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Memorandum

To	Edward Stribula	Page	1
CC			
Subject	Cleveland Innerbelt Sewer Study Addendum (DRAFT)		
From	Daniel Rosenberg		
Date	March 31, 2010		

The purpose of this technical memorandum is to present the results of the hydraulic analysis of the following alternatives for the Easterly Interceptor associated with the proposed I-90 Innerbelt Realignment modifications:

- Alternative 2A - 120-inch/102-inch RCP sewer to reroute the flow to the south of the existing sewer;
- Alternative 2B - 110-inch/104-inch CCFRPMP sewer to reroute the flow to the south of the existing sewer and;
- Alternative 2C - 120-inch/Twin 96-inch RCP sewer to reroute the flow to the south of the existing sewer.

This study is an addendum to the original study dated February, 2006. This technical memorandum discusses the various alternatives considered, hydraulic impacts of the alternatives based on the results of the model simulation and recommendations to maintain the conveyance capacity of the interceptor system. The alternatives evaluated in this memorandum were developed by DLZ Corp. (DLZ). AECOM's services have consisted of hydraulic analysis of the alternatives to evaluate the impact of each on the hydraulic grade line in the Easterly Interceptor.

The goal of this project is to evaluate the three specific alternatives provided by DLZ for the conveyance of waste water and wet weather flows in the Easterly Interceptor between E. 26th St. and E. 33rd St. This was accomplished by:

- Use of the Northeast Ohio Regional Sewer District's (NEORSD) Easterly baseline with Early Action Projects hydraulic model and Advanced Facilities Plan (AFP) hydraulic model as modified in the original study as a basis to construct models of the conceptual alternative plans provided by DLZ;
- Simulate dry weather flow, the 5-year, 6-hour design storm and the District's combined sewer overflow (CSO) control storms for each of the alternatives in the hydraulic model;
- Process and analyze hydraulic grade line (HGL) and velocity results for simulated storm events for each alternative;

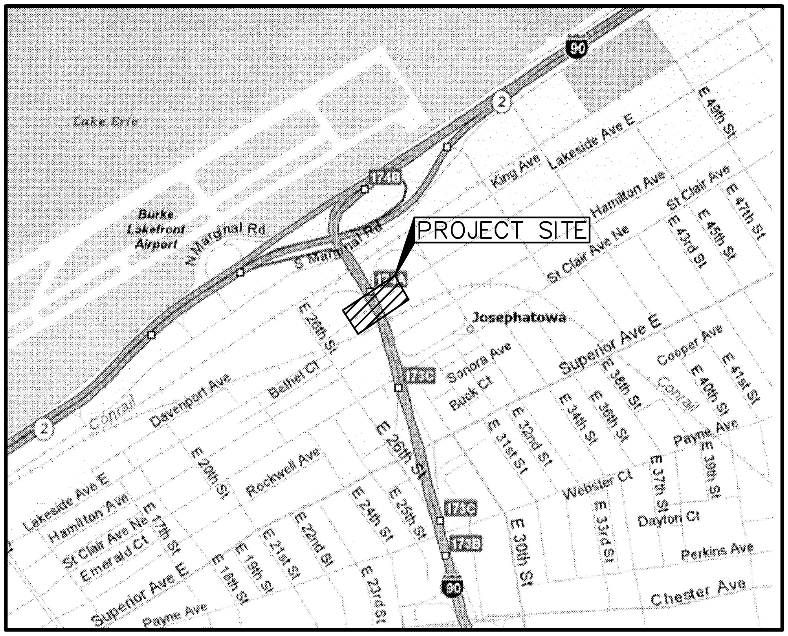
- Compare hydraulic model results with calculations prepared in Excel for the AFP alternatives, and;
- Review CSO control storm results to determine if CSO control at adjacent CSO regulators is maintained.

BACKGROUND

The Northeast Ohio Regional Sewer District's (NEORSD) Easterly Interceptor conveys combined sewer flows to the Easterly Wastewater Treatment Plant (WWTP). It ranges from 8-feet in diameter at Lakeside Avenue and W. 9th Street to 13.5-feet in diameter at the WWTP influent near Lake Shore Boulevard and E. 140th Street.

The Easterly Interceptor Hydraulic Modeling project site, shown in Figure 1, is located at Lakeside Avenue and Interstate-90. The interceptor in the study area is an 11-ft 9-inch circular sewer constructed of four (4) rings of bricks. The sewer transports flow from west to east along Lakeside Avenue and crosses perpendicular under Interstate-90. There is one (1) 12-inch corrugated metal pipe (CMP) connection roughly 30-feet west of the west shoulder of Interstate-90 as proposed by the Innerbelt Realignment project.

Figure 1. Easterly Interceptor Hydraulic Modeling Project Site



The Innerbelt Realignment project proposes modifications that lower the elevation of Interstate-90 at the interceptor crossing. Under these proposed modifications, a portion of the outer brick layer of the existing interceptor crown would protrude into the Interstate-90 pavement section. One of the objectives of this hydraulic modeling project is to evaluate the Easterly Interceptor and nearby CSO regulators to determine whether CSO control is maintained. CSO control will be evaluated under conditions as they exist in the sewer system at the present time. In addition, CSO control will be evaluated under future conditions with the District's CSO control plan in place. The future condition will be assessed by using the hydraulic model developed by NEORSD as part of the Easterly CSO Tunnel Storage Advanced Facilities Plan (AFP).

The hydraulic impact of the three alternatives on the conveyance capacity of the Easterly Interceptor during and after proposed Interstate-90 modifications was considered. The following sections describe the work that was completed and the results. It is important to note that for all of the alternatives evaluated it was assumed that the transitions between the existing and proposed pipe sections would not be abrupt and include rounded edges to minimize head loss through these sections.

REGULATOR E-11 DRY WEATHER OUTLET RECONNECTION

Additionally, the storm water outlet at regulator E-11, located at the just west of the Innerbelt crossing, was recommended to be bulkheaded during the Easterly CSO Phase II Facilities Planning Study (M&E, March, 2002). The dry weather connection currently connects to the Easterly Interceptor in the proposed abandoned section. This connection will have to be reestablished to the realigned interceptor as part of this project. The conveyance capacity of the new pipe will need to be at least as great as the existing connection.

ALTERNATIVES

The three (3) alternatives discussed as part of this technical memorandum are based on conceptual plans provided by DLZ Corp.

