Attachment 2 - PER Williamson Creek Interceptor from 2001

LOWER WILLIAMSON CREEK
RELIEF INTERCEPTOR
PRELIMINARY
ENGINEERING REPORT

PBS&J

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LOWER WILLIAMSON CREEK RELIEF INTERCEPTOR PRELIMINARY ENGINEERING REPORT

Prepared for:

City of Austin
Water and Wastewater Utility
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STATE OF TEXAS
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TABLE OF CONTENTS

Section		Page
	List of Figures	v
	List of Tables	vi
1.0	INTRODUCTION	1-1
1.1	BACKGROUND	1-1
1.2	PURPOSE AND SCOPE	1-3
2.0	STUDY AREA	2-1
2.1	SERVICE AREA	2-1
2.2	PROJECT AREA	2-1
3.0	EXISTING 36-/42-INCH WASTEWATER INTERCEPTOR	3-1
3.1	PHYSICAL CONDITION	3-1
3.2	HYDRAULIC CONDITIONS	3-2
3.2.1	Pipe Capacity	3-2
3.2.2	Existing Flows	3-2
4.0	DESIGN FLOWS	4-1
4.1	METHODOLOGY	4-1
4.2	YEAR 2040 PROJECTED PEAK FLOWS	4-2
5.0	ENVIRONMENTAL SETTING OF THE PROJECT AREA	5-1
5.1	PHYSIOGRAPHY, SOILS, GEOLOGY	5-1
5.2	WATER RESOURCES	5-2
5.3	ECOLOGY	5-2
5.3.1	Vegetation	5-2
5.3.2	Fish and Wildlife	5-3
5.3.3	Endangered and Threatened Species	5-3
5.3.4	Critical Environmental Features	5-6
5.3.5	Environmentally Sensitive Areas	5-6
5.4	CULTURAL RESOURCES	5-7
5.5	SOCIAL SETTING	5-9
6.0	PROJECT REQUIREMENTS - OVERVIEW	6-1
6.1	HYDRAULICS ISSUES	6-1
6.1.1	Capacity and Flow Connections	6-1
6.1.2	Ventilation and Odor Control	6-3
6.2	CONSTRUCTION METHODS AND MATERIALS	6-4
6.2.1	Relief Interceptor	6-4

TABLE OF CONTENTS (Cont'd)

Section		Page
6.2.2	Cut-over Mains	6-5
6.2.3	Rehabilitation	6-5
6.2.4	Access Considerations	6-8
6.2.5	Pipe Materials	6-8
6.3	ENVIRONMENTAL IMPACTS	6-9
6.4	PUBLIC IMPACTS	6-10
6.5	EASEMENTS, PERMITTING AND APPROVALS	6-10
6.5.1	Parkland Use Agreement	6-10
6.5.2	Private Easements	6-11
6.5.3	State of Texas Highway Permit	6-12
6.5.4	Variance for Construction in Critical Water Quality Zone	6-12
6.5.5	City of Austin Site Development Permit	6-12
6.5.6	Corps of Engineers Permit	6-13
6.5.7	Texas Natural Resource Conservation Commission	6-13
6.5.8	Texas Historical Commission	6-13
6.6	PROJECT COST	6-14
7.0	ENGINEERING AND ENVIRONMENTAL EVALUATION OF ALTERNATIVES	7-1
7.1	CONCEPTUAL BASIS OF ALTERNATIVES	7-1
7.2	ALTERNATIVE "A"	7-8
7.3	ALTERNATIVE "B"	7-15
7.4	ALTERNATIVE "C"	7-16
7.5	ALTERNATIVE "D"	7-19
7.6	ALTERNATIVE "E"	7-22
7.7	COMPARISONS OF ALTERNATIVES	7-24
8.0	PRELIMINARY DESIGN OF RELIEF MAIN ALTERNATIVES	8-1
8.1	DESIGN FLOWS FOR IMPROVEMENTS	8-1
8.2	RELIEF INTERCEPTOR TUNNEL	8-1
8.3	CUT-OVER MAINS	8-7
8.4	PIPELINE REHABILITATION	8-7
8.5	FLOW DIVERSION/JUNCTION STRUCTURES	8-8
8.5.1	Existing Diversion Box	8-8
8.5.2	East Junction Structure at 84-inch Tunnel	8-11

TABLE OF CONTENTS (Concluded)

Section		Page
8.5.3	West Junction Structure at 48-inch Interceptor and Tunnel Inlet Shafts	8-11
9.0	SEQUENCE OF CONSTRUCTION AND PROJECT SCHEDULE	9-1
9.1	SEQUENCE OF CONSTRUCTION	9-1
9.2	PROJECTED CONSTRUCTION SCHEDULE	9-2
10.0	CONSTRUCTION COST ESTIMATES	10-1
11.0	CONCLUSIONS AND RECOMMENDATIONS	11-1

APPENDIX A - GEOTECHNICAL INVESTIGATION (Terra-Mar)
APPENDIX B - CASE HISTORIES OF AUSTIN WASTEWATER TUNNELS (P. Nelson)

LIST OF FIGURES

Figure		Page
1-1	Location Map	1-2
2-1	Service Area	2-2
2-2	Project Area	map pocket
3-1	Lower Williamson Ck Interceptor - Capacity and Profile	3-4
3-2	Creekwood Meter -Daily Flows (3/27/97 to 6/9/97)	3-13
3-3	Creekwood Meter -Hourly Average Flows (11/10/97 to 12/12/97)	3-14
3-4	Creekwood Meter -Flows (10/7/98 to 10/27/98 - 5-minute Values)	3-15
3-5	Creekwood Meter - Hourly Average Flows and Levels (2/1/99 to 5/6/99)	3-16
3-6	Creekwood Meter - 6/9/97 Storm (Hourly Max and Min Values)	3-17
3-7	Creekwood Meter - 10/17/98 Storm (5-minute Values)	3-18
3-8	Battlebend Meter - Hourly Average Flows and Levels (2/1/99 to 5/6/99)	3-19
3-9	Wasson Meter - Hourly Average Flows and Levels (2/1/99 to 5/6/99)	3-20
3-10	Comparison of Williamson Creek Interceptor Flow Data	3-21
3-11	Velocity vs Level Scattergraph - Wasson Meter (2/1/99 to 5/6/99)	3-28
3-12	Velocity vs Level Scattergraph - Battlebend Meter (2/1/99 to 5/6/99)	3-29
3-13	Velocity vs Level Scattergraph - Creekwood Meter (2/1/99 to 5/6/99)	3-30
3-14	$\sqrt{s/n}$ Graph - Wasson Meter (2/1/99 to 5/6/99)	3-31
3-15	$\sqrt{s/n}$ Graph - Battlebend Meter (2/1/99 to 5/6/99)	3-32
3-16	$\sqrt{s/n}$ Graph - Creekwood Meter (2/1/99 to 5/6/99)	3-33
7-1	Alternative "A"	7-2
7-2	Alternative "B"	7-3
7-3	Alternative "C"	7-4
7-4	Alternative "D"	7-5
7-5	Alternative "E"	7-6
7-6	Alternatives Composite	map pocket
7-7	Tunnel, Sliplining and Cut-over Main Quantities	7-25
8-1	Relief Interceptor Profile - Alternative "C"	8-6
8-2	Lower Williamson Ck Interceptor Diversion Box Details	8-9

LIST OF TABLES

Table		Page
3-1	Interceptor Design Data and Capacity	3-3
3-2	Dry Weather Flows During TV Inspection	3-11
4-1	Node Service Area Data	4-3
4-2	Disaggregation of Node Service Area Data to Sewer Subbasins	4-4
4-3	Design Flows for the Lower Williamson Creek Interceptor	4-6
7-1	Comparison of Relief Tunnel Alternatives	7-24
8-1	Design Flows for Project Alternatives	8-2
10-1	Unit Costs for Cut-over Mains and Sliplining	10-1
10-2	Preliminary Construction Cost Estimate - Alternative "A"	10-3
10-3	Preliminary Construction Cost Estimate - Alternative "C"	10-5
10-4	Preliminary Construction Cost Estimate - Alternative "D1"	10-7
10-5	Preliminary Construction Cost Estimate - Alternative "D2"	10-9
10-6	Summary of Construction Cost Estimates	10-11

1.0 INTRODUCTION

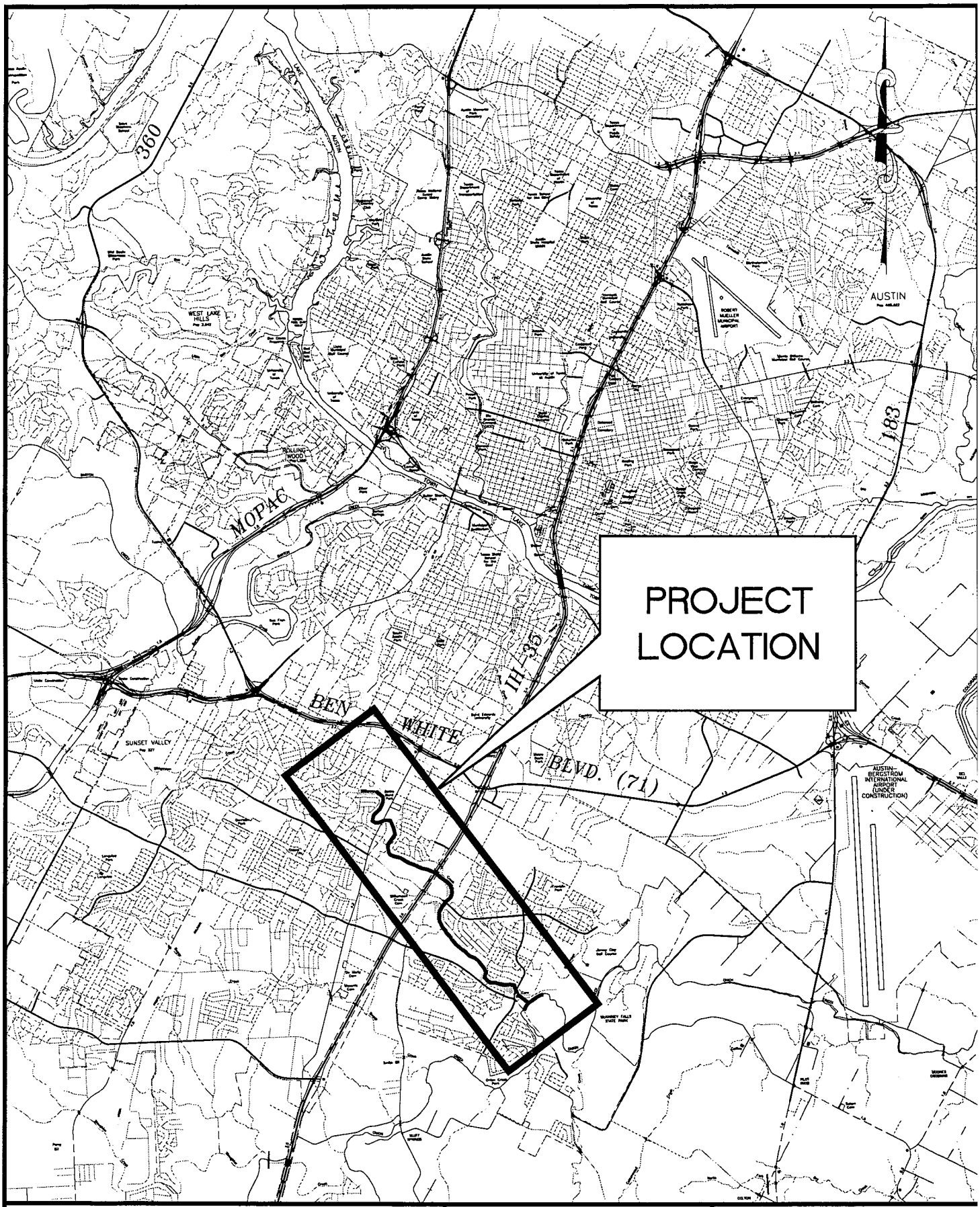
PBS&J (formerly Espey Huston & Associates) was selected by the City of Austin Water and Wastewater Utility (W&WW) to provide preliminary engineering and environmental investigation for the proposed interceptor that will relieve the existing 36-/42-inch interceptor along Williamson Creek between approximately South First Street and Pleasant Valley Road (see Figure 1-1, Location Map). In addition to PBS&J, the project team assembled to cover various aspects of the project consists of KLW for research and data collection, Terra-Mar for geotechnical investigation, Hill Country Environmental for environmental investigation, Hicks and Company for cultural resources investigation, and Fugro South for tunneling issues.

1.1 BACKGROUND

The 36-/42-inch sewer which this project will relieve was constructed in 1962-63 to relieve the Goliad Lane lift station on the north bank of Williamson Creek east of South First Street. This lift station, built in 1958, received flow from an upstream 18-inch gravity main in Williamson Creek and pumped sewage northward to the collection system in the West Bouldin Creek basin. The 36-/42-inch line relieved the Goliad lift station in 1964 and carried flow to the Williamson Creek Wastewater Treatment Plant just east of Nuckols Crossing Road. In 1981 a 48-inch interceptor was constructed in Williamson Creek upstream of the 36-inch main to relieve the old 18-inch sewer. In the 1980's the South Austin Regional Plant and the 84-inch Onion Creek tunnel, with an extension to the 42-inch Williamson Creek main, were constructed in order to abandon the Williamson Creek Treatment Plant. In short, the 36-/42-inch main is a bottleneck with a 48-inch sewer upstream and an 84-inch sewer downstream.

The 1963 Williamson Creek interceptor consisted of 5,104 ft of 36-inch pipe and 14,367 ft of 42-inch pipe. The 1984 bond election included CIP Project No. 237365 for 3,635 ft of 60-inch tunnel that would relieve the upstream 36-inch section and connect to the old downstream 42-inch pipe. That project was designed in 1986 and revised in 1992 but never constructed. The 1992 plans are not in conformance with current permitting and design requirements and need revisions to update for current conditions.

The Utility's Wastewater Collection System Long Range Planning Guide (May 1994) addressed the need for a relief interceptor for the 42-inch Williamson Creek main, which was established as CIP Project No. 448-237-8895. The current Williamson Creek Relief Interceptor project described herein is a combination of these two CIP projects intended to relieve the entire 19,500 ft of 36-/42-inch main. Unifying the two projects frees CIP 237365 from the horizontal and vertical constraints of connecting to the existing 42-inch main and greatly increases the alternatives for a relief main between the upstream 48-inch interceptor and the downstream 84-inch Onion Creek tunnel.



LOWER WILLIAMSON CREEK
RELIEF INTERCEPTOR

FIGURE 1 - 1
LOCATION MAP

PBS&J

206 WILD BASIN ROAD STE. 300
AUSTIN, TEXAS 78746 – (512) 327-6840

K:\PROJECTS\MUTIL\WILLMSON\LWC19005\DWG\LW-FG-10.dw

1.2 PURPOSE AND SCOPE

The objective of this CIP project is construction of a new interceptor to relieve overloading in the existing main and to provide capacity for projected year 2040 design flows. The project is also needed because the existing interceptor is in deteriorated condition due to corrosion and should be either abandoned or rehabilitated in the near future due to its diminishing structural integrity.

The project will progress from the current preliminary engineering phase to the design phase, then bidding and award, and finally construction. The purpose of this preliminary engineering phase is to evaluate alternatives for design of the new relief interceptor and for rehabilitation or abandonment of the existing interceptor. The preliminary phase evaluates the technical alternatives for the relief interceptor in light of the environmental impacts and public impacts of the various alternatives. The subsequent design phase will be authorized after the final recommendations and the City's selection of the alternative for implementation.

The scope of the preliminary engineering and environmental impacts phase, which is reflected in the organization of the report, includes the following issues:

- study area (service area, project area)
- existing interceptor conditions (physical, hydraulic)
- design flows
- project area environmental setting
- project requirements and constraints
- engineering and environmental evaluation of relief main alternatives
- preliminary design
- sequence of construction and construction cost estimates
- recommendations for final design

On account of funding limitations, the relief interceptor may be constructed in phases. However, preliminary engineering addresses each alternative as a complete project between the downstream 84-inch tunnel and upstream 48-inch interceptor.

2.0 STUDY AREA

The study area as used herein actually denotes two types of areas designated with different terms. The "service area" for the project is the area in which the design flows for the relief interceptor are generated. The "project area" is the more immediate vicinity of the improvements to be constructed.

2.1 SERVICE AREA

The service area for the Lower Williamson Creek Relief Interceptor is illustrated in Figure 2-1, which shows wastewater trunk mains 12-inch diameter and larger. The majority of the service area is in the Williamson Creek watershed. Figure 2-1 includes the watershed boundaries in the City's GIS database. The service area also includes portions of the Barton Creek watershed which are served by lift stations that pump over to the Williamson Creek collection system. The City's records indicate 10 lift stations in the Barton Creek watershed pumping to the Williamson Creek system. Travis Country, Barton View, and Barton Ridge Plaza are City lift stations. There are 7 small private lift stations. There is also a lift station (Silverstone) in the South Boggy Creek watershed which currently pumps over to Williamson, but the Utility has plans to relieve it by gravity in the near future.

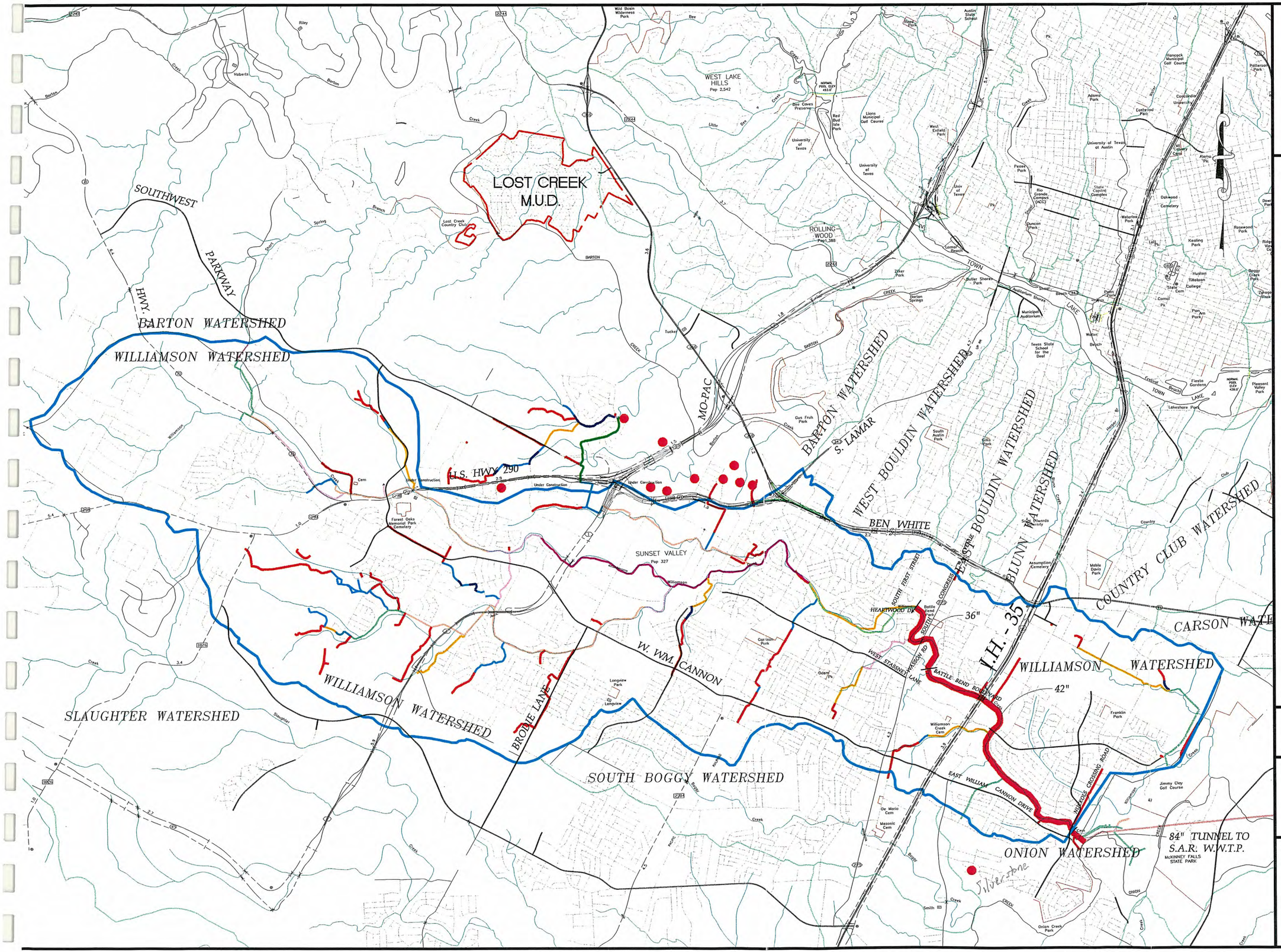
Figure 2-1 includes the Lost Creek Municipal Utility District (MUD) which is located in the Barton Creek watershed. The 1994 Wastewater Collection System Long Range Planning Guide considered the possibility of a future pumpover to Williamson as a means of providing City service to Lost Creek MUD. Wastewater disposal for Lost Creek MUD is currently provided by a treatment plant with land irrigation of the effluent. City service could provide an alternative to treatment and irrigation.

2.2 PROJECT AREA

Figure 2-2 (map pocket) shows the project area, i.e., the more immediate area of the existing 36-/42-inch interceptor and the corridor for potential alignments of the Lower Williamson Creek Relief Interceptor improvements. Manholes on the existing interceptor are labeled starting at the lower end with "A" at Pleasant Valley Road, up to "Z" at the size change from 42-inch to 36-inch. Manholes along the 36-inch are labeled "AA", up to "KK" at the junction of the 36-inch main with the upstream 48-inch interceptor. The manhole identifiers are sequential letters except for "X1" located between "X" and "Y". This manhole was recently constructed on the 42-inch main for connection of a 10-inch lateral serving new development recently completed north of Stassney Lane between Congress Avenue and IH-35.

The manhole identifiers are also used in this report to designate the lateral mains to the interceptor and the contributing subbasins. Figure 2-2 includes the boundaries for the sewer subbasins established by the existing collection system for each lateral.

LOWER WILLIAMSON CREEK RELIEF INTERCEPTOR



LEGEND

MAJOR WW COLLECTION

- 12" WASTEWATER LINE
- 14" WASTEWATER LINE
- 15" WASTEWATER LINE
- 16" WASTEWATER LINE
- 18" WASTEWATER LINE
- 20" WASTEWATER LINE
- 21" WASTEWATER LINE
- 24" WASTEWATER LINE
- 27" WASTEWATER LINE
- 30" WASTEWATER LINE
- 33" WASTEWATER LINE
- 36" WASTEWATER LINE
- 42" WASTEWATER LINE
- 48" WASTEWATER LINE
- 84" WASTEWATER LINE
- 36"/42" LOWER WILLIAMSON CREEK INTERCEPTOR
- EXISTING LIFT STATION PUMP-OVER TO WILLIAMSON CREEK
- WILLIAMSON WATERSHED BOUNDARY
- OTHER WATERSHED BOUNDARIES

5000 2500 0 5000

FIGURE 2-1

SERVICE AREA MAP

PBS&J
206 WILD BASIN ROAD STE. 300
AUSTIN, TEXAS 78746 – (512) 327-6840

Note that the alignment of the 42-inch interceptor shown in Figure 2-2 and other figures in the report was created from the as-built construction plans and differs significantly in some areas from the location of the 42-inch main shown in the City's wastewater collection system map (not shown). Field surveying by project team member McGray & McGray Land Surveyors located the manholes on the 42-inch main and also several property corners shown in the City's base map. The field surveying was used to accurately register the calculated alignment of the 42-inch main in relation to the base map.

Various alternatives have been evaluated for relief interceptor improvements in a wide corridor between the upper and lower ends of the existing interceptor. Separate maps showing each alternative are presented in Section 7 of the report.

3.0 EXISTING 36-/42-INCH WASTEWATER INTERCEPTOR

3.1 PHYSICAL CONDITION

The existing 36-/42-inch interceptor has been in service for 35 years. The pipe material installed was ASTM C-76 reinforced concrete pipe with an interior bitumastic coating for corrosion protection. Evidence of the current physical condition of the pipe is provided by the Utility's closed circuit television inspection of portions of the pipeline on several occasions between October 1997 and April 1998. The Utility has provided PBS&J with video tapes of the TV inspection for evaluation of pipeline integrity. Approximately 60% of the 36-/42-interceptor was inspected. Accessibility problems prevented the Utility from TV'ing the total length of line.

The Williamson Creek interceptor shows a significant amount of deterioration throughout its length. No videotaped section of the interceptor showed significantly more severe problems than the rest of the line. The original bitumastic coating is completely gone except in a very few spots. Corrosion from the sewer environment has removed the smooth interior cement coating, exposing the aggregate in the concrete. The reinforcing wire in the concrete can be seen in a few places. Cracks in the pipe wall are very common at the crown of the pipe. In some locations there are also cracks at the springline (the vertical midpoint or widest part of the pipe).

Corrosion of the concrete has generally been more severe at the pipe joints. The video tapes show some joints where the gasket is exposed, hanging free, or gone completely. The tapes also show infiltration flows and one joint with an intrusion of roots.

One section of the interceptor which was not available for videotape review is reported by the Utility to have more severe corrosion due to a former lift station discharge. Sewage that goes septic from wet well and force main detention time frequently causes corrosion problems in the receiving concrete gravity line. The Boggy Creek lift station (before it was relieved and abandoned) pumped to a 24-inch main tributary to manhole "K" on the 42-inch Williamson Creek interceptor near Wagon Bend Trail and Blackmule Drive. The Utility reports that the interceptor downstream of the Boggy Creek lift station lateral shows more serious corrosion.

Although the video tapes show no section of the interceptor in imminent danger of collapse, deterioration of the pipe will continue since it has no corrosion protection. Either rehabilitation or abandonment of the pipe is inevitable. The Utility has indicated a preference for abandoning the pipe after the relief interceptor is complete. However, it may be necessary to permanently maintain sections of the interceptor in service given the alternatives for handling flows from lateral mains. Sections that will remain in service would be rehabilitated by sliplining as part of the relief interceptor construction project.

Requirements and alternatives for the relief interceptor and for cutting over the lateral mains are discussed in overview in Section 5.1 and in detail in Section 7.

3.2 HYDRAULIC CONDITIONS

3.2.1 Pipe Capacity

This section addresses pipe capacities and observed flow rates in the lower Williamson Creek interceptor. Capacities in the 36-/42-inch interceptor range generally from about 24,000 gpm at the lower end to approximately 16,000 gpm at the upper end. These capacities represent the pipe flowing full based on Mannings equation with the nominal pipe sizes, with pipe slopes from the construction plans, and with an assumed friction coefficient "n" of 0.013. Table 3-1 presents data from the construction plans with pipe slopes calculated from invert elevations and with pipe lengths based on station numbers.

Figure 3-1 shows pipe capacity graphically in association with the pipe profile. Figure 3-1 includes the capacity at 80% full, which is the Utility's current design criteria for sizing new interceptors. Under this guideline, the design capacity of the 36-/42-inch interceptor (versus capacity flowing full) is approximately 19,000 gpm at the lower end and 13,000 gpm at the upper end.

Corrosion of the pipe wall will have slightly increased the pipe diameter and increased the roughness (and friction factor, n) due to the exposed aggregate. With regard to pipe capacity, these factors tend to be somewhat offsetting. For example, an actual inside diameter of 43 inches with n-factor of 0.014 would provide about the same capacity as the nominal 42-inch pipe at 0.013. The true n-factor could indeed be higher. It is likely that the current actual capacities are less than shown in Table 3-1 and Figure 3-1 due to higher n-factor, but determination of actual n-factors is problematical.

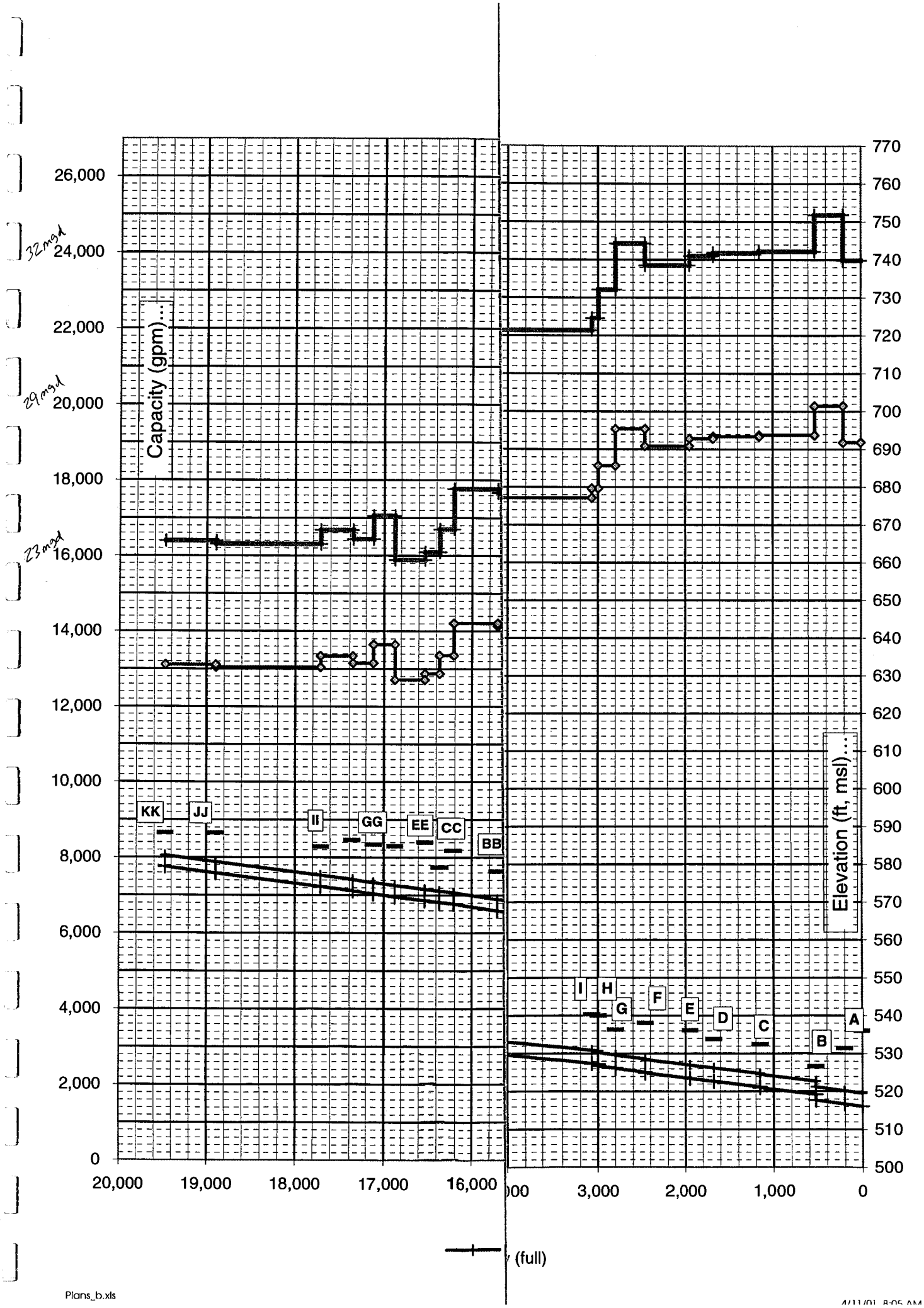
3.2.2 Existing Flows

Information on current actual flows is available from flow monitoring conducted by the City. The Utility has maintained a flow monitor for four years in a manhole at the lower end of the 42-inch main near Teewood Drive off of Creek Bend Drive. This flow monitor, designated the "Creekwood Meter" by the Utility, is in manhole "D" on the 42-inch main (see Figure 2-2). The Utility installed meters for temporary monitoring at manholes in the middle and upper sections of the interceptor. These meters were intended to determine the extent of overloading upstream in order to evaluate potential staging of improvements. The "Battlebend Meter" was installed in manhole "X" on the 42-inch main west of IH-35. The "Wasson Meter" was installed in manhole "CC" on the 36-inch main east of Congress Avenue.

TABLE 3-1. INTERCEPTOR DESIGN DATA AND CAPACITY

36" / 42" LOWER WILLIAMSON CREEK INTERCEPTOR													
CONSTRUCTION PLANS								Bk-	CALCULATIONS				
DownStream MH			Plans		UpStream MH			Fwd			UpMH	DFCR=	80%
		STA_DN	Size	Slope%			STA_UP	EQN	Length	CalcS%	dFL	Qfull	Qdesgn
Beg/Pl	pi	92+52.82	42	0.28%	MH	A	94+58.52		205.70	0.28%	0	23,977	19,181
MH	A	94+58.52	42	0.28%	MH	B	97+80.14		321.62	0.31%	1.5	25,178	20,143
MH	B	97+80.14	42	0.28%	MH	C	104+05.94		625.80	0.29%	0.2	24,217	19,373
MH	C	104+05.94	42	0.28%	newMH	D	109+29.30		523.36	0.29%	0	24,174	19,339
newMH	D	109+29.30	42	0.28%	MH	E	111+99.68		270.38	0.28%	0	24,096	19,277
MH	E	111+99.68	42	0.28%	MH	F	117+12.36		512.68	0.28%	0.2	23,847	19,078
MH	F	117+12.36	42	0.28%	MH	G	120+54.05		341.69	0.29%	0.1	24,428	19,542
MH	G	120+54.05	42	0.28%	MH	H	122+50.00	0.90	196.85	0.26%	0.49	23,208	18,566
MH	H	122+50.00	42	0.24%	newMH	I	123+22.78		72.78	0.25%	0	22,456	17,965
newMH	I	123+22.78	42	0.24%	MH	J	134+50.00		1,127.22	0.24%	0.5	22,140	17,712
MH	J	134+50.00	42	4.35%	GB/pi		135+23.50		73.50	4.35%	0	94,216	75,373
GB/pi		135+23.50	42	0.24%	MH	K	146+56.35	-0.26	1,132.59	0.24%	0	22,047	17,637
MH	K	146+56.35	42	0.24%	newMH	L	154+57.76		801.41	0.24%	0	22,216	17,773
newMH	L	154+57.76	42	0.24%	MH	M	156+20.29		162.53	0.24%	0	22,119	17,695
MH	M	156+20.29	42	0.24%	newMH	N	161+19.33		499.04	0.24%	0	22,121	17,697
newMH	N	161+19.33	42	0.24%	eq/MH	O	163+55.66		735.37	0.24%	0	22,090	17,672
eq/MH	O	163+76.84	42	0.24%	newMH	P	169+88.00		611.16	0.24%	0	22,121	17,697
newMH	P	169+88.00	42	0.24%	eq/MH	Q	172+99.88		923.04	0.24%	1.41	22,144	17,715
eq/MH	Q	172+91.96	42	0.28%	newMH	R	173+62.56		70.60	0.28%	0	24,033	19,226
newMH	R	173+62.56	42	0.28%	MH	S	180+05.61		643.05	0.28%	0	23,890	19,112
MH	S	180+05.61	42	0.28%	End/MH	T	190+00.00		994.39	0.28%	0.07	23,875	19,100
Beg/MH	T	0+99.22	42	0.18%	newMH	U	6+50.00		550.78	0.18%	0	19,144	15,315
newMH	U	6+50.00	42	0.18%	MH	V	14+27.14		777.14	0.18%	0	19,165	15,332
MH	V	14+27.14	42	0.18%	MH	W	23+28.48	-7.05	894.29	0.18%	0	19,218	15,375
MH	W	23+28.48	42	0.18%	MH	X	27+08.21		379.73	0.18%	0.53	19,108	15,286
MH	X	27+08.21	42	0.18%	MH	Y	42+65.16		1,556.95	0.18%	0.15	19,149	15,319
MH	Y	42+65.16	42	0.18%	MH	Z	47+35.59	2.70	473.13	0.18%	0.5	19,026	15,221
MH	Z	47+35.59	36	2.61%	GB/pi		48+52.27		116.68	2.61%	0	48,397	38,718
GB/pi		48+52.27	36	0.35%	MH	AA	56+56.10		803.83	0.35%	0	17,699	14,159
MH	AA	56+56.10	36	0.35%	MH	BB	60+81.00		424.90	0.35%	0	17,667	14,133
MH	BB	60+81.00	36	0.35%	MH	CC	65+83.63		502.63	0.35%	0	17,764	14,211
MH	CC	65+83.63	36	0.30%	MH	DD	67+31.50		147.87	0.31%	0	16,696	13,357
MH	DD	67+31.50	36	0.30%	MH	EE	68+97.65		166.15	0.29%	0	16,089	12,872
MH	EE	68+97.65	36	0.30%	MH	FF	72+42.00		344.35	0.28%	0	15,888	12,710
MH	FF	72+42.00	36	0.30%	MH	GG	74+85.43		243.43	0.32%	0	17,053	13,642
MH	GG	74+85.43	36	0.30%	MH	HH	77+30.79		245.36	0.30%	0.09	16,439	13,151
MH	HH	77+30.79	36	0.30%	MH	II	80+78.74		347.95	0.31%	0	16,677	13,342
MH	II	80+78.74	36	0.30%	MH	JJ	90+10.80	251.13	1,183.19	0.30%	0	16,304	13,043
MH	JJ	90+10.80	36	0.30%	MH	KK	95+88.00		577.20	0.30%		16,388	13,111

32 mgd



Plans_b.xls

4/11/01 8:05 AM

The Utility used Marsh-McBirney open channel area-velocity flow monitors for this data collection. The main components of the system are a data logger module in a waterproof housing, the data acquisition probe that senses flow depth and velocity, and accessories such as the communication cable and mounting hardware. Hydrostatic pressure due to flow depth is measured by a pressure transducer on the probe. The pressure under surcharged conditions is converted into an equivalent depth that is greater than the pipe diameter. Velocity measurement is by the electromagnetic method using three electrodes on the top surface of the probe. The velocity reading is the velocity at the sensor electrodes. The flow monitor software converts the sensor velocity to a mean flow velocity by means of a "site coefficient" that must be determined by manual velocity profiling. The software derives flow rate as the product of mean velocity and flow cross-sectional area (as a function of flow depth and pipe size).

Note that Figures 3-2 through 3-16 referenced in the following discussions are all inserted at the end of Section 3.2.

Creekwood Meter. Pipe capacity at the Creekwood meter is 23,900 gpm flowing full based on the record slope of 0.28% with Mannings n of 0.013. It should be noted that this pipe capacity at the meter is approximate due to the assumed value for the pipe friction coefficient and the assumption that the calculated slope between manholes is valid in the vicinity of the flow monitor. The manhole with the Creekwood meter is 1,675 ft upstream of the 42-inch main's connection with the diversion box.

The Utility furnished various flow monitoring data for four time periods: March 27 to June 9, 1997; November 10 to December 12, 1997; October 7 to October 27, 1998; and February 1 to May 6, 1999. This flow data is presented in Figures 3-2, 3-3, 3-4, and 3-5, respectively. These figures show that the dry weather daily peak flow occasionally reaches 10,000 gpm, or about 42% of pipe capacity. Note that Figures 3-3 and 3-5 show average hourly flows and Figures 3-2 and 3-4 show average 5-minute flows.

During wet weather the peak flows increase significantly. Figure 3-2 for the March to June 1997 period shows daily peak flows greater than 14,000 gpm on six occasions. A storm on June 9, 1997, produced a recorded peak flow of 26,700 gpm and a maximum indicated depth of 42.3 inches. For a storm on October 17, 1998 (Figure 3-3), the flow monitor indicated a peak flow of 30,000 gpm and surcharging to a depth of 111 inches. The manhole remained surcharged until just past midnight on October 21.

The Utility provided data on the 5-minute maximum and minimum flows occurring hourly between midnight and noon for the June 9, 1997 storm. The pattern in this data was not expected and is not the typical spike in flow rate due to inflow from a high intensity rain event. Very dynamic hydraulics occurred at the peak flows resulting from this storm, with rapid changes in velocity and flow, but relatively constant level. For example, between approximately 5 a.m. and 6 a.m. the flow rate dropped from 24,150 gpm to 5,000 gpm in 25 minutes and then increased by 21,700 gpm in the next 20 minutes to

26,700 gpm. The velocity went from 3.9 fps to 0.8 fps to 4.3 fps. There were several other swings of 15,000 to 20,000 gpm within one hour. The recorded level stayed relatively constant at 39.3 to 42.3 inches during the flow pulsations. Figure 3-6 shows the maximum and minimum flows, velocities, and levels each hour.

The same flow dynamics occurred with the October 17, 1998, storm. Figure 3-7 shows 5-minute level and velocity readings. The meter recorded six negative velocities during the period of surcharging, indicating flow reversal, and recorded a velocity change of at least ± 2 fps within 5 minutes on 55 occasions.

This pattern is very unusual in a collection system. The typical flow response to a high intensity rainfall is a rapid increase in flow rate to a peak, followed by gradual reduction in the flow. The cause of the unusual recorded data is not known. The observed surcharging at the Creekwood meter site is apparently not due solely to flow rate exceeding the pipe capacity. The diversion box approximately 1,700 ft downstream has been suspected as a capacity constraint due to limited wall opening area for the discharge to the 84-inch tunnel. Water level in the diversion box submerging the 42-inch influent pipe would be reflected in the water level at the Creekwood meter. However, the rapid velocity variations and the apparent flow reversals indicate that there are additional hydraulic factors that affect the surcharging. The swings in velocity resemble the pressure transients associated with water hammer in a pressurized water system, although the indicated water levels (pressure) shows much greater stability than the velocity. More data collection downstream and upstream of the flow monitoring manhole under peak flow conditions would be needed to investigate and verify an explanation for the phenomenon.

The accuracy of flow monitoring data can have low reliability due to the "hostile environment" for data collection. Meter malfunction, such as fouling of the velocity sensor, is not uncommon. However, the occurrence of the unusual data only during peak flows and the consistency between rain events supports the validity of the data, but not necessarily the accuracy. The peak velocities (and consequently peak flows which are calculated from velocity) may not be accurate since calibration of the flow monitor by manual velocity profiling is done at lower flows. The site coefficient, derived from velocity profiling to convert point velocity at the sensor to mean velocity, may be different under surcharge conditions. The cause of a reported reverse flow of 27,420 gpm (10/17/98 at 6:41pm), based on a recorded negative velocity of -5.5 fps, is difficult to conceive, considering positive downstream flows at the preceding and succeeding 5-minute readings. Turbulence in the flow regime could produce negative velocity at the sensor with a positive mean velocity for the bulk flow.

The results under surcharged conditions indicate discrepancies between pipe size, velocity, and flow rate. The data for velocity and flow rate with the pipe flowing full October 17-20, 1998, correspond to a pipe diameter of 45.3 inches, rather than the nominal size of 42 inches. For the June 9,

1997, storm, the 3:56 am reading indicates a level of 42.3 inches, but the flow and velocity readings (8,440 gpm at 1.34 fps) correspond to cross-sectional area equivalent to 50.7-inch diameter pipe. Again, the dependence of the flow on the selected site coefficient value may be a factor.

In summary, the data for peak flows and surcharged conditions has some accuracy uncertainties, but there is no indication that the flows did not indeed show cyclical surges in velocity at the flow monitoring manhole.

Battle Bend Meter. The Utility furnished flow monitoring data from a temporary meter installed at manhole "X" on the 42-inch main south of Battlebend subdivision approximately 1,400 ft west of IH-35. Records indicate slopes of 0.18% upstream and downstream of the meter, with corresponding capacity of 19,100 gpm flowing full. The meter was installed in the upstream pipe. Construction plans indicate a flowline drop of 6.4 inches across the manhole.

