 60-Foot Width

Local Event (yr)	Flow in Tributary (cfs)	Critical Depth (ft)	Flow Regime	Water Depth Upstream of Hydraulic Jump (ft)*	Water Depth Down-stream of Hydraulic Jump (ft)*	Velocity Down-stream of Hydraulic Jump (ft/s)	Length of Hydraulic Jump (ft)
10	528	1.34	skim	0.5	2.85	3.1	17
50	1601	2.81	skim	0.92	6.49	4.1	39
100	1,887	3.13	skim	1.03	7.24	4.3	43
500	2,178	3.45	skim	1.13	7.94	4.6	*48*

* Tailwater effects were not incorporated into the sizing of the stilling basin.

F-D2.9 DROP STRUCTURE AT RUSH RIVER

A description of the drop structure at the Rush River can be found in Appendix F. This portion of Exhibit D presents a comparison of water surface elevations at the Rush River structure and design calculations of the drop structure.

F-D2.9.1 Comparison of Diversion Channel and Rush River Hydrology

Flows, water surface elevations, flow areas, and average velocities for the Diversion Channel at the Rush River drop structure for the 5-, 10-, 20-, 50-, 100-, 200-, and 500-year events with the Phase 3 hydrology are presented in Table F-D36. Table F-D37 provides the flows, water surface elevations, flow areas, and average velocities for the Diversion Channel at the Rush River drop structure for the 10-, 50-, 100-, and 500- year events with the Phase 4 hydrology. The location shown in the tables is Station 47124 for Phase 3 and Station 42719 for Phase 4 which is where flows diverted from the Rush River enter the Diversion Channel. Water surface elevations in the Diversion Channel at the Rush River structure, in the Rush River for local tributary flood events, and in the Rush River for coincidental flood events, plotted against the cross section data, are presented in Figure F-D52. Tabular results of existing conditions local and coincidental flood events in the tributaries are presented in Appendix F, Exhibit A.

Table F-D36 Phase 3 Hydrology in the Diversion Channel Downstream of the Rush River Structure (Station 47124)

Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
5	3,789	879.16	4,146	0.91
10	9,749	883.09	6,261	1.60
20	16,202	885.45	7,658	2.20
50	25,725	888.06	9,211	2.93
100	32,888	889.79	10,237	3.38
200	38,626	891.12	11,030	3.70
500	44,132	892.48	11,835	3.95

Table F-D37 Phase 4 Hydrology in the Diversion Channel Downstream of the Rush River Structure (Station 47219)

Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
10	7,932	882.46	4,139	1.92
50	22,441	888.11	7,072	3.17
100	26,886	889.35	8,051	3.34
500	29,437	890.14	8,669	3.40

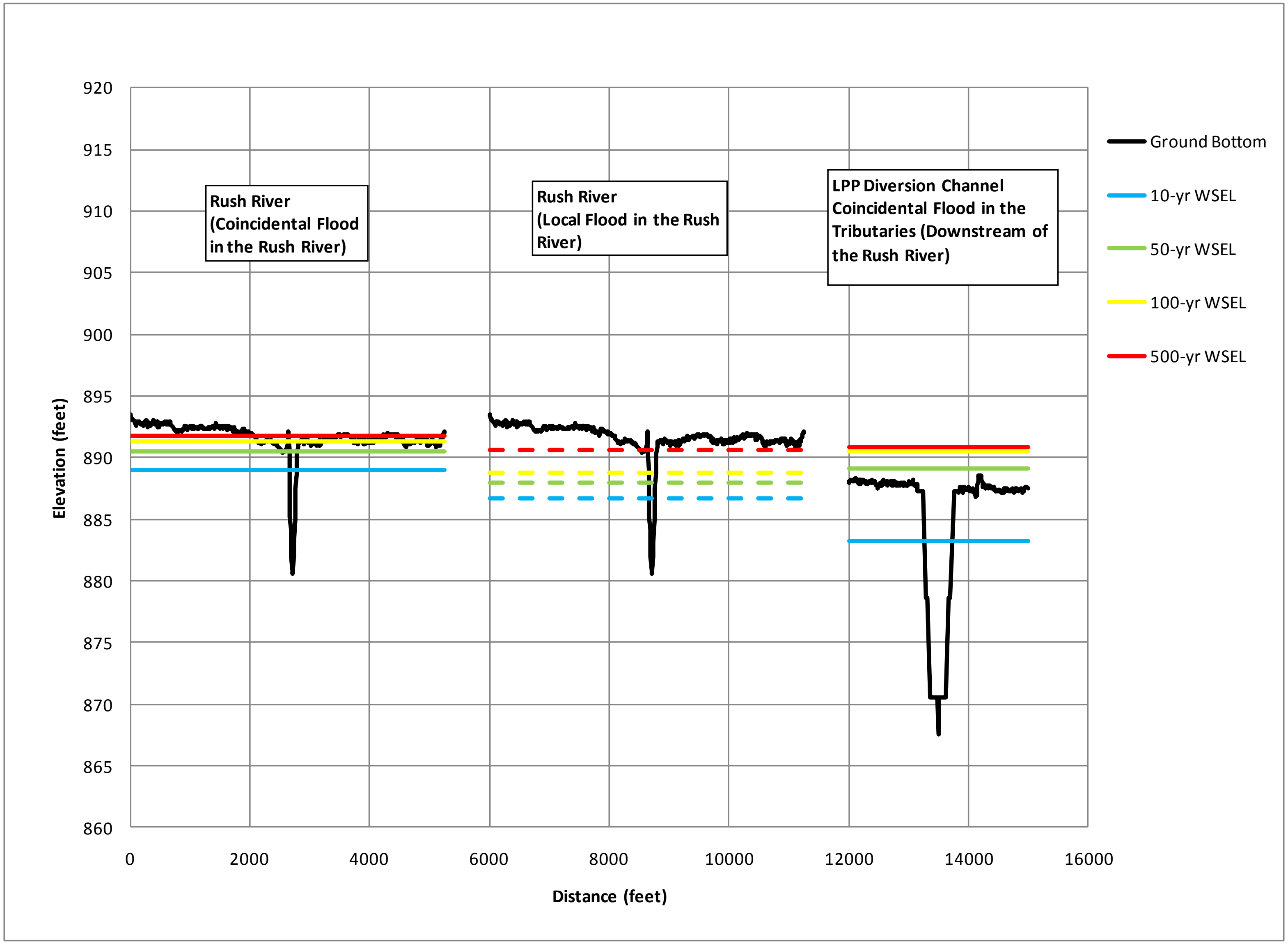


Figure F-D52 Phase 4 Hydrology in the Diversion Channel Downstream of the Rush River Structure and in the Rush River

F-D2.9.2 Design of Rush River Drop Structure

The stepped concrete drop spillway was designed to pass the 500-year local event. For average and low flows, the majority of the flow passes into the Diversion Channel via the fish passage. The fish passage channel is discussed in Appendix F, Exhibit G and the low flow channel in the Diversion Channel extending from the Rush River to the Red River of the North is discussed in Appendix F, Exhibit K. The width of the Rush River stepped drop structure is 100 feet. The drop structure is designed to handle the 500-yr local event with a 50 foot stilling basin. The stilling basin length is dependent on the length of the hydraulic jump. Dimensions of the Rush River stepped spillway are outlined in Table F-D38. The calculations used to determine the 100 foot width of the stepped drop structure are presented in Table F-D39.

Table F-D38 Rush River Stepped Spillway Parameters

Tributary bed invert elevation	879.5	ft
Diversion channel bed elevation	869.62	ft
Diversion channel 500-yr flood elevation (local on the Tributaries)	889.85	ft
Diversion channel 10-yr flood elevation	882.57	ft
10-yr flood elevation on the Rush	889.68	ft
WSEL difference between the 10-yr local event on the Rush and the Diversion Channel bed invert	20.1	ft
Crest of steps	880.9	ft
Number of steps	10	
Height of each step	1.1	ft
Length of each step	1.7	ft
Total height of steps	11.3	ft
Total length of steps	17.0	ft
Spillway width	100	ft

Table F-D39 Flow Regime over the Steps and Stilling Basin Parameters of the Rush River Stepped Spillway with a 100-Foot Width

Local Event (yr)	Flow in Tributary (cfs)	Critical Depth (ft)	Flow Regime	Water Depth Upstream of Hydraulic Jump (ft)*	Water Depth Down-stream of Hydraulic Jump (ft)*	Velocity Down-stream of Hydraulic Jump (ft/s)	Length of Hydraulic Jump (ft)
10	98	0.31	nappe	0.16	0.53	1.9	3
50	588	1.02	trans	0.39	2.18	2.7	13
100	930	1.39	skim	0.48	3.1	3	19
500	4,210	3.8	skim	1.34	8.41	5	*50*

* Tailwater effects were not incorporated into the sizing of the stilling basin.

F-D2.10 OUTLET TO RED RIVER OF THE NORTH

The Phase 3 Outlet of the Diversion Channel into the Red River of the North consisted of riprap over the downstream 300 feet of the Diversion Channel. This outlet configuration was possible because the Diversion Channel outlet invert elevation was near the bottom of the Red River. However, with the introduction of staging in Phase 4, the peak flows diverted through the Diversion Channel downstream of the Wild Rice River was reduced from 35,000 cfs to 19,000 cfs. As a result the cross sectional area of the Diversion Channel was reduced and the bottom invert was raised. As a consequence of these changes the drop into the Red River at the Outlet of the Diversion Channel increased from approximately 11 ft to 20 ft as shown in Figure F-D53.

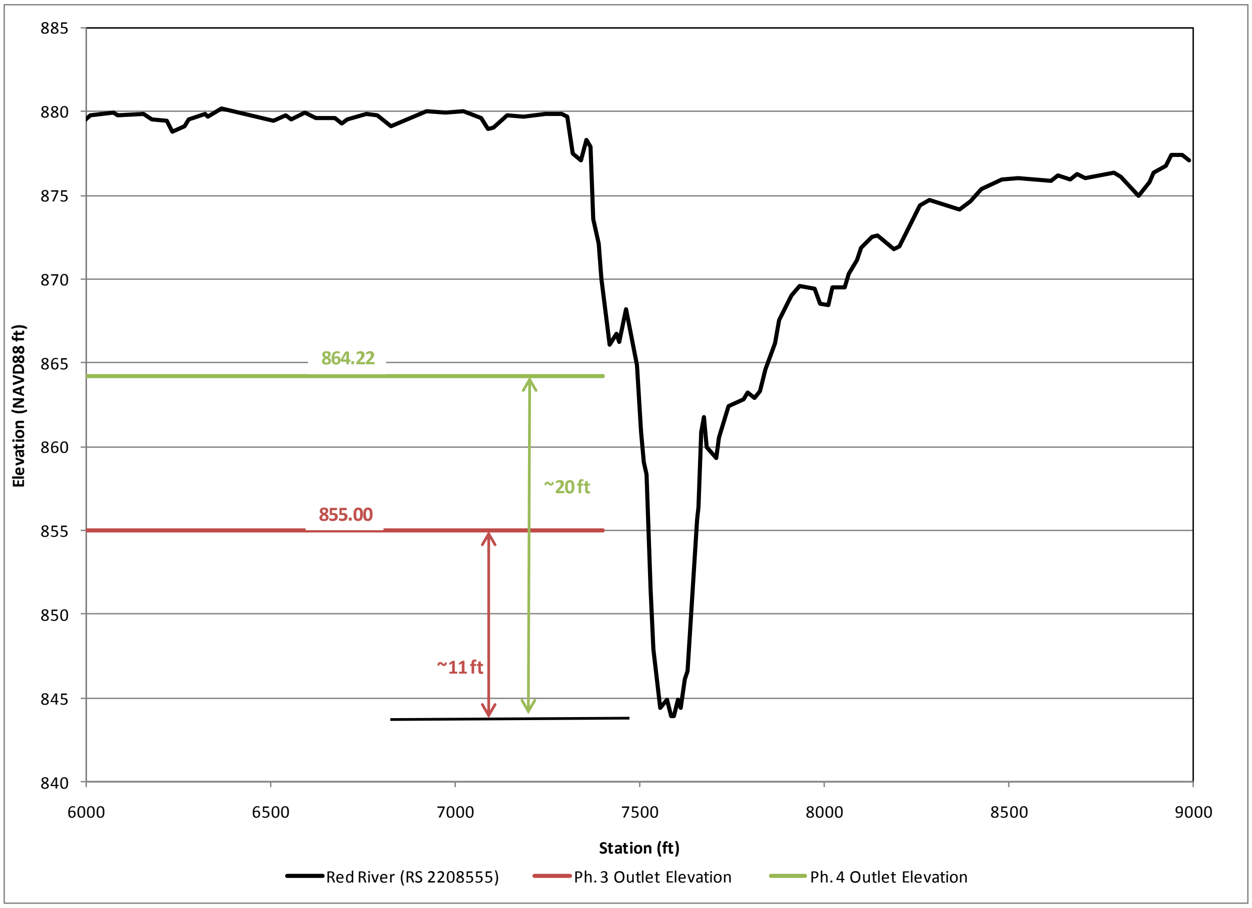


Figure F-D53 Comparison of Phase 3 and Phase 4 Drop from Outlet of Diversion Channel to Red River

In order to securely convey flow over the drop from the Diversion Channel to the Red River, the outlet structure was modified from a riprap channel to a concrete ogee type spillway. To prevent ponding at the outlet from the Diversion Channel the crest of the ogee spillway was set slightly above the main invert of the Diversion Channel. Figure F-D54 and Figure F-D55 show plan and profile schematics of the proposed Diversion Channel Outlet Structure.

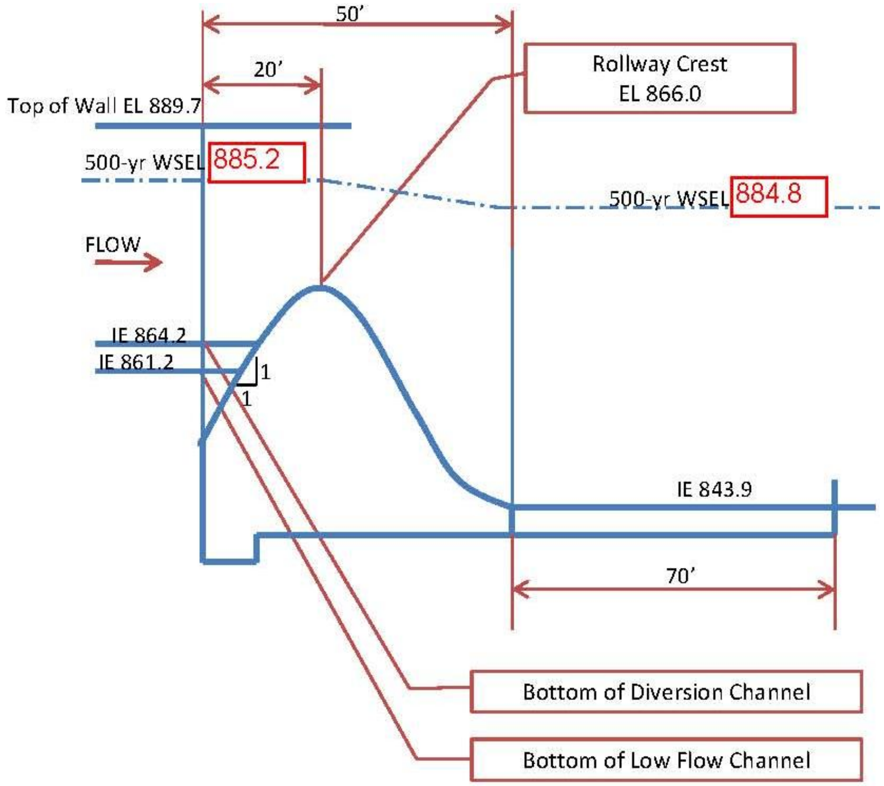


Figure F-D54 Outlet from Diversion Channel Profile View Schematic

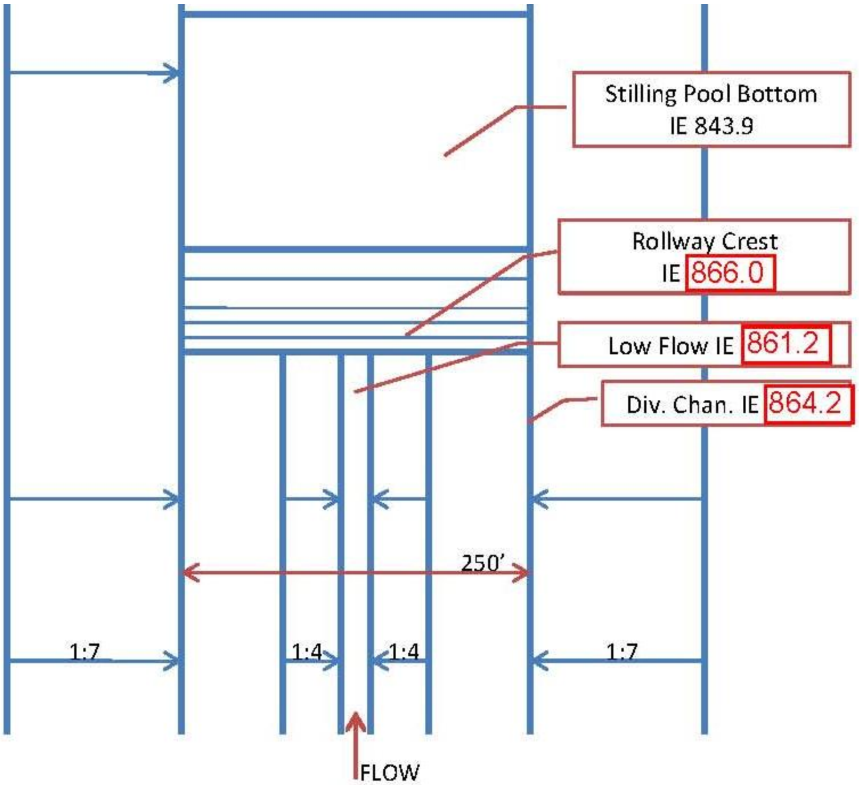


Figure F-D55 Outlet from Diversion Channel Plan View Schematic

F-D2.11 CONTROL STRUCTURE ON WOLVERTON CREEK

A description of the LPP control structure on Wolverton Creek can be found in Appendix F. This portion of Exhibit D presents design concepts and calculations for the Wolverton Creek control structure, detailed HEC-RAS modeling outputs at the control structure, and comparisons of water surface elevations at the control structure using Phase 4 hydrology.

F-D2.11.1 Flow Partitioning

For the events analyzed during Phase 4, the Wolverton Creek control structure functions as an open-close structure. In other words the gates on Wolverton Creek are either completely open or completely closed. During flood events, when the gates are closed, flows on Wolverton Creek, upstream of the control structure discharge into the area of staged water and are conveyed into the protected area either through one of the control structures located on the Red River or Wild Rice River, or through the inlet to the Diversion Channel.

F-D2.11.2 Gate Number and Sizing – Concepts

The number and sizing of the gates in the Wolverton Creek Control Structure is driven by the ability of the design to provide similar conveyance capacity to the culvert crossing that currently exists. Currently two 10x10ft box culverts are located below 130th Avenue South. The proposed control structure provides similar capacity culverts with functionality to close, or restrict, flows conveyed into the protected area.

F-D2.11.3 Gate Number and Sizing

It is desirable to have as little impact on the smaller, more frequent, flood events (i.e. no change in WSEL or gate operation required). For these events the gates on Wolverton Creek will remain fully open. For larger flood events, the flows on Wolverton Creek are very small compared to flows on the Red River and Wild Rice River which determine how high water is staged upstream of the Project. For this reason, the gates on Wolverton Creek were left in the closed position, and flows into the protected area were controlled by the gates located on the Red River and Wild Rice River. In order to maintain computational stability in the HEC-RAS unsteady flow model, however, the Wolverton Creek control structure gates were left partially open. In reality, when the Wolverton Creek control structure gates are operated they will either be fully open or fully closed during a flood event.

The gate height selected for the Wolverton Creek control structure matches the height of the existing culverts, 10-ft tall. However, the top of the control structure was determined based on the water surface elevation upstream of the control structures during a 500-year flood event. A concept drawing for the control structure is presented in Figure F-D56.

Water surface elevations on either side of the Wolverton Creek control structure for varying flood events are presented Table F-D40 and graphically in Figure F-57. The location of the control structure in the LPP HEC-RAS model is inline structure station 9079.5.

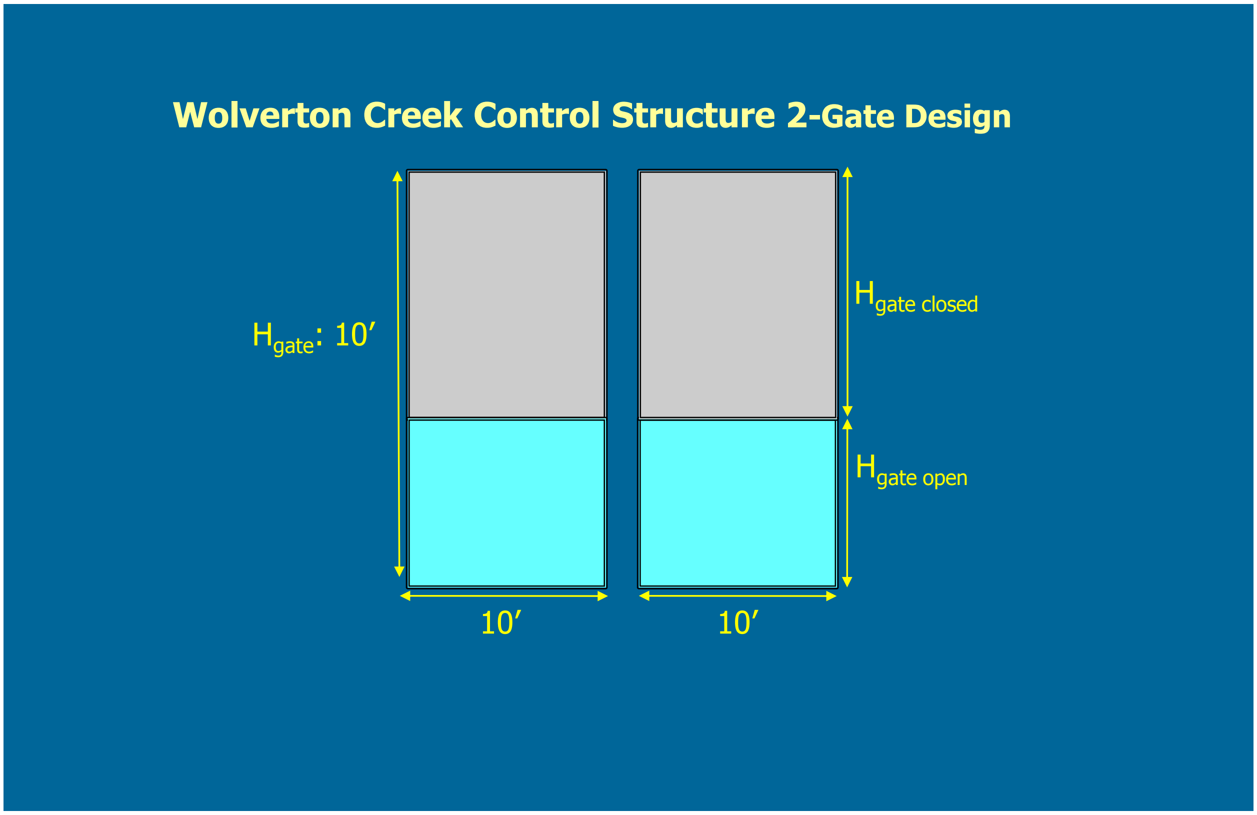


Figure F-D56 Wolverton Creek Control Structure 2-Gate Design

Table F-D40 HEC-RAS Outputs for the Proposed LPP Control Structure on Wolverton Creek

Event (yr)	Existing Water Surface Elevation (ft)[2]	Existing Flow into Protected Area (cfs)[2]	Proposed Headwater Elevation[1] (ft)	Proposed Tailwater Elevation[1] (ft)
10-Year	907.47	1950.39	916.3	903.75
50-Year	912.98	2509.00	920.86	904.44
100-Year	913.80	2731.78	922.88	905.62
500-Year	914.87	4291.07	922.46	913.86

[1] In the Phase 4With Project HEC-RAS model the gates at the Wolverton Creek control structure are only open slightly. In reality it is assumed they will be completely closed

[2] The existing flow and water surface elevations in this portion of Wolverton Creek are controlled by tailwater from the Red River

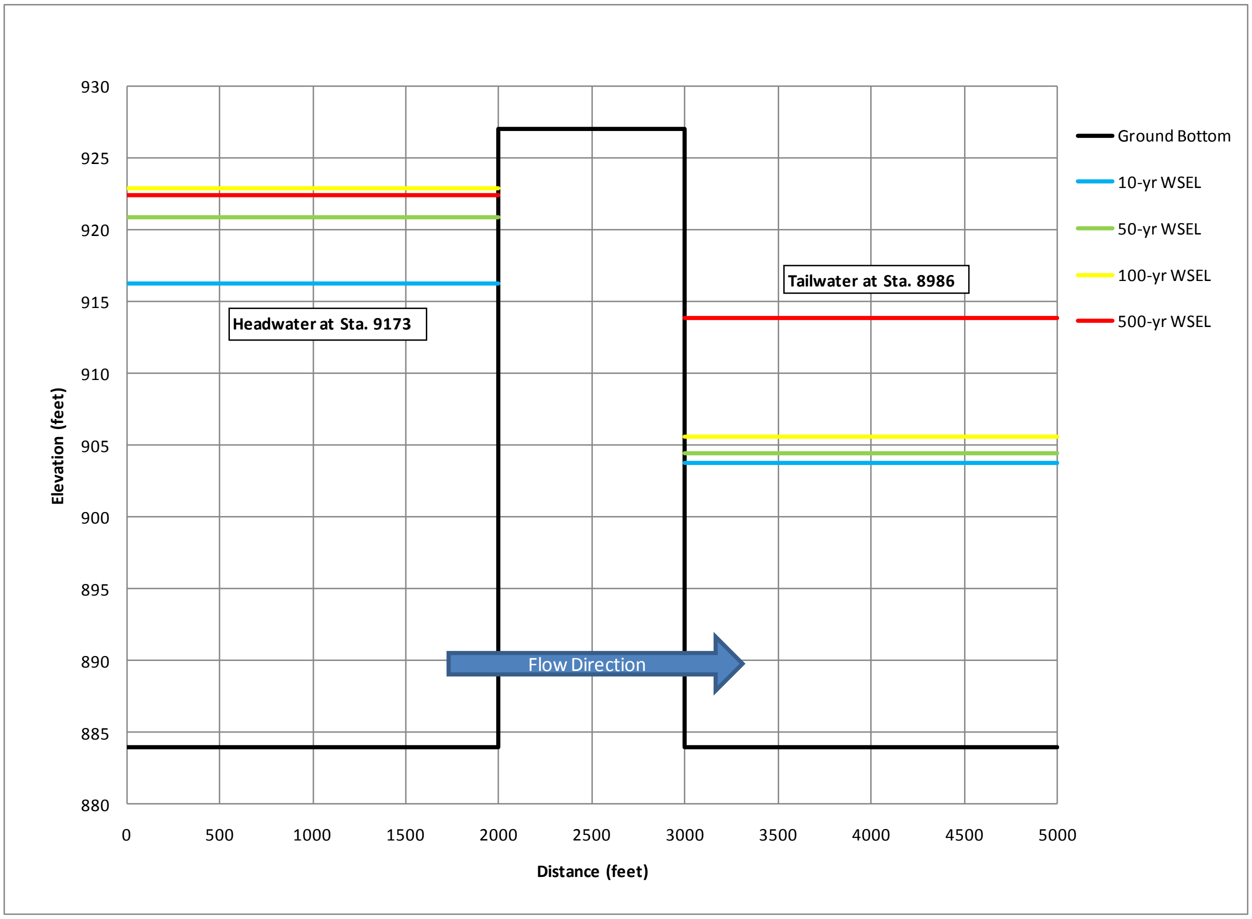


Figure F-D57 Comparison of the Headwater and Tailwater Elevations at the Wolverton Creek Control Structure

F-D2.11.4 <u>Transition from the Natural Channel to the Control Structure</u>

The transition from the natural channel into the control structure is accomplished with 5:1 side slopes, as shown in Figure F-D58.

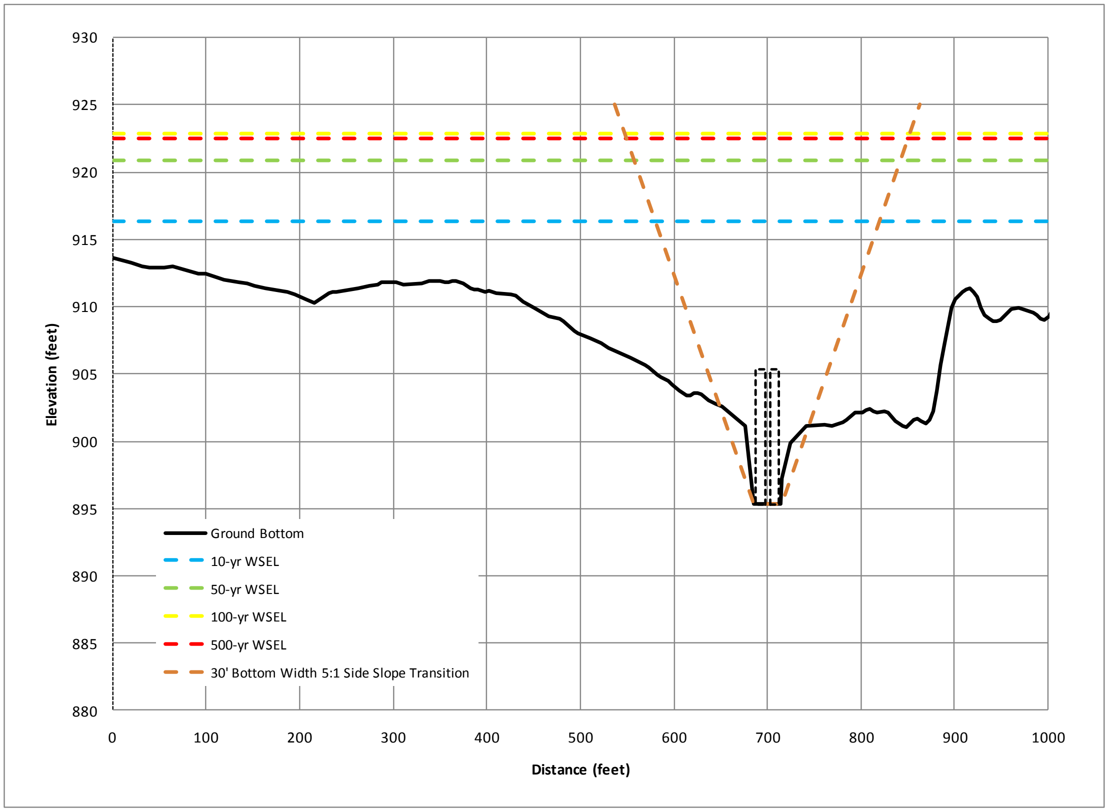


Figure F-D58 **Transition from the Natural Wolverton Creek Cross Section into the Control Structure (Station 9173)**

F-D3.0 WATER SURFACE ELEVATION PROFILES ALONG THE DIVERSION CHANNEL

F-D3.1 WATER SURFACE ELEVATION PROFILES ALONG THE FCP DIVERSION CHANNEL

The following information regarding the FCP was previously presented as part of Exhibit F in the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

The water surface elevation profiles along the FCP Diversion Channel for the 5-, 10-, 20-, 50-, 100-, 200-, and 500-year events are presented in Figures F-D59 to F-D66 for Year 0 Hydrology, Figures F-D67 to F-D74 for Year 25 Hydrology, and Figures F-D75 to F-D82 for Year 50 Hydrology. The location of the Inlet Weir is noted on the figures and head difference over this structure can be seen. Tailwater effects from the Red River of the North are apparent for all events in the downstream portion of the FCP Diversion Channel. No water from the Red River is diverted into the FCP Diversion Channel for the 5-year event for Year 50 hydrology; however, backwater effects from the Red River can be seen in the downstream portion of the FCP Diversion Channel (Figure F-D75 and F-D76).

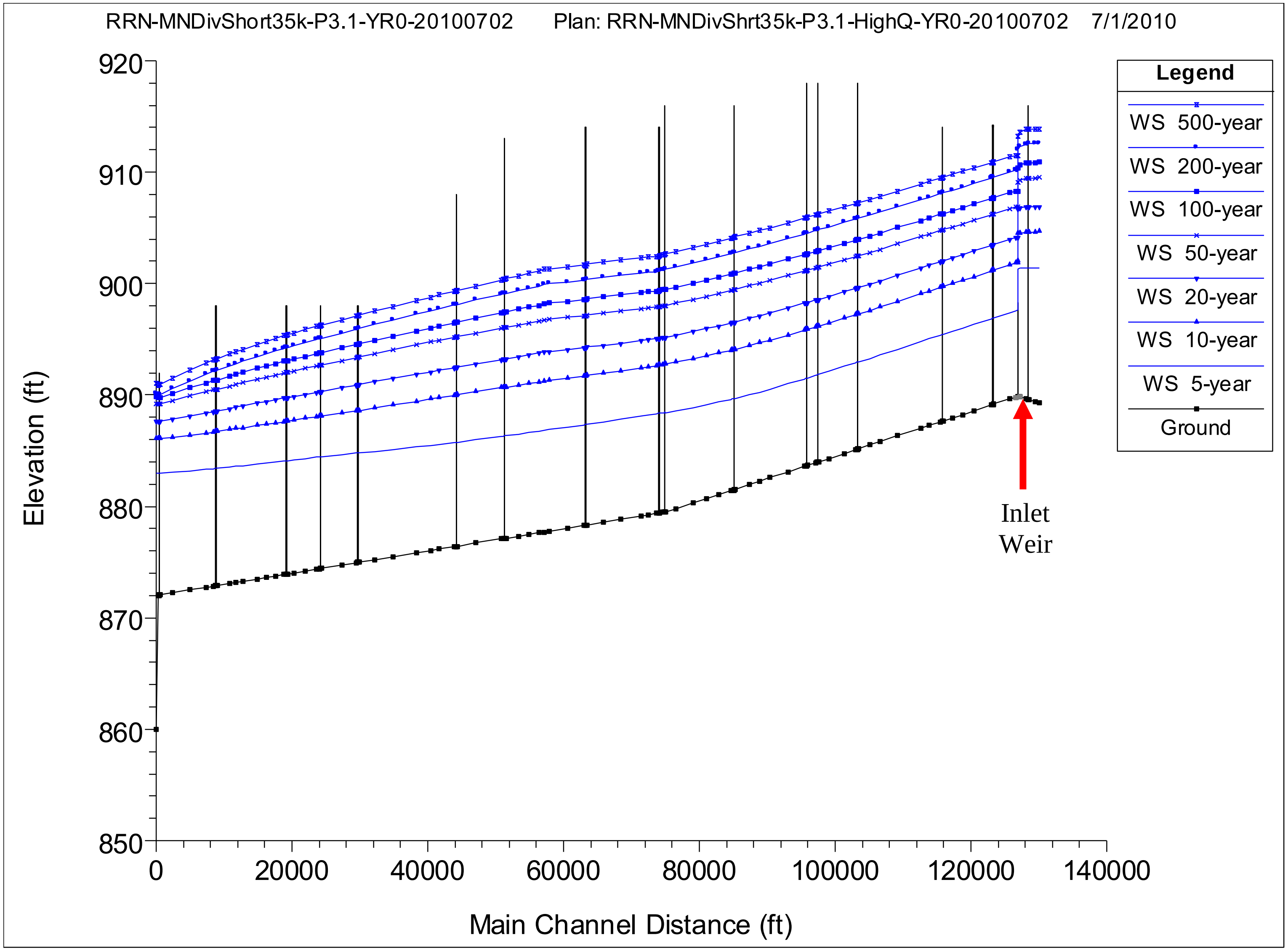


Figure F-D59 FCP Diversion Channel Water Surface Elevation Profile for All Modeled Events for Year 0 Hydrology

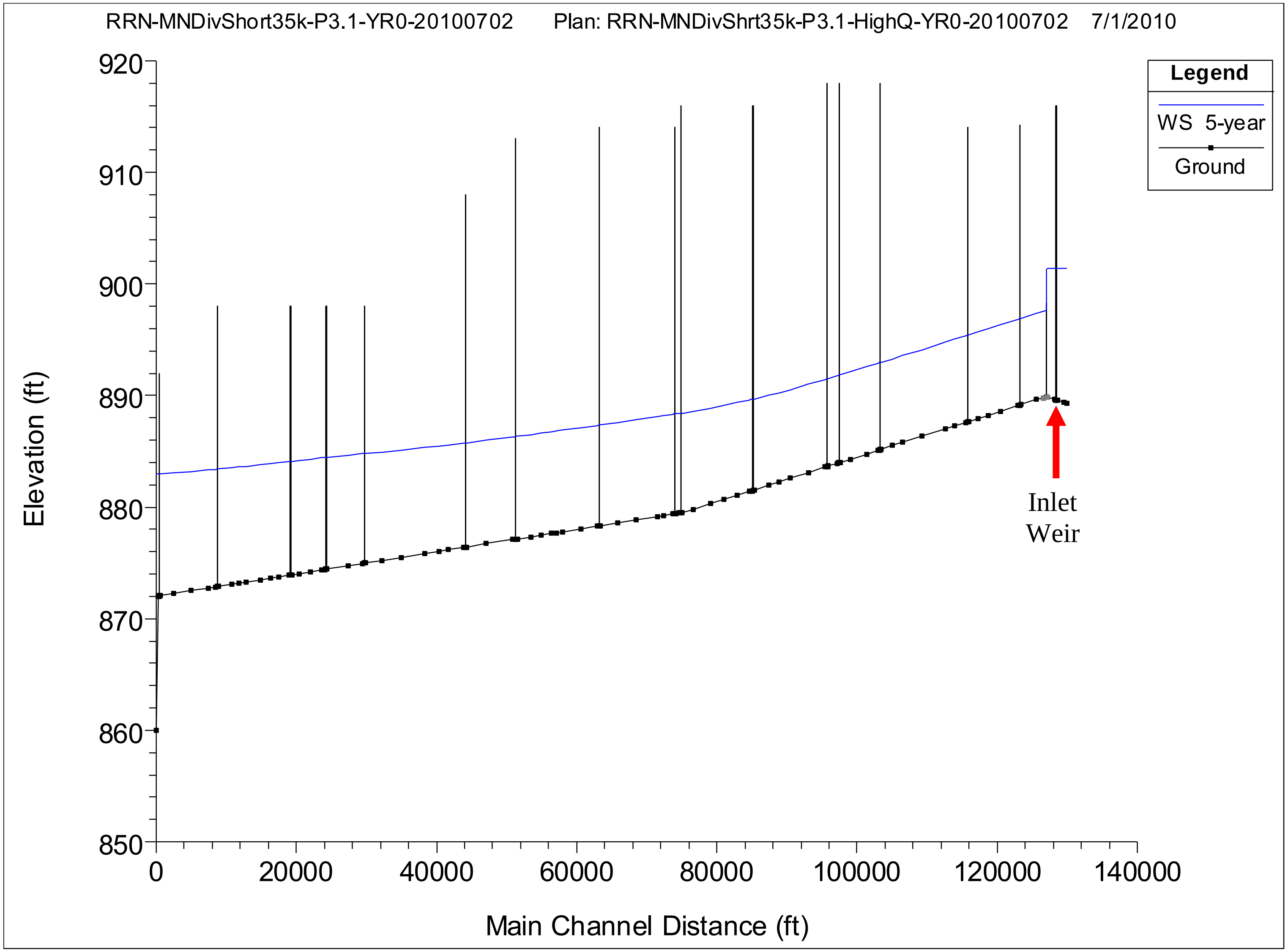


Figure F-D60 FCP Diversion Channel Water Surface Elevation Profile for the 5-Year Event for Year 0 Hydrology

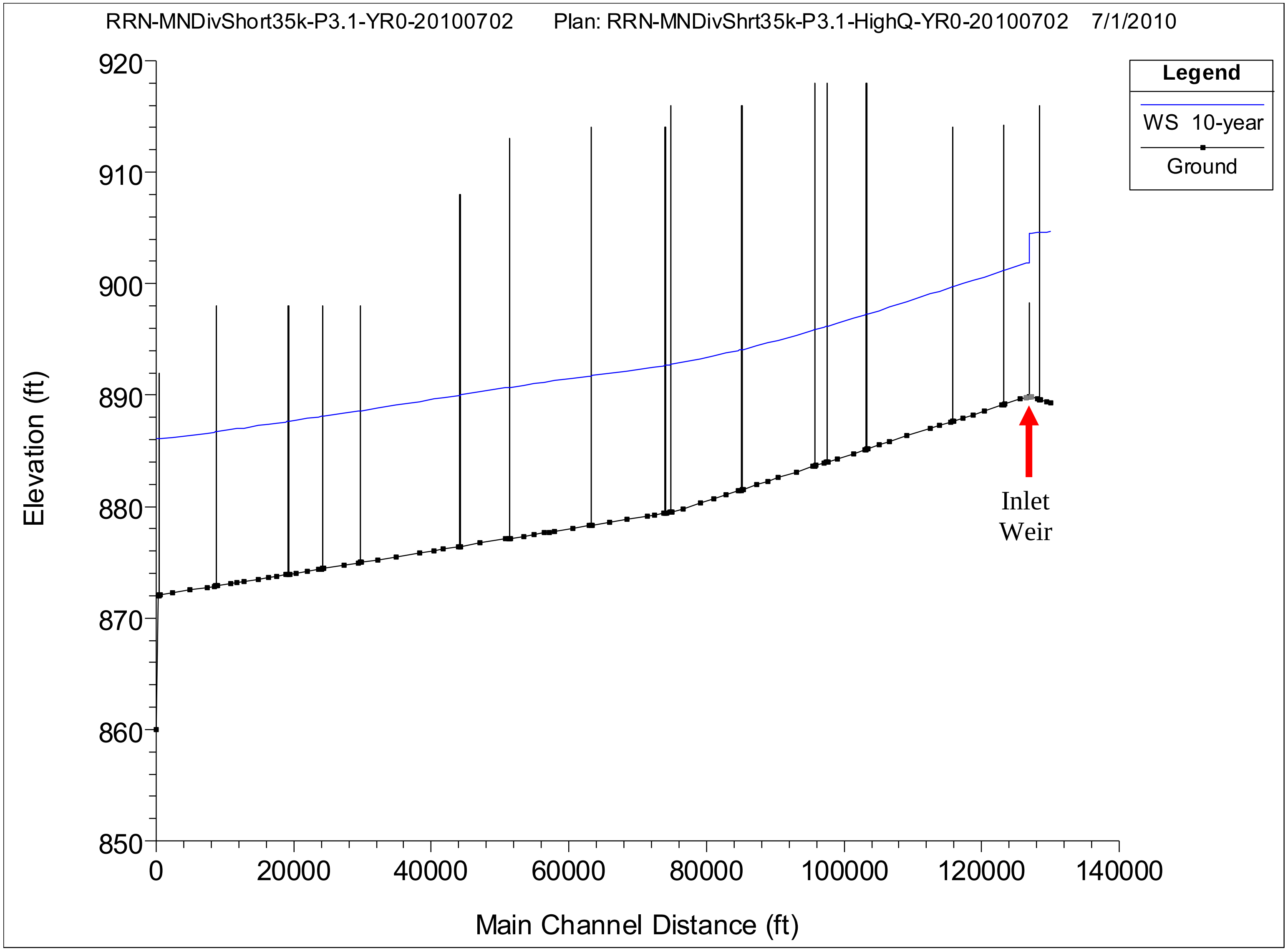


Figure F-D61 **FCP Diversion Channel Water Surface Elevation Profile for the 10-Year Event for Year 0 Hydrology**

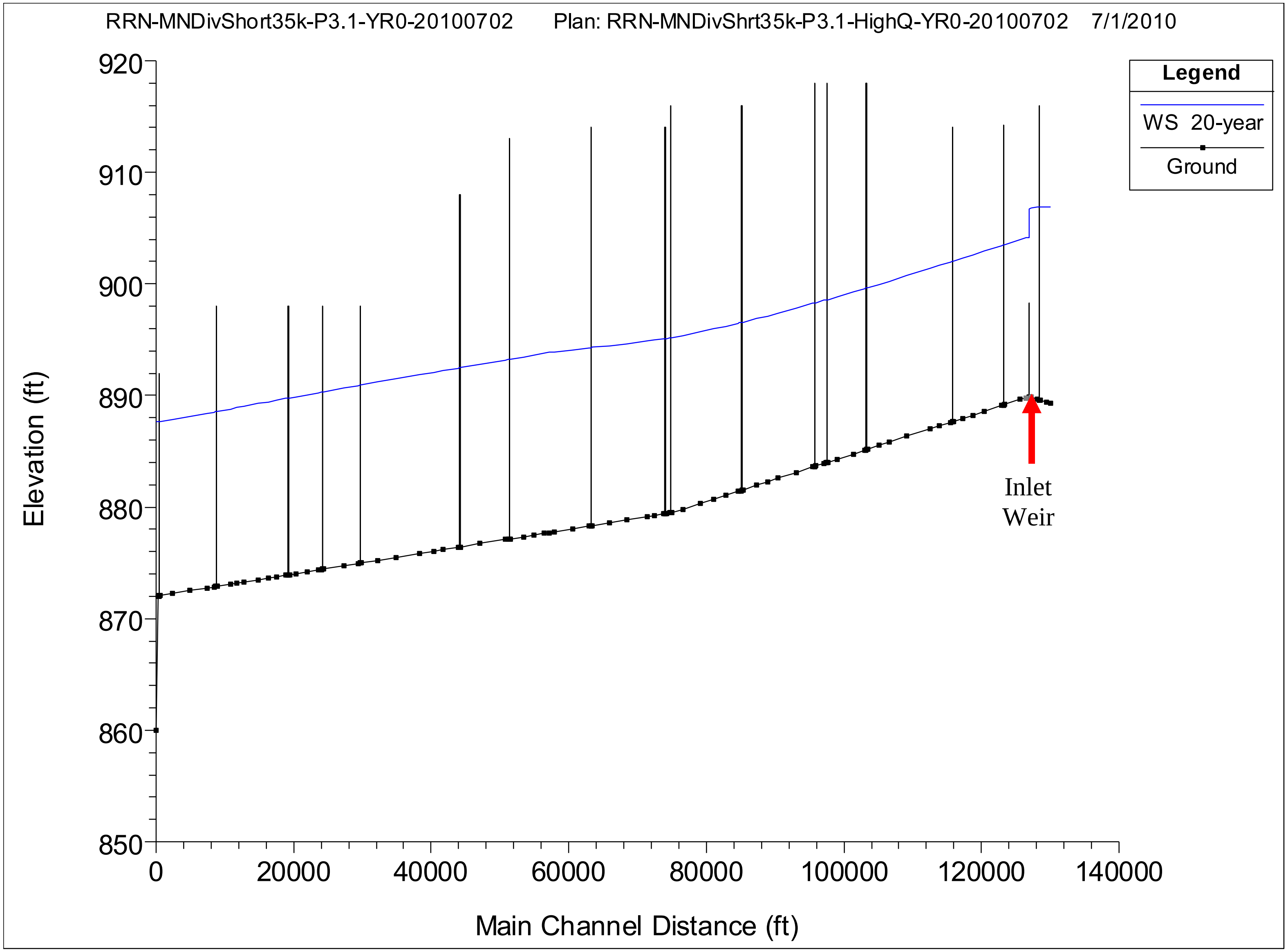


Figure F-D62 FCP Diversion Channel Water Surface Elevation Profile for the 20-Year Event for Year 0 Hydrology

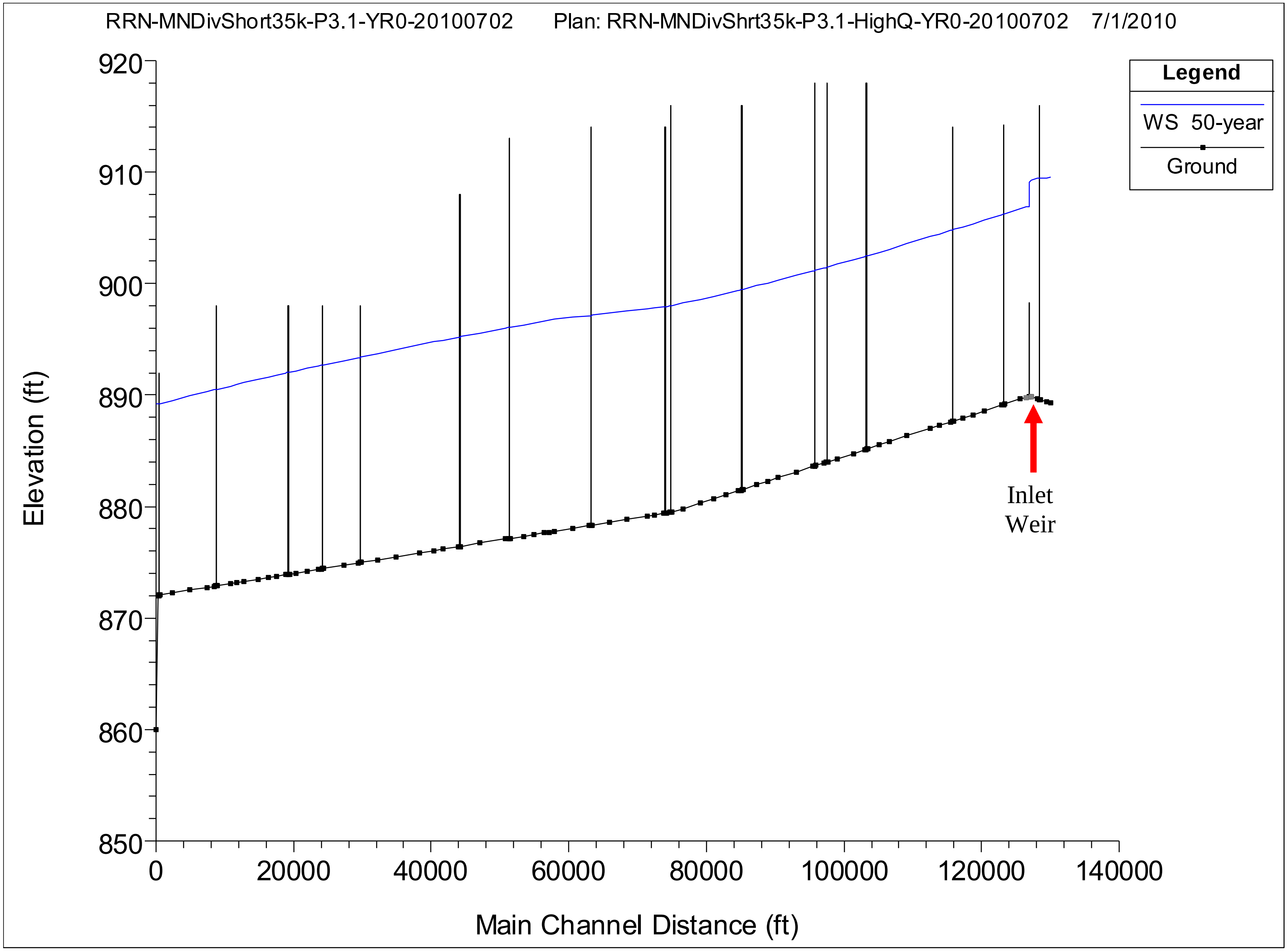


Figure F-D63 FCP Diversion Channel Water Surface Elevation Profile for the 50-Year Event for Year 0 Hydrology

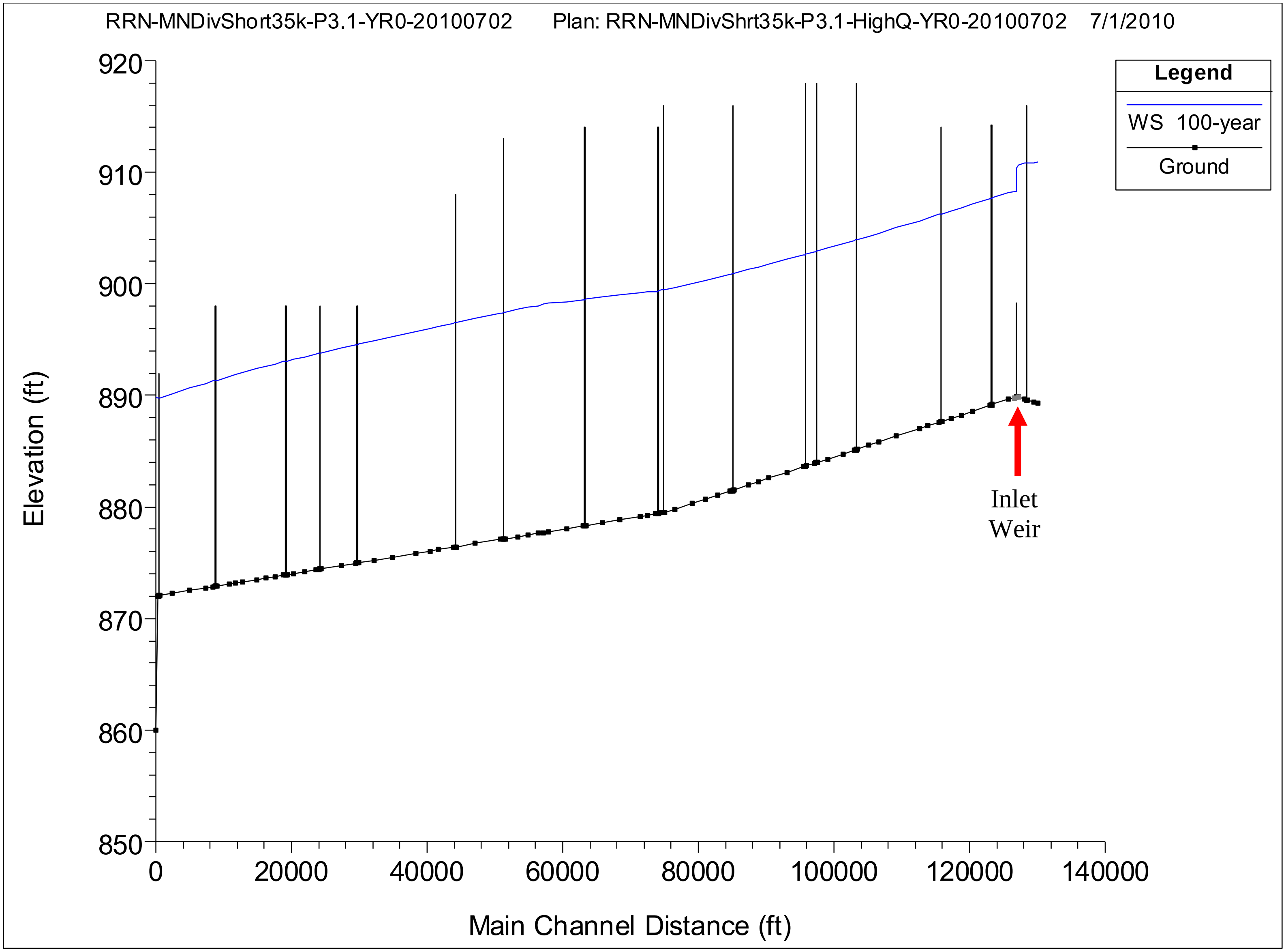


Figure F-D64 FCP Diversion Channel Water Surface Elevation Profile for the 100-Year Event for Year 0 Hydrology

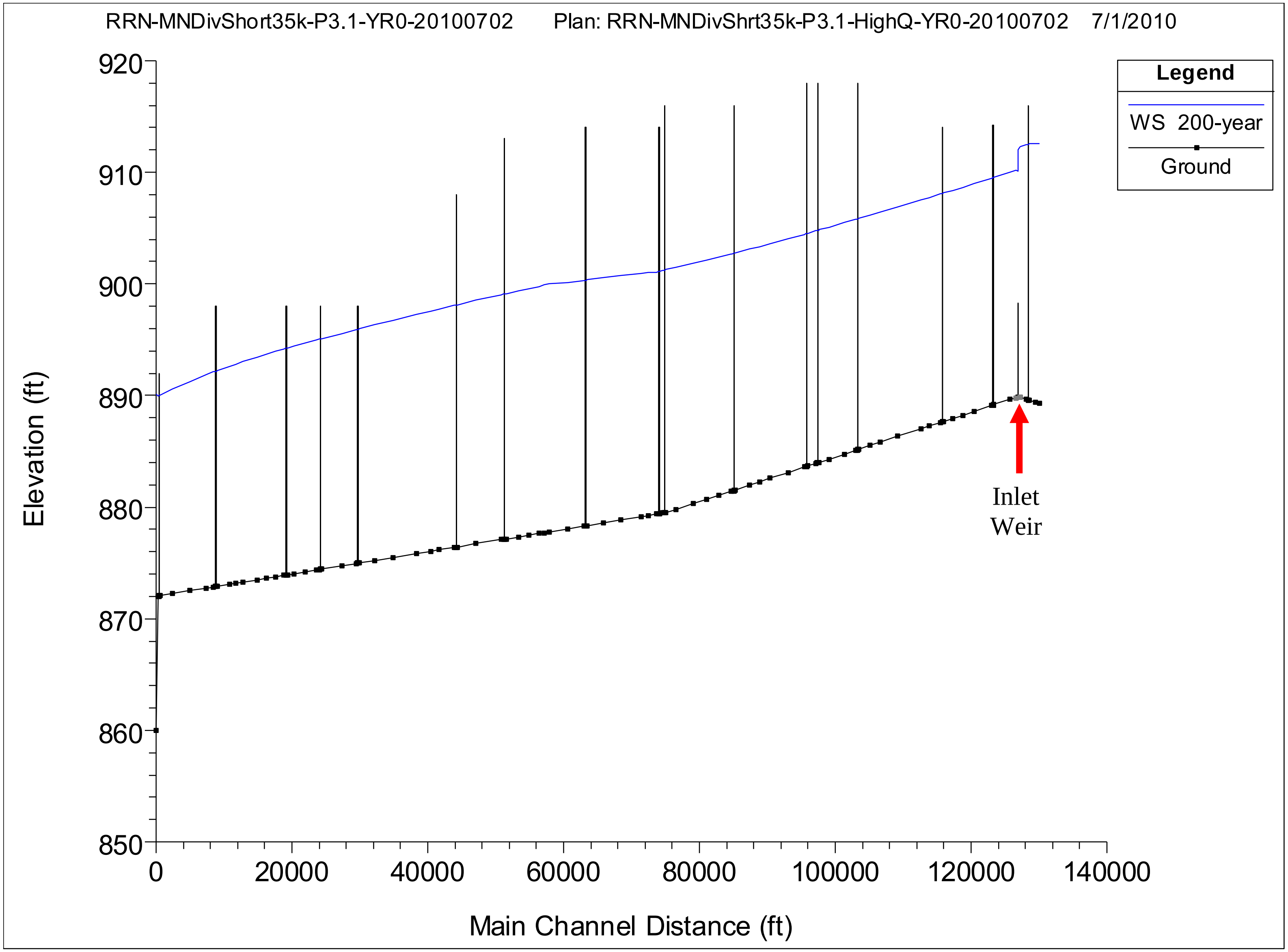


Figure F-D65 FCP Diversion Channel Water Surface Elevation Profile for the 200-Year Event for Year 0 Hydrology

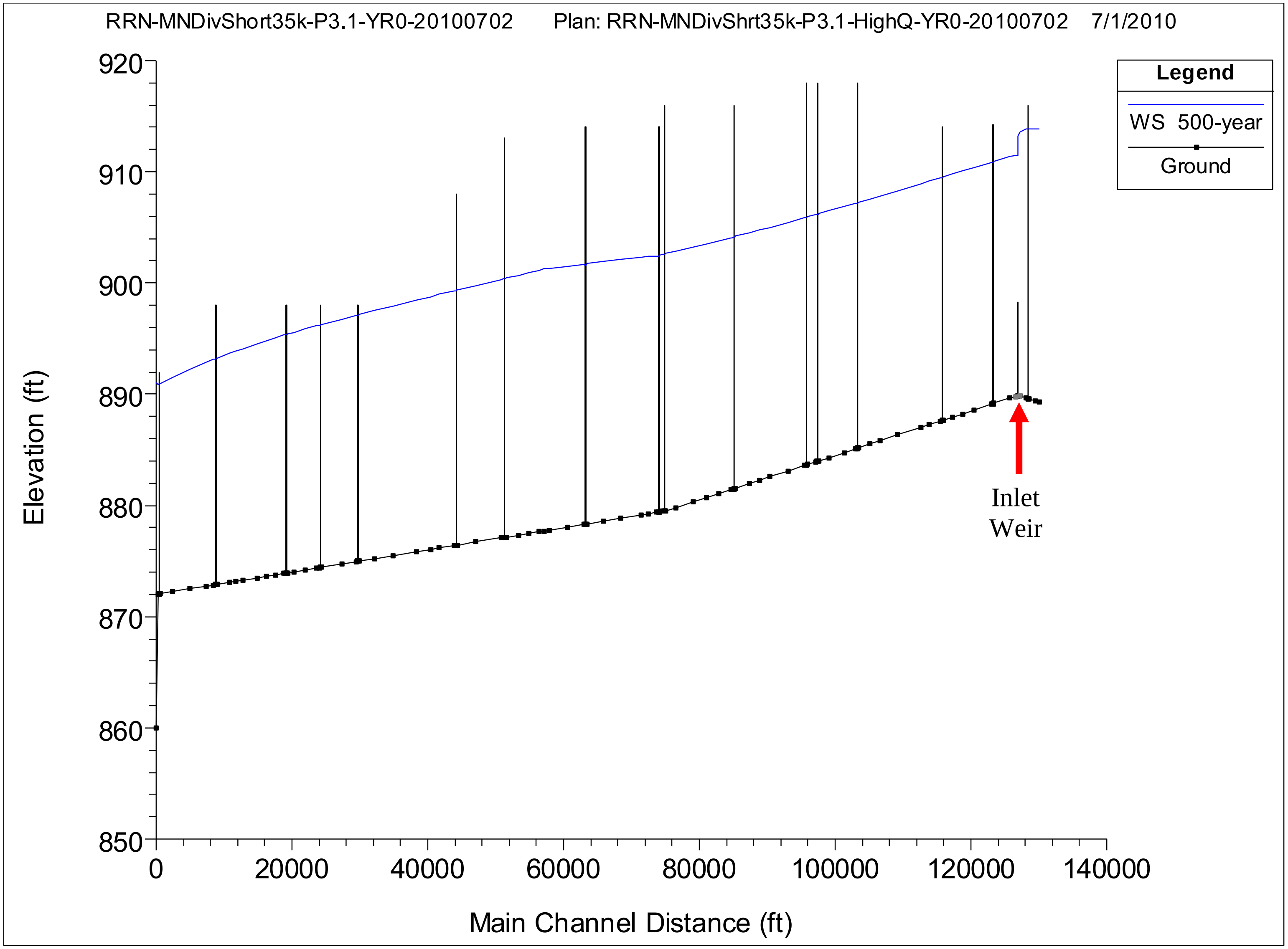


Figure F-D66 **FCP Diversion Channel Water Surface Elevation Profile for the 500-Year Event for Year 0 Hydrology**

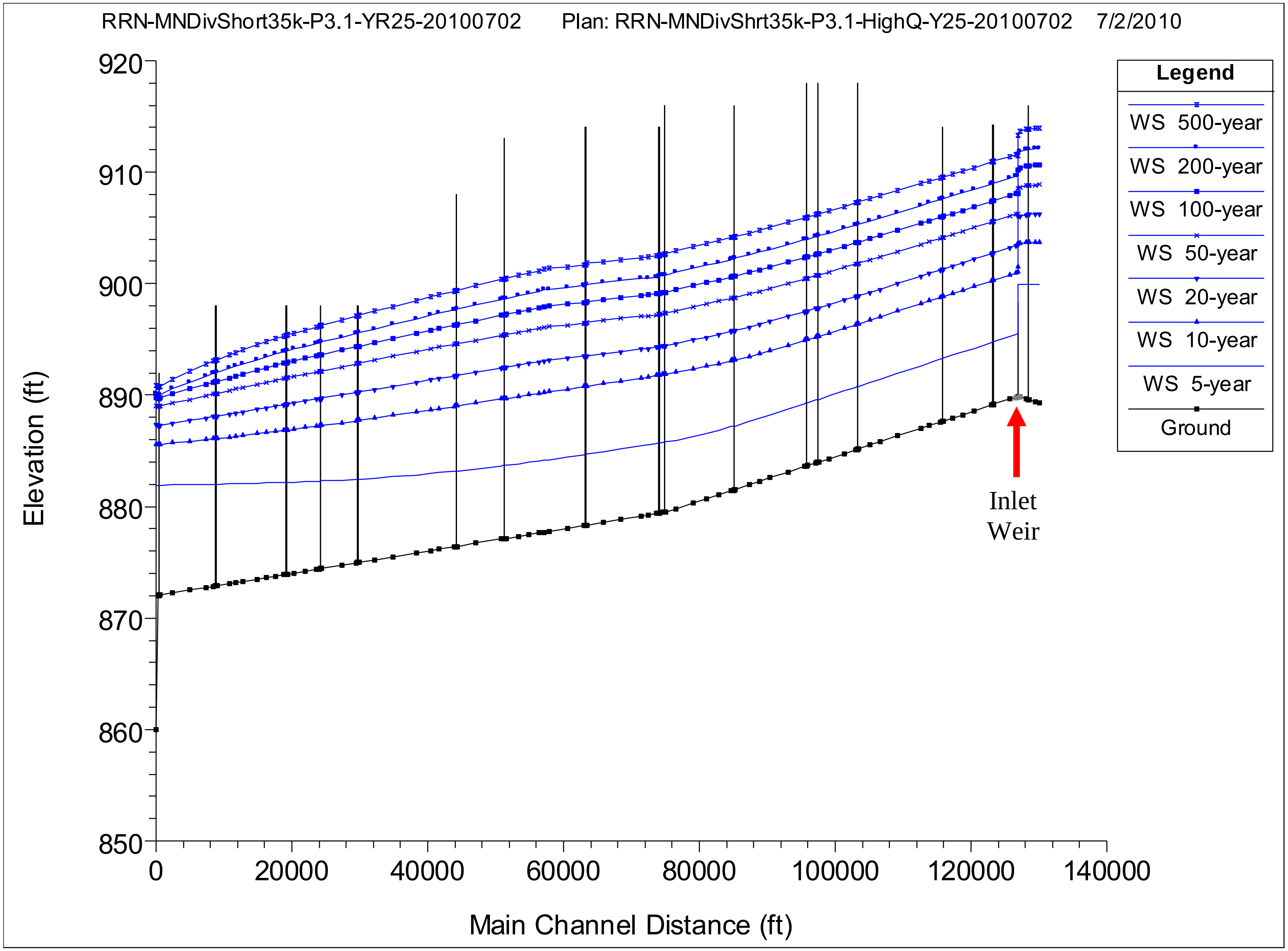


Figure F-D67 FCP Diversion Channel Water Surface Elevation Profile for All Modeled Events for Year 25 Hydrology

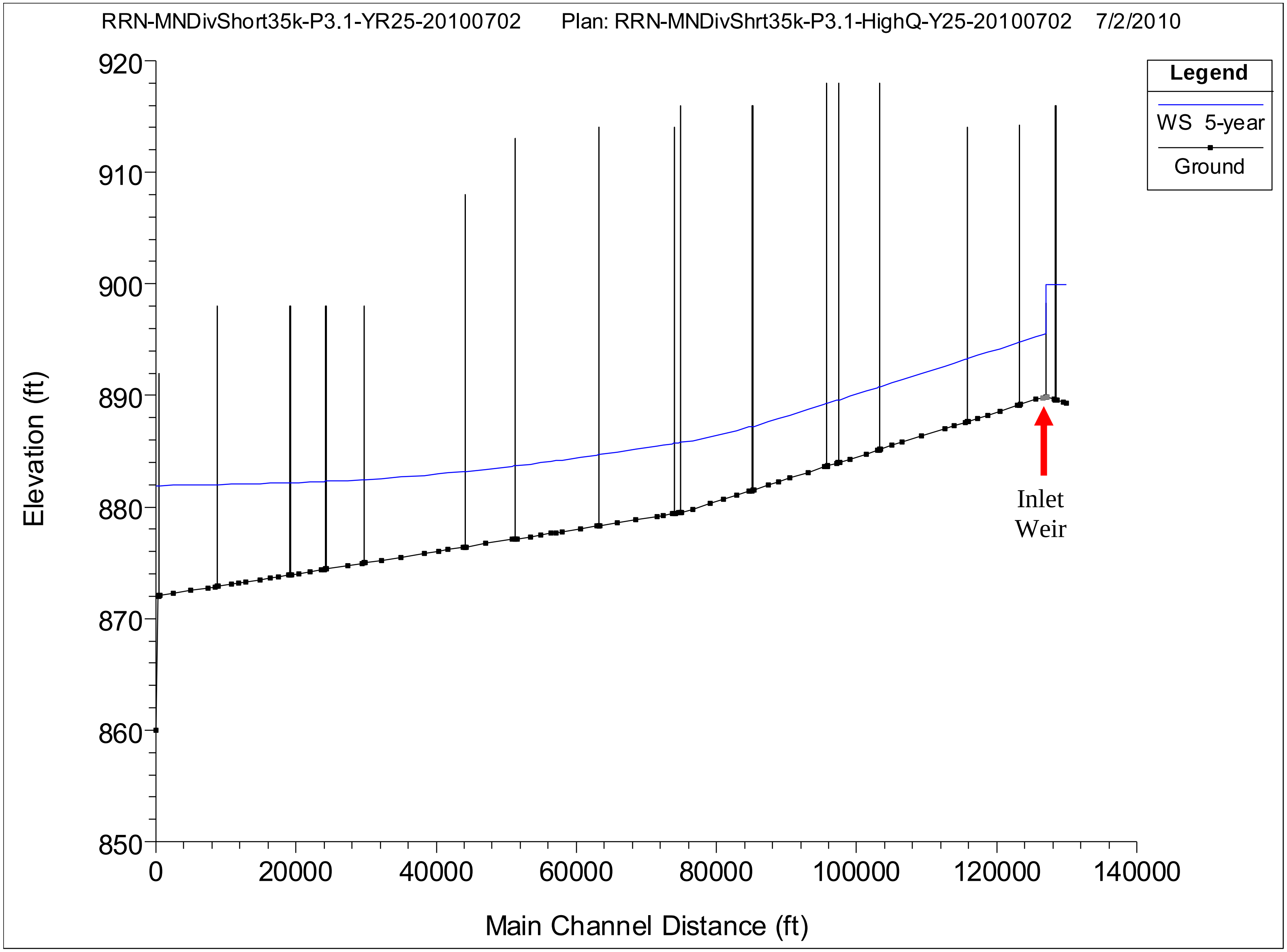


Figure F-D68 FCP Diversion Channel Water Surface Elevation Profile for the 5-Year Event for Year 25 Hydrology

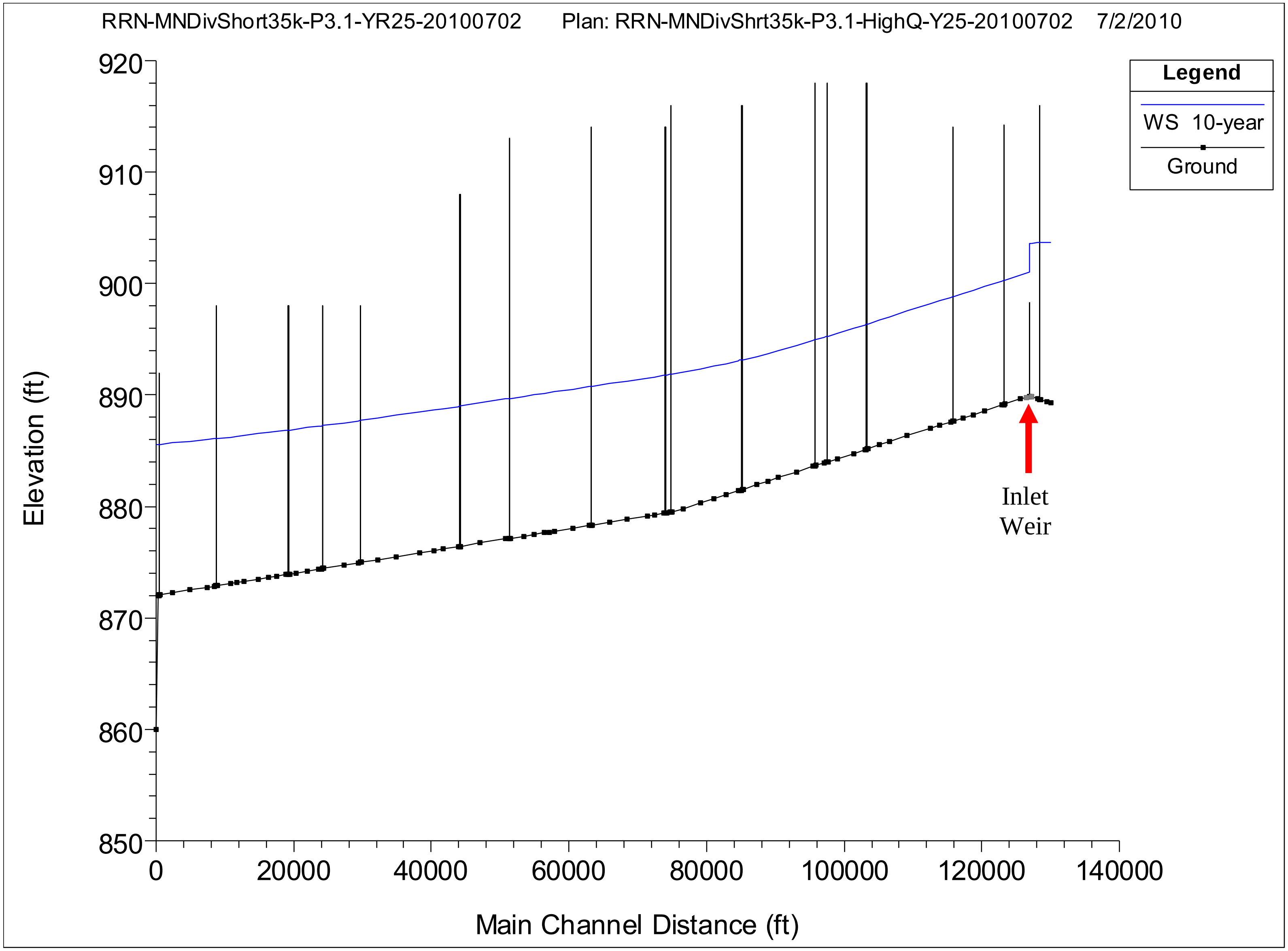


Figure F-D69 FCP Diversion Channel Water Surface Elevation Profile for the 10-Year Event for Year 25 Hydrology

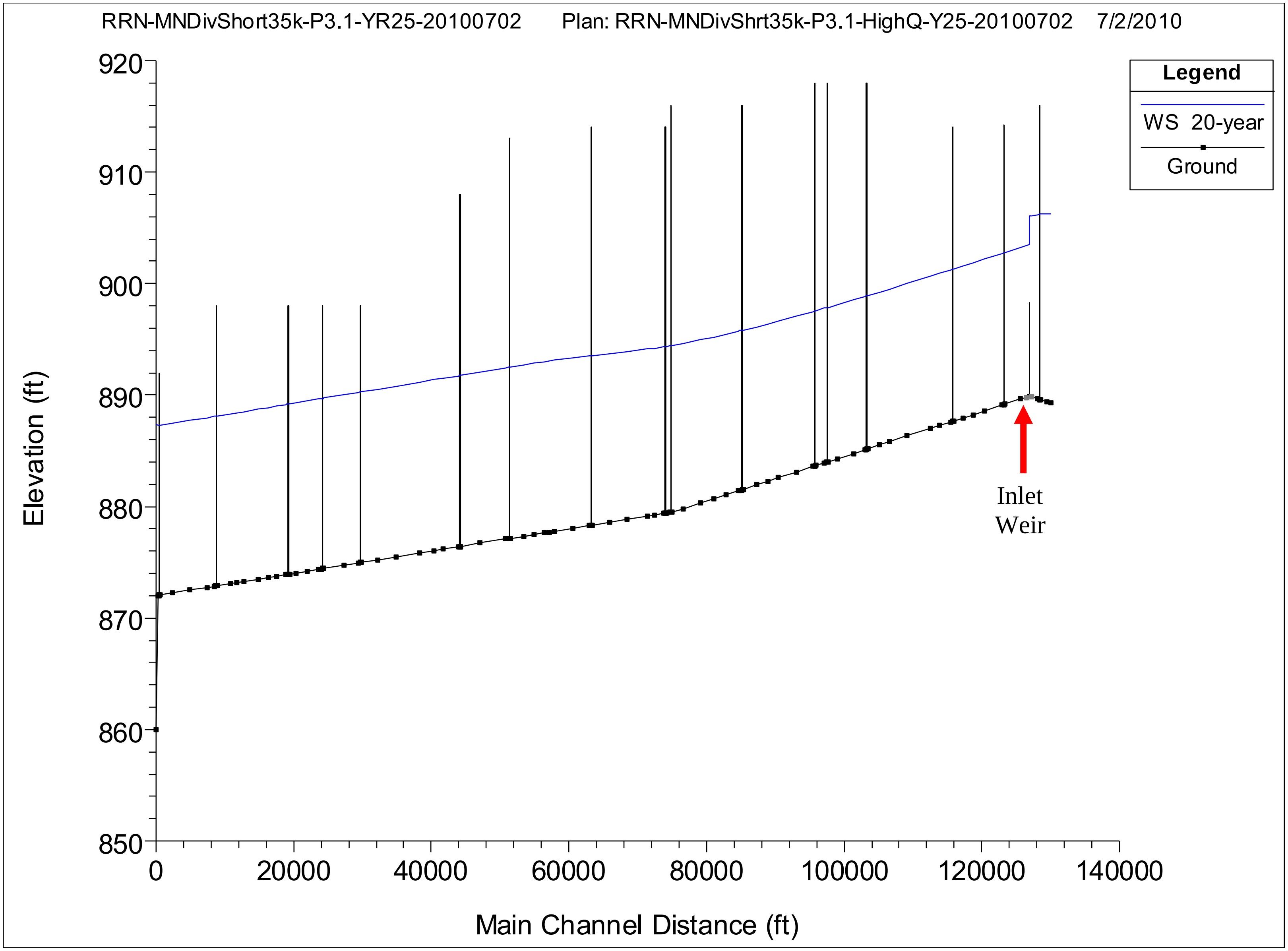


Figure F-D70 FCP Diversion Channel Water Surface Elevation Profile for the 20-Year Event for Year 25 Hydrology

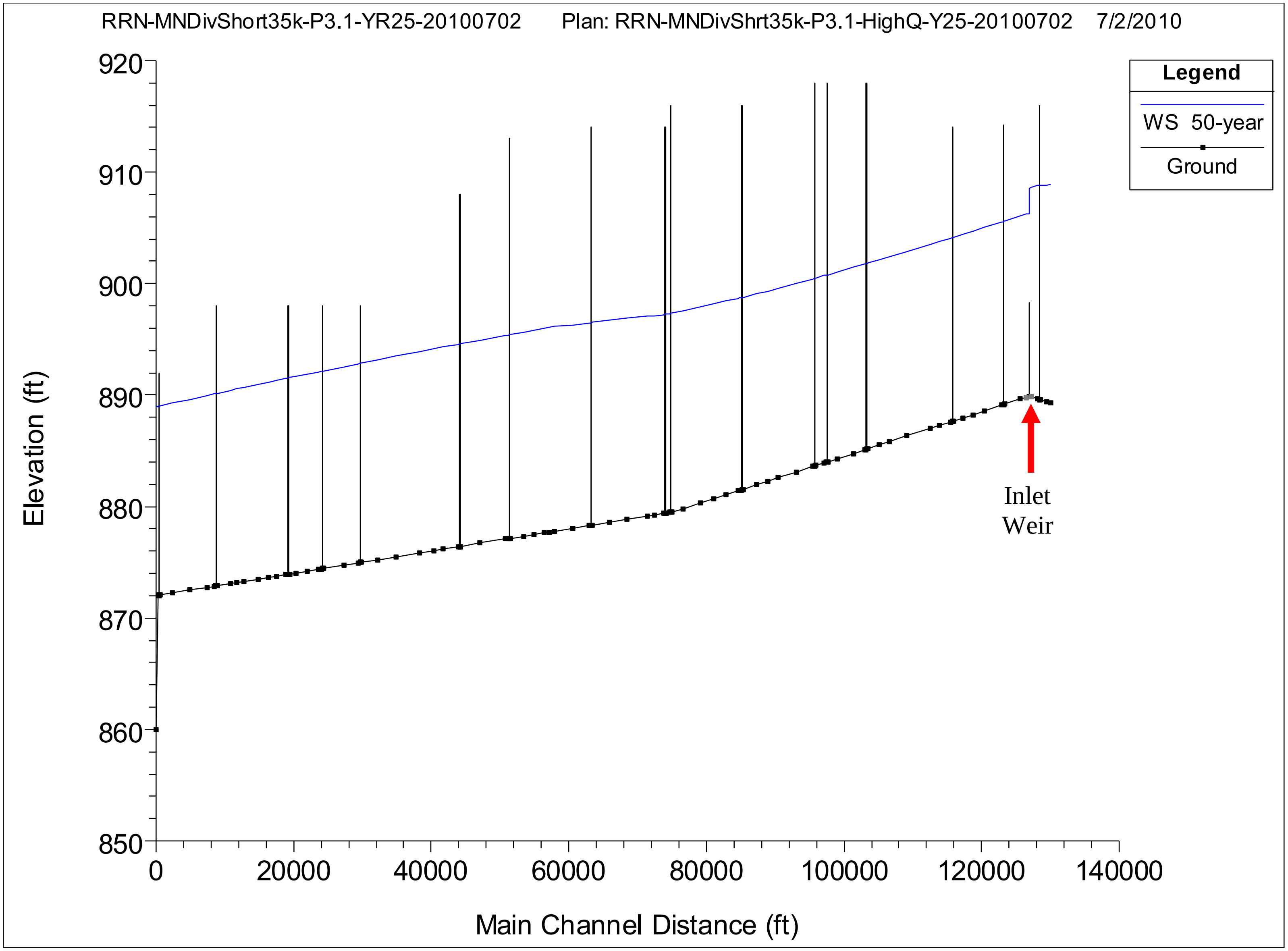


Figure F-D71 FCP Diversion Channel Water Surface Elevation Profile for the 50-Year Event for Year 25 Hydrology

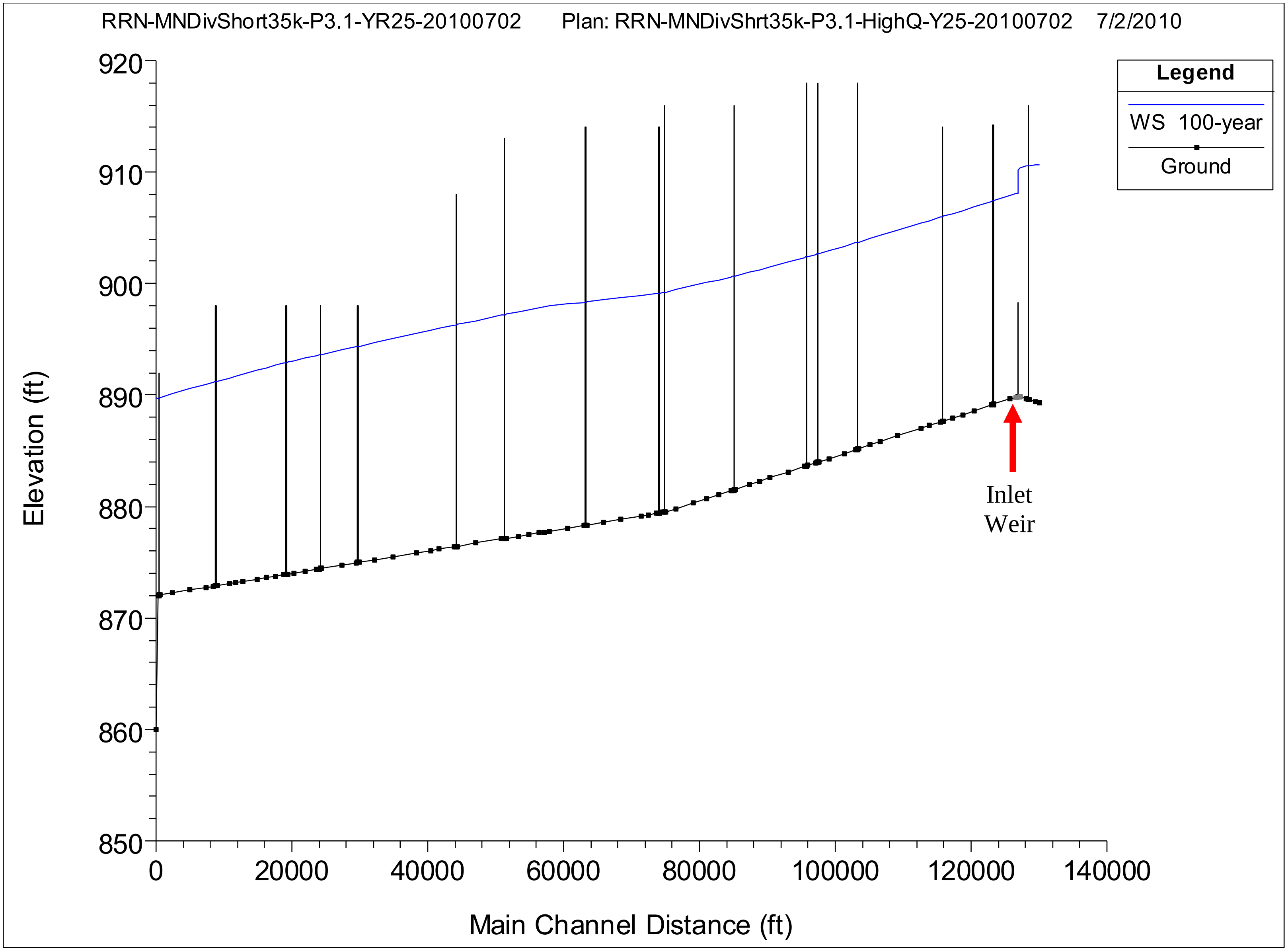


Figure F-D72 FCP Diversion Channel Water Surface Elevation Profile for the 100-Year Event for Year 25 Hydrology

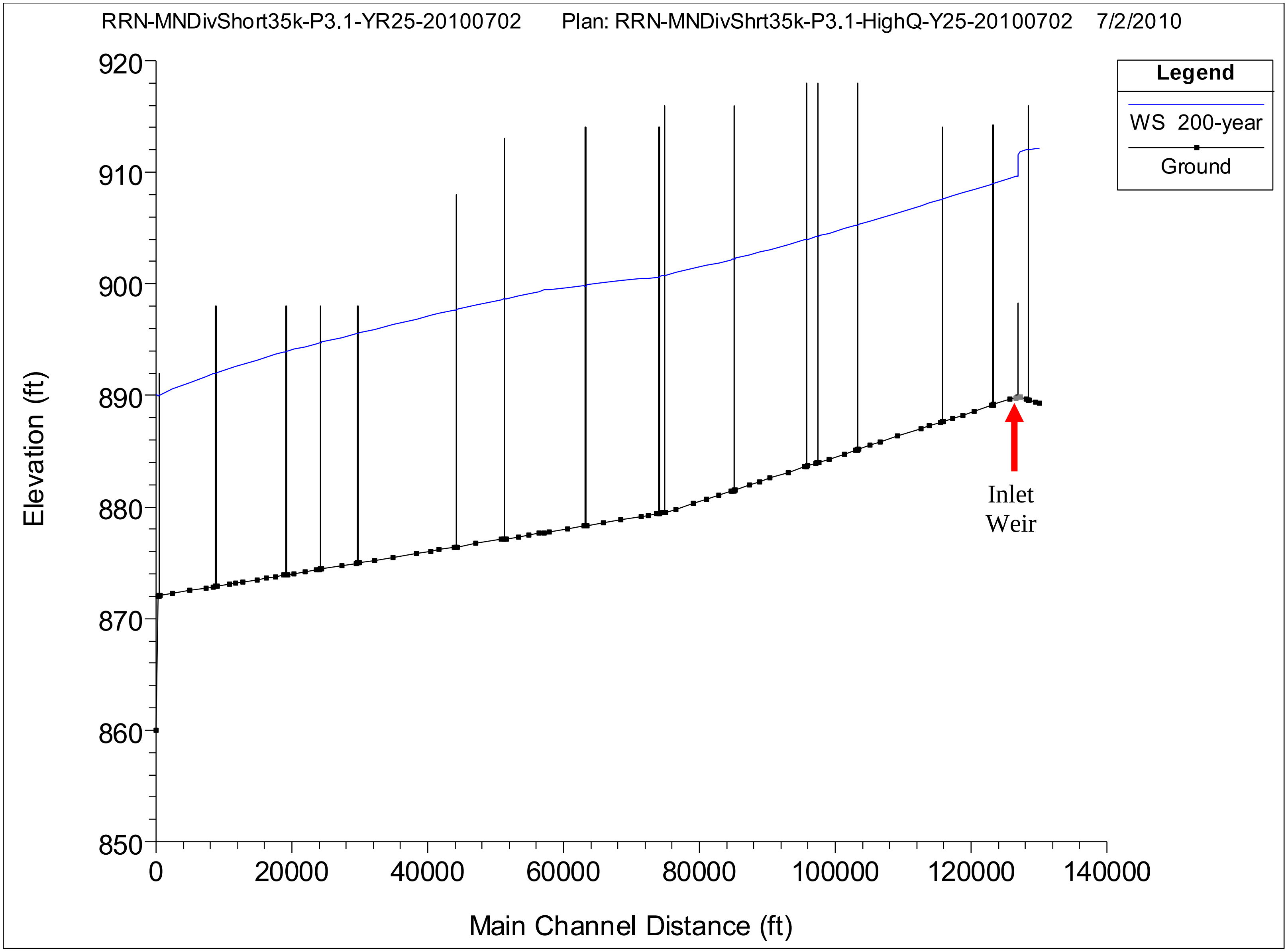


Figure F-D73 FCP Diversion Channel Water Surface Elevation Profile for the 200-Year Event for Year 25 Hydrology

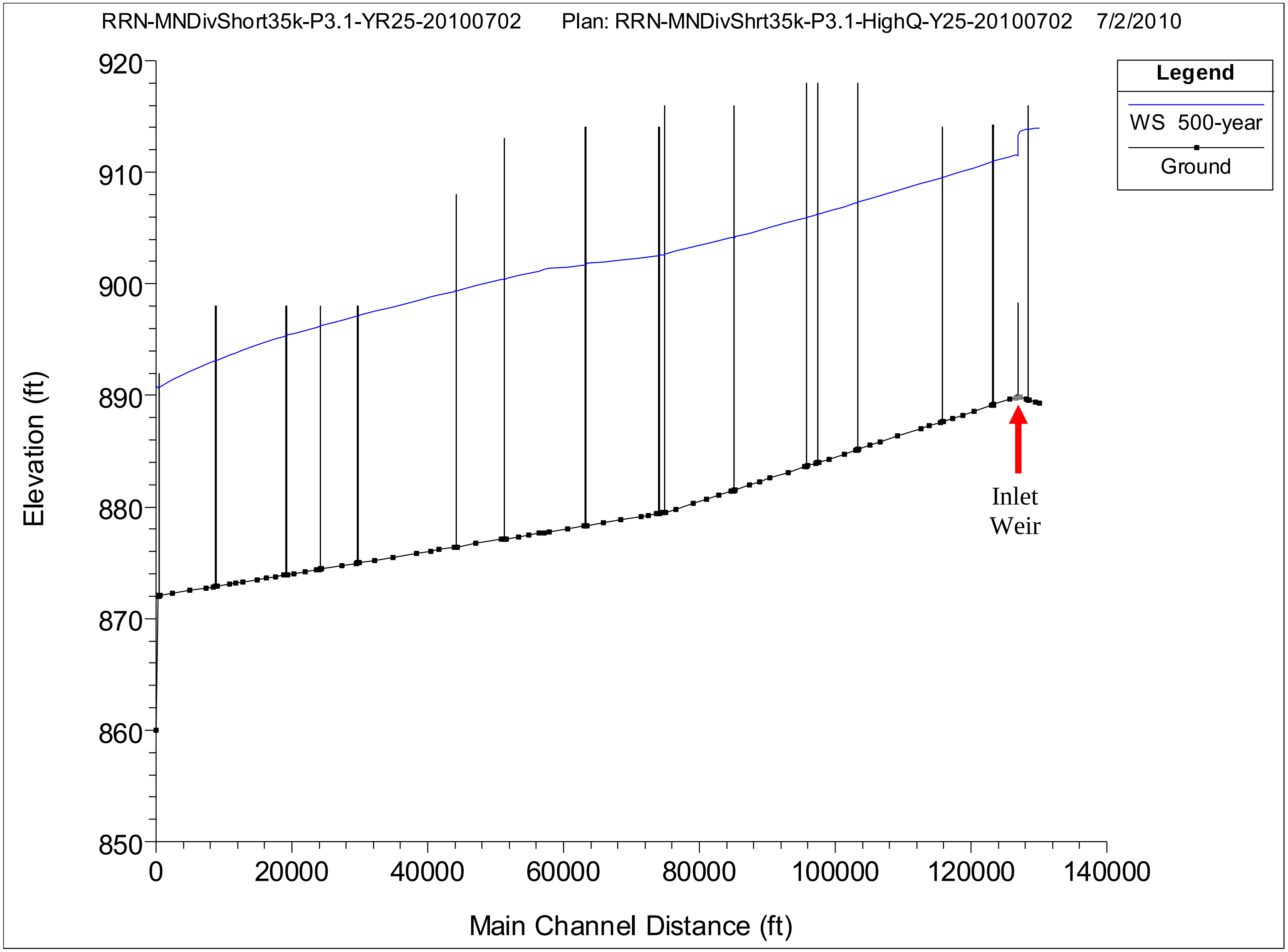


Figure F-D74 FCP Diversion Channel Water Surface Elevation Profile for the 500-Year Event for Year 25 Hydrology

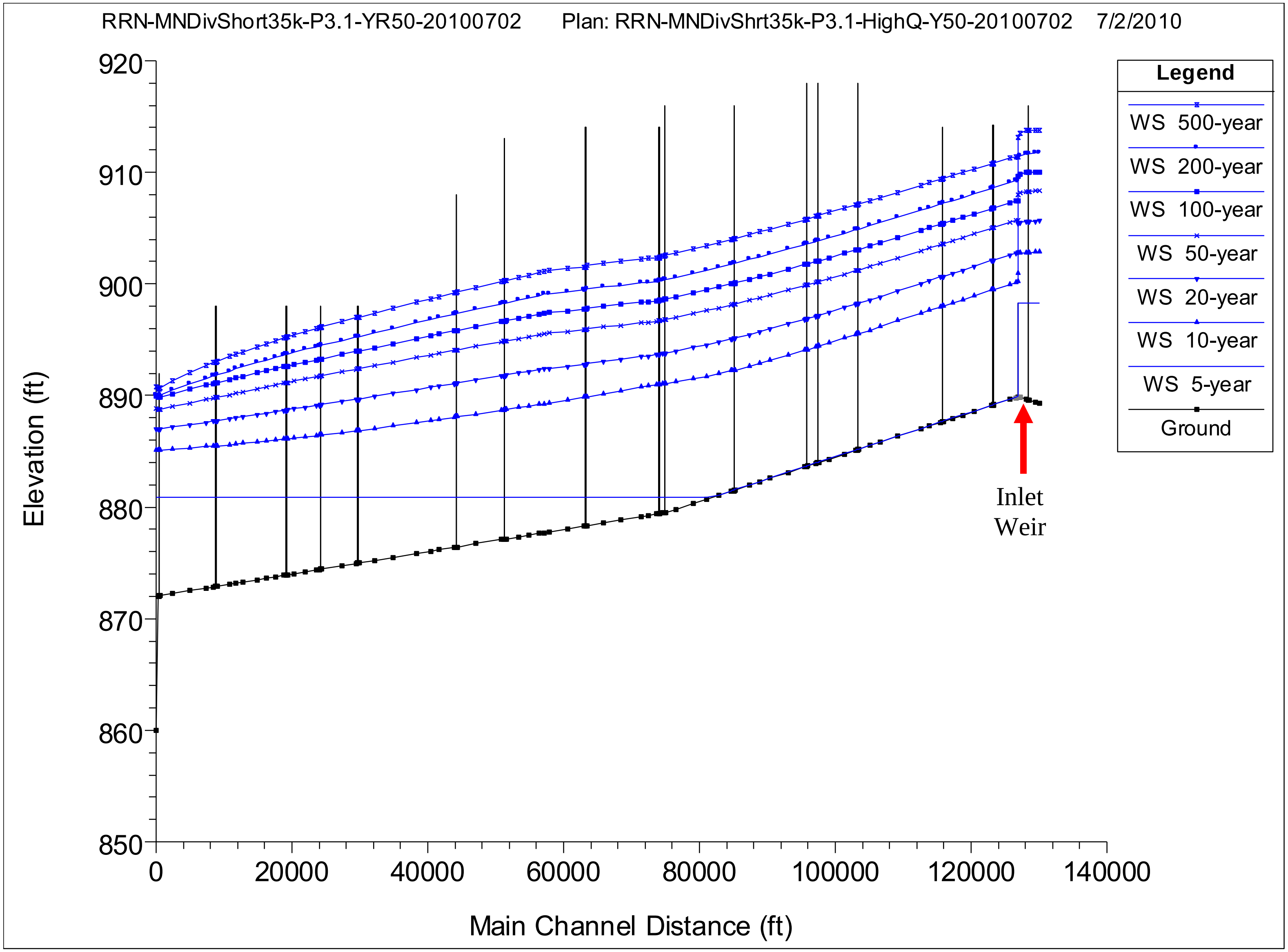


Figure F-D75 FCP Diversion Channel Water Surface Elevation Profile for All Modeled Events for Year 50 Hydrology

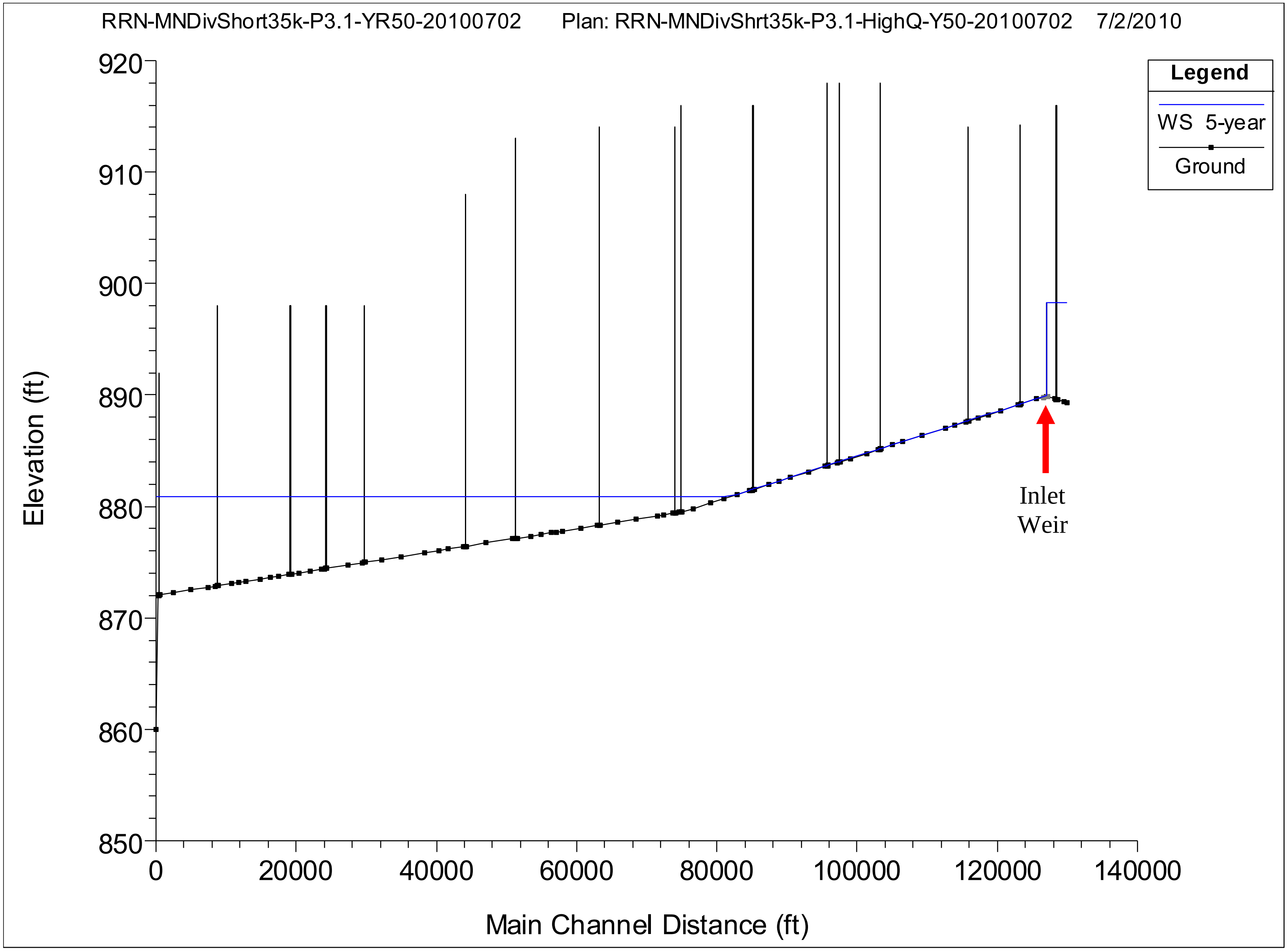


Figure F-D76 FCP Diversion Channel Water Surface Elevation Profile for the 5-Year Event for Year 50 Hydrology

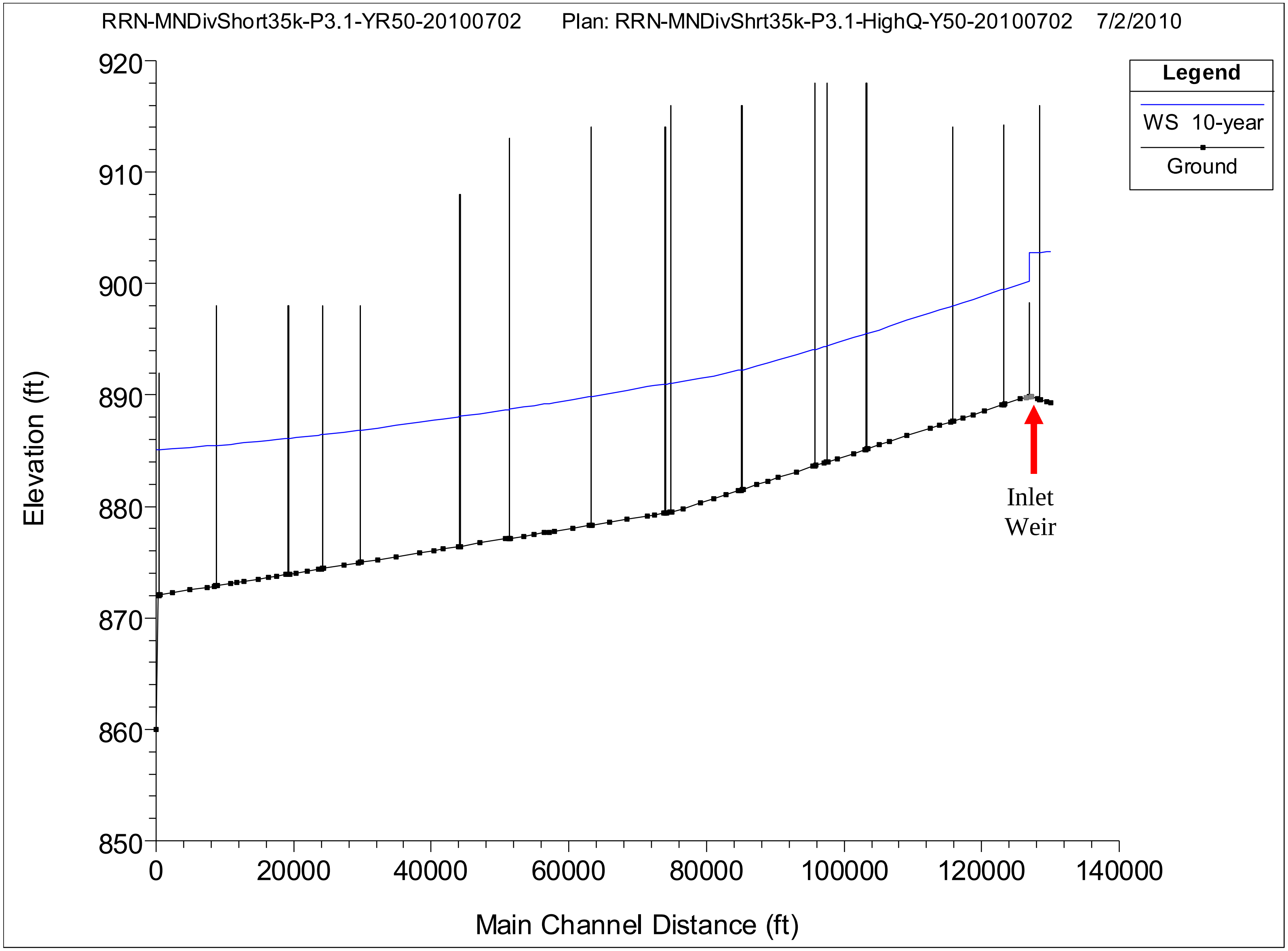


Figure F-D77 FCP Diversion Channel Water Surface Elevation Profile for the 10-Year Event for Year 50 Hydrology

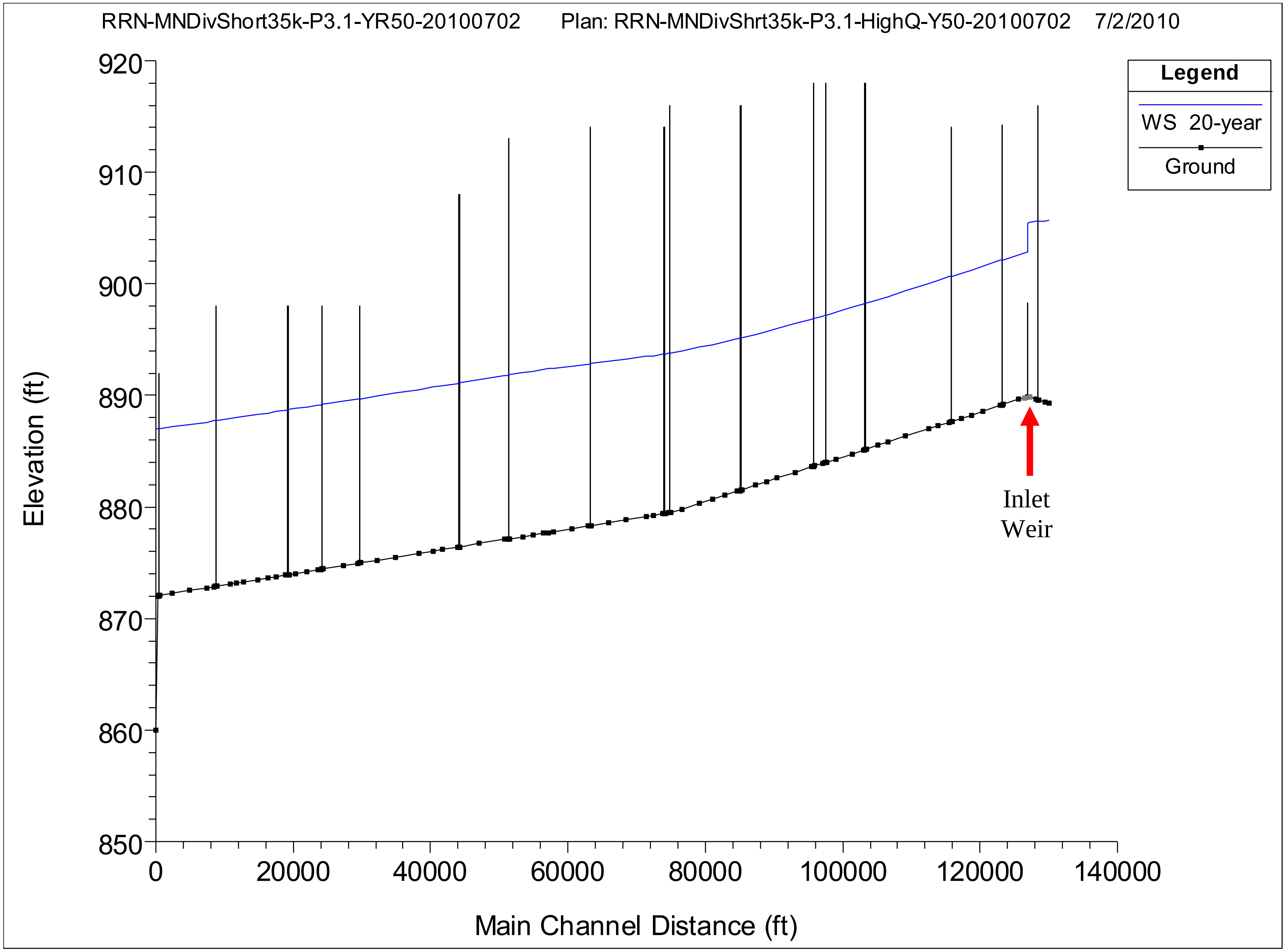


Figure F-D78 FCP Diversion Channel Water Surface Elevation Profile for the 20-Year Event for Year 50 Hydrology

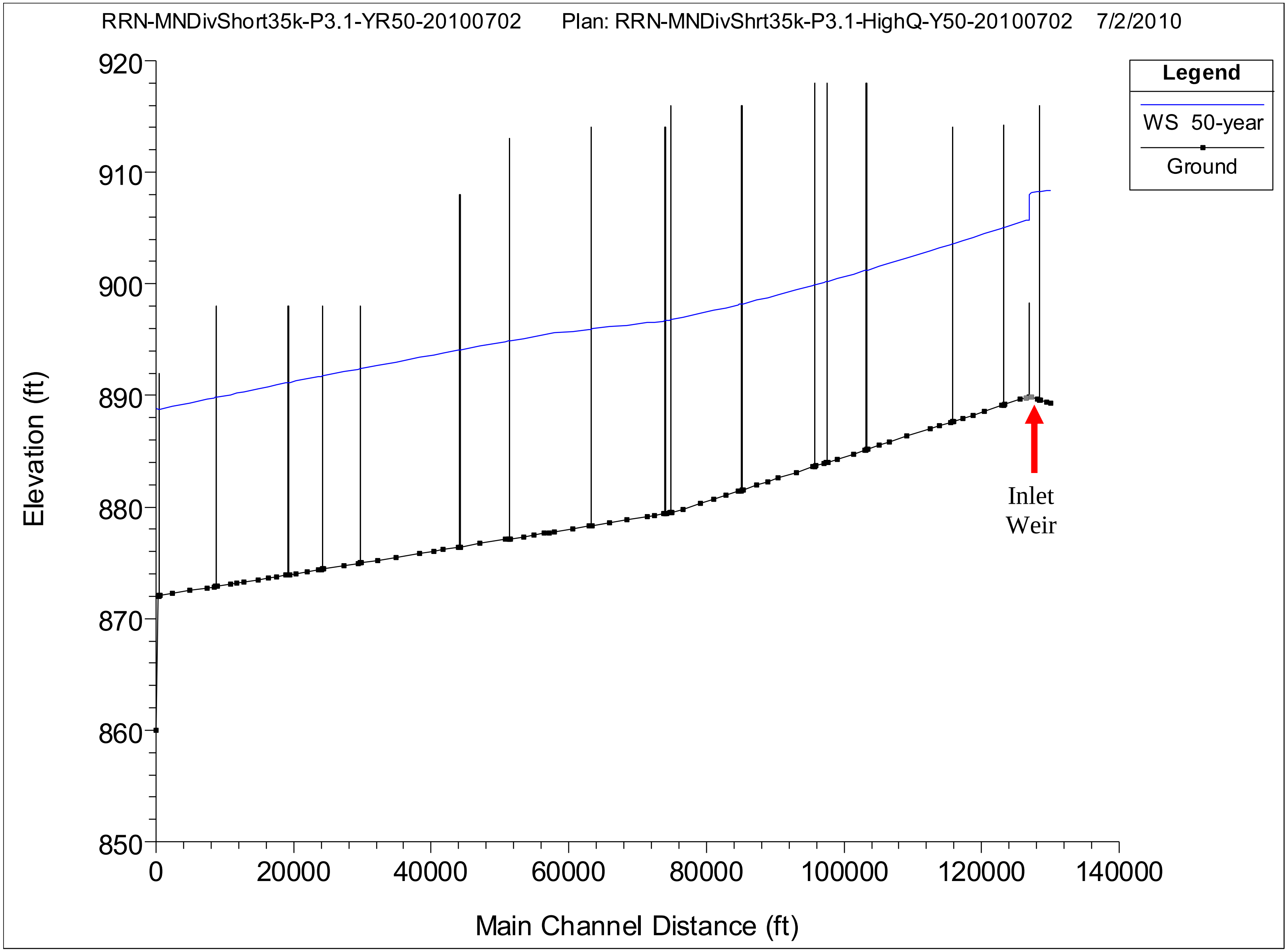


Figure F-D79 FCP Diversion Channel Water Surface Elevation Profile for the 50-Year Event for Year 50 Hydrology

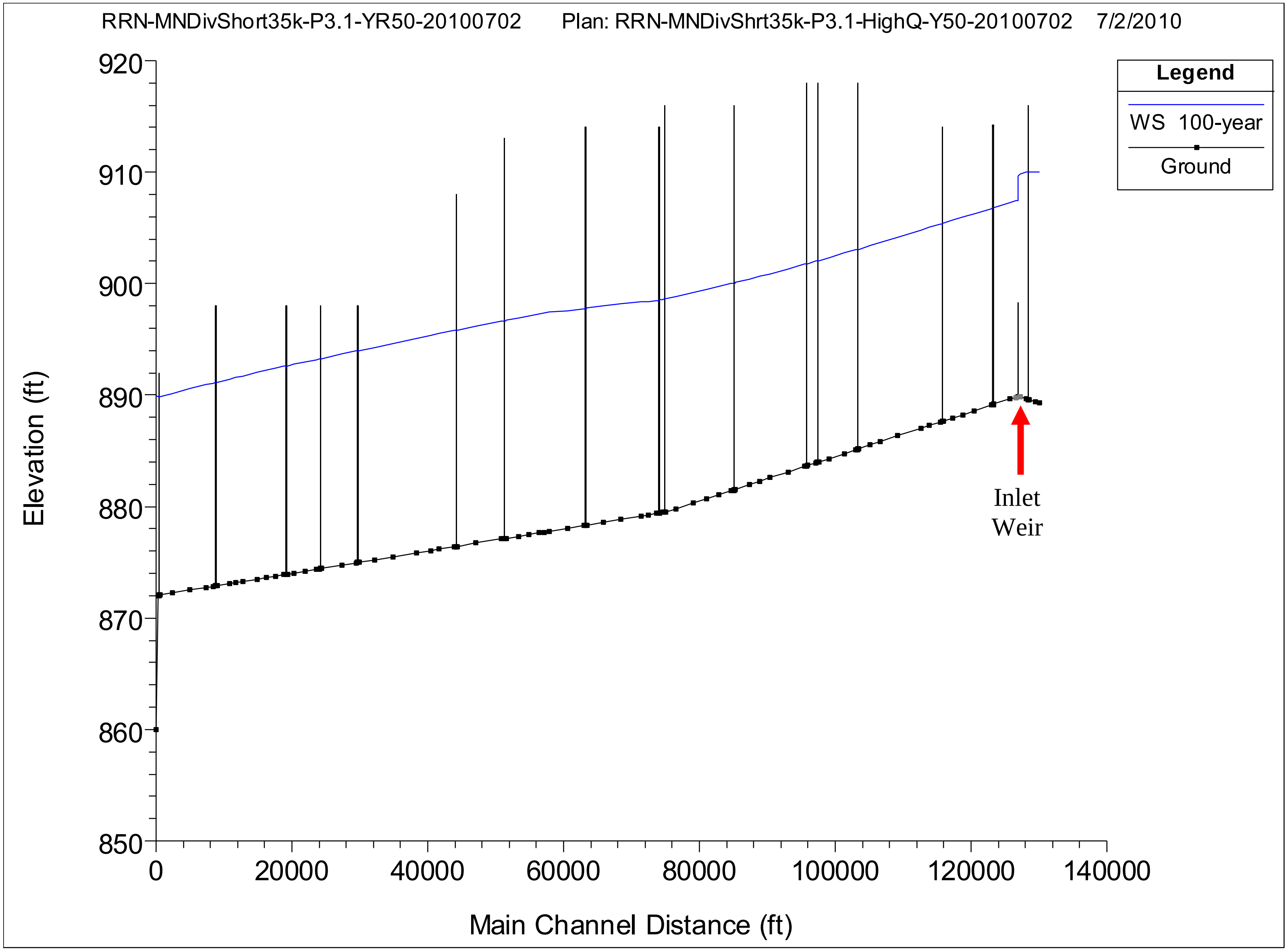


Figure F-D80 FCP Diversion Channel Water Surface Elevation Profile for the 100-Year Event for Year 50 Hydrology

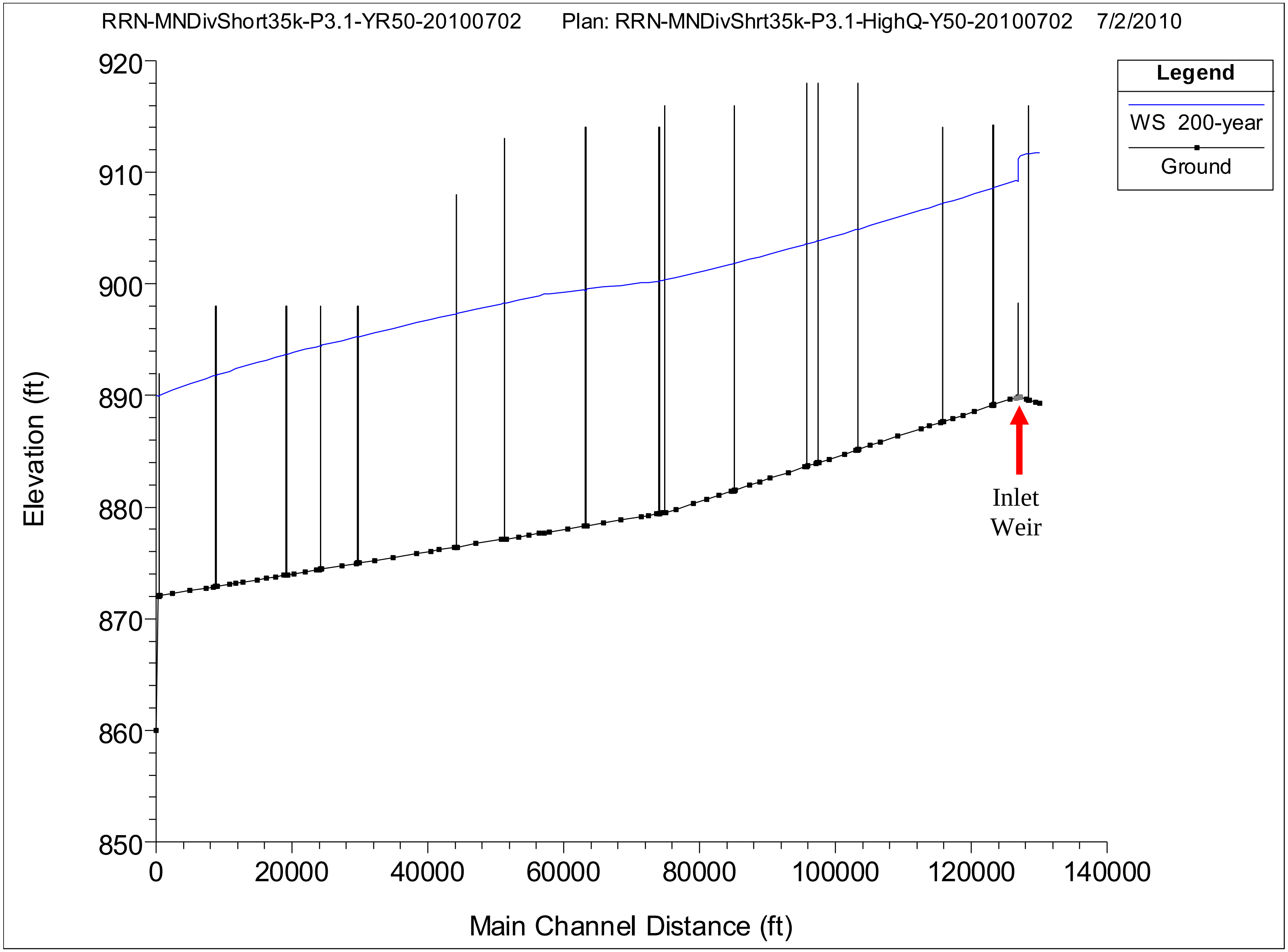


Figure F-D81 FCP Diversion Channel Water Surface Elevation Profile for the 200-Year Event for Year 50 Hydrology

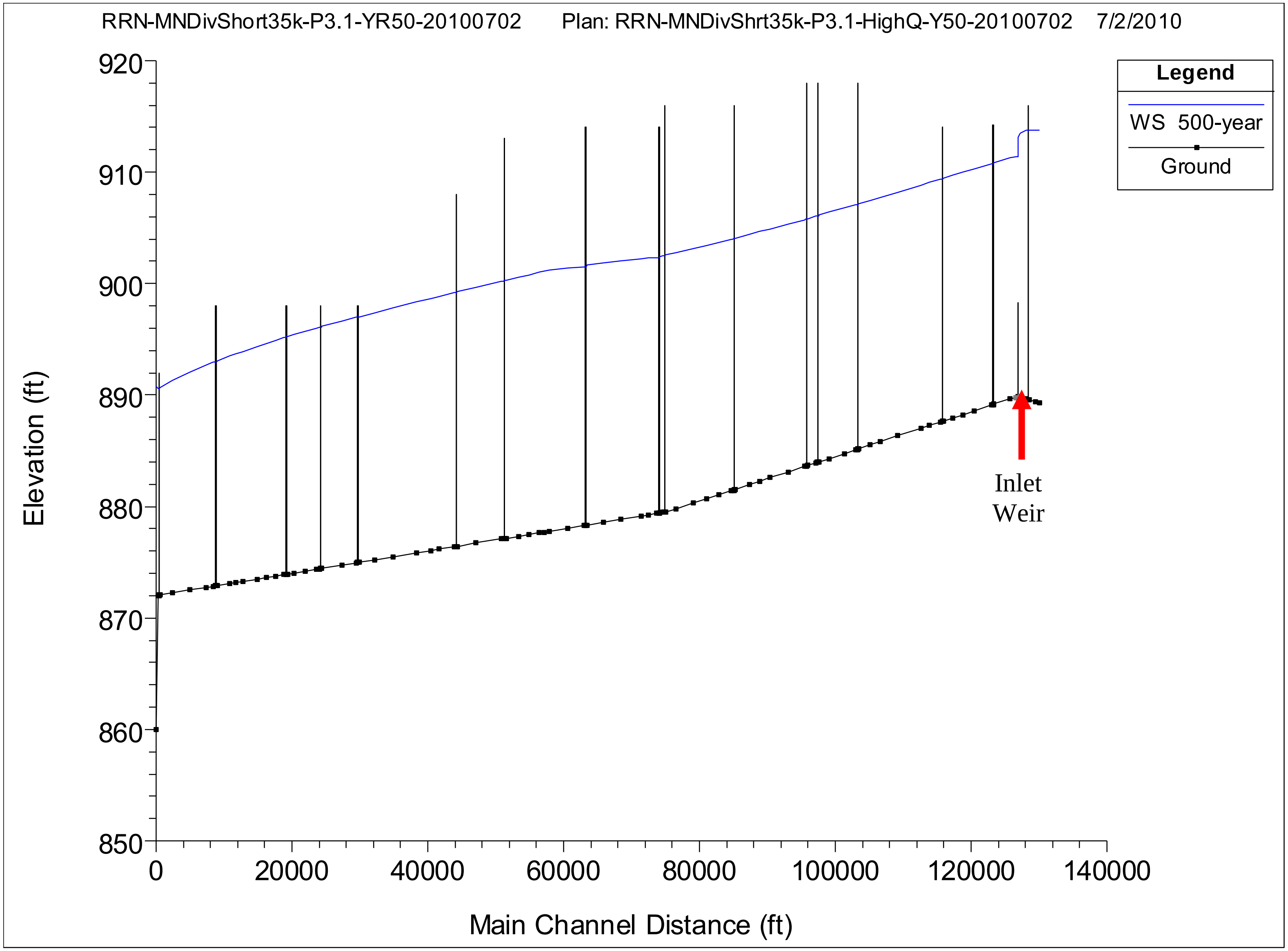


Figure F-D82 **FCP Diversion Channel Water Surface Elevation Profile for the 500-Year Event for Year 50 Hydrology**

F-D3.2 WATER SURFACE ELEVATION PROFILES ALONG THE LPP DIVERSION CHANNEL

The maximum water surface elevation profiles along the LPP Diversion Channel for the 10-, 50-, 100-, and 500-year events are presented in Figures F-D83, F-D88, F-D93, and F-D98. The location of the Inlet Weir (primary Inlet Structure to the Diversion Channel located to the west of the Wild Rice River), Sheyenne River crossing structure, Maple River crossing structure, and Outlet structure are noted on the figures. Figures F-D84 to F-D87, F-D89 to F-D92, F-D94 to F-D97, and F-D99 to F-D102 include stage and flow hydrographs over the duration of the 10, 50-, 100-, and 500-year events in four locations in the Diversion Channel listed below.

1. Upstream of the Inlet Weir (RS 152527)
2. Between the Sheyenne River and Maple River (RS 83654)
3. Between the Maple River and the Lower Rush River (RS 64696)
4. Between the Rush River and the Diversion Outlet (RS 29253)

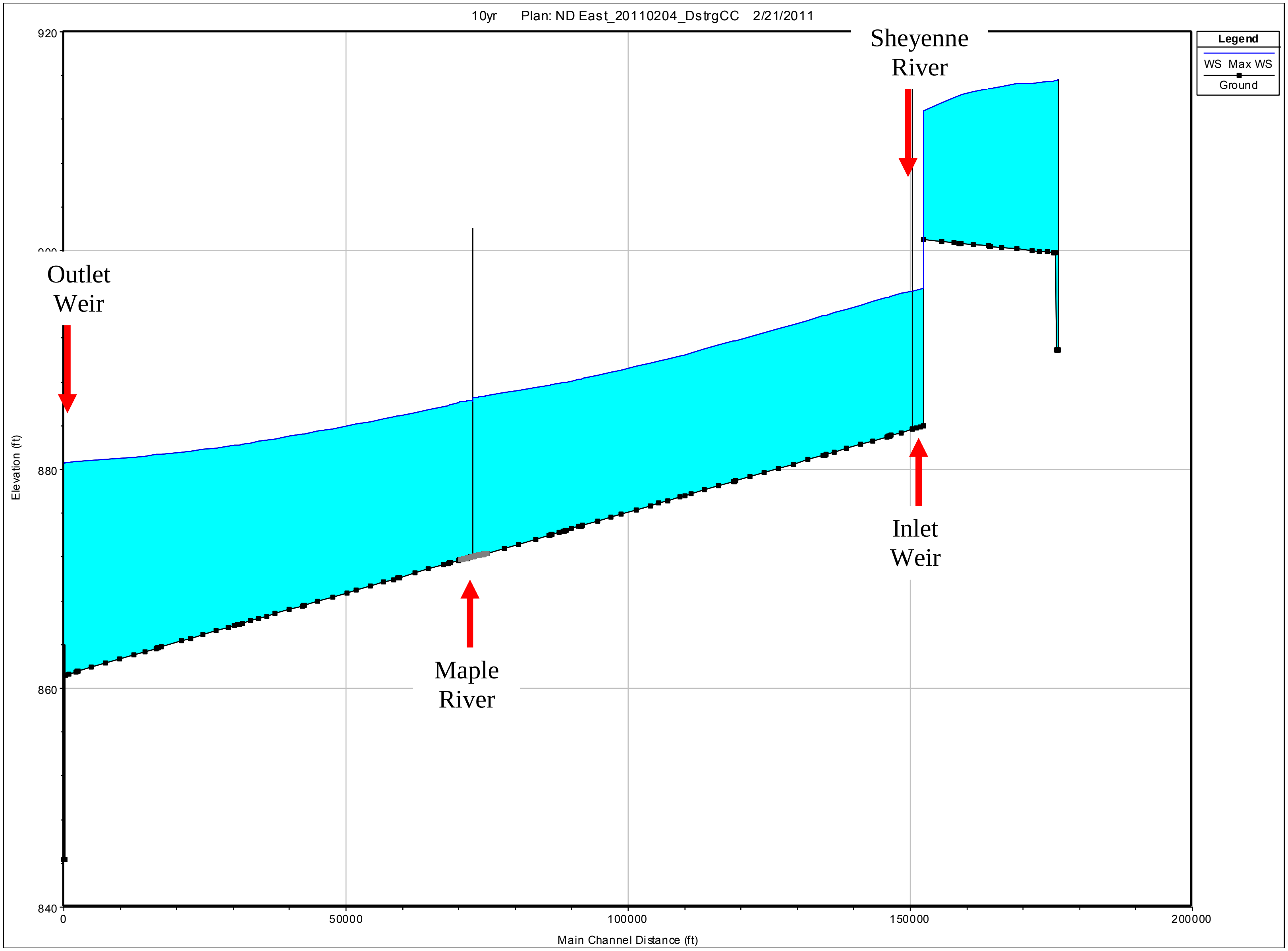


Figure F-D83 LPP Diversion Channel Water Surface Elevation Profile for the 10-Year Event

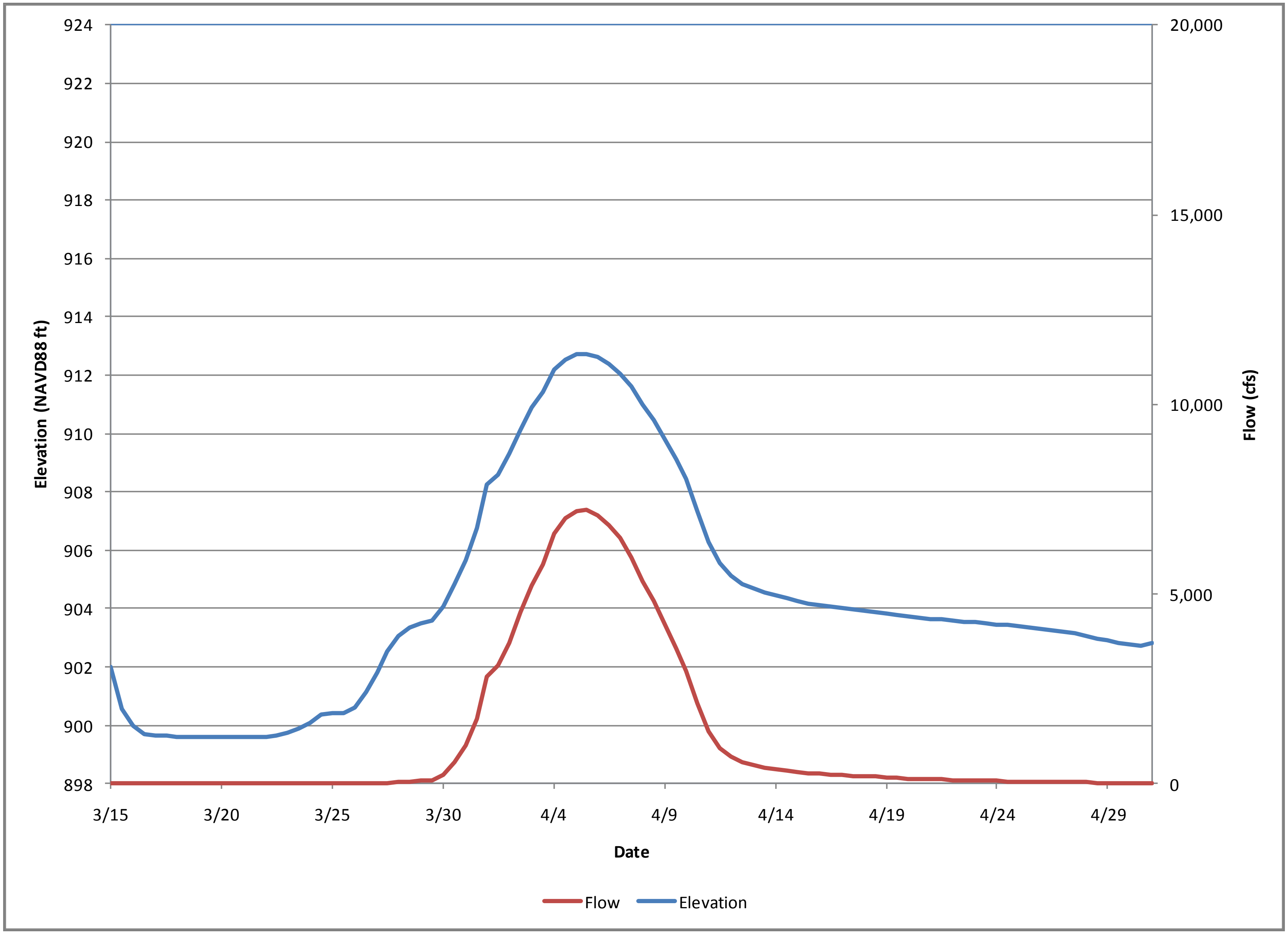


Figure F-D84 Hydrograph Upstream of the Diversion Inlet Structure (RS 152527) for the 10-Year Event

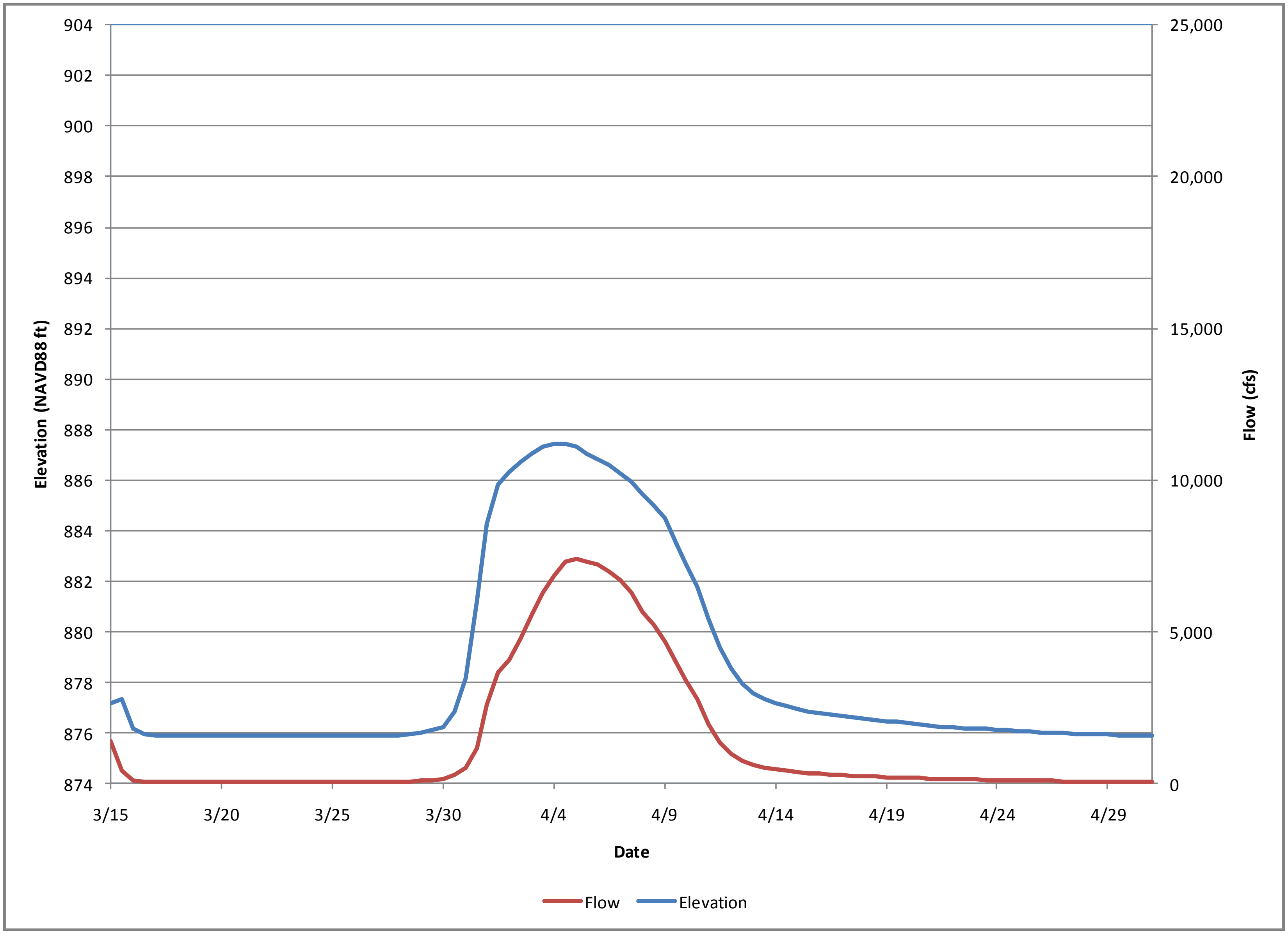


Figure F-D85 Hydrograph between the Sheyenne River and Maple River (RS 83654) for the 10-Year Event

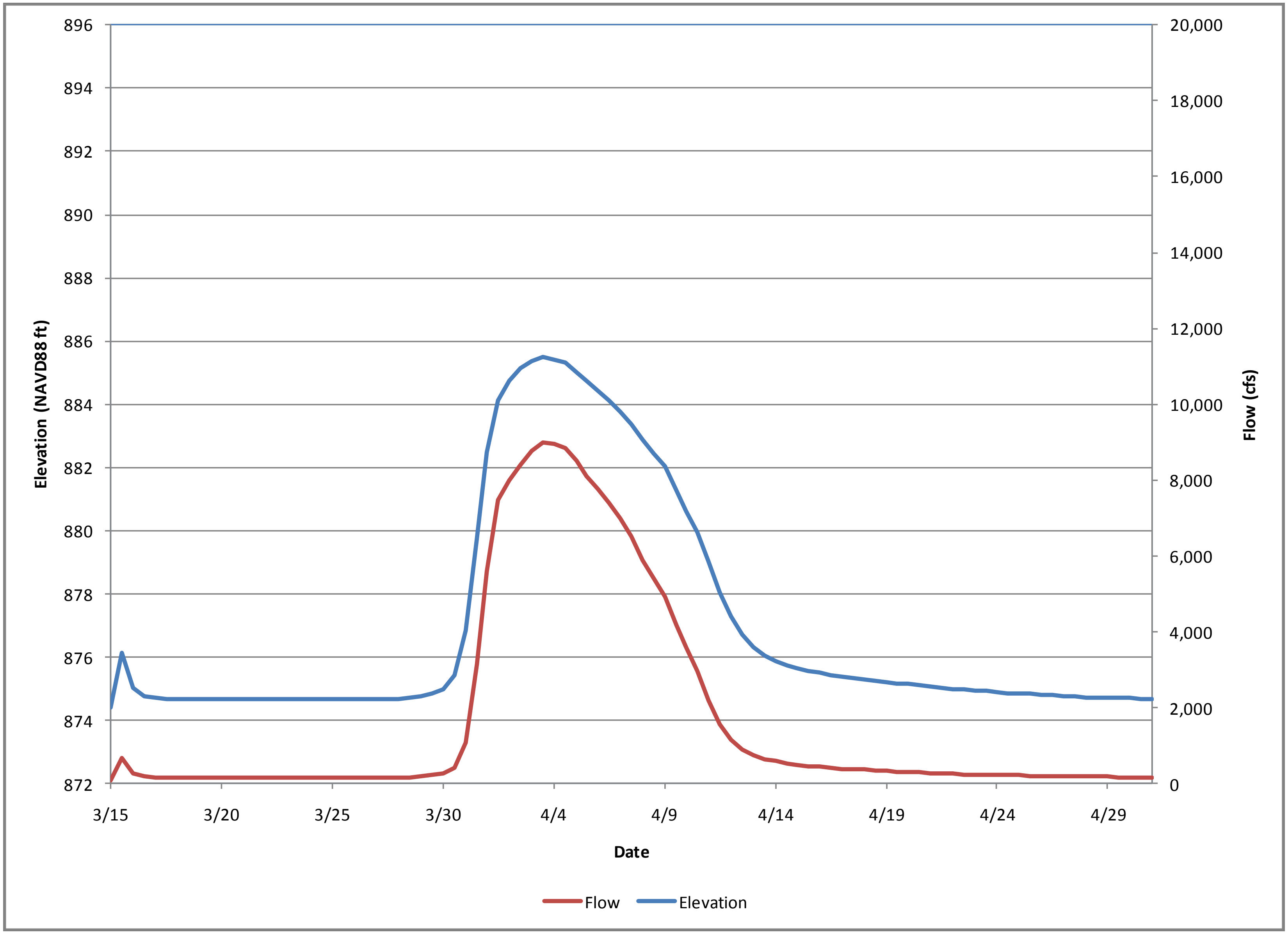


Figure F-D86 Hydrograph between the Maple River and Lower Rush River (RS 64696) for the 10-Year Event

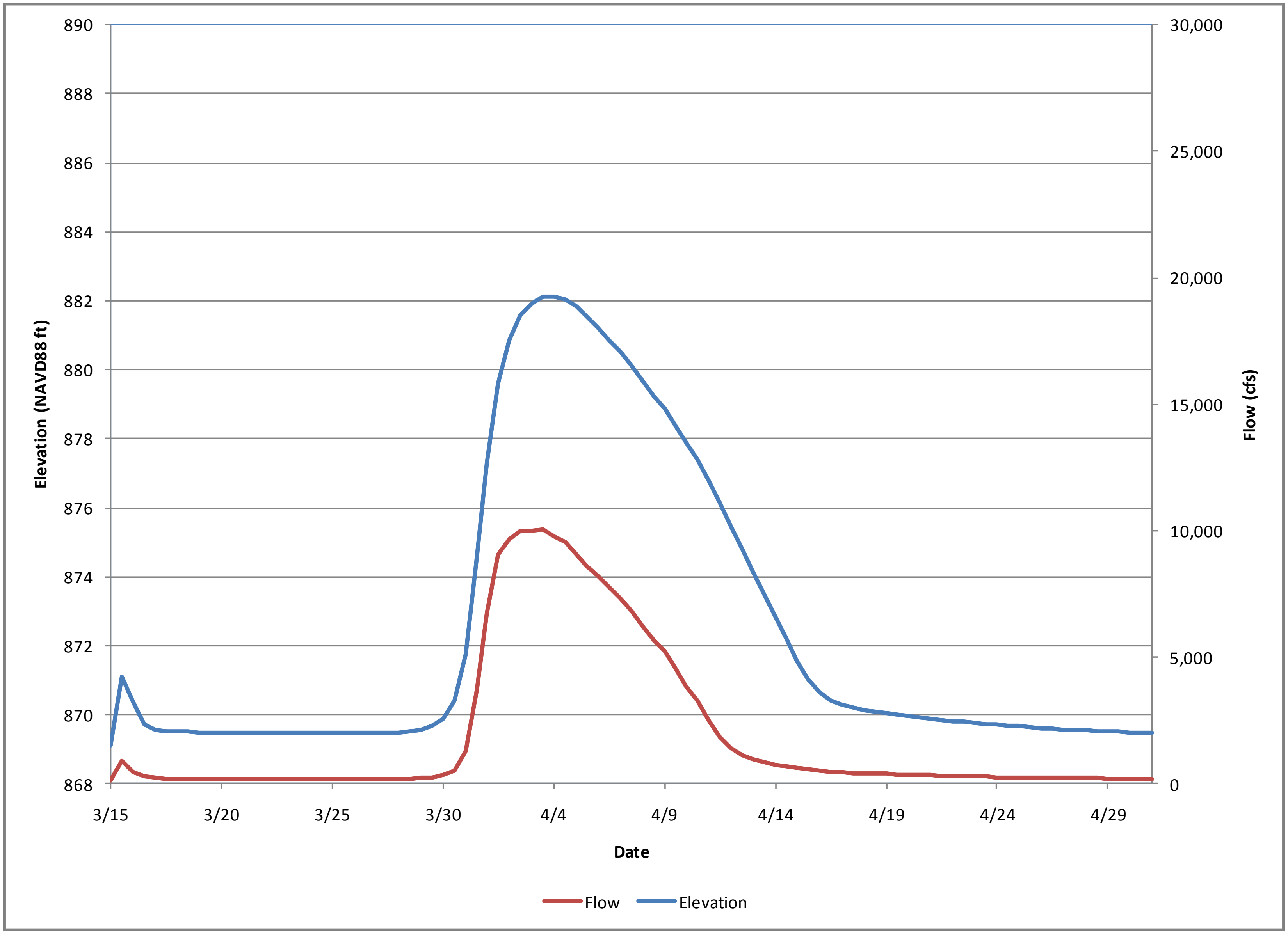


Figure F-D87 Hydrograph between the Rush River and the Outlet Structure (RS 29253) for the 10-Year Event

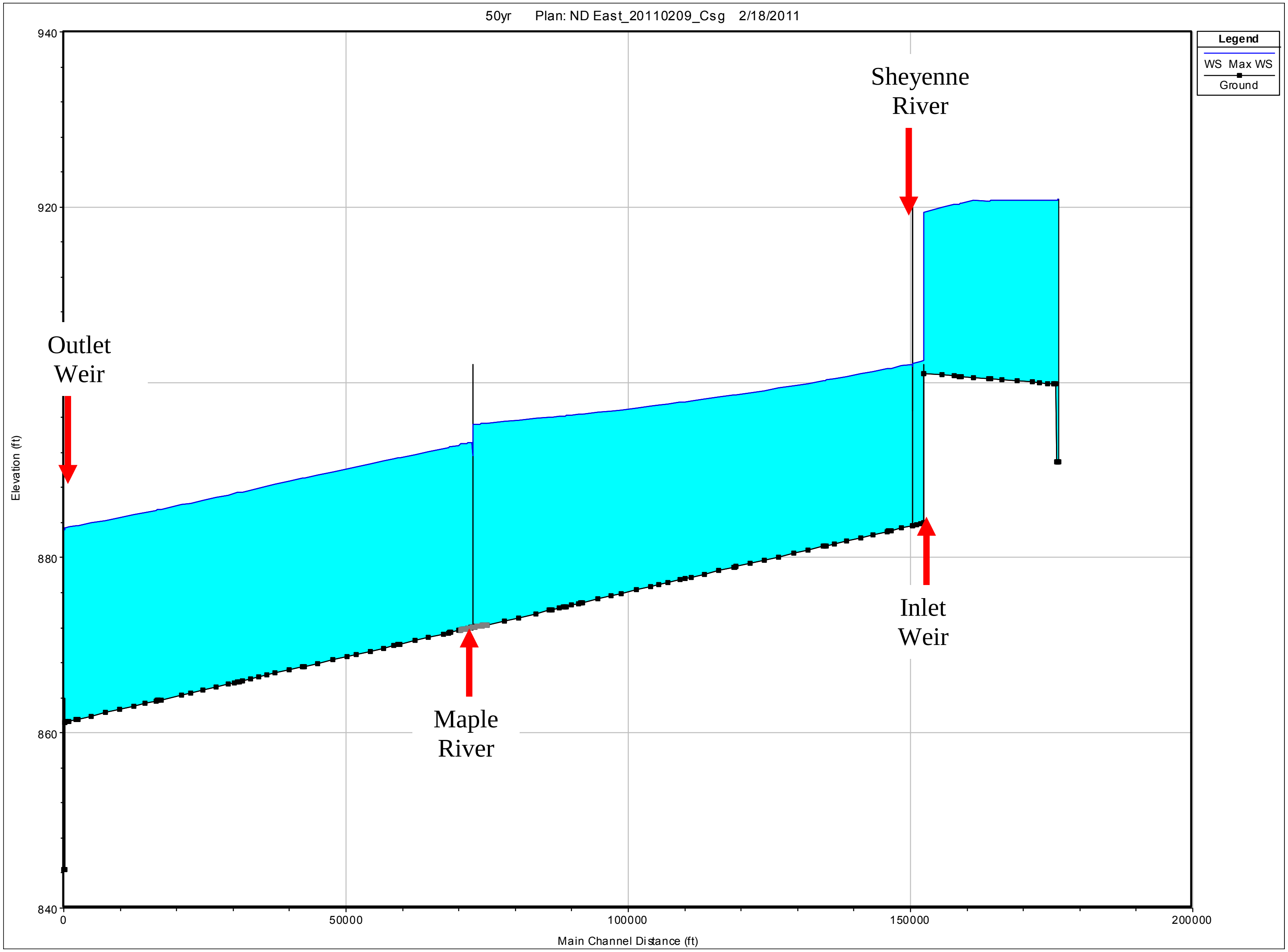


Figure F-D88 LPP Diversion Channel Water Surface Elevation Profile for the 50-Year Event

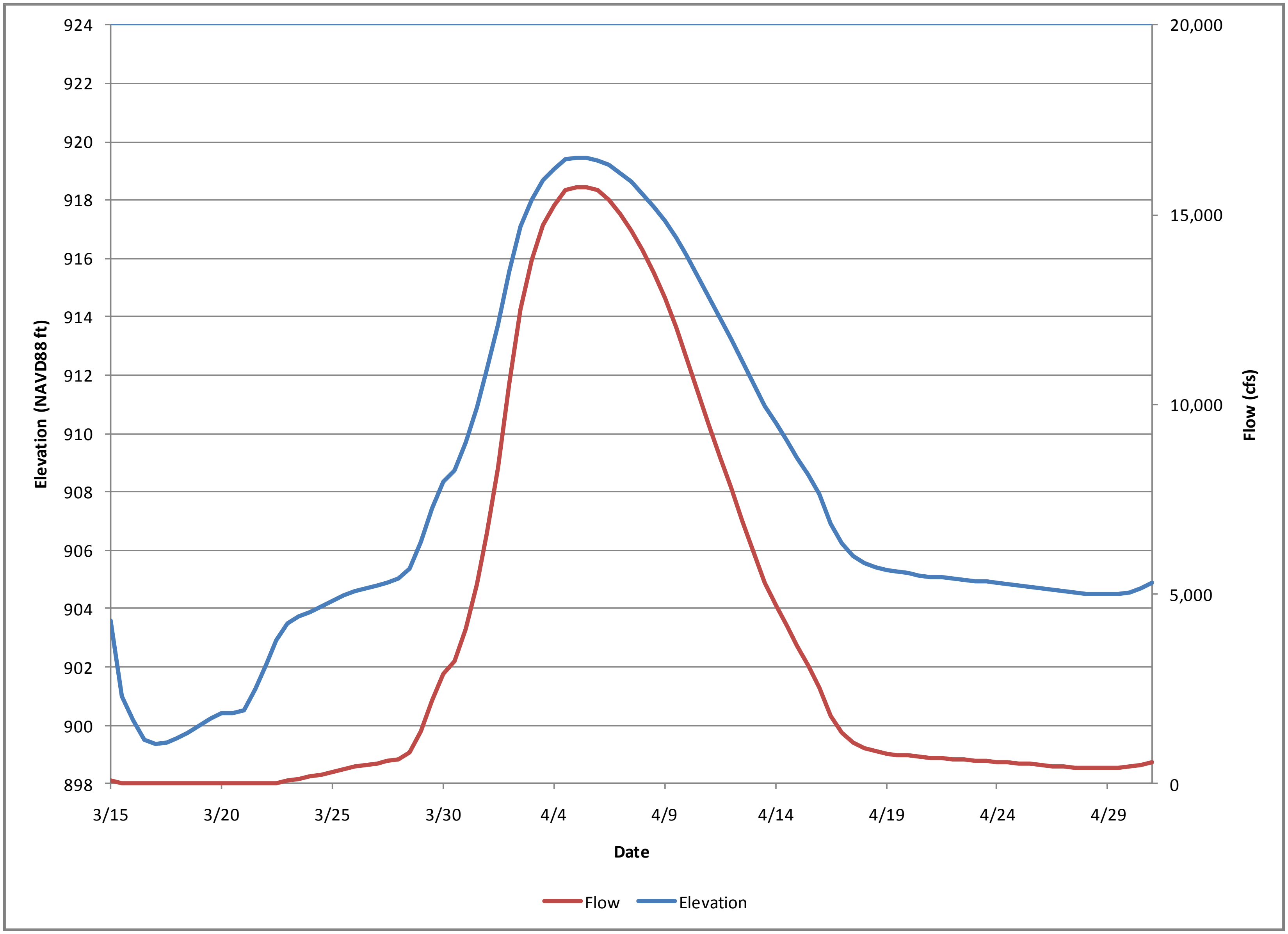


Figure F-D89 Hydrograph Upstream of the Diversion Inlet Structure (RS 152527) for the 50-Year Event

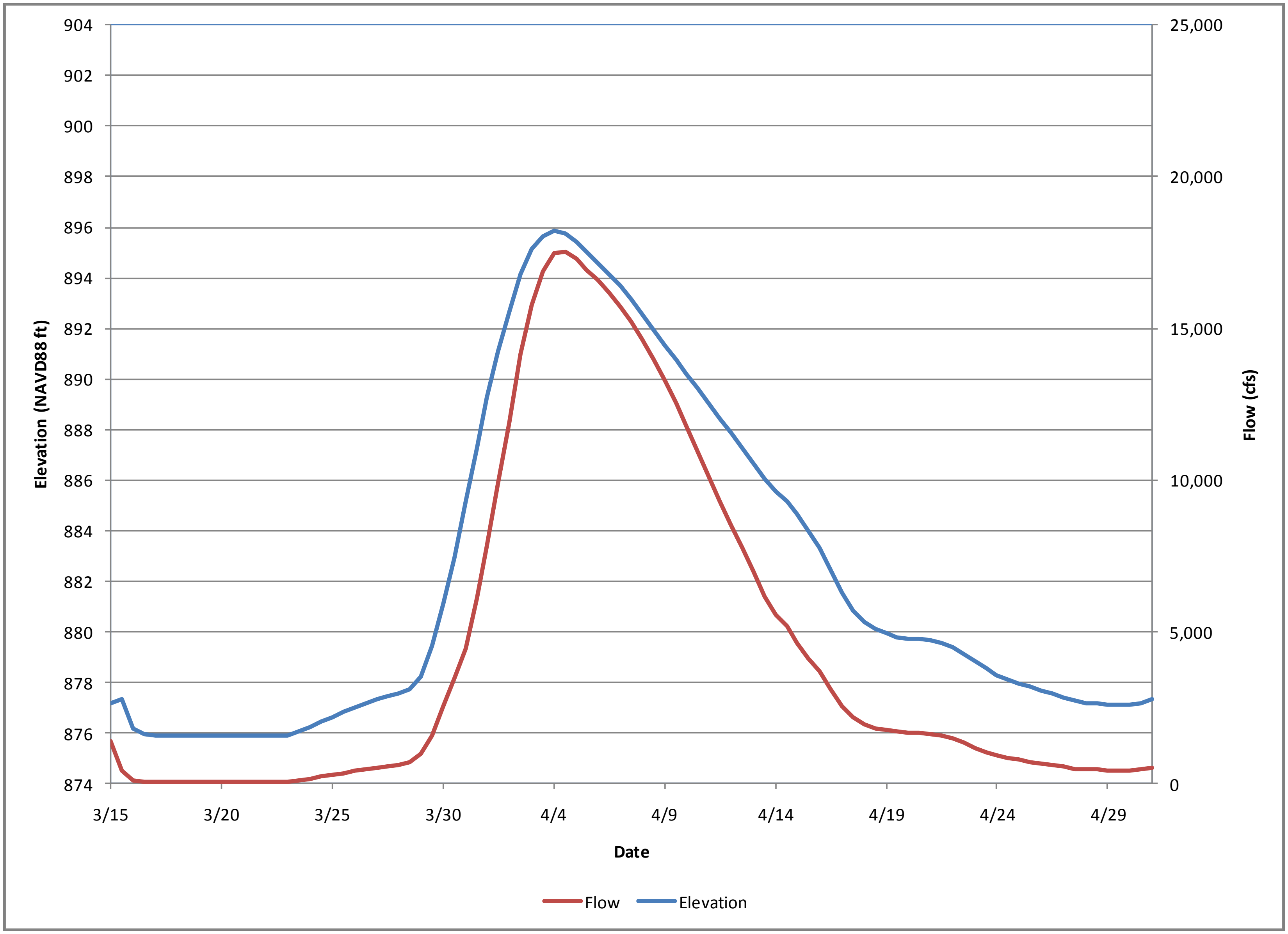


Figure F-D90 Hydrograph between the Sheyenne River and Maple River (RS 83654) for the 50-Year Event

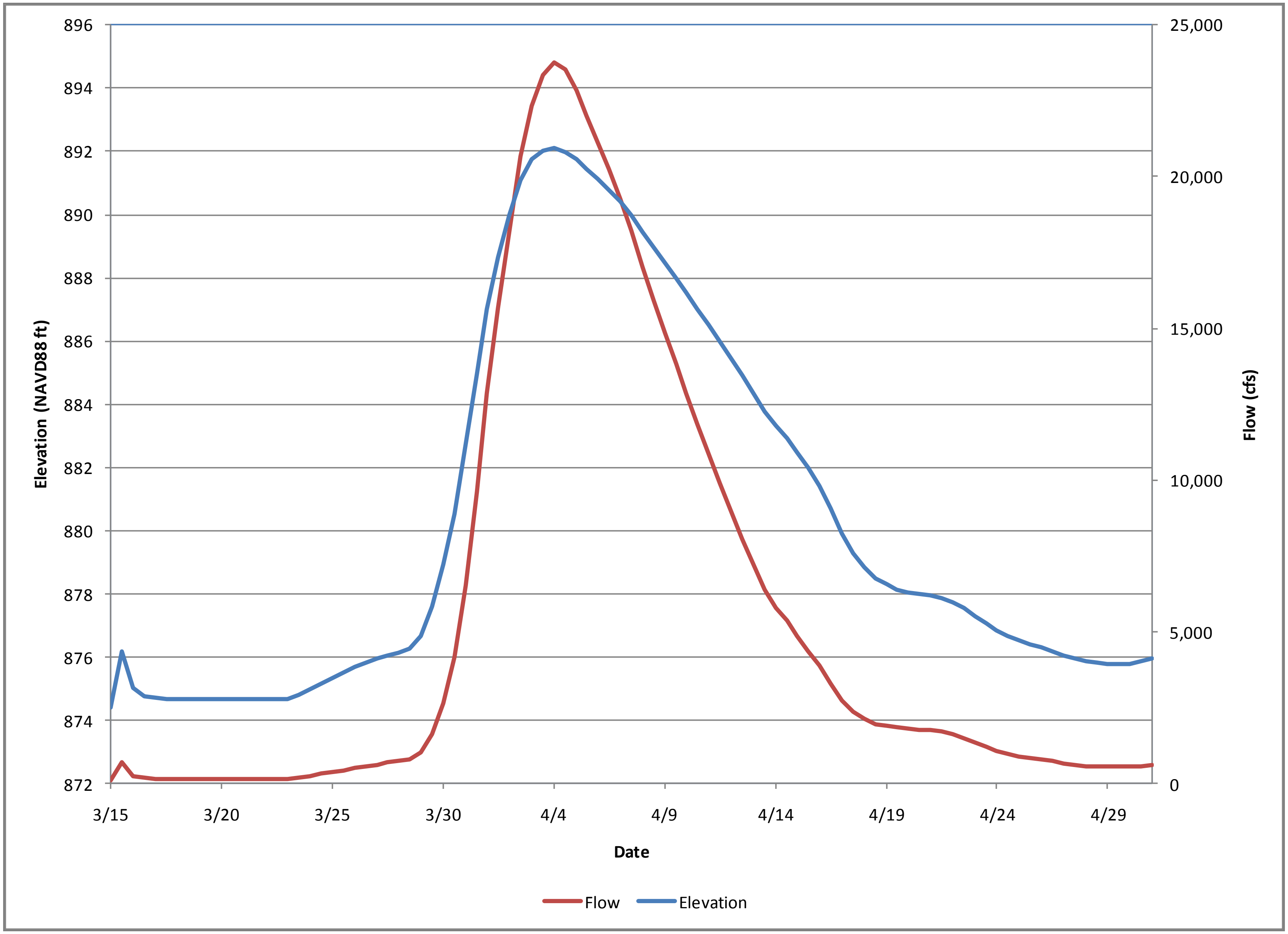


Figure F-D91 Hydrograph between the Maple River and Lower Rush River (RS 64696) for the 50-Year Event

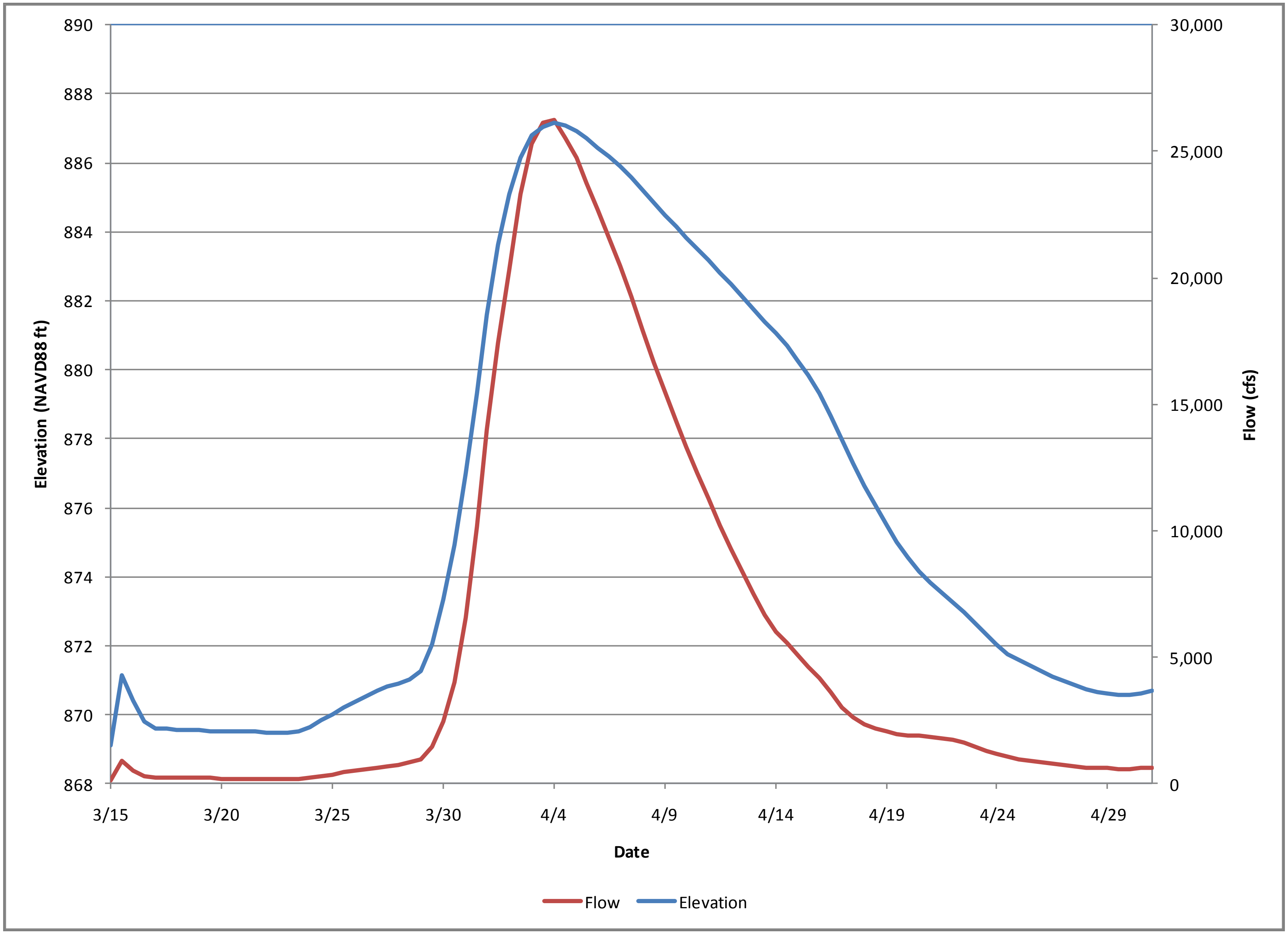


Figure F-D92 Hydrograph between the Rush River and the Outlet Structure (RS 29253) for the 50-Year Event

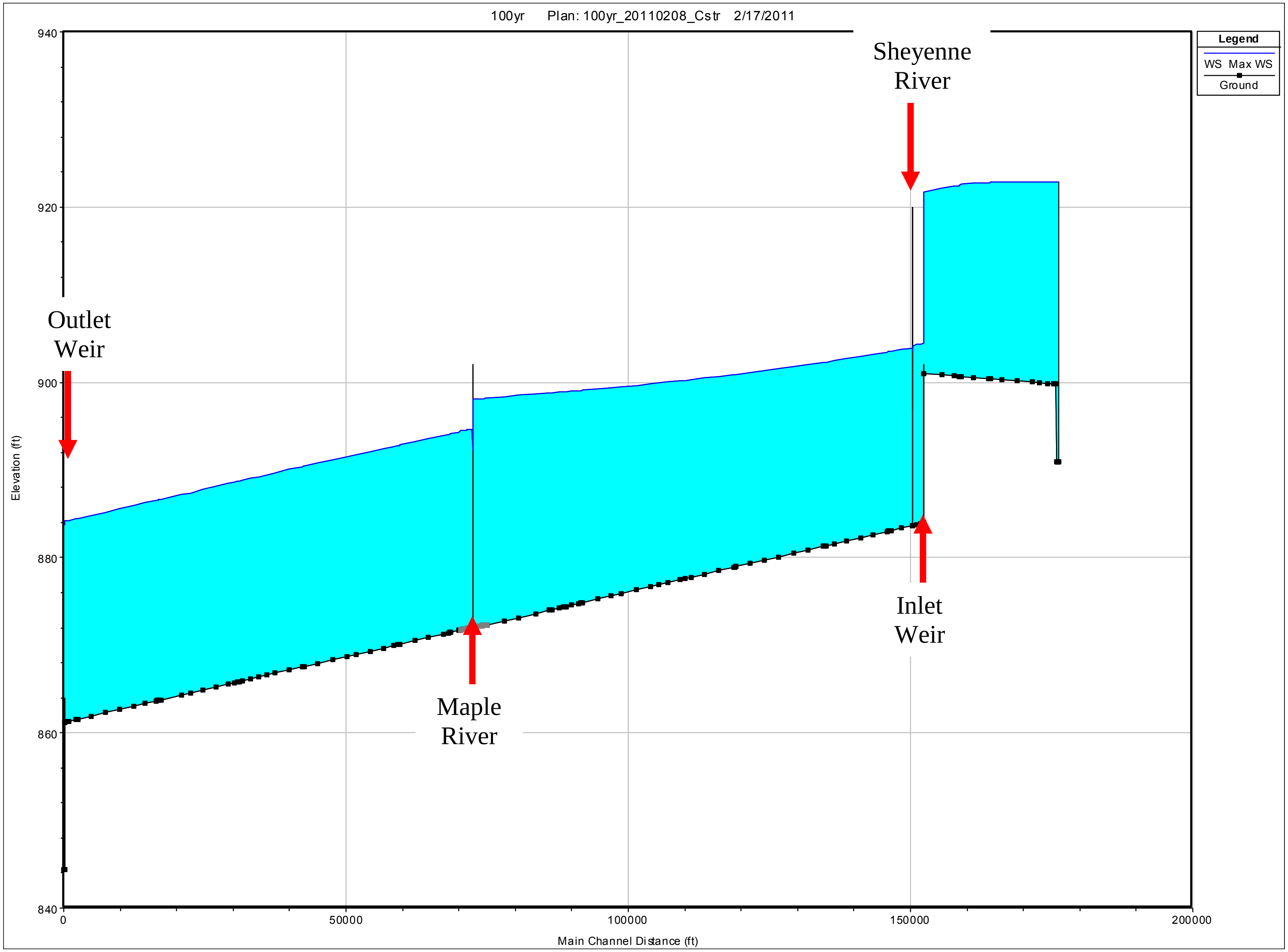


Figure F-D93 LPP Diversion Channel Water Surface Elevation Profile for the 100-Year Event

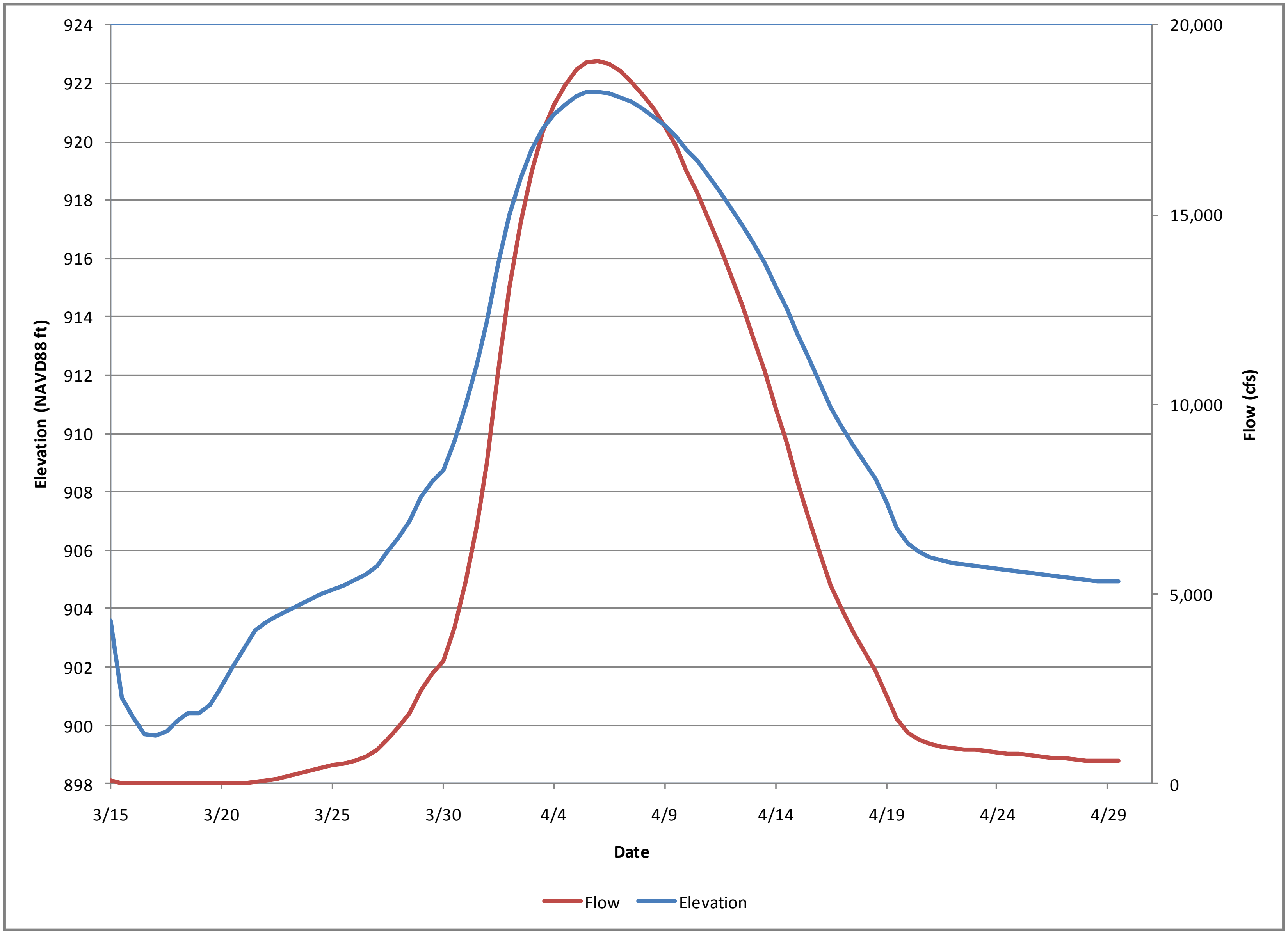


Figure F-D94 Hydrograph Upstream of the Diversion Inlet Structure (RS 152527) for the 100-Year Event

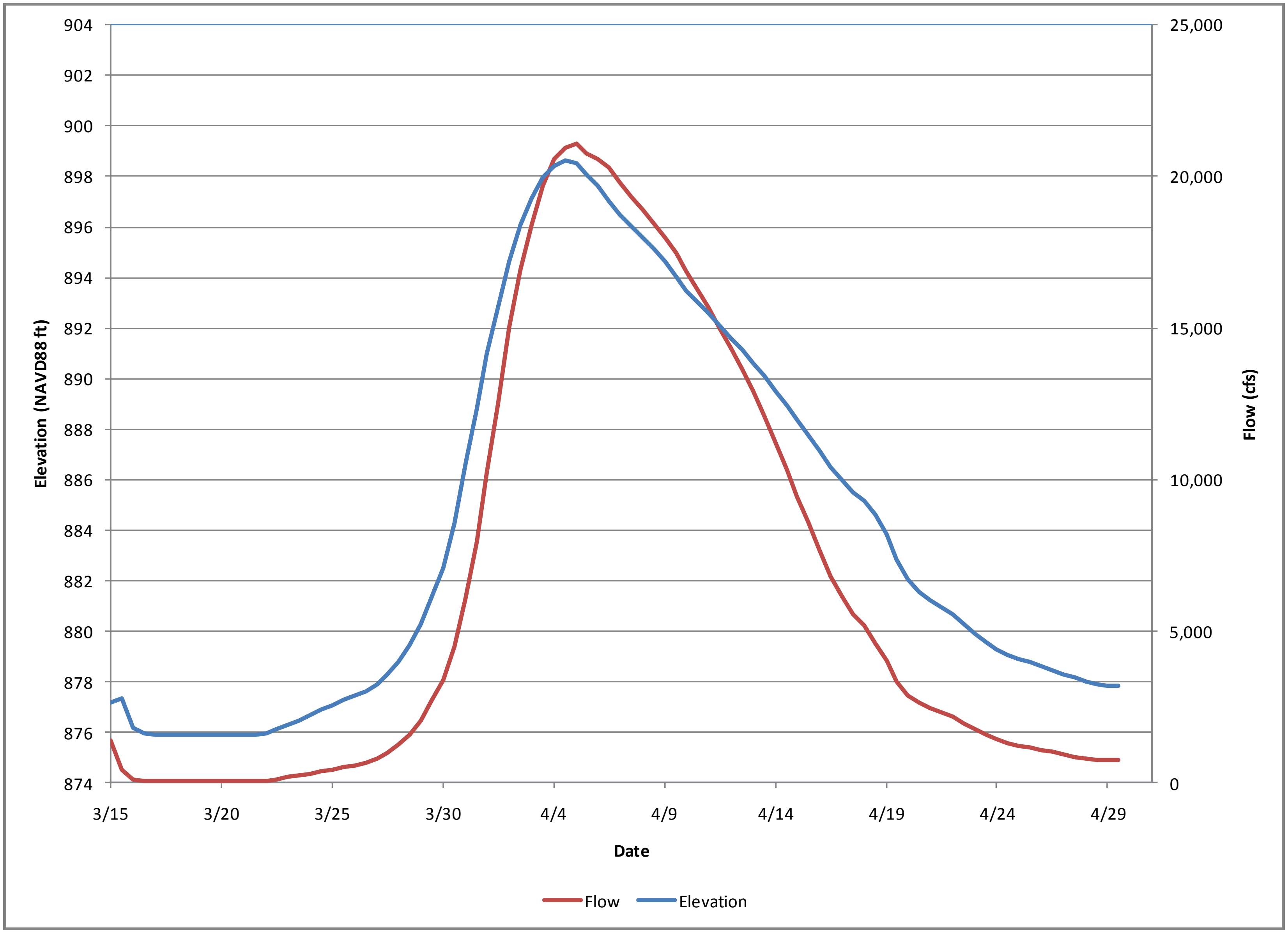


Figure F-D95 Hydrograph between the Sheyenne River and Maple River (RS 83654) for the 100-Year Event

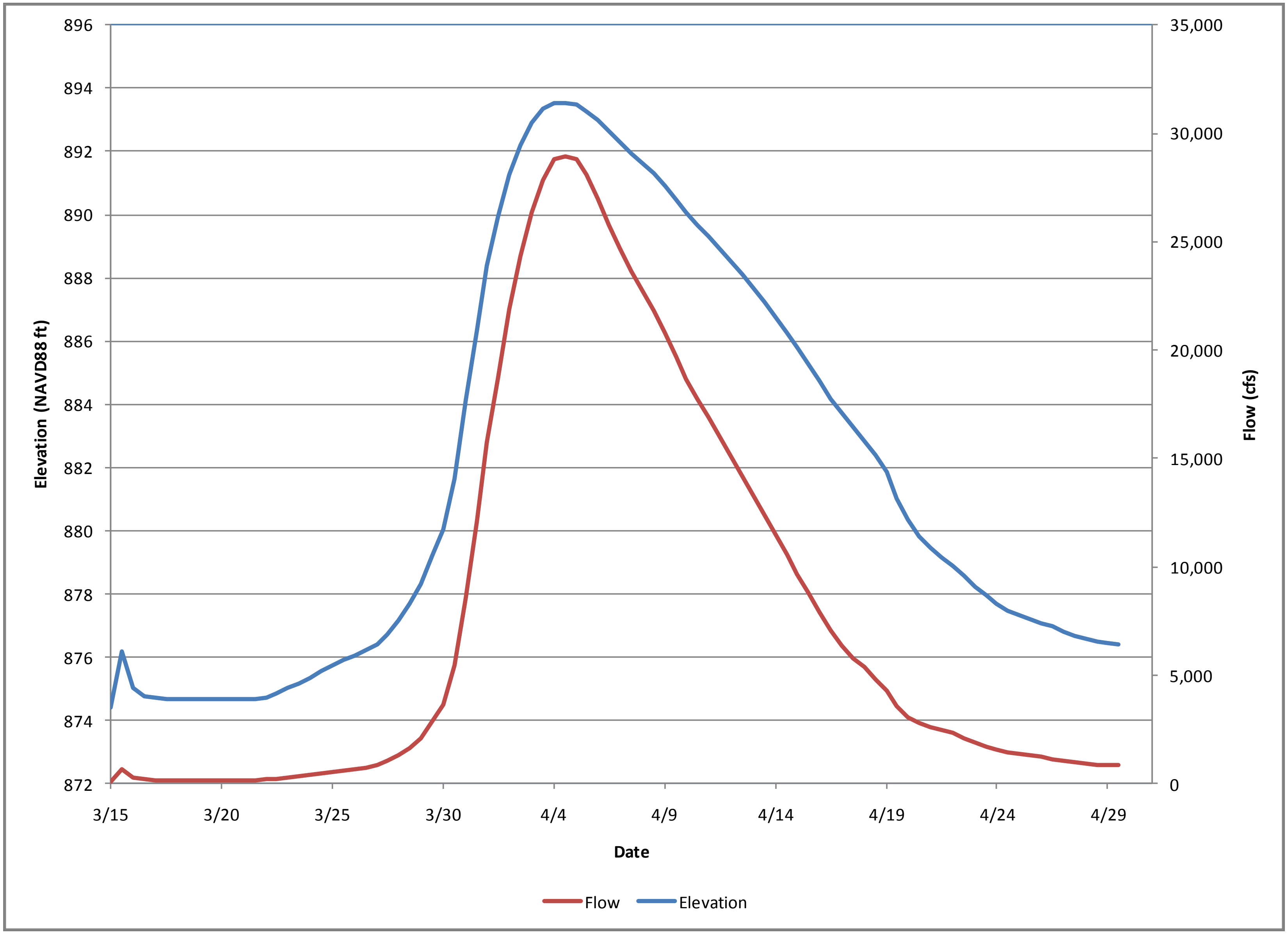


Figure F-D96 Hydrograph between the Maple River and Lower Rush River (RS 64696) for the 100-Year Event

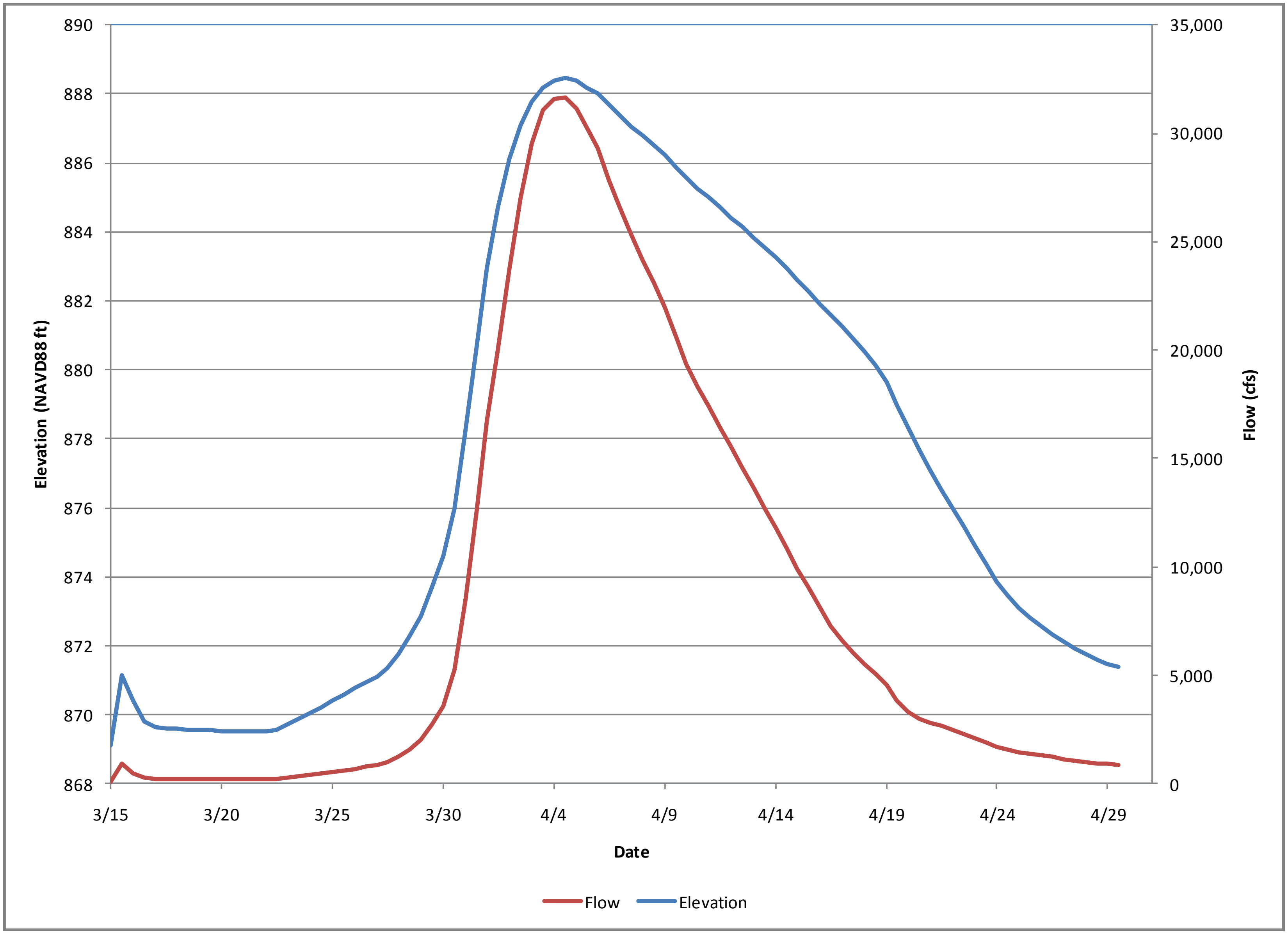


Figure F-D97 Hydrograph between the Rush River and the Outlet Structure (RS 29253) for the 100-Year Event

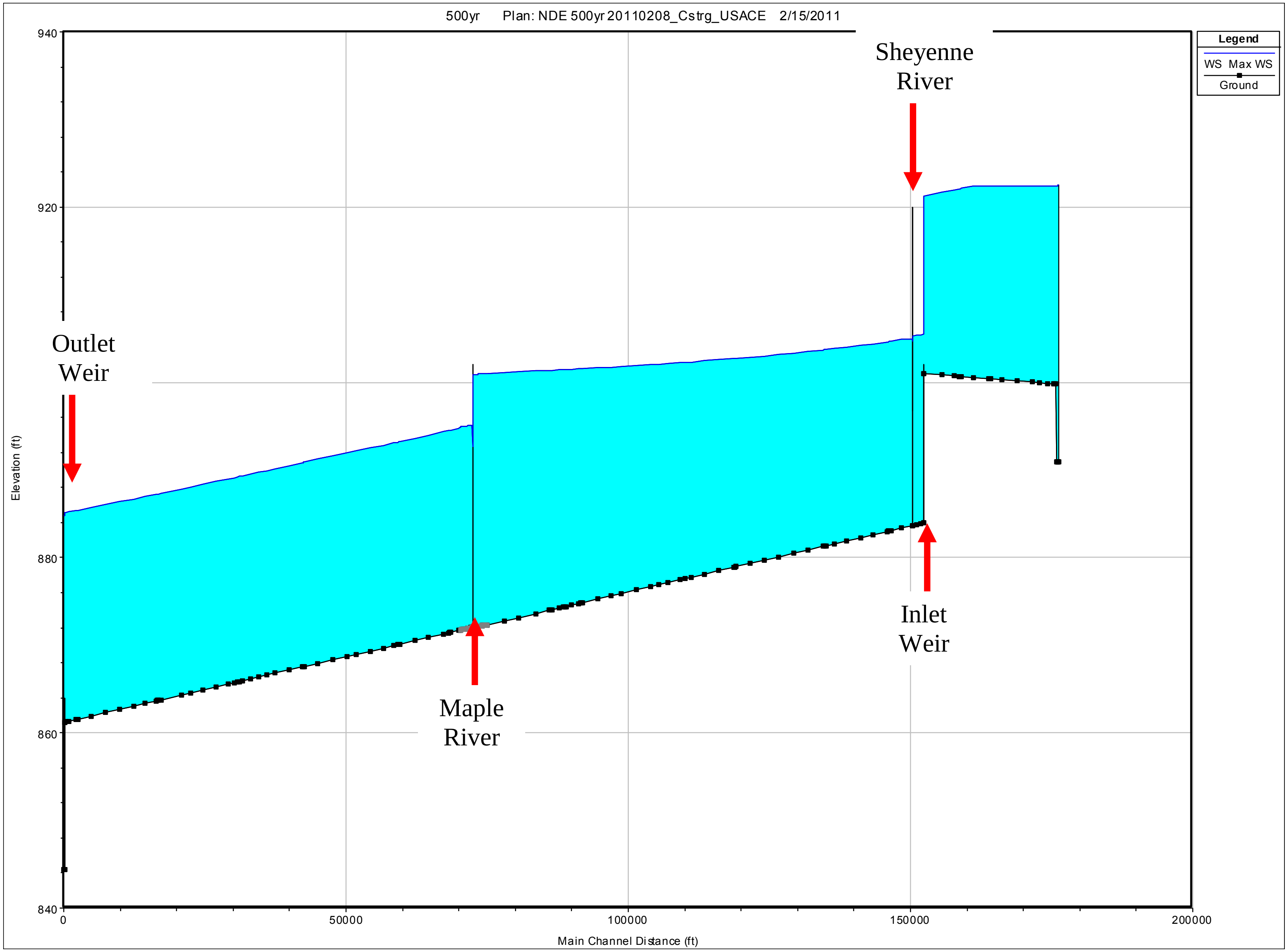


Figure F-D98 LPP Diversion Channel Water Surface Elevation Profile for the 500-Year Event

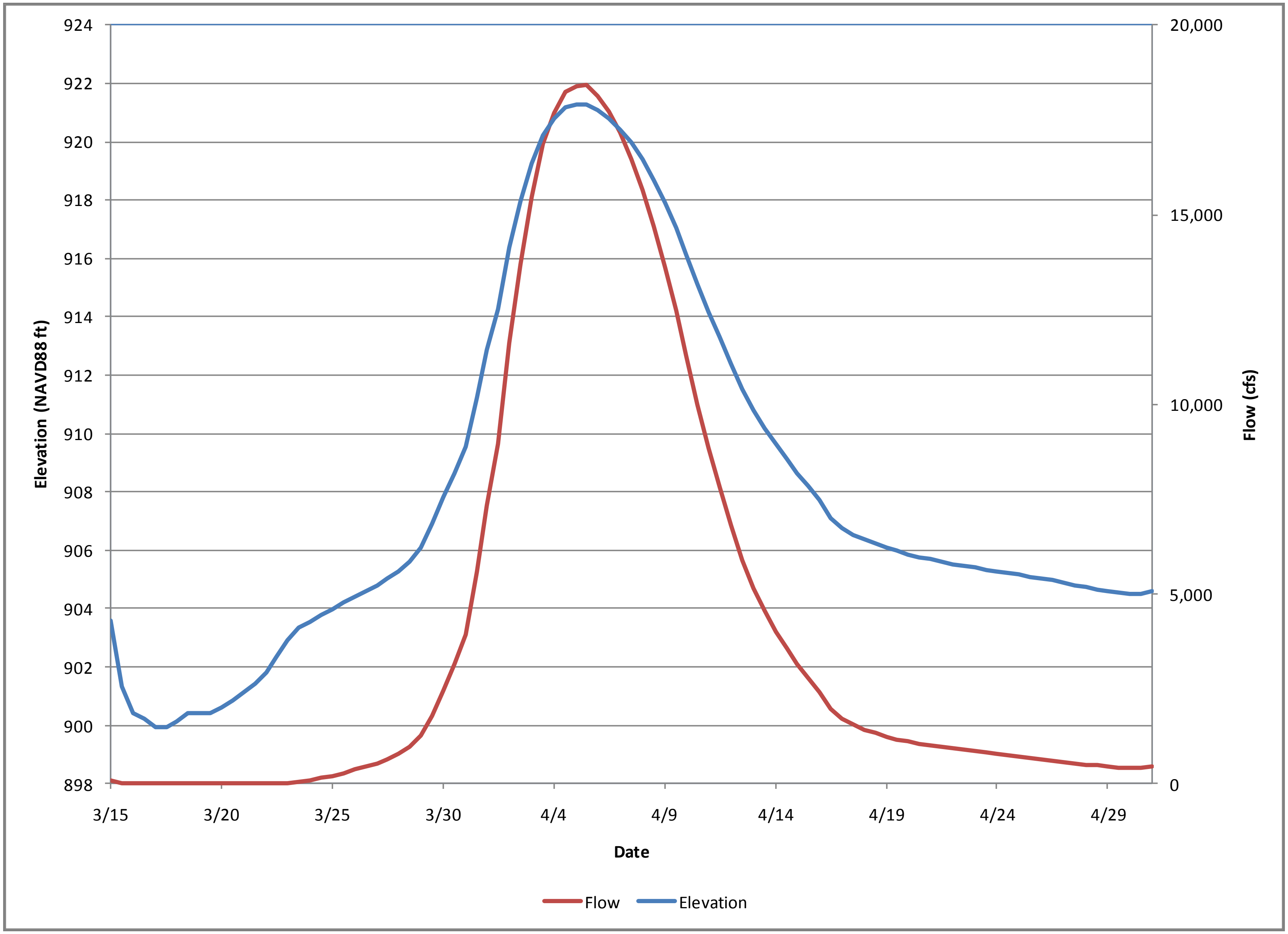


Figure F-D99 Hydrograph Upstream of the Diversion Inlet Structure (RS 152527) for the 500-Year Event

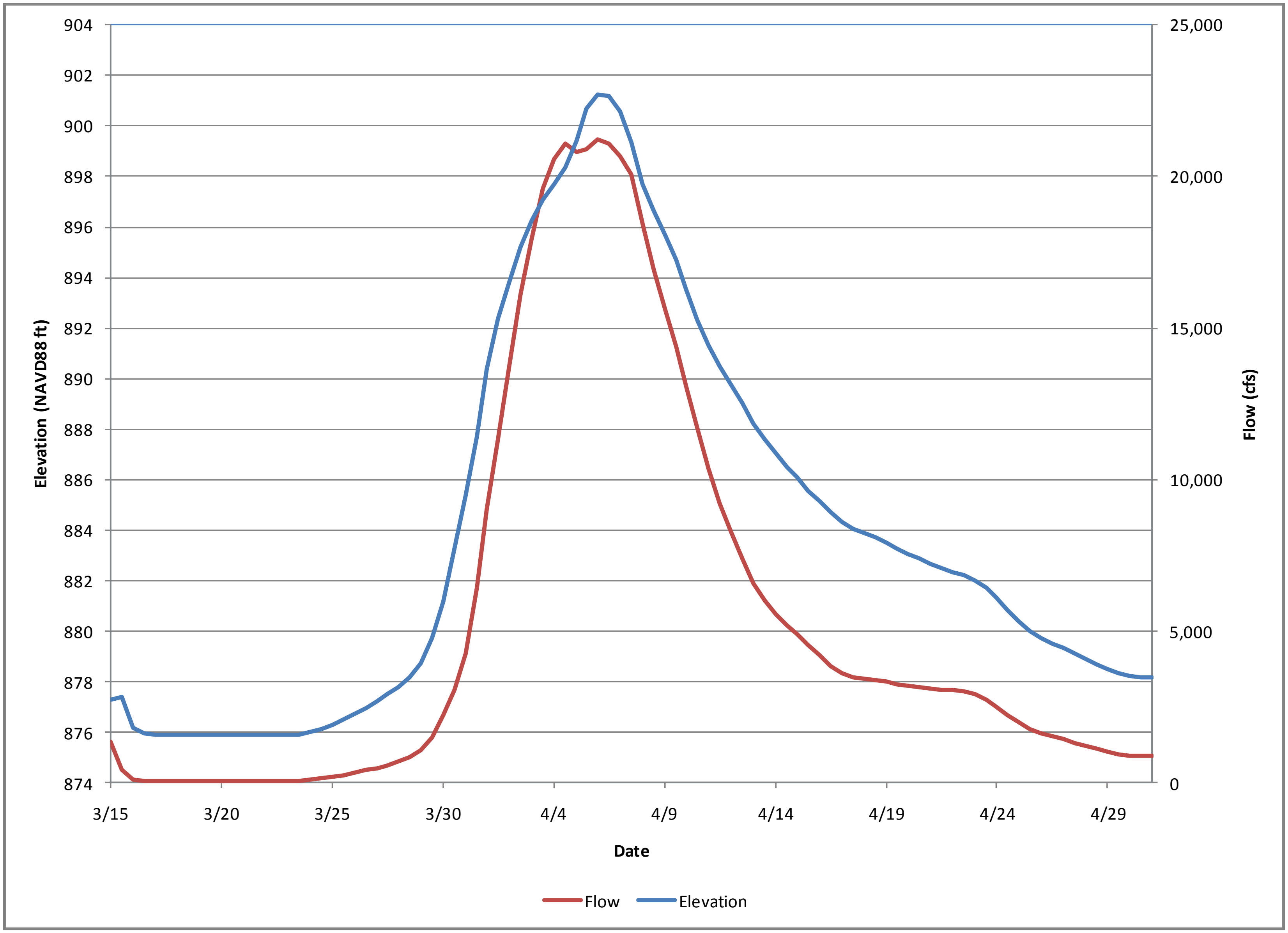


Figure F-D100 Hydrograph between the Sheyenne River and Maple River (RS 83654) for the 500-Year Event

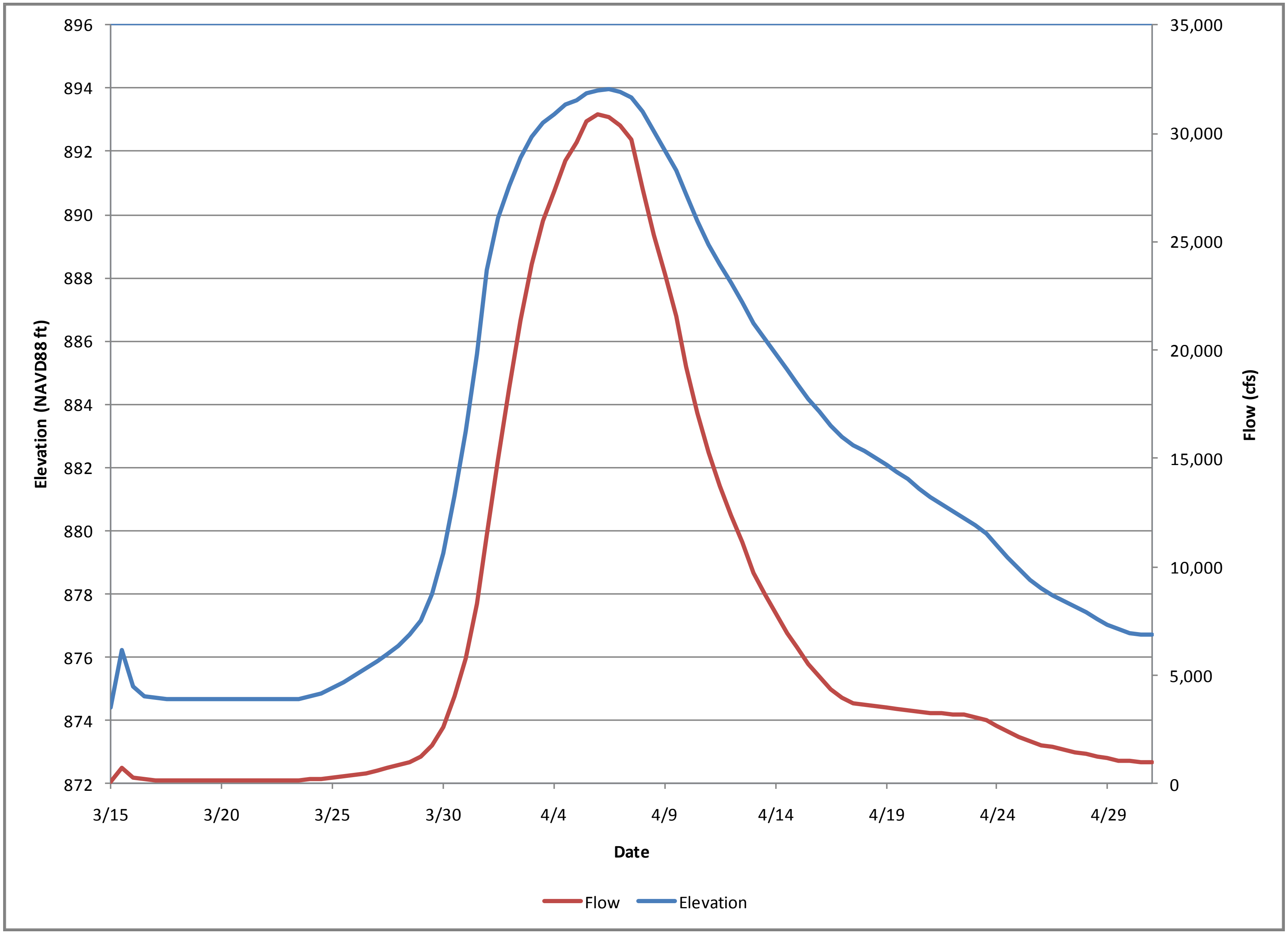


Figure F-D101 Hydrograph between the Maple River and Lower Rush River (RS 64696) for the 500-Year Event

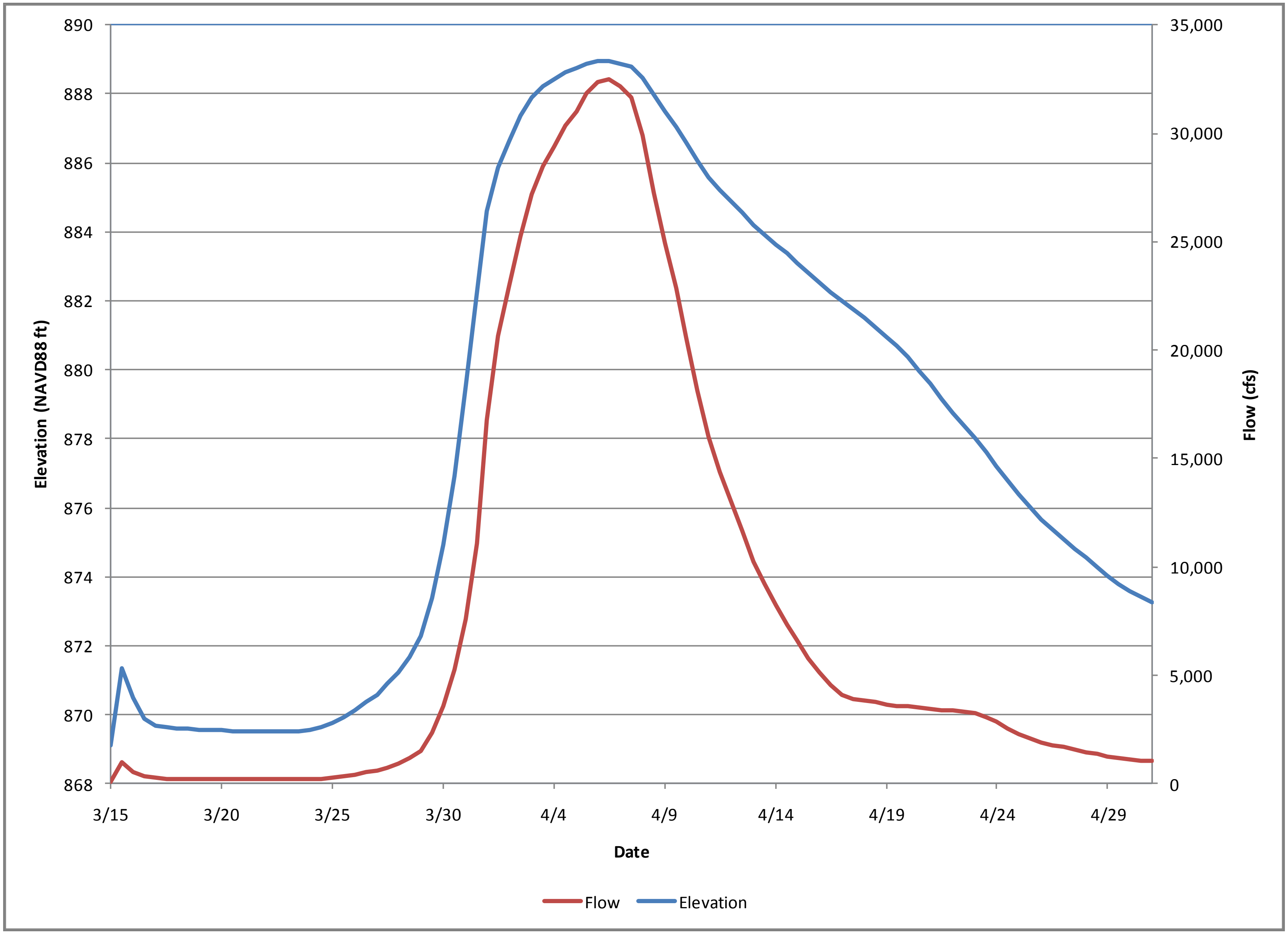


Figure F-D102 Hydrograph between the Rush River and the Outlet Structure (RS 29253) for the 500-Year Event

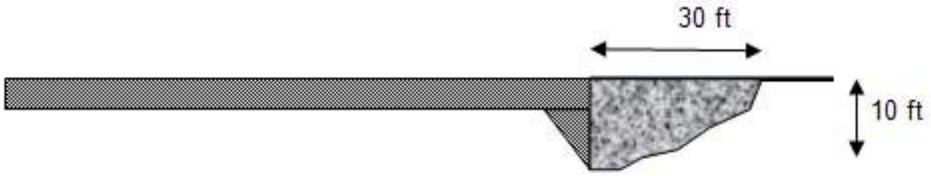
F-D4.0 EROSION PROTECTION CALCULATIONS FOR DOWSTREAM OF THE RED RIVER OF THE NORTH AND WILD RICE RIVER CONTROL STRUCTURES

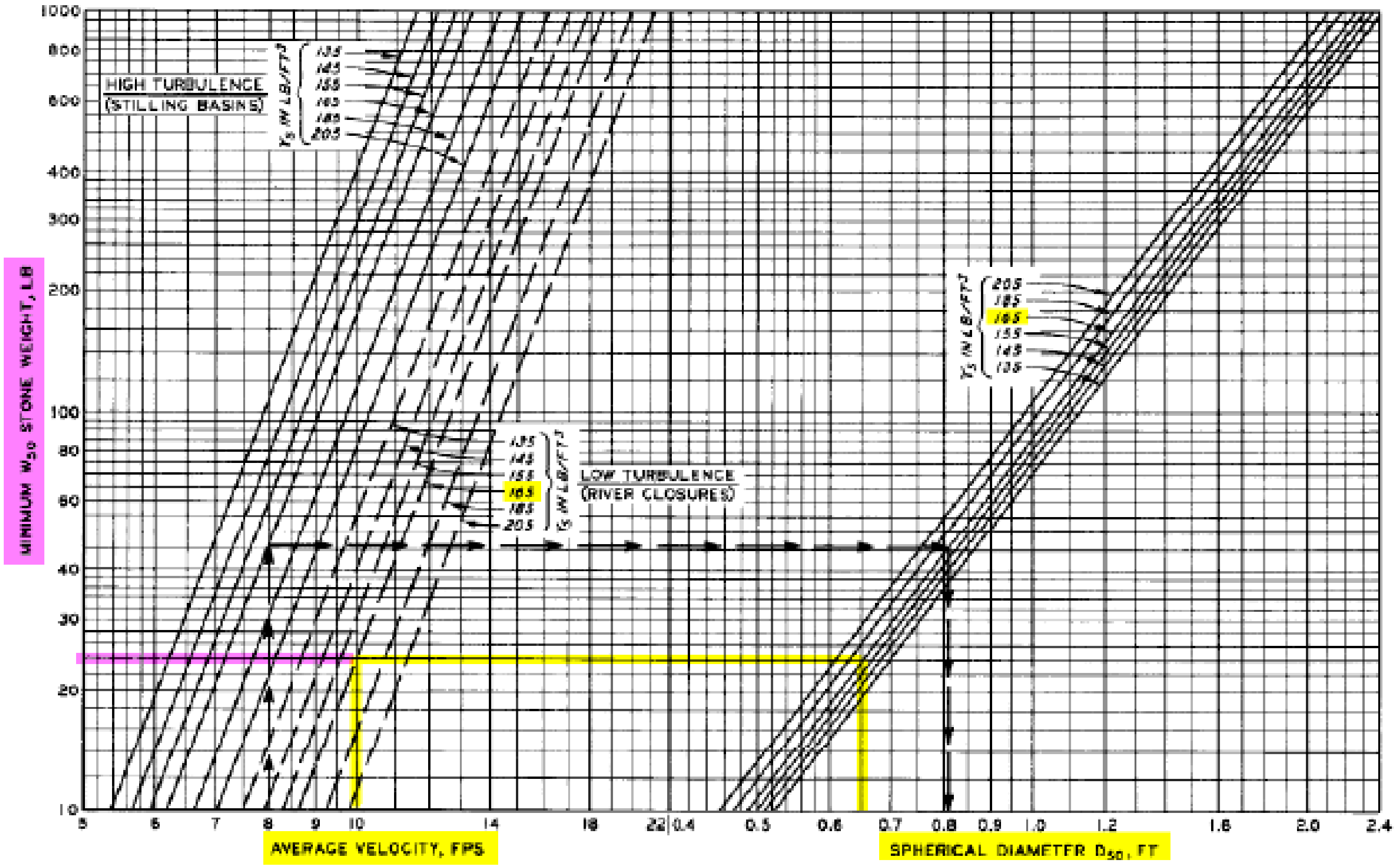
The following information was previously presented as Exhibit D of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness. Flow velocities and return periods have been updated to reflect Phase 4 hydrology unless otherwise noted.

While the range of flow velocities through the gates of the Red River of the North (RRN) and Wild Rice River (WRR) control structures can be quite large, approximately 2-25 ft/sec and approximately 15 – 40 ft/sec, respectively, the velocities significantly decrease once the water is through the gates. For example, at the RRN control structure, HEC-RAS modeling results show that at the bounding cross section on the downstream side of the control structure, velocities range from 1.7 ft/second for the 10-year event to 4.3 ft/second for the 500-year event (downstream flow velocities are less for the WRR structure). In addition, for events greater than the 5-year event (based on Phase 3 hydrology), the gates of the RRN and WRR control structures are submerged; therefore, the jet of water coming through the gates quickly dissipates under so much head. Once the gate configurations for both of the control structures are more well-defined during the next phase of the project, the jet velocities at the end of the concrete slab will be calculated for both structures. For this phase of the project, riprap sizing calculations were performed for erosion protection purposes using the conservative assumption that velocities downstream of the control structures will not exceed 10 ft/sec. The unit weight of stone, which generally varies from 150 to 175 lbs/ft^3, was assumed to be 165 lbs/ft^3. The following page shows the stone stability nomograph from Hydraulic Design Chart 712-1 which shows the relationship between velocity and stone diameter sizing. For a velocity of 10 ft/sec and a unit weight of stone of 165 lbs/ft^3, the spherical diameter of a D50 stone is 0.65 feet.

As discussed in EM 1100-2-1601, stone size should be increased to resist hydrodynamic and a variety of nonhydrodynamic-imposed forces and/or uncontrollable physical conditions. The stone size increase can be accomplished by including a minimum safety factor of 1.1. This safety factor can be increased for a variety of conditions including situations where severe freeze-thaw is anticipated.

For this phase of the project, a minimum median D50 stone size diameter of 1.0 foot has been assumed. A minimum cut-off wall depth of 10 feet has also been assumed, and the length of riprap needed downstream of the cut-off wall has been assumed to be 30 feet (3 times the height of the cut-off wall), as shown conceptually in the drawing below.





BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_S - \gamma_W}{\gamma_W}\right)\right]^{1/2}\left(D_{50}\right)^{1/2}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi\gamma_S}\right)^{1/3}$$

WHERE:

V = VELOCITY, FPS

γ_S = SPECIFIC STONE WEIGHT, LB/FT^3

γ_W = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT^3

W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.

D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}

C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)

g = ACCELERATION OF GRAVITY, FT/SEC^2

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 1 OF 2)

REV 8-58, 9-70 WES 6-57

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT E – HYDRAULIC DESIGN COMPUTATIONS
STORAGE AREA 1

Report for the US Army Corps of Engineers, and the cities of Fargo, ND & Moorhead, MN

By: BARR ENGINEERING

FINAL - February 28, 2011

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F-E1.0 INTRODUCTION

This exhibit documents the analysis used to develop preliminary design recommendations for creating Storage Area 1 (SA1) along the North Dakota East Diversion Channel as part of Phase 4 of the Fargo-Moorhead Metro Flood Risk Management Project.

The design scenarios discussed in this exhibit are preliminary only. The result and conclusions of this analysis are based on the current working hydraulic model for the project. As elements of the hydraulic model are changed, added or removed, portions of this analysis may need to be updated.

F-E2.0 BACKGROUND

F-E2.1 PHASE 4 OBJECTIVES

In Phase 3, this project looked at a design scenario that minimized staging of water upstream of the Protected Area, confining adverse flooding impacts to areas downstream of the Protected Area. In Phase 4, the objective is to develop a design scenario where flood controls keep downstream impacts negligible by staging floodwaters upstream of the Protected Area. Control structures on the major rivers limit the flows passing through and around the control Protected Area. Excess water is stored in designated areas. Most of these areas are to the south of the North Dakota Diversion Channel. The exception being SA1, which is a 4360 acre area on the north side of the diversion channel.

F-E2.2 MODELING APPROACH

F-E2.2.1 Hydrology

The hydrology used to generate flow data for the unsteady HEC-RAS model for the project was developed previously and not modified as part of the SA1 preliminary design task. The four design flood events are the 10-, 50-, 100-, and 500-year. For more information on the project hydrology, see Exhibit A Background Hydrologic Information and Appendix A.

F.E2.2.2 Hydraulics

The hydraulic analysis for Phase 4 is based on results from unsteady HEC-RAS models for each of the design flood events. Models were run using HEC-RAS version 4.1.

F-E2.3 KEY TERMS

The following terms are key components of the flood control system in the vicinity of SA1. A brief definition of these terms is provided below. Each will be discussed in greater detail within the text of this exhibit.

Diversion Channel – Generic term for the channel to be constructed around the City of Fargo. The channel begins and ends on the Red River and intersects the North Dakota tributaries to the Red River. This exhibit focuses on the portion of the Diversion Channel on the south side of SA1 between the Wild Rice Control Structure and the Main Inlet Weir.

Main Inlet Weir – An in-line structure in the Diversion Channel that marks the transition between the reach south of SA1 where major flood events are outside the channel banks, to a deeper channel that contains the design flood events within the channel banks.

Protected Area – Developed area, which includes the cities of Fargo and Moorhead, that will be protected from the design flood events by this project.

Wild Rice Control Structure – Gated structure that controls flow from the Wild Rice River into the Protected Area.

Modified Channel – In order to construct the Wild Rice Control Structure in the dry, a modified channel will be excavated for the Wild Rice River that cuts off a meander near the southeast corner of SA1.

Inlet-Outlet Opening – A large opening in the levee around SA1 that allows water from the Diversion Channel to enter and leave SA1.

East Outlet – A gated structure in the East Levee for SA1 that will allow for natural drainage of SA1 to the Wild Rice River during normal conditions. The structure will be closed during flood events.

North Outlet – A gated structure in the North Levee for SA1 that will allow for natural drainage from SA1 to Rose Coule Drain 27 during normal conditions. The structure will be closed during flood events.

Tie-Back Levee – In addition to SA1 other areas will be inundated during major flood events. To contain this inundation there will be Tie-Back levees on the east and west sides of the staging area. The west Tie-Back levee extends from the Main Inlet Weir to the south at elevation 923.

Wild Rice River – Unless otherwise stated, any mention of the Wild Rice River in this exhibit refers to the North Dakota Wild Rice River, as opposed to the Minnesota Wild Rice River.

F-E3.0 DESIGN OBJECTIVES

The intent of SA1 is to provide contained flood storage upstream of the Protected Area. The design of SA1 must balance the design objectives identified below.

Storage – The goal of providing storage in SA1 is to minimize the depth of staging upstream. The lower the upstream staging the smaller the inundation footprint for major flood events on upstream properties.

Footprint – To achieve a meaningful storage volume in SA1, there will need to be a significant property takes. The layout of SA1 should minimize the number of property owners and structures affected.

Safety – SA1 will be constructed immediately south of the Protected Area. The long term integrity of the levees around SA1 is critical to the safety of the community and the success of the project.

Simplicity – The flood control systems proposed by this project requires some complex elements. The operations and maintenance of SA1 should be kept simple as much as possible, so SA1 can operate effectively with minimal human input.

Flexibility – Design of SA1 should allow for continued use of the area by the community when the area is not flooded. This includes allowing some roads to continue to pass through the Storage Area.

F-E4.0 SITE DESCRIPTION

The area designated for SA1 is predominantly farm fields with a few structures that mostly consist of houses and farm buildings. Topographically SA1 is between the Sheyenne River and the Wild Rice River. Relative to existing infrastructure SA1 is bounded by County Road 17 (Co Rd 17) on the west, Co Rd 14 on the north, and Interstate-29 (I-29) on the east. Figure F-E1 is a map that shows proposed flood control measures and existing roads in the vicinity of SA1.

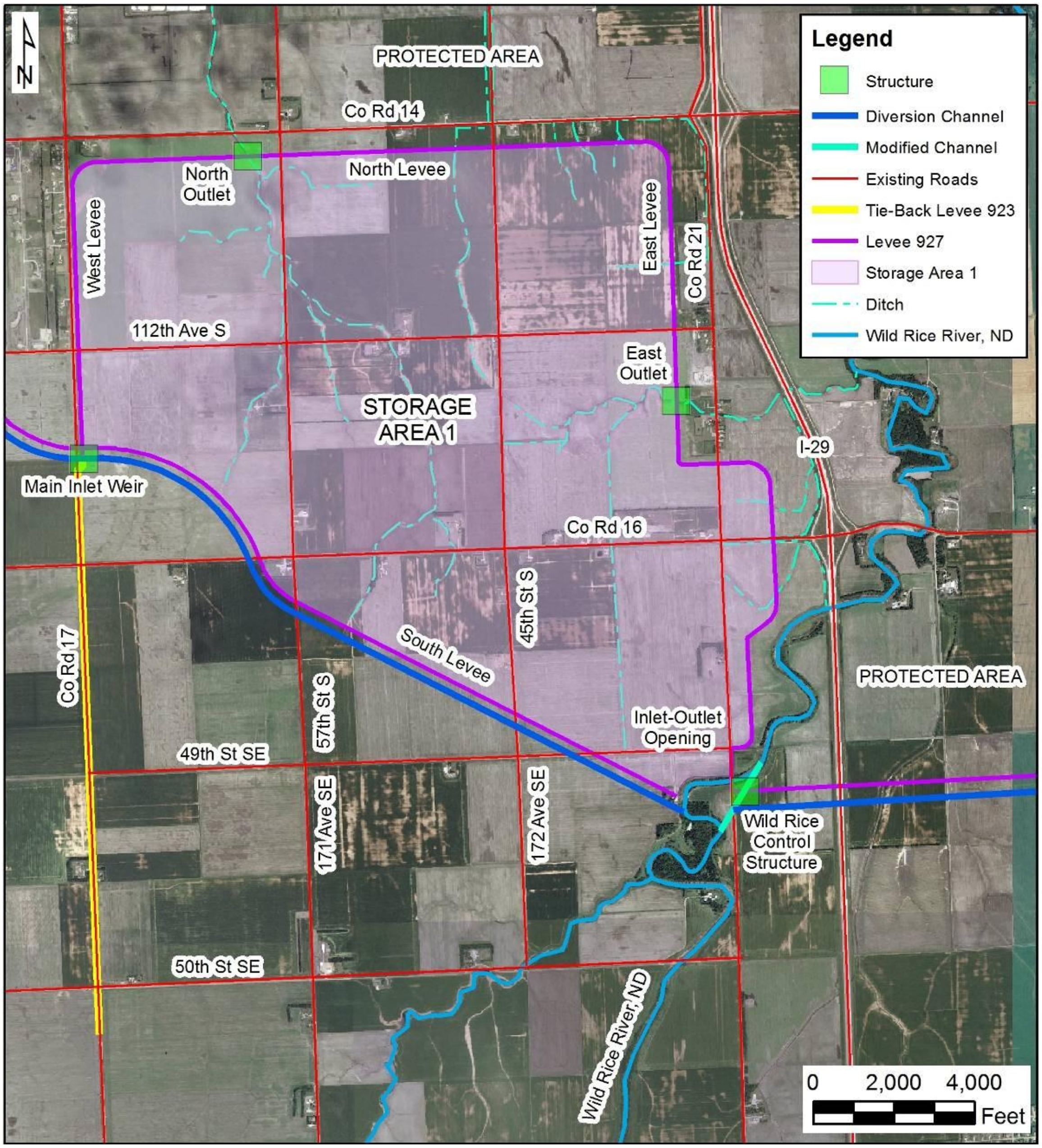

Figure F-E1 Vicinity map for Storage Area 1

Figure F-E2 shows the same area as Figure F-E1, but with a colored elevation gradient ranging from elevation 900 to 923. The topography generally slopes to the north, with some areas draining to the northeast. The low end of this range corresponds to the bottom of the drainage ditch at the North Outlet. The high end of the elevation range corresponds to the crest elevation of the Tie-Back Levee to be constructed along Co Rd 17. The existing ground elevations within SA1 range between 908 and 915.

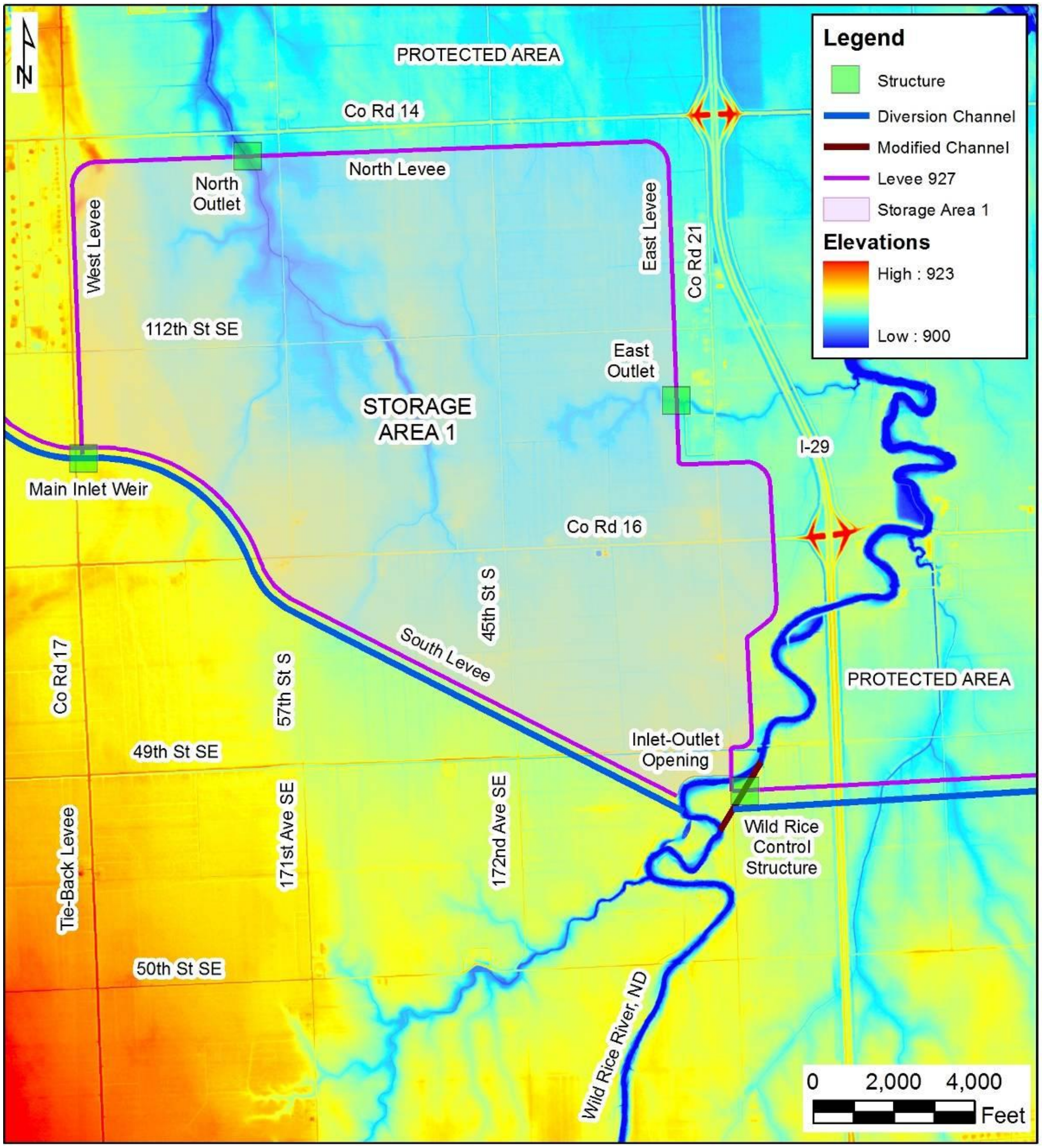

Figure F-E2 Elevation gradient map for Storage Area 1

There are two primary drainage basins within SA1. The larger of the two basins drains north to Rose Coule Drain 27. The smaller basin drains east to the Wild Rice River.

F-E5.0 PRELIMINARY DESIGN

This section describes the recommended preliminary design for SA1 and related infrastructure. A detailed description of how these recommendations were developed is provided in the Design Analysis section below.

The proposed footprint for SA1 is shown in Figure F-E1. This 4360 acre area will be surrounded by a levee at elevation 927. The crest elevation for the levee assumes a minimum of 4-feet of freeboard above the peak stage for the 100-year and 500-year flood events.

SA1 will provide over 55,000 acre-feet of storage during the 100-year and 500-year flood events. Initial modeling results indicated that the inclusion of SA1 results in roughly a 2-foot reduction in the upstream staging elevation during those same flood events.

There will be three openings in the SA1 levees. The Inlet-Outlet Opening will be a 1400-foot gap in the levee at the southeast corner of SA1. This opening hydraulically connects the Wild Rice River and the Diversion Channel with SA1.

The other two openings will be the North Outlet and East Outlet. These will be gated structures that will be left open to allow for natural drainage during non-flood conditions. During major floods the gates will be closed so that flood waters are not released into the Protected Area. After the flood event has passed the gates will be opened to allow low areas within SA1 to drain.

Co Rd 16 and a portion of Co Rd 21 will be maintained through SA1. To accomplish this, the roads will need to be elevated over the levee crest. By maintaining roads within SA1, it will be possible for local property owners to continue to use the land during normal, non-flood, conditions.

F-E6.0 DESIGN ANALYSIS

The preliminary design for SA1 balances an array of factors and design criteria. The major considerations affecting the design are discussed below.

F-E6.1 FOOTPRINT

The area delineated for the proposed storage area is approximately 4360 acres, with a perimeter of about 58,000 feet. It is roughly bounded by the proposed Diversion Channel to the south, Co Rd 17 to the west, Co Rd 14 to the north and I-29 to the East. Geographically, this area is bounded by the Sheyenne River to the west and the Wild Rice River to the east (Figure F-E1).

The footprint for SA1 was selected to minimize impacts to existing infrastructure, maximize storage capacity, and effectively interact with other flood control elements within the project.

The perimeter stays east of Co Rd 17 and south of Co Rd 14, so that the only County Roads affected are 16 and 21. Existing structures within SA1 will need to be removed. The proposed footprint was chosen in part because of the low density of existing structures.

Where practical, the levee alignment was adjusted to avoid unnecessary impacts to structures. The perimeter levees avoid taking the structures along the south side of Co Rd 14 and those along Co Rd 21.

Constructing a basin this large will require significant property takes. Careful consideration was given regarding how to achieve the maximum benefit for the community with minimum adverse impacts to individual property owners. There will no doubt be objections to placement of SA1 by those adversely impacted. The challenge for the project stakeholders is to evaluate the short and long-term impacts of this feature and determine if the benefits are worth the cost.

A separate analysis by Moore Engineering is assessing the hydraulic impact of an even larger SA1 that extends further to the north.

F-E6.2 PROPOSED LEVEES

The levee surrounding SA1 has a consistent template—the 15-foot wide crest is at elevation 927 and the side slopes are 4:1. The levee top width and side slopes were set based on a preliminary geotechnical analysis, discussed in Exhibit N Slope Stability. The crest elevation of the levee assumes a minimum of 4-feet of freeboard above the 100-year and 500-year peak upstream stage.

With the exception of the Inlet-Outlet Opening, the perimeter of SA1 is essentially one continuous levee. For organizational purposes the sections of levee on each of the four sides of SA1 were named South Levee, West Levee, North Levee, and East Levee.

The levees will be constructed from Diversion Channel spoils and from borrow trenches within SA1. A 100-foot buffer will separate the toe of levee from the edge of the adjacent borrow trench. The width and depth of the trenches have been approximated so as to provide sufficient fill material for the levees and to provide internal drainage conveyance for ditches interrupted by the placement of the levees.

Details specific to each section of levee are discussed below.

F-E6.2.1 South Levee

The South Levee parallels the Diversion Channel (Figure F-E1). The south toe of slope for the levee is offset 300-feet from the top of bank on the north side of the channel. Spoils from the excavation of the Diversion Channel will be used to construct the South Levee. The height of the levee above existing grade ranges from 10 feet at the west end to 18 feet near the east end. Figure F-E3 shows the top elevation of the South Levee relative to the Diversion Channel thalweg and bank elevations.

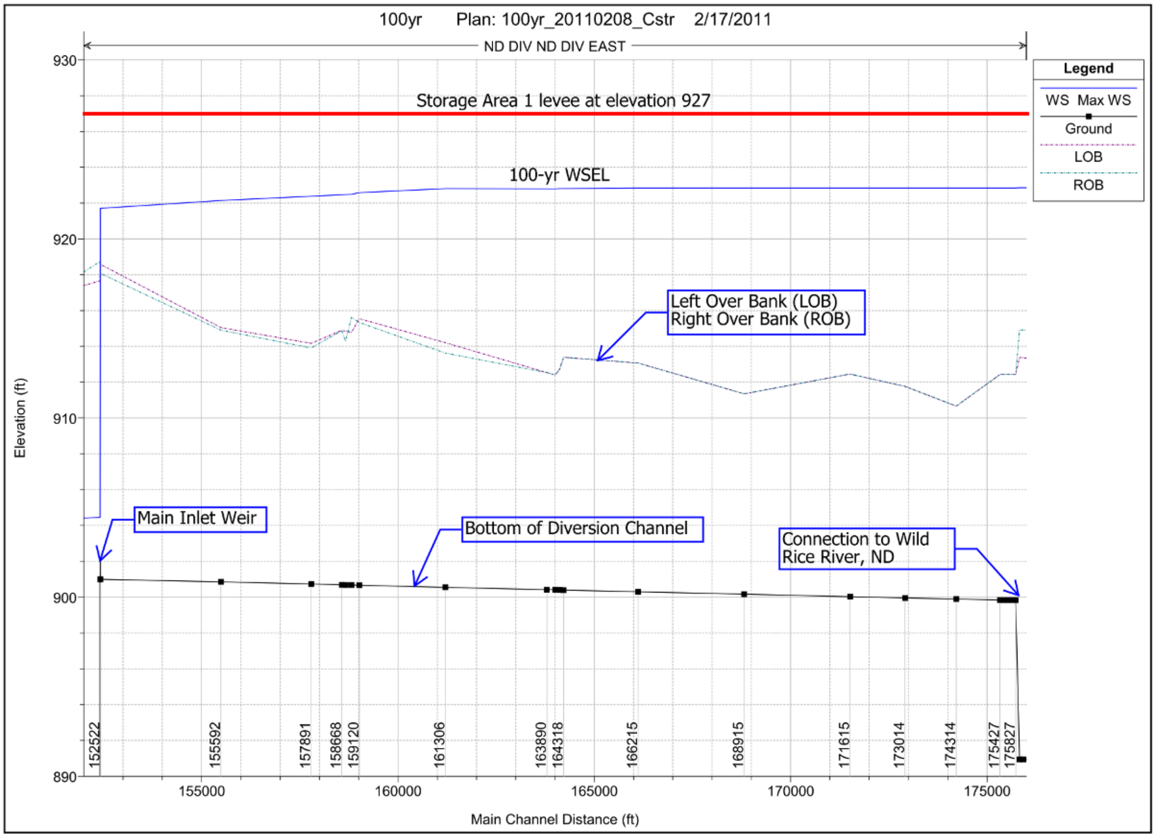

Figure F-E3 Diversion Channel profile along southern edge of Storage Area 1

The profile plot in Figure F-E3 illustrates the average height for the South Levee. The left overbank (LOB) and right overbank (ROB) profiles provide a rough indication of the existing ground elevation along the South Levee alignment. The ground generally slopes from west to east towards the Wild Rice River.

No structures are planned for construction in the South Levee. Co Rd 16 will pass up and over the South Levee. The levee will interrupt 49th St SE, 57th St SE, and 45th St SE. See the Roadway Impacts section in this exhibit for more details.

F-E6.2.2 West Levee

The West Levee runs along the east side of Co Rd 17, with an offset of approximately 150 feet from centerline of road to centerline of levee (Figure F-E1). It is assumed that no structures will be constructed in the West Levee. No roads will be carried over the West Levee. The levee will interrupt 112 St SE.

F-E6.2.3 North Levee

The North Levee is on the south side the Co Rd 14, with an offset of approximately 600-feet from centerline of road to centerline of levee. (Figure F-E1). This offset avoids the taking of five existing residential structures and their associated out-buildings along the south side of Co Rd 14.

The North Outlet will be constructed in the North Levee and in-line with a major drainage ditch that would otherwise be blocked by the levee. The design of this structure is discussed in a separate section below. There are no roadways over the top of the North Levee. The levee interrupts 57th St SE and 45th St SE.

F-E6.2.4 East Levee

The East Levee runs along the west side of I-29 (Figure F-E1). At the north end the levee alignment is on the west side of Co Rd 21. Once the levee gets south of a developed section along Co Rd 21, the alignment shifts to the east side of Co Rd 21. The southern portion of the levee jogs back to the west to avoid impacts to the Wild Rice River, ND.

The East Outlet will be constructed in the East Levee and in-line with the major drainage ditch that would otherwise be blocked by the levee. The design of the structure is discussed in a separate section below.

Both Co Rd 21 and Co Rd 16 will pass up and over the East Levee embankment. The levee interrupts 112th St SE. These changes are discussed further under Roadway Impacts.

F-E6.3 DIVERSION CHANNEL

There are three distinct reaches of the Diversion Channel near SA1. The reach east of SA1 connects the Wild Rice River with the Red River. The reach west of SA1 is a deep channel design to contain and convey all of the design flood events for the project.

The reach most frequently discussed in this exhibit is the portion of the Diversion Channel immediately south of SA1 that extends from the Wild Rice River to the Main Inlet Weir. As seen in Figure F-E3 above, this reach of the Diversion Channel slopes from west to east, towards the Wild Rice River. There are two primary reasons for this design. First, the existing ground for this reach of the Diversion Channel generally slopes to the Wild Rice River. Second, the hydraulic design requires that the Main Inlet Weir crest elevation be above the thalweg for the Diversion Channel. Sloping the channel towards the Main Inlet weir would require a low flow path through the Main Inlet Weir, which would add complexity and cost to the project. Figure F-E4 shows a typical cross-section for the Diversion Channel.

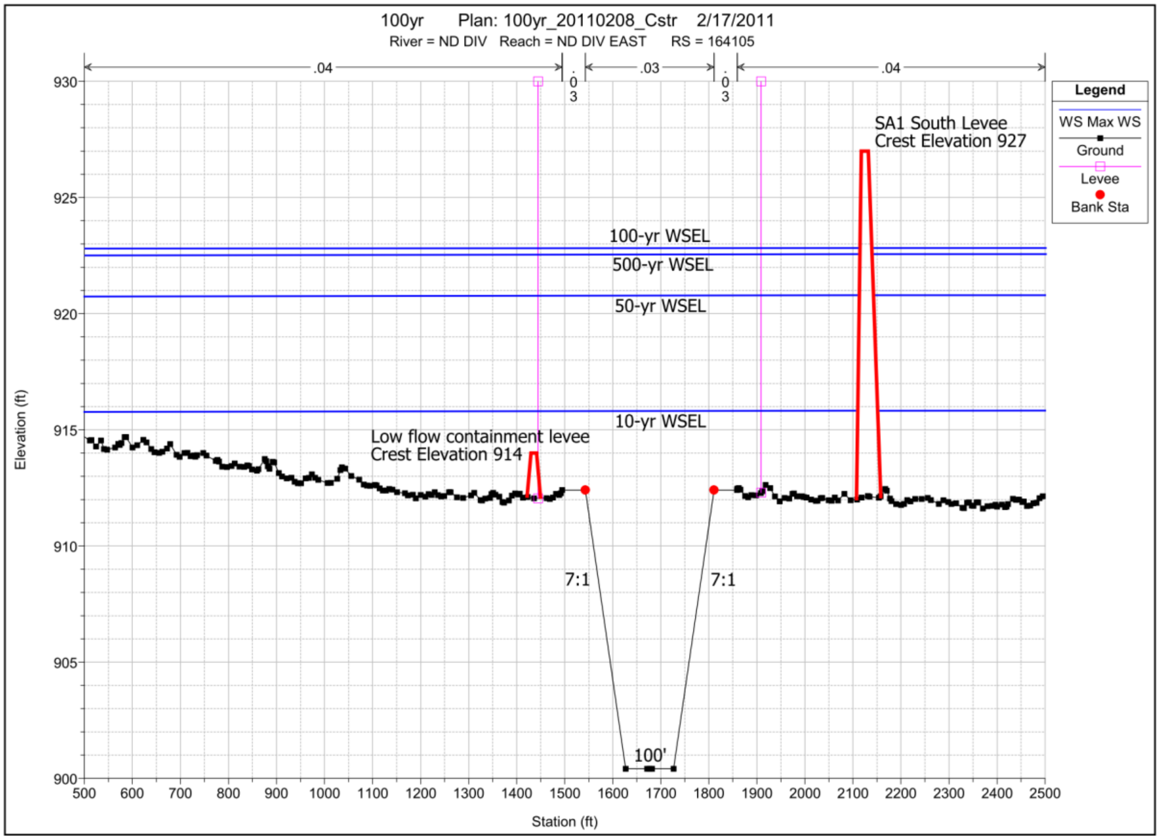

Figure F-E4 Diversion Channel cross-section

Some of the spoils from the excavation of the Diversion Channel will be used to raise the south bank of the diversion channel up to at least elevation 914. As seen in Figure F-E4, fill will be needed for roughly the eastern two-thirds of the Diversion Channel's left overbank. The remainder of the spoils from the Diversion Channel excavation will be used to construct the South Levee and portions of the East and West Levees.

The bottom width for the SA1 section of the Diversion Channel is 100-feet. This width was selected to be slightly wider than the 90 foot crest length for the Main Inlet Weir. The side-slopes for the channel are 7:1 (H:V).

F-E6.3 1 Main Inlet Weir

The purpose of the Main Inlet Weir is to control the flow of water from the staging area into the portion of the Diversion Channel that is designed to contain the four design flood events. The 90-foot Ogee weir has a crest elevation of 903.25. Water surface elevations below the weir are roughly 20 feet lower than those upstream of the weir. The design of the Main Inlet Weir is discussed in detail in Exhibit D – Diversion Structures.

F-E6.4 INTERNAL DRAINAGE

As discussed in the Site Description section above, there are two existing drainage basins within SA1. The larger basin drains north to Rose Coule Drain 27. The smaller basin drains east to the Wild Rice River. Levee construction around SA1 has the potential to cut off these drainage basins from their existing downstream flow paths.

F-E6.4.1 Outlet Options

Two options were considered for dealing with internal drainage within SA1.

Option 1
Construct two outlet control structures through the proposed levees and in-line with the existing drainage ditches.

Advantages
- Maintains existing drainage patterns
- Requires limited grading above and beyond what is already required for excavation of the borrow trenches to construct the levees around SA1

Disadvantages
- Control structures are expensive
- Control structures add complexity to the future operation and maintenance of SA1
- Placing an outlet control structure through the levee creates a potential weak point in the levee embankment

Option 2
Use borrow trenches and modified ditch channels to route all surface drainage south to the diversion channel.

Advantages
- No operation requirements and minimal maintenance
- Leaves levee embankments uninterrupted

Disadvantages
- Alters existing drainage patterns
- May require significant amounts of additional grading to collect and convey water to the south against existing grades
- May require fill in some places to remove low points and make it possible to drain water back to the diversion channel
- Would require the construction of large culverts under existing roads

The preliminary design for SA1 is based on Option 1. It assumes the construction of a North Outlet and East Outlet in-line with the two largest drainage ditches leaving the site. By maintaining existing drainage patterns, the design limits potentially adverse impacts on downstream property owners who may rely on runoff from the watershed for irrigation. The cost to route surface runoff from over 4360 acres against the natural slope of the land would likely be more expensive than constructing two outlet control structures.

While Option 1 takes advantage of the existing ditch drainage system, not all ditches within the SA1 footprint drain to the North Outlet or East Outlet locations. Several smaller ditches, particularly in the northeast corner of SA1, drain away from the site on their own. These ditches will be intercepted by the borrow trenches excavated to construct the West, North and East Levees. Borrow trenches along the West and North Levees will drain to the North Outlet. Borrow trenches along the East Levee will drain to the East Outlet. A small area adjacent to the Wild Rice River will drain back through the Inlet-Outlet Opening for SA1.

F-E6.4.2 North Outlet

The drainage area for the North Outlet is roughly 2810-acres. Concrete retaining walls will create a 10-foot wide break in the levee embankment. A sluice gate and stop logs will control flow through the opening. The outlet will be constructed in-line with the existing drainage ditch. Figure F-E5 is a vicinity map for the North Outlet.

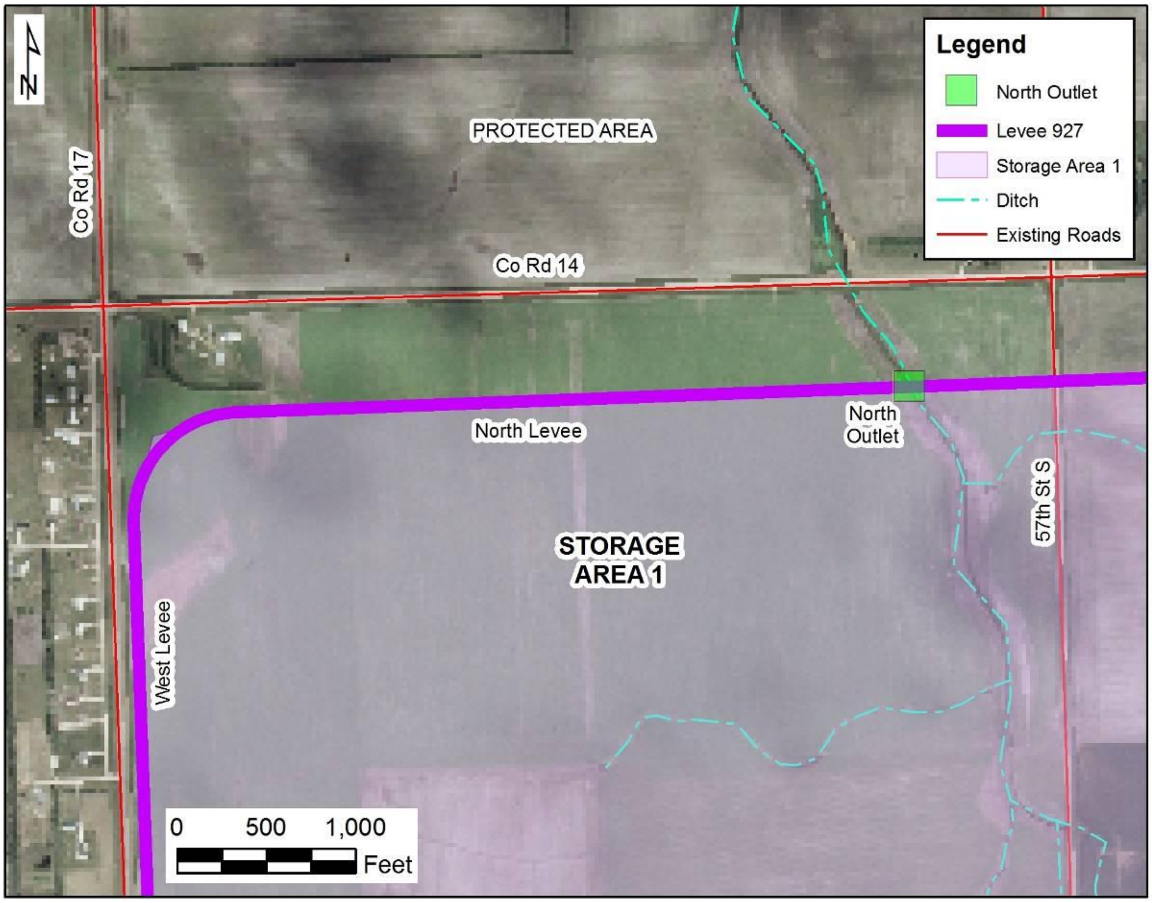

Figure F-E5 Vicinity map for North Outlet from Storage Area 1

Sizing of the North Outlet was based on a hydrologic analysis of the contributing drainage area and an assessment of existing drainage structures along the ditch. From aerial imagery, the culvert crossing beneath County Road 14 (Co Rd 14) appears to be a double 6'x6' or 8'x8' concrete box culvert.

F-E6.4.3 East Outlet

The drainage area for the East Outlet will be roughly 1500 acres. Concrete retaining walls will create a 10-foot wide break in the levee embankment. A sluice gate and stop logs will control flow through the opening. The outlet will be constructed in-line with the existing drainage ditch. Figure F-E6 provides a vicinity map for the East Outlet.

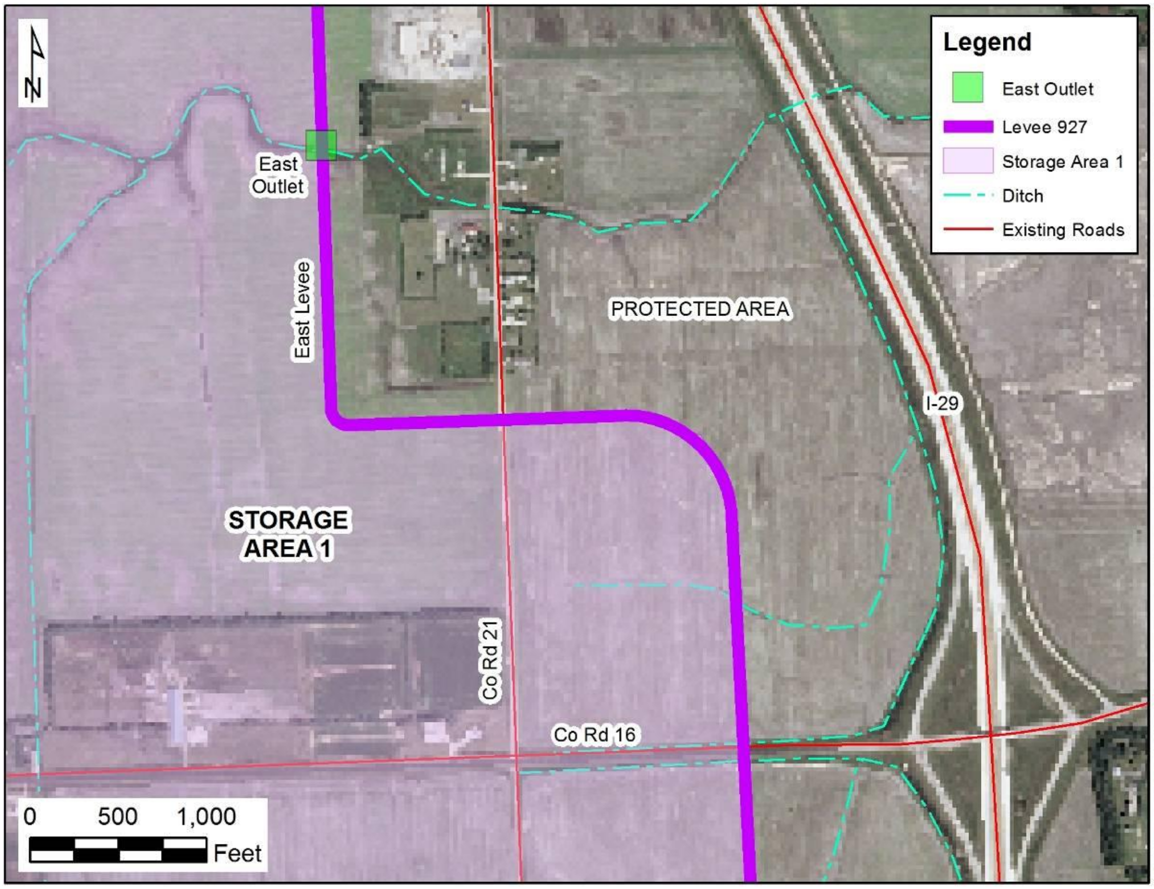

Figure F-E6 Vicinity map for East Outlet from Storage Area 1

Sizing of the East Outlet was based on a hydrologic analysis of the contributing drainage area and an assessment of existing drainage structures along the ditch. From aerial imagery, the culvert under Co Rd 21 appears to be a circular CMP pipe, perhaps 36" in diameter.

F-E6.4.4 Operations and Maintenance

The gates for the East Outlet and North Outlet will generally remain open to allow for the natural drainage of surface runoff from SA1. When major flooding is imminent, the gates to both outlet control structures will be closed for the duration of the flood event. Once the floodwaters have subsided, the gates will be reopened to allow for the drainage of portions of SA1 that are too low to drain out through the Inlet-Outlet Opening.

The final design will need to determine appropriate energy dissipation measures downstream of the outlets. Possible solutions include baffles, stilling basin or sill. The size of the outlet, the gate control rules and the amount of energy dissipation will all depend on the upstream storage depth after a flood and the desired drawdown time.

Access to the North and East Outlet structures will be from inside the Protected Area so that operators can operate the gates as-needed before during and after a flood event. East Outlet access will be from Co Rd 21. North Outlet access will be from Co Rd 14.

F-E6.5 HYDRAULICS

SA1 will be hydraulically connected with the Wild Rice River and the Diversion Channel through the Inlet-Outlet Opening, which will be a 1400-foot gap in the levee perimeter near the Wild Rice Control Structure. Figure F-E7 shows approximate layout for the proposed flood control infrastructure in and around the Inlet-Outlet Opening.

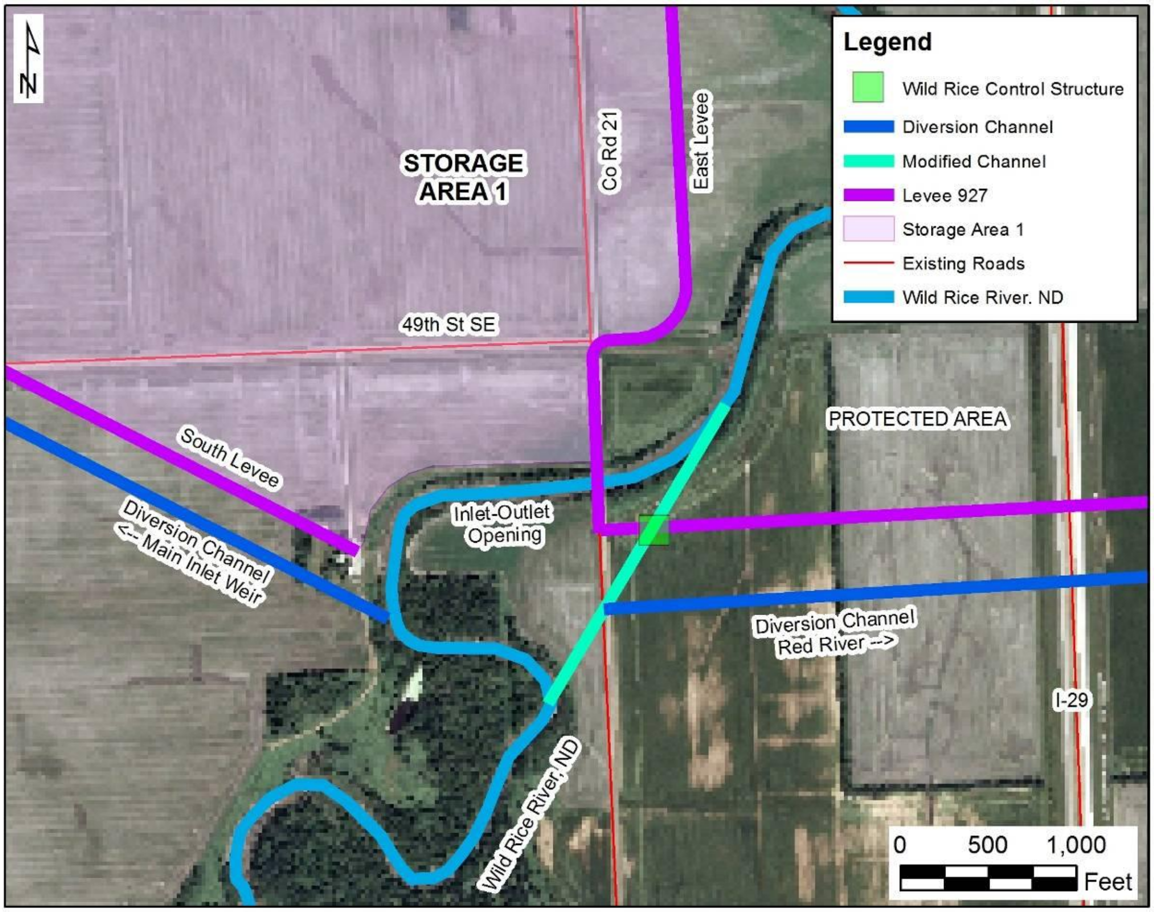

Figure F-E7 Vicinity Map for Inlet-Outlet Opening for Storage Area 1

F-E6.5.1 Wild Rice Control Structure

In order to evaluate the Inlet-Outlet Opening it is necessary to understand how the Wild Rice Control Structure and related flood control measures function. A detailed discussion of the design for the Wild Rice Control Structure is in Exhibit D Hydraulic Design

Computations – Diversion Structures. An abbreviated description follows. In order to construct the Wild Rice Control Structure in the dry, a modified channel for the Wild Rice River will be constructed roughly as shown in Figure F-E7. There are three major areas of disturbance on the Wild Rice River. The two connection points for the Modified Channel and the extension of the levee across the channel on the same alignment as Co Rd 21. There will likely be minor impacts to the channel bank from the portion of the Diversion Channel that extends west along the south side of SA1. The bypassed meander of the Wild Rice River will remain in place as a slough.

With this configuration in place, the Wild Rice River will flow as normal during non-flood events. Surface drainage collected by the Diversion Channel will drain to the Wild Rice River. Surface drainage collected by the east reach of the Diversion Channel will drain to the Red River.

As water levels rise during major flood events (e.g. 10-year or more) the flood levels in the Red River and the Wild Rice River will equalize via the east reach of the Diversion Channel. Water will also begin backing up into the SA1 reach of the diversion channel. The Wild Rice Control Structure and Red River Control Structure limit flows into the Protected Area. When upstream flows exceed the capacity of the gate openings for the Wild Rice and Red River control structures, water will begin backing up into low-lying areas, including SA1.

F-E6.5.2 Inlet-Outlet Options

The two primary options for the flow of floodwaters into and out of SA1 are an uncontrolled opening or a control structure.

Option 1 – Uncontrolled Opening
Leave an opening in the levee that allows water in SA1 to equalize with the water in the diversion channel.

Advantages

- Simple design that does not require operation during flood events
- Low cost alternative since it means less levee construction and little to no engineering required for the Inlet-Outlet
- Minimizes the amount of time that the levee has to withstand elevated flood waters

Disadvantages

- Lack of control over how long water is detained in SA1

Option 2 – Control Structure
A controlled opening that could be opened to allow water into the storage area and temporarily closed until after the peak of the flood has passed and the water can be safely released downstream.

Advantages

- Ability to regulate the rate of inflow and outflow from SA1 in order to maximize flood mitigation benefits

Disadvantages

- Adds complexity to the design
- Creates an additional structure that must be operated during flood events
- Increases initial construction costs and long term maintenance costs
- Retention of water within SA1 extends the time that a large volume of water is being held back by the SA1 levee

Option 2 operates using the following design scenario. The Main Inlet Weir is replaced with a gated structure. During the rising leg of the hydrograph, the Main Inlet Gate is closed and the SA1 control structure gate remains open. Once SA1 is full, the Inlet-Outlet Gate is closed and the Main Inlet Gate is opened. After flood waters recede, the SA1 control structure gates are opened to release the stored water.

Preliminary modeling results indicate that although retaining water in SA1 helps reduce the duration of upstream staging, it has no benefit in relation to downstream impacts. The limited flood mitigation benefit and the other significant disadvantages to using a control structure led to the decision to leave the inlet-outlet for SA1 uncontrolled.

F-E6.5.3 Inlet-Outlet Opening Size and Location

Having opted for an uncontrolled opening, the subsequent design questions for the Inlet-Outlet Opening are how wide should the opening be, where should it occur, and at what elevation should water enter the storage area. For SA1 to be hydraulically connected to the upstream staging area, the opening for SA1 must be along the boundary with the Diversion Channel. The preliminary design analysis looked at various combinations of opening locations, widths, and elevations.

Opening Location

Hydraulically, the location of the opening had minimal effect on upstream and downstream impacts. It was noted that an opening placed further to the west would result in lower peak stage in SA1 because the peak water surface elevations at the Main Inlet Weir are lower than at the Wild Rice River. This drop in the water surface elevation from east to west is shown in Figure F-E3. However, it is somewhat questionable whether this drop will be as pronounced as the model indicates. The RAS model assumes that conveyance occurs in the diversion channel and the adjacent storage areas simply match the channel profile. It is more likely that for most major floods there will be no defined flow path and the water surface elevation will be relatively constant between the Wild Rice River and the Main Inlet Weir. For modeling purposes, the opening location was placed at the southeast corner of SA1 to more accurately model the peak stage within SA1. A more important factor governing the Opening location turns out to be the existing ground elevations and their effect on the opening elevation.

Opening Elevation

A lower opening elevation means that water enters SA1 sooner. It also means that less water will be left in the system once the flooding subsides. Opening elevations as low as 904 and as high as 918 were tested. From a practical standpoint however, the existing ground elevation effectively sets the lower limit for the opening. There is limited storage in SA1 below elevation 908 and the elevations along the Diversion Channel range from 910 to 915 (Figure F-E2). Even if water is allowed to enter the area below the existing grade, there will not be meaningful storage until the water surface gets above 910.

Opening Width

Various opening widths ranging from 1000-feet, to 10,000 feet were tested. The design analysis found that wider openings tend to reduce upstream staging and increase downstream water surface elevations. If the opening is too narrow, the peak elevation in SA1 does not have enough time to match the peak in the Diversion Channel before the peak has passed. However, the differences between the tested scenarios were on the orders of a few hundredths of a foot. While adjusting the opening width could be used to refine the model during final design, the cost of earthwork and site constraints will play the biggest role in determining the opening width.

The opening widths tested all assume that the South Levee exists in some form. However, an alternative scenario would be to leave the south side of SA1—no South Levee at all. The major apparent advantage to this approach would be an 18,000 foot reduction in levee construction. However, there are several compelling reasons to construct the South Levee.

First, spoils from the excavation of the Diversion Channel need to go somewhere. It is assumed that it will be cheaper construct levees from materials immediately adjacent to the levee embankment, rather than hauling the material in from other places. This means using Diversion Channel for embankments adjacent to the channel, and borrow trenches to construct other levees that are not adjacent to a planned excavation.

Second, the staging area will create a vast expanse of water during major flood events. Having the South Levee will help break up wave action created by wind sweeping across the staged water.

Third, it is assumed that in the future local sponsors will try to find a way to reclaim the land used for SA1 through other mitigation measures that eliminate the need for SA1. If this happens it will be much more feasible for them to close a narrow gap between the South Levee and the East Levee than to construct over 3 miles of new levee.

Fourth, if there is ever a problem with one of the other levees for SA1, it would be possible to temporary fill in the Inlet-Outlet Opening to keep floodwaters out of SA1 while the problem is fixed.

Preliminary Design

Given these considerations, the preliminary design for the Inlet-Outlet Opening is a 1400-foot wide gap between at the southeast corner of SA1 near the Wild Rice Control Structure. The length was set primarily based on existing topography near the Wild Rice Control Structure. The meander in the Wild Rice River makes it unnecessary to continue the Diversion Channel excavation further east. With no spoils to place, it becomes a convenient place to leave an opening in the perimeter of the storage area. The existing ground elevations at the opening are between 912 and 914. Figure F-E8 displays a color elevation gradient for the area near the opening.

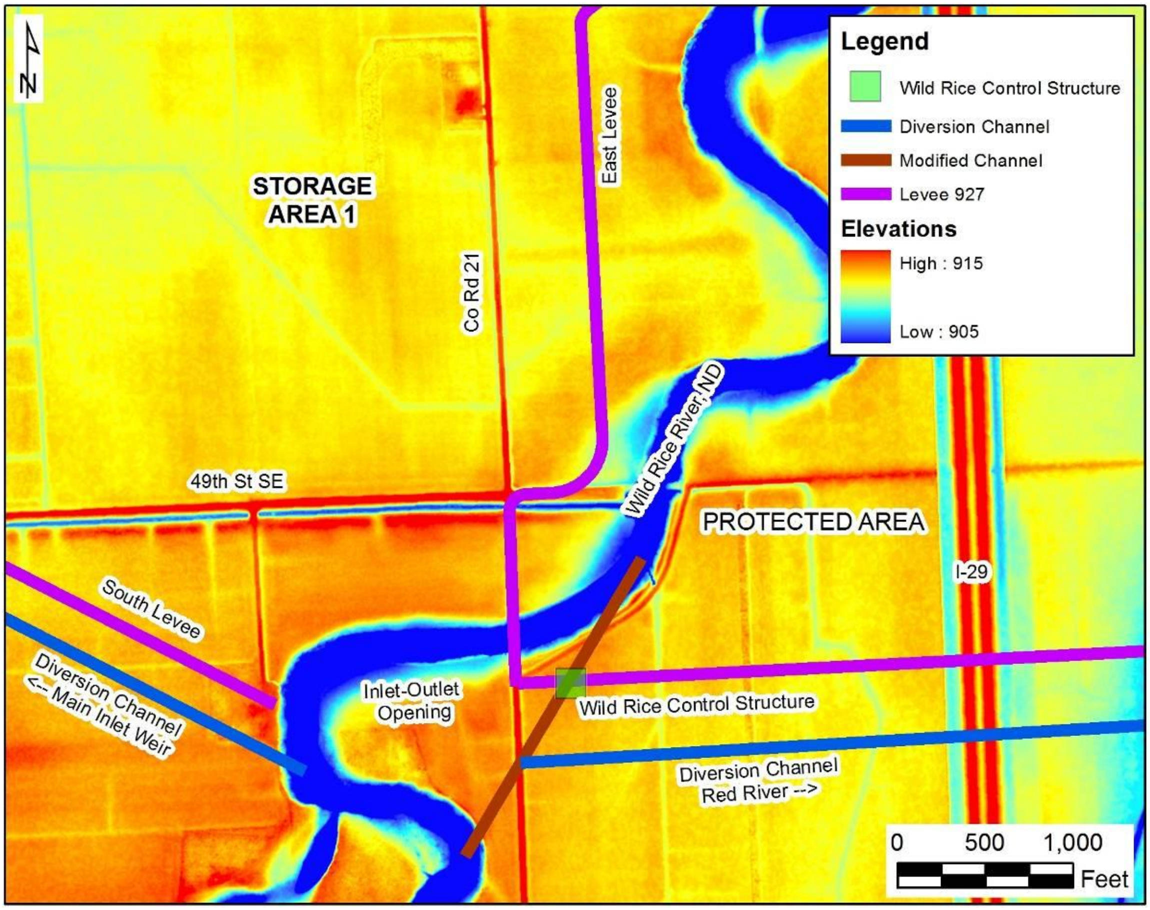

Figure F-E8 Elevation gradient map near Inlet-Outlet Opening for Storage Area 1

The hydraulic model assumes that borrow trenches will bring down the opening elevation to between 910 and 911. it is assumed that some excavation will be required to drain the ditch on the south side of 49th St SE through the Inlet-Outlet Opening to the Wild Rice River. This grading will create the low point where rising flood waters will first enter SA1. Figure F-E9 shows a cross section of the Wild Rice River in the middle of the opening.

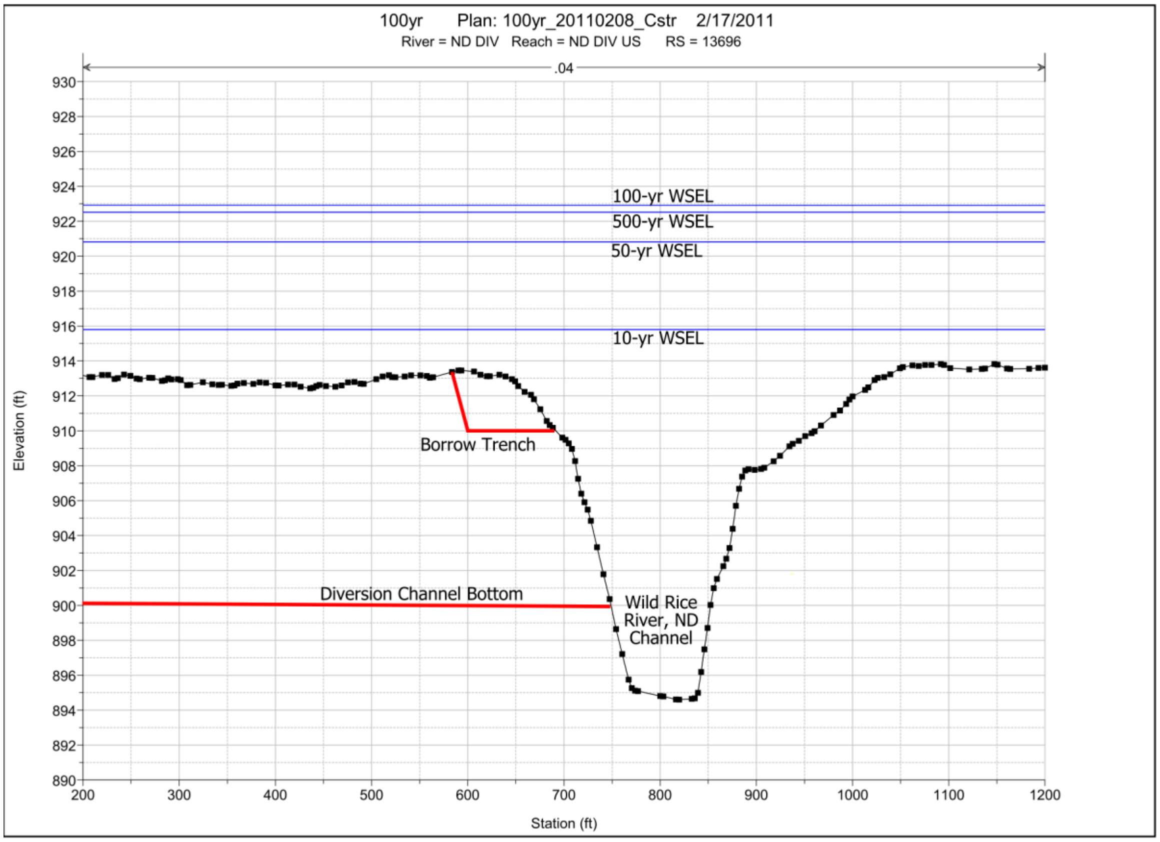

Figure F-E9 Wild Rice River cross-section at Inlet-Outlet Opening

F-E6.6 STORAGE

The storage volume achieved for SA1 is a product of the upstream staging elevation, the existing ground elevation and the footprint of the storage area. Of the available places for storage, the existing ground elevations are lowest on the north side of the Diversion Channel, which corresponds to a larger storage capacity for SA1. As discussed above, the footprint was maximized within the constraints of existing infrastructure.

The depth of staging upstream of the Protected Area is primarily dependent on the operation of the gated control structures on the rivers intersecting the Diversion Channel. Excess water is staged upstream of the gates. Table F-E1 contains the peak stage and storage values for the four design flood events.

Table F-E1 Stage-storage values for design flood events in Storage Area 1

Flood Event (yr)	Peak Stage (ft)	Peak Storage (ac-ft)
10	915.8	24,540
50	920.8	46,370
100	922.9	55,170
500	922.5	53,450

Figure F-E10 presents the data in Table F-E1 graphically in relation to the stage-storage curve for SA1.

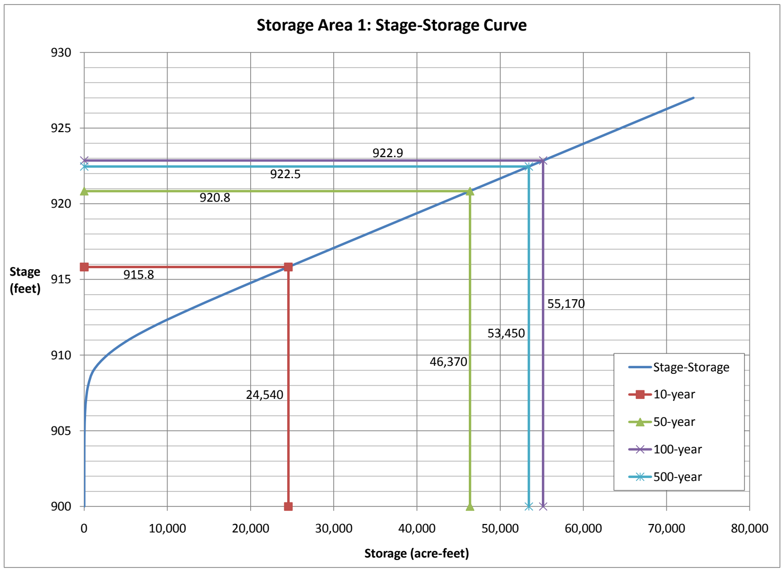

Figure F-E10 Stage-storage provided by Storage Area 1 for the design flood events

Note that the 100-year peak stage is higher than the 500-year peak stage. This phenomenon is caused by the Phase 4 project design objective to have no adverse impact on downstream flood elevations. Since the downstream 500-year floodplain is deeper and wider than the 100-year floodplain, it can receive more water. This corresponds to larger gate openings for the 500-year event than for the 100-year event. As discussed above, the restrictions created by the control structure gates cause water to stage upstream. A smaller opening for a given inflow will result in a higher stage. In this case, the difference in the size of the gate openings is large enough that even with a larger inflow for the 500-year event, the upstream staging requirement is less than that for the 100-year event. For a more detailed discussion of this topic, see Exhibit D Hydraulic Design Computations – Diversion Structures.

The extension of the stage-storage curve to elevation 927 provides an indication as to amount of excess storage available should there ever be a flood that exceeds the design flood events.

F-E6.7 ROADWAY IMPACTS

The footprint of SA1 and the Diversion Channel will have impacts on existing roadways. Roads that intersect the Diversion Channel will either need to bridge the channel or become discontinuous. It is assumed that County Roads through SA1 would be maintain by bringing them up and over the levee embankment, while lesser roads will be interrupted at intersections with the levee. Impacts to specific roads are discussed below.

F-E6.7.1 County Road 21

Co Rd 21 runs north-south on the west side of I-29 (Figure F-E1). There are two major modifications to Co Rd 21 that will occur because of the proposed flood control measures.

The first modification is near the southeast corner of SA1. Under existing conditions, a bridge carries Co Rd 21 over the Wild Rice River. The construction of the Wild Rice Control Structure and adjacent levees will result in the removal of the Co Rd 21 bridge (Figure F-E7). Co Rd 21 will be discontinued from 50th St SE to Co Rd 16.

The second modification is near St. Benedict's Church, where the East Levee jogs to the west in order to avoid impacts to the church and nearby infrastructure (Figure F-E6). Co Rd 21 will be elevated to pass over the levee crest. This means raising the road roughly 15-feet so that it can pass over the levee crest at elevation 927.

F-E6.7.2 Country Road 16

Co Rd 16 runs east-west through SA1 and forms the interchange with I-29 at freeway exit 54 (Figure F-E1). There are three major modifications to Co Rd 16 that will occur because of the proposed flood control measures.

The first modification is at the intersection with the East Levee for SA1, just west of the freeway interchange (Figure F-E8). To maintain continuity for Co Rd 16, the roadway will need to be raised roughly 13 feet so it can pass over the levee crest at elevation 927. Similarly, the intersection with the South Levee for SA1 will mean raising Co Rd 16 roughly 10 feet. Figure F-E11 shows a closer view of the intersection of Co Rd 16 with the South Levee and the Diversion Channel.

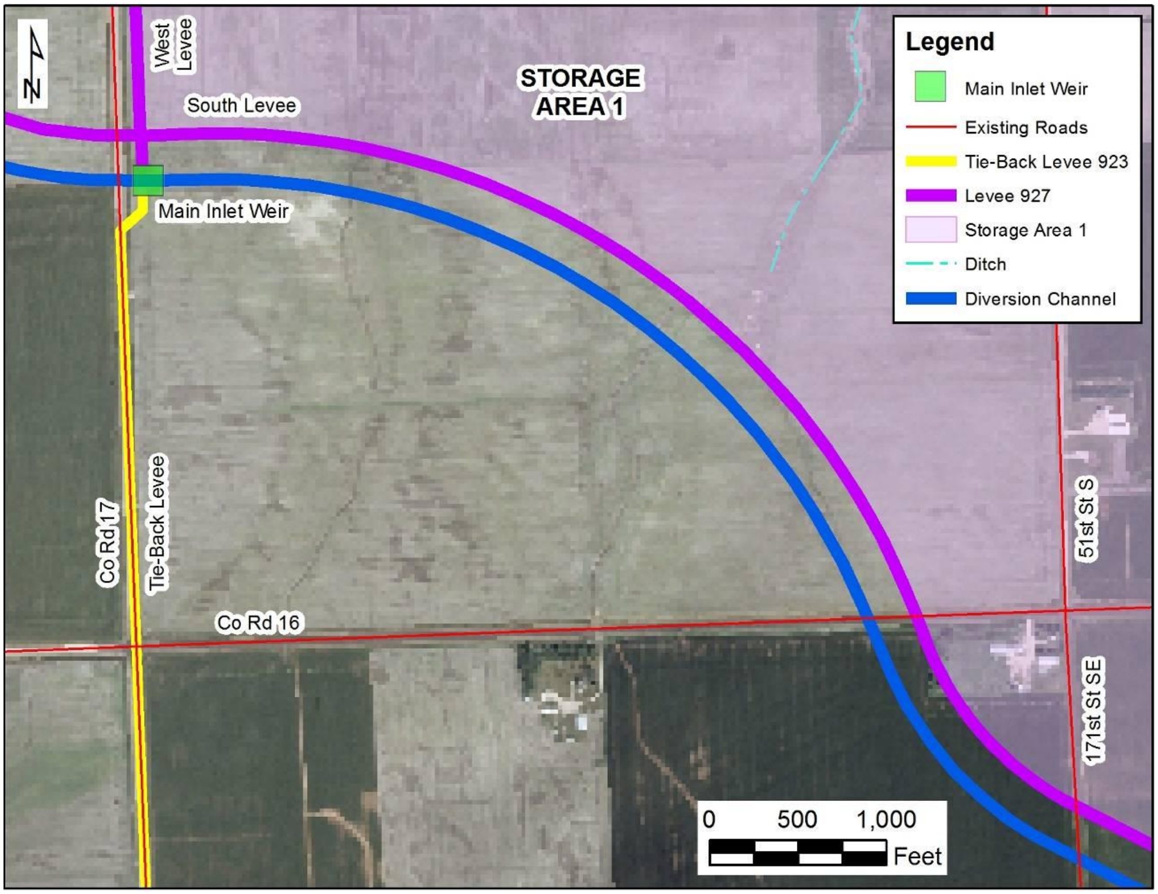

Figure F-E11 Vicinity Map for Main Inlet Weir and Diversion Channel Crossings

The third modification is the intersection of Co Rd 16 with the Diversion Channel (Figure F-E11). To maintain the continuity of Co Rd 16, a bridge will be constructed over the Diversion Channel. Figure F-E4 shows a cross section of the Diversion Channel and adjacent levees near the Co Rd 16 crossing. This reach of the Diversion Channel is not designed to convey or contain the design flood events. Therefore, it is neither necessary or practical to construct a bridge that is above the 100-year or 500-year events. Instead, the bridge deck will be kept near existing grades, and designed to be overtopped during major flood events.

Because peak conveyance of major floods will be primarily in the floodplain, there is no anticipated hydraulic effect on the overall flood control system. As such, the bridge was not included in the hydraulic model. A separate hydraulic analysis for the bridge will be necessary during final design.

F-E6.7.3 County Road 17

Co Rd 17 runs north-south along the west edge of what will be SA1 (Figure F-E1). There are three major modifications of Co Rd 17 that will occur as a result of the proposed flood control measures.

The first modification is the raising of roughly 3 miles of Co Rd 17 to at least elevation 923 to create the west Tie-Back Levee. The purpose of the levee is to limit the extent of the upstream staging area during major flood events. The 923 top elevation is slightly above the modeled peaks staging elevation for the 100-year and 500-year events. At the Diversion Channel the crest of the Tie-Back Levee will be increased 924, so that if the levee is ever overtopped, water will begin spilling over the road several hundred feet south of the Diversion Channel. The elevation of Co Rd 17 will extend approximately 2.5 to 3.0 miles to the south until the existing road is at or above elevation 923. The approximate extent of the Tie-Back Levee can be seen in Figure F-E1.

Figure F-E12 shows a cross section for the Diversion Channel along the alignment for Co Rd 17. The schematic showing the Tie-Back Levee is on the left overbank and shows how the Levee gets higher as it approaches the Diversion Channel.

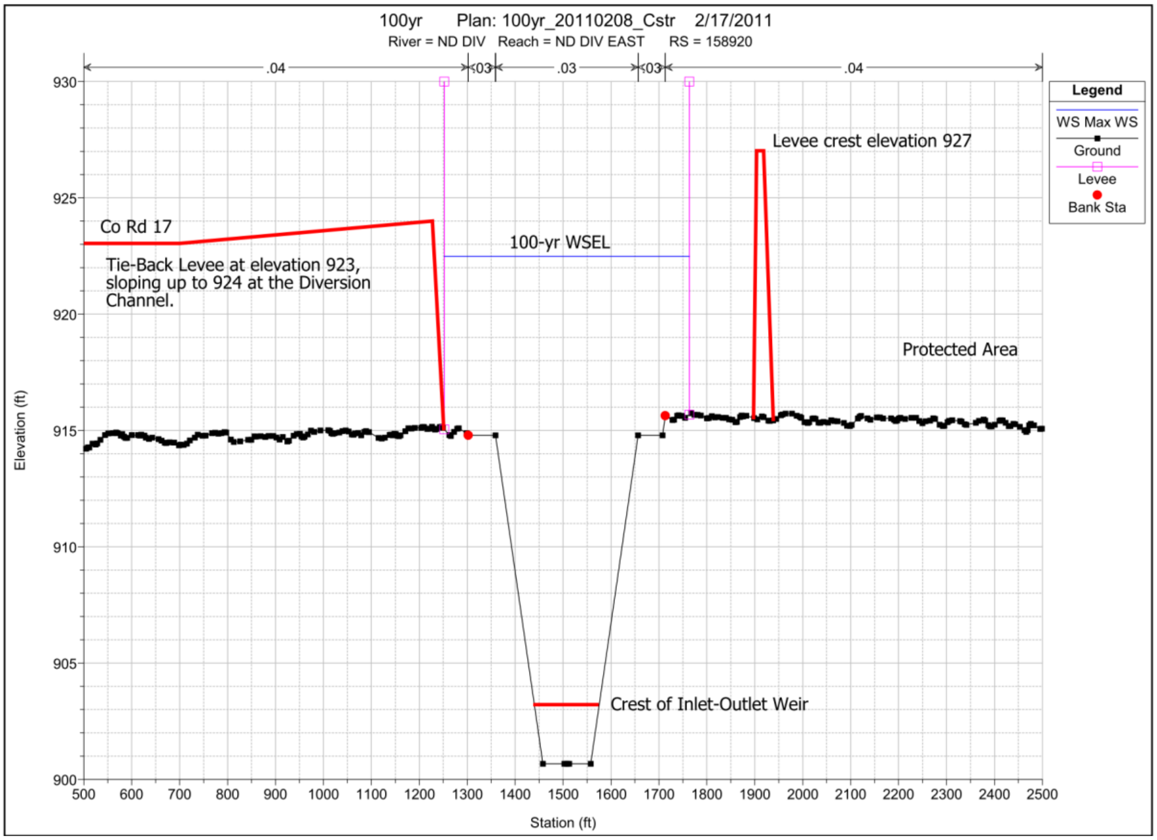

Figure F-E12 Diversion Channel cross-section at Tie-Back Levee

The second modification is a bridge crossing the diversion channel near the southwest corner of what will be SA1 (Figure F-E11). For preliminary design and cost estimating purposes, this bridge was considered an independent structure that would cross the Diversion Channel downstream of the Main Inlet Weir. As discussed above, the Main Inlet Weir is where staged waters upstream of the Protected Area are funneled into the main reach of the Diversion Channel that is designed to contain the design flood events within its banks. The Co Rd 17 Bridge will be constructed such that it is above the 100-year and 500-year water surface elevations and does not otherwise create a constriction for flows in the Diversion Channel. For final design it is anticipated that cost savings could be achieved by combining structural elements of the Main Inlet Weir and the Co Rd 17 Bridge.

The third modification to Co Rd 17 will be raising the roadway so that it clears the levee crest at elevation 927 on the north side of the Diversion Channel (Figure F-E12).

F-E6.7.4 Minor Roads

Construction of SA1 and adjacent flood control infrastructure will interrupt several minor roads. Table F-E3 lists the affected roads. It will be up to the local jurisdictions to determine if these roads will continue to be maintained within SA1. The locations of these roads are shown on in Figure F-E1.

Table F-E2 Effects on minor roads due to proposed flood control measures

Road	**Impact**
112th Ave S	Interrupted by East Levee Interrupted by West Levee
172nd Ave SE	Interrupted by Diversion Channel and South Levee Interrupted by North Levee
171st St Ave SE / 57th St S	Interrupted by Diversion Channel and South Levee Interrupted by North Levee
49th St SE	Interrupted by Diversion Channel and South Levee

Some of these roads may still need to be maintained in order to provide maintenance access to flood control structures. For example, 57th St. S off of Co Rd 14 would be a convenient access point for the North Outlet (Figure F-E5). It is assumed that fields within SA1 will continue to be farmed, which is an additional reason for maintaining minor roads within the storage area.

F-E7.0 SUMMARY

Storage Area 1 is a 4360 acre area on the north side of the Channel between the Wild Rice River and the Sheyenne River in North Dakota. It will be surrounded by a levee at elevation 927 which provides 4-feet of freeboard above the 100-year and 500-year flood events. It will provide over 55,000 acre-feet of storage during the 100-year and 500-year flood events.

There will be three openings in the levees surrounding Storage Area 1. The Inlet-Outlet Opening will be a 1400-foot gap in the South Levee near the Wild Rice Control Structure. The North Outlet and East Outlet will both be gated structures designed to be left open to allow for natural drainage during non-flood conditions, and closed during major flood events.

County Roads 16 and 21 will be maintained through the storage area, but minor roads will be interrupted by the levee embankments.

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APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT F – HYDRAULIC DESIGN COMPUTATIONS – DRAINS (FLOODPLAIN IMPACTS TO THE WEST)

Report for the US Army Corps of Engineers, and the cities of Fargo, ND & Moorhead, MN

By: Barr Engineering Co.

FINAL – February 28, 2011

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F-F1.0 DRAIN 14

A description of the drop structure at Drain 14 can be found in Appendix F. This portion of Exhibit F presents a comparison of water surface elevations at Drain 14 structure and design calculations of the drop structure.

F-F1.1 COMPARISON OF LPP DIVERSION CHANNEL AND DRAIN 14 HYDROLOGY

Flows, water surface elevations, and average velocities for the Diversion Channel at the Drain 14 drop structure for the 10-, 50-, 100-, and 500-year events are presented in Table F-F1. The location shown in the tables is Station 78110 which is where flows from Drain 14 discharge to the Diversion Channel. Water surface elevations in the Diversion Channel at the Drain 14 structure, in Drain 14 for local tributary flood events, and in Drain 14 for coincidental flood events, plotted against the cross section data, are presented in Figures F-F1 and F-F2.

Table F-F1 Phase 4 Hydrology in the Diversion Channel Downstream of the Drain 14 Structure (Station 78110)

Event (yr)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
10	7,545	887.00	1.93
50	19,675	895.53	2.36
100	24,785	898.36	2.46
500	25,074	901.10	2.08

* Flow and velocity information correspond to same time that the maximum water surface elevation occurs and may not be the maximum flow or velocity over the duration of the simulation

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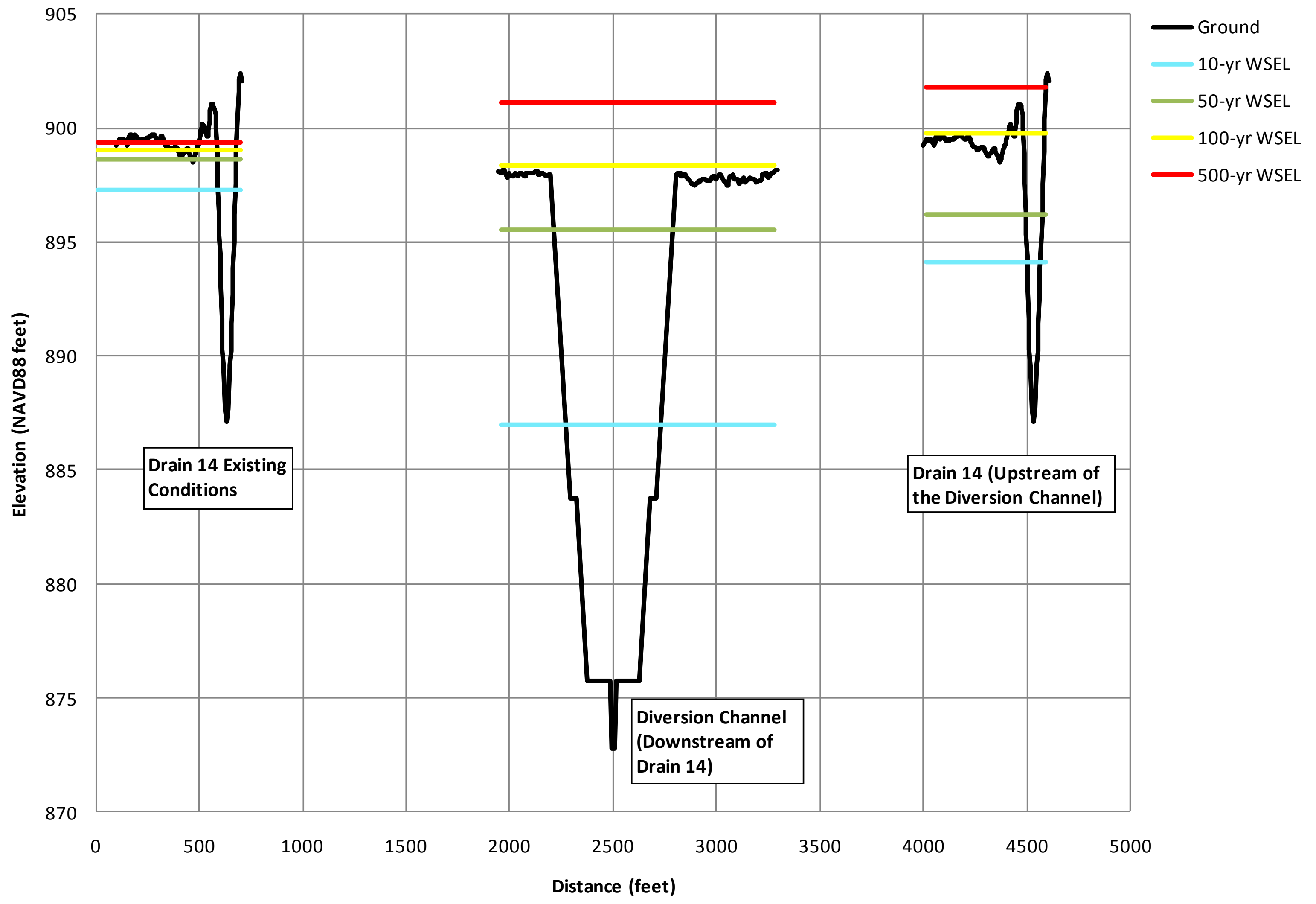

Figure F-F1 Phase IV Hydrology in the Diversion Channel at the Drain 14 Structure and in Drain 14

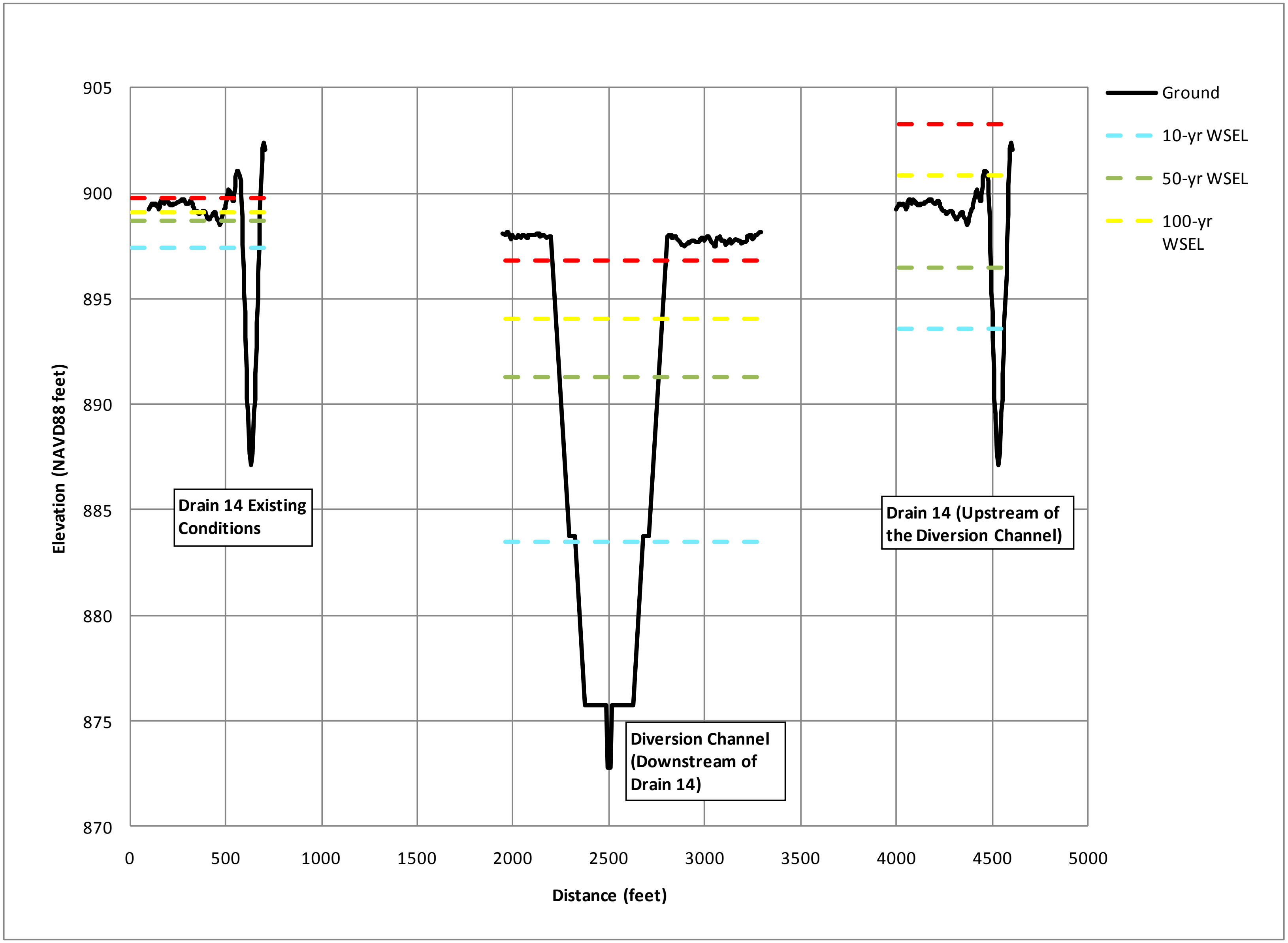

Figure F-F2 Phase IV Local Hydrology in the Diversion Channel at the Drain 14 Structure and in Drain 14

F-F1.2 DESIGN OF THE DRAIN 14 DROP STRUCTURE

The stepped concrete drop spillway was designed to pass the 500-year local event which is approximately 1,200 cfs larger than the 500-year coincidental event. For smaller more frequent events, less than the 50-year event, the water surface elevation in the Diversion Channel remains below the outlet from Drain 14. For larger flow events, the water surface elevation in the Diversion Channel is above the spillcrest of the Drain 14 drop structure, which results in an area of local staging on Drain 14. The water surface profiles upstream of the Drain 14 drop structure for existing conditions and With-Project conditions are shown in Figures F-F3 to F-F6 for the coincidental events and Figures F-F7 to F-F10 for the local events.

The width of the Drain 14 stepped drop structure is 30 feet. The drop structure is designed to handle the 500-yr local event with a 100 foot stilling basin. The stilling basin length is dependent on the length of the hydraulic jump. Dimensions of the Drain 14 stepped spillway are outlined in Table F-F2. The calculations used to determine the 30 foot width of the stepped drop structure are presented in Table F-F3.

Table F-F2 Drain 14 Stepped Spillway Parameters

Tributary bed invert elevation	888.0	Ft
Diversion channel bed elevation	873.26	Ft
Diversion channel 500-yr flood elevation (local on the Tributaries)	903.27	Ft
Diversion channel 10-yr flood elevation (local on the Tributaries)	883.47	Ft
10-yr flood elevation on Drain 14	893.55	Ft
WSEL difference between the 10-yr local event on Drain 14 and the Diversion Channel bed invert	20.3	Ft
Crest of steps	888.0	Ft
Number of steps	21	
Height of each step	0.7	Ft
Length of each step	1.5	Ft
Total height of steps	14.7	Ft
Total length of steps	31.6	Ft
Spillway width	30	Ft

Table F-F3 Flow Regime Over the Steps and Stilling Basin Parameters of the Drain 14 Stepped Spillway with a 30 Foot Width

Local Event (yr)	Flow in Drain 14 (cfs)	Critical Depth (ft)	Flow Regime	Water Depth Upstream of Hydraulic Jump (ft)*	Water Depth Down-stream of Hydraulic Jump (ft)*	Velocity Down-stream of Hydraulic Jump (ft/s)	Length of Hydraulic Jump (ft)
10	1,608	4.47	skim	1.5	10.1	5.3	60
50	2,463	5.94	skim	2.2	12.7	6.5	76
100	4,001	8.21	skim	3.4	16.4	8.2	98
500	4,280	8.58	skim	3.6	16.9	8.4	*102*

* Tailwater effects were not incorporated into the sizing of the stilling basin.

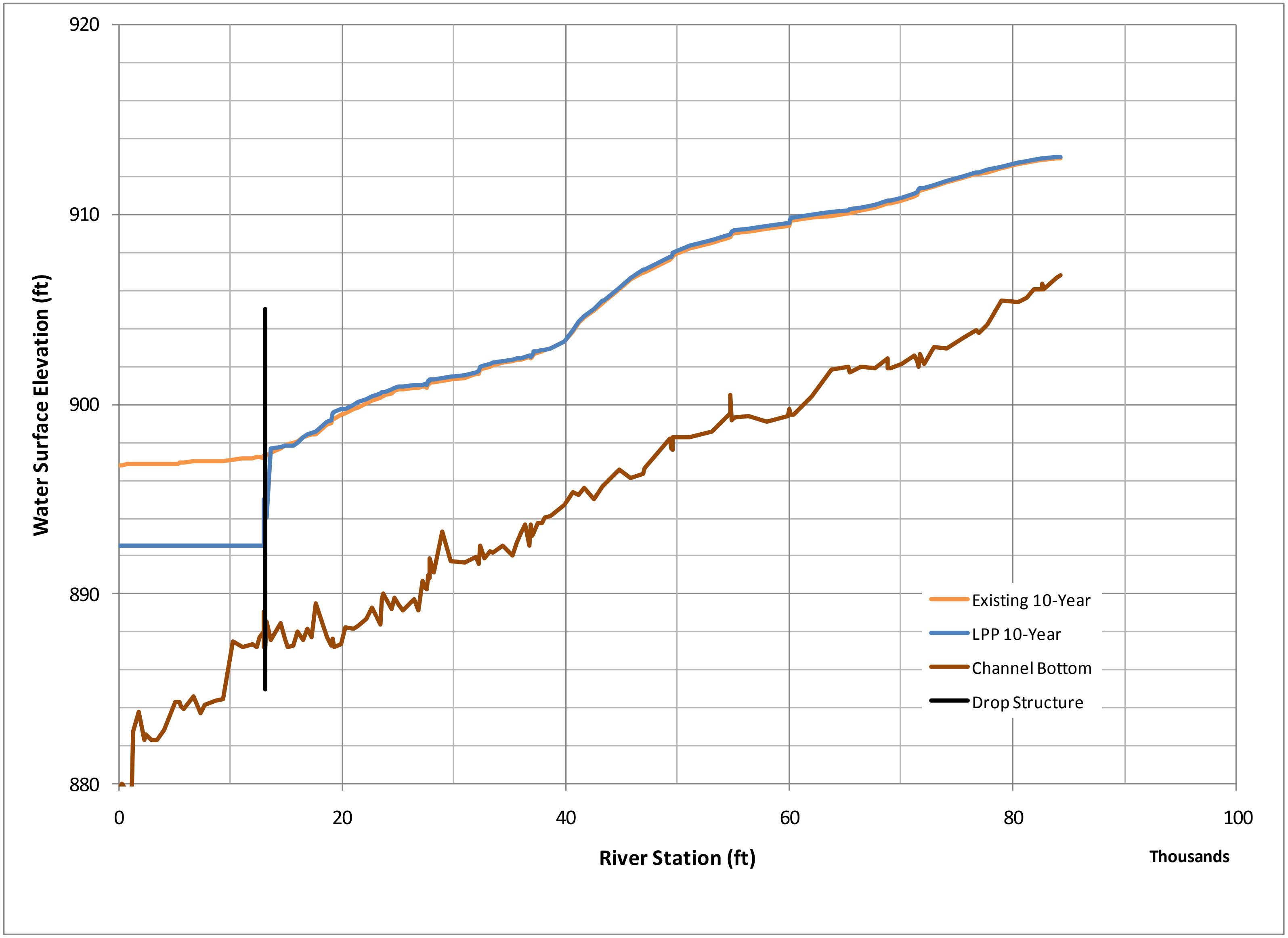

Figure F-F3 **Water Surface Profile Comparison of Existing and With-Project Conditions for the 10-Year Coincidental Event**

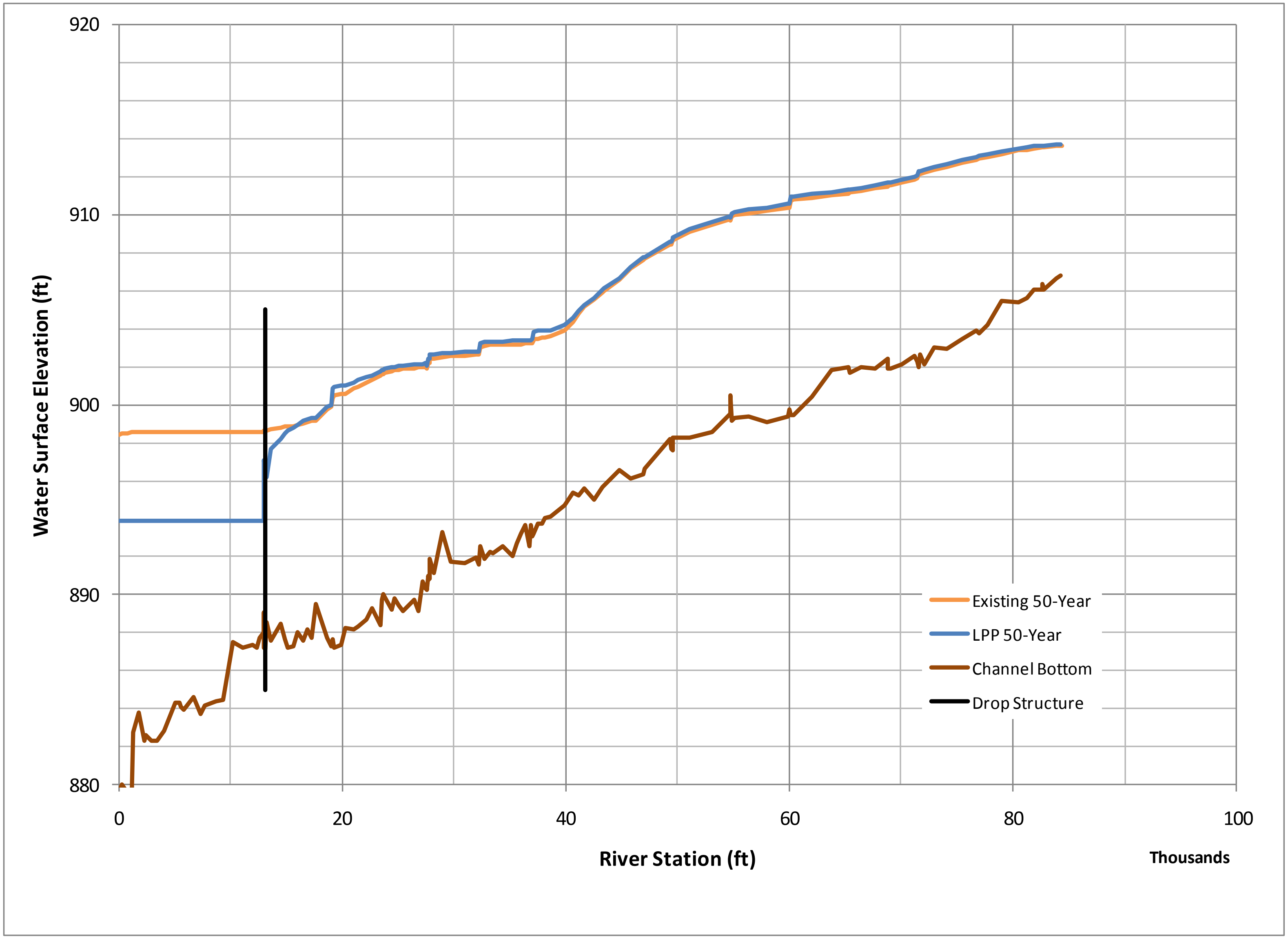

Figure F-F4 Water Surface Profile Comparison of Existing and With-Project Conditions for the 50-Year Coincidental Event

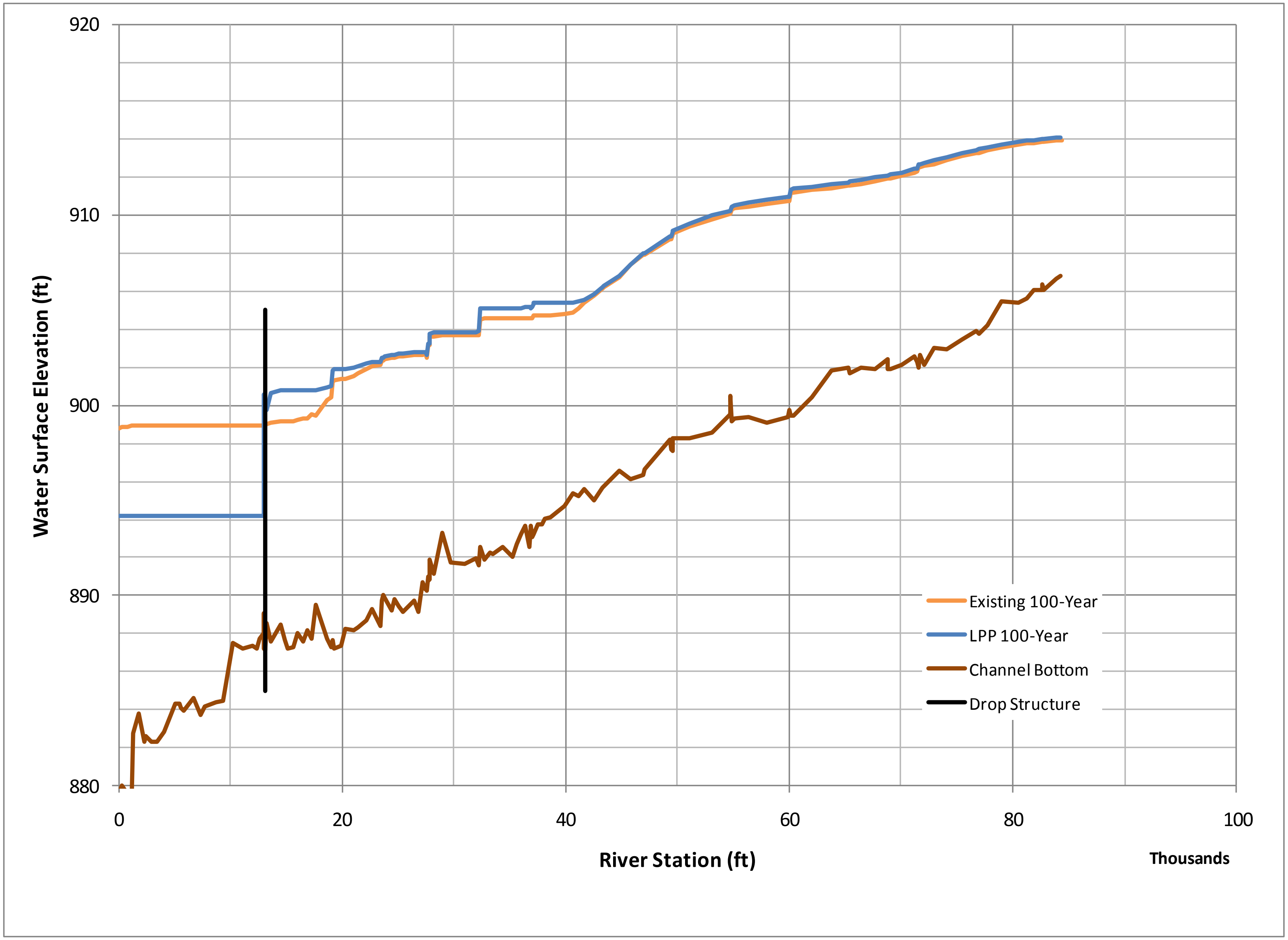

Figure F-F5 Water Surface Profile Comparison of Existing and With-Project Conditions for the 100-Year Coincidental Event

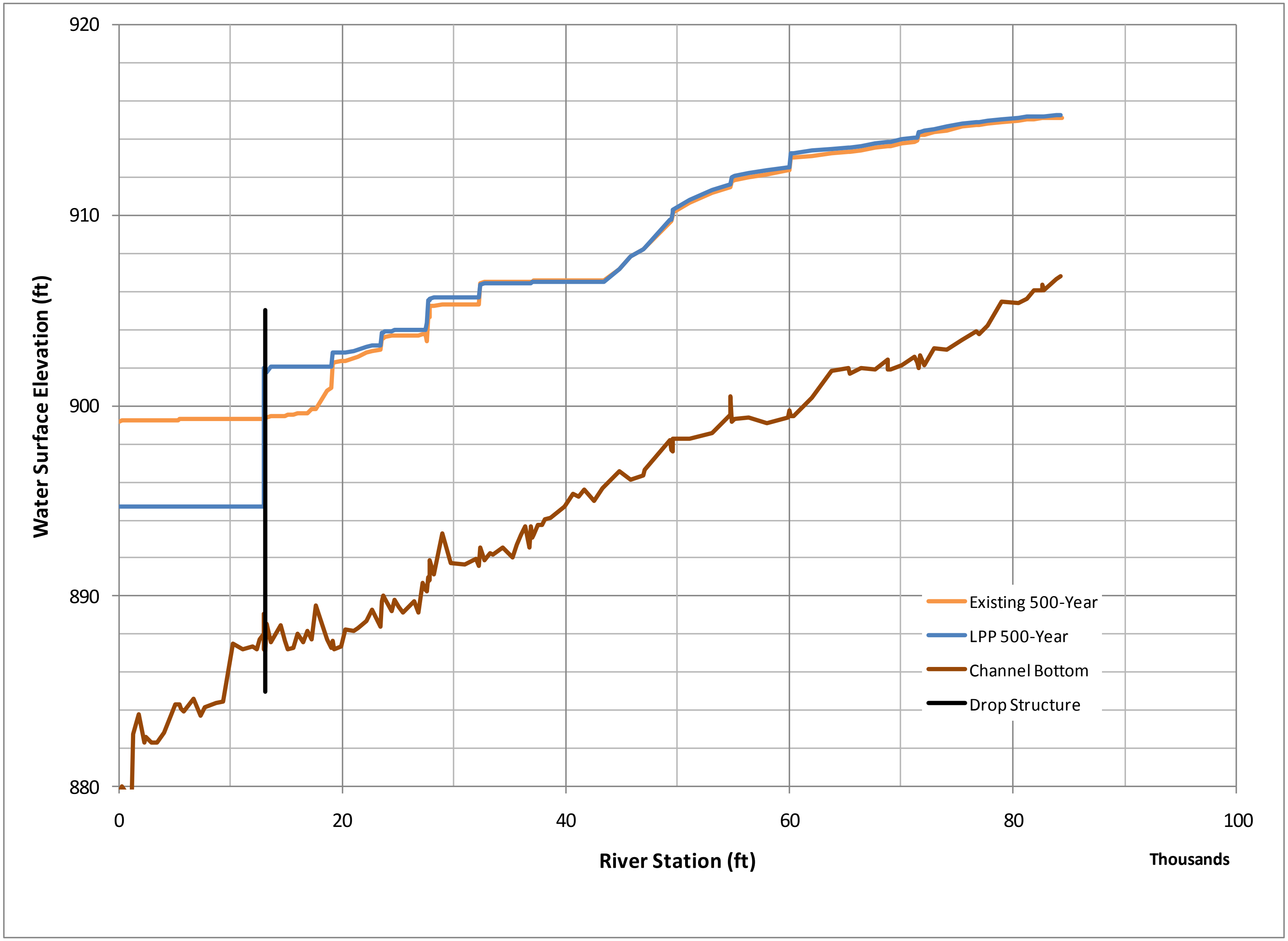

Figure F-F6 Water Surface Profile Comparison of Existing and With-Project Conditions for the 500-Year Coincidental Event

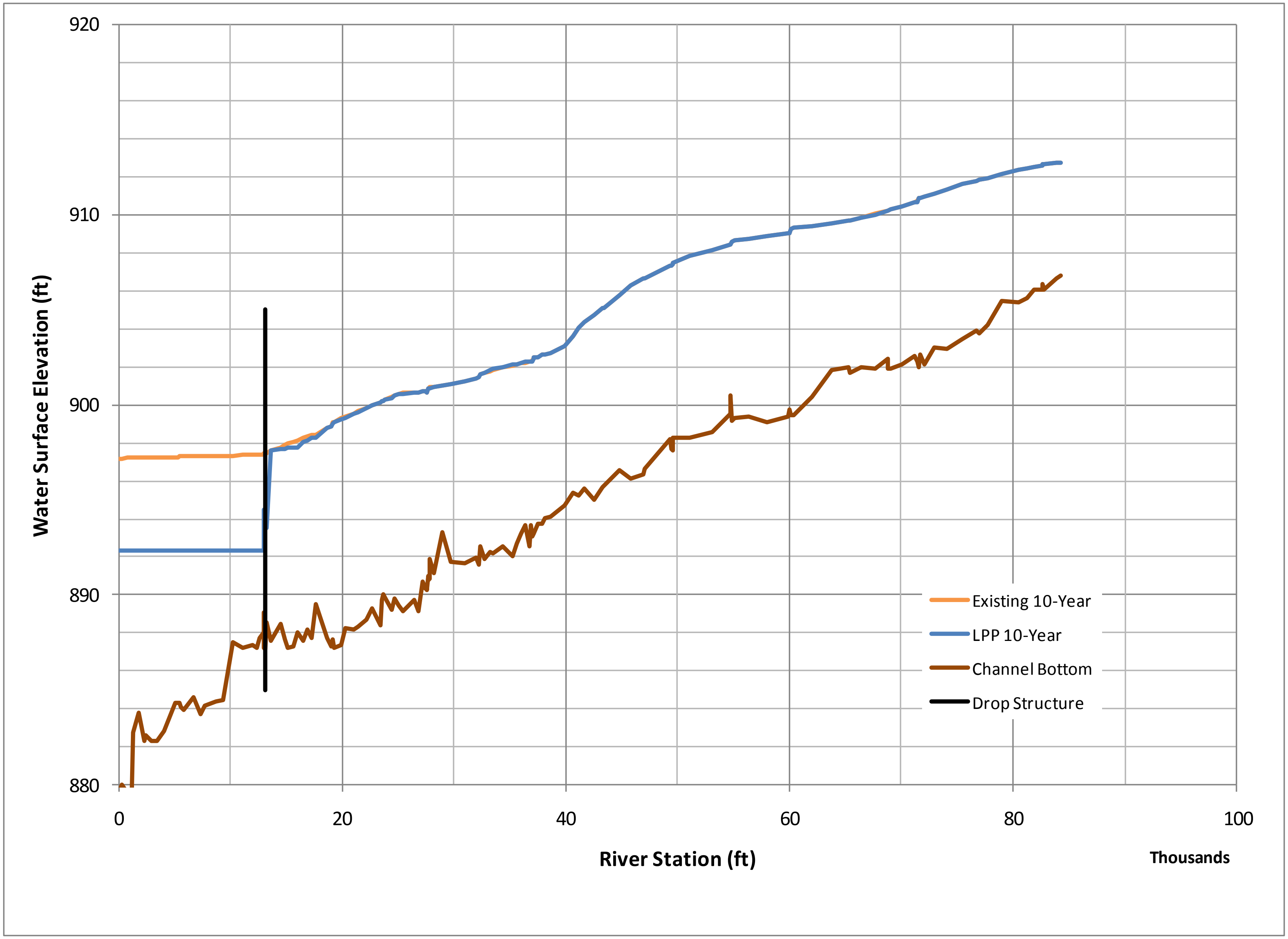

Figure F-F7 Water Surface Profile Comparison of Existing and With-Project Conditions for the 10-Year Local Event

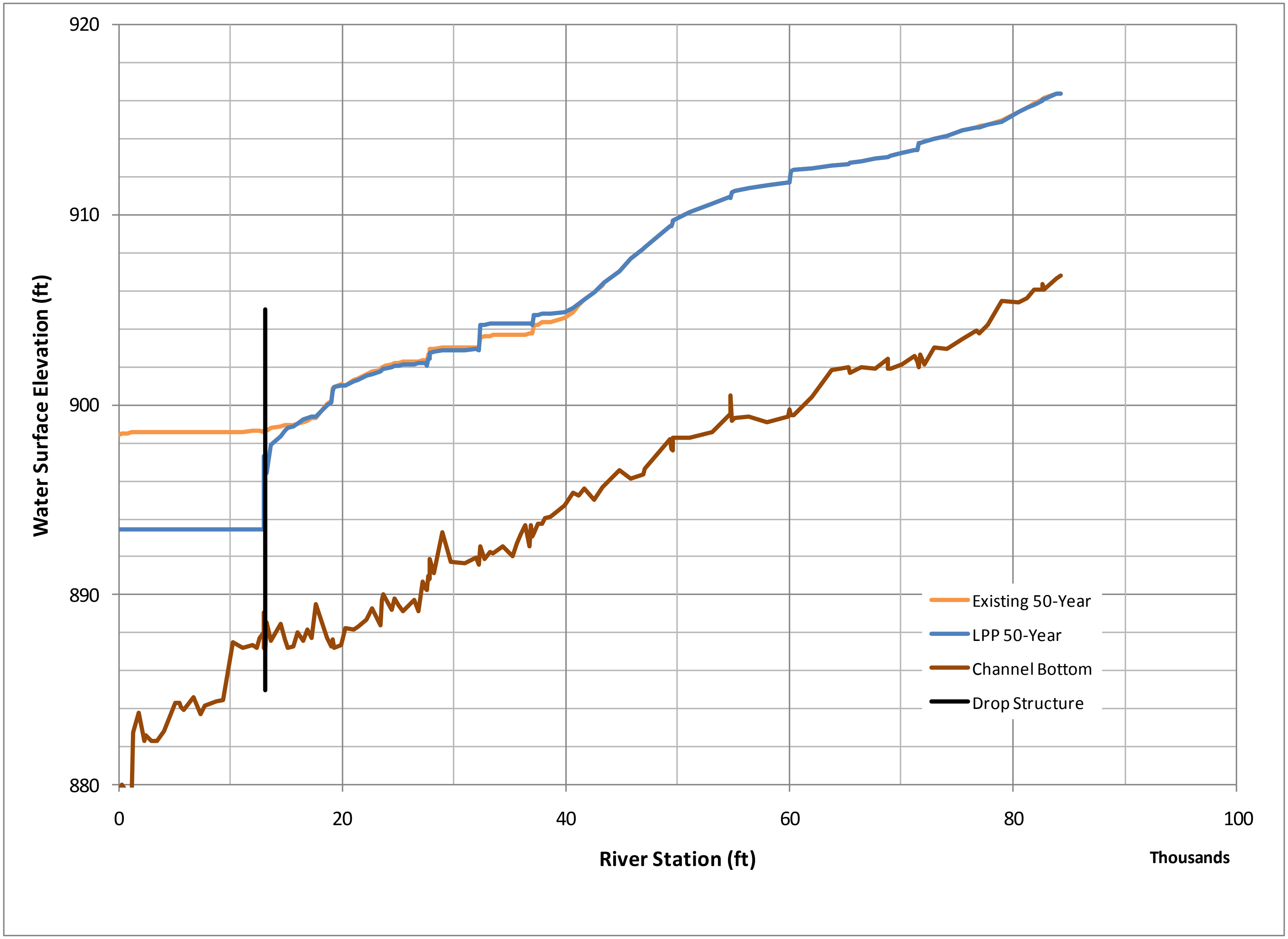

Figure F-F8 Water Surface Profile Comparison of Existing and With-Project Conditions for the 50-Year Local Event

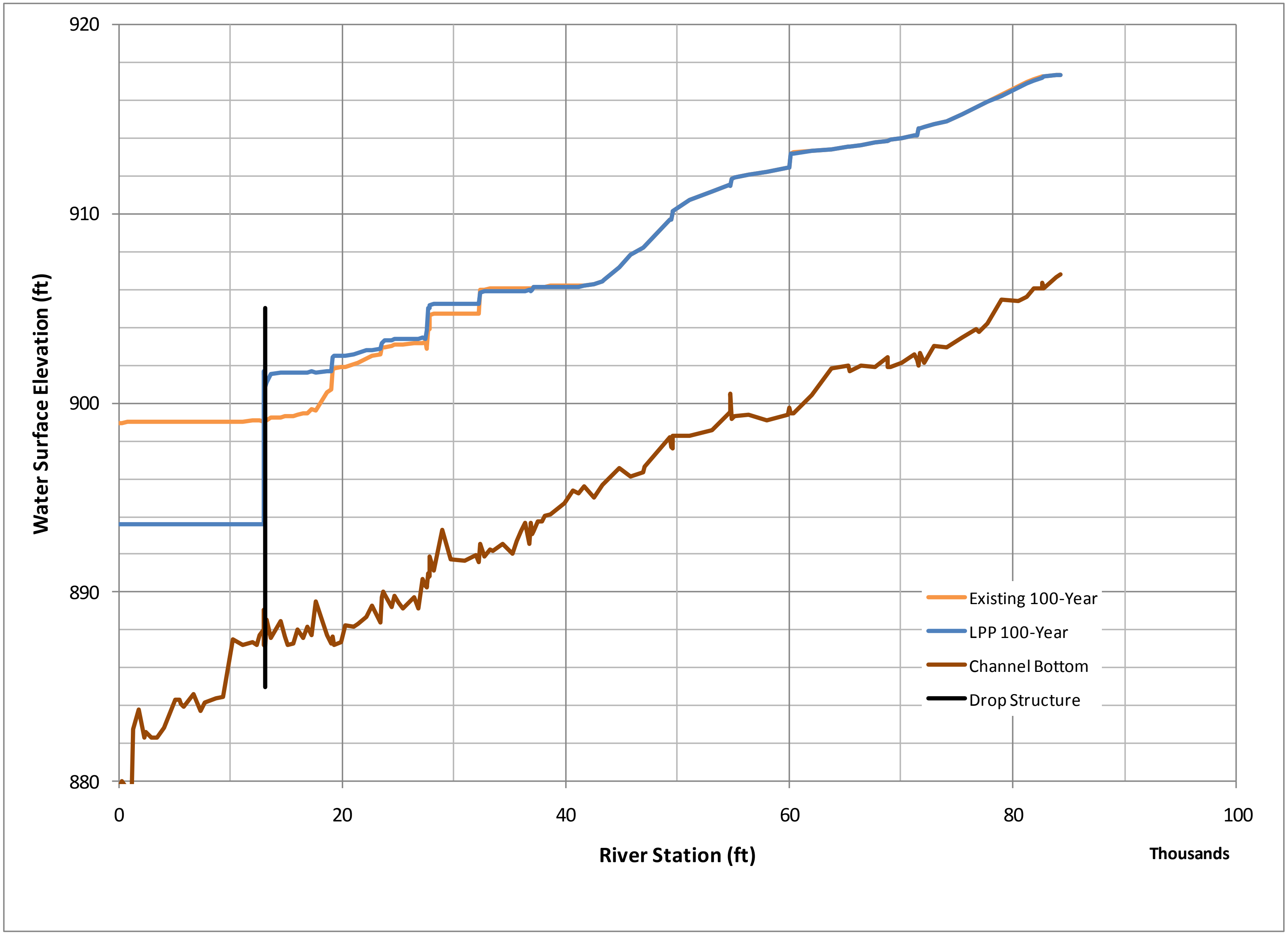

Figure F-F9 Water Surface Profile Comparison of Existing and With-Project Conditions for the 100-Year Local Event

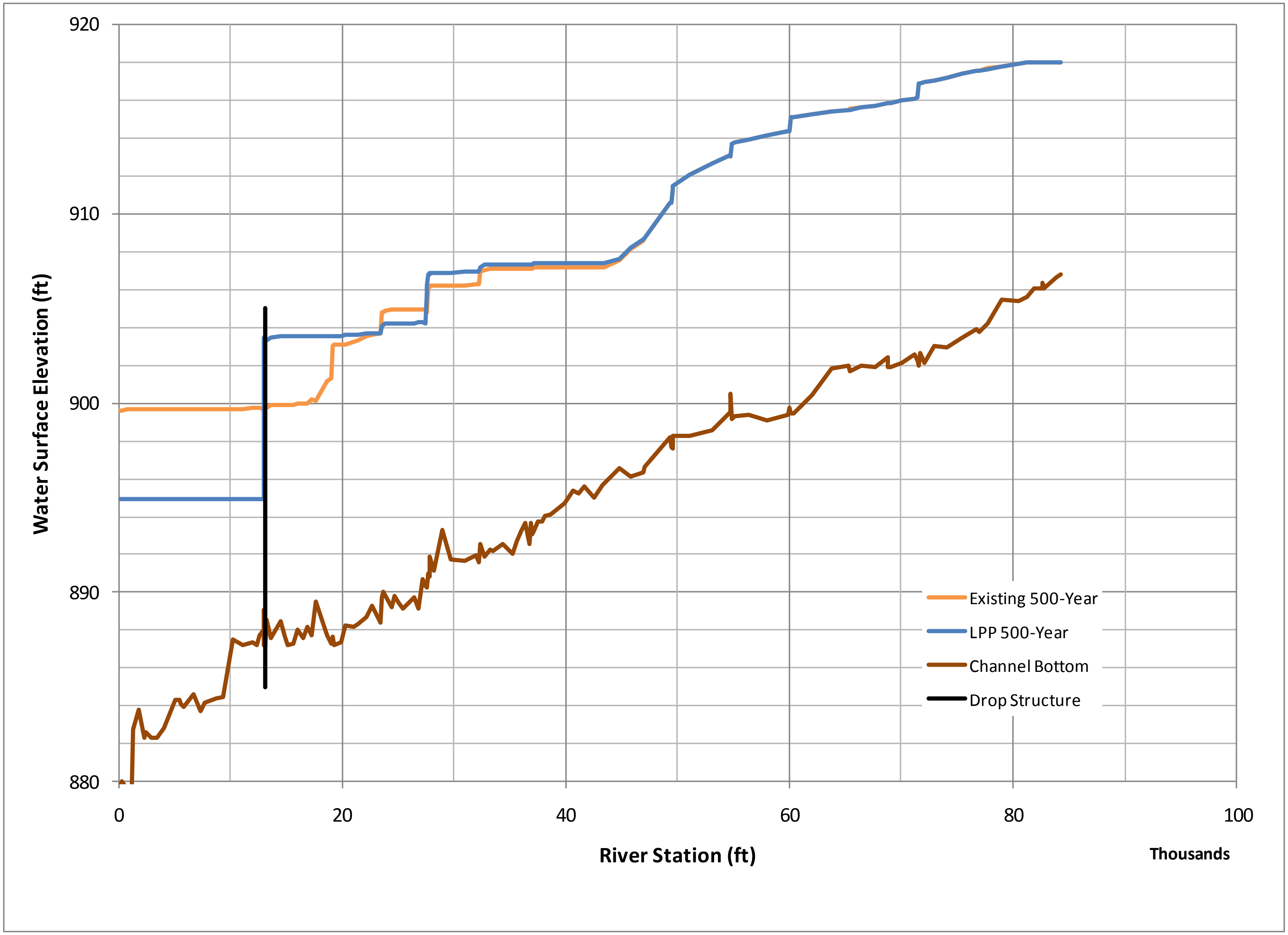

Figure F-F10 Water Surface Profile Comparison of Existing and With-Project Conditions for the 500-YearLocal Event

F-F2.0 LOCAL DRAINAGE

A description of the local drainage inlets to the LPP Diversion Channel can be found in Appendix A of the Phase 3 report submitted July 30th, 2010. This includes other major drainage ditches, in addition to Drain 14, such as Drain 21C and Drains 13 and 30 as well as other minor unnamed local drainage ditches. This portion of Exhibit F presents a summary of the local discharge locations into the Diversion Channel.