Alternative 2A: 102-inch Diameter RCP Sewer

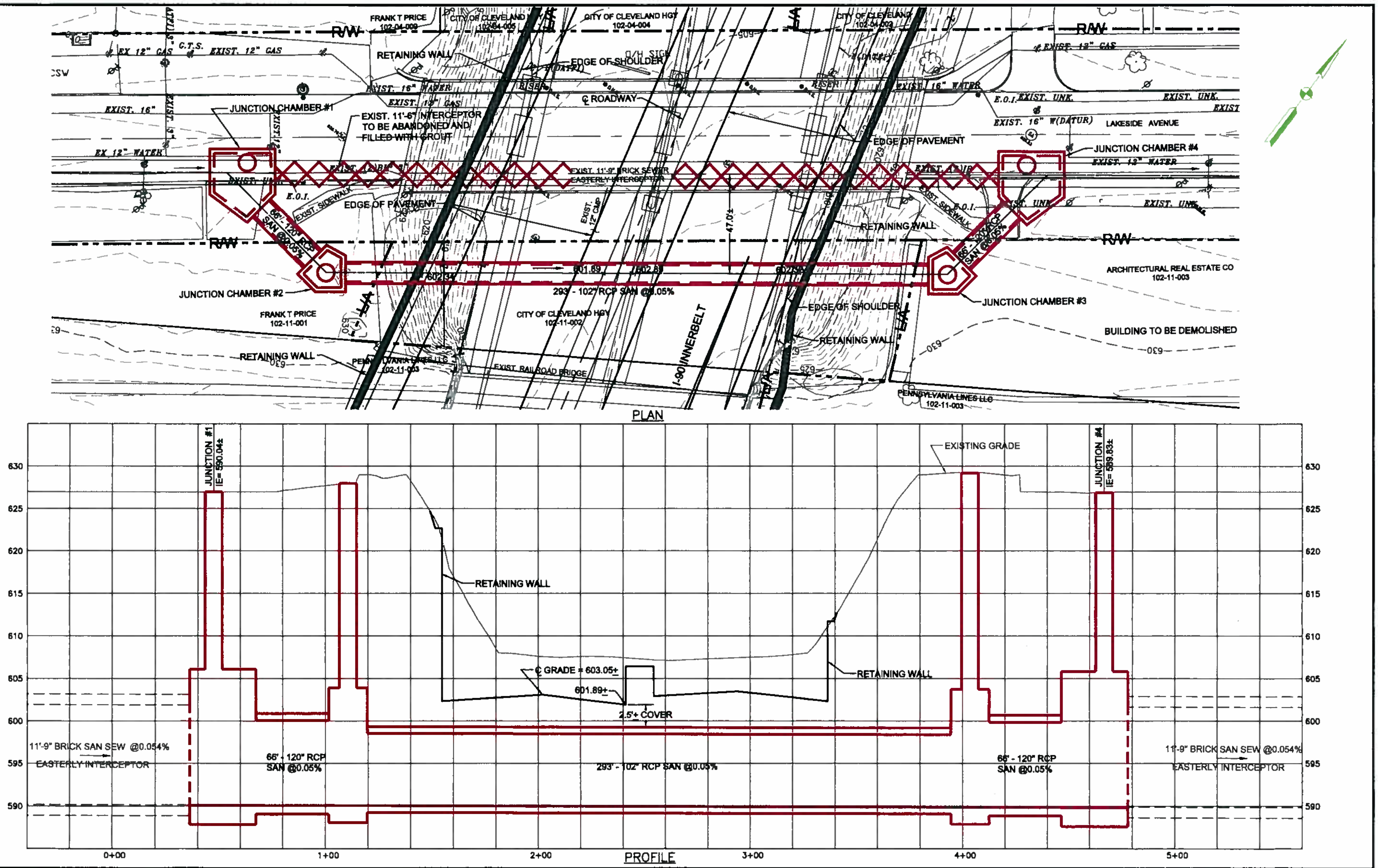
Depicted in Figure 4A, Alternative 2A proposes that the sewer underneath I-90 be realigned to the south and replaced with two 120-inch sections and one 102-inch section of RCP sewer. Four (4) new junction chambers will direct flow to the new sewer sections. The existing interceptor is abandoned. In the hydraulic model, the roughness coefficient of the new RCP sewer was modeled as 0.013, which consistent with the February 2006 study. The upstream and downstream invert elevations of the new sewer match the existing upstream and downstream invert elevations of the brick interceptor.

Alternative 2B: 104-inch Diameter CCFRPMP Sewer

Depicted in Figure 4B, Alternative 2B proposes that the sewer underneath I-90 be realigned to the south and replaced with two 110-inch sections and one 104-inch section of CCFRPMP sewer. Four (4) new junction chambers will direct flow to the new sewer sections. The existing interceptor is abandoned. In the hydraulic model, the roughness coefficient of the new CCFRPMP sewer was modeled as 0.011, which consistent with the February 2006 study. The upstream and downstream invert elevations of the new sewer match the existing upstream and downstream invert elevations of the brick interceptor.

Alternative 2C: Twin 96-inch Diameter RCP Sewer

Depicted in Figure 4C, Alternative 2C proposes that the sewer underneath I-90 be realigned to the south and replaced with two 120-inch sections and one Twin 96-inch section of RCP sewer. Four (4) new junction chambers will direct flow to the new sewer sections. The existing interceptor is



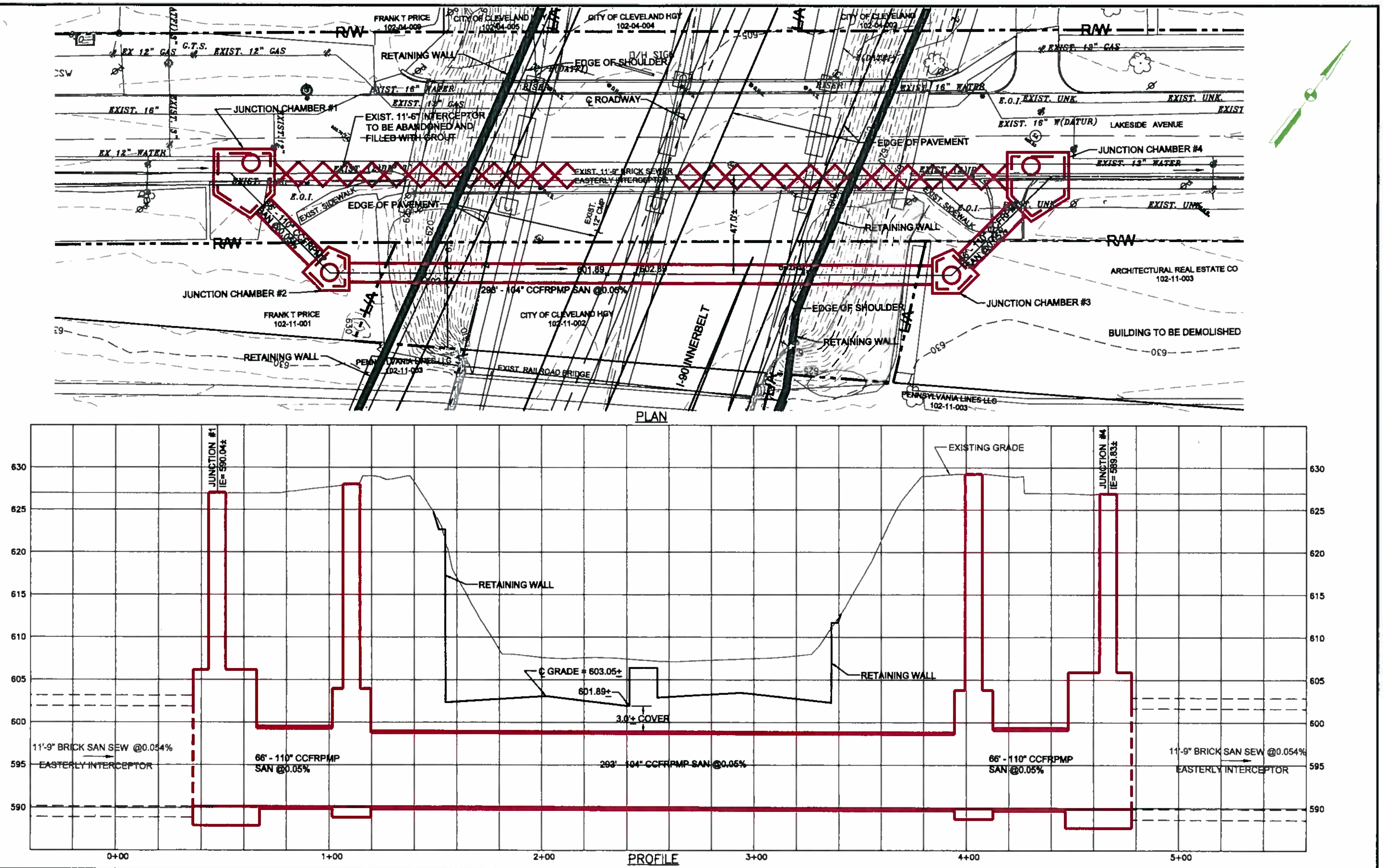
DLZ

0 5 10
VERTICAL SCALE
0 20 40
HORIZONTAL SCALE

CUY-90 INNERBELT PROJECT EASTERLY INTERCEPTOR AT I-90/LAKESIDE AVE.

