**Storm Water Addendum to the Fox Run Redevelopment SWMP
(original SWMP was prepared by Jahnke & Jahnke Associates LLC, last revised
June 5, 2020)**

Re: Fox Den Apartments
City of Waukesha, Waukesha County
Date: November 18, 2020
By: Anthony S. Zanon, PE

Fox Den Apartments is a proposed three building, 72-unit apartment project to be located on Lot 5 of Certified Survey Map No. 12027. The lot is 4.5 acres in size and is part of the Fox Run Redevelopment in the Southeast Quarter (SE 1/4) of Section 8 and the Southwest Quarter (SW 1/4) of Section 9, Town 6 North, Range 19 East, City of Waukesha, Waukesha County. As part of the overall Fox Run Redevelopment, Jahnke & Jahnke Associates LLC (Jahnke) completed a storm water management plan (SWMP) for the entire development. Lot 5 is part of that SWMP. This storm water addendum is being prepared to provide the on-site storm sewer calculations and hydraulic grade lines for said on-site storm sewer. The storm sewer design and hydraulic grade lines were analyzed using a 100-year storm event and tailwater elevations. The tailwater elevations were the elevation provided from the City of Waukesha for a downstream flooding issue at Badger Drive (elevation 24.6) and the peak water elevation from the on-site storm water management pond. The Badger Drive tailwater was used for the storm draining directly off-site. The storm water pond was used for the on-site storm sewer draining to the pond. These calculations correspond to the civil plan set last revised November 18, 2020.

Attached to this letter is the following:

Attachment A: Storm Sewer Computations
Attachment B: Storm Sewer Drainage Map
Attachment C: Hydraulic Grade Line Calculations
Attachment D: Storm Sewer Manhole Sizing
Attachment E: Geotechnical Report

ATTACHMENT A

STORM SEWER COMPUTATIONS

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STORM SEWER COMPUTATIONS
FOR
FOX RUN
WAUKESHA, WI

DESIGN BY: EJM
PROJECT NUMBER: 2206.00
DATE: 11/18/2020

DESIGN DATA

County: Waukesha | **Design Storm:** 100 yr | **Storm Duration:** 5 min | **DESIGN INTENSITY (I):** 10.75 in/hr | Intensity calculated using SEWRPC IDF equations.

STRUCTURE DATA			DRAINAGE AREA AND FLOW DATA (Flow is determined by Rational Method Q = CIA)				PIPE DATA				PIPE CAPACITY INFORMATION (Pipe capacity is determined by Manning's Equation $Q = 1.486/n\ AR^{2/3}\ S^{1/2}$)					ELEVATIONS		
Notes	Upstream Structure	Downstream Structure	Individual Acres A	Individual Coefficient C	Individual Flow Q (cfs)	Cumulative Flow (cfs)	Length (ft)	Diameter (in)	Slope (%)	Manning Coefficient	Required Drop (ft)	Actual Drop (ft)	Percent Full (%)	Actual Velocity (fps)	Max. Capacity (cfs)	Rim/Toc Up	Invert Up	Invert Down
WEST	TD 7.1	MH 7.0	0.18	0.61	1.18	1.18	10.00	12	0.26	0.012	0.01	0.03	57%	2.62	2.12	29.00	26.53	26.50
	MH 7.0	MH 8.0	0.00	0.00	0.00	1.18	81.56	12	0.26	0.012	0.08	0.21	57%	2.62	2.12	29.90	26.50	26.29
	BC 8.1	MH 8.0	0.10	0.90	0.97	0.97	28.85	6	4.16	0.012	0.73	1.20	71%	6.98	1.33	36.60	27.99	26.79
	MH 8.0	MH 9.0	0.00	0.00	0.00	2.15	50.78	15	0.26	0.012	0.05	0.13	58%	3.04	3.84	34.50	26.04	25.91
	BC 9.1	MH 9.0	0.11	0.90	1.06	1.06	32.26	6	4.16	0.011	0.83	1.34	71%	7.63	1.45	36.60	28.00	26.66
	MH 9.0	EX INLET 3	0.00	0.00	0.00	3.21	39.00	15	0.50	0.013	0.10	0.20	65%	4.03	4.91	33.30	25.91	25.71
	EX INLET 3	EX INLET 4	0.37	0.58	2.31	5.52	50.00	15	0.50	0.013	0.36	0.25	Surcharge	---	4.91	29.31	25.61	25.36
	MH IN 10.0	MH 13.0	0.44	0.51	2.41	2.41	169.91	12	0.50	0.012	0.66	0.85	78%	3.92	2.94	31.96	28.18	27.33
	TD 11.0	MH IN 12.0	0.09	0.75	0.73	0.73	8.09	12	0.26	0.012	0.00	0.02	40%	2.32	2.12	30.50	27.43	27.41
	MH IN 12.0	MH 13.0	0.01	0.20	0.02	0.75	30.75	12	0.26	0.012	0.01	0.08	41%	2.33	2.12	30.50	27.41	27.33
	MH 13.0	TEE 14.0	0.00	0.00	0.00	3.16	8.82	18	0.26	0.012	0.01	0.02	53%	3.35	6.24	35.50	26.83	26.81
	BC 14.1	TEE 14.0	0.11	0.90	1.06	1.06	8.27	6	4.16	0.011	0.21	0.34	71%	7.63	1.45	36.60	27.65	27.31
	TEE 14.0	TEE 15.0	0.00	0.00	0.00	4.22	87.79	18	0.26	0.012	0.12	0.23	67%	3.58	6.24	35.90	26.81	26.58
	BC 15.1	TEE 15.0	0.09	0.90	0.87	0.87	8.27	6	4.16	0.011	0.14	0.34	61%	7.32	1.45	36.60	27.42	27.08
	TEE 15.0	TEE 16.0	0.00	0.00	0.00	5.09	62.54	18	0.26	0.012	0.13	0.16	78%	3.70	6.24	34.30	26.58	26.42
	BC 16.1	TEE 16.0	0.09	0.90	0.87	0.87	8.27	6	4.16	0.011	0.14	0.34	61%	7.32	1.45	36.60	27.26	26.92
	TEE 16.0	MH 17.0	0.00	0.00	0.00	5.97	78.92	18	0.26	0.012	0.22	0.21	90%	3.74	6.24	35.10	26.42	26.21
	BC 17.1	MH 17.0	0.10	0.90	0.97	0.97	8.27	6	4.16	0.011	0.18	0.34	66%	7.49	1.45	36.60	27.05	26.71
	MH 17.0	MH 18.0	0.00	0.00	0.00	6.93	18.87	18	0.50	0.012	0.07	0.09	77%	5.12	8.66	32.00	26.21	26.12
	MH 18.0	EX INLET 1	0.00	0.00	0.00	6.93	20.23	18	0.50	0.012	0.08	0.10	77%	5.12	8.66	31.05	26.12	26.02
	EX INLET 1	EX INLET 2	0.04	0.57	0.25	7.18	52.35	18	0.52	0.013	0.24	0.27	83%	4.88	8.15	30.49	25.92	25.65
*	EX INLET 2	MH IN 22.0	0.49	0.41	2.16	9.34	75.88	21	0.49	0.013	0.26	0.37	75%	5.17	11.93	30.33	25.40	25.03
	TD 19.0	MH 21.0	0.26	0.60	1.68	1.68	54.45	12	0.26	0.013	0.12	0.14	81%	2.63	1.95	29.00	26.53	26.39
	TD 20.0	MH 21.0	0.09	0.60	0.58	0.58	12.75	12	0.50	0.013	0.00	0.06	28%	2.61	2.71	29.00	26.45	26.39
	MH 21.0	MH IN 22.0	0.00	0.00	0.00	2.26	121.35	12	0.50	0.013	0.49	0.61	79%	3.63	2.71	29.50	26.39	25.78
	MH IN 22.0	EX INLET 4	0.00	0.00	0.00	11.59	34.01	21	0.49	0.013	0.18	0.17	92%	5.23	11.93	29.63	25.03	24.86
*	EX INLET 4	EX OUTPIPE 1	0.28	0.48	1.44	18.56	11.10	24	0.50	0.013	0.07	0.06	Surcharge	---	17.21	29.31	24.61	24.55

* AREAS AND C-VALUES CALCUATED FROM THE JAHNKE & JAHNKE STORM SEWER COMPUTATIONS SPREADSHEET DATED 6-22-20

NOTE: EXISTING PIPES IN FOX RUN BOULEVARD WERE NOT SIZED FOR THE 100-YEAR STORM. SURCHARGED PIPES HAVE OVERLAND FLOW TO POND.

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STORM SEWER COMPUTATIONS
FOR
FOX RUN

WAUKESHA, WI

DESIGN BY: EJM

PROJECT NUMBER: 2206.00

DATE: 11/18/2020

DESIGN DATA

County: Waukesha | **Design Storm:** 100 yr | **Storm Duration:** 5 min | **DESIGN INTENSITY (I):** 10.75 in/hr | Intensity calculated using SEWRPC IDF equations.

STRUCTURE DATA			DRAINAGE AREA AND FLOW DATA Flow is determined by Rational Method Q = CIA				PIPE DATA				PIPE CAPACITY INFORMATION Pipe capacity is determined by Manning's Equation $Q = 1.486/n\ AR^{2/3}\ S^{1/2}$					ELEVATIONS		
Notes	Upstream Structure	Downstream Structure	Individual Acres A	Individual Coefficient C	Individual Flow Q (cfs)	Cumulative Flow (cfs)	Length (ft)	Diameter (in)	Slope (%)	Manning Coefficient	Required Drop (ft)	Actual Drop (ft)	Percent Full (%)	Actual Velocity (fps)	Max. Capacity (cfs)	Rim/Toc Up	Invert Up	Invert Down
EAST																		
	CB 1.0	MH IN 2.0	0.58	0.49	3.06	3.06	205.20	15	0.50	0.012	0.39	1.03	59%	4.24	5.32	31.96	27.92	26.89
	MH IN 2.0	MH IN 3.0	0.09	0.20	0.19	3.25	146.00	15	0.50	0.012	0.31	0.73	62%	4.30	5.32	31.50	26.89	26.16
	BC 6.1	MH 6.0	0.11	0.90	1.06	1.06	15.61	6	4.16	0.011	0.40	0.65	71%	7.63	1.45	36.60	30.04	29.39
	MH 6.0	TEE 5.0	0.00	0.00	0.00	1.06	60.21	12	1.50	0.012	0.05	0.90	28%	4.86	5.08	32.20	29.14	28.23
	BC 5.1	TEE 5.0	0.09	0.90	0.87	0.87	14.69	6	4.16	0.011	0.25	0.61	61%	7.32	1.45	36.60	29.09	28.48
	TEE 5.0	TEE 4.0	0.00	0.00	0.00	1.94	65.13	12	1.50	0.012	0.16	0.98	43%	5.72	5.08	32.40	28.23	27.26
	BC 4.1	TEE 4.0	0.10	0.90	0.97	0.97	13.85	6	4.16	0.011	0.29	0.58	66%	7.49	1.45	36.60	28.09	27.51
	TEE 4.0	MH IN 3.0	0.00	0.00	0.00	2.90	56.44	12	1.50	0.012	0.32	0.85	58%	6.32	5.08	34.10	27.26	26.41
	MH IN 3.0	EX INLET 7	0.00	0.00	0.00	6.15	19.53	15	0.50	0.013	0.18	0.10	Surcharge	---	4.91	30.10	26.16	26.06
	EX INLET 7	EX INLET 8	0.38	0.56	2.29	8.44	50.00	18	0.50	0.013	0.32	0.25	Surcharge	---	7.99	29.65	25.81	25.56
*	EX INLET 8	EX INLET 6	0.16	0.77	1.32	9.76	194.27	18	0.50	0.013	1.68	0.97	Surcharge	---	7.99	29.65	25.31	24.34
	EX INLET 5	EX INLET 6	0.28	0.61	1.84	1.84	50.00	12	0.50	0.013	0.13	0.25	67%	3.50	2.71	30.31	27.31	27.06
*	EX INLET 6	EX OUTPIPE 2	0.31	0.73	2.43	14.03	17.00	21	0.50	0.013	0.13	0.09	Surcharge	---	12.05	30.31	24.09	24.00
OFF																		
	TD 23.0	EX MH 1	0.06	0.72	0.46	0.46	44.87	12	0.26	0.013	0.01	0.12	31%	1.94	1.95	26.35	19.84	19.72
	TD 24.0	MH 25.0	0.07	0.68	0.51	0.51	24.50	12	0.75	0.013	0.01	0.18	22%	2.91	3.32	26.35	21.93	21.75
	MH 25.0	EX MH 2	0.00	0.00	0.00	0.51	143.42	12	0.75	0.013	0.03	1.08	22%	2.91	3.32	28.50	21.75	20.67
	TD 26.0	EX MH 3	0.02	0.90	0.19	0.19	57.26	12	0.26	0.013	0.00	0.15	16%	1.51	1.95	26.35	21.63	21.48

* AREAS AND C-VALUES CALCUATED FROM THE JAHNKE & JAHNKE STORM SEWER COMPUTATIONS SPREADSHEET DATED 6-22-20

NOTE: EXISTING PIPES IN FOX RUN BOULEVARD WERE NOT SIZED FOR THE 100-YEAR STORM. SURCHARGED PIPES HAVE OVERLAND FLOW TO POND.

ATTACHMENT B

STORM SEWER DRAINAGE MAP

PINNACLE ENGINEERING GROUP

STORM SEWER DRAINAGE AREAS

11-18-20

PINNACLE ENGINEERING GROUP

20725 WATERTOWN ROAD | SUITE 100 | BROOKFIELD, WI 53186 | WWW.PINNACLE-ENGR.COM |

PLAN | DESIGN | DELIVER

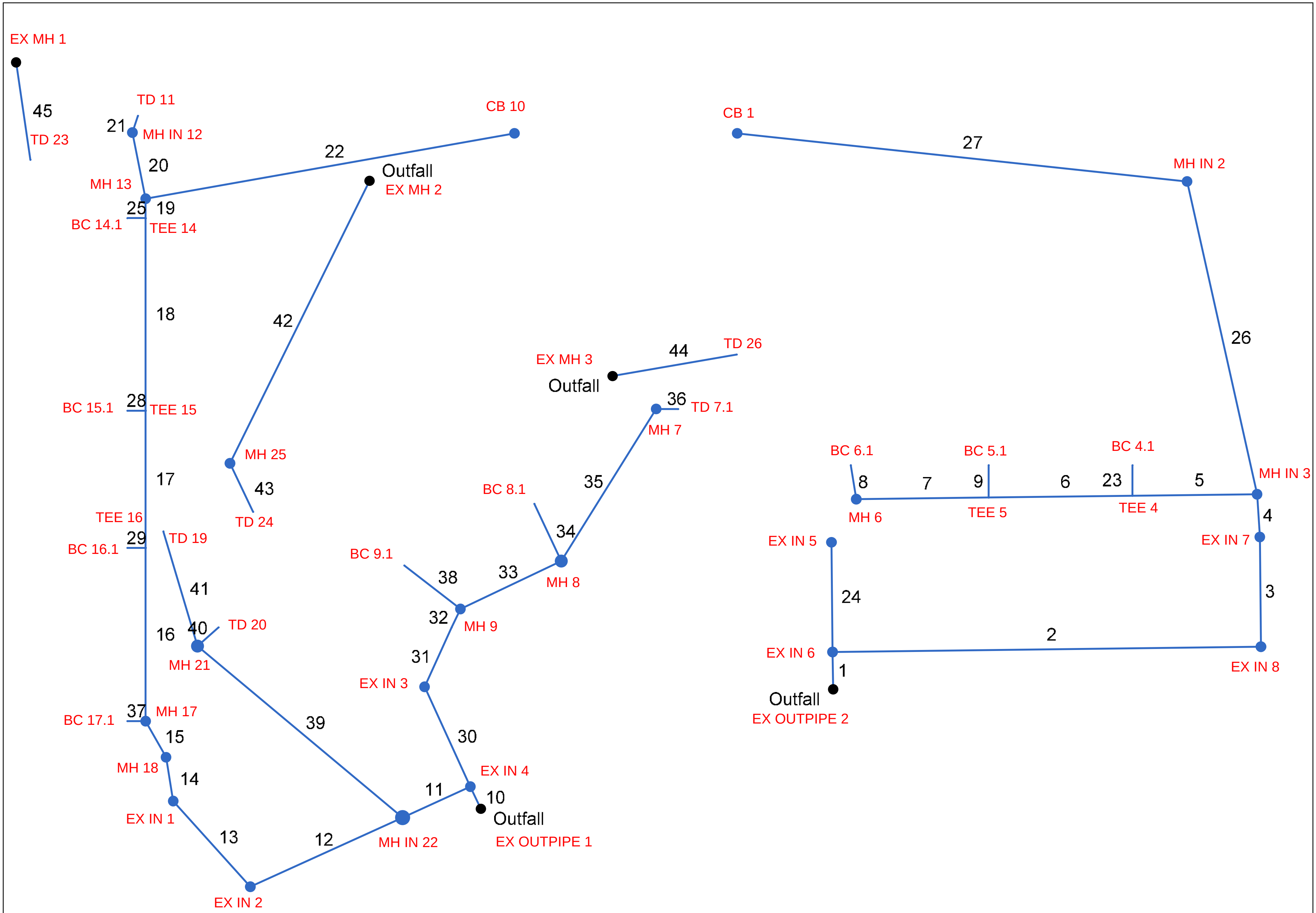
PEG JOB# 2206.00

ATTACHMENT C

HYDRAULIC GRADE LINE CALCULATIONS

PINNACLE ENGINEERING GROUP

Hydraflow Storm Sewers Extension for Autodesk® Civil 3D® Plan



Project File: New.stm	Number of lines: 45	Date: 11/18/2020

Storm Sewers v2019.20

Storm Sewer Inventory Report

Line No.	Alignment				Flow Data				Physical Data								Line ID
	Dnstr Line No.	Line Length (ft)	Defl angle (deg)	Junc Type	Known Q (cfs)	Drng Area (ac)	Runoff Coeff (C)	Inlet Time (min)	Invert El Dn (ft)	Line Slope (%)	Invert El Up (ft)	Line Size (in)	Line Shape	N Value (n)	J-Loss Coeff (K)	Inlet/ Rim El (ft)	
1	End	17.040	-90.707	MH	2.43	0.00	0.00	0.0	24.00	0.53	24.09	21	Cir	0.012	1.00	30.92	Pipe - (509)
2	1	194.270	90.000	MH	1.32	0.00	0.00	0.0	24.34	0.50	25.31	18	Cir	0.012	1.00	30.10	Pipe - (511)
3	2	50.000	-90.000	MH	2.29	0.00	0.00	0.0	25.56	0.50	25.81	18	Cir	0.012	0.15	30.35	Pipe - (512)
4	3	19.529	-3.096	MH	0.00	0.00	0.00	0.0	26.06	0.50	26.16	15	Cir	0.012	1.00	30.10	Pipe - (513)
5	4	56.441	-86.904	None	0.00	0.00	0.00	0.0	26.41	1.51	27.26	12	Cir	0.012	1.00	34.10	Pipe - (518)
6	5	65.130	0.000	None	0.00	0.00	0.00	0.0	27.26	1.49	28.23	12	Cir	0.012	1.00	32.40	Pipe - (517)
7	6	60.213	0.000	MH	0.00	0.00	0.00	0.0	28.23	1.51	29.14	12	Cir	0.012	0.99	32.20	Pipe - (551)
8	7	15.610	82.110	None	1.06	0.00	0.00	0.0	29.39	4.16	30.04	6	Cir	0.012	1.00	36.60	Pipe - (521)
9	6	14.692	90.707	None	0.87	0.00	0.00	0.0	28.48	4.15	29.09	6	Cir	0.012	1.00	36.60	Pipe - (516)
10	End	11.134	-114.440	MH	1.44	0.00	0.00	0.0	24.55	0.54	24.61	24	Cir	0.012	1.00	29.31	Pipe - (522)
11	10	34.014	-90.097	MH	0.00	0.00	0.00	0.0	24.86	0.49	25.03	21	Cir	0.012	0.92	29.63	Pipe - (523) (1)
12	11	75.876	0.000	MH	2.16	0.00	0.00	0.0	25.03	0.49	25.40	21	Cir	0.012	0.96	30.33	Pipe - (523)
13	12	52.349	72.757	MH	0.25	0.00	0.00	0.0	25.65	0.52	25.92	18	Cir	0.012	0.59	30.56	Pipe - (524)
14	13	20.234	32.679	MH	0.00	0.00	0.00	0.0	26.02	0.50	26.12	18	Cir	0.012	0.40	31.05	Pipe - (526)
15	14	18.868	-20.348	MH	0.00	0.00	0.00	0.0	26.12	0.48	26.21	18	Cir	0.012	0.89	32.00	Pipe - (527)
16	15	78.917	29.448	None	0.00	0.00	0.00	0.0	26.21	0.27	26.42	18	Cir	0.012	1.00	35.10	Pipe - (534)
17	16	62.542	0.000	None	0.00	0.00	0.00	0.0	26.42	0.26	26.58	18	Cir	0.012	1.00	34.30	Pipe - (533)
18	17	87.792	0.000	None	0.00	0.00	0.00	0.0	26.58	0.26	26.81	18	Cir	0.012	1.00	35.90	Pipe - (532) (1)
19	18	8.822	0.000	MH	0.00	0.00	0.00	0.0	26.81	0.26	26.83	18	Cir	0.012	0.99	35.50	Pipe - (532)
20	19	30.751	-11.478	MH	0.02	0.00	0.00	0.0	27.33	0.26	27.41	12	Cir	0.012	0.57	30.50	Pipe - (531)
21	20	8.093	30.739	None	0.73	0.00	0.00	0.0	27.41	0.25	27.43	12	Cir	0.012	1.00	30.50	Pipe - (530)
22	19	169.905	79.909	MH	2.41	0.00	0.00	0.0	27.33	0.50	28.18	12	Cir	0.012	1.00	31.96	Pipe - (529)
23	5	13.854	90.707	None	0.97	0.00	0.00	0.0	27.51	4.19	28.09	6	Cir	0.012	1.00	36.60	Pipe - (519)

Project File: New.stm	Number of lines: 45	Date: 11/18/2020

Storm Sewer Inventory Report

Line No.	Alignment				Flow Data				Physical Data								Line ID
	Dnstr Line No.	Line Length (ft)	Defl angle (deg)	Junc Type	Known Q (cfs)	Drng Area (ac)	Runoff Coeff (C)	Inlet Time (min)	Invert El Dn (ft)	Line Slope (%)	Invert El Up (ft)	Line Size (in)	Line Shape	N Value (n)	J-Loss Coeff (K)	Inlet/ Rim El (ft)	
24	1	50.000	0.000	MH	1.84	0.00	0.00	0.0	27.06	0.50	27.31	12	Cir	0.012	1.00	31.31	Pipe - (520)
25	18	8.271	-90.000	None	1.06	0.00	0.00	0.0	27.31	4.11	27.65	6	Cir	0.012	1.00	36.60	Pipe - (539)
26	4	146.000	-8.749	MH	0.19	0.00	0.00	0.0	26.16	0.50	26.89	15	Cir	0.012	0.96	31.50	Pipe - (514)
27	26	205.204	-71.316	MH	3.06	0.00	0.00	0.0	26.89	0.50	27.92	15	Cir	0.012	1.00	31.96	Pipe - (515)
28	17	8.271	-90.000	None	0.87	0.00	0.00	0.0	27.08	4.11	27.42	6	Cir	0.012	1.00	36.60	Pipe - (538)
29	16	8.271	-90.000	None	0.87	0.00	0.00	0.0	26.92	4.11	27.26	6	Cir	0.012	1.00	36.60	Pipe - (537)
30	10	49.992	-0.137	MH	2.31	0.00	0.00	0.0	25.36	0.50	25.61	15	Cir	0.012	0.79	29.31	Pipe - (525)
31	30	29.716	49.021	None	0.00	0.00	0.00	0.0	25.71	0.50	25.86	15	Cir	0.012	0.15	31.00	Pipe - (540)
32	31	9.263	0.018	MH	0.00	0.00	0.00	0.0	25.86	0.54	25.91	15	Cir	0.012	0.98	33.30	Pipe - (541)
33	32	50.777	40.074	MH	0.00	0.00	0.00	0.0	25.91	0.26	26.04	15	Cir	0.012	1.00	34.50	Pipe - (544)
34	33	28.851	-90.000	None	0.97	0.00	0.00	0.0	26.79	4.16	27.99	6	Cir	0.012	1.00	36.60	Pipe - (543)
35	33	81.562	-32.687	MH	0.00	0.00	0.00	0.0	26.29	0.26	26.50	12	Cir	0.012	0.88	29.90	Pipe - (549)
36	35	10.000	58.187	None	1.18	0.00	0.00	0.0	26.50	0.30	26.53	12	Cir	0.012	1.00	29.00	Pipe - (550)
37	15	8.271	-60.552	None	0.97	0.00	0.00	0.0	26.71	4.11	27.05	6	Cir	0.012	1.00	36.60	Pipe - (536)
38	32	32.259	-76.501	None	1.06	0.00	0.00	0.0	26.66	4.15	28.00	6	Cir	0.012	1.00	36.60	Pipe - (542)
39	11	121.349	64.630	MH	0.00	0.00	0.00	0.0	25.78	0.50	26.39	12	Cir	0.012	1.00	29.50	Pipe - (553)
40	39	12.753	97.850	None	0.58	0.00	0.00	0.0	26.39	0.47	26.45	12	Cir	0.012	1.00	29.00	Pipe - (554)
41	39	54.449	33.346	None	1.68	0.00	0.00	0.0	26.39	0.26	26.53	12	Cir	0.012	1.00	29.00	Pipe - (552)
42	End	143.418	116.177	MH	0.00	0.00	0.00	0.0	20.67	0.75	21.75	12	Cir	0.012	0.82	31.00	Pipe - (547)
43	42	24.500	-51.676	None	0.51	0.00	0.00	0.0	21.75	0.73	21.93	12	Cir	0.012	1.00	26.35	Pipe - (546)
44	End	57.261	-9.915	None	0.19	0.00	0.00	0.0	21.48	0.26	21.63	12	Cir	0.012	1.00	26.35	Pipe - (545)
45	End	44.870	81.723	None	0.46	0.00	0.00	0.0	19.72	0.27	19.84	12	Cir	0.012	1.00	26.35	Pipe - (548)

