VEN TE CHOW HYDROSYSTEMS LAB
Technical Memorandum

CONVEYANCE ANALYSIS OF THE MAINLINE TUNNEL USING ICAP

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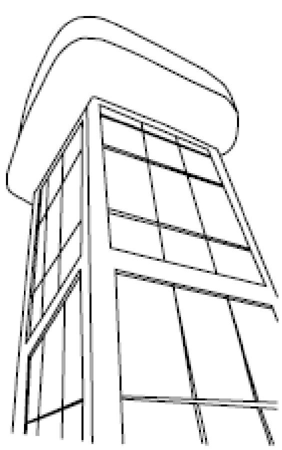

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MEMORANDUM

Introduction

The Faer Havens metropolitan area is primarily a combined sewer system consisting of local sewers, interceptor sewers, and the Reservoirs and Tunnel system (RAT). RAT incorporates many miles of tunnels in addition to multiple reservoirs and was designed to capture and store combined sewer overflows that would otherwise contaminate area waterways. The individual tunnels that are included in RAT are as large as 33 feet in diameter and dozens of miles long. This report summarizes the results of a conveyance analysis on the Mainline tunnel, a reach of RAT approximately 110,000 feet long. The analysis was performed using the Illinois Conveyance Analysis Program (ICAP), and its results were then validated using hand computations and a simple 1D model.

A conveyance system consists of any natural or man-made components of a storm drainage system that transport surface and storm water runoff. These systems contain curbs and gutters, inlets, storm drains, catch basins, channels, ditches, pipes and culverts (Central Oregon Intergovernmental Council, 2010). A conveyance analysis measures the overall efficiency a system has in transporting water, and serves to characterize and understand the conveyance of a system. The conveyance capacity of a system is the largest discharge that it can convey for a given set of boundary conditions without overflow at any point in the system. In modeling and understanding the Mainline RAT system, it is a useful exercise to compute and understand the conveyance of the various tunnels in RAT. The work described in this memo applies the Illinois Conveyance Analysis Program to determine the conveyance capacity of the Mainline Tunnel, at six different locations within the tunnel.

Many approaches exist to perform conveyance studies for sewer systems. Usually, one approach is to discretize the sewer into reaches delimited by manholes, ignoring the water-surface profile within a pipe. Another approach is to perform backwater calculations on spatial steps for every flow condition in the system. Due to the large range of flows and boundary conditions in the Mainline Tunnel, this approach would entail a very large number of combinations of inflow and boundary conditions, as well as mixed flow conditions (Oberg, 2015).

The Illinois Conveyance Analysis Program (ICAP) was written for the Sanitary-Enviornment and Treatment District of Faer Havens (SEAT) and uses hydraulic performance graphs (HPGs) to determine conveyance in a system. HPGs have been proven to be effective in open-channel capacity determination (Yen, 1999), and ICAP extends the HPG approach to describe pressurized flow in sewers (Oberg, 2016).

Development

The range of the system analyzed consists of approximately 21 miles of tunnel from the North Springs confluence to the Deyoung reservoir (Figure 1). The tunnel has a diameter of 33 ft. for the first 47,000 ft. from the reservoir, and a diameter of 30 ft. for the remaining 63,000 ft. until the North Springs confluence. Since the analysis aims to determine maximum conveyance, this work considers discharge entering through only one input to develop flow rating curves, and will be tested for several input locations in the Mainline Tunnel. These locations are the North Springs (110,000 ft. from the reservoir), DROP-77, Central, FHPS, DROP-15, and DROP-13A. In the upstream end, the boundary conditions for the input locations were selected as heads of -50 ft. and -30 ft., respectively. These were chosen to represent the lowest connecting structure elevation and the rim of Deyoung, respectively. At the downstream end (e.g. Deyoung reservoir), maximum discharge is determined when the water surface elevation is at critical depth. The flow rating curve is developed by determining discharge into the reservoir assuming the upstream conditions are held constant but varying the downstream water surface elevation.

Near the reservoir, the system contains a 45° wye for which headloss must be accounted for (Figure 2). Headloss for this wye section was computed using the following equation:

$$h_m = \sum K \frac{v^2}{2g}$$

where K is a headloss coefficient (Jensen; Appendix A).

Summary of Results

Appendix B contains the different results computed by ICAP for the various confluences in the system. The input parameters included upstream head and downstream head and the output was the discharge. These results were used to create the flow rating curves. Table 1 shows a summary of these results, displaying the maximum flow between each confluence and Deyoung, for upstream head conditions of -50 ft. and -30 ft. The maximum discharge for every input location is obtained when the downstream head is -240 ft. or lower.

Table 1: Summary of Results

Node	Flow (cfs)	
	Upstream head: -50 ft.	Upstream head: -30 ft.
DROP-13A	59497	62714
DROP-15	27557	29044
FHPS	20341	21442
Central	15749	16602
DROP-77	13334	14056
North Springs	11888	12532

Figure 3 shows the final flow rating curves developed by ICAP for the Mainline Tunnel, with an upstream head of -50 ft., and taking into consideration the following confluences: North Springs, DROP-77, Central, FHPS, DROP-15, and DROP-13A. The conveyance curves for the Mainline Tunnel show the relationship between the head at Deyoung, and the discharge at Deyoung. As seen from the graph, each curve reaches a maximum discharge after which lowering the Deyoung head will not have any additional effect. Figure 4 shows the final flow rating curves for an upstream head of -30 ft. For an upstream head of -50 ft. and an empty Deyoung reservoir, the maximum possible flow between the North Springs confluence and Deyoung is 11,888 cfs.