The flow monitoring data covers the period of February 1 to May 6, 1999. Figure 3-8 shows 15-minute flow and level readings during the monitoring period. A minor storm occurred on March 18 just before midnight, producing an inflow response, but the total flow was not significantly high due to the time of storm occurrence. The peak flow during the monitoring period did not exceed 8,000 gpm (42% of capacity) and frequently less than 6,000 gpm. Recorded flow depths did not exceed 13 inches in the 42-inch pipe. However, the available data does not reflect peak wet weather flow conditions, and the data for the Battlebend meter has some inconsistencies with the Creekwood and Wasson meters, as discussed below.

Wasson Meter. The Wasson meter was installed in manhole "CC" on the 36-inch main approximately 400 ft east of South Congress Avenue. Records indicate upstream slope of 0.31% and 16,700 gpm capacity, and downstream slope of 0.35% and 17,760 gpm capacity. The meter was installed in the upstream pipe. The Wasson meter is 4,025 ft upstream of the Battlebend meter.

Figure 3-9 shows 15-minute flow and level readings during the monitoring period. Flow monitoring data indicates a maximum recorded flow of 10,380 gpm (62% of capacity). Peak daily flow was typically less than 8,500 gpm. The maximum flow depth recorded was 21.5 inches, and the typical maximum daily level was less than 18 inches.

Although monitoring did not record any overloading at the meter site, the data suggests that the site was subject to backwater effects caused by possible downstream problems. During the March 18 storm, while the flows at the Creekwood and Battlebend meters were peaking, the flows at the Wasson meter went from 10,100 gpm to 7,240 gpm to 10,180 gpm in three consecutive readings. The lower middle flow had increased depth and reduced velocity readings, suggesting a downstream constraint creating a backwater

condition. There were several other occasions with reduced flow and velocity at increased depth when the other meters were peaking.

Comparison of Wasson , Battlebend and Creekwood Data. Flows at the three sites are shown together in Figure 3-10 (sheets a-g). Each sheet shows a two-week Sunday-to-Saturday period. Comparison of the flow data for the Wasson and Battlebend meters shows that the downstream Battlebend flows are consistently less than the upstream Wasson flows. Laterals "X1", "Z", and "BB" contribute flow to the interceptor between the two meters, so the downstream flows should be greater. With periods of obviously bad data factored out (e.g., 4/2/99 to 4/7/99 in Figure 3-9 for Wasson velocity sensor fouling, and 4/20-21 for Battlebend), almost 80% of the hourly flow readings were lower at Battlebend than at Wasson. On a cumulative flow basis over the monitoring period (2084 hours of comparable data), the Battlebend meter recorded a volume of 554 million gallons versus 678 million gallons upstream at Wasson.

Comparison of the Wasson and Creekwood data shows that the Wasson flow readings were greater than the Creekwood readings 53% of the time. Time of travel or clock dissynchronization might be a factor in some instances. The Creekwood meter shows a cumulative volume of 699 million gallons versus 678 million gallons for Wasson, but over the 2084 hours of comparable data, this difference represents an average flow of 167 gpm.

The level, velocity and flow data for the three meters were analyzed in several ways to evaluate the reliability of the data. Figures 3-11, 3-12, and 3-13 show scattergraphs of level versus velocity for the Wasson, Battlebend, and Creekwood meters, respectively. These graphs include the theoretical level versus velocity relationship by Mannings equation for the record pipe size and slope with n=0.013. Scattergraphs reflect the degree to which hydraulic conditions at the meter conform to uniform open channel flow conditions for which Mannings equation applies. Scattergraphs also reveal data outliers due to sensor fouling, equipment malfunction, or backwater conditions. Using hourly average depths and velocities is not as accurate as using 5- or 15-minute readings (not available), but the results reflect the overall nature of the data.

The data was also evaluated by calculating the value of $\sqrt{s}/n$ in Mannings equation for each depth and velocity reading over the monitoring period. Although slope and friction factor can not each be accurately determined separately, the value $\sqrt{s}/n$ is a constant in Mannings equation which can be calculated from velocity and depth readings.

The Wasson scattergraph in Figure 3-11 shows a large number of low velocity readings over the typical 12 to 18 inch depth range. This suggests a velocity sensor probe problem rather than a backwater condition since these low velocities do not show higher levels. The $\sqrt{s}/n$ graph in Figure 3-14 shows relatively steady $\sqrt{s}/n$ values up to about March 12, after which the data becomes much more unsteady. Pipe

capacity flowing full calculated using the average $\sqrt{s}/n$ value through March 12 is 16,200 gpm, compared to Mannings capacity of 16,670 gpm using record slope and assumed n of 0.013. The Wasson flow data through March 12 appears to be consistently reliable, but after March 12 the reliability of the data in general become more uncertain.

The Battlebend scattergraph in Figure 3-12 shows velocities much greater than what would be expected from Mannings equation for the record size and slope, although the level-velocity data points form a tight consistent pattern. Furthermore, the $\sqrt{s}/n$ values in Figure 3-15 remain fairly steady over the monitoring period. However, the pipe capacity flowing full calculated using the average $\sqrt{s}/n$ value for the monitoring period is 29,275 gpm, compared to Mannings capacity of 19,100 gpm using record slope and assumed n of 0.013. The flow monitor level and velocity data points apparently reflect the hydraulic drawdown effect caused by the reported 0.52-ft flowline drop at the manhole. Drawdown would produce lower depths and higher velocities than indicated by Mannings equation for uniform flow. Theoretically the area-velocity flow meter should produce reliable flow data under such conditions since it calculates flow rate as the product of flow cross-sectional area (as a function of depth and pipe diameter) and mean velocity (as determined by the sensor point velocity and the selected site coefficient). The understated flows may be the result of difficulty in determining the appropriate site coefficient, or possibly even input of the wrong pipe size for meter setup.

The Creekwood scattergraph in Figure 3-13 shows a bimodal pattern. At levels between 7 and 13 inches, the level and velocity data points show the expected pattern although with considerably greater velocities (4.5 to 6.2 fps) than indicated by Mannings equation. Records do not indicate a flowline drop at manhole "D" which might produce a drawdown effect at the meter probe to explain the high velocities. The second grouping is concentrated in a range of about 14 to 22 inches with velocities between 3.5 and 4.2 fps. The trend shows reduced velocities at greater depth characteristic of backwater effects. Lower velocity caused by sensor fouling would not show a change in the depths. The apparent backwater effect might be the result of some blockage that developed in the pipe downstream. Evidence of a temporary backwater condition occurs on February 10-11 in Figure 3-5. The graph of $\sqrt{s}/n$ versus time in Figure 3-16 also shows a bimodal pattern which corresponds to the bimodal grouping in Figure 3-13. The $\sqrt{s}/n$ values around 6 are associated with the high velocity/low depth data points. Figure 3-16 shows that the apparent backwater conditions are much more prevalent in the latter half of the monitoring period. The flow rate data up through around March 14, 1999, appears to have the most reliability, except for the February 10-11 period mentioned above.. Review of the flow data in Figure 3-5 shows that higher reported flow rates are associated with the "backwater" data, again probably due to the problem of using an appropriate site coefficient for the different hydraulic conditions. There were no instances in this monitoring period of the severe, cyclical velocity and flow pulsations and surcharging discussed above associated with significant storm events.

Depth Information from TV Inspection. Information on current typical dry weather flow rates is also provided by the recent closed circuit television inspection of the interceptor. The television inspection was conducted during dry weather since lower flows allow better inspection and safer working conditions. These flow conditions may fairly well represent average dry weather flows since they occurred mid-day between the morning and evening daily peak flows. Table 3-2 includes the estimated depth of flow recorded by the operator for each televised segment, which reflects the following hydraulic conditions:

- The 36-inch pipe in the vicinity of Congress Avenue between manholes "Z" and "HH" had 15 to 20 inches of flow on three different inspection days. A 20-inch depth (d/D=0.56) corresponds to a flow-to-capacity (q/Q) ratio of 48% full. For section "CC" to "DD", the operator estimated 15-inch depth during the mid-afternoon inspection. This is generally in agreement with data from the Wasson meter.

- The upper section of the 42-inch pipe at 0.18% slope between the 36-inch pipe (manhole "Z") and Stassney Lane (manhole "S") had 14 to 20 inches of flow, corresponding to q/Q up to 37% full. A depth of 14 inches was reported at manhole "X" for the early afternoon inspection. The Battlebend meter at "X" consistently reported depths of 10 to 12 inches at that time. This tends to confirm that the meter was sensing drawdown conditions.

- The lower section of the 42-inch pipe at 0.28% slope in the Creek Bend area west of Pleasant Valley Road (diversion box to manhole "G") had observed depths of 20 to 27 inches on four different days of inspection, which corresponds to q/Q values of 37% to 62% full. The reported midday depth in section "D" to "E" upstream of the Creekwood meter was 25 inches. In comparison, the typical depths recorded by the Creekwood meter were 10 to 11 inches at this time of day in the February 1 to March 14, 1999, data. The observed depth tends to confirm a drawdown effect at the Creekwood meter implied by the scattergraph in Figure 3-13.

Conclusions. Data from the Creekwood meter confirms that the lower section of the interceptor surcharges under wet weather flow conditions, although there are hydraulic factors involved other than flow exceeding pipe capacity. The observed cyclical pulsations in velocity and flow are not characteristic of conventional overloading. The diversion box to the tunnel at the lower end of the interceptor is a suspected factor. The interceptor does not surcharge under normal dry weather flow conditions, although there is monitoring evidence of backwater conditions.

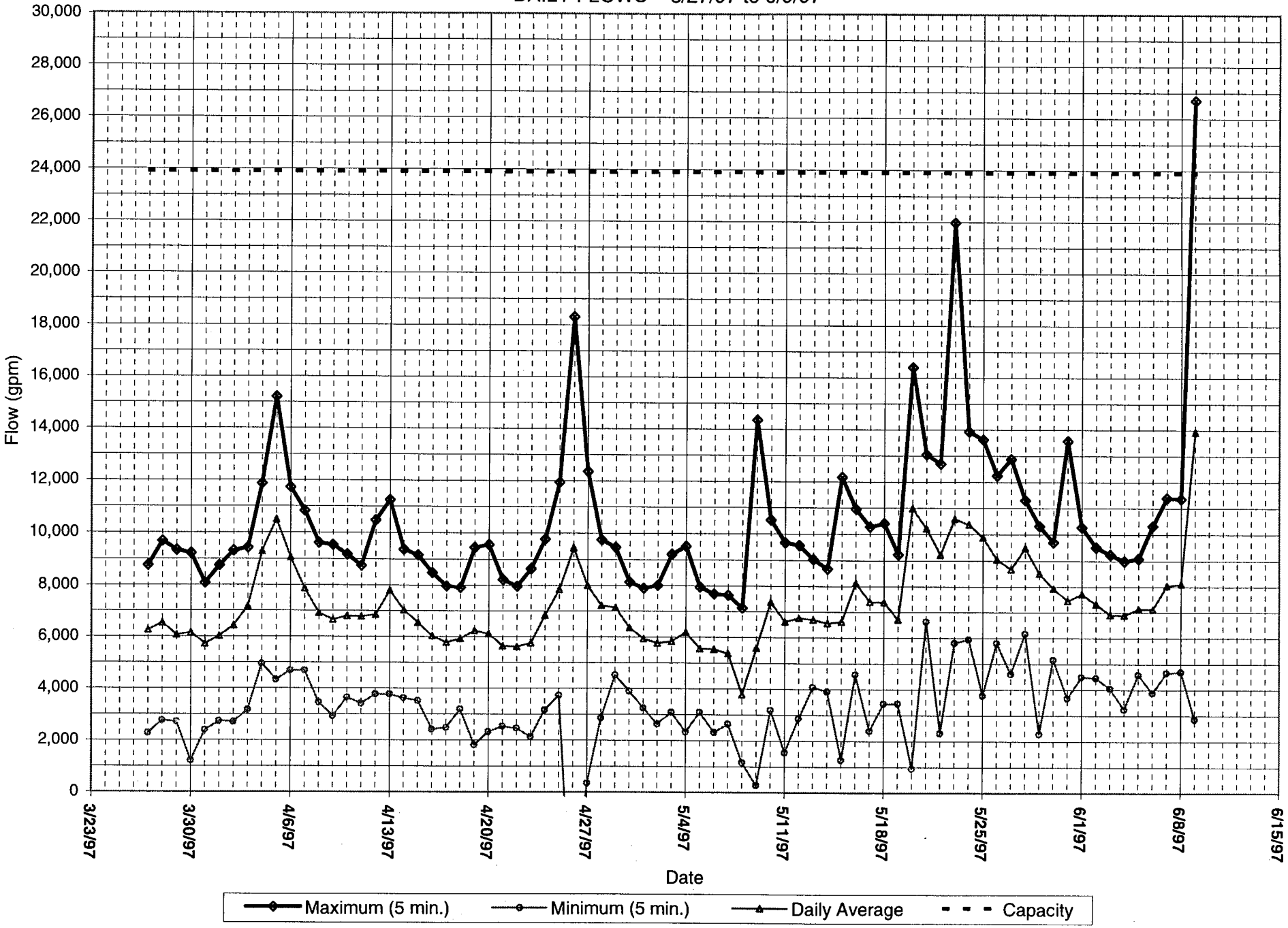
Monitoring at the two upstream locations did not record any surcharging, although the monitoring period did not include any major rainfall events. The Battlebend monitoring data is not considered a reliable indicator of current conditions since it consistently showed lower flow rates than the

TABLE 3-2. DRY WEATHER FLOWS DURING CCTV INSPECTION								
Date	Time	Upstream MH	Dnstream MH	Size	Slope	**Depth**	**d/D**	Comments
21-Oct-97	3:00 PM	HH	GG	36 " @	0.30%	**20 "**	**0.56**	
21-Oct-97	4:00 PM	GG	FF	36 " @	0.30%	**20 "**	**0.56**	
21-Oct-97	5:00 PM	FF	EE	36 " @	0.30%	**20 "**	**0.56**	
22-Oct-97	1:45 PM	EE	DD	36 " @	0.30%	**15 "**	**0.42**	
22-Oct-97	2:00 PM	DD	CC	36 " @	0.30%	**15 "**	**0.42**	Wasson meter @ MH "CC"
24-Oct-97	12:00 PM	CC	BB	36 " @	0.35%	**20 "**	**0.56**	
24-Oct-97	12:30 PM	BB	AA	36 " @	0.35%	**20 "**	**0.56**	
24-Oct-97	2:30 PM	AA	Z	36 " @	0.18%	**20 "**	**0.56**	
25-Oct-97	9:00 AM	Z	Y	42 " @	0.18%	**20 "**	**0.48**	
27-Oct-97	1:00 PM	Y	X	42 " @	0.18%	**14 "**	**0.33**	Battlebend meter @ MH "X"
29-Oct-97	12:00 PM	X	W	42 " @	0.18%	**na**	**na**	
29-Oct-97	2:30 PM	W	V	42 " @	0.18%	**15 "**	**0.36**	
6-Nov-97	2:00 PM	V	U	42 " @	0.18%	**20 "**	**0.48**	
7-Nov-97	10:30 AM	U	T	42 " @	0.18%	**20 "**	**0.48**	
7-Nov-97	2:00 PM	T	S	42 " @	0.28%	**13 "**	**0.31**	
								section not TV'd
22-Apr-98	11:30 AM	O	N	42 " @	0.24%	**20 "**	**0.48**	
								section not TV'd
2-Mar-98	2:00 PM	G	F	42 " @	0.28%	**20 "**	**0.48**	
2-Mar-98	12:00 PM	F	E	42 " @	0.28%	**25 "**	**0.60**	
3-Mar-98	12:30 PM	E	D	42 " @	0.28%	**25 "**	**0.60**	Creekwood meter @ MH "D"
4-Mar-98	10:30 AM	D	C	42 " @	0.28%	**27 "**	**0.64**	
12-Mar-98	1:00 PM	C	B	42 " @	0.28%	**25 "**	**0.60**	
21-Apr-98	1:45 PM	B	A	42 " @	0.28%	**20 "**	**0.48**	
21-Apr-98	2:00 PM	A	Box	42 " @	0.28%	**18 "**	**0.43**	

upstream Wasson meter. Six weeks of reliable Wasson monitoring data showed typical peak dry weather flows reaching about 45% of pipe capacity. With an average dry weather flow during the period of approximately 5,200 gpm, a peak flow that is 3.2 times average flow (a little over 3Q) would overload the 36-inch pipe. Considering that a significant amount of the collection system in the upstream service area is older and more susceptible to I/I, wet weather flows exceeding 3 Q would not be unexpected.

Additional data acquisition with flow monitoring is needed in order to determine the cause of the surging conditions at peak flows in the lower interceptor. Determining the cause might allow a "quick fix" to the surcharging problem in advance of construction of the relief tunnel. Additional monitoring data from the upper end of the interceptor is needed in order to determine the wet weather inflow characteristics in the 36-inch section. With several inflow events recorded along with rainfall intensity data, a "Q vs I" analysis of inflow versus rainfall intensity could determine the sensitivity of the 36-inch section to surcharging as a function of storm recurrence interval, as a measure of the urgency or priority for construction of the relief improvements.

FIGURE 3-2. WILLIAMSON CK INTERCEPTOR - CREEKWOOD METER
DAILY FLOWS - 3/27/97 to 6/9/97



Flowmon.xls

FIGURE 3-3. WILLIAMSON CREEK INTERCEPTOR - CREEKWOOD METER
HOURLY AVERAGE FLOWS (11/10/97 - 12/12/97)

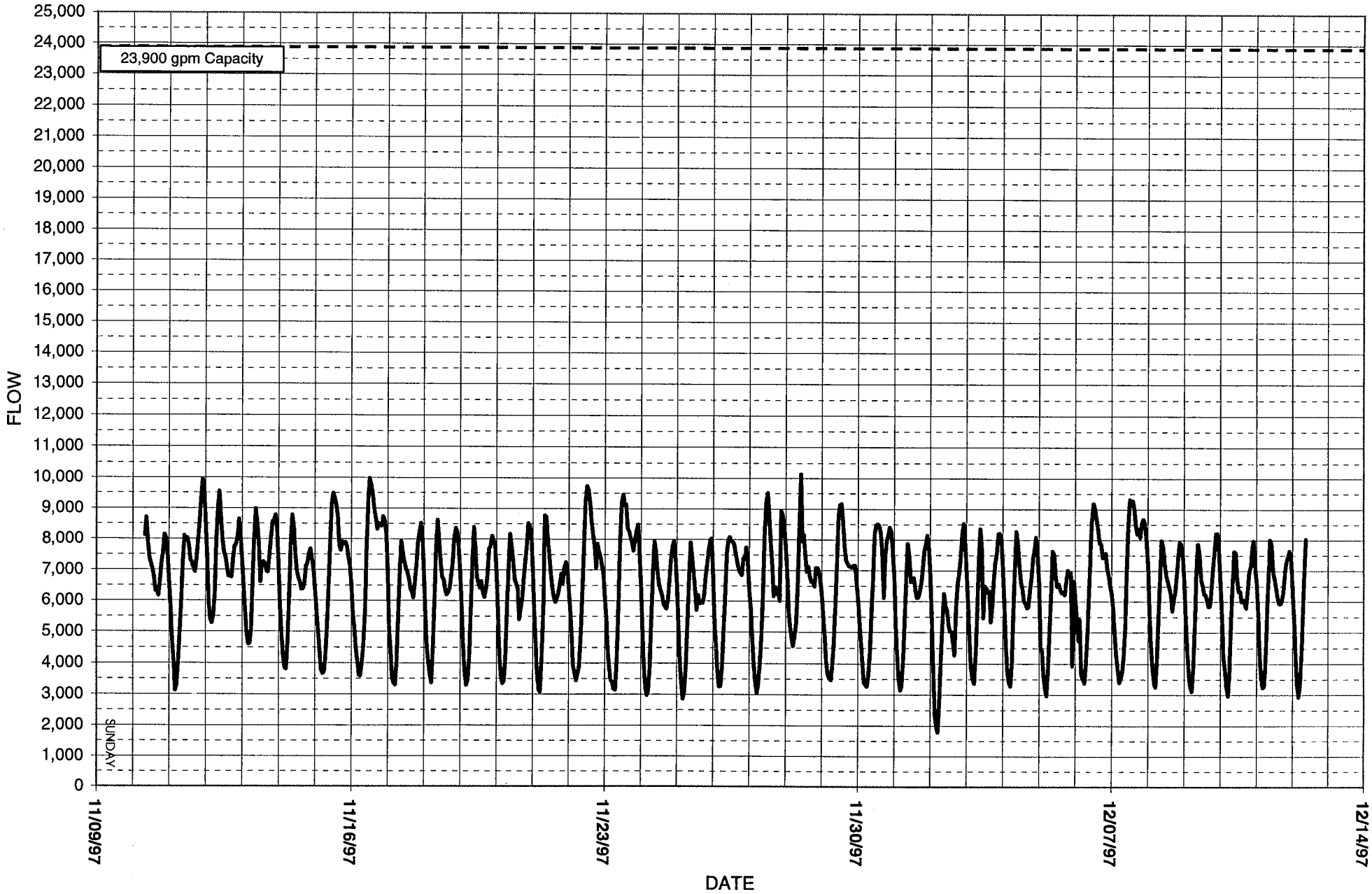
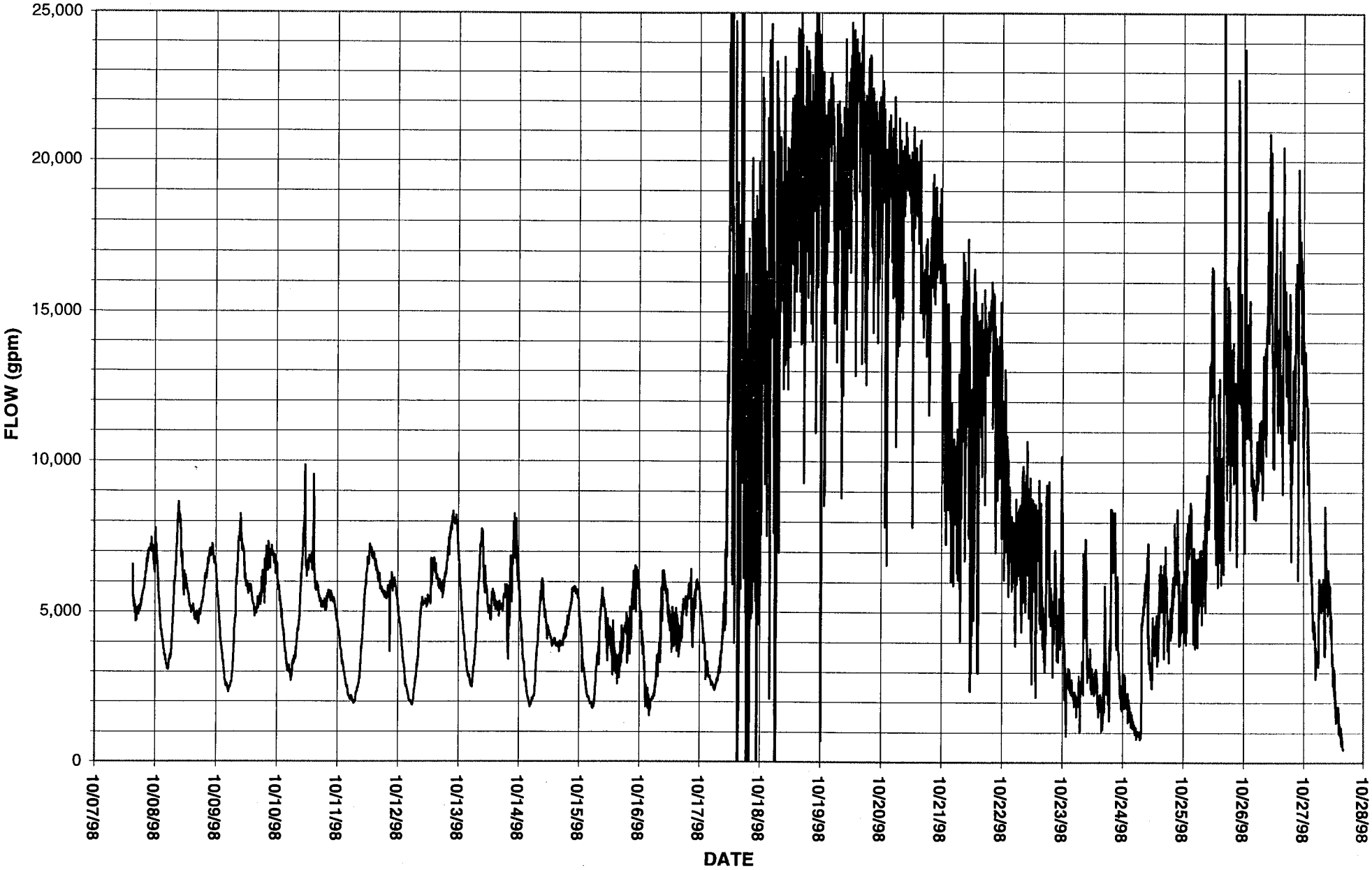


FIGURE 3-4. WILLIAMSON CK INTERCEPTOR - CREEKWOOD METER
10/7/98 -- 10/27/98 (5-minute values)



flowmon2.xls

FIGURE 3-5. WILLIAMSON CREEK INTERCEPTOR - CREEKWOOD METER
HOURLY AVERAGE FLOWS AND LEVELS (2/1/99 - 5/6/99)

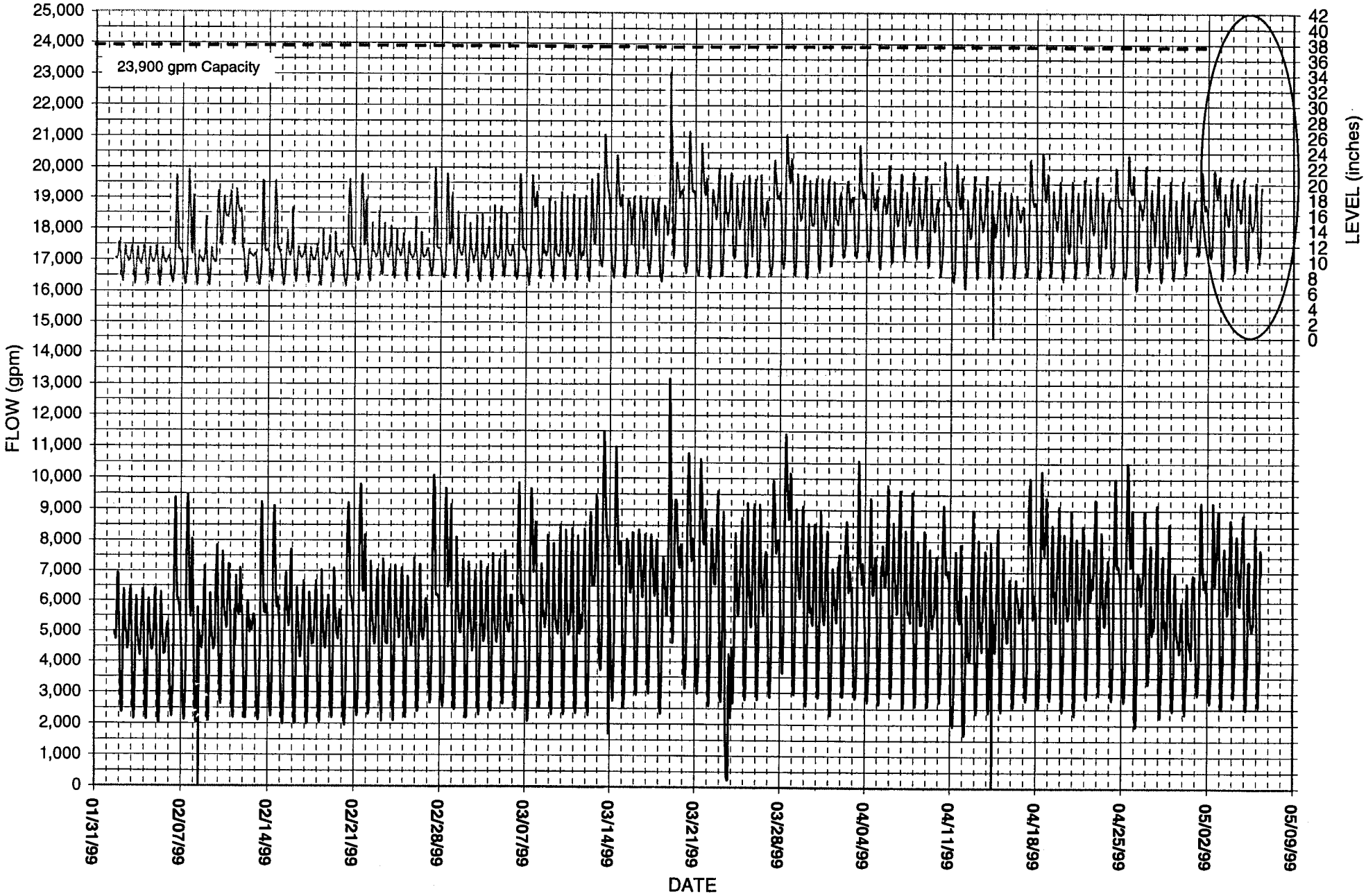


FIGURE 3-6. WILLIAMSON CK INTERCEPTOR - CREEKWOOD METER
6/9/97 STORM (Max and Min 5-minute Values Each Hour)

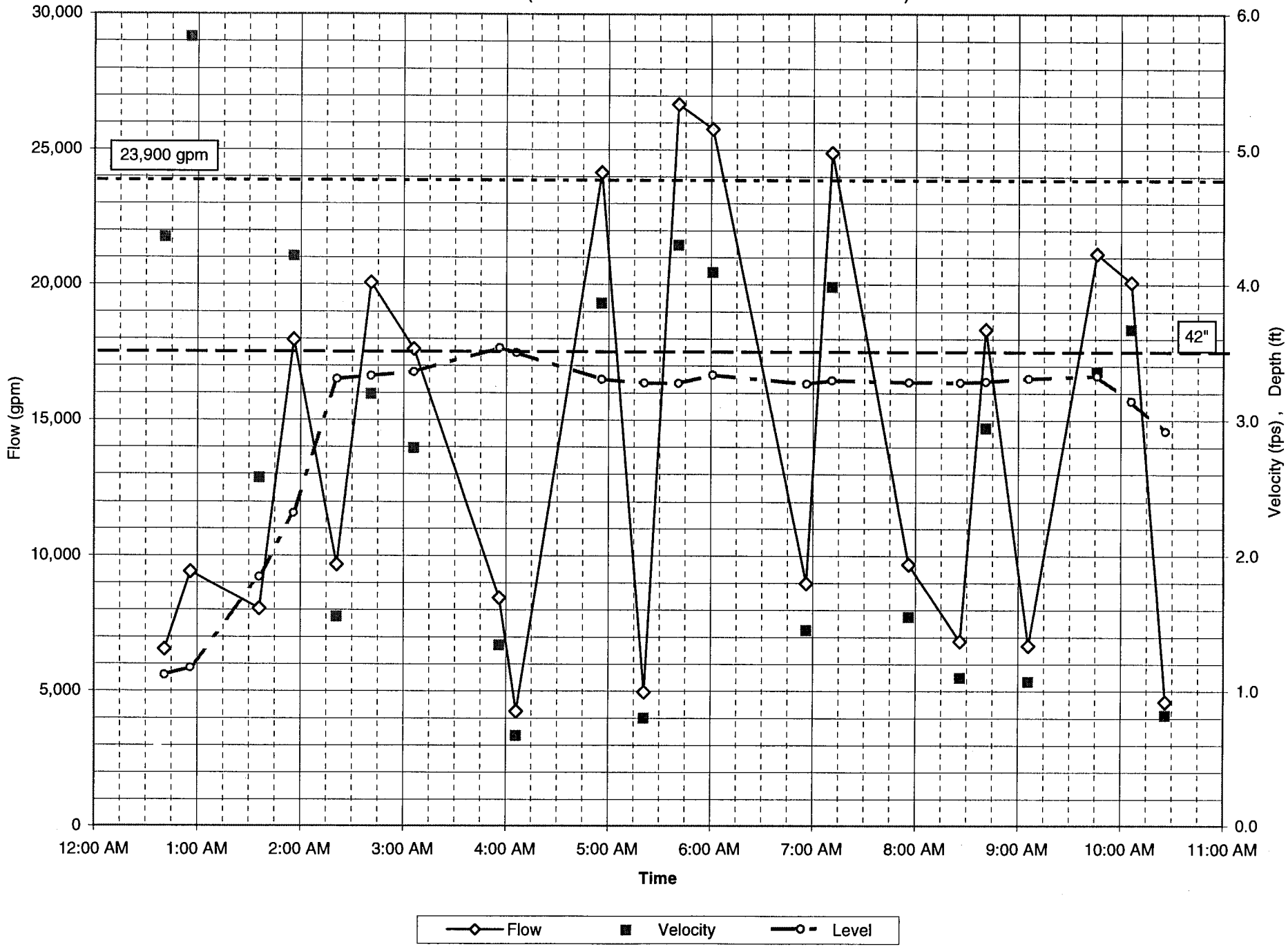
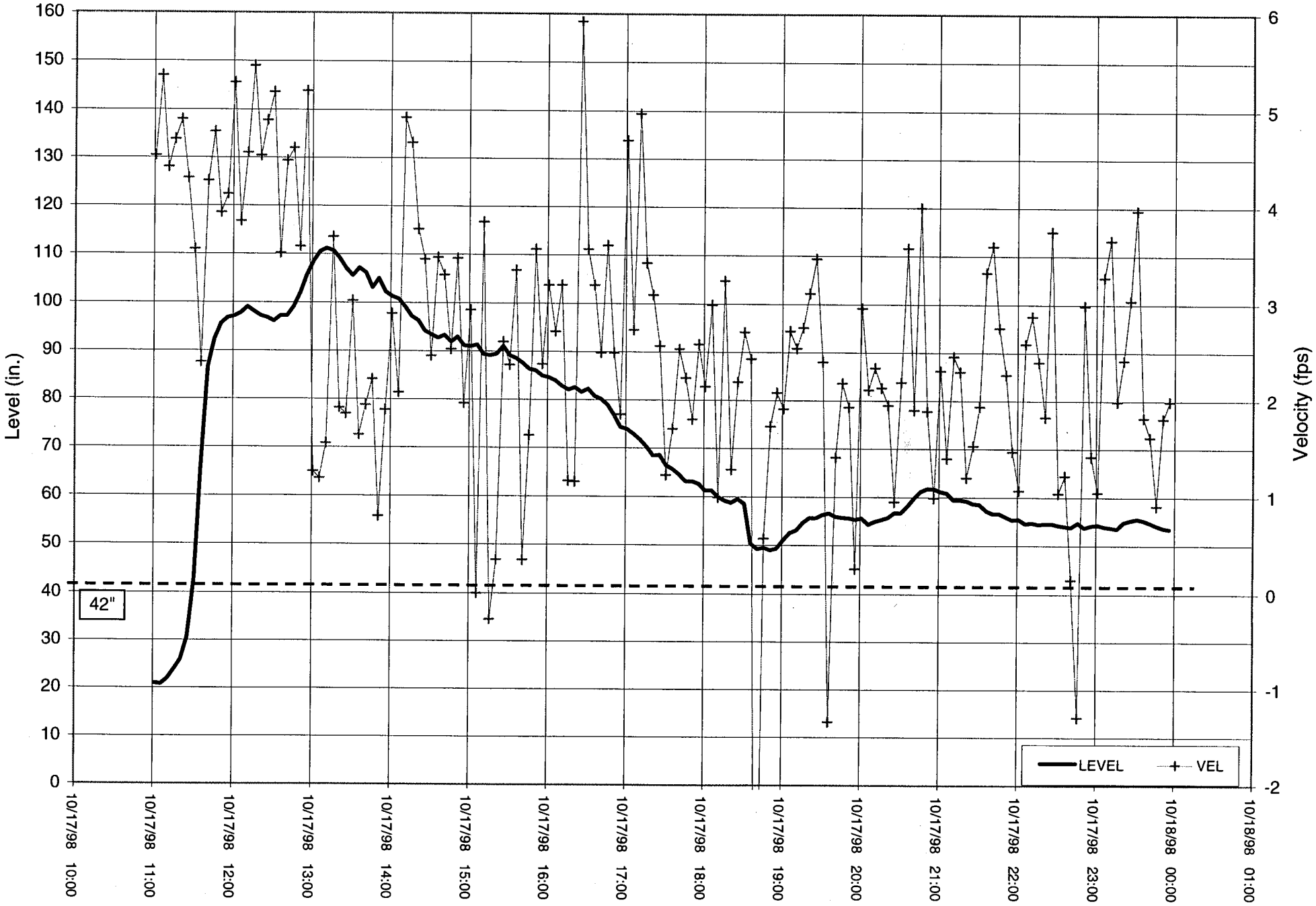


FIGURE 3-7. WILLIAMSON CK INTERCEPTOR -- CREEKWOOD METER
10/17/98 STORM (5-minute Values)



flowmon2.xls

FIGURE 3-8. WILLIAMSON CREEK INTERCEPTOR - BATTLEBEND METER
HOURLY AVERAGE FLOWS AND LEVELS (2/1/99 - 5/6/99)

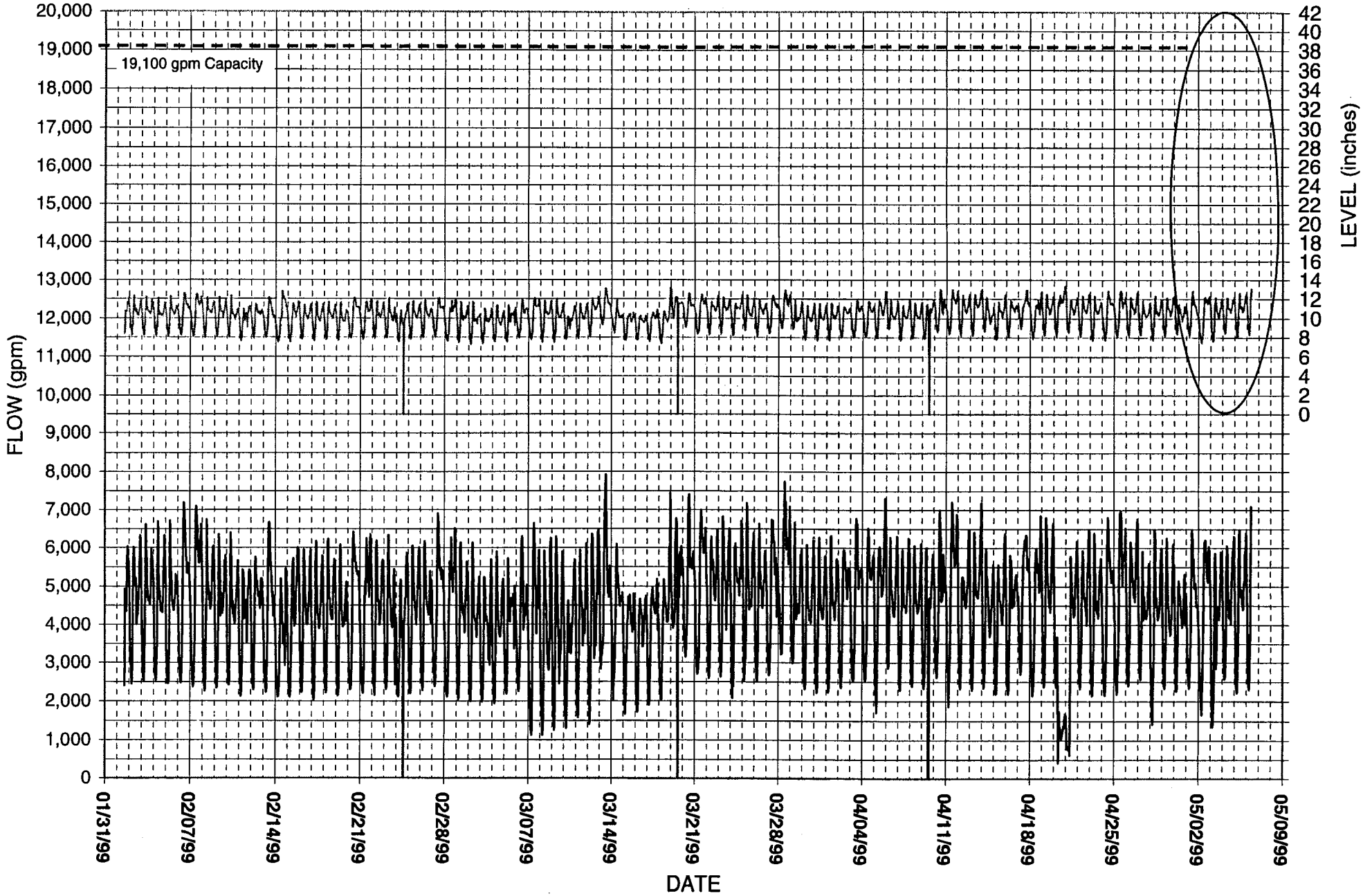


FIGURE 3-9. WILLIAMSON CREEK INTERCEPTOR - WASSON METER
HOURLY AVERAGE FLOWS AND LEVELS (2/1/99 - 5/6/99)