Project File: New.stm	Number of lines: 45	Date: 11/18/2020

Structure Report

Struct No.	Structure ID	Junction Type	Rim Elev (ft)	Structure Shape	Structure Length (ft)	Structure Width (ft)	Line Out Size (in)	Line Out Shape	Line Out Invert (ft)	Line In Size (in)	Line In Shape	Line In Invert (ft)
1	EX INLET 6	Manhole	30.92	Cir	4.00	4.00	21	Cir	24.09	18 12	Cir Cir	24.34 27.06
2	EX INLET 8	Manhole	30.10	Cir	4.00	4.00	18	Cir	25.31	18	Cir	25.56
3	EX INLET 7	Manhole	30.35	Cir	4.00	4.00	18	Cir	25.81	15	Cir	26.06
4	PROP STORM MH IN 3	Manhole	30.10	Cir	4.00	4.00	15	Cir	26.16	12 15	Cir Cir	26.41 26.16
5	TEE 4	None	34.10	n/a	n/a	n/a	12	Cir	27.26	12 6	Cir Cir	27.26 27.51
6	TEE 5	None	32.40	n/a	n/a	n/a	12	Cir	28.23	12 6	Cir Cir	28.23 28.48
7	PROP STORM MH 6	Manhole	32.20	Cir	4.00	4.00	12	Cir	29.14	6	Cir	29.39
8	BC 6.1	None	36.60	n/a	n/a	n/a	6	Cir	30.04			
9	BC 5.1	None	36.60	n/a	n/a	n/a	6	Cir	29.09			
10	EX INLET 4	Manhole	29.31	Cir	4.00	4.00	24	Cir	24.61	21 15	Cir Cir	24.86 25.36
11	PROP STORM MH IN 22	Manhole	29.63	Cir	6.00	6.00	21	Cir	25.03	21 12	Cir Cir	25.03 25.78
12	EX INLET 2	Manhole	30.33	Cir	4.00	4.00	21	Cir	25.40	18	Cir	25.65
13	EX INLET 1	Manhole	30.56	Cir	4.00	4.00	18	Cir	25.92	18	Cir	26.02
14	PROP STORM MH 18	Manhole	31.05	Cir	4.00	4.00	18	Cir	26.12	18	Cir	26.12
15	PROP STORM MH 17	Manhole	32.00	Cir	4.00	4.00	18	Cir	26.21	18 6	Cir Cir	26.21 26.71
16	TEE 16	None	35.10	n/a	n/a	n/a	18	Cir	26.42	18 6	Cir Cir	26.42 26.92
17	TEE 15	None	34.30	n/a	n/a	n/a	18	Cir	26.58	18 6	Cir Cir	26.58 27.08
18	TEE 14	None	35.90	n/a	n/a	n/a	18	Cir	26.81	18 6	Cir Cir	26.81 27.31

Project File: New.stm	Number of Structures: 45	Run Date: 11/18/2020

Structure Report

Struct No.	Structure ID	Junction Type	Rim Elev (ft)	Structure Shape	Structure Length (ft)	Structure Width (ft)	Line Out Size (in)	Line Out Shape	Line Out Invert (ft)	Line In Size (in)	Line In Shape	Line In Invert (ft)
19	PROP STORM MH 13	Manhole	35.50	Cir	4.00	4.00	18	Cir	26.83	12 12	Cir Cir	27.33 27.33
20	PROP STORM MH IN 12	Manhole	30.50	Cir	4.00	4.00	12	Cir	27.41	12	Cir	27.41
21	TRENCH DRAIN 11	None	30.50	n/a	n/a	n/a	12	Cir	27.43			
22	PROP STORM CB 10	Manhole	31.96	Cir	4.00	4.00	12	Cir	28.18			
23	BC 4.1	None	36.60	n/a	n/a	n/a	6	Cir	28.09			
24	EX INLET 5	Manhole	31.31	Cir	4.00	4.00	12	Cir	27.31			
25	BC 14.1	None	36.60	n/a	n/a	n/a	6	Cir	27.65			
26	PROP STORM MH IN 2	Manhole	31.50	Cir	4.00	4.00	15	Cir	26.89	15	Cir	26.89
27	PROP STORM CB 1	Manhole	31.96	Cir	4.00	4.00	15	Cir	27.92			
28	BC 15.1	None	36.60	n/a	n/a	n/a	6	Cir	27.42			
29	BC 16.1	None	36.60	n/a	n/a	n/a	6	Cir	27.26			
30	EX INLET 3	Manhole	29.31	Cir	4.00	4.00	15	Cir	25.61	15	Cir	25.71
31	Structure - (744)	None	31.00	n/a	n/a	n/a	15	Cir	25.86	15	Cir	25.86
32	PROP STORM MH 9	Manhole	33.30	Cir	4.00	4.00	15	Cir	25.91	15 6	Cir Cir	25.91 26.66
33	PROP STORM MH 8	Manhole	34.50	Cir	5.00	5.00	15	Cir	26.04	6 12	Cir Cir	26.79 26.29
34	BC 8.1	None	36.60	n/a	n/a	n/a	6	Cir	27.99			
35	PROP STORM MH 7	Manhole	29.90	Cir	4.00	4.00	12	Cir	26.50	12	Cir	26.50
36	TRENCH DRAIN 7.1	None	29.00	n/a	n/a	n/a	12	Cir	26.53			
37	BC 17.1	None	36.60	n/a	n/a	n/a	6	Cir	27.05			
38	BC 9.1	None	36.60	n/a	n/a	n/a	6	Cir	28.00			
39	PROP STORM MH 21	Manhole	29.50	Cir	5.00	5.00	12	Cir	26.39	12 12	Cir Cir	26.39 26.39

Project File: New.stm	Number of Structures: 45	Run Date: 11/18/2020

Structure Report

Struct No.	Structure ID	Junction Type	Rim Elev (ft)	Structure			Line Out			Line In		
				Shape	Length (ft)	Width (ft)	Size (in)	Shape	Invert (ft)	Size (in)	Shape	Invert (ft)
40	TRENCH DRAIN 20	None	29.00	n/a	n/a	n/a	12	Cir	26.45			
41	TRENCH DRAIN 19	None	29.00	n/a	n/a	n/a	12	Cir	26.53			
42	PROP STORM MH 25	Manhole	31.00	Cir	4.00	4.00	12	Cir	21.75	12	Cir	21.75
43	TRENCH DRAIN 24	None	26.35	n/a	n/a	n/a	12	Cir	21.93			
44	TRENCH DRAIN 26	None	26.35	n/a	n/a	n/a	12	Cir	21.63			
45	TRENCH DRAIN 23	None	26.35	n/a	n/a	n/a	12	Cir	19.84			

Project File: New.stm	Number of Structures: 45	Run Date: 11/18/2020

Hydraulic Grade Line Computations

Line	Size	Q	Downstream								Len	Upstream								Check		JL coeff	Minor loss
			Invert elev	HGL elev	Depth	Area	Vel	Vel head	EGL elev	Sf		Invert elev	HGL elev	Depth	Area	Vel	Vel head	EGL elev	Sf	Ave Sf	Enrgy loss		
	(in)	(cfs)	(ft)	(ft)	(ft)	(sqft)	(ft/s)	(ft)	(ft)	(%)	(ft)	(ft)	(ft)	(ft)	(sqft)	(ft/s)	(ft)	(ft)	(%)	(%)	(ft)	(K)	(ft)
1	21	14.03	24.00	26.93	1.75	2.40	5.83	0.53	27.46	0.669	17.040	24.09	27.04	1.75	2.41	5.83	0.53	27.57	0.668	0.669	0.114	1.00	0.53
2	18	9.76	24.34	27.57	1.50	1.77	5.52	0.47	28.05	0.736	194.270	25.31	29.00	1.50	1.77	5.52	0.47	29.48	0.736	0.736	1.430	1.00	0.47
3	18	8.44	25.56	29.48	1.50	1.77	4.78	0.35	29.83	0.551	50.000	25.81	29.75	1.50	1.77	4.78	0.35	30.11	0.550	0.551	0.275	0.15	0.05
4	15	6.15	26.06	29.81	1.25	1.23	5.01	0.39	30.20	0.773	19.529	26.16	29.96	1.25	1.23	5.01	0.39	30.35	0.773	0.773	0.151	1.00	0.39
5	12	2.90	26.41	30.35	1.00	0.79	3.69	0.21	30.56	0.565	56.441	27.26	30.67	1.00	0.79	3.69	0.21	30.88	0.565	0.565	0.319	1.00	0.21
6	12	1.93	27.26	30.88	1.00	0.79	2.46	0.09	30.97	0.250	65.130	28.23	31.04	1.00	0.79	2.46	0.09	31.14	0.250	0.250	0.163	1.00	0.09
7	12	1.06	28.23	31.14	1.00	0.79	1.35	0.03	31.16	0.076	60.213	29.14	31.18	1.00	0.79	1.35	0.03	31.21	0.075	0.076	0.045	0.99	0.03
8	6	1.06	29.39	31.21	0.50	0.20	5.40	0.45	31.66	3.046	15.610	30.04	31.69	0.50	0.20	5.40	0.45	32.14	3.045	3.046	0.475	1.00	0.45
9	6	0.87	28.48	31.14	0.50	0.20	4.43	0.31	31.44	2.052	14.692	29.09	31.44	0.50	0.20	4.43	0.31	31.74	2.051	2.052	0.301	1.00	0.31
10	24	18.56	24.55	26.93	2.00	3.14	5.91	0.54	27.47	0.574	11.134	24.61	26.99	2.00	3.14	5.91	0.54	27.54	0.574	0.574	0.064	1.00	0.54
11	21	11.60	24.86	27.54	1.75	2.40	4.82	0.36	27.90	0.457	34.014	25.03	27.69	1.75	2.41	4.82	0.36	28.05	0.457	0.457	0.155	0.92	0.33
12	21	9.34	25.03	28.03	1.75	2.40	3.88	0.23	28.26	0.296	75.876	25.40	28.25	1.75	2.41	3.88	0.23	28.48	0.296	0.296	0.225	0.96	0.23
13	18	7.18	25.65	28.47	1.50	1.77	4.06	0.26	28.73	0.399	52.349	25.92	28.68	1.50	1.77	4.06	0.26	28.94	0.398	0.398	0.209	0.59	0.15
14	18	6.93	26.02	28.84	1.50	1.77	3.92	0.24	29.07	0.371	20.234	26.12	28.91	1.50	1.77	3.92	0.24	29.15	0.371	0.371	0.075	0.40	0.10
15	18	6.93	26.12	29.01	1.50	1.77	3.92	0.24	29.25	0.371	18.868	26.21	29.08	1.50	1.77	3.92	0.24	29.32	0.371	0.371	0.070	0.89	0.21
16	18	5.96	26.21	29.29	1.50	1.77	3.37	0.18	29.47	0.275	78.917	26.42	29.51	1.50	1.77	3.37	0.18	29.68	0.274	0.275	0.217	1.00	0.18
17	18	5.09	26.42	29.68	1.50	1.77	2.88	0.13	29.81	0.200	62.542	26.58	29.81	1.50	1.77	2.88	0.13	29.94	0.200	0.200	0.125	1.00	0.13
18	18	4.22	26.58	29.94	1.50	1.77	2.39	0.09	30.03	0.138	87.792	26.81	30.06	1.50	1.77	2.39	0.09	30.15	0.138	0.138	0.121	1.00	0.09
19	18	3.16	26.81	30.15	1.50	1.77	1.79	0.05	30.20	0.077	8.822	26.83	30.15	1.50	1.77	1.79	0.05	30.20	0.077	0.077	0.007	0.99	0.05
20	12	0.75	27.33	30.20	1.00	0.79	0.96	0.01	30.22	0.038	30.751	27.41	30.21	1.00	0.79	0.95	0.01	30.23	0.038	0.038	0.012	0.57	0.01
21	12	0.73	27.41	30.22	1.00	0.79	0.93	0.01	30.24	0.036	8.093	27.43	30.22	1.00	0.79	0.93	0.01	30.24	0.036	0.036	0.003	1.00	0.01
22	12	2.41	27.33	30.20	1.00	0.79	3.07	0.15	30.35	0.390	169.905	28.18	30.87	1.00	0.79	3.07	0.15	31.01	0.390	0.390	0.663	1.00	0.15

Project File: New.stm	Number of lines: 45	Run Date: 11/18/2020

; c = cir e = ellip b = box

Hydraulic Grade Line Computations

Line	Size	Q	Downstream								Len	Upstream								Check		JL coeff	Minor loss
	(in)	(cfs)	Invert elev (ft)	HGL elev (ft)	Depth (ft)	Area (sqft)	Vel (ft/s)	Vel head (ft)	EGL elev (ft)	Sf (%)	(ft)	Invert elev (ft)	HGL elev (ft)	Depth (ft)	Area (sqft)	Vel (ft/s)	Vel head (ft)	EGL elev (ft)	Sf (%)	Ave Sf (%)	Enrgy loss (ft)	(K)	(ft)
23	6	0.97	27.51	30.88	0.50	0.20	4.94	0.38	31.26	2.551	13.854	28.09	31.23	0.50	0.20	4.94	0.38	31.61	2.550	2.550	0.353	1.00	0.38
24	12	1.84	27.06	27.66	0.60*	0.49	3.73	0.22	27.88	0.500	50.000	27.31	27.91	0.60	0.49	3.73	0.22	28.13	0.499	0.499	0.250	1.00	0.22
25	6	1.06	27.31	30.15	0.50	0.20	5.40	0.45	30.60	3.046	8.271	27.65	30.40	0.50	0.20	5.40	0.45	30.85	3.045	3.046	0.252	1.00	0.45
26	15	3.25	26.16	30.35	1.25	1.23	2.65	0.11	30.46	0.216	146.000	26.89	30.66	1.25	1.23	2.65	0.11	30.77	0.216	0.216	0.315	0.96	0.10
27	15	3.06	26.89	30.77	1.25	1.23	2.49	0.10	30.86	0.191	205.204	27.92	31.16	1.25	1.23	2.49	0.10	31.26	0.191	0.191	0.393	1.00	0.10
28	6	0.87	27.08	29.94	0.50	0.20	4.43	0.31	30.24	2.052	8.271	27.42	30.11	0.50	0.20	4.43	0.31	30.41	2.051	2.052	0.170	1.00	0.31
29	6	0.87	26.92	29.68	0.50	0.20	4.43	0.31	29.99	2.052	8.271	27.26	29.85	0.50	0.20	4.43	0.31	30.16	2.051	2.052	0.170	1.00	0.31
30	15	5.52	25.36	27.54	1.25	1.23	4.50	0.31	27.85	0.623	49.992	25.61	27.85	1.25	1.23	4.50	0.31	28.16	0.623	0.623	0.311	0.79	0.25
31	15	3.21	25.71	28.10	1.25	1.23	2.62	0.11	28.20	0.211	29.716	25.86	28.16	1.25	1.23	2.62	0.11	28.27	0.211	0.211	0.063	0.15	0.02
32	15	3.21	25.86	28.18	1.25	1.23	2.62	0.11	28.28	0.211	9.263	25.91	28.19	1.25	1.23	2.62	0.11	28.30	0.211	0.211	0.020	0.98	0.10
33	15	2.15	25.91	28.30	1.25	1.23	1.75	0.05	28.35	0.094	50.777	26.04	28.35	1.25	1.23	1.75	0.05	28.39	0.094	0.094	0.048	1.00	0.05
34	6	0.97	26.79	28.39	0.50	0.20	4.94	0.38	28.77	2.551	28.851	27.99	29.13	0.50	0.20	4.94	0.38	29.51	2.550	2.550	0.736	1.00	0.38
35	12	1.18	26.29	28.39	1.00	0.79	1.50	0.04	28.43	0.094	81.562	26.50	28.47	1.00	0.79	1.50	0.04	28.51	0.094	0.094	0.076	0.88	0.03
36	12	1.18	26.50	28.50	1.00	0.79	1.50	0.04	28.54	0.094	10.000	26.53	28.51	1.00	0.79	1.50	0.04	28.55	0.094	0.094	0.009	1.00	0.04
37	6	0.97	26.71	29.29	0.50	0.20	4.94	0.38	29.67	2.551	8.271	27.05	29.50	0.50	0.20	4.94	0.38	29.88	2.550	2.550	0.211	1.00	0.38
38	6	1.06	26.66	28.30	0.50	0.20	5.40	0.45	28.75	3.046	32.259	28.00	29.28	0.50	0.20	5.40	0.45	29.73	3.045	3.046	0.983	1.00	0.45
39	12	2.26	25.78	28.03	1.00	0.79	2.88	0.13	28.15	0.343	121.349	26.39	28.44	1.00	0.79	2.88	0.13	28.57	0.343	0.343	0.417	1.00	0.13
40	12	0.58	26.39	28.57	1.00	0.79	0.74	0.01	28.58	0.023	12.753	26.45	28.57	1.00	0.79	0.74	0.01	28.58	0.023	0.023	0.003	1.00	0.01
41	12	1.68	26.39	28.57	1.00	0.79	2.14	0.07	28.64	0.190	54.449	26.53	28.67	1.00	0.79	2.14	0.07	28.74	0.190	0.190	0.103	1.00	0.07
42	12	0.51	20.67	24.60	1.00	0.79	0.65	0.01	24.61	0.017	143.418	21.75	24.63	1.00	0.79	0.65	0.01	24.63	0.017	0.017	0.025	0.82	0.01
43	12	0.51	21.75	24.63	1.00	0.79	0.65	0.01	24.64	0.017	24.500	21.93	24.63	1.00	0.79	0.65	0.01	24.64	0.017	0.017	0.004	1.00	0.01
44	12	0.19	21.48	24.60	1.00	0.79	0.24	0.00	24.60	0.002	57.261	21.63	24.60	1.00	0.79	0.24	0.00	24.60	0.002	0.002	0.001	1.00	0.00

Project File: New.stm | Number of lines: 45 | Run Date: 11/18/2020

Notes: * depth assumed ; c = cir e = ellip b = box

Hydraulic Grade Line Computations

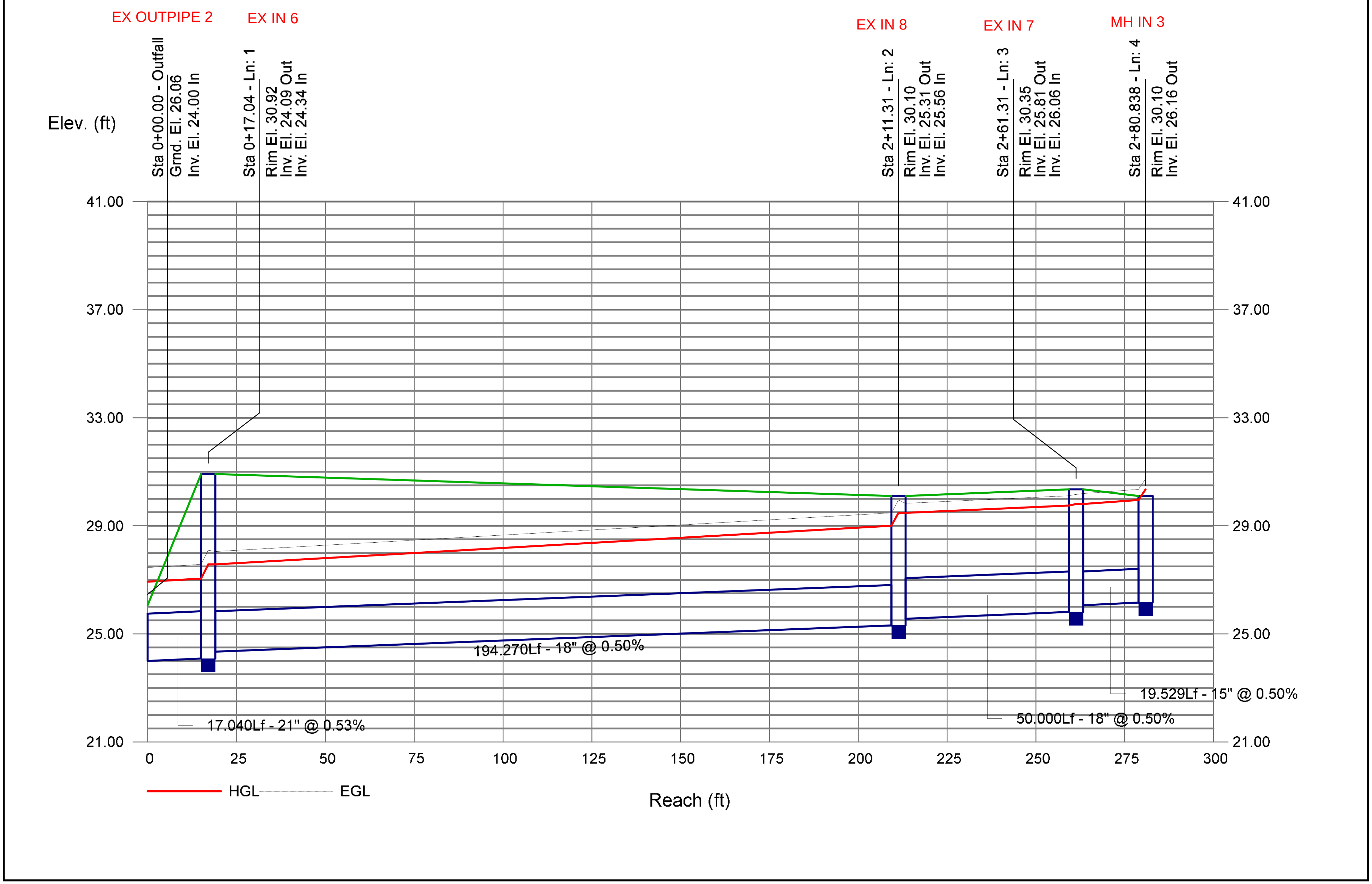
Line	Size	Q	Downstream								Len	Upstream								Check		JL coeff	Minor loss
	(in)	(cfs)	Invert elev (ft)	HGL elev (ft)	Depth (ft)	Area (sqft)	Vel (ft/s)	Vel head (ft)	EGL elev (ft)	Sf (%)	(ft)	Invert elev (ft)	HGL elev (ft)	Depth (ft)	Area (sqft)	Vel (ft/s)	Vel head (ft)	EGL elev (ft)	Sf (%)	Ave Sf (%)	Enrgy loss (ft)	(K)	(ft)
45	12	0.46	19.72	24.60	1.00	0.79	0.59	0.01	24.61	0.014	44.870	19.84	24.61	1.00	0.79	0.59	0.01	24.61	0.014	0.014	0.006	1.00	0.01