ALTERNATE NO. 2A - REALIGNED EASTERLY INTERCEPTOR W/ 102" RCP

FIG. 4A



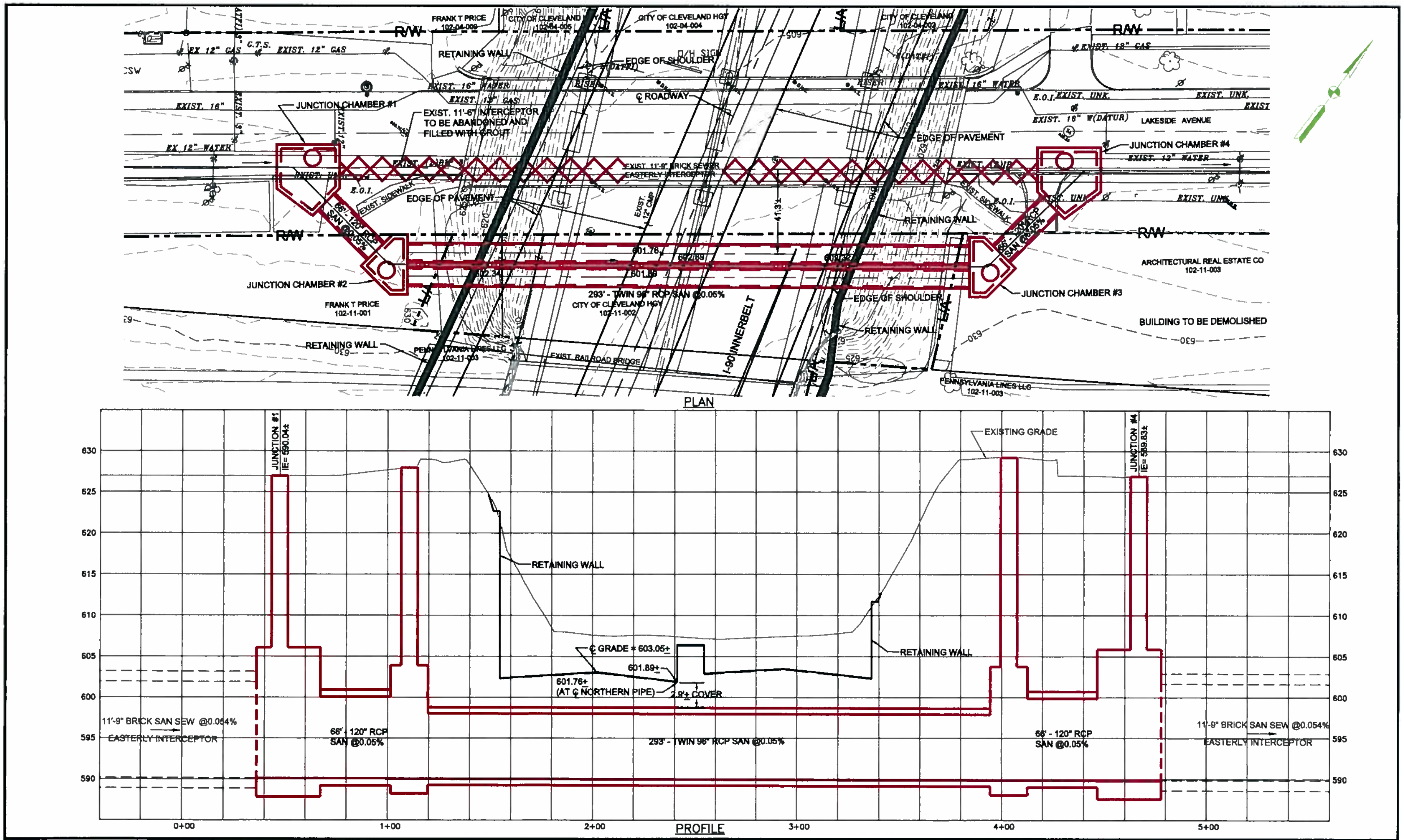
DLZ

0 5 10
VERTICAL SCALE
0 20 40
HORIZONTAL SCALE

CUY-90 INNERBELT PROJECT EASTERLY INTERCEPTOR AT I-90/LAKESIDE AVE.

ALTERNATE NO. 2B - REALIGNED EASTERLY INTERCEPTOR W/ 104" CCFRPMP

FIG. 4B



DLZ

VERTICAL SCALE 0 5 10

HORIZONTAL SCALE 0 20 40

CUY-90 INNERBELT PROJECT EASTERLY INTERCEPTOR AT I-90/LAKESIDE AVE.

ALTERNATE NO. 2C - REALIGNED EASTERLY INTERCEPTOR W/ TWIN 96" RCP

FIG. 4C

abandoned. In the hydraulic model, the roughness coefficient of the new RCP sewer was modeled as 0.013, which consistent with the February 2006 study. The upstream and downstream invert elevations of the new sewer match the existing upstream and downstream invert elevations of the brick interceptor.

EVALUATION PROCEDURE

The NEORSD Easterly baseline with Early Action Projects hydraulic model and AFP hydraulic model were used to simulate the Easterly Interceptor's response to the three (3) alternatives under dry-weather flow and various storm flows. The baseline hydraulic model conditions were developed under the Easterly CSO Phase II Facilities Planning Study. For more information on development of the Easterly baseline hydraulic model, see the Easterly CSO Phase II Hydraulic Modeling Report (Metcalf & Eddy, 2002). Since the sewer network tributary to the Easterly Interceptor under the AFP will be different than the existing sewer network, two sewer models were simulated as part of this project. The baseline hydraulic model network was constructed using the baseline sewer network plus the Early Action Projects and is called the "baseline with early action model" in this technical memorandum. The AFP hydraulic model represents future conditions of the sewer system under full CSO compliance and is called the "AFP model" in this technical memorandum. It represents a conservative future flow scenario in the Easterly Interceptor for design.

Dry-weather flow, the 5-year, 6-hour design storm and the top five (5) NEORSD CSO control storms were simulated in the baseline with early action model and the AFP model for the existing 11-ft 9-inch Easterly Interceptor brick sewer and for each of the five (5) alternatives.

The results of the baseline plus early action model and the AFP model were processed and analyzed for the simulated storm events for the existing brick interceptor and each of the alternatives. To accomplish this

- The CSO control storm results were reviewed to determine if CSO control at adjacent regulator E-12 was being maintained

- The peak HGL for the most severe hydraulic scenario was reviewed at key points along the interceptor

The model results were then compared with calculations done in Excel that calculated the friction loss through each conduit and the headloss at each manhole based on the change in direction and pipe size. The Excel calculations are provided in Appendix A.

CSO IMPACTS

The DWO from regulator E-12 is located approximately 500-feet upstream of the project site on the Easterly Interceptor and connects into the interceptor at an elevation of approximately 614.3-feet. If proposed alternatives raise the peak HGL above this elevation at the connection point, the overflow volume and frequency at this regulator may increase. The HGL of the recommended alternatives remains below the crown of the interceptor (approximately 602.0-feet) at the connection point for all of the CSO control storms; therefore, additional overflow does not occur as a result of the proposed alternatives.