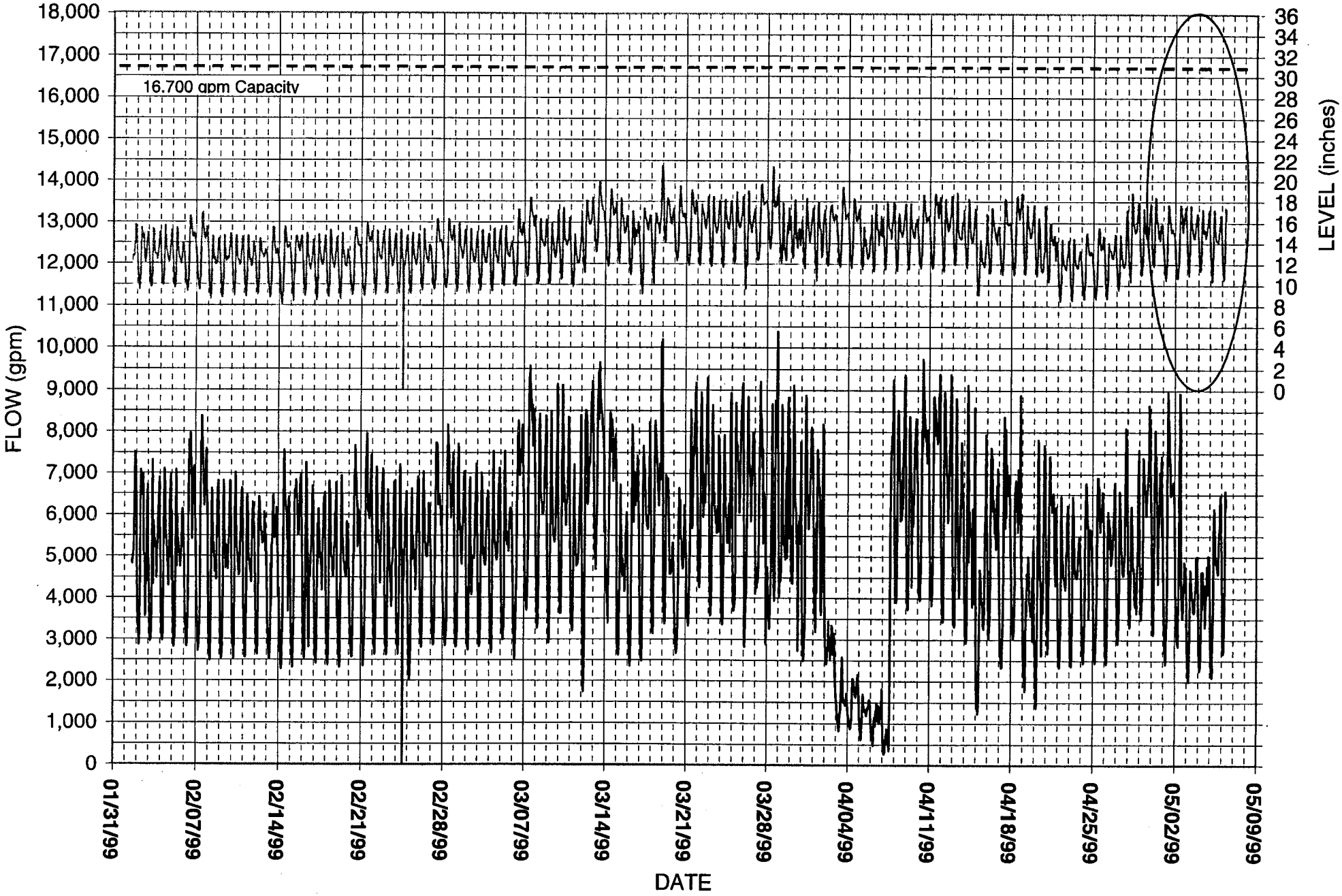


FIGURE 3-10a. COMPARISON OF WILLIANSON CK INTERCEPTOR FLOWS

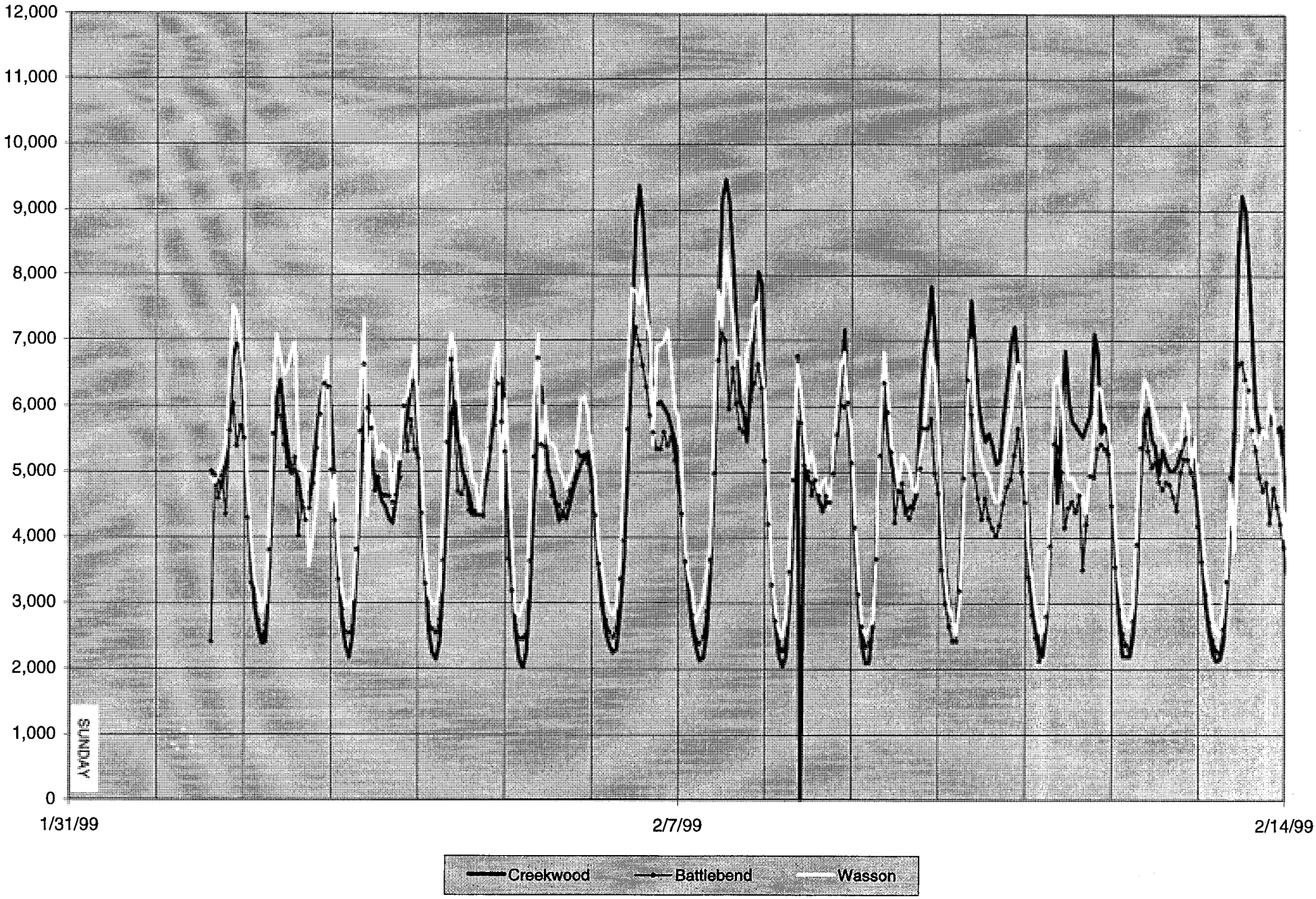


FIGURE 3-10b. COMPARISON OF WILLIAMSON CK INTERCEPTOR FLOWS

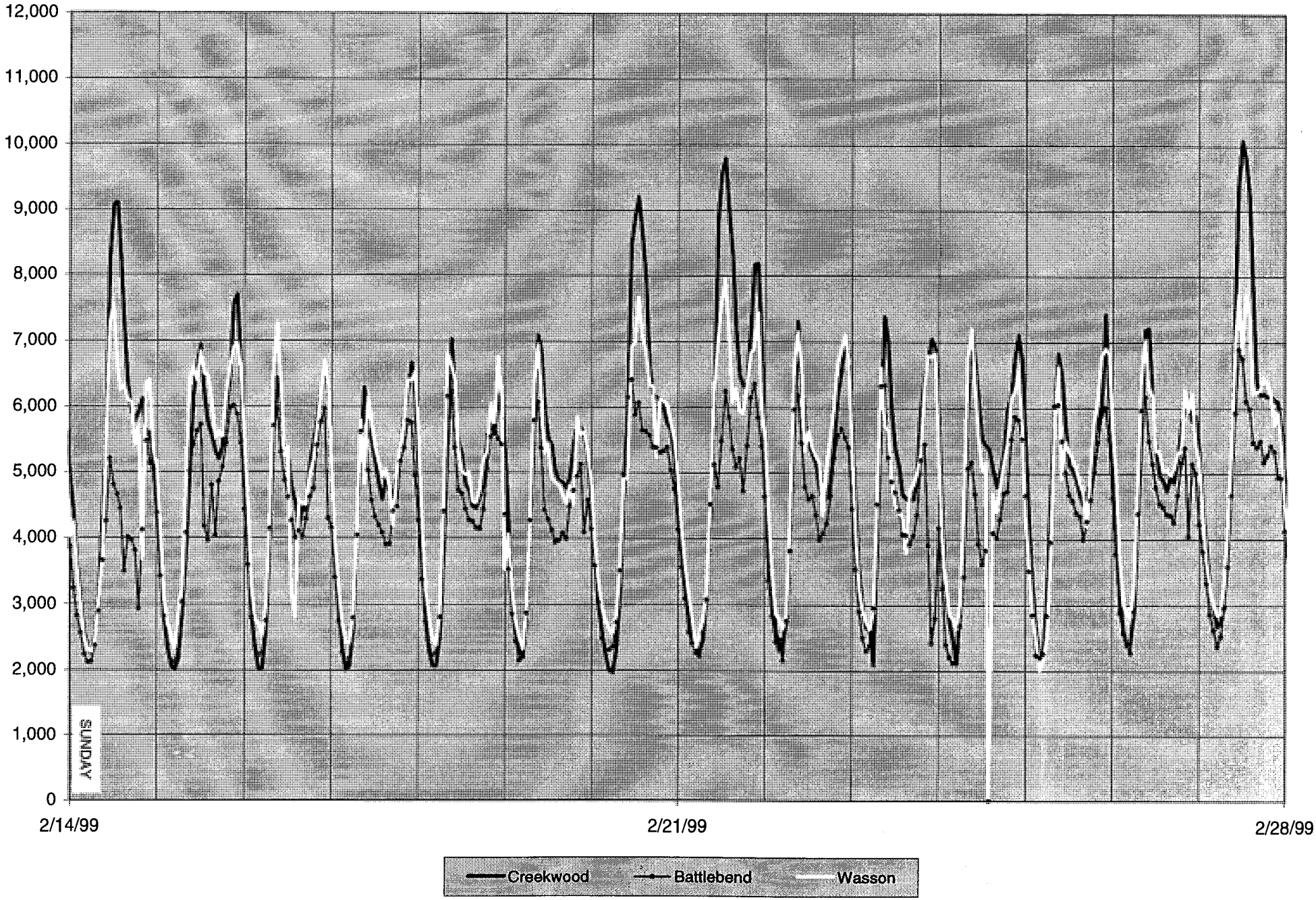


FIGURE 3-10c. COMPARISON OF WILLIAMSON CK INTERCEPTOR FLOWS

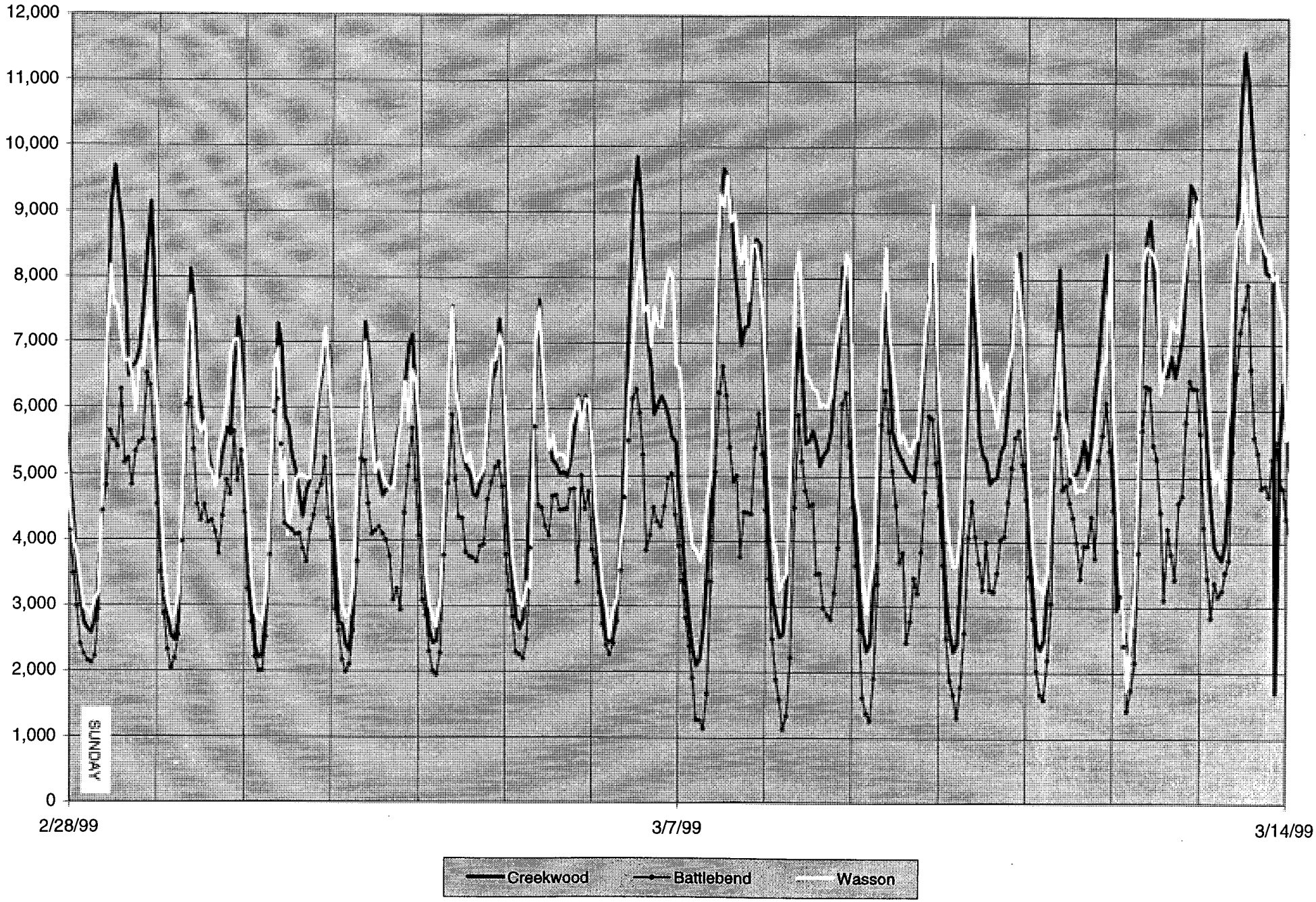


FIGURE 3-10d. COMPARISON OF WILLIAMSON CK INTERCEPTOR FLOWS

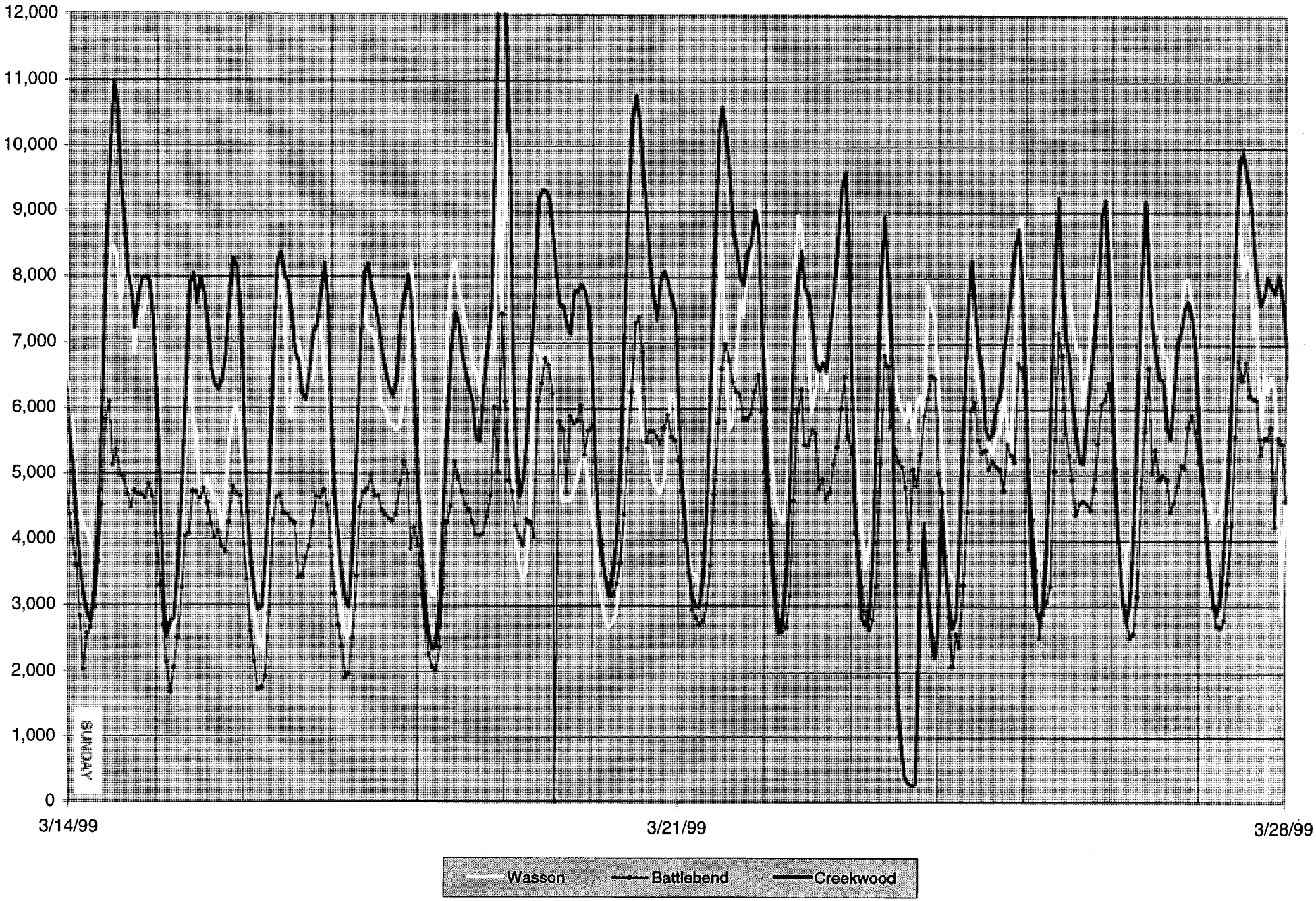


FIGURE 3-10e. COMPARISON OF WILLIAMSON CK INTERCEPTOR FLOWS

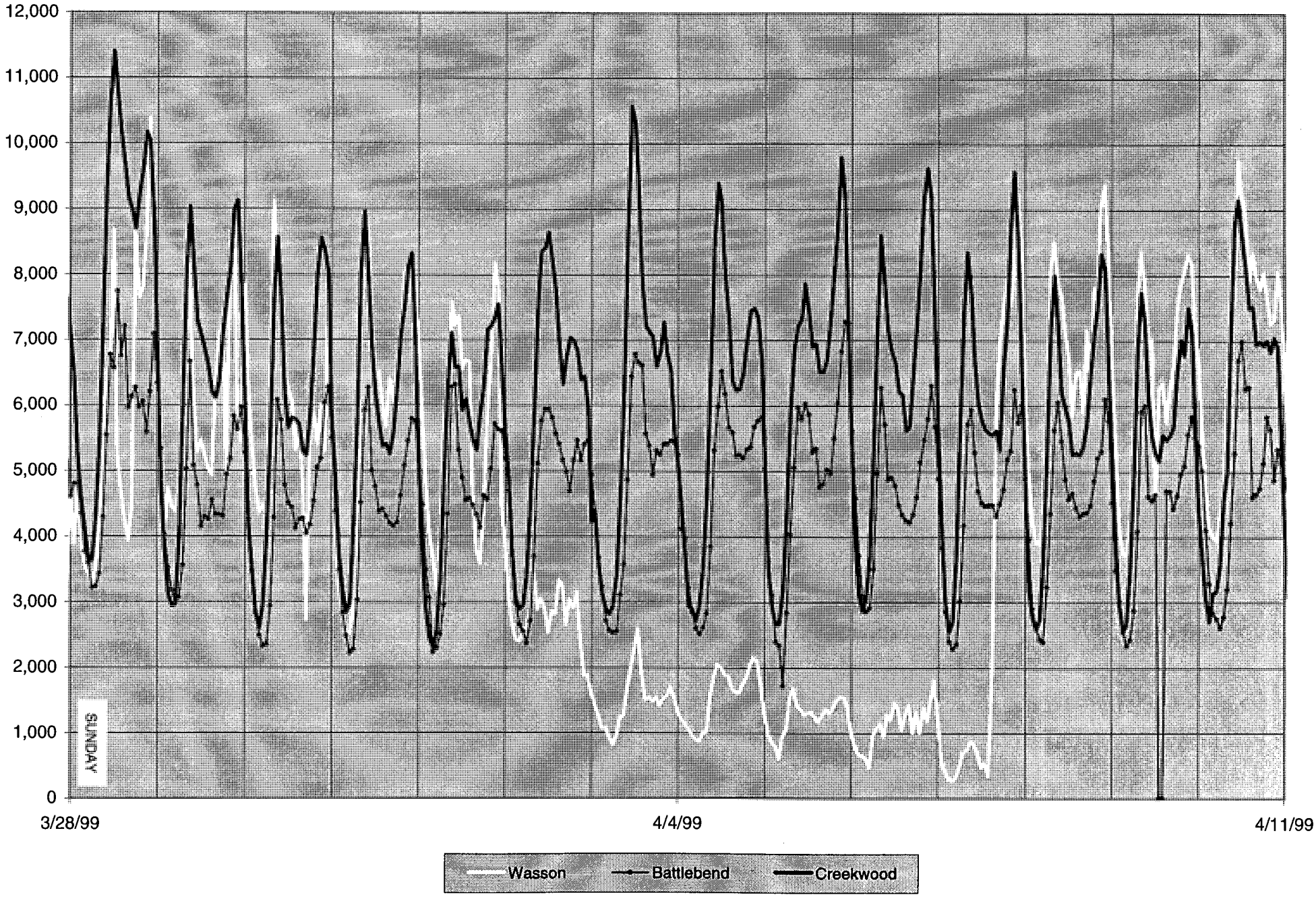


FIGURE 3-10f. COMPARISON OF WILLIAMSON CK INTERCEPTOR FLOWS

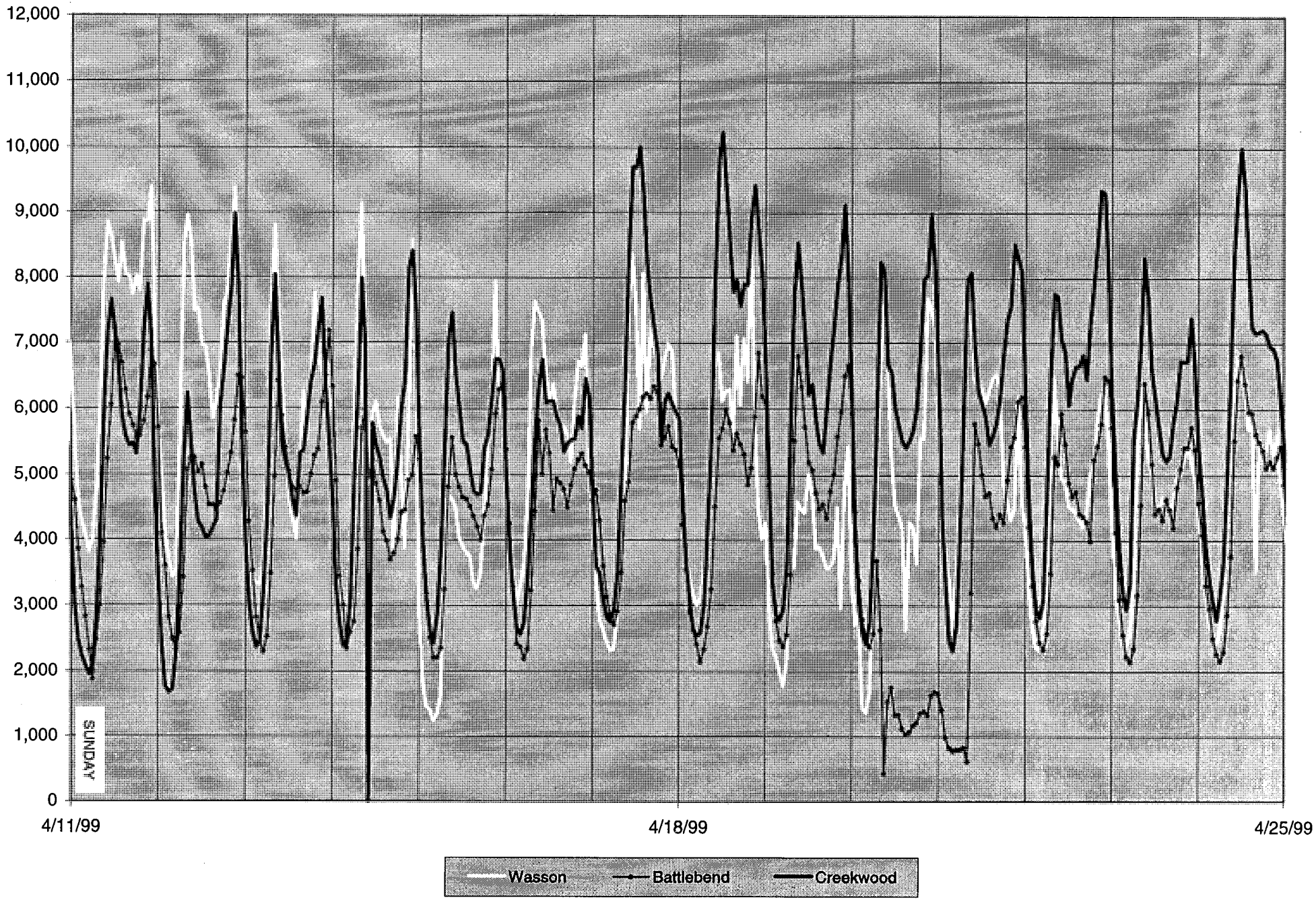


FIGURE 3-10g. COMPARISON OF WILLIAMSON CK INTERCEPTOR FLOWS

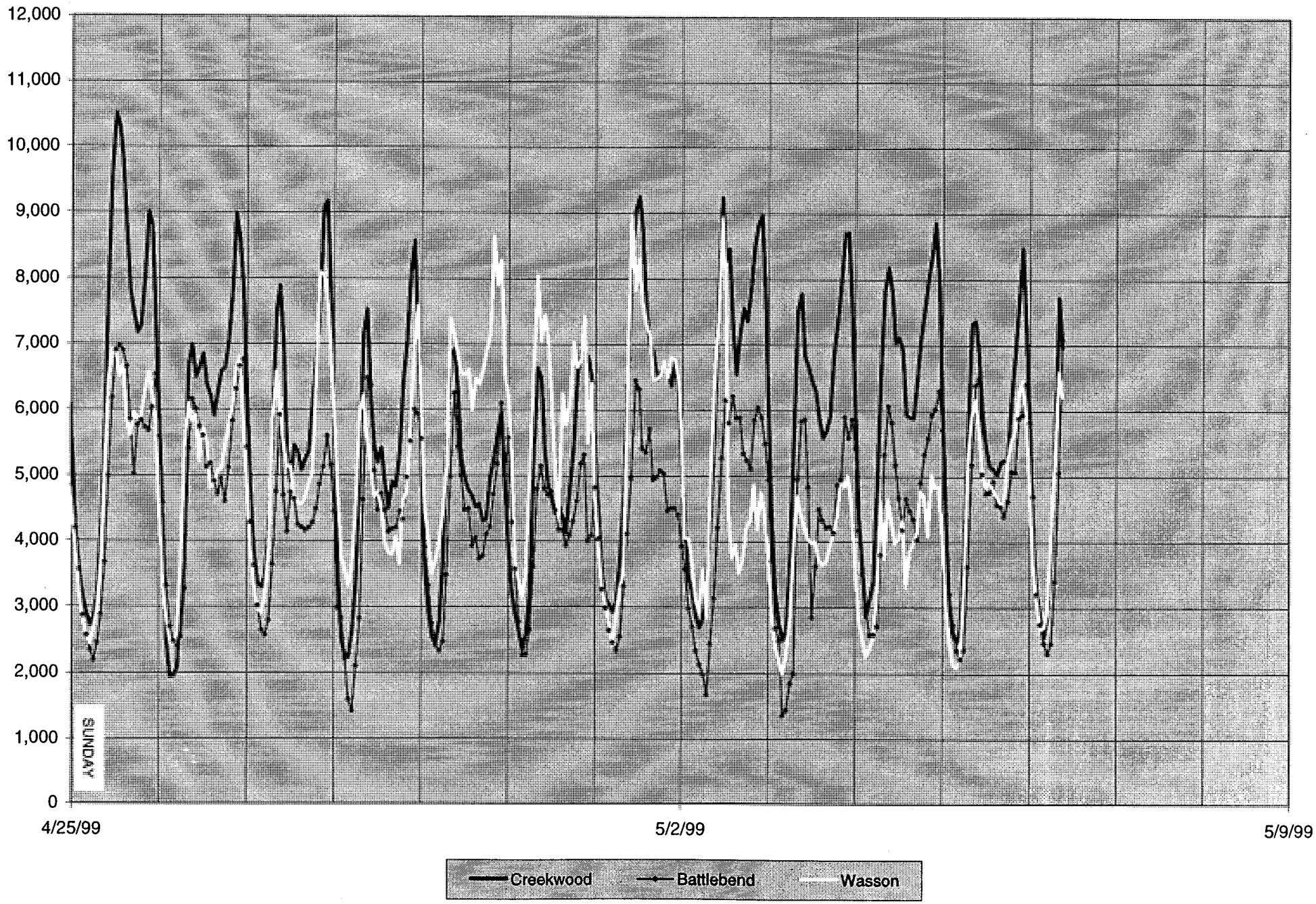


FIGURE 3-11. SCATTERGRAPH OF LEVEL VS VELOCITY - WASSON METER
(2/1/99 to 5/6/99)

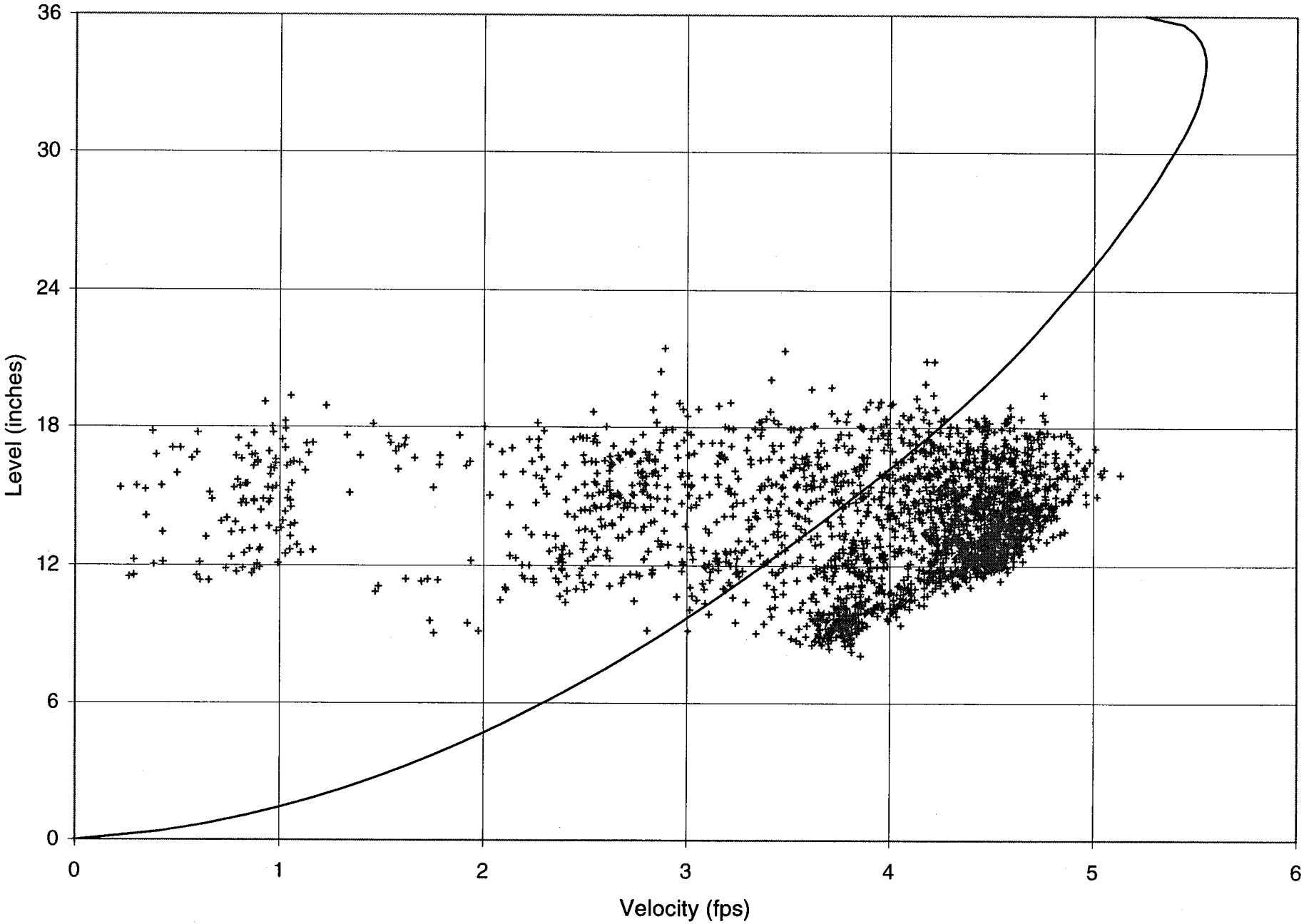


FIGURE 3-12. SCATTERGRAPH OF LEVEL VS VELOCITY - BATTLEBEND METER
(2/1/99 to 5/6/99)

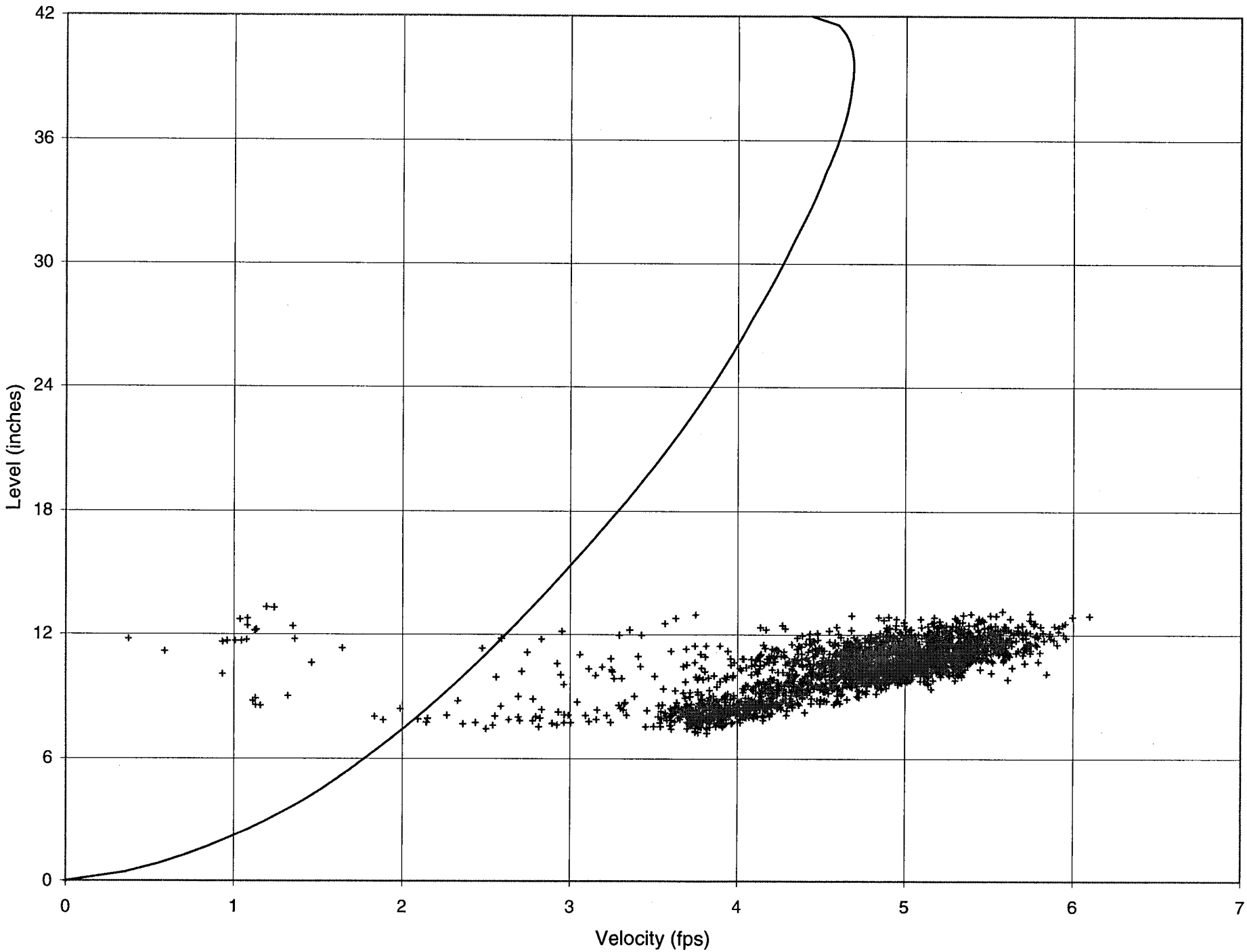


FIGURE 3-13. SCATTERGRAPH OF LEVEL VS VELOCITY - CREEKWOOD METER
(2/1/99 to 5/6/99)

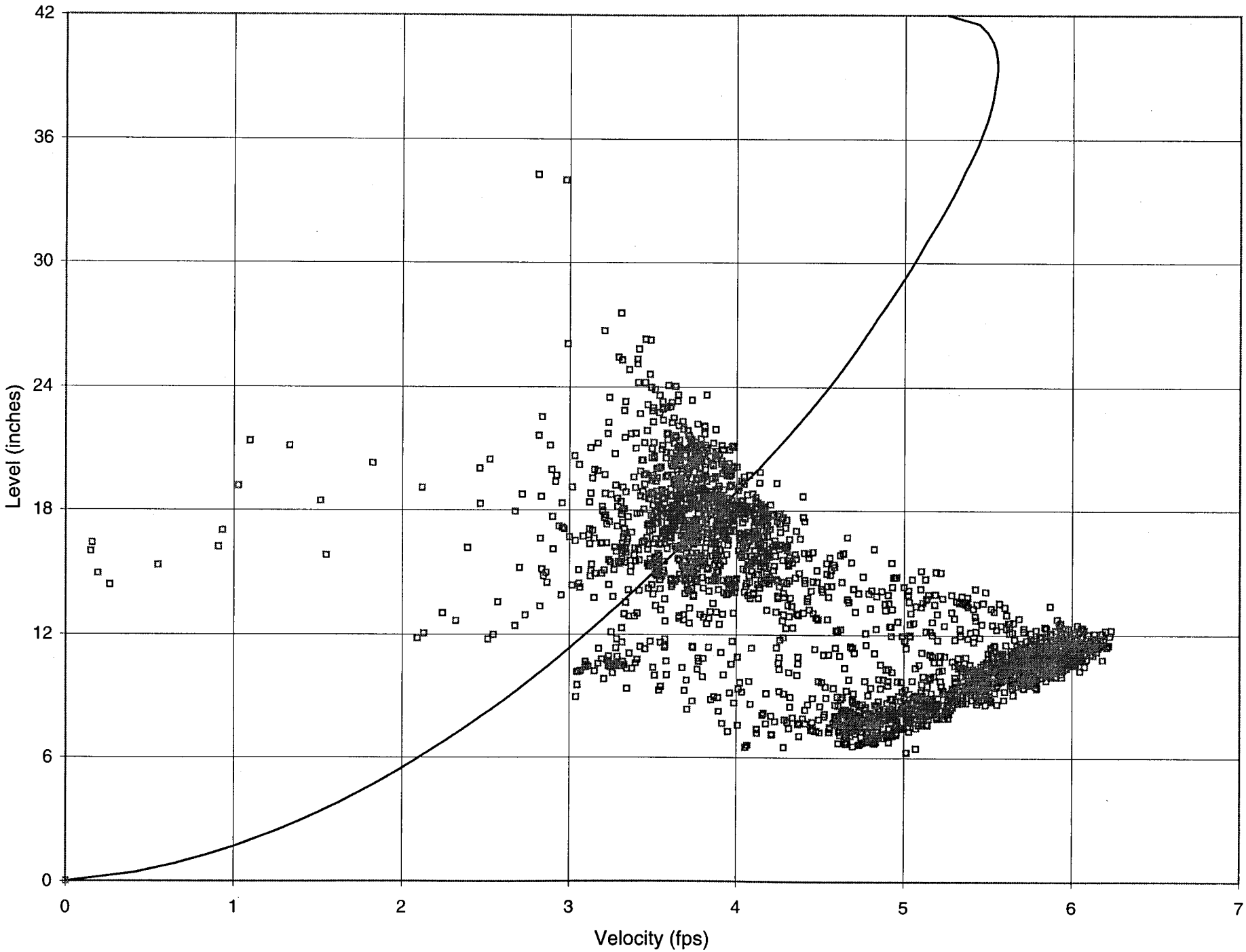


FIGURE 3-14. √s/n VALUES - WASSON METER
(2/1/99 to 5/6/99)

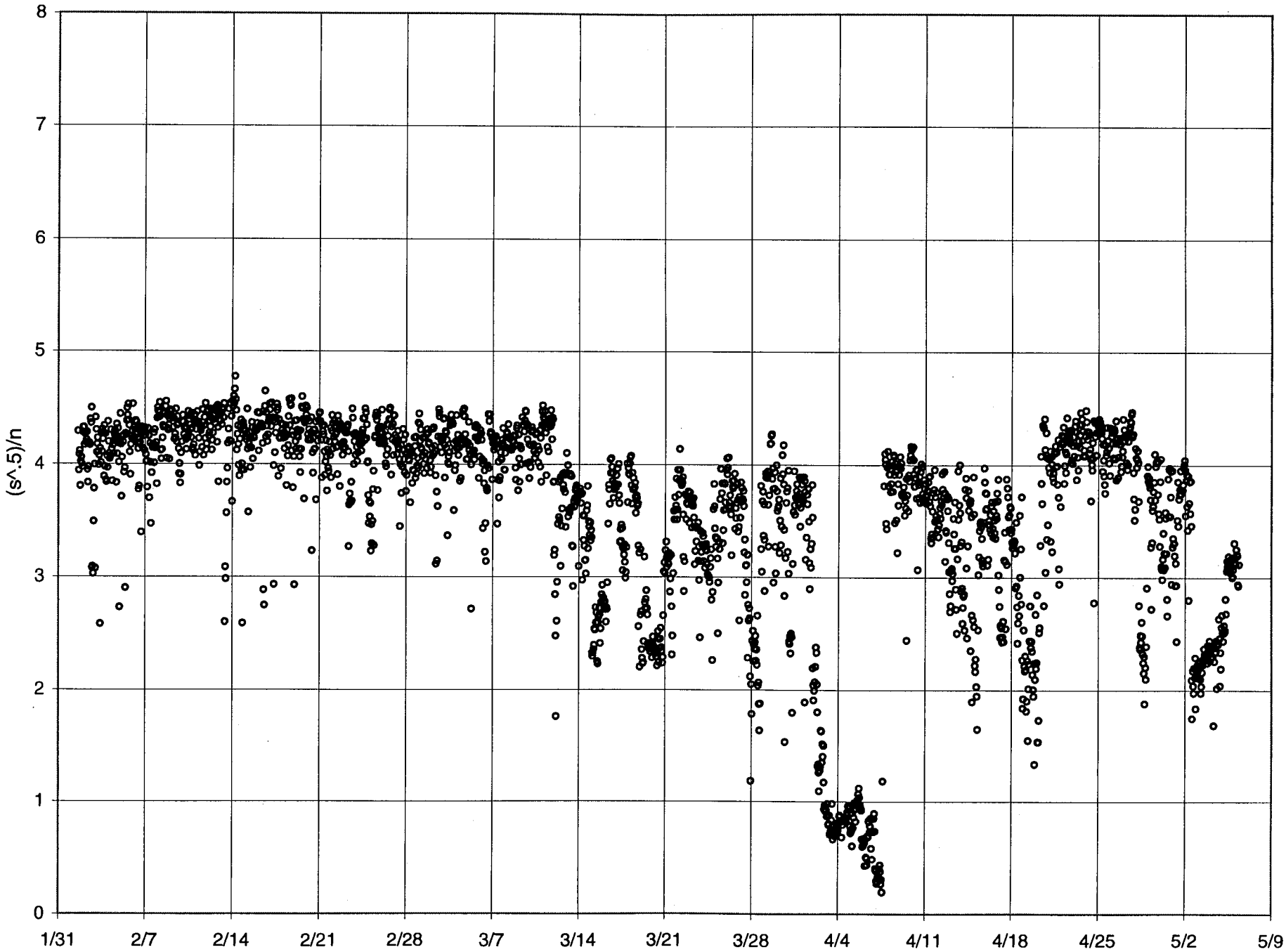


FIGURE 3-15. √s/n VALUES - BATTLEBEND METER
(2/1/99 to 5/6/99)