Project File: New.stm	Number of lines: 45	Run Date: 11/18/2020

Notes: * depth assumed ; c = cir e = ellip b = box

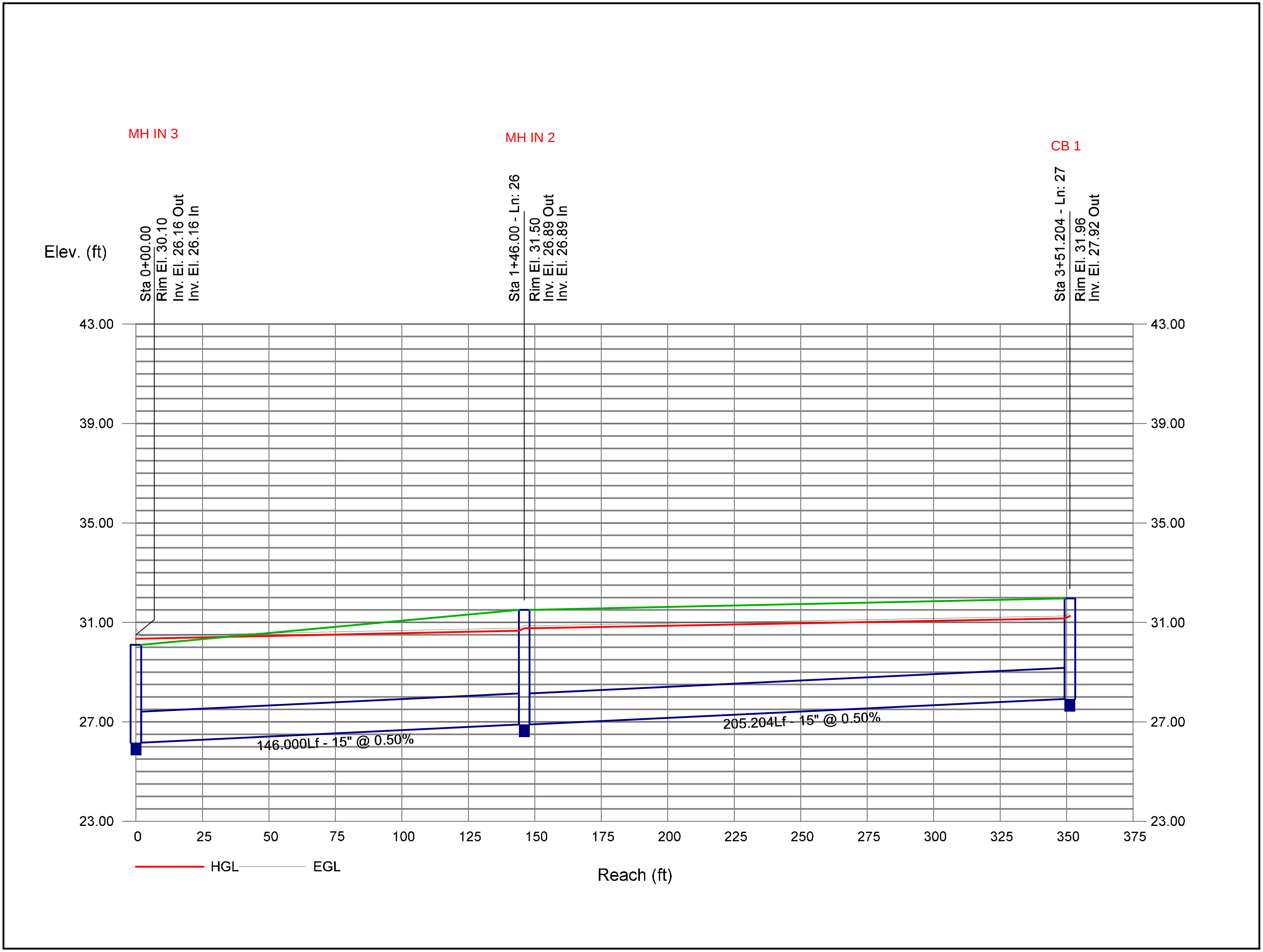
Storm Sewer Profile

Proj. file: New.stm



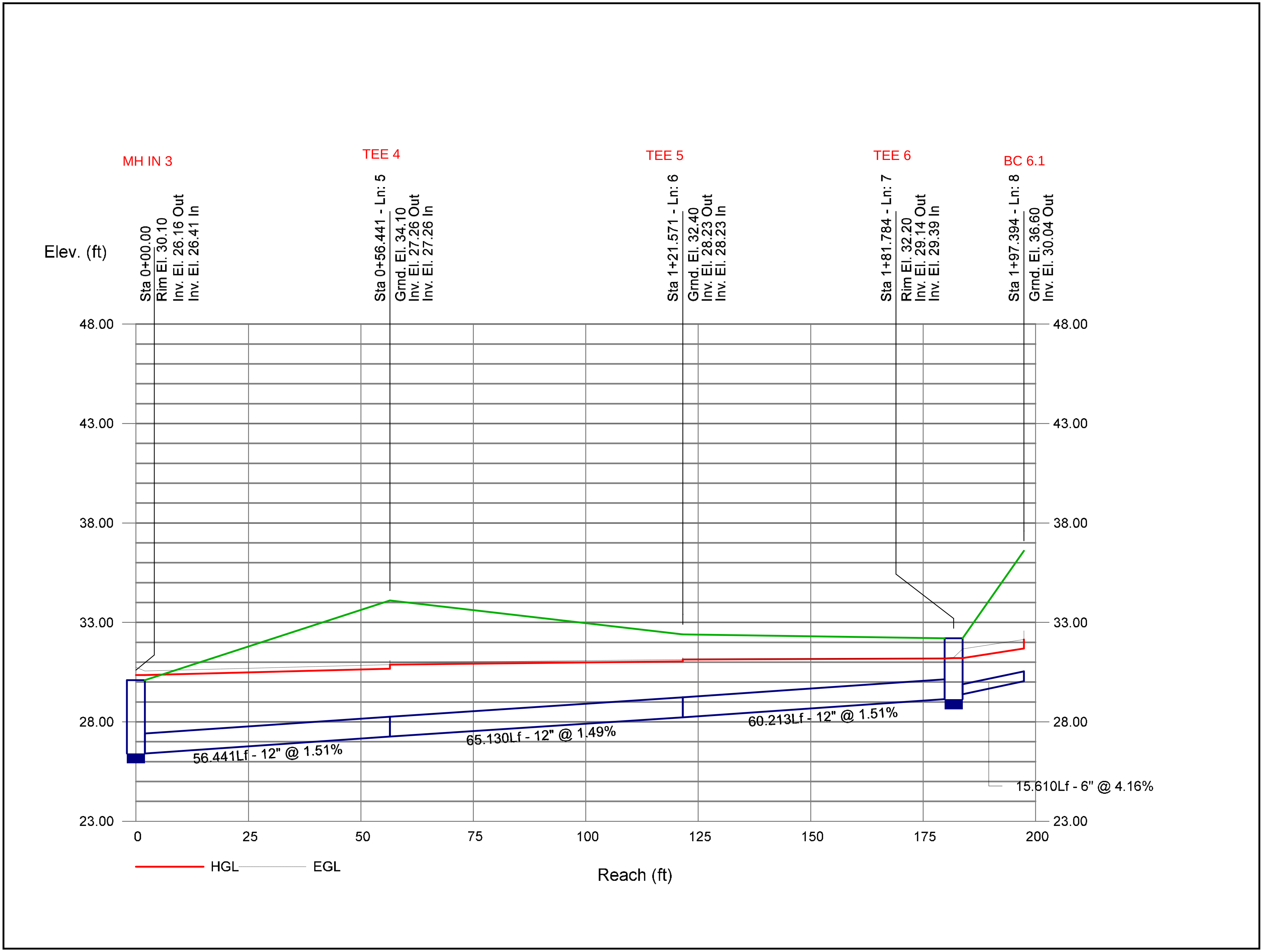
Storm Sewer Profile

Proj. file: New.stm



Storm Sewer Profile

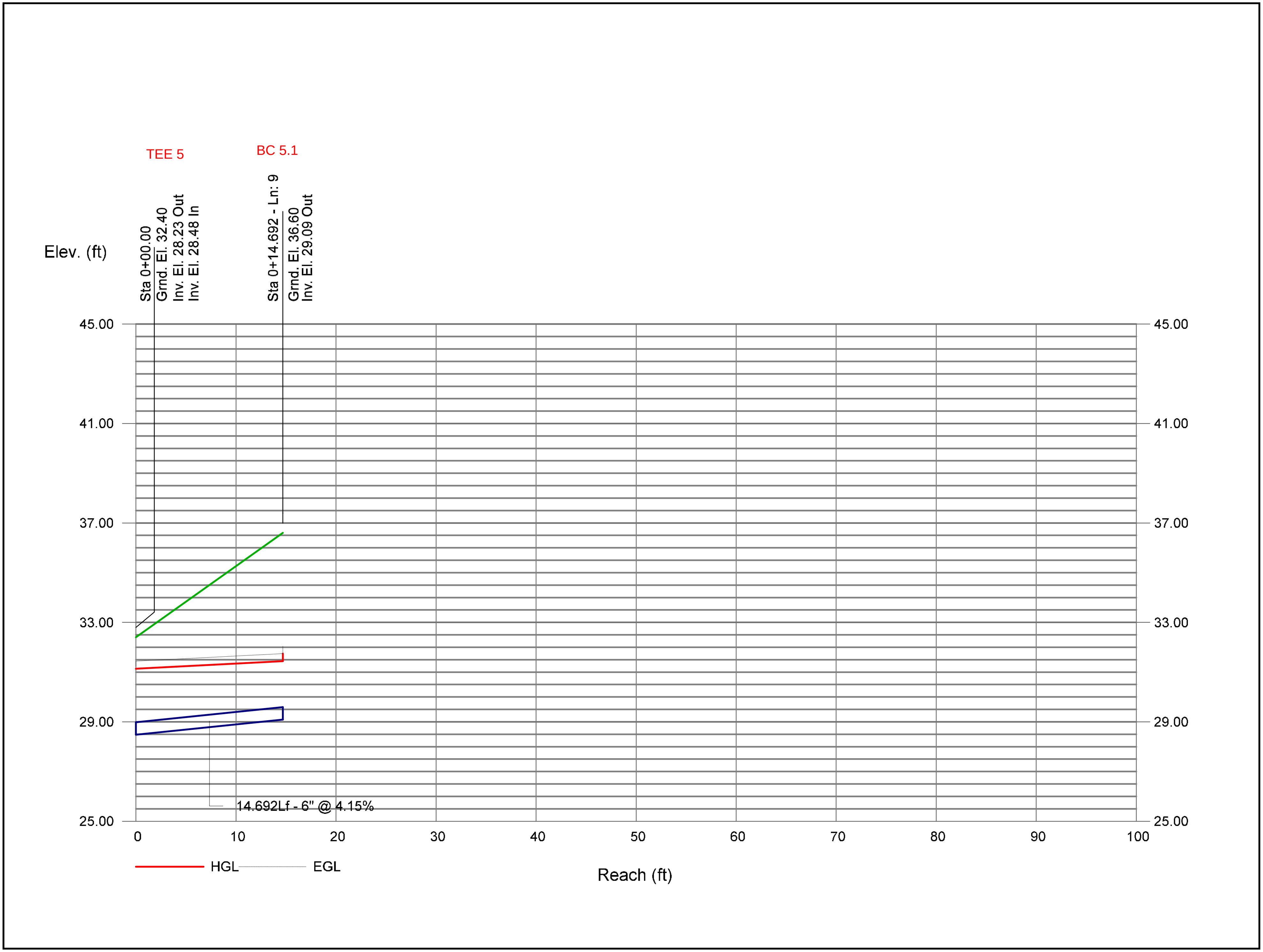
Proj. file: New.stm



Storm Sewers

Storm Sewer Profile

Proj. file: New.stm



Storm Sewer Profile

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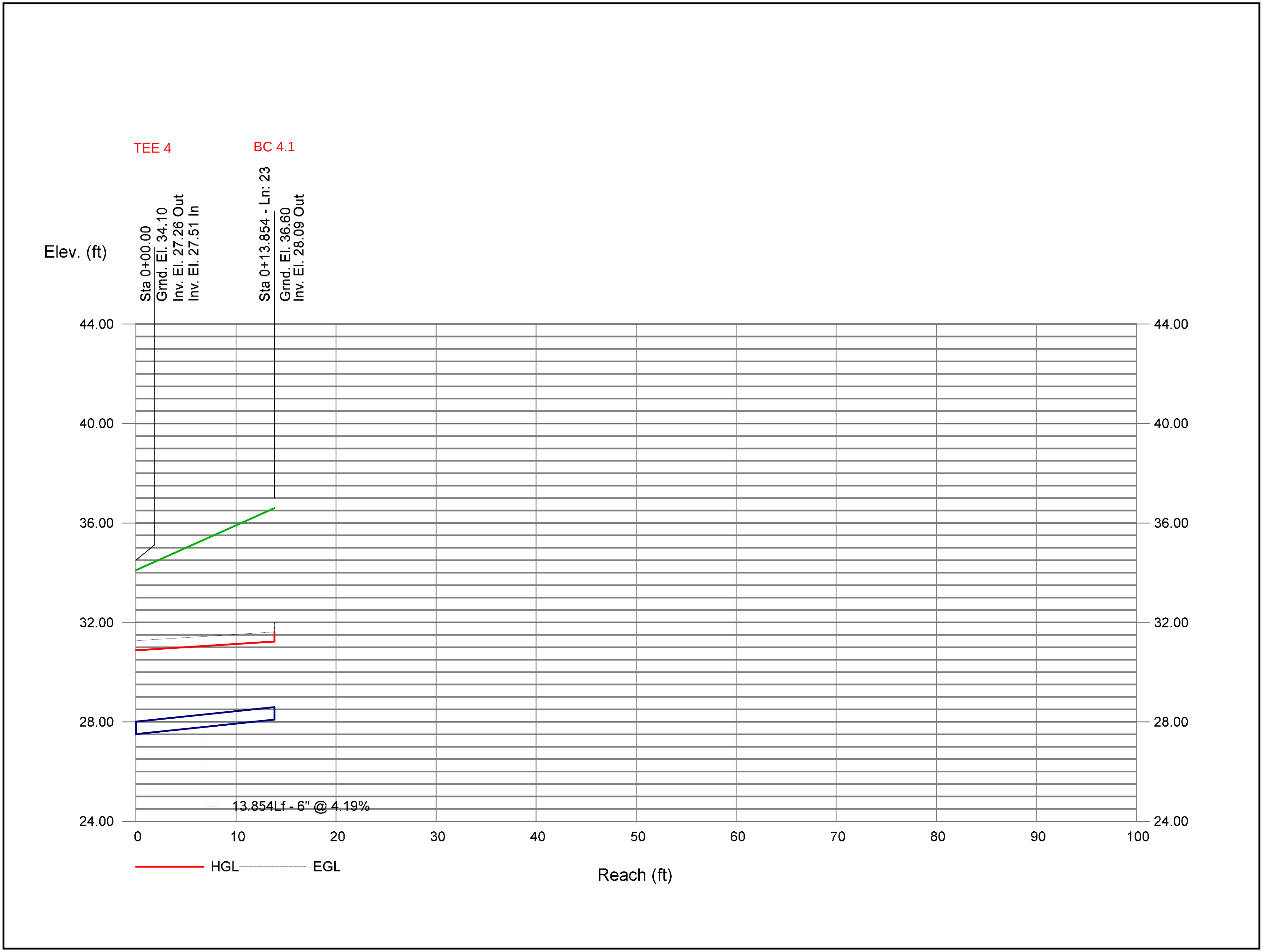

Storm Sewer Profile

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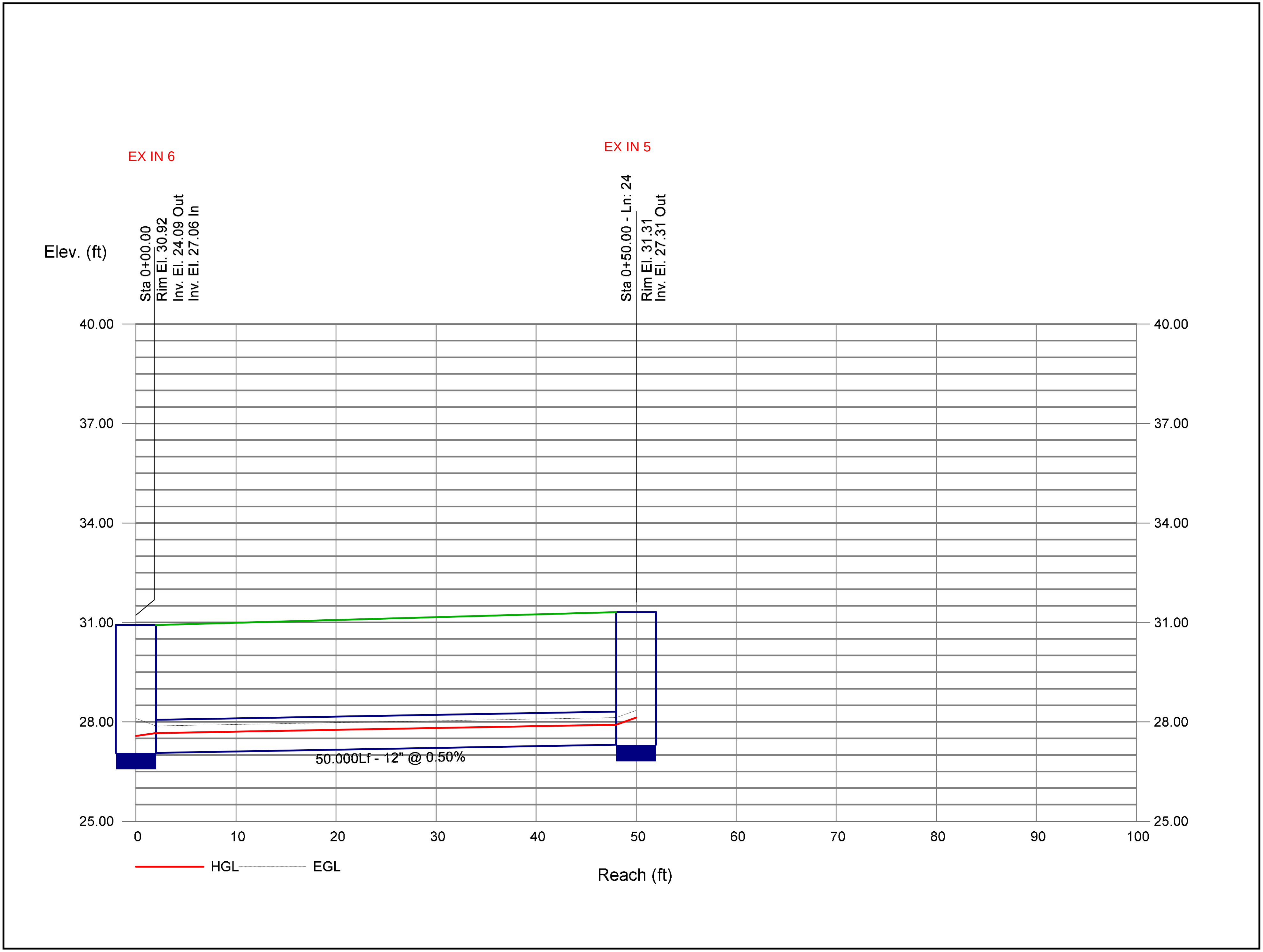

Storm Sewer Profile

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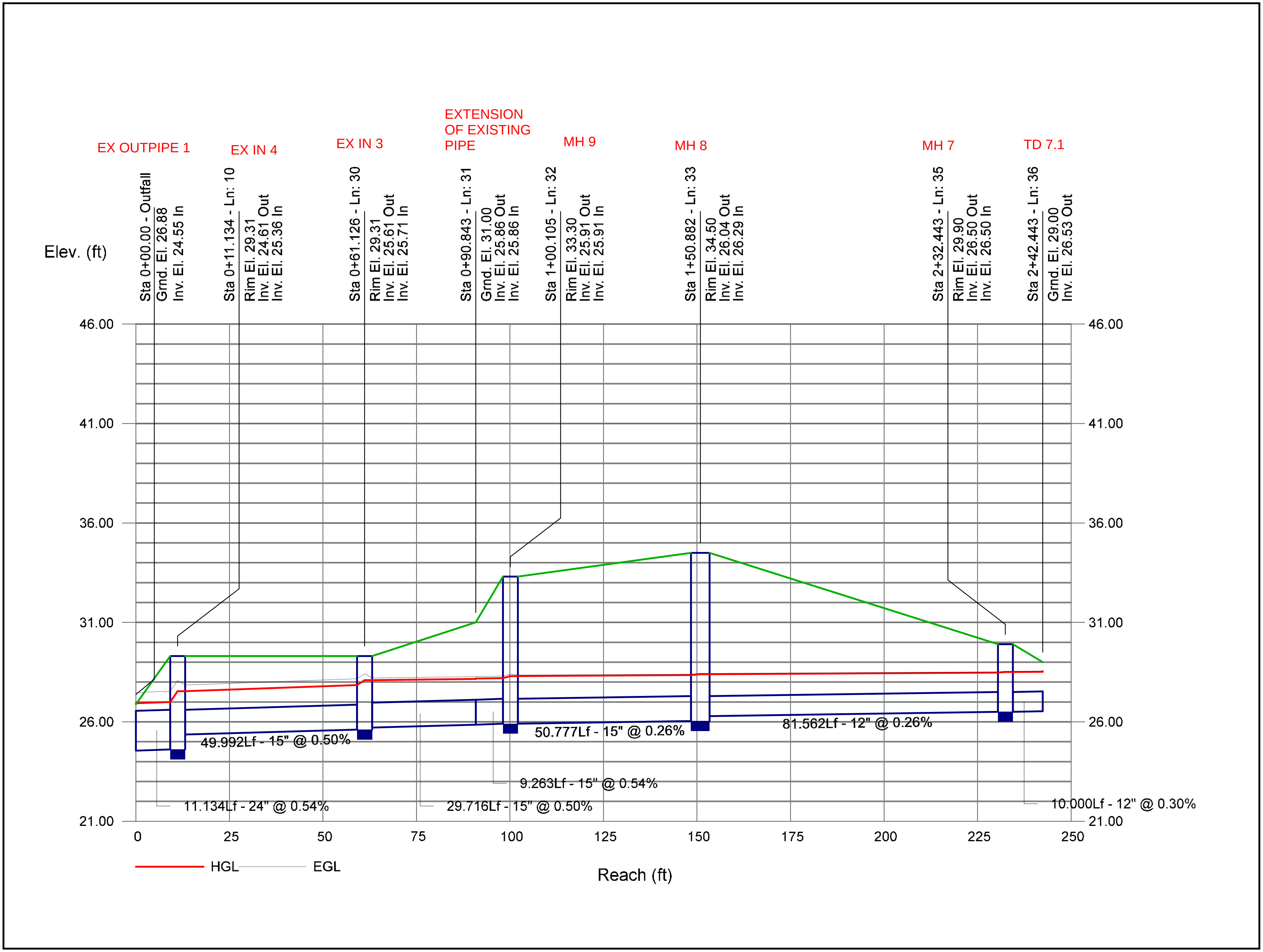

Storm Sewers

Storm Sewer Profile

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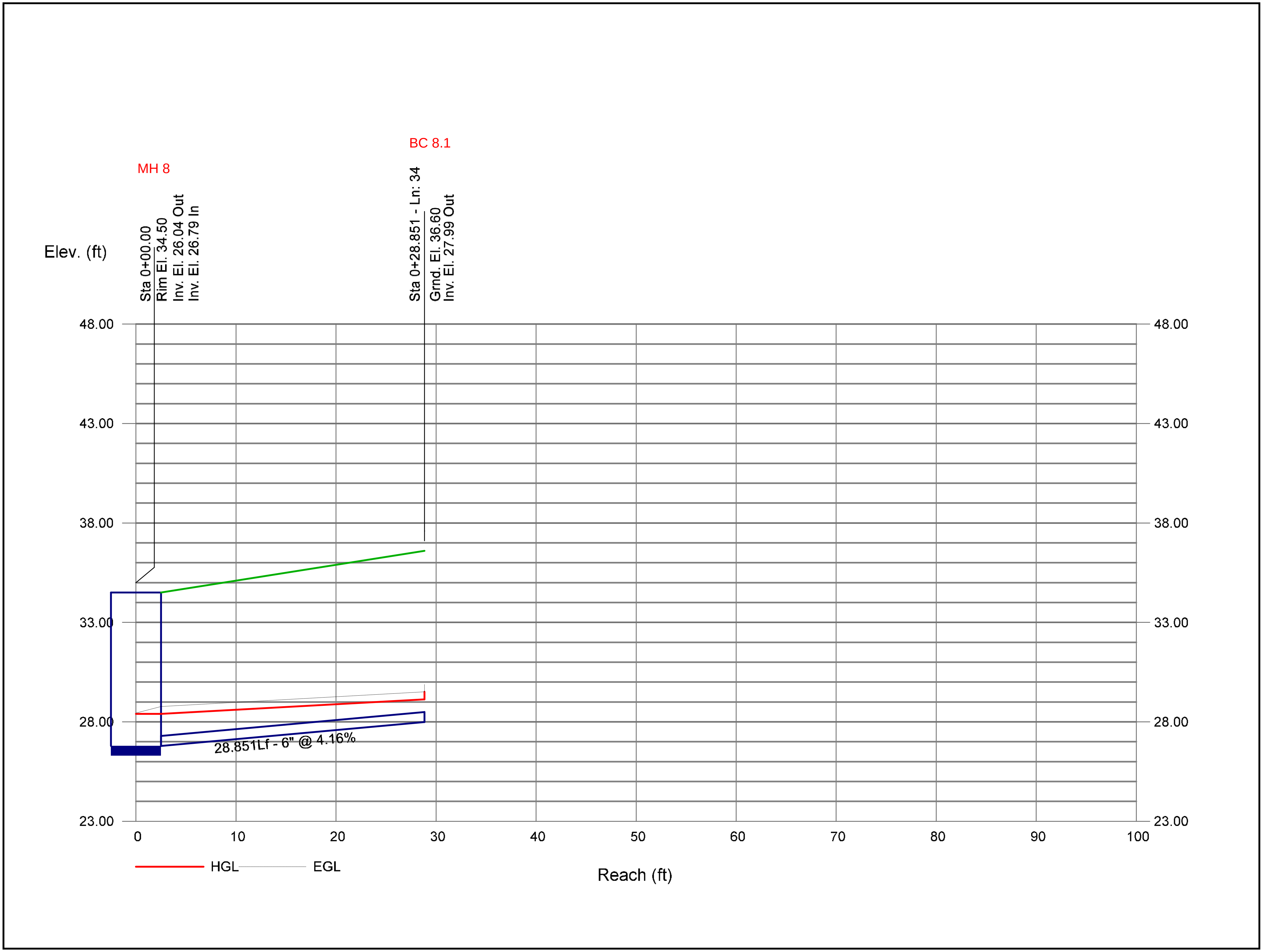
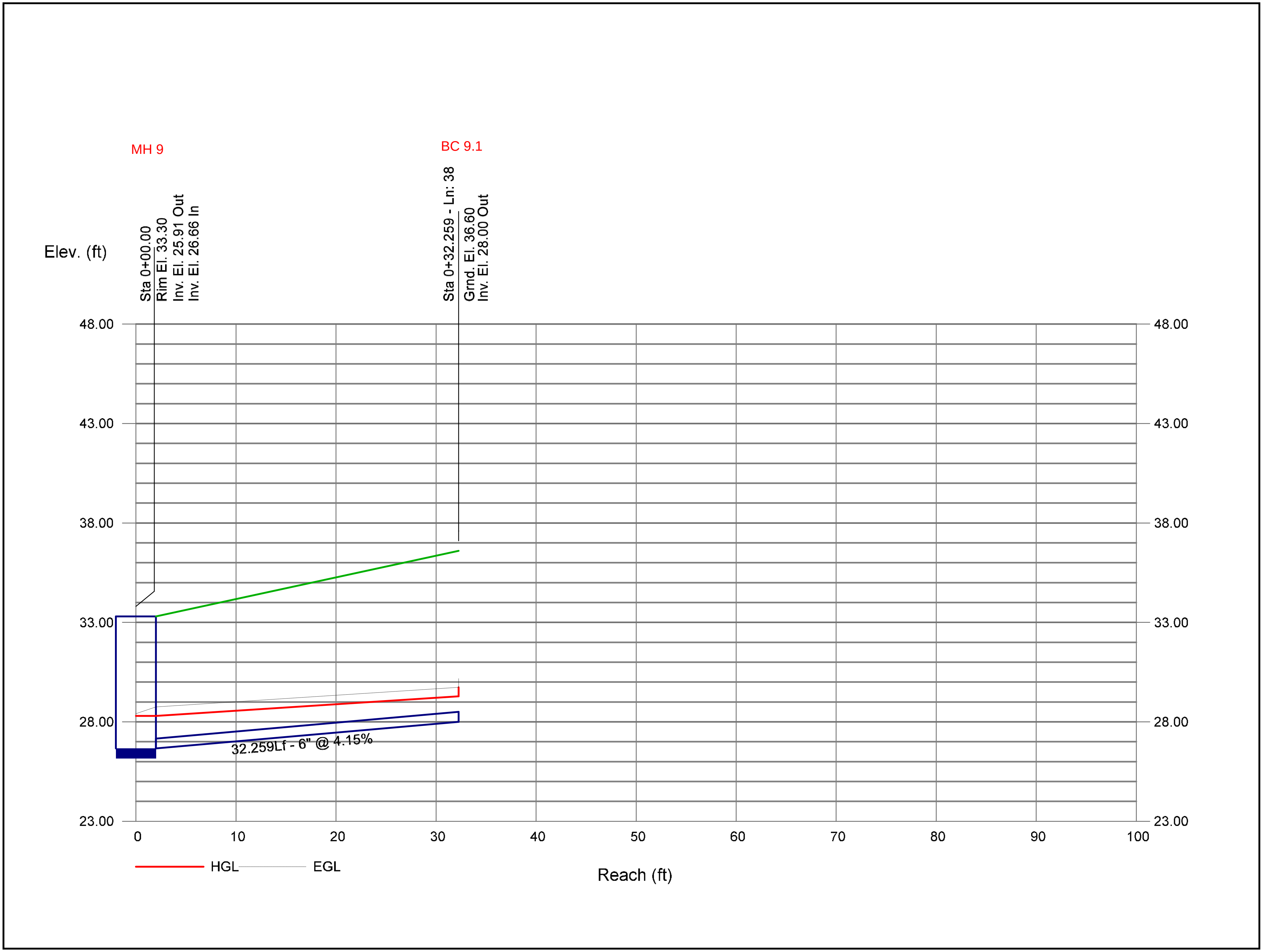