Local drainage inlets into the Diversion Channel were not resized as part of the Phase 4 analysis. During development of the HEC-RAS unsteady flow model, the outlet elevations for local drainage inlets assumed during previous phases were verified to ensure that they were above the existing topography, and conveyed drainage from the adjacent storage areas in the HEC-RAS unsteady flow model. Impacts to the floodplain upstream of each of these drainage locations were verified and are discussed in Section F-F3.0. Table F-F4 includes a summary of the local discharge locations included in the Phase 4 HEC-RAS unsteady flow model.

Table F-F4 Summary of Local Drains that Discharge into the LPP Diversion Channel

HEC-RAS Station	Bank[1]	Structure Type	Diameter (ft)	Number of Culverts[2]
8322	L	Circular Culvert	5.5	1
50657	L	Circular Culvert	6	1
54245	L	Circular Culvert	4.5	1
64646	L	Circular Culvert	4.5	1
69218	L	Circular Culvert	3.5	1
73260	L	Circular Culvert	3.5	1
77104	L	Circular Culvert	4.5	2
89561	L	Circular Culvert	4	1
92149	L	Circular Culvert	5.5	2
95154	L	Circular Culvert	4	1
99921	L	Circular Culvert	3	1
101491	L	Circular Culvert	6	1
105869	L	Circular Culvert	3.5	1
108024	L	Circular Culvert	3.5	1
110137	L	Circular Culvert	6	3
114743	R	Circular Culvert	4	1
118893	R	Circular Culvert	3.5	1
119402	L	Circular Culvert	4.5	2
119407	R	Circular Culvert	4	1
123907	R	Circular Culvert	4	1
125423	L	Box Culvert 17'x25'	NA	1
130363	L	Circular Culvert	4	1
136016	L	Circular Culvert	3.5	1

[1] Facing downstream

[2] If more than one culvert is required, the station is defined as the center location of the culverts

F-F3.0 FLOODPLAIN IMPACTS TO THE WEST

As part of the Phase 4 analysis, impacts to the floodplain west of the Diversion Structure, between the Sheyenne River and Maple River were characterized to quantify the affect the Diversion Channel has on the existing floodplain. This portion of Exhibit F presents a summary of the impacts to the 100-year floodplain to the west of the Diversion Channel.

Based on discussions with the USACE the 100-year local event on the tributaries was selected to characterize impacts to the floodplain west of the Diversion Channel. The HEC-RAS unsteady flow model developed for Phase 4 was used with local 100-year hydrographs on the tributaries provided by the USACE for this analysis. Figure F-F11 presents how the floodplain west of the Diversion Channel between the Sheyenne River and the Maple River is impacted by the Project.

In general the majority of the floodplain located to the west of the Diversion Channel is not impacted by the Project. There are only 10 locations not immediately adjacent to the Diversion Channel where the floodplain is altered by more that 1ft.

The portion of floodplain most impacted by the project is located along Drain 14, immediately upstream of the Diversion Channel. In this location the water surface elevation in the Diversion Channel is higher than the spillcrest of Drain 14, and water from the Diversion Channel flows into Drain 14 resulting in an increase to the 100-year flood elevation in this area. Potential methods to mitigate this increase in flood elevations in this area could include

1. Constructing a backflow prevention device on the Drain 14 outlet
2. Grading an overflow to the Maple River
3. Grading an overflow to the local drains to the south

Methods for addressing impacts in this location were not analyzed during the Phase 4, but should be completed during final design.

Additional impacted areas along the Sheyenne River and Maple River where 100-year flood elevations decrease are due to grade control not being included in the HEC-RAS unsteady flow model. Grade control structures are included in the Project cost estimates, but were not included in the hydraulic modeling completed during Phase 4. Finally, other minor decreases in flood elevation immediately west of the Diversion Channel may be attributed to local drainage discharging immediately into the Diversion Channel. The HEC-RAS unsteady flow model developed for Phase 4 does not include a berm on the west side of the Diversion Channel to control how local drainage enters the Diversion Channel. This is a project feature that should be given additional consideration during final design.

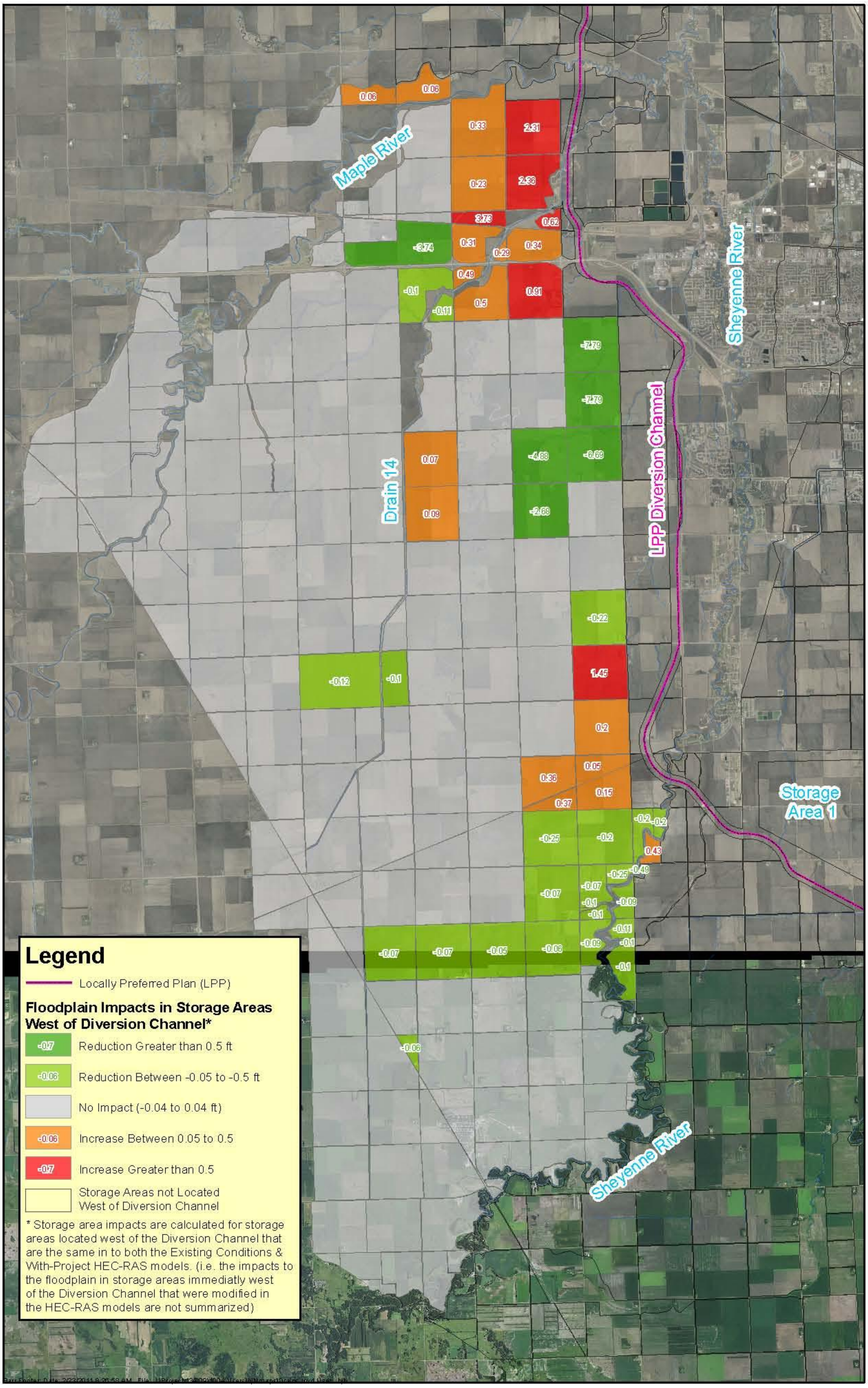

Figure F-F11 Floodplain Impacts West of the Diversion Channel between the Sheyenne River and Maple River for the 100-Year Local Tributary Event

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APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT G – HYDRAULIC DESIGN COMPUTATIONS - FISH PASSAGE

Report for the US Army Corps of Engineers, and the cities of Fargo, ND and Moorhead, MN

By: Barr Engineering Co.

FINAL – February 28, 2011

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EXHIBIT G – HYDRAULIC DESIGN COMPUTATIONS – FISH PASSAGE

F-G1.0 FISH PASSAGE DESIGN

F-G1.1 INTRODUCTION

This exhibit describes the design of fish passages at the proposed control structures on the Red River, Wild Rice River, Rush River, and Lower Rush River. At the Wild Rice River and Red River control structures, the fish passages are designed to allow fish to move from the protected areas of the Wild Rice River and Red River to the upstream side of the respective control structures. At the Rush River and Lower Rush River control structures, the fish passages are designed to allow fish within the Diversion to pass into the upstream sides of the Rush and Lower Rush Rivers, as well as to allow low flows in the Rush and Lower Rush Rivers to drain into the Diversion Channel when water surface elevations are below the invert of the drop structures. Fish passages have not been designed for the Sheyenne or Maple Rivers as the aqueduct allows for adequate fish passage. Fish passages over the Main Inlet (Locally Preferred Plan, or LPP), and diversion inlet weir (Federally Comparable Plan, or FCP) do not include additional structures.

At the Red River Control Structure (FCP and LPP) and Wild Rice Control Structure (LPP), fish passages have been designed to function between the 5-year and 50-year events on the Red River. The Red River and Wild Rice River control structures are designed such that fish may pass through the primary control structures during events less than the 5-year event on the Red River. For larger events, fish passage structures next to the control structures are necessary, as the velocities through the control structures are expected to inhibit upstream fish movement. At the Rush River and Lower Rush River drop structures, fish passage will function for flows less than approximately that which is coincidental to the 10-year event on the Red River.

This exhibit describes the design of preliminary fish passages at several locations along the diversion channel. The design is similar for all fish passage locations, although the number, length, and elevations of fish passage structures vary between sites.

F-G1.2 DESIGN CONSIDERATIONS

The fish passage designs presented in this section are guided by several design considerations based on discussions with the USACE and the USACE’s Lock and Dam 22 Fish Passage Improvement Project Implementation Report – Appendix H. These design goals include:

- Design incorporating pool-riffle sequences
- Maximum velocity through riffles of 6 feet per second

- Average velocity through pools of ~1.5 feet per second
- Minimum depth of 5 feet in pools
- Average slope of 1 to 3 percent
- Minimum depth of 1 foot at entrance to fish passage
- Desired flow of 1-2 percent of the flow through the control structure
- Downstream invert ~1.5 feet below the 5-year tailwater elevation

The designs presented in this section meet these goals with the exception of the desired flow through the fish passage structure. The extreme variability of the upstream water surface elevation creates a wide range of flows through the fish passage structures, including flows less than 1 percent of the total flow as well as flows greater than 2 percent of the total flow through the control structure.

At the Rush and Lower Rush River, the fish passage was also designed to allow low flows in the Rush and Lower Rush River to pass into the Diversion Channel without cresting the drop structures (preventing the pooling of water at these structures).

The above parameters have remained unchanged from the Phase 3 analysis and design. However, changes in the general design and modeling of the diversion have required an update in the fish passage design. The changes in the fish passage design can be attributed to the following:

- An increase in the diversion bottom elevation due to the bottom width change from 100-ft to 250-ft,
- Implementation of a new hydrologic model (unsteady flow modeling and inclusion of storage areas) has changed the predicted water surface elevations,
- Greater upstream staging and maintained water surface elevations in the protected area have created larger water surface elevation differences across the control structures

F-G1.3 DESIGN CONCEPT – RED RIVER AND WILD RICE RIVER FISH PASSAGE

The fish passage design at each control structure includes one or more parallel sequences of pools and riffles (see Appendix F), referred to in this section as “channels”. A one foot drop in elevation occurs in the riffle between pools. The number of pools and riffles is based on the difference in headwater and tailwater elevations at each structure. The number of channels necessary is based on the variability in headwater elevations. The headwater and tailwater elevations used for fish passage design are presented in Table F-G1.

Table F-G1 Headwater and Tailwater Elevations for Fish Passage Design

Plan	Location/Control Structure	10-year Event (on Red River)			50-year Event (on Red River)		
		Headwater (ft)	Tailwater (ft)	HW-TW (ft)	Headwater (ft)	Tailwater (ft)	HW-TW (ft)
FCP	Red River	900.17	898.60	1.57	909.33	899.43	9.90
LPP	Red River	916.29	904.40	11.9	920.86	905.15	15.7
LPP	Wild Rice River	915.61	901.43	14.2	920.84	900.73	20.1

Multiple fish passage channels are necessary at the Red River Control Structure and Wild Rice Control Structure to operate between the 5-year and 50-year events due to headwater elevations that vary by several feet. As the headwater rises, the velocity in the channel increases. At some point, the headwater results in velocities too high for fish passage. At this elevation, flow through the channel is prevented by closing gates at the upstream end of the channel, and flow into another channel with a higher upstream invert elevation is allowed by opening another set of gates. Thus, each channel is designed to operate for a range of headwater elevations. The number of channels may be minimized by maximizing the range of headwater elevations over which each fish passage channel functions. Figure F-G1 shows the water surface elevation and velocity operation ranges for the complete (two fish passages) Phase III design at the Red River of the North control structure.

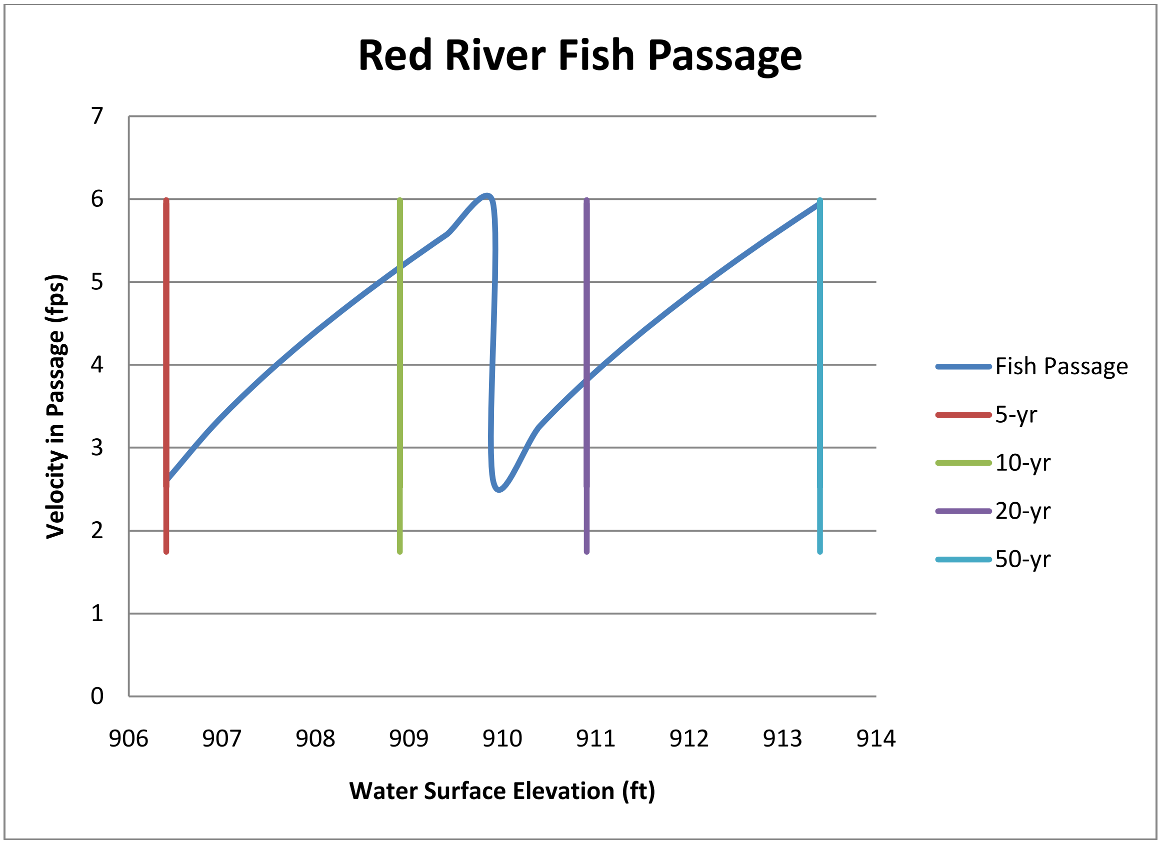

Figure F-G1 **Phase III Fish Passage Operation at the Red River of the North LPP Control Structure**

This design concept requires the operation of gates at some structures during flood events. At various flood stages, gates will be either open or closed (the system is not designed such that gates remain partially open, as is the case at the Red River Control Structure).

F-G1.3.1 Gate Design

At the upstream end of the Red River and Wild Rice River fish passage channels, a gated area connects the upstream river reach to a pool downstream of the gates (see Appendix F). The preliminary gate design includes three gates 10 feet wide by 5 feet tall, for a total gated area of 150 square feet at the upstream end of each fish passage channel. The gate height of 5 feet is based on the maximum headwater at the upstream end of the fish passage for which velocities in the fish passage channel remain acceptable while the gates are open.

F-G1.3.2 Fish Passage Inlet Invert Design

The invert elevations for each set of fish passage gates are listed in Table F-G2 below:

Table F-G2 Fish Passage Gate Invert Elevations by Structure

Plan	Location/Control Structure	Fish Passage Channel	Gate Invert (ft)	Number of Gates	Gate Width (ft)	Gate Height (ft)
FCP	Red River	1	899.2	3	10	5
FCP	Red River	2	902.7	3	10	5
FCP	Red River	3	906.2	3	10	5
LPP	Red River	1	912.9	3	10	5
LPP	Red River	2	916.4	3	10	5
LPP	Wild Rice River	1	912.9	3	10	5
LPP	Wild Rice River	2	916.4	3	10	5

Determination of the above Phase 4 inlet inverts involved a lengthy optimization process. Three alternatives were evaluated during this optimization process:

- Alternative 1: Inlet invert based on Phase 3 guidelines (1-ft below the 5-year event water surface elevation).
- Alternative 2: Inlet invert based on operating during the peak water surface elevation of the 50-year design storm with two fish passages.
- Alternative 3: Inlet invert set at the approximate water surface elevation for which the gates become fully opened during the receding limb of the hydrograph.

Alternative 1 represents the design approach used in Phase 3, in which it is assumed that fish passage will be maintained through the control structure until flows reach the 5-year event, at which point the control structure gates will be closed, necessitating a dedicated fish passage. Therefore, fish passage is designed to begin at the 5-year event and maximize fish passage through larger events. In Phase 4, it is estimated that larger flood events may require shutting the control structure gates earlier in the hydrograph (before water surface elevations reach the 5-year event). Thus, fish passage may be required at lower elevations. Alternative 2 lowers the invert of the lower fish passage relative to Alternative 1 to minimize this impact, at the cost of providing no fish passage above the 50-year event. As indicated by Phase 4 HEC-RAS modeling, the control structure gates may be opened following the peak water surface elevation of larger flood events but prior to the water surface elevation falling below that of the 5-year event. Thus, there may be periods along the falling limb of the hydrograph were fish passage is inhibited. Alternative 3 above places the invert of the lowest fish passage at an elevation to maximize fish passage along the falling limb of the hydrograph.

F-G1.3.2.1 Alternative 1 Red River of the North Fish Passage Inlet Invert Design

For Alternative 1, the inlet and outlet inverts were set at 1-ft and 1.5-ft, respectively, below the 5-year water surface elevation. Since the modeling conducted in Phase 4 does not include the 5-year event, this event water surface elevation was chosen to be 1-ft below the 10-year event for this design. Based on this assumption, the invert of the first fish passage on the Red River is at 914.3-ft. The downstream invert is at 901.9-ft. The fish passage becomes operable when the water depth reaches one foot (915.3-ft). The fish passage becomes inoperable and the inflow gate must be shut when the water depth reaches 4.5-ft (918.8-ft). This span allows one fish passage to cover both the 5-year and 10-year design floods.

To reach the 50-year flood event a second passage is needed. The upstream invert of the second passage will be located one foot below the peak elevation for the first passage (917.8-ft). The downstream invert of the second passage is designed conservatively and will be 1.5 feet below the 5-year flood event (901.9-ft). Similarly to the first passage, once the water surface upstream reaches a depth of 4.5-ft (922.3) the velocities in the channel become too high for fish passage. The water surface elevation for the 50-year flood (920.9-ft) lies within the operable range of the second passage. Figures F-G2 and F-G3 present the water surface elevation hydrographs for the 10-year and 50-year storms respectively and the inlet invert elevations for the two fish passage structures at the diversion control structure.

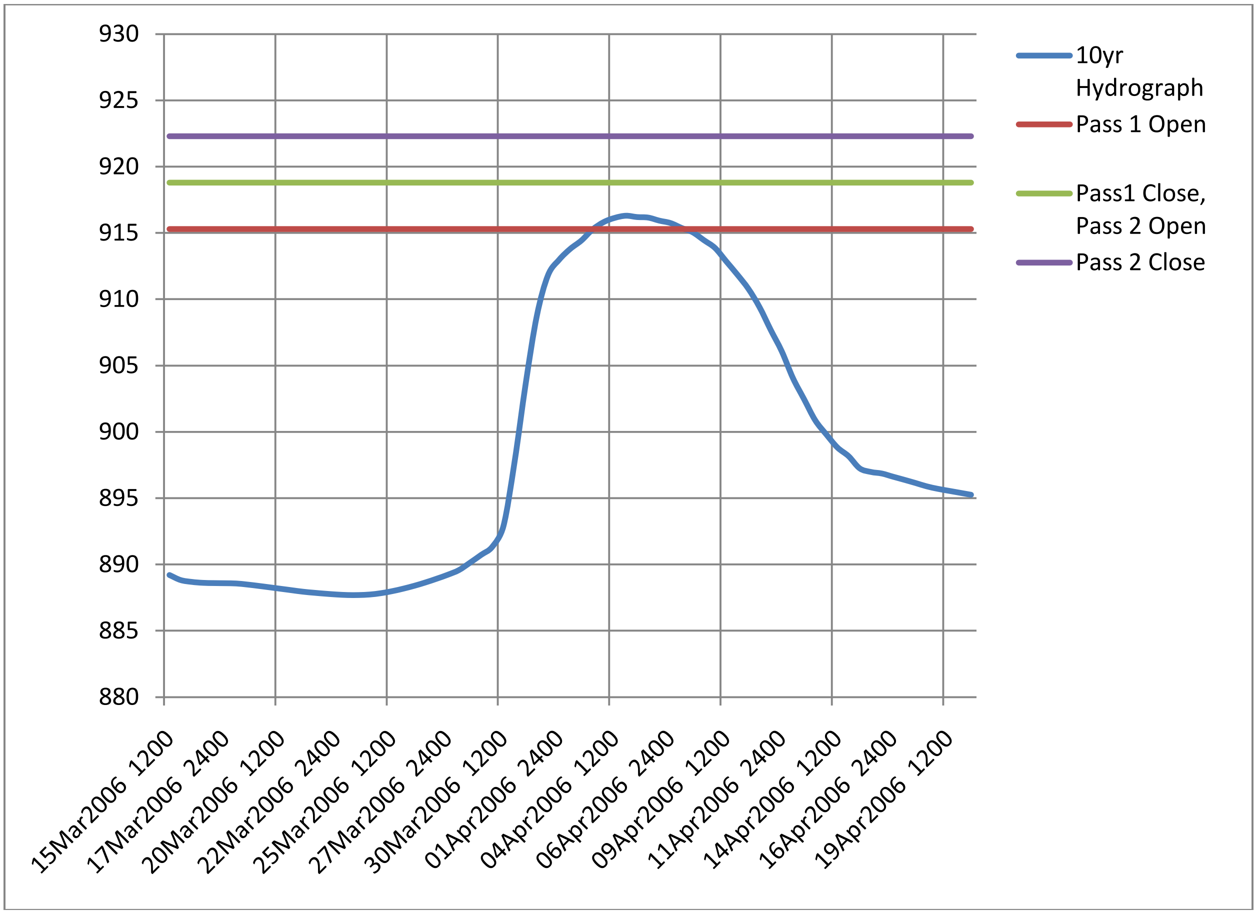

Figure F-G2 Red River Alternative 1 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 10-year Event

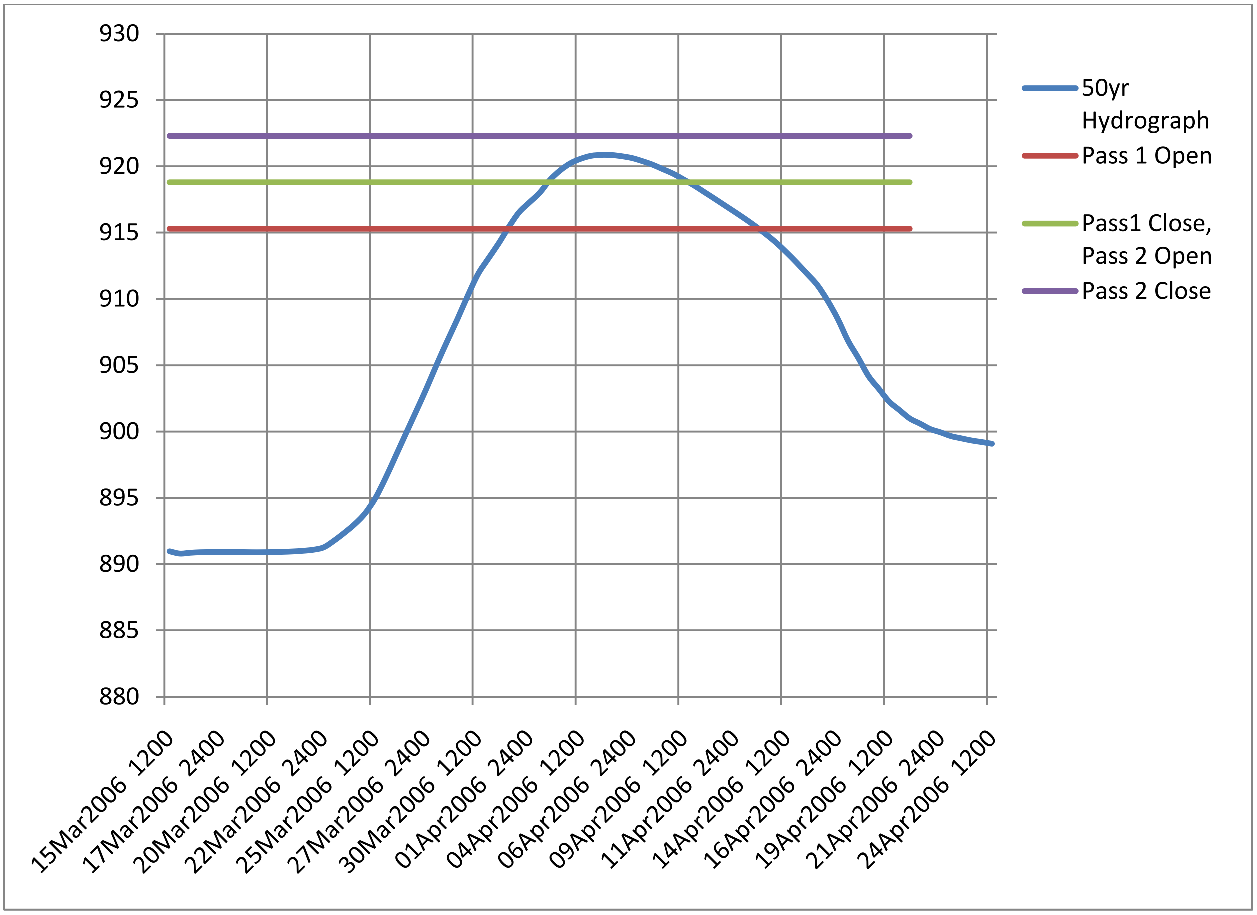

Figure F-G3 **Red River Alternative 1 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 50-year Event**

The Phase 3 design approach with the inlet invert at the 5-year storm allows for fish passage for the 5-year, 10-year, and 50-year flood event peaks, as shown in Figures F-G2 and F-G3. However, the performance of the fish passage for events outside of this range, and how they perform for the entire storm hydrograph, should be determined. To evaluate this, three time parameters are defined; rising limb, peak, and falling limb. The rising limb time parameter represents the duration of time along the hydrograph from the control structure gate closing (fish passage is stopped) to the start of fish passage operation (water surface elevation reaches 1-ft above the fish passage invert). The peak parameter represents the duration of time the hydrograph exceeds the maximum water surface elevation for fish passage on the upper passage. The falling limb parameter represents the duration of time from the water surface elevation occurring below the first fish passage invert to the water surface elevation when the control structure gates are fully opened.

Evaluating the above three parameters is further complicated by the gate operation. When a large, event is predicted (greater than 9,600 cfs), the gates must be closed on the rising limb of the hydrograph (approximately 7-10 days before the 9,600 cfs discharge is reached at the Fargo gauge). In the HEC-RAS models, the gates have been closed slowly over the course of several days to improve stability in the model. In reality, the gates could be closed within a shorter duration. However, the data presented in Table F-G3 for the rising limb closure duration is based on the approximate inflection point for the rising limb of the synthetic, modeled hydrograph. This represents a worst case scenario. It is also important to note that the actual duration of fish passage closure for the rising limb will vary depending on the shape and peak flow of the hydrograph.

The peak and falling limb fish passage closure durations are easier to predict. The peak is simply determined by the maximum fish passage water surface elevation and the hydrograph peak. For the falling limb fish passage closure duration, the models can be used to predict the time of gate opening and the hydrograph will provide the duration of closure. Table F-G3 presents the results of the fish passage closure analysis in terms of number of days and approximate days per year (days closed multiplied by flood event probability).

Table F-G3 Red River Alternative 1 – Duration in Days of Fish Passage Closure Due to Velocities Exceeding 6-fps

	Rising Limb		Peak		Falling Limb		Total	
Flood Event	**Days Out[(a)]**	**Days Per Year Out**	**Days Out**	**Days Per Year Out**	**Days Out**	**Days Per Year Out**	**Days Out**	**Days Per Year Out**
10	9	1	0	0	10	1	19	2.0
50	10	<1	0	0	10	<1	20	<1
100	10	<1	4	<1	10	<1	24	<1
500	7	<1	1	<1	0	0	8	<1

(a) Please see the preceding paragraphs for a discussion about the rising limb closure quantification

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4

From Table F-G3, it is evident that fish passage would not be possible for 10 days on the falling limb of the hydrograph for the 10-year and 50-year events. The 100-year event will have four days when fish are unable to pass during the peak. Due to the narrowing of the hydrograph for the 500-year event, fish pass will only be restricted for 1 day at the peak. Figure F-G4 presents a comparison of the 100-yr water surface elevation hydrograph and the 500-yr water surface elevation hydrograph at the Red River of the North control structure. The plot provides a clear view of the difference in the receding limbs of the hydrographs which produces the decrease in closure days for the 500-yr event in Table F-G3. This plot is characteristic of only the project conditions. Finally, the gates are expected to opened near the peak in the hydrograph for the 500-year storm, so fish passage is not expected to be restricted for the falling limb (peak velocities through the gates should be approximately 5.4 fps).

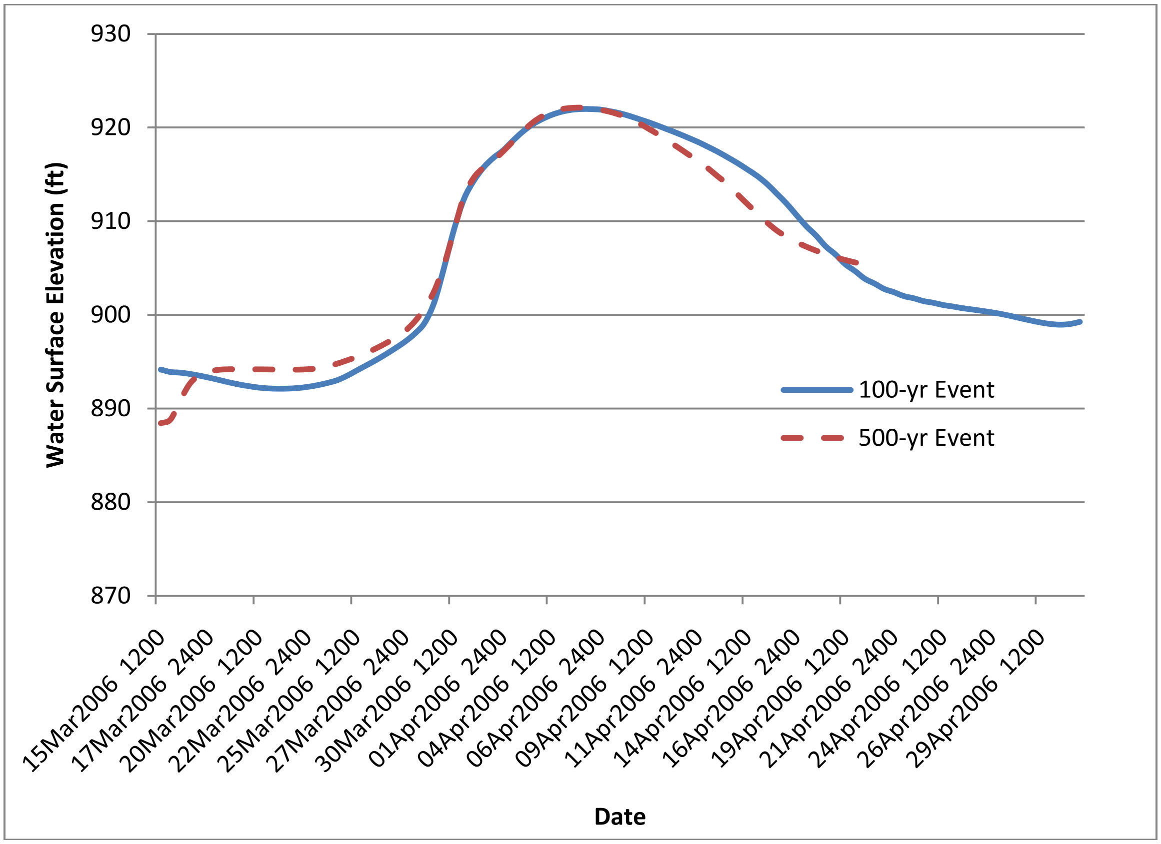

Figure F-G4 **Comparison off RRN Control Structure Water Surface Elevation for the 100-year Event and 500-year Event**

Alternatively, it is desirable to know how often each fish passage will be open. From this information it is possible to compare the advantage of allowing fish passage for a set duration to the associated cost of this infrastructure. Table F-G4 presents the duration in days the Alternative 1 fish passage (both fish passage 1 and 2) will be open. For instance, a 10-year event will only allow fish passage for a total of 4 days in passage 1. This appears short, however it is also providing passage for events down to the 5-year event. The second passage will provide fish movement for 7 days during the 50-year and 100-year events.

Table F-G4 **Red River Alternative 1 – Duration in Days Fish Passage is Operational**

Flood Event	Pass 1		Pass 2	
	Days in Operation	Days Per Year in Operation	Days in Operation	Days Per Year in Operation
10	4	<1	0	0
50	6	<1	7	<1
100	5	<1	7	<1
500	4	<1	6	<1

F-G 1.3.2.2 Alternative 1 Red River of the North Fish Passage Frequency Analysis

Detailed records exist of the Red River of the North discharges at Fargo, ND for each year back to the early 20th century. Correlating the maximum operable water surface elevation of each fish passage to a discharge at the Red River of the North Fargo, ND gauge (based on exceedance probability) allows for the quantification of the fish passage's closure frequency using the historic flow record at Fargo. Table F-G5 provides the upstream operational WSEL's for each fish passage and correlates it with a discharge at the Red River of the North Fargo, ND gauge. This was accomplished by noting the peak WSEL's for each Phase 4 flood event (10-year, 50-year, 100-year, 500-year) and pulling the corresponding peak discharge for the Fargo, ND gauge from the HEC-RAS models. An exponential function was fit to this data and used to interpolate the Fargo, ND discharge for each fish passage operational WSEL range. By correlating the fish passage operation ranges with a discharge at Fargo, ND a frequency analysis can be conducted with the historic data. Table F-G5 includes the number of events and number of days which exceeded each fish passage's design open (individual passage allows fish movement) and closed (individual passage prohibits fish movement) WSEL's for the 1901 to 2009 historic record. Figure F-G5 is a plot of the fitted exponential model for the Red River of the North fish passage WSEL's and the Fargo, ND gauge.

Table F-G5 Red River Alternative 1 – Fish Passage Operational Range WSEL, Corresponding Interpolated Discharges, and Flood Frequency Data for 1901-2009 at the Red River of the North Fargo, ND Gauge

	Fish Passage Open El. (ft)	Fish Passage Closed El. (ft)	Pre-project Q at Fargo for the Fish Passage Open, Q_{open}[(a)] (cfs)	Number of Events Q_{open} is Exceeded [(b)]	Number of Days Q_{open} is Exceeded [(b)]	Pre-project Q at Fargo for the Fish Passage Closed , Q_{closed}[(c)] (cfs)	Number of Events Q_{closed} is Exceeded [(b)]	Number of Days Q_{closed} is Exceeded [(b)]
Pass 1 (~5-10 yr)	913.7	917.2	12653	10	116	17551	6	46
Pass 2 (~20-50 yr)	917.2	920.7	17551	6	46	24346	3	14
Pass 3 (>50 yr)	920.7	924.2	24346	3	14	33772	0	0

(a) Flow refers to the pre-project flow at the Fargo USGS Gage with the same recurrence interval as the post-project flow (and corresponding water elevation) at the Red River Control Structure at which the fish passage opens.

(b) As measured in the existing Fargo USGS gage record.

(c) Flow refers to the pre-project flow at the Fargo USGS Gage with the same recurrence interval as the post-project flow (and corresponding water elevation) at the Red River Control Structure at which the fish passage is closed.

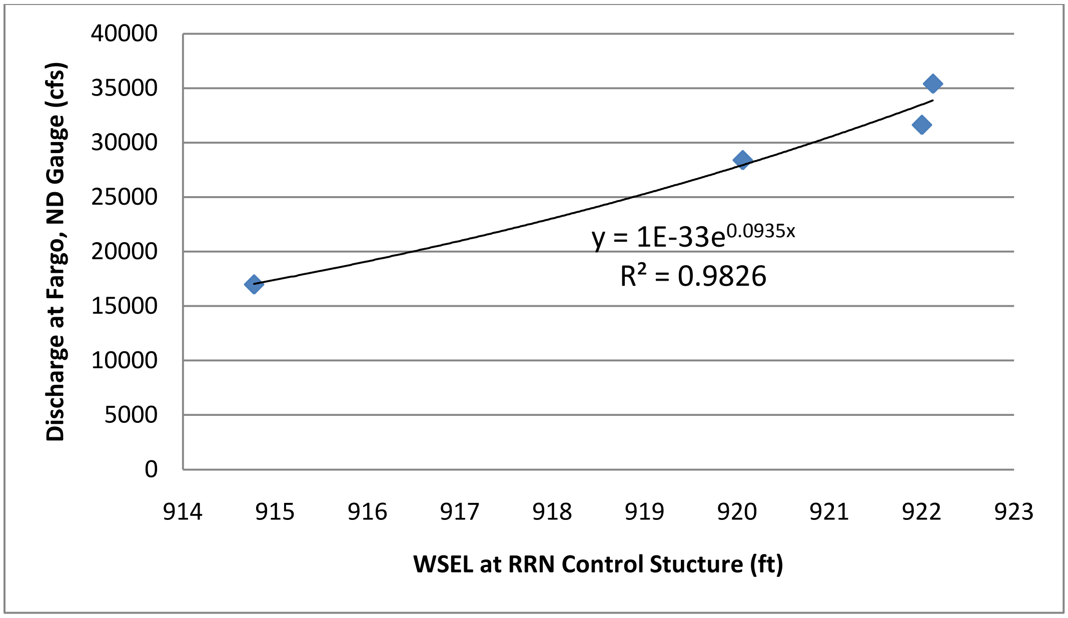

Figure F-G5 RRN Control Structure Water Surface Elevation versus Discharge at the RRN Fargo, ND Gauge

It is important to note for this analysis, there were only four data points available for the curve fitting. In addition, the lowest and highest water surface elevations in Table F-G5 extend outside of the available data range.

F-G1.3.2.3 Alternative 2 Red River of the North Fish Passage Inlet Invert Design

For Alternative 2, the maximum water surface elevation for the second passage was set at the peak of the 50-year event. Based on this assumption, the invert of the first fish passage on the Red River is at 912.9-ft. The downstream invert is at 901.9-ft (1.5-ft below the 5-year tailwater elevation). The fish passage becomes operable when the water depth reaches one foot (913.9-ft). The fish passage becomes inoperable and the inflow gate must be shut when the water depth reaches 4.5-ft (917.4-ft). This span allows one fish passage to cover both the 5-year and 10-year design floods. This design also allows for operation at water surface elevations less than the 5-year event. This is beneficial as larger flood events would require shutting the control structure before the water surface elevation rises to the 5-year event. In addition, the fish passage will be more flexible for use during smaller, more frequent events between 9,600 cfs (approximately the 3.5-year event) and the 5-year event.

The upstream invert of the second passage will be located one foot below the peak elevation for the first passage (917.4-ft). The downstream invert of the second passage is designed conservatively and will be 1.5 feet below the 5-year flood event (901.9-ft). Similarly to the first passage, once the water surface upstream reaches a depth of 4.5-ft (920.9) the velocities in the channel become too high for fish passage. The water surface elevation for the 50-year flood (920.9-ft) lies within the operable range of the second passage. Figures F-G6 and F-G7 present the water surface elevation hydrograph for the

10-year and 50-year storms respectively and the inlet invert elevations for the two fish passage structures at the diversion control structure.

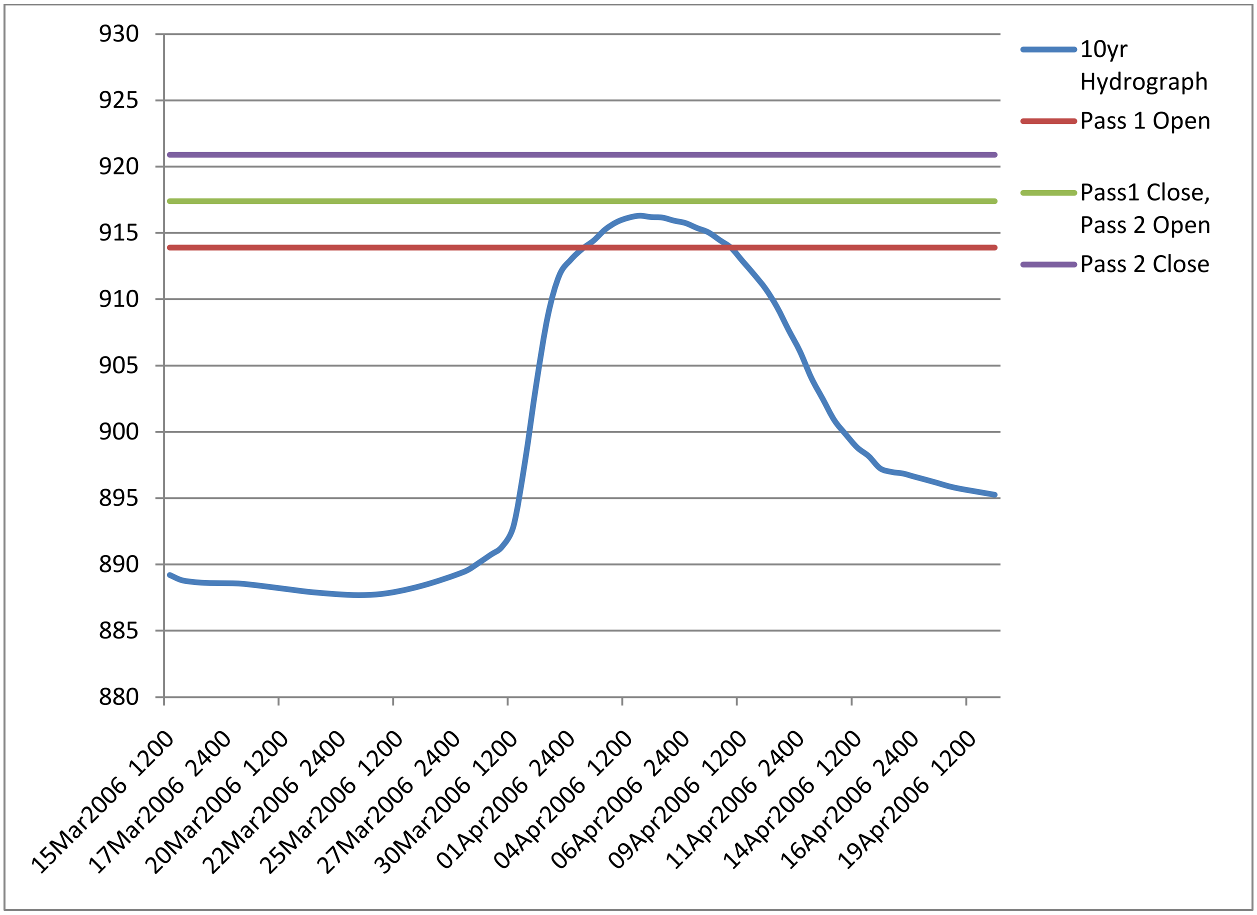

Figure F-G6 **Red River Alternative 2 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 10-year Event**

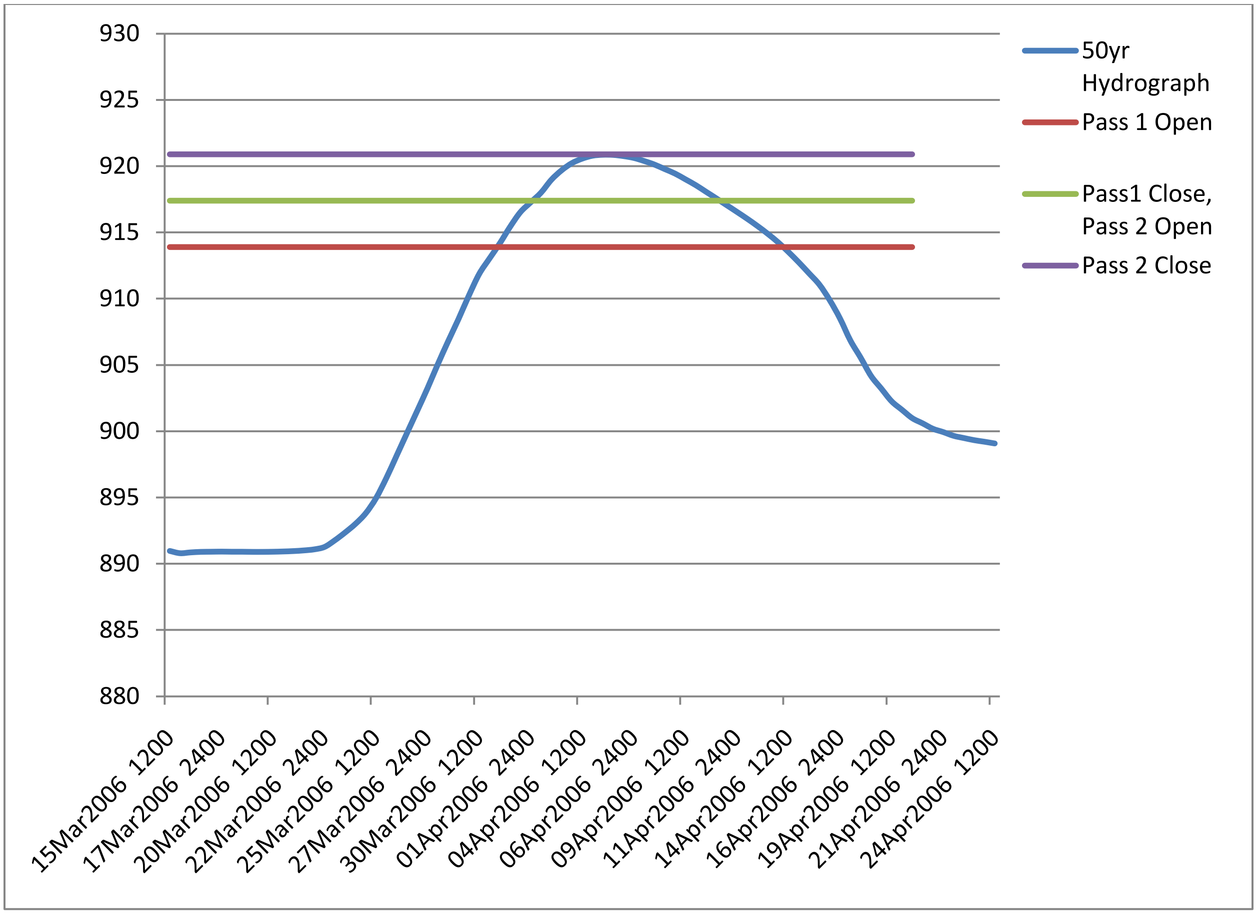

Figure F-G7 **Red River Alternative 2 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 50-year Event**

As discussed for Alternative 1, Table F-G6 presents the results of the fish passage closure analysis in terms of number of days and approximate days per year (days closed multiplied by flood event probability).

Table F-G6 Red River Alternative 2 – Duration in Days of Fish Passage Closure Due to Velocities Exceeding 6-fps

Flood Event	Rising Limb		Peak		Falling Limb		Total	
	Days Out[(a)]	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out
10	8	1	0	0	9	1	17	2
50	10	<1	0	0	9	<1	19	<1
100	9	<1	8	<1	9	<1	26	<1
500	6	<1	5	<1	0	0	11	<1

(a) Please see the Red River of the North Alternative 1 design discussion preceding Table F-G3 about the rising limb closure quantification.

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

Table F-G6 shows that fish passage will not be possible for nine days on the falling limb of the hydrograph for the 10-year, 50-year, and 100-year events. The 100-year event will have eight days when fish are unable to pass during the peak. Due to the narrowing of the hydrograph for the 500-year event, fish passage will only be restricted for 5 days at the peak (See explanation in Section F-G1.3.2.1). Also, the gates are expected to opened near the peak in the hydrograph for the 500-year event, so fish passage is not expected to be restricted for falling limb (peak velocities through the control structure gates should be approximately 5.4 fps). Since this passage has been lowered as compared to Alternative 1, it has slightly fewer days closed. In addition, this passage will have fewer days closed for the more frequent events smaller than the 10-year flood.

Alternatively, it is desirable to know how often each fish passage will be open. From this information it is possible to compare the advantage of allowing fish passage for a set duration to the cost of the associated infrastructure. Table F-G7 presents the duration in days the Alternative 2 fish passage (both passages 1 and 2) will be open. For instance, a 10-year event will only allow fish passage for a total of 6 days in passage 1. This appears short, however it is also providing passage for events down to the 5-year event. The second passage will only provide fish movement for 9 days during the 50-year and 100-year events.

Table F-G7 Red River Alternative 2 – Duration in Days Fish Passage is Operational

Flood Event	Pass 1		Pass 2	
	Days in Operation	Days Per Year in Operation	Days in Operation	Days Per Year in Operation
10	6	1	0	0
50	5	<1	9	1
100	5	<1	5	<1
500	4	<1	4	<1

F-G 1.3.2.4 Alternative 2 Red River of the North Fish Passage Frequency Analysis

Detailed records exist of the Red River of the North discharges at Fargo, ND for each year back to the early 20th century. Correlating the maximum operable WSEL of each fish passage to a discharge at the Red River of the North Fargo, ND gauge (based on exceedance probability) allows for the quantification of the fish passage's closure frequency using the historic flow record at Fargo. Table F-G8 provides the upstream operational WSEL's for each fish passage and correlates it with a discharge at the Red River of the North Fargo, ND gauge. This was accomplished by noting the peak WSEL's for each Phase 4 flood event (10-year, 50-year, 100-year, 500-year) and pulling the corresponding peak discharge for the Fargo, ND gauge from the HEC-RAS models. An exponential function was fit to this data and used to interpolate the Fargo, ND discharge for each fish passage operational WSEL range. By correlating the fish passage operation ranges with a discharge at Fargo, ND a frequency analysis can be conducted with the historic data. Table F-G8 includes the number of events and number of days which exceeded each fish passage's design open (individual passage allows fish movement) and closed (individual passage prohibits fish movement) WSEL's for the 1901 to 2009 historic record. Figure F-G8 is a plot of the fitted exponential model for the Red River of the North fish passage WSEL's and the Fargo, ND gauge.

Table F-G8 Red River Alternative 2 – Fish Passage Operational Range WSEL, Corresponding Interpolated Discharges, and Flood Frequency Data for 1901-2009 at the Red River of the North Fargo, ND Gauge

	Fish Passage Open El. (ft)	Fish Passage Closed El. (ft)	Pre-project Q at Fargo for the Fish Passage Open, Q_{open}[(a)] (cfs)	Number of Events Q_{open} is Exceeded (b)	Number of Days Q_{open} is Exceeded (b)	Pre-project Q at Fargo for the Fish Passage Closed , Q_{closed}[(c)] (cfs)	Number of Events Q_{closed} is Exceeded (b)	Number of Days Q_{closed} is Exceeded (b)
Pass 1 (~5-10 yr)	913.9	917.4	12892	10	116	17883	6	44
Pass 2 (~20-50 yr)	917.4	920.9	17883	6	44	24806	2	10

(a) Flow refers to the pre-project flow at the Fargo USGS Gage with the same recurrence interval as the post-project flow (and corresponding water elevation) at the Red River Control Structure at which the fish passage opens.

(b) As measured in the existing Fargo USGS gage record.

(c) Flow refers to the pre-project flow at the Fargo USGS Gage with the same recurrence interval as the post-project flow (and corresponding water elevation) at the Red River Control Structure at which the fish passage is closed.

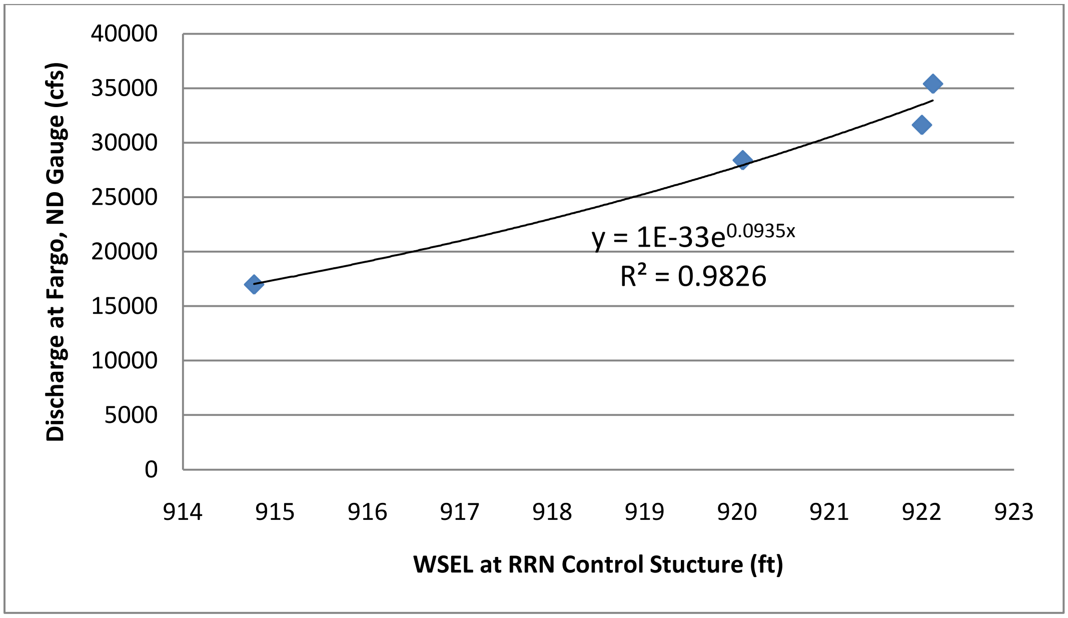

Figure F-G8 RRN Control Structure Water Surface Elevation versus Discharge at the RRN Fargo, ND Gauge

It is important to note for this analysis, there were only four data points available for the curve fitting. In addition, the lowest and highest WSEL's in Table F-G5 extend outside of the available data range.

F-G1.3.2.5 Alternative 3 Red River of the North Fish Passage Inlet Invert Design

For Alternative 3, the minimum water surface elevation for the first passage was set at the approximate water surface elevation that coincides with the full opening of the control structure gates on the receding limb of the 50-year event hydrograph. Based on this assumption, the invert of the first fish passage on the Red River is at 898.3-ft. The downstream invert is at 901.9-ft (1.5-ft below the 5-year tailwater elevation). The fish passage becomes operable when the water depth reaches one foot (899.3-ft). The fish passage becomes inoperable and the inflow gate must be shut when the water depth reaches 4.5-ft (902.8-ft). This span does not allow the fish passage to cover the peak of the 5-year and 10-year design floods.

The upstream invert of the second passage will be located one foot below the peak elevation for the first passage (901.8-ft). The downstream invert of the second passage is designed conservatively and will be 1.5 feet below the 5-year flood event (901.9-ft). Similarly to the first passage, once the water surface upstream reaches a depth of 4.5-ft (906.3) the velocities in the channel become too high for fish passage. The water surface elevation for the 10-year flood (916.29-ft) lies outside the operable range of the second passage, thus the second passage will not handle the peak of the 10-year event. Figures F-G9 and F-G10 present the water surface elevation hydrograph for the 10-year and 50-year events respectively and the inlet invert elevations for the two fish passage structures at the diversion control structure.

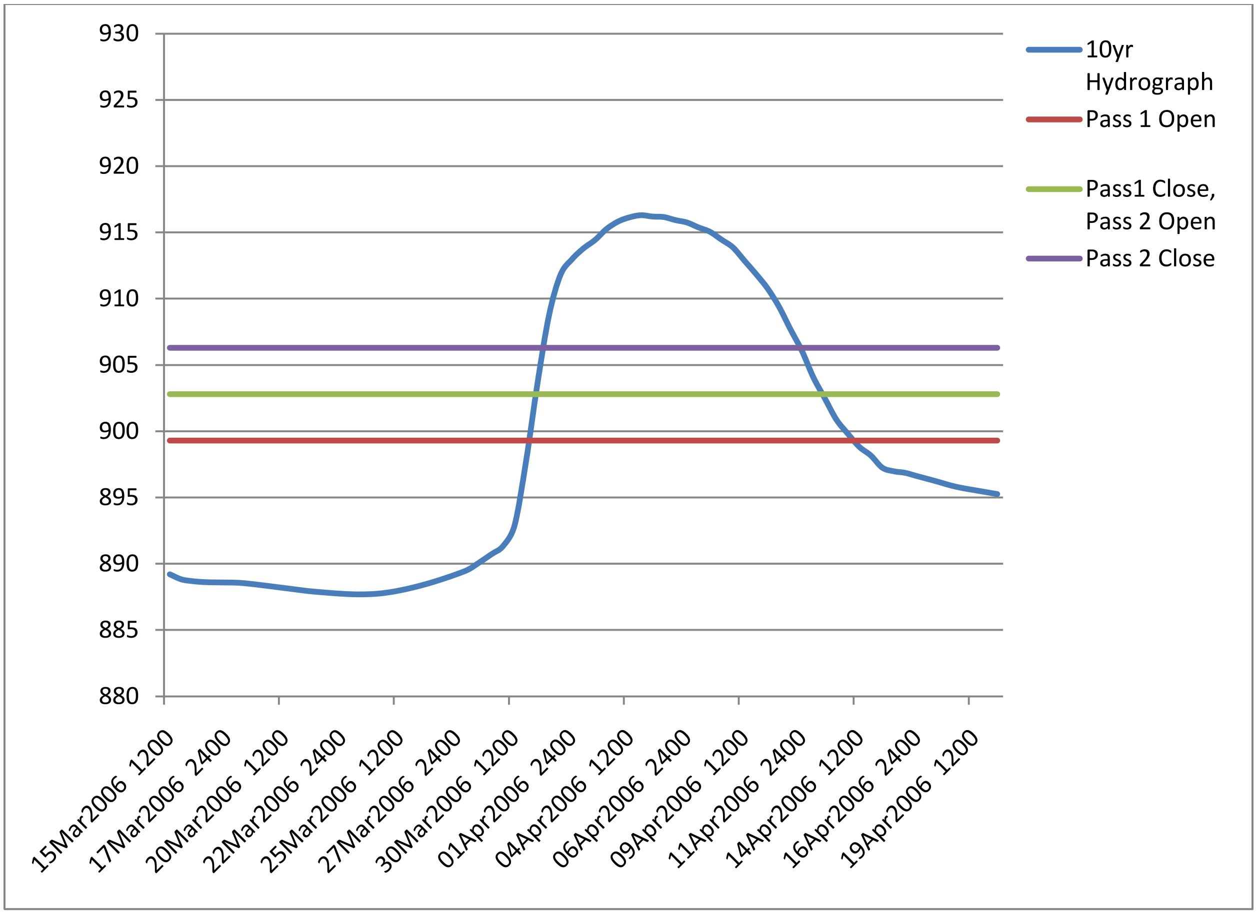

Figure F-G9 **Red River Alternative 3 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 10-year Event**

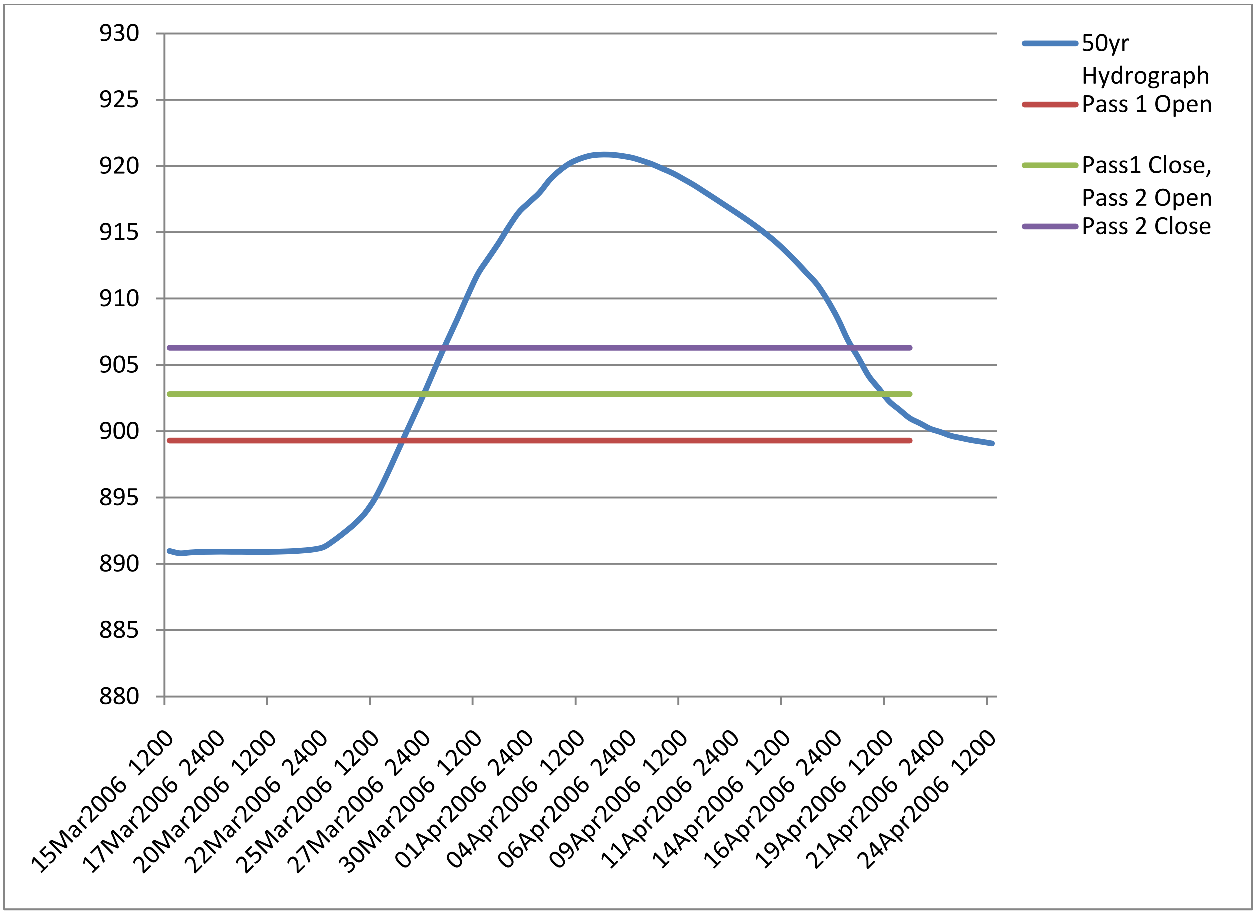

Figure F-G10 Red River Alternative 3 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 50-year Event

As discussed for Alternative 1, Table F-G9 presents the results of the fish passage closure analysis in terms of number of days and approximate days per year (days closed multiplied by flood event probability).

Table F-G9 Red River Alternative 3 – Duration in Days of Fish Passage Closure Due to Velocities Exceeding 6-fps

	Rising Limb		Peak		Falling Limb		Total	
Flood Event	Days Out(a)	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out
10	6	<1	11	1	3	<1	20	2
50	5	<1	20	<1	0	0	25	<1
100	3	<1	24	<1	0	0	27	<1
500	3	<1	19	<1	0	0	22	<1

(a) Please see the Red River of the North Alternative 1 design discussion preceding Table F-G3 about the rising limb closure quantification.

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

Table F-G9 shows that fish passage will not be possible for 3 days on the falling limb of the hydrograph for the 10-year event. The 50-, 100-, and 500-year events will not have any days closed for the falling limb. However, there will be 11, 20, 24, and 19 days of closure for the 10-, 50-, 100-, and 500-year events, respectively. This passage is effective at eliminating the days of closure for the falling limb, however, there is a significant increase in the number of closure days at the peak of the hydrograph.

Alternatively, it is desirable to know how often each fish passage will be open. From this information it is possible to compare the advantage of allowing fish passage for a set duration to the cost of the associated infrastructure. Table F-G10 presents the duration in days the Alternative 3 fish passage (both passages 1 and 2) will be open. For instance, a 10-year event will only allow fish passage for a total of 2 days in passage 1. The second passage will only provide fish movement for 3 days during the 50-year and 100-year events.

Table F-G10 Red River Alternative 3 –Duration in Days Fish Passage is Operational

	Pass 1		Pass 2	
Flood Event	Days in Operation	Days Per Year in Operation	Days in Operation	Days Per Year in Operation
10	2	<1	2	<1
50	6	<1	3	<1
100	8	<1	3	<1
500	5	<1	11	<1

F-G 1.3.2.6 Alternative 1 Wild Rice River Fish Passage Inlet Invert Design

The design of the Wild Rice River Fish passage was conducted in the same manner as was done for the Red River of the North Fish Passages. Three alternatives were evaluated with the goal of optimizing the number of days the fish passage is in operation and keep the number of days it is closed for each event to a minimum.

For Alternative 1, the inlet and outlet inverts were set at 1-ft and 1.5-ft, respectively, below the 5-year water surface elevation. Because the unsteady flow modeling conducted in Phase 4 does not include the 5-year event, the 5-year event water surface elevation was chosen to be 1-ft below the 10-year event for this design. Based on this assumption, the invert of the first fish passage on the Wild Rice River is at 913.6-ft. The downstream invert is at 898.9-ft. The fish passage becomes operable when the water depth reaches one foot (914.6-ft). The fish passage becomes inoperable and the inflow gate must be shut when the water depth reaches 4.5-ft (918.1-ft). This span allows one fish passage to cover both the 5-year and 10-year design floods.

To reach the 50-year flood event a second passage is needed. The upstream invert of the second passage will be located one foot below the peak elevation for the first passage (917.1-ft). The downstream invert of the second passage is designed conservatively and will be 1.5 feet below the 5-year flood event (898.9-ft). Similarly to the first passage, once the water surface upstream reaches a depth of 4.5-ft (921.6) the velocities in the channel become too high for fish passage. The water surface elevation for the 50-year flood (920.9-ft) lies within the operable range of the second passage. Figures F-G11 and F-G12 present the water surface elevation hydrograph for the 10-year and 50-year events respectively and the inlet invert elevations for the two fish passage structures at the Wild Rice River control structure.

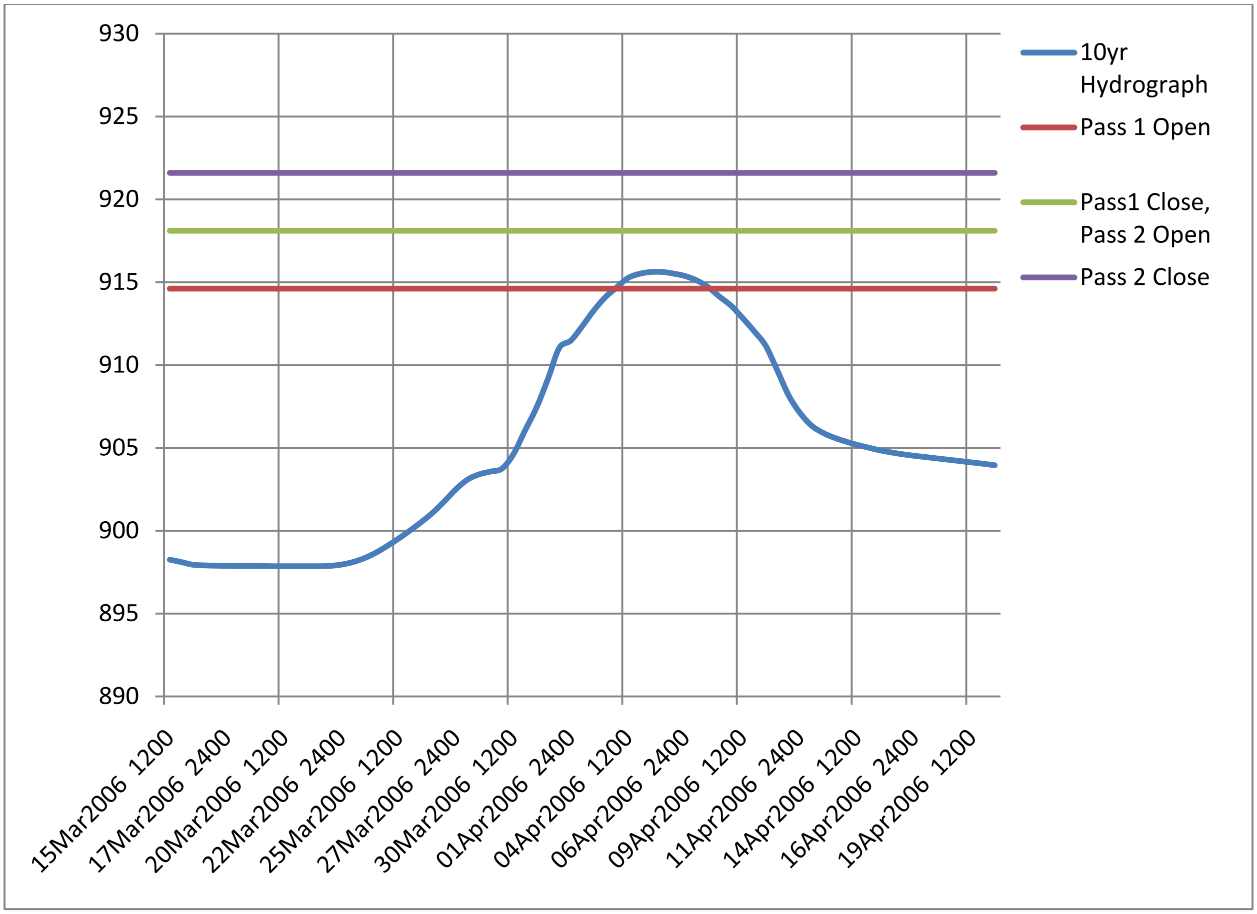

Figure F-G11 **Wild Rice River Alternative 1 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Wild Rice River Control Structure for the 10-year Event**

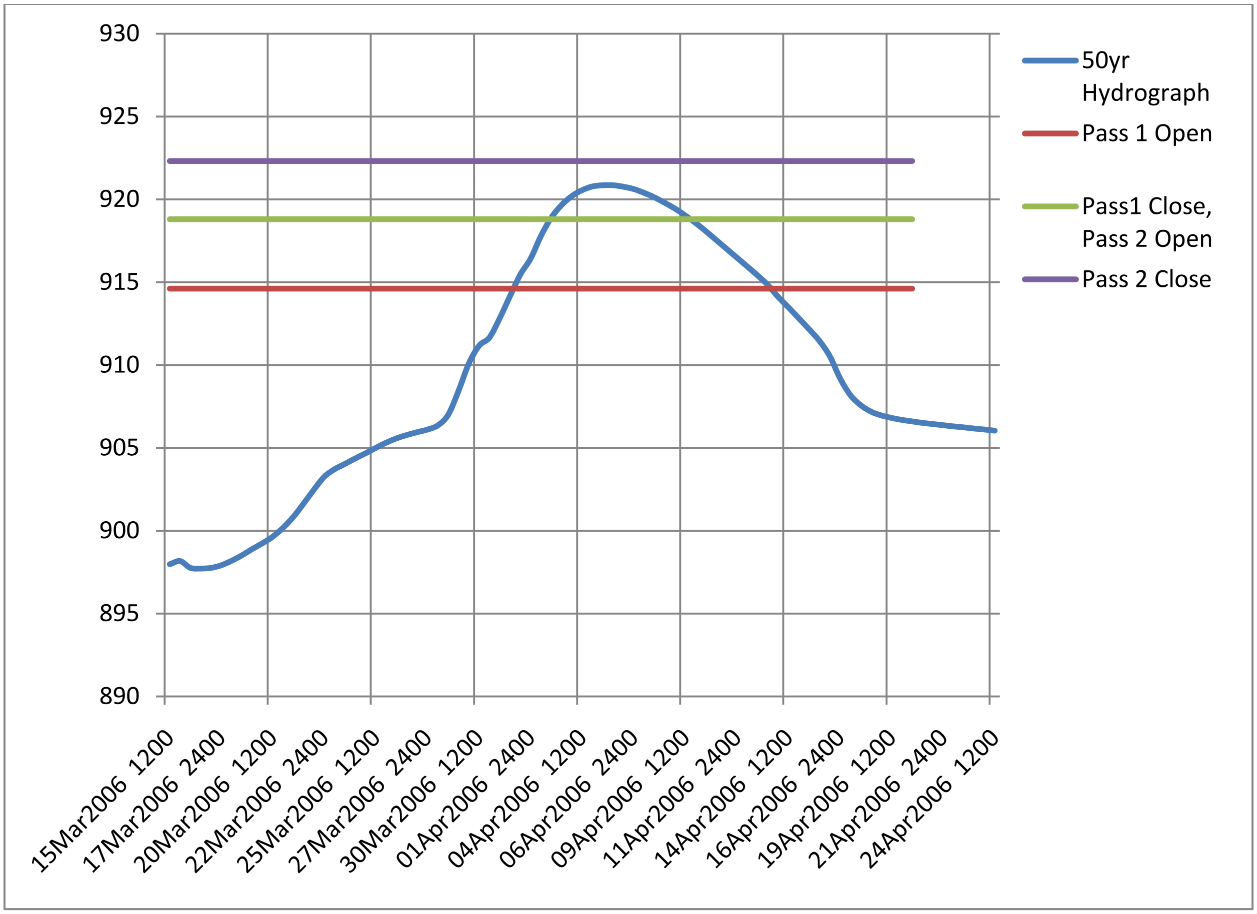

Figure F-G12 Wild Rice River Alternative 1 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Wild Rice River Control Structure for the 50-year Event

As discussed for Red River of the North Alternative 1, Table F-G11 presents the results of the fish passage closure analysis in terms of number of days and approximate days per year (days closed multiplied by flood event probability). The gate operation for the Wild Rice River begins partially closed in the model: similar to the Red River control structure. However, the Wild Rice River control structure does not open back up on the falling limb of the hydrograph, which increases the number of fish passage closure days for the falling limb of the Wild Rice hydrograph when compared to the Red River of the North data.

Table F-G11 Wild Rice River Alternative 1 – Duration in Days of Fish Passage Closure Due to Velocities Exceeding 6-fps

	Rising Limb		Peak		Falling Limb		Total	
Flood Event	**Days Out***	**Days Per Year Out**	**Days Out**	**Days Per Year Out**	**Days Out**	**Days Per Year Out**	**Days Out**	**Days Per Year Out**
10	12	1	0	0	13	1	25	3
50	12	<1	0	0	11	<1	23	<1
100	13	<1	6	<1	14	<1	33	<1
500	13	<1	4	<1	19	<1	36	<1

(a) Please see the Red River of the North Alternative 1 design discussion preceding Table F-G3 about the rising limb closure quantification.

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

Table F-G11, shows that fish passage will not be possible for 13, 11, 14, and 19 days on the falling limb of the hydrograph for the 10-year, 50-year, 100-year, and 500-year events. The 100-year event will have 6 days when fish are not able to pass during the peak. Due to the narrowing of the hydrograph for the 500-year event, fish pass will only be restricted for 4 days at the peak (See explanation in Section F-G1.3.2.1). Also, the gates are expected to opened near the peak in the hydrograph for the 500-year storm, so fish passage is not expected to be restricted for the falling limb (peak velocities through the control structure gates should be approximately 5.4 fps).

Alternatively, it is desirable to know how often each fish passage will be open. From this information it is possible to compare the advantage of allowing fish passage for a set duration to the cost of the associated infrastructure. Table F-G12 presents the duration in days the Alternative 1 fish passage (both passages 1 and 2) will be open. For instance, a 10-year event will only allow fish passage for a total of 5 days in passage 1. This appears short, however it is also providing passage for events down to the 5-year event. The second passage will only provide fish movement for 8 days during the 50-year and 100-year events.

Table F-G12 Wild Rice River Alternative 1 – Duration in Days Fish Passage is Operational

Flood Event	Pass 1		Pass 2	
	Days in Operation	Days Per Year in Operation	Days in Operation	Days Per Year in Operation
10	5	<1	0	0
50	5	<1	8	<1
100	5	<1	8	<1
500	4	<1	7	<1

F-G 1.3.2.7 Alternative 2 Wild Rice River Fish Passage Inlet Invert Design

For Alternative 2, the maximum water surface elevation for the second passage was set at the peak of the 50-year event. Based on this assumption, the invert of the first fish passage on the Wild Rice River is at 912.9-ft. The downstream invert is at 901.9-ft (1.5-ft below the 5-year tailwater elevation). The fish passage becomes operable when the water depth reaches one foot (913.9-ft). The fish passage becomes inoperable and the inflow gate must be shut when the water depth reaches 4.5-ft (917.4-ft). This span allows one fish passage to cover both the 5-year and 10-year design floods.

The upstream invert of the second passage will be located one foot below the peak elevation for the first passage (917.4-ft). The downstream invert of the second passage is designed conservatively and will be 1.5 feet below the 5-year flood event (901.9-ft). Similarly to the first passage, once the water surface upstream reaches a depth of 4.5-ft (920.9) the velocities in the channel become too high for fish passage. The water surface elevation for the 50-year flood (920.9-ft) lies within the operable range of the second passage. Figures F-G13 and F-G14 present the water surface elevation hydrograph for the 10-year and 50-year storms respectively and the inlet invert elevations for the two fish passage structures at the Wild Rice River control structure.

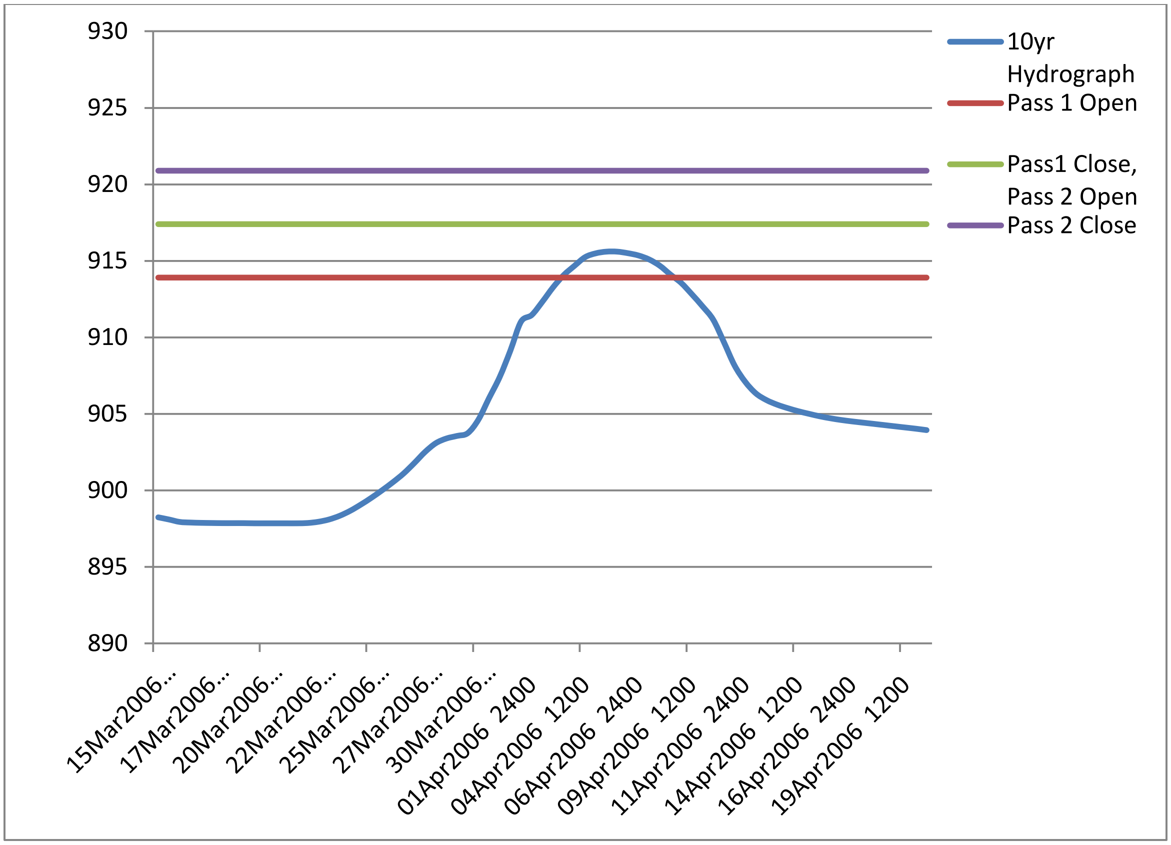

Figure F-G13 **Wild Rice River Alternative 2 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Wild Rice River Control Structure for the 10-year Event**

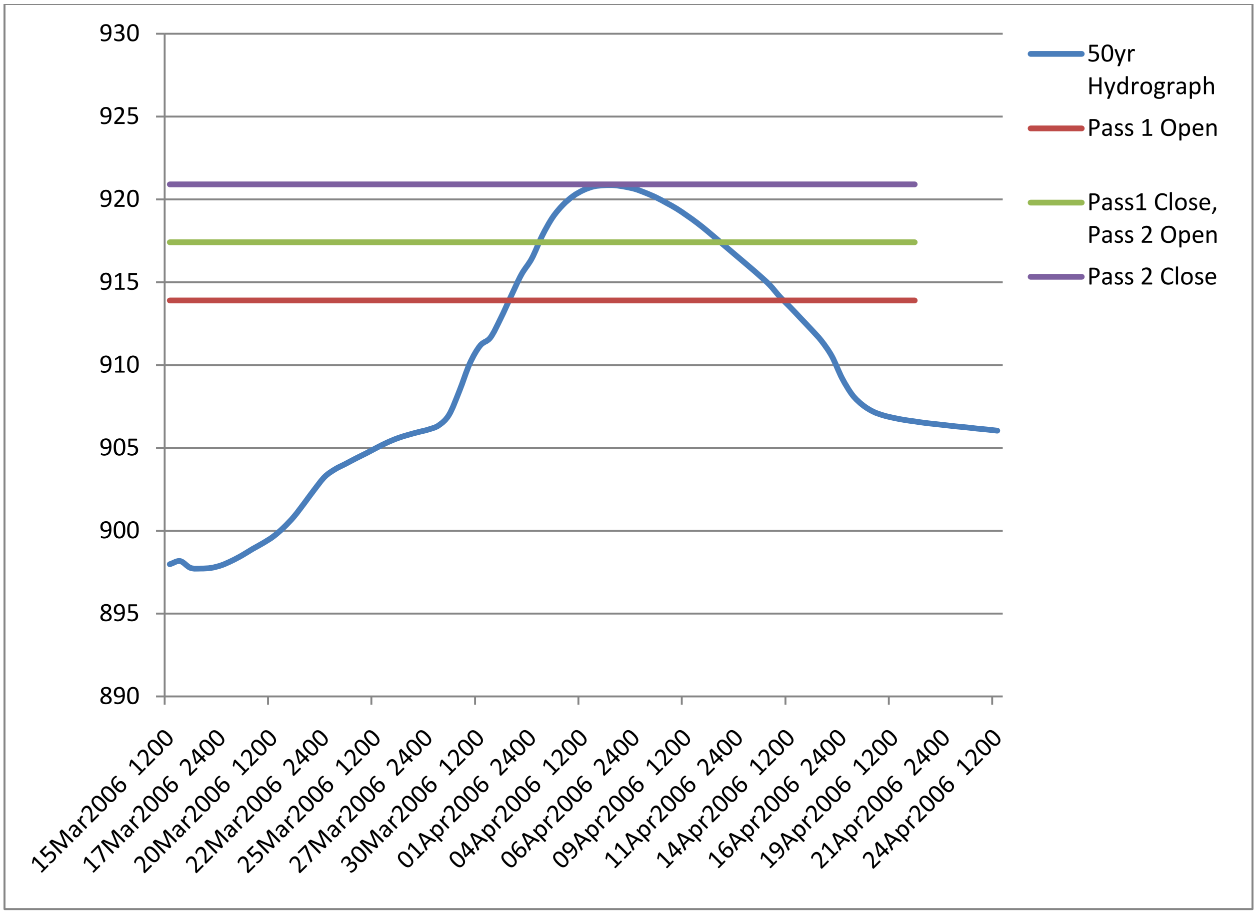

Figure F-G14 Wild Rice River Alternative 2 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Wild Rice River Control Structure for the 50-year Event

As discussed for the Red River of the North Alternative 1, Table F-G13 presents the results of the fish passage closure analysis in terms of number of days and approximate days per year (days closed multiplied by flood event probability). The gate operation for the Wild Rice River begins partially closed in the model, similar to the Red River control structure. However, the Wild Rice River control structure does not open back up on the falling limb of the hydrograph, which increases the number of fish passage closure days for the falling limb of the Wild Rice hydrograph when compared to the Red River of the North data.

Table F-G13 Wild Rice River Alternative 2 – Duration in Days of Fish Passage Closure Due to Velocities Exceeding 6-fps

Flood Event	Rising Limb		Peak		Falling Limb		Total	
	Days Out[(a)]	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out
10	11	1	0	0	12	1	23	2
50	11	<1	0	0	11	<1	22	<1
100	12	<1	6	<1	13	<1	31	<1
500	12	<1	4	<1	19	<1	34	<1

(a) Please see the Red River of the North Alternative 1 design discussion preceding Table F-G3 about the rising limb closure quantification.

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

From Table F-G13, it is evident fish passage will not be possible for 12, 11, 13, and 19 days on the falling limb of the hydrograph for the 10-year, 50-year, 100-year, and 500-year events, respectively. The 100-year event will have six days when fish are not able to pass during the peak (see Section F-G1.3.2.1 for explanation). Due to the narrowing of the hydrograph for the 500-year event, fish passage will only be restricted for 4 days at the peak. Since this passage has been lowered as compared to Alternative 1, it has slightly fewer days closed. In addition, this passage will have fewer days closed for the more frequent events smaller than the 10-year flood.

Alternatively, it is desirable to know how often each fish passage will be open. From this information it is possible to compare the advantage of allowing fish passage for a set duration to the cost of the associated infrastructure. Table F-G14 presents the duration in days the Alternative 2 fish passage (both passages 1 and 2) will be open. For instance, a 10-year event will only allow fish passage for a total of 6 days in passage 1. This seems short, however it is also providing passage for events down to the 5-year event. The second passage will only provide fish movement for 9 days during the 50-year and 100-year events.

Table F-G14 Wild Rice River Alternative 2 – Duration in Days Fish Passage is Operational

Flood Event	Pass 1		Pass 2	
	Days in Operation	Days Per Year in Operation	Days in Operation	Days Per Year in Operation
10	6	1	0	0
50	5	<1	8	<1
100	5	<1	8	<1
500	4	<1	7	<1

F-G1.3.2.8 Alternative 3 Wild Rice River Fish Passage Inlet Invert Design

For Alternative 3 on the Red River of the North, the minimum water surface elevation for the first passage was set at the approximate water surface elevation that coincides with the full opening of the control structure gates on the receding limb of the 50-year event hydrograph. The control structure on the Wild Rice River however, does not open its gates on the receding limb in the same way the Red River of the North control structure does. Instead, the Wild Rice River control structure leaves its gates only slightly open until the end of the event. Thus, the Alternative 3 assessment is not necessary.

F-G1.3.3 Channel Design

Each set of gates is connected to a separate fish passage channel that descends from the upstream pool to the downstream tailwater. Thus, three fish passage channels are necessary to cover the 5-year to 50-year flows at the Red River control structure for the FCP. Two fish passage channels are necessary to cover the 5-year to 50-year flows at the Red River Control Structure and the Wild Rice River Control Structure.

Although each fish passage channel has a different upstream elevation (based on the gate invert), each fish passage channel at a control structure ties into the same downstream elevation, set at least 1.5 feet below the tailwater at that location coincidental to the 5-year event on the Red River.

Each fish passage channel uses a sequence of pools and riffles to achieve the elevation drop between the upstream gates and downstream tailwater. A one foot drop in elevation occurs in the riffle between pools. The number of pools and riffles included in each fish passage channel is based on the difference in gate invert and tailwater elevation. Thus, the length of each fish passage channel varies. The average slope of each fish passage channel is 2.2 percent (see discussion of pool and riffle design below). The number of pools and riffles, and length of each fish passage channel is summarized in Table F-G15.

Table F-G15 Red River and Wild Rice River Fish Passage Channel Pools and Riffles

Plan	Location/ Control Structure	Fish Passage Channel	Riffle Length (ft)	Number of Riffles	Pool Length Top (ft)	Pool Length Bottom (ft)	Number of Pools	Length (ft)	Slope (%)
FCP	Red River	1	20	3	40	20	3	135	2.2
FCP	Red River	2	20	6	40	20	6	270	2.2
FCP	Red River	3	20	10	40	20	10	450	2.2
LPP	Red River	1	20	11	40	20	11	660	2.2
LPP	Red River	2	20	15	40	20	15	900	2.2
LPP	Wild Rice River	1	20	14	40	20	14	840	2.2
LPP	Wild Rice River	2	20	18	40	20	18	1080	2.2

F-G1.3.4 Riffle Design

The riffles are designed to achieve a 1 foot drop over a 20 foot length (5 percent slope). The invert of the first riffle downstream of the gate is equal to the gate invert. The first riffle downstream of the gates is designed with a 10 foot bottom width and 3:1 horizontal to vertical sideslopes (see Figure F-G15).

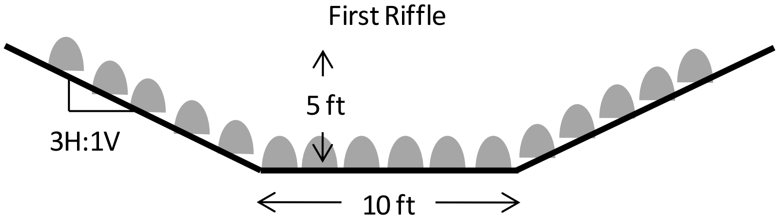

Figure F-G15 Cross Section of First Riffle in Fish Passage

The dimensions of the first riffle are chosen to limit flow over the riffle for larger headwater elevations. When the water depth over the gate (and thus over the riffle) is 4.5 feet (the upper limit of the gate operational range, see section F-G1.5), the flow over the riffle is approximately 600 cfs, or 6 percent of the flow through the Red River control structure. To limit flow velocities to less than 6 feet per second over the first riffle, a high channel roughness (Manning's n of approximately 0.11) must be achieved.

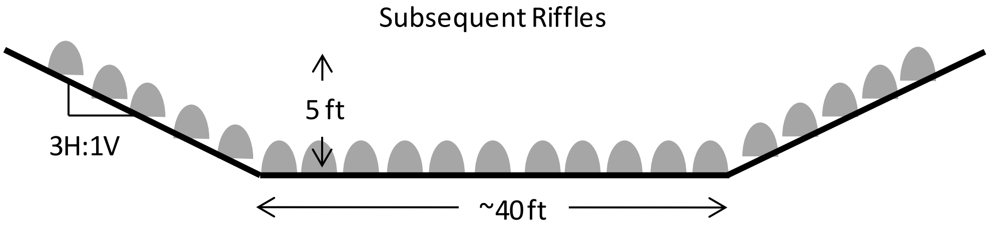

Figure F-G16 Cross Section of Subsequent Riffles in Fish Passage

Subsequent riffles are designed with dimensions similar to the pool dimensions for ease of construction. Thus, the water depths (and velocities) over these riffles are less than the first riffle. Riffles downstream of the first riffle are designed with a bottom width on the order of 40 feet and 3:1 side slopes (see Figure F-G16). The upstream invert of each subsequent riffle is 1 foot below that of the upstream riffle. The last riffle in the pool-riffle sequence of each channel is located below the 5-year tailwater elevation.

F-G1.3.5 <u>Pool Design</u>

Level pools located between riffles provide areas of low velocity for fish to rest between areas of high velocity while moving upstream. The pool design includes a bottom width of 40 feet and 3:1 side slopes (see Figures F-G17 and F-G18). The minimum length of the pools in the direction of flow is calculated using volumetric energy calculation used in the USACE's Lock and Dam 22 Fish Passage Improvement Project Implementation Report – Appendix H.

where:

K = volumetric energy (foot pounds per second per cubic foot)
= specific weight of water (1.94 slugs per cubic foot)
g = gravitational acceleration (32.2 feet per second per second)
Q = discharge (cubic feet per second)
= head difference between pools (feet)
Vp = volume of pool (cubic feet)

Pools should be sized to achieve a volumetric energy (K) of between 3.1 to 4.2 foot pounds per second per cubic foot. Solving the equation for K = 3.1 and using a discharge of 625 cfs (see discussion of fish passage operation, Section F-G1.5) yields a pond volume of 12,600 cubic feet. Using a pool geometry with a 40 foot bottom width, 3:1 side slopes in the direction perpendicular to the flow, 2:1 side slopes in the flow direction, and a maximum pool depth of 7.6 feet (see discussion of fish passage operation, Section F-G1.5), the required pool length is 26 feet. For design feasibility, the bottom width will be increased to 20 feet in the flow direction, and the top width will be 40 feet.

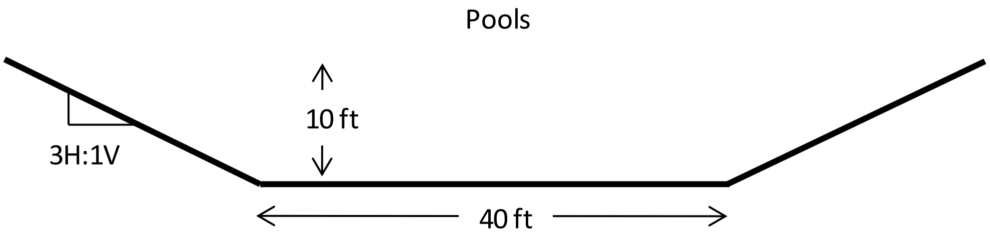

Figure F-G17 Cross Section of Pools in Fish Passage Perpendicular to Flow Direction

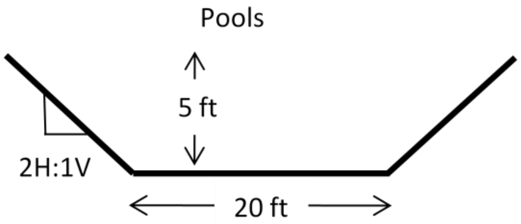

Figure F-G18 Cross Section of Pools in Fish Passage Parallel to Flow Direction

F-G1.4 FISH PASSAGE DESIGN – RUSH RIVER AND LOWER RUSH RIVER FISH PASSAGE

The fish passage design at the Rush River and Lower Rush River also utilizes sequences of pools and riffles (see Appendix F) with one opening at the upstream end of the sequence (or channel). The pool and riffle designs are identical to those discussed for the Red River and Wild Rice River fish passages (see Section F-F1.3.4 and Section F-F1.3.5) with the exception of the initial 10 foot wide riffle. The Rush and Lower Rush Rivers do not need to restrict flow through the fish passages, which allows all of the riffles to be 40-ft wide.

At each tributary, a single fish passage channel is used, thereby minimizing the need for operational control during flood events. At these locations, the fish passage will remain open during all flood events. Thus, for some events, the upstream head on the fish passage will result in velocities through the fish passage channel that prevent fish passage (i.e. head greater than 4.5 feet, see Table F-G22). The headwater and tailwater elevations used for fish passage design at the Rush River and Lower Rush River are summarized in Table F-G16.

Table F-G16 Invert, Headwater and Tailwater Elevations for Fish Passage Design at the Rush River and Lower Rush River

Location/Control Structure	Fish Passage Upstream Invert (ft)	Fish Passage Downstream Invert (ft)	10-year Event (on Red River)		
			Headwater (ft)	Tailwater (ft)	HW-TW (ft)
Rush River[(a)]	880.0[(a)]	866.58	883.52	883.35	0.17
Lower Rush River[(a)]	885.9[(a)]	869.12	887.77	885.0	2.8

(a) Upstream inverts based on channel invert, as opposed to water surface elevations corresponding to an event on the Red River, as is the design concept for the Red River and Wild Rice River fish passages.

At the Rush River, the inlet invert is set at an elevation of 880.0, the approximate channel bottom. The fish passage channel includes a sequence of 14 pools and riffles; the fish passage drops from the tributary channel bottom to an elevation equal to the invert of the low flow channel in the Diversion Channel (see Table F-G17). This allows low flows in the Rush River to pass into the Diversion Channel while allowing fish to pass upstream into the Rush River. This structure will allow fish passage until slightly above the 10-year coincidental event on the Red River.

At the Lower Rush River, the inlet invert is set at an elevation of 885.9, the approximate channel bottom. The fish passage channel includes a sequence of 17 pools and riffles; the fish passage drops from the tributary channel bottom to an elevation equal to the invert of the low flow channel in the Diversion Channel (see Table F-G17). This allows low flows in the Lower Rush River to pass into the Diversion Channel while allowing fish to pass upstream into the Lower Rush River. This structure will allow fish passage up to approximately 1 foot below the 50-year event on the Red River.

Table F-G17 Rush River and Lower Rush River Fish Passage Channel Pools and Riffles

Location/Control Structure	Fish Passage Channel	Riffle Length (ft)	Number of Riffles	Pool Length Top (ft)	Pool Length Bottom (ft)	Number of Pools	Length (ft)	Slope (%)
Rush River	1	20	14	40	20	14	840	2.2
Lower Rush River	1	20	17	40	20	17	1020	2.2

Tables F-G18 and F-G20 provide the duration in days the Rush and Lower Rush fish passage will be closed due to average velocities exceeding 6-fps. The Rush and Lower Rush fish passages will remain fully passable for the 10 year storm, however there will be days the fish passage will be closed for the 50-, 100-, and 500-year design events. Tables F-G19 and F-G21 display the number of days the Rush and Lower Rush fish passages will be open for fish movement upstream for each representative design event.

Table F-G18 Duration in Days of Rush Fish Passage Closure Due to Velocities Exceeding 6-fps

Flood Event	Peak	
	Days Out	Days Per Year Out
10	0	0
50	10	<1
100	14	<1
500	13	<1

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

Table F-G19 Duration in Days Rush Fish Passage is Operational

Flood Event	Pass 1	
	Days in Operation	Days Per Year in Operation
10	10	1
50	7	<1
100	7	<1
500	10	<1

Table F-G20 Duration in Days of Lower Rush Fish Passage Closure Due to Velocities Exceeding 6-fps

Flood Event	Peak	
	Days Out	Days Per Year Out
10	0	0
50	4	<1
100	8	<1
500	8	<1

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

Table F-G21 Duration in Days Lower Rush Fish Passage is Operational

Flood Event	Pass 1	
	Days in Operation	Days Per Year in Operation
10	4	<1
50	6	<1
100	5	<1
500	6	<1

F-G1.5 HYDRAULICS OF FISH PASSAGE OPERATION

The hydraulics of each fish passage channel varies as the headwater elevation at the entrance to the fish passage channel increases. Thus, the flow through a fish passage channel, velocity through the pools and riffles, and depth through the pools and riffles vary between the 5-, 10-, 20-, and 50-year events. The hydraulics were evaluated for the full range of relative headwater elevations for which a fish passage channel is intended to function, and are presented in Table F-G22. Calculations were performed using Manning's equation through the first riffle (the limiting channel geometry), and extrapolating those results to other pool and riffle geometries using continuity of flow.

Table F-G22 Hydraulics of Fish Passage Channels for Varying Headwater Conditions

Head on Gate Invert (ft)	Velocity through Gates (ft/sec)	Depth in First Pool (ft)	Velocity in First Pool (ft/sec)	Depth in Later Pools (ft)	Velocity in Later Pools (ft/sec)	Depth in First Riffle (ft)	Velocity in First Riffle (ft/sec)	Depth in Later Riffles (ft)	Velocity in Later Riffles (ft/sec)	Flow (cfs)
0.0(a)	--	--	--	--	--	--	--	--	--	--
0.5(a)	--	--	--	--	--	--	--	--	--	--
1.0	1.2	6.0	0.1	5.5	0.1	1.0	2.6	0.5	1.8	35
1.5	1.6	6.5	0.2	5.7	0.2	1.5	3.3	0.7	2.4	71
2.0	2.1	7.0	0.3	6.0	0.4	2.0	3.8	1.0	3.0	123
2.5	2.5	7.5	0.4	6.3	0.5	2.5	4.3	1.3	3.5	187
3.0	3.0	8.0	0.5	6.6	0.7	3.0	4.8	1.6	4.0	271
3.5	3.5	8.5	0.7	6.9	0.9	3.5	5.2	2.0	4.5	371
4.0	4.1	9.0	0.8	7.2	1.1	4.0	5.6	2.3	5.0	489
4.5	4.6	9.5	1.0	7.6	1.3	4.5	5.9	2.7	5.5	625

(a) Gate to fish passage remains closed until headwater is 1 foot greater than gate invert

The critical condition (i.e. highest velocities and highest flows) occur when the headwater is greatest. The use of only two sets of gates and fish passage channels at the Red River and Wild Rice River for the LPP result in a wide range of possible hydraulic conditions during operation. Limiting the range of flow and velocity through the fish passage would require more fish passage channels with a smaller gap in invert upstream elevations. Single fish passage channels were selected for the Lower Rush and Rush Rivers so that the passages do not have to be actively managed, minimizing the operational complexity of the system.

Throughout the range of potential headwaters and flows through each fish passage channel, the maximum velocities in the riffles and gates remain below 6 feet per second. The average flow through the pools is 1.3 feet per second or less. In the case of the Red River and Wild Rice River fish passages, only one set of gates is opened at a time, and gates are closed as the upstream water surface elevation increases. At the Lower Rush River and Rush River, the fish passage gates remain open regardless of upstream headwater conditions (although velocities prohibit fish passage above a given water surface elevation).

F-G1.6 LOCATION OF FISH PASSAGE CHANNELS

Similar design concepts have been used for fish passage at the Red River control structures and tributary control structures. Each location, however, presents unique topography and control structure design, among which fish passage must be situated.

F-G1.6.1 Fish Passage Location – Red River Control Structure (FCP)

The design described in this subsection includes three fish passage channels of varying lengths at the Red River control structure for the FCP. The gates at the upstream ends of the fish passage channels will be placed in the east wingwall of the control structure (see Appendix F), and extend north to the constructed Red River channel downstream of the control structure. The downstream ends of the fish passage channels will merge at the Red River channel, downstream of the hydraulic jump, at an elevation of 896.7 feet. The downstream end of the fish passage will be located as close to the control structure as possible provide an alternative route to fish that reach that point but cannot proceed through the control structure due to high velocities.

F-G1.6.2 Fish Passage Location – Red River Control Structure (LPP)

The design described in this subsection includes two fish passage channels of varying lengths at the Red River control structure. The gates at the upstream ends of the fish passage channels will be placed in the east wingwall of the control structure (see Appendix F), and extend north to the constructed Red River channel downstream of the control structure. Placing the fish passage on the east side of the control structure will reduce the chances of fish emerging from the fish passage channel and being swept into the connecting channel on the west side of the Red River. The downstream ends of the fish passage channels will merge at the Red River channel, downstream of the hydraulic jump. The fish passage includes 11 and 15 sequences of pools and riffles for the lower and upper channels, respectively. The downstream end of the fish passage will be located as close to the control structure as possible to provide an alternative route for fish that reach that point but cannot proceed through the control structure due to high velocities.

F-G1.6.3 Fish Passage Location – Wild Rice River Control Structure (LPP)

The design described in this subsection includes two fish passage channels of varying lengths at the Wild Rice River control structure. The gates at the upstream ends of the fish passage channels will be placed in the west wingwall of the control structure (see

Appendix F). Placing the fish passage on the west side of the control structure will prevent fish already swimming upstream from having to move downstream to reach the unprotected reach of the Wild Rice River. The downstream ends of the fish passage channels will merge in the protected reach of the Wild Rice River, downstream of the hydraulic jump. The fish passage includes 14 and 18 sequences of pools and riffles for the lower and upper channels, respectively. The downstream end of the fish passage will be located as close to the control structure as possible to provide an alternative route for fish that reach that point but cannot proceed through the control structure due to high velocities.

F-G1.6.4 Fish Passage Location – Rush River Control Structure (LPP)

The design described in this subsection includes one fish passage channel including 14 sequences of pools and riffles at the Rush River control structure. The inlet invert at the upstream end of the fish passage channel will be located on the Rush River upstream of the drop structure (see Appendix F) with an invert elevation set at the approximate tributary invert. The fish passage includes 14 sequences of pools and riffles. The downstream end of the fish passage channel will merge with the diversion channel downstream of the drop structure, at an elevation of approximately 866.58 feet.

F-G1.6.5 Fish Passage Location – Lower Rush River Control Structure (LPP)

The design described in this subsection includes one fish passage channel including 17 sequences of pools and riffles at the Lower Rush River control structure. The inlet invert at the upstream end of the fish passage channel will be located on the Lower Rush River upstream of the drop structure (see Appendix F) with an invert elevation set at the approximate tributary invert. The fish passage includes 17 sequences of pools and riffles. The downstream end of the fish passage channel will merge with the diversion channel downstream of the drop structure, at an elevation of approximately 869.12 feet.

F-G1.7 FISH PASSAGE DESIGN DISCUSSION

It was necessary to update the Phase 3 design due to the use of a HEC-RAS unsteady flow model and the need for upstream staging. Due to the increase in the upstream staging in Phase 4, the inlet inverts for the Red River and the Wild Rice River fish passages were necessarily raised to provide proper operation. However, the downstream water surface elevations remained similar to the Phase 3 water surface elevations, which necessitated an increase in the length of each fish passage. The length, and subsequently the cost, has nearly doubled when compared to Phase 3.

Three inlet invert elevation alternatives were evaluated for the Red River of the North:

- Alternative 1: Inlet invert based on Phase 3 guidelines (1-ft below the 5-year event water surface elevation).
- Alternative 2: Inlet invert based on operation during the peak water surface elevation of the 50-year design storm with two fish passages.
- Alternative 3: Inlet invert set at the approximate water surface elevation for which the gates become fully opened during the receding limb of the hydrograph.

Since the gate operations of the Wild Rice River remain relatively closed for the duration of the storm event, Alternative 3 was not evaluated for this control structure fish passage. In addition, the Rush and Lower Rush have set inlet and outlet invert elevations and will remain similar to their Phase 3 design.

Each alternative was evaluated for the number of days the control structure and fish passage combination will not allow fish passage and the number of days it will allow fish passage for the 10-year, 50-year, 100-year, and 500-year hydrographs. Based on this information, the alternative which provides the greatest amount of service can be chosen. Alternative 1 and 2 for both the Red River and Wild Rice River control structures produced nearly identical results for flood events evaluated. Alternative 2 has a slightly lower inlet invert which allowed it to include more days on the falling limb of the hydrograph. Conversely, Alternative 1 had a higher inlet invert, which produced more days of operation for the higher events. However, since the smaller events are more frequent, the lower invert of Alternative 2 will provide more days of operation than Alternative 1 over time. In other words, if Alternative 2 had been evaluated for smaller events, i.e. the 5-year design event, Alternative 2 would have provided more days of available fish passage. Alternative 2 is the feasibility design selected for the Red River of the North and the Wild Rice River fish passages.

During this evaluation, it became clear that the second fish passage at both the Red River of the North and Wild Rice River may not be the most cost effective form of environmental impact mitigation. Tables F-G7 and F-G14 show the Red River of the North lower fish passage will be open for 6 days in the 10-year event and the Wild Rice River lower fish passage will remain open 5 days in the 10-year event. This is a very short duration of operation, however, the lower fish passage will also be effective for smaller events, and these events will be more frequent. On the other hand, the upper fish passage at both the Red River of the North and the Wild Rice River will only be open for 9 days and 8 days, respectively, for the 50-year event. This event will be infrequent, and providing 9 or 8 days of fish passage may not be worth the price tag for each fish passage.

F-G2.0 VELOCITIES IN THE DIVERSION CHANNEL

The following section presents velocity profiles in the LPP and FCP Diversion Channels. The velocity profiles present expected average velocities that fish would encounter if they were to enter the Diversion Channel. The plots are taken from the HEC-RAS models.

In general, positive spikes represent bridges where roads cross the diversion channel. The negative spike for the 10-, 50-, and 500-year events occurs upstream of the diversion inlet structure. The slope of the diversion channel upstream of the inlet structure is opposite the flow direction, which makes this area a large pool for these flood events.

Figures F-G19 to F-G23 present velocities along the LPP Diversion Channel. Figures F-G24 to F-G38 present the velocity profiles for the 10-, 50-, 100-, and 500-year events for four locations along the diversion channel. Maximum velocity in the LPP Diversion Channel occurs downstream of the I-29 bridge crossing for the 50-, 100-, and 500-year events. Maximum velocities for these events are approximately 3.7 fps. For the 10-year event the maximum velocity occurs at the inlet weir located upstream of the Sheyenne River aqueduct. The maximum velocity for the 10-year event is approximately 3.4 fps. For the 10-year event, the higher velocities in the LPP Diversion Channel occur upstream in the diversion channel. For the 50-year event and greater, the higher velocities in the LPP Diversion Channel occur downstream in the diversion channel, where the flows are higher.

Figures F-G39 to F-G46 present velocities along the FCP Diversion Channel for Year 0 hydrology. Figures F-G47 to F-G54 present velocities along the FCP Diversion Channel for Year 25 hydrology. Figures F-G55 to F-G62 present velocities along the FCP Diversion Channel for Year 50 hydrology. Maximum velocity in the FCP Diversion Channel occurs at the inlet weir for all modeled events. Bench width in the Diversion Channel affects the velocities. For events greater than the 10-year, the lowest velocities in the FCP Diversion Channel occur in the reach with 250’ wide benches.

F-G2.1 VELOCITIES IN THE LPP DIVERSION CHANNEL

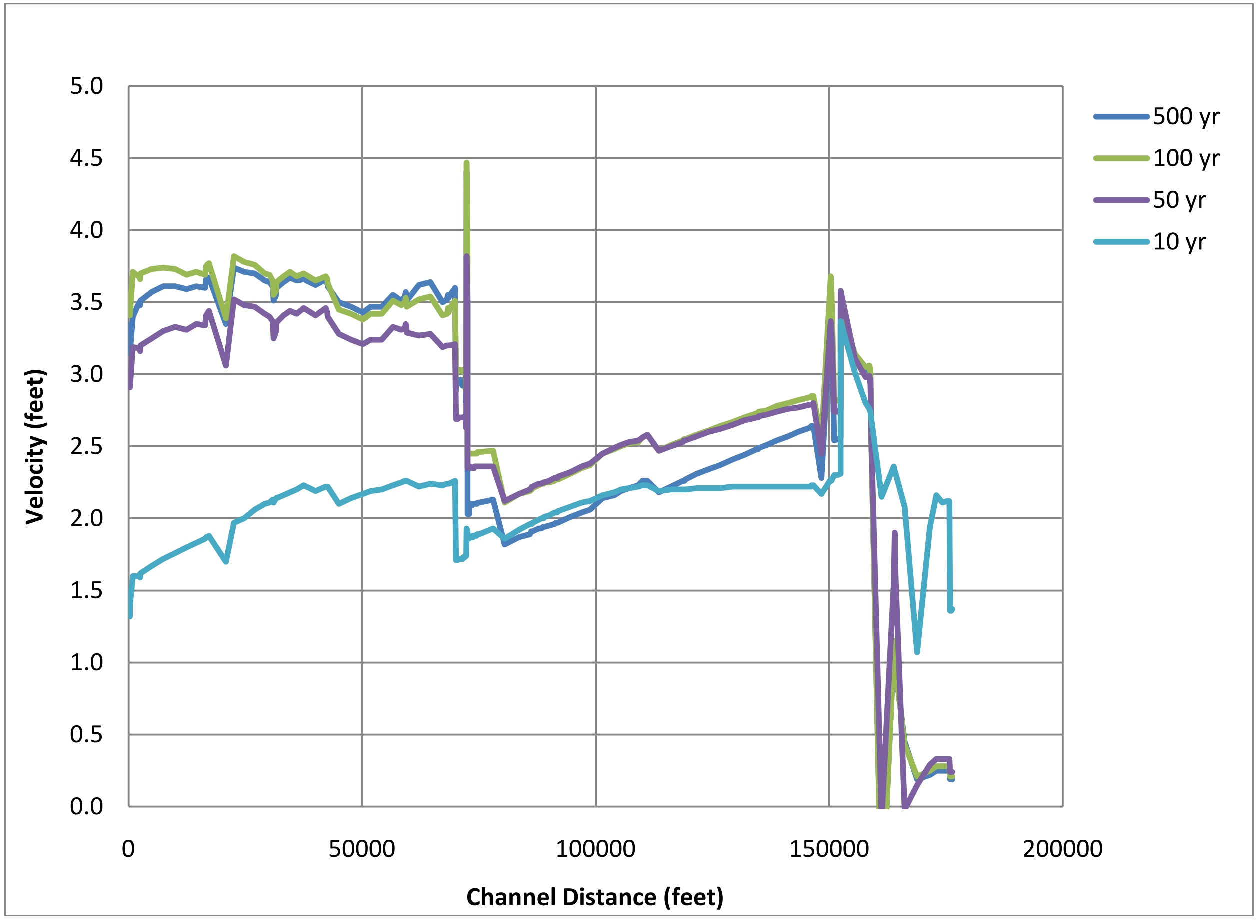

Figure F-G19 LPP Diversion Channel Velocities for All Modeled Events

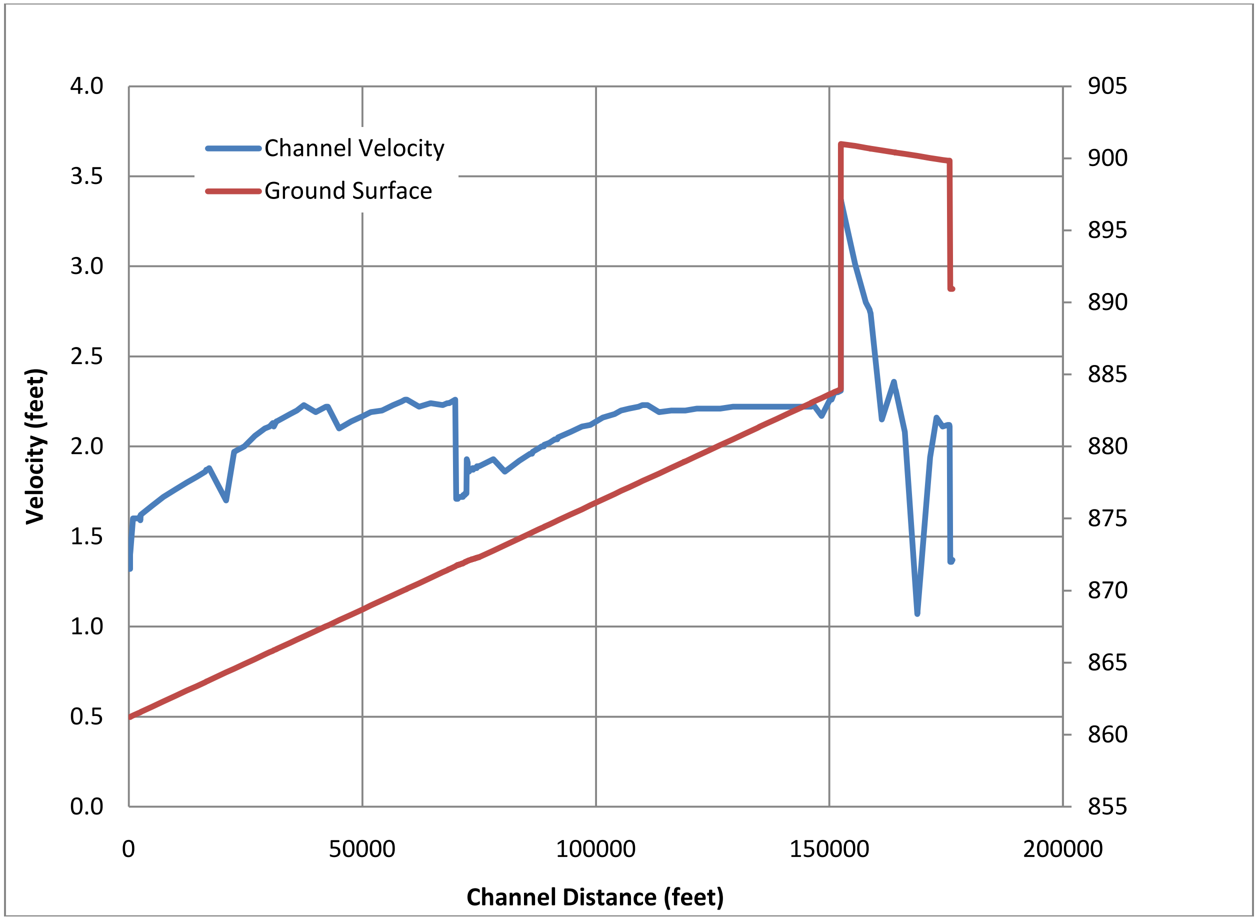

Figure F-G20 LPP Diversion Channel Velocities for the 10-Year Event

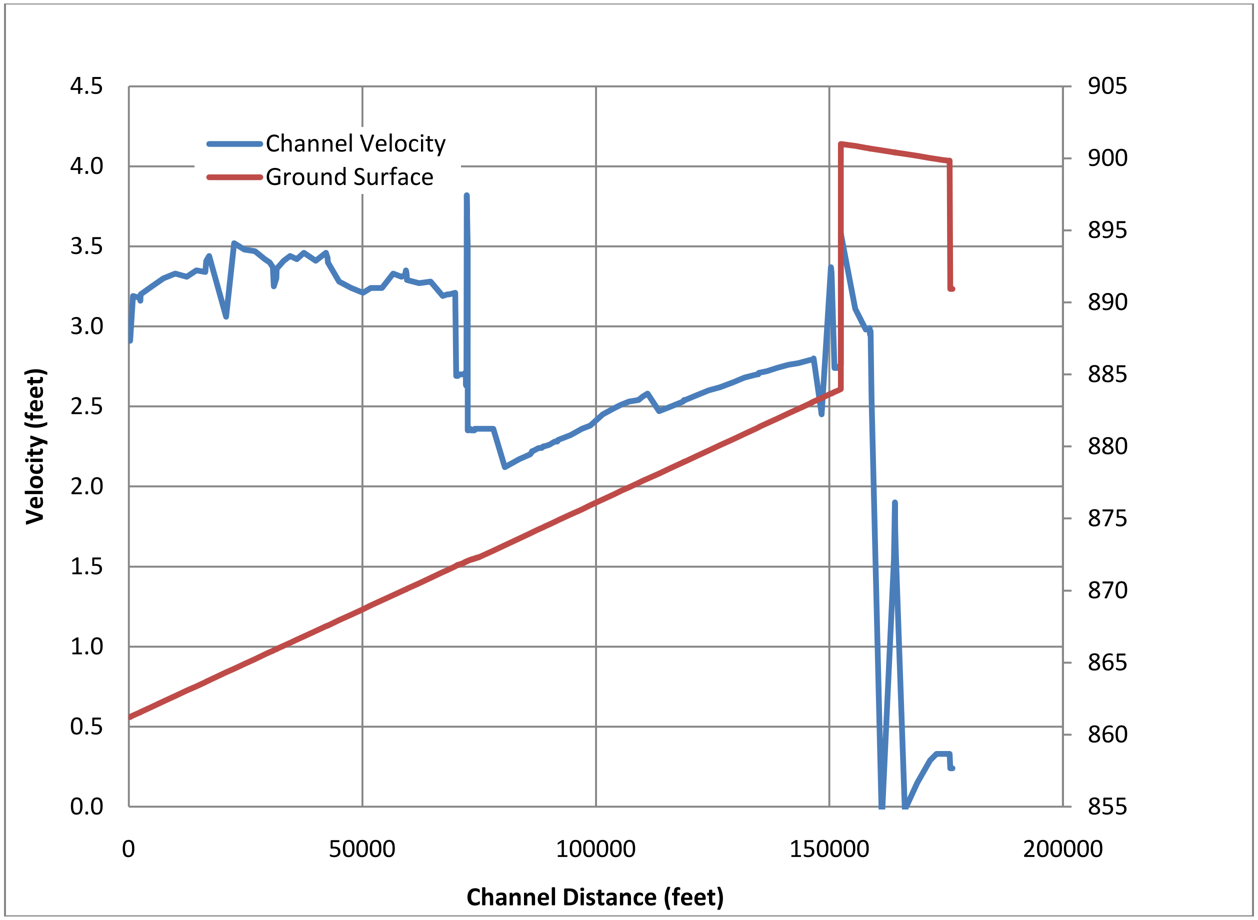

Figure F-G21 LPP Diversion Channel Velocities for the 50-Year Event

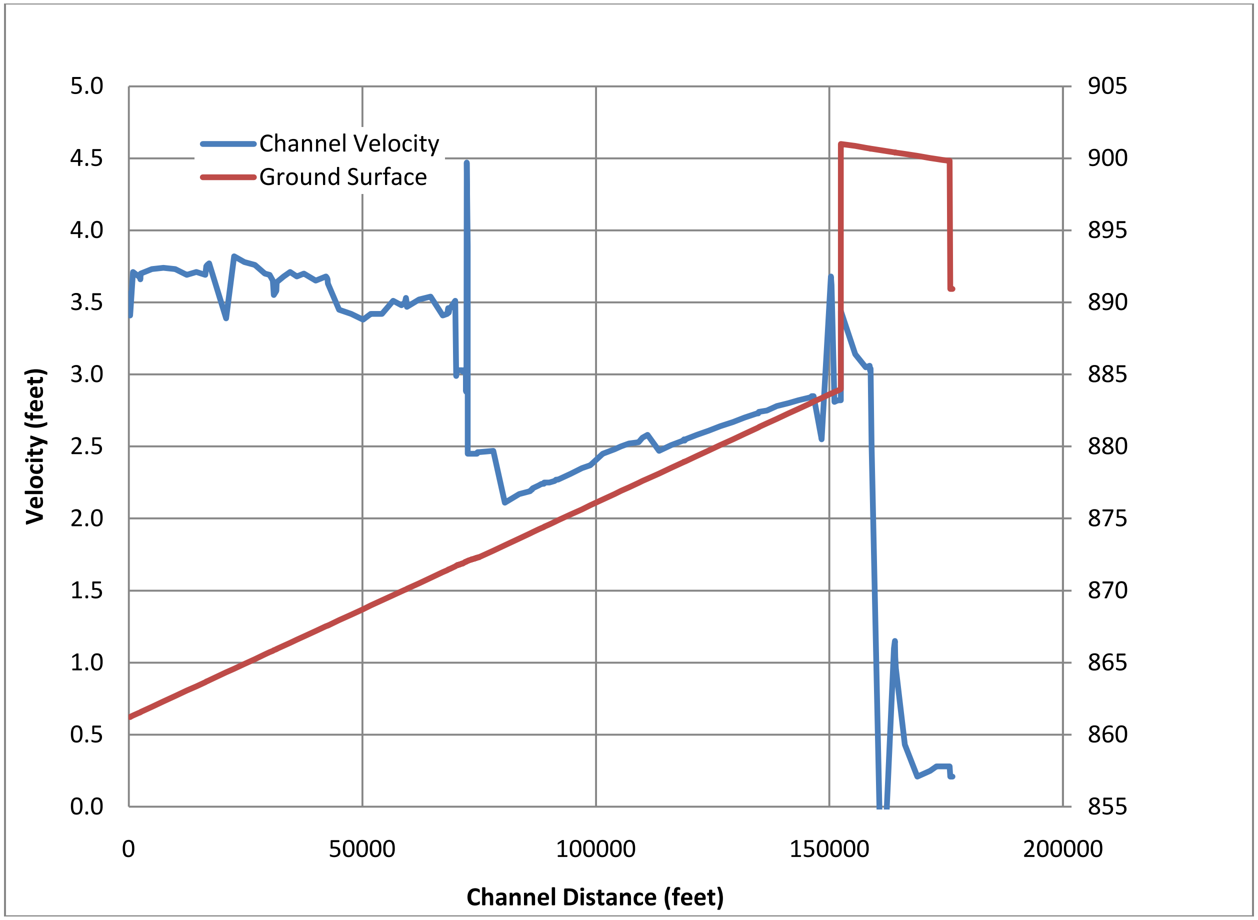

Figure F-G22 LPP Diversion Channel Velocities for the 100-Year Event

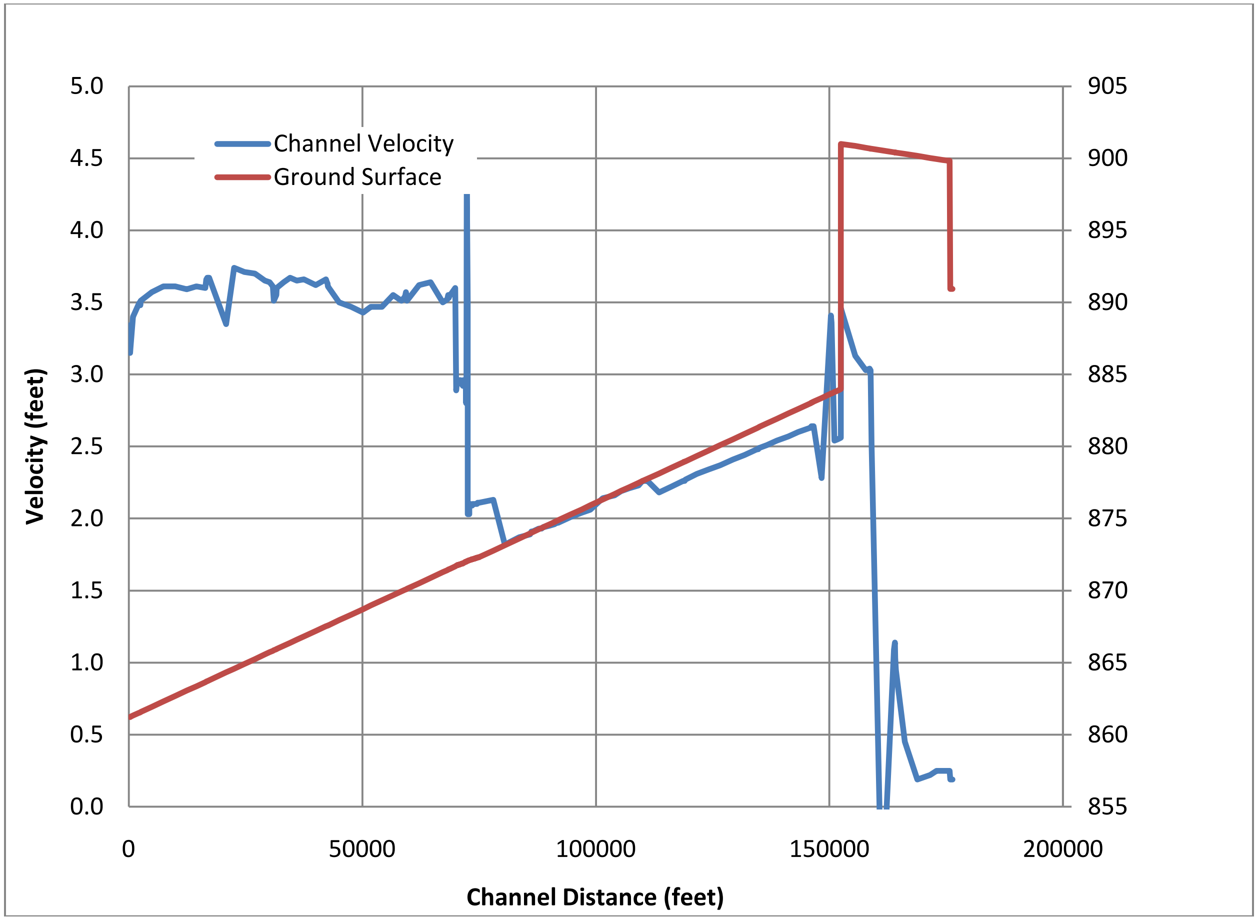

Figure F-G23 LPP Diversion Channel Velocities for the 500-Year Event

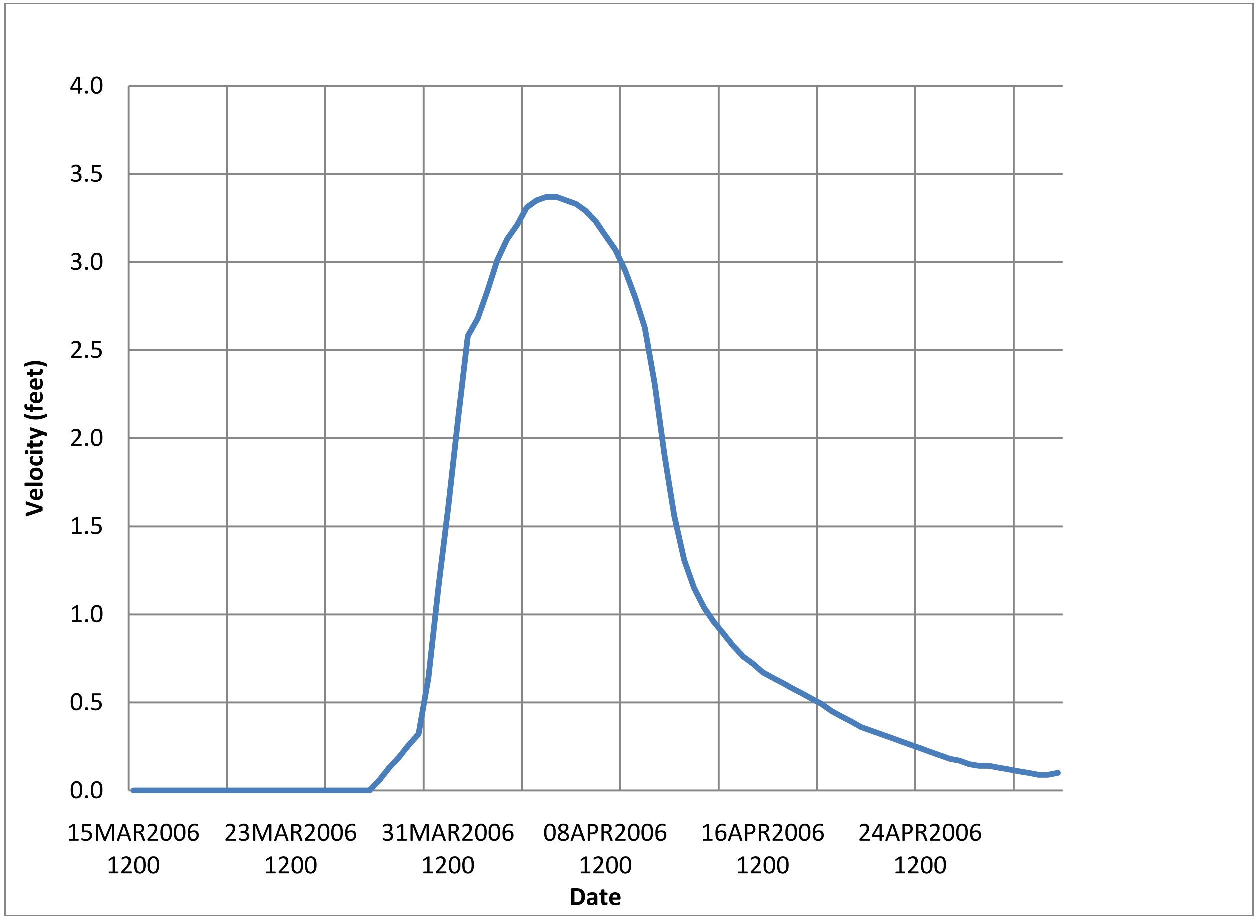

Figure F-G24 LPP Diversion Channel Velocity Profile for the 10-Year Event at Station 152527

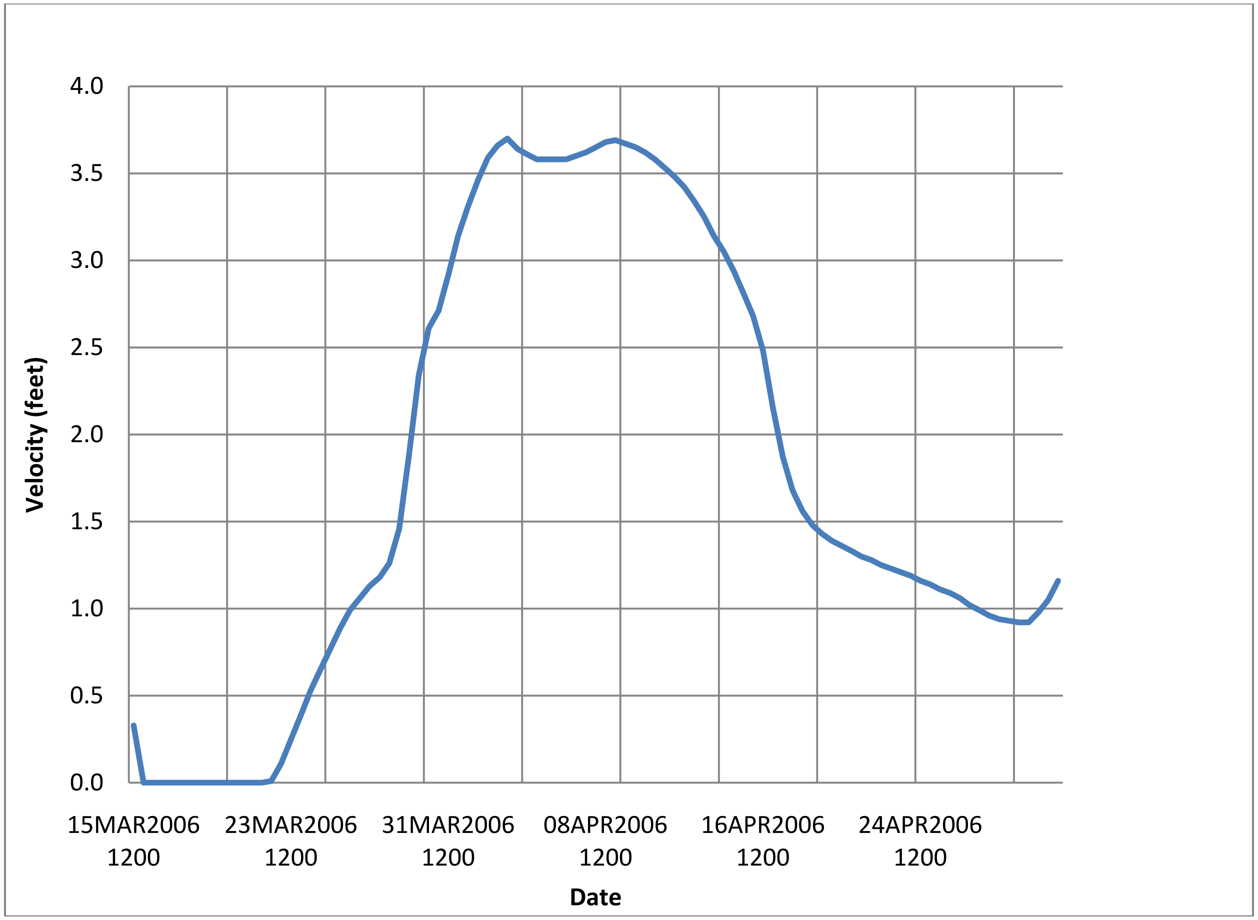

Figure F-G25 LPP Diversion Channel Velocity Profile for the 50-Year Event at Station 152527

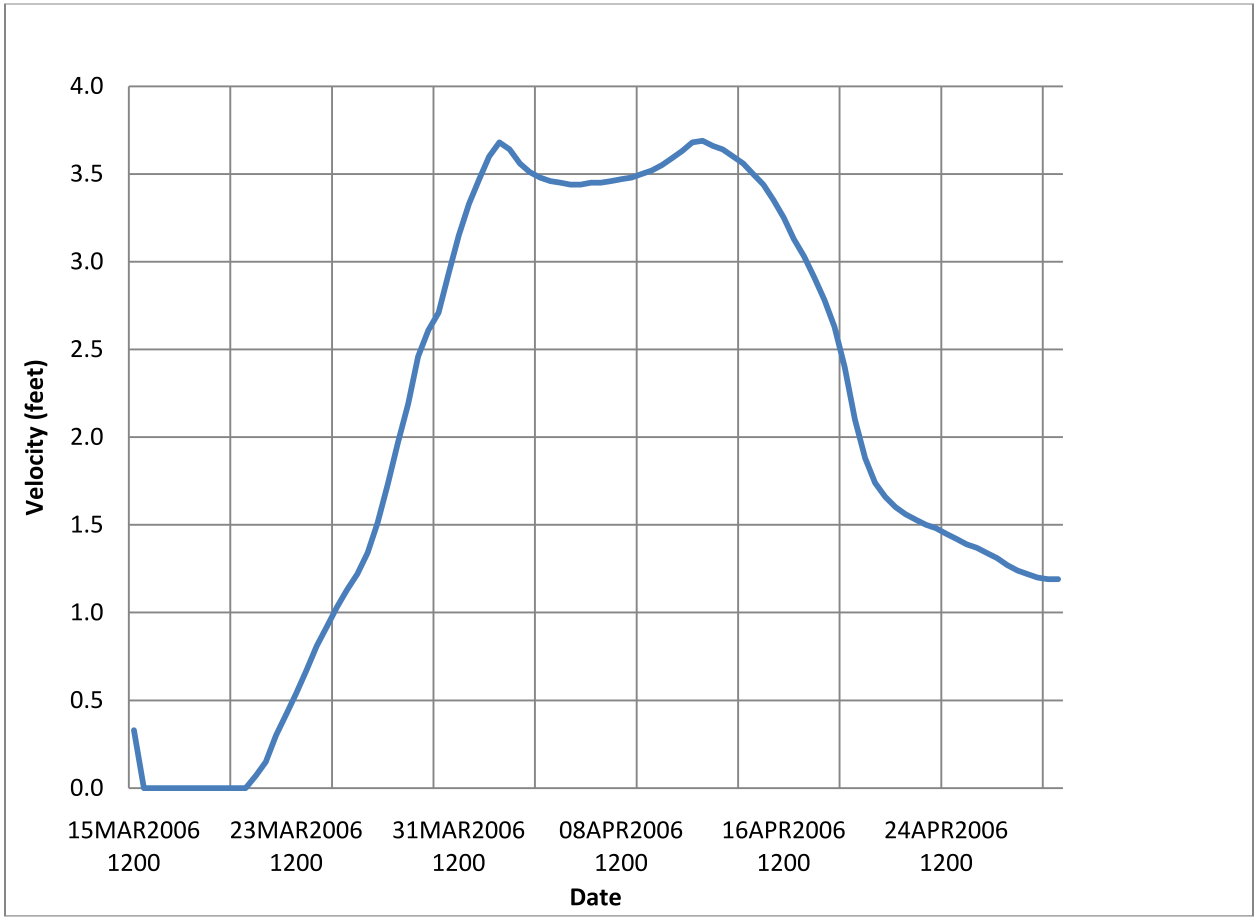

Figure F-G26 LPP Diversion Channel Velocity Profile for the 100-Year Event at Station 152527

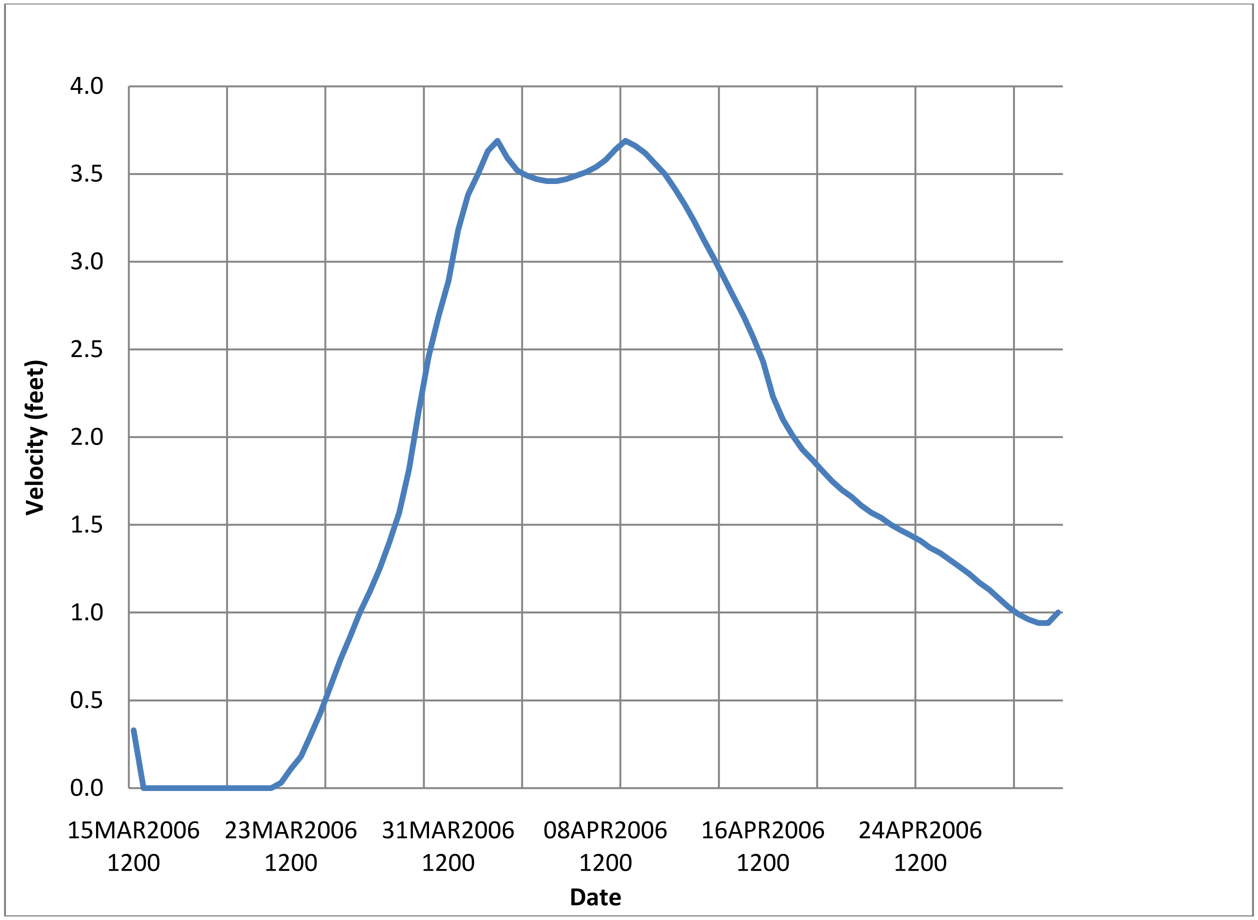

Figure F-G27 LPP Diversion Channel Velocity Profile for the 500-Year Event at Station 152527

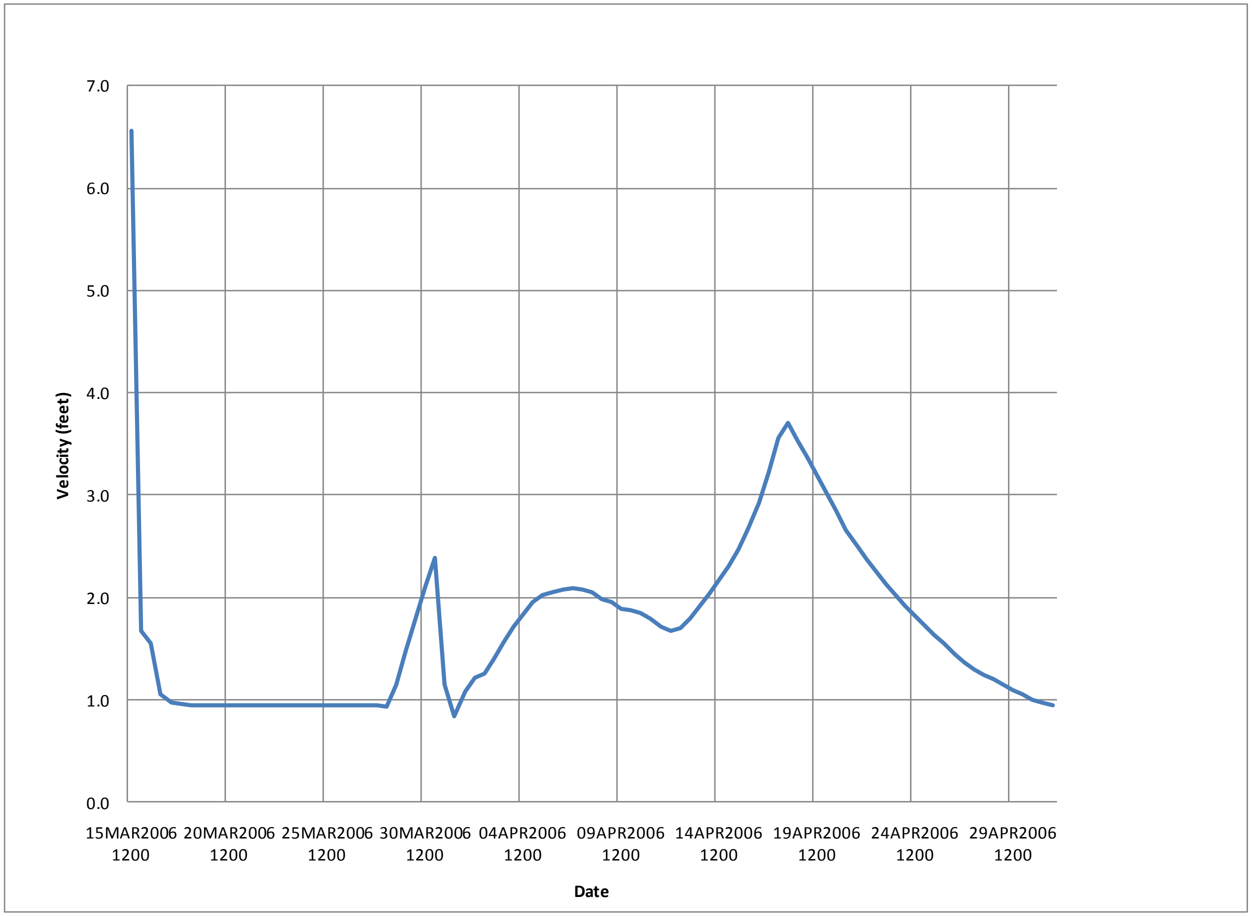

Figure F-G28 LPP Diversion Channel Velocity Profile for the 10-Year Event at Station 83654

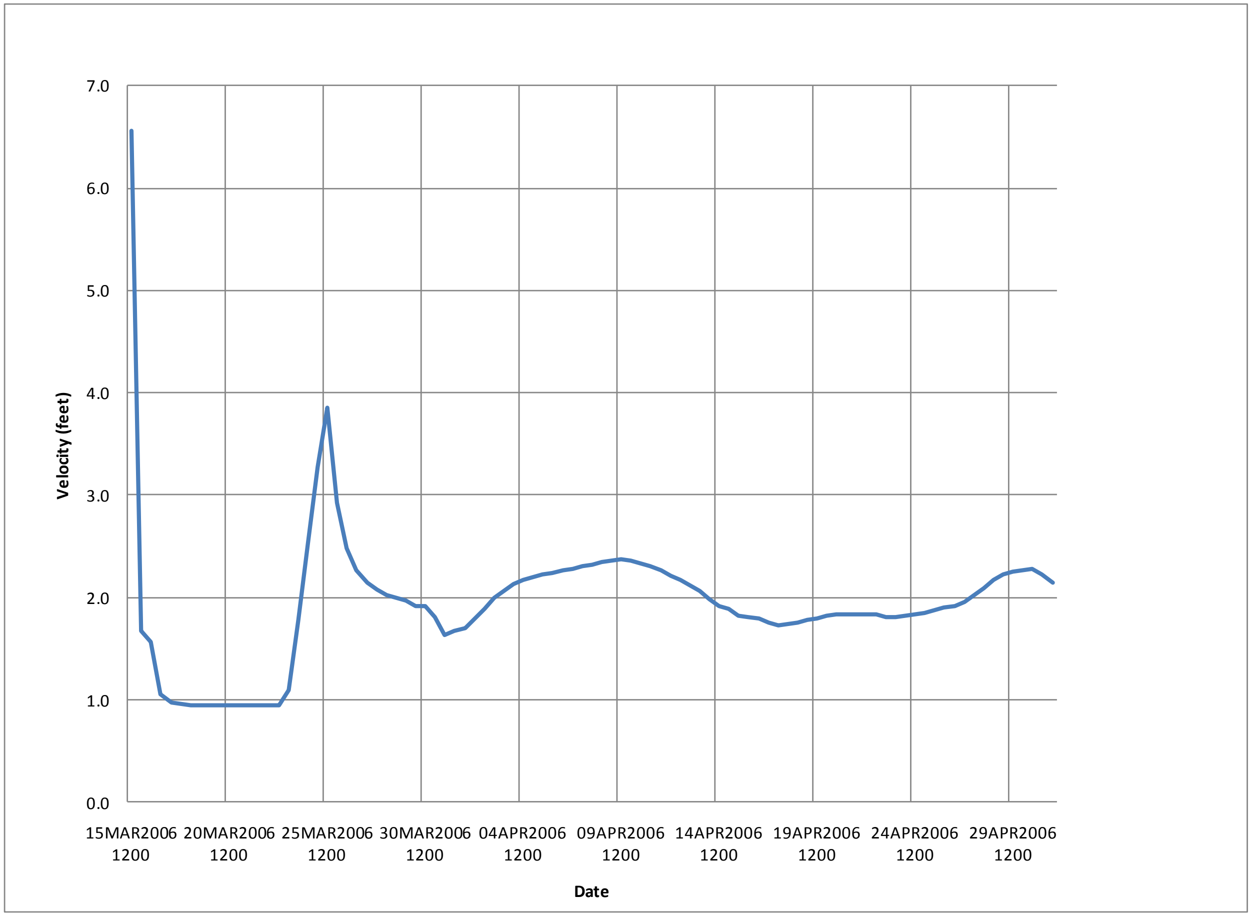

Figure F-G29 LPP Diversion Channel Velocity Profile for the 50-Year Event at Station 83654

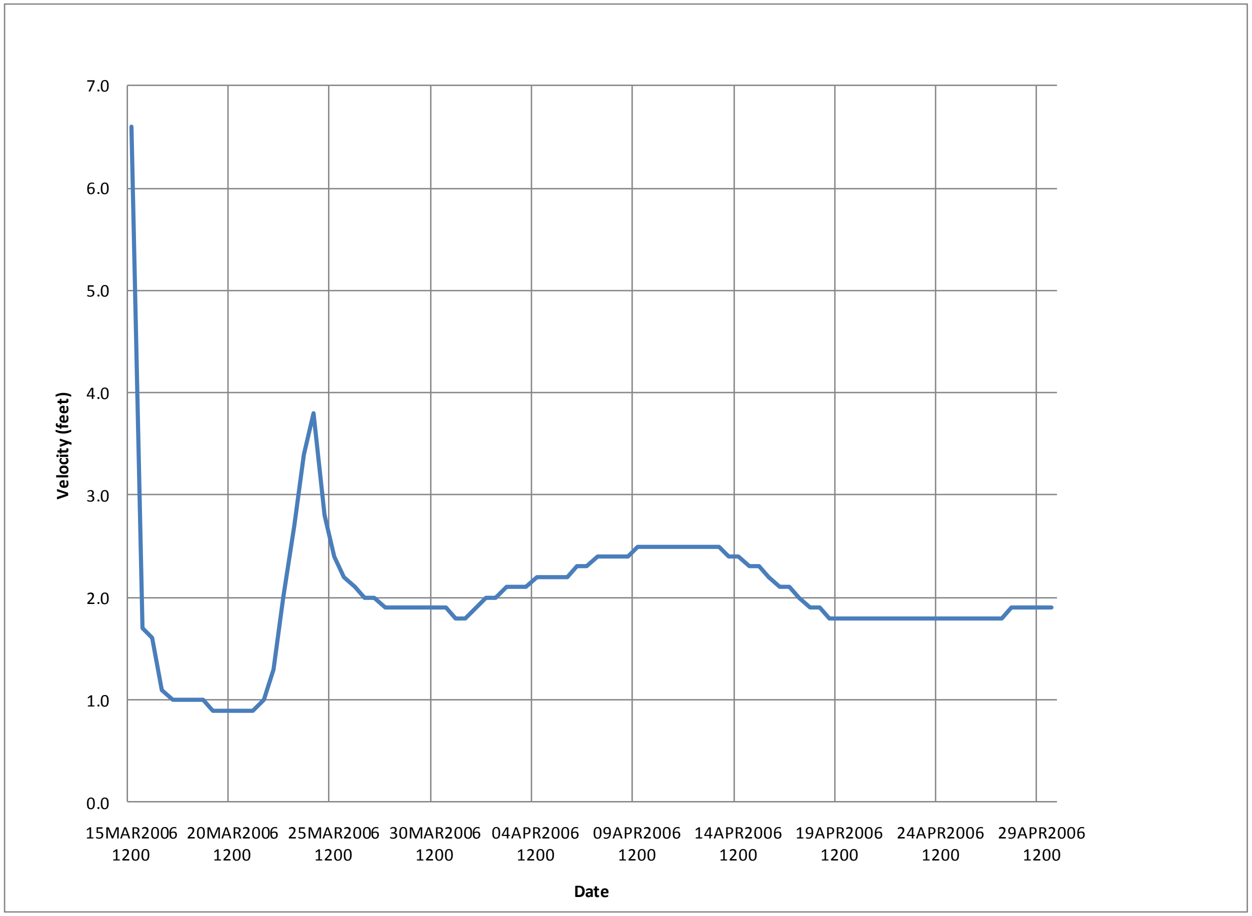

Figure F-G30 LPP Diversion Channel Velocity Profile for the 100-Year Event at Station 83654

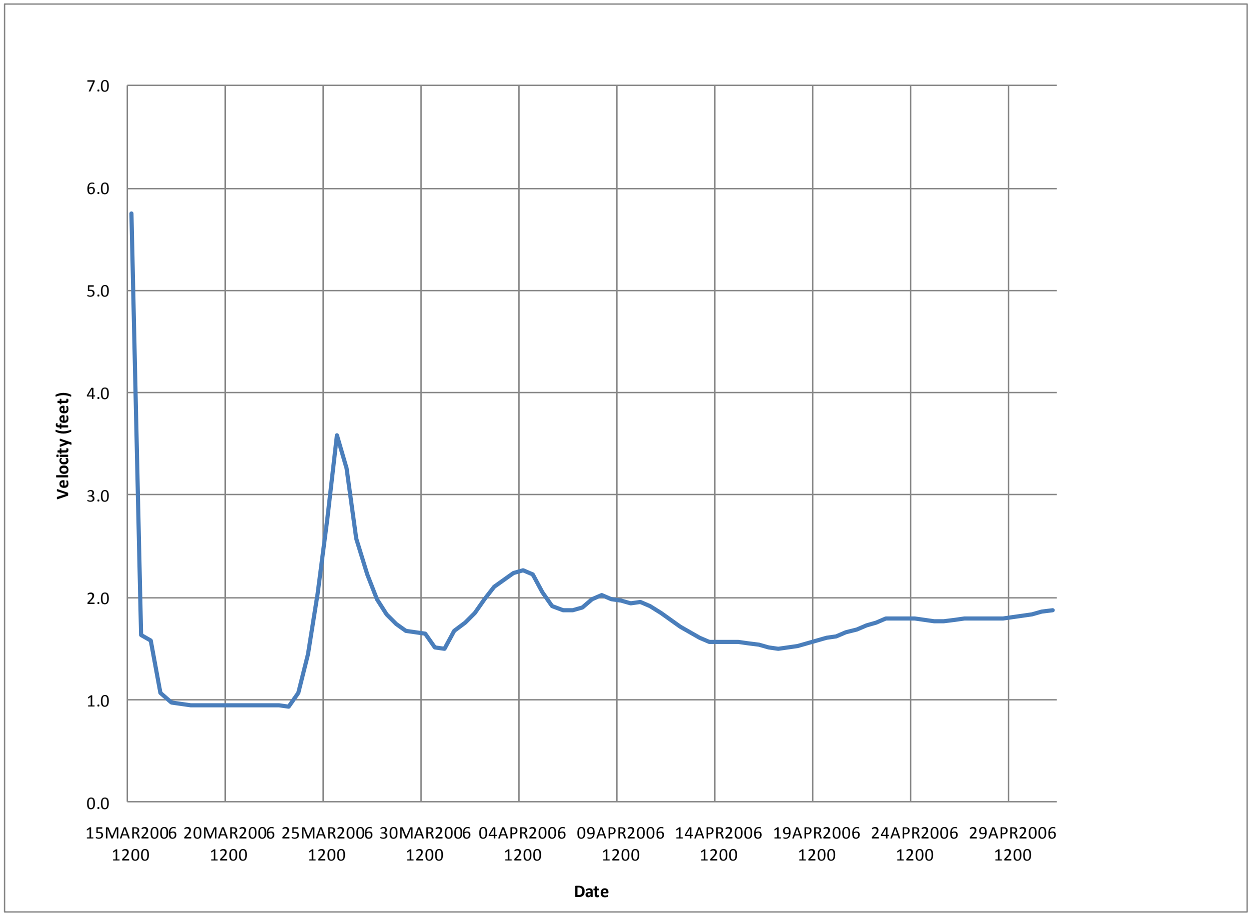

Figure F-G31 LPP Diversion Channel Velocity Profile for the 500-Year Event at Station 83654

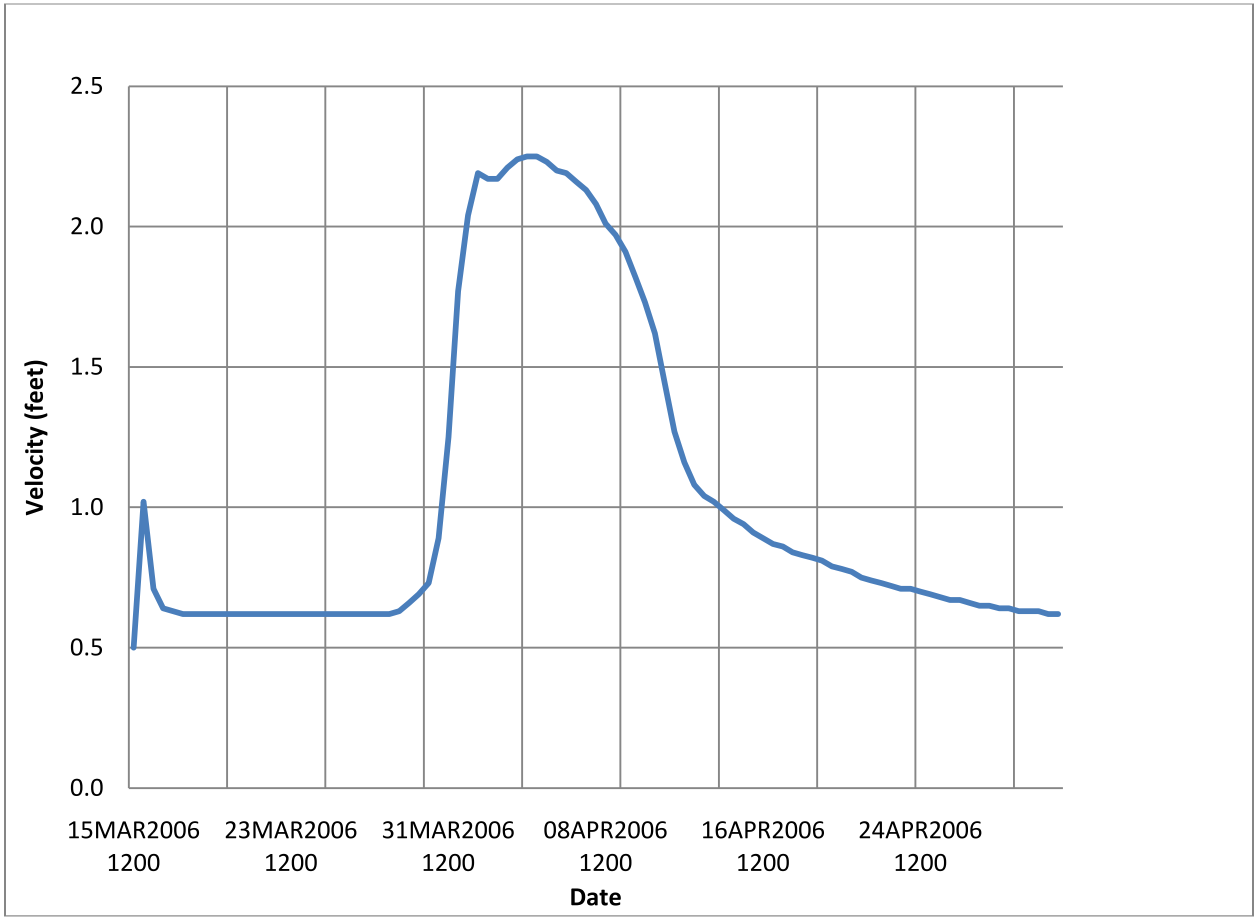

Figure F-G32 LPP Diversion Channel Velocity Profile for the 10-Year Event at Station 64696

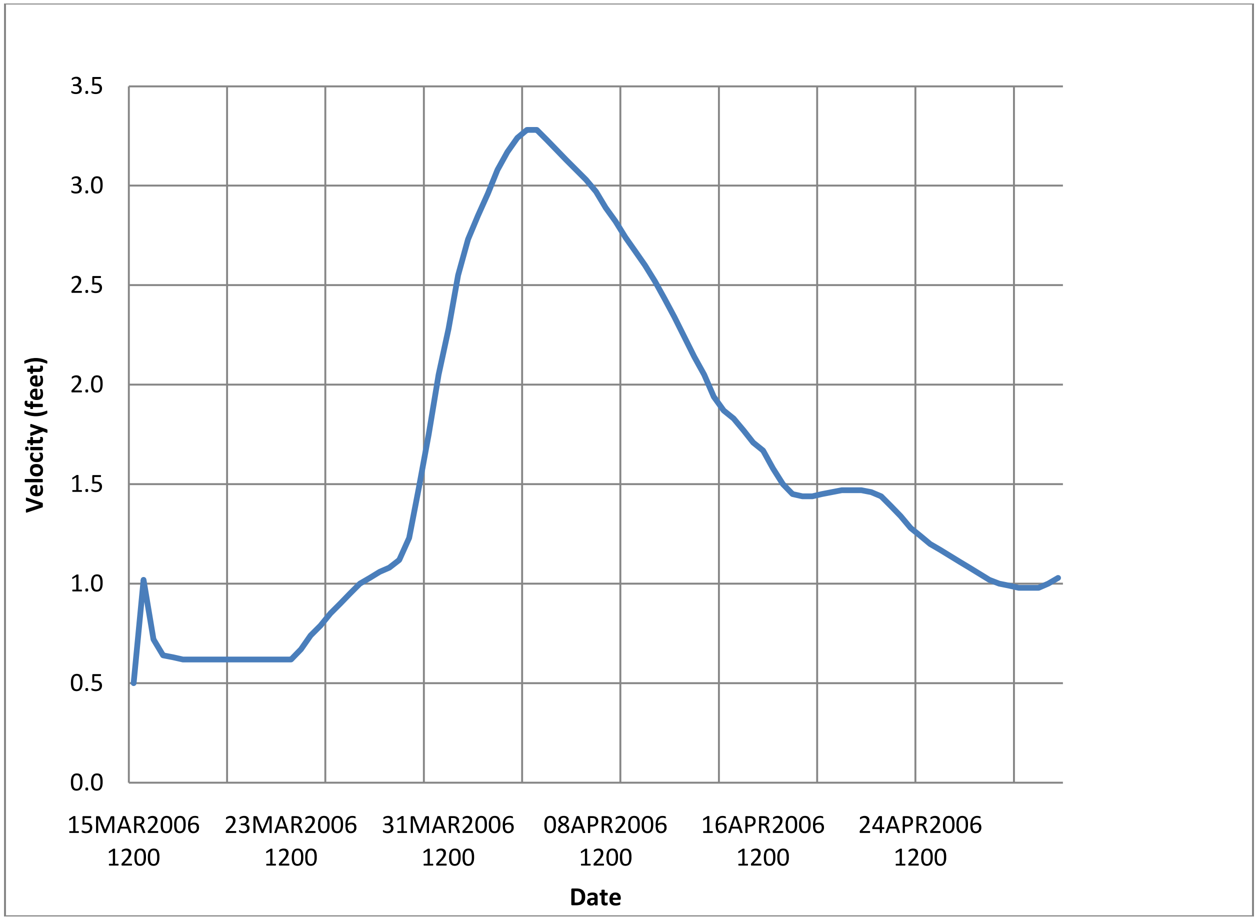

Figure F-G33 LPP Diversion Channel Velocity Profile for the 50-Year Event at Station 64696

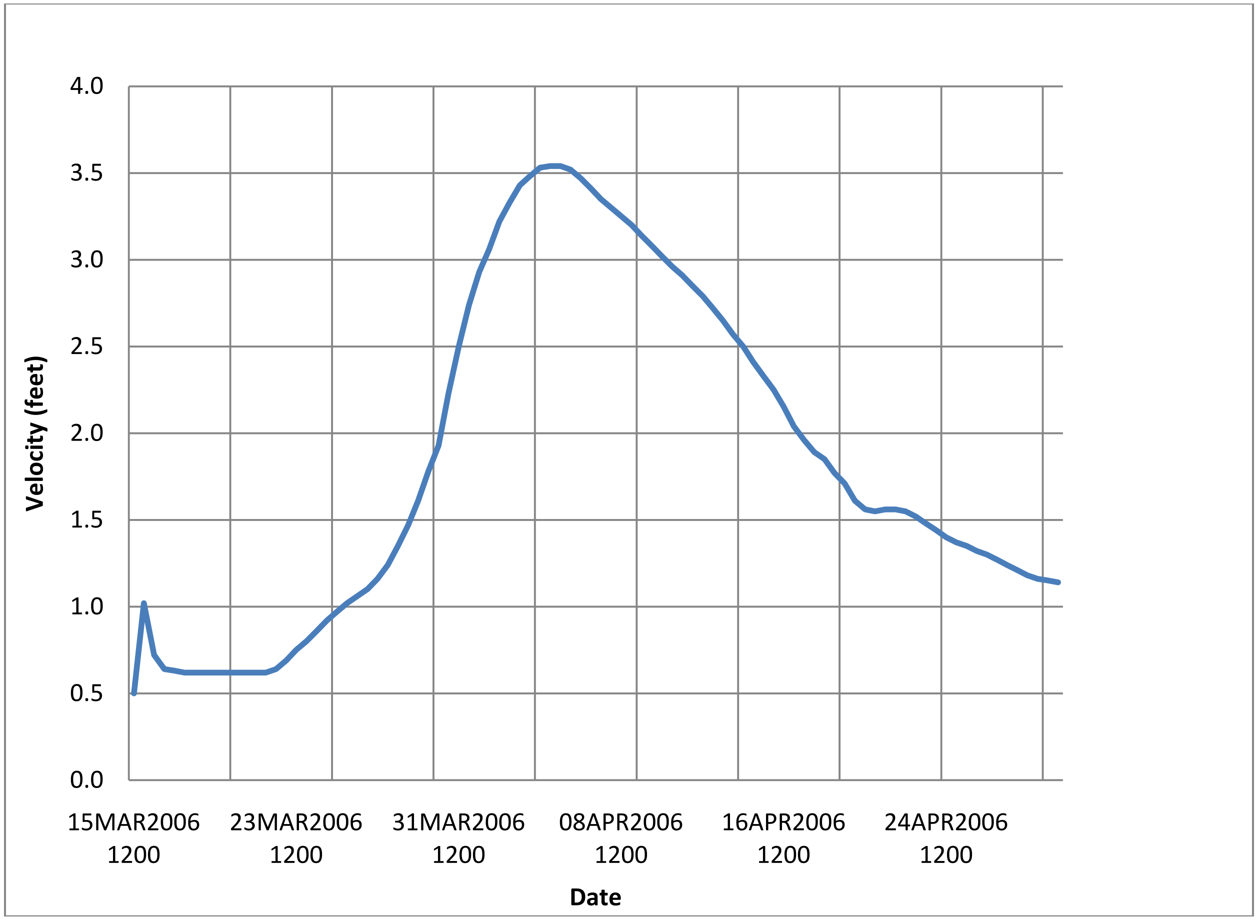

Figure F-G34 LPP Diversion Channel Velocity Profile for the 100-Year Event at Station 64696

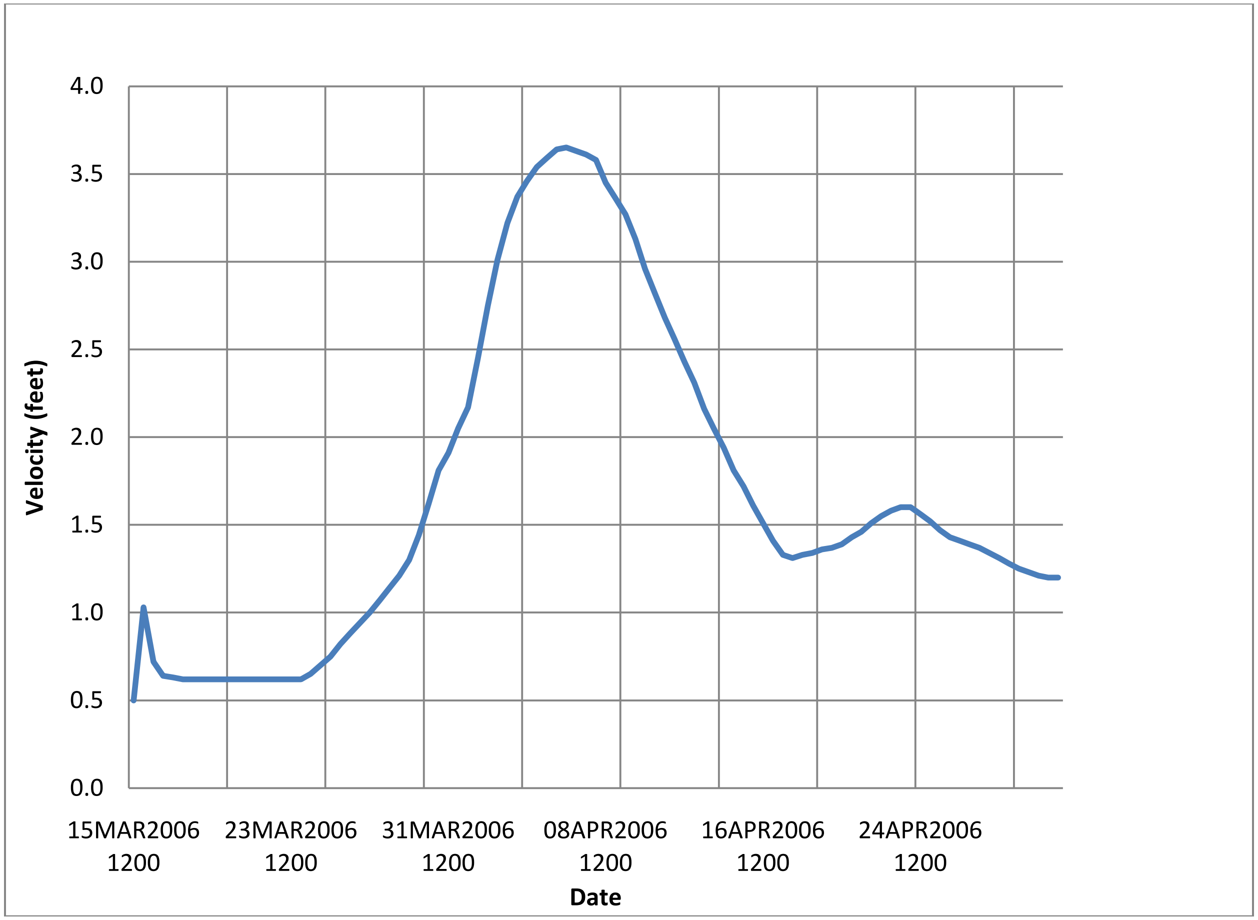

Figure F-G35 LPP Diversion Channel Velocity Profile for the 500-Year Event at Station 64696

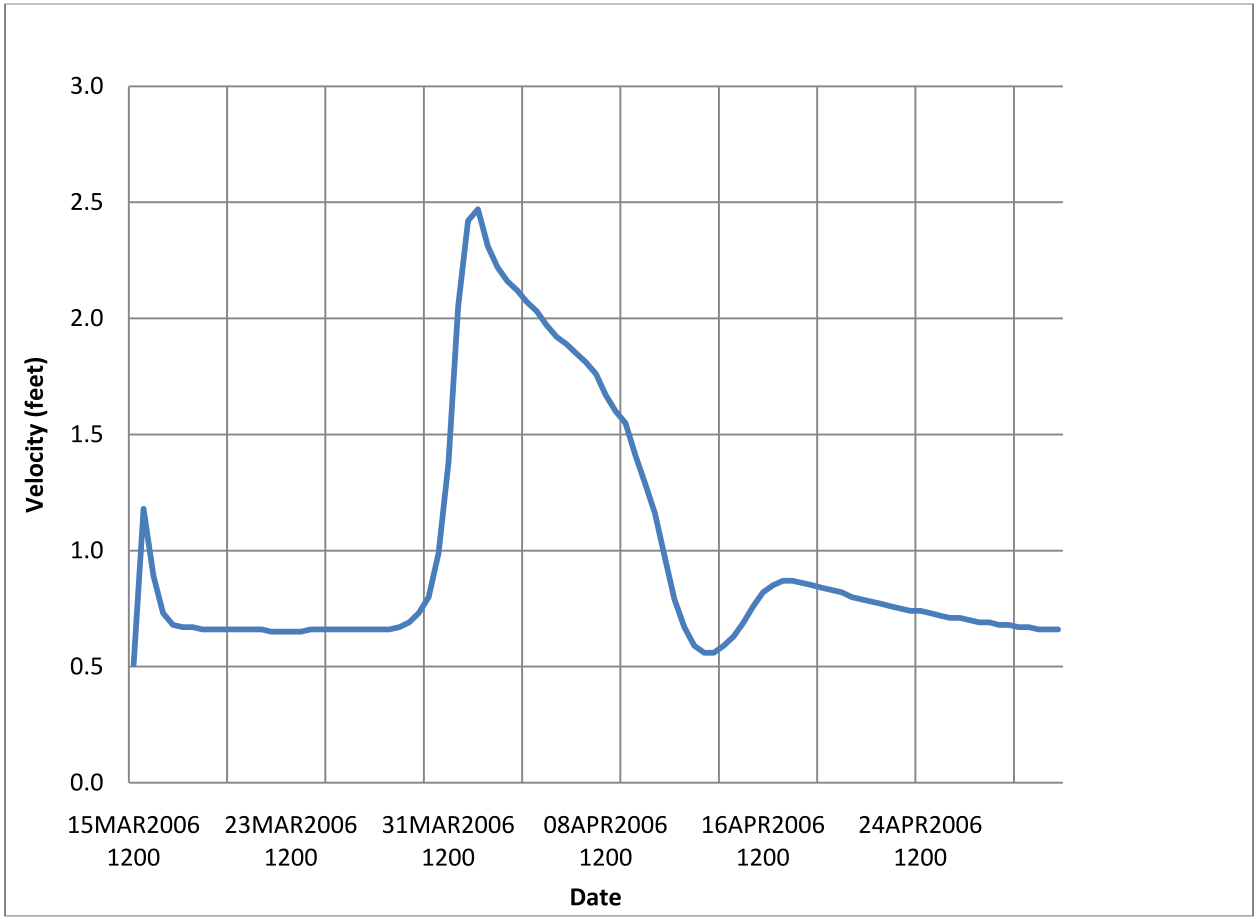

Figure F-G36 LPP Diversion Channel Velocity Profile for the 10-Year Event at Station 29253

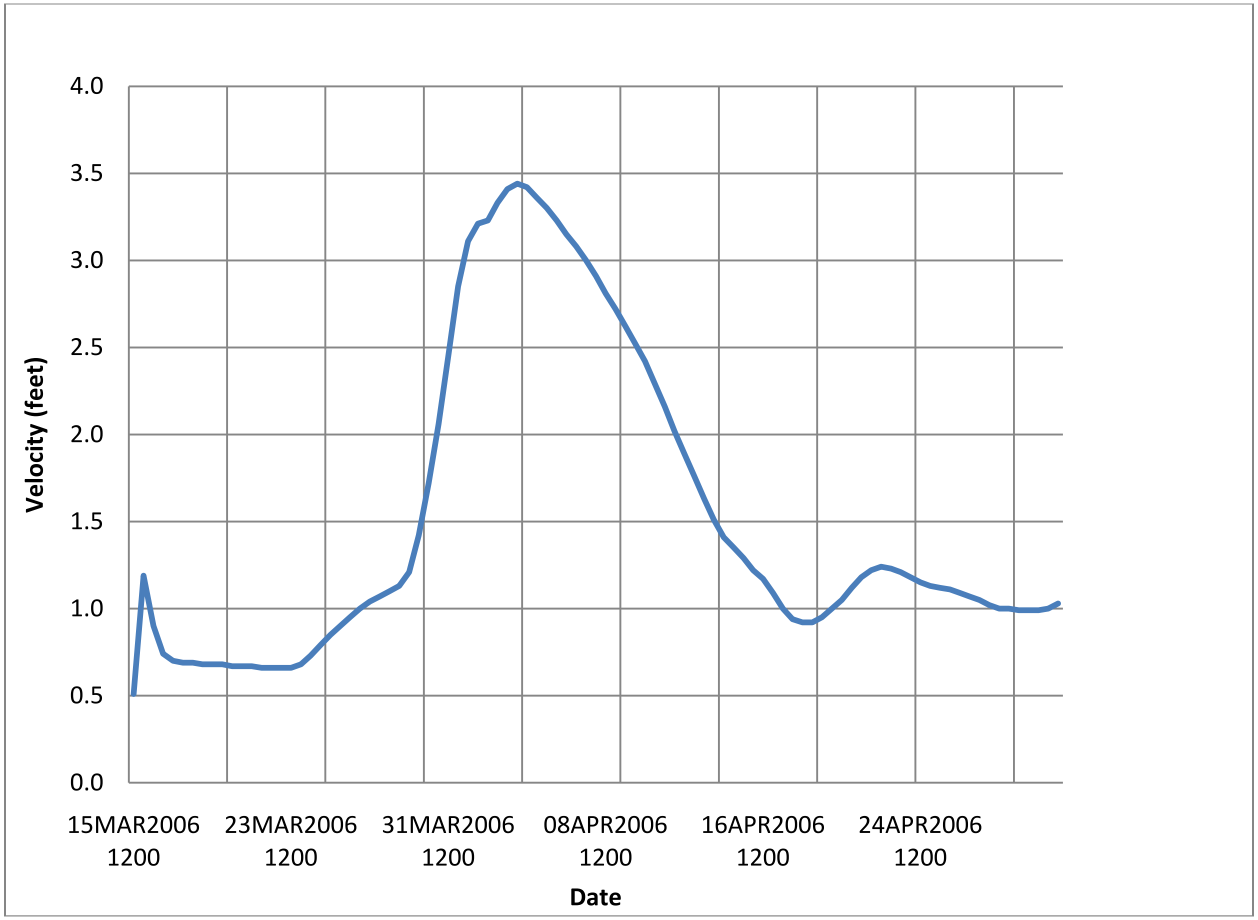

Figure F-G37 LPP Diversion Channel Velocity Profile for the 50-Year Event at Station 29253

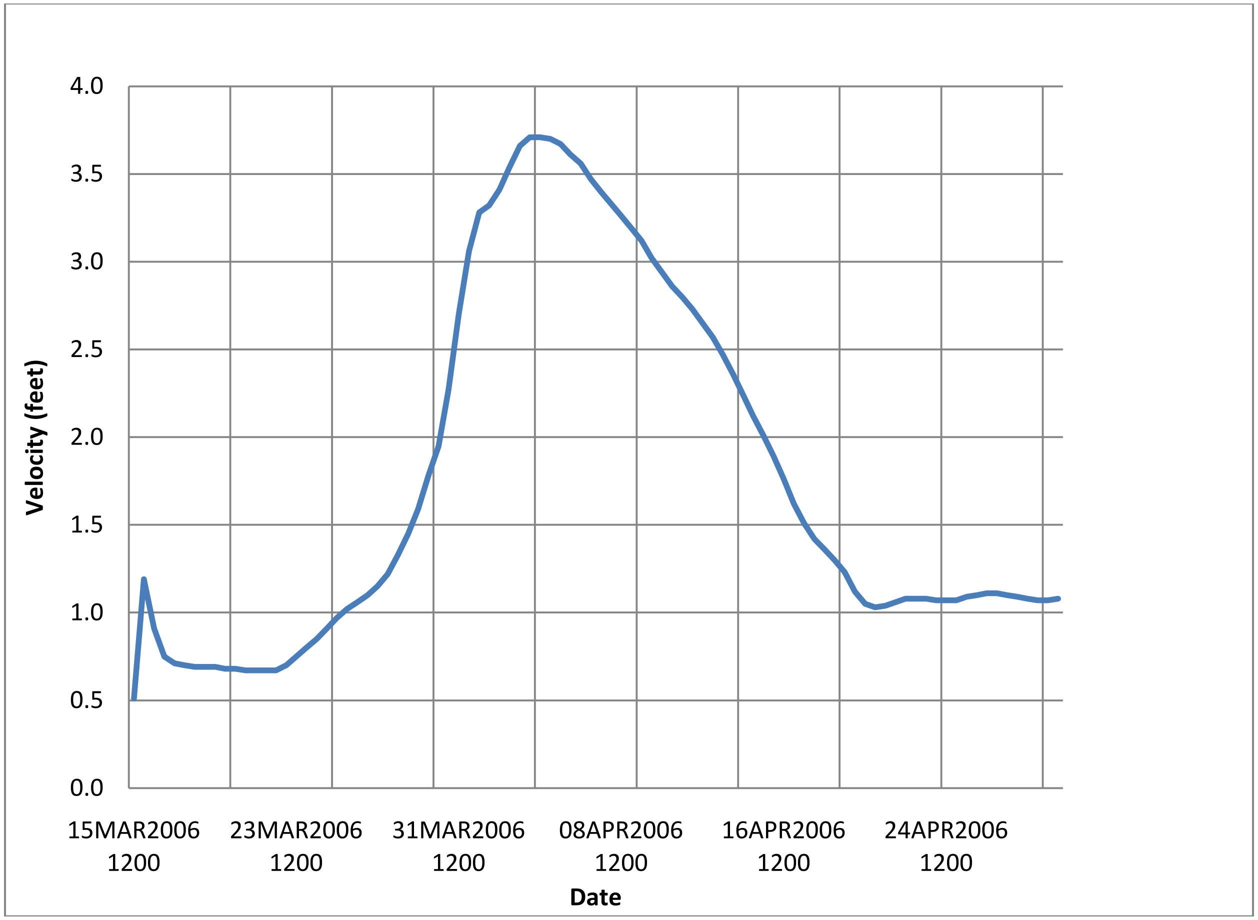

Figure F-G38 LPP Diversion Channel Velocity Profile for the 100-Year Event at Station 29253

F-G2.2 VELOCITIES IN THE FCP DIVERSION CHANNEL – YEAR 0 HYDROLOGY

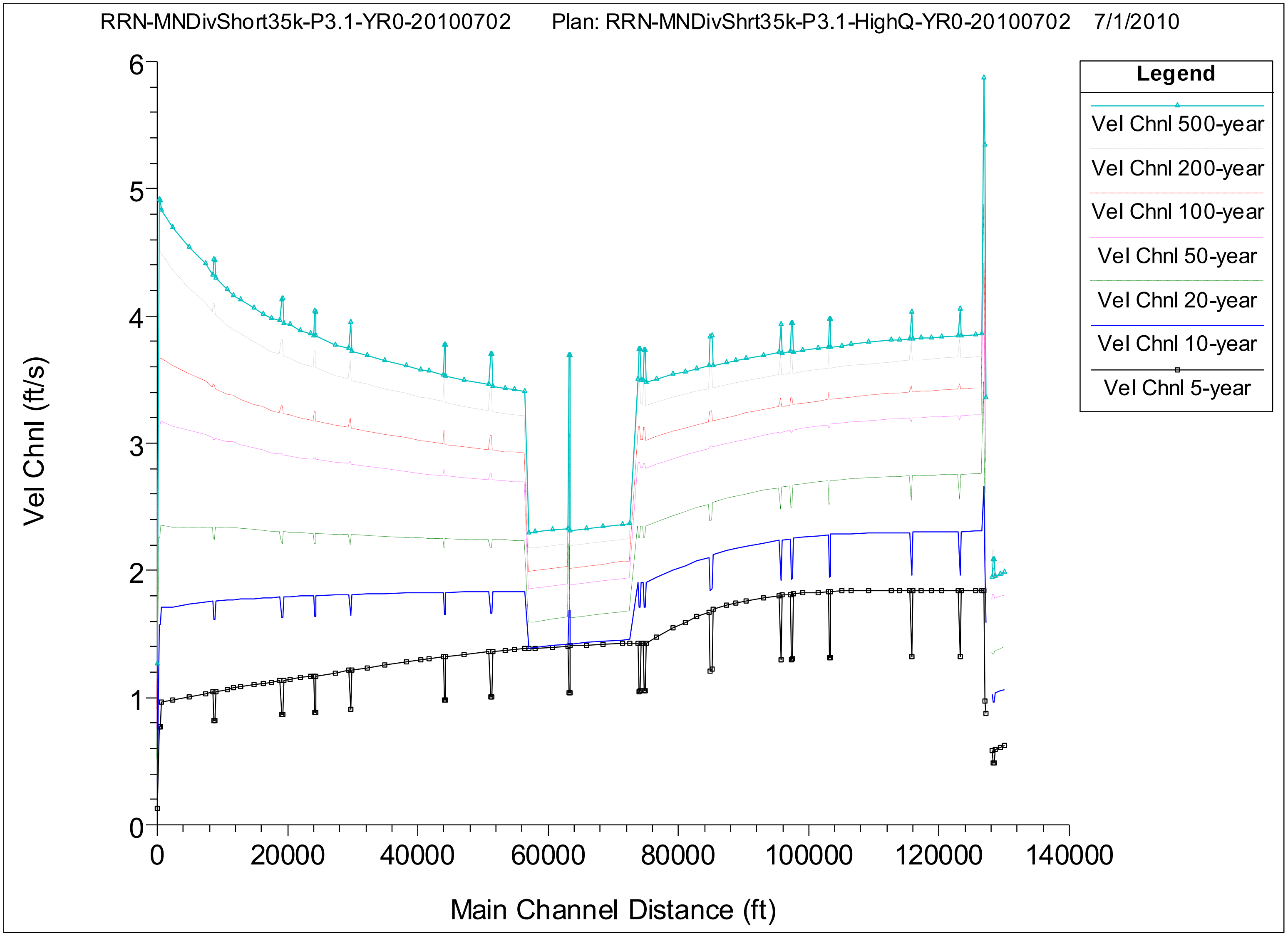

Figure F-G39 FCP Diversion Channel Velocities for All Modeled Events for Year 0 Hydrology

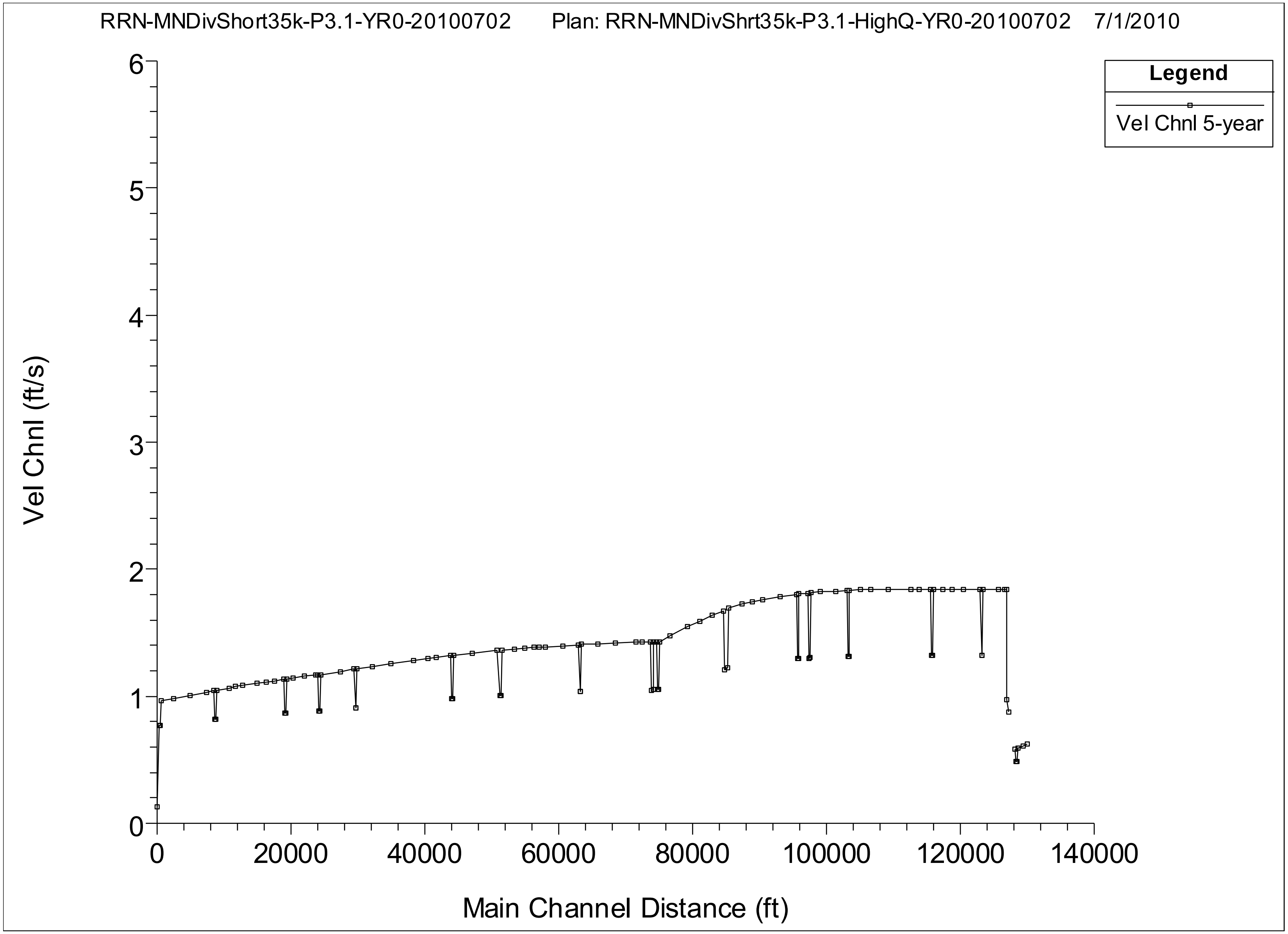

Figure F-G40 **FCP Diversion Channel Velocities for the 5-Year Event for Year 0 Hydrology**

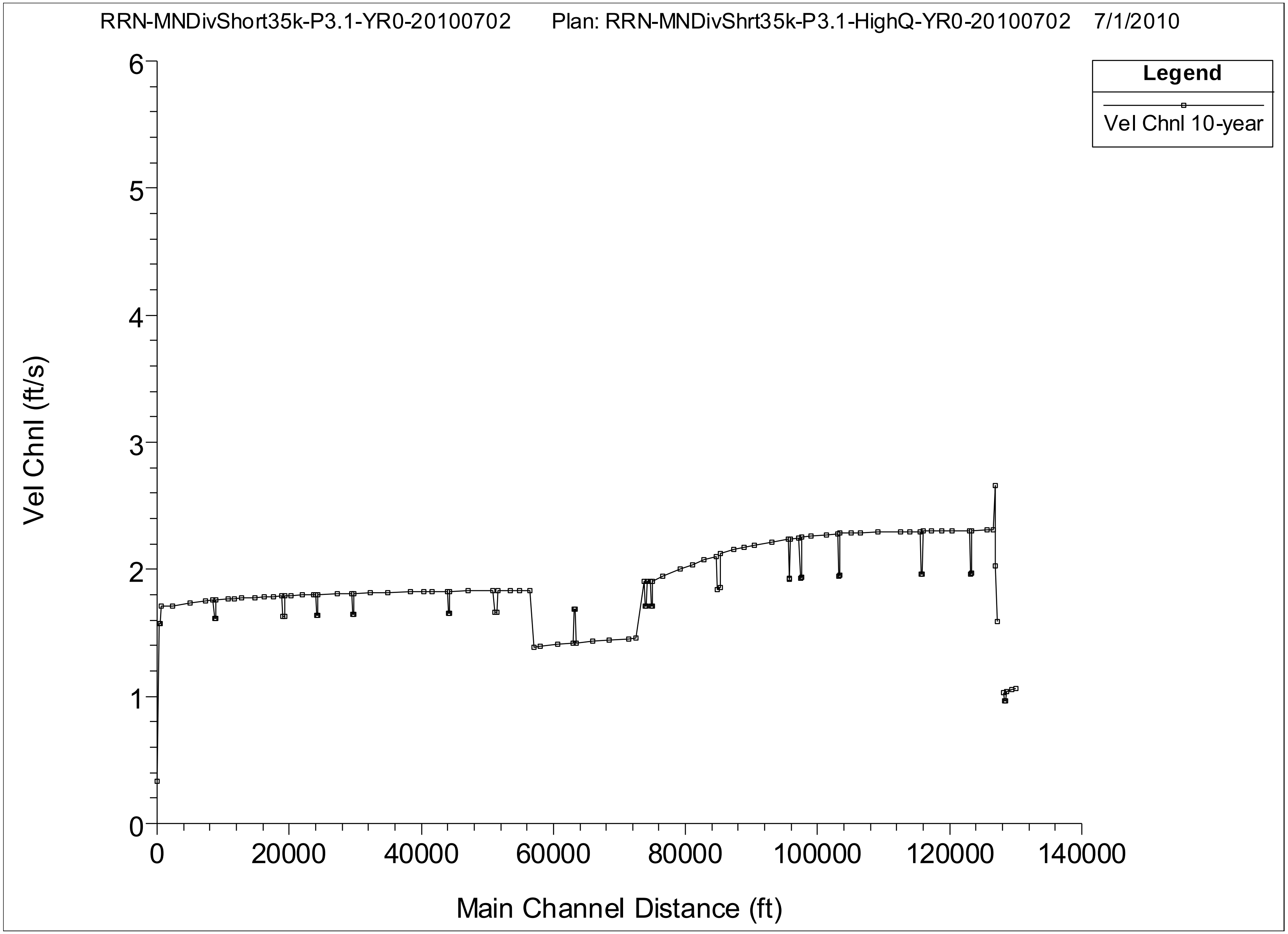

Figure F-G41 FCP Diversion Channel Velocities for the 10-Year Event for Year 0 Hydrology

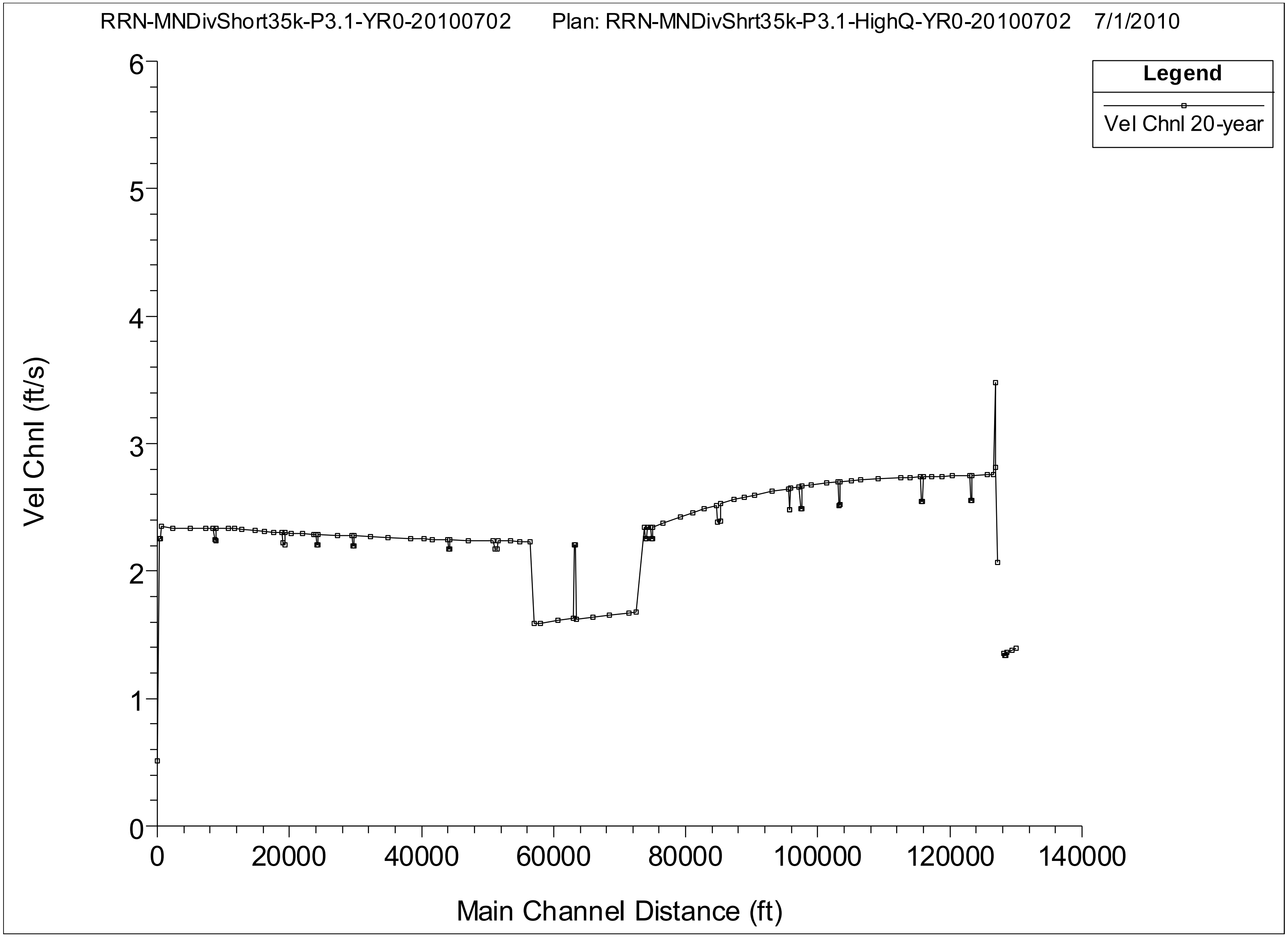

Figure F-G42 FCP Diversion Channel Velocities for the 20-Year Event for Year 0 Hydrology

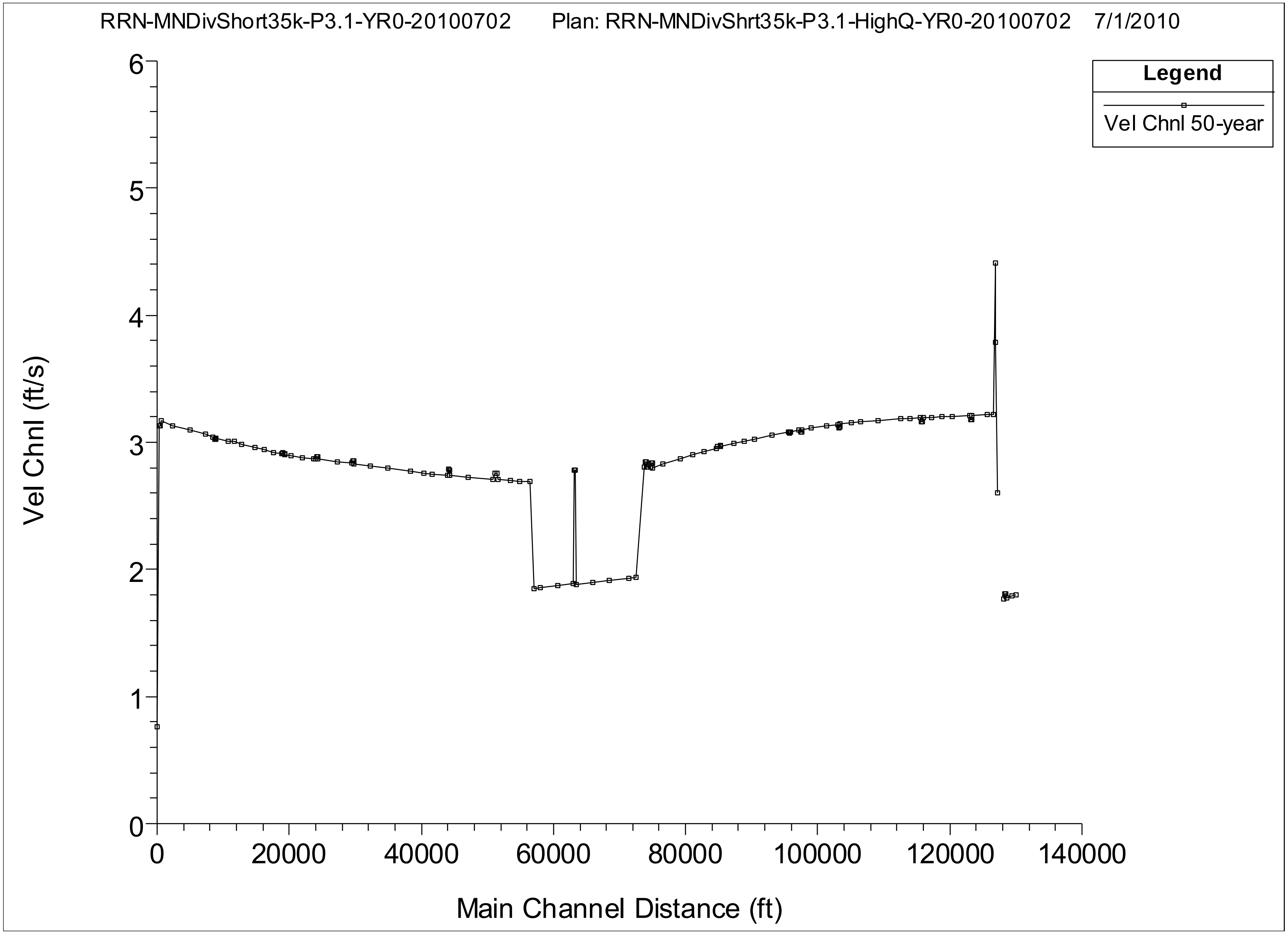

Figure F-G43 **FCP Diversion Channel Velocities for the 50-Year Event for Year 0 Hydrology**

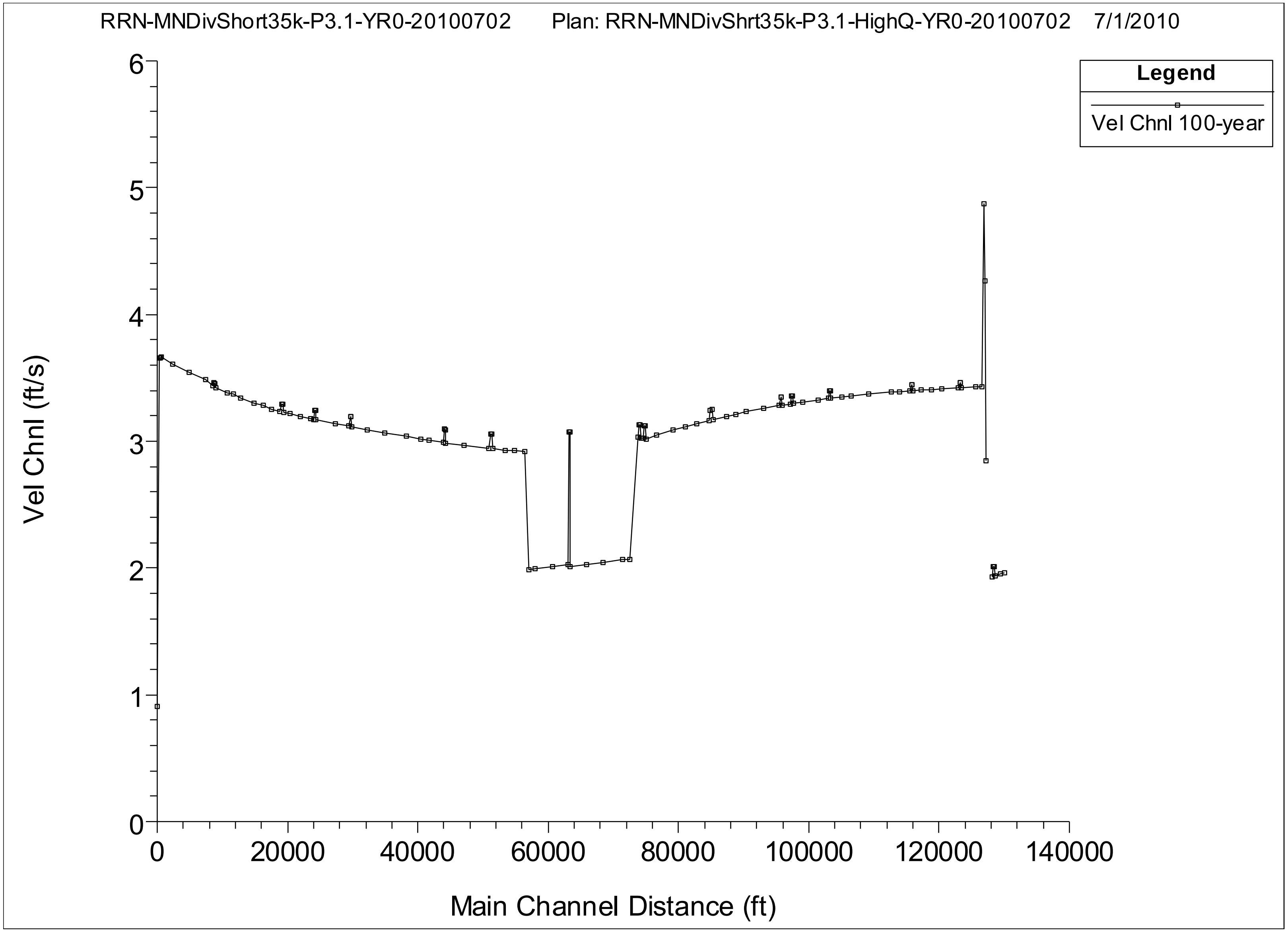

Figure F-G44 **FCP Diversion Channel Velocities for the 100-Year Event for Year 0 Hydrology**

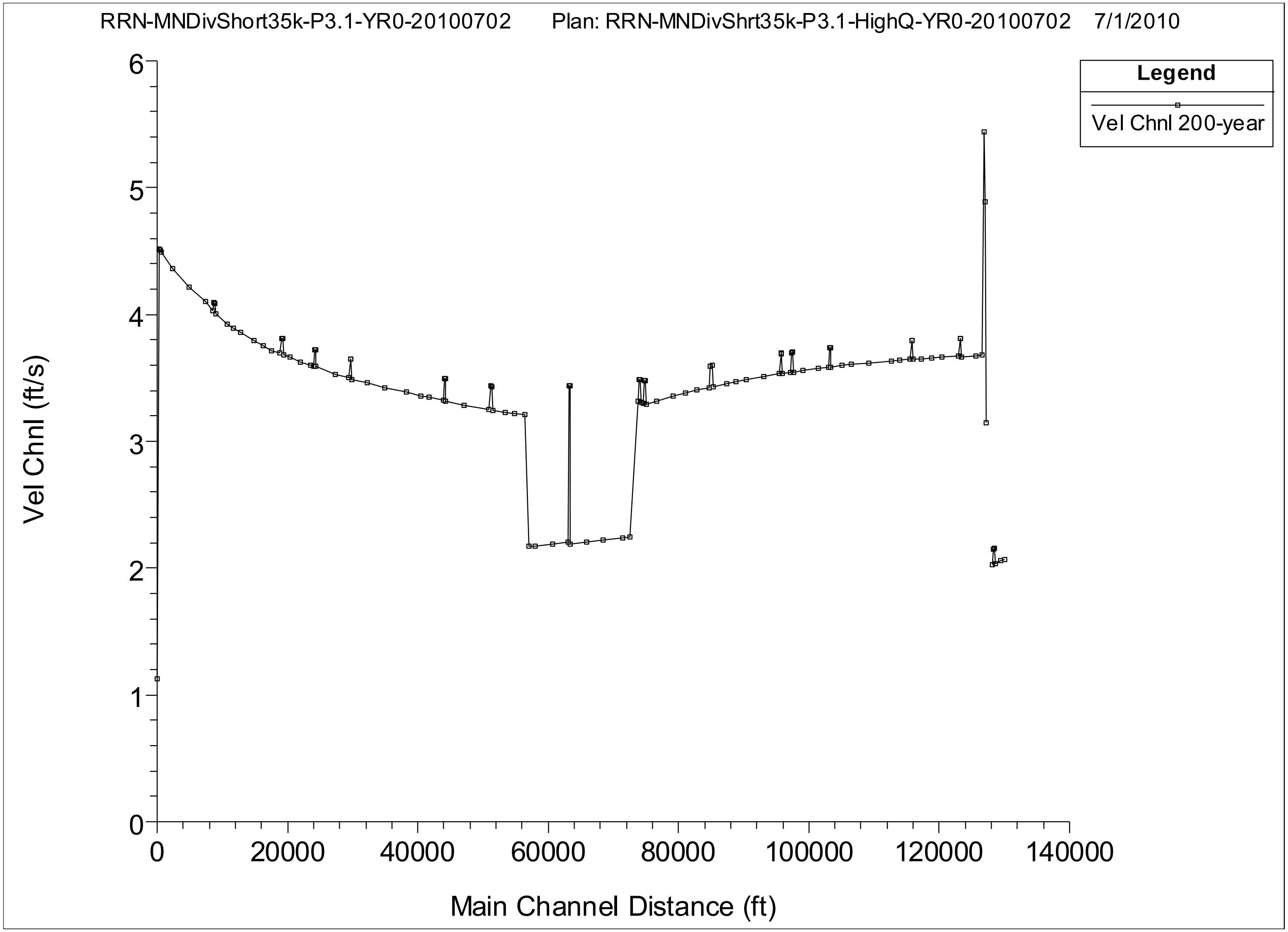

Figure F-G45 **FCP Diversion Channel Velocities for the 200-Year Event for Year 0 Hydrology**

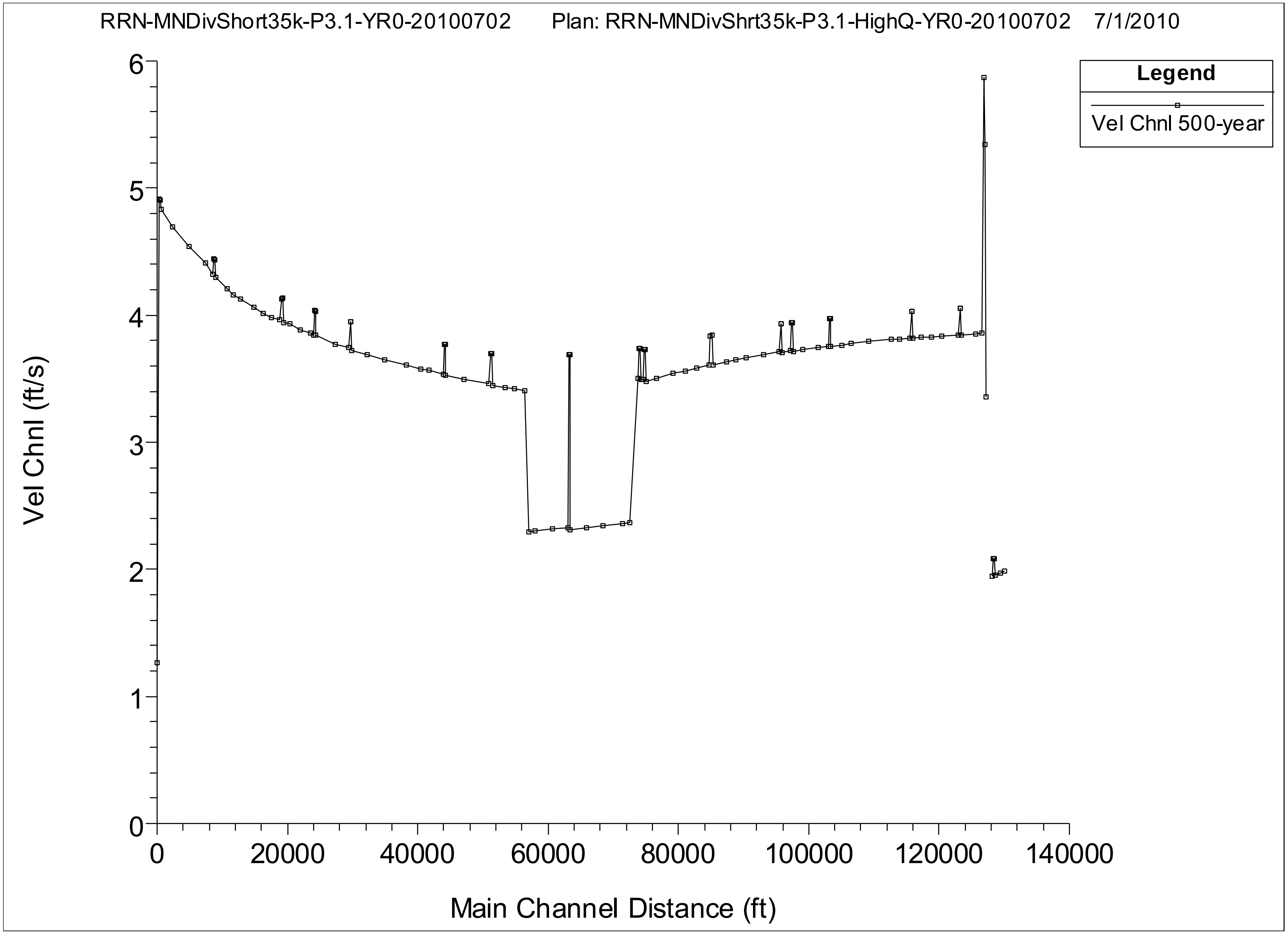

Figure F-G46 **FCP Diversion Channel Velocities for the 500-Year Event for Year 0 Hydrology**

F-G2.3VELOCITIES IN THE FCP DIVERSION CHANNEL – YEAR 25 HYDROLOGY

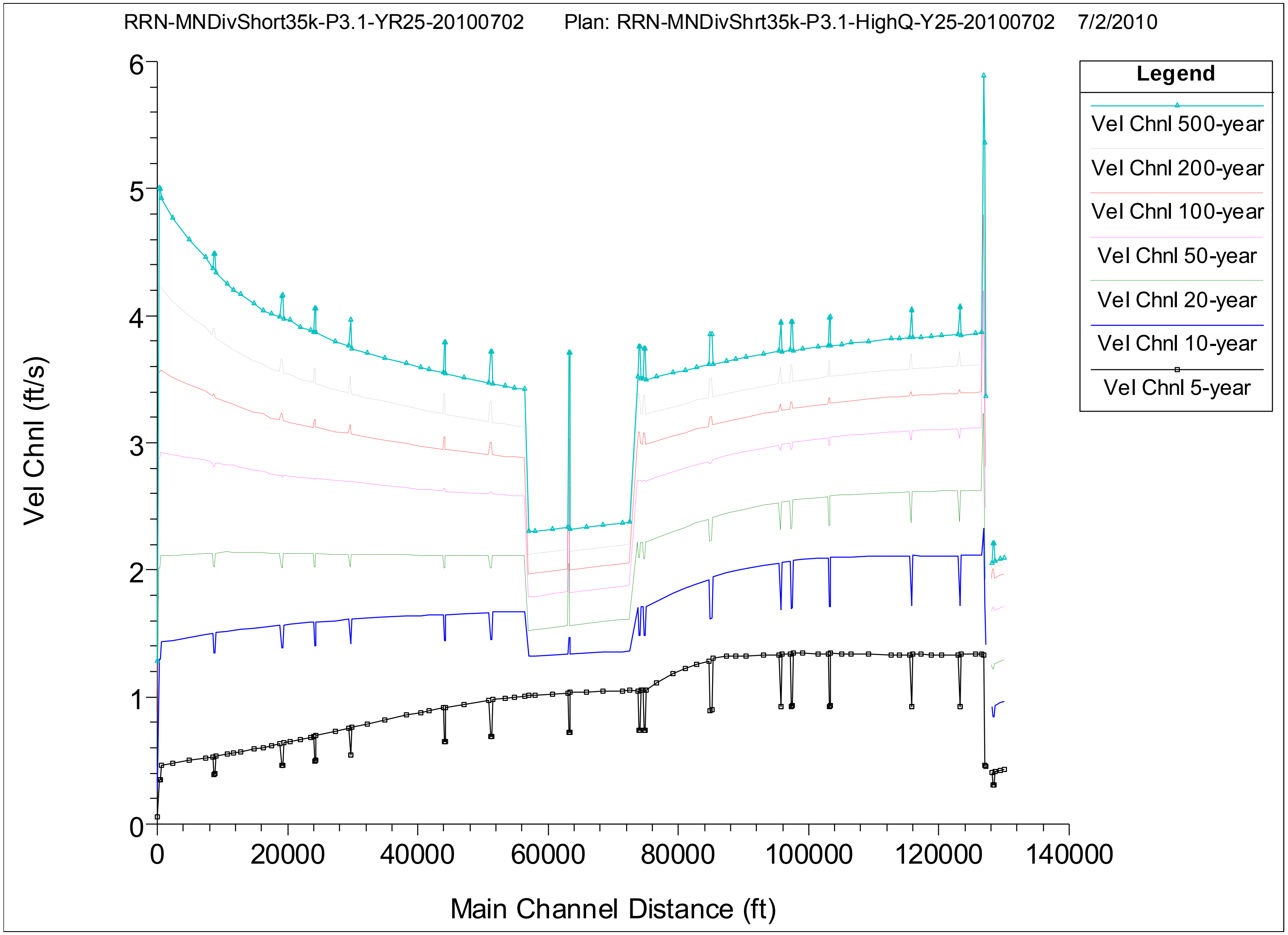

Figure F-G47 FCP Diversion Channel Velocities for All Modeled Events for Year 25 Hydrology

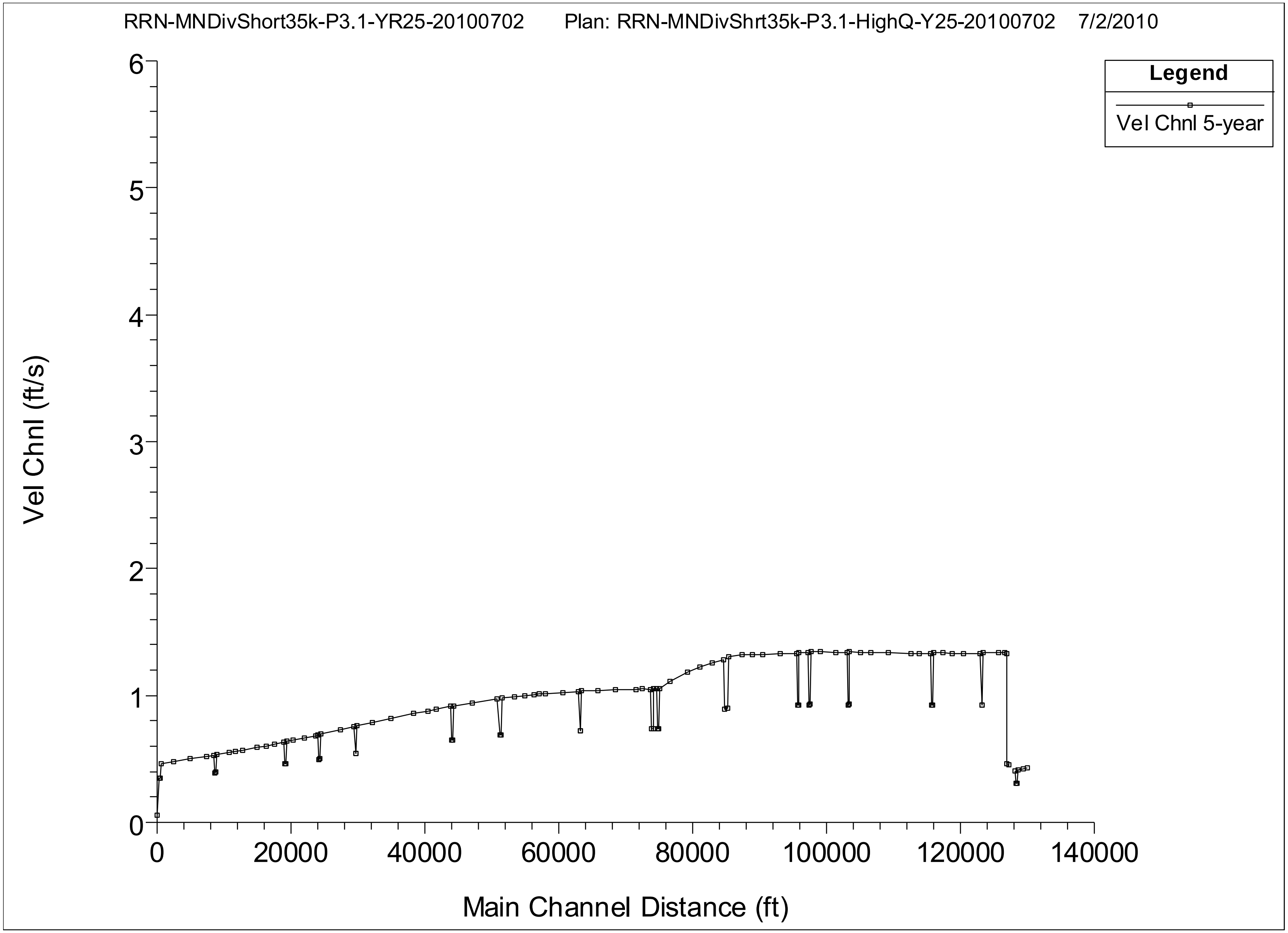

Figure F-G48 FCP Diversion Channel Velocities for the 5-Year Event for Year 25 Hydrology

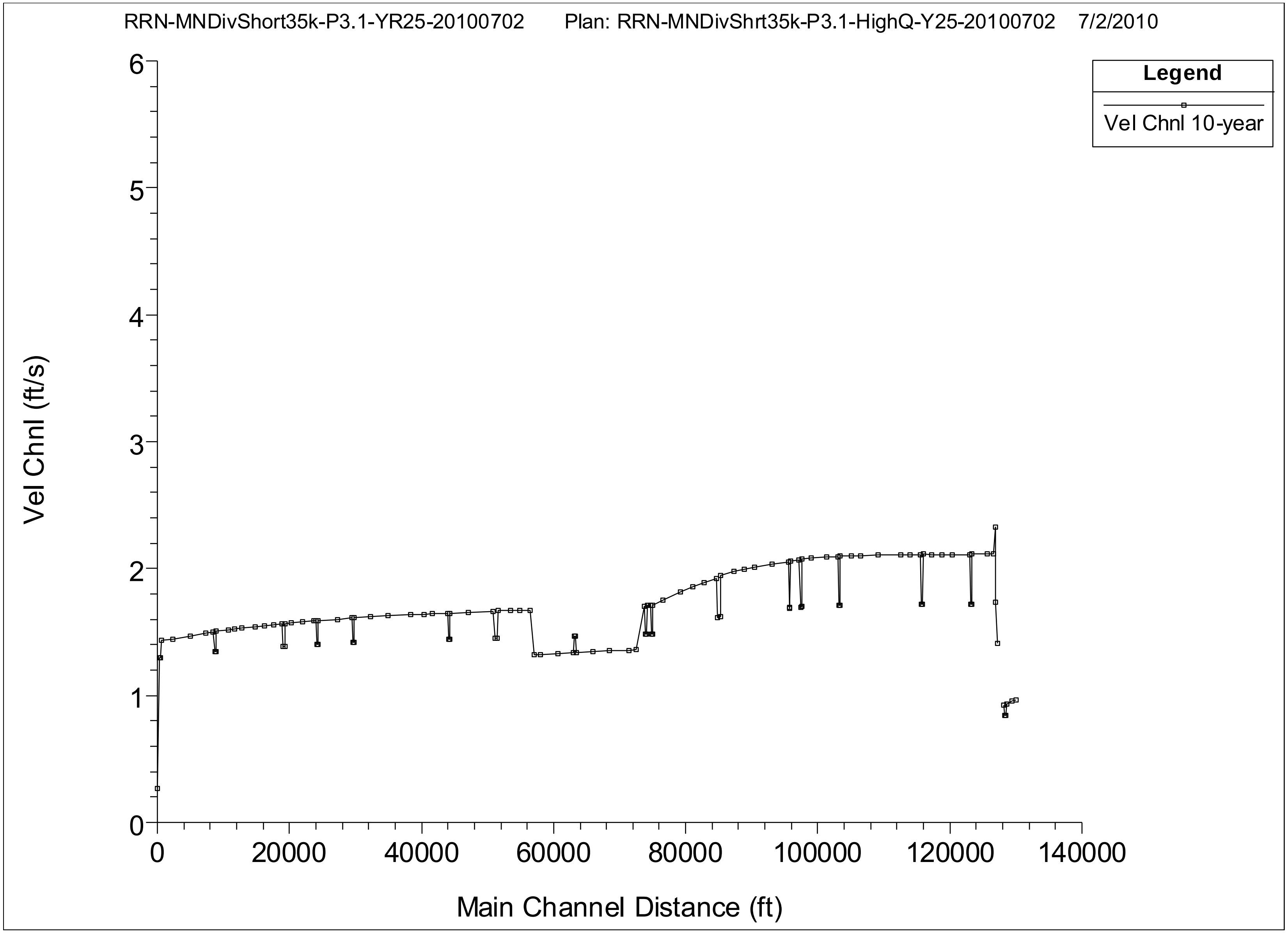

Figure F-G49 **FCP Diversion Channel Velocities for the 10-Year Event for Year 25 Hydrology**

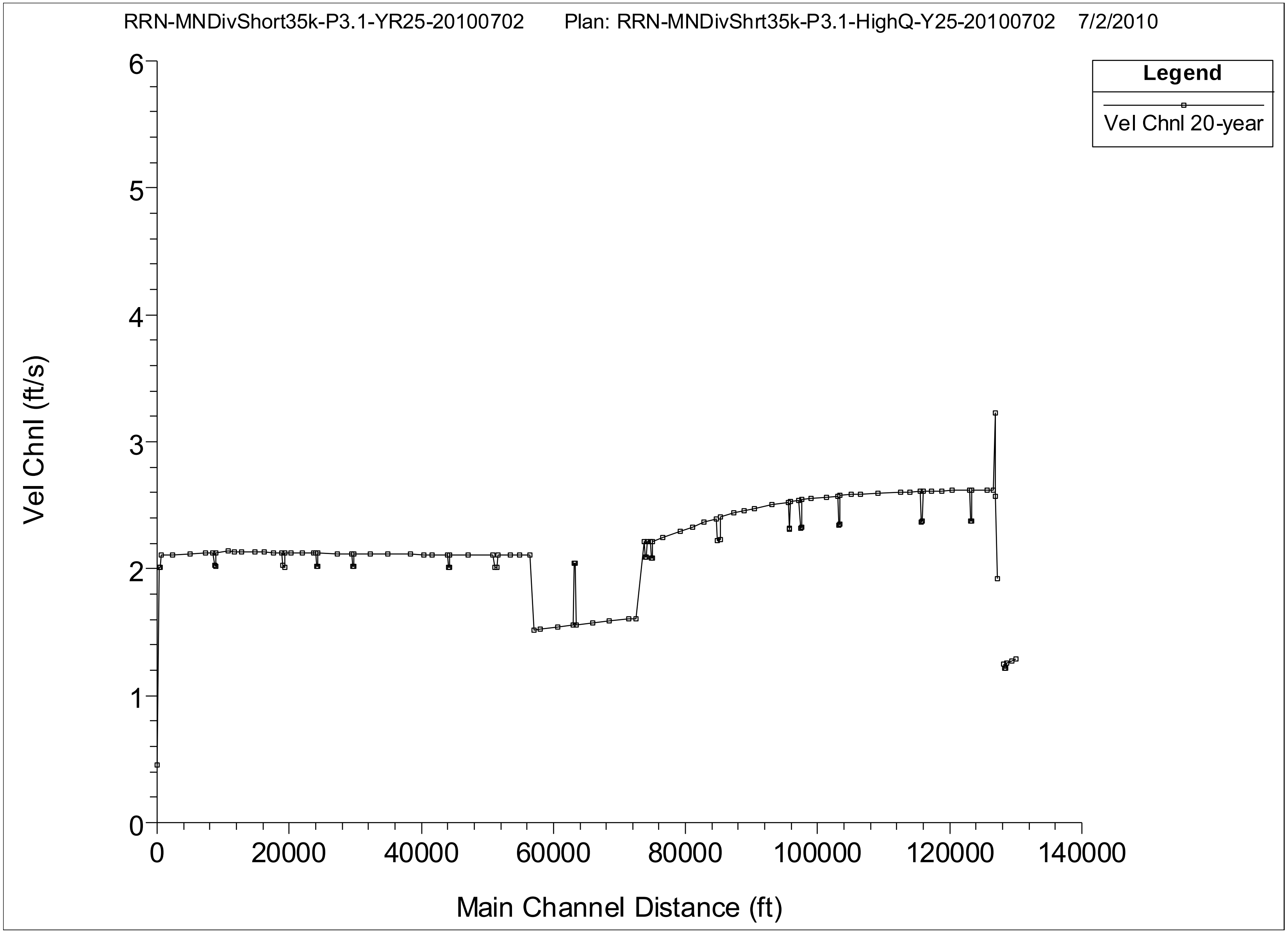

Figure F-G50 **FCP Diversion Channel Velocities for the 20-Year Event for Year 25 Hydrology**

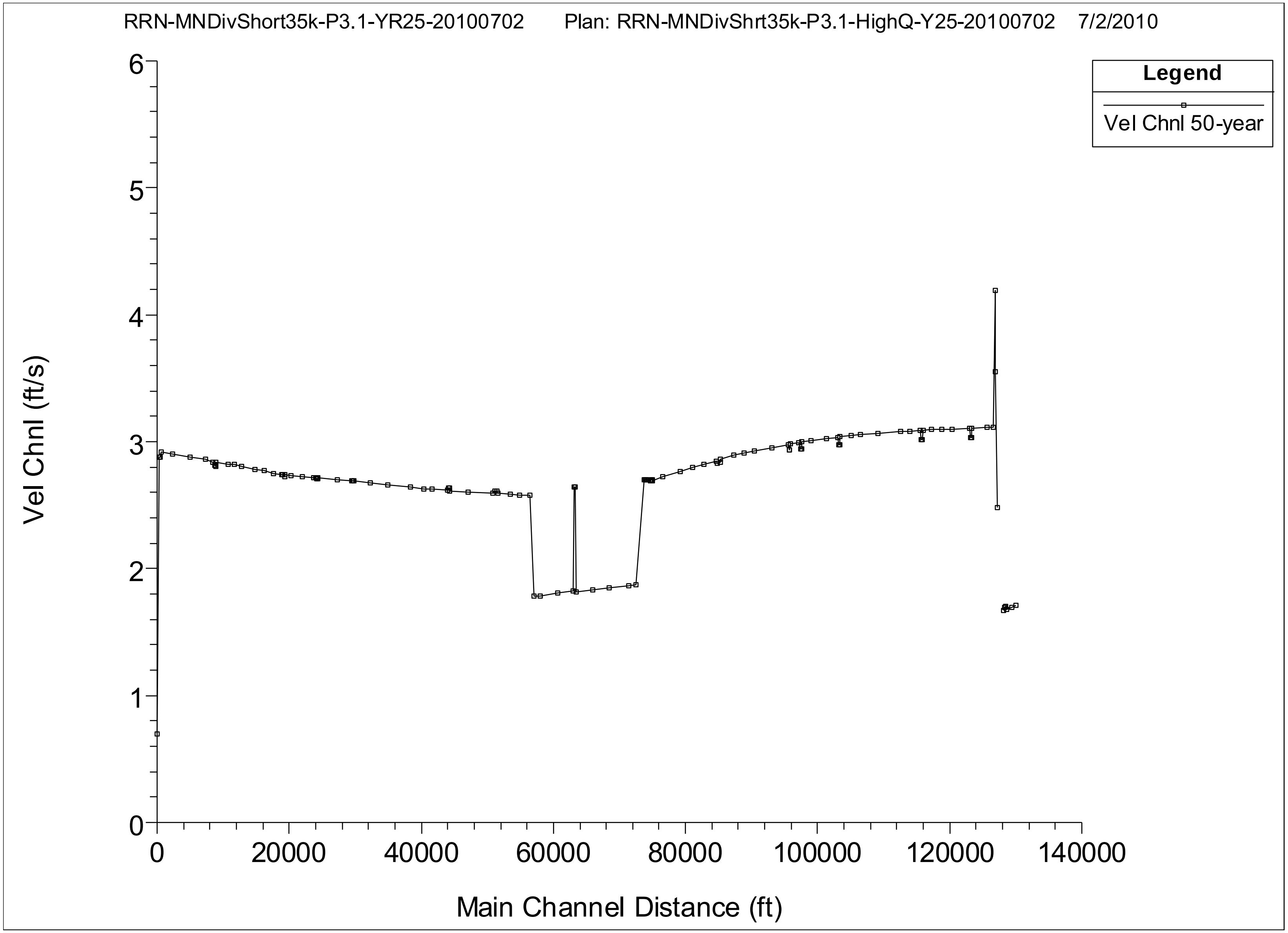

Figure F-G51 **FCP Diversion Channel Velocities for the 50-Year Event for Year 25 Hydrology**

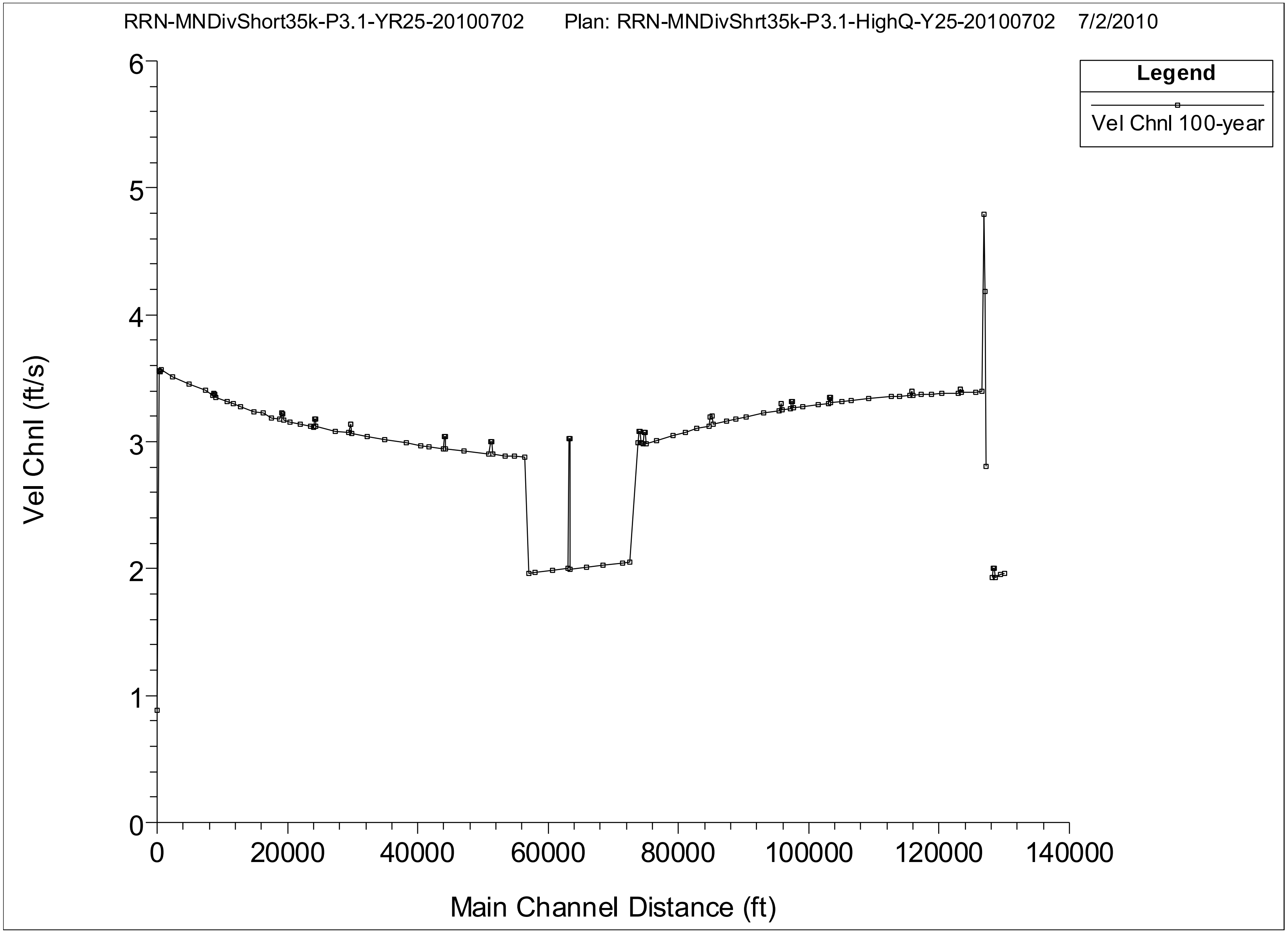

Figure F-G52 FCP Diversion Channel Velocities for the 100-Year Event for Year 25 Hydrology

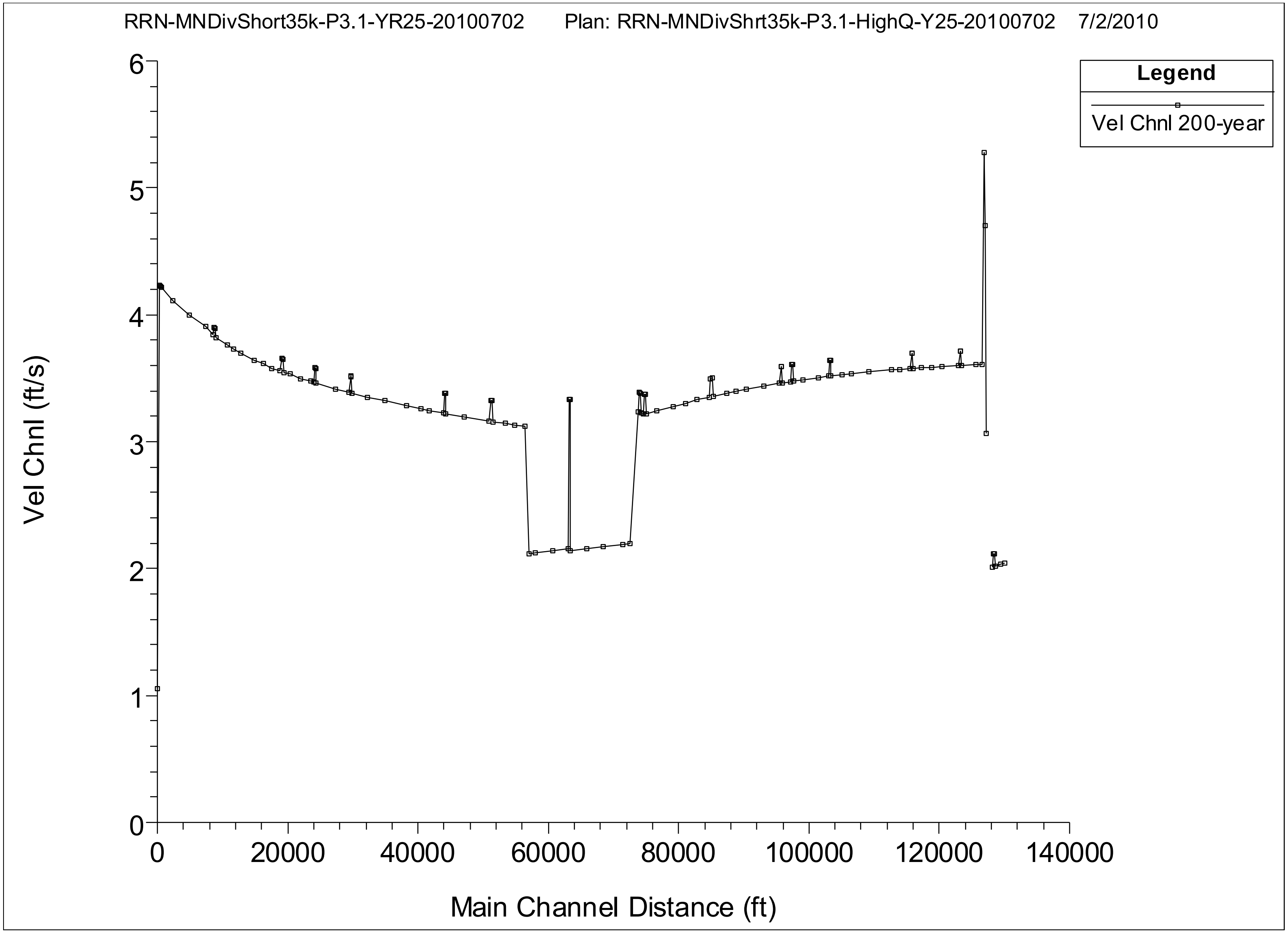

Figure F-G53 FCP Diversion Channel Velocities for the 200-Year Event for Year 25 Hydrology

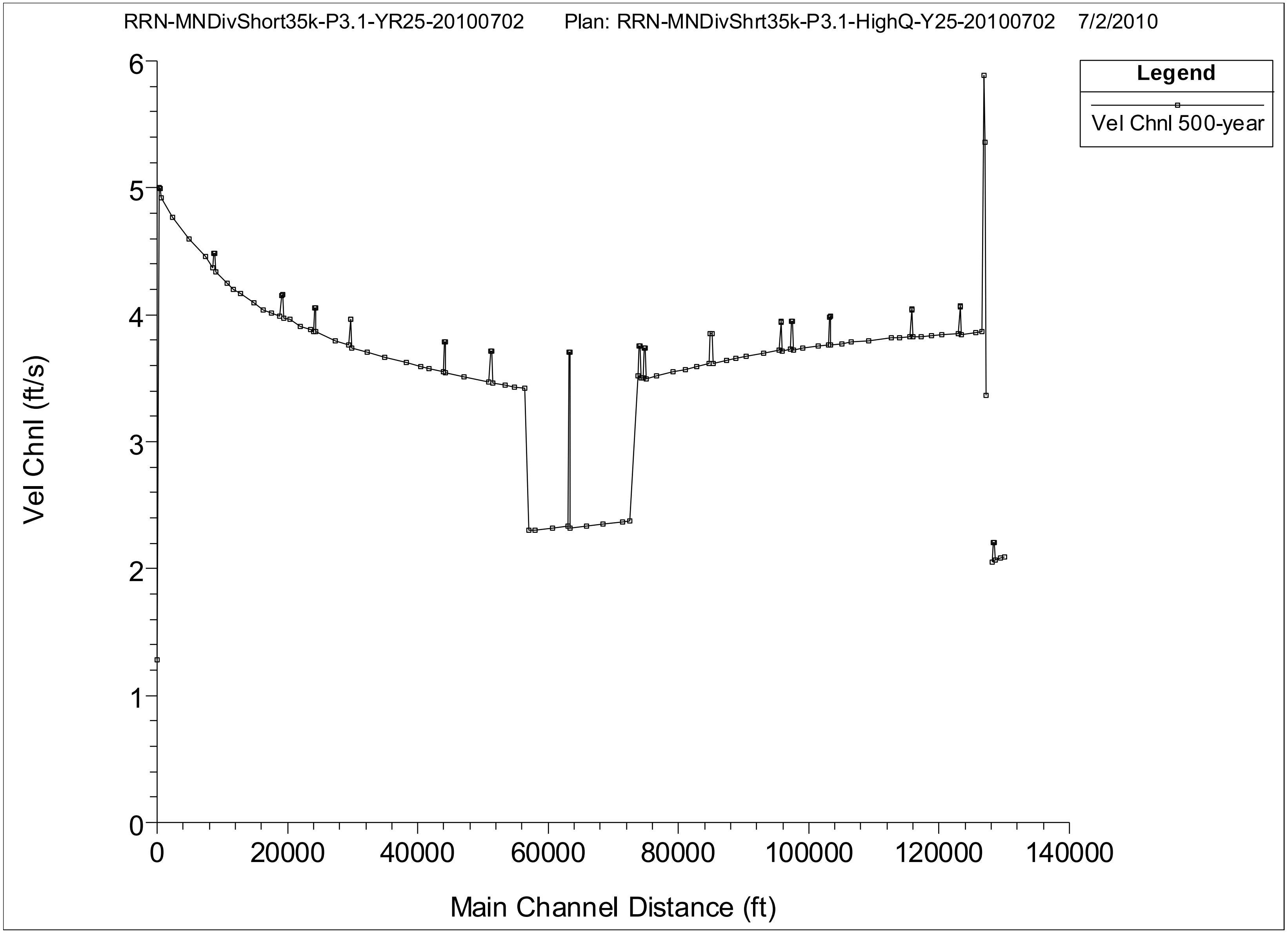

Figure F-G54 **FCP Diversion Channel Velocities for the 500-Year Event for Year 25 Hydrology**

F-G2.4VELOCITIES IN THE FCP DIVERSION CHANNEL – YEAR 50 HYDROLOGY

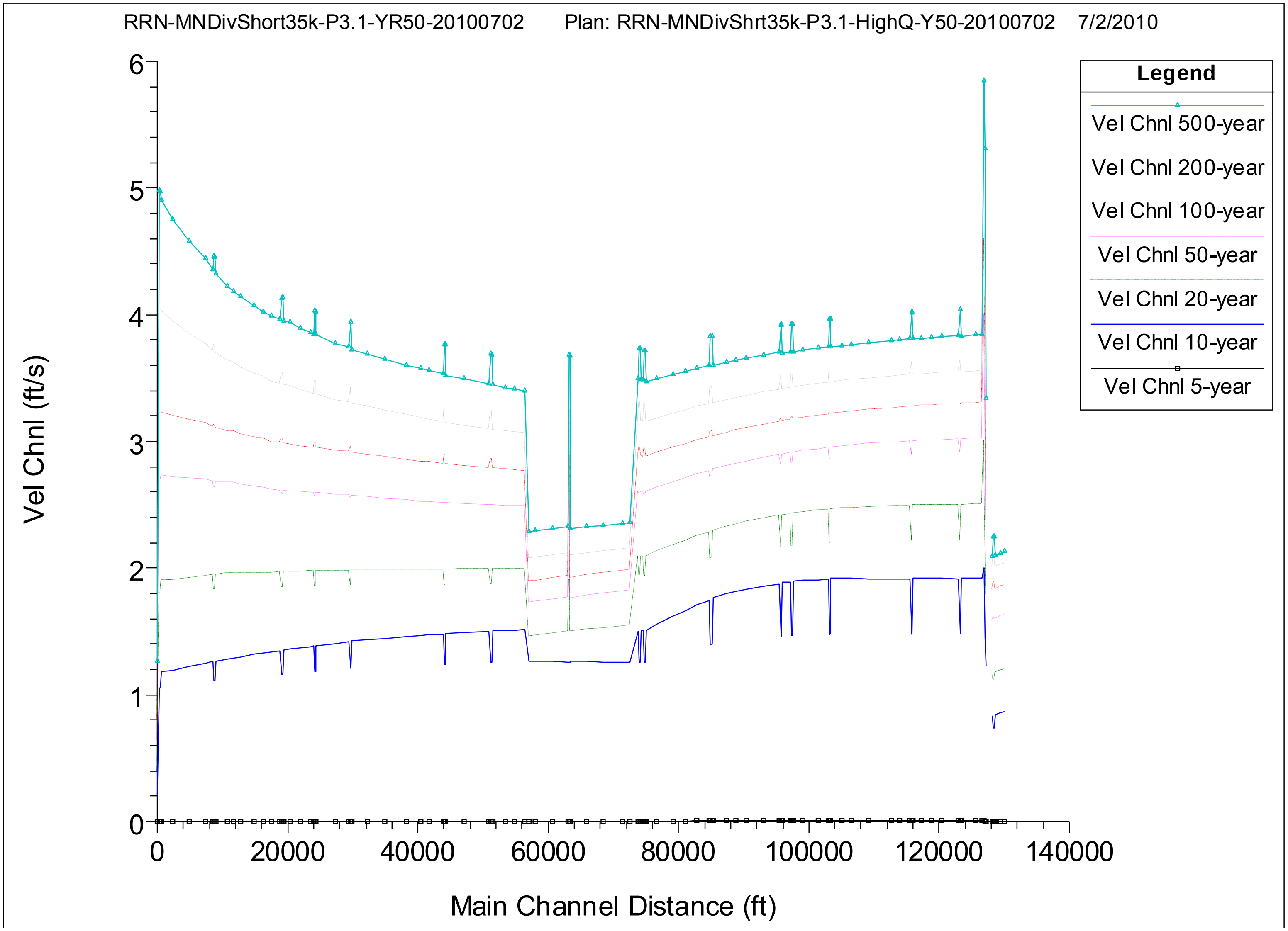

Figure F-G55 FCP Diversion Channel Velocities for All Modeled Events for Year 50 Hydrology

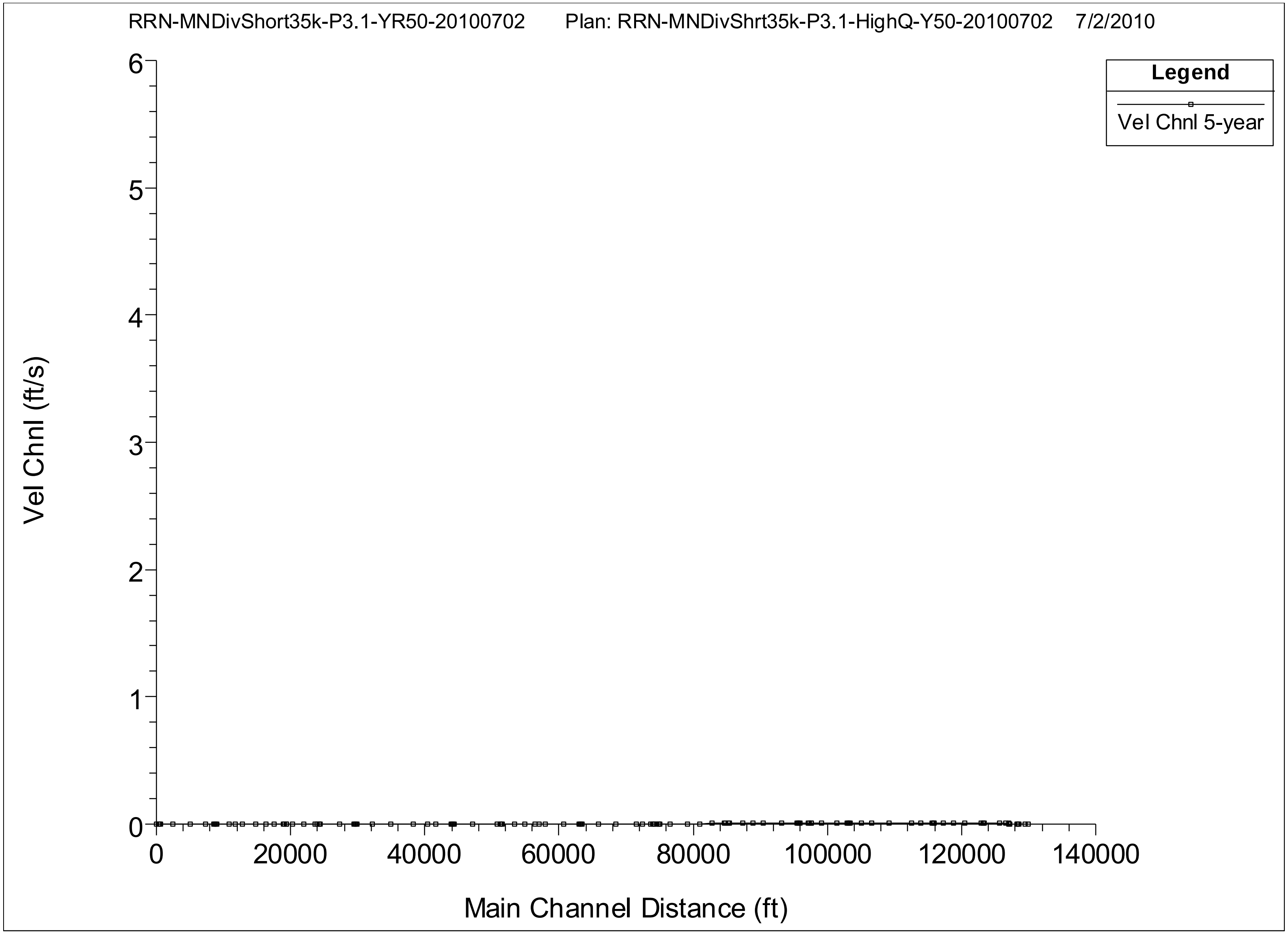

Figure F-G56 **FCP Diversion Channel Velocities for the 5-Year Event for Year 50 Hydrology**

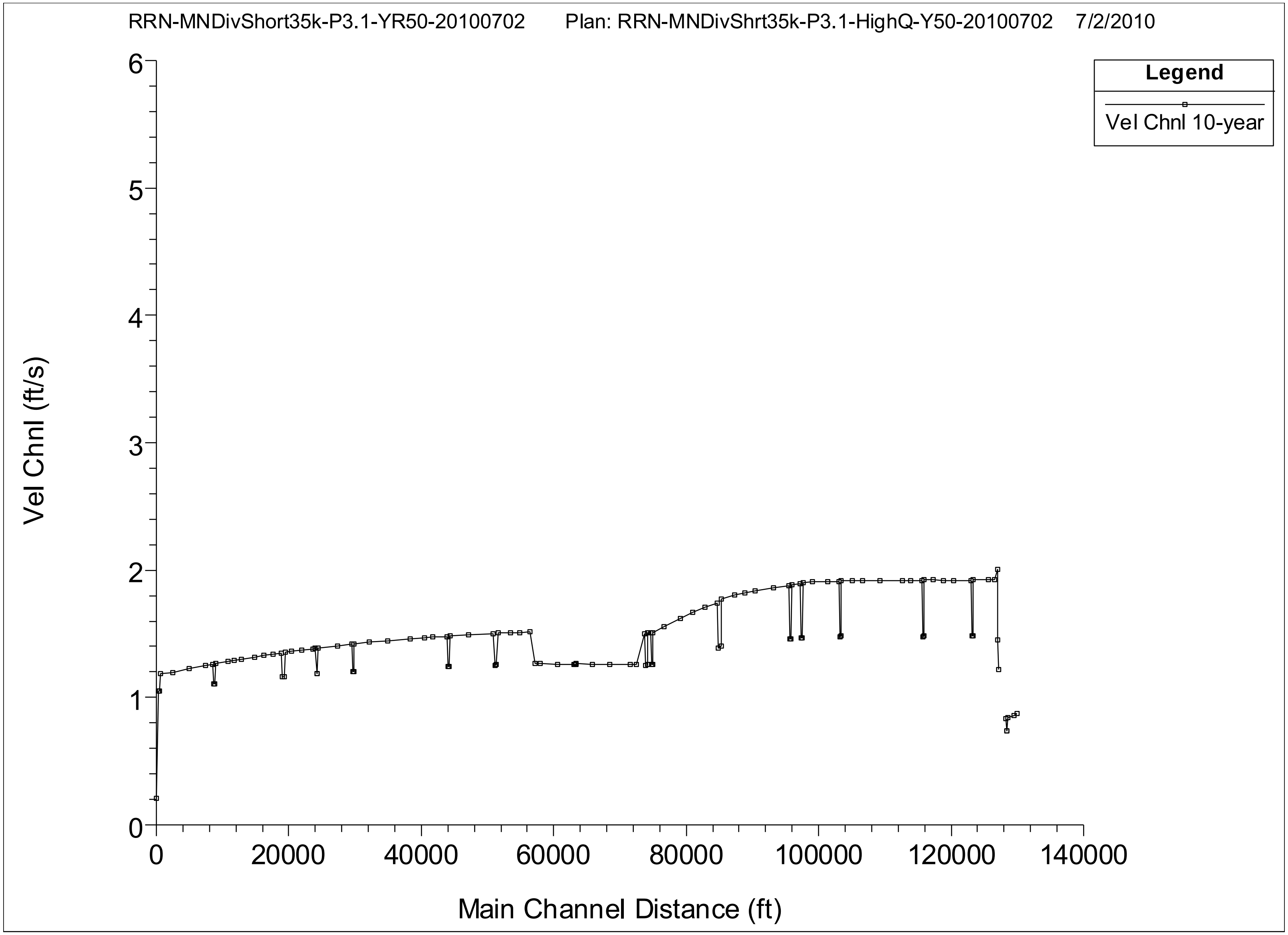

Figure F-G57 **FCP Diversion Channel Velocities for the 10-Year Event for Year 50 Hydrology**

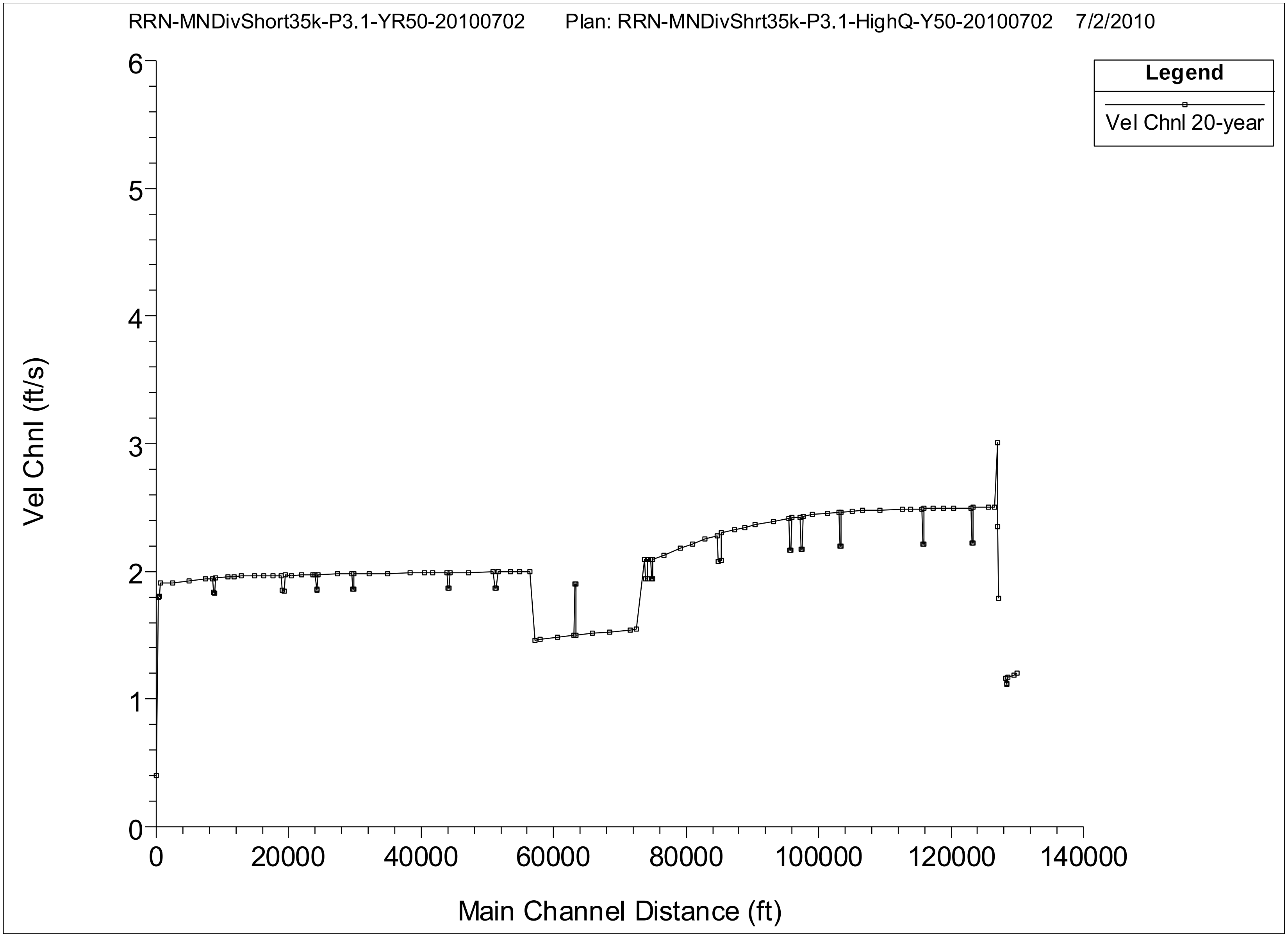

Figure F-G58 **FCP Diversion Channel Velocities for the 20-Year Event for Year 50 Hydrology**

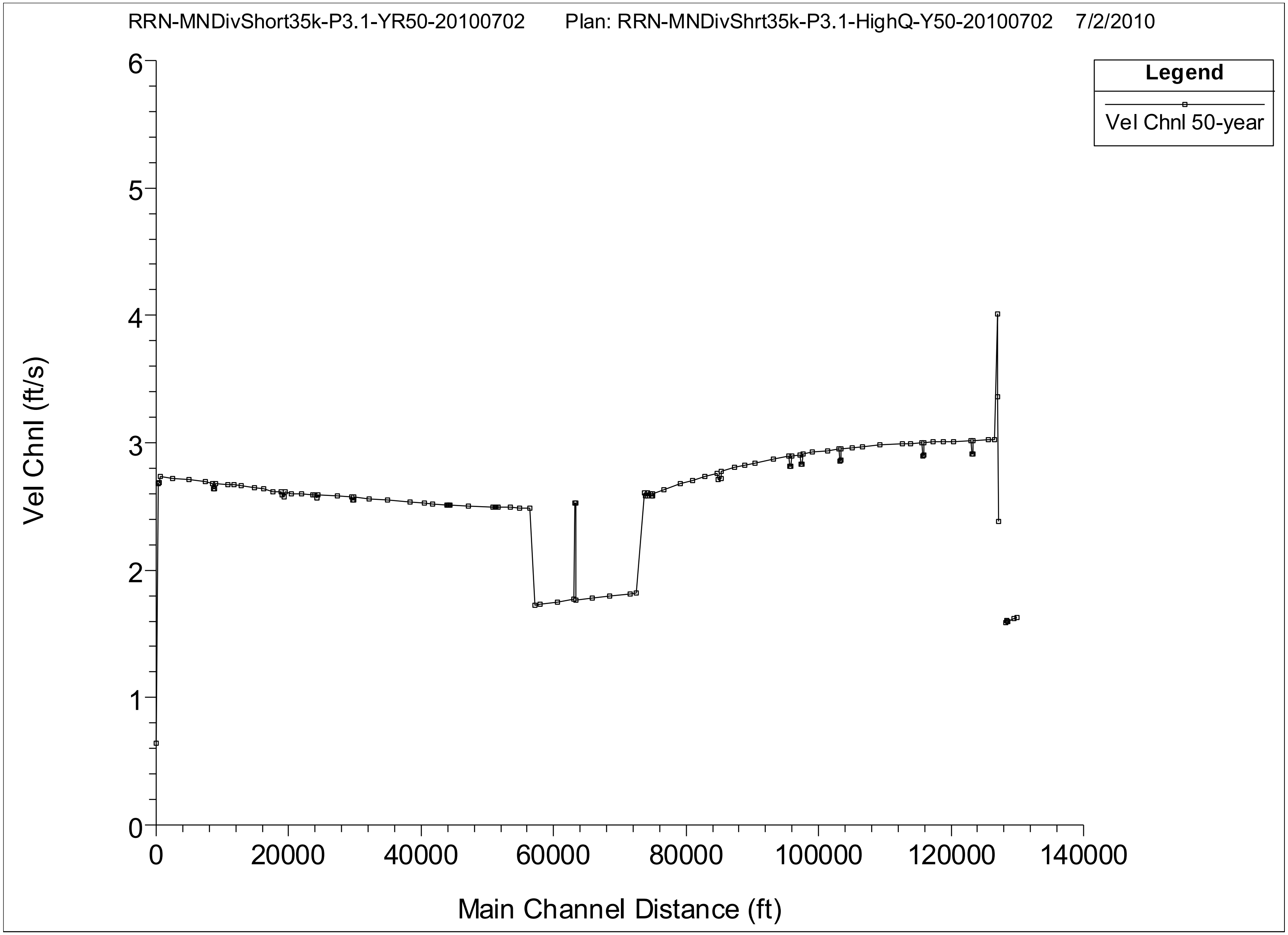

Figure F-G59 **FCP Diversion Channel Velocities for the 50-Year Event for Year 50 Hydrology**

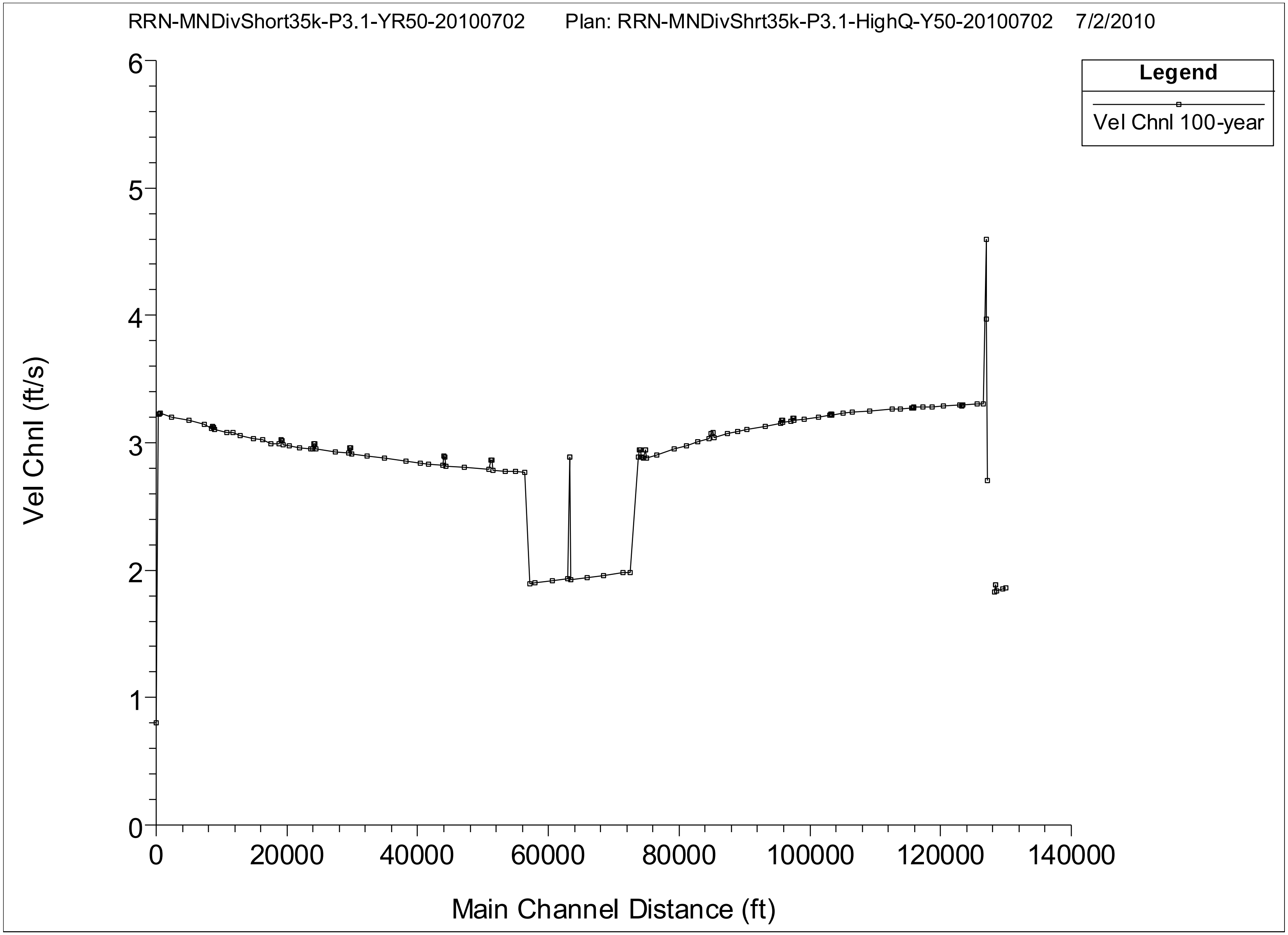

Figure F-G60 **FCP Diversion Channel Velocities for the 100-Year Event for Year 50 Hydrology**

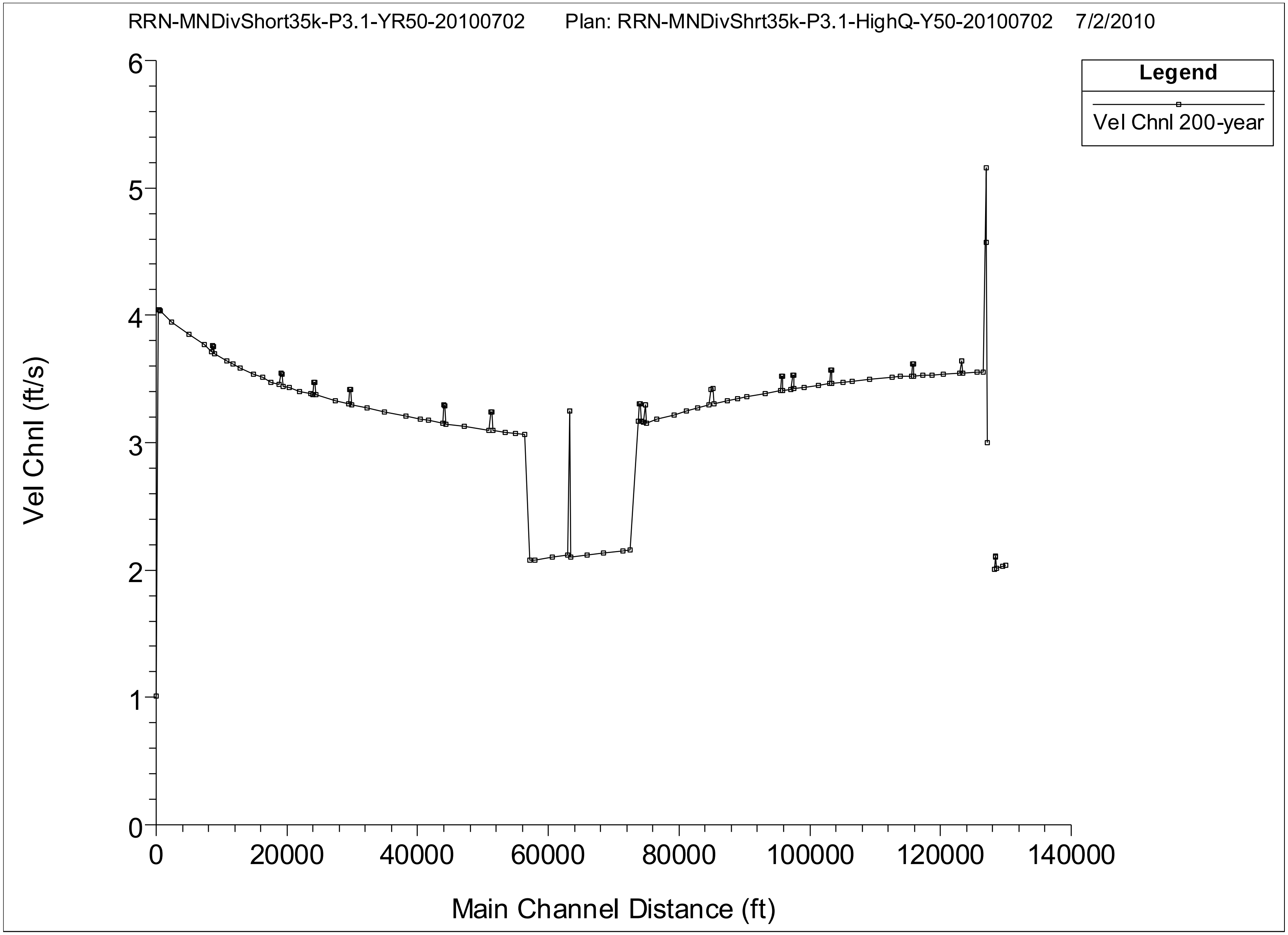

Figure F-G61 **FCP Diversion Channel Velocities for the 200-Year Event for Year 50 Hydrology**

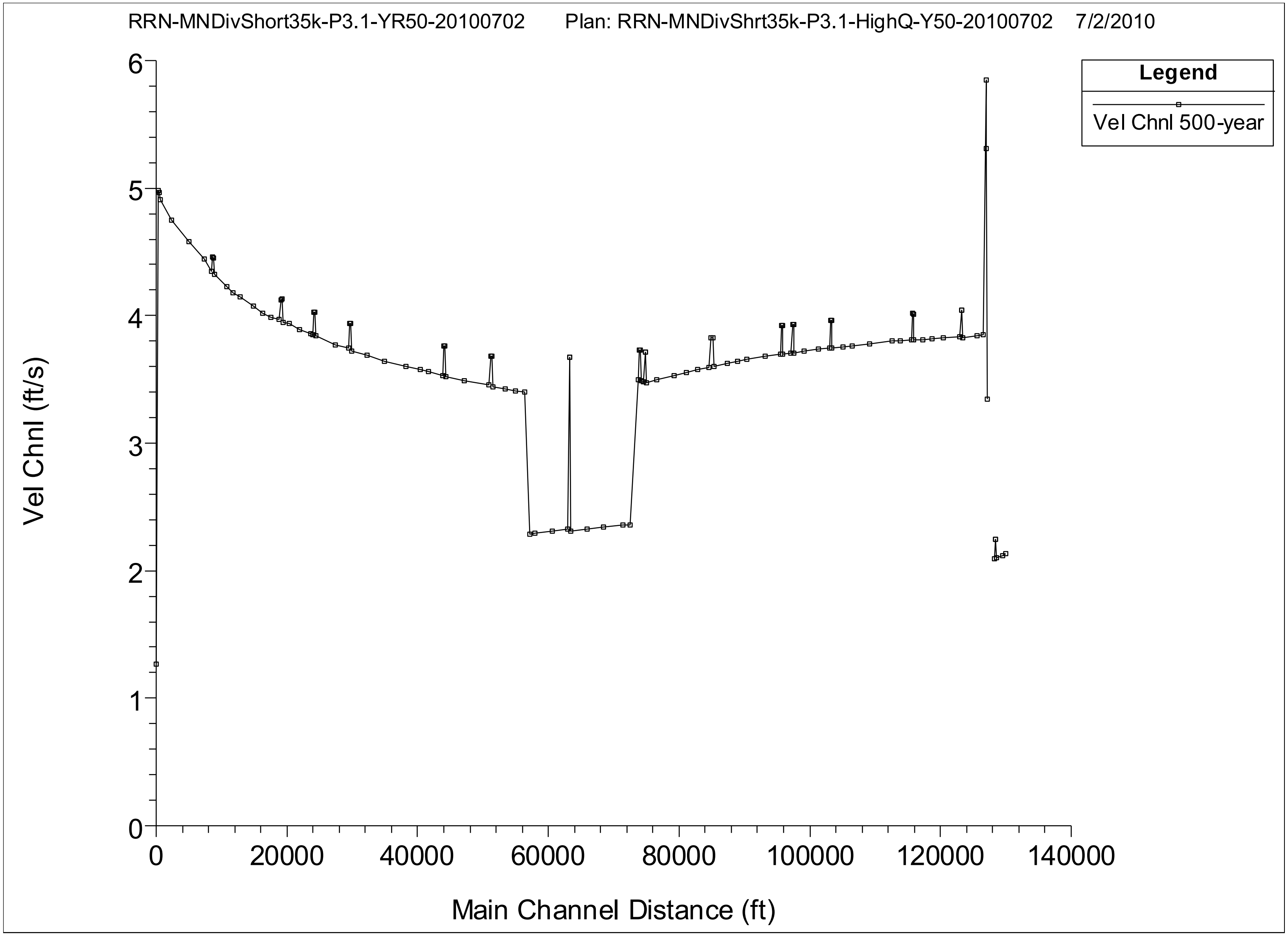

Figure F-G62 **FCP Diversion Channel Velocities for the 500-Year Event for Year 50 Hydrology**

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT H – 2D HYDRAULIC MODELING OF DIVERSION STRUCTURES AND FISHWAYS

Report for the US Army Corps of Engineers, and the cities of Fargo, ND and Moorhead, MN

By: Barr Engineering Co.

FINAL – February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT H – 2D HYDRAULIC MODELING OF DIVERSION STRUCTURES AND FISHWAYS

Notwithstanding the changes in water surface elevations and flows at, or dimensions of, some of the primary project features, the general configuration and concept functioning of these features is maintained in Phase 4. Therefore, the results presented in Exhibit G of the Phase 3 report submitted on August 6, 2010, are repeated in the Phase 4 report because they provide an approximate representation of flow velocity patterns, in particular during the most frequent events.

F-H1.0 INTRODUCTION

Two-dimensional (2D) hydraulic models of the diversion structures and fish passage ways were created to capture the (vertically-averaged) velocity distribution across channels or structures as it relates to fish passage. For each structure, pre-project (existing) conditions were compared to post-construction (with structures in place) conditions. Models of post-construction conditions were based on Phase 3 structure and channel modification plans. This analysis included modeling seven diversion/crossing structures in addition to the two outlets from the diversion channel for a total of nine structures:

- RRN control structure for Minnesota plan
- RRN control structure for North Dakota plan
- Wild Rice control structure
- Sheyenne River crossing of diversion channel
- Maple River crossing of diversion channel
- Lower Rush River connection to diversion channel
- Rush River connection to diversion channel
- Outlet to RRN, Minnesota Plan
- Outlet to RRN, North Dakota Plan

Multiple flow events were analyzed for each structure. For most structures, a low flow event and a high flow event were selected. For some structures, multiple flow events representing various flow regimes were selected. Only Year 0 of the Phase 3 hydrology (e.g., Year 0 scenario, but not Year 25 or Year 50 scenarios) was used in modeling, and only the Locally Preferred Plan (LPP) and Federally Comparable Plan (FCP) plans (Phase 3) were analyzed. Unless otherwise noted, the flow events (e.g., 100-year event) in all the tributaries and the Diversion Channel refer to coincidental events in the RRN.

F-H2.0 METHODOLOGY

F-H2.1 2D FLOW MODEL

The 2D flow model used for this analysis was Adaptive Hydraulics (ADH) revision # 5939. ADH-5939 is a two-dimensional model developed by the coastal and hydraulic laboratory (CHL), engineer research and development center (ERDC) and the USACE. ADH-5939 was selected by the USACE as the numerical model for this analysis because of its ability to adapt numerical meshes to efficiently compute a steady state solution. All models completed as part of this analysis were steady state models that used US customary units.

Preprocessing of model inputs including developing the mesh was completed using AquaVeo's Surface-Water Modeling System Version 10.1 (SMS). ADH-5939 was run independently of SMS from the Microsoft command line. ADH boundary conditions files were created in Microsoft Notepad and manually adjusted for each model. Finally, ADH model results were imported back into SMS for post-processing and viewing of the results.

F-H2.2 MODEL COMPLEXITY AND DOMAIN

The level of detail incorporated into the models was consistent with a feasibility-level analysis. The intent was to capture the effects of the location and the general type/shape of major features of structures. The feasibility-level analysis assumes more detailed 2D flow models (and possibly physical scale models, or 3D flow models) will be completed during future phases of the overall project (e.g., detailed design phase). The models are not intended to be used to analyze other aspects of the project (e.g., ice loading/passage, sediment transport, etc.). This analysis also assumes that the models developed will be used only for this current phase of analysis.

Numerous assumptions were necessary to model the complicated geometry of structures. For example, ADH does not have a routine to model flow under gates. The size and complexity of the models (i.e., mesh size and density, number of nodes, and extent of model domains) was established at a level of detail consummate with Phase 3 analysis. The models were developed and refined only to a level required to achieve convergence, and at which the resulting velocity distribution appeared to be sensible.

Modeling of structures (or parts of structures) was done only where necessary in order to achieve the objectives of this phase, namely at flow junctions and at flow transitions. Model domains were set to include approximately 1,000-2,000 ft of channel upstream/downstream of the structure location.

For low flow conditions, model domains were established to capture the extents of the inundated area. However, for larger flow conditions that result in larger inundated areas, this was not always possible due to the flat terrain. In these locations model domains were clipped along roads, topographic ridges (that may be inundated), or at distance sufficiently away from the area of interest in locations where it was assumed that flow outside the model domain could largely be considered ineffective. Ineffective area was defined by reviewing existing RRN existing FIS HEC-RAS models.

F-H2.3 MESH/ELEMENT SIZE

ADH required the use of triangular elements. Element size was varied at different areas of mesh in order to capture detail of the topography and structures. At gate openings or through structures, elements were set as small as 10 ft; out in middle of flood plain, where bathymetry is very flat and flow is slow, elements were set at roughly 50.

F-H2.4 AERIAL IMAGES AND LAND USE

Aerial images were used to assign material types in the 2D models. Areas of similar land use were identified based on the 2009 Minnesota aerial photography collected by the U.S. Department of Agriculture, Farm Service Agency, as part of the Farm Service Agency's National Agriculture Imagery Program (NAIP). Based on the aerial photographs, existing conditions land use was divided into three categories, riverbed, grass, and trees, which were then assigned material properties in the 2D models. Land use for post-construction conditions was assigned as either constructed channel or concrete, based the structure's grading plan.

F-H2.5 MANNING'S VALUES

Manning's values were assigned based on material type. Three material types were used for modeling pre-construction conditions (riverbed, trees, and grass). Additional materials (concrete, constructed channel, and water surface) were added for post-constriction conditions. Initial runs used Manning's values that were consistent with available HEC-RAS models. Manning's values used in the HEC-RAS models ranged between 0.04-0.055 for the riverbed, 0.06-0.16 for grass and tree land use types in the overbanks, and 0.030 for constructed channels. Manning's values greater than 0.080 caused ADH to have a difficult time converging on a solution. Additionally, the higher Manning's values resulted in upstream water surface elevations that were higher than the HEC-RAS models. Consequently, Manning's values used in the 2D models were significantly lower than those used in the HEC-RAS models.

Final values of Manning's n were determined after calibrating the models both to upstream water surface from available HEC-RAS models and to available measured velocity distributions. See Section F-H3.3 below for a discussion of model calibration. Final values of Manning's n used in the models are shown in Table F-H1.

Table F-H1 Final Manning's Values

Material	Manning's Value
Riverbed	0.030
Grass	0.040
Trees	0.060
Concrete	0.014
Constructed Channel	0.030
Water Surface*	0.010

*Water surface material was only used for the Maple River Case 4 model where only the upper portion of the diversion channel was modeled.

F-H2.6 EDDY VISCOSITY

The Estimated Eddy Viscosity (EEV) card was used to calculate eddy viscosity in the models. ADH uses the estimated eddy viscosity coefficient to weight each eddy viscosity term calculated by the model. The EEV may range between 0.1-1.0. The EEV card used for this modeling analysis was set to 0.5 for all models.

F-H2.7 TOPOGRAPHY/BATHYMETRY

Meshes for the 2D models were created by combining available topographic and bathymetric data. Models of existing conditions were based on available LiDAR data for areas above the normal waterline and on interpolated HEC-RAS sections for areas below the normal water line. The LiDAR data gathered in 2008 was used to create the ground surface profile in the overbank portions of the models. The LiDAR data was gathered as part of the Red River Basin Mapping Initiative. Bathymetric data from HEC-RAS cross sections were merged with the LiDAR data for areas of the channel where LiDAR data did not survey the channel below the water level. All elevation data used for this analysis is presented in North American Vertical Datum 1988 (NAVD88).

For post-construction conditions sections of river that were impacted by construction were replaced by the appropriate grading plan. Grading plans at the structures dated July 10, 2010 were used to develop topography for all post-construction conditions.

F-H2.8 MESH REFINEMENT

In areas where initial element sizes large, or areas with complex flow patterns ADH has the ability to automatically refine the mesh during the model simulation. The number of times ADH can refine a mesh element, or the number of times an original mesh element may be subdivided by ADH, is specified in the boundary conditions file with the MP ML card. The maximum number of times ADH may subdivide a mesh element is determined by $2^{MP\ ML.}$ For example, for a MP ML card value of 3 one element may be split into 8 elements during the model run.

For this analysis the MP ML card ranged between 1 and 3. Increasing the MP ML card past 3 significantly increased model run time. Therefore in areas with complex flow patterns, or in areas of interest, smaller elements were used. This allowed the MP ML card to stay below 3 and allowed the model to converge on a solution.

F-H2.9 ERROR SETTINGS

During model runs ADH determines if refinement of the mesh (i.e. subdividing elements) is required by calculating the error for each element and comparing the calculated error to the MP SRT card in the boundary conditions file.

For this analysis the MP SRT card was set between 10 and 100. Decreasing the MP SRT card below 10 allowed ADH to subdivide more elements which significantly increased model run time. Therefore, in areas with complex flow patterns or areas of interest, smaller elements were used rather than further reducing the MP SRT card. This allowed the MP SRT card to stay within the specified range for this analysis and allowed the model to converge on a solution.

F-H2.10 WETTING/DRYING

During model runs some elements become wet or dry as the water surface rises and falls as the model searches for a solution. ADH accounts for this wetting and drying of elements by gradually turning elements on and off as the model works to converge on solution. The MP DTL card controls when elements are turned on and off within ADH. For this analysis the MP DTL card drying depth was set at 0.3, and the wetting depth was set at 0.5-0.6.

F-H2.11 CONVERGENCE AND RUN TIME

ADH will continue to run until the simulation either reaches the final time step or the spatial residual is less than the tolerance specified in the boundary conditions file. For this analysis the tolerance was specified by the IP NTL card as 0.001, and the model run time was set at 100,000 seconds. If the models reached the final run time of 100,000 seconds before converging, the error, velocity, and depth output files were compared. If the maximum error was less than 100 and the velocity and depth results remained constant between 90,000 -100,000 seconds, then the models were considered to have reached a solution. If these criteria were not achieved, then adjustments to model parameters were completed and the model was rerun.

The hot start files for all the models assumed that the starting water-surface elevation was level. This resulted in ADH having a difficult time starting the model (at time 0 seconds) with the desired steady flow rate. This was because introducing a high flow rate to a level pool resulted in a quick change to water-surface elevations (and elements quickly wetting or drying) which in some cases resulted in the model crashing. Therefore, to ramp the model up one of two methodologies were used:

1. The downstream water-surface elevation boundary condition was set and remained constant throughout the model run. The upstream flow rate boundary

condition was initially set low, and was gradually increased until it reached its final value.

2. The upstream flow boundary condition was set and remained constant throughout the model run. The downstream water-surface elevation boundary condition was initially set high, and was gradually decreased until it reached its final value.

F-H2.12 SMOOTH INFLOW AREAS

ADH had trouble running if there were dry elements along the inflow boundary. ADH also had trouble appropriately distributing flow across the inflow boundary over varying topography. Therefore, for many of the models an inflow ramp was added at the upstream end of the model mesh such that the upstream boundary would remain wet throughout the duration of the model run. This also allowed the inflow to spread out across the mesh prior to reaching the study area, rather than being artificially distributed by the boundary condition. The Manning's values at the ramps were set between 0.030-0.035

F-H2.13 LID ELEVATIONS

The post-construction conditions at the Maple River crossing and Sheyenne River crossing were each simulated with pressure flow below the aqueduct crossing. Pressure flow below the aqueduct was modeled using the Lid function in ADH. The Lid elevation was set at an elevation that resulted in the steady state water depth below the aqueduct crossing that corresponds to the available area below the crossing. In both cases it was necessary to set the Lid elevation lower than the low chord of the aqueduct crossing in order to achieve the proper area of flow under the aqueduct since the current algorithms in ADH solve for the pressure head and do not directly solve for the depth of water.

F-H3.0 MODEL DEVELOPMENT

F-H3.1 STRUCTURES MODELED

This analysis included modeling 7 diversion/crossing structures in addition to the two outlets from the diversion channel for a total of 9 structures:

- RRN control structure for Minnesota plan
- RRN control structure for North Dakota plan
- Wild Rice control structure
- Sheyenne River crossing of diversion channel
- Maple River crossing of diversion channel
- Lower Rush River connection to diversion channel
- Rush River connection to diversion channel
- Outlet to RRN, Minnesota Plan
- Outlet to RRN, North Dakota Plan

Areas of each structure modeled and flows conditions used in this analysis are described in the following sections.

F-H3.1.1 RRN Control Structure for Minnesota Plan

Approximately 17,000 linear feet of RRN channel and floodplain were modeled for existing conditions at the location of the RRN control structure for the MN plan. The model domain for this location includes four major bends in the RRN. A low and a high flow condition were modeled for existing conditions. The low flow event for this location was 9,600 cfs in the RRN. This flow event corresponds to the flow rate in the RRN just prior to flow from the RRN being diverted into the Diversion Channel for post-construction conditions. The high flow event selected for this location was the 20-year event.

Approximately the same section of RRN was modeled for post-construction conditions, and the same flow conditions were modeled. For the 9,600 cfs flow event, the control structure gates will be completely open, there will be open channel flow through control structure, and there will be no water flowing over the diversion channel inlet weir or into the fish pass entrance on the RRN. For the 20-year flow, the control structure gates will be partially closed, restricting flow through the structure and creating a submerged hydraulic jump downstream of the control structure. Flow will also enter the fish pass, and there will be flow over the inlet weir into the diversion channel. Since ADH does not have a gate-flow routine, flow through the control structure gates was not included in this analysis. Consequently, for the 20-year flow event the model domain was divided into area upstream of the control structure and area downstream of the control structure. The fish pass will consist of a series of riffles and pools, which will be designed following the guidance obtained from the Natural Resource Agencies and the main findings of work done by the Corps of Engineers for Lock and Dam 22. Only the connection of the fish pass to the main channel of the RRN was modeled for this analysis.

F-H3.1.2 RRN Control Structure for ND Plan

Approximately 7,000 linear feet of RRN channel and floodplain were modeled for existing conditions at the location of the RRN control structure for the ND plan. The model domain for this location on the RRN includes two major bends in the RRN. A low and a high flow condition were modeled for existing conditions. The low flow event for this location was 6,100 cfs in the RRN. This flow event corresponds to the flow rate in the RRN just prior to flow in the WRR overtopping the east weir into the connecting channel for post-construction conditions. The high flow event selected for this location was the 20-year event.

Approximately the same section of RRN was modeled for post-construction conditions; the same flow conditions were modeled. For the 6,100 cfs flow event, the control structure gates will be completely open and there will be open channel flow through control structure. For this low flow event there will be no water flowing over the connecting channel inlet weir east of the Wild Rice River (however approximately 2,000 ft of backwater in the connecting channel was included in the 2D model domain) or into the fish pass entrance on the RRN. For the 20-year flow the control structure gates will be partially closed, restricting flow through the structure and creating a submerged hydraulic jump downstream of the control structure. Flow will also enter the fish pass and flow over the inlet weir into the connecting channel. Since ADH does not have a gate-flow routine, flow through the control structure gates was not included in this analysis. Consequently for the 20-year flow event the model domain was divided into area upstream of the control structure and an area downstream of the control structure. The fish pass will consist of a series of riffles and pools, which will be designed following the guidance obtained from the Natural Resource Agencies and the main findings of work done by the Corps of Engineers for Lock and Dam 22. Only the connection of the fish pass to the main channel of the RRN was modeled for this analysis.

F-H3.1.3 Wild Rice Control Structure

Approximately 10,800 linear feet of the Wild Rice River were modeled for existing conditions. Two flow conditions were modeled for the existing conditions. . The flow rates were 3,021 cfs and 8,648 cfs, which are the 5-year and 20-year coincidental flows, respectively (i.e. the flow rate on the Wild Rice River that coincides with the respective flow event in the RRN).

The post-construction conditions analysis included a 2,352 cfs flow event and the same 20-year flow as the existing conditions analysis. The 2,352 cfs flow is the approximate flow before water begins to be diverted into the Connecting Channel between the Wild Rice River and the Red River. This flow event covered the same upstream and downstream domains as existing conditions, and included the short diversion through the control structure. The model also included the entire channel between the weirs connecting the Wild Rice River to the Connecting Channel and the Diversion Channel.

Since ADH does not have a gate-flow routine, and since there were concerns about model stability for flow over weirs, the 20-year flow event for the post-construction conditions was divided into four models sections: 1) approximately 6,750 feet upstream of the control structure and 3,250 feet between the two weirs; 2) 2,700 feet downstream of the control structure; 3) 1,350 feet upstream of the weir between the Connecting Channel and the Wild Rice River; and 4) 1,500 feet downstream of the weir between the Wild Rice River and the Diversion Channel. The analysis used flows of 8,648 cfs in the Wild Rice River; 5,985 cfs over the weir from the Connecting Channel; 12,283 cfs to the Diversion Channel; 24 cfs to the fish pass downstream; and a water surface elevation of 910.67 at the control structure. A flow of 2.350 cfs was modeled to pass through the gates, and a water surface elevation of 904.9 was used as the downstream boundary condition.

F-H3.1.4 Sheyenne River Crossing of Diversion Channel

This study reach consisted of approximately 7,800 feet of the Sheyenne River at the crossing of the Diversion Channel. The analysis for the existing conditions used the 2-year local and 50-year coincidental flows of 1,200 cfs and 4,510 cfs, respectively.

The post-construction conditions considered four flow cases, two through the aqueduct and two in the diversion channel. The two cases in the aqueduct included the 2-year local flow (1,200 cfs) and the 10-year flow (2,565 cfs). The two cases in the Diversion Channel included Case 2 (5-year event) for the Diversion Channel (which is 2,554 cfs in the Diversion Channel and 735 cfs over the weir from the Sheyenne River,) and Case 3

(50-year event) on the Diversion channel which is 19,547 cfs in the Diversion Channel and 2,814 cfs over the weir from the Sheyenne River.