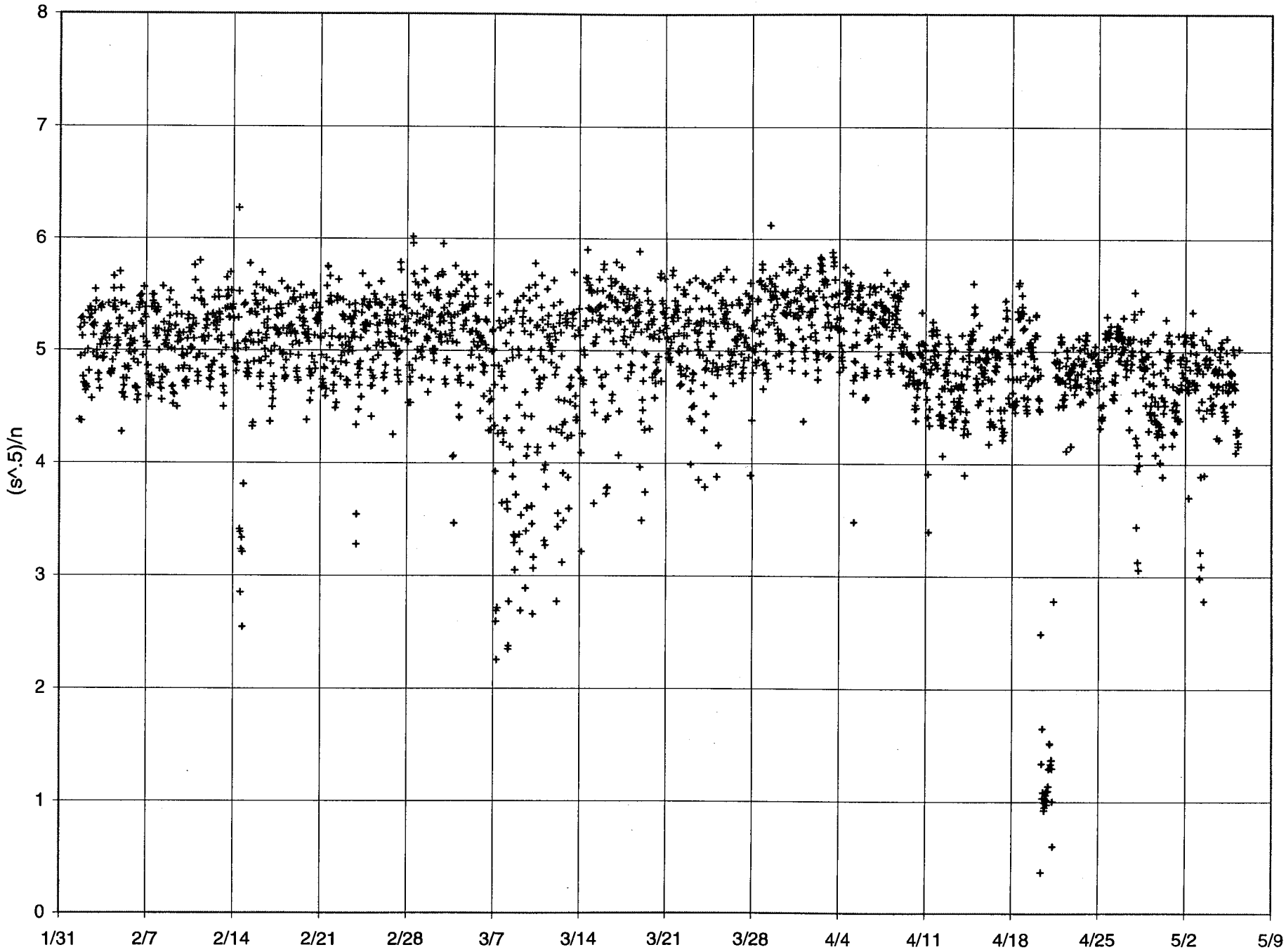
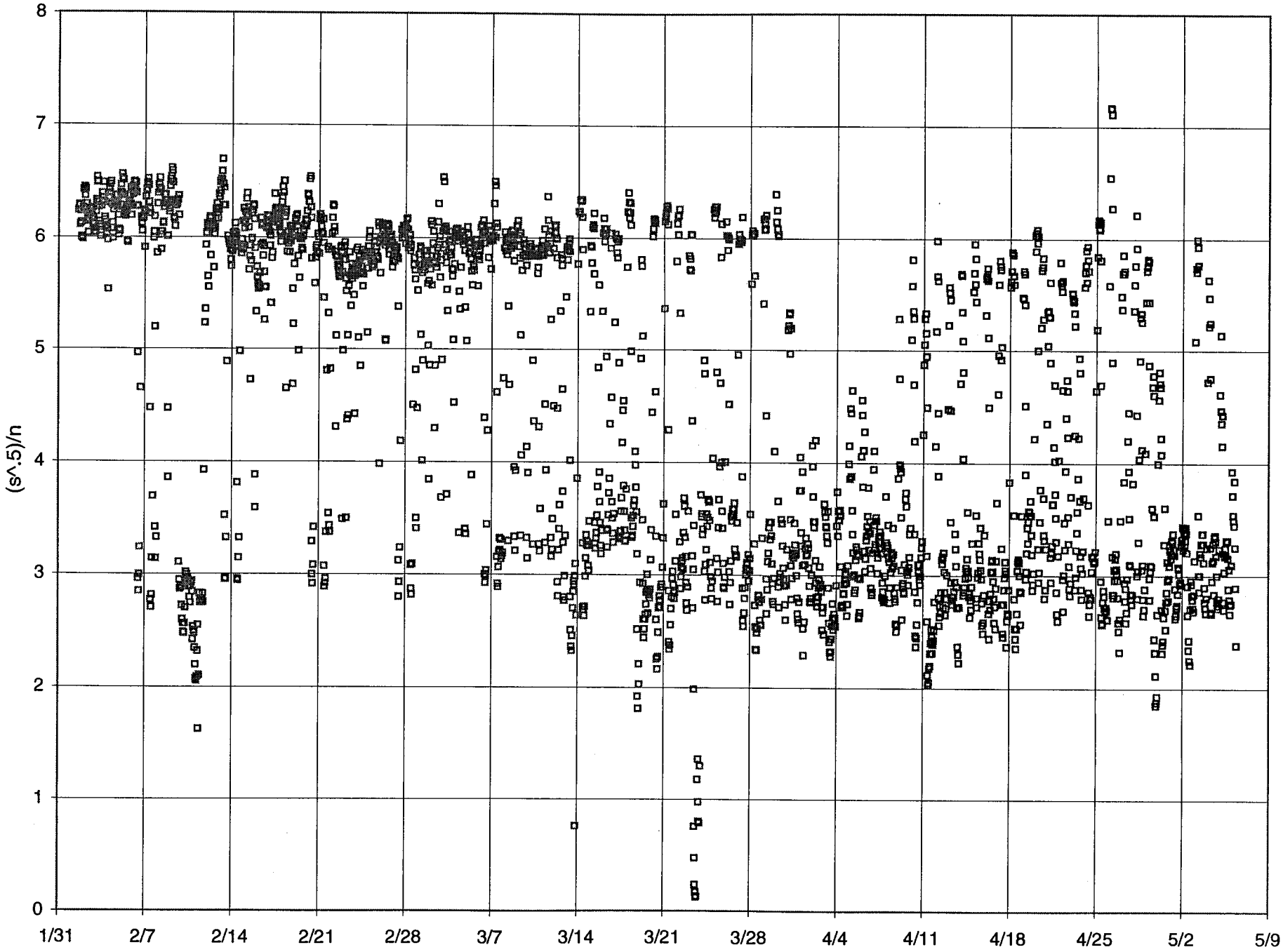


FIGURE 3-16. √s/n VALUES - CREEKWOOD METER
(2/1/99 to 5/6/99)



4.0 DESIGN FLOWS

Design flows are used for two purposes in this study, namely, (1) to size the relief interceptor for projected Year 2040 peak flows, and (2) to evaluate piping improvements ("cut-overs") needed to divert flows to the relief interceptor from existing lateral mains that connect to the 36-/42-inch main. There are two categories for the service areas that generate the design flows. The "Upstream Area" consists of all land upstream of the existing 36-/42-inch main that generates flow that will impact the relief interceptor. In addition to the Williamson Creek watershed, the Upstream Area includes portions of the Barton Creek basin with existing or future pumpover of flows to the Williamson Creek basin. The other service area category consists of the subbasins for 24 lateral sewers that connect at 21 manholes along the existing 36-/42-inch main. The subbasin flows affect the cut-over requirements, and add to the Upstream Area design flow to produce the cumulative flows for sizing the relief interceptor.

In past years there have been discussions among various groups about the possibility of extending City of Austin wastewater service to Lost Creek Municipal Utility District (MUD) via the Williamson Creek interceptor system. Lost Creek MUD is located between Barton Creek and Loop 360 in the Barton Creek basin. It is currently almost fully developed. The MUD's wastewater treatment plant provides effluent disposal by means of golf course irrigation. Pumping the MUD's wastewater to the City of Austin collection system has been considered as a way to eliminate the possibility that golf course irrigation could degrade water quality in Barton Creek. Final resolution of this issue by all parties concerned is not likely in the near future. Therefore, the design flows for the Williamson Creek relief interceptor include an alternative with service to Lost Creek MUD. As shown in the following sections, service to Lost Creek MUD at the extreme upstream end of the collection system would have a very small impact on design flows for the relief interceptor at the lower end of the system.

4.1 METHODOLOGY

Design flows for sizing the relief interceptor are based on data developed by the W&WW Systems Planning group for the Wastewater Collection System Long-Range Planning Guide (W&WW Utility, May 1994). W&WW updated the projections for this study. This data is based on projected development in the service area for year 2040.

The W&WW Systems Planning group developed sewer models for the wastewater collection system using Node Service Areas (NSA) as the basic unit for generating design flows. Each NSA represents one or more actual collection system subbasins, although the NSA boundaries were not established at the lot-line level of detail for the purposes of the Utility's collection system modeling. There are a total of 55 NSA's in the Williamson Creek watershed. There are 10 NSA's in the Barton Creek watershed to account for pump-over's to the Williamson Creek system. The Upstream Area for this study consists of

47 NSA's. The subbasins for the laterals in the project area are contained in 17 NSA's. One NSA is downstream of the project.

Each of the Utility's Node Service Areas is made up of smaller parcels that are generated by intersecting the NSA boundary with the city's Traffic Serial Zones (TSZ). Population, employment and acreage estimates are quantified for each NSA/TSZ parcel as follows. Each TSZ has population and employment projections by decade out to year 2040 which are assumed to have uniform areal distribution in the TSZ. The population and employment of each parcel is equal to its area times the population and employment densities of its TSZ. The NSA total population and employment is the sum of the population and employment of each TSZ parcel in the NSA. Table 4-1 summarizes the Node Service Area data used in this analysis. The City's projected total population and employment in the Upstream Area for year 2040 are 87,628 residents and 39,649 employment. The scenario discussed above regarding Lost Creek MUD adds 3,770 residents and 1,169 employment. The subbasins within the project area add 33,315 residents and 13,699 employment for totals at the downstream end of the project of 124,713 residents and 54,517 employment.

In order to be consistent in using the Utility's projected Yr. 2040 data for both the Upstream Area and the lateral subbasins, the population, employment and acreage for the 17 NSA's in the project area were disaggregated to the 21 subbasins contributing flow to the existing 36-/42-inch main. Each lateral subbasin was assigned to the appropriate NSA, and the NSA data was proportioned among the subbasins on the basis of subbasin area. The subbasin boundaries are shown in Figure 2-2, Project Area Map, discussed in Section 2.2. This disaggregation of year 2040 NSA parameters to subbasins is shown in Table 4-2.

4.2 YEAR 2040 PROJECTED PEAK FLOWS

For design purposes the peak flow consists of the peak wastewater flow plus the peak extraneous inflow/infiltration (I/I) flows. Wastewater flows are based on the projected population and employment numbers and unit flow data derived by W&WW from treatment plant records for average dry weather flows. The unit flows for the South Austin Regional WWTP service area, which includes the Williamson Creek basin, are 84 gal/day per resident and 42 gal/day per employee. An allowance of 1,500 gal/day per acre was provided for the I/I component.

The average wastewater flow is cumulative as lateral subbasins make contributions to the interceptor. A peaking factor is applied to the cumulative average wastewater flows to derive the peak wastewater flows. The peaking factor formula is $(18+\sqrt{0.0144*Q_{avg}})/(4+\sqrt{0.0144*Q_{avg}})$. This equation produces a peaking factor which decreases as the average flow increases, so that the peak wastewater flow does not increase as much as the average wastewater flow.

TABLE 4-1. NODE SERVICE AREA DATA

	NODE SERVICE AREA	POP2040	EMP2040	TOTAL GPM 2040	ACRES	
	WI100040 Total	1,466	429	98	125.7	
	WI100041 Total	2,081	439	134	183.2	
	WI100043 Total	2,879	1,311	206	304.1	
	WI100047 Total	3,447	603	219	349.5	
	WI100051 Total	1,677	1,802	150	350.1	
	WI100074 Total	1,512	486	102	330.6	
	WI100103 Total	1,018	228	66	193.7	
	WI100112 Total	2,066	1,634	168	334.9	
	WI100117 Total	887	883	77	207.6	
	WI100124 Total	306	226	24	91.0	
	WI100136 Total	906	478	67	236.5	
	WI100144 Total	1,972	893	141	424.2	
	WI10025N Total	831	777	71	369.6	
	WI10037N Total	6,252	2,528	438	1,891.9	
	WI10077N Total	1,711	161	104	213.2	
	WI10081N Total	1,199	587	87	300.6	
	WI10090N Total	1,198	74	72	134.9	
	WI122008 Total	3,972	1,836	285	459.7	
	WI126012 Total	955	257	63	101.6	
	WI126023 Total	2,817	805	188	350.6	
	WI126032 Total	3,145	421	196	365.9	
	WI12633N Total	710	101	44	84.2	
	WI136011 Total	2,951	567	189	361.8	
	WI136027 Total	2,407	227	147	271.8	
	WI143016 Total	3,726	204	223	409.8	
	WI147003 Total	2,127	152	129	224.0	
	WI149005 Total	3,382	282	206	381.9	
	WI151016 Total	2,221	506	144	300.2	
	WI15115N Total	10,611	4,454	749	2,506.0	
	WI152015 Total	2,926	813	194	421.6	
	WI160015 Total	1,604	2,958	180	799.6	
	WI1MOTOR Total	111	239	13	52.0	
	WI1SS024 Total	1,558	2,596	167	580.2	
	WI1VALVI Total	923	439	67	212.4	
	WI300020 Total	365	439	34	126.0	
	WI30018N Total	874	1,584	97	388.2	
	WI30019N Total	678	546	55	166.0	
Williamson basin - upstream of 36"		79,471	32,966	5,597	14,604.4	**1 Total**
36"	**WI100025 Total**	1,471	487	100	203.5	
36"	**WI100027 Total**	1,440	1,346	123	233.1	
36"	**WI100033 Total**	1,123	342	75	113.2	
36"	**WI119020 Total**	2,734	516	175	222.7	
36"	**WI119024 Total**	2,065	414	133	200.4	
36" section		8,833	3,104	606	972.9	**2 Total**
42"	**WI100005 Total**	2,258	403	144	286.6	
42"	**WI100013 Total**	847	71	51	64.7	
42"	**WI100020 Total**	2,977	2,986	261	531.6	
42"	**WI10010N Total**	451	163	31	57.3	
42"	**WI10021N Total**	725	533	58	214.1	
42"	**WI101008 Total**	4,967	1,628	337	689.2	
42"	**WI10101A Total**	487	439	41	215.5	
42"	**WI10107N Total**	2,310	1,413	176	389.7	
42"	**WI105007 Total**	4,616	1,172	303	514.2	
42"	**WI107006 Total**	1,130	365	77	143.0	
42"	**WI10713N Total**	121	124	11	30.6	
42"	**WI109015 Total**	3,593	1,298	247	488.6	
42" section		24,482	10,595	1,737	3,625.0	**3 Total**
	WI80001N Total	2,704	3,243	252	1,249.7	
Williamson basin - downstream		2,704	3,243	252	1,249.7	**4 Total**
	BA3CHAPN Total	167	41	11	153.9	
	BA3MOPXN Total	1,687	1,542	143	589.7	
	BA3PATTN Total	409	666	43	217.9	
	BA3PATXN Total	317	116	22	226.8	
	BABAVIEW Total	884	2,424	122	234.7	
	BAGAINAN Total	804	194	53	761.1	
	BAGAINBN Total	1,025	310	69	326.1	
	BAMOP290 Total	288	612	35	185.5	
	F2BA3TRACO Total	2,575	777	173	836.1	
Barton basin pumpover		8,157	6,683	671	3,531.7	**5 Total**
	BA3LCMUN Total	3,770	1,169	254	812.0	
Lost Creek MUD		3,770	1,169	254	812.0	**8 Total**
	Grand Total	127,417	57,759	9,117	24,795.8	

TABLE 4-2. DISAGGREGATION OF NODE SERVICE AREA DATA TO SEWER SUBBASINS

NSA	0025	0027	0033	9020	9024	0005	0013	0020	010N	021N	1008	101A	107N	5007	7006	713N	9015	Totals
Popn	1471	1440	1123	2734	2065	2,258	847	2,977	451	725	4,967	487	2,310	4,616	1,130	121	3,593	33,315
Empl	487	1346	342	516	414	403	71	2,986	163	533	1,628	439	1,413	1,172	365	124	1,298	13,699
AvgFlow	100	123	75	175	133	144	51	261	31	58	337	41	176	303	77	11	247	2,343
Area	203	233	113	223	200	287	65	532	57	214	689	215	390	514	143	31	489	4,598
Subbasin areas from PBSJ base map allocated to NodeServiceAreas on COA drawing (acres)																		
I I			113.2															113
HH				222.7	200.4													423
FF		51.7																52
EE		51.7																52
BB		129.9																130
Z	163.7									42.7								206
W								223.0		77.0								300
V								164.6		48.7								213
U								171.3										171
S										30.7				27.7				58
R																	567.3	567
P															13.9			14
N															66.7			67
L							36.4											36
K							9.8		77.2							43.7		131
I														401.5				401
E						272.4												272
D											18.0							18
A						8.3					430.4	171.3	389.7					1,000
Total	164	233	113	223	200	281	46	559	77	199	448	171	390	429	81	44	567	4,225
NSA Average Flow distributed to subbasins proportional to PBSJ subbasin areas (gpm)																		
I I			75															75
HH				175	133													307
FF		27																27
EE		27																27
BB		69																69
Z	100									12								112
W								104		22								126
V								77		14								91
U								80										80
S										9				20				28
R																	247	247
P															13			13
N															63			63
L							41											41
K							11		31							11		53
I														284				284
E						139												139
D											14							14
A						4					324	41	176					545
Total	100	123	75	175	133	144	51	261	31	58	337	41	176	303	77	11	247	2,343
NSA Acreage distributed to subbasins proportional to PBSJ subbasin areas (acres)																		
I I			113															113
HH				223	200													423
FF		52																52
EE		52																52
BB		130																130
Z	203									46								249
W								212		83								295
V								157		52								209
U								163										163
S										33				33				66
R																	489	489
P															25			25
N															118			118
L							51											51
K							14		57							31		102
I														481				481
E						278												278
D											28							28
A						8					662	215	390					1,275
Total	203	233	113	223	200	287	65	532	57	214	689	215	390	514	143	31	489	4,598

The extraneous flow component of the design flow is the 1,500 gal/day/acre I/I unit flow rate applied to the contributing area for I/I determined by the Utility for each node service area. The I/I component is cumulative with no depeaking factor. The design flow assumes that the peak wastewater component and the peak I/I component occur simultaneously.

The unit flow rates and peaking factors are applied to the data in Tables 4-1 and 4-2 to derive the design flows presented in Table 4-3 on a segment by segment basis for the 36-/42-inch interceptor, with and without Lost Creek MUD. The design flow at the upper end of the project is 31,660 gpm without Lost Creek MUD design flow and 32,929 gpm with the MUD. At the lower end the design flows are 40,257 gpm without the MUD and 41,505 gpm with the MUD. The design flow for just the MUD's Node Service Area is 1,700 gpm, but the wastewater peaking factor that decreases with increasing average flow reduces the impact of the MUD's flow to about 1,250 gpm, or 3% to 4% of the design flow, for the downstream 36-/42-inch interceptor.

These design flows for the existing interceptor are not directly applicable to the relief improvements. Various alternatives for the project improvements leave portions of the existing interceptor in service. Design flows for the relief main alternatives and associated cut-over mains are presented in Section 8.0. However, those design flows all use the population, average flow, and acreage data presented in this section to derive the design flows for the existing interceptor.

TABLE 4-3. DESIGN FLOWS FOR THE LOWER WILLIAMSON CREEK INTERCEPTOR

SCENARIO 'A' - WILLIAMSON CREEK BASIN PLUS BARTON CREEK PUMPOVER

SubBasins (Segment Upstream Manhole)	Added Avg Flow (gpm)	Cumul Avg Flow (gpm)	Peaking Factor	Peak Dry Weathr Flow	Added Srvce Area (ac)	I/I criteria (gpd/ac)	Added I/I Flow (gpm)	Cumul I/I Flow	Trib Peak WetW Flow	Cum Peak WetW Flow
LostCk MUD pumpover	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
BartonCk pumpover	671	671	2.97	1,992	3,532	1500	3,679	3,679	5,671	5,671
KK	5,597	6,268	2.04	12,768	14,604	1500	15,213	18,892	26,848	31,660
I I	75	6,343	2.03	12,894	113	1500	118	19,010	403	31,904
HH	307	6,651	2.02	13,404	423	1500	441	19,451	1,452	32,855
FF	27	6,678	2.01	13,450	52	1500	54	19,504	164	32,954
EE	27	6,705	2.01	13,495	52	1500	54	19,558	164	33,053
BB	69	6,774	2.01	13,608	130	1500	135	19,693	396	33,301
Z	112	6,886	2.00	13,793	249	1500	260	19,953	671	33,746
W	126	7,013	2.00	14,001	295	1500	307	20,260	764	34,261
V	91	7,104	1.99	14,150	209	1500	218	20,478	556	34,628
U	80	7,184	1.99	14,281	163	1500	170	20,648	470	34,928
S	28	7,212	1.99	14,327	66	1500	69	20,717	183	35,044
R	247	7,459	1.97	14,730	489	1500	509	21,226	1,345	35,955
P	13	7,473	1.97	14,751	25	1500	26	21,251	80	36,002
N	63	7,536	1.97	14,854	118	1500	123	21,374	366	36,228
L	41	7,577	1.97	14,920	51	1500	53	21,428	213	36,347
K	53	7,629	1.97	15,005	102	1500	106	21,533	310	36,538
I	284	7,913	1.95	15,462	481	1500	501	22,035	1,445	37,497
E	139	8,052	1.95	15,686	278	1500	290	22,324	789	38,010
D	14	8,066	1.95	15,708	28	1500	29	22,353	85	38,061
A	545	8,611	1.92	16,576	1,275	1500	1,328	23,681	2,995	40,257

SCENARIO 'B' = SCENARIO 'A' PLUS LOST CREEK M.U.D. PUMPOVER

SubBasins (Segment Upstream Manhole)	Added Avg Flow (gpm)	Cumul Avg Flow (gpm)	Peaking Factor	Peak Dry Weathr Flow	Added Srvce Area (ac)	I/I criteria (gpd/ac)	Added I/I Flow (gpm)	Cumul I/I Flow	Trib Peak WetW Flow	Cum Peak WetW Flow
LostCk MUD pumpover	254	254	3.37	855	812	1500	846	846	1,701	1,701
BartonCk pumpover	671	925	2.83	2,617	3,532	1500	3,679	4,525	5,671	7,142
KK	5,597	6,522	2.02	13,191	14,604	1500	15,213	19,738	26,848	32,929
I I	75	6,597	2.02	13,316	113	1500	118	19,856	403	33,172
HH	307	6,905	2.00	13,823	423	1500	441	20,296	1,452	34,120
FF	27	6,932	2.00	13,868	52	1500	54	20,350	164	34,218
EE	27	6,959	2.00	13,913	52	1500	54	20,404	164	34,317
BB	69	7,028	2.00	14,026	130	1500	135	20,539	396	34,565
Z	112	7,140	1.99	14,210	249	1500	260	20,799	671	35,009
W	126	7,267	1.98	14,416	295	1500	307	21,106	764	35,522
V	91	7,358	1.98	14,564	209	1500	218	21,324	556	35,888
U	80	7,438	1.98	14,694	163	1500	170	21,493	470	36,188
S	28	7,466	1.97	14,740	66	1500	69	21,562	183	36,303
R	247	7,713	1.96	15,141	489	1500	509	22,071	1,345	37,212
P	13	7,727	1.96	15,162	25	1500	26	22,097	80	37,259
N	63	7,790	1.96	15,264	118	1500	123	22,220	366	37,485
L	41	7,831	1.96	15,330	51	1500	53	22,273	213	37,603
K	53	7,883	1.96	15,414	102	1500	106	22,379	310	37,794
I	284	8,167	1.94	15,869	481	1500	501	22,880	1,445	38,750
E	139	8,306	1.94	16,092	278	1500	290	23,170	789	39,262
D	14	8,320	1.94	16,113	28	1500	29	23,199	85	39,312
A	545	8,865	1.92	16,978	1,275	1500	1,328	24,527	2,995	41,505

÷0.80
41,161

51,881

5.0 ENVIRONMENTAL SETTING OF THE PROJECT AREA

5.1 PHYSIOGRAPHY, SOILS, GEOLOGY

The study area is mapped as lying within the Blackland Prairie physiographic area of Texas (BEG, 1970, 1977), also known as the Black Prairie. The Black and Grand prairies together form a southwest trending belt in northeast Texas, averaging about 65 miles wide and 275 miles long. The Black Prairie has a gently undulating surface characterized by deep, waxy, fertile soil. The soil is derived from the decomposition of Cretaceous marl that dips beneath the Tertiary rocks of the East Texas Coastal Plain. Although there is no marked topographic change at the boundary between the Cretaceous and Tertiary strata, the vegetation and soils are distinctly different, with the East Texas Timbered belt ending abruptly as the lime deficient red clay and sandy soils give way to the more alkaline thick Black Prairie soils (Maxwell, 1970).

Soils of the entire study area are mainly deep, gently sloping soils of the Blackland Prairies, with soils of the Austin-Eddy Association predominating to the northwest of IH 35, and soils of the Houston Black-Heiden Association becoming more prominent to the Southeast of IH 35. Soils of the Austin-Eddy Association are characterized as moderately deep and shallow, calcareous, clayey and loamy soils overlying chalk. Soils of the Houston Black-Heiden Association are characterized as deep, nearly level and gently sloping, calcareous, clayey soils overlying marl (SCS, 1974).

The Geologic Atlas of Texas maps the study area as occupying four different geological mapping types. The northwestern, or upstream portions of the study area are predominantly Upper Cretaceous chalk and marl formations of the Austin Chalk. In the vicinity of Interstate Highway 35 (IH 35), the drainage enters an area of Fluviatile Terraces deposited in the Pleistocene. These deposits typically contain three or more levels, with gravel, silt, and clay in various proportions. Gravels tend to be more prominent in the older, higher terraces. A short distance downstream from IH 35, Williamson Creek becomes a boundary marker between two geologic types, with the Fluviatile Terrace deposits on the north side of the creek. From this point to the downstream, or southeastern end of the study area, the majority of the remaining study area is mapped as the Ozan formation, which was formed in the Upper Cretaceous. This formation is also locally known also as the Sprinkle Formation or Lower Taylor Marl. Another type of Fluviatile terrace deposit from the Pleistocene, typified as high gravel deposits, is most dominant within the study area in the vicinity of IH 35 (BEG, 1974).

A preliminary geotechnical investigation was conducted by team member Terra-Mar, Inc., (TMI) to obtain information required for engineering evaluations. The TMI report on the preliminary geotechnical investigation, which is included as Appendix A, presents more detailed information on the geological setting of the project area.

5.2 WATER RESOURCES

The study area is drained by Williamson Creek and its tributaries. Approximately 1.3 miles northeast of the study area, Williamson Creek flows into Onion Creek, which then flows to the Colorado River. Aquatic habitats in the portion of Williamson Creek within the study area are highly impacted by urban runoff, as evidenced by the large amount of water borne litter and detritus throughout the floodplain.

5.3 ECOLOGY

5.3.1 Vegetation

The study area is located in an area of intergrade between the Blackland Prairies vegetational area and the Edwards Plateau vegetational area, and shares characteristics of both areas. It is also very close to the southern fringes of the Cross Timbers and Prairies vegetational area and is influenced by that vegetative regime. The Blackland Prairies are a rolling and well-dissected prairie which represents the southern extension of the true prairie that occurs from Texas to Canada. This once luxuriant tallgrass prairie was originally dominated by little bluestem (*Schizachyrium scoparium*), big bluestem (*Andropogon gerardii*), yellow indiangrass (*Sorghastrum nutans*), tall dropseed (*Sporobolis asper* var. *asper*), and Silveus dropseed (*S. silveanus*). Oak (*Quercus* spp), elm (*Ulmus* spp), eastern cottonwood (*Populus deltoides*), and pecan (*Carya illinoinensis*) are common along drainages. Modern grazing practices have resulted in the increase of formerly minor grass species such as sideoats grama (*Bouteloua curtipendula*), hairy grama (*B. hirsuta*), Mead's sedge (*Carex meadii*), Texas wintergrass (*Stipa leucotricha*), and buffalograss (*Buchloe dactyloides*). More extreme grazing pressure has resulted in the invasion of former grasslands by honey mesquite (*Prosopis glandulosa*), huisache (*Acacia farnesiana*), and elms (Hatch et al., 1990).

Although the eastern and southern boundaries of the Edwards Plateau are officially delimited by the Balcones Escarpment (Hatch et al., 1990), several species typical of the Edwards Plateau extend well beyond the physical boundaries of the Edwards Plateau, especially along high banks, bluffs, and ridges. The best examples of this within the study area are Ashe juniper (*Juniperus ashei*), which is abundant in well drained upland areas, and elbowbush (*Forestiera pubescens*) and kidneywood (*Eysenhardtia texana*).

Floodplain and riparian tree species observed within the study area include sugar hackberry (*Celtis laevigata*), American elm (*Ulmus americana*), green ash (*Fraxinus pennsylvanica*), boxelder (*Acer negundo*), cedar elm (*Ulmus crassifolia*), pecan, osage orange (*Maclura pomifera*), gum bumelia (*Bumelia lanuginosa*), and Chinese tallow (*Sapium sebiferum*), with occasional black willow (*Salix nigra*) and American sycamore (*Platanus occidentalis*). Many areas supported a dense understory of waxleaf ligustrum (*Ligustrum quihoui*), with sawtooth greenbriar (*Smilax bona-nox*), peppervine (*Ampelopsis arborea*), and poison ivy (*Toxicodendron radicans*), while other areas have been maintained by frequent mowing to

produce grassy park-like areas under the canopy. Other shrub species include possum-haw (*Ilex decidua*) and Roosevelt weed (*Baccharis neglecta*). Herbaceous species within the riparian zone include Johnsongrass (*Sorghum halepense*), broadleaf woodoats (*Chasmanthium latifolium*), giant ragweed (*Ambrosia trifida*), frostweed (*Verbesina virginica)*, hedgeparsley *(Torilis* sp), onions (*Allium* sp), henbit (*Lamium amplexicaule*), bedstraw (*Galium* sp), dock (*Rumex* sp), geranium (*Geranium* sp), wood sorrel (*Oxalis* sp), bermudagrass (*Cynodon dactylon*), Drummond's waxmallow (*Malaviscus arboreus*), prostrate lawnflower (*Calyptocarpus vialis*), spiderwort (*Tradescantia* sp), broadleaf woodoats (*Chasmanthium latifolium*), wildrye *(Elymus* sp), switchgrass (*Panicum virgatum*), and vetch (*Vicia* sp).

The upland woodlands were variously dominated by sugar hackberry, cedar elm, and Ashe juniper. Other species included plateau liveoak (*Quercus fusiformis*), honey mesquite, chinaberrry (*Melia azedarach*), retama (*Parkinsonia aculeata*), and Chinese tallow. Shrub species included elbowbush, kidneywood, sugar hackberry, cedar elm, and waxleaf ligustrum. Herbaceous species included Drummond's waxmallow, frostweed, bedstraw, hedgeparsley, and vetch. Some upland areas lacked a tree canopy , and were dominated by a mixture of small trees, shrubs, and herbaceous species.

5.3.2 Fish and Wildlife

The study area lies near the western edge of the Texan biotic province as defined by Blair (1950). This biotic province is a transitional area between the forested Austroriparian province to the east and the western provinces, which were, at least originally dominated by grasslands. There are no endemic vertebrate species here, and the most outstanding biogeographic phenomenon here is the interdigitation of forest and grassland associations (Blair, 1950). The fauna of the study area is heavily influenced by that of the Balconian biotic province, which bounds the Texan province closely to the west of the study area. Wildlife observed in the study area included whitetail deer (*Odocoileus virginianus*), eastern fox squirrel (*Sciurus niger*), common snapping turtle (*Chelydra serpentina serpentina*), blotched water snake (*Nerodia erythrogaster transversa*), great blue heron (*Ardea herodias*), and numerous passerine bird species typical of suburban habitats. Turkey vultures (*Cathartes aura*) were observed roosting in the riparian woodlands.

5.3.3 Endangered and Threatened Species

Thirteen species of animals of potential occurrence in Travis County are federally listed as endangered or threatened. No plants which are federally listed as endangered or threatened are known to occur in Travis County (FWS, 2000; TPWD, 2000).

Six of the endangered species are troglodytic invertebrates which are limited to certain cave systems of the Balcones Escarpment, and would not be expected to occur in the study area: the Bee Creek Cave harvestman (*Texella reddelli*), Bone Cave harvestman (*Texella reyesi*), Kretschmarr Cave mold beetle

(*Texamaurops reddelli*), tooth Cave pseudoscorpion (*Tartarocreagris texana*), Tooth Cave ground beetle (*Rhadine persephone*), and Tooth Cave spider (*Neoleptoneta (=Leptoneta) myopica*). Warton's cave spider (*Cicurina wartoni*) is a candidate for federal listing, meaning the U.S. Fish and Wildlife Service has enough substantial information on file to warrant listing. It currently is not a protected species, and is not likely to occur in the study area, because of the lack of suitable habitat.

Similarly, the Barton Springs salamander (*Eurycea sosorum*) is limited to aquatic habitats in the immediate vicinities of spring outflows of Barton Springs and would not be expected to occur in the study area.

The endangered whooping crane (*Grus americana*), least tern (*Sterna antillarum athalassos*), and the threatened piping plover (*Charadrius melodus*) are very rare migrants through the area (TAS, 1994). All of these species sometimes occur at small impoundments, and the piping plover and least tern have been observed at the Hornsby Bend sewage treatment ponds (Kutac and Caran, 1994). None of these species would be expected to occur in the study area due to the lack of suitable habitat.

The bald eagle (*Haliaeetus leucocephalus*) is currently federally listed as threatened (60 FR 36000), but has been proposed for delisting (64 FR 36453). It is an uncommon winter resident of this area, and can often be observed at Lake Buchanan, Lake Bastrop, Lake Somerville, and other perennial waterways (Kutac and Caran, 1994). One active nesting territory is known from Bastrop County, which produced one offspring in 1999 (Mitchell, 1999). The bald eagle would not be expected to occur in the study area due to the lack of large bodies of water, and the high level of human activity.

The black-capped vireo (*Vireo atricapillus*) and golden-cheeked warbler (*Dendroica chrysoparia*) are endangered birds which are uncommon nesting species (TAS, 1994) in western Travis County on the Balcones Escarpment. They would not be expected to occur in the study area, due to the lack of suitable habitat.

The mountain plover (*Charadrius montanus*) currently is not federally listed and has no special protection, but it has been proposed for listing as threatened. It is a rare to very rare migrant and winter resident in Travis County (TAS, 1994). It is typically observed in South Central Texas in croplands, range, and pastures (Kutac and Caran, 1994), and would not be expected to occur in the study area.

Six species of plants and nine species of animals of potential occurrence in Travis County are categorized as species of concern (SOCs), which are species for which there is some information showing evidence of vulnerability, but not enough evidence to support federal listing at this time (FWS, 2000). These species currently receive no special protection, but could be listed in the future. The plant SOC species are: the big red sage (*Salvia penstemonoides*), bracted twistflower (*Streptanthus bracteatus*), canyon mock orange

(*Philadelphus ernestii*), Correl's false dragon-head (*Physostegia correllii*), Glass Mountain coral-root (*Hexalectris nitida*), and Texabama (Fort Hood) croton (*Croton alabamensis* var. *texensis*). The animal SOC species are: the Balcones cave amphipod (*Stygobromus balconis*), bifurcated cave amphipod (*Stygobromus bifurcatus*), Jollyville Plateau salamander (*Eurycea* sp), blue sucker (*Cycleptus elongatus*), Texas horned lizard (*Phrynosoma cornutum*), Texas garter snake (*Thamnophis sirtalis annectans*), white-faced ibis (*Plegadis chihi*), loggerhead shrike (*Lanius ludovicianus*), and Texas olive sparrow (*Arremonops rufivirgatus rufivirgatus*).

Four of the federal SOC species are also state listed as threatened: the blue sucker, Texas horned lizard, Texas garter snake, and white-faced ibis. The arctic peregrine falcon (*Falco peregrinus tundrius*), and the American peregrine falcon (*Falco peregrinus anatum*) had formerly been federally listed as threatened and endangered, but have recently been delisted. They are still listed by the state of Texas, the arctic peregrine falcon listed as threatened, and the American peregrine falcon listed as threatened (TPWD, 2000).

The blue sucker is typically limited to the largest rivers and lower parts of their tributaries (Lee et al., 1980), and is of low potential occurrence in the study area.

The Texas horned lizard was historically found throughout Texas in areas with open, flat terrain with scattered vegetation and sandy or loamy soils. Over the past 25 years, it has almost vanished from the eastern half of the state, but still maintains relatively stable numbers in West Texas. It has been recorded in Travis County in the past (Dixon, 2000), although its presence within the study area is currently unlikely.

The Texas garter snake is found in a wide range of habitats, though nearly always in the vicinity of moisture, whether along the margins of streams, rivers, ponds, lakes, or marshes or around damp soil some distance from such bodies of water. Although even drainage ditches and irrigation canals relatively free of plant life sometimes attract this moisture dependent serpent, its most typical haunts include a ground cover of grass, weeds, or other brushy streamside vegetation. It is known from Travis County (Werler and Dixon, 2000), and could occur in the project area. No significant impacts to this species are expected from this project.

The white-faced ibis is uncommon in Travis County in July and August, and rare at other times of year (TAS, 1994). It could occur in the study area as a transient in streamside habitats, but no impacts to this species are expected.

The arctic peregrine falcon and American peregrine falcon occur as rare migrants or transients in Travis County (TAS, 1994). These transient peregrines are most often associated with large

concentrations of birds suitable as prey, such as waterfowl and shorebirds. They are unlikely to linger in the study area due to the lack of suitable habitat.

5.3.4 Critical Environmental Features

Williamson Creek within the study area was searched for Critical Environmental Features (CEFs) such as bluffs, springs, canyon rimrocks, caves, sinkholes, and wetlands; as defined in the City of Austin Environmental Criteria Manual (ECM) (COA, 1998). The only CEFs observed were several bluffs, some of which had associated rimrocks. Bluffs were mapped at six locations: on the north side of the creek, between Heartwood Drive and Goliad Lane; on the south side of the creek, immediately west of Congress Avenue; on the east side of the creek along Manchester Circle and Suburban Drive; on the south side of the creek, across from Battle Bend Boulevard and Chihuahua Trail; on the southeast side of the creek to the north of Stassney Lane, near Conestoga Trail; and on the southwest side of the creek, below the apartments at Sneed Cove and Nelms Drive. These bluffs should not be impacted by the proposed project, as no work will occur on these bluffs and they are too steep to be used for equipment passage. These bluffs and rimrocks would be out of the zone of impacts for this project.

No caves, sinkholes, or springs were observed. No areas were observed to meet wetland criteria outside of the incised channel banks of Williamson Creek. Several washover areas were observed, which were generally associated with bends in the creek channel. The substrate in these areas was generally composed of well drained depositional gravels and sands. These washover areas supported disturbance type plant communities dominated by predominately facultative plant species such as Johnsongrass, giant ragweed, hedge parsley, poison ivy, dewberry (*Rubus trivialis*), and saplings of honey mesquite, huisache, and green ash. No hydrophytic plant communities were observed outside of the incised stream channel.

5.3.5 Environmentally Sensitive Areas

Williamson Creek within the study area was searched for environmentally sensitive areas (ESAs). ESAs are defined as areas of high priority for preservation and special consideration. ESAs include habitat for the golden-cheeked warbler and black-capped vireo, and high quality riparian and upland woodlands (COA, 1998). ESAs may also include, but are not limited to rare, threatened or endangered biological communities such as priority grasslands and priority canyons, priority aquatic habitat, and unique geological features. No suitable habitat for the golden-cheeked warbler or the black-capped vireo was observed. No unique geological features, priority grasslands, or priority canyons were observed. Aquatic habitats within the study area are already heavily impacted by urban runoff. The study area contains both riparian and upland woodlands, although none of unusually high quality. The riparian woodlands are typically either highly modified by clearing of understory and mowing, or contain a dense understory of primarily exotic species such as waxleaf ligustrum. Both riparian and upland woodlands along some of these

sections of Williamson Creek have reportedly been designated as "priority woodlands" (H&C, 1996), but inquiries to the City of Austin have not clarified this. The glossary of the ECM (COA, 1999) does not address priority woodlands.

The majority of this portion of Williamson Creek is designated as parkland, forming the Williamson Creek Greenbelt. As such, it is subject to regulations concerning construction in parks (COA, 1999, Section 5). The ECM requires tree surveys and tree evaluations within and adjacent to all construction and access easements. The ECM prescribes guidelines pertaining to route selection, site clearing, erosion control, and site cleanup. Areas of construction within the creek are subject to further regulations concerning discharge of pumped water, in-channel erosion and siltation controls, restoration of creek banks, excavation in creek channels, and trench caps in creek channels. As a greenbelt, these sections of Williamson Creek will require restoration as detailed in Appendix L of the ECM. Some floodplain areas not within designated greenbelt parklands may qualify as protected riparian areas if they meet the criteria of species composition, canopy coverage, and canopy extent as defined in the ECM glossary. This protection is also extended to wetlands other than springs, and permanent natural pools in perennial or intermittent waterways (COA, 1999).