Storm Sewer Profile

Proj. file: New.stm



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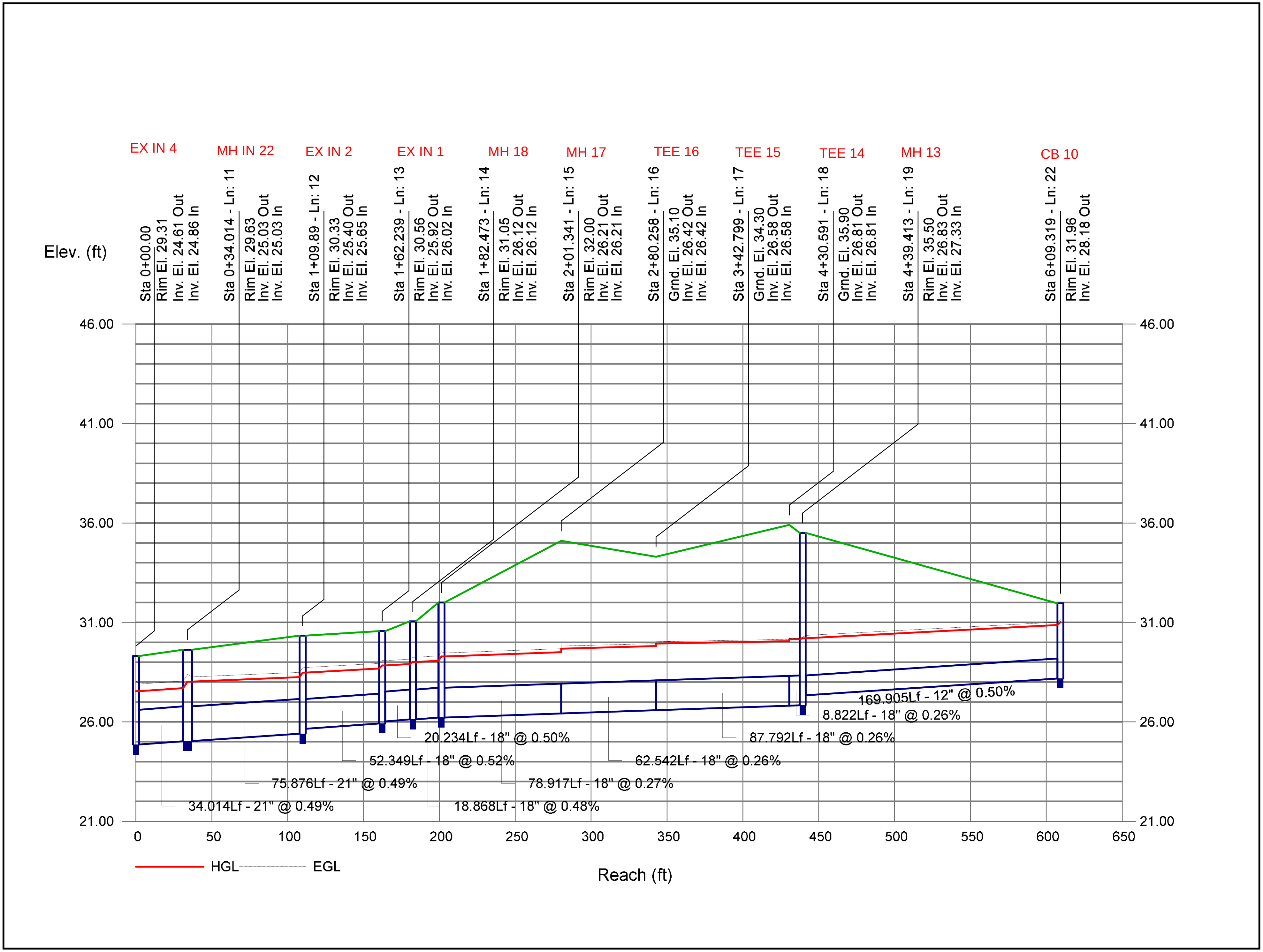

Storm Sewers

Storm Sewer Profile

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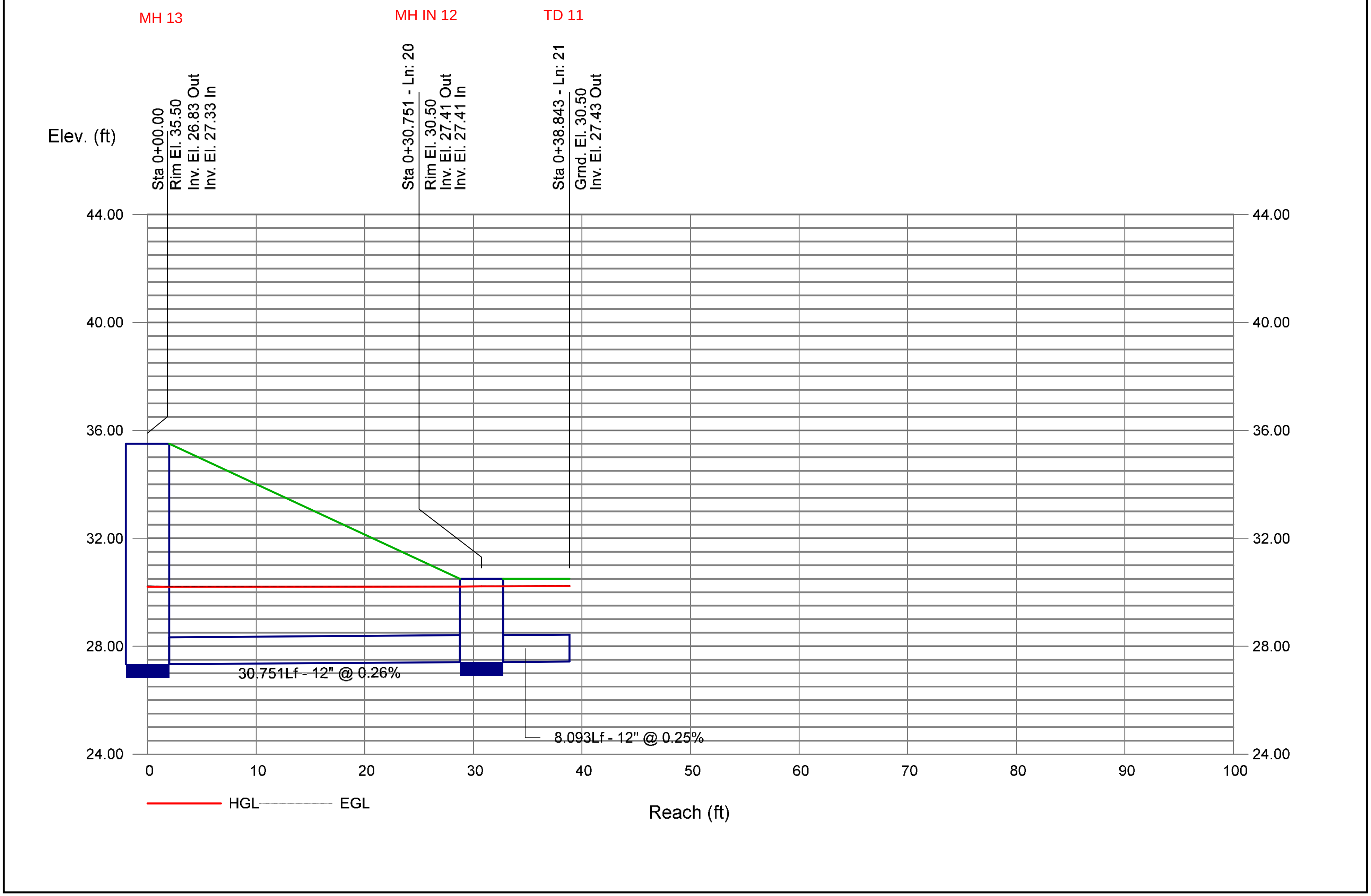

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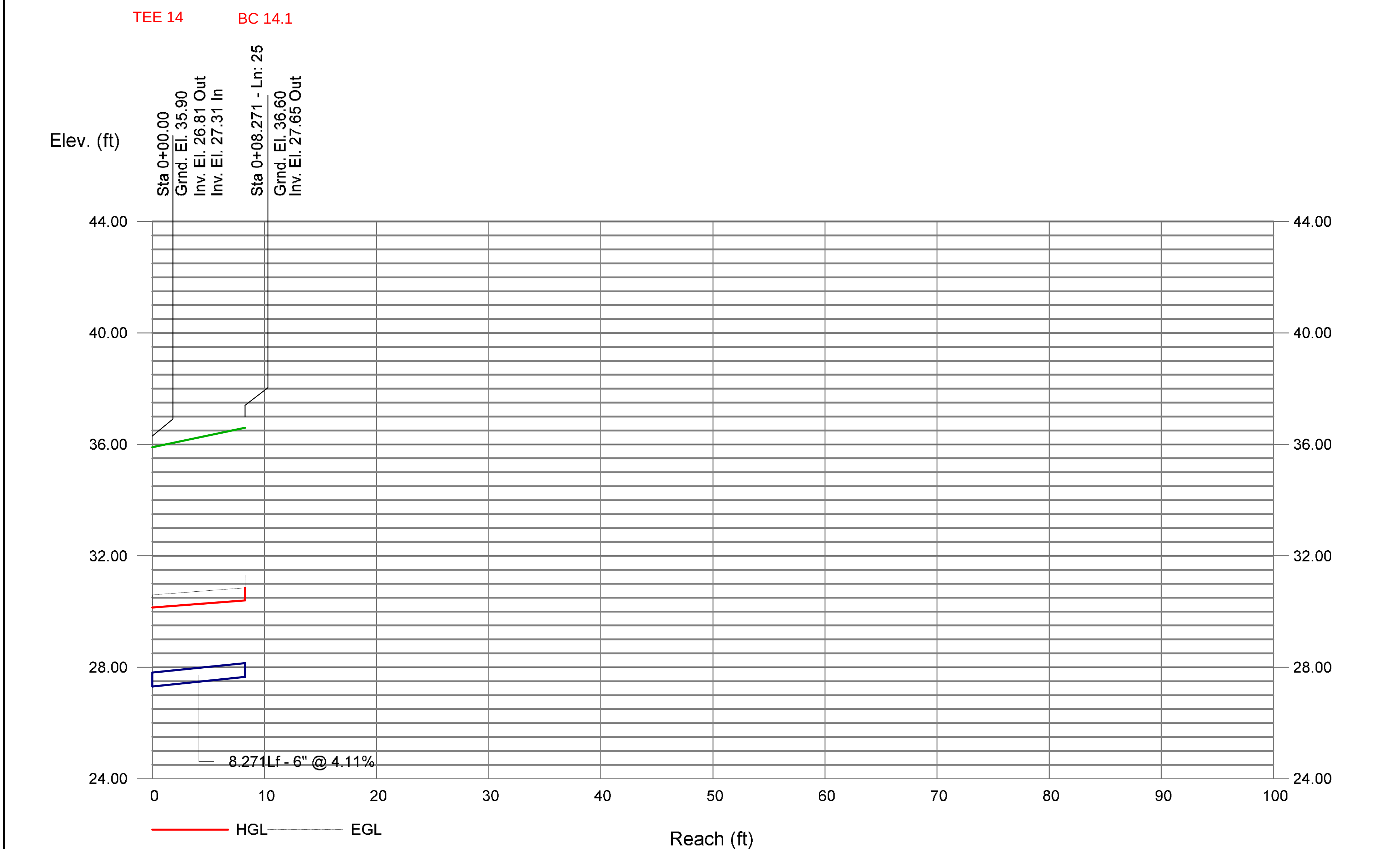

Storm Sewer Profile

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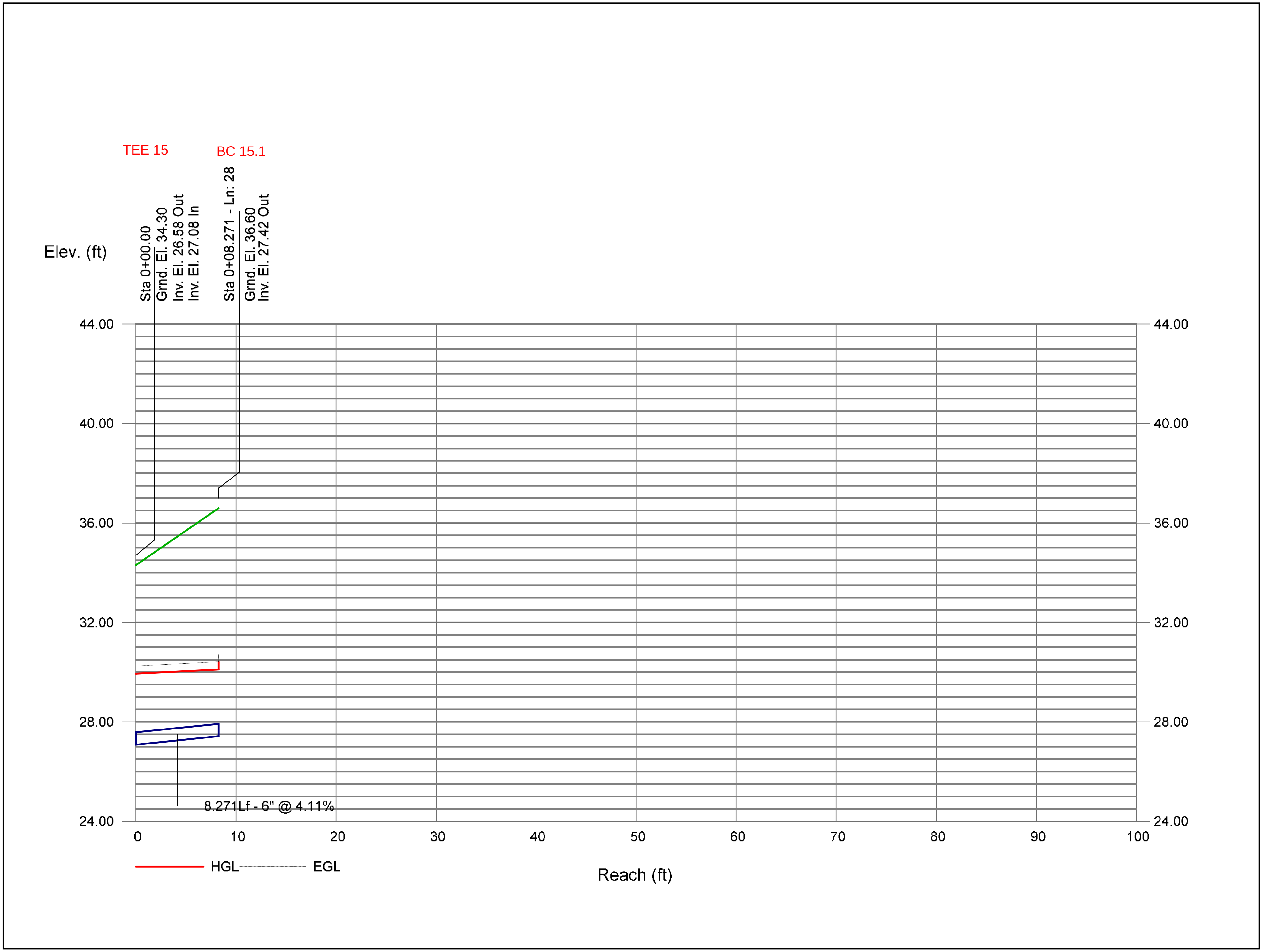

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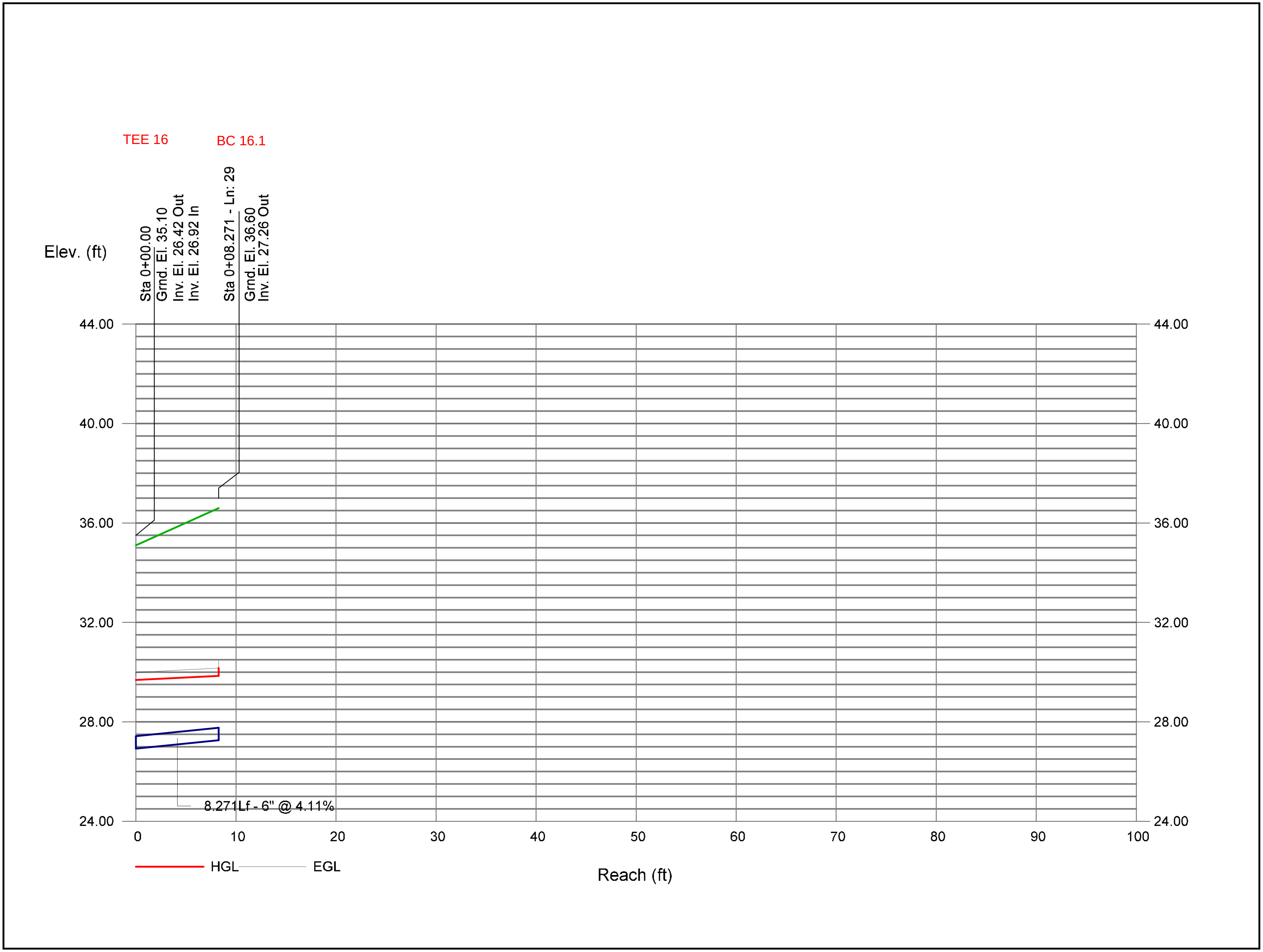

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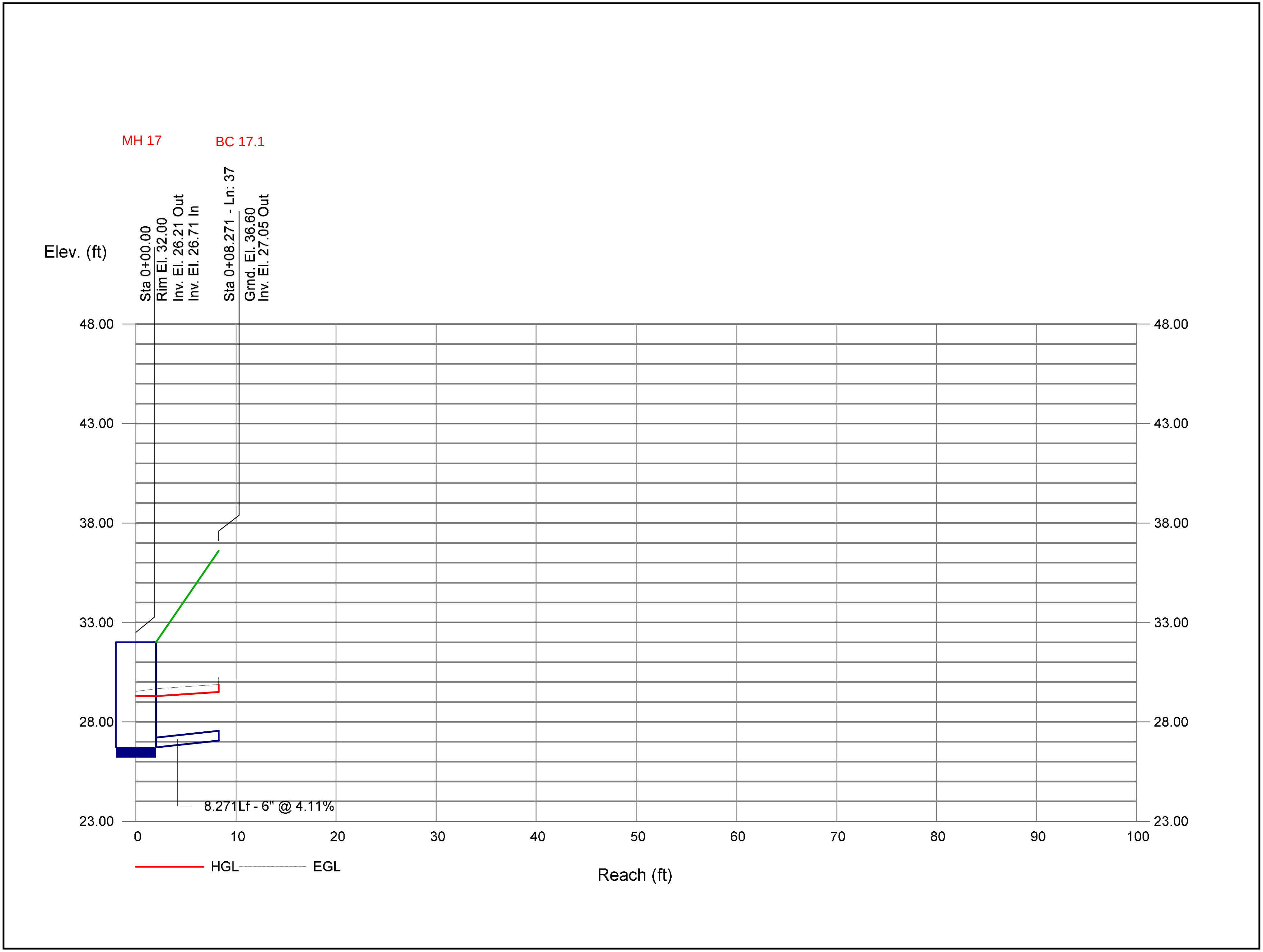

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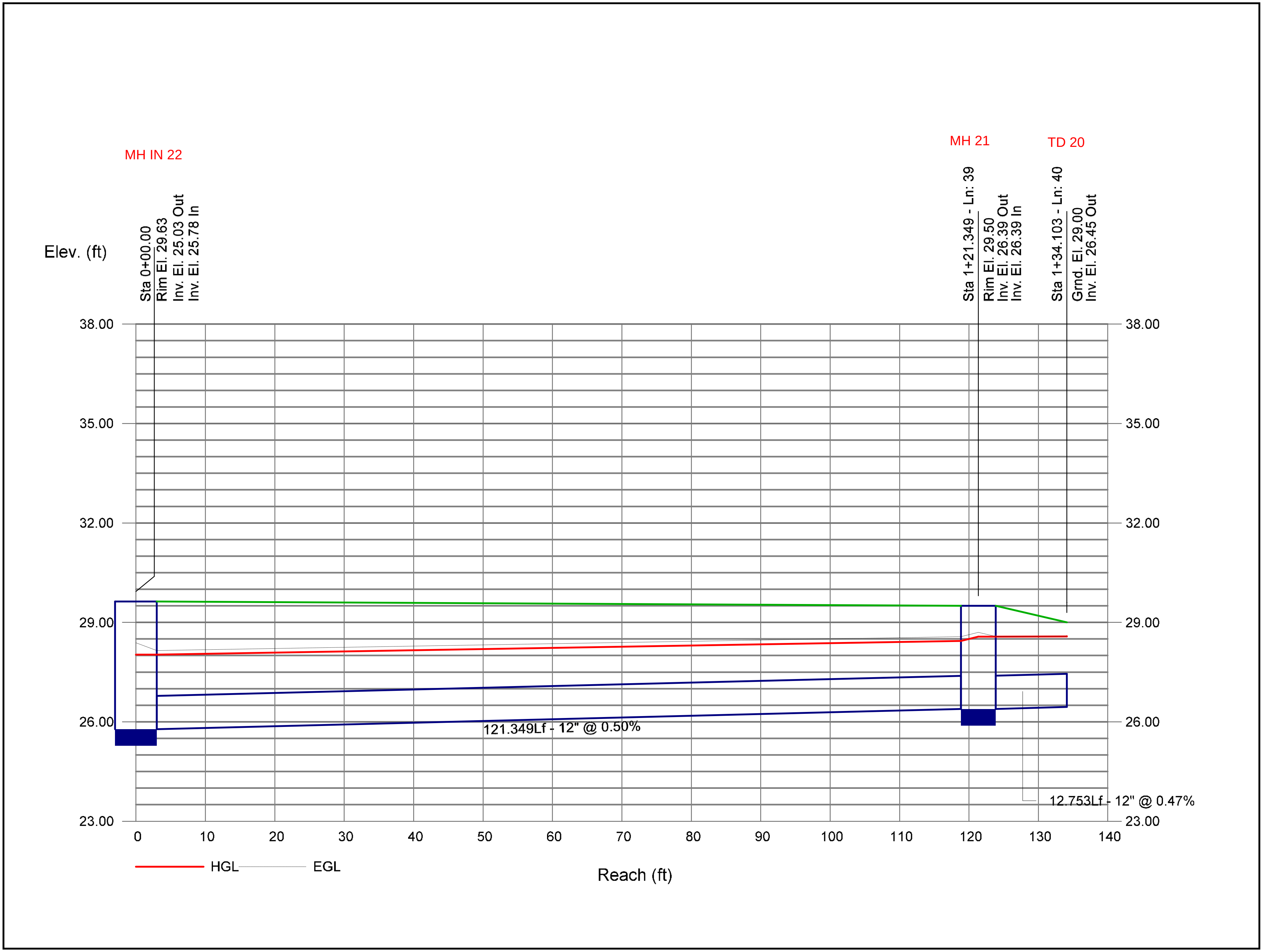