Validation of Results

Background

A backwater computation was made to aid and corroborate the results of the analysis for the Mainline Tunnel. The purpose of this computation was to determine the maximum possible flow into the reservoir, located at the downstream end of the tunnel, assuming all flow enters the tunnel through the North Springs confluence. To perform this computation, geometry information for the tunnel, as well as hydraulic results from previous model runs, were simplified and used. The following is the data used in the computation. Refer to Figure 1 for the interpretation of the data:

Invert at DS end (STA 0): -263 ft
Invert at US end (STA 110,000): -236 ft
Diameter from STA 0 to STA 47000: 33 ft
Diameter from STA 47000 to STA 110000: 30 ft
Water depth at DS end: critical depth, -230 ft
Head at US end: -50 ft
Assumed Manning's roughness (n): 0.015

Following the given information, as well as the tunnel's profile, the remaining geometrical information was computed and/or assumed, and will be specified in this section.

Hand Computations using MS Excel and a Calculator

Since there is sufficient geometry and hydraulic information available, hand computations aided by a calculator or a spreadsheet could be made. This process was done by constructing a theoretical "open channel" using the Step Method (Henderson, 1966). This open channel will have dimensions large enough to fit a circular "culvert" (tunnel). The step method consists of calculating the head loss of different sections within the channel through fiction loss, energy equation and iteration.

The following considerations were made in order to develop a base rectangular "open channel" in which culverts representing the tunnel will be incorporated: The channel will have three cross sections: The first located at the downstream end, the second at the point of diameter change, and third at the upstream end (at sta. 0, sta. 47,000, and sta. 110,000, respectively). The invert elevation at sta. 47,000 was taken to be -255 ft, from the profile plot. The channel width will be 50 ft, and its height will be equal to the surface elevation at each cross section. To represent the transitions at each cross section, a portion of a 50 ft long channel will be adopted at each cross section.

Now that an open channel has been developed, two circular culverts will be inserted between their respective cross-sections. These culverts will represent the tunnels themselves. The culverts will have diameters of 33 ft between cross sections 1 and 2, and 30 ft between 2 and 3. The head loss in the three 50 ft long channel segments will be estimated by Manning's formula. The loss at the culverts will be computed considering both inlet control, and friction (outlet control), and selecting the most disadvantageous head loss. The incorporation of the two culverts into the open channel model results in 6 cross sections that will represent the tunnels and the open "sections" between the tunnels (i.e. manholes).

First, inlet control was tested for the culverts. Inlet control limits were computed using the extrapolation of the Federal Highway Administration's culvert charts (USACE, 2010). For a faster computation, free software provided by the FHWA can be used, such as the Hydraulic Design Series Number 5, and a paper on Hydraulic Design of Highway Culverts (3^{rd} Ed) provides information on this calculator software (FHWA, 2012). This computation takes into account the diameter of the culvert (or tunnel) and the flow entering through it. For the 30-ft diameter pipe, the capacity seems to be only 8,234 cfs; however, when verified with Torrecelli's orifice equation, the capacity of the pipe without a head over its top is 11,424 cfs. The latter is more trustworthy for the result using the FHWA charts, since there is no assurance that the FHWA charts can be extrapolated, as they were. Similar computations were made for the 33 ft pipe and proved that minimum capacity of this pipe without head over its top is 14,497 cfs.

With the above results in hand, the model was set in a spreadsheet. The water stage was estimated by the Bernoulli equation, and was computed from the downstream end to the upstream end, as the accumulation of the losses of the channel segments at the "manholes" and the friction losses at the culverts. Examples of the spreadsheet tables used to compute head losses for inlet and outlet control are displayed in Appendices C and D. Appendix E contains the sheets used to calculate the "open channel" sections or manholes. Appendix F contains an example of the Step Method table used in these computations. The final maximum flow into the reservoir considering inflow at the North Brach only resulted in **11,568.4cfs**, after various iterations and revisions. This flow is very close to the maximum capacity of the culverts at inlet control; so the model can be used for representing the hydraulics of the analyzed system up to 11,500 cfs. Over this flow value, inlet control must be considered at least for the 30-ft diameter culvert.

Revision using a HEC-RAS model

In order to check the accuracy of this computation, a HEC-RAS model of the tunnel was developed to determine maximum flow. This method also includes iterations, as flows will be given as input, in order to reach a condition identical to the given data for this problem. The methodology is very similar to the hand calculations, and in fact uses some of the same formulas (see *HEC-RAS Hydraulic Reference Manual*). As one may know, the HEC-RAS model is for open channels and irregular cross sections, but can incorporate culverts between two cross sections. This convenient functionality is utilized to simulate the effect to the water surface elevations from flooded pipes. The model verifies the hydraulics to inlet control and outlet control (friction), and uses the most disadvantageous.

To begin, an open channel model is constructed using the software's Geometry Editor. This channel will have similar dimensions as the hand-made channel (width of 50 ft, height of ground elevation at given section). After constructing this open channel, two culverts will be inserted with dimensions identical to the tunnels we are analyzing. Figure 5 shows an example of the geometry editor and the constructed tunnels.