5.4 CULTURAL RESOURCES

Cultural resource surveys were conducted by Hicks & Company, Inc. upon a portion of the study area. The portion surveyed is a corridor approximately 200 ft wide centered on Williamson Creek. This corridor begins at a point approximately 3000 ft west of IH 35, near the intersection of Suburban Drive and Battlebend Boulevard, and extends approximately 14,400 ft downstream to a point just east of Pleasant Valley Road. An investigation of files at the Texas Historical Commission (THC) and the Texas Archeological Research Laboratory (TARL) was undertaken to identify extant prehistoric and historic archeological sites and determine the locations of previous surveys. In addition, records were searched for properties listed on the National Register of Historic Places (NRHP), State Archeological Landmarks (SALs), and Texas Historical Markers. The 1936 and 1958 Travis County General Highway maps were consulted to locate any additional historical structures. Field surveys were then performed on the entire archeological project area for that project (H&C, 1998).

Results of the background research indicate that no NRHP properties, SALs, or Texas Historical Markers exist within the portion of the study area examined. Archival research revealed that two prehistoric sites (41TV409 and 41TV1666) and one probable historical site (41TV1713) were located within the archeological project area. The two prehistoric sites were located on the upper terrace to the north of the creek, one in the area of the Pleasant Valley bridge, and the other approximately 2400 ft west of Pleasant Valley Road. Both of these sites appear to have been destroyed by construction activities for the bridge and a flood control levee. The status of the probable historical site, located on the east side of Pleasant Valley

Road on a bluff overlooking Williamson Creek from the south, was not determined, due to lack of access (H&C, 1998).

PBS&J performed similar records searches at TARL and THC for the remainder of the study area not covered by the H&C report, and searched for new records in the area previously covered. These searches revealed two prehistoric sites (41TV679 and 41TV684) near a section proposed to be tunneled for Alternative C. This area is not expected to have any surface impacts. Site 41TV684 consisted of several piles of debitage, and site 41TV679 consisted of chipped flint and a small number of burned rocks. Both sites were in the vicinity of the Williamson Creek Cemetery on the north side of Stassney Lane, and the west side of IH 35.

Williamson Creek within the study area appears to be a highly dynamic environment which has experienced significant episodes of both deposition and retrenching during modern times. Recent debris is buried at various depths in the cut banks. The only non-modern cultural materials seen during Hicks & Company's field visits were historic ceramic sherds located in point bar deposits and one ceramic sherd and one chert flake in flood deposited gravels eroding out of a cut bank. The highly dynamic nature of Williamson Creek precludes the presence of intact cultural deposits within the floodplain. Due to the magnitude of flooding experienced by the creek, long term occupation of the creek bottom would be unlikely. In addition, any cultural occupation deposits in the creek bottom would most likely be either washed away during a flood event or eventually mixed up during repeated episodes of deposition and retrenching as Williamson Creek meanders across its floodplain (H&C, 1998).

Prehistoric and historic settlements would be expected to occur only on the upper terraces outside of the floodplain. Within the areas surveyed by Hicks & Company only a few areas along the upper terrace have not been disturbed by modern development. One such area exists which would warrant subsurface investigations if the proposed project were to impact it. This area lies along the section of Williamson Creek that runs between IH 35 and East Stassney Lane. On the south and west side of a 90-degree bend in the creek a section of the upper terrace appears to be undisturbed and could contain intact cultural deposits. Only about 5 to 10 ft of the upper terrace projects into the archeological project area as defined by Hicks & Company. The rest of this upper terrace is being commercially developed (H&C, 1998). This site now contains a movie theater complex where an open trench through the parking lot has been proposed.

The portion of the study area which was surveyed by Hicks & Company was determined to have a low potential for the presence of intact cultural deposits. In order to prevent the proposed wastewater line from possibly impacting cultural materials, it was recommended that the line avoid the upper terrace along the entire project corridor as much as possible. It was also recommended that access be obtained to

allow site 41TV1713 to be revisited to elucidate its nature and evaluate the possible impact on it by the proposed project (H&C, 1998).

Construction activities in upland terrace areas, including construction of ramps and roads for access to construction sites, could impact cultural sites. Archeological investigations might be warranted in some of these areas.

5.5 SOCIAL SETTING

Although much of the existing interceptor is in greenbelt and undeveloped properties, the project area is in an urban setting. Much of the existing interceptor is within 200 ft of single-family residential lots in the Creek Bend, Dove Springs, Wagon Crossing, and Battle Bend neighborhoods. Neighborhood Planning Areas designated by the Planning, Environmental and Conservation Services Department in the area include the South Manchaca, West Congress, East Congress, Franklin Park, and McKinney neighborhoods. The project area includes commercial and multi-family land use in the proximity of IH-35, Stassney Lane, William Cannon Drive, and South Congress Avenue. Whereas the project will have environmental impacts in natural areas, the project will also have public impacts in the developed areas.

6.0 PROJECT REQUIREMENTS - OVERVIEW

6.1 HYDRAULICS ISSUES

6.1.1 Capacity and Flow Connections

The Lower Williamson Creek relief interceptor must extend upstream to a connection point on the existing 48-inch main and deliver flow downstream to the existing 84-inch Onion Creek interceptor. A fundamental requirement for the project improvements is that it provide sufficient capacity for the design flows and good hydraulic conditions to minimize operating problems. The project improvements subject to this requirement are the relief interceptor itself, the cut-over lines carrying flow to the relief interceptor, and any sections of the existing interceptor kept in service with rehabilitation to transport lateral flows to a collective cut-over point.

A major objective of the project is diverting the flows in the interceptor's laterals to the relief interceptor, and then abandoning the existing interceptor. There are 24 lateral mains currently connected to the 36-/42-inch interceptor at 21 manholes. The cut-over mains will be constructed from a junction manhole on the relief interceptor either to the existing interceptor itself or to a diversion point on the lateral upstream from the interceptor.

Following are several advantages of cutting the lateral flows over to the relief interceptor:

- It avoids the cost and environmental impacts of rehabilitating the existing interceptor if it can be abandoned instead.
- It eliminates the need for future maintenance of the existing interceptor by the Utility if it were not abandoned but rehabilitated and kept in service for local flows.
- It avoids the environmental and public impacts of access to the interceptor for future maintenance activities.

On the other hand, there are several site-specific problems associated with installing cut-over mains:

- Construction of the cut-over main itself in some areas would have major environmental and public impacts.
- It may increase the length and cost of the relief interceptor by forcing it to follow the alignment of the existing main in order to minimize the length, cost, and impacts of constructing the cut-overs.

- A cut-over main requires construction of a junction manhole (i.e., tunnel shaft) on the relief interceptor where one might not be required without the cut-over.

A shorter relief interceptor tunnel would be possible on a straighter alignment that diverges from the existing main, but such alignment would require long cut-over mains in some areas. The higher cost and greater impacts of long cut-over mains offset the cost savings of a shorter tunnel. This problem can be avoided by not constructing a cut-over and conveying the flow from one or more laterals through the existing interceptor to a downstream point for a collective cut-over to the relief interceptor. Any laterals that are not cut over will require rehabilitation of the existing interceptor in order to keep it in service for local flows on account of its deteriorated condition described in Section 3.1. This study investigates a number of tunnel alignments and cut-over combinations (in Section 7) in order to evaluate total project costs (in Section 10).

There are limitations from a hydraulic standpoint on where it is feasible to keep sections of the existing interceptor in service for local flows. Flows from the lateral(s) should be great enough to produce cleaning velocities in the interceptor. Since the interceptor is a relatively large pipe at relatively flat slope designed for high flows from its entire service area, low flows in the interceptor from just one or more laterals could produce low velocities that would allow deposition of solids, maintenance problems, septic conditions, and odors.

Therefore, a project requirement is that any flows remaining in the interceptor be sufficient to prevent maintenance problems. A typical guideline is that the high flow on an average day should produce cleaning velocity. The effect of this requirement is that laterals with low flows should be cut over to the relief interceptor even if the cut-over presents construction or environmental problems, unless there is a lateral to the interceptor upstream that provides sufficient flows for cleaning velocities. Sliplining the existing main with a smaller diameter pipe would help to produce higher velocities than in the full-size pipe, but the smaller pipe must also have capacity for ultimate peak wet weather flows.

The design flow in different sections of the relief interceptor is affected by the location of cut-overs for the various tunnel alignment and cut-over alternatives. Regarding pipe capacity, the Utility's recently revised standard design criteria says that peak flows should not exceed 80% of the pipe's capacity flowing full ($q/Q < 80\%$), or conversely, the pipe capacity should be at least 25% greater than the design flow. This guideline is intended to ensure free ventilation in the pipe under all flow conditions and to provide an allowance for future conditions not foreseen under the current planning scenarios. Based on the design flows for the existing interceptor in Table 4-3, the required pipe capacity for a relief main carrying the entire flow is 41,200 gpm at the upper end and 51,900 gpm at the lower end. As discussed below, pipe sizes for the relief interceptor to provide these required capacities could conceivably range from 48-inch to 108-inch depending on design alternatives.

Pipe capacity for the relief interceptor in this analysis is based on Manning's equation for steady, uniform flow with a friction factor "n" equal to 0.013. Manning's equation relates pipe capacity, size, slope and velocity. Pipe size is typically determined given the required capacity and the available slope. The maximum potential slope for the relief interceptor is 0.4% based on a straight-line distance of 15,130 ft between the upstream 48-inch interceptor and the downstream junction box with the Onion Creek tunnel, and an elevation difference of 61 feet between these two control points. At that average slope, the required pipe size for the design flows would be 54-inch. If the relief interceptor tied into the Onion Creek tunnel approximately 600 feet downstream of the existing junction box, the available fall would be 78 ft for an average slope of 0.5%, and an upper portion of the relief interceptor could be 48-inch pipe. However, for practical purposes, flatter slopes will be used in at least some portions of the project because the length will be greater than the straight-line distance between the upper and lower control points and because there are obstacles causing vertical interference with the relief interceptor at the maximum available slope (and the interceptor must be deeper, i.e., flatter slope, to avoid those obstacles). Pipe slope and size with construction by tunneling can be controlled by the geological conditions, e.g., a flatter slope may be needed to maintain good rock conditions for tunneling. Furthermore, tunneling with cost-efficient working conditions requires a certain minimum size tunnel that can be the controlling factor for pipe size.

Maximum pipe size would occur at the minimum allowable slope, which is based on city and state criteria for a minimum velocity of 2 fps flowing full. However, minimum slope should be avoided if possible because most of the time the flows will be much less than the peak flow, and the velocities would be significantly less than the 2 fps needed for sewer self-cleaning. Based on standard pipe sizes, the design flows, and $q/Q < 80\%$, the pipe sizes would reach 96-inch at the upper end and 108-inch at the lower end at the minimum allowable slopes for the required capacities.

6.1.2 Ventilation and Odor Control

The large size of a tunnel compared to other sewers in the collection system means that tunnels transport a large volume of air, as the wastewater flow drags air along with it. The air flow rate depends on tunnel size, depth and velocity of the wastewater flow, lateral connections, and drop inlet conditions. The air pressure in the tunnel changes constantly in response to changes in the flow depth and in atmospheric temperature and barometric pressure. The pressure differential can cause the air to exhaust in certain locations frequently causing odor complaints. Good ventilation is beneficial in preventing corrosive sewer gas, but it can aggravate potential odor problems.

An appropriate requirement is to design the project with consideration of ventilation and odor control in conjunction with hydraulic capacity requirements. Drop inlet structures are key factors since they

can potentially inject large amounts of air to the tunnel, in addition to causing high turbulence which releases odorous gases. Certain design features can reduce these conditions.

6.2 CONSTRUCTION METHODS AND MATERIALS

6.2.1 Relief Interceptor

The existing 36-/42-inch interceptor was installed by excavation of a trench and burial of the pipe. Its route closely follows the creek alignment and its depth is great enough for laterals to reach the interceptor from both sides of the creek. The existing interceptor crosses the creek in about 17 locations.

From a construction method standpoint, it would be practical to install the relief interceptor by open cut construction, except at major street and highway crossings where tunneling would be required. Open cut construction would require the relief interceptor alignment to closely follow the creek in order to keep depth of excavation within feasible limits. Since the proposed relief interceptor is a gravity sewer with its elevation controlled by hydraulic capacity and flow cut-over requirements, the pipe elevation can not be raised to reduce depth of excavation on an alignment farther away from the creek where ground elevations are higher. A route for the interceptor at some distance from the creek would make the pipe too deep for open cut construction to be practical. In the past, relief mains were typically installed parallel to the original pipe to reduce the construction cost and also facilitate diversion of flows to the relief main.

Although cut-and-cover construction of the relief interceptor would be feasible as a construction method, it is not a desirable alternative from the standpoint of environmental and public impacts and permitting and approvals. Factors such as removal of protected trees and construction in the critical water quality zone would make the environmental approvals exceptionally difficult for a relief interceptor design for open cut construction.

Elimination of cut-and-cover construction for the relief interceptor due to environmental and public impacts requires tunneling for installation of the main. A number of wastewater interceptors have been constructed in recent years by tunneling in Austin Chalk limestone that is similar to the geology in the Williamson Creek interceptor project area. These tunnel projects include the 84-inch Onion Creek Wastewater Interceptor, the 54-inch Slaughter Creek Wastewater Interceptor, the 96-inch Govalle Wastewater Service Area Interception and Diversion System, and the 96/84-inch Austin Crosstown Wastewater Interceptor. These tunnels are located north, south and east of the Williamson Creek project. Case histories for several of these projects are presented in "Soft Rock Tunneling: Equipment Selection Concepts and Performance Case Histories" by Priscilla P. Nelson in the proceedings of the 1987 Rapid Excavation and Tunneling Conference. This article describes problems encountered during tunnel excavation in weak sedimentary rock and compares the performance of different excavation systems. The nature of

these previous sewer tunnel projects in Austin Chalk is relevant to evaluation of tunneling for the Williamson Creek relief interceptor. Appendix B is a summary of the information presented in the Nelson article.

As discussed in Section 4.1 and Appendix A, subsurface exploration for this project has included borings at nine locations along a preliminary alignment for a tunnel between Pleasant Valley Road and Congress Avenue. The rock strata encountered in all of these borings was the Austin Chalk limestone with compressive strength in the range of 660 psi to 3670 psi, moisture content of 5% to 11%, and rock quality designation (RQD) of 50% to 100%. Geotechnical data in the Nelson article on tunnel projects in Austin Chalk shows compressive strengths in the range of 490 psi to 6560 psi, moisture content of 2% to 17%, and rock quality designation (RQD) of 40% to 100%.

Given the length of the relief tunnel and the good tunneling conditions provided by Austin Chalk, a tunnel boring machine (TBM) would be used for the project rather than conventional tunneling by drilling and blasting. In addition to excavation of the tunnel bore with a TBM, construction activities would include: excavation of the main tunnel shaft, the terminal shaft to remove the TBM, and intermediate shafts for drop inlets and ventilation; mucking (spoil removal using a muck train); tunnel ventilation ducts and fans; seepage water handling; a high voltage power supply for the TBM and power for the muck train, hoist, ventilation fans, lighting, and pumping; a surface plant for staging area, materials storage, contractor's field office, and maintenance and repair shop; and a spoil storage area for off-site hauling.

6.2.2 Cut-over Mains

Cut-over mains are required for diversion of flow the relief interceptor. Although open cut construction is not feasible for the relief interceptor, trenching for pipe laying is required in some areas for installation of the cut-over mains. Among the different tunnel alternatives, there are a large number of site-specific conditions for the cut-over mains. Some cut-over mains will probably be bored or a combination of open cut and boring. The cut-over mains will be much smaller than the relief interceptor and their relatively short length allows some flexibility is selecting alignments and construction methods that meet environmental standards.

Section 7 of the report addresses the cut-over mains specific to each Alternative.

6.2.3 Rehabilitation

Another project requirement is rehabilitation of the pipeline and manholes for any sections of the existing 36/42-inch interceptor that remain in service. Various methods and materials are available for lining the existing pipe, but some are not applicable for this project. A primary factor is that the liners

will be smaller size than the existing pipe in order to improve hydraulic conditions for the much lower flows remaining in the interceptor.

Sliplining is a common rehab method which pulls or pushes a new pipe into an existing line in between an insertion pit and a manhole or receiving pit. The inserted pipe can be either a continuous pipe or individual pipe segments. Segmental pipe sliplining pushes the liner pipes into the host pipe from an insertion pit, typically using 10 or 20 ft pipe lengths. Pipe materials used for segmental sliplining include PVC, high density polyethylene (HDPE), and centrifugally cast fiberglass reinforced plastic pipe (FRP) such as Hobas. A jacking machine in the insertion pit, or a backhoe bucket for smaller and shorter lining, is used to push in each pipe segment, which advances the entire pipe string. The insertion pit is sized for the liner pipe length and jacking machine. The sliplining pipe can be jacked both directions from one location to reduce the number of insertion pits. The existing wastewater flow typically continues through the host pipe, liner, and insertion pit as the sliplining progresses, avoiding the need for bypass pumping. After installation the annular space between the liner and the host pipe is grouted. Segmental sliplining requires an alignment that is essentially straight between the insertion pit and receiving manhole, or gradual curves but no point bends. Bends in the interceptor alignment with no manhole require excavation of a receiving pit and construction of a manhole. The 36-/42-inch interceptor has 53 bends with no existing manhole, with deflection angles ranging from 33° to less than 1°.

Continuous pipe sliplining uses a string of butt-welded HDPE pipe lengths which is pulled into the host pipe at the insertion pit by means of a winch located at the receiving pit or manhole. The HDPE pipe joints are fusion welded into a single jointless pipe string which is typically installed in one continuous operation if a long narrow workspace is available to position the string for the pull. An alternative is to weld and pull the pipe one or several joints at a time depending on workspace. The insertion pit for continuous sliplining is ramped from the host pipe up to natural grade with curves to support the pipe as it is pulled into place. This generally requires excavation of a longer insertion pit depending on the depth of the host pipe. Continuous sliplining requires bypass pumping when full size liner is installed. However, if a small liner is installed with relatively low flows, the annular space may be sufficient to avoid bypass pumping. The annular space is grouted after slipliner installation. A small liner pipe may also allow the pipe string to be pulled through minor bends.

Another common rehab method is cured-in-place pipe (CIPP). However, with CIPP the resin-impregnated felt lining tube is expanded tightly against the existing pipe and cured with steam or hot water to harden the resin. Therefore, CIPP cannot produce smaller diameter liners desired on this project for hydraulic conditions. Advantages of CIPP include insertion of the liner through existing manholes (i.e., no excavation of insertion pits) and ability to negotiate minor bends. Installation of CIPP requires bypass pumping for the existing wastewater flows.

A procedure similar to CIPP uses PVC "fold and form" pipe. A reel of the deformed pipe is heated to soften the PVC and then pulled into the host pipe through an existing manhole and pressurized with steam to reform the pipe tightly against the original pipe wall. This method has advantages and disadvantages similar to CIPP.

A rehab method making a new appearance in the Austin area is the Channeline system. Advantages are installation through existing manholes, segments manufactured to negotiate bends, and typically no bypass pumping. The system uses interlocking reinforced plastic pipe segments to form the lining. The one- or multi-piece segments are introduced through existing manholes and assembled manually inside the host pipe. Once assembled, the lining is grouted in place. The system is applicable for larger-diameter pipes allowing manned entry, with 36-inch being the minimum intended size. Bypass pumping is not required for low flows. However, some sections of the existing 36-/42-inch interceptor may have sufficient flow remaining after the cut-overs that bypass pumping would be required, e.g., the section of 36-inch pipe in the vicinity of Congress Avenue which will continue to receive flow from a 20-inch lateral in all Alternatives. The Channeline system was selected for the Barton Creek Interceptor Rightsizing project in Austin, which includes smaller liners in the 36-/42-/54-inch interceptor in order to improve hydraulics. This new procedure does not have the historical performance record in Austin that sliplining and CIPP have.

Another new method for the Austin area is rehab using spiral wound pipe. In this procedure a hydraulically operated winding machine placed in an existing manhole (without excavation) spirally winds a continuous strip of PVC into a new pipe which is pushed into the existing pipe during the winding process. The PVC strip has a cross-section profile that produces a continuous interlocked joint sealed with adhesive. After insertion the pipe is grouted in place. Deflection of the pipe can negotiate curves but not point bends. Bypass pumping is not required. The PVC profile strip and installation methods are covered by ASTM specifications, but there is no performance history for this rehab method in the Austin area.

Manhole rehabilitation is required concurrent with pipeline rehab. The unlined original manholes as well as the pipe are showing corrosion and structural deterioration. Installation of a structural lining system will reinforce the manholes, provide corrosion protection, and prevent I/I and root intrusion.

The existing pipe and manholes must be thoroughly cleaned prior to the rehabilitation procedures. A high pressure cleaning truck will dislodge debris from manhole to manhole. Any sections with hard packed deposits may require mechanical cleaning. Debris will be removed from the line and trucked out for disposal. Television inspection will be made prior to installation of the liner pipe to confirm adequate cleaning. Spot repairs will be required for any offset joints or other defects which would interfere with sliplining.

Rehabilitation has been considered maintenance work that can be performed under the Utility's General Permit, rather than construction which would require permitting on a project by project basis. As maintenance it would be exempt from the requirement for a variance from the Planning Commission for construction in the Critical Water Quality Zone. Appropriate environmental protection and restoration measures will be applied due to the ecological features of the maintenance workspaces and access routes. Once a final Alternative has been selected for construction, the design phase of the project will include a formal Environmental Assessment that will be submitted to the City's Environmental Resource Management department to determine the permitting requirements for the rehabilitation work.

6.2.4 Access Considerations

One of the biggest above-ground concerns during design and construction of a tunnel is access to the tunnel shafts. For this project, access must be obtained to the existing interceptor (pipe, lateral, manholes) for cutovers, sliplining, and permanent access easements (for the new alignment as well as any portion of the existing wastewater line that will stay in service). In many cases, access is the primary factor in deciding which alternative to choose due to environmental and public impacts and implications for permitting and approvals. The access routes will have their own set of construction requirements including possible clearing, erosion and sedimentation control during use, and permanent restoration once the project is completed.

Potential access routes are included in the discussion of each Alternative in Section 7 of the report.

6.2.5 Pipe Materials

It is required that pipe materials used on the project not corrode in the sewer environment and have sufficient strength for any loading on the pipe during and after installation. Pipe installed in the tunnel will be required to withstand axial jacking loads during installation as well as long-term radial loads from the surrounding earth. The various products discussed above regarding rehabilitation are all non-corrodible.

Reinforced concrete pipe (RCP) was frequently used in the past for sewer tunnels. RCP can be manufactured with an anchored PVC liner (e.g., "T-Lock") locked into the concrete that provides protection from corrosion and abrasion. Protective polymer coatings can also be applied after pipe manufacture. Such systems require that the pipe joints be totally sealed after the pipe is installed in the tunnel. Large tunnels frequently have a cast-in-place concrete lining, which can be constructed inside the tunnel with a protective anchored liner or coating.

Centrifugally cast fiberglass reinforced polyester pipe (FRP), e.g., Hobas pipe, is commonly used for sewer tunnels. The material is inherently corrosion resistant. The lighter weight and longer joints of FRP can provide some installation advantages over RCP. Polymer concrete ("polycrete") is a new pipe material for the Austin area that has potential for sewer tunnels. It is similar to concrete pipe except that cement is replaced by a polymer compound to provide inherent corrosion protection similar to FRP.

Tunnel inlet structures require special attention to corrosion protection on account of the potential for release of hydrogen sulfide due to turbulence. Embedded PVC liner and polymer coatings can be used for protection of structural concrete, and non-corrodible pipe can be used for riser sections.

The smaller cut-over mains will be similar to conventional wastewater lines that are installed according to standard city specifications. Manholes that are constructed on existing mains to implement a cutover must have a protective coating or non-corrodible risers, e.g., fiberglass.

6.3 ENVIRONMENTAL IMPACTS

The project requirements include minimizing environmental impacts. This consideration is a large part of the preliminary engineering and design process. Each alternative for the relief interceptor will have a certain level of environmental impact associated with it - both during construction and then, in the future, for continued maintenance. Avoiding a cut-over eliminates the environmental and public impacts of constructing both the cut-over main and the junction manhole on the relief interceptor. However, rehabilitation of the existing interceptor is not without environmental impact. Sliplining generally requires access by vehicular equipment to all manholes on the section of the interceptor that will be sliplined and, depending on the pipe material, a significant amount of workspace is required.

Much of the project lies within the Critical Water Quality Zone (CWQZ), i.e., zones along creek and tributaries as defined in Section 25-8-92 of the Land Development Code. For a Major waterway, the boundaries of the CWQZ are located not less than 200 feet and not more than 400 feet from the centerline of the waterway. Any Critical Environmental Features (features which have been determined to be of importance to the protection of one or more environmental resources, such as bluffs, springs, canyon rimrocks, caves, sinkholes and wetlands (Section 25-8-281)) have been identified and are further described in this report.

6.4 PUBLIC IMPACTS

Project requirements also include minimizing public impacts. To some extent, the different alternatives offer trade-offs between environmental and public impacts in the vicinity of the construction workspace, but the project will have public impacts in a larger area. Construction will have an impact on neighborhood traffic - increased volume and noise due to both with contractor employees and traffic associated with operations (hauling spoil, delivery of materials and equipment). Although it will be required that dust and operational noise be kept to a minimum, there will be some, as well as some vibration which may be sensed - depending on the neighborhood's vicinity to the actual tunneling activity. While tunneling has less of a "noticeable" impact in extent, it's duration of impact is actually longer. A tunnel shaft could be required from anywhere from 24 to 32 months - operations extending 24 hours/day.

6.5 EASEMENTS, PERMITTING AND APPROVALS

6.5.1 Parkland Use Agreement

A parkland use agreement between the Water and Wastewater Utility and the Parks and Recreation Department is required for construction of any project facilities in the Williamson Creek Greenbelt. Approximately 80% of the existing 36-/42-inch interceptor is located or immediately adjacent to the greenbelt. Much of the greenbelt was apparently acquired by the City after construction of the interceptor in 1963. The amount of the various relief interceptor alternatives located in the greenbelt ranges from approximately 10% to 60%.

Some construction in parkland is unavoidable on this project due to the need to install cut-over mains that will connect to existing sewers that are located in parkland. Some alternatives being considered require construction of tunnel shafts in parkland. In addition to excavation for these construction activities, access to the work sites through parkland will be required for haul routes. Sliplining for rehabilitation of the existing interceptor is considered as maintenance by the Utility rather than construction activity, and does not require a parkland use agreement.

The request for a parkland use agreement is supported by an information packet which discusses the project description and need, alternatives to use of parkland, project schedule, and the short and long term effects of construction. This report should address Chapter 26 of the Texas Parks and Wildlife Code which states that the use of parkland shall be approved only if there are no feasible and prudent alternatives to using parkland and all reasonable planning to minimize harm to the parkland has been carried out. A parkland use agreement is granted by City Council resolution following a public hearing for citizen input, after approval of the use agreement by the Parks Board based on review of plans and

recommendations by the Land and Facilities Committee. The project also requires survey field notes for description of a parkland easement that will be recorded in county Real Property Records.

Use of parkland requires that the plans and specifications be in compliance with the "Construction in Parks Specifications" adopted by the Parks Board. Approval of the engineering drawings is required and indicated by sign-off of the Parks and Recreation Department on the cover sheet. The Construction in Parks Specifications specifically address requirements for tree surveys; evaluation, preservation, and replacement of trees; construction procedures; and restoration and revegetation. Special conditions may also be imposed by the Parks Board as condition for approval of use of parkland, for example, for certain park improvements to be included in the utility project, such as restoration of a haul route as a hike and bike trail.

6.5.2 Private Easements

Easements from private property owners are required where the project is not city right-of-way or parkland. An easement for construction of the tunnel under private property is required even if there is no encroachment on the property for construction activity. Easements are also required where cut-over mains or access routes cross private property. Typically some temporary easements are required in addition to the permanent easements in order to provide sufficient workspace for the contractor during construction.

The easement acquisition process requires identifying and contacting the appropriate property owners, explanation of the project and description of the requested easement, preparation of easement documents including survey field notes, and recording of the easements in county property records. The process can take anywhere from several weeks for cooperative property owners to many months for cases involving protracted negotiation and possibly eventual condemnation proceedings.

Easement requirements can be a significant factor in evaluating the project Alternatives. Impact on project schedule for easement acquisition is one consideration. Cost is another factor. For planning purposes, the Department of Transportation and Public Works suggests using an average cost to the project of $6,000 per residential lot for the administrative work of obtaining uncontested easements (excluding condemnation easements). This does not include any cost for land or easement surveying. Tunnel easements under developed property also introduce a risk of liability for damage claims due to vibration or settlement. The project should require photographic documentation of pre-existing property conditions and vibration and settlement monitoring during construction, as evidence for any lawsuits for property damage due to tunneling.

6.5.3 State of Texas Highway Permit

Highway permits will be needed from the Texas Highway Department of Transportation for construction of the relief interceptor beneath IH-35 and Congress Avenue. The highway permit specifies construction methods and materials which might be required for the highway crossing. The time required for obtaining a highway permit between submittal of plans and receipt of permit is typically 4 to 6 weeks.

6.5.4 Variance for Construction in Critical Water Quality Zone

The primary sections of the Land Development Code (LDC) relevant to this project are Land Development Procedures - Water Quality Related Development Intensities, and Environmental Protection and Management. "Development" in a broad sense is any activity which causes land disturbance, which includes utility line construction as well as subdivision development.

Requirements for allowable development activities and intensities within the City's jurisdiction depend on the watershed in which the development is located. The Williamson Creek basin in the project area, which is entirely east or downstream of the Edwards Aquifer recharge zone, is classified as a Suburban Watershed.

In regard to the regulatory framework affecting design and construction of the Lower Williamson Creek Relief Interceptor, Critical Water Quality Zones, defines critical water quality zones along waterways and identifies allowable activities and development. For Williamson Creek in the project area (classified here as a major waterway on the basis of drainage area), the boundary of the critical water quality zone is the 100-year floodplain, with the limitation that this boundary is located not less than 200 feet nor more than 400 feet either side of the centerline of the waterway).

No construction activity, development or alterations are allowed in the critical water quality zone except for the types and conditions explicitly listed in the LDC. Utility lines, including sewer lines, are among the permitted types of development. Construction of sewer utility lines in the critical water quality zone is subject to special requirements. Specifically, "a sewer line shall not be located in a critical water quality zone other than for necessary crossings, except upon approval of a variance from this section by the Planning Commission which shall be considered pursuant to an environmental assessment and the applicant evaluating the environmental impacts of alternative sewer alignments".

6.5.5 City of Austin Site Development Permit

In addition to compliance with provisions in the Land Development Code, the design and construction of the Lower Williamson Creek relief interceptor must satisfy the requirements of other City

of Austin guidelines and regulations. These include the requirements of the Environmental Criteria Manual and the Utilities Criteria Manual and the W&WW Standard Technical Specifications. Special relevance applies to Floodplain Modification Criteria, Section 1.7 of the Environmental Criteria Manual. The floodplain modification criteria are intended "to prevent and reduce direct degradation of water quality by minimizing disturbance of vegetative cover in order to reduce construction associated pollutants, shoreline, channel and floodplain erosion, and sedimentation problems."

6.5.6 Corps of Engineers Permit

The Lower Williamson Creek Relief Interceptor would be constructed in jurisdictional waters (which include waters and wetlands) under regulation of the U.S. Army Corps of Engineers (USACE). Any specific action within these waters may be subject to authorization by the USACE prior to any construction activities. The type of permit that might be required for a particular project is based on several factors, including the type of activity proposed, the location of the jurisdictional area within the watershed or in relation to a water body, and the extent of impacts to the jurisdictional areas. The regulatory program of the USACE provides for permitting through several procedures with the most common permitting procedures including a standardized individual permit, a nationwide permit, and regional general permits.

6.5.7 Texas Natural Resource Conservation Commission

Once designed, the project plans and specifications should comply with State regulations and should be submitted to the Texas Natural Resource Conservation Commission for review and approval. The TNRCC may choose to comment, however, if all State Design Guidelines have been followed and are adequately documented, there is often little to no response from the Agency.

6.5.8 Texas Historical Commission

Because the project is funded by a state municipality, the Antiquities Code of Texas requires that an Antiquity Permit be applied for. In addition, since the project occurs along Lower Williamson Creek, it falls within the jurisdiction of the U.S. Army Corps of Engineers, and will require a federal permit according to Section 106 of the National Historic Preservation Act of 1966. The Texas Historical Commission serves as the issuing agency for both state and federal permits.

This preliminary engineering phase of the project includes a preliminary assessment of cultural resources in a corridor that contains the various alternative routes. It is intended to identify factors which could affect selection of the alternative for design. The design phase of the project for the selected alternative will require application for a formal Antiquities Permit, additional archeological investigations as stipulated in the permit, and preparation of a report detailing the investigations.

6.6 PROJECT COST

An obvious requirement for completion of the project is adequate funding for engineering, construction and administrative costs. The Utility's 1994 Wastewater Collection System Long Range Planning Guide included project cost estimates of $2.547 million for the Williamson Creek Tunnel and $4.72 million for the Lower Williamson Creek Interceptor Upgrade. As discussed in Section 1.2, these two projects have been combined into the current project. The cost estimate for the Williamson Creek Tunnel was based on tunneling, but the cost estimate for the Lower Williamson Creek Interceptor Upgrade was based on open cut construction. Current costs are considerably higher than the 1994 estimates due to inflation and due to tunneling rather than open cut for the portion of the current project that had been the Lower Williamson Creek Interceptor Upgrade.

The general description of the Facilities Plan in the Long Range Planning Guide did not address rehabilitation of the existing interceptor or construction of diversion mains to cut over flow to the relief main. The 1994 cost estimates apparently did not include costs for these parts of the current project.

The Utility reports that bond elections have provided funding of $5 million specifically for the Lower Williamson Creek Interceptor improvements, and also that $7 million is reserved from other sources. Additional funding may be available if the Utility considers the project need critical compared to other W&WW projects. The cost estimates presented in Section 10 for the various alternatives significantly exceed the current available funding.

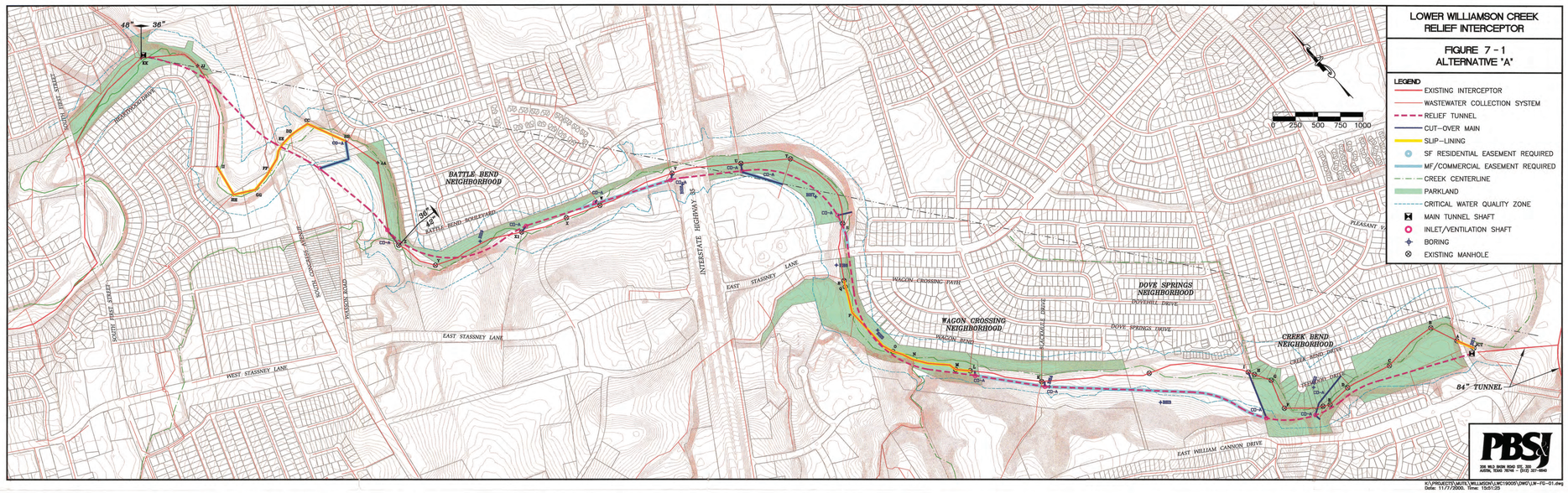
7.0 ENGINEERING AND ENVIRONMENTAL EVALUATION OF ALTERNATIVES

7.1 CONCEPTUAL BASIS OF ALTERNATIVES

A number of alternatives have been evaluated for the project with consideration given to the various project requirements as discussed in Section 6. As described in Section 6.2.1, cut and cover construction constrains the alignment of the relief interceptor close to the existing main in order for open cut excavation to remain practical. There is a narrow corridor close to the creek and the existing line where there could be some variations in the relief main alignment, but the environmental and public impacts of the open cut alignment alternatives would not be significantly different. Alternatives with open cut construction for the relief interceptor have not been pursued due to the magnitude of the environmental and public impacts.

Since tunneling largely, but not entirely, detaches the relief main alignment from ground surface elevation, there is a wide corridor for alternative tunnel alignments between the required upstream and downstream connection points. Different tunnel alignments affect the requirements for new cut-overs main and for sliplining the existing interceptor, all of which affect the project cost, environmental and public impacts, and easement and approvals requirements. This section discusses five conceptual alternatives for relief tunnel alignments and the corresponding cut-over main and sliplining requirements, and the resulting environmental and engineering impacts. Figures 7-1 to 7-5 show the tunnel, cut-over main, and sliplining features for each alternative, and Figure 7-6 (map pocket) provides a schematic showing the tunnel alignments in relation to each other. It should be noted that these five alternatives represent general concepts. There are numerous variations possible within each alternative, and the overall project could ultimately be a combination of portions of different alternatives.