F-H3.1.5 Maple River Crossing of Diversion Channel

Approximately 8,500 linear feet of Maple River channel and floodplain were modeled for existing conditions at the location the Maple River crosses the diversion channel. The model domain for this location on the Maple River includes nine major bends in the river. A low and a high flow condition were modeled for existing conditions. The low flow event for this location was 1,685 cfs. This flow event corresponds to the minimum flow that will be allowed to pass through to the protected area during a flood event. This flow rate is the combined 2-year flows in the Maple River, Lower Rush River, and Rush River. The high flow event selected for this location was the 50-year coincidental event (i.e. the flow rate on the Maple that coincides with the 50-year flow in the diversion channel).

For post-construction conditions two flow events were modeled for the aqueduct crossing the diversion channel. The two flow rates for the Maple River and aqueduct were Case 1 (1685 cfs) and Case 2 (the 5-year coincidental event). For Case 1, all flow is allowed to pass through the Maple River aqueduct into the protected area and no flow is diverted over the spillway into the diversion channel. For Case 2, flow is allowed to pass over the spillway into the diversion channel as well as be conveyed through the aqueduct into the protected area.

In addition, three flow conditions were modeled in the diversion channel at the Maple River. The three flow rates modeled for the diversion channel were Case 2 (2,000 cfs), Case 3 (50-year event), and Case 4 (100-year event). Case 2 corresponds approximately to the 4-year event in the diversion channel. For this flow rate there is open channel flow in the diversion channel below the aqueduct and no inflow from the Maple River spillway. (Note: Case 2 for the diversion channel does not correspond to Case 2 for the aqueduct. This is because the 5-year flow in the diversion channel results in pressurized flow below the Maple River aqueduct, which is analyzed as part of Case 3). Case 3 is the 50-year flow in the diversion channel. For this event there is pressurized flow below the aqueduct and inflow from the Maple River spillway. Finally, for Case 4 there is pressurized flow below the aqueduct and inflow from the Maple River spillway, and overtopping of the diversion channel into the aqueduct. The model for Case 4 includes the upper layers the diversion channel that overtops the Maple River aqueduct.

F-H3.1.6 Lower Rush River Connection to Diversion Channel

Approximately 600ft of Lower Rush River was modeled upstream of the spillway to the diversion channel, and approximately 1,000 ft for the diversion channel upstream and downstream of the spillway. Two different flow conditions were modeled, the mean annual flow in the Lower Rush and the coincidental 20-year flow event. Both flow conditions were modeled for the area upstream of the spillway and in the diversion channel. For the model of the Lower Rush River upstream of the spillway, the inflow and water surface elevation boundary conditions were set based on the Phase 3 hydrology rating curve. These boundary conditions are the same for existing conditions as for post-construction conditions. Since the existing channel geometry is also the same as the post-construction geometry, the velocity distributions in the Lower Rush channel will be identical for existing versus post-construction conditions. Consequently, only the post-construction conditions were modeled.

The 20-year coincidental flow on the Lower Rush River (210 cfs) is less than the 2-year local flow on the Lower Rush River (302 cfs). Therefore for this analysis it has been assumed that all flow will be directed over the spillway to the diversion channel and no flow will be directed over the drop structure to the diversion channel.

F-H3.1.7 Rush River Connection to Diversion Channel

Approximately 600ft of Rush River was modeled upstream of the spillway to the diversion channel, and approximately 1,000 ft for the diversion channel upstream and downstream of the spillway. Two different flow conditions were modeled, the mean annual flow in the Rush River and the coincidental 20-year flow event. Both flow conditions were modeled for the area upstream of the spillway and in the diversion channel. For the model of the Rush River upstream of the spillway, the inflow and water surface elevation boundary conditions were set based on the Phase 3 hydrology rating curve. These boundary conditions are the same for existing conditions as for post-construction conditions. Since the existing channel geometry is also the same as the post-construction geometry, the velocity distributions in the Rush channel will be identical for existing versus post-construction conditions. Consequently, only the post-construction conditions were modeled.

The 20-year coincidental flow on the Rush River (420 cfs) is approximately equal to the 2-year local flow on the Rush River (415 cfs). Therefore for this analysis it has been assumed that all flow will be directed over the spillway to the diversion channel and no flow will be directed over the drop structure.

F-H3.1.8 Outlet to RRN, Minnesota Plan

Approximately 6,000 feet of the RRN was modeled at the location where the diversion channel outlets to the RRN for the MN plan. Two flow conditions were modeled for the MN outlet for both existing and post-construction conditions, the 2-year and 20-year events. During the 2-year event there is no inflow from the diversion channel to the RRN. However, during the 2-year event the RRN backs up into the downstream portion of the diversion channel. Approximately 1,600 linear feet of diversion channel was included to model the velocity distribution in this backwater area. For the 20-year event there are inflows from both the RRN and diversion channel.

F-H3.1.9 Outlet to RRN, ND Plan

Approximately 6,000 feet of the RRN was modeled at the location where diversion channel outlets to the RRN for the ND plan. Two flow conditions were modeled for the ND outlet for both existing and post-construction conditions, the 2-year and 20-year flow events. There is no flow from the diversion channel to the RRN during the 5-year event. During this low flow event the RRN will back up into the downstream portion of the diversion channel. Approximately 1,600 linear feet of diversion channel was included to model the velocity distribution in this backwater area. For the 20-year event there are inflows from both the RRN and diversion channel.

F-H3.2 BOUNDARY CONDITIONS

Boundary conditions for the 2D models were assigned based on the available data for each flow condition. Boundary conditions were determined using Phase 3 hydrology, tributary rating curves, Phase 3 HEC-RAS models, and from the hydraulic structure designs.

The existing conditions boundary conditions for the inlet and outlet models along the RRN were determined based on the existing conditions HEC-RAS models. Existing conditions boundary conditions on the tributaries were determined based on Phase 3 hydrology and the rating curve at each tributary.

The post-construction boundary conditions at the inlet and outlet structures were determined based on the Phase 3 – Year 0 HEC-RAS models. The post-construction boundary conditions in the diversion channel at each of the tributaries were determined based on the post-construction Phase 3 Year 0 HEC-RAS models. The post-construction boundary conditions along each tributary were determined based on calculations completed during the design of the hydraulic structures. Boundary conditions for all models are shown in Table F-H2.

Table F-H2 Boundary Conditions

Structure Location	Development Condition	Structure Sub-Part	Flow Condition	Upstream Boundary Condition - 1	Upstream Boundary Condition - 2	Downstream Boundary Condition - 1	Downstream Boundary Condition - 2	Downstream Boundary Condition - 3
MN RRN Control Structure	Existing Conditions	-	9,600 cfs	Q RRN (cfs) 9,600		WSEL RRN (ft) 898.09		
MN RRN Control Structure	Existing Conditions	-	20-year	Q RRN (cfs) 22,000		WSEL RRN (ft) 907.13		
MN RRN Control Structure	Post-Construction	Structure	9,600 cfs	Q RRN (cfs) 9,429	Q Connecting Channel (cfs) 171	WSEL RRN (ft) 898.09		
MN RRN Control Structure	Post-Construction	Structure - Upstream	20-year	Q RRN (cfs) 17,758	Q Connecting Channel (cfs) 4,242	WSEL Weir to Diversion(ft) 906.64	Q Control Structure (cfs) 9,815	Q Fish Pass (cfs) 98
MN RRN Control Structure	Post-Construction	Structure – Downstream	20-year	Q Control Structure (cfs) 9,815	Q Fish Pass(cfs) 98	WSEL RRN (ft) 899.08		
ND RRN Control Structure	Existing Conditions	-	6,100 cfs	Q RRN (cfs) 6,100		WSEL RRN (ft) 901.98		
ND RRN Control Structure	Existing Conditions	-	20-year	Q RRN (cfs) 14,800		WSEL RRN (ft) 910.93		
ND RRN Control Structure	Post-Construction	Structure	6,100 cfs	Q RRN (cfs) 6,100		WSEL RRN (ft) 901.98		
ND RRN Control Structure	Post-Construction	Structure - Upstream	20-year	Q RRN (cfs) 14,800		WSEL Connecting Channel (cfs) 910.91	Q Control Structure (cfs) 8,815	Q Fish Pass(cfs) 88
ND RRN Control Structure	Post-Construction	Structure – Downstream	20-year	Q Control Structure (cfs) 8,815	Q Fish Pass(cfs) 88	WSEL RRN (ft) 905.7		
Wild Rice River	Existing Conditions	-	5-year	Q WRR (cfs) 3,021		WSEL WRR (ft) 906.76		
Wild Rice River	Existing Conditions	-	20-year	Q WRR (cfs) 8,648		WSEL WRR (ft) 914.59		
Wild Rice River	Post-Construction	Structure	2,352 cfs	Q WRR (cfs) 2,352		WSEL WRR (ft) 904.9		
Wild Rice River	Post-Construction	Structure - Upstream	20-year	Q WRR (cfs) 8,648	Q Connecting Channel (cfs) 5,985	WSEL Control Structure (ft) 910.67	Q Diversion Channel (cfs) 12,283	Q Fish Pass(cfs) 24
Wild Rice River	Post-Construction	Structure - Connecting	20-year	Q Connecting Channel (cfs) 5,985		WSEL Connecting Channel(ft) 910.67		
Wild Rice River	Post-Construction	Structure - Diversion	20-year	Q Diversion Channel (cfs) 12,283		WSEL Diversion Channel(ft) 904.6		
Wild Rice River	Post-Construction	Structure - Downstream	20-year	Q Control Structure (cfs) 2,350	Q Fish Pass(cfs) 24	WSEL RRN (ft) 904.9		
Sheyenne River	Existing Conditions	-	2-year (Local)	Q Sheyenne (cfs) 1,200		WSEL Sheyenne (ft) 912.71		
Sheyenne River	Existing Conditions	-	50-year	Q Sheyenne (cfs) 4,510		WSEL Sheyenne (ft) 919.37		
Sheyenne River	Post-Construction	Aqueduct	Case1 (2-year - Local)	Q Sheyenne (cfs) 1,200		WSEL Sheyenne (ft) 912.71		
Sheyenne River	Post-Construction	Aqueduct	Case2 (10-year)	Q Sheyenne (cfs) 2,565		WSEL Sheyenne (ft) 915.12	Q Spillway (cfs) 1,122	
Sheyenne River	Post-Construction	Diversion Channel	Case2 (5-year)	Q Diversion Channel (cfs) 2,554	Q Spillway (cfs) 735	WSEL Diversion Channel (ft) 892.83		
Sheyenne River	Post-Construction	Diversion Channel	Case3 (50-year)	Q Diversion Channel (cfs) 19,547	Q Spillway (cfs) 2,814	WSEL Diversion Channel (ft) 904.82		
Maple River	Existing Conditions	-	2-year (Min. pass through Q)	Q Maple (cfs) 1,685		WSEL Maple (ft) 893.32		
Maple River	Existing Conditions	-	50-year	Q Maple (cfs)		WSEL Maple (ft)		

Table F-H2 Boundary Conditions

Structure Location	Development Condition	Structure Sub-Part	Flow Condition	Upstream Boundary Condition - 1	Upstream Boundary Condition - 2	Downstream Boundary Condition - 1	Downstream Boundary Condition - 2	Downstream Boundary Condition - 3
				4,400		898.83		
Maple River	Post-Construction	Aqueduct	Case1 (2-year - Min. pass through Q)	Q Maple (cfs) 1,685		WSEL Maple (ft) 893.32		
Maple River	Post-Construction	Aqueduct	Case2 (5-year)	Q Maple (cfs) 2,000		WSEL Maple (ft) 892.45	Q Spillway (cfs) 301	
Maple River	Post-Construction	Diversion Channel	Case2 (2,000 cfs)	Q Diversion Channel (cfs) 2,000		WSEL Diversion Channel (ft) 877.2		
Maple River	Post-Construction	Diversion Channel	Case3 (50-year)	Q Diversion Channel (cfs) 23,118	Q Spillway (cfs) 2,408	WSEL Diversion Channel (ft) 891.68		
Maple River	Post-Construction	Diversion Overtopping	Case4 (100-year)	Q Upper Layer Div. Channel (cfs) 3,513	Q Maple (cfs) 2,043	WSEL Diversion Channel (ft) 895.56	Q Maple Gates (cfs) 2,043	
Lower Rush River	Post-Construction	Lower Rush Fish Pass	MAF	Q Lower Rush (cfs) 10		WSEL Fish Pass (ft) 886.7		
Lower Rush River	Post-Construction	Lower Rush Fish Pass	20-year	Q Lower Rush (cfs) 210		WSEL Fish Pass (ft) 891.8		
Lower Rush River	Post-Construction	Diversion Channel	MAF	Q Fish Pass (cfs) 10		WSEL Diversion Channel (ft) 863.46		
Lower Rush River	Post-Construction	Diversion Channel	20-year	Q Diversion Channel (cfs) 16,249	Q Fish Pass (cfs) 210	WSEL Diversion Channel (ft) 886.3		
Rush River	Post-Construction	Rush Fish Pass	MAF	Q Rush (cfs) 17		WSEL Fish Pass (ft) 880.1		
Rush River	Post-Construction	Rush Fish Pass	20-year	Q Rush in (cfs) 420		WSEL Fish Pass (ft) 891.09		
Rush River	Post-Construction	Diversion Channel	MAF	Q Fish Pass (cfs) 17		WSEL Diversion Channel (ft) 861.26		
Rush River	Post-Construction	Diversion Channel	20-year	Q Diversion Channel (cfs) 16,879	Q Fish Pass (cfs) 420	WSEL Diversion Channel (ft) 885.13		
MN RRN Outlet	Existing Conditions	-	2-year	Q RRN (cfs) 8,328		WSEL RRN (ft) 874.29		
MN RRN Outlet	Existing Conditions	-	20-year	Q RRN (cfs) 28,491		WSEL RRN (ft) 887.17		
MN RRN Outlet	Post-Construction	Outlet	2-year	Q RRN (cfs) 8,328		WSEL RRN (ft) 874.29		
MN RRN Outlet	Post-Construction	Outlet	20-year	Q RRN (cfs) 16,375	Q Diversion Channel (cfs) 12,116	WSEL RRN (ft) 887.17		
ND RRN Outlet	Existing Conditions	-	2-year	Q RRN (cfs) 8328		WSEL RRN (ft) 870.05		
ND RRN Outlet	Existing Conditions	-	20-year	Q RRN (cfs) 28,491		WSEL RRN (ft) 883.21		
ND RRN Outlet	Post-Construction	Outlet	2-year	Q RRN (cfs) 8,688		WSEL RRN (ft) 870.26		
ND RRN Outlet	Post-Construction	Outlet	20-year	Q RRN (cfs) 13,362	Q Diversion Channel (cfs) 16,879	WSEL RRN (ft) 883.37		

F-H3.3 MODEL CALIBRATION

Models were calibrated by visually comparing the resulting velocity distributions from models of existing conditions to available measured velocity distributions as well as existing HEC-RAS results. Typically, in calibrating 2D models, the Manning's n values and the EEV card values can be adjusted to best match velocity distributions and water surface elevations. In this analysis, it was decided to hold the EEV card constant at a value of 0.5. Only the Manning's n values for the various materials were adjusted during the calibration process.

F-H3.3.1 Compare upstream water surface elevations to HEC-RAS

Due to the relatively short stream-wise length of the models and low slope of the energy grade line (EGL), generally there was little variation in upstream water surface elevation as a function of user-supplied model coefficients. For 2D models of the RRN and Diversion Channel, the upstream water-surface elevation in the 2D model was compared to the water-surface elevation calculated in the Phase 3-Year 0 or existing conditions HEC-RAS models. The length of the model reach and the land use materials in the model affected the match of the 2D model elevations to the HEC-RAS elevations. For example the water surface elevation on the RRN for the 20-year event at the Minnesota inlet at the upstream end of the 2D model was only 0.1-ft higher than the HEC-RAS model developed for this phase of the study. For reaches where trees lined the channel, the 2D model results deviated further from the HEC-RAS model results. One example is the 20-year event on the RRN at the location of the ND outlet. In this area the water surface elevation at the upstream end of the 2D model is 0.3-ft higher than the HEC-RAS model.

F-H3.3.2 Compare velocity distribution to measured data

Data available from USGS stations on the RRN and the modeled tributaries were used to compare the velocity distributions at locations along the portion of modeled channels to measured velocity distributions for similar flow rates for existing conditions. This was done to verify that the model was calculating velocity distributions within the channel approximated past measured velocity distributions. It was not expected that the modeled velocity distributions would match the measured distributions exactly, due to differences in geometry (e.g. the stream may have been straighter where the velocity was measured, and have more bends in the modeled portion), roughness of the stream bed, or bathymetry. However, this did provide validation that the existing conditions models approximated the measured data. For example, the velocities on the Maple for the existing conditions 2-year event (1,685 cfs) were 2.0-2.2 ft/s near the thalweg and 0.5 ft/s near the banks. In comparison the measured velocities at USGS station 05060100 on April 1, 2009 at 2,510 cfs show channel velocities ranging from 2.0-3.0 ft/s. In a few locations modeled velocities did not match the measured velocities as well. For example, for the 20-year event (22,000 cfs) at the MN inlet location for existing conditions the modeled velocity was 1.7-2.0 ft/s. Measured velocity data from USGS gage 05051522 at Hickson, from March 25, 2009, (21,500 cfs) ranged from approximately 2.0-2.5 ft/sec.

F-H4.0 RESULTS

The following section discusses the results of the two-dimensional flow models. The section is organized by structure. The results are presented in two formats: screen captures from computer simulations, and graphs. The screen captures from computer simulations show the resulting velocity fields for the various structures and flow conditions. The results are superimposed over aerial images. Color is used indicate the magnitude of the velocity, blue corresponds to slow velocities and red corresponds to high velocities. Vectors are plotted on the screen captures to show the direction of flow throughout the velocity field. For the most part the areas covered by color and vectors are the wetted areas under the given flow condition; the areas not covered by color and vectors are dry. Exceptions include some areas of standing water with velocities very close to zero that, for some reason, SMS does not color. Graphs are shown to highlight and compare flow velocities and/or depths for one flow condition to another (e.g., existing conditions versus post-construction conditions), or for one location to another on a given structure (e.g., through-structure location to downstream location). Graphs show either the velocity or depth plotted as a function of distance across a river/channel section. The location of the river channel sections from which data in the graphs is taken is shown on the screen captures as solid black lines.

F-H4.1 RRN CONTROL STRUCTURE FOR MINNESOTA PLAN

F-H4.1.1 Existing Conditions

Results for the Minnesota inlet control structure are shown in Figures F-H1 to F-H14. Two flow events were analyzed for the Minnesota inlet control structure, 9,600 cfs and the 20-year event. During the 9,600 cfs event flows are primarily contained within the river banks and follow the general river alignment. Along the main channel velocities remain consistent throughout the section of river analyzed varying from 2.0-2.3 ft/s along the thalweg and 0.0-0.5 at the banks. During the 20-year event flow overtop the river banks and flows in the floodplain before reconnecting back to the Red River. Because there is more flow in the overbanks during the 20-year event velocities along the thalweg vary throughout the section of river analyzed and range between 0.7-2.5 ft/s and between 0.3-1.5 ft/s along the banks. In floodplain velocities range between 0-0.5 ft/s, and at large river bends, flow begins to flow overland short circuiting the bend rather than following the channel alignment.

F-H4.1.2 Post-Construction Conditions

Velocity distributions post-construction were analyzed for the same two flow conditions, a 9,600 cfs event and the 20-year event. During the 9,600 cfs event all flow is allowed to pass through the structure on the Red River into the protected area; no flow is directed over the weir in the diversion channel. Therefore water in the connecting channel flows west towards the Red River during low flow events. Velocities in the connecting channel and immediately upstream of the weir to the diversion channel are approximately 0.0-0.3 ft/s. Velocities along the Red River remain similar to existing conditions, 2.0-2.3 ft/s along the thalweg of the channel, and approximately 2.0 ft/s through the control structure. Immediately upstream and downstream from the Red River structure in the constructed channels velocities are slightly lower, 1.3-1.5 ft/s, than existing conditions because the size of the constructed channel is slightly larger than the adjacent river section. During the 9,600 cfs event there are storage areas of inundation in the diversion channel and in the portion of Red River channel that was disconnected to construct the control structure that have very low velocities 0.0-0.2 ft/s.

During the 20-year event flow is directed into the diversion channel. Velocities upstream and downstream of the control structure on the Red River along the channel thalweg are approximately 1.0-1.5 ft/s. The velocities in the Red River are lower than existing conditions primarily because a significant amount of the flow is conveyed in the connecting channel which has velocities between 1.0-1.5 ft/s. During the 20-year event some flow is directed into the diversion channel. The velocity at the entrance to the diversion channel is 2.0-2.5 ft/s. Similar to the 9,600 cfs event there is an area of inundation upstream and downstream of the control channel in the portion of Red River that was disconnected during construction that act as storage areas and have velocities of 0.0-0.2 ft/s. Finally, the velocity in the downstream portion of the fish pass, where it connects to the constructed channel is approximately 0.7 ft/s. The area of higher velocity in the fish pass is a result of model boundary conditions and does not reflect the velocity in the fish pass. Because the entrance to the fish pass is submerged during the 20-year event, varying the geometry of the fish pass had no impact on the modeled velocities.

F-H4.2 RRN CONTROL STRUCTURE FOR NORTH DAKOTA PLAN

F-H4.2.1 Existing Conditions

Results for the North Dakota inlet control structure are shown in Figures F-H15 to F-H26. Two flow events were analyzed for the North Dakota Inlet structure, 6,100 cfs and the 20-year event. During the 6,100 cfs event flows just begin to overtop the river banks, which create inundated areas in the overbanks that have very low velocities. Along the main channel velocities remain consistent throughout the section of river analyzed varying from 1.5-2.0 ft/s along the thalweg and 0.5-1.0 at the banks. During the 20-year event flow overtop the river banks and flows in the floodplain before reconnecting back to the Red River and the downstream end of the modeled area. Velocities along the thalweg range between 1.7-2.2 ft/s and between 0.6-1.1 ft/s along the banks. In floodplain velocities range between 0-0.5 ft/s, and at large river bends, flow begins to flow overland short circuiting the bend rather than following the channel.

F-H4.2.2 Post-Construction Conditions

Velocity distributions post-construction were analyzed for the same two flow conditions, a 6,100 cfs event and the 20-year event. During the 6,100 cfs event all flow is allowed to pass through the structure on the Red River into the protected area; no flow is directed over the weir in the connecting channel located east of the Wild Rice River. Velocities along the Red River remain similar to existing conditions, 1.5-2.0 ft/s along the thalweg of the channel, and approximately 1.5 ft/s through the control structure. Immediately upstream and downstream from the Red River structure in the constructed channels velocities are slightly lower, 1.0-1.5 ft/s, than existing conditions because the size of the constructed channel is slightly larger than the adjacent river section. During the 6,100 cfs event there are storage areas of inundation in the connecting channel and in the portion of

Red River channel that was disconnected to construct the control structure that have very low velocities 0.0-0.2 ft/s.

During the 20-year event some flow is directed into the connecting channel. Velocities upstream of the control structure along the channel thalweg are approximately 2.0-2.5 ft/s and decrease to 1.5-2.0 ft/s in the channel downstream of the structure in the protected area. The velocity in the entrance to the connecting channel is 1.0-1.5 ft/s. Similar to the 6,100 cfs event there is an area of inundation downstream of the control channel in the portion of Red River that was disconnected during construction. Finally, the velocity in the downstream portion of the fish pass, where it connects to the constructed channel is approximately 0.1-0.3 ft/s. Because the entrance to the fish pass is submerged during the 20-year event varying the geometry of the fish pass had no impact on the modeled velocities.

F-H4.3 WILD RICE CONTROL STRUCTURE

F-H4.3.1 Existing Conditions

Results for the Wild Rice River structure are shown in Figures F-H27 to F-H42. Two existing conditions flow events, the 5-year and 20-year coincidental events, were simulated for the Wild Rice River. During the 5-year event, flows are starting to spill over the river banks. There is a location just downstream from the location of the proposed diversion channel crossing where flow leaves the main channel and then returns the river channel at the next river bend. Velocities along the channel thalweg are approximately 2.0 - 2.2 ft/s and 0.5 ft/s near the banks. During the 20-year event the river banks are overtopped and there are large inundated areas where velocities are very small. During the 20-year event velocities along the channel thalweg are approximately 1.5-2.0 ft/s and less than 0.5 ft/s near the banks and in the floodplain. Overall, the velocity distribution across the channel remains relatively constant for both the 5-year and 20-year existing conditions models throughout the area of river analyzed.

F-H4.3.2 Post-Construction Conditions

Velocity distributions for the post-construction aqueduct crossing were analyzed for two flow conditions, the 2,352 cfs and 20-year events. During the 2,352 cfs event, all flow is allowed to pass between the two weirs in the diversion channel and through the control structure on the Wild Rice River into the protected area. For larger events the weir in the diversion channel east of the Wild Rice River is over topped. Velocities upstream and downstream from the diversion channel are approximately 2.0 ft/s along the thalweg. In the constructed connecting channel velocities are slightly lower, 1.2-1.5 ft/s due to the increase in channel cross sectional area. Because there is no flow over the east or west weirs the section of diversion channel between the two weirs becomes a large backwater area, similar to the two sections of Wild Rice River that were abandoned to construct the crossing, where the velocities are less than 0.1 ft/s. Velocity on the Wild Rice River increase to 2.5-2.7 ft/s as flow is restricted and passes through the control structure.

During the 20-year event, water from the RRN connecting channel flows over the east weir and mixes with inflows from the Wild Rice River. Some flow is allowed to pass through the Wild Rice River control structure into the protected area, and the rest of the flow is directed over the west weir into the diversion channel. The velocities in the connecting channel vary between 1.5 ft/s at the center of the channel to less than 0.5 ft/s near the sides. During the 20-year event, all of the flow is contained within the connecting channel, and given the uniform channel geometry the velocity distribution remains constant throughout the section of channel velocity analyzed.

Velocities upstream of the diversion channel area vary between 2.5-4.0 ft/s along the thalweg, and 0.0-0.5 ft/s near the banks and in the floodplain. Velocities are higher compared to existing conditions because the downstream water surface elevation has been lowered for the same flow rate. Flow from the Wild Rice River mixes with inflows from the connecting channel. In the diversion channel velocities east of the Wild Rice River are approximately 1.5 ft/s and increase to 3.5-4.0 ft/s in the western portion of the diversion channel. Velocities near the crest of the west weir reach 6.0-7.0 ft/s. Velocities through the control structure on the Wild Rice River are 2.0-2.5 ft/s.

Velocities downstream of the Wild Rice River control structure in the constructed channel vary between 1.5 ft/s near the middle of the channel to 0.5 ft/s closer to the banks. Flow in the constructed channel mixes with the small inflow from the fish pass downstream of the control structure. As a result of the relatively small amount of flow in the fish pass the velocity vector field in the connecting channel is not greatly disturbed by flows from the fish pass. Velocities near the fish pass entrance are approximately 0.3 ft/s. After the constructed channel rejoins the undisturbed section of the Wild Rice River velocities increase to 1.5-2.0 ft/s which is similar to existing conditions. Finally, there is a large area of backwater area in the section of Wild Rice River that was abandoned to construct the control structure. In these area velocities are less than 0.1 ft/s.

F-H4.4 SHEYENNE RIVER CROSSING OF DIVERSION CHANNEL

F-H4.4.1 Existing Conditions

Results for the Sheyenne River structure are shown in Figures F-H43 to F-H62. Two existing conditions flow events, the 2-year and 50-year events, were analyzed for the Sheyenne River. During the 2-year event (which was defined as the 2-year local flow event for the Sheyenne River) flow stays primarily within the river banks. Velocities along the channel thalweg are approximately 1.5-2.0 ft/s and less than 0.5 ft/s near the banks. During the 50-year event, the river banks are overtopped and there are large inundated areas where velocities are very small. For this analysis only the effective flow area defined by the FIS HEC-RAS models adjacent to the River was analyzed. During the 50-year event velocities along the channel thalweg are approximately 2.5-3.0 ft/s and 0.5 ft/s near the banks. Overall the velocity distribution across the channel remains relatively constant for both existing conditions events throughout the area of river analyzed.

F-H4.4.2 Post-Construction Conditions – Aqueduct Crossing

Velocity distributions for the post-construction aqueduct crossing was analyzed for two flow conditions, Case 1 (2-year local event) and Case 2 (10-year coincidental event). During the 2-year event all flow is allowed to pass over the aqueduct into the protected area. The area upstream of the spillway acts as a backwater storage area and velocities are less than 0.1 ft/s. Velocities upstream and downstream from the aqueduct are similar to the existing conditions event; approximately 1.5-1.8 ft/s along the thalweg. Near the aqueduct velocities increase slightly to 1.8 ft/s as flow is restricted and passes through the constructed aqueduct channel.

During the 10-year event, some flow is directed over the spillway into the diversion channel. Velocities upstream of the spillway along the channel thalweg are approximately 2.5-3.0 ft/s and decrease to 1.5-2.0 ft/s downstream of the spillway as some flow is redirected to the spillway. Velocities over the aqueduct are approximately 2.0 ft/s and decrease to 1.5-2.0 ft/s in the channel downstream of the aqueduct in the protected area. The velocity at the entrance to the spillway is 1.5 ft/s and increases to approximately 5.0 ft/s at the near the crest of the spillway as the water depth decreases.

F-H4.4.3 Post-Construction Conditions – Diversion Channel

Two flow conditions in the diversion channel were analyzed for post-construction conditions, the 5- and 50-year events. During the 5-year event the water surface elevation in the diversion channel remains below the bottom of the aqueduct crossing. During this event the velocities along the center of the diversion channel are 1.5-2.0 ft/s, and remain constant throughout the section of diversion channel modeled. Model results indicate that flow from the Sheyenne spillway enters the diversion channel at velocities near 5.0-7.0 ft/s. This high velocity is due to the assumed slope of the spillway as it connects to the Diversion Channel.

During the 50-year event the water surface elevation in the diversion channel is higher than the bottom of the aqueduct. This results in pressurized flow below the Sheyenne River aqueduct crossing. Velocities in the diversion channel upstream of the aqueduct are approximately 2.5-3.3 ft/s and increase to 6.0-7.0 ft/s below the aqueduct crossing. Downstream of the aqueduct crossing the velocities return to 2.5-3.0 ft/s as the channel expands. The spillway velocity during the 50-year event is 2.5 ft/s which is similar to the velocities in the diversion channel.

F-H4.5 MAPLE RIVER CROSSING OF DIVERSION CHANNEL

F-H4.5.1 Existing Conditions

Results for the Maple River structure are shown in Figures F-H63 to F-H83. Two existing conditions flow events, the 2-year and 50-year events, were analyzed for the Maple River. During the 2-year event (which is defined as the minimum pass through flow of the 2-year events on the Maple, Rush, and Lower Rush Rivers) flow stays primarily within the river banks. Velocities along the channel thalweg are approximately 2.0 - 2.2 ft/s and 0.5 ft/s near the banks. During the 50-year event the river banks are overtopped and there are large inundated areas where velocities are very small. During the 50-year event velocities along the channel thalweg are approximately 2.0-3.0 ft/s and 0.5 ft/s near the banks. Overall the velocity distribution across the channel remains relatively constant for both the 2-year and 50-year existing conditions models throughout the area of river analyzed.

F-H4.5.2 Post-Construction Conditions – Aqueduct Crossing

Velocity distributions for the post-construction aqueduct crossing were analyzed for two flow conditions, the 2-year (Case 1) and 5-year (Case 2) events. During the 2-year event all flow is allowed to pass over the aqueduct into the protected area. Velocities upstream and downstream from the aqueduct are similar to the existing conditions event; approximately 2.0 ft/s along the thalweg. Near the aqueduct velocities increase slightly to 2.5 ft/s as flow is restricted and passes through the constructed aqueduct.

During the 5-year event some flow is directed over the spillway into the diversion channel. Velocities upstream of the aqueduct along the channel thalweg are approximately 2.0-2.5 ft/s and increase to 2.8 ft/s over the aqueduct before returning to 2.0-2.5 ft/s in the channel downstream of the aqueduct in the protected area. The velocity at the entrance to the spillway is 0.5 ft/s and increases to 2.5 ft/s at the near the crest of the spillway as the water depth decreases. For both the 2- and 5-year events there are areas of inundation to the east and west of the diversion channel in the portion of Maple River channel that was disconnected to construct the aqueduct crossing.

F-H4.5.3 Post-Construction Conditions – Diversion Channel

Three flow conditions in the diversion channel were analyzed for post-construction conditions, a 2000 cfs event (Case 2), 50-year (Case 3) and 100-year (Case 4) events. During the 2000 cfs event the water surface elevation in the diversion channel is below the bottom of the aqueduct crossing. During this event the velocities along the center of the diversion channel are 1.5-2.0 ft/s, and remain constant below the aqueduct.

During the 50-year event the water surface elevation in the diversion channel is higher than the bottom of the aqueduct. This results in pressurized flow below the Maple River crossing. Velocities in the diversion channel upstream of the aqueduct are approximately 2.5-3.0 ft/s and increase to 7.0-8.0 ft/s below the aqueduct crossing. Downstream of the aqueduct crossing the velocities return to 2.5-3.0 ft/s as the channel expands. The spillway velocity during the 50-year event is 1.5-2.0 ft/s which is similar to the velocities in the diversion channel.

Finally, during the 100-year event water in the diversion channel overtops the aqueduct and flow from the diversion channel mixes with flow in the aqueduct. For this case only the upper layers of the diversion channel were included in this analysis. During this event water flows into the diversion channel aqueduct at velocities of 7.0-10.0 ft/s and mixes with flow in the aqueduct which is traveling at approximately 1.5-2.0 ft/s. The flows mix and some flow is pass through gates into the protected area with velocities between 2.5-3.0 ft/s and the remaining flow is conveyed downstream in the diversion channel at velocities of 6.0-7.0 ft/s. Downstream of the aqueduct velocities return to 2.5-3.0 ft/s in the full diversion channel section.

F-H4.6 LOWER RUSH RIVER CONNECTION TO DIVERSION CHANNEL

F-H4.6.1 Post-Construction Conditions – Spillway

Results for the Lower Rush River are shown in Figures F-H84 to F-H87. Two flow conditions were analyzed for the Lower Rush River upstream of the spillway; the mean average flow (MAF) and the 20-year coincidental event. During the MAF event all flow is contained within the banks and is directed over the spillway. Velocities in the channel are approximately 0.1 ft/s and approach 0.3 ft/s near the spillway. During the 20-year event flow just beings to overtop the river banks (i.e. in the Lower Rush River the 20-year coincidental flow rate 210 cfs is less than the 2-year local event 302 cfs). During the 20-year event velocities along the channel thalweg are approximately 0.4 ft/s and approach 0.8 ft/s near the spillway. Overall the velocity distribution across the channel remains relatively constant for both the 2-year and 20-year existing conditions models throughout the section of river analyzed.

F-H4.6.2 Post-Construction Conditions – Diversion Channel

Two flow conditions were analyzed for the diversion channel adjacent to the Lower Rush River spillway; the mean average flow (MAF) in the Lower Rush and the 20-year event in the diversion channel. During the MAF event there is no flow in the diversion channel upstream of the Lower Rush connecting structure. Velocities in the channel downstream of the spillway are approximately 0.3 ft/s and 0.2 ft/s upstream of the spillway. During the 20-year event there is a relatively large amount of flow in the diversion channel compared to the contributing flow from the Lower Rush River spillway, and the addition of the flows from the spillway have no visible effect on the velocities in the diversion channel. During the 20-year event velocities in the diversion channel are approximately 2.5 ft/s and approach 0.2 ft/s in the spillway.

F-H4.7 RUSH RIVER CONNECTION TO DIVERSION CHANNEL

F-H4.7.1 Post-Construction Conditions – Spillway

Results for the Rush River are shown in Figures F-H88 to F-H91 Two flow conditions were analyzed for the Rush River upstream of the spillway; the mean average flow (MAF) and the 20-year coincidental event. During the MAF event all flow is contained within the banks and is directed over the spillway. Velocities in the channel are approximately 0.3 ft/s and approach 1.0 ft/s near the spillway. During the 20-year event flow just beings to overtop the river banks (i.e. in the Rush River the 20-year coincidental flow rate 410 cfs is approximately equal to the 2-year local event 415 cfs). During the 20-year event velocities along the channel thalweg are approximately 0.2 ft/s and approach 0.6 ft/s near the spillway. Overall the velocity distribution across the channel remains relatively constant for both the 2-year and 20-year existing conditions models throughout the section of river analyzed.

F-H4.7.2 Post-Construction Conditions – Diversion Channel

Two flow conditions were analyzed for the diversion channel adjacent to the Rush River spillway; the mean average flow (MAF) in the Lower Rush and the 20-year event in the diversion channel. During the MAF event there is no flow in the diversion channel and all of the flow is contained in the bottom of the diversion channel. Velocities in the channel downstream of the spillway are approximately 0.9 ft/s and less than 0.1ft/s in the diversion channel upstream of the spillway. During the 20-year event there is a relatively large amount of flow in the diversion channel compared to the contributing flow from the Rush River spillway, and the addition of the flows from the spillway have no visible effect on the velocities in the diversion channel. During the 20-year event velocities in the diversion channel are approximately 2.7 ft/s and approach 0.3 ft/s in the spillway.

F-H4.8 OUTLET TO RRN, MINNESOTA PLAN

F-H4.8.1 Existing Conditions

Results for the Minnesota alignment outlet to the RRN are shown in Figures F-H92 to F-H99. Two flow conditions were analyzed for the outlet of the MN Plan, the 2- and 20-year events. During the 2-year event the flow is primarily contained within the river banks and follows the river alignment. Velocities along the channel thalweg are approximately 2.0 - 2.5 ft/s and 1.5-2.0 ft/s near the banks. During the 20-year event the river banks are overtopped, and water flows through the overbanks rather than following the river alignment. During the 20-year event velocities along the channel thalweg are approximately 2.5-3.0 ft/s, 1.0-1.5 ft/s near the banks, and 0-0.5 ft/s in the floodplain. Overall the velocity distribution across the channel remains relatively constant for both the 2-year and 20-year existing conditions models throughout the section of river analyzed.

F-H4.8.2 Post-Construction Conditions

During the 2-year event there is no flow in the diversion channel. During this event water from the RRN backs up into the lower portion of the diversion channel. Velocities in the RRN upstream and downstream from the diversion channel outlet remain similar to existing conditions velocities along the thalweg, 2.0-2.5 ft/s, and near the banks1.5-2.0 ft/s. During this flow condition the downstream portion of the diversion channel is a backwater area and velocities are near zero.

During the 20-year event there is flow in the diversion channel. Velocities in the RRN upstream of the diversion channel outlet are between 1.0-1.5 ft/s along the thalweg and 0.5-1.0 near the banks, which is slightly lower than existing conditions due to the lower flow rate in the RRN. Velocities in the RRN increase downstream of the outlet of the diversion channel to 2.5-3.0 ft/s along the thalweg and 1.5-2.0 near the banks, which is similar to the existing conditions velocity distribution. Velocities at the center of the diversion channel are between 2.0-2.5 ft/s.

F-H4.9 OUTLET TO RRN, NORTH DAKOTA PLAN

F-H4.9.1 Existing Conditions

Results for the North Dakota alignment outlet to the RRN are shown in Figures F-H100 to F-H107. Two flow conditions were analyzed for the outlet of the ND Plan, the 2- and 20-year events. During the 2-year event the flow is primarily contained within the river banks and follows the river alignment. At locations of river meanders flow overtops the banks and flows back into the RRN. Velocities along the channel thalweg are approximately 2.0 - 2.5 ft/s and 1.5-2.0 ft/s near the banks. Overall the velocity distribution across the channel remains relatively constant for both the 2-year existing conditions model throughout the section of river analyzed. During the 20-year event the river banks are overtopped, and water flows through the overbanks rather than following the river alignment. During the 20-year event velocities near the channel thalweg vary between 0.5-1.5 ft/s, and are generally less than 0.5 ft/s in the floodplain. In general, during the 20-year event velocities are remain relatively uniform across the throughout the floodplain. This is due to the large amount of flow that occurs in the overbanks, which does not follow the alignment of the channel due to the large meanders in this section of river.

F-H4.9.2 Post-Construction Conditions

During the 2-year event there is no flow in the diversion channel. During this event water from the RRN backs up into the lower portion of the diversion channel. Velocities in the RRN upstream and downstream from the diversion channel outlet remain similar to existing conditions velocities along the thalweg, 2.0-2.5 ft/s, and near the banks1.5-2.0 ft/s. During this flow condition the downstream portion of the diversion channel is a backwater area and velocities are near zero.

During the 20-year event there is flow in the diversion channel. Velocities in the RRN upstream and downstream of the diversion channel outlet are between 0.5-1.0 ft/s, which is similar to existing conditions due to the large amount of flow through the floodplain. Velocities at the center of the diversion channel are between 1.5-2.0 ft/s.

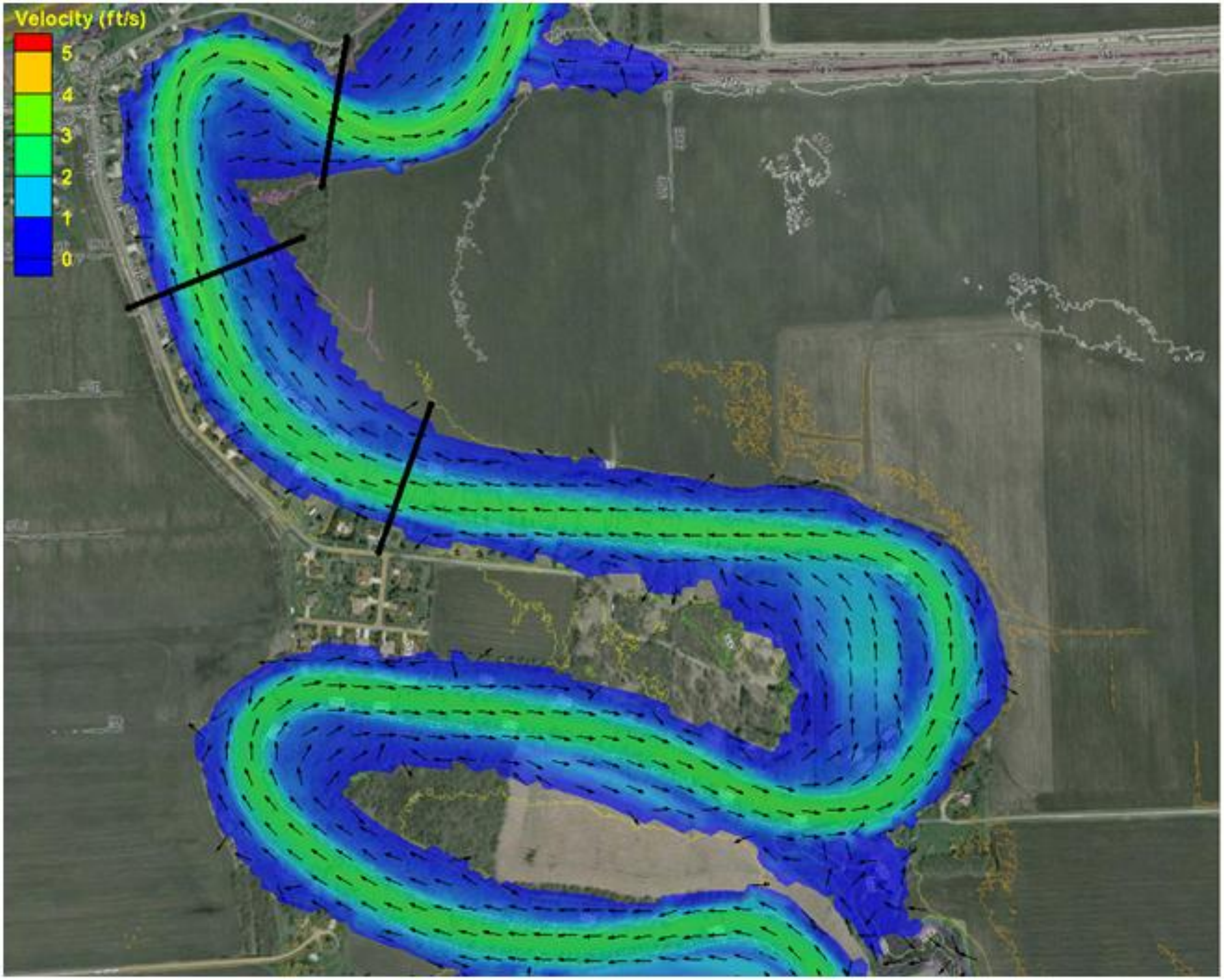

Figure F-H1 Velocity Field for Minnesota Control Structure Existing Conditions 9,600 cfs Event Flow

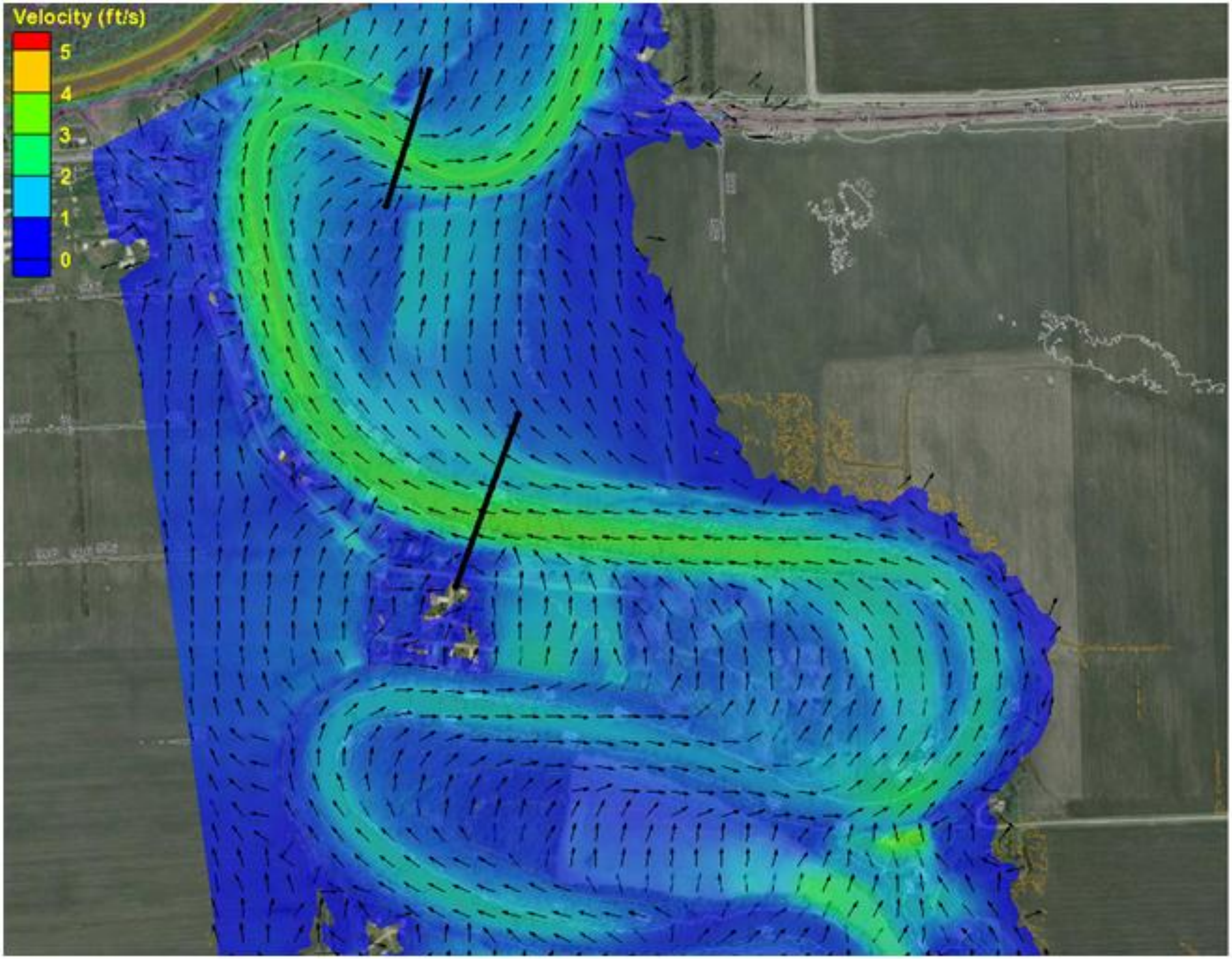

Figure F-H2 Velocity Field for Minnesota Control Structure Existing Conditions 20-Year Event Flow

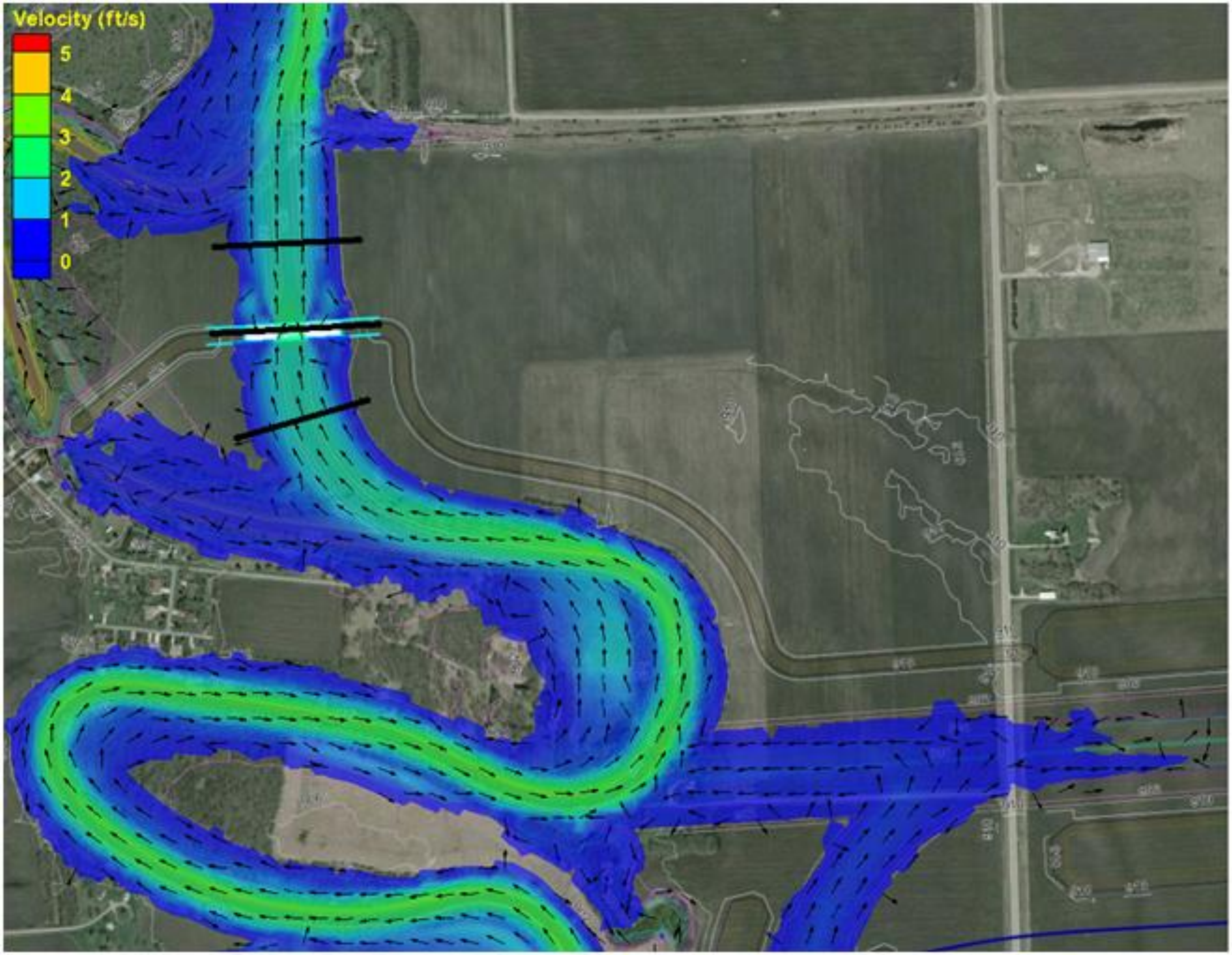

Figure F-H3 Velocity Field for Minnesota Control Structure Post-construction Conditions 9,600 cfs Event Flow

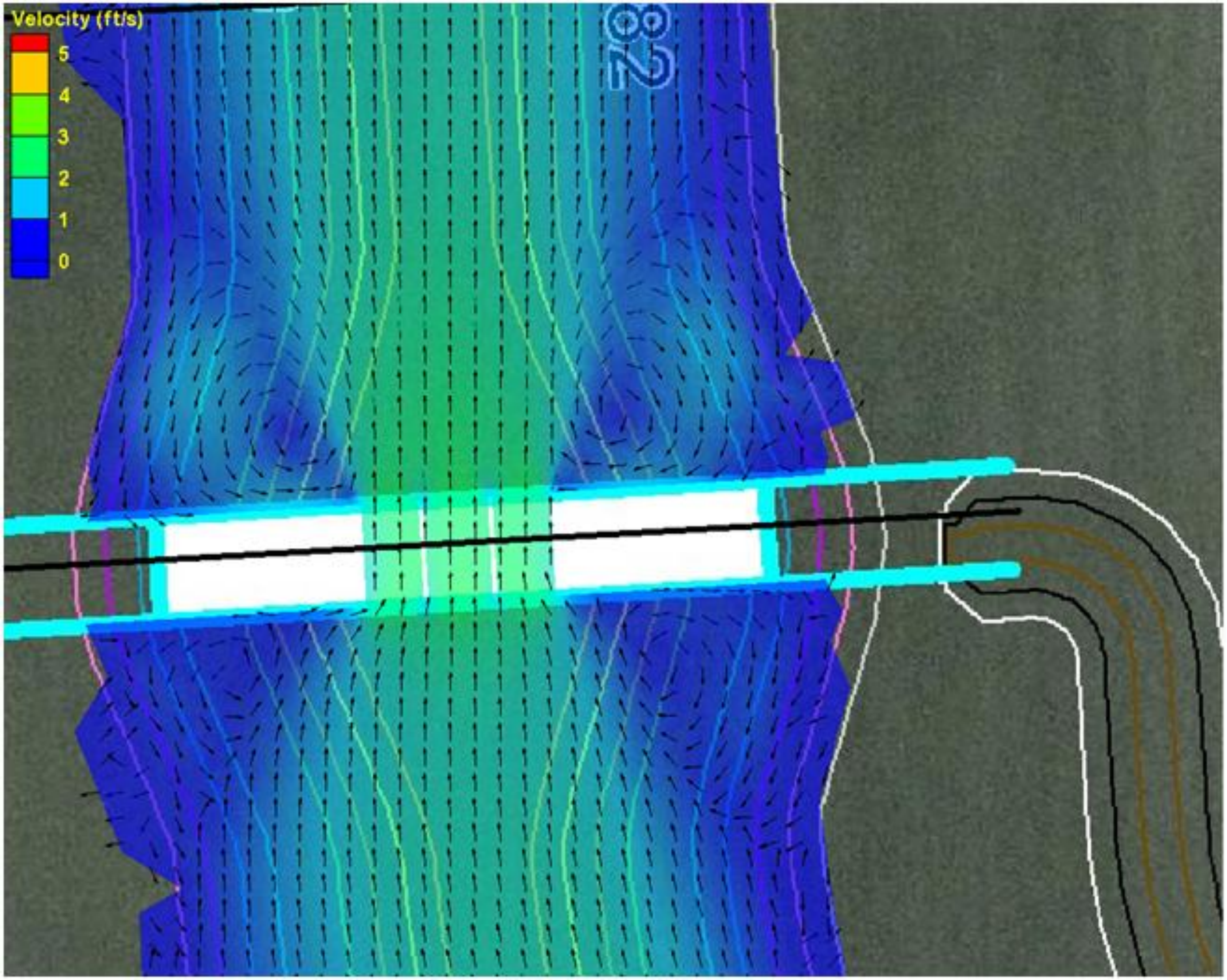

Figure F-H4 Velocity Field for Minnesota Control Structure Post-construction Conditions Control Structure Close Up 9,600 cfs Event Flow

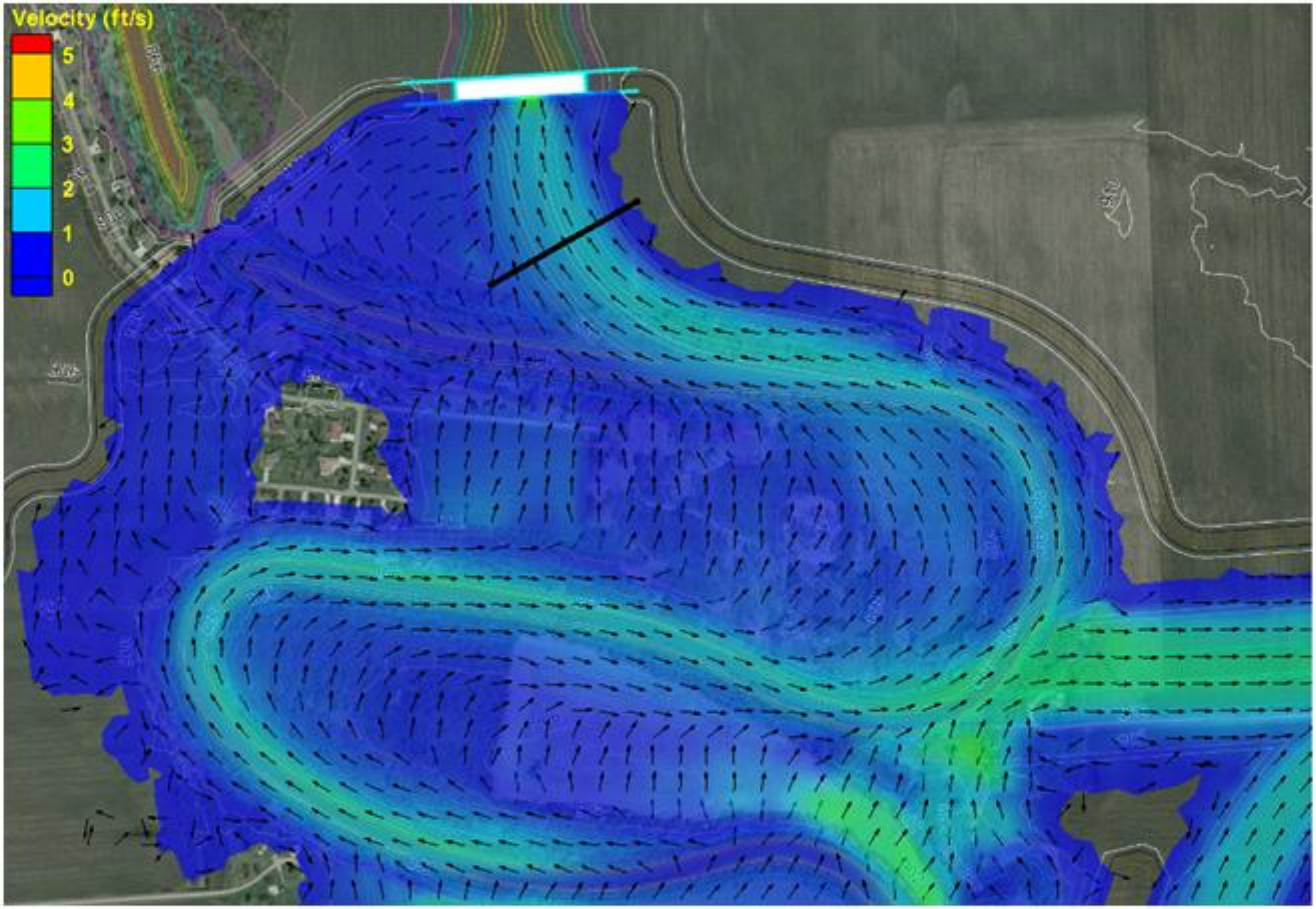

Figure F-H5 Velocity Field for Minnesota Control Structure Post-construction Conditions Upstream 20-Year Event Flow

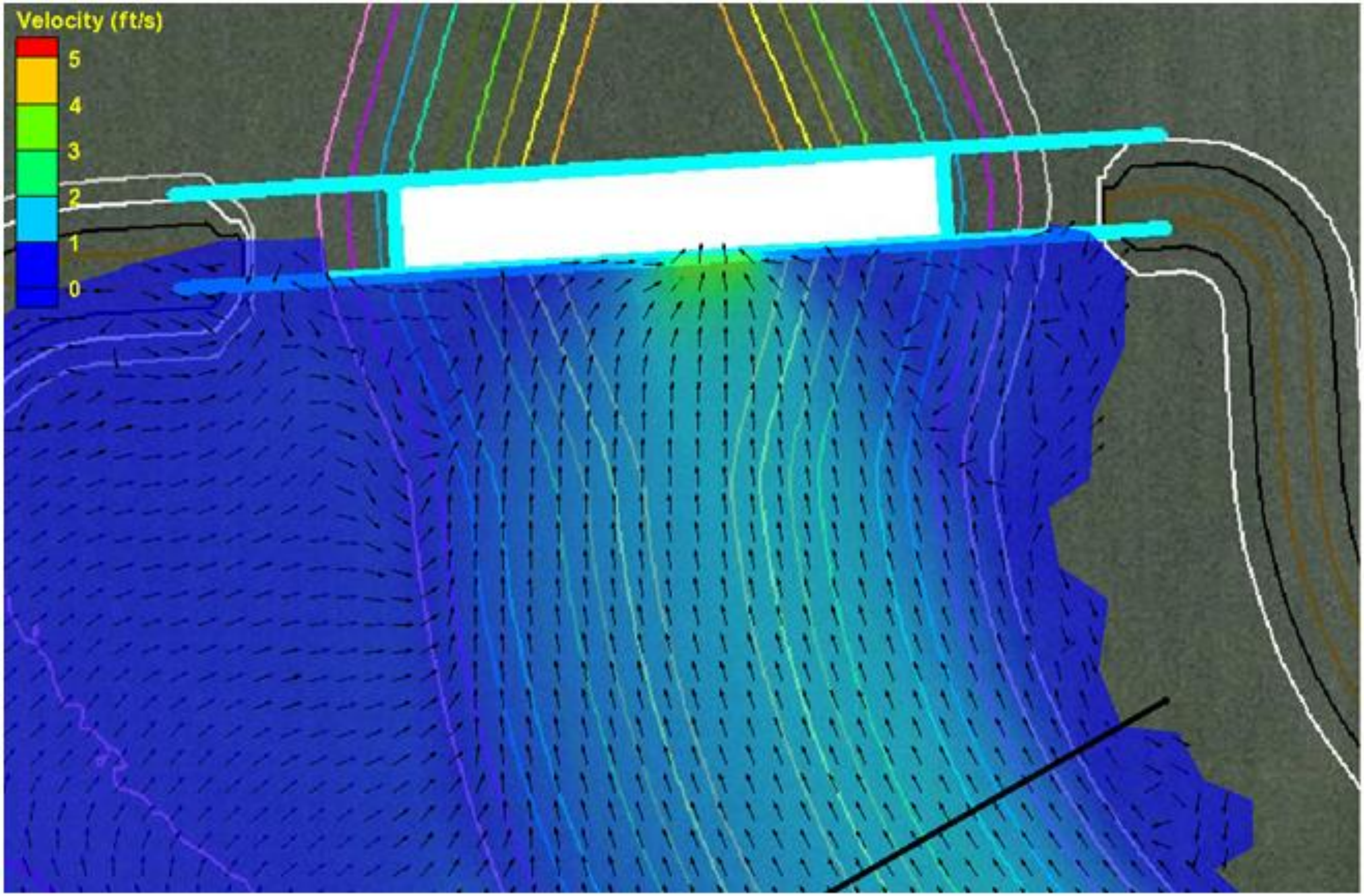

Figure F-H6 Velocity Field for Minnesota Control Structure Post-construction Conditions Upstream Control Structure Close Up 20-Year Event Flow

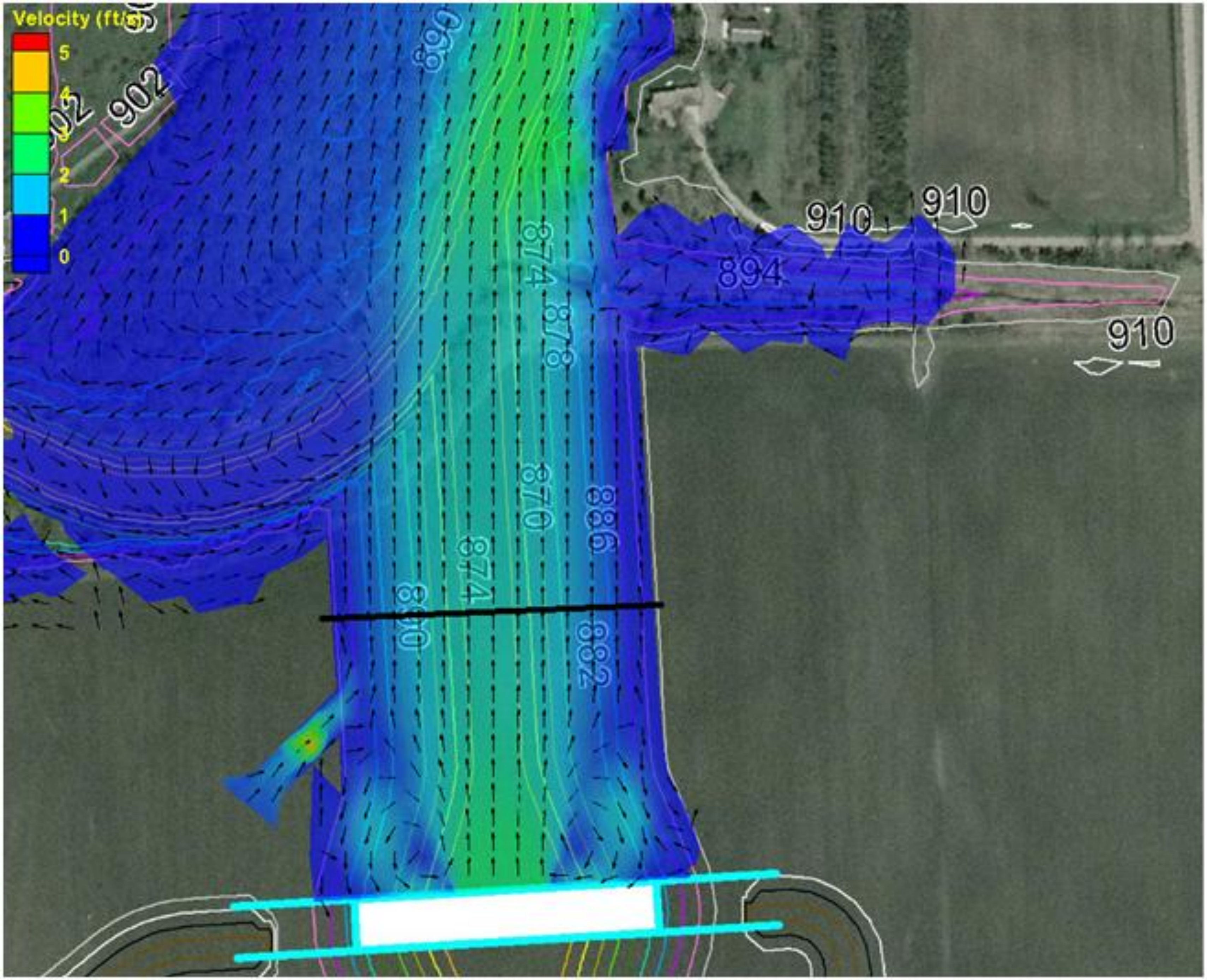

Figure F-H7 Velocity Field for Minnesota Control Structure Post-construction Conditions Downstream 20-Year Event Flow

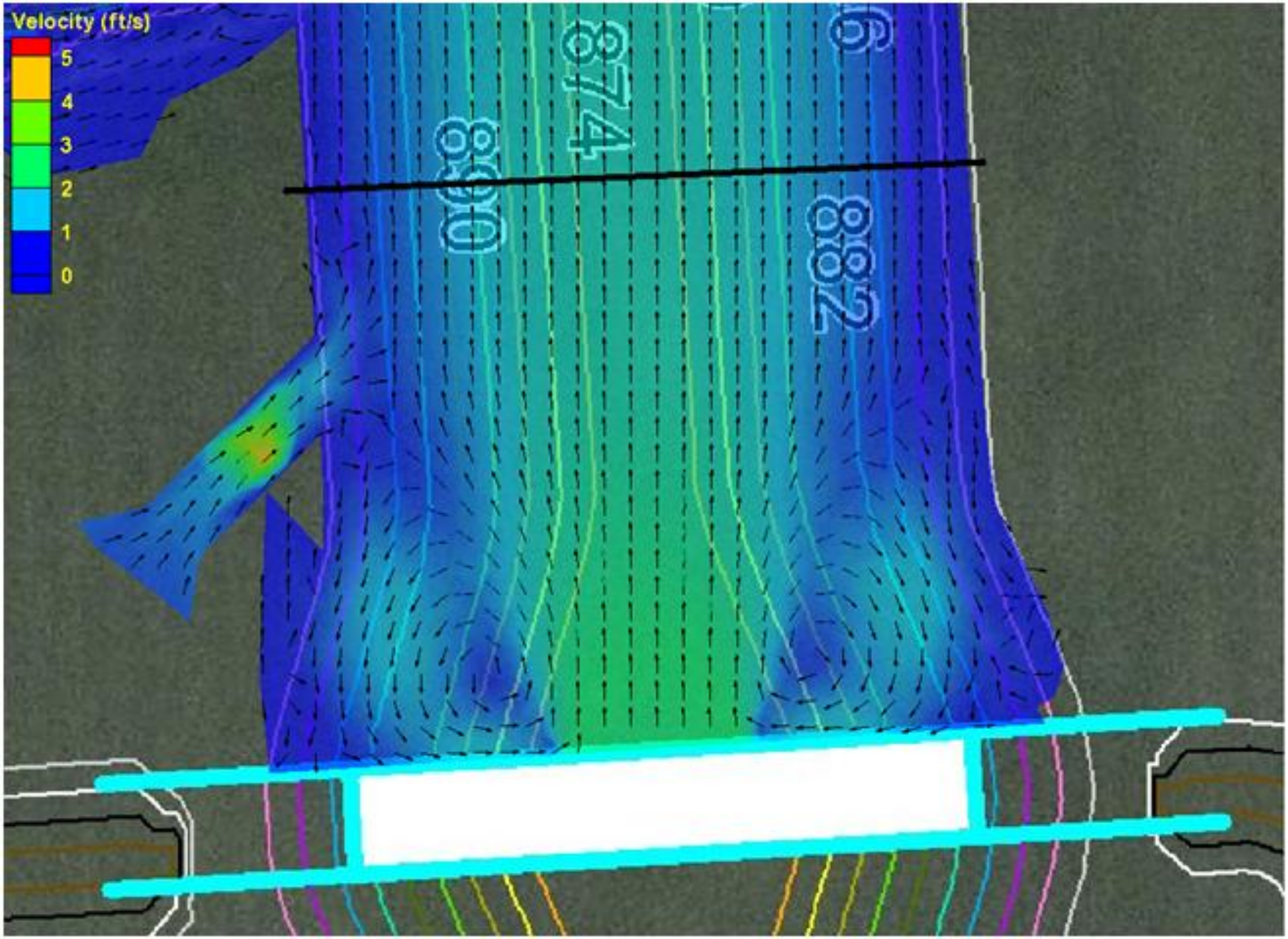

Figure F-H8 Velocity Field for Minnesota Control Structure Post-construction Conditions Downstream Control Structure Close Up 20-Year Event Flow

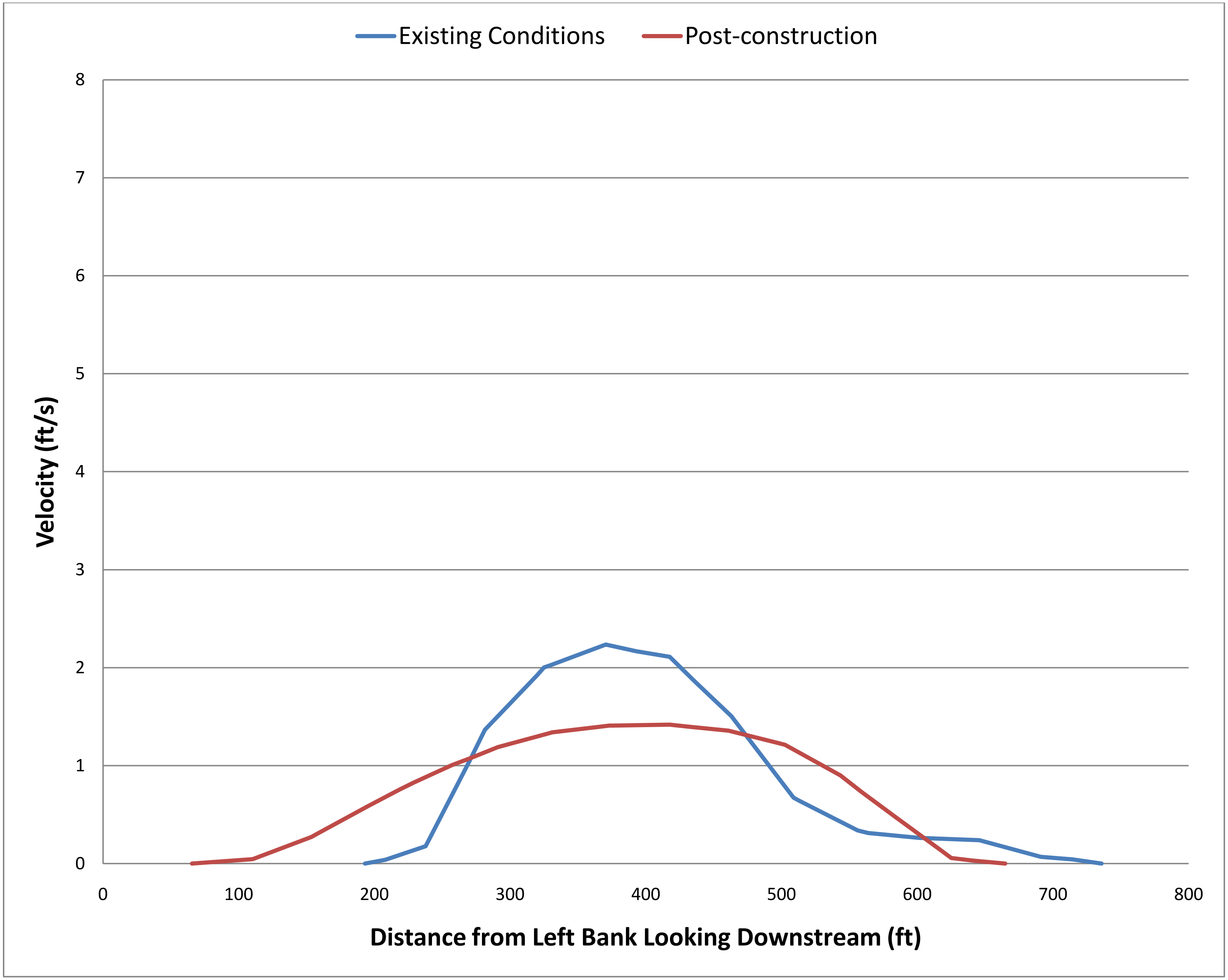

Figure F-H9 Cross Sectional Velocity Data for Minnesota Control Structure Upstream During 9,600 cfs Event

Cross Sectional Velocity Data for Minnesota Control Structure (9,600 cfs). Compares velocity at existing conditions to post-construction upstream of structure.

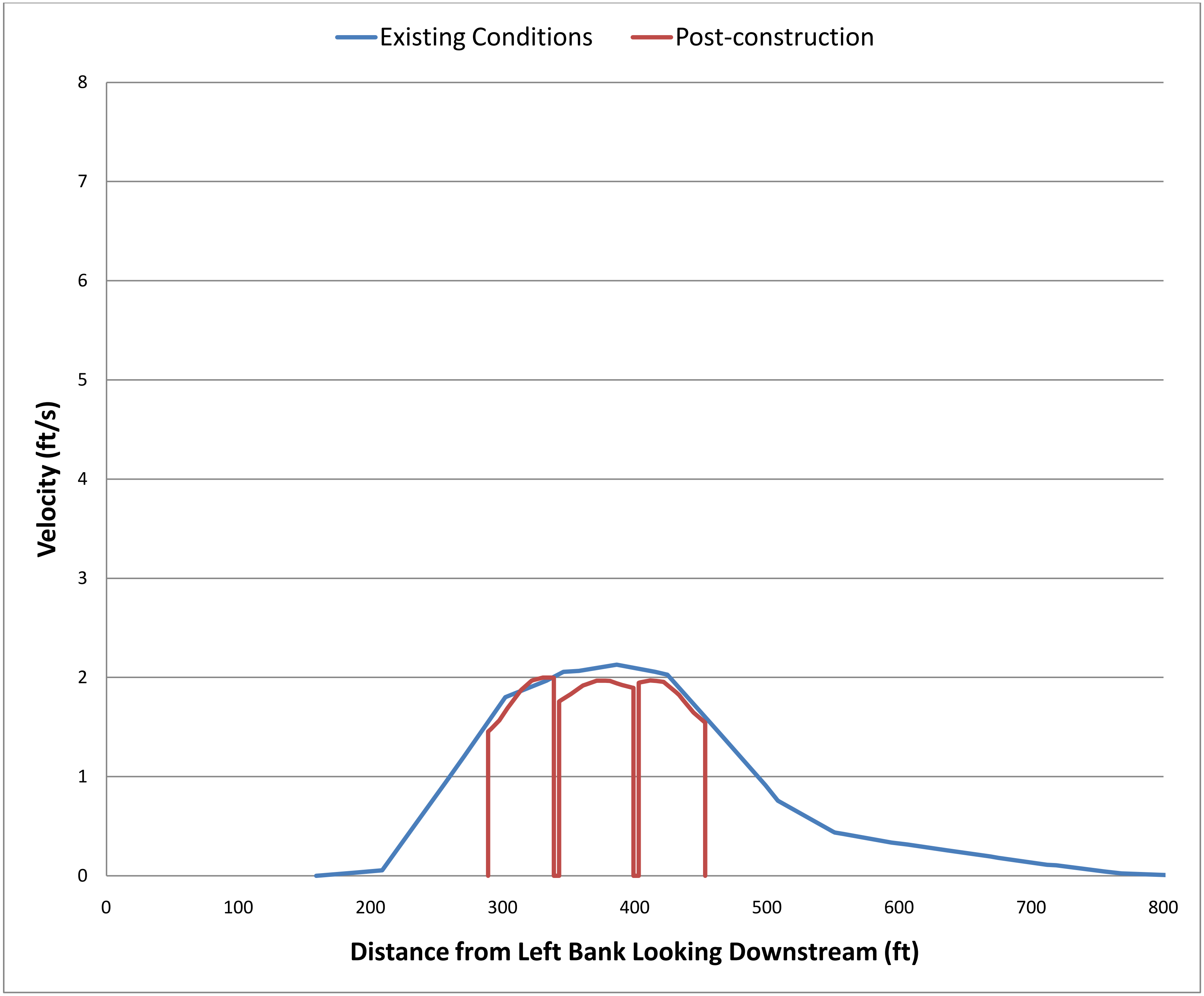

Figure F-H10 Cross Sectional Velocity Data for Minnesota Control Structure Through Structure During 9,600 cfs Event

Cross Sectional Velocity Data for Minnesota Control Structure (9,600 cfs). Compares velocity at existing conditions to post-construction through structure.

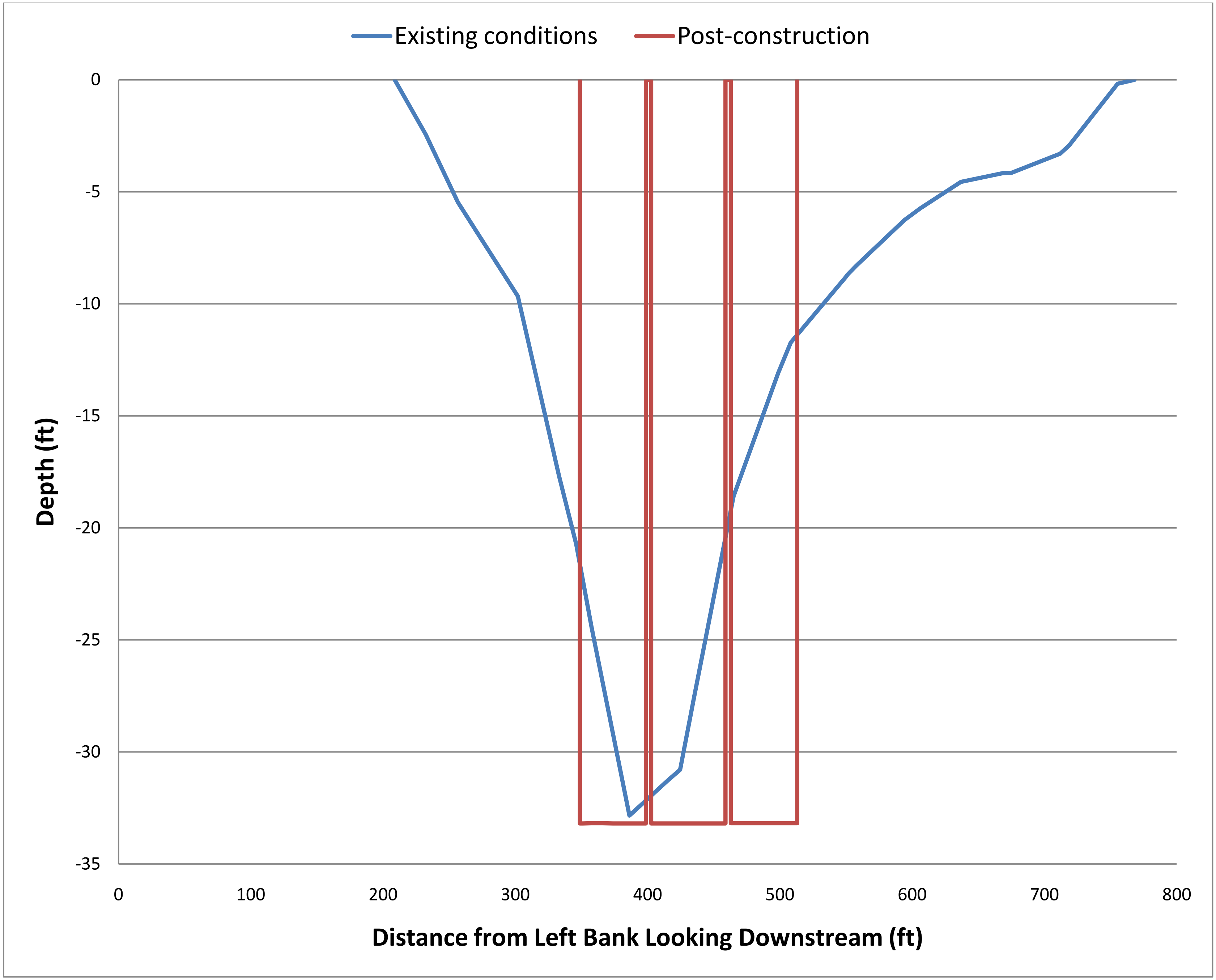

Figure F-H11 Cross Sectional Depth Data for Minnesota Control Structure Through Structure During 9,600 cfs Event

Cross Sectional Depth Data for Minnesota Control Structure (9,600 cfs). Compares depth at existing conditions to post-construction through structure.

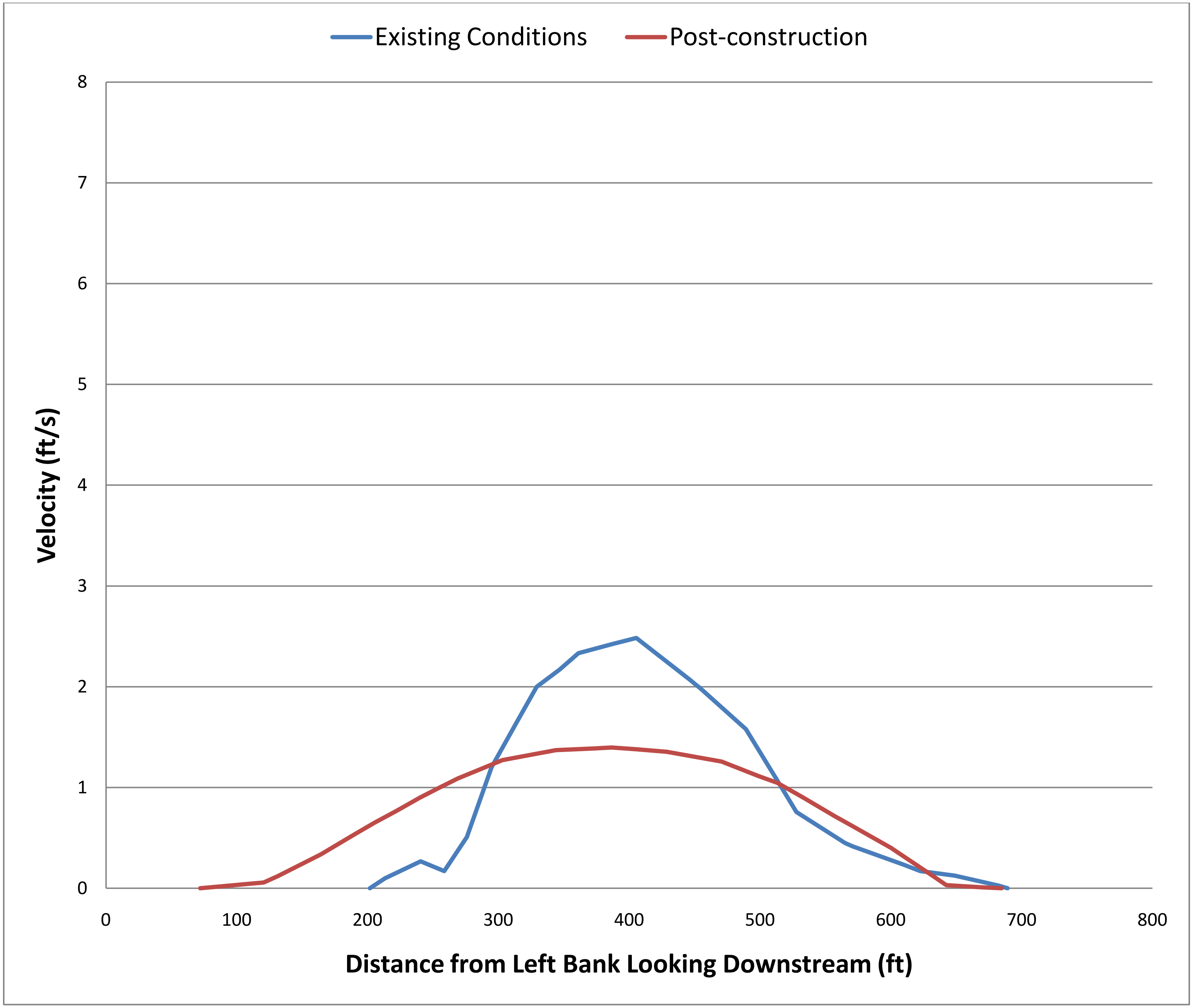

Figure F-H12 Cross Sectional Velocity Data for Minnesota Control Structure Downstream During 9,600 cfs Event

Cross Sectional Velocity Data for Minnesota Control Structure (9,600 cfs). Compares velocity at existing conditions to post-construction downstream of structure.

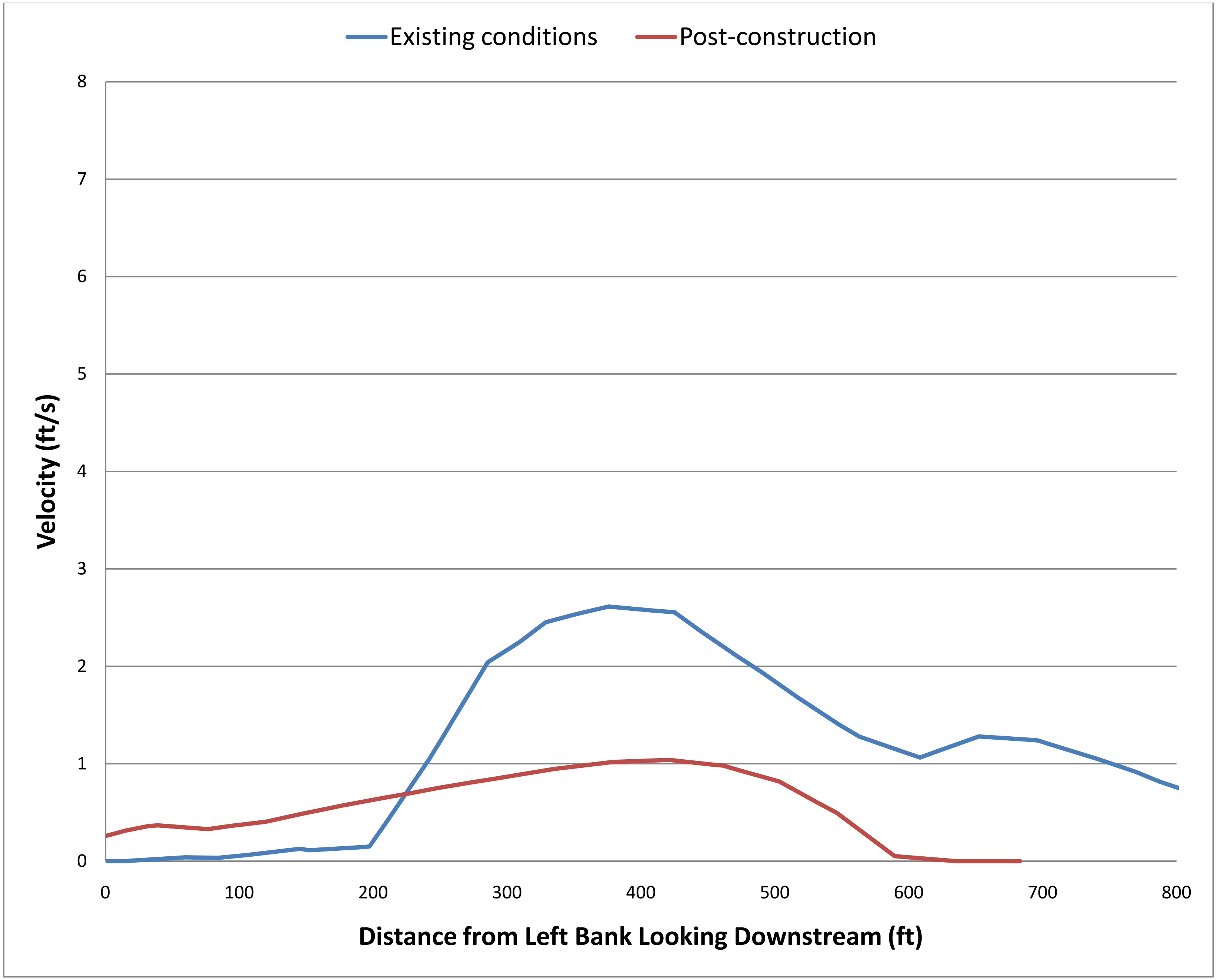

Figure F-H13 Cross Sectional Velocity Data for Minnesota Control Structure Upstream During 20-Year Event

Cross Sectional Velocity Data for Minnesota Control Structure (20-Year). Compares velocity at existing conditions to post-construction upstream of structure.

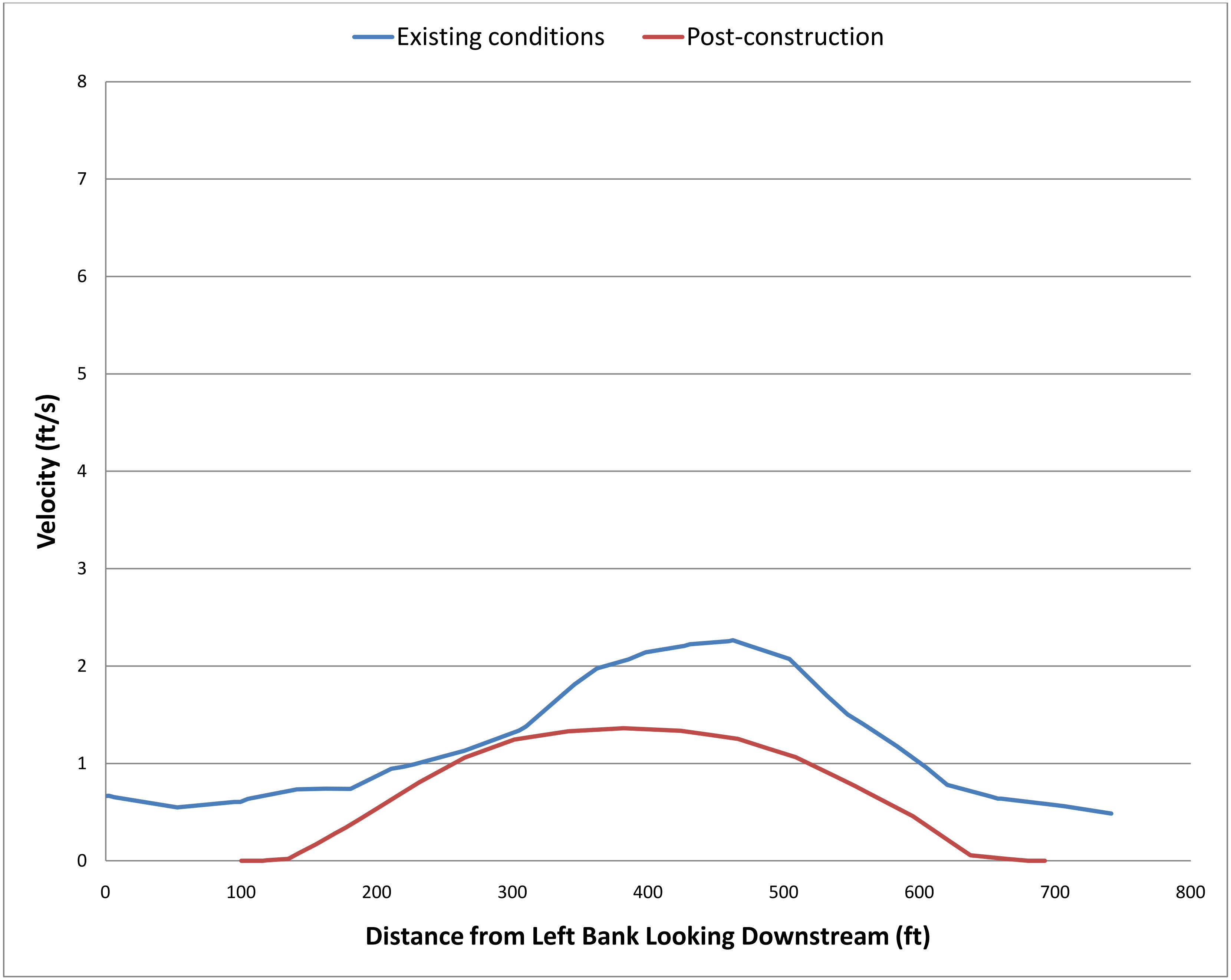

Figure F-H14 Cross Sectional Velocity Data for Minnesota Control Structure Downstream During 20-Year Event

Cross Sectional Velocity Data for Minnesota Control Structure (20-Year). Compares velocity at existing conditions to post-construction downstream of structure.

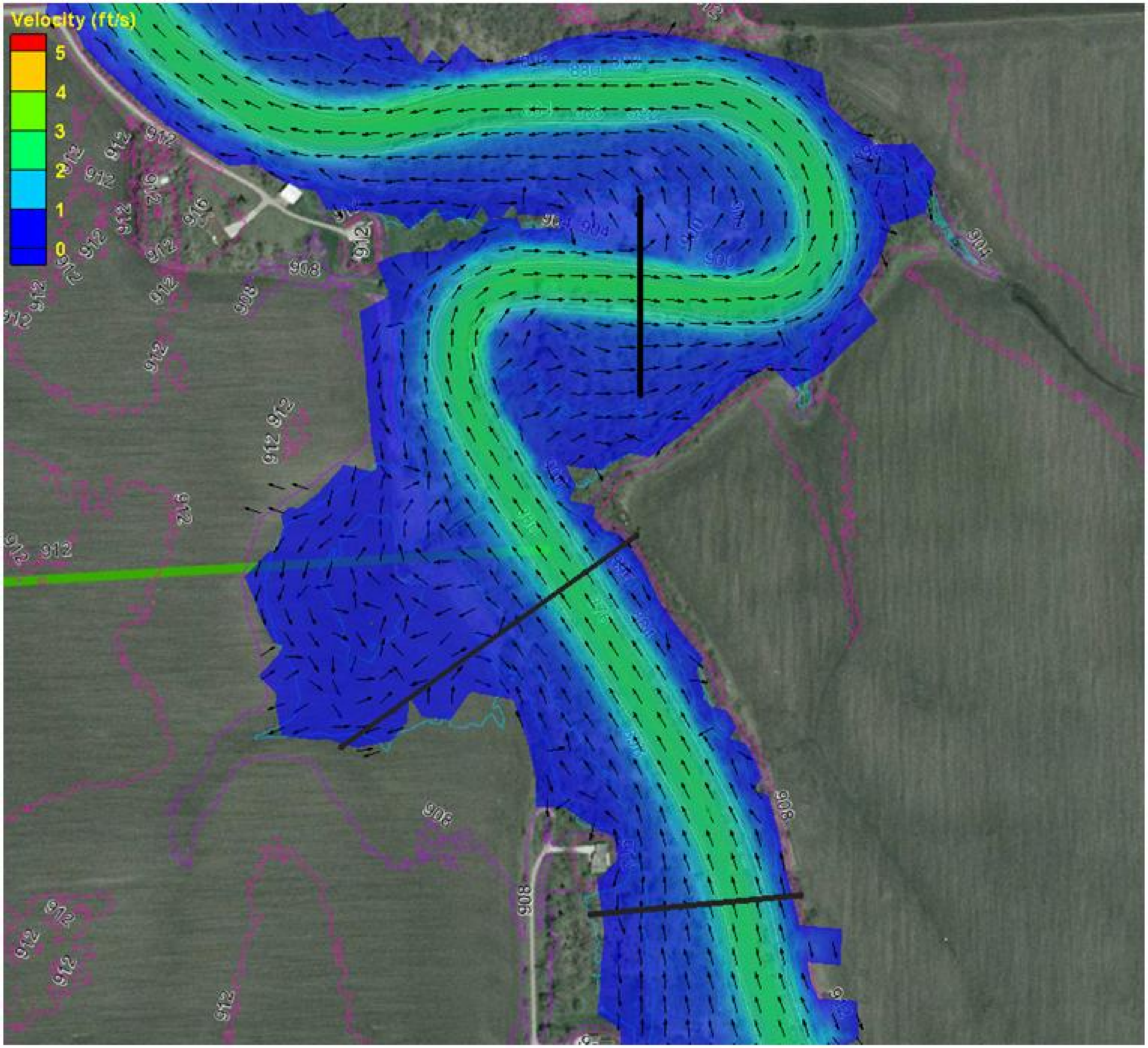

Figure F-H15 Velocity Field for North Dakota Control Structure Existing Conditions 6,100 cfs Event Flow

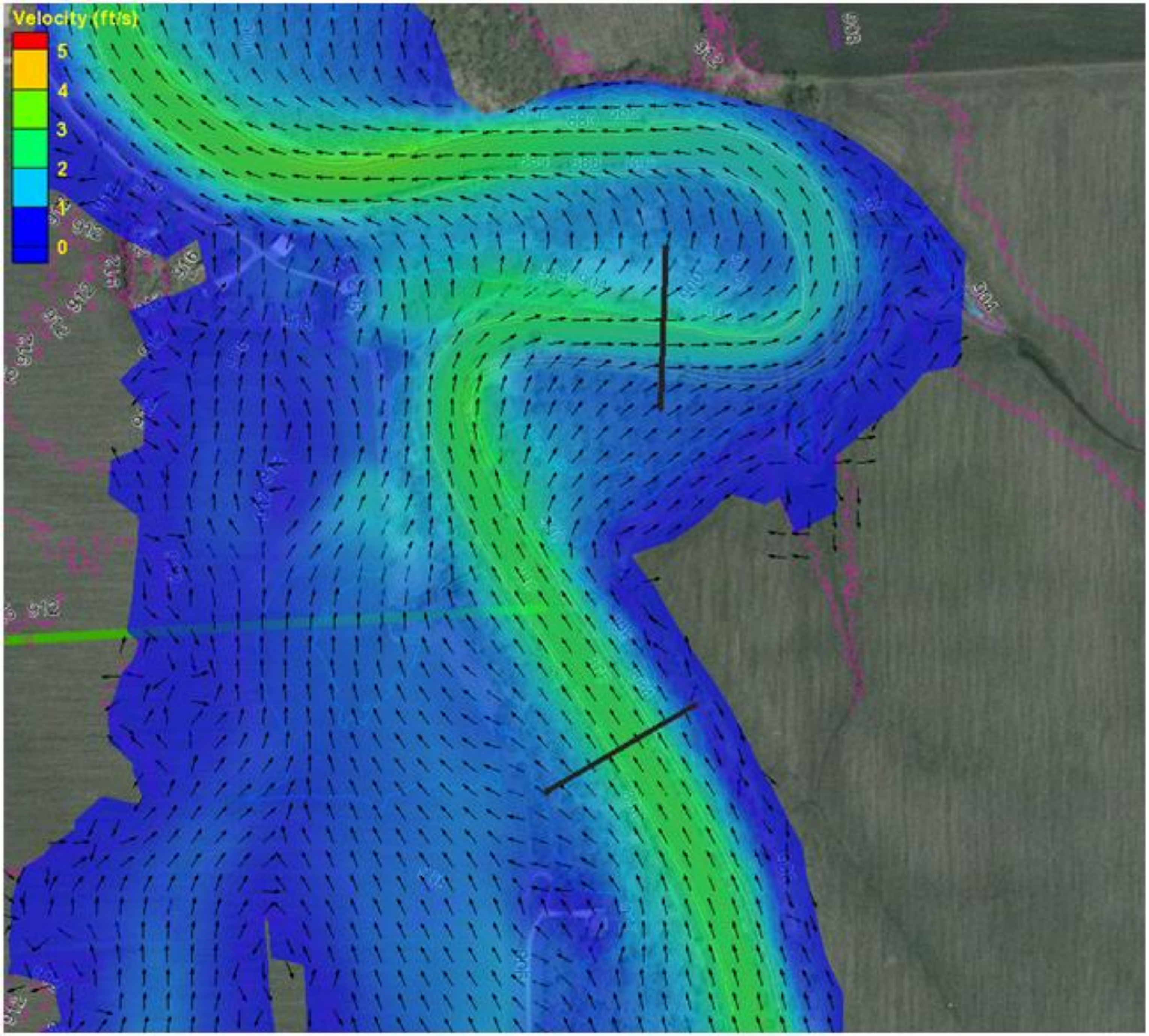

Figure F-H16 Velocity Field for North Dakota Control Structure Existing Conditions 20-Year Event Flow

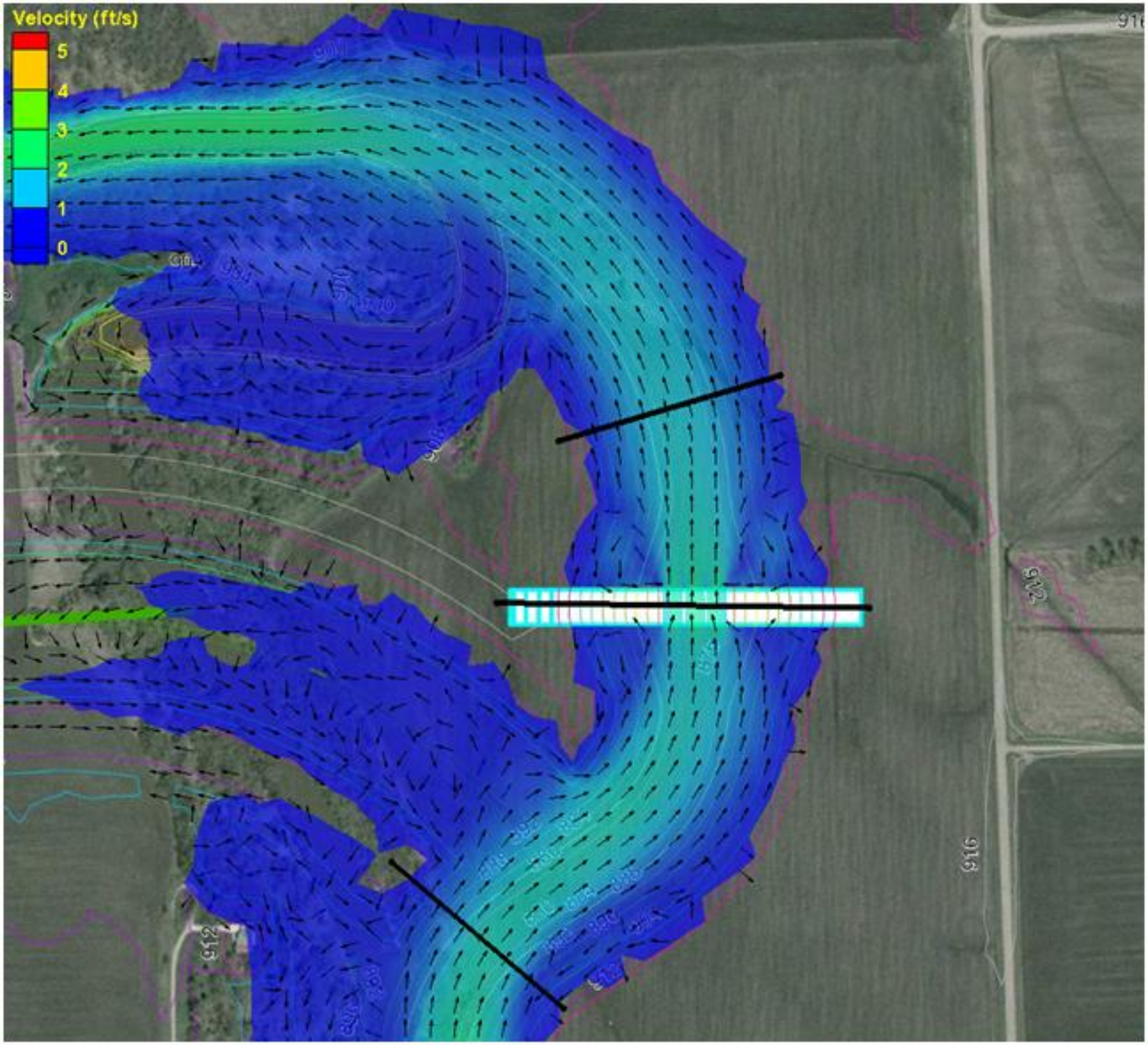

Figure F-H17 Velocity Field for North Dakota Control Structure Post-construction Conditions 6,100 cfs Event Flow

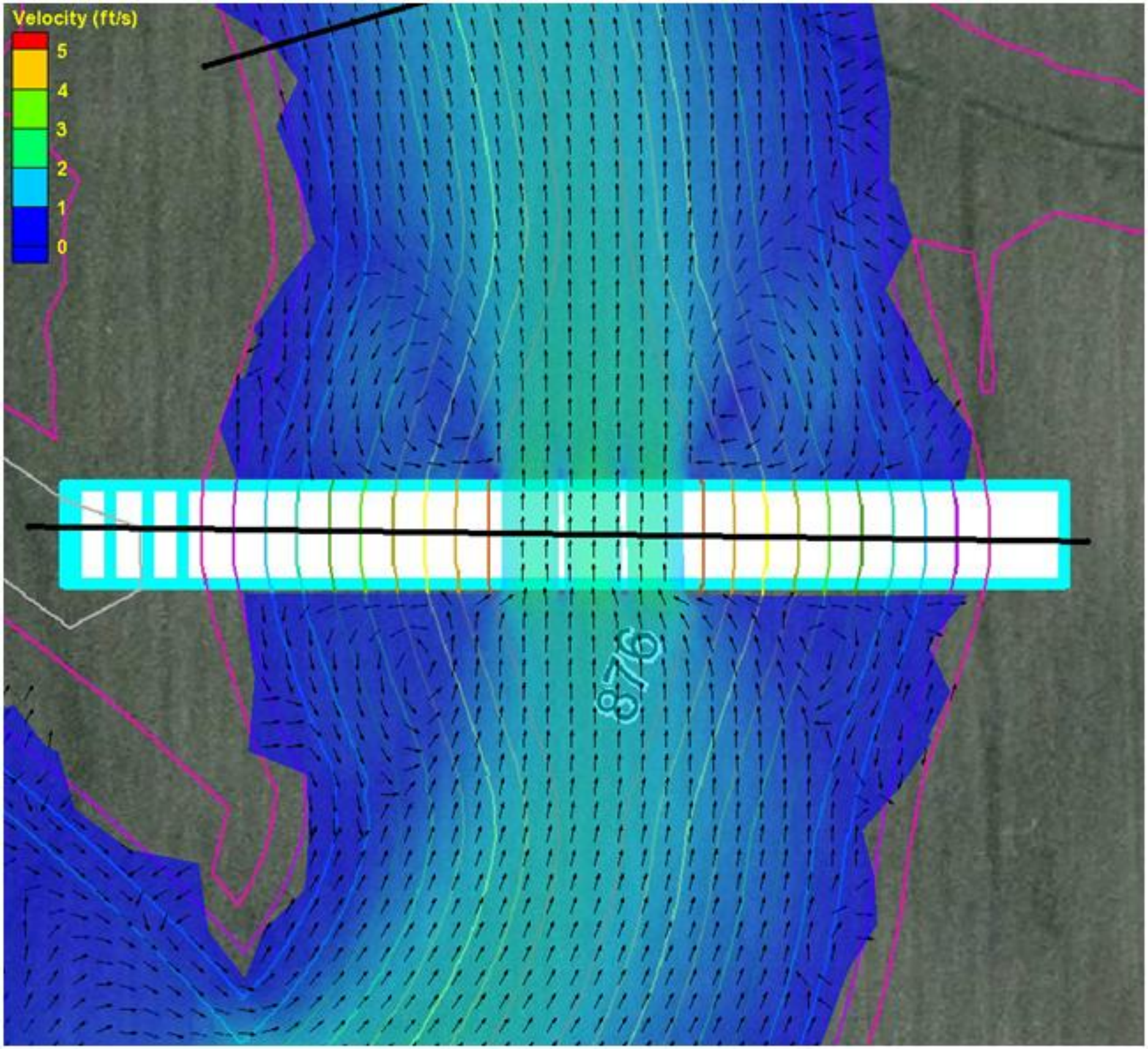

Figure F-H18 Velocity Field for North Dakota Control Structure Post-construction Conditions Close Up 6,100 cfs Event Flow

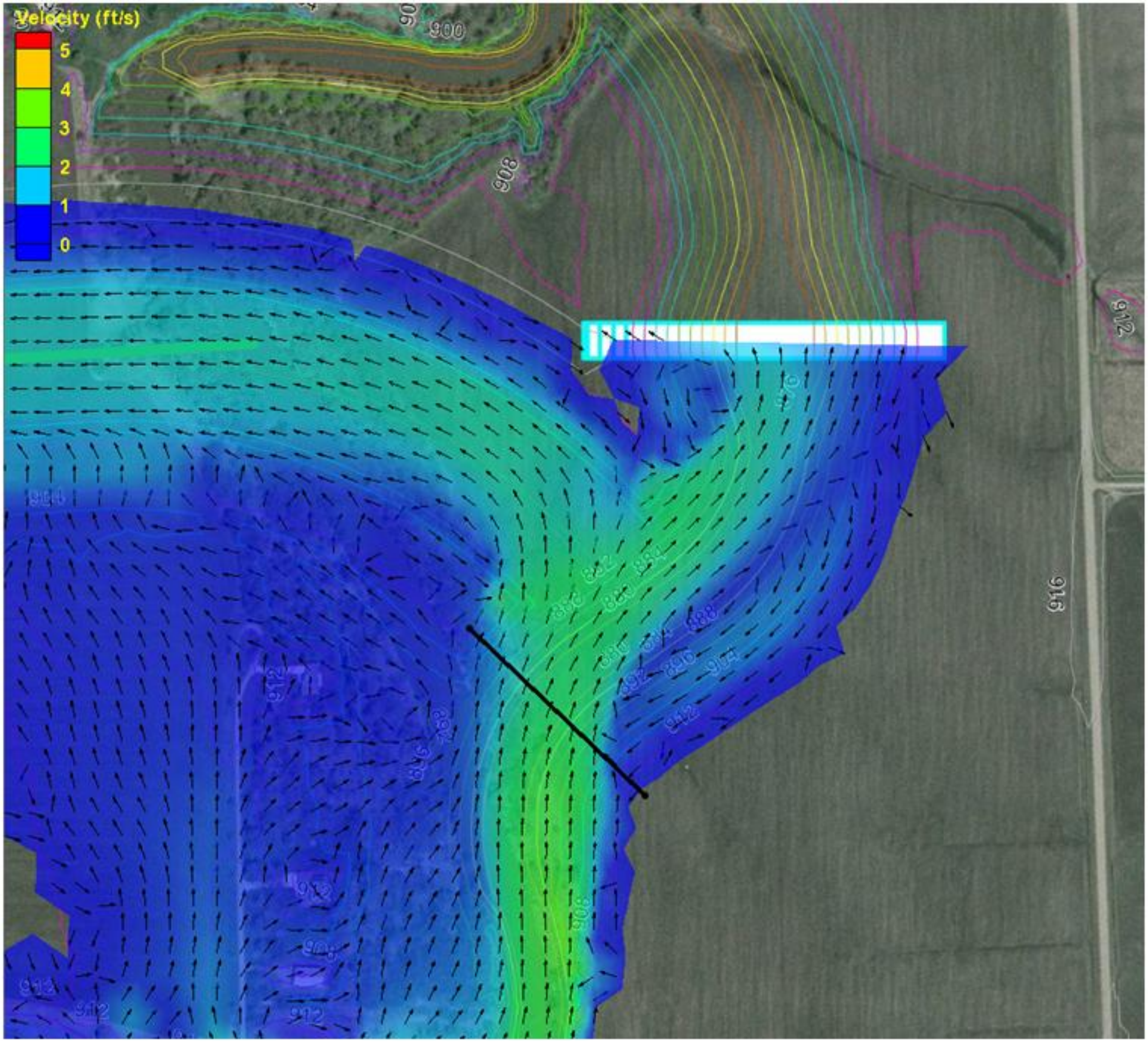

Figure F-H19 Velocity Field for North Dakota Control Structure Post-construction Conditions Upstream 20-Year Event Flow

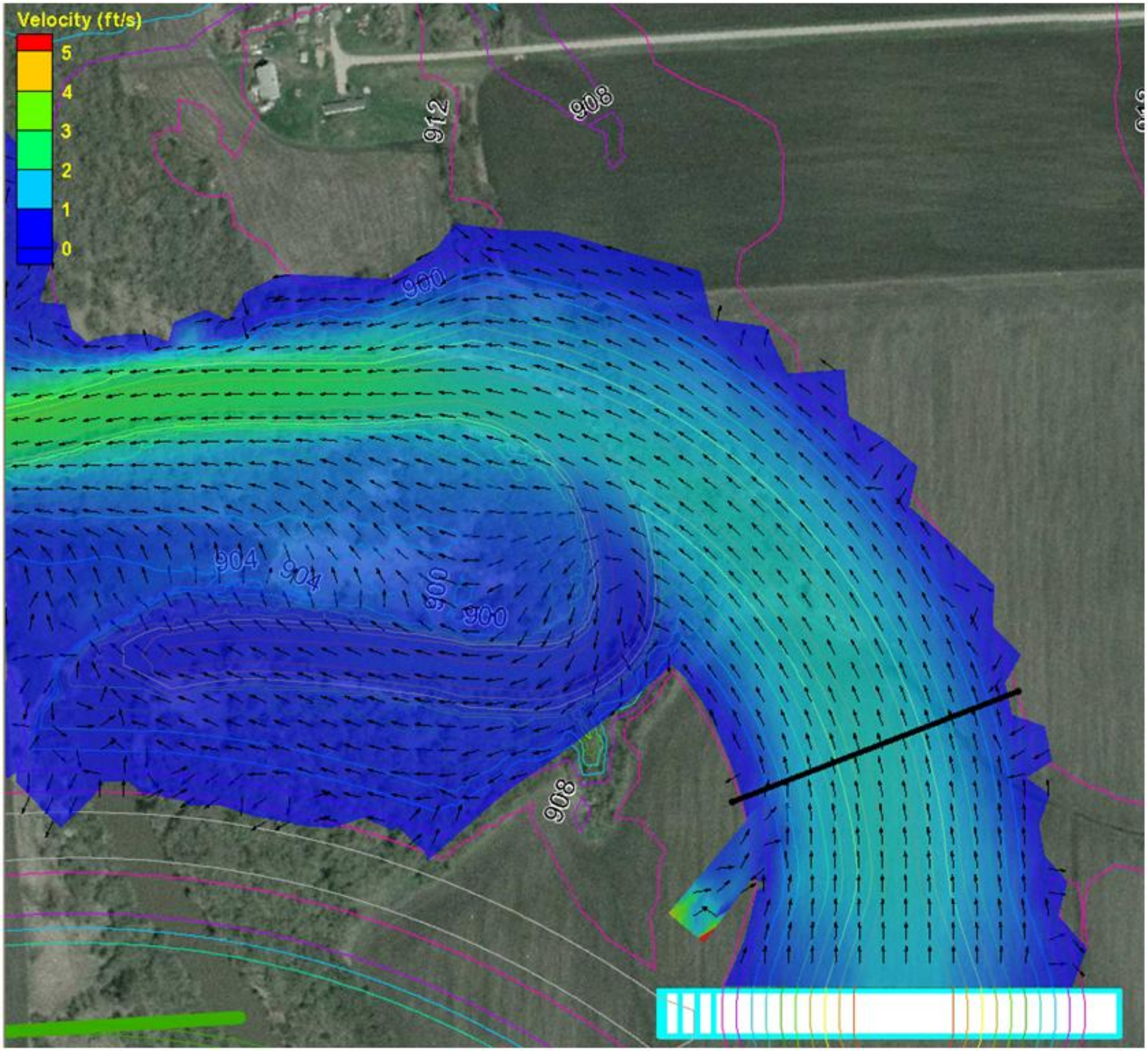

Figure F-H20 Velocity Field for North Dakota Control Structure Post-construction Conditions Downstream 20-Year Event Flow

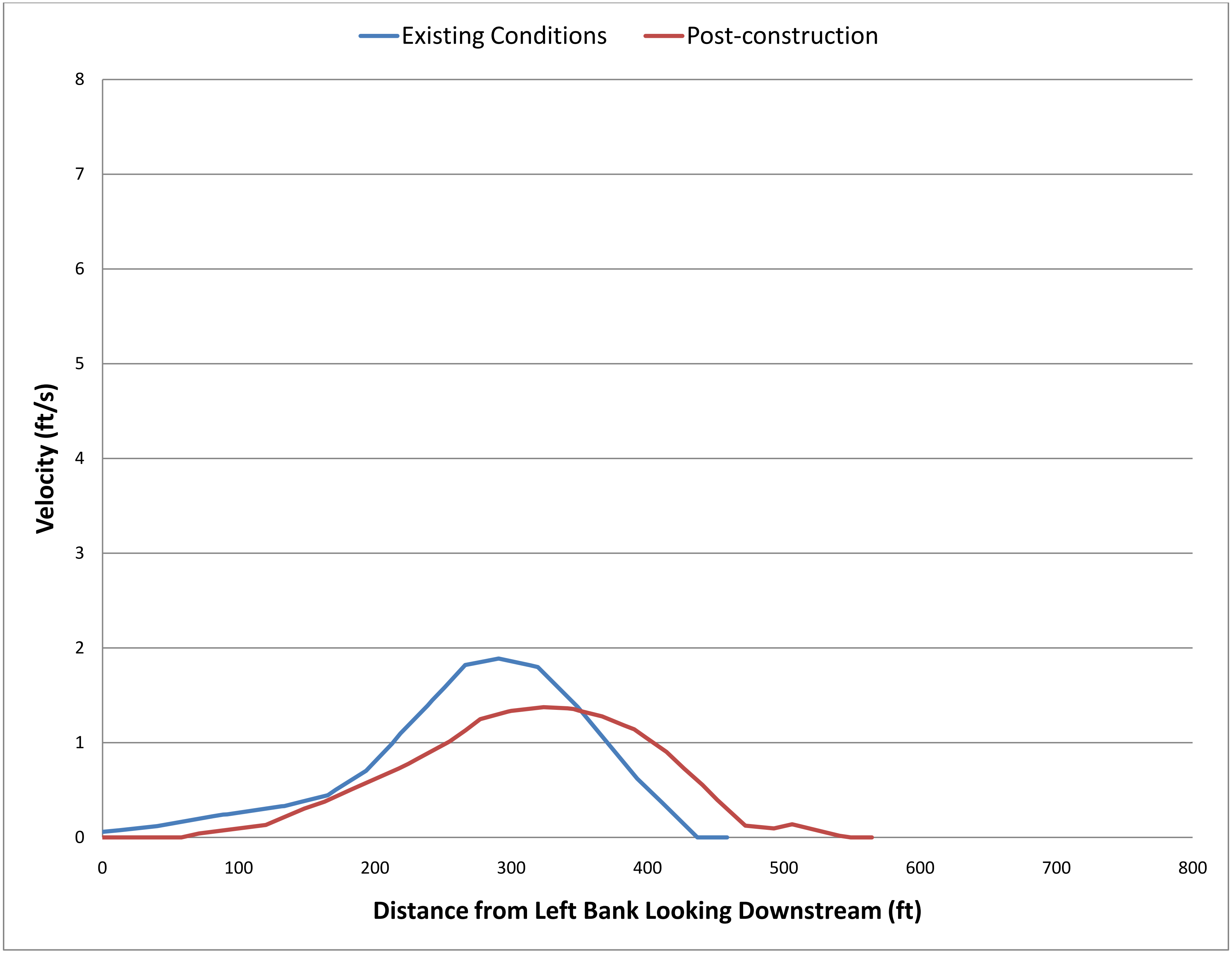

Figure F-H21 Cross Sectional Velocity Data for North Dakota Control Structure Upstream During 6,100 cfs Event

Cross Sectional Velocity Data for North Dakota Control Structure (6,100 cfs). Compares velocity at existing conditions to post-construction upstream of structure.

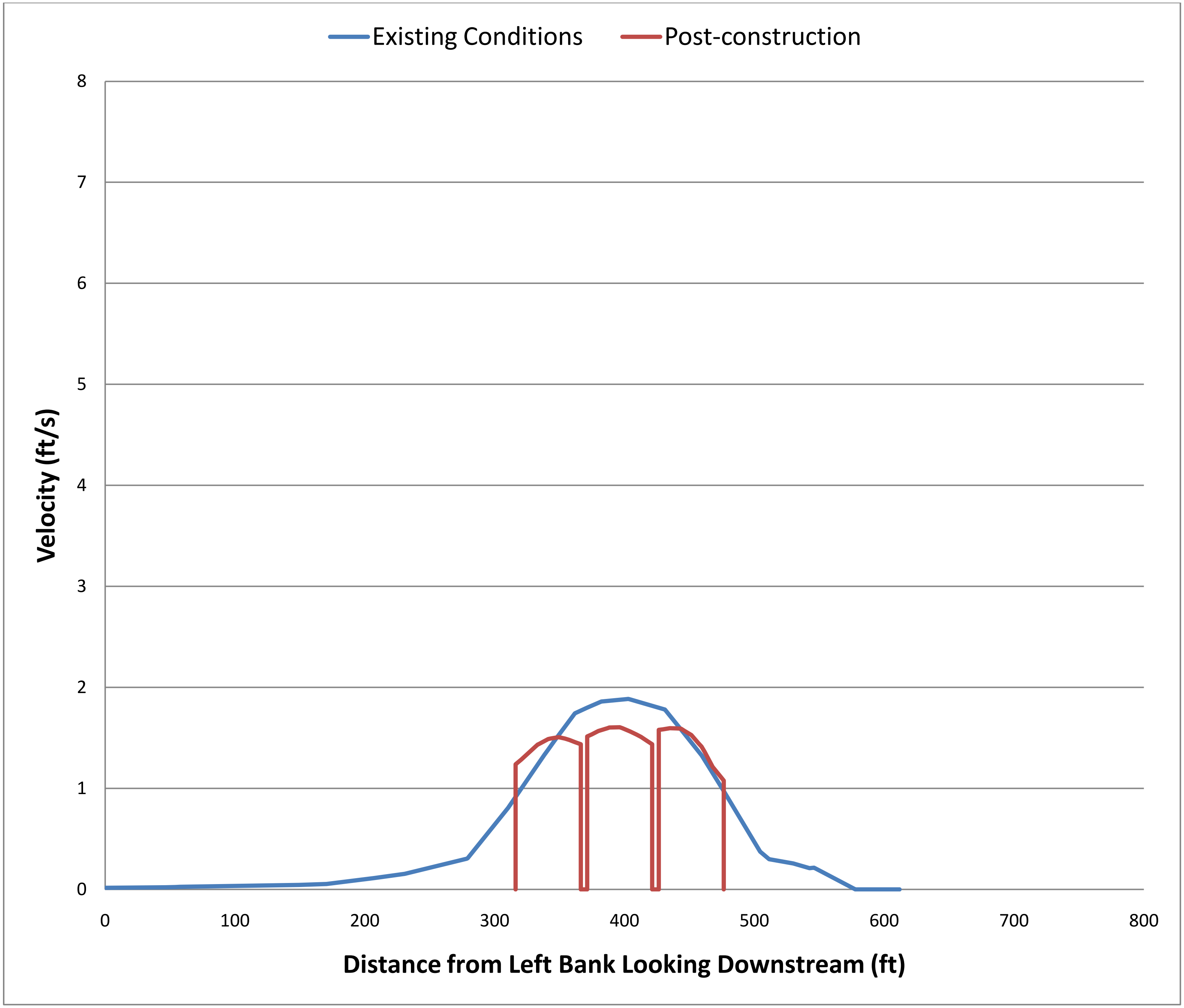

Figure F-H22 Cross Sectional Velocity Data for North Dakota Control Structure Through Structure During 6,100 cfs Event

Cross Sectional Velocity Data for North Dakota Control Structure (6,100 cfs). Compares velocity at existing conditions to post-construction through structure.

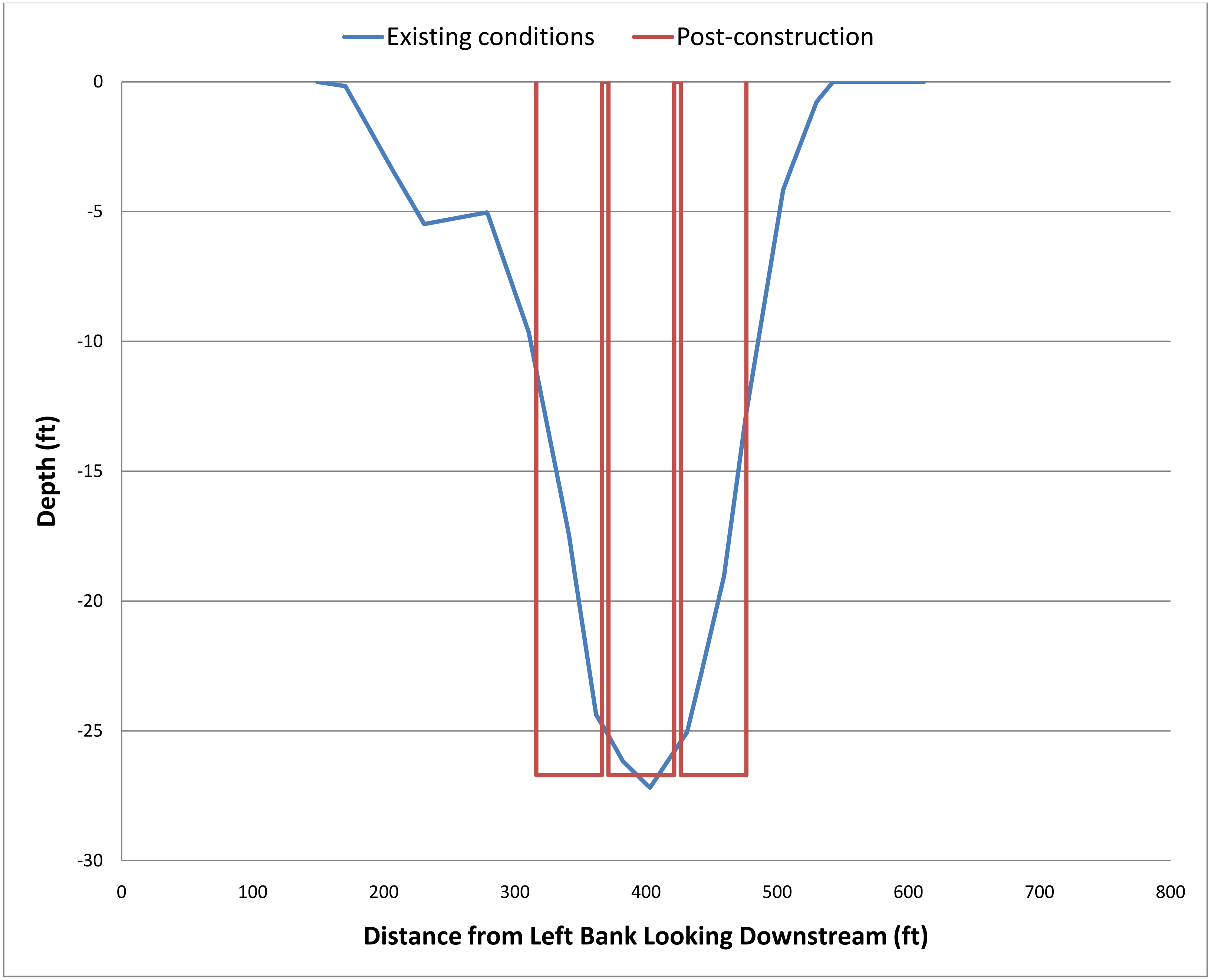

Figure F-H23 Cross Sectional Depth Data for North Dakota Control Structure Through Structure During 6,100 cfs Event

Cross Sectional Depth Data for North Dakota Control Structure (6,100 cfs). Compares depth at existing conditions to post-construction through structure.

Figure F-H24 Cross Sectional Velocity Data for North Dakota Control Structure Downstream During 6,100 cfs Event

Cross Sectional Velocity for North Dakota Control Structure (6,100 cfs) Compares velocity at existing conditions to post-construction downstream of structure.

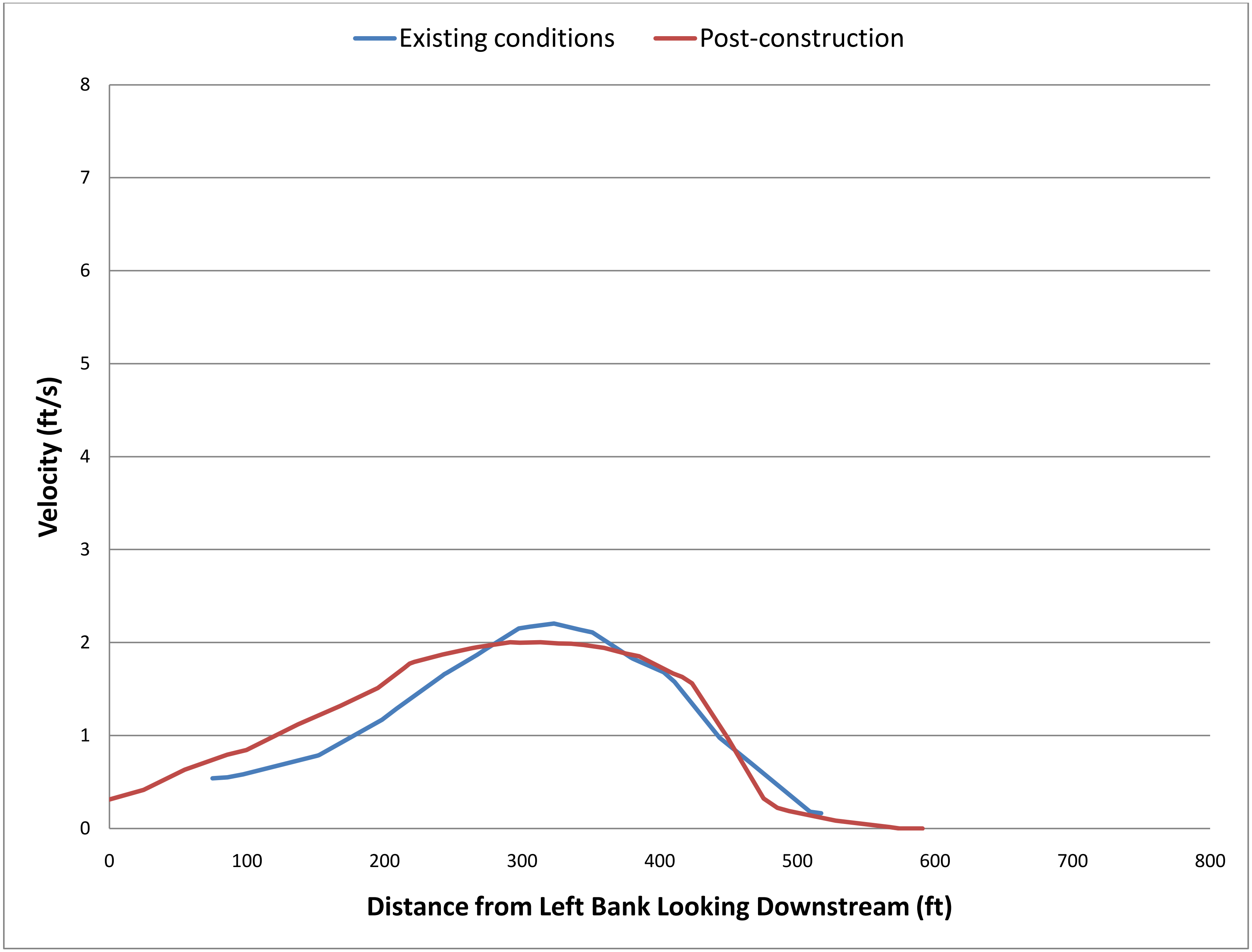

Figure F-H25 Cross Sectional Velocity Data for North Dakota Control Structure Upstream During 20-Year Event

Cross Sectional Velocity for North Dakota Control Structure (20-Year cfs) Compares velocity at existing conditions to post-construction upstream of structure.

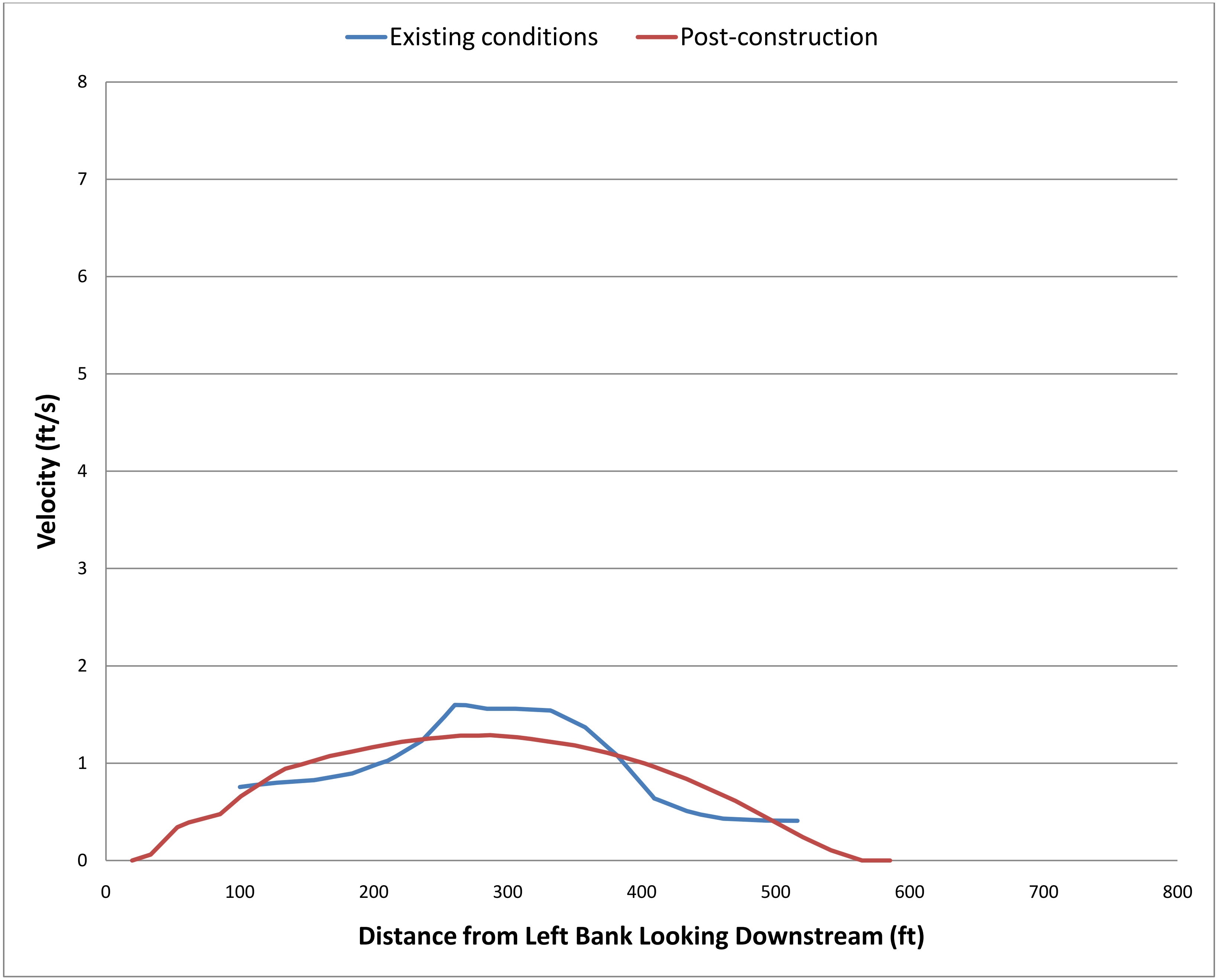

Figure F-H26 Cross Sectional Velocity Data for North Dakota Control Structure Downstream During 20-Year Event

Cross Sectional Velocity for North Dakota Control Structure (20-Year) Compares velocity at existing conditions to post-construction downstream of structure.

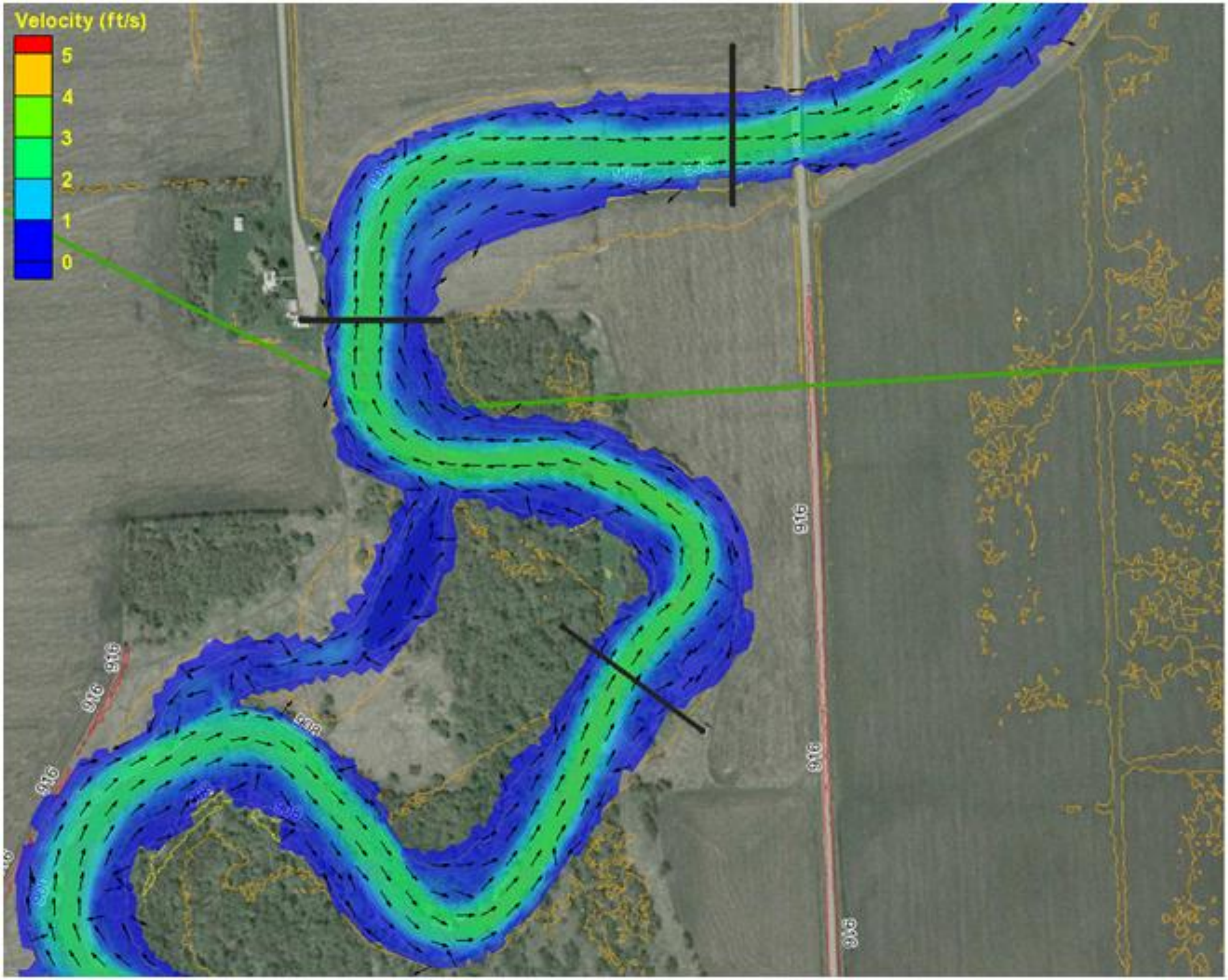

Figure F-H27 Velocity Field for Wild Rice River Control Structure Existing Conditions 5-Year Event Flow

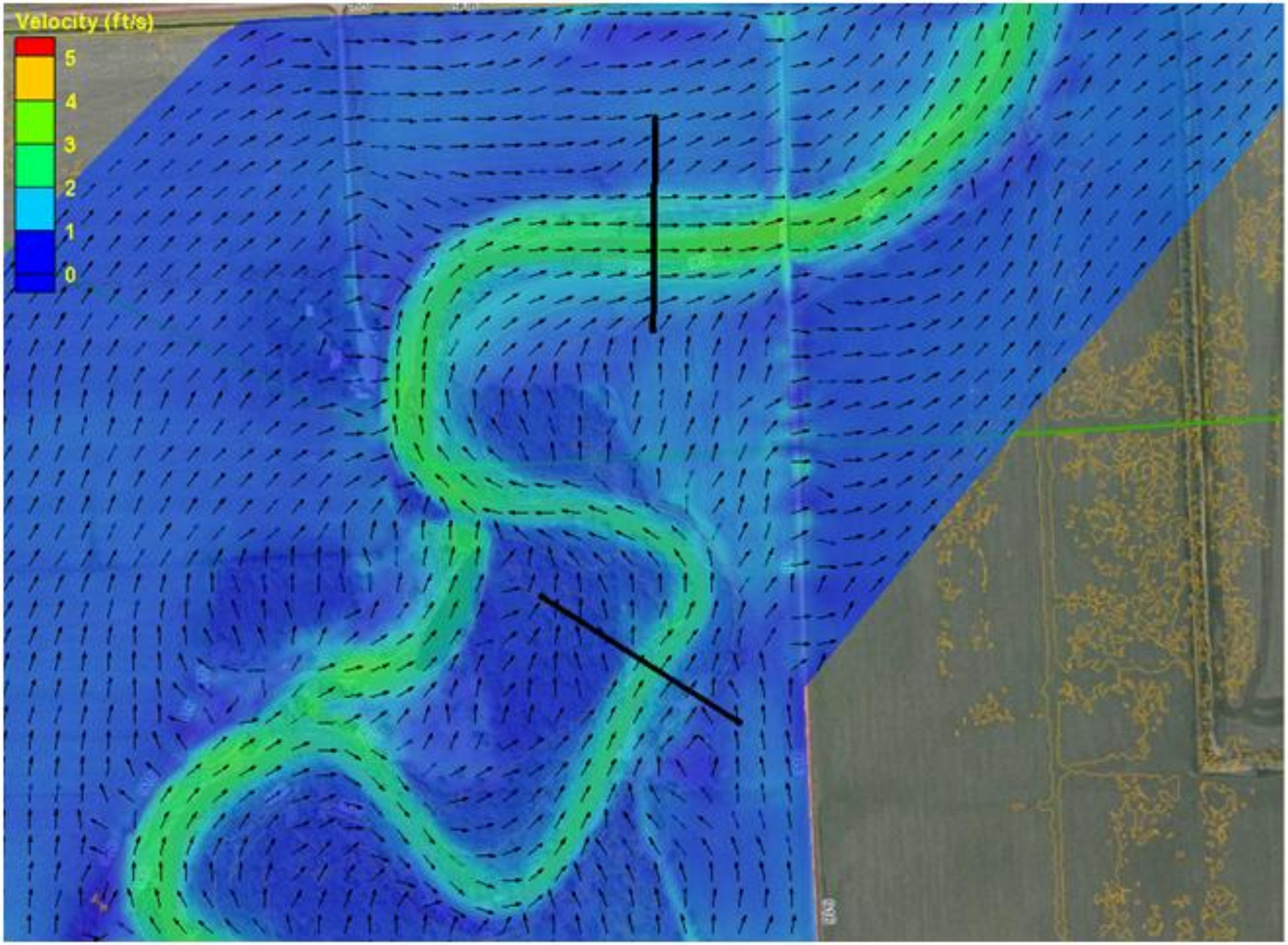

Figure F-H28 Velocity Field for Wild Rice River Existing Conditions 20-Year Event Flow

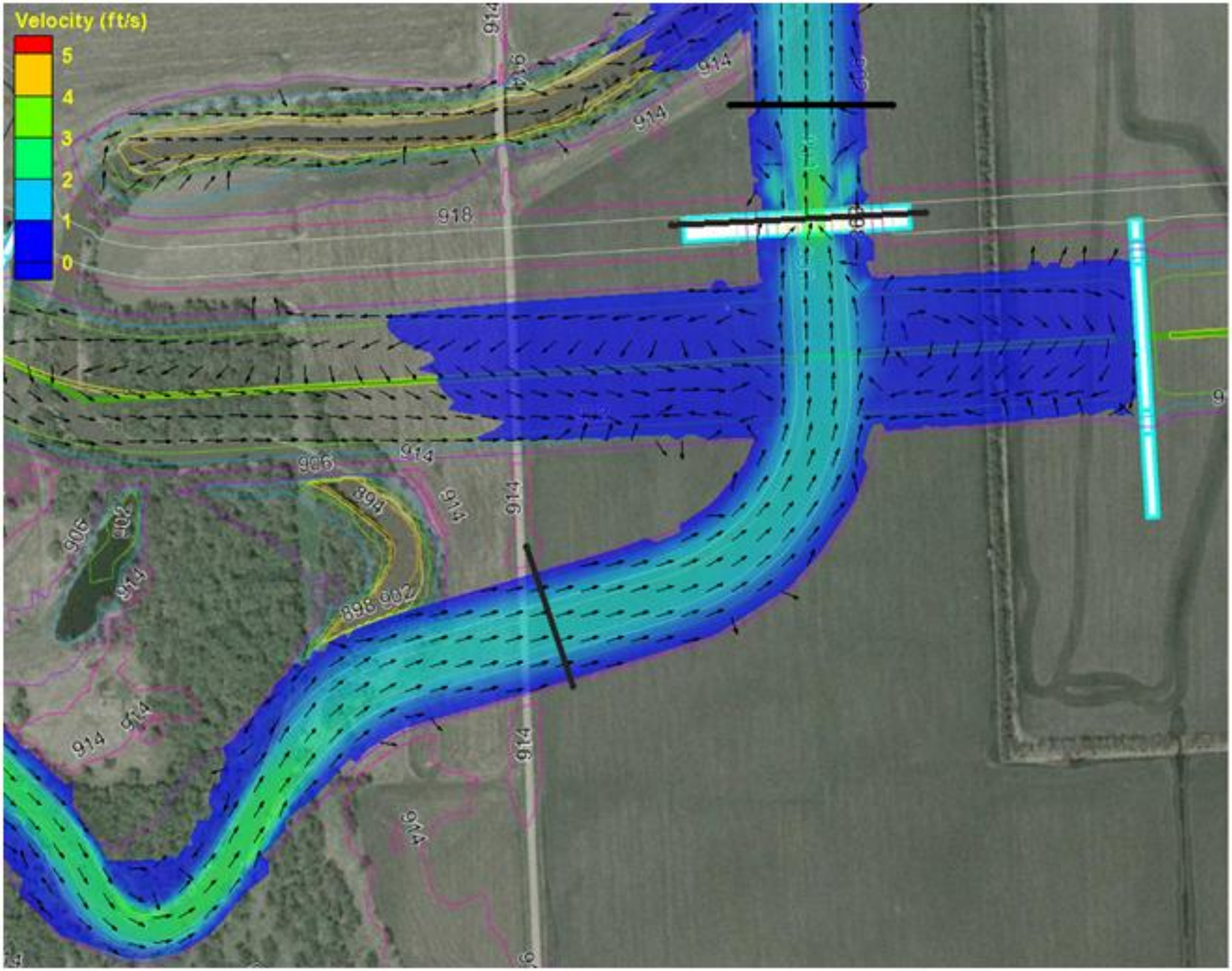

Figure F-H29 Velocity Field for Wild Rice River Post-construction Conditions 5-Year Event Flow

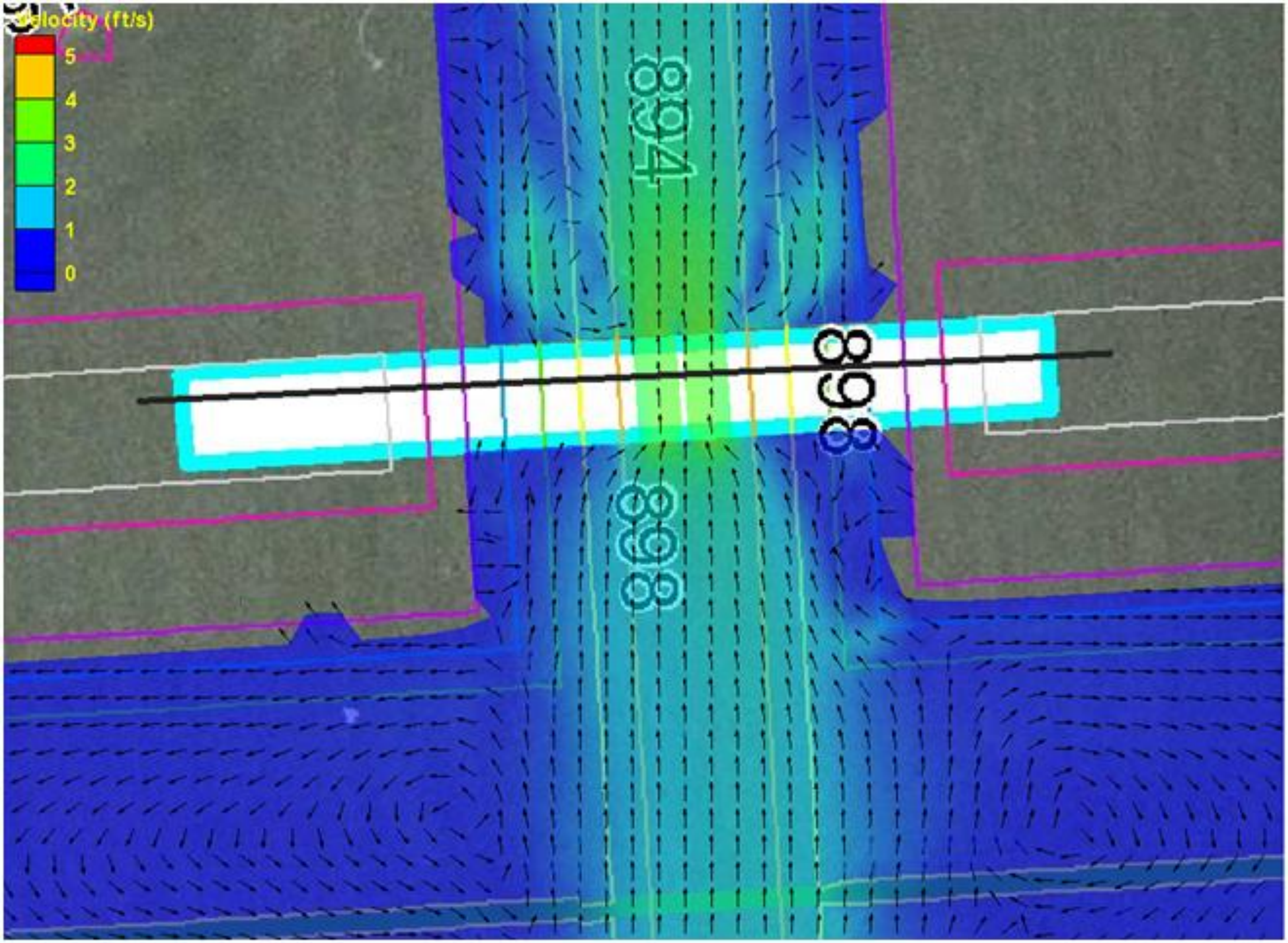

Figure F-H30 Velocity Field for Wild Rice River Post-construction Conditions Close Up 5-Year Event Flow

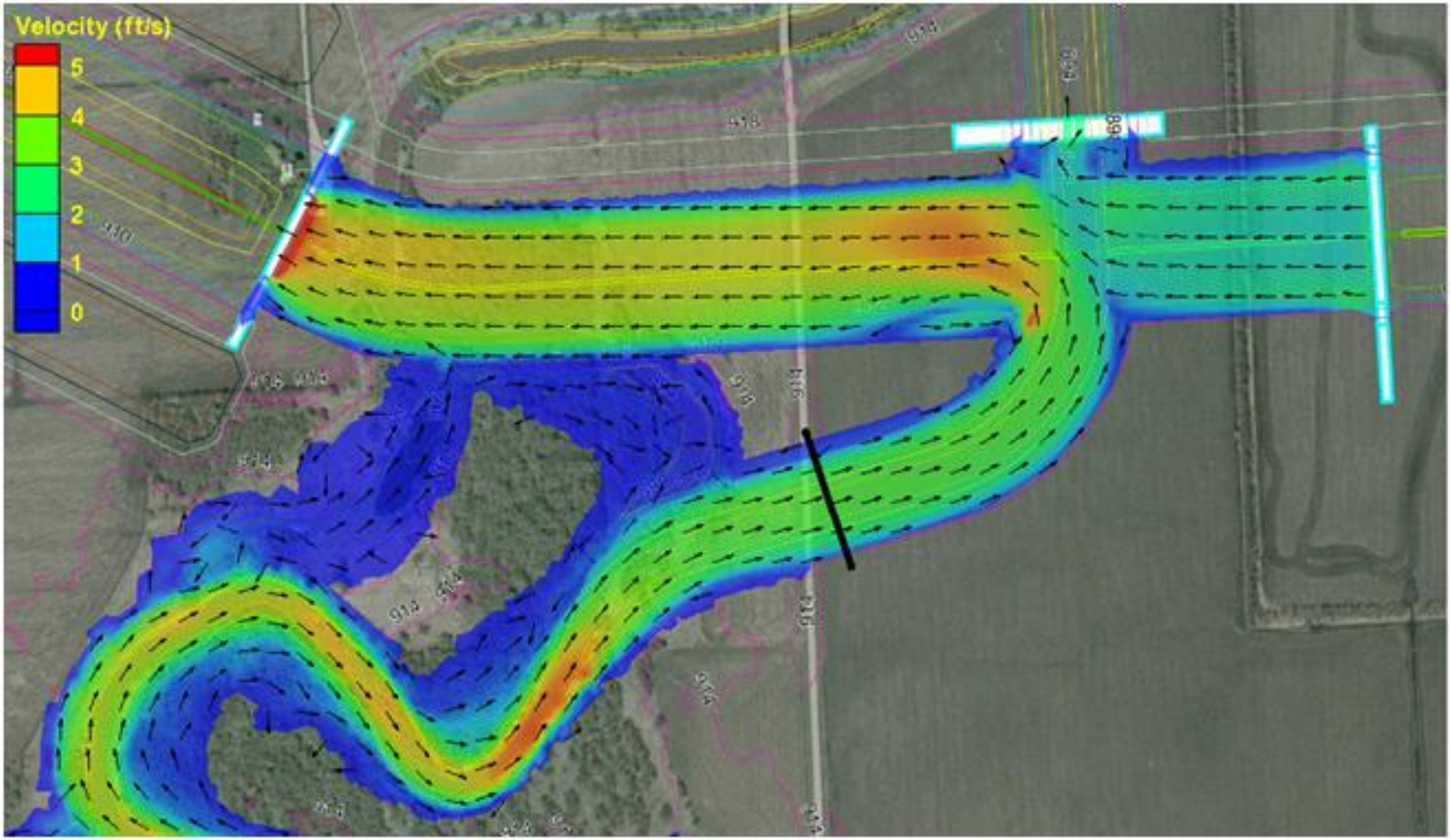

Figure F-H31 Velocity Field for Wild Rice River Post-construction Conditions Upstream 20-Year Event Flow

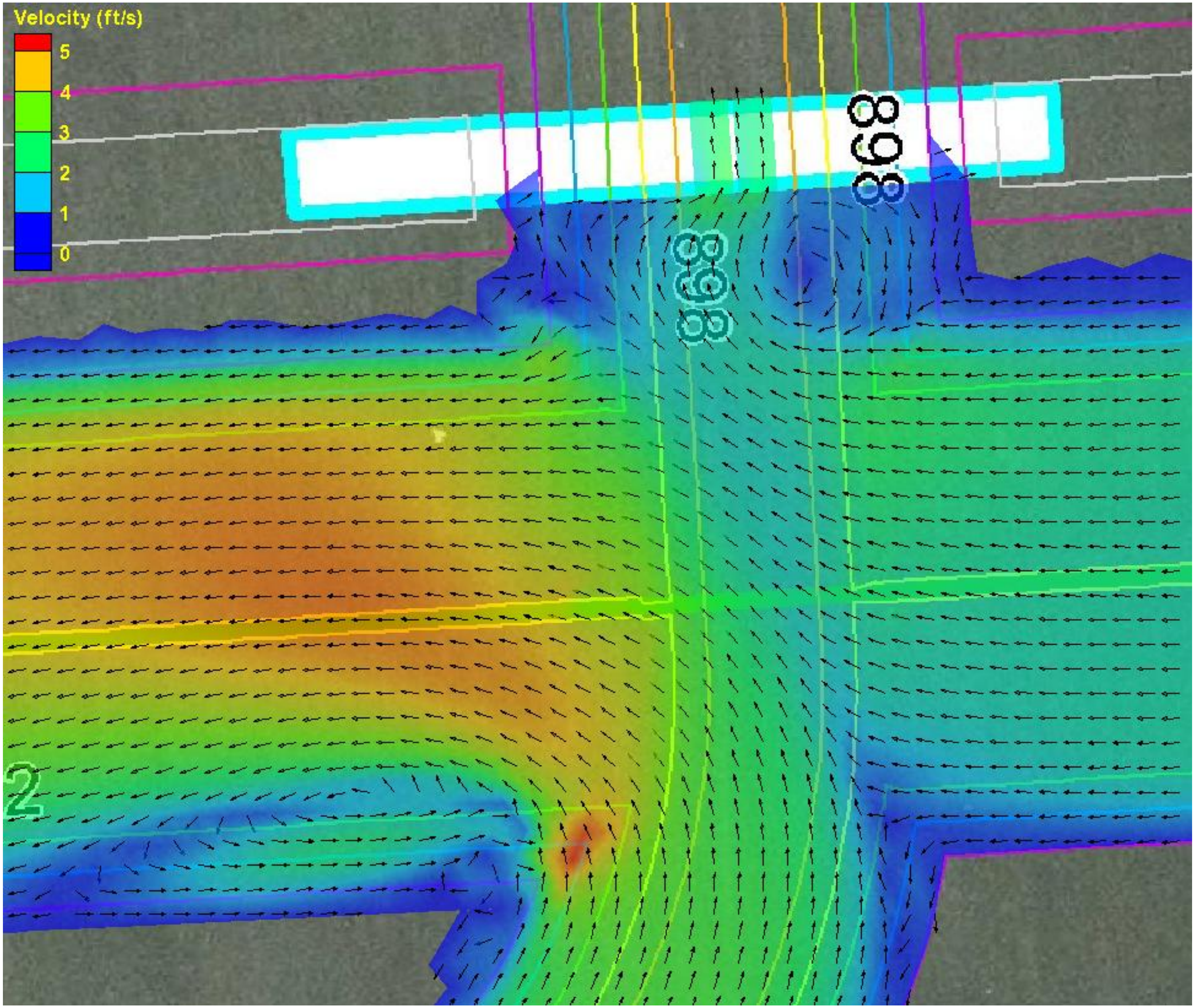

Figure F-H32 Velocity Field for Wild Rice River Post-construction Conditions Upstream Close Up 20-Year Event Flow

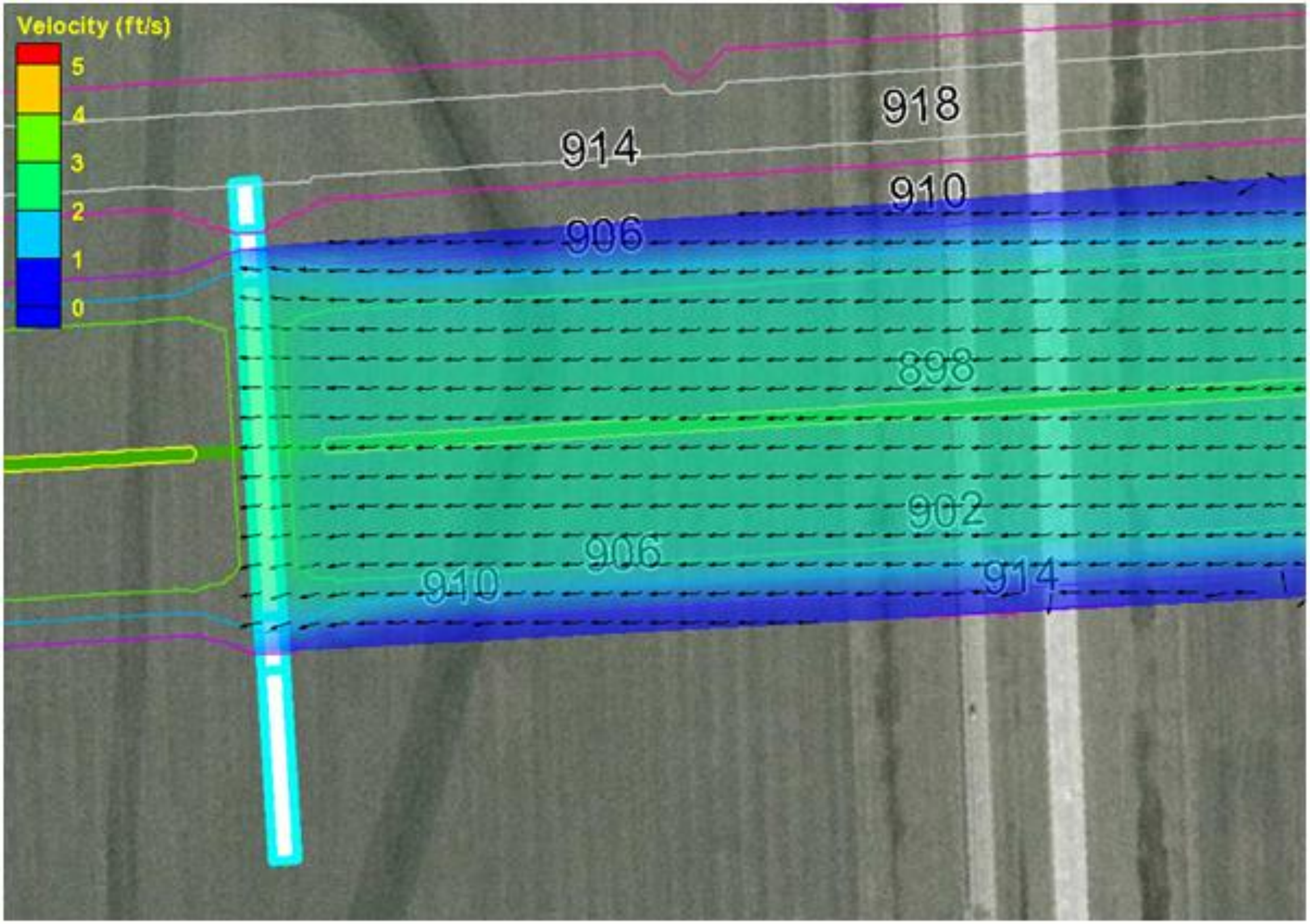

Figure F-H33 Velocity Field for Wild Rice River Post-construction Connecting Channel 20-Year Event Flow

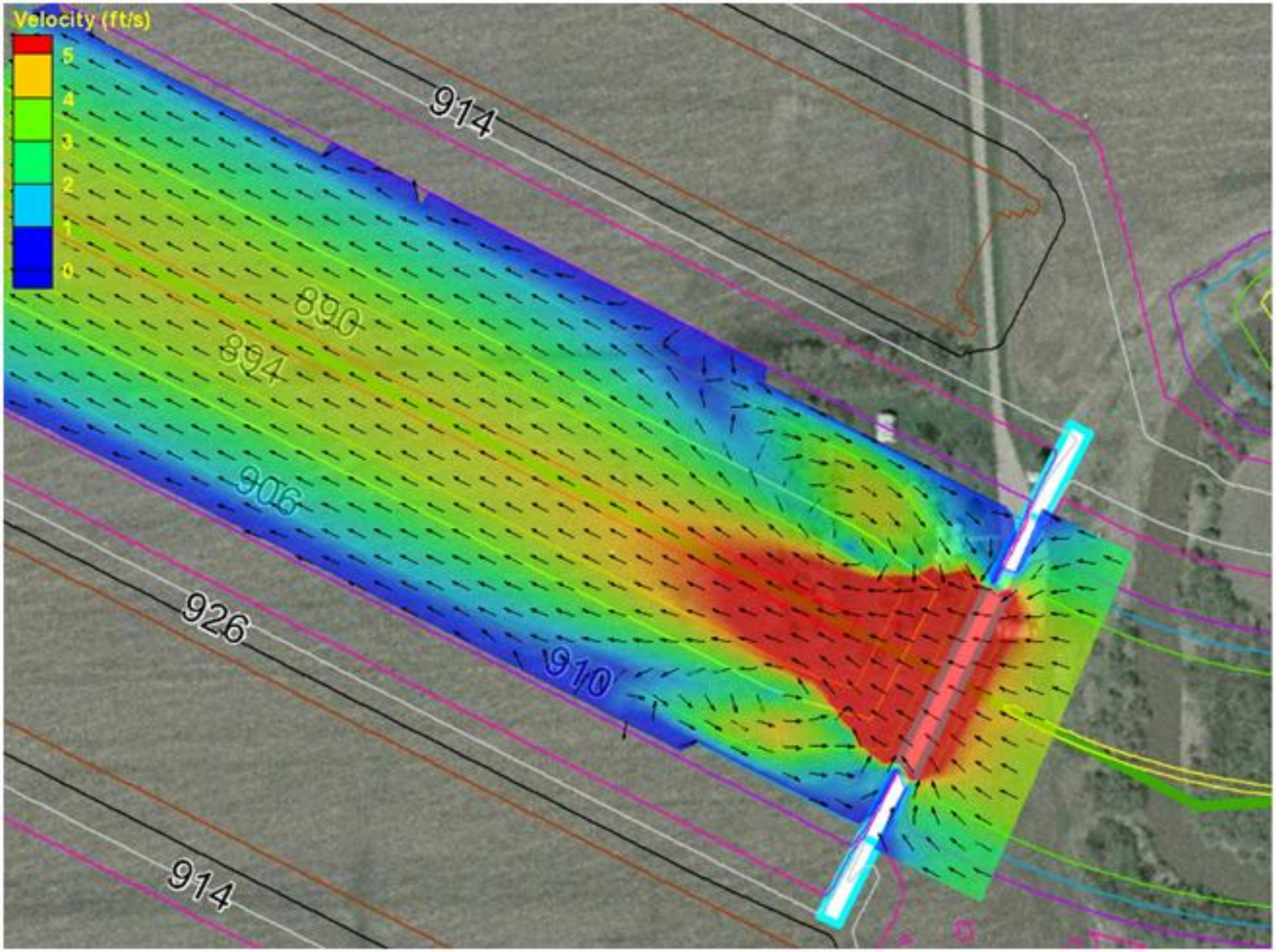

Figure F-H34 Velocity Field for Wild Rice River Post-construction Diversion Channel 20-Year Event Flow

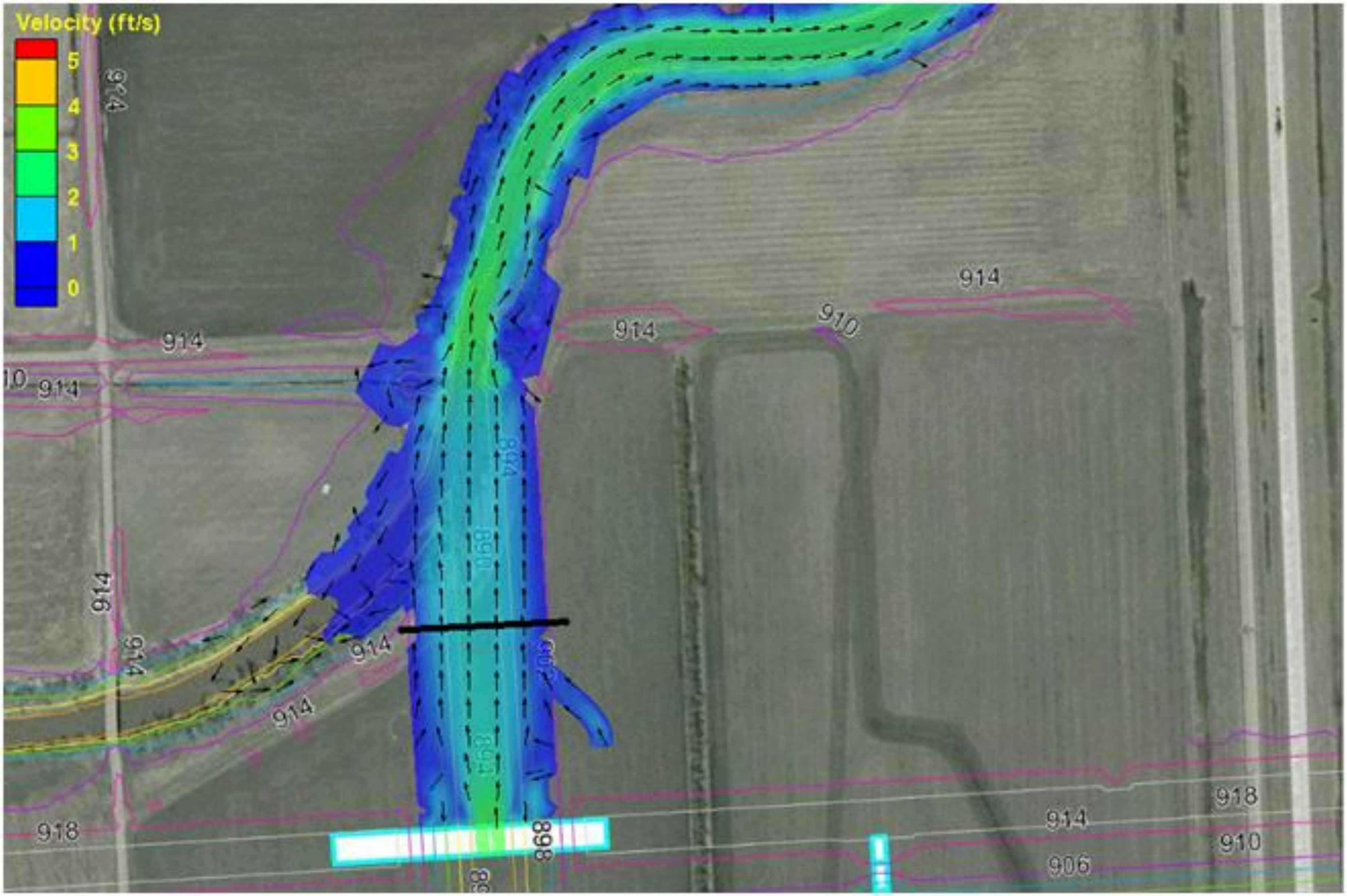

Figure F-H35 Velocity Field for Wild Rice River Post-construction Conditions Downstream 20-Year Event Flow

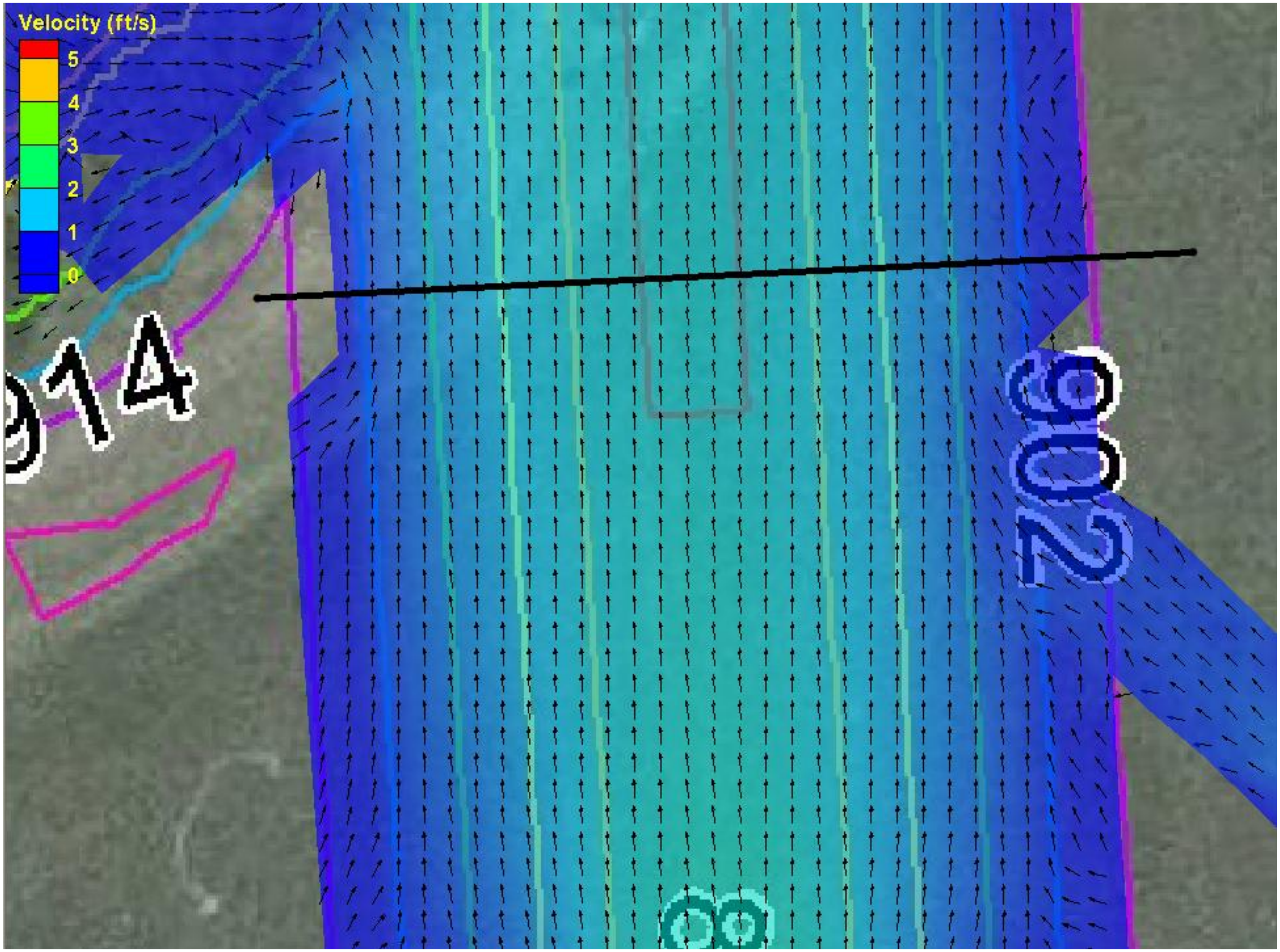

Figure F-H36 Velocity Field for Wild Rice River Post-construction Conditions Downstream Close Up 20-Year Event Flow

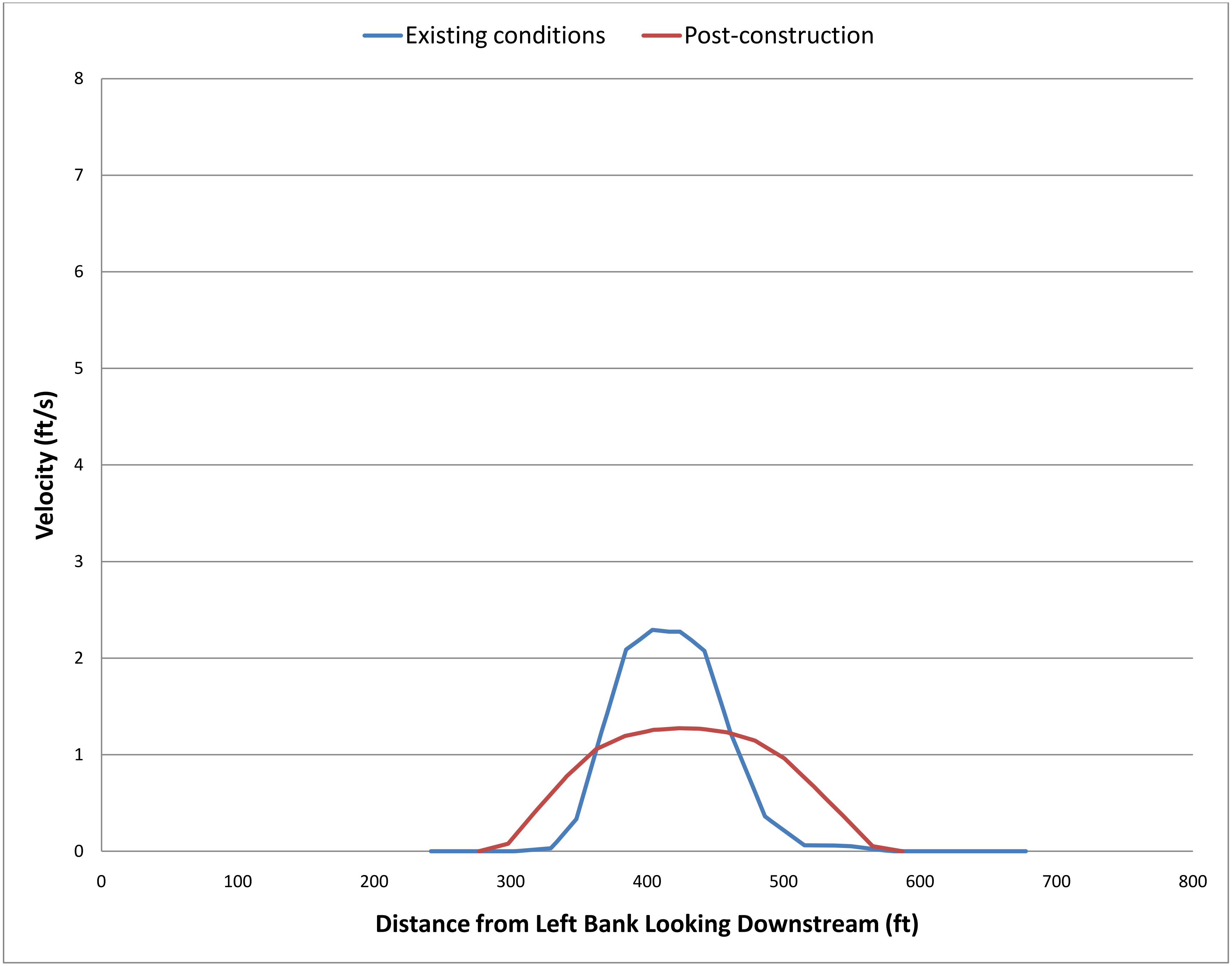

Figure F-H37 Cross Sectional Velocity Data for Wild Rice River Control Structure Upstream During 5-Year Event

Cross Sectional Velocity Data for Wild Rice Control Structure (5-Year) Compares velocity at existing conditions to post-construction upstream of structure.

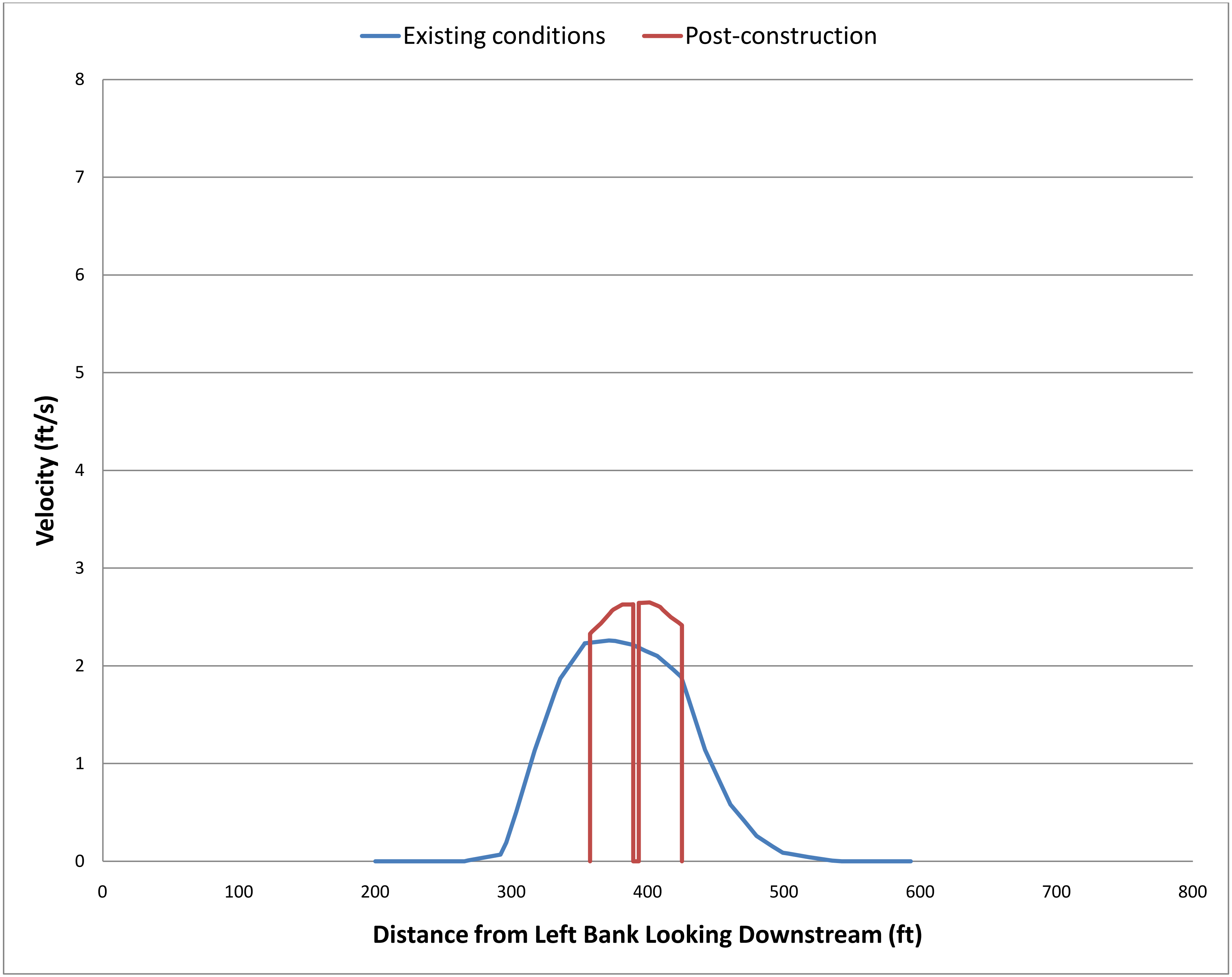

Figure F-H38 Cross Sectional Velocity Data for Wild Rice River Control Structure Through Structure During 5-Year Event

Cross Sectional Velocity Data for Wild Rice Control Structure (5-Year). Compares velocity at existing conditions to post-construction through structure.

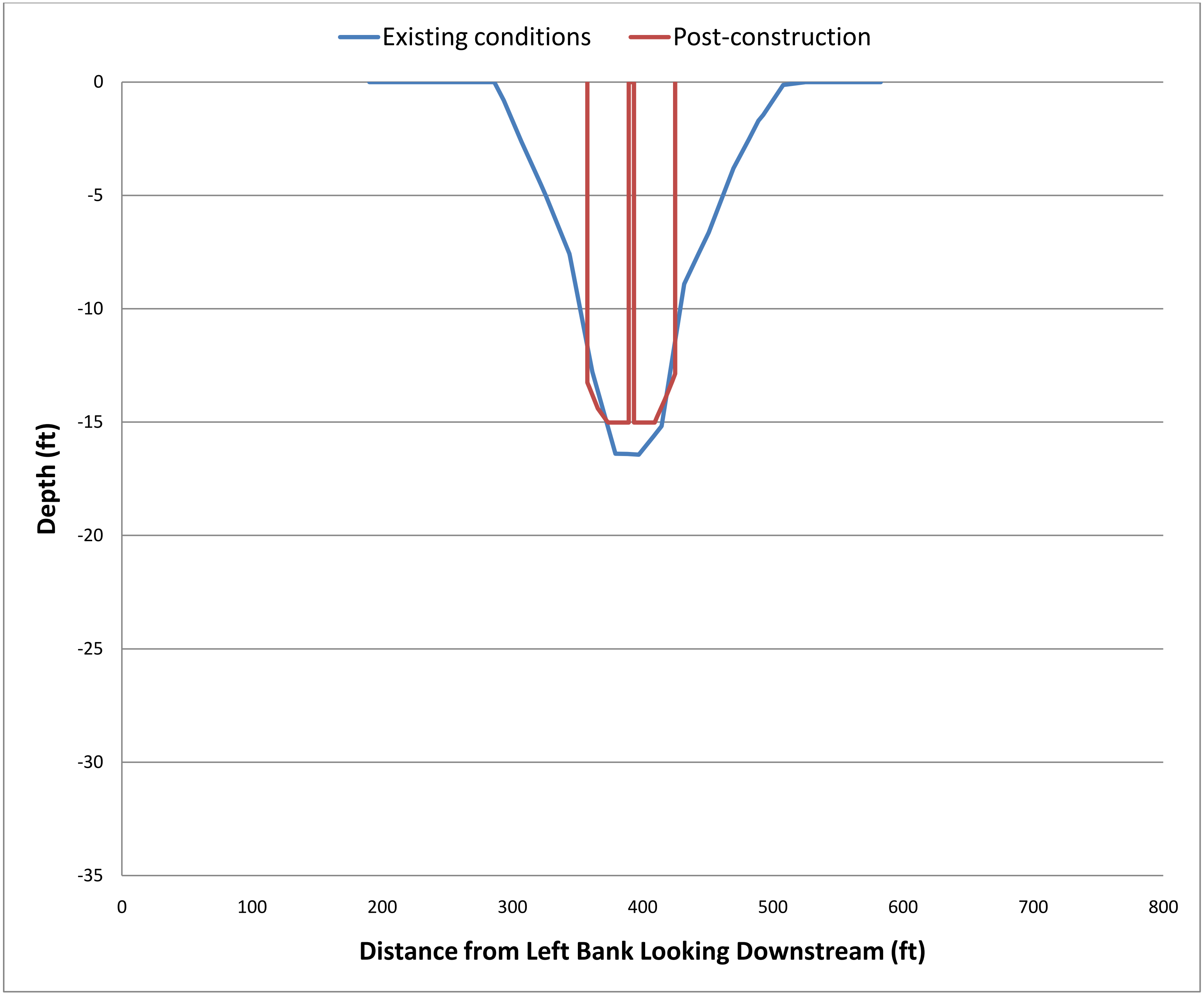

Figure F-H39 Cross Sectional Depth Data for Wild Rice River Control Structure Downstream During 5-Year Event

Cross Sectional Depth Data for Wild Rice Control Structure (5-Year). Compares depth at existing conditions to post-construction downstream of structure.

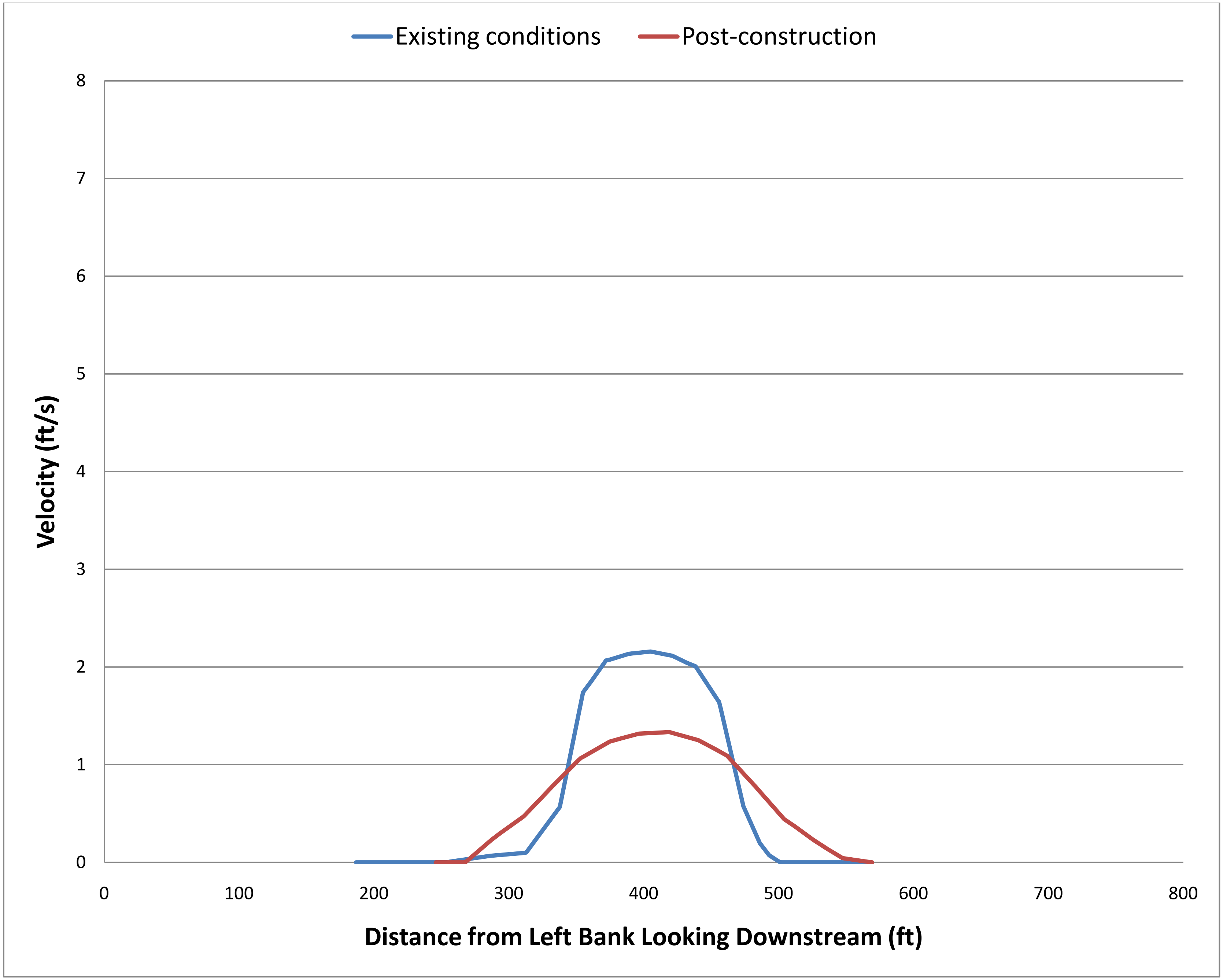

Figure F-H40 Cross Sectional Velocity Data for Wild Rice River Control Structure Downstream During 5-Year Event

Cross Sectional Velocity Data for Wild Rice Control Structure (5-Year). Compares velocity at existing conditions to post-construction downstream of structure.

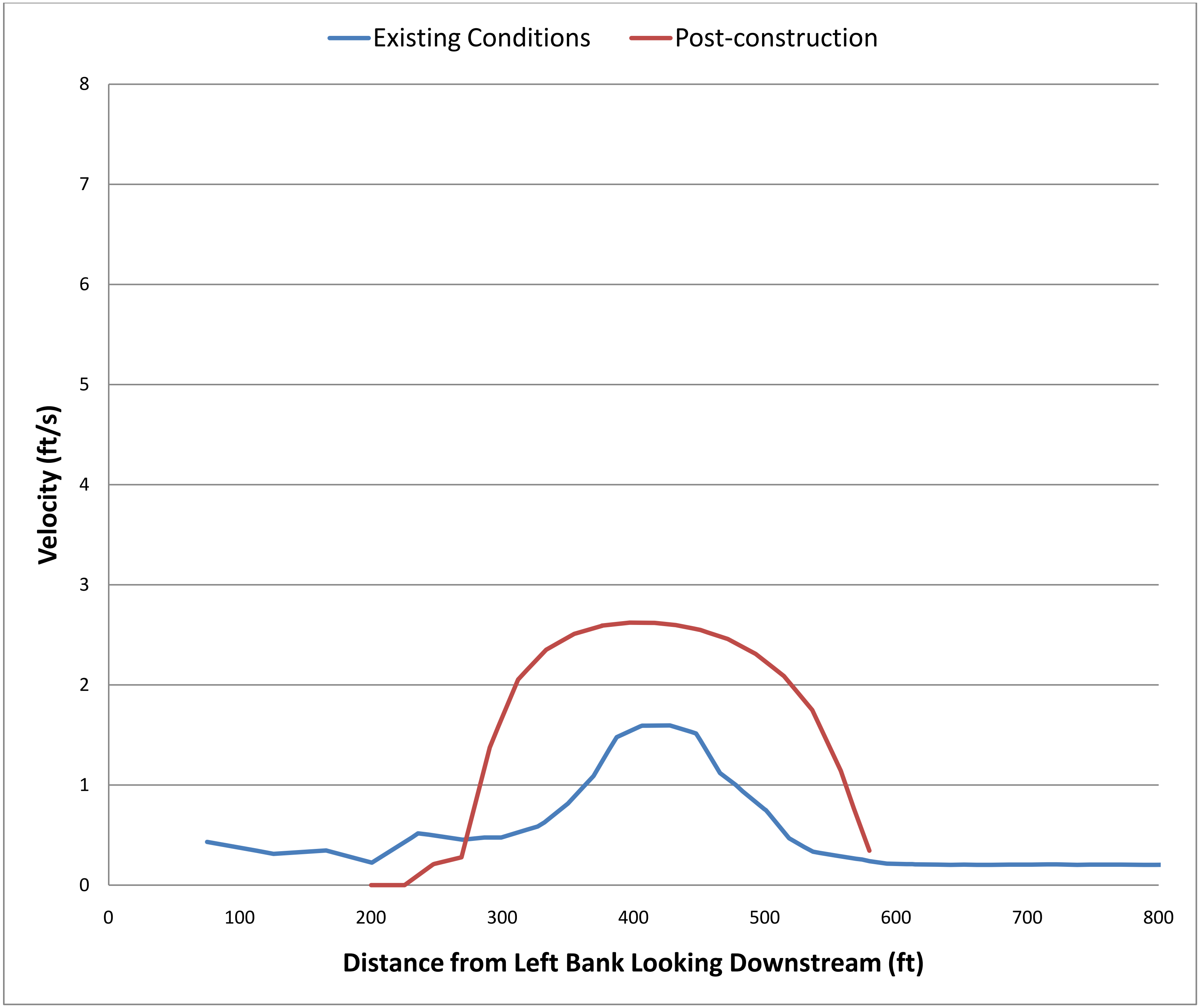

Figure F-H41 Cross Sectional Velocity Data for Wild Rice River Control Structure Upstream During 20-Year Event

Cross Sectional Velocity Data for Wild Rice Control Structure (20-Year). Compares velocity at existing conditions to post-construction upstream of structure.

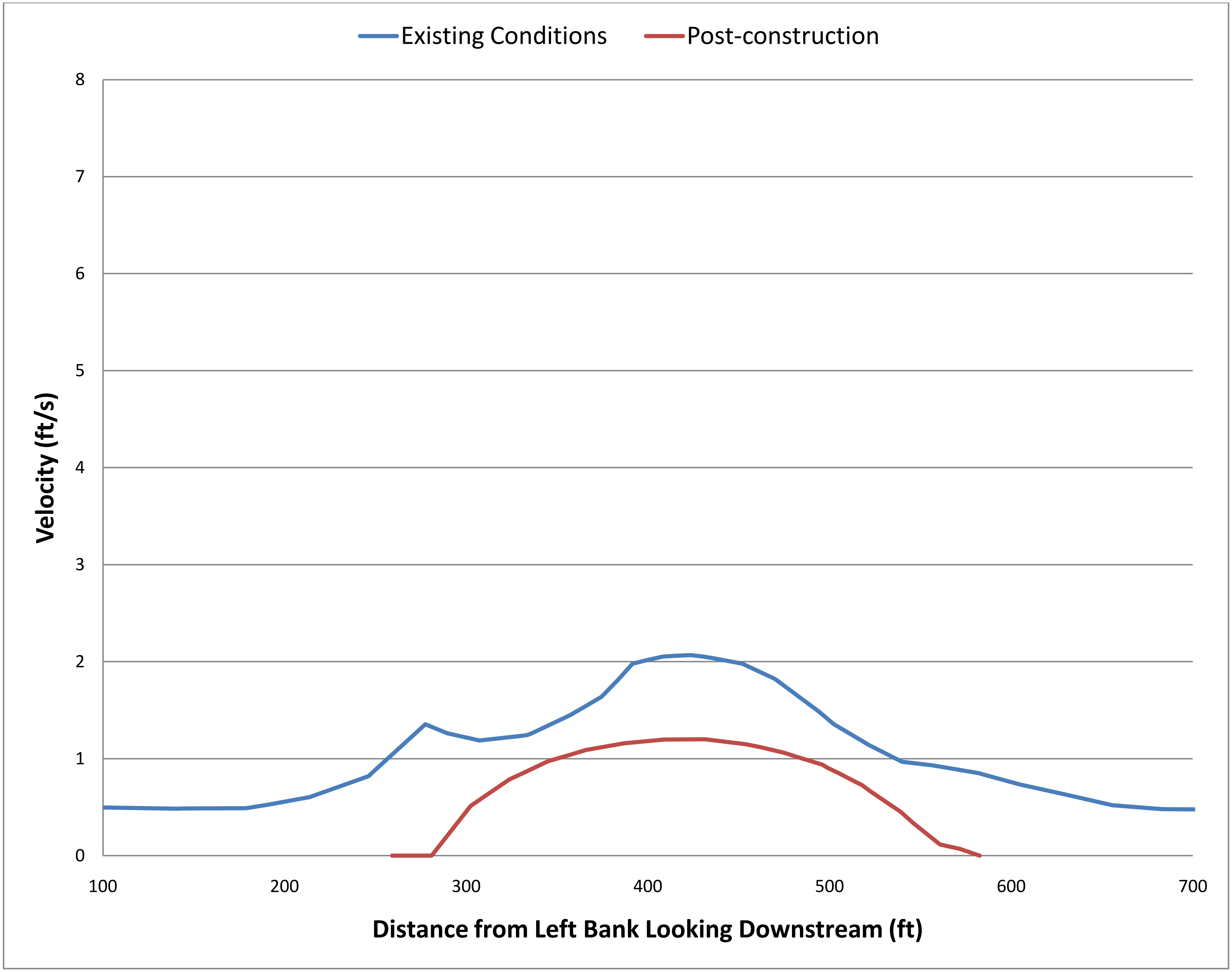

Figure F-H42 Cross Sectional Velocity Data for Wild Rice River Control Structure Downstream During 20-Year Event

Cross Sectional Velocity Data for Wild Rice Control Structure (20-Year). Compares velocity at existing conditions to post-construction downstream of structure.

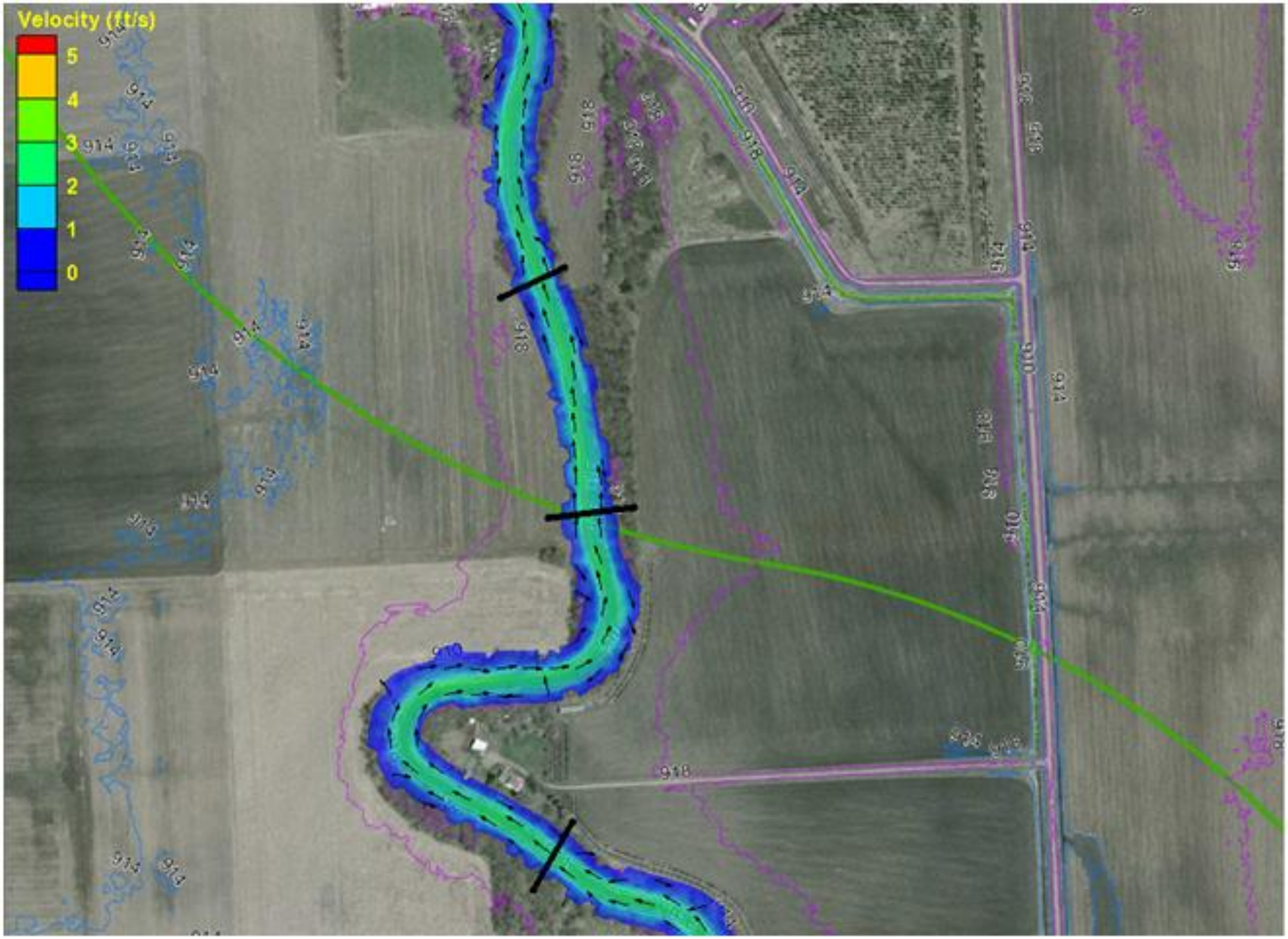

Figure F-H43 Velocity Field for Sheyenne River Existing Conditions 2-Year Event Flow

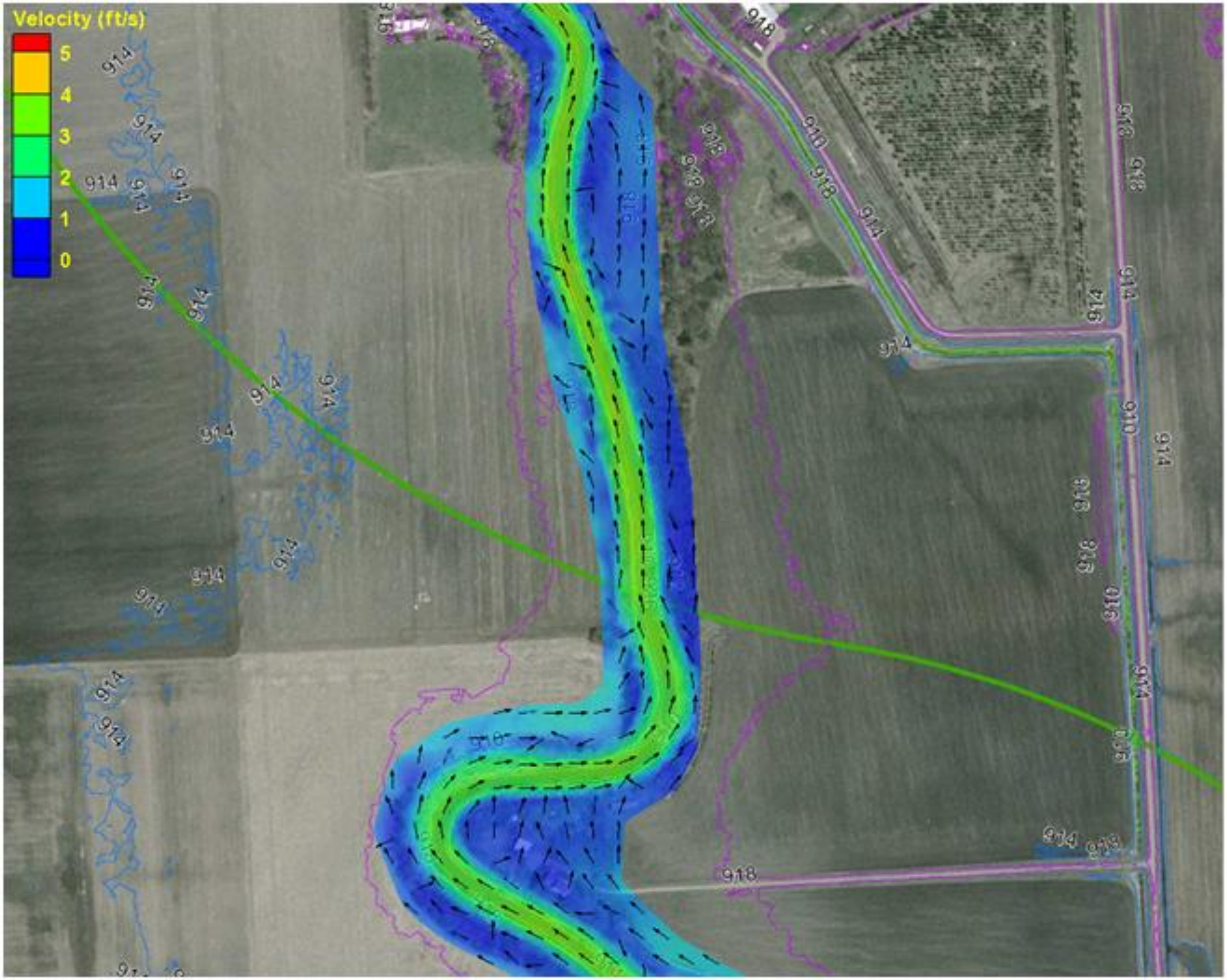

Figure F-H44 Velocity Field for Sheyenne River Existing Conditions 50-Year Event Flow

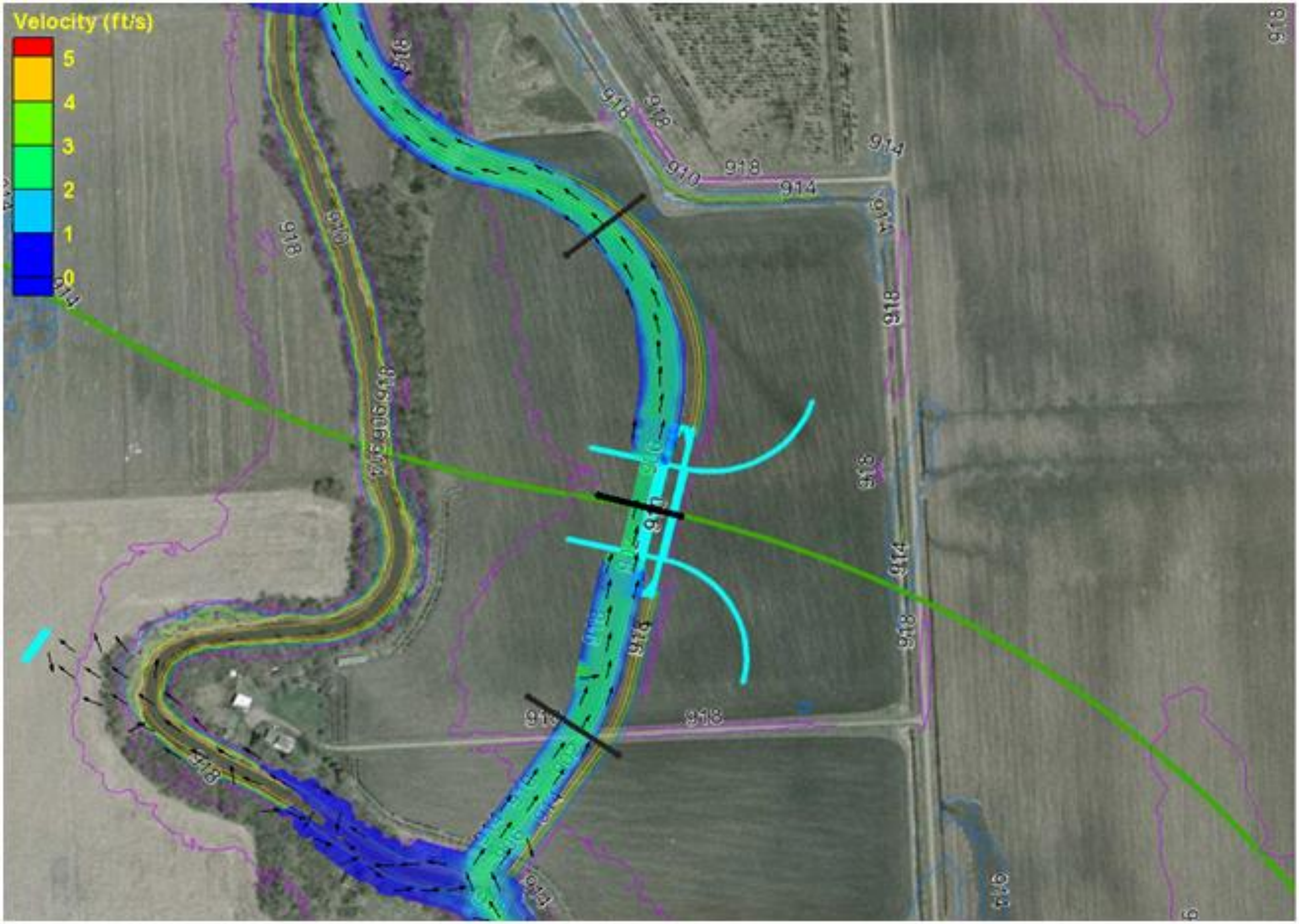

Figure F-H45 Velocity Field for Sheyenne River Aqueduct Post-construction Conditions 2-Year Event Flow

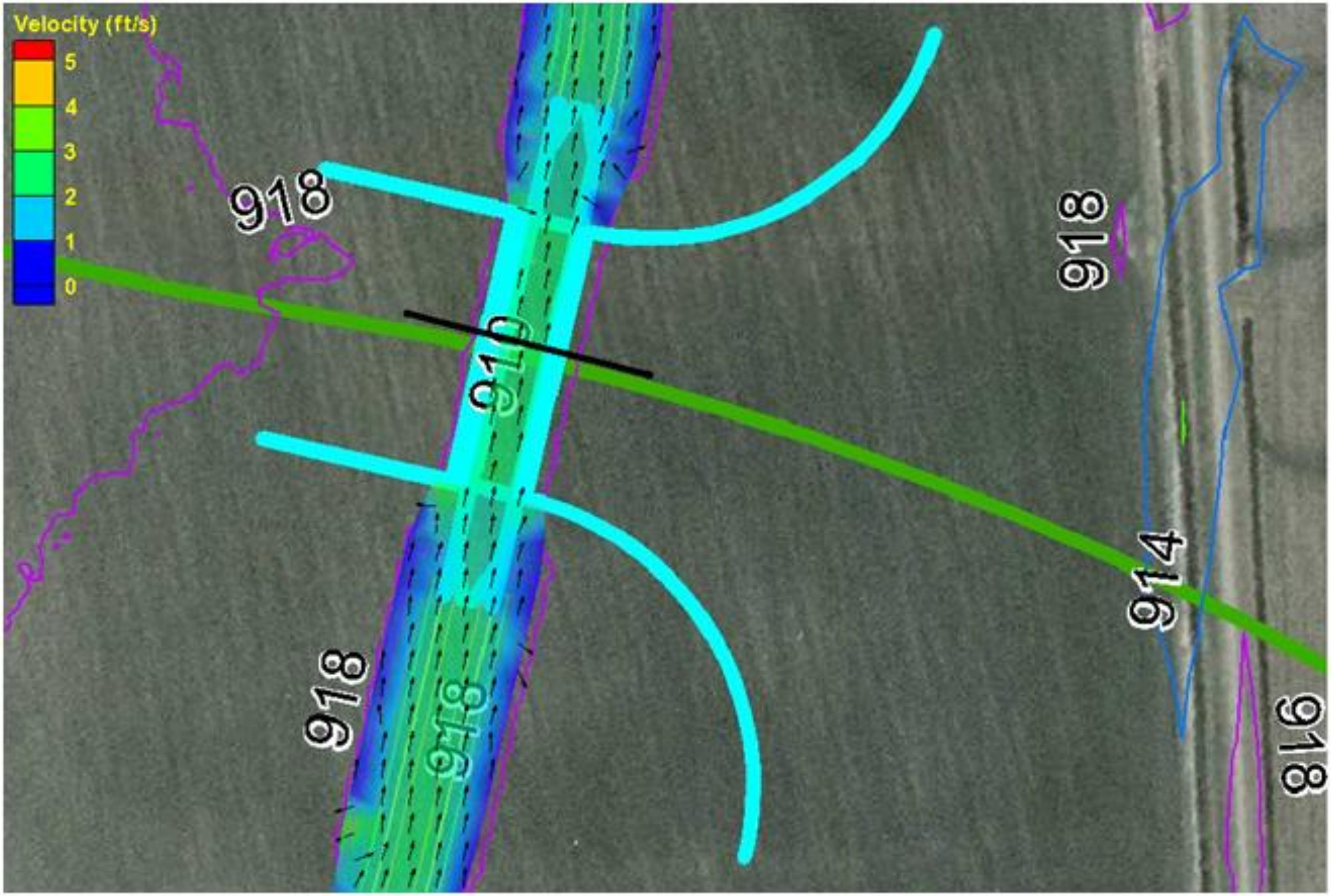

Figure F-H46 Velocity Field for Sheyenne River Aqueduct Post-construction Conditions Close Up 2-Year Event Flow

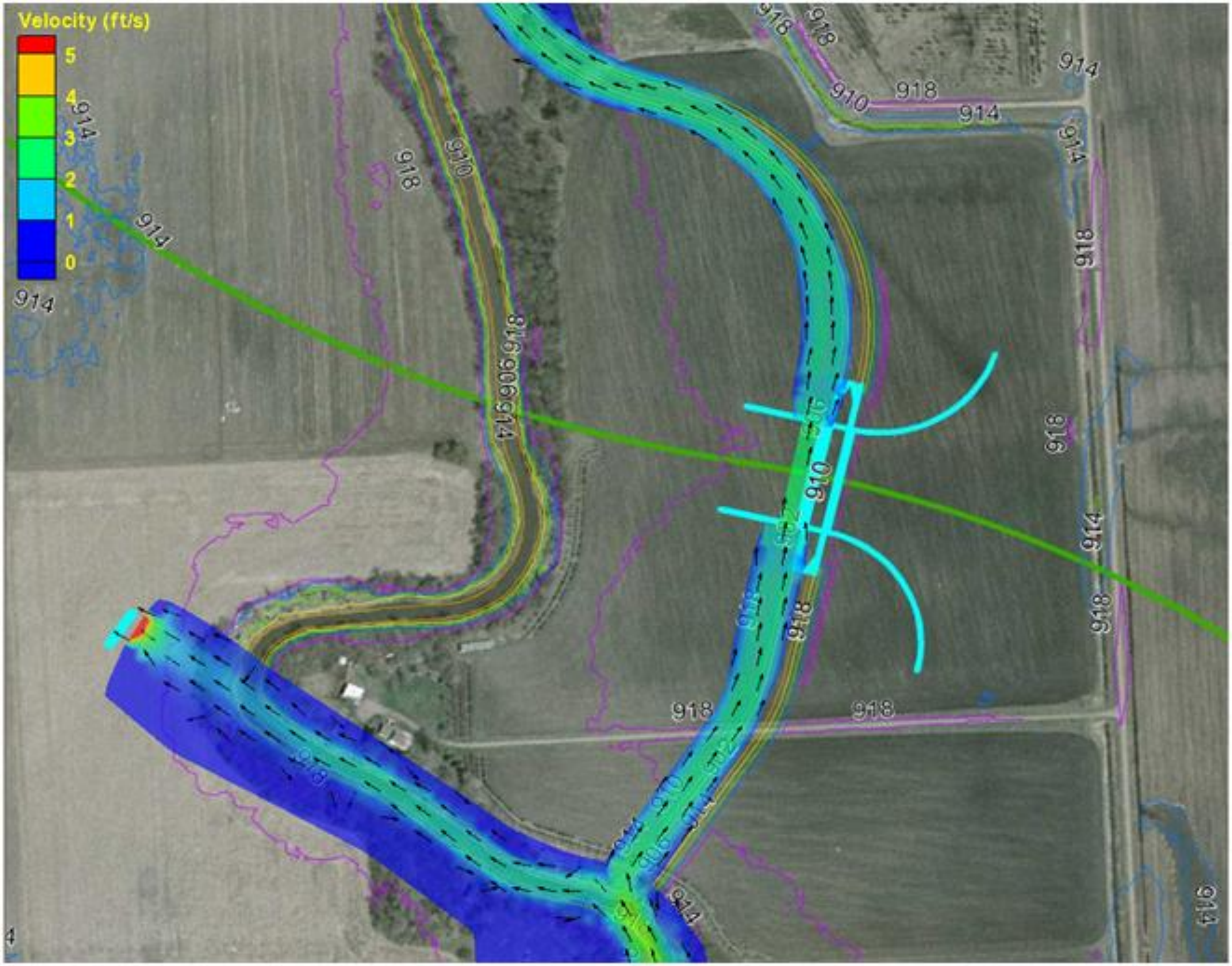

Figure F-H47 Velocity Field for Sheyenne River Aqueduct Post-construction Conditions 10-Year Event Flow

Figure F-H48 Velocity Field for Sheyenne River Aqueduct Post-construction Conditions Close Up 10-Year Event Flow

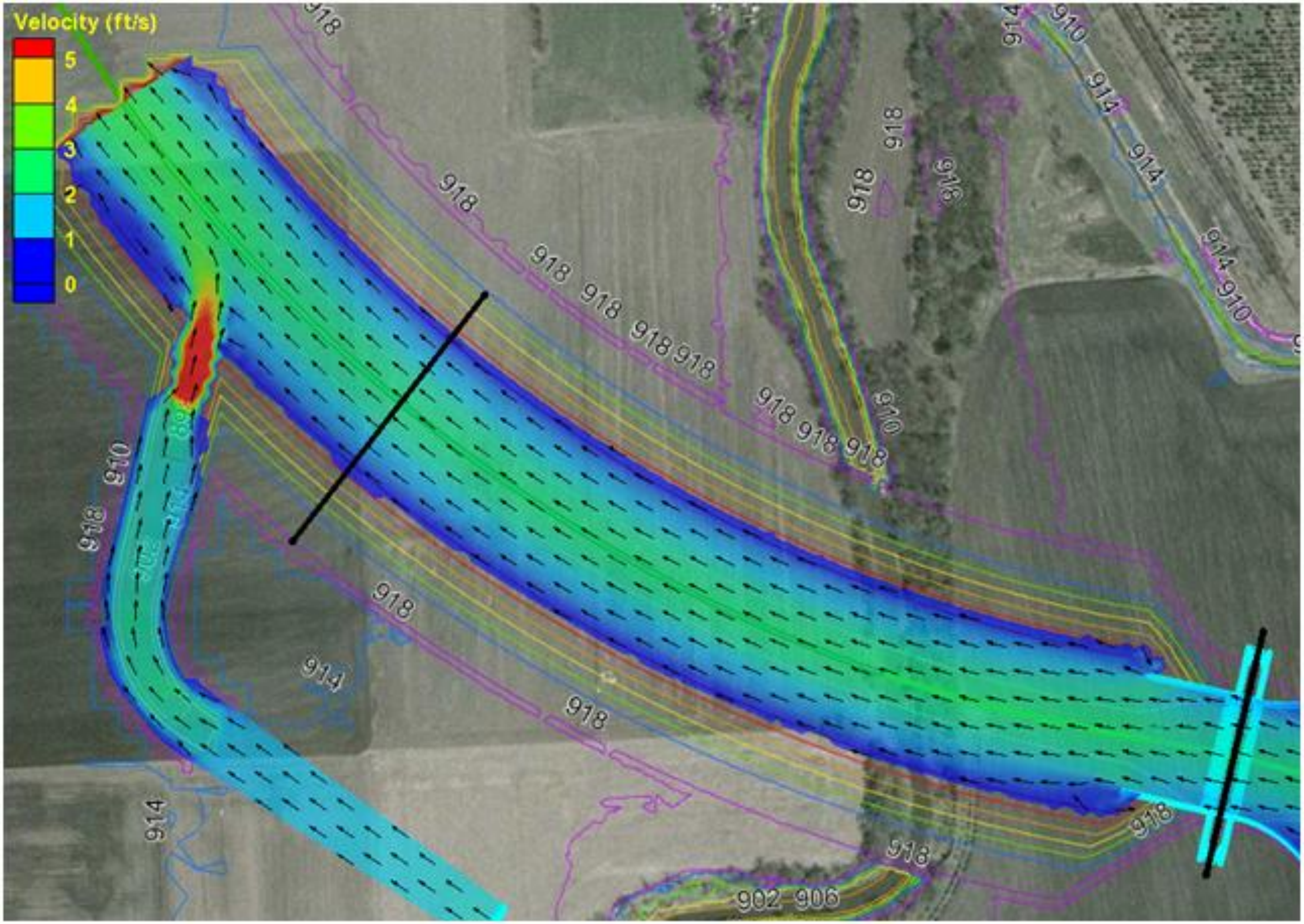

Figure F-H49 Velocity Field for Sheyenne River Diversion Channel Post-construction Conditions 5-Year Event Flow

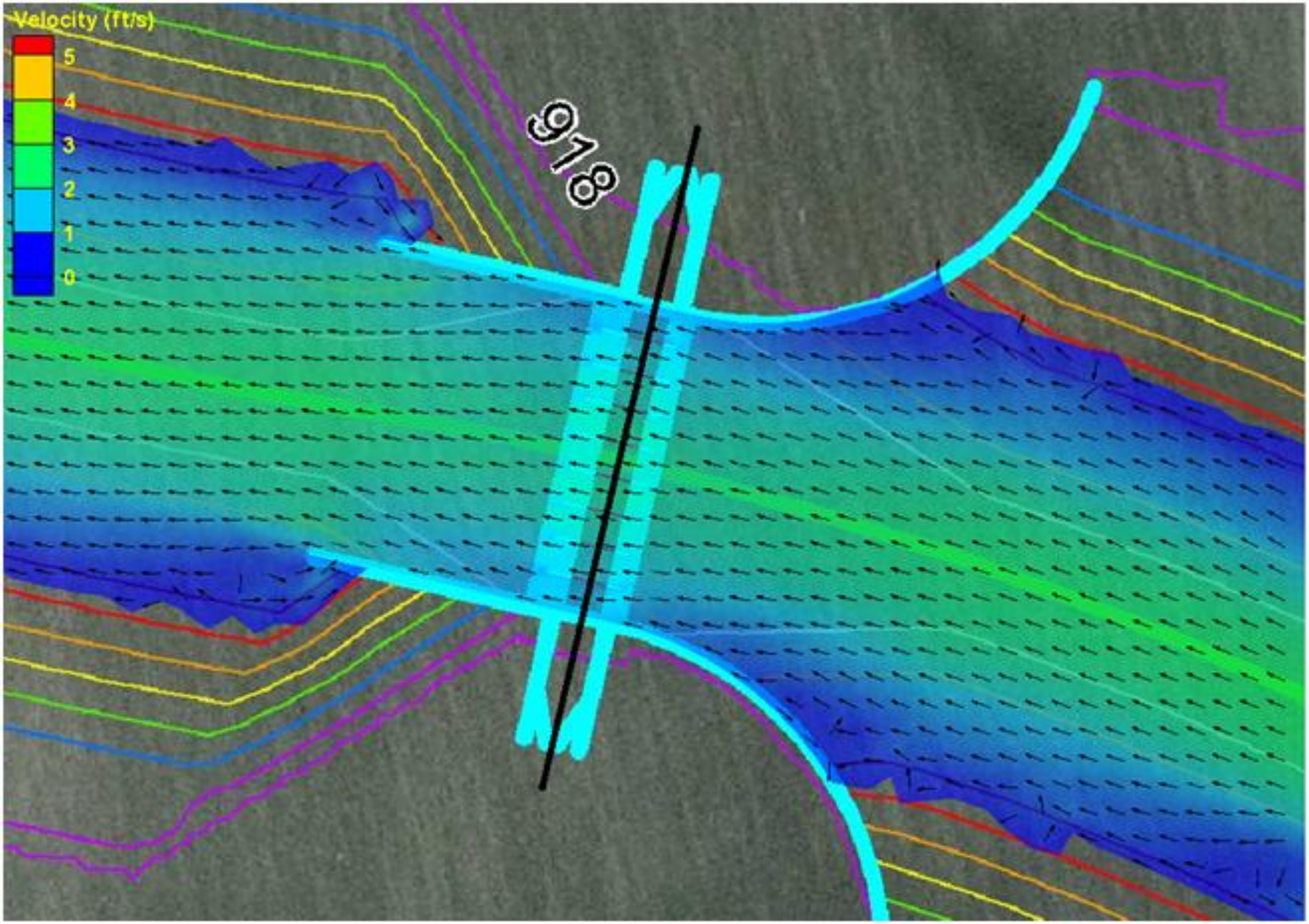

Figure F-H50 Velocity Field for Sheyenne River Diversion Channel and Aqueduct Crossing Post-construction Conditions Close Up 5-Year Event Flow

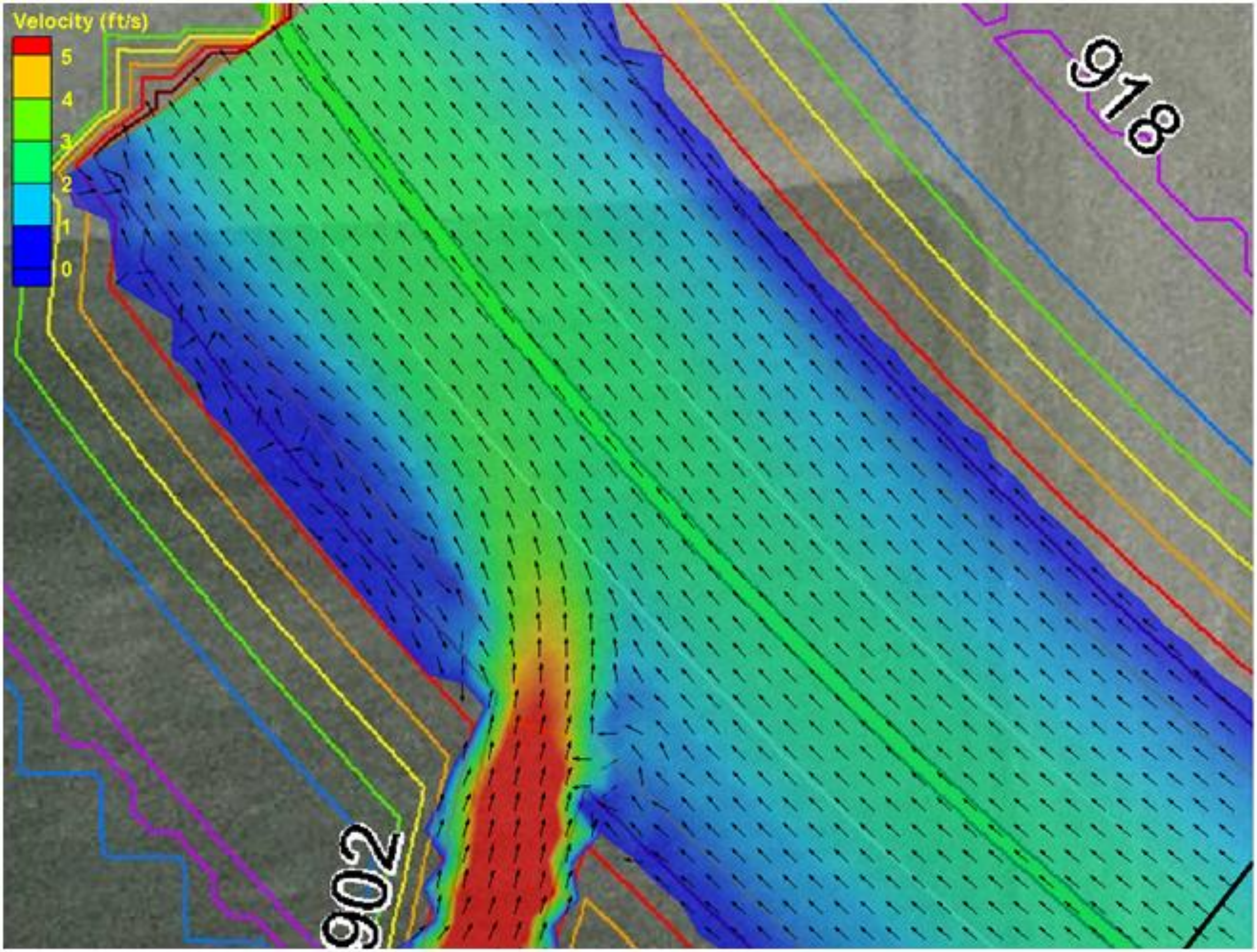

Figure F-H51 Velocity Field for Sheyenne River Diversion Channel and Spillway Post-construction Conditions Close Up 5-Year Event Flow

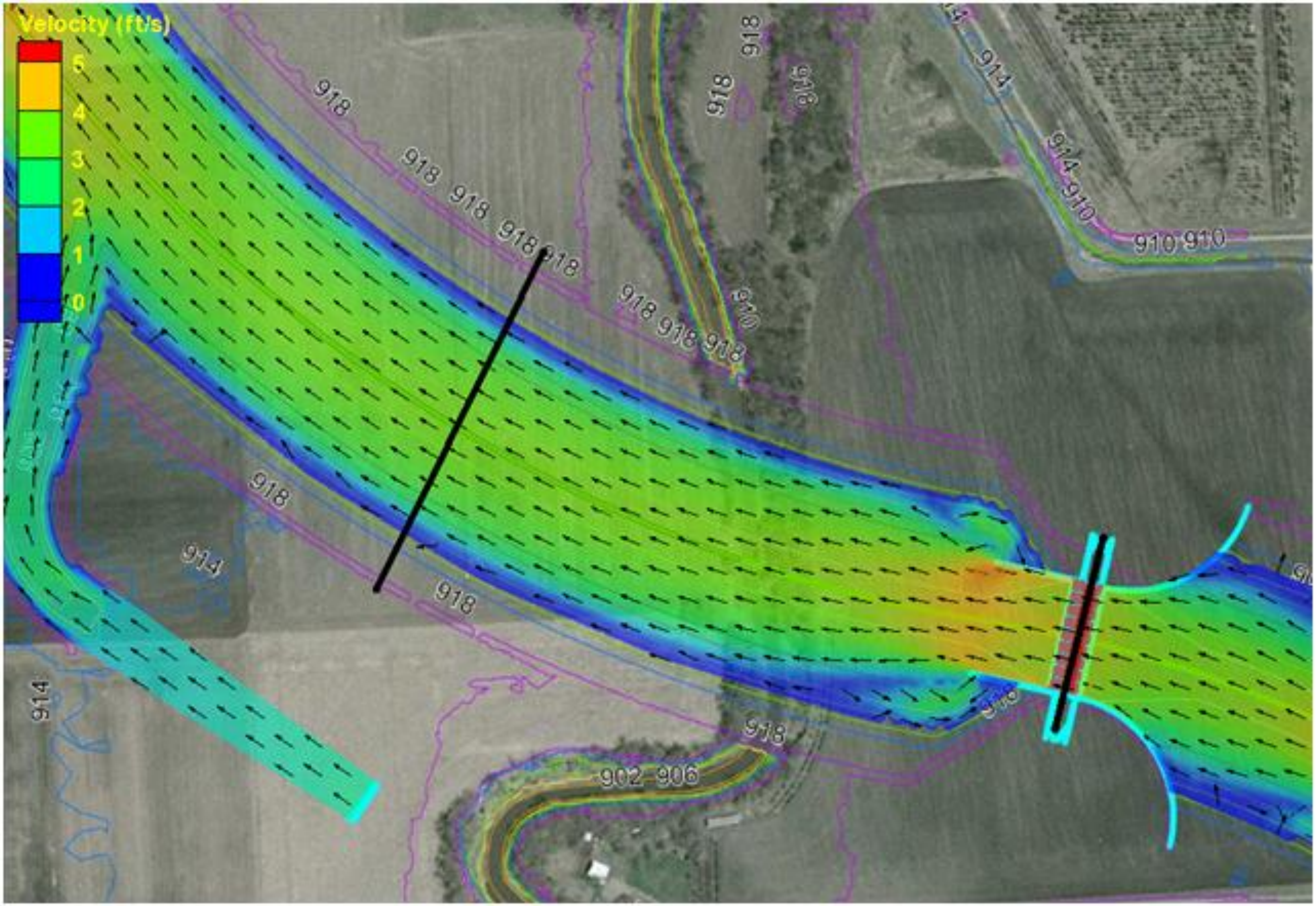

Figure F-H52 Velocity Field for Sheyenne River Diversion Channel Post-construction Conditions 50-Year Event Flow

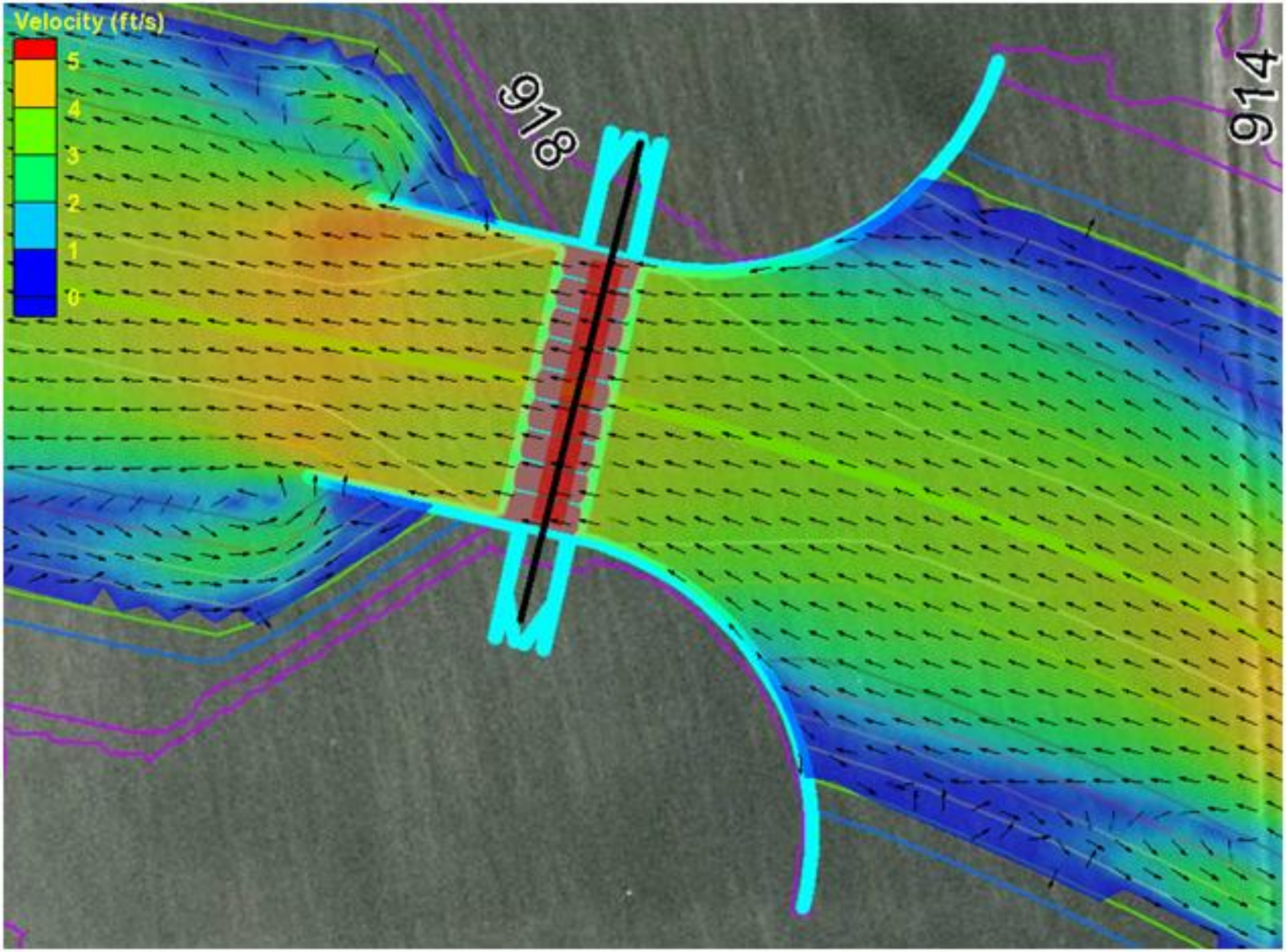

Figure F-H53 Velocity Field for Sheyenne River Diversion Channel and Aqueduct Crossing Post-construction Conditions Close Up 50-Year Event Flow

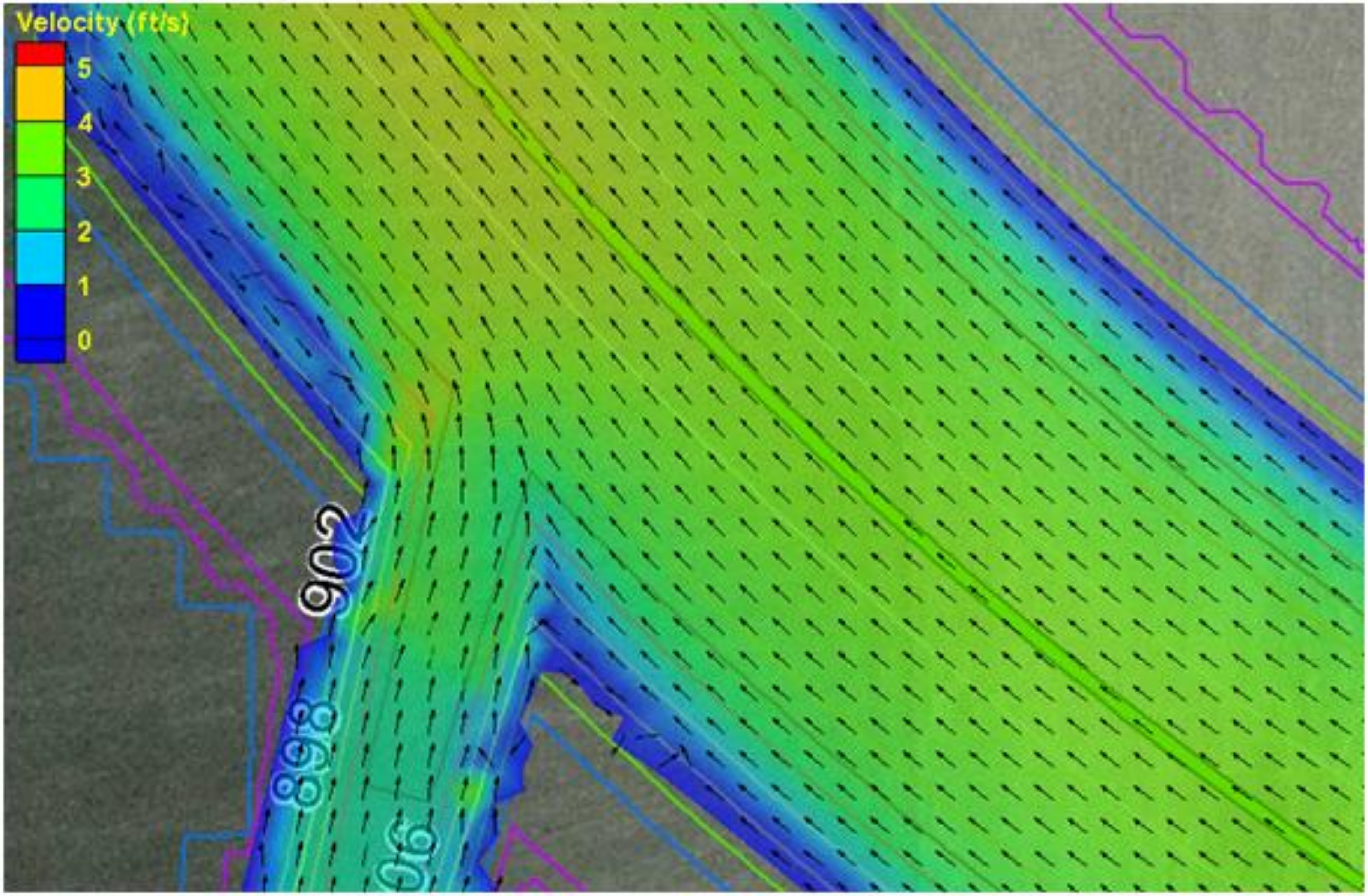

Figure F-H54 Velocity Field for Sheyenne River Diversion Channel and Spillway Post-construction Conditions Close Up 50-Year Event Flow

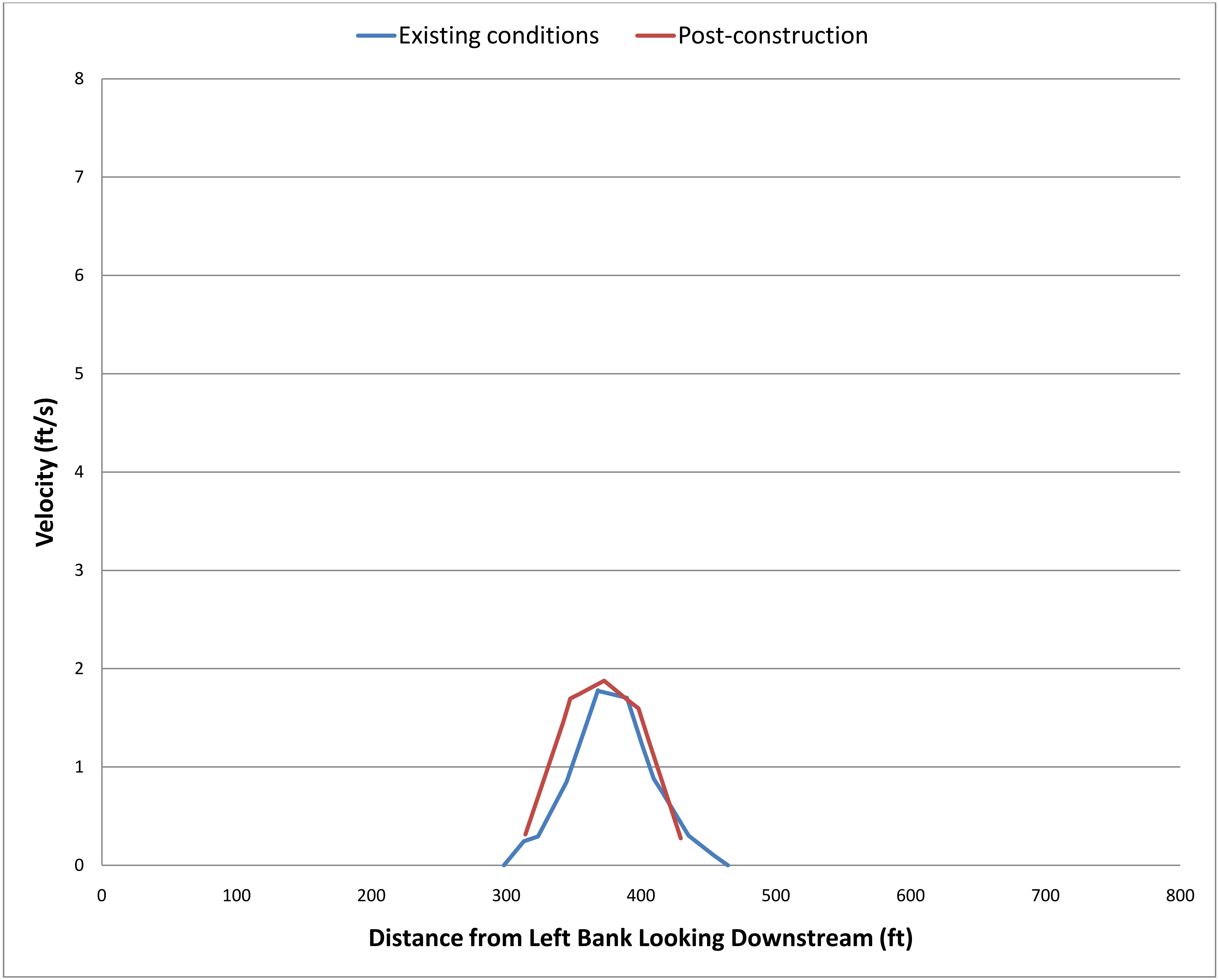

Figure F-H55 Cross Sectional Velocity Data for Sheyenne River Aqueduct Upstream During 2-Year Event

Cross Sectional Velocity Data for Sheyenne River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction upstream of structure.

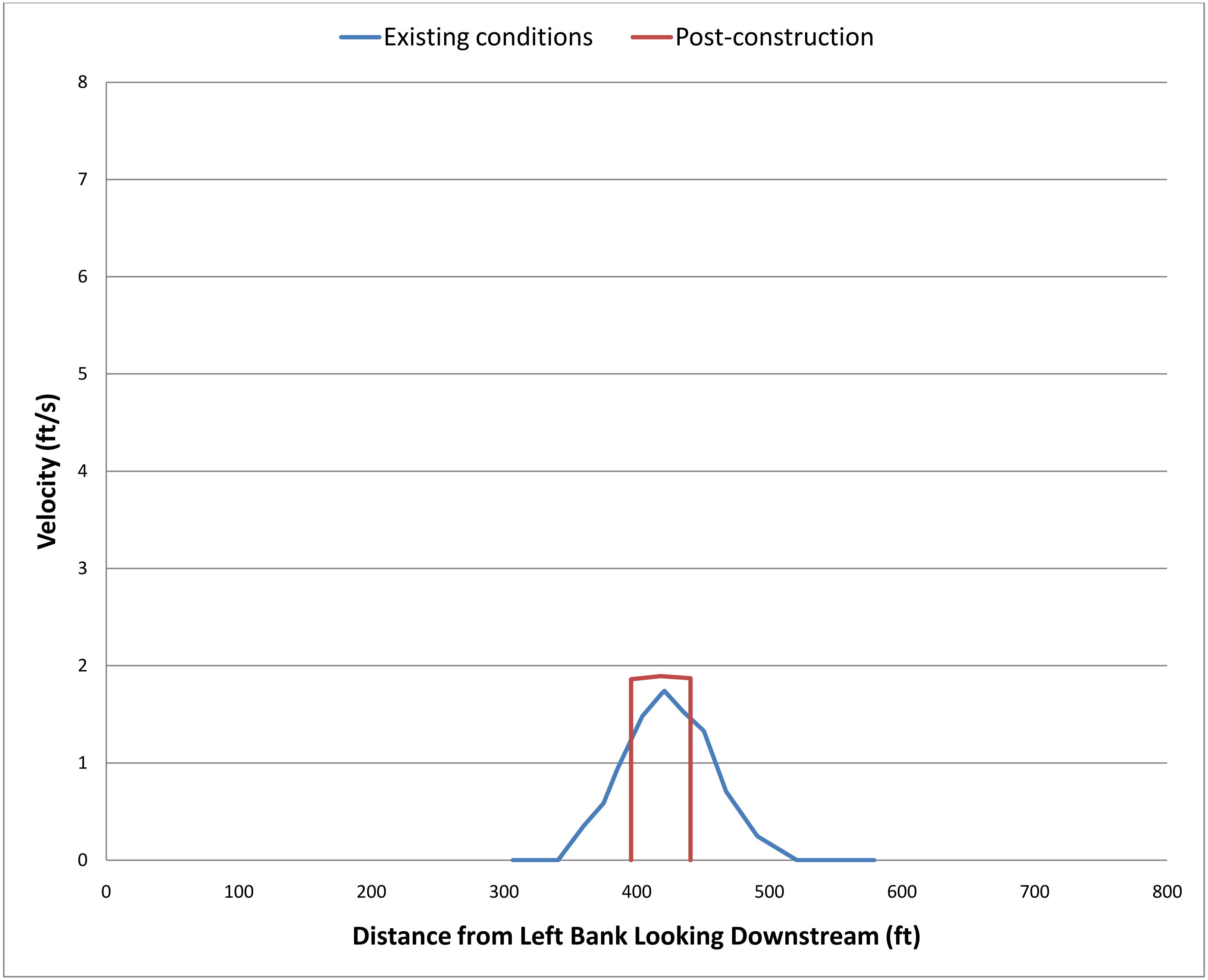

Figure F-H56 Cross Sectional Velocity Data for Sheyenne River Aqueduct Through Structure During 2-Year Event

Cross Sectional Velocity Data for Sheyenne River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction through structure.

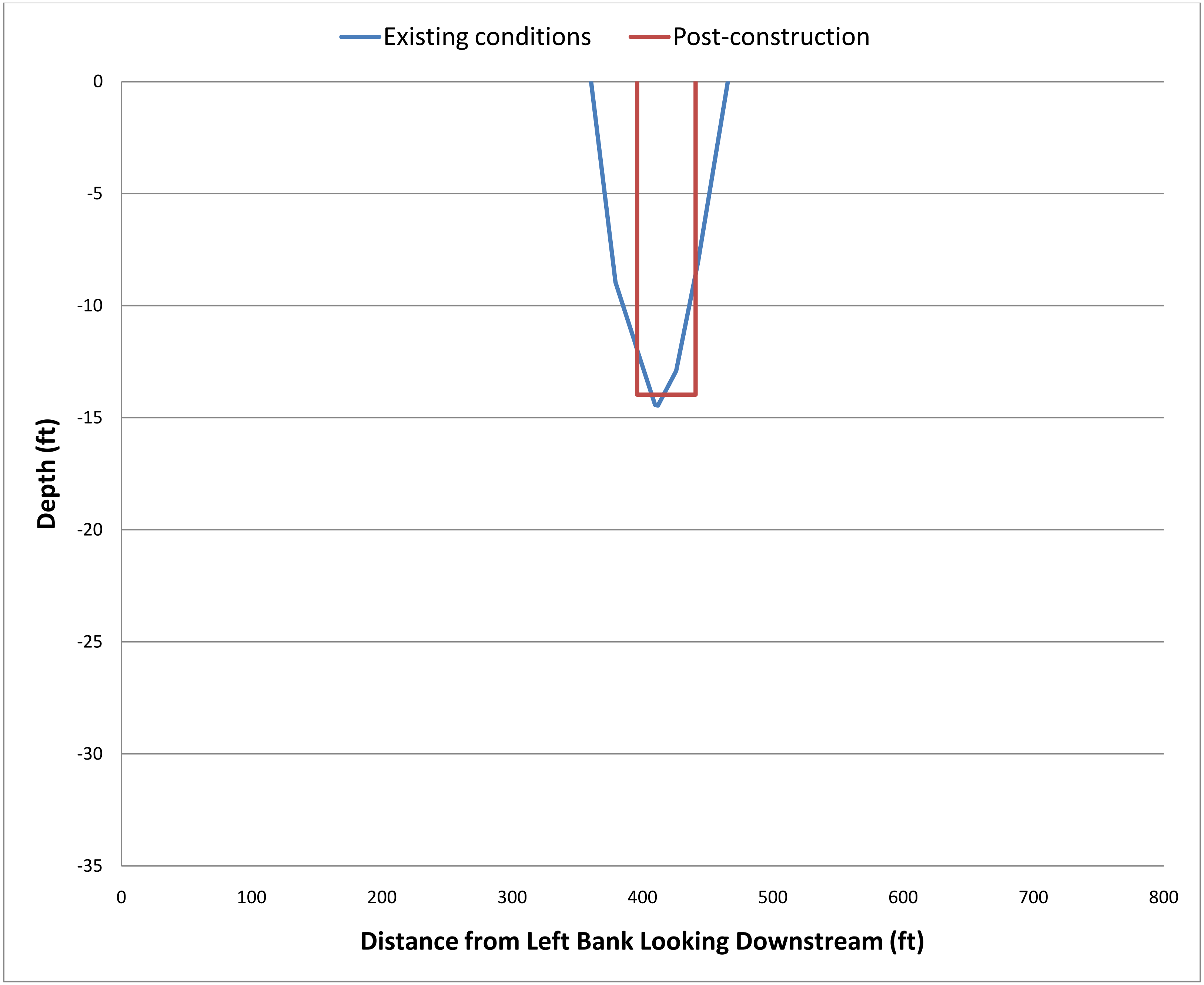

Figure F-H57 Cross Sectional Depth Data for Sheyenne River Aqueduct Through Structure During 2-Year Event

Cross Sectional Depth Data for Sheyenne River Aqueduct (2-Year). Compares depth at existing conditions to post-construction through structure.

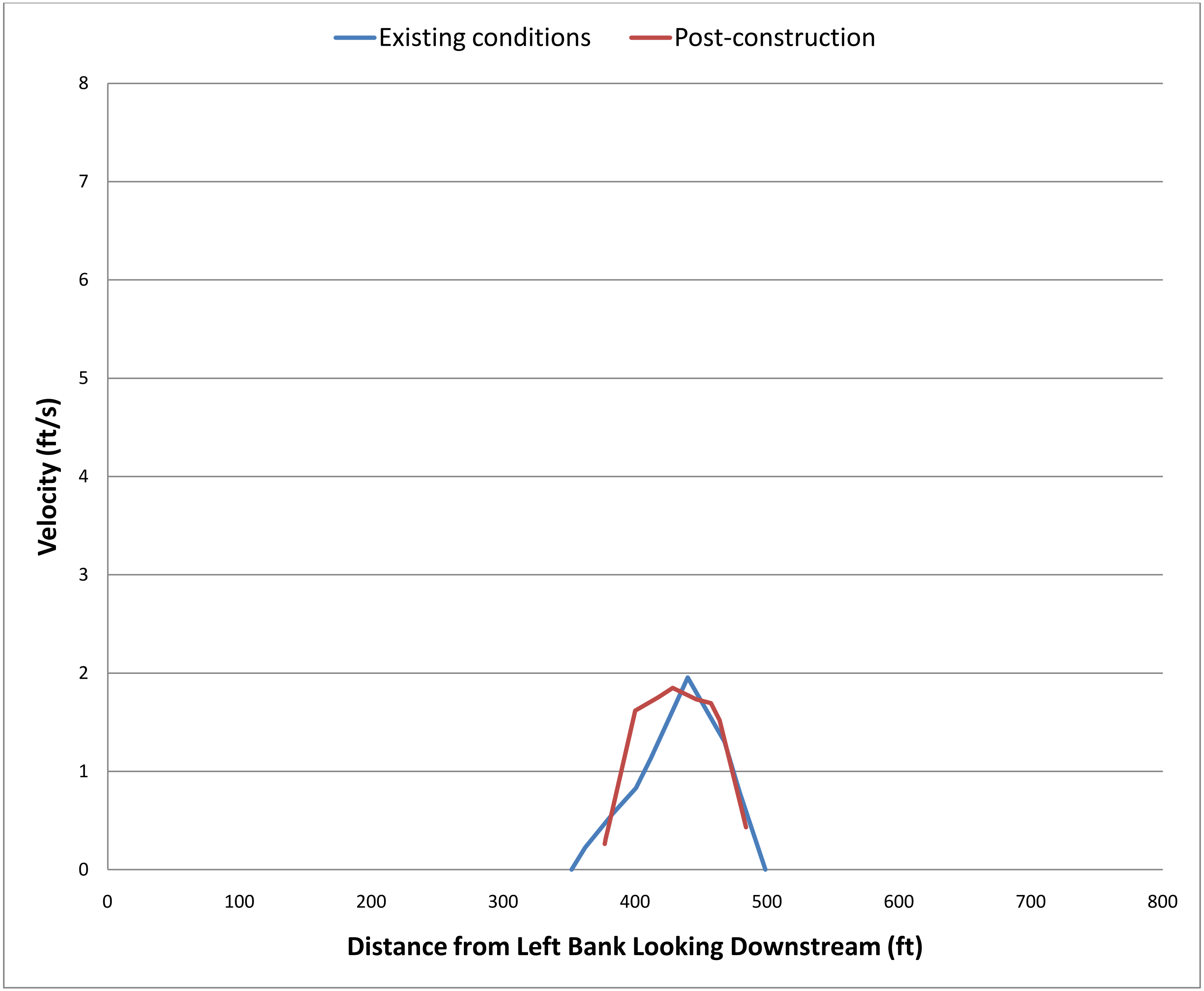

Figure F-H58 Cross Sectional Velocity Data for Sheyenne River Aqueduct Downstream During 2-Year Event

Cross Sectional Velocity Data for Sheyenne River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction downstream of structure.

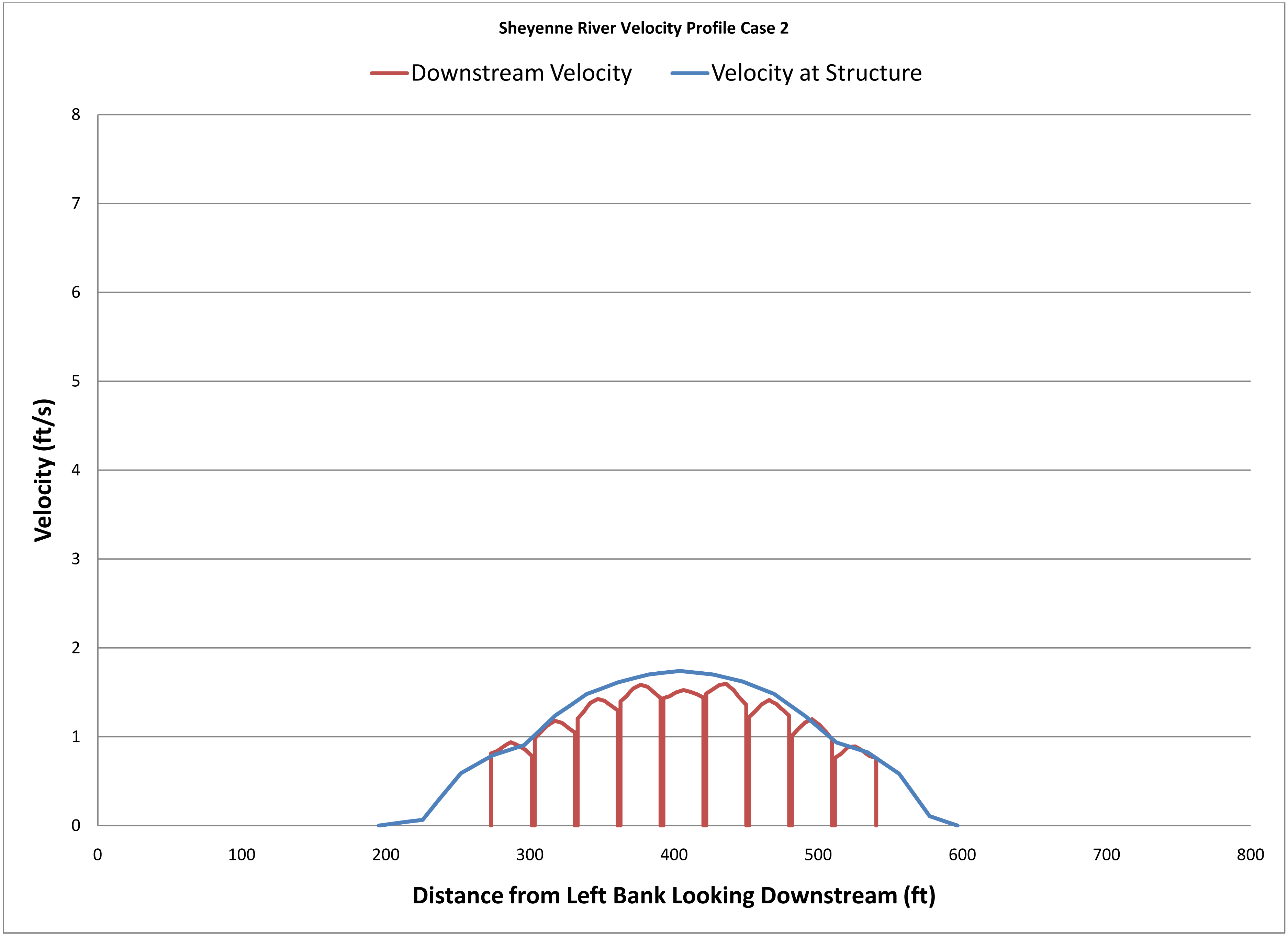

Figure F-H59 Cross Sectional Velocity Data for Sheyenne River Diversion Channel Through Structure During 5-Year Event

Cross Sectional Velocity Data for Sheyenne River Diversion Channel (5-Year). Compares velocity at structure to velocity downstream.

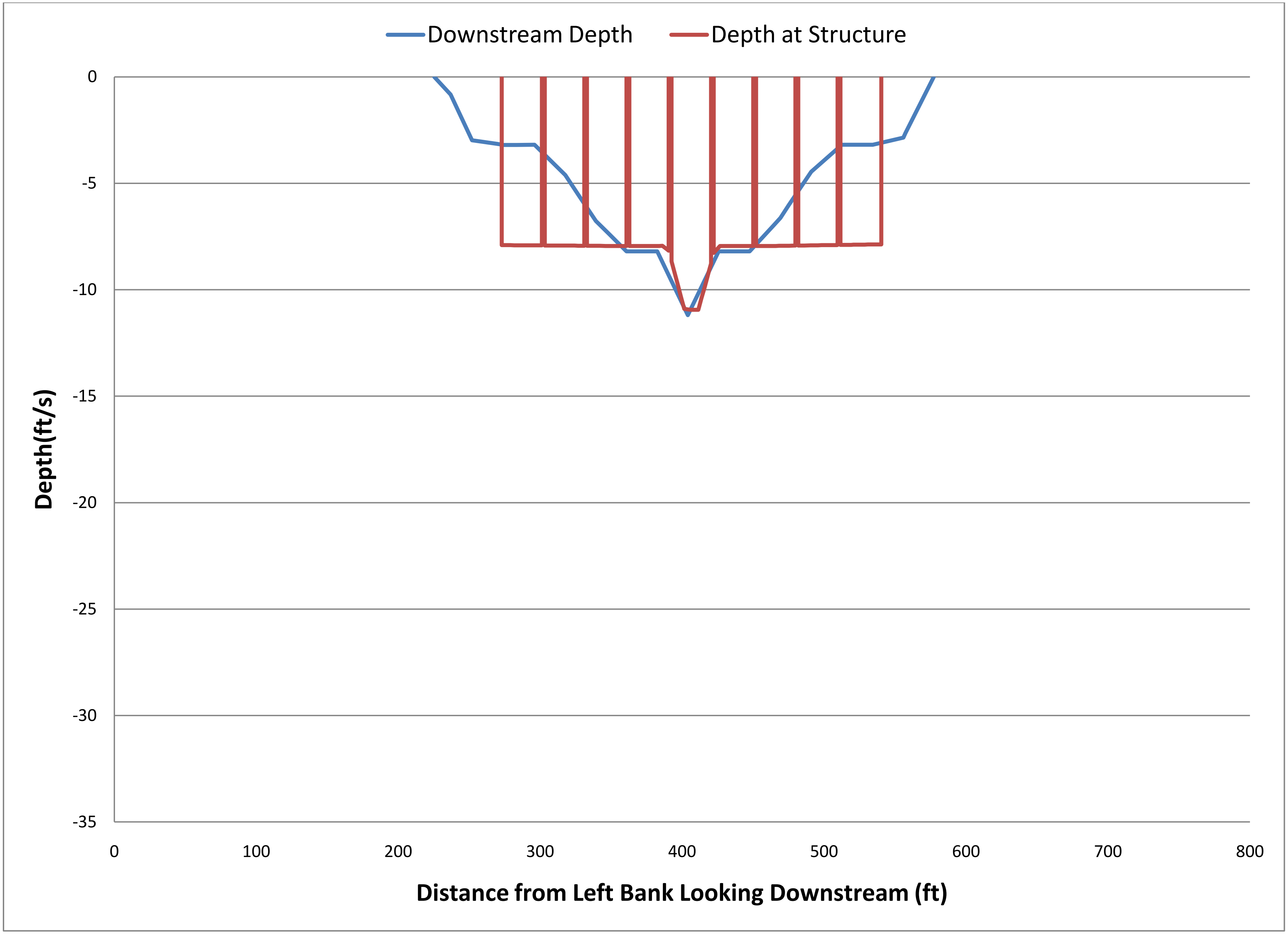

Figure F-H60 Cross Sectional Depth Data for Sheyenne River Diversion Channel Through Structure During 5-Year Event

Cross Sectional Depth Data for Sheyenne River Diversion Channel (5-Year). Compares depth at structure to depth downstream.

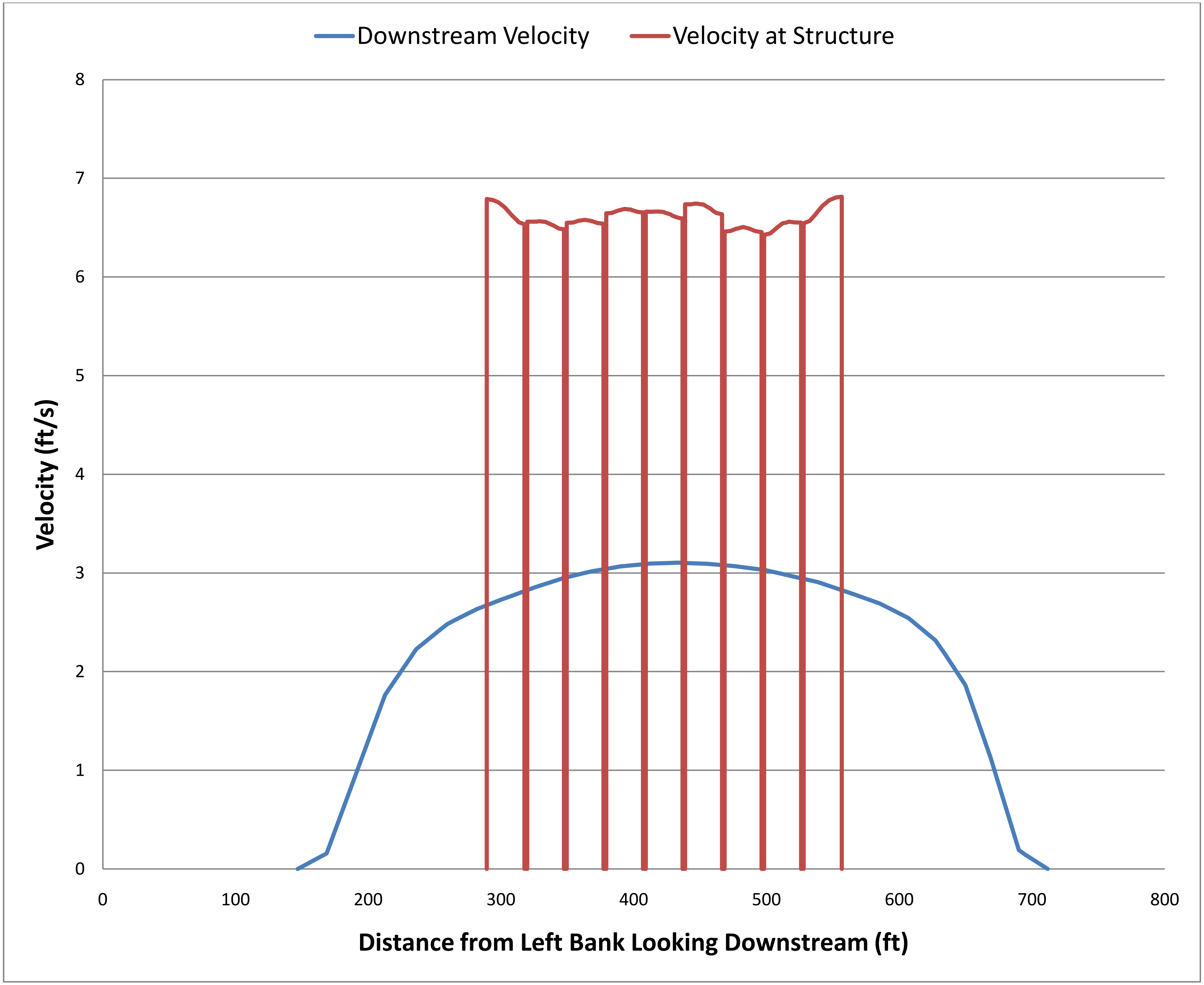

Figure F-H61 Cross Sectional Velocity Data for Sheyenne River Diversion Channel Through Structure During 50-Year Event

Cross Sectional Velocity Data for Sheyenne River Diversion Channel (50-Year). Compares velocity at structure to velocity downstream.

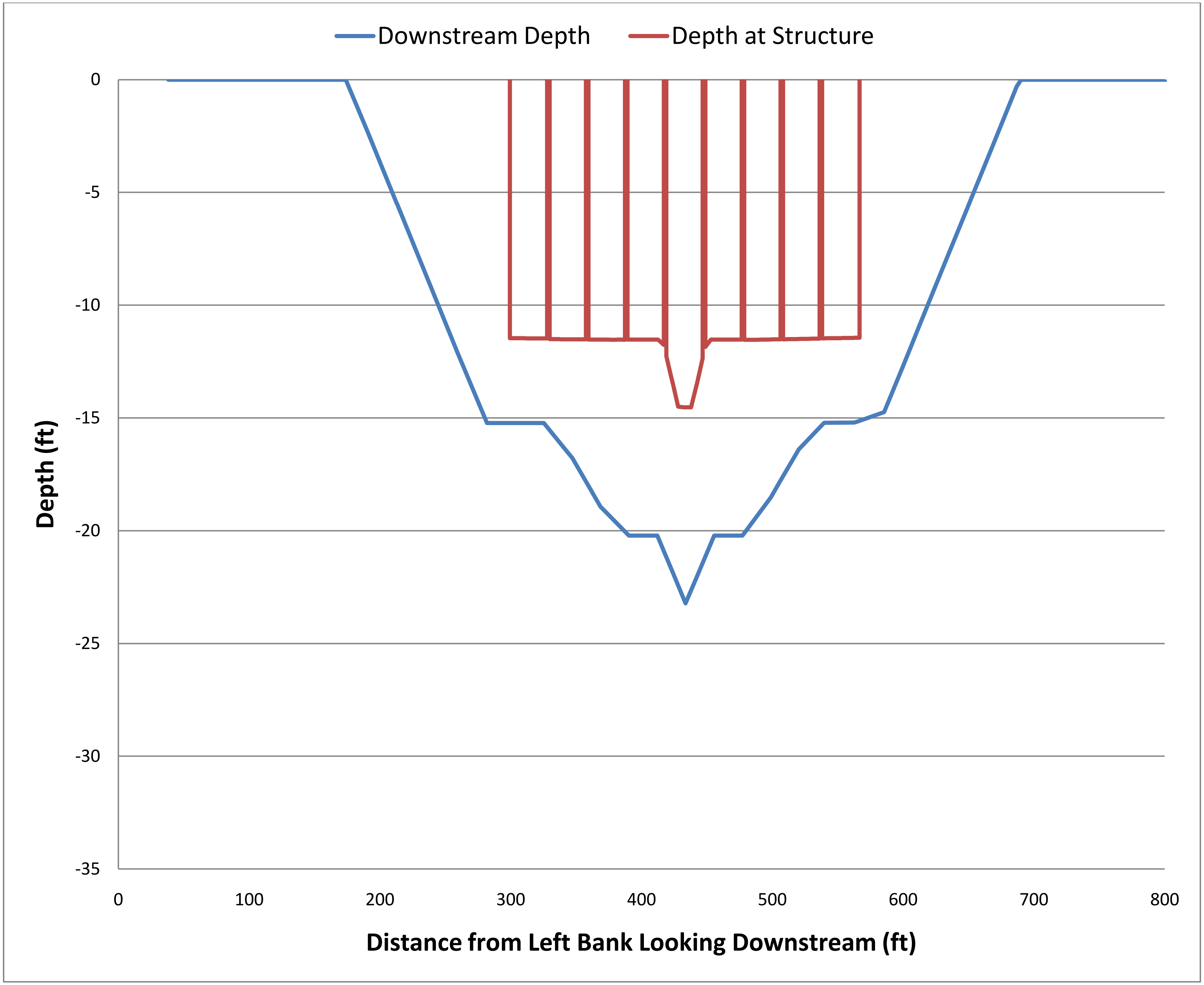

Figure F-H62 Cross Sectional Depth Data for Sheyenne River Diversion Channel Through Structure During 50-Year Event

Cross Sectional Depth Data for Sheyenne River Diversion Channel (50-Year). Compares depth at structure to depth downstream.

Figure F-H63 Velocity Field for Maple River Existing Conditions 2-Year Event Flow

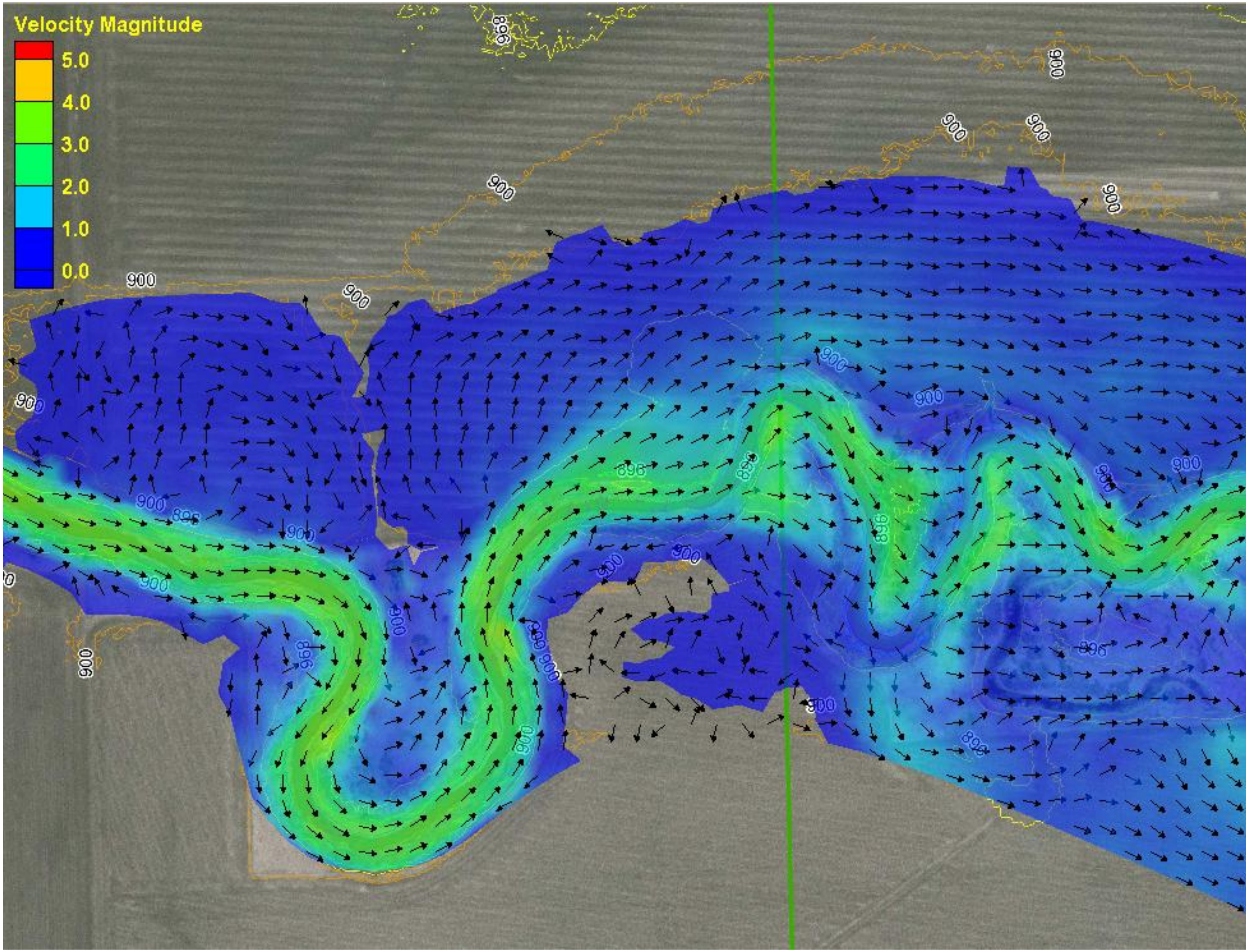

Figure F-H64 Velocity Field for Maple River Existing Conditions 50-Year Event Flow

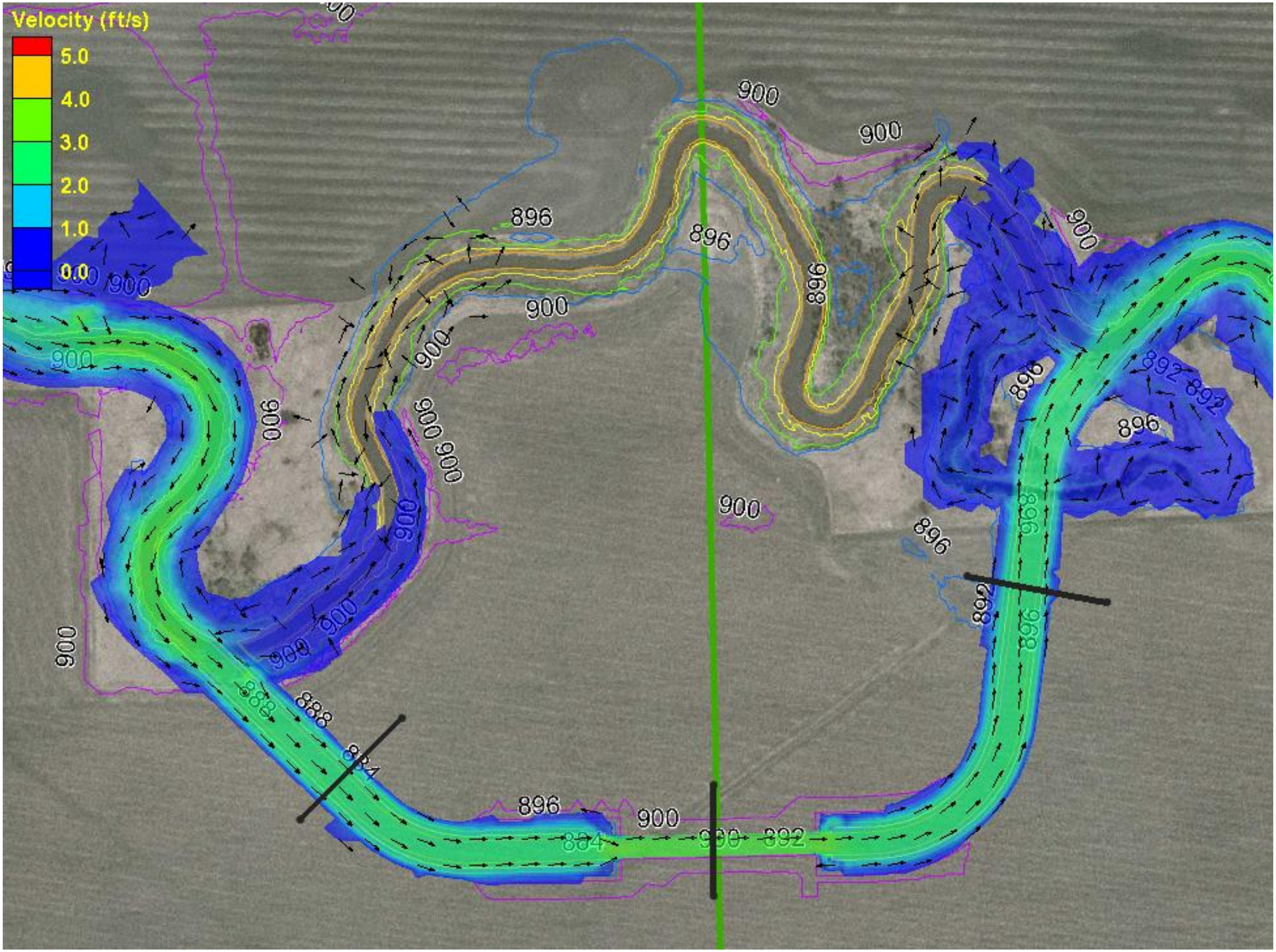

Figure F-H65 Velocity Field for Maple River Aqueduct Post-construction Conditions 2-Year Event Flow

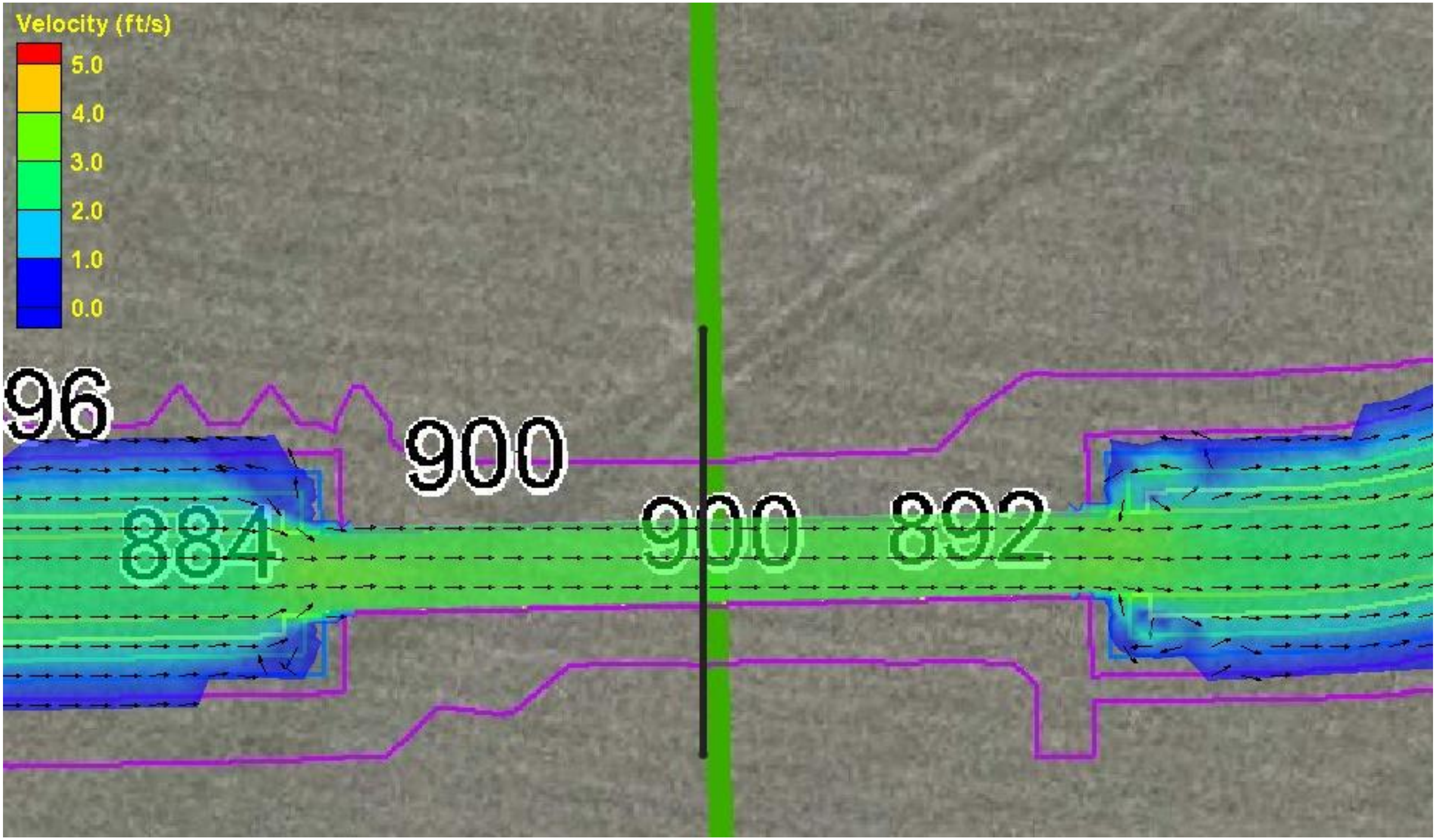

Figure F-H66 Velocity Field for Maple River Aqueduct Post-construction Conditions Close Up 2-Year Event Flow

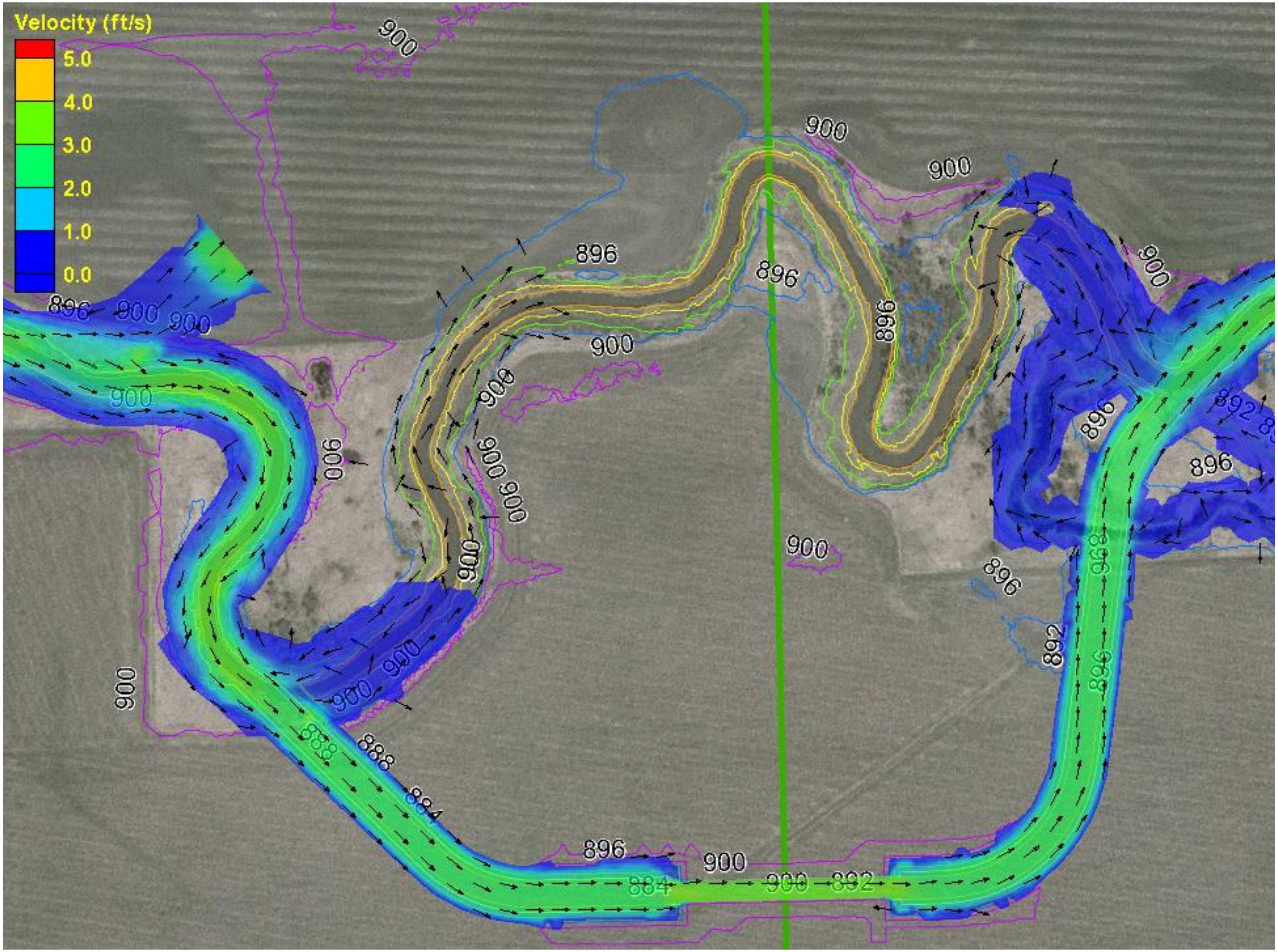

Figure F-H67 Velocity Field for Maple River Aqueduct Post-construction Conditions 5-Year Event Flow

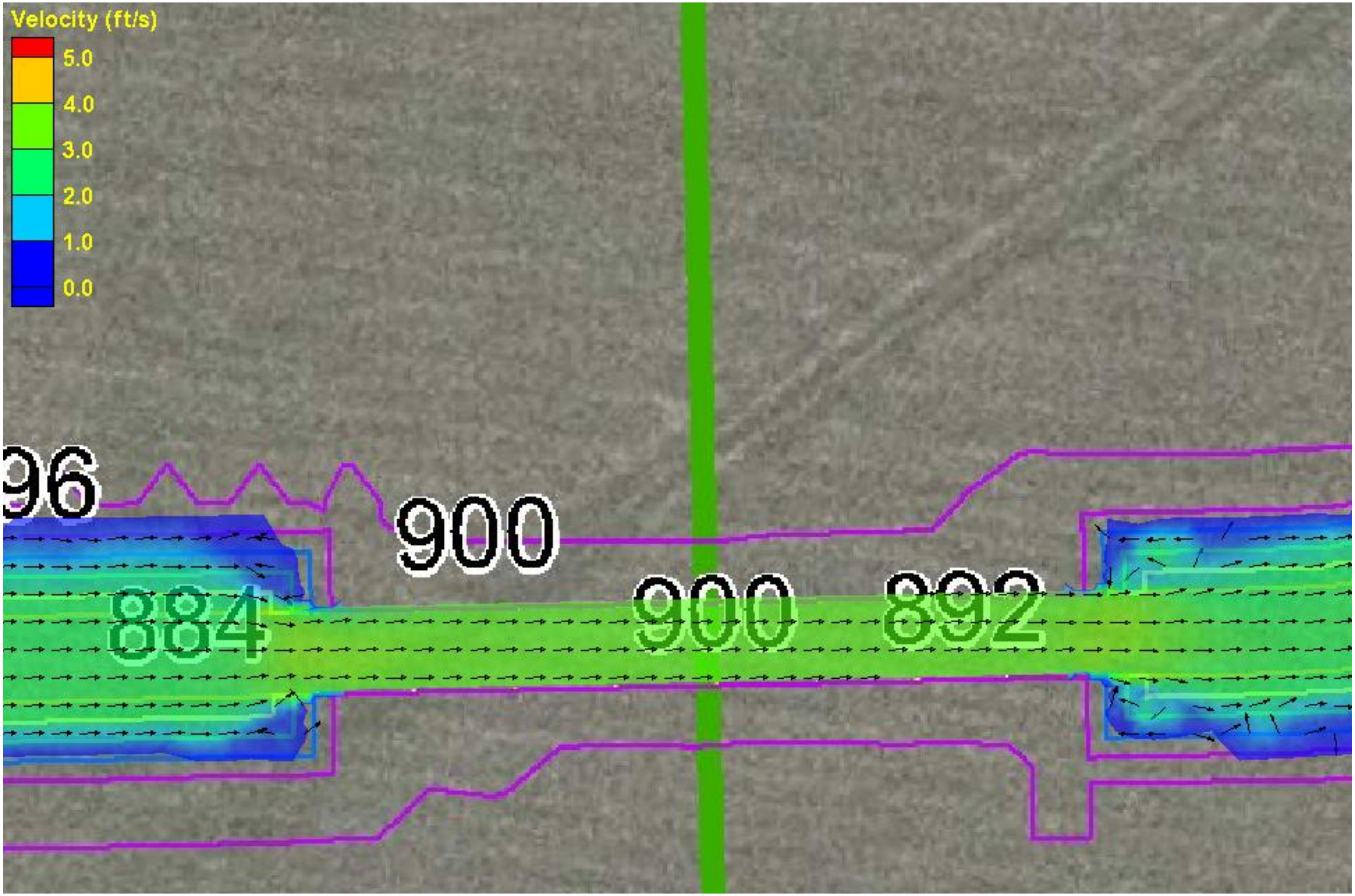

Figure F-H68 Velocity Field for Maple River Aqueduct Post-construction Conditions Close Up 5-Year Event Flow

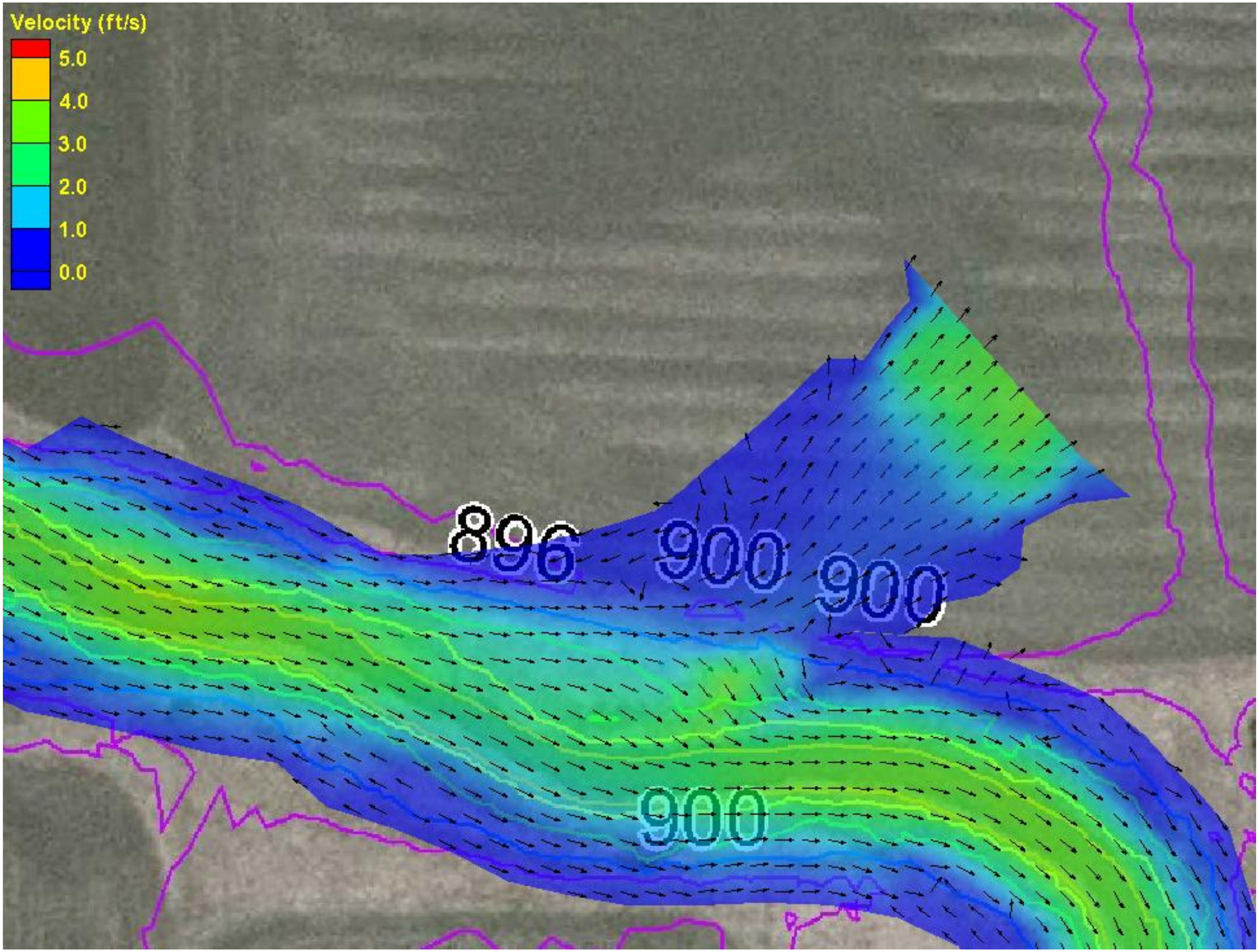

Figure F-H69 Velocity Field for Maple River Spillway Post-construction Conditions Close Up 5-Year Event Flow

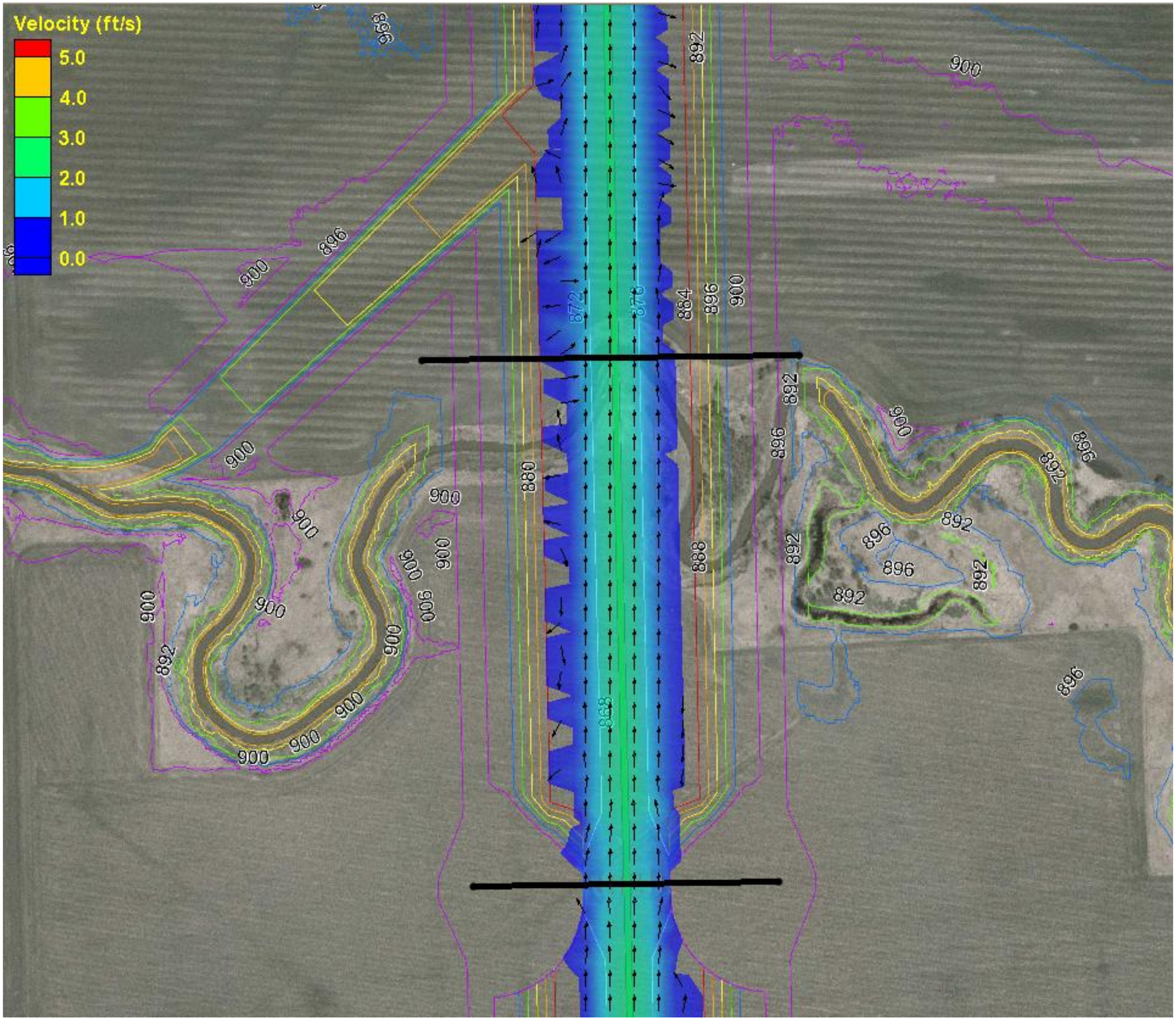

Figure F-H70 Velocity Field for Maple River Diversion Channel Post-construction Conditions 2,000 cfs Event Flow

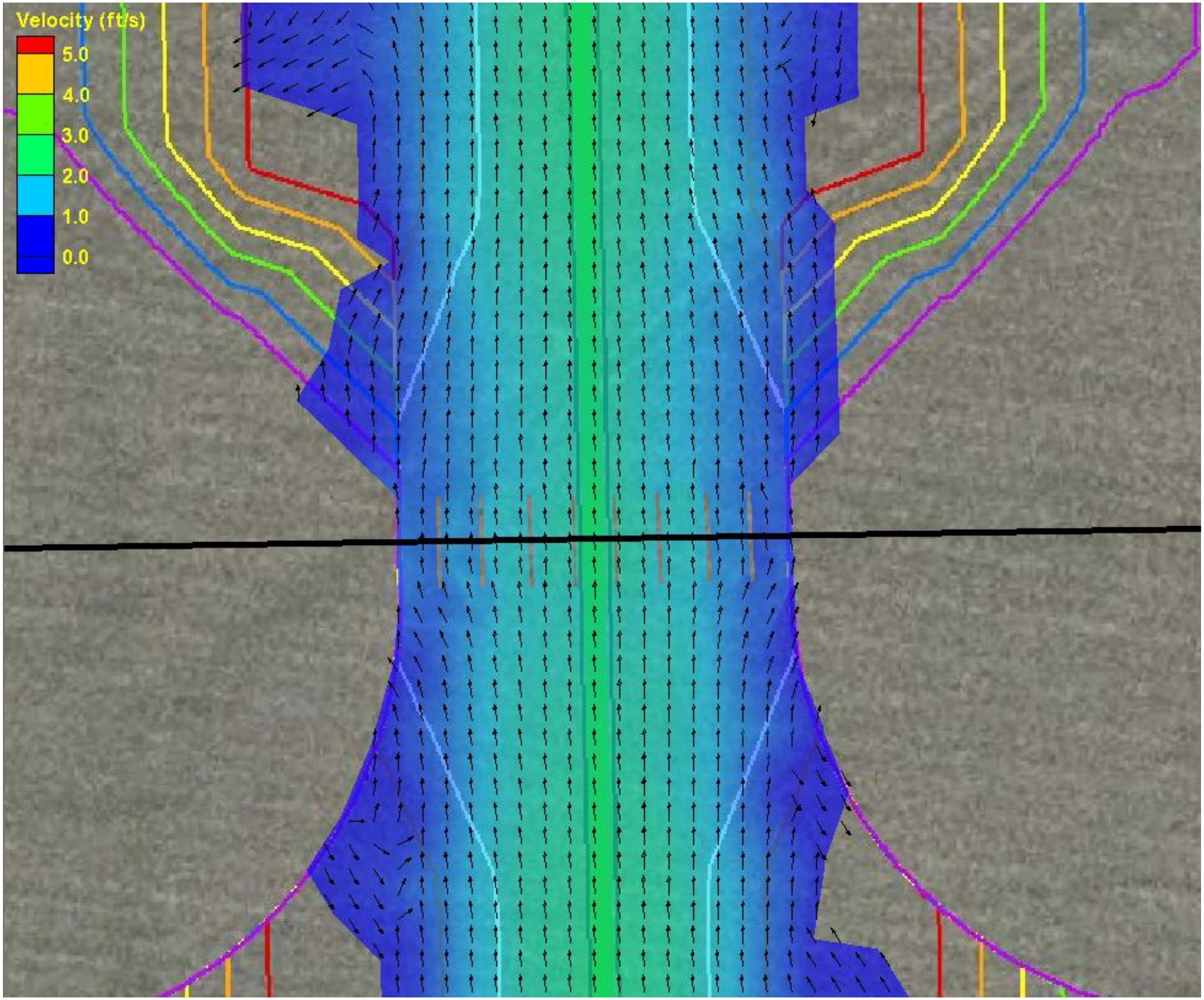

Figure F-H71 Velocity Field for Maple River Diversion Channel Post-construction Conditions Close Up 2,000 cfs Event Flow

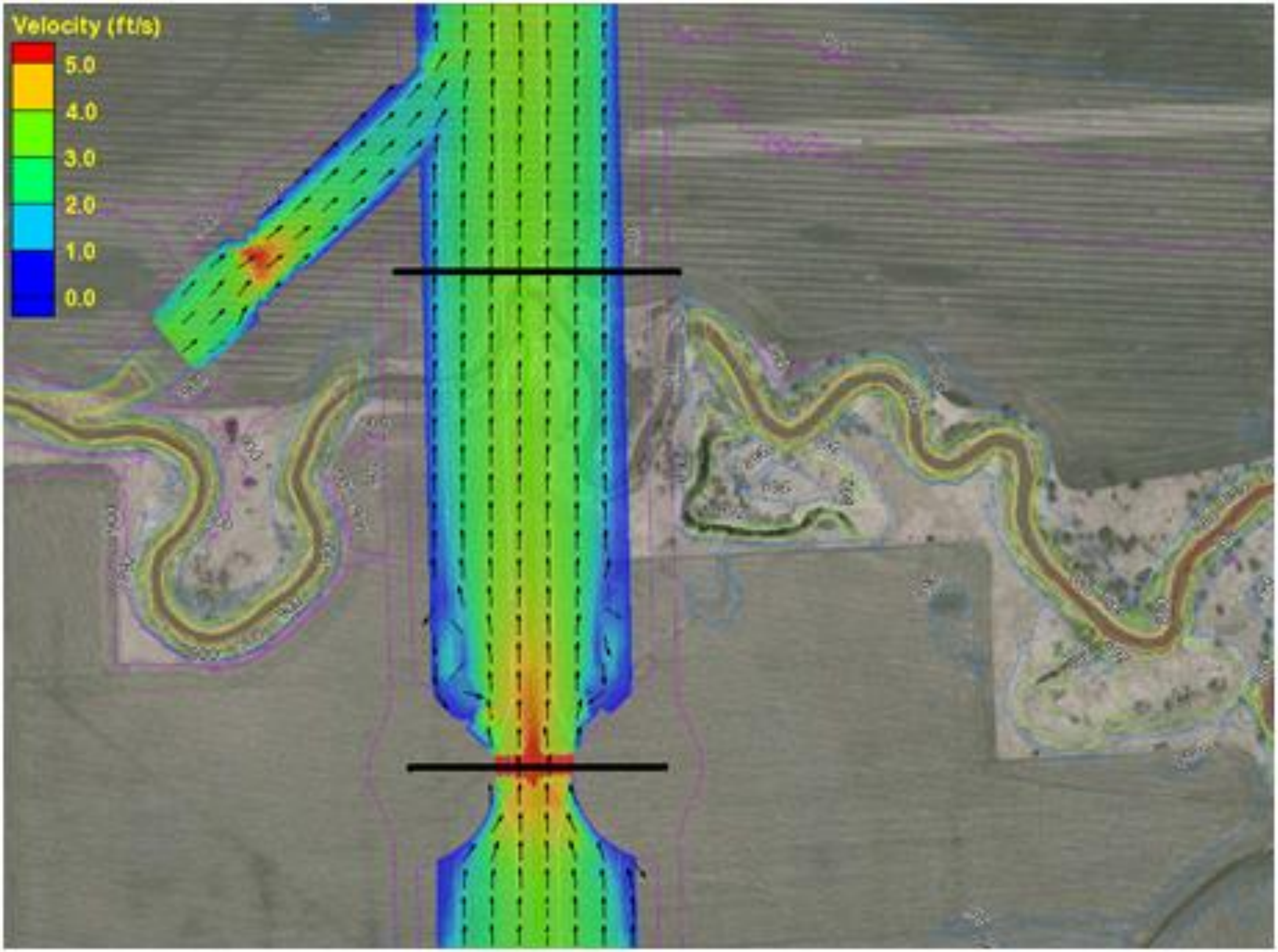

Figure F-H72 Velocity Field for Maple River Diversion Channel Post-construction Conditions 50-Year Event Flow

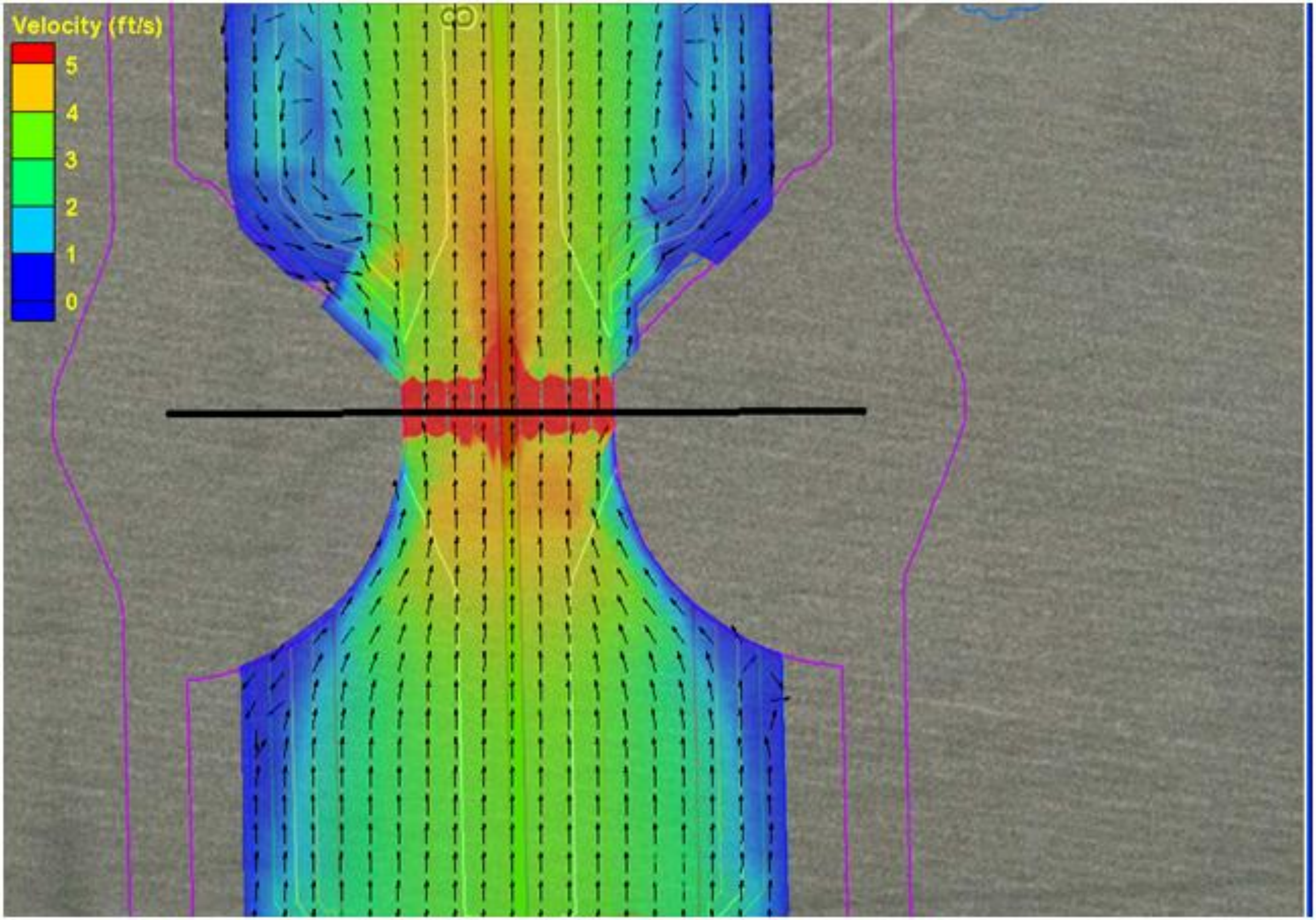

Figure F-H73 Velocity Field for Maple River Diversion Channel Crossing Post-construction Conditions Close Up 50-Year Event Flow

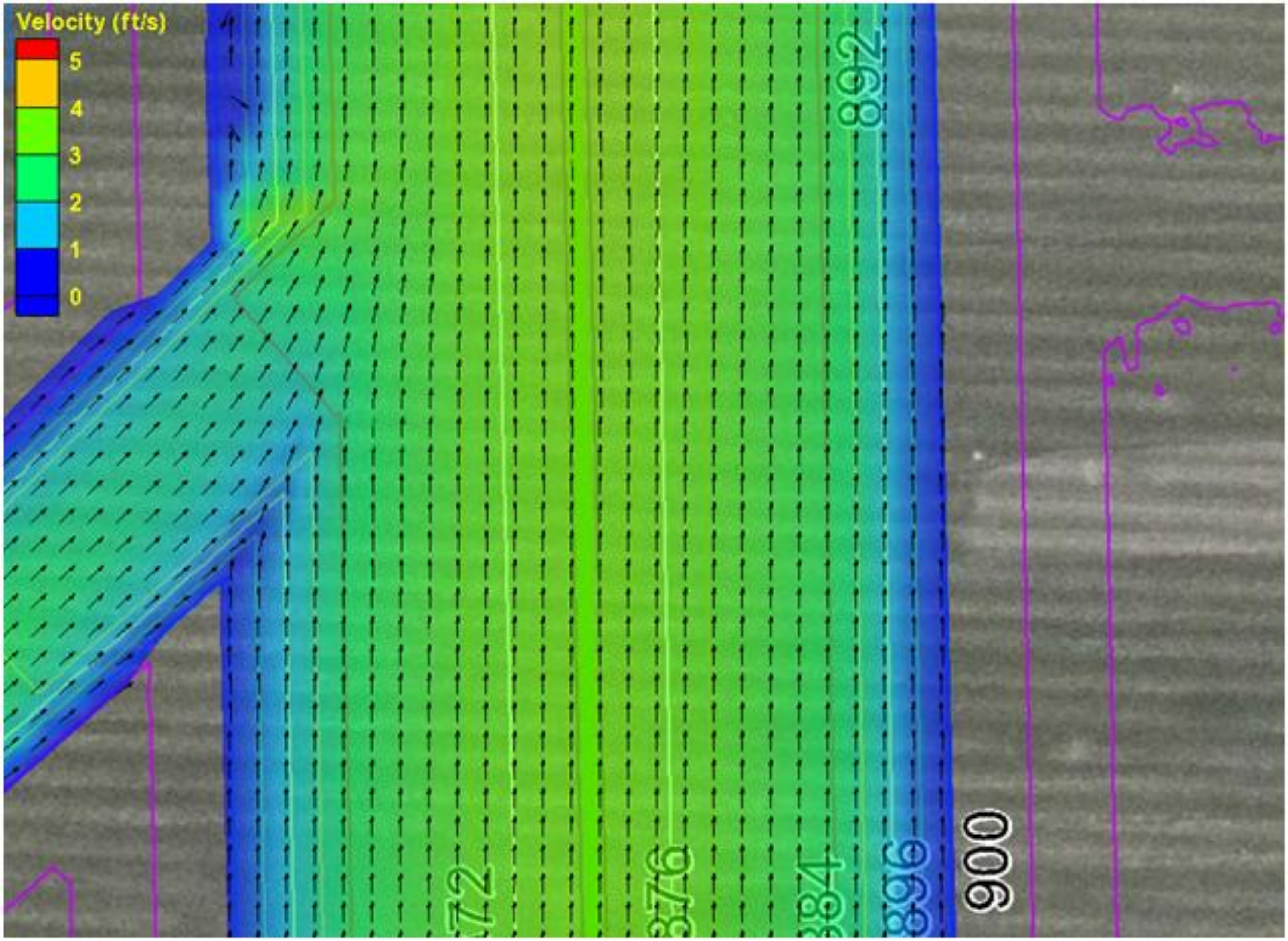

Figure F-H74 Velocity Field for Maple River Spillway Post-construction Conditions Close Up 50-Year Event Flow

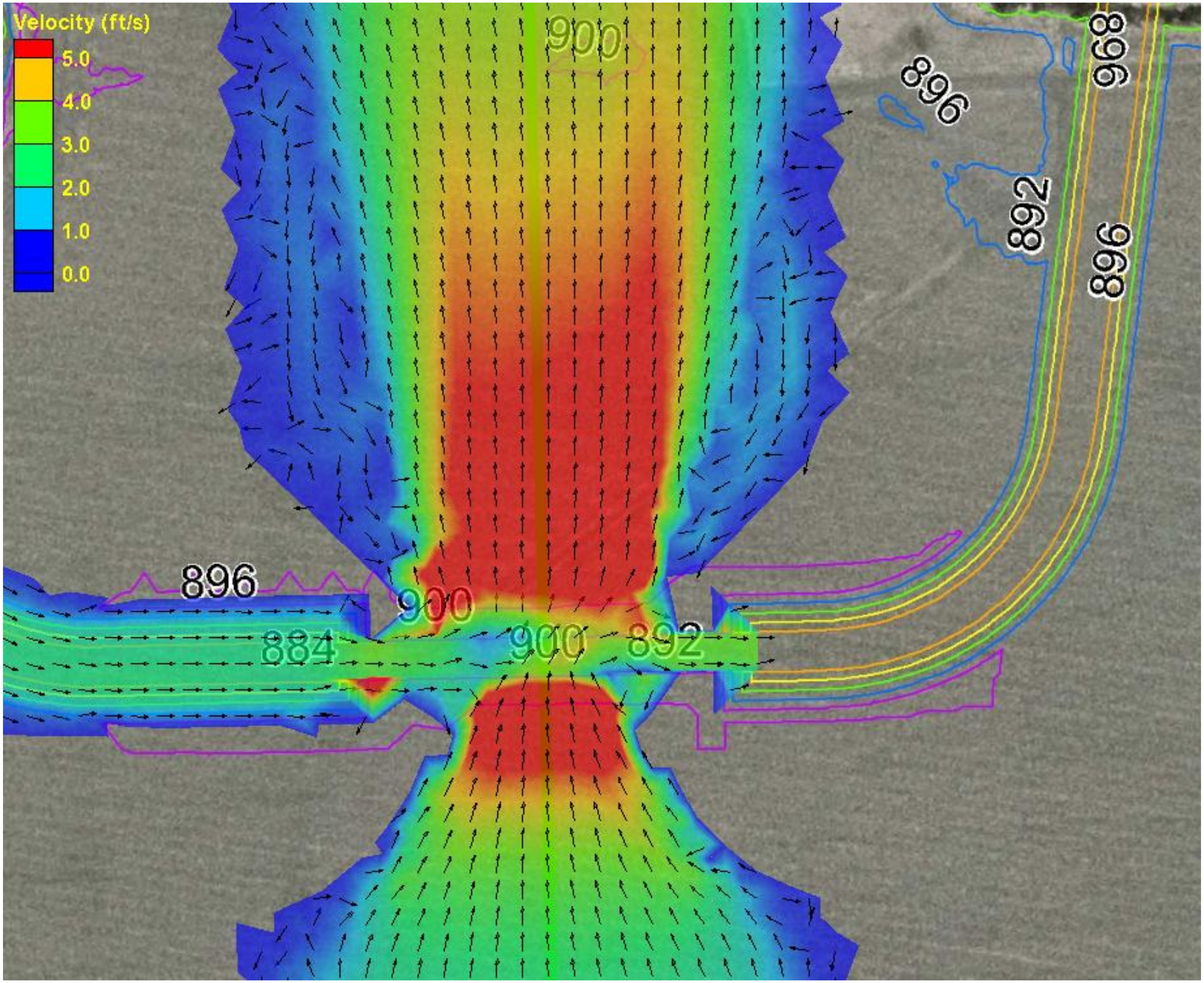

Figure F-H75 Velocity Field for Maple River Diversion Channel Post-construction Conditions 100-Year Event Flow

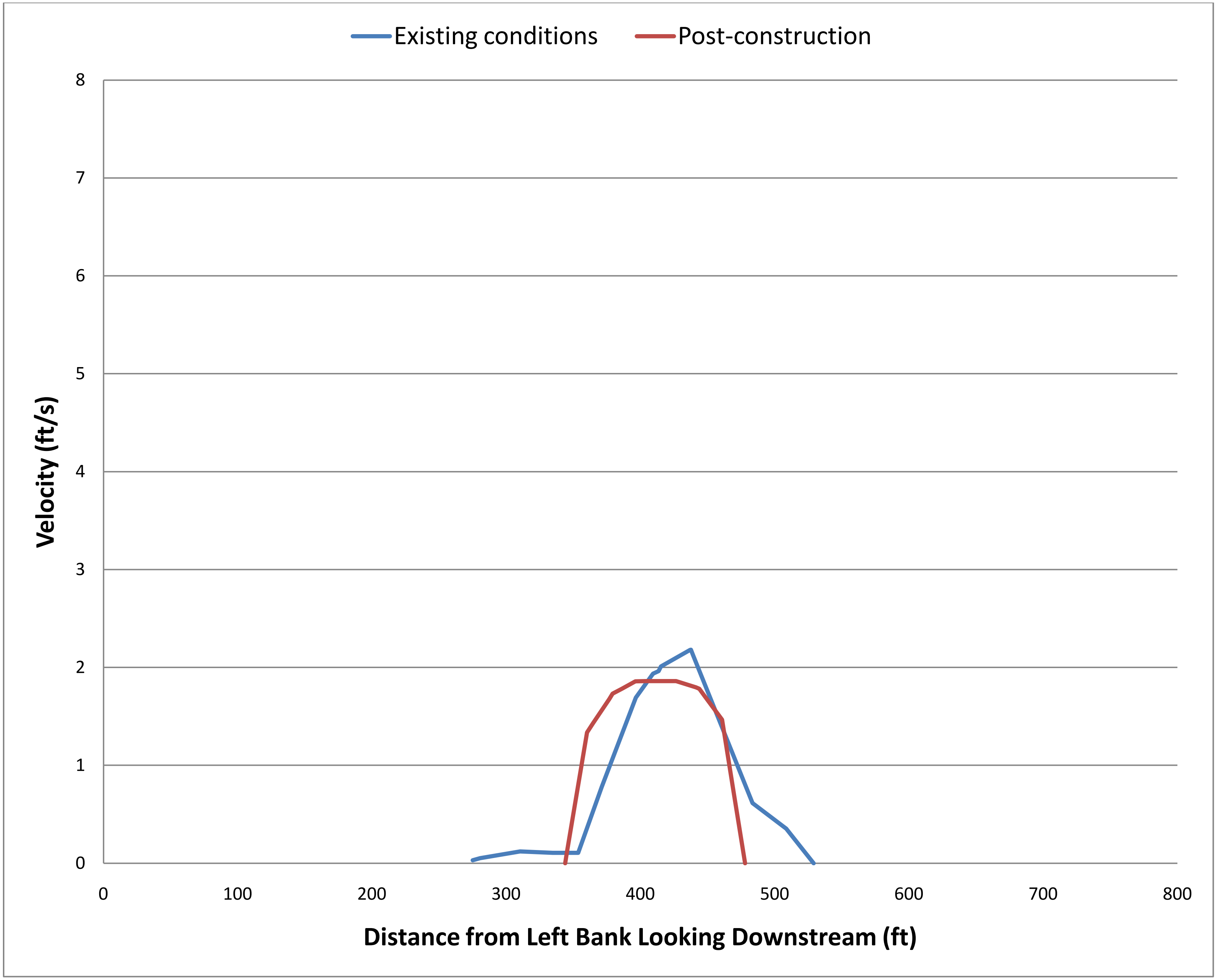

Figure F-H76 Cross Sectional Velocity Data for Maple River Aqueduct Upstream During 2-Year Event

Cross Sectional Velocity Data for Maple River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction upstream of structure.