After constructing the tunnels, boundary and flow conditions were set for the model; these included the head of -50 ft at the upstream station, the water elevation of -230 ft at the downstream elevation. Several arbitrary flow values were assumed. The purpose of this model will be to run a backwater flow computation and revise the results of the water surface elevation at the upstream station. This elevation should be equal to -50 ft. When none of the given flows achieved this elevation, the flows were changed accordingly, and after several iterations and attempts, the correct flow was found. Figure 6 shows screenshots of the results tables and profile.

The final flow that achieved the desired head (-50 ft) at the upstream section was 11,568 cfs (which resulted in a head of -50.01 ft). After completing this step, several lower flows were evaluated to develop a rating curve for the water surface elevation at the reservoir (downstream end). Figure 7 shows the flow rating curve drawn by HEC-RAS for the North Springs confluence. This curve can be seen to be similar to the curves generated by ICAP.

Conclusion

In conclusion, the ICAP program predicts a maximum possible discharge of 11,888 from the North Springs confluence to Deyoung. This value was validated separately twice, via hand calculations and a HEC-RAS model, providing researchers with the assurance that the conveyance analysis values from ICAP are correct. ICAP was also shown to be a useful tool for understanding and characterizing the conveyance of Mainline tunnel.

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FIGURES

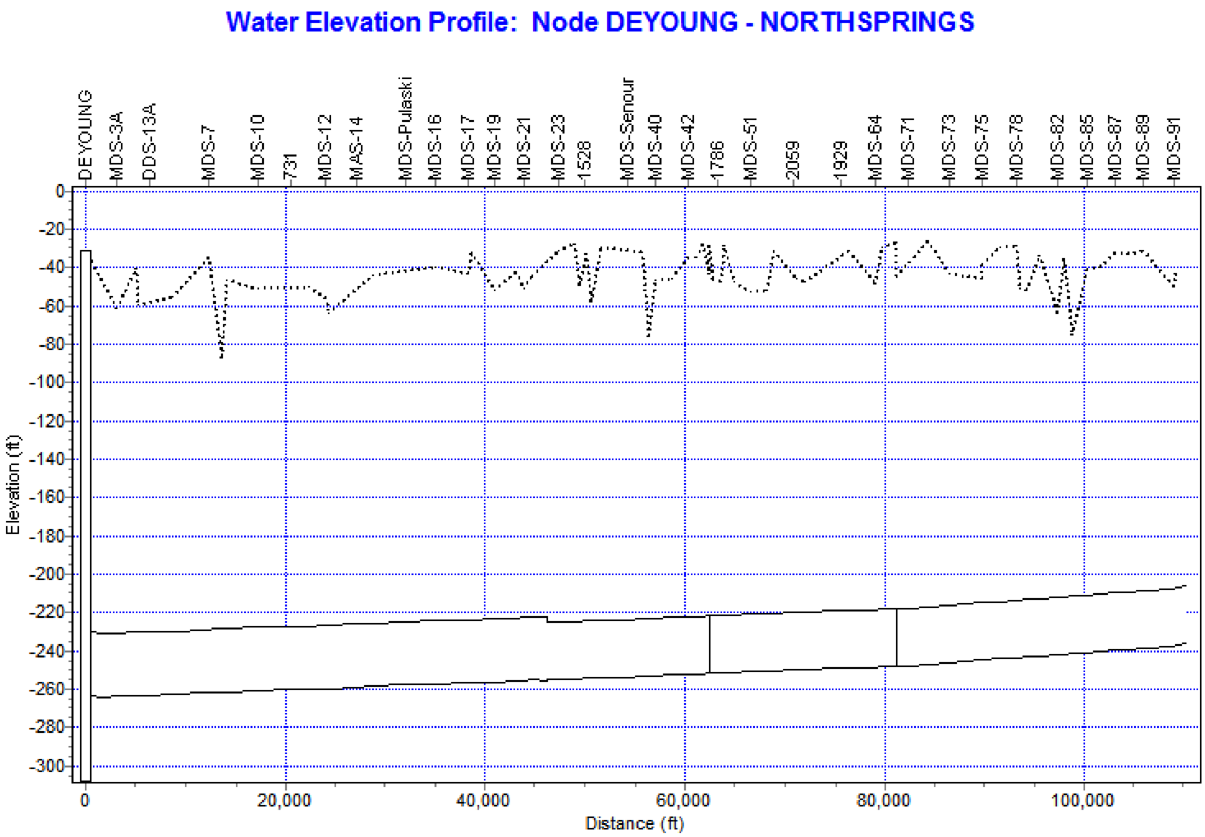


Figure 1: Mainline Tunnel Profile

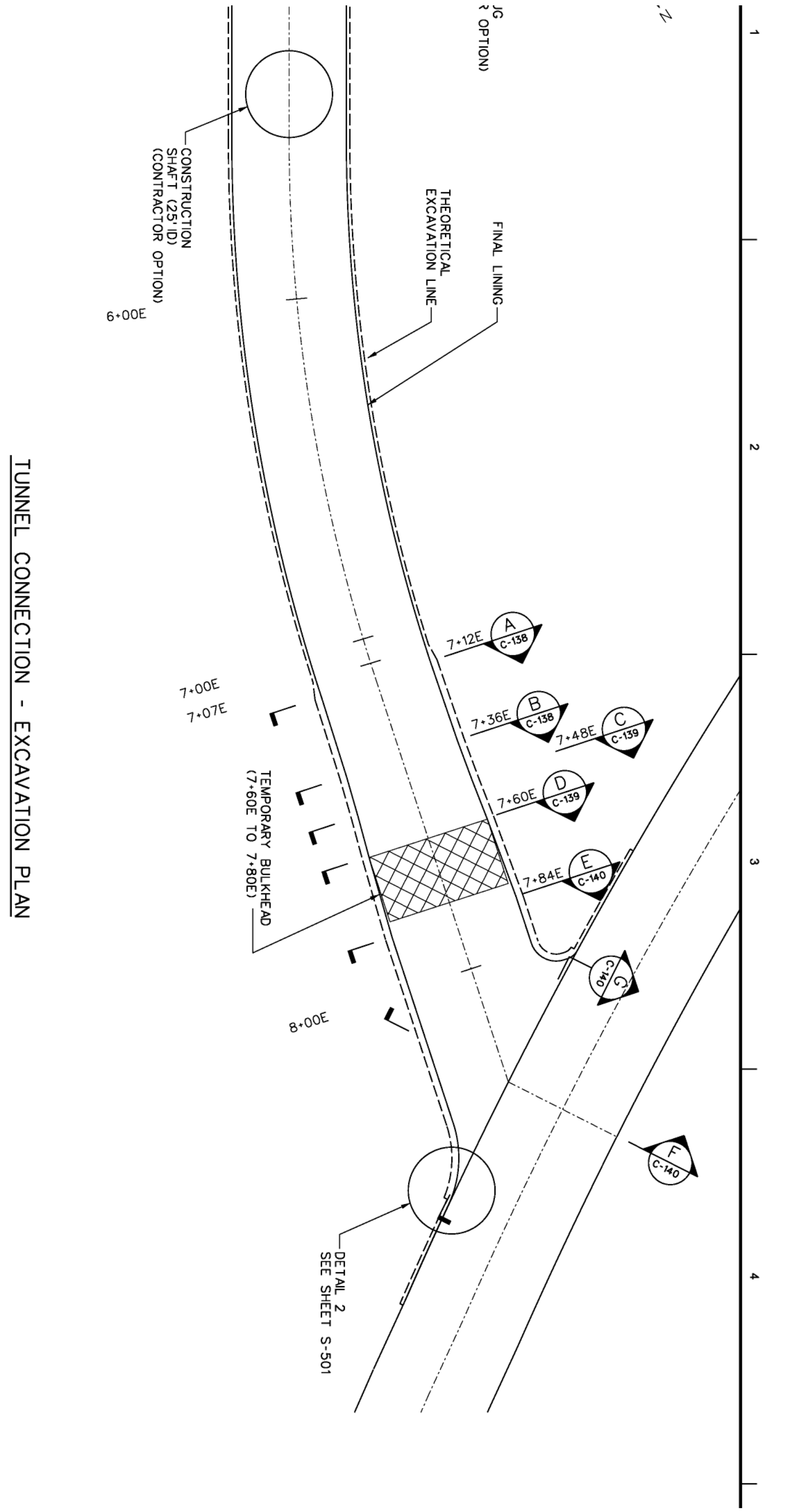


Figure 2: Wye, 45°

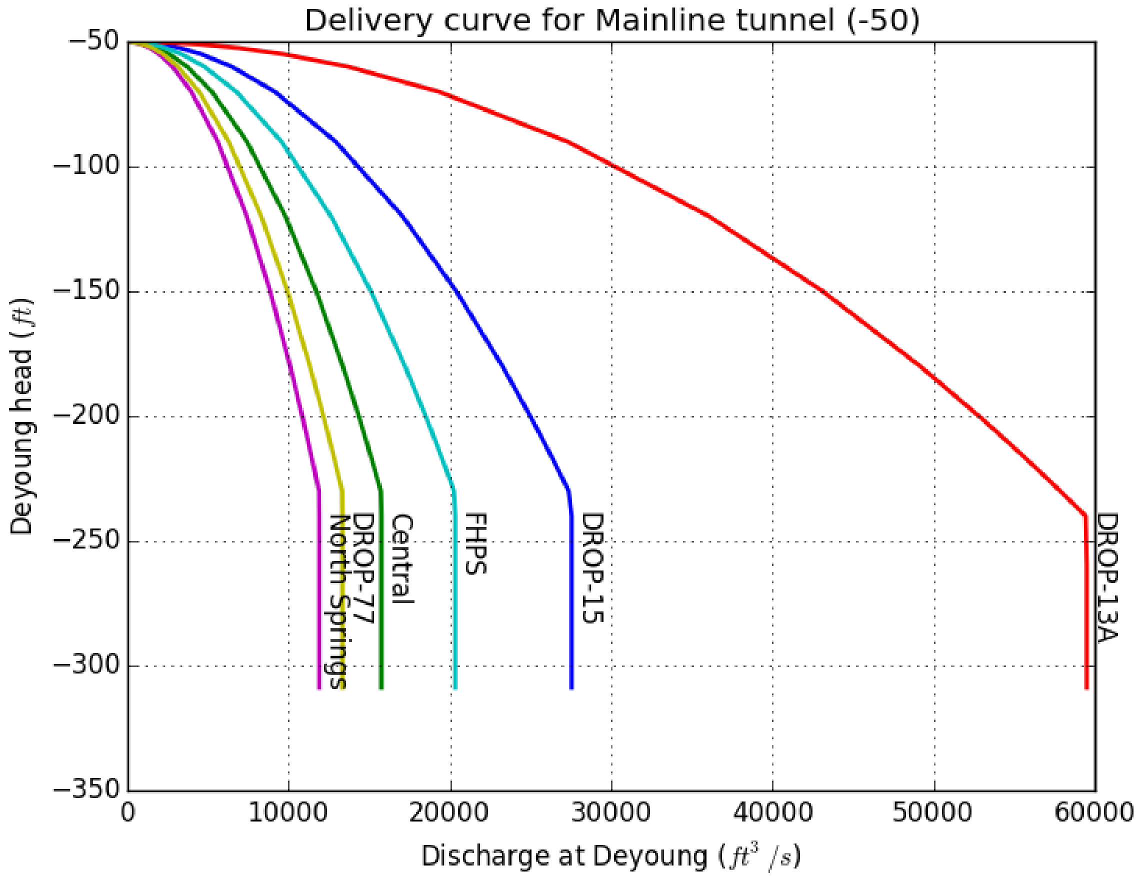


Figure 3: Delivery curve for Mainline Tunnel (head of -50 ft)

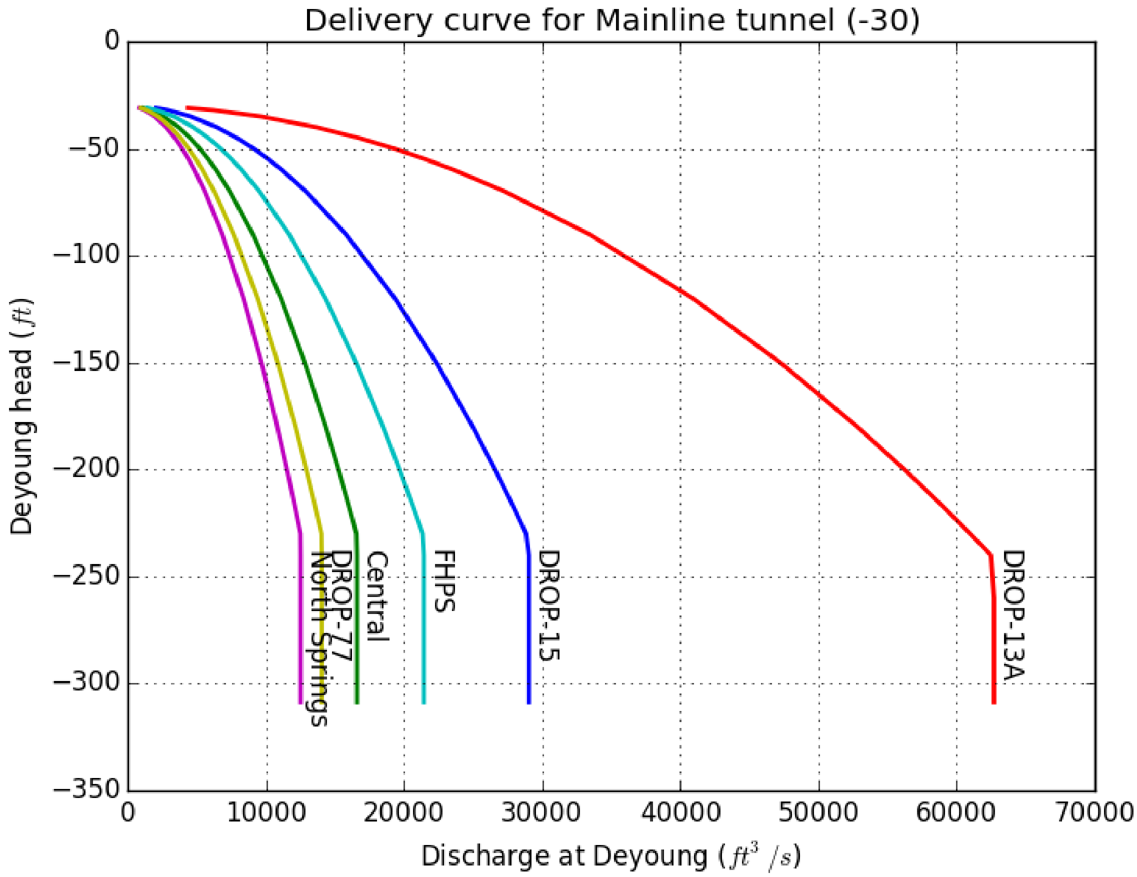


Figure 4: Delivery curve for Mainline Tunnel (head of -30 ft)