EVALUATION RESULTS

Hydraulic scenarios were reviewed for the 5-year design storm and the CSO control storms for all of the alternatives. It was determined that the 5-year design storm had more severe hydraulic impacts on the alternatives than any of the CSO control storms. Therefore, the peak HGL under the 5-year design storm condition was evaluated along the interceptor to determine if proposed alternatives caused flooding or surcharging in the hydraulic models. In the AFP model, proposed sewer segments were surcharged for all of the alternatives, but the existing 11-feet 9-inch diameter interceptor was under free flow conditions. Although surcharging was present, the HGL remained between 0.9 and 1.4 feet below the minimum proposed ground surface elevation of Interstate-90 (approximately 601.76-feet) for the sewer section at the proposed Innerbelt crossing.

The Easterly Interceptor sewer system is designed to convey the 5-year 6-hour design storm. For storms larger than this, additional flow will be relieved from the system through the overflows. However, during these higher intensity or duration storms, some additional surcharging may be present in the Easterly Interceptor under any of the alternative conditions.

Manholes along the Easterly Interceptor between E. 26th Street and E. 33rd Street were chosen as key points for hydraulic review of the 5-year design storm. The sewer system further upstream or downstream did not appear to be affected by any of the alternatives. The peak water level and velocity during dry-weather flow and during the 5-year storm simulations are shown in Table 1 and Table 2 at the key points. Table 1 represents results from the baseline with early action model and Table 2 represents results from the AFP model. The peak water level at each key point is represented as depth above the manhole invert and is shown in feet. The velocity is determined in the downstream pipe and is shown in feet per second (ft/sec).

The shaded cells in Table 2 represent sewer sections of the alternatives where surcharging was evident during the 5-year storm in the hydraulic model, or where low velocities occur in the proposed sections. The HGL in each sewer section is higher than the crown of the pipe, but none exceed the ground surface elevation of Interstate-90.

Evaluation of the capacity at each of the proposed alternatives was compared to the peak flow rate in the Easterly Interceptor. All of the options have the capacity to convey the 5-year design storm with surcharging to ground surface under I-90. However, option 2C shows a dramatic reduction in velocity during dry weather flow.

CONCLUSIONS

Based on the information provided, Alternatives 2A and 2B effectively convey flows in the Easterly Interceptor at Lakeside Avenue and Interstate-90 for the 5-year design storm, maintain CSO control at adjacent regulator E-12 and provide velocities above 1.5 feet per second during dry weather flow. Alternative 2C (twin 96-inch diameter pipes) resulted in velocities less than 1.5 feet per second during dry weather flow. These velocities will further exacerbate the deposition of grit and debris through this portion of the Easterly Interceptor.

One further option that could be developed would be a combination of 120-inch RCP and 104-inch CCFRPMP.

Table 1. Baseline Flow Rates and Velocities

Location on Lakeside Avenue	Manhole Name	Profile Baseline						Profile Baseline Alternative 2A						Profile Baseline Alternative 2B						Profile Baseline Alternative 2C					
		Existing Brick Sewer						102-inch RCP Sewer						104-inch CCFRPMP Sewer						Twin 96-inch RCP Sewers					
		Pipe Size (ft)	Invert Elev. (ft)	Water Level* (ft)		Velocity (ft/sec)		Pipe Size (ft)	Invert Elev. (ft)	Water Level* (ft)		Velocity (ft/sec)		Pipe Size (ft)	Invert Elev. (ft)	Water Level* (ft)		Velocity (ft/sec)		Pipe Size (ft)	Invert Elev. (ft)	Water Level* (ft)		Velocity (ft/sec)	
				DWF	5-year	DWF	5-year			DWF	5-year	DWF	5-year			DWF	5-year	DWF	5-year			DWF	5-year	DWF	5-year
E. 26th Street	EAA325	11.75	590.26	1.2	8.0	2.0	4.3	11.75	590.26	1.3	9.3	1.5	3.3	11.75	590.26	1.3	9.4	1.6	3.3	11.75	590.26	1.2	8.8	1.7	3.6
Temporary Pit w/Permanent Manhole Access/ Junction Chamber #1	IBUPSTM	-	-	-	-	-	-	10	589.96	1.5	9.6	1.6	4.0	9.17	589.96	1.5	9.7	1.8	4.5	10	589.96	1.4	9.1	2.1	4.3
Interstate 90	EAA320	11.75	590	1.2	8.2	1.6	3.9	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Junction Chamber #2	RLIGNUP	-	-	-	-	-	-	8.5	589.93	1.4	9.3	1.9	5.3	8.67	589.93	1.4	9.3	1.9	5.1	2@8.0	589.93	1.3	8.8	1.1	3.0
Junction Chamber #3	RLIGNDS	-	-	-	-	-	-	8.5	589.8	1.4	8.7	1.8	4.6	8.67	589.8	1.4	8.8	1.9	4.9	2@8.0	589.8	1.4	8.7	1.8	4.6
Temporary Receiving Pit w/ Junction Chamber #4	IBDSTRM	-	-	-	-	-	-	10	589.75	1.3	8.5	1.5	3.9	9.17	589.75	1.3	8.5	1.5	3.9	10	589.75	1.3	8.5	1.5	3.9
E.33rd Street	EAA315	11.75	589.31	1.5	8.8	1.6	4.3	11.75	589.31	1.5	8.8	1.5	4.2	11.75	589.31	1.5	8.7	1.5	4.2	11.75	589.31	1.5	8.8	1.6	4.2

* Depth above pipe invert

Water Level above crown of pipe

Low velocity

Table 2. AFP Flow Rates and Velocities

Location on Lakeside Avenue	Manhole Name	Profile AFP						Profile AFP Alternative 2A						Profile AFP Alternative 2B						Profile AFP Alternative 2C					
		Existing Brick Sewer						102-inch RCP Sewer						104-inch CCFRPMP Sewer						Twin 96-inch RCP Sewers					
		Pipe Size (ft)	Invert Elev. (ft)	Water Level* (ft)		Velocity (ft/sec)		Pipe Size (ft)	Invert Elev. (ft)	Water Level* (ft)		Velocity (ft/sec)		Pipe Size (ft)	Invert Elev. (ft)	Water Level* (ft)		Velocity (ft/sec)		Pipe Size (ft)	Invert Elev. (ft)	Water Level* (ft)		Velocity (ft/sec)	
				DWF	5-year	DWF	5-year			DWF	5-year	DWF	5-year			DWF	5-year	DWF	5-year			DWF	5-year	DWF	5-year
E. 26th Street	EAA325	11.75	590.26	1.3	9.7	2.0	4.4	11.75	590.26	1.4	10.9	1.5	3.4	11.75	590.26	1.4	11.0	1.6	3.4	11.75	590.26	1.3	10.4	1.7	3.7
Temporary Pit w/Permanent Manhole Access/ Junction Chamber #1	IBUPSTM	-	-	-	-	-	-	10	589.96	1.5	11.1	1.7	4.1	9.17	589.96	1.5	11.2	1.8	4.8	10	589.96	1.4	10.7	2.1	4.5
Interstate 90	EAA320	11.75	590	1.2	9.9	1.6	4.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Junction Chamber #2	RLIGNUP	-	-	-	-	-	-	8.5	589.93	1.5	10.9	2.0	5.7	8.67	589.93	1.5	10.8	2.0	5.5	2@8.0	589.93	1.4	10.4	1.1	3.4
Junction Chamber #3	RLIGNDS	-	-	-	-	-	-	8.5	589.8	1.4	10.2	1.9	4.7	8.67	589.8	1.4	10.3	2.0	5.2	2@8.0	589.8	1.4	10.4	1.9	4.8
Temporary Receiving Pit w/ Junction Chamber #4	IBDSTRM	-	-	-	-	-	-	10	589.75	1.4	10.1	1.6	4.0	9.17	589.75	1.4	10.0	1.6	4.0	10	589.75	1.4	10.1	1.6	4.0
E.33rd Street	EAA315	11.75	589.31	1.5	10.4	1.6	4.3	11.75	589.31	1.5	10.3	1.6	4.3	11.75	589.31	1.5	10.3	1.6	4.3	11.75	589.31	1.5	10.4	1.6	4.3