Storm Sewer Profile

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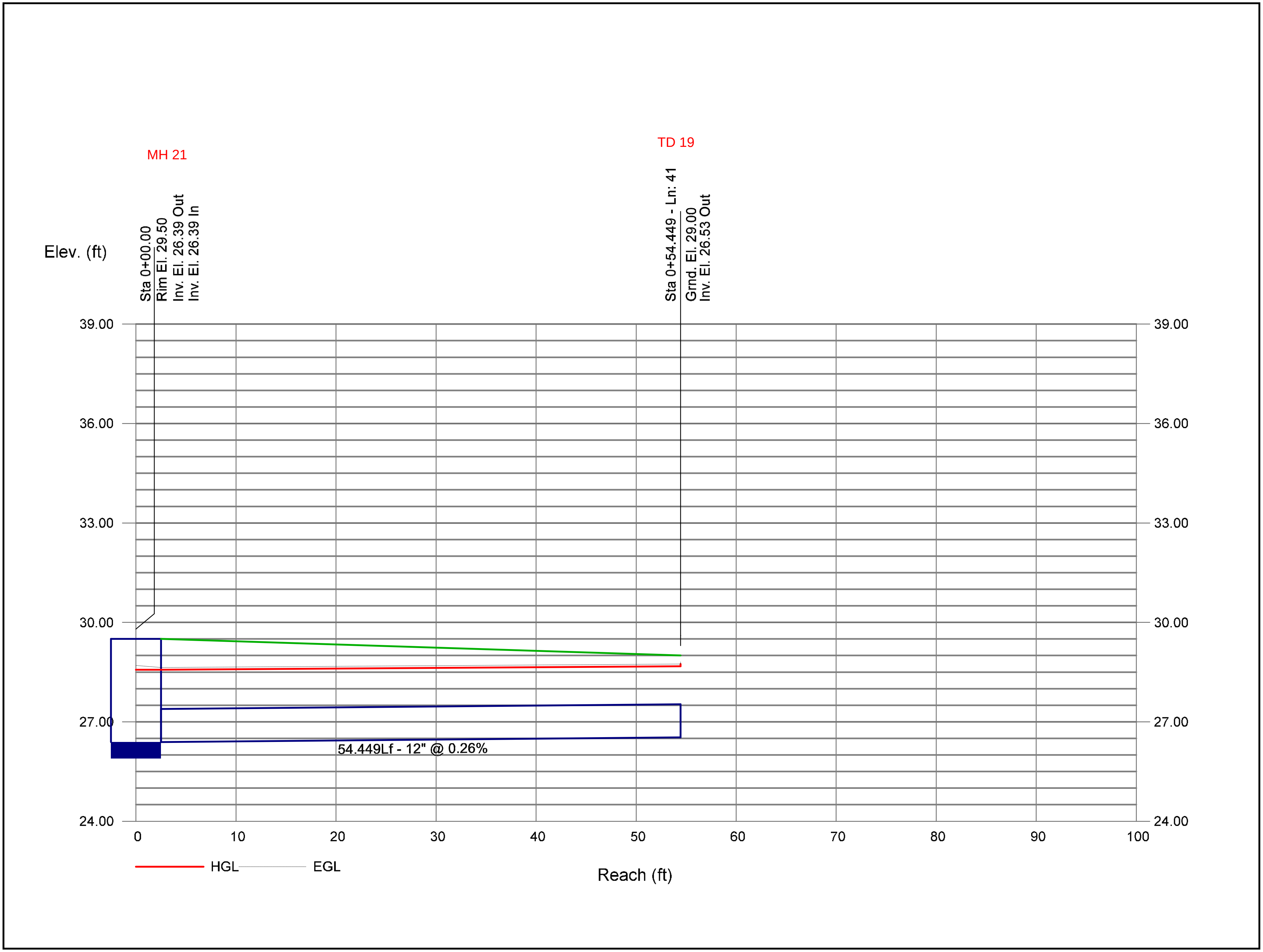

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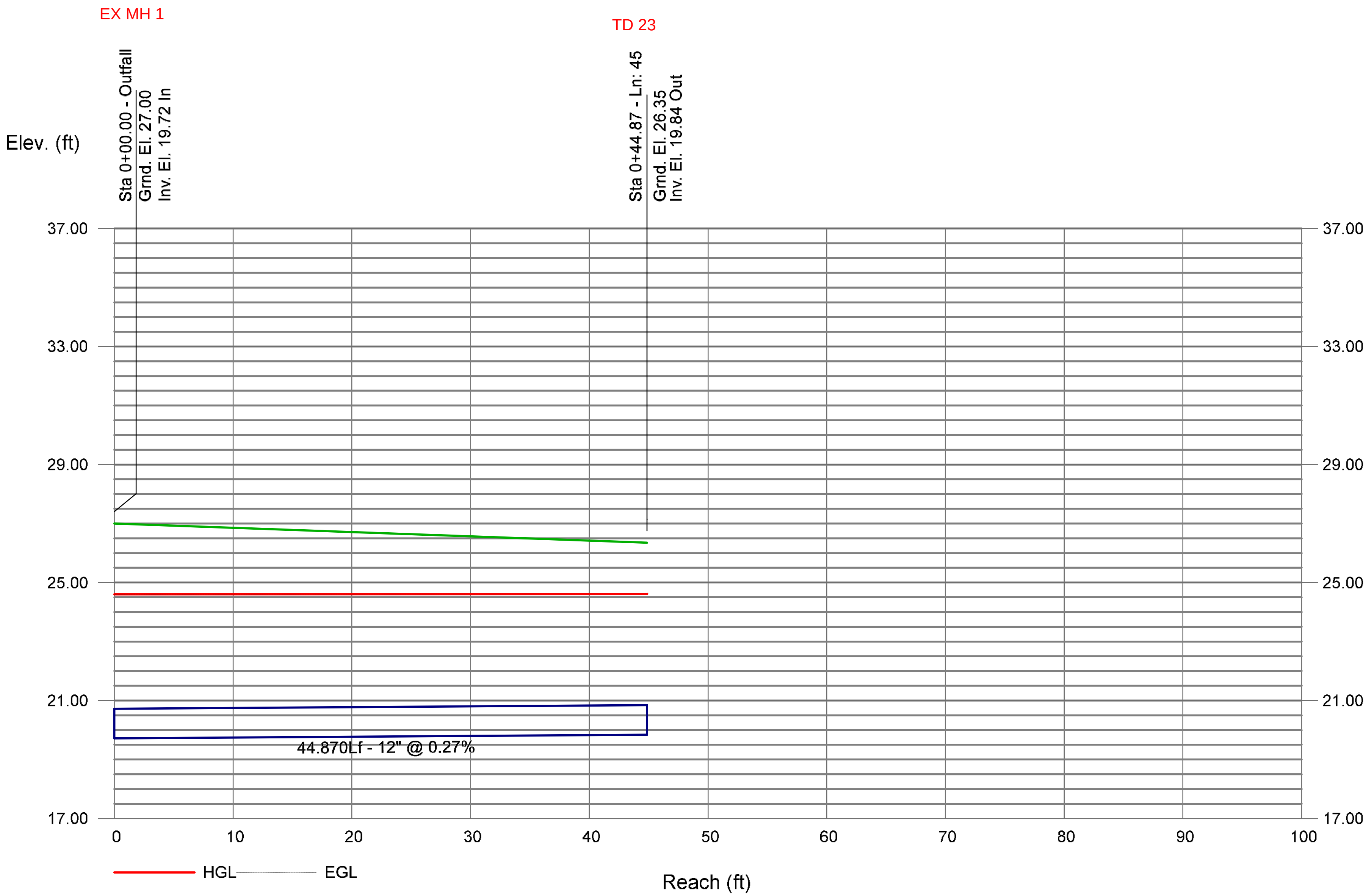

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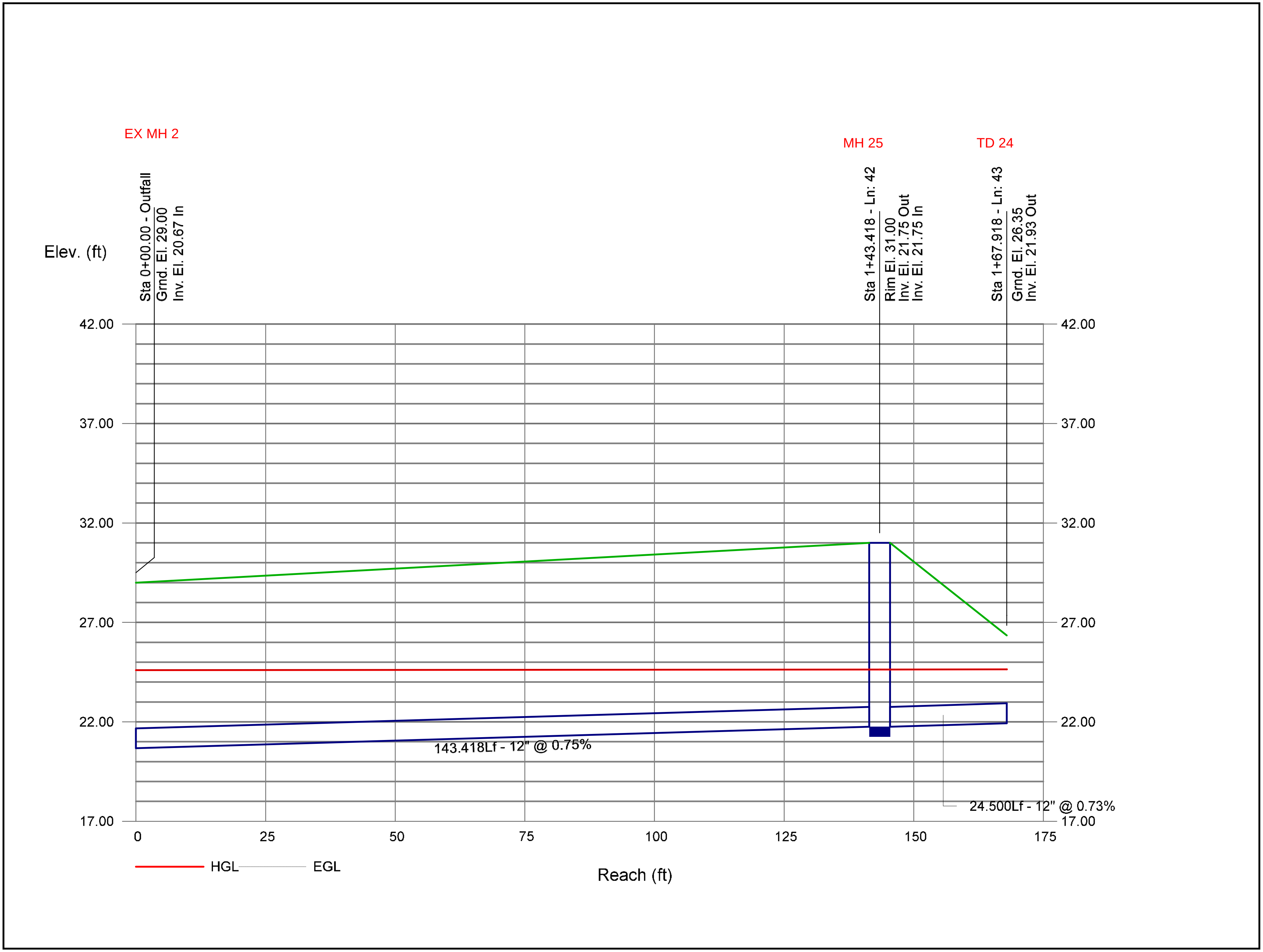

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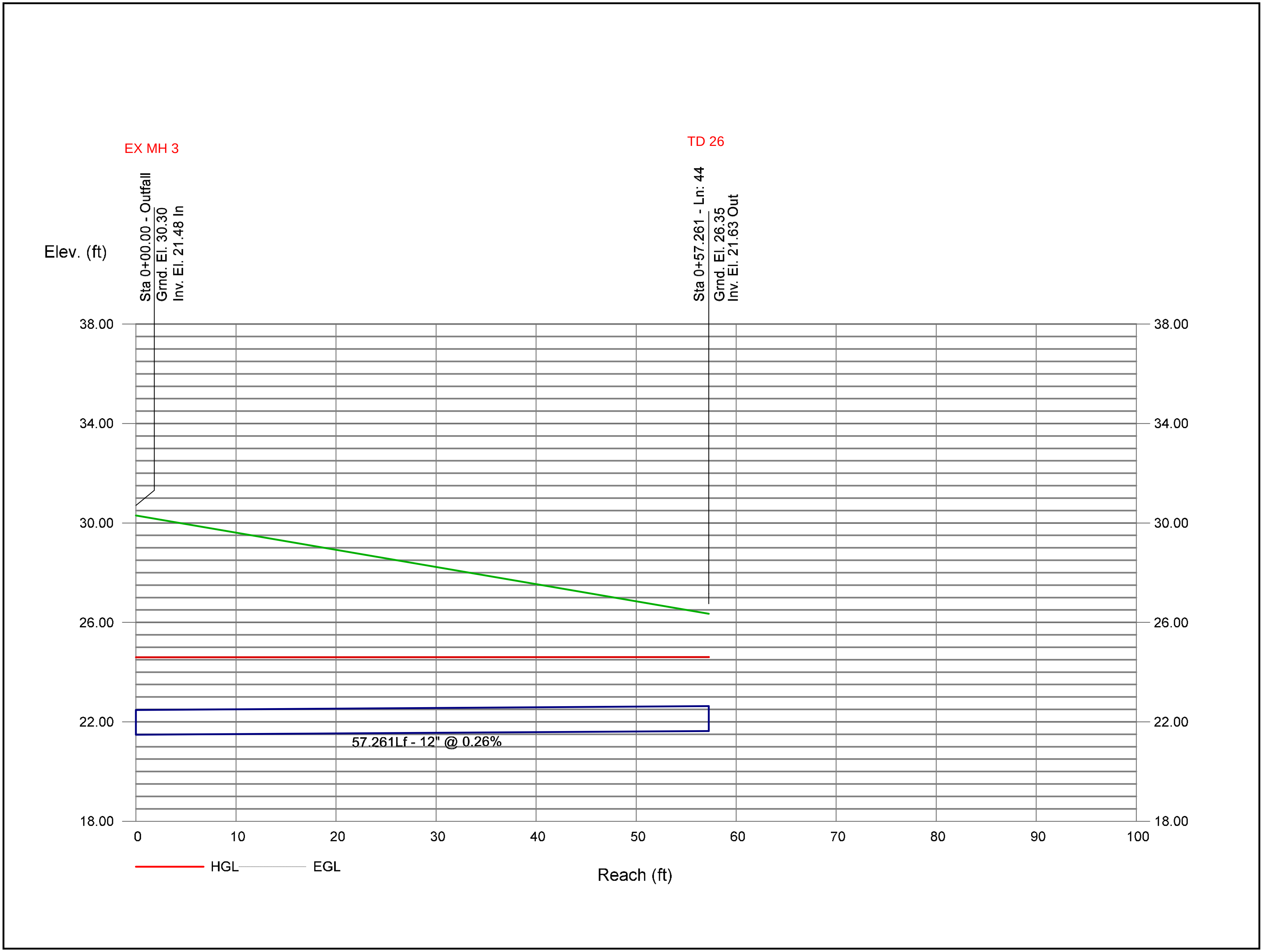


ATTACHMENT D

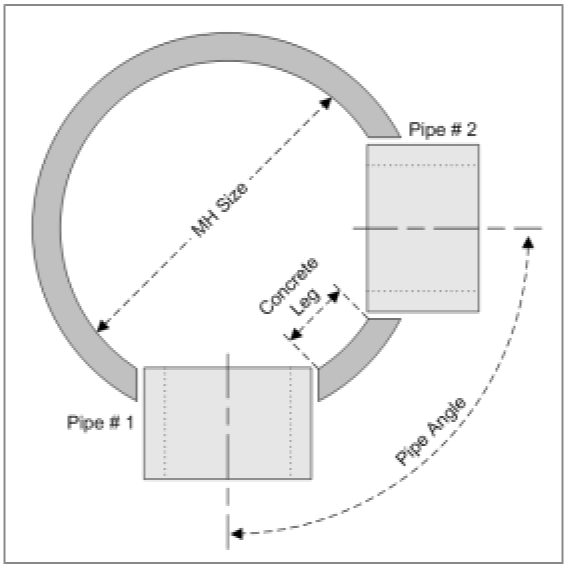
STORM SEWER
MANHOLE SIZING

PINNACLE ENGINEERING GROUP

MH IN 2

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 31.5

Pipe 1 Type: RCP

Size (inches): 15

Inv Elevation (feet): 26.89

Hole Req'd (inches): 24

48" Minimum Manhole Required For Size Specified

Pipe 2 Type: RCP

Size (inches): 15

Inv Elevation (feet): 26.89

Hole Req'd (inches): 24

48" Minimum Manhole Required For Size Specified

Pipe Angle (deg): 108

48" MANHOLE
Minimum Diameter Required
Leg Width (inches): 20.1

Print | Calculate | Reset

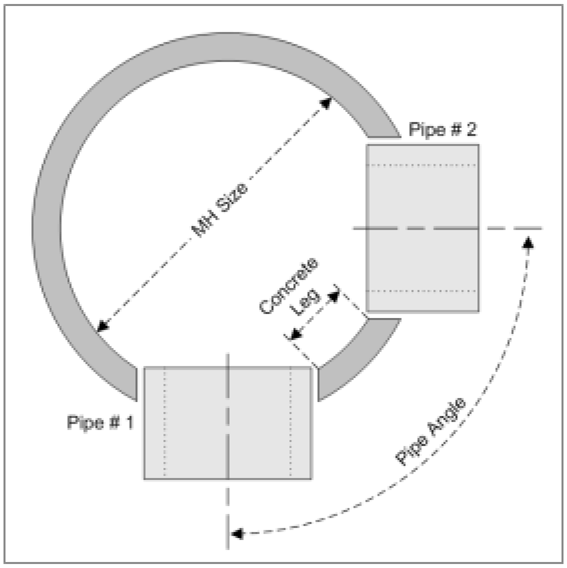
Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

MH IN 3

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 30.1

Pipe 1 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 12

Inv Elevation (feet): 26.41

Hole Req'd (inches): 22

Pipe 2 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 15

Inv Elevation (feet): 26.16

Hole Req'd (inches): 24

Pipe Angle (deg): 78

60" MANHOLE
Minimum Diameter Required
Leg Width (inches): 17.3

Print | Calculate | Reset

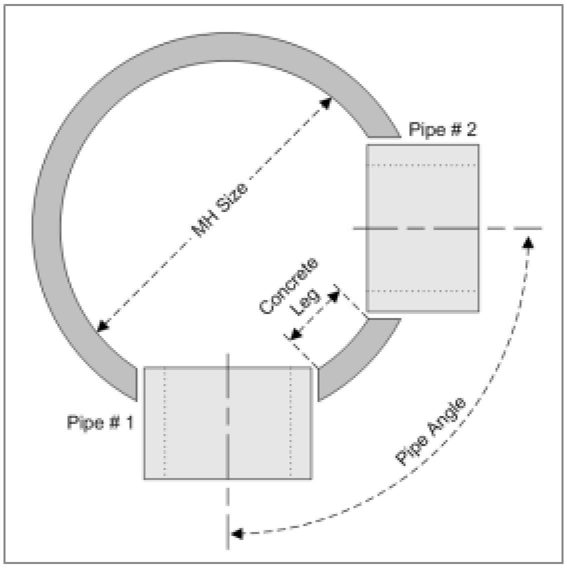
Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

MH 6

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 32.2

Pipe 1 Type: RCP

Size (inches): 12

Inv Elevation (feet): 29.14

Hole Req'd (inches): 22

48" Minimum Manhole Required For Size Specified

Pipe 2 Type: PVC

Size (inches): 6

Inv Elevation (feet): 29.39

Hole Req'd (inches): 11

48" Minimum Manhole Required For Size Specified

Pipe Angle (deg): 97

48" MANHOLE
Minimum Diameter Required
Leg Width (inches): 23.8

Print | Calculate | Reset

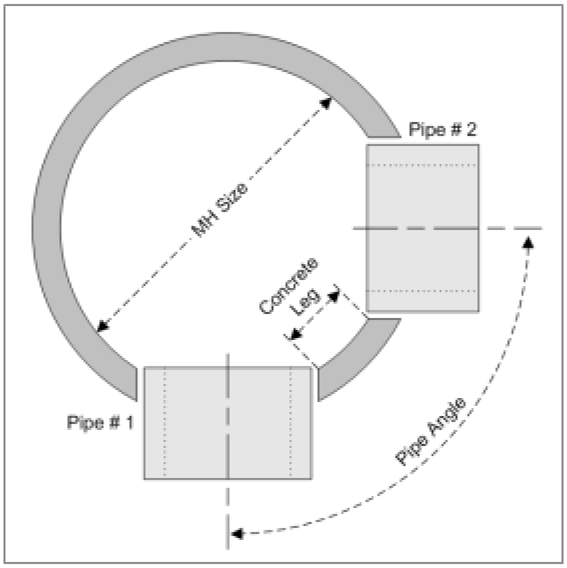
Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

MH 7

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 29.9

Pipe 1 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 12

Inv Elevation (feet): 26.5

Hole Req'd (inches): 22

Pipe 2 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 12

Inv Elevation (feet): 26.5

Hole Req'd (inches): 22

Pipe Angle (deg): 121

48" MANHOLE
Minimum Diameter Required
Leg Width (inches): 27.8

Print Calculate Reset

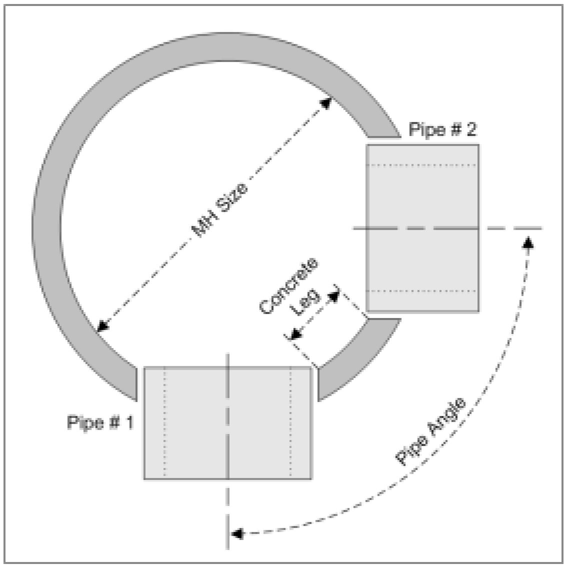
Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

MH 8

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 34.5

Pipe 1 Type: RCP

Size (inches): 12

48" Minimum Manhole Required For Size Specified

Inv Elevation (feet): 26.29

Hole Req'd (inches): 22

Pipe 2 Type: PVC

Size (inches): 6

48" Minimum Manhole Required For Size Specified

Inv Elevation (feet): 26.79

Hole Req'd (inches): 11

Pipe Angle (deg): 57

60" MANHOLE
Minimum Diameter Required
Leg Width (inches): 13.1

Print | Calculate | Reset

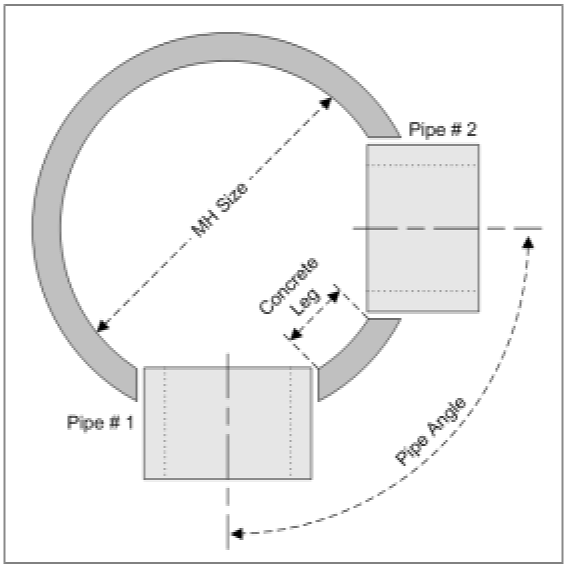
Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

MH 9

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 33.3

Pipe 1 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 15

Inv Elevation (feet): 25.91

Hole Req'd (inches): 24

Pipe 2 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 15

Inv Elevation (feet): 25.91

Hole Req'd (inches): 24

Pipe Angle (deg): 140

48" MANHOLE
Minimum Diameter Required
Leg Width (inches): 33.5

Print | Calculate | Reset

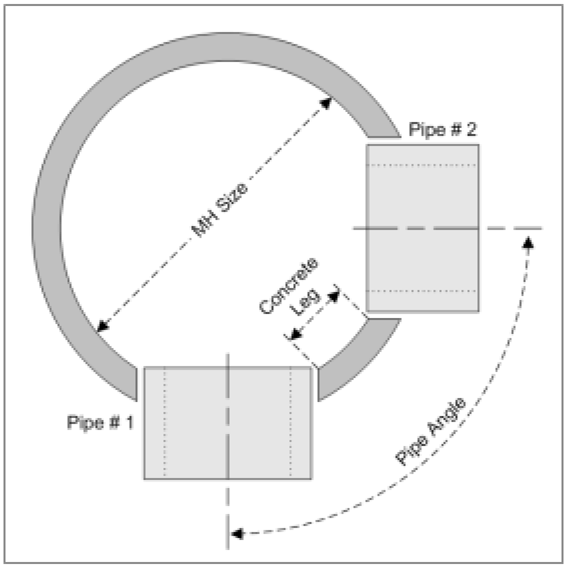
Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

MH IN 12

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 30.50

Pipe 1 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 12

Inv Elevation (feet): 27.41

Hole Req'd (inches): 22

Pipe 2 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 12

Inv Elevation (feet): 27.41

Hole Req'd (inches): 22

Pipe Angle (deg): 149

48" MANHOLE
Minimum Diameter Required
Leg Width (inches): 39.6

Print **Calculate** **Reset**

Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

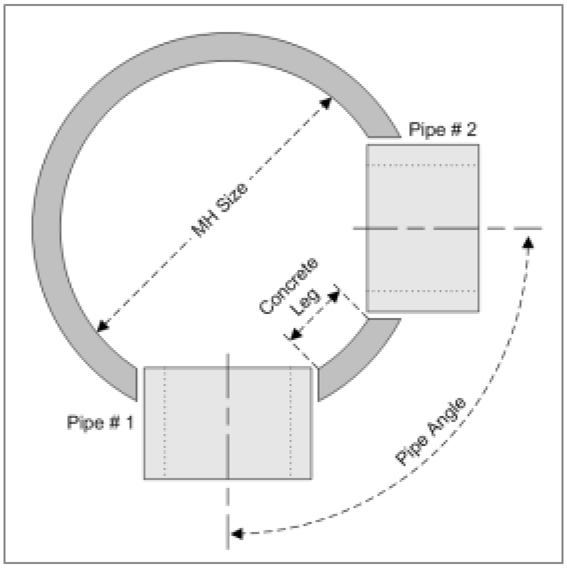
Contact Us

MH 13

Version 2.0.6.0

Cretex Concrete Products
the shape of solutions

Manhole Sizing Calculations



Top of Casting (feet): 35.5

Pipe 1 Type: RCP

Size (inches): 18

Inv Elevation (feet): 26.83

Hole Req'd (inches): 28

48" Minimum Manhole Required For Size Specified

Pipe 2 Type: RCP

Size (inches): 12

Inv Elevation (feet): 27.33

Hole Req'd (inches): 22

48" Minimum Manhole Required For Size Specified

Pipe Angle (deg): 100

48" MANHOLE
Minimum Diameter Required
Leg Width (inches): 15.6

Print | Calculate | Reset

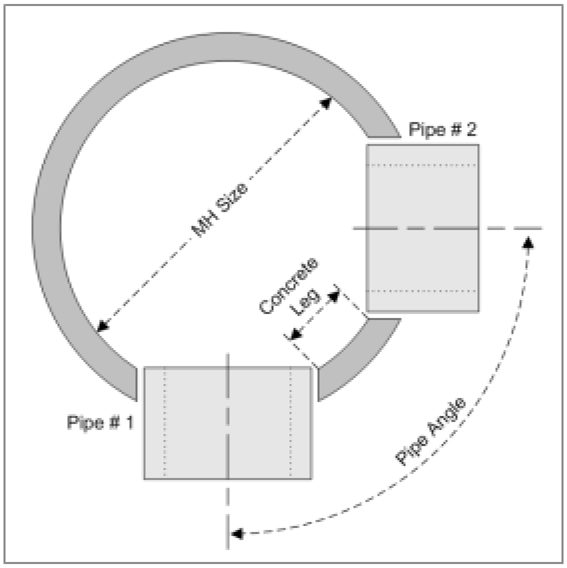
Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