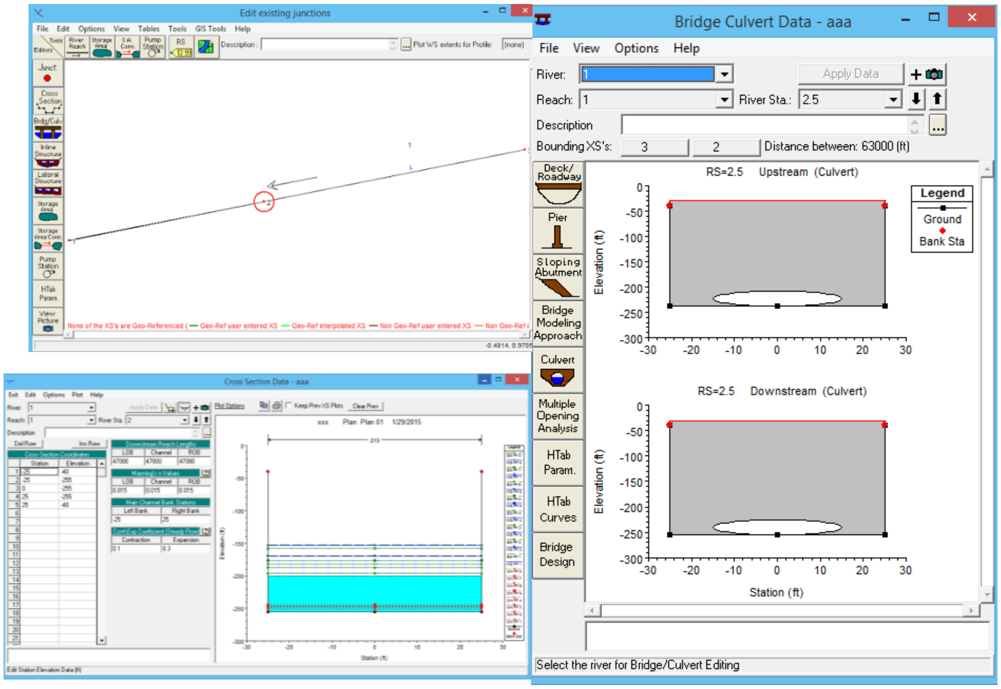
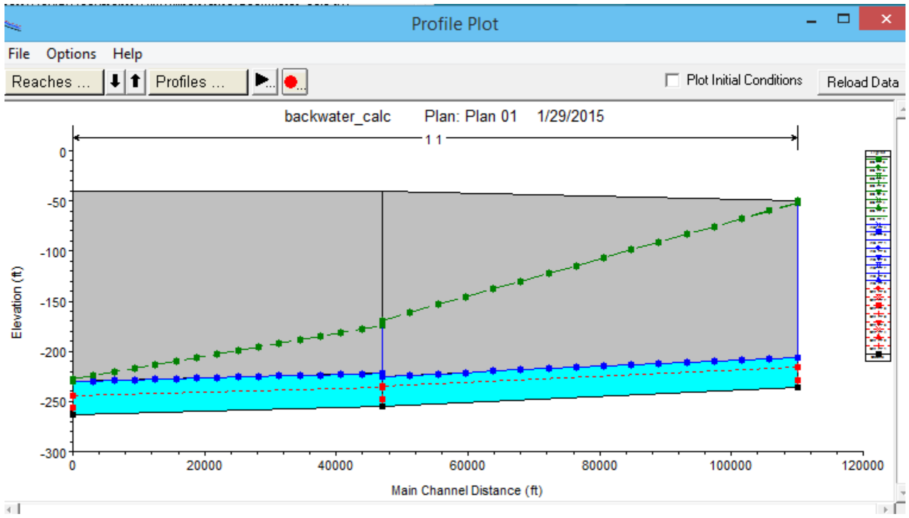


Figure 5: HEC-RAS Geometry Editor

Note: Water elevations are present because screenshots were taken after the model was run.



Profile Output Table - Standard Table 1

HEC-RAS Plan: Plan 01 River: 1 Reach: 1

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
1	3	PF 1	11568.00	-236.00	-50.01	-228.54	-50.00	0.000000	0.62	18599.15	100.00	0.01
1	3	PF 2	11568.00	-236.00	-50.01	-228.54	-50.00	0.000000	0.62	18599.15	100.00	0.01
1	3	PF 3	11569.00	-236.00	-49.98	-228.54	-49.97	0.000000	0.62	18602.26	100.00	0.01
1	3	PF 4	11569.00	-236.00	-49.98	-228.54	-49.97	0.000000	0.62	18602.26	100.00	0.01
1	3	PF 5	11570.00	-236.00	-49.95	-228.54	-49.94	0.000000	0.62	18605.38	100.00	0.01
1	3	PF 6	11570.00	-236.00	-49.95	-228.54	-49.94	0.000000	0.62	18605.38	100.00	0.01
1	3	PF 7	11570.00	-236.00	-49.95	-228.54	-49.94	0.000000	0.62	18605.38	100.00	0.01
1	3	PF 8	11570.00	-236.00	-49.95	-228.54	-49.94	0.000000	0.62	18605.38	100.00	0.01
1	2.1		Culvert									
1	2	PF 1	11568.00	-255.00	-173.27	-247.56	-173.24	0.000002	1.42	8173.10	100.00	0.03
1	2	PF 2	11568.00	-255.00	-173.27	-247.56	-173.24	0.000002	1.42	8173.10	100.00	0.03
1	2	PF 3	11569.00	-255.00	-173.26	-247.56	-173.23	0.000002	1.42	8174.08	100.00	0.03
1	2	PF 4	11569.00	-255.00	-173.26	-247.56	-173.23	0.000002	1.42	8174.08	100.00	0.03
1	2	PF 5	11570.00	-255.00	-173.25	-247.55	-173.22	0.000002	1.42	8175.06	100.00	0.03
1	2	PF 6	11570.00	-255.00	-173.25	-247.55	-173.22	0.000002	1.42	8175.06	100.00	0.03
1	2	PF 7	11570.00	-255.00	-173.25	-247.55	-173.22	0.000002	1.42	8175.06	100.00	0.03
1	2	PF 8	11570.00	-255.00	-173.25	-247.55	-173.22	0.000002	1.42	8175.06	100.00	0.03
1	1.1		Culvert									
1	1	PF 1	11568.00	-263.00	-230.00	-255.56	-229.81	0.000023	3.51	3300.00	100.00	0.11
1	1	PF 2	11568.00	-263.00	-230.00	-255.56	-229.81	0.000023	3.51	3300.00	100.00	0.11
1	1	PF 3	11569.00	-263.00	-230.00	-255.56	-229.81	0.000023	3.51	3300.00	100.00	0.11
1	1	PF 4	11569.00	-263.00	-230.00	-255.56	-229.81	0.000023	3.51	3300.00	100.00	0.11
1	1	PF 5	11570.00	-263.00	-230.00	-255.56	-229.81	0.000023	3.51	3300.00	100.00	0.11
1	1	PF 6	11570.00	-263.00	-230.00	-255.56	-229.81	0.000023	3.51	3300.00	100.00	0.11
1	1	PF 7	11570.00	-263.00	-230.00	-255.56	-229.81	0.000023	3.51	3300.00	100.00	0.11
1	1	PF 8	11570.00	-263.00	-230.00	-255.56	-229.81	0.000023	3.51	3300.00	100.00	0.11