* Depth above pipe invert

Water Level above crown of pipe

Low velocity

APPENDIX A

Number of manholes	8	Invert	Ground	Water Level	Distance upstream	Cumulative Distance upstream	Pipe Crown
Downstream	EAA315	589.31	627.11	599.62	0	0	601.06
Downstream of Expansion	IBDSTRM	589.75	628	599.82	723	723	601.5
Upstream of Expansion	IBDSTRM	589.75	628	599.90	0	723	599.75
Realigned Pipe Section Downstream Expansion	RLIGNDS	589.8	628	599.92	66	789	599.8
Realigned Pipe Section Upstream Expansion	RLIGNDS	589.8	628	600.09	0	789	598.3
Realigned Pipe Section Downstream Contraction	RLIGNUP	589.93	628	600.39	293	1082	598.43
Realigned Pipe Section Upstream Contraction	RLIGNUP	589.93	628	600.53	0	1082	599.93
Downstream of Contraction	IBUPSTM	589.96	628	600.56	66	1148	599.96
Upstream of Contraction	IBUPSTM	589.96	628	600.61	0	1148	601.71
	EAA325	590.26	628.26	600.74	490.53	1638.53	602.01
	EAA330	590.81	629.11	600.80	248.72	1887.25	602.56
Upstream	EAA335	591.48	630.97	600.97	594	2481.25	603.23
Flow Rate through System (cfs)	340	Uses 3% Safety Factor					
Downstream Water Level at EAA315 ()	599.623	Calculate Water Level based on Normal Flow =		595.29	Flow is therefore downstream controlled, use model water level		
Friction loss between IBDSTRM and EAA315							
$h_f=2.87n^2(LV^2/D^{4/3})$							
n~	Manning's roughness coefficient						
L~	Length of Pipe						
V~	Velocity						
D~	Diameter of Pipe						
n=	0.015						
L=	723	Fill in for Q and V will be calculated					
V=	3.37	Q=	340				
D=	11.75						
DEPTH OF FLOW=	10.31	4.85	**100.856473**				
h_f=	0.199						
Slope of Pipe	0.0006						
INVERT AT DOWNSTREAM END (EAA315)	589.31						
INVERT AT UPSTREAM END (IBDSTRM)	589.75						
Depth at downstream end (EAA315)	599.62						
Depth at upstream end (IBDSTRM)	599.82						
Compute losses due to Change in Direction							
	$h_L= (h_{dir})$						
	$h_{dir} =K_{dir}*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)			
V1=	4.33	ft/sec		78.5398163			
V2=	3.37	ft/sec		100.856473			
K_{dir}=	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, Appendix C					
g=	32.2						
h_{dir} =	0.050	ft					
Depth upstream of Direction Change (IBDSTRM)	**599.87**						
Compute losses due to Expansion at IBDSTRM							
	$h_L = (h_{expansion})$						
	$h_{expansion} =K_e*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)			
V1=	4.33	ft/sec		78.5398163			
V2=	3.37	ft/sec		100.856473			
K_e=	0.2	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, pg 43 Table 2-7					
g=	32.2						
$h_{expansion}$ =	0.023	ft					
Depth upstream of expansion (IBDSTRM)	**599.90**						

$h_f=2.87n^2(LV^2/D^{4/3})$				
n=	0.013			
L=	66	Fill in for Q and V will be calculated		
V=	4.33	Q=	340	
D=	10			
DEPTH OF FLOW=	10.15	6.28	**78.53981627**	
h_f=	0.028			
INVERT AT DOWNSTREAM END (IBDSTRM)	589.75			
INVERT AT UPSTREAM END (RLIGNDS)	589.80			
Depth at downstream end (IBDSTRM)	599.90			
Depth at upstream end (RLIGNDS)	**599.92**			
Compute losses due to Change in Direction				
	$h_L= (h_{dir})$			
	$h_{dir} =K_{dir}*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)
V1=	5.99	ft/sec		56.7450173
V2=	4.33	ft/sec		78.5398163
K_{dir}=	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, Appendix C		
g=	32.2			
h_{dir} =	0.117	ft		
Depth upstream of Direction Change (RLIGNDS)	**600.04**			
Compute losses due to Expansion at RLIGNDS				
	$h_L = (h_{expansion})$			
	$h_{expansion} =K_e*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)
V1=	5.99	ft/sec		56.7450173
V2=	4.33	ft/sec		78.5398163
K_e=	0.2	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, pg 43 Table 2-7		
g=	32.2			
$h_{expansion}$ =	0.053	ft		
Depth upstream of expansion (RLIGNDS)	**600.09**			
$h_f=2.87n^2(LV^2/D^{4/3})$				
n=	0.013			
L=	293	Fill in for Q and V will be calculated		
V=	5.99	Q=	340	
D=	8.5			
DEPTH OF FLOW=	10.29	6.28	**56.74501726**	
h_f=	0.294			
INVERT AT DOWNSTREAM END (RLIGNDS)	589.8			
INVERT AT UPSTREAM END (RLIGNUP)	589.96			
Depth at downstream end (RLIGNDS)	600.09			
Depth at upstream end (RLIGNUP)	**600.39**			

Compute losses due to Change in Direction							
	h_L= (h_{dir})						
	h_{dir} =K_{dir}*((V_2^2/2g)-(V_1^2/2g))			Area (ft^2)			
V1=	4.33	ft/sec		78.5398163			
V2=	5.99	ft/sec		56.7450173			
K_{dir}=	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, Appendix C					
g=	32.2						
h_{dir} =	0.117	ft					
Depth upstream of Direction Change (RLIGNUP)	**600.50**						
Compute losses due to Contraction at RLIGNUP							
	h_L = ($h_{contraction}$)						
	$h_{contraction}$ =K_c*((V_2^2/2g)-(V_1^2/2g))			Area (ft^2)			
V1=	4.33	ft/sec		78.5398163			
V2=	5.99	ft/sec		56.7450173			
K_c=	0.1	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, pg 43 Table 2-7					
g=	32.2						
$h_{contraction}$ =	0.027	ft					
Depth upstream of contraction (RLIGNUP)	**600.53**						
h_f=2.87n^2($LV^2/D^{4/3}$)							
n=	0.013						
L=	66	Fill in for Q and V will be calculated					
V=	4.33		Q=	340			
D=	10						
DEPTH OF FLOW=	10.60		6.28	**78.53981627**			
h_f=	0.028						
INVERT AT DOWNSTREAM END (RLIGNUP)	589.93						
INVERT AT UPSTREAM END (IBUPSTM)	589.96						
Depth at downstream end (RLIGNUP)	600.53						
Depth at upstream end (IBUPSTM)	**600.56**						
Compute losses due to Change in Direction							
	h_L= (h_{dir})						
	h_{dir} =K_{dir}*((V_2^2/2g)-(V_1^2/2g))			Area (ft^2)			
V1=	3.41	ft/sec		102.765952			
V2=	4.14	ft/sec		66.0432676			
K_{dir}=	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, Appendix C					
g=	32.2						
h_{dir} =	0.038	ft					
Depth upstream of Direction Change (IBUPSTM)	**600.60**						