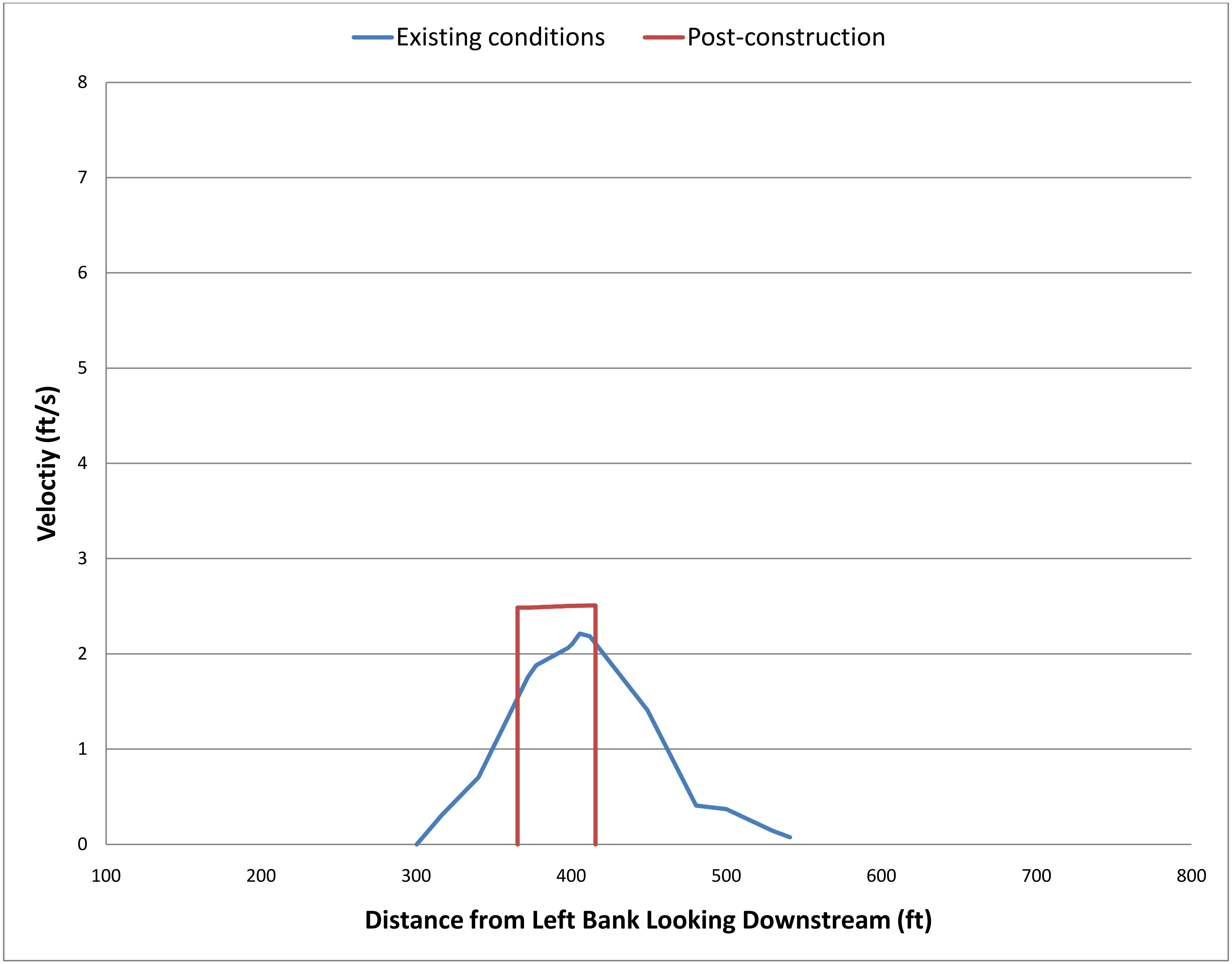

Figure F-H77 Cross Sectional Velocity Data for Maple River Aqueduct Through Structure 2-Year Event

Cross Sectional Velocity data for Maple River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction through structure.

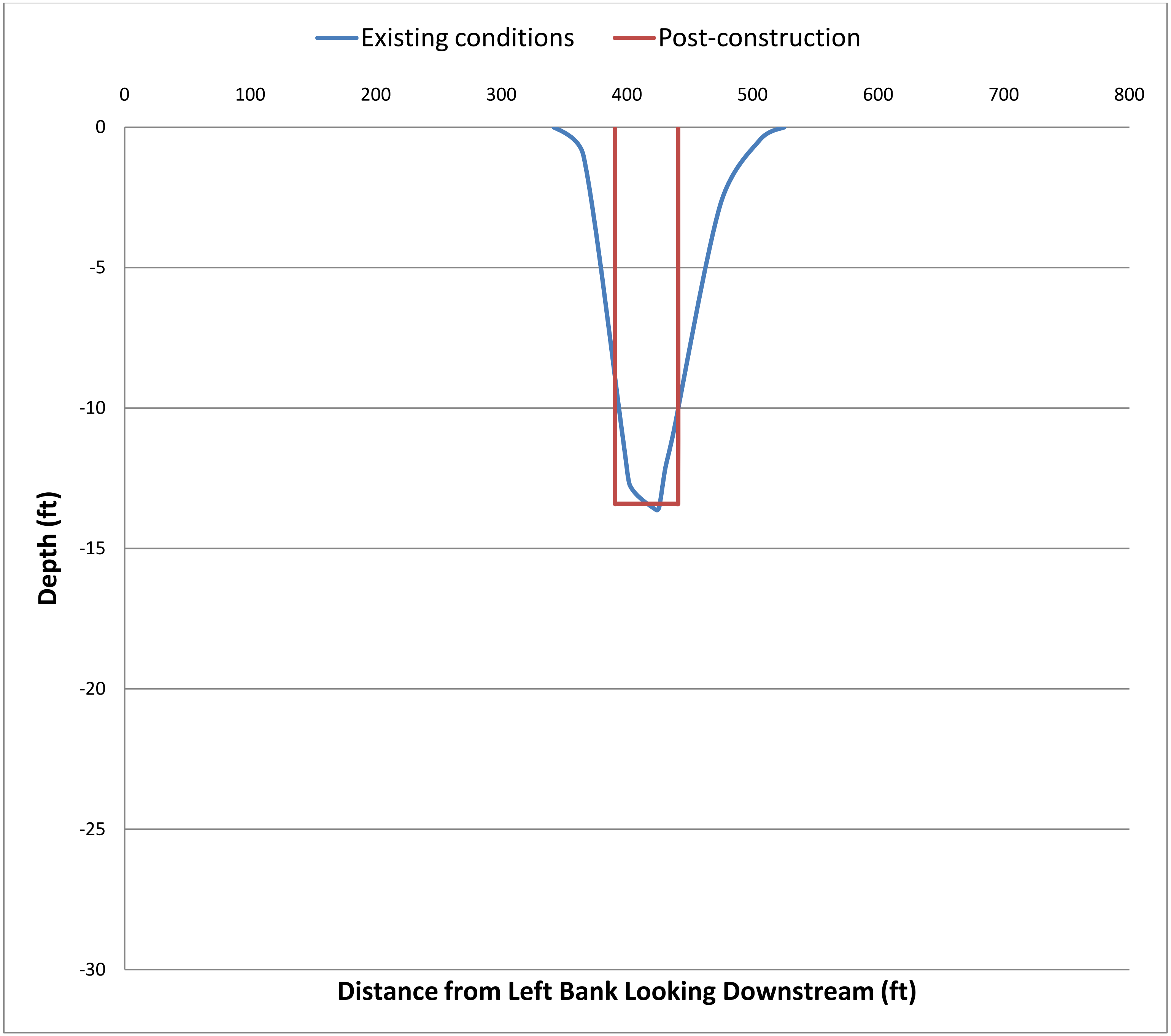

Figure F-H78 Cross Sectional Depth Data for Maple River Aqueduct Through Structure 2-Year Event

Cross Sectional Depth Data for Maple River Aqueduct (2-Year). Compares depth at existing conditions to post-construction through structure.

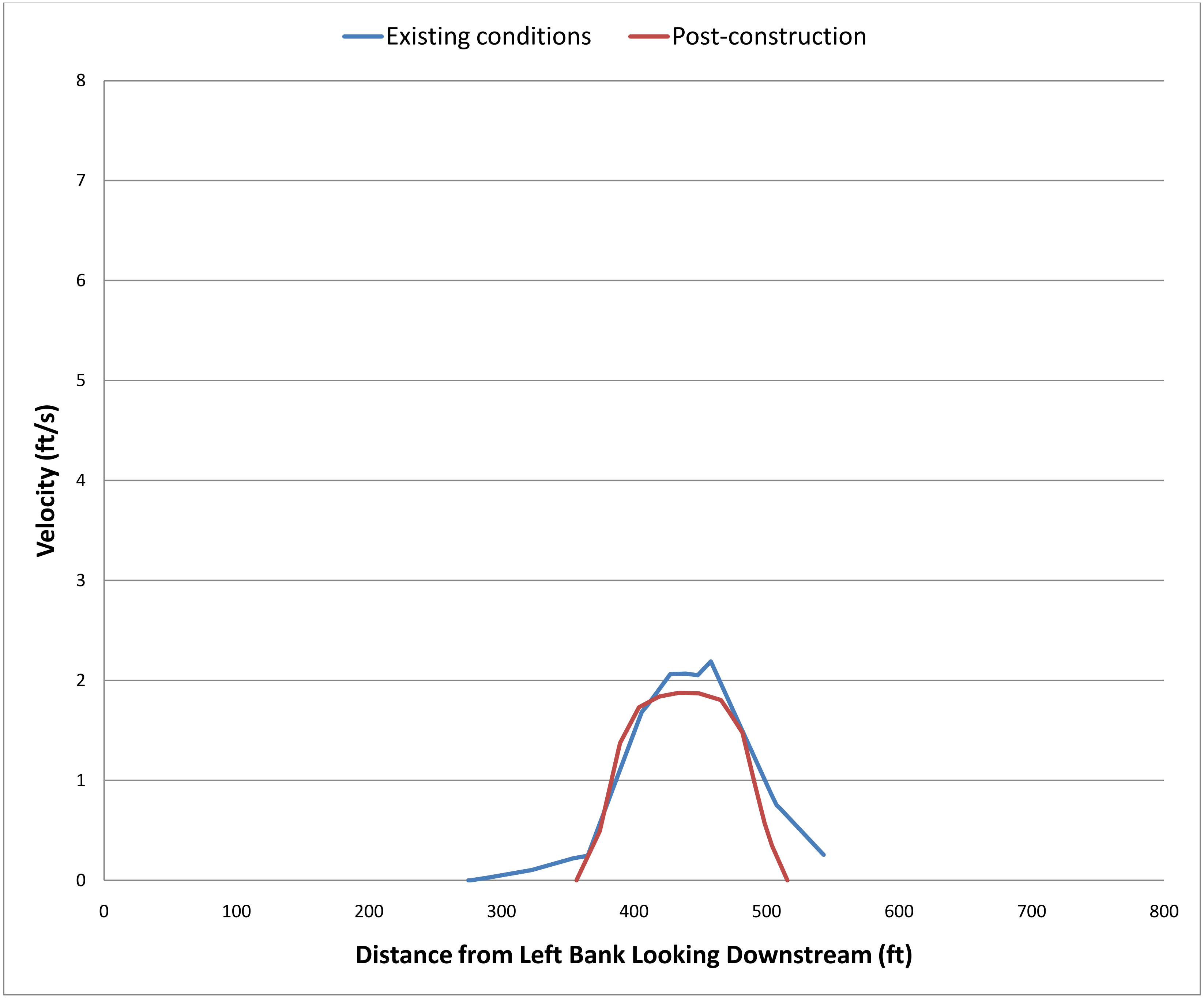

Figure F-H79 Cross Sectional Velocity Data for Maple River Aqueduct Downstream During 2-Year Event

Cross Sectional Velocity Data for Maple River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction through structure.

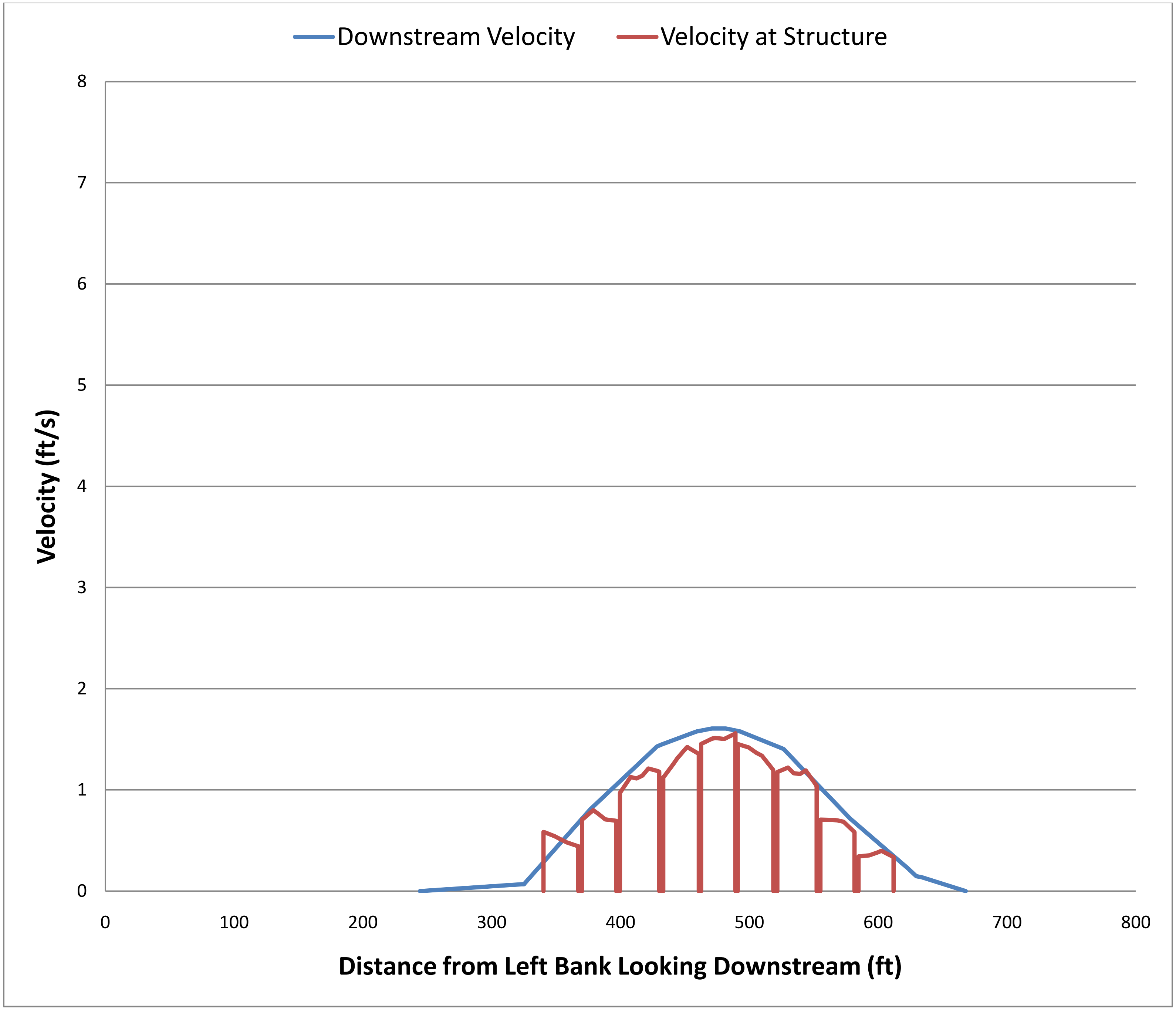

Figure F-H80 Cross Sectional Velocity Data for Maple River Diversion Channel and Downstream During 2,000 cfs Event

Cross Sectional Velocity Data for Maple River Aqueduct (2,000 cfs). Compares velocity at structure to downstream.

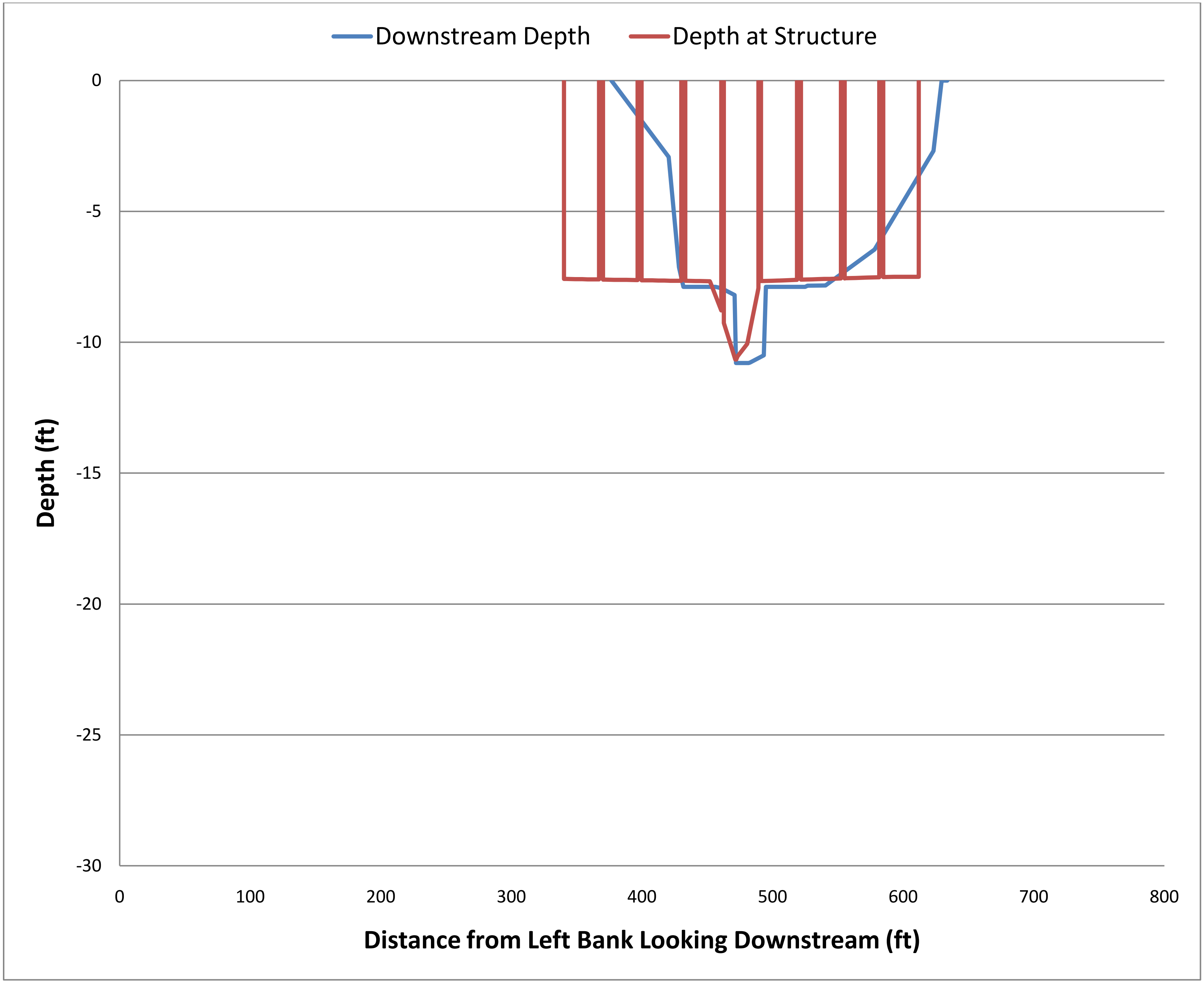

Figure F-H81 Cross Sectional Depth Data for Maple River Diversion Channel and Downstream During 2,000 cfs Event

Cross Sectional Depth Data for Maple River Aqueduct (2,000 cfs). Compares depth at structure to downstream.

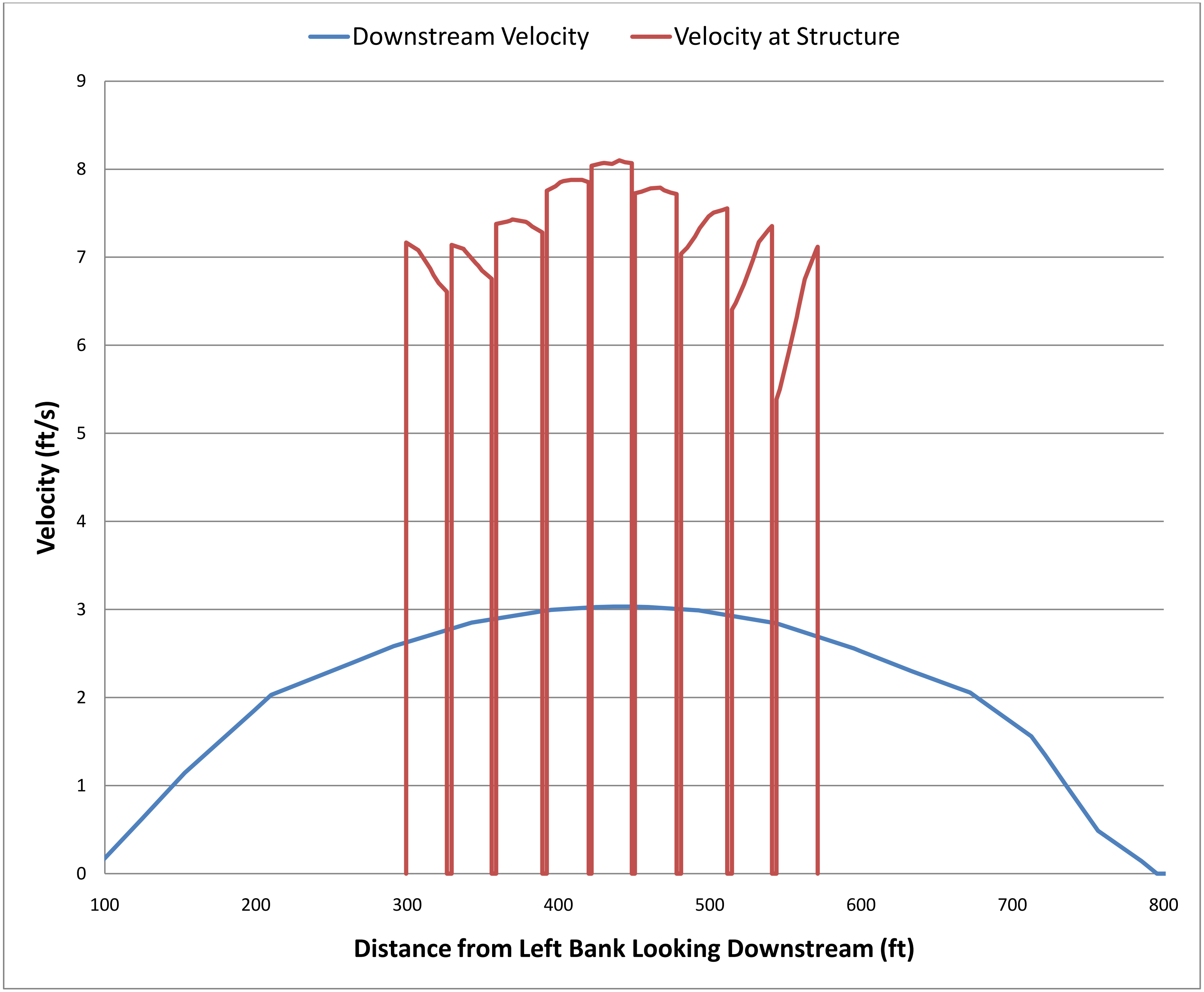

Figure F-H82 Cross Sectional Velocity Data for Maple River Diversion Channel and Downstream During 50-Year Event

Cross Sectional Velocity Data for Maple River Aqueduct (50-Year). Compares velocity at structure to downstream.

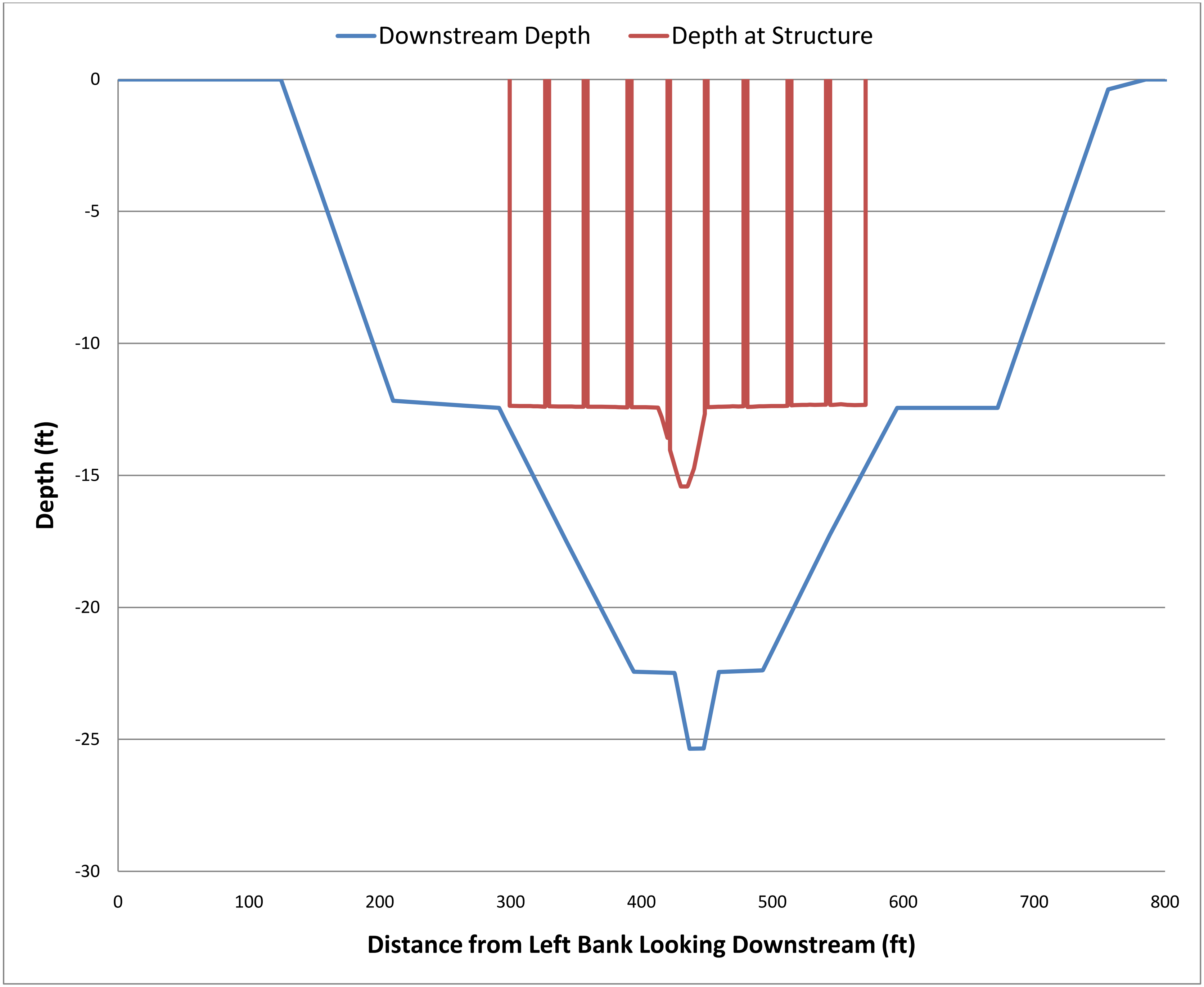

Figure F-H83 Cross Sectional Depth Data for Maple River Diversion Channel and Downstream During 50-Year Event

Cross Sectional Depth Data for Maple River Aqueduct (50-Year). Compares depth at structure to downstream.

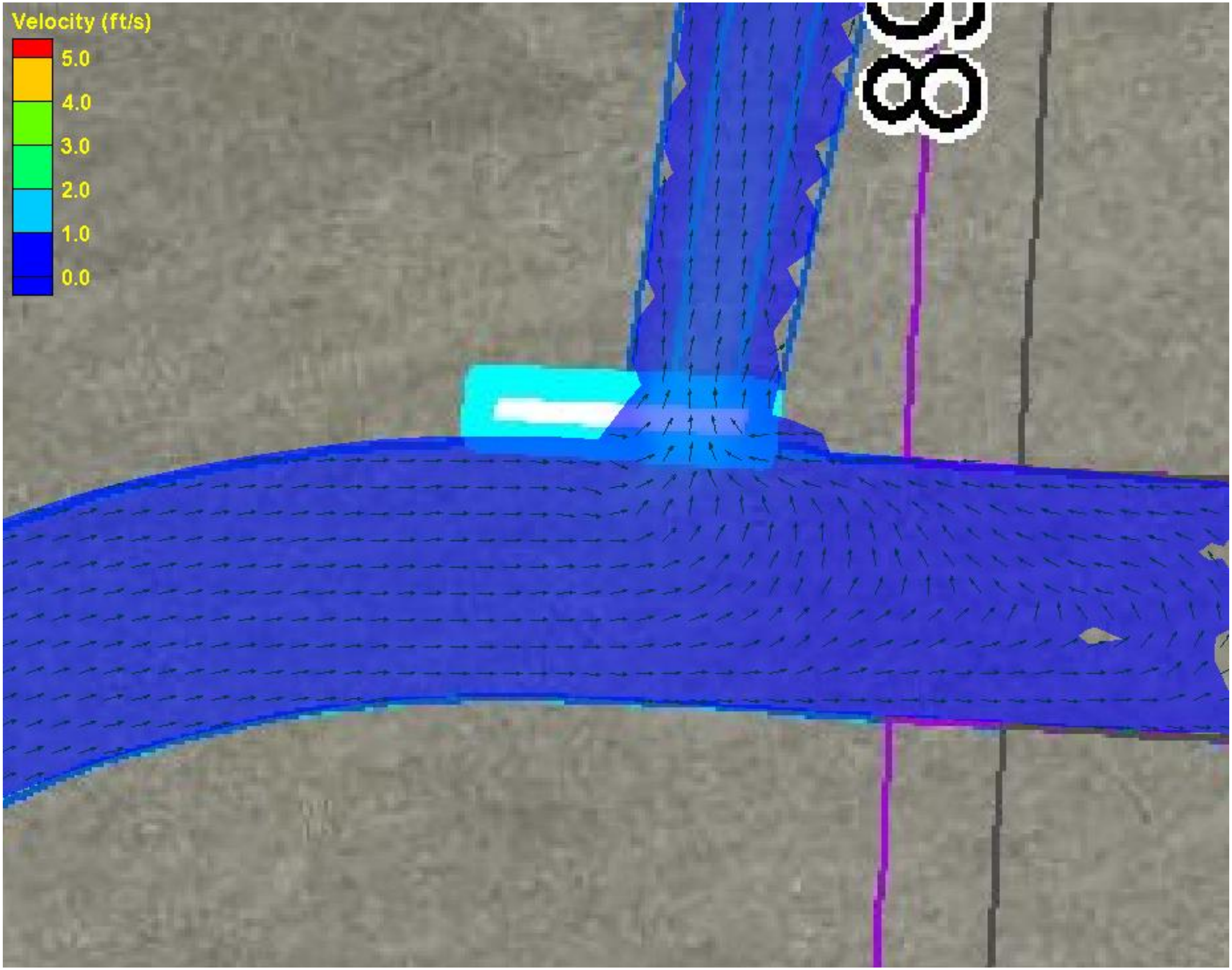

Figure F-H84 Velocity Field for Lower Rush River Structure Post-construction Conditions Mean Annual Flow Event

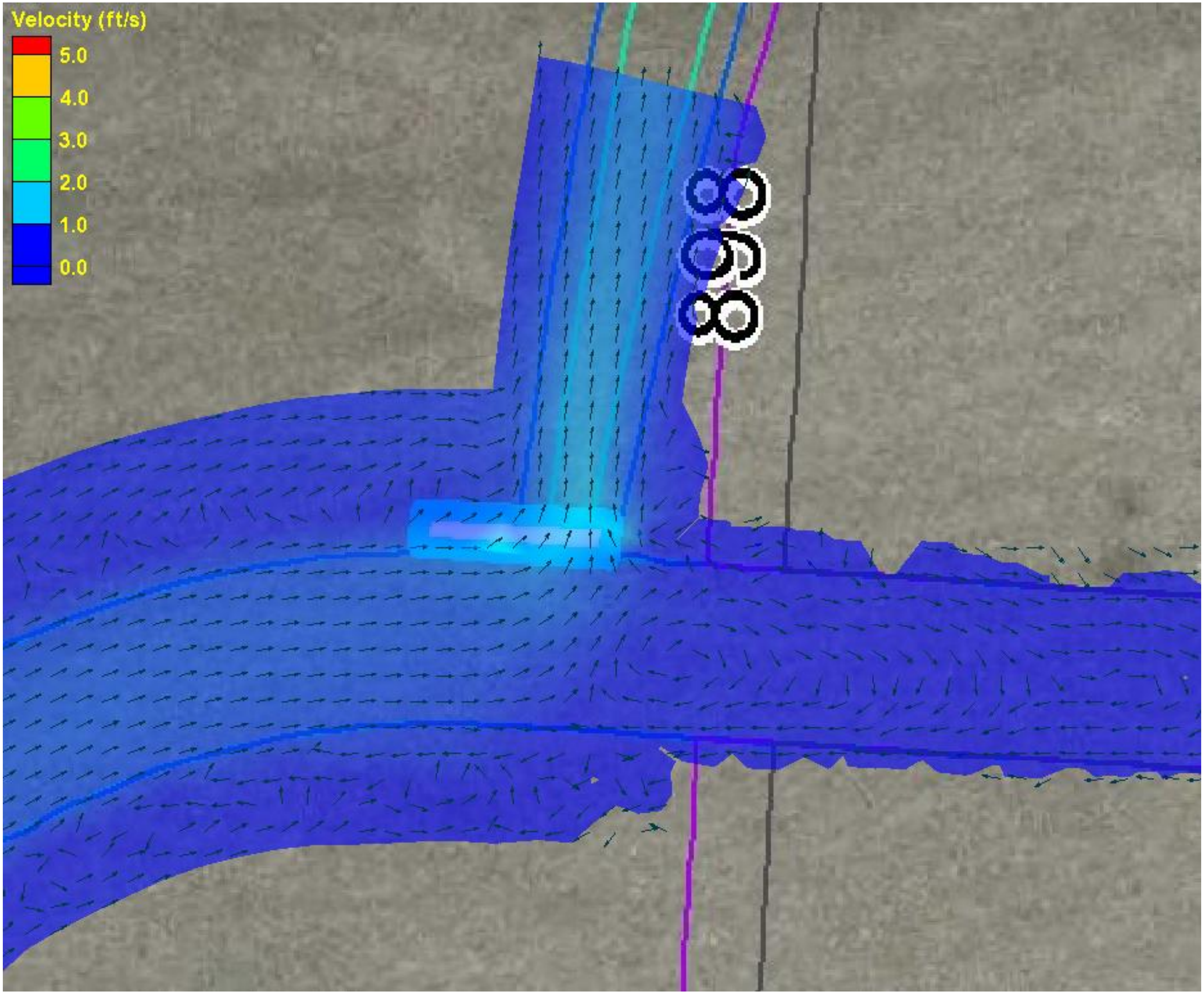

Figure F-H85 Velocity Field for Lower Rush River Structure Post-construction Conditions 20-Year Event Flow

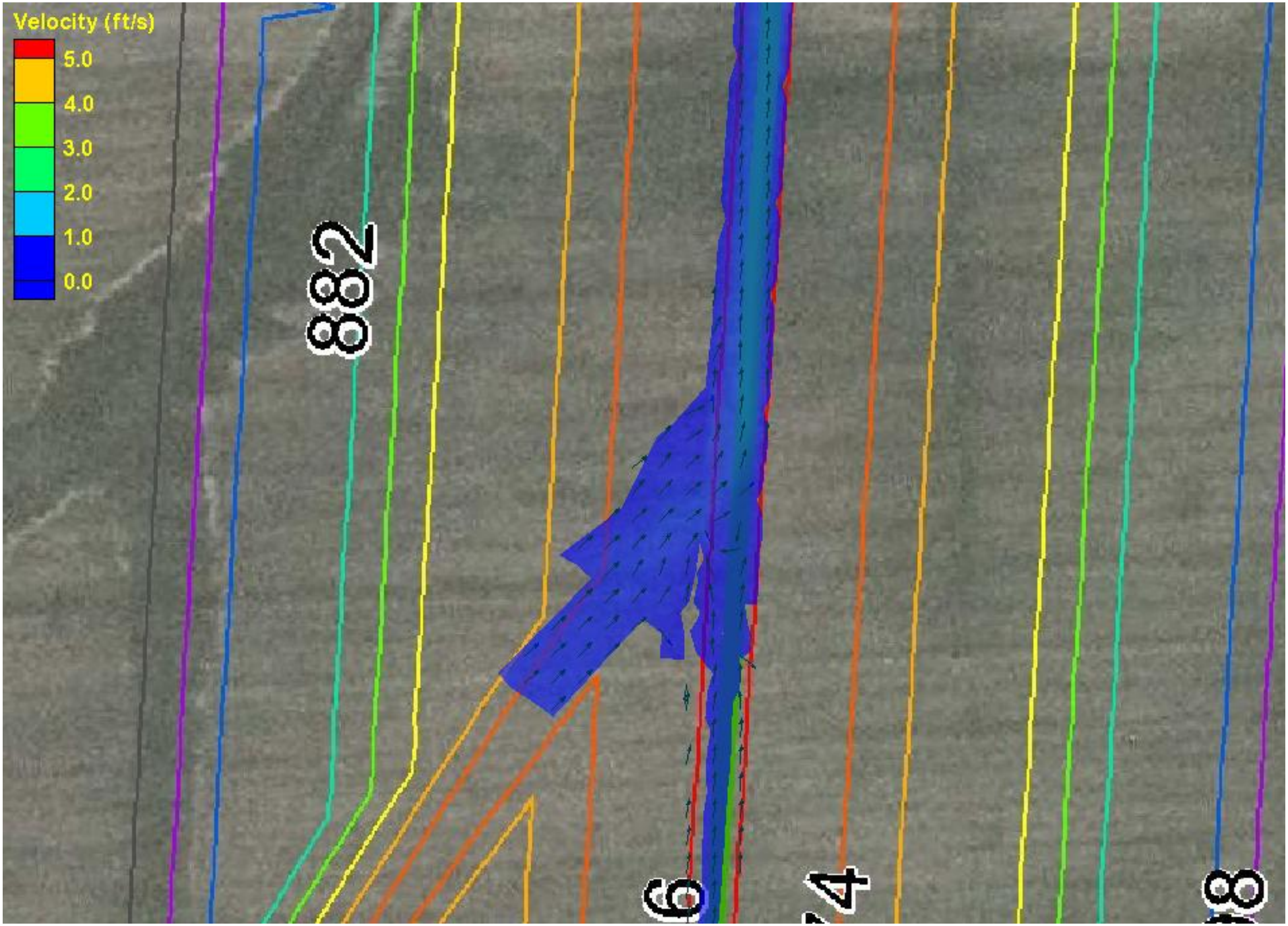

Figure F-H86 Velocity Field for Lower Rush River Diversion Channel Post-construction Conditions Mean Annual Flow Event

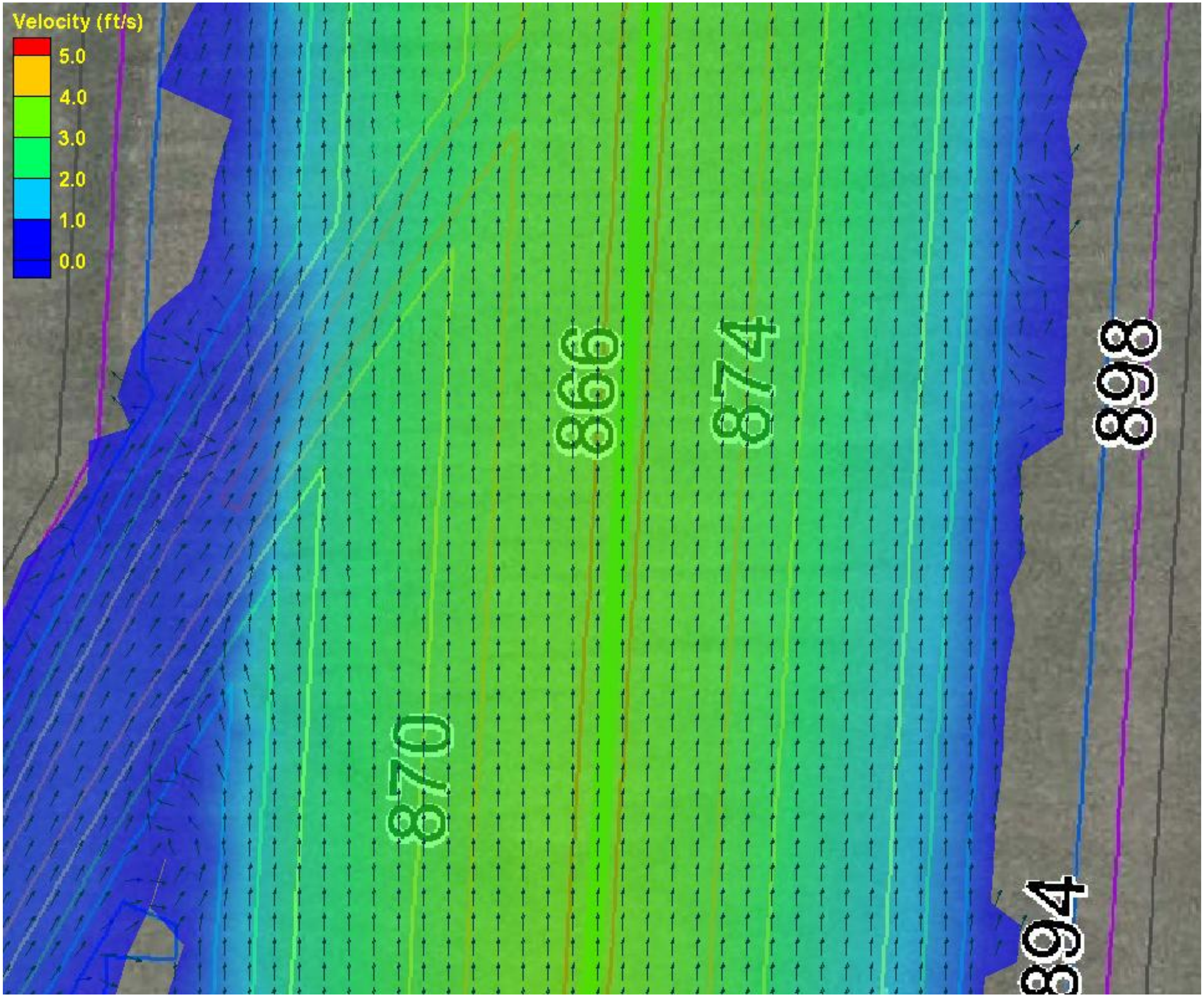

Figure F-H87 Velocity Field for Lower Rush River Diversion Channel Post-construction Conditions 20-Year Event Flow

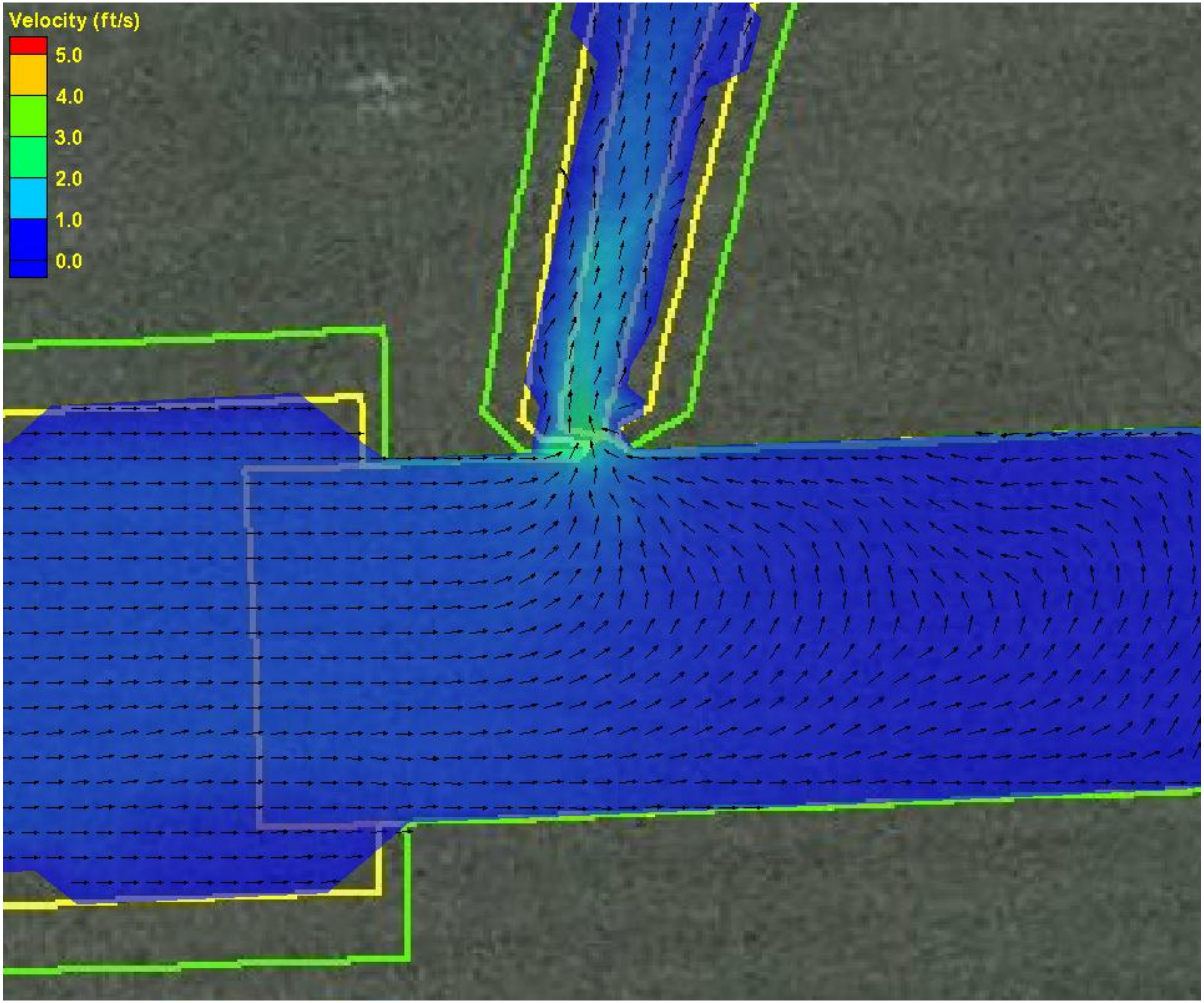

Figure F-H88 Velocity Field for Rush River Structure Post-construction Conditions Mean Annual Flow Event

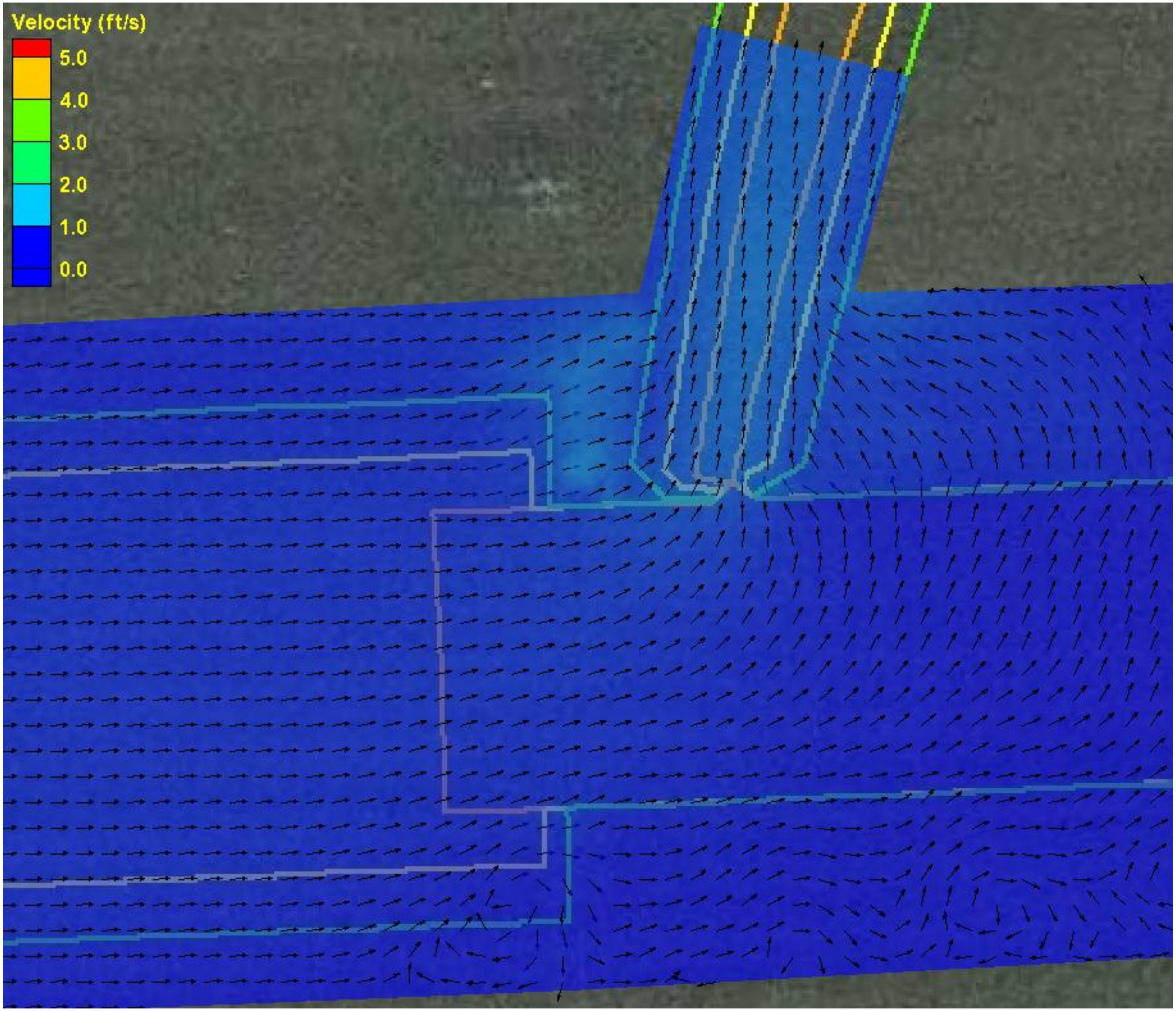

Figure F-H89 Velocity Field for Rush River Structure Post-construction Conditions 20-Year Event Flow

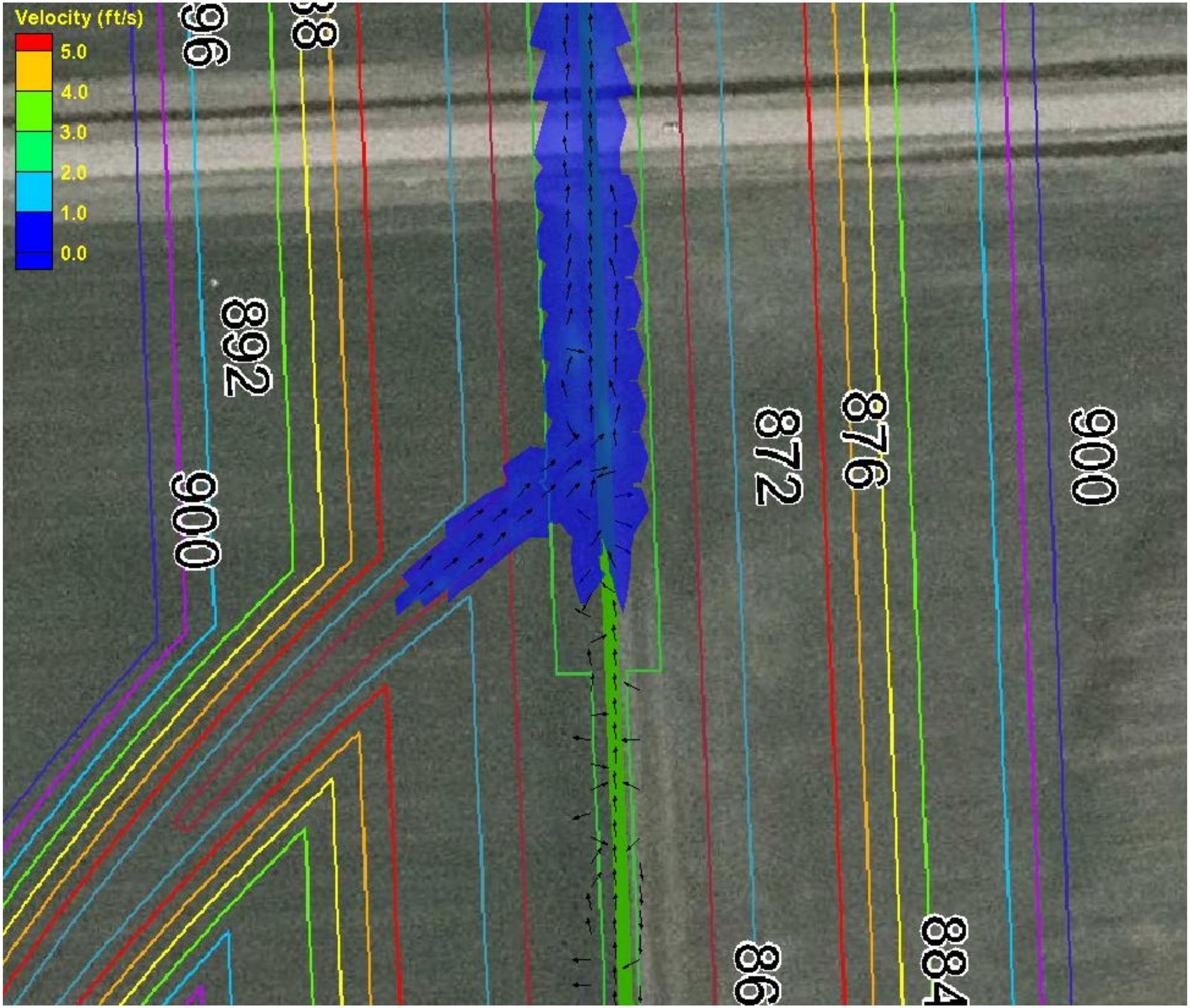

Figure F-H90 Velocity Field for Rush River Diversion Channel Post-construction Conditions Mean Annual Flow Event

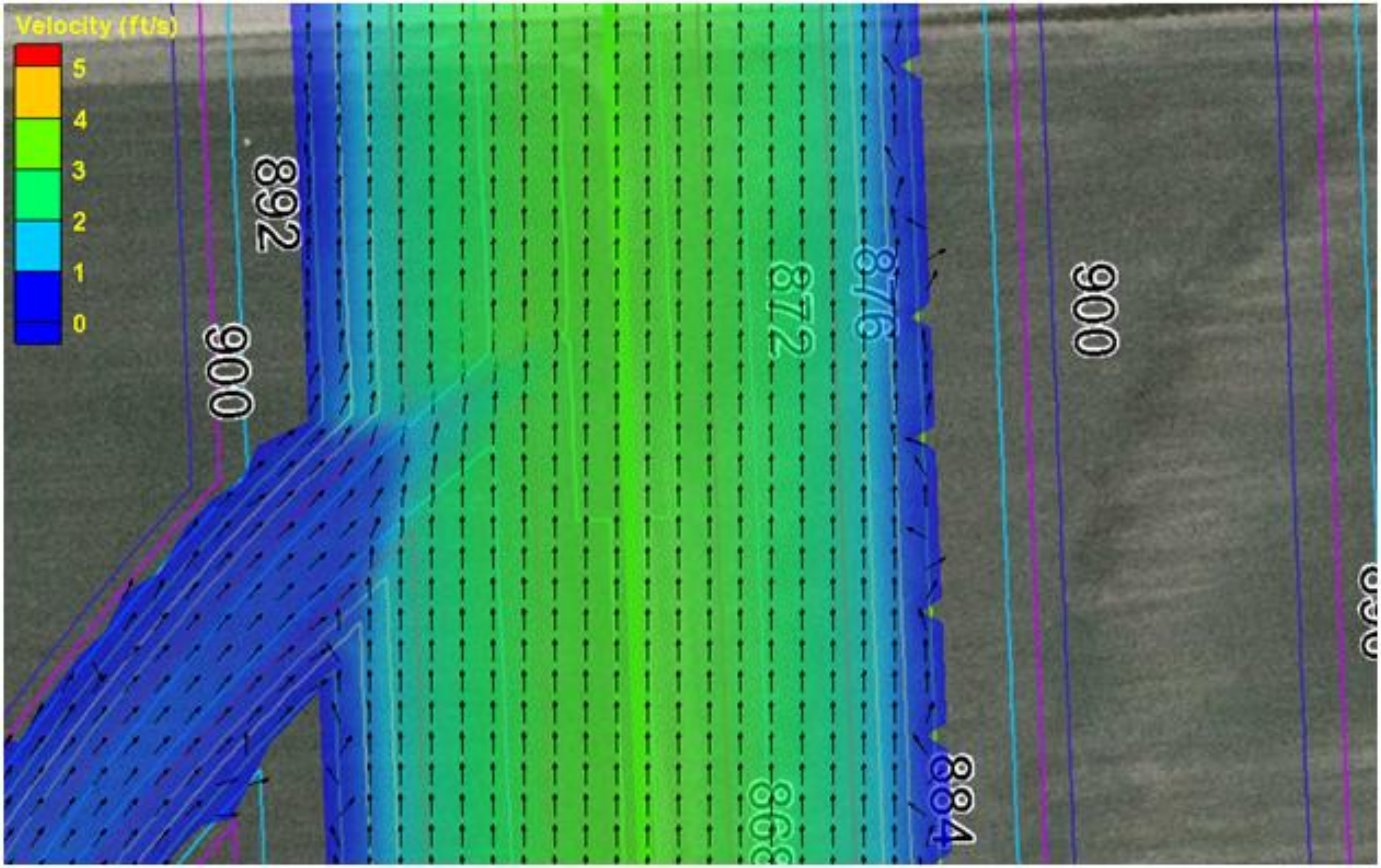

Figure F-H91 Velocity Field for Rush River Diversion Channel Post-construction Conditions 20-Year Event Flow

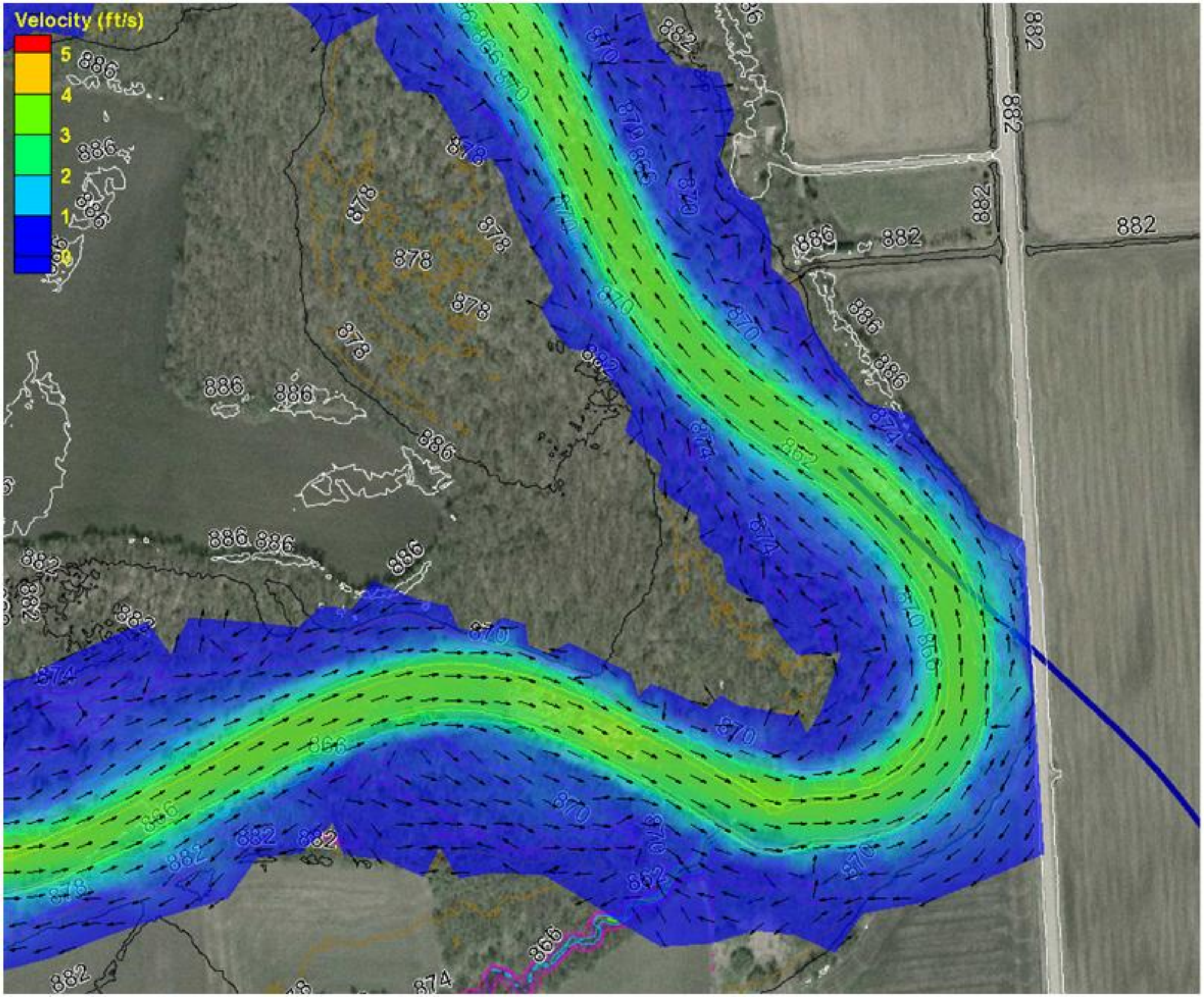

Figure F-H92 Velocity Field for Minnesota Outlet to Red River of the North Existing Conditions 2-Year Event Flow

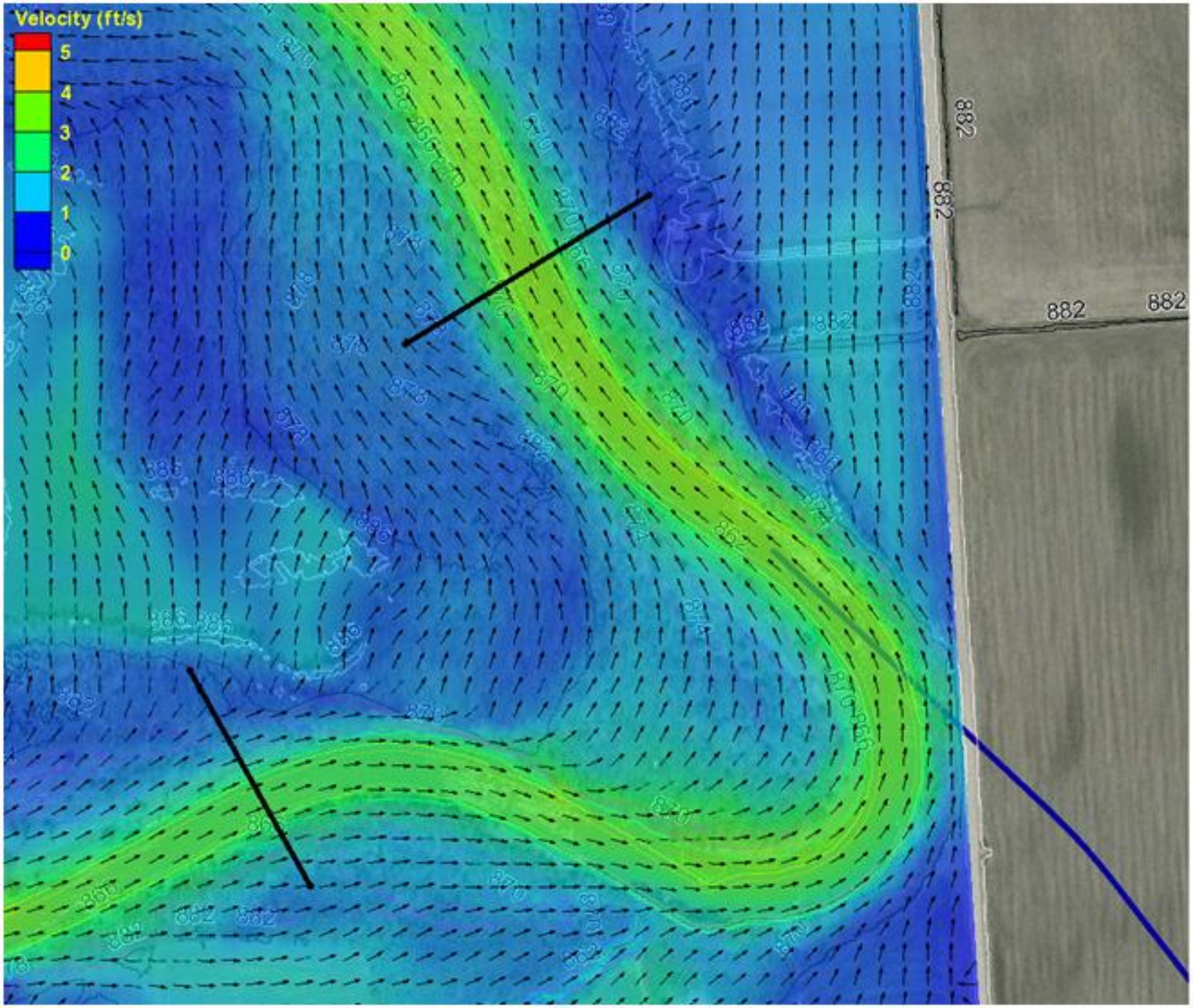

Figure F-H93 Velocity Field for Minnesota Outlet to Red River of the North Existing Conditions 20-Year Event Flow

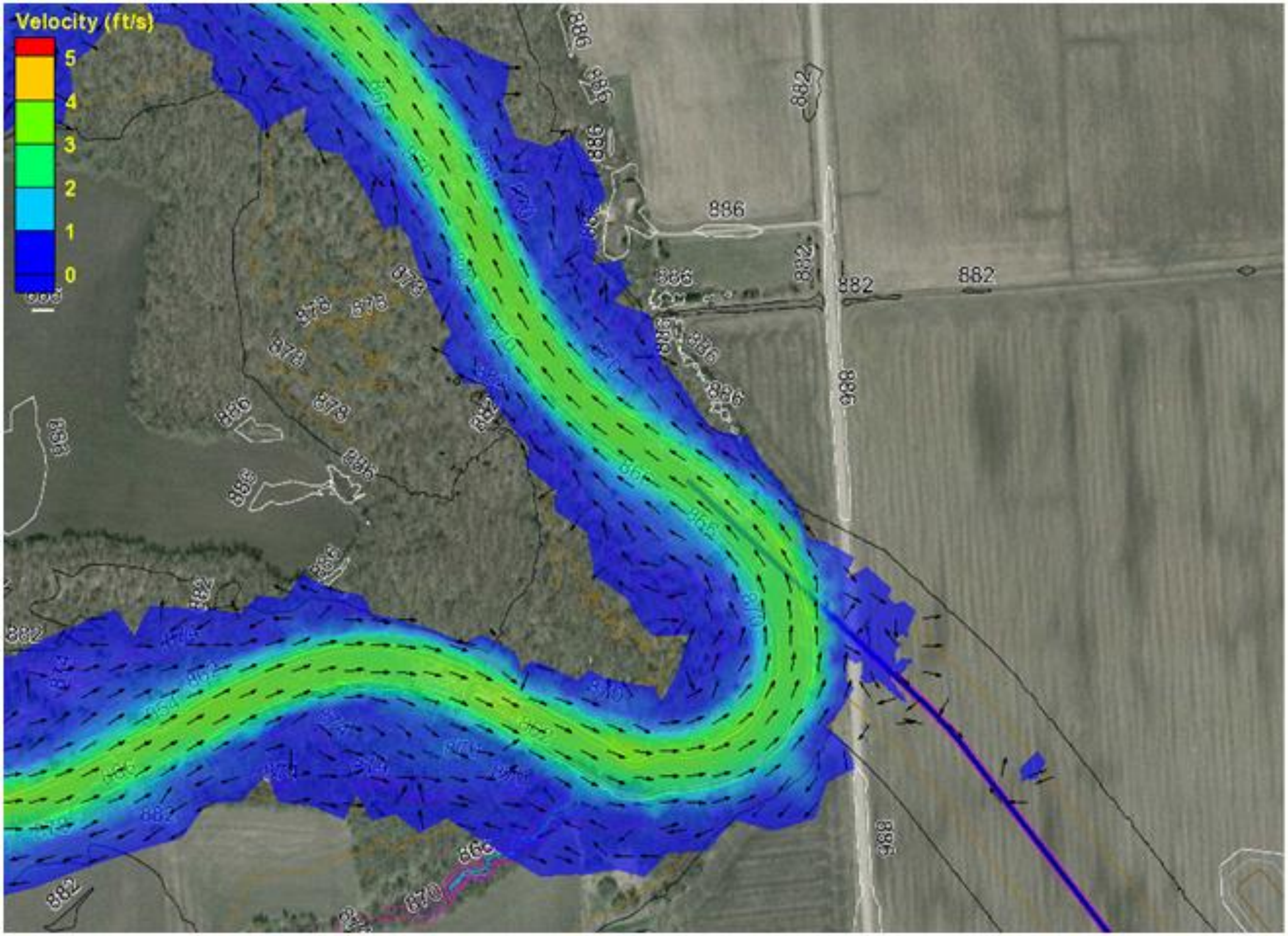

Figure F-H94 Velocity Field for Minnesota Outlet to Red River of the North Post-construction Conditions 2-Year Event Flow

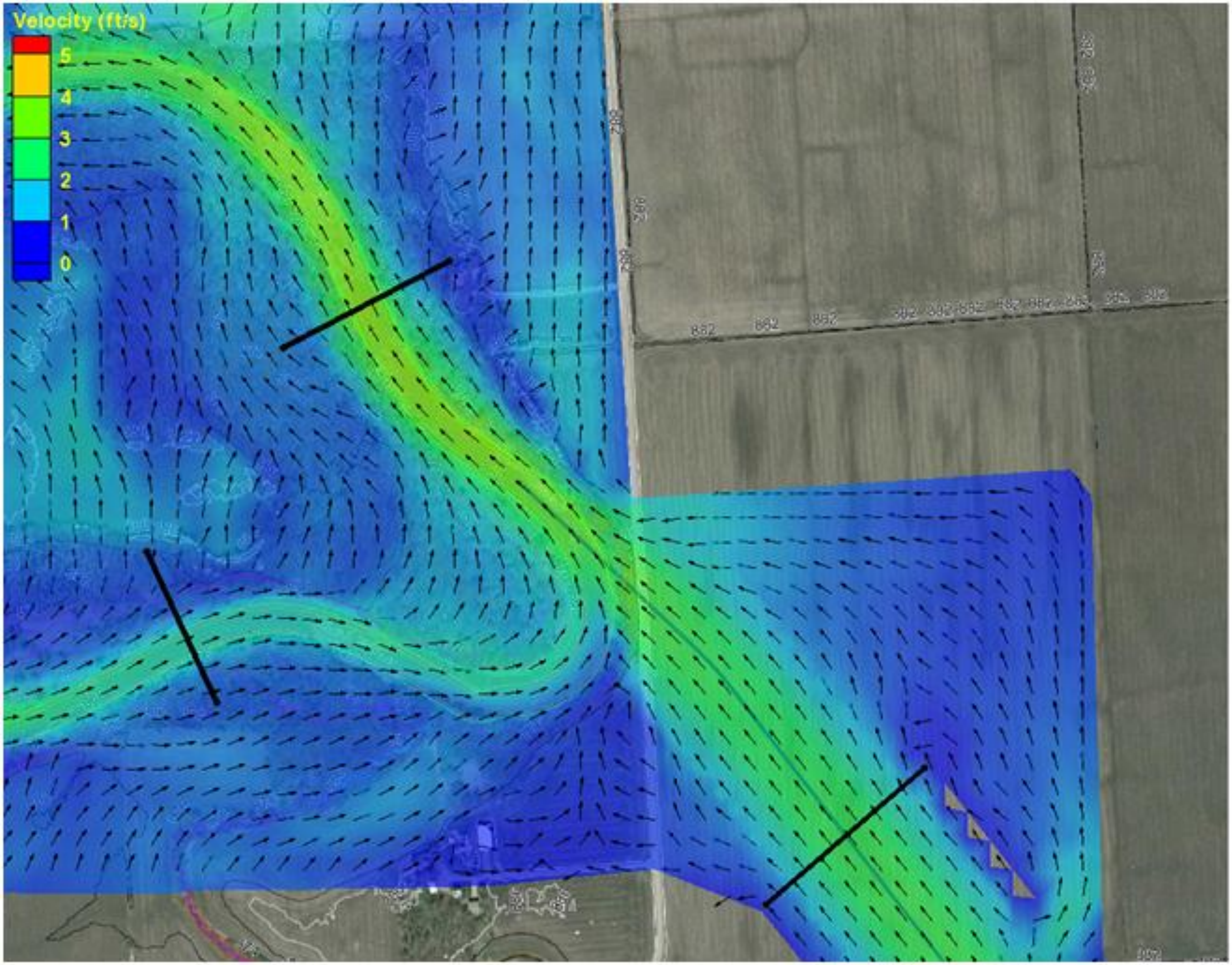

Figure F-H95 Velocity Field for Minnesota Outlet to Red River of the North Post-construction Conditions 20-Year Event Flow

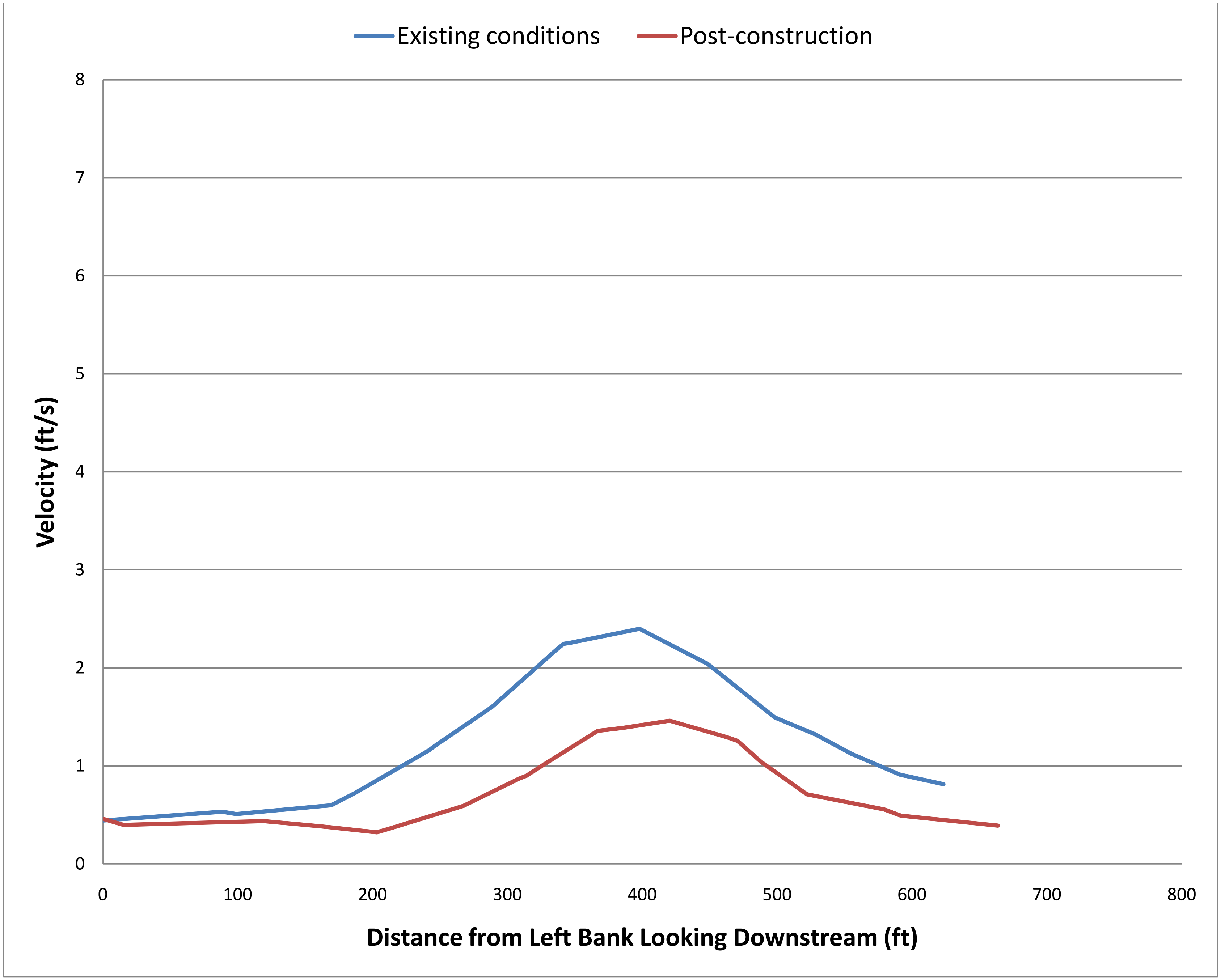

Figure F-H96 Cross Sectional Velocity Data for Minnesota Outlet to Red River of the North Upstream During 20-Year Event

Cross Sectional Velocity Data for Minnesota Outlet (20-Year). Compares velocity at existing conditions to post-construction upstream of structure.

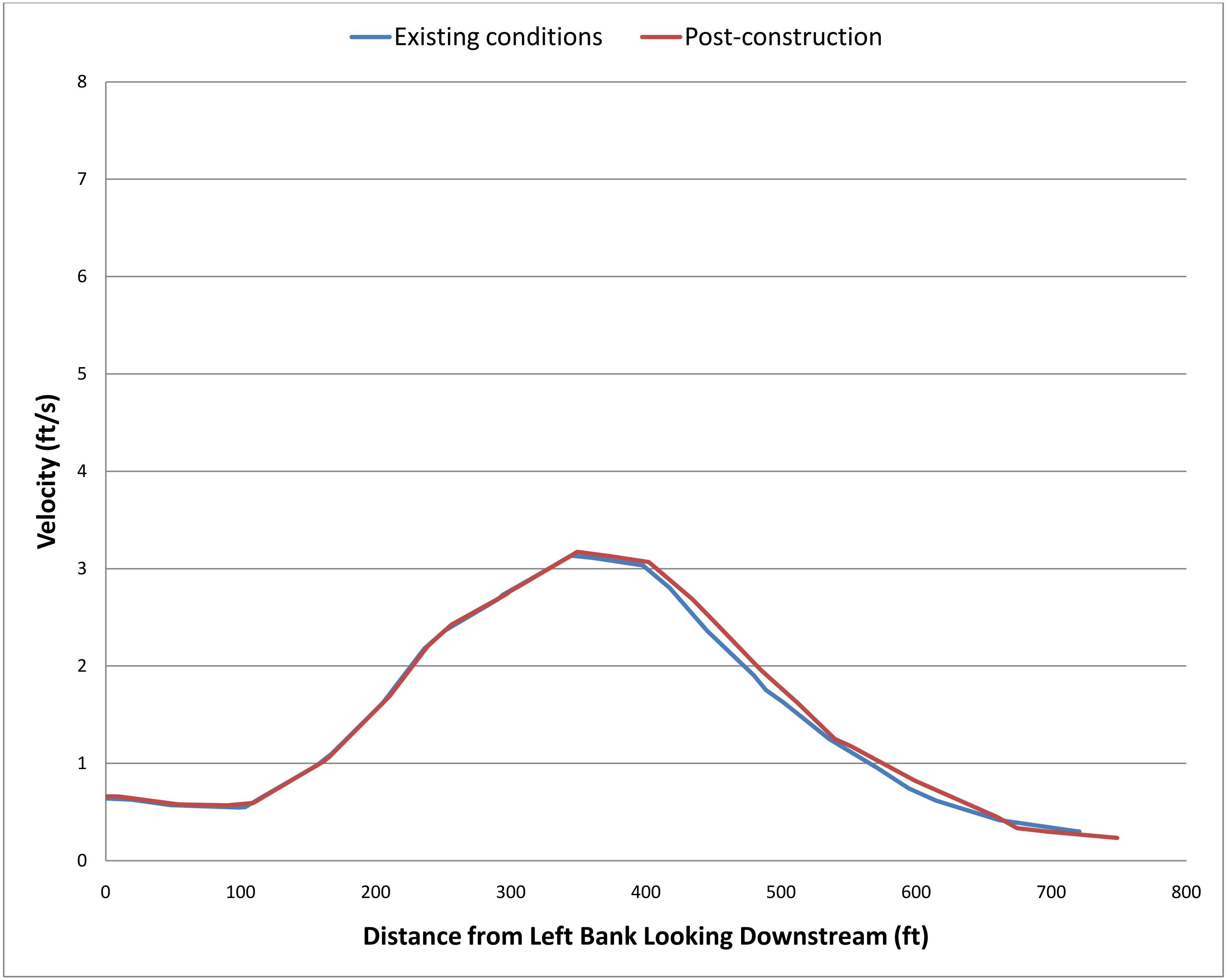

Figure F-H97 Cross Sectional Velocity Data for Minnesota Outlet to Red River of the North Downstream During 20-Year Event

Cross Sectional Velocity Data for Minnesota Outlet (20-Year). Compares velocity at existing conditions to post-construction downstream of structure.

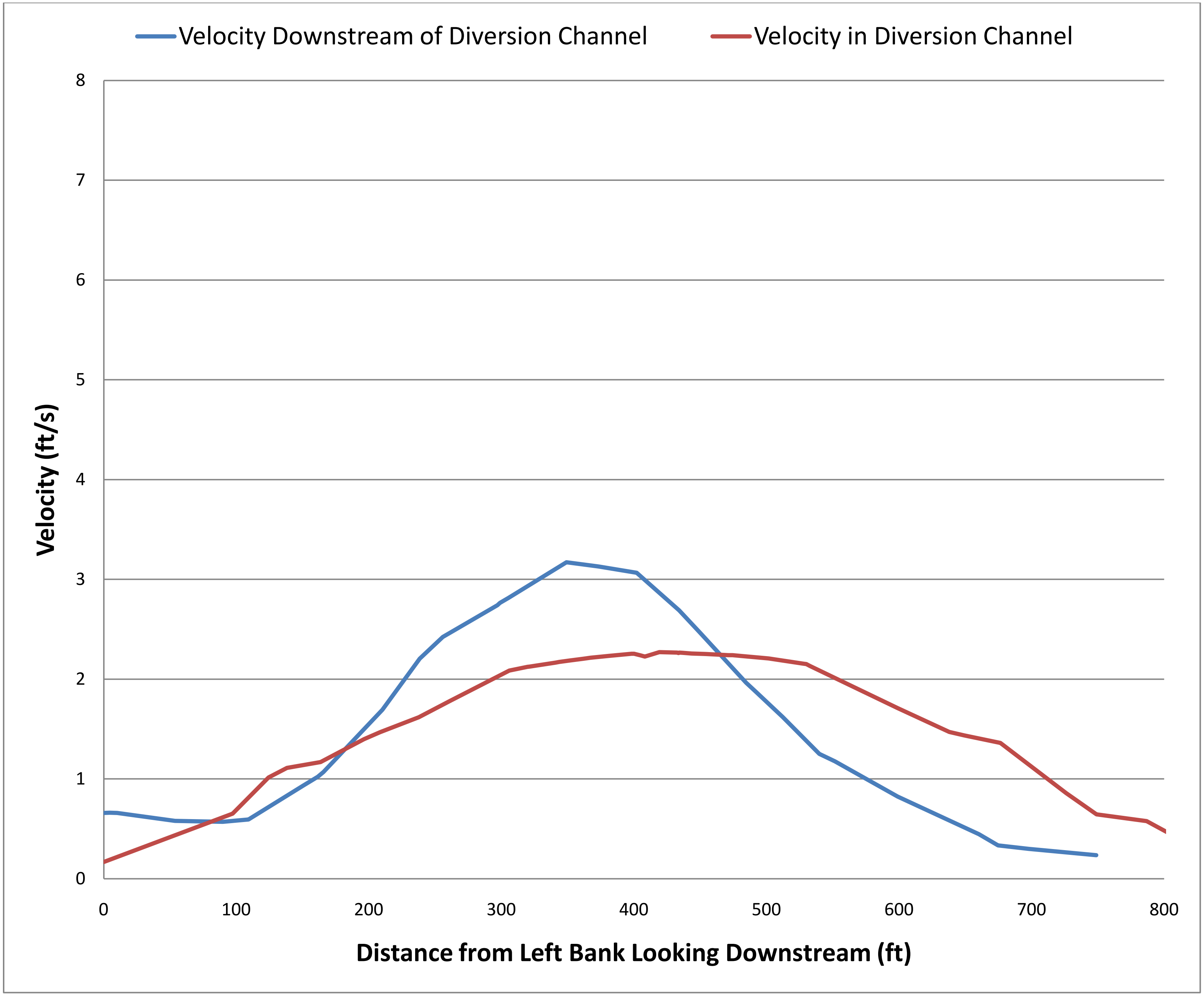

Figure F-H98 Cross Sectional Velocity Data for Minnesota Outlet to Red River of the North Diversion Channel During 20-Year Event

Cross Sectional Velocity Data for Minnesota Outlet (20-Year). Compares velocity in diversion channel to downstream.

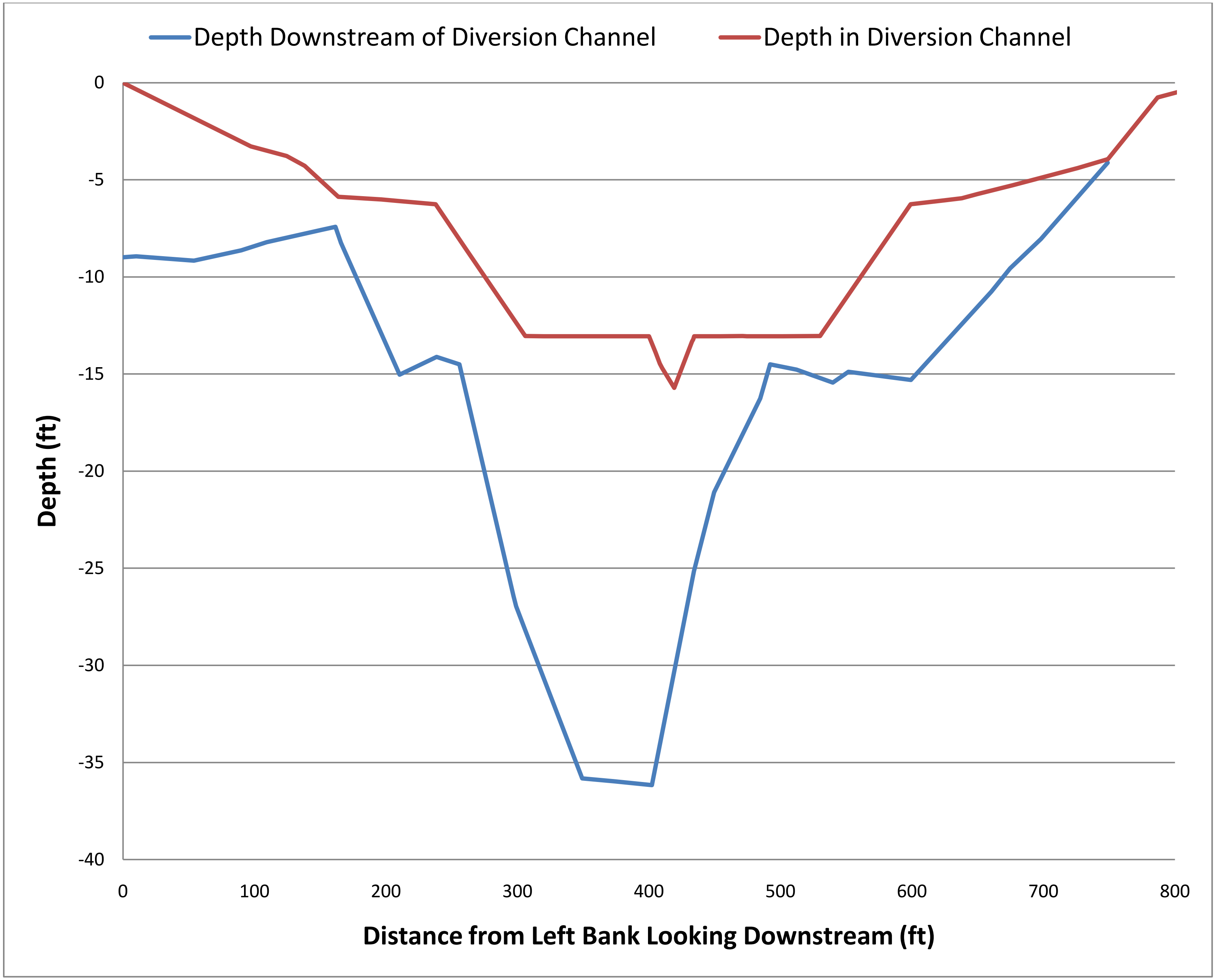

Figure F-H99 Cross Sectional Depth Data for Minnesota Outlet to Red River of the North Diversion Channel During 20-Year Event

Cross Sectional Depth Data for Minnesota Outlet (20-Year). Compares depth in diversion channel to downstream.

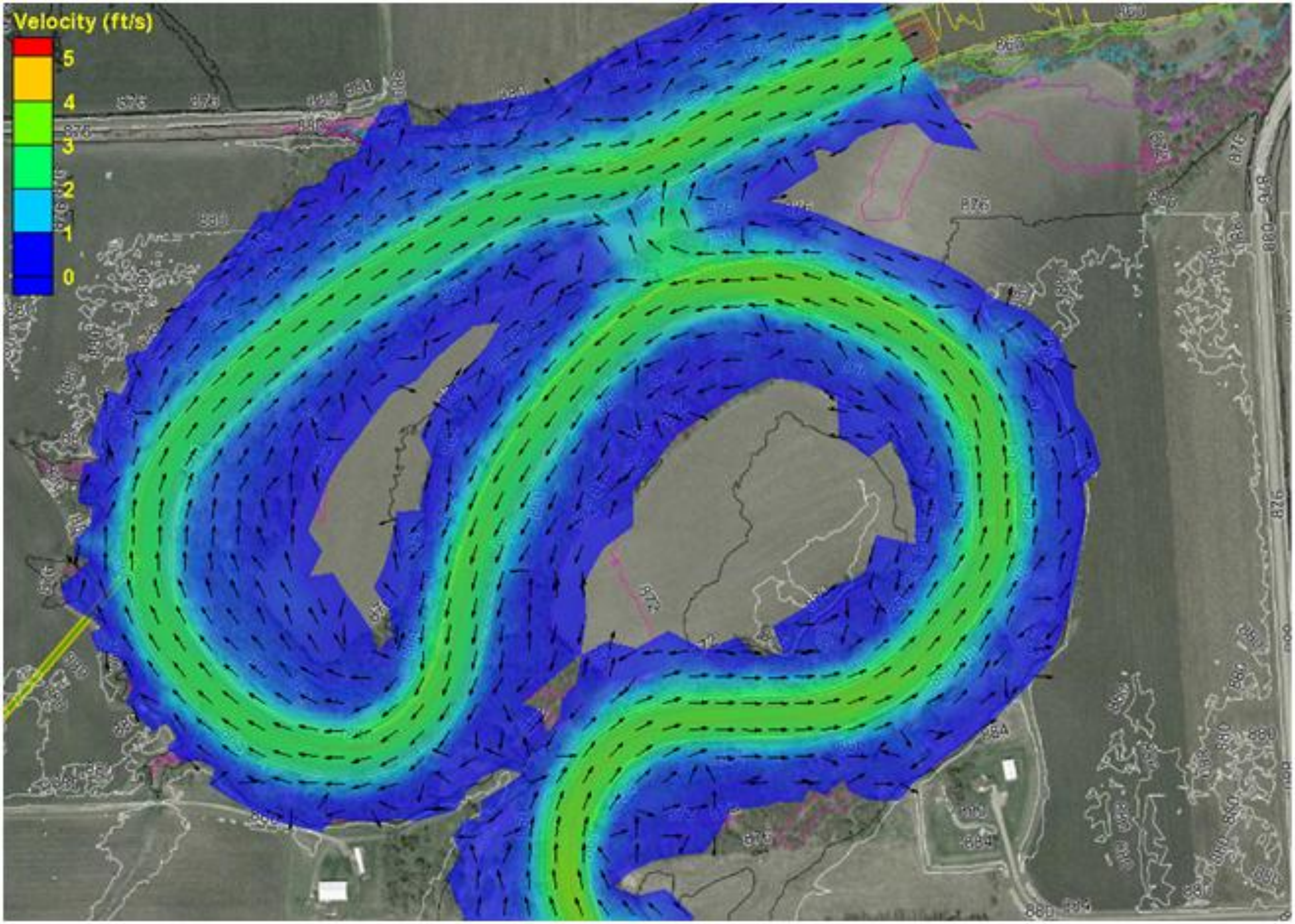

Figure F-H100 Velocity Field for North Dakota Outlet to Red River of the North Existing Conditions 2-Year Event Flow

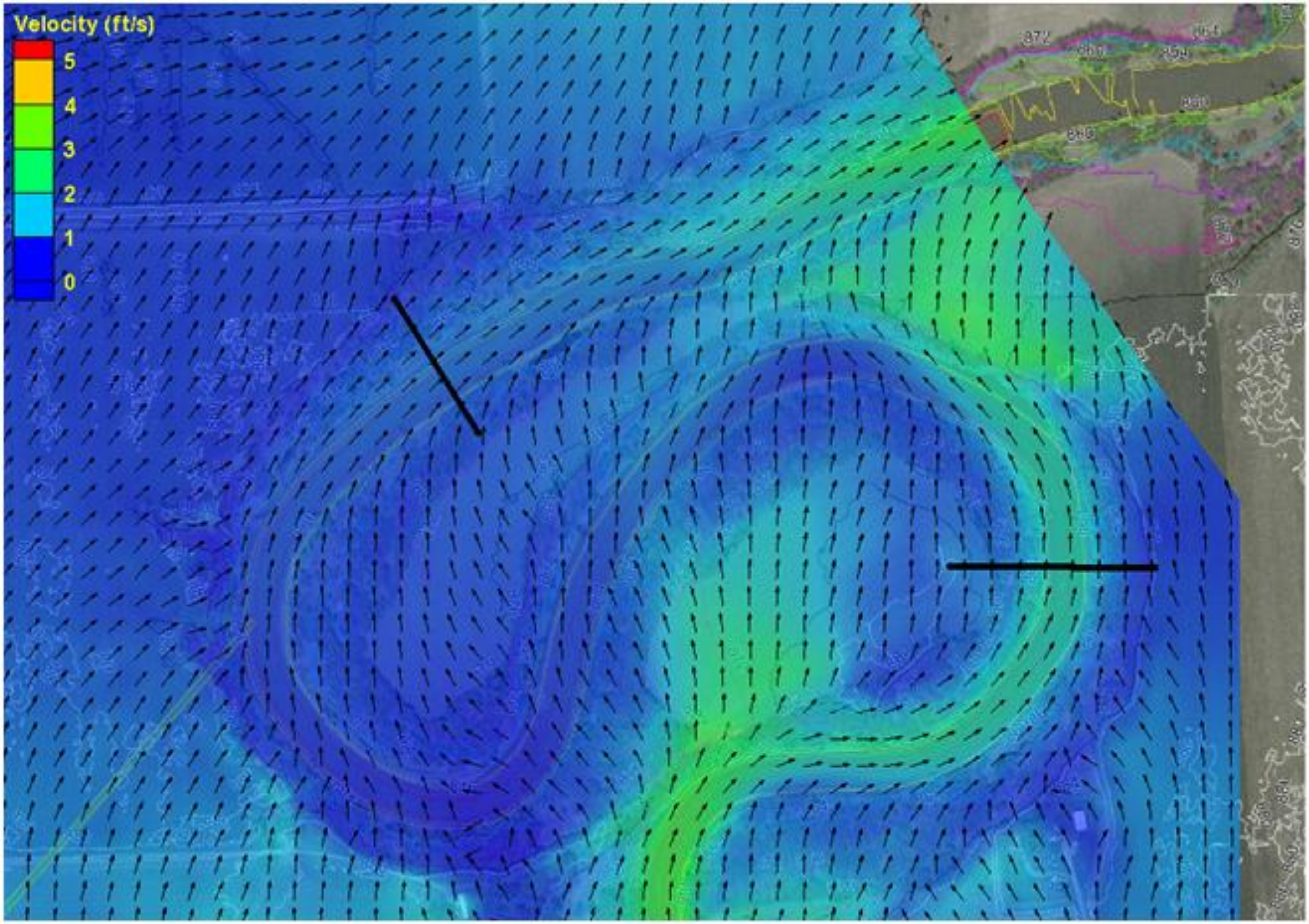

Figure F-H101 Velocity Field for North Dakota Outlet to Red River of the North Existing Conditions 20-Year Event Flow

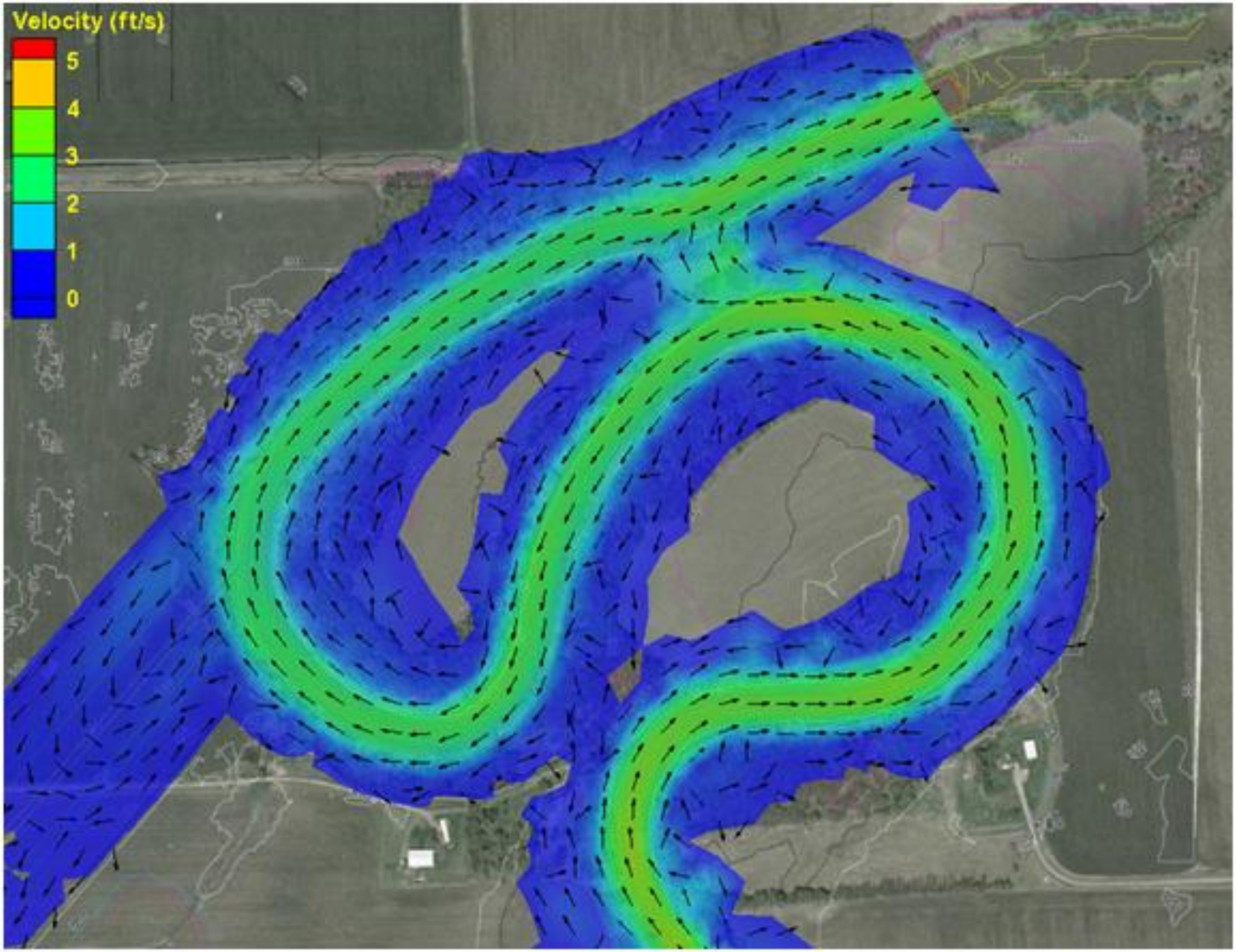

Figure F-H102 Velocity Field for North Dakota Outlet to Red River of the North Post-construction Conditions 2-Year Event Flow

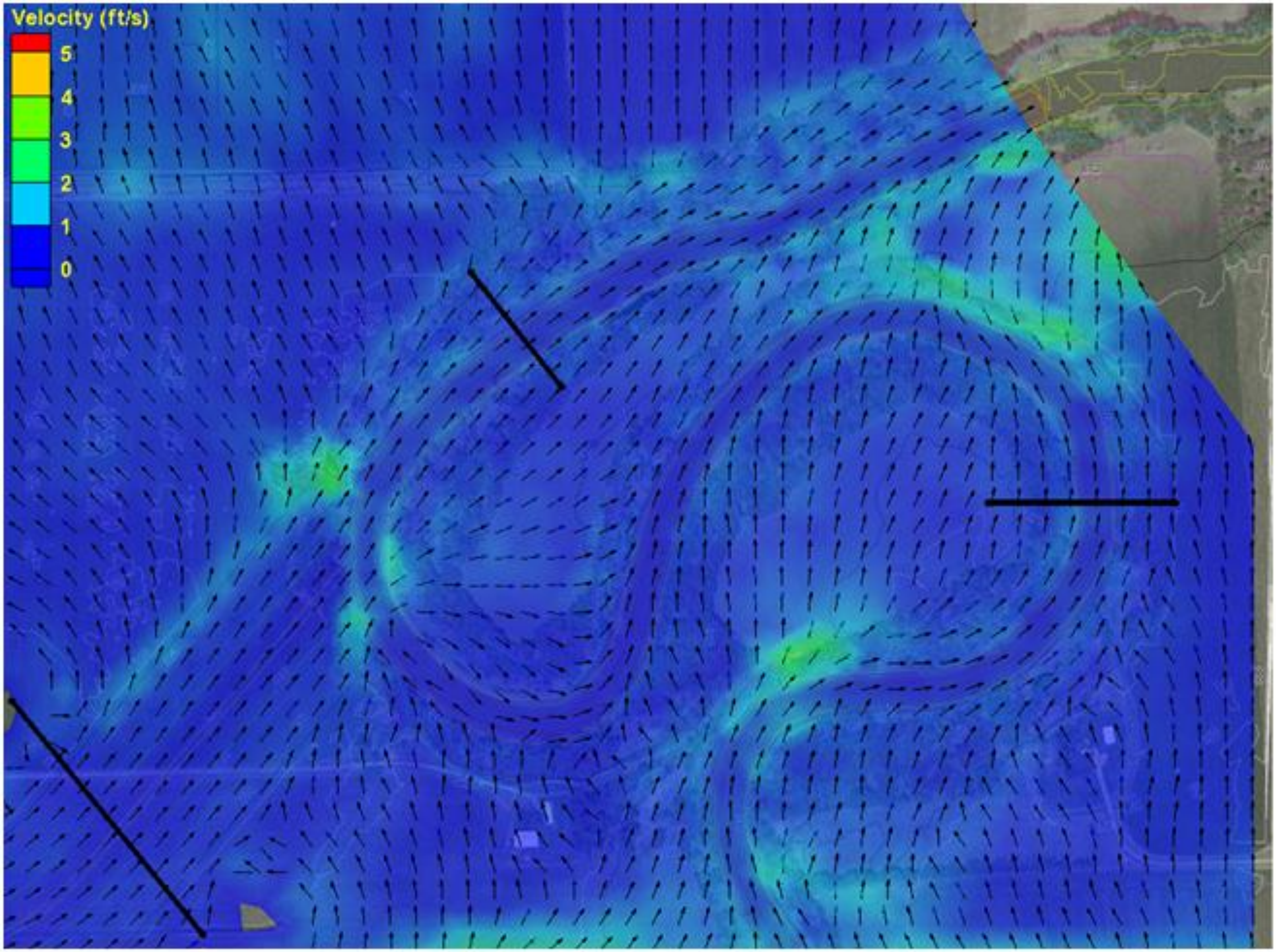

Figure F-H103 Velocity Field for North Dakota Outlet to Red River of the North Post-construction Conditions 20-Year Event Flow

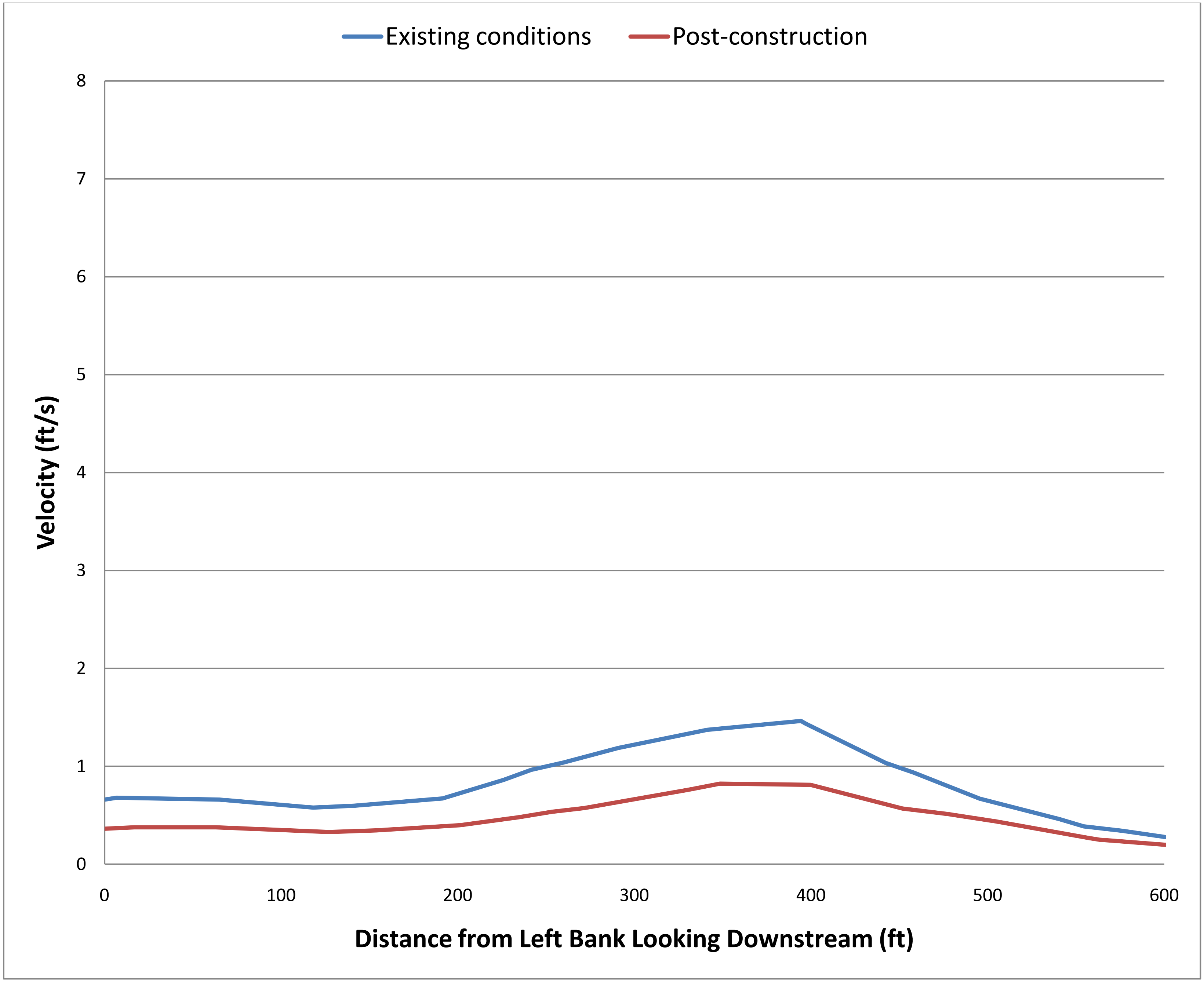

Figure F-H104 Cross Sectional Velocity Data for North Dakota Outlet to Red River of the North Upstream During 20-Year Event

Cross Sectional Velocity Data for North Dakota Outlet (20-Year). Compares velocity at existing conditions to post-construction upstream of structure.

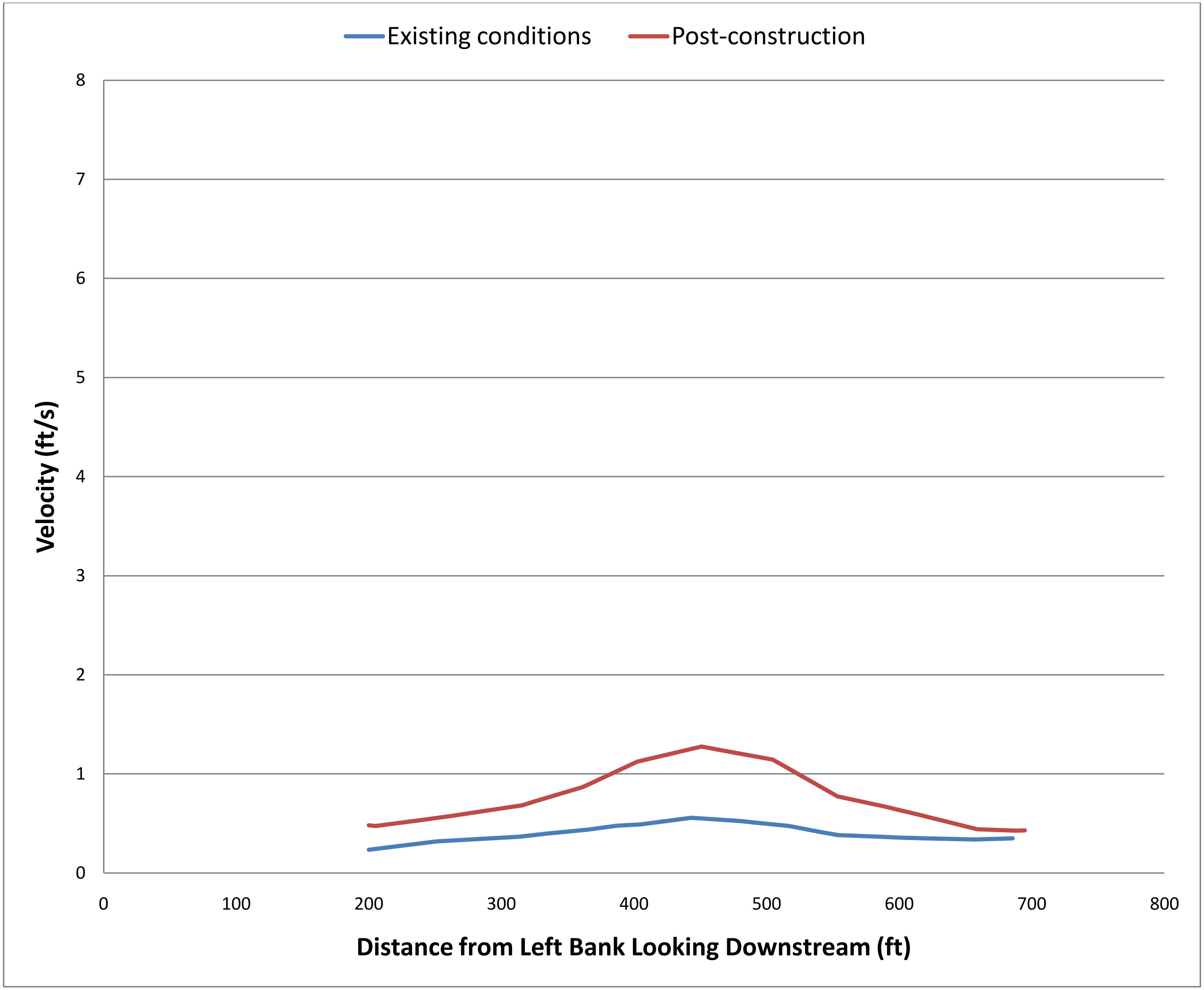

Figure F-H105 Cross Sectional Velocity Data for North Dakota Outlet to Red River of the North Downstream During 20-Year Event

Cross Sectional Velocity Data for North Dakota Outlet (20-Year). Compares velocity at existing conditions to post-construction downstream of structure.

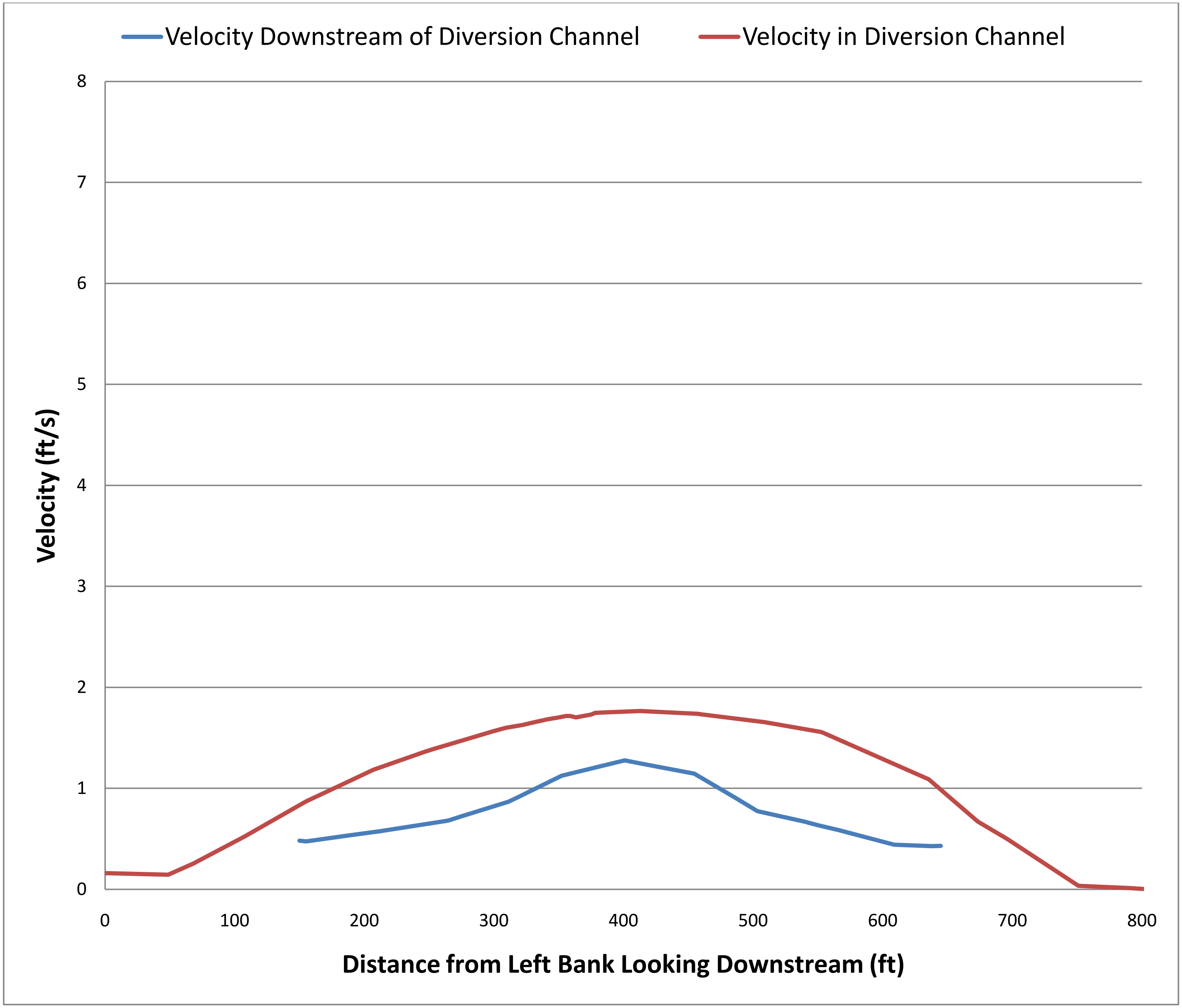

Figure F-H106 Cross Sectional Velocity Data for North Dakota Outlet to Red River of the North Diversion Channel During 20-Year Event

Cross Sectional Velocity Data for North Dakota Outlet (20-Year). Compares velocity in diversion channel to downstream.

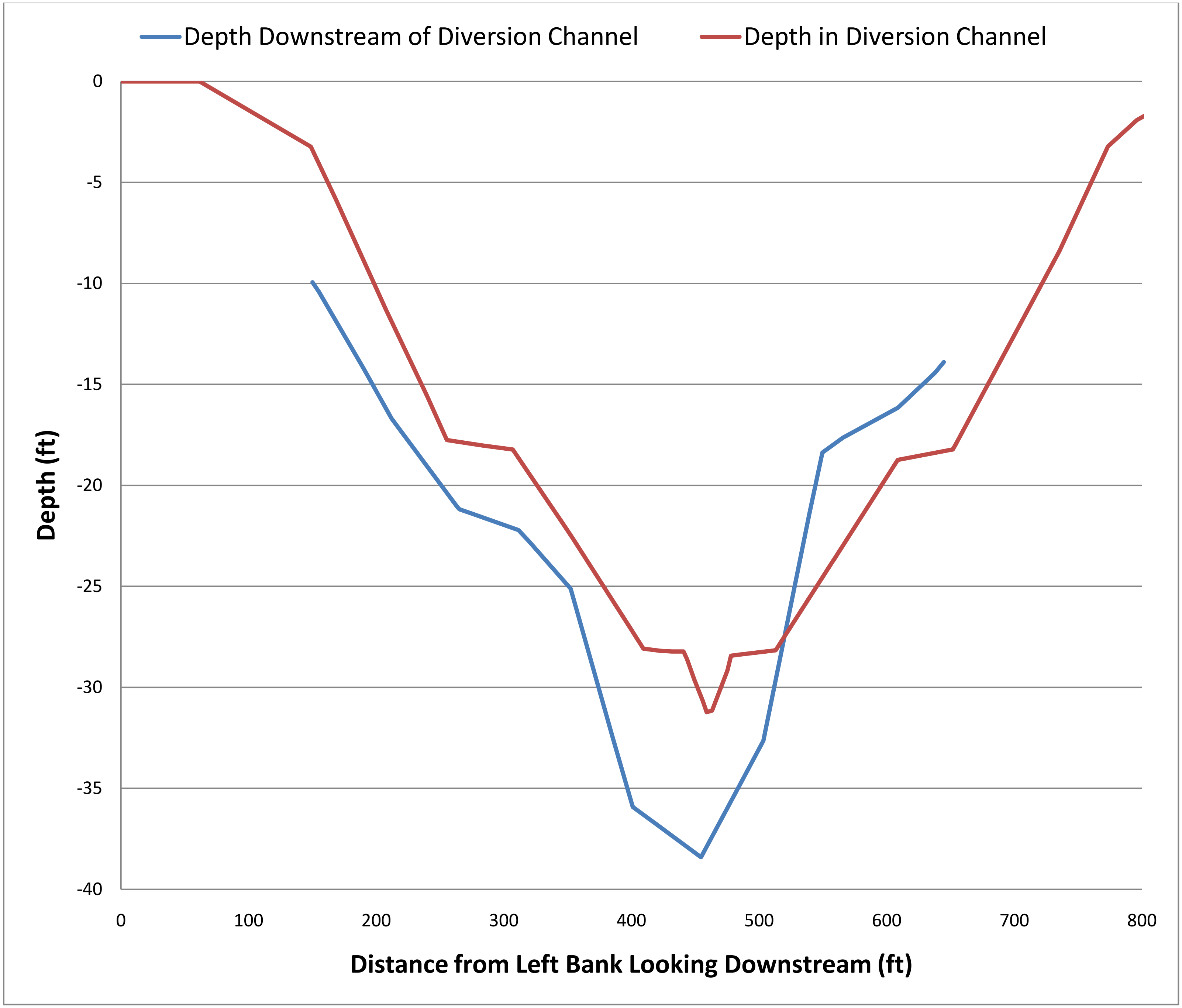

Figure F-H107 Cross Sectional Depth Data for North Dakota Outlet to Red River of the North Diversion Channel During 20-Year Event

Cross Sectional Depth Data for North Dakota Outlet (20-Year). Compares depth in diversion channel to downstream.

RED RIVER DIVERSION

FARGO–MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT I – SEDIMENT TRANSPORT

Report for the US Army Corps of Engineers, and the Cities of Fargo, ND and Moorhead, MN

By: Barr Engineering Co.

VERSION 2 – March 9, 2011

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HYDRAULIC STRUCTURES

EXHIBIT I – SEDIMENT TRANSPORT

F-I1.0 INTRODUCTION AND GEOLOGIC SETTING

In order to characterize the sediment transport and geomorphology of the rivers in the vicinity of the cities of Fargo and Moorhead, it is important to first understand the geologic setting of the greater Red River Basin. The geology, topography, and soils of the area are the fundamental influences (along with hydrology) that govern river shape and morphology.

F-I1.1 STUDY LOCATION IN REGIONAL GEOLOGY

The central feature of the Red River Basin is the Red River Valley, the flat plain that once was the bed of Glacial Lake Agassiz. The lake formed at the southern edge of the Laurentide Ice Sheet and remained in existence from approximately 11,500 to 7,500 years before present (Teller and Clayton, 1983). Within the study area and over much of the old lake bed, the lake left behind a 150 to 300 foot layer of primarily silts and clays (Klausing, 1968; Fenton et al., 1983; Tornes and Brigham, 1994) over a 50 to 60 mile wide area stretching from south of Breckenridge, Minnesota to Winnipeg, Manitoba. This area is known as the "lake plain" in descriptions of the Red River Basin (see Figure F-I1). Within the lake plain, topographic relief is minimal (Figure F-I2) and the typical slope is less than 5 feet per mile (0.1%, Figure F-I3).

The lake plain is bordered by steeper beach ridges, which formed the shoreline of Glacial Lake Agassiz. Glacial rivers flowing into the lake deposited coarser sediment (sands and gravels) in these areas (Christensen, 2007), creating deltas that are mostly buried beneath later lake-deposited fine sediment. The surficial geology of the study area is shown in Figure F-I4. Regional soil survey information (Figure F-I5) shows that the sandiest soils in the Red River Basin are concentrated along the shoreline areas, approximately 20 miles from the proposed LPP Diversion Channel.

The Cities of Fargo and Moorhead sit at the center of the Red River Valley and the lake plain, although the beach ridges with higher slopes and sandier soils are closer to the Red River of the North on the eastern (Minnesota) side.

F-I1.2 GENERAL STREAM CHARACTERISTICS

F-I1.2.1 Wild Rice River (North Dakota)

The Wild Rice River (hereafter WRR) enters the lake plain near Wahpeton, ND and flows northward for more than 60 miles before joining the Red River of the North (hereafter RRN) approximately 10 miles south of the Cities of Fargo and Moorhead. Like the RRN, which it parallels, the WRR is highly meandering and has a very low gradient of approximately 0.7 feet per mile (0.01%). The banks of the WRR are typically gently sloped from 4-5H:1V. There is one low-head dam on the WRR approximately 3.5 miles upstream of the confluence with the RRN.

F-I1.2.2 Sheyenne River

The Sheyenne River enters the lake plain near Kindred, ND and flows northward for approximately 75 miles before joining the RRN near Harwood. Note from Figures F-I1 to F-I5 that the Sheyenne River enters the lake plain relatively closer to the cities of Fargo and Moorhead than the other rivers in this study. According to Klausing (1968), the glacial Lake Agassiz deposits that form the Sheyenne River delta is located approximately 4 miles upstream from where the Sheyenne River crosses Kindred, and approximately 12 miles southwest from the upstream reach of the proposed LPP Diversion Channel. The Sheyenne River delta includes well-sorted deposits of thinly laminated silt and fine to medium sand, and it ranges in thickness from 0 to about 37 meters (120 feet). The reach of the Sheyenne River downstream of Kindred appears to be fluvial (not glacial) in origin, with few or no sand and gravel deposits.

Midway along the portion of the Sheyenne River within the lake plain (42 miles upstream of the RRN) the existing Horace/West Fargo Diversion channel routes a portion of the Sheyenne River water around Horace and areas of West Fargo during high flow conditions. Farther downstream, near Interstate 94 (29.5 miles upstream of the RRN) the West Fargo/Riverside Diversion channel routes additional flow around West Fargo and Riverside. Under the highest flow conditions all of the flow in the Sheyenne River is transferred to this diversion channel and direct flow down the main stem of the Sheyenne River is stopped entirely. The combined diversion channel rejoins the main stem of the Sheyenne River near the confluence with the Maple River, approximately 19 river miles upstream of the RRN.

The Sheyenne River is highly meandering, with a gradient of approximately 0.8 to 1.1 feet per mile (0.01% to 0.02%) upstream of the confluence with the Maple River. The river gradient steepens somewhat downstream of the diversion channels near the confluence with the Maple (to about 2.8 feet per mile or 0.05%), then returns to its previous range for the rest of the distance to the RRN. The banks of the Sheyenne River are steeper than those of the WRR, with the lower part of the banks typically sloped from 1-2H:1V. In addition to the two diversion structures that control flow into the Horace/West Fargo and West Fargo/Riverside Diversions, there is a low-head dam downstream of the Main Avenue bridge on the portion of the Sheyenne River bypassed by the West Fargo/Riverside Diversion.

F-I1.2.3 Maple River

The Maple River enters the lake plain near Leonard, ND and flows northwest for approximately 68 miles before joining the Sheyenne River near Riverside, ND. The Maple River delta is located at the edge of the lake plain, approximately 60 miles upstream from the confluence with the Sheyenne River (see Figure F-I4). This delta includes deposits of silt, sand, gravel and a few boulders (probably ice-rafted erratic), and it ranges in thickness from 0 to about 15 meters (50 feet). Klausing (1968) suggests that the lower reach of the Maple River, near the proposed Diversion Channel, is of post-glacial origin and is characterized by the absence of sand and gravel deposits.

Similar to the WRR, the gradient of the Maple River is approximately 0.7 feet per mile (0.01%), and the banks are gently sloped from 3-5H:1V. There are two existing low-head dams on the Maple River, one upstream of the Interstate 94 crossing near Mapleton and one immediately upstream of the confluence with the Sheyenne River.

F-I1.2.4 Red River of the North

The RRN originates in the cities of Wahpeton and Breckenridge at the confluence of the Otter Tail and Bois de Sioux Rivers, approximately 187 miles upstream of Fargo and Moorhead. Similar to the Wild Rice and Maple Rivers, the gradient of this portion of the RRN is approximately 0.6 feet per mile (0.01%), and the banks are sloped near 3H:1V.

Brooks (2003a, 2005) indicates that the suspended sediment load of the RRN is composed primarily of silt with some clay. Paakh et al. (2006) state that the fine clay and silt sediments in the Red River Valley Lake Plain are easily suspended and tend to stay in suspension even during relatively low-flow conditions. Lauer et al. (2006) hypothesize that although some of the RRN sediment moves as bedload in the form of aggregated pellets of fine sand size, most of the bed sediment is transported in disaggregated form as silt and clay in suspension. Thus over engineering time scales, unless there is a significant change in the sediment supply from the watershed, potential changes of the RRN channel geometry would be associated with channel migration rather than with bed aggradation or degradation. However, Brooks (2003b) reports a very slow net expansion of RRN (meander) bends with channel migration rates in the order of 4 centimeters per year (1.6 inches per year) over the past 1,000 years. Therefore, the RRN can be considered a stable riverine system, an opinion that is shared by Prof. Gary Parker (personal communication), one of the world leading experts in river morphodynamics.

There are several existing low-head dams on the RRN in the area of interest, two upstream of the confluence with the WRR (near Christine and Hickson) and three in the vicinity of Fargo and Moorhead (Fargo South, Midtown, and North dams). The three dams in the cities of Fargo and Moorhead have been retrofitted with rock spillways to increase public safety and to improve fish passage up the RRN during low flow conditions. Similar retrofitting is proposed for the dams at Christine and Hickson, but has not been constructed.

F-I2.0 PREVIOUS STUDIES

The sections below provide a brief summary of several previous studies consulted for the current analysis from the literature and project data available for the RRN and its tributaries. This is not intended to be an exhaustive list of the available literature on the geomorphology of the RRN.

F-I2.1 SHEYENNE RIVER GEOMORPHOLOGY STUDY

A study of the geomorphology of the entire length of the Sheyenne River was prepared for the USACE in 2001 by West Consultants (West 2001). The study evaluated existing conditions at various locations along the Sheyenne River and provided predictions of the behavior of the river under several hydrologic and Devils Lake discharge future scenarios.

Of relevance to the current study are the findings at two of the monitored sub-reaches near the proposed diversion channel for the RRN (see Figure F-I6). At both of these locations the stream bed was found to consist of primarily fine sand, with finer material (silt and clay) in the banks at the more downstream location. At both locations the surveyed river cross sections showed relatively steep stream banks with slopes ranging from 1-3H:1V. Analysis of the sediment transport processes at both sub-reaches implied the river channel has widened slightly over the last 50 years and will likely continue to widen to achieve equilibrium conditions (West 2001).

F-I2.2 SOUTH BRANCH OF THE BUFFALO RIVER SEDIMENT MODEL

In 2006 the St. Anthony Falls Laboratory at the University of Minnesota performed sediment production modeling on the watershed of the South Branch of the Buffalo River for Houston Engineering and the Buffalo-Red River Watershed District (Lauer et al. 2006). The study used a sediment production model developed by the US Department of Agriculture to quantify surface and bank erosion rates for various segments of the Buffalo River watershed.

Although the Buffalo River is outside of the project area for the current study (see Figure F-I6), it occupies a similar position in the watershed to the Sheyenne River system. The stream bed sediment collected on the Buffalo River and its tributaries shows generally fine-grained bed material (silt and clay) at sites located within the glacial lake plain. At sites located along the glacial beach line and in the uplands, the stream bed material is typically fine to medium sand. The sediment budget for the watershed indicates that the majority of the sediment load to the river system is generated in the steeper portions of the watershed, outside of the glacial lake plain. The majority of the sandy material is deposited quickly once the streams enter the lake plain and is not transported to the confluence with the RRN.

F-I2.3 SOUTHSIDE FLOOD CONTROL STUDY

As part of a larger study performed by Moore Engineering of potential flood control measures on the RRN in south Fargo, Barr Engineering assessed the geomorphology of the river in 2008 (Barr 2008).

A general review of literature on the RRN and analysis of the available suspended sediment data indicated that the river is in morphodynamic equilibrium, with no appreciable net aggradation or erosion. Eight bed sediment samples were collected for this project downstream of the confluence of the RRN and the WRR (see Figure F-I6). With the exception of the sampling location nearest to the mouth of the WRR, all samples indicate that the bed of the RRN consists of silt and clay with very little sand (less than 10% by weight). The slightly greater quantity of sand in the remaining sample is interpreted as being delivered by the WRR but deposited in a short stretch of the RRN.

These observations confirm the interpretation of the RRN as being a stable riverine system.

F-I3.0 SEDIMENT TRANSPORT DATA

The sediment transport data considered in this analysis was collected by the USGS at various locations in the study area, as described below.

F-I3.1 USGS SEDIMENT SAMPLING (PRE-2010)

Historical sediment data is available from the USGS for the RRN and several of its tributaries. The data was generally collected at established gage locations, and includes observations throughout the calendar year and for a range of flow conditions. See Figure F-I6 for the monitoring locations considered in this analysis, and Table F-I1 for a summary of the available data.

The historical sediment data used in this study includes numerous measurements of depth-integrated suspended sediment concentration (SSC) and the fraction of the suspended material that is finer than sand. Locations with only total suspended solids (TSS) data or with only a single measurement were not included in this analysis. Complete gradations of suspended sediment and bed material are available for a limited set of monitoring locations and dates, primarily in the late 1970s.

F-I3.2 USGS SEDIMENT SAMPLING (SPRING 2010)

The USACE contracted with the USGS to determine sediment concentrations, loads, and particle size distributions at six sites in the RRN and its tributaries during the spring high flow of 2010 (Blanchard et al. 2010). See Figure F-I6 for the monitoring locations. Sampling began on March 24, 2010 and the last measurement was taken on April 7, 2010; data was collected only when the rivers were not covered by ice, resulting in different numbers of samples for the different locations.

The sediment data collected in 2010 included the following measurements during the peak and recession of spring flood waters: depth-integrated suspended sediment concentration (SSC), suspended sediment concentration at discrete points in the water column, suspended sediment gradation, bedload magnitude and gradation, and bed material gradation. Figures and tables presented in the 2010 USGS report will not be reproduced here but should be used for reference.

A second season of sediment data collection at the same sites is planned by the USGS for the spring high flow of 2011. This additional data will be used in further analysis of the proposed diversion project and in the concurrent study discussed below.

F-I3.3 DATA CONSIDERED BY CONCURRENT STUDIES

Concurrent with this feasibility study the USACE has contracted with West Consultants for a detailed study of the geomorphology of the RRN and its tributaries within the project area. This study will involve field data collection including channel cross-sectional surveys and bank and bed material sampling (some of which is referenced in

Section 4.1) as well as review of historic aerial photos, surveys, construction documents, hydrologic models, and sediment borings. This data will be used to develop a detailed morphological classification of each stream reach, assess the stability of the channel and the current sediment transport conditions, and evaluate the effects of future conditions with the proposed diversion project in place.

F-I4.0 CHARACTERIZATION OF EXISTING CONDITIONS

F-I4.1 SEDIMENT DATA ANALYSIS

F-I4.1.1 Wild Rice River (North Dakota)

As discussed in Section F-I1.2, the WRR is a meandering, low-gradient river that flows across the lake plain for many miles before approaching the monitoring locations near the cities of Fargo and Moorhead. The fine soils in the lake plain and along the river banks contribute sediment to the river that is transported primarily in suspension because of its low settling velocity. In this regard conditions on the WRR at the monitoring locations shown in Figure F-I6 are typical of many of the other streams within the glacial lake plain.

Historic monitoring data from the USGS shows that the SSC in the WRR is typically below 100 mg/L (the median SSC is 42 mg/L), although concentrations have been observed as high as 540 mg/L (Figure F-I7). Although there are limited historic measurements of the suspended sediment particle size in the WRR, the available data consistently show that over 90% of the suspended sediment is finer than sand (i.e. silts and clays). The 2010 USGS sampling data for SSC shows results similar to the historic data (over 90% finer than sand), albeit with a slightly higher range of observed SSC (136 to 186 mg/L). Measurements of SSC at different points in the water column show no significant vertical differences in SSC or in the fraction of the suspended material that is finer than sand.

Both the historic and the 2010 USGS data show that the bed material in the WRR is medium sand, with a typical median particle size of approximately 1 mm (Figures F-I9 and F-I10). There are two observations in the historic record of more finely grained bed material, both under lower-flow conditions in the summer (as opposed to during spring floods). The material transported as bedload is fine to medium sand, similarly sized to the majority of the samples of bed material (Figure F-I11).

The SSC and bedload data collected by the USGS in the spring of 2010 show that the vast majority of the sediment load in the WRR, over 99.9% by weight, is transported in suspension (Table F-I2). The total sediment load in the WWR in the spring of 2010 peaked at or possibly before the peak flow, driven by the highest measured SSC at and before the peak of the hydrograph.

In summary, these data indicate that the sediment load carried by the WRR in the project area is overwhelmingly fine suspended material, which is likely transported long distances from its origin in overland and bank erosion. Some of this material may settle to the bed of the river during periods of lower river velocity, but typical high flows are likely sufficient to re-suspend any settled material, leading to minimal net change in channel dimensions over time.

F-I4.1.2 Sheyenne River

Like the WRR, the Sheyenne River is a meandering, low-gradient river that transports sediment primarily in suspension. However, as noted in Section F-I1.2, the Sheyenne has several characteristics that make it distinct from the other rivers considered in this analysis. The banks of the Sheyenne River are generally steeper than those of the WRR, and the gradient of the Sheyenne River is slightly greater. Also significant to this study is the fact that flows in the Sheyenne River are managed by two existing diversion projects, which significantly alter the flow regime through portions of the river channel.

The highest SSC levels observed by the USGS in the 2010 study were on the Sheyenne River just upstream of the Horace/ West Fargo Diversion, ranging from 476 to 1120 mg/L with average values near 700 mg/L. The suspended sediment was observed to contain more coarse material than at the WRR, with fine sand representing between 15 and 30% of the material in suspension. The historic USGS data on the Sheyenne River at Kindred (i.e. approximately 30 miles upstream of the 2010 sampling location) show generally lower SSC (median 73 mg/L and maximum 880 mg/L), although the fraction of fine material was observed to vary even more widely (Figures F-I7 and F-I8). As for the WRR, the 2010 measurements of SSC at different points in the water column show no significant vertical differences in SSC or in the fraction of the suspended material that is finer than sand.

The 2010 USGS data indicates that the bed material in the Sheyenne River at Horace is fine to medium sand, with a typical median particle size of approximately 0.4 mm (Figure F-I13). The material transported as bedload is similarly sized (Figure F-I14). These characterizations of bed material are somewhat coarser than indicated by the historic data at Kindred (typically fine sand as shown in Figure F-I12).

As observed for the WRR, the 2010 measurements show that the vast majority of the sediment load in the Sheyenne River, over 99.9% by weight, is transported in suspension (Table F-I2). The total sediment load in the spring of 2010 peaked at the peak flow, driven by the highest measured SSC at the peak of the hydrograph.

The 2010 USGS data collected on the Sheyenne River at Horace (downstream of the Horace/West Fargo Diversion) show SSC and bed material characteristics that are similar to the upstream Sheyenne River. The observed SSC downstream of the diversion is nearly identical to that upstream (average values near 700 mg/L), and the suspended material is slightly finer than in the upstream Sheyenne River (78 to 83% finer than sand, compared to 70 to 84% above the diversion). The bed and bedload material in the

Sheyenne River downstream of the diversion are slightly finer than in the Sheyenne River upstream of the diversion (Figures F-I15 and F-I6).

The sediment load data for the Sheyenne River downstream of the Horace/West Fargo Diversion indicate that the suspended material is diverted from the main Sheyenne River in proportion to the flow diversion (Table F-I2), and with similar timing. The small quantity of bedload, however, was not diverted proportionally: the measured bedload downstream of the diversion was approximately one-quarter of that measured upstream, while the total flow passing into the protected areas was approximately half of that in the upstream Sheyenne River. This observation may indicate that a portion of the bedload material is deposited in the Sheyenne River upstream of the diversion structure, which could contribute to the slightly coarser bed material upstream of the diversion (compare Figure F-I13 with Figure F-I15). The bed material samples collected in 2010 by West Consultants along the Horace/West Fargo diversion channel (see Figure F-I6 for locations) show gradations generally similar to the bed material sampled by the USGS in the Sheyenne River downstream of the diversion, suggesting that coarser bedload is not transported into the diversion channel. Any deposition of coarser bedload upstream of the diversion structure is likely minor.

Like the WRR, the data indicate that the sediment load carried by the Sheyenne River is primarily fine material. The higher SSC levels and slightly coarser material suspended in the Sheyenne River suggest that there is more active bank erosion in the Sheyenne River system than in the WRR, providing a source of more sediment close to the monitoring locations. This finding is consistent with the results of the West 2001 geomorphology study, which predicted that the lower portions of the Sheyenne River would slowly widen by eroding the steep banks.

The available sediment data does not suggest that the geomorphology of the Sheyenne River has been significantly impacted since the construction of the Horace/West Fargo and West Fargo/Riverside diversions. Bed and bank samples collected by the USGS and West Consultants in 2010 show similar gradations on the Sheyenne River upstream and downstream of the diversion channels: bed material of fine sand and banks of primarily silt and clay. The West 2010 data for the Horace/West Fargo Diversion channel shows some evidence of deposition of fine material at the upstream end of the diversion channel, likely left behind as floodwaters recede and flow ceases in the diversion channel. However, there is no indication of significant net deposition of fine material in the main Sheyenne River as a result of the diversion projects.

F-I4.1.3 Maple River

The available cross-sectional and sediment transport data indicates that the Maple River is very similar to the WRR. Both rivers have similar gradients and bank slopes, and both rivers traverse the lake plain for significant distances before approaching the study area. There is no historic data available for the Maple River (see Table F-I1), so the discussion here is limited to analysis of the 2010 USGS sediment data.

The 2010 USGS monitoring data shows that SSC in the Maple River is somewhat higher than in the WRR, with a range of 172 to 325 mg/L (median 211 mg/L). Similar to the WRR, the 2010 data show that over 90% of the suspended sediment in the Maple River is finer than sand. Measurements of SSC at different points in the water column show no significant vertical differences in SSC or in the fraction of the suspended material that is finer than sand, except for one mid-depth sample that may represent equipment malfunction.

Similar to the WRR, the 2010 USGS data shows that the bed material in the Maple River is medium sand, with a typical median particle size of approximately 0.7 mm (Figure F-I17). The material transported as bedload is medium sand, slightly coarser than the majority of the samples of bed material with a typical median particle size of approximately 1 mm (Figure F-I18).

Like the WRR and the Sheyenne River, the majority of the sediment load in the Maple River is transported in suspension. Bedload was a slightly higher portion of the total load in the Maple River, but suspended material still accounted for 99.8% by weight of the sediment transport (Table F-I2). The total sediment load in the Maple River in the spring of 2010 peaked at or possibly before the peak flow, although there was a secondary peak of high SSC observed as the flood waters receded.

In summary, these data indicate that the sediment load carried by the Maple River in the project area is overwhelmingly fine suspended material, which is likely transported long distances from its origin in overland and bank erosion. Some of this material may settle to the bed of the river during periods of lower river velocity, but typical high flows are likely sufficient to re-suspend any settled material, leading to minimal net change in channel dimensions over time.

F-I4.1.4 Red River of the North

Like its tributaries, the RRN transports the large majority of its sediment load as suspended fine material (silts and clays). The historic USGS data indicates that SSC levels and their variability both increase moving downstream from Hickson (median SSC 72 mg/L) to Fargo (median SSC 112 mg/L), as shown in Figure F-I7. The majority of samples at all three historic USGS monitoring locations on the RRN show that greater than 90% of the suspended material is finer than sand, although there are occasional (11 out of a combined 160 samples) outliers with higher sand fractions (Figure F-I8). In 2010, the monitoring locations on the RRN had the lowest SSC observed, with concentrations ranging from 74 to 156 mg/L and median values from 93 to 116 mg/L. As in the other rivers in this study, there are no significant or consistent vertical differences in SSC or in the fraction of the suspended material that is finer than sand.

The bed material sampled by the USGS in 2010 at both locations on the RRN was medium sand, with typical median particle size of approximately 1 mm (Figures F-I20 and F-I22). The material transported as bedload is similarly sized, although slightly finer and with a very sporadic distribution for the samples collected on the RRN in Fargo

(Figures F-I21 and F-I23). These observations compare well with the historic bed sediment gradations measured at the USGS gage on the RRN at Hickson (Figure F-I19).

The sandy bed material collected by the USGS from the RRN is somewhat inconsistent with the results of other sampling efforts in the sampling area. As described in Section 2.3, samples collected for the Southside Flood Control Study (Barr 2008) show primarily fine-grained bed material with less than 10% sand. In addition, recent sampling by West Consultants as a part of the study described in Section F-I3.4 show predominantly fine bed material at locations both upstream and downstream of the confluence of the RRN with the WRR (see Figure F-I6 for locations). Additional bed samples by West Consultants at locations in Fargo show mostly fine material, although one sample contains 47% sand.

The observed differences in bed material gradation may simply be due to spatial heterogeneity along the river bed or they may be a reflection of the bias in bed sediment sampling locations. The USGS samples (both in 2010 and in the historic data set) are collected from bridges, and the bed material may be affected by road runoff, channel armoring, and constricted flows and may not represent conditions in the remainder of the river channel. In addition, the monitoring locations near Hickson (historic) and Christine (2010) are both upstream of low-head dams on the RRN, which may cause atypical deposition in these areas. Further study, including borings of river bed sediment profiles, is warranted to better characterize the conditions in the study area.

Regardless of the exact nature of the RRN bed material, the data collected by the USGS in 2010 demonstrates that bedload is a small fraction of the total sediment transport in the RRN (Table F-I2). At the upstream site near Christine, 99.5% of the measured total sediment load was from suspended material, dominated by a single high measurement of bedload at the peak of the hydrograph. At the downstream site in Fargo, the bedload was of similar magnitude to the other sampling locations and the suspended sediment accounted for over 99.9% of the total sediment transport. At both locations the total sediment load peaked at or just before the peak flow, and the peak SSC was observed as the flood waters receded.

Like the tributaries considered in this analysis, sediment transport in the RRN is dominated by the movement of suspended fine material. This suspended material is well-distributed throughout the vertical water column and is transported through the study area with minimal interaction with the stream bed.

F-I4.2 SEDIMENT BALANCE ANALYSIS

Total sediment loading for the period of monitoring is reported in the summary of the USGS 2010 sediment data collection (Blanchard et al. 2010). Total flows and sediment loads for concurrent periods are shown in Table F-I2 to facilitate interpretation of the sediment balance for several of the study rivers.

F-I4.2.1 Red River of the North and Wild Rice River

The data collected in 2010 on the RRN and WRR is sufficient to perform a simple flow and sediment balance for the portion of the RRN upstream of the confluence with the Sheyenne River. In the absence of other data, the assumption for this analysis is that no additional flow or sediment enters the WRR or RRN between the upstream monitoring locations (WRR near St. Benedict and RRN near Christine) and the downstream location (RRN near Fargo).

As shown in Figure F-I24, both the flow and sediment balance for this system are reasonably close, given the simplifying assumptions made above. The total flow measured at the upstream gaging locations from March 18, 2010 to March 31, 2010 was 18,810 million cubic feet (MCF). The total flow measured in Fargo during the same period was 19,810 MCF, a difference of approximately 5%. The total sediment load (suspended and bedload) measured at the upstream gaging locations was 74,250 tons; the total sediment load measured in Fargo was 72,110, a difference of approximately 3%.

This simple sediment balance indicates that, as expected based on the discussion above, the sediment load in the RRN through the cities of Fargo and Moorhead is neither increasing or decreasing. The RRN does not appear to be gaining sediment (via erosion) or losing sediment (via aggradation) over this reach. This corroborates the description of the RRN as a stable riverine system, with sediment loading from fine suspended material that is primarily washed through the system.

F-I4.2.2 Sheyenne River and Horace/West Fargo Diversion

A similar sediment balance can be performed for the Sheyenne River system at the Horace/West Fargo Diversion. Because there was not a monitoring location on the diversion channel itself, the flow and sediment balance must be assumed to close. The flow and sediment load on the Horace/West Fargo Diversion can then be calculated.

As shown in Figure F-I25, the Horace/West Fargo Diversion is calculated to transport just over half (52-53%) of the total flow and sediment load from the upstream Sheyenne River, with the remainder passing into the protected area in the Sheyenne River. The total flow measured on the Sheyenne River upstream of the diversion from March 24, 2010 to April 7, 2010 was 5,340 MCF, while the total flow measured downstream of the diversion channel during the same period was 2,580 MCF. The total sediment load (suspended and bedload) measured on the Sheyenne River upstream of the diversion was 119,630 tons; the total sediment load measured downstream of the diversion channel was 56,380.

The sediment load passed from the main Sheyenne River to the protected area (through the diversion structure) is almost exactly proportional to the transmitted flow. This is a result of the nearly-identical measured SSC at the monitoring locations upstream and downstream of the diversion structure, and the minimal contribution of bedload to the total sediment load. Note in Table F-I2 that although approximately half of the flow and total sediment load are passed to the diversion channel, the measured bedload in the Sheyenne River downstream of the Horace/West Fargo Diversion is only 23% of that measured upstream of the diversion. This result may indicate that some of the small quantity of bedload material is preferentially retained in the Sheyenne River upstream of the diversion structure.

F-I5.0 POTENTIAL PROJECT IMPACTS

The proposed project to divert flood waters from the RRN and its tributaries around the cities of Fargo and Moorhead involves the construction of a diversion channel and a number of hydraulic structures. Water from the RRN and selected tributaries (Wild Rice, Sheyenne, and Maple Rivers) will be passed into the protected area, with the portion of the flow diverted depending on the magnitude of the flood. This diversion will effectively reduce the flood flows through the protected area for events greater than the 2-yr to the 10-yr return period event, potentially impacting the sediment transport and geomorphology of the streams in the protected area.

F-I5.1 IMPACTS TO SEDIMENT TRANSPORT

The proposed changes to the flow regime in the RRN and its tributaries within the protected area will change the capacity of the rivers to transport sediment. However, because all of the affected rivers are dominated by the transport of fine suspended material (as shown in this analysis), the diversion of a fraction of the river flow is expected to divert a proportional fraction of the total sediment load transported as suspended sediment. This suspended sediment, being fine-grained with very slow settling velocities, can be expected to move through the diversion system and return to the RRN downstream of Fargo and Moorhead. The changes to river flows within the protected area are not expected to be sufficient for the remaining fraction of the suspended sediment to settle. Therefore, the total sediment budget of the system will be essentially unchanged.

The Horace/West Fargo Diversion of the Sheyenne River provides an example of the potential maximum impacts that can be expected from the proposed diversion. As discussed in this analysis, the Sheyenne River system has coarser bed material and more coarse suspended sediment than the other affected rivers, meaning that the impacts of diversion on sediment transport would be expected to be the most significant. However, even the somewhat coarser suspended sediment in the Sheyenne River is passed into the protected area and to the diversion channel in proportion to the flow, validating the description of expected future conditions proposed above. Although the bed material of the Sheyenne River (and most of the other rivers in this study) appears to consist of fine or medium sand, this more coarse material is not transported in significant quantities through the system. What little bedload sediment is in the Sheyenne River appears to move primarily past the diversion structure and into the protected area, although some sediment may be retained upstream of the diversion structure.

F-I5.2 IMPACTS TO GEOMORPHOLOGY

The potential project impacts to the geomorphology of the RRN and its tributaries from the discussed changes in the sediment transport regime are expected to be negligible. The RRN is a stable riverine system, neither aggrading or degrading, with sediment transport primarily in suspension. These characteristics are not expected to change significantly following implementation of the proposed diversion works.

Again, the existing Horace/West Fargo Diversion of the Sheyenne River provides an example of the potential maximum impacts to geomorphology expected from the proposed diversion. The presence of the diversion channel and diversion control structures does not appear to have altered the geomorphic behavior of the river within the protected area. Recent modeling (West 2001) has shown that monitored locations on the lower Sheyenne River experienced only slight adjustments to channel shape over more than 50 years, including periods after the construction of the two Sheyenne River diversion projects, which have been in place for nearly 20 years. The slight widening of the river in these areas was expected based on the hydrology of the larger Sheyenne River, and was not interpreted as a response to local changes in the flow regime caused by the diversion. It is expected that the diversion of additional flood water proposed by the current project will have a similarly small impact on the geomorphology of the Sheyenne River (and by extension the other rivers in the study). This initial conclusion will be examined more thoroughly in the ongoing geomorphic study of the area rivers being performed by West Consultants.

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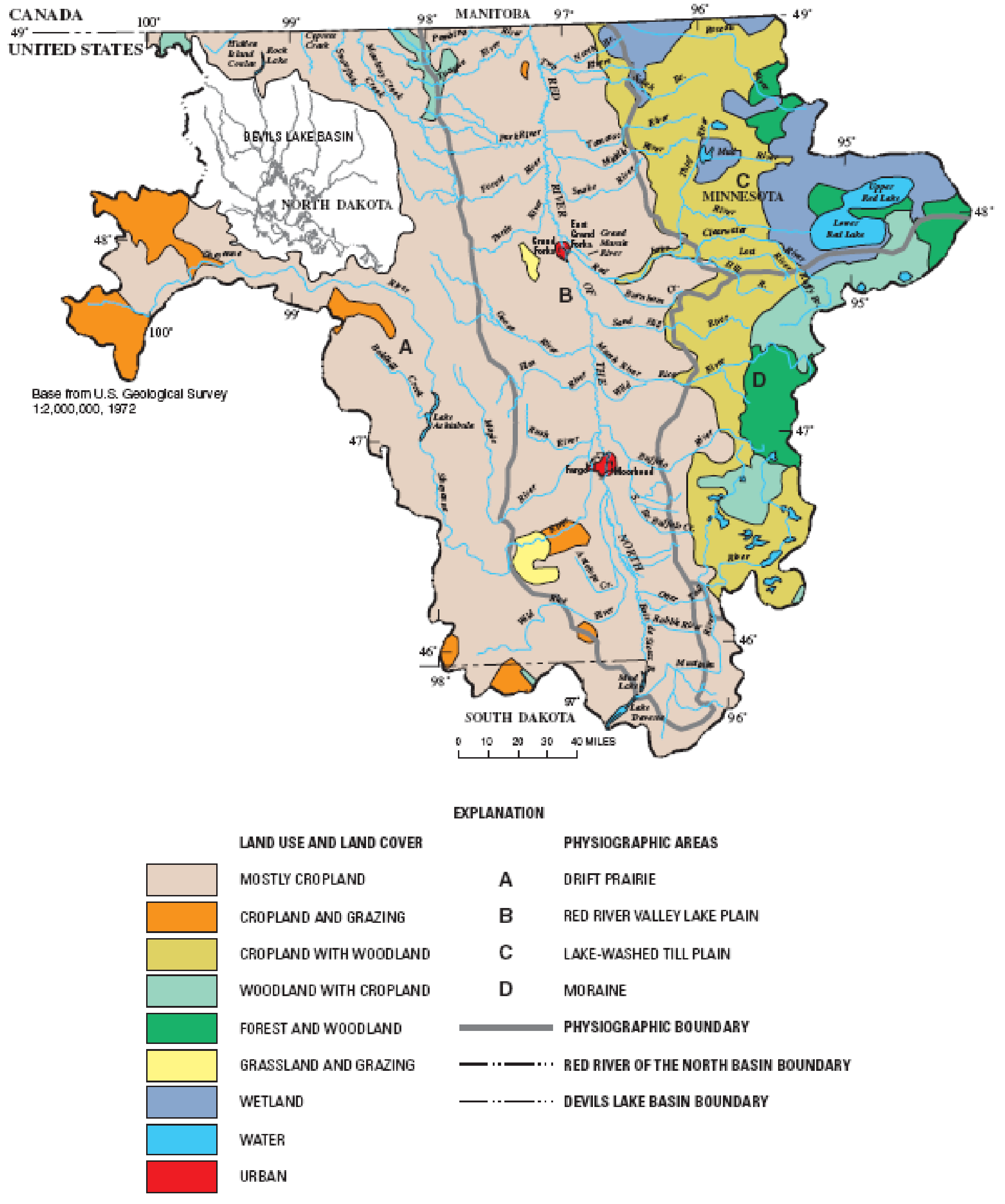
Blanchard, R.A., Ellison, C.A., Galloway, J.M., and Evans, D.A. (2010). Sediment concentrations, loads and particle size distributions in the Red River of the North and selected tributaries near Fargo, North Dakota during the 2010 spring high-flow event. *U.S. Geological Survey*, Scientific Investigations Report 2010-XXXX, XX pp. (pending final review)

West Consultants (2001). Sheyenne River Geomorphology Study. 122pp.

Figure F-I1 Land Use and Land Cover and Physiographic Areas in the Red River of the North Basin



Source: Christensen (2007). Used with permission.

Figure F-I 2 Topography of the Red River of the North Basin

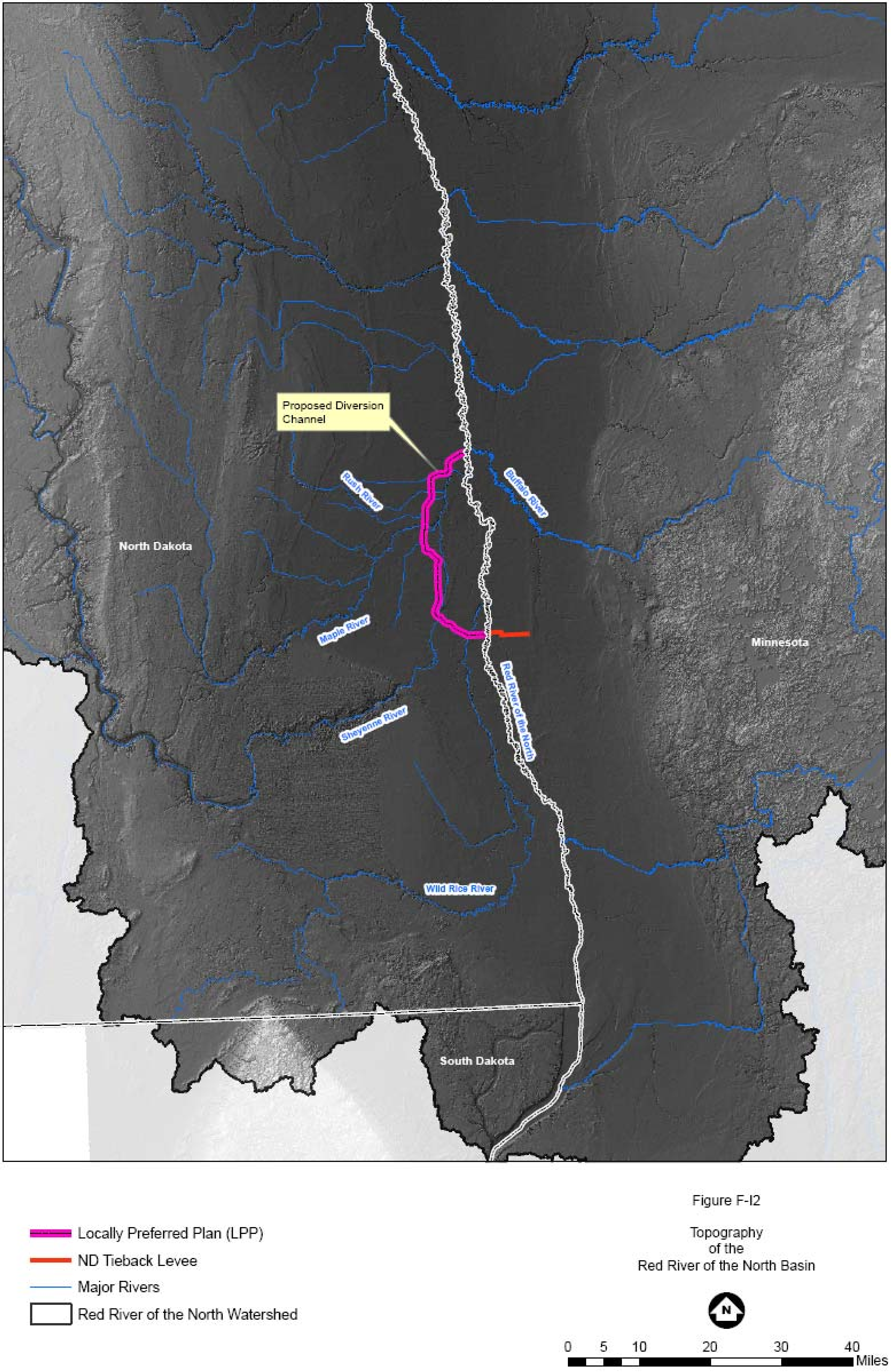
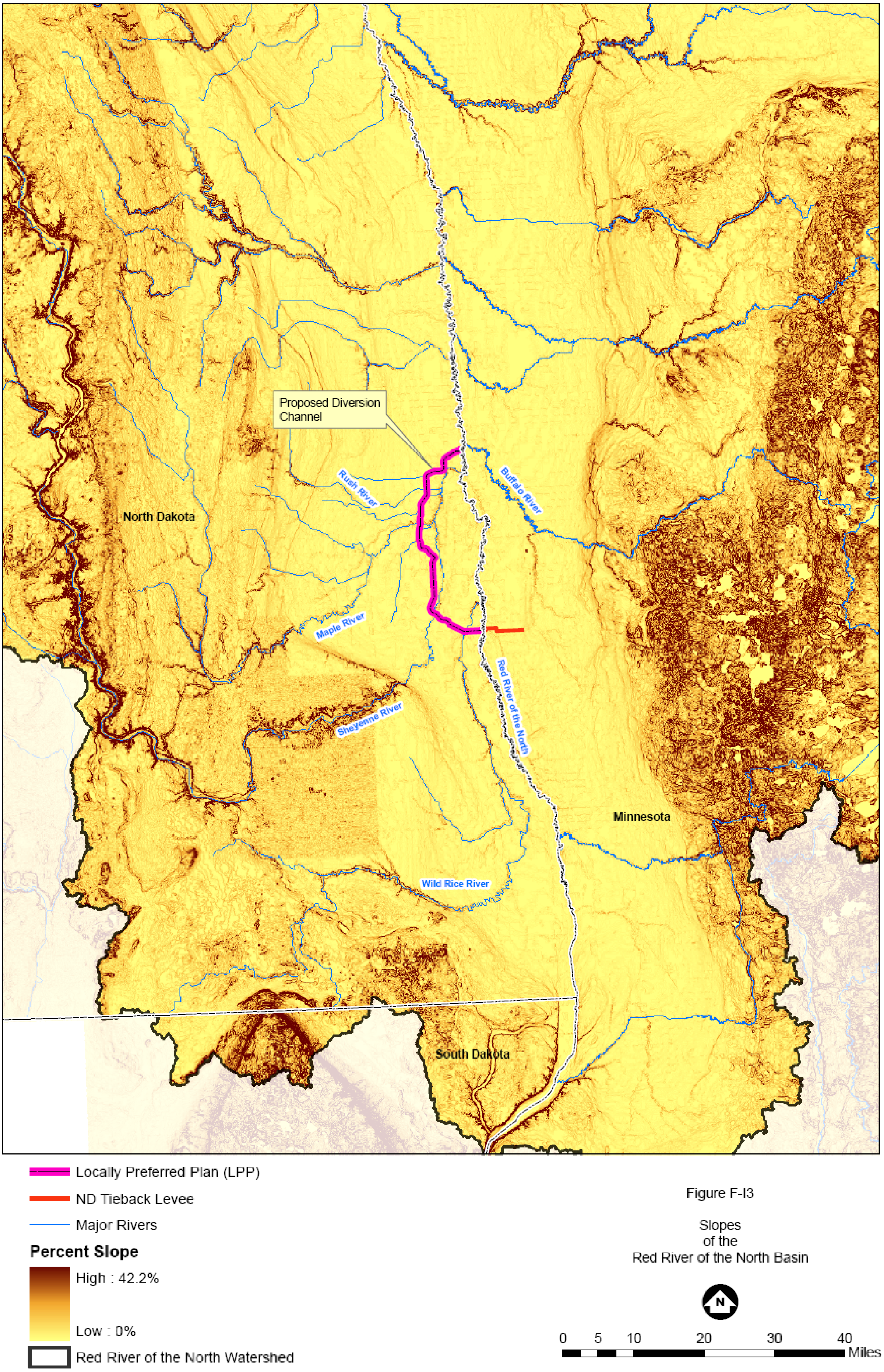


Figure F-I 3 Slopes in the Red River of the North Basin



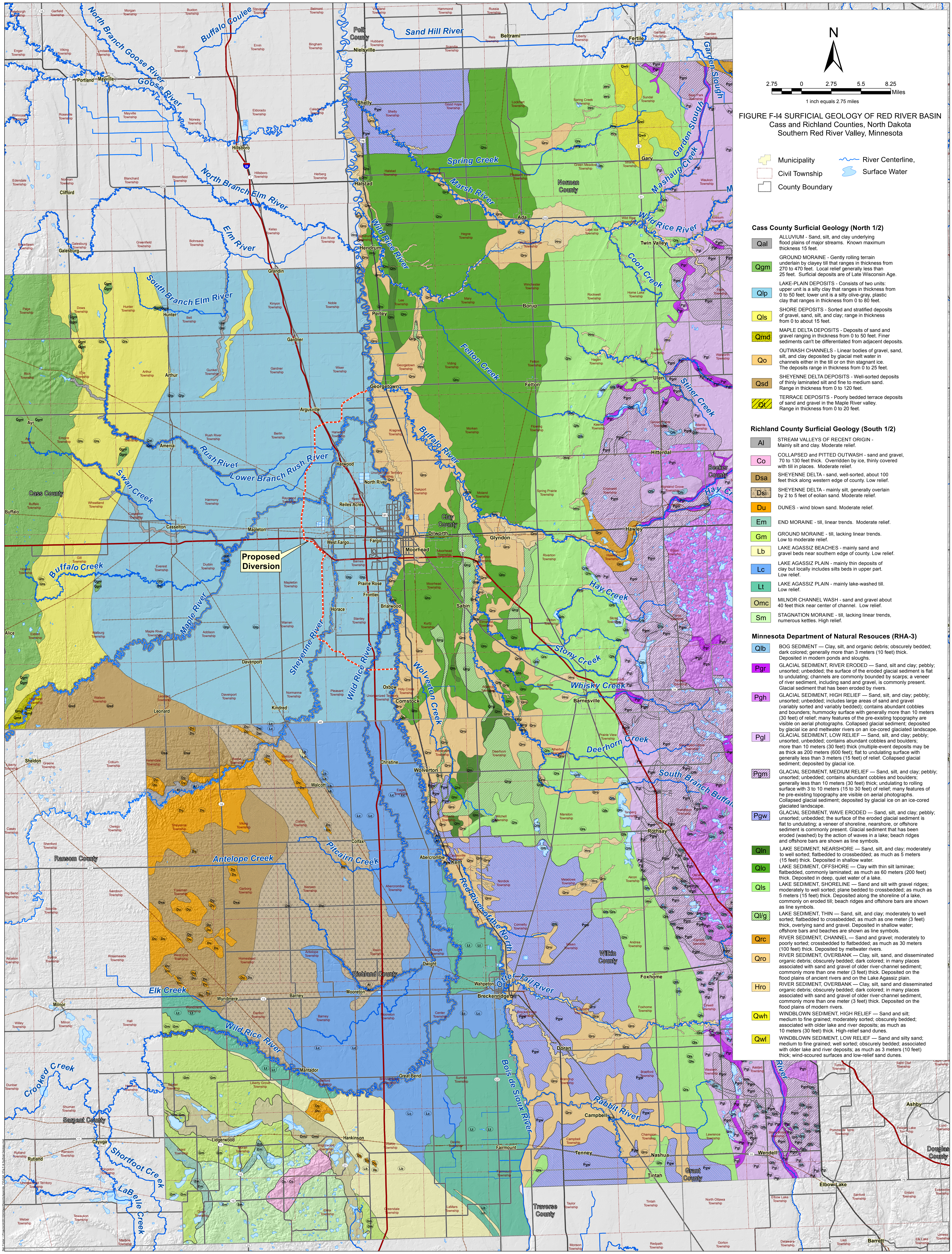


FIGURE F-I4 SURFICIAL GEOLOGY OF RED RIVER BASIN
Cass and Richland Counties, North Dakota
Southern Red River Valley, Minnesota

Municipality
Civil Township
County Boundary
River Centerline,
Surface Water

Cass County Surficial Geology (North 1/2)

Qal	ALLUVIUM - Sand, silt, and clay underlying flood plains of major streams. Known maximum thickness 15 feet.
Qgm	GROUND MORAINE - Gently rolling terrain underlain by clayey till that ranges in thickness from 270 to 470 feet. Local relief generally less than 25 feet. Surficial deposits are of Late Wisconsin Age.
Qlp	LAKE-PLAIN DEPOSITS - Consists of two units: upper unit is a silty clay that ranges in thickness from 0 to 50 feet; lower unit is a silty olive-gray, plastic clay that ranges in thickness from 0 to 80 feet.
Qls	SHORE DEPOSITS - Sorted and stratified deposits of gravel, sand, silt, and clay; range in thickness from 0 to about 15 feet.
Qmd	MAPLE DELTA DEPOSITS - Deposits of sand and gravel ranging in thickness from 0 to 50 feet. Finer sediments can't be differentiated from adjacent deposits.
Qo	OUTWASH CHANNELS - Linear bodies of gravel, sand, silt, and clay deposited by glacial melt water in channels either in the till or on thin stagnant ice. The deposits range in thickness from 0 to 25 feet.
Qsd	SHEYENNE DELTA DEPOSITS - Well-sorted deposits of thinly laminated silt and fine to medium sand. Range in thickness from 0 to 120 feet.
Qt	TERRACE DEPOSITS - Poorly bedded terrace deposits of sand and gravel in the Maple River valley. Range in thickness from 0 to 20 feet.

Richland County Surficial Geology (South 1/2)

Al	STREAM VALLEYS OF RECENT ORIGIN - Mainly silt and clay. Moderate relief.
Co	COLLAPSED and PITTED OUTWASH - sand and gravel, 70 to 130 feet thick. Overridden by ice, thinly covered with till in places. Moderate relief.
Dsa	SHEYENNE DELTA - sand, well-sorted, about 100 feet thick along western edge of county. Low relief.
Dsi	SHEYENNE DELTA - mainly silt, generally overlain by 2 to 5 feet of eolian sand. Moderate relief.
Du	DUNES - wind blown sand. Moderate relief.
Em	END MORAINE - till, linear trends. Moderate relief.
Gm	GROUND MORAINE - till, lacking linear trends. Low to moderate relief.
Lb	LAKE AGASSIZ BEACHES - mainly sand and gravel beds near southern edge of county. Low relief.
Lc	LAKE AGASSIZ PLAIN - mainly thin deposits of clay but locally includes silts beds in upper part. Low relief.
Lt	LAKE AGASSIZ PLAIN - mainly lake-washed till. Low relief.
Omc	MILNOR CHANNEL WASH - sand and gravel about 40 feet thick near center of channel. Low relief.
Sm	STAGNATION MORAINE - till, lacking linear trends, numerous kettles. High relief.

Minnesota Department of Natural Resouces (RHA-3)

Qlb	BOG SEDIMENT — Clay, silt, and organic debris; obscurely bedded; dark colored; generally more than 3 meters (10 feet) thick. Deposited in modern ponds and sloughs.
Pgr	GLACIAL SEDIMENT, RIVER ERODED — Sand, silt and clay; pebbly; unsorted; unbedded; the surface of the eroded glacial sediment is flat to undulating; channels are commonly bounded by scarps; a veneer of river sediment, including sand and gravel, is commonly present. Glacial sediment that has been eroded by rivers.
Pgh	GLACIAL SEDIMENT, HIGH RELIEF — Sand, silt, and clay; pebbly; unsorted; unbedded; includes large areas of sand and gravel (variably sorted and variably bedded); contains abundant cobbles and bounders; hummocky surface with generally more than 10 meters (30 feet) of relief; many features of the pre-existing topography are visible on aerial photographs. Collapsed glacial sediment; deposited by glacial ice and meltwater rivers on an ice-cored glaciated landscape.
Pgl	GLACIAL SEDIMENT, LOW RELIEF — Sand, silt, and clay; pebbly; unsorted, unbedded; contains abundant cobbles and boulders; more than 10 meters (30 feet) thick (multiple-event deposits may be as thick as 200 meters (600 feet); flat to undulating surface with generally less than 3 meters (15 feet) of relief. Collapsed glacial sediment; deposited by glacial ice.
Pgm	GLACIAL SEDIMENT, MEDIUM RELIEF — Sand, silt, and clay; pebbly; unsorted; unbedded; contains abundant cobbles and boulders; generally less than 10 meters (30 feet) thick; undulating to rolling surface with 3 to 10 meters (15 to 30 feet) of relief; many features of he pre-existing topography are visible on aerial photographs. Collapsed glacial sediment; deposited by glacial ice on an ice-cored glaciated landscape.
Pgw	GLACIAL SEDIMENT, WAVE ERODED — Sand, silt, and clay; pebbly; unsorted; unbedded; the surface of the eroded glacial sediment is flat to undulating; a veneer of shoreline, nearshore, or offshore sediment is commonly present. Glacial sediment that has been eroded (washed) by the action of waves in a lake; beach ridges and offshore bars are shown as line symbols.
Qln	LAKE SEDIMENT, NEARSHORE — Sand, silt, and clay; moderately to well sorted; flatbedded to crossbedded; as much as 5 meters (15 feet) thick. Deposited in shallow water.
Qlo	LAKE SEDIMENT, OFFSHORE — Clay with thin silt laminae; flatbedded, commonly laminated; as much as 60 meters (200 feet) thick. Deposited in deep, quiet water of a lake.
Qls	LAKE SEDIMENT, SHORELINE — Sand and silt with gravel ridges; moderately to well sorted; plane bedded to crossbedded; as much as 5 meters (15 feet) thick. Deposited along the shoreline of a lake, commonly on eroded till; beach ridges and offshore bars are shown as line symbols.
Ql/g	LAKE SEDIMENT, THIN — Sand, silt, and clay; moderately to well sorted; flatbedded to crossbedded; as much as one meter (3 feet) thick, overlying sand and gravel. Deposited in shallow water; offshore bars and beaches are shown as line symbols.
Qrc	RIVER SEDIMENT, CHANNEL — Sand and gravel; moderately to poorly sorted; crossbedded to flatbedded; as much as 30 meters (100 feet) thick. Deposited by meltwater rivers.
Qro	RIVER SEDIMENT, OVERBANK — Clay, silt, sand, and disseminated organic debris; obscurely bedded; dark colored; in many places associated with sand and gravel of older river-channel sediment; commonly more than one meter (3 feet) thick. Deposited on the flood plains of ancient rivers and on the Lake Agassiz plain.
Hro	RIVER SEDIMENT, OVERBANK — Clay, silt, sand and disseminated organic debris; obscurely bedded; dark colored; in many places associated with sand and gravel of older river-channel sediment, commonly more than one meter (3 feet) thick. Deposited on the flood plains of modern rivers.
Qwh	WINDBLOWN SEDIMENT, HIGH RELIEF — Sand and silt; medium to fine grained; moderately sorted; obscurely bedded; associated with older lake and river deposits; as much as 10 meters (30 feet) thick. High-relief sand dunes.
Qwl	WINDBLOWN SEDIMENT, LOW RELIEF — Sand and silty sand; medium to fine grained; well sorted; obscurely bedded; associated with older lake and river deposits; as much as 3 meters (10 feet) thick; wind-scoured surfaces and low-relief sand dunes.

Figure F-I 5 Percent Sand in Surface Soils

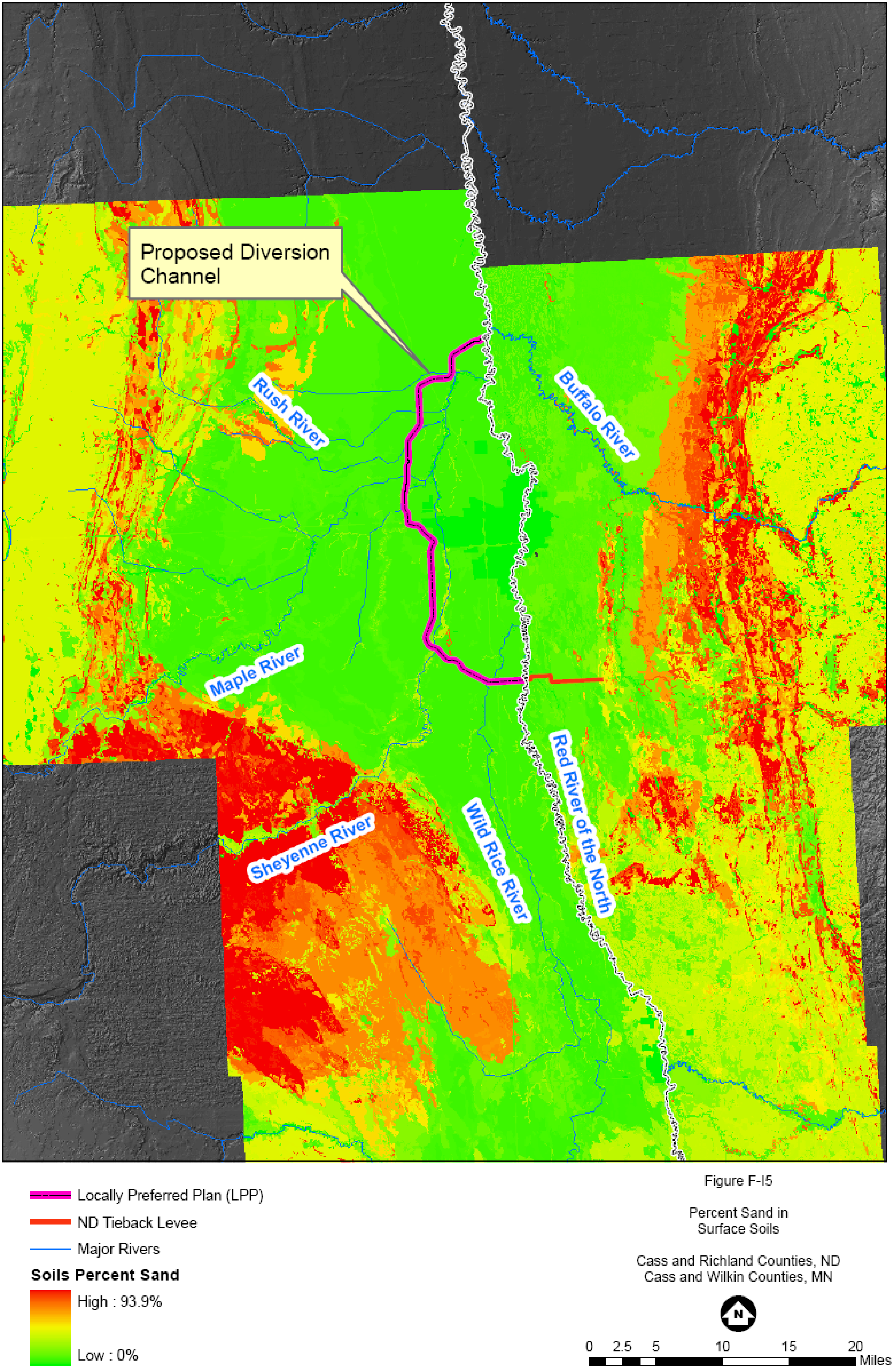


Figure F-I 6 Locations of Available Sediment Data

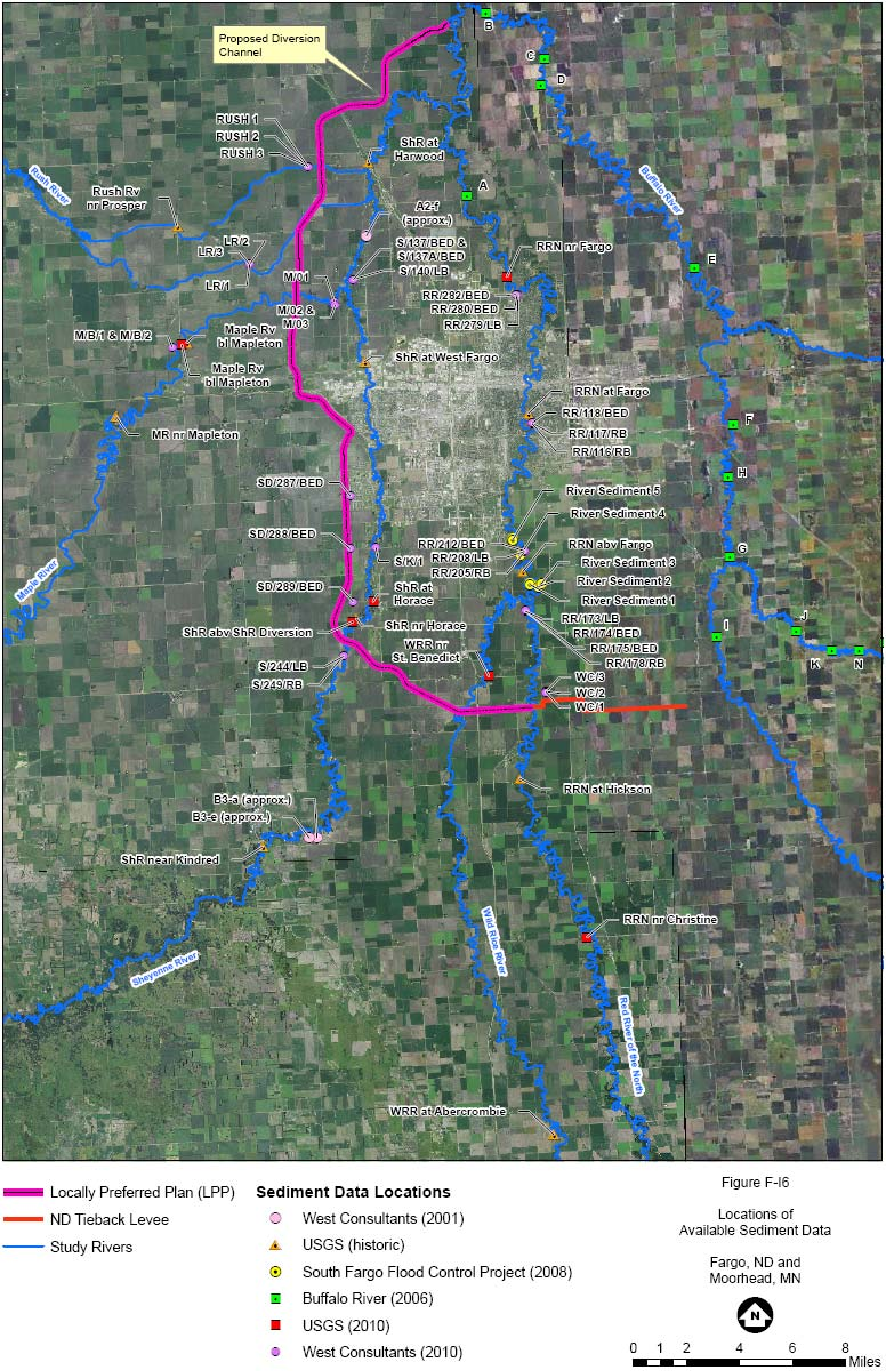


Table F-I1 Available Historic Sediment Data

USGS Site ID	Site Name	Suspended Sediment Concentration	Suspended Sediment % Finer than Sand	Suspended Sediment Gradation	Bed Sediment Gradation	Used in this Analysis
05053000	Wild Rice River near Abercrombie, ND	1975-1981, 1994 (n = 62)	N/A	1975-1979. 1994 (n = 9)	N/A	YES
05059000	Sheyenne River near Kindred, ND	1975-1995, 2001 (n = 112)	1975-1995, 2001 (n = 106)	1977-1978, 1980, 2001 (n = 11)	1976-1978 (n = 12)	YES
05060400	Sheyenne River at Harwood, ND	1997-1999 (n = 21)	1997-1999 (n = 20)	1997-1999 (n = 7)	N/A	YES
05060000	Maple River near Mapleton	1975 (n = 1)	N/A	N/A	N/A	NO
05051522	Red River of the North at Hickson	1975-1981, 1997-1999, 2003 (n = 102)	1975-1980, 1997-1999, 2003 (n = 45)	1978-1980, 1998-1999 (n = 10)	1976, 1978-1980 (n = 7)	YES
05053800	Red River of the North above Fargo	1994-1999 (n = 38)	1994-1999 (n = 38)	1998-1999 (n = 4)	N/A	YES
05054000	Red River of the North at Fargo	1975, 2001, 2003-2010 (n = 78)	1975, 2001, 2003-2010 (n = 77)	N/A	N/A	YES

Table F-I2 2010 Observed Sediment Loading

Site Name	Time Period[1]	Total Flow (million ft^3)	Total Suspended Sediment Load (tons)	Total Bedload (tons)[2]	Total Sediment Load (tons)
Wild Rice River near St. Benedict	March 18, 2010 - March 31, 2010	8,780	43,260	31.8 (0.07%)	43,300
Sheyenne River above Sheyenne River Diversion near Horace	March 24, 2010 - April 7, 2010	5,340	119,590	40.7 (0.03%)	119,630
Sheyenne River at Horace	March 24, 2010 - April 7, 2010	2,580	56,370	9.3 (0.01%)	56,380
Maple River below Mapleton	March 19, 2010 - April 6, 2010	4,660	31,520	70.9 (0.2%)	31,600
Red River of the North near Christine	March 18, 2010 - March 31, 2010	10,030	30,780	171 (0.6%)	30,950
Red River of the North near Fargo	March 18, 2010 - March 31, 2010	19,810	72,080	27.6 (0.04%)	72,110

[1] Time period shown does not represent the complete monitoring period for all sites. The periods shown represent concurrent data on the WRR and RRN (March 18, 2010 to March 31, 2010) and on the Sheyenne River above and below the Sheyenne River Diversion (March 24, 2010 to April 7, 2010).

[2] Percentage values represent bedload as a fraction of total sediment load for the time periods shown.

Figure F-I7 Suspended Sediment Concentrations (historic)

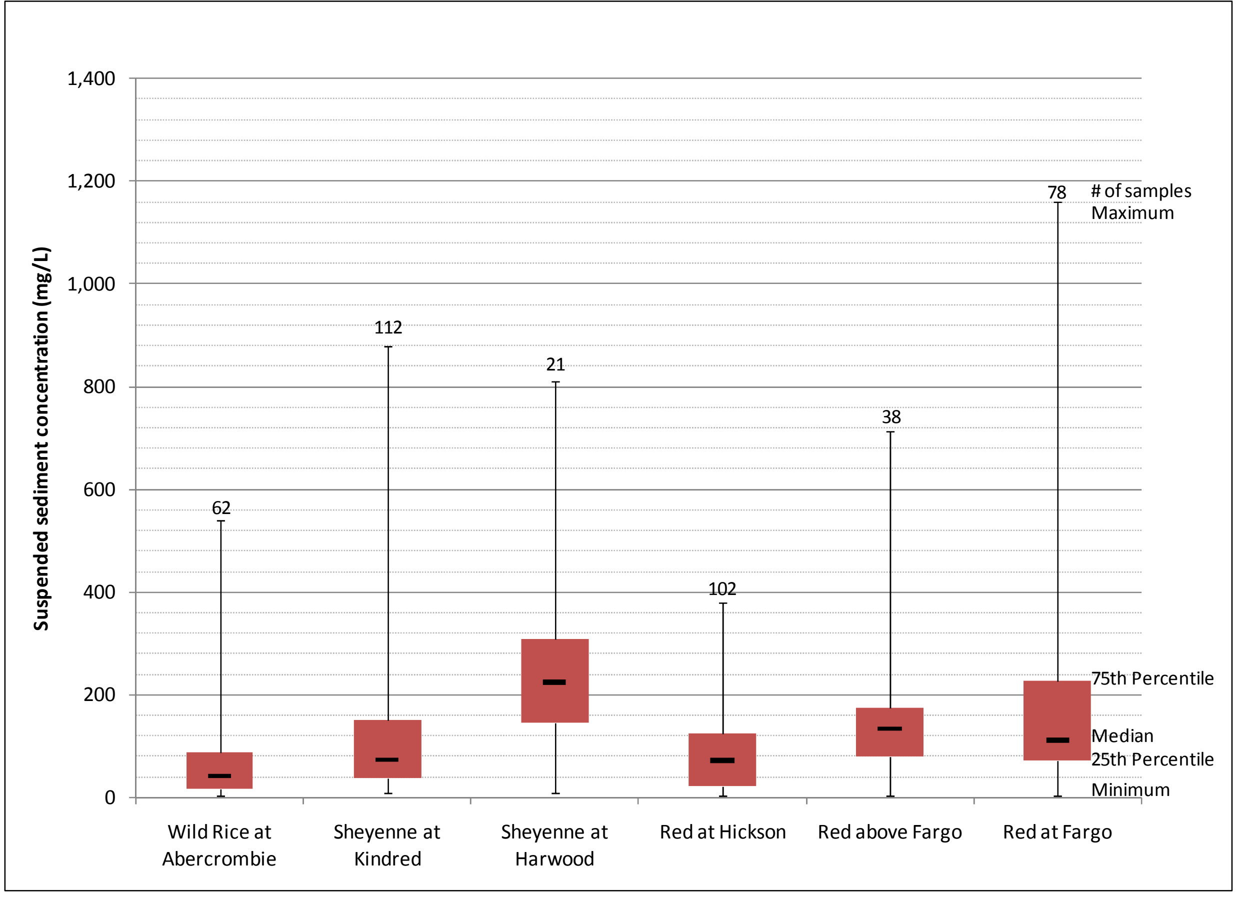


Figure F-I8 Suspended Sediment Percent Finer than Sand (historic)

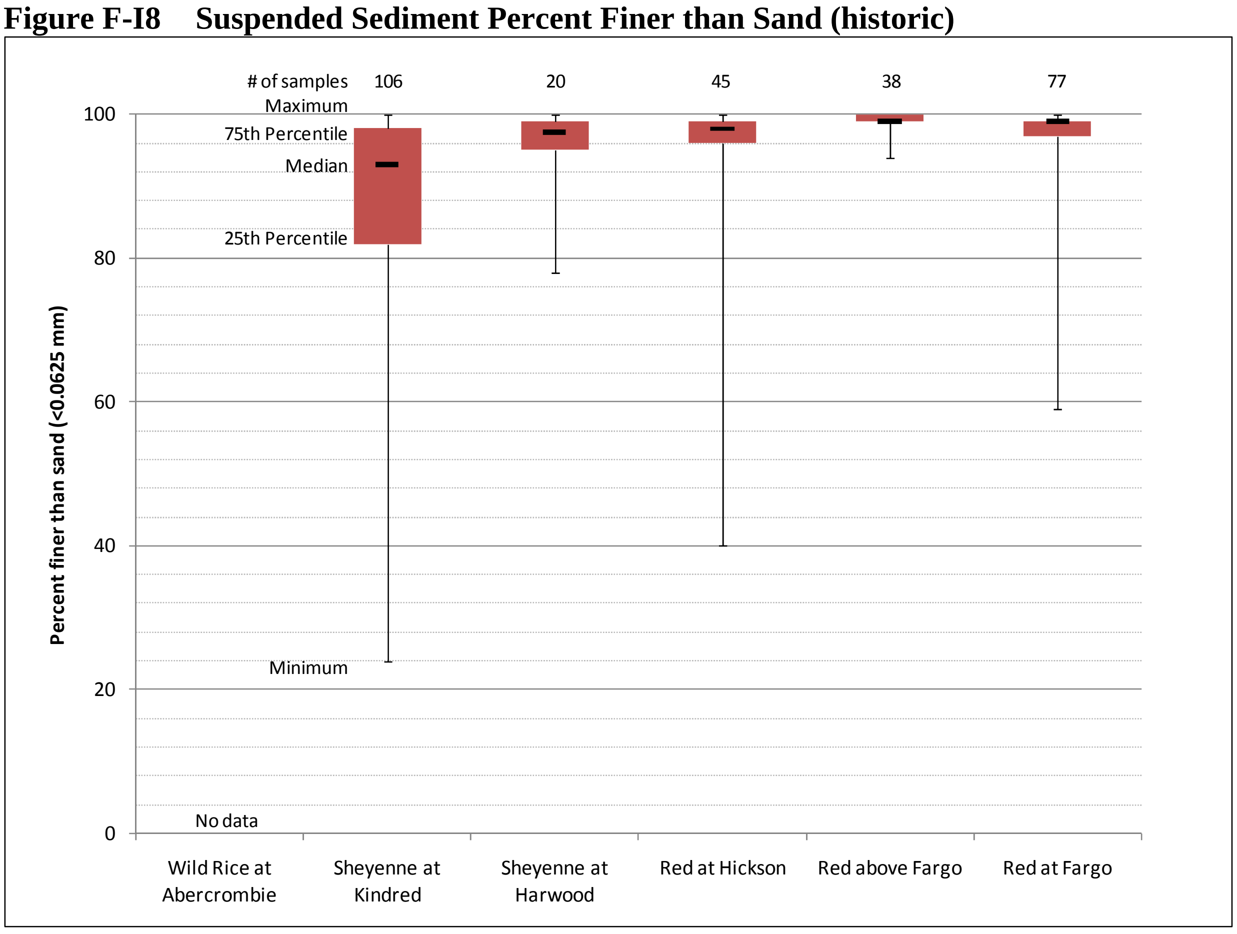
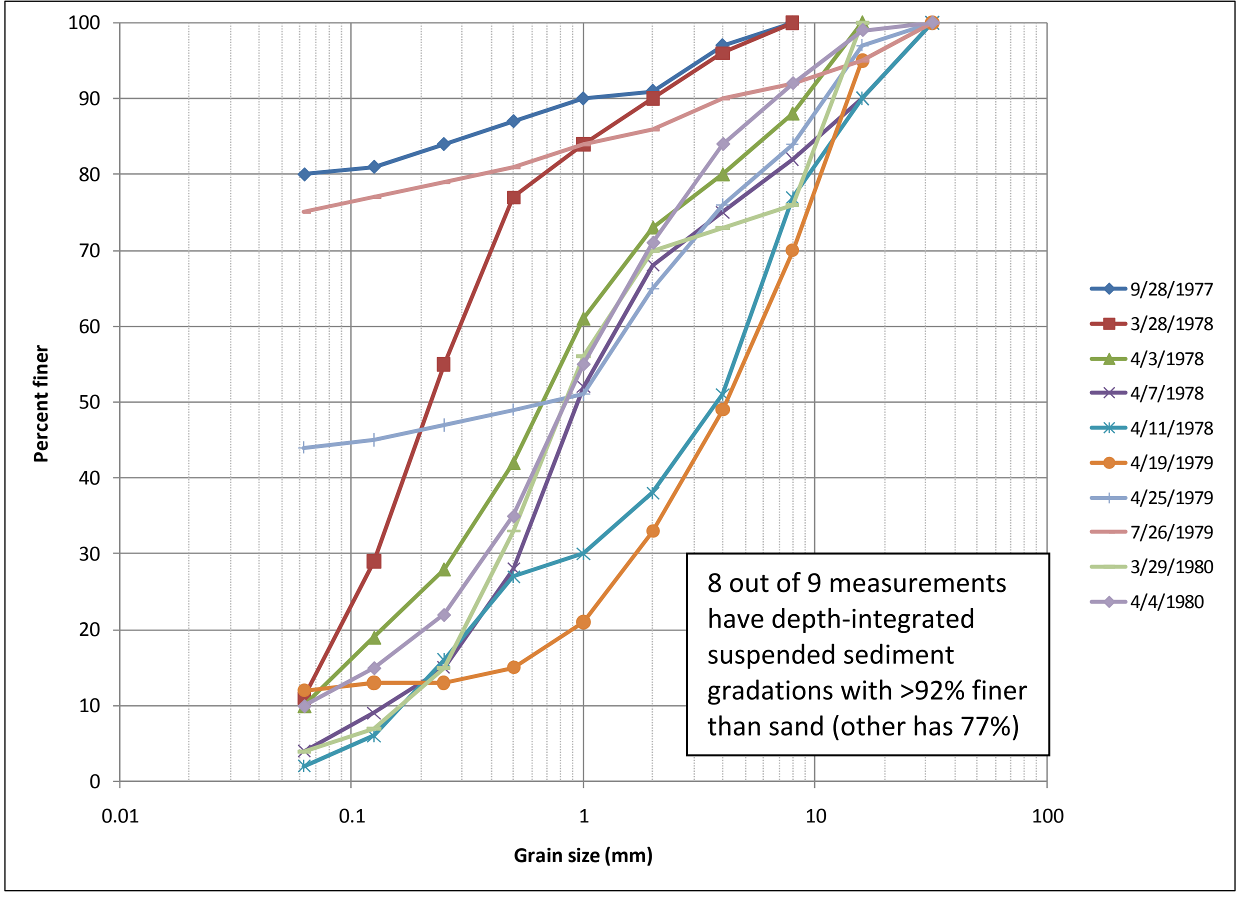
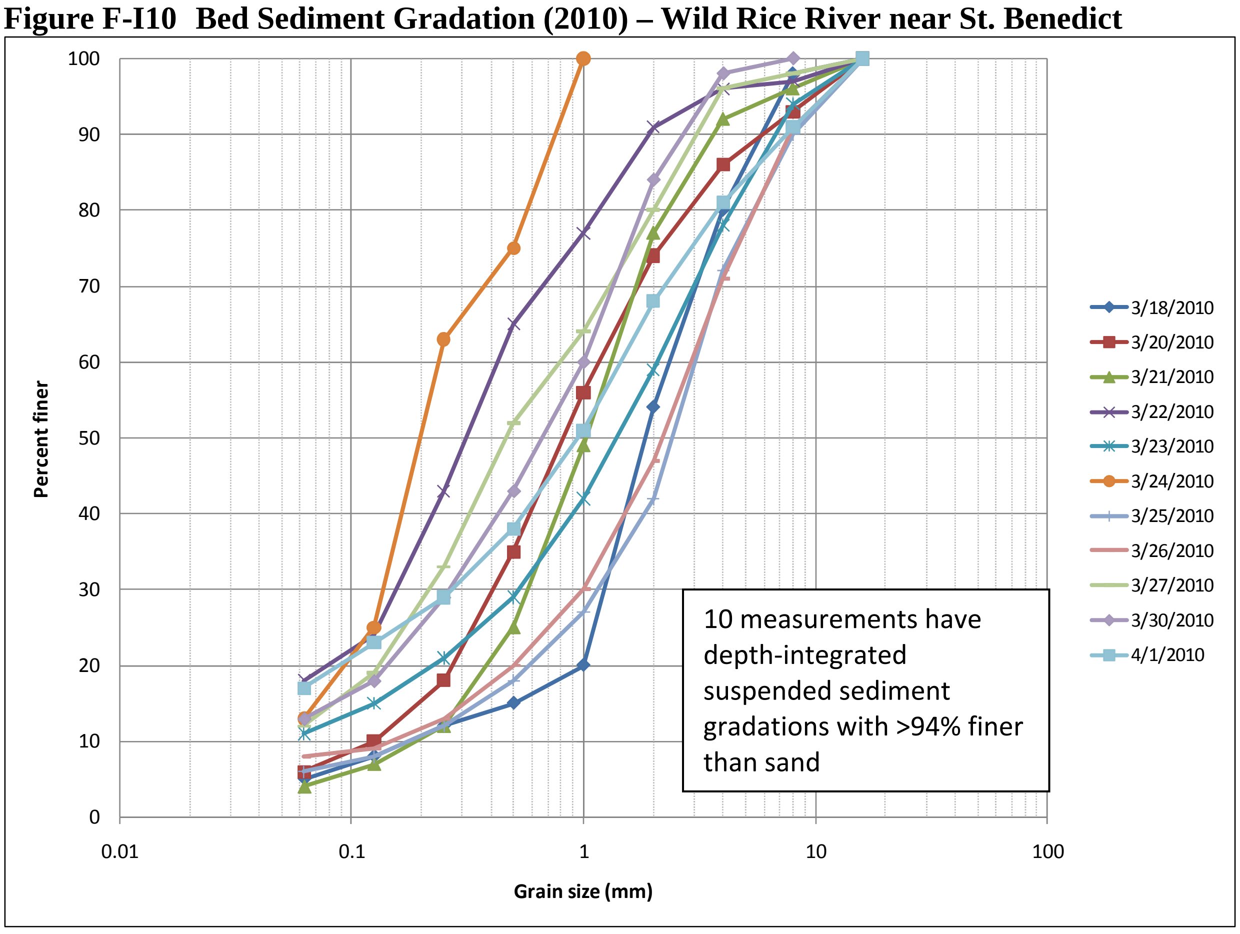


Figure F-I9 Bed Sediment Gradation (historic) – Wild Rice River near Abercrombie



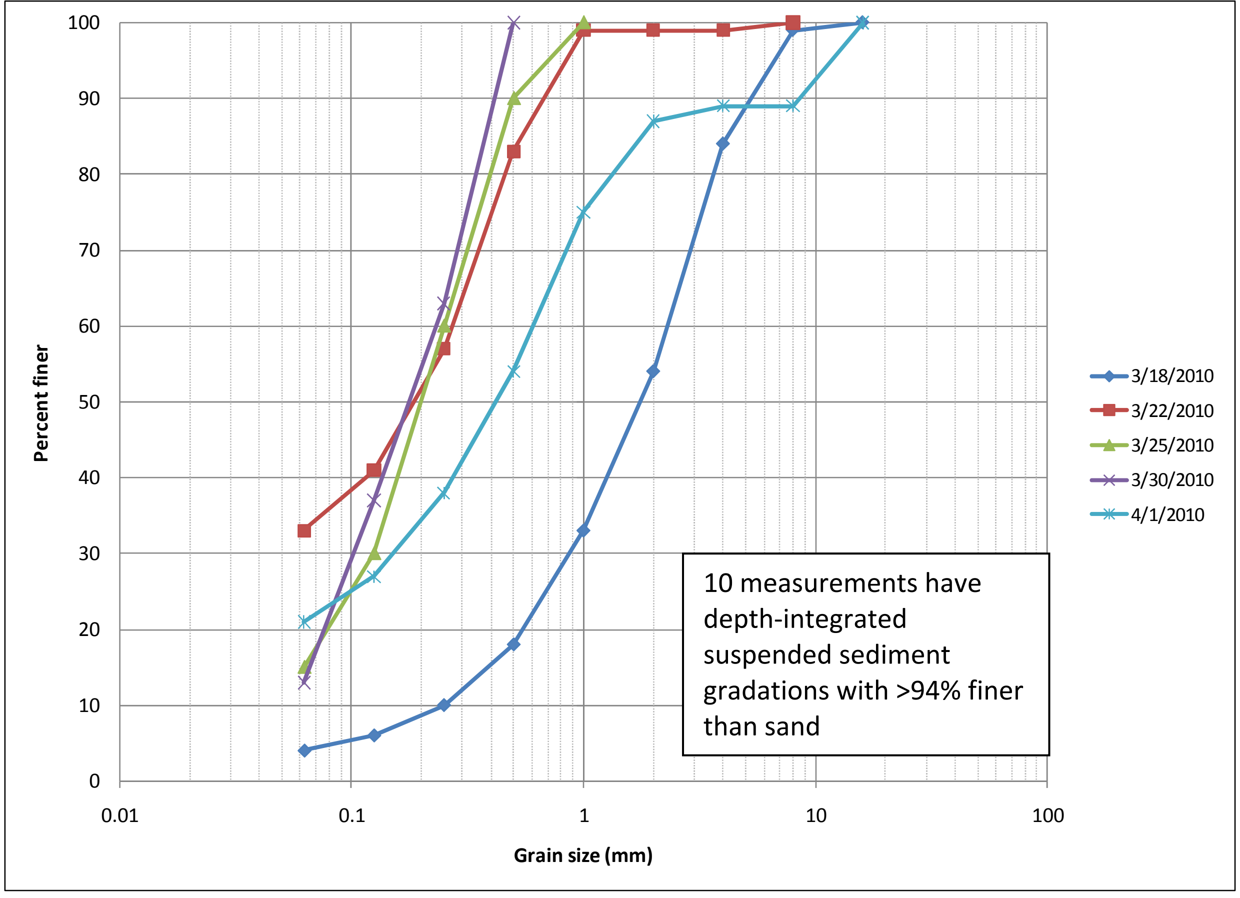
8 out of 9 measurements have depth-integrated suspended sediment gradations with >92% finer than sand (other has 77%)

Figure F-I10 Bed Sediment Gradation (2010) – Wild Rice River near St. Benedict



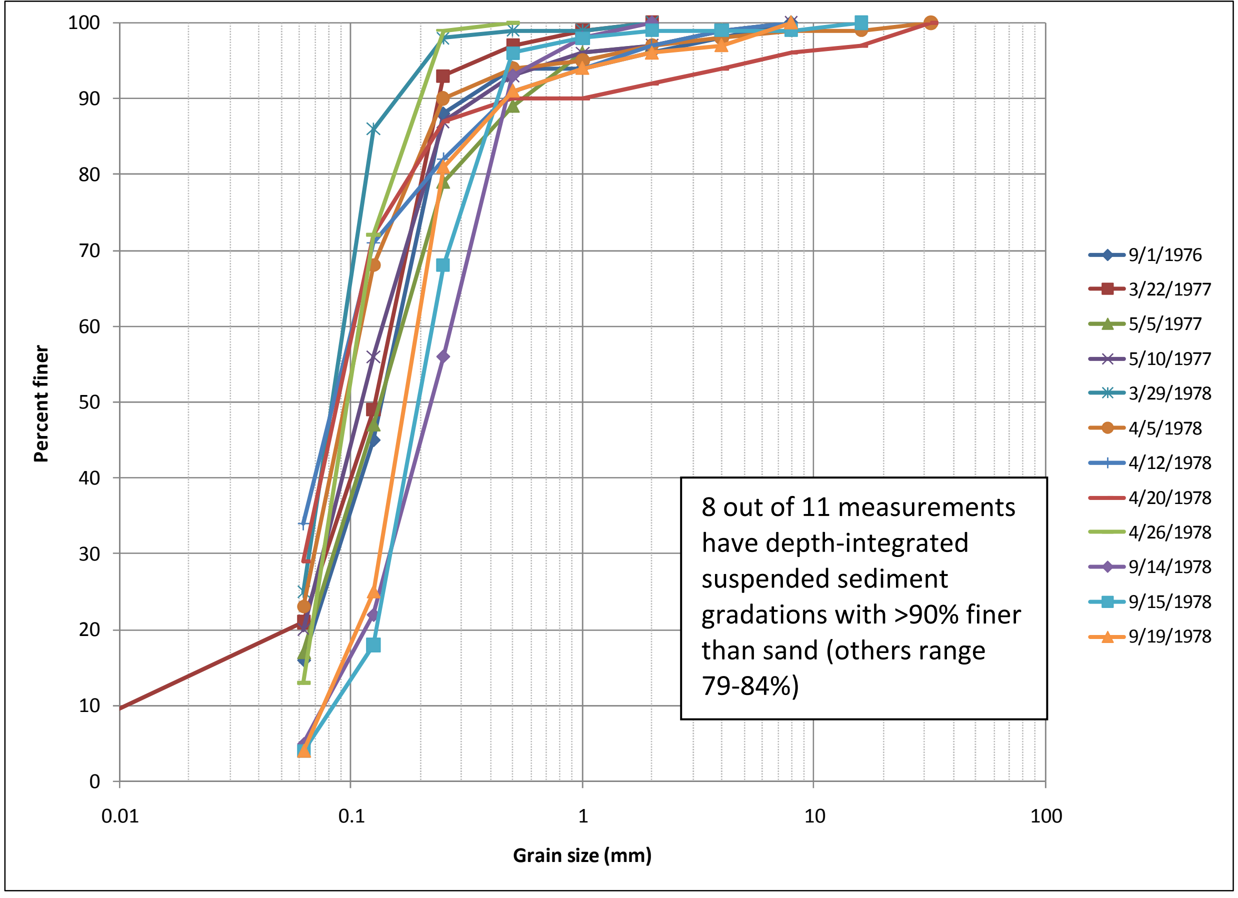
10 measurements have depth-integrated suspended sediment gradations with >94% finer than sand

Figure F-I11 Bedload Gradation (2010) – Wild Rice River near St. Benedict



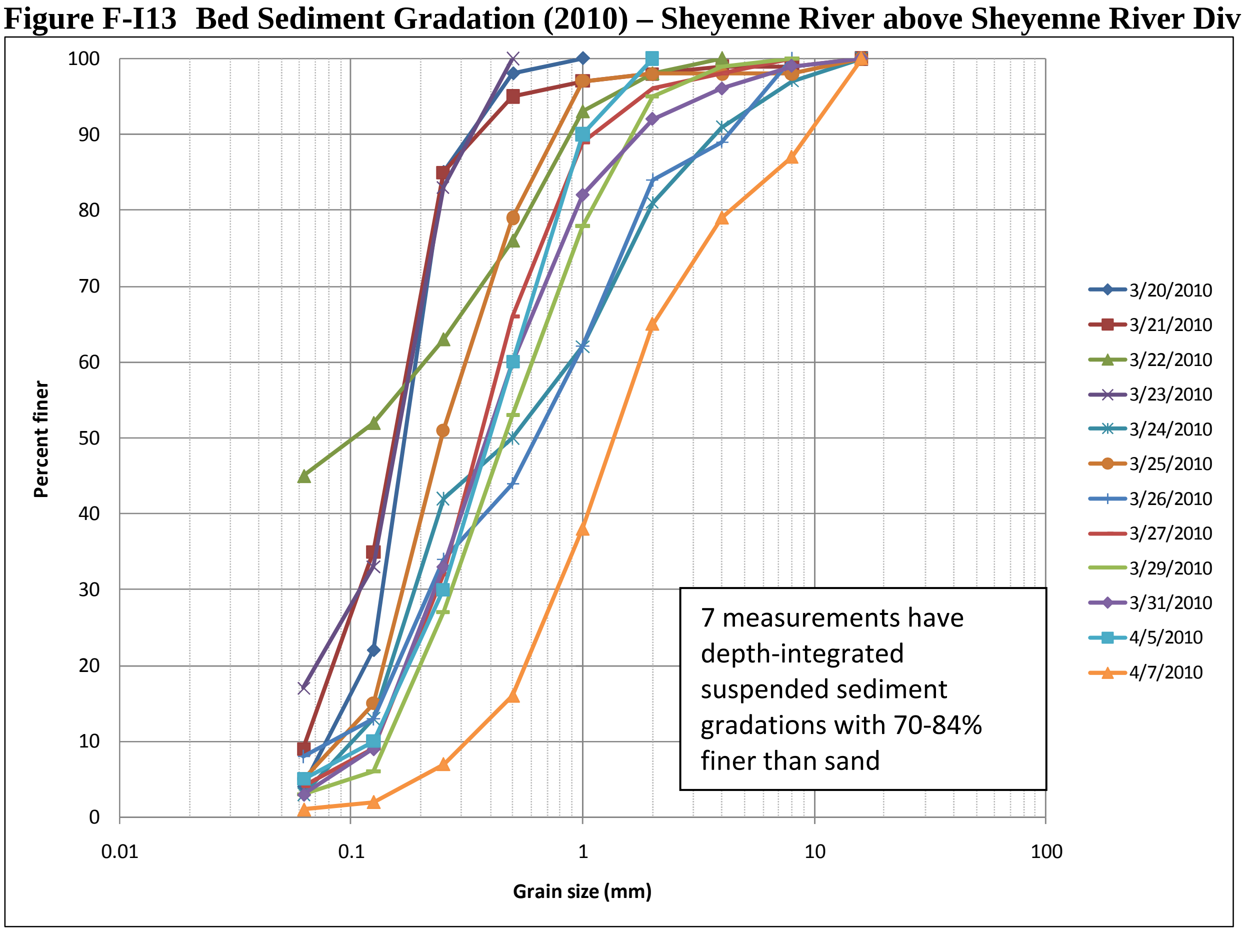
10 measurements have depth-integrated suspended sediment gradations with >94% finer than sand

Figure F-I12 Bed Sediment Gradation (historic) – Sheyenne River near Kindred



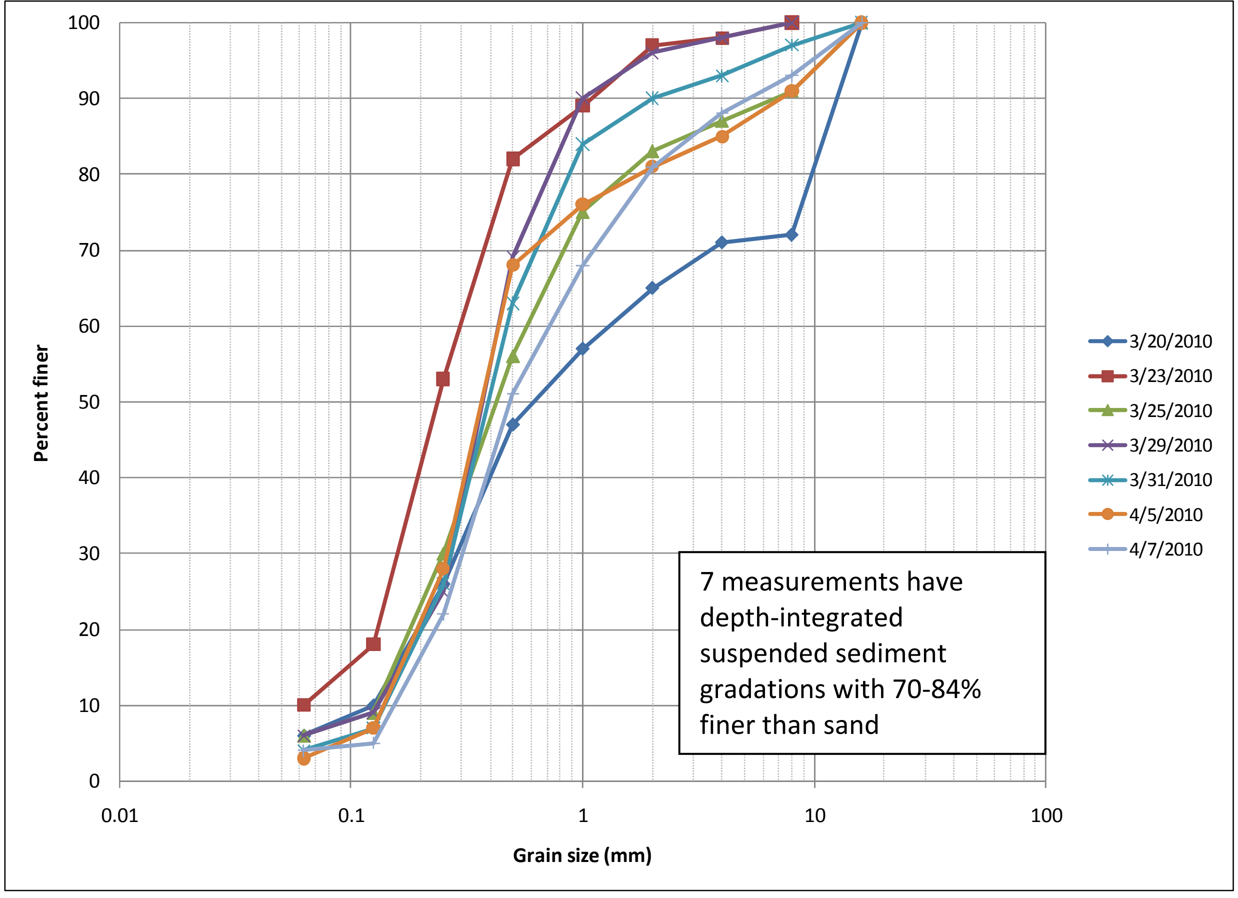
8 out of 11 measurements have depth-integrated suspended sediment gradations with >90% finer than sand (others range 79-84%)

Figure F-I13 Bed Sediment Gradation (2010) – Sheyenne River above Sheyenne River Diversion near Horace



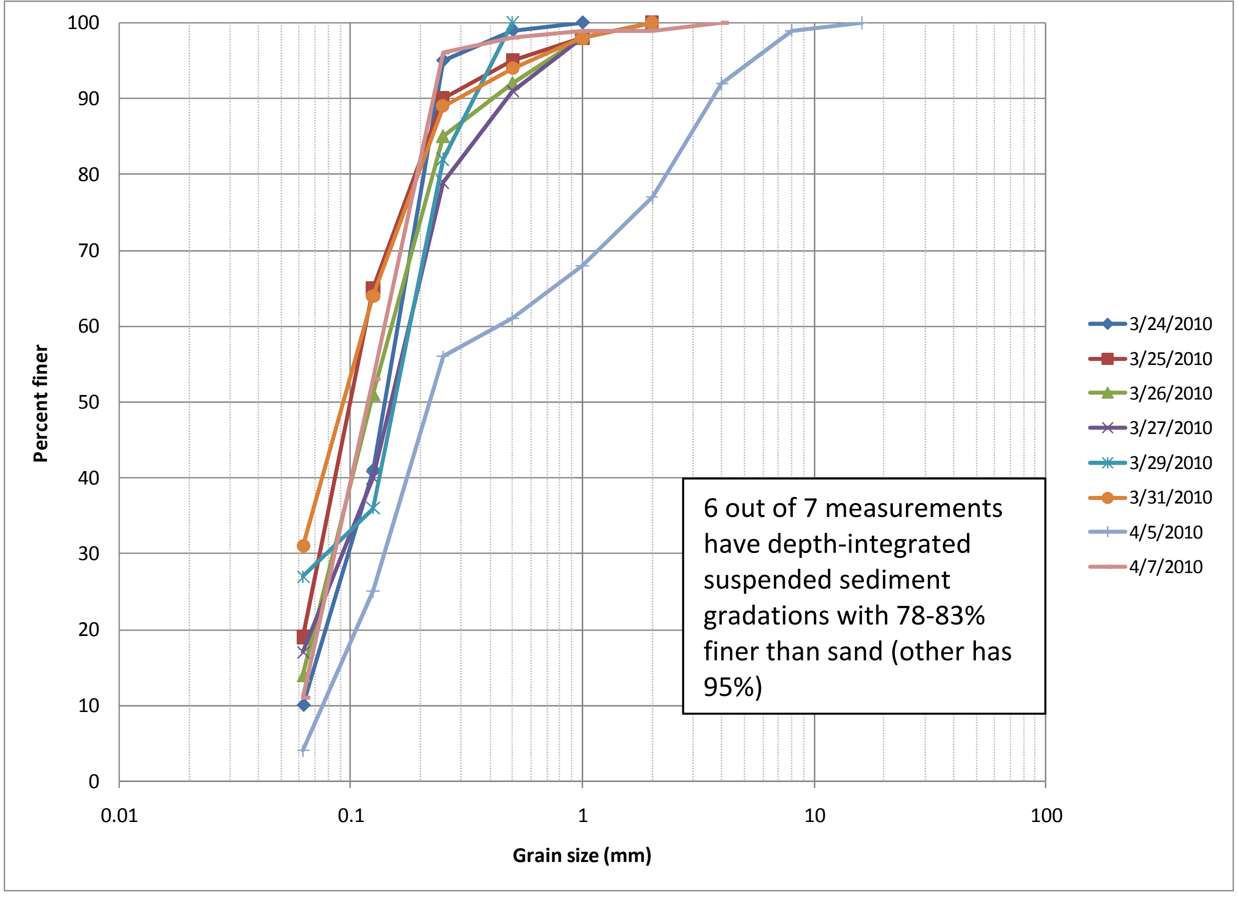
7 measurements have depth-integrated suspended sediment gradations with 70-84% finer than sand

Figure F-I14 Bedload Gradation (2010) – Sheyenne River above Sheyenne River Diversion near Horace



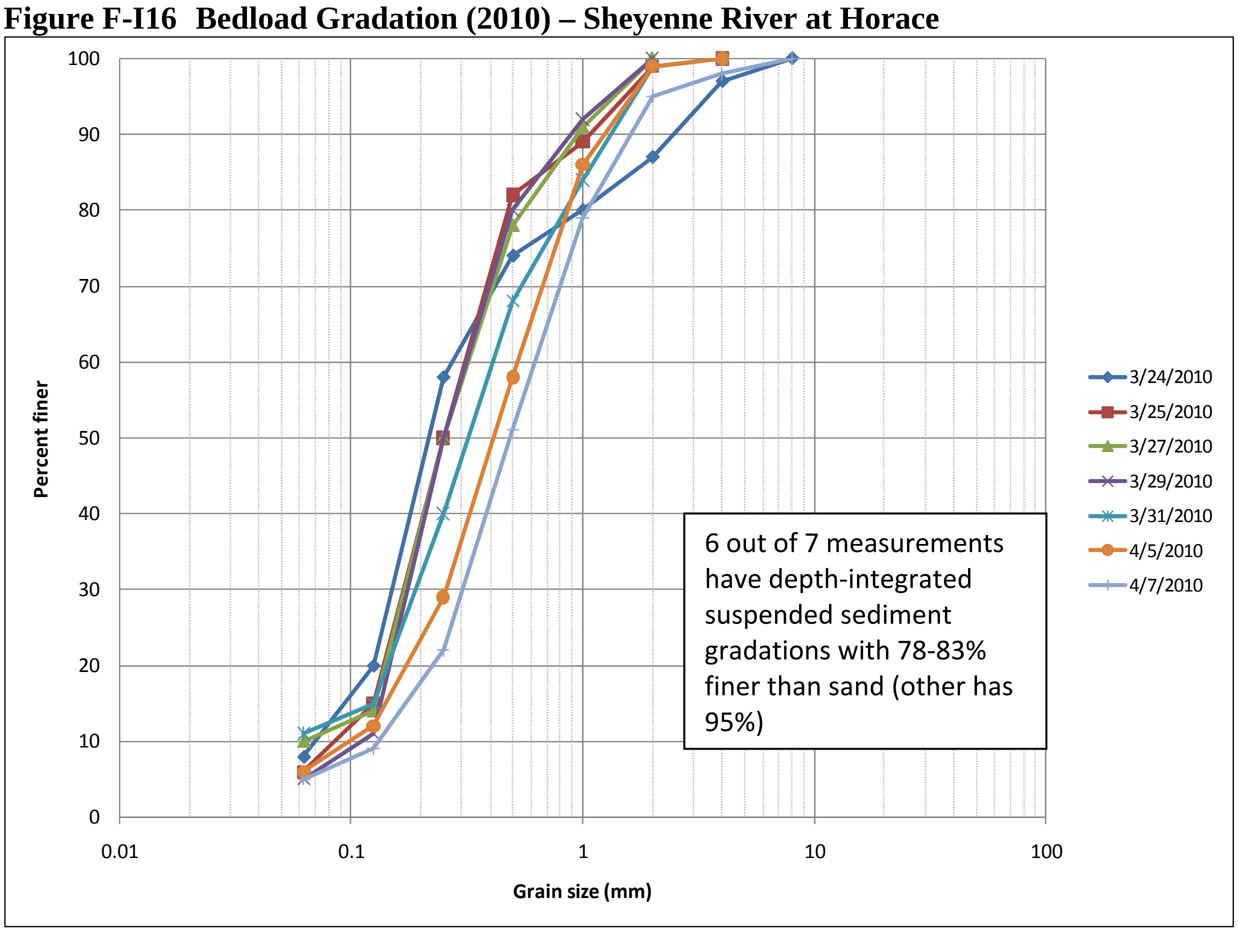
7 measurements have depth-integrated suspended sediment gradations with 70-84% finer than sand

Figure F-I15 Bed Sediment Gradation (2010) – Sheyenne River at Horace



6 out of 7 measurements have depth-integrated suspended sediment gradations with 78-83% finer than sand (other has 95%)

Figure F-I16 Bedload Gradation (2010) – Sheyenne River at Horace



6 out of 7 measurements have depth-integrated suspended sediment gradations with 78-83% finer than sand (other has 95%)

Figure F-I17 Bed Sediment Gradation (2010) – Maple River below Mapleton

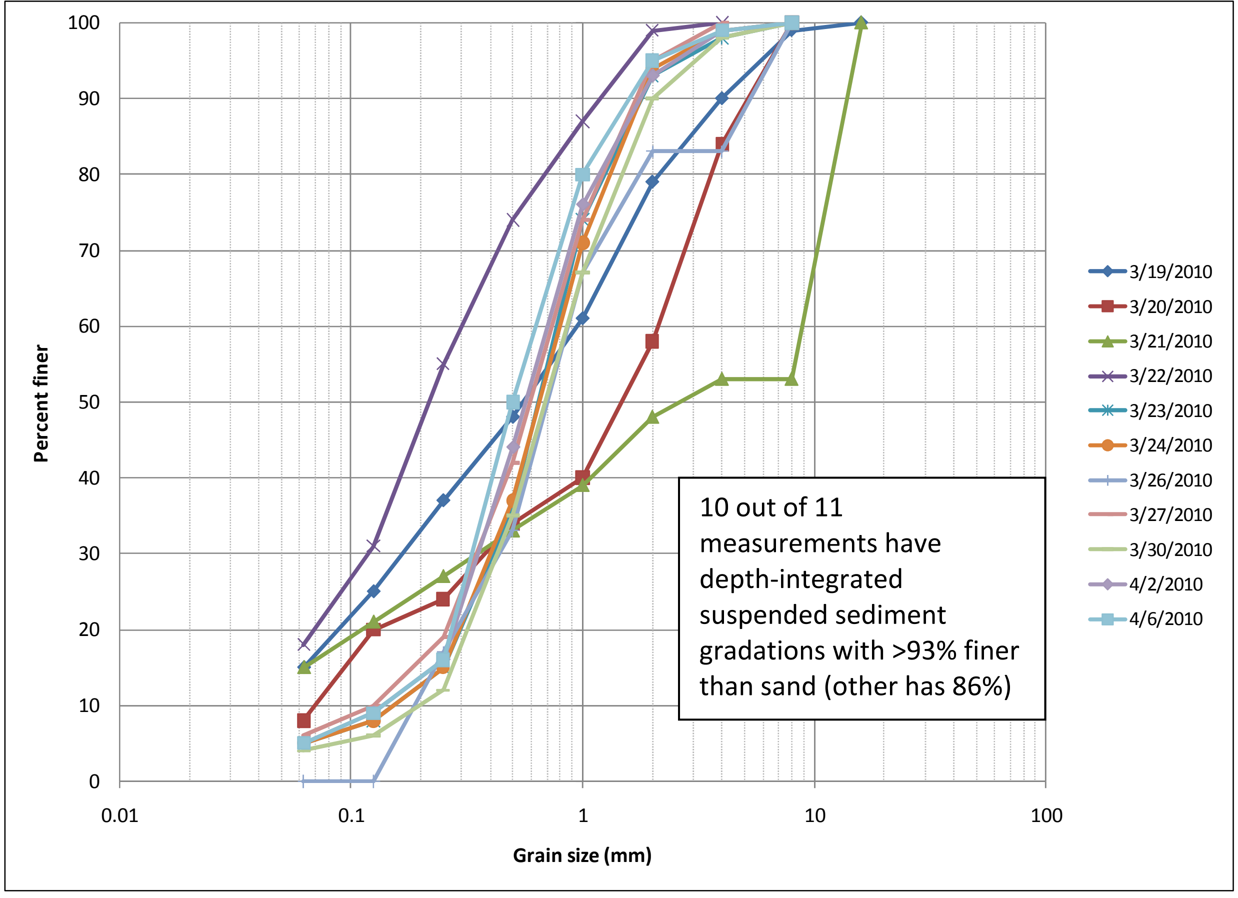


Figure F-I18 Bedload Gradation (2010) – Maple River below Mapleton

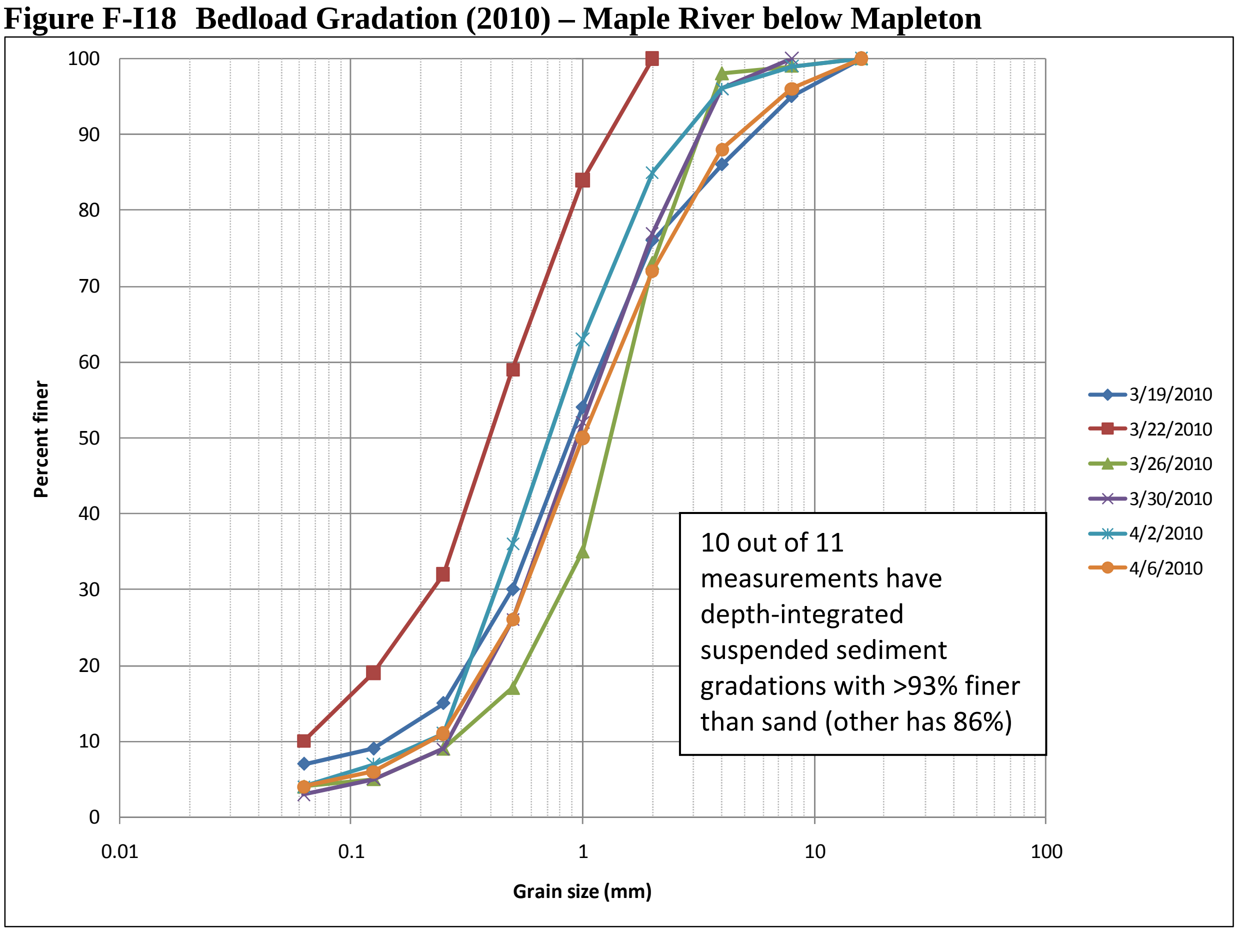
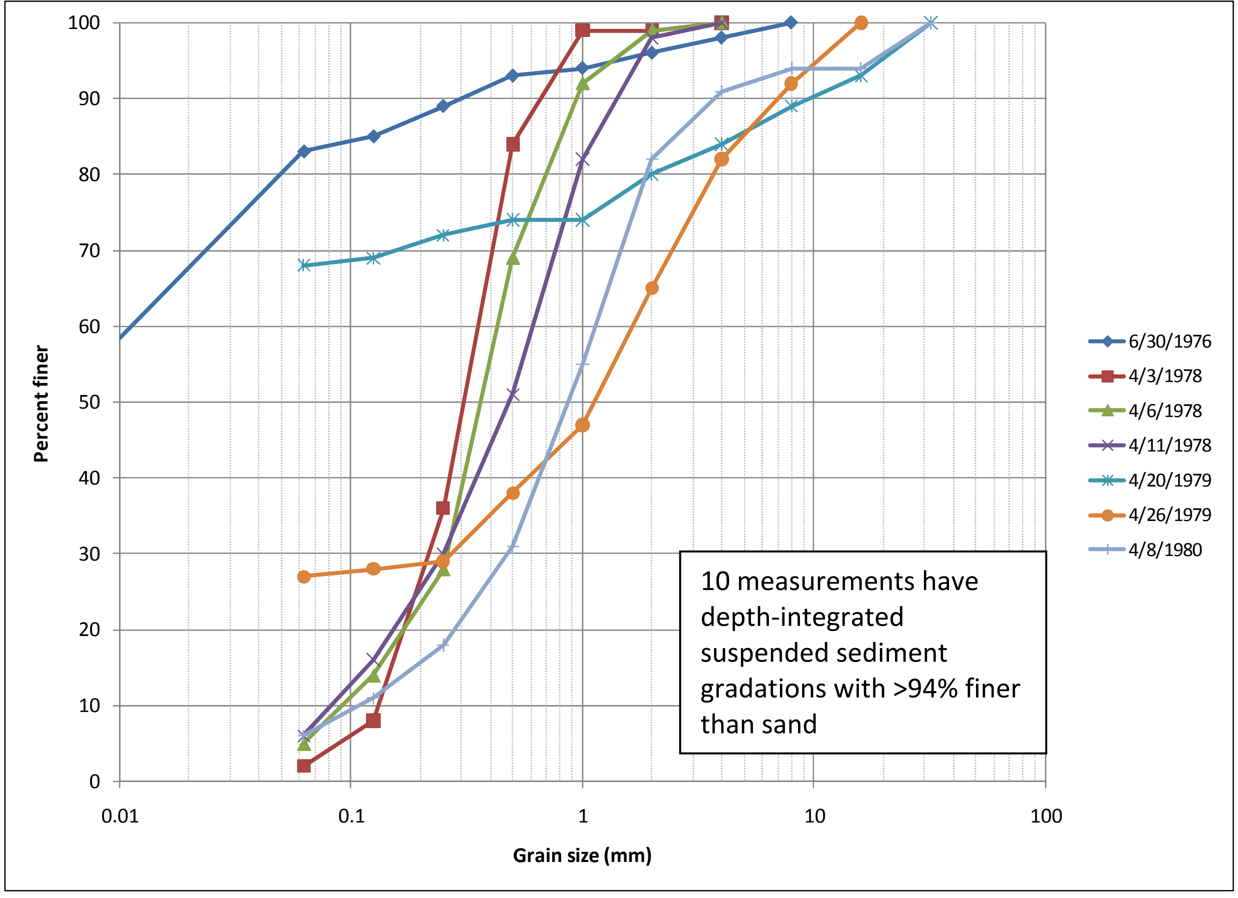
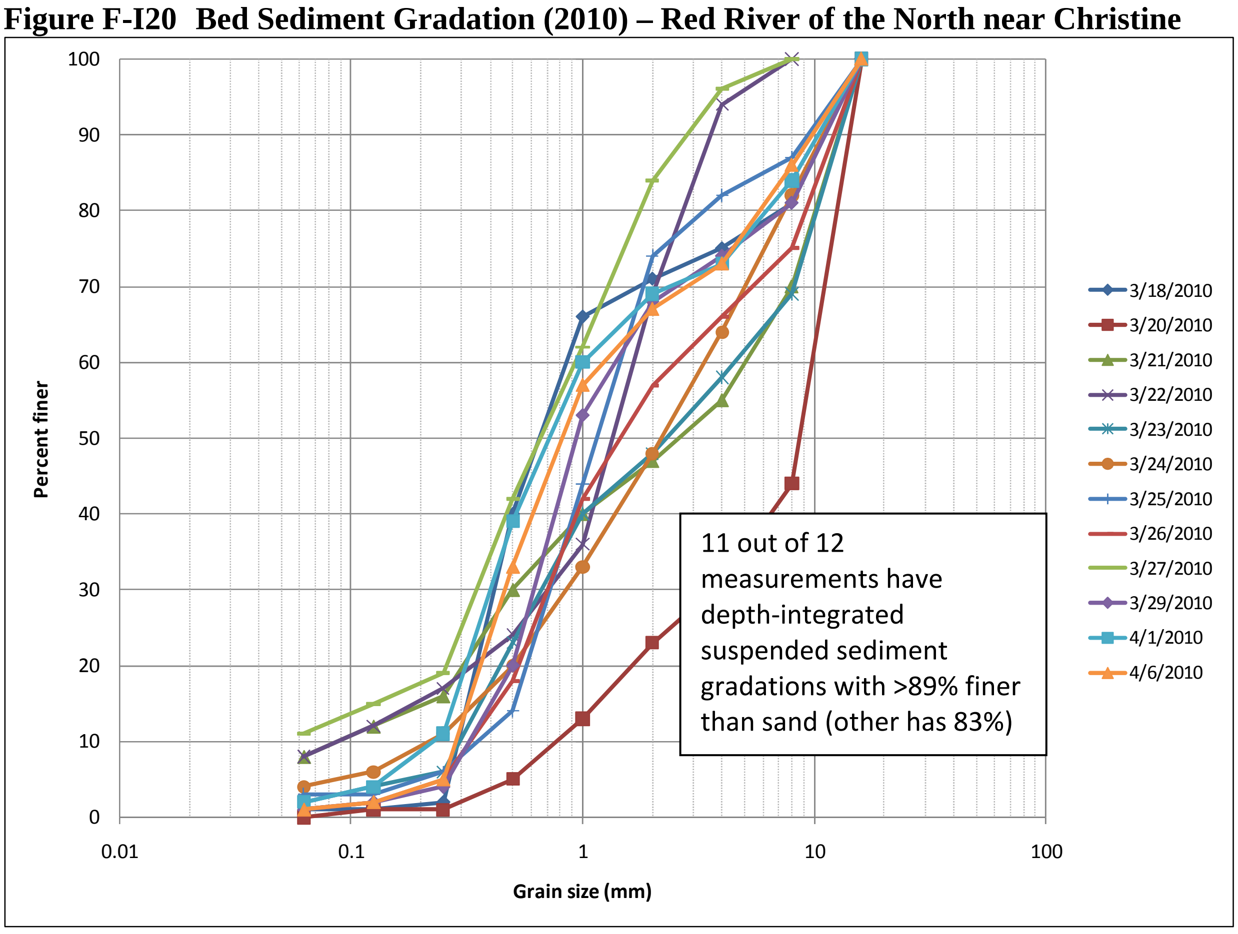


Figure F-I19 Bed Sediment Gradation (historic) – Red River of the North near Hickson



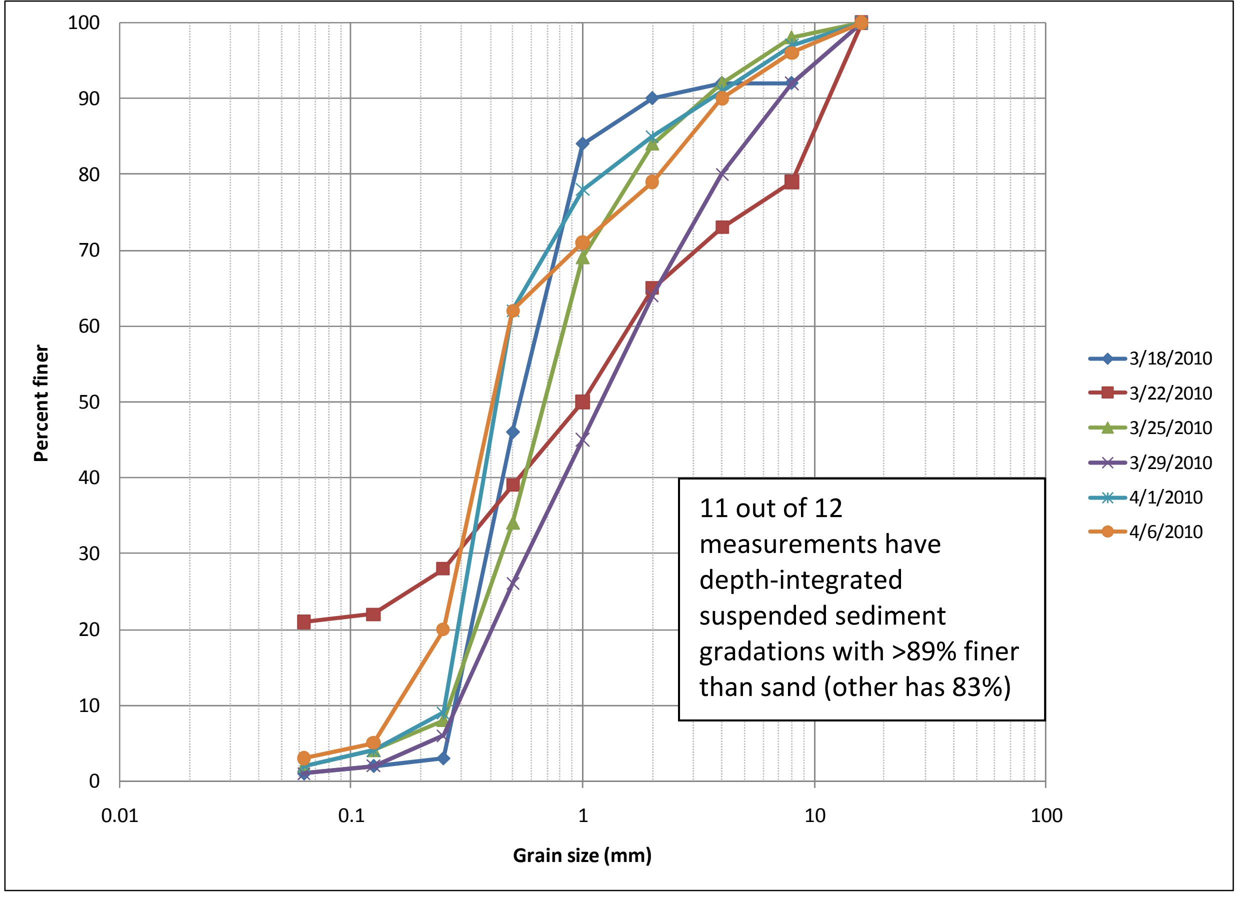
10 measurements have depth-integrated suspended sediment gradations with >94% finer than sand

Figure F-I20 Bed Sediment Gradation (2010) – Red River of the North near Christine



11 out of 12 measurements have depth-integrated suspended sediment gradations with >89% finer than sand (other has 83%)

Figure F-I21 Bedload Gradation (2010) – Red River of the North near Christine



11 out of 12 measurements have depth-integrated suspended sediment gradations with >89% finer than sand (other has 83%)

Figure F-I22 Bed Sediment Gradation (2010) – Red River of the North at Fargo

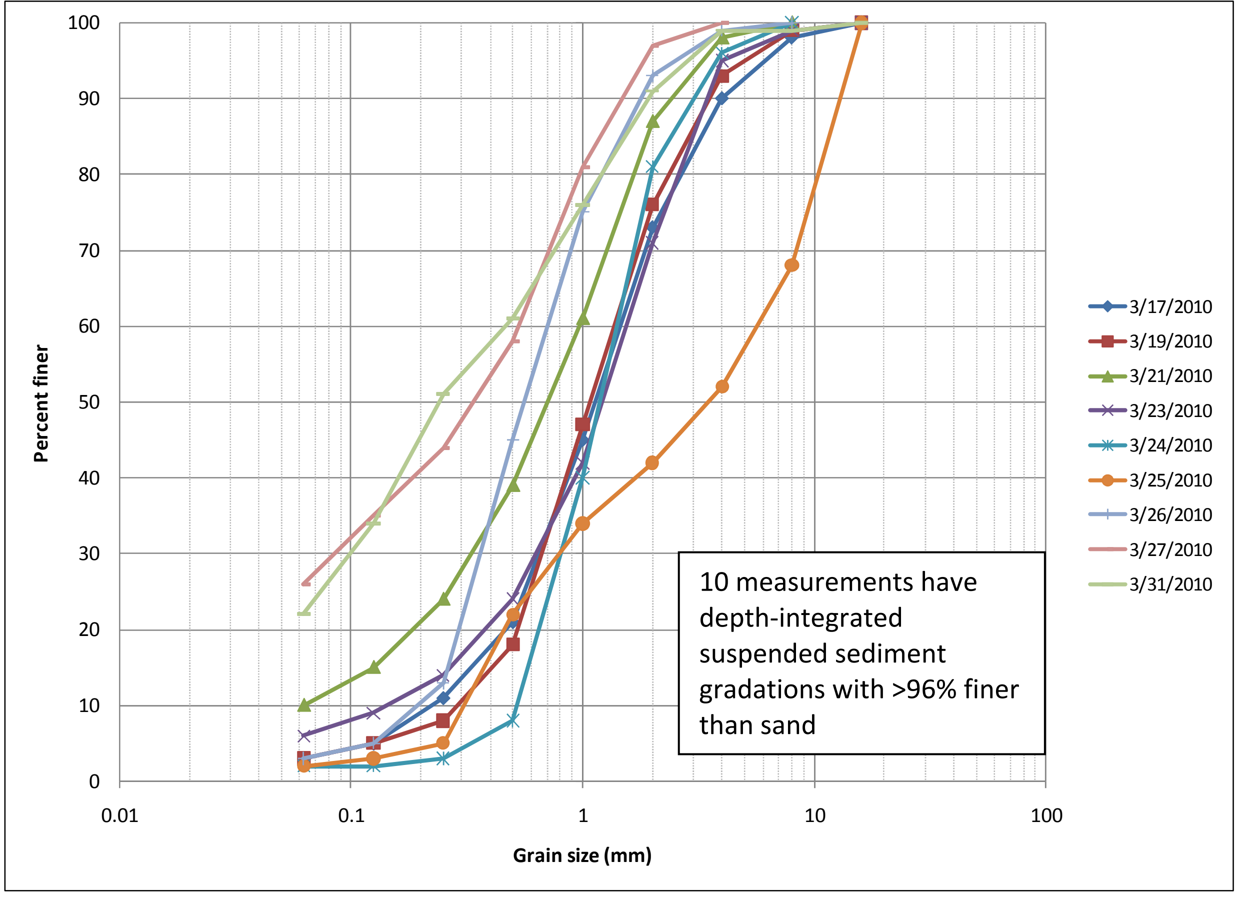


Figure F-I23 Bedload Gradation (2010) – Red River of the North at Fargo

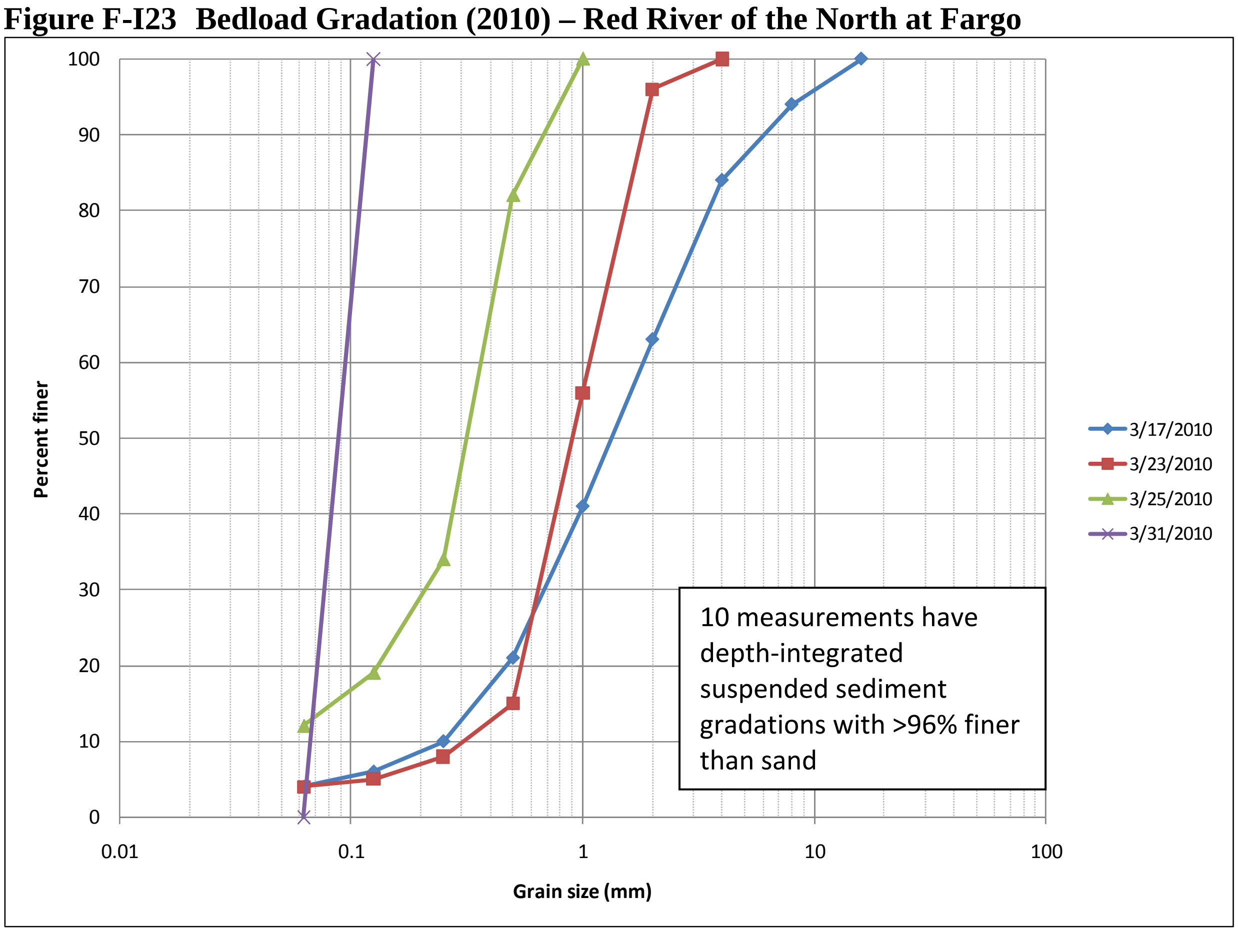


Figure F-I24 Flow and Sediment Balance (2010) – Red River of the North

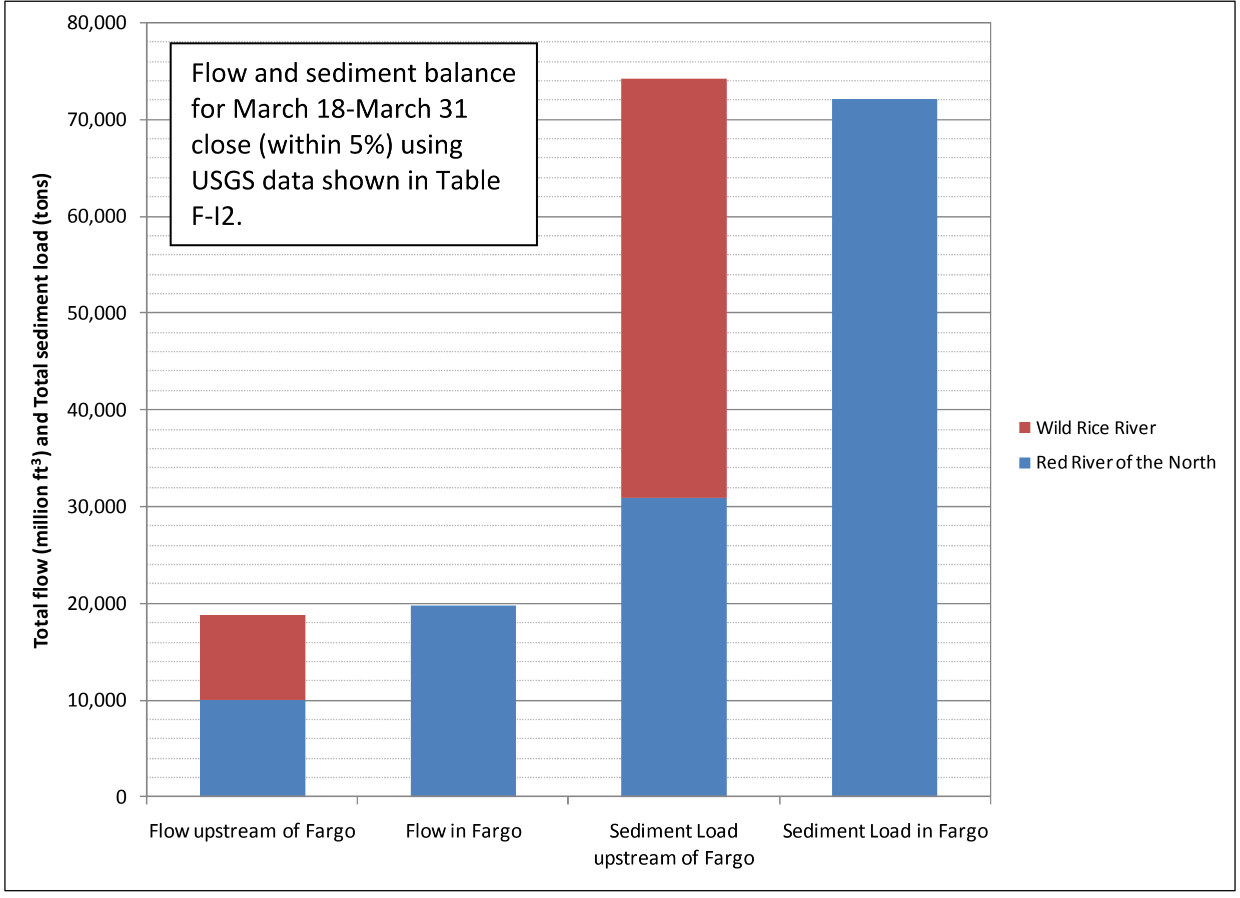
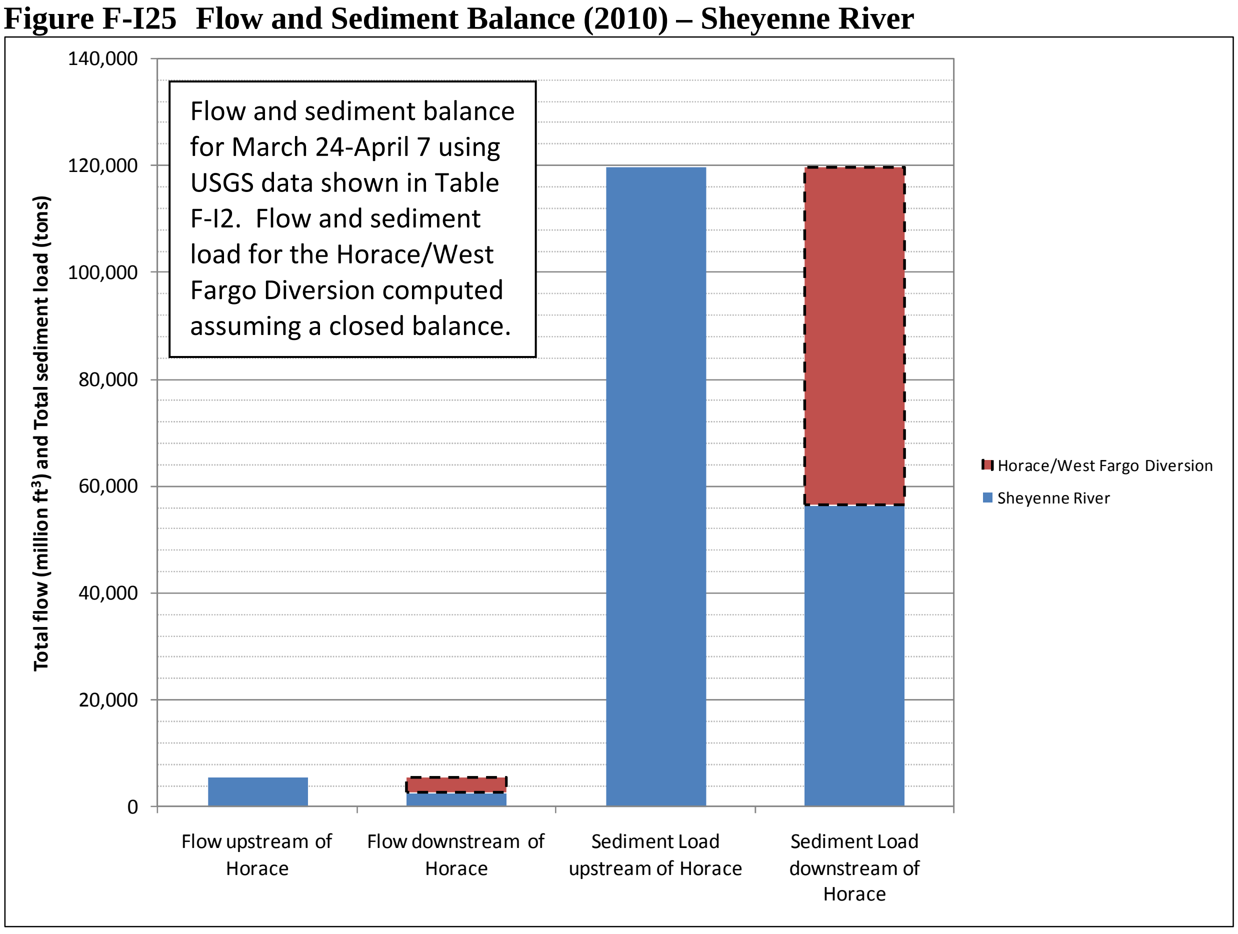


Figure F-I25 Flow and Sediment Balance (2010) – Sheyenne River



RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT J – ICE ASPECTS OF PRELIMINARY DESIGNS

Report for the US Army Corps of Engineers, and the cities of Fargo, ND and Moorhead, MN

By: Barr Engineering Co.

FINAL – Version February 28, 2010

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT J – ICE ASPECTS OF PRELIMINARY DESIGNS

F-J1.0 MEMORANDUM FROM ANDREW TUTHILL, USACE-CRREL, FEBRUARY 16, 2011

MEMO FOR RECORD:
RE: Preliminary assessment of the ice and debris control included in the Phase 3 report. based on details in the Phase 3-Appendix F drawings.
DATE: Feb. 16, 2011.

Review of additional information in Appendix F of the Phase 3 report, support the preliminary findings of the December 31, 2009 "Ice Aspects of Preliminary Designs" **(**Exhibit E). The biggest concern is still that the diversion and crossing structures will retain large ice floes and possibly debris and, under certain conditions, ice and debris will be drawn into the diversion canal. The preventative measures proposed in the 12/31/09 report included retention booms where surface water velocities are expected to be low (≤ 2-1/4 ft/s) and possibly piers in areas where velocities are expected to be higher.

Measures to control drifting ice using booms are similar to those used to control debris, but there are important differences. The main one is that the retained ice (and the ice that may get past the boom into the canal) melts, particularly under conditions of high flow and a rising hydrograph. This is a mitigating factor should the ice control measures not perform as designed. This is not true with debris which may remain in place to cause problems throughout the event. Following the event, the debris becomes an O&M issue as it will need to be removed. It is not certain what the debris load is on the RRN and tributaries in the project area. More needs to be learned about this. DPW's charged with debris removal after flood events are potentially good information sources.

The ongoing ice analysis will help narrow the discharge range for which the ice control measures will be needed. A mitigating factor is that, above some return interval discharge (Tr ≥ 10-20?), very little ice remains in the system and ice control becomes increasingly less important. We are currently trying to determine the dates for each year when ice essentially disappeared. This date, or time window, is estimated from USGS discharge measurement and water temperature data, aerial and satellite imagery and an ice formation and decay model that is being developed. Our aim is to relate discharge and stage to a probability of maximum ice thickness.

For estimating ice-structure interaction, useful new information in Phase 3 Appendix F includes the graphics showing how the structures look under various discharge and diversion scenarios. Also useful were the water velocity distributions from the ADH modeling. From analysis of historic ice breakups completed so far, it appears that much of the RRN ice remains as large floes during the breakup period. This may be less true for the tributary ice. Based on these information sources, ice-structure interactions and possible ice control measures are discussed. It should be noted that this assessment is preliminary and section 2.3 of the ongoing ice study will treat this subject in greater detail.

RRN Diversion Structure

The greatest ice concerns at the RRN diversion structure will probably be for discharges in the Tr = 5-10 year range where about 2/3 of the flow goes through the structure and 1/3 diverts into the canal (Fig. F06). Here large floes may accumulate against the structure and some may be drawn into the canal entrance. Fig. F-G17 shows ADH calculated velocities in the 1-2 ft/s range upstream of the structure at a ‹ 5-year Q, with no flow entering the canal. This could cause a single layer accumulation of floes upstream of the structure and the canal entrance area. As the velocity increases to the Q=20 yr level, provided the ice had not melted, it would be drawn into the canal. Here the ADH predicted velocities are on the order of 2-3 ft/s (Fig. F-G19). At the 20 yr Q, the gates would be open about 5 ft which is not sufficient to draw floes beneath. If the floes were large as field evidence suggests, a floating boom would probably keep most of this ice out of the canal.

WRR Diversion Structure

The WRR structure will probably accumulate a single layer of large floes and not pass much ice at the 5-year discharge. Figure F-G29 shows average water velocities upstream of the structure to be in the 1-2 ft/s range for this condition. At the 20-year flow level, any of the accumulated ice that remains would be drawn over the downstream weir and into the canal as a result of water velocities as high as 4-5 ft/s. If our analysis indicates the likelihood of significant ice remaining in the system for Q › 20 yr flow levels, then the WRR ice will need to be retained upstream of the canal junction. ADH calculated velocities upstream of the structure are on the order of 2-3 ft/s, indicating that a boom might work provided the ice floes were large (Fig. F-G31).

Sheyenne and Maple River Crossing Structures

Under Flow Scenario 2 (Q ~ 10 yr?) (Fig F26), ice floes would accumulate upstream of the aqueduct and ice would at some point be drawn over the grade control structure and spillway into the canal. The same would be true for Flow Scenario 3 (Q~20 yr ?) if ice still remained in the system. Due to the width reduction, it is unlikely that the aqueduct would not convey any ice downstream; the ice would probably jam at the aqueduct entrance which would be undesirable as it would increase upstream stage and the diversion flow into the canal. To prevent ice from jamming in the aqueduct entrance and entering the canal, the ice would need to be retained somewhere upstream of the rock grade control structure. The existing conditions ADH simulations at 2 and 50 year flows show average velocities upstream of the structures in the 2-3 ft/s range (Figs. F-G43, F-G44, F-G63 and F-G64). This indicates that floating booms would likely to retain the bulk of the ice upstream of these crossing structures under the estimated range of hydraulic conditions of the Phase 3 design. The booms would be at their upper limit of their performance range though.

Respectfully Submitted,

Andrew M. Tuthill, P. E.
Research Hydraulic Engineer
U S Army Cold Regions Research and Engineering Laboratory
72 Lyme Rd.
Hanover, NH 03755
603-646-4225 office
603-306-6699 cell

F-J2.0 BARR ENGINEERING JULY 8, 2010, MEMORANDUM FROM DISCUSSION WITH ANDREW TUTHILL, USACE-CRREL

The following information was previously presented as Exhibit E of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

Brian LeMon and Sarah Stratton had a conference call with ice expert Andy Tuthill to discuss general ice control strategies for the FMM diversion project. Notes from that conversation follow:

- Approximate Costs:
  - Floating ice boom (e.g. Lake Erie) – roughly $1 million and under[1]
  - Ice control piers – roughly $1 million and over[2]
  - Allegheny River ice boom - $1000 to $1500/foot (note that this number represents the cost of the boom itself, i.e. does not include anchors). Matt Metzger has a cost estimate for the Allegheny Boom which is roughly $3,000/ft[3].

- We explained that our focus has been to keep ice out of the diversion, and Andy agreed that this should be the approach.

- Andy stated that the most important single variable with ice control is the velocity of the water conveying the ice. Also important are water surface slope, ice thickness and ice strength.
  - These factors determine whether the sheet ice cover will bring into small or large pieces
  - Another variable is the water surface elevation (WSEL) during break-up events and the increase in WSEL from winter base flow levels to break-up levels.
  - General rule of thumb is that the higher the velocity, the more difficult it is to control ice
    - Higher velocities make it harder to retain ice and make the ice more prone to move

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[1] Based on a recent estimate to replace the Oil City, PA ice boom, a ballpark cost for an ice or debris boom consisting of steel pipe pontoons ~2' in diameter is about $2500/ft (including the anchors).

[2] Based on the Cazenovia Creek ice control structure completed in 2005, the cost of a pier-type ice control structure is about $ 10,000/ft of river width. These concrete piers are about 10' high and 5' in diameter with 12' gaps between. A large part of the expense is the foundation and post tensioned anchors to the underlying bedrock. This type of structure also requires considerable bank armoring where flow exits and reenters the main channel. Debris accumulation and bank erosion problems are currently being addressed.

[3] This was the approximate cost of the original boom which was over-designed and overly complex. Modern designs are simpler and more efficient and constructed of off-the shelf components such as steel pipe.

- We asked Andy to provide some general guidelines for how to choose ice control measures and here are his responses:
    - An ice expert needs to study the river and ice floes (historic data, talking to people who know about the river, hydrology, etc.)
    - A time-tested rule of thumb is that ice floes will accumulate edge-to-edge (juxtapose) when the water surface velocity is ≤ 2.25 ft/second (for deeper water):
        - Generally speaking, ice booms will retain ice when velocities are equal to or less than 2.25 ft/sec
        - For velocities above 2.25 ft/sec, piers are an ice retention option – but piers are more complicated (and expensive) and often get plugged with debris and ice. Piers (or timber cribs) have been used successfully for stabilizing sheet ice in relatively quiescent hydraulic settings.

    - The second time-tested criteria for ice retention is a Froude number of 0.08 or less (for shallow water) – a boom will typically work if the Froude number is 0.08 or less.
    - If ice floes are greater than ¼ of the channel width, they are more likely to create "arching" (<2.5 ft/sec) (*Calkins, D.J., and G.D. Ashton (1975) Arching of Fragmented Ice Covers. US Army Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire, Special Report 222).*

- We asked Andy about how the ice booms function when the water surface elevation and width of the water surface vary a lot. Andy mentioned that this is done a lot in flood control reservoirs (although the booms are usually to keep debris away from the outlet), and cited the West Hill Dam as an example – Andy sent photos of that:


9 2:49PM

  - In this situation, the boom is adjustable and the excess length of boom is just strung along the ground.
  - Andy did state that ice booms and trash booms are pretty much the same thing.

- As mentioned previously, piers or timber cribs can be used for stabilizing sheet ice. In this application they are relatively widely spaced (50 ft or greater). This may not apply to the FMM situation where water velocity could be sufficiently high to fragment the sheet ice and push the floes between widely spaced piers or, in the case of narrower spacing, back up ice rubble and flow until sufficient head is developed to push the ice between the piers.
  - When piers are "dolphined" (w/ pier spacing closer to 50' apart), the piers are used the keep big sheets of ice from getting into navigation channels. Andy cited the St. Lawrence River at Lake St. Peter as an example of this.

- Brian asked about the possibility of using heat at some structures to help with ice control.
  - Andy said that heat can be used where a power plant is present – he cited the Dresden Island nuclear power plant on the Kankakee River in IL as an example.

- Brian asked if Andy could share any horror stories and/or tell us about additional things we should be concerned about:
  - Cazennova Creek (written-up in many of the papers and presentations we have regarding ice control)
    - Concrete piers 12 ft apart.
    - Concept is that when ice gets caught up behind piers, the water spreads onto the floodplain then back into the channel.
      - A physical model at CRREL showed relatively uniform lateral outflow and inflow of the water that bypassed the ice jam via the floodplain.
      - The main problem with these piers is that debris quantities were underestimated. Following high flow events "the ice control structure looked like a beaver dam". More flow than anticipated bypassed the structure via floodplain and that flow tended to concentrate at low points rather than distribute uniformly.
      - As a result, the riprap protection along the adjacent bank was severely eroded. Measures are currently being developed to correct the problems and avoid the development of a new channel flanking the structure.
  - Flow beneath an ice jam caries the potential for "under ice hydraulic scour"
    - The scour occurs since the ice jam decreases the flow area increasing the near-bed water velocities. The roughness of the ice

        - cover also increases the turbulent kinetic energy of the flow, increasing the bed shear.

- Recommendations from Andy:
    - Put effort into trying to determine where ice control is actually needed (i.e. don’t over-design for ice control).
        - “Probably don’t get a big ice run on the RRN.”
        - Further study needs to be done, specifically for ice potential at/near the RRN and WRR control structures.
        - Andy also commented that he is not currently comfortable with his lack of knowledge of the ice situation for the FMM project. He mentioned in a follow-up email that he thinks this is an extremely interesting project and he is glad to be involved.
    - With regards to having permanent ice removal (e.g. cranes) on site, Andy said this seems like an “emergency” measure and does not recommend relying heavily on this approach.
        - Too expensive, too much maintenance.
        - Use an emergency response protocol instead.
    - With regards to trying to contain the ice somewhere then remove it with equipment, Andy does not recommend this either – “it’s probably futile to pull ice out of the river during an ice run and it’s never been done.”
        - However, Andy also mentioned that equipment is often used to help move ice through structures such as bridge openings during an ice event.
    - At the Sheyenne and Maple structures, a sheer boom could be used at the entrance to the spillway channel (perhaps even if velocities along the tributary are higher than 2.25 ft/sec since the boom would be parallel to the tributary.
    - Keep under ice hydraulic scour in mind.
        - Ice/sediment/channel bottom interface.
        - We’ll want to control where the ice jams (i.e. we want to make sure that if a jam occurs, it does not occur where scour may impact a structure – in this case, most likely at the RRN and WRR).

F-J3.0 ICE ASPECTS OF PRELIMINARY DESIGNS, RED RIVER OF THE NORTH DIVERSION PROJECT, BY ANDREW TUTHILL, USACE-CRREL

The following information was previously presented as Exhibit E of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness. The tables, figures, and appendices referred to in the text are included within this section F-J3.0.

Ice Aspects of Preliminary Designs
Red River of the North Diversion Project
Fargo ND, Moorhead, MN

to

Barr Engineering
and
St. Paul District USACE

by:

Ice Engineering Group,
Remote Sensing/ GIS Branch
US Army Engineer Research and Development Center
Cold Regions Research and Engineering Laboratory
72 Lyme Rd., Hanover, NH 03755

Dec. 31, 2009

1. Introduction

The all-time peak stage on the Red River of the North at Fargo, ND occurred on March 28, 2009, causing major flooding in the Fargo and Moorhead area. In terms of river discharge, this event had a return interval of about 100 years. The RRN also experienced major flooding in the early spring of 1997 with the greatest damages occurring farther downstream at Grand Forks. Though discharge was the predominant factor, ice and ice jams played a role in both these events. To mitigate future flooding, plans are being developed for diversion channels to bypass a large portion of RRN flood flows around Fargo and Moorhead. The diversion alternatives being evaluated call for a gated structure located on the RRN about 27 river miles (RM) south of Fargo that would divert as much as two-thirds of the 100-year flow into bypass canals around the cities. The diversion flow would re-enter the RRN about 34 RM north of Fargo, about near the near Georgetown, MN. Fig. 1 shows the project area with one of the preliminary design alternatives. The objective of this review is to identify important ice issues associated with the preliminary project plans.

This report describes ice processes on the RRN in the vicinity of Fargo and how ice may impact operation of the proposed project. Much of the preliminary ice analysis is based on observations and data from the 2009 and 1997 floods as well as review of flow and stage records. Preliminary plans of diversion structures by Barr Engineering are also assessed in terms of their expected ice-passage performance.

2. General Ice Processes on the RRN

The RRN is a flat and meandering river with low water velocities during the ice formation period favoring the growth of sheet ice that often exceeds 2 ft in thickness by late winter. Although mainstem flood flow velocities can be as high as 5-6 ft/s, spring breakup on the RRN is typically gradual without dynamic ice runs. While water levels may rise well above the top of bank elevation, the thick sheet ice typically melts in place, often lodged in tight bends or contained within the channel by trees that line the banks. Fig. 2. shows a satellite image of the RRN at the time of the peak stages on March 28, 2009. In the vicinity of the proposed diversion, the flooded width is about 1/3 mile, increasing to over 1 mile farther the north. One can see snake-like remnants of the sheet ice cover indicating the location of the river channel before the flow went out of bank. Through the settled part of Fargo and Moorhead and northward, the floodway narrows to about ¼ mile or less with little ice visible. Downstream of the Sheyenne confluence, the flooded width increases to about ½ mile up to the tight S-bend where the proposed channel would re-enter the RRN. In this bend, one can see the remaining trace of the sheet ice cover.

This interpretation of RRN ice processes based on the 3/28/09 satellite image agrees with general observations by Kate White of CRREL from helicopter flights on April 1 and 3, 2009. Appendix A contains summary of Dr. White's observations.

The daily stage record for the 2009 flood event shows no evidence of significant ice movement on the RRN (Fig. 3.). Daily discharge is somewhat more peaked than the stage hydrographs, but still not characteristic of a breakup involving ice jamming or ice releases (Fig. 4). It may be that the extremely high discharges in 2009 overwhelmed the ice effects. In the more moderate breakup of April 2, 2005 where stage at Fargo reached 19.2 ft, the stage hydrograph saw rapid changes as great as 2 ft attributed to ice jams and releases. From the 2009 discharge data, the river became ice free at Hickson on 3/24 and at Halstad on 3/25.

Winter period flood problems on the RNN appear to be increasing with time. In 2009, in addition to Fargo and Moorhead, ice-related flooding occurred at Oslo, Grand Forks, Drayton and Pembina, though the potential impacts at Grand Forks were greatly reduced by flood control works built since 1997. In 1997, flooding was widespread along the RRN at locations such as Wahpeton, Tyler, Breckenridge, Hickson, Fargo, Drayton, Grand Forks and Emerson. At Fargo, the 1997 flood crested on April 10 at 37.6 ft, only 3.4 ft lower than the 2009 peak. The CRREL Ice Jam Database (IJDB) reports less severe ice jam flooding along the RRN in 1989, 1969, 1961, 1960, 1959, 1957, 1955, 1954, 1952, 1948, 1946, and 1945, though many of these are ice-affected gage reports with no mention of damages. One IJDB report of interest is a "large ice obstruction" at Hickson, ND on April 9, 1997 which is about 2 RM upstream of the proposed diversion shown in Fig.1. The timing of the peaks is March 26, 28 and 30 for Hickson, Fargo and Halstad respectively. Hickson is about 14 air miles upstream of Fargo while Halstad is 35 air miles downstream. The Wild Rice River which enters the RRN about 7 air miles above Fargo and the Sheyenne and Buffalo River which enter the RRN 11 and 14 miles downstream respectively have a large influence on the stage and discharge data shown in Figs. 3 and 4.

If the ice period on the RRN is defined as December 1 – April 15, then 61 of 109 peak stages at Fargo occurred with some ice present on the river. Fig. 5 plots open-water and ice season peak discharges at Fargo with time, showing increasing trendlines, particularly for the winter-season events. Fig. 6 shows a discharge frequency curve with the data sorted into ice-season and open water season. Of the 21 events above the 5-year discharge, two-thirds occurred during the ice season which emphasizes the importance of winter hydrology and ice processes in the project design.

3. Ice Issues for the Proposed RRN Diversion Structure

A gated structure is proposed to divert flow from the RRN into bypass canal(s) around Fargo and Moorhead. This structure would have three openings, 40-ft-wide × 44.1-ft-high Fig. 7. Underflow radial gates would close off all but the lowest 4.1 ft. of each opening. Table 1, based on data from Barr Engineering for one of the diversion alternatives being considered, lists flood discharges and the flow split between the upstream diversion(s) and the RRN Diversion Structure. For the 5-year flow and lower, the gates would fully open and all flow would pass the through the structure. For the 10 to 100-year discharge range, all but about 10,000 cfs would be diverted around Fargo and Moorhead. For the 200 and 500 return interval flows, about two-thirds of the total river flow would be diverted into the bypass canal(s).

Table 1. Diversion Structure Data and Calculations

Event (Years)	Q tot (cfs)	Q thru (cfs)	Qdiv (cfs)	Upstream Depth (ft)	WL Change Thru Gate (ft)	Gate Opening (ft)	Total Opening (ft)	Average Velocity at Gates (ft/s)	Approach Flow Area (ft^2)	Average Approach Velocity (ft/s)	Opening/ u/s Depth
2	3500	3500	0	30.2	0.7	40.0	44.1	1.0	5,294	0.66	1
5	9600	9600	0	39.8	1.1	40.0	44.1	2.0	7,550	1.27	1
10	14500	10274	4226	43.5	3.9	2.6	6.7	12.8	8,493	1.21	0.15
20	19000	9917	9083	45.7	6.5	0.9	5	16.5	9,074	1.09	0.11
50	25500	9528	15972	48.1	9.1	0.0	4.1	19.4	9,723	0.98	0.09
100	30000	9792	20208	49	9.6	0.0	4.1	19.9	9,972	0.98	0.08
200	39000	12806	26194	49.9	8.1	1.7	5.8	18.4	10,222	1.25	0.12
500	53000	17940	35060	51.5	6.3	5.2	9.3	16.1	10,673	1.68	0.18

In terms of ice, it is good that the gates start in the open position and be progressively closed as flow increases up to the 100-year level. This mode of operation would avoid gate freezing problems since, by the time the 10-year flow occurred, any ice on the gate abutments would probably have melted. In the opposite scenario of opening the gates as the flow increases, freezing of gates and gate seals would likely be a problem.

Water velocity provides a good indicator of how the ice floes will accumulate upstream of the structure. The average approach velocities in Table 1 were calculated from the 1-D continuity. The upstream flow areas were calculated based on the listed depths from Barr Engineering and an assumed a trapezoidal channel cross section with a 120-ft base width and 1V:2H side slopes. A long-held rule of thumb predicts that ice floes will accumulated edge-to edge (juxtapose) and not under-turn and submerge where water velocity is less than about 2-1/4 ft/s. In all the above cases, the juxtaposition criterion is easily met, so, if the upstream ice cover were fractured and free from the banks, the floes would accumulate edge to edge upstream of the gate rather than form a multi-layer (shoved) ice accumulation.

In the case of the 5 year and lower discharges, these floes would either pass the gate or the ice fragments would arch and stop upstream. Calkins and Ashton 1975 predict that concentrated ice floes will arch across an opening where the average floe diameter is less than about one quarter of the opening width, so if the floe were smaller than about 10 ft, the ice pieces would probably go through.

In terms of passing ice or debris, a larger opening of the central gate is preferable to lesser but equal openings of all three gates. A central gate opening is also better than having the middle gate closed and the side ones open. This may not be an important factor since ice will likely only pass the structure for flows ≤ the 5-year discharge (explained below). A minimum width of 40 ft for the central gate would ensure optimal ice passage. Assuming that ice floes will be passed through the central gate, decreasing the width of the side gates would not have a negative effect on ice passage.

For these lower flows all three gates will be fully open. Ice and debris booms would be beneficial in terms of channeling the ice and debris through the central gate, as suggested by Barrr in Fig. 8. Many off-the-shelf debris boom designs cannot withstand serious ice action. Recent steel boom designs used on Canadian Rivers do however, and this technology might be useful in the current project (Appendix B).

For all the other discharges and gate opening conditions listed, the ice will most likely not pass the structure. This is because the gate opening heights are relatively small fraction the upstream depth (Table 1). This prediction of non-passage of ice is based on experience with ice passage on large river dams were the gate opening needs to be from one-half to one-third the upstream depth in order to pass ice. An example is the Mel Price Dam on the Mississippi River where the normal pool depth is about 40 ft and an under-flow tainter gate opening needs to be at least 15 ft to draw broken ice pieces beneath.

The design intent is to preserve existing ice passage conditions on the RRN main channel while preventing ice from entering and jamming in the diversion canals. For this reason, the sheet ice cover on the RRN may need to be stabilized and retained at the canal entrances. This could be accomplished with rows of piers across the canal entrances but detailed analyses of expected ice type, floe diameters and approach velocities, etc. would be needed for the designs under consideration.

4. Ice Issues at Diversion Canal-Tributary Crossings

The diversion canal will cross three tributaries, the Wild Rice (WR), the Sheyenne and the Maple. In the case of the Wild Rice, a diversion structure similar to the RRN gated one is proposed. This would have two-30-ft-wide by 22-ft-high gates that would divert most of the WR flood flow over a weir into the canal. In the proposed design, the Sheyenne and Maple Rivers would be conveyed over the diversion canal by concrete aqueducts. Diagonal canals would divert a portion of the flood flows from these two tributaries over weirs into the canal. Rock arch drop structures would be built across the tributaries upstream of the diversions. The Maple river aqueduct would have gates to regulate diversion flow while the Sheyenne outflow would be governed by water levels at the weir. In a non-flood situation, the diversion canal will be dry with the exception of a low flow channel to convey local runoff.

The Wild Rice, Sheyenne and Maple Rivers, like the RRN are low gradient tightly meandering rivers, not known for dynamic breakup ice runs. Still some breakup ice jamming occurs, and, combined with elevated water levels due to high discharge, small jams can exacerbate the flooding. The CRREL Ice Jam Database reports a number of ice jams at locations on the Wild Rice River such as Mantador, Great Bend, Rutland and Cayuga, all too far upstream to influence ice processes in the vicinity of the diversion canal crossing. On the Sheyenne River, the IJDB reported major ice jam flooding at Harwood in 2009 and a jam in West Fargo. Backwater due to ice was reported at Kindred in 1969 and West Fargo and Kindred in 1961. Kindred is about 5 RM upstream of the proposed canal crossing structure.

At the Wild Rice crossing, one would expect ice issues similar to those at the RRN diversion structure. For all but the 2- and 5-year discharges, the gates will be closed, leaving only the 2-ft bedload opening. Assuming a trapezoidal approach channel with 2H:1 V side slopes, Calculated approach velocities at the depths provided by Barr Engineering would be at or below about 1.5 ft /s, favoring accumulation of ice floes by juxtaposition and no underturning. Depending on surface concentration and floe diameter, the ice pieces would either pass the 30-ft-wide gate opening or arch and stop upstream, as discussed in the previous section. For the higher flood flows with upstream depths in the 17-25 ft range, the 2-ft opening would be too small draw ice floes downs and through the gates. Similar to the RRN diversion structure, provisions would be needed to prevent ice floes on the Wild Rice River from entering the diversion canal. This could likely be accomplished by row of piers upstream of the canal entrance weir.

At the Sheyenne and Maple crossings, assuming the ice cover fractures, the ice pieces may accumulate at the constriction at the aqueduct entrances. The flow diverted from the river upstream of the aqueduct will also decrease the water velocity and ice conveyance capacity downstream favoring jamming. Provided the water velocity in this area does not exceed about 2-1/4 ft/s, the ice floes will accumulate edge to edge (juxtapose) and not shove-thicken[1] to block off much flow area. Again, some type of ice retention piers will likely be needed to keep ice pieces from passing into the diversion canal upstream.

At the Maple crossing, gates and operator deck abutments will tend to retain large ice floes in addition to the ice retaining effects of the channel constriction and the discharge reduction.

Both crossing structures are supported by concrete walls spaced about 20-ft apart. Diversion channel flow passes beneath the tributary through these conduits, in either an open channel flow or pressurized flow condition. It will be critical that these conduits do not become ice or debris blocked. That the canal is dry (except for low flow drainage) in

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[1] Shove-thickening is a process of ice jam formation where floes under-turn to form a multi-layer ice accumulation.

non-flood conditions is fortunate in that it will be ice free at the start of the breakup period. One concern is that the spring thaw might occur in several phases. Under this scenario, the drainage canal might fill. A return to colder air temperatures might cause an ice cover to form on the canal as discharge and water levels subside. A second thaw and rise in canal discharge could then break up the ice cover on the canal and jam the ice in the conduit openings.

5. Additional Ice Concerns Related to the RRN Diversion Project

One concern is that, for large flow events, the bypassed flow may move the hydrograph downstream more quickly and change how the ice cover releases where it re-enters the RRN. Much of the water volume that now fills the overbank areas in a large flood event would be transposed downstream under post-project conditions. The canal outflow would amount to the addition of a large tributary. This large discharge influx might break up the local ice cover and cause ice jams where none occurred before. For example, the satellite photo in Fig. 2 shows an S-shaped section of sheet ice at the proposed re-entry point that might break up and re-jam downstream.

6. Preliminary Conclusions

1. Although peak stages on the RRN are often ice-affected, the ice breakup on the RRN is typically gradual with little dynamic movement of ice floes. Ice jams do occur however, and when the river is already at or above flood stage, these jams can greatly exacerbate the situation. During the peak floods 1997 and 2009, much flow was out of bank with short sections of sheet ice remaining in the channel bends.

2. The long-term RRN gage data suggest an increasing trend in annual peak discharge magnitude and variability, with the bulk of the peak flow events occurring during the ice season. If it is decided that these trends are real, some conservatism may need to be added to design to accommodate increased future peak discharges.

3. Under flood conditions, in the area of the proposed diversion structure, the 40-50 ft depths and low approach velocities will cause sheet ice and large floes to accumulate upstream of the structure without passing the gates. Provided the floes accumulate edge-to-edge and do not thicken into a multi-layer ice accumulation, this does not pose a problem in terms of ice passing into the diversion channel, provided the ice can be retained in the RRN (see Item 4 below).

Ice may pass the gates at the 2 and 5 year discharges provided the ice floes do not arch and stop upstream of the gate opening(s).

4. The project design must include provisions to prevent RRN ice floes from entering the diversion canal(s). This might consist of rows of piers or piles to retain the floes in the main channel of the RRN. Depending on hydraulic conditions it may be possible to retain the ice floes in the main river channel using the less expensive alternative of ice

booms. The necessity and design of ice retention schemes will depend on further analysis of expected ice conditions and ice processes in the vicinity of the canal entrances need to be examined in detail.

5. The crossing structure at the Wild Rice River will have similar ice issues as the RRN diversion structure. It is predicted that the gated structure will retain Wild Rice river ice under all but the lowest flood discharges. At the Sheyenne and Maple crossing structures, ice accumulations are possible at the transitions from the natural channels to the aqueducts. This will be acceptable as long as the accumulated ice does not shove-thicken into a multi-layer ice jam. As with the RRN diversion structure, provisions will be needed retain ice in tributary channels and prevent it from passing into the diversion canal.

6. Changes in the breakup period hydrograph as a result of the project, and how the bypassed flow re-enters the RRN need to be examined, particularly in terms of causing ice jams where none occurred before.

Respectfully Submitted,

Andrew M. Tuthill, P. E.
Ice Engineering Group
U S Army Cold Regions Research and Engineering Laboratory
72 Lyme Rd.
Hanover, NH 03755
603-646-4225

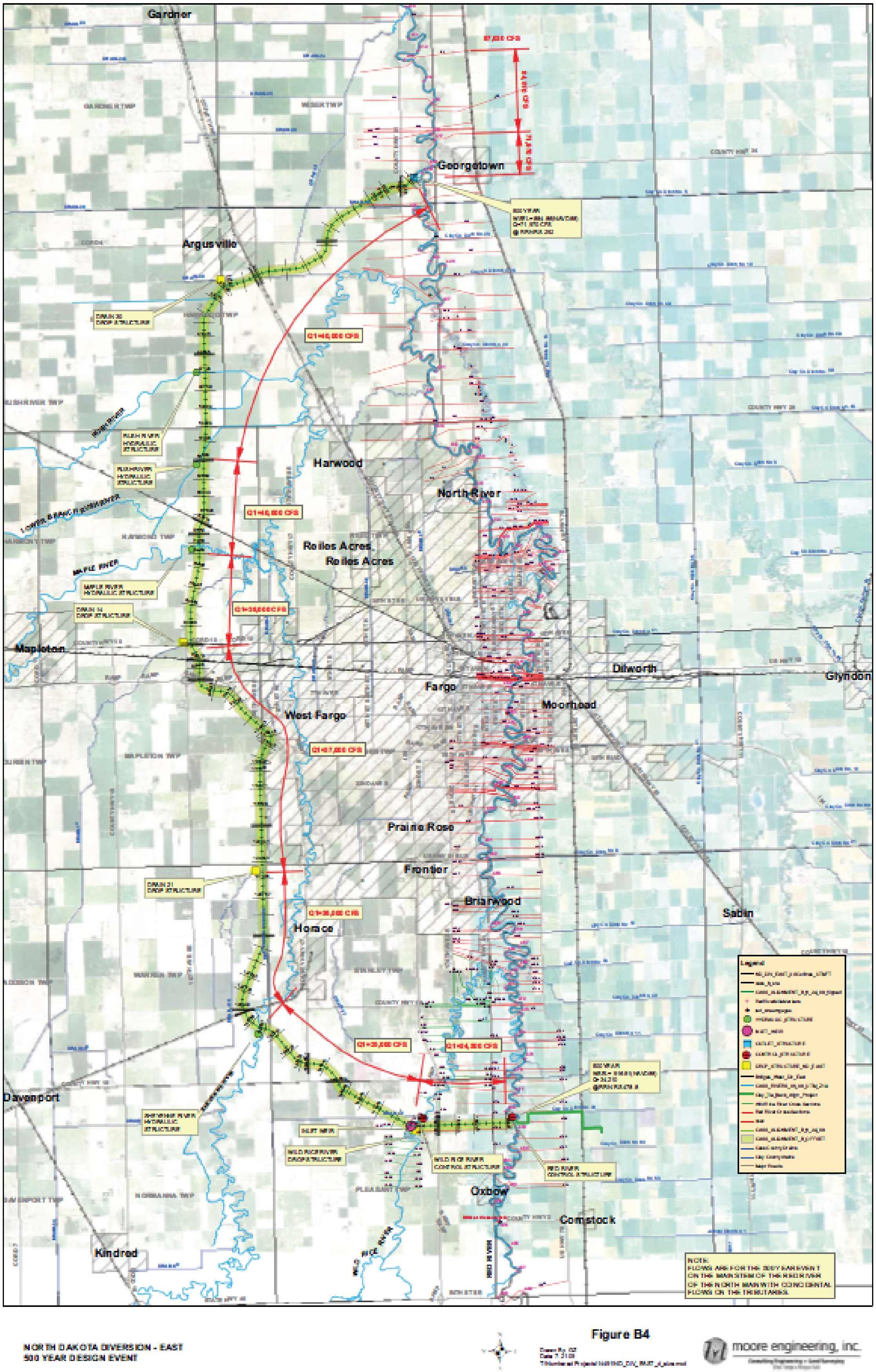


Fig. 1. Map of project area showing one of the diversion alternatives.

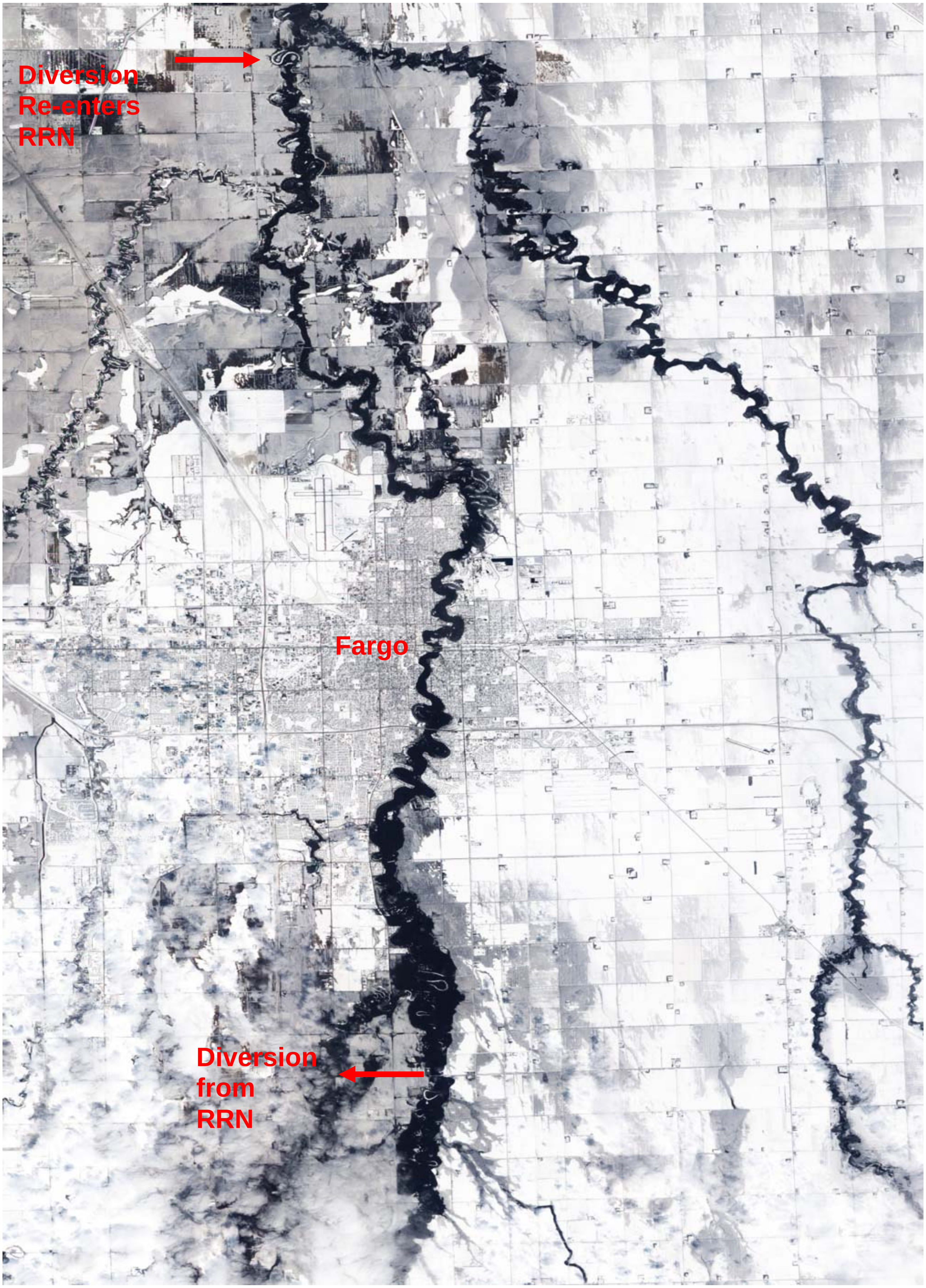


Fig. 2. Satellite image showing e96xtent of flooding on March 28, 2009

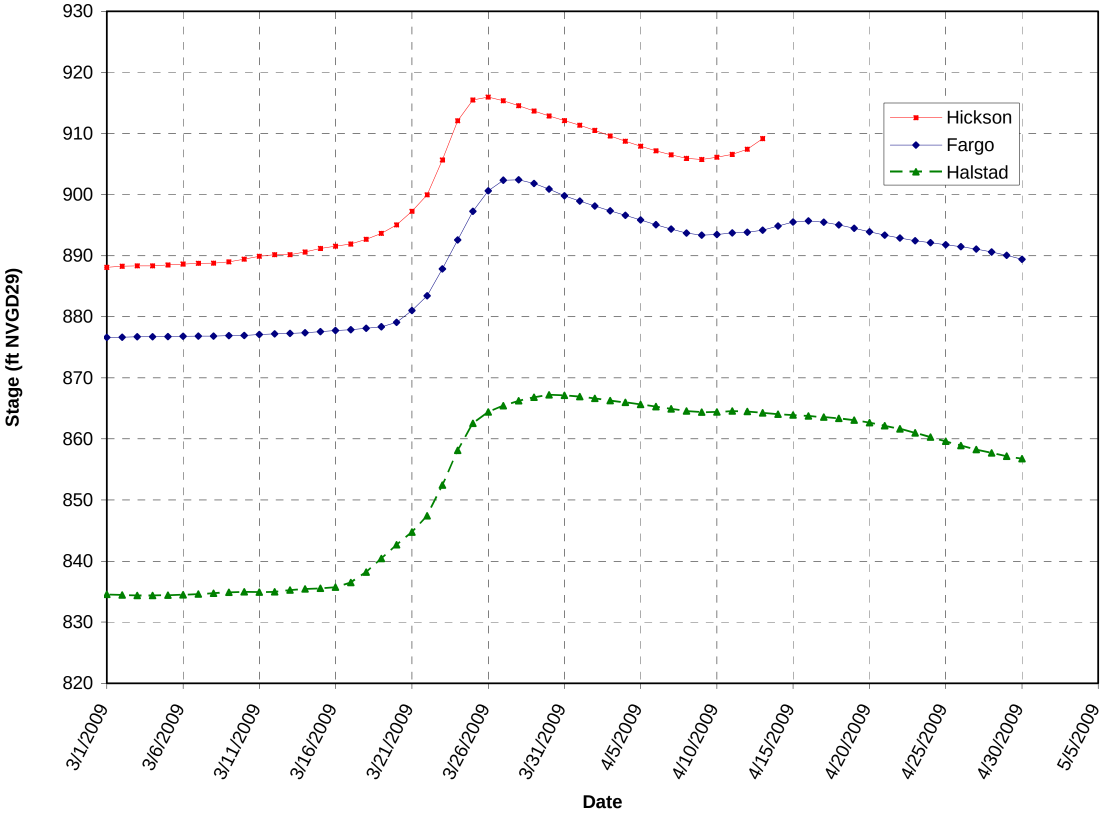


Fig. 3. RRN water surface elevation at Hickson, Fargo and Halstad during the 2009 flood.