Figure 6: Screenshots of Profile and Results Table

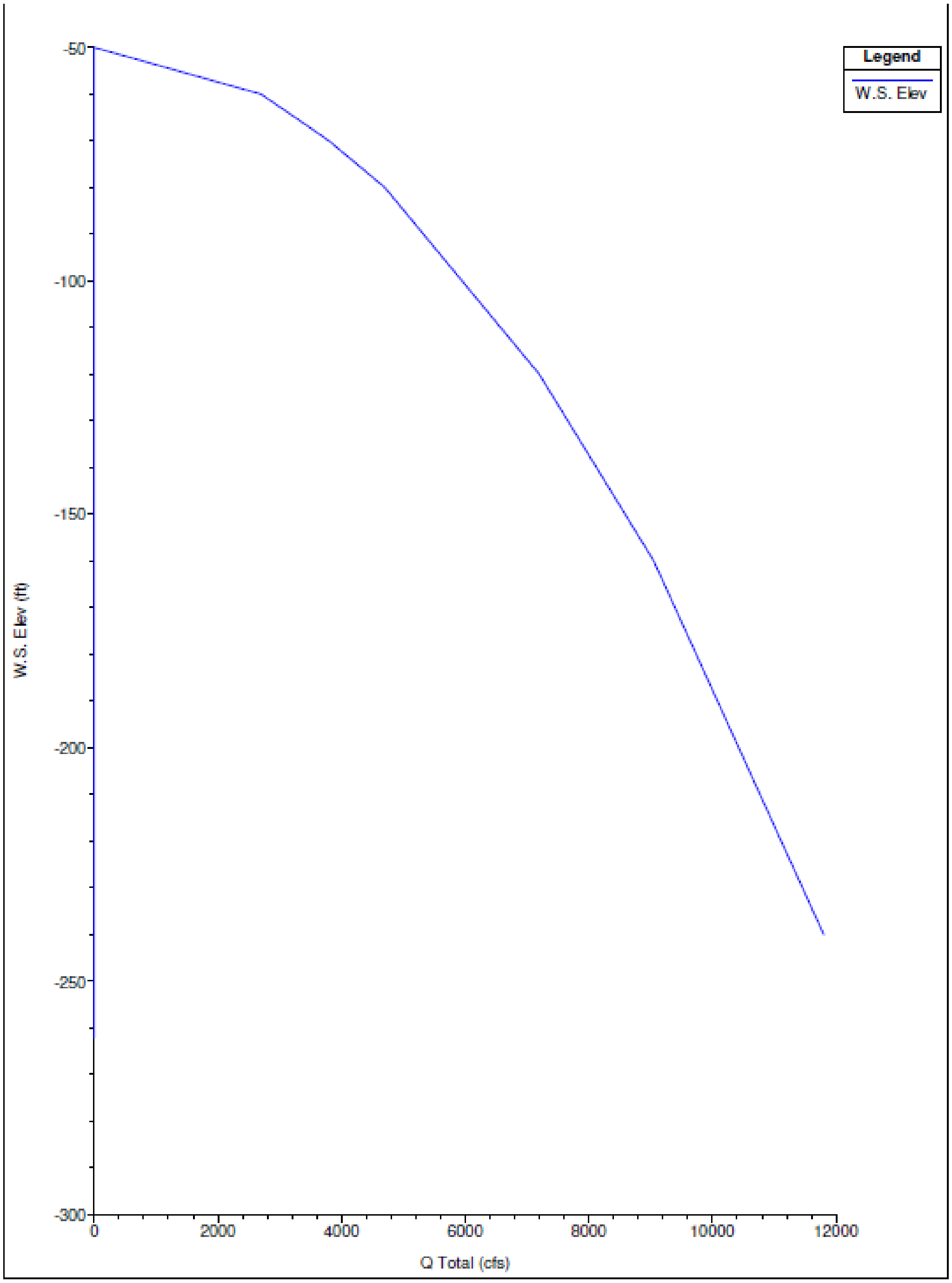


Figure 7: Flow Rating Curve for the North Springs confluence drawn by HEC-RAS

APPENDICES

Appendix A: K-Values Table, from Jensen Eng. Pump Station Design Guidelines

Entrance	Bellmouth	0.005
	Rounded	0.25
	Sharp-Edged	0.5
	Projecting	0.8
Exits		1.0
90° Bend		0.25
45° Bend		0.18
Tee, line flow		0.30
Tee, branch flow		0.75
Cross, line flow		0.50
Cross, branch flow		0.75
Wye, 45°		0.50

Ball		0.04
Check Valves	Ball	0.9-1.7
	Rubber flapper (v < ft/s)	2.0
	Rubber flapper (v > ft/s)	1.1
	Swing	0.6-2.2
Gate	Double Disc	0.1-0.2
	Resilient seat	0.3
Knife	Metal seat	0.2
	Resilient seat	0.3
Eccentric	Rectangular (80%) opening	1.0
	Full bore opening	0.5

Table 2 – K Values, Source: Pumping Station Design

Appendix B: Example spreadsheet (Inlet Control)

Inlet Control Limits				**Inlet Control Limits**		
DIAMETER	FLOW			DIAMETER	FLOW	
inch	cfs	gpm		inch	cfs	gpm
360	8234.63	3695702		**396**	10252.42	4601288
Orifice Formula				Orifice Formula		
360	11424.19			**396**	14497.96	
over	**0**			over	**0**	
Head,ft	15			Head,ft	16.5	
360	11424.19			**396**	14497.96	

Appendix C: Example Spreadsheet (Outlet Control)

CIRCULAR PIPES (OPEN)		
	Culvert 1-2	Culvert 2-3
Diameter (inch)	360	396
Diameter (Feet)	30.00	33.00
Water Depth/Diameter Ratio	**1.00**	**1.00**
Water Depth (feet)	**30.00**	**33.00**
n	0.015	0.015
Slope	0.0014	0.0009
Chord c (ft)	0.346	0.363
Tg ALFA	0.012	0.011
*ANGLE 2*ALFA (Radians)*	0.023	0.022
Wetted Perimeter (ft)	93.902	103.309
Area (sqft)	706.860	855.300
Hydraulic Radio (ft)	7.528	8.279
Flow, cfs	10063.87077	10109.77101
Velocity, fps	14.24	11.82
m/sec	4.340681728	3.603701589
Area, sqft	706.8597691	855.3003578
sq.mt.	65.70305706	79.5007025
Velocity head (ft)	3.147586736	2.169499042
LENGTH, ft	46950	62950