MH 17

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 32

Pipe 1 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 18

Inv Elevation (feet): 26.21

Hole Req'd (inches): 28

Pipe 2 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 18

Inv Elevation (feet): 26.21

Hole Req'd (inches): 28

Pipe Angle (deg): 150

48" MANHOLE
Minimum Diameter Required
Leg Width (inches): 32.9

Print | Calculate | Reset

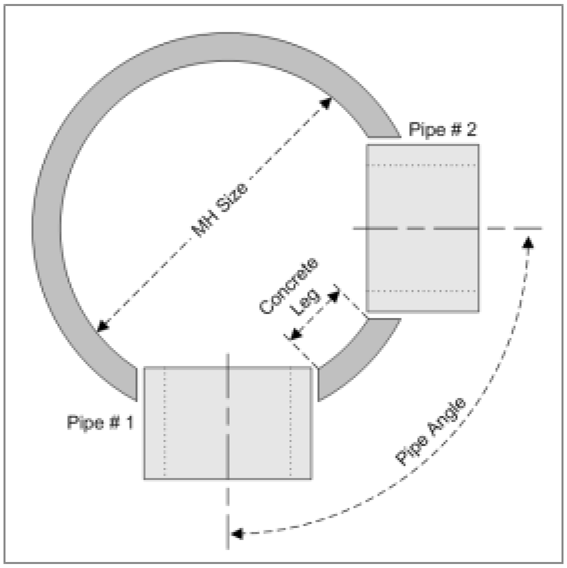
Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

MH 18

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 31.05

Pipe 1 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 18

Inv Elevation (feet): 26.12

Hole Req'd (inches): 28

Pipe 2 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 18

Inv Elevation (feet): 26.12

Hole Req'd (inches): 28

Pipe Angle (deg): 159

48" MANHOLE
Minimum Diameter Required
Leg Width (inches): 36.7

Print | Calculate | Reset

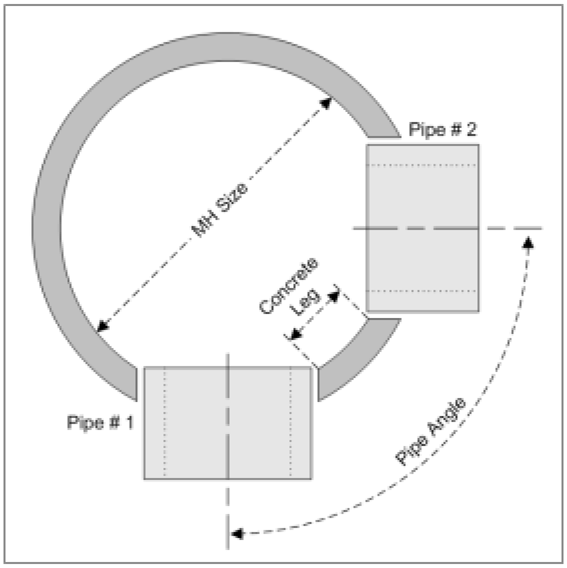
Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

MH 21

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 29.5

Pipe 1 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 12

Inv Elevation (feet): 26.39

Hole Req'd (inches): 22

Pipe 2 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 12

Inv Elevation (feet): 26.39

Hole Req'd (inches): 22

Pipe Angle (deg): 64

60" MANHOLE
Minimum Diameter Required
Leg Width (inches): 11.0

Print Calculate Reset

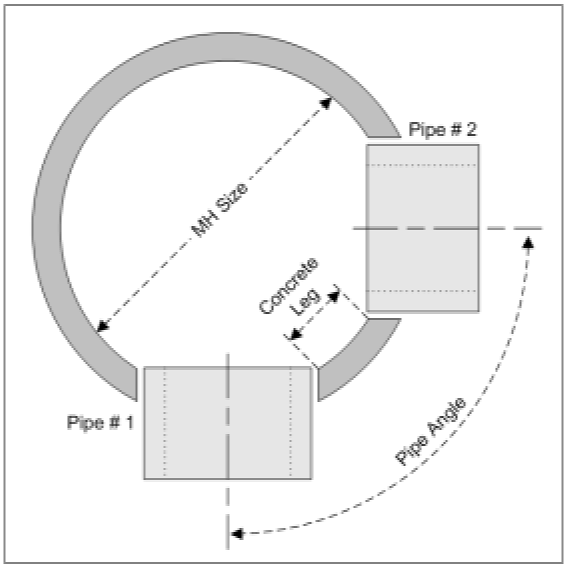
Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

MH IN 22

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 29.63

Pipe 1 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 12

Inv Elevation (feet): 25.78

Hole Req'd (inches): 22

Pipe 2 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 21

Inv Elevation (feet): 25.03

Hole Req'd (inches): 30

Pipe Angle (deg): 64

72" MANHOLE
Minimum Diameter Required
Leg Width (inches): 13.8

Print **Calculate** **Reset**

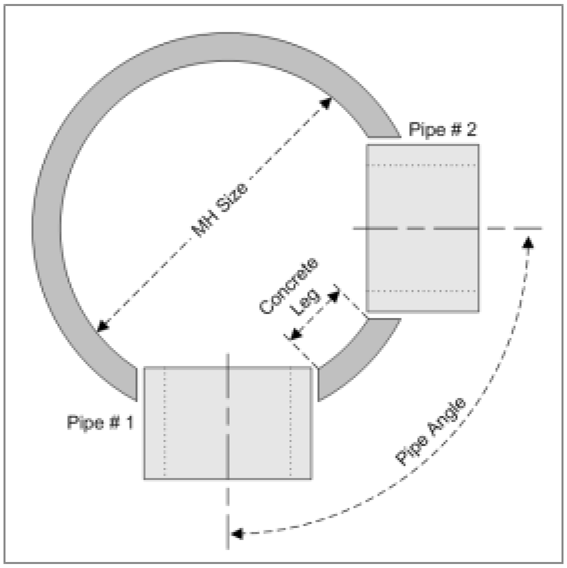
Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

MH 25

Version 2.0.6.0

Manhole Sizing Calculations



Top of Casting (feet): 31

Pipe 1 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 12

Inv Elevation (feet): 21.75

Hole Req'd (inches): 22

Pipe 2 Type: RCP

48" Minimum Manhole Required For Size Specified

Size (inches): 12

Inv Elevation (feet): 21.75

Hole Req'd (inches): 22

Pipe Angle (deg): 128

48" MANHOLE
Minimum Diameter Required
Leg Width (inches): 30.8

Print **Calculate** **Reset**

Manhole size calculations are approximate and for information only.
Please contact your local office for possible alternate design options.

Contact Us

ATTACHMENT E

Geotechnical Report

PINNACLE ENGINEERING GROUP

Geotechnical Engineering Exploration and Analysis

Proposed Apartment Buildings
2300 W. St. Paul Avenue
Waukesha, Wisconsin

Prepared for:

VJS Construction Services
Pewaukee, Wisconsin

November 5, 2020
Giles Project No. 1G-2009018

GILES
ENGINEERING ASSOCIATES, INC.

GILES
ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

• Atlanta, GA
• Dallas, TX
• Los Angeles, CA
• Manassas, VA
• Milwaukee, WI

November 4, 2020

VJS Construction Services
W233 N2847 Roundy Circle West
Pewaukee, WI 53072

Attention: Mr. Adam Lewis
Senior Project Manager

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Apartment Buildings
2300 W. St. Paul Avenue
Waukesha, Wisconsin
Giles Project No. 1G-2009018

Dear Mr. Lewis:

As requested, Giles Engineering Associates, Inc. conducted a *Geotechnical Engineering Exploration and Analysis* for the proposed project. The accompanying report describes the services that were performed, and it provides geotechnical-related findings, conclusions, and recommendations that were derived from those services.

We sincerely appreciate the opportunity to provide geotechnical services for the proposed project. Please contact the undersigned if there are questions about the report, or if we may be of further service.

Very truly yours,

GILES ENGINEERING ASSOCIATES, INC.

Andrew J. Globig
Andrew J. Globig, E.I.T. DA
Staff Professional I

D M. Cornale
David M. Cornale, P.E.
Sr. Geotechnical Consultant

WISCONSIN
DAVID M. CORNALE
E-43336
GREENFIELD
WI
PROFESSIONAL ENGINEER

Distribution: VJS Construction Services
Attn: Mr. Adam Lewis (pdf via email: alewis@vjscs.com)

N8 W22350 Johnson Drive • Suite A1 • Waukesha, WI 53186
262/544-0118 • Fax 262/549-5868 • E-Mail milwaukee@gilesengr.com

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED APARTMENT BUILDINGS
2300 W ST. PAUL AVENUE
WAUKESHA, WISCONSIN
GILES PROJECT NO. 1G-2009018

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED APARTMENT BUILDINGS
2300 W ST. PAUL AVENUE
WAUKESHA, WISCONSIN
GILES PROJECT NO. 1G-2009018

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. ("Giles") conducted for the proposed development. The *Geotechnical Engineering Exploration and Analysis* included a geotechnical subsurface exploration program, geotechnical laboratory services, and geotechnical engineering. The scope of each service area was narrow and limited, as directed by our client, and based on our understanding and assumptions about the proposed project. Service areas are briefly described later. Environmental consulting services were beyond our authorized scope for this project.

Geotechnical-related recommendations for design and construction of the foundations, below-ground parking level, and elevator pits for the buildings are provided in this report. Site preparation recommendations are given, but are only preliminary, as the means and methods of site preparation will depend on factors that were unknown when this report was prepared. Those factors include, but are not limited to, the weather before and during construction, subsurface conditions that are exposed during construction, and finalized details of the proposed development. Environmental consulting was beyond our scope of services for this project.

2.0 SITE DESCRIPTION

The subject site is located north of Fox Run Boulevard, approximately 650 feet west of W. St. Paul Avenue in Waukesha, Wisconsin. The site area is shown on the *Test Boring Location Plan*, enclosed as Figure 1 in Appendix A. When the test borings (described later) were performed, the site was vacant. Topographically, the site was relatively flat and level. Ground elevations at the test borings varied between ±El. 30 to ±El. 31.5; those elevations are referenced to the *Existing Boundry Survey*, revised March 12, 2020, prepared by Jahnke & Jahnke Associates, LLC. Historical aerial photographs show that a former multi-tenant building existed at the site, and that the structures were razed before subsurface exploration was performed.

3.0 PROJECT DESCRIPTION

Three separate apartment buildings will be constructed at the site. It is understood that the proposed apartments will be two-story masonry structure with a wood-truss or bar-joist roof system. The apartments will have a full below-ground parking level with an entrance/exit ramp. The apartment buildings will also have elevators. Elevator pits are assumed to be a maximum of 4 feet deep, measured from the parking-level floor. Bearing walls and columns will assumedly support the buildings. Maximum foundation loads are unknown, but are assumed to be 6,000 pounds per lineal foot (plf) from bearing walls and 100 kips per column. The floors of the parking level are planned to be ground-bearing concrete slabs; the maximum floor load is expected to be 100 pounds per square foot (psf).

The proposed project will include the construction of a parking lot on the north side of the development. Also, certain sections of existing drives will be reconstructed. The proposed development is shown on the *Test Boring Location Plan*, which was prepared using the *Preliminary Site Plan* (Sheet C1.01), revised September 8, 2020, prepared by VJS Construction Services.

Final elevations for the proposed buildings and parking lot were not provided to Giles; however, based on the existing site grades, it is anticipated that only minor grading (four-feet maximum) is expected in the proposed building and pavement areas. Excluding the excavations for the below grade parking level area and ramp entrance/ exit for the below grade parking level.

Due to the relatively moderately shallow water table (discussed later), the proposed development is recommended to be situated as high as practical. Giles must be notified if the actual floor elevations and/or pavement grades will differ from those described above. Also, it is critical that Giles review the project plans before construction begins, thereby allowing us the opportunity to revise this report, if needed. If the building is too low, severe groundwater-related problems may occur during and after construction.

4.0 GEOTECHNICAL SUBSURFACE EXPLORATION PROGRAM

To explore subsurface conditions, ten test borings were conducted at the site, using a mechanical drill-rig. Test Borings 1 through 8 were in the excavated proposed building areas. Those test borings were advanced to ±21 feet below-ground. Test Borings 9 and 10 were in the parking lot area and were ±11 feet deep. Test boring locations were positioned on-site relative to apparent property lines, features of the site, and by estimating right angles. Approximate locations of the test borings are shown on the *Test Boring Location Plan*.

Samples were collected from each test boring, at certain depths, using the Standard Penetration Test (SPT), conducted with the drill rig. A brief description of the SPT is given in Appendix B, along with descriptions of other field procedures. Immediately after sampling, select portions of the SPT samples were placed in containers that were labeled at the site for identification. A Standard Penetration Resistance value (N-value) was determined from each SPT. N-values are reported on the *Test Boring Logs* (in Appendix A), which are records of the test borings.

Ground elevations at the test borings were estimated by topographic contour lines on the provided *Existing Boundry Survey*. The test boring elevations are noted on the *Test Boring Logs*, and are considered accurate within about one foot.

The boreholes were backfilled upon completion. However, backfill material will likely settle and/or heave, creating a hazard that can injure people and animals. Borehole areas should, therefore, be carefully and routinely monitored by the property owner or others; settlement and/or heave of backfill materials should be repaired immediately. Giles will not monitor or repair boreholes.

5.0 GEOTECHNICAL LABORATORY SERVICES

Soil samples that were retained from the test borings were transported to Giles' geotechnical laboratory, where they were classified using the descriptive terms and particle-size criteria shown on the *General Notes* in Appendix D, and by using the Unified Soil Classification System (ASTM D 2488) as a general guide. The classifications are shown on the *Test Boring Logs*, along with horizontal lines that show estimated depths of material change. Field-related information pertaining to the test borings is also shown on the *Test Boring Logs*. For simplicity and abbreviation, terms and symbols are used on the *Test Boring Logs;* the terms and symbols are defined on the *General Notes*.

Calibrated penetrometer resistance, unconfined compression (without controlled strain), and water content tests were performed on select SPT samples to evaluate their general engineering properties. Results of the laboratory tests are shown on the *Test Boring Logs*, included in Appendix A. Because SPT samples were used, which are categorized as being disturbed samples, results of the unconfined compression and calibrated penetrometer resistance tests are considered to be approximate.

6.0 MATERIAL CONDITIONS

Because material sampling at the test borings was discontinuous, it was necessary to estimate conditions between sample intervals. Estimated conditions at the test borings are briefly discussed in this section and are described in more detail on the *Test Boring Logs*. The conclusions and recommendations in this report are only based on the estimated conditions.

6.1. Fill Material

Soil classified as fill was at the surface of Test Borings 1 through 6 and 9. The fill material extended to about 2 feet below-ground, except at Test Borings 9, where the fill extended to about 9 feet below grade surface. The fill material generally consisted of gravelly silty sand. Fill material was variable, with low to relatively high strength characteristics.

6.2. Native Soil

Native soil was below the materials described above, and was present to the ±11- to ±21-foot termination depths at the test borings. In general, native soil mostly consisted of sand (variable gradations with variable amounts of silt and gravel) underlain by sandy silt with lenses of silty fine sand. However, native silty clay with sandy silt lenses was at deeper depths at the test borings. Based on SPT N-values, native granular soil (sand and sandy silt) had relative densities of loose and firm, whereas native cohesive soil (silty clay) had comparative consistencies of medium stiff to stiff, based on laboratory testing.

7.0 GROUNDWATER CONDITIONS

Based the colors and moisture conditions of the retained soil samples, and the depth that groundwater was identified within the test borings, it is estimated that the water table varied between about 6½ and 9 feet below-ground at the test boring locations, when the test borings were conducted. In general, the water table was shallower in lower areas and deeper in areas of higher elevation. Using the test boring elevations, it is estimated that the water table was between ±El. 22 and ±El. 25. Also, the site is likely subject to perched-groundwater conditions, where groundwater perches above the water table, such as within existing fill.

The estimated water table depth/elevation is only an approximation. The water table could be higher or lower than estimated. If a more precise determination of the water table depth/elevation is needed, groundwater observation wells are recommended to be installed and monitored at the site. Giles can install and monitor observation wells, if it is decided that a more detailed determination of the water table depth/elevation is needed.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1. Site Development Considerations

As noted above, it is estimated that the water table was between ±El. 22 and ±El. 25 at the locations of the test borings, when our field services were conducted; those elevations are referenced to the *Existing Boundry Survey*. Considering the groundwater conditions, the below grade parking-level is recommended to be at or above El. 27, El. 27, and El. 26 for the east, central and west building, respectively. Because of the groundwater conditions, it is recommended that Giles review the finalized plans and specifications for the proposed development prior to its construction. Depending on that review, this report might need to be revised. This report is strictly based on Giles' assumption that the below-grade parking floor will be at El. 27, El. 27, and El. 26 for the east, central and west building, respectively.

Depending on the conditions during construction, it might be necessary to install a layer of crushed stone in the below grade parking level excavation to stabilize the subgrade, and to develop a working mat for construction. The actual thickness and gradation of a crushed stone layer should be determined by a geotechnical engineer, based on the conditions within the excavation.

It is understood that currently or previously environmentally impacted areas are present within the site, as shown on the provided plans. Therefore, special measures may be needed for site development and building construction due to environmental considerations. Environmental evaluation is not within the scope of this report. It is recommended that any special measures required for environmental considerations be provided for our review to determine the impacts on the geotechnical engineering recommendations provided in this report. Revision to this report may be needed, dependent on specific measures that may be required.

8.2. Seismic Design Considerations

A soil Site Class D is recommended for seismic design. By definition, Site Class is based on the average properties of subsurface materials to 100 feet below-ground. Because 100-foot test borings were not requested or authorized, it was necessary to estimate the Site Class based on the test borings, presumed area geology, and the International Building Code.

8.3. Building Foundation Recommendations

Spread-footing foundations are recommended for the proposed apartment buildings. However, existing fill is unsuitable for direct or indirect support of foundations. Each footing must bear on suitable native soil or on new engineered fill or lean-concrete backfill (both discussed below) placed on suitable native soil. The foundations for the central and western building are recommended to be designed using a 4,000 pound per square foot (psf) maximum, net, allowable soil bearing pressure. Additionally, a maximum, net, allowable soil bearing pressure of 3,000 psf is recommended for the eastern building foundations. For geotechnical considerations, and regardless of the calculated foundation-bearing stress, strip footings are recommended to be at least 18 inches wide and isolated footings are recommended to be at least 24 inches wide/long. Also, from a geotechnical perspective, and because of the variable subsurface conditions, foundation walls are recommended to be constructed of reinforced cast-in-place concrete. It is recommended and assumed that a structural engineer will provide specific foundation details, including footing dimensions, reinforcing, etc.

A minimum 48-inch foundation-embedment depth is required by the local building code. Footings for perimeter walls and other exterior elements of the proposed building are, therefore, recommended to bear at least 48 inches below the finished ground-grade, or to the depth required by the governing building code. Interior footings could be directly below the parking-level floor slab, since it is assumed that the parking level will be heated and foundation-support soil will not freeze. Because the apartments will have a full below-ground parking level, it is assumed that foundations for the east, central and west buildings will bear at ±El. 26, ±El. 26, and ±El. 25, respectively. However, foundations in the area of the entrance/exit ramp will step down to ±El. 23, ±El. 23, and ±El. 22 to meet the 48-inch embedment-depth criteria, respectively.

The following table shows estimated depths and elevations of suitable-bearing native soil at Test Borings 1 through 8, conducted in or near the proposed building areas. It is important to note that the depth of suitable-bearing native soil could be deeper or shallower away from the test borings. Because fill material and lower-strength native materials were encountered at the test borings, testing and approval of foundation-support materials by a geotechnical engineer during construction is critical.

TABLE 1 ESTIMATED DEPTH AND ELEVATION OF SUITABLE-BEARNG NATIVE SOIL			
Apartment Building	**Test Boring Number**	**Estimated Depth of Suitable-Bearing Native Soil**	**Estimated Elevation of Suitable-Bearing Native Soil**
East	1	±2 feet	±El. 28.5
	2	±2 feet	±El. 27.8
	3	±2 feet	±El. 27.5
Central	4	±2 feet	±El. 29.2
	5	±2 feet	±El. 29.5
West	6	±2 feet	±El. 28.8
	7	±½ foot	±El. 30.7
	8	±½ foot	±El. 30.7

Notes:
- Based on a 4,000 psf maximum, net, allowable soil bearing capacity (3,000 psf maximum bearing pressure at the western building)
- Depths are referenced to the site grades during the geotechnical subsurface exploration program.
- Elevations are referenced to the topographic contour lines on the *Existing Boundry Survey*.

Based on the assumed foundation-bearing elevations noted above, and considering the estimated depths and elevations of suitable-bearing native soil shown in Table 1, extensive over-excavation is not expected to be needed for foundation construction. However, some overexcavation/stabilization might be necessary to develop suitable support for foundations in the below grade parking level of the building, due to the shallow groundwater and deeper moisture sensitive soils. The actual areas and depths of over-excavation are recommended to be determined during construction, on a location-by-location basis, with the assistance of a geotechnical engineer during full-time observation and testing.

Foundation excavations are recommended to be dug with a smooth-edge backhoe bucket to develop a relatively undisturbed bearing grade. A toothed bucket will likely disturb foundation-bearing soil more than a smooth-edge bucket, thereby making soil at the excavation base more susceptible to saturation and instability, especially during adverse weather. It is critical that contractors protect foundation-support soil and foundation construction materials (concrete and reinforcing). Also, engineered fill is recommended to be placed and compacted in benched excavations along foundation walls immediately after the foundation walls are capable of supporting lateral pressures from backfill, compaction, and compaction equipment. The use of earth-formed footing construction methods is not expected to be feasible, due to the granular site soil.

<u>Foundation Support Soil Requirements</u>

Existing fill is unsuitable for direct or indirect support of foundations. Each footing must be directly supported by suitable native soil or by new engineered fill or lean-concrete backfill (both discussed below) placed directly on suitable native soil. Based on the recommended 4,000 psf maximum, net, allowable soil bearing capacity, native non-cohesive soil, such as sand, within foundation influence zones is recommended to have a corrected N-value (determined from SPTs and correlated from other in-situ tests) of at least 14. For the recommended 3,000 psf maximum, net, allowable soil bearing capacity at the western building, native non-cohesive soil, such as sand, within foundation influence zones is recommended to have a corrected N-value (determined from SPTs and correlated from other in-situ tests) of at least 10, based on the recommended bearing capacity. It is further recommended that the strength characteristics of soil within all foundation influence zones (determined by a geotechnical engineer during construction) meet or exceed the recommended values, unless Giles approves other values.

Because of the existing fill material, shallow groundwater, and moisture sensitive soils, full-time evaluation of foundation-support soil by a geotechnical engineer during foundation excavation and foundation construction is recommended. The purpose of the recommended evaluation is (1) to confirm that the foundations will be properly supported by suitable native soil, (2) to determine areas and depths of over-excavation, and (3) to confirm that the subsurface conditions are similar to those described on the *Test Boring Logs*. If a firm other than Giles performs the recommended support-soil evaluation, Giles must be notified if the composition or strength characteristics of foundation-support soil differ from the subsurface conditions shown on the *Test Boring Logs*, thereby allowing us the opportunity to revise this report, if needed. Without evaluation and approval of foundation-support soil by a geotechnical engineer, the proposed buildings could be improperly supported, which could lead to excessive settlement and other structural problems. All OSHA requirements must be strictly followed when evaluating foundation-support soil. Excavations that do not meet OSHA safety guidelines must not be entered.

Unsuitable materials beneath foundation areas could be replaced with engineered fill consisting of properly compacted well-graded aggregate. Aggregate fill is recommended to consist of dense-graded crushed stone that meets the gradation requirements of *dense-graded base* (1¼-inch) in Section 305 of the Wisconsin Department of Transportation Standard Specifications (2019). Aggregate with other gradation characteristics could possibly be used, but should be approved by a geotechnical engineer before the material is placed. Also, aggregate with other gradation characteristics might need to be underlain by geotextile, which will serve as a separator. If engineered fill is used to replace unsuitable materials, lateral over-excavation of the unsuitable materials will also be required, in addition to the required vertical over-excavation. The overall width of lateral over-excavation will depend on the vertical over-excavation depth. For estimating purposes, the minimum lateral over-excavation could be determined by extending an imaginary line outward and downward at a ratio of 1(horizontal):2(vertical) from the bottom edges of a footing pad, but the actual lateral extents of over-excavation are recommended to be approved by a geotechnical engineer during construction.

Lean Portland cement concrete (minimum 28-day compressive strength of 500 psi) could also be used to replace unsuitable materials beneath foundation areas, and is generally Giles' preferred backfill material within deeper over-excavations. Where lean concrete is used as backfill, footing construction must not begin until the lean concrete has gained sufficient strength. Also, over-excavations that are filled with lean concrete are recommended to be at least as wide (on all sides) as the footing pad that will be supported by the concrete, and excavation sidewalls are recommended to be plumb and parallel. To help control caving, lean-concrete backfill is recommended to be placed immediately after excavation. This trench-and-pour method requires close communication and scheduling between the general contractor, foundation contractor, concrete supply company, and geotechnical engineer. With a trench-and-pour method, a geotechnical engineer must observe excavations as they are made. Full-time observation by a geotechnical engineer is therefore recommended, as noted above.

Estimated Foundation Settlement

The post-construction total and differential settlements of a spread-footing foundation designed and constructed based on this report are estimated to be less than about 1 inch and ½ inch, respectively. The post-construction angular distortion is estimated to be less than about 0.002 inch per inch across a distance of 20 feet or more. Estimated settlements assume that the recommendations provided in this report will be followed, and that foundation-support soil will be evaluated and approved by a geotechnical engineer.

8.4. Parking-Level (Below-Ground) Recommendations

Geotechnical-related recommendations regarding the below-ground parking level for the proposed apartment buildings are provided below. The recommendations are based on the assumed elevation discussed above.