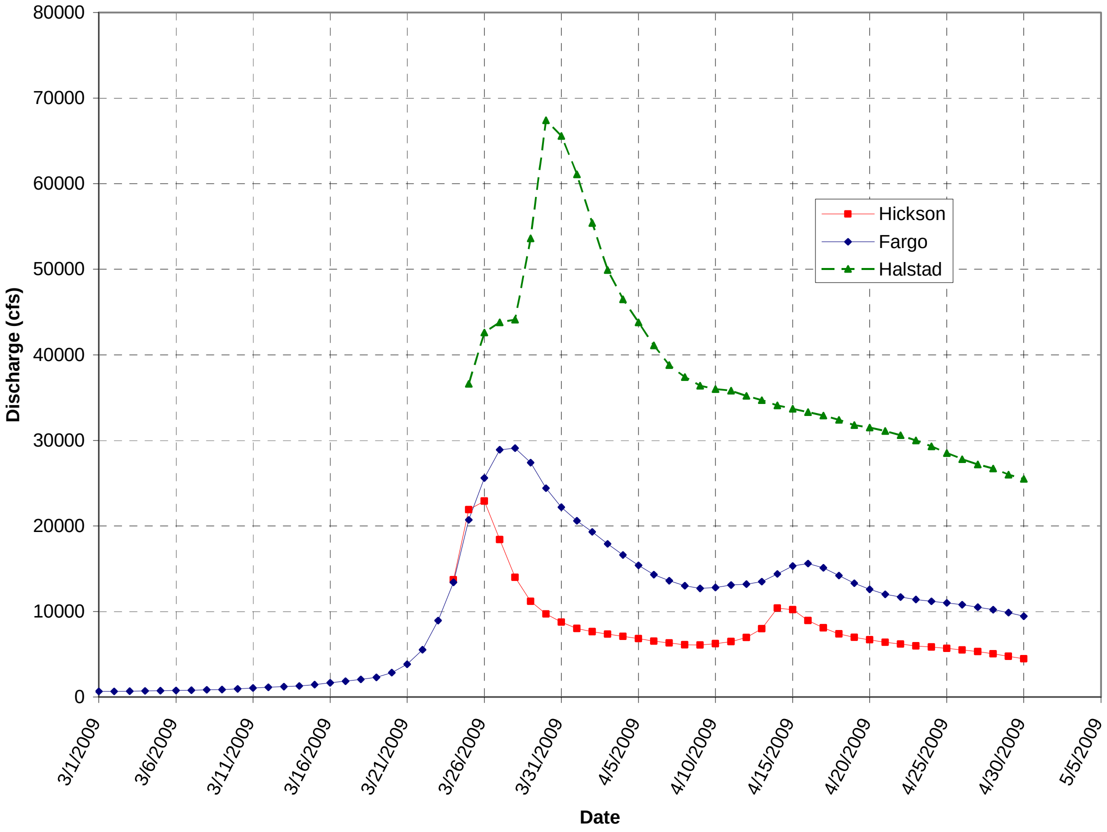


Fig. 4. RRN discharge at Hickson, Fargo and Halstad during the 2009 flood.

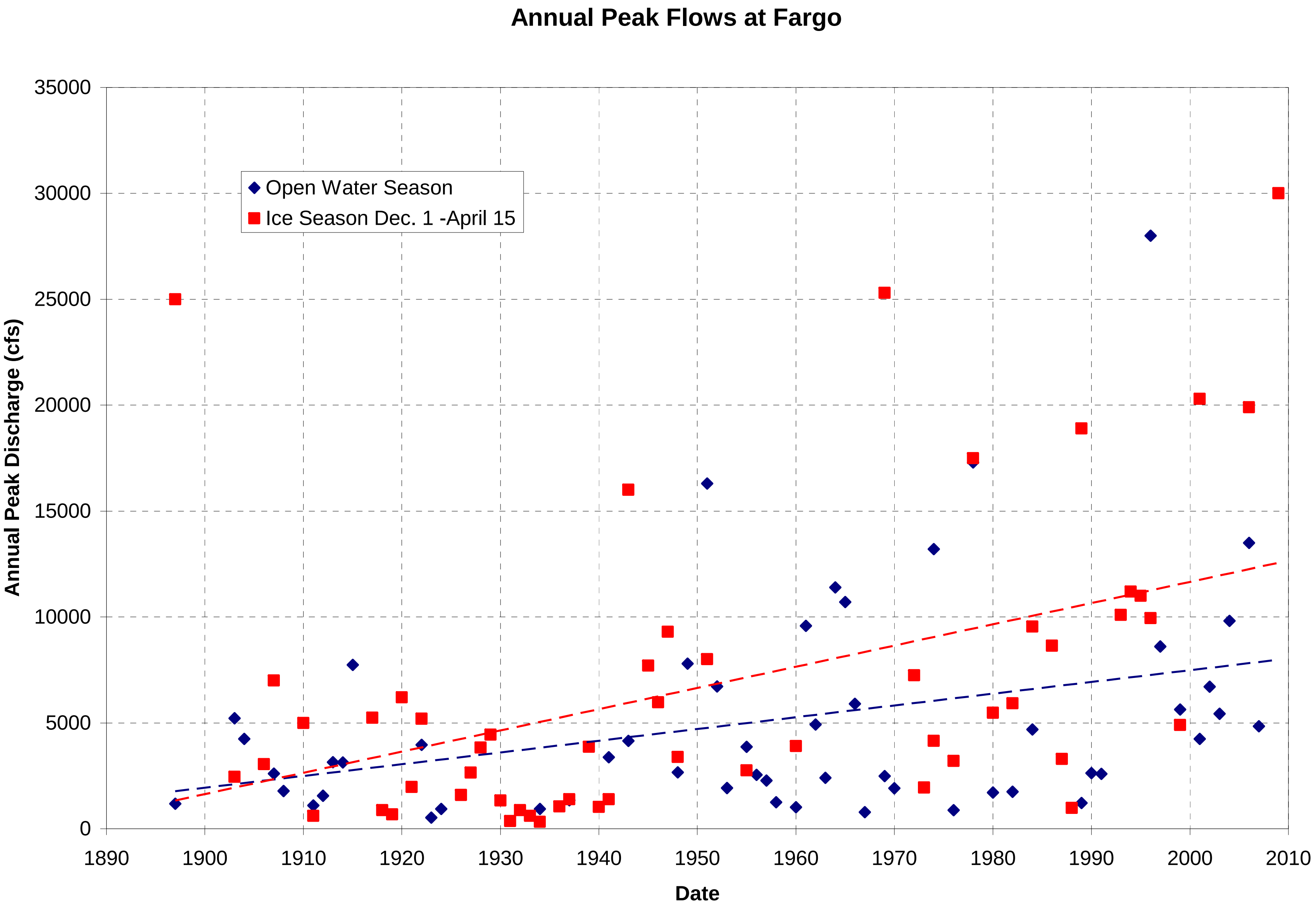


Fig. 5. Annual peak discharges at Fargo.

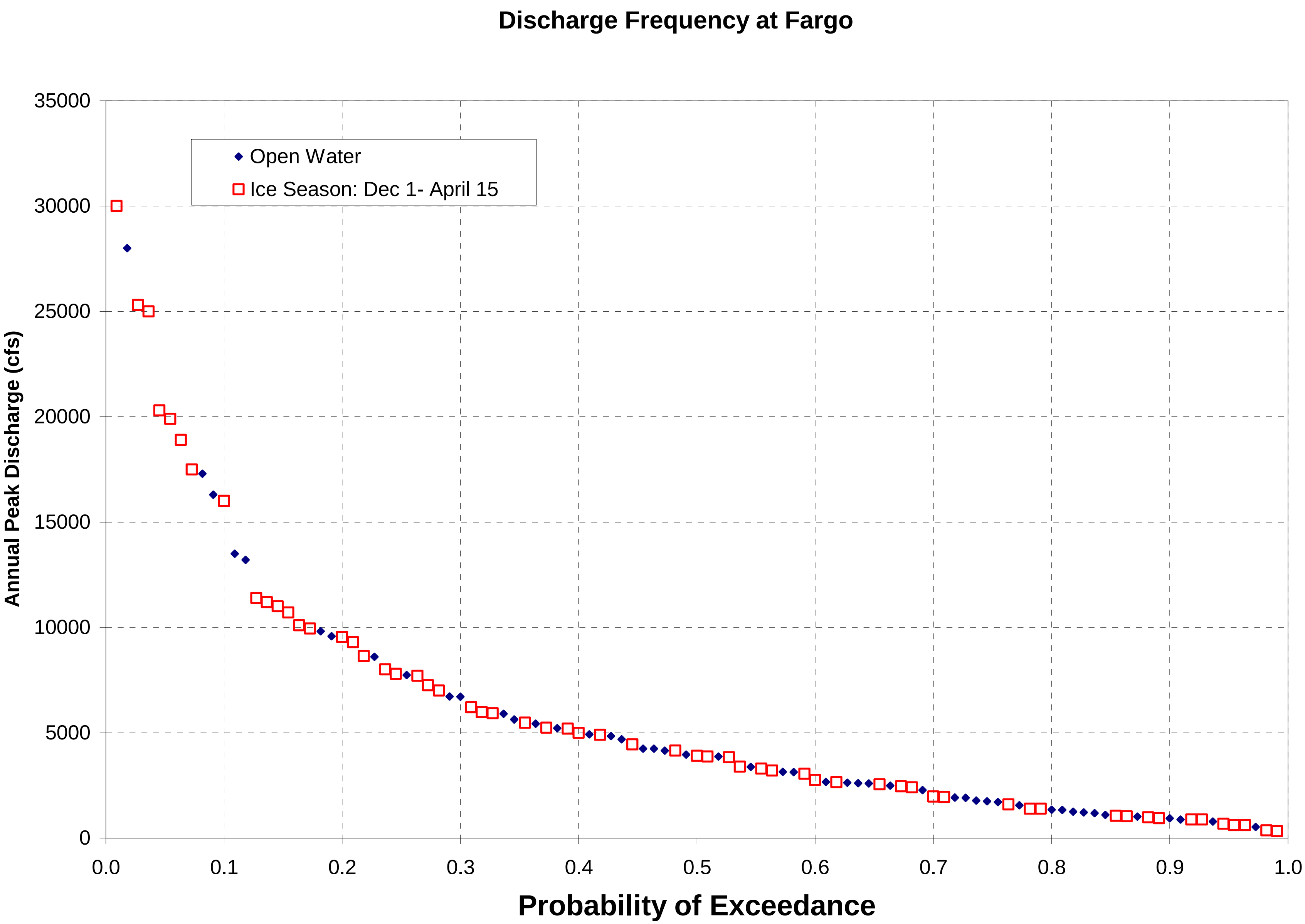
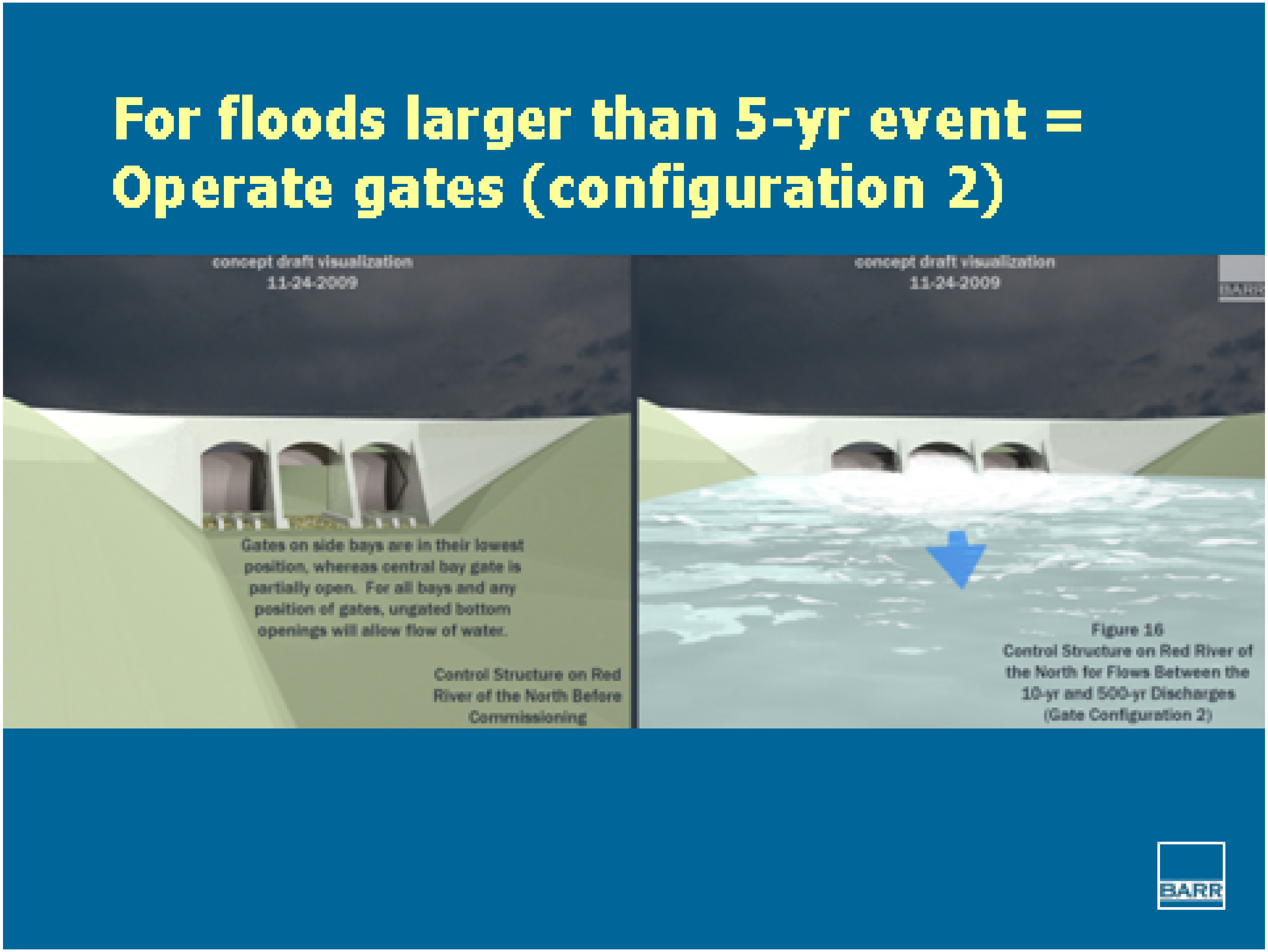


Fig. 6. Probability distribution of peak discharges at Fargo.



For floods larger than 5-yr event = Operate gates (configuration 2)

concept draft visualization
11-24-2009

Gates on side bays are in their lowest position, whereas central bay gate is partially open. For all bays and any position of gates, ungated bottom openings will allow flow of water.

Control Structure on Red River of the North Before Commissioning

concept draft visualization
11-24-2009

Figure 16
Control Structure on Red River of the North for Flows Between the 10-yr and 500-yr Discharges
(Gate Configuration 2)

Fig. 7. Proposed RRN Diversion Structure.

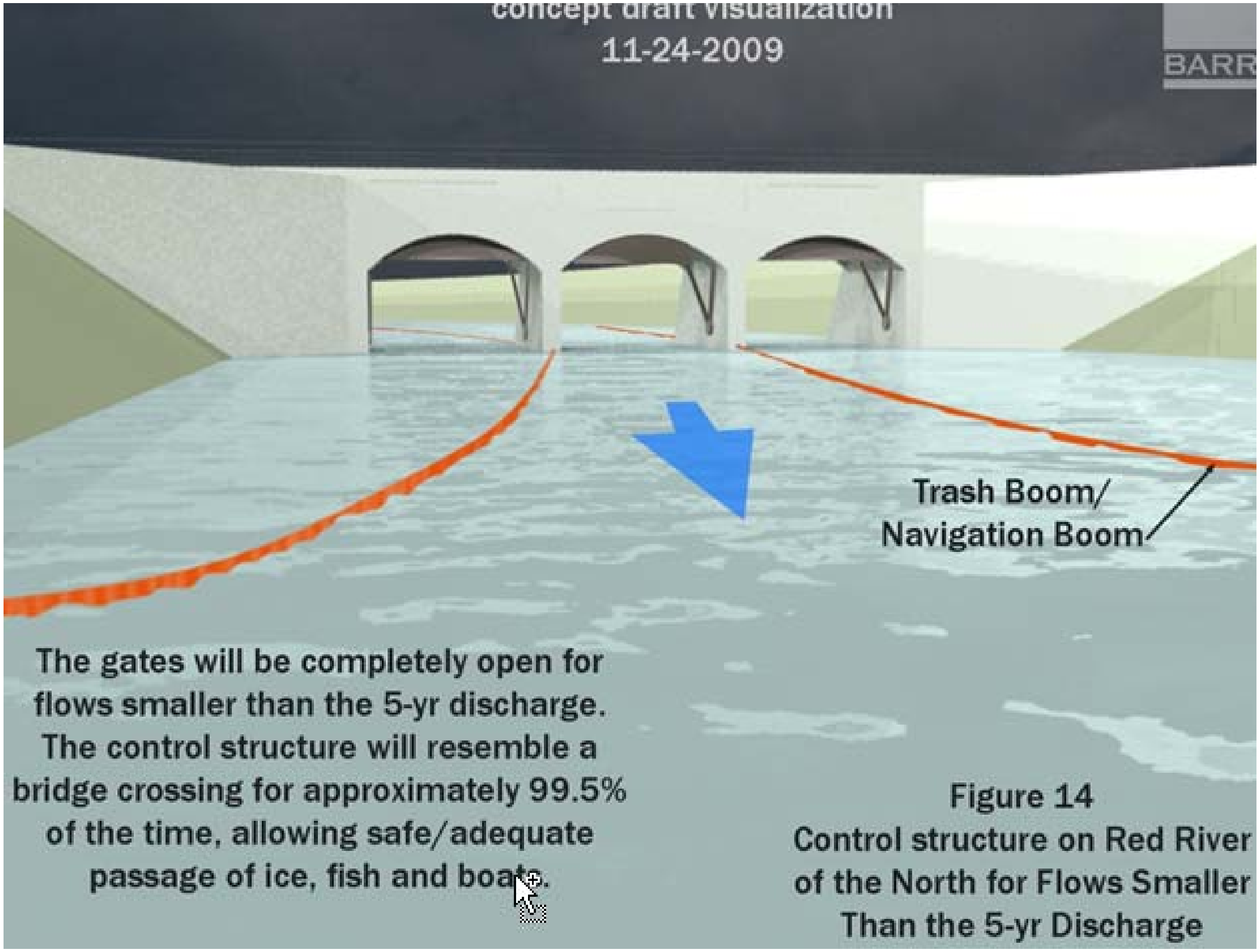
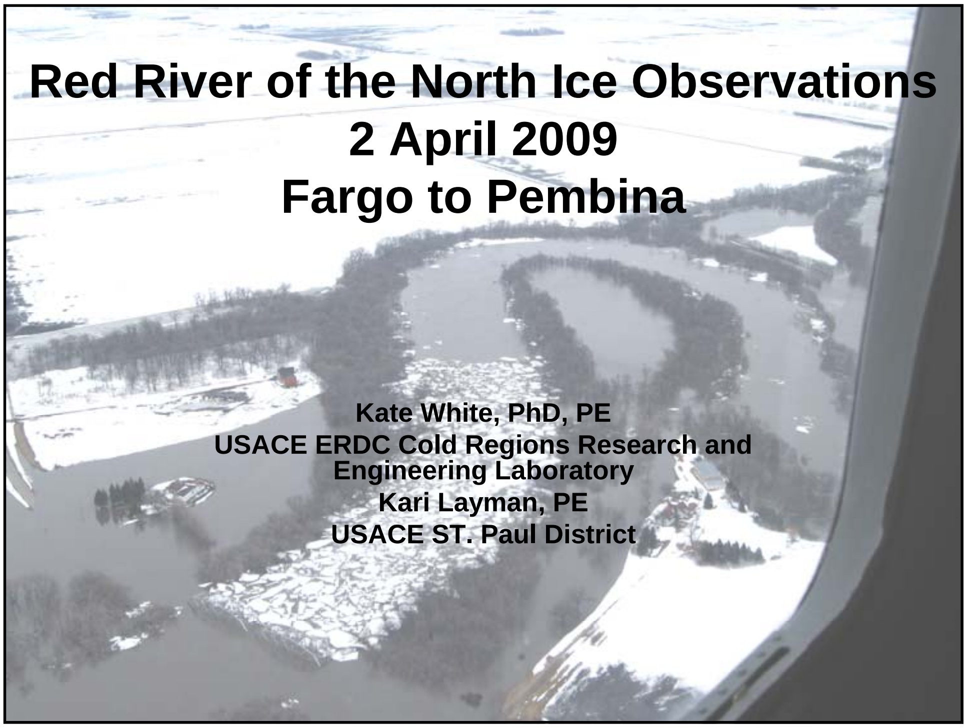


Fig. 8. Possible ice and debris boom configuration.

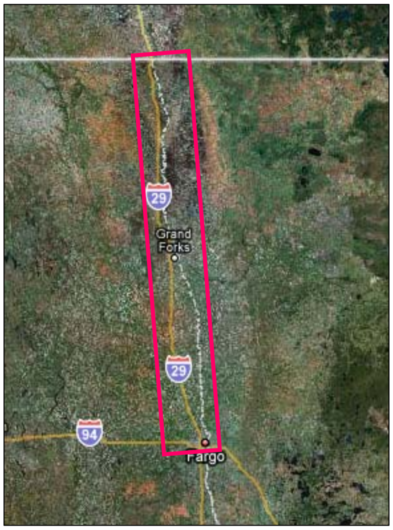


Red River of the North Ice Observations
2 April 2009
Fargo to Pembina

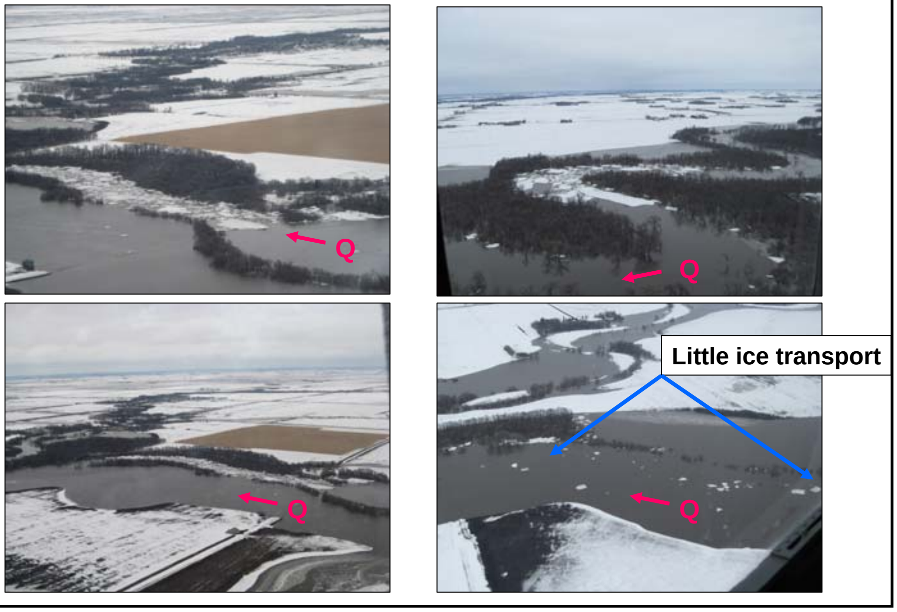
Kate White, PhD, PE
USACE ERDC Cold Regions Research and Engineering Laboratory
Kari Layman, PE
USACE ST. Paul District

Ice Observations 2 April 2009
Fargo to Pembina

- **BLUF: No Ice Jams**
  - **Fargo to Grand Forks:**
    - **little ice, some big sheets but mostly juxtaposed, single-layer ice pieces, decay evident, some ice transport**
  - **Grand Forks to Oslo:**
    - **more large ice sheets but open water as well, ice decay evident**
  - **Oslo to Drayton:**
    - **mostly large ice sheets, less decay**
  - **Drayton to Pembina:**
    - **large ice sheets interspersed with open water, little decay**



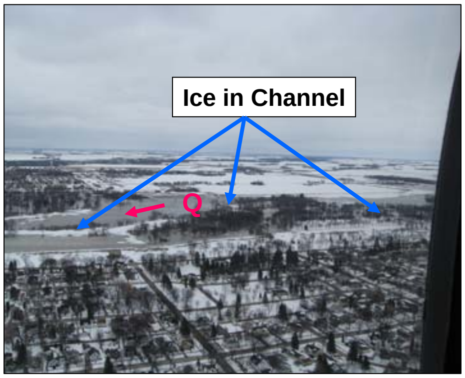
Typical ice accumulations Fargo to Grand Forks



Little ice transport

Grand Forks

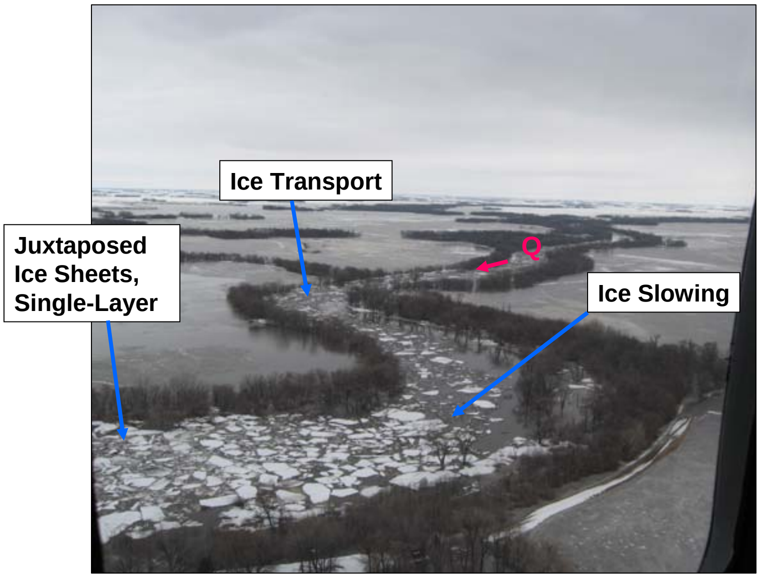
Grand Forks/East Grand Forks



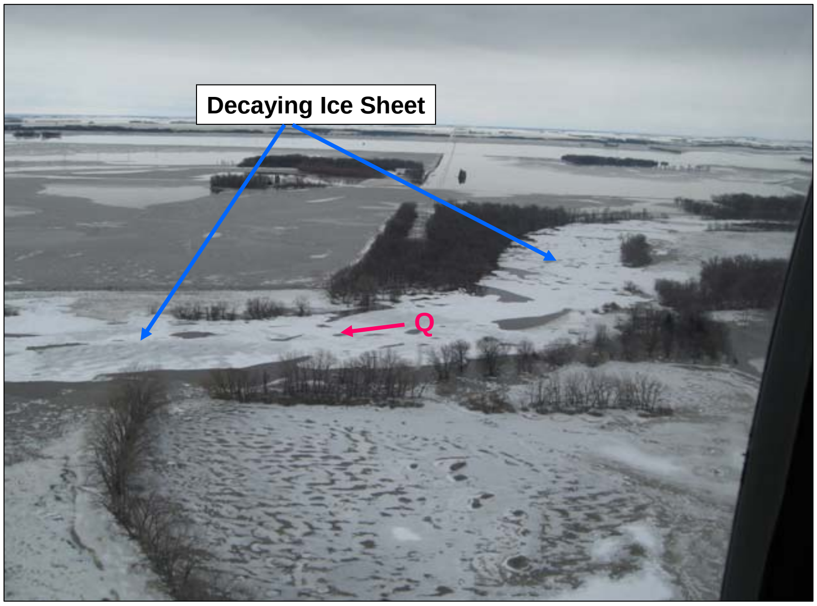
Ice in Channel

Upstream portion of Grand Forks

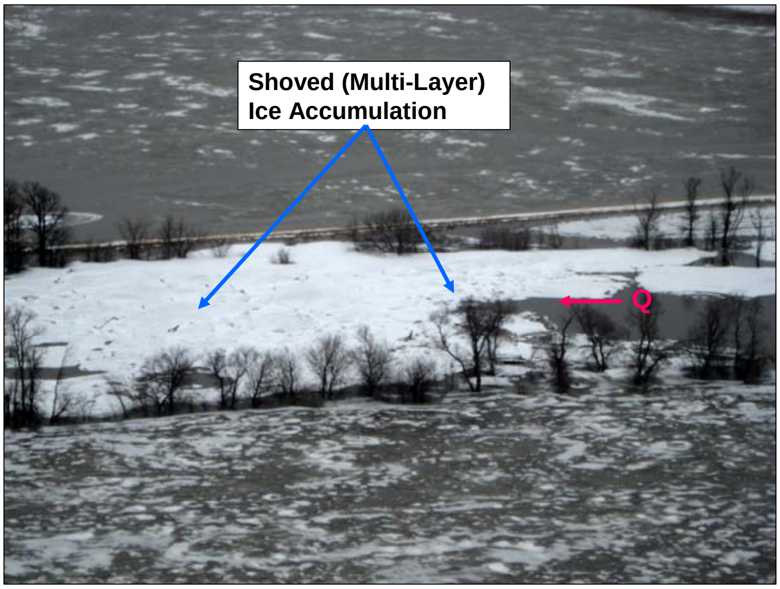
Ice transport between Grand Forks and Oslo



Typical Ice Conditions Oslo to Drayton



Typical Ice Conditions Oslo to Drayton



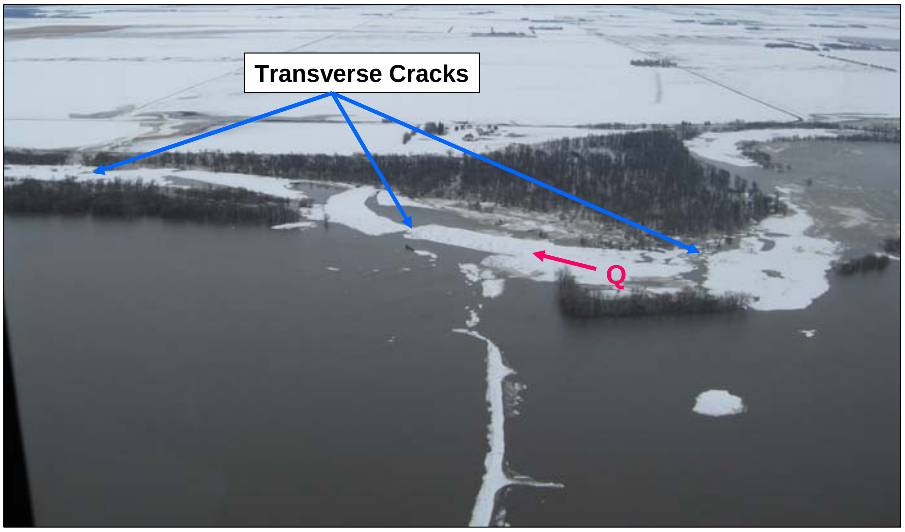
Shoved (Multi-Layer)
Ice Accumulation

Drayton

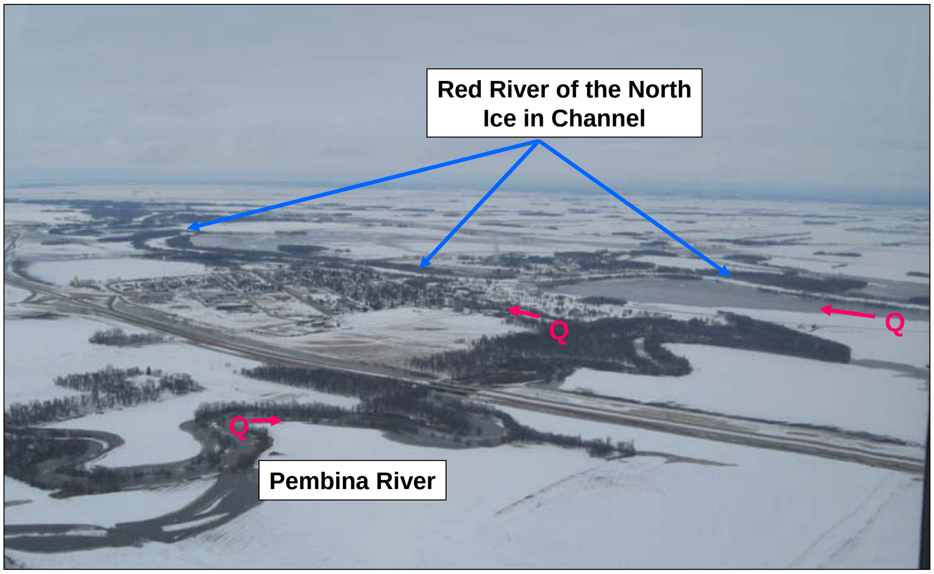
Bridge North of Drayton

Second Bridge North of Drayton

Typical Ice Conditions Drayton to Pembina



Pembina



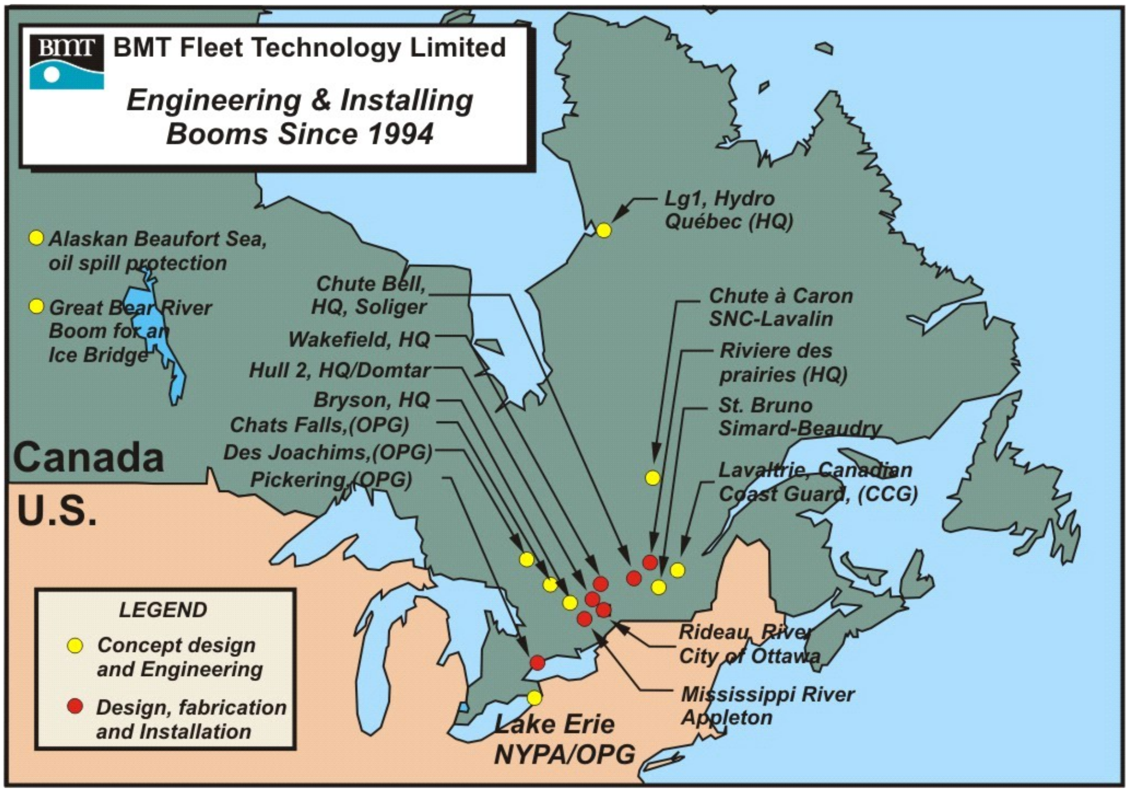
Boom Projects

BMT Fleet Technology Limited

Over the past ten years, BMT Fleet Technology Limited has gained significant experience in the re-engineering of some of the longest ice control booms in North America and in the construction of several small ones.

Without exception, steel booms, using cylindrical pipes with capped ends, replaced the timber pontoons for several reasons:

- Lower Capital Costs
- Lower Maintenance Costs
- Constant density/buoyancy, for optimization to achieve the best results for each particular site
- Better visibility for improved safety of boaters
- Environmentally friendly



Boom Projects

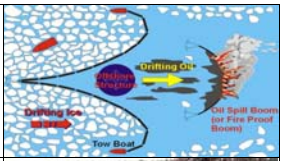
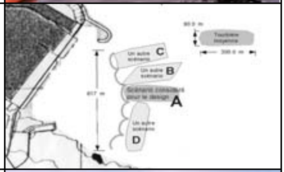
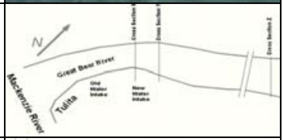
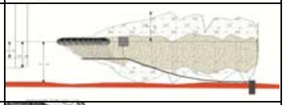
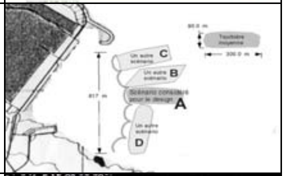
BMT Fleet Technology Limited

Client	Contact	Description	Location *	Duration	A view
New York Power Authority, Niagara Power Project	Mr. Randy Crissman	Engineering services for the Lake Erie – Upper Niagara River Ice Boom. Assessment, monitoring, concept design.	North-east end of Lake Erie *	5 years, 1993 to 1997.	
Canadian Coast Guard, Laurentian Region	Mr. Stéphane Dumont	Engineering services for the development of concept design for the Lavaltrie Ice Boom.	St. Lawrence River at Lavaltrie *	12 weeks, from Aug 1993 to Oct 1993.	
SNC-LAVALIN	Mr. Bertrand Massé	Design of a replacement ice boom upstream of Chute à Caron hydroelectric dam.	Chicoutimi, Quebec *	4 weeks, 1997	
Merol Power	Mr. Ed Olshesky	Ice observations and design of ice booms for the protection of Appleton Power Plant.	Appleton, Ontario	Jan to April 1998	
Hydro Quebec, Maisonneuve Region	Mr. Alain Cyr	Design, fabrication and deployment of an ice boom to protect the Hull 2 and E.B. Eddy hydroelectric power plants' intakes from ice.	Ottawa River near Hull, Quebec	10 weeks, Sep 1997 to Nov 1997.	
City of Ottawa	Mr. Tony Garnett	Design, fabrication and deployment of an ice boom to reduce the amount of ice generated in the Rideau River.	Rideau River in Ottawa	8 weeks, Sep to Oct 1998.	
IRAP	Dr. M. Bishop	A study to evaluate a system to sink and re-float ice boom pontoons.	The Canadian Coast Guard Base in Prescott	January to July 1999.	
Soliger, Hydro Québec, Région Maisonneuve	Mr. Marc Richard	Design, fabrication and delivery of an ice boom to protect the Bell Falls hydroelectric power plant intake from ice.	Rivière Rouge at Bell Falls. To be installed in June 1999 *	8 weeks, Sep 1998 to Oct 1998.	
Ontario Hydro, Pickering Nuclear GS	Mr. Mark Arnone	Design, fabrication and deployment of an ice boom to protect the Pickering Nuclear Power Plant intake.	Lake Ontario, at Pickering *	3 weeks, Jan 27 to Feb 20 1999.	
Hydro Quebec, Wakefield	Mr. André Chouinard	Concept development, design, fabrication and deployment of an ice boom to accelerate the formation of a smooth ice cover leading to higher power production.	Gatineau River, Downstream of Paugan Dam *	April 1999 to November 1999.	

311 Legget Drive, Kanata, Ontario, Canada K2K 1Z8
Tel : +1 613 592 2830 Fax : +1 613 592 4950
Email : booms@fleetech.com Website : www.fleetech.com

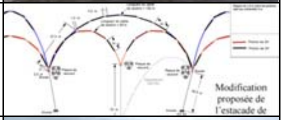
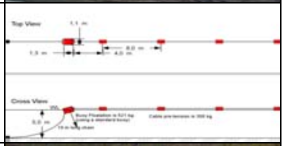
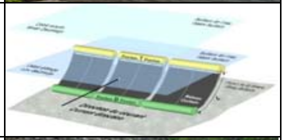
Boom Projects

BMT Fleet Technology Limited

Minerals Management Services, Arctic Region	Mr. Joseph Mullin	Develop a method for the recovery of oil spilled in ice covered water. A modified ice boom design will be used.	Prudhoe Bay	September 1999	
Ontario Hydro	M. Tony Bennett	A study to define the factors that caused the failure of two safety booms.	Chats Falls, Des Joachims*	November 1999 (Project in progress).	
Hydro-Québec	M. Gaétan Lesage	Design of a debris boom to retain floating debris masses.	Complex Lagrande, LG1	December 1999.	
Ontario Hydro, Pickering Nuclear GS	Mr. Bob Ross	Design, fabrication and deployment of an ice and safety boom to protect the Pickering Nuclear Power Plant intake from ice and from people entering the intake.	Lake Ontario, Pickering*	August 2000.	
Canadian Hydro Developers	Mr. Mike Stockton	Design, fabrication and deployment of an ice boom to minimize power losses due to ice blockages.	Appleton, Ontario	November 2000 In progress	
Simard Beaudry	Mr. Denis Robitaille	Concept design of an ice boom that was installed to facilitate the construction of a bridge on the Richelieu River.	South of Montreal, St Bruno	September 2000	
Canadian Coast Guard, Laurentian Region	Mr. Stéphane Dumont	A study to evaluate the feasibility of applying a new technique to sink and re-float ice booms for the Yamachiche boom.	The Port of Prescott and in Lac St. Pierre at Yamachiche	June 2000 – May 2001	
Ministry of Transport, NWT	Mr. Les Shaw	Site visit and concept design development for an ice boom to stabilize an ice bridge, along the Mackenzie Highway.	Tulita, Great Bear River	September 2000	
Bue Offshore	M. C Grolaston	Design of ice boom systems to protect an offshore living facility to be deployed in the Caspian Sea.	Caspian Sea	September 2001 to Nov 2001	
Hydro-Québec	M. Gaétan Lesage	Phase II: Detailed design of a debris boom to retain floating debris masses.	Complex Lagrande, LG1	January to March 2002.	
Hydro Quebec, Maisonneuve Region	Mr. Alain Cyr/ Andre Chouinard	Ice observations and assessment of Hull 2 hydroelectric power plant's winter performance.	Ottawa River near Hull, Quebec.	September 2001 to March 2002.	

Boom Projects

BMT Fleet Technology Limited

Canadian Hydro Developers and IRAP	Mr. Mike Stockton / Ms. Liza Medec	Development of a boom using a net to deploy in fast moving currents	Appleton, Ontario.	November 2001 to February 2002	
Hydro Quebec, Maisonneuve Region	Mr. Alain Cyr/ Andre Chouinard	Design of an ice and debris boom for the Bryson Power Plant.	Ottawa River near Bryson.	September 2001 to March 2002.	
Hydro-Québec	M. Raymond-Marie Tremblay	Construction of an ice boom with a net to reduce the amount of frazil drifting toward the Rivière Des Prairies Power Plant.	Rivière des Prairies	September 2001 to April 2002	
Hydro-Québec	M. Raymond-Marie Tremblay	Improve the efficiency of the boom without a net.	Rivière des Prairies	April to September 2002	
Hydro Quebec, Maisonneuve Region	Mr. Alain Cyr/ Andre Chouinard	Construction of an ice boom for the Bryson Power Plant.	Ottawa River near Bryson.	October to December 2002.	
Domtar Inc., Eddy Specialty Papers.	Mr. Jim Collings	The development of a safety boom design criteria and the development of a design for implementation in 2003.	Ottawa River, Ottawa-Gatineau	September to November 2002	
Hydro Quebec, Maisonneuve Region	Mr. Andre Chouinard	Installation of a web camera via satellite communication system to observe Bryson Ice Boom.	Ottawa River at Bryson.	November to December 2002.	
Domtar Inc., Eddy Specialty Papers.	Mr. Jim Collings	The installation of an 800m long safety boom. The boom consisted of eight spans and nine anchors.	Ottawa River, Ottawa-Gatineau	September to November 2003	
Ministry of Environment of Québec	Mr. Jean-Francois Cyr	Feasibility study to assess the effectiveness of a headloss boom to control the outflow of water from a lake.	Lac Des Deux Montagnes, Montreal	January to April 2003 (in progress)	
Ontario Power Generation	Andre Bourbonnais	Approximately twenty-six booms were designed and installed by others on a number of watercourses for Ontario Power Generation during summer 2003. Several of the booms were damaged or broke and some were completely removed from the sites. BMT analysed the failure mechanisms and recommended corrective actions.	Renfrew, Ontario	June 2004	
Hydro Quebec, Maisonneuve Region	André Chouinard	Design, fabrication and installation of two booms, one for ice retention and one for security purposes. This project included the installation of a web camera to facilitate observation of the boom site.	Gatineau River at Farmers Rapids	October to December 2004	

Boom Projects

BMT Fleet Technology Limited

Hydro Québec, Région Beauharnois	Raymond-Marie Tremblay	This project consisted of the design and installation of a boom to control ice and debris. The boom included a section of pontoons with neutrally-buoyant pipes below the surface pontoon in order to retain submerged debris up to one metre underwater.	Red River at Chute Bell	July to October 2004	
Hydro Québec, Région Beauharnois	Yves Leduc	Concept design of three safety booms; two to be placed upstream and downstream of the Carillon hydroelectric station, and; one to be placed downstream of the Chute Bell station.	Red River and Ottawa River	November to December 2004	
Domtar Inc., Eddy Specialty Papers.	Brian Overton	The study developed design criteria for the construction of three safety booms downstream of the Chaudière Dam and surrounding hydroelectric stations. The booms keep the users of the Ottawa River from coming close to the dangerous flows that can originate from the powerhouses and spillways.	Ottawa River, Ottawa-Gatineau.	September 2004 to May 2005	
Hydro-Québec, Région Montréal	Raymond-Marie Tremblay	This study measured the noise levels generated by the boom deployed on the Des Prairies River during the 2004/055 winter. The noise level was found to be directly proportional to the flow in the river.	Rivière des Prairies, Laval QC	September 2004 to April 2005	
Ontario Power Generation	Steve Chu	This study looked at options to improve the effectiveness of an oil-skimming boom system currently deployed downstream of an electrical generating station on the north shore of Lake Erie. The boom captures oil that is accidentally spilled or leaked into the cooling water system for the plant.	Lake Érie	September 2004 to April 2005	
Ontario Power Generation	Barrie Askew	This project consisted of the design, fabrication and installation of a boom to control ice and debris.	Seymour	April to December 2005	
Hydro Quebec, Maisonneuve Region	Henri Bertrand	This project consisted of the design, fabrication and installation of a boom to control divert debris to a desired location.	Bryson	September to November 2005	

** These projects were existing booms that BMT Fleet Technology Limited re-engineered to improve their performance in retaining ice.*

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT K – EROSION CONTROL METHODS FOR LOW FLOW CHANNEL

Report for the US Army Corps of Engineers, and the cities of Fargo, ND and Moorhead, MN

By: Barr Engineering Co.

FINAL – February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT K – EROSION CONTROL METHODS FOR LOW FLOW CHANNEL

The following information was previously presented as Exhibit H of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

F-K1.0 BACKGROUND INFORMATION

The proposed low-flow channel would consist of a meandering channel within the larger flow diversion, at least for the reach from the confluence of the diversion channel with the Lower Rush River downstream to the confluence of the diversion channel with the Red River of the North (RRN). This reach would receive all flow from the Lower Rush River and the Rush River, and the meandering low-flow channel would provide some measure of mitigation for the loss of the tributaries east of the diversion. The low-flow channel will also extend upstream of the Lower Rush River, since this reach would receive direct precipitation runoff as well as runoff from tile inlets and other miscellaneous inflows. The focus here is on the portion that is downstream of the Lower Rush River, since it will convey the most flow and will be subject to backwater effects from the RRN.

F-K2.0 HYDROLOGY OF THE LOW FLOW CHANNEL

It is assumed that the low-flow channel would convey flows up to the bankfull flow in either the Rush or Lower Rush, plus the coincidental flow in the other channel and the direct runoff from the upstream diversion channel. The bankfull flow rate is often cited as having a 1.5 year return frequency, but may typically range from an approximate 1-year to 2-year return frequency. The bankfull flow rate for either stream has not been measured or calculated, however individual 2-year flows are estimated to be 415 cfs and 302 cfs for the Rush and Lower Rush, respectively. The bankfull flow in either stream is likely to be somewhat less; the coincidental flow from the two channels and watershed inputs would need to be considered together. For now, we will assume that the minimum design flow for the low-flow channel is 415 cfs.

F-K3.0 HYDRAULICS OF THE LOW FLOW CHANNEL

The conceptual design for the low-flow channel consists of a 10-foot bottom width, 4:1 side slopes and 3-foot depth. The low-flow channel would have a top width of approximately 34 feet. The average slope of the diversion channel is 0.0002. However, because the low-flow channel will be meandering it will have a lower slope – perhaps as low as 0.00013 for a sinuosity of 1.5. This slope is very mild, and would result in a bankfull capacity of 40 to 60 cfs for the range of likely channel slopes, assuming a Manning's "n" value of 0.035. In order to convey the assumed design flow, the channel would need to be significantly larger in cross-section. The current channel geometry of the Rush River would provide a useful starting point of reference. Preliminary review indicates that it is indeed a significantly larger channel than the current low-flow channel.

F-K4.0 ALTERNATIVE LOW FLOW CHANNEL DESIGN

If the low-flow channel size is indeed increased significantly, the top width will rapidly approach that of the primary channel, which will have a bottom width of 100-feet. An alternative design for the low-flow channel would be to simply create a “V” shaped bottom to the primary channel. The low point of the “V” could follow a sinuous pattern within the primary channel. Such a channel would accommodate the anticipated large range of inflows from the Rush and Lower Rush in a controlled fashion.

F-K5.0 BACKWATER FROM THE RED RIVER OF THE NORTH

As the diversion channel approaches the RRN on the downstream side, it will become influenced to a greater degree by periodic inundation from sub 5-year flows in the RRN. Those flows are large enough to submerge the downstream end of the diversion channel, but not large enough to flow into the diversion from the upstream side. Inundation here is defined as overtopping the banks of the low-flow channel. As a point of reference, the 90-percent exceedance flow (1,633 cfs) will inundate approximately 2,000 feet or 3 percent of the diversion channel (as measured from the RRN to the Lower Rush). The 80-percent exceedance flow (2,515 cfs) will inundate approximately 12,000 feet or 20 percent of the diversion channel. The duration of flooding will need to be considered in the selection of appropriate vegetation for these areas, or hard armoring may be required if it is determined that the vegetation cannot survive the inundation.

F-K6.0 STABILIZATION ALTERNATIVES

Riprap lining of the low-flow channel has been suggested as a stabilization measure. The cost of armoring the 10-foot wide low-flow channel has been estimated at $20 million for the reach from the Lower Rush to the RRN, and $60 million for a low flow channel throughout the entire diversion channel (not accounting for sinuosity of the low flow channel).

With the alternative channel design recommended above, it is suggested instead that only the lower banks of the primary channel be armored with riprap to a depth below that of anticipated scour. The armoring should extend to a height that would protect against very frequent flows. The proposed armoring method would extend 3 feet below the primary channel bottom, 4 feet wide at the primary channel bottom, with a 12-inch layer extending 10-feet up the primary channel bank from the bottom (protecting 1-foot vertically above the primary channel bottom). It is estimated that this will require approximately 35% less riprap compared to armoring a narrower low-flow channel. The riprap should be covered with topsoil and vegetated in order to provide a natural appearance.

A number of measures could be introduced to stabilize the bottom of the primary channel/low-flow channel while preserving ecological function. Some portion of the bottom would not be amenable to vegetation establishment since it would have flowing water for much of the time. The most important stabilization measure would be native grasses that are appropriate for the soil and climate conditions. Suitable grasses may include cord grass, switch grass, bluestem and Indian grass. These deep-rooted grasses would help to maintain the integrity of the low-flow channel banks.

Grade control may be necessary depending on the final geometry of the low-flow channel and detailed channel hydraulics. For cost-estimating purposes it is recommended that grade-control measures be installed across the entire width of the primary channel every 5,000 feet of channel length (1-foot of primary channel drop). The grade-control measures would be largely buried unless erosion exposes them. It is assumed they would be constructed of rock material but sheet pile could also be used. The rock grade control is assumed to be 100-feet wide (primary channel width) by 20 feet wide by 5 feet deep (370 cubic yards per grade control structure). Approximately 12 structures would be required in the reach downstream of the Lower Rush River. As with the bank protection, the rock should be covered with topsoil and vegetated in order to provide a natural appearance.

F-K7.0 CONCLUSION

The intent of the low-flow channel is to mitigate the loss of habitat from diverting the Rush and Lower Rush Rivers into the diversion channel. It appears that the low-flow channel as currently designed is significantly undersized and would overtop its banks very frequently. The hydrology of the Rush and Lower Rush will have to be considered carefully, and other contributing watershed area will need to be considered to develop an appropriate channel size to more closely mimic natural conditions. An alternative low-flow channel design has been proposed consisting of a shallow "V" shaped bottom to the primary channel. The bottom would be stabilized with native grasses and supplemented by periodic grade-control measures for additional protection. The duration of inundation of the diversion channel by the RRN will need to be considered, but should impact a relatively small portion of the channel.

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT L – REVIEW OF GEOTECHNICAL DATA

Report for the US Army Corps of Engineers, and the Cities of Fargo, ND and Moorhead, MN

By: Barr Engineering Co.

FINAL – February 28, 2010

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HYDRAULIC STRUCTURES

EXHIBIT L - REVIEW OF GEOTECHNICAL DATA

F-L1.0 INTRODUCTION

This exhibit presents a review and interpretation of the existing geotechnical data to generate the parameters used in the preliminary design. The data described herein were collected during The Fargo-Moorhead Metro Feasibility Study (FMMFS) initiated by the U.S. Army Corps of Engineers (USACE) in 2008. Most of the data collected for use during Phase 3 of the FMMFS are associated with what was known as the In-Town Levee Alternative and the Minnesota Diversion Alternative. Additional data were collected in 2010 for use during Phase 4 of the FMMFS. The locations for data collection focused on the proposed structure locations where minimal information was available.

Currently, there are two alternatives being evaluated and they are the North Dakota Diversion (LPP) Alternative and the Minnesota Diversion (FCP) Alternative. The existing data were utilized for this Phase 4 report and it is considered representative of the overall ground conditions for the concept feasibility designs presented in Appendix F.

F-L2.0 GEOTECHNICAL EXPLORATION

The USACE performed a geotechnical exploration as part of the FMMFS. The results of the exploration are summarized in Appendix I of the report USACE (2010). The regional design issues associated with geology, topography, hydrogeology, and seismic risk are discussed in detail in that report and thus are not addressed herein. This exhibit focuses on the site specific data utilized for feasibility design purposes. As a result, the following review discusses the field exploration and laboratory testing as they relate to the design parameters.

F-L2.1 FIELD EXPLORATION

The field exploration conducted by the USACE included machine borings for disturbed sample collection and offset boreholes for undisturbed sample collection. Additionally, Cone Penetration Test (CPT) soundings were conducted to obtain in-situ soils information. The boring logs of the machine borings and CPT logs of the CPT soundings are included in Appendix I of the USACE (2010) report.

F-L2.1.1 Soil Borings

The initial investigation program during the Spring/Summer 2009 included 51 exploratory borings (machine borings) for the In-Town Levee Alternative. There were a total of 10 offset holes for undisturbed sample collection during this initial investigation. Subsequently, there were 85 machine holes completed also for the In-Town Levee Alternative distributed as 45 holes on the Fargo side and 40 on the Moorhead side.

The investigation for the Minnesota Diversion Alternative included 40 exploratory borings. A subsequent subsurface investigation was completed in December 2009 along the North Dakota diversion alternative. This investigation also included 2 offset holes for collection of undisturbed samples.

The exploratory borings were conducted using an all-terrain vehicle (ATV) rig. The borings were conducted using continuous sampling alternated with split spoon sampling. The undisturbed samples were classified in the field by a geologist and stored in jars for later testing in the laboratory.

The offset holes were performed near the machine borings with the purpose of obtaining 5-inch undisturbed samples. One undisturbed soil sample was obtained for each formation in each offset hole. A total of 12 undisturbed borings were completed, obtaining 46 undisturbed samples. These undisturbed borings were distributed throughout the project, with four being completed for the in-town levee alignment in Fargo, three for the in-town levee alignment in Moorhead, three along the Minnesota Diversion alternative, and two along the North Dakota Diversion alternative.

The subsequent Phase 4 program carried out during the Spring/Summer 2010 included a total of 47 borings, with 17 on the Fargo side and 30 on the Moorhead side.

F-L2.1.2 CPT Soundings

A total of 64 CPT soundings were performed during the Phase 3 field investigation. A total of 45 soundings were performed on the Moorhead side and 19 soundings on the Fargo side. All the CPT soundings were performed as part of the In-Levee Alternative. The CPT soundings were conducted using a 20-ton CPT truck mounted rig from the USACE Savannah District.

In the CPT a cylindrical cone is pushed vertically into the ground at a constant rate of penetration of 20 mm/s. During penetration, measurements are made of cone tip resistance, side friction of the cylindrical shaft just above the tip, and pore-water pressure generated by cone penetration.

F-L2.2 LABORATORY TESTING

Disturbed samples collected during the field exploration were taken to the laboratory for index property testing. Undisturbed samples collected during the field exploration were taken to the laboratory for undisturbed laboratory testing including shear strength and compressibility testing. The following briefly describes in more detail the laboratory testing performed on disturbed and undisturbed samples.

F-L2.2.1 Index Properties

Index property testing was mainly performed on disturbed samples and included water content, unit weight, specific gravity, liquid limit, plastic limit, and grain size. The tests were performed following the appropriate ASTM standard. The results of this testing helped identify characteristics of the soils and define the stratigraphy. This characterization allowed identification of the different soil formations at the site including: Alluvium, Sherack, Poplar River, Brenna, Oxidized Brenna, Argusville, and Till. Section F-L3.0 discusses the findings regarding index properties.

F-L2.2.2 Shear Strength Tests
The majority of the laboratory testing had the objective of determining the shear strength of the different soils involved. The shear strength testing involved the following tests: 1) isotropically consolidated undrained triaxial compression test with pore-water pressure measurements (R-Bar tests); 2) direct shear test (DS); and 3) unconsolidated undrained tests (Q tests). Some limited index property testing was also performed on undisturbed samples. Section F-L3.0 discusses the findings regarding strength tests.

F-L2.2.3 Compressibility
A limited number of consolidation tests were performed on undisturbed samples. Section F-L3.0 discusses the findings regarding compressibility.

F-L3.0 INTERPRETATION OF GEOTECHNICAL DATA

F-L3.1 INDEX PROPERTIES

This section discusses the results of the index properties of the different formations encountered at the site. Some of the data include test results of the same formations in samples taken in other USACE projects involving Lake Agassiz clays.

F-L3.1.1 Water Content
Figure F-L1a shows the results of the moisture content versus depth for the different formations in the study area. It can be seen from Figure F-L1a that water content ranges between 8 and 86 percent depending on the formation. Figure F-L1b shows the results of the moisture content versus elevation for the different formations.

Table F-L1 summarizes the average water content and the standard deviation for each formation. It can be seen from Table F-L1 that the Brenna Formation exhibits the highest average water content with a value of 59.2 percent whereas the Till exhibits the lowest average water content with a value of 18.1 percent.

Table F-L1 Average Water Content Per Formation

Formation	Average Water Content (%)	Standard Deviation (%)
Alluvium	31.78	5.13
Argusville	47.13	8.16
Brenna	59.24	10.65
Oxidized Brenna	49.00	7.43
PL Sherack	50.75	7.01
Poplar River	36.98	10.14
Sherack	36.48	5.45
Till	18.14	4.41
Unit "A" Till	18.26	7.60
Topsoil/Alluvium	35.19	4.72

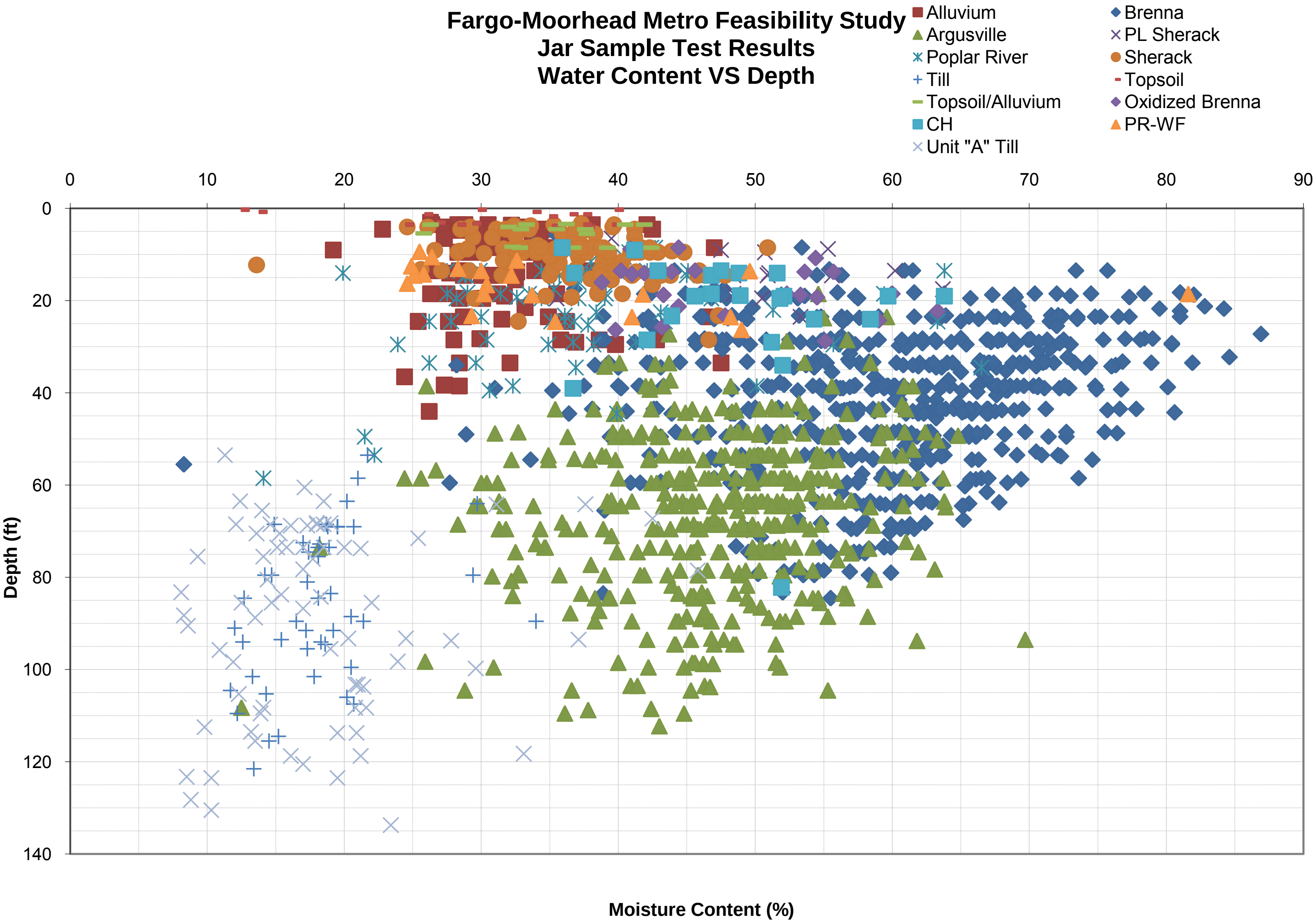


Figure F-L1a Water Content vs. Depth

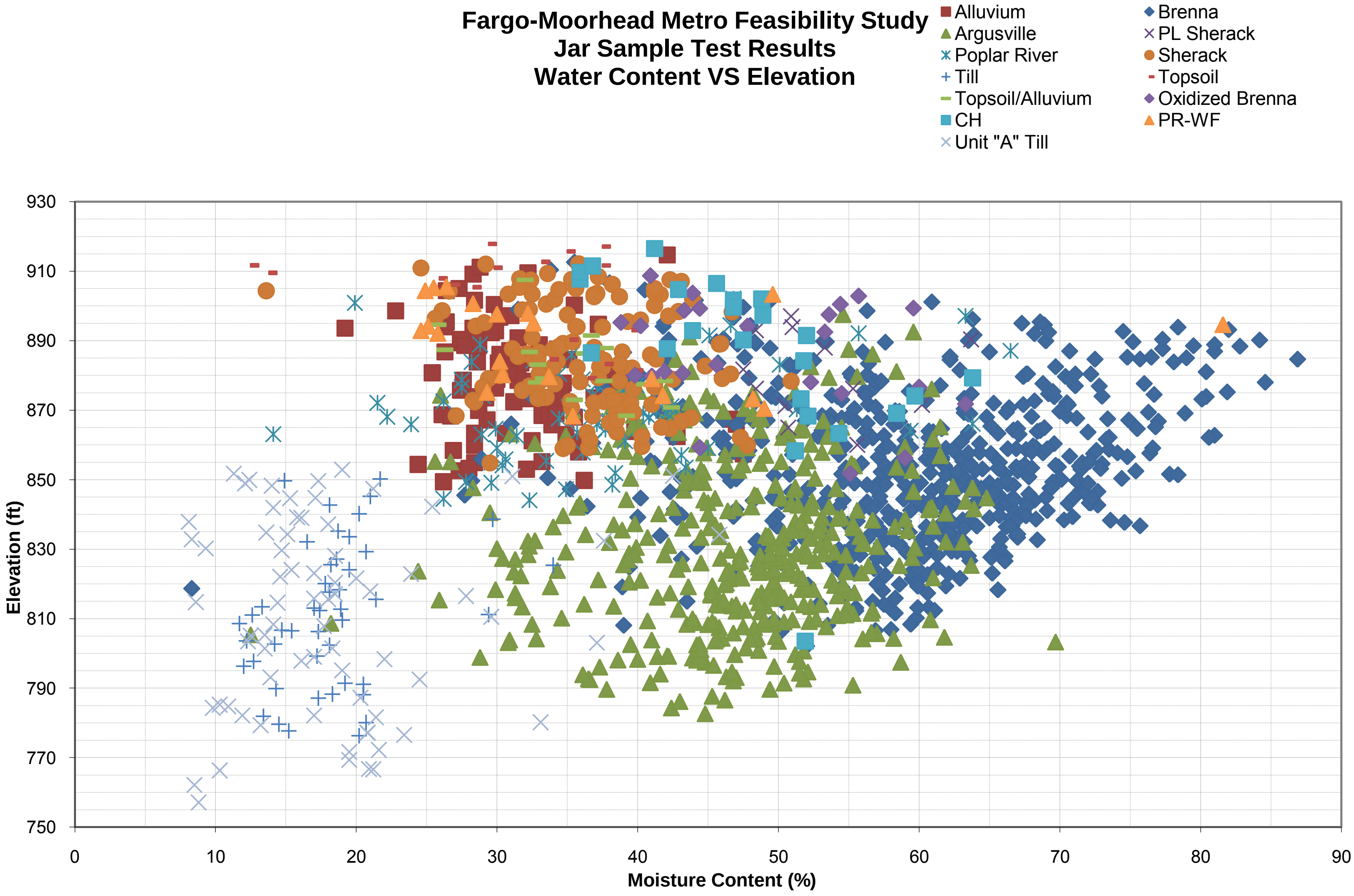


Figure F-L1b Water Content vs. Elevation

F-L3.1.2 Unit Weight

Table F-L2 summarizes the average saturated unit weight and the standard deviation for each formation. It can be seen from Table F-L2 that the Brenna Formation exhibits the lowest average saturated unit weight with a value of 104.1 pcf whereas the Poplar River-West Fargo exhibits the highest average saturated unit weight with a value of 123.2 pcf.

Table F-L2 Average Unit Weight Per Formation

Formation	Average Unit Weight (pcf)	Standard Deviation (pcf)
Alluvium	119.9	4.8
Sherack	117.6	4.0
PL Sherack	113.9	0.7
Poplar River - West Fargo	123.2	1.3
Poplar River - Harwood	115.6	0.5
Poplar River	119.4	3.9
Oxidized Brenna	111.3	6.2
Brenna	104.1	9.0
Brenna/Argusville Transition	107.8	3.8
Argusville	106.5	3.2

F-L3.1.3 Atterberg Limits

Figure F-L2a shows the results of the liquid limit versus depth for the different formations in the study area. It can be seen from Figure F-L2a that water content ranges between 1 and 134 percent depending on the formation. Figure F-L2b shows the results of the liquid limit versus elevation for the different formations.

Table F-L3 summarizes the liquid limit and the standard deviation for each formation. It can be seen from Table F-L3 that the PL Sherack exhibits the highest average liquid limit with a value of 100.3 percent and the Brenna Formation exhibits the second highest average liquid limit with a value of 95.4 percent whereas the Till exhibits the lowest average liquid limit with a value of 29.6 percent.

Table F-L3 Average Liquid Limit Per Formation

Formation	**Average Liquid Limit (%)**	**Standard Deviation (%)**
Alluvium	57.9	11.2
Argusville	72.0	14.2
Brenna	95.4	15.0
Oxidized Brenna	93.5	17.6
PL Sherack	100.3	8.6
Poplar River	50.6	18.1
Sherack	64.9	14.3
Till	29.6	6.2
Unit "A" Till	27.8	7.1
Topsoil/Alluvium	58.9	7.8

The liquid limit and water content data were used to compute the liquidity index. Figures F-L3a and F-L3b show the liquidity index versus depth and elevation for the different formations in the study area, respectively. It can be seen from the figures that most of the data are between a liquidity index of 0.0 and 0.7.

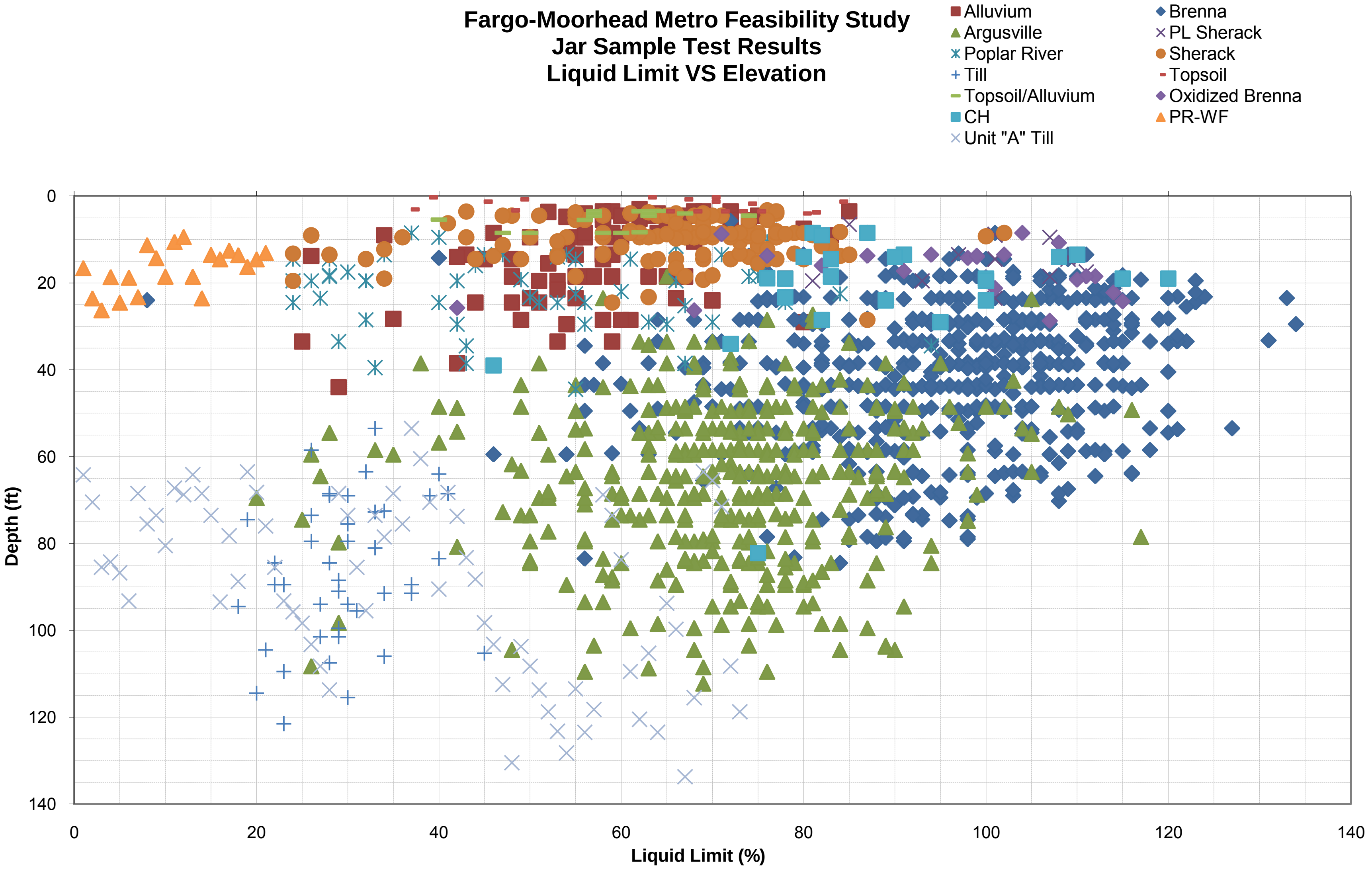


Figure F-L2a Liquid Limit vs. Depth

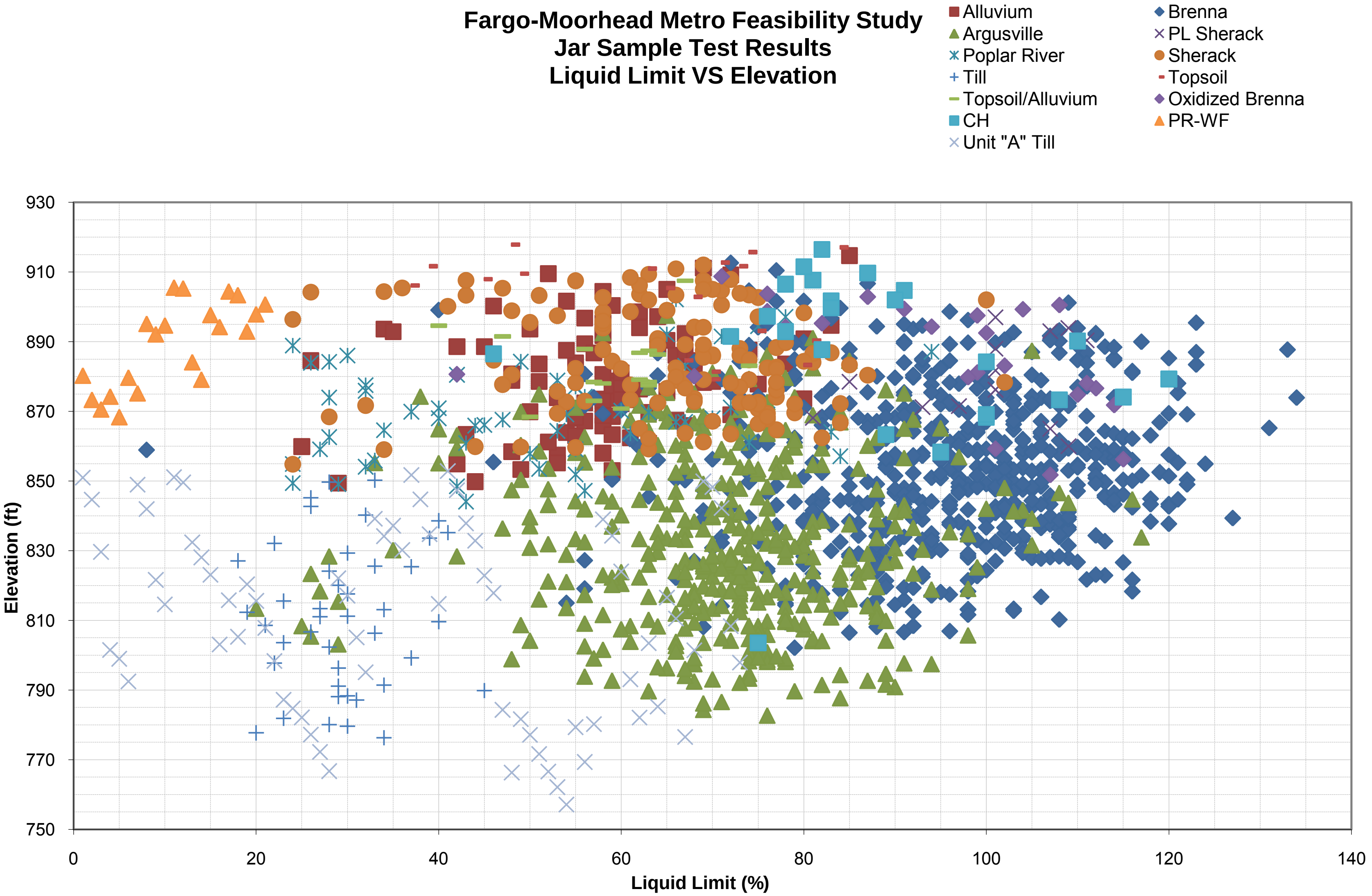


Figure F-L2b Liquid Limit vs. Elevation

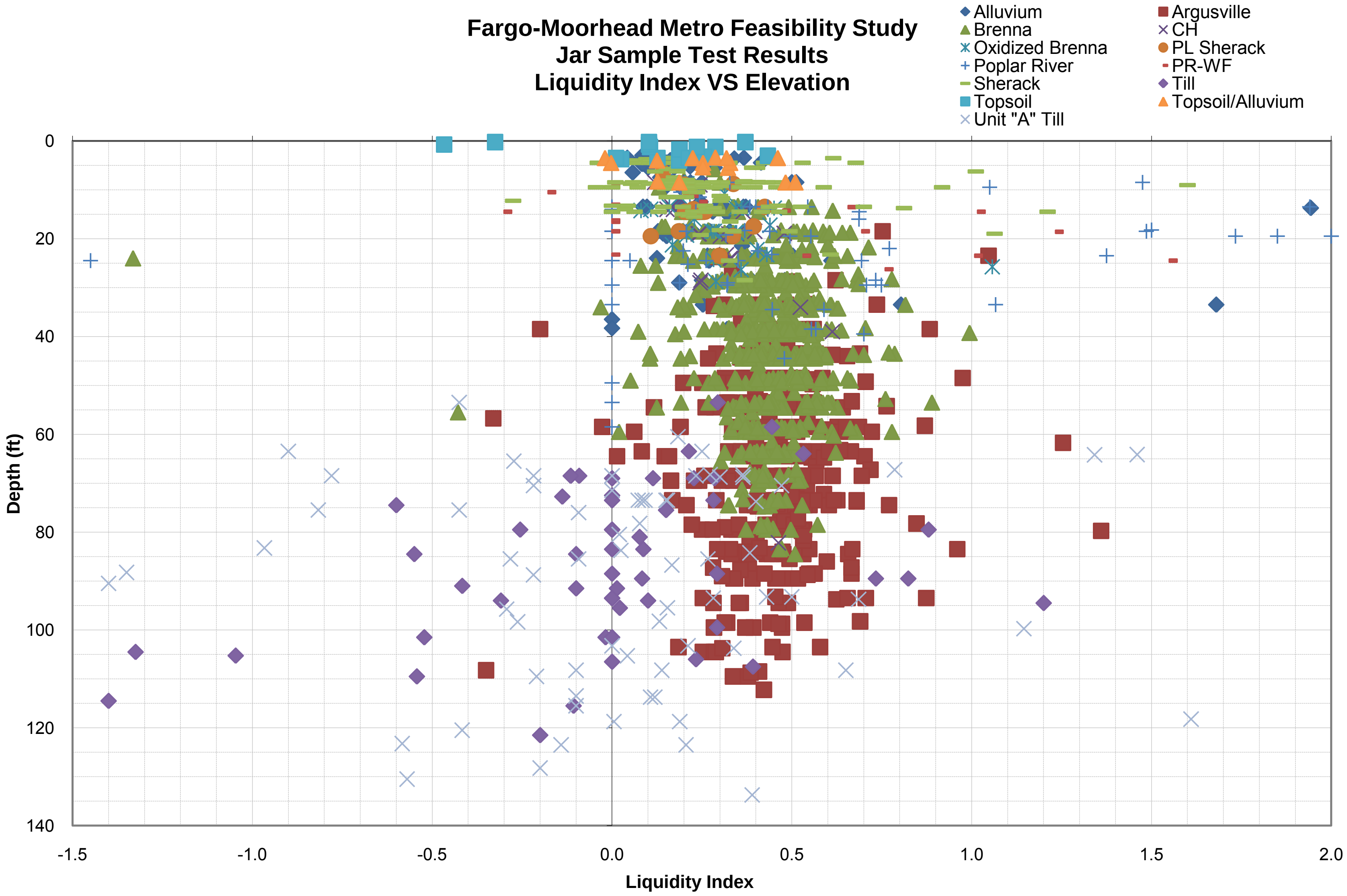


Figure F-L3a Liquidity Index vs. Depth

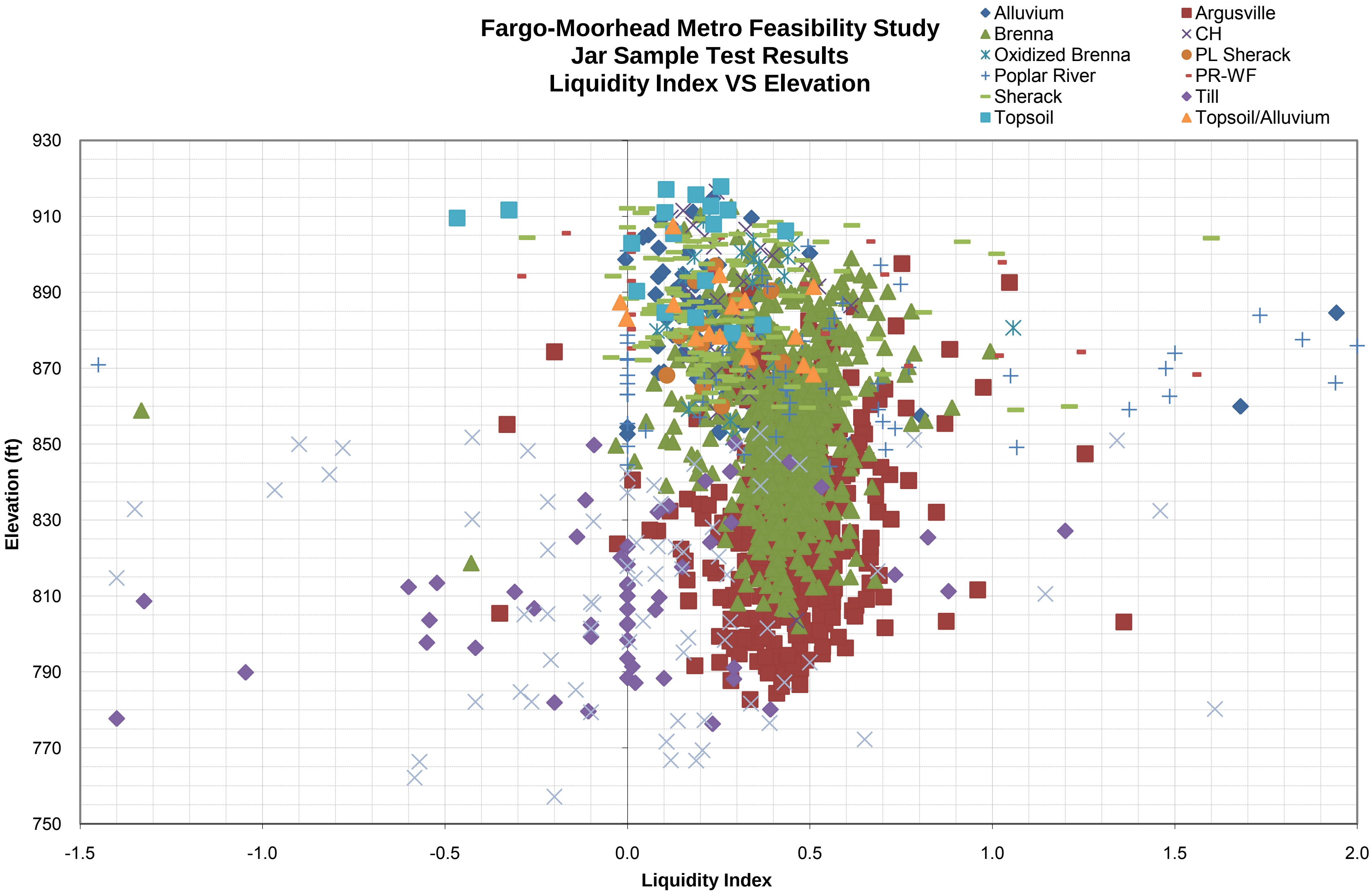


Figure F-L3b Liquidity Index vs. Elevation

F-L3.2 SHEAR STRENGTH

This section of the report discusses the shear strength of the soils involved in the project. The shear strength discussion utilizes the laboratory tests and CPT results. The interpretation of the shear strength evaluates the undrained and drained shear strength.

F-L3.2.1 Undrained Shear Strength

The undrained shear strength was initially interpreted based on the laboratory test results. The FMMFS included a significant amount of laboratory undrained shear strength data. The data were collected from Unconsolidated Undrained (UU) tests on undisturbed samples from different formations. The standard procedure used in the FMMFS consisted of testing three specimens at a given depth.

Figure F-L4 shows the results of UU tests versus depth of the different formations encountered at the study area. All of the tested samples included in the FMMFS are within the upper 85 ft. Each point in Figure F-L4 represents the average of the three specimens tested at that depth. It can be seen in Figure F-L4 that the shear strength plot shows the typical results of a clay deposit with a desiccated crust in the upper 15 to 20 ft and then the undrained shear strength tests increases linearly with depth. Figure F-L4 also includes two lines that represent the upper and lower bound of the data. In the upper portion of Figure F-L4, generally above 15 ft to 20 ft, data correspond to the Sherack, Poplar, and Oxidized Brenna. On the other hand, below 20 ft, most of the data are associated with the Brenna and Argusville formations.

The undrained shear strength was also interpreted using the CPT sounding results. An extensive amount of CPT soundings were available from the FMMFS and were utilized to estimate the undrained shear strength using the following equation:

where

q_t = is the corrected cone tip resistance
σ_{vo} = total vertical stress
N_{kt} = cone factor

The laboratory test data included in Figure F-L4 were used to estimate the cone factor N_{kt} as 19. Figure F-L5 shows the undrained shear strength vs. depth obtained from the CPT soundings in North Dakota (using an N_{kt} of 19). Figure F-L5 also includes the upper and lower bound from the CPT soundings included in Figure F-L4. It can be seen from Figure F-L5 that the undrained shear strength from the CPT shows the general tendency of a typical clay deposit with a desiccated crust at the top and then increasing with depth until it encounters the till at depths varying between 70 ft to 110 ft.

Similarly, Figures F-L6 and F-L7 show the undrained shear strength vs. depth obtained from laboratory testing and the CPT soundings in Minnesota (using an N_{kt} of 19), respectively. Figure F-L7 also includes the upper and lower bound from the CPT soundings included in Figure F-L6.

Figures F-L4 through Figure F-L7 validate the selected cone factor N_{kt} of 19. Furthermore, these figures are used to estimate undrained shear strength for design.

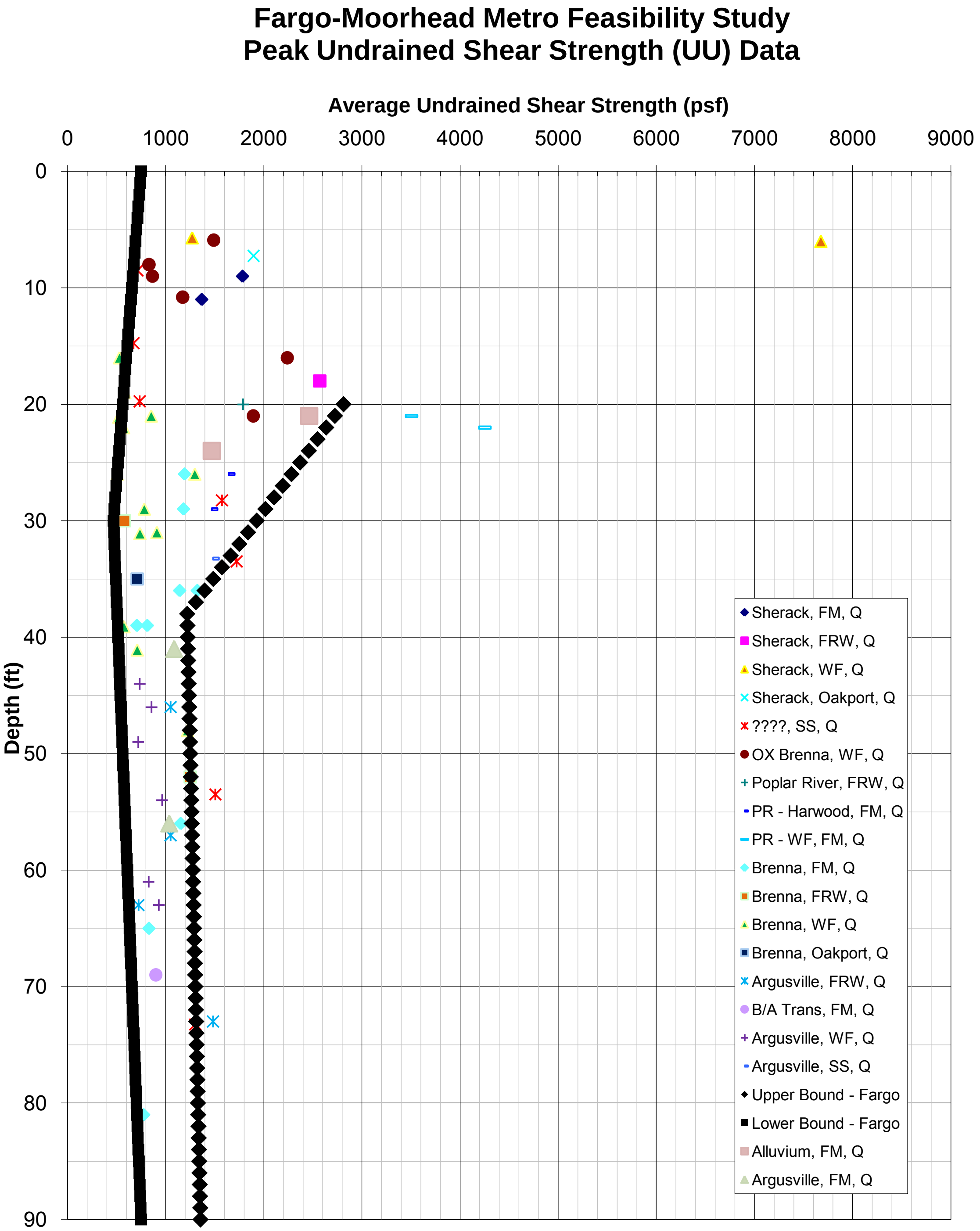


Figure F-L4 **Peak Undrained Strength vs. Depth with Upper and Lower Bounds Taken From North Dakota CPT Soundings**

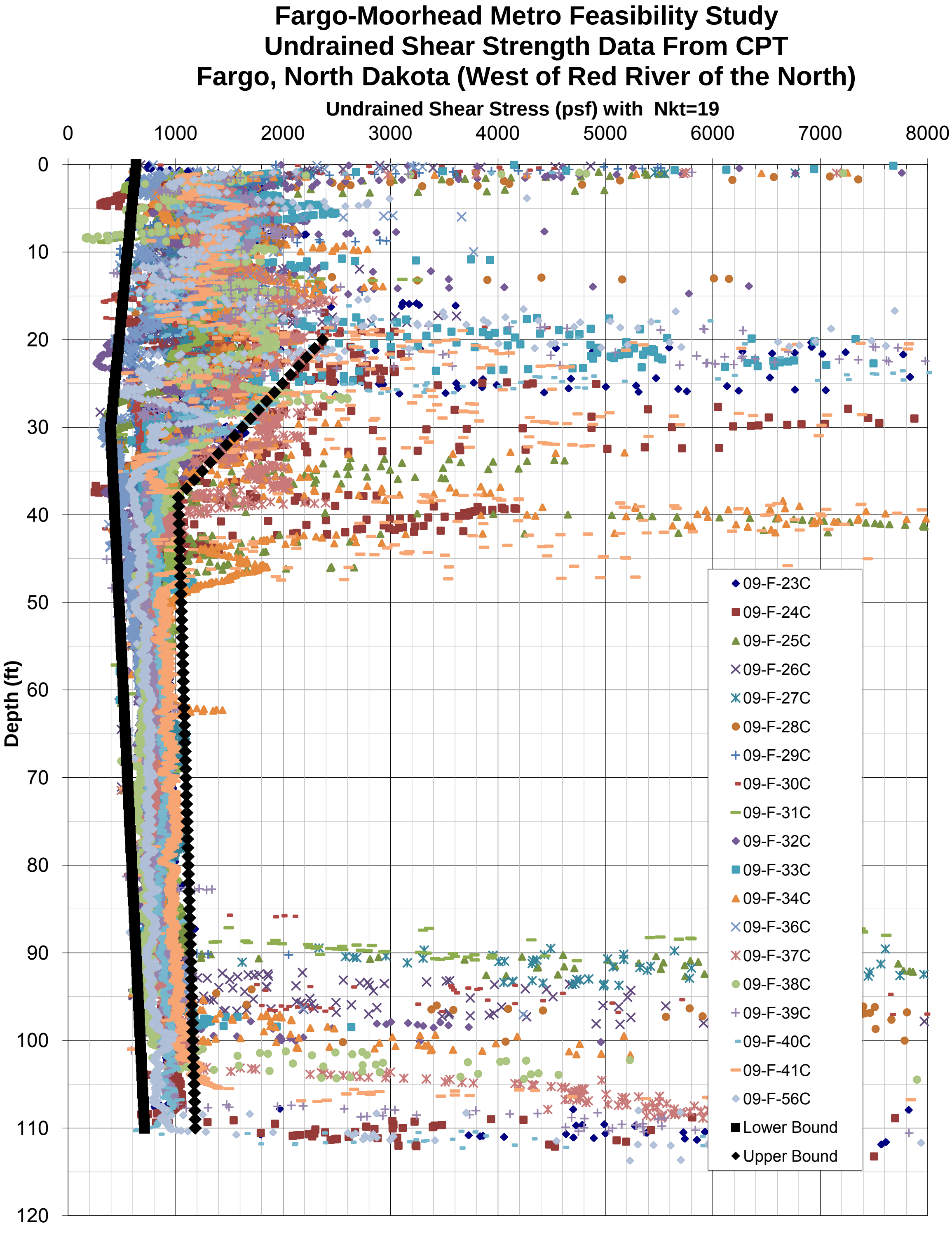


Figure F-L5 **Undrained Shear Strength vs. Depth Obtained from CPT Soundings in North Dakota**

Fargo-Moorhead Metro Feasibility Study
Peak Undrained Shear Strength (UU) Data

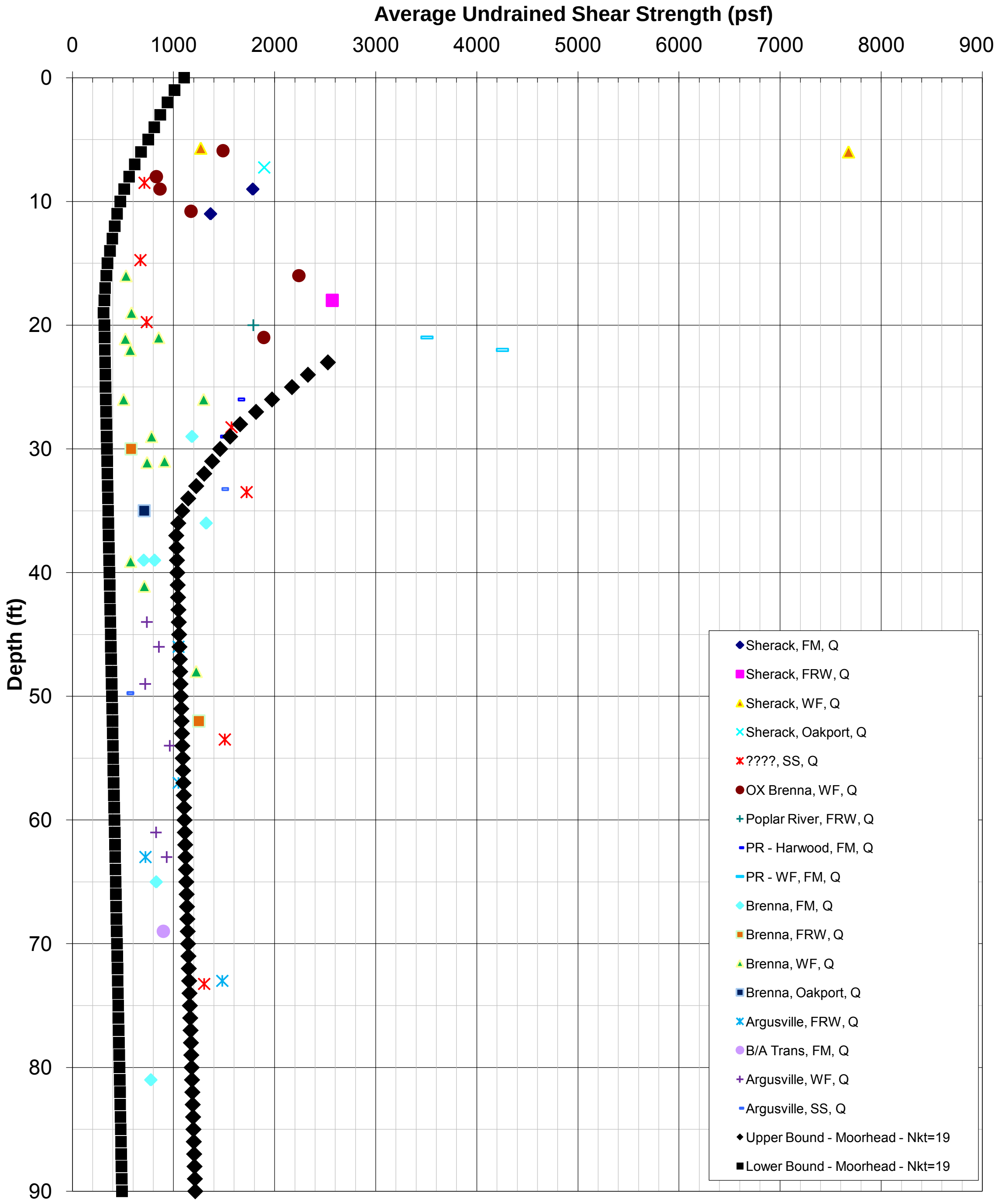
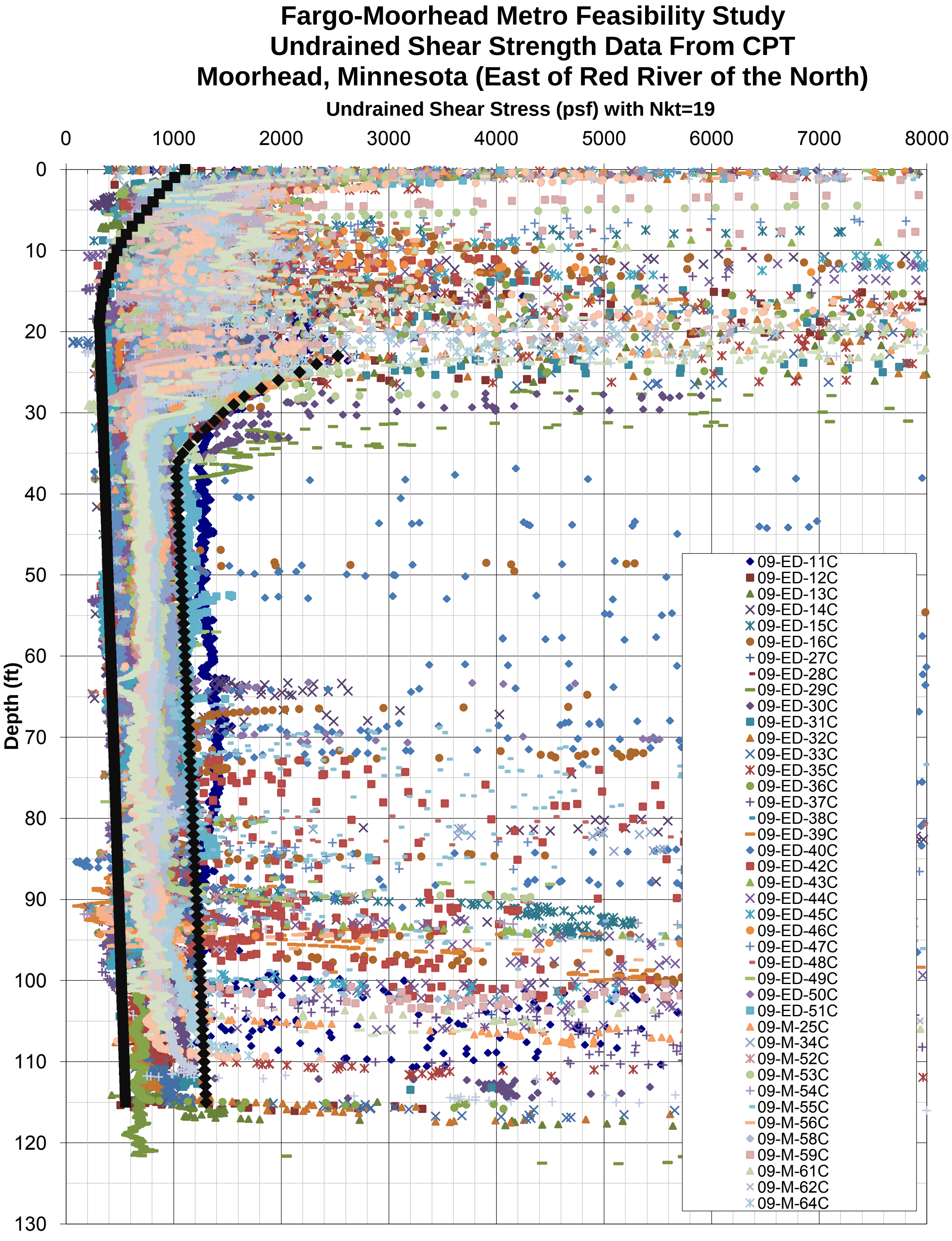


Figure F-L6 Peak Undrained Strength vs. Depth with Upper and Lower Bounds Taken From Minnesota CPT Soundings



Fargo-Moorhead Metro Feasibility Study
Undrained Shear Strength Data From CPT
Moorhead, Minnesota (East of Red River of the North)

Figure F-L7 Undrained Shear Strength vs. Depth Obtained from CPT Soundings in Minnesota

F-L3.2.2 Drained Shear Strength

The drained shear strength was interpreted based on the laboratory test results on undisturbed samples. The laboratory testing included consolidated undrained triaxial shear tests with pore-water pressure measurements (R-bar) and direct shear tests (DS). All the laboratory data used to interpret the drained shear strength were included in the FMMFS and are part of a large database which contains information on Lake Agassiz soils from this and other projects.

Actual laboratory testing includes samples from the Sherack, Poplar River, Brenna, and Argusville formations. Data included in the FMMFS is summarized in Table F-L4.

Table F-L4 Summary of Drained Shear Strength

	Effective Stress (1)	
Formation	**ϕ'**	**c' (psf)**
Alluvium	31	0
Sherack	28	0
Poplar River-West Fargo	34	0
Poplar River-Harwood	26	0
Oxidized Brenna (2)	19	0
Brenna (2)	13	0
Argusville (2)	15	0
Till	34	0

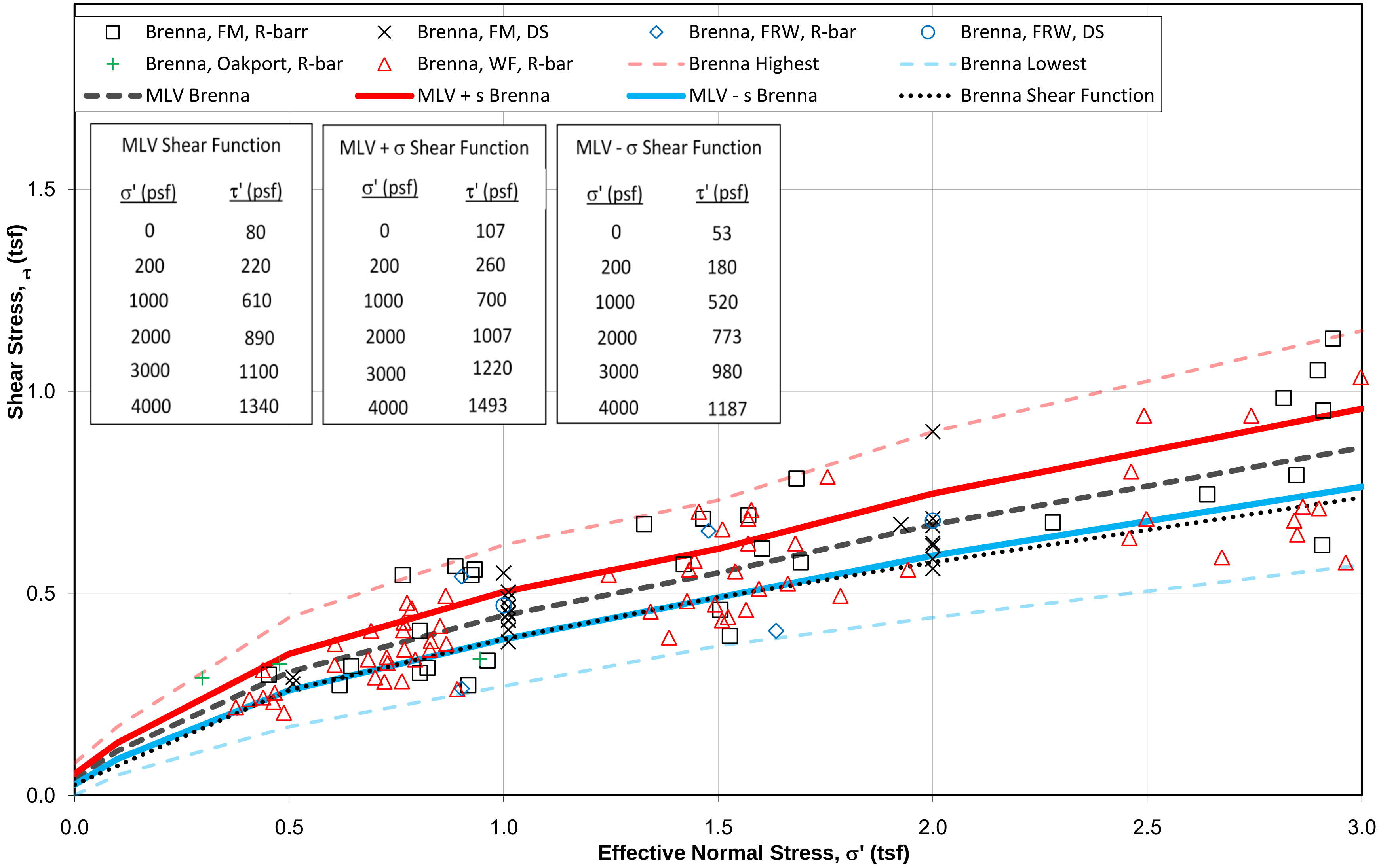
(1) The effective stress parameters for the formations are based on the R-Bar triaxial and direct shear tests. The failure criterion is defined as ultimate deviator stress which equates to the deviator stress at 15% and 20% axial strain. It is assumed that there is no cohesion intercept for the Mohr-Coulomb shear strength envelope.

(2) A curvilinear shear strength envelope was developed for the effective stress analysis of the diversion channel excavated slope. The curvilinear envelope is defined as the line one standard deviation less than the most likely value (MLV).

In the interpretation of the triaxial test data in terms of effective stress in Table F-L4, the failure criteria utilized the stress at 15% strain. This is known as the large strain failure criterion. The reason for utilization of this failure criterion is based on the USACE experience with the Red River Valley clays which exhibit the strain softening phenomena. The large strain failure criterion is applicable when analyzing channels, slopes, and levees which are subject to strain softening. In the case of analysis and design of shallow or deep foundations under drained conditions, the existing design methodology utilizes the drained strength defined by the peak deviator stress criterion. As a result, the triaxial laboratory data were revisited and new failure envelopes were developed using the peak deviator stress as failure criterion. More discussion on this topic is included in Exhibit O.

Figure F-L8 shows the results of the reinterpretation of the triaxial test results of the Brenna formation using the peak failure criterion. It can be seen from Figure F-L8 that utilizing the peak stress as failure criterion results in a higher effective friction angle than using the 15% strain criterion.

A similar reinterpretation of the triaxial test results using the peak failure criterion was performed for the Oxidized Brenna and Argusville formations. The use of the peak stress as failure criterion resulted in higher friction angles than using the 15% strain criterion. These reinterpreted results were used in the analysis and design of deep foundations under drained conditions. This aspect is further discussed in Exhibit O.



MLV Shear Function

σ' (psf)	τ' (psf)
0	80
200	220
1000	610
2000	890
3000	1100
4000	1340

MLV + σ Shear Function

σ' (psf)	τ' (psf)
0	107
200	260
1000	700
2000	1007
3000	1220
4000	1493

MLV - σ Shear Function

σ' (psf)	τ' (psf)
0	53
200	180
1000	520
2000	773
3000	980
4000	1187

Figure F-L8 Shear Strength of Brenna Formation using Peak Stress as Failure Criterion

F-L3.3 COMPRESSIBILITY

The compressibility characteristics of the different soils involved in the study area were tested using the incremental loading test procedure. The results of the tests are summarized in Table F-L5.

Table F-L5 Summary of Consolidation Tests

Project, Formation, Boring, Sample No., Specimen No.	p_c' (tsf)	p_o' (tsf)	OCR	Cc	Cr	USCS Soil Type	Formation	Project
FRW PL Sherack 01-5MU, 1	1.95	0.56	3.48	0.22	0.07		PL Sherack	FRW
FRW PL Sherack 01-5MU, 1	1.95	0.84	2.32	0.22	0.07		PL Sherack	FRW
FRW Sherack 01-12MU, 1	3.20	0.72	4.44	0.17	0.04		Sherack	FRW
FRW Brenna 01-5MU, 2	2.95	1.60	1.84	1.16	0.19		Brenna	FRW
FRW Brenna 01-5MU, 2	2.95	1.80	1.64	1.16	0.19		Brenna	FRW
FRW Argusville 01-5MU, 4	2.60	2.20	1.18	1.05	0.18		Argusville	FRW
FRW Argusville 01-5MU, 4	2.60	2.30	1.13	1.05	0.18		Argusville	FRW
SS ST-10,	2.52	0.63	3.97	0.30	0.10	CH		SS
SS ST-10,	4.38	0.78	5.63	0.80	0.23	CH		SS
SS ST-10,	2.48	0.96	2.60	0.84	0.16	CH		SS
SS SB-11,	4.30	1.19	3.62	0.60	0.10	CH		SS
SS SB-17,	3.70	1.12	3.30	0.57	0.11	CH		SS
SS-WRL ST-7,	1.04	0.70	1.48	0.26	0.10	CH		SS-WRL
SS-WRL ST-7,	3.1	0.86	3.60	0.67	0.19	CH		SS-WRL
SS-WRL ST-9,	2.3	0.56	4.08	0.32	0.12	CH		SS-WRL
SA-52ndAVE ST-1,	4.0	1.09	3.65	0.76	0.15	CH		SA-52ndAVE
SA-52ndAVE ST-2,	4.5	0.72	6.22	0.33	0.04	CH-ML		SA-52ndAVE
SA-52ndAVE SL-3,	1.8	1.44	1.25	0.24	0.04	CH		SA-52ndAVE
SS-32ND ST-13,	2.0	0.82	2.43	0.17	0.03	CH		SS-32ND
SS-32ND ST-13,	2.7	0.97	2.80	0.83	0.15	CH		SS-32ND
FM PR - WF 09-25MU-M, 2	1.6	0.93	1.72	0.05	0.01	ML	PR - WF	FM
FM PR - Harwood 09-25MU-M, 3	3.4	1.04	3.27	0.29	0.07	CH	PR - Harwood	FM
FM Brenna 09-25MU-M, 4	2.9	1.34	2.15	1.28	0.24	CH	Brenna	FM
FM Argusville 09-25MU-M, 5	2.8	1.95	1.44	0.80	0.13	CH	Argusville	FM
FM Brenna 09-25MU-F, 4	2.8	1.70	1.62	1.10	0.23	CH	Brenna	FM
FM Sherack 09-26MU-F, 1	1.4	0.51	2.80	0.20	0.07	CH	Sherack	FM
FM Brenna 09-26MU-F, 3	3.7	1.04	3.56	0.77	0.19	CH	Brenna	FM
FM Alluvium 09-27MU-F, 1	1.6	0.42	3.81	0.23	0.08	CH	Alluvium	FM
FM Argusville 09-27MU-F, 4	2.8	1.82	1.51	0.87	0.16	CH	Argusville	FM
FM PL Sherack 09-34MU-M, 2	3.8	0.80	4.75	0.80	0.22	CH	PL Sherack	FM
FM OX Brenna 09-59MU-F, 2	3.0	0.95	3.17	0.65	0.17	CH	Brenna	FM
FM Brenna/Argusville 09-59MU, 3	2.8	1.30	2.16	0.41	0.08	CH	Brenna	FM
FM OX Brenna 09-60MU, 2	5.6	0.90	6.20	0.65	0.12	CH	Brenna	FM
FM Brenna 09-60MU, 3	4.7	1.10	4.25	1.06	0.17	CH	Brenna	FM
FM Alluvium 10-78MU, 1	2.9	0.61	4.75	0.28	0.03	CH	Brenna	FM
FM Brenna 10-78MU, 2	3.6	0.90	4.00	0.54	0.04	CH	Brenna	FM
FM Argusville 10-78MU, 3	4.5	1.60	2.81	0.82	0.05	CH	Brenna	FM
FM Alluvium 10-79MU, 1	4.0	0.90	4.44	0.31	0.05	CH	Brenna	FM

Project, Formation, Boring, Sample No., Specimen No.	**p_c' (tsf)**	**p_o' (tsf)**	**OCR**	**Cc**	**Cr**	**USCS Soil Type**	**Formation**	**Project**
FM Argusville 10-79MU, 2	3.9	1.44	2.71	0.69	0.05	CH	Brenna	FM
FM Alluvium 10-80MU, 1	2.0	1.00	2.00	0.14	0.02	CL-SM	Alluvium	FM
FM Brenna 10-80MU, 2	5.0	1.30	3.85	0.63	0.04	CH	Brenna	FM
FM Argusville 101-80MU, 3	4.4	1.70	2.59	0.73	0.09	CH	Brenna	FM

F-L4.0 SUMMARY

The existing geotechnical information from the FMMFS was reviewed. This included the field and laboratory data including index properties, shear strength, and compressibility. The information available was fairly complete for this concept feasibility design phase (additional data will be required for future phases and final design) and only two additions or revisions of the existing data were made to use in this Phase 4. The first addition was the interpretation of the CPT to estimate the undrained shear strength. In this regard, the existing information was further expanded with additional values of undrained shear strength from the CPT. The undrained shear strength is a very important parameter for deep foundation design. All the structures will be supported by deep foundations for this project. The other aspect reviewed was the drained shear strength of the Brenna, Oxidized Brenna, and Argusville formations to be used in the analysis and design of deep foundations under drained conditions. The data presented in the FMMFS utilized the large strain (15% strain) failure criterion, which accounts for the softening phenomena mainly applicable in design of slope stability problems. The data were revisited and a new failure envelopes were developed using the peak stress failure criterion. This design failure criterion is applicable in the design of deep foundations under drained conditions.

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT M – SEEPAGE ANALYSIS

Report for the US Army Corps of Engineers, and the cities of Fargo, ND & Moorhead, MN

By: Barr Engineering Co.

FINAL –February 28, 2010

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT M - SEEPAGE ANALYSIS

F-M1.0 SEEPAGE ANALYSIS

The following information was previously presented as Exhibit J of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

A transient seepage analysis was initially performed as part of Phase 3 to compute the anticipated uplift pressures along the bottom of the control structures on the Red River of the North and the Wild Rice River. In the transient seepage model (SEEP/W in the GeoStudio 2007 suite), sheet pile cutoffs were assumed to extend a variable distance into the foundation soils and the width of the structure was also varied. Using full hydrographs for upstream and downstream of the Red River Control Structure (LPP) for the 100-year event, the development of increased porewater pressures was computed versus time. Then, the highest uplift pressure distribution was provided to the structural engineers performing the feasibility study structural design.

A meeting was held on July 1st, 2010, between Barr Engineering, the US Army Corps of Engineers, Moore Engineering, Houston Engineering, and the City of Fargo at the offices of Barr Engineering in Edina, MN. The uplift pressures on the foundations were discussed in this meeting. It was decided that vertical holes would be installed through the foundation such that the increased pressures from the floodwaters would be transmitted to the foundation bottom to increase the uplift pressures. This would act to reduce the structural loads. Consequently, because the uplift pressures are virtually identical to the increased downward pressures from the floodwaters, the results from the transient seepage analysis were made obsolete. No further description of the seepage analysis is included herein. It should be noted that 10-foot sheet piles are still anticipated to be installed on the upstream and downstream edges of the structures for piping and scour protection.

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT N – SLOPE STABILITY

Report for the US Army Corps of Engineers, and the cities of Fargo, ND & Moorhead, MN

By: Barr Engineering

FINAL –February 28, 2010

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT N: SLOPE STABILITY

F-N1.0 INTRODUCTION

Slope stability is a major concern for many natural and engineered slopes in the Red River Valley of North Dakota and Minnesota. The soils deposited by glacial Lake Agassiz are medium to stiff consistency and exhibit high plasticity, which leads to low shear strengths. The shear strengths can be especially low under drained conditions (long-term) because of the mineralogical composition of the material, and thus the drained strength of the material typically controls the design of stable slopes. As such, the slopes along the diversion channel and tributary approach channels at and near the hydraulic structures required a separate stability analysis in addition to the overall channel stability analysis performed by the US Army Corps of Engineers (USACE). The structures for which these additional analyses were performed were:

- Tributary Hydraulic Structure at the Maple River
- Tributary Hydraulic Structure at the Sheyenne River
- Tributary Hydraulic Structure at the Wild Rice River
- Red River Control Structure
- Road and Rail Bridges

The stability of the tie-back levees associated with proposed Storage Area 1 were also analyzed.

The slope stability analysis presented in this exhibit provides methodology, input parameters, results, and recommendations.

F-N2.0 GEOLOGY

The Fargo-Moorhead area is covered by 200 to 300 feet of clays, tills, and granular sediments from past glaciation associated with the formation of glacial Lake Agassiz. The Sherack Formation, Brenna Formation, Argusville Formation, and glacial till are the primary soil units in the area, though other more localized soil units have been identified such as the Poplar River Formation, Plastic Laminated Sherack Formation, and Oxidized Brenna Formation. The material types of most interest for geotechnical engineers in the Red River Valley are the Brenna and Argusville Formations, which exhibit high plasticity . The clay was deposited after the most recent glacial event, and it is characterized as a lacustrine clay but it was altered at some locations during lower lake levels where erosion channels were filled with sediments. Additionally, exposure to the atmosphere may have caused overconsolidation of the formation. The Sherack Formation was deposited during higher lake levels and may contain clays and silts as well as organics. The Sherack Formation is continuously present except in areas where relatively recent alluvial or fluvial processes have eroded it away.

F-N3.0 SLOPE STABILITY MODELING

A slope stability analysis was carried out to ensure that slopes near the hydraulic structures and the tie-back levees associated with Storage Area 1 exhibit adequate factors of safety.

F-N3.1 METHODOLOGY

The main objective of the slope stability analysis was to evaluate the stability of earth slopes and associated structural elements near the hydraulic structures to ensure that slope instability does not impact the integrity of the hydraulic structures and impede their intended functions. The stability of the tie-back levees associated with Storage Area 1 is also critical as they will contain a very large volume of water. The impact of groundwater flow on stability was also assessed, and steady-state seepage conditions were used in the stability analysis.

Slope stability factors of safety are computed as the summation of forces (or moments) that resist movement divided by the summation of forces (or moments) that cause movement. Conditions that may decrease stability in a slope include increasing water levels or pore-water pressure, loading on the crest of a slope, or removal of material or water from the slope toe.

Two types of stability analyses are typically performed for slopes: the Undrained Strength Stability Analysis (USSA) and the Effective Stress Stability Analysis (ESSA). The USSA is performed to analyze the case in which loading or unloading is applied rapidly and excess porewater pressures do not have time to dissipate during shearing. This scenario typically applies to rapid loading from, for example, embankment construction where the loading takes place quickly relative to the permeability of the soils. It is often referred to as the “end-of-construction” case.

The ESSA is performed to account for much slower loading or unloading, or no external loading, in which the drained shear strength of the materials is mobilized and no excess pore-water pressures are developed. For example, a slowly moving landslide is best analyzed using the ESSA. For this reason, the ESSA is often referred to as the “long term” case.

Only the ESSA was performed as part of the slope stability analysis for the hydraulic structures because previous analysis of the main channel slopes identified that using the drained (long term) strength of the soils resulted in lower factors of safety. Thus, the ESSA was the controlling case for slope stability. However, for the Storage Area 1 tie-back levees, both cases were examined because embankment construction could possibly mobilize the undrained shear strength of the material and the USSA could be the controlling case.

For typical long-term conditions, such as with the normal river conditions, the minimum recommended factor of safety for levees and embankments is 1.40 according to USACE standard EM 1110-2-1913, Table 6-1b (USACE, 2003). For typical flood conditions, assuming steady-state seepage, the minimum recommended minimum factor of safety is 1.40 (USACE, 2003).

Stability analyses for transient (sudden drawdown) conditions are performed for drained and undrained strength parameters. A factor of safety of 1.0 to 1.2 is accepted according to USACE standard EM 1110-2-1913, Table 6-1b (USACE, 2003). A minimum factor of safety of 1.20 was used for the transient analyses.

F-N3.2 SOFTWARE

The slope stability analysis was conducted using GeoStudio 2007, a computer-modeling program developed by GEO-SLOPE International, Ltd. Within GeoStudio 2007, two modules were used: SEEP/W and SLOPE/W.

SEEP/W uses the finite-element analysis technique to model groundwater flow and porewater pressure distribution within porous materials such as soil. This method was chosen because its comprehensive formulation makes possible to analyze both simple and highly complex seepage problems. It can model saturated and unsaturated flow, steady-state and transient conditions, and a variety of boundary conditions. SEEP/W generates an output file containing the heads at the nodes of the finite element mesh. Integration of the modules within the software package allows the use of a SEEP/W head file in the slope stability program to compute effective stresses. In this manner, the impact of seepage on stability can be evaluated.

The slope stability analysis was conducted using SLOPE/W. This model uses limit equilibrium theory to compute a factor of safety for earth and rock slopes. It is capable of using a variety of methods to compute the factor of safety of a slope while analyzing complex geometry, stratigraphy, and loading conditions. SLOPE/W allows the user to import the groundwater head file from the seepage analysis to compute effective stresses.

Spencer’s method was used as the search technique to determine the factor of safety of the embankment in this stability analysis. This method is considered the most adequate because it satisfies all conditions of static equilibrium and provides a factor of safety based on both force and moment equilibrium.

F-N3.3 PARAMETERS

The input parameters for modeling were developed by St. Paul District of the USACE. They are summarized below.

F-N3.3.1 Seepage Analysis: Mesh and Boundary Conditions

The mesh for the seepage analysis consisted of triangular and quadrilateral, unstructured finite elements. The mesh was made fine enough to capture the effects of any areas of high hydraulic gradient and to ensure that the boundary conditions were applied effectively to the model.

Boundary conditions applied in the seepage model include:

- Total heads applied along the upstream model boundaries for the approach channel and hydraulic structure modeling. The total heads were fixed at various values as discussed later in this exhibit. Model boundaries were placed far from the area of interest (250 feet from channel centerline for tributary approach channels, 500 feet from the channel centerline for the Red River Control Structure approach channel, and 250 feet from the radial wall centerline for hydraulic structures) such that boundary effects could be minimized.

- Potential seepage face review nodes placed along the slopes. During model cycling, these nodes check for any pressure heads or total flux values above zero and, if found, resets them to zero iteratively. This makes it possible for a realistic phreatic surface to develop.

- Default no-flow boundary conditions on the bottom of the model.

- Default no-flow boundary conditions with infinite elements were placed on the left and right boundaries for the Storage Area 1 modeling.

The steady-state analysis included the following assumptions:

- Storage Area 1: Two water levels on the interior of the model were assumed: (a) at the ground surface of 908 and (2) at Elevation 922 to simulate flood conditions.

- Tributary Approach Channels at Maple, Sheyenne, and Wild Rice Rivers: There is always water in the approach channel corresponding to the normal water elevation. This elevation corresponds to the mean annual flow. The phreatic surface was assumed to be hydrostatic.

- Red River Control Structure Approach Channel: There is always water in the approach channel corresponding to the normal water elevation. This elevation corresponds to the mean annual flow. The phreatic surface was assumed to slope toward the channel because this channel is significantly deeper than the tributary approach channels. The total head boundary condition was fixed at Elevation 893.

- Maple River and Sheyenne Hydraulic Structures: There is no water in the diversion channel. This represents worst-case conditions because having the weight of the water acting against the slope only increases the factor of safety.

- There is no infiltration on the ground surface.

F-N3.3.2 Seepage Analysis: Material Properties

The USACE performed an extensive seepage analysis for the main channel slopes. The parameters used in modeling presented herein were obtained from testing and interpretation performed by the USACE.

Three additional material types were used in seepage modeling in addition to the material types used by the USACE. They were Levee Fill, Granular Backfill, and Concrete.

The seepage properties used in modeling are shown in Table F-N1.

Table F-N1 Seepage Parameters Used in Modeling

Material	Soil Class	Saturated Hydraulic Conductivity, k		Porosity, n	Coefficient of Volume Compressibility, m_v	Residual Volumetric Water Content, $(\theta_{vw})_r$
		cm/sec	ft/day	-	1/psf	-
Sherack	CH	1.0E-06	2.8E-03	0.50	9.0E-06	0.050
Oxidized Brenna	CH	5.0E-07	1.4E-03	0.55	1.0E-05	0.055
Brenna	CH	1.0E-07	2.8E-04	0.63	3.0E-05	0.063
Argusville	CH	1.0E-07	2.8E-04	0.60	3.0E-05	0.060
Till	CL	5.0E-06	1.4E-02	0.45	3.0E-05	0.045
Levee Fill	CH	2.0E-07(1)	5.7E-04(1)	0.50	9.0E-06	0.050
Granular Backfill	SP	1.0E-02	2.8E+01	0.40	3.0E-05	0.040

(1)- Assumed to be half order of magnitude lower than Sherack permeability

The Concrete was assumed to have very low permeability, and the same permeability characteristics as the Sherack Formation were used for this material.

F-N3.3.3 Stability Analysis: Material Properties

As for the stability analysis, the USACE also performed an extensive slope stability analysis for the main channel slopes. The material properties determined by the USACE for their analysis were used in the stability modeling described herein. The strength parameters used by the USACE were defined using the large strain failure criterion which is appropriate in the Red River Valley clays to simulate the softening phenomenon.

The unit weight and strength properties used in the models are shown in Table F-N2.

Table F-N2 Unit Weight and Shear Strength Parameters Used in Modeling

Material	Soil Class	Saturated Unit Weight	ESSA (Drained)		USSA (Undrained)	
			Cohesion	Friction Angle	Cohesion	Friction Angle
		pcf	psf	deg	psf	deg
Sherack	CH	117	0	28	1400	0
Oxidized Brenna (1)	CH	111	0	19	1000	0
Brenna (1)	CH	103	0	13	650	0
Argusville (1)	CH	107	0	15	825	0
Till (2)	CL	122	0	34	1900	0
Levee Fill (3)	CH	117	200	28	1400	0
Granular Backfill	SP	125	0	30	0	30

(1)- Non-linear failure envelope used for these materials (drained)
(2)- Assumed to be impenetrable for wedge (composite) failure
(3)- Assumed to consist of compacted Sherack Formation; a nominal cohesion was added to account for compaction (drained)

As noted, the glacial till material was selected as an impenetrable material so that wedge failures could be modeled accurately. When weaker soils overlie much stronger soils, such as the Argusville Formation overlying the glacial till, there is a tendency for the lower portion of the potential failure surface to be truncated and much of the failure surface is coincident with the contact between the weak and strong soils. Modeling the stronger soil as impenetrable allows the model to find these potential failure surfaces, which in many instances are the controlling cases. Impenetrable materials do not require unit weight input values.

F-N4.0 STORAGE AREA 1 LEVEES

A slope stability analysis was carried out to demonstrate that proposed levees associated with Storage Area 1 are stable.

F-N4.1 GEOMETRY

The levee geometry was assumed to have a 15-foot crest width, a crest elevation of 927, and 4H:1V slopes. The ground surface was assumed to be Elevation 908, which is the lowest ground surface elevation found along the levee alignment (along the northern portion). This leads to the highest embankment and represents the most critical embankment cross-section. Figure F-N1 shows typical levee geometry.

Boring logs in the vicinity of the proposed Storage Area 1 indicate the general stratigraphy found elsewhere along the diversion channel alignment, but some variability existed. To account for this variability in the modeling and its impact on levee stability, four stratigraphic cases were analyzed. Table F-N3 shows the four stratigraphy types and the elevations for the top contact of each material.

Table F-N3 Stratigraphy Types for Storage Area 1 Levees

	Top of Unit Elevation (feet)			
Material	**Type 1**	**Type 2**	**Type 3**	**Type 4**
Sherack/Alluvium	908	908	908	908
Oxidized Brenna	None	900	900	None
Brenna	900	895	None	900
Argusville	880	880	880	890
Till	850	850	850	850

F-N4.2 RESULTS

Using the stratigraphy types shown in Table F-N3, stability was analyzed for steady-state seepage assuming flood conditions on the interior of Storage Area 1 with the goal of achieving a factor of safety of 1.40 for the ESSA case. A factor of safety of 1.30 was desired for the USSA case. The ESSA and USSA factor of safety results are shown in Table F-N4.

Table F-N4 Factor of Safety Results for Storage Area 1 Levees (Steady-State)

	Factor of Safety			
Analysis Type	**Type 1**	**Type 2**	**Type 3**	**Type 4**
ESSA	1.29	1.39	1.67	1.33
USSA	2.21	2.28	2.37	2.26

Because the Type 1 and Type 4 stratigraphy resulted in inadequate factors of safety, a transient seepage analysis was performed. This procedure (established in a meeting with the USACE on December 13th, 2010) involved instantly placing a total head boundary condition at 922 on the upstream face of the levee and then determining the permeability values required for the flow to reach close to steady-state at an arbitrary time of 30 days. Steady-state seepage was defined by tracking pore pressures at randomly placed nodes throughout the levee. All permeability values were adjusted up and down together using the same multiplication factor such that the relative permeabilities remained unchanged. These permeability values were then used with the 100-year hydrograph and a full transient analysis resulting in computed factors of safety with time. The minimum factor of safety was then reported for the upstream and downstream levee faces (ESSA and USSA). The Type 1 and Type 4 stratigraphy factors of safety are shown in Table F-N5. Type 2 and Type 3 were not analyzed because their steady-state factors of safety were considered adequate.

Table F-N5 Factor of Safety Results for Storage Area 1 Levees (Transient)

Analysis Type	Factor of Safety	
	Type 1	Type 4
ESSA Downstream	1.45	1.43
ESSA Upstream	1.53	1.59
USSA Downstream	2.24	2.31
USSA Upstream	2.37	2.44

It can be seen in Table F-N5 that the Type 1 and Type 4 factors of safety are adequate because they all exceed 1.40, which is the most stringent factor of safety requirement for levees. All factor of safety and seepage model outputs are attached.

F-N5.0 APPROACH CHANNELS

A slope stability analysis was performed on several proposed approach channels to assess their stability. The tributary channels that were evaluated include the Maple River, Sheyenne River, and Wild Rice River as well as the Red River approach. Stratigraphy and soil properties used in the models were provided by the USACE. The initial models were designed with river channel slopes of 3H:1V.

Transient conditions were also evaluated for each river considering the 100- and 500-year flood conditions and using the Phase 4 hydrographs that provided water levels for flood conditions over a period of 36 days. These analyses did not represent the governing case for slope stability, so the results are not presented in this exhibit.

F-N5.1 MAPLE RIVER

The Maple River stratigraphy from the ground surface consists of Sherack, Oxidized Brenna, Brenna, Argusville, and glacial till. The normal river elevation is at 881.5 feet above sea level. For steady-state conditions at a slope of 3H:1V, the left and right bank were found to have a factor of safety of 1.25 for the ESSA. This value does not meet USACE requirements of a factor of safety greater than or equal to 1.40. Figure F-N2 shows the initial model created with slopes of 3H:1V. Because the factor of safety was not met, two alternatives were developed in order to obtain a factor of safety of at least 1.40.

The first alternative was designed to decrease the slope steepness by increasing the horizontal to vertical displacement. Slopes were increased until a steady state factor of safety of 1.40 or greater was reached. The Maple River slopes were found to have a stable factor of safety when slopes were at 4.25H:1V.

The second alternative is to install steel piles into the 3H:1V river bank slopes. Two options were evaluated within this alternative. The first option was to use two rows of steel pipe piles with smaller diameter and lower shear capacity spaced every 12 feet on both sides of the river bank. The second option was to use one row of piles on each side of the river bank having a larger diameter and higher shear capacity. Both alternatives were evaluated and the results are discussed below. A cost comparison for each option was created in order to determine the most economical solution.

F-N5.1.1 Alternative 1: Change in Slope

The first alternative flattens the slope while maintaining a river channel bottom width of 20 feet and a depth of 19.3 feet. The required factor of safety was met when the channel slopes were at 4.25H:1V. Because the other tributary approach channels required flatter slopes and for consistency, a slope of 5H:1V was be used for all tributary approach channels. Figure F-N3 shows the Maple River steady-state analysis geometry with river bank slopes of 5H:1V. The required excavation to create a 3H:1V channel (including the channel and both banks) is 70.0 CY per linear foot of channel. At an estimated $3.49 per CY, the total excavation cost would be around $244.35 per linear foot of channel. The required excavation to create a 5H:1V channel (including channel and both banks) is 97.6 CY per linear foot of channel. At an estimated $3.49 per CY, the total excavation cost would be around $340.78 per foot along the alignment. A summary of all the cost estimates is included in this Exhibit.

F-N5.1.2 Alternative 2-Option 1: Two Rows of Piles

The option consists of placing steel piles in the river bank slope of 3H:1V. This option consists of two rows of 24-inch hollow steel pipe piles with a wall thickness of ½ inch spaced at 12 feet along the length of the channel. The estimated channel length is about 3,000 feet. The piles are 57 feet long and 61 feet long, allowing them to extend 5 feet into the glacial till. Figure F-N4 shows the river cross-section with a single row of piles in place. The cost of excavation for 3H:1V slopes would be 70 CY per foot along the alignment at $3.49 per CY for a total of $244.35 per foot along the alignment. The piles would result in 19.7 VLF (vertical linear feet) per foot along the alignment at $117.77 with a total pile cost of $2,316.18 per foot along the alignment. The total cost of this alternative is $2,560.54 per foot along the alignment.

F-N5.1.3 Alternative 2-Option 2: One Row of Piles

The option consists of one row of piles on both sides of the river bank spaced every 12 feet along the channel length. Figure F-N5 shows the river cross-section with a single row of piles in place. The steel pipe design consists of 30-inch hollow steel pipe piles with a wall thickness of ½ inch. The piles are 59 feet long allowing it to extend 5 feet into the glacial till. The cost of excavation for 3H:1V slopes would be 70 CY per foot along the alignment at $3.49 per CY for a total of $244.35 per foot along the alignment. The piles would result in 9.8 VLF per foot along the alignment at $157.80 with a total pile cost of $1,551.73 per foot along the alignment. The total cost of this alternative is $1796.08 per foot along the alignment.

F-N5.1.4 Cost Comparison

A cost comparison for the three slope stability options confirms that the most economical alternative would be to excavate the tributary river channel slopes to 5H:1V. Once all the river tributaries were modeled, the minimum slope of 5H:1V was required for the Red River and Wild Rice River.

F-N5.2 SHEYENNE RIVER

The Sheyenne River stratigraphy from the ground surface consists of Sherack, Oxidized Brenna, Brenna, Argusville, and glacial till. The normal river elevation is at 903.24 feet above sea level. The initial model consisted of 3H:1V slopes. The left and right bank had a factor of safety of 1.16, not meeting the USACE requirement of 1.40. Figure F-N6 shows a river cross-section of the initial model. The slopes were then flattened while maintaining a river channel bottom width of 20 feet and a depth of 21.9 feet. The required factor of safety was met when the channel slopes were 5H:1V. Figure F-N7 shows the new cross-section for a river channel with slopes 5H:1V.

F-N5.3 WILD RICE RIVER

The Wild Rice River stratigraphy from the ground surface consists of Sherack, Oxidized Brenna, Argusville, and glacial till. The normal river elevation is at 894.08 feet above sea level. The initial model evaluated at a slope of 3H:1V resulted in a right and left bank factor of safety of 1.27. This does not meet the USACE requirement of 1.40. Figure F-N8 shows the initial model river cross-section. The river bank slopes on the Wild Rice River were then flattened while maintaining a river channel bottom width of 40 feet and depth of 21.7 feet. The required factor of safety was met when the channel slopes were at 3.5H:1V. Figure F-N9 shows the new cross-section for a river channel with slopes 5H:1V.

F-N5.4 RED RIVER CONTROL STRUCTURE

The Red River stratigraphy from the ground surface consists of Sherack, Oxidized Brenna, Argusville Formation, and glacial till. The normal river elevation is at 882.96 feet above sea level. The initial model evaluated at a slope of 3H:1V resulted in a right and left bank factor of safety of 0.99. This does not meet stability nor the minimum USACE required factor of safety of 1.40. Figure F-N10 shows the initial model river cross-section. The slopes were then flattened while maintaining a river channel bottom width of 50 feet and a depth of 38.4 feet. The required factor of safety was met when the channel slopes were 7H:1V. Figure F-N11 shows the new cross-section for a river channel with slopes 7H:1V.

F-N6.0 HYDRAULIC STRUCTURES AT MAPLE AND SHEYENNE RIVERS

The slope stability analysis addressed the global stability of the radial walls at the immediate entrance and exit of the diversion channel crossing of the Maple River and Sheyenne River aqueducts. These structures are critical and the slopes along the approach channels near the structures must remain stable to ensure that no damage to the structures occur especially during diversion channel operation in a flood event.

F-N6.1 GEOMETRY

Radial walls are required at these structures to minimize head loss as the flow in the diversion channel passes underneath the aqueducts at their respective river crossings. After some discussion regarding the geometry of these walls and based on the input from the USACE for the Red River Control Structure, it was decided that the walls should extend all the way to the top of the aqueduct channel. This created a very high wall, and the global stability of this wall needed to be checked.

The structural analysis of the wall resulted in a 4-foot-thick reinforced concrete stem and footing, with the toe and heel extending 12 feet from the stem. This resulted in an overall footing width of 28 feet. Five rows of piles are spaced at 6 feet nominally parallel to the wall (longitudinally) and 6.25 feet perpendicular to the wall (transversely). The H-piles were sized as HP 14x73 piles for structural purposes with HP 14x89 piles used in the cost estimate to allow for corrosion. Granular backfill was assumed to be placed behind the wall and above a 1H:1V cut slope extending upward from the heel of the footing. Drainage was provided via weep holes in the wall.

At the Maple River, the stem extends from Elevation 872.06 (channel invert) upward to Elevation 903.5 (top of wall). At the Sheyenne River, the stem extends from Elevation 883.68 (channel invert) upward to Elevation 917.5 (top of wall). See Figures F-N12 and F-N13 for representations of the geometry used in modeling for the Maple and Sheyenne hydraulic structures, respectively.

F-N6.2 PILES

The piles described above were accounted for in the limit equilibrium slope stability model using the parameters shown in Table F-N6 below.

Table F-N6 Pile Properties

Parameter	Maple	Sheyenne
Pile Length [ft]	40.6*	37.0*
Pile Spacing (longitudinal) [ft]	6.0	6.0
Pile Spacing (lateral) [ft]	6.25	6.25
Shear Capacity in Yield, Unfactored [kips]	106	118
Factor of Safety	1.4	1.4
Shear Capacity in Yield, Allowable [kips]	74.3	90.0

*- Piles extend about 7.5 feet into glacial till

The shear force for the piles was applied parallel to the slip surface rather than perpendicular to the reinforcement. The unfactored shear capacity in yield was changed in the model until an adequate stability factor of safety was achieved. A factor of safety of 1.40 was used for the pile shear capacity because it represents the highest required factor of safety for global slope/wall stability and it can be considered conservative for any other stability cases in which lower factors of safety are required.

F-N6.3 RESULTS

Total heads for the steady-state seepage model for the Maple River hydraulic structure are shown in this Exhibit. The plot shows that the phreatic surface gradually decreases from its far-field value (250 feet back from wall; total head of 881.50; normal flow for the Maple River) along the left to the granular backfill where it slopes steeply toward the weep hole at the bottom of the wall. Total heads for the steady-state seepage model for the Sheyenne River hydraulic structure are also shown in this Exhibit. The plot shows that the phreatic surface gradually decreases from its far-field value (250 feet back from wall; total head of 903.24; normal flow for the Sheyenne River) along the left to the granular backfill where it slopes steeply toward the weep hole at the bottom of the wall.

For the Maple River hydraulic structure, stability modeling results are shown in this Exhibit for the cases without and with piles, respectively. It should be noted that the entry and exit ranges and the strength of the concrete was established such that the method found a potential failure surface corresponding to global stability. Output reports are attached for the modeling runs.

The ESSA factors of safety are shown in Table F-N7 below. USSA factors of safety were much higher and are not presented here.

Table F-N7 Hydraulic Structure Factors of Safety

Scenario	Modeled Factor of Safety		Required Factor of Safety
	Maple	Sheyenne	
No Piles	0.50	0.55	1.40
With Piles	1.41	1.40	1.40

As discussed previously, the unfactored shear capacities presented in Table F-N6 were obtained by changing the capacities until the required global factor of safety was achieved. The actual shear capacity of the piles is about is 206 kips based on 50 ksi steel using only the area of the web for shear capacity. The web area for HP 14x73 piles is 6.87 in^2.

It can be seen that piles are required to ensure adequate global stability. It should be noted, however, that using the shear capacity for the global stability analysis takes only the structural capacity of the pile into account and not the soil-structure interaction. In other words, the effect of the piles being forced into the soil within the sliding mass is not considered.

Using a simplified limit equilibrium approach is considered adequate for this feasibility study. In final design, more sophisticated analysis methods will be warranted and the soil-structure interaction will be addressed. If this analysis indicates that the factor of safety against global instability is inadequate, additional piles and/or tie-backs may be required.

F-N7.0 ROAD AND RAIL BRIDGES

Discussions were had with the USACE regarding the costs associated with improvement techniques to ensure a stable 5H:1V slope at the bridge abutments relative to extending the bridge length to cross the full channel width. Because the improvement techniques could incur significant costs, the cost estimate presented herein uses the extended bridge lengths across the full channel width. This approach is considered somewhat conservative, and improvement techniques could be the less expensive option for final design.

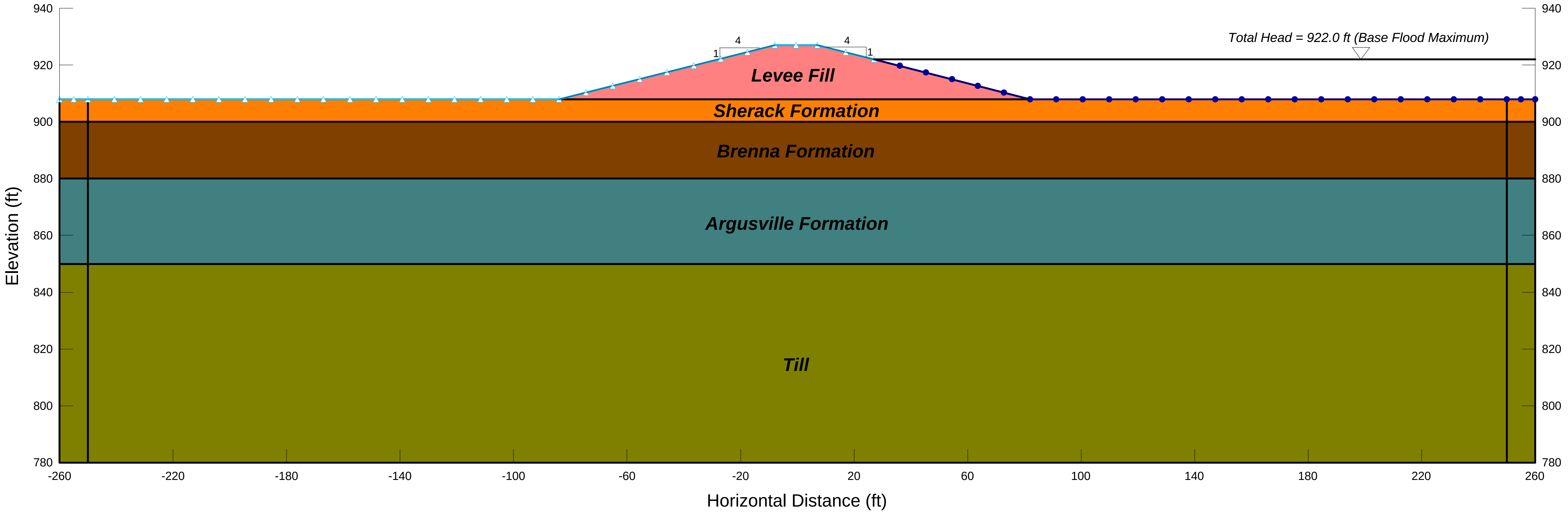
Possible improvement techniques to consider in final design are:

- Piles
- Deep soil mixing
- Lightweight fill

These techniques could be used in association with retaining walls to ensure stability of the abutments.

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1_FIGURE_F-N1.gsz
Last Saved Date: 2/24/2011

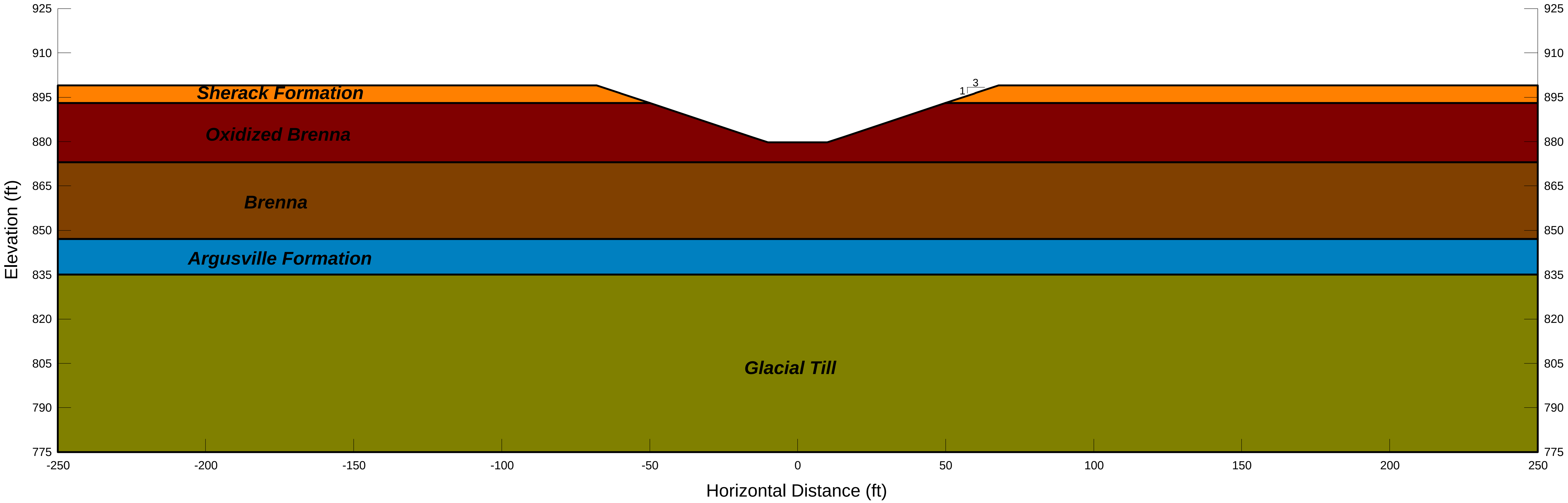
Figure F-N1. Typical Storage Area 1 Levee Geometry



Fargo-Moorhead, Phase 4, Maple River Upstream Channel Stability Analysis

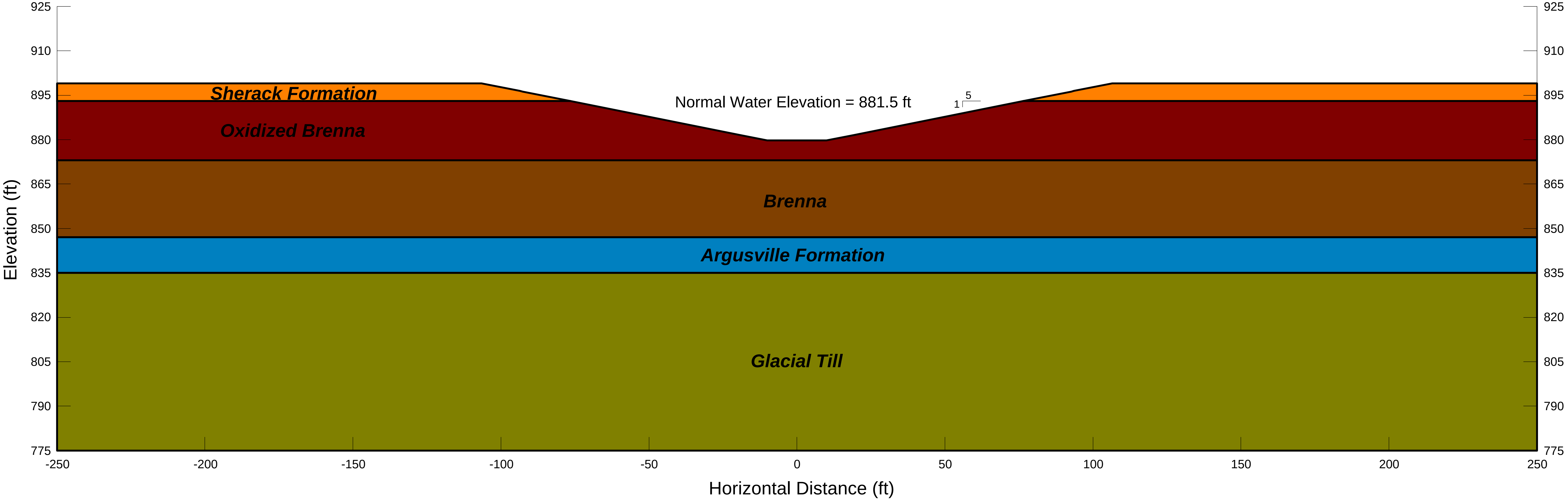
File Name: Maple River Channel_3to1slope_FIGURES.gsz

Figure F-N2. Maple River Approach Channel Geometry (3H:1V)



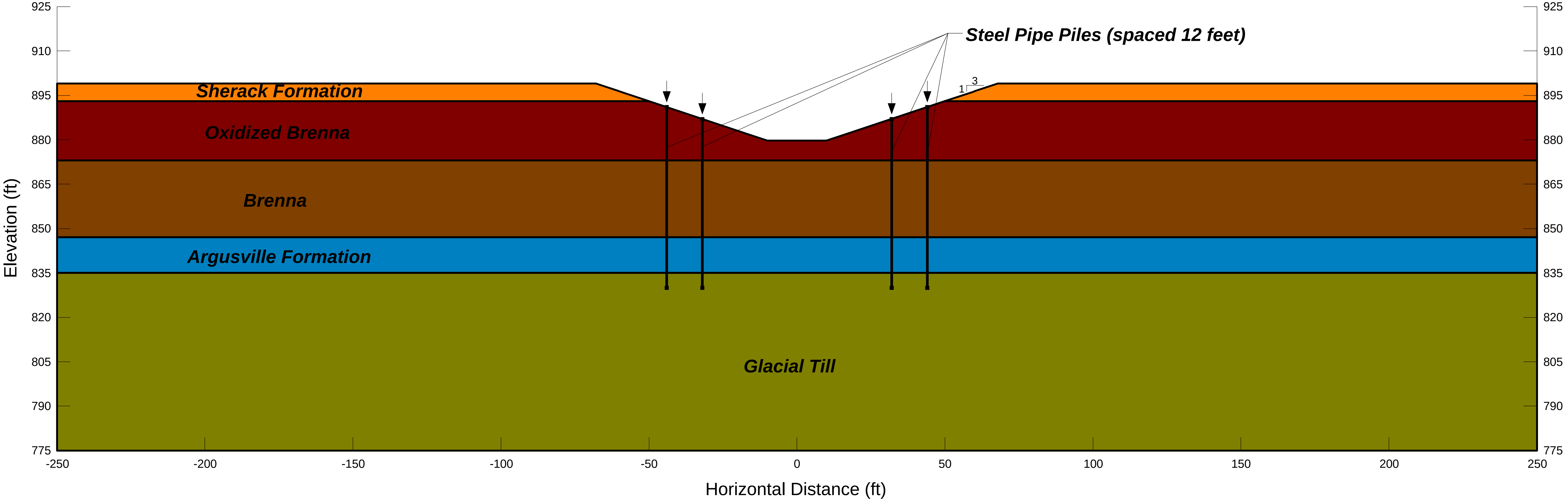
Fargo-Moorhead, Phase 4, Maple River Upstream Channel Stability Analysis
File Name: Maple River Channel_5to1slope_FIGURES.gsz

Figure F-N3. Maple River Approach Channel Geometry (5H:1V)
Option 1: Flatten Slopes



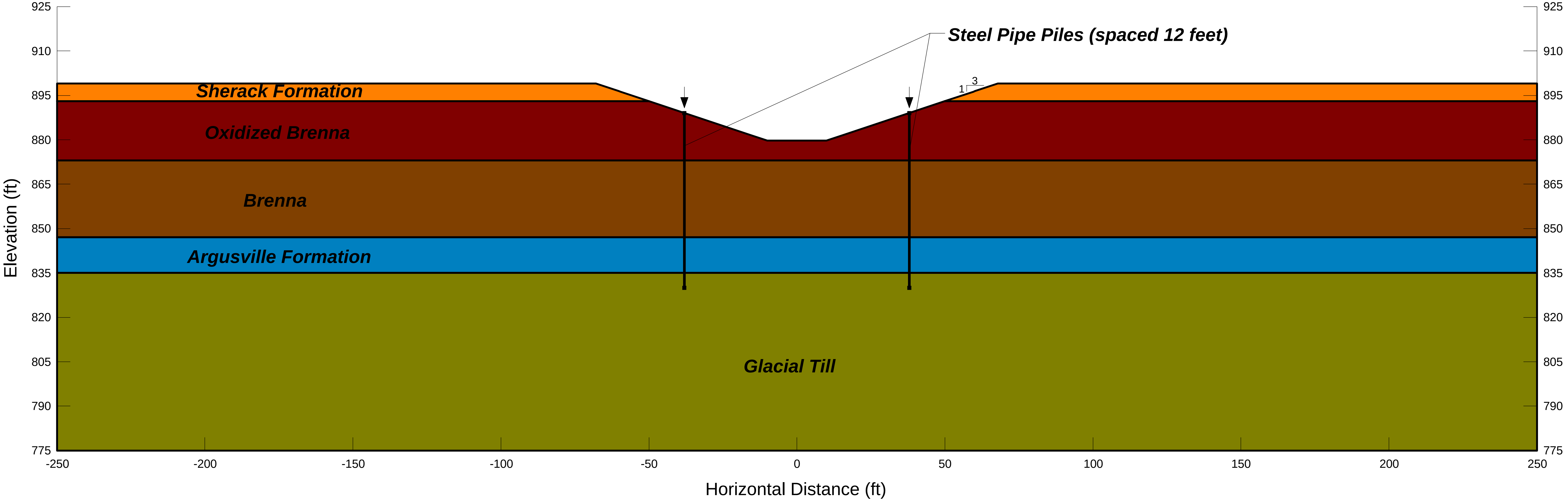
Fargo-Moorhead, Phase 4, Maple River Upstream Channel Stability Analysis
File Name: Maple River Channel_3to1slope_FIGURES.gsz

Figure F-N4. Maple River Approach Channel Geometry (3H:1V) Option 2: Two Rows of Piles



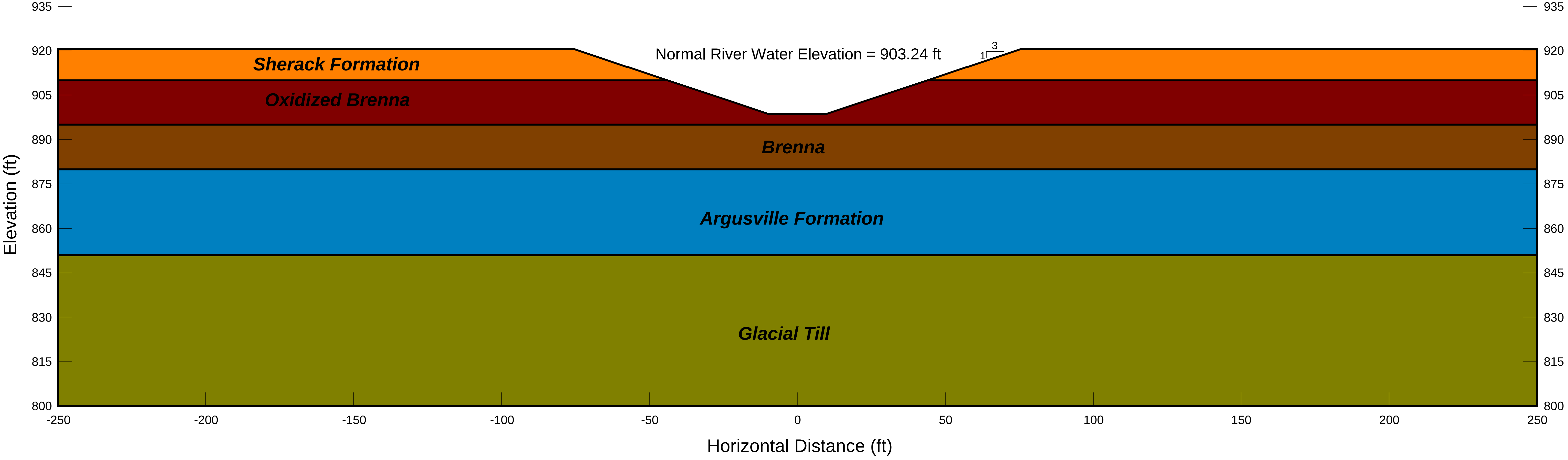
Fargo-Moorhead, Phase 4, Maple River Upstream Channel Stability Analysis
File Name: Maple River Channel_3to1slope_FIGURES.gsz

Figure F-N5. Maple River Approach Channel Geometry (3H:1V)
Option 3: One Row of Piles



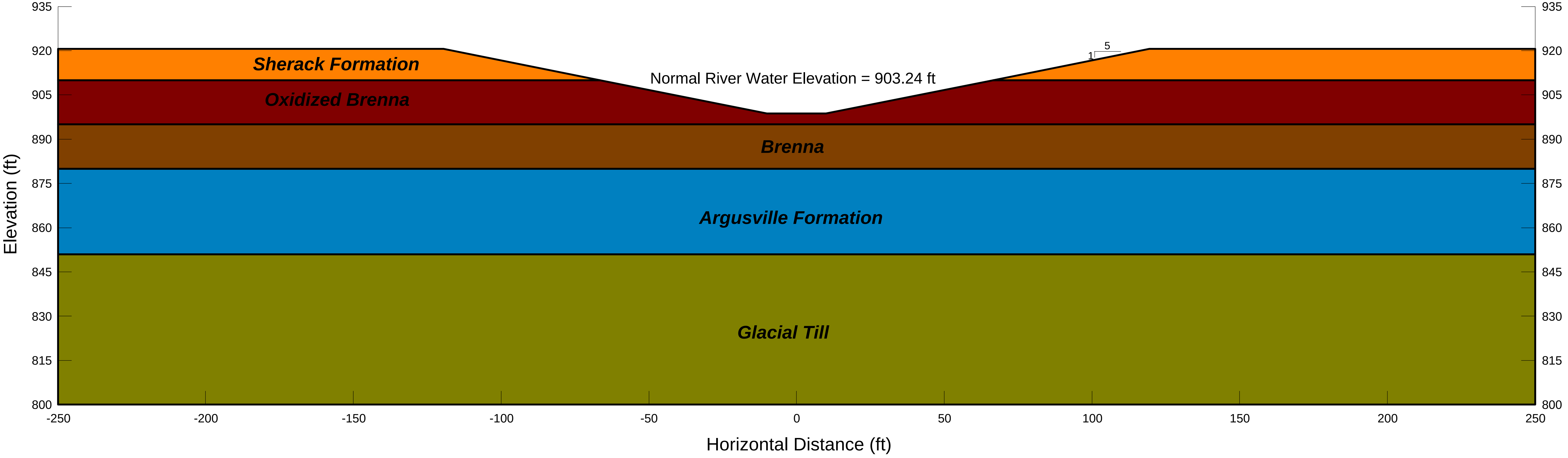
Fargo-Moorhead, Phase 4, Sheyenne River Upstream Channel Stability Analysis
File Name: Sheyenne River Channel_3to1_FIGURES.gsz

Figure F-N6. Sheyenne River Approach Channel Geometry (3H:1V)



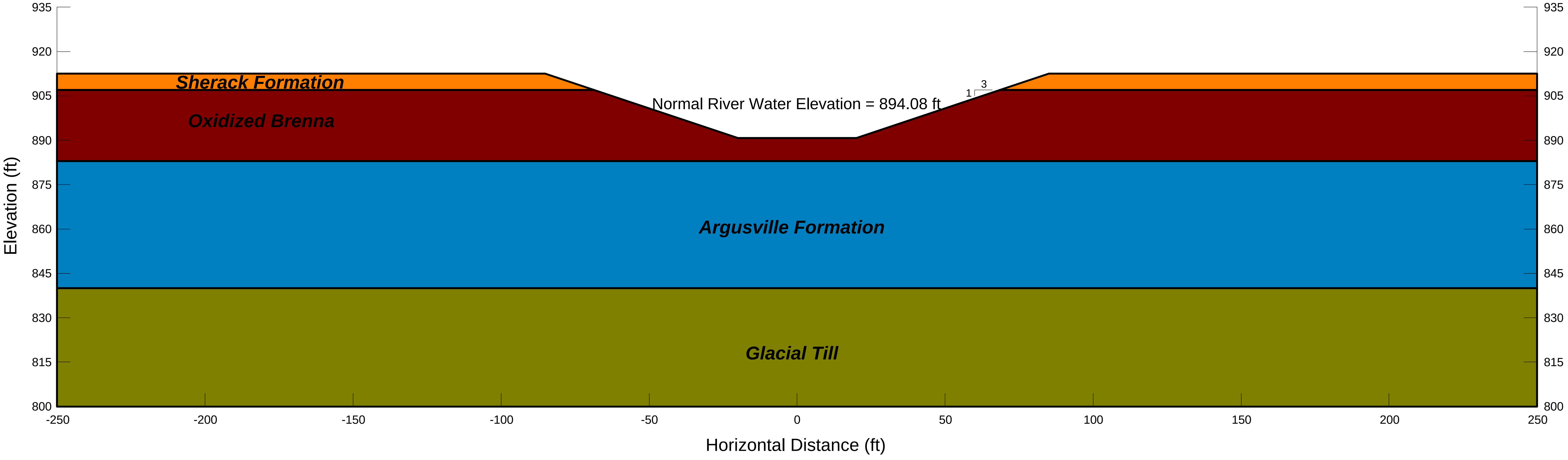
Fargo-Moorhead, Phase 4, Sheyenne River Upstream Channel Stability Analysis
File Name: Sheyenne River Channel_5to1_FIGURES.gsz

Figure F-N7. Sheyenne River Approach Channel Geometry (5H:1V)



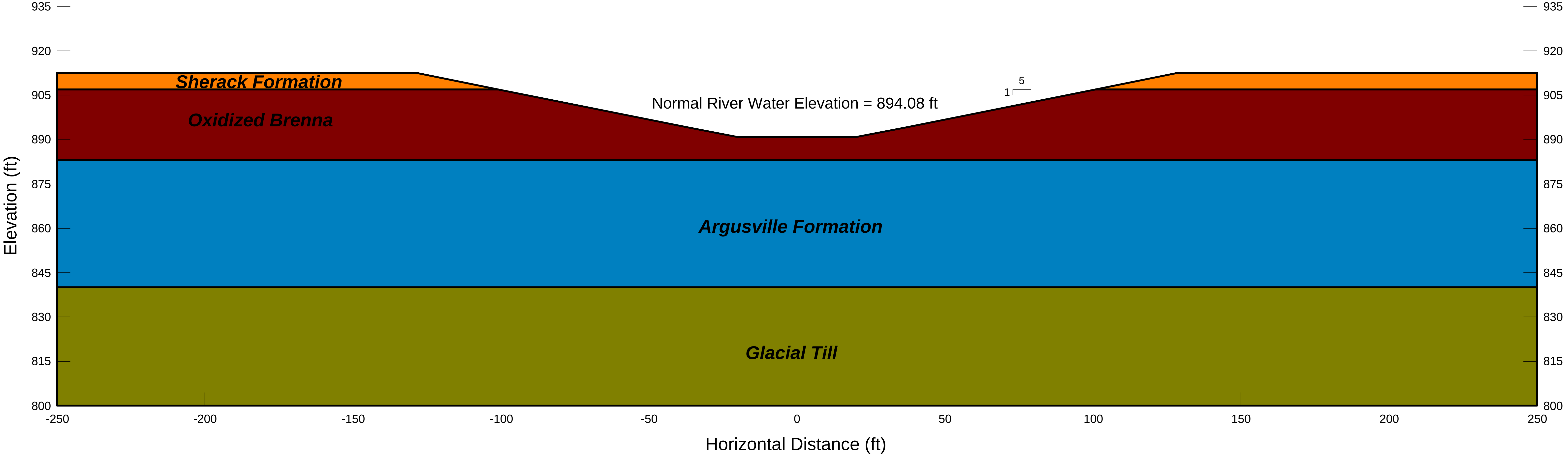
Fargo-Moorhead, Phase 4, Wild Rice River Upstream Channel Stability Analysis
File Name: Wild Rice River Channel_3to1_FIGURES.gsz

Figure F-N8. Wild Rice River Approach Channel Geometry (3H:1V)



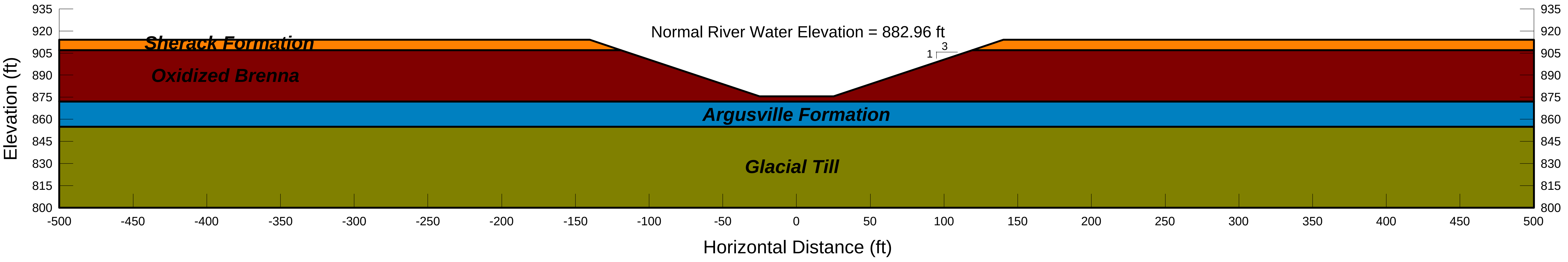
Fargo-Moorhead, Phase 4, Wild Rice River Upstream Channel
Stability Analysis
File Name: Wild Rice River Channel_5to1_FIGURES.gsz

Figure F-N9. Wild Rice River Approach Channel Geometry (5H:1V)



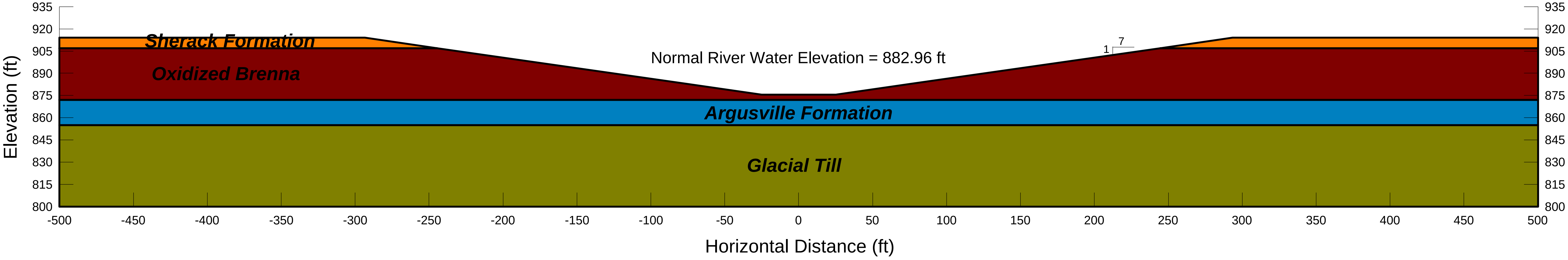
Fargo-Moorhead, Phase 4, Red River Upstream Channel
Stability Analysis
File Name: Red River Channel_3to1_FIGURES.gsz

Figure F-N10. Red River Approach Channel Geometry (3H:1V)



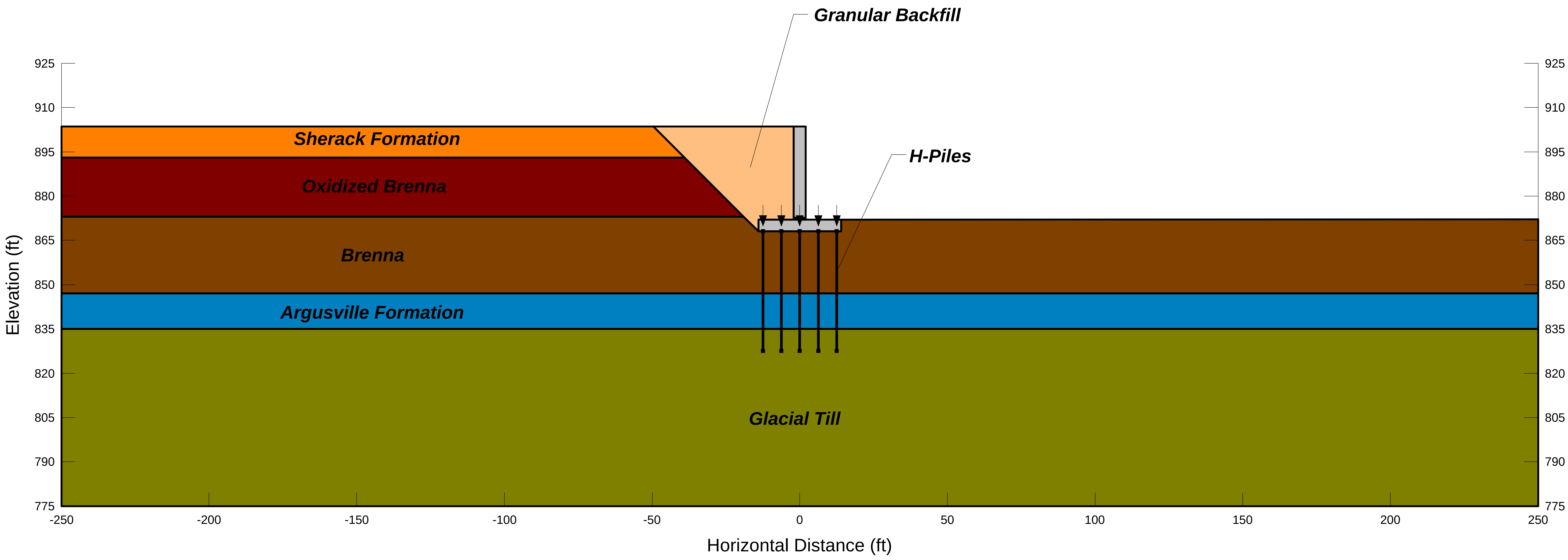
Fargo-Moorhead, Phase 4, Red River Upstream Channel Stability Analysis
File Name: Red River Channel_7to1_FIGURES.gsz

Figure F-N11. Red River Approach Channel Geometry (7H:1V)



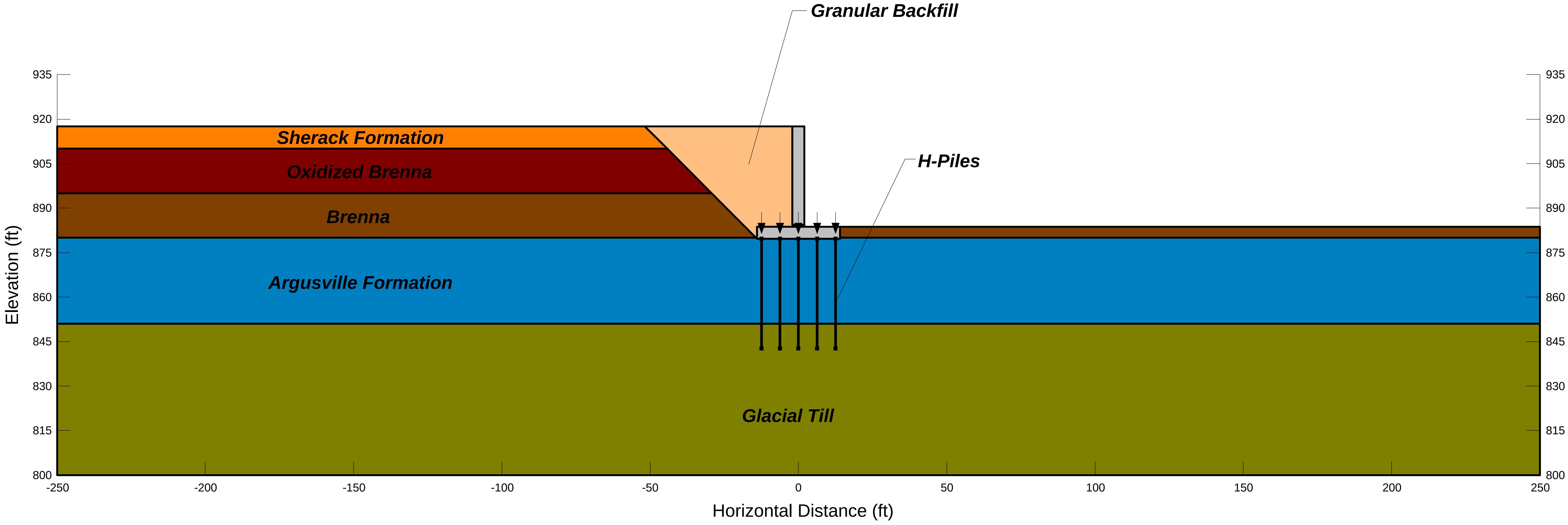
Fargo-Moorhead, Phase 4, Maple River Wall
Stability Analysis
File Name: Maple River Wall_FIGURES.gsz

Figure F-N12. Maple River Hydraulic Structure Global Stability Geometry



Fargo-Moorhead, Phase 4, Sheyenne River Wall
Stability Analysis
File Name: Sheyenne River Wall.gsz

Figure F-N13. Sheyenne River Hydraulic Structure Global Stability Geometry



Levee Storage Area 1

Type 1

2/2/2011

Levee Fill

8 ft Sherack

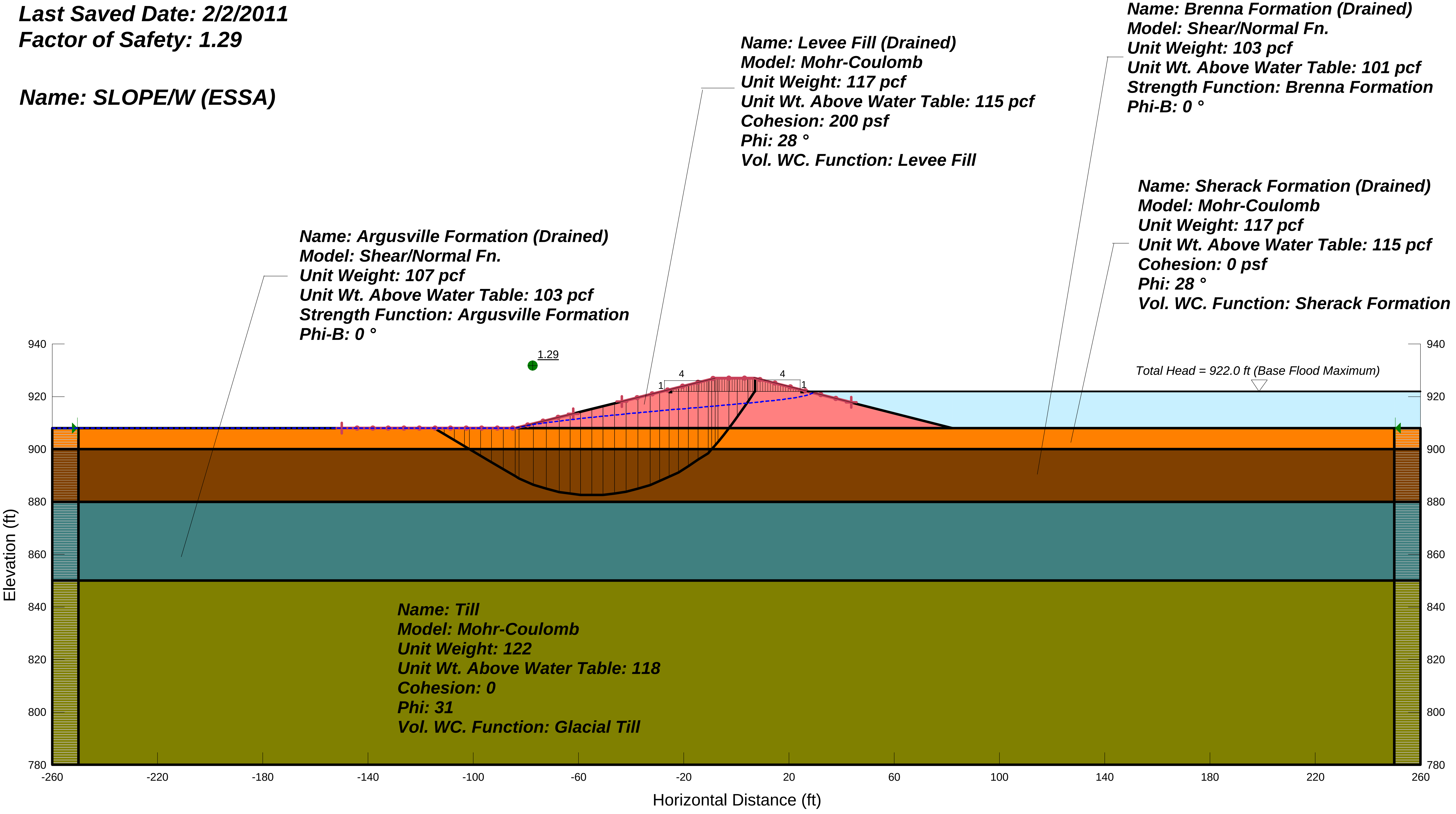
20 ft Brenna

30 ft Argusville

Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.29

Name: SLOPE/W (ESSA)



Name: Levee Fill (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 200 psf
Phi: 28 °
Vol. WC. Function: Levee Fill

Name: Brenna Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna Formation
Phi-B: 0 °

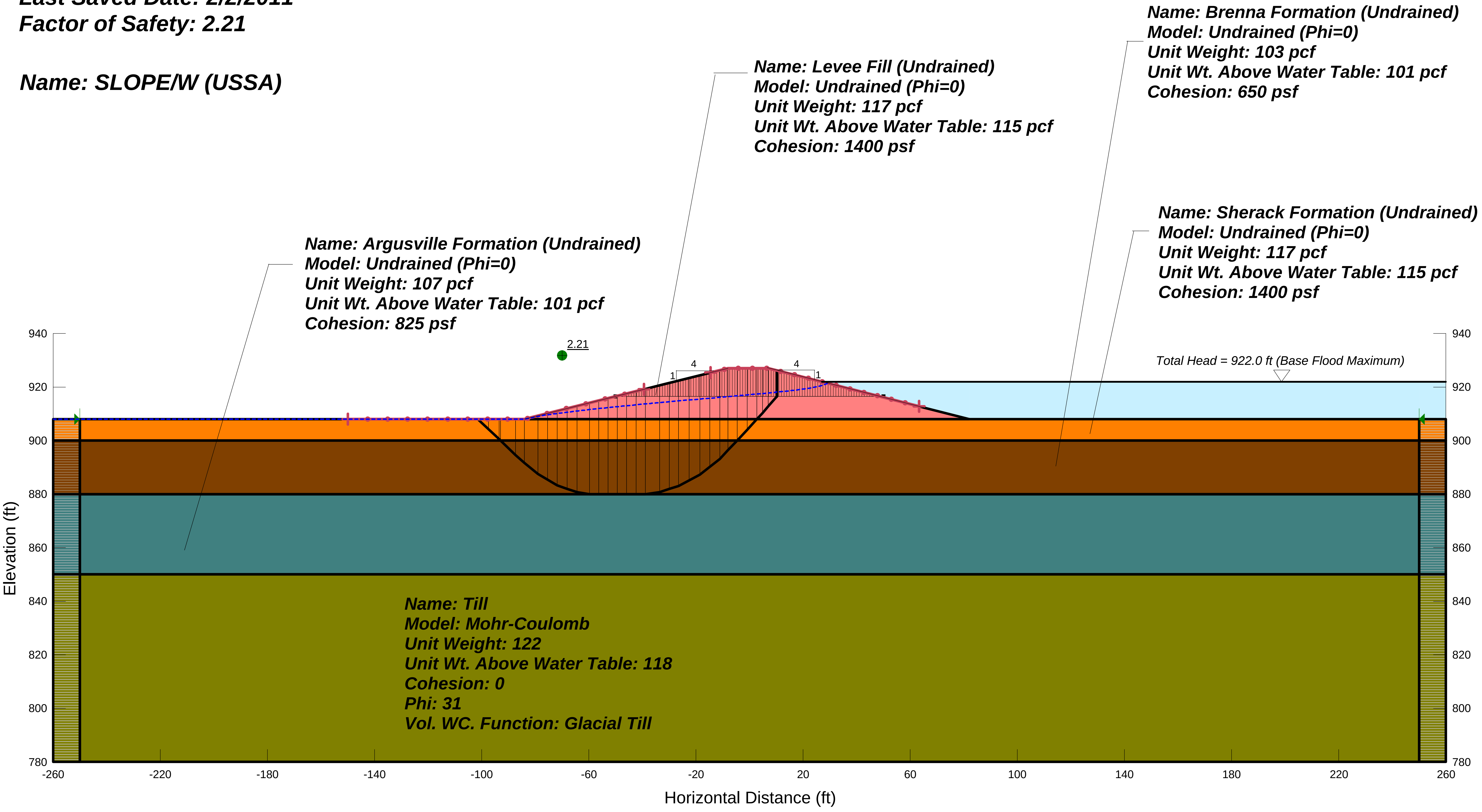
Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack Formation

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville Formation
Phi-B: 0 °

Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 2.21

Name: SLOPE/W (USSA)



Name: Brenna Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 650 psf

Name: Levee Fill (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

Name: Sherack Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

Name: Argusville Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 825 psf

Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Levee Storage Area 1

Type 2

2/2/2011

Levee Fill

8 ft Sherack

5 ft Oxidized Brenna

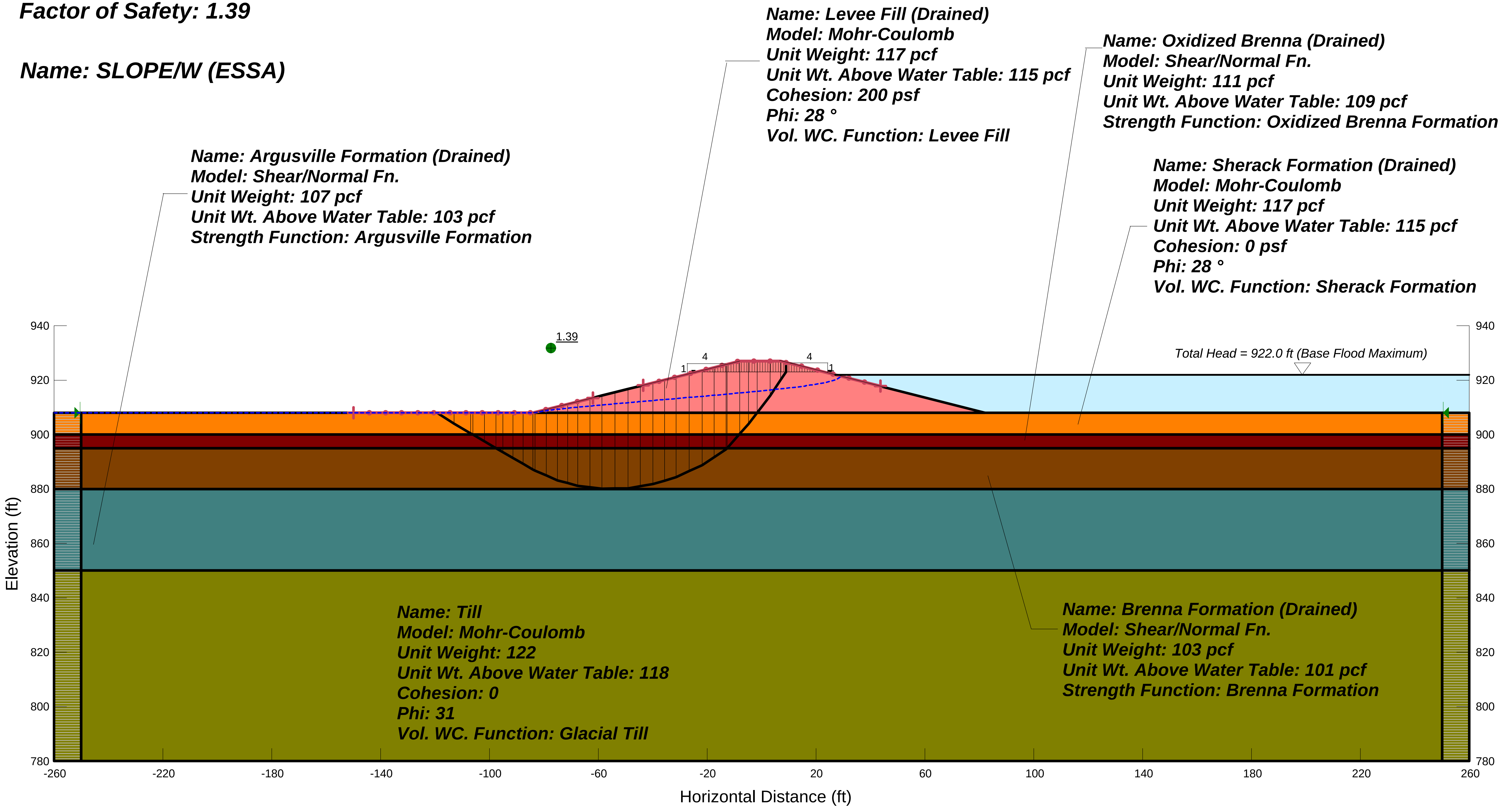
15 ft Brenna

30 ft Argusville

Till

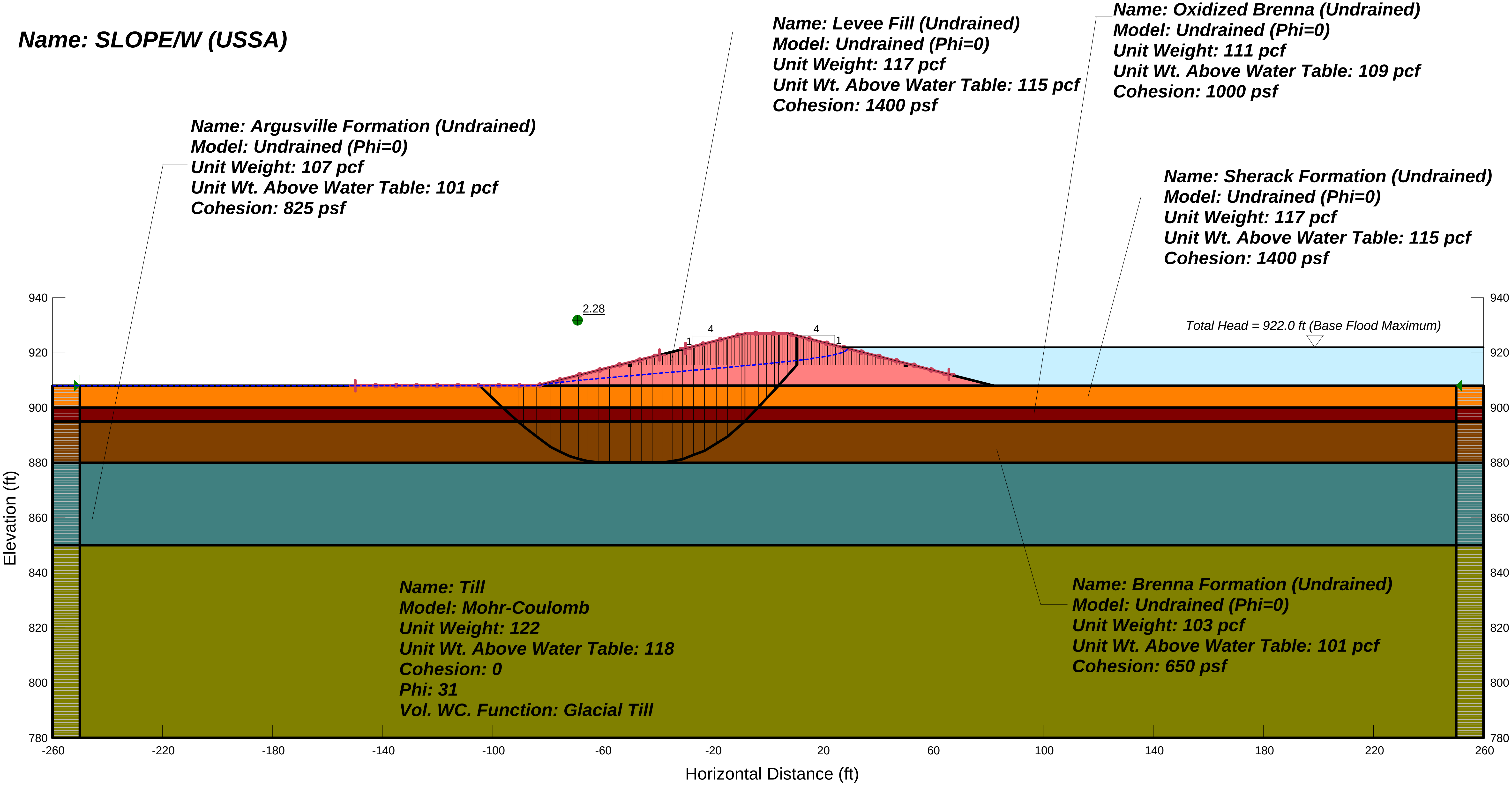
Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_2.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.39

Name: SLOPE/W (ESSA)



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_2.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 2.28

Name: SLOPE/W (USSA)



Name: Levee Fill (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

Name: Oxidized Brenna (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Cohesion: 1000 psf

Name: Argusville Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 825 psf

Name: Sherack Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Name: Brenna Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 650 psf

Levee Storage Area 1

Type 3

2/2/2011

Levee Fill

8 ft Sherack

20 ft Oxidized Brenna

30 ft Argusville

Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_3.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.67

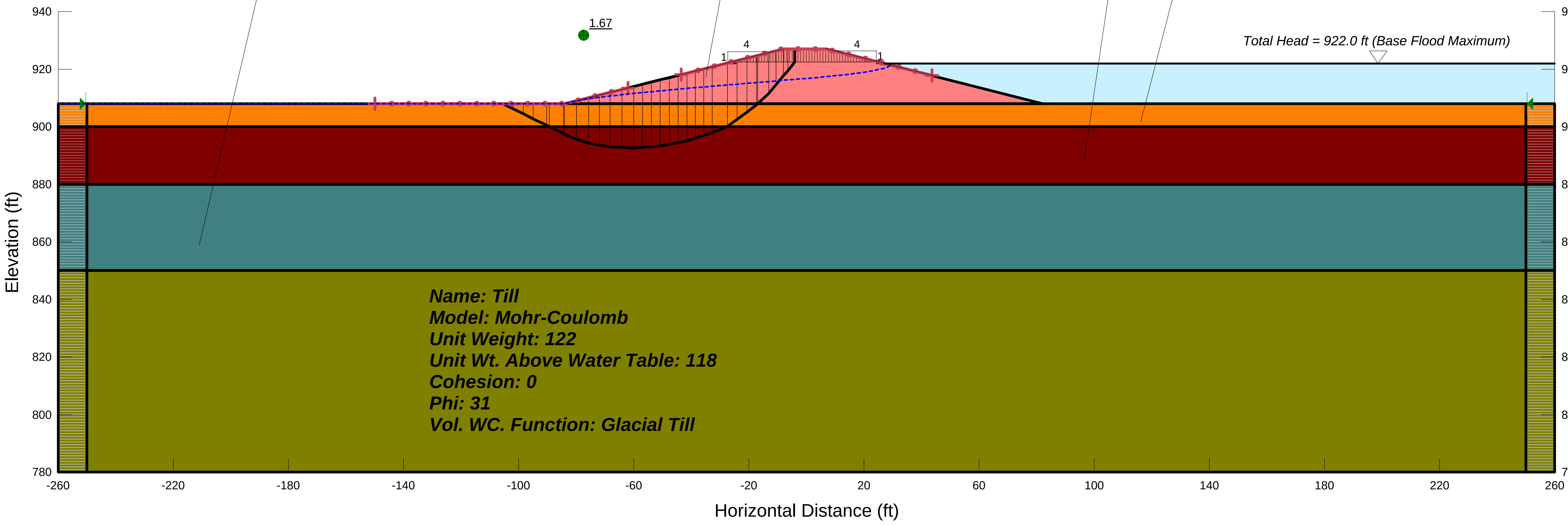
Name: SLOPE/W (ESSA)

Name: Levee Fill (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 200 psf
Phi: 28 °
Vol. WC. Function: Levee Fill

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: Oxidized Brenna Formation

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville Formation

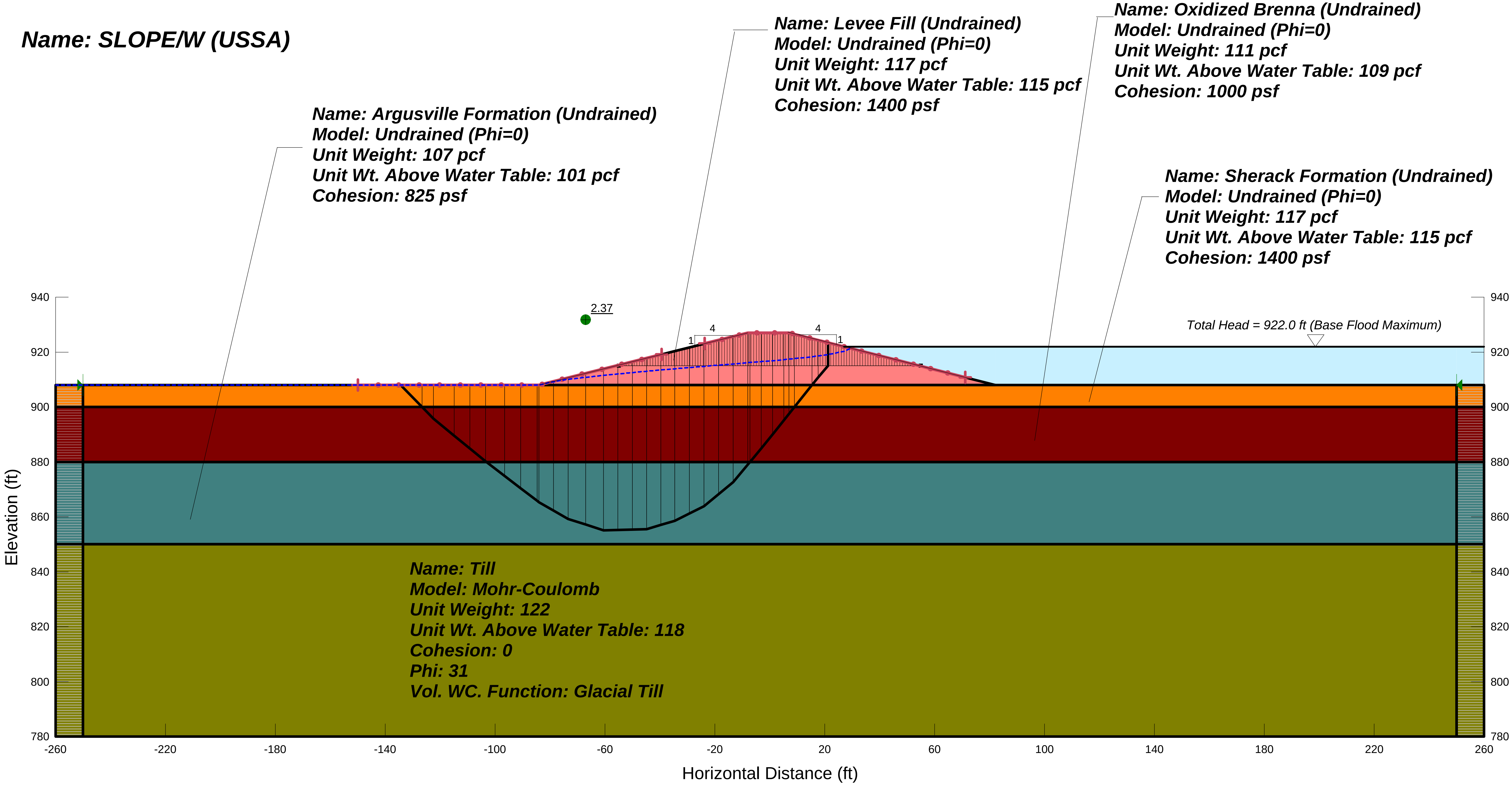
Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack Formation



Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_3.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 2.37

Name: SLOPE/W (USSA)


Name: Argusville Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 825 psf

Name: Levee Fill (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

Name: Oxidized Brenna (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Cohesion: 1000 psf

Name: Sherack Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Total Head = 922.0 ft (Base Flood Maximum)

Levee Storage Area 1

Type 4

2/2/2011

Levee Fill

8 ft Sherack

10 ft Brenna

40 ft Argusville

Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.33

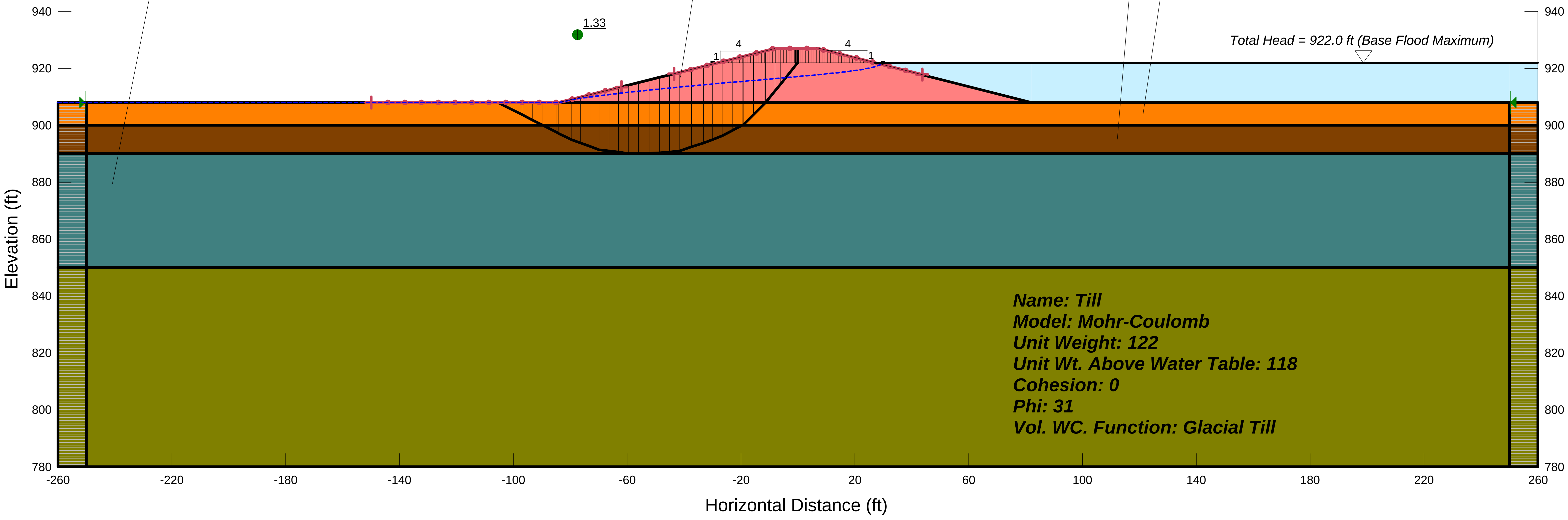
Name: SLOPE/W (ESSA)

Name: Levee Fill (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 200 psf
Phi: 28 °
Vol. WC. Function: Levee Fill

Name: Brenna Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna Formation

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville Formation

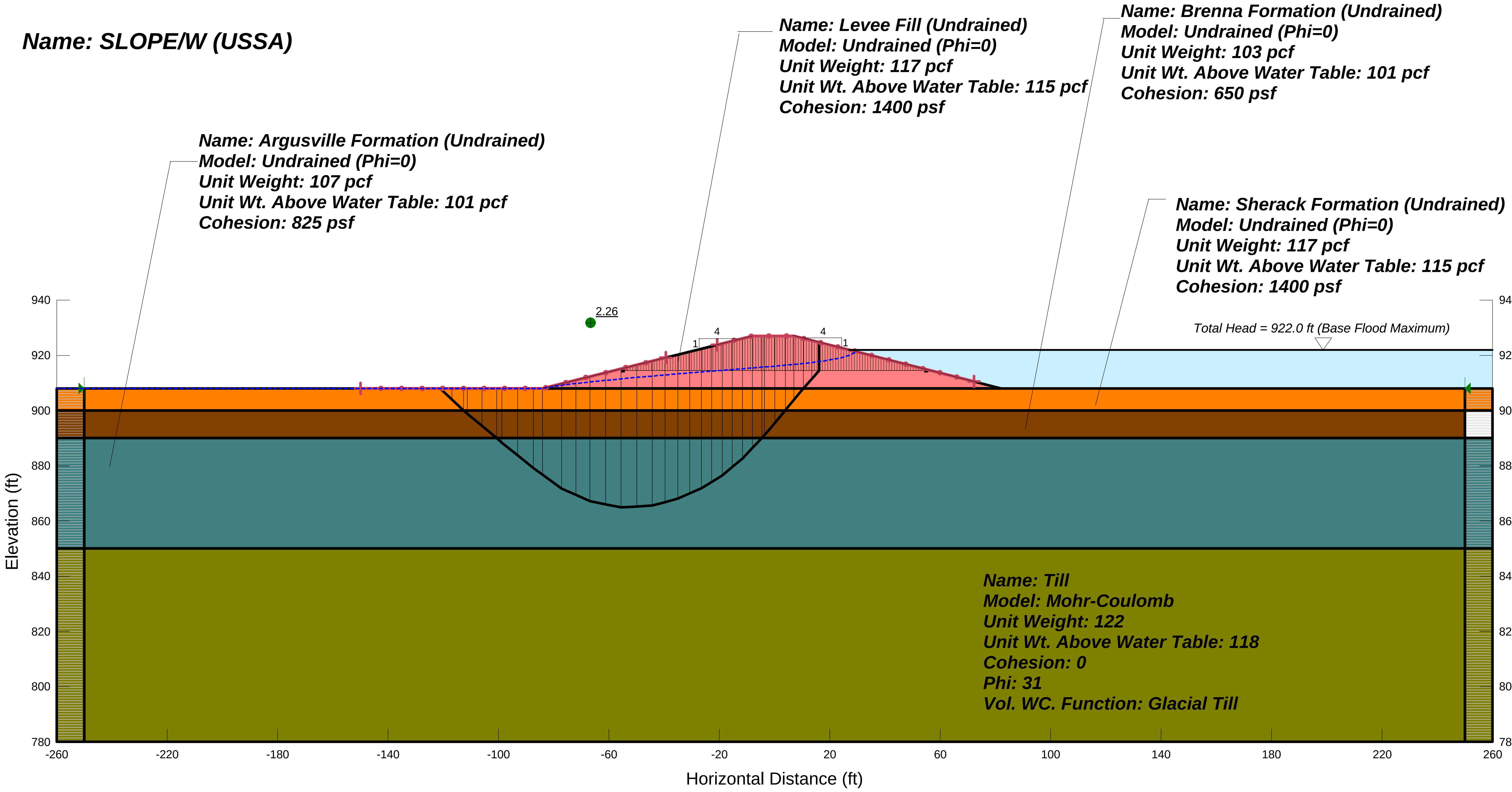
Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack Formation



Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 1/25/2011
Factor of Safety: 2.26

Name: SLOPE/W (USSA)

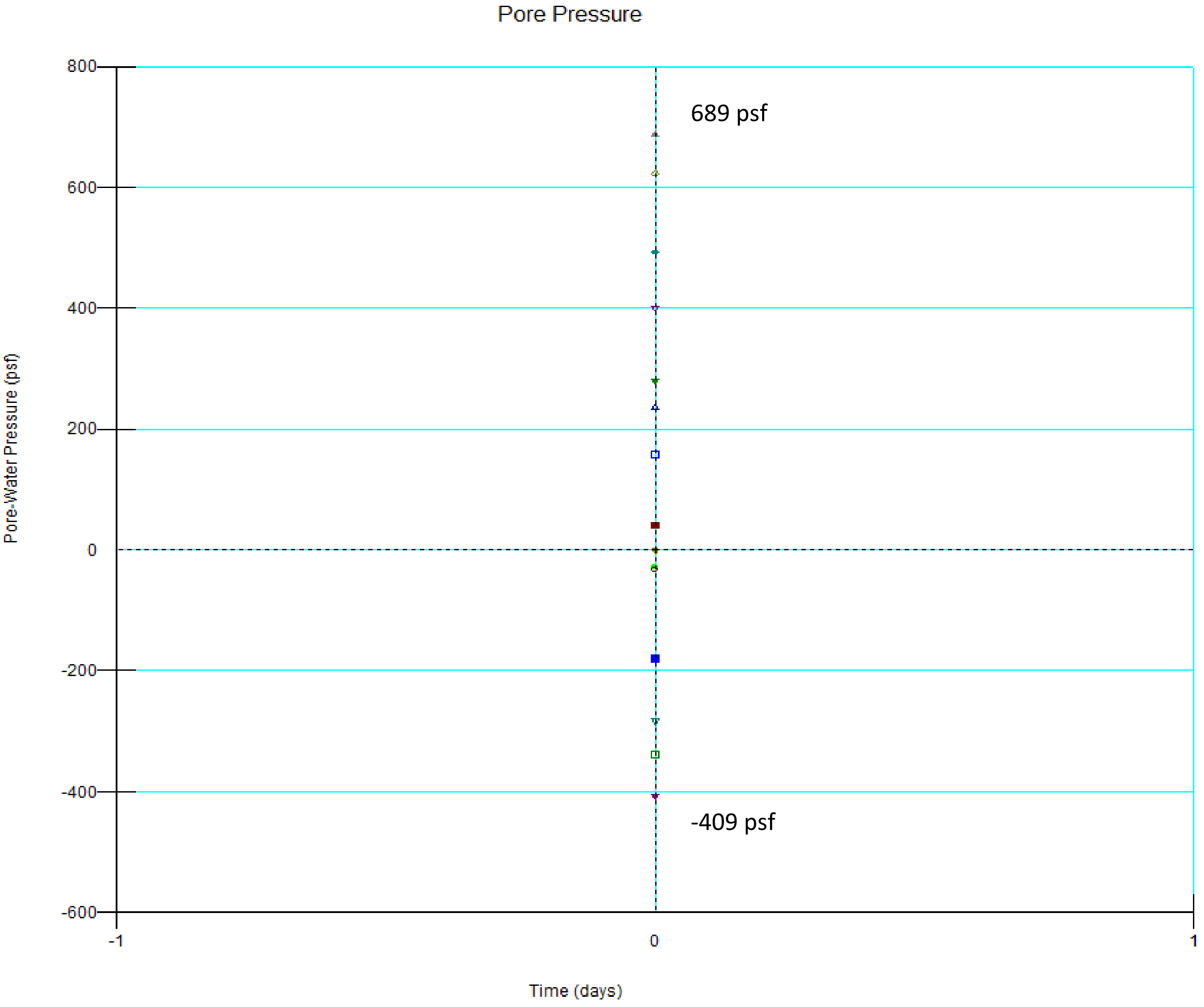


Type 1 (FS = 1.29)

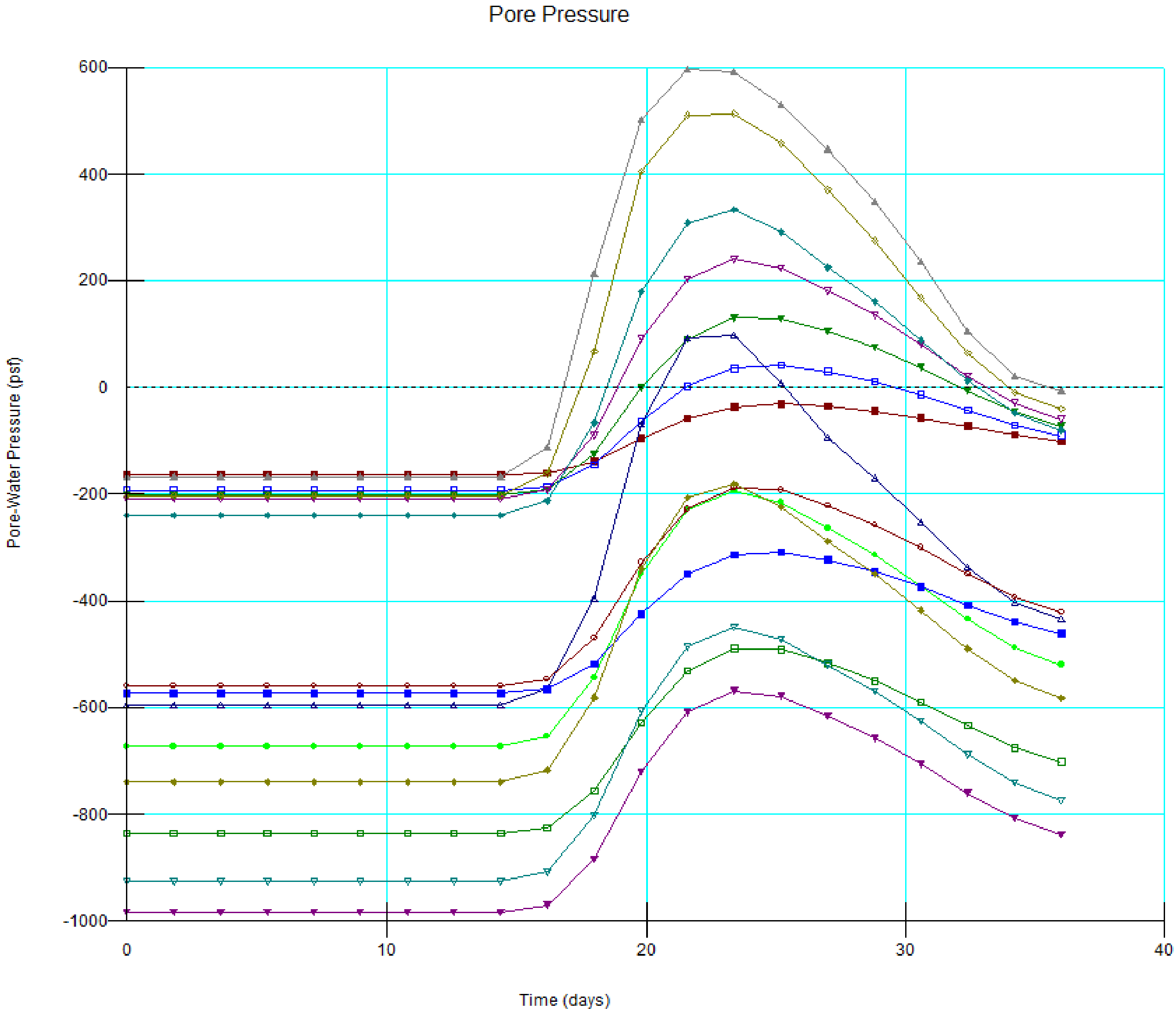
and

Type 4 (FS = 1.33)

Ideal Steady State Condition with Water Elevation at 922ft

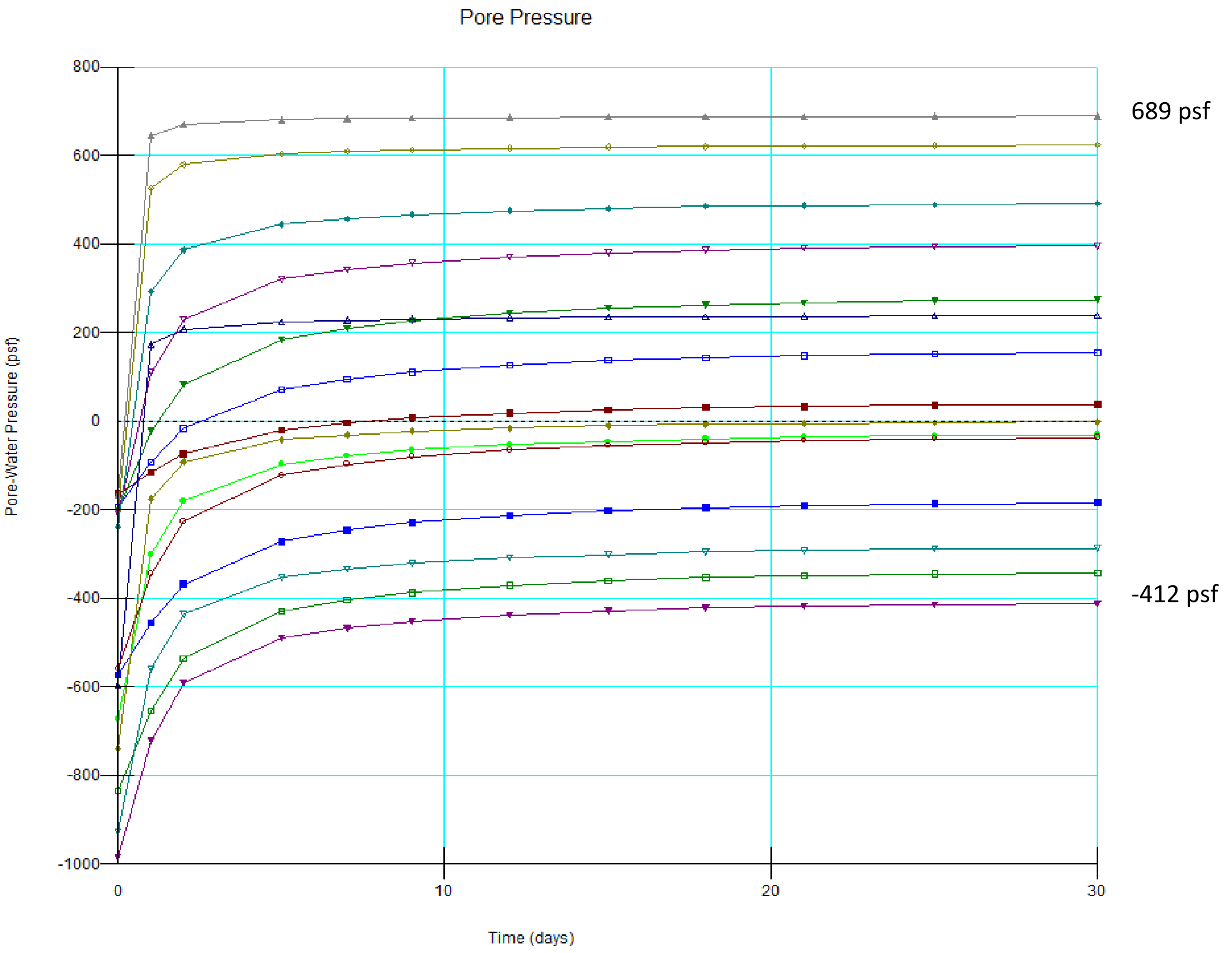


Hydrograph Conditions



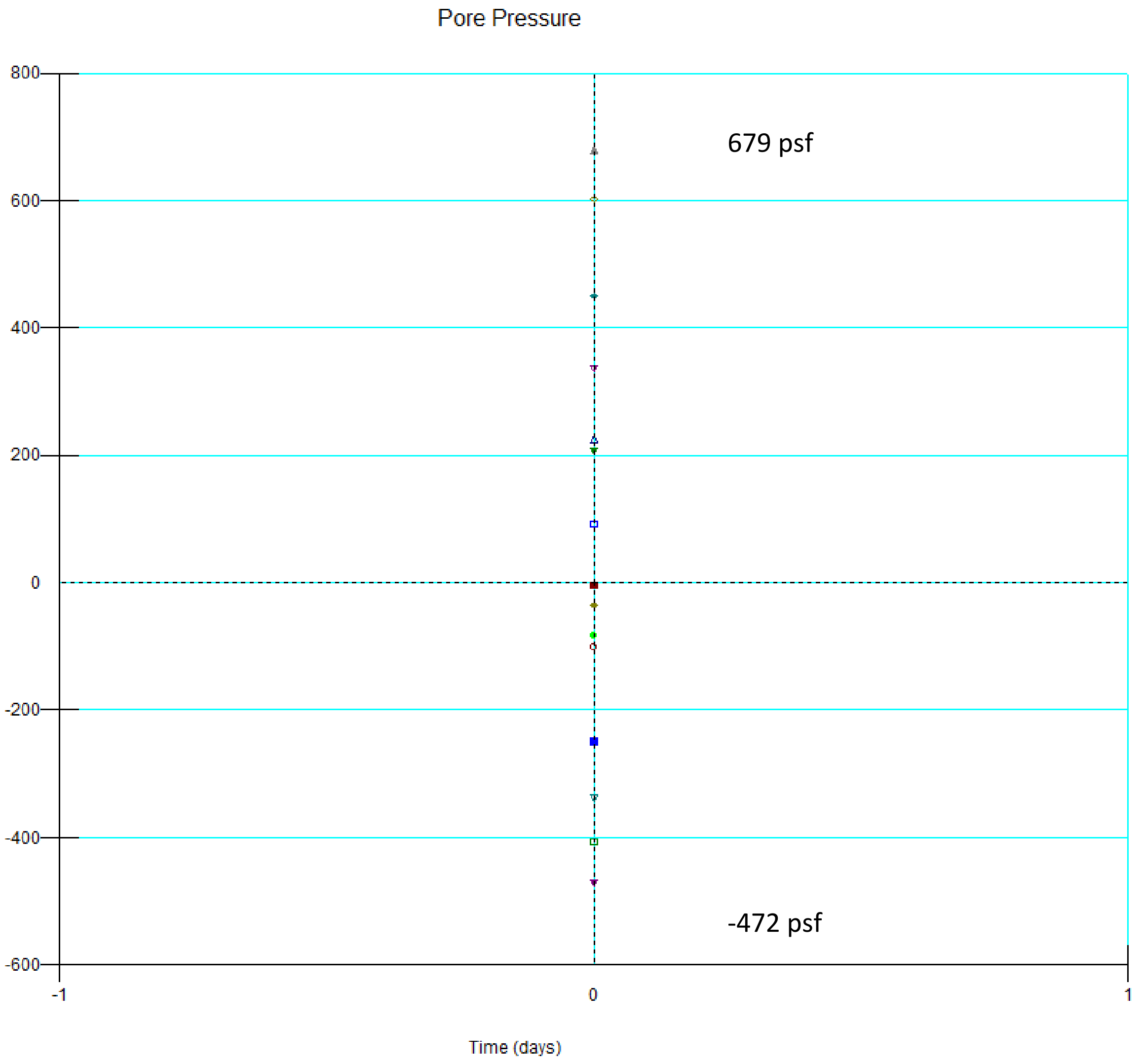
Steady State Condition in 30 Days

Embankment Fill = 18.6 ft/day **Sherack =** 93 ft/day **Brenna =** 9.3 ft/day **Argusville =** 9.3 ft/day **Till =** 1860 ft/day

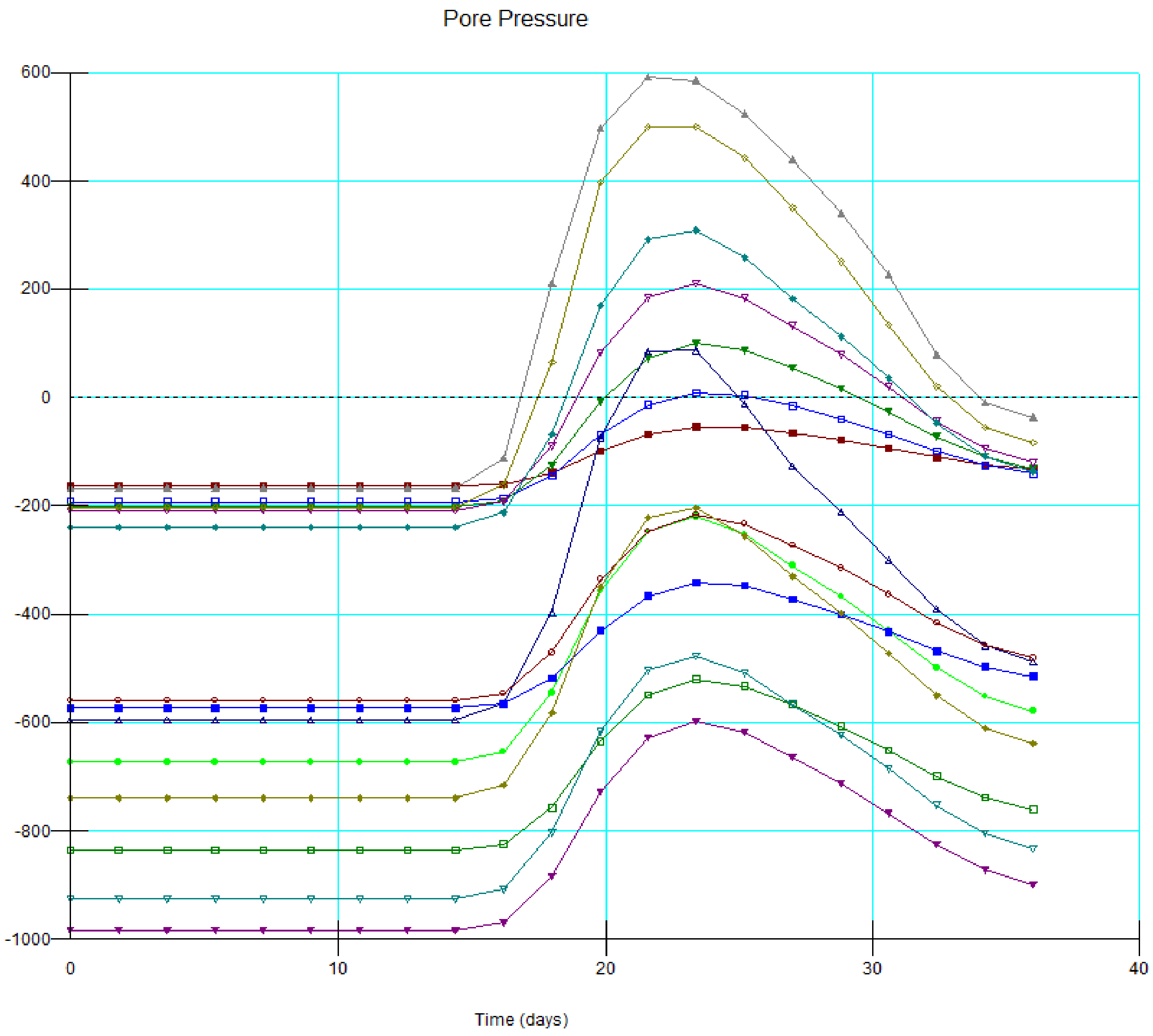


Type 2 (FS = 1.39)

Ideal Steady State Condition with Water Elevation at 922ft



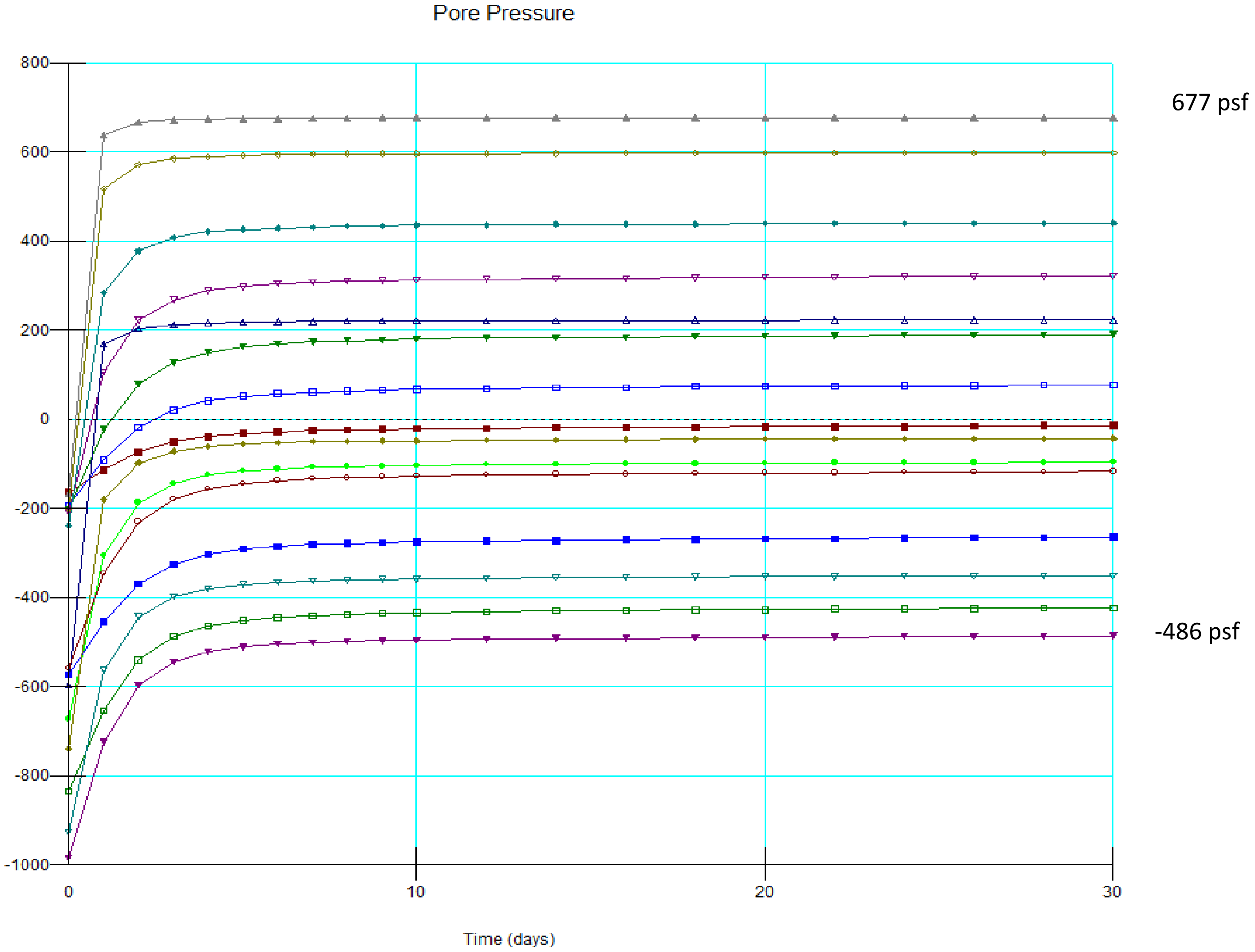
Hydrograph Conditions



Steady State Condition in 30 Days

Embankment Fill = 18.6 ft/day **Sherack =** 93 ft/day **Oxidized Brenna =** 46.5 ft/day **Brenna =** 9.3 ft/day

Argusville = 9.3 ft/day **Till =** 1860 ft/day



Levee Storage Area 1

Type 1

2/2/2011

Levee Fill

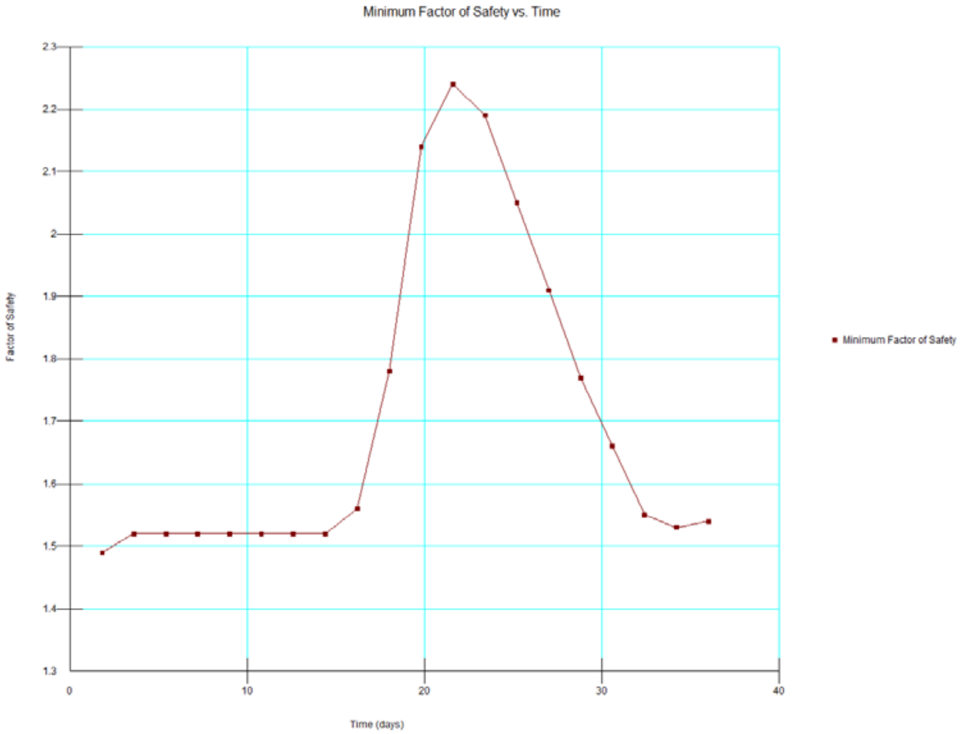
8 ft Sherack

20 ft Brenna

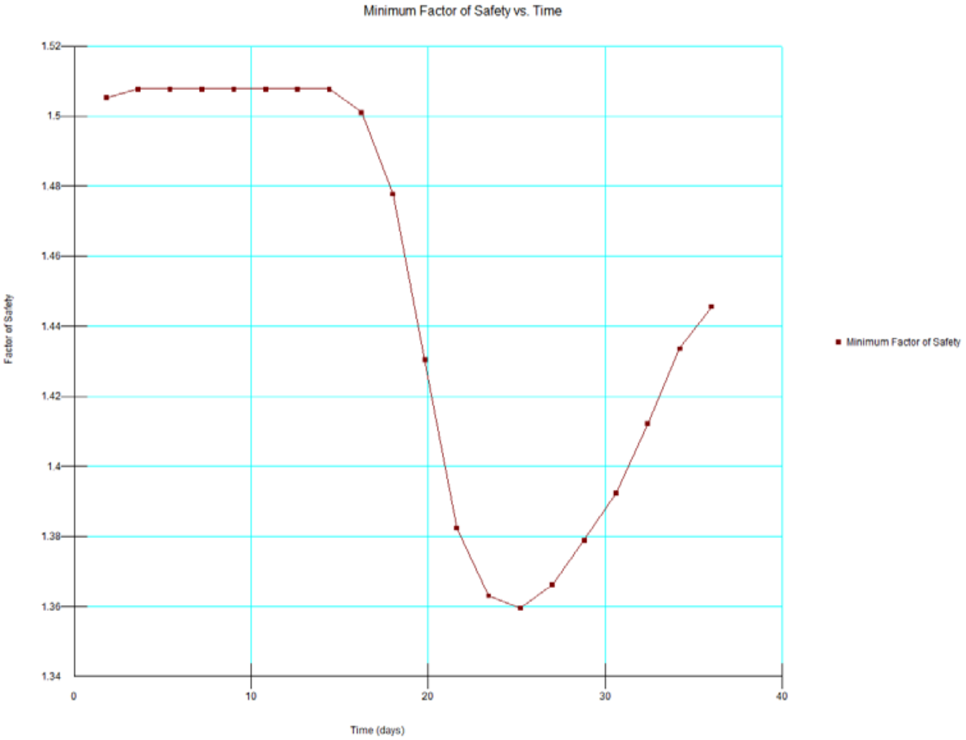
30 ft Argusville

Till

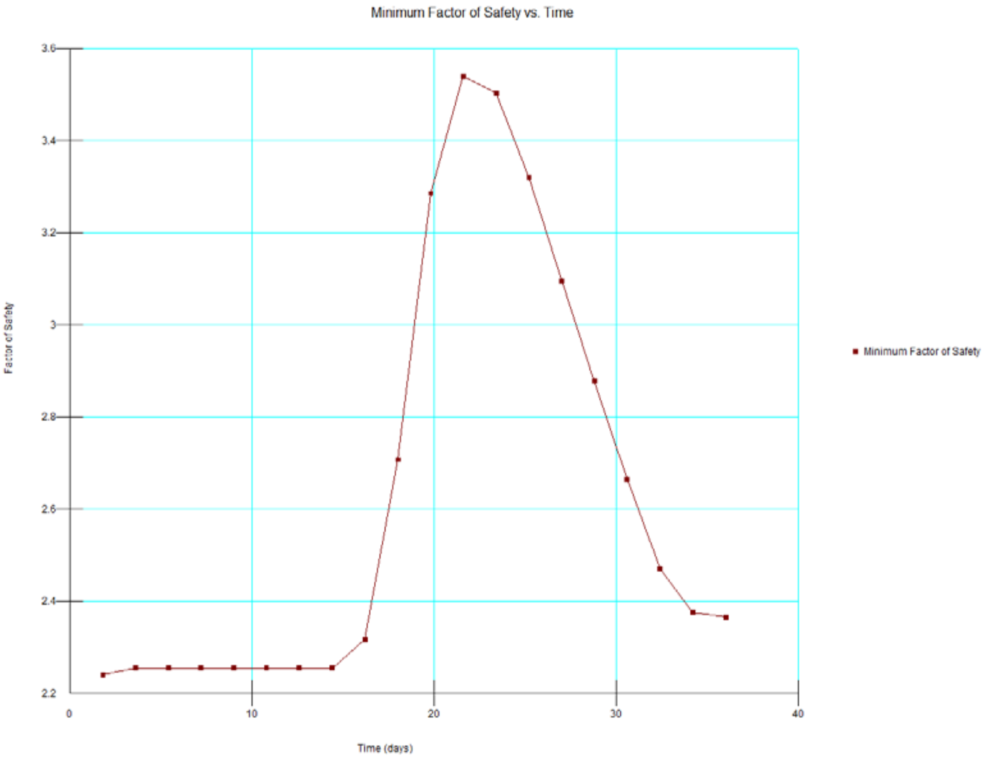
Factor of Safety for ESSA Transient Hydrograph (36 days) Left to Right



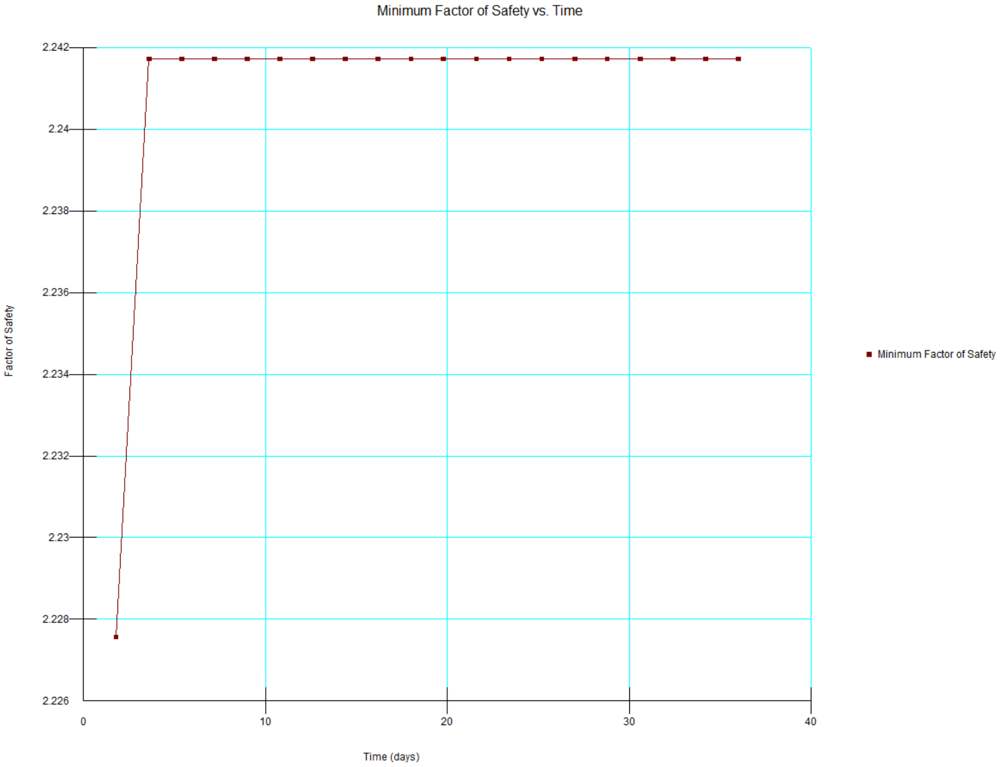
Factor of Safety for ESSA Transient Hydrograph (36 days) Right to Left



Factor of Safety for USSA Transient Hydrograph (36 days) Left to Right



Factor of Safety for USSA Transient Hydrograph (36 days) Right to Left



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.54

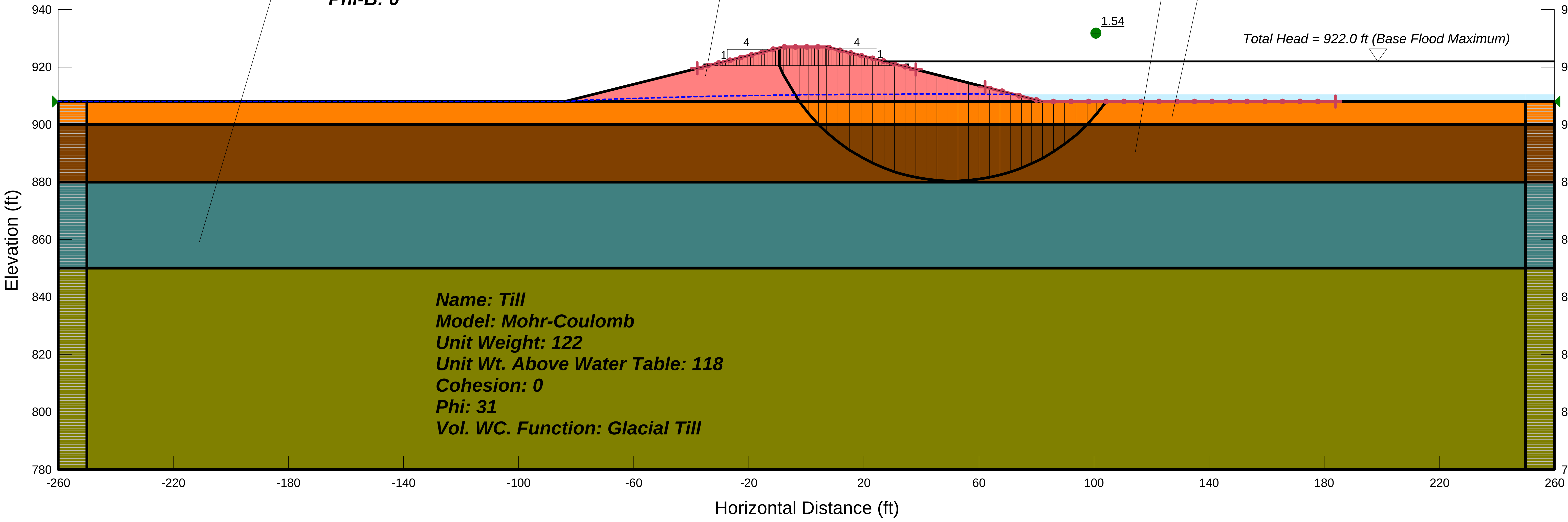
Name: ESSA Slope Stability Hydrograph L-R

Name: Levee Fill (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 200 psf
Phi: 28 °
Vol. WC. Function: Levee Fill

Name: Brenna Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna Formation
Phi-B: 0 °

Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack Formation

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville Formation
Phi-B: 0 °



Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.36

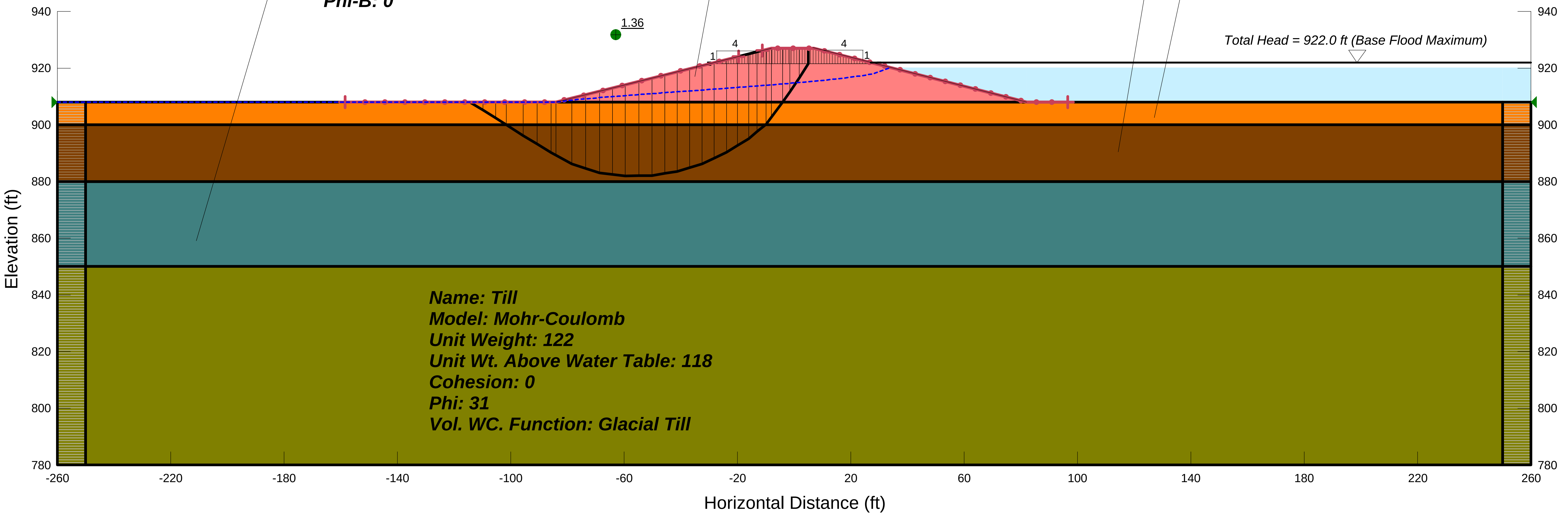
Name: ESSA Slope Stability Hydrograph R-L

Name: Brenna Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna Formation
Phi-B: 0 °

Name: Levee Fill (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 200 psf
Phi: 28 °
Vol. WC. Function: Levee Fill

Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack Formation

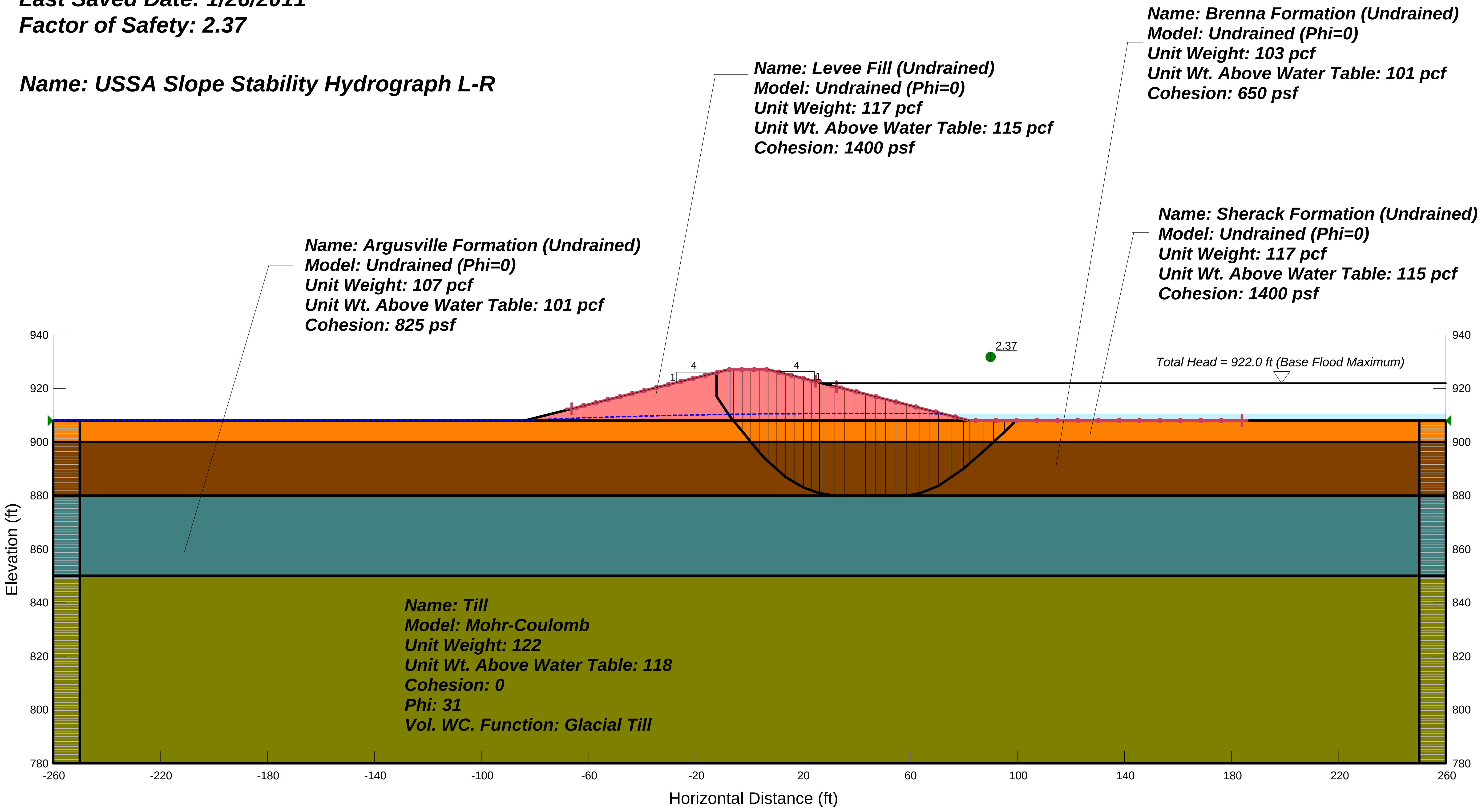
Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville Formation
Phi-B: 0 °



Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

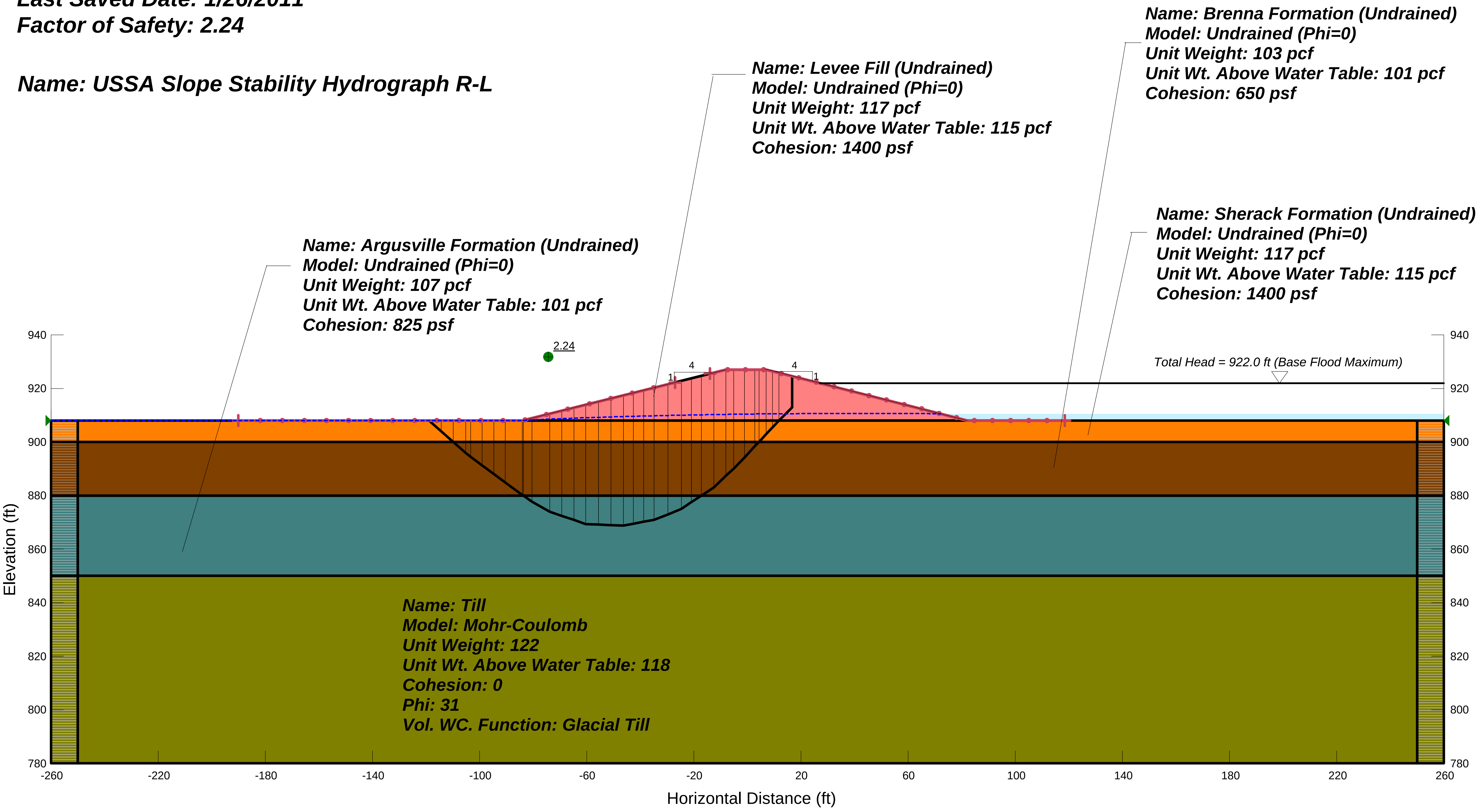
Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1.gsz
Last Saved Date: 1/26/2011
Factor of Safety: 2.37

Name: USSA Slope Stability Hydrograph L-R



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1.gsz
Last Saved Date: 1/26/2011
Factor of Safety: 2.24

Name: USSA Slope Stability Hydrograph R-L



Name: Brenna Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 650 psf

Name: Levee Fill (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

Name: Sherack Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

Name: Argusville Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 825 psf

Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Levee Storage Area 1

Type 2

2/2/2011

Levee Fill

8 ft Sherack

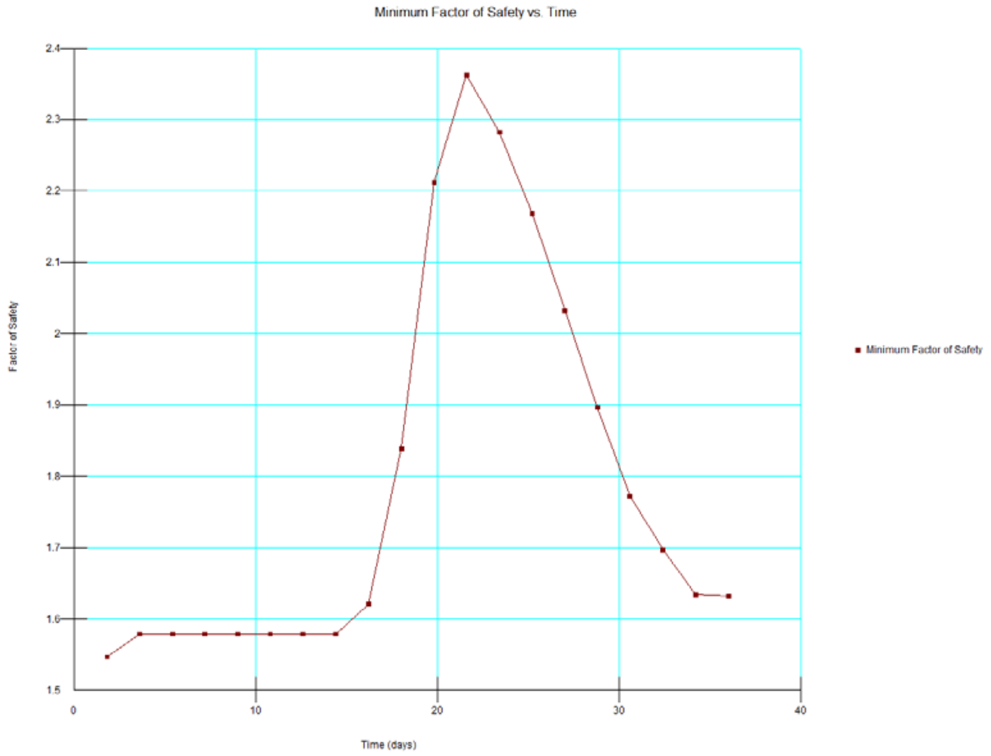
5 ft Oxidized Brenna

15 ft Brenna

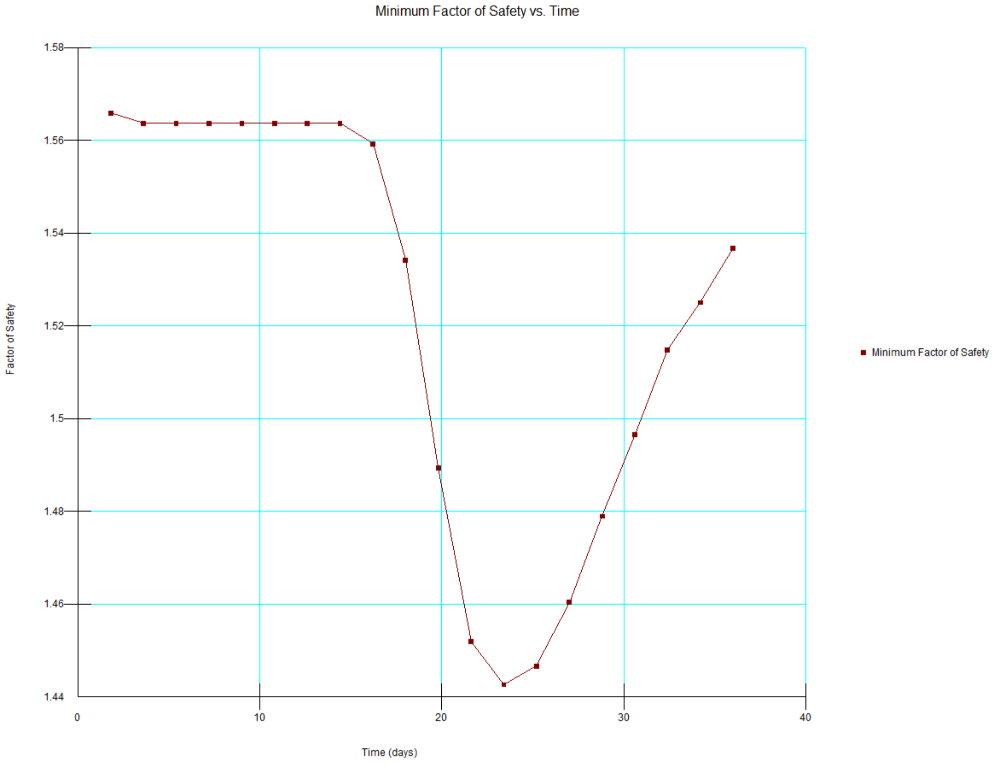
30 ft Argusville

Till

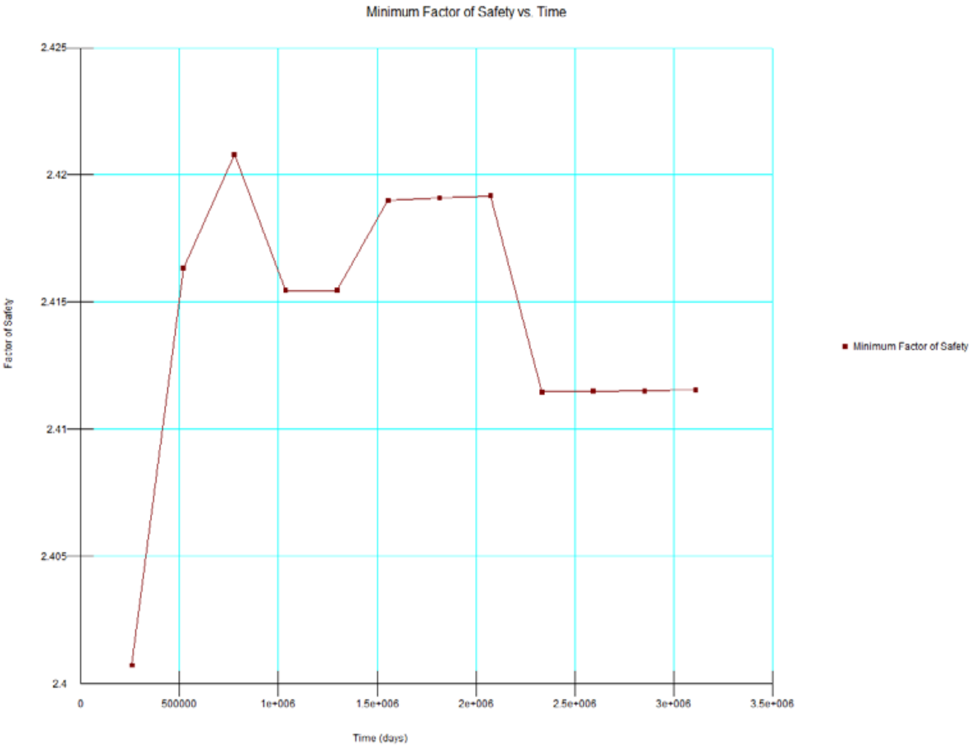
Factor of Safety for ESSA Transient Hydrograph (36 days) Left to Right



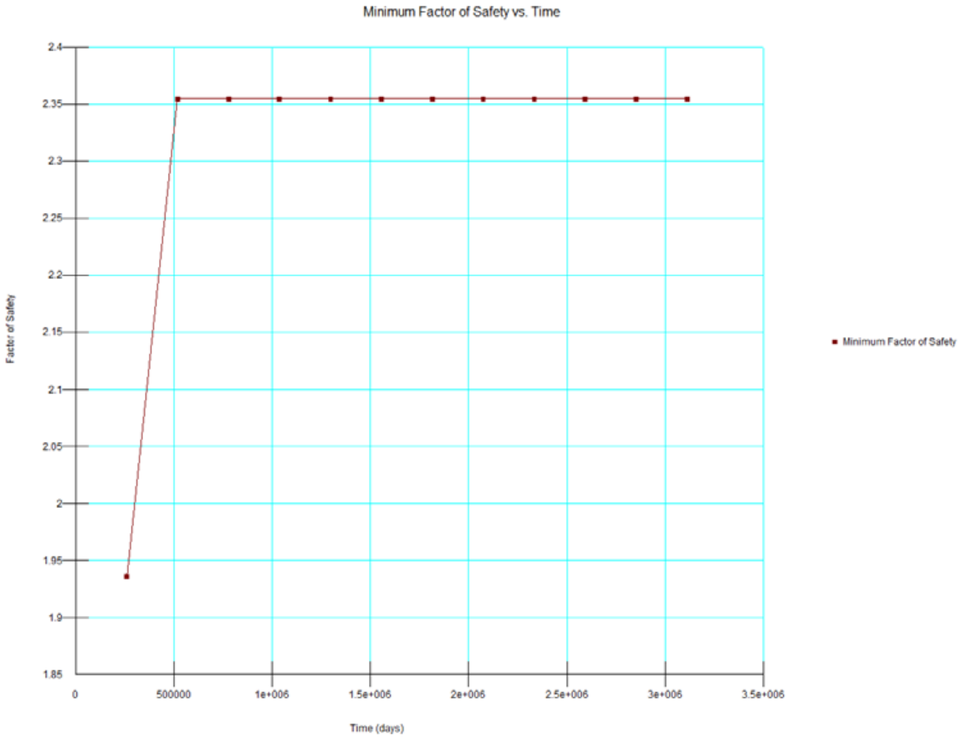
Factor of Safety for ESSA Transient Hydrograph (36 days) Right to Left



Factor of Safety for USSA Transient Hydrograph (36 days) Left to Right

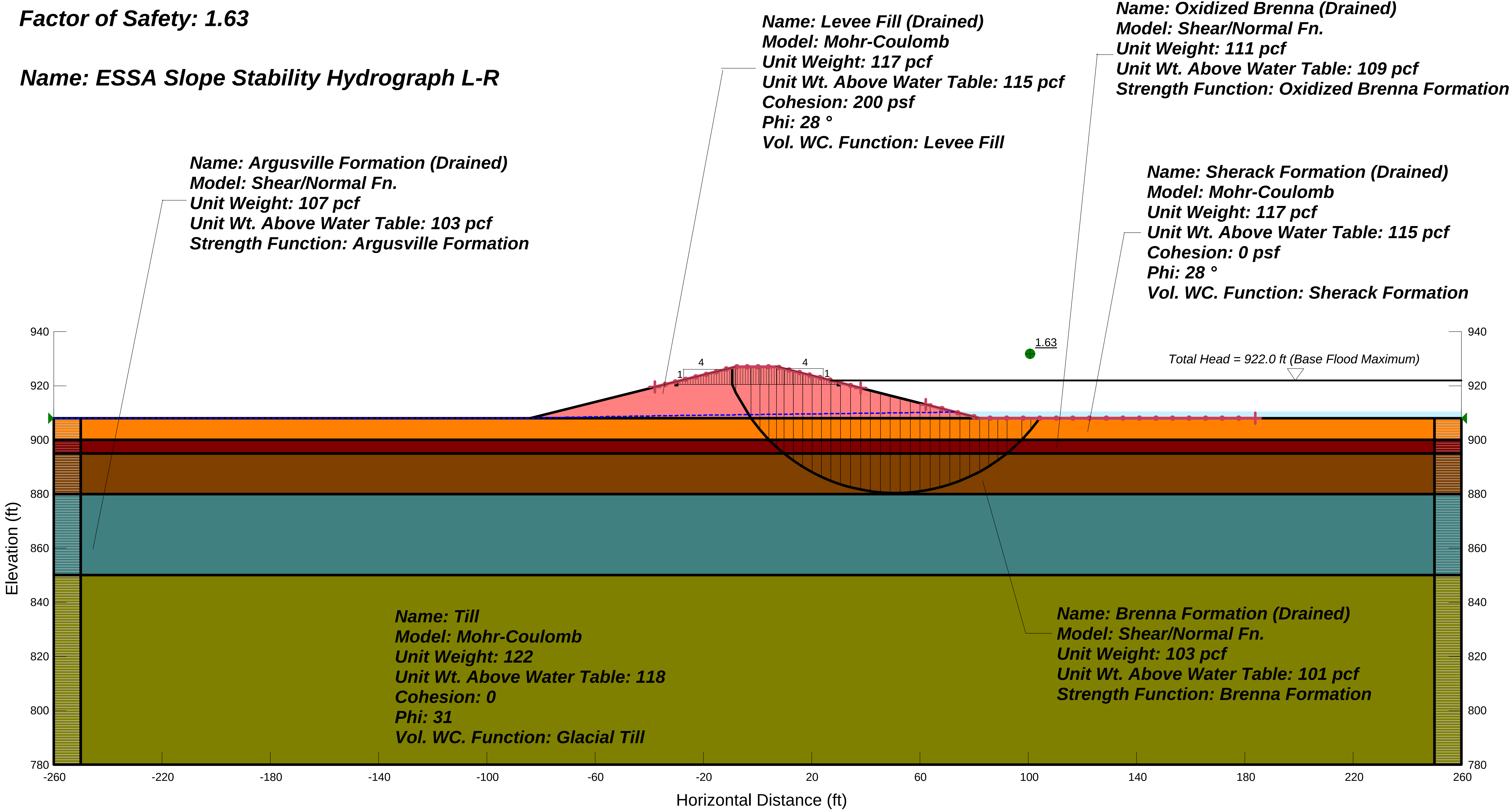


Factor of Safety for USSA Transient Hydrograph (36 days) Right to Left



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_2.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.63

Name: ESSA Slope Stability Hydrograph L-R



Name: Levee Fill (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 200 psf
Phi: 28 °
Vol. WC. Function: Levee Fill

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: Oxidized Brenna Formation

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville Formation

Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack Formation

Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Name: Brenna Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna Formation

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_2.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.44

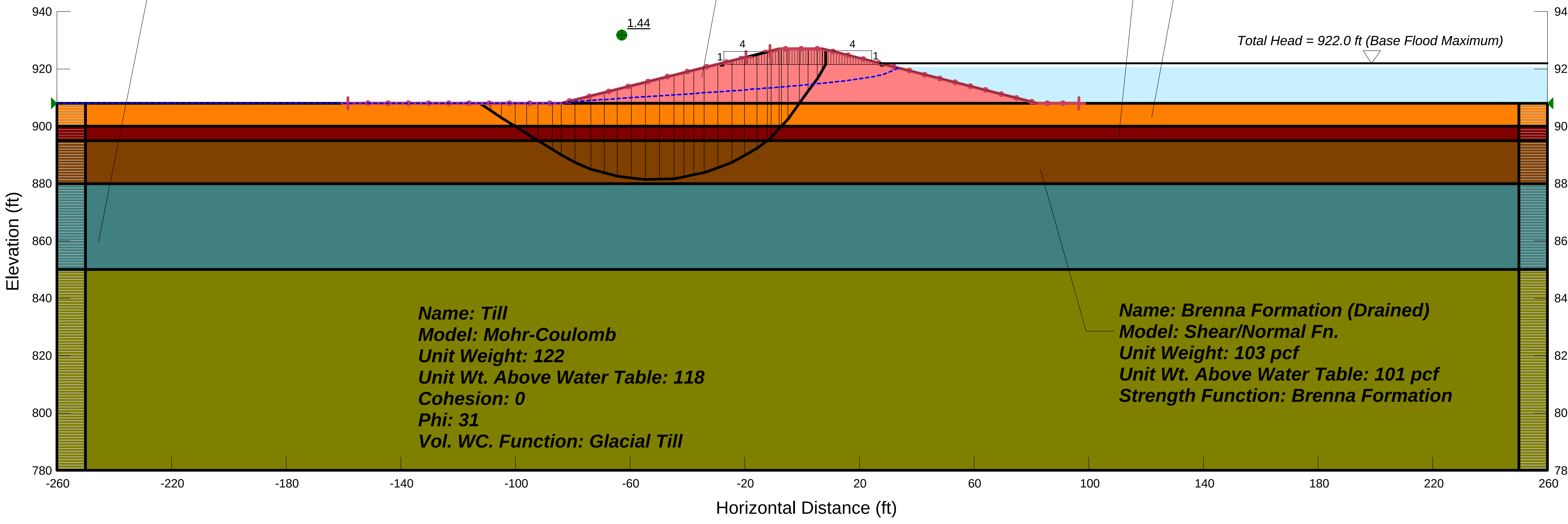
Name: ESSA Slope Stability Hydrograph R-L

Name: Levee Fill (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 200 psf
Phi: 28 °
Vol. WC. Function: Levee Fill

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: Oxidized Brenna Formation

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville Formation

Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack Formation

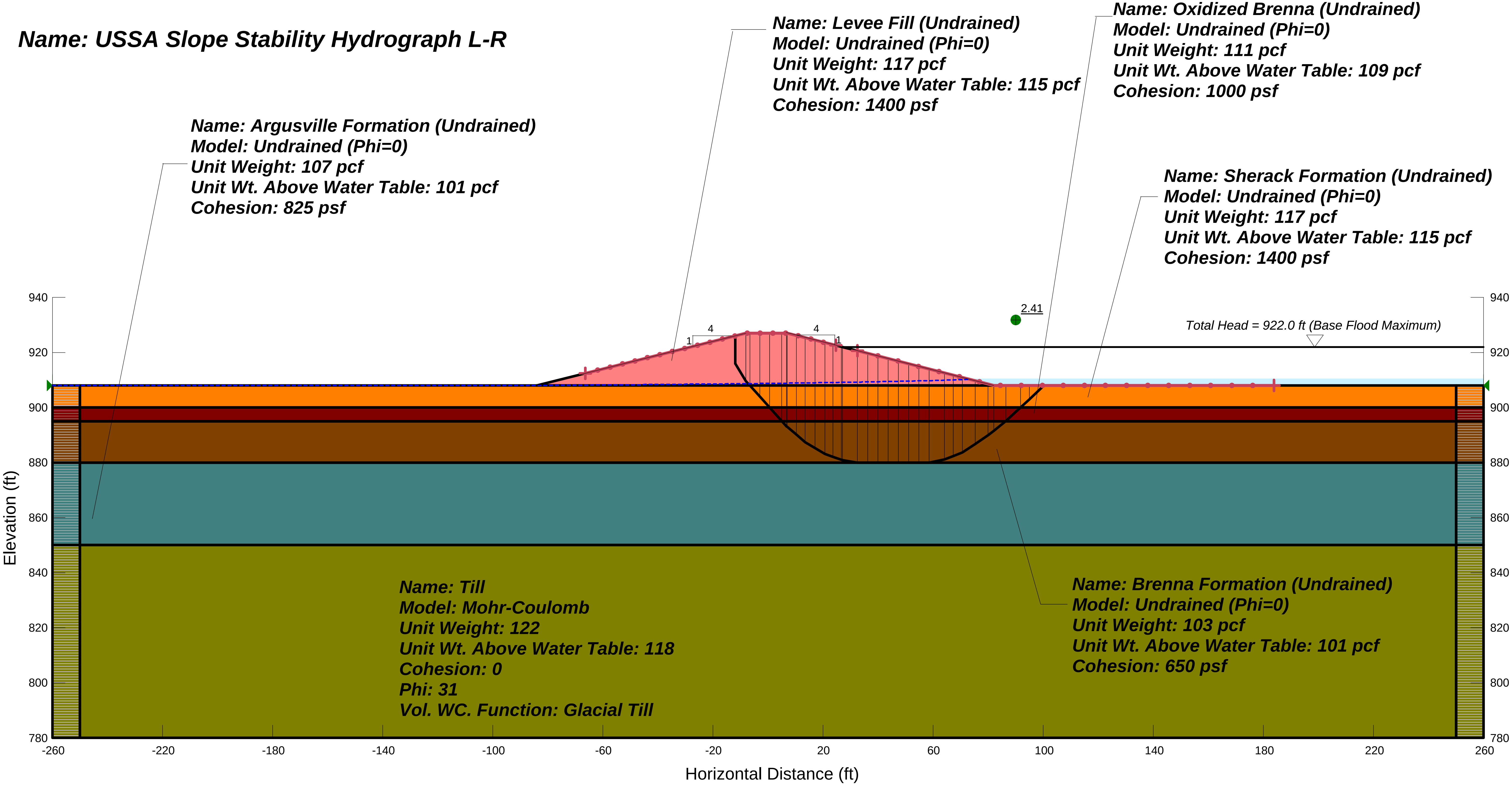


Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Name: Brenna Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna Formation

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_2.gsz
Last Saved Date: 2/1/2011
Factor of Safety: 2.41

Name: USSA Slope Stability Hydrograph L-R



Name: Argusville Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 825 psf

Name: Levee Fill (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

Name: Oxidized Brenna (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Cohesion: 1000 psf

Name: Sherack Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

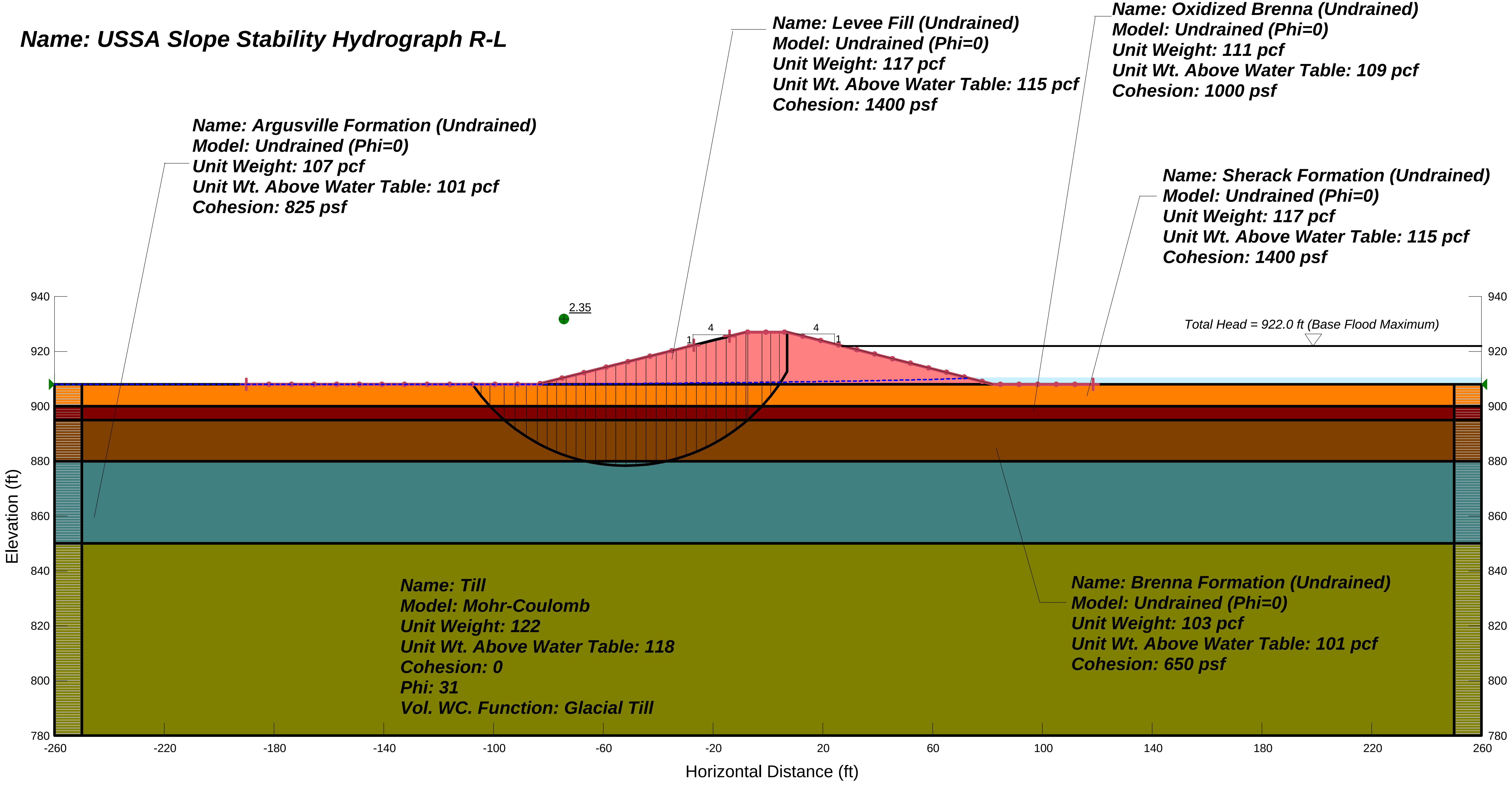
Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Name: Brenna Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 650 psf

Total Head = 922.0 ft (Base Flood Maximum)

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_2.gsz
Last Saved Date: 2/1/2011
Factor of Safety: 2.35

Name: USSA Slope Stability Hydrograph R-L



Levee Storage Area 1

Type 4

2/2/2011

Levee Fill

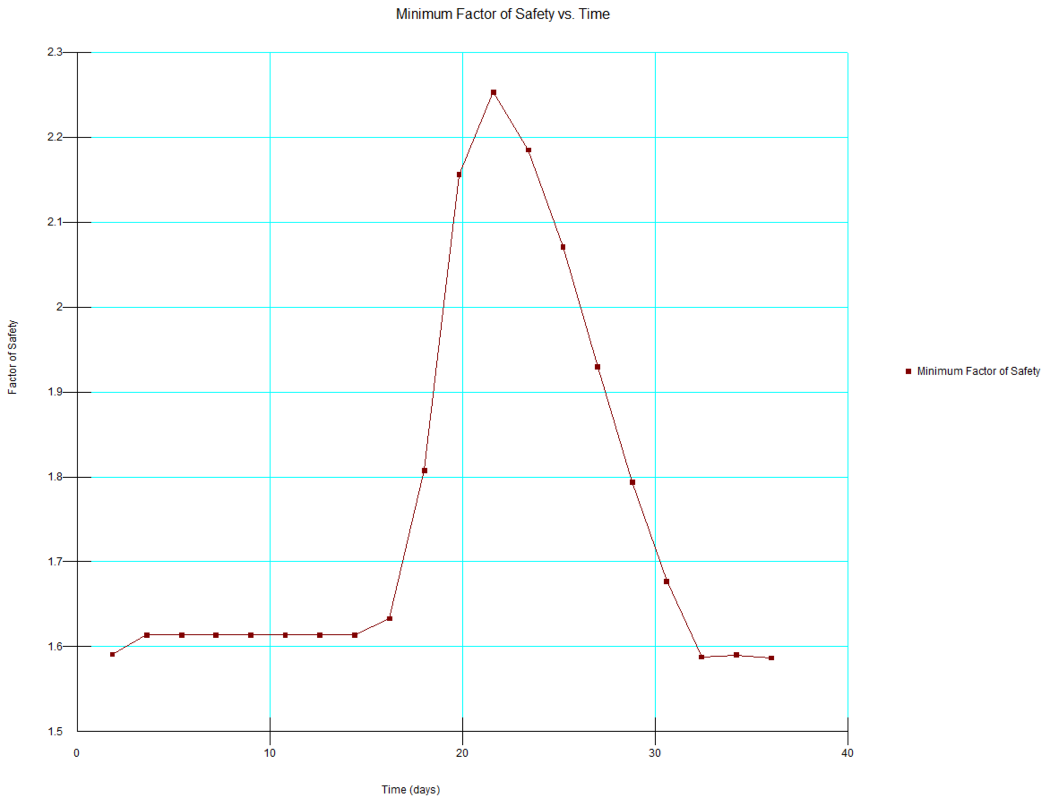
8 ft Sherack

10 ft Brenna

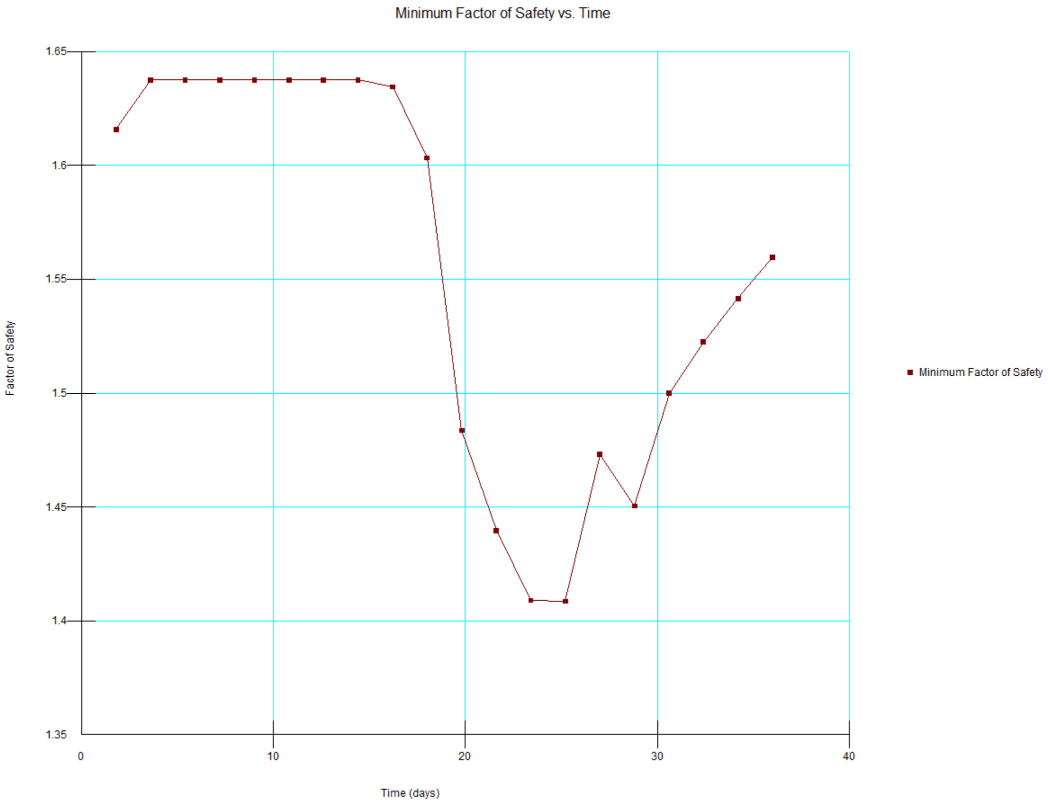
40 ft Argusville

Till

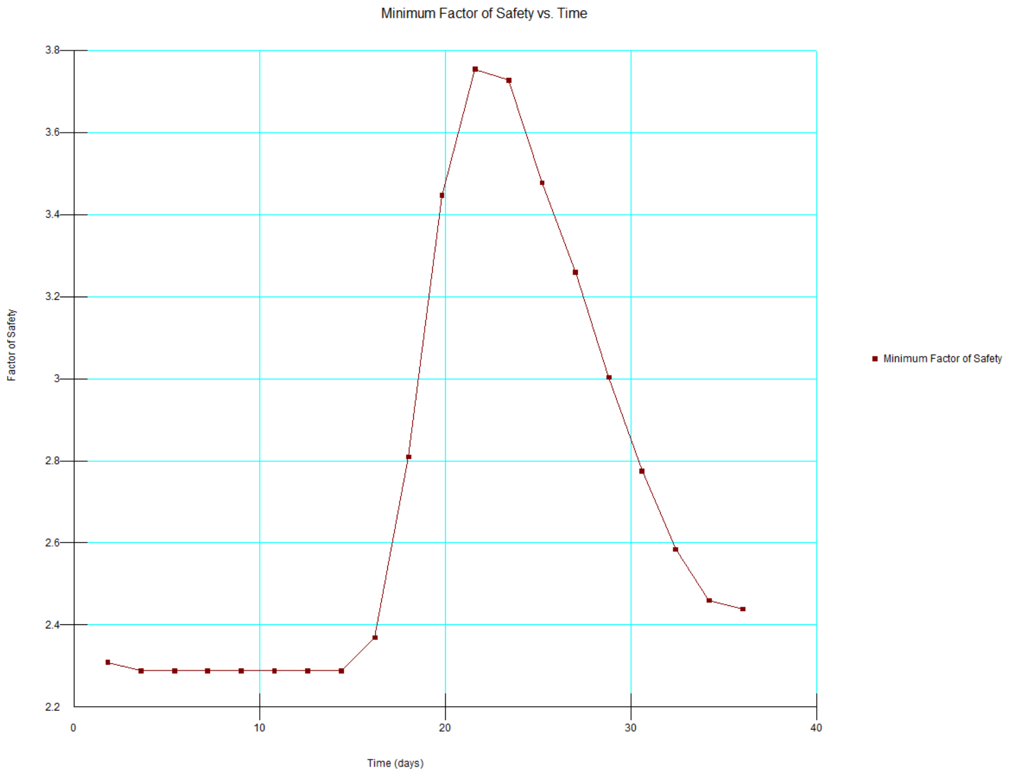
Factor of Safety for ESSA Transient Hydrograph (36 days) Left to Right



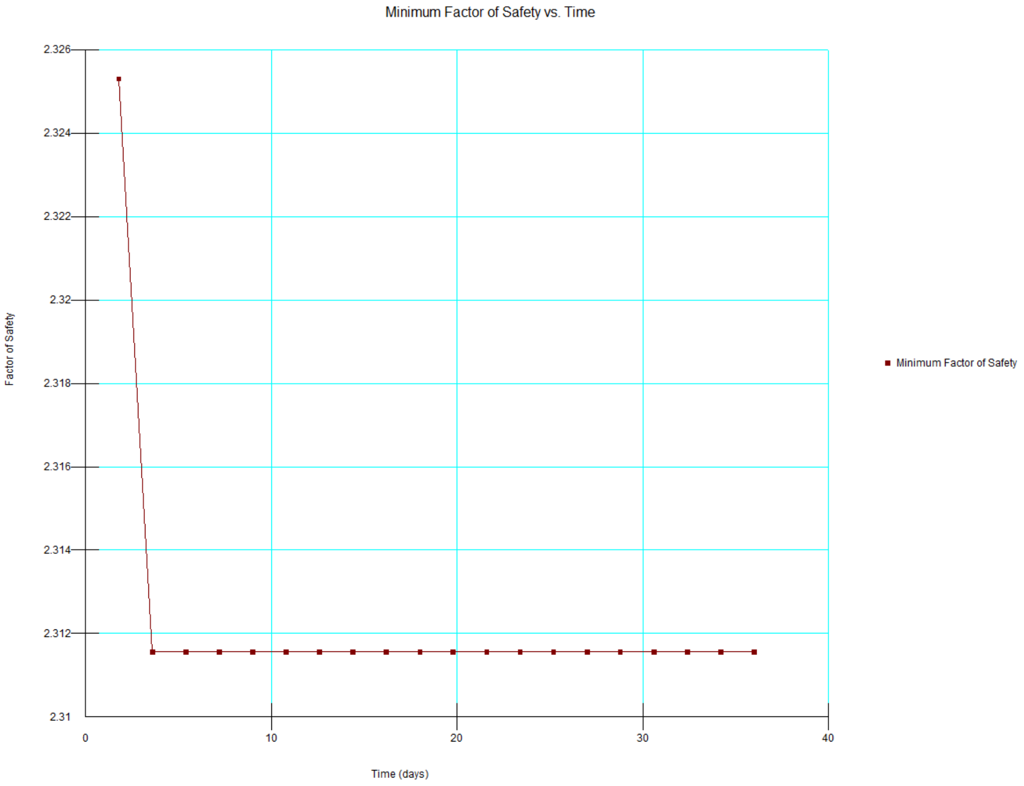
Factor of Safety for ESSA Transient Hydrograph (36 days) Right to Left



Factor of Safety for USSA Transient Hydrograph (36 days) Left to Right



Factor of Safety for USSA Transient Hydrograph (36 days) Right to Left



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.59

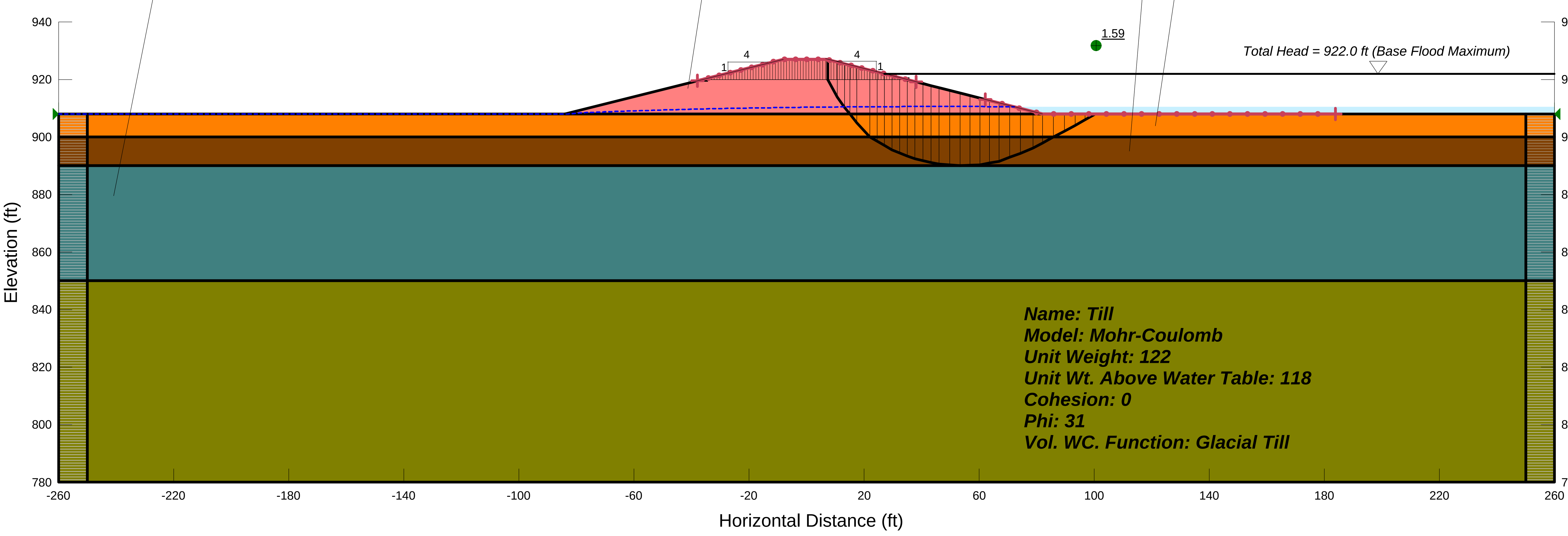
Name: ESSA Slope Stability Hydrograph L-R

Name: Levee Fill (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 200 psf
Phi: 28 °
Vol. WC. Function: Levee Fill

Name: Brenna Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna Formation

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville Formation

Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack Formation



Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.42

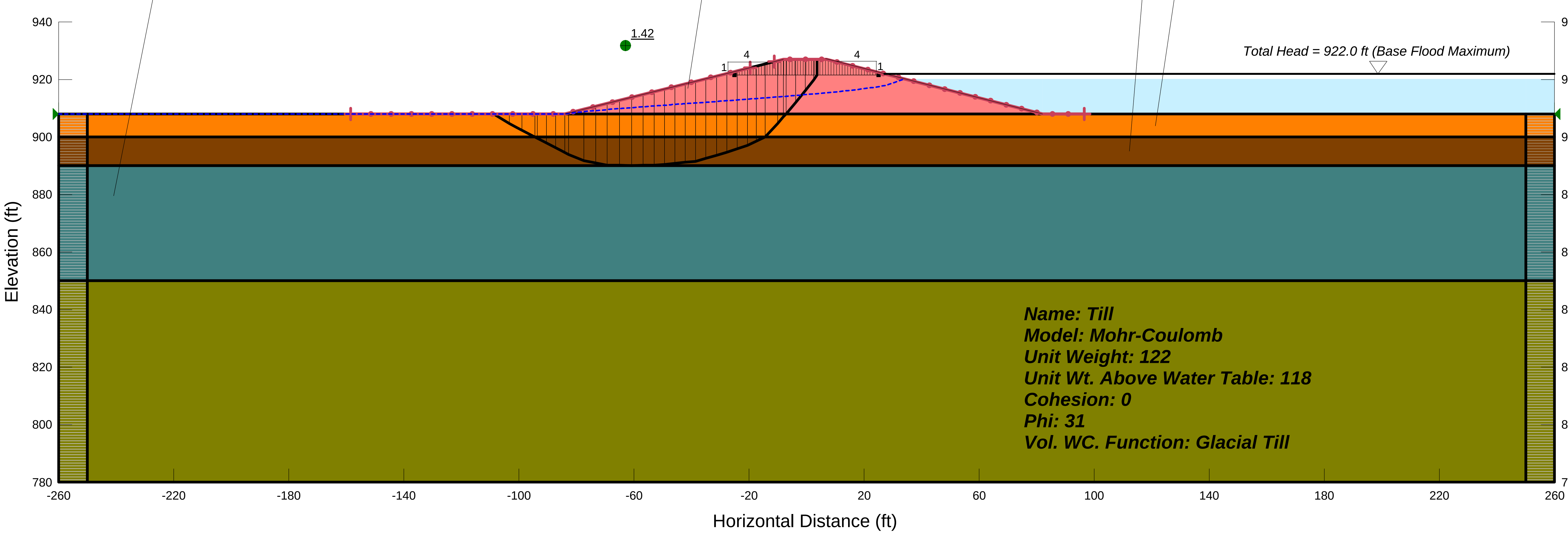
Name: ESSA Slope Stability Hydrograph R-L

Name: Brenna Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna Formation

Name: Levee Fill (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 200 psf
Phi: 28 °
Vol. WC. Function: Levee Fill

Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack Formation

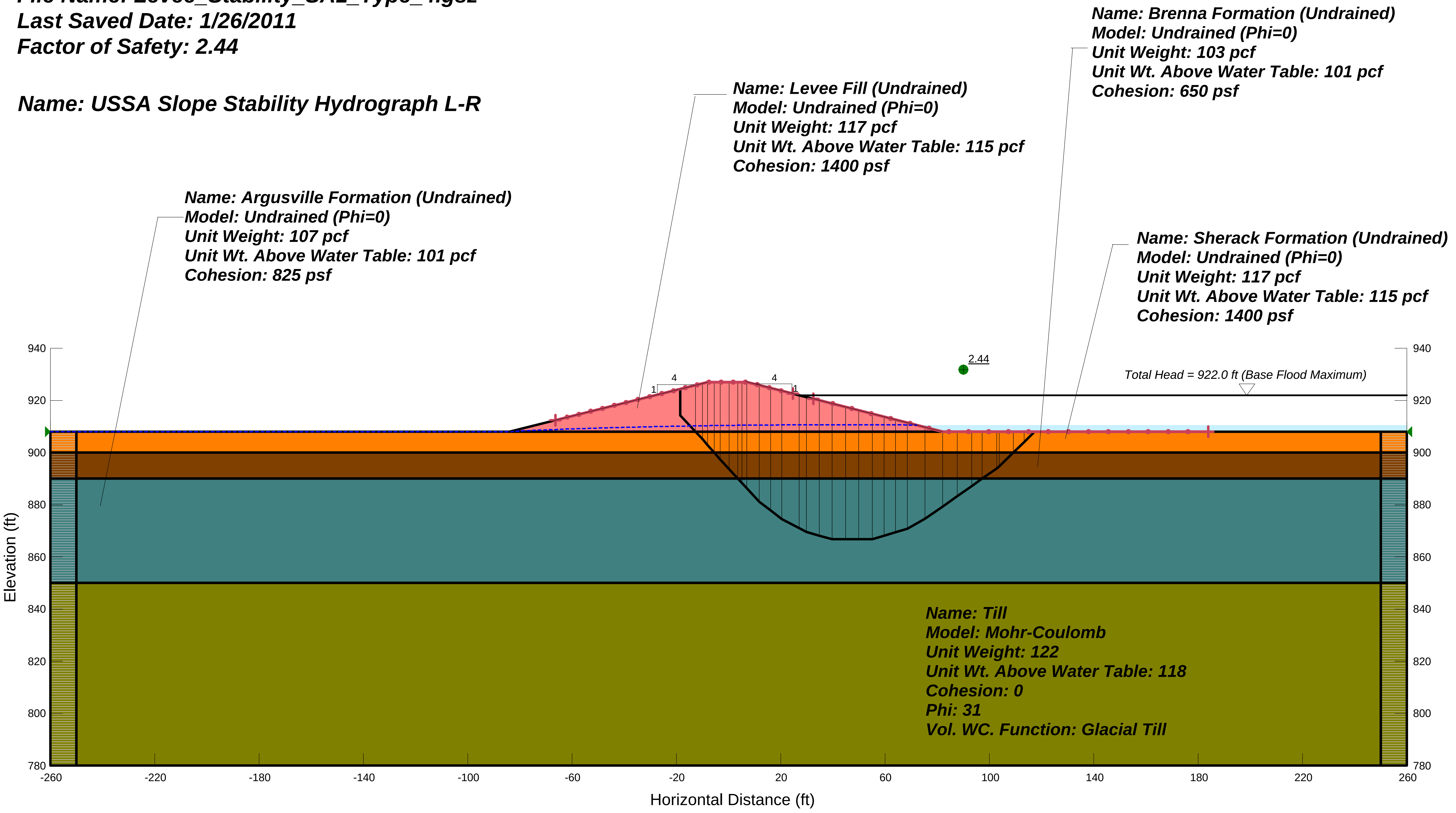
Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville Formation



Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

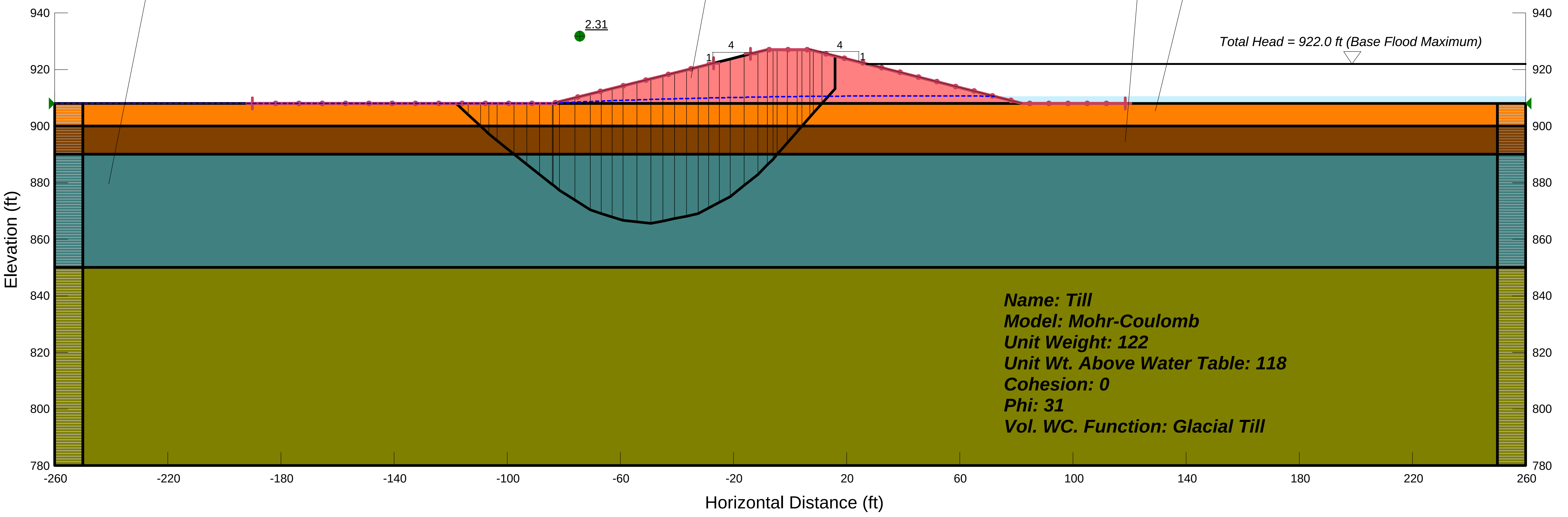
Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 1/26/2011
Factor of Safety: 2.44

Name: USSA Slope Stability Hydrograph L-R



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 1/26/2011
Factor of Safety: 2.31

Name: USSA Slope Stability Hydrograph R-L



Total Head = 922.0 ft (Base Flood Maximum)

Name: Argusville Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 825 psf

Name: Levee Fill (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

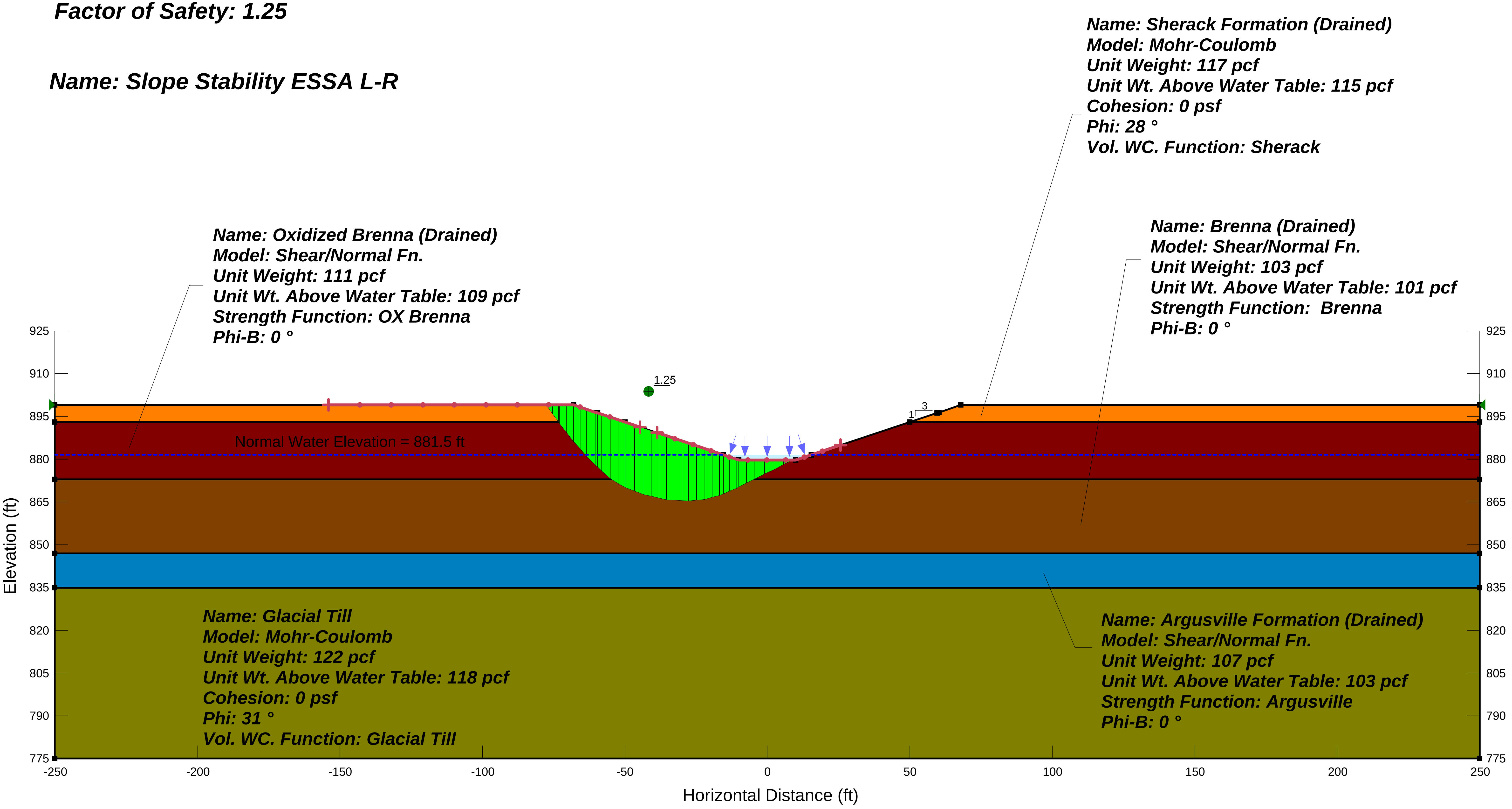
Name: Brenna Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 650 psf

Name: Sherack Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

Name: Till
Model: Mohr-Coulomb
Unit Weight: 122
Unit Wt. Above Water Table: 118
Cohesion: 0
Phi: 31
Vol. WC. Function: Glacial Till

Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Stability Analysis
File Name: Maple River Channel_3to1slope.gsz
Last Saved Date: 2/4/2011
Factor of Safety: 1.25

Name: Slope Stability ESSA L-R



Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna
Phi-B: 0 °

Name: Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna
Phi-B: 0 °

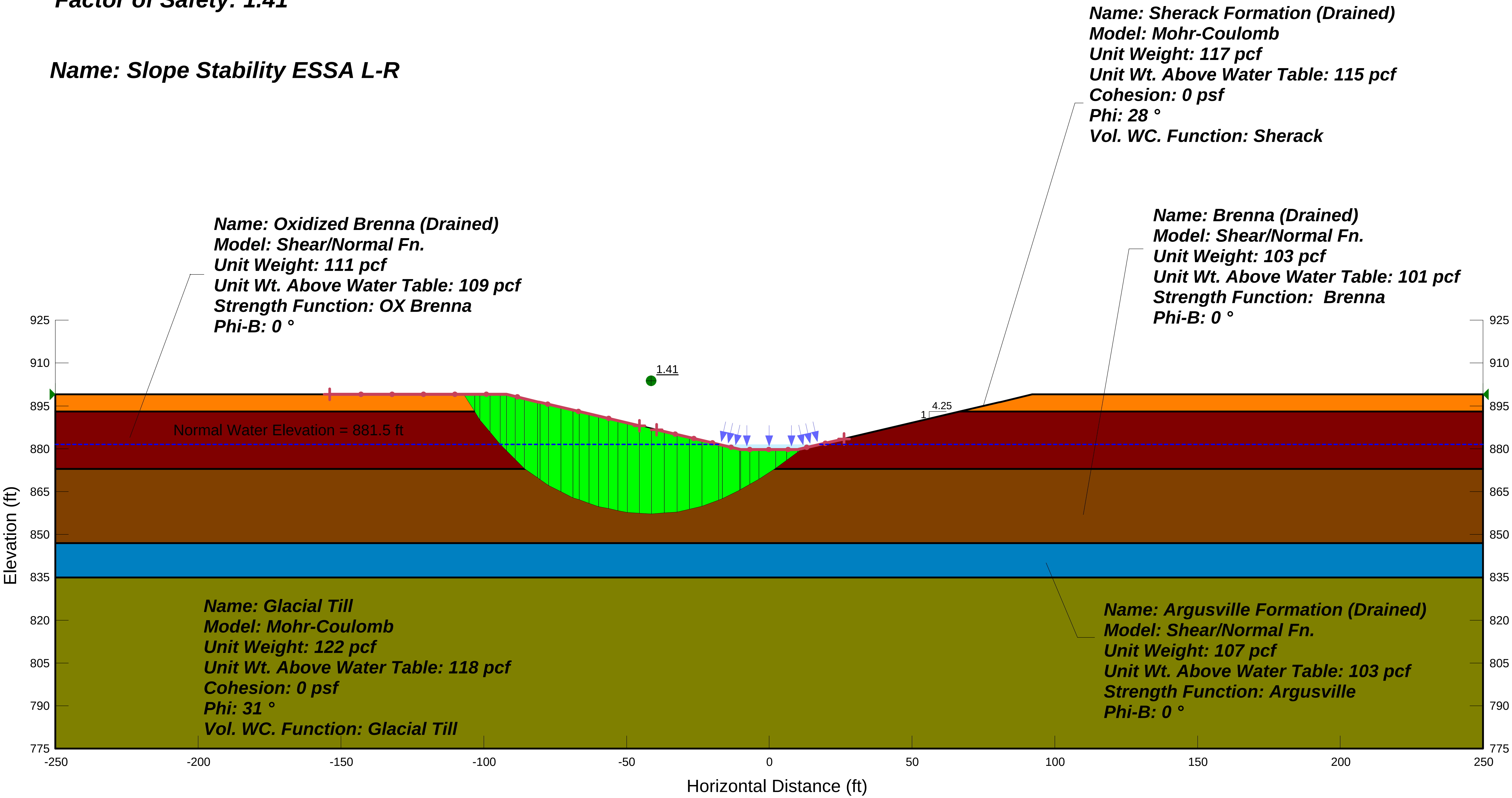
Normal Water Elevation = 881.5 ft

Name: Glacial Till
Model: Mohr-Coulomb
Unit Weight: 122 pcf
Unit Wt. Above Water Table: 118 pcf
Cohesion: 0 psf
Phi: 31 °
Vol. WC. Function: Glacial Till

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville
Phi-B: 0 °

Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Stability Analysis
File Name: Maple River Channel_4.25to1slope.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.41

Name: Slope Stability ESSA L-R



Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna
Phi-B: 0 °

Name: Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna
Phi-B: 0 °

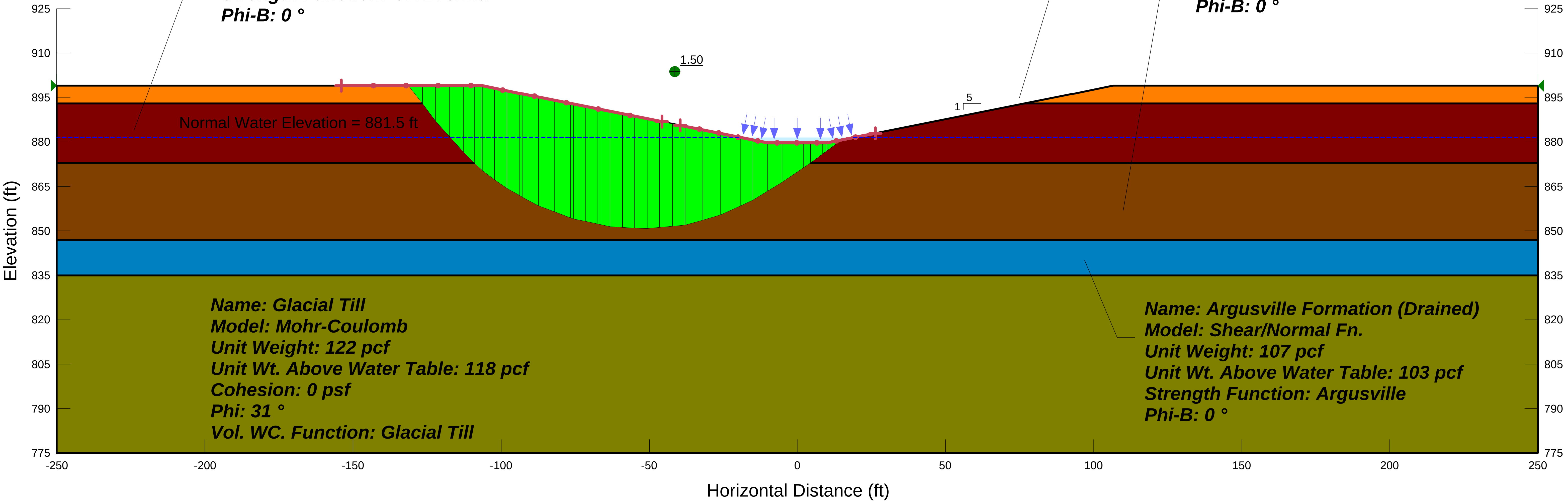
Normal Water Elevation = 881.5 ft

Name: Glacial Till
Model: Mohr-Coulomb
Unit Weight: 122 pcf
Unit Wt. Above Water Table: 118 pcf
Cohesion: 0 psf
Phi: 31 °
Vol. WC. Function: Glacial Till