Compute losses due to Contraction at IBUPSTM				
	$h_L = (h_{contraction})$			
	$h_{contraction} = K_c*((V_2^2/2g)-(V_1^2/2g))$			Area (ft^2)
V1=	3.41	ft/sec		102.765952
V2=	4.14	ft/sec		66.0432676
K_c=	0.1	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, pg 43 Table 2-7		
g=	32.2			
$h_{contraction}$ =	0.009	ft		
Depth upstream of contraction (IBUPSTM)	**600.61**			
Friction loss between IBUPSTM and EAA325				
$h_f=2.87n^2(LV^2/D^{4/3})$				
n~	Manning's roughness coefficient			
L~	Length of Pipe			
V~	Velocity			
D~	Diameter of Pipe			
n=	0.015			
L=	490.53	Fill in for Q and V will be calculated		
V=	3.31	Q=	340	
D=	11.75			
DEPTH OF FLOW=	10.57	4.99	**102.7659522**	Area
h_f=	0.130			
INVERT AT DOWNSTREAM END (IBUPSTM)	589.96			
INVERT AT UPSTREAM END (EAA325)	589.96			
Depth at downstream end (IBUPSTM)	600.61			
Depth at upstream end (EAA325)	600.74			
Friction loss between EAA325 and EAA330				
$h_f=2.87n^2(LV^2/D^{4/3})$				
n~	Manning's roughness coefficient			
L~	Length of Pipe			
V~	Velocity			
D~	Diameter of Pipe			
n=	0.015			
L=	248.72	Fill in for Q and V will be calculated		
V=	3.33	Q=	340	
D=	11.75			
DEPTH OF FLOW=	10.48	4.94	**102.0755526**	Area
h_f=	0.067			
INVERT AT DOWNSTREAM END (EAA325)	590.26			
INVERT AT UPSTREAM END (EAA330)	590.26			
Depth at downstream end (EAA325)	600.74			
Depth at upstream end (EAA330)	600.80			

Friction loss between EAA330 and EAA335							
$h_f=2.87n^2(LV^2/D^{4/3})$							
n~	Manning's roughness coefficient						
L~	Length of Pipe						
V~	Velocity						
D~	Diameter of Pipe						
n=	0.015						
L=	594	Fill in for Q and V will be calculated					
V=	3.46	Q=	340				
D=	11.75						
DEPTH OF FLOW=	9.99	4.69	**98.27336366**	Area			
h_f=	0.172						
INVERT AT DOWNSTREAM END (EAA330)	590.81						
INVERT AT UPSTREAM END (EAA335)	590.81						
Depth at downstream end (EAA330)	600.80						
Depth at upstream end (EAA335)	600.97						

Number of manholes	7	Invert	Ground	Water Level	Distance upstream	Cumulative Distance upstream	Pipe Crown
Downstream	EAA315	589.31	627.11	599.62	0	0	601.06
Downstream of Expansion	IBDSTRM	589.75	628	599.82	723	723	601.5
Upstream of Expansion	IBDSTRM	589.75	628	599.97	0	723	598.92
Realigned Pipe Section Downstream Expansion	RLIGNDS	589.8	628	600.00	66	789	598.97
Realigned Pipe Section Upstream Expansion	RLIGNDS	589.8	628	600.07	0	789	598.47
Realigned Pipe Section Downstream Contraction	RLIGNUP	589.93	628	600.26	293	1082	598.6
Realigned Pipe Section Upstream Contraction	RLIGNUP	589.93	628	600.32	0	1082	599.1
Downstream of Contraction	IBUPSTM	589.96	628	600.35	66	1148	599.13
Upstream of Contraction	IBUPSTM	589.96	628	600.45	0	1148	601.71
	EAA325	590.26	628.26	600.58	490.53	1638.53	602.01
	EAA330	590.81	629.11	600.65	248.72	1887.25	602.56
Upstream	EAA335	591.48	630.97	600.82	594	2481.25	603.23
Flow Rate through System (cfs)	340	Uses 3% Safety Factor					
Downstream Water Level at EAA315 ()	599.623	Calculate Water Level based on Normal Flow =		595.29	Flow is therefore downstream controlled, use model water level		
Friction loss between IBDSTRM and EAA315							
$h_f=2.87n^2(LV^2/D^{4/3})$							
n~	Manning's roughness coefficient						
L~	Length of Pipe						
V~	Velocity						
D~	Diameter of Pipe						
n=	0.015						
L=	723	Fill in for Q and V will be calculated					
V=	3.37	Q=	340				
D=	11.75						
DEPTH OF FLOW=	10.31	4.85	**100.856473**				
h_f=	0.199						
Slope of Pipe	0.0006						
INVERT AT DOWNSTREAM END (EAA315)	589.31						
INVERT AT UPSTREAM END (IBDSTRM)	589.75						
Depth at downstream end (EAA315)	599.62						
Depth at upstream end (IBDSTRM)	599.82						
Compute losses due to Change in Direction							
	$h_L= (h_{dir})$						
	$h_{dir} =K_{dir}*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)			
V1=	5.15	ft/sec		66.0432676			
V2=	3.37	ft/sec		100.856473			
K_{dir}=1.5*(1-cos(θ)	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, Appendix C Figure (C-9)					
g=	32.2						
$h_{expansion}$ =	0.103	ft					
Depth upstream of Direction Change (IBDSTRM)	**599.93**						

Compute losses due to Expansion at IBDSTRM				
	h_L = ($h_{expansion}$)			
	$h_{expansion}$ =K_e*((V_1^2/2g)-(V_2^2/2g))			Area (ft^2)
V1=	5.15	ft/sec		66.0432676
V2=	3.37	ft/sec		100.856473
K_e=	0.20	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, pg 43 Table 2-7		
g=	32.2			
$h_{expansion}$ =	0.047	ft		
Depth upstream of expansion (IBDSTRM)	**599.97**			
h_f=2.87n^2($LV^2/D^{4/3}$)				
n=	0.011			
L=	66	Fill in for Q and V will be calculated		
V=	5.15	Q=	340	
D=	9.17			
DEPTH OF FLOW=	10.22	6.28	**66.04326757**	
h_f=	0.032			
INVERT AT DOWNSTREAM END (IBDSTRM)	589.75			
INVERT AT UPSTREAM END (RLIGNDS)	589.80			
Depth at downstream end (IBDSTRM)	599.97			
Depth at upstream end (RLIGNDS)	**600.00**			
Compute losses due to Change in Direction				
	h_L= (h_{dir})			
	h_{dir} =K_{dir}*((V_1^2/2g)-(V_2^2/2g))			Area (ft^2)
V1=	5.76	ft/sec		59.037516
V2=	5.15	ft/sec		66.0432676
K_{dir}=	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, Appendix C Figure (C-9)		
g=	32.2			
$h_{expansion}$ =	0.045	ft		
Depth upstream of Direction Change (RLIGNDS)	**600.05**			