Alternatives "A" and "B" generally follow the alignment of the existing interceptor in order to maximize the number of short cut-over mains and to minimize the amount of sliplining (and maximize abandonment) of the existing interceptor. The main difference between "A" and "B" is that Alternative "A" has curves in the tunnel alignment and "B" does not. This has significant environmental implications. Following the general alignment of the existing interceptor to maximize cutovers requires a number of changes in direction for the relief tunnel. Straight tunnel sections are desirable to reduce the difficulty of maintaining the correct line and grade, but a tunnel with straight sections requires sharp bends to change direction rather than gradual curves. A tunnel boring machine can be steered on a gradual curve, but a sharp bend to change the heading of the TBM requires excavation of a shaft in order to reorient the machine. Furthermore, the muck train can not negotiate sharp bends without time-consuming skip handling. Therefore, each major bend would normally be a shaft with a hoist or crane for muck removal. Without curves, Alternative "B" has five more shafts than Alternative "A". Excavation of the shafts, access for


LOWER WILLIAMSON CREEK
RELIEF INTERCEPTOR
FIGURE 7 - 1
ALTERNATIVE "A"
LEGEND
EXISTING INTERCEPTOR
WASTEWATER COLLECTION SYSTEM
RELIEF TUNNEL
CUT-OVER MAIN
SLIP-LINING
SF RESIDENTIAL EASEMENT REQUIRED
MF/COMMERCIAL EASEMENT REQUIRED
CREEK CENTERLINE
PARKLAND
CRITICAL WATER QUALITY ZONE
MAIN TUNNEL SHAFT
INLET/VENTILATION SHAFT
BORING
EXISTING MANHOLE
0 250 500 750 1000
48" 36"
36" 48"
BATTLE BEND NEIGHBORHOOD
BATTLE BEND BOULEVARD
INTERSTATE HIGHWAY 35
EAST STASSNEY LANE
WEST STASSNEY LANE
WAGON CROSSING PATH
WAGON CROSSING NEIGHBORHOOD
WAGON BEND
DOVE SPRINGS NEIGHBORHOOD
DOVEHILL DRIVE
DOVE SPRINGS DRIVE
CLOVELY DRIVE
CREEK BEND NEIGHBORHOOD
CREEK BEND DRIVE
PEASANT VIEW DRIVE
PLEASANT V
EAST WILLIAM CANNON DRIVE
SOUTH FIRST STREET
SOUTH CONGRESS HILLS
84" TUNNEL
PBSJ
305 WILD BURN ROAD STE. 200
AUSTIN, TEXAS 78746 - (512) 327-6940
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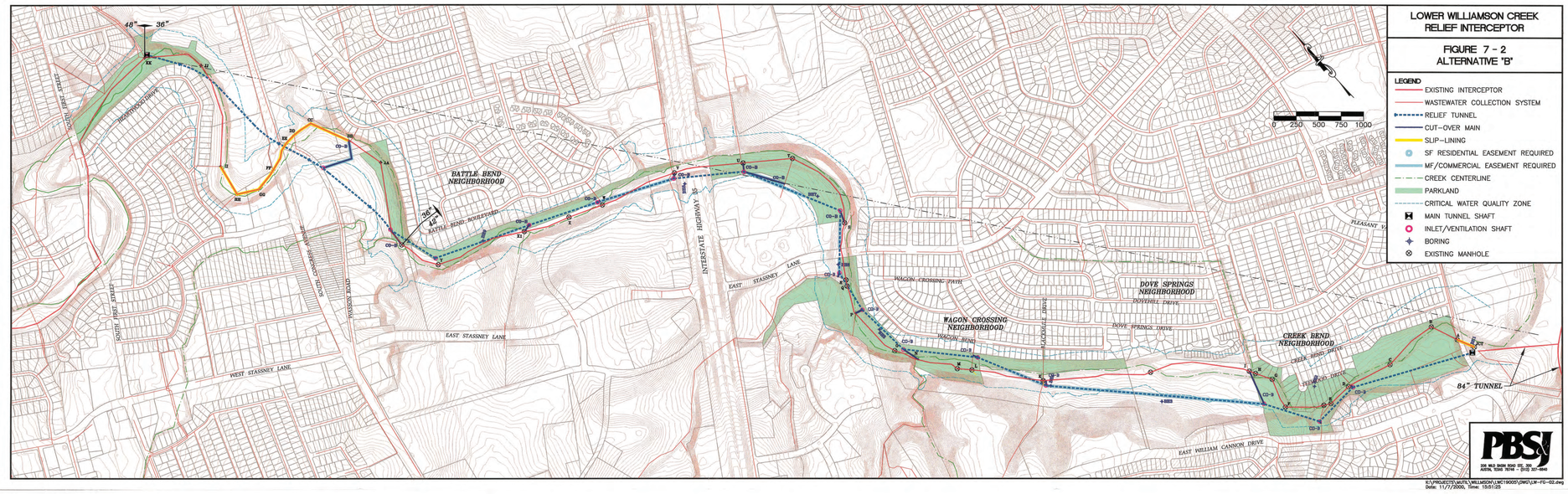

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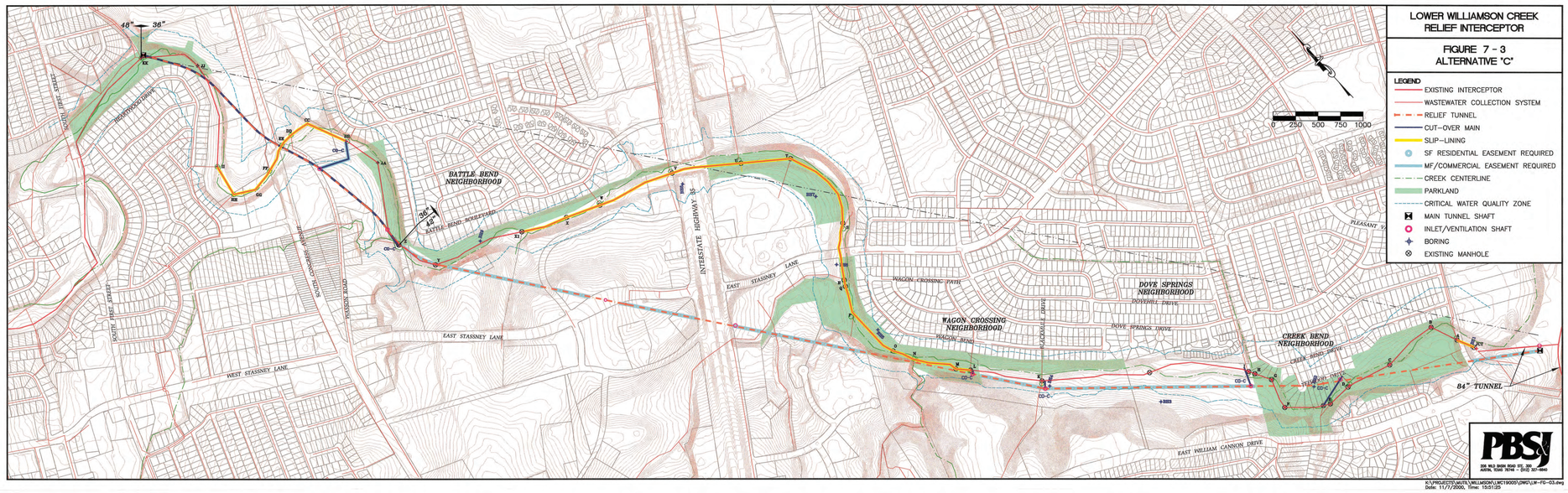
LOWER WILLIAMSON CREEK
RELIEF INTERCEPTOR
FIGURE 7 - 3
ALTERNATIVE "C"
LEGEND
EXISTING INTERCEPTOR
WASTEWATER COLLECTION SYSTEM
RELIEF TUNNEL
CUT-OVER MAIN
SLIP-LINING
SF RESIDENTIAL EASEMENT REQUIRED
MF/COMMERCIAL EASEMENT REQUIRED
CREEK CENTERLINE
PARKLAND
CRITICAL WATER QUALITY ZONE
MAIN TUNNEL SHAFT
INLET/VENTILATION SHAFT
BORING
EXISTING MANHOLE
0 250 500 750 1000
48"
36"
BATTLE BEND NEIGHBORHOOD
BATTLE BEND BOULEVARD
INTERSTATE HIGHWAY 35
EAST STASSNEY LANE
WEST STASSNEY LANE
WAGON CROSSING PATH
WAGON CROSSING NEIGHBORHOOD
WAGON BEND
DOVE SPRINGS NEIGHBORHOOD
DOVE SPRINGS DRIVE
CREEK BEND NEIGHBORHOOD
CREEK BEND DRIVE
PLEASANT VALLEY DRIVE
EAST WILLIAM CANNON DRIVE
SOUTH 1ST STREET
84" TUNNEL
PBSJ
206 MLK BURN ROAD (R), 200
AUSTIN, TEXAS 78745 - (512) 327-6940
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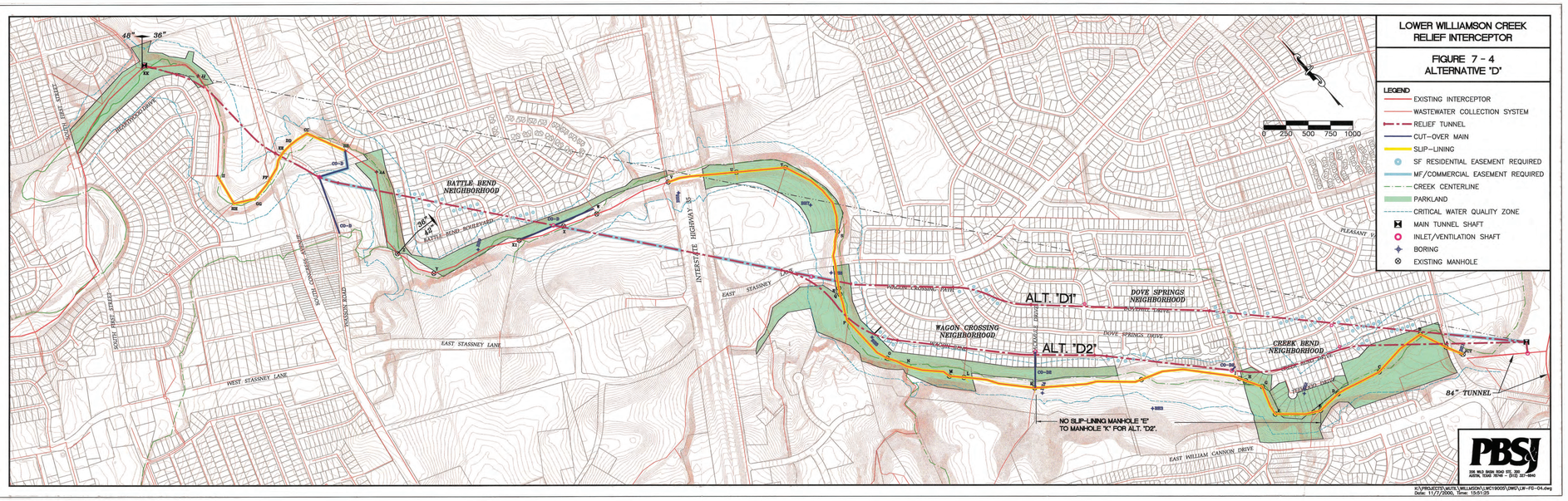
LOWER WILLIAMSON CREEK
RELIEF INTERCEPTOR
FIGURE 7 - 4
ALTERNATIVE "D"
LEGEND
EXISTING INTERCEPTOR
WASTEWATER COLLECTION SYSTEM
RELIEF TUNNEL
CUT-OVER MAIN
SLIP-LINING
SF RESIDENTIAL EASEMENT REQUIRED
MF/COMMERCIAL EASEMENT REQUIRED
CREEK CENTERLINE
PARKLAND
CRITICAL WATER QUALITY ZONE
MAIN TUNNEL SHAFT
INLET/VENTILATION SHAFT
BORING
EXISTING MANHOLE
0 250 500 750 1000
48" 36"
HEATHERSTONE
CREEK HEIGHTS
BATTLE BEND NEIGHBORHOOD
BATTLE BEND BOULEVARD
36" 48"
INTERSTATE HIGHWAY 35
EAST STASSNEY LANE
WEST STASSNEY LANE
SOUTH PINE STREET
WOODSTOCK HILLS
CREEK MANOR
WAGON CROSSING PATH
WAGON CROSSING NEIGHBORHOOD
ALT. "D1"
ALT. "D2"
DOVE SPRINGS NEIGHBORHOOD
DOVE SPRINGS DRIVE
CREEK BEND NEIGHBORHOOD
PLEASANT DRIVE
PLEASANT VALLEY
EAST WILLIAM CANNON DRIVE
84" TUNNEL
NO SLIP-LINING MANHOLE "E"
TO MANHOLE "K" FOR ALT. "D2".
PBSJ
305 WEST BARTON SPRINGS ROAD, STE. 200
AUSTIN, TEXAS 78704 - (512) 327-8840
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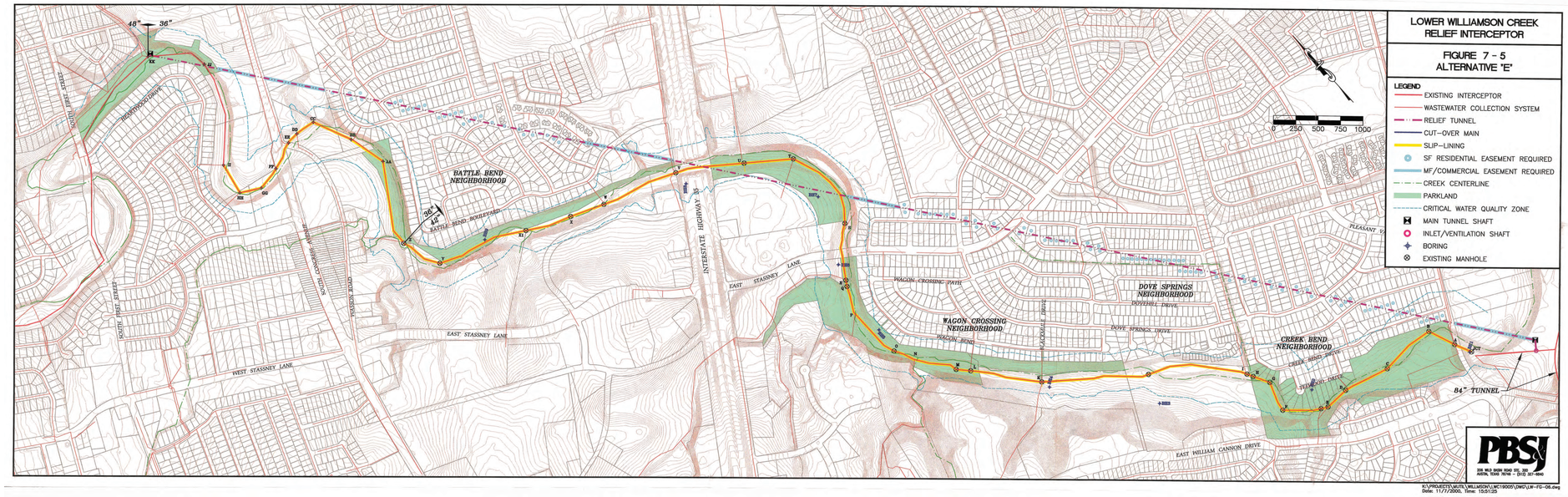
LOWER WILLIAMSON CREEK
RELIEF INTERCEPTOR
FIGURE 7 - 5
ALTERNATIVE "E"
LEGEND
EXISTING INTERCEPTOR
WASTEWATER COLLECTION SYSTEM
RELIEF TUNNEL
CUT-OVER MAIN
SLIP-LINING
SF RESIDENTIAL EASEMENT REQUIRED
MF/COMMERCIAL EASEMENT REQUIRED
CREEK CENTERLINE
PARKLAND
CRITICAL WATER QUALITY ZONE
MAIN TUNNEL SHAFT
INLET/VENTILATION SHAFT
BORING
EXISTING MANHOLE
0 250 500 750 1000
48"
36"
84" TUNNEL
BATTLE BEND NEIGHBORHOOD
BATTLE BEND BOULEVARD
WAGON CROSSING NEIGHBORHOOD
WAGON CROSSING PATH
WAGON BEND
DOVE SPRINGS NEIGHBORHOOD
DOVEHILL DRIVE
DOVE SPRINGS DRIVE
CREEK BEND NEIGHBORHOOD
CREEK BEND DRIVE
INTERSTATE HIGHWAY 35
EAST STASSNEY LANE
WEST STASSNEY LANE
SOUTH FIRST STREET
CERES HILL PATH
HEATHER GLEN
HIGHLAND HILLS
STASSNEY ROAD
CLOVELLY DRIVE
PLEASANT
EAST WILLIAM CANNON DRIVE
PBSJ
305 WEST BARTON SPRINGS STREET, STE. 300
AUSTIN, TEXAS 78704 • (512) 327-1840
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muck hauling, and construction of cut-over mains to the shafts would have significant public and environmental impacts in two sensitive areas for alternative "B", as discussed below in more detail.

Alternatives "A" and "B" produce the greatest tunnel lengths. Both alternatives begin at the existing junction box just east of Pleasant Valley Road, where the 84-inch tunnel receives flow from the 42-inch main. The City would like to abandon the diversion box if possible due to hydraulic problems with the existing structure (see Section 8.5.1) and also due to the upcoming Creek Bend drainage improvements project which would leave the diversion box elevated and isolated in an improved channel. The "A" and "B" tunnel alignments generally are located within the boundaries of the 100-year flood plain, which minimizes the need for tunnel easement under developed property. Due to depth constraints, the "A" and "B" alignments stay on the south side of Williamson Creek at the lower end of the project.

Alternatives "C", "D" and "E" achieve shorter tunnel lengths by diverging from the existing interceptor alignment on more direct routes between upstream and downstream connection points to the existing system. However, the more direct routes require the relief main to cross Williamson Creek fairly close to Pleasant Valley Road. The diversion box 48-inch stub that is the connection point of Alternatives "A" and "B" is too high in elevation for a tunnel connecting at that point to cross under Williamson Creek. In addition, there is inadequate usable workspace at the diversion box to construct a junction directly on the 84-inch tunnel out of the diversion box that is 8 ft lower than the 48-inch stub. Therefore, Alternatives "C", "D" and "E" originate at a point approximately 600 ft east of the existing diversion box, where workspace is available for a new diversion structure to be constructed for a junction with the 84-inch tunnel. The tunnel at that point is approximately 19 feet lower than the inlet at the diversion box. The additional 600 ft tunnel length required for the new connection point significantly offsets the tunnel length savings on the straighter alignments compared to the Alternatives "A" and "B" along the existing interceptor.

Alternative "C" reduces the tunnel length by bypassing the bend in Williamson Creek in the vicinity of IH-35. It remains fairly close to the existing 42-inch main up to a point about 2,000 ft east of IH-35 and then takes a straight alignment to a point about 3,000 ft west of IH-35 where it again is in proximity to the 42-inch main. Alternative "D" is somewhat shorter than "C" by taking an even more direct route that crosses parts of Creek Bend, Wagon Crossing, Dove Springs, and Battle Bend Springs subdivisions. Alternatives "C" and "D" both have a few minor curves in the tunnel alignment. Alternative "D", unlike Alternative "C", requires subsurface tunnel easements from a substantial number of home owners, even though much of the alignment can follow street right-of-way. Both Alternatives "C" and "D" require tunnel easements from multi-family and commercial tract owners. Alternative "D", with its route diverging more from the alignment of the 42-inch main, has fewer cut-overs, but a greater total length for cut-over mains, and more sliplining. Alternative "E" features a straight tunnel between the lower and upper connection points, i.e., the shortest tunnel possible. This alignment significantly increases the

number of tunnel easements required. The greater divergence of Alternative "E" from the existing main results in the fewest cut-overs and the most sliplining for the existing interceptor.

Alternatives "A", "B", and "C" utilize the alignment for the 60-inch tunnel that was designed in 1986 (and revised in 1992) to relieve the existing 36-inch interceptor. The Utility has already obtained the easements for this alignment. Alternative "D" includes only the portion of the 60-inch tunnel from Wasson Road upstream to the 48-inch interceptor connection. Alternative "E" abandons this previous tunnel alignment.

7.2 ALTERNATIVE "A"

The Alternative "A" tunnel alignment closely follows the existing interceptor. Alternative "A" consists of 17,260 ft of tunnel, 11 inlet shafts for 13 cut-over mains with total length of approximately 2,810 ft, and 4,100 ft of sliplining (or 20% of the total length of the existing interceptor). Approximately 50% of the tunnel alignment is in curves. The tunnel alignment, cut-overs, and sliplining are shown in Figure 7-1.

The alignment for Alternative "A" starts at the existing diversion box just east of Pleasant Valley Road, with connection to the existing 48-inch stub at flowline elevation 516.1. Workspace for a tunnel construction shaft is available but limited around the diversion box and abandoned lift station structure at this location. The site is in the 100-year flood plain. The City's Creek Bend Drainage Improvements Phase 2 includes grading to enlarge the channel in the area that would be the tunnel construction shaft workspace.

An alternate tunnel shaft site is just west of Pleasant Valley Road, but this is parkland with scattered trees, and also in the 100-year flood plain. A short tunnel would be bored eastward under Pleasant Valley Road to the diversion box, with the main tunnel heading west. Accessibility to the area west of Pleasant Valley Road is a problem. The steep embankment for the road would require a considerable area for placement of fill to build up a ramp for the construction traffic. Obtaining adequate workspace for the ramp and main shaft west of Pleasant Valley Road would require removal of a number of mature trees. There are also two sewer lines in this area which impose some limitation on shaft location.

There is an alternate access route from the diversion box to the west side of Pleasant Valley Road under the Williamson Creek bridge, but it is not suitable for heavy construction equipment. However, this route may be used for access to manhole "A" for sliplining. Given the Alternative "A" tunnel alignment, no cut-over is proposed for the lateral mains to manhole "A". The 42-inch interceptor would be sliplined approximately 200 ft between the diversion box and manhole "A".

The elevation constraint for the tunnel tie-in imposed by the 48-inch stub at the diversion box creates a problem for tunneling due to the geology. Boring B-1 adjacent to the diversion box shows that the strata at the pipe elevation is clayey gravel overlain by lean clay with some limestone gravel. The water level at boring B-1 was about 4 ft above the rock/soil interface. A TBM that would bore efficiently in Austin Chalk would not work in this soft ground material. Depending on the TBM design, a contractor might be able to modify the excavation and muck handling equipment in order to make the transition, or a shield tunnel could be advanced by hand mining to a point where a rock tunnel could begin with a TBM. The water table level is a problem and would require dewatering or grouting. The case histories in Appendix D for several Austin tunnels illustrate tunneling problems that can occur in these conditions.

An alternative to soft ground and mixed face tunneling in this area would be a bore under Pleasant Valley Road and then open cut construction in the parkland to the west. Excavation depth would reach over 25 ft. Dewatering would be required. The workspace needed would remove a substantial number of trees. The environmental impacts and restoration requirements are much greater for open cut in this section than for the tunneling alternative.

Determining the extent of the soft ground tunneling conditions west of Pleasant Valley Road requires additional borings in the area. The TBM should have at least one tunnel diameter of sound unweathered rock above the crown of the tunnel. At boring B-2 approximately 1,700 ft west of Pleasant Valley Road, sound rock is still be too deep for reliable TBM tunneling, but B-2 is approximately 300 ft north, and not representative of the Alternative "A" alignment. Exposed rock on the steep slope north of Brushy Ridge Drive just south of Alignment "A" suggests that rock on Alignment "A" is higher than at B-2. Mixed face tunneling or open cut construction might be required for 1,000 to 1,500 ft or more west of Pleasant Valley Road before good Austin Chalk tunneling conditions are encountered.

The Alternative "A" alignment has two compound curves through the greenbelt south of Creek Bend in order to stay in City parkland and avoid obtaining tunnel easements under private residential lots to the south. The first inlet shaft (shaft #A1) is south of Williamson Creek in the vicinity of manhole "E" on the 42-inch interceptor. The depth limitation on the tunnel due to the diversion box tie-in prevents a straight alignment that crosses the creek and locates the shaft in City property on the north bank, which would have much easier access. Access to shaft #A1 from the north would require a temporary stream crossing/baseflow diversion structure and reconstruction of recent Creek Bend drainage improvements (e.g., concrete retaining wall, flexible channel liner, and slope protection blanket). Access to the inlet shaft from the south from William Cannon Drive would require building up a ramp to reduce the slope off of William Cannon sufficiently for construction equipment.

Two cut-over mains would connect to the first inlet shaft. Approximately 50 ft of 8-inch main would cut over flows in the lateral to manhole "E". Cutting over the flows currently going to manhole "D"

would require approximately 550 ft of 8-inch main partially installed by boring in order to cross under Williamson Creek and a number of protected trees. Note that the flows are intercepted from the laterals and not from the 42-inch main.

A second inlet shaft (#A2) is proposed at the western boundary of the Creek Bend greenbelt in order to cut over the flow in the 12-inch main going to manhole "I". This cut-over is desired in order to (a) avoid sliplining the 42-inch downstream of "I" and (b) avoid construction of a cut-over main tying directly into the 42-inch. The 42-inch interceptor is largely inaccessible now without causing damage to the recent Creek Bend drainage improvements. Excavation of the 42-inch main for sliplining insertion pits or for a cut-over main tie-in would require reconstruction of the damaged drainage improvements. Insertion pits would be required at manholes "F" and "G" due to bends in the alignment of the 42-inch main.

The length required for the lateral "I" cut-over main is approximately 500 ft. The shaft can not be located closer to manhole "I" due to the 1,000-ft limitation on tunnel radius. Boring and jacking the cut-over main would probably require the contractor to use an intermediate bore pit to keep the pipe adequately on line and grade. The intermediate bore pit and access to it would require removal of some trees in this heavily wooded area.

An alternative to a lateral "I" cut-over main is a cut-over to manhole "F", plus sliplining the 42-inch main between "F" and "I" and reconstruction of the drainage improvements. This would reduce the length of the cut-over main to about 150 ft. This is presented as an option since an objective of Alternative "A" is to minimize sliplining and maximize abandonment of the existing interceptor.

The inlet shaft for either the lateral "I" or lateral "F" cut-over main would be on the slope between William Cannon Drive and the creek floodplain due to the limitation on tunnel radius. Some site grading would probably be required at the shaft site. As with shaft "A1", there are potential access routes from William Cannon Drive to the south or from Teewood Drive to the north. The route from the south requires some tree removal and grading for the drive. The route from Teewood Drive is on the same alignment as used by the construction equipment that built the drainage improvements, i.e., crossing Williamson Creek approximately 100 ft southeast of manhole "F" with a temporary stream crossing/baseflow diversion structure. Some clearing and grading would be required between the creek and the shaft site.

Inlet shaft #A2 would be a good site for a ventilation shaft during tunnel construction. It is 2,500 ft upstream from the main construction shaft at the diversion box and approximately 400 ft from the nearest neighbor.

West of the Creek Bend greenbelt, the Alternative "A" alignment enters undeveloped private property (Las Maderas Subdivision and 81 William Cannon Joint Venture Subdivision). The tunnel alignment curves to the north on a 1,000-ft radius in order to get into an existing drainage and public utility easement, avoiding the need for a tunnel easement under future land improvements. Inlet shaft **#A3** would be just southeast of manhole "K" on the 42-inch main. Manhole "K" receives flow from a 24-inch lateral from the south and from an 8-inch lateral crossing the creek from the north. A cut-over main from the inlet shaft would extend approximately 50 ft to a new manhole constructed on the 42-inch main immediately downstream of the existing manhole "K".

The tentative flowline of the relief interceptor tunnel would be approximately 32 ft deep at inlet shaft #A3. Boring B-4 was made in the vicinity of inlet shaft #A3, and boring B-3 was drilled about half-way between inlet #A2 and inlet #A3 approximately 150 ft south of the Alternative "A" alignment. Boring B-3 shows about 8 ft of sound Austin Chalk above the proposed tunnel crown, with about 12 ft at boring B-4 (assuming an 84-inch tunnel bore).

A possible access route to inlet shaft #A3 is off of William Cannon Drive along the existing easement for the 24-inch wastewater main. This is the route that the drill rig reported using to get to B-4. For use by construction traffic, portions of this route would require some grading and stabilization due to the slopes, possibly with temporary workspace for additional width. An alternate access route from the south might be along the east property line of the 81 William Cannon Joint Venture subdivision in a temporary access easement and then along the existing easement for the 42-inch interceptor. Either route would require some grading and tree and brush removal. More detailed site investigation and negotiations with the property owner would be necessary to settle on an access route to shaft #A3 from William Cannon. The properties have posted "For Sale" signs. Development of the sites prior to relief tunnel construction could eliminate the feasibility of access from William Cannon Drive.

The easiest route for construction traffic would be from the north with a stream crossing/baseflow diversion structure across Williamson Creek. The existing 8-inch sewer main from the north is located in City-owned property between Wagon Bend Drive right-of-way and manhole "K". The City owns a 20-ft wide strip of land between two existing houses that essentially forms an extension of Blackmule Drive, and the property between Wagon Crossing subdivision and Williamson Creek is City greenbelt. While this route provides the least off-road access to inlet shaft #A3, it has more public impact due to construction traffic through the residential neighborhood and between two houses.

Inlet shaft #A3 is 2,500 ft upstream from shaft #A2 and is a good candidate for a ventilation shaft during construction. It is approximately 300 to 350 ft from the nearest residence. As a mucking shaft, it could potentially lower the project cost by maintaining a tunnel advance rate controlled by the TBM rather

than muck removal. However, drawbacks as a mucking shaft are difficult haul conditions and additional workspace requirements.

Alternative "A" has inlet shaft #A4 and a proposed cut-over at manhole "L" on the 42-inch main. This cut-over pipe would carry flow from the upstream, sliplined 42-inch main and from an 8-inch lateral from the north. Similar to the shaft #A3 cut-over, a new manhole would be constructed on the 42-inch main and approximately 50 ft of cut-over main installed to the inlet shaft. Inlet shaft #A4 would provide slightly better spacing for ventilation shafts during construction than shaft #A3. Access to inlet shaft #A4 and manhole "L" would be from shaft #A3 along the existing easement for the 42-inch main. Access would require removal of trees and brush that have grown in the area since the interceptor was installed 37 years ago. Manhole "L" and the shaft and cut-over pipe are just inside City parkland, avoiding the need for a private easement at the shaft site.

If the cut-over was not installed at manhole "L", the 42-inch main would require sliplining between manholes "L" and "K". Access for construction equipment to manholes upstream from manhole "K" would still be required for the sliplining work, so avoiding an inlet shaft at manhole "L" does not eliminate environmental impacts upstream. However, the excavation for a shaft and cut-over main at manhole "L" would have more environmental impact than excavation for a sliplining insertion pit. With a cut-over at "L", the 42-inch main can be abandoned between manhole "L" and Pleasant Valley Road.

The next cut-over proposed for Alternative "A" is near manhole "S" north of Stassney, although there are three lateral mains along the 42-inch at manholes "N", "P", and "R". A cut-over for the 18-inch lateral "R" would be relatively easy with access south off of Stassney. However, the downstream laterals "P" and "N" carry relatively low flows. Without the big flow from lateral "R", the low flows from "N" and "P" would not produce adequate cleaning velocity in a sliplined 42-inch main, so either all three laterals should be cut over or none. This section at a bend in Williamson Creek has tight workspace between the creek and bluffs on the south and west and the Wagon Crossing neighborhood on the north and east. Clearing and grading will be required for access by construction equipment for either inlet shafts or sliplining. In some areas construction activity will occur within 100 ft of adjoining houses. Everything considered, sliplining the 42-inch should have less environmental and neighborhood impact than construction of three inlet shafts and cut-over mains. Furthermore, sliplining is classified as maintenance activity which is not subject to some of the environmental ordinance conditions. Construction of shafts and cut-over mains would have greater difficulty receiving permitting and approvals, e.g., a Planning Commission variance for construction in the Critical Water Quality Zone.

Alternative "A" includes an inlet shaft #A5 and cut-over main for lateral "S", which serves a 30-acre mobile home park located between Williamson Creek and Ponciana Drive. The 6-inch lateral has an inverted siphon under Williamson Creek to reach the 42-inch main. Since the 42-inch main upstream of

manhole "S" will be abandoned, it is necessary to cut-over lateral "S" because it does not deliver enough flow to produce cleaning velocities in a downstream, sliplined 42-inch main. The inlet shaft is in City greenbelt adjacent to an apartment complex and a multiplex theater. The cut-over main could extend directly from the inlet shaft to manhole "S" on the 42-inch main. An alternative cut-over could eliminate the existing 6-inch inverted siphon under the creek by boring a deeper line from the inlet shaft under the creek to a new manhole on the lateral on the east bank. Inlet shaft #A5 is approximately 2,600 ft upstream from inlet shaft #A4. The site should be allowed as a ventilation shaft during construction provided that adequate measures are taken for sound attenuation due to the proximity of the apartment complex.

Inlet shaft #A6 for the relief tunnel is proposed at manhole "U" on the 42-inch main approximately 400 ft east of the IH-35 east right-of-way line. The shaft would be in City greenbelt with construction access through the theater parking lot. Two cut-over mains would connect to the inlet shaft manhole. Approximately 100 ft of main would connect to manhole "U" to pick up flow in a 12-inch lateral coming from the north. Approximately 400 ft of 8-inch main is needed to intercept flow from the theater that currently goes to manhole "T". A private easement is needed for most of the theater cut-over main. These two cut-overs allow abandonment of the downstream 42-inch main.

Use of shaft #A6 as a mucking shaft could increase the tunnel contractor's efficiency and potentially lower the tunneling cost for Alternative "A". This site has good access from the northbound IH-35 frontage road, there is high voltage power in the area, and the site is not adjacent to a residential area. A temporary easement for a greater amount of workspace in the theater parking lot would be required to use #A6 as a mucking shaft.

Alternative "A" has a proposed inlet shaft #A7 at manhole "V"in order to cut over two 8-inch laterals that discharge to the 42-inch main from the north and the south at the IH-35 west right-of-way line. Within the last year, the property to the south has been developed as a shopping center, which has effectively eliminated construction access on gentle slopes from the south off of Stassney. An alternative access route is from the north off of Battle Bend Boulevard at its intersection with Presidio Road on the trail currently used by PARD and W&WW for access to the Williamson Creek greenbelt and existing interceptor west of IH-35. There are residences on the lots to the east and west of the trail entrance. The current steep slope of the trail into the greenbelt would probably require cut and fill for ramping suitable for heavy construction equipment. This route to the manhole "V" cut-over would also require a temporary stream crossing/ baseflow diversion structure across Williamson Creek for access to manhole "V" via the existing wastewater line easement along the flatter south bank of Williamson Creek. The inlet shaft would be located close to manhole "V" to minimize the length of cut-over mains. A temporary workspace easement would be needed around the shaft site. The tunnel alignment itself in this area is south of the narrow strip of greenbelt, which will require acquisition of a tunnel easement.

The Alternative "A" relief tunnel includes inlet shafts #A8 and #A9 in the greenbelt west of IH-35 for cut-overs of laterals "W" and "X1". The 8-inch lateral "W" connects to the existing 42-inch main just west of the aforementioned access trail into the greenbelt off of Battle Bend Boulevard. Manhole "X1" is approximately 900 ft west of manhole "W". Manhole "X1" was constructed on the 42-inch main in 1999 for connection of a 10-inch main serving the recent development to the south. The greenbelt has a large number of protected trees. Access to the work sites for the "W" and "X1" inlet shafts and cut-over mains would follow the existing trail which largely avoids the major trees. The inlet shafts would be near the existing manholes to minimize the length of cut-over mains. These two shaft sites are approximately 200 ft from the nearest residences.

The next cut-over for Alternative "A" is near manhole "Z", which is located on a narrow peninsula between Williamson Creek and a tributary. Manhole "Z" has a lateral 8-inch sewer from the west and is the break point between the 36- and 42-inch portions of the interceptor. The Williamson Creek Tunnel designed in 1986 and 1992 for CIP Project No. 237265 (mentioned in Section 1.1) tied into manhole "Z".

Due to very limited workspace near manhole "Z", the proposed inlet shaft #A10 for the "Z" cut-over is approximately 200 ft north of manhole "Z" at a PI in the 36-inch interceptor. This area is in City greenbelt. With the right construction sequencing, the cut-over main could be installed by removing the existing 36-inch pipe and replacing it with the cut-over main carrying flow northward to the inlet shaft. A key factor is that the proposed upstream cut-over at manhole "BB" be implemented first in order to remove flow from the 36-inch pipe and allow the remove-and-replace operation for the cut-over main. The remove-and-replace approach is intended to minimize the environmental disruption. Access to the shaft site would be off of Wasson Road in an access easement crossing land that is currently vacant in order to reach the 36-inch main and use its existing easement for access to the shaft.

Upstream of the "Z" cut-over, the Alternative "A" alignment follows the Williamson Creek Tunnel alignment established in the 1986/1992 design. The City obtained a permanent wastewater easement for this tunnel route which will be used for the current project.

The next cutover from the existing interceptor is at manhole "BB". This cutover intercepts flow in the 36-inch main from laterals at manholes "BB", "DD", "EE", "FF", "HH", and "II". The 36-inch main between manholes "BB" and "II", with length of 2,000 ft, would be sliplined and remain in service. The inlet shaft #A11 is proposed adjacent to Wasson Road. Approximately 600 ft of cutover main is required, installed mostly by bore and jack due to depth. A potential cutover alignment shown in Figure 7-1 requires easements for three properties. The easement for the cutover main for the Wasson Road Addition property could provide permanent access to the downstream tunnel inlet shaft "Z" if the existing easement is a tunnel easement which does not include surface access rights. The 36-inch main at "BB" is

south of Williamson Creek, so a creek crossing with flow diversion is not required for access to the 36-inch main for construction of the "BB" tie-in.

The tunnel construction terminates at manhole "KK" with a tie-in to the existing 48-inch interceptor. This feature is common to all of the alternatives. This will be a major work site with a large shaft. Activity during construction will include removal of the TBM and associated ventilation and mucking equipment, and installation of the final carrier pipe for the tunnel. Due to elevation difference between the upstream 48-inch interceptor and the tunnel, the shaft will be finished as an energy dissipating inlet structure.

The work site is located in City greenbelt that extends westward to South First Street. The 1986/1992 Williamson Creek Tunnel plans indicate a 20-ft permanent easement for the 48-inch interceptor that would be used for access during construction. The easement extends 1,300 ft to the shaft site from the intersection of South First Street and Heartwood Drive. The shaft site and access route are 200 to 400 ft from the nearest residences, except near the entrance to the greenbelt off of Heartwood. The work area and access route are located in the 100-year flood plain and Critical Water Quality Zone. The easement/access route crosses the wide streambed on a diagonal near South First Street. Extensive erosion and sedimentation control design and implementation will be required in order to mitigate water quality impacts.

An alternative access route to the terminal shaft off of South Congress would avoid a creek crossing. However, it would require an access easement from one or more businesses on the west side of South Congress with the construction traffic through the property causing major disruption. The slopes for access from the east would also require some cut and fill and tree removal for a ramp to the shaft site that would be suitable for the construction equipment. Due to these complications, access from South First Street has better feasibility provided that proper erosion and sedimentation control is assured.

7.3 ALTERNATIVE "B"

The alignment of Alternative "B" is similar to Alternative "A" except that the tunnel is straight between manholes. Some concern had been expressed about problems with TBM's being able to maintain line and grade in tunnels with curves, as mentioned in the Appendix D case histories. The objective of Alternative "B" is a tunnel without curves, and also further reduction in the amount of sliplining (i.e., abandon more of the existing interceptor) by installing more cut-overs. Alternative "B" consists of 17,200 ft of tunnel, 16 inlet shafts for 16 cut-over mains with total length of approximately 3,200 ft, and 2,230 ft of sliplining. The tunnel alignment, cut-overs, and sliplining for Alternative "B" are shown in Figure 7-2. Additional cut-overs at manholes "N", "P", and "R" eliminate 1,900 ft of sliplining in the Wagon Crossing area, but each cut-over requires an additional tunnel inlet shaft. Two more shafts are required at tunnel bends, with the one near manhole "D" also serving as an inlet manhole.

Having no curves in the alignment raises the possibility of excavating the tunnel by the bore and jack method instead of a steerable, self-propelled tunnel boring machine. Equipment cost for boring and jacking is less than for a TBM with its associated mucking and ventilation equipment. In the bore and jack method, a jacking rig in a pit jacks casing pipe to advance a cutterhead. An augar in the casing removes the muck and provides torque for the cutterhead. The augar and jacked casing pipe require a straight alignment, which limits the bore and jack method to Alternative "B". However, the length of a bore and jack tunnel between pits is usually limited to well under 1,000 ft on account of increasing load on the jacking unit with greater length. The shorter length is also required because bore and jack can not maintain line and grade as well as TBM tunneling. Therefore, the Alternative "B" tunnel excavated by bore and jack instead of TBM would require more tunnel shafts, and the cost of extra shafts just for jacking is offsetting to the costs savings of the bore and jack equipment. Extra shafts also increase the environmental and public impacts as discussed below.

Each shaft for Alternative "B" would be larger diameter than the inlet shafts for Alternative "A" because the shafts are needed to change the direction of the tunnel by reorienting the TBM. This would be done with a crane to lift and rotate the TBM. Shafts for Alternative "B" would be on the order of 20 ft in diameter (depending on TBM design), whereas inlet/ventilation shafts on the other tunnel alternatives could be about 5 to 10 ft in diameter. Furthermore, since the muck train could not negotiate point bends, each shaft would be used for mucking.

Environmental impacts of Alternative "B" would be much greater than for Alternative "A" due to excavation of larger shafts, muck hauling from each shaft, and access to additional shafts that are in more sensitive areas (e.g., near manholes "N", "P", "R", and "Y"). Access for construction would have greater impacts than access for sliplining the existing main in more or less the same area. Construction traffic through the Wagon Crossing neighborhood and construction shafts within 100 ft of homes would produce more adverse public impact with Alternative "B".

Alternative "B" is not pursued further as a favorable alternative due to the environmental impacts, public impacts, limited reduction in the amount of sliplining given the increase in the number and size of tunnel shafts, and greater ability of modern TBM's to accurately maintain line and grade for tunnels with curves.

7.4 ALTERNATIVE "C"

The objective of Alternative "C" is to lower the total project cost by reducing the quantity of the highest cost item, the tunnel. The length of tunnel is reduced by diverging from the existing meandering creek and interceptor alignment for a straight run between approximately manholes "O" and "Y". A straighter tunnel alignment is also achieved by crossing to the north of Williamson Creek in the

Creek Bend area west of Pleasant Valley Road. The tunnel alignment, cut-overs, and sliplining for Alternative "C" are shown in Figure 7-3. Alternative "C" consists of 16,510 ft of tunnel, 6 inlet shafts for 7 cut-over mains with total length of approximately 1,420 ft, and 8,870 ft of sliplining (or 45% of the total interceptor length). Approximately 15% of the tunnel alignment is in curves.

Shortening the tunnel length by crossing to the north side of Williamson Creek near Pleasant Valley Road prevents a tunnel tie-in to the existing 42-inch stub on the diversion box. The depth is too shallow to tunnel under the creek, and open cutting across the creek has been eliminated as a feasible option based on discussions with City environmental reviewers. The 84-inch tunnel that connects to the diversion box is almost 8 ft deeper than the 42-inch stub, but there is inadequate workspace at the diversion box for a tie-in to the 84-inch pipe due to its proximity to the creek. Therefore, the Alternative "C" tunnel would start at a shaft approximately 600 ft east of the diversion box for a tie-in directly to the 84-inch tunnel in an area with ample workspace for the main tunnel shaft. The location is within the bounds of the City's former Williamson Creek wastewater treatment plant. The site is accessed off of William Cannon Drive east of Pleasant Valley Road. High voltage electricity is readily available for the tunneling operations, and the site is remote from residential areas.

The 84-inch tunnel invert at the Alternative "C" tie-in point is about elevation 500 ft. At Pleasant Valley Road the Alternative "C" tunnel would be about 15 ft deeper than the diversion box. Boring B-1 indicates that the tunnel would be in rock, avoiding the soft ground tunneling problems of Alternatives "A" and "B". With the added depth the tunnel crosses under Williamson Creek in the Creek Bend area.

An alternative that will investigated further, in conjunction with the Creek Bend Drainage Improvements Phase 2, is an inlet shaft on the tunnel in the vicinity of the diversion box. A cut-over line from the inlet shaft to the 42-inch main just upstream of the box would allow the diversion box to be abandoned. The 42-inch main between the cut-over and the sewer lines in Pleasant Valley Road would be sliplined and remain in service. Grading around the box in the preliminary plans for the drainage improvements leaves the box standing as an island in the floodplain. The diversion box has serious hydraulic constraints and maintenance problems under existing conditions, which might well be eliminated at the very reduced remaining flows in Alternative "C", but the box may still need some modifications and rehab if kept in service (see Section 8.5.1). The alignment of the Alternative "C" tunnel and the location of a potential inlet shaft should be coordinated closely with the Creek Bend drainage improvements project. One concern is plans for drilled piers to support two or three new spans to the Pleasant Valley Road bridge as part of the drainage project. There could potentially be some interference between the relief tunnel and the bridge piers. Another concern is location of an inlet shaft in relation to bank stabilization improvements. For the time being, Alternative "C" does not include an inlet shaft in this area.

Inlet shaft #C1 is in City property south of Teewood Drive adjacent to the 8-inch lateral to manhole "D". Shaft #C1 would have a short open-cut cut-over main to intercept the flow in lateral "D" and a 400-ft bore and jack cut-over main to intercept the lateral to manhole "E". These intercepts allow abandonment of the 42-inch main down to manhole "A" and avoid sliplining. (The 42-inch would be sliplined 200 ft between "A" and the diversion box due to the two laterals discharging to manhole "A".) Access to the #C1 shaft site would be off Teewood Drive through the Creek Bend neighborhood via Dove Springs Drive from Pleasant Valley Road. The main work at the shaft would be north of Williamson Creek, but the lateral "E" cut-over would require access with a creek crossing/baseflow diversion structure and repair of resulting damage to the recent drainage improvements along Williamson Creek.

Inlet shaft #C2 allows cut-over of the 12-inch lateral going to manhole "I". The shaft site is south of the creek. The proposed cut-over main would be bored approximately 250 ft under the creek and 42-inch main from the shaft to the north side of the creek for a tie-in to a manhole on the 12-inch main. Access to the tie-in manhole would be south from Dove Springs Drive in the existing easement. Access to the shaft site would require a creek crossing/baseflow diversion at the creek coming off of Teewood Drive. Due to the alignment of the cut-over main, the proposed shaft site is just west of the Creek Bend greenbelt in Las Maderas subdivision property. Access to the shaft would require some clearing of trees and brush. #C2 would be a good site for a tunnel ventilation shaft during construction if so desired by the contractor. It is approximately 3,200 ft from the main construction shaft and 350 ft from the nearest residence.

From shaft #C2 the Alternative "C" tunnel extends 2,300 ft across Las Maderas and 81 William Cannon Joint Venture subdivisions in existing drainage and public utility easements to inlet shaft #C3 near manhole "K" on the 42-inch main. Shaft #C3 is for the cut-over of 24-inch and 8-inch laterals to manhole "K". Inlet shaft #C4 approximately 800 ft upstream of #C3 allows cut-over of the 8-inch lateral to manhole "L". These two shaft and cut-over sites are in the same locations as proposed for Alternative "A". See Section 7.2 for discussion of cut-over requirements, access alternatives and impacts, and provision for tunnel ventilation.

From this point the Alternative "C" tunnel alignment diverges from the 42-inch main and takes a heading more or less straight for manhole "Z" on the 42-inch main west of Battle Bend subdivision near Wasson Road. This alignment requires tunnel easements for boring the tunnel beneath three apartment complexes and two shopping centers. Tunnel depth below ground level would vary between approximately 50 and 120 ft in the developed areas. No borings were made on this alignment, but the soil overburden should be shallow. Good tunneling conditions in sound Austin Chalk would be expected. If this alignment is selected for design and construction, additional borings will be made to provide the required geotechnical information.

A shaft site should be provided for the contractor in the vicinity of IH-35 and Stassney Lane, e.g., in the open area at the southeast corner of the intersection. The contractor may elect to use the shaft for ventilation and emergency access. If sufficient workspace can be acquired from right-of-way and adjoining property for a construction shaft for mucking and pipe installation, the project could receive lower bids due to shorter construction time. This site is approximately 9,000 ft from the main construction shaft at the old Williamson Creek treatment plant, and just past half-way to the upstream terminus. High voltage power lines are available in the area, and the hauling would not impact residential neighborhoods.