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville
Phi-B: 0 °

Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Stability Analysis
File Name: Maple River Channel_5to1slope.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.50

Name: Slope Stability ESSA L-R



Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna
Phi-B: 0 °

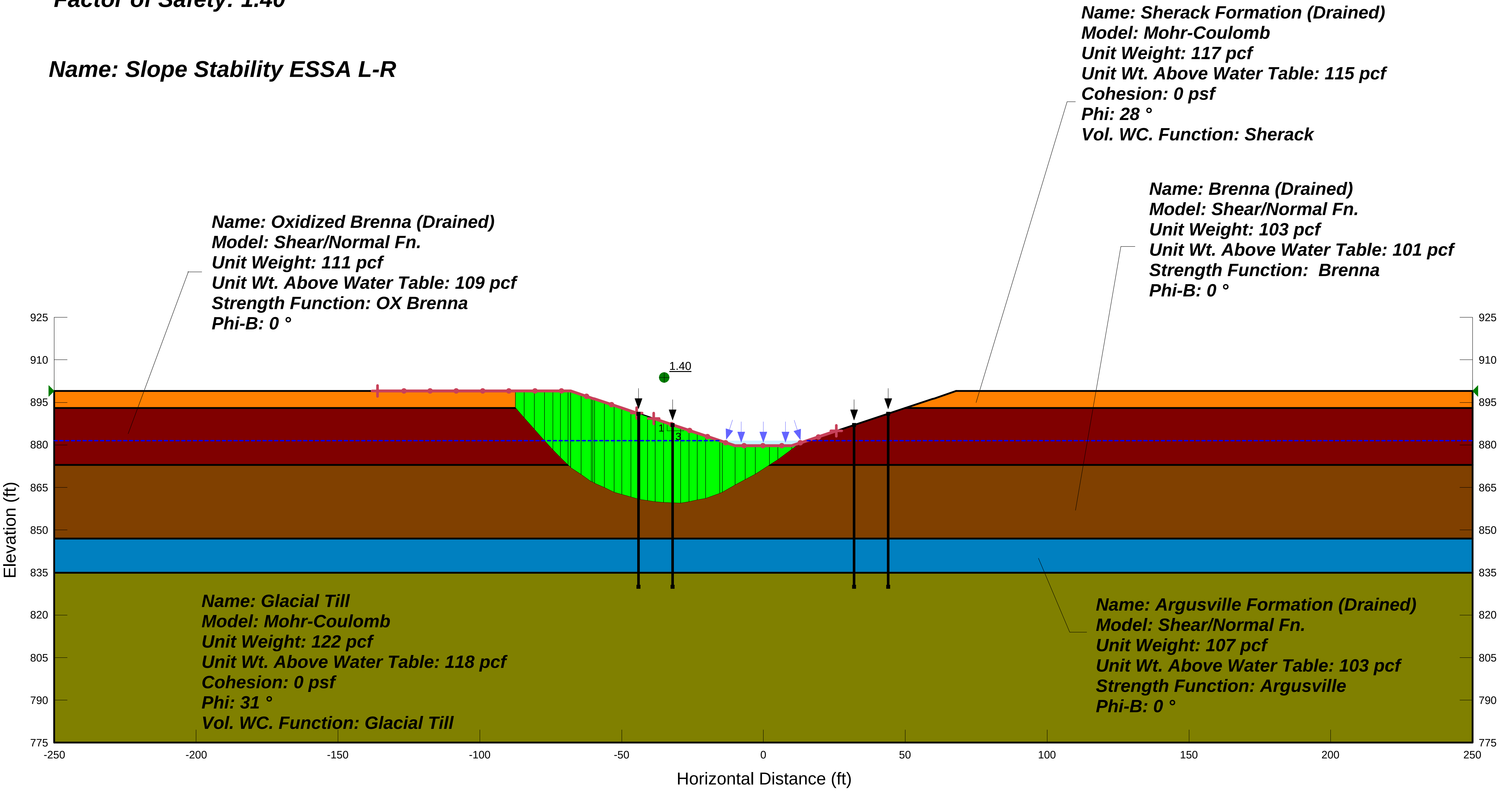
Name: Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna
Phi-B: 0 °

Name: Glacial Till
Model: Mohr-Coulomb
Unit Weight: 122 pcf
Unit Wt. Above Water Table: 118 pcf
Cohesion: 0 psf
Phi: 31 °
Vol. WC. Function: Glacial Till

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville
Phi-B: 0 °

Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Piles: 12ft spacing, 38,000 Shear Force
File Name: Maple River Channel_3to1_piles.gsz
Last Saved Date: 2/4/2011
Factor of Safety: 1.40

Name: Slope Stability ESSA L-R



Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack

Name: Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna
Phi-B: 0 °

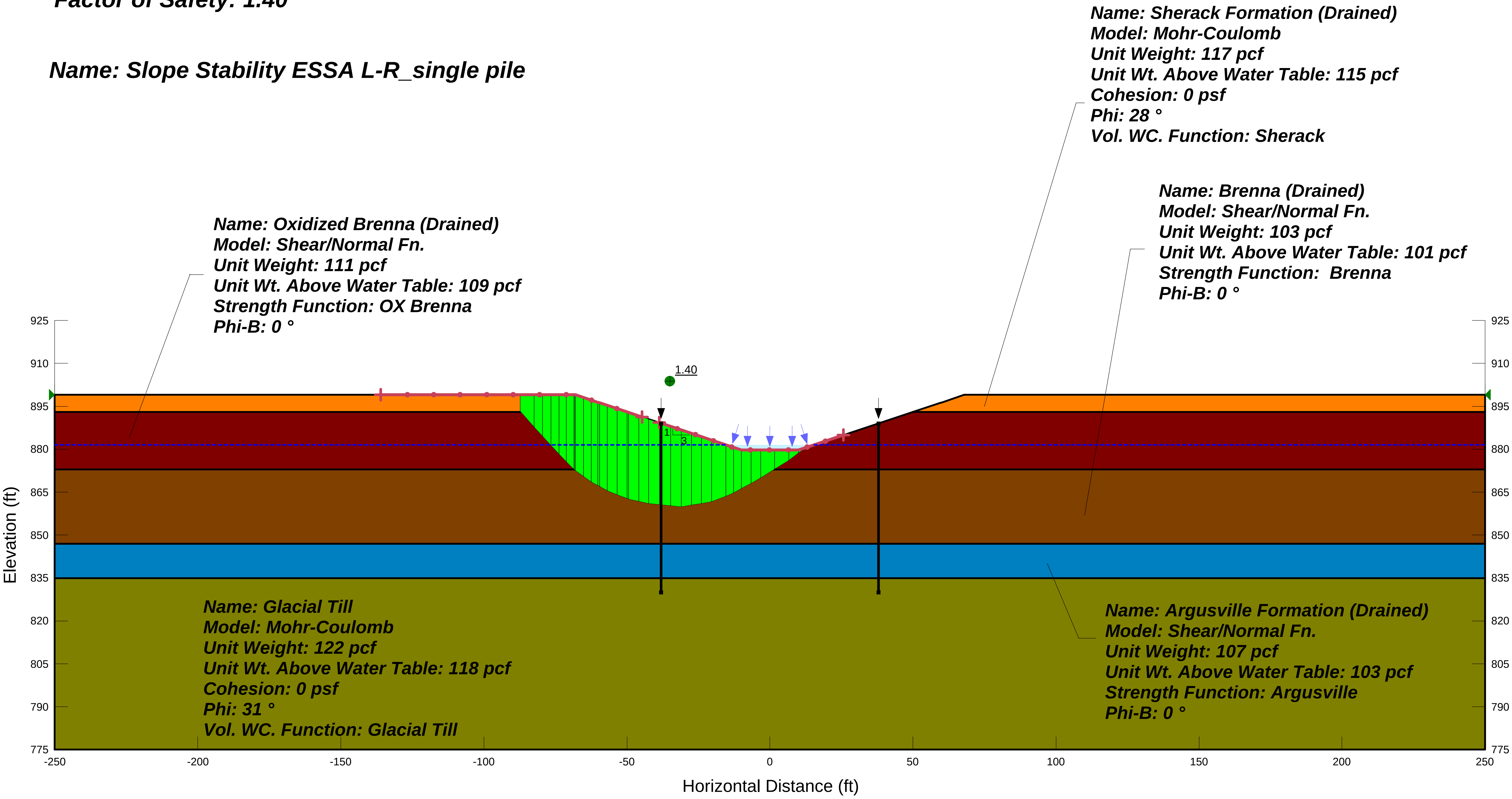
Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna
Phi-B: 0 °

Name: Glacial Till
Model: Mohr-Coulomb
Unit Weight: 122 pcf
Unit Wt. Above Water Table: 118 pcf
Cohesion: 0 psf
Phi: 31 °
Vol. WC. Function: Glacial Till

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville
Phi-B: 0 °

Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Piles: 12ft spacing, 38,000 Shear Force
File Name: Maple River Channel_3to1_piles.gsz
Last Saved Date: 2/7/2011
Factor of Safety: 1.40

Name: Slope Stability ESSA L-R_single pile



Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack

Name: Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna
Phi-B: 0 °

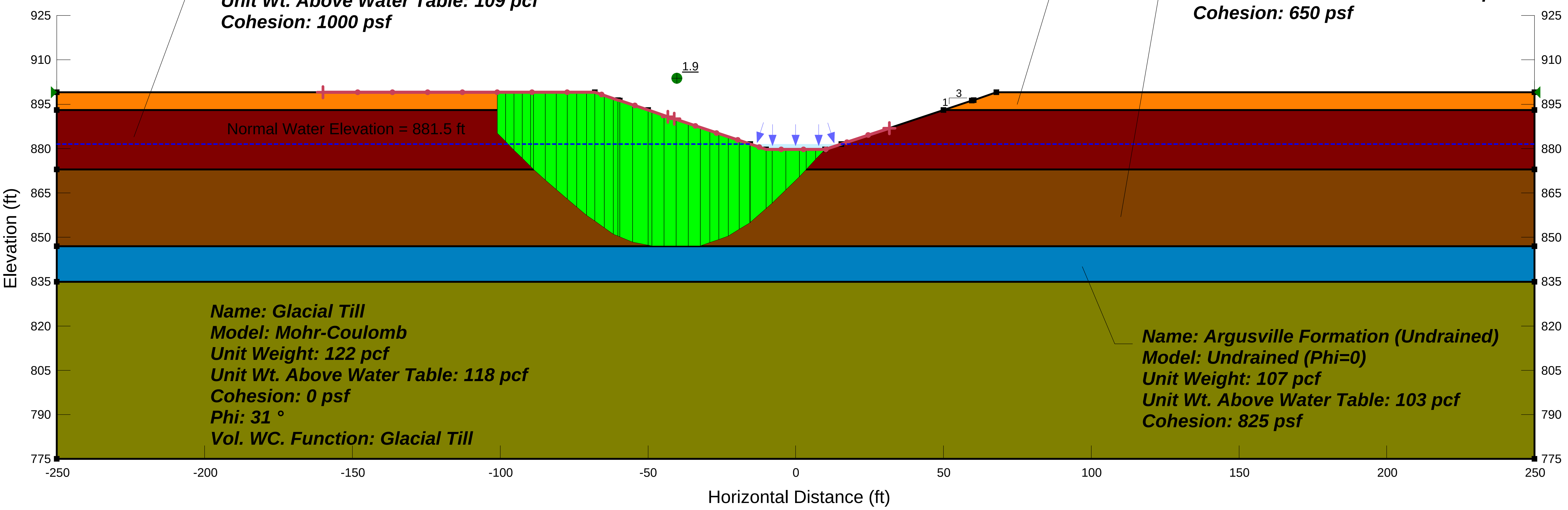
Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna
Phi-B: 0 °

Name: Glacial Till
Model: Mohr-Coulomb
Unit Weight: 122 pcf
Unit Wt. Above Water Table: 118 pcf
Cohesion: 0 psf
Phi: 31 °
Vol. WC. Function: Glacial Till

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville
Phi-B: 0 °

Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Stability Analysis
File Name: Maple River Channel_3to1slope.gsz
Last Saved Date: 2/4/2011
Factor of Safety: 1.9

Name: Slope Stability USSA L-R



Name: Sherack Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 1400 psf

Name: Oxidized Brenna (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Cohesion: 1000 psf

Name: Brenna (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Cohesion: 650 psf

Name: Glacial Till
Model: Mohr-Coulomb
Unit Weight: 122 pcf
Unit Wt. Above Water Table: 118 pcf
Cohesion: 0 psf
Phi: 31 °
Vol. WC. Function: Glacial Till

Name: Argusville Formation (Undrained)
Model: Undrained (Phi=0)
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Cohesion: 825 psf

Fargo-Moorhead Metro Flood Risk Management Project
Phase 4

Last Updated MRM 09Feb2011 **DRAFT**

Tributary Channels Slope Stability Cost Alternatives on a per-linear-foot-basis

Quantities and concept design by JDG, KNA, DMH2

Note: costs are Phase 3 contract costs and do not include contingency, costs are for comparitive purposes only

Engineer's Opinion of Cost

Option 1 - Grade Excavated Side Slopes 5H:1V

Item (per LF channel)	Qty	Unit	Unit Cost	Extended Cost	
Excavation and Spoil (full channel xs)	97.6	BCY	$ 3.49	$ 340.78	5H:1V side slopes, 20' bottom width, 19.3' depth
Total (per LF channel)				$ 340.78	

Option 2 - Grade Excavated Side Slopes 3H:1V, 1); Install two 24" piles each bank, spacing 12 ft apart. The Shear Force per pile is 38,000 lbs

Item (per LF channel)	Qty	Unit	Unit Cost	Extended Cost	
Excavation and Spoil (full channel xs)	70.0	BCY	$ 3.49	$ 244.35	3H:1V side slopes, 20' bottom width, 19.3' depth
Drilled Shaft Piles (38,000 lbs shear force per pile)	19.7	VLF	$ 117.77	$ 2,316.18	2 rows 57 LF and 61 LF of 24" diameter piles (each bank), 1/2" wall thickness, no concrete; spacing 12' o.c.
Total (per LF channel)				$ 2,560.54	

Option 3 - Grade Excavated Side Slopes 3H:1V; Install one 30" pile per bank, 12 ft spacing with a Shear Force per pile of 75,000 lbs

Item (per LF channel)	Qty	Unit	Unit Cost	Extended Cost	
Excavation and Spoil (full channel xs)	70.0	BCY	$ 3.49	$ 244.35	3H:1V side slopes, 20' bottom width, 19.3' depth
Drilled Shaft Piles (75,000 lbs shear force per pile)	9.8	VLF	$ 157.80	$ 1,551.73	1 row 59 LF of 30" diameter piles (each bank), 1/2" wall thickness, no concrete; spacing 12' o.c.
Total (per LF channel)				$ 1,796.08	

Estimated Unit Cost Calculation

2006		*2010 (x1.13)*		size		
$ 0.55	per vlf	$ 0.62	per vlf	Mobilization	2006 Barr #23-60-0024	
$ 123.00	per vlf	$ 138.99	per vlf	Drilling 48"	2006 Barr #23-60-0024	
$ 143.00	per vlf	$ 161.59	per vlf	Steel pipe piles 48"	2006 Barr #23-60-0024	253.89 lbs/ft
$ 84.00	CY	$ 94.92	CY	Concrete ($84/CY)	2006 Barr #23-60-0024	
$ 4.83	CY	$ 5.46	CY	Load and haul excess ($5/CY)	2006 Barr #23-60-0024	
$ 0.56	lb	$ 0.63	lb	pile cost	2006 Barr #23-60-0024	
		2010	*2010*			
		75k shear (30")	*38k shear 24"*			
		$ 0.62	$ 0.62	Mobilization		
		$ 54.29	$ 34.75	Drilling	xs area ratio	
		$ 101.89	$ 81.50	Steel pipe piles 1/2" wall thickness	161.02 lbs/vlf for 30" and 128.8 lbs/vlf for 24"	
		$ -	$ -	Concrete ($84/CY)		
		$ 0.99	$ 0.90	Load and haul excess ($5/CY)		
		$ 157.80	$ 117.77	Estimated cost per VLF per pile		

Notes:

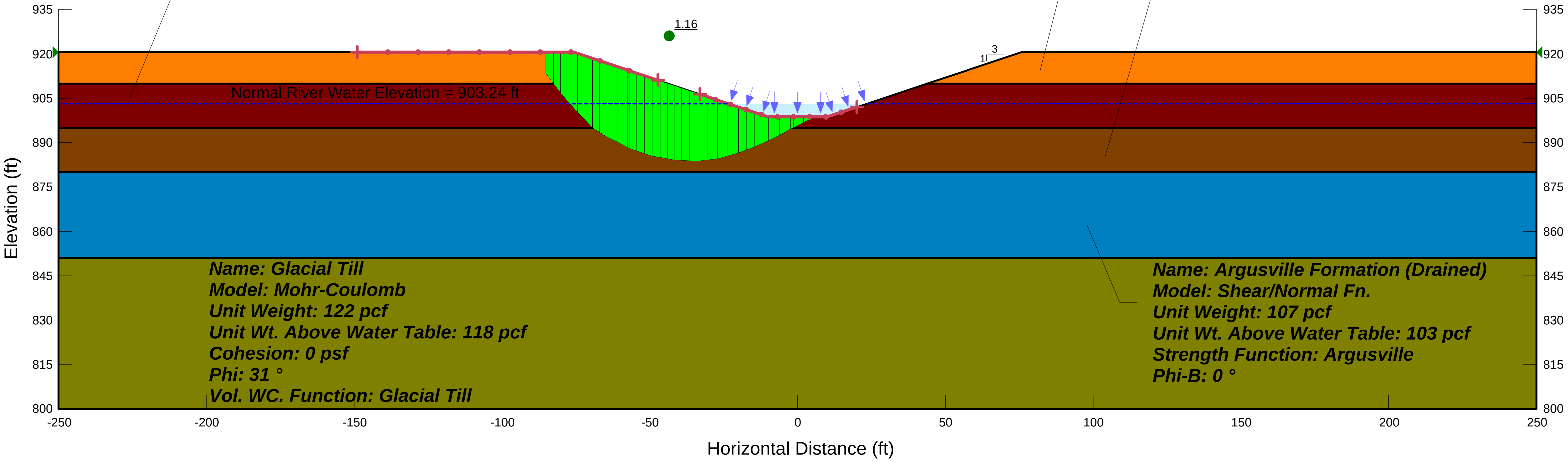
Barr Engineering #23-60-0024 (Crookston), Cost Quote: 2006 Skyline Steel $142/VLF for 48" diameter, 1/2" wall thickness, delivered (Skyline Steel, $1,120/ton, 253.89 lb.ft)

Cost estimate 2006 from P:\Mpls\23 MN\60\2360024_MovedFromMpls_P\ $2.1 Million for 6510 LF, or $322/LF for 48" dia. Concrete filled shaft piles

Time Factor 2010/2006 = 1.13 RS Means

Fargo-Moorhead, Phase 4, Sheyenne River Upstream Channel
Stability Analysis
File Name: Sheyenne River Channel_3to1.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.16

Name: Slope Stability ESSA L-R



Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna
Phi-B: 0 °

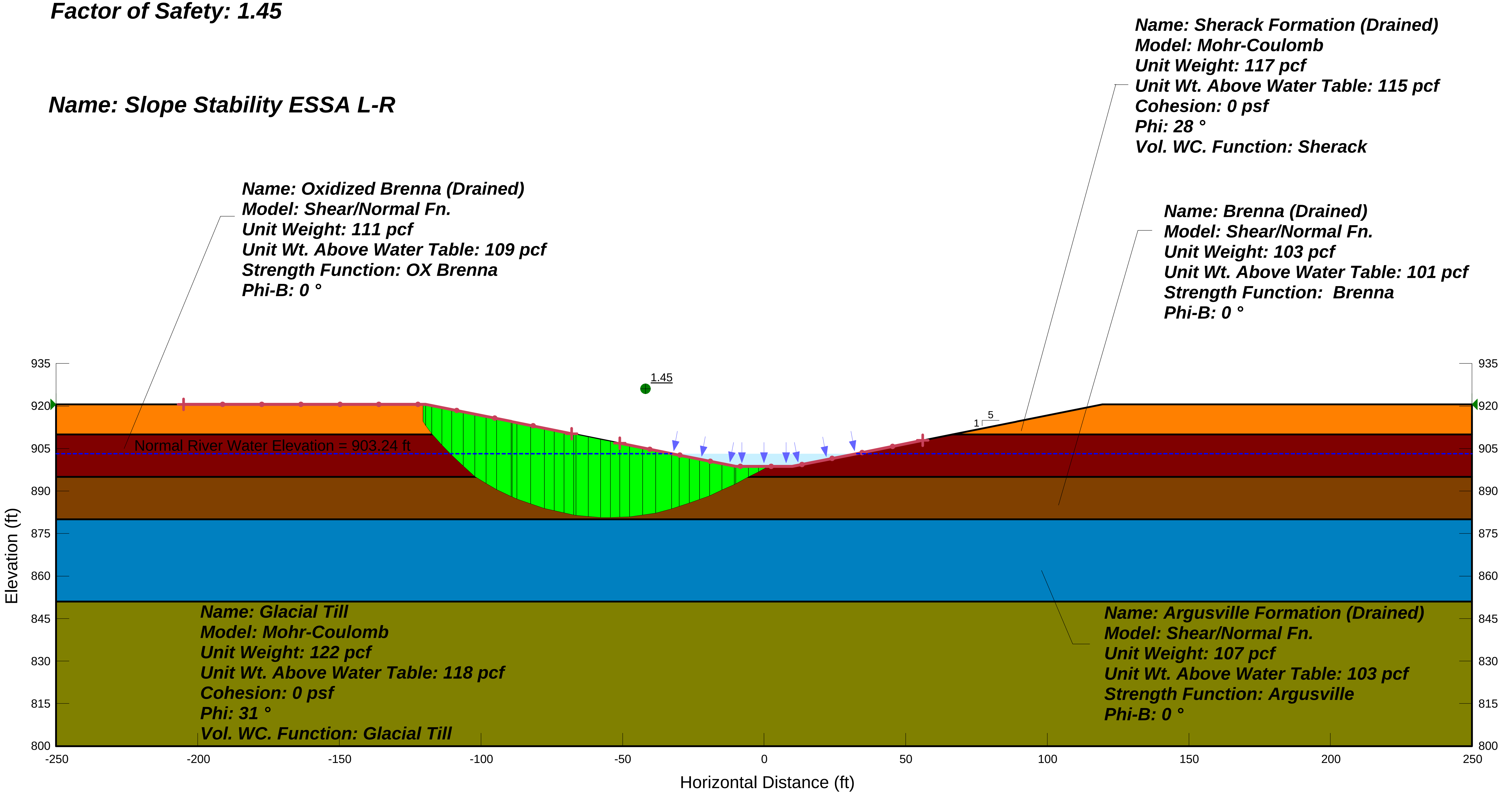
Name: Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna
Phi-B: 0 °

Name: Glacial Till
Model: Mohr-Coulomb
Unit Weight: 122 pcf
Unit Wt. Above Water Table: 118 pcf
Cohesion: 0 psf
Phi: 31 °
Vol. WC. Function: Glacial Till

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville
Phi-B: 0 °

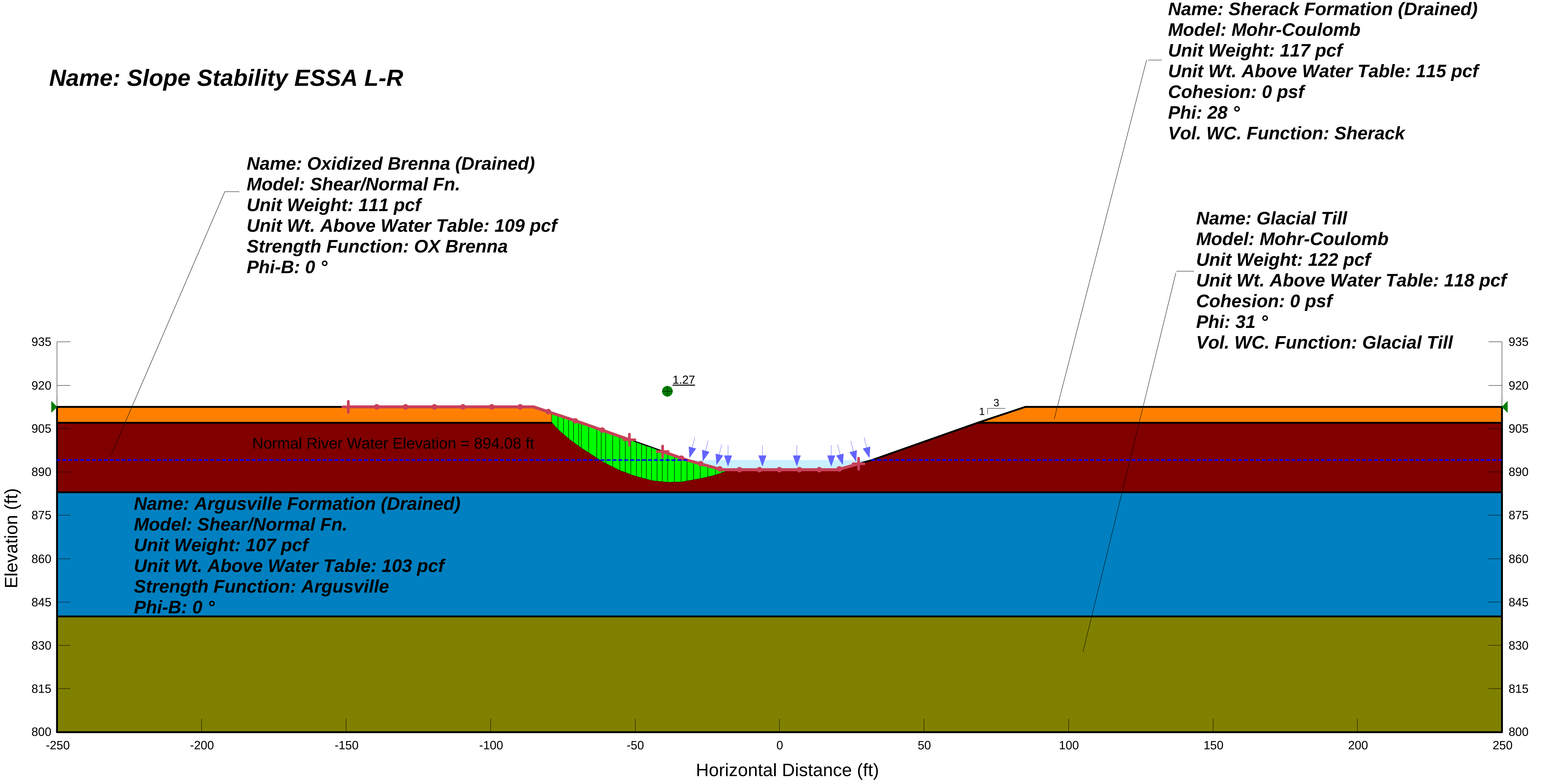
Fargo-Moorhead, Phase 4, Sheyenne River Upstream Channel
Stability Analysis
File Name: Sheyenne River Channel_5to1.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.45

Name: Slope Stability ESSA L-R



Fargo-Moorhead, Phase 4, Wild Rice River Upstream Channel
Stability Analysis
File Name: Wild Rice River Channel_3to1.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.27

Name: Slope Stability ESSA L-R



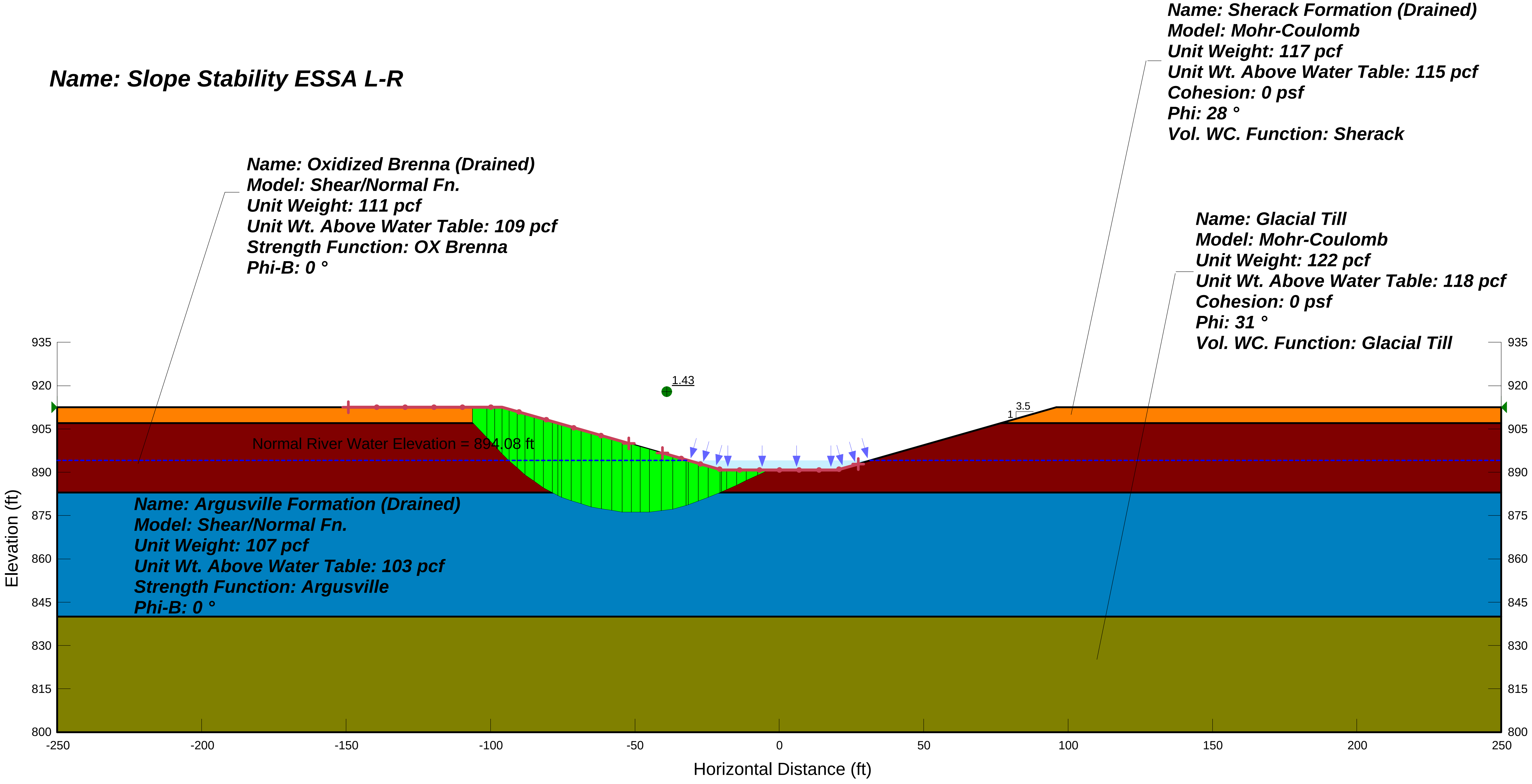
Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna
Phi-B: 0 °

Name: Glacial Till
Model: Mohr-Coulomb
Unit Weight: 122 pcf
Unit Wt. Above Water Table: 118 pcf
Cohesion: 0 psf
Phi: 31 °
Vol. WC. Function: Glacial Till

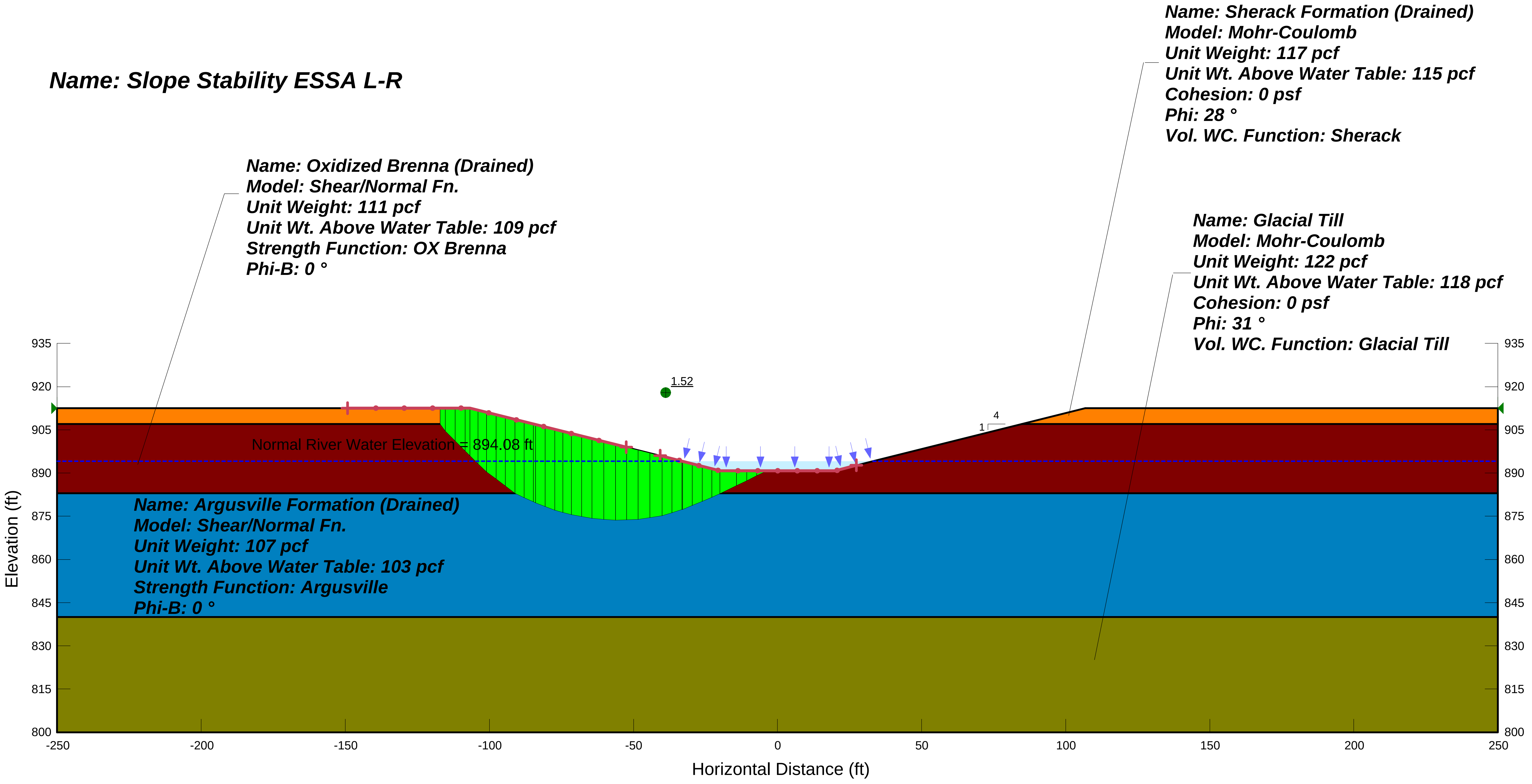
Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville
Phi-B: 0 °

Fargo-Moorhead, Phase 4, Wild Rice River Upstream Channel
Stability Analysis
File Name: Wild Rice River Channel_3.5to1.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.43



Fargo-Moorhead, Phase 4, Wild Rice River Upstream Channel
Stability Analysis
File Name: Wild Rice River Channel_4to1.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.52

Name: Slope Stability ESSA L-R



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Stability Analysis
File Name: Red River Channel_3to1_UPDATED.gsz
Last Saved Date: 2/27/2011
Factor of Safety: 0.99

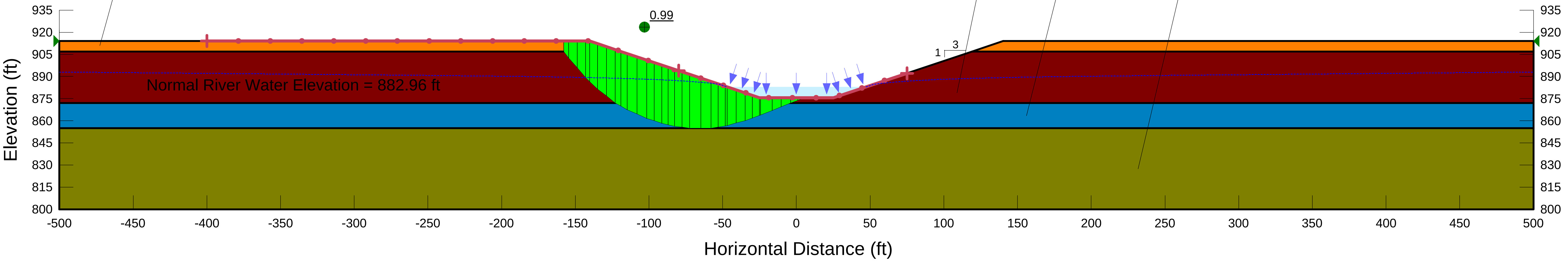
Name: Slope Stability ESSA L-R

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna
Phi-B: 0 °

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville
Phi-B: 0 °

Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack_Formation

Name: Glacial Till
Model: Bedrock (Impenetrable)



Fargo-Moorhead, Phase 4, Red River Upstream Channel
Stability Analysis
File Name: Red River Channel_7to1_UPDATED.gsz
Last Saved Date: 2/26/2011
Factor of Safety: 1.62

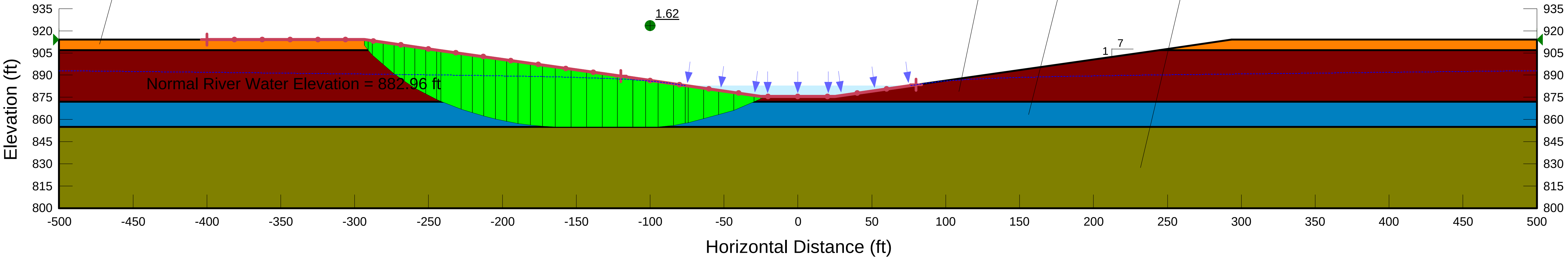
Name: Slope Stability ESSA L-R

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna
Phi-B: 0 °

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville
Phi-B: 0 °

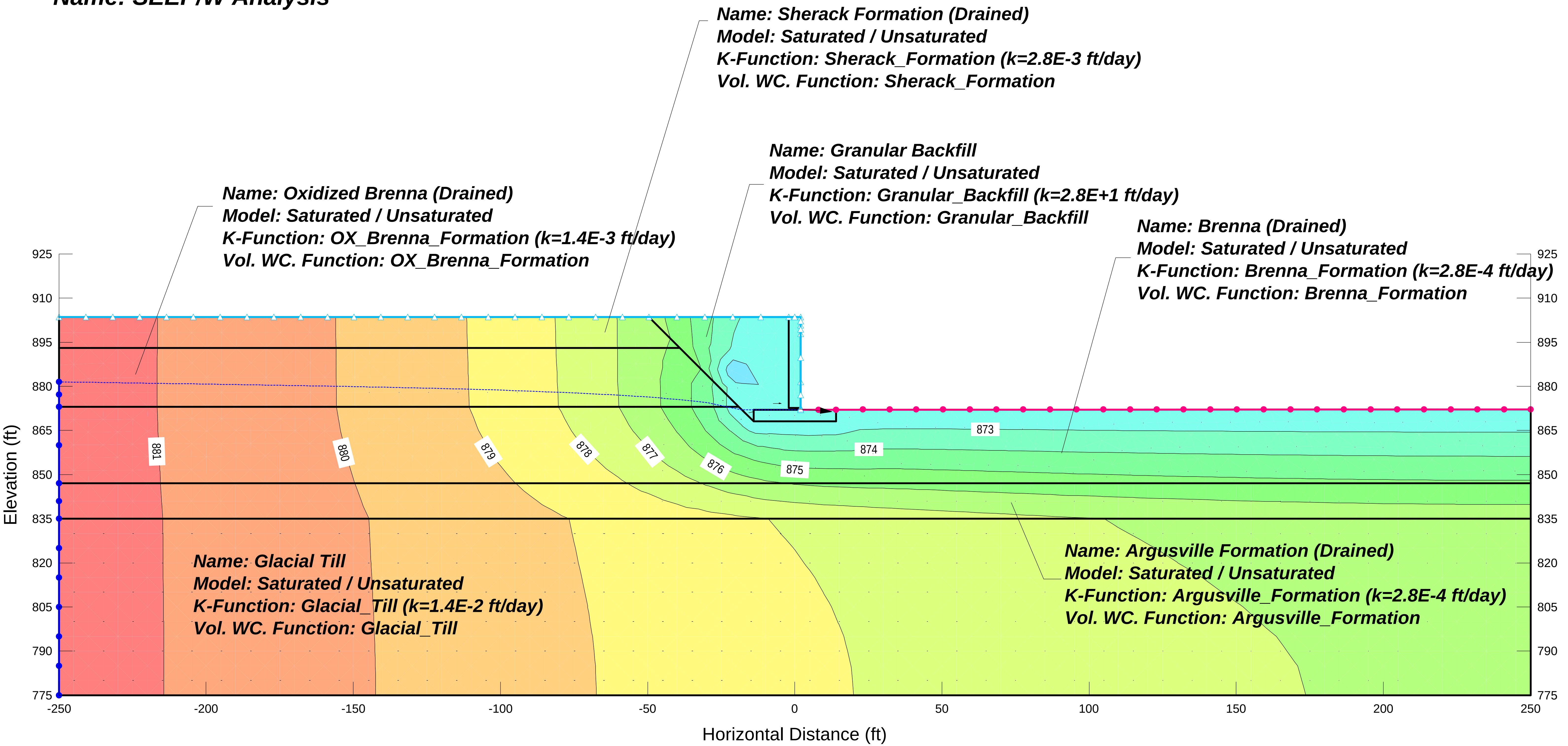
Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack_Formation

Name: Glacial Till
Model: Bedrock (Impenetrable)



Fargo-Moorhead, Phase 4, Maple River Wall
Stability Analysis
File Name: Maple River Wall_UPDATED.gsz
Last Saved Date: 2/26/2011

Name: SEEP/W Analysis



Name: Sherack Formation (Drained)
Model: Saturated / Unsaturated
K-Function: Sherack_Formation (k=2.8E-3 ft/day)
Vol. WC. Function: Sherack_Formation

Name: Granular Backfill
Model: Saturated / Unsaturated
K-Function: Granular_Backfill (k=2.8E+1 ft/day)
Vol. WC. Function: Granular_Backfill

Name: Oxidized Brenna (Drained)
Model: Saturated / Unsaturated
K-Function: OX_Brenna_Formation (k=1.4E-3 ft/day)
Vol. WC. Function: OX_Brenna_Formation

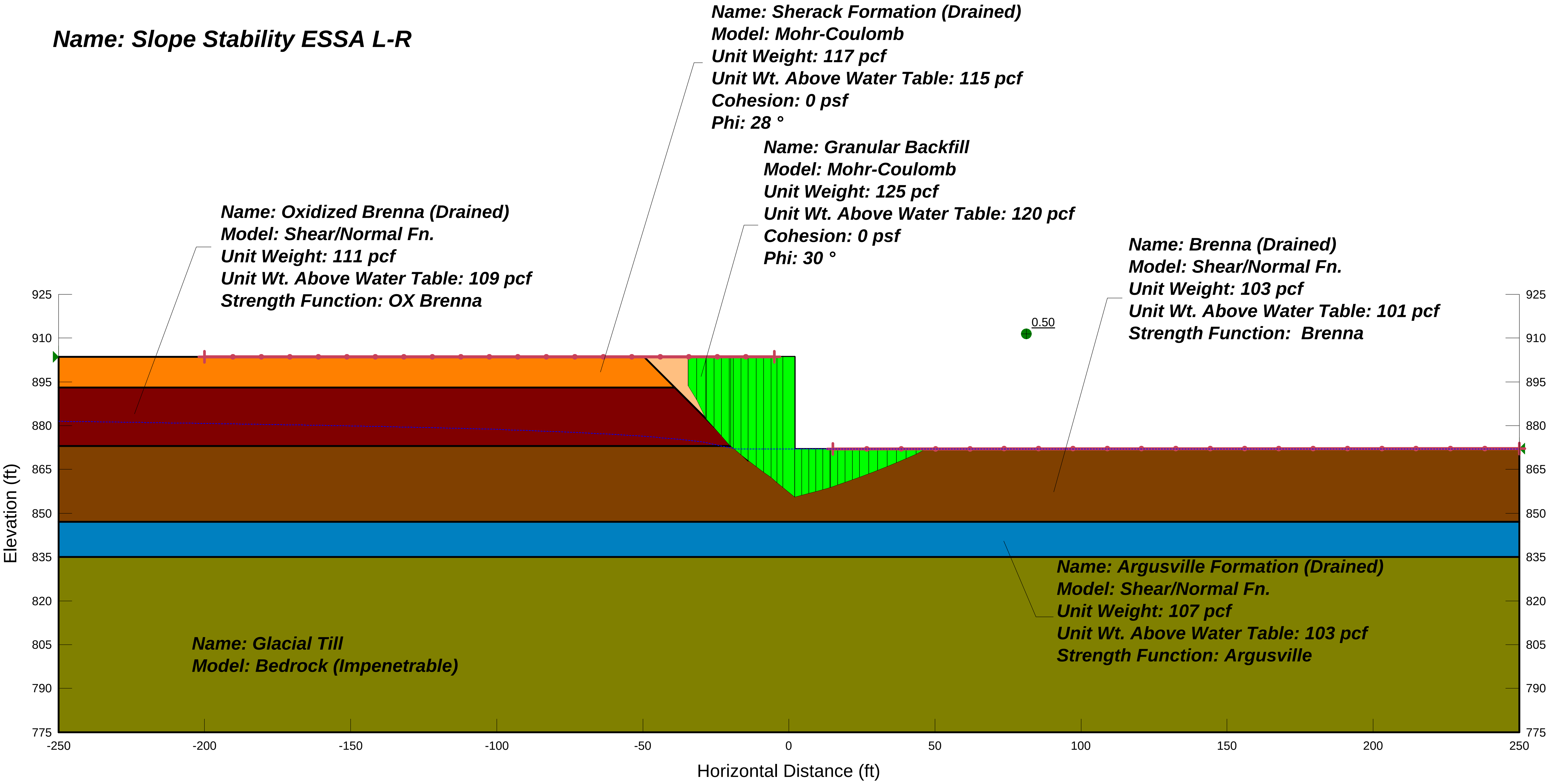
Name: Brenna (Drained)
Model: Saturated / Unsaturated
K-Function: Brenna_Formation (k=2.8E-4 ft/day)
Vol. WC. Function: Brenna_Formation

Name: Glacial Till
Model: Saturated / Unsaturated
K-Function: Glacial_Till (k=1.4E-2 ft/day)
Vol. WC. Function: Glacial_Till

Name: Argusville Formation (Drained)
Model: Saturated / Unsaturated
K-Function: Argusville_Formation (k=2.8E-4 ft/day)
Vol. WC. Function: Argusville_Formation

Fargo-Moorhead, Phase 4, Maple River Wall
Stability Analysis
File Name: Maple River Wall_UPDATED.gsz
Last Saved Date: 2/26/2011
Factor of Safety: 0.50

Name: Slope Stability ESSA L-R



Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °

Name: Granular Backfill
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Unit Wt. Above Water Table: 120 pcf
Cohesion: 0 psf
Phi: 30 °

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna

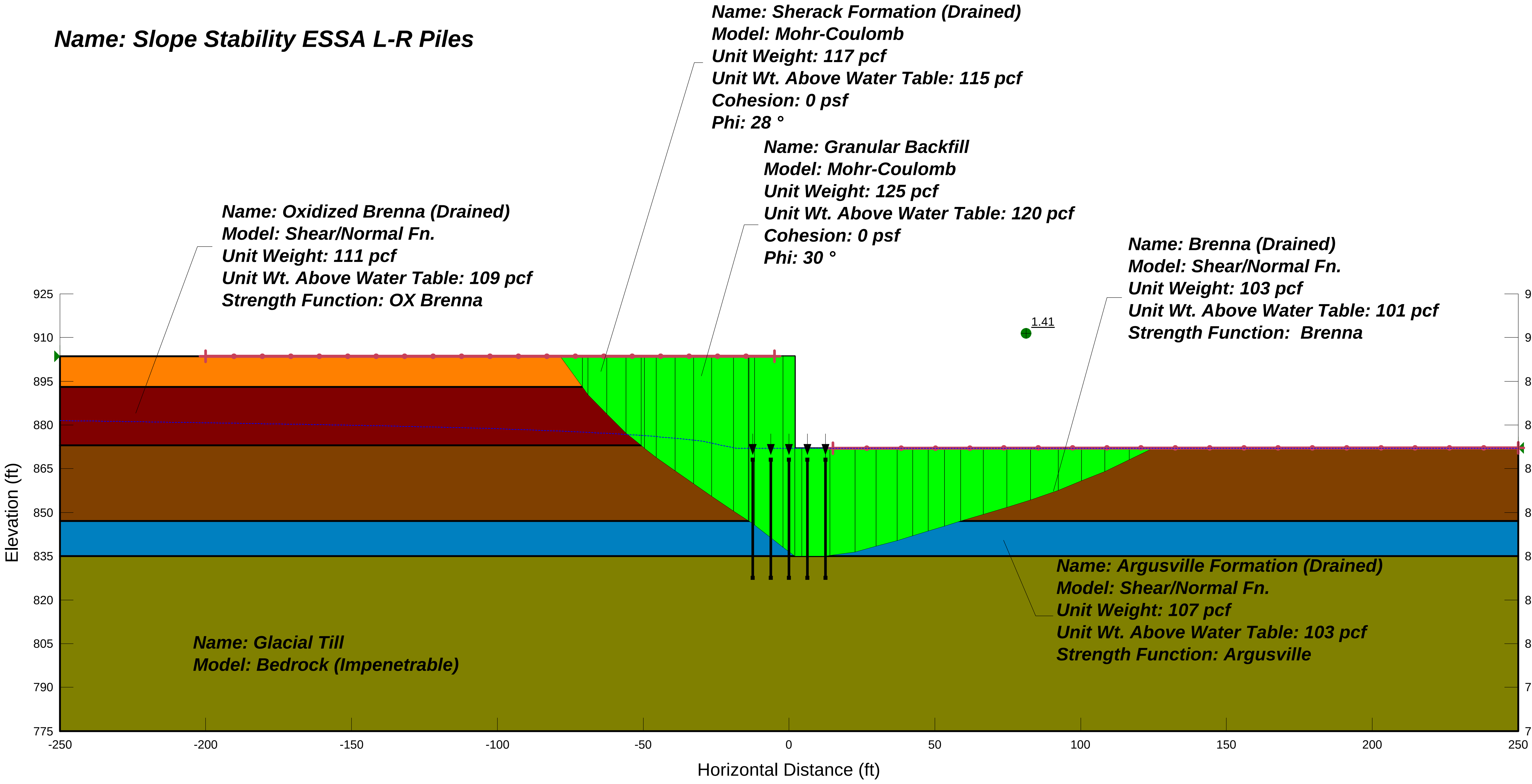
Name: Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 103 pcf
Unit Wt. Above Water Table: 101 pcf
Strength Function: Brenna

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville

Name: Glacial Till
Model: Bedrock (Impenetrable)

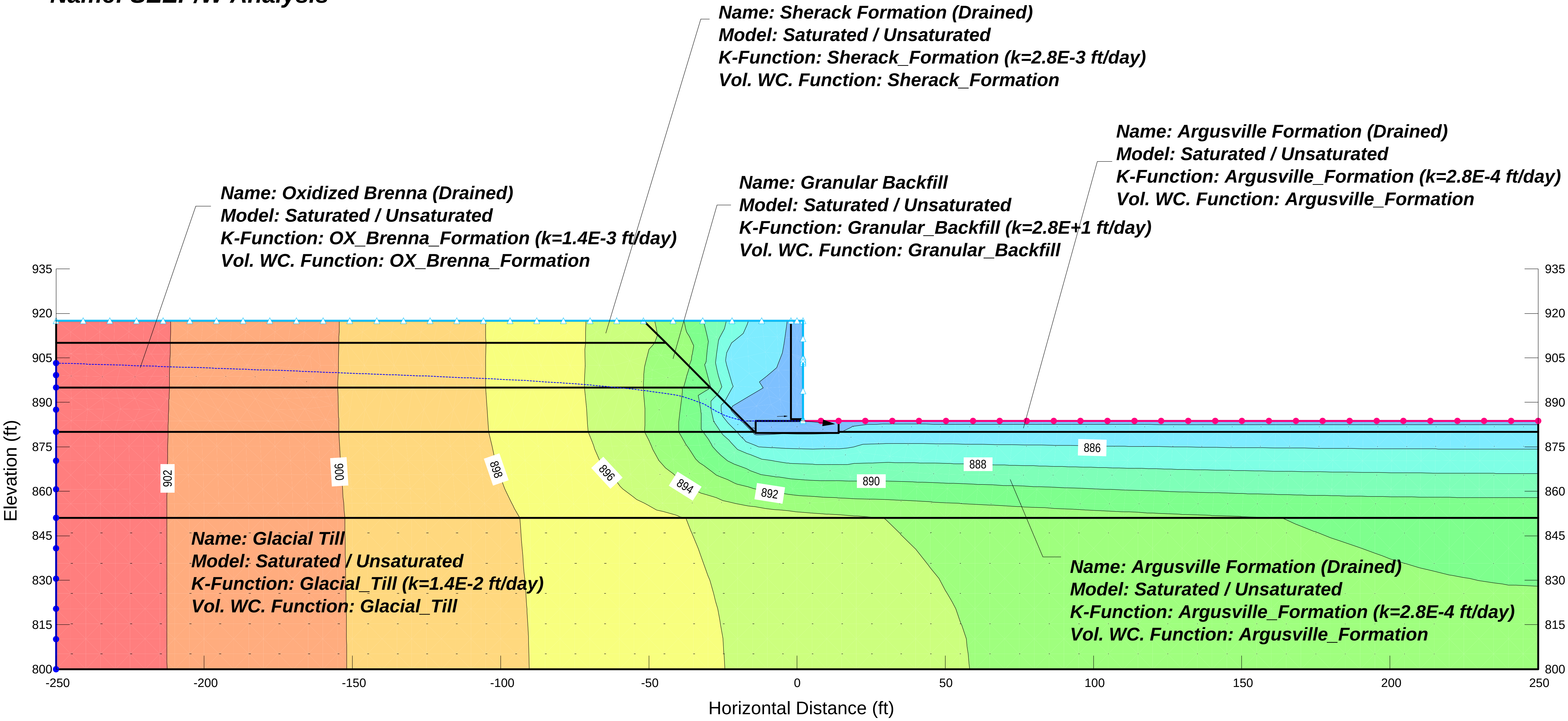
Fargo-Moorhead, Phase 4, Maple River Wall
Stability Analysis
File Name: Maple River Wall_UPDATED.gsz
Last Saved Date: 2/26/2011
Factor of Safety: 1.41

Name: Slope Stability ESSA L-R Piles



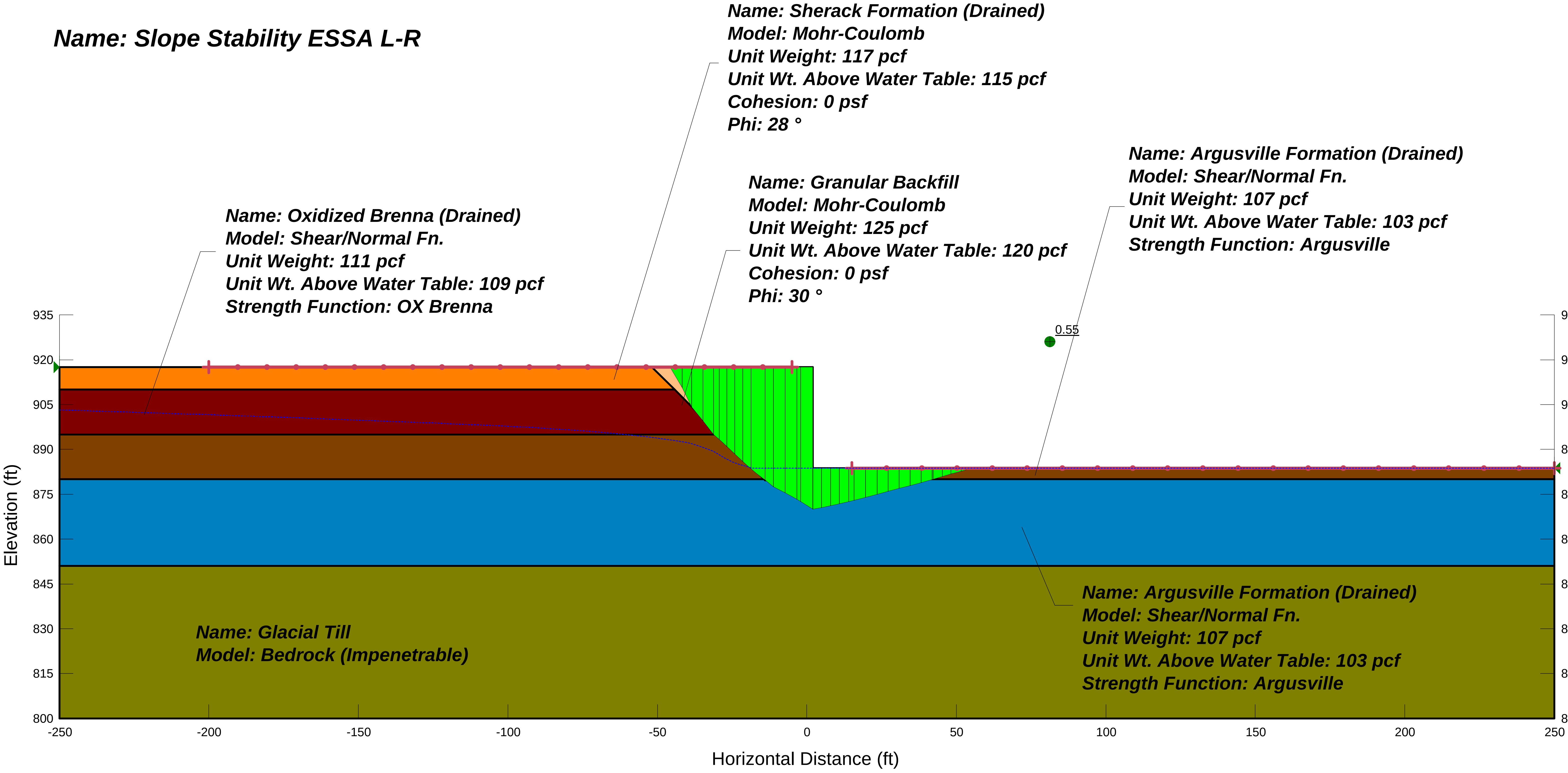
Fargo-Moorhead, Phase 4, Sheyenne River Wall
Stability Analysis
File Name: Sheyenne River Wall_UPDATED.gsz
Last Saved Date: 2/26/2011

Name: SEEP/W Analysis



Fargo-Moorhead, Phase 4, Sheyenne River Wall
Stability Analysis
File Name: Sheyenne River Wall_UPDATED.gsz
Last Saved Date: 2/26/2011
Factor of Safety: 0.55

Name: Slope Stability ESSA L-R



Fargo-Moorhead, Phase 4, Sheyenne River Wall
Stability Analysis
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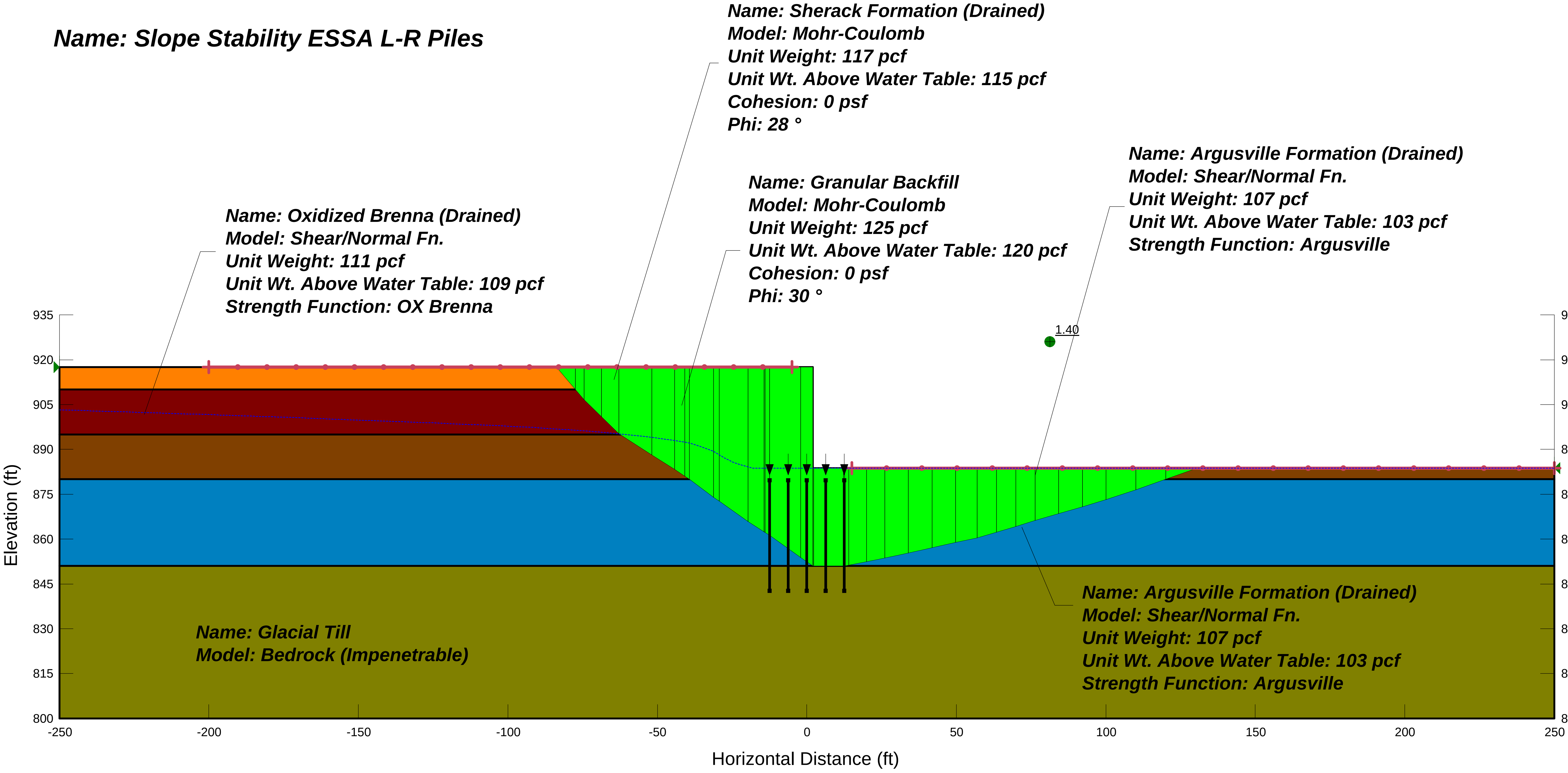
Last Saved Date: 2/26/2011
Factor of Safety: 1.40

Name: Slope Stability ESSA L-R Piles



RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT O – PILE FOUNDATIONS

Report for the US Army Corps of Engineers, and the Cities of Fargo, ND and Moorhead, MN

By: Barr Engineering Co.

FINAL – February 28, 2010

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT O – PILE FOUNDATIONS

F-O1.0 INTRODUCTION TO PILE CAPACITY ANALYSES

Deep foundations are required for support of the water control and aqueduct structures along the proposed project alignments. The structures and location along the project alignments are shown in Table F-O1. Each of the proposed structures is situated with a base foundation elevation that is at or very near the weak, compressible Brenna or Argusville formations which are incapable of supporting the heavy concrete structures. The intent of the deep foundations is to eliminate the foundation bearing loads on the weaker clays and transfer the load through the clays into a glacial till bearing layer. It is worthwhile acknowledging, however, that the pile will still develop side-resistance capacity in the clay layers. For this project, HP14X73 H-piles were used and the structural evaluation will be discussed in Appendix F.

Table F-O1 Structures Requiring Pile Foundations

Structure	Station Location
Red River Control Structure (FCP) MN Foundations and Wing walls	MN 128703
Red River Control Structure (LPP) ND Foundations and Wing walls	RRN 2529023
Wild Rice River Control Structure	WRR 66102
Sheyenne River Aqueduct Crossing Foundations	150467
Maple River Aqueduct Crossing Foundations	72543
Lower Rush (vertical drop)	60755
Rush (vertical drop)	45110
Inlet Ogee	152522
Outlet Ogee	363
Wolverton	WC 9079.5
Storage Area Outlet	WRR 75000

F-O2.0 REGIONAL FOUNDATION EXPERIENCE

Deep foundations are routinely used in this region of the country to support heavily-loaded structures due to the deeper deposits of medium to stiff yet compressible, Brenna and Argusville clay formations. Historically, heavily-loaded structures founded on shallow foundations bearing on or near these layers have performed poorly due to shear failures and long-term settlement. Because each of the proposed structures is situated with a base elevation that is at or near the weak, compressible layers formation, poor performance would result.

Commonly, deep foundations such as driven pipe-pile or H-pile are used throughout the region to support bridges or other heavily loaded structures. Typical axial allowable pile capacities, used by governmental transportation agencies in the area and other organizations, generally range from 60 to 100 tons per pile depending on length and size of pile. All piles used in this region are supported by the glacial till layer which exists below the Brenna and Argusville formations at depths ranging from 80 to 100 feet or more below the existing ground surface. The glacial till layer is very dense or hard and it has been found during investigations conducted by the US Army Corps of Engineers and through other information compiled from the Minnesota and North Dakota Departments of Transportation, that the Standard Penetration (SPT) N-values are greater than 50 blows per foot and even higher values of 100 blows per foot have been reported. Historically, sampling in the layer for laboratory testing has not been performed other than SPT samples so there is no documented strength data on the layer because only estimates of the glacial till strength can be made using the N-values.

When the piles are driven into the glacial till layer, refusal conditions generally prevail and piles can only be advanced a few feet into the glacial till. Pile penetrations in the glacial till are reportedly a maximum of 5 to 7 feet before meeting refusal. Deeper penetrations are a result of slightly weathered glacial till layers that are found above the very dense or hard layer. This indicates that the glacial till has a very high undocumented strength and the use of the undrained pile capacity based on the undrained strength provided by the US Army Corps of Engineers is conservative. Typical axial pile capacities range from 60 tons to greater than 100 tons for 9-5/8-inch to 12-3/4-inch pipe-piles or HP12X60 to HP14X89 H-piles. Often, pile designers use the structural capacity of the pile element itself when founded on the till.

Because of the stratigraphy, difficult foundation conditions, and regional familiarity of deep foundations on governmental projects, deep foundations will be also used for support of each of the structures along the project alignments. For structural reasons, an HP14X73 was used in the analysis. This decision to use the HP14X73 is discussed in Appendix F and it is consistent with many foundations constructed in the region.

F-O3.0 GEOLOGY

The Fargo-Moorhead area is covered by 200 to 300 feet of clays, tills, and granular sediments from past glaciation associated with the formation of glacial Lake Agassiz.

The Sherack Formation, Brenna Formation, Argusville Formation, and glacial till are the primary soil units in the area, though other more localized soil units have been identified such as the Poplar River Formation, Plastic Laminated Sherack Formation, and Oxidized Brenna Formation. The material types of most interest for the design of the deep foundations are the Brenna and Argusville Formations, which are relatively medium to stiff and highly plastic materials. The clay was deposited after an additional glacial event, and it is characterized as a lacustrine clay but it was altered at some locations during lower lake levels where erosion channels were filled with sediments. Due to the depth of the foundations, the Sherack and other surficial layers are of little importance. The deeper glacial till layer encountered below the Brenna and Argusville is hard to dense, depending on the constituents, and it will provide the bearing resistance for the piles. It should be noted that although the Brenna and Argusville are relatively weak compared to the glacial till layer, they are overconsolidated meaning that each layer has experienced a pressure greater than the existing overburden pressure. Overconsolidation can be caused by changes in groundwater levels and erosion of surficial soils among other mechanisms. Overconsolidation tends to affect the strength and behavior of soils in regard to foundation design and settlement.

F-O4.0 PILE CAPACITY ANALYSIS

An analysis of driven pile foundations was conducted at each structure to determine the appropriate pile length and capacity for structural design.

F-O4.1 METHODOLOGY

The main objective of the pile capacity analysis was to evaluate the axial and lateral capacity and settlement of pile foundations at each of the structures. The engineering manual, EM 1110-2-2906 “Design of Pile Foundations” (15 Jan 1991), was used as guidance for estimation of axial pile capacities. Although the manual is outdated (with regard to certain current practices) and other design methods and manuals are available, the methodology presented in this manual provided enough relevant guidance to complete the analysis. The manual requires that for the calculation of axial and lateral pile capacity both the drained and undrained strengths are used. The governing condition is then used for each case to determine the pile spacing and layout during structural design.

The undrained or short-term analysis is performed to analyze the case in which loading or unloading is applied rapidly and excess pore-water pressures do not have time to dissipate during load application. This scenario typically applies to loading from transient conditions such as floods where the load is applied for a short period of time relative to the time that takes excess pore-water pressure dissipation which is related to the permeability of the soils.

The drained, effective stress, or long-term analysis is performed to account for much slower loading or unloading, or no external loading, in which the drained shear strength of the materials is mobilized and shear induced pore-water pressures are not generated. The dead load of the structure is considered a long-term case.

Similar to the axial analysis, the drained and undrained lateral capacity of the piles were also computed for use in resisting the lateral design loads. The capacity was determined for the piles at three acceptable displacements which were established as the lateral movement criteria. The criteria are 0.5 inches, 0.67, and 0.875 inches of lateral movement. The criteria were developed based on structural requirements discussed in other sections of Appendix F.

The pile settlement under working loads was also estimated at each of the structures for a single pile. For the detailed design, it is recommended that the capacity of the piles be adjusted to account for group effects.

F-O4.2 SOFTWARE

Three different design software packages were used to facilitate the calculation of pile capacities and settlement. Each software and calculation methods are discussed in the following sections.

AllPile

The program, AllPile version 7 analyzes pile load capacity efficiently and accurately. AllPile can handle all types of piles: drilled shaft, driven pile, augercast pile, steel pipe pile, H-pile, timber pile, tapered pile, bell pile, shallow foundation, etc. The program can define new pile types and use customized input parameters based on local practices and experience. The program is capable of performing calculations for lateral capacity and deflection, axial capacity and settlement. The lateral capacity calculation uses COM624S, which is the same method as FHWA's COM624P and is comparable with Ensoft's Lpile. The settlement is calculated using the Vesic methodology.

APILE

The APILE software, developed by Ensoft Inc., is used to compute the axial capacity, as a function of depth, of a driven pile in clay, sand, or mixed-soil profiles. The main computational methods used by *APILE* are those established by the American Petroleum Institute (API) in their manual on recommended practice, API-RP2A. The procedures from API have been adopted by a number of organizations.

In addition to computations based on API-RP2A criteria, APILE offers alternative methods for computing the axial-capacity of driven piles, including: (i) U.S. Army Corps of Engineers (USACE) method, (ii) U.S. Federal Highway Administration (FHWA) method, and (iii) the Lambda-Method.

The APILE program also provides some flexibility in allowing users to specify any set of values for load transfer in side resistance and end bearing as a function of depth. This feature is useful for cases when site measurements were made from instrumented axial load tests.

LPILE

The LPILE Version 5 software, developed by Ensoft Inc., is used to compute the lateral capacity of pile foundations in varying stratigraphy. LPILE uses the p-y method to calculate the shear, moment, and deformation of the pile under various load conditions. LPILE is an updated version of lateral pile analysis compared to AllPile which uses COM624.

F-O4.3 DESIGN STRENGTH PARAMETERS

Geotechnical data and a summary of strength parameters for each soil layer were provided by the St. Paul District of the US Army Corps of Engineers. The Corps has completed a preliminary geotechnical investigation program along the project alignments to evaluate the strength of the soils for slope stability modeling. These strength parameters vary significantly from the parameters required for pile capacity analyses. The soil shear strength parameters (failure envelopes) used in slope stability analyses were defined using the large strain failure criterion to incorporate effect of softening phenomenon applicable to slope stability problems. The pile capacity analyses methods are based on the use of peak strengths for the drained and undrained strength parameters and therefore a reanalysis of the geotechnical data was required. However, the unit weight of the soils provided by the Corps for the stability analyses were considered adequate for the purposes of the pile analyses. The revised design parameters are summarized in Exhibit L of Appendix F.

F-O4.3.1 Peak Undrained Strength Parameters

The peak undrained strength of the Brenna and Argusville plays an important role in the pile capacity calculations because these formations occur along the entire side of the pile and provide significant side resistance. Therefore, it was necessary to develop the peak parameters from the soil test results performed by the USACE. The process to evaluate the strength has been discussed in Exhibit L of Appendix F and the results presented indicated that at the depths of the pile foundation, the undrained shear strength of the Brenna formation is about 900 psf. The strength of the Argusville formation was slightly higher at 950 psf. Other soil layers encountered less frequently were the Sherack and the West Fargo. These parameters were also estimated based on the strength results by the USACE and are shown in Table F-O2.

The peak undrained shear strength of the glacial till layer was more difficult to define due to the lack of strength testing of this material. Standard penetration tests indicate the material is generally medium dense to very dense or hard with N-values ranging from 30 to over 100 blows per foot. The N-values relate to an undrained shear strength of about 7500 psf. Slightly lower strength values were observed in the upper, weather glacial till layers and are accounted for by a relationship with N-values. Table F-O2 presents the design undrained strength parameters for the preliminary pile capacity analyses.

F-O4.3.2 Peak Drained Strength Parameters

The analysis of long-term or drained pile capacity requires the friction angle of the soil material and the use of the “sand” design methodology. Therefore, the friction angles of the Brenna and Argusville formations are important to the evaluation of pile capacity for the long-term condition. The analysis of these two layers was discussed previously in Exhibit L of Appendix F. The glacial till layer is represented by friction angles from actual laboratory tests. The strength of the till is expected to vary based on the degree of weathering and was modeled using two till layers. The first is the top layer that simulates the weathered till (about 5 ft thick) and the second is the non-weathered till underneath it. The results of the analysis and the design parameters in provided in Table F-O2.

Table F-O2 Design Soil Strengths

Fargo Moorhead Flood Control Preliminary Pile Design Parameters			
Formation	**Unit Weight**	**Selected Shear Strength Parameters**	
	γ_m (pcf)	**Effective Friction Angle ϕ'**	**Undrained Shear Strength Su (psf)**
Sherack	117.6	30	1000
West Fargo Poplar River	123.2	30	3000
Oxidized Brenna	111.3	25	1000
Brenna	104.1	25	800~900
Argusville	106.5	26	950
Weathered Till	130	31	6000(1)
Till	130	34	7500

(1) Weathered till surficial layer generally 5 feet thick.

Table F-O3 Lateral Pile Design Parameters

Fargo Moorhead Flood Control Lateral Pile Design Parameters			
Formation	**Undrained**		**Drained**
	c (psi)	**ε_{50} (%)**	**k (pci)**
Sherack	6.9	0.01	60
West Fargo Poplar River	6.9	0.01	60
Oxidized Brenna and Brenna	5.55-6.9	0.01	20
Argusville	6.6	0.01	20
Till	41.7-52	0.004	125

F-O4.3.3 Properties for Lateral Loading Analyses

Both the drained and undrained soil properties were estimated for the lateral pile analyses using documentation provided in the AllPile 7 software manual or LPile Version 5. The correlations for the undrained lateral load parameters were based on undrained shear strength of the soils. For the drained strength parameters, correlations between N-values and friction angle were used to determine the moduli.

F-O5.0 SITE CONDITIONS

As discussed in section F-O3.0, there are about three to four soil layers at each site. For the purpose of the deep foundations analyses each site was reviewed along the project profile to determine the individual soil layers that exist below the foundation level. Generally, the foundation levels of the structures are at or very near the top of the Brenna layer except in one case where the Argusville was encountered at the foundation level. Where found at each of the locations, the Brenna is about 10 to 30 feet thick. Where the Argusville exists, it is about 18 feet thick.

F-O6.0 RESULTS

The results of the axial pile capacity calculations are summarized in Table F-O4. Table F-O4 includes the ultimate axial pile capacity and skin friction under drained and undrained conditions for individual piles. These pile capacities were computed using the design methodology previously described. Pile capacity calculations under undrained conditions used the clay-type soil modeling in the software. Pile capacity calculations under drained conditions used the sand-type soil modeling in the computational software.

The results of the axial pile capacity analyses are shown in Table F-O4. The table shows the range in capacity for each of the pile capacity analysis methods. Considerable variation is observed in the results as expected due to the differences in calculation methodology utilized in each analysis method. At each structure the axial pile capacity was computed and reported. The design pile capacity was calculated by computing the average between the maximum and minimum capacity values computed from the four calculation methods. This reporting procedure for the pile capacity was requested by the USACE and was used to reduce the sometimes large range difference in estimated/computed pile capacities for this preliminary evaluation.

Table F-O4 Summary of Individual Pile Capacities by Method

		Ultimate Drained Capacity(1)	**Skin Friction Drained**	**Ultimate Undrained Capacity(1)**	**Skin Friction Undrained**
	Analysis Method	**(kips)**	**(kips)**	**(kips)**	**(kips)**
Phase 3 Analysis RRN ND	FHWA (Apile)	137.6	36.1	268.7	175.5
	USACE (Apile)	95.5	31	267.7	174.5
2' into hard till	Lambda 2 (Apile)	157.5	40.8	213.7	120.5
	API RP-2A (Apile)	158.9	42.2	209.5	127.7
	Navfac (Allpile)	120.2		237.4	
	Average (2)	**127.2**	**36.6**	**239.1**	**148**
Phase 4 Analysis RRN ND	FHWA (Apile)	125.8	35.9	267.8	174.6
	USACE (Apile)	86.7	30.7	274.2	181.0
	Lambda 2 (Apile)	163.0	41	212.8	119.6
	API RP-2A (Apile)	164.3	42.4	221.3	128.1
	Average (2)	**125.5**	**36.6**	**243.5**	**150.3**
Phase 3 Analysis Wild Rice	FHWA (Apile)	206.5	66.4	339.6	220.5
	USACE (Apile)	107.5	43.1	321.3	196.7
1' into hard till	Lambda 2 (Apile)	227	66	252.7	128.9
	API RP-2A (Apile)	242.2	81.3	265.6	172.4
	Navfac (Allpile)	153.4		283.6	
	Average (2)	**174.85**	**62.2**	**296.15**	**174.7**
Phase 4 Analysis Wild Rice	FHWA (Apile)	173.8	71.8	349.9	256.7
	USACE (Apile)	101.1	45.1	349.4	256.2
	Lambda 2 (Apile)	237.6	67.2	269.0	175.8
	API RP-2A (Apile)	255.9	85.5	290.0	196.8
	Average (2)	**178.5**	**65.3**	**309.5**	**216.3**
Phase 3 Analysis Sheyenne	FHWA (Apile)	165.5	49.3	296.2	203.1
	USACE (Apile)	101.5	37	260.8	167.6
3' into hard till	Lambda 2 (Apile)	183.4	49.9	199.3	106.1
	API RP-2A (Apile)	190.7	57.2	209.7	116.6
	Navfac (Allpile)	131.4		234.9	

		Ultimate Drained Capacity(1)	**Skin Friction Drained**	**Ultimate Undrained Capacity(1)**	**Skin Friction Undrained**
	Analysis Method	**(kips)**	**(kips)**	**(kips)**	**(kips)**
	Average (2)	**146.1**	**47.1**	**247.75**	**154.6**
Phase 4 Analysis Sheyenne	FHWA (Apile)	145	45.5	287.1	193.9
	USACE (Apile)	90.6	34.6	291.3	198.1
	Lambda 2 (Apile)	185.5	49.4	220.0	126.8
	API RP-2A (Apile)	191.1	54.8	234.1	140.9
	Average (2)	**140.9**	**44.7**	**255.7**	**169.5**
Phase 3 Analysis Maple River	FHWA (Apile)	133.3	35	261.3	168.1
	USACE (Apile)	97.8	33.3	252	158.8
5' into hard till	Lambda 2 (Apile)	153.1	40.1	196.1	102.9
	API RP-2A (Apile)	153.6	40.7	202.5	109.3
	Navfac (Allpile)	120.2		228	
	Average (2)	**125.7**	**37**	**228.7**	**135.5**
Phase 4 Analysis Maple River	FHWA (Apile)	136.6	41.5	278.5	185.3
	USACE (Apile)	92.6	36.6	261.1	167.9
	Lambda 2 (Apile)	177.1	48	198.4	105.3
	API RP-2A (Apile)	179.6	50.5	207.3	114.1
	Average (2)	**136.1**	**43.5**	**238.5**	**145.3**
Phase 3 Analysis Lower Rush	FHWA (Apile)	229.3	82.7	373.1	279.9
	USACE (Apile)	113.3	48.9	322	229
1' into hard till	Lambda 2 (Apile)	238.7	69	254.8	161.6
	API RP-2A (Apile)	262.8	93.1	269.3	176.1
	Navfac (Allpile)	153.4		283.8	
	Average (2)	**188.05**	**71**	**313.95**	**220.75**
Phase 4 Analysis Lower Rush	FHWA (Apile)	184.2	82.7	374.6	281.4
	USACE (Apile)	104.2	48.2	384.7	291.6
	Lambda 2 (Apile)	251.9	71.4	303.2	210
	API RP-2A (Apile)	275.8	95.3	324.9	231.7
	Average (2)	**190**	**71.8**	**343.9**	**250.8**

		Ultimate Drained Capacity(1)	**Skin Friction Drained**	**Ultimate Undrained Capacity(1)**	**Skin Friction Undrained**
	Analysis Method	**(kips)**	**(kips)**	**(kips)**	**(kips)**
Phase 3 Analysis Rush River	FHWA (Apile)	232.3	84.1	212.7	287.7
	USACE (Apile)	109.9	45.4	205.9	212.7
3' into hard till	Lambda 2 (Apile)	245.2	70.6	238.5	145.4
	API RP-2A (Apile)	270.6	96	256.6	163.5
	Navfac (Allpile)	153.4		269.9	
	Average (2)	**190.25**	**70.7**	**237.9**	**216.55**
Phase 4 Analysis Rush River	FHWA (Apile)	184.6	83.1	379.1	285.9
	USACE (Apile)	100	44	413.2	320.1
	Lambda 2 (Apile)	256.2	71.5	323.1	229.9
	API RP-2A (Apile)	281.1	96.3	352.0	258.8
	Average (2)	**190.6**	**70.1**	**368.2**	**275.0**
Phase 4 Analysis Wolverton Structure	FHWA (Apile)	480.4	379.0	618.8	525.8
	USACE (Apile)	255.2	178.7	549.4	456.4
5' into hard till	Lambda 2 (Apile)	725.0	380.4	499.5	406.5
	API RP-2A (Apile)	817.2	472.6	543.4	450.4
	Average (2)	**536.2**	**325.7**	**559.2**	**566.2**
Phase 4 Analysis Inlet Structure	FHWA (Apile)	119.3	32.1	255.4	162.4
	USACE (Apile)	83.6	28.1	256.3	163.3
1' into hard till	Lambda 2 (Apile)	158.1	39.8	193.9	100.8
	API RP-2A (Apile)	157.7	39.5	203	110
	Average (2)	**120.9**	**33.9**	**225.1**	**132.1**
Phase 4 Analysis Outlet Structure	FHWA (Apile)	214.3	113	417.3	324.3
	USACE (Apile)	113.3	57.8	396.4	303.4
5' into hard till	Lambda 2 (Apile)	313.1	102	310.8	217.8
	API RP-2A (Apile)	344.7	133.5	339.3	246.2
	Average (2)	**229.0**	**95.7**	**363.8**	**211.0**

	Analysis Method	Ultimate Drained Capacity$^{(1)}$ (kips)	Skin Friction Drained (kips)	Ultimate Undrained Capacity$^{(1)}$ (kips)	Skin Friction Undrained (kips)
Phase 4 Analysis Storage Area 1 Structure	FHWA (Apile)	181.6	80.3	360.6	267.6
	USACE (Apile)	101.4	44.9	328.4	235.3
0' into hard till	Lambda 2 (Apile)	262.5	78.9	246.0	152.9
	API RP-2A (Apile)	283.5	99.8	272.8	179.7
	Average (2)	**192.5**	**72.4**	**303.3**	**210.3**

(1) Pile capacity shall be the minimum of the soil capacity as reported in the table or the structural capacity of the pile with loads applied.
(2) The reported average capacity represent the average between the maximum and minimum capacity computed from the four calculation methods.

A pile tip penetration was estimated and is reported for each structure and ranges from about 1 to 5 feet of embedment into the hard or very dense glacial till bearing layer. Additional penetration during driving is unlikely due to hard driving conditions which could damage the pile or driving equipment.

A detailed summary of the pile capacities at each structure is presented in Exhibit O Table F-O6 and should be used for the preliminary design and layout. The pile tip elevation is presented in the table and the total length ranges from 25 to 116 feet depending on the structure foundation depth and depth to the glacial till layer. The undrained capacities are similar to those used in regional pile design when back-calculations are performed to compare and check skin friction and adhesion as well as end bearing of the piles. Since most of the piles for this project are much shorter, by 40 to 50 feet, than most bridge foundation piles, the skin friction component is much less than for bridge foundations but the end bearing is similar for the same size pile. The ultimate drained or long-term axial pile capacities ranged from 125.7 to greater than 500 kips and are significantly less than the capacities determined for the undrained analysis. It should be noted that these analyses are based on the ultimate soil capacity and the structural capacity of the pile section may govern in some instances depending on the axial load and moment applied computed.

A preliminary sensitivity analysis was performed at the Wild Rice River structure to evaluate the effect of upward or downward groundwater flow on the axial pile capacity. The upward and downward flow primarily affects the drained pile capacity. The analysis using the Allpile software used an assumed value of 10 feet head above or below the estimated static level at the structure location. In order to evaluate this condition using the software, the unit weights were adjusted to account for increased effective unit weights where the groundwater flow was downward. In the case where upward flow was simulated, a reduced effective stress was applied in each soil layer. The ultimate static drained capacity of the pile foundation is 153.4 kips. Using this methodology described the effect of upward flow reduces the ultimate capacity to 111.6 kips and the effect of downward flow increases the ultimate capacity to about 195.3 kips. This results in an increase or decrease from the static capacity in the drained condition as shown in Table F-O5.

Table F-O5 Sensitivity Analysis of Assumed Groundwater Conditions at Wild Rice River Control Structure (Drained Analysis)

Flow Condition	Head Conditions	Ultimate Axial Pile Capacity (kips)	% Difference from Static Level
Upward Flow	+10 feet of head	111.6	73%
Existing Conditions	-	153.4	0
Downward Flow	-10 feet of head	195.3	127%

The lateral pile capacities were computed at allowable pile top movement for a fixed head condition. The fixed head condition is appropriate due to the thickness and size of the pile caps and mats. The capacities were evaluated at the design pile length and for lateral deflections of 0.5 inches, 0.67 inches, and 0.875 inches which represent potential different structural load case scenarios. The results of the analyses show that lateral pile capacities range from 35 to 48 kips for the undrained loading condition. For the drained loading condition, the lateral pile capacities range from 21 to 36 kips.

Negative skin friction could occur due to the existence of the softer, compressible Brenna and Argusville formations under foundations if fill is placed adjacent to the structures in excess of the existing ground surface. The amount and extent of the negative skin friction varies depending on the individual site conditions. The effect of negative skin friction should be evaluated on a case by case basis during final design.

F-O7.0 RECOMMENDATIONS FOR FINAL DESIGN

The pile capacities presented for the drained or long-term condition are less than are typically used in the region. Additional sampling and testing is recommended to validate the assumed strength parameters of the glacial till. Such sampling could consist of Pitcher barrel sampling to cut undisturbed samples from the glacial till. The samples should be tested in both drained and undrained triaxial tests with pore pressure measurements. It is likely these tests will show an increase in available toe resistance within the glacial till.

Pile load tests are also recommended prior to final design. The soil formations located in this area are susceptible to strength gain over time otherwise called pile setup. A long-term load test concept should be developed to install test piles at the structures and then load them dynamically and statically to evaluate both the short-term and long-term load carrying capabilities. These tests could show that pile set up and strength gain could contribute additional capacity that reduces the number of piles and increases their spacing.

Table F-O6 Design Pile Capacities

Fargo-Moorhead Food Control Structures
Preliminary Pile Foundation Analyses
HP 14X73

A_{tip} = 198.5 in^2, A_{steel} = 21.4 in^2, perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in^4

Structure	Diversion Channel Station Location	Approximate Ground (Bank) Surface Elevation (ft)	Invert Elevation (ft)	Estimated Foundation Elevation (ft)	Estimated Ground Water Elevation (ft)	Design Condition/Tip Elevations			Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips) (fixed head - single pile)			Estimated Settlement at allowable load
										0.5”	0.67”	0.875”	
Red River Control Structure (LPP) ND Foundations and Wing walls	RRN 2529023	906	875.51	869.51	876.7	Phase 3	Undrained Analysis 835.8’	Total	239.1	35	41	48	<0.5”
								Uplift Resistance	148				
							Drained Analysis	Total	127.2	22	28	35	
								Uplift Resistance	36.6				
						Phase 4	Undrained Analysis 835.8’	Total	243.5	-			
								Uplift Resistance	150.3				
							Drained Analysis	Total	125.5				
								Uplift Resistance	36.6				
Red River Control Structure (FCP) MN Foundations and Wing walls PHASE 3	MN 128703	Right 906 Left 893	861	857	880.17	Undrained Analysis 815’		Total	269.4	35	41	47	<0.5”
								Uplift Resistance	176.2				
						Drained Analysis		Total	141.6	21	29	33	
								Uplift Resistance	26.9				
Wild Rice River Control Structure	WRR 66102	913	890.8	884.80	908	Phase 3	Undrained Analysis 836.8’	Total	296.2	35	41	48	<0.5”
								Uplift Resistance	174.7				
							Drained Analysis	Total	174.9	23	29	34	
								Uplift Resistance	62.2				
						Phase 4	Undrained Analysis 836.8’	Total	309.5	-			
								Uplift Resistance	216.3				
							Drained Analysis	Total	178.5				
								Uplift Resistance	65.3				

Fargo-Moorhead Food Control Structures
Preliminary Pile Foundation Analyses
HP 14X73

A_{tip} = 198.5 in^2, A_{steel} = 21.4 in^2, perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in^4

Structure	Diversion Channel Station Location	Approximate Ground (Bank) Surface Elevation (ft)	Invert Elevation (ft)	Estimated Foundation Elevation (ft)	Estimated Ground Water Elevation (ft)	Design Condition/Tip Elevations			Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips) (fixed head - single pile)			Estimated Settlement at allowable load
										0.5”	0.67”	0.875”	
Sheyenne River Aqueduct Crossing Foundations	150467	916	883.68	879.68	908	Phase 3	Undrained Analysis 842.8’	Total	247.8	36	41	48	<0.5”
								Uplift Resistance	154.6				
							Drained Analysis	Total	146.1	23	29	36	
								Uplift Resistance	47.1				
						Phase 4	Undrained Analysis 842.8’	Total	255.7	-			
								Uplift Resistance	169.5				
							Drained Analysis	Total	140.9				
								Uplift Resistance	44.7				
Maple River Aqueduct Crossing Foundations	72543	898	872.06	868.06	893	Phase 3	Undrained Analysis 827.5’	Total	228.7	36	42	48	<0.5”
								Uplift Resistance	135.5				
							Drained Analysis	Total	125.7	23	29	33	
								Uplift Resistance	37				
						Phase 4	Undrained Analysis 827.5’	Total	238.5	-			
								Uplift Resistance	145.3				
							Drained Analysis	Total	136.1				
								Uplift Resistance	43.5				

Fargo-Moorhead Food Control Structures
Preliminary Pile Foundation Analyses
HP 14X73

A_{tip} = 198.5 in^2, A_{steel} = 21.4 in^2, perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in^4

Structure	Diversion Channel Station Location	Approximate Ground (Bank) Surface Elevation (ft)	Invert Elevation (ft)	Estimated Foundation Elevation (ft)	Estimated Ground Water Elevation (ft)	Design Condition/Tip Elevations			Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips) (fixed head - single pile)			Estimated Settlement at allowable load
										0.5”	0.67”	0.875”	
Lower Rush River Drop Structure	60755	891.5	872.2	860.7	885	Phase 3	Undrained Analysis 809.4’	Total	314	35	42	48	<0.5”
								Uplift Resistance	220.8				
							Drained Analysis	Total	188	22	26	33	
								Uplift Resistance	71.0				
						Phase 4	Undrained Analysis 809.4’	Total	343.9	-			
								Uplift Resistance	250.8				
							Drained Analysis	Total	190				
								Uplift Resistance	71.8				
Rush River Drop Structure	45110	889	869.6	858.1	885.3	Phase 3	Undrained Analysis 806.9’	Total	237.9	35	42	48	<0.5”
								Uplift Resistance	216.6				
							Drained Analysis	Total	190.3	23	28	34	
								Uplift Resistance	70.7				
						Phase 4	Undrained Analysis 806.9’	Total	368.2	-			
								Uplift Resistance	275.0				
							Drained Analysis	Total	190.6				
								Uplift Resistance	70.1				
Storage Area 1 Outlet PHASE 4	WRR 75000			900+/-	908+/-	Undrained Analysis 845’		Total	303.3	-	-	-	-
								Uplift Resistance	210.3				
						Drained Analysis		Total	192.5	-	-	-	
								Uplift Resistance	72.4				

Fargo-Moorhead Food Control Structures
Preliminary Pile Foundation Analyses
HP 14X73

A_{tip} = 198.5 in^2, A_{steel} = 21.4 in^2, perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in^4

Structure	Diversion Channel Station Location	Approximate Ground (Bank) Surface Elevation (ft)	Invert Elevation (ft)	Estimated Foundation Elevation (ft)	Estimated Ground Water Elevation (ft)	Design Condition/Tip Elevations		Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips) (fixed head - single pile)			Estimated Settlement at allowable load
									0.5”	0.67”	0.875”	
Diversion Inlet Structure PHASE 4	152522	918.3	884	880	910.3	Undrained Analysis 845’	Total	225.1	30	35	41	<0.5”
							Uplift Resistance	132.1				
						Drained Analysis	Total	120.6	22	27	35	
							Uplift Resistance	33.9				
Diversion Outlet Ogee Structure PHASE 4	363	881	843.9	840	878.7	Undrained Analysis 778’	Total	363.8	35	41	48	<0.5”
							Uplift Resistance	211.0				
						Drained Analysis	Total	229.0	22	28	35	
							Uplift Resistance	95.7				
Wolverton Structure PHASE 4	WC 9079.5		895.3	891	898.4	Undrained Analysis 775’	Total	559.2	37	43	50	<0.5”
							Uplift Resistance	566.2				
						Drained Analysis	Total	536.2	28	35	42	
							Uplift Resistance	325.7				

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT P – STRUCTURAL DESIGN CRITERIA

Report for the US Army Corps of Engineers, and the cities of Fargo, ND and Moorhead, MN

By: BARR ENGINEERING

FINAL – February 28, 2011

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ATTACHMENTS

F-P2	Section 10.7.2.4 – Horizontal Pile Foundation Movement – from the AASHTO LRFD Bridge Design Specifications Manual
F-P3	June 7, 2010 Memorandum from Paul Nielsen and Miguel Wong, Barr Engineering to Tony Fares, Corps of Engineers MVP entitled Structural Design Criteria
F-P4	Emails between Barr Engineering and Corps of Engineers MVP discussing structural design criteria

APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT P – STRUCTURAL DESIGN CRITERIA

F-P1.0 STRUCTURAL DESIGN CRITERIA

The attachments to this Exhibit P include:

- Emails between Barr Engineering and Corps of Engineers MVP discussing structural design criteria – February 2011
- Emails between Barr Engineering and Corps of Engineers MVP discussing structural design criteria – June and July 2010
- June 7, 2010 Memorandum from Paul Nielsen and Miguel Wong, Barr Engineering to Tony Fares, Corps of Engineers MVP entitled Structural Design Criteria
- Section 10.7.2.4 – Horizontal Pile Foundation Movement – from the AASHTO LRFD Bridge Design Specifications Manual

From: Miguel Wong
Sent: Tuesday, February 01, 2011 9:54 AM
To: 'Fares, Tony S MVP'; 'Hokens, Kent D MVP'; Buesing, Aaron W MVP; Brian K. LeMon; Tor S. Hansen; Paul Nielsen; Mustafa B. Igdelioglu
Cc: Aaron Grosser; Jed Greenwood; Brandon J. Barnes
Subject: FW: Ice forces on structures (UNCLASSIFIED)

fyi

Miguel Wong, PE, PhD

Senior Water Resources Engineer
Minneapolis office: 952.832.2632 | cell: 612.703.6562 mailto:mwong@barr.com www.barr.com

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-----Original Message-----
From: Tuthill, Andrew M ERDC-CRREL-NH [mailto:Andrew.M.Tuthill@usace.army.mil]
Sent: Tuesday, February 01, 2011 9:52 AM
To: Miguel Wong
Subject: Ice forces on structures (UNCLASSIFIED)

Classification: UNCLASSIFIED
Caveats: NONE

Miguel:

Here is a preliminary answer to the ice force question. The 5000 psf (34.7 psi) ice pressure with a 2-ft-thick ice cover is equivalent to 10 kips per lineal ft, a commonly used design number for static ice loads on rigid structures due to thermal expansion of an ice cover. It is usually applied uniformly to long structural elements such as the crest of a dam or a lock wall. I assume that the 100 and 500 year flood discharges would be used to estimate the elevations that these ice loads would be expected to act on the structures.

In terms static ice forces, the approach may be overly conservative because, as the stage rises, the ice cover progressively thins and weakens. The 2009 flood, which is approximately the 100-year event, is an example. The photos taken a the time of peak show very little remaining ice along the RRN or tribs. Since the near-peak channel is so much wider than the freezeup channel, the remaining ice is unconfined by the banks or structures, so static ice forces would not apply. The 5000 psf x 2-ft-thick ice would be an appropriate structural design number at freezeup discharges and water levels that are well within banks.

At higher water levels, dynamic forces due to crushing or bending failure of ice floes would govern. These ice pressures are usually higher than the static ones but applied to smaller areas such as the pier noses. Calculation methods are described in the AASHTO (1998) "LRFD Bridge Design Specifications" which are excerpted below along with typical ice strength values. This loading scenario assumes an environmental driving force such as current and/or wind sufficient to push a large ice floe against the structure and cause failure either in bending or crushing. In case of the RRN spring floods, t and p both likely decrease as the water level rises. For now we could use the following:

-For late winter conditions at the start of the rise, one could use t = 2 ft and p = 167 psi ("where breakup or major ice movement occurs at melting temperatures but the ice moves in large pieces and is internally sound")

-By the time the stage reaches top of bank (2-year discharge ?), one could assume that the ice cover had thinned to 1 ft and the ice had weakened to 111 psi (" where breakup occurs at melting temperatures and the ice structure is somewhat disintegrated:)

-Once the river is well out of bank, dynamic ice forces would likely be negligible since the ice "structure would be substantially disintegrated". We could assume p <= 56 psi and t<= 1 ft. Here the ice forces on the structures would probably result not from ice failure but from ice accumulating upstream due to driving forces of water current and wind drag. Forces exerted by accumulated ice can be calculated from drag equations and would be much less
than static or dynamic ice forces.

This is just a start. We can develop better relationships between ice thickness, ice strength and river stage by comparing the results of the ice thickness model (based on historic air temperature) with historic stage hydrographs. This will improve our estimate of ice forces as water levels rise under melting conditions.

Please call if you have questions or would like to discuss.

Thanks,

Andy

Andrew M. Tuthill, P. E.
U S Army Cold Regions Research and Engineering Laboratory
72 Lyme Rd.
Hanover, NH 03755
603-646-4225 office
603-643-3354 home
603-306-6699 cell

Classification: UNCLASSIFIED
Caveats: NONE

Classification: UNCLASSIFIED
Caveats: NONE

Paul Nielsen

From: Tor S. Hansen
Sent: Tuesday, February 01, 2011 2:39 PM
To: 'tony.s.fares@mvp02.usace.army.mil'; 'kent.d.hokens@usace.army.mil'; 'Aaron.W.Buesing@usace.army.mil'
Cc: Mustafa B. Igdelioglu; Miguel Wong; Paul Nielsen; Brian K. LeMon; Aaron Grosser
Subject: Summary of Conclusions from February 1 structural meeting for Fargo- Moorehead Metro Flood Risk Management Project Phase 4

All:

Below is Barr's summary of today's meeting regarding load cases, ice loads, and structure design elevations for the structures associated with the Fargo-Moorehead Flood Risk Management Project for Phase 4. If you disagree with any of the following conclusions please let me know. Also, please let me know if there is anything I have forgotten or not included from the meeting today. The following conclusions will govern the design of the structures for Phase 4 of the project.

Table 1 – Gated Structures Summary of Load Cases

Load Case	Event Category	Allowable Pile Deflection (inches)	Factor of Safety for Piles	Soil Condition
1 – 100 yr flood	Usual	0.67 (note 1)	2.00	Undrained
1.1 – 100 yr + ice (note 4 and 5)	Unusual	0.875 (note 2)	1.50	Undrained
2 – 500 yr flood	Unusual	0.875 (note 2)	1.50	Undrained
2.1 – 500 yr + 5ft	Extreme	1.000 (note 3)	1.15	Undrained
3 – construction	Unusual	0.67	1.50	Undrained
4 – Normal low flow	Usual	0.50	2.00	Drained

Note 1: It was agreed that an allowable deflection of 0.67-inches (as opposed to 0.50 inches) is acceptable even though this is considered a usual load case.
Note 2: It was agreed that an allowable deflection of 0.875-inches (as opposed to 0.67 inches) is acceptable even though this is considered an unusual load case.
Note 3: It was agreed that for this extreme event an allowable deflection of 1.0-inch is acceptable.
Note 4: Ice loads on the gated structure during the 100 year flood will be considered as dynamic forces due to crushing or bending of ice floes as provided by Andrew Tuthill from the USACE via email to Miguel Wong dated February 1, 2011 at 9:52AM. These loads will be applied to the piers.
Note 5: An ice/debris load of 500 PLF along the structure will be used for the wing and retaining wall structures.
Note 6: A 10-foot long SSP cut-off will be provided on upstream and downstream edges of gated structures. (i.e. 2 – rows of SSP)
Note 7: A 10-foot long SSP cut-off will be provided at the center of the wing and retaining wall structures. (i.e. 1 – row of SSP)
Note 8: The structure freeboard will extend 2 –additional feet for a total height of 7-feet above the 500 year flood elevation.
Note 9: Barr will complete a group pile analysis for 2-sections of the wing wall structures at the Red River of the North gated structure to better estimate the number and batter of piles required for the wing walls. Based on the results for these 2-sections at the Red River structure, we will estimate the piles required for the other gated structures and other wing wall sections.

Table 2 – Aqueduct Structures

Load Case	Event Category	Factor of Safety for Piles	Soil Condition
1 – 100 yr flood	Usual	2.00	Undrained
2 – 100 yr + ice	Unusual	1.50	Undrained

(note 1,2,4)			
3 – 500 yr flood	Unusual	1.50	Undrained
4 – 500 yr (+match levee elevation) (note 7)	Extreme	1.15	Undrained
5 – construction	Unusual	1.50	Undrained
6 – Normal low flow (note 3)	Usual	2.00	Drained

Note 1: Ice loads on the structure in the diversion channel during the 100 year flood will be considered as dynamic forces due to crushing or bending of ice floes as provided by Andrew Tuthill from the USACE via email to Miguel Wong dated February 1, 2011 at 9:52AM. These loads will be applied to the piers.
Note 2: Ice loads within the crossing channel above the diversion will be considered for the 100-year flood elevation and be considered static 5,000 psf for 2-foot thick ice.
Note 3: Ice loads within the minimum flow channel will be considered static 5,000 psf for 2-foot thick ice at the top of the low flow walls.
Note 4: An ice/debris load of 500 PLF along the structure will be used for the wing and retaining wall structures.
Note 5: A 10-foot long SSP cut-off will be provided on upstream and downstream edges of the crossing structures. (i.e. 2 – rows of SSP)
Note 6: A 10-foot long SSP cut-off will be provided at the center of the wing and retaining wall structures. (i.e. 1 – row of SSP)
Note 7: The top of structure design elevation will match the elevation of the highest levee adjacent to the structure.

Thank you all again for attending today's meeting. It was very productive.

Tor S. Hansen, PE

Vice President
Senior Civil Engineer
Minneapolis office: 952.832.2758
thansen@barr.com
www.barr.com

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From: Miguel Wong
Sent: Monday, June 07, 2010 4:31 PM
To: 'tony.s.fares@usace.army.mil'
Cc: Paul Nielsen; Brian K. LeMon; Evans, Craig O MVP
Subject: Structural Design Criteria
Attachments: Phase 3_Structural Design Criteria_07June2010.pdf

Dear Tony,

Find attached a memorandum summarizing the primary structural design considerations that we propose to follow for the Phase 3 feasibility design of the main hydraulic structures. This memorandum is in addition to the discussion with Ken Hoakens on the specs for piles (that you confirmed to agree with), and the ongoing work on Geotechnical Engineering that is being completed with input from Kurt Heckendorf.

In general, we are following the framework used by the Corps of Engineers for this type of work, but we would still appreciate very much to have your buy-in on the methodology before the calcs are completed.

We have tentatively assumed that you will be able to provide feedback by this Friday, June 11th (we are adding a task to the Critical Path Items Checklist). Let us know if you would be able to meet this tight schedule, otherwise please let us know an alternative due date.

Thanks a lot,

Miguel

Miguel Wong, PE, PhD

Senior Water Resources Engineer
Minneapolis office: 952.832.2632
cell: 612.703.6562
mwong@barr.com
www.barr.com

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Paul Nielsen

From: Miguel Wong
Sent: Monday, June 21, 2010 11:31 AM
To: Paul Nielsen; Mustafa B. Igdelioglu
Cc: Tor S. Hansen
Subject: FW: FMM Critical Path Checklist Item

fyi

Miguel Wong, PE, PhD

Senior Water Resources Engineer
Minneapolis office: 952.832.2632
cell: 612.703.6562
mwong@barr.com
www.barr.com

resourceful. naturally. BARR

From: Sarah Stratton
Sent: Monday, June 21, 2010 11:13 AM
To: Craig.O.Evans@usace.army.mil; tony.s.fares@usace.army.mil; Brian K. LeMon; Miguel Wong
Cc: Sarah Stratton; Miguel Wong
Subject: FMM Critical Path Checklist Item

A task has been added or updated:

Follow-up on structural design criteria
Requestor: Brian LeMon(952-832-2774; blemon@barr.com). USACE to provide feedback on email from Paul Nielsen, Barr to Tony Fares, USACE on June 18th 4.24 pm. The email summarized conference call on Tuesday, June 15th at 3.30 pm - including:

Red River Control Structure

Case = 1; Flood Event = 100 yr; Allow. Stress Increase = 1.0; Lateral Pile deflection = 0.67 inches; comments = w/ 2 ft ice pressure

Case = 2; Flood Event = Top of Structure; Allow. Stress Increase = 1.33; Lateral Pile deflection = 0.875 inches; comments = HW = 500 yr + 2 ft free board, TW = 500yr, w/2 ft ice pressure

Case = 3; Flood Event = Construction; Allow. Stress Increase = 1.33; Lateral Pile deflection = 0.67 inches; comments = no HW, TW, ice

Please verify that ice pressure is required for Case 2. We assume that the Flood Events listed for each case are maximum levels and that lower flood events may control.

Aqueducts, Wild Rice River Control structure, & Drop structures

Controlling cases will need to be evaluated differently from Red River Control Structure to determine stability and structural components since the diversion channel and tributary rivers could be at different flood events. Allowable pile loading and deflections will be based on original structural criteria sent by Barr to USACE.

Sheet pile weirs

Will be designed base on undrained & drained soil parameters determined under Barr's geotechnical work in Phase 3.

Reply by Tony Fares on 6/21/10.

I confirm the 3 loading cases as described for the Red River Control Structure. According to Hydraulics, the 100 year event will produce the highest head differential on the structure.
Ice pressure is required for case 2 if HW ice level is at or below the top of the structure.
The Wild Rice River Control Structure should be designed for the same loading cases as the Red River Control Structure: 100 year, 500 year and construction (structure is totally built before letting water into it). For allowable pile loading and deflections, use the same as for the Red River Control Structure.
For the rest of the structures allowable pile loading and deflections will be based on original structural criteria sent by Barr to USACE.
The assumption for Sheet Pile design is acceptable. Take into consideration expected scour and erosion. For straight line sheet pile cantilevered wall, they better be short due to the soft clay. Since the weir walls are like U shape walls you can consider anchoring the opposite walls together if that produce more stable U walls.

Category: Geotechnical and Structural

Due Date: 6/23/2010

Assigned To:
Craig Evans
Tony Fares
Brian LeMon
Miguel Wong

Login to Barr Project Management websites at: http://www.barr.com/clientre/Login.asp to update the completed date.

Paul Nielsen

From: Fares, Tony S MVP [tony.s.fares@mvp02.usace.army.mil]
Sent: Friday, July 02, 2010 12:59 PM
To: Paul Nielsen
Cc: Hokens, Kent D MVP; Heckendorf, Kurt A MVP
Subject: FW: Load Cases

Paul.
Please update the load cases per Kent Hoken's email as included below.
Please relay the info to Mustafa so he can utilize them in his design of the aqua duct structures.
Let us know if you need further assistance.

Tony Fares, P.E.
Structural Engineer
USACE St. Paul District
email: tony.s.fares@usace.army.mil
Tel: (651) 290 5568

-----Original Message-----
From: Hokens, Kent D MVP
Sent: Friday, July 02, 2010 7:09 AM
To: Fares, Tony S MVP
Cc: Wittine, Eric A MVP
Subject: Load Cases

Tony,

Thinking about this last night, I would suggest the following load cases for the Fargo feasibility design.

100 yr flood - Usual
100 yr flood + ice - Unusual
500 yr flood - Unusual
500 yr flood + ice - Extreme
Normal low flow - Usual

This would be mostly consistent with the latest direction and provide a reasonable but conservative cost estimate.

Kent

From:	Heckendorf, Kurt A MVP
To:	Miguel Wong; Brian K. LeMon
Cc:	Sarah Stratton; Matthew R. Metzger; Jed Greenwood; Aaron Grosser; Ivan Contreras; Paul Nielsen; Mustafa B. Igdelioglu; mbittner@cityoffargo.com; awalker@cityoffargo.com; bob.zimmerman@ci.moorhead.mn.us; Lesher, Michael D MVP; Pang, Edith P MVP; Hansen, Jeffrey L MVP; Fares, Tony S MVP; Stefanik, Elliott L MVP; Snyder, Aaron M MVP; Evans, Craig O MVP; Lee Beauvais; Stuart Dobberpuhl; C. Gregg Thielman; Hokens, Kent D MVP; Wittine, Eric A MVP; Dahlquist, Michael S MVP; Behling, Christopher W MVP; Rydeen, David W MVP; Morey, Darrell W MVP
Subject:	FMMFS: Allowable Capacity for Piles
Date:	Friday, July 02, 2010 2:22:45 PM

Everyone,

The St. Paul PDT members had a meeting this morning to discuss the pile capacities for the Fargo-Moorhead project. We feel that for the feasibility study, we need to follow EM 1110-2-2906 and be computing capacities using both undrained and drained shear strengths. We would have to obtain a waiver if we were to neglect the drained capacity and this would take some time. For the feasibility study, we want to make sure we capture the foundation costs and do not want to under estimate the costs.

Given the time frame, the Corps was hoping that Barr could look into what pile spacings are required for the different load cases (i.e. end-of-construction, normal river elevation & structural dead load, and the flood events). The appropriate uplift pressures should be taken into account with each load case. If the pile spacing is physically possible (i.e. spacing is 4 feet or more) we feel we could live with this. If not, then the alternative would be drilled shafts / caissons.

Along a parallel track, the Corps was also hoping that Barr could look into drilled shaft / caisson foundation for the structures and what the associated costs would be. This would help the entire team decide which direction to go for the final.

The biggest thing is that whatever direction we take, we need to have a cost estimate completed towards the end of July to ensure that we can meet the schedule.

If any of the calculations for the pile spacing can be sent to Kurt, Tony, Eric, and Kent for our review, it would be greatly appreciated. (Aaron Grosser proved the pile capacity calcs for the Maple River Structure in email today at 13:00 and we'll be looking at this.)

Please contact myself or Tony Fares if you have any questions or concerns.

We can discuss this in further details at the Weekly PDT meeting on Tuesday 6 JUL at 10:00 AM.

Respectfully,
Kurt
Kurt A. Heckendorf, P.E.
U.S. Army Corps of Engineers, St. Paul District
Geotechnical and Geology Section
Engineering and Construction Division, Design Branch
Phone: 651-290-5411
Fax: 651-290-5805

We have moved, new mailing address, effective June 14, 2010
180 5th St. East, Suite 700

St. Paul, MN 55101-1678

BARR

Barr Engineering Company
4700 West 77th Street • Minneapolis, MN 55435-4803
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Memorandum

To: Tony Fares – Corps of Engineers MVP
From: Paul Nielsen, Miguel Wong – Barr Engineering
Subject: Structural Design Criteria
Date: June 7, 2010
Project: Fargo-Moorhead Metro Flood Risk Management Project, Feasibility Study, Phase 3
c: Craig Evans – Corps of Engineers MVP; Brian LeMon – Barr Engineering

Introduction

The purpose of this memorandum is to introduce the methodology and design criteria that will be used by the A-E team to complete the feasibility structural design of the main hydraulic structures that are part of the Red River diversion project (Fargo Moorhead Metro Feasibility Study – Phase 3). It is the expectation of the A-E team that the U.S. Army Corps of Engineers (USACE) will provide feedback on the contents of this memorandum before the actual Phase 3 feasibility structural design calculations are completed.

The first section of this memo identifies the USACE documents that were reviewed to set up the constraints and criteria that will be used to guide the structural calculations for this project. Following this will be a number of sections that summarize the main constraints and criteria as they pertain to each specific element of design. The elements of design considered in this memo are:

- Stability for Concrete Structures including those at:
    - Red River of the North
    - Wild Rice River
    - Sheyenne River
    - Maple River
    - Lower Rush River
    - Rush River
- Loads and Load Combinations
- Hydraulic Loads

- Geotechnical
- Sheet Pile Weirs
- Reinforced Concrete Design

General Structural Guidance

The USACE Engineer Regulation ER 1110-2-1150 "Engineering and Design for Civil Work Projects", Section 13.5 (Engineering during Feasibility Phase) indicates that the first engineering objective is to "Provide engineering data and analyses sufficient to develop the complete project schedule and cost estimate." The USACE Engineer Regulation ER 1110-2-1150, Appendix C-7 formulates the structural requirements that need to be included in the feasibility engineering appendix.

The USACE is governed by Engineer Regulations (ER's), Engineer Manuals (EM's), Engineer Technical Letters (ETL's) and Engineering Circulars (EC's). These documents are available online at the following web site: http://www.usace.army.mil/inet/usace-docs. It is our understanding that industry standards shall apply when USACE criteria are not applicable.

The USACE documents to be used in Phase 3 designs include:

- EM 1110-2-1612 "Ice Engineering"
- EM 1110-2-2100 "Stability Analysis of Concrete Structures"
- EM 1110-2-2104 "Structural Design for Reinforced-Concrete Hydraulic Structures"
- EM 1110-2-2200 "Gravity Dam Design"
- EM 1110-2-2502 "Retaining and Flood Walls"
- EM 1110-2-2504 "Design of Sheet Pile Walls"
- EM 1110-2-2702 "Design of Spillway Tainter Gates"
- EM 1110-2-2906 "Design of Pile Foundations
- ER 1110-2-1806 "Earthquake Design and Analysis for Corps of Engineers Projects"

Stability for Concrete Structures

The USACE Engineer Manual EM 1110-2-2100 provides guidance for stability of concrete gravity structures. Where there are conflicting stability criteria with other documents, this manual will govern. Appendix B of this manual describes load classifications (Usual, Unusual, Extreme) and all loading-condition requirements for each structure as specified in various manuals. This manual will be followed for Phase 3 feasibility designs except as noted below, for pile analysis and design.

The soil conditions along the entire project (see Geotechnical), are underlain by "soft" clay, susceptible to large settlements. Therefore, piles will be used at all concrete structures. Stability requirements outlined in USACE Engineer Manual EM 1110-2-2100 do not cover pile supported structures. Piles will be designed per USACE Engineer Manual EM 1110-2-2906, and this design will include the following considerations:

- The purpose of a pile foundation is to transfer and distribute load through a material or stratum with inadequate bearing, sliding or uplift capacity to a firmer stratum capable of supporting the load without detrimental displacement. The conventional stability check of sliding, rotation, floatation and bearing per USACE Engineer Manual EM 1110-2-2100 is no longer applicable.
- A rigid cap analysis will determine the axial load to each pile. Horizontal loads will be assumed to be evenly resisted by each vertical pile. For battered piles, the horizontal component of the batter provides an additional lateral resistance to horizontal loads.
- Vertical and lateral loads on each pile will be checked to make sure the pile capacity (with Factor of Safety) and stresses are not exceeded in accordance with USACE Engineer Manual EM 1110-2-2906, Section 4-2 (Loading Conditions). The number of piles and spacing are adjusted to meet the required pile capacity and stresses.
- In general, stresses for the usual and extreme case may be increased up to 33% and 75% respectively. However, checks must be made to ensure that the structure will not catastrophically fail during or after extreme loading conditions.
- The Factor of Safety (F.S.) associated with each load case varies with the frequency (probability) of each flood event. Assuming that piles will be test loaded, the Usual case is based on a 10 yr event with a F.S. =2.0, the Unusual case is based on 300 yr event with a F.S. =1.5, and the Extreme case is based on the maximum design flood (MDF) with a F.S. = 1.15. The MDF is the designation used to represent the maximum loading condition (as judged by the minimum factor of safety) and must be determined for each structure. The MDF may be represented by any large flood event up to the probable maximum flood (PMF). For this Phase 3 we will assume the MDF occurs when the headwater is at the top of the structure prior to overtopping. This will represent a flood greater than the 500 yr event due to freeboard and will be classified as an Extreme event.

Loads and Load Combinations

Load classifications and combinations described in USACE Engineer Manual EM 1110-2-2100 Appendix B will be used to determine the loads and applicable load combinations, including:

- The load conditions that a structure may encounter during its service life are grouped into the load condition categories of Usual, Unusual and Extreme (per USACE Engineer Manual EM 1110-2-2100, Section 3-2).
- USACE Engineer Manual EM 1110-2-2100 Table B-1 lists 7 basic Loading Conditions for a gravity dam (Structure Type: Gravity Dams, EM 1110-2-2200; Navigation Dams, EM 1110-2-2607):

Load Case	Loading Description	Classification
1	Construction Condition	UN
2	Normal Operating	U
3	Infrequent Flood	UN
4	Construction with Operational Basis Earthquake (OBE)	E
5	Coincident Pool with OBE	UN
6	Coincident Pool with Maximum Design Earthquake (MDE)	E
7	Maximum Design Flood (MDF)	U/UN/E

* U = usual, UN = unusual, E = extreme

- Load Conditions 2 (Usual), 3 (Unusual), and 7 (Extreme) will be checked during the stability analysis of the piles. Conditions 1 (construction), 4, 5, and 6 will not be checked. USACE Engineer Regulation ER 1110-2-1806 indicates that the project is located in Seismic Zone 0. Conditions 4, 5, and 6 all include earthquake loading, which typically does not control in Seismic Zone 0. For final design all load conditions should be checked.
- Basic Loads used for stability include:
    - Vertical dead load of concrete.
    - Vertical dead load of water on foundation, openings, and downstream back face of structure.
    - Uplift loads on foundation (see Hydraulic Loads & Geotechnical).

- Horizontal water pressures (static) based on headwater and tailwater elevations (see Hydraulic Loads).
- USACE Engineer Manual EM 1110-2-1612: Ice loads assume 2 ft thick at 5,000 psf acting at headwater elevation. No ice pressure is assumed on downstream face. For final design impact loads from floating ice and debris should be investigated.

Hydraulic Loads

Hydraulic information will be provided based on revised models for the Phase 3 work. Both horizontal and vertical loads due to water pressures will be considered during feasibility designs in Phase 3. Hydrostatic loads from HEC-RAS modeling using Phase 3 hydrology during10-yr, 200-yr, and 500-yr events will be provided for each structure. Uplift loads will be evaluated for structure based on the hydraulic gradient, permeability of soil and cut-off wall recommendations provided during Geotechnical evaluation.

Geotechnical

Soil properties for the soil profile at a given structure will be modeled using current industry standard software, such as DRIVEN, ALLPILE, A-PILE and/or L-PILE to determine the vertical and lateral capacity of the piles in accordance with USACE Engineer Manual EM 1110-2-2906. Allowable displacement at the pile cap will depend on the load classification as follows:

Load Classification	Vertical Displacement (inches)	Horizontal Displacement (inches)
Usual	0.25	0.50
Unusual	0.33	0.67
Extreme	0.44	0.875

The larger displacements for the Unusual and Extreme load classifications have been assumed by Barr Engineering. They correspond to an increase of 133% and 175% of the deformation allowed for the Usual case based on Barr's interpretation of the allowable stress increase of 33% and 75% allowed in USACE Engineer Manual EM 1110-2-2906. The deflections above represent deflection limits for gated structures like dams; larger deflections may be allowed for less sensitive structures if the stresses in the structure and piles are not excessive. For final design, allowable deflection limits should be established for each structure.

A seepage analysis will be made to determine the requirements of cut-off walls and uplift pressures on the Red River Control Structure and Maple Hydraulic Structure. No site specific seepage analysis will be performed at the other concrete structures. Results from the two analyses above will be used as a reference to determine approximate cut-off wall requirements at the other structures. For final design, a seepage analysis should be made for each structure to determine uplift mitigation requirements like drains and cut-off walls.

Sheet Pile Weirs

Sheet pile structures will be designed in accordance with the requirements of USACE Engineer Manual EM 1110-2-2504. It is anticipated that all test results will not be completed prior to the submission of the Phase 3 report. Soil parameters provide by the USACE in Table I-2 of the January 2010 "Geotechnical Design and Geology" will be used to determine the lateral earth pressures for both the undrained and drained parameters sheeting driven in clay.

Reinforced Concrete Design

Preliminary sizing of reinforced concrete components will be made in accordance with USACE Engineer Manual EM 1110-2-2104, including the following design considerations:

- A hydraulic load factor, Hf = 1.3 (accept members in direct tension use Hf = 1.65), will be applied in addition to the standard load factors found in ACI 318.
- Fluid pressure shall have a 1.7 load factor.
- For preliminary design of members use a maximum tension reinforcement ratio between $0.25\rho_b$ and $0.375\rho_b$.

I = influence factor of the effective group embedment (dim.)

D' = effective depth taken as $2D_b/3$ (ft.)

D_b = depth of embedment of piles in layer that provides support, as specified in Figure 10.7.2.3.1-1 (ft.)

$N1_{60}$ = *SPT* blow count corrected for both overburden and hammer efficiency effects (blows/ft.) as specified in Article 10.4.6.2.4.

q_c = static cone tip resistance (ksf)

Alternatively, other methods for computing settlement in cohesionless soil, such as the Hough method as specified in Article 10.6.2.4.2 may also be used in connection with the equivalent footing approach.

The corrected *SPT* blow count or the static cone tip resistance should be averaged over a depth equal to the pile group width *B* below the equivalent footing. The *SPT* and *CPT* methods (Eqs. 1 and 2) shall only be considered applicable to the distributions shown in Figure 10.7.2.3.1-1b and Figure 10.7.2.3.1-2.

10.7.2.4 Horizontal Pile Foundation Movement

Horizontal movement induced by lateral loads shall be evaluated. The provisions of Article 10.5.2.1 shall apply regarding horizontal movement criteria.

The horizontal movement of pile foundations shall be estimated using procedures that consider soil-structure interaction. Tolerable lateral movements of piles shall be established on the basis of confirming compatible movements of structural components, e.g., pile to column connections, for the loading condition under consideration.

The effects of the lateral resistance provided by an embedded cap may be considered in the evaluation of horizontal movement.

The orientation of nonsymmetrical pile cross-sections shall be considered when computing the pile lateral stiffness.

Lateral resistance of single piles may be determined by static load test. If a static lateral load test is to be performed, it shall follow the procedures specified in ASTM D 3966.

The effects of group interaction shall be taken into account when evaluating pile group horizontal movement. When the *P-y* method of analysis is used, the values of *P* shall be multiplied by *P*-multiplier values, P_m, to account for group effects. The values of P_m provided in Table 1 should be used.

C10.7.2.4

Pile foundations are subjected to horizontal loads due to wind, traffic loads, bridge curvature, vessel or traffic impact and earthquake. Batter piles are sometimes used but they are somewhat more expensive than vertical piles, and vertical piles are more effective against dynamic loads.

Methods of analysis that use manual computation were developed by Broms (*1964a and 1964b*). They are discussed in detail by Hannigan et al. (*2005*). Reese developed analysis methods that model the horizontal soil resistance using *P-y* curves. This analysis has been well developed and software is available for analyzing single piles and pile groups (*Reese, 1986*; *Williams et al., 2003*; and *Hannigan et al., 2005*).

Deep foundation horizontal movement at the foundation design stage may be analyzed using computer applications that consider soil-structure interaction. Application formulations are available that consider the total structure including pile cap, pier and superstructure (*Williams et al., 2003*).

If a static load test is used to assess the site specific lateral resistance of a pile, information on the methods of analysis and interpretation of lateral load tests presented in the *Handbook on Design of Piles and Drilled Shafts Under Lateral Load* (*Reese, 1984*) and *Static Testing of Deep Foundations* (*Kyfor et al., 1992*) should be used.

Table 10.7.2.4-1 Pile P-Multipliers, P_m, for Multiple Row Shading (averaged from *Hannigan et al., 2005*).

Pile *CTC* spacing (in the direction of loading)	P-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
$3B$	0.7	0.5	0.35
$5B$	1.0	0.85	0.7

Loading direction and spacing shall be taken as defined in Figure 1. If the loading direction for a single row of piles is perpendicular to the row (bottom detail in the figure), a group reduction factor of less than 1.0 should only be used if the pile spacing is $5B$ or less, i.e., a P_m of 0.7 for a spacing of $3B$, as shown in Figure 1.

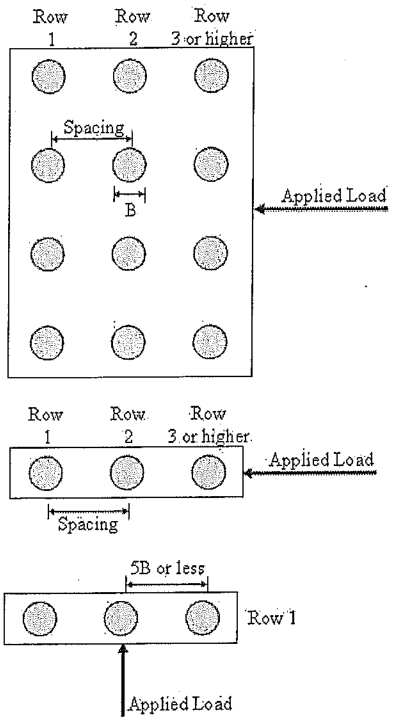

Figure 10.7.2.4-1 Definition of loading direction and spacing for group effects.

Since many piles are installed in groups, the horizontal resistance of the group has been studied and it has been found that multiple rows of piles will have less resistance than the sum of the single pile resistance. The front piles "shade" rows that are further back.

The P-multipliers, P_m, in Table 1 are a function of the center-to-center (*CTC*) spacing of piles in the group in the direction of loading expressed in multiples of the pile diameter, B. The values of P_m in Table 1 were developed for vertical piles only.

Horizontal load tests have been performed on pile groups, and multipliers have been determined that can be used in the analysis for the various rows. Those multipliers have been found to depend on the pile spacing and the row number in the direction of loading. To establish values of P_m for other pile spacing values, interpolation between values should be conducted.

The multipliers on the pile rows are a topic of current research and may change in the future. Values from recent research have been tabulated by Hannigan et al. (*2005*). Averaged values are provided in Table 1.

Note that these *P-y* methods generally apply to foundation elements that have some ability to bend and deflect. For large diameter, relatively short foundation elements, e.g., drilled shafts or relatively short stiff piles, the foundation element rotates rather than bends, in which case strain wedge theory (*Norris, 1986*; *Ashour et al., 1998*) may be more applicable. When strain wedge theory is used to assess the lateral load response of groups of short, large diameter piles or shaft groups, group effects should be addressed through evaluation of the overlap between shear zones formed due to the passive wedge that develops in front of each shaft in the group as lateral deflection increases. Note that P_m in Table 1 is not applicable if strain wedge theory is used.

Batter piles provide a much stiffer lateral response than vertical piles when loaded in the direction of the batter.

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT Q – STRUCTURAL DESIGN COMPUTATIONS— CONTROL STRUCTURES

Report for the US Army Corps of Engineers, and the cities of Fargo, ND & Moorhead, MN

By: BARR ENGINEERING

FINAL – Version February 28, 2011

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ELECTRONIC ATTACHMENTS ONLY

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT Q – STRUCTURAL DESIGN COMPUTATIONS—CONTROL STRUCTURES

F-Q1.0 GROUP AND RIGID CAP ANALYSIS RESULTS

Attachments F-Q1.1 through F-Q1.5 of this Exhibit Q present the results from the group and rigid cap structural design analyses for the Red River of the North (LPP) control structure wing walls A and F.

F-Q2.0 STRUCTURAL PILE CAPACITY RESULTS

Attachment F-Q2.1 of this Exhibit Q presents the structural pile capacities for the following structural components: HP14x73 (usual case), HP14x89 (usual case), Red River (LPP) Control Structure Wing Wall A HP14x89, and Red River (LPP) Control Structure Wing Wall F HP14x89.

F-Q3.0 ICE LOADS ON PIERS

Attachment F-Q3.1 of this Exhibit Q presents the ice loads on piers for the following structural components: Red River of the North (LPP) and Wild Rice River Gated Control Structures.

F-Q4.0 COMPLETE SET OF STRUCTURAL COMPUTATIONS

The complete set of structural computations for all load conditions for the Red River of the North (LPP) control structure and Wild Rice River control structure is included electronically on the attached DVD.

ATTACHMENT

F-Q1.1 Summary of Group and Rigid Cap Analyses Results

BARR ENGINEERING			DATE	2/21/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/21/11		**PKN**	SUBJECT	**GOUP vs RIGID Cap Analysis**	

Control Structure: ND RRN Wing Wall SECTION A Summary Pile Reactions

103% of ALLOWABLE O.K.

Load Case	Pile Row	1	2	3	4	5	6	7	8	Allowable Pile Loads
	Rigid Cap Analysis									
1	Axial Load (tons/pile)	57.10	48.50	39.91	29.59	19.28	8.96	-1.35	-11.67	59.775
	GROUP									
	Axial Load (tons/pile)	***61.59***	53.28	44.97	34.93	20.13	5.52	-8.91	-23.57	59.775
	Horiz. Displace (in.)	-0.5488	-0.5488	-0.5488	-0.5488	-0.5488	-0.5488	-0.5488	-0.5488	
	Mz max (k-ft)	193.33	173.33	173.33	181.67	183.33	183.33	183.33	183.33	
	Fy (k)	-33.57	-28.87	-28.91	-30.80	-31.32	-31.40	-31.47	-31.54	Horiz Loads/pile
	Rigid Cap Analysis									
1.1	Axial Load (tons/pile)	57.90	49.09	40.27	29.70	19.13	8.55	-2.02	-12.59	79.70
	GROUP									
	Axial Load (tons/pile)	53.50	47.16	40.82	32.27	20.97	9.80	-1.23	-12.29	
	Horiz. Displace (in.)	-0.5513	-0.5513	-0.5513	-0.5513	-0.5513	-0.5513	-0.5513	-0.5513	
	Mz max (k-ft)	196.67	176.67	176.67	185.00	186.67	186.67	186.67	186.67	
	Fy (k)	-33.922	-29.177	-29.206	-31.107	-31.619	-31.674	-31.73	-31.783	Horiz Loads/pile

Control Structure: ND RRN Wing Wall SECTION F Summary Pile Reactions

Load Case	Pile Row	1	2	3	4	5	Allowable Pile Loads
	Rigid Cap Analysis						
1	Axial Load (tons/pile)	47.04	34.70	22.36	10.02	-2.31	59.775
	GROUP						
	Axial Load (tons/pile)	55.62	40.70	25.78	5.42	-1.51	59.775
	Horiz. Displace (in.)	-0.4354	-0.4354	-0.4354	-0.4354	-0.4354	
	Mz max (k-ft)	152.50	140.00	139.17	140.83	139.17	
	Fy (k)	-28.61	-25.32	-25.23	-25.85	-25.55	Horiz Loads/pile

Control Structure: Wolverton Creek Wing Wall SECTION A Summary Pile Reactions

Load Case	Pile Row	1	2	3	4		Allowable Pile Loads
	Rigid Cap Analysis						
1	Axial Load (tons/pile)	63.29	36.74	12.07	-12.59		100
	Horiz Resist Capacity(k)	61.70	31.00	31.00	31.00	Row 1 includes 3": 12" batter	
	GROUP						
	Axial Load (tons/pile)	69.88	38.17	12.25	-16.37	Local Axis	100
	Horiz. Displace (in.)	-0.1033	-0.1033	-0.1033	-0.1033	Global	
	Mz max (k-ft)	41.17	63.92	64.00	63.92		
	Fy (k)	-60.78	-22.18	-22.24	-22.28		Horiz Loads/pile

BATTERED Row

ATTACHMENT

F-Q1.2 Rigid Cap Analyses Results for the Red River (LPP) Control Structure Wing Wall A

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/4/11		PKN	SUBJECT	Control Structure: ND RRN 35k s35k Load Cases: 1	SECTION A

Case 1
Event 100 years

File:

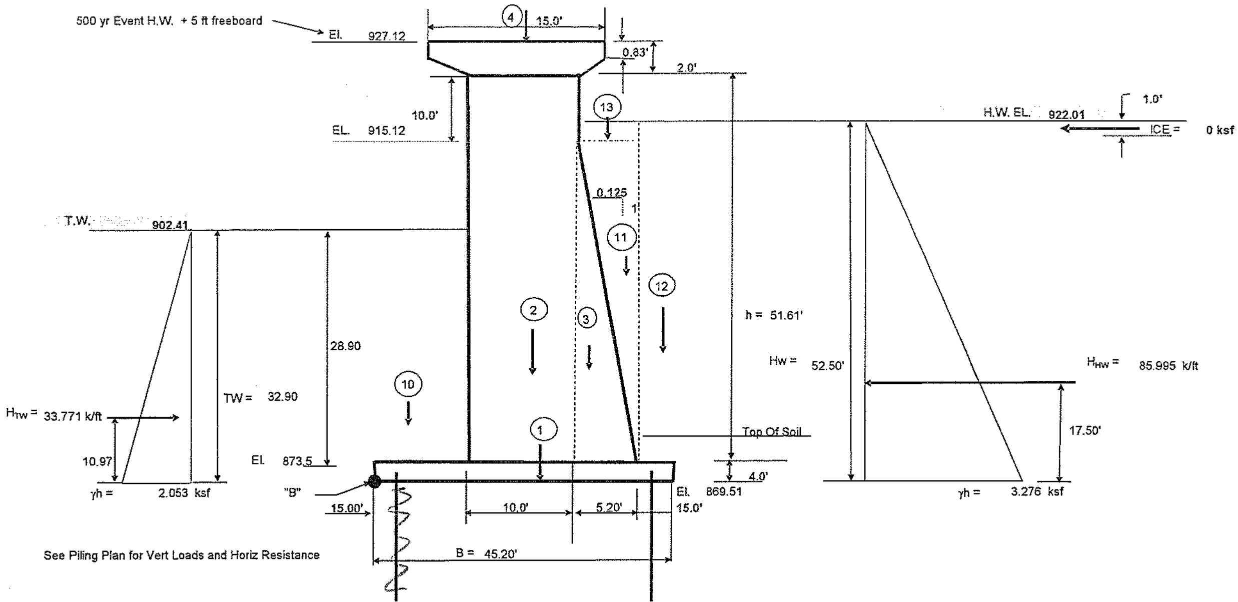
H.W. EL = 922.01 ft
T.W. EL. = 902.41 ft

500 yr H.W. EL = 922.12 ft

Top of Ftg EL. = 873.51 ft

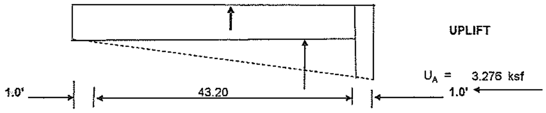
State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section
Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model

U_B = 2.053 ksf



enter "0" if no cut-off

REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

Non-Overflow Dam

		L	W	H	γ	shape	V	arm	Mv
Vertical Loads	Section	ft	ft	ft	kcf		K	ft	ft-k
Ftg concrete	1	30	45.20	4.00	0.15	rec	813.6	22.60	18,388.4
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	20.00	1,120.5
Bridge Slab	4	30	12.50	1.17	0.15	rec	65.8	20.00	1,316.3
Monolith	2	30	10.00	51.61	0.15	rec	2322.5	20.00	46,449.0
Monolith	3	30	5.20	41.61	0.15	tri	487.0	26.73	13,018.1
					D.L. Concrete	ΣVc =	3744.9	ΣM_V =	80,292.2
T.W on ftg Monolith	10	30	15.00	28.90	0.0624	rec	811.5	7.50	6,086.3
H.W. on Monolith Slope	11	30	5.20	41.61	0.0624	tri	202.6	26.73	5,415.5
H.W. Above Slope	13	30	5.20	6.89	0.0624	rec	67.1	27.60	1,851.6
H.W. on Mono. ftg	12	30	15.00	48.50	0.0624	rec	1361.9	37.70	51,344.6
					D.L. Water	ΣVw =	2443.1	ΣM_V =	64,698.1

← CONSTANT FOR ALL LOAD CASES

	L	W	Pressure		U	arm	Mu
Uplift Loads	ft	ft	ksf		K	ft	ft-k
U_B	30	45.20	2.053	rec	-2783.9	22.60	-62,918
U_A	30	45.20	1.223	tri	-829.2	30.13	-24,989
				ΣU =	-3613.1	ΣM_U =	-87,906

Horizontal Loads	L	H	Pressure		ICE	arm	Mu
	ft	ft	ksf		K	ft	ft-k
ICE	30	1.00	0.00	rec	0.0	52	0.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

	L	Force		H	arm	Mw
Water Loads	ft	k/ft		K	ft	ft-k
H_{TW}	30	33.771	tri	1013.14	10.97	11110.72
H_{HW}	30	-85.995	tri	-2579.85	17.50	-45147.38
			ΣW =	-1566.71	ΣM_W =	-34036.7

Overturning Moments $\Sigma M_{OT} = M_U + M_W + M_{ICE}$ = -121943 kip-ft
Resisting Moments $\Sigma M_R = M_V$ = 144990 kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**23,047**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**2,575**	kips
Sum of Horizontal Forces	H = Ice + water =	**-1,567**	kips

Location of Resultant $Xr = \Sigma M / P$ = 8.95 ft from Toe
$e = B/2 - Xr$ = 13.65 ft
$B/6$ = 7.534 ft

BARR ENGINEERING			DATE	2/4/2011 SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/4/11		PKN	SUBJECT	Control Structure: ND RRN 35k s35k Load Cases: 1.1 SECTION A

Case 1.1
Event 100 years

H.W. EL = 922.01 ft
T.W. EL = 902.41 ft

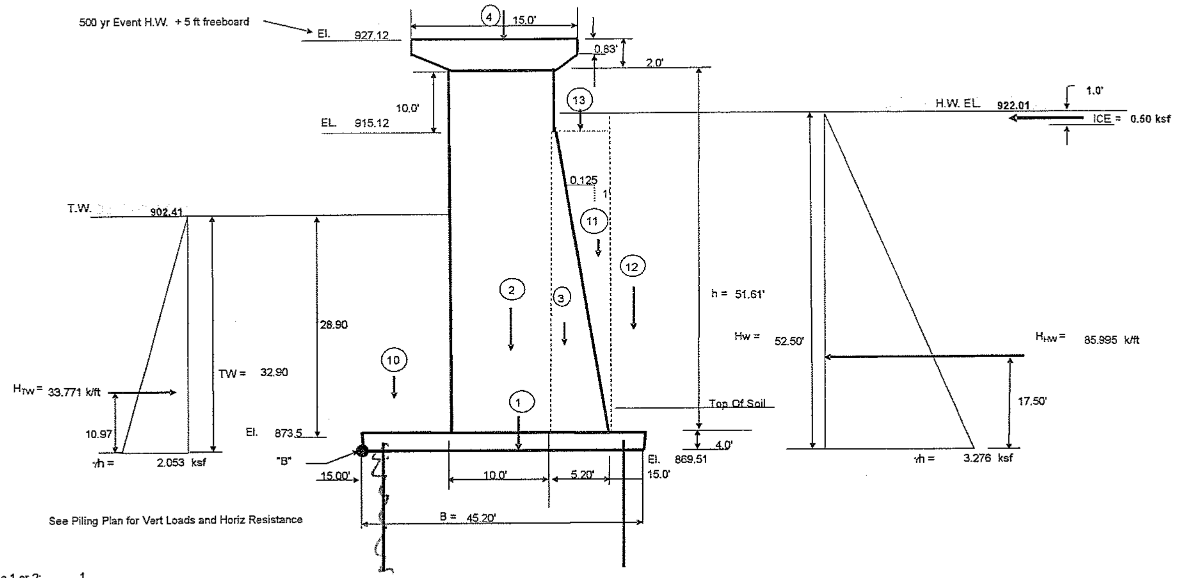
500 yr H.W. EL = 922.12 ft

Top of Ftg EL = 873.51 ft

File:

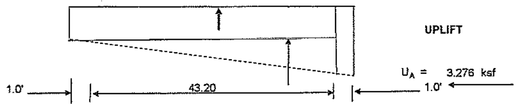
State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section
Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model

U_B = 2.053 ksf



enter "0" if no cut-off

REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	45.20	4.00	0.15	rec	813.6	22.60	18,388.4
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	20.00	1,120.5
Bridge Slab	4	30	12.50	1.17	0.15	rec	65.8	20.00	1,316.3
Monolith	2	30	10.00	51.61	0.15	rec	2322.5	20.00	46,449.0
Monolith	3	30	5.20	41.61	0.15	tri	487.0	26.73	13,018.1
					D.L. Concrete	ΣVc =	3744.9	ΣMv =	80,292.2

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	15.00	28.90	0.0624	rec	811.5	7.50	6,086.3
H.W. on Monolith Slope	11	30	5.20	41.61	0.0624	tri	202.6	26.73	5,415.5
H.W. Above Slope	13	30	5.20	6.89	0.0624	rec	67.1	27.60	1,851.6
H.W. on Mono. ftg	12	30	15.00	48.50	0.0624	rec	1361.9	37.70	51,344.6
					D.L. Water	ΣVw =	2443.1	ΣMv =	64,698.1

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	45.20	2.053	rec	-2783.9	22.60	-62,918
	U_A	30	45.20	1.223	tri	-829.2	30.13	-24,989
					ΣU =	-3613.1	ΣMu =	-87,906

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	1.00	-0.50	rec	-15.0	52	-780.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{TW}	30	33.771	tri	1013.14	10.97	11110.72
H_{HW}	30	-85.995	tri	-2579.85	17.50	-45147.38
			ΣW =	-1566.71	ΣMw =	-34036.7

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-122723	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	144990	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**22,267**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**2,575**	kips
Sum of Horizontal Forces	H = Ice + water =	**-1,582**	kips

Location of Resultant
Xr = ΣM / P = 8.65 ft from Toe
e = B/2 - Xr = 13.95 ft
B/6 = 7.534 ft

BARR ENGINEERING			DATE	2/4/2011		SHEET NO.
			PROJECT NAME	**Flood Control ND Diversion Inlet - Corps Of Engineers**		
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER			
PKN 2/4/11		PKN	SUBJECT	Control Structure: ND RRN 35k s35k Load Cases: 2	SECTION A	

Case 2
Event 500 years

H.W. EL = 922.12 ft
T.W. EL. = 914.41 ft

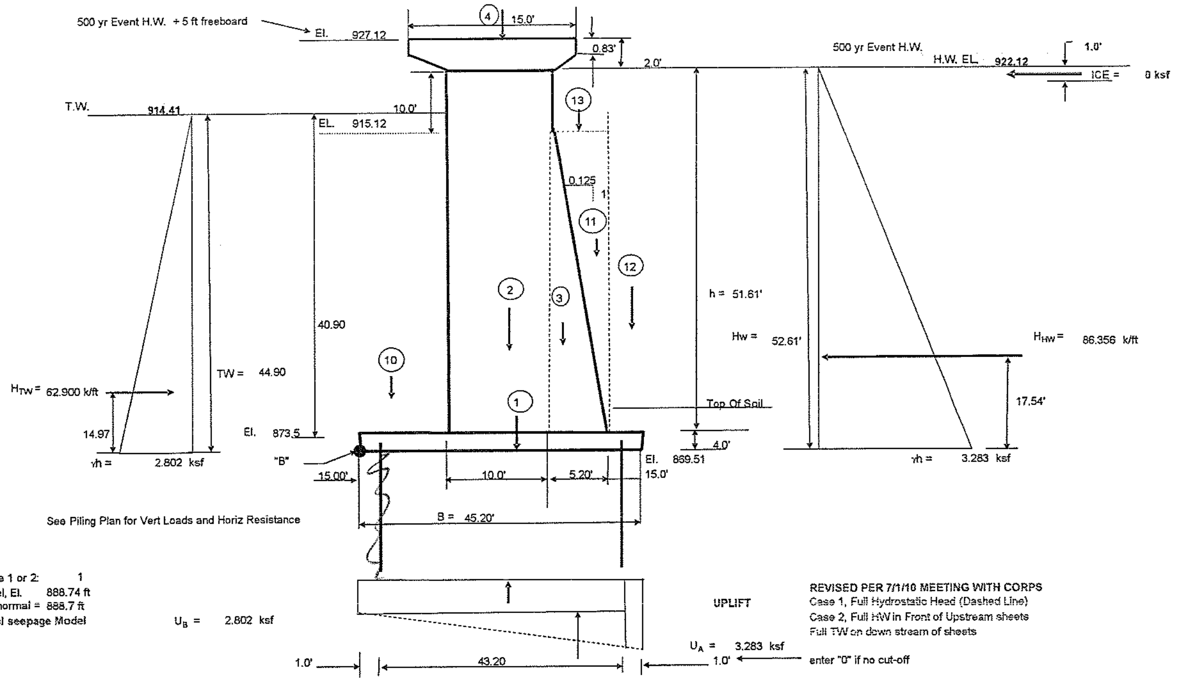
500 yr H.W. EL = 922.12 ft

Top of Ftg EL. = 873.51 ft

File:

State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section
Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model

U_B = 2.802 ksf

REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	45.20	4.00	0.15	rec	813.6	22.60	18,388.4
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	20.00	1,120.5
Bridge Slab	4	30	12.50	1.17	0.15	rec	65.8	20.00	1,316.3
Monolith	2	30	10.00	51.61	0.15	rec	2322.5	20.00	46,449.0
Monolith	3	30	5.20	41.61	0.15	tri	487.0	26.73	13,018.1
					D.L. Concrete	ΣVc =	**3744.9**	ΣM_V =	**80,292.2**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	15.00	40.90	0.0624	rec	1148.5	7.50	8,613.5
H.W. on Monolith Slope	11	30	5.20	41.61	0.0624	tri	202.6	26.73	5,415.5
H.W. Above Slope	13	30	5.20	7.00	0.0624	rec	68.2	27.60	1,881.2
H.W. on Mono. ftg	12	30	15.00	48.61	0.0624	rec	1365.0	37.70	51,461.0
					D.L. Water	ΣVw =	**2784.2**	ΣM_V =	**67,371.3**

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	45.20	2.802	rec	-3799.3	22.60	-85,866
	U_A	30	45.20	0.481	tri	-326.2	30.13	-9,830
					ΣU =	**-4125.5**	ΣM_U =	**-95,696**

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	1.00	0.00	rec	0.0	52.11	0.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{TW}	30	62.900	tri	1886.99	14.97	28241.88
H_{HW}	30	-86.356	tri	-2590.67	17.54	-45431.75
			ΣW =	**-703.69**	ΣM_W =	**-17189.9**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-112886	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	147664	kip-ft

Sum of Moments	$\Sigma M net = M_R + M_{OT}$ =	**34,778**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**2,404**	kips
Sum of Horizontal Forces	H = Ice + water =	**-704**	kips

Location of Resultant
Xr = ΣM / P = 14.47 ft from Toe
e = B/2 - Xr = 8.13 ft
B/6 = 7.534 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/4/11		PKN	SUBJECT	Control Structure: ND RRN 35k s35k Load Cases: 2.1	SECTION A

Case 2.1
Event 500 years

Per COE 2/1/11: HW = 500yr + 5' w/ No ice

File:

H.W. EL = 927.12 ft
T.W. EL. = 914.41 ft

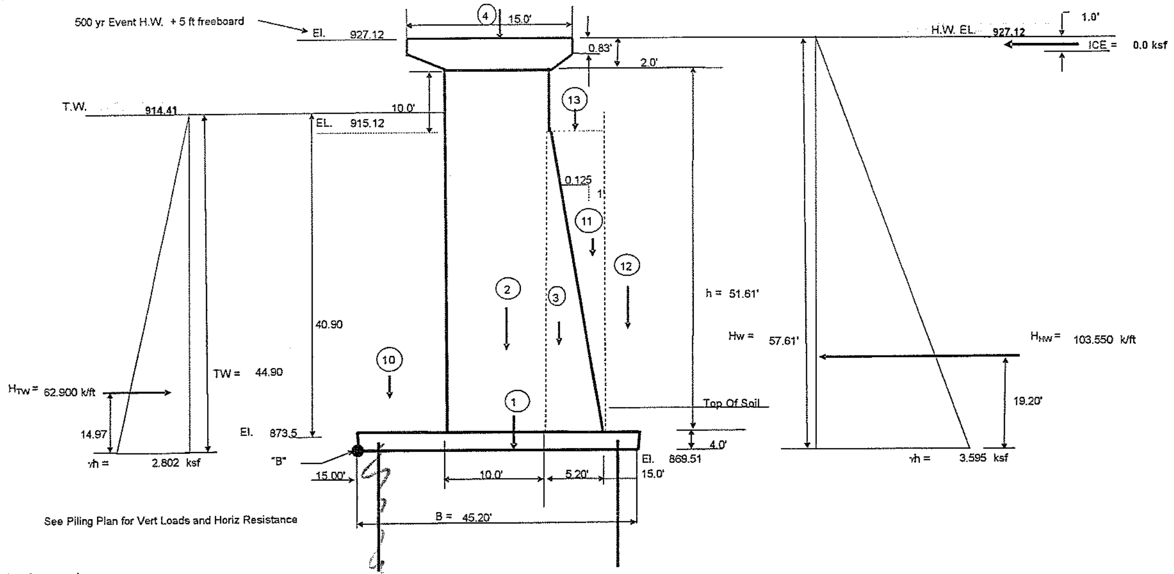
500 yr H.W. EL = 922.12 ft

Top of Ftg EL. = 873.51 ft

State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

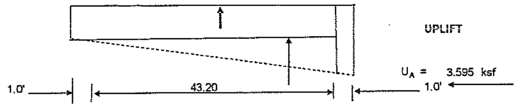
Non-Overflow Section

Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model

U_B = 2.802 ksf



REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	45.20	4.00	0.15	rec	813.6	22.60	18,388.4
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	20.00	1,120.5
Bridge Slab	4	30	12.50	1.17	0.15	rec	65.8	20.00	1,316.3
Monolith	2	30	10.00	51.61	0.15	rec	2322.5	20.00	46,449.0
Monolith	3	30	5.20	41.61	0.15	tri	487.0	26.73	13,018.1
					D.L. Concrete	ΣVc =	3744.9	ΣM$_V$ =	80,292.2

CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	15.00	40.90	0.0624	rec	1148.5	7.50	8,613.5
H.W. on Monolith Slope	11	30	5.20	41.61	0.0624	tri	202.6	26.73	5,415.5
H.W. Above Slope	13	30	5.20	12.00	0.0624	rec	116.8	27.60	3,224.9
H.W. on Mono. ftg	12	30	15.00	53.61	0.0624	rec	1505.4	37.70	56,754.3
					D.L. Water	ΣVw =	2973.3	ΣM$_V$ =	74,008.2

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	45.20	2.802	rec	-3799.3	22.60	-85,866
	U_A	30	45.20	0.793	tri	-537.7	30.13	-16,204
					ΣU =	-4337.0	ΣM$_U$ =	-102,071

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	1.00	0.00	rec	0.0	57.11	0.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{TW}	30	62.900	tri	1886.99	14.97	28241.88
H_{HW}	30	-103.550	tri	-3106.50	19.20	-59655.19
			ΣW =	-1219.52	ΣM$_W$ =	-31413.3

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-133484	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	154300	kip-ft

Sum of Moments	$\Sigma M_{net} = M_R + M_{OT}$ =	20,816	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	2,381	kips
Sum of Horizontal Forces	H = Ice + water =	-1,220	kips

Location of Resultant
$X_r = \Sigma M / P$ = 8.74 ft from Toe
$e = B/2 - X_r$ = 13.86 ft
$B/6$ = 7.534 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/4/11		PKN	SUBJECT	Control Structure: ND RRN 35k s35k Load Cases: 3	SECTION A

Case 3
Event 2 years

H.W. EL = ft
T.W. EL. = ft

500 yr H.W. EL = 922.12 ft

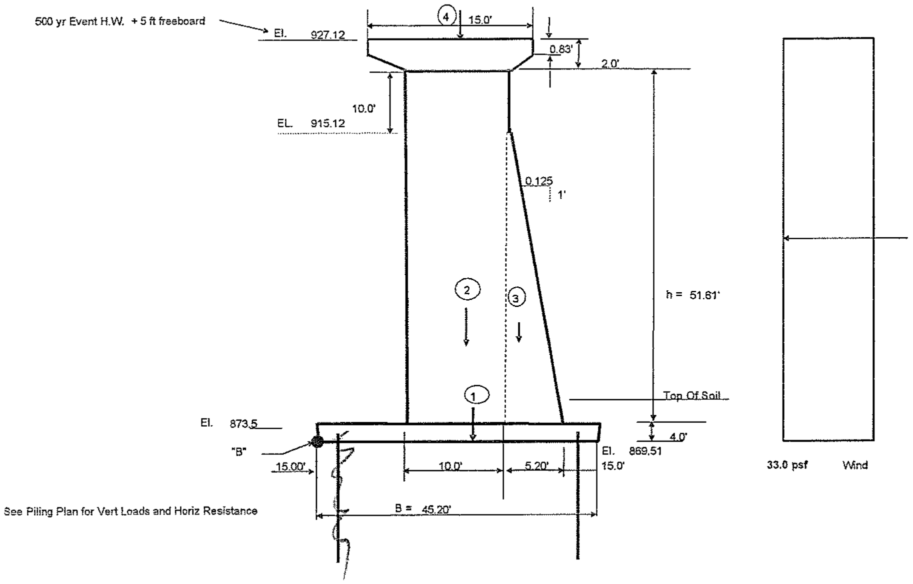
Top of Ftg EL. = 873.51 ft

File:

State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section

Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Non-Overflow Dam

		L	W	H	γ	shape	V	arm	Mv
Vertical Loads	**Section**	ft	ft	ft	kcf		K	ft	ft-k
Ftg concrete	**1**	30	45.20	4.00	0.15	rec	813.6	22.60	18,388.4
Bridge Slab	**4**	30	15.00	0.83	0.15	rec	56.0	20.00	1,120.5
Bridge Slab	**4**	30	12.50	1.17	0.15	rec	65.8	20.00	1,316.3
Monolith	**2**	30	10.00	51.61	0.15	rec	2322.5	20.00	46,449.0
Monolith	**3**	30	5.20	41.61	0.15	tri	487.0	26.73	13,018.1
					D.L. Concrete	$\Sigma Vc =$	**3744.9**	$\Sigma M_V =$	**80,292.2**

← CONSTANT FOR ALL LOAD CASES

Horizontal Loads		L	H	Pressure		H	arm	Mw
		ft	ft	ksf		K	ft	ft-k
	Wind	30	57.61	-0.03	rec	-57.0	28.805	-1,642.9

Overturning Moments $\Sigma M_{OT} = M_W =$ -1643 kip-ft
Resisting Moments $\Sigma M_R = M_V =$ 80292 kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT} =$	**78,649**	kip-ft
Sum of Vertical Forces	P = Conc =	**3,745**	kips
Sum of Horizontal Forces	H = Wind =	**-57**	kips

Location of Resultant $Xr = \Sigma M / P =$ 21.00 ft from Toe
$e = B/2 - Xr =$ 1.60 ft
$B/6 =$ 7.534 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	**Flood Control ND Diversion Inlet - Corps Of Engineers**	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER		
			SUBJECT	**Control Structure:** ND RRN 35k s35k Load Cases: **4**	SECTION A

Case 4

Event years

File:

H.W. EL = 888.74 ft **Normal Water Level**

T.W. EL. = 888.74 ft

500 yr H.W. EL = 922.12 ft

Top of Ftg EL. = 873.51 ft

State Building Codes

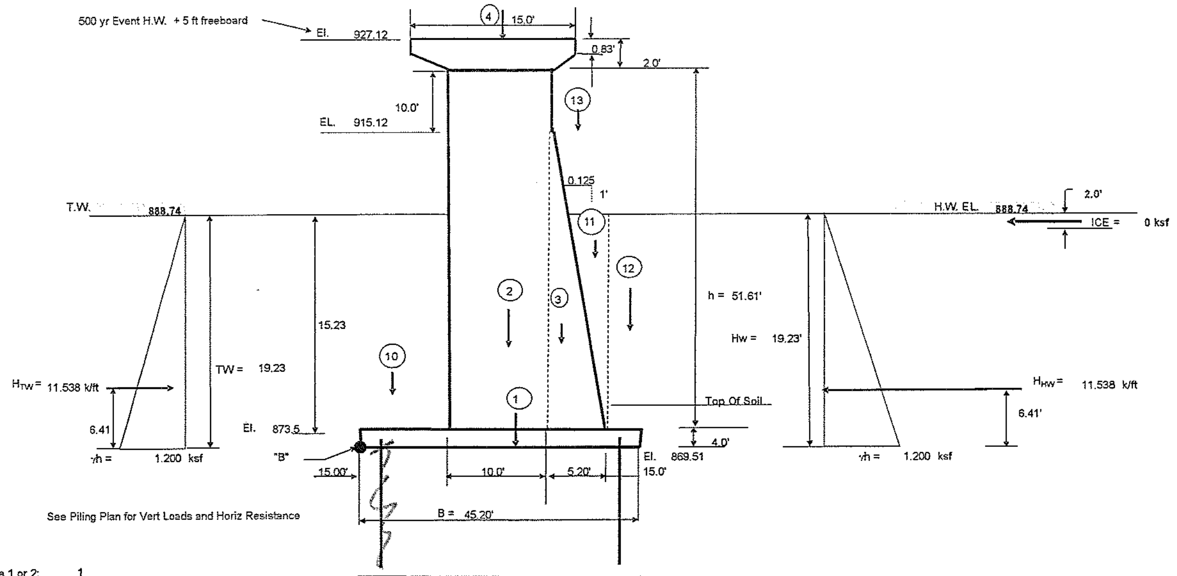
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March

Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section

Length = 30.0 ft

Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



See Piling Plan for Vert Loads and Horiz Resistance

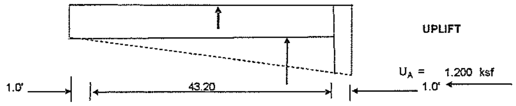
Case 1 or 2: 1

Normal Water Level, El. 888.74 ft

Δh normal = 888.7 ft

See Geotechnical seepage Model

U_B = 1.200 ksf



enter "0" if no cut-off

REVISED PER 7/1/10 MEETING WITH CORPS

Case 1, Full Hydrostatic Head (Dashed Line)

Case 2, Full HW in Front of Upstream sheets

Full TW on down stream of sheets

Non-Overflow Dam

Vertical Loads	**Section**	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	**1**	30	45.20	4.00	0.15	rec	813.6	22.60	18,388.4
Bridge Slab	**4**	30	15.00	0.83	0.15	rec	56.0	20.00	1,120.5
Bridge Slab	**4**	30	12.50	1.17	0.15	rec	65.8	20.00	1,316.3
Monolith	**2**	30	10.00	51.61	0.15	rec	2322.5	20.00	46,449.0
Monolith	**3**	30	5.20	41.61	0.15	tri	487.0	26.73	13,018.1
					D.L. Concrete	ΣVc =	**3744.9**	ΣM_V =	**80,292.2**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	**10**	30	15.00	15.23	0.0624	rec	427.7	7.50	3,207.4
H.W. on Monolith Slope	**11**	30	1.90	15.23	0.0624	tri	27.1	25.63	695.7
H.W. Above Slope	**13**	30	0.00	-26.38	0.0624	rec	0.0	25.00	0.0
H.W. on Mono. ftg	**12**	30	15.00	15.23	0.0624	rec	427.7	37.70	16,123.3
					D.L. Water	ΣVw =	**882.5**	ΣM_V =	**20,026.4**

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	45.20	1.200	rec	-1627.2	22.60	-36,775
	U_A	30	45.20	0.000	tri	0.0	30.13	0
					ΣU =	**-1627.2**	ΣM_U =	**-36,775**

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	2.00	0.00	rec	0.0	18.23	0.0

REVISED PER 2/1/11 MEETING WITH CORPS

Ice Loading is for Ice Floe on single Pier Only on Gated Structure

use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{TW}	30	11.538	tri	346.13	6.41	2218.67
H_{HW}	30	-11.538	tri	-346.13	6.41	-2218.67
			ΣW =	**0.00**	ΣM_W =	**0.0**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-36775	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	100319	kip-ft

Sum of Moments	$\Sigma M_{net} = M_R + M_{OT}$ =	**63,543**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**3,000**	kips
Sum of Horizontal Forces	H = Ice + water =	**0**	kips

Location of Resultant

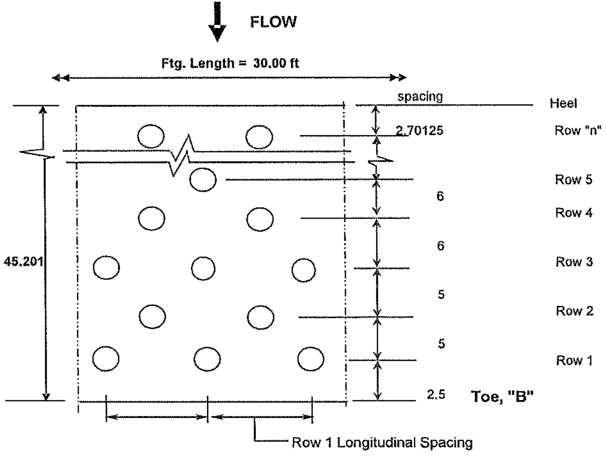
$X_r = \Sigma M / P$ = 21.18 ft from Toe

$e = B/2 - X_r$ = 1.42 ft

$B/6$ = 7.534 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/4/11		PKN	SUBJECT	**Pile Capacity** **Control Structure: 35k**	SECTION A

PILE FOUNDATION DESIGN



L = 30.00 ft

PILE PATTERN GEOMETRY

Transverse Spacing		Distance to Toe, d_{toe}	Longitudinal Spacing	Batter	Piles per Row (N)	Edge Dist (ft)		Trial N	
Row 1 to Toe	**2.50 ft**	2.5 ft	**4.25 ft**	0 "/12"	7	2.25	1	8	
Row 1 to Row 2	**5.00 ft**	7.5 ft	**4.25 ft**	0 "/12"	7	2.25	2	8	14
Row 2 to Row 3	**5.00 ft**	12.5 ft	**4.25 ft**	0 "/12"	7	2.25	3	8	21
Row 3 to Row 4	**6.00 ft**	18.5 ft	**5.00 ft**	0 "/12"	6	2.50	4	7	27
Row 4 to Row 5	**6.00 ft**	24.5 ft	**5.00 ft**	0 "/12"	6	2.50	5	7	33
Row 5 to Row 6	**6.00 ft**	30.5 ft	**5.00 ft**	0 "/12"	6	2.50	6	7	39
Row 6 to Row 7	**6.00 ft**	36.5 ft	**5.00 ft**	0 "/12"	6	2.50	7	7	45
Row 7 to Row 8	**6.00 ft**	42.5 ft	**5.00 ft**	0 "/12"	6	2.50	8	7	51
Row 8 to Row 9	**0.00 ft**	0.0 ft	**0.00 ft**	0 "/12"	0	15.00	0	0	
Row 9 to Row 10	**0.00 ft**	0.0 ft	**0.00 ft**	0 "/12"	0	15.00	0	0	
Row 10 to Row 11	**0.00 ft**	0.0 ft	**0.00 ft**	0 "/12"	0	15.00	0	0	
Row 11 to Row 12	**0.00 ft**	0.0 ft	**0.00 ft**	0 "/12"	0	15.00	0	0	
Row 12 to Row 13	**0.00 ft**	0.0 ft	**0.00 ft**	0 "/12"	0	15.00	0	0	
Row 13 to Row 14	**0.00 ft**	0.0 ft	**0.00 ft**	0 "/12"	0	15.00	0	0	
Row 14 to Row 15	**0.00 ft**	0.0 ft	**0.00 ft**	0 "/12"	0	15.00	0	0	
Last Row to Heel	2.70 ft								
	45.20 ft				ΣN = 51			59	

Note: Enter 0 for Longitudinal Spacing for Rows Not Used)

Pile Properties: Pile Type: **HP** (C.I.P or HP) Pile Length = **34.7 ft**

HP Nominal Depth, h = **14.0 in**

Wt. per ft, plf **74**

Total pile Length = 1,770 LF

Ftg EL. 869.51

Pile Tip El. 835.8

Pile Cap Embed = **1.00 ft**

Pile Group Properties

N.A. of Pile Group to Toe

$X_{NA} = (\Sigma N * d_{toe}) / \Sigma N =$ **21.03 ft**

	Dist. From N.A. to Pile Row	d	N	I = N * d²
1	Dist. To Row 1	18.53 ft	7	2403.4
2	Dist. To Row 2	13.53 ft	7	1281.3
3	Dist. Row 3	8.53 ft	7	509.3
4	Dist. Row 4	2.53 ft	6	38.4
5	Dist. Row 5	-3.47 ft	6	72.3
6	Dist. Row 6	-9.47 ft	6	538.2
7	Dist. Row 7	-15.47 ft	6	1436.0
8	Dist. Row 8	-21.47 ft	6	2765.9
0	Row 9 (not used)	0.00 ft	0	0.0
0	Row 10 (not used)	0.00 ft	0	0.0
0	Row 11 (not used)	0.00 ft	0	0.0
0	Row 12 (not used)	0.00 ft	0	0.0
0	Row 13 (not used)	0.00 ft	0	0.0
0	Row 14 (not used)	0.00 ft	0	0.0
0	Row 15 (not used)	0.00 ft	0	0.0
			51	Σ I = **9044.7**

ALLOWABLE LOADS (from Geotechnical)

Service Case	Allowable Pile Loads						
	1 Usual	1.1 Unsual	2 Unusual	2.1 Extreme	3 Unusual	4 Usual	
Axial	59.8 tons	79.7 tons	79.7 tons	104.0 tons	79.7 tons	31.8 tons	
Tension	40.6 tons	54.1 tons	54.1 tons	70.6 tons	54.1 tons	7.6 tons	
Lateral	31.0 kips/pile	35.0 kips/pile	35.0 kips/pile	35.0 kips/pile	30.0 kips/pile	22.0 kips/pile	**w/o Group effects**

Summary Pile Reactions

̲oad Combinations	Allowable Pile Loads	Pile Loads (tons/pile) 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	Max. Vertical Load (Tons)	Horiz Pile Group Capacity (k)	
1	**59.8 tons**	57.1	48.5	39.9	29.6	19.3	9.0	-1.4	-11.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	57.1	1,581	OK
1.1	**79.7 tons**	57.9	49.1	40.3	29.7	19.1	8.6	-2.0	-12.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	57.9	1,785	OK
2	**79.7 tons**	39.7	35.4	31.0	25.8	20.5	15.3	10.1	4.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	39.7	1,785	OK
2.1	**104.0 tons**	53.3	45.2	37.1	27.4	17.7	8.0	-1.7	-11.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	53.3	1,785	OK
3	**79.7 tons**	36.8	36.8	36.8	36.7	36.7	36.7	36.6	36.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	36.8	1,785	OK
4	**31.8 tons**	28.9	29.1	29.2	29.3	29.5	29.6	29.8	29.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	29.9	1,122	OK
																Max Service : P =	**57.9**		

Using solid mechanics equations adapted for discrete elements, the forces in the pile rows for different load combinations are determined.
The force in each pile row is found using:

$$\text{Pile Load} = P / N + M_{NA} / I$$

First, the moment about the toe must be translated to get the moment about the neutral axis of the pile group.

$$e_{toe} = M_{toe} / P$$

Then the eccentricity about the neutral axis of the pile group is

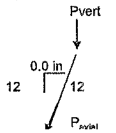
$$e_{NA} = X_{NA} - e_{toe}$$

The moment about the neutral axis of the pile group becomes

$$M_{NA} = P * e_{NA}$$

For battered pile, the Vertical pile load needs to be transformed to the axial load along the pile axis

$$P_{axial} = 1.000 P_{vert}$$



BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER		
			SUBJECT	**Pile Capacity** **Control Structure: 35k**	SECTION A

axial

FORCE RESULTANT (see Stability Analysis)

CASE	Event	Vertical Load P (kips)	Horizontal Load H (kips)	ΣM $_{toe}$ (kip-ft)	e $_{toe}$ = M $_{toe}$ / P (ft)	e $_{NA}$ = X_{NA} - e $_{toe}$ (ft)	M_{NA} = P * e $_{NA}$ (kip-ft)	
1	**100**	2,575	1,567	23,047	8.95	12.08	31099	Usual
1.1	**100**	2,575	1,582	22,267	8.65	12.38	31879	Unusual
2	**500**	2,404	704	34,778	14.47	6.56	15768	Unusual
2.1	**500**	2,381	1,220	20,816	8.74	12.29	29256	Extreme
3	**Dry**	3,745	57	78,649	21.00	0.03	103	Unusual
4	**Daily**	3,000	0	63,543	21.18	-0.15	-452	Usual

SERVICE

Case **1**
Flood Event **100**
Usual

Vertical Load, P = 2575 kips
Horizontal Load, H = 1567 kips
M_{NA} = 31099 kip-ft 51

	Vertical Pile Loading	P / N	+ M_{NA}* d / Σ I	= Pile Loads		**Axial Pile Load**
1	Row 1	50.5	63.7	114.2 kips/pile	57.1 tons/pile	57.1 tons/pile
2	Row 2	50.5	46.5	97.0 kips/pile	48.5 tons/pile	48.5 tons/pile
3	Row 3	50.5	29.3	79.8 kips/pile	39.9 tons/pile	39.9 tons/pile
4	Row 4	50.5	8.7	59.2 kips/pile	29.6 tons/pile	29.6 tons/pile
5	Row 5	50.5	-11.9	38.6 kips/pile	19.3 tons/pile	19.3 tons/pile
6	Row 6	50.5	-32.6	17.9 kips/pile	9.0 tons/pile	9.0 tons/pile
7	Row 7	50.5	-53.2	-2.7 kips/pile	-1.4 tons/pile	-1.4 tons/pile
8	Row 8	50.5	-73.8	-23.3 kips/pile	-11.7 tons/pile	-11.7 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	**57.1 tons/pile**	max: **57.1 tons/pile**

Assumed lateral Capacity: 31.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	7	0.0	217	1.000	217 kips	
2	Row 2	0	7	0.0	217	1.000	217 kips	
3	Row 3	0	7	0.0	217	1.000	217 kips	
4	Row 4	0	6	0.0	186	1.000	186 kips	
5	Row 5	0	6	0.0	186	1.000	186 kips	
6	Row 6	0	6	0.0	186	1.000	186 kips	
7	Row 7	0	6	0.0	186	1.000	186 kips	
8	Row 8	0	6	0.0	186	1.000	186 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			51		1581		**1581 kips**	**OK**

Case **1.1**
Flood Event **100**
Unusual

Vertical Load, P = 2575 kips
Horizontal Load, H = 1582 kips
M_{NA} = 31879 kip-ft 51

	Vertical Pile Loading	P / N	+ M_{NA}* d / Σ I	= Pile Loads		**Axial Pile Load**
1	Row 1	50.5	65.3	115.8 kips/pile	57.9 tons/pile	57.9 tons/pile
2	Row 2	50.5	47.7	98.2 kips/pile	49.1 tons/pile	49.1 tons/pile
3	Row 3	50.5	30.1	80.5 kips/pile	40.3 tons/pile	40.3 tons/pile
4	Row 4	50.5	8.9	59.4 kips/pile	29.7 tons/pile	29.7 tons/pile
5	Row 5	50.5	-12.2	38.3 kips/pile	19.1 tons/pile	19.1 tons/pile
6	Row 6	50.5	-33.4	17.1 kips/pile	8.6 tons/pile	8.6 tons/pile
7	Row 7	50.5	-54.5	-4.0 kips/pile	-2.0 tons/pile	-2.0 tons/pile
8	Row 8	50.5	-75.7	-25.2 kips/pile	-12.6 tons/pile	-12.6 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile

12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	**57.9 tons/pile**	max: **57.9 tons/pile**

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	7	0.0	245	1.000	245 kips	
2	Row 2	0	7	0.0	245	1.000	245 kips	
3	Row 3	0	7	0.0	245	1.000	245 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	6	0.0	210	1.000	210 kips	
7	Row 7	0	6	0.0	210	1.000	210 kips	
8	Row 8	0	6	0.0	210	1.000	210 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			51		1785		**1785 kips**	**OK**

Case **2**
Flood Event **500**
Unusual

Vertical Load, P = 2404 kips
Horizontal Load, H = 704 kips
M_{NA} = 15768 kip-ft

	Vertical Pile Loading	P / N +	M_{NA} * d / Σ I	= Pile Loads		Axial Pile Load
1	Row 1	47.1	32.3	79.4 kips/pile	39.7 tons/pile	39.7 tons/pile
2	Row 2	47.1	23.6	70.7 kips/pile	35.4 tons/pile	35.4 tons/pile
3	Row 3	47.1	14.9	62.0 kips/pile	31.0 tons/pile	31.0 tons/pile
4	Row 4	47.1	4.4	51.5 kips/pile	25.8 tons/pile	25.8 tons/pile
5	Row 5	47.1	-6.1	41.1 kips/pile	20.5 tons/pile	20.5 tons/pile
6	Row 6	47.1	-16.5	30.6 kips/pile	15.3 tons/pile	15.3 tons/pile
7	Row 7	47.1	-27.0	20.2 kips/pile	10.1 tons/pile	10.1 tons/pile
8	Row 8	47.1	-37.4	9.7 kips/pile	4.8 tons/pile	4.8 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	**39.7 tons/pile**	max: **39.7 tons/pile**

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	7	0.0	245	1.000	245 kips	
2	Row 2	0	7	0.0	245	1.000	245 kips	
3	Row 3	0	7	0.0	245	1.000	245 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	6	0.0	210	1.000	210 kips	
7	Row 7	0	6	0.0	210	1.000	210 kips	
8	Row 8	0	6	0.0	210	1.000	210 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			51		1785		**1785 kips**	**OK**

Case **2.1**
Flood Event **500**
Extreme

Vertical Load, P = 2381 kips
Horizontal Load, H = 1220 kips
M_{NA} = 29256 kip-ft

Vertical Pile Loading	P / N +	M_{NA} * d / Σ I	= Pile Loads	Axial Pile Load

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			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/4/11		PKN	SUBJECT	**Pile Capacity** **Control Structure: 35k**	SECTION A

1	Row 1	46.7	59.9	106.6 kips/pile	53.3 tons/pile	53.3 tons/pile
2	Row 2	46.7	43.8	90.5 kips/pile	45.2 tons/pile	45.2 tons/pile
3	Row 3	46.7	27.6	74.3 kips/pile	37.1 tons/pile	37.1 tons/pile
4	Row 4	46.7	8.2	54.9 kips/pile	27.4 tons/pile	27.4 tons/pile
5	Row 5	46.7	-11.2	35.5 kips/pile	17.7 tons/pile	17.7 tons/pile
6	Row 6	46.7	-30.6	16.1 kips/pile	8.0 tons/pile	8.0 tons/pile
7	Row 7	46.7	-50.0	-3.4 kips/pile	-1.7 tons/pile	-1.7 tons/pile
8	Row 8	46.7	-69.4	-22.8 kips/pile	-11.4 tons/pile	-11.4 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	**53.3 tons/pile**	max: **53.3 tons/pile**

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	7	0.0	245	1.000	245 kips	
2	Row 2	0	7	0.0	245	1.000	245 kips	
3	Row 3	0	7	0.0	245	1.000	245 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	6	0.0	210	1.000	210 kips	
7	Row 7	0	6	0.0	210	1.000	210 kips	
8	Row 8	0	6	0.0	210	1.000	210 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			51		1785		**1785 kips**	**OK**

Case **3**
Flood Event **Dry**
Unusual

Vertical Load, P = 3745 kips
Horizontal Load, H = 57 kips
M_{NA} = 103 kip-ft

	Vertical Pile Loading	P / N +	M_{NA} * d / Σ I	= Pile Loads			**Axial Pile Load**
1	Row 1	73.4	0.2	73.6 kips/pile	36.8 tons/pile		36.8 tons/pile
2	Row 2	73.4	0.2	73.6 kips/pile	36.8 tons/pile		36.8 tons/pile
3	Row 3	73.4	0.1	73.5 kips/pile	36.8 tons/pile		36.8 tons/pile
4	Row 4	73.4	0.0	73.5 kips/pile	36.7 tons/pile		36.7 tons/pile
5	Row 5	73.4	0.0	73.4 kips/pile	36.7 tons/pile		36.7 tons/pile
6	Row 6	73.4	-0.1	73.3 kips/pile	36.7 tons/pile		36.7 tons/pile
7	Row 7	73.4	-0.2	73.3 kips/pile	36.6 tons/pile		36.6 tons/pile
8	Row 8	73.4	-0.2	73.2 kips/pile	36.6 tons/pile		36.6 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
				max:	**36.8 tons/pile**	max:	**36.8 tons/pile**

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance
1	Row 1	0	7	0.0	245	1.000	245 kips
2	Row 2	0	7	0.0	245	1.000	245 kips
3	Row 3	0	7	0.0	245	1.000	245 kips
4	Row 4	0	6	0.0	210	1.000	210 kips
5	Row 5	0	6	0.0	210	1.000	210 kips
6	Row 6	0	6	0.0	210	1.000	210 kips
7	Row 7	0	6	0.0	210	1.000	210 kips
8	Row 8	0	6	0.0	210	1.000	210 kips
9	Row 9	0	0	0.0	0	1.000	0 kips
10	Row 10	0	0	0.0	0	1.000	0 kips
11	Row 11	0	0	0.0	0	1.000	0 kips
12	Row 12	0	0	0.0	0	1.000	0 kips
13	Row 13	0	0	0.0	0	1.000	0 kips
14	Row 14	0	0	0.0	0	1.000	0 kips
15	Row 15	0	0	0.0	0	1.000	0 kips

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER		
			SUBJECT	**Pile Capacity** **Control Structure: 35k**	SECTION A

51	1785	**1785 kips**	**OK**

Case **4**
Flood Event **Daily**
Usual

Vertical Load, P = 3000 kips
Horizontal Load, H = 0 kips
M_{NA} = -452 kip-ft

	Vertical Pile Loading	P / N +	M_{NA} * d / Σ I	= Pile Loads		**Axial Pile Load**
1	Row 1	58.8	-0.9	57.9 kips/pile	28.9 tons/pile	28.9 tons/pile
2	Row 2	58.8	-0.7	58.1 kips/pile	29.1 tons/pile	29.1 tons/pile
3	Row 3	58.8	-0.4	58.4 kips/pile	29.2 tons/pile	29.2 tons/pile
4	Row 4	58.8	-0.1	58.7 kips/pile	29.3 tons/pile	29.3 tons/pile
5	Row 5	58.8	0.2	59.0 kips/pile	29.5 tons/pile	29.5 tons/pile
6	Row 6	58.8	0.5	59.3 kips/pile	29.6 tons/pile	29.6 tons/pile
7	Row 7	58.8	0.8	59.6 kips/pile	29.8 tons/pile	29.8 tons/pile
8	Row 8	58.8	1.1	59.9 kips/pile	29.9 tons/pile	29.9 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	**29.9 tons/pile**	**max:** **29.9 tons/pile**

Assumed lateral Capacity: 22.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	7	0.0	154	1.000	154 kips	
2	Row 2	0	7	0.0	154	1.000	154 kips	
3	Row 3	0	7	0.0	154	1.000	154 kips	
4	Row 4	0	6	0.0	132	1.000	132 kips	
5	Row 5	0	6	0.0	132	1.000	132 kips	
6	Row 6	0	6	0.0	132	1.000	132 kips	
7	Row 7	0	6	0.0	132	1.000	132 kips	
8	Row 8	0	6	0.0	132	1.000	132 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			51		1122		**1122 kips**	**OK**

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.	
			PROJECT NAME	RRN Control Structure		
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER			
PKN		PKN	SUBJECT	ACOE Stability Requirements		
2/4/11				SECTION A		

EM 1110-2-2100	Stability Analysis of Concrete Structures	12/1/2005		* Pile-founded structures are not included
EM 1110-2-2200	Gravity Dam Design	6/30/1995		
EM 1110-2-2104	Strength Design for Reinforced-Concrete Hydraulic Structures	6/30/1992	8/20/2003	
EM 1110-2-2906	Design of Pile Foundations	1/15/1991		

See 6/21/10, response from COE to critical path check list item:
Follow-up on structural design criteria

Load Condition Pobabilities			
Case:	1	2	3
	Usual	**Unusual**	**Unusual**
Return Period, yrs:	100	500 + free Bd	
allowable stress increase:	1	33%	33%
Lateral pile deflection, in.:	0.67	0.875	0.67
Ice thickness, ft:	2	2	0	Ice

Piles		**Minimum Factor of Saftey, Ultimate axial capacity**			EM 1110-2-2906	Ratios	
		Usual	**Unusual**	**Extreme**		Usual/ Unusual	Usual/ Extreme
	compression	2.00	1.50	1.15	verified by load tests	1.333	1.739
	tension	2.00	1.50	1.15		1.333	1.739
	compression	2.50	1.90	1.40	verified by pile driver analyzer	1.316	1.786
	tension	3.00	2.25	1.70		1.333	1.765
	compression	3.00	2.25	1.70	not verified by load test	1.333	1.765
	tension	3.00	2.25	1.70		1.333	1.765

	Usual	**Unusual**	**Extreme**	
EM 1110-2-2906, P. 4-9		1.33	1.75	← Increase per allowable stress increase
Vertical displacemet	**0.25 in**	0.33 in	0.44 in	
Horizonatl Displacement	**0.50 in**	0.67 in	0.88 in	

Pile Capacity:	HP14x73							
	Usual	**Unusual**	**Unusual**	**Extreme**	**Unusual**	**Usual**		
Case	**1**	**1.1**	**2**	**2.1**	**3**	**4**	←	**Note**
Pile Tip El.	**835.8**	835.8	835.8	835.8	835.8	835.8		Case 1, 2, 3: Flood & Construction loading using Undrained Analysis
Ultimate Axial capacity:	**239.1 k**	239.1 k	239.1 k	239.1 k	239.1 k	**127.2 k**		Case 4: Long-term loading using Drained Analysis
Ultimate Uplift capacity:	**162.4 k**	162.4 k	162.4 k	162.4 k	162.4 k	30.2 k		Cases 1.1 & 2.1 include ice, use next higher load classification
Horizonatl Displacement:	0.67 in	0.875 in	0.875 in	1.000 in	0.67 in	0.50 in	←	Per Corp meeting 2/1/11
Allow. Lateral Capacity:	**31 k**	**35 k**	**35 k**	**35 k**	**30 k**	**22 k**	←	FOR PHASE 4 : Use GROUP FOR Lateral capacity
FS:	2.00	1.50	1.50	1.15	1.50	2.00	←	Per Corp meeting 2/1/11
Allow. Axial Compression:	119.6 k	159.4 k	159.4 k	207.9 k	159.4 k	63.6 k		
Allow. Axial Tension:	81.2 k	108.3 k	108.3 k	141.2 k	108.3 k	15.1 k		

Pile P-Multipliers, Pm for multiple Row Shading (ave from Hannigan....2005) **See AASHTO P. 10-88**

	1	2	3	4										
Pile Spacing	Row 1	Row 2	Row 3	Row n		B =	**1.1666667**					Ensoft		
3 B	0.7	0.5	0.35	0.35			Row 1	Row 2	Row 3	Row n			s/B	Formula
5 B	1	0.85	0.7	0.7		Pile Spacing	5.00 ft	5.00 ft	6.00 ft	**6.00 ft**	parallel	trailing Pile	5.37	0.5791 $(s/B)^{.3251}$
η_{Long}	0.893	1.000	1.000	1.000	parallel	Ratio s/B	4.286	4.286	5.143	5.143		leading Pile	3.37	0.7309 $(s/B)^{[illegible]}$
η_{Trans}	0.796	0.613	0.463	0.575	perp		4.25 ft	4.25 ft	4.25 ft	**5.00 ft**				
η_{Total}	1.000	1.000	1.000	1.000	Total Lateral reduction Ratio s/B		3.643	3.643	3.643	4.286	perp	side side	3.28	0.5292 $(s/B)^{[illegible]}$

FOR PHASE 4 : NO Pm applied for Lateral capacity

Hydraulic Data - RRN Mainstream - Optimized w/Control Structures

1/20/2011

		Structure:	ND East Red River 35k		
		Event	Headwatei	Tailwater	
		Year	ft	ft	Case
1/20/2011		2	896.85	896.82	
		5	906.37	902.2	
1/20/2011		**10**	914.77	903.05	
		20	910.87	903.23	
1/20/2011		50	920.02	902.11	
1/20/2011		100	922.01	902.41	1
		200	916.79	908.37	
	**	300	918.567	910.383	
1/20/2011		**500**	**922.12**	**914.41**	**2**

** **

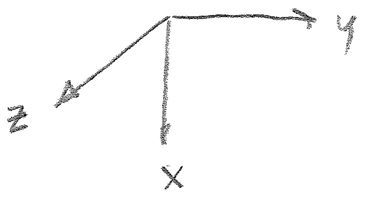
**300 yr event is linearly interploated between 200 & 500 yr event

ATTACHMENT

F-Q1.3 Group Analyses Results for the Red River (LPP) Control Structure Wing Wall A

RRN
WING WALL
SEC A

```
RRN Wingwall A
GROUP8.0.4
* Analysis Parameters
1 3 0 0 0 0 0 1 0 1 0 0
* Pile Cap Dimensions
0.0000000E+00 0.0000000E+00 0.0000000E+00
* Load Cases
2 0 0 1
0 1 0 1 1 100 100
0.0001 0.0001 1 1
Load Case  1
0 0 0 0 1 0 0
2 1 1 0
1   Fx      Fy                  Xc  Yc   Zc
2575000 -1567000 0 0 0 0 0 107.4 180
0                         LOADING LOCATION
0
0 0 0 1 1 100 100
0.0001 0.0001 1 1
Load Case 1.1 w/ice
0 0 0 0 1 0 0
0 1 1 0
1   Fx      Fy                  Xc  Yc    Zc
2575000 -1582000 0 0 0 0 0 148.56 180
0
0
* Load Combinations
0 0 0 1
* Distributed Load Sets
0
* Pile Group Configuration
51               <------  51 PILES
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 27 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 78 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 129 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 180 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 231 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 282 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 333 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 27 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 78 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 129 0 90 -36
2 2 1 0 0 0
```



```
0 0 1 1 1 1 1
-12 90 180 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 231 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 282 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 333 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 27 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 78 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 129 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 180 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 231 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 282 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 333 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 294 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 294 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 294 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 294 210 0 90 -36
2 2 1 0 0 0
```

```
0 0 1 1 1 1 1
-12 294 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 294 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
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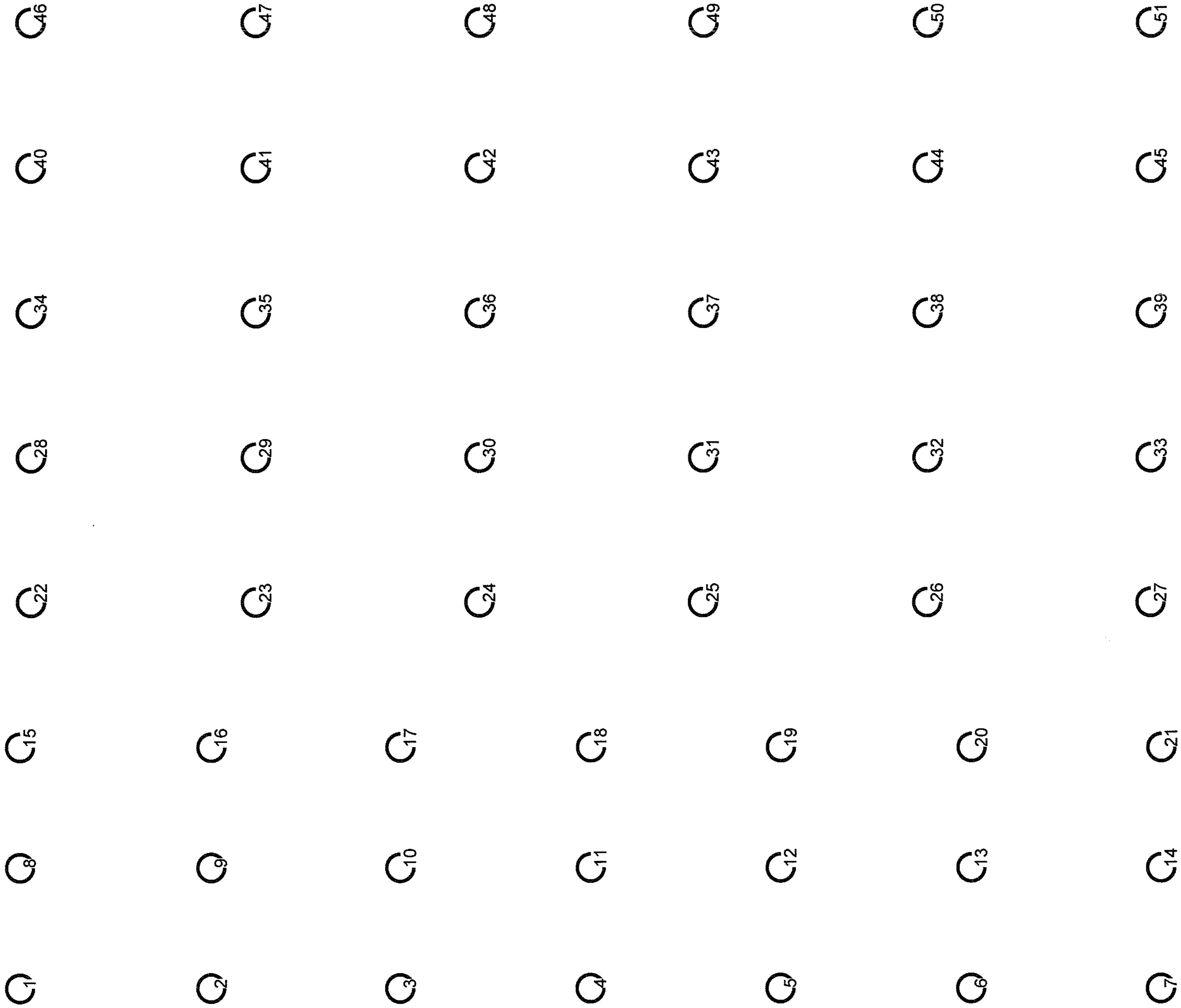
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 438 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 438 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 438 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 438 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 438 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 330 0 90 -36
* Pile Properties
1
0 1
100 900 29000000 1
```

```
0 900 1 0 0
* Pile Cross Sections
1
0 0 4
HP14x89
13.83 0 14.585 13.83 0 0 0.615 0.615
1 26.1 904 326 1808 2.09728E10 0 0 0 0
* Soil Layers
3
0 1
0 324
2 10
0.0235 4.514 0 0 0 0.01 0 0 0 0
0.0235 4.514 0 0 0 0.01 0 0 0 0
1 3
324 384
2 10
0.03449 13.194 0 0 0 0.007 0 0 0 0
0.03449 13.194 0 0 0 0.007 0 0 0 0
2 3
384 1200
2 10
0.03449 13.194 0 0 0 0.007 0 0 0 0
0.03449 13.194 0 0 0 0.007 0 0 0 0
* End of file
```


46
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31
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6
7

```
--------------------------------------------------------------------------
                     Time and Date of Analysis
--------------------------------------------------------------------------

          Date:  February 03, 2011     Time:  13:05:52



            *****     COMPUTATION RESULTS     *****


RRN Wingwall A



            *****     LOAD CASES RESULTS      *****


LOAD CASE :      1
CASE NAME : Load Case  1
LOAD TYPE : Dead, DL
```

CASE 1 100 YR FLOOD

```
  REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS, COMBINED Y AND Z DIRECTIONS
  ESTIMATED USING MOVEMENT IN THE DIRECTION OF PILE CAP DISPLACEMENTS

       GROUP NO      P-FACTOR      Y-FACTOR

           1          0.9974        1.0000
           2          0.9948        1.0000
           3          0.9948        1.0000
           4          0.9948        1.0000
           5          0.9948        1.0000
           6          0.9948        1.0000
           7          0.9974        1.0000
           8          0.8025        1.0000
           9          0.7682        1.0000
          10          0.7682        1.0000
          11          0.7682        1.0000
          12          0.7682        1.0000
          13          0.7682        1.0000
          14          0.8025        1.0000
          15          0.8025        1.0000
          16          0.7682        1.0000
          17          0.7682        1.0000
          18          0.7682        1.0000
          19          0.7682        1.0000
          20          0.7682        1.0000
          21          0.8025        1.0000
          22          0.8764        1.0000
          23          0.8666        1.0000
          24          0.8622        1.0000
          25          0.8622        1.0000
          26          0.8666        1.0000
          27          0.8764        1.0000
          28          0.8949        1.0000
          29          0.8913        1.0000
          30          0.8913        1.0000
          31          0.8913        1.0000
          32          0.8913        1.0000
          33          0.8949        1.0000
          34          0.8949        1.0000
          35          0.8913        1.0000
          36          0.8913        1.0000
          37          0.8913        1.0000
          38          0.8913        1.0000
          39          0.8949        1.0000
          40          0.8949        1.0000
          41          0.8913        1.0000
          42          0.8913        1.0000
          43          0.8913        1.0000
          44          0.8913        1.0000
          45          0.8949        1.0000
          46          0.8949        1.0000
          47          0.8913        1.0000
          48          0.8913        1.0000
```

CASE 2 INFO
STARTS ON P. 11

```
          49            0.8913        1.0000
          50            0.8913        1.0000
          51            0.8949        1.0000


 * TABLE L *   COMPUTATION ON PILE CAP

 * EQUIVALENT CONC. LOAD AT ORIGIN *

       VERT. LOAD, LBS     HOR. LOAD Y, LBS    HOR. LOAD Z, LBS
         2.57500E+06         -1.56700E+06            0.00000

      MOMENT X ,IN-LBS     MOMENT Y,IN-LBS     MOMENT Z,IN-LBS
         2.82060E+08          4.63500E+08        -2.76555E+08

 * DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

        VERTICAL ,IN        HORIZONTAL Y,IN      HORIZONTAL Z,IN
             0.12245             -0.54503          2.00672E-08

      ANGLE ROT. X,RAD     ANGLE ROT. Y,RAD     ANGLE ROT. Z,RAD
        -7.87989E-11          2.29369E-14          3.11077E-04


 * TABLE M *   COMPUTATION ON INDIVIDUAL PILE

   THE GLOBAL STRUCTURAL COORDINATE SYSTEM
   ---------------------------------------

   * PILE TOP DISPLACEMENTS *

  PILE GROUP    DISP. X,IN    DISP. Y,IN    DISP. Z,IN    ROT. X,RAD    ROT. Y,RAD    ROT. Z,RAD
  **********   ***********   ***********   **********    ***********   ***********   ***********
       1           0.1131       -0.5488    1.7704E-08   -7.8799E-11    2.2937E-14    3.1108E-04
       2           0.1131       -0.5488    1.7704E-08   -7.8799E-11    2.2937E-14    3.1108E-04
       3           0.1131       -0.5488    1.7704E-08   -7.8799E-11    2.2937E-14    3.1108E-04
       4           0.1131       -0.5488    1.7704E-08   -7.8799E-11    2.2937E-14    3.1108E-04
       5           0.1131       -0.5488    1.7704E-08   -7.8799E-11    2.2937E-14    3.1108E-04
       6           0.1131       -0.5488    1.7704E-08   -7.8799E-11    2.2937E-14    3.1108E-04
       7           0.1131       -0.5488    1.7704E-08   -7.8799E-11    2.2937E-14    3.1108E-04
       8       9.4449E-02       -0.5488    1.2976E-08   -7.8799E-11    2.2937E-14    3.1108E-04
       9       9.4449E-02       -0.5488    1.2976E-08   -7.8799E-11    2.2937E-14    3.1108E-04
      10       9.4449E-02       -0.5488    1.2976E-08   -7.8799E-11    2.2937E-14    3.1108E-04
      11       9.4449E-02       -0.5488    1.2976E-08   -7.8799E-11    2.2937E-14    3.1108E-04
      12       9.4449E-02       -0.5488    1.2976E-08   -7.8799E-11    2.2937E-14    3.1108E-04
      13       9.4449E-02       -0.5488    1.2976E-08   -7.8799E-11    2.2937E-14    3.1108E-04
      14       9.4449E-02       -0.5488    1.2976E-08   -7.8799E-11    2.2937E-14    3.1108E-04
      15       7.5785E-02       -0.5488    8.2477E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      16       7.5785E-02       -0.5488    8.2477E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      17       7.5785E-02       -0.5488    8.2477E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      18       7.5785E-02       -0.5488    8.2477E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      19       7.5785E-02       -0.5488    8.2477E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      20       7.5785E-02       -0.5488    8.2477E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      21       7.5785E-02       -0.5488    8.2477E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      22       5.3387E-02       -0.5488    2.5741E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      23       5.3387E-02       -0.5488    2.5741E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      24       5.3387E-02       -0.5488    2.5741E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      25       5.3387E-02       -0.5488    2.5741E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      26       5.3387E-02       -0.5488    2.5741E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      27       5.3387E-02       -0.5488    2.5741E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      28       3.0989E-02       -0.5488   -3.0994E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      29       3.0989E-02       -0.5488   -3.0994E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      30       3.0989E-02       -0.5488   -3.0994E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      31       3.0989E-02       -0.5488   -3.0994E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      32       3.0989E-02       -0.5488   -3.0994E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      33       3.0989E-02       -0.5488   -3.0994E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      34       8.5919E-03       -0.5488   -8.7729E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      35       8.5919E-03       -0.5488   -8.7729E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      36       8.5919E-03       -0.5488   -8.7729E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      37       8.5919E-03       -0.5488   -8.7729E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      38       8.5919E-03       -0.5488   -8.7729E-09   -7.8799E-11    2.2937E-14    3.1108E-04
      39       8.5919E-03       -0.5488   -8.7729E-09   -7.8799E-11    2.2937E-14    3.1108E-04
```

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   40      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   41      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   42      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   43      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   44      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   45      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   46      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   47      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   48      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   49      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   50      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   51      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04

 MINIMUM   -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
 Pile N.       46              1         46            1            1            1
 MAXIMUM      0.1131        -0.5488   1.7704E-08  -7.8799E-11   2.2937E-14   3.1108E-04
 Pile N.        1              1          1            1            1            1
                              Axial          Horiz
 * PILE TOP REACTIONS *

PILE GROUP   FOR. X,LBS   FOR. Y,LBS    FOR. Z,LBS  MOM X,LBS-IN MOM Y,LBS-IN MOM Z,LBS-IN   STRESS,LBS/IN**2
**********   **********   **********    **********   **********   **********   **********   ****************
Ton   1      1.2317E+05  -3.3566E+04   7.4621E-04  -7.1459E-10  -4.2804E-02  -2.3170E+06   2.2443E+04
61.58 2      1.2317E+05  -3.3506E+04   7.4485E-04  -7.1459E-10  -4.2752E-02  -2.3141E+06   2.2421E+04
      3      1.2317E+05  -3.3506E+04   7.4485E-04  -7.1459E-10  -4.2752E-02  -2.3141E+06   2.2421E+04
      4  Row 1.2317E+05  -3.3506E+04   7.4485E-04  -7.1459E-10  -4.2752E-02  -2.3141E+06   2.2421E+04
      5      1.2317E+05  -3.3506E+04   7.4485E-04  -7.1459E-10  -4.2752E-02  -2.3141E+06   2.2421E+04
      6   1  1.2317E+05  -3.3506E+04   7.4485E-04  -7.1459E-10  -4.2752E-02  -2.3141E+06   2.2421E+04
      7      1.2317E+05  -3.3566E+04   7.4621E-04  -7.1459E-10  -4.2804E-02  -2.3170E+06   2.2443E+04
      8      1.0656E+05  -2.8865E+04   4.7253E-04  -7.1459E-10  -2.8383E-02  -2.0848E+06   2.0030E+04
      9      1.0656E+05  -2.8003E+04   4.5886E-04  -7.1459E-10  -2.7822E-02  -2.0421E+06   1.9703E+04
53.28 10     1.0656E+05  -2.8003E+04   4.5886E-04  -7.1459E-10  -2.7822E-02  -2.0421E+06   1.9703E+04
     11      1.0656E+05  -2.8003E+04   4.5886E-04  -7.1459E-10  -2.7822E-02  -2.0421E+06   1.9703E+04
     12   2  1.0656E+05  -2.8003E+04   4.5886E-04  -7.1459E-10  -2.7822E-02  -2.0421E+06   1.9703E+04
     13      1.0656E+05  -2.8003E+04   4.5886E-04  -7.1459E-10  -2.7822E-02  -2.0421E+06   1.9703E+04
     14      1.0656E+05  -2.8865E+04   4.7253E-04  -7.1459E-10  -2.8383E-02  -2.0848E+06   2.0030E+04
     15      8.9943E+04  -2.8906E+04   3.0112E-04  -7.1459E-10  -1.8042E-02  -2.0848E+06   1.9393E+04
     16      8.9943E+04  -2.8043E+04   2.9238E-04  -7.1459E-10  -1.7684E-02  -2.0420E+06   1.9066E+04
44.97 17     8.9943E+04  -2.8043E+04   2.9238E-04  -7.1459E-10  -1.7684E-02  -2.0420E+06   1.9066E+04
     18      8.9943E+04  -2.8043E+04   2.9238E-04  -7.1459E-10  -1.7684E-02  -2.0420E+06   1.9066E+04
     19   3  8.9943E+04  -2.8043E+04   2.9238E-04  -7.1459E-10  -1.7684E-02  -2.0420E+06   1.9066E+04
     20      8.9943E+04  -2.8043E+04   2.9238E-04  -7.1459E-10  -1.7684E-02  -2.0420E+06   1.9066E+04
     21      8.9943E+04  -2.8906E+04   3.0112E-04  -7.1459E-10  -1.8042E-02  -2.0848E+06   1.9393E+04
     22      6.9869E+04  -3.0797E+04   1.0008E-04  -7.1459E-10  -5.8619E-03  -2.1763E+06   1.9324E+04
     23      6.9869E+04  -3.0557E+04   9.9297E-05  -7.1459E-10  -5.8314E-03  -2.1645E+06   1.9234E+04
34.93 24     6.9869E+04  -3.0449E+04   9.8948E-05  -7.1459E-10  -5.8176E-03  -2.1592E+06   1.9193E+04
     25   4  6.9869E+04  -3.0449E+04   9.8948E-05  -7.1459E-10  -5.8176E-03  -2.1592E+06   1.9193E+04
     26      6.9869E+04  -3.0557E+04   9.9297E-05  -7.1459E-10  -5.8314E-03  -2.1645E+06   1.9234E+04
     27      6.9869E+04  -3.0797E+04   1.0008E-04  -7.1459E-10  -5.8619E-03  -2.1763E+06   1.9324E+04
     28      4.0257E+04  -3.1321E+04  -1.2294E-04  -7.1459E-10   7.1398E-03  -2.1979E+06   1.8355E+04
     29      4.0257E+04  -3.1234E+04  -1.2257E-04  -7.1459E-10   7.1261E-03  -2.1938E+06   1.8323E+04
     30      4.0257E+04  -3.1234E+04  -1.2257E-04  -7.1459E-10   7.1261E-03  -2.1938E+06   1.8323E+04
     31   5  4.0257E+04  -3.1234E+04  -1.2257E-04  -7.1459E-10   7.1261E-03  -2.1938E+06   1.8323E+04
20.13 32     4.0257E+04  -3.1234E+04  -1.2257E-04  -7.1459E-10   7.1261E-03  -2.1938E+06   1.8323E+04
     33      4.0257E+04  -3.1321E+04  -1.2294E-04  -7.1459E-10   7.1398E-03  -2.1979E+06   1.8355E+04
     34      1.1048E+04  -3.1395E+04  -3.4925E-04  -7.1459E-10   2.0203E-02  -2.1979E+06   1.7235E+04
     35      1.1048E+04  -3.1308E+04  -3.4825E-04  -7.1459E-10   2.0164E-02  -2.1936E+06   1.7203E+04
5.524 36     1.1048E+04  -3.1308E+04  -3.4825E-04  -7.1459E-10   2.0164E-02  -2.1936E+06   1.7203E+04
     37   6  1.1048E+04  -3.1308E+04  -3.4825E-04  -7.1459E-10   2.0164E-02  -2.1936E+06   1.7203E+04
     38      1.1048E+04  -3.1308E+04  -3.4825E-04  -7.1459E-10   2.0164E-02  -2.1936E+06   1.7203E+04
     39      1.1048E+04  -3.1395E+04  -3.4925E-04  -7.1459E-10   2.0203E-02  -2.1979E+06   1.7235E+04
     40     -1.7814E+04  -3.1467E+04  -5.7734E-04  -7.1459E-10   3.3269E-02  -2.1977E+06   1.7494E+04
     41     -1.7814E+04  -3.1382E+04  -5.7580E-04  -7.1459E-10   3.3208E-02  -2.1937E+06   1.7463E+04
-8.907 42   -1.7814E+04  -3.1382E+04  -5.7580E-04  -7.1459E-10   3.3208E-02  -2.1937E+06   1.7463E+04
     43   7 -1.7814E+04  -3.1382E+04  -5.7580E-04  -7.1459E-10   3.3208E-02  -2.1937E+06   1.7463E+04
     44     -1.7814E+04  -3.1382E+04  -5.7580E-04  -7.1459E-10   3.3208E-02  -2.1937E+06   1.7463E+04
     45     -1.7814E+04  -3.1467E+04  -5.7734E-04  -7.1459E-10   3.3269E-02  -2.1977E+06   1.7494E+04
     46     -4.7139E+04  -3.1542E+04  -8.0744E-04  -7.1459E-10   4.6342E-02  -2.1977E+06   1.8617E+04
     47     -4.7139E+04  -3.1455E+04  -8.0521E-04  -7.1459E-10   4.6256E-02  -2.1935E+06   1.8585E+04
     48   8 -4.7139E+04  -3.1455E+04  -8.0521E-04  -7.1459E-10   4.6256E-02  -2.1935E+06   1.8585E+04
-23.569 49  -4.7139E+04  -3.1455E+04  -8.0521E-04  -7.1459E-10   4.6256E-02  -2.1935E+06   1.8585E+04
     50     -4.7139E+04  -3.1455E+04  -8.0521E-04  -7.1459E-10   4.6256E-02  -2.1935E+06   1.8585E+04
     51     -4.7139E+04  -3.1542E+04  -8.0744E-04  -7.1459E-10   4.6342E-02  -2.1977E+06   1.8617E+04

 MINIMUM    -4.7139E+04  -3.3566E+04  -8.0744E-04  -7.1459E-10  -4.2804E-02  -2.3170E+06   1.7203E+04
 Pile N.        46            1           46            1            1            1             35
 MAXIMUM     1.2317E+05  -2.8003E+04   7.4621E-04  -7.1459E-10   4.6342E-02  -2.0420E+06   2.2443E+04
 Pile N.         1            9            1            1           46           16              1
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 THE PILE COORDINATE SYSTEM (LOCAL AXES)
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 * PILE TOP DISPLACEMENTS *

PILE GROUP   DISP. x,IN   DISP. y,IN   DISP. z,IN    ROT. x,RAD   ROT. y,RAD   ROT. z,RAD
**********  ***********  ***********  ***********  ***********  ***********  ***********
    1           0.1131      -0.5488   1.7704E-08  -7.8799E-11   2.2937E-14   3.1108E-04
    2           0.1131      -0.5488   1.7704E-08  -7.8799E-11   2.2937E-14   3.1108E-04
    3           0.1131      -0.5488   1.7704E-08  -7.8799E-11   2.2937E-14   3.1108E-04
    4           0.1131      -0.5488   1.7704E-08  -7.8799E-11   2.2937E-14   3.1108E-04
    5           0.1131      -0.5488   1.7704E-08  -7.8799E-11   2.2937E-14   3.1108E-04
    6           0.1131      -0.5488   1.7704E-08  -7.8799E-11   2.2937E-14   3.1108E-04
    7           0.1131      -0.5488   1.7704E-08  -7.8799E-11   2.2937E-14   3.1108E-04
    8       9.4449E-02      -0.5488   1.2976E-08  -7.8799E-11   2.2937E-14   3.1108E-04
    9       9.4449E-02      -0.5488   1.2976E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   10       9.4449E-02      -0.5488   1.2976E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   11       9.4449E-02      -0.5488   1.2976E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   12       9.4449E-02      -0.5488   1.2976E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   13       9.4449E-02      -0.5488   1.2976E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   14       9.4449E-02      -0.5488   1.2976E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   15       7.5785E-02      -0.5488   8.2477E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   16       7.5785E-02      -0.5488   8.2477E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   17       7.5785E-02      -0.5488   8.2477E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   18       7.5785E-02      -0.5488   8.2477E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   19       7.5785E-02      -0.5488   8.2477E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   20       7.5785E-02      -0.5488   8.2477E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   21       7.5785E-02      -0.5488   8.2477E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   22       5.3387E-02      -0.5488   2.5741E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   23       5.3387E-02      -0.5488   2.5741E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   24       5.3387E-02      -0.5488   2.5741E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   25       5.3387E-02      -0.5488   2.5741E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   26       5.3387E-02      -0.5488   2.5741E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   27       5.3387E-02      -0.5488   2.5741E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   28       3.0989E-02      -0.5488  -3.0994E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   29       3.0989E-02      -0.5488  -3.0994E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   30       3.0989E-02      -0.5488  -3.0994E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   31       3.0989E-02      -0.5488  -3.0994E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   32       3.0989E-02      -0.5488  -3.0994E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   33       3.0989E-02      -0.5488  -3.0994E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   34       8.5919E-03      -0.5488  -8.7729E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   35       8.5919E-03      -0.5488  -8.7729E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   36       8.5919E-03      -0.5488  -8.7729E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   37       8.5919E-03      -0.5488  -8.7729E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   38       8.5919E-03      -0.5488  -8.7729E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   39       8.5919E-03      -0.5488  -8.7729E-09  -7.8799E-11   2.2937E-14   3.1108E-04
   40      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   41      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   42      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   43      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   44      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   45      -1.3806E-02      -0.5488  -1.4446E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   46      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   47      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   48      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   49      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   50      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
   51      -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04

 MINIMUM   -3.6203E-02      -0.5488  -2.0120E-08  -7.8799E-11   2.2937E-14   3.1108E-04
 Pile N.        46             1          46            1            1            1
 MAXIMUM        0.1131      -0.5488   1.7704E-08  -7.8799E-11   2.2937E-14   3.1108E-04
 Pile N.         1             1           1            1            1            1

 * PILE TOP REACTIONS *

PILE GROUP    AXIAL,LBS   LAT. y,LBS   LAT. z,LBS MOM x,LBS-IN MOM y,LBS-IN MOM z,LBS-IN  STRESS,LBS/IN**2
**********  ***********  ***********  *********** ***********  ***********  ***********  ****************
    1       1.2317E+05  -3.3566E+04   7.4621E-04  -7.1459E-10  -4.2804E-02  -2.3170E+06    2.2443E+04
    2       1.2317E+05  -3.3506E+04   7.4485E-04  -7.1459E-10  -4.2752E-02  -2.3141E+06    2.2421E+04
    3       1.2317E+05  -3.3506E+04   7.4485E-04  -7.1459E-10  -4.2752E-02  -2.3141E+06    2.2421E+04
    4       1.2317E+05  -3.3506E+04   7.4485E-04  -7.1459E-10  -4.2752E-02  -2.3141E+06    2.2421E+04
    5       1.2317E+05  -3.3506E+04   7.4485E-04  -7.1459E-10  -4.2752E-02  -2.3141E+06    2.2421E+04
    6       1.2317E+05  -3.3506E+04   7.4485E-04  -7.1459E-10  -4.2752E-02  -2.3141E+06    2.2421E+04
    7       1.2317E+05  -3.3566E+04   7.4621E-04  -7.1459E-10  -4.2804E-02  -2.3170E+06    2.2443E+04
    8       1.0656E+05  -2.8865E+04   4.7253E-04  -7.1459E-10  -2.8383E-02  -2.0848E+06    2.0030E+04
    9       1.0656E+05  -2.8003E+04   4.5886E-04  -7.1459E-10  -2.7822E-02  -2.0421E+06    1.9703E+04
   10       1.0656E+05  -2.8003E+04   4.5886E-04  -7.1459E-10  -2.7822E-02  -2.0421E+06    1.9703E+04
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  11      1.0656E+05  -2.8003E+04   4.5886E-04  -7.1459E-10  -2.7822E-02  -2.0421E+06   1.9703E+04
  12      1.0656E+05  -2.8003E+04   4.5886E-04  -7.1459E-10  -2.7822E-02  -2.0421E+06   1.9703E+04
  13      1.0656E+05  -2.8003E+04   4.5886E-04  -7.1459E-10  -2.7822E-02  -2.0421E+06   1.9703E+04
  14      1.0656E+05  -2.8865E+04   4.7253E-04  -7.1459E-10  -2.8383E-02  -2.0848E+06   2.0030E+04
  15      8.9943E+04  -2.8906E+04   3.0112E-04  -7.1459E-10  -1.8042E-02  -2.0848E+06   1.9393E+04
  16      8.9943E+04  -2.8043E+04   2.9238E-04  -7.1459E-10  -1.7684E-02  -2.0420E+06   1.9066E+04
  17      8.9943E+04  -2.8043E+04   2.9238E-04  -7.1459E-10  -1.7684E-02  -2.0420E+06   1.9066E+04
  18      8.9943E+04  -2.8043E+04   2.9238E-04  -7.1459E-10  -1.7684E-02  -2.0420E+06   1.9066E+04
  19      8.9943E+04  -2.8043E+04   2.9238E-04  -7.1459E-10  -1.7684E-02  -2.0420E+06   1.9066E+04
  20      8.9943E+04  -2.8043E+04   2.9238E-04  -7.1459E-10  -1.7684E-02  -2.0420E+06   1.9066E+04
  21      8.9943E+04  -2.8906E+04   3.0112E-04  -7.1459E-10  -1.8042E-02  -2.0848E+06   1.9393E+04
  22      6.9869E+04  -3.0797E+04   1.0008E-04  -7.1459E-10  -5.8619E-03  -2.1763E+06   1.9324E+04
  23      6.9869E+04  -3.0557E+04   9.9297E-05  -7.1459E-10  -5.8314E-03  -2.1645E+06   1.9234E+04
  24      6.9869E+04  -3.0449E+04   9.8948E-05  -7.1459E-10  -5.8176E-03  -2.1592E+06   1.9193E+04
  25      6.9869E+04  -3.0449E+04   9.8948E-05  -7.1459E-10  -5.8176E-03  -2.1592E+06   1.9193E+04
  26      6.9869E+04  -3.0557E+04   9.9297E-05  -7.1459E-10  -5.8314E-03  -2.1645E+06   1.9234E+04
  27      6.9869E+04  -3.0797E+04   1.0008E-04  -7.1459E-10  -5.8619E-03  -2.1763E+06   1.9324E+04
  28      4.0257E+04  -3.1321E+04  -1.2294E-04  -7.1459E-10   7.1398E-03  -2.1979E+06   1.8355E+04
  29      4.0257E+04  -3.1234E+04  -1.2257E-04  -7.1459E-10   7.1261E-03  -2.1938E+06   1.8323E+04
  30      4.0257E+04  -3.1234E+04  -1.2257E-04  -7.1459E-10   7.1261E-03  -2.1938E+06   1.8323E+04
  31      4.0257E+04  -3.1234E+04  -1.2257E-04  -7.1459E-10   7.1261E-03  -2.1938E+06   1.8323E+04
  32      4.0257E+04  -3.1234E+04  -1.2257E-04  -7.1459E-10   7.1261E-03  -2.1938E+06   1.8323E+04
  33      4.0257E+04  -3.1321E+04  -1.2294E-04  -7.1459E-10   7.1398E-03  -2.1979E+06   1.8355E+04
  34      1.1048E+04  -3.1395E+04  -3.4925E-04  -7.1459E-10   2.0203E-02  -2.1979E+06   1.7235E+04
  35      1.1048E+04  -3.1308E+04  -3.4825E-04  -7.1459E-10   2.0164E-02  -2.1936E+06   1.7203E+04
  36      1.1048E+04  -3.1308E+04  -3.4825E-04  -7.1459E-10   2.0164E-02  -2.1936E+06   1.7203E+04
  37      1.1048E+04  -3.1308E+04  -3.4825E-04  -7.1459E-10   2.0164E-02  -2.1936E+06   1.7203E+04
  38      1.1048E+04  -3.1308E+04  -3.4825E-04  -7.1459E-10   2.0164E-02  -2.1936E+06   1.7203E+04
  39      1.1048E+04  -3.1395E+04  -3.4925E-04  -7.1459E-10   2.0203E-02  -2.1979E+06   1.7235E+04
  40     -1.7814E+04  -3.1467E+04  -5.7734E-04  -7.1459E-10   3.3269E-02  -2.1977E+06   1.7494E+04
  41     -1.7814E+04  -3.1382E+04  -5.7580E-04  -7.1459E-10   3.3208E-02  -2.1937E+06   1.7463E+04
  42     -1.7814E+04  -3.1382E+04  -5.7580E-04  -7.1459E-10   3.3208E-02  -2.1937E+06   1.7463E+04
  43     -1.7814E+04  -3.1382E+04  -5.7580E-04  -7.1459E-10   3.3208E-02  -2.1937E+06   1.7463E+04
  44     -1.7814E+04  -3.1382E+04  -5.7580E-04  -7.1459E-10   3.3208E-02  -2.1937E+06   1.7463E+04
  45     -1.7814E+04  -3.1467E+04  -5.7734E-04  -7.1459E-10   3.3269E-02  -2.1977E+06   1.7494E+04
  46     -4.7139E+04  -3.1542E+04  -8.0744E-04  -7.1459E-10   4.6342E-02  -2.1977E+06   1.8617E+04
  47     -4.7139E+04  -3.1455E+04  -8.0521E-04  -7.1459E-10   4.6256E-02  -2.1935E+06   1.8585E+04
  48     -4.7139E+04  -3.1455E+04  -8.0521E-04  -7.1459E-10   4.6256E-02  -2.1935E+06   1.8585E+04
  49     -4.7139E+04  -3.1455E+04  -8.0521E-04  -7.1459E-10   4.6256E-02  -2.1935E+06   1.8585E+04
  50     -4.7139E+04  -3.1455E+04  -8.0521E-04  -7.1459E-10   4.6256E-02  -2.1935E+06   1.8585E+04
  51     -4.7139E+04  -3.1542E+04  -8.0744E-04  -7.1459E-10   4.6342E-02  -2.1977E+06   1.8617E+04

MINIMUM  -4.7139E+04  -3.3566E+04  -8.0744E-04  -7.1459E-10  -4.2804E-02  -2.3170E+06   1.7203E+04
Pile N.      46            1           46            1            1            1           35
MAXIMUM   1.2317E+05  -2.8003E+04   7.4621E-04  -7.1459E-10   4.6342E-02  -2.0420E+06   2.2443E+04
Pile N.       1            9            1            1           46           16            1

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

     PILE         DEFLECTION           BENDING MOMENT          SHEAR FORCE          SOIL REACTION
TOTAL      FLEXURAL RIGIDITY
           y-DIR      z-DIR        z-DIR      y-DIR        y-DIR      z-DIR        y-DIR      z-DIR
STRESS       z-DIR      y-DIR
             IN         IN        LBS-IN     LBS-IN         LBS        LBS        LBS/IN     LBS/IN
LBS/IN**2  LBS-IN**2  LBS-IN**2
     *****  **********  **********  **********  **********  **********  **********  **********  **********
**********  **********  **********
        1     -0.5490 -1.1000E-10 -7.3800E+05 -4.2800E-02 -3.3600E+04 -1.4900E-04     -248.00 -7.4900E-06
4720.0  2.6200E+10  9.4500E+09
     x(IN)     0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00     0.0000      0.0000
        2     -0.5490 -1.1500E-10 -7.3800E+05 -4.2800E-02 -3.3600E+04 -1.4800E-04     -247.00 -7.0700E-06
4720.0  2.6200E+10  9.4500E+09
     x(IN)     0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00     0.0000      0.0000
        3     -0.5490 -1.1500E-10 -7.3800E+05 -4.2800E-02 -3.3600E+04 -1.4800E-04     -247.00 -7.0700E-06
4720.0  2.6200E+10  9.4500E+09
     x(IN)     0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00     0.0000      0.0000
        4     -0.5490 -1.1500E-10 -7.3800E+05 -4.2800E-02 -3.3600E+04 -1.4800E-04     -247.00 -7.0700E-06
4720.0  2.6200E+10  9.4500E+09
     x(IN)     0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00     0.0000      0.0000
        5     -0.5490 -1.1500E-10 -7.3800E+05 -4.2800E-02 -3.3600E+04 -1.4800E-04     -247.00 -7.0700E-06
4720.0  2.6200E+10  9.4500E+09
     x(IN)     0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00     0.0000      0.0000
```

Fvy

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        6      -0.5490 -1.1500E-10 -7.3800E+05 -4.2800E-02 -3.3600E+04 -1.4800E-04      -247.00 -7.0700E-06
4720.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00     0.0000     0.0000
        7      -0.5490 -1.1000E-10 -7.3800E+05 -4.2800E-02 -3.3600E+04 -1.4900E-04      -248.00 -7.4900E-06
4720.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00     0.0000     0.0000
        8      -0.5490 -8.3100E-11 -6.6400E+05 -2.8400E-02 -2.8900E+04 -9.1700E-05      -205.00 -4.1700E-06
4080.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      243.00      180.00      144.00      0.0000      216.00      108.00      252.00
468.00     0.0000     0.0000
        9      -0.5490 -5.9400E-11 -6.5200E+05 -2.7800E-02 -2.8000E+04 -9.1700E-05      -197.00 -5.9800E-06
4080.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00     0.0000     0.0000
       10      -0.5490 -5.9400E-11 -6.5200E+05 -2.7800E-02 -2.8000E+04 -9.1700E-05      -197.00 -5.9800E-06
4080.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00     0.0000     0.0000
       11      -0.5490 -5.9400E-11 -6.5200E+05 -2.7800E-02 -2.8000E+04 -9.1700E-05      -197.00 -5.9800E-06
4080.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00     0.0000     0.0000
       12      -0.5490 -5.9400E-11 -6.5200E+05 -2.7800E-02 -2.8000E+04 -9.1700E-05      -197.00 -5.9800E-06
4080.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00     0.0000     0.0000
       13      -0.5490 -5.9400E-11 -6.5200E+05 -2.7800E-02 -2.8000E+04 -9.1700E-05      -197.00 -5.9800E-06
4080.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00     0.0000     0.0000
       14      -0.5490 -8.3100E-11 -6.6400E+05 -2.8400E-02 -2.8900E+04 -9.1700E-05      -205.00 -4.1700E-06
4080.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      243.00      180.00      144.00      0.0000      216.00      108.00      252.00
468.00     0.0000     0.0000
       15      -0.5490 -5.0700E-11 -6.6400E+05 -1.8000E-02 -2.8900E+04 -5.8100E-05      -205.00 -2.7800E-06
3450.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      243.00      180.00      144.00      0.0000      216.00      108.00      252.00
477.00     0.0000     0.0000
       16      -0.5490 -4.0500E-11 -6.5100E+05 -1.7700E-02 -2.8100E+04 -5.7700E-05      -197.00 -3.5200E-06
3450.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
468.00     0.0000     0.0000
       17      -0.5490 -4.0500E-11 -6.5100E+05 -1.7700E-02 -2.8100E+04 -5.7700E-05      -197.00 -3.5200E-06
3450.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00     0.0000     0.0000
       18      -0.5490 -4.0500E-11 -6.5100E+05 -1.7700E-02 -2.8100E+04 -5.7700E-05      -197.00 -3.5200E-06
3450.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
468.00     0.0000     0.0000
       19      -0.5490 -4.0500E-11 -6.5100E+05 -1.7700E-02 -2.8100E+04 -5.7700E-05      -197.00 -3.5200E-06
3450.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
468.00     0.0000     0.0000
       20      -0.5490 -4.0500E-11 -6.5100E+05 -1.7700E-02 -2.8100E+04 -5.7700E-05      -197.00 -3.5200E-06
3450.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00     0.0000     0.0000
       21      -0.5490 -5.0700E-11 -6.6400E+05 -1.8000E-02 -2.8900E+04 -5.8100E-05      -205.00 -2.7800E-06
3450.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      243.00      180.00      144.00      0.0000      216.00      108.00      252.00
477.00     0.0000     0.0000
       22      -0.5490 -1.5100E-11 -6.9300E+05 -5.8600E-03 -3.0800E+04 -1.9500E-05      -221.00 -1.0000E-06
2680.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
459.00     0.0000     0.0000
       23      -0.5490 -1.7300E-11 -6.8900E+05 -5.8300E-03 -3.0600E+04 -1.9000E-05      -219.00 -7.8200E-07
2680.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
459.00     0.0000     0.0000
       24      -0.5490 -1.7900E-11 -6.8700E+05 -5.8200E-03 -3.0500E+04 -1.8900E-05      -218.00 -7.1000E-07
2680.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
459.00     0.0000     0.0000
       25      -0.5490 -1.7900E-11 -6.8700E+05 -5.8200E-03 -3.0500E+04 -1.8900E-05      -218.00 -7.1000E-07
2680.0  2.6200E+10  9.4500E+09
    X(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
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468.00      0.0000      0.0000
       26      -0.5490 -1.7300E-11 -6.8900E+05 -5.8300E-03 -3.0600E+04 -1.9000E-05      -219.00 -7.8200E-07
2680.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
459.00      0.0000      0.0000
       27      -0.5490 -1.5100E-11 -6.9300E+05 -5.8600E-03 -3.0800E+04 -1.9500E-05      -221.00 -1.0000E-06
2680.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
459.00      0.0000      0.0000
       28      -0.5490 -3.1000E-09 -6.9900E+05 -2.0300E-03 -3.1300E+04 -1.2300E-04      -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
       29      -0.5490 -3.1000E-09 -6.9700E+05 -2.0200E-03 -3.1200E+04 -1.2300E-04      -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
       30      -0.5490 -3.1000E-09 -6.9700E+05 -2.0200E-03 -3.1200E+04 -1.2300E-04      -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
       31      -0.5490 -3.1000E-09 -6.9700E+05 -2.0200E-03 -3.1200E+04 -1.2300E-04      -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
       32      -0.5490 -3.1000E-09 -6.9700E+05 -2.0200E-03 -3.1200E+04 -1.2300E-04      -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
       33      -0.5490 -3.1000E-09 -6.9900E+05 -2.0300E-03 -3.1300E+04 -1.2300E-04      -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
       34      -0.5490 -8.7700E-09 -6.9700E+05 -5.7100E-03 -3.1400E+04 -3.4900E-04      -226.00 -3.0600E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
       35      -0.5490 -8.7700E-09 -6.9600E+05 -5.6900E-03 -3.1300E+04 -3.4800E-04      -225.00 -3.0500E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
       36      -0.5490 -8.7700E-09 -6.9600E+05 -5.6900E-03 -3.1300E+04 -3.4800E-04      -225.00 -3.0500E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
       37      -0.5490 -8.7700E-09 -6.9600E+05 -5.6900E-03 -3.1300E+04 -3.4800E-04      -225.00 -3.0500E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
       38      -0.5490 -8.7700E-09 -6.9600E+05 -5.6900E-03 -3.1300E+04 -3.4800E-04      -225.00 -3.0500E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
       39      -0.5490 -8.7700E-09 -6.9700E+05 -5.7100E-03 -3.1400E+04 -3.4900E-04      -226.00 -3.0600E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
       40      -0.5490 -1.4400E-08 -6.9600E+05 -9.3500E-03 -3.1500E+04 -5.7700E-04      -226.00 -5.0300E-06
683.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
       41      -0.5490 -1.4400E-08 -6.9500E+05 -9.3400E-03 -3.1400E+04 -5.7600E-04      -225.00 -5.0200E-06
683.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
       42      -0.5490 -1.4400E-08 -6.9500E+05 -9.3400E-03 -3.1400E+04 -5.7600E-04      -225.00 -5.0200E-06
683.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
       43      -0.5490 -1.4400E-08 -6.9500E+05 -9.3400E-03 -3.1400E+04 -5.7600E-04      -225.00 -5.0200E-06
683.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
       44      -0.5490 -1.4400E-08 -6.9500E+05 -9.3400E-03 -3.1400E+04 -5.7600E-04      -225.00 -5.0200E-06
683.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
       45      -0.5490 -1.4400E-08 -6.9600E+05 -9.3500E-03 -3.1500E+04 -5.7700E-04      -226.00 -5.0300E-06
683.00  2.6200E+10  9.4500E+09
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     x(IN)     0.0000     0.0000     171.00     144.00     9.0000     9.0000     108.00     63.000
468.00    0.0000     0.0000
       46     -0.5490 -2.0100E-08 -6.9500E+05 -1.3000E-02 -3.1500E+04 -8.0700E-04     -226.00 -7.0100E-06
1810.0  2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     171.00     144.00     9.0000     9.0000     108.00     63.000
459.00    0.0000     0.0000
       47     -0.5490 -2.0100E-08 -6.9300E+05 -1.3000E-02 -3.1500E+04 -8.0500E-04     -225.00 -6.9900E-06
1810.0  2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     171.00     144.00     9.0000     9.0000     108.00     63.000
459.00    0.0000     0.0000
       48     -0.5490 -2.0100E-08 -6.9300E+05 -1.3000E-02 -3.1500E+04 -8.0500E-04     -225.00 -6.9900E-06
1810.0  2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     171.00     144.00     9.0000     9.0000     108.00     63.000
459.00    0.0000     0.0000
       49     -0.5490 -2.0100E-08 -6.9300E+05 -1.3000E-02 -3.1500E+04 -8.0500E-04     -225.00 -6.9900E-06
1810.0  2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     171.00     144.00     9.0000     9.0000     108.00     63.000
459.00    0.0000     0.0000
       50     -0.5490 -2.0100E-08 -6.9300E+05 -1.3000E-02 -3.1500E+04 -8.0500E-04     -225.00 -6.9900E-06
1810.0  2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     171.00     144.00     9.0000     9.0000     108.00     63.000
459.00    0.0000     0.0000
       51     -0.5490 -2.0100E-08 -6.9500E+05 -1.3000E-02 -3.1500E+04 -8.0700E-04     -226.00 -7.0100E-06
1810.0  2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     171.00     144.00     9.0000     9.0000     108.00     63.000
459.00    0.0000     0.0000

      Min.    -0.5490 -2.0100E-08 -7.3800E+05 -4.2800E-02 -3.3600E+04 -8.0700E-04     -248.00 -7.4900E-06
423.00  2.6200E+10  9.4500E+09
   Pile N.        1         46          1          1          1         46          1          1
34            1          1

       * MAXIMUM VALUES AND LOCATIONS *                    193.33 K-ft

     PILE          DEFLECTION           BENDING MOMENT          SHEAR FORCE          SOIL REACTION
TOTAL        FLEXURAL RIGIDITY
               y-DIR      z-DIR      z-DIR      y-DIR      y-DIR      z-DIR      y-DIR      z-DIR
STRESS        z-DIR      y-DIR
                  IN         IN     LBS-IN     LBS-IN        LBS        LBS     LBS/IN     LBS/IN
LBS/IN**2   LBS-IN**2  LBS-IN**2
     *****  **********  ********** ********** ********** ********** ********** ********** **********
********** **********  **********
         1    -0.5490  1.7700E-08 2.3200E+06 1.2300E-02     7800.0 7.4900E-04     196.00 6.8000E-06
2.2400E+04 2.6200E+10  9.4500E+09
     x(IN)     270.00     0.0000     0.0000     0.0000     234.00     0.0000     342.00     63.000
0.0000     0.0000     0.0000
         2    -0.5490  1.7700E-08 2.3100E+06 1.2300E-02     7780.0 7.4700E-04     195.00 6.7800E-06
2.2400E+04 2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     0.0000     0.0000     234.00     0.0000     342.00     63.000
0.0000     0.0000     0.0000
         3    -0.5490  1.7700E-08 2.3100E+06 1.2300E-02     7780.0 7.4700E-04     195.00 6.7800E-06
2.2400E+04 2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     0.0000     0.0000     234.00     0.0000     342.00     63.000
0.0000     0.0000     0.0000                                                           Row 1
         4    -0.5490  1.7700E-08 2.3100E+06 1.2300E-02     7780.0 7.4700E-04     195.00 6.7800E-06
2.2400E+04 2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     0.0000     0.0000     234.00     0.0000     342.00     63.000
0.0000     0.0000     0.0000
         5    -0.5490  1.7700E-08 2.3100E+06 1.2300E-02     7780.0 7.4700E-04     195.00 6.7800E-06
2.2400E+04 2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     0.0000     0.0000     234.00     0.0000     342.00     63.000
0.0000     0.0000     0.0000
         6    -0.5490  1.7700E-08 2.3100E+06 1.2300E-02     7780.0 7.4700E-04     195.00 6.7800E-06
2.2400E+04 2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     0.0000     0.0000     234.00     0.0000     342.00     63.000
0.0000     0.0000     0.0000
         7    -0.5490  1.7700E-08 2.3200E+06 1.2300E-02     7800.0 7.4900E-04     196.00 6.8000E-06
2.2400E+04 2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     0.0000     0.0000     234.00     0.0000     342.00     63.000
0.0000     0.0000     0.0000
         8    -0.5490  1.3000E-08 2.0800E+06 8.1000E-03     6690.0 4.7400E-04     176.00 4.0900E-06
2.0000E+04 2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
0.0000     0.0000     0.0000                                                           Row 2
         9    -0.5490  1.3000E-08 2.0400E+06 7.9500E-03     6520.0 4.6000E-04     174.00 3.9300E-06
1.9700E+04 2.6200E+10  9.4500E+09
     x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
0.0000     0.0000     0.0000
        10    -0.5490  1.3000E-08 2.0400E+06 7.9500E-03     6520.0 4.6000E-04     174.00 3.9300E-06
```

```
 1.9700E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         11     -0.5490  1.3000E-08  2.0400E+06  7.9500E-03      6520.0  4.6000E-04      174.00  3.9300E-06
 1.9700E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         12     -0.5490  1.3000E-08  2.0400E+06  7.9500E-03      6520.0  4.6000E-04      174.00  3.9300E-06
 1.9700E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         13     -0.5490  1.3000E-08  2.0400E+06  7.9500E-03      6520.0  4.6000E-04      174.00  3.9300E-06
 1.9700E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         14     -0.5490  1.3000E-08  2.0800E+06  8.1000E-03      6690.0  4.7400E-04      176.00  4.0900E-06
 2.0000E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
```

Row 3

```
         15     -0.5490  8.2500E-09  2.0800E+06  5.1400E-03      6670.0  3.0200E-04      176.00  2.6000E-06
 1.9400E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         16     -0.5490  8.2500E-09  2.0400E+06  5.0400E-03      6500.0  2.9300E-04      173.00  2.5000E-06
 1.9100E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         17     -0.5490  8.2500E-09  2.0400E+06  5.0400E-03      6500.0  2.9300E-04      173.00  2.5000E-06
 1.9100E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         18     -0.5490  8.2500E-09  2.0400E+06  5.0400E-03      6500.0  2.9300E-04      173.00  2.5000E-06
 1.9100E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         19     -0.5490  8.2500E-09  2.0400E+06  5.0400E-03      6500.0  2.9300E-04      173.00  2.5000E-06
 1.9100E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         20     -0.5490  8.2500E-09  2.0400E+06  5.0400E-03      6500.0  2.9300E-04      173.00  2.5000E-06
 1.9100E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         21     -0.5490  8.2500E-09  2.0800E+06  5.1400E-03      6670.0  3.0200E-04      176.00  2.6000E-06
 1.9400E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
```

Row 4

```
         22     -0.5490  2.5700E-09  2.1800E+06  1.6700E-03      7080.0  1.0000E-04      186.00  8.7900E-07
 1.9300E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      243.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         23     -0.5490  2.5700E-09  2.1600E+06  1.6600E-03      7030.0  9.9500E-05      184.00  8.7000E-07
 1.9200E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      243.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         24     -0.5490  2.5700E-09  2.1600E+06  1.6500E-03      7000.0  9.9100E-05      184.00  8.6600E-07
 1.9200E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      243.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         25     -0.5490  2.5700E-09  2.1600E+06  1.6500E-03      7000.0  9.9100E-05      184.00  8.6600E-07
 1.9200E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      243.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         26     -0.5490  2.5700E-09  2.1600E+06  1.6600E-03      7030.0  9.9500E-05      184.00  8.7000E-07
 1.9200E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      243.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
         27     -0.5490  2.5700E-09  2.1800E+06  1.6700E-03      7080.0  1.0000E-04      186.00  8.7900E-07
 1.9300E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      0.0000      0.0000      0.0000      243.00      0.0000      342.00      63.000
 0.0000      0.0000      0.0000
```

Row 5

```
         28     -0.5490  8.6400E-12  2.2000E+06  7.1400E-03      7150.0  2.4800E-05      187.00  2.0700E-06
 1.8400E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      252.00      0.0000      0.0000      243.00      225.00      342.00      243.00
 0.0000      0.0000      0.0000
         29     -0.5490  1.2200E-11  2.1900E+06  7.1300E-03      7130.0  2.4200E-05      187.00  1.7200E-06
 1.8300E+04  2.6200E+10  9.4500E+09
      X(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
 0.0000      0.0000      0.0000
```

```
          30      -0.5490   1.2200E-11   2.1900E+06   7.1300E-03     7130.0   2.4200E-05      187.00   1.7200E-06
  1.8300E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          31      -0.5490   1.2200E-11   2.1900E+06   7.1300E-03     7130.0   2.4200E-05      187.00   1.7200E-06
  1.8300E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          32      -0.5490   1.2200E-11   2.1900E+06   7.1300E-03     7130.0   2.4200E-05      187.00   1.7200E-06
  1.8300E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          33      -0.5490   8.6400E-12   2.2000E+06   7.1400E-03     7150.0   2.4800E-05      187.00   2.0700E-06
  1.8400E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       252.00       0.0000       0.0000     243.00       225.00      342.00       243.00
  0.0000       0.0000       0.0000
```

Row 6

```
          34      -0.5490   3.6500E-11   2.2000E+06   2.0200E-02     7130.0   6.7900E-05      187.00   4.6300E-06
  1.7200E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          35      -0.5490   4.2900E-11   2.1900E+06   2.0200E-02     7110.0   6.6900E-05      187.00   4.0400E-06
  1.7200E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          36      -0.5490   4.2900E-11   2.1900E+06   2.0200E-02     7110.0   6.6900E-05      187.00   4.0400E-06
  1.7200E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          37      -0.5490   4.2900E-11   2.1900E+06   2.0200E-02     7110.0   6.6900E-05      187.00   4.0400E-06
  1.7200E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          38      -0.5490   4.2900E-11   2.1900E+06   2.0200E-02     7110.0   6.6900E-05      187.00   4.0400E-06
  1.7200E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          39      -0.5490   3.6500E-11   2.2000E+06   2.0200E-02     7130.0   6.7900E-05      187.00   4.6300E-06
  1.7200E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
```

Row 7

```
          40      -0.5490   7.5000E-11   2.2000E+06   3.3300E-02     7100.0   1.0900E-04      188.00   6.1100E-06
  1.7500E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          41      -0.5490   7.5200E-11   2.1900E+06   3.3200E-02     7080.0   1.0900E-04      187.00   6.0800E-06
  1.7500E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          42      -0.5490   7.5200E-11   2.1900E+06   3.3200E-02     7080.0   1.0900E-04      187.00   6.0800E-06
  1.7500E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          43      -0.5490   7.5200E-11   2.1900E+06   3.3200E-02     7080.0   1.0900E-04      187.00   6.0800E-06
  1.7500E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          44      -0.5490   7.5200E-11   2.1900E+06   3.3200E-02     7080.0   1.0900E-04      187.00   6.0800E-06
  1.7500E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          45      -0.5490   7.5000E-11   2.2000E+06   3.3300E-02     7100.0   1.0900E-04      188.00   6.1100E-06
  1.7500E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
```

Row 8

```
          46      -0.5490   1.0700E-10   2.2000E+06   4.6300E-02     7070.0   1.5000E-04      187.00   8.0900E-06
  1.8600E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          47      -0.5490   1.1300E-10   2.1900E+06   4.6300E-02     7050.0   1.4900E-04      186.00   7.4700E-06
  1.8600E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          48      -0.5490   1.1300E-10   2.1900E+06   4.6300E-02     7050.0   1.4900E-04      186.00   7.4700E-06
  1.8600E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
  0.0000       0.0000       0.0000
          49      -0.5490   1.1300E-10   2.1900E+06   4.6300E-02     7050.0   1.4900E-04      186.00   7.4700E-06
  1.8600E+04   2.6200E+10   9.4500E+09
       x(IN)       0.0000       243.00       0.0000       0.0000     243.00       216.00      342.00       243.00
```

```
0.0000     0.0000     0.0000
       50    -0.5490  1.1300E-10  2.1900E+06  4.6300E-02      7050.0  1.4900E-04      186.00  7.4700E-06
1.8600E+04  2.6200E+10  9.4500E+09
    x(IN)     0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000     0.0000     0.0000
       51    -0.5490  1.0700E-10  2.2000E+06  4.6300E-02      7070.0  1.5000E-04      187.00  8.0900E-06
1.8600E+04  2.6200E+10  9.4500E+09
    x(IN)     0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000     0.0000     0.0000

     Max.    -0.5490  1.7700E-08  2.3200E+06  4.6300E-02      7800.0  7.4900E-04      196.00  8.0900E-06
2.2400E+04  2.6200E+10  9.4500E+09
   Pile N.         1           1           1          46           1           1           1          46
1          1          1


     LOAD CASE :      2
     CASE NAME : Load Case 1.1 w/ice
     LOAD TYPE : Dead, DL

       REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS, COMBINED Y AND Z DIRECTIONS
       ESTIMATED USING MOVEMENT IN THE DIRECTION OF PILE CAP DISPLACEMENTS

            GROUP NO      P-FACTOR      Y-FACTOR

                1           0.9974        1.0000
                2           0.9948        1.0000
                3           0.9948        1.0000
                4           0.9948        1.0000
                5           0.9948        1.0000
                6           0.9948        1.0000
                7           0.9974        1.0000
                8           0.8025        1.0000
                9           0.7682        1.0000
               10           0.7682        1.0000
               11           0.7682        1.0000
               12           0.7682        1.0000
               13           0.7682        1.0000
               14           0.8025        1.0000
               15           0.8025        1.0000
               16           0.7682        1.0000
               17           0.7682        1.0000
               18           0.7682        1.0000
               19           0.7682        1.0000
               20           0.7682        1.0000
               21           0.8025        1.0000
               22           0.8764        1.0000
               23           0.8666        1.0000
               24           0.8622        1.0000
               25           0.8622        1.0000
               26           0.8666        1.0000
               27           0.8764        1.0000
               28           0.8949        1.0000
               29           0.8913        1.0000
               30           0.8913        1.0000
               31           0.8913        1.0000
               32           0.8913        1.0000
               33           0.8949        1.0000
               34           0.8949        1.0000
               35           0.8913        1.0000
               36           0.8913        1.0000
               37           0.8913        1.0000
               38           0.8913        1.0000
               39           0.8949        1.0000
               40           0.8949        1.0000
               41           0.8913        1.0000
               42           0.8913        1.0000
               43           0.8913        1.0000
               44           0.8913        1.0000
               45           0.8949        1.0000
               46           0.8949        1.0000
               47           0.8913        1.0000
               48           0.8913        1.0000
               49           0.8913        1.0000
               50           0.8913        1.0000
               51           0.8949        1.0000
```

```
* TABLE L *   COMPUTATION ON PILE CAP


* EQUIVALENT CONC. LOAD AT ORIGIN *


      VERT. LOAD, LBS     HOR. LOAD Y, LBS     HOR. LOAD Z, LBS
        2.57500E+06        -1.58200E+06              0.00000

     MOMENT X ,IN-LBS     MOMENT Y,IN-LBS      MOMENT Z,IN-LBS
        2.84760E+08         4.63500E+08         -3.82542E+08


* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *


       VERTICAL ,IN        HORIZONTAL Y,IN       HORIZONTAL Z,IN
            0.10208              -0.54842         5.38292E-13

     ANGLE ROT. X,RAD     ANGLE ROT. Y,RAD      ANGLE ROT. Z,RAD
       -2.11583E-15         1.00455E-18          2.37446E-04


  THE GLOBAL STRUCTURAL COORDINATE SYSTEM
  ---------------------------------------

  * PILE TOP DISPLACEMENTS *

 PILE GROUP    DISP. X,IN    DISP. Y,IN    DISP. Z,IN    ROT. X,RAD    ROT. Y,RAD    ROT. Z,RAD
 **********   ***********   ***********   ***********   ***********   ***********   ***********
      1        9.4952E-02       -0.5513    4.7483E-13   -2.1158E-15    1.0045E-18    2.3745E-04
      2        9.4952E-02       -0.5513    4.7483E-13   -2.1158E-15    1.0045E-18    2.3745E-04
      3        9.4952E-02       -0.5513    4.7483E-13   -2.1158E-15    1.0045E-18    2.3745E-04
      4        9.4952E-02       -0.5513    4.7483E-13   -2.1158E-15    1.0045E-18    2.3745E-04
      5        9.4952E-02       -0.5513    4.7483E-13   -2.1158E-15    1.0045E-18    2.3745E-04
      6        9.4952E-02       -0.5513    4.7483E-13   -2.1158E-15    1.0045E-18    2.3745E-04
      7        9.4952E-02       -0.5513    4.7483E-13   -2.1158E-15    1.0045E-18    2.3745E-04
      8        8.0705E-02       -0.5513    3.4788E-13   -2.1158E-15    1.0045E-18    2.3745E-04
      9        8.0705E-02       -0.5513    3.4788E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     10        8.0705E-02       -0.5513    3.4788E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     11        8.0705E-02       -0.5513    3.4788E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     12        8.0705E-02       -0.5513    3.4788E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     13        8.0705E-02       -0.5513    3.4788E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     14        8.0705E-02       -0.5513    3.4788E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     15        6.6458E-02       -0.5513    2.2093E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     16        6.6458E-02       -0.5513    2.2093E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     17        6.6458E-02       -0.5513    2.2093E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     18        6.6458E-02       -0.5513    2.2093E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     19        6.6458E-02       -0.5513    2.2093E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     20        6.6458E-02       -0.5513    2.2093E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     21        6.6458E-02       -0.5513    2.2093E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     22        4.9362E-02       -0.5513    6.8590E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     23        4.9362E-02       -0.5513    6.8590E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     24        4.9362E-02       -0.5513    6.8590E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     25        4.9362E-02       -0.5513    6.8590E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     26        4.9362E-02       -0.5513    6.8590E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     27        4.9362E-02       -0.5513    6.8590E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     28        3.2266E-02       -0.5513   -8.3750E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     29        3.2266E-02       -0.5513   -8.3750E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     30        3.2266E-02       -0.5513   -8.3750E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     31        3.2266E-02       -0.5513   -8.3750E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     32        3.2266E-02       -0.5513   -8.3750E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     33        3.2266E-02       -0.5513   -8.3750E-14   -2.1158E-15    1.0045E-18    2.3745E-04
     34        1.5170E-02       -0.5513   -2.3609E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     35        1.5170E-02       -0.5513   -2.3609E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     36        1.5170E-02       -0.5513   -2.3609E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     37        1.5170E-02       -0.5513   -2.3609E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     38        1.5170E-02       -0.5513   -2.3609E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     39        1.5170E-02       -0.5513   -2.3609E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     40       -1.9264E-03       -0.5513   -3.8843E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     41       -1.9264E-03       -0.5513   -3.8843E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     42       -1.9264E-03       -0.5513   -3.8843E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     43       -1.9264E-03       -0.5513   -3.8843E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     44       -1.9264E-03       -0.5513   -3.8843E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     45       -1.9264E-03       -0.5513   -3.8843E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     46       -1.9023E-02       -0.5513   -5.4077E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     47       -1.9023E-02       -0.5513   -5.4077E-13   -2.1158E-15    1.0045E-18    2.3745E-04
     48       -1.9023E-02       -0.5513   -5.4077E-13   -2.1158E-15    1.0045E-18    2.3745E-04
```

```
         49     -1.9023E-02      -0.5513  -5.4077E-13 -2.1158E-15   1.0045E-18   2.3745E-04
         50     -1.9023E-02      -0.5513  -5.4077E-13 -2.1158E-15   1.0045E-18   2.3745E-04
         51     -1.9023E-02      -0.5513  -5.4077E-13 -2.1158E-15   1.0045E-18   2.3745E-04

      MINIMUM   -1.9023E-02      -0.5513  -5.4077E-13 -2.1158E-15   1.0045E-18   2.3745E-04
      Pile N.        46             1          46          1            1            1
      MAXIMUM    9.4952E-02      -0.5513   4.7483E-13 -2.1158E-15   1.0045E-18   2.3745E-04
      Pile N.         1             1           1          1            1            1

      * PILE TOP REACTIONS * AXIAL  HORIZ

     PILE GROUP   FOR. X,LBS   FOR. Y,LBS   FOR. Z,LBS MOM X,LBS-IN MOM Y,LBS-IN MOM Z,LBS-IN  STRESS,LBS/IN**2
     **********  ***********  ***********  *********** ************ ************ ************  ****************
          1      1.0700E+05  -3.3922E+04   1.9960E-08  -1.9187E-14  -1.1441E-06  -2.3568E+06    2.2127E+04
 TONS     2      1.0700E+05  -3.3859E+04   1.9924E-08  -1.9187E-14  -1.1427E-06  -2.3537E+06    2.2104E+04
          3      1.0700E+05  -3.3859E+04   1.9924E-08  -1.9187E-14  -1.1427E-06  -2.3537E+06    2.2104E+04
 53.5     4      1.0700E+05  -3.3859E+04   1.9924E-08  -1.9187E-14  -1.1427E-06  -2.3537E+06    2.2104E+04
          5      1.0700E+05  -3.3859E+04   1.9924E-08  -1.9187E-14  -1.1427E-06  -2.3537E+06    2.2104E+04
          6      1.0700E+05  -3.3859E+04   1.9924E-08  -1.9187E-14  -1.1427E-06  -2.3537E+06    2.2104E+04
          7      1.0700E+05  -3.3922E+04   1.9960E-08  -1.9187E-14  -1.1441E-06  -2.3568E+06    2.2127E+04
          8      9.4323E+04  -2.9177E+04   1.2638E-08  -1.9187E-14  -7.5837E-07  -2.1224E+06    1.9849E+04
          9      9.4323E+04  -2.8306E+04   1.2261E-08  -1.9187E-14  -7.4314E-07  -2.0792E+06    1.9519E+04
         10      9.4323E+04  -2.8306E+04   1.2261E-08  -1.9187E-14  -7.4314E-07  -2.0792E+06    1.9519E+04
 47.16   11      9.4323E+04  -2.8306E+04   1.2261E-08  -1.9187E-14  -7.4314E-07  -2.0792E+06    1.9519E+04
         12      9.4323E+04  -2.8306E+04   1.2261E-08  -1.9187E-14  -7.4314E-07  -2.0792E+06    1.9519E+04
         13      9.4323E+04  -2.8306E+04   1.2261E-08  -1.9187E-14  -7.4314E-07  -2.0792E+06    1.9519E+04
         14      9.4323E+04  -2.9177E+04   1.2638E-08  -1.9187E-14  -7.5837E-07  -2.1224E+06    1.9849E+04
         15      8.1641E+04  -2.9206E+04   8.0420E-09  -1.9187E-14  -4.8160E-07  -2.1223E+06    1.9362E+04
         16      8.1641E+04  -2.8338E+04   7.8013E-09  -1.9187E-14  -4.7192E-07  -2.0793E+06    1.9033E+04
         17      8.1641E+04  -2.8338E+04   7.8013E-09  -1.9187E-14  -4.7192E-07  -2.0793E+06    1.9033E+04
 40.82   18      8.1641E+04  -2.8338E+04   7.8013E-09  -1.9187E-14  -4.7192E-07  -2.0793E+06    1.9033E+04
         19      8.1641E+04  -2.8338E+04   7.8013E-09  -1.9187E-14  -4.7192E-07  -2.0793E+06    1.9033E+04
         20      8.1641E+04  -2.8338E+04   7.8013E-09  -1.9187E-14  -4.7192E-07  -2.0793E+06    1.9033E+04
         21      8.1641E+04  -2.9206E+04   8.0420E-09  -1.9187E-14  -4.8160E-07  -2.1223E+06    1.9362E+04
         22      6.4547E+04  -3.1107E+04   2.6548E-09  -1.9187E-14  -1.5556E-07  -2.2151E+06    1.9417E+04
         23      6.4547E+04  -3.0864E+04   2.6343E-09  -1.9187E-14  -1.5476E-07  -2.2031E+06    1.9325E+04
 32.27   24      6.4547E+04  -3.0755E+04   2.6252E-09  -1.9187E-14  -1.5439E-07  -2.1977E+06    1.9284E+04
         25      6.4547E+04  -3.0755E+04   2.6252E-09  -1.9187E-14  -1.5439E-07  -2.1977E+06    1.9284E+04
         26      6.4547E+04  -3.0864E+04   2.6343E-09  -1.9187E-14  -1.5476E-07  -2.2031E+06    1.9325E+04
         27      6.4547E+04  -3.1107E+04   2.6548E-09  -1.9187E-14  -1.5556E-07  -2.2151E+06    1.9417E+04
         28      4.1944E+04  -3.1619E+04  -3.3048E-09  -1.9187E-14   1.9232E-07  -2.2373E+06    1.8721E+04
         29      4.1944E+04  -3.1529E+04  -3.2953E-09  -1.9187E-14   1.9195E-07  -2.2329E+06    1.8687E+04
 20.97   30      4.1944E+04  -3.1529E+04  -3.2953E-09  -1.9187E-14   1.9195E-07  -2.2329E+06    1.8687E+04
         31      4.1944E+04  -3.1529E+04  -3.2953E-09  -1.9187E-14   1.9195E-07  -2.2329E+06    1.8687E+04
         32      4.1944E+04  -3.1529E+04  -3.2953E-09  -1.9187E-14   1.9195E-07  -2.2329E+06    1.8687E+04
         33      4.1944E+04  -3.1619E+04  -3.3048E-09  -1.9187E-14   1.9232E-07  -2.2373E+06    1.8721E+04
         34      1.9593E+04  -3.1674E+04  -9.3414E-09  -1.9187E-14   5.4180E-07  -2.2372E+06    1.7864E+04
         35      1.9593E+04  -3.1586E+04  -9.3155E-09  -1.9187E-14   5.4079E-07  -2.2329E+06    1.7831E+04
         36      1.9593E+04  -3.1586E+04  -9.3155E-09  -1.9187E-14   5.4079E-07  -2.2329E+06    1.7831E+04
 9.796   37      1.9593E+04  -3.1586E+04  -9.3155E-09  -1.9187E-14   5.4079E-07  -2.2329E+06    1.7831E+04
         38      1.9593E+04  -3.1586E+04  -9.3155E-09  -1.9187E-14   5.4079E-07  -2.2329E+06    1.7831E+04
         39      1.9593E+04  -3.1674E+04  -9.3414E-09  -1.9187E-14   5.4180E-07  -2.2372E+06    1.7864E+04
         40         -2459.5  -3.1730E+04  -1.5416E-08  -1.9187E-14   8.9139E-07  -2.2371E+06    1.7207E+04
         41         -2459.5  -3.1642E+04  -1.5373E-08  -1.9187E-14   8.8973E-07  -2.2328E+06    1.7174E+04
         42         -2459.5  -3.1642E+04  -1.5373E-08  -1.9187E-14   8.8973E-07  -2.2328E+06    1.7174E+04
 -1.23   43         -2459.5  -3.1642E+04  -1.5373E-08  -1.9187E-14   8.8973E-07  -2.2328E+06    1.7174E+04
         44         -2459.5  -3.1642E+04  -1.5373E-08  -1.9187E-14   8.8973E-07  -2.2328E+06    1.7174E+04
         45         -2459.5  -3.1730E+04  -1.5416E-08  -1.9187E-14   8.9139E-07  -2.2371E+06    1.7207E+04
         46     -2.4587E+04  -3.1783E+04  -2.1527E-08  -1.9187E-14   1.2410E-06  -2.2368E+06    1.8052E+04
         47     -2.4587E+04  -3.1694E+04  -2.1468E-08  -1.9187E-14   1.2387E-06  -2.2324E+06    1.8019E+04
         48     -2.4587E+04  -3.1694E+04  -2.1468E-08  -1.9187E-14   1.2387E-06  -2.2324E+06    1.8019E+04
 -12.29  49     -2.4587E+04  -3.1694E+04  -2.1468E-08  -1.9187E-14   1.2387E-06  -2.2324E+06    1.8019E+04
         50     -2.4587E+04  -3.1694E+04  -2.1468E-08  -1.9187E-14   1.2387E-06  -2.2324E+06    1.8019E+04
         51     -2.4587E+04  -3.1783E+04  -2.1527E-08  -1.9187E-14   1.2410E-06  -2.2368E+06    1.8052E+04

      MINIMUM   -2.4587E+04  -3.3922E+04  -2.1527E-08  -1.9187E-14  -1.1441E-06  -2.3568E+06    1.7174E+04
      Pile N.        46            1          46            1            1            1            41
      MAXIMUM    1.0700E+05  -2.8306E+04   1.9960E-08  -1.9187E-14   1.2410E-06  -2.0792E+06    2.2127E+04
      Pile N.         1            9           1            1           46            9             1

      THE PILE COORDINATE SYSTEM (LOCAL AXES)
      ---------------------------------------

      * PILE TOP DISPLACEMENTS *

     PILE GROUP   DISP. x,IN   DISP. y,IN   DISP. z,IN   ROT. x,RAD   ROT. y,RAD   ROT. z,RAD
     **********  ***********  ***********  ***********  ***********  ***********  ***********
          1      9.4952E-02      -0.5513   4.7483E-13  -2.1158E-15   1.0045E-18   2.3745E-04
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    2        9.4952E-02        -0.5513   4.7483E-13  -2.1158E-15    1.0045E-18    2.3745E-04
    3        9.4952E-02        -0.5513   4.7483E-13  -2.1158E-15    1.0045E-18    2.3745E-04
    4        9.4952E-02        -0.5513   4.7483E-13  -2.1158E-15    1.0045E-18    2.3745E-04
    5        9.4952E-02        -0.5513   4.7483E-13  -2.1158E-15    1.0045E-18    2.3745E-04
    6        9.4952E-02        -0.5513   4.7483E-13  -2.1158E-15    1.0045E-18    2.3745E-04
    7        9.4952E-02        -0.5513   4.7483E-13  -2.1158E-15    1.0045E-18    2.3745E-04
    8        8.0705E-02        -0.5513   3.4788E-13  -2.1158E-15    1.0045E-18    2.3745E-04
    9        8.0705E-02        -0.5513   3.4788E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   10        8.0705E-02        -0.5513   3.4788E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   11        8.0705E-02        -0.5513   3.4788E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   12        8.0705E-02        -0.5513   3.4788E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   13        8.0705E-02        -0.5513   3.4788E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   14        8.0705E-02        -0.5513   3.4788E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   15        6.6458E-02        -0.5513   2.2093E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   16        6.6458E-02        -0.5513   2.2093E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   17        6.6458E-02        -0.5513   2.2093E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   18        6.6458E-02        -0.5513   2.2093E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   19        6.6458E-02        -0.5513   2.2093E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   20        6.6458E-02        -0.5513   2.2093E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   21        6.6458E-02        -0.5513   2.2093E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   22        4.9362E-02        -0.5513   6.8590E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   23        4.9362E-02        -0.5513   6.8590E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   24        4.9362E-02        -0.5513   6.8590E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   25        4.9362E-02        -0.5513   6.8590E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   26        4.9362E-02        -0.5513   6.8590E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   27        4.9362E-02        -0.5513   6.8590E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   28        3.2266E-02        -0.5513  -8.3750E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   29        3.2266E-02        -0.5513  -8.3750E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   30        3.2266E-02        -0.5513  -8.3750E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   31        3.2266E-02        -0.5513  -8.3750E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   32        3.2266E-02        -0.5513  -8.3750E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   33        3.2266E-02        -0.5513  -8.3750E-14  -2.1158E-15    1.0045E-18    2.3745E-04
   34        1.5170E-02        -0.5513  -2.3609E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   35        1.5170E-02        -0.5513  -2.3609E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   36        1.5170E-02        -0.5513  -2.3609E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   37        1.5170E-02        -0.5513  -2.3609E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   38        1.5170E-02        -0.5513  -2.3609E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   39        1.5170E-02        -0.5513  -2.3609E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   40       -1.9264E-03        -0.5513  -3.8843E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   41       -1.9264E-03        -0.5513  -3.8843E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   42       -1.9264E-03        -0.5513  -3.8843E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   43       -1.9264E-03        -0.5513  -3.8843E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   44       -1.9264E-03        -0.5513  -3.8843E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   45       -1.9264E-03        -0.5513  -3.8843E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   46       -1.9023E-02        -0.5513  -5.4077E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   47       -1.9023E-02        -0.5513  -5.4077E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   48       -1.9023E-02        -0.5513  -5.4077E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   49       -1.9023E-02        -0.5513  -5.4077E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   50       -1.9023E-02        -0.5513  -5.4077E-13  -2.1158E-15    1.0045E-18    2.3745E-04
   51       -1.9023E-02        -0.5513  -5.4077E-13  -2.1158E-15    1.0045E-18    2.3745E-04

 MINIMUM    -1.9023E-02        -0.5513  -5.4077E-13  -2.1158E-15    1.0045E-18    2.3745E-04
 Pile N.        46                1         46            1             1             1
 MAXIMUM     9.4952E-02        -0.5513   4.7483E-13  -2.1158E-15    1.0045E-18    2.3745E-04
 Pile N.         1                1          1            1             1             1

 * PILE TOP REACTIONS *

PILE GROUP   AXIAL,LBS    LAT. y,LBS    LAT. z,LBS  MOM x,LBS-IN  MOM y,LBS-IN  MOM z,LBS-IN   STRESS,LBS/IN**2
**********  ***********  ***********  ***********   ***********   ***********   ***********   ****************
    1        1.0700E+05  -3.3922E+04   1.9960E-08  -1.9187E-14   -1.1441E-06   -2.3568E+06     2.2127E+04
    2        1.0700E+05  -3.3859E+04   1.9924E-08  -1.9187E-14   -1.1427E-06   -2.3537E+06     2.2104E+04
    3        1.0700E+05  -3.3859E+04   1.9924E-08  -1.9187E-14   -1.1427E-06   -2.3537E+06     2.2104E+04
    4        1.0700E+05  -3.3859E+04   1.9924E-08  -1.9187E-14   -1.1427E-06   -2.3537E+06     2.2104E+04
    5        1.0700E+05  -3.3859E+04   1.9924E-08  -1.9187E-14   -1.1427E-06   -2.3537E+06     2.2104E+04
    6        1.0700E+05  -3.3859E+04   1.9924E-08  -1.9187E-14   -1.1427E-06   -2.3537E+06     2.2104E+04
    7        1.0700E+05  -3.3922E+04   1.9960E-08  -1.9187E-14   -1.1441E-06   -2.3568E+06     2.2127E+04
    8        9.4323E+04  -2.9177E+04   1.2638E-08  -1.9187E-14   -7.5837E-07   -2.1224E+06     1.9849E+04
    9        9.4323E+04  -2.8306E+04   1.2261E-08  -1.9187E-14   -7.4314E-07   -2.0792E+06     1.9519E+04
   10        9.4323E+04  -2.8306E+04   1.2261E-08  -1.9187E-14   -7.4314E-07   -2.0792E+06     1.9519E+04
   11        9.4323E+04  -2.8306E+04   1.2261E-08  -1.9187E-14   -7.4314E-07   -2.0792E+06     1.9519E+04
   12        9.4323E+04  -2.8306E+04   1.2261E-08  -1.9187E-14   -7.4314E-07   -2.0792E+06     1.9519E+04
   13        9.4323E+04  -2.8306E+04   1.2261E-08  -1.9187E-14   -7.4314E-07   -2.0792E+06     1.9519E+04
   14        9.4323E+04  -2.9177E+04   1.2638E-08  -1.9187E-14   -7.5837E-07   -2.1224E+06     1.9849E+04
   15        8.1641E+04  -2.9206E+04   8.0420E-09  -1.9187E-14   -4.8160E-07   -2.1223E+06     1.9362E+04
   16        8.1641E+04  -2.8338E+04   7.8013E-09  -1.9187E-14   -4.7192E-07   -2.0793E+06     1.9033E+04
   17        8.1641E+04  -2.8338E+04   7.8013E-09  -1.9187E-14   -4.7192E-07   -2.0793E+06     1.9033E+04
   18        8.1641E+04  -2.8338E+04   7.8013E-09  -1.9187E-14   -4.7192E-07   -2.0793E+06     1.9033E+04
   19        8.1641E+04  -2.8338E+04   7.8013E-09  -1.9187E-14   -4.7192E-07   -2.0793E+06     1.9033E+04
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    20         8.1641E+04  -2.8338E+04   7.8013E-09  -1.9187E-14  -4.7192E-07  -2.0793E+06   1.9033E+04
    21         8.1641E+04  -2.9206E+04   8.0420E-09  -1.9187E-14  -4.8160E-07  -2.1223E+06   1.9362E+04
    22         6.4547E+04  -3.1107E+04   2.6548E-09  -1.9187E-14  -1.5556E-07  -2.2151E+06   1.9417E+04
    23         6.4547E+04  -3.0864E+04   2.6343E-09  -1.9187E-14  -1.5476E-07  -2.2031E+06   1.9325E+04
    24         6.4547E+04  -3.0755E+04   2.6252E-09  -1.9187E-14  -1.5439E-07  -2.1977E+06   1.9284E+04
    25         6.4547E+04  -3.0755E+04   2.6252E-09  -1.9187E-14  -1.5439E-07  -2.1977E+06   1.9284E+04
    26         6.4547E+04  -3.0864E+04   2.6343E-09  -1.9187E-14  -1.5476E-07  -2.2031E+06   1.9325E+04
    27         6.4547E+04  -3.1107E+04   2.6548E-09  -1.9187E-14  -1.5556E-07  -2.2151E+06   1.9417E+04
    28         4.1944E+04  -3.1619E+04  -3.3048E-09  -1.9187E-14   1.9232E-07  -2.2373E+06   1.8721E+04
    29         4.1944E+04  -3.1529E+04  -3.2953E-09  -1.9187E-14   1.9195E-07  -2.2329E+06   1.8687E+04
    30         4.1944E+04  -3.1529E+04  -3.2953E-09  -1.9187E-14   1.9195E-07  -2.2329E+06   1.8687E+04
    31         4.1944E+04  -3.1529E+04  -3.2953E-09  -1.9187E-14   1.9195E-07  -2.2329E+06   1.8687E+04
    32         4.1944E+04  -3.1529E+04  -3.2953E-09  -1.9187E-14   1.9195E-07  -2.2329E+06   1.8687E+04
    33         4.1944E+04  -3.1619E+04  -3.3048E-09  -1.9187E-14   1.9232E-07  -2.2373E+06   1.8721E+04
    34         1.9593E+04  -3.1674E+04  -9.3414E-09  -1.9187E-14   5.4180E-07  -2.2372E+06   1.7864E+04
    35         1.9593E+04  -3.1586E+04  -9.3155E-09  -1.9187E-14   5.4079E-07  -2.2329E+06   1.7831E+04
    36         1.9593E+04  -3.1586E+04  -9.3155E-09  -1.9187E-14   5.4079E-07  -2.2329E+06   1.7831E+04
    37         1.9593E+04  -3.1586E+04  -9.3155E-09  -1.9187E-14   5.4079E-07  -2.2329E+06   1.7831E+04
    38         1.9593E+04  -3.1586E+04  -9.3155E-09  -1.9187E-14   5.4079E-07  -2.2329E+06   1.7831E+04
    39         1.9593E+04  -3.1674E+04  -9.3414E-09  -1.9187E-14   5.4180E-07  -2.2372E+06   1.7864E+04
    40            -2459.5  -3.1730E+04  -1.5416E-08  -1.9187E-14   8.9139E-07  -2.2371E+06   1.7207E+04
    41            -2459.5  -3.1642E+04  -1.5373E-08  -1.9187E-14   8.8973E-07  -2.2328E+06   1.7174E+04
    42            -2459.5  -3.1642E+04  -1.5373E-08  -1.9187E-14   8.8973E-07  -2.2328E+06   1.7174E+04
    43            -2459.5  -3.1642E+04  -1.5373E-08  -1.9187E-14   8.8973E-07  -2.2328E+06   1.7174E+04
    44            -2459.5  -3.1642E+04  -1.5373E-08  -1.9187E-14   8.8973E-07  -2.2328E+06   1.7174E+04
    45            -2459.5  -3.1730E+04  -1.5416E-08  -1.9187E-14   8.9139E-07  -2.2371E+06   1.7207E+04
    46        -2.4587E+04  -3.1783E+04  -2.1527E-08  -1.9187E-14   1.2410E-06  -2.2368E+06   1.8052E+04
    47        -2.4587E+04  -3.1694E+04  -2.1468E-08  -1.9187E-14   1.2387E-06  -2.2324E+06   1.8019E+04
    48        -2.4587E+04  -3.1694E+04  -2.1468E-08  -1.9187E-14   1.2387E-06  -2.2324E+06   1.8019E+04
    49        -2.4587E+04  -3.1694E+04  -2.1468E-08  -1.9187E-14   1.2387E-06  -2.2324E+06   1.8019E+04
    50        -2.4587E+04  -3.1694E+04  -2.1468E-08  -1.9187E-14   1.2387E-06  -2.2324E+06   1.8019E+04
    51        -2.4587E+04  -3.1783E+04  -2.1527E-08  -1.9187E-14   1.2410E-06  -2.2368E+06   1.8052E+04

  MINIMUM     -2.4587E+04  -3.3922E+04  -2.1527E-08  -1.9187E-14  -1.1441E-06  -2.3568E+06   1.7174E+04
  Pile N.          46            1           46            1            1            1             41
  MAXIMUM      1.0700E+05  -2.8306E+04   1.9960E-08  -1.9187E-14   1.2410E-06  -2.0792E+06   2.2127E+04
  Pile N.           1            9            1            1           46            9              1

  * EFFECTS FOR LATERALLY LOADED PILE *

  * MINIMUM VALUES AND LOCATIONS *

     PILE         DEFLECTION            BENDING MOMENT          SHEAR FORCE            SOIL REACTION
TOTAL       FLEXURAL RIGIDITY
              y-DIR      z-DIR       z-DIR       y-DIR       y-DIR       z-DIR       y-DIR       z-DIR
STRESS        z-DIR      y-DIR
                IN          IN       LBS-IN      LBS-IN        LBS         LBS        LBS/IN      LBS/IN
LBS/IN**2   LBS-IN**2  LBS-IN**2
     *****  **********  **********  **********  **********  **********  **********  **********  **********
**********  **********  **********
         1     -0.5510 -3.5000E-15 -7.4100E+05 -1.1400E-06 -3.4000E+04 -3.8800E-09     -249.00 -1.3800E-10
4100.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00      0.0000      0.0000
         2     -0.5510 -3.5600E-15 -7.4000E+05 -1.1400E-06 -3.3900E+04 -3.8600E-09     -249.00 -1.2900E-10
4100.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00      0.0000      0.0000
         3     -0.5510 -3.5600E-15 -7.4000E+05 -1.1400E-06 -3.3900E+04 -3.8600E-09     -249.00 -1.2900E-10
4100.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00      0.0000      0.0000
         4     -0.5510 -3.5600E-15 -7.4000E+05 -1.1400E-06 -3.3900E+04 -3.8600E-09     -249.00 -1.2900E-10
4100.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00      0.0000      0.0000
         5     -0.5510 -3.5600E-15 -7.4000E+05 -1.1400E-06 -3.3900E+04 -3.8600E-09     -249.00 -1.2900E-10
4100.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00      0.0000      0.0000
         6     -0.5510 -3.5600E-15 -7.4000E+05 -1.1400E-06 -3.3900E+04 -3.8600E-09     -249.00 -1.2900E-10
4100.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00      0.0000      0.0000
         7     -0.5510 -3.5000E-15 -7.4100E+05 -1.1400E-06 -3.4000E+04 -3.8800E-09     -249.00 -1.3800E-10
4100.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00      0.0000      0.0000
         8     -0.5510 -1.3700E-15 -6.6700E+05 -7.5800E-07 -2.9200E+04 -2.5100E-09     -206.00 -1.8200E-10
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3610.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
         9       -0.5510 -2.2800E-15 -6.5400E+05 -7.4300E-07 -2.8300E+04 -2.3500E-09      -198.00 -9.8900E-11
3610.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       216.00       108.00       252.00
468.00       0.0000       0.0000
        10       -0.5510 -2.2800E-15 -6.5400E+05 -7.4300E-07 -2.8300E+04 -2.3500E-09      -198.00 -9.8900E-11
3610.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       216.00       108.00       252.00
477.00       0.0000       0.0000
        11       -0.5510 -2.2800E-15 -6.5400E+05 -7.4300E-07 -2.8300E+04 -2.3500E-09      -198.00 -9.8900E-11
3610.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       216.00       108.00       252.00
477.00       0.0000       0.0000
        12       -0.5510 -2.2800E-15 -6.5400E+05 -7.4300E-07 -2.8300E+04 -2.3500E-09      -198.00 -9.8900E-11
3610.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       216.00       108.00       252.00
468.00       0.0000       0.0000
        13       -0.5510 -2.2800E-15 -6.5400E+05 -7.4300E-07 -2.8300E+04 -2.3500E-09      -198.00 -9.8900E-11
3610.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       216.00       108.00       252.00
468.00       0.0000       0.0000
        14       -0.5510 -1.3700E-15 -6.6700E+05 -7.5800E-07 -2.9200E+04 -2.5100E-09      -206.00 -1.8200E-10
3610.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
477.00       0.0000       0.0000
        15       -0.5510 -7.0000E-16 -6.6600E+05 -4.8200E-07 -2.9200E+04 -1.6100E-09      -206.00 -1.2900E-10
3130.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
        16       -0.5510 -1.4500E-15 -6.5400E+05 -4.7200E-07 -2.8400E+04 -1.4900E-09      -198.00 -6.1400E-11
3130.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
477.00       0.0000       0.0000
        17       -0.5510 -1.4500E-15 -6.5400E+05 -4.7200E-07 -2.8400E+04 -1.4900E-09      -198.00 -6.1400E-11
3130.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
        18       -0.5510 -1.4500E-15 -6.5400E+05 -4.7200E-07 -2.8400E+04 -1.4900E-09      -198.00 -6.1400E-11
3130.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
        19       -0.5510 -1.4500E-15 -6.5400E+05 -4.7200E-07 -2.8400E+04 -1.4900E-09      -198.00 -6.1400E-11
3130.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
        20       -0.5510 -1.4500E-15 -6.5400E+05 -4.7200E-07 -2.8400E+04 -1.4900E-09      -198.00 -6.1400E-11
3130.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
        21       -0.5510 -7.0000E-16 -6.6600E+05 -4.8200E-07 -2.9200E+04 -1.6100E-09      -206.00 -1.2900E-10
3130.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
        22       -0.5510 -4.7000E-16 -6.9500E+05 -1.5600E-07 -3.1100E+04 -5.0500E-10      -222.00 -1.9700E-11
2470.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
        23       -0.5510 -4.9800E-16 -6.9100E+05 -1.5500E-07 -3.0900E+04 -4.9800E-10      -220.00 -1.6300E-11
2470.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
        24       -0.5510 -5.0500E-16 -6.8900E+05 -1.5400E-07 -3.0800E+04 -4.9500E-10      -219.00 -1.5000E-11
2470.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
        25       -0.5510 -5.0500E-16 -6.8900E+05 -1.5400E-07 -3.0800E+04 -4.9500E-10      -219.00 -1.5000E-11
2470.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
        26       -0.5510 -4.9800E-16 -6.9100E+05 -1.5500E-07 -3.0900E+04 -4.9800E-10      -220.00 -1.6300E-11
2470.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
        27       -0.5510 -4.7000E-16 -6.9500E+05 -1.5600E-07 -3.1100E+04 -5.0500E-10      -222.00 -1.9700E-11
2470.0  2.6200E+10  9.4500E+09
   x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
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        28       -0.5510 -8.3700E-14 -7.0100E+05 -5.4400E-08 -3.1600E+04 -3.3100E-09     -227.00 -2.9000E-11
1610.0  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
        29       -0.5510 -8.3700E-14 -7.0000E+05 -5.4200E-08 -3.1500E+04 -3.3000E-09     -226.00 -2.8900E-11
1610.0  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
        30       -0.5510 -8.3700E-14 -7.0000E+05 -5.4200E-08 -3.1500E+04 -3.3000E-09     -226.00 -2.8900E-11
1610.0  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
        31       -0.5510 -8.3700E-14 -7.0000E+05 -5.4200E-08 -3.1500E+04 -3.3000E-09     -226.00 -2.8900E-11
1610.0  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
        32       -0.5510 -8.3700E-14 -7.0000E+05 -5.4200E-08 -3.1500E+04 -3.3000E-09     -226.00 -2.8900E-11
1610.0  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
        33       -0.5510 -8.3700E-14 -7.0100E+05 -5.4400E-08 -3.1600E+04 -3.3100E-09     -227.00 -2.9000E-11
1610.0  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
        34       -0.5510 -2.3600E-13 -7.0000E+05 -1.5300E-07 -3.1700E+04 -9.3500E-09     -227.00 -8.1700E-11
751.00  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
        35       -0.5510 -2.3600E-13 -6.9900E+05 -1.5200E-07 -3.1600E+04 -9.3200E-09     -226.00 -8.1400E-11
751.00  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
        36       -0.5510 -2.3600E-13 -6.9900E+05 -1.5200E-07 -3.1600E+04 -9.3200E-09     -226.00 -8.1400E-11
751.00  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
        37       -0.5510 -2.3600E-13 -6.9900E+05 -1.5200E-07 -3.1600E+04 -9.3200E-09     -226.00 -8.1400E-11
751.00  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
        38       -0.5510 -2.3600E-13 -6.9900E+05 -1.5200E-07 -3.1600E+04 -9.3200E-09     -226.00 -8.1400E-11
751.00  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
        39       -0.5510 -2.3600E-13 -7.0000E+05 -1.5300E-07 -3.1700E+04 -9.3500E-09     -227.00 -8.1700E-11
751.00  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
        40       -0.5510 -3.8800E-13 -6.9900E+05 -2.5100E-07 -3.1700E+04 -1.5400E-08     -227.00 -1.3500E-10
94.200  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
        41       -0.5510 -3.8800E-13 -6.9800E+05 -2.5000E-07 -3.1600E+04 -1.5400E-08     -226.00 -1.3400E-10
94.200  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
        42       -0.5510 -3.8800E-13 -6.9800E+05 -2.5000E-07 -3.1600E+04 -1.5400E-08     -226.00 -1.3400E-10
94.200  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
        43       -0.5510 -3.8800E-13 -6.9800E+05 -2.5000E-07 -3.1600E+04 -1.5400E-08     -226.00 -1.3400E-10
94.200  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
        44       -0.5510 -3.8800E-13 -6.9800E+05 -2.5000E-07 -3.1600E+04 -1.5400E-08     -226.00 -1.3400E-10
94.200  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
        45       -0.5510 -3.8800E-13 -6.9900E+05 -2.5100E-07 -3.1700E+04 -1.5400E-08     -227.00 -1.3500E-10
94.200  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
        46       -0.5510 -5.4100E-13 -6.9800E+05 -3.4800E-07 -3.1800E+04 -2.1500E-08     -227.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
        47       -0.5510 -5.4100E-13 -6.9600E+05 -3.4700E-07 -3.1700E+04 -2.1500E-08     -226.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
     X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
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468.00       0.0000      0.0000
          48       -0.5510 -5.4100E-13 -6.9600E+05 -3.4700E-07 -3.1700E+04 -2.1500E-08     -226.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00       0.0000      0.0000
          49       -0.5510 -5.4100E-13 -6.9600E+05 -3.4700E-07 -3.1700E+04 -2.1500E-08     -226.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00       0.0000      0.0000
          50       -0.5510 -5.4100E-13 -6.9600E+05 -3.4700E-07 -3.1700E+04 -2.1500E-08     -226.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00       0.0000      0.0000
          51       -0.5510 -5.4100E-13 -6.9800E+05 -3.4800E-07 -3.1800E+04 -2.1500E-08     -227.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00       0.0000      0.0000

        Min.       -0.5510 -5.4100E-13 -7.4100E+05 -1.1400E-06 -3.4000E+04 -2.1500E-08     -249.00 -1.8700E-10
94.200  2.6200E+10  9.4500E+09
      Pile N.          1          46           1           1           1          46           1          46
40             1           1

        * MAXIMUM VALUES AND LOCATIONS *

       PILE           DEFLECTION            BENDING MOMENT           SHEAR FORCE            SOIL REACTION
TOTAL         FLEXURAL RIGIDITY
                 y-DIR       z-DIR       z-DIR       y-DIR       y-DIR       z-DIR       y-DIR       z-DIR
STRESS          z-DIR       y-DIR
                    IN          IN      LBS-IN      LBS-IN         LBS         LBS      LBS/IN      LBS/IN
LBS/IN**2    LBS-IN**2   LBS-IN**2
       *****  **********  **********  **********  **********  **********  **********  **********  **********
**********  **********  **********
           1       -0.5510  4.7500E-13  2.3600E+06  3.2700E-07      7790.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000       0.0000      0.0000
           2       -0.5510  4.7500E-13  2.3500E+06  3.2600E-07      7770.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000       0.0000      0.0000
           3       -0.5510  4.7500E-13  2.3500E+06  3.2600E-07      7770.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000       0.0000      0.0000
           4       -0.5510  4.7500E-13  2.3500E+06  3.2600E-07      7770.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000       0.0000      0.0000
           5       -0.5510  4.7500E-13  2.3500E+06  3.2600E-07      7770.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000       0.0000      0.0000
           6       -0.5510  4.7500E-13  2.3500E+06  3.2600E-07      7770.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000       0.0000      0.0000
           7       -0.5510  4.7500E-13  2.3600E+06  3.2700E-07      7790.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000       0.0000      0.0000
           8       -0.5510  3.4800E-13  2.1200E+06  2.1600E-07      6700.0  1.2700E-08      177.00  1.0900E-10
1.9800E+04  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
0.0000       0.0000      0.0000
           9       -0.5510  3.4800E-13  2.0800E+06  2.1100E-07      6530.0  1.2300E-08      175.00  1.0500E-10
1.9500E+04  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
0.0000       0.0000      0.0000
          10       -0.5510  3.4800E-13  2.0800E+06  2.1100E-07      6530.0  1.2300E-08      175.00  1.0500E-10
1.9500E+04  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
0.0000       0.0000      0.0000
          11       -0.5510  3.4800E-13  2.0800E+06  2.1100E-07      6530.0  1.2300E-08      175.00  1.0500E-10
1.9500E+04  2.6200E+10  9.4500E+09
       X(IN)       0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
0.0000       0.0000      0.0000
          12       -0.5510  3.4800E-13  2.0800E+06  2.1100E-07      6530.0  1.2300E-08      175.00  1.0500E-10
1.9500E+04  2.6200E+10  9.4500E+09
```

Row 1

Row 2

```
      x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         13    -0.5510 3.4800E-13 2.0800E+06 2.1100E-07     6530.0 1.2300E-08     175.00 1.0500E-10
 1.9500E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         14    -0.5510 3.4800E-13 2.1200E+06 2.1600E-07     6700.0 1.2700E-08     177.00 1.0900E-10
 1.9800E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
```

Row 3

```
         15    -0.5510 2.2100E-13 2.1200E+06 1.3700E-07     6690.0 8.0600E-09     177.00 6.9200E-11
 1.9400E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         16    -0.5510 2.2100E-13 2.0800E+06 1.3400E-07     6520.0 7.8200E-09     175.00 6.6500E-11
 1.9000E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         17    -0.5510 2.2100E-13 2.0800E+06 1.3400E-07     6520.0 7.8200E-09     175.00 6.6500E-11
 1.9000E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         18    -0.5510 2.2100E-13 2.0800E+06 1.3400E-07     6520.0 7.8200E-09     175.00 6.6500E-11
 1.9000E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         19    -0.5510 2.2100E-13 2.0800E+06 1.3400E-07     6520.0 7.8200E-09     175.00 6.6500E-11
 1.9000E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         20    -0.5510 2.2100E-13 2.0800E+06 1.3400E-07     6520.0 7.8200E-09     175.00 6.6500E-11
 1.9000E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         21    -0.5510 2.2100E-13 2.1200E+06 1.3700E-07     6690.0 8.0600E-09     177.00 6.9200E-11
 1.9400E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     252.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
```

Row 4

```
         22    -0.5510 6.8600E-14 2.2200E+06 4.4100E-08     7090.0 2.6600E-09     187.00 2.3300E-11
 1.9400E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     243.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         23    -0.5510 6.8600E-14 2.2000E+06 4.3900E-08     7030.0 2.6400E-09     185.00 2.3000E-11
 1.9300E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     243.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         24    -0.5510 6.8600E-14 2.2000E+06 4.3800E-08     7000.0 2.6300E-09     185.00 2.2900E-11
 1.9300E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     243.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         25    -0.5510 6.8600E-14 2.2000E+06 4.3800E-08     7000.0 2.6300E-09     185.00 2.2900E-11
 1.9300E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     243.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         26    -0.5510 6.8600E-14 2.2000E+06 4.3900E-08     7030.0 2.6400E-09     185.00 2.3000E-11
 1.9300E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     243.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
         27    -0.5510 6.8600E-14 2.2200E+06 4.4100E-08     7090.0 2.6600E-09     187.00 2.3300E-11
 1.9400E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     0.0000     0.0000     0.0000     243.00     0.0000     342.00     63.000
   0.0000     0.0000     0.0000
```

Row 5

```
         28    -0.5510 4.5900E-16 2.2400E+06 1.9200E-07     7180.0 6.3600E-10     189.00 3.4900E-11
 1.8700E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     243.00     0.0000     0.0000     243.00     216.00     342.00     243.00
   0.0000     0.0000     0.0000
         29    -0.5510 5.0800E-16 2.2300E+06 1.9200E-07     7160.0 6.2800E-10     189.00 3.0200E-11
 1.8700E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     243.00     0.0000     0.0000     243.00     216.00     342.00     243.00
   0.0000     0.0000     0.0000
         30    -0.5510 5.0800E-16 2.2300E+06 1.9200E-07     7160.0 6.2800E-10     189.00 3.0200E-11
 1.8700E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     243.00     0.0000     0.0000     243.00     216.00     342.00     243.00
   0.0000     0.0000     0.0000
         31    -0.5510 5.0800E-16 2.2300E+06 1.9200E-07     7160.0 6.2800E-10     189.00 3.0200E-11
 1.8700E+04 2.6200E+10 9.4500E+09
      x(IN)     0.0000     243.00     0.0000     0.0000     243.00     216.00     342.00     243.00
   0.0000     0.0000     0.0000
         32    -0.5510 5.0800E-16 2.2300E+06 1.9200E-07     7160.0 6.2800E-10     189.00 3.0200E-11
```

MZ

```
1.8700E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        33     -0.5510  4.5900E-16  2.2400E+06  1.9200E-07      7180.0  6.3600E-10      189.00  3.4900E-11
1.8700E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
```

Row 6

```
        34     -0.5510  1.3800E-15  2.2400E+06  5.4200E-07      7160.0  1.7700E-09      189.00  8.7900E-11
1.7900E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        35     -0.5510  1.4500E-15  2.2300E+06  5.4100E-07      7140.0  1.7500E-09      188.00  8.1000E-11
1.7800E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        36     -0.5510  1.4500E-15  2.2300E+06  5.4100E-07      7140.0  1.7500E-09      188.00  8.1000E-11
1.7800E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        37     -0.5510  1.4500E-15  2.2300E+06  5.4100E-07      7140.0  1.7500E-09      188.00  8.1000E-11
1.7800E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        38     -0.5510  1.4500E-15  2.2300E+06  5.4100E-07      7140.0  1.7500E-09      188.00  8.1000E-11
1.7800E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        39     -0.5510  1.3800E-15  2.2400E+06  5.4200E-07      7160.0  1.7700E-09      189.00  8.7900E-11
1.7900E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
```

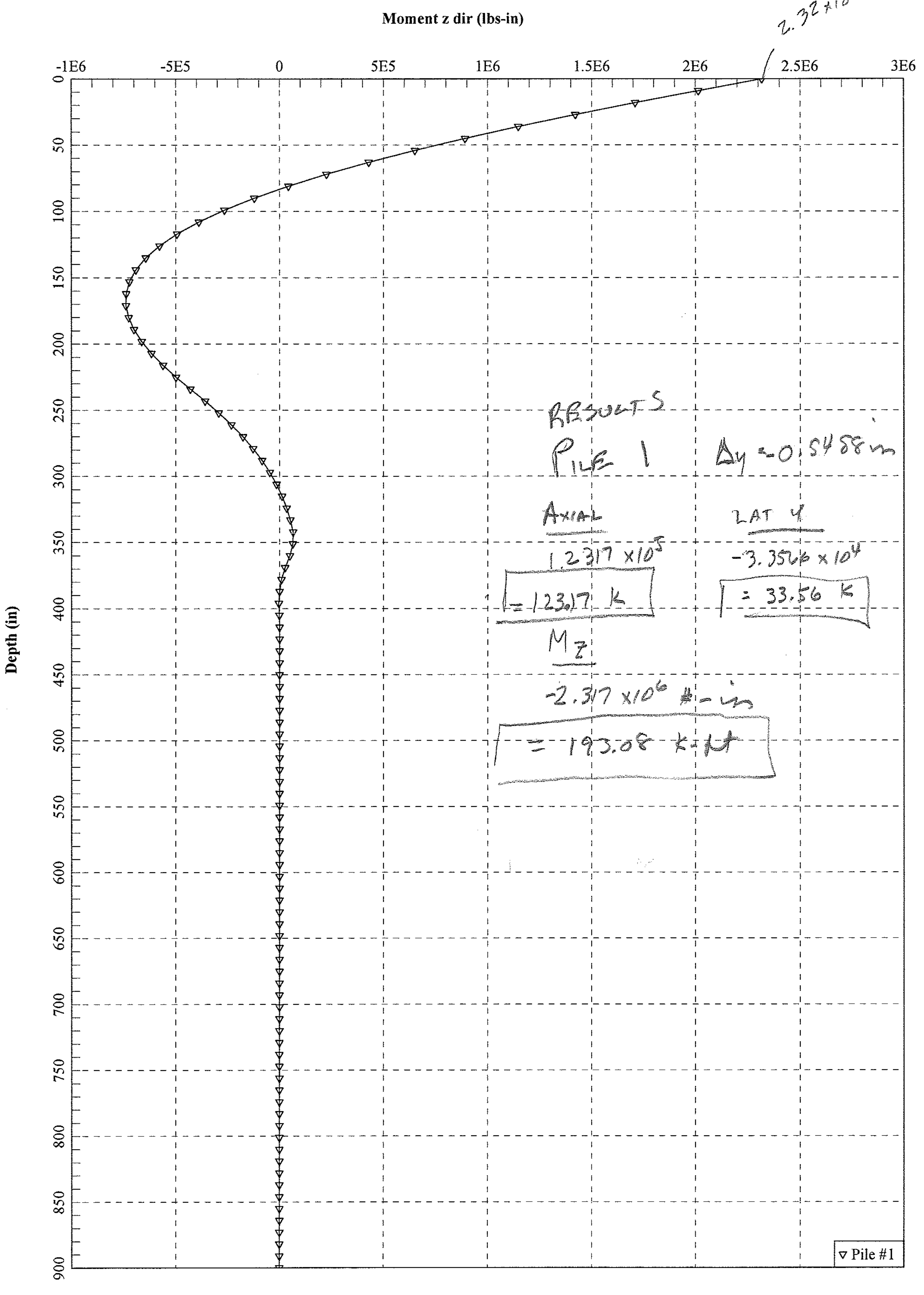
Row 7

```
        40     -0.5510  2.3600E-15  2.2400E+06  8.9100E-07      7130.0  2.8800E-09      188.00  1.3400E-10
1.7200E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        41     -0.5510  2.4700E-15  2.2300E+06  8.9000E-07      7110.0  2.8600E-09      188.00  1.2300E-10
1.7200E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        42     -0.5510  2.4700E-15  2.2300E+06  8.9000E-07      7110.0  2.8600E-09      188.00  1.2300E-10
1.7200E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        43     -0.5510  2.4700E-15  2.2300E+06  8.9000E-07      7110.0  2.8600E-09      188.00  1.2300E-10
1.7200E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        44     -0.5510  2.4700E-15  2.2300E+06  8.9000E-07      7110.0  2.8600E-09      188.00  1.2300E-10
1.7200E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        45     -0.5510  2.3600E-15  2.2400E+06  8.9100E-07      7130.0  2.8800E-09      188.00  1.3400E-10
1.7200E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
```

Row 8

```
        46     -0.5510  3.4900E-15  2.2400E+06  1.2400E-06      7110.0  3.9500E-09      188.00  1.6000E-10
1.8100E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        47     -0.5510  3.5900E-15  2.2300E+06  1.2400E-06      7090.0  3.9200E-09      187.00  1.4800E-10
1.8000E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        48     -0.5510  3.5900E-15  2.2300E+06  1.2400E-06      7090.0  3.9200E-09      187.00  1.4800E-10
1.8000E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        49     -0.5510  3.5900E-15  2.2300E+06  1.2400E-06      7090.0  3.9200E-09      187.00  1.4800E-10
1.8000E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        50     -0.5510  3.5900E-15  2.2300E+06  1.2400E-06      7090.0  3.9200E-09      187.00  1.4800E-10
1.8000E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
        51     -0.5510  3.4900E-15  2.2400E+06  1.2400E-06      7110.0  3.9500E-09      188.00  1.6000E-10
1.8100E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      243.00      216.00      342.00      243.00
0.0000      0.0000      0.0000
```

```
       Max.     -0.5510  4.7500E-13  2.3600E+06  1.2400E-06      7790.0  2.0000E-08       197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
     Pile N.         1           1           1          46           1           1           1           1
1           1           1
```



2.32×10^6 #-in

RESULTS

PILE 1 $\Delta y = -0.5488$ in

AXIAL

1.2317×10^5

$\boxed{= 123.17 \text{ K}}$

LAT Y

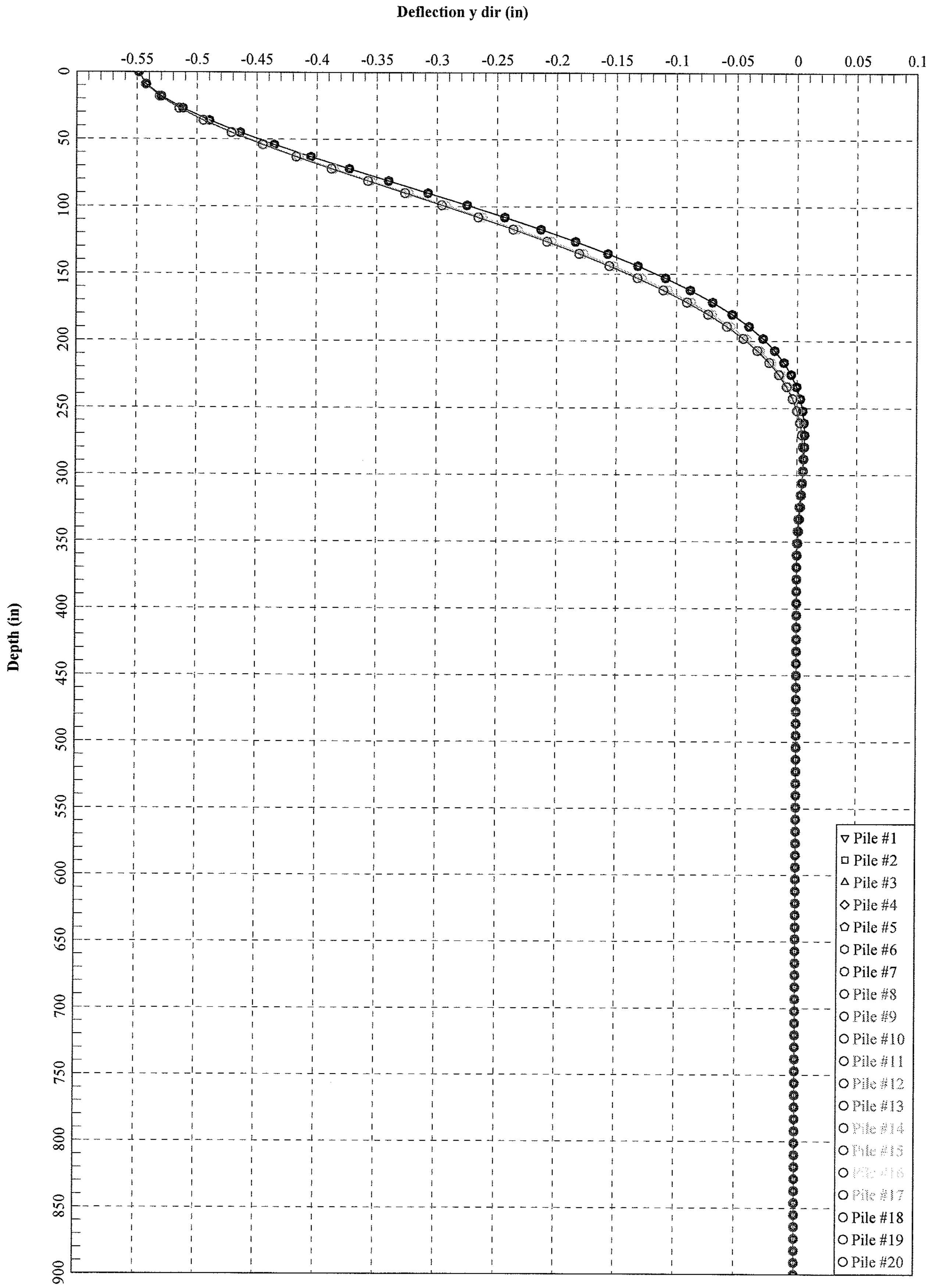
-3.356×10^4

$\boxed{= 33.56 \text{ K}}$

M_Z

-2.317×10^6 #-in

$\boxed{= 193.08 \text{ K-ft}}$


Deflection y dir (in)
-0.55
-0.5
-0.45
-0.4
-0.35
-0.3
-0.25
-0.2
-0.15
-0.1
-0.05
0
0.05
0.1
Depth (in)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
850
900
▽ Pile #1
□ Pile #2
△ Pile #3
◇ Pile #4
⬠ Pile #5
○ Pile #6
○ Pile #7
○ Pile #8
○ Pile #9
○ Pile #10
○ Pile #11
○ Pile #12
○ Pile #13
○ Pile #14
○ Pile #15
○ Pile #16
○ Pile #17
○ Pile #18
○ Pile #19
○ Pile #20

ATTACHMENT

F-Q1.4 Rigid Cap Analyses Results for the Red River (LPP) Control Structure Wing Wall F

BARR ENGINEERING			DATE 2/4/2011	SHEET NO.
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER SUBJECT Control Structure: ND RRN 35k s35k Load Cases: 1	SECTION F

Case 1
Event 100 years

H.W. EL = 922.01 ft
T.W. EL = 902.41 ft

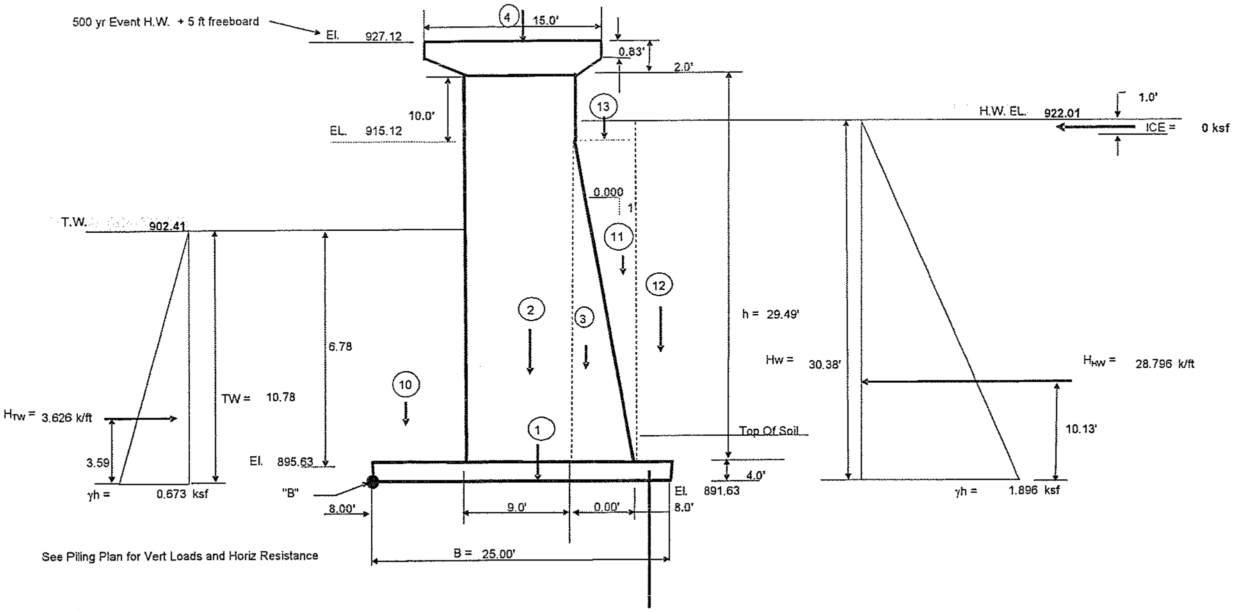
500 yr H.W. EL = 922.12 ft

Top of Ftg EL. = 895.63 ft

File:

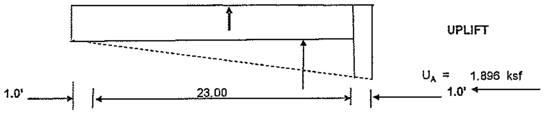
State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model

U_B = 0.673 ksf



REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	4	30	12.00	1.17	0.15	rec	63.2	12.50	789.8
Monolith	2	30	9.00	29.49	0.15	rec	1194.3	12.50	14,929.3
Monolith	3	30	0.00	19.49	0.15	tri	0.0	17.00	0.0
					D.L. Concrete	ΣVc =	1763.6	ΣM_V =	22,044.4

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	8.00	6.78	0.0624	rec	101.5	4.00	406.1
H.W. on Monolith Slope	11	30	0.00	19.49	0.0624	tri	0.0	17.00	0.0
H.W. Above Slope	13	30	0.00	6.89	0.0624	rec	0.0	17.00	0.0
H.W. on Mono. ftg	12	30	8.00	26.38	0.0624	rec	395.1	21.00	8,296.4
					D.L. Water	ΣVw =	496.6	ΣM_V =	8,702.6

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	25.00	0.673	rec	-504.5	12.50	-6,306
	U_A	30	25.00	1.223	tri	-458.6	16.67	-7,644
					ΣU =	-963.1	ΣM_U =	-13,950

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	1.00	0.00	rec	0.0	29.88	0.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{TW}	30	3.626	tri	108.77	3.59	390.85
H_{HW}	30	-28.796	tri	-863.88	10.13	-8748.18
			ΣW =	-755.10	ΣM_W =	-8357.3

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-22308	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	30747	kip-ft

Sum of Moments	$\Sigma M_{net} = M_R + M_{OT}$ =	8,439	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	1,297	kips
Sum of Horizontal Forces	H = Ice + water =	-755	kips

Location of Resultant
Xr = ΣM / P = 6.51 ft from Toe
e = B/2 - Xr = 5.99 ft
B/6 = 4.167 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER		
			SUBJECT	Control Structure: ND RRN 35k s35k Load Cases: 1.1	SECTION F

Case 1.1
Event 100 years

H.W. EL = 922.01 ft
T.W. EL. = 902.41 ft

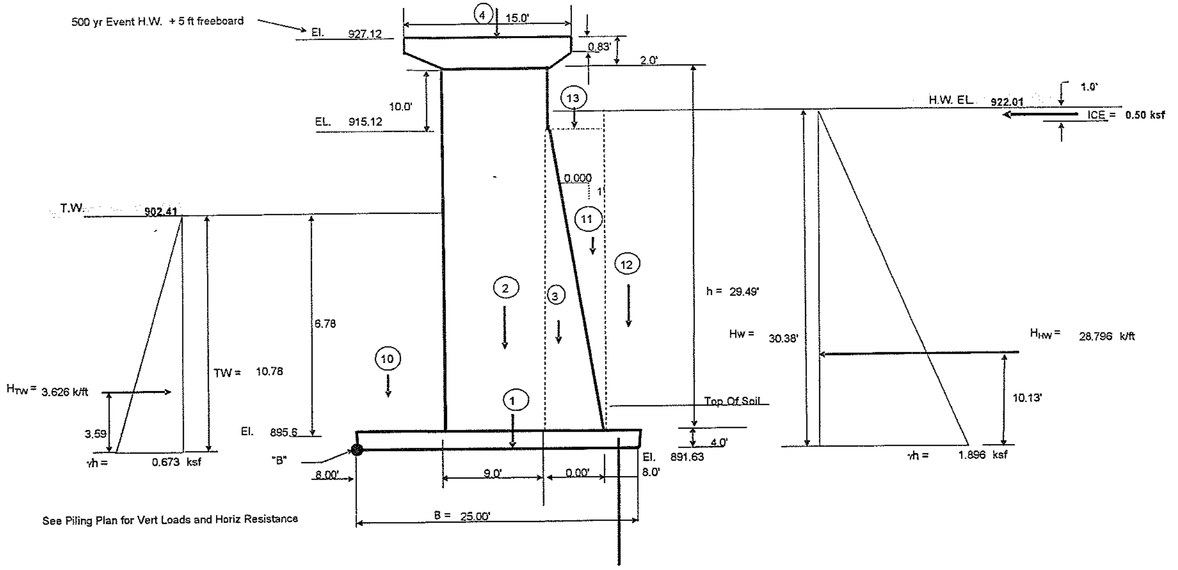
500 yr H.W. EL = 922.12 ft

Top of Ftg EL. = 895.63 ft

File:

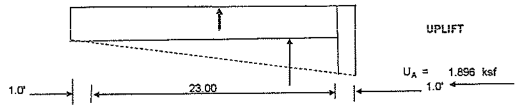
State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section
Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model

U_B = 0.673 ksf



REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	4	30	12.00	1.17	0.15	rec	63.2	12.50	789.8
Monolith	2	30	9.00	29.49	0.15	rec	1194.3	12.50	14,929.3
Monolith	3	30	0.00	19.49	0.15	tri	0.0	17.00	0.0
					D.L. Concrete	ΣVc =	1763.6	ΣM$_V$ =	22,044.4

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	8.00	6.78	0.0624	rec	101.5	4.00	406.1
H.W. on Monolith Slope	11	30	0.00	19.49	0.0624	tri	0.0	17.00	0.0
H.W. Above Slope	13	30	0.00	6.89	0.0624	rec	0.0	17.00	0.0
H.W. on Mono. ftg	12	30	8.00	26.38	0.0624	rec	395.1	21.00	8,296.4
					D.L. Water	ΣVw =	496.6	ΣM$_V$ =	8,702.6

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	25.00	0.673	rec	-504.5	12.50	-6,306
	U_A	30	25.00	1.223	tri	-458.6	16.67	-7,644
					ΣU =	-963.1	ΣM$_U$ =	-13,950

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	1.00	-0.50	rec	-15.0	29.88	-448.2

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{TW}	30	3.626	tri	108.77	3.59	390.85
H_{HW}	30	-28.796	tri	-863.88	10.13	-8748.18
			ΣW =	-755.10	ΣM$_W$ =	-8357.3

Overturning Moments $\Sigma M_{OT} = M_U + M_W + M_{ICE}$ = -22756 kip-ft
Resisting Moments $\Sigma M_R = M_V$ = 30747 kip-ft

Sum of Moments	$\Sigma M_{net} = M_R + M_{OT}$ =	7,991	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	1,297	kips
Sum of Horizontal Forces	H = Ice + water =	-770	kips

Location of Resultant
Xr = ΣM / P = 6.16 ft from Toe
e = B/2 - Xr = 6.34 ft
B/6 = 4.167 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	**Flood Control ND Diversion Inlet - Corps Of Engineers**	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/4/11		PKN	SUBJECT	**Control Structure:** ND RRN 35k s35k Load Cases: **2**	SECTION F

Case **2**
Event **500** years

H.W. EL = 922.12 ft
T.W. EL. = 914.41 ft

500 yr H.W. EL = 922.12 ft

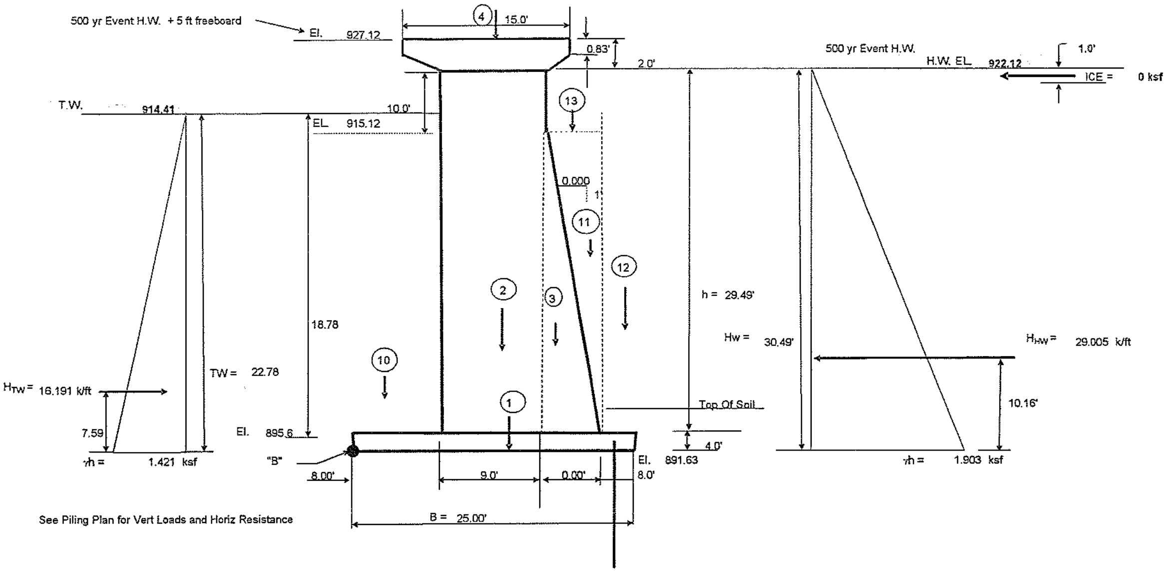
Top of Ftg EL. = 895.63 ft

File:

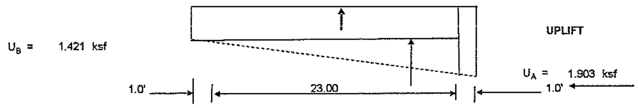
State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section

Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model



REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	4	30	12.00	1.17	0.15	rec	63.2	12.50	789.8
Monolith	2	30	9.00	29.49	0.15	rec	1194.3	12.50	14,929.3
Monolith	3	30	0.00	19.49	0.15	tri	0.0	17.00	0.0
					D.L. Concrete	ΣVc =	1763.6	ΣM_V =	22,044.4

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	8.00	18.78	0.0624	rec	281.2	4.00	1,125.0
H.W. on Monolith Slope	11	30	0.00	19.49	0.0624	tri	0.0	17.00	0.0
H.W. Above Slope	13	30	0.00	7.00	0.0624	rec	0.0	17.00	0.0
H.W. on Mono. ftg	12	30	8.00	26.49	0.0624	rec	396.7	21.00	8,331.0
					D.L. Water	ΣVw =	678.0	ΣM_V =	9,456.0

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	25.00	1.421	rec	-1066.1	12.50	-13,326
	U_A	30	25.00	0.481	tri	-180.4	16.67	-3,007
					ΣU =	-1246.5	ΣM_U =	-16,333

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	1.00	0.00	rec	0.0	29.99	0.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{TW}	30	16.191	tri	485.72	7.59	3688.21
H_{HW}	30	-29.005	tri	-870.14	10.16	-8843.55
			ΣW =	-384.43	ΣM_W =	-5155.3

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-21489	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	31500	kip-ft

Sum of Moments	$\Sigma M_{net} = M_R + M_{OT}$ =	**10,012**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**1,195**	kips
Sum of Horizontal Forces	H = Ice + water =	**-384**	kips

Location of Resultant
Xr = ΣM / P = 8.38 ft from Toe
e = B/2 - Xr = 4.12 ft
B/6 = 4.167 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/4/11		PKN	SUBJECT	Control Structure: ND RRN 35k s35k Load Cases: 2.1	SECTION F

Case 2.1
Event 500 years

Per COE 2/1/11: HW = 500yr + 5' w/ No ice

File:

H.W. EL = 927.12 ft
T.W. EL. = 914.41 ft

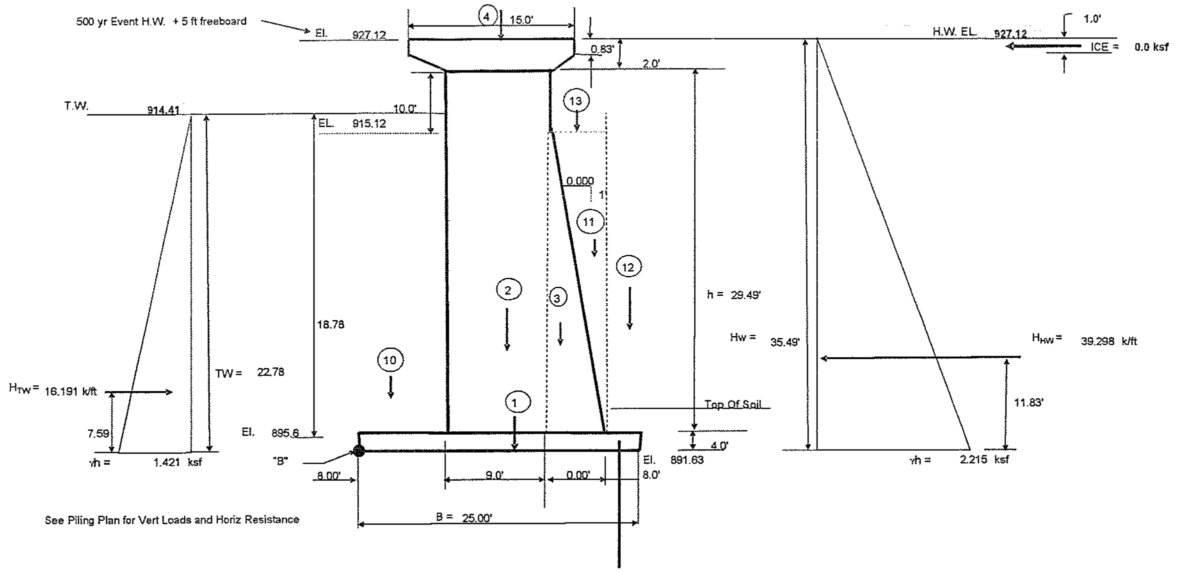
500 yr H.W. EL = 922.12 ft

Top of Ftg EL. = 895.63 ft

State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section

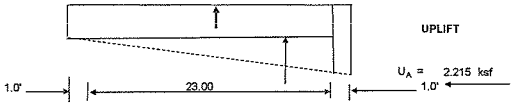
Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model

U_B = 1.421 ksf



UPLIFT

U_A = 2.215 ksf

REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	4	30	12.00	1.17	0.15	rec	63.2	12.50	789.8
Monolith	2	30	9.00	29.49	0.15	rec	1194.3	12.50	14,929.3
Monolith	3	30	0.00	19.49	0.15	tri	0.0	17.00	0.0
					D.L. Concrete	ΣVc =	1763.6	ΣM$_V$ =	22,044.4

CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	8.00	18.78	0.0624	rec	281.2	4.00	1,125.0
H.W. on Monolith Slope	11	30	0.00	19.49	0.0624	tri	0.0	17.00	0.0
H.W. Above Slope	13	30	0.00	12.00	0.0624	rec	0.0	17.00	0.0
H.W. on Mono. ftg	12	30	8.00	31.49	0.0624	rec	471.6	21.00	9,903.5
					D.L. Water	ΣVw =	752.8	ΣM$_V$ =	11,028.5

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	25.00	1.421	rec	-1066.1	12.50	-13,326
	U_A	30	25.00	0.793	tri	-297.4	16.67	-4,957
					ΣU =	-1363.5	ΣM$_U$ =	-18,283

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	1.00	0.00	rec	0.0	34.99	0.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{TW}	30	16.191	tri	485.72	7.59	3688.21
H_{HW}	30	-39.298	tri	-1178.93	11.83	-13946.74
			ΣW =	-693.21	ΣM$_W$ =	-10258.5

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-28542	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	33073	kip-ft

Sum of Moments	$\Sigma M_{net} = M_R + M_{OT}$ =	4,531	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	1,153	kips
Sum of Horizontal Forces	H = Ice + water =	-693	kips

Location of Resultant
$X_r = \Sigma M / P$ = 3.93 ft from Toe
$e = B/2 - X_r$ = 8.57 ft
$B/6$ = 4.167 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	**Flood Control ND Diversion Inlet - Corps Of Engineers**	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/4/11		PKN	SUBJECT	**Control Structure:** ND RRN 35k s35k Load Cases: 3	SECTION F

Case 3
Event 2 years

H.W. EL = ft
T.W. EL. = ft

500 yr H.W. EL = 922.12 ft

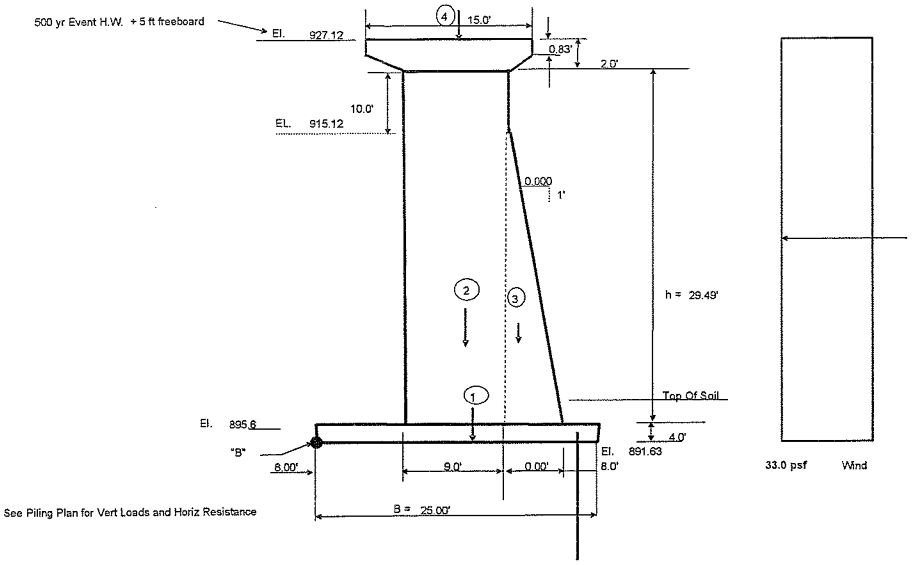
Top of Ftg EL. = 895.63 ft

File:

State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section

Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



See Piling Plan for Vert Loads and Horiz Resistance

Non-Overflow Dam

Vertical Loads	**Section**	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	**1**	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	**4**	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	**4**	30	12.00	1.17	0.15	rec	63.2	12.50	789.8
Monolith	**2**	30	9.00	29.49	0.15	rec	1194.3	12.50	14,929.3
Monolith	**3**	30	0.00	19.49	0.15	tri	0.0	17.00	0.0
					D.L. Concrete	$\Sigma Vc =$	**1763.6**	$\Sigma M_V =$	**22,044.4**

← CONSTANT FOR ALL LOAD CASES

Horizontal Loads		L ft	H ft	Pressure ksf	shape	H K	arm ft	Mw ft-k
	Wind	30	35.49	-0.03	rec	-35.1	17.745	-623.5

Overturning Moments	$\Sigma M_{OT} = M_W =$	-623	kip-ft
Resisting Moments	$\Sigma M_R = M_V =$	22044	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT} =$	**21,421**	kip-ft
Sum of Vertical Forces	P = Conc =	**1,764**	kips
Sum of Horizontal Forces	H = Wind =	**-35**	kips

Location of Resultant $Xr = \Sigma M / P =$ 12.15 ft from Toe
$e = B/2 - Xr =$ 0.35 ft
$B/6 =$ 4.167 ft

BARR ENGINEERING			DATE 2/4/2011	SHEET NO.
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/4/11		PKN	SUBJECT **Control Structure:** ND RRN 35k s35k Load Cases: **4**	SECTION F

Case 4
Event years

File:

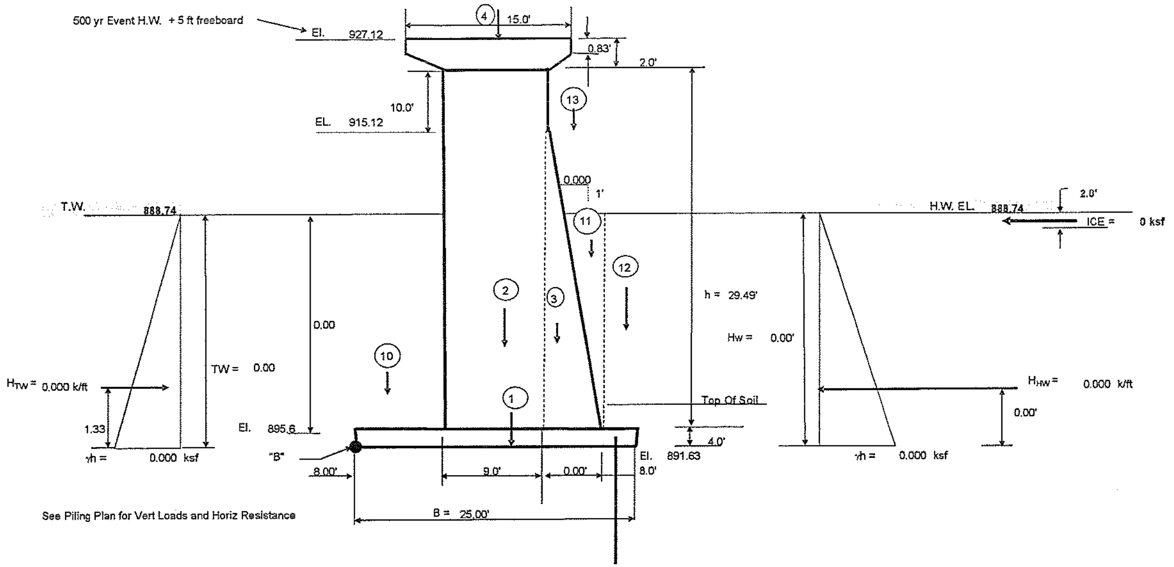
H.W. EL = 888.74 ft **Normal Water Level**
T.W. EL. = 888.74 ft

State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft DEC, JAN, FEB Mean Water Elevation

500 yr H.W. EL = 922.12 ft

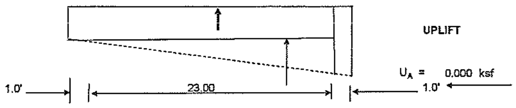
Non-Overflow Section
Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg

Top of Ftg EL. = 895.63 ft



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model

U_B = 0.000 ksf



REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	4	30	12.00	1.17	0.15	rec	63.2	12.50	789.8
Monolith	2	30	9.00	29.49	0.15	rec	1194.3	12.50	14,929.3
Monolith	3	30	0.00	19.49	0.15	tri	0.0	17.00	0.0
					D.L. Concrete	ΣVc =	**1763.6**	ΣM_V =	**22,044.4**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	8.00	0.00	0.0624	rec	0.0	4.00	0.0
H.W. on Monolith Slope	11	30	0.00	-6.89	0.0624	tri	0.0	17.00	0.0
H.W. Above Slope	13	30	0.00	-26.38	0.0624	rec	0.0	17.00	0.0
H.W. on Mono. ftg	12	30	8.00	0.00	0.0624	rec	0.0	21.00	0.0
					D.L. Water	ΣVw =	**0.0**	ΣM_V =	**0.0**

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	25.00	0.000	rec	0.0	12.50	0
	U_A	30	25.00	0.000	tri	0.0	16.67	0
					ΣU =	**0.0**	ΣM_U =	**0**

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	2.00	0.00	rec	0.0	-1	0.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{TW}	30	0.000	tri	0.00	1.33	0.00
H_{HW}	30	0.000	tri	0.00	0.00	0.00
			ΣW =	**0.00**	ΣM_W =	**0.0**

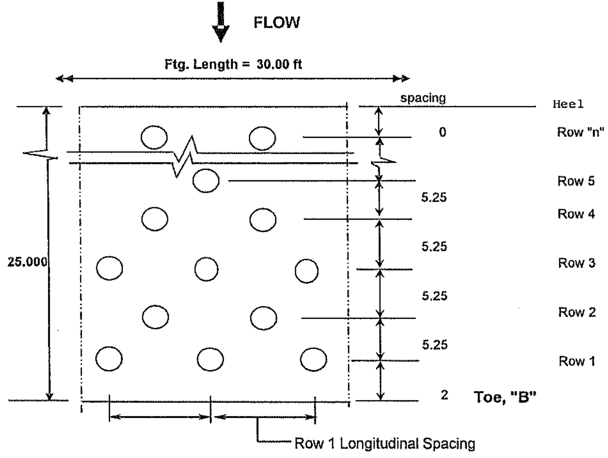
Overturning Moments $\Sigma M_{OT} = M_U + M_W + M_{ICE}$ = 0 kip-ft
Resisting Moments $\Sigma M_R = M_V$ = 22044 kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**22,044**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**1,764**	kips
Sum of Horizontal Forces	H = Ice + water =	**0**	kips

Location of Resultant $Xr = \Sigma M / P$ = 12.50 ft from Toe
e = B/2 - Xr = - ft
B/6 = 4.167 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers		
			PROJECT NUMBER			
			SUBJECT	Pile Capacity Control Structure: 35k	SECTION	F

PILE FOUNDATION DESIGN



L = 30.00 ft

PILE PATTERN GEOMETRY

Transverse Spacing		Distance to Toe, d_{toe}	Longitudinal Spacing	Batter	Piles per Row (N)	Edge Dist (ft)		Trial N	
Row 1 to Toe	2.00 ft	2.0 ft	4.25 ft	0 "/12"	6	4.38	1	8	
Row 1 to Row 2	5.25 ft	7.3 ft	4.25 ft	0 "/12"	6	4.38	2	8	12
Row 2 to Row 3	5.25 ft	12.5 ft	5.00 ft	0 "/12"	5	5.00	0	7	17
Row 3 to Row 4	5.25 ft	17.8 ft	5.00 ft	0 "/12"	6	2.50	1	7	23
Row 4 to Row 5	5.25 ft	23.0 ft	4.25 ft	0 "/12"	6	4.38	2	8	29
Row 5 to Row 6	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	15.00	0	0	35
Row 6 to Row 7	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	15.00	0	0	41
Row 7 to Row 8	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	15.00	0	0	47
Row 8 to Row 9	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	15.00	0	0	
Row 9 to Row 10	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	15.00	0	0	
Row 10 to Row 11	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	15.00	0	0	
Row 11 to Row 12	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	15.00	0	0	
Row 12 to Row 13	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	15.00	0	0	
Row 13 to Row 14	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	15.00	0	0	
Row 14 to Row 15	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	15.00	0	0	
Last Row to Heel	2.00 ft								
	25.00 ft				ΣN = 29			38	

Note: Enter 0 for Longitudinal Spacing for Rows Not Used)

				Ftg EL.	891.63
Pile Properties:	Pile Type: HP	(C.I.P or HP)	Pile Length = **56.8 ft**	Pile Tip El.	835.8
	HP Nominal Depth, h = 14.0 in			Pile Cap Embed =	1.00 ft
	Wt. per ft, plf 74		**Total pile Length =**	**1,648**	LF

ALLOWABLE LOADS (from Geotechnical)

Service	Allowable Pile Loads						
Case	1 Usual	1.1 Unsual	2 Unusual	2.1 Extreme	3 Unusual	4 Usual	
Axial	59.8 tons	79.7 tons	79.7 tons	104.0 tons	79.7 tons	31.8 tons	
Tension	40.6 tons	54.1 tons	54.1 tons	70.6 tons	54.1 tons	7.6 tons	
Lateral	31.0 kips/pile	35.0 kips/pile	35.0 kips/pile	35.0 kips/pile	31.0 kips/pile	22.0 kips/pile	w/o Group effects

Pile Group Properties

N.A. of Pile Group to Toe

$X_{NA} = (\Sigma N * d_{toe}) / \Sigma N =$ **12.50 ft**

	Dist. From N.A. to Pile Row	d	N	$I = N * d^2$
1	Dist. To Row 1	10.50 ft	6	661.5
2	Dist. To Row 2	5.25 ft	6	165.4
0	Dist. Row 3	0.00 ft	5	0.0
1	Dist. Row 4	-5.25 ft	6	165.4
2	Dist. Row 5	-10.50 ft	6	661.5
0	Row 6 (not used)	0.00 ft	0	0.0
0	Row 7 (not used)	0.00 ft	0	0.0
0	Row 8 (not used)	0.00 ft	0	0.0
0	Row 9 (not used)	0.00 ft	0	0.0
0	Row 10 (not used)	0.00 ft	0	0.0
0	Row 11 (not used)	0.00 ft	0	0.0
0	Row 12 (not used)	0.00 ft	0	0.0
0	Row 13 (not used)	0.00 ft	0	0.0
0	Row 14 (not used)	0.00 ft	0	0.0
0	Row 15 (not used)	0.00 ft	0	0.0
			29	Σ I = 1653.8

Summary Pile Reactions

.oad Combinations		Pile Loads (tons/pile)															Max. Vertical Load (Tons)	Horiz Pile Group Capacity (k)	
	Allowable Pile Loads	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15			
1	59.8 tons	47.0	34.7	22.4	10.0	-2.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	47.0	899	OK
1.1	79.7 tons	48.5	35.4	22.4	9.3	-3.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	48.5	1,015	OK
2	79.7 tons	36.2	28.4	20.6	12.8	5.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	36.2	1,015	OK
2.1	104.0 tons	51.2	35.6	19.9	4.2	-11.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	51.2	1,015	OK
3	79.7 tons	32.4	31.4	30.4	29.4	28.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	32.4	1,015	OK
4	31.8 tons	30.4	30.4	30.4	30.4	30.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	30.4	638	OK
																Max Service : P =	51.2		

Using solid mechanics equations adapted for discrete elements, the forces in the pile rows for different load combinations are determined.
The force in each pile row is found using:

$$\text{Pile Load} = P / N + M_{NA} / I$$

First, the moment about the toe must be translated to get the moment about the neutral axis of the pile group.

$$e_{toe} = M_{toe} / P$$

Then the eccentricity about the neutral axis of the pile group is

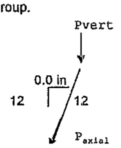
$$e_{NA} = X_{NA} - e_{toe}$$

The moment about the neutral axis of the pile group becomes

$$M_{NA} = P * e_{NA}$$

For battered pile, the Vertical pile load needs to be transformed to the axial load along the pile axis

Paxial = 1.000 Pvert



FORCE RESULTANT (see Stability Analysis)

CASE	Event	Vertical Load P (kips)	Horizontal Load H (kips)	ΣM_{toe} (kip-ft)	$e_{toe} = M_{toe} / P$ (ft)	$e_{NA} = X_{NA} - e_{toe}$ (ft)	$M_{NA} = P * e_{NA}$ (kip-ft)	
1	**100**	1,297	755	8,439	6.51	5.99	7773	Usual
1.1	**100**	1,297	770	7,991	6.16	6.34	8222	Unusual
2	**500**	1,195	384	10,012	8.38	4.12	4926	Unusual
2.1	**500**	1,153	693	4,531	3.93	8.57	9880	Extreme
3	**Dry**	1,764	35	21,421	12.15	0.35	623	Unusual
4	**Daily**	1,764	0	22,044	12.50	0.00	0	Usual

SERVICE

Case **1**
Flood Event **100**
Usual

Vertical Load, P = 1297 kips
Horizontal Load, H = 755 kips
M_{NA} = 7773 kip-ft

29

	Vertical Pile Loading	P / N +	M_{NA}* d / Σ I	= Pile Loads		Axial Pile Load
1	Row 1	44.7	49.4	94.1 kips/pile	47.0 tons/pile	47.0 tons/pile
2	Row 2	44.7	24.7	69.4 kips/pile	34.7 tons/pile	34.7 tons/pile
3	Row 3	44.7	0.0	44.7 kips/pile	22.4 tons/pile	22.4 tons/pile
4	Row 4	44.7	-24.7	20.0 kips/pile	10.0 tons/pile	10.0 tons/pile
5	Row 5	44.7	-49.4	-4.6 kips/pile	-2.3 tons/pile	-2.3 tons/pile
6	Row 6	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	**47.0 tons/pile**	max: **47.0 tons/pile**

Assumed lateral Capacity: 31.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	6	0.0	186	1.000	186 kips	
2	Row 2	0	6	0.0	186	1.000	186 kips	
3	Row 3	0	5	0.0	155	1.000	155 kips	
4	Row 4	0	6	0.0	186	1.000	186 kips	
5	Row 5	0	6	0.0	186	1.000	186 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			29		899		**899 kips**	**OK**

Case **1.1**
Flood Event **100**
Unusual

Vertical Load, P = 1297 kips
Horizontal Load, H = 770 kips
M_{NA} = 8222 kip-ft

29

	Vertical Pile Loading	P / N +	M_{NA}* d / Σ I	= Pile Loads		Axial Pile Load
1	Row 1	44.7	52.2	96.9 kips/pile	48.5 tons/pile	48.5 tons/pile
2	Row 2	44.7	26.1	70.8 kips/pile	35.4 tons/pile	35.4 tons/pile
3	Row 3	44.7	0.0	44.7 kips/pile	22.4 tons/pile	22.4 tons/pile
4	Row 4	44.7	-26.1	18.6 kips/pile	9.3 tons/pile	9.3 tons/pile
5	Row 5	44.7	-52.2	-7.5 kips/pile	-3.7 tons/pile	-3.7 tons/pile
6	Row 6	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile

12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	**48.5 tons/pile**	max: **48.5 tons/pile**

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	6	0.0	210	1.000	210 kips	
2	Row 2	0	6	0.0	210	1.000	210 kips	
3	Row 3	0	5	0.0	175	1.000	175 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			29		1015		**1015 kips**	OK

Case **2**
Flood Event **500**
Unusual

Vertical Load, P = 1195 kips
Horizontal Load, H = 384 kips
M_{NA} = 4926 kip-ft

	Vertical Pile Loading	P / N +	M_{NA} * d / Σ I	= Pile Loads		Axial Pile Load
1	Row 1	41.2	31.3	72.5 kips/pile	36.2 tons/pile	36.2 tons/pile
2	Row 2	41.2	15.6	56.8 kips/pile	28.4 tons/pile	28.4 tons/pile
3	Row 3	41.2	0.0	41.2 kips/pile	20.6 tons/pile	20.6 tons/pile
4	Row 4	41.2	-15.6	25.6 kips/pile	12.8 tons/pile	12.8 tons/pile
5	Row 5	41.2	-31.3	9.9 kips/pile	5.0 tons/pile	5.0 tons/pile
6	Row 6	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	**36.2 tons/pile**	max: **36.2 tons/pile**

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	6	0.0	210	1.000	210 kips	
2	Row 2	0	6	0.0	210	1.000	210 kips	
3	Row 3	0	5	0.0	175	1.000	175 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			29		1015		**1015 kips**	OK

Case **2.1**
Flood Event **500**
Extreme

Vertical Load, P = 1153 kips
Horizontal Load, H = 693 kips
M_{NA} = 9880 kip-ft

	Vertical Pile Loading	P / N +	M_{NA} * d / Σ I	= Pile Loads		Axial Pile Load

1	Row 1	39.8	62.7	102.5 kips/pile	51.2 tons/pile		51.2 tons/pile
2	Row 2	39.8	31.4	71.1 kips/pile	35.6 tons/pile		35.6 tons/pile
3	Row 3	39.8	0.0	39.8 kips/pile	19.9 tons/pile		19.9 tons/pile
4	Row 4	39.8	-31.4	8.4 kips/pile	4.2 tons/pile		4.2 tons/pile
5	Row 5	39.8	-62.7	-23.0 kips/pile	-11.5 tons/pile		-11.5 tons/pile
6	Row 6	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
				max:	**51.2 tons/pile**	max:	**51.2 tons/pile**

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	6	0.0	210	1.000	210 kips	
2	Row 2	0	6	0.0	210	1.000	210 kips	
3	Row 3	0	5	0.0	175	1.000	175 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			29		1015		**1015 kips**	OK

Case 3
Flood Event Dry
Unusual

Vertical Load, P = 1764 kips
Horizontal Load, H = 35 kips
M_{NA} = 623 kip-ft

	Vertical Pile Loading	P / N +	M_{NA} * d / Σ I	= Pile Loads			Axial Pile Load
1	Row 1	60.8	4.0	64.8 kips/pile	32.4 tons/pile		32.4 tons/pile
2	Row 2	60.8	2.0	62.8 kips/pile	31.4 tons/pile		31.4 tons/pile
3	Row 3	60.8	0.0	60.8 kips/pile	30.4 tons/pile		30.4 tons/pile
4	Row 4	60.8	-2.0	58.8 kips/pile	29.4 tons/pile		29.4 tons/pile
5	Row 5	60.8	-4.0	56.9 kips/pile	28.4 tons/pile		28.4 tons/pile
6	Row 6	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
				max:	**32.4 tons/pile**	max:	**32.4 tons/pile**

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance
1	Row 1	0	6	0.0	210	1.000	210 kips
2	Row 2	0	6	0.0	210	1.000	210 kips
3	Row 3	0	5	0.0	175	1.000	175 kips
4	Row 4	0	6	0.0	210	1.000	210 kips
5	Row 5	0	6	0.0	210	1.000	210 kips
6	Row 6	0	0	0.0	0	1.000	0 kips
7	Row 7	0	0	0.0	0	1.000	0 kips
8	Row 8	0	0	0.0	0	1.000	0 kips
9	Row 9	0	0	0.0	0	1.000	0 kips
10	Row 10	0	0	0.0	0	1.000	0 kips
11	Row 11	0	0	0.0	0	1.000	0 kips
12	Row 12	0	0	0.0	0	1.000	0 kips
13	Row 13	0	0	0.0	0	1.000	0 kips
14	Row 14	0	0	0.0	0	1.000	0 kips
15	Row 15	0	0	0.0	0	1.000	0 kips

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
			PROJECT NUMBER		
			SUBJECT	**Pile Capacity** **Control Structure: 35k**	SECTION F

29 1015 **1015 kips** **OK**

Case **4**
Flood Event **Daily**
Usual

Vertical Load, P = 1764 kips
Horizontal Load, H = 0 kips
M_{NA} = 0 kip-ft

	Vertical Pile Loading	P / N +	M_{NA}* d / Σ I	= Pile Loads		**Axial Pile Load**
1	Row 1	60.8	0.0	60.8 kips/pile	30.4 tons/pile	30.4 tons/pile
2	Row 2	60.8	0.0	60.8 kips/pile	30.4 tons/pile	30.4 tons/pile
3	Row 3	60.8	0.0	60.8 kips/pile	30.4 tons/pile	30.4 tons/pile
4	Row 4	60.8	0.0	60.8 kips/pile	30.4 tons/pile	30.4 tons/pile
5	Row 5	60.8	0.0	60.8 kips/pile	30.4 tons/pile	30.4 tons/pile
6	Row 6	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	**30.4 tons/pile**	**max:** **30.4 tons/pile**

Assumed lateral Capacity: 22.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	6	0.0	132	1.000	132 kips	
2	Row 2	0	6	0.0	132	1.000	132 kips	
3	Row 3	0	5	0.0	110	1.000	110 kips	
4	Row 4	0	6	0.0	132	1.000	132 kips	
5	Row 5	0	6	0.0	132	1.000	132 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			29		638		**638 kips**	**OK**

ATTACHMENT

F-Q1.5 Group Analyses Results for the Red River (LPP) Control Structure Wing Wall F

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/4/11		PKN	SUBJECT	**Pile Coordinates** **Control Structure: 35k**	SECTION F

Pile Coordinates

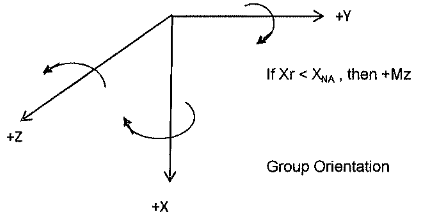
	x (in)	z (in)	Batter
1	24	30.00	
2	24	90.00	
3	24	150.00	
4	24	210.00	
5	24	270.00	
6	24	330.00	
7	87	30.00	
8	87	90.00	
9	87	150.00	
10	87	210.00	
11	87	270.00	
12	87	330.00	
13	150	36.00	
14	150	108.00	
15	150	180.00	
16	150	252.00	
17	150	324.00	
18	213	30.00	
19	213	90.00	
20	213	150.00	
21	213	210.00	
22	213	270.00	
23	213	330.00	
24	276	30.00	
25	276	90.00	
26	276	150.00	
27	276	210.00	
28	276	270.00	
29	276	330.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	
0	0	0.00	

from pile CG

Pile Coordinates

	x (in)	z (in)
1	-10.50	-12.50
2	-10.50	-7.50
3	-10.50	-2.50
4	-10.50	2.50
5	-10.50	7.50
6	-10.50	12.50
7	-5.25	-12.50
8	-5.25	-7.50
9	-5.25	-2.50
10	-5.25	2.50
11	-5.25	7.50
12	-5.25	12.50
13	0.00	-12.00
14	0.00	-6.00
15	0.00	0.00
16	0.00	6.00
17	0.00	12.00
18	5.25	-12.50
19	5.25	-7.50
20	5.25	-2.50
21	5.25	2.50
22	5.25	7.50
23	5.25	12.50
24	10.50	-12.50
25	10.50	-7.50
26	10.50	-2.50
27	10.50	2.50
28	10.50	7.50
29	10.50	12.50
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00

Load 1.1		GROUP INPUT	Load 1.2		GROUP INPUT
Px =	1,297 k	1,297,010 lbs	Px =	1,297 k	1,297,010 lbs
Hy =	-755 k	(755,105) lbs	Hy =	-770 k	(770,105) lbs
ΣMnet =	-8,439 k-ft	lb-in	ΣMnet =	-7,991 k-ft	lb-in
Xr = Yr =	6.51 ft	78.08 in	Xr =	6.16 ft	73.93 in
X_{NA} =	12.50 ft		X_{NA} =	12.50 ft	
e_{NA} =	5.99 ft		e_{NA} =	6.34 ft	
Mz = M_{NA} =	7773 k-ft		M_{NA} =	8222 k-ft	lbs
Zr =	15 ft		Zr =	15 ft	



If Xr < X_{NA} , then +Mz

Group Orientation

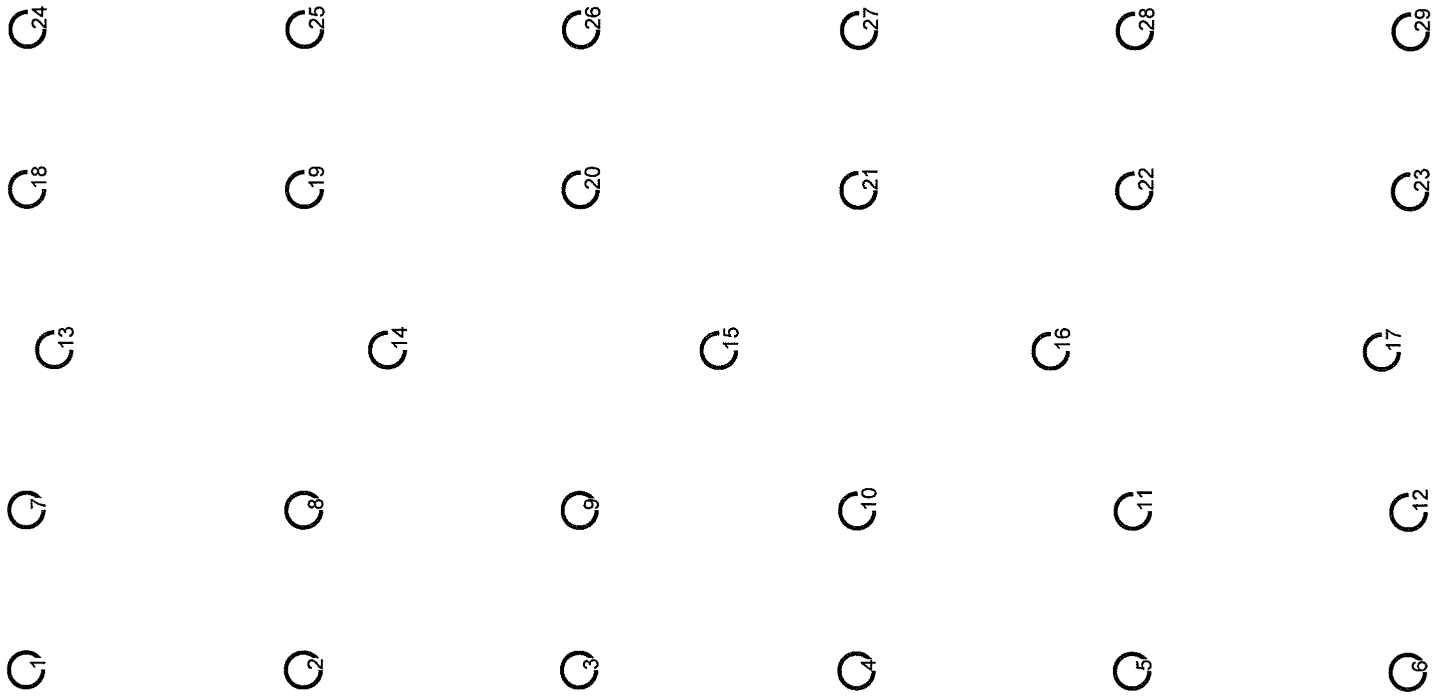
Translated Moment about 0 , 0 Axis (Compare with GROUP Translation)

My = Px * Zr = +233461828.8 lb-in
Mz = Px * Yr = -101271544.9 lb-in

My = Px * Zr = +233461828.8 lb-in
Mz = Px * Yr = -95893144.95 lb-in

Bot FTG EL. 891.63
CONTROL STRUCT " " EL 869.50
22.13 FT

ADD 22' of
BRENNA THICKNESS
COMPARED W/ WING WALL A


24
25
26
27
28
29
18
19
20
21
22
23
13
14
15
16
17
7
8
9
10
11
12
1
2
3
4
5
6

```
--------------------------------------------------------------------------
                    Time and Date of Analysis
--------------------------------------------------------------------------

          Date:  February 16, 2011     Time:  11:52:01



            *****     COMPUTATION RESULTS     *****


RRN Wingwall F



            *****     LOAD CASES RESULTS      *****


LOAD CASE :     1
CASE NAME : Load Case  1
LOAD TYPE : Dead, DL

  REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS, COMBINED Y AND Z DIRECTIONS
  ESTIMATED USING MOVEMENT IN THE DIRECTION OF PILE CAP DISPLACEMENTS

       GROUP NO      P-FACTOR      Y-FACTOR

           1           1.0000        1.0000
           2           1.0000        1.0000
           3           1.0000        1.0000
           4           1.0000        1.0000
           5           1.0000        1.0000
           6           1.0000        1.0000
           7           0.8387        1.0000
           8           0.8237        1.0000
           9           0.8237        1.0000
          10           0.8237        1.0000
          11           0.8237        1.0000
          12           0.8387        1.0000
          13           0.8316        1.0000
          14           0.8272        1.0000
          15           0.8274        1.0000
          16           0.8272        1.0000
          17           0.8316        1.0000
          18           0.8569        1.0000
          19           0.8517        1.0000
          20           0.8574        1.0000
          21           0.8574        1.0000
          22           0.8517        1.0000
          23           0.8569        1.0000
          24           0.8387        1.0000
          25           0.8237        1.0000
          26           0.8237        1.0000
          27           0.8237        1.0000
          28           0.8237        1.0000
          29           0.8387        1.0000


* TABLE L *   COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *                                        100 YR
            1,297 K               755 K
      VERT. LOAD, LBS     HOR. LOAD Y, LBS    HOR. LOAD Z, LBS        LC  1
        1.29701E+06 ✓      -7.55105E+05          0.00000

     MOMENT X ,IN-LBS     MOMENT Y,IN-LBS     MOMENT Z,IN-LBS
        1.35919E+08         2.33462E+08 ✓      -1.01271E+08 ✓

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *
```

```
RRN Wingwall F
GROUP8.0.4
* Analysis Parameters
1 3 0 0 0 0 0 1 0 1 0 0
* Pile Cap Dimensions
0.0000000E+00 0.0000000E+00 0.0000000E+00
* Load Cases
1 0 0 1
0 1 0 1 1 100 100
0.0001 0.0001 1 1
Load Case  1
0 0 0 0 1 0 0
2 1 1 0
1
1297010 -755105 0 0 0 0 0 78.08 180
0
0
* Load Combinations
0 0 0 1
* Distributed Load Sets
0
* Pile Group Configuration
29
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 36 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 108 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 180 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 252 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 324 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 213 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
```

```
-12 213 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 213 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 213 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 213 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 213 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 330 0 90 -36
* Pile Properties
1
0 1
100 900 29000000 1
0 900 1 0 0
* Pile Cross Sections
1
0 0 4
HP14x89
13.83 0 14.585 13.83 0 0 0.615 0.615
1 26.1 904 326 1808 2.09728E10 0 0 0 0
* Soil Layers
3
0 1
0 590
2 10
0.0235 4.514 0 0 0 0.01 0 0 0 0
0.0235 4.514 0 0 0 0.01 0 0 0 0
1 3
590 650
2 10
0.03449 13.194 0 0 0 0.007 0 0 0 0
0.03449 13.194 0 0 0 0.007 0 0 0 0
2 3
650 1200
2 10
0.03449 13.194 0 0 0 0.007 0 0 0 0
0.03449 13.194 0 0 0 0.007 0 0 0 0
* End of file
```

$\gamma_b=40.61$

$c=650$ PSF

} BRENNA

```
  VERTICAL ,IN       HORIZONTAL Y,IN     HORIZONTAL Z,IN
     0.13173             -0.42851        -3.21107E-11

 ANGLE ROT. X,RAD    ANGLE ROT. Y,RAD    ANGLE ROT. Z,RAD
    2.17671E-13        2.19705E-17         5.73412E-04


* TABLE M *   COMPUTATION ON INDIVIDUAL PILE

  THE GLOBAL STRUCTURAL COORDINATE SYSTEM
  ---------------------------------------

  * PILE TOP DISPLACEMENTS *

 PILE GROUP   DISP. X,IN   DISP. Y,IN   DISP. Z,IN   ROT. X,RAD   ROT. Y,RAD   ROT. Z,RAD
 **********  ***********  ***********  ***********  ***********  ***********  ***********
      1           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
      2           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
      3           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
      4           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
      5           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
      6           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
      7       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
      8       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
      9       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
     10       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
     11       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
     12       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
     13       4.5718E-02      -0.4354   5.4017E-13   2.1767E-13   2.1970E-17   5.7341E-04
     14       4.5718E-02      -0.4354   5.4017E-13   2.1767E-13   2.1970E-17   5.7341E-04
     15       4.5718E-02      -0.4354   5.4017E-13   2.1767E-13   2.1970E-17   5.7341E-04
     16       4.5718E-02      -0.4354   5.4017E-13   2.1767E-13   2.1970E-17   5.7341E-04
     17       4.5718E-02      -0.4354   5.4017E-13   2.1767E-13   2.1970E-17   5.7341E-04
     18       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
     19       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
     20       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
     21       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
     22       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
     23       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
     24      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
     25      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
     26      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
     27      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
     28      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
     29      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04

  MINIMUM    -2.6532E-02      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
  Pile N.         24             1           1            1            1            1
  MAXIMUM         0.1180      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
  Pile N.          1             1          24            1            1            1
```

EQUAL LOCAL AXIS FOR VERT. PILES

.4354" < 0.67"

AXIAL

HORIZ.