Parking-Level Floor Slab

The parking-level floor slab is recommended to be directly supported by suitable-bearing native soil and/or by new engineered fill placed on suitable-bearing native soil. Assuming a maximum 100 psf floor load, and from a geotechnical perspective, the parking level floor slab is recommended to be at least 5 inches thick; that thickness assumes that the 28-day compressive strength of concrete will be at least 3,500 psi. The parking level floor slab may be designed using a *Modulus of Subgrade Reaction* (k_{v1}) of 150 pounds per cubic inch (pci). It is recommended and assumed that a structural engineer will specify the floor slab thickness, reinforcing, joint details, and other parameters.

For moisture control only, a minimum 10-mil vapor retarder is recommended to be directly below the parking-level floor slab. It is recommended that the vapor retarder completely underlie the entire parking-level area and extend to all foundation walls. Abutting vapor retarders are recommended to be overlapped and taped. The vapor retarder is recommended to be in accordance with ASTM E 1745, entitled: *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*. If the base course has sharp, angular aggregate, protecting the retarder with a geotextile (or by other means) is recommended.

A minimum 6-inch-thick base course is recommended to be directly below the minimum 10-mil vapor retarder to serve as a capillary break and for drainage. Because the base course will be a component of the recommended drainage system (discussed below), the base-course material is recommended to consist of free-draining crushed stone approved by a geotechnical engineer. Base-course materials are recommended to be properly compacted. Also, depending on subgrade conditions, geotextile might need to be below the base materials; the need for a geotextile should be determined during construction with the assistance of a geotechnical engineer.

As described in Section 8.1, due to the shallow water table and depending on the conditions during construction, it might be necessary to install a layer of crushed stone in the parking level excavation to stabilize the subgrade, and to develop a working mat for construction.

The post-construction total and differential settlements of an isolated floor slab constructed according to this report are estimated to be less than about 0.5 inch and 0.3 inch, respectively, over a distance of about 20 feet. Estimated settlements assume that support materials will be will be approved by a geotechnical engineer immediately before floor slab construction.

<u>Drainage System Recommendations</u>

Continuous drainpipes are recommended to be along the interior and exterior sides of perimeter strip footings, thereby creating interior and exterior drainage loops around the parking level. Drainpipes could consist of conduits specifically manufactured for foundation drainage applications, such as Form-A-Drain® conduits. Manufactured foundation drains are recommended to be installed per the manufacturer’s recommendations. Circular drainpipes could also be used and are recommended to be minimum 4-inch-diameter perforated pipes suitable for foundation drainage. Circular drains are recommended to be directly adjacent to the footing pads, not atop footing flanges. Interior drainpipes are to be properly situated within the base-course layer below the parking level floor slab. It is recommended that a minimum 12-inch-thick layer of free-draining crushed stone surround exterior drainpipes, except that the crushed stone must not extend below the foundations and into the foundation-influence zone. Bleeder pipes are recommended to be cast in the perimeter strip-footing pads to serve as water conduits between interior and exterior drainpipes. Bleeder pipes are recommended to be 3 inches in diameter and about 8 to 12 feet on-center.

It is recommended that the drainage system discharge to a dedicated sump-pump basin situated within the parking level. The basin location should be determined based on architectural and structural details of the building. It is recommended that the basin have a sealed-and-bolted, airtight lid to prevent in-flow of subsurface gases, such as radon. Also, the basin is recommended to be equipped with a sump pump that has sufficient capacity. It is recommended that the sump pump be equipped with a battery backup to temporarily maintain pump operation in the event of a power failure. Piping for the sump pump should discharge a sufficient distance away from the proposed building (and other structures) to a suitable location where the possibility of ponded water will not be a nuisance or hazard, especially during cold weather when ponded water could freeze.

Perimeter Drainage Layer

Free-draining aggregate is recommended to be along the exterior side of the parking level walls. Crushed stone meeting the gradation requirements of ASTM No. 57 aggregate is recommended. The aggregate will serve as drainage media for the recommended drainage system, and is recommended to be at least 24 inches wide, measured from the outside face of the parking level walls. Also, the aggregate layer is recommended to be continuous along the length and height of the walls, except that pavement or a ±6-inch-thick layer of relatively impervious material is recommended to be above the drainage aggregate to reduce surface-water intrusion. Furthermore, the aggregate layer must extend to the base of the parking level-area footing pads, thereby creating a continuous drainage path to the perimeter drainage conduits.

Drainage aggregate that is placed adjacent to parking level walls is recommended to be compacted in maximum 8-inch-thick lifts, measured loose. Use of manual compaction equipment must be in accordance with current OSHA excavation and trench safety standards, and other applicable requirements. Manual compaction equipment should not be used within spaces that do not meet OSHA requirements. Drainage aggregate should not be excessively compacted. Excavations for parking level walls must be properly shored, sloped, or restrained. Also, parking level walls are recommended to be adequately braced before placing backfill to prevent the walls from moving or possibly even overturning during backfilling. Bracing must remain in-place until the top and bottom of the parking level walls are structurally restrained.

Lateral Pressure Design Parameters

The below-ground parking-level walls must be designed to resist lateral pressures from drainage backfill, adjacent soil, and any surface and subsurface surcharges. An equivalent "at-rest" fluid pressure of 60 pounds per square foot per foot of depth (psf/ft) is recommended for design of parking level walls. The recommended “at-rest” value is based on Giles’ assumption that drainage backfill will continuously abut the parking level walls and that the recommended drainage system will be installed and will remain functional. If drainage backfill and/or the drainage system are not installed, lateral pressures could exceed the recommended "at-rest" fluid pressure, possibly exceeding the lateral capacity of the walls.

If free-draining crushed stone is not installed along the parking level walls as recommended, and soil that is not free-draining abuts the walls, lateral pressures will likely exceed the recommended "at-rest" fluid pressure, possibly exceeding the lateral capacity of the walls. Cohesive (silty clay) soil should not be near below-ground walls due to potentially excessive pressures and insufficient drainage.

Lateral pressures caused by surface and subsurface surcharge loads must be added to the "at-rest" fluid pressure. Giles could provide supplemental recommendations regarding surface and subsurface surcharge loads on a case-by-case basis, but would require specific structural information. Parking level walls that are not designed to resist actual pressures could move laterally and possibly fail. It is recommended and assumed that a structural engineer will design the parking level walls.

8.5. Elevator Pit Recommendations

This report assumes that elevator pits will be a maximum of 4 feet deep, and that the floor of each pit will be at or above El. 22 for the western building, El. 23 for the central and eastern building. Based on that floor elevations, and considering the water-table depth, elevator pits are recommended to be watertight. Watertight construction is recommended to include permanent water-stops at all control joints, construction joints, cold joints, and at all other junctures where water could enter the elevator pits. Furthermore, it is expected that each elevator pit will need to be surrounded by a waterproof membrane. Waterproofing materials are recommended to be specified by a structural engineer or architect, and installed in accordance with the manufacturer's recommendations.

Elevator pits are recommended to be designed based on fully submerged conditions, assuming that water will completely surround the pits. Because the elevator pits are recommended to be designed based on submerged conditions, elevator-pit walls are recommended to be designed to resist lateral earth pressure and hydraulic lateral pressure, and the floor of each pit is recommended to be designed to resist buoyant uplift. Buoyant uplift must, however, be determined by a structural engineer based on final details of the elevator pits. The structural engineer should also determine if anchors or increased concrete thickness are needed to resist uplift of the elevator pits, including the elevator-pit floors.

It is assumed that the elevator-pit walls will be cast against or near existing soil, without a surrounding layer of free-draining aggregate. Elevator-pit walls are recommended to be designed for an equivalent "at-rest" fluid pressure of 97 psf/ft. Lateral pressures caused by surface and subsurface surcharge loads (such as the parking-level floor load) must be added to the "at-rest" fluid pressure. Giles could provide supplemental recommendations regarding surface and subsurface surcharge loads on a case-by-case basis, but would require specific structural information. Elevator-pit walls that are not designed to resist the actual pressures could move laterally and possibly fail.

8.6. Pavement Recommendations

Giles was not given information regarding traffic conditions for the proposed parking lot. Therefore, to provide pavement recommendations, it was necessary to use an arbitrarily selected traffic condition. The pavement section given below is for a maximum daily traffic condition consisting of five 18,000-pound equivalent single axle loads (ESALs). The pavement section is only for light-duty areas, such as areas that are subject to passenger vehicles along with infrequent heavy vehicles. Evaluation for design of roadway pavements at the site was not performed. Giles could provide recommendations for a heavier traffic condition after specific details regarding the expected traffic are provided to us.

It is recommended that the project owner, developer, civil engineer, and other design professionals involved with the project confirm that the arbitrarily selected traffic condition is appropriate. If requested, Giles can provide supplemental pavement recommendations based on other traffic conditions, such as if a heavy-duty pavement section is needed. If the pavement section is subject to traffic greater than assumed, increased maintenance and premature failure could occur.

A California Bearing Ratio (CBR) test is used to determine soil support parameters for pavement design. Since a CBR test was not authorized for this project, it was necessary for Giles to assume a CBR design value. Considering that gravelly silty sand that was encountered at the test borings, the following pavement sections are based on a gravelly silty sand subgrade and an assumed field CBR value of 10. Engineered fill that is placed in proposed pavement areas is recommended to have a field CBR value equal to or greater than 10, and the fill is recommended to be placed and compacted per this report.

The recommended pavement section is shown below, and is based on the assumed traffic condition and the assumed CBR value. Depending on the site conditions during construction, the subgrade might need to be improved, especially if construction is during or after adverse weather. There are various methods of subgrade improvement, including the use of geosynthetics (geogrids or geotextile), coarse aggregate modification, and soil stabilization with hydrated lime or Portland cement. The need for subgrade improvement should be determined during construction with the assistance of a geotechnical engineer.

TABLE 1
RECOMMENDED ASPHALT-CONCRETE PAVEMENT

Materials	Thickness	Wisconsin DOT Standard Specifications
Hot-Mix Asphalt Surface Course	1.5 inches	Section 460
Hot-Mix Asphalt Binder Course	2.0 inches	Section 460
Dense-Graded Aggregate Base-Course	6 inches	Section 305 1¼-inch Crushed Stone

Portland cement concrete pavement is recommended in high-stress areas, such as the lot entrance/exit aprons and other high stress areas. Concrete pavement is recommended to be at least 6 inches thick, and is recommended to be underlain by a minimum 6-inch-thick aggregate base-course. The concrete should have a minimum 28-day compressive strength of 4,000 psi with 4 to 7 percent air entrainment. Control-joint spacing should be determined in accordance with the current ACI code. Expansion joints should be provided where pavement abuts fixed objects. Materials and construction procedures for concrete pavement are recommended to be per Wisconsin DOT Standard Specifications Section 415 for concrete and Section 305 for base course.

Pavement Drainage Considerations

Because of the potential for shallow groundwater conditions, an under-pavement drain system is recommended for the entire pavement area to collect and remove water beneath the pavement. Installing an under-pavement drain system could increase the service life of the new pavement, it could help preserve the condition of the pavement, and it could reduce the need for non-routine maintenance and repair of the pavement.

It is recommended that a civil engineer design the under-pavement drainage system based on details of the site. The under-pavement drain system should, at a minimum, consist of finger or circular drains, installed within free drainage aggregate backfill along with a sloped subgrade, that discharges water to the stormwater system at catch basins and other low areas. Additional under-drains, such as drainage trenches, may be needed, dependent on-site grades, the locations and quantities of stormwater inlets and other aspects of the proposed pavement areas. While the primary purpose of the drainage system is to collect groundwater from pavement areas, the drainage system is recommended to be configured to collect (intercept) water from beneath sidewalks. Even with the recommended under-pavement drainage system, pavement damage and other problems should be expected due to frost-heave and subsequent thaw-related strength loss of subgrade soil. In some areas, frost-heave could be significant.

General Pavement Considerations

The pavement recommendations assume that the subgrade will be prepared per report, the base course will be properly drained, and a geotechnical engineer will observe pavement construction. Pavement was designed based on AASHTO parameters for a twenty-year design period. Pavement maintenance along with a major rehabilitation after about 8 to 10 years should be expected. Local codes may require specific testing to determine soil support characteristics and/or minimum pavement section thickness might be required. Additional pavement maintenance might be needed due to the low strength native soils.

8.7. Generalized Site Preparation Recommendations

This section deals with site preparation, including preparation of floor slab, and engineered fill areas. The means and methods of site preparation will greatly depend on the weather conditions before and during construction, the subsurface conditions that are exposed during earthwork operations, and the final details of the proposed development. Therefore, only generalized site preparation recommendations are given.

In addition to being general, the following site preparation recommendations are abbreviated; the *Guide Specifications* in Appendix D gives further recommendations. The *Guide Specifications* should be read along with this section. Also, the *Guide Specifications* are recommended to be used as an aid to develop the project specifications.

Removal and Stripping

Structures formerly existed at the site, as discussed in Section 2.0. Remnants (if any) of the former structures are recommended to be completely removed to at least several feet beyond the proposed building footprint. Disposal of rubble and debris is recommended to be in accordance with local, state, and federal regulations for the material type. Outside the proposed building area, it might be feasible for existing foundations to remain, provided the foundations are stable, are cut off at least three feet below the planned subgrade, and hollow cores are grouted solid. Remaining floor slabs that are outside the proposed building area could possibly also stay in-place, provided that the slabs are at least three feet below the planned finished grade, are perforated (broken) on a maximum two-foot grid, are “seated” into the subgrade for stability, and are covered with a minimum 12-inch-thick layer of well-graded, free-draining granular material for drainage. It is important to note that structural remnants that are left in-place might cause excavation difficulties for new utilities and landscape plantings, and for future construction. Excavations created during removal of structural remnants must be backfilled with engineered fill, benched as needed into the surrounding soil, as noted in Item No. 3 of the *Guide Specifications* enclosed in Appendix D.

Surface vegetation, trees and bushes (including root-balls), topsoil with adverse organic content, and otherwise unsuitable materials are recommended to be removed from the proposed building and pavement areas, and other structural areas. Stripping should extend at least several feet beyond the proposed development area, where feasible.

Soil Evaluation and Fill Placement

After the recommended building area is excavated to the planned elevation, the subgrade throughout the entire building areas are recommended to be evaluated by a geotechnical engineer to confirm that the soil is suitable to support the proposed structures. The subgrade is recommended to be evaluated by a geotechnical engineer using appropriate means and methods, which could include proof-rolling, depending on accessibility into the parking-level excavation. Proof-rolling, if performed, is recommended to be done with a fully-loaded, tandem-axle dump truck, or other suitable construction equipment, to help locate unstable areas based on subgrade deflection caused by the wheel loads of the proof-roll equipment. However, proof-roll equipment must be kept a sufficient distance from existing construction, as existing construction could be damaged during proof-rolling. For safety, proof-roll equipment must be kept a sufficient distance from excavation walls. It is recommended that a geotechnical engineer observe proof-roll operations, and evaluate subgrade stability based on those observations.

Unsuitable materials are recommended to be replaced with engineered fill, or otherwise improved. Recommendations for subgrade improvement should, however, be made by a geotechnical engineer based on the site conditions during construction. Depending on the conditions that are encountered, areas requiring soil improvement might be large, and improvement methods might need to extend up to several feet below the planned subgrade. Areas requiring subgrade improvement should be defined during construction with the assistance of a geotechnical engineer. Also, specific improvement methods should be determined during construction on an area-by-area basis.

Low areas (if any) are recommended to be raised, where necessary, to the planned finished grade with engineered fill immediately after the subgrade is confirmed to be stable and suitable to support the proposed structure. Engineered fill is recommended to be placed in uniform, relatively thin layers (lifts). And each layer of engineered fill is recommended to be compacted to at least 95 percent of the fill material's maximum dry density determined from the Standard Proctor compaction test (ASTM D698). As an exception, the in-place dry density of engineered fill within one foot of the pavement subgrade is recommended to be compacted to at least 100 percent of the fill's maximum dry density. The water content of fill material is recommended to be uniform and within a narrow range of the optimum moisture content, also determined by the Standard Proctor compaction test. Item Nos. 4 and 5 of the *Guide Specifications* give move information pertaining to selection and compaction of engineered fill.

Engineered fill that does not meet the density and water content requirements is recommended to be replaced with new fill, or scarified to a sufficient depth (likely 6 to 12 inches, or more), moisture-conditioned, and compacted to the required density. A subsequent lift of fill should only be placed after a geotechnical engineer confirms that the previous lift was properly placed and compacted. Subgrade soil will likely need to be recompacted immediately before construction since equipment traffic and adverse weather may reduce soil stability.

Because of the moderately shallow groundwater, extreme caution is recommended to be taken when using vibratory compaction equipment at the site. Vibratory compaction could cause soils to become unstable; therefore, in some cases, it might be necessary to use static compaction equipment.

<u>Use of Site Soil as Engineered Fill</u>

Site soils that do not contain adverse organic content or other deleterious materials, as noted in the *Guide Specifications,* could be used as engineered fill. However, site soil will likely need to be moisture-conditioned (uniformly moistened or dried) prior to use as engineered fill. If construction is during adverse weather (discussed in the following section), drying site soil will likely not be feasible. In that case, aggregate fill (or other fill material with a low water-sensitivity) might need to be imported to the site. Recommendations regarding fill selection, placement, and compaction are given in the *Guide Specifications*.

8.8. General Construction Considerations

Adverse Weather

Site soil is moisture sensitive and will become unstable when exposed to adverse weather such as rain, snow, and freezing temperatures. Therefore, it might be necessary to remove or stabilize the upper 8 to 15 inches (or more) of soil due to adverse weather, which commonly occurs during late fall, winter, and early spring. At least some over-excavation and/or stabilization of unstable soil should be expected if construction is during or after adverse weather. Because site preparation is weather dependent, bids for site preparation, and other earthwork activities, should consider the time of year that construction will be conducted.

To protect soil from adverse weather, the site is recommended to be smoothly graded and contoured during construction to divert surface water away from construction areas. Contoured subgrades are recommended to be rolled with a smooth-drum compactor, before precipitation, to "seal" the surface. Furthermore, construction traffic should be restricted to certain aggregate-covered areas to control traffic-related soil disturbance. Foundation and floor slab construction should begin immediately after suitable support is confirmed.

Dewatering

Filtered sump pumps drawing water from sump pits excavated in the bottom of construction trenches are expected to be adequate to remove water that collects in the excavations. Excavated sump pits should be fully lined with geotextile and filled with free-draining crushed stone, such as crushed stone that meets the gradation requirements of ASTM No. 57 aggregate. More specialized dewatering methods might be necessary to dewater deeper excavations that extend below the groundwater table. Improper dewatering could cause support-related problems at the site and in the surrounding area.

Excavation Stability

Excavations are recommended to be made in accordance with current OSHA excavation and trench safety standards, and other applicable requirements. Sides of excavations might need to be sloped, benched, and/or braced to maintain or develop a safe work environment. Temporary shoring must be designed according to applicable regulatory requirements. Contractors are responsible for excavation safety.

Existing Fill Considerations

Existing fill materials were encountered at the site. Questionable materials, if encountered, are recommended to be evaluated by a geotechnical engineer to determine if removal and replacement with engineered fill is necessary. Disposal of materials should be in accordance with local, state, and federal regulations for the material type. This report might need to be revised if subsurface conditions differ from those shown on the *Test Boring Logs*.

8.9. Recommended Construction Materials Testing Services

This report was prepared assuming that a geotechnical engineer will perform Construction Materials Testing ("CMT") services during construction of the proposed development. It might be necessary for Giles to provide supplemental geotechnical recommendations based on the results of CMT services and specific details of the project not known at this time.

9.0 BASIS OF REPORT

This report is strictly based on the project description given earlier in this report. Giles must be notified if any parts of the project description or our assumptions about the proposed project are not accurate so that this report can be amended, if needed. This report is based on the assumption that the facility will be designed and constructed according to the codes that govern construction at the site.

The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Test Boring Logs*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the *Test Boring Logs* because this report will likely need to be revised. General comments and limitations of this report are given in the appendix.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

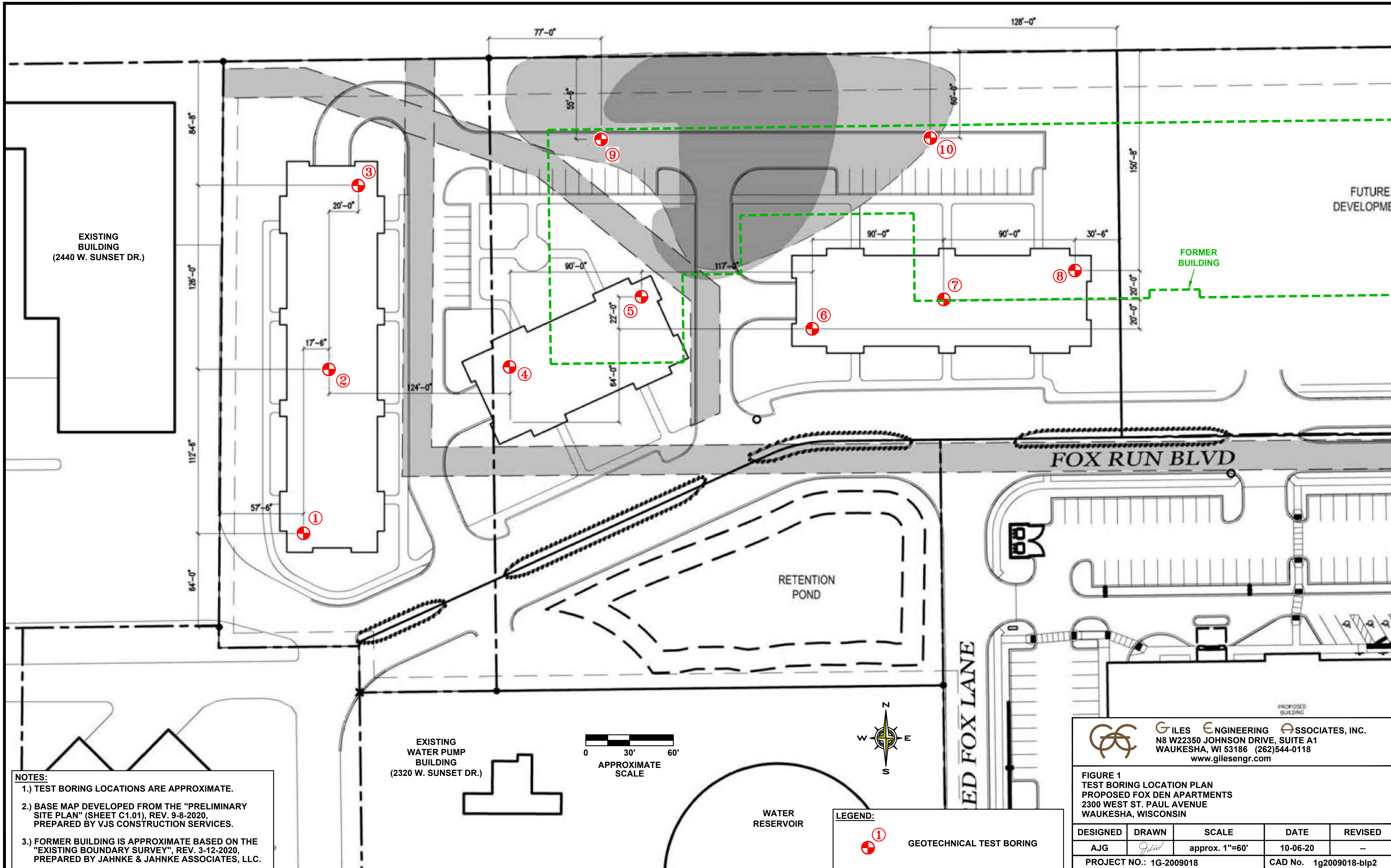
 1G-2009018/Report/20geo03/ajg

APPENDIX A

FIGURES AND TEST BORING LOGS

The Test Boring Location Plan contained herein was prepared based upon information supplied by *Giles'* client, or others, along with *Giles'* field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation.

The Test Boring Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific boring locations on the date that the exploration was performed. Subsurface conditions may differ between boring locations and within areas of the site that were not explored with test borings. The subsurface conditions may also change at the boring locations over the passage of time.



NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) BASE MAP DEVELOPED FROM THE "PRELIMINARY SITE PLAN" (SHEET C1.01), REV. 9-8-2020, PREPARED BY VJS CONSTRUCTION SERVICES.

3.) FORMER BUILDING IS APPROXIMATE BASED ON THE "EXISTING BOUNDARY SURVEY", REV. 3-12-2020, PREPARED BY JAHNKE & JAHNKE ASSOCIATES, LLC.