The next inlet shaft, #C5, would provide a cut-over for flow in the 8-inch lateral to manhole "Z" on the 42-inch main. From this location upstream, Alternative "C" shares the same alignment as Alternatives "A" and "B" in order to utilize the existing easement obtained for the 1986 Williamson Creek Tunnel project. There are cut-over mains to manholes "Z" and "BB" and a tie-in to the existing 48-inch interceptor at manhole "KK". The existing 36-inch interceptor between manholes "BB" and "II" would be sliplined. Section 7.2 for Alternative "A" describes the cut-over mains, access alternatives, environmental impacts, sliplining, and the upstream junction with the 48-inch main, which is also applicable for Alternative "C".

The shortcut alignment of Alternative "C" between manholes "L" and "Z" requires sliplining for the existing 42-inch interceptor between manholes "L" and "X1". This section would continue to receive flows from ten laterals to manholes "N", "P", "R", "S", "T", "U", "V", "W", and "X1". The length of sliplining for this section is 6,660 ft.

7.5 ALTERNATIVE "D"

The intent of Alternative "D" is further reduction in tunneling cost from Alternative "C" by means of a more direct route and shorter length. This places the tunnel north of Williamson Creek for most of the section between Pleasant Valley Road and IH-35, crossing the Creek Bend, Dove Springs, and Wagon Crossing neighborhoods. West of IH-35 the alignment crosses the Battle Bend neighborhood in order to shorten the length. For Alternative "D" the tunnel will be bored beneath a number of single family lots. Tunnel easements will have to be obtained from all of the affected property owners.

There can be a number of variations in the alignment east of IH-35 that have trade-offs between the amount in street right-of-way versus the number of tunnel easements, the amount in curves, potential ventilation shaft locations, and the number of cut-overs and amount of sliplining. Two variations are presented herein. Alignment "D1" is the shorter, more direct route, but it requires more easements. Alignment "D2" is approximately 600 ft south with fewer easements, and it allows two cut-overs that eliminate some sliplining for the existing 42-inch interceptor, but the tunnel is approximately 330 feet longer than "D1". The alignment is the same west of IH-35. The street layout in the Battle Bend area does not

provide opportunities to align the tunnel with street right-of-way. In any case the tunnel starts, as in Alternative "C", approximately 600 ft east of the existing diversion box so that it can tie directly into the 84-inch tunnel and obtain sufficient depth to cross under Williamson Creek west of Pleasant Valley Road.

Alternative "D1" has a total tunnel length of 16,045 ft, four cut-over mains to two inlet shafts, 13,060 ft of sliplining for the existing interceptor, approximately 60 tunnel easements for single-family residential lots, and 4 easements for commercial and multi-family properties. The alignment utilizes approximately 3,400 ft of Dovehill Drive and Wagon Crossing Path street right-of-way to reduce the number of easements. The alignment in residential neighborhoods presents a problem for obtaining the recommended ±2,500 ft spacing for ventilation shafts. Ventilation shafts would still be possible at certain street intersections, such as Brassiewood at Softwood Drive or Dove Hill Drive at Wagon Crossing Path, as shown in Figure 7-4 . At these locations, closing the side street could provide workspace for drilling the shaft. The shaft work sites would not block driveways of adjacent houses, however, there would be associated traffic and noise impacts on the neighborhood.

Alternative "D1" is removed from the 42-inch interceptor to the extent that there would not be any cut-overs to the tunnel east of IH-35. There is a potential cut-over at manhole "R" on the 42-inch but this is not recommended because (1) only 400 ft of the 42-inch downstream to manhole "P" would be abandoned, and (2) manhole "P" has a very small tributary flow which, without the upstream flow, would result in very low velocity in the sliplined 42-inch main downstream of manhole "P". A ventilation shaft for the tunnel would be needed in the vicinity of manhole "R" near Stassney Lane in keeping with the ±2,500 ft spacing.

The Alternative "D2" alignment, about 600 ft south of the "D1" alignment but still north of Williamson Creek, uses approximately 3,300 ft of Creek Bend Drive and Wagon Crossing Drive right-of-way. Alternative "D2" has a total tunnel length of 16,375 ft, six cut-over mains to four inlet shafts, 9,600 ft of sliplining for the existing interceptor, approximately 32 tunnel easements for single-family residential lots, and 6 easements for commercial and multi-family properties.

Alternative "D2" is close enough to the 42-inch main to permit two cut-overs. With an inlet shaft near manhole "I" (in Las Maderas subdivision), a short open-cut cut-over main could intercept the flow in the 12-inch main going to manhole "I". Access to the shaft site would be through the existing easement south of Dove Springs Drive. A second cut-over main would be possible from the tunnel to the 42-inch interceptor just downstream of manhole "K". This cut-over would divert the flows in the upstream 42-inch pipe and the 24-inch and 8-inch laterals to manhole "K". The tunnel inlet shaft would be in Blackmule Drive at Wagon Crossing Trail. Blackmule Drive would be closed between Wagon Crossing and Dove Springs Drive for workspace. The adjoining lots do not have driveways on Blackmule, but the work activity would have noise and traffic impacts on the surrounding neighborhood. Approximately 400 ft of cut-over

main would be installed by bore and jack from the inlet shaft to cross the creek. Access for constructing the cut-over manhole on the 42-inch main is possible through the existing 20-ft wide strip of City property between residential lots south of Wagon Bend Trail opposite Blackmule Drive. A temporary stream crossing/baseflow diversion structure in the greenbelt would be required to reach the 42-inch main. A new manhole at the end of the cut-over main would be in the 81 William Cannon Joint Venture subdivision within the 100-year flood plain. These two inlet shafts could be used for tunnel ventilation during construction if so desired by the contractor. The two cut-overs would allow 3,460 ft of the 42-inch main to be abandoned (avoiding sliplining) between manholes "K" and "E", which is not possible with Alternative "D1". However, abandoning this section of the 42-inch interceptor, with the relief tunnel north of Williamson Creek, could present problems for the Las Maderas subdivision to obtain gravity wastewater service for future development

Besides "D1" and "D2", other alignment variations would be possible using portions of other streets in the area, namely, Stassney Lane, Dovewood Drive, Covered Wagon Pass, Dove Springs Drive, and Brassiewood Drive. Selection of a final alignment for Alternative "D" could depend on the route for which all of the necessary easements could be obtained with the least expense and legal proceedings.

North of Stassney Lane, Alternative "D" has a single alignment. An inlet shaft with two cut-over mains is proposed in the greenbelt south of the Battle Bend neighborhood. A cut-over main approximately 500 ft in length would be installed by bore and jack between the inlet shaft and the 8-inch lateral to manhole "W". Another cut-over main would be bored from the shaft approximately 400 ft to manhole "X1" for flow in the new 12-inch lateral to the south. There would be no flow in the 42-inch main at manhole "X1" due to an upstream cut-over. Access for construction of this inlet shaft and the cut-over mains would be from Battle Bend Boulevard at Presidio Road via the trail currently used by PARD and W&WW for access to the Williamson Creek greenbelt and existing interceptor west of IH-35. This access route and associated impacts are discussed in Section 7.2 for the "W" and "X1" cut-overs for Alternative "A".

From the "W"/"X1" inlet shaft, the Alternative "D" alignment crosses Battle Bend subdivision. Very little of the alignment can fall in street right-of-way. Approximately 20 single-family lots would be required to grant tunnel easements. Tunnel depth of cover in this reach would range from approximately 45 ft for lots near the creek to a maximum of about 80 ft.

Unlike Alternatives "A", "B", and "C", the Alternative "D" alignment does not merge into the 1986 Williamson Creek Tunnel alignment (and easement) at manhole "Z" on the 42-inch main, but rather at Wasson Road, where there will be an inlet shaft for two cut-over mains. An inlet shaft at this location is also part of Alternatives "A", "B", and "C". Since the Alternative "D" alignment is not in proximity to manhole "Z", flow in the 8-inch "Z" lateral would be intercepted by a cut-over main constructed south

in Wasson Road from the inlet shaft. The cut-over main would be approximately 670 ft in length, with a maximum depth of about 30 ft (it has to cross a minor ridge line). There are two lots on the east side of Wasson Road that have service taps on the 8-inch lateral that are downstream of the cut-over point. These lots would require installation of grinder pumps discharging to the sewer main in Wasson Road.

Another cut-over main from the Wasson Road inlet shaft to manhole "BB" would intercept flow in the 36-inch main and pick up an 8-inch lateral to "BB". This cut-over main (approximately 530 ft in length, needing three easements, and installed by bore and jack) is discussed in Section 7.2 for Alternative "A".

With cut-over of the flows in the laterals to manholes "W", "X1", and "Z" and the 36-inch main at manhole "BB", 4,650 ft of the existing 36-/42-inch interceptor can be abandoned, to avoid sliplining, between manhole "BB" and manhole "V" on the west side of IH-35 at Williamson Creek.

At the Wasson Road shaft, Alternative "D" gets on the alignment of 1986 Williamson Creek Tunnel and uses the existing easement. Of the 3,760 ft of 36-inch interceptor upstream of manhole "BB", 2,000 ft would be sliplined up to manhole "II" for flows from five lateral mains, and 1,760 ft between manholes "II" and "KK" could be abandoned. The tie-in of the tunnel to the existing 48-inch interceptor at manhole "KK" is described in Section 7.2 for Alternative "A" with regard to structures, access alternatives, and associated impacts.

In summary for Alternative "D", the more direct route reduces the tunnel length and cost, but it requires a significant number of tunnel easements for boring under single family homes. Potential variations in the alignment east of IH-35 affect the number of easements, the number of cut-overs, the amount of sliplining required for the existing interceptor, and the overall tunnel length.

7.6 ALTERNATIVE "E"

Alternative "E" provides the shortest possible tunnel, a straight line between the downstream 84-inch tunnel and the upstream 48-inch interceptor. This alternative was evaluated to determine if the shortest tunnel produced the lowest total project cost, including costs for sliplining and cut-overs. The tunnel length as shown in Figure 7-5 is 15,780 ft, which is 91% of the Alternative "A" tunnel length, approximately 96% of Alternative "C" and "D2" tunnel lengths, and 98% of Alternative "D1" tunnel length.

This alignment is north of the existing interceptor by as much as 1,600 ft in a couple of locations. But the interceptor crosses the "E" alignment at IH-35 between manholes "U" and "V" and also between manholes "S" and "T". The only feasible location for an inlet shaft to divert flow from the

interceptor to the relief tunnel is near manhole "T" or "U", which would allow 1,550 ft of the interceptor to be abandoned between manholes "U" and "S". Flows in the existing 12-inch lateral to manhole "U" and the 8-inch lateral to manhole "T" could be diverted to the inlet shaft with approximately 500 to 600 ft total length of cut-over main depending on the exact inlet shaft location. However, the cutover is not recommended in keeping with the project objective of maintaining good hydraulic conditions in the rehab'ed 42-inch main. Low flows in lateral "S" would allow sediment deposition in the sliplined 42-inch between manholes "S" and "R", with resulting odor problems and maintenance requirements. Without this cutover, 17,710 ft (or 91%) of the existing 36-/42-inch interceptor would be sliplined.

Workspace for a shaft should be provided for project bidders near manhole "U" even if it is not used as an inlet shaft. A shaft available for ventilation, emergency access, and even mucking could increase the tunnel contractor's productivity and produce lower construction bids. Section 7.2 for Alternative "A" mentions good conditions for access, availability of high voltage power, and isolation from residential areas in regard to a mucking and pipe installation shaft at this location. This area is near the midpoint of the Alternative "E" tunnel alignment.

The alignment of Alternative "E" through residential areas presents problems with obtaining sites for potential ventilation shafts. Target sites would be side streets with light traffic and with sufficient workspace in the right-of-way to avoid blocking a driveway. Significant traffic and noise impacts on the public would result from construction activity to drill the shaft and operation of the site during tunnel construction.

Alternative "E" avoids the public and environmental impacts and construction costs of the cut-over mains that are used in the other alternatives. However, sliplining almost the entire length of the 36-/42-inch interceptor will have more environmental impact. Especially impacted areas with only Alternative "E" are the Creek Bend area where recent drainage improvements will need to be reconstructed (following excavation for sliplining insertion pits) and the greenbelt south and west of the Battle Bend neighborhood.

As mentioned above, much of the Alternative "E" tunnel alignment is located in residential areas. Approximately 115 tunnel easements would be required from single-family residential lot owners as shown in Figure 7-5. Tunnel easements would be required from 8 commercial and multi-family properties. Modifying the alignment for portions to fall in street right-of-way would reduce the number of easements somewhat, but this measure increases the tunnel length and violates the objective of this alternative. There are several significant problems with obtaining a large number of easements from homeowners:

- Time and expense for City staff (or a private contractor for easement acquisition) to identify and contact the property owner (who may be different from the occupant), negotiate the easement conditions, and prepare and record the easement documents.
- Potential cost of owners' demands for compensation, or possibly the cost of condemnation proceedings to obtain the easement.
- Liability for structural damage claimed by the owner as being caused by the tunneling (e.g., cracks in the walls or foundation caused by vibration), which could be difficult to prove was not a pre-existing condition.
- Delay of the project schedule caused by only a few reluctant owners out of the total number of properties.

For Alternative "E", and to a lesser extent Alternative "D", the cost of easement acquisition from individual homeowners can become a significant factor in the total project cost.

7.7 COMPARISON OF ALTERNATIVES

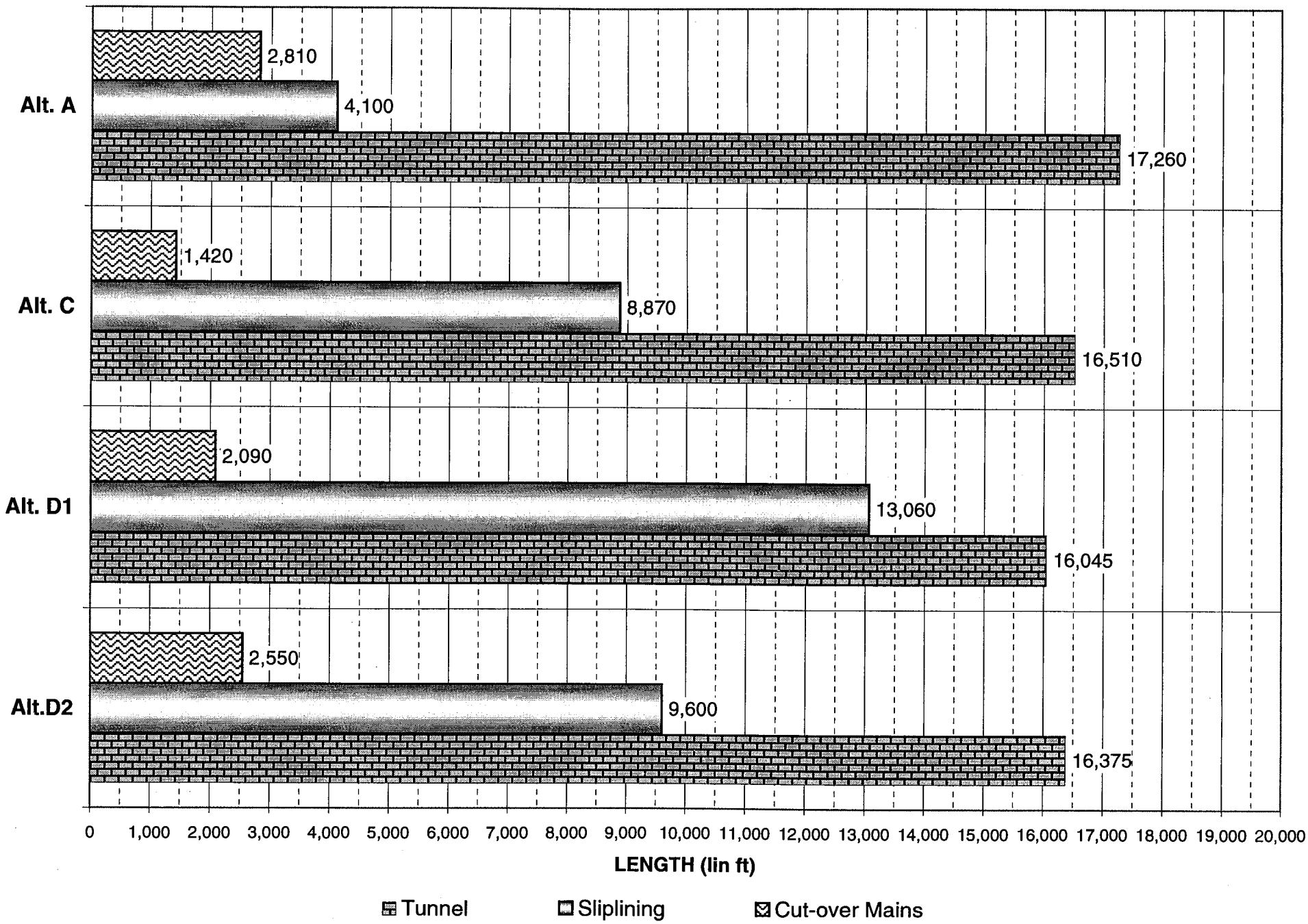
Figure 7-6 is a composite map of all the Alternatives' alignments. Alternatives B and E are not developed further due to the problems with these alternatives discussed in Sections 7.3 and 7.6. For Alternatives A, C, D1, and D2, Table 7-1 presents a comparison of the lengths of tunnel, sliplining, and cut-over mains, as well as the number of tunnel shafts, cut-over mains, and easements. The length comparisons are presented graphically in Figure 7-7.

TABLE 7-1

COMPARISON OF RELIEF TUNNEL ALTERNATIVES

	Alt. A	Alt. C	Alt. D1	Alt. D2
Tunnel (LF)	17,260	16,510	16,045	16,375
# of Tunnel Shafts	13	8	7	7
Cut-over Mains (LF)	2,810	1,420	2,090	2,550
# of Cut-over Mains	13	7	4	6
SlipLining (LF)	4,100	8,870	13,060	9,600
# of SFR Easements	0	0	60±	32±
# of MF/Com Easements	6±	8±	4±	6±

FIGURE 7-7. TUNNEL, SLIPLINING AND CUT-OVER MAIN QUANTITIES



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4/10/01

8.0 PRELIMINARY DESIGN OF RELIEF ALTERNATIVES

8.1 DESIGN FLOWS FOR IMPROVEMENTS

The design flows for the components of each alternative vary on account of the different cut-over connections. For each Alternative, Table 8-1 develops the design flows for each section of tunnel and sliplining and for each cut-over main. These design flows are derived from the data for wastewater flows and I/I flows that are contained in Table 4-3. Note that Scenario "B" with Lost Creek MUD is used since the MUD's flows are almost inconsequential in the total flow at the lower end of the Williamson Creek interceptor. Note also that the new laterals to manholes "T" and "X1" are treated separately for preliminary design since the small service areas for these laterals are not included in the disaggregation of the City's Node Service Area data to the project subbasins.

8.2 RELIEF INTERCEPTOR TUNNEL

The design flow at the upper end of the relief tunnel for all Alternatives is 32,930 gpm, which corresponds to a required capacity of approximately 41,200 gpm based on the criteria for q/Q not to exceed 80%. At the lower end the design flows are 39,312 gpm for Alternatives "A" and "C" and slightly less, 38,750 gpm, for "D2". The corresponding capacity requirements are approximately 49,200 gpm and 48,500 gpm, respectively. The capacity requirement for the lower half of Alternative "D1" is 44,400 gpm (35,522 gpm design flow) since "D1" has no flow cut-overs to the relief tunnel east of IH-35.

Obtaining the pipe capacity needed for these design flows at the lower end of the tunnel requires a 60-inch pipe at 0.15% to 0.18% slope or a 66-inch pipe at 0.09% to 0.11%. Considering that the flatter slopes provide some slight benefit in increasing the amount of rock cover over the tunnel at the lower end, a 66-inch pipe is recommended for the tunnel carrier pipe size. A steeper slope would be possible further west where the rock elevations are higher, but it is assumed that the same pipe size will be used for the complete tunnel length.

It is expected that the tunnel contractor will use a TBM that will excavate a tunnel bore about 8 feet in diameter. Smaller tunnels can create more difficult working conditions that reduce productivity and are not cost effective, due to less space for workers, muck train, ventilation duct, power cables, dewatering pipe, etc. Also, there is a greater availability of TBMs in the 8 to 10 ft diameter range.

Figure 8-1 shows the ground profile along Alternative "C" with a 96-inch tunnel and a 66-inch carrier pipe at 0.11% slope. This figure shows about 12 ft of rock over the top of the tunnel in the vicinity of Pleasant Valley Road, although it should be noted that boring B-1 at the diversion box is about

TABLE 8-1. DESIGN FLOWS FOR PROJECT IMPROVEMENTS

Alternative A																					
			Relief Tunnel					Cut-Over Lines						SlipLining							
	Added Avg Flow (gpm)	Added I/I Flow (gpm)	Tunnl MH	Cum Avg Q	Cum I/I	Tunnel Design Flow	C/O to Tunnel	Cum Avg Q	Cum I/I	C/O Design Flow	Vel	Calc Size	Pipe Size	Cum Avg Q	Cum I/I	Design Flow	Minimum Capacity	Pipe slope	Calc S'Liner pipe size	Use slipliner size (ID)	Vel @ Peak Dry Weather Flow
LostCk MUD pumpover	254	846																			
BartonCk pumpover	671	3,679																			
KK	5,597	15,213	A-12	6,522	19,738	32,929															
II	75	118		6,522	19,738	32,929								75	118	403	537	0.30%	10.0	18	1.8
HH	307	441		6,522	19,738	32,929								383	559	1,785	2,380	0.30%	17.5	27	2.6
FF	27	54		6,522	19,738	32,929								410	613	1,915	2,553	0.30%	17.9	27	2.6
EE	27	54		6,522	19,738	32,929								437	666	2,044	2,725	0.30%	18.4	27	2.7
BB	69	135	A-11	7,028	20,539	34,565	1	506	802	2,364	4	17.94	18								
Z	112	260	A-10	7,140	20,799	35,009	1	112	260	671	4	9.56	10								
W	126	307	A-9	7,267	21,106	35,522	1	126	307	764	4	10.20	12								
V	91	218	A-8	7,358	21,324	35,888	1	91	218	556	4	8.70	10								
U	80	170	A-7	7,438	21,493	36,188	1	80	170	470	4	8.00	10								
S	28	69	A-6	7,466	21,562	36,303	1	28	69	183	4	5.00	8								
R	247	509		7,466	21,562	36,303								247	509	1,345	1,793	0.28%	15.9	27	2.3
P	13	26		7,466	21,562	36,303								261	535	1,410	1,880	0.24%	16.7	27	2.2
N	63	123		7,466	21,562	36,303								324	658	1,718	2,291	0.24%	17.9	27	2.3
L	41	53	A-5	7,831	22,273	37,603	1	365	711	1,887	4	16.03	18								
K	53	106	A-4	7,883	22,379	37,794	1	53	106	310	4	6.49	8								
I	284	501	A-3	8,167	22,880	38,750	1	284	501	1,445	4	14.03	15								
E	139	290	A-2	8,320	23,199	39,312	1	139	290	789	4	10.37	12								
D	14	29		8,320	23,199	39,312	1	14	29	85	4	3.40	8								
A	545	1,328		8,320	23,199	39,312								545	1,328	2,995	3,994	0.28%	21.5	36	2.7

TABLE 8-1. DESIGN FLOWS FOR PROJECT IMPROVEMENTS

Alternative C																						
			Relief Tunnel					Cut-Over Lines						SlipLining							
	Added Avg Flow (gpm)	Added I/I Flow (gpm)	Tunnl MH	Cum Avg Q	Cum I/I	Tunnel Design Flow	C/O to Tunnel	Cum Avg Q	Cum I/I	C/O Design Flow	Vel	Calc Size	Pipe Size	Cum Avg Q	Cum I/I	SlipLiner Design Flow	Minimum Capacity	Pipe slope	Calc S'Liner pipe size	Use slipliner size (ID)	Vel @ Peak Dry Weather Flow
LostCk MUD pumpover	254	846																			
BartonCk pumpover	671	3,679																			
KK	5,597	15,213	C-10	6,522	19,738	32,929															
II	75	118		6,522	19,738	32,929								75	118	403	537	0.30%	10.0	18	1.8
HH	307	441		6,522	19,738	32,929								383	559	1,785	2,380	0.30%	17.5	27	2.6
FF	27	54		6,522	19,738	32,929								410	613	1,915	2,553	0.30%	17.9	27	2.6
EE	27	54		6,522	19,738	32,929								437	666	2,044	2,725	0.30%	18.4	27	2.7
BB	69	135	C-9	7,028	20,539	34,565	1	506	802	2,364	5	16.05	18								
Z	112	260	C-8	7,140	20,799	35,009	1	112	260	671	5	8.55	10								
W	126	307		7,140	20,799	35,009								126	307	764	1,019	0.18%	14.0	18	1.7
V	91	218		7,140	20,799	35,009								217	525	1,270	1,693	0.18%	16.9	18	2.0
U	80	170		7,140	20,799	35,009								297	695	1,678	2,237	0.18%	18.8	21	2.1
S	28	69		7,140	20,799	35,009								326	763	1,829	2,439	0.28%	17.8	21	2.5
R	247	509		7,140	20,799	35,009								573	1,272	3,013	4,018	0.24%	22.2	33	2.6
P	13	26		7,140	20,799	35,009								586	1,298	3,073	4,098	0.24%	22.3	33	2.6
N	63	123		7,140	20,799	35,009								650	1,421	3,360	4,480	0.24%	23.1	33	2.7
L	41	53	C-5	7,831	22,273	37,603	1	690	1,474	3,516	5	19.57	21								
K	53	106	C-4	7,883	22,379	37,794	1	53	106	310	5	5.81	8								
I	284	501	C-3	8,167	22,880	38,750	1	284	501	1,445	5	12.55	15								
E	139	290	C-2	8,320	23,199	39,312	1	139	290	789	5	9.27	10								
D	14	29		8,320	23,199	39,312	1	14	29	85	5	3.04	8								
A	545	1,328		8,320	23,199	39,312								545	1,328	2,995	3,994	0.28%	21.5	36	2.7

TABLE 8-1. DESIGN FLOWS FOR PROJECT IMPROVEMENTS

Alternative D1																						
			Relief Tunnel					Cut-Over Lines						SlipLining							
	Added Avg Flow (gpm)	Added I/I Flow (gpm)	Tunnl MH	Cum Avg Q	Cum I/I	Tunnel Design Flow	C/O to Tunnel	Cum Avg Q	Cum I/I	C/O Design Flow	Vel	Calc Size	Pipe Size	Cum Avg Q	Cum I/I	SlipLiner Design Flow	Minimum Capacity	Pipe slope	Calc S'Liner pipe size	Use slipliner size (ID)	Vel @ Peak Dry Weather Flow
LostCk MUD pumpover	254	846																			
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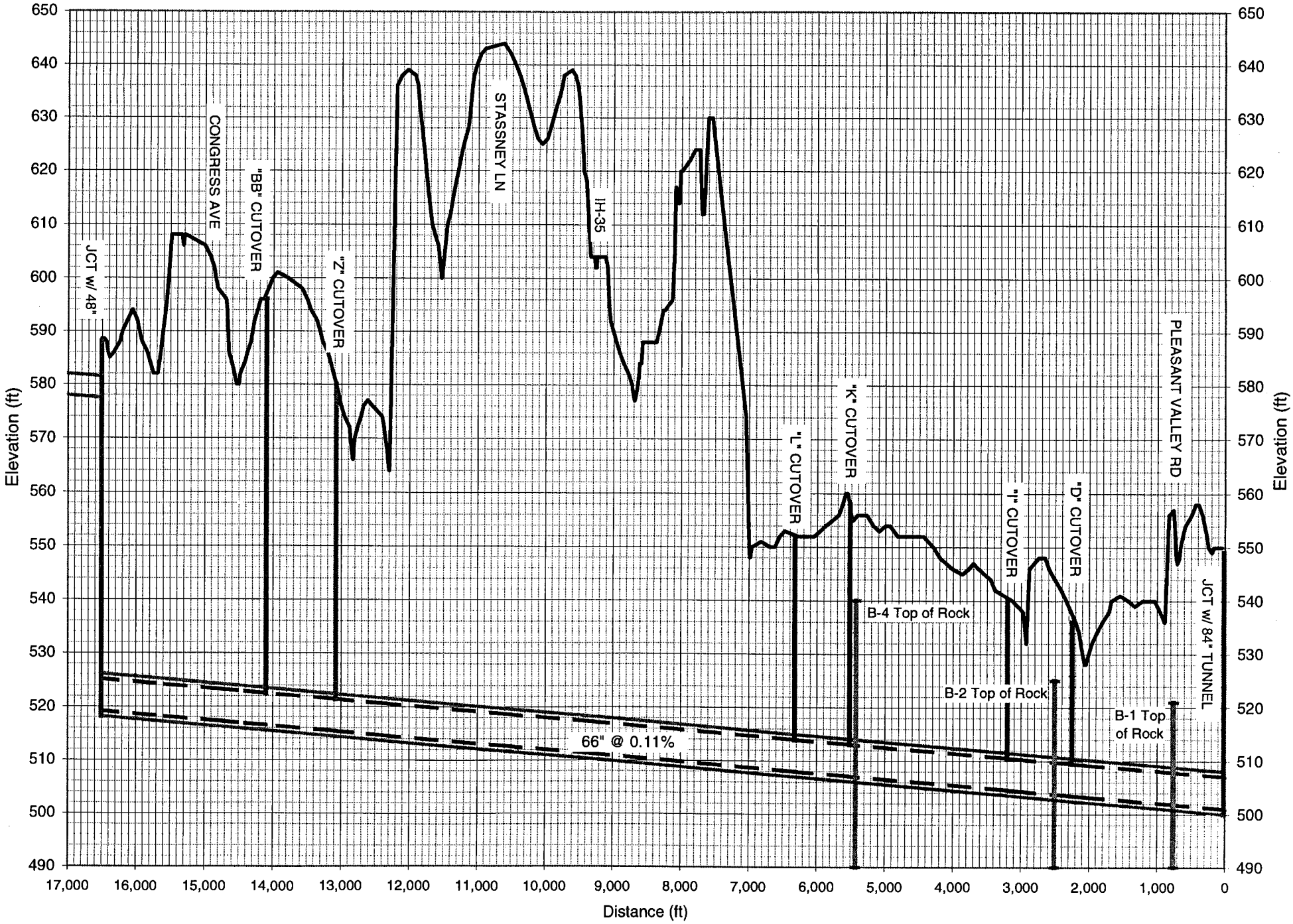
KK	5,597	15,213		6,522	19,738	32,929															
II	75	118		6,522	19,738	32,929								75	118	403	537	0.30%	10.0	18	1.8
HH	307	441		6,522	19,738	32,929								383	559	1,785	2,380	0.30%	17.5	27	2.6
FF	27	54		6,522	19,738	32,929								410	613	1,915	2,553	0.30%	17.9	27	2.6
EE	27	54		6,522	19,738	32,929								437	666	2,044	2,725	0.30%	18.4	27	2.7
BB	69	135		7,028	20,539	34,565	1	506	802	2,364	5	16.05	18								
Z	112	260		7,140	20,799	35,009	1	112	260	671	5	8.55	10								
W	126	307		7,267	21,106	35,522	1	126	307	764	5	9.13	10								
V	91	218		7,267	21,106	35,522								91	218	556	741	0.18%	12.4	18	1.6
U	80	170		7,267	21,106	35,522								171	387	988	1,317	0.18%	15.4	18	1.8
S	28	69		7,267	21,106	35,522								199	456	1,146	1,528	0.28%	15.0	18	2.2
R	247	509		7,267	21,106	35,522								447	965	2,369	3,159	0.24%	20.2	30	2.5
P	13	26		7,267	21,106	35,522								460	991	2,431	3,241	0.24%	20.4	30	2.5
N	63	123		7,267	21,106	35,522								523	1,114	2,724	3,632	0.24%	21.3	30	2.6
L	41	53		7,267	21,106	35,522								564	1,167	2,884	3,845	0.24%	21.8	36	2.6
K	53	106		7,267	21,106	35,522								617	1,273	3,126	4,169	0.24%	22.5	36	2.6
I	284	501		7,267	21,106	35,522								900	1,774	4,333	5,778	0.24%	25.4	36	2.9
E	139	290		7,267	21,106	35,522								1,040	2,064	4,954	6,605	0.28%	25.9	36	3.1
D	14	29		7,267	21,106	35,522								1,053	2,093	5,014	6,685	0.28%	26.1	36	3.2
A	545	1,328		7,267	21,106	35,522								1,598	3,421	7,563	10,084	0.28%	30.4	36	3.5

TABLE 8-1. DESIGN FLOWS FOR PROJECT IMPROVEMENTS

Alternative D2																					
			Relief Tunnel					Cut-Over Lines						SlipLining							
	Added Avg Flow (gpm)	Added I/I Flow (gpm)	Tunnl MH	Cum Avg Q	Cum I/I	Tunnel Design Flow	C/O to Tunnel	Cum Avg Q	Cum I/I	C/O Design Flow	Vel	Calc Size	Pipe Size	Cum Avg Q	Cum I/I	SlipLiner Design Flow	Minimum Capacity	Pipe slope	Calc S'Liner pipe size	Use slipliner size (ID)	Vel @ Peak Dry Weather Flow
LostCk MUD pumpover	254	846																			
BartonCk pumpover	671	3,679																			
KK	5,597	15,213	D-7	6,522	19,738	32,929															
II	75	118		6,522	19,738	32,929								75	118	403	537	0.30%	10.0	18	1.8
HH	307	441		6,522	19,738	32,929								383	559	1,785	2,380	0.30%	17.5	27	2.6
FF	27	54		6,522	19,738	32,929								410	613	1,915	2,553	0.30%	17.9	27	2.6
EE	27	54		6,522	19,738	32,929								437	666	2,044	2,725	0.30%	18.4	27	2.7
BB	69	135	D-6	7,140	20,799	35,009	1	506	802	2,364	5	16.05	18								
Z	112	260		7,140	20,799	35,009	1	112	260	671	5	8.55	10								
W	126	307	D-5	7,267	21,106	35,522	1	126	307	764	5	9.13	10								
V	91	218		7,267	21,106	35,522								91	218	556	741	0.18%	12.4	18	1.6
U	80	170		7,267	21,106	35,522								171	387	988	1,317	0.18%	15.4	18	1.8
S	28	69		7,267	21,106	35,522								199	456	1,146	1,528	0.28%	15.0	18	2.2
R	247	509		7,267	21,106	35,522								447	965	2,369	3,159	0.24%	20.2	36	2.4
P	13	26		7,267	21,106	35,522								460	991	2,431	3,241	0.24%	20.4	36	2.4
N	63	123		7,267	21,106	35,522								523	1,114	2,724	3,632	0.24%	21.3	36	2.5
L	41	53		7,267	21,106	35,522								564	1,167	2,884	3,845	0.24%	21.8	36	2.6
K	53	106	D-3	7,883	22,379	37,794	1	617	1,273	3,126	5	18.46	21								
I	284	501	D-2	8,167	22,880	38,750	1	284	501	1,445	5	12.55	15								
E	139	290		8,167	22,880	38,750								139	290	789	1,052	0.28%	13.0	18	2.0
D	14	29		8,167	22,880	38,750								153	319	862	1,149	0.28%	13.5	18	2.1
A	545	1,328		8,167	22,880	38,750								698	1,647	3,707	4,943	0.28%	23.3	36	2.9

FIGURE 8-1. RELIEF INTERCEPTOR PROFILE - ALTERNATIVE "C"



130 ft north of the Alternative "C" alignment and not reliable for conditions at the "C" tunnel. At least one tunnel diameter of competent rock over the top of the tunnel bore is typically recommended. (It might be noted in reference to Figure 8-1, that the Alternative "A" tunnel with an invert elevation of 516 ft tying into the diversion box, would be a mixed face tunnel at the diversion box.) Borings B-2 and B-4 are close to the "C" alignment and show increasing rock cover further west. At the multi-family and commercial tracts that are east and west of IH-35, the tunnel at 0.11% slope would be 80 ft to 120 deep.

The Utility may wish to consider a larger carrier pipe than 66-inch for the relief tunnel. The cost of larger pipe size is offset by a reduced amount of grouting. With the flow rates involved here, a larger pipe would not present hydraulic problems (e.g., an 84-inch at 0.10% slope would provide 2.5 fps velocity at the typical average flow of 6,000 gpm versus 2.6 fps for a 66-inch). A larger pipe could also provide storage volume that could be used to some extent for flow equalization during peak flows for possible benefit of treatment operations at the South Austin Regional WWTP. The lowest cost for the completed tunnel might be obtained if the bidders have some flexibility in the size of the tunnel bore and carrier pipe.

8.3 CUT-OVER MAINS

As stated above, Table 8-1 presents the design flow for each cut-over main for each Alternative. A pipe size is derived for each cut-over assuming pipe capacity for q/Q of 75% and velocity flowing full of 5 fps. Since the cut-over mains are going to tunnel inlet shafts and a number of the cut-overs will be installed by bore and jack, slope is generally not a limiting factor for the cut-over pipe size.

The calculated cut-over pipe size is 8 inches or less for some of the smaller subbasins. For the purpose of cost estimates, a minimum cut-over pipe size of 12 inches is used.

8.4 PIPELINE REHABILITATION

Table 8-1 shows the minimum theoretical pipe size for the slipliner design flow based on the slope of the existing interceptor. This size is the most hydraulically efficient size in terms of providing the best velocity characteristics. Larger slipliner would produce slightly lower velocities. The small slipliner in the existing 36-/42-inch pipe in not practical in many cases. One reason is the cost of grouting. A larger slipliner can reduce the total sliplining cost by reducing the amount of grout required. For example, with a 16-inch (O.D.) slipliner in a 42-inch pipe, 85% of the 42-inch pipe would be filled with grout. With a 30-inch liner in a 42-inch pipe, grout would fill one-half of the pipe volume. Besides the cost factor, a large grout volume can affect constructability. The amount of heat generated by a large volume of grout can weaken the liner pipe and increase the potential for collapse of the liner under grouting pressure. Installation of grout in lifts to avoid this problem increases the grouting time and cost. Fortunately, velocity is not very

sensitive to pipe slope, so larger slipliner pipe can reduce the rehab cost without significantly reducing velocities.

Table 8-1 shows a selected slipliner size (inside diameter) and the resulting velocity at the peak dry weather design flow. The guideline used for size was the largest liner that would provide velocity at peak dry weather (i.e., wastewater) flow between 2.0 and 2.5 fps. In cases where this could not be achieved, the guideline was slipliner (outside diameter) not smaller than one-half the host pipe size. Each Alternative has at least one section of sliplining where the velocity at peak dry weather flow would not reach 2.0 fps, as shown in Table 8-1.

None of the sliplining pipe is the maximum possible size for the lined pipe. The smaller sliplining pipe has a couple of constructability benefits for HDPE sliplining pipe. Some contractors report that they would avoid bypass pumping and use the annular space between liner and the pipe when the flow requirements and pipe sizes are suitable. (For segmented sliplining pipe, bypass pumping is not required in any case.) However, the requirement for potential bypass pumping depends on sequence of construction for the project, which is discussed in Section 9. An additional benefit of smaller HDPE sliplining pipe is the ability for the liner pipe to be pulled through some of the minor bends in the interceptor where full size HDPE or segmented liner pipe would require excavation of an insertion pit.

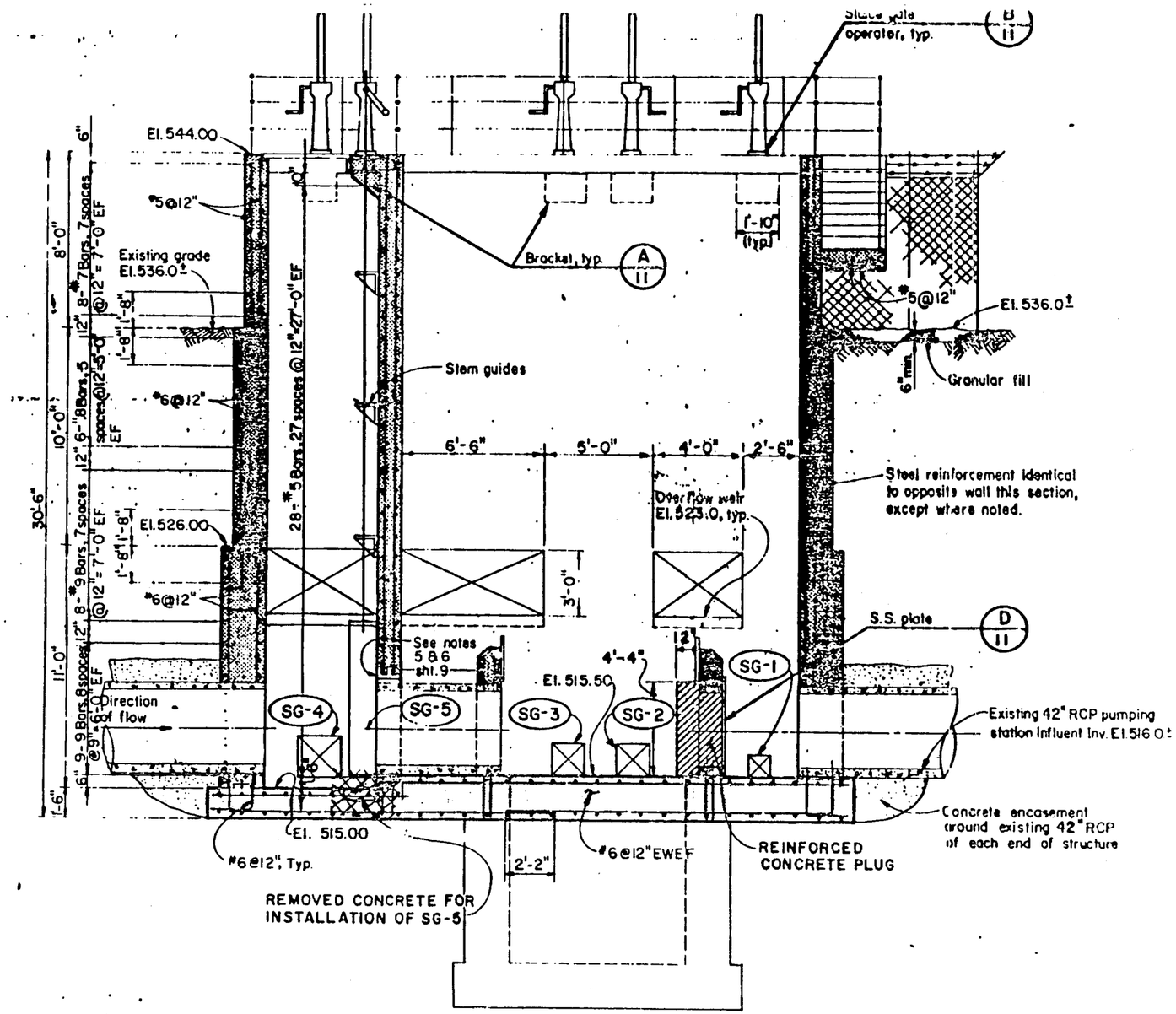
8.5 FLOW DIVERSION/JUNCTION STRUCTURES

8.5.1 Existing Diversion Box

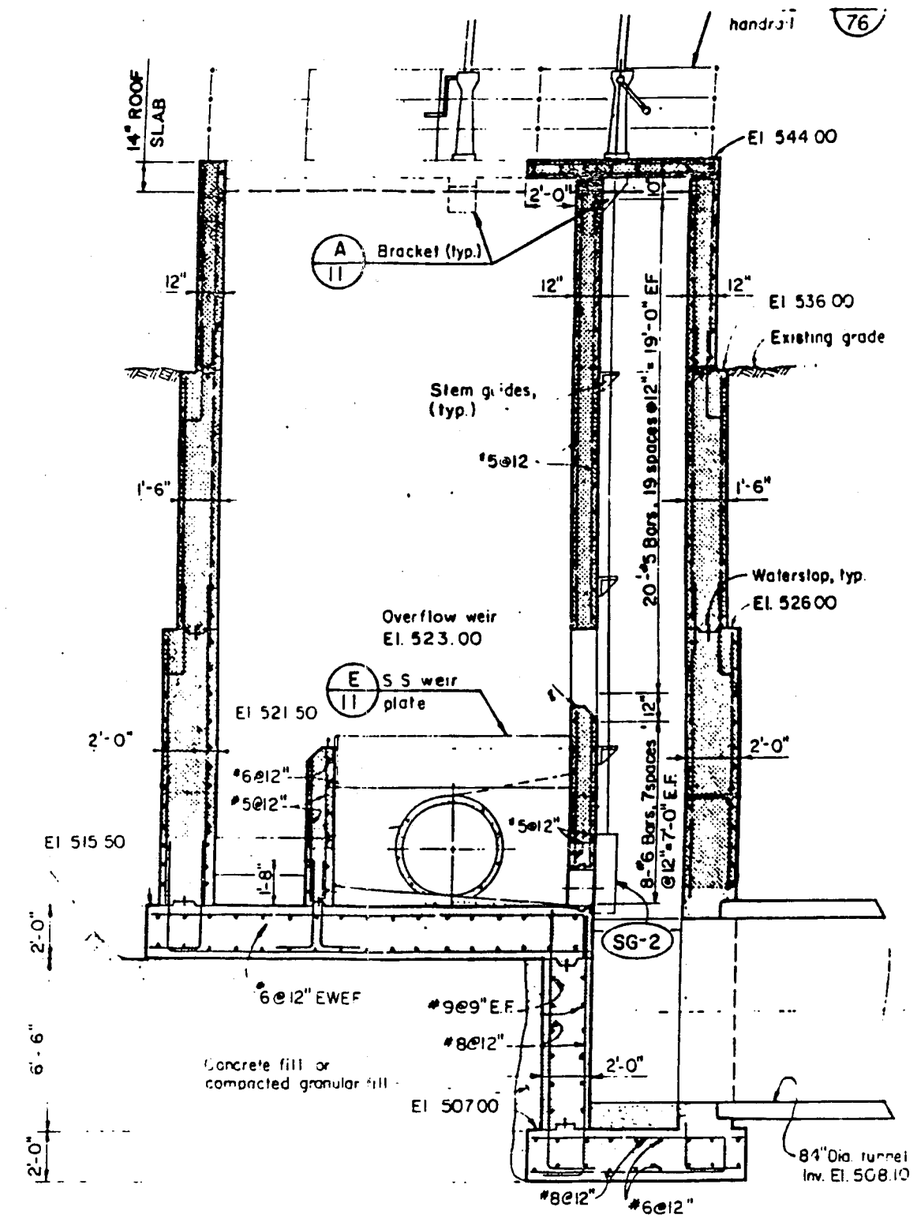
The relief tunnel for Alternative "A" ties into the existing diversion box where flow in the 42-inch interceptor goes to the 84-inch tunnel. The diversion box has a plugged 48-inch stub at flowline elevation 516.1 which would be a connection point for the relief tunnel. However, additional inlet capacity is needed at the box, so it is planned to have a splitter box on the tunnel pipe outside the diversion box and a junction with the 42-inch pipe in order to use both the 42-inch and 48-inch wall openings for flow from the relief tunnel. The 42-inch main would be sliplined from the diversion box to manhole "A" and carry flow from the sewer mains in Pleasant Valley Road.