Compute losses due to Expansion at RLIGNDS				
	$h_L = (h_{expansion})$			
	$h_{expansion} = K_e*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)
V1=	5.76	ft/sec		59.037516
V2=	5.15	ft/sec		66.0432676
K_e=	0.2	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, pg 43 Table 2-7		
g=	32.2			
$h_{expansion}$ =	0.021	ft		
Depth upstream of expansion (RLIGNDS)	**600.07**			
$h_f=2.87n^2(LV^2/D^{4/3})$				
n=	0.011			
L=	293	Fill in for Q and V will be calculated		
V=	5.76		Q=	340
D=	8.67			
DEPTH OF FLOW=	10.27		6.28	**59.03751595**
h_f=	0.189			
INVERT AT DOWNSTREAM END (RLIGNDS)	589.8			
INVERT AT UPSTREAM END (RLIGNUP)	589.96			
Depth at downstream end (RLIGNDS)	600.07			
Depth at upstream end (RLIGNUP)	**600.26**			
Compute losses due to Change in Direction				
	$h_L = (h_{dir})$			
	$h_{dir} = K_{dir}*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)
V1=	5.15	ft/sec		66.0432676
V2=	5.76	ft/sec		59.037516
K_{dir}=	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, Appendix C Figure (C-9)		
g=	32.2			
h_{dir} =	0.045	ft		
Depth upstream of Direction Change (RLIGNUP)	**600.31**			

Compute losses due to Contraction at RLIGNUP							
	$h_L = (h_{contraction})$						
	$h_{contraction} = K_c*((V_2^2/2g)-(V_1^2/2g))$			Area (ft^2)			
V1=	5.15	ft/sec		66.0432676			
V2=	5.76	ft/sec		59.037516			
K_c=	0.10	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, pg 43 Table 2-7					
g=	32.2						
$h_{contraction}$ =	0.010	ft					
Depth upstream of contraction (RLIGNUP)	**600.32**						
$h_f=2.87n^2(LV^2/D^{4/3})$							
n=	0.011						
L=	66	Fill in for Q and V will be calculated					
V=	5.15		Q=	340			
D=	9.17						
DEPTH OF FLOW=	10.39		6.28	**66.04326757**			
h_f=	0.032						
INVERT AT DOWNSTREAM END (RLIGNUP)	589.93						
INVERT AT UPSTREAM END (IBUPSTM)	589.96						
Depth at downstream end (RLIGNUP)	600.32						
Depth at upstream end (IBUPSTM)	**600.35**						
Compute losses due to Change in Direction							
	$h_L = (h_{dir})$						
	$h_{dir} = K_{dir}*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)			
V1=	3.38	ft/sec		108.434034			
V2=	4.81	ft/sec		66.0432676			
K_{dir}=	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, Appendix C Figure (C-9)					
g=	32.2						
h_{dir} =	0.080	ft					
Depth upstream of Direction Change (IBUPSTM)	**600.43**						
Compute losses due to Contraction at IBUPSTM							
	$h_L = (h_{contraction})$						
	$h_{contraction} = K_c*((V_2^2/2g)-(V_1^2/2g))$			Area (ft^2)			
V1=	3.38	ft/sec		108.434034			
V2=	4.81	ft/sec		66.0432676			
K_c=	0.10	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, pg 43 Table 2-7					
g=	32.2						
$h_{contraction}$ =	0.018	ft					
Depth upstream of contraction (IBUPSTM)	**600.45**						

Friction loss between IBUPSTM and EAA325				
$h_f=2.87n^2(LV^2/D^{4/3})$				
n~	Manning's roughness coefficient			
L~	Length of Pipe			
V~	Velocity			
D~	Diameter of Pipe			
n=	0.015			
L=	490.53	Fill in for Q and V will be calculated		
V=	3.36	Q=	340	
D=	11.75			
DEPTH OF FLOW=	10.36	4.88	**101.1823758**	Area
h_f=	0.134			
INVERT AT DOWNSTREAM END (IBUPSTM)	589.96			
INVERT AT UPSTREAM END (EAA325)	589.96			
Depth at downstream end (IBUPSTM)	600.45			
Depth at upstream end (EAA325)	600.58			
Friction loss between EAA325 and EAA330				
$h_f=2.87n^2(LV^2/D^{4/3})$				
n~	Manning's roughness coefficient			
L~	Length of Pipe			
V~	Velocity			
D~	Diameter of Pipe			
n=	0.015			
L=	248.72	Fill in for Q and V will be calculated		
V=	3.37	Q=	340	
D=	11.75			
DEPTH OF FLOW=	10.32	4.86	**100.9090364**	Area
h_f=	0.068			
INVERT AT DOWNSTREAM END (EAA325)	590.26			
INVERT AT UPSTREAM END (EAA330)	590.26			
Depth at downstream end (EAA325)	600.58			
Depth at upstream end (EAA330)	600.65			
Friction loss between EAA330 and EAA335				
$h_f=2.87n^2(LV^2/D^{4/3})$				
n~	Manning's roughness coefficient			
L~	Length of Pipe			
V~	Velocity			
D~	Diameter of Pipe			
n=	0.015			
L=	594	Fill in for Q and V will be calculated		
V=	3.51	Q=	340	
D=	11.75			
DEPTH OF FLOW=	9.84	4.62	**96.95962669**	Area
h_f=	0.177			
INVERT AT DOWNSTREAM END (EAA330)	590.81			
INVERT AT UPSTREAM END (EAA335)	590.81			
Depth at downstream end (EAA330)	600.65			
Depth at upstream end (EAA335)	600.82			

Number of manholes	8	Invert	Ground	Water Level	Distance upstream	Cumulative Distance upstream	Pipe Crown
Downstream	EAA315	589.31	627.11	599.62	0	0	601.06
Downstream of Expansion	IBDSTRM	589.75	628	599.82	723	723	601.5
Upstream of Expansion	IBDSTRM	589.75	628	599.90	0	723	599.75
Realigned Pipe Section Downstream Expansion	RLIGNDS	589.8	628	599.92	66	789	599.8
Realigned Pipe Section Upstream Expansion	RLIGNDS	589.8	628	600.05	0	789	597.8
Realigned Pipe Section Downstream Contraction	RLIGNUP	589.93	628	600.15	293	1082	597.93
Realigned Pipe Section Upstream Contraction	RLIGNUP	589.93	628	600.27	0	1082	599.93
Downstream of Contraction	IBUPSTM	589.96	628	600.30	66	1148	599.96
Upstream of Contraction	IBUPSTM	589.96	628	600.36	0	1148	601.71
	EAA325	590.26	628.26	600.50	490.53	1638.53	602.01
	EAA330	590.81	629.11	600.57	248.72	1887.25	602.56
Upstream	EAA335	591.48	630.97	600.74	594	2481.25	603.23
Flow Rate through System (cfs)	340	Uses 3% Safety Factor					
Downstream Water Level at EAA315 ()	599.623	Calculate Water Level based on Normal Flow =		595.29	Flow is therefore downstream controlled, use model water level		
Friction loss between IBDSTRM and EAA315							
$h_f=2.87n^2(LV^2/D^{4/3})$		From Hydraulics (King)					
n~	Manning's roughness coefficient						
L~	Length of Pipe						
V~	Velocity						
D~	Diameter of Pipe						
n=	0.015						
L=	723	Fill in for Q and V will be calculated					
V=	3.37	Q=	340				
D=	11.75						
DEPTH OF FLOW=	10.31	4.85	**100.856473**				
h_f=	0.199						
Slope of Pipe	0.0006						
INVERT AT DOWNSTREAM END (EAA315)	589.31						
INVERT AT UPSTREAM END (IBDSTRM)	589.75						
Depth at downstream end (EAA315)	599.62						
Depth at upstream end (IBDSTRM)	599.82						
Compute losses due to Change in Direction							
	$h_L= (h_{dir})$						
	$h_{dir} =K_{dir}*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)			
V1=	4.33	ft/sec		78.5398163			
V2=	3.37	ft/sec		100.856473			
K_{dir}=	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, Appendix C Figure (C-9)					
g=	32.2						
$h_{expansion}$ =	0.050	ft					
Depth upstream of Direction Change (IBDSTRM)	**599.87**						
Compute losses due to Expansion at IBDSTRM							
	$h_L = (h_{expansion})$						
	$h_{expansion} =K_e*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)			
V1=	4.33	ft/sec		78.5398163			
V2=	3.37	ft/sec		100.856473			
K_e=	0.2	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, pg 43 Table 2-7					
g=	32.2						
$h_{expansion}$ =	0.023	ft					
Depth upstream of expansion (IBDSTRM)	**599.90**						