hf, ft	65.73	53.5075
Entrance loss		
k.	1	1
he, ft	3.15	2.17
Exist loss		
k.	1	1
hex, ft	3.15	2.17

Outlet control was computed using friction formulas. See the *HEC-RAS Hydraulic Reference Manual*, Ch. 2 for more information.

Appendix D: Example Spreadsheet (Open Channel Sections)

RECTANGULAR CHANNEL (OPEN)			
	MH-1	MH-2	MH-3
Heigth (feet)	**62**	**115**	**152**
Width (feet)	**50**	**50**	**50**
Free Board (feet)	0	0	0
n	0.015	0.015	0.015
slope	0.000026	0.00001	0.00001
Water Depth (feet)	62	115	152
Flow, cfs	10661.47	13481.15	18353.83
Velocity, fps	3.44	2.34	2.41
m/sec	1.05	0.71	0.74
Area, sqft	3100	5750	7600
sq.mt	288.1469363	534.4661	706.4247
Velocity Head, feet	0.183664305	0.085356	0.090561
meters	0.055995215	0.026023	0.02761
Froude Number	0.07697178	0.038529	0.034519
Length, ft	**50**	**50**	**50**
Hf, ft	0.0013	0.0005	0.0005

Note: The "open channel" sections of the system constitute the manhole sections of the tunnel. These include the input section (sta. 110000), the diameter change section (sta. 47000) and the downstream section which discharges into the reservoir (sta. 0). Computations were made using open channel head loss formulas from Henderson's *Open Channel Flow*.

Appendix E: Example Spreadsheet (Step Method Table)

Xsection	Stage,ft	channel											
		IE	width, ft	Area,ft2	Q, cfs	v, fps	v^2/2g	ToTal Head, tf	friction loss, ft	Exit loss, ft	Entrance loss,ft		Delta
1	-230	-262	50	1600	10000	6.25	0.606561	-229.393					-32
1.1	-226.851	-262	50	1757.444	10000	5.690081	0.502749	-226.348	0.0013	3.15			
	Culvert												
2	-157.974								65.73		3.147587		
2.1	-155.804	-255	50	4959.824	10000	2.016201	0.063122	-155.740	0.0005	2.169499			-99.1965
	Culvert												
3	-100.127								53.5075		2.169499		
3.1	-100.126	-238	50	6893.699	10000	1.4506	0.032675	-100.093	0.0005				-137.874