LEGEND:

① GEOTECHNICAL TEST BORING

GILES ENGINEERING ASSOCIATES, INC.
N8 W22350 JOHNSON DRIVE, SUITE A1
WAUKESHA, WI 53186 (262)544-0118
www.gilesengr.com

FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED FOX DEN APARTMENTS
2300 WEST ST. PAUL AVENUE
WAUKESHA, WISCONSIN

DESIGNED	DRAWN	SCALE	DATE	REVISED
AJG		approx. 1"=60'	10-06-20	--

PROJECT NO.: 1G-2009018 CAD No. 1g2009018-blp2

BORING NO. & LOCATION:	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
1		
SURFACE ELEVATION: 30.5 feet	FOX DEN APARTMENTS	
COMPLETION DATE: 09/30/20	2300 W. ST. PAUL AVENUE WAUKESHA, WISCONSIN	
FIELD REP: CHARLES RENS	PROJECT NO: 1G-2009018	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Fill: Dark Brown Gravelly Silty Sand-Moist		30	1-SS	7						
Brown Gravelly Silty Sand-Moist			2-SS	57						
	5	25	3-SS	53						
Gray Sandy Silt-Wet			4-SS	10				22		
	10	20	5-SS	7				18		
Gray Silty Clay with Sandy Silt lenses-Wet										
	15	15	6-SS	8		2.0		22		
	20	10	7-SS	10	3.5	3.1		23		

Boring Terminated at about 21 feet (EL. 9.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 6.5 ft.	
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

TEST BORING LOG

BORING NO. & LOCATION: 2	FOX DEN APARTMENTS
SURFACE ELEVATION: 29.8 feet	2300 W. ST. PAUL AVENUE WAUKESHA, WISCONSIN
COMPLETION DATE: 09/30/20	
FIELD REP: CHARLES RENS	PROJECT NO: 1G-2009018

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Fill: Dark Brown Silty fine Sand, trace Gravel-Moist			1-SS	8						
Brown Gravelly Silty Sand-Moist			2-SS	15						
	5	25	3-SS	30						
Gray Sandy Silt, trace Clay-Wet			4-SS	14				21		
	10	20	5-SS	12				18		
	15	15	6-SS	10				20		
Gray Silty Clay with Sandy Silt lenses-Wet	20	10	7-SS	7		1.3		23		

Boring Terminated at about 21 feet (EL. 8.8')

Water Observation Data	Remarks:
Water Encountered During Drilling: 6.5 ft.	
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After Drilling:	
Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 3	# TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 29.5 feet	FOX DEN APARTMENTS	
COMPLETION DATE: 09/30/20	2300 W. ST. PAUL AVENUE WAUKESHA, WISCONSIN	
FIELD REP: CHARLES RENS	PROJECT NO: 1G-2009018	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Fill: Dark Brown Gravelly Silty Sand-Moist			1-SS	17						
Brown Gravelly Silty Sand-Moist			2-SS	66						
	5	25	3-SS	30						
Gray Sandy Silt-Wet			4-SS	17				19		
	10	20	5-SS	11				21		
Gray Silty Clay with Sandy Silt lenses-Wet	15	15	6-SS	11				19		
	20	10	7-SS	11		1.3		21		

Boring Terminated at about 21 feet (EL. 8.5')

Water Observation Data	Remarks:
Water Encountered During Drilling: 6.5 ft.	
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After Drilling:	
Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 4	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 31.2 feet	FOX DEN APARTMENTS	
COMPLETION DATE: 09/30/20	2300 W. ST. PAUL AVENUE WAUKESHA, WISCONSIN	
FIELD REP: CHARLES RENS	PROJECT NO: 1G-2009018	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Fill: Dark Brown Gravelly Silty Sand-Moist		30	1-SS	12						
Brown fine to medium Sand, trace Silt and Gravel-Moist to Very Moist			2-SS	10						
	5	25	3-SS	16						
			4-SS	14						
Gray Sandy Silt, trace Clay-Wet	10	20	5-SS	8				23		
Gray Silty Clay, little Sand - Wet	15	15	6-SS	7		1.0		20		
	20		7-SS	9	1.8	1.8		23		

Boring Terminated at about 21 feet (EL. 10.2')

	Water Observation Data	Remarks:
∇	Water Encountered During Drilling: 9 ft.	
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 5

SURFACE ELEVATION: 31.5 feet

COMPLETION DATE: 09/30/20

FIELD REP: CHARLES RENS

TEST BORING LOG

FOX DEN APARTMENTS

2300 W. ST. PAUL AVENUE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-2009018

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Fill: Brown Silty fine to medium Sand, trace Silt and Gravel-Moist		30	1-SS	11						
Brown Silty fine Sand-Moist			2-SS	10						
	5		3-SS	17						
Brown and Gray mottled Sandy Silt-Wet	∇	25	4-SS	7				19		
	10	20	5-SS	13						
Gray Sandy Clay with Sandy Silt lenses-Wet	15	15	6-SS	7		0.7		19		
	20		7-SS	10	3.1	2.7		22		

Boring Terminated at about 21 feet (EL. 10.5')

	Water Observation Data	Remarks:
∇	Water Encountered During Drilling: 6.5 ft.	
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 6	# TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 30.8 feet	FOX DEN APARTMENTS	
COMPLETION DATE: 09/30/20	2300 W. ST. PAUL AVENUE WAUKESHA, WISCONSIN	
FIELD REP: CHARLES RENS	PROJECT NO: 1G-2009018	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Fill: Very Dark Brown Gravelly Silty Sand-Moist		30	1-SS	28						
Brown Gravelly Silty Sand-Moist			2-SS	27						
	5	25	3-SS	20						
Brown Sandy Silt with Silty fine Sand lenses-Wet			4-SS	15				19		
	10	20	5-SS	21						
Gray Silty Clay with Sandy Silt lenses-Wet	15	15	6-SS	14		1.3		22		
	20	10	7-SS	10	2.0	1.6		22		

Boring Terminated at about 21 feet (EL. 9.8')

Water Observation Data	Remarks:
▽ Water Encountered During Drilling: 6.5 ft.	
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
▼ Water Level After Drilling:	
Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 7	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 31.2 feet	FOX DEN APARTMENTS	
COMPLETION DATE: 09/30/20	2300 W. ST. PAUL AVENUE WAUKESHA, WISCONSIN	
FIELD REP: CHARLES RENS	PROJECT NO: 1G-2009018	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown Gravelly Silty Sand-Moist to Wet at 6.5 feet		30	1-SS	27						
			2-SS	12						
	5		3-SS	10						
	▽	25								
			4-SS	47						
Gray Sandy Silt-Wet	10		5-SS	28						
		20								
Gray Silty Clay with Sandy Silt lenses-Wet	15		6-SS	16						
		15								
	20		7-SS	13						

Boring Terminated at about 21 feet (EL. 10.2')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 6.5 ft.	
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 8	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 31.2 feet	FOX DEN APARTMENTS	
COMPLETION DATE: 09/30/20	2300 W. ST. PAUL AVENUE WAUKESHA, WISCONSIN	
FIELD REP: CHARLES RENS	PROJECT NO: 1G-2009018	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown Gravelly Silty Sand-Moist		30	1-SS	35						
			2-SS	31						
	5		3-SS	10						
	∇	25								
Brown and Gray mottled Gravelly Silty Sand-Moist to Very Moist			4-SS	5						
	10		5-SS	24						
		20								
Gray Silty Clay with Sandy Silt lenses-Wet	15		6-SS	12	2.6	2.5		21		
		15								
	20		7-SS	10	3.7	3.3		25		

Boring Terminated at about 21 feet (EL. 10.2')

	Water Observation Data	Remarks:
∇	Water Encountered During Drilling: 6.5 ft.	
	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 9	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 31.2 feet	FOX DEN APARTMENTS	
COMPLETION DATE: 09/30/20	2300 W. ST. PAUL AVENUE WAUKESHA, WISCONSIN	
FIELD REP: CHARLES RENS	PROJECT NO: 1G-2009018	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Fill: Brown Gravelly Silty Sand (Includes Concrete rubble)-Moist		30	1-SS	30						
			2-SS	32						
	5		3-SS	41						
		25	4-SS	23						
Gray Gravelly Silty Sand-Very Moist	10		5-SS	22						

Boring Terminated at about 11 feet (EL. 20.2')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 9 ft.	
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 10	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 31.2 feet	FOX DEN APARTMENTS	
COMPLETION DATE: 09/30/20	2300 W. ST. PAUL AVENUE WAUKESHA, WISCONSIN	
FIELD REP: CHARLES RENS	PROJECT NO: 1G-2009018	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown Gravelly Silty Sand-Moist		30	1-SS	17						
			2-SS	19						
Brown Silty fine to medium Sand, trace Gravel-Moist	5		3-SS	7						
		25								
			4-SS	21						
Brown and Gray mottled Silty fine to medium Sand-Wet	10		5-SS	28						

Boring Terminated at about 11 feet (EL. 20.2')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 9 ft.	
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

APPENDIX B

FIELD PROCEDURES

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D

420 entitled “Standard Guide for Sampling Rock and Rock” and/or other relevant specifications. Soil samples were preserved and transported to *Giles’* laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled “Standard Practice for Preserving and Transporting Soil Samples.” Brief descriptions of the sampling, testing and field procedures commonly performed by *Giles* are provided herein.

GENERAL FIELD PROCEDURES

Test Boring Elevations

The ground surface elevations reported on the Test Boring Logs are referenced to the assumed benchmark shown on the Boring Location Plan (Figure 1). Unless otherwise noted, the elevations were determined with a conventional hand-level and are accurate to within about 1 foot.

Test Boring Locations

The test borings were located on-site based on the existing site features and/or apparent property lines. Dimensions illustrating the approximate boring locations are reported on the Boring Location Plan (Figure 1).

Water Level Measurement

The water levels reported on the Test Boring Logs represent the depth of "free" water encountered during drilling and/or after the drilling tools were removed from the borehole. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation with cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must therefore be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfilled with an "impervious" material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were "capped" with Portland Cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by *Giles'* client or the property owner may be required.

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximate soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

Split-Barrel Sampling (SS) – (ASTM D-1586)

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the “Standard Penetration Resistance” or N-value is an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

Shelby Tube Sampling (ST) – (ASTM D-1587)

A relatively undisturbed soil sample is collected by hydraulically advancing a thin-walled Shelby Tube sampler into a soil mass. Shelby Tubes have a sharp cutting edge and are commonly 2 to 5 inches in diameter.

Bulk Sample (BS)

A relatively large volume of soils is collected with a shovel or other manually-operated tool. The sample is typically transported to *Giles’* materials laboratory in a sealed bag or bucket.

Dynamic Cone Penetration Test (DC) – (ASTM STP 399)

This test is conducted by driving a 1.5-inch-diameter cone into the subsoil using a 15-pound steel ring (hammer), free-falling a vertical distance of 20 inches. The number of hammer-blows required to drive the cone 1¾ inches is an indication of the soil strength and density, and is defined as “N”. The Dynamic Cone Penetration test is commonly conducted in hand auger borings, test pits and within excavated trenches.

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

In this procedure, a ring-lined barrel sampler is used to collect soil samples for classification and laboratory testing. This method provides samples that fit directly into laboratory test instruments without additional handling/disturbance.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-values) are reported on the Test Boring Logs. Explanations of the terms and symbols shown on the logs are provided on the appendix enclosure entitled “General Notes”.

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Brief descriptions of laboratory tests commonly performed by *Giles* are provided herein.

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

In this procedure, soil samples are “scanned” in *Giles’* analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of **certain** Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNu (manufacturer’s) units rather than actual concentration.

Moisture Content (w) (ASTM D 2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Unconfined Compressive Strength (qu) (ASTM D 2166)

An axial load is applied at a uniform rate to a cylindrical soil sample. The unconfined compressive strength is the maximum stress obtained or the stress when 15% axial strain is reached, whichever occurs first.

Calibrated Penetrometer Resistance (qp)

The small, cylindrical tip of a hand-held penetrometer is pressed into a soil sample to a prescribed depth to measure the soils capacity to resist penetration. This test is used to evaluate unconfined compressive strength.

Vane-Shear Strength (qs)

The blades of a vane are inserted into the flat surface of a soil sample and the vane is rotated until failure occurs. The maximum shear resistance measured immediately prior to failure is taken as the vane-shear strength.

Loss-on-Ignition (ASTM D 2974; Method C)

The Loss-on-Ignition (L.O.I.) test is used to determine the organic content of a soil sample. The procedure is conducted by heating a dry soil sample to 440°C in order to burn-off or “ash” organic matter present within the sample. The L.O.I. value is the ratio of the weight loss due to ignition compared to the initial weight of the dry sample. L.O.I. is expressed as a percentage.

Particle Size Distribution (ASTB D 421, D 422, and D 1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a “sieve analysis,” which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a “hydrometer analysis” which is based on the sedimentation of particles suspended in water.

Consolidation Test (ASTM D 2435)

In this procedure, a series of cumulative vertical loads are applied to a small, laterally confined soil sample. During each load increment, vertical compression (consolidation) of the sample is measured over a period of time. Results of this test are used to estimate settlement and time rate of settlement.

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM D-2488-75). The classifications are reported on the Test Boring Logs.

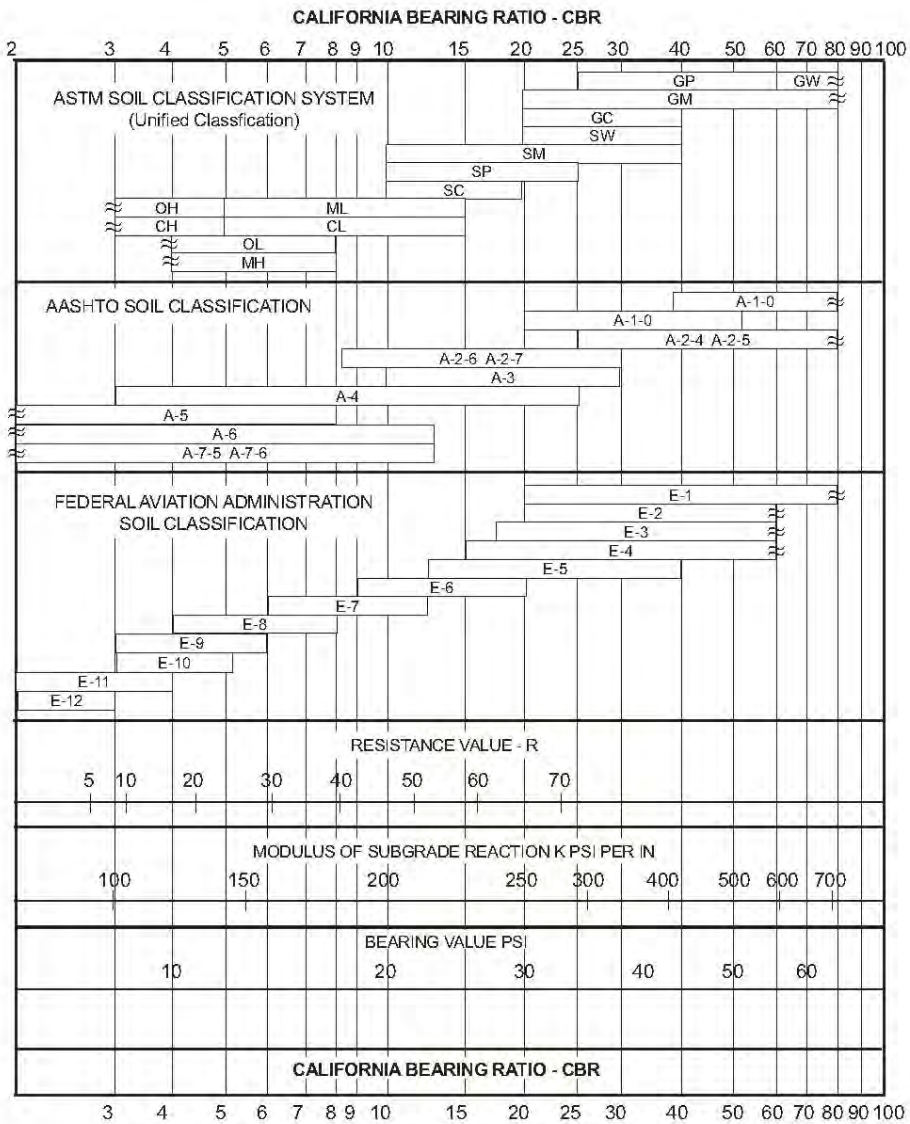
Laboratory Testing

The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Boring Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs is provided on the appendix enclosure entitled “General Notes.”

California Bearing Ratio (CBR) Test ASTM D-1833

The CBR test is used for evaluation of a soil subgrade for pavement design. The test consists of measuring the force required for a 3-square-inch cylindrical piston to penetrate 0.1 or 0.2 inch into a compacted soil sample. The result is expressed as a percent of force required to penetrate a standard compacted crushed stone.

Unless a CBR test has been specifically requested by the client, the CBR is estimated from published charts, based on soil classification and strength characteristics. A typical correlation chart is below.



APPENDIX D

GENERAL INFORMATION

AND
IMPORTANT INFORMATION ABOUT
YOUR GEOTECHNICAL REPORT

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. The project plans and specifications may also be submitted to *Giles* for review to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, *Giles* must be contacted immediately to determine if the conditions alter the recommendations contained herein.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

GUIDE SPECIFICATIONS FOR SUBGRADE AND GRADE PREPARATION FOR FILL, FOUNDATION, FLOOR SLAB AND PAVEMENT SUPPORT; AND SELECTION, PLACEMENT AND COMPACTION OF FILL SOILS USING STANDARD PROCTOR PROCEDURES

1. Construction monitoring and testing of subgrades and grades for fill, foundation, floor slab and pavement; and fill selection, placement and compaction shall be performed by an experienced soils engineer and/or his representatives.

2. All compaction fill, subgrades and grades shall be (a) underlain by suitable bearing material; (b) free of all organic, frozen, or other deleterious material, and (c) observed, tested and approved by qualified engineering personnel representing an experienced soils engineer. Preparation of subgrades after stripping vegetation, organic or other unsuitable materials shall consist of (a) proof-rolling to detect soil, wet yielding soils or other unstable materials that must be undercut, (b) scarifying top 6 to 8 inches, (c) moisture conditioning the soils as required, and (d) recompaction to same minimum in-situ density required for similar materials indicated under Item 5. Note: compaction requirements for pavement subgrade are higher than other areas. Weather and construction equipment may damage compacted fill surface and reworking and retesting may be necessary to assure proper performance.

3. In overexcavation and fill areas, the compacted fill must extend (a) a minimum 1 foot lateral distance beyond the exterior edge of the foundation at bearing grade or pavement subgrade and down to compacted fill subgrade on a maximum 0.5(H):1(V) slope, (b) 1 foot above footing grade outside the building, and (c) to floor subgrade inside the building. Fill shall be placed and compacted on a 5(H):1(V) slope or must be stepped or benched as required to flatten if not specifically approved by qualified personnel under the direction of an experienced soil engineer.

4. The compacted fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated", and shall be low-expansive with a maximum Liquid Limit (ASTM D-423) and Plasticity Index (ASTM D-424) of 30 and 15, respectively, unless specifically tested and found to have low expansive properties and approved by an experienced soils engineer. The top 12 inches of compacted fill should have a maximum 3-inch-particle diameter and all underlying compacted fill a maximum 6-inch-diameter unless specifically approved by an experienced soils engineer. All fill materials must be tested and approved under the direction of an experienced soils engineer prior to placement. If the fill is to provide non-frost susceptible characteristics, it must be classified as a clean GW, GP, SW or SP per the Unified Soil Classification System (ASTM D-2487).

5. For structural fill depths less than 20 feet, the density of the structural compacted fill and scarified subgrade and grades shall not be less than 95 percent of the maximum dry density as determined by Standard Proctor (ASTM-698) with the exception of the top 12 inches of pavement subgrade which shall have a minimum in-situ density of 100 percent of maximum dry density, or 5 percent higher than underlying fill materials. Where the structural fill depth is greater than 20 feet, the portions below 20 feet should have a minimum in-place density of 100 percent of its maximum dry density of 5 percent greater than the top 20 feet. The moisture content of cohesive soil shall not vary by more than -1 to +3 percent and granular soil ±3 percent of the optimum when placed and compacted or recompacted, unless specifically recommended/approved by the soils engineer monitoring the placement and compaction. Cohesive soils with moderate to high expansion potentials (PI>15) should, however, be placed, compacted and maintained prior to construction at a moisture content 3±1 percent above optimum moisture content to limit further heave. The fill shall be placed in layers with a maximum loose thickness of 8 inches for foundations and 10 inches for floor slabs and pavement, unless specifically approved by the soils engineer taking into consideration the type of materials and compaction equipment being used. The compaction equipment should consist of suitable mechanical equipment specifically designed for soil compaction. Bulldozers or similar tracked vehicles are typically not suitable for compaction.

6. Excavation, filling, subgrade and grade preparation shall be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs and seepage water encountered shall be pumped or drained to provide a suitable working platform. Springs or water seepage encountered during grading/foundation construction must be called to the soil engineer's attention immediately for possible construction procedure revision or inclusion of an underdrain system.

7. Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below-grade walls (i.e. basement walls and retaining walls) must be properly tested and approved by an experienced soils engineer with consideration for the lateral pressure used in the wall design.

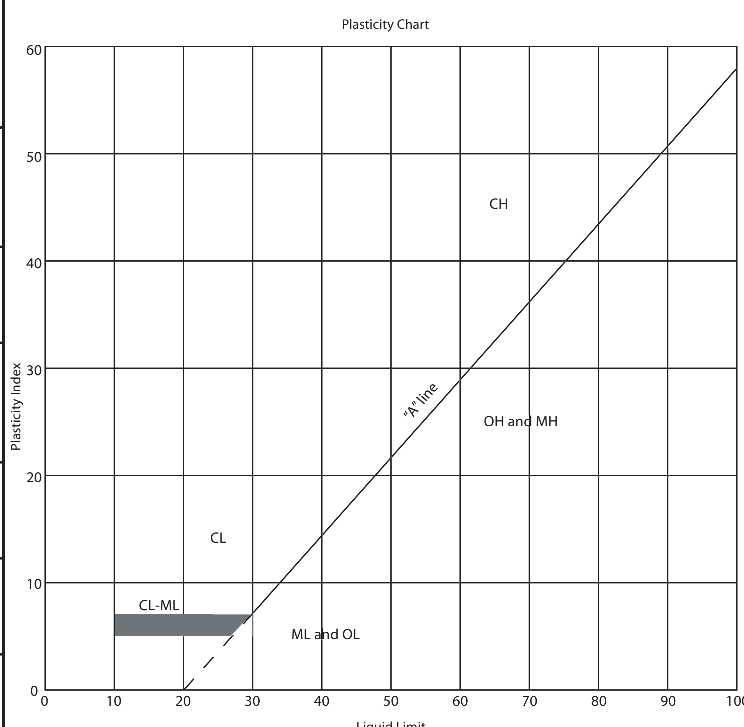
8. Whenever, in the opinion of the soils engineer or the Owner's Representatives, an unstable condition is being created either by cutting or filling, the work shall not proceed into that area until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised, if found necessary.

CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *									
Class	**Compaction Characteristics**	**Max. Dry Density Standard Proctor (pcf)**	**Compressibility and Expansion**	**Drainage and Permeability**	**Value as an Embankment Material**	**Value as Subgrade When Not Subject to Frost**	**Value as Base Course**	**Value as Temporary Pavement**	
								With Dust Palliative	**With Bituminous Treatment**
GW	Good: tractor, rubber-tired, steel wheel or vibratory roller	125-135	Almost none	Good drainage, pervious	Very stable	Excellent	Good	Fair to poor	Excellent
GP	Good: tractor, rubber-tired, steel wheel or vibratory roller	115-125	Almost none	Good drainage, pervious	Reasonably stable	Excellent to good	Poor to fair	Poor	
GM	Good: rubber-tired or light sheepsfoot roller	120-135	Slight	Poor drainage, semipervious	Reasonably stable	Excellent to good	Fair to poor	Poor	Poor to fair
GC	Good to fair: rubber-tired or sheepsfoot roller	115-130	Slight	Poor drainage, impervious	Reasonably stable	Good	Good to fair **	Excellent	Excellent
SW	Good: tractor, rubber-tired or vibratory roller	110-130	Almost none	Good drainage, pervious	Very stable	Good	Fair to poor	Fair to poor	Good
SP	Good: tractor, rubber-tired or vibratory roller	100-120	Almost none	Good drainage, pervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SM	Good: rubber-tired or sheepsfoot roller	110-125	Slight	Poor drainage, impervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SC	Good to fair: rubber-tired or sheepsfoot roller	105-125	Slight to medium	Poor drainage, impervious	Reasonably stable	Good to fair	Fair to poor	Excellent	Excellent
ML	Good to poor: rubber-tired or sheepsfoot roller	95-120	Slight to medium	Poor drainage, impervious	Poor stability, high density required	Fair to poor	Not suitable	Poor	Poor
CL	Good to fair: sheepsfoot or rubber-tired roller	95-120	Medium	No drainage, impervious	Good stability	Fair to poor	Not suitable	Poor	Poor
OL	Fair to poor: sheepsfoot or rubber-tired roller	80-100	Medium to high	Poor drainage, impervious	Unstable, should not be used	Poor	Not suitable	Not suitable	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70-95	High	Poor drainage, impervious	Poor stability, should not be used	Poor	Not suitable	Very poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80-105	Very high	No drainage, impervious	Fair stability, may soften on expansion	Poor to very poor	Not suitable	Very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65-100	High	No drainage, impervious	Unstable, should not be used	Very poor	Not suitable	Not suitable	Not suitable
Pt	Not suitable		Very high	Fair to poor drainage	Should not be used	Not suitable	Not suitable	Not suitable	Not suitable

* "The Unified Classification: Appendix A - Characteristics of Soil, Groups Pertaining to Roads and Airfields, and Appendix B - Characteristics of Soil Groups Pertaining to Embankments and Foundations," Technical Memorandum 357, U.S. Waterways Ixperiment Station, Vicksburg, 1953.

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Major Divisions			*Group Symbols*		*Typical Names*	*Laboratory Classification Criteria*		
Coarse-grained soils (more than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent: GW, GP, SW, SP; More than 12 percent: GM, GC, SM, SC; 5 to 12 percent: *Borderline* cases requiring dual symbols[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtrues, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (appreciable amount of fines)	GM[a]	d	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve size)	Silts and clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt		Peat and other highly organic soils			

[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group sympols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

GENERAL NOTES

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders:	8 inch and larger
Cobbles:	3 inch to 8 inch
Gravel:	coarse - ¾ to 3 inch
	fine – No. 4 (4.76 mm) to ¾ inch
Sand:	coarse – No. 4 (4.76 mm) to No. 10 (2.0 mm)
	medium – No. 10 (2.0 mm) to No. 40 (0.42 mm)
	fine – No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt:	No. 200 (0.074 mm) and smaller (non-plastic)
Clay:	No 200 (0.074 mm) and smaller (plastic)

SOIL PROPERTY SYMBOLS

Dd:	Dry Density (pcf)
LL:	Liquid Limit, percent
PL:	Plastic Limit, percent
PI:	Plasticity Index (LL-PL)
LOI:	Loss on Ignition, percent
Gs:	Specific Gravity
K:	Coefficient of Permeability
w:	Moisture content, percent
qp:	Calibrated Penetrometer Resistance, tsf
qs:	Vane-Shear Strength, tsf
qu:	Unconfined Compressive Strength, tsf
qc:	Static Cone Penetrometer Resistance (correlated to Unconfined Compressive Strength, tsf)
PID:	Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-Units. (BDL=Below Detection Limit)
N:	Penetration Resistance per 12 inch interval, or fraction thereof, for a standard 2 inch O.D. (1⅜ inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in general accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N-Values where plus sign (+) is shown.
Nc:	Penetration Resistance per 1¾ inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.
Nr:	Penetration Resistance per 12 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches per ASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon
ST:	Shelby Tube – 3 inch O.D. (except where noted)
CS:	3 inch O.D. California Ring Sampler
DC:	Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
AU:	Auger Sample
DB:	Diamond Bit
CB:	Carbide Bit
WS:	Wash Sample
RB:	Rock-Roller Bit
BS:	Bulk Sample
Note:	Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

COMPARATIVE CONSISTENCY	**BLOWS PER FOOT (N)**	**UNCONFINED COMPRESSIVE STRENGTH (TSF)**
Very Soft	0 - 2	0 - 0.25
Soft	3 - 4	0.25 - 0.50
Medium Stiff	5 – 8	0.50 - 1.00
Stiff	9 – 15	1.00 - 2.00
Very Stiff	16 – 30	2.00 - 4.00
Hard	31+	4.00+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	**BLOWS PER FOOT (N)**
Very Loose	0 - 4
Loose	5 - 10
Firm	11 - 30
Dense	31 - 50
Very Dense	51+

DEGREE OF PLASTICITY	**PI**
None to Slight	0 - 4
Slight	5 - 10
Medium	11 - 30
High to Very High	31+

DEGREE OF EXPANSIVE POTENTIAL	**PI**
Low	0 - 15
Medium	15 - 25
High	25+

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

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