h_f=2.87n^2(LV^2/$D^{4/3}$)							
n=	0.013						
L=	66	Fill in for Q and V will be calculated					
V=	4.33		Q=	340			
D=	10						
DEPTH OF FLOW=	10.15		6.28	**78.53981627**			
h_f=	0.028						
INVERT AT DOWNSTREAM END (IBDSTRM)	589.75						
INVERT AT UPSTREAM END (RLIGNDS)	589.80						
Depth at downstream end (IBDSTRM)	599.90						
Depth at upstream end (RLIGNDS)	**599.92**						
Compute losses due to Change in Direction							
	h_L= (h_{dir})						
	h_{dir} =K_{dir}*((V_2^2/2g)-(V_1^2/2g))			Area (ft^2)			
V1=	3.38	ft/sec		50.2654825			
V2=	4.33	ft/sec		78.5398163			
K_{dir}=	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf					
g=	32.2	and Eddy, 1981, Appendix C Figure (C-9)					
h_{dir} =	0.050	ft					
Depth upstream of Direction Change (RLIGNDS)	**599.97**						
Compute losses due to Contraction at RLIGNDS							
	h_L = ($h_{contraction}$)						
	$h_{contraction}$ =K_c*((V_2^2/2g)-(V_1^2/2g))			Area (ft^2)			
V1=	3.38	ft/sec		50.2654825			
V2=	4.33	ft/sec		78.5398163			
K_e=	0.7	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf					
g=	32.2	and Eddy, 1981, pg 43 Table 2-7					
$h_{contraction}$ =	0.079	ft					
Depth upstream of expansion (RLIGNDS)	**600.05**						
h_f=2.87n^2(LV^2/$D^{4/3}$)							
n=	0.013						
L=	293	Fill in for Q and V will be calculated					
V=	3.38		Q=	170			
D=	8						
DEPTH OF FLOW=	10.25		6.28	**50.26548241**			
h_f=	0.102						
INVERT AT DOWNSTREAM END (RLIGNDS)	589.8						
INVERT AT UPSTREAM END (RLIGNUP)	589.96						
Depth at downstream end (RLIGNDS)	600.05						
Depth at upstream end (RLIGNUP)	**600.15**						
Compute losses due to Change in Direction							
	h_L= (h_{dir})						
	h_{dir} =K_{dir}*((V_1^2/2g)-(V_2^2/2g))			Area (ft^2)			
V_1=	4.33	ft/sec		78.5398163			
V_2=	3.38	ft/sec		50.2654825			
K_{dir}=	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf					
g=	32.2	and Eddy, 1981, Appendix C Figure (C-9)					

h_{dir} =	0.050	ft		
Depth upstream of Direction Change (RLIGNUP)	**600.20**			
Compute losses due to Expansion at RLIGNUP				
	$h_L = (h_{expansion})$			
	$h_{expansion} = K_e*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)
V1=	4.33	ft/sec		78.5398163
V2=	3.38	ft/sec		50.2654825
K_c=	0.6	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, pg 43 Table 2-7		
g=	32.2			
$h_{expansion}$ =	0.068	ft		
Depth upstream of contraction (RLIGNUP)	**600.27**			
$h_f=2.87n^2(LV^2/D^{4/3})$				
n=	0.013			
L=	66	Fill in for Q and V will be calculated		
V=	4.33	Q=	340	
D=	10			
DEPTH OF FLOW=	10.34	6.28	**78.53981627**	
h_f=	0.028			
INVERT AT DOWNSTREAM END (RLIGNUP)	589.93			
INVERT AT UPSTREAM END (IBUPSTM)	589.96			
Depth at downstream end (RLIGNUP)	600.27			
Depth at upstream end (IBUPSTM)	**600.30**			
Compute losses due to Change in Direction				
	$h_L = (h_{dir})$			
	$h_{dir} = K_{dir}*((V_1^2/2g)-(V_2^2/2g))$			Area (ft^2)
V1=	3.37	ft/sec		100.848276
V2=	4.33	ft/sec		78.5398163
K_{dir}=	0.44	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, Appendix C Figure (C-9)		
g=	32.2			
h_{dir} =	0.050	ft		
Depth upstream of Direction Change (IBUPSTM)	**600.35**			
Compute losses due to Contraction at IBUPSTM				
	$h_L = (h_{contraction})$			
	$h_{contraction} = K_c*((V_2^2/2g)-(V_1^2/2g))$			Area (ft^2)
V1=	3.37	ft/sec		100.848276
V2=	4.33	ft/sec		78.5398163
K_c=	0.1	From - Wastewater Engineering: Collection and Pumping of Wastwater, Metcalf and Eddy, 1981, pg 43 Table 2-7		
g=	32.2			
$h_{contraction}$ =	0.011	ft		
Depth upstream of contraction (IBUPSTM)	**600.36**			
Friction loss between IBUPSTM and EAA325				
$h_f=2.87n^2(LV^2/D^{4/3})$				
n~	Manning's roughness coefficient			
L~	Length of Pipe			
V~	Velocity			

D~	Diameter of Pipe			
n=	0.015			
L=	490.53	Fill in for Q and V will be calculated		
V=	3.37	Q=	340	
D=	11.75			
DEPTH OF FLOW=	10.31	4.85	**100.848276**	Area
h_f=	0.135			
INVERT AT DOWNSTREAM END (IBUPSTM)	589.96			
INVERT AT UPSTREAM END (EAA325)	589.96			
Depth at downstream end (IBUPSTM)	600.36			
Depth at upstream end (EAA325)	600.50			
Friction loss between EAA325 and EAA330				
$h_f=2.87n^2(LV^2/D^{4/3})$				
n~	Manning's roughness coefficient			
L~	Length of Pipe			
V~	Velocity			
D~	Diameter of Pipe			
n=	0.015			
L=	248.72	Fill in for Q and V will be calculated		
V=	3.39	Q=	340	
D=	11.75			
DEPTH OF FLOW=	10.24	4.81	**100.259402**	Area
h_f=	0.069			
INVERT AT DOWNSTREAM END (EAA325)	590.26			
INVERT AT UPSTREAM END (EAA330)	590.26			
Depth at downstream end (EAA325)	600.50			
Depth at upstream end (EAA330)	600.57			
Friction loss between EAA330 and EAA335				
$h_f=2.87n^2(LV^2/D^{4/3})$				
n~	Manning's roughness coefficient			
L~	Length of Pipe			
V~	Velocity			
D~	Diameter of Pipe			
n=	0.015			
L=	594	Fill in for Q and V will be calculated		
V=	3.53	Q=	340	
D=	11.75			
DEPTH OF FLOW=	9.76	4.58	**96.23676423**	Area
h_f=	0.179			
INVERT AT DOWNSTREAM END (EAA330)	590.81			
INVERT AT UPSTREAM END (EAA335)	590.81			
Depth at downstream end (EAA330)	600.57			
Depth at upstream end (EAA335)	600.74			