28.61 K

152.60 K-FT

```
  * PILE TOP REACTIONS *

 PILE GROUP   FOR. X,LBS   FOR. Y,LBS   FOR. Z,LBS MOM X,LBS-IN MOM Y,LBS-IN MOM Z,LBS-IN  STRESS,LBS/IN**2
 **********  ***********  ***********  *********** ***********  ***********  ***********  ****************
      1       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06    1.8270E+04
      2       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06    1.8270E+04
      3       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06    1.8270E+04
      4       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06    1.8270E+04
      5       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06    1.8270E+04
      6       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06    1.8270E+04
      7       8.1402E+04  -2.5323E+04  -5.5921E-07   3.5978E-12   3.1944E-05  -1.6757E+06    1.5937E+04
      8       8.1402E+04  -2.4997E+04  -5.5278E-07   3.5978E-12   3.1683E-05  -1.6602E+06    1.5818E+04
      9       8.1402E+04  -2.4997E+04  -5.5278E-07   3.5978E-12   3.1683E-05  -1.6602E+06    1.5818E+04
     10       8.1402E+04  -2.4997E+04  -5.5278E-07   3.5978E-12   3.1683E-05  -1.6602E+06    1.5818E+04
     11       8.1402E+04  -2.4997E+04  -5.5278E-07   3.5978E-12   3.1683E-05  -1.6602E+06    1.5818E+04
     12       8.1402E+04  -2.5323E+04  -5.5921E-07   3.5978E-12   3.1944E-05  -1.6757E+06    1.5937E+04
     13       5.1568E+04  -2.5228E+04   2.2853E-08   3.5978E-12  -1.3000E-06  -1.6677E+06    1.4732E+04
     14       5.1568E+04  -2.5134E+04   2.2768E-08   3.5978E-12  -1.2968E-06  -1.6634E+06    1.4700E+04
     15       5.1568E+04  -2.5139E+04   2.2773E-08   3.5978E-12  -1.2970E-06  -1.6637E+06    1.4702E+04
     16       5.1568E+04  -2.5134E+04   2.2768E-08   3.5978E-12  -1.2968E-06  -1.6634E+06    1.4700E+04
     17       5.1568E+04  -2.5228E+04   2.2853E-08   3.5978E-12  -1.3000E-06  -1.6677E+06    1.4732E+04
     18       1.0840E+04  -2.5854E+04   6.1971E-07   3.5978E-12  -3.4909E-05  -1.6930E+06    1.3366E+04
     19       1.0840E+04  -2.5745E+04   6.1724E-07   3.5978E-12  -3.4813E-05  -1.6879E+06    1.3327E+04
     20       1.0840E+04  -2.5863E+04   6.1992E-07   3.5978E-12  -3.4917E-05  -1.6934E+06    1.3369E+04
     21       1.0840E+04  -2.5863E+04   6.1992E-07   3.5978E-12  -3.4917E-05  -1.6934E+06    1.3369E+04
     22       1.0840E+04  -2.5745E+04   6.1724E-07   3.5978E-12  -3.4813E-05  -1.6879E+06    1.3327E+04
```

TONS 55.62

40.7

25.78

5.42

AXIA LOADS TONS

	Row 1	Row 2	Row 3	Row 4	Row 5	
RIGID CAP	47.0	34.7	22.4	10.0	-2.3	CASE 1.1
GROUP	55.62	40.7	25.78	5.42	-15.14	

```
         23       1.0840E+04  -2.5854E+04   6.1971E-07   3.5978E-12  -3.4909E-05  -1.6930E+06   1.3366E+04
         24      -3.0283E+04  -2.5550E+04   1.2060E-06   3.5978E-12  -6.7856E-05  -1.6743E+06   1.3968E+04
         25      -3.0283E+04  -2.5230E+04   1.1922E-06   3.5978E-12  -6.7306E-05  -1.6595E+06   1.3855E+04
         26      -3.0283E+04  -2.5230E+04   1.1922E-06   3.5978E-12  -6.7306E-05  -1.6595E+06   1.3855E+04
-15.14   27      -3.0283E+04  -2.5230E+04   1.1922E-06   3.5978E-12  -6.7306E-05  -1.6595E+06   1.3855E+04
         28      -3.0283E+04  -2.5230E+04   1.1922E-06   3.5978E-12  -6.7306E-05  -1.6595E+06   1.3855E+04
         29      -3.0283E+04  -2.5550E+04   1.2060E-06   3.5978E-12  -6.7856E-05  -1.6743E+06   1.3968E+04

       MINIMUM   -3.0283E+04  -2.8611E+04  -1.2801E-06   3.5978E-12  -6.7856E-05  -1.8312E+06   1.3327E+04
       Pile N.       24            1            1            1           24            1            19
       MAXIMUM    1.1124E+05  -2.4997E+04   1.2060E-06   3.5978E-12   7.0633E-05  -1.6595E+06   1.8270E+04
       Pile N.        1            8           24            1            1           25             1


       THE PILE COORDINATE SYSTEM (LOCAL AXES)
       ---------------------------------------

       * PILE TOP DISPLACEMENTS *

      PILE GROUP   DISP. x,IN   DISP. y,IN   DISP. z,IN   ROT. x,RAD   ROT. y,RAD   ROT. z,RAD
      **********  ***********  ***********  ***********  ***********  ***********  ***********
          1           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
          2           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
          3           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
          4           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
          5           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
          6           0.1180      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
          7       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
          8       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
          9       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
         10       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
         11       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
         12       8.1843E-02      -0.4354  -1.3173E-11   2.1767E-13   2.1970E-17   5.7341E-04
         13       4.5718E-02      -0.4354   5.4017E-13   2.1767E-13   2.1970E-17   5.7341E-04
         14       4.5718E-02      -0.4354   5.4017E-13   2.1767E-13   2.1970E-17   5.7341E-04
         15       4.5718E-02      -0.4354   5.4017E-13   2.1767E-13   2.1970E-17   5.7341E-04
         16       4.5718E-02      -0.4354   5.4017E-13   2.1767E-13   2.1970E-17   5.7341E-04
         17       4.5718E-02      -0.4354   5.4017E-13   2.1767E-13   2.1970E-17   5.7341E-04
         18       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
         19       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
         20       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
         21       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
         22       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
         23       9.5931E-03      -0.4354   1.4253E-11   2.1767E-13   2.1970E-17   5.7341E-04
         24      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
         25      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
         26      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
         27      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
         28      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
         29      -2.6532E-02      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04

       MINIMUM   -2.6532E-02      -0.4354  -2.6886E-11   2.1767E-13   2.1970E-17   5.7341E-04
       Pile N.       24            1            1            1            1            1
       MAXIMUM        0.1180      -0.4354   2.7967E-11   2.1767E-13   2.1970E-17   5.7341E-04
       Pile N.        1            1           24            1            1            1

       * PILE TOP REACTIONS *

      PILE GROUP    AXIAL,LBS    LAT. y,LBS   LAT. z,LBS MOM x,LBS-IN MOM y,LBS-IN MOM z,LBS-IN   STRESS,LBS/IN**2
      **********  ***********  ***********  ***********  ***********  ***********  ***********   ****************
          1       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06   1.8270E+04
          2       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06   1.8270E+04
          3       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06   1.8270E+04
          4       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06   1.8270E+04
          5       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06   1.8270E+04
          6       1.1124E+05  -2.8611E+04  -1.2801E-06   3.5978E-12   7.0633E-05  -1.8312E+06   1.8270E+04
          7       8.1402E+04  -2.5323E+04  -5.5921E-07   3.5978E-12   3.1944E-05  -1.6757E+06   1.5937E+04
          8       8.1402E+04  -2.4997E+04  -5.5278E-07   3.5978E-12   3.1683E-05  -1.6602E+06   1.5818E+04
          9       8.1402E+04  -2.4997E+04  -5.5278E-07   3.5978E-12   3.1683E-05  -1.6602E+06   1.5818E+04
         10       8.1402E+04  -2.4997E+04  -5.5278E-07   3.5978E-12   3.1683E-05  -1.6602E+06   1.5818E+04
         11       8.1402E+04  -2.4997E+04  -5.5278E-07   3.5978E-12   3.1683E-05  -1.6602E+06   1.5818E+04
         12       8.1402E+04  -2.5323E+04  -5.5921E-07   3.5978E-12   3.1944E-05  -1.6757E+06   1.5937E+04
         13       5.1568E+04  -2.5228E+04   2.2853E-08   3.5978E-12  -1.3000E-06  -1.6677E+06   1.4732E+04
         14       5.1568E+04  -2.5134E+04   2.2768E-08   3.5978E-12  -1.2968E-06  -1.6634E+06   1.4700E+04
         15       5.1568E+04  -2.5139E+04   2.2773E-08   3.5978E-12  -1.2970E-06  -1.6637E+06   1.4702E+04
         16       5.1568E+04  -2.5134E+04   2.2768E-08   3.5978E-12  -1.2968E-06  -1.6634E+06   1.4700E+04
         17       5.1568E+04  -2.5228E+04   2.2853E-08   3.5978E-12  -1.3000E-06  -1.6677E+06   1.4732E+04
         18       1.0840E+04  -2.5854E+04   6.1971E-07   3.5978E-12  -3.4909E-05  -1.6930E+06   1.3366E+04
         19       1.0840E+04  -2.5745E+04   6.1724E-07   3.5978E-12  -3.4813E-05  -1.6879E+06   1.3327E+04
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         20         1.0840E+04  -2.5863E+04   6.1992E-07   3.5978E-12  -3.4917E-05  -1.6934E+06   1.3369E+04
         21         1.0840E+04  -2.5863E+04   6.1992E-07   3.5978E-12  -3.4917E-05  -1.6934E+06   1.3369E+04
         22         1.0840E+04  -2.5745E+04   6.1724E-07   3.5978E-12  -3.4813E-05  -1.6879E+06   1.3327E+04
         23         1.0840E+04  -2.5854E+04   6.1971E-07   3.5978E-12  -3.4909E-05  -1.6930E+06   1.3366E+04
         24        -3.0283E+04  -2.5550E+04   1.2060E-06   3.5978E-12  -6.7856E-05  -1.6743E+06   1.3968E+04
         25        -3.0283E+04  -2.5230E+04   1.1922E-06   3.5978E-12  -6.7306E-05  -1.6595E+06   1.3855E+04
         26        -3.0283E+04  -2.5230E+04   1.1922E-06   3.5978E-12  -6.7306E-05  -1.6595E+06   1.3855E+04
         27        -3.0283E+04  -2.5230E+04   1.1922E-06   3.5978E-12  -6.7306E-05  -1.6595E+06   1.3855E+04
         28        -3.0283E+04  -2.5230E+04   1.1922E-06   3.5978E-12  -6.7306E-05  -1.6595E+06   1.3855E+04
         29        -3.0283E+04  -2.5550E+04   1.2060E-06   3.5978E-12  -6.7856E-05  -1.6743E+06   1.3968E+04

      MINIMUM      -3.0283E+04  -2.8611E+04  -1.2801E-06   3.5978E-12  -6.7856E-05  -1.8312E+06   1.3327E+04
      Pile N.          24            1            1            1           24            1            19
      MAXIMUM       1.1124E+05  -2.4997E+04   1.2060E-06   3.5978E-12   7.0633E-05  -1.6595E+06   1.8270E+04
      Pile N.           1            8           24            1            1           25             1

      * EFFECTS FOR LATERALLY LOADED PILE *

      * MINIMUM VALUES AND LOCATIONS *

     PILE         DEFLECTION           BENDING MOMENT          SHEAR FORCE          SOIL REACTION
TOTAL      FLEXURAL RIGIDITY
             y-DIR       z-DIR       z-DIR       y-DIR       y-DIR       z-DIR       y-DIR       z-DIR
STRESS       z-DIR       y-DIR
               IN          IN       LBS-IN      LBS-IN        LBS         LBS       LBS/IN      LBS/IN
LBS/IN**2   LBS-IN**2   LBS-IN**2
     *****  **********  **********  **********  **********  **********  **********  **********  **********
**********  **********  **********
        1      -0.4350 -2.6900E-11 -6.3000E+05 -2.0300E-05 -2.8600E+04 -1.2800E-06     -222.00 -1.2100E-08
4260.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      153.00      135.00      0.0000      0.0000      99.000      54.000
531.00      0.0000      0.0000
        2      -0.4350 -2.6900E-11 -6.3000E+05 -2.0300E-05 -2.8600E+04 -1.2800E-06     -222.00 -1.2100E-08
4260.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      153.00      135.00      0.0000      0.0000      99.000      54.000
522.00      0.0000      0.0000
        3      -0.4350 -2.6900E-11 -6.3000E+05 -2.0300E-05 -2.8600E+04 -1.2800E-06     -222.00 -1.2100E-08
4260.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      153.00      135.00      0.0000      0.0000      99.000      54.000
522.00      0.0000      0.0000
        4      -0.4350 -2.6900E-11 -6.3000E+05 -2.0300E-05 -2.8600E+04 -1.2800E-06     -222.00 -1.2100E-08
4260.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      153.00      135.00      0.0000      0.0000      99.000      54.000
531.00      0.0000      0.0000
        5      -0.4350 -2.6900E-11 -6.3000E+05 -2.0300E-05 -2.8600E+04 -1.2800E-06     -222.00 -1.2100E-08
4260.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      153.00      135.00      0.0000      0.0000      99.000      54.000
522.00      0.0000      0.0000
        6      -0.4350 -2.6900E-11 -6.3000E+05 -2.0300E-05 -2.8600E+04 -1.2800E-06     -222.00 -1.2100E-08
4260.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      153.00      135.00      0.0000      0.0000      99.000      54.000
531.00      0.0000      0.0000
        7      -0.4350 -1.3200E-11 -5.7800E+05 -9.1600E-06 -2.5300E+04 -5.6000E-07     -190.00 -5.0700E-09
3120.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      162.00      135.00      0.0000      0.0000      108.00      63.000
549.00      0.0000      0.0000
        8      -0.4350 -1.3200E-11 -5.7300E+05 -9.1200E-06 -2.5000E+04 -5.5400E-07     -187.00 -4.9900E-09
3120.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      162.00      135.00      0.0000      0.0000      108.00      63.000
558.00      0.0000      0.0000
        9      -0.4350 -1.3200E-11 -5.7300E+05 -9.1200E-06 -2.5000E+04 -5.5400E-07     -187.00 -4.9900E-09
3120.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      162.00      135.00      0.0000      0.0000      108.00      63.000
558.00      0.0000      0.0000
       10      -0.4350 -1.3200E-11 -5.7300E+05 -9.1200E-06 -2.5000E+04 -5.5400E-07     -187.00 -4.9900E-09
3120.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      162.00      135.00      0.0000      0.0000      108.00      63.000
558.00      0.0000      0.0000
       11      -0.4350 -1.3200E-11 -5.7300E+05 -9.1200E-06 -2.5000E+04 -5.5400E-07     -187.00 -4.9900E-09
3120.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      162.00      135.00      0.0000      0.0000      108.00      63.000
558.00      0.0000      0.0000
       12      -0.4350 -1.3200E-11 -5.7800E+05 -9.1600E-06 -2.5300E+04 -5.6000E-07     -190.00 -5.0700E-09
3120.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      162.00      135.00      0.0000      0.0000      108.00      63.000
549.00      0.0000      0.0000
       13      -0.4350  1.0000E-14 -5.7400E+05 -1.3000E-06 -2.5200E+04 -4.7400E-09     -189.00 -3.7200E-10
1980.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      198.00      162.00      135.00      0.0000      216.00      108.00      234.00
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549.00      0.0000      0.0000
       14       -0.4350  1.0000E-14 -5.7300E+05 -1.3000E-06 -2.5100E+04 -4.6700E-09     -188.00 -3.4000E-10
1980.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      198.00      162.00      135.00      0.0000      216.00      108.00      234.00
558.00      0.0000      0.0000
       15       -0.4350  1.0000E-14 -5.7300E+05 -1.3000E-06 -2.5200E+04 -4.6700E-09     -188.00 -3.3900E-10
1980.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      198.00      162.00      135.00      0.0000      216.00      108.00      234.00
558.00      0.0000      0.0000
       16       -0.4350  1.0000E-14 -5.7300E+05 -1.3000E-06 -2.5100E+04 -4.6700E-09     -188.00 -3.4000E-10
1980.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      198.00      162.00      135.00      0.0000      216.00      108.00      234.00
558.00      0.0000      0.0000
       17       -0.4350  1.0000E-14 -5.7400E+05 -1.3000E-06 -2.5200E+04 -4.7400E-09     -189.00 -3.7200E-10
1980.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      198.00      162.00      135.00      0.0000      216.00      108.00      234.00
549.00      0.0000      0.0000
       18       -0.4350 -9.8700E-14 -5.8200E+05 -3.4900E-05 -2.5900E+04 -1.1800E-07     -194.00 -5.0600E-09
415.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      162.00      135.00      0.0000      207.00      108.00      234.00
558.00      0.0000      0.0000
       19       -0.4350 -9.4600E-14 -5.8000E+05 -3.4800E-05 -2.5700E+04 -1.1900E-07     -193.00 -5.4200E-09
415.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      162.00      135.00      0.0000      207.00      108.00      234.00
549.00      0.0000      0.0000
       20       -0.4350 -9.9100E-14 -5.8200E+05 -3.4900E-05 -2.5900E+04 -1.1800E-07     -194.00 -5.0200E-09
415.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      162.00      135.00      0.0000      207.00      108.00      234.00
558.00      0.0000      0.0000
       21       -0.4350 -9.9100E-14 -5.8200E+05 -3.4900E-05 -2.5900E+04 -1.1800E-07     -194.00 -5.0200E-09
415.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      162.00      135.00      0.0000      207.00      108.00      234.00
558.00      0.0000      0.0000
       22       -0.4350 -9.4600E-14 -5.8000E+05 -3.4800E-05 -2.5700E+04 -1.1900E-07     -193.00 -5.4200E-09
415.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      162.00      135.00      0.0000      207.00      108.00      234.00
549.00      0.0000      0.0000
       23       -0.4350 -9.8700E-14 -5.8200E+05 -3.4900E-05 -2.5900E+04 -1.1800E-07     -194.00 -5.0600E-09
415.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      162.00      135.00      0.0000      207.00      108.00      234.00
558.00      0.0000      0.0000
       24       -0.4350 -1.2300E-13 -5.7400E+05 -6.7900E-05 -2.5600E+04 -2.3700E-07     -190.00 -1.6200E-08
1160.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      162.00      135.00      9.0000      216.00      108.00      234.00
558.00      0.0000      0.0000
       25       -0.4350 -5.7000E-14 -5.6900E+05 -6.7300E-05 -2.5200E+04 -2.5200E-07     -187.00 -2.3200E-08
1160.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      261.00      162.00      135.00      9.0000      225.00      108.00      234.00
558.00      0.0000      0.0000
       26       -0.4350 -5.7000E-14 -5.6900E+05 -6.7300E-05 -2.5200E+04 -2.5200E-07     -187.00 -2.3200E-08
1160.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      261.00      162.00      135.00      9.0000      225.00      108.00      234.00
558.00      0.0000      0.0000
       27       -0.4350 -5.7000E-14 -5.6900E+05 -6.7300E-05 -2.5200E+04 -2.5200E-07     -187.00 -2.3200E-08
1160.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      261.00      162.00      135.00      9.0000      225.00      108.00      234.00
558.00      0.0000      0.0000
       28       -0.4350 -5.7000E-14 -5.6900E+05 -6.7300E-05 -2.5200E+04 -2.5200E-07     -187.00 -2.3200E-08
1160.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      261.00      162.00      135.00      9.0000      225.00      108.00      234.00
558.00      0.0000      0.0000
       29       -0.4350 -1.2300E-13 -5.7400E+05 -6.7900E-05 -2.5600E+04 -2.3700E-07     -190.00 -1.6200E-08
1160.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      162.00      135.00      9.0000      216.00      108.00      234.00
558.00      0.0000      0.0000

      Min.      -0.4350 -2.6900E-11 -6.3000E+05 -6.7900E-05 -2.8600E+04 -1.2800E-06     -222.00 -2.3200E-08
415.00  2.6200E+10  9.4500E+09
   Pile N.        1           1           1          24           1           1           1          25
18          1           1
                                                  M3
       * MAXIMUM VALUES AND LOCATIONS *

     PILE         DEFLECTION            BENDING MOMENT          SHEAR FORCE           SOIL REACTION
TOTAL       FLEXURAL RIGIDITY
                y-DIR       z-DIR       z-DIR       y-DIR       y-DIR       z-DIR       y-DIR       z-DIR
STRESS        z-DIR       y-DIR
                 IN          IN        LBS-IN      LBS-IN        LBS         LBS       LBS/IN      LBS/IN
LBS/IN**2   LBS-IN**2   LBS-IN**2
```

Mz

Row 1

```
     *****  **********  **********  **********  **********  **********  **********  **********  **********
**********  **********  **********
         1     -0.4350  2.0300E-13  1.8300E+06  7.0600E-05      6860.0  2.5500E-07      71.300  1.0200E-08
1.8300E+04  2.6200E+10  9.4500E+09
     x(IN)      261.00      225.00      0.0000      0.0000      225.00      198.00      261.00      225.00
0.0000      0.0000      0.0000
         2     -0.4350  2.0300E-13  1.8300E+06  7.0600E-05      6860.0  2.5500E-07      71.300  1.0200E-08
1.8300E+04  2.6200E+10  9.4500E+09
     x(IN)      261.00      225.00      0.0000      0.0000      225.00      198.00      261.00      225.00
0.0000      0.0000      0.0000
         3     -0.4350  2.0300E-13  1.8300E+06  7.0600E-05      6860.0  2.5500E-07      71.300  1.0200E-08
1.8300E+04  2.6200E+10  9.4500E+09
     x(IN)      261.00      225.00      0.0000      0.0000      225.00      198.00      261.00      225.00
0.0000      0.0000      0.0000
         4     -0.4350  2.0300E-13  1.8300E+06  7.0600E-05      6860.0  2.5500E-07      71.300  1.0200E-08
1.8300E+04  2.6200E+10  9.4500E+09
     x(IN)      261.00      225.00      0.0000      0.0000      225.00      198.00      261.00      225.00
0.0000      0.0000      0.0000
         5     -0.4350  2.0300E-13  1.8300E+06  7.0600E-05      6860.0  2.5500E-07      71.300  1.0200E-08
1.8300E+04  2.6200E+10  9.4500E+09
     x(IN)      261.00      225.00      0.0000      0.0000      225.00      198.00      261.00      225.00
0.0000      0.0000      0.0000
         6     -0.4350  2.0300E-13  1.8300E+06  7.0600E-05      6860.0  2.5500E-07      71.300  1.0200E-08
1.8300E+04  2.6200E+10  9.4500E+09
     x(IN)      261.00      225.00      0.0000      0.0000      225.00      198.00      261.00      225.00
0.0000      0.0000      0.0000
```

Row 2

```
         7     -0.4350  8.7400E-14  1.6800E+06  3.1900E-05      6020.0  1.1100E-07      59.700  5.3000E-09
1.5900E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      0.0000      0.0000      234.00      207.00      270.00      234.00
0.0000      0.0000      0.0000
         8     -0.4350  4.3500E-14  1.6600E+06  3.1700E-05      5940.0  1.1700E-07      58.400  9.4100E-09
1.5800E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      234.00      216.00      279.00      234.00
0.0000      0.0000      0.0000
         9     -0.4350  4.3500E-14  1.6600E+06  3.1700E-05      5940.0  1.1700E-07      58.400  9.4100E-09
1.5800E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      234.00      216.00      279.00      234.00
0.0000      0.0000      0.0000
        10     -0.4350  4.3500E-14  1.6600E+06  3.1700E-05      5940.0  1.1700E-07      58.400  9.4100E-09
1.5800E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      234.00      216.00      279.00      234.00
0.0000      0.0000      0.0000
        11     -0.4350  4.3500E-14  1.6600E+06  3.1700E-05      5940.0  1.1700E-07      58.400  9.4100E-09
1.5800E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      0.0000      0.0000      234.00      216.00      279.00      234.00
0.0000      0.0000      0.0000
        12     -0.4350  8.7400E-14  1.6800E+06  3.1900E-05      6020.0  1.1100E-07      59.700  5.3000E-09
1.5900E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      234.00      0.0000      0.0000      234.00      207.00      270.00      234.00
0.0000      0.0000      0.0000
```

Row 3

```
        13     -0.4350  5.4000E-13  1.6700E+06  3.7400E-07      5950.0  2.2900E-08      58.600  2.0600E-10
1.4700E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      270.00      63.000
0.0000      0.0000      0.0000
        14     -0.4350  5.4000E-13  1.6600E+06  3.7300E-07      5940.0  2.2800E-08      58.500  2.0500E-10
1.4700E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      270.00      63.000
0.0000      0.0000      0.0000
        15     -0.4350  5.4000E-13  1.6600E+06  3.7300E-07      5940.0  2.2800E-08      58.500  2.0500E-10
1.4700E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      270.00      63.000
0.0000      0.0000      0.0000
        16     -0.4350  5.4000E-13  1.6600E+06  3.7300E-07      5940.0  2.2800E-08      58.500  2.0500E-10
1.4700E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      270.00      63.000
0.0000      0.0000      0.0000
        17     -0.4350  5.4000E-13  1.6700E+06  3.7400E-07      5950.0  2.2900E-08      58.600  2.0600E-10
1.4700E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      270.00      63.000
0.0000      0.0000      0.0000
```

Row 4

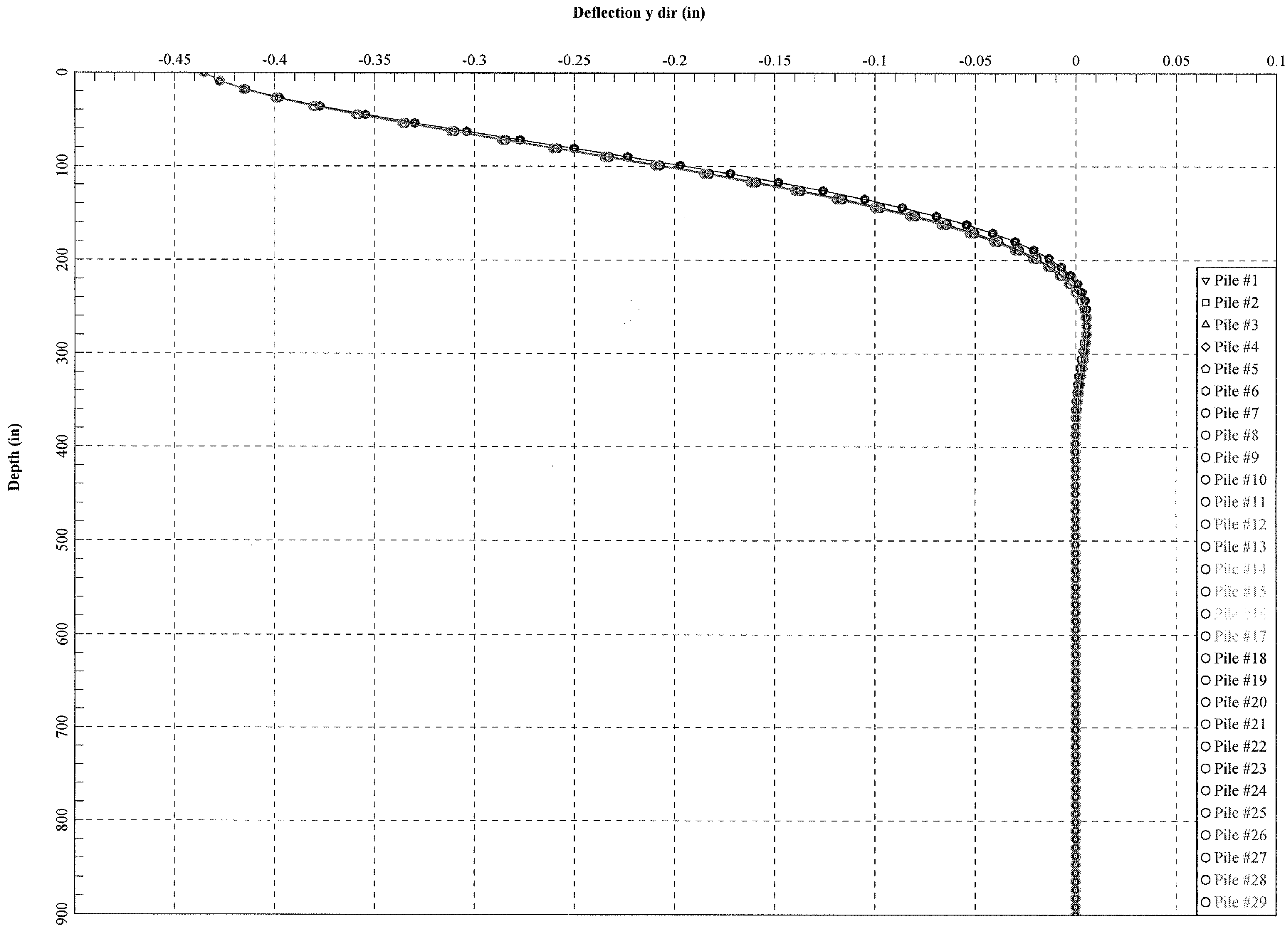
```
        18     -0.4350  1.4300E-11  1.6900E+06  9.9300E-06      6050.0  6.2000E-07      60.600  5.5900E-09
1.3400E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      270.00      63.000
0.0000      0.0000      0.0000
        19     -0.4350  1.4300E-11  1.6900E+06  9.9000E-06      6030.0  6.1700E-07      60.200  5.5600E-09
1.3300E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      270.00      63.000
0.0000      0.0000      0.0000
        20     -0.4350  1.4300E-11  1.6900E+06  9.9300E-06      6060.0  6.2000E-07      60.700  5.6000E-09
```

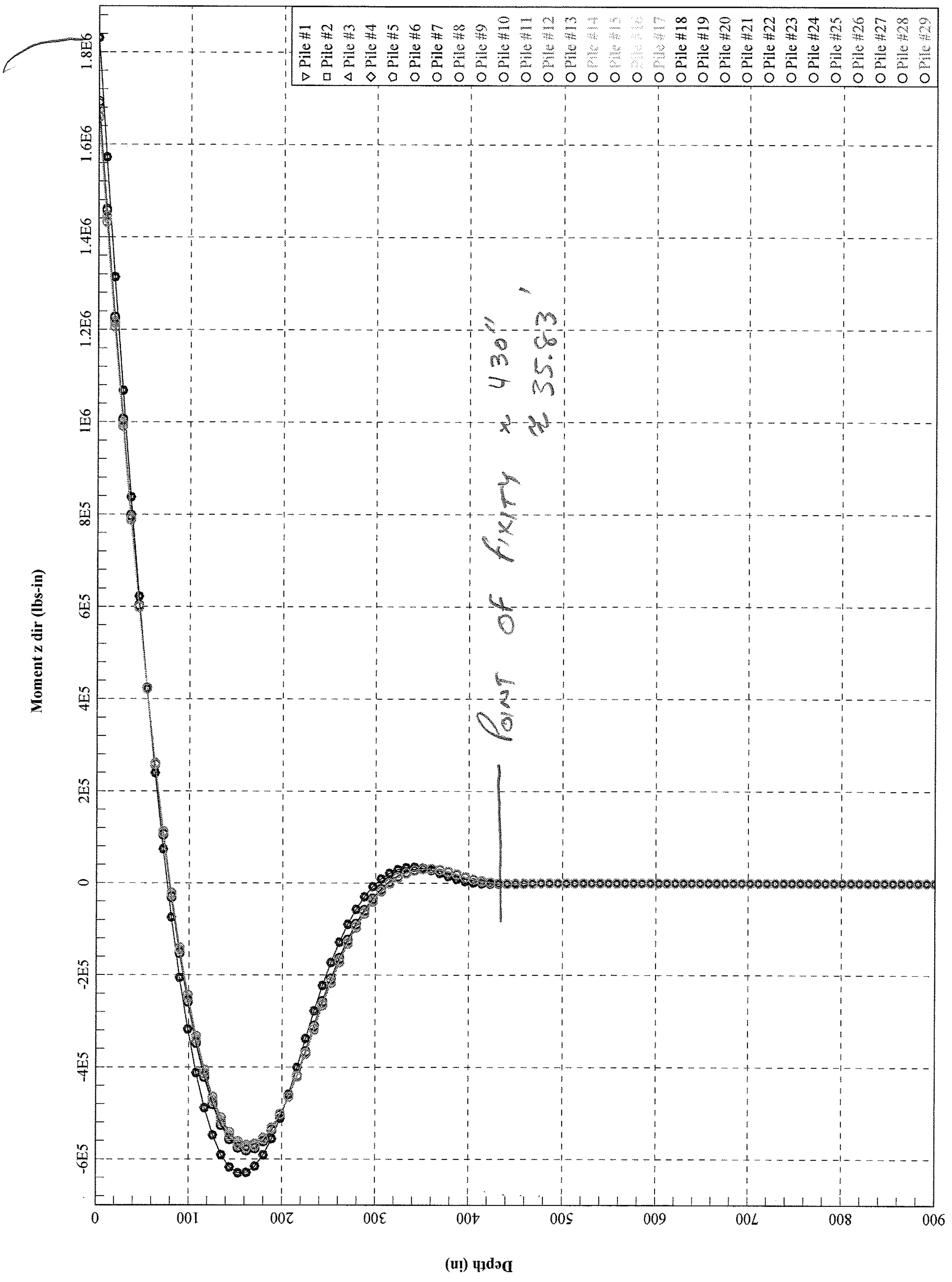
MZ

Row 5

```
1.3400E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      270.00      63.000
0.0000      0.0000      0.0000
        21     -0.4350  1.4300E-11  1.6900E+06  9.9300E-06      6060.0  6.2000E-07      60.700  5.6000E-09
1.3400E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      270.00      63.000
0.0000      0.0000      0.0000
        22     -0.4350  1.4300E-11  1.6900E+06  9.9000E-06      6030.0  6.1700E-07      60.200  5.5600E-09
1.3300E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      270.00      63.000
0.0000      0.0000      0.0000
        23     -0.4350  1.4300E-11  1.6900E+06  9.9300E-06      6050.0  6.2000E-07      60.600  5.5900E-09
1.3400E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      270.00      63.000
0.0000      0.0000      0.0000
        24     -0.4350  2.8000E-11  1.6700E+06  1.9300E-05      5930.0  1.2100E-06      58.800  1.0800E-08
1.4000E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      9.0000      270.00      63.000
0.0000      0.0000      0.0000
        25     -0.4350  2.8000E-11  1.6600E+06  1.9200E-05      5860.0  1.1900E-06      58.200  1.0600E-08
1.3900E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      9.0000      279.00      63.000
0.0000      0.0000      0.0000
        26     -0.4350  2.8000E-11  1.6600E+06  1.9200E-05      5860.0  1.1900E-06      58.200  1.0600E-08
1.3900E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      9.0000      279.00      63.000
0.0000      0.0000      0.0000
        27     -0.4350  2.8000E-11  1.6600E+06  1.9200E-05      5860.0  1.1900E-06      58.200  1.0600E-08
1.3900E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      9.0000      279.00      63.000
0.0000      0.0000      0.0000
        28     -0.4350  2.8000E-11  1.6600E+06  1.9200E-05      5860.0  1.1900E-06      58.200  1.0600E-08
1.3900E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      9.0000      279.00      63.000
0.0000      0.0000      0.0000
        29     -0.4350  2.8000E-11  1.6700E+06  1.9300E-05      5930.0  1.2100E-06      58.800  1.0800E-08
1.4000E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      9.0000      270.00      63.000
0.0000      0.0000      0.0000

      Max.     -0.4350  2.8000E-11  1.8300E+06  7.0600E-05      6860.0  1.2100E-06      71.300  1.0800E-08
1.8300E+04  2.6200E+10  9.4500E+09
   Pile N.           1          24           1           1           1          24           1          24
1           1           1
```


Deflection y dir (in)
-0.45
-0.4
-0.35
-0.3
-0.25
-0.2
-0.15
-0.1
-0.05
0
0.05
0.1
Depth (in)
0
100
200
300
400
500
600
700
800
900
▽ Pile #1
□ Pile #2
△ Pile #3
◇ Pile #4
⬠ Pile #5
⬡ Pile #6
○ Pile #7
○ Pile #8
○ Pile #9
○ Pile #10
○ Pile #11
○ Pile #12
○ Pile #13
○ Pile #14
○ Pile #15
○ Pile #16
○ Pile #17
○ Pile #18
○ Pile #19
○ Pile #20
○ Pile #21
○ Pile #22
○ Pile #23
○ Pile #24
○ Pile #25
○ Pile #26
○ Pile #27
○ Pile #28
○ Pile #29



1.83×10^6

Point of fixity ≈ 430"
≅ 35.83'

ATTACHMENT

F-Q2.1 Structural Pile Capacity Results

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTE	PROJECT NUMBER	
PKN		PKN	SUBJECT EM 1110-2-2906	
2/9/11			HP 14X73 Axial Capacity compression	

EM 1110-2-2906 Design of Pile Foundations 1/15/1991

(1) Steel Piles. Allowable tension and compression stresses are given for both the lower and upper regions of the pile. Since the lower region of the pile is subject to damage during driving, the basic allowa stress should reflect a high factor of safety. The distribution of allowable tension or compression stress along the length of the pile is shown in Figure 4-1. This factor of safety may be decreased if more is known about the actual driving conditions. Pile shoes should be used when driving in dense sand strata, gravel strata, cobble-boulder zones, and when driving piles to refusal on a hard layer of bedrock. Bending effects are usually minimal in the lower region of the pile. The upper region of the pile may be subject to the effects of bending and buckling as well as axial load. Since damage in the upper region usually apparent during driving, a higher allowable stress is permitted. The upper region of the pile is actually designed as a beam-column, with due consideration to lateral support conditions. The allowable stresses for fully supported piles are as follows:

STEEL PROPERTIES:

Fy= **50** ksi
E= **29000** ksi

BEAM PROPERTIES:

Beam Type: **HP** W or HP
depth= **14** nominal depth code: 14073
wt/ft = **73**
member HP 14X73

Ag=	21.4 in^2	T =	11.22 in
d=	13.6 in	Ix=	729 in^4
tw=	0.505 in	Sx=	107 in^3
bf=	14.6 in	rx=	5.84 in
tf=	0.505 in	Iy=	261 in^4
rt=	3.9	Sy=	35.8 in^3
d/Af=	1.85	ry=	3.49 in

			Usual	Unusual	Extreme	
Load Type :	**Usual**	enter Usual, Unusual or Extreme	**Usual**	**Unusual**	**Extreme**	
allow. stress increase	1		1	1.33	1.75	allowable stress increase EM 1110-2-2906, 4-1

HP 14x73 ✱

120 TON

Mx ALLOW = 122.9 K-ft

Loading:

Mx= **122.92** k-ft
My= **0.00** k-ft
V = **0.00** k
Pa= **240.00** k

Lb= **1.00** ft
Cb = **1.00**

Cb = 1.75 + 1.05 (M1/M2) + 0.3 (M1/M2) <= 2.3
M1/M2 is ratio of smaller to larger moments at ends of unbraced member.
M1/M2 is positive when bent in reverse curvature.
If Bending moment within an unbraced length is larger than at both ends than Cb = 1.

Kx= **1.00**
Ky= **1.00**
Lx= **1.00** ft
Ly= **1.00** ft

M1= **0.00**
M2= **1.00**
Cb = 1.75

Tension or Compression in Lower region

Fa = 1/3 * Fy * 5/6 =	13.89 ksi	Concentric axial tension or compression
Fa = 1/3 * Fy =	16.67 ksi	Concentric axial tension or compression only with driving shoe
Fa = 1 / 2.5 * Fy =	20.00 ksi	Concentric axial tension or compression only with driving shoe, at least one axial load test and use of a pile driving analyzer to verify the pile capacity and integrity
Select Fa =	13.89 ksi	Select from above values depending on driving shoe & testing
stress increase * Fa =	13.89 ksi	Select from above values depending on driving shoe & testing
fa = Pa / Ag =	11.21 ksi	**OK.....Fa > fa**

fa / Fa = 0.81 **Lower Region OK** **Pmax = Fa*A = 297.2 k**

Combined bending and axial compression in Upper Region

$fa/Fa \pm fbx / Fb \pm fby / Fb \leq 1.0$ allow. stress increase 1

			w/o Stress Increase	
allowable axial stress				
Fa = 5/6 * 3/5 Fy =	25.00 ksi			25 ksi
allowable bending stress				
Fb = 5/6 * 3/5 Fy =	25.00 ksi	for noncompact sections		25 ksi
Fb = 5/6 * 2/3 Fy =	27.78 ksi	for compact sections		27.777778 ksi

Bending Calculations: AISC Chapter F

AISC TABLE B5.1			Compact	Noncompact	
bf/2*tf=	14.46	noncompact	9.19	13.44	
d/tw=	26.93	ok	90.51	-	webs in flexural compression

Slender - Check AISC Appendix B5

Fbx = 25.00 ksi includes allowable stress increase **Fby = 25.00 ksi** includes allowable stress increase

fbx= Mx/Sx = 13.79 ksi OK **fby= My/Sy = 0.00 ksi OK**

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN		PKN	SUBJECT EM 1110-2-2906	
2/9/11			HP 14X73 Axial Capacity compression	

fbx / Fbx = 0.551 **fby / Fby =** 0.000

Axial Load Calculation: Upper Pile Region

Fa= **25.000 ksi** includes allowable stress increase

fa = Pa/A = 11.21 ksi

Pmax = Fa*A = 535.0 k

fa/Fa = 0.45 ksi

Combined Bending and Axial Compression in Upper Pile Region

$fa/Fa \pm fbx / Fb \pm fby / Fb \leq 1.0$

fa / Fa = 0.45
fbx / Fbx = 0.551
fby / Fby = 0.000
1.00 OK

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/9/11		PKN	SUBJECT EM 1110-2-2906 HP 14X89 Axial Capacity compression	

EM 1110-2-2906 Design of Pile Foundations 1/15/1991

(1) Steel Piles. Allowable tension and compression stresses are given for both the lower and upper regions of the pile. Since the lower region of the pile is subject to damage during driving, the basic allowa stress should reflect a high factor of safety. The distribution of allowable tension or compression stress along the length of the pile is shown in Figure 4-1. This factor of safety may be decreased if more is known about the actual driving conditions. Pile shoes should be used when driving in dense sand strata, gravel strata, cobble-boulder zones, and when driving piles to refusal on a hard layer of bedrock. Bending effects are usually minimal in the lower region of the pile. The upper region of the pile may be subject to the effects of bending and buckling as well as axial load. Since damage in the upper region usually apparent during driving, a higher allowable stress is permitted. The upper region of the pile is actually designed as a beam-column, with due consideration to lateral support conditions. The allowable stresses for fully supported piles are as follows:

STEEL PROPERTIES:

Fy= **50** ksi
E= **29000** ksi

BEAM PROPERTIES:

Beam Type: HP W or HP
depth= **14** nominal depth code: 14089
wt/ft = **89**
member HP 14X89

Ag=	26.1 in^2	T =	11.18 in
d=	13.8 in	Ix=	904 in^4
tw=	0.615 in	Sx=	131 in^3
bf=	14.7 in	rx=	5.88 in
tf=	0.615 in	Iy=	326 in^4
rt=	3.94	Sy=	44.3 in^3
d/Af=	1.53	ry=	3.53 in

			Usual	Unusual	Extreme	
Load Type :	**Usual**	enter Usual, Unusual or Extreme				
allow. stress increase	1		1	1.33	1.75	allowable stress increase EM 1110-2-2906, 4-1

Loading:

Mx= **172.53** k-ft
My= **0.00** k-ft
V = **0.00** k
Pa= **240.00** k

Cb = 1.75 + 1.05 (M1/M2) + 0.3 (M1/M2) <= 2.3
M1/M2 is ratio of smaller to larger moments at ends of unbraced member.
M1/M2 is positive when bent in reverse curvature.
If Bending moment within an unbraced length is larger than at both ends than Cb = 1.

Lb= **1.00** ft
Cb = **1.00**

Kx= **1.00**
Ky= **1.00**
Lx= **1.00** ft
Ly= **1.00** ft

M1= **0.00**
M2= **1.00**
Cb = 1.75

HP 14x89 ✗

P_y = 120 TON

M_x = 172.5 K-ft

Tension or Compression in Lower region

Fa = 1/3 * Fy * 5/6 =	13.89 ksi	Concentric axial tension or compression
Fa = 1/3 * Fy =	16.67 ksi	Concentric axial tension or compression only with driving shoe
Fa = 1 / 2.5 * Fy =	20.00 ksi	Concentric axial tension or compression only with driving shoe, at least one axial load test and use of a pile driving analyzer to verify the pile capacity and integrity

Select Fa = **13.89** ksi Select from above values depending on driving shoe & testing

stress increase * Fa = **13.89** ksi Select from above values depending on driving shoe & testing

fa = Pa / Ag = 9.20 ksi **OK.....Fa > fa**

fa / Fa = 0.66 Lower Region OK Pmax = Fa*A = 362.5 k

Combined bending and axial compression in Upper Region

fa/Fa ± fbx / Fb ± fby / Fb ≤ 1.0 allow. stress increase 1

allowable axial stress				
Fa = 5/6 * 3/5 Fy =	25.00 ksi		w/o Stress Increase	25 ksi
allowable bending stress				
Fb = 5/6 * 3/5 Fy =	25.00 ksi	for noncompact sections		25 ksi
Fb = 5/6 * 2/3 Fy =	27.78 ksi	for compact sections		27.777778 ksi

Bending Calculations: AISC Chapter F

AISC TABLE B5.1		Compact	Noncompact	
bf/2*tf=	11.95 noncompact	9.19	13.44	
d/tw=	22.44 ok	90.51	-	webs in flexural compression

Noncompact Section

Fbx = 25.00 ksi includes allowable stress increase **Fby = 25.00 ksi** includes allowable stress increase

fbx= Mx/Sx = 15.80 ksi OK fby= My/Sy = 0.00 ksi OK

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PKN 2/9/11		PKN	SUBJECT EM 1110-2-2906 HP 14X89 Axial Capacity compression	

fbx / Fbx = 0.632 **fby / Fby = 0.000**

Axial Load Calculation: Upper Pile Region

Fa= 25.000 ksi includes allowable stress increase

fa = Pa/A = 9.20 ksi

Pmax = Fa*A = 652.5 k

fa/Fa = 0.37 ksi

Combined Bending and Axial Compression in Upper Pile Region

fa/Fa $\pm$ fbx / Fb $\pm$ fby / Fb $\leq$ 1.0

fa / Fa =	0.37
fbx / Fbx =	0.632
fby / Fby =	0.000
	1.00 OK

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/9/11		PKN	SUBJECT EM 1110-2-2906 HP 14X89 Axial Capacity compression	

EM 1110-2-2906 Design of Pile Foundations 1/15/1991

(1) Steel Piles. Allowable tension and compression stresses are given for both the lower and upper regions of the pile. Since the lower region of the pile is subject to damage during driving, the basic allowa stress should reflect a high factor of safety. The distribution of allowable tension or compression stress along the length of the pile is shown in Figure 4-1. This factor of safety may be decreased if more is known about the actual driving conditions. Pile shoes should be used when driving in dense sand strata, gravel strata, cobble-boulder zones, and when driving piles to refusal on a hard layer of bedrock. Bending effects are usually minimal in the lower region of the pile. The upper region of the pile may be subject to the effects of bending and buckling as well as axial load. Since damage in the upper region usually apparent during driving, a higher allowable stress is permitted. The upper region of the pile is actually designed as a beam-column, with due consideration to lateral support conditions. The allowable stresses for fully supported piles are as follows:

ARN WINGWALL A

STEEL PROPERTIES:

Fy= 50 ksi
E= 29000 ksi

BEAM PROPERTIES:

Beam Type: HP W or HP
depth= 14 nominal depth code: 14089
wt/ft = 89

member HP 14X89

Ag=	26.1	in^2	T =	11.18	in
d=	13.8	in	Ix=	904	in^4
tw=	0.615	in	Sx=	131	in^3
bf=	14.7	in	rx=	5.88	in
tf=	0.615	in	Iy=	326	in^4
rt=	3.94		Sy=	44.3	in^3
d/Af=	1.53		ry=	3.53	in

			Usual	Unusual	Extreme	
Load Type :	Usual	enter Usual, Unusual or Extreme				
allow. stress increase	1		1	1.33	1.75	allowable stress increase EM 1110-2-2906, 4-1

Loading:

Mx= **193.08** k-ft
My= **0.00** k-ft
V = **0.00** k
Pa= **123.20** k

PILE 1 LOADING FROM GROUP

61.58 TON

Lb= **1.00** ft
Cb = **1.00**

Kx= **1.00**
Ky= **1.00**
Lx= **1.00** ft
Ly= **1.00** ft

Cb = 1.75 + 1.05 (M1/M2) + 0.3 (M1/M2) <= 2.3
M1/M2 is ratio of smaller to larger moments at ends of unbraced member.
M1/M2 is positive when bent in reverse curvature.
If Bending moment within an unbraced length is larger than at both ends than Cb = 1.

M1= **0.00**
M2= **1.00**
Cb = 1.75

Tension or Compression in Lower region

Fa = 1/3 * Fy * 5/6 =	13.89	ksi	Concentric axial tension or compression
Fa = 1/3 * Fy =	16.67	ksi	Concentric axial tension or compression only with driving shoe
Fa = 1 / 2.5 * Fy =	20.00	ksi	Concentric axial tension or compression only with driving shoe, at least one axial load test and use of a pile driving analyzer to verify the pile capacity and integrity
Select Fa =	**13.89**	ksi	Select from above values depending on driving shoe & testing
stress increase * Fa =	**13.89**	ksi	Select from above values depending on driving shoe & testing
fa = Pa / Ag =	4.72	ksi	**OK.....Fa > fa**

fa / Fa = 0.34 **Lower Region OK** **Pmax = Fa*A = 362.5 k**

Combined bending and axial compression in Upper Region

fa/Fa ± fbx / Fb ± fby / Fb ≤ 1.0 allow. stress increase 1

				w/o Stress Increase	
allowable axial stress					
Fa = 5/6 * 3/5 Fy =	25.00	ksi		25	ksi
allowable bending stress					
Fb = 5/6 * 3/5 Fy =	25.00	ksi	for noncompact sections	25	ksi
Fb = 5/6 * 2/3 Fy =	27.78	ksi	for compact sections	27.777778	ksi

Bending Calculations: AISC Chapter F

AISC TABLE B5.1			Compact	Noncompact	
bf/2*tf=	11.95	noncompact	9.19	13.44	
d/tw=	22.44	ok	90.51	-	webs in flexural compression

Noncompact Section

Fbx = 25.00 ksi includes allowable stress increase **Fby = 25.00 ksi** includes allowable stress increase

fbx= Mx/Sx = 17.69 ksi OK **fby= My/Sy = 0.00 ksi** OK

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fbx / Fbx = 0.707 **fby / Fby = 0.000**

Axial Load Calculation: Upper Pile Region

Fa= 25.000 ksi includes allowable stress increase

fa = Pa/A = 4.72 ksi

Pmax = Fa*A = 652.5 k

fa/Fa = 0.19 ksi

Combined Bending and Axial Compression in Upper Pile Region

fa/Fa ± fbx / Fb ± fby / Fb ≤ 1.0

fa / Fa = 0.19
fbx / Fbx = 0.707
fby / Fby = 0.000
0.90 OK

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/16/11		PKN	SUBJECT EM 1110-2-2906 HP 14X89 Axial Capacity compression RRN Wing Wall F	

EM 1110-2-2906 Design of Pile Foundations 1/15/1991

(1) Steel Piles. Allowable tension and compression stresses are given for both the lower and upper regions of the pile. Since the lower region of the pile is subject to damage during driving, the basic allowa stress should reflect a high factor of safety. The distribution of allowable tension or compression stress along the length of the pile is shown in Figure 4-1. This factor of safety may be decreased if more is known about the actual driving conditions. Pile shoes should be used when driving in dense sand strata, gravel strata, cobble-boulder zones, and when driving piles to refusal on a hard layer of bedrock. Bending effects are usually minimal in the lower region of the pile. The upper region of the pile may be subject to the effects of bending and buckling as well as axial load. Since damage in the upper region usually apparent during driving, a higher allowable stress is permitted. The upper region of the pile is actually designed as a beam-column, with due consideration to lateral support conditions. The allowable stresses for fully supported piles are as follows:

STEEL PROPERTIES:

Fy= 50 ksi
E= 29000 ksi

BEAM PROPERTIES:

Beam Type: **HP** W or HP
depth= **14** nominal depth code: 14089
wt/ft = **89**
member HP 14X89

Ag=	26.1	in^2	T =	11.18	in
d=	13.8	in	Ix=	904	in^4
tw=	0.615	in	Sx=	131	in^3
bf=	14.7	in	rx=	5.88	in
tf=	0.615	in	Iy=	326	in^4
rt=	3.94		Sy=	44.3	in^3
d/Af=	1.53		ry=	3.53	in

GROUP OUT PUT
FOR ROW 1

P = 55.62 TON
Mx = 152.6 K-ft

			Usual	Unusual	Extreme	
Load Type :	**Usual**	enter Usual, Unusual or Extreme	**Usual**	**Unusual**	**Extreme**	
allow. stress increase	1		1	1.33	1.75	allowable stress increase EM 1110-2-2906, 4-1

Loading:

Mx= **189.26** k-ft
My= **0.00** k-ft
V = **0.00** k
Pa= **200.00** k = 100 TON

FOR 100 TON PILE MX MAX = 189.26 K-ft

Cb = 1.75 + 1.05 (M1/M2) + 0.3 (M1/M2) <= 2.3

Lb= **1.00** ft M1/M2 is ratio of smaller to larger moments at ends of unbraced member.
Cb = **1.00** M1/M2 is positive when bent in reverse curvature.
If Bending moment within an unbraced length is larger than at both ends than Cb = 1.
Kx= **1.00**
Ky= **1.00** M1= **0.00**
Lx= **1.00** ft M2= **1.00**
Ly= **1.00** ft Cb = 1.75

Tension or Compression in Lower region

Fa = 1/3 * Fy * 5/6 =	13.89	ksi	Concentric axial tension or compression
Fa = 1/3 * Fy =	16.67	ksi	Concentric axial tension or compression only with driving shoe
Fa = 1 / 2.5 * Fy =	20.00	ksi	Concentric axial tension or compression only with driving shoe, at least one axial load test and use of a pile driving analyzer to verify the pile capacity and integrity

Select Fa = 13.89 ksi Select from above values depending on driving shoe & testing

stress increase * Fa = 13.89 ksi Select from above values depending on driving shoe & testing

fa = Pa / Ag = 7.66 ksi **OK.....Fa > fa**

fa / Fa = 0.55 Lower Region OK Pmax = Fa*A = 362.5 k

Combined bending and axial compression in Upper Region

fa/Fa ± fbx / Fb ± fby / Fb ≤ 1.0 allow. stress increase 1

allowable axial stress

Fa = 5/6 * 3/5 Fy = 25.00 ksi 25 ksi

allowable bending stress w/o Stress Increase

Fb = 5/6 * 3/5 Fy = 25.00 ksi for noncompact sections 25 ksi
Fb = 5/6 * 2/3 Fy = 27.78 ksi for compact sections 27.777778 ksi

Bending Calculations: AISC Chapter F

AISC TABLE B5.1			Compact	Noncompact	
bf/2*tf=	11.95	noncompact	9.19	13.44	
d/tw=	22.44	ok	90.51	-	webs in flexural compression

Noncompact Section

Fbx = 25.00 ksi includes allowable stress increase **Fby = 25.00 ksi** includes allowable stress increase

fbx= Mx/Sx = 17.34 ksi OK fby= My/Sy = 0.00 ksi OK

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PKN 2/16/11		PKN	SUBJECT EM 1110-2-2906 HP 14X89 Axial Capacity compression RRN Wing Wall F	

fbx / Fbx = 0.693 **fby / Fby =** 0.000

Axial Load Calculation: Upper Pile Region

Fa= **25.000 ksi** includes allowable stress increase

fa = Pa/A = **7.66** ksi

Pmax = Fa*A = **652.5 k**

fa/Fa = **0.31** ksi

Combined Bending and Axial Compression in Upper Pile Region

$fa/Fa \pm fbx / Fb \pm fby / Fb \leq 1.0$

fa / Fa = 0.31

fbx / Fbx = 0.693

fby / Fby = 0.000

1.00 OK

ATTACHMENT

F-Q3.1 Ice Loads on Piers

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/2/11		PKN	SUBJECT **ICE LOADS on Piers**	

AASHTO 3.9 Ice Loads

RRN & WILD RICE

3.9.2 Dynamic Ice Forces on Piers

3.9.2.2 Crushing and Flexing

if $w/t \leq 6.0$, then

F = lesser of either Fc or, when ice failure by flexure is considered applicable as described herein, Fb,
and

if $w/t > 6.0$

$F = F_c$

$F_c = C_a\, p\, t\, w$

$F_b = C_n\, p\, t^2$

$C_a = (5\, t/w + 1)^{0.5}$

$C_n = 0.5 / [\tan(\alpha - 15)]$

where:

t = thickness of ice (ft)
α = inclination of the nose to the vertical (deg)
p = effective ice crushing stength as specified in Article 3.9.2.1 (ksf)
w = pier width at level of ice action (ft)
Fc = horizontal ice force caused by ice floes that fail by crushing over the full width of the pier (kip)
Fb = horizontal ice force caused by ice floes that fail by flexure as they ride up the inclined pier nose (kip)
Ca = coefficient accounting for the effect of the pier width/ice thickness ratio where the floe fails by crushing
Cn = coefficient accounting for the inclination of the pier nose with respect to a vertical

where $\alpha \leq 15$ deg, ice failure by flexure shall not be considered to be a possible ice failure mode for the purpose of caculating the horizontal force, F, in which case F shall be taken as Fc

t = **1** ft
α = **90** deg
p = **56** psi = **8.064 ksf**
PIER WIDTH – w = **8** ft
t/w = 0.125 w/t = 8 > 6 Use Fc
Ca = 1.274755
Cn = 0.133975
Fc = 82.24 k
Fb = 1.08 k **F = 82.24 k**

3.9.2.4 Combination of Longitudinal and Transverse Forces (Piers Parallel to Flow)

$F_t = F / [2 \tan(\beta/2 + \theta_f)]$

β = **100** deg, nose angle in horizontal plane for a round nose taken as 100 deg
θ_f = **10** deg, friction angle between ice and pier nose, deg

Ft = 23.74 k

		↓ F (k)	→ Ft (k)
A)	A longitudinal force equal to F shall be combined with the transverse force of 0.15F, or	**82.24**	**12.34**
B)	A longitudinal force equal to 0.5F shall be combined with the transverse force of Ft	**41.12**	**23.74**

Both the longitudinal and transverse forces shall be assumed to act at the pier nose

3.9.4 Hanging Dams and Ice Jams

The frazil accumulation in a hanging dam may be taken to exert a pressure of 0.2 to 2.0 ksf as it moves by the pier.
An Ice jam may be taken to exert a pressure of 0.02 to 0.20 ksf.

RED RIVER DIVERSION

FARGO–MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT R – STRUCTURAL DESIGN COMPUTATIONS – AQUEDUCTS

Report for the US Army Corps of Engineers, and the Cities of Fargo, ND and Moorhead, MN

By: Barr Engineering Co.

FINAL – February 28, 2011

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F-R4.4 Sheyenne River Aqueduct Structure Drawings

F-R5.1 Sheyenne River Aqueduct Structure Pile Computations
F-R5.2 Sheyenne River Aqueduct Structure Wall Panel E Computations
F-R5.3 Maple River Aqueduct Structure Drawings

ELECTRONIC ATTACHMENTS ONLY

The complete set of structural computations for all load conditions for the Sheyenne River aqueduct structure and Maple River aqueduct structure is included electronically on the attached DVD.

F-R6.1 Sheyenne River Aqueduct Structure – Low Flow Channel Walls Computations
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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT R – STRUCTURAL DESIGN COMPUTATIONS - AQUEDUCTS

F-R1.0 BASIS OF DESIGN

Structural design of the aqueducts at, and the diversion channel crossings of the Sheyenne and Maple Rivers, will be based on the following assumptions.

F-R1.1 APPLICABLE CODES, STANDARDS AND GUIDELINES

United State Army Corps of Engineers (USACE) Engineer Manuals:

- EM 1110-2-1612 “Ice Engineering”
- EM 1110-2-2100 “Stability Analysis of Concrete Structures”
- EM 1110-2-2104 “Structural Design for Reinforced-Concrete Hydraulic Structures”
- EM 1110-2-2502 “Retaining and Flood Walls”
- EM 1110-2-2504 “Design of Sheet Pile Walls”
- EM 1110-2-2906 “Design of Pile Foundations”

American Concrete Institute (ACI)

- ACI 301 Standard Specification for Structural Concrete
- ACI 302.1R Guide for Concrete Floor & Slab Construction
- ACI 318 Building Code Requirements for Structural Concrete
- ACI 350R, Environmental Engineering Concrete Structures
- ACI 350.3/350.3R, Seismic Design of Liquid-Containing Concrete Structures
- ACI 360R, Design of Slabs on Grade

American Society of Civil Engineers (ASCE)

- ASCE 7-05 Minimum Design Loads for Buildings and Other Structures

F-R1.2 DESIGN LOADS

F-R1.2.1 <u>Loading Conditions</u>

According to the discussions with USACE, the following seven loading cases were evaluated to generate the design conditions for the different structures and their components:

- Case 1 – Usual Loading Condition – 100-Year Flood
- Case 2 – Unusual Loading Condition – 100-Year Flood + Ice Loading
- Case 3 – Unusual Loading Condition – 500-Year Flood
- Case 4 – Extreme Loading Condition – Water Level is at the top of the structures
- Case 5 – Usual Loading Condition – Normal Flow + Ice
- Case 6 – Unusual Loading Condition – End of Construction condition

F-R1.2.2 Load Combinations

Based on EM1110-2-2104 "Structural Design for Reinforced-Concrete Hydraulic Structures," the following load combinations are assumed for the concrete design. Load combinations are increased by the hydraulic factor Hf = 1.3 except for members in direct tension.

Factored Load Combinations are listed below where:

D: Dead Loads
L: Live Loads
F1: Hydrostatic Load (Water level to the top of the structure on the Diversion Channel side only)
F2: Hydrostatic Load (Water level to the top of the structure inside the Tributary Channel only)
F3: Hydrostatic Load (Water level at the 100 year event + ice on both the Tributary Channel and Diversion Channel)
F4: Hydrostatic Load (Water level to the top of the Low Flow Channel walls + ice loading applied at the top of the Low Flow Channel walls)
H: Earth Load

- LOAD COMBINATION #24 1.3(1.4D+1.7L)
- LOAD COMBINATION #27 1.3(1.4D+1.7L+1.7F1)
- LOAD COMBINATION #32 1.3(1.4D+1.7L+1.7F2)
- LOAD COMBINATION #33 1.3(1.4D+1.7L+1.7F3)
- LOAD COMBINATION #34 1.3(1.4D+1.7L+1.7F4)
- LOAD COMBINATION #37 1.3(1.4D+1.7L+1.7F1&F2)
- LOAD COMBINATION #42 1.3(1.4D+1.7L+1.7H)
- LOAD COMBINATION #45 1.3(1.4D+1.7L+1.7F1+1.7H)
- LOAD COMBINATION #47 1.3(1.4D+1.7L+1.7F2+1.7H)
- LOAD COMBINATION #50 1.3(1.4D+1.7L+1.7F3+1.7H)
- LOAD COMBINATION #53 1.3(1.4D+1.7L+1.7F4+1.7H)
- LOAD COMBINATION #55 1.3(1.4D+1.7L+1.7 F1&F2+1.7H)

F-R1.2.3 Loads

Dead Loads: Selfweight of the concrete structure is assumed as dead loads by assuming normal weight concrete density of 150 lbs/ft^3.

Live Loads: HS-20 loading is assumed on the access bridge. HS-20 load is assumed to be 640 psf for a 12-ft wide strip. HS-20 loading governs over any other live loads that can be subjected on the access bridge slab.

Wind Loads: No wind loads are assumed since the hydrostatic load on the walls are significantly larger than the wind loads.

Seismic Loads: The peak ground acceleration with 2% probability of exceedance in 50 years for the project site is between 0.02g and 0.08g according to the 2003 National Seismic Hazard Maps provided by the United States Geological Survey. Any anticipated earthquake damage in this zone is considered to be minor and is not expected to have an influence on the future stability of the project structures. In accordance with Chapter III of the FERC *Engineering Guidelines for Evaluation of Hydropower Projects* seismic loading is not considered when the peak ground accelerations for the site are less than 0.1g at the site. The mapped spectral accelerations based on USGS Seismic Maps for Latitude = 46° 43’ 59.78” (46.733271°) and Longitude = -96° 48’ 22.95” (-96.806374°) are as follows:

- Short Period, S_s = 0.078g
- 1-second Period, S_1 = 0.022g

Hydrostatic Loads: Internal and external fluid loading is determined by the specific gravity of the water of 62.4 pcf. The tributary channel walls are designed for maximum water levels at the top of the walls assuming no fluid pressure on the opposite side of the wall. Different uplift pressures are assumed underneath the mat foundation depending on the load conditions as shown on Table F-R1 and F-R2.

Hydrostatic fluid load behind the approach and wing wall is neglected during the design. Weep holes and drainage system are assumed to be installed behind these retaining walls to avoid any accumulation of water.

Ice Loading: Ice loading of 10,000 pounds per linear foot (plf) for an ice thickness of 2-feet applied to the contact surface of the structure is assumed in the tributary channel for normal flow case. Ice load is applied at El. 903.24. As a secondary protection, de-icers will be installed inside the tributary walls to avoid ice building. Impact structures will be installed in the diversion channel to avoid any big ice particles hitting the structure.

Recent review of the ice conditions during the 2009 flood in the Fargo/Moorhead area work by ice engineering expert Andrew Tuthill of the US Army Cold Regions Research and Engineering Laboratory (CRREL), revealed that the static ice loads assumed during 100 and 500 year events were unrealistic and conservative. Mr. Tuthill provided recommended ice thicknesses and effective ice crushing strengths to be used for ice floes acting on piers. Calculations for the dynamic ice force follow the methods outlined in AASHTO LRFD Bridge Design Specifications.

Ice loads on the structure in the diversion channel during the 100 year flood will be considered as dynamic forces due to crushing or bending of ice floes as provided by Andrew Tuthill from the USACE via email to Miguel Wong dated February 1, 2011 at 9:52 a.m. These loads will be applied to the piers.

Earth Loading: The below grade walls and retaining walls around the aqueducts is designed using the maximum exterior soil loads and the minimum internal loads. No surcharge assumed.

Lateral Soil Pressures:

- Friction Angle, ϕ = 30
- Active Pressure Coefficient, Ka = 0.33 (Rankine)
- Moist Soil Unit Weight, ϑs = 120 pcf
- Saturated Soil Unit Weight, ϑs = 125 pcf
- Moist Soil Active Pressure, Pa = Ka ϑs Z = 0.27 x 125 = 34 pcf
- Saturated Soil =(ϑs Z – u) Ka + u = (125–62.4) 0.33 = 19 pcf

F-R1.3 DESIGN APPROACH AND ASSUMPTIONS

Finite Element Analysis (FEA) computer software is used to analyze the portion of the aqueduct structures. One FEA model is generated using STAAD Pro V8i structural analysis program (STAAD) since both Sheyenne and Maple structures are very similar as far as the geometry. The model included the maintenance access bridge, tributary channel slab and wall, diversion channel mat foundation and walls supporting the tributary channel. 90-feet section of the structure is modeled. Diversion channel mat foundation is assumed to be supported on H-piles. Walls supporting the Tributary channel are extended all the way to the top of the Tributary channel wall elevation on both upstream and downstream sides to act as pilasters and provide strength to the Tributary channel walls. These walls/pilasters will also act as impact resisting elements to protect the Tributary channel walls for any ice or debris during high flows. These walls/pilasters are also acting as beams and reducing the stresses on the Tributary channel walls. Concrete walls and slabs are modeled by using quadrilateral plate elements. Different thicknesses are assigned to the plates. STAAD uses the stiffness matrix method for the beam elements and Mindlin – Reissner's thick plate theory and finite elements for plates and shell elements. STAAD uses the stiffness matrix to distribute the loads between beams, columns and plates.

Models created in STAAD were used to generate the maximum envelope shear (V_u), moment (M_u), and axial (P_u) forces on the structural elements. Shear (ϕV_n) and moment (ϕM_n) capacities of the beams and slabs are calculated based on ACI 318 using Microsoft Excel and MathCAD spreadsheets.

Lateral loads such as such as wind and seismic forces were not included in the analysis as indicated above.

1) Geometry
    a) Aqueduct structure, approach and wing walls are supported by concrete foundations on H-piles.
    b) Bridge is assumed 15 ft wide to allow maintenance vehicle access
    c) Top of the concrete walls is assumed to be same at the same elevations as the surrounding levees.
    d) Tributary channel width is assumed to be 50 ft.

e) Diversion Channel width is assumed to be 250 ft with 6 bays at 30-ft on center and two bays at both ends are 35 ft.
f) Approach walls are assumed to be curved with radius of 205 ft.
g) Wing walls are assumed to be straight and extending 350 ft beyond the downstream face of the Diversion channel.
h) Stem of the approach and wing walls varies from 14 to 33 feet with 4 ft thick.

2) Reinforcing Steel
Concrete reinforcing was designed preliminarily. The quantity of reinforcement was based on the following:

i. Footings (pile cap)
    1. #9 @ 6" Top & Bottom Transversely
    2. #9 @ 6" Top & Bottom Longitudinally
ii. Piers
    1. #9 @ 6" Ea. Way & Ea. Face
iii. Walls
    1. #9 @ 6" Ea. Way & Ea. Face or #9 @ 12" at lower heights.
iv. Elevated Slabs / Deck
    1. #9 @ 6" Top & Bottom Transversely
    2. #9 @ 6" Top & Bottom Longitudinally

3) Piles – Undrained strength pile capacities were used for Cases 1, 2, 3, 4, and 5. Drained strength pile capacities were used for Case 6 only.

4) Sheet Pile Cut-offs – The sheet pile cut-off walls were assumed 10 ft long installed along both the upstream and downstream faces of the aqueduct mat foundation. For the approach and wing walls it was assumed a single upstream cut-off wall would be used.

5) The soil conditions along the entire project are underlain by "soft" clay, susceptible to large settlements. Therefore, aqueducts structures, approach and wing walls are supported by H-piles.

F-R2.0 SHEYENNE RIVER AQUEDUCT STRUCTURE

Table F-R1 presents the key elevations and loading conditions for the Sheyenne River Aqueduct Structure. Table F-R2 presents the pile capacity of the Sheyenne River Aqueduct Structure. Figure F-R1 presents a typical cross section of the Sheyenne River Aqueduct Structure.

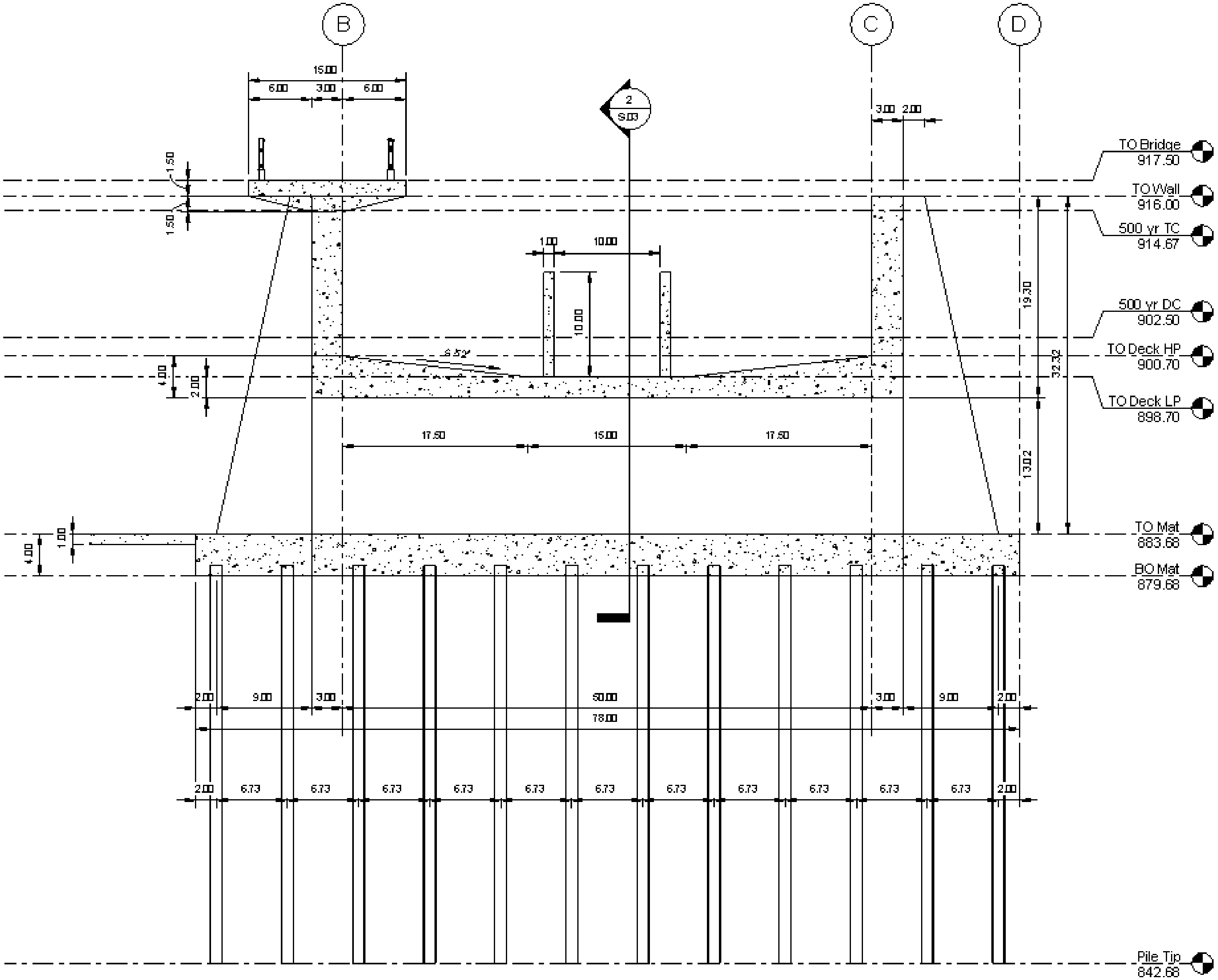
Table F-R1 Sheyenne River Aqueduct – Key Elevations and Loading Conditions

ID#	**Case 1**	**Case 2**	**Case 3**	**Case 4**	**Case 5**	**Case 6**
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	914.56	914.56	914.67	917.5	903.24	**NA**
Diversion - Head Water El. (ft)	902.12	902.12	903.32	917.5	**NA**	**NA**
Diversion - Tail Water El. (ft)	901.91	901.91	903.06	917.5	**NA**	**NA**
Tributary - T.O. Wall El. (ft)	917.5					
Tributary - T.O. Deck L.P. El.(ft)	898.7					
Tributary - T.O. Deck H.P. El.(ft)	900.7					
Diversion - T.O. Mat El. (ft)	901.68					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tributary - Water height (ft)	**15.86**	**15.86**	**15.97**	**18.8**	**4.54**	**NA**
Diversion - Head Water height (ft)	**0.44**	**0.44**	**1.64**	**15.82**	**NA**	**NA**
Ice	**NA**	2ft Ice	**NA**	**NA**	2ft Ice	**NA**
Ice Load	**NA**	10 kips/ft	**NA**	**NA**	10 kips/ft	**NA**
Ice Load El. (ft)	**NA**	**914.56**	**NA**	**NA**	**903.24**	**NA**
Uplift @ HW (ft)	**4.44**	**4.44**	**5.64**	**19.82**	**NA**	**NA**
Uplift @ TW (ft)	**4.23**	**4.23**	**5.38**	**19.82**	**NA**	**NA**
Pile Condition	**Undrained**	**Undrained**	**Undrained**	**Undrained**	**Drained**	**Undrained**
Load Category	**Usual**	**Unusual**	**Unusual**	**Extreme**	**Usual**	**Unusual**
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allowable Lateral Capacity (tons)	**18**	**20.5**	**20.5**	**24**	**11.5**	**20.5**
Allowable Pile Capacity (tons) - Axial	**61.95**	**82.60**	**82.60**	**107.74**	**36.525**	**82.60**
Allowable Pile Capacity (tons) - Uplift	**38.65**	**51.53**	**51.53**	**67.22**	**5.9**	**51.53**

Table F-R2 Sheyenne River Aqueduct – Pile Capacity

Pile Capacity	Ultimate Axial Capacity	Allowable Lateral Capacity (kips)		
		0.5" (Usual)	0.67" (Unusual)	0.875" (Extreme)
Undrained - Axial	247.8	36	41	48
Undrained - Uplift	154.6			
Drained - Axial	146.1	23	29	36
Drained - Uplift	23.6			

Figure F-R1 Sheyenne River Aqueduct Structure Typical Section (Looking North)



F-R3.0 MAPLE RIVER AQUEDUCT STRUCTURE

Table F-R3 presents the key elevations and loading conditions for the Maple River Aqueduct Structure. Table F-R4 presents the pile capacity of the Maple River Aqueduct Structure. Figure F-R2 presents a typical cross section of the Maple River Aqueduct Structure.

Table F-R3 Maple River Aqueduct – Key Elevations and Loading Conditions

ID#	**Case 1**	**Case 2**	**Case 3**	**Case 4**	**Case 5**	**Case 6**
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	914.56	914.56	914.67	917.5	903.24	**NA**
Diversion - Head Water El. (ft)	902.12	902.12	903.32	917.5	**NA**	**NA**
Diversion - Tail Water El. (ft)	901.91	901.91	903.06	917.5	**NA**	**NA**
Tributary - T.O. Wall El. (ft)	917.5					
Tributary - T.O. Deck L.P. El.(ft)	898.7					
Tributary - T.O. Deck H.P. El.(ft)	900.7					
Diversion - T.O. Mat El. (ft)	901.68					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	**15.86**	**15.86**	**15.97**	**18.8**	**4.54**	**NA**
Diversion - Head Water height (ft)	**0.44**	**0.44**	**1.64**	**15.82**	**NA**	**NA**
Ice	**NA**	2ft Ice	**NA**	**NA**	2ft Ice	**NA**
Ice Load	**NA**	10 kips/ft	**NA**	**NA**	10 kips/ft	**NA**
Ice Load El. (ft)	**NA**	**914.56**	**NA**	**NA**	**903.24**	**NA**
Uplift @ HW (ft)	**4.44**	**4.44**	**5.64**	**19.82**	**NA**	**NA**
Uplift @ TW (ft)	**4.23**	**4.23**	**5.38**	**19.82**	**NA**	**NA**

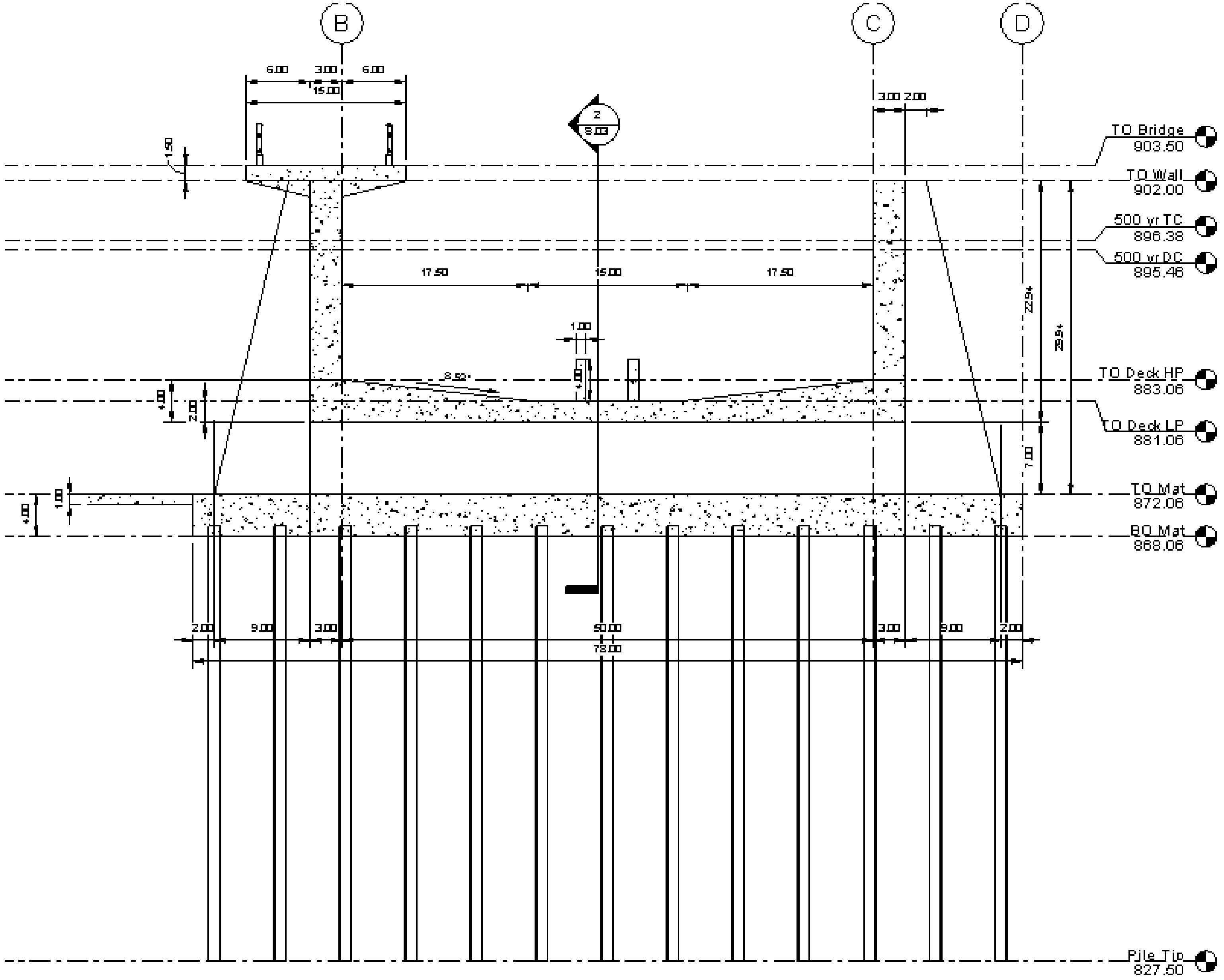
Table F-R3 Maple River Aqueduct – Key Elevations and Loading Conditions (continued)

Pile Condition	**Undrained**	**Undrained**	**Undrained**	**Undrained**	**Drained**	**Undrained**
Load Category	**Usual**	**Unusual**	**Unusual**	**Extreme**	**Usual**	**Unusual**
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allowable Lateral Capacity (tons)	**18**	**20.5**	**20.5**	**24**	**11.5**	**20.5**
Allowable Pile Capacity (tons) - Axial	**61.95**	**82.60**	**82.60**	**107.74**	**36.525**	**82.60**
Allowable Pile Capacity (tons) - Uplift	**38.65**	**51.53**	**51.53**	**67.22**	**5.9**	**51.53**

Table F-R4 Maple River Aqueduct – Pile Capacity

Pile Capacity	**Ultimate Axial Capacity**	**Allowable Lateral Capacity (kips)**		
		0.5" (Usual)	**0.67" (Unusual)**	**0.875" (Extreme)**
Undrained - Axial	228.7	36	42	48
Undrained - Uplift	135.5			
Drained - Axial	125.7	23	29	33
Drained - Uplift	18.5			

Figure F-R2   Maple River Aqueduct Structure Typical Section (Looking North)



ATTACHMENT

F-R4.1 Sheyenne River Aqueduct Structural Computations

DATE
SHEET NO. 1
PROJECT NAME Fargo Moorhead Div. Chn.
PROJECT NUMBER 34091004
SUBJECT Aqueduct Preliminary Reinforcement Design.

COMPUTED BY mbl DATE 2/21/11
CHECKED BY DATE
SUBMITTED TO DATE

Aqueduct MAT FOUNDATION

STAAD Results.

<u>Moment</u>

Mxmax = 104.34 k-ft/ft LC# 5. Pl# 14951 Mxy = 1.375 k-ft/ft

Mxmin = -94.78 k-ft/ft LC# 32 Pl# 9534 Mxy = -4.322 k-ft/ft

Mymax = 106.209 k-ft/ft LC# 5. Pl# 13278 Mxy = 40.59 k-ft/

Mymin = -90.415 k-ft/ft LC# 47 Pl# 9479 Mxy = -2.619 k-ft/ft

$\boxed{\text{Governs}}$

Plate orientation + → Bottom
− → Top

My = 106.209 + 40.59 = 146.78 k-ft/ft

b = 12"
h = 48" } As req = 1.021 in²/ft.

cover = 15"
(For Bottom Bars Due to Piles.)

As min T&S > #9 @ 12" As = 1.0 in²/ft

As prelim selected #9@6"
As = 2.0 in²/ft

WALL / SLAB REINFORCEMENT DESIGN TEMPLATE

REF. *1. ACI 318 Building Code and Commentary*
2. ACI 350 Code Requirements for Environmental Structures

Password: obgmbi
Please Fill in the gray boxes.

Client Name:	**U.S. ARMY CORPS OF ENGINEERS**	**Design By:**	**MBI**
Project Name:	**FARGO – MOORHEAD , FEASIBILITY STUDY, PHASE 4**	**Review By:**	
Work Describtion:	**Sheyenne Aquaduct Structure - Mat Foundation**	**Date:**	**2/14/2011**
		Job #:	**34091004**

Cross Section Properties			
Height	h (in)	48	in
Width	b (in)	12	in
Section as		Slab Section	
Section Modulus	S (in3)	4608	in^3
Design Forces			
Shear	Vu (kips)	35	kip
Moment	Mu (k-ft/ft)	146.78	kip-ft / ft

Material Properties			
Concrete	f_c' (ksi)	4	ksi
	E (ksi)		ksi
Steel	f_y (ksi)	60	ksi
	E (ksi)	29000	ksi
	n	9	

Factors	
φFlex	0.9
φShear	0.75
Governing Code	
ACI 350- 01	

Flexural Reinforcement Design — *Minimum Required Steel*

Cover	15	in	$3fc'^{1/2}/fy$ =	0.00316		200/fy =	0.00333	
Length Bet. Mov. Jt.	50	ft	$\rho_{min\ Flex}$ =	0.0033333		$A_{s\ min\ Flex}$ =	1.308	in^2
Bar index	# 5		$\rho_{min\ T\&S}$ =	0.005		$A_{s\ min\ T\&S}$ =	0.720	in^2
d (Tension Reinf.)	32.6875	in	$\rho_{Balance}$ =	0.0285068		$A_{s\ max}$ =	11.1818	in^2
Design Mom. (Fact.)	146.78	kip-ft / ft	Rn	0.153		Constant	0.090	
						OK		
a	1.502	in	$A_{s\ Required}$ =	1.021	in^2	$\rho_{Required}$ =	0.0026	
k	0.046		$A_{s\ Provide}$ =	**1.308**	in^2			
j	0.98							
Moment Calculated	146.78	kip-ft	$A_{s\ Selected}$ =	# 9		$A_{s\ Selected}$ =	2.000	in^2
Difference Δ	**0.000**		Spacing @	6	in	**< Amax**		
a'	2.941		**Spacing is OK per ACI 318**					
β_1	0.85							
c	3.460		**Tension controlled**			φ	**0.9**	
ε_t	0.0253	>	0.005			**Same as f assumed initially, OK.**		
ϕM_n	280.95	kip-ft / ft	> Design Moment, Section OK					

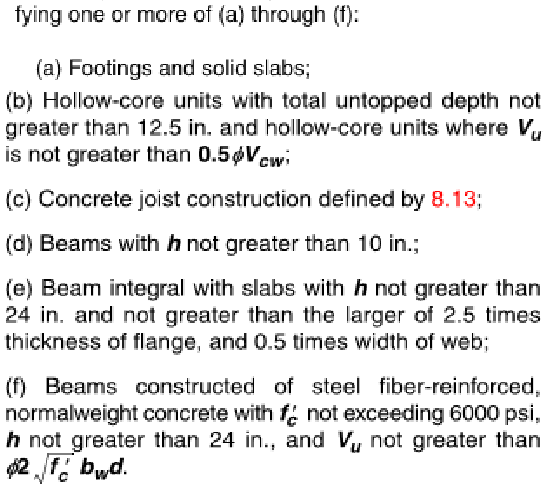
Cracking moment Capacity — *Flexural Moment Capacity*

f_r =	474.34	psi	$A_{s\ Selected}$ =	2.000	in^2
$M_{r\ Cracking}$ =	182.15	k-ft	$M_{u\ Flex}$ =	280.95	k-ft

Shear Reinforcement Design

Vc	49.62	kip			
φShear	0.75				
φVc	37.21	kip	>	Vu =	35.00 kip
Vs	9.45	kip			

Minimum Shear Reinforcement



11.4.6 — Minimum shear reinforcement

11.4.6.1 — A minimum area of shear reinforcement, $A_{v,min}$, shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where V_u exceeds $0.5\phi V_c$, except in members satisfying one or more of (a) through (f):

(a) Footings and solid slabs;

(b) Hollow-core units with total untopped depth not greater than 12.5 in. and hollow-core units where V_u is not greater than $0.5\phi V_{cw}$;

(c) Concrete joist construction defined by 8.13;

(d) Beams with h not greater than 10 in.;

(e) Beam integral with slabs with h not greater than 24 in. and not greater than the larger of 2.5 times thickness of flange, and 0.5 times width of web;

(f) Beams constructed of steel fiber-reinforced, normalweight concrete with f_c' not exceeding 6000 psi, h not greater than 24 in., and V_u not greater than $\phi 2\sqrt{f_c'}\,b_w d$.

Is any of the requirements (a thru f) satisfied? YES

Av Min / s =	0.0095	OR	50bw/fyt=	0.010
Av Min/s =	0.01			

No Shear reinforcement required.

Spacing Requirement

s = d/2	16.34375 in	OR		24 in
4fc'^0.5*bw*d	99.23 kip	>	Vs =	9.45 kip
smax =	17.00 in			
$A_{v\ Min}$ =	0.17 in^2			
Av/s = Vs/fyt*d	0.0048			
$A_{v\ Required}$ =	0.08 in^2		$A_{v\ Provide}$ =	0.17 in^2
Stirrup Size	# 3		$A_{s\ Selected}$ =	0.220 in^2
# Legs	2		**> Av Provide, OK**	

Calculation of Moment Capacity with Min. Reinforcement

Analize as a 8

1	$\rho_{min\ Flex.}$	0.00333	Flexural Members
2	$\rho_{min\ Wall\ Ver.}$	0.0015	Wall Vertical (ACI 318 Section 14.3.2)
3	$\rho_{min\ Wall\ Hor.}$	0.0025	Wall Horizontal (ACI 318 Section 14.3.2)
4	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink for Slabs (ACI 318 Section 7.12.2.1)
5	$\rho_{min\ Wall\ Ver.}$	0.003	Wall Vertical (ACI 350 Section 14.3.2)
6	$\rho_{min\ Wall\ Hor.}$	0.005	Wall Horizontal (ACI 350 Section 14.3.3)
7	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink (ACI 318 Section 7.12.2.1)
8	Select Reinforcement Below		

$A_{s\ Selected}$ # 9 @ 12 in $A_{s\ Selected}$ 1 in^2

Than ρ_{min} **0.00000**

$A_s = \rho_{min}$*bd or bh	1.0000	in^2	***One Face Only*** As=	0.5	in^2
T = A_s*fy	60.000	kip	T = A_s*fy	30.000	kip
a = T / (0.85 b fc')	1.471	in	a = T / (0.85 b fc')	0.735	
M = T*(d-a/2)	159.761	kip-ft	M = T*(d-a/2)	80.800	kip-ft
M_u = φ*M	143.785	kip-ft	M_u = φ*M	72.720	kip-ft

COMPUTED	CHECKED	SUBMITTED	DATE
BY mbl	BY	TO	PROJECT NAME Fargo Moorhead Div. Ch.
DATE 2/21/11	DATE	DATE	PROJECT NUMBER 34091004

SHEET NO. 5

SUBJECT

TRIBUTARY CHANNEL ELEVATED SLAB

STAAD Results

moments

$M_{x\,max}$ = 93.81 k-ft/ft LC# 32 Pl# 1133 M_{xy} = -0.494 k-ft/ft

$M_{x\,min}$ = -106.32 k-ft/ft LC# 45 Pl# 637 M_{xy} = -26.221 k-ft/ft

$M_{y\,max}$ = 142.858 k-ft/ft LC# 55 Pl# 718 M_{xy} = -2.547 k-ft/ft

$M_{y\,min}$ = -77.671 k-ft/ft LC# 37 Pl# 713 M_{xy} = 0.713 k-ft/ft

→ Governs

Plate orientation

y → Longitudinal
x → Transverse
+ → Top
- → Bottom

$M = 142.858 + 2.547 = 145.40$ k-ft/ft.

b = 12"
h = 24"

As required = 1.666 in²/ft.

As min = 1 in²/ft.

As prelim selected #9@6' As = 2.0 in²/ft ✓ ok.

WALL / SLAB REINFORCEMENT DESIGN TEMPLATE

REF. *1. ACI 318 Building Code and Commentary*
2. ACI 350 Code Requirements for Environmental Structures

Password: obgmbi
Please Fill in the gray boxes.

Client Name:	**U.S. ARMY CORPS OF ENGINEERS**	**Design By:**	**MBI**
Project Name:	**FARGO – MOORHEAD , FEASIBILITY STUDY, PHASE 4**	**Review By:**	
Work Describtion:	**Sheyenne Aquaduct Structure - Tributary Channel Deck**	**Date:**	**2/14/2011**
		Job #:	**34091004**

Cross Section Properties			
Height	h (in)	24	in
Width	b (in)	12	in
Section as		Slab Section	
Section Modulus	S (in3)	1152	in^3
Design Forces			
Shear	Vu (kips)	35	kip
Moment	Mu (k-ft/ft)	145.4	kip-ft / ft

Material Properties			
Concrete	f_c' (ksi)	4	ksi
	E (ksi)		ksi
Steel	f_y (ksi)	60	ksi
	E (ksi)	29000	ksi
	n	9	

Factors	
ϕFlex	0.9
ϕShear	0.75
Governing Code	
ACI 350- 01	

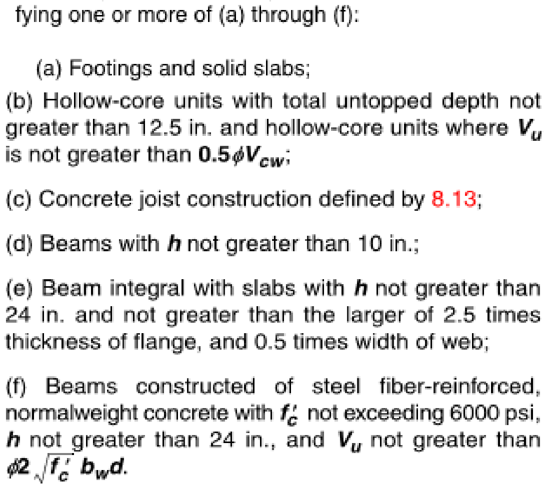
Flexural Reinforcement Design			*Minimum Required Steel*					
Cover	3	in	$3fc'^{1/2}/fy$ =	0.00316		200/fy =	0.00333	
Length Bet. Mov. Jt.	50	ft	$\rho_{min\ Flex}$ =	0.0033333		$A_{s\ min\ Flex}$ =	0.828	in^2
Bar index	# 5		$\rho_{min\ T\&S}$ =	0.005		$A_{s\ min\ T\&S}$ =	0.720	in^2
d (Tension Reinf.)	20.6875	in	$\rho_{Balance}$ =	0.0285068		$A_{s\ max}$ =	7.07681	in^2
Design Mom. (Fact.)	145.4	kip-ft / ft	Rn	0.377		Constant	0.222	
						OK		
a	2.441	in	$A_{s\ Required}$ =	1.660	in^2	$\rho_{Required}$ =	0.00669	
k	0.118		$A_{s\ Provide}$ =	**1.660**	in^2			
j	0.94							
Moment Calculated	145.40	kip-ft	$A_{s\ Selected}$ =	# 9		$A_{s\ Selected}$ =	2.000	in^2
Difference Δ	**0.000**		Spacing @	6	in	**< Amax**		
a'	2.941		**Spacing is OK per ACI 318**					
β_1	0.85							
c	3.460		**Tension controlled**			ϕ	**0.9**	
ε_t	0.0149	>	0.005			**Same as f assumed initially, OK.**		
ϕM_n	172.95	kip-ft / ft	>	Design Moment, Section OK				

Cracking moment Capacity			*Flexural Moment Capacity*		
f_r =	474.34	psi	$A_{s\ Selected}$ =	2.000	in^2
$M_{r\ Cracking}$ =	45.54	k-ft	$M_{u\ Flex}$ =	172.95	k-ft

Shear Reinforcement Design

Vc	31.40	kip				
ϕShear	0.75					
ϕVc	23.55	kip	<	Vu =	35.00	kip
Vs	23.12	kip				

Minimum Shear Reinforcement



11.4.6 — Minimum shear reinforcement

11.4.6.1 — A minimum area of shear reinforcement, $\boldsymbol{A_{v,min}}$, shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where $\boldsymbol{V_u}$ exceeds $\boldsymbol{0.5\phi V_c}$, except in members satisfying one or more of (a) through (f):

(a) Footings and solid slabs;

(b) Hollow-core units with total untopped depth not greater than 12.5 in. and hollow-core units where $\boldsymbol{V_u}$ is not greater than $\boldsymbol{0.5\phi V_{cw}}$;

(c) Concrete joist construction defined by 8.13;

(d) Beams with $\boldsymbol{h}$ not greater than 10 in.;

(e) Beam integral with slabs with $\boldsymbol{h}$ not greater than 24 in. and not greater than the larger of 2.5 times thickness of flange, and 0.5 times width of web;

(f) Beams constructed of steel fiber-reinforced, normalweight concrete with $\boldsymbol{f_c'}$ not exceeding 6000 psi, $\boldsymbol{h}$ not greater than 24 in., and $\boldsymbol{V_u}$ not greater than $\boldsymbol{\phi 2\sqrt{f_c'}\, b_w d}$.

Is any of the requirements (a thru f) satisfied?			YES	
Av Min / s =	0.0095	OR	50bw/fyt=	0.010
Av Min/s =	0.01			

Shear Reinforcement Required

Spacing Requirement					
s = d/2	10.34375 in	OR		24 in	
4fc'^0.5*bw*d	62.80 kip	>	Vs =		23.12 kip
smax =	11.00 in				
$A_{v\ Min}$ =	0.11 in^2				
Av/s = Vs/fyt*d	0.0186				
$A_{v\ Required}$ =	0.20 in^2		$A_{v\ Provide}$ =	0.20 in^2	
Stirrup Size	# 3		$A_{s\ Selected}$ =	0.220	in^2
# Legs	2		**> Av Provide, OK**		

Calculation of Moment Capacity with Min. Reinforcement

Analize as a 8

1	$\rho_{min\ Flex.}$	0.00333	Flexural Members
2	$\rho_{min\ Wall\ Ver.}$	0.0015	Wall Vertical (ACI 318 Section 14.3.2)
3	$\rho_{min\ Wall\ Hor.}$	0.0025	Wall Horizontal (ACI 318 Section 14.3.2)
4	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink for Slabs (ACI 318 Section 7.12.2.1)
5	$\rho_{min\ Wall\ Ver.}$	0.003	Wall Vertical (ACI 350 Section 14.3.2)
6	$\rho_{min\ Wall\ Hor.}$	0.005	Wall Horizontal (ACI 350 Section 14.3.3)
7	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink (ACI 318 Section 7.12.2.1)
8	Select Reinforcement Below		

$A_{s\ Selected}$ # 9 @ 12 in $A_{s\ Selected}$ 1 in^2

Than ρ_{min} **0.00000**

$A_s = \rho_{min}$*bd or bh	1.0000	in^2	***One Face Only*** As=	0.5	in^2
T = A_s*fy	60.000	kip	T = A_s*fy	30.000	kip
a = T / (0.85 b fc')	1.471	in	a = T / (0.85 b fc')	0.735	
M = T*(d-a/2)	99.761	kip-ft	M = T*(d-a/2)	50.800	kip-ft
$M_u = \phi$*M	89.785	kip-ft	$M_u = \phi$*M	45.720	kip-ft

BARR
MINNEAPOLIS, MINNESOTA - HIBBING, MINNESOTA
DULUTH, MINNESOTA
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

COMPUTED	CHECKED	SUBMITTED
BY mbl	BY	TO
DATE 2/21/11	DATE	DATE

DATE
SHEET NO. 3
PROJECT NAME Big Morhead Div. Ch.
PROJECT NUMBER 32/091004
SUBJECT

PIERS / WALLS

moments

Mxmax = 94.17 k-ft/ft LC#37 PL#6519 Mxy = 45.904 k-ft/ft

Mxmin = -111.728 k-ft/ft LC#55 PL#8241 Mxy = -45.16 k-ft/ft

Mymax = 105.502 k-ft/ft LC#37 PL#6517 Mxy = 38.741 k-ft/ft

Mymin = -130.012 k-ft/ft LC#27 PL#1176 Mxy = 2.753 k-ft/ft

Govern

M = 111.728 + 45.16 = 156.88 k-ft/ft.

b = 12"
h = 36"
Cover = 3"

As required = 1.093 in²/ft

As min flex = 1.308 in²/ft

As provided selected #9@6" As = 2.0 in²/ft

WALL / SLAB REINFORCEMENT DESIGN TEMPLATE

REF. *1. ACI 318 Building Code and Commentary*
2. ACI 350 Code Requirements for Environmental Structures

Password: obgmbi
Please Fill in the gray boxes.

Client Name:	**U.S. ARMY CORPS OF ENGINEERS**	**Design By:**	**MBI**
Project Name:	**FARGO – MOORHEAD , FEASIBILITY STUDY, PHASE 4**	**Review By:**	
Work Describtion:	**Sheyenne Aquaduct Structure - Walls and Piers**	**Date:**	**2/14/2011**
		Job #:	**34091004**

Cross Section Properties				**Material Properties**				**Factors**	
Height	h (in)	36	in	*Concrete*	f_c' (ksi)	4	ksi	ϕFlex	0.9
Width	b (in)	12	in		E (ksi)		ksi	ϕShear	0.75
Section as		Wall Section		*Steel*	f_y (ksi)	60	ksi	**Governing Code**	
Section Modulus	S (in3)	2592	in^3		E (ksi)	29000	ksi	ACI 350- 01	
Design Forces					n	9			
Shear	Vu (kips)	35	kip						
Moment	Mu (k-ft/ft)	156.88	kip-ft / ft						

Flexural Reinforcement Design			*Minimum Required Steel*					
Cover	3	in	$3fc'^{1/2}/fy$ =	0.00316		200/fy =	0.00333	
Length Bet. Mov. Jt.	50	ft	$\rho_{min\ Flex}$ =	0.0033333		$A_{s\ min\ Flex}$ =	1.308	in^2
Bar index	# 5		$\rho_{min\ T\&S}$ =	0.005		$A_{s\ min\ T\&S}$ =	0.720	in^2
d (Tension Reinf.)	32.6875	in	$\rho_{Balance}$ =	0.0285068		$A_{s\ max}$ =	11.1818	in^2
Design Mom. (Fact.)	156.88	kip-ft / ft	Rn	0.163		Constant	0.096	
						OK		
a	1.608	in	$\mathbf{A_{s\ Required}}$ =	1.093	in^2	$\rho_{Required}$ =	0.00279	
k	0.049		$A_{s\ Provide}$ =	**1.308**	in^2			
j	0.98							
Moment Calculated	156.88	kip-ft	$A_{s\ Selected}$ =	# 9		$A_{s\ Selected}$ =	2.000	in^2
Difference Δ	**0.000**		Spacing @	6	in	**< Amax**		
a'	2.941		**Spacing is OK per ACI 318**					
β_1	0.85							
c	3.460		**Tension controlled**			ϕ	**0.9**	
ε_t	0.0253	>	0.005			**Same as f assumed initially, OK.**		
ϕM_n	280.95	kip-ft / ft	>	Design Moment, Section OK				

Cracking moment Capacity			*Flexural Moment Capacity*		
f_r =	474.34	psi	$A_{s\ Selected}$ =	2.000	in^2
$M_{r\ Cracking}$ =	102.46	k-ft	$M_{u\ Flex}$ =	280.95	k-ft

Shear Reinforcement Design

Vc	49.62	kip				
ϕShear	0.75					
ϕVc	37.21	kip	>	Vu =	35.00	kip
Vs	9.45	kip				

Minimum Shear Reinforcement

11.4.6 — Minimum shear reinforcement

11.4.6.1 — A minimum area of shear reinforcement, $A_{v,min}$, shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where V_u exceeds $0.5\phi V_c$, except in members satisfying one or more of (a) through (f):

(a) Footings and solid slabs;

(b) Hollow-core units with total untopped depth not greater than 12.5 in. and hollow-core units where V_u is not greater than $0.5\phi V_{cw}$;

(c) Concrete joist construction defined by 8.13;

(d) Beams with *h* not greater than 10 in.;

(e) Beam integral with slabs with *h* not greater than 24 in. and not greater than the larger of 2.5 times thickness of flange, and 0.5 times width of web;

(f) Beams constructed of steel fiber-reinforced, normalweight concrete with f_c' not exceeding 6000 psi, *h* not greater than 24 in., and V_u not greater than $\phi 2\sqrt{f_c'}\,b_w d$.

Is any of the requirements (a thru f) satisfied?			YES	
Av Min / s =	0.0095	OR	50bw/fyt=	0.010
Av Min/s =	0.01			

No Shear reinforcement required.

Spacing Requirement					
s = d/2	16.34375	in	OR	24 in	
4fc'^0.5*bw*d	99.23	kip	>	Vs =	9.45 kip
smax =	17.00	in			
$A_{v\ Min}$ =	0.17	in^2			
Av/s = Vs/fyt*d	0.0048				
$\mathbf{A_{v\ Required}}$ =	0.08	in^2	$A_{v\ Provide}$ =	0.17	in^2
Stirrup Size	# 3		$A_{s\ Selected}$ =	0.220	in^2
# Legs	2		**> Av Provide, OK**		

Calculation of Moment Capacity with Min. Reinforcement

Analize as a 8

1	$\rho_{min\ Flex.}$	0.00333	Flexural Members
2	$\rho_{min\ Wall\ Ver.}$	0.0015	Wall Vertical (ACI 318 Section 14.3.2)
3	$\rho_{min\ Wall\ Hor.}$	0.0025	Wall Horizontal (ACI 318 Section 14.3.2)
4	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink for Slabs (ACI 318 Section 7.12.2.1)
5	$\rho_{min\ Wall\ Ver.}$	0.003	Wall Vertical (ACI 350 Section 14.3.2)
6	$\rho_{min\ Wall\ Hor.}$	0.005	Wall Horizontal (ACI 350 Section 14.3.3)
7	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink (ACI 318 Section 7.12.2.1)
8	Select Reinforcement Below		

$A_{s\ Selected}$	# 9	@	12	in	$A_{s\ Selected}$	1 in^2
Than ρ_{min}		**0.00000**				

$A_s = \rho_{min}$*bd or bh	1.0000	in^2	***One Face Only*** As=	0.5	in^2
$T = A_s$*fy	60.000	kip	$T = A_s$*fy	30.000	kip
a = T / (0.85 b fc')	1.471	in	a = T / (0.85 b fc')	0.735	
M = T*(d-a/2)	159.761	kip-ft	M = T*(d-a/2)	80.800	kip-ft
$M_u = \phi$*M	143.785	kip-ft	$M_u = \phi$*M	72.720	kip-ft

DATE

SHEET NO. 4

PROJECT NAME Fargo Moorhead Div. ch.

PROJECT NUMBER 34091004

COMPUTED	CHECKED	SUBMITTED
BY mbl	BY	TO
DATE 2/21/11	DATE	DATE

SUBJECT

Low Flow CHANNEL WALLS

moments

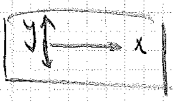
Mxmax = 4.011 k-ft/ft LC# 53 PL# 8693 Mxy = 0.652 k-ft/ft

Mxmin = −0.232 k-ft/ft LC# 55 PL# 8739 Mxy = 0.036 k-ft/ft

Mymax = 85.159 k-ft/ft LC# 53 PL# 8665 Mxy = 1.157 k-ft/ft

Mymin = −0.551 k-ft/ft LC# 55 PL# 8679 Mxy = 0.051 k-ft/ft

Covers flat situation



y : vertical
x : horizontal
+ → Inside
− → outside

M = 85.159 + 1.157 = 86.316 k-ft/ft

b = 12"
h = 15"
cover = 3"

Asreq = 1.658 in²/ft.

Asmin = #9@12" = 1 in²/ft

As prelim select #9@6" As = 2.0 in²/ft.

WALL / SLAB REINFORCEMENT DESIGN TEMPLATE

REF. *1. ACI 318 Building Code and Commentary*
2. ACI 350 Code Requirements for Environmental Structures

Password: obgmbi
Please Fill in the gray boxes.

Client Name:	**U.S. ARMY CORPS OF ENGINEERS**	**Design By:**	**MBI**
Project Name:	**FARGO – MOORHEAD , FEASIBILITY STUDY, PHASE 4**	**Review By:**	
Work Describtion:	**Sheyenne Aquaduct Structure - Low flow Channel Walls**	**Date:**	**2/14/2011**
		Job #:	**34091004**

Cross Section Properties			
Height	h (in)	15	in
Width	b (in)	12	in
Section as		Wall Section	
Section Modulus	S (in3)	450	in^3
Design Forces			
Shear	Vu (kips)	35	kip
Moment	Mu (k-ft/ft)	86.316	kip-ft / ft

Material Properties			
Concrete	f_c' (ksi)	4	ksi
	E (ksi)		ksi
Steel	f_y (ksi)	60	ksi
	E (ksi)	29000	ksi
n		9	

Factors	
ϕFlex	0.9
ϕShear	0.75
Governing Code	
ACI 350- 01	

Flexural Reinforcement Design

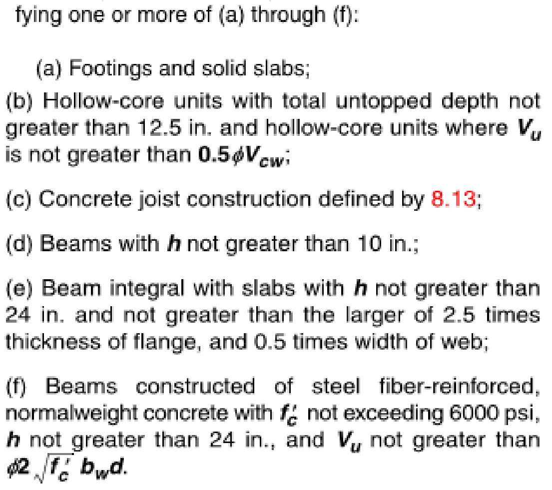
Cover	3	in	$3fc'^{1/2}/fy =$	0.00316		200/fy =	0.00333	
Length Bet. Mov. Jt.	50	ft	$\rho_{min\ Flex} =$	0.0033333		$A_{s\ min\ Flex} =$	0.468	in^2
Bar index	# 5		$\rho_{min\ T\&S} =$	0.005		$A_{s\ min\ T\&S} =$	0.450	in^2
d (Tension Reinf.)	11.6875	in	$\rho_{Balance} =$	0.0285068		$A_{s\ max} =$	3.99808	in^2
Design Mom. (Fact.)	86.316	kip-ft / ft	Rn	0.702		Constant	0.413	
						OK		
a	2.733	in	$\mathbf{A_{s\ Required}} =$	1.858	in^2	$\rho_{Required} =$	0.01325	
k	0.234		$A_{s\ Provide} =$	**1.858**	in^2			
j	0.88							
Moment Calculated	86.32	kip-ft	$A_{s\ Selected} =$	# 9		$A_{s\ Selected} =$	2.000	in^2
Difference Δ	**0.000**		Spacing @	6	in	**< Amax**		
a'	2.941		**Spacing is OK per ACI 318**					
β_1	0.85							
c	3.460		**Tension controlled**			ϕ	**0.9**	
ε_t	0.0071	>	0.005			**Same as f assumed initially, OK.**		
ϕM_n	91.95	kip-ft / ft	>	Design Moment, Section OK				

Cracking moment Capacity			*Flexural Moment Capacity*		
$f_r =$	474.34	psi	$A_{s\ Selected} =$	2.000	in^2
$M_{r\ Cracking} =$	17.79	k-ft	$M_{u\ Flex} =$	91.95	k-ft

Shear Reinforcement Design

Vc	17.74	kip				
ϕShear	0.75					
ϕVc	13.31	kip	<	Vu =	35.00	kip
Vs	33.36	kip				

Minimum Shear Reinforcement



11.4.6 — Minimum shear reinforcement

11.4.6.1 — A minimum area of shear reinforcement, $A_{v,min}$, shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where V_u exceeds $0.5\phi V_c$, except in members satisfying one or more of (a) through (f):

(a) Footings and solid slabs;

(b) Hollow-core units with total untopped depth not greater than 12.5 in. and hollow-core units where V_u is not greater than $0.5\phi V_{cw}$;

(c) Concrete joist construction defined by 8.13;

(d) Beams with *h* not greater than 10 in.;

(e) Beam integral with slabs with *h* not greater than 24 in. and not greater than the larger of 2.5 times thickness of flange, and 0.5 times width of web;

(f) Beams constructed of steel fiber-reinforced, normalweight concrete with f_c' not exceeding 6000 psi, *h* not greater than 24 in., and V_u not greater than $\phi 2\sqrt{f_c'}\,b_w d$.

Is any of the requirements (a thru f) satisfied? YES

Av Min / s =	0.0095		OR	50bw/fyt=	0.010
Av Min/s =	0.01				

Shear Reinforcement Required

Spacing Requirement

s = d/2	5.84375	in	OR	24	in	
4fc'^0.5*bw*d	35.48	kip	>	Vs =	33.36	kip
smax =	6.00	in				
$A_{v\ Min} =$	0.06	in^2				
Av/s = Vs/fyt*d	0.0476					
$\mathbf{A_{v\ Required}} =$	0.29	in^2	$A_{v\ Provide} =$	0.29	in^2	
Stirrup Size	# 3		$A_{s\ Selected} =$	0.220	in^2	
# Legs	2		**> Av Provide, OK**			

Calculation of Moment Capacity with Min. Reinforcement

Analize as a 8

1	$\rho_{min\ Flex.}$	0.00333	Flexural Members
2	$\rho_{min\ Wall\ Ver.}$	0.0015	Wall Vertical (ACI 318 Section 14.3.2)
3	$\rho_{min\ Wall\ Hor.}$	0.0025	Wall Horizontal (ACI 318 Section 14.3.2)
4	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink for Slabs (ACI 318 Section 7.12.2.1)
5	$\rho_{min\ Wall\ Ver.}$	0.003	Wall Vertical (ACI 350 Section 14.3.2)
6	$\rho_{min\ Wall\ Hor.}$	0.005	Wall Horizontal (ACI 350 Section 14.3.3)
7	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink (ACI 318 Section 7.12.2.1)
8	Select Reinforcement Below		

$A_{s\ Selected}$ # 9 @ 12 in $A_{s\ Selected}$ 1 in^2

Than ρ_{min} **0.00000**

$A_s = \rho_{min}$*bd or bh	1.0000	in^2	***One Face Only*** As=	0.5	in^2
T = A_s*fy	60.000	kip	T = A_s*fy	30.000	kip
a = T / (0.85 b fc')	1.471	in	a = T / (0.85 b fc')	0.735	
M = T*(d-a/2)	54.761	kip-ft	M = T*(d-a/2)	28.300	kip-ft
$M_u = \phi$*M	49.285	kip-ft	$M_u = \phi$*M	25.470	kip-ft

Barr Engineering

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Job No **34091009**	Sheet No **1**		Rev 2
Job Title Fargo- Moorhead Feasibility Study - Phase4	Part Sheyene Aquaduct - Partial		
	Ref Phase4		
	By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers	File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	

Job Information

	Engineer	Checked	Approved
Name:	mbi		
Date:	1-24-11		

Comments

Only (3) 30-ft bays is modeled for simplicity.
Wall thickness = 36-in
Tributary Slab thickness = 32" (ave)
Access Bridge slab thickness = 27" (avarage)
Matt foundation thickness = 48"
Low flow channel walls = 12"
REV1. Geometry Update
REV2. Wall Thickness ad pile loc update

Structure Type	SPACE FRAME

Number of Nodes	11893	Highest Node	17741
Number of Plates	11666	Highest Plate	17054

Number of Basic Load Cases	19
Number of Combination Load Cases	0

Included in this printout are data for:

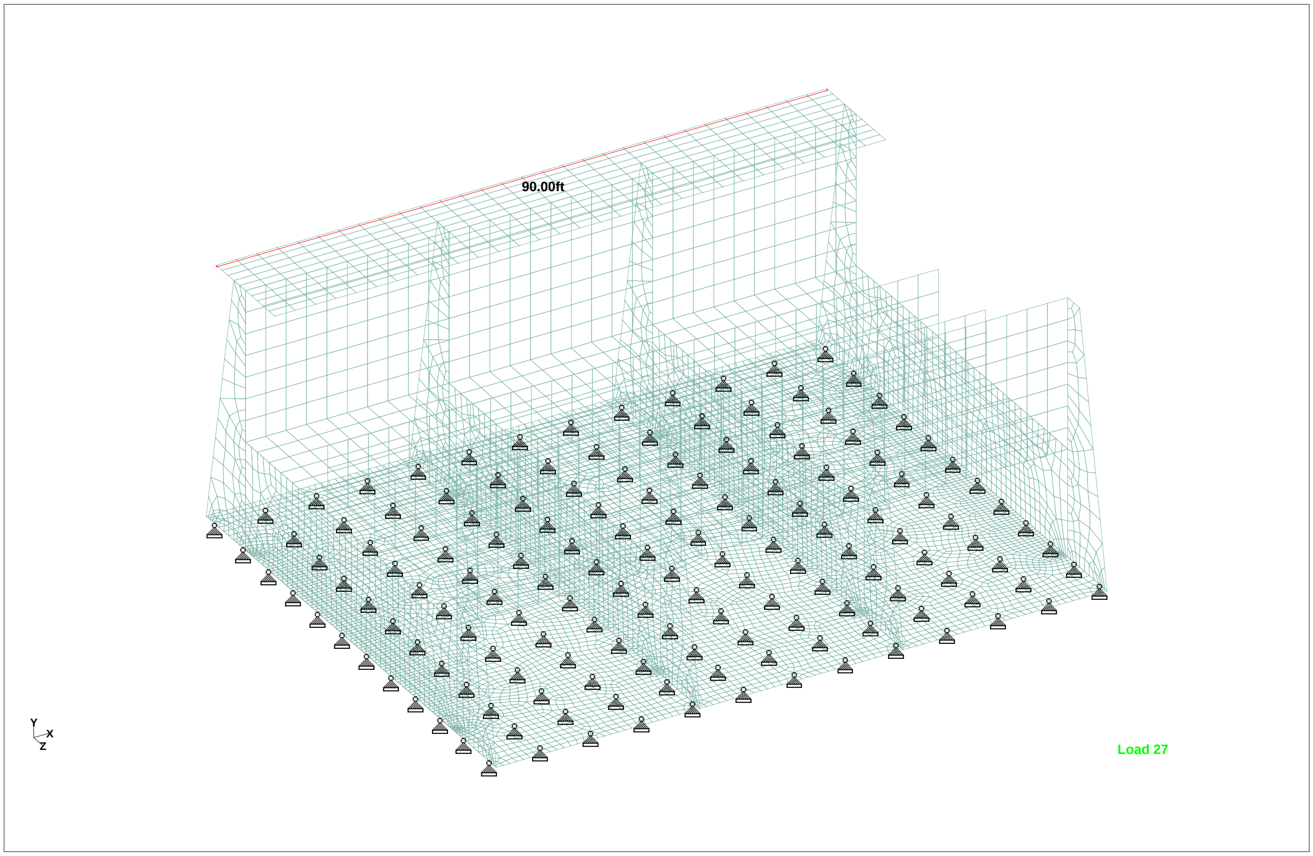
All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	3	DEAD LOAD D
Primary	4	LIVE LOAD L
Primary	5	FLUID LOAD F1 (OVERTOPPING DC FUL
Primary	6	FLUID LOAD F2 (OVERTOPPING TC FUL
Primary	7	FLUID LOAD F3 (500YR+ICE)
Primary	8	SOIL LOAD H
Primary	9	FLUID LOAD F4 (LOWFLOW CHANNEL F
Primary	24	1.3(1.4D+1.7L)
Primary	27	1.3(1.4D+1.7L+1.7F1)
Primary	32	1.3(1.4D+1.7L+1.7F2)
Primary	33	1.3(1.4D+1.7L+1.7F3)
Primary	34	1.3(1.4D+1.7L+1.7F4)
Primary	37	1.3(1.4D+1.7L+1.7F1&F2)
Primary	42	1.3(1.4D+1.7L+1.7H)
Primary	45	1.3(1.4D+1.7L+1.7F1+1.7H)
Primary	47	1.3(1.4D+1.7L+1.7F2+1.7H)
Primary	50	1.3(1.4D+1.7L+1.7F3+1.7H)
Primary	53	1.3(1.4D+1.7L+1.7F4+1.7H)
Primary	55	1.3(1.4D+1.7L+1.7 F1&F2+1.7H)

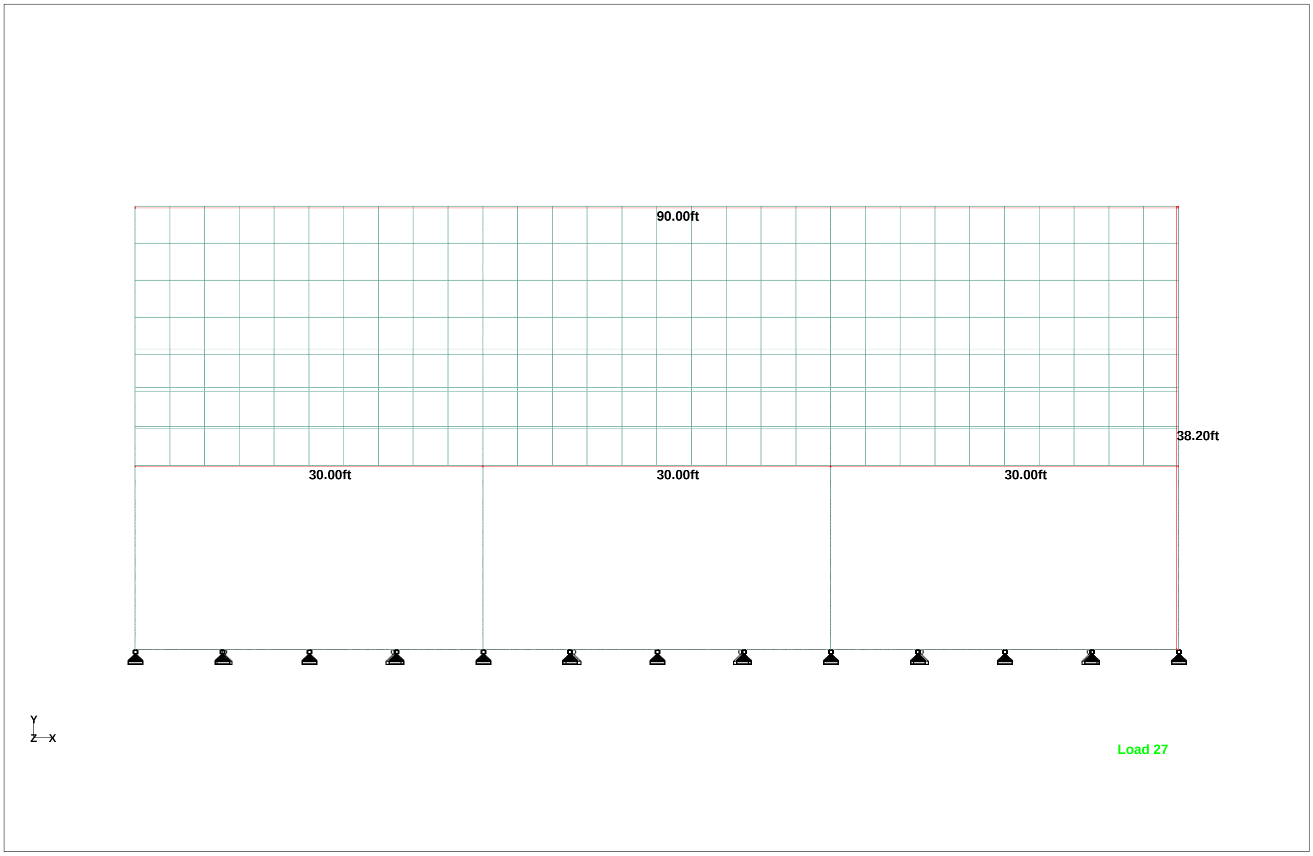
Materials

Mat	Name	E (kip/in^2)	ν	Density (kip/in^3)	α (/°F)
1	STEEL	29E+3	0.300	0.000	6E -6
2	STAINLESSSTEEL	28E+3	0.300	0.000	10E -6
3	ALUMINUM	10E+3	0.330	0.000	13E -6
4	CONCRETE	3.15E+3	0.170	0.000	5E -6

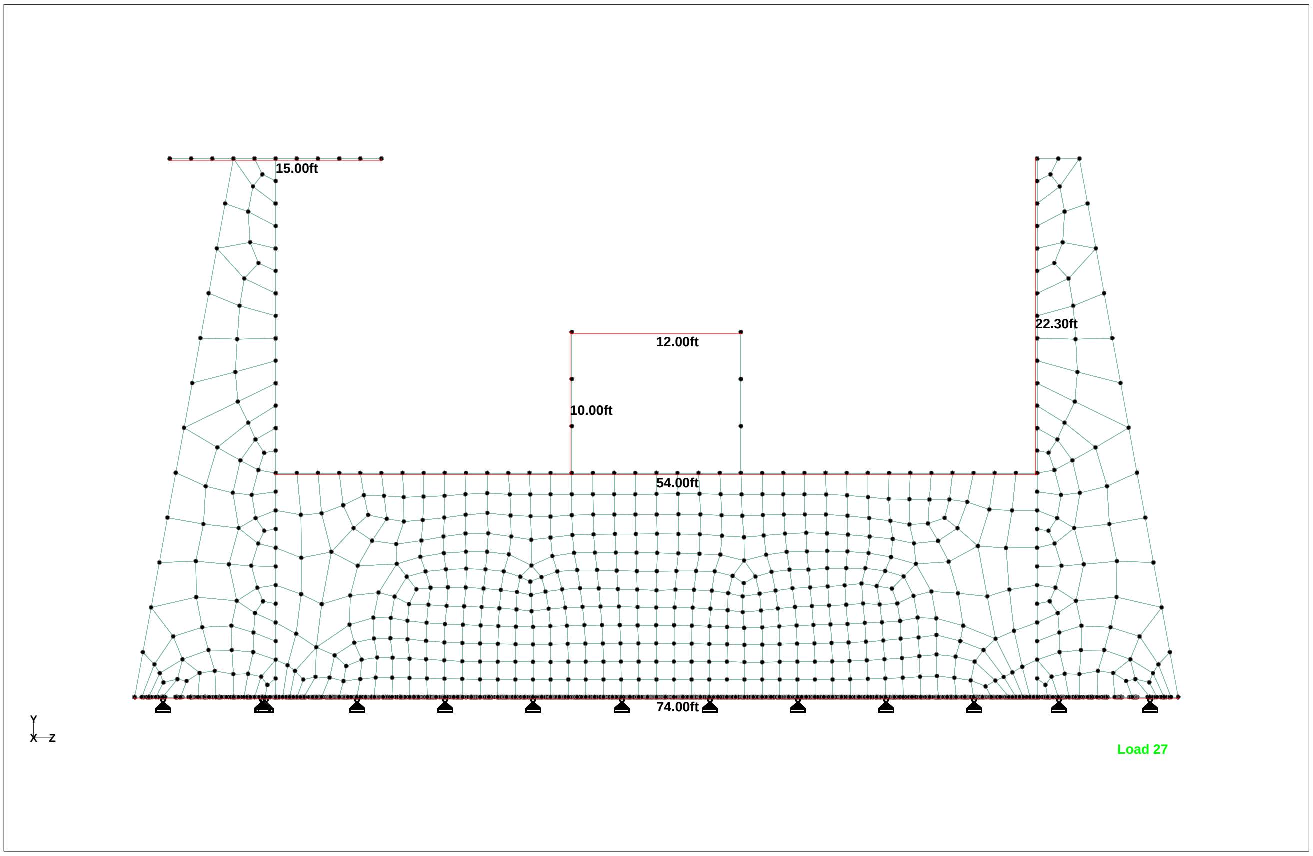


Whole Structure (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft	Job No **34091009**	Sheet No **3**	Rev 2
	Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4	Ref Phase4		
	By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers	File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



Whole Structure - Longitudinal Section (Input data was modified after picture taken)



Whole Structure - Transverse Section (Input data was modified after picture taken)

Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip˙ft/deg)	rY (kip˙ft/deg)	rZ (kip˙ft/deg)
186	Fixed	Fixed	Fixed	-	-	-
190	Fixed	Fixed	Fixed	-	-	-
242	Fixed	Fixed	Fixed	-	-	-
246	Fixed	Fixed	Fixed	-	-	-
298	Fixed	Fixed	Fixed	-	-	-
302	Fixed	Fixed	Fixed	-	-	-
354	Fixed	Fixed	Fixed	-	-	-
358	Fixed	Fixed	Fixed	-	-	-
678	Fixed	Fixed	Fixed	-	-	-
726	Fixed	Fixed	Fixed	-	-	-
774	Fixed	Fixed	Fixed	-	-	-
822	Fixed	Fixed	Fixed	-	-	-
2307	Fixed	Fixed	Fixed	-	-	-
2312	Fixed	Fixed	Fixed	-	-	-
2314	Fixed	Fixed	Fixed	-	-	-
2364	Fixed	Fixed	Fixed	-	-	-

Supports Cont...

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip˙ft/deg)	rY (kip˙ft/deg)	rZ (kip˙ft/deg)
2366	Fixed	Fixed	Fixed	-	-	-
2371	Fixed	Fixed	Fixed	-	-	-
6973	Fixed	Fixed	Fixed	-	-	-
6978	Fixed	Fixed	Fixed	-	-	-
6983	Fixed	Fixed	Fixed	-	-	-
6988	Fixed	Fixed	Fixed	-	-	-
7488	Fixed	Fixed	Fixed	-	-	-
7587	Fixed	Fixed	Fixed	-	-	-
7661	Fixed	Fixed	Fixed	-	-	-
7772	Fixed	Fixed	Fixed	-	-	-
7894	Fixed	Fixed	Fixed	-	-	-
7923	Fixed	Fixed	Fixed	-	-	-
7956	Fixed	Fixed	Fixed	-	-	-
8018	Fixed	Fixed	Fixed	-	-	-
8117	Fixed	Fixed	Fixed	-	-	-
8191	Fixed	Fixed	Fixed	-	-	-
8302	Fixed	Fixed	Fixed	-	-	-
8424	Fixed	Fixed	Fixed	-	-	-
8453	Fixed	Fixed	Fixed	-	-	-
8486	Fixed	Fixed	Fixed	-	-	-
8645	Fixed	Fixed	Fixed	-	-	-
8717	Fixed	Fixed	Fixed	-	-	-
8826	Fixed	Fixed	Fixed	-	-	-
8972	Fixed	Fixed	Fixed	-	-	-
9556	Fixed	Fixed	Fixed	-	-	-
9557	Fixed	Fixed	Fixed	-	-	-
9558	Fixed	Fixed	Fixed	-	-	-
9559	Fixed	Fixed	Fixed	-	-	-
9574	Fixed	Fixed	Fixed	-	-	-
9683	Fixed	Fixed	Fixed	-	-	-
9767	Fixed	Fixed	Fixed	-	-	-
9849	Fixed	Fixed	Fixed	-	-	-
9904	Fixed	Fixed	Fixed	-	-	-
9905	Fixed	Fixed	Fixed	-	-	-
9906	Fixed	Fixed	Fixed	-	-	-
9907	Fixed	Fixed	Fixed	-	-	-
9908	Fixed	Fixed	Fixed	-	-	-
9909	Fixed	Fixed	Fixed	-	-	-
9910	Fixed	Fixed	Fixed	-	-	-
9911	Fixed	Fixed	Fixed	-	-	-
9912	Fixed	Fixed	Fixed	-	-	-
9913	Fixed	Fixed	Fixed	-	-	-
9914	Fixed	Fixed	Fixed	-	-	-
9916	Fixed	Fixed	Fixed	-	-	-
9917	Fixed	Fixed	Fixed	-	-	-

Supports Cont...

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip˙ft/deg)	rY (kip˙ft/deg)	rZ (kip˙ft/deg)
9918	Fixed	Fixed	Fixed	-	-	-
9919	Fixed	Fixed	Fixed	-	-	-
9920	Fixed	Fixed	Fixed	-	-	-
9921	Fixed	Fixed	Fixed	-	-	-
9922	Fixed	Fixed	Fixed	-	-	-
9923	Fixed	Fixed	Fixed	-	-	-
9924	Fixed	Fixed	Fixed	-	-	-
9925	Fixed	Fixed	Fixed	-	-	-
9926	Fixed	Fixed	Fixed	-	-	-
9928	Fixed	Fixed	Fixed	-	-	-
9929	Fixed	Fixed	Fixed	-	-	-
9930	Fixed	Fixed	Fixed	-	-	-
9931	Fixed	Fixed	Fixed	-	-	-
9932	Fixed	Fixed	Fixed	-	-	-
9933	Fixed	Fixed	Fixed	-	-	-
9934	Fixed	Fixed	Fixed	-	-	-
9935	Fixed	Fixed	Fixed	-	-	-
9936	Fixed	Fixed	Fixed	-	-	-
9937	Fixed	Fixed	Fixed	-	-	-
9938	Fixed	Fixed	Fixed	-	-	-
9940	Fixed	Fixed	Fixed	-	-	-
9941	Fixed	Fixed	Fixed	-	-	-
9942	Fixed	Fixed	Fixed	-	-	-
9943	Fixed	Fixed	Fixed	-	-	-
9944	Fixed	Fixed	Fixed	-	-	-
9945	Fixed	Fixed	Fixed	-	-	-
9946	Fixed	Fixed	Fixed	-	-	-
9947	Fixed	Fixed	Fixed	-	-	-
9948	Fixed	Fixed	Fixed	-	-	-
9949	Fixed	Fixed	Fixed	-	-	-
9950	Fixed	Fixed	Fixed	-	-	-
9952	Fixed	Fixed	Fixed	-	-	-
9953	Fixed	Fixed	Fixed	-	-	-
9954	Fixed	Fixed	Fixed	-	-	-
9955	Fixed	Fixed	Fixed	-	-	-
9956	Fixed	Fixed	Fixed	-	-	-
9957	Fixed	Fixed	Fixed	-	-	-
9958	Fixed	Fixed	Fixed	-	-	-
9959	Fixed	Fixed	Fixed	-	-	-
9960	Fixed	Fixed	Fixed	-	-	-
9961	Fixed	Fixed	Fixed	-	-	-
9962	Fixed	Fixed	Fixed	-	-	-
9964	Fixed	Fixed	Fixed	-	-	-
9965	Fixed	Fixed	Fixed	-	-	-
9966	Fixed	Fixed	Fixed	-	-	-

Supports Cont...

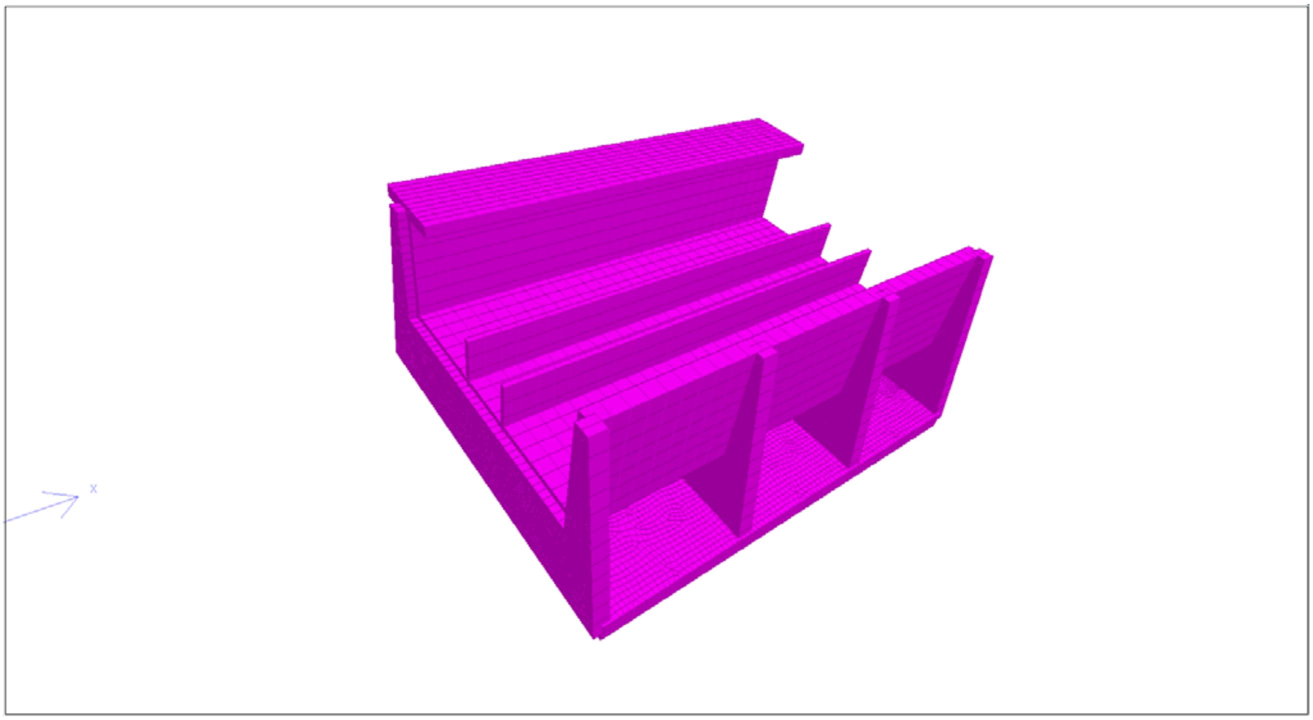
Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip˙ft/deg)	rY (kip˙ft/deg)	rZ (kip˙ft/deg)
9967	Fixed	Fixed	Fixed	-	-	-
9968	Fixed	Fixed	Fixed	-	-	-
9969	Fixed	Fixed	Fixed	-	-	-
9970	Fixed	Fixed	Fixed	-	-	-
9971	Fixed	Fixed	Fixed	-	-	-
9972	Fixed	Fixed	Fixed	-	-	-
9973	Fixed	Fixed	Fixed	-	-	-
9974	Fixed	Fixed	Fixed	-	-	-
9976	Fixed	Fixed	Fixed	-	-	-
9977	Fixed	Fixed	Fixed	-	-	-
9978	Fixed	Fixed	Fixed	-	-	-
9979	Fixed	Fixed	Fixed	-	-	-
9980	Fixed	Fixed	Fixed	-	-	-
9981	Fixed	Fixed	Fixed	-	-	-
9982	Fixed	Fixed	Fixed	-	-	-
9983	Fixed	Fixed	Fixed	-	-	-
9984	Fixed	Fixed	Fixed	-	-	-
9985	Fixed	Fixed	Fixed	-	-	-
9986	Fixed	Fixed	Fixed	-	-	-
9988	Fixed	Fixed	Fixed	-	-	-
9989	Fixed	Fixed	Fixed	-	-	-
9990	Fixed	Fixed	Fixed	-	-	-
9991	Fixed	Fixed	Fixed	-	-	-
9992	Fixed	Fixed	Fixed	-	-	-
9993	Fixed	Fixed	Fixed	-	-	-
9994	Fixed	Fixed	Fixed	-	-	-
9995	Fixed	Fixed	Fixed	-	-	-
9996	Fixed	Fixed	Fixed	-	-	-
9997	Fixed	Fixed	Fixed	-	-	-
9998	Fixed	Fixed	Fixed	-	-	-
10000	Fixed	Fixed	Fixed	-	-	-
10001	Fixed	Fixed	Fixed	-	-	-
10002	Fixed	Fixed	Fixed	-	-	-
10003	Fixed	Fixed	Fixed	-	-	-
10004	Fixed	Fixed	Fixed	-	-	-
10005	Fixed	Fixed	Fixed	-	-	-
10006	Fixed	Fixed	Fixed	-	-	-
10007	Fixed	Fixed	Fixed	-	-	-
10008	Fixed	Fixed	Fixed	-	-	-
10009	Fixed	Fixed	Fixed	-	-	-
10010	Fixed	Fixed	Fixed	-	-	-
11533	Fixed	Fixed	Fixed	-	-	-
12640	Fixed	Fixed	Fixed	-	-	-
13521	Fixed	Fixed	Fixed	-	-	-
13562	Fixed	Fixed	Fixed	-	-	-

Supports Cont...

Node	**X** (kip/in)	**Y** (kip/in)	**Z** (kip/in)	**rX** (kip˙ft/deg)	**rY** (kip˙ft/deg)	**rZ** (kip˙ft/deg)
13915	Fixed	Fixed	Fixed	-	-	-
14085	Fixed	Fixed	Fixed	-	-	-
15692	Fixed	Fixed	Fixed	-	-	-
16287	Fixed	Fixed	Fixed	-	-	-
16500	Fixed	Fixed	Fixed	-	-	-

Plate Thickness

Prop	**Node A** (in)	**Node B** (in)	**Node C** (in)	**Node D** (in)	**Material**
1	27.000	27.000	27.000	27.000	CONCRETE
2	36.000	36.000	36.000	36.000	CONCRETE
3	12.000	12.000	12.000	12.000	CONCRETE
4	32.000	32.000	32.000	32.000	CONCRETE
5	48.000	48.000	48.000	48.000	CONCRETE



3D Rendered View (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft	Job No **34091009**	Sheet No **9**	Rev 2
	Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4	Ref Phase4		
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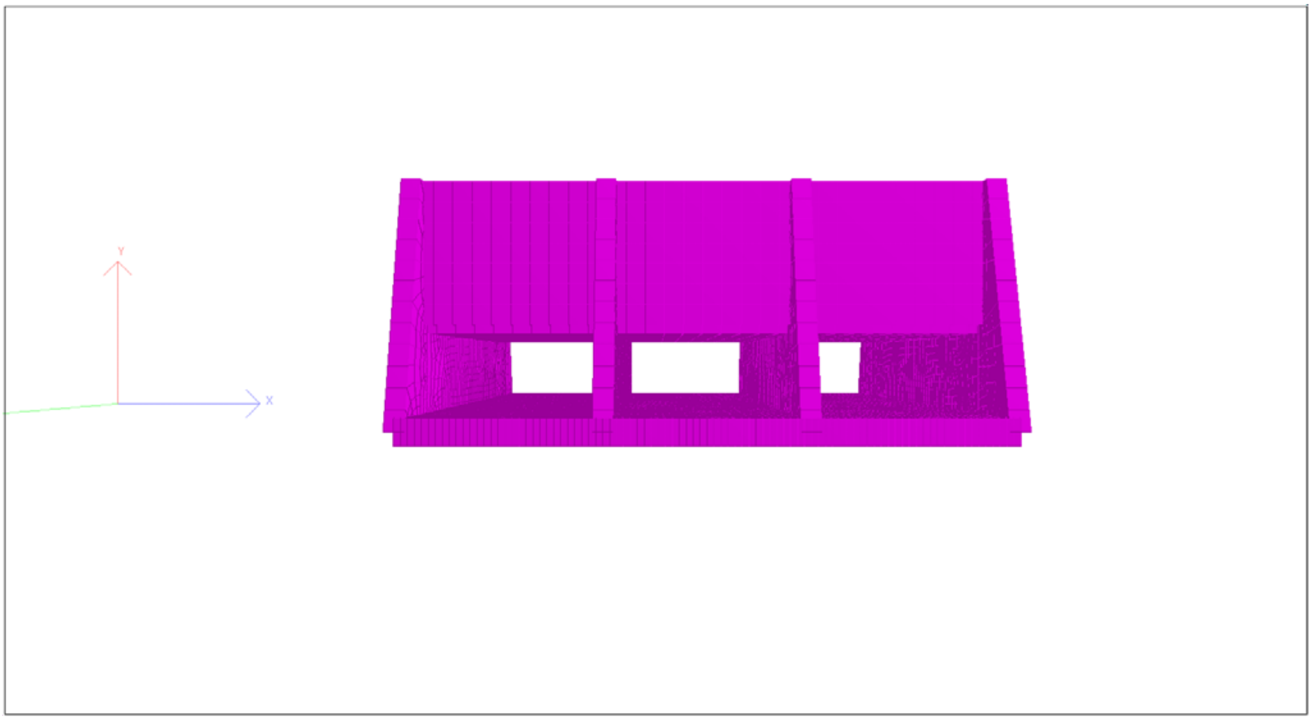
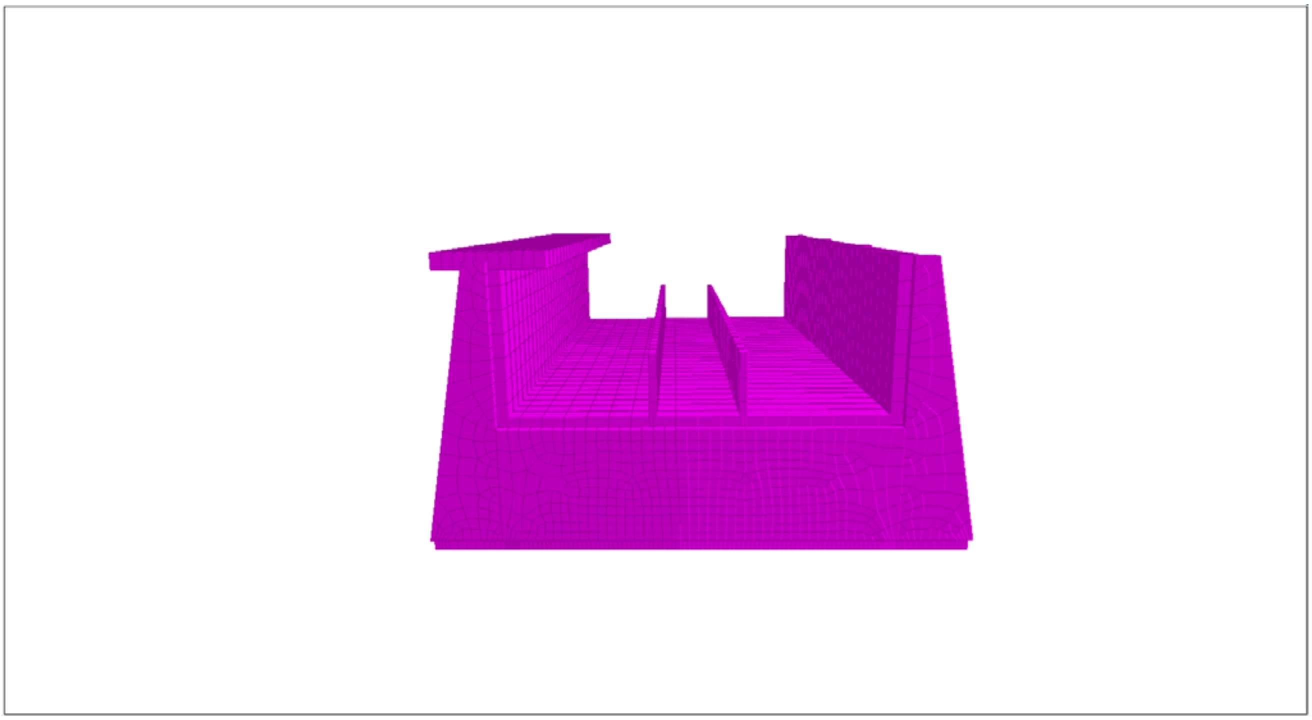
Client US Army Corps of Engineers	File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



3D Rendered View - Longitudinal Section (Input data was modified after picture taken)



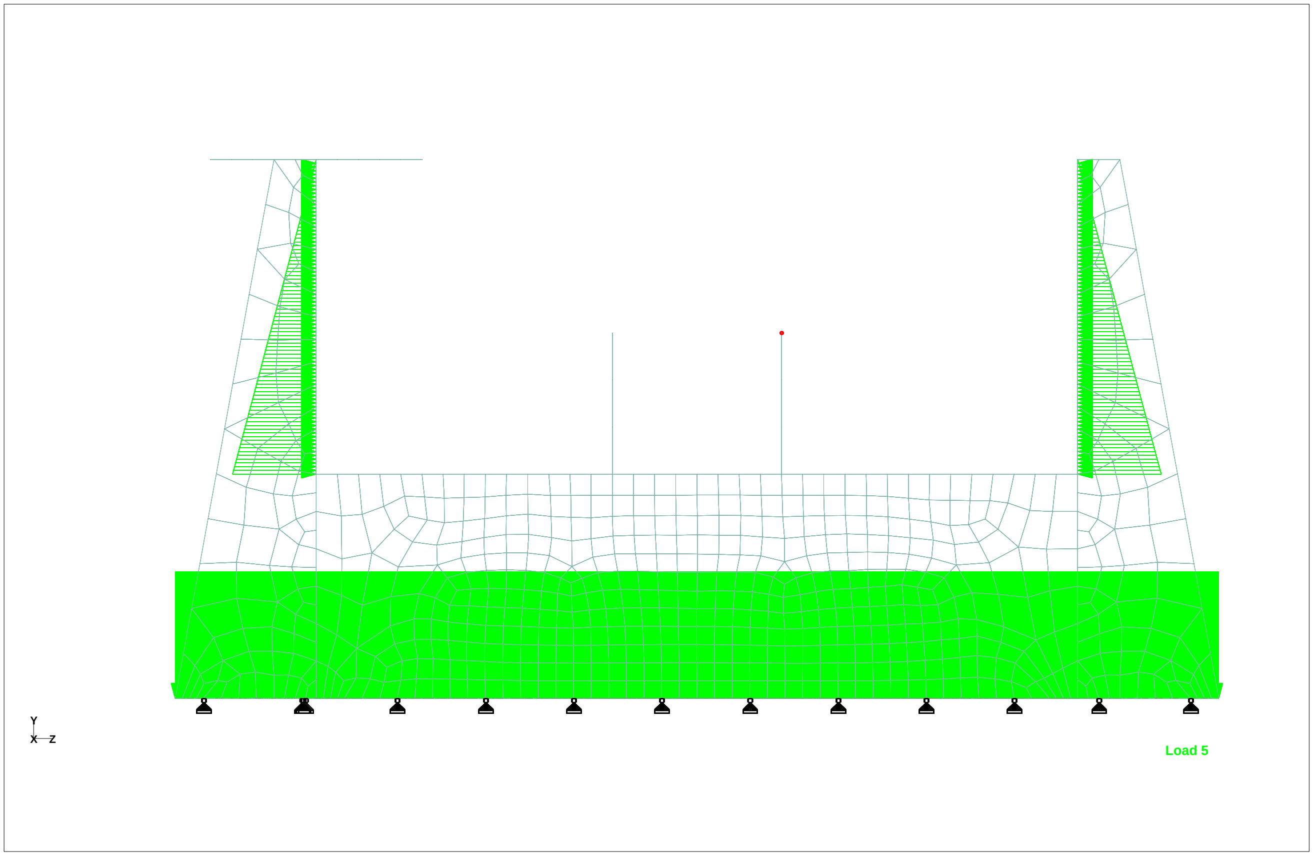
3D Rendered View - Tranverse Section (Input data was modified after picture taken)

Basic Load Cases

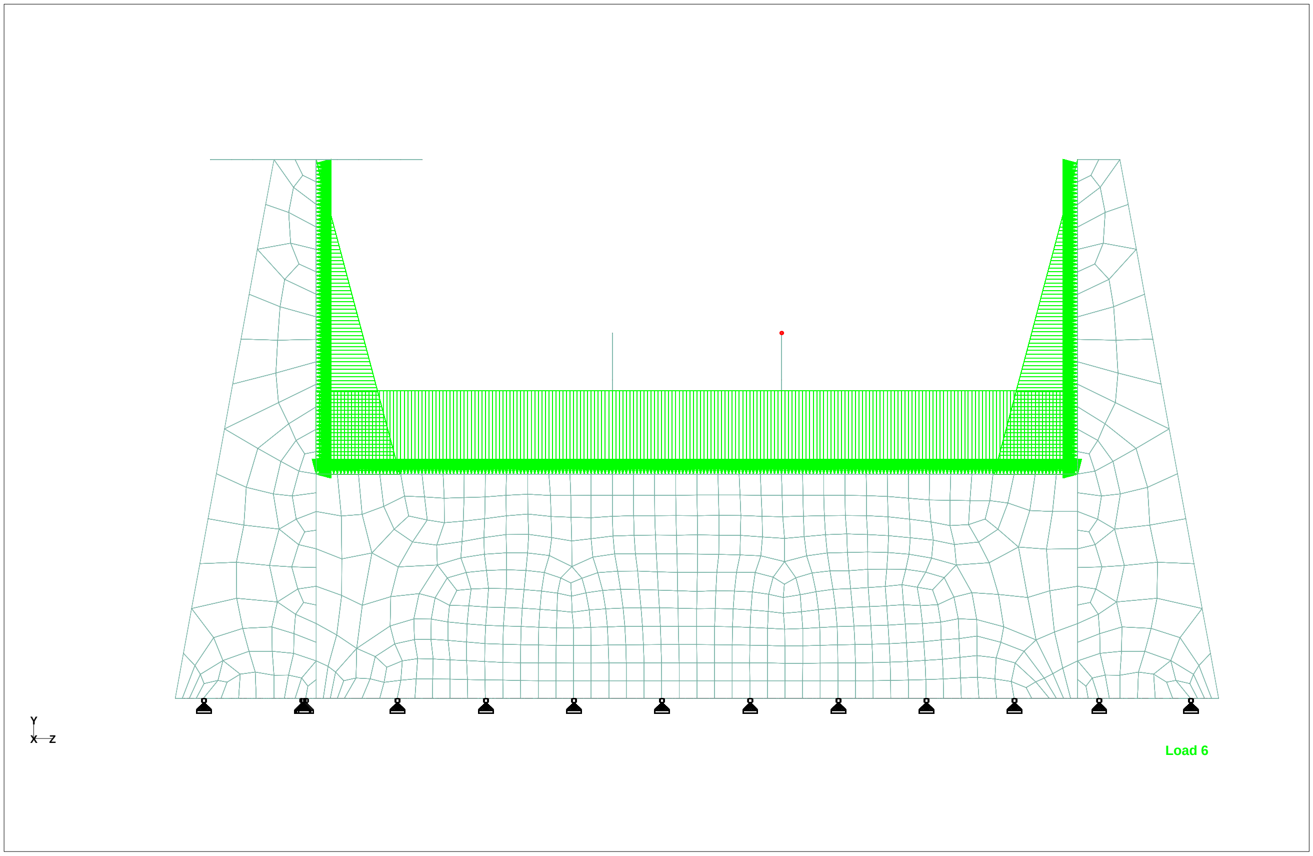
Number	Name
3	DEAD LOAD D
4	LIVE LOAD L
5	FLUID LOAD F1 (OVERTOPPING DC FUL
6	FLUID LOAD F2 (OVERTOPPING TC FUL
7	FLUID LOAD F3 (500YR+ICE)
8	SOIL LOAD H
9	FLUID LOAD F4 (LOWFLOW CHANNEL F
24	1.3(1.4D+1.7L)
27	1.3(1.4D+1.7L+1.7F1)
32	1.3(1.4D+1.7L+1.7F2)
33	1.3(1.4D+1.7L+1.7F3)
34	1.3(1.4D+1.7L+1.7F4)
37	1.3(1.4D+1.7L+1.7F1&F2)
42	1.3(1.4D+1.7L+1.7H)
45	1.3(1.4D+1.7L+1.7F1+1.7H)
47	1.3(1.4D+1.7L+1.7F2+1.7H)
50	1.3(1.4D+1.7L+1.7F3+1.7H)
53	1.3(1.4D+1.7L+1.7F4+1.7H)
55	1.3(1.4D+1.7L+1.7 F1&F2+1.7H)

Combination Load Cases

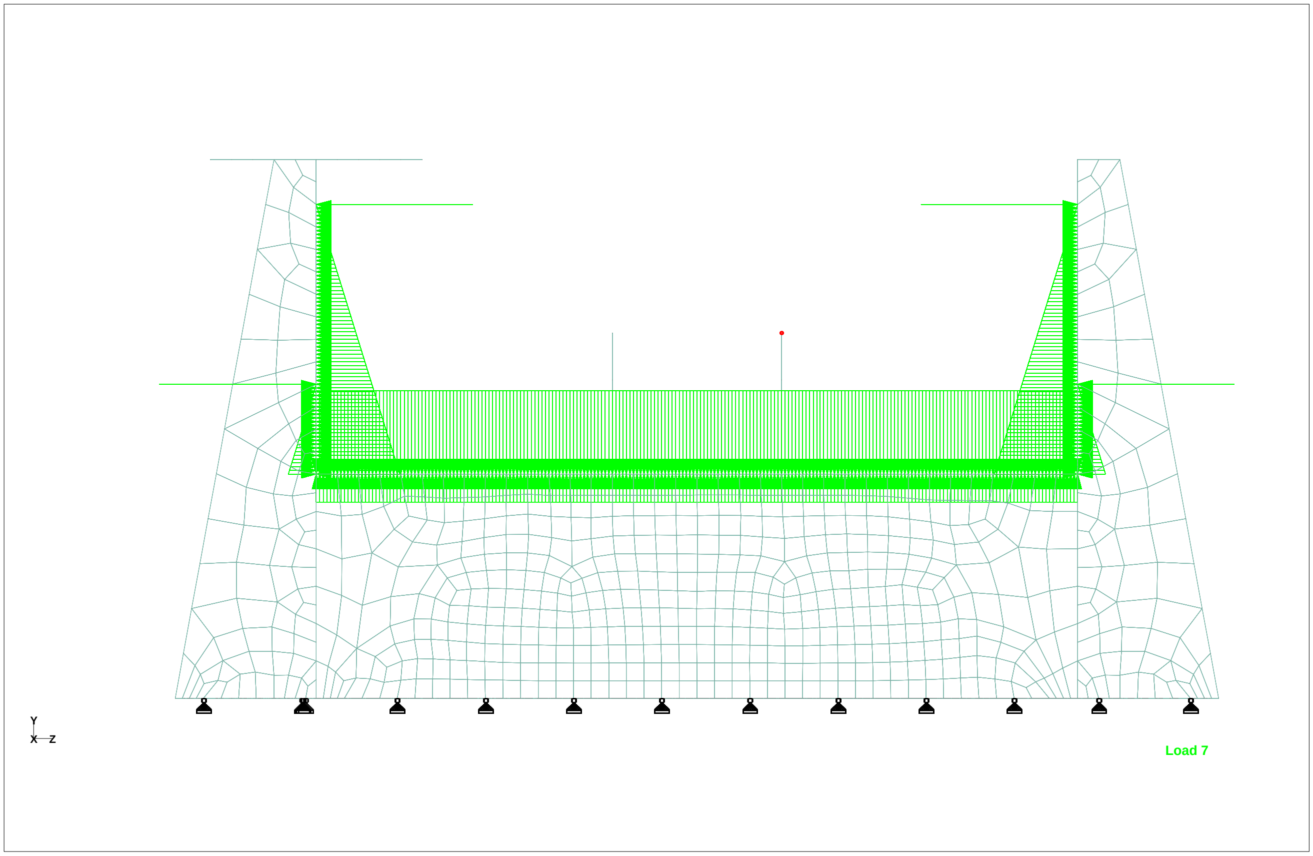
There is no data of this type.



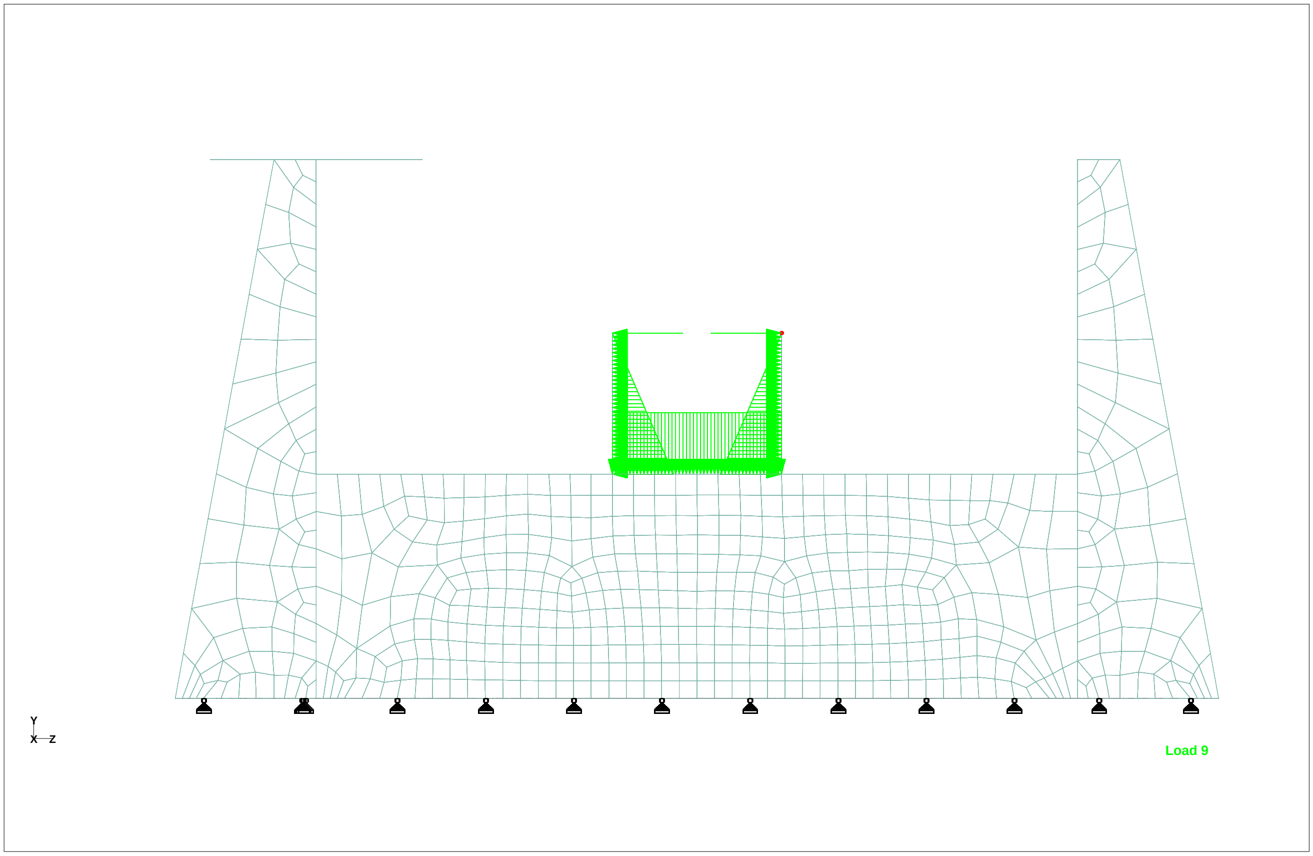
Whole Structure Loads LC#5 FLUID LOAD F1 (OVERTOPPING DC FULL) (Input data was modified after picture taken)



Whole Structure Loads LC#6 FLUID LOAD F2 (OVERTOPPING TC FULL) (Input data was modified after picture taken)



Whole Structure Loads LC#7 FLUID LOAD F3 (500YR+ICE) (Input data was modified after picture taken)



Whole Structure Loads LC#9 FLUID LOAD F4 (LOWFLOW CHANNEL FULL W/ICE) (Input data was modified after picture taken)

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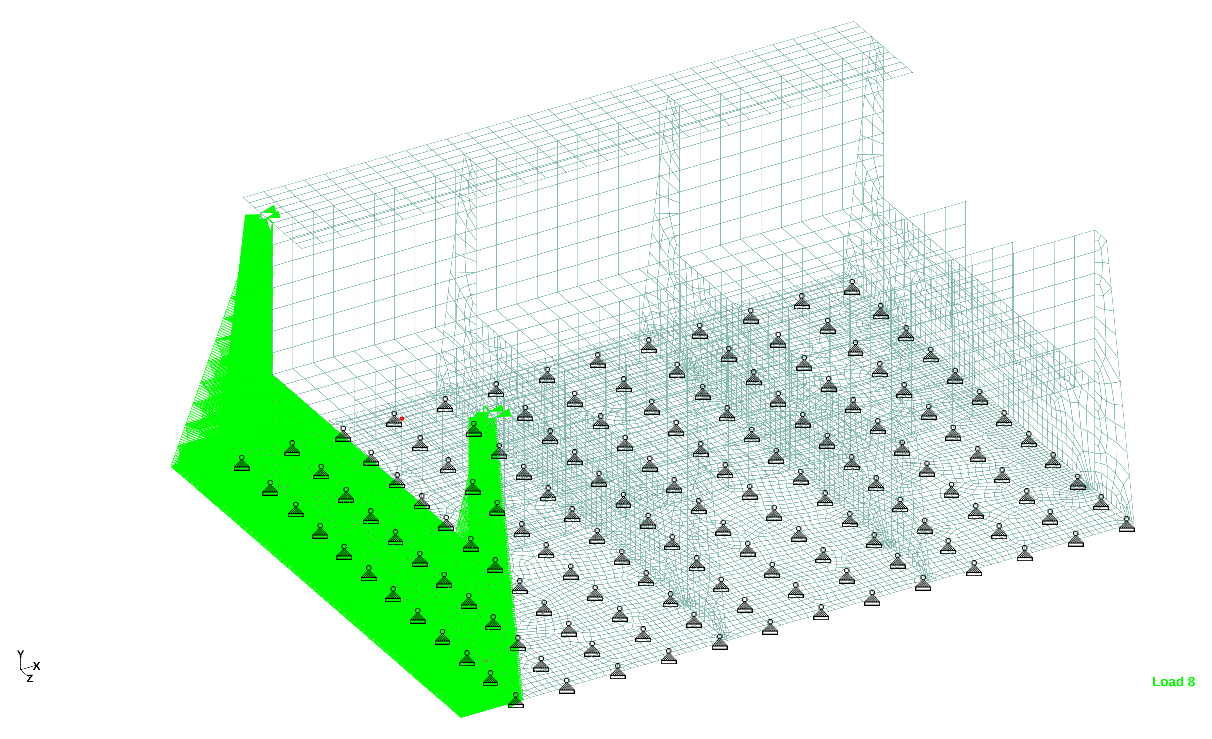

Whole Structure Loads LC#8 SOIL LOAD H (Input data was modified after picture taken)

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	9370	55:1.3(1.4D+1.	**0.067**	-0.032	-0.008	0.075	-0.000	0.000	-0.000
Min X	9373	37:1.3(1.4D+1.	**-0.031**	-0.032	-0.008	0.045	-0.000	-0.000	0.000
Max Y	9091	27:1.3(1.4D+1.	-0.010	**0.037**	0.225	0.228	0.001	0.000	0.000
Min Y	9074	27:1.3(1.4D+1.	-0.002	**-0.154**	0.253	0.296	0.001	-0.000	0.000
Max Z	9371	53:1.3(1.4D+1.	0.046	-0.011	**1.071**	1.072	0.013	0.001	-0.000
Min Z	9370	53:1.3(1.4D+1.	0.046	-0.012	**-1.074**	1.075	-0.013	-0.001	-0.000
Max rX	9371	53:1.3(1.4D+1.	0.046	-0.011	1.071	1.072	**0.013**	0.001	-0.000
Min rX	9370	53:1.3(1.4D+1.	0.046	-0.012	-1.074	1.075	**-0.013**	-0.001	-0.000
Max rY	9371	53:1.3(1.4D+1.	0.046	-0.011	1.071	1.072	0.013	**0.001**	-0.000
Min rY	9372	34:1.3(1.4D+1.	-0.009	-0.011	1.071	1.071	0.013	**-0.001**	0.000
Max rZ	1498	37:1.3(1.4D+1.	0.001	-0.040	-0.000	0.040	-0.000	-0.000	**0.001**
Min rZ	1470	55:1.3(1.4D+1.	0.037	-0.040	-0.001	0.055	-0.000	0.000	**-0.001**
Max Rst	9370	53:1.3(1.4D+1.	0.046	-0.012	-1.074	**1.075**	-0.013	-0.001	-0.000

Plate Centre Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (psi)	Qy (psi)	Sx (psi)	Sy (psi)	Sxy (psi)	Mx (Kip¯ft/ft)	My (Kip¯ft/ft)	Mxy (Kip¯ft/ft)
Max Qx	13085	55:1.3(1.4D+1.	**304.139**	278.927	-45.353	-17.709	-5.714	-60.833	-53.322	-20.557
Min Qx	16314	37:1.3(1.4D+1.	**-345.812**	226.390	-90.256	-13.919	-1.314	-56.291	-27.658	-25.704
Max Qy	14875	37:1.3(1.4D+1.	201.221	**468.462**	-5.563	-34.894	-19.051	-65.071	-77.953	-15.974
Min Qy	13355	55:1.3(1.4D+1.	168.919	**-421.178**	-16.666	-64.264	-16.616	-75.182	-78.306	-7.321
Max Sx	7302	45:1.3(1.4D+1.	-5.043	-1.222	**430.095**	-3.116	-57.900	-16.099	0.011	1.466
Min Sx	9021	37:1.3(1.4D+1.	-2.195	-2.682	**-827.458**	-262.897	-242.399	3.133	0.788	0.630
Max Sy	7301	45:1.3(1.4D+1.	-0.985	0.976	58.983	**443.315**	181.106	-1.542	-18.779	-6.499
Min Sy	8936	32:1.3(1.4D+1.	-0.672	-1.162	44.753	**-939.464**	150.521	0.332	2.034	-0.125
Max Sxy	8933	32:1.3(1.4D+1.	-0.827	-0.049	-654.309	-187.818	**436.748**	1.501	0.504	-0.630
Min Sxy	8686	37:1.3(1.4D+1.	-0.472	-0.399	-55.953	-174.622	**-424.720**	0.132	0.377	0.173
Max Mx	14951	50:1.3(1.4D+1.	85.131	148.824	17.621	-62.184	28.029	**104.339**	-2.101	1.375
Min Mx	8241	55:1.3(1.4D+1.	26.371	21.812	-116.699	-36.040	-58.880	**-111.728**	-13.127	-45.160
Max My	718	55:1.3(1.4D+1.	2.182	116.785	-6.971	-13.460	6.312	29.416	**142.858**	-2.547
Min My	1178	27:1.3(1.4D+1.	32.598	87.277	-63.202	-387.845	128.829	-17.675	**-130.012**	2.753
Max Mxy	15175	55:1.3(1.4D+1.	-25.779	-19.878	1.021	8.958	23.471	43.864	38.920	**56.657**
Min Mxy	7978	55:1.3(1.4D+1.	5.315	30.499	-25.811	-101.556	-27.764	-13.327	-119.389	**-54.827**

Plate Centre Principal Stress Summary

			Principal		Von Mis		Tresca	
	Plate	L/C	Top (psi)	Bottom (psi)	Top (psi)	Bottom (psi)	Top (psi)	Bottom (psi)
Max (t)	8666	53:1.3(1.4D+1.	**3.48E+3**	531.860	3.25E+3	3.24E+3	3.48E+3	3.51E+3
Max (b)	9085	55:1.3(1.4D+1.	1.14E+3	**-1.14E+3**	1.14E+3	529.589	1.15E+3	594.623
Max VM (t)	8666	53:1.3(1.4D+1.	3.48E+3	531.860	**3.25E+3**	3.24E+3	3.48E+3	3.51E+3
Max VM (b)	8665	53:1.3(1.4D+1.	3.33E+3	364.839	3.16E+3	**3.56E+3**	3.33E+3	3.78E+3
Tresca (t)	8666	53:1.3(1.4D+1.	3.48E+3	531.860	3.25E+3	3.24E+3	**3.48E+3**	3.51E+3
Tresca (b)	8665	53:1.3(1.4D+1.	3.33E+3	364.839	3.16E+3	3.56E+3	3.33E+3	**3.78E+3**

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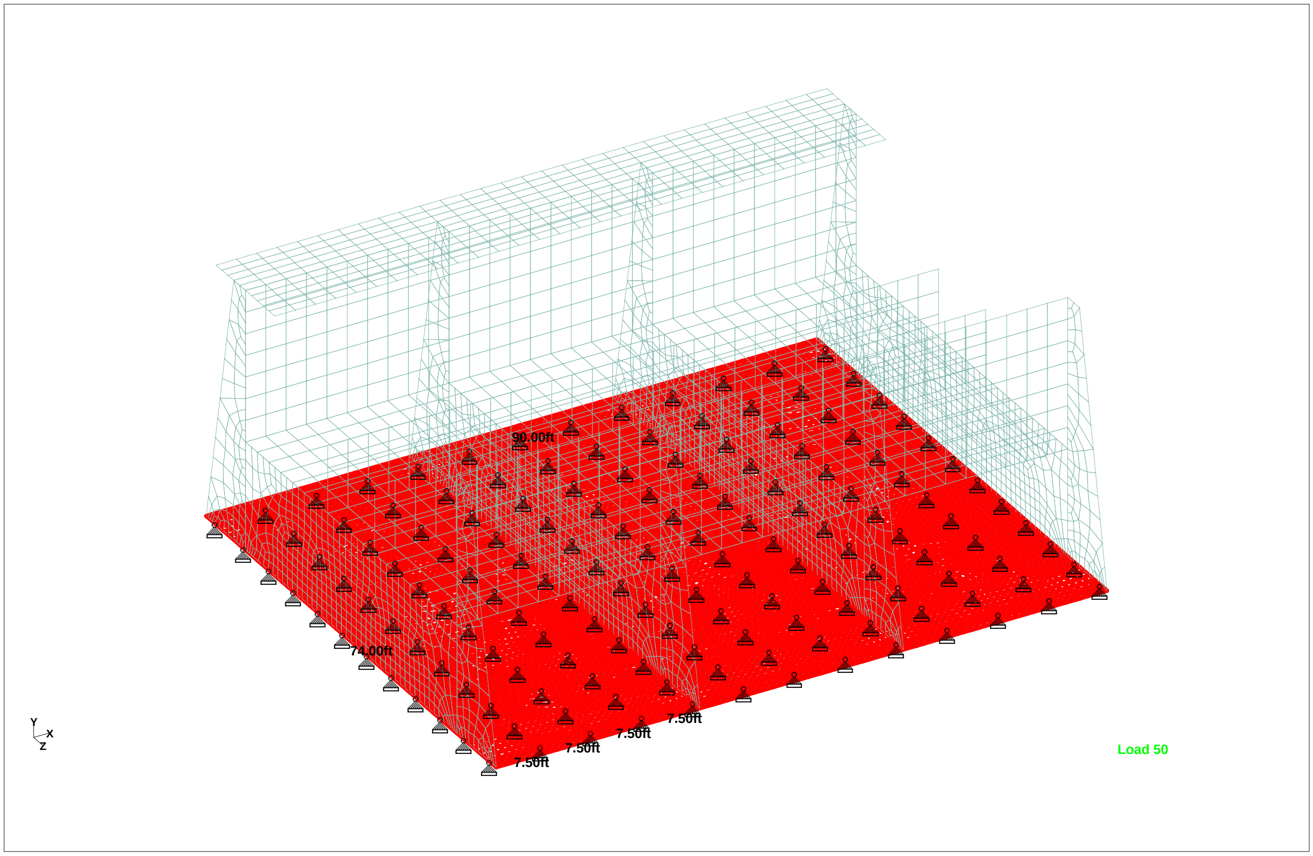
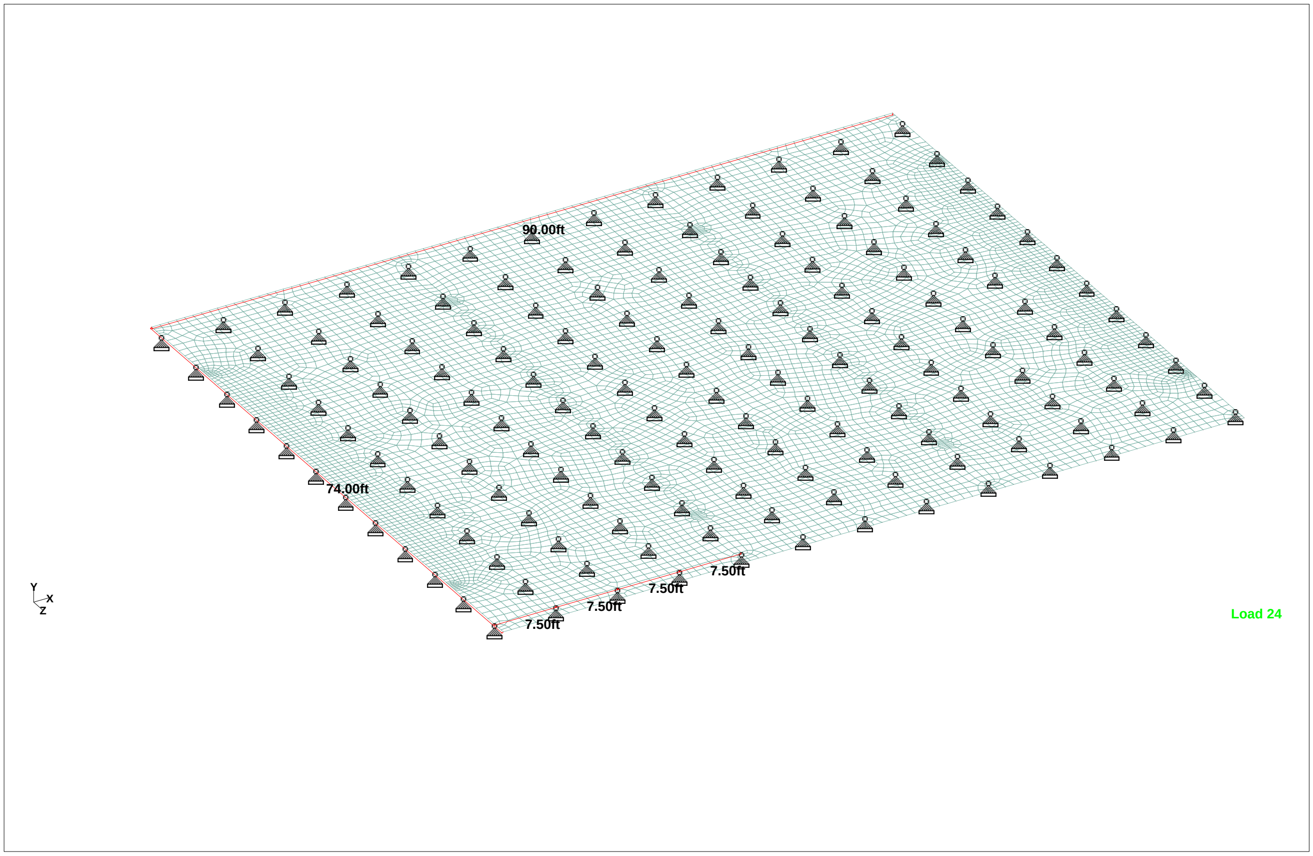


Whole Structure - Aqeuduct Mat Foundation (Input data was modified after picture taken)



Aqeuduct Mat Foundation (Input data was modified after picture taken)

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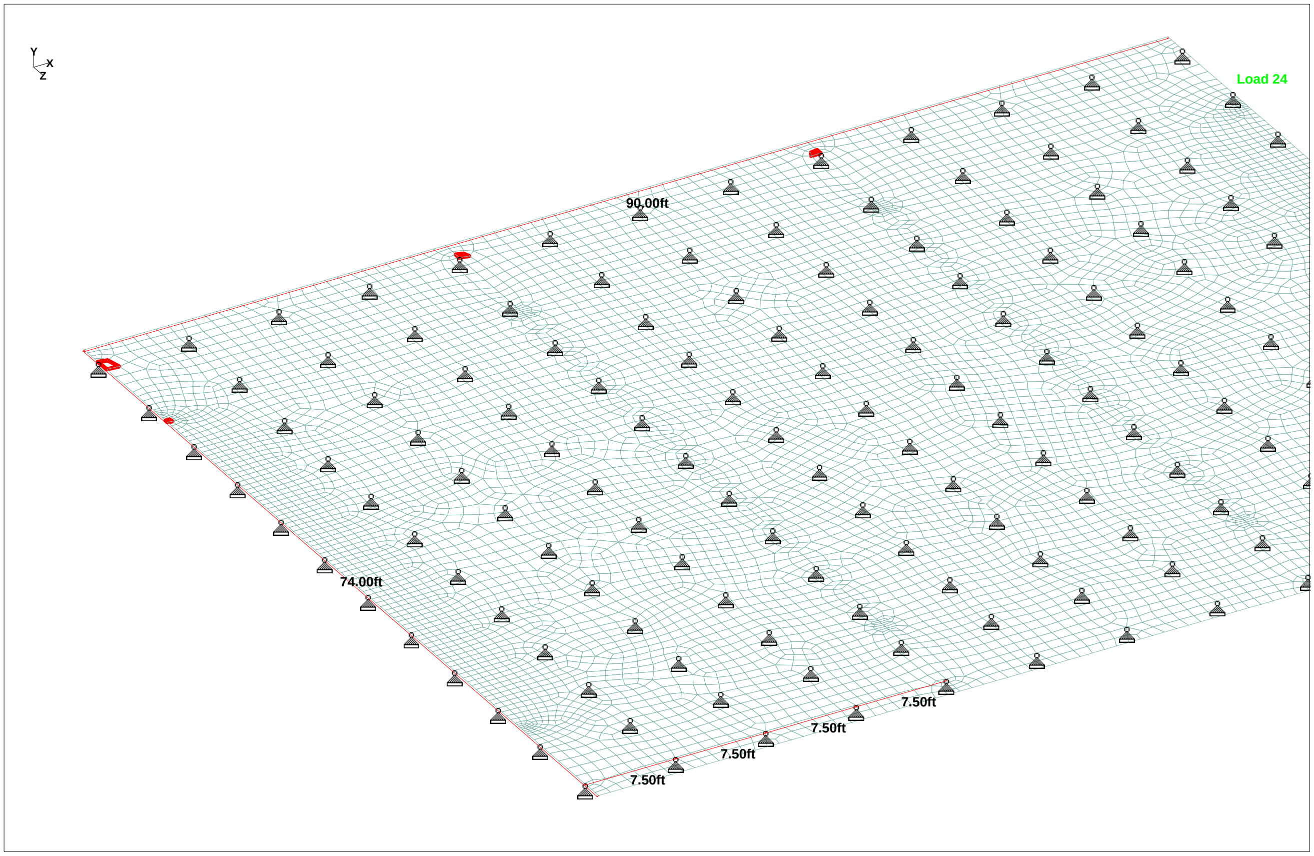
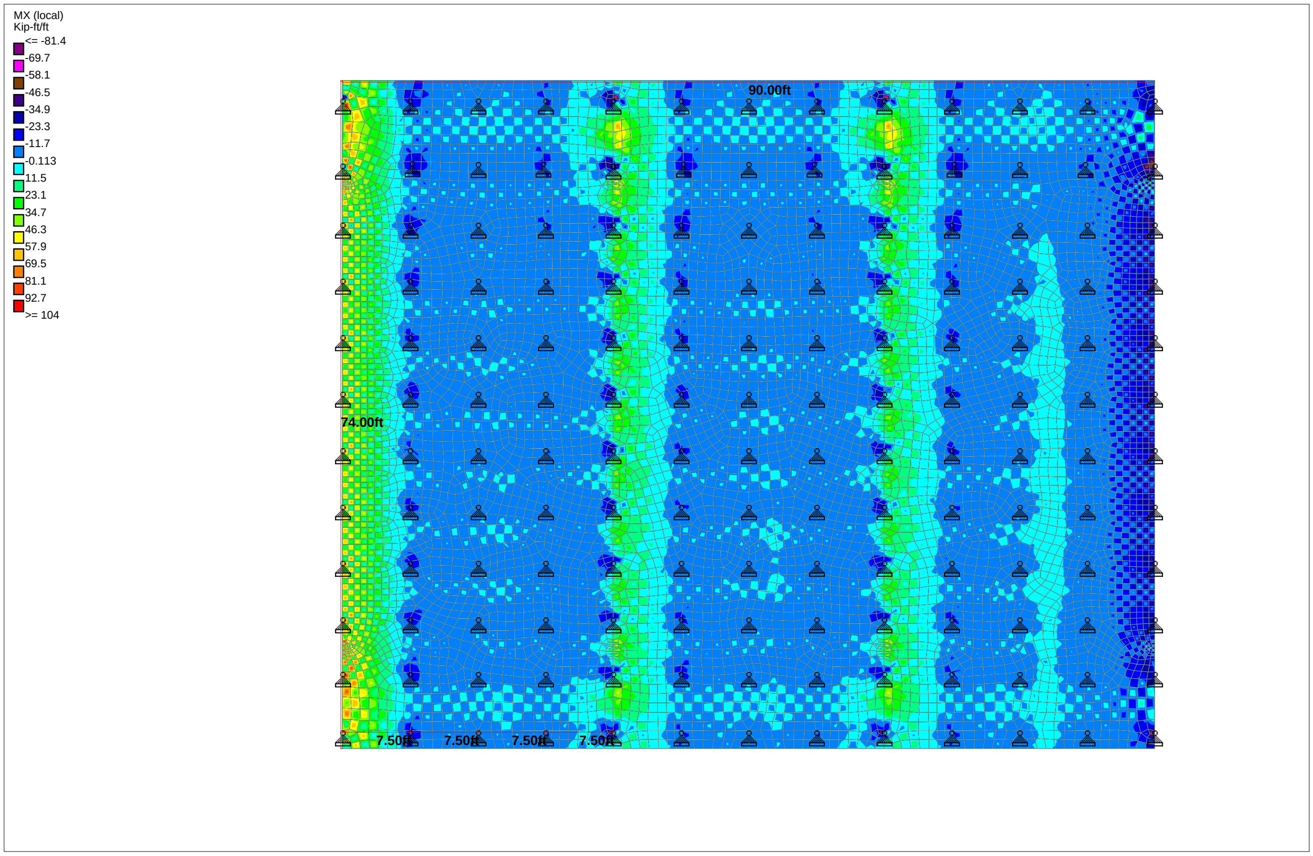


Aqeuduct Mat Foundation - Max Moments Critical Plates (Input data was modified after picture taken)



Aqeuduct Mat Foundation LC#50 Mx (Input data was modified after picture taken)

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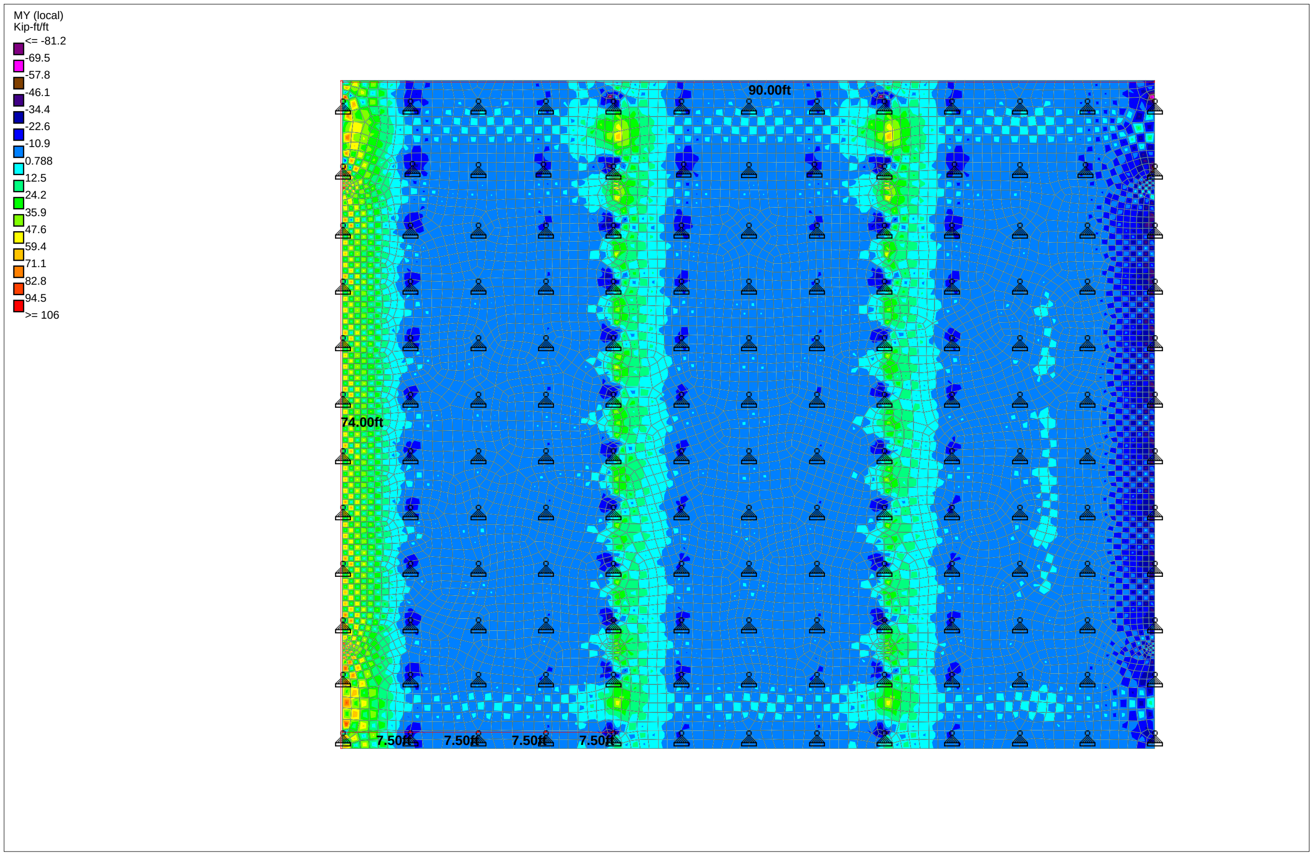

Aqeuduct Mat Foundation LC#50 My (Input data was modified after picture taken)

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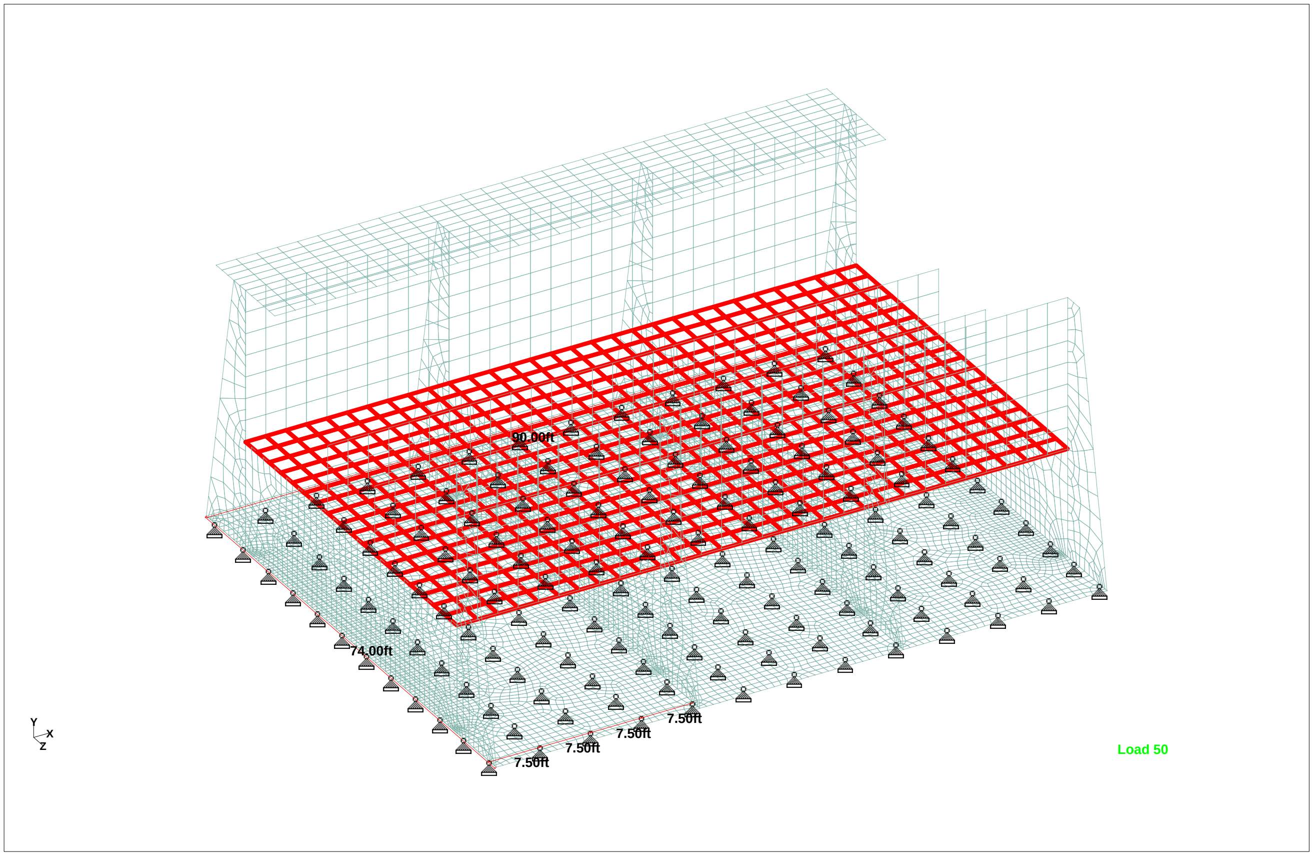
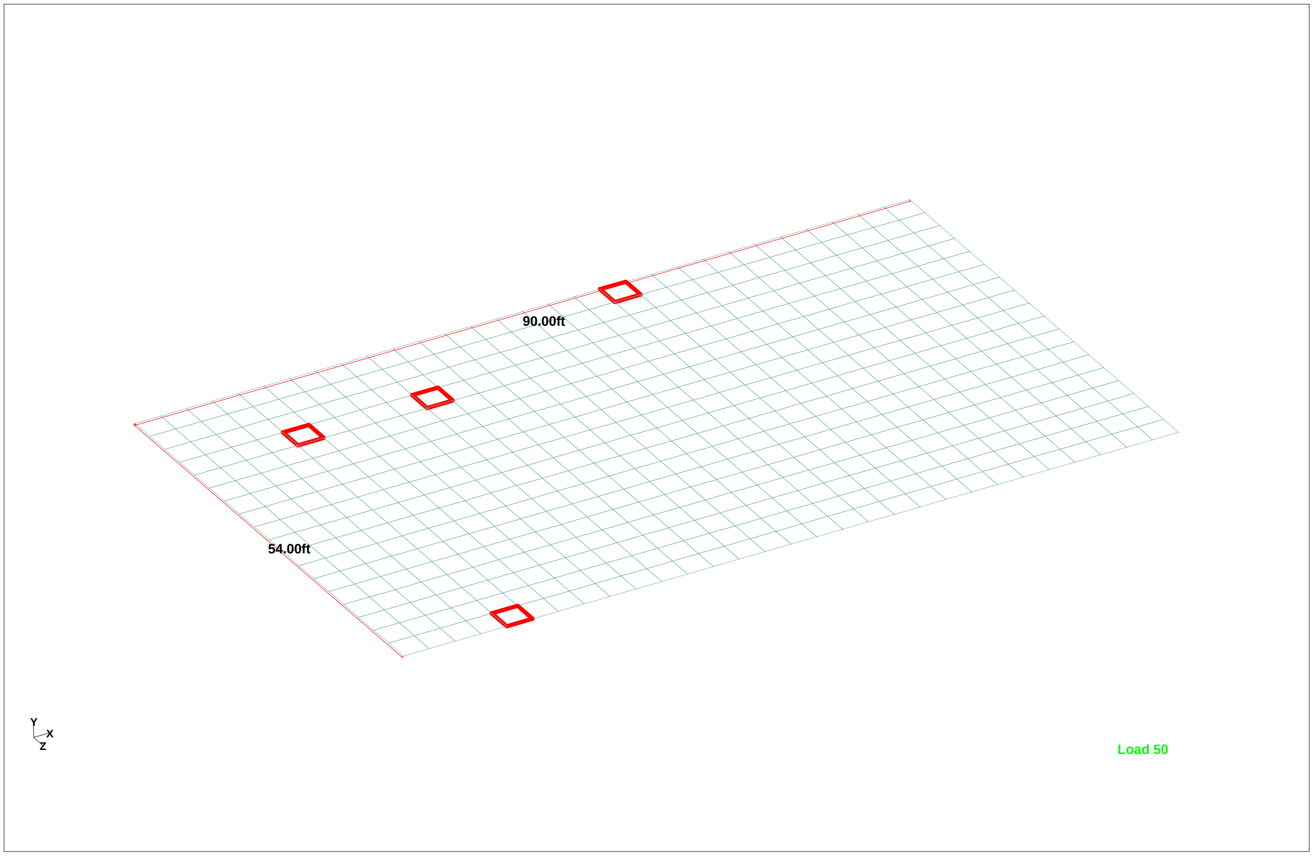


Whole Structure - Tributary Channel Deck (Input data was modified after picture taken)



Tributary Channel Deck - Max Moment Critical Plates (Input data was modified after picture taken)

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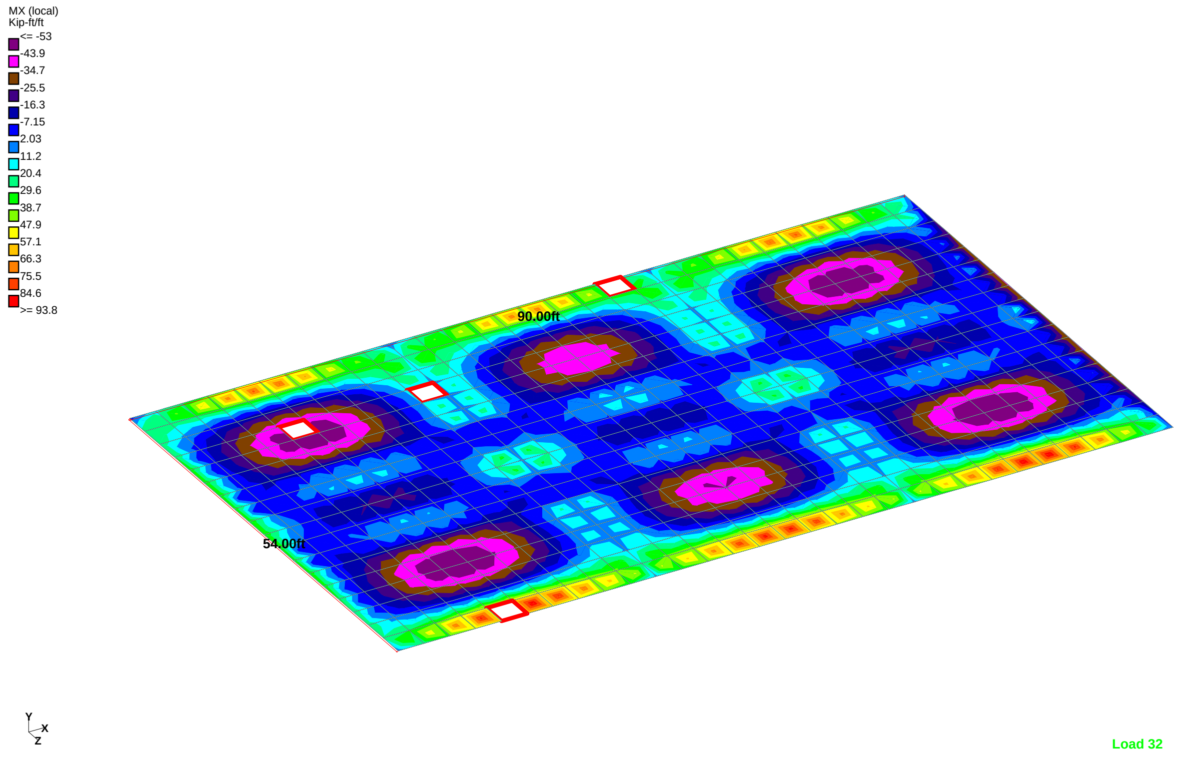
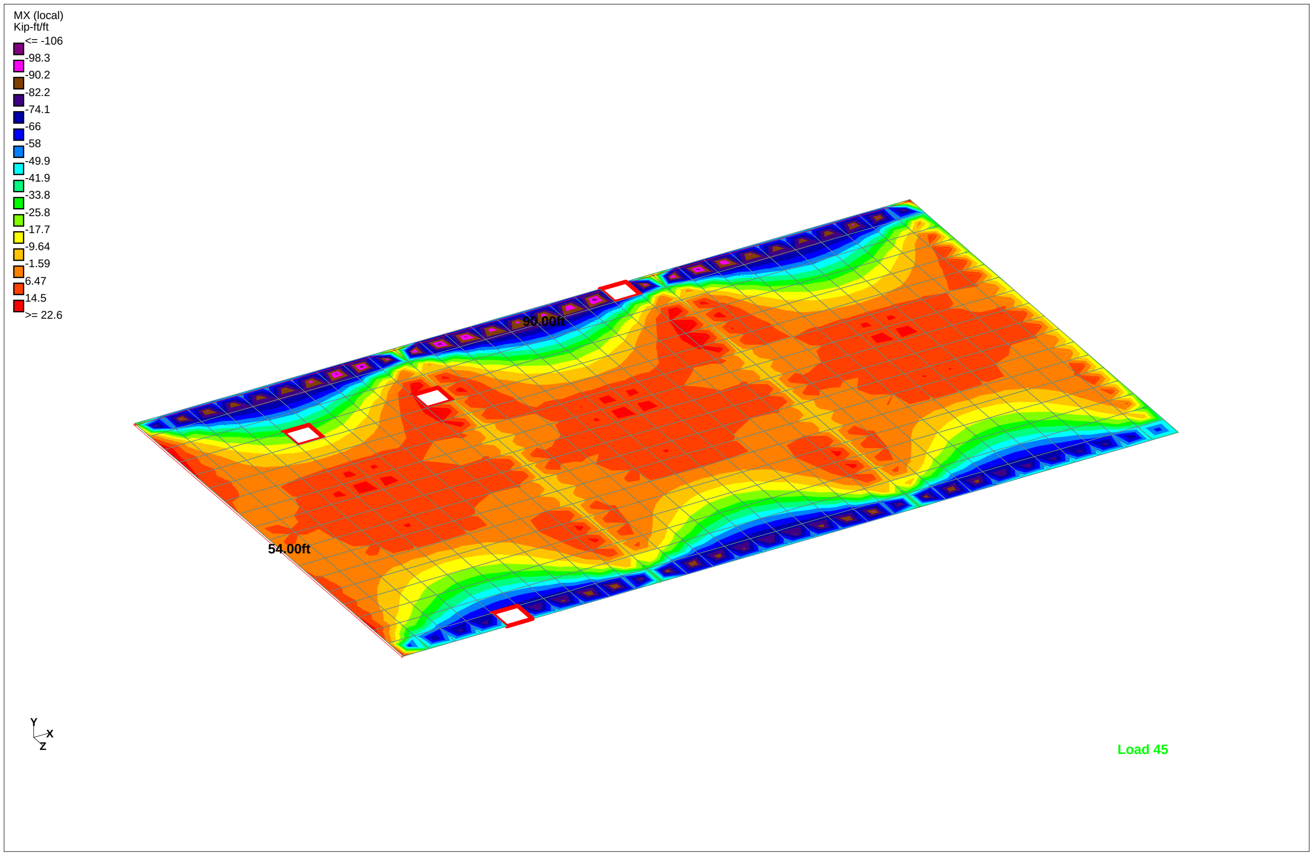


Tributary Channel Deck - LC#32 Mxmax (Input data was modified after picture taken)



Tributary Channel Deck - LC#45 Mxmin (Input data was modified after picture taken)

Barr Engineering

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Job No	Sheet No	Rev
34091009	**26**	2

Part Sheyene Aquaduct - Partial

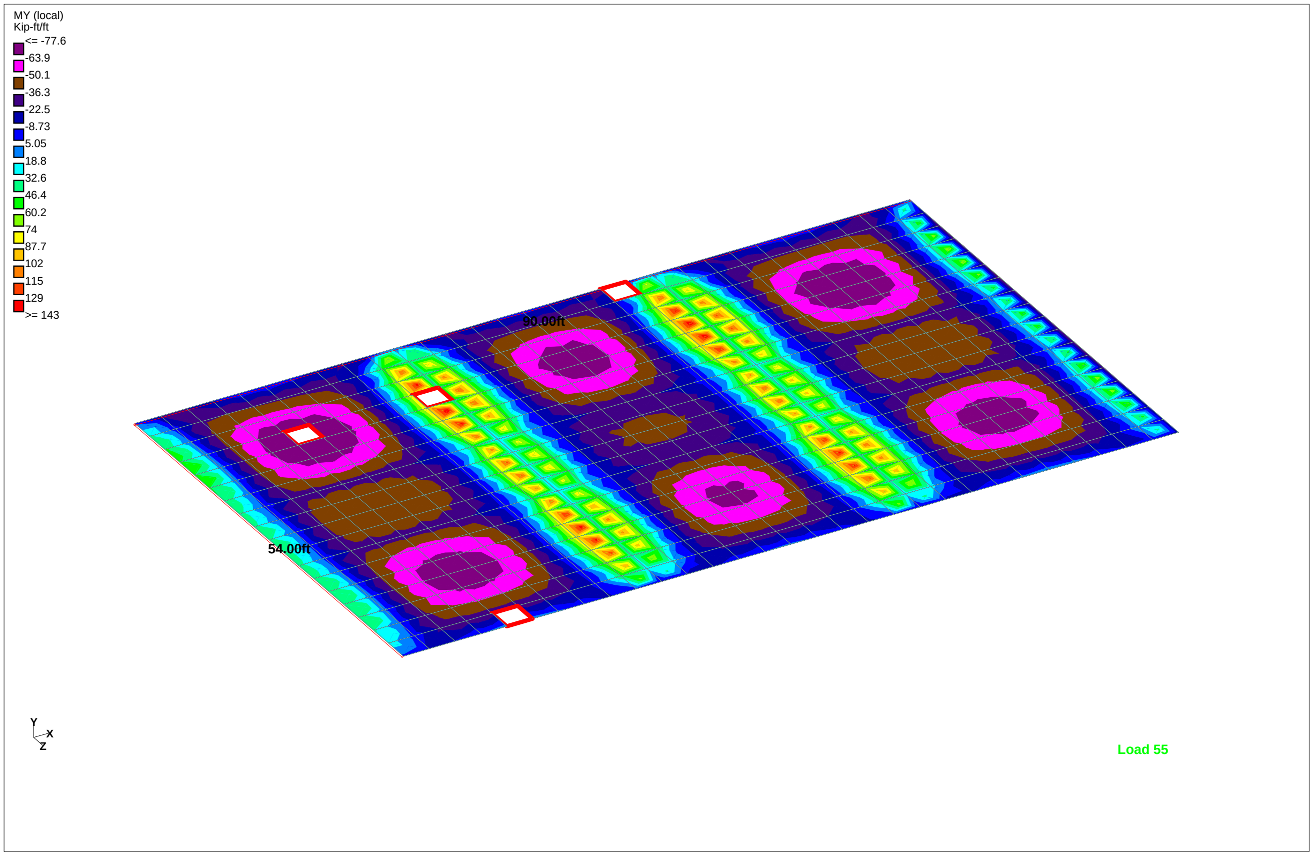
Job Title Fargo- Moorhead Feasibility Study - Phase4

Ref Phase4

By mbi Date 1-24-11 Chd

Client US Army Corps of Engineers

File PH4 Sheyene Aquaduct F Date/Time 24-Feb-2011 15:13



Tributary Channel Deck - LC#55 Mymax (Input data was modified after picture taken)

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Job No **34091009**

Sheet No **27**

Rev 2

Part Sheyene Aquaduct - Partial

Job Title Fargo- Moorhead Feasibility Study - Phase4

Ref Phase4

By mbi

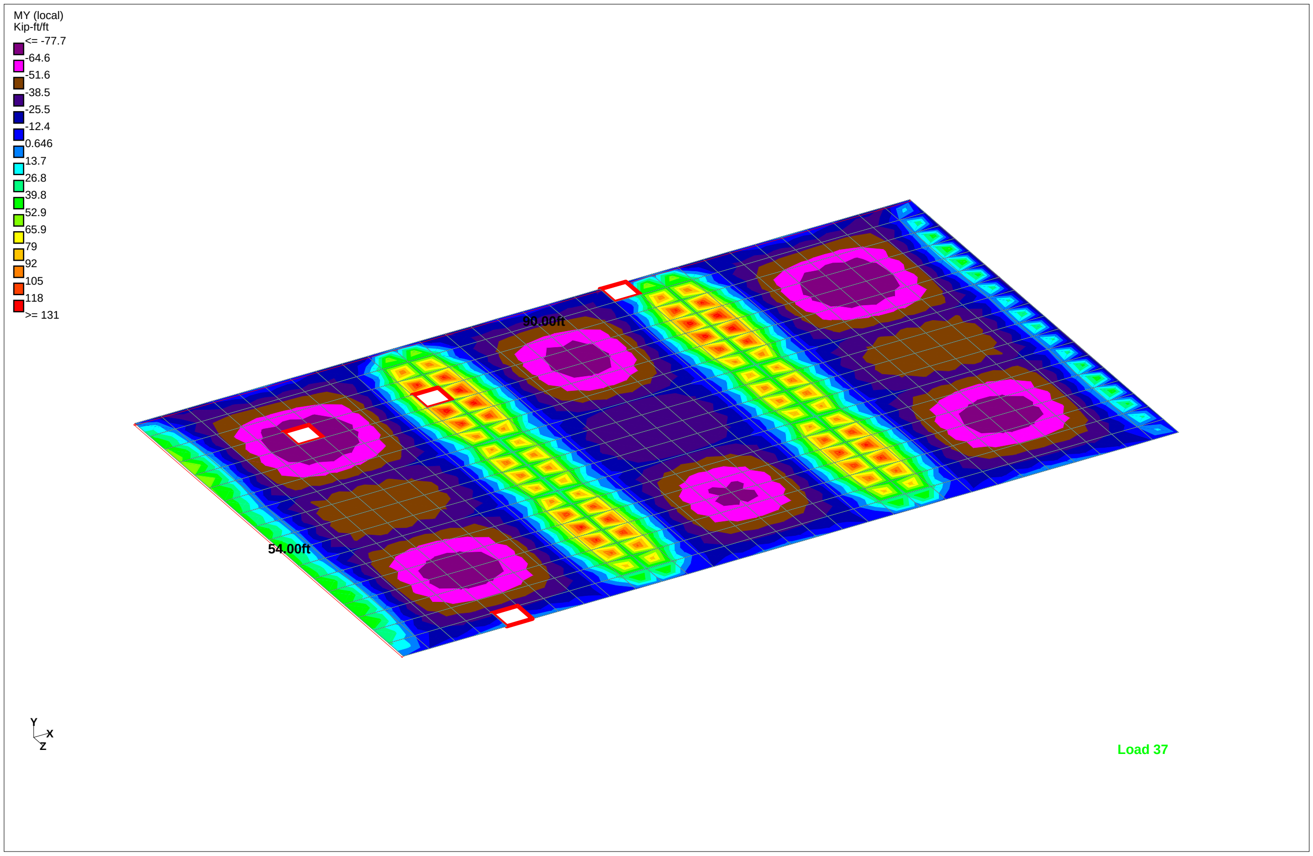
Date 1-24-11

Chd

Client US Army Corps of Engineers

File PH4 Sheyene Aquaduct F

Date/Time 24-Feb-2011 15:13



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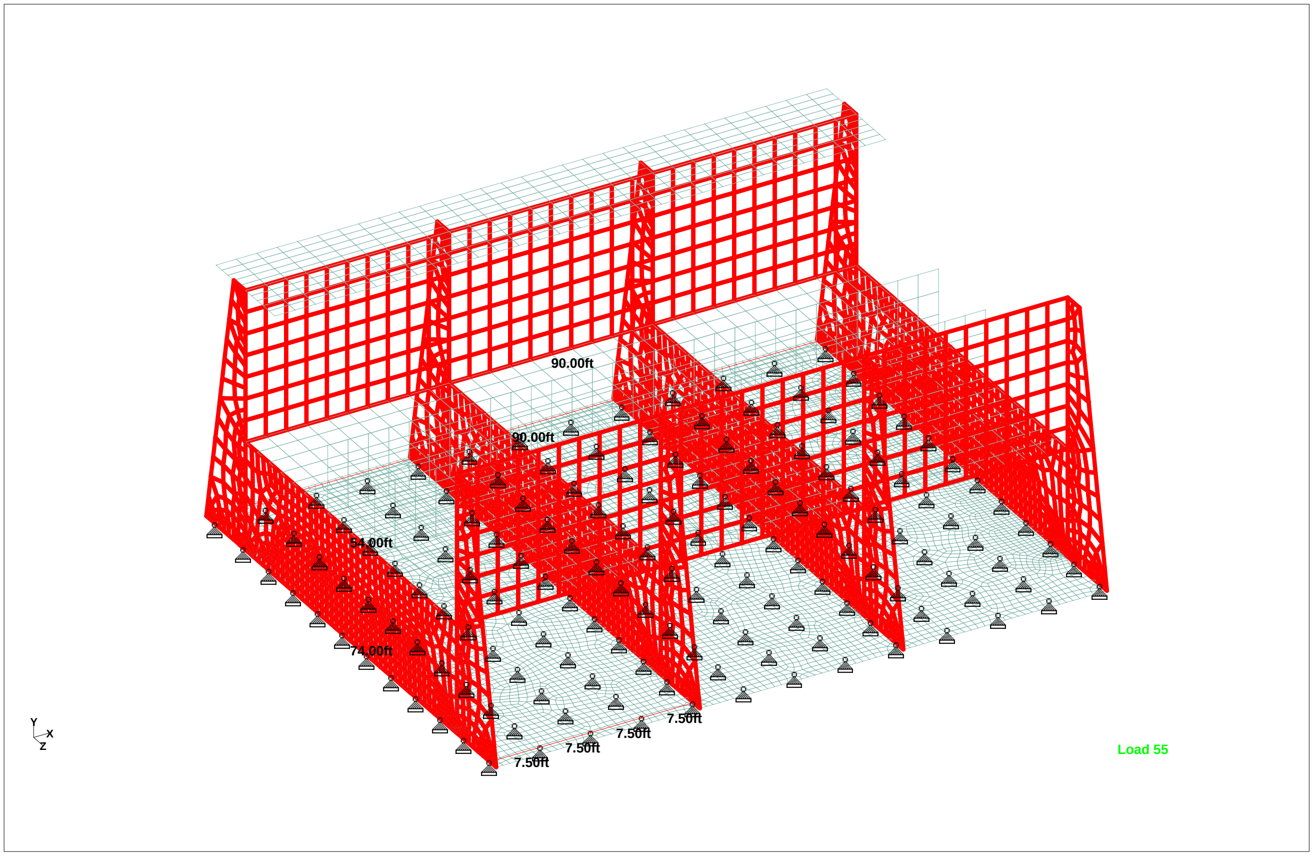
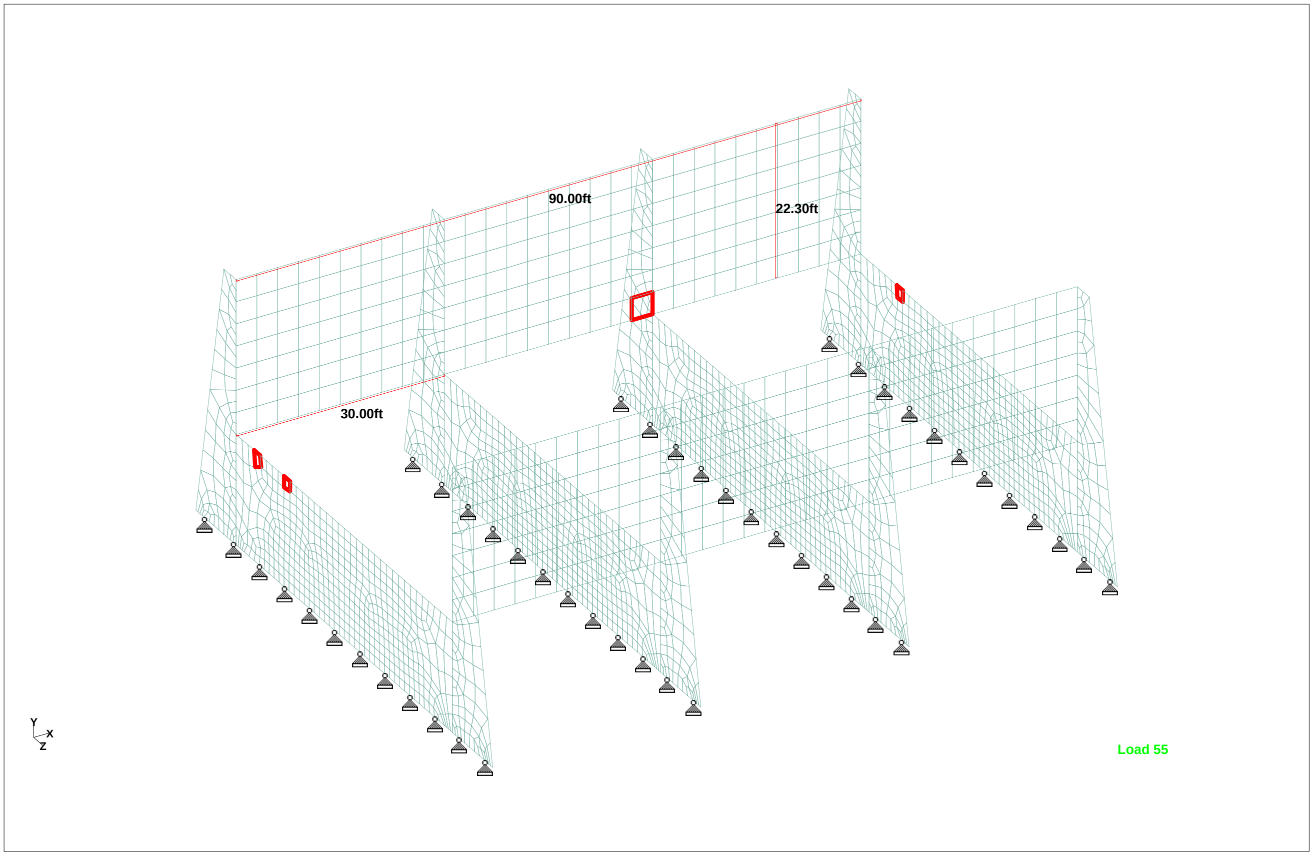


Whole Structure - Piers/Walls (Input data was modified after picture taken)



Piers/Walls Max Moment Critical Plates (Input data was modified after picture taken)

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Job No 34091009

Sheet No **30**

Rev 2

Part Sheyene Aquaduct - Partial

Job Title Fargo- Moorhead Feasibility Study - Phase4

Ref Phase4

By mbi

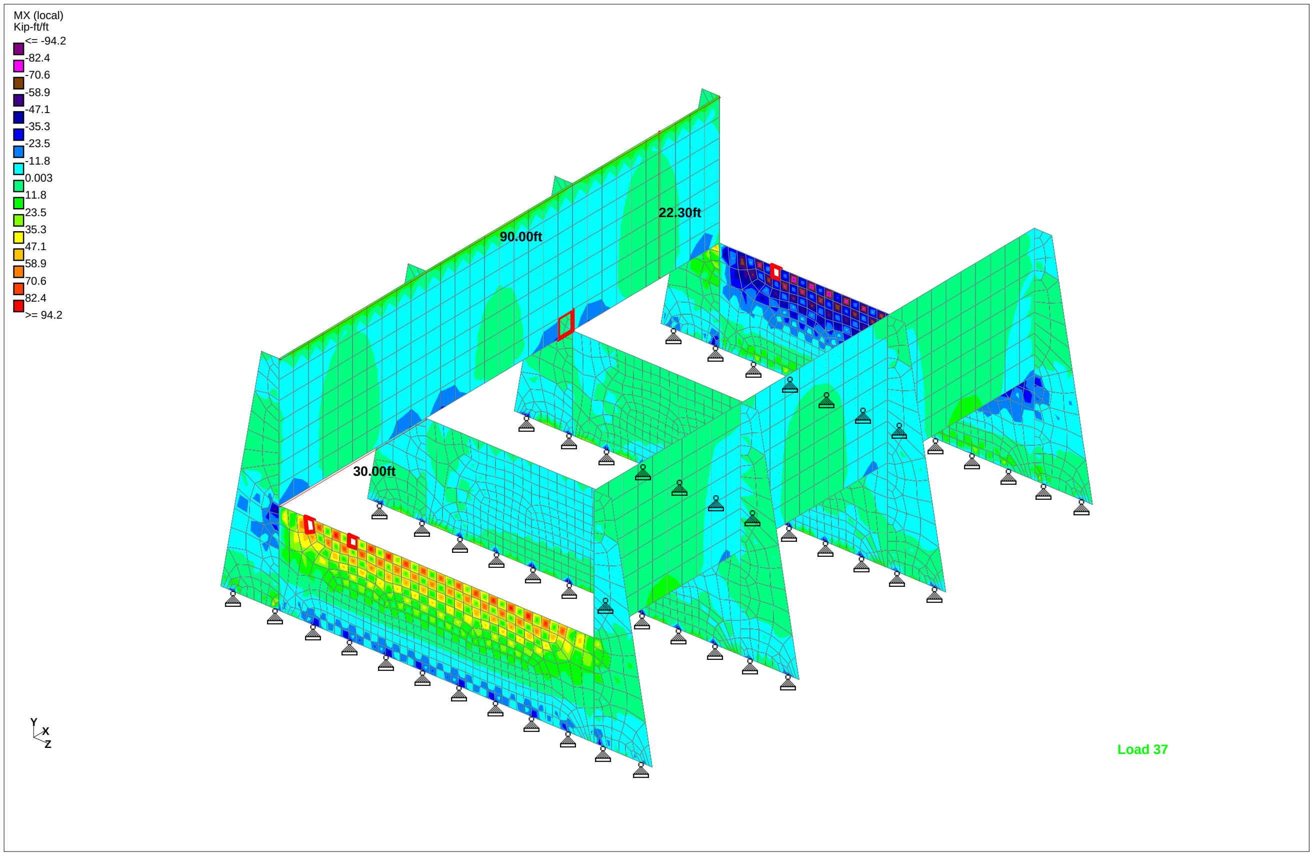
Date 1-24-11

Chd

Client US Army Corps of Engineers

File PH4 Sheyene Aquaduct F

Date/Time 24-Feb-2011 15:13



Piers/Walls LC#37 Mxmax (Input data was modified after picture taken)

Barr Engineering

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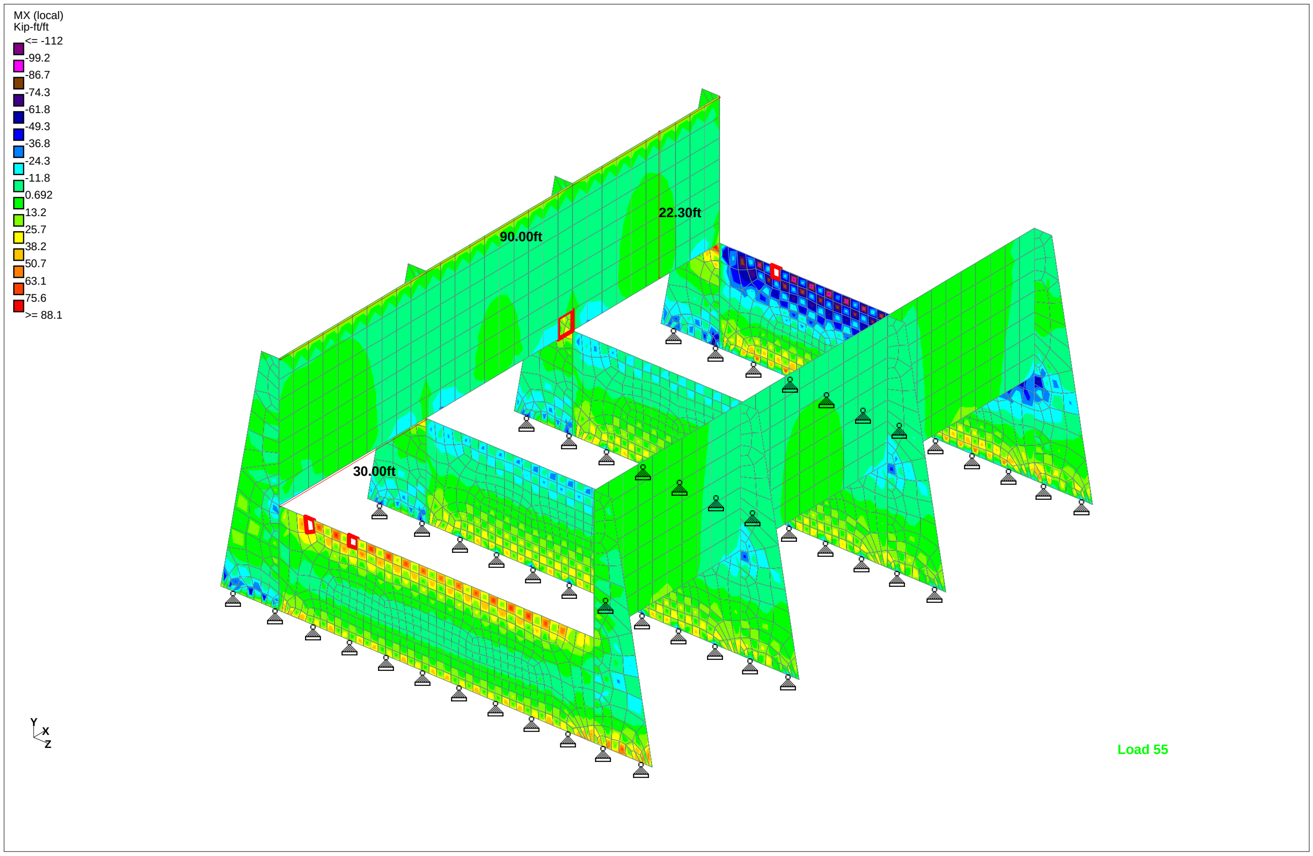
Job No	34091009	Sheet No	31	Rev	2
Part	Sheyene Aquaduct - Partial				
Job Title	Fargo- Moorhead Feasibility Study - Phase4	Ref	Phase4		
By	mbi	Date	1-24-11	Chd	
Client	US Army Corps of Engineers	File	PH4 Sheyene Aquaduct F	Date/Time	24-Feb-2011 15:13



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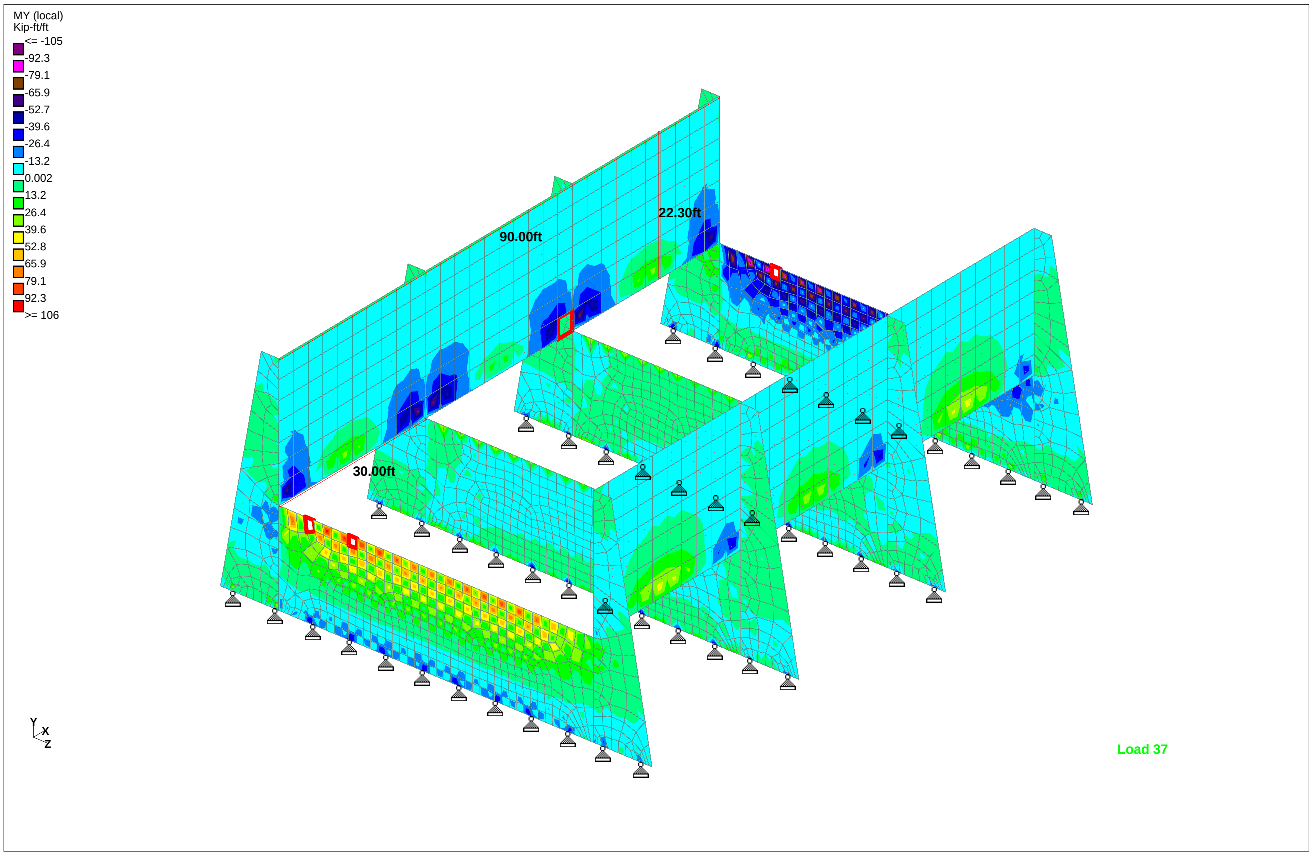
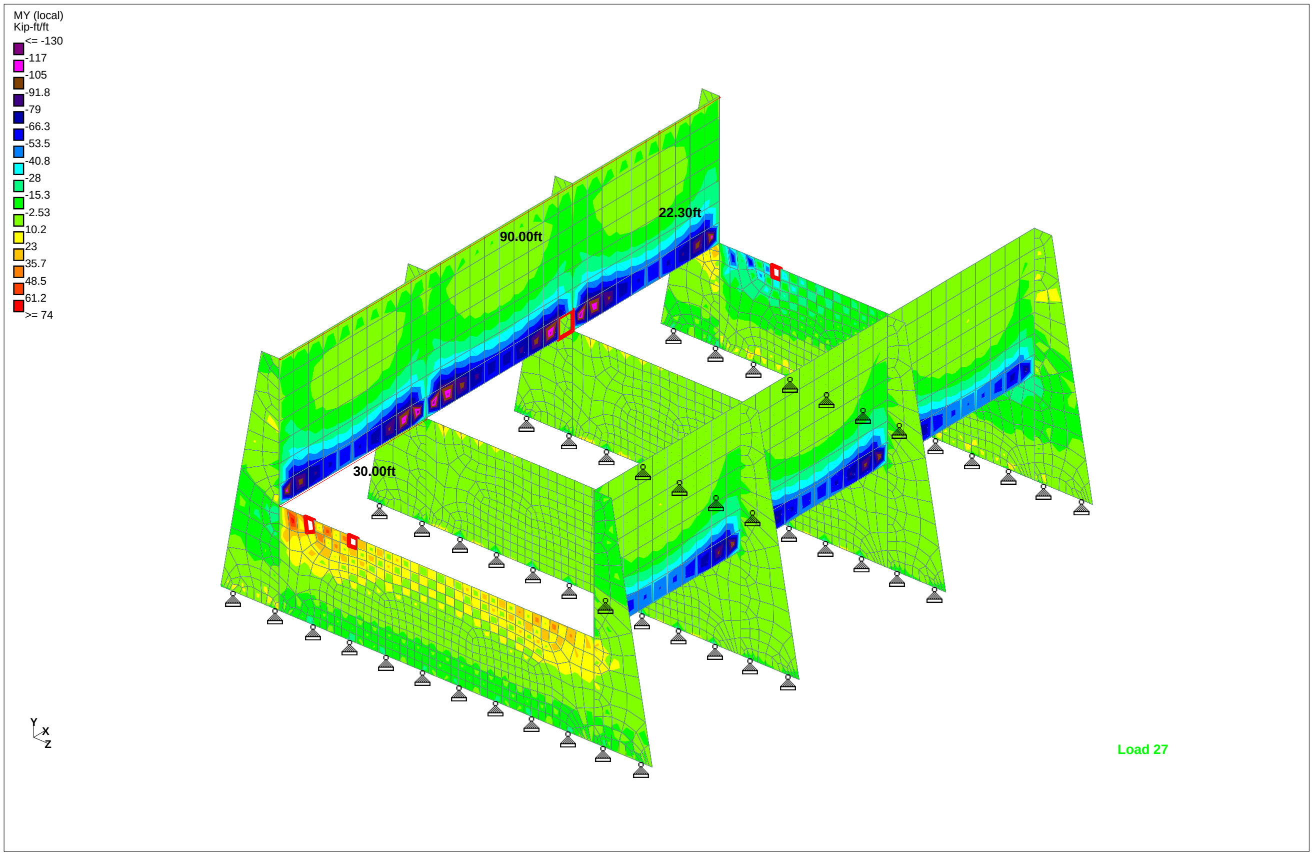
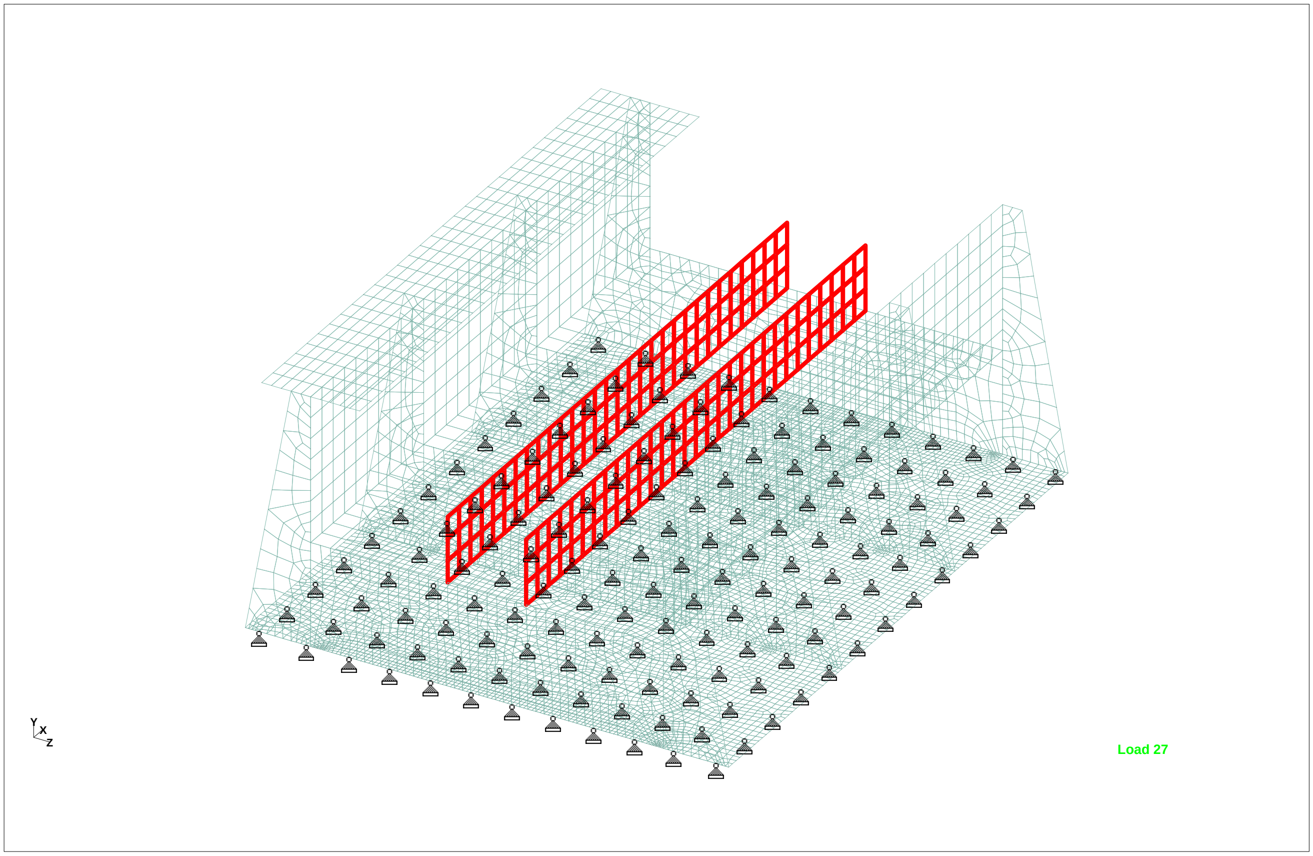


Piers/Walls LC#37 Mymax (Input data was modified after picture taken)

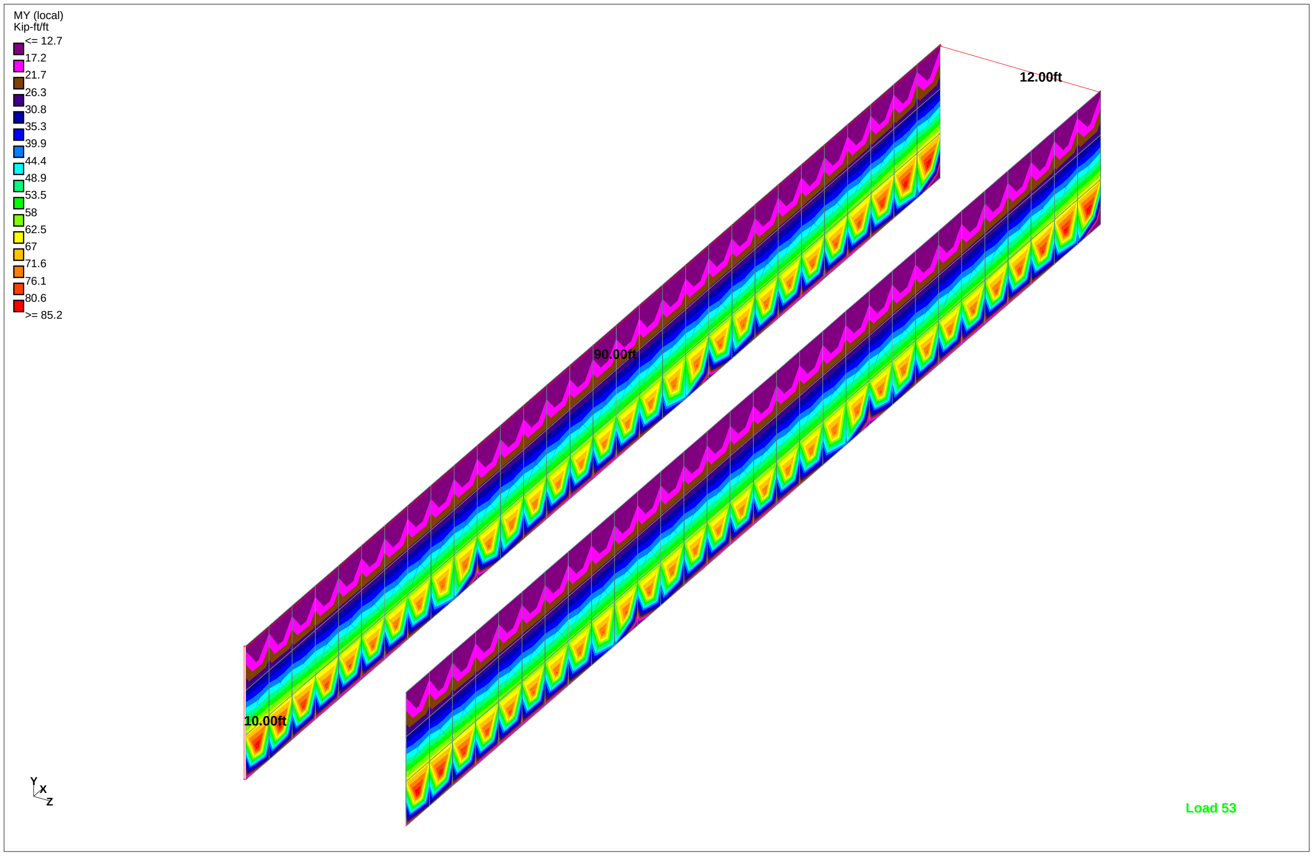
Barr Engineering Software licensed to Microsoft	Job No **34091009**	Sheet No **33**	Rev 2
	Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4	Ref Phase4		
	By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers	File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



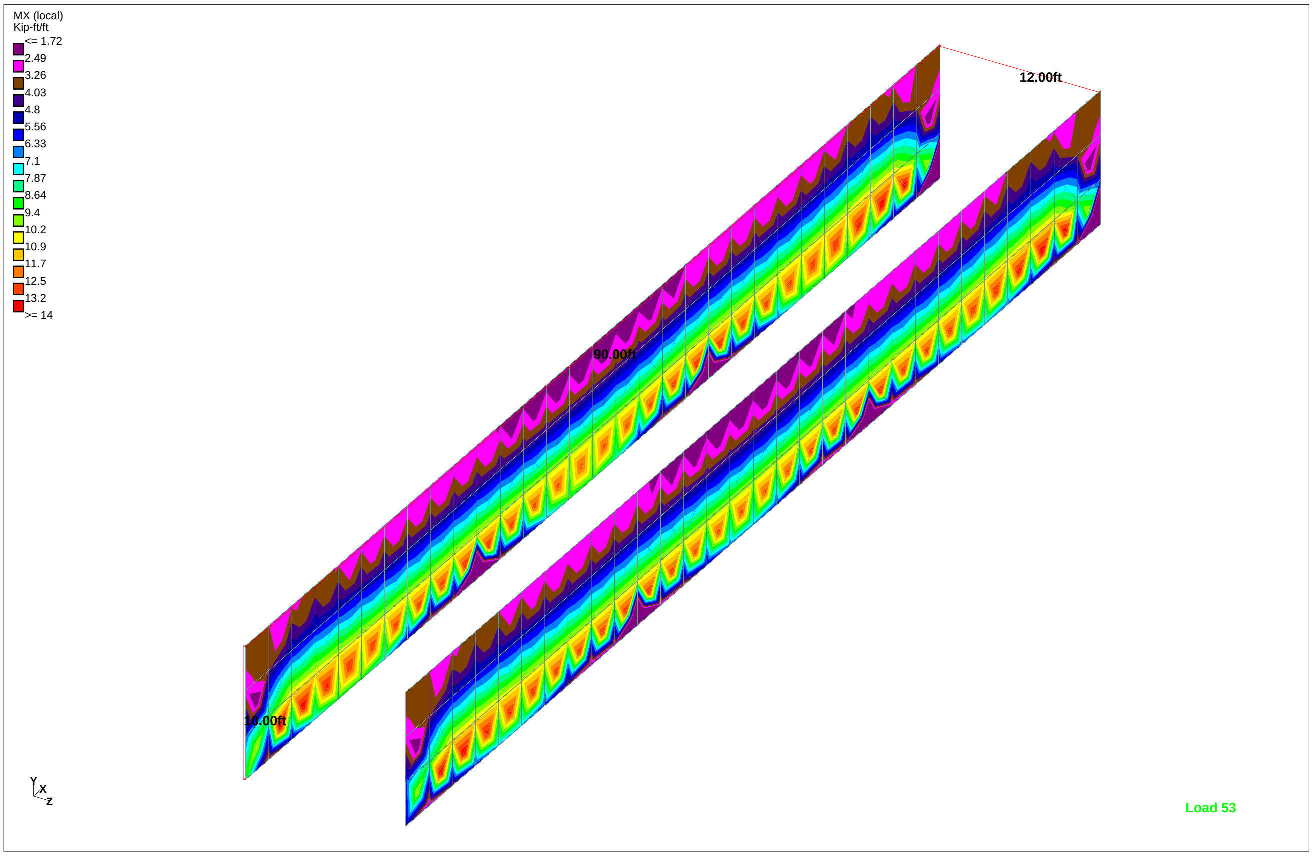
Piers/Walls LC#27 Mymin (Input data was modified after picture taken)



Whole Structure - Low flow Channel Walls (Input data was modified after picture taken)



Low Flow Channel Walls - LC53 Mymax (Input data was modified after picture taken)



Low Flow Channel Walls LC53 Mxmax (Input data was modified after picture taken)

ATTACHMENT

F-R4.2 Sheyenne River Aqueduct Structure Pile Computations

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
Work Describtion:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	1/24/2011
		34091004
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Sheyene\[34091004 PH4 Sheyene Pile Calcs.xlsx]Load Cases	

REF.	1
	2

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	914.56	914.56	914.67	916	903.24	NA
Diversion - Head Water El. (ft)	902.12	902.12	903.32	916	NA	NA
Diversion - Tail Water El. (ft)	901.91	901.91	903.06	916	NA	NA
Tributary - T.O. Wall El. (ft)	916					
Tributary - T.O. Deck L.P. El.(ft)	898.7					
Tributary - T.O. Deck H.P. El.(ft)	900.7					
Diversion - T.O. Mat El. (ft)	883.68					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	15.86	15.86	15.97	17.3	4.54	NA
Diversion - Head Water height (ft)	18.44	18.44	19.64	32.32	NA	NA
Ice	NA	2ft Ice	NA	NA	2ft Ice	NA
Ice Load	NA	10 kips/ft	NA	NA	10 kips/ft	NA
Ice Laod El. (ft)	NA	914.56	NA	NA	903.24	NA
Uplift @ HW (ft)	22.44	22.44	23.64	36.32	NA	NA
Uplift @ TW (ft)	22.23	22.23	23.38	36.32	NA	NA
Pile Condition	Undrained	Undrained	Undrained	Undrained	Drained	Undrained
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	18	20.5	20.5	24	11.5	20.5
Allowable Pile Capacity (tons) - Axial	61.95	82.60	82.60	107.74	36.525	82.60
Allowable Pile Capacity (tons) - Uplift	38.65	51.53	51.53	67.22	5.9	51.53

Pile Capacity	Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips)		
		0.5" (Usual)	0.67" (Unusual)	0.875" (Extreme)
Undrained - Axial	247.8	36	41	48
Undrained - Uplift	154.6			
Drained - Axial	146.1	23	29	36
Drained - Uplift	23.6			

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	**Design By:**	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	**Review By:**	
		Date:	1/24/2011
Work Describtion:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	**Job #:**	34091004'
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Sheyene\[34091004 PH4 Sheyene Pile Calcs.xlsx]Load Cases		

REF. 1

2

Hydrolic Profile

ID#	**Case 1**	**Case 2**	**Case 3**	**Case 4**	**Case 5**	**Case 6**
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	914.56	914.56	914.67	916	903.24	NA
Diversion - Head Water El. (ft)	902.12	902.12	903.32	916	NA	NA
Diversion - Tail Water El. (ft)	901.91	901.91	903.06	916	NA	NA
Tributary - T.O. Wall El. (ft)	916					
Tributary - T.O. Deck L.P. El.(ft)	898.7					
Tributary - T.O. Deck H.P. El.(ft)	900.7					
Diversion - T.O. Mat El. (ft)	883.68					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	15.86	15.86	15.97	17.3	4.54	NA
Diversion - Head Water height (ft)	18.44	18.44	19.64	32.32	NA	NA
Ice	NA	2ft Ice	NA	NA	2ft Ice	NA
Ice Load	NA	10 kips/ft	NA	NA	10 kips/ft	NA
Ice Laod El. (ft)	NA	914.56	NA	NA	903.24	NA
Uplift @ HW (ft)	22.44	22.44	23.64	36.32	NA	NA
Uplift @ TW (ft)	22.23	22.23	23.38	36.32	NA	NA

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	**U.S. ARMY CORPS OF ENGINEERS**	**Design By:**	**MBI**
Project Name:	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	**Review By:**	
		Date:	**1/24/2011**
Work Describtion:	**LOAD CASES - SHEYENNE AQUEDUCT - Phase 4**	**Job #:**	**34091004'**
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Sheyene\[34091004 PH4 Sheyene Pile Calcs.xlsx]Load Cases		

Quantity Take Off (Revit)

Volume of Walls (ft3)	Vw (ft3)	51100	ft3
Volume Tributary Deck Slab (ft3)	Vs (ft3)	41022	ft3
Volume of Bridge Deck (ft3)	Vs (ft3)	8127	ft3
Volume Diversion Mat Slab (ft3)	Vs (ft3)	81744	ft3
Total		**181993**	ft3

Material Properties

Concrete	γ Concrete (pcf)	150
Steel	γ Steel (pcf)	495
Soil Dry	γs Dry (pcf)	120
Soil Saturated	γs Sat. (pcf)	130
Water	γ Water (pcf)	62.4

Geometry

Tributary Channel

Tributary - T.O. Wall El. (ft)		**916**	ft
Tributary - T.O. Deck L.P. El.(ft)		**898.7**	ft
Tributary - T.O. Deck H.P. El.(ft)		**900.7**	ft
Tributary - Clear Width (ft)	w TC	50	ft
Tributary - Wall Thickness (ft)	twall TC	3	ft
Tributary - Deck Slope Width (ft)	lslab slope TC	17.5	ft
Tributary - Deck Slab thickness @ L.P. (ft)	tslab TC	**2**	ft
Tributary - Deck Slab thickness @ H.P. (ft)	tslab TC	**4**	ft
Tributary - Low Flow Channel height (ft)	hlowflow TC	10	ft
Tributary - Low Flow Channel width (ft)	wlowflow TC	10	ft
Tributary - Low Flow Channel thickness (ft)	tlowwall TC	1	ft
Tributary - Wall Height (ft)	hwall TC	**19.3**	ft

Diversion Channel

Diversion - T.O. Wall El. (ft)		**896.7**	ft
Diversion - T.O. Mat El. (ft)		**883.68**	ft
Diversion - Clear Opening Width	wopen DC	30	ft
Diversion - # of Openings	#open DC	6	
Diversion - Wall Thickness (ft)	twall DC	3	ft
Diversion - Mat Slab thickness (ft)	tslab DC	**4**	ft
Diversion - Butress height (ft)	hbutress	**32.32**	ft
Diversion - Butress Top width (ft)	wbutress Top	2	ft
Diversion - Butress Top width (ft)	wbutress Bot	9	ft
Diversion - Wall Height (ft)	hwall DC	**13.02**	ft

Mat Foundation

Overall Width (ft)	wmat	**78**	ft
Overall Length (ft)	lmat	**262**	ft
Triburtary - Channel Length (ft)	lslab TC	**258**	ft
Triburtary - Channel Width (ft)	wslab TC	**56**	ft

Access Bridge

Overall Width (ft)	wbridge	15	ft
Overall Length (ft)	lbridge	**258**	ft
Minimum Deck Thickness	tbridge	1.5	ft
Maximum Deck Thickness	tbridge	3	ft

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	**Design By:**	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	**Review By:**	
		Date:	1/24/2011
Work Describtion:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	**Job #:**	34091004'
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Sheyene\[34091004 PH4 Sheyene Pile Calcs.xlsx]Load Cases		

Weight of Structure

Tributary	Volume (ft3)	Weight (tons)
Walls	23684	1776
Deck	41022	3077
Low Flow Chanel	5160	387
Bridge	8127	860
Sub Total	77993	6100

Diversion	Volume (ft3)	Weight (tons)
Walls	16770	1258
Mat	81744	6131
Butress Walls	5333	400
Sub Total	103847	7788
Total	181840	13638

Whole Structure	Volume (ft3)	Weight (tons)
Walls	50947	3821
Deck	41022	3077
Bridge	8127	860
Mat	81744	6131
Total	181840	13638

Take off (Revit)	Volume (ft3)	Weight (tons)
Walls	51100	3833
Deck	41022	3077
Bridge	8127	610
Mat	81744	6131
Total	181993	13649

Ratio	0.999159089

Forces

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tibutary - Water height (ft)	15.86	15.86	15.97	17.30	4.54	NA
Triburtary - Channel Length (ft)	258.00					
Tributary - Clear Width (ft)	50.00					
Tributary - Water force (psf)	989.66	989.66	996.53	1079.52	283.30	NA
Tributary - Water Volume (ft3)	204594.00	204594.00	206013.00	223170.00	11713.20	NA
Tributary - Water Weight (tons)	6383.33	6383.33	6427.61	6962.90	365.45	NA
Diversion - Head Water height (ft)	18.44	18.44	19.64	32.32	NA	NA
Mat Foundation - Overall Width (ft)	78.00					
Mat Foundation - Clear Length (ft)	162.00					
Diversion - Water force (psf)	1150.66	1150.66	1225.54	2016.77	NA	NA
Diversion - Water Volume (ft3)	233007.84	233007.84	248171.04	408395.52	NA	NA
Diversion - Water Weight (tons)	7269.84	7269.84	7742.94	12741.94	NA	NA
Total Water Weight on the Structure (tons)	13653.18	13653.18	14170.54	19704.84	365.45	NA
Tributary - Uplift on the Deck (ft)	5.42	5.42	6.62	19.30	NA	NA
Tributary - Uplift force (psf)	338.21	338.21	413.09	1204.32	NA	NA
Tributary - Uplift force (tons)	-2443.21	-2443.21	-2984.15	-8700.01	NA	NA
Uplift @ HW (ft)	22.44	22.44	23.64	36.32	NA	NA
Uplift @ TW (ft)	22.23	22.23	23.38	36.32	NA	NA
Diversion - Uplift force on the Mat (psf)	1393.70	1393.70	1467.02	2266.37	NA	NA
Diversion - Uplift force on the Mat (tons)	-14240.87	-14240.87	-14990.05	-23157.75	NA	NA
Total Uplif Force on the Structure (tons)	-16684.08	-16684.08	-17974.20	-31857.76	NA	NA
Weight of Structure (tons)	13649.5					

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name: U.S. ARMY CORPS OF ENGINEERS

Project Name: FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

Work Describtion: LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

File Path: P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Sheyene\[34091004 PH4 Sheyene Pile Calcs.xlsx]Load Cases

Design By: MBI

Review By:

Date: 1/24/2011

Job #: 34091004'

Flotation

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Downward force on the Structure (tons)	10618.57	10618.57	9845.82	1496.56	14014.93	13649.48
Uplift Ratio	1.64	1.64	1.55	1.05	NA	NA
Uplift Ratio (No water in the Tributary)	1.25	1.25	1.19	0.83	NA	NA
Condition	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors - Flotation	1.30	1.20	1.20	1.10	1.30	1.20
Check	OK	OK	OK	NG!!!	OK	OK

Pile Computation

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Pile Condition	Undrained	Undrained	Undrained	Undrained	Drained	Undrained
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	18	20.5	20.5	24	11.5	20.5
Allowable Pile Capacity (tons) - Axial	61.95	82.60	82.60	107.74	36.53	82.60
# of Piles Required	171.41	128.55	119.20	13.89	383.71	165.25
Uniform Spacing	10.92	12.61	13.09	38.36	7.30	11.12
# of Columns (along length)	35.00	35.00	35.00	35.00	35.00	35.00
Pile Spacing (along length)	7.53	7.53	7.53	7.53	7.53	7.53
# of Rows (along width)	4.90	3.67	3.41	0.40	10.96	4.72
# of Rows (along widht) provided	12.00	12.00	12.00	12.00	12.00	12.00
Actual Pile Spacing (along width)	6.55	6.55	6.55	6.55	6.55	6.55
Total number of pile provided	420	420	420	420	420	420
Pile Load	25.28	25.28	23.44	3.56	33.37	32.50
Utilization Ratio	0.41	0.31	0.28	0.03	0.91	0.39
Check	OK	OK	OK	OK	OK	OK
ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Final # of Piles	420					
Pile Load	25.28	25.28	23.44	3.56	33.37	32.50
Utilization Ratio	0.41	0.31	0.28	0.03	0.91	0.39
Check	OK	OK	OK	OK	OK	OK

Fargo-Moorhead Food Control Structures
Preliminary Pile Foundation Analyses
HP 14X73

A_{tip} = 198.5 in^2, A_{steel} = 21.4 in^2, perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in^4

Structure	Diversion Channel Station Location	Approximate Ground (Bank) Surface Elevation (ft)	Invert Elevation (ft)	Estimated Foundation Elevation (ft)	Estimated Ground Water Elevation (ft)	Design Condition/Tip Elevations		Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips) (fixed head - single pile)			Estimated Settlement at allowable load
									0.5″	0.67″	0.875″	
Sheyenne River Aqueduct Crossing Foundations	1493+98	916	~~881.8~~ 883.68	~~877.83~~ 879.68	908	Undrained Analysis 842.8′	Total	247.8	36	41	48	<0.5″
							Uplift Resistance	154.6				
						Drained Analysis 842.8′	Total	146.1	23	29	36	
							Uplift Resistance	23.6				

See following link for details
P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\060 Geotech\Deep Foundations

ATTACHMENT

F-R4.3 Sheyenne River Aqueduct Structure Retaining Wall Panel E Computations

SHEYENE AQUADUCT STRUCTURE

Client Name:	**U.S. ARMY CORPS OF ENGINEERS**	**Design By:**	**MBI**
Project Name:	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	**Review By:**	
Work Describtion:	**Sheyenne Aquaduct Structure - Retaining Walls**	**Date:**	**2/10/2011**
	Panel E	**Job #:**	**34091004**
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Sheyene\[34091004 PH4 Sheyene Retaining Walls Panel E.xlsx]Load Cases		
REF.	**1**		
	2		

ID#	**Case 1**	**Case 2**	**Case 3**	**Case 4**	**Case 5**	**Case 6**
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	914.56	914.56	914.67	917.5	903.24	**NA**
Diversion - Head Water El. (ft)	902.12	902.12	903.32	917.5	**NA**	**NA**
Diversion - Tail Water El. (ft)	901.91	901.91	903.06	917.5	**NA**	**NA**
Tributary - T.O. Wall El. (ft)	917.5					
Tributary - T.O. Deck L.P. El.(ft)	898.7					
Tributary - T.O. Deck H.P. El.(ft)	900.7					
Diversion - T.O. Mat El. (ft)	883.68					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	**15.86**	**15.86**	**15.97**	**18.8**	**4.54**	**NA**
Diversion - Head Water height (ft)	**18.44**	**18.44**	**19.64**	**33.82**	**NA**	**NA**
Ice	**NA**	2ft Ice	**NA**	**NA**	2ft Ice	**NA**
Ice Load	**NA**	10 kips/ft	**NA**	**NA**	10 kips/ft	**NA**
Ice Laod El. (ft)	**NA**	**914.56**	**NA**	**NA**	**903.24**	**NA**
Uplift @ HW (ft)	**22.44**	**22.44**	**23.64**	**37.82**	**NA**	**NA**
Uplift @ TW (ft)	**22.23**	**22.23**	**23.38**	**37.82**	**NA**	**NA**
Pile Condition	**Undrained**	**Undrained**	**Undrained**	**Undrained**	**Drained**	**Undrained**
Load Category	**Usual**	**Unusual**	**Unusual**	**Extreme**	**Usual**	**Unusual**
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	**18**	**20.5**	**20.5**	**24**	**11.5**	**20.5**
Allowable Pile Capacity (tons) - Axial	**61.95**	**82.60**	**82.60**	**107.74**	**36.525**	**82.60**
Allowable Pile Capacity (tons) - Uplift	**38.65**	**51.53**	**51.53**	**67.22**	**5.9**	**51.53**

Pile Capacity	**Ultimate Axial Capacity (kips)**	**Allowable Lateral Capacity (kips)**		
		0.5" (Usual)	**0.67" (Unusual)**	**0.875" (Extreme)**
Undrained - Axial	247.8	36	41	48
Undrained - Uplift	154.6			
Drained - Axial	146.1	23	29	36
Drained - Uplift	23.6			

BARR ENGINEERING			DATE	2/11/2011 SHEET NO.
			PROJECT NAME	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	**34091004**
MBI 2/11/11		**MBI**	SUBJECT	**Sheyenne Aquaduct Structure - Retaining Walls** **Panel E**

Monolith Structure

ITEM		UNIT	QUANITY	UNIT COST	TOTAL Cost
FURNISH HP14x73 WALL PILING		LF	4,370	0	$0
INSTALL HP14x73 WALL PILING		LF	4,370	0	$0
PILE TEST, 48.0 ft	Long	EA	6	0	$0
FOOTING CONCRETE		CY	618	0	$0
	Forming	SF	1,428		
STEM CONCRETE		CY	701	0	$0
	Forming	SF	9,812		
STEEL REINFORCEMENT		LB	253,760	0	$0
WALL RAILING		LF	140	0	$0
SHEET PILE CUT-OFF WALL		SF	2,800	0	$0
					$0

Structure Length = 140 ft

No. piles = 115 Each
Length = 38 ft

Note: HP14x73 pile used for design,
use HP14x73 to allow for corrosion

	LENGTH
(FRONT & Back FACE)	10 FT

Native Soil has low permeability assume cut-off
minimal to prevent scour

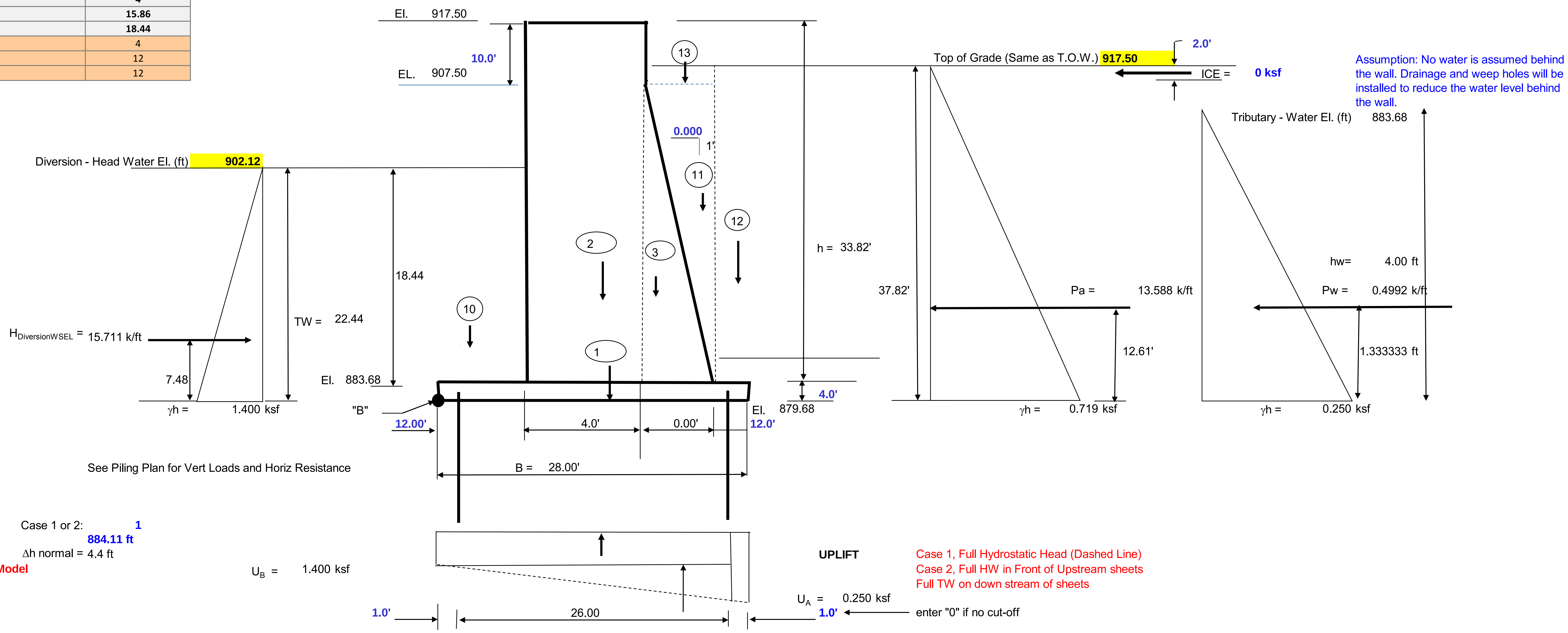
BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls Load Cases: Case 1 100 yr. flood	Panel E

ID#	Case 1
Name	100 yr. flood
Load Category	Usual
Tributary - Water El. (ft)	914.56
Diversion - Head Water El. (ft)	902.12
Diversion - Tail Water El. (ft)	901.91
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El.(ft)	898.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	15.86
Diversion - Head Water height (ft)	18.44
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 903.24 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section
Length = 140.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Assumption: No water is assumed behind the wall. Drainage and weep holes will be installed to reduce the water level behind the wall.

See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1
Normal Water Level, El. 884.11 ft
Δh normal = 4.4 ft

See Geotechnical seepage Model

UPLIFT

Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Vertical Loads	**Section**	L ft	W ft	H ft	γ kcf	shape	**V** K	arm ft	**Mv** ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**5192.9**	ΣM$_V$ =	**72,700.3**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	18.44	0.0624	rec	1933.1	6.00	11,598.6
H.W. on Stem Slope	11	140	0.00	23.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0626	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**5489.9**	ΣM$_V$ =	**89,847.8**

Uplift Loads		L ft	W ft	Pressure ksf		**U** K	arm ft	**Mu** ft-k
	U_B	140	28.00	1.400	rec	-5489.0	14.00	-76,846
	U_A	140	28.00	-1.151	tri	2255.3	18.67	42,099
					ΣU =	**-3233.7**	ΣM$_U$ =	**-34,747**

BARR ENGINEERING			DATE	**2/11/2011**	SHEET NO.
			PROJECT NAME	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	**34091004**	
MBI		**MBI**	SUBJECT	**Sheyenne Aquaduct Structure - Retaining Walls**	
2/11/11				**Load Cases:** **Case 1 100 yr. flood**	**Panel E**

Horizontal Loads

	L	H	Pressure		ICE	arm	Mu
	ft	ft	ksf		K	ft	ft-k
ICE	140	2.00	0.00	**rec**	0.0	36.82	0.0

	L	Force	H	arm	Mw
	ft	k/ft	K	ft	ft-k
SOIL	140	-13.588	-1902.37	12.61	-23982.53

Water Loads

H_{TW}	140	15.711	**tri**	2199.52	7.48	16452.43
H_{HW}	140	-0.499	**tri**	-69.89	1.33	-93.18
			ΣWater =	**2129.63**	ΣM_W =	**-7623.3**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-42371	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	162548	kip-ft
Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**120,177**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,449**	kips
Sum of Horizontal Forces	H = Σhorizontal	**227**	kips

Location of Resultant Xr = ΣM / P = 16.13 ft from Toe

e = B/2 - Xr = (2.13) ft

B/6 = 4.667 ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	701	cy	9812	sf
Total =	1,290			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing		Length	# of bars	Total wt		
	a) Footing		in	LB /ft	ft	ea	lb		
Top mat	Transverse:	**9**	**6**	3.40	27.5	284	26,554		
	Longitudinal:	**9**	**6**	3.40	141.5	56	26,942		
Bot mat	Transverse:	**9**	**6**	3.40	27.5	284	26,554		
	Longitudinal:	**9**	**6**	3.40	141.5	56	26,942		
								cy	**LB/cy**
							106,991	589	181.6374748
	b) Skin Reinf. On Monolith								
Vert Face	Vertical	**9**	**6**	3.40	33.32	280	31,721	63,441.28	
	Longitudinal:	**9**	**6**	3.40	139.5	67	31,778	63,556.20	
Top Face	Transverse:	**9**	**6**	3.40	3.5	280	3,332		
	Longitudinal:	**9**	**6**	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	**9**	**6**	3.40	33.3	280	31,721		
	Vertical O.F.	**9**	**6**	3.40	33.3	280	31,721		
								cy	**LB/cy**
							134,066	701	191.1270455
						Σ =	**241,058**		
	Lap Splices (long. Bars)		**9**	3.40	8	467	12,702		
						Σ Bar Wt=	253,760	lb	

FORCES AT THE BOTTOM OF THE STEM

		H	γ	Pbase	V	arm	Mv
Diversion Face		ft	kcf		K	ft	ft-k
Diversion WSEL		18.44	**0.0624**	1.150656	10.609	6.147	65.21028
Tributary SEL =		33.82	**0.019**	0.64258	10.866	11.273	122.4964
Tributary WSEL =		0.00	**0.0624**	0	0.000	0.000	0
	Sum				10.866		122.4964
	Net Forces				0.257		57.28607

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls Load Cases: Case 2 100 yr. flood + ice	Panel E

ID#	Case 2
Name	100 yr. flood + ice
Load Category	Unusual
Tributary - Water El. (ft)	914.56
Diversion - Head Water El. (ft)	902.12
Diversion - Tail Water El. (ft)	901.91
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El.(ft)	898.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	15.86
Diversion - Head Water height (ft)	18.44
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

MN State Building Codes

Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March

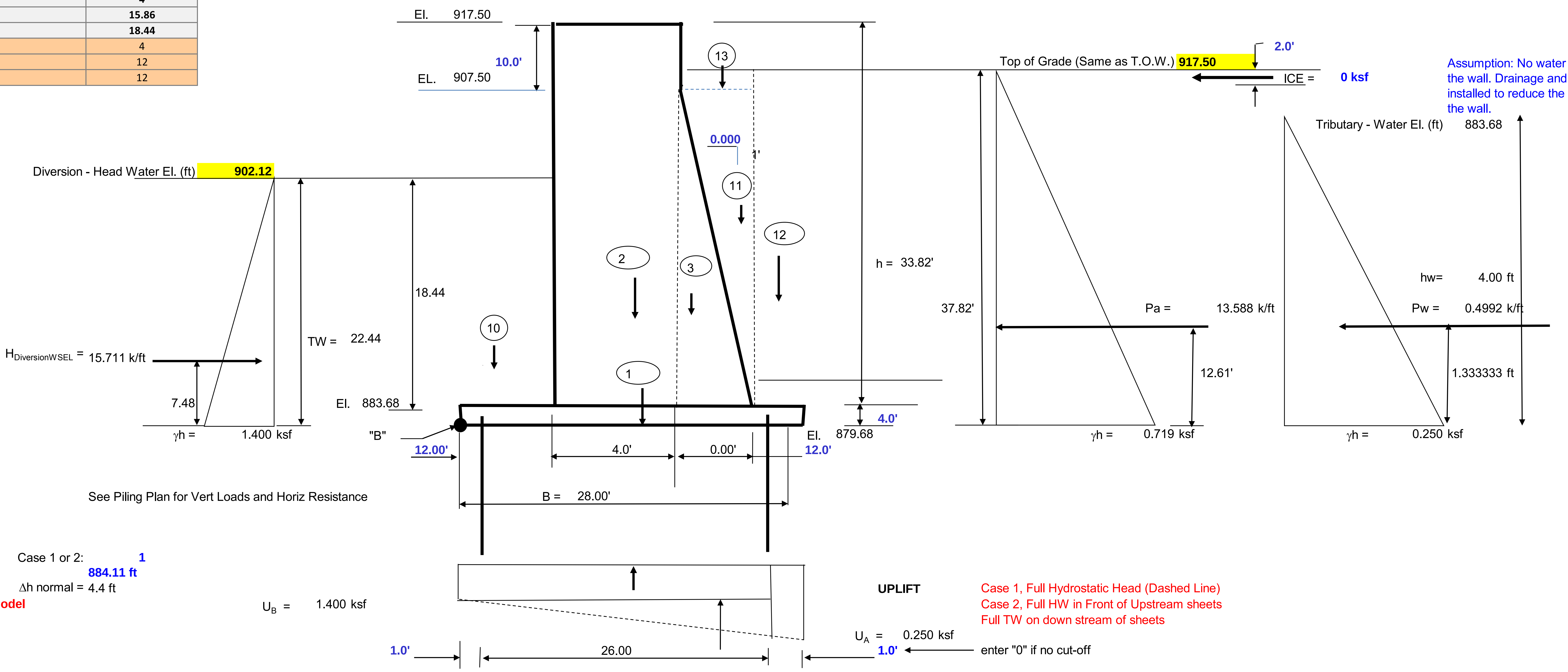
Water El. = 903.24 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section

Length = 140.0 ft

Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg

Assumption: No water is assumed behind the wall. Drainage and weep holes will be installed to reduce the water level behind the wall.



Case 1 or 2: 1

Normal Water Level, El. 884.11 ft

Δh normal = 4.4 ft

See Geotechnical seepage Model

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	**V** K	arm ft	**Mv** ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
				D.L. Concrete		ΣVc =	**5192.9**	ΣM$_V$ =	**72,700.3**

CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	18.44	0.0624	rec	1933.1	6.00	11,598.6
H.W. on Stem Slope	11	140	0.00	23.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0626	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
				D.L. Water		ΣVw =	**5489.9**	ΣM$_V$ =	**89,847.8**

Uplift Loads		L ft	W ft	Pressure ksf			**U** K	arm ft	**Mu** ft-k
	U$_B$	140	28.00	1.400		rec	-5489.0	14.00	-76,846
	U$_A$	140	28.00	-1.151		tri	2255.3	18.67	42,099
						ΣU =	**-3233.7**	ΣM$_U$ =	**-34,747**

Horizontal Loads		L ft	H ft	Pressure ksf			**ICE** K	arm ft	**Mu** ft-k

BARR ENGINEERING			DATE	2/11/2011 SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls Load Cases: Case 2 100 yr. flood + ice Panel E

	L		Force		H	arm	Mw
ICE	140	2.00	0.00	rec	0.0	36.82	0.0
	L ft		Force k/ft		**H** K	arm ft	**Mw** ft-k
SOIL	140		-13.588		-1902.37	12.61	-23982.53
Water Loads							
H_{TW}	140		15.711	tri	2199.52	7.48	16452.43
H_{HW}	140		-0.499	tri	-69.89	1.33	-93.18
				ΣWater =	**2129.63**	ΣM_W =	**-7623.3**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-42371	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	162548	kip-ft
Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**120,177**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,449**	kips
Sum of Horizontal Forces	H = Σhorizontal	**227**	kips
Location of Resultant	Xr = ΣM / P =	16.13	ft from Toe
	e = B/2 - Xr =	(2.13)	ft
	B/6 =	4.667	ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	701	cy	9812	sf
Total =	1,290			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing in	LB /ft	Length ft	# of bars ea	Total wt lb		
	a) Footing								
Top mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
								cy	**LB/cy**
							106,991	589	181.6374748
	b) Skin Reinf. On Monolith								
Vert Face	Vertical	9	6	3.40	33.32	280	31,721	63441.28	
	Longitudinal:	9	6	3.40	139.5	67	31,778	63556.2	
Top Face	Transverse:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	33.3	280	31,721		
	Vertical O.F.	9	6	3.40	33.3	280	31,721		
								cy	**LB/cy**
							134,066	701	191.1270455
						Σ =	**241,058**		
Lap Splices (long. Bars)		9	3.40	8		467	12,702		
						Σ Bar Wt=	253,760	lb	

FORCES AT THE BOTTOM OF THE STEM

Diversion Face		H ft	γ kcf	Pbase	**V** K	arm ft	**Mv** ft-k
Diversion WSEL		18.44	0.0624	1.150656	10.609	6.147	65.21028
Tributary SEL =		33.82	0.019	0.64258	10.866	11.273	122.4964
Tributary WSEL =		0.00	0.0624	0	0.000	0.000	0
	Sum				10.866		122.4964
	Net Forces				0.257		57.28607

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
				Load Cases: Case 3 500 yr. flood	Panel E

ID#	Case 3
Name	500 yr. flood
Load Category	Unusual
Tributary - Water El. (ft)	914.67
Diversion - Head Water El. (ft)	903.32
Diversion - Tail Water El. (ft)	903.06
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El.(ft)	898.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	15.97
Diversion - Head Water height (ft)	19.64
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

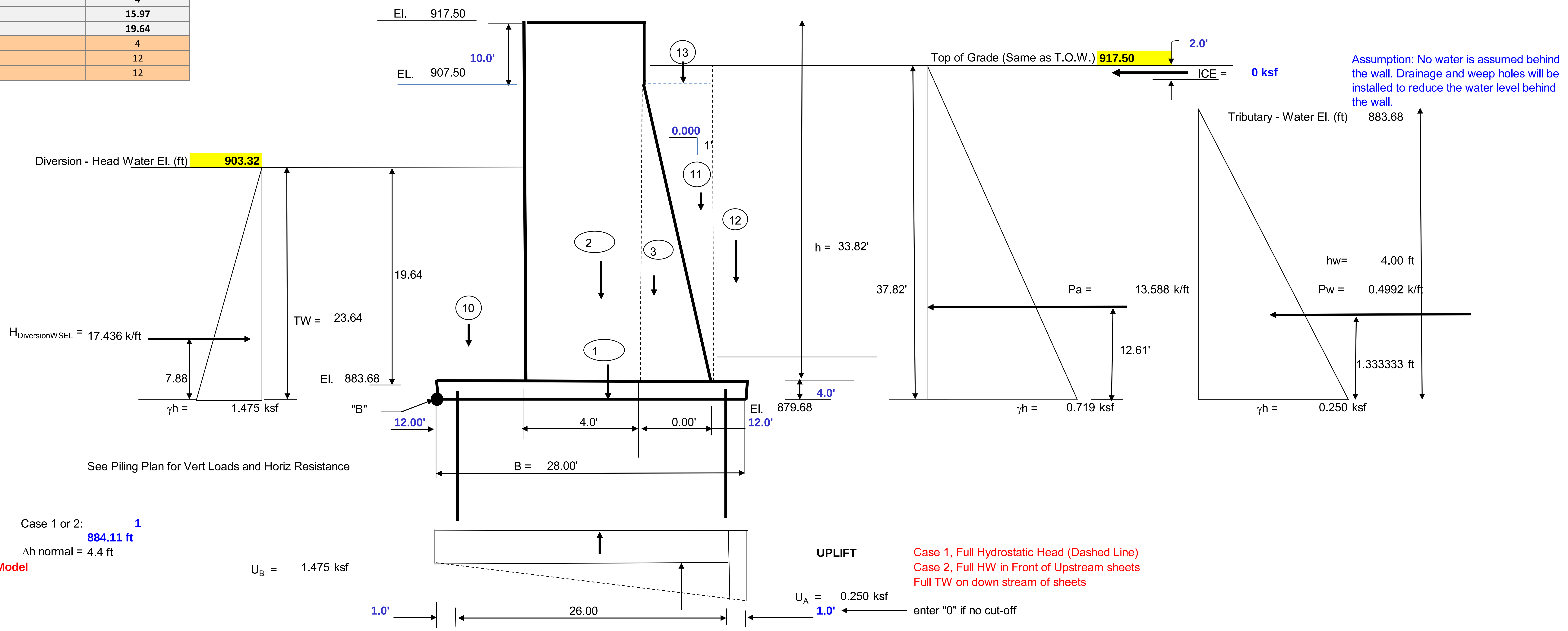
MN State Building Codes

Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March

Water El. = 903.24 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section Length = 140.0 ft

Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Assumption: No water is assumed behind the wall. Drainage and weep holes will be installed to reduce the water level behind the wall.

See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1

Normal Water Level, El. 884.11 ft

Δh normal = 4.4 ft

See Geotechnical seepage Model

Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

	Section	L	W	H	γ	shape	V	arm	Mv
Vertical Loads		ft	ft	ft	kcf		K	ft	ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**5192.9**	ΣM$_V$ =	**72,700.3**

CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	19.64	0.0624	rec	2058.9	6.00	12,353.4
H.W. on Stem Slope	11	140	0.00	23.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0626	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**5615.7**	ΣM$_V$ =	**90,602.6**

		L	W	Pressure		U	arm	Mu
Uplift Loads		ft	ft	ksf		K	ft	ft-k
	U$_B$	140	28.00	1.475	rec	-5782.5	14.00	-80,955
	U$_A$	140	28.00	-1.226	tri	2402.1	18.67	44,838
					ΣU =	**-3380.5**	ΣM$_U$ =	**-36,117**

	L	H	Pressure	ICE	arm	Mu
Horizontal Loads	ft	ft	ksf	K	ft	ft-k

BARR ENGINEERING			DATE	2/11/2011 SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls Load Cases: Case 3 500 yr. flood Panel E

	L		Force		H	arm	Mw
ICE	140	2.00	0.00	rec	0.0	36.82	0.0
	ft		k/ft		K	ft	ft-k
SOIL	140		-13.588		-1902.37	12.61'	-23982.53
Water Loads							
H_{TW}	140		17.436	tri	2441.06	7.88	19235.51
H_{HW}	140		-0.499	tri	-69.89	1.33	-93.18
				ΣWater =	**2371.17**	ΣM_W =	**-4840.2**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-40957	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	163303	kip-ft
Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**122,346**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,428**	kips
Sum of Horizontal Forces	H = Σhorizontal	**469**	kips

Location of Resultant

Xr = ΣM / P = 16.47 ft from Toe

e = B/2 - Xr = (2.47) ft

B/6 = 4.667 ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	701	cy	9812	sf
Total =	1,290			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing	LB /ft	Length	# of bars	Total wt		
	a) Footing		in		ft	ea	lb		
Top mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
								cy	**LB/cy**
							106,991	589	181.6374748
	b) Skin Reinf. On Monolith								
Vert Face	Vertical	9	6	3.40	33.32	280	31,721	63441.28	
	Longitudinal:	9	6	3.40	139.5	67	31,778	63556.2	
Top Face	Transverse:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	33.3	280	31,721		
	Vertical O.F.	9	6	3.40	33.3	280	31,721		
								cy	**LB/cy**
							134,066	701	191.1270455
						Σ =	**241,058**		
	Lap Splices (long. Bars)		9	3.40	8	467	12,702		
						Σ Bar Wt=	253,760	lb	

FORCES AT THE BOTTOM OF THE STEM

Diversion Face	H	γ	Pbase	**V**	arm	**Mv**
	ft	kcf		K	ft	ft-k
Diversion WSEL	19.64	0.0624	1.225536	12.035	6.547	78.78759
Tributary SEL =	33.82	0.019	0.64258	10.866	11.273	122.4964
Tributary WSEL =	0.00	0.0624	0	0.000	0.000	0
Sum				10.866		122.4964
Net Forces				-1.169		43.70877

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls Load Cases: Case 4 T.O. Levee	Panel E

ID#	Case 4
Name	T.O. Levee
Load Category	Extreme
Tributary - Water El. (ft)	NA
Diversion - Head Water El. (ft)	917.5
Diversion - Tail Water El. (ft)	917.5
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El.(ft)	898.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	18.8
Diversion - Head Water height (ft)	33.82
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

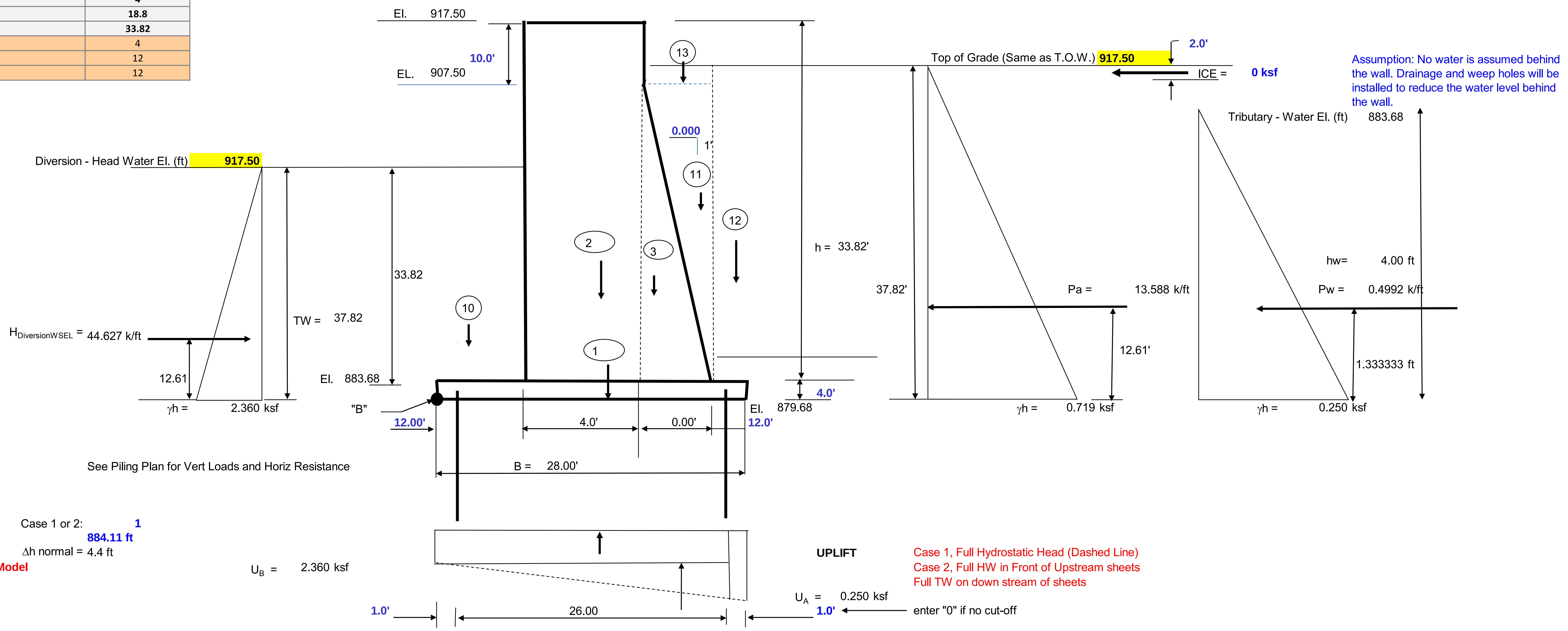
MN State Building Codes

Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March

Water El. = 903.24 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section Length = 140.0 ft

Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Assumption: No water is assumed behind the wall. Drainage and weep holes will be installed to reduce the water level behind the wall.

See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1

Normal Water Level, El. 884.11 ft

Δh normal = 4.4 ft

See Geotechnical seepage Model

UPLIFT

Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

		L	W	H	γ	shape	V	arm	Mv
Vertical Loads	**Section**	ft	ft	ft	kcf		K	ft	ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**5192.9**	ΣM_V =	**72,700.3**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	33.82	0.0624	rec	3545.4	6.00	21,272.5
H.W. on Stem Slope	11	140	0.00	23.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0626	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**7102.2**	ΣM_V =	**99,521.7**

		L	W	Pressure		U	arm	Mu
Uplift Loads		ft	ft	ksf		K	ft	ft-k
	U_B	140	28.00	2.360	rec	-9251.1	14.00	-129,515
	U_A	140	28.00	-2.110	tri	4136.3	18.67	77,211
					ΣU =	**-5114.8**	ΣM_U =	**-52,304**

	L	H	Pressure	ICE	arm	Mu
Horizontal Loads	ft	ft	ksf	K	ft	ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls Load Cases: Case 4 T.O. Levee	Panel E

	L		Force		H	arm	Mw
ICE	140	2.00	0.00	rec	0.0	36.82	0.0
	ft		k/ft		K	ft	ft-k
SOIL	140		-13.588		-1902.37	12.61'	-23982.53
Water Loads							
H_{TW}	140		44.627	tri	6247.78	12.61	78763.67
H_{HW}	140		-0.499	tri	-69.89	1.33	-93.18
				ΣWater =	**6177.89**	ΣM_W =	**54688.0**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	2384	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	172222	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**174,606**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,180**	kips
Sum of Horizontal Forces	H = Σhorizontal	**4,276**	kips

Location of Resultant	Xr = ΣM / P =	24.32	ft from Toe
	e = B/2 - Xr =	(10.32)	ft
	B/6 =	4.667	ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	701	cy	9812	sf
Total =	1,290			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing	LB /ft	Length	# of bars	Total wt		
	a) Footing		in		ft	ea	lb		
Top mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
								cy	**LB/cy**
							106,991	589	181.6374748
	b) Skin Reinf. On Monolith								
Vert Face	Vertical	9	6	3.40	33.32	280	31,721	63441.28	
	Longitudinal:	9	6	3.40	139.5	67	31,778	63556.2	
Top Face	Transverse:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	33.3	280	31,721		
	Vertical O.F.	9	6	3.40	33.3	280	31,721		
								cy	**LB/cy**
							134,066	701	191.1270455
						Σ =	**241,058**		
	Lap Splices (long. Bars)		9	3.40	8	467	12,702		
						Σ Bar Wt=	253,760	lb	

FORCES AT THE BOTTOM OF THE STEM

	H	γ	Pbase	V	arm	Mv
Diversion Face	ft	kcf		K	ft	ft-k
Diversion WSEL	33.82	0.0624	2.110368	35.686	11.273	402.3038
Tributary SEL =	33.82	0.019	0.64258	10.866	11.273	122.4964
Tributary WSEL =	0.00	0.0624	0	0.000	0.000	0
Sum				10.866		122.4964
Net Forces				-24.820		-279.807

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls Load Cases: Case 5 Normal flow + ice	Panel E

ID#	Case 5
Name	Normal flow + ice
Load Category	Usual
Tributary - Water El. (ft)	903.24
Diversion - Head Water El. (ft)	NA
Diversion - Tail Water El. (ft)	NA
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El.(ft)	898.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	4.54
Diversion - Head Water height (ft)	NA
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

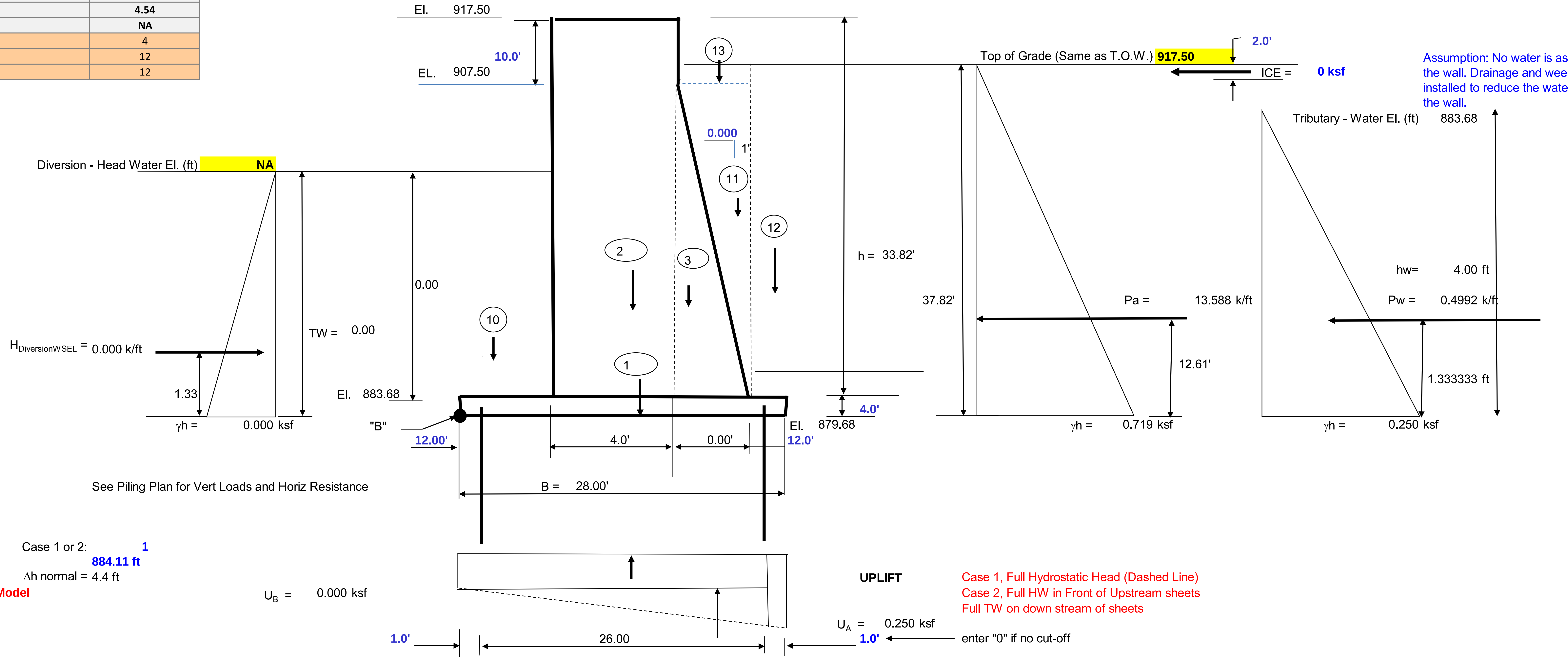
MN State Building Codes

Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March

Water El. = 903.24 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length =** 140.0 ft

Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Assumption: No water is assumed behind the wall. Drainage and weep holes will be installed to reduce the water level behind the wall.

Case 1 or 2: 1

Normal Water Level, El. 884.11 ft

Δh normal = 4.4 ft

See Geotechnical seepage Model

Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Vertical Loads	**Section**	L ft	W ft	H ft	γ kcf	shape	**V** K	arm ft	**Mv** ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**5192.9**	ΣM_V =	**72,700.3**
T.W on ftg Stem	10	140	12.00	0.00	0.0624	rec	0.0	6.00	0.0
H.W. on Stem Slope	11	140	0.00	23.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0626	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**3556.8**	ΣM_V =	**78,249.2**

← CONSTANT FOR ALL LOAD CASES

Uplift Loads	L ft	W ft	Pressure ksf		**U** K	arm ft	**Mu** ft-k
U_B	140	28.00	0.000	rec	0.0	14.00	0
U_A	140	28.00	0.250	tri	-489.2	18.67	-9,132
				ΣU =	**-489.2**	ΣM_U =	**-9,132**

Horizontal Loads	L ft	H ft	Pressure ksf	**ICE** K	arm ft	**Mu** ft-k

BARR ENGINEERING			DATE	**2/11/2011**	SHEET NO.
			PROJECT NAME	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	**34091004**	
MBI		**MBI**	SUBJECT	**Sheyenne Aquaduct Structure - Retaining Walls**	
2/11/11				**Load Cases:** **Case 5** **Normal flow + ice**	**Panel E**

ICE	140	2.00	0.00	**rec**	0.0	36.82	0.0	
	L		Force		**H**	arm	**Mw**	
	ft		k/ft		K	ft	ft-k	
SOIL	140		-13.588		-1902.37	12.61'	-23982.53	
Water Loads								
H_{TW}	140		0.000	**tri**	0.00	1.33	0.00	
H_{HW}	140		-0.499	**tri**	-69.89	0.00	0.00	
				ΣWater =	**-69.89**	ΣM_W =	**-23982.5**	

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-33115	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	150950	kip-ft
Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**117,835**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**8,260**	kips
Sum of Horizontal Forces	H = Σhorizontal	**-1,972**	kips

Location of Resultant

Xr = ΣM / P = 14.26 ft from Toe

e = B/2 - Xr = (0.26) ft

B/6 = 4.667 ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	701	cy	9812	sf
Total =	1,290			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing in	LB /ft	Length ft	# of bars ea	Total wt lb			
	a) Footing									
Top mat	Transverse:	9	6	3.40	27.5	284	26,554			
	Longitudinal:	9	6	3.40	141.5	56	26,942			
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554			
	Longitudinal:	9	6	3.40	141.5	56	26,942			
								cy		**LB/cy**
							106,991		589	181.6374748
	b) Skin Reinf. On Monolith									
Vert Face	Vertical	9	6	3.40	33.32	280	31,721	63441.28		
	Longitudinal:	9	6	3.40	139.5	67	31,778	63556.2		
Top Face	Transverse:	9	6	3.40	3.5	280	3,332			
	Longitudinal:	9	6	3.40	139.5	8	3,794			
Dowels	Vertical I.F.	9	6	3.40	33.3	280	31,721			
	Vertical O.F.	9	6	3.40	33.3	280	31,721			
								cy		**LB/cy**
							134,066		701	191.1270455
						Σ =	**241,058**			
	Lap Splices (long. Bars)		9	3.40	8	467	12,702			
						Σ Bar Wt=	253,760	lb		

FORCES AT THE BOTTOM OF THE STEM

Diversion Face		H	γ	Pbase	**V**	arm	**Mv**
		ft	kcf		K	ft	ft-k
Diversion WSEL		0.00	**0.0624**	0	0.000	0.000	0
Tributary SEL =		33.82	**0.019**	0.64258	10.866	11.273	122.4964
Tributary WSEL =		0.00	**0.0624**	0	0.000	0.000	0
	Sum				10.866		122.4964
	Net Forces				10.866		122.4964

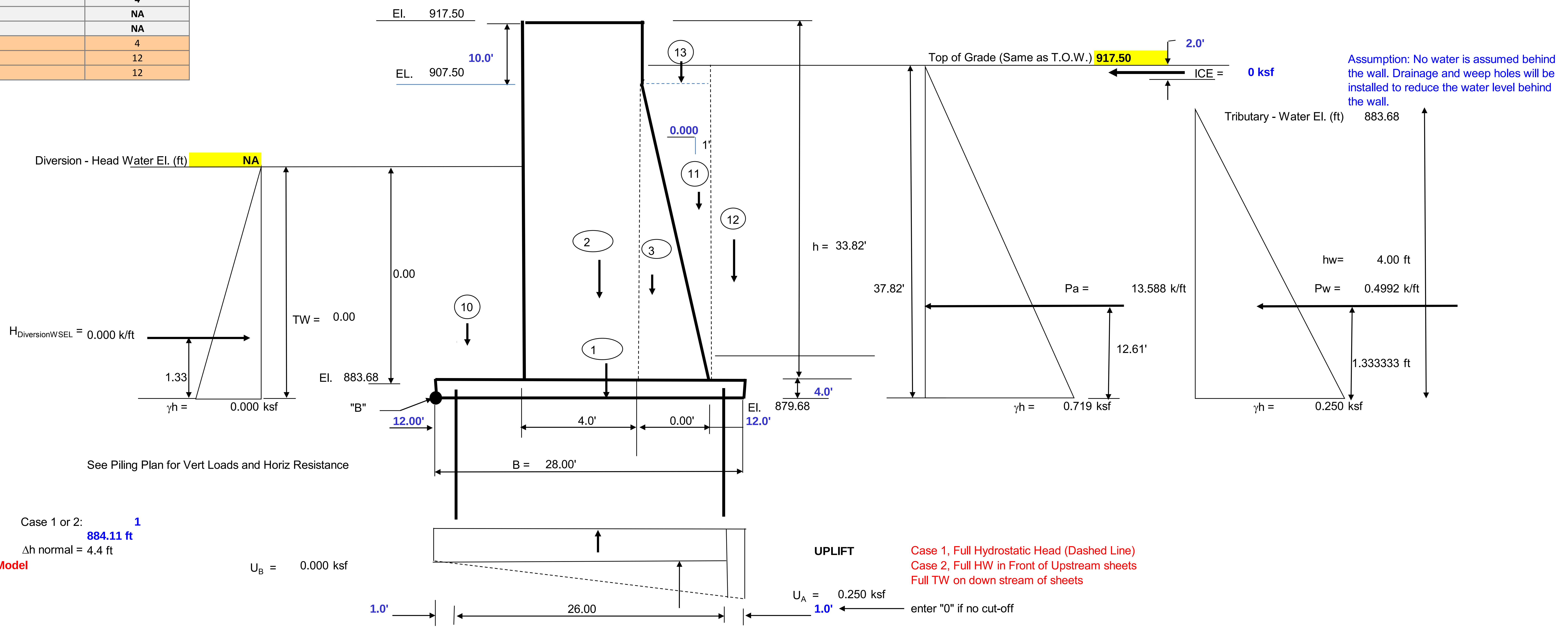
BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls Load Cases: Case 6 Construction	Panel E

ID#	Case 6
Name	Construction
Load Category	Unusual
Tributary - Water El. (ft)	NA
Diversion - Head Water El. (ft)	NA
Diversion - Tail Water El. (ft)	NA
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El.(ft)	898.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	NA
Diversion - Head Water height (ft)	NA
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 903.24 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section
Length = 140.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



Assumption: No water is assumed behind the wall. Drainage and weep holes will be installed to reduce the water level behind the wall.

See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1
Normal Water Level, El. 884.11 ft
Δh normal = 4.4 ft
See Geotechnical seepage Model

Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**5192.9**	ΣMV =	**72,700.3**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	0.00	0.0624	rec	0.0	6.00	0.0
H.W. on Stem Slope	11	140	0.00	23.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0626	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**3556.8**	ΣMV =	**78,249.2**

Uplift Loads		L ft	W ft	Pressure ksf			**U** K	arm ft	**Mu** ft-k
	UB	140	28.00	0.000		rec	0.0	14.00	0
	UA	140	28.00	0.250		tri	-489.2	18.67	-9,132
						ΣU =	**-489.2**	ΣMU =	**-9,132**

Horizontal Loads		L ft	H ft	Pressure ksf			**ICE** K	arm ft	**Mu** ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls Load Cases: Case 6 Construction Panel E	

ICE	140	2.00	0.00	rec	0.0	36.82	0.0
	L ft		Force k/ft		**H** K	arm ft	**Mw** ft-k
SOIL	140		-13.588		-1902.37	12.61'	-23982.53
Water Loads							
H_{TW}	140		0.000	tri	0.00	1.33	0.00
H_{HW}	140		-0.499	tri	-69.89	1.33	-93.18
				ΣWater =	**-69.89**	ΣM_W =	**-24075.7**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-33208	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	150950	kip-ft
Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**117,742**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**8,260**	kips
Sum of Horizontal Forces	H = Σhorizontal	**-1,972**	kips
Location of Resultant	Xr = ΣM / P =	14.25	ft from Toe
	e = B/2 - Xr =	(0.25)	ft
	B/6 =	4.667	ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	701	cy	9812	sf
Total =	1,290			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing in	LB /ft	Length ft	# of bars ea	Total wt lb		
	a) Footing								
Top mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
								cy	**LB/cy**
							106,991	589	181.6374748
	b) Skin Reinf. On Monolith								
Vert Face	Vertical	9	6	3.40	33.32	280	31,721	63441.28	
	Longitudinal:	9	6	3.40	139.5	67	31,778	63556.2	
Top Face	Transverse:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	33.3	280	31,721		
	Vertical O.F.	9	6	3.40	33.3	280	31,721		
								cy	**LB/cy**
							134,066	701	191.1270455
						Σ =	**241,058**		
	Lap Splices (long. Bars)		9	3.40	8	467	12,702		
						Σ Bar Wt=	253,760	lb	

FORCES AT THE BOTTOM OF THE STEM

Diversion Face	H ft	γ kcf	Pbase	**V** K	arm ft	**Mv** ft-k
Diversion WSEL	0.00	0.0624	0	0.000	0.000	0
Tributary SEL =	33.82	0.019	0.64258	10.866	11.273	122.4964
Tributary WSEL =	0.00	0.0624	0	0.000	0.000	0
Sum				10.866		122.4964
Net Forces				10.866		122.4964

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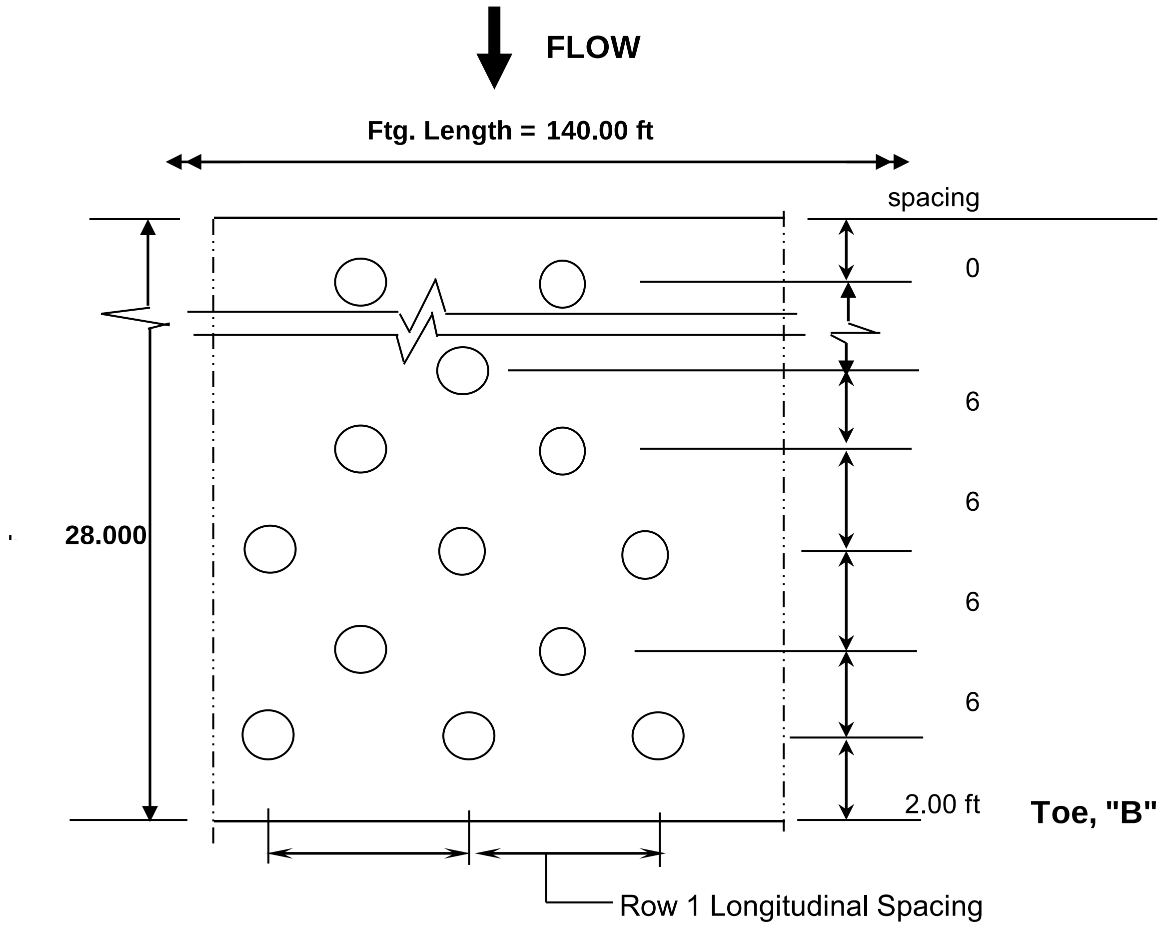
2/11/11				**Panel E**		

PILE FOUNDATION DESIGN



L = 140.00 ft

PILE PATTERN GEOMETRY

Transverse Spacing		**Distance to Toe, d_{toe}**	**Longitudinal Spacing**	**Batter**	Piles per Row (**N**)	Edge Dist (ft)		**Trial N**
Row 1 to Toe	2.00 ft	2.0 ft	2.50 ft	0 "/12"	23	42.50	1	57
Row 1 to Row 2	6.00 ft	8.0 ft	5.00 ft	0 "/12"	22	17.50	2	29
Row 2 to Row 3	6.00 ft	14.0 ft	5.00 ft	0 "/12"	22	17.50	3	29
Row 3 to Row 4	6.00 ft	20.0 ft	5.00 ft	0 "/12"	24	12.50	4	29
Row 4 to Row 5	6.00 ft	26.0 ft	5.00 ft	0 "/12"	24	12.50	5	29
Row 5 to Row 6	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 6 to Row 7	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 7 to Row 8	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 8 to Row 9	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 9 to Row 10	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 10 to Row 11	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 11 to Row 12	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 12 to Row 13	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 13 to Row 14	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 14 to Row 15	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Last Row to Heel	2.00 ft							
	28.00 ft		Note: Enter 0 for Longitudinal Spacing for Rows Not Used)		ΣN = 115			173

Pile Properties:

Pile Type: HP (C.I.P or HP)

Pile Length = 38.0 ft

Ftg EL. 879.68

Pile Tip El. 842.68

HP Nominal Depth, h = 14.0 in

Pile Cap Embed = 1.00 ft

Wt. per ft, plf 73

Total pile Length = 4,370 LF

ALLOWABLE LOADS (from Geotechnical)

	Service	Allowable Pile Loads					
	ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
	Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
	Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Allwable Lateral Capacity (tons)		18.0 tons	20.5 tons	20.5 tons	24.0 tons	11.5 tons	20.5 tons
Allowable Pile Capacity (tons) - Axial		62.0 tons	82.6 tons	82.6 tons	107.7 tons	36.5 tons	82.6 tons
Safety Factors		2.00	1.50	1.50	1.15	2.00	1.50

w/o Group effects

Pile Group Properties

N.A. of Pile Group to Toe

$X_{NA} = (\Sigma N * d_{toe}) / \Sigma N =$ **14.21 ft**

	Dist. From N.A. to Pile Row	d	N	$I = N * d^2$
1	Dist. To Row 1	12.21 ft	23	3428.2
2	Dist. To Row 2	6.21 ft	22	848.1
3	Dist. Row 3	0.21 ft	22	1.0
4	Dist. Row 4	-5.79 ft	24	804.9
5	Dist. Row 5	-11.79 ft	24	3336.8
0	Row 6 (not used)	0.00 ft	0	0.0
0	Row 7 (not used)	0.00 ft	0	0.0
0	Row 8 (not used)	0.00 ft	0	0.0
0	Row 9 (not used)	0.00 ft	0	0.0
0	Row 10 (not used)	0.00 ft	0	0.0
0	Row 11 (not used)	0.00 ft	0	0.0
0	Row 12 (not used)	0.00 ft	0	0.0
0	Row 13 (not used)	0.00 ft	0	0.0
0	Row 14 (not used)	0.00 ft	0	0.0
0	Row 15 (not used)	0.00 ft	0	0.0
			115	Σ I = **8419.0**

Summary Pile Reactions

Load Combinations	Allowable Pile Capacity (tons) - Axial	Pile Loads (tons/pile)												Max. Vertical Load (Tons)	Horiz Pile Group Capacity (k)	Check
		1	2	3	4	5	6	7	8	9	10	11	12			
Case 1	**62.0 tons**	22.0	27.1	32.2	37.3	42.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	42.4	4,140	**OK**
Case 2	**82.6 tons**	22.0	27.1	32.2	37.3	42.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	42.4	4,715	**OK**
Case 3	**82.6 tons**	20.1	26.1	32.1	38.1	44.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	44.1	4,715	**OK**
Case 4	**107.7 tons**	-21.4	4.5	30.3	56.2	82.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	82.0	5,520	**OK**
Case 5	**36.5 tons**	35.6	35.7	35.9	36.1	36.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	36.2	2,645	**OK**
Case 6	**82.6 tons**	35.6	35.8	35.9	36.0	36.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	36.2	2,645	**OK**

Max Service : P = **82.0**

Using solid mechanics equations adapted for discrete elements, the forces in the pile rows for different load combinations are determined.
The force in each pile row is found using:

$$\text{Pile Load} = P / N + M_{NA} / I$$

First, the moment about the toe must be translated to get the moment about the neutral axis of the pile group.

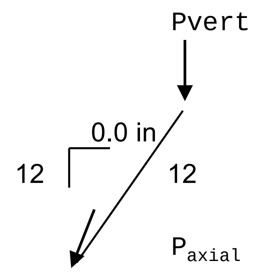
$$e_{toe} = M_{toe} / P$$

Then the eccentricity about the neutral axis of the pile group is

$$e_{NA} = X_{NA} - e_{toe}$$

The moment about the neutral axis of the pile group becomes

$$M_{NA} = P * e_{NA}$$



For battered pile, the Vertical pile load needs to be transformed to the axial load along the pile axis

Paxial = 1.000 Pvert

FORCE RESULTANT (see Stability Analysis)

CASE	Event		Vertical Load P (kips)	Horizontal	ΣM_{toe} (kip-	$e_{toe} = M_{toe} / P$	$e_{NA} = X_{NA} - e_{toe}$	$M_{NA} = P * e_{NA}$

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2/11/11				**Panel E**	

Case 1	**100 yr. flood**	**Usual**	7,449	-227	120,177	16.13	-1.92	-14336
Case 2	**100 yr. flood + ice**	**Unusual**	7,449	-227	120,177	16.13	-1.92	-14336
Case 3	**500 yr. flood**	**Unusual**	7,428	-469	122,346	16.47	-2.26	-16802
Case 4	**T.O. Levee**	**Extreme**	7,180	-4,276	174,606	24.32	-10.11	-72583
Case 5	**Normal flow + ice**	**Usual**	8,260	1,972	117,835	14.26	-0.06	-465
Case 6	**Construction**	**Unusual**	8,260	1,972	117,742	14.25	-0.04	-372

SERVICE

Case **Case 1**
Flood Event **100 yr. flood**
Usual

Vertical Load, P = 7449 kips
Horizontal Load, H = -227 kips
M_{NA} = -14336 kip-ft 115

	Vertical Pile Loading	P / N	+	M_{NA}* d / Σ I	= Pile Loads				**Axial Pile Load**
1	Row 1	64.8		-20.8	44.0 kips/pile		22.0 tons/pile		22.0 tons/pile
2	Row 2	64.8		-10.6	54.2 kips/pile		27.1 tons/pile		27.1 tons/pile
3	Row 3	64.8		-0.4	64.4 kips/pile		32.2 tons/pile		32.2 tons/pile
4	Row 4	64.8		9.9	74.6 kips/pile		37.3 tons/pile		37.3 tons/pile
5	Row 5	64.8		20.1	84.9 kips/pile		42.4 tons/pile		42.4 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
						max:	**42.4 tons/pile**	**max:**	**42.4 tons/pile**

Assumed lateral Capacity: 36.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	23	0.0	828	1.000	828 kips	
2	Row 2	0	22	0.0	792	1.000	792 kips	
3	Row 3	0	22	0.0	792	1.000	792 kips	
4	Row 4	0	24	0.0	864	1.000	864 kips	
5	Row 5	0	24	0.0	864	1.000	864 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			115		4140		**4140 kips**	**OK**

Case **Case 2**
Flood Event **100 yr. flood + ice**
Unusual

Vertical Load, P = 7449 kips
Horizontal Load, H = -227 kips
M_{NA} = -14336 kip-ft 115

	Vertical Pile Loading	P / N	+	M_{NA}* d / Σ I	= Pile Loads				**Axial Pile Load**
1	Row 1	64.8		-20.8	44.0 kips/pile		22.0 tons/pile		22.0 tons/pile
2	Row 2	64.8		-10.6	54.2 kips/pile		27.1 tons/pile		27.1 tons/pile
3	Row 3	64.8		-0.4	64.4 kips/pile		32.2 tons/pile		32.2 tons/pile
4	Row 4	64.8		9.9	74.6 kips/pile		37.3 tons/pile		37.3 tons/pile
5	Row 5	64.8		20.1	84.9 kips/pile		42.4 tons/pile		42.4 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
						max:	**42.4 tons/pile**	**max:**	**42.4 tons/pile**

Assumed lateral Capacity: 41.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance
1	Row 1	0	23	0.0	943	1.000	943 kips
2	Row 2	0	22	0.0	902	1.000	902 kips
3	Row 3	0	22	0.0	902	1.000	902 kips

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4	Row 4	0	24	0.0	984	1.000	984 kips	
5	Row 5	0	24	0.0	984	1.000	984 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			115		4715		**4715 kips**	**OK**

Case **Case 3**
Flood Event **500 yr. flood**
Unusual

Vertical Load, P = 7428 kips
Horizontal Load, H = -469 kips
M_{NA} = -16802 kip-ft

	Vertical Pile Loading	P / N	+	M_{NA}* d / Σ I	= Pile Loads			**Axial Pile Load**
1	Row 1	64.6		-24.4	40.2 kips/pile	20.1 tons/pile		20.1 tons/pile
2	Row 2	64.6		-12.4	52.2 kips/pile	26.1 tons/pile		26.1 tons/pile
3	Row 3	64.6		-0.4	64.2 kips/pile	32.1 tons/pile		32.1 tons/pile
4	Row 4	64.6		11.6	76.2 kips/pile	38.1 tons/pile		38.1 tons/pile
5	Row 5	64.6		23.5	88.1 kips/pile	44.1 tons/pile		44.1 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
					max:	**44.1 tons/pile**	**max:**	**44.1 tons/pile**

Assumed lateral Capacity: 41.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	23	0.0	943	1.000	943 kips	
2	Row 2	0	22	0.0	902	1.000	902 kips	
3	Row 3	0	22	0.0	902	1.000	902 kips	
4	Row 4	0	24	0.0	984	1.000	984 kips	
5	Row 5	0	24	0.0	984	1.000	984 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			115		4715		**4715 kips**	**OK**

Case **Case 4**
Flood Event **T.O. Levee**
Extreme

Vertical Load, P = 7180 kips
Horizontal Load, H = -4276 kips
M_{NA} = -72583 kip-ft

	Vertical Pile Loading	P / N	+	M_{NA}* d / Σ I	= Pile Loads			**Axial Pile Load**
1	Row 1	62.4		-105.3	-42.8 kips/pile	-21.4 tons/pile		-21.4 tons/pile
2	Row 2	62.4		-53.5	8.9 kips/pile	4.5 tons/pile		4.5 tons/pile
3	Row 3	62.4		-1.8	60.6 kips/pile	30.3 tons/pile		30.3 tons/pile
4	Row 4	62.4		49.9	112.4 kips/pile	56.2 tons/pile		56.2 tons/pile
5	Row 5	62.4		101.7	164.1 kips/pile	82.0 tons/pile		82.0 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
					max:	**82.0 tons/pile**	**max:**	**82.0 tons/pile**

Assumed lateral Capacity: 48.0 kips/pile

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	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	23	0.0	1104	1.000	1104 kips	
2	Row 2	0	22	0.0	1056	1.000	1056 kips	
3	Row 3	0	22	0.0	1056	1.000	1056 kips	
4	Row 4	0	24	0.0	1152	1.000	1152 kips	
5	Row 5	0	24	0.0	1152	1.000	1152 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			115		5520		**5520 kips**	**OK**

Case **Case 5**
Flood Event **Normal flow + ice**
Usual

Vertical Load, P = 8260 kips
Horizontal Load, H = 1972 kips
M_{NA} = -465 kip-ft

	Vertical Pile Loading	P / N	+	M_{NA}* d / Σ I	= Pile Loads			**Axial Pile Load**
1	Row 1	71.8		-0.7	71.2 kips/pile		35.6 tons/pile	35.6 tons/pile
2	Row 2	71.8		-0.3	71.5 kips/pile		35.7 tons/pile	35.7 tons/pile
3	Row 3	71.8		0.0	71.8 kips/pile		35.9 tons/pile	35.9 tons/pile
4	Row 4	71.8		0.3	72.1 kips/pile		36.1 tons/pile	36.1 tons/pile
5	Row 5	71.8		0.7	72.5 kips/pile		36.2 tons/pile	36.2 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
						max:	**36.2 tons/pile**	**max: 36.2 tons/pile**

Assumed lateral Capacity: 23.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	23	0.0	529	1.000	529 kips	
2	Row 2	0	22	0.0	506	1.000	506 kips	
3	Row 3	0	22	0.0	506	1.000	506 kips	
4	Row 4	0	24	0.0	552	1.000	552 kips	
5	Row 5	0	24	0.0	552	1.000	552 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			115		2645		**2645 kips**	**OK**

Case **Case 6**
Flood Event **Construction**
Unusual

Vertical Load, P = 8260 kips
Horizontal Load, H = 1972 kips
M_{NA} = -372 kip-ft

	Vertical Pile Loading	P / N	+	M_{NA}* d / Σ I	= Pile Loads		**Axial Pile Load**
1	Row 1	71.8		-0.5	71.3 kips/pile	35.6 tons/pile	35.6 tons/pile
2	Row 2	71.8		-0.3	71.6 kips/pile	35.8 tons/pile	35.8 tons/pile
3	Row 3	71.8		0.0	71.8 kips/pile	35.9 tons/pile	35.9 tons/pile
4	Row 4	71.8		0.3	72.1 kips/pile	36.0 tons/pile	36.0 tons/pile
5	Row 5	71.8		0.5	72.4 kips/pile	36.2 tons/pile	36.2 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile

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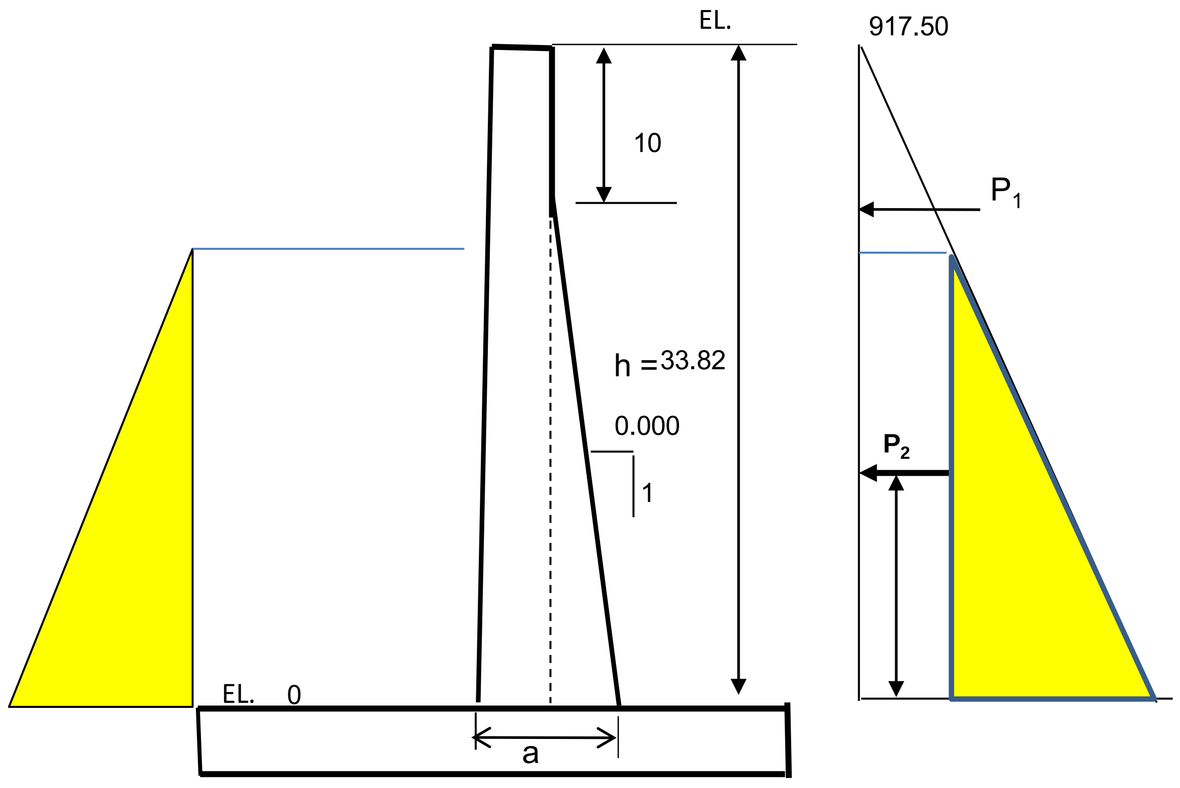
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
2/11/11				Panel E	

11	Row 11	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
					max:	**36.2 tons/pile**	**max:**	**36.2 tons/pile**

Assumed lateral Capacity: 41.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	23	0.0	529	1.000	529 kips	
2	Row 2	0	22	0.0	506	1.000	506 kips	
3	Row 3	0	22	0.0	506	1.000	506 kips	
4	Row 4	0	24	0.0	552	1.000	552 kips	
5	Row 5	0	24	0.0	552	1.000	552 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			115		2645		**2645 kips**	**OK**

BARR ENGINEERING			DATE	**2/11/2011**	SHEET NO.
			PROJECT NAME	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	**34091004**	
MBI **2/11/11**		**MBI**	SUBJECT	**Sheyenne Aquaduct Structure - Retaining Walls** **Panel E 0**	



CASE	Event		HW	TW	Dh	TW -ftg
Case 1	100 yr. flood	Usual	902.12	901.91	0.21	901.91
Case 2	100 yr. flood + ice	Unusual	902.12	901.91	0.21	901.91
Case 3	500 yr. flood	Unusual	914.670	903.06	11.61	903.06
Case 4	T.O. Levee	Extreme	917.500	917.50	0.00	917.50
Case 5	Normal flow + ice	Usual	0.000	0.000	0.00	0.00
Case 6	Construction	Unusual	0.000	0.000	0.00	0.00

LOAD FACTORS

Hf =	1.30	hydraulic Factor	
LF =	1.70		
Unsual & Extreme =	0.75		
TOP THICK =	**4.0 ft**	48.0 in	
Batter at Base =	0.00 ft	0.0 in	
a =	4.00 ft	48.0 in	

Load Factors - Hydraulic Structures

live load, LL =	**1.7**
dead load, DL =	**1.4**
flood level , FL =	**1**
Fluid, F =	**1.7**
hydraulic, Hf =	**1.3**
direct tension hydraulic, Hf =	**1.65**
ICE =	**1.7**

WALL DESIGN:

Horizontal Load Components and Moments about Bottom of Stem (Service)

CASE	Event	Condition	Load Factor	H (kips/ft)	Moment (kip-ft/ ft)	Vu (kips/ft)	Mu (kip-ft/ ft)
Case 1	100 yr. flood	Usual	1	0.26	57.286	0.57	126.60
Case 2	100 yr. flood + ice	Unusual	0.75	0.26	57.286	0.43	94.95
Case 3	500 yr. flood	Unusual	0.75	-1.17	43.709	-1.94	72.45
Case 4	T.O. Levee	Extreme	0.75	-24.82	-279.807	41.14	463.78
Case 5	Normal flow + ice	Usual	1	10.87	122.496	24.01	270.72
Case 6	Construction	Unusual	0.75	10.87	122.496	18.01	203.04

	STEM DESIGN VALUES	
MU, k-ft/ft	463.78	k-ft/ft
VU, k/ft	41.14	k/ft

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MBI **2/11/11**		**MBI**	SUBJECT	**Sheyenne Aquaduct Structure - Retaining Walls** **Panel E 0**	

ACI 318-05 w/ Modifications per EM 1110-2-2104 ref. EM 110-2-2104

9.3 - Design Strength

ϕ = **0.9** 9.3.2.1 - Tension Controlled sections

0.75 9.3.2.3 - Shear and torsion

FLEXURAL STEEL FOR RECTANGULAR CONCRETE SECTIONS

fy= **60** ksi

Fc'= **4** ksi

B1= 0.85

Muh = 464 k-ft /ft **Includes: hf = 1.3**

Vuh= 41.14 k / ft

bw= 12 in.

h= 48 in

cover= **4** in (include correct stirrup bar dia.)

d= **43.50** in

pb= 0.0285 pb=0.85*B1*Fc'/ fy*(87 / (87+fy))

.75*pb= 0.0214

m=fy / 0.85*Fc'= 17.647

TRIAL

Ru=Mn/bd^2 =	272.328		**ACI 10.5.1**			**ACI 10.5.3**
REQ'D p=	0.0047	**O.K.**	p(min)= 3*SQRT(Fc')/fy		200* / fy	4/3*p
p=	FALSE	N.G.		0.00316	0.00333	0.0063

EM 110-2-2104 2-8 c. (not less than Temp & Shrinkage, half in each face)

As (REQ'D)= 0.81 in^2 p(min)= 0.0028 /2 → As =0.5*$p_{T\&S}$bh = 0.8064 in^2

As = #9 @ 12 = 1.00 in^2

SELECT STEEL

bar #= **9**

spacing, s= **6** in

\# OF BAR= **1** (ENTER 1 IF PER FT, b=12") a

As= 1.999 in^2

d= 43.4375 in

p = As/bd = 0.0038 **O.K. < 0.375pb** **EM 110-2-2104**

p = 0.135 pb **MAXIMUM TENSILE REINFORCEMENT**

a) For singly reinforced flexural members

1) p = **0.25** pb Recommended limit

2) p = **0.375** pb Max. permitted upper limit not requiring special study

3) p = **0.5** pb Max. permitted upper limit when excessive deflections are not predicted In ACI 318

4) p = >**0.5** pb but ≤ 0.375 pb permitted only if detailed serviceability analysis incl. deflect. Calc.

b) Use of compression reinf. shall be per ACI 318

T= As*fy = 119.9 k

C= B1*Fc' *b*a = 1576.9 **a**

a= T/C = 0.076 in

Mn= T(d - a/2)/12 = 433.7 ft-k

ϕ Mn= 390.3 ft-k **< Mu N.G.**

CHECK SHEAR REINFORCEMENT (ACI 11.3 & EM 110-2-2104 3-3a)

Vuh = 41.1 k **NO SHEAR REINF. REQUIRED**

Vn = Vuh / ϕ = 54.9 k

Vc = 2*sqrt(Fc') bw * d = 65.9 k **11.3.1.1**

Vs =Vuh / f - 1.3Vc = No Shear Reinf. Req. k NG Vs(max) ≤ 8*sqrt(fc')bd = 263.7 k

Trial Stirrup Sizes:

\# of stirrup legs = **2** (single stirrup = 2, Dbl stirrup = 4………)

Stirrup bar size = **4**

Av= 0.393 in^2

s= 0.000 in s = Av * fy * d / (Vu / f - Vc)

11.5.5 - Spacing limits for shear reinforcement

s = d/2 = 21.719 in OR 24 in

s(max)= 10.859 in

4*sqrt(Fc')*bw*d= 131.9 k < Vs Reduce Spacing

USE s= **0.00** in

Vs = (Av * Fy * d) / s = #DIV/0! k

11.5.6 - MINIMUM SHEAR REINFORCEMENT

A minimum area of shear reinforcement, Av,min shall be provided in all reinforced concrete flexural members where **Vu** exceeds **0.5 f Vc**

NOT REQUIRED IF:

a) SLAB OR FOOTING, vc>vn **O.K.**

b) CONCRETE JOIST ACI 8.11

c) BEAMS W/ h <= 10"

h <= 2.5*Bf

h <= 0.5*tw

d) WALLS (SEE ACI 11.10.1); vc>vn **O.K.**

11.5.6.3

Av,min = 0.75 sqrt(fc') bw*s/fy = 0.70 * s

but not less than 50bw*s/ fy = 23.33333333 * s

s max = Av fy / 0.75 sqrt(fc') bw = 0.00 in

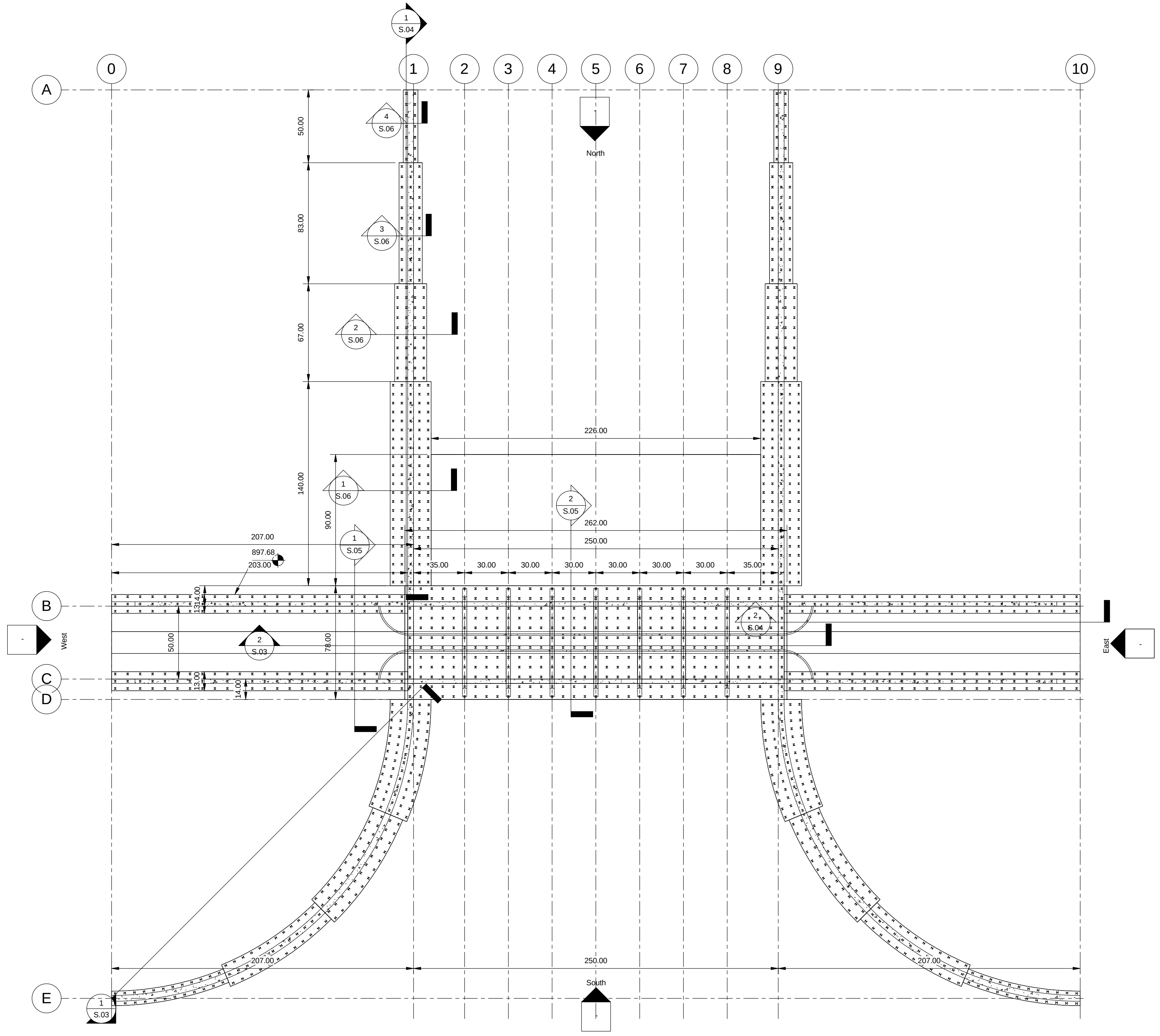
s max = Av fy /50 bw = 0.00 in

11.5.5.3

Where Vs exceeds 4*sqrt(Fc')*bw*d maximum spacings shall be reduced by one-half

ATTACHMENT

F-R4.4 Sheyenne River Aqueduct Structure Drawings



① S01 - FOUNDATION PLAN
1" = 40'-0"

02/14/11

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FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – SHEYENNE AQUEDUCT –
FOUNDATION PLAN

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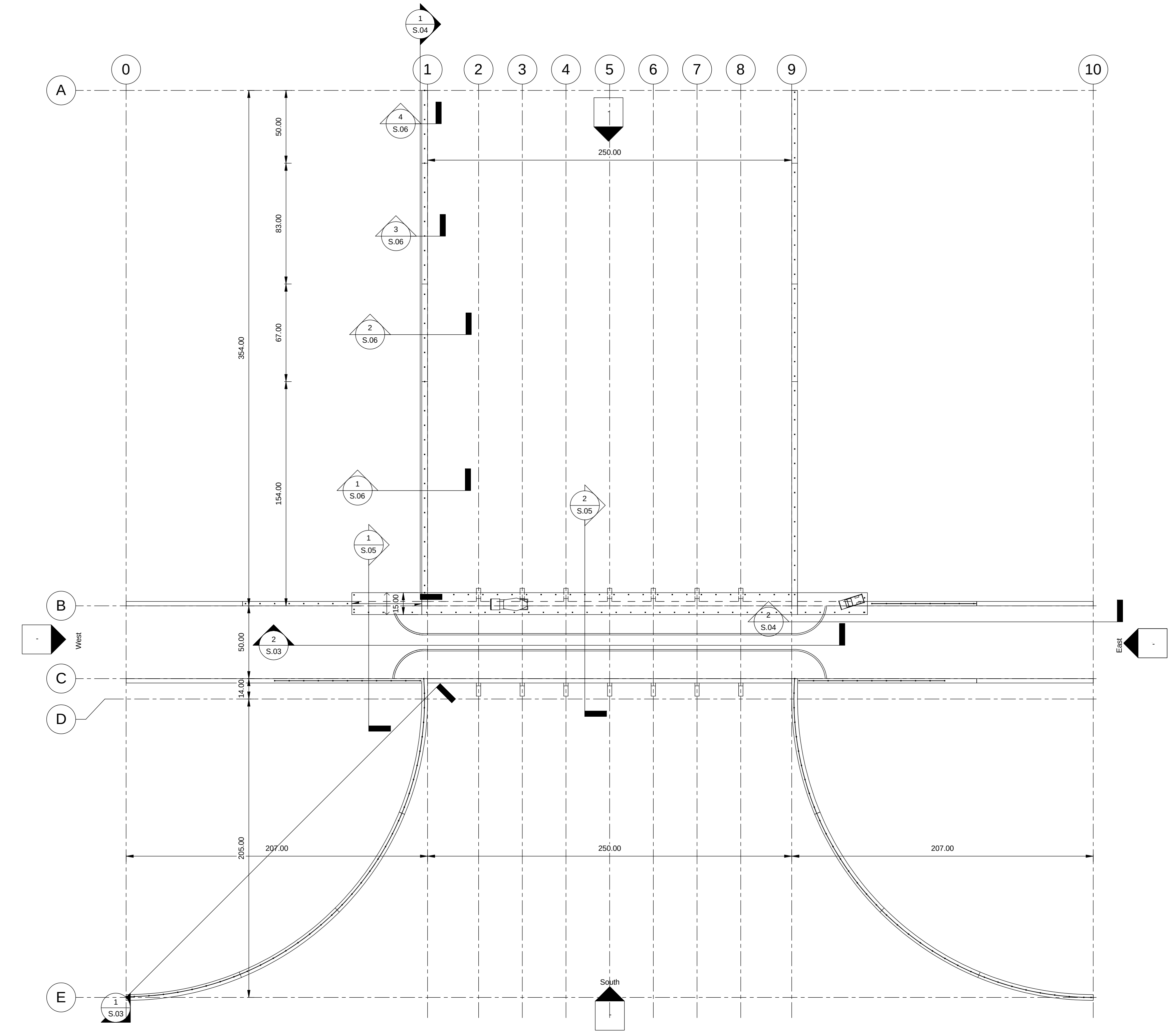

① S02 - TOP PLAN
1" = 40'-0"

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FLOOD CONTROL – SHEYENNE AQUEDUCT –
GENERAL ARRANGEMENT PLAN

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CLIENT PROJECT No.

DWG No.
S.02

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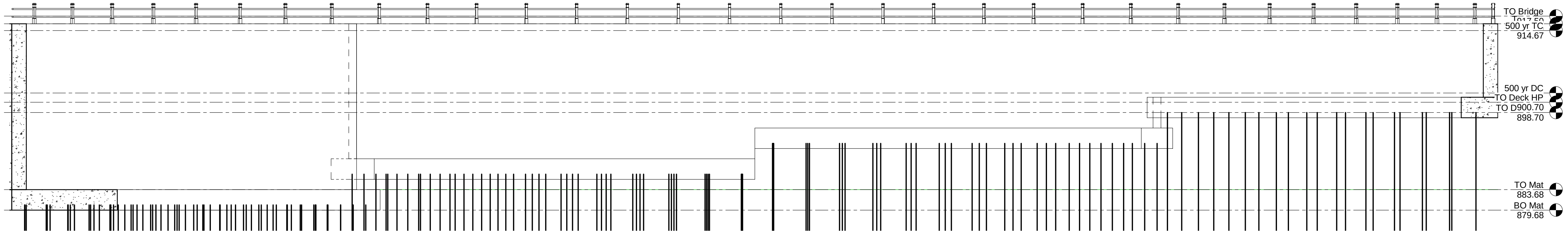
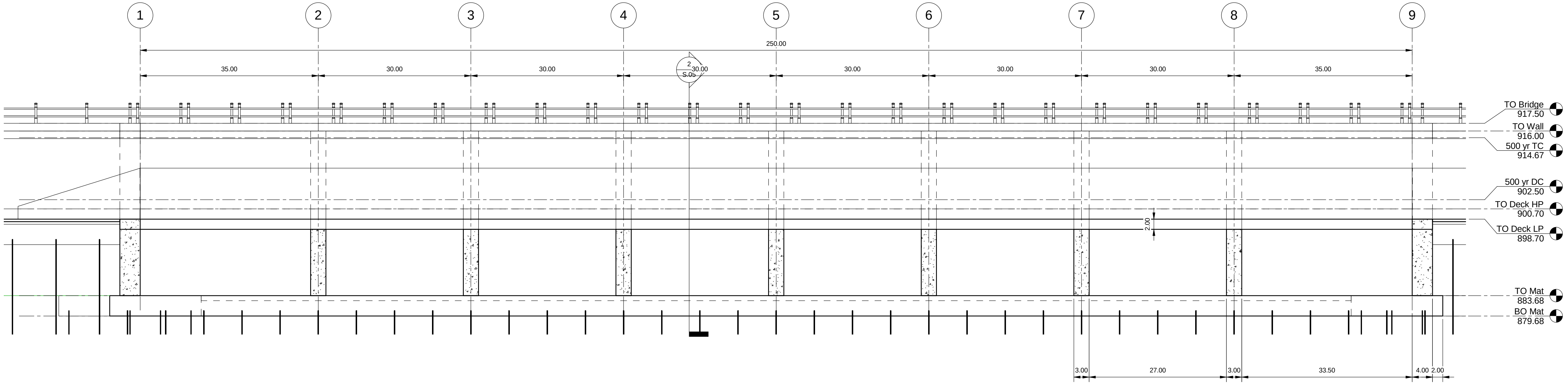


① Section 1
1" = 10'-0"



② Section 2
1" = 10'-0"

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FLOOD CONTROL – SHEYENNE AQUEDUCTT –
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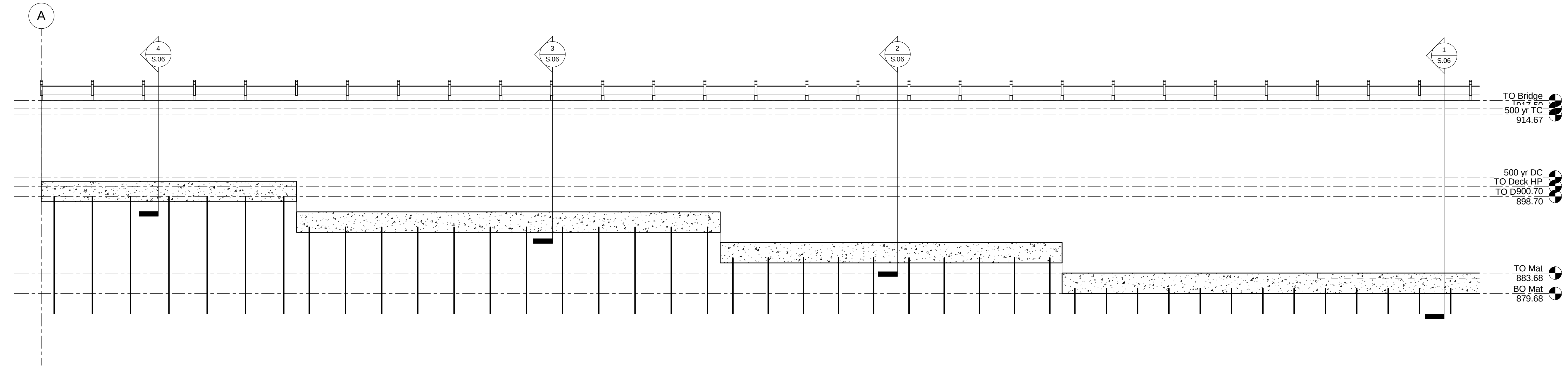
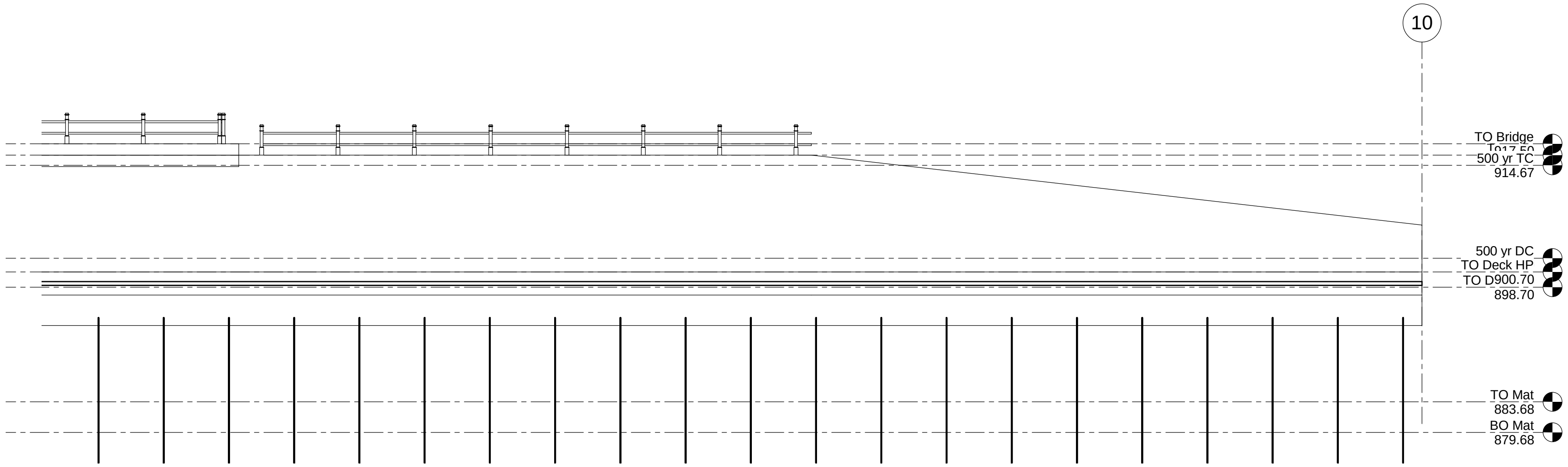


① Section 5
1" = 10'-0"



② Section 11
1" = 10'-0"

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FLOOD CONTROL – SHEYENNE AQUEDUCTT – ELEVATIONS

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DWG No. S.04
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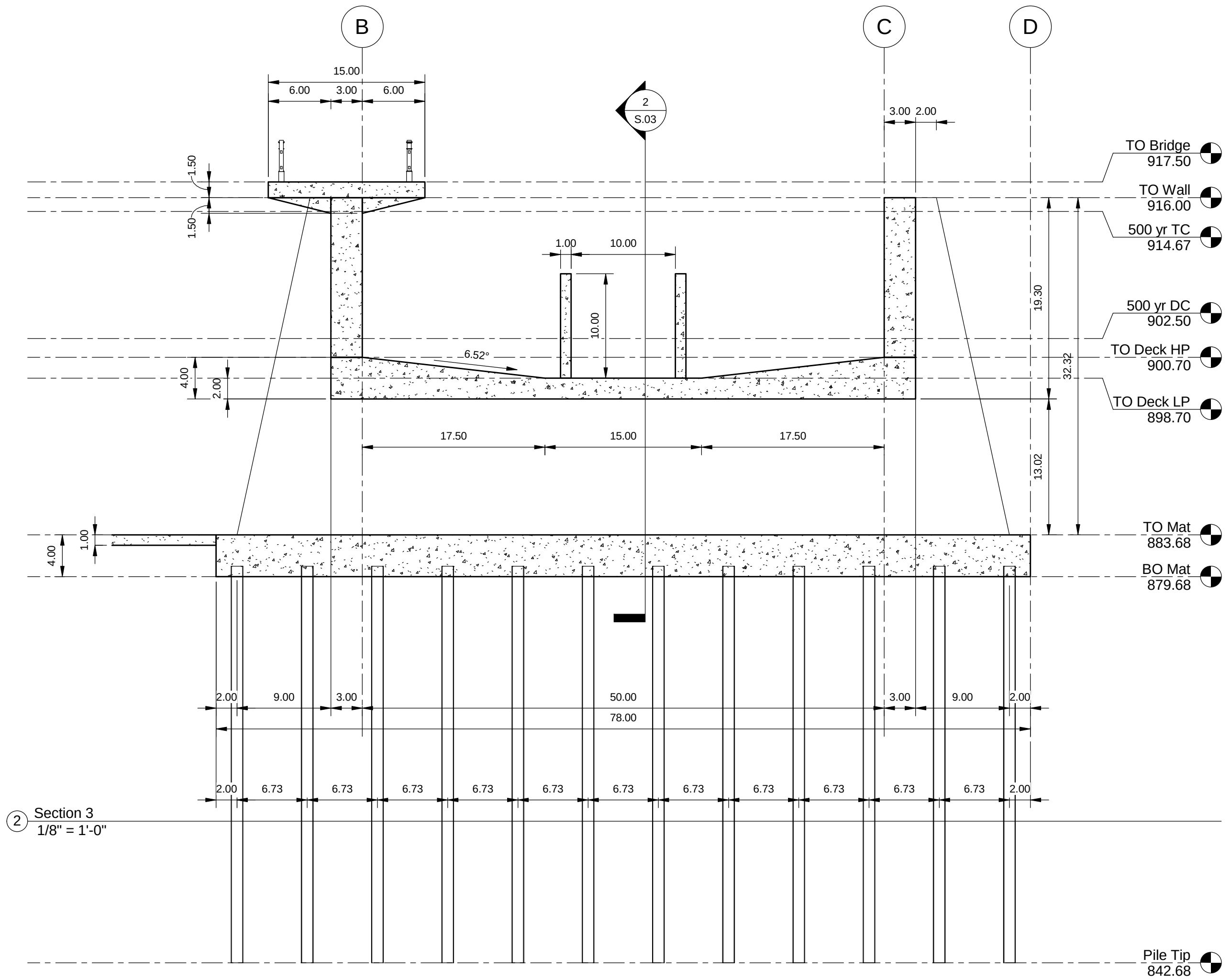
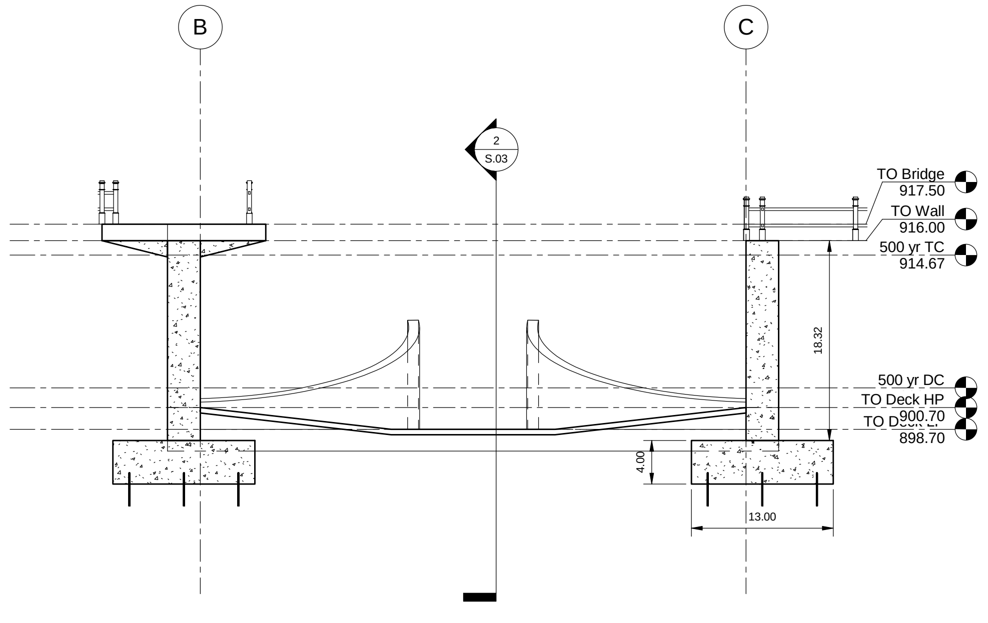


② Section 3
1/8" = 1'-0"



① Section 7
1/8" = 1'-0"

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RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – SHEYENNE AQUEDUCT –
OVERALL SECTIONS

BARR PROJECT No.
34091004

CLIENT PROJECT No.

DWG No.
S.05

REV No.

Structural Foundation Schedule

Type	Structural Usage	Count	Width	Length	Foundation Thickness	Area	Forming	Volume	Volume CY	Reinf. (lbs/sf)	Total Reinf. (lbs)
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	50.00	4.00	500 SF	510 SF	2000.00 CF	74.07 CY	27.2	13600
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	80.50	4.00	805 SF	769 SF	3220.13 CF	119.26 CY	27.2	21897
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	50.00	4.00	500 SF	510 SF	2000.00 CF	74.07 CY	27.2	13600
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	80.50	4.00	805 SF	769 SF	3220.13 CF	119.26 CY	27.2	21897
Retaining Footing - 36" x 36" x 48": 4						2610 SF	2559 SF	10440.25 CF	386.68 CY		70994
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	123.00	4.00	1599 SF	1156 SF	6396.00 CF	236.89 CY	27.2	43493
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	123.00	4.00	1599 SF	1156 SF	6396.00 CF	236.89 CY	27.2	43493
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	123.00	4.00	1599 SF	1156 SF	6396.00 CF	236.89 CY	27.2	43493
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	123.00	4.00	1599 SF	1156 SF	6396.00 CF	236.89 CY	27.2	43493
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	80.00	4.00	1040 SF	791 SF	4160.00 CF	154.07 CY	27.2	28288
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	80.00	4.00	1040 SF	791 SF	4160.00 CF	154.07 CY	27.2	28288
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	80.00	4.00	1040 SF	791 SF	4160.00 CF	154.07 CY	27.2	28288
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	80.00	4.00	1040 SF	791 SF	4160.00 CF	154.07 CY	27.2	28288
Retaining Footing - 60" x 60" x 48": 8						10556 SF	7786 SF	42224.00 CF	1563.85 CY		287123
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	83.00	4.00	1328 SF	842 SF	5312.00 CF	196.74 CY	27.2	36122
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	80.50	4.00	1288 SF	820 SF	5152.22 CF	190.82 CY	27.2	35035
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	83.00	4.00	1328 SF	842 SF	5312.00 CF	196.74 CY	27.2	36122
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	80.50	4.00	1288 SF	820 SF	5152.22 CF	190.82 CY	27.2	35035
Retaining Footing - 72" x 72" x 48": 4						5232 SF	3324 SF	20928.43 CF	775.13 CY		142313
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	67.00	4.00	1474 SF	757 SF	5896.00 CF	218.37 CY	27.2	40093
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	80.50	4.00	1771 SF	871 SF	7084.30 CF	262.38 CY	27.2	48173
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	67.00	4.00	1474 SF	757 SF	5896.00 CF	218.37 CY	27.2	40093
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	80.50	4.00	1771 SF	871 SF	7084.30 CF	262.38 CY	27.2	48173
Retaining Footing - 108" x 108" x 48": 4						6490 SF	3256 SF	25960.59 CF	961.50 CY		176532
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	140.00	4.00	3920 SF	1428 SF	15680.00 CF	580.74 CY	27.2	106624
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	80.50	4.00	2254 SF	922 SF	9016.38 CF	333.94 CY	27.2	61311
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	140.00	4.00	3920 SF	1428 SF	15680.00 CF	580.74 CY	27.2	106624
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	80.50	4.00	2254 SF	922 SF	9016.38 CF	333.94 CY	27.2	61311
Retaining Footing - 144" x 144" x 48": 4						12348 SF	4701 SF	49392.76 CF	1829.36 CY		335871
Grand total: 24						37237 SF	21624 SF	148946.04 CF	5516.52 CY		1012833

Floor Schedule

Type	Comments	Elevated	Type Comments	Structural Usage	Count	Thickness	Perimeter	Area	Forming	Volume	Volume (cy)	Est. Reinf.	Total Reinf. (lbs)	Takeoff
12" Concrete	SOG	No	Apron Slab	Slab	1	1.00	632.00	20340 SF	632	20340.00 CF	753.33 CY	13.6	276624	Yes
Apron Slab: 1							632.00	20340 SF	632	20340.00 CF	753.33 CY		276624	
18" Concrete	Deck	Yes	Bridge	Slab	1	1.50	546.00	3870 SF	4689	5805.00 CF	215.00 CY	27.2	105264	Yes
18" Concrete	Deck	Yes	Bridge	Slab	1	1.50	126.00	720 SF	909	1080.00 CF	40.00 CY	27.2	19584	Yes
18" Concrete	Deck	Yes	Bridge	Slab	1	1.50	126.00	720 SF	909	1080.00 CF	40.00 CY	27.2	19584	Yes
Bridge: 3							798.00	5310 SF	6507	7965.00 CF	295.00 CY		144432	
24" Concrete	Deck	Yes	Deck Slab	Slab	1	2.00	628.00	14507 SF	15763	41022.00 CF	1519.33 CY	27.2	394584	Yes
Deck Slab: 1							628.00	14507 SF	15763	41022.00 CF	1519.33 CY		394584	
6" Concrete	RipRap	No	Grading	Slab	1	0.50	506.00	10196 SF	253	5098.12 CF	0.00 CY	0	0	No
6" Concrete	RipRap	No	Grading	Slab	1	0.50	765.40	21978 SF	383	10988.91 CF	0.00 CY	0	0	No
6" Concrete	RipRap	No	Grading	Slab	1	0.50	506.00	10196 SF	253	5098.12 CF	0.00 CY	0	0	No
Grading: 3							1777.40	42370 SF	889	21185.15 CF	0.00 CY		0	
48" Concrete	SOG	No	Mat Foundation	Slab	1	4.00	680.00	20436 SF	2720	81744.00 CF	3027.56 CY	27.2	555859	Yes
Mat Foundation: 1							680.00	20436 SF	2720	81744.00 CF	3027.56 CY		555859	
Grand total: 9							4515.40	102963 SF	26510	172256.15 CF	5595.22 CY		1371500	

Structural Column Schedule

Type	Count	Length	Top Level	Base Level	Base Offset	Top Offset	Volume	Height
HP14X89	790	30020.00	BO Mat	Pile Tip	0.00	1.00	6.81 CF	38
38.00: 790	790	30020.00						
HP14X89	176	7744.00	BO Mat	Pile Tip	0.00	7.00	7.89 CF	44
44.00: 176	176	7744.00						
HP14X89	144	7200.00	BO Mat	Pile Tip	0.00	13.00	8.96 CF	50
50.00: 144	144	7200.00						
HP14X89	288	14976.00	BO Mat	Pile Tip	0.00	15.00	9.32 CF	52
52.00: 288	288	14976.00						
HP14X89	76	4256.00		Pile Tip	0.00		10.04 CF	56
56.00: 76	76	4256.00						
Grand total: 1474	1474	64196.00						

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#	BY	CHK.	DATE	REVISION DESCRIPTION

SIGNATURE ____
PRINTED NAME ____
DATE ____ REG. NO. ____

CLIENT							
BID							
CONSTRUCTION							
RELEASED TO/FOR	A	B	C	0	1	2	3
	DATE RELEASED						

BARR

Corporate Headquarters:
Minneapolis, Minnesota
Ph: 1-800-632-2277

Project Office:
BARR ENGINEERING CO.
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
Ph: 1-800-632-2277
Fax: (952) 832-2600
www.barr.com

Scale	
Date	Issue Date
Drawn	MBI
Checked	TSH
Designed	MBI
Approved	Approver

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL - SHEYENNE AQUEDUCT -
SCHEDULES

BARR PROJECT No.
34091004

CLIENT PROJECT No.

DWG No.
S.07

REV No.

Wall Schedule												
Type	Type Comments	Comments	Length	Width	Height	Area	Forming	Volume	Volume CY	Reinf. (lbs/sf)	Total Reinf. (lbs)	Takeoff
Exterior - 72" Concrete	Aquaduct	Deck	258.00	6.00	1.50	387 SF	792 SF	1161.00 CF	43.00 CY	27.2	10526	
Exterior - 72" Concrete	Aquaduct	Deck	254.00	6.00	1.48	375 SF	768 SF	375.00 CF	13.89 CY	27.2	10200	
Exterior - 72" Concrete	Aquaduct	Deck	48.00	6.00	1.50	72 SF	162 SF	216.00 CF	8.00 CY	27.2	1958	
Exterior - 72" Concrete	Aquaduct	Deck	48.00	6.00	1.50	72 SF	162 SF	216.00 CF	8.00 CY	27.2	1958	
Exterior - 72" Concrete	Aquaduct	Deck	50.00	6.00	1.44	72 SF	161 SF	72.00 CF	2.67 CY	27.2	1958	
Exterior - 72" Concrete	Aquaduct	Deck	50.00	6.00	1.44	72 SF	161 SF	72.00 CF	2.67 CY	27.2	1958	
Deck: 6			708.00			1050 SF	2206 SF	2112.00 CF	78.22 CY		28560	
Exterior - 12" Concrete	Aquaduct	Low Flow Walls	250.05	1.00	10.00	2501 SF	5022 SF	2500.49 CF	92.61 CY	27.2	68026	
Exterior - 12" Concrete	Aquaduct	Low Flow Walls	250.05	1.00	10.00	2501 SF	5022 SF	2500.49 CF	92.61 CY	27.2	68026	
Exterior - 12" Concrete	Aquaduct	Low Flow Walls	35.46	1.00	5.29	187 SF	385 SF	186.32 CF	6.90 CY	27.2	5098	
Exterior - 12" Concrete	Aquaduct	Low Flow Walls	35.46	1.00	5.29	187 SF	385 SF	186.32 CF	6.90 CY	27.2	5098	
Exterior - 12" Concrete	Aquaduct	Low Flow Walls	35.46	1.00	9.80	348 SF	715 SF	339.55 CF	12.58 CY	27.2	9456	
Exterior - 12" Concrete	Aquaduct	Low Flow Walls	35.46	1.00	9.80	348 SF	715 SF	339.55 CF	12.58 CY	27.2	9456	
Low Flow Walls: 6			641.92			6072 SF	12245 SF	6052.70 CF	224.17 CY		165160	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Pier: 21			546.00			7592 SF	17089 SF	22777.44 CF	843.61 CY		206515	

Wall Schedule												
Type	Type Comments	Comments	Length	Width	Height	Area	Forming	Volume	Volume CY	Reinf. (lbs/sf)	Total Reinf. (lbs)	Takeoff
Exterior - 36" Concrete	Aquaduct	Wall	254.00	3.00	15.06	3825 SF	7740 SF	11475.00 CF	425.00 CY	27.2	104040	
Exterior - 36" Concrete	Aquaduct	Wall	254.00	3.00	15.06	3825 SF	7740 SF	11475.00 CF	425.00 CY	27.2	104040	
Wall: 2			508.00			7650 SF	15481 SF	22950.00 CF	850.00 CY		208080	
Aquaduct: 35			2403.92			22365 SF	47021 SF	53892.14 CF	1996.01 CY		608316	
Retaining - 36" Concrete	Retaining	Wall	125.00	3.00	18.03	2253 SF	4615 SF	6760.08 CF	250.37 CY	27.2	61291	
Retaining - 36" Concrete	Retaining	Wall	125.00	3.00	18.03	2253 SF	4615 SF	6760.08 CF	250.37 CY	27.2	61291	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	34.52	431 SF	1139 SF	1725.92 CF	63.92 CY	27.2	11736	
Exterior - 48" Concrete	Retaining	Wall	140.00	4.00	33.82	4735 SF	9740 SF	18939.20 CF	701.45 CY	27.2	128787	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	35.84	448 SF	1183 SF	1791.92 CF	66.37 CY	27.2	12185	
Exterior - 48" Concrete	Retaining	Wall	67.00	4.00	27.82	1864 SF	3950 SF	7455.76 CF	276.14 CY	27.2	50699	
Exterior - 48" Concrete	Retaining	Wall	83.00	4.00	21.82	1811 SF	3797 SF	7244.24 CF	268.31 CY	27.2	49261	
Exterior - 48" Concrete	Retaining	Wall	50.00	4.00	15.82	791 SF	1709 SF	3164.00 CF	117.19 CY	27.2	21515	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	32.64	2627 SF	5516 SF	10407.49 CF	385.46 CY	27.2	71461	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	26.58	2140 SF	4492 SF	8475.40 CF	313.90 CY	27.2	58195	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	20.52	1652 SF	3468 SF	6543.31 CF	242.34 CY	27.2	44929	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	14.46	1164 SF	2444 SF	4611.22 CF	170.79 CY	27.2	31662	
Exterior - 48" Concrete	Retaining	Wall	53.00	4.00	15.02	796 SF	1712 SF	3184.24 CF	117.93 CY	27.2	21653	
Retaining - 36" Concrete	Retaining	Wall	125.00	3.00	18.03	2253 SF	4615 SF	6760.08 CF	250.37 CY	27.2	61291	
Retaining - 36" Concrete	Retaining	Wall	125.00	3.00	18.03	2253 SF	4615 SF	6760.08 CF	250.37 CY	27.2	61291	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	34.40	430 SF	1135 SF	1719.80 CF	63.70 CY	27.2	11695	
Exterior - 48" Concrete	Retaining	Wall	140.00	4.00	33.82	4735 SF	9740 SF	18939.20 CF	701.45 CY	27.2	128787	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	36.08	451 SF	1191 SF	1803.80 CF	66.81 CY	27.2	12266	
Exterior - 48" Concrete	Retaining	Wall	67.00	4.00	27.82	1864 SF	3950 SF	7455.76 CF	276.14 CY	27.2	50699	
Exterior - 48" Concrete	Retaining	Wall	83.00	4.00	21.82	1811 SF	3797 SF	7244.24 CF	268.31 CY	27.2	49261	
Exterior - 48" Concrete	Retaining	Wall	50.00	4.00	15.82	791 SF	1709 SF	3164.00 CF	117.19 CY	27.2	21515	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	32.64	2627 SF	5516 SF	10407.49 CF	385.46 CY	27.2	71461	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	26.58	2140 SF	4492 SF	8475.40 CF	313.90 CY	27.2	58195	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	20.52	1652 SF	3468 SF	6543.31 CF	242.34 CY	27.2	44929	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	14.46	1164 SF	2444 SF	4611.22 CF	170.79 CY	27.2	31662	
Exterior - 48" Concrete	Retaining	Wall	53.00	4.00	14.96	793 SF	1706 SF	3172.00 CF	117.48 CY	27.2	21570	
Retaining - 36" Concrete	Retaining	Wall	80.00	3.00	13.74	1099 SF	2281 SF	2564.80 CF	94.99 CY	27.2	29898	
Retaining - 36" Concrete	Retaining	Wall	80.00	3.00	13.74	1099 SF	2281 SF	2564.80 CF	94.99 CY	27.2	29898	
Retaining - 36" Concrete	Retaining	Wall	80.00	3.00	13.74	1099 SF	2281 SF	3297.60 CF	122.13 CY	27.2	29898	
Retaining - 36" Concrete	Retaining	Wall	80.00	3.00	13.74	1099 SF	2281 SF	3297.60 CF	122.13 CY	27.2	29898	
Wall: 30			2300.03			50326 SF	105877 SF	185844.05 CF	6883.11 CY		1368880	
Retaining: 30			2300.03			50326 SF	105877 SF	185844.05 CF	6883.11 CY		1368880	
Grand total: 65			4703.95			72691 SF	152898 SF	239736.18 CF	8879.12 CY		1977196	

02/18/11

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#	BY	CHK.	DATE	REVISION DESCRIPTION

SIGNATURE ____________
PRINTED NAME ____________
DATE ________ REG. NO. ________

CLIENT							
BID							
CONSTRUCTION							
RELEASED TO/FOR	A	B	C	0	1	2	3
	DATE RELEASED						

BARR
Corporate Headquarters:
Minneapolis, Minnesota
Ph: 1-800-632-2277

Project Office:
BARR ENGINEERING CO.
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
Ph: 1-800-632-2277
Fax: (952) 832-2600
www.barr.com

Scale	
Date	Issue Date
Drawn	Author
Checked	Checker
Designed	Designer
Approved	Approver

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – SHEYENNE AQUEDUCT –
SCHEDULES – II

BARR PROJECT No.
34091004
CLIENT PROJECT No.
DWG No.
S.08
REV No.

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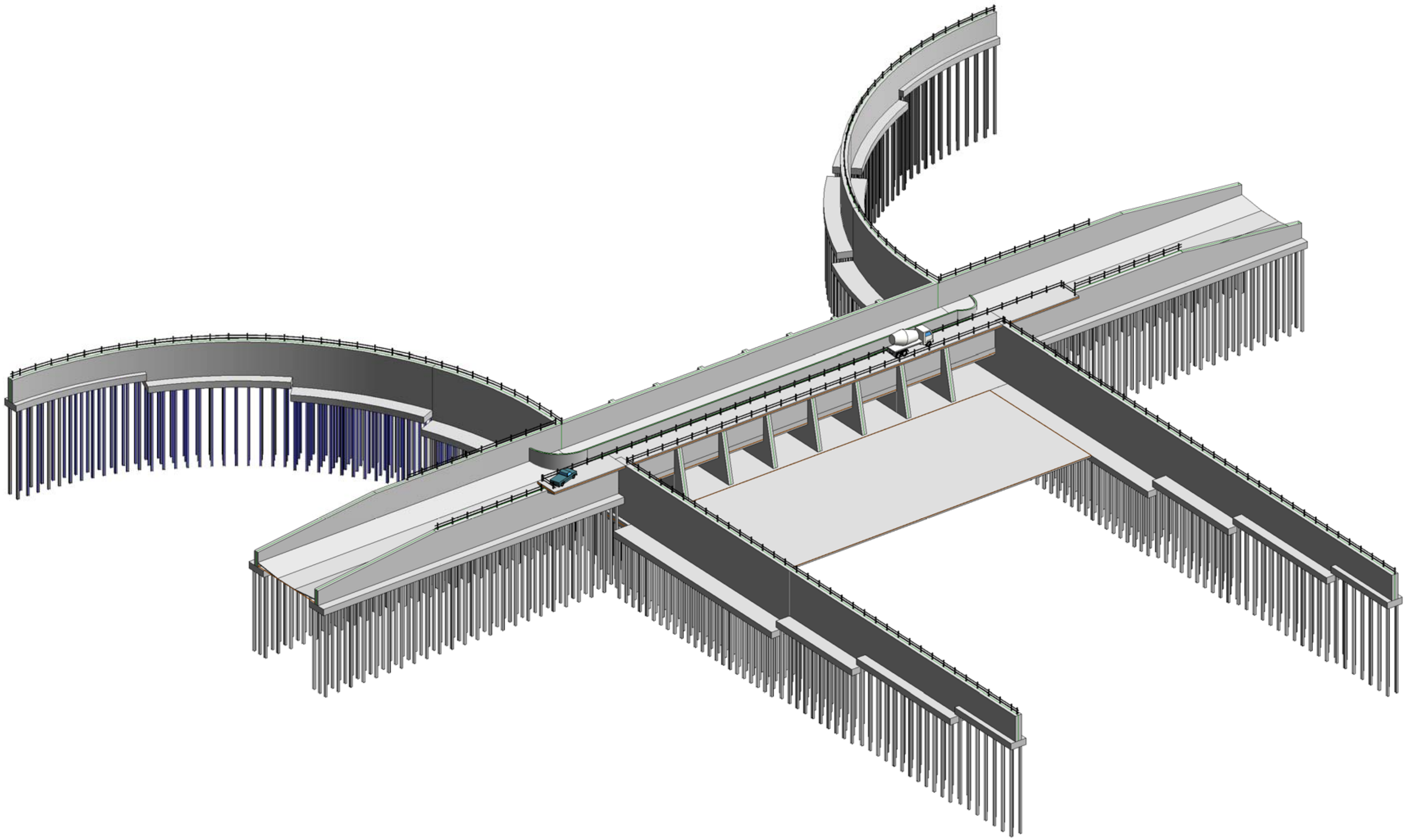

① 3D VIEW OVERALL D/S

02/14/11

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SIGNATURE
PRINTED NAME
DATE REG. NO.

CLIENT							
BID							
CONSTRUCTION							
RELEASED TO/FOR	A	B	C	0	1	2	3
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MINNEAPOLIS, MN 55435

Ph: 1-800-632-2277
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Scale	
Date	Issue Date
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Checked	TSH
Designed	MBI
Approved	Approver

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – SHEYENNE AQUEDUCT –
3D VIEWS

BARR PROJECT No.
34091004

CLIENT PROJECT No.

DWG No.
S.09

REV No.

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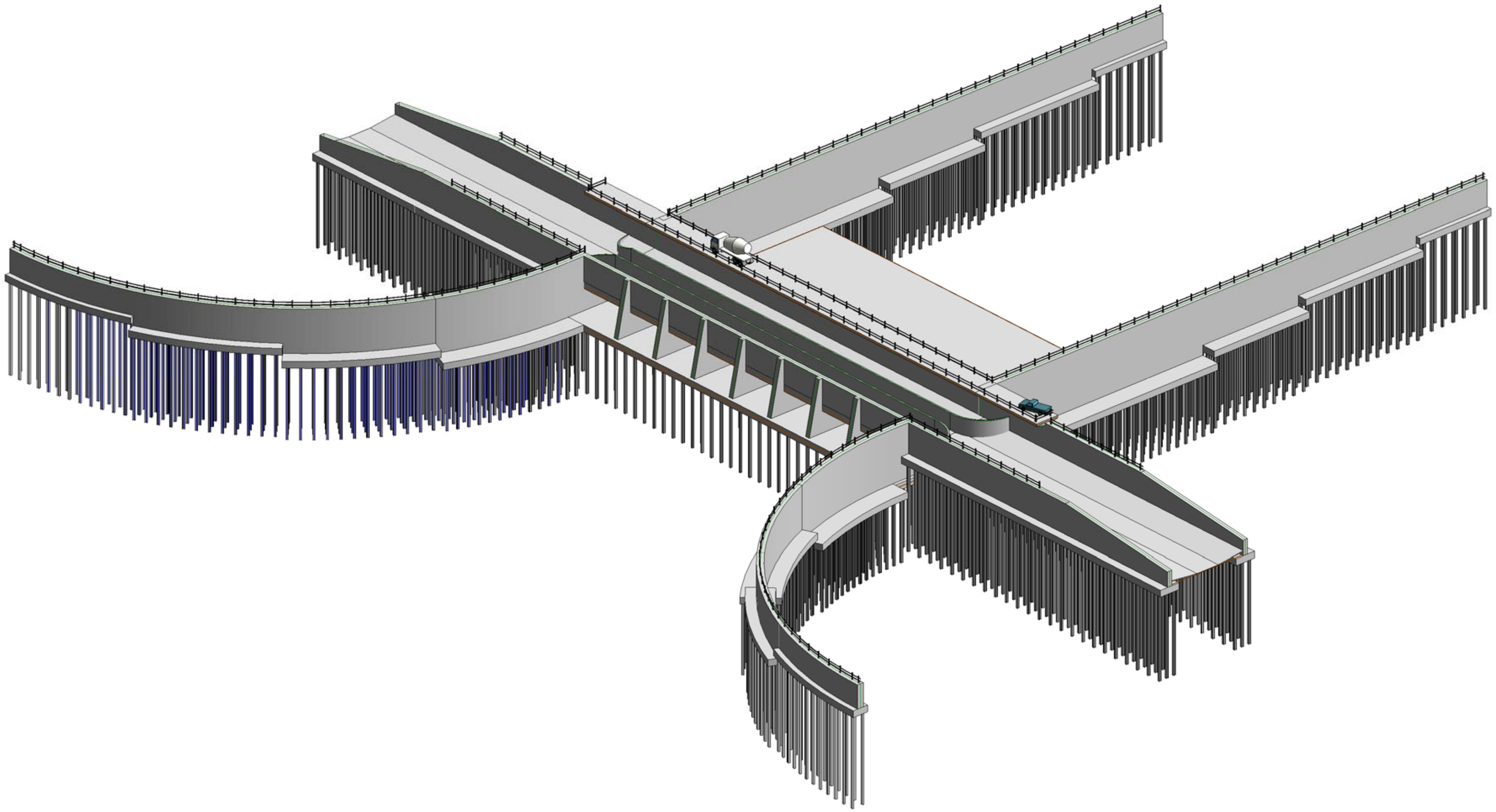


① 3D VIEW OVERALL U/S

2/26/2011 2:09:31 PM M:\cad\34091004\Feasibility-Phase4\Engineering and Design\(07) S- Structural\Revit Models\34091004 PH4 Sheyene Aqueduct Rev2.rvt

02/26/11

PRELIMINARY - FOR DESIGN PURPOSES ONLY

#	BY	CHK.	DATE	REVISION DESCRIPTION

SIGNATURE ________
PRINTED NAME ________
DATE ________ REG. NO. ________

CLIENT							
BID							
CONSTRUCTION							
RELEASED TO/FOR	A	B	C	0	1	2	3
	DATE RELEASED						

BARR

Corporate Headquarters:
Minneapolis, Minnesota
Ph: 1-800-632-2277

Project Office:
BARR ENGINEERING CO.
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

Ph: 1-800-632-2277
Fax: (952) 832-2600
www.barr.com

Scale	
Date	Issue Date
Drawn	Author
Checked	Checker
Designed	Designer
Approved	Approver

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – SHEYENNE AQUEDUCT –
3D VIEWS

BARR PROJECT No.
34091004

CLIENT PROJECT No.

DWG No.
S.10

REV No.

ATTACHMENT

F-R5.1 Maple River Aqueduct Structure Pile Computations

LOAD CASES - MAPLE AQUEDUCT - Phase 4

Client Name:	**U.S. ARMY CORPS OF ENGINEERS**	**MBI**
Project Name:	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	
Work Describtion:	**LOAD CASES - MAPLE AQUEDUCT - Phase 4**	**1/24/2011**
		34091004
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Maple\[34091004 PH4 Maple Pile Calcs.xlsx]Load Cases	

REF.	**1**
	2

ID#	**Case 1**	**Case 2**	**Case 3**	**Case 4**	**Case 5**	**Case 6**
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	895.99	895.99	896.38	902	881.5	**NA**
Diversion - Head Water El. (ft)	893.89	893.89	895.46	902	**NA**	**NA**
Diversion - Tail Water El. (ft)	892.57	892.57	893.66	902	**NA**	**NA**
Tributary - T.O. Wall El. (ft)	902					
Tributary - T.O. Deck L.P. El.(ft)	881.06					
Tributary - T.O. Deck H.P. El.(ft)	883.06					
Diversion - T.O. Mat El. (ft)	872.06					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	**14.93**	**14.93**	**15.32**	**20.94**	**0.44**	**NA**
Diversion - Head Water height (ft)	**21.83**	**21.83**	**23.4**	**29.94**	**NA**	**NA**
Ice	**NA**	2ft Ice	**NA**	**NA**	2ft Ice	**NA**
Ice Load	**NA**	10 kips/ft	**NA**	**NA**	10 kips/ft	**NA**
Ice Laod El. (ft)	**NA**	**895.99**	**NA**	**NA**	**881.5**	**NA**
Uplift @ HW (ft)	**25.83**	**25.83**	**27.4**	**33.94**	**NA**	**NA**
Uplift @ TW (ft)	**24.51**	**24.51**	**25.6**	**33.94**	**NA**	**NA**
Pile Condition	**Undrained**	**Undrained**	**Undrained**	**Undrained**	**Drained**	**Undrained**
Load Category	**Usual**	**Unusual**	**Unusual**	**Extreme**	**Usual**	**Unusual**
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	**18**	**21**	**21**	**24**	**11.5**	**21**
Allowable Pile Capacity (tons) - Axial	**57.18**	**76.23**	**76.23**	**99.43**	**31.425**	**76.23**
Allowable Pile Capacity (tons) - Uplift	**33.88**	**45.17**	**45.17**	**58.91**	**4.625**	**45.17**

Pile Capacity	**Ultimate Axial Capacity (kips)**	**Allowable Lateral Capacity (kips)**		
		0.5" (Usual)	**0.67" (Unusual)**	**0.875" (Extreme)**
Undrained - Axial	228.7	36	42	48
Undrained - Uplift	135.5			
Drained - Axial	125.7	23	29	33
Drained - Uplift	18.5			

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	**Design By:**	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	**Review By:**	
		Date:	1/24/2011
Work Describtion:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	**Job #:**	34091004'
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Maple\[34091004 PH4 Maple Pile Calcs.xlsx]Load Cases		

REF.	1
	2

Hydrolic Profile

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	895.99	895.99	896.38	902	881.5	NA
Diversion - Head Water El. (ft)	893.89	893.89	895.46	902	NA	NA
Diversion - Tail Water El. (ft)	892.57	892.57	893.66	902	NA	NA
Tributary - T.O. Wall El. (ft)	902					
Tributary - T.O. Deck L.P. El.(ft)	881.06					
Tributary - T.O. Deck H.P. El.(ft)	883.06					
Diversion - T.O. Mat El. (ft)	872.06					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	14.93	14.93	15.32	20.94	0.44	NA
Diversion - Head Water height (ft)	21.83	21.83	23.4	29.94	NA	NA
Ice	NA	2ft Ice	NA	NA	2ft Ice	NA
Ice Load	NA	10 kips/ft	NA	NA	10 kips/ft	NA
Ice Laod El. (ft)	NA	895.99	NA	NA	881.5	NA
Uplift @ HW (ft)	25.83	25.83	27.4	33.94	NA	NA
Uplift @ TW (ft)	24.51	24.51	25.6	33.94	NA	NA

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	**U.S. ARMY CORPS OF ENGINEERS**	**Design By:**	**MBI**
Project Name:	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	**Review By:**	
		Date:	**1/24/2011**
Work Describtion:	**LOAD CASES - SHEYENNE AQUEDUCT - Phase 4**	**Job #:**	**34091004'**
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Maple\[34091004 PH4 Maple Pile Calcs.xlsx]Load Cases		

Quantity Take Off (Revit)				**Material Properties**		
Volume of Walls (ft3)	Vw (ft3)	45558.53	ft3	*Concrete*	γ Concrete (pcf)	150
Volume Tributary Deck Slab (ft3)	Vs (ft3)	41022	ft3	*Steel*	γ Steel (pcf)	495
Volume of Bridge Deck (ft3)	Vs (ft3)	8127	ft3	*Soil Dry*	γs Dry (pcf)	120
Volume Diversion Mat Slab (ft3)	Vs (ft3)	81744	ft3	*Soil Saturated*	γs Sat. (pcf)	130
Total		**176451.53**	ft3	*Water*	γ Water (pcf)	62.4

Geometry

Tributary Channel			
Tributary - T.O. Wall El. (ft)		**902**	ft
Tributary - T.O. Deck L.P. El.(ft)		**881.06**	ft
Tributary - T.O. Deck H.P. El.(ft)		**883.06**	ft
Tributary - Clear Width (ft)	w TC	50	ft
Tributary - Wall Thickness (ft)	twall TC	3	ft
Tributary - Deck Slope Width (ft)	lslab slope TC	17.5	ft
Tributary - Deck Slab thickness @ L.P. (ft)	tslab TC	**2**	ft
Tributary - Deck Slab thickness @ H.P. (ft)	tslab TC	**4**	ft
Tributary - Low Flow Channel height (ft)	hlowflow TC	4	ft
Tributary - Low Flow Channel width (ft)	wlowflow TC	4	ft
Tributary - Low Flow Channel thickness (ft)	tlowwall TC	1	ft
Tributary - Wall Height (ft)	hwall TC	**22.94**	ft
Diversion Channel			
Diversion - T.O. Wall El. (ft)		**879.06**	ft
Diversion - T.O. Mat El. (ft)		**872.06**	ft
Diversion - Clear Opening Width	wopen DC	30	ft
Diversion - # of Openings	#open DC	6	
Diversion - Wall Thickness (ft)	twall DC	3	ft
Diversion - Mat Slab thickness (ft)	tslab DC	**4**	ft
Diversion - Butress height (ft)	hbutress	**29.94**	ft
Diversion - Butress Top width (ft)	wbutress Top	2	ft
Diversion - Butress Top width (ft)	wbutress Bot	9	ft
Diversion - Wall Height (ft)	hwall DC	**7**	ft
Mat Foundation			
Overall Width (ft)	wmat	**78**	ft
Overall Length (ft)	lmat	**262**	ft
Triburtary - Channel Length (ft)	lslab TC	**258**	ft
Triburtary - Channel Width (ft)	wslab TC	**56**	ft
Access Bridge			
Overall Width (ft)	wbridge	15	ft
Overall Length (ft)	lbridge	**258**	ft
Minimum Deck Thickness	tbridge	1.5	ft
Maximum Deck Thickness	tbridge	3	ft

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	**Design By:**	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	**Review By:**	
		Date:	1/24/2011
Work Describtion:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	**Job #:**	34091004'
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Maple\[34091004 PH4 Maple Pile Calcs.xlsx]Load Cases		

Weight of Structure

Tributary	Volume (ft3)	Weight (tons)
Walls	29319	2199
Deck	41022	3077
Low Flow Chanel	2064	155
Bridge	8127	860
Sub Total	80532	6290

Diversion	Volume (ft3)	Weight (tons)
Walls	9016	676
Mat	81744	6131
Butress Walls	4940	371
Sub Total	95700	7178
Total	176232	13217

Whole Structure	Volume (ft3)	Weight (tons)
Walls	45339	3400
Deck	41022	3077
Bridge	8127	860
Mat	81744	6131
Total	176232	13217

Take off (Revit)	Volume (ft3)	Weight (tons)
Walls	45559	3417
Deck	41022	3077
Bridge	8127	610
Mat	81744	6131
Total	176452	13234

Ratio 0.998757109

Forces

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water height (ft)	14.93	14.93	15.32	20.94	0.44	NA
Triburtary - Channel Length (ft)	258.00					
Tributary - Clear Width (ft)	50.00					
Tributary - Water force (psf)	931.63	931.63	955.97	1306.66	27.46	NA
Tributary - Water Volume (ft3)	192597.00	192597.00	197628.00	270126.00	454.08	NA
Tributary - Water Weight (tons)	6009.03	6009.03	6165.99	8427.93	14.17	NA
Diversion - Head Water height (ft)	21.83	21.83	23.40	29.94	NA	NA
Mat Foundation - Overall Width (ft)	78.00					
Mat Foundation - Clear Length (ft)	162.00					
Diversion - Water force (psf)	1362.19	1362.19	1460.16	1868.26	NA	NA
Diversion - Water Volume (ft3)	275843.88	275843.88	295682.40	378321.84	NA	NA
Diversion - Water Weight (tons)	8606.33	8606.33	9225.29	11803.64	NA	NA
Total Water Weight on the Structure (tons)	14615.36	14615.36	15391.28	20231.57	14.17	NA
Tributary - Uplift on the Deck (ft)	14.83	14.83	16.40	22.94	NA	NA
Tributary - Uplift force (psf)	925.39	925.39	1023.36	1431.46	NA	NA
Tributary - Uplift force (tons)	-6685.03	-6685.03	-7392.75	-10340.84	NA	NA
Uplift @ HW (ft)	25.83	25.83	27.40	33.94	NA	NA
Uplift @ TW (ft)	24.51	24.51	25.60	33.94	NA	NA
Diversion - Uplift force on the Mat (psf)	1570.61	1570.61	1653.60	2117.86	NA	NA
Diversion - Uplift force on the Mat (tons)	-16048.47	-16048.47	-16896.48	-21640.25	NA	NA
Total Uplif Force on the Structure (tons)	-22733.50	-22733.50	-24289.24	-31981.09	NA	NA
Weight of Structure (tons)	13233.9					

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	**Design By:**	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	**Review By:**	
		Date:	1/24/2011
Work Describtion:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	**Job #:**	34091004'
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Maple\[34091004 PH4 Maple Pile Calcs.xlsx]Load Cases		

Flotation

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Downward force on the Structure (tons)	5115.72	5115.72	4335.91	1484.35	13248.03	13233.86
Uplift Ratio	1.23	1.23	1.18	1.05	NA	NA
Uplift Ratio (No water in the Tributary)	0.96	0.96	0.92	0.78	NA	NA
Condition	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors - Flotation	1.30	1.20	1.20	1.10	1.30	1.20
Check	NG!!!	OK	NG!!!	NG!!!	OK	OK

Pile Computation

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Pile Condition	Undrained	Undrained	Undrained	Undrained	Drained	Undrained
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	18	21	21	24	11.5	21
Allowable Pile Capacity (tons) - Axial	57.18	76.23	76.23	99.43	31.43	76.23
# of Piles Required	89.47	67.11	56.88	14.93	421.58	173.60
Uniform Spacing	15.11	17.45	18.96	37.00	6.96	10.85
# of Columns (along length)	35.00	35.00	35.00	35.00	35.00	35.00
Pile Spacing (along length)	7.53	7.53	7.53	7.53	7.53	7.53
# of Rows (along width)	2.56	1.92	1.63	0.43	12.05	4.96
# of Rows (along widht) provided	13.00	13.00	13.00	13.00	13.00	13.00
Actual Pile Spacing (along width)	6.00	6.00	6.00	6.00	6.00	6.00
Total number of pile provided	455	455	455	455	455	455
Pile Load	11.24	11.24	9.53	3.26	29.12	29.09
Utilization Ratio	0.20	0.15	0.13	0.03	0.93	0.38
Check	OK	OK	OK	OK	OK	OK
ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Final # of Piles	455					
Pile Load	11.24	11.24	9.53	3.26	29.12	29.09
Utilization Ratio	0.20	0.15	0.13	0.03	0.93	0.38
Check	OK	OK	OK	OK	OK	OK

Fargo-Moorhead Food Control Structures
Preliminary Pile Foundation Analyses
HP 14X73

A_{tip} = 198.5 in^2, A_{steel} = 21.4 in^2, perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in^4

Structure	Diversion Channel Station Location	Approximate Ground (Bank) Surface Elevation (ft)	Invert Elevation (ft)	Estimated Foundation Elevation (ft)	Estimated Ground Water Elevation (ft)	Design Condition/Tip Elevations		Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips) (fixed head - single pile)			Estimated Settlement at allowable load
									0.5″	0.67″	0.875″	
Maple River Aqueduct Crossing Foundations	725+92	898	~~866.5~~ 872.06	~~862.46~~ 868.06	893	Undrained Analysis 827.5′	Total	228.7	36	42	48	<0.5″
							Uplift Resistance	135.5				
						Drained Analysis	Total	125.7	23	29	33	
							Uplift Resistance	18.5				

See following link for details
P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\060 Geotech\Deep Foundations

ATTACHMENT

F-R5.2 Maple River Aqueduct Structure Retaining Wall Panel E Computations

MAPLE AQUADUCT STRUCTURE

Client Name:	**U.S. ARMY CORPS OF ENGINEERS**	**Design By:**	**MBI**
Project Name:	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	**Review By:**	
Work Describtion:	**Maple Aquaduct Structure - Retaining Walls**	**Date:**	**2/10/2011**
	Panel E	**Job #:**	**34091004**
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Maple\[34091004 PH4 Maple Retaining Walls Panel E.xlsx]Piling		
REF.	**1**		
	2		

ID#	**Case 1**	**Case 2**	**Case 3**	**Case 4**	**Case 5**	**Case 6**
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	895.99	895.99	896.38	903.5	881.5	**NA**
Diversion - Head Water El. (ft)	893.89	893.89	895.46	903.5	**NA**	**NA**
Diversion - Tail Water El. (ft)	892.57	892.57	893.66	903.5	**NA**	**NA**
Tributary - T.O. Wall El. (ft)	903.5					
Tributary - T.O. Deck L.P. El.(ft)	881.06					
Tributary - T.O. Deck H.P. El.(ft)	883.06					
Diversion - T.O. Mat El. (ft)	872.06					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	**14.93**	**14.93**	**15.32**	**22.44**	**0.44**	**NA**
Diversion - Head Water height (ft)	**21.83**	**21.83**	**23.4**	**31.44**	**NA**	**NA**
Ice	**NA**	2ft Ice	**NA**	**NA**	2ft Ice	**NA**
Ice Load	**NA**	10 kips/ft	**NA**	**NA**	10 kips/ft	**NA**
Ice Laod El. (ft)	**NA**	**895.99**	**NA**	**NA**	**881.5**	**NA**
Uplift @ HW (ft)	**25.83**	**25.83**	**27.4**	**35.44**	**NA**	**NA**
Uplift @ TW (ft)	**24.51**	**24.51**	**25.6**	**35.44**	**NA**	**NA**
Pile Condition	**Undrained**	**Undrained**	**Undrained**	**Undrained**	**Drained**	**Undrained**
Load Category	**Usual**	**Unusual**	**Unusual**	**Extreme**	**Usual**	**Unusual**
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	**18**	**21**	**21**	**24**	**11.5**	**21**
Allowable Pile Capacity (tons) - Axial	**57.18**	**76.23**	**76.23**	**99.43**	**31.425**	**76.23**
Allowable Pile Capacity (tons) - Uplift	**33.88**	**45.17**	**45.17**	**58.91**	**4.625**	**45.17**

Pile Capacity	**Ultimate Axial Capacity (kips)**	**Allowable Lateral Capacity (kips)**		
		0.5" (Usual)	**0.67" (Unusual)**	**0.875" (Extreme)**
Undrained - Axial	228.7	36	42	48
Undrained - Uplift	135.5			
Drained - Axial	125.7	23	29	33
Drained - Uplift	18.5			

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	**34091004**	
MBI 2/11/11		**MBI**	SUBJECT	**Maple Aquaduct Structure - Retaining Walls** **Panel E**	

Monolith Structure

ITEM		UNIT	QUANITY	UNIT COST	TOTAL Cost
FURNISH HP14x73 WALL PILING		LF	3,297	0	$0
INSTALL HP14x73 WALL PILING		LF	3,297	0	$0
PILE TEST, 36.4 ft	Long	EA	6	0	$0
FOOTING CONCRETE		CY	618	0	$0
	Forming	SF	1,428		
STEM CONCRETE		CY	652	0	$0
	Forming	SF	9,127		
STEEL REINFORCEMENT		LB	244,455	0	$0
WALL RAILING		LF	140	0	$0
SHEET PILE CUT-OFF WALL		SF	1,400	0	$0
					$0

Structure Length =	140	ft
No. piles =	125	Each
Length =	26.38	ft

Note: HP14x73 pile used for design,
use HP14x73 to allow for corrosion

	LENGTH
(FRONT FACE)	10 FT

Native Soil has low permeability assume cut-off
minimal to prevent scour

BARR ENGINEERING

COMPUTED	CHECKED	SUBMITTED		
MBI 2/11/11		MBI	DATE	2/11/2011
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4
			PROJECT NUMBER	34091004
			SUBJECT	Maple Aquaduct Structure - Retaining Walls Load Cases: Case 1 100 yr. flood Panel E

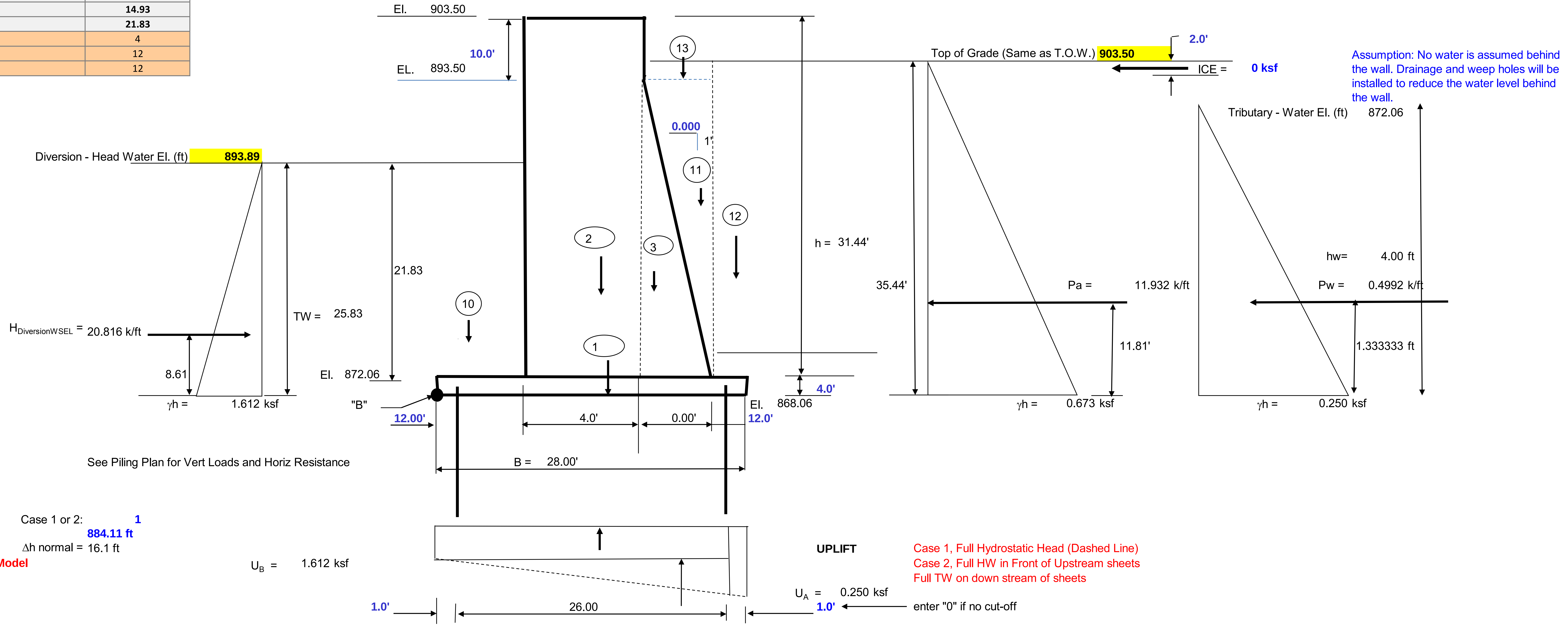
SHEET NO.

ID#	Case 1
Name	100 yr. flood
Load Category	Usual
Tributary - Water El. (ft)	895.99
Diversion - Head Water El. (ft)	893.89
Diversion - Tail Water El. (ft)	892.57
Tributary - T.O. Wall El. (ft)	903.5
Tributary - T.O. Deck L.P. El.(ft)	881.06
Tributary - T.O. Deck H.P. El.(ft)	883.06
Diversion - T.O. Mat El. (ft)	872.06
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	14.93
Diversion - Head Water height (ft)	21.83
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.50 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section
Length = 140.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1
Normal Water Level, El. 884.11 ft
Δh normal = 16.1 ft

See Geotechnical seepage Model

Vertical Loads	Section	L	W	H	γ	shape	V	arm	Mv
		ft	ft	ft	kcf		K	ft	ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	4993.0	ΣM_V =	69,901.4

CONSTANT FOR ALL LOAD CASES

	Section	L	W	H	γ	shape	V	arm	Mv
T.W on ftg Stem	10	140	12.00	21.83	0.0624	rec	2288.5	6.00	13,730.9
H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	31.44	0.0626	rec	3306.5	22.00	72,742.6
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	5595.0	ΣM_V =	86,473.5

Uplift Loads		L	W	Pressure	shape	U	arm	Mu
		ft	ft	ksf		K	ft	ft-k
	U_B	140	28.00	1.612	rec	-6318.2	14.00	-88,455
	U_A	140	28.00	-1.362	tri	2669.9	18.67	49,838
					ΣU =	-3648.3	ΣM_U =	-38,617

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls Load Cases: Case 1 100 yr. flood	Panel E

Horizontal Loads

	L	H	Pressure		ICE	arm	Mu
	ft	ft	ksf		K	ft	ft-k
ICE	140	2.00	0.00	rec	0.0	34.44	0.0
	L		Force		**H**	arm	**Mw**
	ft		k/ft		K	ft	ft-k
SOIL	140		-11.932		-1670.47	11.81	-19733.84
Water Loads							
H_{TW}	140		20.816	tri	2914.28	8.61	25091.96
H_{HW}	140		-0.499	tri	-69.89	1.33	-93.18
				ΣWater =	**2844.39**	ΣM_W =	**5264.9**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-33352	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	156375	kip-ft
Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**123,023**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**6,940**	kips
Sum of Horizontal Forces	H = Σhorizontal	**1,174**	kips

Location of Resultant

Xr = ΣM / P = 17.73 ft from Toe

e = B/2 - Xr = (3.73) ft

B/6 = 4.667 ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	652	cy	9127	sf
Total =	1,241			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing	LB /ft	Length	# of bars	Total wt		
	a) Footing		in		ft	ea	lb		
Top mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
								cy	**LB/cy**
							106,991	589	181.6374748
	b) Skin Reinf. On Monolith								
Vert Face	Vertical	9	6	3.40	30.94	280	29,455	58,909.76	
	Longitudinal:	9	6	3.40	139.5	62	29,407	58,813.20	
Top Face	Transverse:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	30.9	280	29,455		
	Vertical O.F.	9	6	3.40	30.9	280	29,455		
								cy	**LB/cy**
							124,898	652	191.5346851
						Σ =	**231,889**		
	Lap Splices (long. Bars)		9	3.40	8	462	12,566		
						Σ Bar Wt=	244,455	lb	

FORCES AT THE BOTTOM OF THE STEM

Diversion Face		H	γ	Pbase	V	arm	Mv
		ft	kcf		K	ft	ft-k
Diversion WSEL		21.83	0.0624	1.362192	14.868	7.277	108.1918
Tributary SEL =		31.44	0.019	0.59736	9.390	10.480	98.41243
Tributary WSEL =		0.00	0.0624	0	0.000	0.000	0
	Sum				9.390		98.41243
	Net Forces				-5.478		-9.77942

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls Load Cases: Case 2 100 yr. flood + ice	Panel E

ID#	Case 2
Name	100 yr. flood + ice
Load Category	Unusual
Tributary - Water El. (ft)	895.99
Diversion - Head Water El. (ft)	893.89
Diversion - Tail Water El. (ft)	892.57
Tributary - T.O. Wall El. (ft)	903.5
Tributary - T.O. Deck L.P. El.(ft)	881.06
Tributary - T.O. Deck H.P. El.(ft)	883.06
Diversion - T.O. Mat El. (ft)	872.06
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	14.93
Diversion - Head Water height (ft)	21.83
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

MN State Building Codes

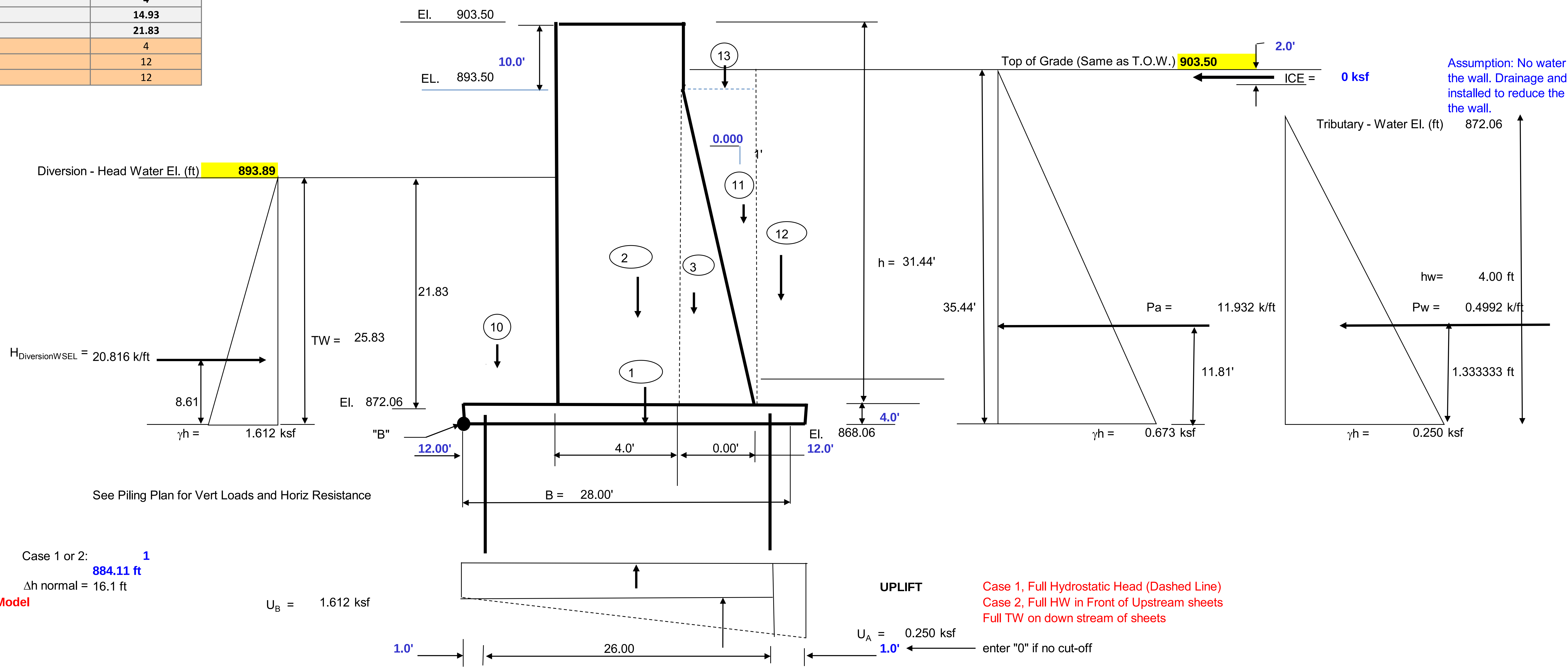
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March

Water El. = 881.50 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length =** 140.0 ft

Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg

Assumption: No water is assumed behind the wall. Drainage and weep holes will be installed to reduce the water level behind the wall.



Case 1 or 2: 1

Normal Water Level, El. 884.11 ft

Δh normal = 16.1 ft

See Geotechnical seepage Model

Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	4993.0	ΣMv =	69,901.4

← CONSTANT FOR ALL LOAD CASES

	Section	L	W	H	γ	shape	V	arm	Mv
T.W on ftg Stem	10	140	12.00	21.83	0.0624	rec	2288.5	6.00	13,730.9
H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	31.44	0.0626	rec	3306.5	22.00	72,742.6
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	5595.0	ΣMv =	86,473.5

Uplift Loads	L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
U_B	140	28.00	1.612	rec	-6318.2	14.00	-88,455
U_A	140	28.00	-1.362	tri	2669.9	18.67	49,838
				ΣU =	-3648.3	ΣMu =	-38,617

Horizontal Loads	L ft	H ft	Pressure ksf	ICE K	arm ft	Mu ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	**34091004**	
MBI		**MBI**	SUBJECT	**Maple Aquaduct Structure - Retaining Walls**	
2/11/11				**Load Cases:** **Case 2** **100 yr. flood + ice**	**Panel E**

ICE	140	2.00	0.00	**rec**	0.0	34.44	0.0
	L ft		Force k/ft		**H** K	arm ft	**Mw** ft-k
SOIL	140		-11.932		-1670.47	11.81	-19733.84
Water Loads							
H_{TW}	140		20.816	**tri**	2914.28	8.61	25091.96
H_{HW}	140		-0.499	**tri**	-69.89	1.33	-93.18
				ΣWater =	**2844.39**	ΣM_W =	**5264.9**

Overturning Moments $\Sigma M_{OT} = M_U + M_W + M_{ICE} =$ -33352 kip-ft

Resisting Moments $\Sigma M_R = M_V =$ 156375 kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT} =$	**123,023**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**6,940**	kips
Sum of Horizontal Forces	H = Σhorizontal	**1,174**	kips

Location of Resultant

Xr = ΣM / P = 17.73 ft from Toe

e = B/2 - Xr = (3.73) ft

B/6 = 4.667 ft

FORCES AT THE BOTTOM OF THE STEM

Diversion Face	H ft	γ kcf	Pbase	**V** K	arm ft	**Mv** ft-k
Diversion WSEL	21.83	**0.0624**	1.362192	14.868	7.277	108.1918
Tributary SEL =	31.44	**0.019**	0.59736	9.390	10.480	98.41243
Tributary WSEL =	0.00	**0.0624**	0	0.000	0.000	0
Sum				9.390		98.41243
Net Forces				-5.478		-9.77942

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls Load Cases: Case 3 500 yr. flood Panel E	

ID#	Case 3
Name	500 yr. flood
Load Category	Unusual
Tributary - Water El. (ft)	896.38
Diversion - Head Water El. (ft)	895.46
Diversion - Tail Water El. (ft)	893.66
Tributary - T.O. Wall El. (ft)	903.5
Tributary - T.O. Deck L.P. El.(ft)	881.06
Tributary - T.O. Deck H.P. El.(ft)	883.06
Diversion - T.O. Mat El. (ft)	872.06
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	15.32
Diversion - Head Water height (ft)	23.4
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

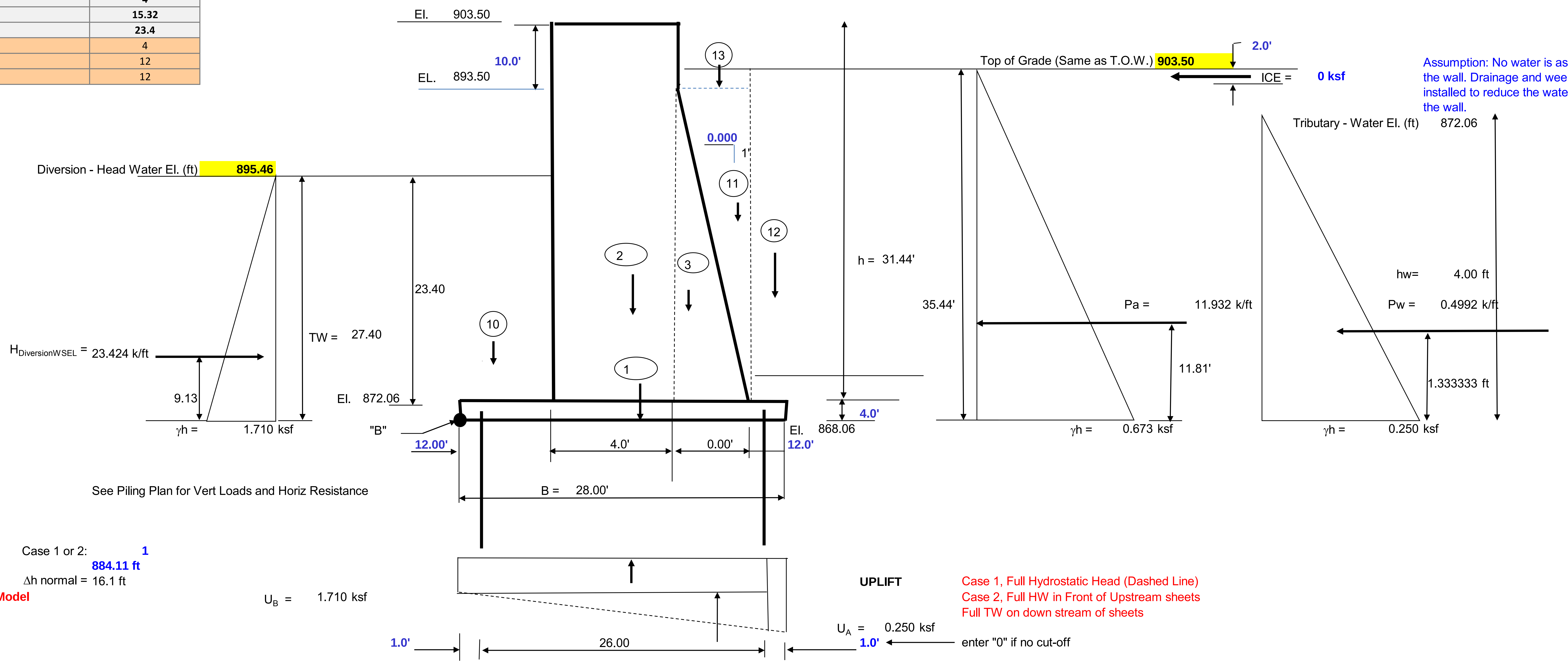
MN State Building Codes

Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March

Water El. = 881.50 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length = 140.0 ft**

Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Assumption: No water is assumed behind the wall. Drainage and weep holes will be installed to reduce the water level behind the wall.

Normal Water Level, El.

See Geotechnical seepage Model

Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**4993.0**	ΣM_V =	**69,901.4**

CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	23.40	0.0624	rec	2453.1	6.00	14,718.4
H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	31.44	0.0626	rec	3306.5	22.00	72,742.6
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**5759.6**	ΣM_V =	**87,461.0**

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	140	28.00	1.710	rec	-6702.3	14.00	-93,832
	U_A	140	28.00	-1.460	tri	2861.9	18.67	53,422
					ΣU =	**-3840.3**	ΣM_U =	**-40,409**

Horizontal Loads	L ft	H ft	Pressure ksf	ICE K	arm ft	Mu ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls Load Cases: **Case 3** **500 yr. flood**	**Panel E**

	L		Force		H	arm	Mw
ICE	140	2.00	0.00	rec	0.0	34.44	0.0
	ft		k/ft		K	ft	ft-k
SOIL	140		-11.932		-1670.47	11.81'	-19733.84
Water Loads							
H_{TW}	140		23.424	tri	3279.32	9.13	29951.12
H_{HW}	140		-0.499	tri	-69.89	1.33	-93.18
				ΣWater =	**3209.43**	ΣM_W =	**10124.1**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-30285	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	157362	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**127,077**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**6,912**	kips
Sum of Horizontal Forces	H = Σhorizontal	**1,539**	kips

Location of Resultant	Xr = ΣM / P =	18.38	ft from Toe
	e = B/2 - Xr =	(4.38)	ft
	B/6 =	4.667	ft

FORCES AT THE BOTTOM OF THE STEM

Diversion Face	H	γ	Pbase	**V**	arm	**Mv**
	ft	kcf		K	ft	ft-k
Diversion WSEL	23.40	**0.0624**	1.46016	17.084	7.800	133.2542
Tributary SEL =	31.44	**0.019**	0.59736	9.390	10.480	98.41243
Tributary WSEL =	0.00	**0.0624**	0	0.000	0.000	0
Sum				9.390		98.41243
Net Forces				-7.693		-34.8418

BARR ENGINEERING		DATE	2/11/2011 SHEET NO.
		PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4
COMPUTED MBI 2/11/11	CHECKED	SUBMITTED MBI	PROJECT NUMBER 34091004
			SUBJECT Maple Aquaduct Structure - Retaining Walls Load Cases: Case 4 T.O. Levee Panel E

ID#	**Case 4**
Name	**T.O. Levee**
Load Category	**Extreme**
Tributary - Water El. (ft)	**NA**
Diversion - Head Water El. (ft)	**903.5**
Diversion - Tail Water El. (ft)	**903.5**
Tributary - T.O. Wall El. (ft)	**903.5**
Tributary - T.O. Deck L.P. El.(ft)	**881.06**
Tributary - T.O. Deck H.P. El.(ft)	**883.06**
Diversion - T.O. Mat El. (ft)	**872.06**
Tributary - Deck Slab thickness @ L.P. (ft)	**2**
Tributary - Deck Slab thickness @ H.P. (ft)	**4**
Diversion - Mat Slab thickness (ft)	**4**
Tibutary - Water height (ft)	**22.44**
Diversion - Head Water height (ft)	**31.44**
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

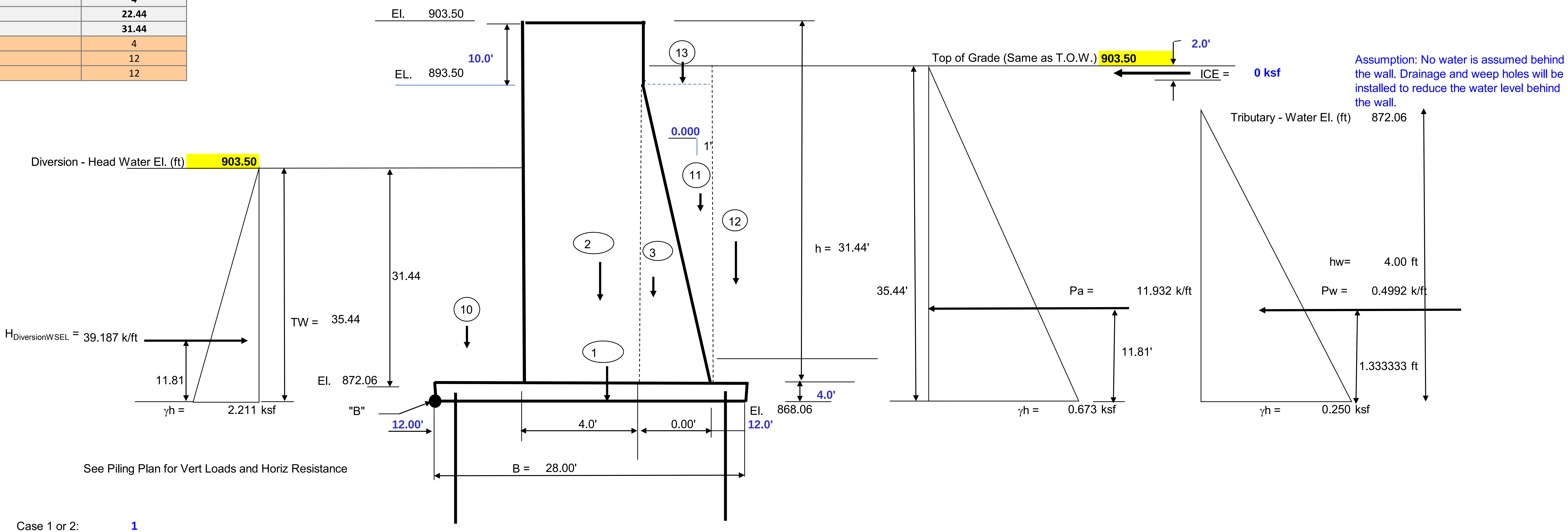
MN State Building Codes

Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March

Water El. = 881.50 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length =** 140.0 ft

Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Assumption: No water is assumed behind the wall. Drainage and weep holes will be installed to reduce the water level behind the wall.

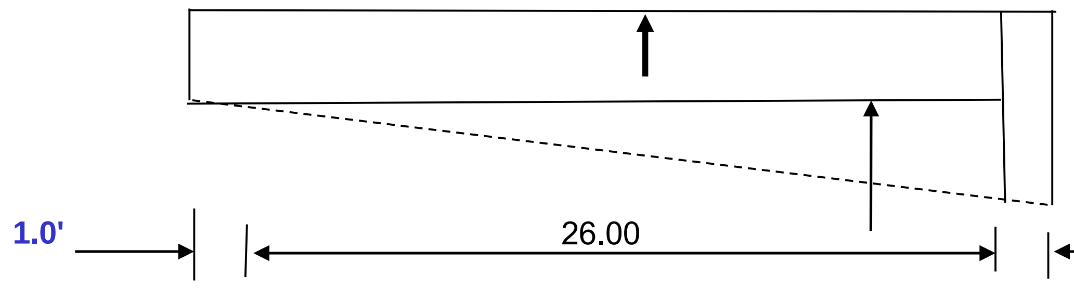
See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1

Normal Water Level, El. 884.11 ft

Δh normal = 16.1 ft

See Geotechnical seepage Model



UPLIFT

Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Vertical Loads	**Section**	L ft	W ft	H ft	γ kcf	shape	**V** K	arm ft	**Mv** ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**4993.0**	ΣM_V =	**69,901.4**
T.W on ftg Stem	10	140	12.00	31.44	0.0624	rec	3295.9	6.00	19,775.5
H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	31.44	0.0626	rec	3306.5	22.00	72,742.6
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**6602.4**	ΣM_V =	**92,518.1**

← CONSTANT FOR ALL LOAD CASES

Uplift Loads	L ft	W ft	Pressure ksf		**U** K	arm ft	**Mu** ft-k
U_B	140	28.00	2.211	rec	-8668.9	14.00	-121,365
U_A	140	28.00	-1.962	tri	3845.2	18.67	71,778
				ΣU =	**-4823.7**	ΣM_U =	**-49,587**

Horizontal Loads	L ft	H ft	Pressure ksf	**ICE** K	arm ft	**Mu** ft-k

BARR ENGINEERING			DATE	**2/11/2011**	SHEET NO.
			PROJECT NAME	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	**34091004**	
MBI		**MBI**	SUBJECT	**Maple Aquaduct Structure - Retaining Walls**	
2/11/11				**Load Cases:** **Case 4** **T.O. Levee**	**Panel E**

ICE	140	2.00	0.00	**rec**	0.0	34.44	0.0
	L		Force		**H**	arm	**Mw**
	ft		k/ft		K	ft	ft-k
SOIL	140		-11.932		-1670.47	11.81'	-19733.84
Water Loads							
H_{TW}	140		39.187	**tri**	5486.18	11.81	64810.07
H_{HW}	140		-0.499	**tri**	-69.89	1.33	-93.18
				ΣWater =	**5416.29**	ΣM_W =	**44983.1**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE} =$	-4604	kip-ft
Resisting Moments	$\Sigma M_R = M_V =$	162420	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT} =$	**157,816**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**6,772**	kips
Sum of Horizontal Forces	H = Σhorizontal	**3,746**	kips

Location of Resultant	Xr = ΣM / P =	23.31	ft from Toe
	e = B/2 - Xr =	(9.31)	ft
	B/6 =	4.667	ft

FORCES AT THE BOTTOM OF THE STEM

		H	γ	Pbase	**V**	arm	**Mv**
Diversion Face		ft	kcf		K	ft	ft-k
Diversion WSEL		31.44	**0.0624**	1.961856	30.840	10.480	323.2071
Tributary SEL =		31.44	**0.019**	0.59736	9.390	10.480	98.41243
Tributary WSEL =		0.00	**0.0624**	0	0.000	0.000	0
	Sum				9.390		98.41243
	Net Forces				-21.450		-224.795

BARR ENGINEERING			DATE	2/11/2011 SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls Load Cases: Case 5 Normal flow + ice Panel E

ID#	**Case 5**
Name	**Normal flow + ice**
Load Category	**Usual**
Tributary - Water El. (ft)	**881.5**
Diversion - Head Water El. (ft)	**NA**
Diversion - Tail Water El. (ft)	**NA**
Tributary - T.O. Wall El. (ft)	**903.5**
Tributary - T.O. Deck L.P. El.(ft)	**881.06**
Tributary - T.O. Deck H.P. El.(ft)	**883.06**
Diversion - T.O. Mat El. (ft)	**872.06**
Tributary - Deck Slab thickness @ L.P. (ft)	**2**
Tributary - Deck Slab thickness @ H.P. (ft)	**4**
Diversion - Mat Slab thickness (ft)	**4**
Tibutary - Water height (ft)	**0.44**
Diversion - Head Water height (ft)	**NA**
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

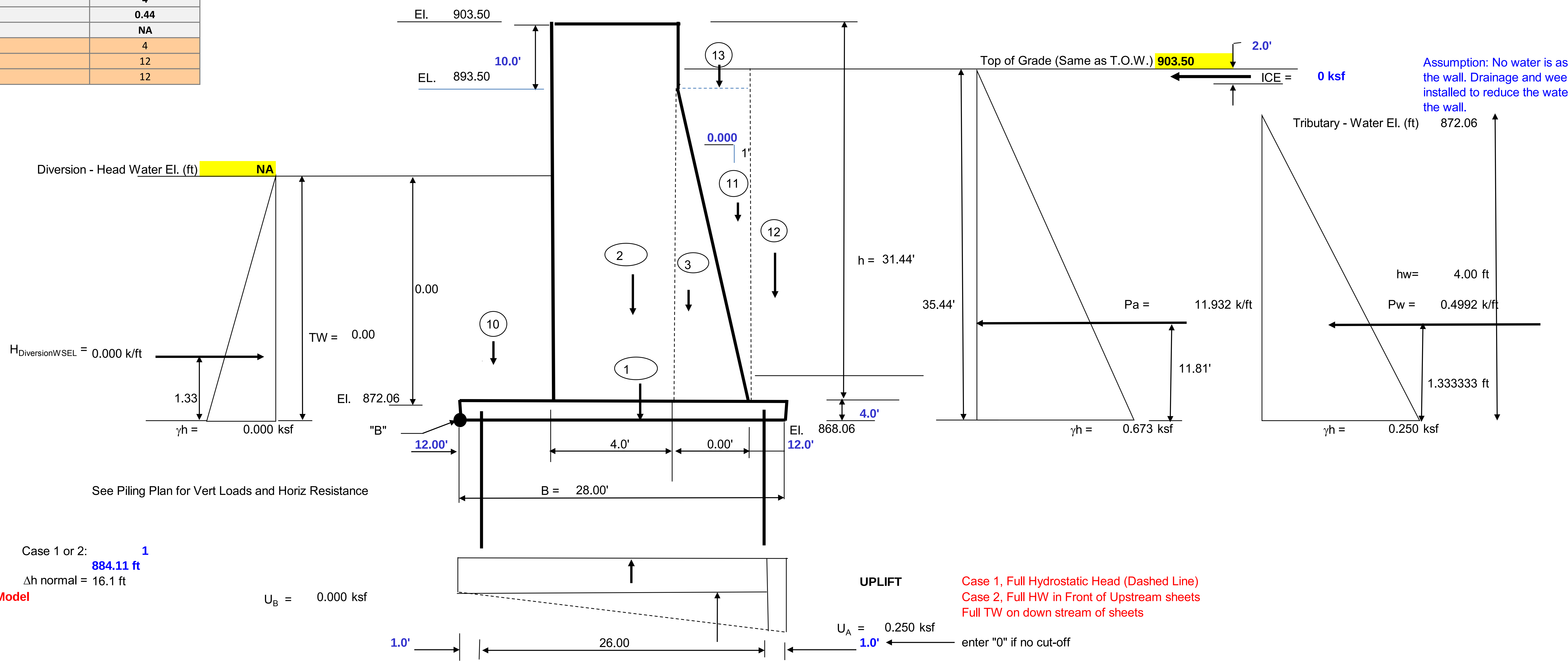
MN State Building Codes

Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March

Water El. = 881.50 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length =** 140.0 ft

Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Assumption: No water is assumed behind the wall. Drainage and weep holes will be installed to reduce the water level behind the wall.

See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1

Normal Water Level, El. 884.11 ft

Δh normal = 16.1 ft

See Geotechnical seepage Model

Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Vertical Loads	**Section**	L ft	W ft	H ft	γ kcf	shape	**V** K	arm ft	**Mv** ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**4993.0**	ΣM$_V$ =	**69,901.4**

CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	0.00	0.0624	rec	0.0	6.00	0.0
H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	31.44	0.0626	rec	3306.5	22.00	72,742.6
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**3306.5**	ΣM$_V$ =	**72,742.6**

Uplift Loads		L ft	W ft	Pressure ksf		**U** K	arm ft	**Mu** ft-k
	U$_B$	140	28.00	0.000	rec	0.0	14.00	0
	U$_A$	140	28.00	0.250	tri	-489.2	18.67	-9,132
					ΣU =	**-489.2**	ΣM$_U$ =	**-9,132**

Horizontal Loads	L ft	H ft	Pressure ksf	**ICE** K	arm ft	**Mu** ft-k

BARR ENGINEERING		DATE	2/11/2011	SHEET NO.
		PROJECT NAME	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	**34091004**
MBI **2/11/11**		**MBI**	SUBJECT	**Maple Aquaduct Structure - Retaining Walls** **Load Cases:** **Case 5** **Normal flow + ice** **Panel E**

ICE	140	2.00	0.00	**rec**	0.0	34.44	0.0
	L ft		Force k/ft		**H** K	arm ft	**Mw** ft-k
SOIL	140		-11.932		-1670.47	11.81'	-19733.84
Water Loads							
H_{TW}	140		0.000	**tri**	0.00	1.33	0.00
H_{HW}	140		-0.499	**tri**	-69.89	0.00	0.00
				ΣWater =	**-69.89**	ΣM_W =	**-19733.8**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-28866	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	142644	kip-ft
Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**113,778**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,810**	kips
Sum of Horizontal Forces	H = Σhorizontal	**-1,740**	kips

Location of Resultant

Xr = ΣM / P = 14.57 ft from Toe

e = B/2 - Xr = (0.57) ft

B/6 = 4.667 ft

FORCES AT THE BOTTOM OF THE STEM

Diversion Face		H ft	γ kcf	Pbase	**V** K	arm ft	**Mv** ft-k
Diversion WSEL		0.00	**0.0624**	0	0.000	0.000	0
Tributary SEL =		31.44	**0.019**	0.59736	9.390	10.480	98.41243
Tributary WSEL =		0.00	**0.0624**	0	0.000	0.000	0
	Sum				9.390		98.41243
	Net Forces				9.390		98.41243

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls Load Cases: Case 6 Construction Panel E	

ID#	Case 6
Name	Construction
Load Category	Unusual
Tributary - Water El. (ft)	NA
Diversion - Head Water El. (ft)	NA
Diversion - Tail Water El. (ft)	NA
Tributary - T.O. Wall El. (ft)	903.5
Tributary - T.O. Deck L.P. El.(ft)	881.06
Tributary - T.O. Deck H.P. El.(ft)	883.06
Diversion - T.O. Mat El. (ft)	872.06
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	NA
Diversion - Head Water height (ft)	NA
Wall Thickness (ft)	4
Toe (Ft)	12
Heel (ft)	12

File:

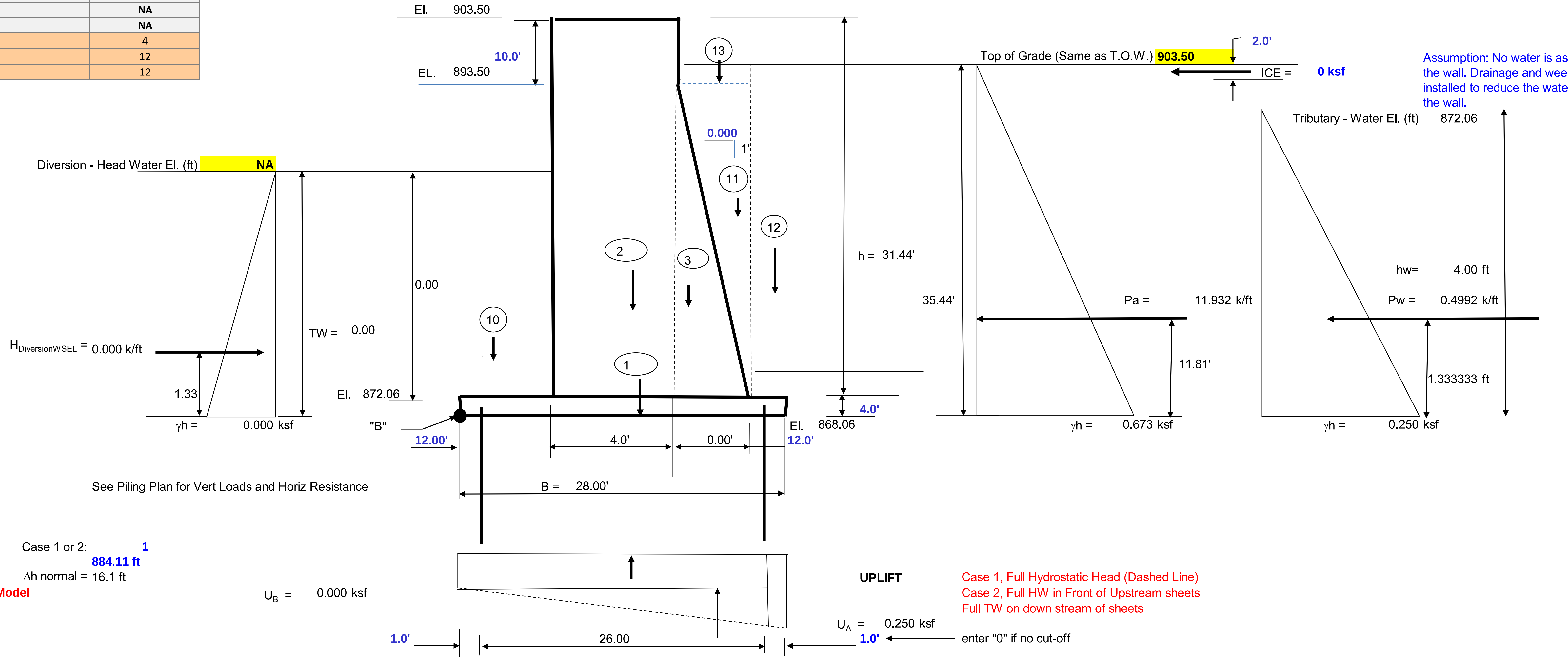
MN State Building Codes

Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March

Water El. = 881.50 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section Length = 140.0 ft

Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Assumption: No water is assumed behind the wall. Drainage and weep holes will be installed to reduce the water level behind the wall.

See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1

Normal Water Level, El. 884.11 ft

Δh normal = 16.1 ft

See Geotechnical seepage Model

UPLIFT

Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

enter "0" if no cut-off

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**4993.0**	ΣMV =	**69,901.4**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	0.00	0.0624	rec	0.0	6.00	0.0
H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	31.44	0.0626	rec	3306.5	22.00	72,742.6
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**3306.5**	ΣMV =	**72,742.6**

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	UB	140	28.00	0.000	rec	0.0	14.00	0
	UA	140	28.00	0.250	tri	-489.2	18.67	-9,132
					ΣU =	**-489.2**	ΣMU =	**-9,132**

Horizontal Loads	L ft	H ft	Pressure ksf	ICE K	arm ft	Mu ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	**34091004**	
MBI **2/11/11**		**MBI**	SUBJECT	**Maple Aquaduct Structure - Retaining Walls** **Load Cases: Case 6 Construction**	**Panel E**

ICE	140	2.00	0.00	rec	0.0	34.44	0.0
	L ft		Force k/ft		**H** K	arm ft	**Mw** ft-k
SOIL	140		-11.932		-1670.47	11.81'	-19733.84
Water Loads							
H_{TW}	140		0.000	tri	0.00	1.33	0.00
H_{HW}	140		-0.499	tri	-69.89	1.33	-93.18
				ΣWater =	**-69.89**	ΣM_W =	**-19827.0**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-28959	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	142644	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**113,685**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,810**	kips
Sum of Horizontal Forces	H = Σhorizontal	**-1,740**	kips

Location of Resultant

Xr = ΣM / P = 14.56 ft from Toe

e = B/2 - Xr = (0.56) ft

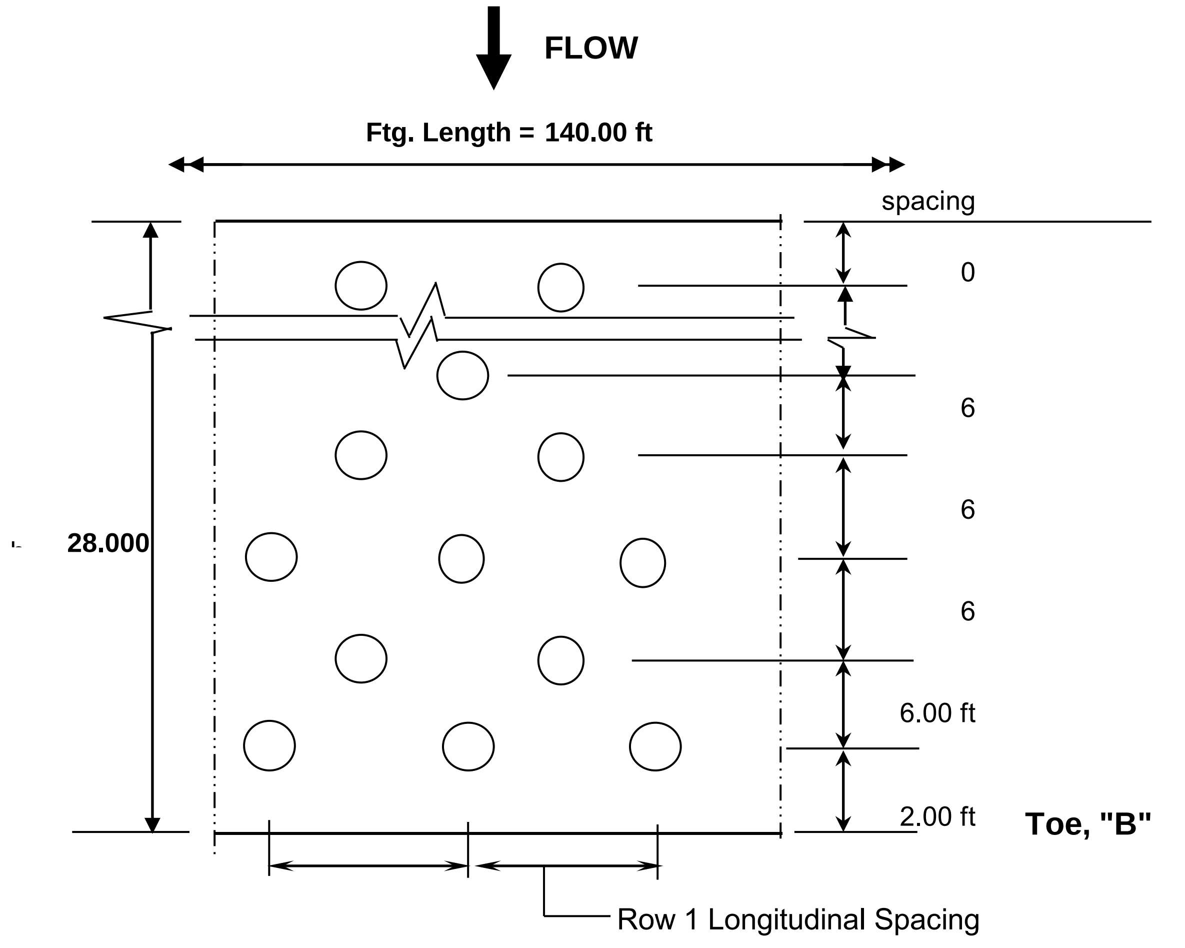
B/6 = 4.667 ft

FORCES AT THE BOTTOM OF THE STEM

Diversion Face	H ft	γ kcf	Pbase	**V** K	arm ft	**Mv** ft-k
Diversion WSEL	0.00	**0.0624**	0	0.000	0.000	0
Tributary SEL =	31.44	**0.019**	0.59736	9.390	10.480	98.41243
Tributary WSEL =	0.00	**0.0624**	0	0.000	0.000	0
Sum				9.390		98.41243
Net Forces				9.390		98.41243

BARR ENGINEERING			DATE	**2/11/2011**	SHEET NO.
			PROJECT NAME	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	
COMPUTED **MBI** **2/11/11**	CHECKED	SUBMITTED **MBI**	PROJECT NUMBER	**34091004**	
			SUBJECT	**Maple Aquaduct Structure - Retaining Walls** **Panel E**	

PILE FOUNDATION DESIGN



L = 140.00 ft

PILE PATTERN GEOMETRY

Transverse Spacing		**Distance to Toe, d_{toe}**	**Longitudinal Spacing**	**Batter**	Piles per Row (**N**)	Edge Dist (ft)		**Trial N**
Row 1 to Toe	**2.00 ft**	2.0 ft	**2.50 ft**	**0** "/12"	**23**	42.50	1	57
Row 1 to Row 2	**6.00 ft**	8.0 ft	**5.00 ft**	**0** "/12"	**24**	12.50	2	29
Row 2 to Row 3	**6.00 ft**	14.0 ft	**5.00 ft**	**0** "/12"	**24**	12.50	3	29
Row 3 to Row 4	**6.00 ft**	20.0 ft	**5.00 ft**	**0** "/12"	**26**	7.50	4	29
Row 4 to Row 5	**6.00 ft**	26.0 ft	**0.00 ft**	**0** "/12"	**28**	70.00	5	#DIV/0!
Row 5 to Row 6	**0.00 ft**	0.0 ft	**0.00 ft**	**0** "/12"	**0**	70.00	0	0
Row 6 to Row 7	**0.00 ft**	0.0 ft	**0.00 ft**	**0** "/12"	**0**	70.00	0	0
Row 7 to Row 8	**0.00 ft**	0.0 ft	**0.00 ft**	**0** "/12"	**0**	70.00	0	0
Row 8 to Row 9	**0.00 ft**	0.0 ft	**0.00 ft**	**0** "/12"	**0**	70.00	0	0
Row 9 to Row 10	**0.00 ft**	0.0 ft	**0.00 ft**	**0** "/12"	**0**	70.00	0	0
Row 10 to Row 11	**0.00 ft**	0.0 ft	**0.00 ft**	**0** "/12"	**0**	70.00	0	0
Row 11 to Row 12	**0.00 ft**	0.0 ft	**0.00 ft**	**0** "/12"	**0**	70.00	0	0
Row 12 to Row 13	**0.00 ft**	0.0 ft	**0.00 ft**	**0** "/12"	**0**	70.00	0	0
Row 13 to Row 14	**0.00 ft**	0.0 ft	**0.00 ft**	**0** "/12"	**0**	70.00	0	0
Row 14 to Row 15	**0.00 ft**	0.0 ft	**0.00 ft**	**0** "/12"	**0**	70.00	0	0
Last Row to Heel	2.00 ft							
	28.00 ft		**Note: Enter 0 for Longitudinal Spacing for Rows Not Used)**		ΣN = **125**			#DIV/0!

Pile Properties:	Pile Type: **HP**	(C.I.P or HP)	Pile Length = **26.4 ft**	Ftg EL.	868.06
				Pile Tip El.	842.68
	HP Nominal Depth, h = **14.0 in**			Pile Cap Embed =	**1.00 ft**
	Wt. per ft, plf **73**		**Total pile Length =**	**3,298** LF	

Pile Group Properties

N.A. of Pile Group to Toe

$X_{NA} = (\Sigma N * d_{toe}) / \Sigma N =$ **14.58 ft**

	Dist. From N.A. to Pile Row	d	N	I = N * d^2
1	Dist. To Row 1	12.58 ft	23	3637.6
2	Dist. To Row 2	6.58 ft	24	1037.9
3	Dist. Row 3	0.58 ft	24	8.0
4	Dist. Row 4	-5.42 ft	26	764.9
5	Dist. Row 5	-11.42 ft	28	3654.2
0	Row 6 (not used)	0.00 ft	0	0.0
0	Row 7 (not used)	0.00 ft	0	0.0
0	Row 8 (not used)	0.00 ft	0	0.0
0	Row 9 (not used)	0.00 ft	0	0.0
0	Row 10 (not used)	0.00 ft	0	0.0
0	Row 11 (not used)	0.00 ft	0	0.0
0	Row 12 (not used)	0.00 ft	0	0.0
0	Row 13 (not used)	0.00 ft	0	0.0
0	Row 14 (not used)	0.00 ft	0	0.0
0	Row 15 (not used)	0.00 ft	0	0.0
			125	Σ I = **9102.5**

ALLOWABLE LOADS (from Geotechnical)

	Service	**Allowable Pile Loads**						
	ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	
	Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction	
	Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual	
Allwable Lateral Capacity (tons)		18.0 tons	21.0 tons	21.0 tons	24.0 tons	11.5 tons	21.0 tons	
Allowable Pile Capacity (tons) - Axial		57.2 tons	76.2 tons	76.2 tons	99.4 tons	31.4 tons	76.2 tons	
Safety Factors		2.00	1.50	1.50	1.15	2.00	1.50	**w/o Group effects**

Summary Pile Reactions

Load Combinations	Allowable Pile Capacity (tons) - Axial	Pile Loads (tons/pile) 1	2	3	4	5	6	7	8	9	10	11	12	Max. Vertical Load (Tons)	Horiz Pile Group Capacity (k)	Check
Case 1	**57.2 tons**	12.6	19.9	27.1	34.3	41.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	41.5	4,500	**OK**
Case 2	**76.2 tons**	12.6	19.9	27.1	34.3	41.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	41.5	5,250	**OK**
Case 3	**76.2 tons**	9.5	18.1	26.8	35.5	44.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	44.2	5,250	**OK**
Case 4	**99.4 tons**	-13.7	5.7	25.2	44.7	64.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	64.2	6,000	**OK**
Case 5	**31.4 tons**	31.3	31.3	31.2	31.2	31.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	31.3	2,875	**OK**
Case 6	**76.2 tons**	31.3	31.3	31.2	31.2	31.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	31.3	2,875	**OK**
													Max Service : P =	**64.2**		

Using solid mechanics equations adapted for discrete elements, the forces in the pile rows for different load combinations are determined.
The force in each pile row is found using:

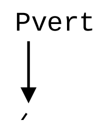
$$\text{Pile Load} = P / N + M_{NA} / I$$

First, the moment about the toe must be translated to get the moment about the neutral axis of the pile group.

$$e_{toe} = M_{toe} / P$$

Then the eccentricity about the neutral axis of the pile group is

$$e_{NA} = X_{NA} - e_{toe}$$



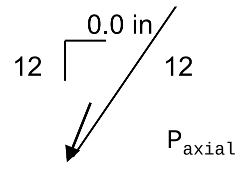
BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	**FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4**	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI		MBI	SUBJECT	**Maple Aquaduct Structure - Retaining Walls**	
2/11/11				**Panel E**	

The moment about the neutral axis of the pile group becomes

$$M_{NA} = P * e_{NA}$$

For battered pile, the Vertical pile load needs to be transformed to the axial load along the pile axis

Paxial = 1.000 `Pvert`



FORCE RESULTANT (see Stability Analysis)

CASE	Event		Vertical Load P (kips)	Horizontal	ΣM_{toe} (kip-	$e_{toe} = M_{toe} / P$	$e_{NA} = X_{NA} - e_{toe}$	$M_{NA} = P * e_{NA}$
Case 1	**100 yr. flood**	**Usual**	6,940	-1,174	123,023	17.73	-3.15	-21871
Case 2	**100 yr. flood + ice**	**Unusual**	6,940	-1,174	123,023	17.73	-3.15	-21871
Case 3	**500 yr. flood**	**Unusual**	6,912	-1,539	127,077	18.38	-3.81	-26326
Case 4	**T.O. Levee**	**Extreme**	6,772	-3,746	157,816	23.31	-8.73	-59112
Case 5	**Normal flow + ice**	**Usual**	7,810	1,740	113,778	14.57	0.01	64
Case 6	**Construction**	**Unusual**	7,810	1,740	113,685	14.56	0.02	157

SERVICE

Case **Case 1**
Flood Event **100 yr. flood**
Usual

Vertical Load, P = 6940 kips
Horizontal Load, H = -1174 kips
M_{NA} = -21871 kip-ft 125

	Vertical Pile Loading	P / N +	M_{NA}* d / Σ I	= Pile Loads		**Axial Pile Load**
1	Row 1	55.5	-30.2	25.3 kips/pile	12.6 tons/pile	12.6 tons/pile
2	Row 2	55.5	-15.8	39.7 kips/pile	19.9 tons/pile	19.9 tons/pile
3	Row 3	55.5	-1.4	54.1 kips/pile	27.1 tons/pile	27.1 tons/pile
4	Row 4	55.5	13.0	68.5 kips/pile	34.3 tons/pile	34.3 tons/pile
5	Row 5	55.5	27.4	83.0 kips/pile	41.5 tons/pile	41.5 tons/pile
6	Row 6	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	**41.5 tons/pile**	**max: 41.5 tons/pile**

Assumed lateral Capacity: 36.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	23	0.0	828	1.000	828 kips	
2	Row 2	0	24	0.0	864	1.000	864 kips	
3	Row 3	0	24	0.0	864	1.000	864 kips	
4	Row 4	0	26	0.0	936	1.000	936 kips	
5	Row 5	0	28	0.0	1008	1.000	1008 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			125		4500		**4500 kips**	**OK**

Case **Case 2**
Flood Event **100 yr. flood + ice**
Unusual

Vertical Load, P = 6940 kips
Horizontal Load, H = -1174 kips
M_{NA} = -21871 kip-ft 125

	Vertical Pile Loading	P / N +	M_{NA}* d / Σ I	= Pile Loads		**Axial Pile Load**
1	Row 1	55.5	-30.2	25.3 kips/pile	12.6 tons/pile	12.6 tons/pile
2	Row 2	55.5	-15.8	39.7 kips/pile	19.9 tons/pile	19.9 tons/pile
3	Row 3	55.5	-1.4	54.1 kips/pile	27.1 tons/pile	27.1 tons/pile
4	Row 4	55.5	13.0	68.5 kips/pile	34.3 tons/pile	34.3 tons/pile

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls	
2/11/11				Panel E	

5	Row 5	55.5	27.4	83.0 kips/pile	41.5 tons/pile		41.5 tons/pile
6	Row 6	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
				max:	**41.5 tons/pile**	**max:**	**41.5 tons/pile**

Assumed lateral Capacity: 42.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	23	0.0	966	1.000	966 kips	
2	Row 2	0	24	0.0	1008	1.000	1008 kips	
3	Row 3	0	24	0.0	1008	1.000	1008 kips	
4	Row 4	0	26	0.0	1092	1.000	1092 kips	
5	Row 5	0	28	0.0	1176	1.000	1176 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			125		5250		**5250 kips**	**OK**

Case **Case 3**
Flood Event **500 yr. flood**
Unusual

Vertical Load, P = 6912 kips
Horizontal Load, H = -1539 kips
M_{NA} = -26326 kip-ft

	Vertical Pile Loading	P / N	+ M_{NA}* d / Σ I	= Pile Loads			**Axial Pile Load**
1	Row 1	55.3	-36.4	18.9 kips/pile	9.5 tons/pile		9.5 tons/pile
2	Row 2	55.3	-19.0	36.3 kips/pile	18.1 tons/pile		18.1 tons/pile
3	Row 3	55.3	-1.7	53.6 kips/pile	26.8 tons/pile		26.8 tons/pile
4	Row 4	55.3	15.7	71.0 kips/pile	35.5 tons/pile		35.5 tons/pile
5	Row 5	55.3	33.0	88.3 kips/pile	44.2 tons/pile		44.2 tons/pile
6	Row 6	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
				max:	**44.2 tons/pile**	**max:**	**44.2 tons/pile**

Assumed lateral Capacity: 42.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	23	0.0	966	1.000	966 kips	
2	Row 2	0	24	0.0	1008	1.000	1008 kips	
3	Row 3	0	24	0.0	1008	1.000	1008 kips	
4	Row 4	0	26	0.0	1092	1.000	1092 kips	
5	Row 5	0	28	0.0	1176	1.000	1176 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			125		5250		**5250 kips**	**OK**

Case **Case 4**
Flood Event **T.O. Levee**
Extreme

Vertical Load, P = 6772 kips
Horizontal Load, H = -3746 kips
M_{NA} = -59112 kip-ft

	Vertical Pile Loading	P / N	+	M_{NA}* d / Σ I	= Pile Loads		**Axial Pile Load**
1	Row 1	54.2		-81.7	-27.5 kips/pile	-13.7 tons/pile	-13.7 tons/pile
2	Row 2	54.2		-42.7	11.5 kips/pile	5.7 tons/pile	5.7 tons/pile
3	Row 3	54.2		-3.7	50.4 kips/pile	25.2 tons/pile	25.2 tons/pile
4	Row 4	54.2		35.2	89.4 kips/pile	44.7 tons/pile	44.7 tons/pile
5	Row 5	54.2		74.2	128.4 kips/pile	64.2 tons/pile	64.2 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
					max:	**64.2 tons/pile**	**max: 64.2 tons/pile**

Assumed lateral Capacity: 48.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	23	0.0	1104	1.000	1104 kips	
2	Row 2	0	24	0.0	1152	1.000	1152 kips	
3	Row 3	0	24	0.0	1152	1.000	1152 kips	
4	Row 4	0	26	0.0	1248	1.000	1248 kips	
5	Row 5	0	28	0.0	1344	1.000	1344 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			125		6000		**6000 kips**	**OK**

Case **Case 5**
Flood Event **Normal flow + ice**
Usual

Vertical Load, P = 7810 kips
Horizontal Load, H = 1740 kips
M_{NA} = 64 kip-ft

	Vertical Pile Loading	P / N	+	M_{NA}* d / Σ I	= Pile Loads		**Axial Pile Load**
1	Row 1	62.5		0.1	62.6 kips/pile	31.3 tons/pile	31.3 tons/pile
2	Row 2	62.5		0.0	62.5 kips/pile	31.3 tons/pile	31.3 tons/pile
3	Row 3	62.5		0.0	62.5 kips/pile	31.2 tons/pile	31.2 tons/pile
4	Row 4	62.5		0.0	62.4 kips/pile	31.2 tons/pile	31.2 tons/pile
5	Row 5	62.5		-0.1	62.4 kips/pile	31.2 tons/pile	31.2 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
					max:	**31.3 tons/pile**	**max: 31.3 tons/pile**

Assumed lateral Capacity: 23.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance
1	Row 1	0	23	0.0	529	1.000	529 kips
2	Row 2	0	24	0.0	552	1.000	552 kips

3	Row 3	0	24	0.0	552	1.000	552 kips	
4	Row 4	0	26	0.0	598	1.000	598 kips	
5	Row 5	0	28	0.0	644	1.000	644 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			125		2875		**2875 kips**	**OK**

Case **Case 6**
Flood Event **Construction**
Unusual

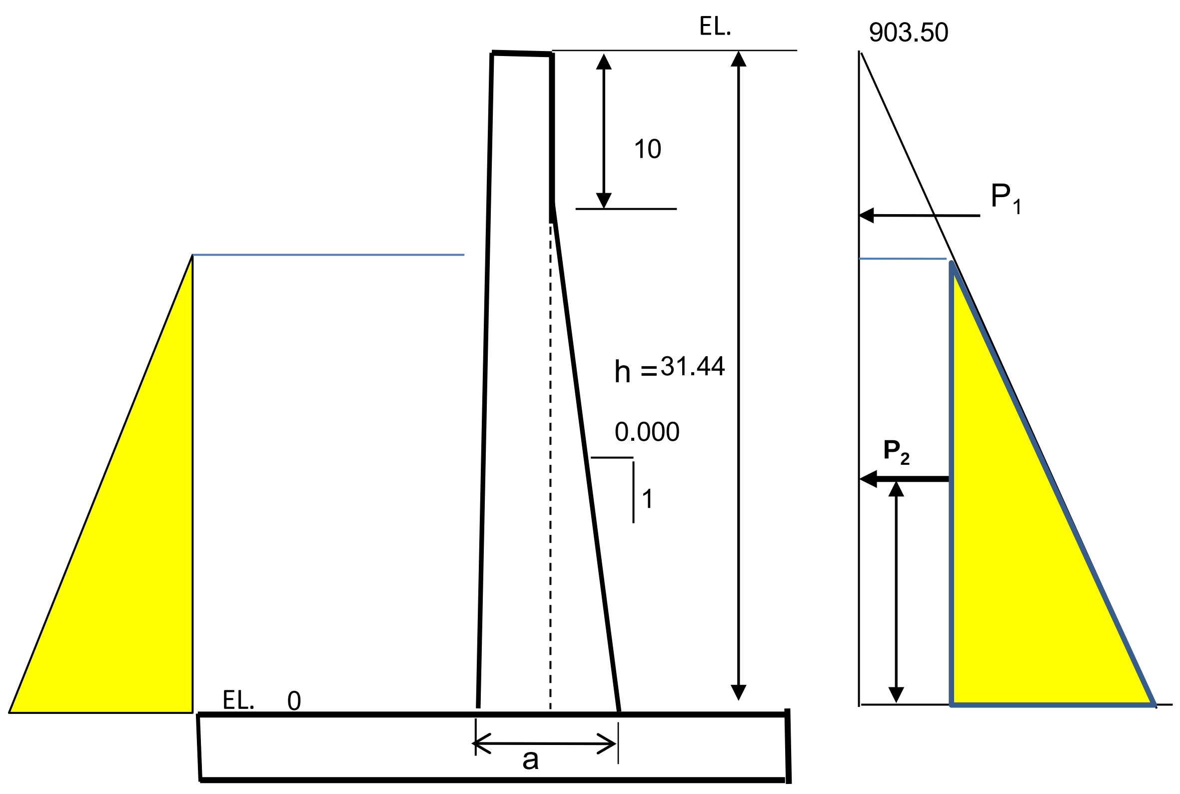
Vertical Load, P = 7810 kips
Horizontal Load, H = 1740 kips
M_{NA} = 157 kip-ft

	Vertical Pile Loading	P / N	+	M_{NA}* d / Σ I	= Pile Loads				**Axial Pile Load**
1	Row 1	62.5		0.2	62.7 kips/pile		31.3 tons/pile		31.3 tons/pile
2	Row 2	62.5		0.1	62.6 kips/pile		31.3 tons/pile		31.3 tons/pile
3	Row 3	62.5		0.0	62.5 kips/pile		31.2 tons/pile		31.2 tons/pile
4	Row 4	62.5		-0.1	62.4 kips/pile		31.2 tons/pile		31.2 tons/pile
5	Row 5	62.5		-0.2	62.3 kips/pile		31.1 tons/pile		31.1 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
						max:	**31.3 tons/pile**	**max:**	**31.3 tons/pile**

Assumed lateral Capacity: 42.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	23	0.0	529	1.000	529 kips	
2	Row 2	0	24	0.0	552	1.000	552 kips	
3	Row 3	0	24	0.0	552	1.000	552 kips	
4	Row 4	0	26	0.0	598	1.000	598 kips	
5	Row 5	0	28	0.0	644	1.000	644 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			125		2875		**2875 kips**	**OK**

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls Panel E 0	



CASE	Event		HW	TW	Dh	TW -ftg
Case 1	100 yr. flood	Usual	893.89	892.57	1.32	892.57
Case 2	100 yr. flood + ice	Unusual	893.89	892.57	1.32	892.57
Case 3	500 yr. flood	Unusual	896.380	893.66	2.72	893.66
Case 4	T.O. Levee	Extreme	903.500	903.50	0.00	903.50
Case 5	Normal flow + ice	Usual	0.000	0.000	0.00	0.00
Case 6	Construction	Unusual	0.000	0.000	0.00	0.00

LOAD FACTORS

Hf = 1.30 hydraulic Factor
LF = 1.70
Unsual & Extreme = 0.75

TOP THICK =	4.0 ft	48.0 in
Batter at Base =	0.00 ft	0.0 in
a =	4.00 ft	48.0 in

Load Factors - Hydraulic Structures
live load, LL = 1.7
dead load, DL = 1.4
flood level , FL = 1
Fluid, F = 1.7
hydraulic, Hf = 1.3
direct tension hydraulic, Hf = 1.65
ICE = 1.7

WALL DESIGN:

Horizontal Load Components and Moments about Bottom of Stem (Service)

CASE	Event	Condition	Load Factor	H (kips/ft)	Moment (kip-ft/ ft)	Vu (kips/ft)	Mu (kip-ft/ ft)
Case 1	100 yr. flood	Usual	1	-5.48	-9.779	-12.11	-21.61
Case 2	100 yr. flood + ice	Unusual	0.75	-5.48	-9.779	-9.08	-16.21
Case 3	500 yr. flood	Unusual	0.75	-7.69	-34.842	-12.75	-57.75
Case 4	T.O. Levee	Extreme	0.75	-21.45	-224.795	35.55	372.60
Case 5	Normal flow + ice	Usual	1	9.39	98.412	20.75	217.49
Case 6	Construction	Unusual	0.75	9.39	98.412	15.56	163.12

	STEM DESIGN VALUES	
MU, k-ft/ft	372.60	k-ft/ft
VU, k/ft	35.55	k/ft

ACI 318-05 w/ Modifications per EM 1110-2-2104

ref. EM 110-2-2104

9.3 - Design Strength

ϕ = **0.9** 9.3.2.1 - Tension Controlled sections
0.75 9.3.2.3 - Shear and torsion

FLEXURAL STEEL FOR RECTANGULAR CONCRETE SECTIONS

fy= 60 ksi
Fc'= 4 ksi
B1= 0.85

Muh = 373 k-ft /ft **Includes: hf = 1.3**
Vuh= 35.55 k / ft

bw= 12 in.
h= 48 in
cover= 4 in (include correct stirrup bar dia.)
d= 43.50 in

pb= 0.0285 pb=0.85*B1*Fc'/ fy*(87 / (87+fy))
.75*pb= 0.0214

m=fy / 0.85*Fc'= 17.647

TRIAL

$Ru=Mn/bd^2$ = 218.786 **ACI 10.5.1** **ACI 10.5.3**
REQ'D p= 0.0038 **O.K.** p(min)= 3*SQRT(Fc')/fy 200* / fy 4/3*p
p= FALSE N.G. 0.00316 0.00333 0.0050

EM 110-2-2104 2-8 c. (not less than Temp & Shrinkage, half in each face)

As (REQ'D)= 0.81 in^2 p(min)= 0.0028 /2 ► As =0.5*$p_{T\&S}$bh = 0.8064 in^2
As = #9 @ 12 = 1.00 in^2

SELECT STEEL

bar #= 9
spacing, s= 6 in
# OF BAR= 1 (ENTER 1 IF PER FT, b=12") a
As= 1.999 in^2
d= 43.4375 in
p = As/bd = 0.0038 **O.K. < 0.375pb** **EM 110-2-2104**
p = 0.135 pb **MAXIMUM TENSILE REINFORCEMENT**

a) For singly reinforced flexural members
1) p = **0.25** pb Recommended limit
2) p = **0.375** pb Max. permitted upper limit not requiring special study
3) p = **0.5** pb Max. permitted upper limit when excessive deflections are not predicted In ACI 318
4) p = >**0.5** pb but ≤ 0.375 pb permitted only if detailed serviceability analysis incl. deflect. Calc.
b) Use of compression reinf. shall be per ACI 318

T= As*fy = 119.9 k
C= B1*Fc' *b*a = 1266.8 **a**
a= T/C = 0.095 in
Mn= T(d - a/2)/12 = 433.6 ft-k
ϕ Mn= 390.2 ft-k **> Mu O.K.**

CHECK SHEAR REINFORCEMENT (ACI 11.3 & EM 110-2-2104 3-3a)

Vuh = 35.6 k **NO SHEAR REINF. REQUIRED**
Vn = Vuh / ϕ = 47.4 k
Vc = 2*sqrt(Fc') bw * d = 65.9 k **11.3.1.1**
Vs =Vuh / f - 1.3Vc = No Shear Reinf. Req. k NG Vs(max) ≤ 8*sqrt(fc')bd = 263.7 k

Trial Stirrup Sizes:

# of stirrup legs = 2 (single stirrup = 2, Dbl stirrup = 4.........)
Stirrup bar size = 4
Av= 0.393 in^2
s= 0.000 in s = Av * fy * d / (Vu / f - Vc)

11.5.5 - Spacing limits for shear reinforcement

s = d/2 = 21.719 in OR 24 in
s(max)= 10.859 in
4*sqrt(Fc')*bw*d= 131.9 k < Vs Reduce Spacing

USE s= **0.00** in

Vs = (Av * Fy * d) / s = #DIV/0! k

11.5.6 - MINIMUM SHEAR REINFORCEMENT

A minimum area of shear reinforcement, Av,min shall be provided in all reinforced concrete flexural members where **Vu** exceeds **0.5 f Vc**

NOT REQUIRED IF:
a) SLAB OR FOOTING, vc>vn **O.K.**
b) CONCRETE JOIST ACI 8.11
c) BEAMS W/ h <= 10"
h <= 2.5*Bf
h <= 0.5*tw
d) WALLS (SEE ACI 11.10.1); vc>vn **O.K.**

11.5.6.3

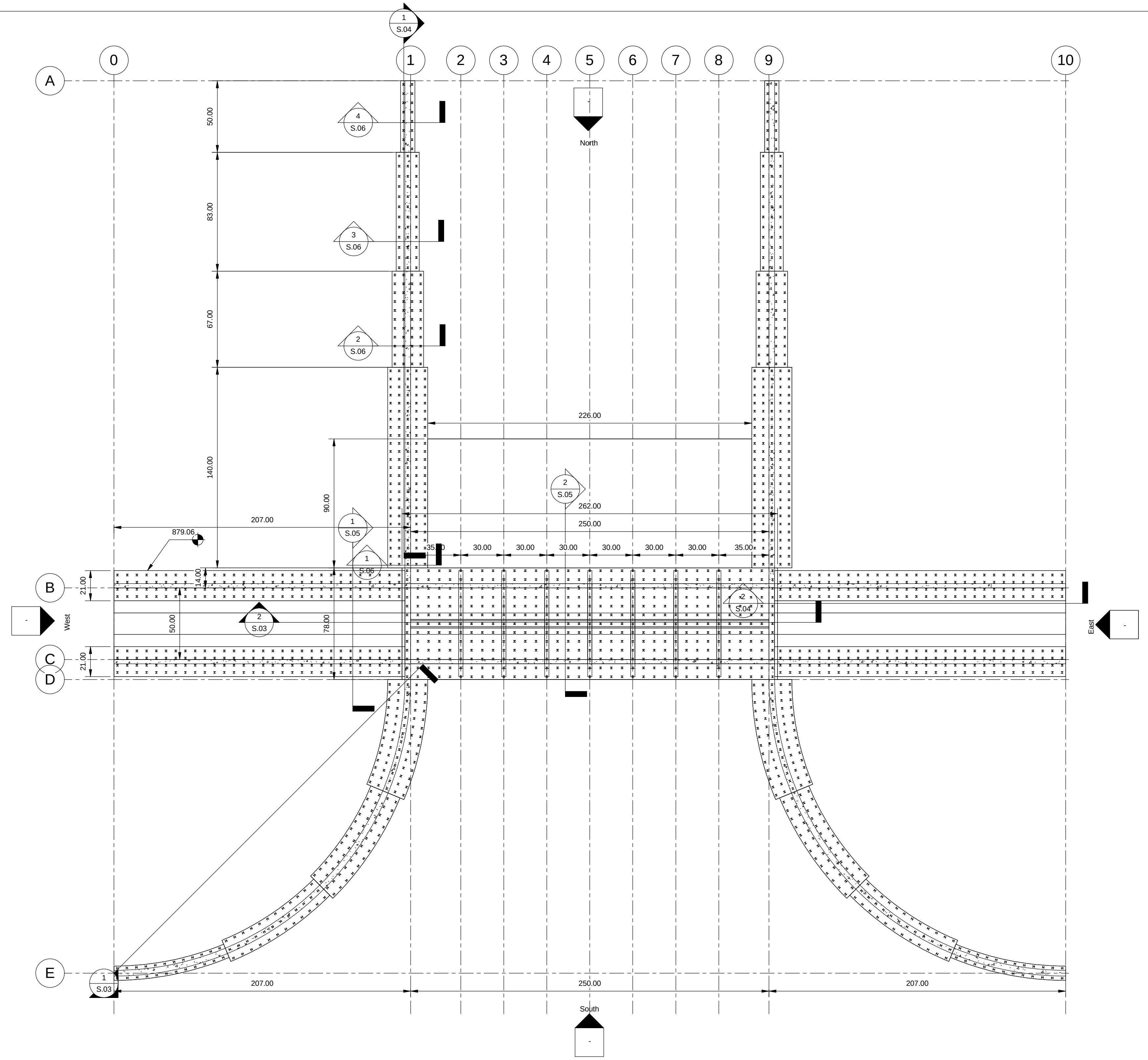
Av,min = 0.75 sqrt(fc') bw*s/fy = 0.70 * s
but not less than 50bw*s/ fy = 23.33333333 * s
s max = Av fy / 0.75 sqrt(fc') bw = 0.00 in
s max = Av fy /50 bw = 0.00 in

11.5.5.3

Where Vs exceeds 4*sqrt(Fc')*bw*d maximum spacings shall be reduced by one-half

ATTACHMENT

F-R5.3 Maple River Aqueduct Structure Drawings



① S01 - FOUNDATION PLAN
1" = 40'-0"

02/14/11

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ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – MAPLE AQUEDUCT –
FOUNDATION PLAN

BARR PROJECT No.
34091004
CLIENT PROJECT No.
DWG No.
S.01
REV No.

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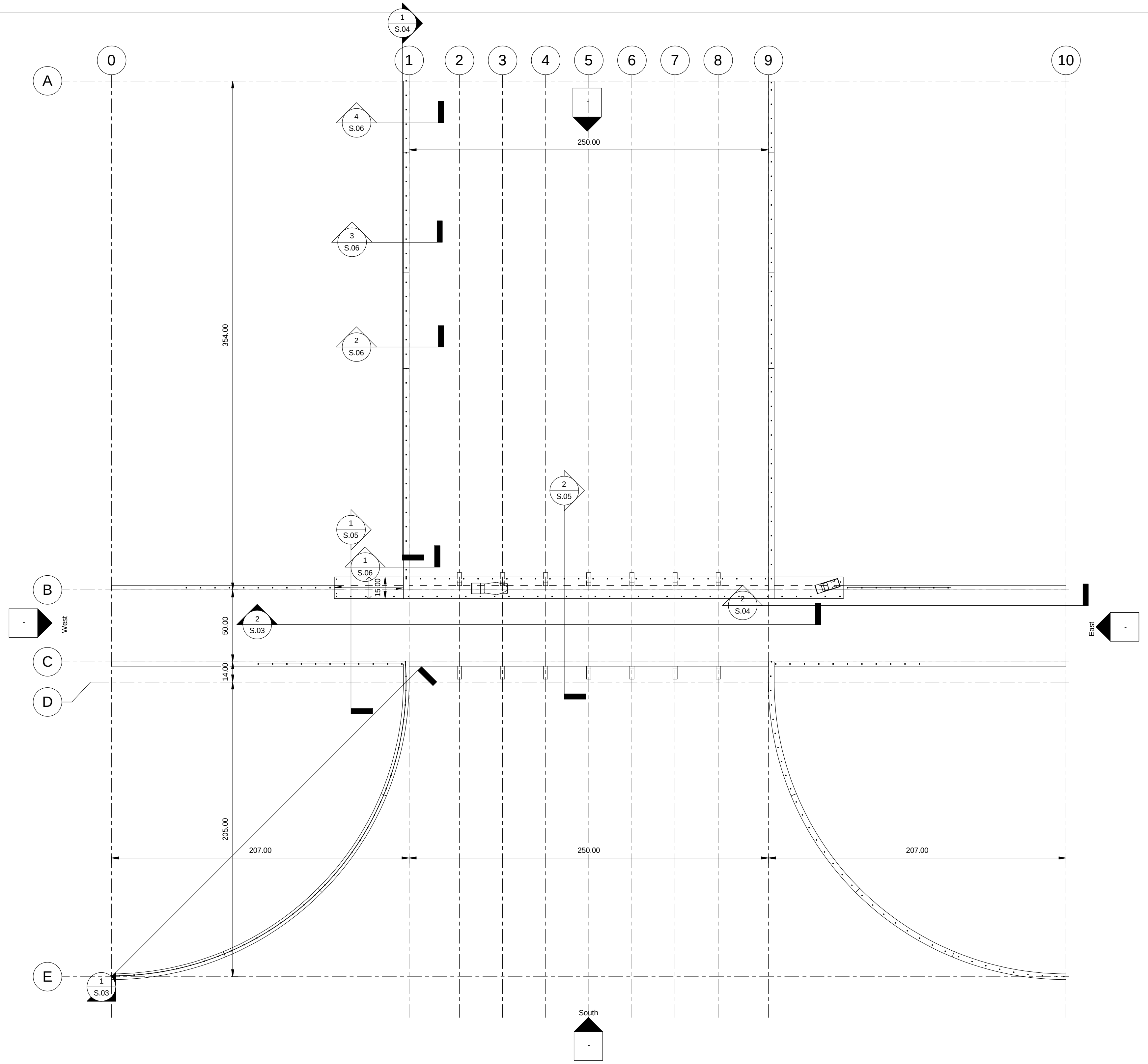

① S02 - TOP PLAN
1" = 40'-0"

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FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL - MAPLE AQUEDUCT -
GENERAL ARRANGEMENT PLAN

BARR PROJECT No.
34091004

CLIENT PROJECT No.

DWG No.
S.02

REV No.

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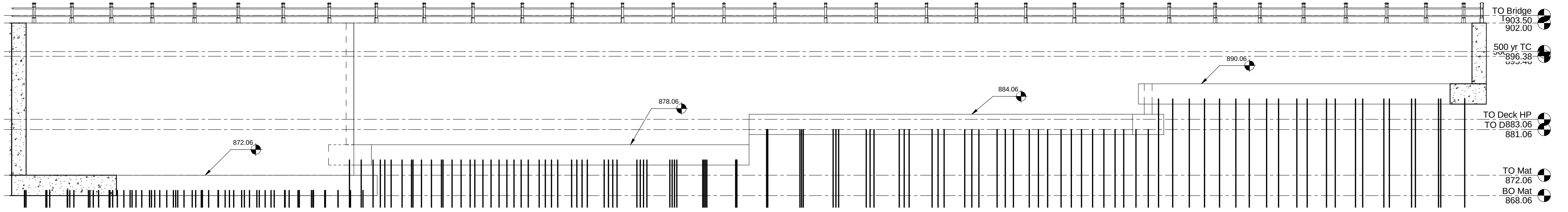
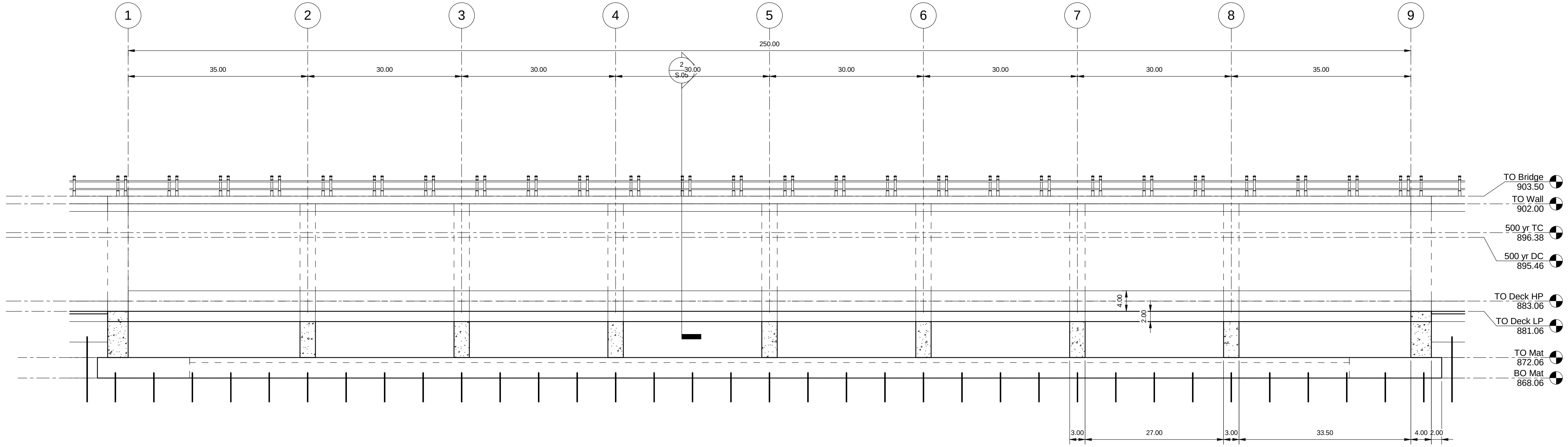


① Section 1
1" = 10'-0"



② Section 2
1" = 10'-0"

02/14/11

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Date	Issue Date
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Checked	TSH
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FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
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FLOOD CONTROL – MAPLE AQUEDUCTT –
ELEVATIONS

BARR PROJECT No.
34091004
CLIENT PROJECT No.
DWG No.
S.03
REV No.

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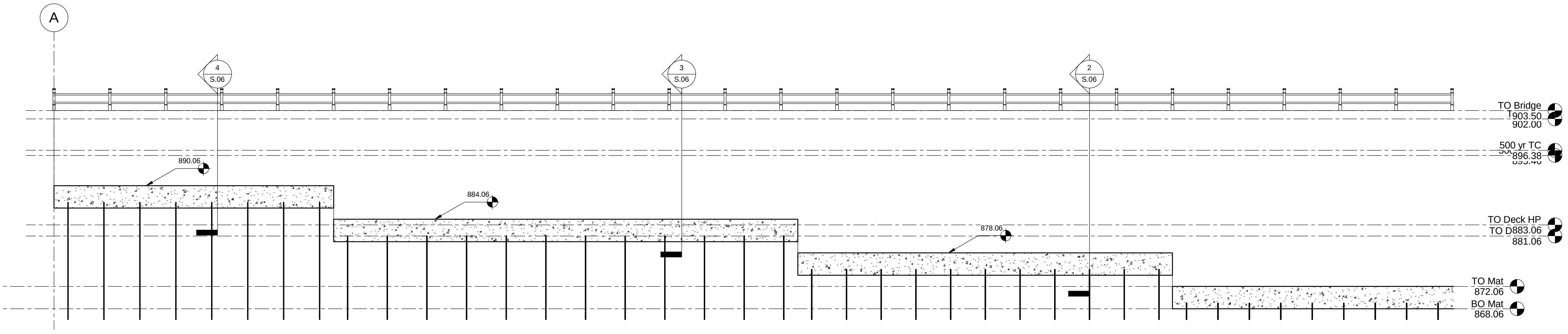
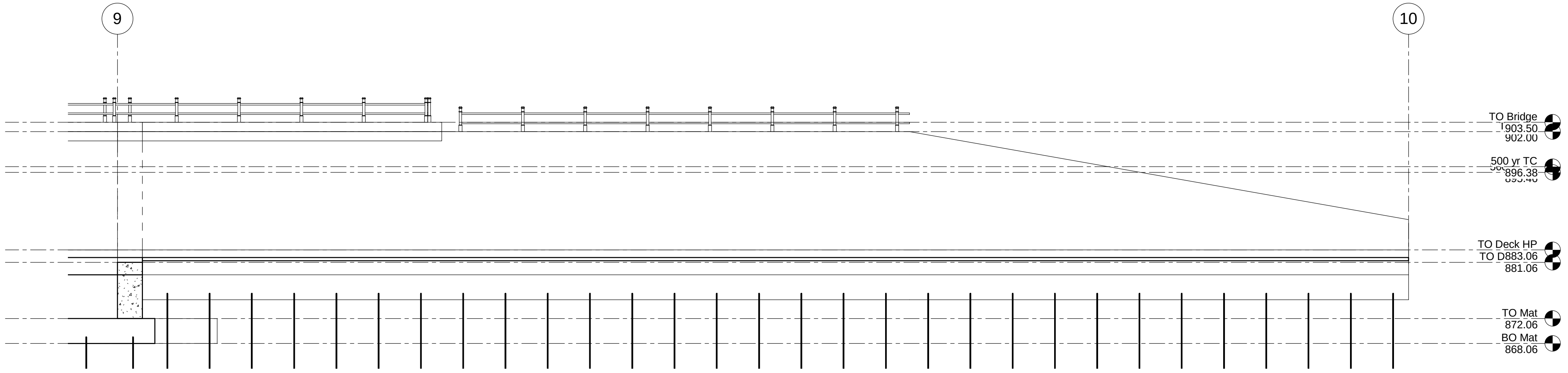


① Section 5
1" = 10'-0"



② Section 15
1" = 10'-0"

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Fax: (952) 832-2600
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Date	Issue Date
Drawn	Author
Checked	Checker
Designed	Designer
Approved	Approver

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FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
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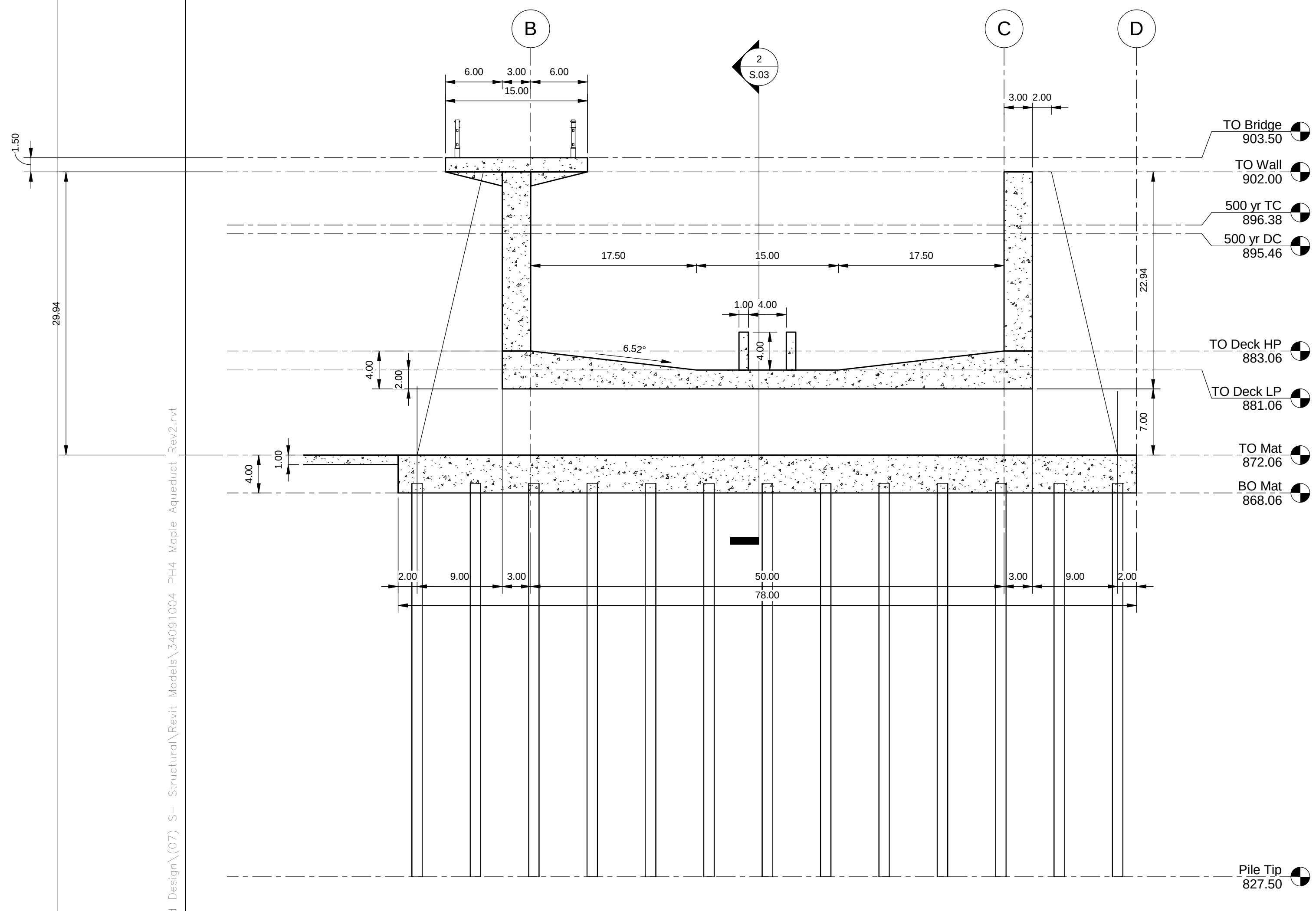
FLOOD CONTROL – MAPLE AQUEDUCTT –
ELEVATIONS

BARR PROJECT No.
34091004

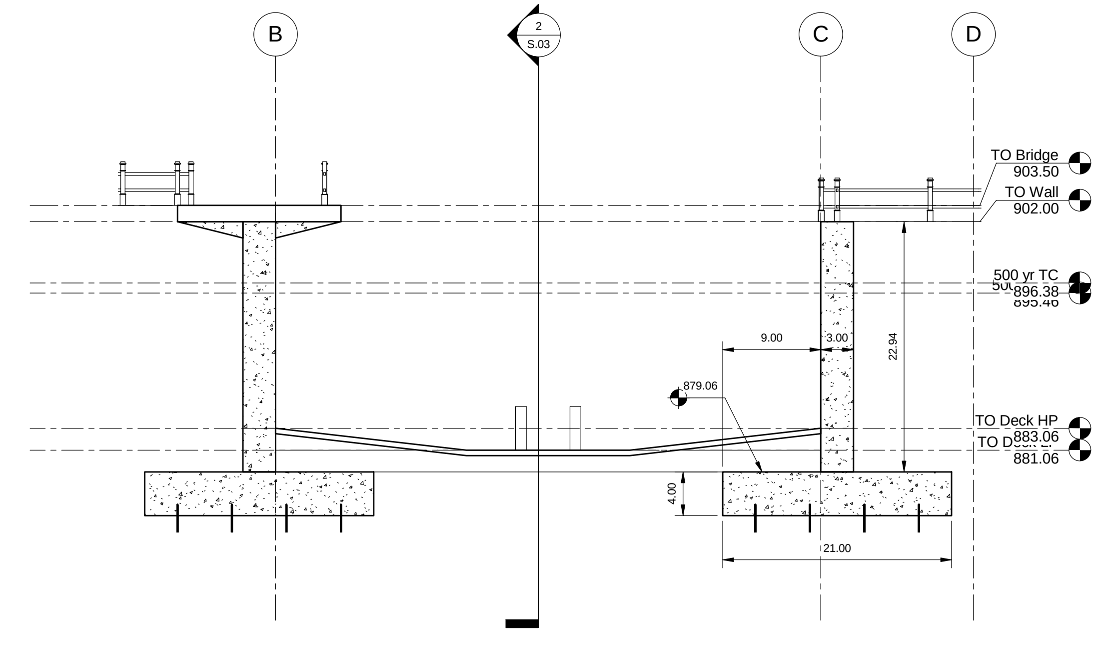
CLIENT PROJECT No.

DWG No.
S.04

REV No.



② Section 3
1/8" = 1'-0"



① Section 7
1/8" = 1'-0"

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Scale	1/8" = 1'-0"
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FEASIBILITY STUDY, PHASE 4
RED RIVER
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FLOOD CONTROL - MAPLE AQUEDUCT -
OVERALL SECTIONS

BARR PROJECT No.
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CLIENT PROJECT No.
DWG No.
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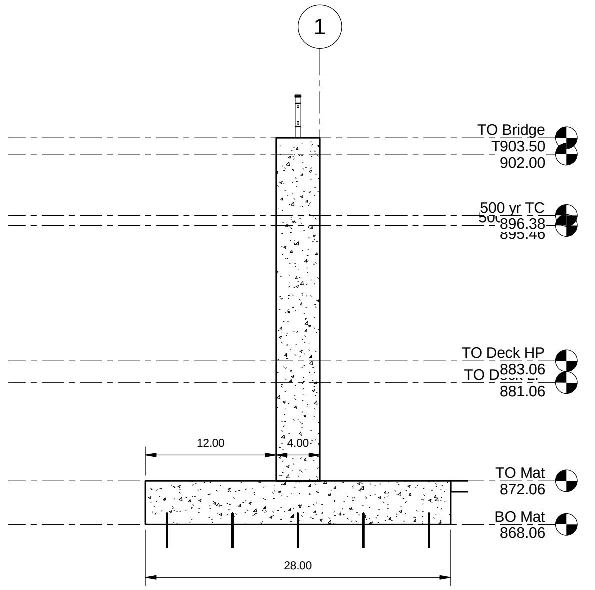
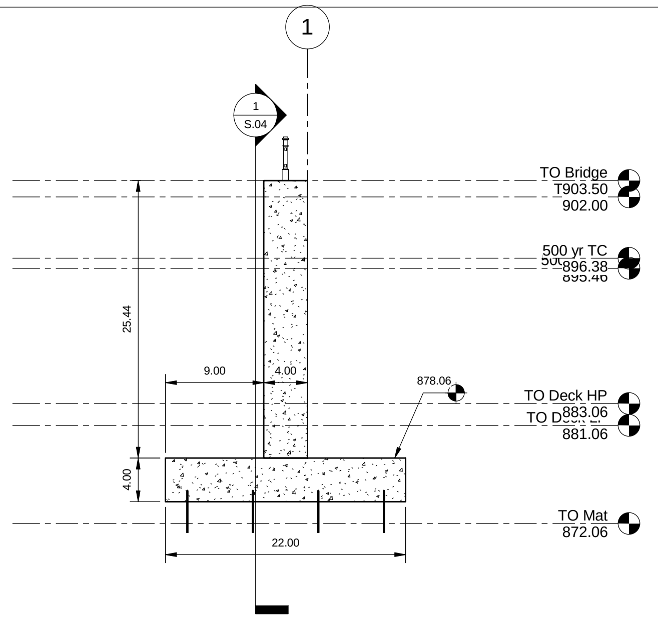
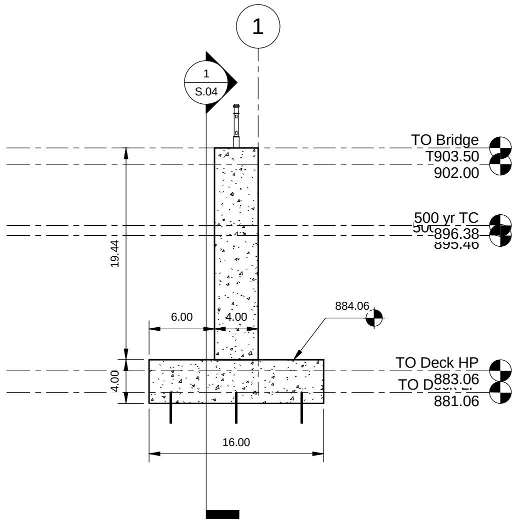


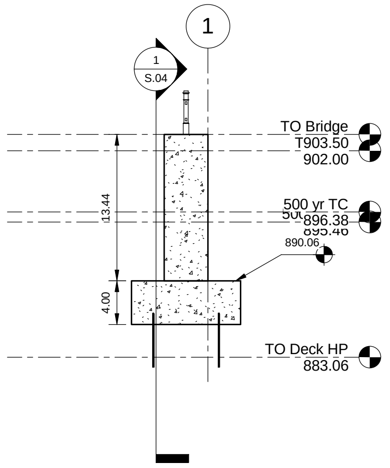
① Section 4
1/8" = 1'-0"



② Section 8
1/8" = 1'-0"



③ Section 9
1/8" = 1'-0"



④ Section 14
1/8" = 1'-0"

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FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – MAPLE AQUEDUCT – WALL
SECTIONS

BARR PROJECT No.
34091004

CLIENT PROJECT No.

DWG No.
S.06

REV No.

Structural Foundation Schedule

Type	Structural Usage	Count	Width	Length	Foundation Thickness	Area	Forming	Volume	Volume CY	Reinf. (lbs/sf)	Total Reinf. (lbs)
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	50.00	4.00	500 SF	510 SF	2000.00 CF	74.07 CY	27.2	13600
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	80.50	4.00	805 SF	769 SF	3220.13 CF	119.26 CY	27.2	21897
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	50.00	4.00	500 SF	510 SF	2000.00 CF	74.07 CY	27.2	13600
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	80.50	4.00	805 SF	769 SF	3220.13 CF	119.26 CY	27.2	21897
Retaining Footing - 36" x 36" x 48": 4						2610 SF	2559 SF	10440.25 CF	386.68 CY		70994
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	83.00	4.00	1328 SF	842 SF	5312.00 CF	196.74 CY	27.2	36122
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	80.50	4.00	1288 SF	820 SF	5152.22 CF	190.82 CY	27.2	35035
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	83.00	4.00	1328 SF	842 SF	5312.00 CF	196.74 CY	27.2	36122
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	80.50	4.00	1288 SF	820 SF	5152.22 CF	190.82 CY	27.2	35035
Retaining Footing - 72" x 72" x 48": 4						5232 SF	3324 SF	20928.43 CF	775.13 CY		142313
Retaining Footing - 108" x 108" x 48"	Retaining	1	21.00	203.00	4.00	4263 SF	1904 SF	17052.00 CF	631.56 CY	27.2	115954
Retaining Footing - 108" x 108" x 48"	Retaining	1	21.00	203.00	4.00	4263 SF	1904 SF	17052.00 CF	631.56 CY	27.2	115954
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	67.00	4.00	1474 SF	757 SF	5896.00 CF	218.37 CY	27.2	40093
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	80.50	4.00	1771 SF	871 SF	7084.30 CF	262.38 CY	27.2	48173
Retaining Footing - 108" x 108" x 48"	Retaining	1	21.00	203.00	4.00	4263 SF	1904 SF	17052.00 CF	631.56 CY	27.2	115954
Retaining Footing - 108" x 108" x 48"	Retaining	1	21.00	123.00	4.00	2583 SF	1224 SF	10332.00 CF	382.67 CY	27.2	70258
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	67.00	4.00	1474 SF	757 SF	5896.00 CF	218.37 CY	27.2	40093
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	80.50	4.00	1771 SF	871 SF	7084.30 CF	262.38 CY	27.2	48173
Retaining Footing - 108" x 108" x 48"	Retaining	1	21.00	80.00	4.00	1680 SF	859 SF	6720.00 CF	248.89 CY	27.2	45696
Retaining Footing - 108" x 108" x 48": 9						23542 SF	11050 SF	94168.59 CF	3487.73 CY		640346
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	140.00	4.00	3920 SF	1428 SF	15680.00 CF	580.74 CY	27.2	106624
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	80.50	4.00	2254 SF	922 SF	9016.38 CF	333.94 CY	27.2	61311
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	140.00	4.00	3920 SF	1428 SF	15680.00 CF	580.74 CY	27.2	106624
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	80.50	4.00	2254 SF	922 SF	9016.38 CF	333.94 CY	27.2	61311
Retaining Footing - 144" x 144" x 48": 4						12348 SF	4701 SF	49392.76 CF	1829.36 CY		335871
Grand total: 21						43733 SF	21633 SF	174930.04 CF	6478.89 CY		1189524

Floor Schedule

Type	Comments	Elevated	Type Comments	Structural Usage	Count	Thickness	Perimeter	Area	Forming	Volume	Volume (cy)	Est. Reinf.	Total Reinf. (lbs)	Takeoff
12" Concrete	SOG	No	Apron Slab	Slab	1	1.00	632.00	20340 SF	632	20340.00 CF	753.33 CY	13.6	276624	Yes
Apron Slab: 1							632.00	20340 SF	632	20340.00 CF	753.33 CY		276624	
18" Concrete	Deck	Yes	Bridge	Slab	1	1.50	546.00	3870 SF	4689	5805.00 CF	215.00 CY	27.2	105264	Yes
18" Concrete	Deck	Yes	Bridge	Slab	1	1.50	126.00	720 SF	909	1080.00 CF	40.00 CY	27.2	19584	Yes
18" Concrete	Deck	Yes	Bridge	Slab	1	1.50	126.00	720 SF	909	1080.00 CF	40.00 CY	27.2	19584	Yes
Bridge: 3							798.00	5310 SF	6507	7965.00 CF	295.00 CY		144432	
24" Concrete	Deck	Yes	Deck Slab	Slab	1	2.00	628.00	14507 SF	15763	41022.00 CF	1519.33 CY	27.2	394584	Yes
Deck Slab: 1							628.00	14507 SF	15763	41022.00 CF	1519.33 CY		394584	
6" Concrete	RipRap	No	Grading	Slab	1	0.50	506.00	10196 SF	253	5098.12 CF	0.00 CY	0	0	No
6" Concrete	RipRap	No	Grading	Slab	1	0.50	506.00	10196 SF	253	5098.12 CF	0.00 CY	0	0	No
Grading: 2							1012.00	20392 SF	506	10196.25 CF	0.00 CY		0	
48" Concrete	SOG	No	Mat Foundation	Slab	1	4.00	680.00	20436 SF	2720	81744.00 CF	3027.56 CY	27.2	555859	Yes
Mat Foundation: 1							680.00	20436 SF	2720	81744.00 CF	3027.56 CY		555859	
Grand total: 8							3750.00	80985 SF	26128	161267.25 CF	5595.22 CY		1371500	

Structural Column Schedule

Type	Count	Length	Top Level	Base Level	Base Offset	Top Offset	Volume	Height
HP14X89	845	35118.20	BO Mat	Pile Tip	0.00	1.00	7.45 CF	41.54
41.56: 845		35118.20						
HP14X89	192	9131.52	BO Mat	Pile Tip	0.00	7.00	8.52 CF	47.54
47.56: 192		9131.52						
HP14X89	480	23308.80	BO Mat	Pile Tip	0.00	8.00	8.70 CF	48.54
48.56: 480		23308.80						
HP14X89	144	7712.64	BO Mat	Pile Tip	0.00	13.00	9.60 CF	53.54
53.56: 144		7712.64						
HP14X89	80	4764.80		Pile Tip	0.00		10.68 CF	59.54
59.56: 80		4764.80						
Grand total: 1741		80035.96						

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FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL - MAPLE AQUEDUCT - SCHEDULES

BARR PROJECT No. 34091004
CLIENT PROJECT No.
DWG No. S.07
REV No.

Wall Schedule

Type	Type Comments	Comments	Length	Width	Height	Area	Forming	Volume	Volume CY	Reinf. (lbs/sf)	Total Reinf. (lbs)	Takeoff
Exterior - 72" Concrete	Aquaduct	Deck	258.00	6.00	1.50	387 SF	792 SF	1161.00 CF	43.00 CY	27.2	10526	
Exterior - 72" Concrete	Aquaduct	Deck	254.00	6.00	1.48	375 SF	768 SF	375.00 CF	13.89 CY	27.2	10200	
Exterior - 72" Concrete	Aquaduct	Deck	48.00	6.00	1.50	72 SF	162 SF	432.00 CF	16.00 CY	27.2	1958	
Exterior - 72" Concrete	Aquaduct	Deck	48.00	6.00	1.50	72 SF	162 SF	216.00 CF	8.00 CY	27.2	1958	
Exterior - 72" Concrete	Aquaduct	Deck	50.00	6.00	1.44	72 SF	161 SF	72.00 CF	2.67 CY	27.2	1958	
Exterior - 72" Concrete	Aquaduct	Deck	50.00	6.00	1.44	72 SF	161 SF	432.00 CF	16.00 CY	27.2	1958	
Deck: 6			708.00			1050 SF	2206 SF	2688.00 CF	99.56 CY		28560	
Exterior - 12" Concrete	Aquaduct	Low Flow Walls	250.05	1.00	4.00	1000 SF	2008 SF	1000.19 CF	37.04 CY	27.2	27205	
Exterior - 12" Concrete	Aquaduct	Low Flow Walls	250.05	1.00	4.00	1000 SF	2008 SF	1000.19 CF	37.04 CY	27.2	27205	
Low Flow Walls: 2			500.10			2000 SF	4017 SF	2000.39 CF	74.09 CY		54410	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aquaduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Pier: 21			546.00			5049 SF	11650 SF	15148.14 CF	561.04 CY		137343	
Exterior - 36" Concrete	Aquaduct	Wall	254.00	3.00	18.64	4735 SF	9582 SF	14205.00 CF	526.11 CY	27.2	128792	
Exterior - 36" Concrete	Aquaduct	Wall	254.00	3.00	18.64	4735 SF	9582 SF	14205.00 CF	526.11 CY	27.2	128792	
Wall: 2			508.00			9470 SF	19164 SF	28410.00 CF	1052.22 CY		257584	
Aquaduct: 31			2262.10			17570 SF	37037 SF	48246.53 CF	1786.91 CY		477898	

Wall Schedule

Type	Type Comments	Comments	Length	Width	Height	Area	Forming	Volume	Volume CY	Reinf. (lbs/sf)	Total Reinf. (lbs)	Takeoff
Retaining - 36" Concrete	Retaining	Wall	205.00	3.00	22.72	4657 SF	9450 SF	13970.46 CF	517.42 CY	27.2	126666	
Retaining - 36" Concrete	Retaining	Wall	205.00	3.00	22.72	4657 SF	9450 SF	13970.46 CF	517.42 CY	27.2	126666	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	32.69	409 SF	1079 SF	1634.64 CF	60.54 CY	27.2	11116	
Exterior - 48" Concrete	Retaining	Wall	140.00	4.00	31.44	4402 SF	9055 SF	17606.40 CF	652.09 CY	27.2	119724	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	34.37	430 SF	1134 SF	1718.64 CF	63.65 CY	27.2	11687	
Exterior - 48" Concrete	Retaining	Wall	67.00	4.00	25.44	1704 SF	3612 SF	6817.92 CF	252.52 CY	27.2	46362	
Exterior - 48" Concrete	Retaining	Wall	83.00	4.00	19.44	1614 SF	3383 SF	6454.08 CF	239.04 CY	27.2	43888	
Exterior - 48" Concrete	Retaining	Wall	50.00	4.00	13.44	672 SF	1452 SF	2688.00 CF	99.56 CY	27.2	18278	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	30.23	2434 SF	5109 SF	9641.10 CF	357.08 CY	27.2	66199	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	24.17	1946 SF	4085 SF	7709.01 CF	285.52 CY	27.2	52933	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	18.12	1458 SF	3062 SF	5776.92 CF	213.96 CY	27.2	39666	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	12.06	971 SF	2038 SF	3844.83 CF	142.40 CY	27.2	26400	
Exterior - 48" Concrete	Retaining	Wall	53.00	4.00	9.00	477 SF	1026 SF	1908.00 CF	70.67 CY	27.2	12974	
Retaining - 36" Concrete	Retaining	Wall	125.00	3.00	22.57	2822 SF	5779 SF	8464.86 CF	313.51 CY	27.2	76748	
Retaining - 36" Concrete	Retaining	Wall	205.00	3.00	22.72	4657 SF	9450 SF	13970.46 CF	517.42 CY	27.2	126666	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	32.45	406 SF	1071 SF	1622.64 CF	60.10 CY	27.2	11034	
Exterior - 48" Concrete	Retaining	Wall	140.00	4.00	31.44	4402 SF	9055 SF	17606.40 CF	652.09 CY	27.2	119724	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	34.13	427 SF	1126 SF	1706.64 CF	63.21 CY	27.2	11605	
Exterior - 48" Concrete	Retaining	Wall	67.00	4.00	25.44	1704 SF	3612 SF	6817.92 CF	252.52 CY	27.2	46362	
Exterior - 48" Concrete	Retaining	Wall	83.00	4.00	19.44	1614 SF	3383 SF	6454.08 CF	239.04 CY	27.2	43888	
Exterior - 48" Concrete	Retaining	Wall	50.00	4.00	13.44	672 SF	1452 SF	2688.00 CF	99.56 CY	27.2	18278	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	30.23	2434 SF	5109 SF	9641.10 CF	357.08 CY	27.2	66199	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	24.17	1946 SF	4085 SF	7709.01 CF	285.52 CY	27.2	52933	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	18.12	1458 SF	3062 SF	5776.92 CF	213.96 CY	27.2	39666	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	12.06	971 SF	2038 SF	3844.83 CF	142.40 CY	27.2	26400	
Exterior - 48" Concrete	Retaining	Wall	53.00	4.00	8.89	471 SF	1013 SF	1884.00 CF	69.78 CY	27.2	12811	
Retaining - 36" Concrete	Retaining	Wall	80.00	3.00	15.89	1271 SF	2637 SF	3812.86 CF	141.22 CY	27.2	34570	
Wall: 27			2300.03			51082 SF	106806 SF	185740.17 CF	6879.27 CY		1389440	
Retaining: 27			2300.03			51082 SF	106806 SF	185740.17 CF	6879.27 CY		1389440	
Grand total: 58			4562.12			68652 SF	143843 SF	233986.70 CF	8666.17 CY		1867338	

02/18/11

PRELIMINARY - FOR DESIGN PURPOSES ONLY

#	BY	CHK.	DATE	REVISION DESCRIPTION

SIGNATURE
PRINTED NAME
DATE
REG. NO.

CLIENT
BID
CONSTRUCTION
RELEASED TO/FOR
A B C 0 1 2 3
DATE RELEASED

BARR
Corporate Headquarters:
Minneapolis, Minnesota
Ph: 1-800-632-2277

Project Office:
BARR ENGINEERING CO.
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
Ph: 1-800-632-2277
Fax: (952) 832-2600
www.barr.com

Scale	
Date	Issue Date
Drawn	MBI
Checked	TSH
Designed	MBI
Approved	Approver

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – MAPLE AQUEDUCT –
SCHEDULES – II

BARR PROJECT No.
34091004
CLIENT PROJECT No.
DWG No.
S.08
REV No.

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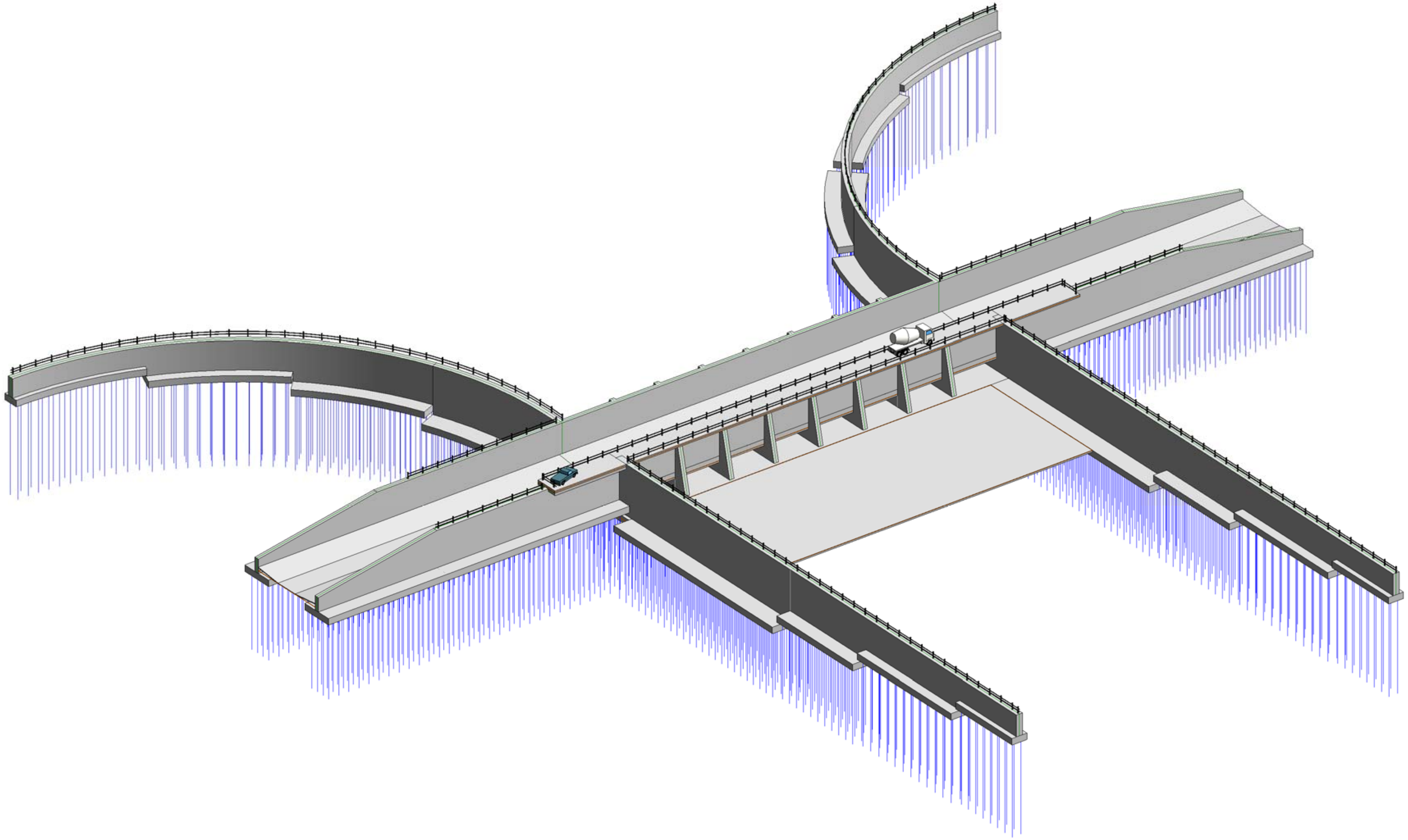

① 3D VIEW OVERALL D/S

02/14/11

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SIGNATURE
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DATE REG. NO.

CLIENT							
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CONSTRUCTION							
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Drawn	MBI
Checked	TSH
Designed	MBI
Approved	Approver

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – MAPLE AQUEDUCT – 3D VIEWS

BARR PROJECT No.
34091004

CLIENT PROJECT No.

DWG No.
S.09

REV No.

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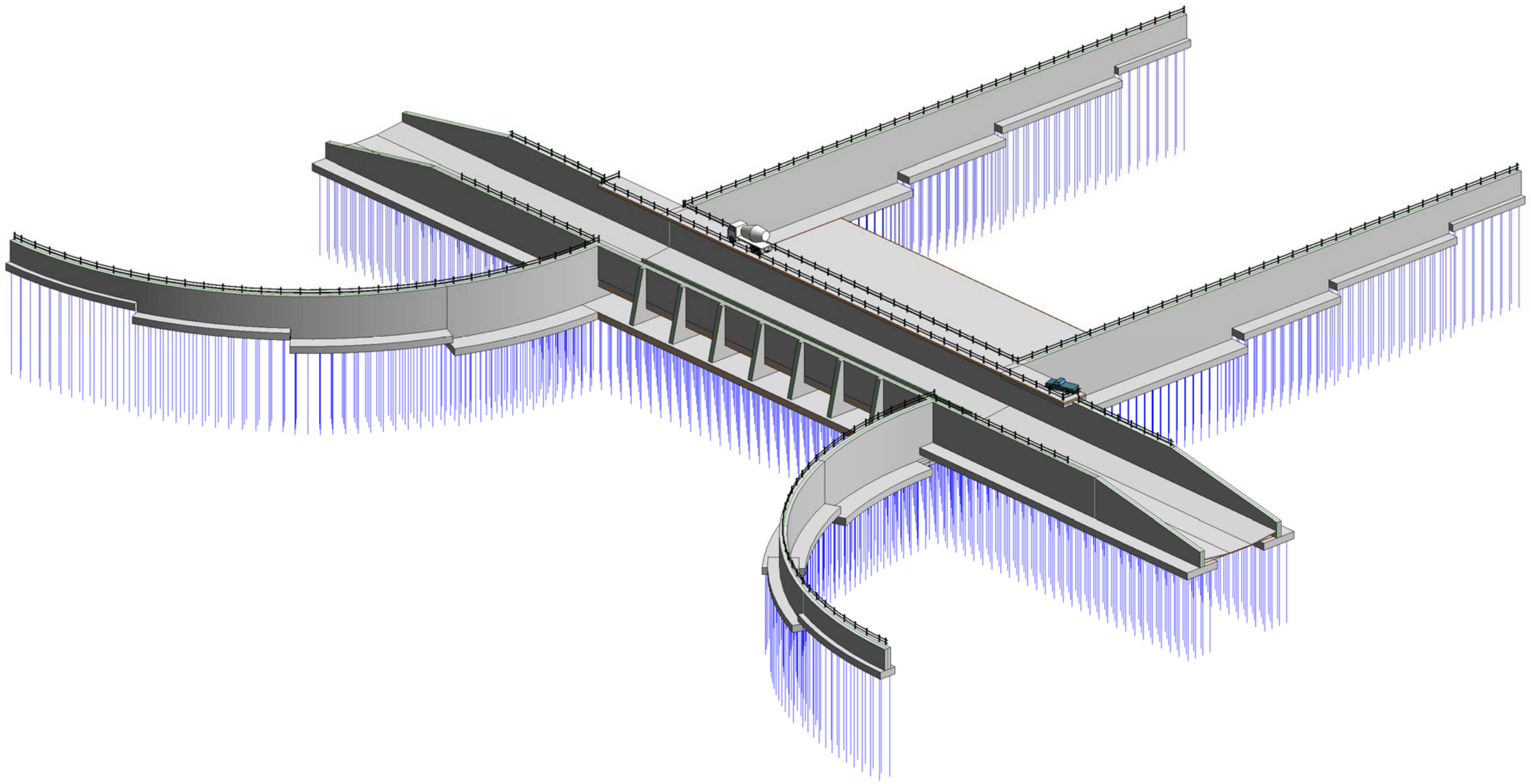

① 3D VIEW OVERALL U/S

02/24/11

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SIGNATURE ____
PRINTED NAME ____
DATE ____ REG. NO. ____

CLIENT							
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CONSTRUCTION							
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	DATE RELEASED						

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MINNEAPOLIS, MN 55435

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Fax: (952) 832-2600
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Scale	
Date	Issue Date
Drawn	Author
Checked	Checker
Designed	Designer
Approved	Approver

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – MAPLE AQUEDUCT – 3D VIEWS

BARR PROJECT No.
34091004

CLIENT PROJECT No.

DWG No.
S.10

REV No.

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES
EXHIBIT S – STRUCTURAL DESIGN COMPUTATIONS—SHEETPILE/ROCKFILL SPILLWAYS

Report for the US Army Corps of Engineers, and the cities of Fargo, ND and Moorhead, MN

By: Barr Engineering Co.

FINAL – February 28, 2011

TABLE OF CONTENTS

ATTACHMENTS

APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT S – STRUCTURAL DESIGN COMPUTATIONS—
SHEETPILE/ROCKFILL SPILLWAYS

F-S1.0 SHEETPILE/ROCKFILL WEIR SPILLWAYS

A description of the structural design concepts and criteria for the sheetpile weirs can be found in Appendix F. *Attachments F-S1.1 through F-S1.2 of this Exhibit S were previously presented as part of Exhibit P of Appendix F of the Phase 3 report submitted on August 6, 2010 and present the structural design computations for the sheetpile/rockfill weir spillway structures.* This section is applicable for the following structures:

1) FCP Primary Inlet Weir
2) Sheyenne River Spillway Weir
3) Maple River Spillway Weir

ATTACHMENT

F-S1.1 Mohr-Coulomb Envelope for Soils Backfill Behind Weirs

Paul Nielsen

From: Jed Greenwood
Sent: Tuesday, June 15, 2010 2:04 PM
To: Aaron Grosser; Paul Nielsen
Cc: Miguel Wong; Ivan Contreras
Subject: Brenna Peak Strength Envelope

All,

I came up with a drained envelope for use in the Brenna Formation clays. It is based on the peak strength of the material from CIU triax and direct shear testing. It is intended to be used only in design of piles where a sustained load will be present, e.g. soil backfill behind a sheet pile wall, etc. Between 0.5 and 1.0 tsf (1,000 to 2,000 psf), the Mohr-Coulomb envelope is:

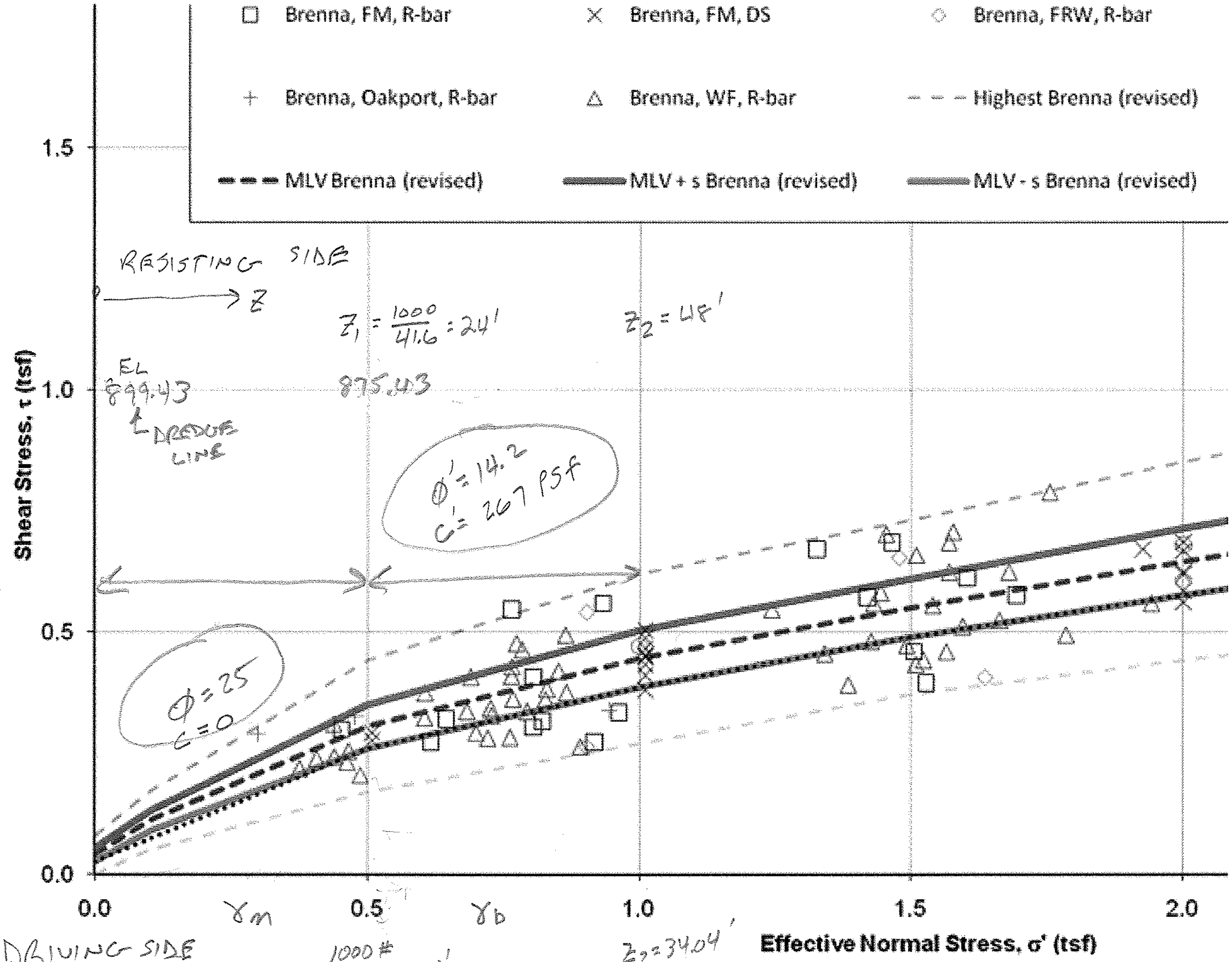
ϕ'=14.2 degrees, c'=267 psf

This envelope was determined based on the methodology the Corps used, which is the most likely value (MLV) minus 1 standard deviation. The plot is below.

Jed

- USE EM 1110-2-2504
- P. 6-1 FOR ALLOW, STRESS
- P. 9-2 CORROSION
- P. 5-3 FOR F.S. (PASSIVE SIDE) IN STABILITY COMP

USED 1.3 INCREASE ON DEPTH FOR F.S. COE METHOD NOT USED IN FEASIBILITY



DRIVING SIDE

Z →

$Z_1 = \frac{1000\#}{100\ pcf} = 10'$

$Z_2 = 34.04'$

TOP EL 909.50

EL 899.43

$\Delta Z\ \frac{1000\ \#}{41.6} = 24.04'$

Jed Greenwood, PE, PG

$\gamma_M = 100\ pcf$

Geotechnical Engineer
Minneapolis office: 952.832.2773
cell: 952.769.7488
jgreenwood@barr.com
www.barr.com

$\gamma_{SAT} = 104\ pcf$

$\gamma_b = 41.6\ pcf$

resourceful. naturally. BARR

ATTACHMENT

F-S1.2 Design Computations for Sheetpile Weirs

X-Tra DEPTH FOR F.S.

$d_1 = 1.3 * 19.44 = 25.3$ ft

$S_{X\,REQ'D} = \frac{M}{F_b} = \frac{24.349 \times 12}{0.5 * 50 \text{ ksf}} = 11.69$ in³/ft

USE LARGER SECTION FOR CORROSION IN FUTURE

USE PZ27
$S_x = 30.2$ in³/ft
$I_x = 184.2$ in⁴/ft

```
PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                          BY CLASSICAL METHODS
DATE: 16-JUN-2010                                              TIME: 11.41.00

                              ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
                              º  INPUT DATA  º
                              ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¼

I.--HEADING:
  'FLOOD CONTROL METRO FEASIBILITY STUDY, DRAINED PHI=25, C=0
  'DIVERSION INLET WEIR, H=10.07 FT

II.--CONTROL
  CANTILEVER WALL DESIGN

  LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES  =  1.00
  LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES =  1.00
```

DEFLECTIONS SHOULD USE SOIL-INTERACTION PROGRAM.

$M = 24.349$ k-ft/ft
$d = 19.44'$
$\Delta = \frac{1.0604 \times 10^{10}}{EI}$
$\Delta = \frac{365.66}{I_x}$

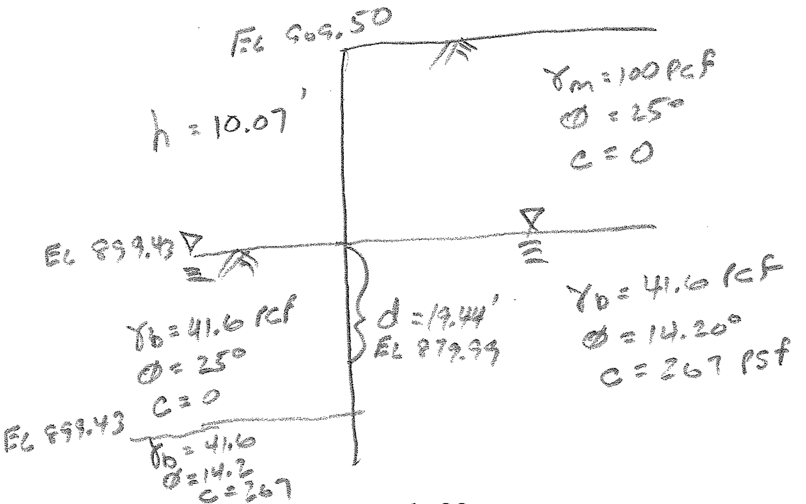
```
III.--WALL DATA
  ELEVATION AT TOP OF WALL     =   909.50 (FT)


IV.--SURFACE POINT DATA

  IV.A--RIGHTSIDE
     DIST. FROM      ELEVATION
     WALL (FT)         (FT)
           .00        909.50

  IV.B-- LEFTSIDE
     DIST. FROM      ELEVATION
     WALL (FT)         (FT)
           .00        899.43


V.--SOIL LAYER DATA
```



```
  V.A.--RIGHTSIDE LAYER DATA
     LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES  =  1.00
     LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES =  1.00

                ANGLE OF               ANGLE OF                              <-SAFETY->
  SAT.   MOIST  INTERNAL    COH-        WALL       ADH-       <--BOTTOM--> <-FACTOR->
 WGHT.   WGHT.  FRICTION    ESION     FRICTION     ESION      ELEV.  SLOPE ACT. PASS.
 (PCF)   (PCF)    (DEG)     (PSF)      (DEG)       (PSF)      (FT)  (FT/FT)
104.00  100.00    25.00        .0        .00          .0     899.43    .00 1.00 1.00
104.00  104.00    14.20     267.0        .00          .0                    DEF  DEF


  V.B.-- LEFTSIDE LAYER DATA
     LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES  =  1.00
     LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES =  1.00

                ANGLE OF               ANGLE OF                              <-SAFETY->
  SAT.   MOIST  INTERNAL    COH-        WALL       ADH-       <--BOTTOM--> <-FACTOR->
```

```
  WGHT.    WGHT.   FRICTION   ESION  FRICTION    ESION    ELEV.  SLOPE ACT. PASS.
  (PCF)    (PCF)    (DEG)     (PSF)    (DEG)     (PSF)    (FT)  (FT/FT)
 104.00   104.00    25.00       .0       .00        .0   875.43     .00 1.00 1.00
 104.00   104.00    14.20    267.0       .00        .0                  DEF  DEF


     VI.--WATER DATA

          UNIT WEIGHT          =    62.40 (PCF)
          RIGHTSIDE ELEVATION =   899.43 (FT)
           LEFTSIDE ELEVATION =   899.43 (FT)
          NO SEEPAGE


     VII.--SURFACE LOADS
          NONE


     VIII.--HORIZONTAL LOADS
          NONE
```

```
     PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                               BY CLASSICAL METHODS
     DATE: 16-JUN-2010                                               TIME: 11.41.12

                           ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
                           º     SOIL PRESSURES FOR     º
                           º   CANTILEVER WALL DESIGN   º
                           ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍͼ

     I.--HEADING

       'FLOOD CONTROL METRO FEASIBILITY STUDY, DRAINED PHI=25, C=0
       'DIVERSION INLET WEIR, H=10.07 FT

     II.--SOIL PRESSURES

          RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
          AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

           LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
          AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.


                                         <---NET PRESSURES---->
           <-LEFTSIDE PRESSURES->       (SOIL PLUS WATER)       <RIGHTSIDE PRESSURES->
  ELEV.     PASSIVE      ACTIVE       ACTIVE       PASSIVE       ACTIVE       PASSIVE
  (FT)       (PSF)        (PSF)        (PSF)         (PSF)        (PSF)         (PSF)
 909.50        .00          .00          .000          .000          .00           .00
 908.50        .00          .00        40.586       246.391        40.59        246.39
 907.50        .00          .00        81.172       492.783        81.17        492.78
 906.50        .00          .00       121.758       739.174       121.76        739.17
 905.50        .00          .00       162.343       985.565       162.34        985.57
```

```
904.50          .00         .00     202.929   1231.956      202.93      1231.96
903.50          .00         .00     243.515   1478.348      243.52      1478.35
902.50          .00         .00     284.101   1724.739      284.10      1724.74
901.50          .00         .00     324.687   1971.130      324.69      1971.13
900.50          .00         .00     365.273   2217.522      365.27      2217.52
899.50          .00         .00     405.859   2463.913      405.86      2463.91
899.43+         .00         .00     408.700   2481.160      408.70      2481.16
899.43-         .00         .00     194.564   2347.589      194.56      2347.59
898.50        95.32       15.70     122.686   2395.726      218.01      2411.43
898.43       102.50       16.88     117.276   2399.349      219.77      2416.23
897.50       197.82       32.59      45.398   2447.486      243.22      2480.07
896.91       258.03       42.50        .000   2477.889      258.03      2520.39
896.71       279.18       45.99     -15.945   2488.567      263.23      2534.55
896.50       300.32       49.47     -31.890   2499.246      268.43      2548.71
895.50       402.82       66.35    -109.178   2551.006      293.64      2617.36
894.50       505.32       83.24    -186.466   2602.765      318.85      2686.00
893.50       607.82      100.12    -263.754   2654.525      344.06      2754.65
892.50       710.32      117.00    -341.042   2706.285      369.27      2823.29
891.50       812.82      133.89    -418.330   2758.045      394.49      2891.93
890.50       915.31      150.77    -495.618   2809.805      419.70      2960.58
889.50      1017.81      167.66    -572.906   2861.565      444.91      3029.22
888.50      1120.31      184.54    -650.194   2913.324      470.12      3097.86
887.50      1222.81      201.42    -727.482   2965.084      495.33      3166.51
886.50      1325.31      218.31    -804.770   3016.844      520.54      3235.15
885.50      1427.81      235.19    -882.058   3068.604      545.75      3303.79
884.50      1530.31      252.07    -959.346   3120.364      570.96      3372.44
883.50      1632.81      268.96   -1036.633   3172.124      596.17      3441.08
882.50      1735.30      285.84   -1113.921   3223.884      621.38      3509.72
881.50      1837.80      302.72   -1191.209   3275.643      646.59      3578.37
880.50      1940.30      319.61   -1268.497   3327.403      671.80      3647.01
879.50      2042.80      336.49   -1345.785   3379.163      697.02      3715.66
878.50      2145.30      353.38   -1423.073   3430.923      722.23      3784.30
877.50      2247.80      370.26   -1500.361   3482.683      747.44      3852.94
876.50      2350.30      387.14   -1577.649   3534.443      772.65      3921.59
875.50      2452.80      404.03   -1654.937   3586.202      797.86      3990.23
875.43+     2459.97      405.21   -1597.061   3697.754      799.62      3995.03
875.43-     2333.40      189.35   -1597.061   3697.754      799.62      3995.03
874.50      2397.24      212.80   -1574.168   3846.075      823.07      4058.87
873.50      2465.88      238.01   -1617.601   3889.508      848.28      4127.52
872.50      2534.52      263.22   -1661.033   3932.941      873.49      4196.16
871.50      2603.17      288.43   -1704.466   3976.374      898.70      4264.80
870.50      2671.81      313.64   -1747.899   4019.806      923.91      4333.45
869.50      2740.45      338.85   -1791.332   4063.239      949.12      4402.09
868.50      2809.10      364.06   -1834.764   4106.672      974.33      4470.73




        PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                                  BY CLASSICAL METHODS
        DATE: 16-JUN-2010                                             TIME: 11.41.31

                           ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
                           º   SUMMARY OF RESULTS FOR   º
                           º   CANTILEVER WALL DESIGN   º
                           ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¼
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I.--HEADING

  'FLOOD CONTROL METRO FEASIBILITY STUDY, DRAINED PHI=25, C=0
  'DIVERSION INLET WEIR, H=10.07 FT

II.--SUMMARY


     RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
     AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

      LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
     AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

WALL BOTTOM ELEV. (FT)     :       879.99
      PENETRATION (FT)     :        19.44

MAX. BEND. MOMENT (LB-FT) :        24349.
      AT ELEVATION (FT)    :       889.19

MAX. SCALED DEFL. (LB-IN3):    1.0604E+10
      AT ELEVATION (FT)    :       909.50

     (NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF
     ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA
     IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)







PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                          BY CLASSICAL METHODS
DATE: 16-JUN-2010                                             TIME: 11.41.31

                        ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
                        º    COMPLETE RESULTS FOR    º
                        º   CANTILEVER WALL DESIGN   º
                        ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¼

I.--HEADING

  'FLOOD CONTROL METRO FEASIBILITY STUDY, DRAINED PHI=25, C=0
  'DIVERSION INLET WEIR, H=10.07 FT

II.--RESULTS

             BENDING                          SCALED         NET
 ELEVATION   MOMENT          SHEAR          DEFLECTION     PRESSURE
   (FT)      (LB-FT)          (LB)           (LB-IN3)        (PSF)
  909.50          0.             0.        1.0604E+10          .00
  908.50          7.            20.        1.0035E+10        40.59
  907.50         54.            81.        9.4668E+09        81.17
  906.50        183.           183.        8.8982E+09       121.76
  905.50        433.           325.        8.3300E+09       162.34
  904.50        846.           507.        7.7625E+09       202.93
```

```
      903.50          1461.           731.       7.1965E+09        243.52
      902.50          2320.           994.       6.6331E+09        284.10
      901.50          3463.          1299.       6.0738E+09        324.69
      900.50          4931.          1644.       5.5204E+09        365.27
      899.50          6764.          2029.       4.9757E+09        405.86
      899.43          6907.          2058.       4.9379E+09        408.70
      899.43          6907.          2058.       4.9379E+09        194.56
      898.50          8895.          2205.       4.4426E+09        122.69
      898.43          9050.          2214.       4.4058E+09        117.28
      897.50         11149.          2289.       3.9250E+09         45.40
      896.91         12499.          2303.       3.6297E+09           .00
      896.71         12974.          2301.       3.5277E+09        -15.94
      896.50         13448.          2296.       3.4266E+09        -31.89
      895.50         15715.          2226.       2.9515E+09       -109.18
      894.50         17873.          2078.       2.5035E+09       -186.47
      893.50         19845.          1853.       2.0863E+09       -263.75
      892.50         21553.          1550.       1.7035E+09       -341.04
      891.50         22920.          1171.       1.3578E+09       -418.33
      890.50         23868.           714.       1.0516E+09       -495.62
      889.50         24321.           179.       7.8665E+08       -572.91
      888.50         24201.          -432.       5.6362E+08       -650.19
      887.50         23431.         -1121.       3.8231E+08       -727.48
      886.50         21933.         -1887.       2.4140E+08       -804.77
      885.50         19631.         -2731.       1.3826E+08       -882.06
      884.50         16446.         -3651.       6.8922E+07       -959.35
      883.75         13422.         -4395.       3.5758E+07      -1017.49
      883.50         12306.         -4611.       2.7864E+07       -729.71
      882.50          7523.         -4760.       7.9692E+06        432.33
      881.50          3173.         -3747.       1.1365E+06       1594.37
      880.50           417.         -1571.       1.6290E+04       2756.41
      879.99             0.             0.       0.0000E+00       3354.02

        (NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF
        ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA
        IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)


   III.--SOIL PRESSURES
    ELEVATION    < LEFTSIDE PRESSURE (PSF)>    <RIGHTSIDE PRESSURE (PSF)>
      (FT)          PASSIVE         ACTIVE          ACTIVE         PASSIVE
      909.50             0.             0.              0.              0.
      908.50             0.             0.             41.            246.
      907.50             0.             0.             81.            493.
      906.50             0.             0.            122.            739.
      905.50             0.             0.            162.            986.
      904.50             0.             0.            203.           1232.
      903.50             0.             0.            244.           1478.
      902.50             0.             0.            284.           1725.
      901.50             0.             0.            325.           1971.
      900.50             0.             0.            365.           2218.
      899.50             0.             0.            406.           2464.
      899.43+            0.             0.            409.           2481.
      899.43-            0.             0.            195.           2348.
      898.50            95.            16.            218.           2411.
      898.43           102.            17.            220.           2416.
      897.50           198.            33.            243.           2480.
      896.91           258.            43.            258.           2520.
      896.71           279.            46.            263.           2535.
      896.50           300.            49.            268.           2549.
      895.50           403.            66.            294.           2617.
      894.50           505.            83.            319.           2686.
      893.50           608.           100.            344.           2755.
      892.50           710.           117.            369.           2823.
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```
  891.50       813.        134.        394.       2892.
  890.50       915.        151.        420.       2961.
  889.50      1018.        168.        445.       3029.
  888.50      1120.        185.        470.       3098.
  887.50      1223.        201.        495.       3167.
  886.50      1325.        218.        521.       3235.
  885.50      1428.        235.        546.       3304.
  884.50      1530.        252.        571.       3372.
  883.75      1607.        265.        590.       3424.
  883.50      1633.        269.        596.       3441.
  882.50      1735.        286.        621.       3510.
  881.50      1838.        303.        647.       3578.
  880.50      1940.        320.        672.       3647.
  879.99      2043.        336.        697.       3716.
  878.50      2145.        353.        722.       3784.
```

RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT T – STRUCTURAL DESIGN COMPUTATIONS—STEPPED SPILLWAYS

Report for the US Army Corps of Engineers, and the cities of Fargo, ND & Moorhead, MN

By: BARR ENGINEERING

FINAL – Version February 28, 2011

TABLE OF CONTENTS

ATTACHMENTS

F-T1.1	Design Computations for Stepped Spillway Structures at the Lower Rush and Rush Rivers from Phase 3 submittal
F-T1.2	Design Computations for Stepped Spillway Structures at the Lower Rush and Rush Rivers for Phase 4 submittal

APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT T – STRUCTURAL DESIGN COMPUTATIONS—STEPPED SPILLWAYS

F-T1.0 STEPPED SPILLWAYS

A description of the structural design concepts and criteria for the stepped spillways at the Lower Rush River and Rush River can be found in Appendix F Section 4.0. *Attachment F-T1.1 of this Exhibit T presents the structural design computations for the Lower Rush River and Rush River stepped spillway structures previously presented as Exhibit P of Appendix F of the Phase 3 report submitted on August 6th, 2010.* Attachment F-T1.2 of this Exhibit T presents the structural design and quantity computations for the Lower Rush River and Rush River stepped spillway structures for Phase 4.

BARR

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DULUTH, MINNESOTA
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

COMPUTED	CHECKED	SUBMITTED
BY	BY	TO
DATE	DATE	DATE

DATE
SHEET NO. 0
PROJECT NAME RED RIVER DIVERSION
PROJECT NUMBER 34/09-1004
SUBJECT
RUSH & LOWER RUSH DIVERSION WALL STRUCTURES - QUANTITY CALCULATION

INDEX

			DATE
			SHEET NO. 1
			PROJECT NAME RED RIVER DIVERSION
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER 34/09-1004
BY JGT	BY	TO	SUBJECT RUSH & LOWER RUSH RIVER DIV.
DATE 7-22-10	DATE	DATE	

PILING DESIGN

LC 1 - ACTIVE SOIL PRESSURE RETAINING WALL W/ WATER TABLE @ 5' DOWN - NO FLOW IN RIVER

USE UNDRAINED PILE VALUES - USUAL LC FS=2

LC 2 - WATER PRESSURE ONLY AT LOCATION OF NO SOIL - NOT A REAL L.C., BUT SOMETHING ALONG THESE LINES MAY OCCUR NEAR FISH PASSAGE -

USE UNDRAINED PILE VALUES - USUAL LC FS=2

LC 3 - ACTIVE SOIL PRESSURE W/ WATER TABLE LOW - NO FLOW IN RIVER - USE DRAINED PILE VALUES - ASSUME UNUSUAL LC IS APPROPRIATE FOR THIS FS =

BASIS

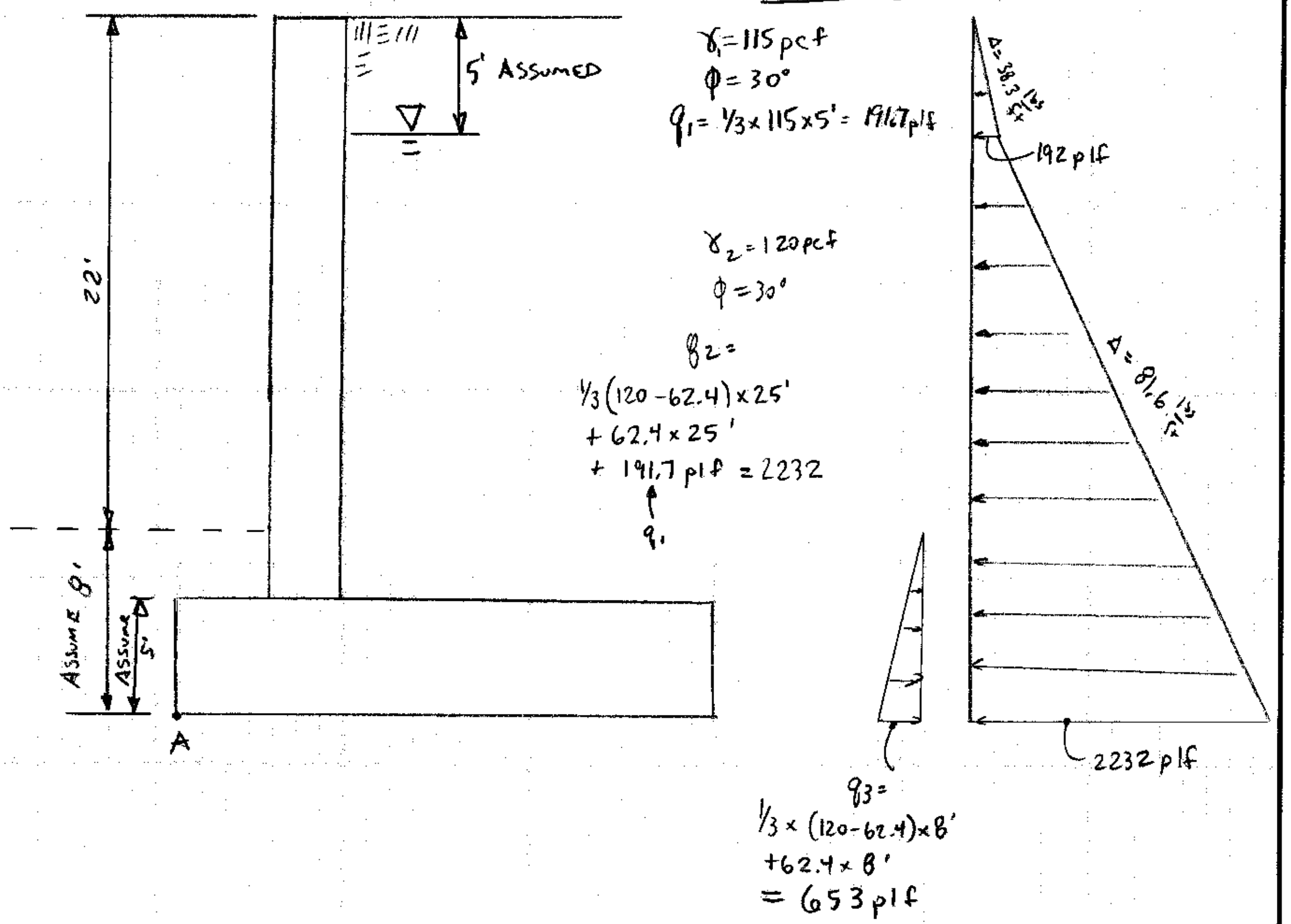
RETAINING WALLS FOR THE RUSH & LOWER RUSH RIVERS HAVE SIMILAR GEOMETRIES. THIS CALCULATION CONSERVATIVELY TAKES THE WORST CASE WALL & COMPUTES QUANTITIES FOR BOTH RIVER DIVERSIONS BASED ON THIS CRITICAL WALL GEOMETRY (FOOTING WIDTH & THICKNESS + WALL THICKNESS - WALL HEIGHT IS MODIFIED TO MATCH CURRENT LAYOUT). THESE CALCULATIONS & DESIGN WORK ASSOCIATED WITH THESE CALCULATIONS IS ONLY REQ'D IN ORDER TO SUPPORT THE CALCULATION OF CONCRETE & REBAR QUANTITIES. DESIGN SHOULD NOT BE CONSIDERED "FOR CONSTRUCTION"

BARR MINNEAPOLIS, MINNESOTA - HIBBING, MINNESOTA DULUTH, MINNESOTA ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI			DATE | SHEET NO. 2
			PROJECT NAME RED RIVER DIVERSION
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER 34/09-1004
BY JGT	BY	TO	SUBJECT
DATE 7-22-10	DATE	DATE	RUSH & LOWER RUSH RIVER WALLS

LC 1

LARGEST RETAINED SOIL HT = 22'

ACTIVE PRESSURE



$\gamma_1 = 115$ pcf

$\phi = 30°$

$q_1 = 1/3 \times 115 \times 5' = 191.7$ plf

$\gamma_2 = 120$ pcf

$\phi = 30°$

$q_2 =$
$1/3(120 - 62.4) \times 25'$
$+ 62.4 \times 25'$
$+ 191.7$ plf $= 2232$
↑
q_1

$q_3 =$
$1/3 \times (120 - 62.4) \times 8'$
$+ 62.4 \times 8'$
$= 653$ plf

TOTAL LATERAL SHEAR

$= 1/2 \times 192\text{plf} \times 5' + 192\text{plf} \times 25' + 1/2 \times 2040\text{ plf} \times 25' - 1/2 \times 653\text{plf} \times 8'$

$480\text{ lbs} + 4800\text{ lbs} + 25,500^{\text{lbs}} - 2612^{\text{lbs}} = \underline{28,168^{\text{lbs}}}$

TOTAL MOMENT ABOUT A

$480^{\text{lbs}} \times 26.67' + 4800^{\text{lbs}} \times \frac{25'}{2} + 25,500^{\text{lbs}} \times \frac{25'}{3} - 2612^{\text{lbs}} \times \frac{8'}{3} = \underline{278.3^{\text{ft-k}}}$

MOMENT AT BOTTOM OF WALL

$M_u = 1.6 \times \left[480^{\text{lbs}} \times 21.67' + 3840^{\text{lbs}} \times \frac{20'}{2} + 1632\text{ plf} \times 1/2 \times 20' \times \frac{20'}{3}\right] = \underline{252.2^{\text{ft-k}}}$

↰ LOAD FACTOR

DATE
SHEET NO. 3
PROJECT NAME RED RIVER DIVERSION
PROJECT NUMBER 34/09-1004
SUBJECT Rush & Lower Rush River Diversion

COMPUTED	CHECKED	SUBMITTED
BY JGT	BY	TO
DATE 7-22-10	DATE	DATE

ASSUME: 3' THICK WALL, d = 32" ⇒ A_s REQD = 1.83 in^2 SEE ATTACHED SHEET
#11 @ 9" = 2.08 in^2

3'-6" THICK WALL, d = 38" ⇒ A_s REQD = 1.52 in^2
#11 @ 12" = 1.56 in^2

$$\underset{(3'-6")}{\Delta_g} \approx \frac{(480 + 3840 + 16{,}320) \times (20')^3 \times (12^{in}/_{ft})^3}{15 \times (57000 \times \sqrt{4000}) \times (1/12 \times 12" \times 42^3)} = 0.07"$$

OK SMALL - ASSUME OK FOR PARTIALLY CRACKED SECTION ALSO

$$\underset{(3'-0")}{\Delta_g} = 0.113"$$

ALSO OK - SMALL

* USE 3' THICK WALL

CHECK OVERTURNING RESISTANCE MOMENT

SEE SPREADSHEET

SHEET #4 34/09-1004

Flexural Concrete Design (As unknown)

Mu	252.2 [ft-k/ft]
d	32 [in]

If f'c=3000psi, fy=60000psi

Asreq'd	1.855798 [in^2/ft]
As (min)	1.28 [in^2/ft]

If f'c=3000psi, fy=40000psi

Asreq'd	2.784408 [in^2/ft]
As (min)	1.28 [in^2/ft]

If f'c=4000psi, fy=60000psi

Asreq'd	1.828861 [in^2/ft] *
As (min)	1.28 [in^2/ft]

If f'c=4000psi, fy=40000psi

Asreq'd	2.770602 [in^2/ft]
As (min)	1.28 [in^2/ft]

If f'c=2500psi, fy=60000psi

Asreq'd	1.892169 [in^2/ft]
As (min)	2.522892 [in^2/ft]

If f'c=2500psi, fy=40000psi

Asreq'd	2.832228 [in^2/ft]
As (min)	1.28 [in^2/ft]

Flexural Concrete Design (Mu cap)

As	1.56 [in^2/ft]
d	42 [in]

If f'c=3000psi, fy=60000psi

Mu	215.2273 [in^2/ft]	(uses 3/4 A
	284.2836	(uses full A

If f'c=3000psi, fy=40000psi

Mu	144.7917 [in^2/ft]	(uses 3/4 A
	191.8623 [in^2/ft]	(uses full A

If f'c=4000psi, fy=60000psi

Mu	216.5257 [in^2/ft]	(uses 3/4 A
	286.6879 [in^2/ft]	(uses full A

If f'c=4000psi, fy=40000psi

Mu	143.9438 [in^2/ft]	(uses 3/4 A
	191.0479 [in^2/ft]	(uses full A

If f'c=2500psi, fy=60000psi

Mu	212.7602 [in^2/ft]	(uses 3/4 A
	280.4804 [in^2/ft]	(uses full A

If f'c=2500psi, fy=40000psi

Mu	143.7385 [in^2/ft]	(uses 3/4 A
	190.2231 [in^2/ft]	(uses full A

IGNORE

* THIS CALC JUSTIFIES #11@9" FOR 252,2 ft-k & 36" WIDE WALL AT WORST CASE WALL MOMENT.

NOTE: REBAR QUANTITY TOTALS ASSUME #11@12" OVER ENTIRE WALL AS THIS CRITICAL MOMENT LIKELY ONLY OCCURS IN SMALL REGION

JGT 7-22-10

RED RIVER DIVERSION
34/09-1004
RUSH & LOWER RUSH DIVERSION
SHEET #5

WALL RESISTANCE AGAINST OVERTURNING

	LOAD CASE 1 RETAINING SOIL	LOAD CASE 2 RETAINING WATER	LC3 Drained
width ftg	22 ft	22 ft	22 ft
thickness ftg	5 ft	5 ft	5 ft
wall ht	25 ft	25 ft	25 ft
wall thickness	3 ft	3 ft	3 ft
location of wall from pt A	8 ft	14 ft	8 ft
Mr	837.5156 ft-kips	689.3906	837 ft-kips
Moverturn	278.3 ft-kips	231.5	165.5347 ft-kips
safety factor against OT	3.009399	2.977929	5.056343

BEARING PRESSURE

Net Moment	LC1	LC2
About center of ftg	141.3469 ft-kips	119.165 ft-kips
e	2.219382 ft	3.145025 ft
B/6	3.666667 ft	3.666667 ft
Q	63.6875 kips	37.89 ft
q max	4.64712 ksf	3.199525 ksf
q min	1.142652 ksf	0.245021 ksf
q norm	2.894886 ksf	1.722273 ksf

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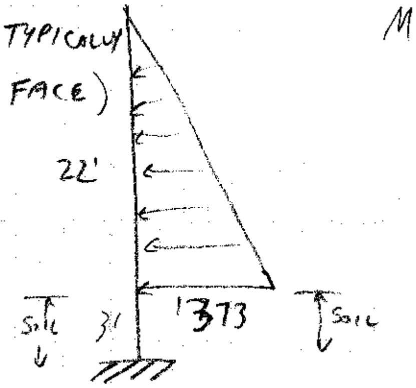
DATE
SHEET NO. 6
PROJECT NAME RED RIVER DIVERSION
PROJECT NUMBER 34/09-1004
SUBJECT RUSH & LOWER RUSH DIVERSION

COMPUTED	CHECKED	SUBMITTED
BY JGT	BY	TO
DATE 7/22/10	DATE	DATE

LC2

CHECK MOMENT UNDER WATER

(NOT REAL REALISTIC CASE
AS SOIL IS TYPICALLY
ON OPPOSITE FACE)



LOAD FACTOR

$M_u = 1.6 \times 10.33' \times 15.1^k = 249.5^{ft\text{-}k}$

$< 252.2^{ft\text{-}k}$ ok

Doesn't Control WALL DESIGN

$M_A = 15.1^k \times 15.33' = 231.5^{ft\text{-}k}$

$< 278.3^{ft\text{-}k}$

BARR

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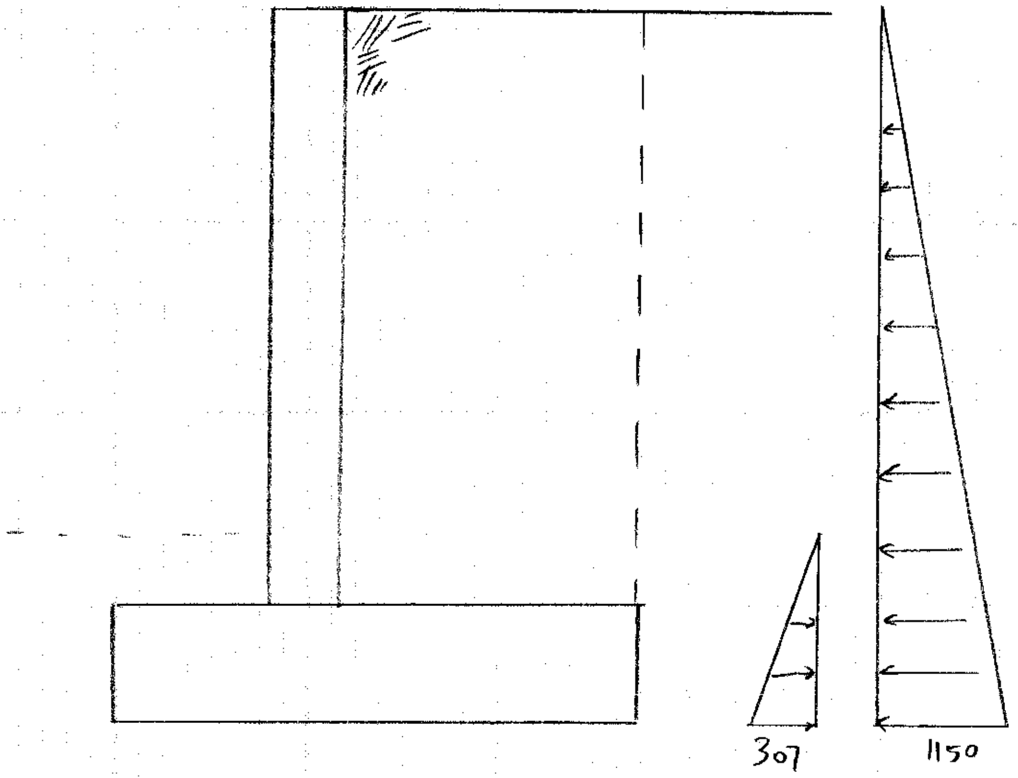
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

DATE
SHEET NO. 7
PROJECT NAME RED RIVER DIVERSION
PROJECT NUMBER 34/09-1004
SUBJECT RUSH & LOWER RUSH RIVER DIVERSIONS

COMPUTED	CHECKED	SUBMITTED
BY JGT	BY	TO
DATE 7/22/10	DATE	DATE

LC3 - DRAINED - NO H_2O TABLE



$$M_{ot} = \frac{1}{2} \times 1150 \times 30 \times \frac{30}{3} - \frac{1}{2} \times 307 \times 8 \times \frac{8}{3} = 169.2^{ft\text{-}k}$$

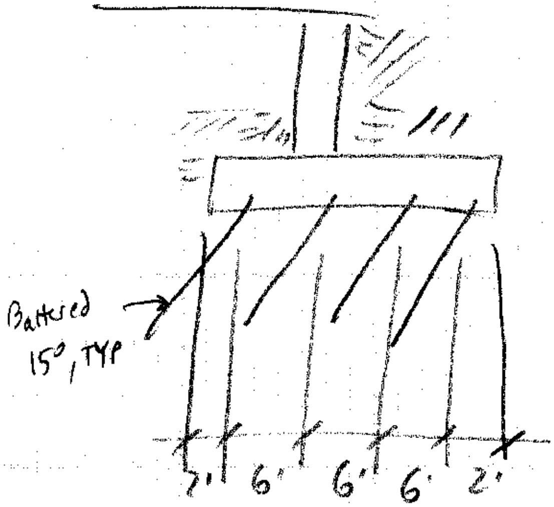
$$M_r = .15 \times 5' \times 22' \times 11' + .15 \times 3' \times 25' \times 8' + .115 \times 12.5' \times 25' \times 15.75' = 837^{ft\text{-}k}$$

BARR MINNEAPOLIS, MINNESOTA - HIBBING, MINNESOTA
DULUTH, MINNESOTA
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

COMPUTED	CHECKED	SUBMITTED	DATE
BY JGT	BY	TO	SHEET NO. 8
DATE 7-22-10	DATE	DATE	PROJECT NAME RED RIVER DIVERSION
			PROJECT NUMBER 34/09-1004
			SUBJECT RUSH & LOWER RUSH RIVER DIVERSIONS

LC 1 PILING

USE SPACING TO NOT REQUIRE PILING STRENGTH REDUCTIONS [6' > 5×14/12 = 5.83']
@ 6' LONGITUDINAL SPACING



(@ 6' SPACING)

Horiz. $4 \times [(237.9 \times \sin 15) + 35]$

$= 386.3^{k}$

REQ'D $= 28.2^{k} \times 6' = 169.2^{k}$

F.S. > 2 ok

VERT $4 \times 237.9 \times \cos 15 = 919^{k}$

$2.89^{ksf} \times 6' \times 22' = 382^{k}$

Ave bearing   F.S. = 2.4

MAX ⇒ 4.65^{ksf}

$6' \times 5' \times \left(4.647 - \left(\frac{2.5}{22} \times (4.647 - 1.143)\right)\right) = 127.5^{k}$

$1 \times 237.9 \times \cos 15 = 230$

F.S. = 1.8

FOR ESTIMATING PURPOSES, ASSUME OK - THIS IS WORST CASE LOAD CONDITION, PILE ARRANGEMENT COULD POSSIBLY BE ADJUSTED LOCALLY IF FOUND UNACCEPTABLE IN FINAL CALCS.

BARR	MINNEAPOLIS, MINNESOTA - HIBBING, MINNESOTA DULUTH, MINNESOTA ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI	DATE	SHEET NO. 9
		PROJECT NAME RED RIVER DIVERSION	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER 34/09-1004
BY JGT	BY	TO	SUBJECT
DATE 7-22-10	DATE	DATE	RUSH & LOWER RUSH RIVER DIVERSIONS

LC 2

$$\text{HORIZ} - 4 \times (216.6 \times \sin 15 + \overset{\text{UPLIFT}}{\overset{\downarrow}{35}}) = 364.2^{k}$$

$$\text{Force} = \tfrac{1}{2} \times 1373 \times 22 = 15.1^{k} \times 6' = 90.6^{k}$$

OK

$$\text{VERT} - 4 \times 216.6 \times \cos 15 = 837^{k} > 22' \times 1.72^{\text{KSF}} = 37.8^{k}$$

$$\times 6 = 227^{k}$$

ok

VERT MAX - SAY OK AT THIS POINT OF ESTIMATE
SEE LC 1

H:\RLG\WNHOUSE\COMPPADS.CDR

REV.12/1/98

DATE
SHEET NO. 10
PROJECT NAME RED RIVER DIVERSION
PROJECT NUMBER 34/09-1004
SUBJECT RUSH & LOWER RUSH RIVER DIVERSIONS

COMPUTED	CHECKED	SUBMITTED
BY JGT	BY	TO
DATE 7-22-10	DATE	DATE

LC 3

Horiz Force = $1/2\ (1/3 \times 115\ pcf \times 30') \times 30' = 17.25^k$

$17.25^k \times 6'$ PILE SPACING = 103.5^k

PILE RESISTANCE = $4 \times 188^k \times \sin 15 + 4 \times 22 = 282.6^k$

F.S. > 2 ok

VERT

$63.69^k \times 6' = 382.1^k$

4 PILES × $188^k \times \cos 15 = 726.4^k$

F.S. = 1.90 > 1.5 ok

COMPUTED	CHECKED	SUBMITTED
BY JGT	BY	TO
DATE 7-22-10	DATE	DATE

DATE

SHEET NO. 11

PROJECT NAME RED RIVER DIVERSION

PROJECT NUMBER 34/09-1004

SUBJECT
RUSH & LOWER RUSH RIVER DIVERSIONS

STEPS (LOWER RUSH)

$341.9\ ft^2 \times 150 pcf = 51.3\ {}^{k}/_{ft}$ of width

20 ft of WATER $= 43.1\ {}^{k}/_{ft}$ of with

$94.4\ {}^{k}/_{ft}$ of width

ASSUME 6' SPACING

$94.4 \times 6 = 566.4^{k}$ PER ROW

$314 \times 6 = 1884$

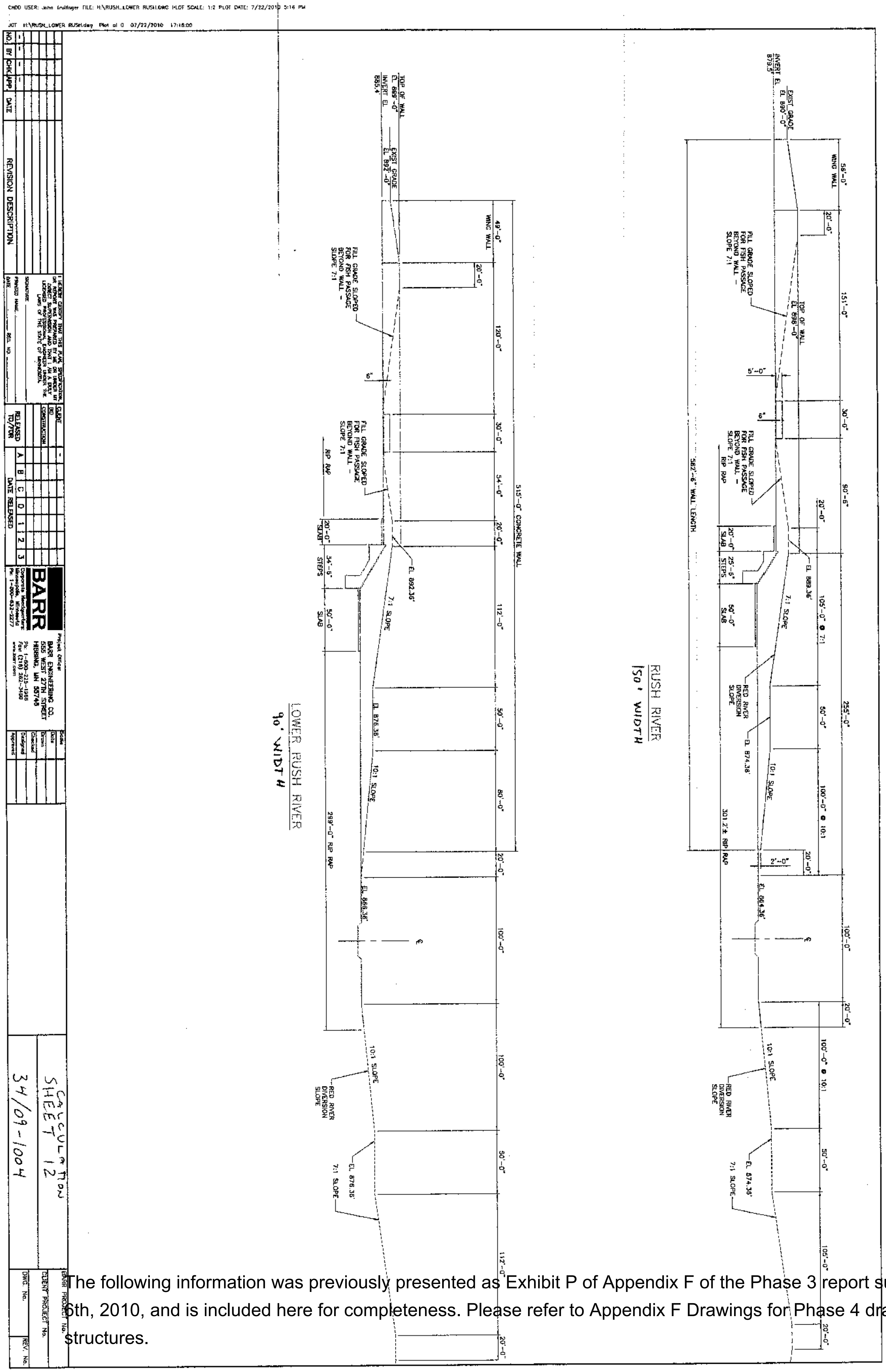
$237.9 \times 6 = 1427 > 566.4 \times \underset{\uparrow}{2} = 1132$

F. of Safety

USE 6 PILES PER STEPS
@ 6' O.C. SPACING

WALL FTG

LEAVE AT 5' THICK FROM PREVIOUS ESTIMATE TO ACT AS PILE CAP. MOMENT AT THIS THICKNESS SHOULD BE OK.



The following information was previously presented as Exhibit P of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness. Please refer to Appendix F Drawings for Phase 4 drawings of these structures.

Sheet 13

Background

This section is applicable for the following structures as part of the Red River Diversion:

1) ND Concrete Structure for Diversion and Termination of Rush River
2) ND Concrete Structure for Diversion and Termination of Lower Rush River

The Rush and Lower Rush Rivers will not continue to the actual Red River under the plan being analyzed in this computation. They will terminate into the Red River Diversion channel, and the Red River Diversion channel will then terminate in the Red River downstream of the Fargo / Moorehead area.

This calculation is used in order to compute quantities to be used as part of a budgetary estimate for the project. This computation is not for final design.

These two structures are very similar, so are analyzed in tandem. Both structures consist of retaining walls on each side of the river. These retaining walls will confine the flow of the river with a top of wall elevation 3 feet above the elevation of a 500 year flood for each river. Once these walls hit the sloping sides of the Red River Diversion, the top of wall is dropped to follow the slope of the diversion sides. At the sloping sides, river flow is required to pool up and pass over and down a set of steps. Prior to running into the Red River Diversion, the north wall has a 30 feet wide x 5 feet high "fish passage" leading to a man-made stream to travel down to the bottom of the Red River Diversion on a much more gradual path.# The "fish passage" has a lower invert elevation than the top step, so will be the usual flow path in the system with the steps providing flood relief.

The soil conditions along the entire project (see Appendix F3.0), are underlain by "soft" clay, susceptible to large settlements. Therefore, piles will be used at the concrete wall and concrete step structures.

Load Cases used for the Rush & Lower Rush River Structures

1. The primary load case for retaining walls is generally the case where the retained soil pushes on the wall with nothing on the opposite side. In this case, soil on one side with no water on the other side. Under load case 1, the water table in the soil was assumed at 5 feet below top of wall to produce an increased horizontal force in the retained soil. This is considered a usual load case, and uses undrained pile values.
2. Since the fish passage has no retained soil behind it, this load case looked at 22 feet of water on the river side of the wall with no retained soil on the back side of the wall. Even though unrealistic to impossible, the calculations for this load case assume this to be a usual load case, using undrained pile values.
3. The third load case is the same as load case 1 with the exception of no water table in the soil. This load case uses the drained soil pile values, so it was assumed this load case would occur in a dry river and thus be considered unusual.

For the retaining walls, the largest retained soil height for either river is 22 feet above final grade. Wall thicknesses, footing size, and piling design were computed to satisfy 22 feet retained height. Conservatively, this same concrete thicknesses and pile requirements were continued throughout the design. Footings for walls were set at 5 feet thick to act as a pile cap. Foundation bearing pressures were computed and piles were placed to withstand the foundation pressures. Piles were computed to require battering to withstand the horizontal

#The design changed from Phase 3 to Phase 4. For Phase 4 this sentence should read: Prior to running into the Red River Diversion, the north wall has a 40 foot wide "fish passage" leading to a series of pools and riffles to travel down to the bottom of the Red River Diversion on a more gradual path sliding force of the wall. Bottom of wall footings were placed 8 feet below grade for frost protection.

The step weight was also supported by piles. Slabs were provided for 20 feet leading into the steps and 50 feet leaving the steps. No piles were assumed necessary for these steps.

Reinforcing bar quantities were roughly estimated considering 12" bar spacing in both faces each way. Vertical bars in wall sections were assumed to weigh 5.3 pounds per foot of bar length in tall wall sections, 4 pounds per foot in medium height wall sections, and 3 pounds per foot in short wall sections. Horizontal bars were estimated at 3 pounds per foot. Footing bars were estimated at 4 pounds per foot each direction. The slabs on grade were estimated at 3 pounds per foot in each direction. The steps were computed at 4 pounds per foot based on the concrete perimeter distance.

This calculation assumes that concrete would be 4000 psi concrete compressive strength with 60,000 psi reinforcing bar yield strength.

Sheet 15

Job Number: 34-091-004.300.700

Barr Engineering Company

Project: Fargo-Moorhead Metro Flood Risk Management Project

Subject: Phase 3 Cost Estimate

Task: Cost Estimate - Quantities - ND East 35K Erosion Control and Diversion-Wide

Status: *draft*

Computed: JGT

Date: 7/23/2010

Checked:

Date:

Sheet:

of

Hydraulic Structure - Lower Rush - Structure						
	Earthwork - Structural	Earthwork - Structural Excavation	0	BCY		
		Earthwork - Structural Haul	0	BCY		
		Earthwork - Structural Levee Embankment	0	BCY		
	Drop Structure and Walls		0	LF	number of piles	0
	Steel Piles	Steel Piles - Furnish Material	53,701	LF		
		Steel Piles - Set up Rig	0	EA		
		Steel Piles - Unload Piles	99	EA	pilesx73 / 40000	
		Steel Piles - Weld Splices	837	EA	cutoffsx.25	
		Steel Piles - Drive Piling	53,701	LF		
		Steel Piles - Cut Offs	837	EA	no. of piles	
		Steel Piles - Pile Test	1	EA		
	Concrete - Footing	Concrete - Footing - Form Strip and Cure	10,300	SF		
		Concrete - Footing - Pour	4,196	CY		
		Concrete - Footing - 5% Waste	210	CY		
	Concrete - Walls	Concrete - Walls - Build Forms	32,329	SF		
		Concrete - Walls - Assemble and Modify Forms	4	EA		
		Concrete - Walls - Form, Strip and Cure	32,329	SF		
		Concrete - Walls - Pour	1,779	CY		
		Concrete - Walls, Piers and Elevated Deck - 5% Waste	89	CY		
	Concrete - Steps	Concrete - Steps - Build Forms	5,140	SF		
		Concrete - Steps - Assemble and Modify Forms	25	EA		
		Concrete - Steps - Form, Strip and Cure	5,140	SF		
		Concrete - Steps - Pour	1,161	CY		
		Concrete - Steps - 5% Waste	58	CY		
	Concrete - Slabs on Grade	Concrete - Slab - Build Forms	270	SF		
		Concrete - Slab - Assemble and Modify Forms	2	EA		
		Concrete - Slab - Form, Strip and Cure	270	SF		
		Concrete - Slab - Pour	608	CY		
		Concrete - Slab - 5% Waste	30	CY		
	Concrete - Elevated Deck	Concrete - Elevated Deck - Form Strip and Cure	0	SF		
		Concrete - Elevated Deck - Pour	0	CY		
		Concrete - Walls, Piers and Elevated Deck - 5% Waste	0	CY		
	Steel Reinforcement Bars	Steel Reinforcement Bars - Furnish and Install	665,000	LB		
		Steel Reinforcement Bars - Hoist	665,000	LB		
	Hydraulic Gates - Steel	Hydraulic Gates - Steel - Furnish	0	SF		
		Hydraulic Gates - Steel - Install	0	EA		
		Hydraulic Gate Hoist - Furnish and Install	0	EA		
	Bulkheads - Steel	Bulkheads - Steel - Furnish	0	LS		
	Bridge Railing	Bridge Railing - Furnish and Install	0	LF		
	Sheet Piling	Sheet Piling - Furnish	0	SF		
		Sheet Piling - Unload Sheeting	0	EA		
		Sheet Piling - Set up Rig	0	EA		
		Sheet Piling - Drive Sheets	0	EA		
		Sheet Piling - Cut Sheets	0	EA		
	Structural Aggregate Backfill	Aggregate Base - Class V - Furnish and Install	#REF!	ECY		
Hydraulic Structure - Rush - Structure						
	Earthwork - Structural	Earthwork - Structural Excavation	0	BCY		
		Earthwork - Structural Haul	0	BCY		
		Earthwork - Structural Levee Embankment	0	BCY		
	Drop Structure and Walls		0	LF	number of piles	0
	Steel Piles	Steel Piles - Furnish Material	56,599	LF		
		Steel Piles - Set up Rig	0	EA		
		Steel Piles - Unload Piles	104	EA	pilesx73 / 40000	
		Steel Piles - Weld Splices	850	EA	cutoffsx.25	
		Steel Piles - Drive Piling	56,599	LF		
		Steel Piles - Cut Offs	900	EA	no. of piles	
		Steel Piles - Pile Test	1	EA		

Sheet 16

			Concrete - Footing	Concrete - Footing - Form Strip and Cure	11,250	SF		
				Concrete - Footing - Pour	4,583	CY		
				Concrete - Footing - 5% Waste	229	CY		
			Concrete - Walls	Concrete - Walls - Build Forms	41,359	SF		
				Concrete - Walls - Assemble and Modify Forms	4	EA		
				Concrete - Walls - Form, Strip and Cure	41,359	SF		
				Concrete - Walls - Pour	2,281	CY		
				Concrete - Walls, Piers and Elevated Deck - 5% Waste	114	CY		
			Concrete - Steps	Concrete - Steps - Build Forms	7,710	SF		
				Concrete - Steps - Assemble and Modify Forms	17	EA		
				Concrete - Steps - Form, Strip and Cure	7,710	SF		
				Concrete - Steps - Pour	1,537	CY		
				Concrete - Steps - 5% Waste	77	CY		
			Concrete - Slabs on Grade	Concrete - Slab - Build Forms	900	SF		
				Concrete - Slab - Assemble and Modify Forms	2	EA		
				Concrete - Slab - Form, Strip and Cure	900	SF		
				Concrete - Slab - Pour	608	CY		
				Concrete - Slab - 5% Waste	30	CY		
			Concrete - Elevated Deck	Concrete - Elevated Deck - Form Strip and Cure	0	SF		
				Concrete - Elevated Deck - Pour	0	CY		
				Concrete - Walls, Piers and Elevated Deck - 5% Waste	0	CY		
			Steel Reinforcement Bars	Steel Reinforcement Bars - Furnish and Install	831,800	LB		
				Steel Reinforcement Bars - Hoist	831,800	LB		
			Hydraulic Gates - Steel	Hydraulic Gates - Steel - Furnish	0	SF		
				Hydraulic Gates - Steel - Install	0	EA		
				Hydraulic Gate Hoist - Furnish and Install	0	EA		
			Bulkheads - Steel	Bulkheads - Steel - Furnish	0	LS		
			Bridge Railing	Bridge Railing - Furnish and Install	0	LF		
			Sheet Piling	Sheet Piling - Furnish	0	SF		
				Sheet Piling - Unload Sheeting	0	EA		
				Sheet Piling - Set up Rig	0	EA		
				Sheet Piling - Drive Sheets	0	EA		
				Sheet Piling - Cut Sheets	0	EA		
			Structural Aggregate Backfill	Aggregate Base - Class V - Furnish and Install	#REF!	ECY		

WALLS (Both Sides) PHASE 4

Lower Rush River

Panel	Stem			Ftg			Concrete Volume		Steel	Piles (Both sides)			
	W	Ht	L	B	Toe	L	Ftg	Stem	Reinf	No.	No. rows	Length	Total
		ft			ft		cy	cy	lbs			ft	ft
A	3	13.1	144	22	6	144	587	210	104,903	96	24	74	7104
B	3	16.5	792	22	6	832	3390	1452	639,613	560	140	74	41440
C	3	16	170	22	6	170	693	302	132,521	112	28	55	6160
D	2.5	11	30	22	6	30	122	31	20,746	24	6	55	1320
E	2.5	8	84	22	6	84	342	62	53,654	28	7	55	1540
						Totals:	5133	2057	951,437	820			57,564

WALLS (Both Sides) PHASE 4

Rush River

Panel	Stem			Ftg			Concrete Volume		Steel	Piles (Both Sides)			
	W	Ht	L	B	T	L	Ftg	Stem	Reinf	No.	No. rows	Length	Total
		ft			ft		cy	cy	lbs			ft	ft
A	3	17.5	164	22	6	164	668	319	132,173	112	28	71	7952
B	3	21.5	778	22	6	818	3333	1859	697,041	544	136	71	38624
C	3	16.7	166	22	6	166	676	308	131,448	112	28	55.5	6216
D	2.5	11	30	22	6	30	122	31	20,746	24	6	55.5	1332
E	2.5	8	84	22	6	84	342	62	53,654	28	7	55.5	1554
						Totals:	5141	2578	1,035,061	820			55,678

Note: For Stem Panel B, The Length deducts a 40 ft wide fish bypass on one wall only.
Footing is 5 ft thick.
Vertical stem bars dowels included by adding 11' to vertical bars. (4 feet into footing + 2 ft hook and 5 ft lap)
Horizontal stem bars assume a 5 ft lap every 30 ft.
Footing longitudinal barrs assume 5 ft laps every 30 ft.
Vertical Stem rebar assumed to weigh 3, 4 or 5.3 plf at 12" oc
Horizontal Stem rebar assumed to weigh 3 plf at 12" oc.
Footing rebar is assumed to weigh 4 plf each way at 12" oc (Top and bottom bar layers).

DROP STRUCTURE PHASE 4

Lower Rush River

Concrete Volumes			Steel	Piles			
Approach	Stilling	Steps	Reinf	No	No	Length	Total
cy	cy	cy	lbs		Rows	ft	ft
70	170	517	78523	20	10	52	1040
				20	10	60	1200
			Total:	40			2240

DROP STRUCTURE PHASE 4

Rush River

Concrete Volumes			Steel	Piles			
Approach	Stilling	Steps	Reinf	No	No	Length	Total
cy	cy	cy	lbs		Rows	ft	ft
117	283	794	118720	34	17	45.5	1547
				17	17	56	952
			Total:	51			2499

Notes: Horizontal rebars assume a 4 ft lap every 30 ft.
Total Slab rebar assumed to weigh 3 plf at 12" oc.
Step rebar assumed to weigh 4 plf at 12" oc based on the total perimeter.
10% added to Step rebar to account for laps.