There is a severe capacity constraint inside the diversion box due to interior walls and inadequate wall openings (see Figure 8-2 with excerpts from the construction plans). The first chamber with the 42- and 48-inch inlet wall openings has (1) a 24x24-inch square orifice to a channel carrying flow to the 84-inch tunnel pipe, and (2) a short 42-inch pipe stub to a second chamber. The second chamber has two 18x18-inch square orifices to the 84-inch. The second chamber has an overflow to the 84-inch that is 3 ft higher than the crown of the 48-inch stub. Poor hydraulic conditions in the diversion box are suspected as

FIGURE 8-2. LOWER WILLIAMSON CREEK INTERCEPTOR DIVERSION BOX DETAILS



SECTION 5/10
Scale: 1/4" = 1'-0"



SECTION 2/10
Scale: 1/4" = 1'-0"

one potential cause of the flow instability that has been observed at peak flows in monitoring data from manhole "D" (see Section 3.2.2).

Utilization of the diversion box as part of Alternative "A" will require major structural alterations to obtain sufficient capacity for the design flows. This work will involve demolition of interior walls and enlargement of wall openings discharging to the trough for the 84-inch. The work must be carefully planned and staged to keep the box in service for existing flows. Bypass pumping from a manhole on the 42-inch main just outside the diversion box to the 84-inch could handle a portion of the flow. If the relief tunnel is completed prior to work on the diversion box, it would be possible, for short durations, to divert all flow into storage in the tunnel to allow critical demolition or construction activities inside the diversion box under safer working conditions. Renovation of the diversion box will involve some hazardous situations for manual labor due to existing flows and constrained workspace, and it is expected to be a time-consuming and expensive part of the project.

Rebuilding the diversion box for increased capacity would allow incorporation of flow monitoring features, such as a sharp-crested weir with aerated nappe for accurate flow measurements, with instrumentation and telemetry equipment. A primary device such as a weir would have some maintenance and reliability benefits compared to an area-velocity meter for manhole installation.

For Alternatives "C" and "D" the diversion box could remain in service for local flows without the extensive modifications required for Alternative "A". For Alternative "C" the diversion box would receive flow from the sewer mains in Pleasant Valley Road via sliplined 42-inch main between the diversion box and manhole "A". In Alternative "D2" the diversion box and sliplined 42-inch main would receive flows from laterals to manholes "A", "D", and "E" on the 42-inch. Alternative "D1" imposes considerably more flow on the diversion box. It keeps the 42-inch in service for flows from 16 laterals to the interceptor up to manhole "V" at IH-35. Further investigation of the current hydraulic problems at the diversion box would be needed to determine if the remaining flows in the interceptor in Alternative "D1" would require improvements to the diversion box.

Although the diversion box may have adequate capacity for the reduced flows in Alternatives "C" and "D", abandoning the structure would be a benefit to the Utility by eliminating the need for rehab and maintenance. In addition, the diversion box as is creates an obstruction in the modified floodplain that is planned with the Creek Bend Drainage Improvements Phase 2. Abandoning the box would require another inlet shaft near Pleasant Valley Road and a cut-over main between the shaft and the 42-inch main. Although this is not part of the current planning for Alternatives "C" and "D", further coordination between the relief tunnel and drainage improvements projects is likely to eliminate the diversion box.

8.5.2 East Junction Structure at 84-inch Tunnel

Alternatives "C" and "D" tie the relief tunnel into the existing 84-inch tunnel approximately 600 ft east of the diversion box. Although the primary purpose is to obtain better rock conditions for tunneling, it also avoids the problems described above for rebuilding the existing diversion box.

The main construction shaft for the relief tunnel would be immediately south of the 84-inch tunnel for Alternative "C" and immediately north for Alternative "D". A junction box constructed on the 84-inch tunnel would be about 40 ft deep. The diversion pipe from the relief tunnel would be about the same elevation as the 84-inch, e.g., a crown match of the pipes with no energy dissipation required. During construction the 84-inch would be carrying the flows from the 42-inch interceptor, but the flow depths would be low due to the steep (1.84%) slope of the 84-inch pipe. (At current peak dry weather flow of approximately 10,000 gpm, the level in the 84-inch pipe would be less than 11 inches.)

One means of providing accurate, permanent flow measurement would be a flume built in the construction shaft for the relief tunnel. This could provide good hydraulic conditions upstream of the flume and require minimal head. Manholes will provide access to both the junction box and the flume.

8.5.3 West Junction Structure at 48-inch Interceptor and Tunnel Inlet Shafts

The relief tunnel in all of the Alternatives ties into the existing 48-inch main that currently discharges to the 36-inch interceptor. Depending on the Alternative, the relief tunnel will be about 40 to 60 ft deep. The 48-inch pipe is 10 ft deep. The junction box will include controlled energy dissipation such as use of a vortex drop structure. An inlet flume directs the flow tangentially into a vertical drop pipe. High velocity spiral flow in the drop pipe dissipates energy through friction losses. With proper design there is a stable air core in the vortex shaft that should minimize escape of odorous gases. The design is intended to minimize release of hydrogen sulfide.

Tunnel inlet shafts for flows from cut-over mains will also have energy dissipating drop structures. Both the tunnel inlets and the upstream junction can be designed with flumes and telemetry for permanent flow monitoring.

9.0 SEQUENCE OF CONSTRUCTION AND PROJECT SCHEDULE

9.1 SEQUENCE OF CONSTRUCTION

Extensive preparation is required in order to commence the tunnel construction. Mobilization by the Contractor includes securing the workspace at the construction shaft, setting up the project office and shop facilities, preparing the tunnel boring machine (e.g., cutterheads and propulsion system) for project conditions and shipping its components to the job site, assembly of mucking and ventilation equipment, bringing in high voltage power, and stockpiling related equipment and supplies (tunnel secondary power and lighting, dewatering, primary lining materials, etc.).

Excavation and support of the main tunnel shaft starts construction. The shaft can be prepared during TBM mobilization. The TBM is launched from the construction shaft and bores the tunnel upstream. Depending on the rock conditions and the type of TBM, the contractor may need to erect primary lining as the tunnel progresses. As the tunnel length increases, if the Contractor is not provided a site for an intermediate mucking shaft (e.g., at IH-35), the Contractor may construct a muck train siding in the tunnel and run two muck trains simultaneously so that mucking does not limit the production of the TBM.

As the tunnel progresses the Contractor will excavate shafts for the drop inlets at cut-overs. The cut-over mains could be constructed at the same time as the shafts in order to limit the duration of construction activity at each site. Some cut-overs would be bored from the tunnel shaft to the diversion point on the lateral or interceptor. Shafts that are used for the ventilation system would remain in service for the duration of tunnel construction. Either drop inlets or manholes will be constructed at all the tunnel shafts to provide future access to the relief tunnel.

A shaft will be constructed at the upper end of the tunnel for removal of the TBM and for the junction structure with the upstream 48-inch interceptor. After completion of the tunnel bore, placement of the carrier pipe in the tunnel will begin. Typically the pipe sections are lowered to the tunnel at the shafts and jacked or pulled into position. After the pipe is installed, the annular space between the carrier pipe and the tunnel bore is grouted. A potential alternative to this method is construction of a cast-in-place concrete secondary tunnel lining, with suitable corrosion protection, which functions as the carrier pipe.

Following construction of the tunnel, inlet shafts, junction structures, and cut-over mains, the relief tunnel can be placed in service by making the final "wet connections" to the existing system. Progressing with the cut-overs starting from the upstream end will reduce the amount of flow involved in the wet connection where the cut-over is made directly on the existing interceptor (e.g., manholes "K", "L", "BB"). This is especially applicable for the cut-over at manhole "Z". The plan for this cut-over is to remove the existing 36-inch pipe between manhole "Z" and the tunnel shaft, and then install the cut-over

main in the same trench for flow in the reverse direction. Completing the cut-over at manhole "BB" allows the 36-inch main downstream to be abandoned, facilitating the flow cut-over at manhole "Z".

Depending on the Alternative, various sections of the existing interceptor will continue to carry flow from lateral mains. Rehabilitation of the existing main after the relief tunnel is placed in service will greatly reduced requirements for flow handling during sliplining.

9.2 PROJECTED CONSTRUCTION SCHEDULE

The contractor's mobilization from notice to proceed to delivery and assembly of the TBM typically takes 3 to 6 months. Availability of suitable TBM's can be quite variable.

Tunneling progress will probably be limited the first few weeks as equipment adjustments and work routines are refined. At full production the estimated advance rate in the Austin Chalk is projected to be an average of 80 to 100 ft per working day. Depending on a particular contractor's equipment versus labor costs, the contractor may prefer to work either 5 or 6 days a week. A workday is typically two 10-hour tunneling shifts and a 4-hour maintenance shift. The contractor may take some downtime at about the half-way mark for TBM refurbishing, e.g., cutterheads, drive gear, etc. Downtime can be major variable. One of the case histories in Appendix B notes that TBM repairs took 40% of the tunneling shift time. The Contractor should have sufficient information available for selection of proper tunneling machinery for the rock conditions in order to minimize unplanned downtime. Based on a tunnel length of 16,000 to 17,000 ft, the estimated tunnel construction time is 40 to 48 weeks, or about 10 to 12 months. Maintaining a good production rate can be very dependent on the contractor's selection of appropriate equipment, as illustrated in the case histories in Appendix B. An accelerated schedule could be imposed on the contractor, but this could limit the number of bidders and result in higher bids, for example, if the schedule forced the contractor to acquire and use two TBM's.

After tunnel excavation is complete, installation of the carrier pipe and grouting typically takes ½ to ¾ as long as the tunnel boring time. Additional time is required after completion of the tunnel for construction of the drop inlet structures and manholes at the tunnel shafts and completion of the upstream and downstream junction structures. The cut-over mains could be installed during tunnel construction as stated above. The estimated time for installation of the tunnel carrier pipe, construction of drop inlets and junction structures, and completion of the cut-overs is 12 to 18 months. Contractor's demobilization for vacation and restoration of all of the worksites is estimated at 2 months.

The potential duration for tunnel construction given the factors discussed above is 27 to 36 months. To some extent the schedule will depend on the Alternative selected for final design and construction, since the tunnel length and number of inlets varies among Alternatives. This schedule is aimed

at obtaining good bids rather than meeting a time frame constraint. The schedule does not account for any special improvements that might be required related to construction in parkland.

As discussed in Section 9.1, it is assumed that rehabilitation of the existing interceptor will occur after the relief tunnel is placed in service. The duration for the rehab work is dependent on the tunnel Alternative that is selected, since the amount of sliplining ranges from 4,100 ft for Alternative "A" to 13,000 ft for Alternative "D1". Rehabilitation of the existing interceptor concurrent with construction of the relief tunnel is possible but not recommended due to complications with multiple contractors in the same workspace and increased requirement for bypass pumping.

10.0 CONSTRUCTION COST ESTIMATES

Construction cost estimates are prepared for each Alternative which include costs for the relief tunnel, the cut-over mains, and sliplining for the existing interceptor. Each component has costs for pipe, structures, and miscellaneous factors.

Cost of the tunnel, carrier pipe, and grouting is based on $850/lf for all alternatives. Note that Alternative "A" does not include an allowance for some amount of soft ground and mixed face tunneling at the lower end as discussed in Section 7.2. Excavation costs for the construction shafts and inlet/ventilation shafts are estimated at $700/vf and $200/vf, respectively. The inlet shaft costs include $30,000 for a vortex drop structure and manhole. The ventilation shaft costs include $10,000 for equipment and a standard manhole. Allowance for mobilization/demobilization is typically 5% to 10% of the tunnel construction cost. Each Alternative includes $1,000,000 for mobilization/demobilization.

Unit prices for cut-over mains and for sliplining are presented in Table 10-1. Due to the short length of pipe for many of the cut-over mains, significant additional costs are incurred for connections to the existing laterals or interceptor and for fixed or miscellaneous costs. Since the cut-over mains are associated with tunnel inlet shafts, some shared costs are assigned to the cut-over mains. These costs include construction access to the shaft site and cut-over point, creek crossing/baseflow diversion structures where required, stabilized roadway, and restoration of the Creek Bend drainage improvements where required. Costs are estimated separately for manholes needed for the flow diversions to the cut-over main.

TABLE 10-1

UNITS COSTS FOR CUT-OVER MAINS AND SLIPLINING

CUT-OVER MAIN UNIT COSTS			SLIPLINING UNIT COSTS ($/LF)					
Pipe			Liner	Pipe	Grouting Cost		Pipe & Grouting Cost	
Size	Open Cut	Bore & Jack	Size	Cost	36"	42"	36"	42"
12	60	146	15	30.40	36.10	51.80	66.50	82.20
15	75	174	18	32.10	32.70	48.50	64.80	80.60
18	90	201	21	33.60	28.80	44.50	62.40	78.10
21	105	227	24	35.00	24.20	40.00	59.20	75.00
24	120	251	27	36.30	19.10	34.80	55.40	71.10
			30	37.40	13.30	29.10	50.70	66.50
			33	38.50	7.00	22.70	45.50	61.20
			36	39.50	na	15.80	39.50	55.30

The unit costs for sliplining in Table 10-1 reflect the tradeoff between liner pipe size and grouting cost. An additional $5/lf is added for cleaning and TV inspection prior to sliplining. Significant

costs are incurred for insertion pits and new manholes. The existing 36-/42-inch interceptor was constructed with a number of bends (up to 33°) with no manhole. There are 15 bends greater than 5° with no manhole. It is possible to pull smaller liner pipe through slight bends, but there can be a problem with the liner pipe riding up the side of the lined pipe at the bend due to tension on the liner pipe. This raises the liner pipe invert at the bend and interferes with uniform gravity flow. The sliplining cost estimates include a number of new manholes at the problem bends. Estimated costs are $5,000 for a new manhole, $5,000 for an insertion pit, and $10,000 for a site with both. The miscellaneous costs included in the cost estimate tables for each section of sliplining provide an allowance for mobilization, bypass pumping, access improvements, environmental controls and restoration.

The preliminary construction cost estimates for each Alternative are presented in Tables 10-2, 10-3, 10-4, and 10-5. It should be noted that the estimates in these tables do not include an amount for contingencies. Furthermore, the estimates do not include easement acquisition costs, which could be significant for Alternatives "D1" and "D2" on account of the number of tunnel easements required for single-family residences.

Table 10-6 summarizes the preliminary construction cost estimates and includes amounts for easement acquisition and contingencies. The City advised using an allowance of $6,000 per property as the easement administrative cost for the City's property agents and appraisers. This does not include costs for land acquisition or easement surveying. The number of easements (which must be obtained prior to construction) can also affect project schedule. The City advises allowing a minimum of six months for any easement acquisition. Obtaining all of the easements required for Alternatives "D1" (±64) and "D2" (±38) may take one to two years. These cost estimates also do not include potential costs for parkland improvements which might be required as a result of negotiations with PARD for obtaining Parkland Use Agreements. The totals include an allowance of 30% for contingencies such as changes in final design and inflation in construction costs, as well as land acquisition and parkland improvements that are undetermined costs at this time.

TABLE 10-2. PRELIMINARY CONSTRUCTION COST ESTIMATE - ALTERNATIVE 'A' - $17,730,000

TUNNEL - $16,710,000				CUT-OVER MAINS - $551,200							SLIPLINING - $466,800						
Comment	Tunnel & Liner	Structures	Exist. MH	Length	Size	Bore/ O.Cut	Pipe Cost	Manholes/ Connections	Fixed/Misc Costs	Total	Exist. MH	--	Length	Size	Sliplining Cost	Insertion Pits & MHs	Fixed/Misc Costs
$1,000,000	$14,671,000	$1,039,000	Totals	2,810			$319,680	$113,500	$118,000	$551,180	Totals		4,096		$311,757	$110,000	$45,000
@ Diversion Box		329,000	Jct								Jct	42	-----			$5,000	
												42	206	36	$12,422		
Construction Shaft		44,000	A								A	42	-----				
Mobe/Demobe			B								B						
1,000,000																	
			C								C						
Tunnel Length= 17,260 ft																	
	14,671,000		D	540	12	OC	32,400	12,500	22,000	66,900	D						
Inlet Shaft		35,000	E	70	12	OC	4,200	7,500	12,000	23,700	E						
			F								F						
			G								G						
			H								H						
Inlet Shaft		40,000	I	550	15	BJ	95,700	7,500	17,000	120,200	I						
			J								J						
Inlet Shaft		37,000	K	50	12	OC	3,000	10,000	10,000	23,000	K						
Inlet Shaft		38,000	L	50	15	OC	3,750	10,000	5,000	18,750	L	42	-----			5,000	
												42	I				
			M								M	42	I				
												42	I			10,000	
			N								N	42	I			5,000	
												42	I			5,000	
			O								O	42	I				
												42	I			5,000	
			P								P	42	I			10,000	
												42	I			5,000	
			Q								Q	42	I				
												42	1,892	27	$143,981		25,000
			R								R	42	-----			5,000	
Inlet Shaft		38,000	S	150	12	BJ	21,900	7,500	10,000	39,400	S						
			T	470	12	OC	28,200	5,000	2,000	35,200	T						
Inlet Shaft		38,000	U	80	12	OC	4,800	7,500	5,000	17,300	U						
Inlet Shaft		37,000	V	60	12	OC	3,600	10,000	12,000	25,600	V						

TUNNEL - $16,710,000				CUT-OVER MAINS - $551,200							SLIPLINING - $466,800						
Comment	Tunnel & Liner	Structures	Exist. MH	Length	Size	Bore/ O.Cut	Pipe Cost	Manholes/ Connections	Fixed/Misc Costs	Total	Exist. MH	--	Length	Size	Sliplining Cost	Insertion Pits & MHs	Fixed/Misc Costs
Inlet Shaft		39,000	W	40	12	OC	2,400	7,500	3,000	12,900	W						
			X								X						
Inlet Shaft		39,000	X1	50	12	OC	3,000	7,500	5,000	15,500	X1						
			Y								Y						
Inlet Shaft		38,000	Z	170	12	OC	10,200	7,500	10,000	27,700	Z						
			AA								AA						
Inlet Shaft		44,000	BB	530	18	BJ	106,530	13,500	5,000	125,030	BB	36	-----			5,000	
												36	I				
			CC								CC	36	I			10,000	
												36	I				
			DD								DD	36	I			5,000	
												36	I				
			EE								EE	36	I				
												36	I			10,000	
			FF								FF	36	I				
												36	I				
			GG								GG	36	I			10,000	
												36	1,650	27	$125,565		
			HH								HH	36	-----			10,000	
												36	348	18	$29,789		20,000
			II								II	36	-----			5,000	
			JJ								JJ						
Inlet Shaft		243,000	KK								KK						

TABLE 10-3. PRELIMINARY CONSTRUCTION COST ESTIMATE - ALTERNATIVE 'C' - $16,310,000

TUNNEL - $15,033,500				CUT-OVER MAINS - $341,400							SLIPLINING - $931,000						
Comment	Tunnel & Liner	Structures	Exist. MH	Length	Size	Bore/ O.Cut	Pipe Cost	Manholes/ Connection	Fixed/Misc Costs	Total	Exist. MH	--	Length	Size	Sliplining Cost	Insertion Pits & MHs	Fixed/Misc Costs
$1,000,000	$14,033,500	$847,500	Totals	1,420			$217,360	$66,000	$58,000	$341,360	Totals		8,867		$695,954	$165,000	$70,000
Jct @ 84"		228,000	Jct								Jct	42	-----			$5,000	
												42	206	36	$12,422		
Construction Shaft		68,000	A								A	42	-----				
Mobe/Demobe			B								B						
1,000,000																	
Tunnel Length= 16,510 ft			C								C						
	14,033,500																
Inlet Shaft		38,000	D	50	12	OC	3,000	7,500	10,000	20,500	D						
			E	350	12	BJ	51,100	5,000	8,000	64,100	E						
			F								F						
			G								G						
			H								H						
Inlet Shaft		39,000	I	220	15	BJ	38,280	11,500	10,000	59,780	I						
			J								J						
Inlet Shaft		40,000	K	50	12	OC	3,000	10,000	10,000	23,000	K						
Inlet Shaft		41,000	L	50	21	OC	5,250	10,000	5,000	20,250	L	42	-----			5,000	
												42	I				
			M								M	42	I				
												42	I			10,000	
			N								N	42	I			5,000	
												42	I			5,000	
			O								O	42	I				
												42	I			5,000	
			P								P	42	I			10,000	
												42	I			5,000	
			Q								Q	42	I				
												42	1,892	33	$125,250		
			R								R	42	-----			5,000	
												42	I			5,000	
			S								S	42	I			5,000	
Vent. Shaft @ IH-35		64,500										42	I			10,000	
(? Mucking)			T								T	42	I				
												42	2,188	21	$181,823	5,000	
			U								U	42	-----			5,000	
												42	I			5,000	
			V								V	42	I				

TUNNEL - $15,033,500				CUT-OVER MAINS - $341,400							SLIPLINING - $931,000						
Comment	Tunnel & Liner	Structures	Exist. MH	Length	Size	Bore/ O.Cut	Pipe Cost	Manholes/ Connection	Fixed/Misc Costs	Total	Exist. MH	--	Length	Size	Sliplining Cost	Insertion Pits & MHs	Fixed/Misc Costs
												42	I			10,000	
			W								W	42	I				
												42	I				
			X								X	42	I				
												42	2,583	18	$221,105	10,000	50,000
			X1								X1	42	-----				
			Y								Y						
Inlet Shaft		40,000	Z	170	12	OC	10,200	8,500	10,000	28,700	Z						
			AA								AA						
Inlet Shaft		46,000	BB	530	18	BJ	106,530	13,500	5,000	125,030	BB	36	-----			5,000	
												36	I				
			CC								CC	36	I			10,000	
												36	I				
			DD								DD	36	I			5,000	
												36	I				
			EE								EE	36	I				
												36	I			10,000	
			FF								FF	36	I				
												36	I				
			GG								GG	36	I			10,000	
												36	1,650	27	$125,565		
			HH								HH	36	-----			10,000	
												36	348	18	$29,789		20,000
			II								II	36	-----			5,000	
			JJ								JJ						
Inlet Shaft		243,000	KK								KK						

TABLE 10-4. PRELIMINARY CONSTRUCTION COST ESTIMATE - ALTERNATIVE 'D1' - $16,240,000

TUNNEL - $14,638,250				CUT-OVER MAINS - $394,300							SLIPLINING - $1,208,700						
Comment	Tunnel & Liner	Structures	Exist. MH	Length	Size	Bore/ O.Cut	Pipe Cost	Manholes/ Connection	Fixed/Misc Costs	Total	Exist. MH	--	Length	Size	Sliplining Cost	Insertion Pits & MHs	Fixed/Misc Costs
$1,000,000	$13,638,250	$688,000	Totals	2,090			$334,290	$35,000	$25,000	$394,290	Totals		13,061		$918,658	$210,000	$80,000
Jct @ 84"		228,000	Jct								Jct	42	-----			5,000	
												42	I				
Construction Shaft		68,000	A								A	42	I				
												42	I				
Mobe/Demobe			B								B	42	I				
1,000,000												42	I			5,000	
			C								C	42	I				
Tunnel Length= 16,045 ft												42	I			5,000	
	13,638,250		D								D	42	I			5,000	
												42	I				
			E								E	42	I				
												42	I			5,000	
			F								F	42	I				
Vent. Shaft		18,000										42	I			5,000	
			G								G	42	I				
												42	I				
			H								H	42	I				
												42	I				
			I								I	42	I			5,000	
												42	I			10,000	
			J								J	42	I				
												42	I			10,000	
			K								K	42	I				
Vent. Shaft		23,000										42	6,206	36	$374,222	5,000	
			L								L	42	-----			5,000	
												42	I				
			M								M	42	I				
												42	I			10,000	
			N								N	42	I			5,000	
												42	I			5,000	
			O								O	42	I				
												42	I			5,000	
			P								P	42	I			10,000	
												42	I			5,000	
			Q								Q	42	I				
Vent. Shaft		22,000										42	1,892	30	$135,278		
			R								R	42	-----			5,000	
												42	I			5,000	
			S								S	42	I			5,000	
												42	I			10,000	
			T								T	42	I				
												42	I			5,000	
			U								U	42	I			5,000	
												42	2,965	18	$253,804	5,000	60,000
			V								V	42	-----				

TUNNEL - $14,638,250				CUT-OVER MAINS - $394,300							SLIPLINING - $1,208,700						
Comment	Tunnel & Liner	Structures	Exist. MH	Length	Size	Bore/ O.Cut	Pipe Cost	Manholes/ Connection	Fixed/Misc Costs	Total	Exist. MH	--	Length	Size	Sliplining Cost	Insertion Pits & MHs	Fixed/Misc Costs
																10,000	
			W	470	12	BJ	68,620	7,500	5,000	81,120	W						
Inlet Shaft		42,000	X						5,000		X						
			X1	420	12	BJ	61,320	7,500	5,000	73,820	X1						
			Y								Y						
			Z	670	12	BJ	97,820	7,500	5,000	110,320	Z						
			AA								AA						
Inlet Shaft		44,000	BB	530	18	BJ	106,530	12,500	5,000	124,030	BB	36	-----			5,000	
												36	I				
			CC								CC	36	I			10,000	
												36	I				
			DD								DD	36	I			5,000	
												36	I				
			EE								EE	36	I				
												36	I			10,000	
			FF								FF	36	I				
												36	I				
			GG								GG	36	I			10,000	
												36	1,650	27	$125,565		
			HH								HH	36	-----			10,000	
												36	348	18	$29,789		20,000
			II								II	36	-----			5,000	
			JJ								JJ						
Inlet Shaft		243,000	KK								KK						

TABLE 10-5. PRELIMINARY CONSTRUCTION COST ESTIMATE - ALTERNATIVE 'D2' - $16,440,000

TUNNEL - $14,918,750				CUT-OVER MAINS - $520,100							SLIPLINING - $998,000						
Comment	Tunnel & Liner	Structures	Exist. MH	Length	Size	Bore/ O.Cut	Pipe Cost	Manholes/ Connection	Fixed/Misc Costs	Total	Exist. MH	--	Length	Size	Sliplining Cost	Insertion Pits & MHs	Fixed/Misc Costs
$1,000,000	$13,918,750	$747,000	Totals	2,550			$429,590	$52,500	$38,000	$520,090	Totals		9,603		$732,997	$175,000	$90,000
Jct @ 84"		228,000	Jct								Jct	42	206	36	$12,422	5,000	
												42	-----				
Construction Shaft		68,000	A								A	42	I				
												42	I				
Mobe/Demobe			B								B	42	I				
1,000,000												42	I			5,000	
			C								C	42	I				
Tunnel Length= 16,375 ft												42	I			5,000	
	13,918,750		D								D	42	I			5,000	
												42	1,741	18	$149,030		10,000
			E								E	42	-----				
			F								F						
			G								G						
			H								H						
Inlet Shaft		50,000	I	60	15	OC	4,500	7,500	5,000	17,000	I						
			J								J						
Inlet Shaft		50,000	K	400	21	BJ	90,800	10,000	8,000	108,800	K	42	-----				
												42	I			5,000	
			L								L	42	I			5,000	
												42	I				
			M								M	42	I				
												42	I			10,000	
			N								N	42	I			5,000	
												42	I			5,000	
			O								O	42	I				
												42	I			5,000	
			P								P	42	I			10,000	
												42	I			5,000	
			Q								Q	42	I				
												42	2,693	36	$162,388		
			R								R	42	-----			5,000	
Vent. Shaft		22,000										42	I			5,000	
			S								S	42	I			5,000	
												42	I			10,000	
			T								T	42	I				
												42	I			5,000	
			U								U	42	I			5,000	
												42	2,965	18	$253,804	5,000	60,000
			V								V	42	-----				

TUNNEL - $14,918,750				CUT-OVER MAINS - $520,100							SLIPLINING - $998,000						
Comment	Tunnel & Liner	Structures	Exist. MH	Length	Size	Bore/ O.Cut	Pipe Cost	Manholes/ Connection	Fixed/Misc Costs	Total	Exist. MH	--	Length	Size	Sliplining Cost	Insertion Pits & MHs	Fixed/Misc Costs
																10,000	
			W	470	12	BJ	68,620	7,500	5,000	81,120	W						
Inlet Shaft		42,000	X						5,000		X						
			X1	420	12	BJ	61,320	7,500	5,000	73,820	X1						
			Y								Y						
			Z	670	12	BJ	97,820	7,500	5,000	110,320	Z						
			AA								AA						
Inlet Shaft		44,000	BB	530	18	BJ	106,530	12,500	5,000	124,030	BB	36	-----			5,000	
												36					
			CC								CC	36				10,000	
												36					
			DD								DD	36				5,000	
												36					
			EE								EE	36					
												36				10,000	
			FF								FF	36					
												36					
			GG								GG	36				10,000	
												36	1,650	27	$125,565		
			HH								HH	36	-----			10,000	
												36	348	18	$29,789		
			II								II	36	-----			5,000	20,000
			JJ								JJ						
Inlet Shaft		243,000	KK								KK						

TABLE 10-6

SUMMARY OF CONSTRUCTION COST ESTIMATES

Alternative	A	C	D1	D2
Tunnel	$16,710,000	$15,033,500	$14,638,250	$14,918,750
Cut-Over Mains	551,180	341,360	394,290	520,090
Sliplining	466,760	930,950	1,208,660	998,000
Const. Sub-Total	17,727,940	16,305,810	16,241,200	16,436,840
Easements (Admin Only)	36,000	48,000	384,000	228,000
30% Contingencies	5,329,182	4,906,143	4,987,560	4,999,452
Total (rounded)	$23,093,100	$21,260,000	$21,612,800	$21,664,300

11.0 CONCLUSIONS AND RECOMMENDATIONS

Estimated construction cost is not a decisive factor favoring any single Alternative. Alternative "A" with the longest tunnel has the highest total cost even though its costs for sliplining and cut-over mains are $250,000 to almost $600,000 less than the other Alternatives. Among Alternatives "C", "D1", and "D2", the cost estimates are very close. The estimated total project cost (including easements) for "D2" is only 2% more than for "C". Compared to the allowance for contingencies, this difference is not reliable for using cost estimates as a controlling factor for selection of the final Alternative.

Alternative "C" is the recommendation of this report due to lesser public impact than Alternatives "D1" and "D2". Alternative "C" would have more environmental impacts on account of access to the tunnel shaft sites and more cut-over mains. However, Alternative "C" may have better prospects for obtaining permits and approvals, with less public opposition at Planning Commission hearings for the required variances. Alternative "C" avoids the single-family easements and eliminates or reduces haul routes through residential neighborhoods.

It is also recommended that the complete project be constructed rather than staged in two or more phases. The flow monitoring data that was reviewed does not demonstrate overloading at the upper end of the 36-/42-inch interceptor (due to dry weather conditions), but the current level of dry weather flows strongly suggests that the 36-inch main does not have much spare capacity for wet weather I/I flows. Aside from observed overloading at the lower end apparently related to problems at the diversion box, the 36-inch may be more sensitive to wet weather overloading than the 42-inch main.

If staging is required due to limited funding, it is not feasible to construct the upper section of the relief tunnel in the first phase due to elevation constraints imposed by the need to tie into the existing interceptor. Alternative "A" has benefits if staging is required since it could relieve more of the existing interceptor for the currently available funding. Extension of Alternative "C" to manhole "L" would greatly reduce the flow at the existing diversion box, but without extension all the way to manhole "Z", it would not relieve any of the interceptor upstream of manhole "L". Any surplus from current funding could not be used for rehabilitation of the 42-inch pipe upstream of "L" since sliplining would reduce its capacity for current flows. Alternative "A" could provide more relief with available funds, but the ultimate cost for completion of the entire project is higher. With regard to cash flow, it might be possible to delay rehabilitation work on the existing interceptor, but this is only 3% to 7% of the total construction cost estimates.

It should be noted that there are possible variations within each Alternative and even different alternatives that are combinations of the selected Alternatives. For example, a combination of Alternatives "A" and "C" could start the relief tunnel at the 84-inch tunnel (to avoid the diversion box modification and

soft ground tunneling problems) and follow the "C" alignment to around manhole "N" where it would change to the Alternative "A" alignment in order to reduce the amount of sliplining. Such combinations mix the original objectives for each Alternative (e.g., maximize the cut-overs in order to abandon the 42-inch and minimize sliplining, shorten the tunnel length to reduce the project cost, etc.), but they may have practical applications considering funding and schedule. For portions of each Alternative, there can be some wiggle in the alignment in order to facilitate easement acquisition or to reduce environmental impacts in the design phase.

In this preliminary engineering phase, various alignment alternatives in a wide corridor were evaluated against different objectives. In the design phase a complete Environmental Assessment should be prepared for the selected Alternative and associated shaft sites, cut-over mains, and access routes. At that point detailed tree surveys along with additional geotechnical information can refine the horizontal and vertical alignment of the selected tunnel and its associated cut-over mains.

APPENDIX A

TERRA-MAR, INC.
Consulting – Geotechnical - Environmental
Construction Materials Testing

PRELIMINARY GEOTECHNICAL INVESTIGATION
LOWER WILLIAMSON CREEK RELIEF INTERCEPTOR
AUSTIN, TEXAS

TMI REPORT NO. AE97-033

TO

ESPEY, HUSTON & ASSOCIATES, INC.

AUSTIN, TEXAS

BY

TERRA-MAR, INC.

DALLAS / FORT WORTH / HOUSTON / AUSTIN / LONGVIEW / LUBBOCK

APRIL 16, 1999

TERRA-MAR

Consulting Engineers • Geotechnical • Environmental • Construction Materials Testing

DALLAS • FORT WORTH • HOUSTON • AUSTIN • LONGVIEW • LUBBOCK

April 16, 1999
TMI Report No. AE97-033

Mr. James L. Meara, P.E.
PBS&J
206 Wild Basin Road, Suite 300
Austin, Texas 78746-3343

Re: Preliminary Geotechnical Investigation
Lower Williamson Creek Relief Interceptor
Austin, Texas

Dear Mr. Meara:

Submitted here is our preliminary geotechnical report for the above referenced project. The study was performed in accordance with our agreement dated September 2, 1998, with authorization for project commencement provided on December 30, 1997. A draft copy of this report was submitted for review on July 22, 1998. Review comments by you and Mr. Peter H. Bush, P.E., of Fugro South, Inc. have been addressed herein. This report describes the results of our field and laboratory investigations together with preliminary recommendations for the design and construction of the referenced project.

We appreciate the opportunity to assist on this project and trust that our field and laboratory data and engineering recommendations will be of assistance in your design efforts. Do not hesitate to call if there are questions, or when we may be of further service.

Sincerely,

TERRA-MAR, INC.

Donald L. Anderson, P.E.
Project Engineer

Scott M. Pettit, P.E.
Vice President

STATE OF TEXAS
SCOTT M. PETTIT
62843
REGISTERED PROFESSIONAL ENGINEER
4-16-99

2621 Ridgepoint Drive, Suite #110, Austin, Texas 78754 Phone: 512-926-8224 Fax: 512-926-8768

TABLE OF CONTENTS

Page

1.0 INTRODUCTION — 1
1.1 PROJECT DESCRIPTION — 1
1.2 PURPOSE AND SCOPE — 2
2.0 FIELD AND LABORATORY INVESTIGATIONS — 3
2.1 FIELD INVESTIGATION — 3
2.2 LABORATORY INVESTIGATION — 5
3.0 SITE AND SUBSURFACE CONDITIONS — 6
3.1 GEOLOGICAL ELEMENTS — 6
3.2 HYDROLOGICAL ELEMENTS — 8
4.0 PRELIMINARY DESIGN RECOMMENDATIONS — 10
4.1 Tunnel considerations — 10
4.1.1 General — 10
4.1.2 Description of Potential Tunneling Methods and Procedures — 10
4.1.3 Requirements for Ventilation and Alignment Shafts — 11
4.1.4 Power requirements — 11
4.1.5 Tunnel Size — 12
4.1.6 Contractor Workspace — 12
4.1.7 Staging Areas — 12
4.1.8 Recommendations for Feasible Trenching and Tunneling Construction Methods — 13
4.1.9 Ventilation and Construction Shafts — 14
4.1.10 Vertical Tunnel Alignments — 14
4.1.11 Tunnel Support, Rock Tunnel — 14
4.1.12 Tunnel Support, Soft Ground Tunnels — 15
4.1.13 Muck Removal — 15
4.1.14 Groundwater Removal and Disposal — 15
4.1.15 Tunnel Costs — 16
4.1.16 Trench Safety Program — 18
4.1.17 Horizontal Tunnel Alignment — 18
4.2 OPEN CUT INSTALLATION — 19
4.2.1 Existing Conditions and OSHA Classifications — 19
4.2.2 Shoring, Bracing and Sidewall Slope Recommendations — 21
4.2.3 Construction and Other Considerations — 22
5.0 INSPECTION AND TESTING — 24
6.0 LIMITATIONS — 24

TABLE OF CONTENTS, continued

FIELD & LABORATORY PROCEDURES ---------- Appendix A
SUMMARIES OF SIMILAR TUNNELING PROJECTS ---------- Appendix B
ROCK CORE SAMPLE PHOTOGRAPHS ---------- Appendix C

Figure

PLAN OF BORINGS ---------- 1
LOG OF BORINGS ---------- 2-18
LEGEND - LOG OF BORINGS ---------- 19-20
BORING LOG PROFILE ---------- 21
RECOMMENDED SLOPE RATIOS ---------- 22
UNCONFINED COMPRESSION TEST RESULTS ---------- 23-29

1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION

The City of Austin Water and Wastewater Department is planning to install a new wastewater interceptor to relieve an existing 42-inch wastewater line along Williamson Creek in south Austin, Texas. Currently, a 60-inch diameter relief interceptor is planned. The new line will be approximately 14,400 feet in length. The new line will typically parallel Williamson Creek and will extend from just east of Pleasant Valley Road to approximately 3,200 feet west of Interstate Highway 35.

The relief interceptor project is in the preliminary design stage. Currently, it is anticipated that the pipe invert elevation will be established near elevation 545 feet at the west end of the planned alignment. The pipeline will slope downward at an approximate 0.02% grade to elevation 515 feet at the east end. The east end of the line will then tie into an existing junction box, although some consideration is being given to extending the line further east.

The pipeline will typically be embedded 20 to 35 feet below the ground surface. A possible alternative being considered by the city places the invert elevation well below 35 feet (this alternative is to be addressed at a later date). In some areas, the pipe may be installed in an open trench cut. However, in other areas, it is likely that the pipe will be installed by horizontal boring, or tunneling, methods. A horizontal boring will most likely be needed to install the new pipeline beneath IH-35. Additionally, extensive horizontal boring may be desirable in other areas to limit disturbance to City of Austin parkland along the creek.

The flow in the existing 42-inch wastewater line will be designed to discharge into the new relief interceptor. Depending upon the installation method chosen along the various sections of the new alignment, the existing line may be redirected to discharge into connector shafts emptying into the new pipeline, or feeder lines presently connected to the existing line will be extended to discharge into the new relief interceptor.

The western end of the proposed alignment begins between South Congress Avenue and Interstate Highway 35 in the Williamson Creek Greenbelt adjacent to Battle Bend Drive. The alignment proceeds easterly along the north side of Williamson Creek approximately 2000 feet. The alignment then crosses to the south side of the creek and crosses Interstate Highway 35. After crossing Interstate Highway 35, the alignment remains in the Williamson Creek Greenbelt

LOG OF BORING
BORING B-2

Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
510 — 40										
505 — 45										
500 — 50										
495 — 55										
490 — 60										
485 — 65										
480 — 70										
475 — 75										
470 — 80										

FIGURE NO.: 4

LOG OF BORING
BORING B-3

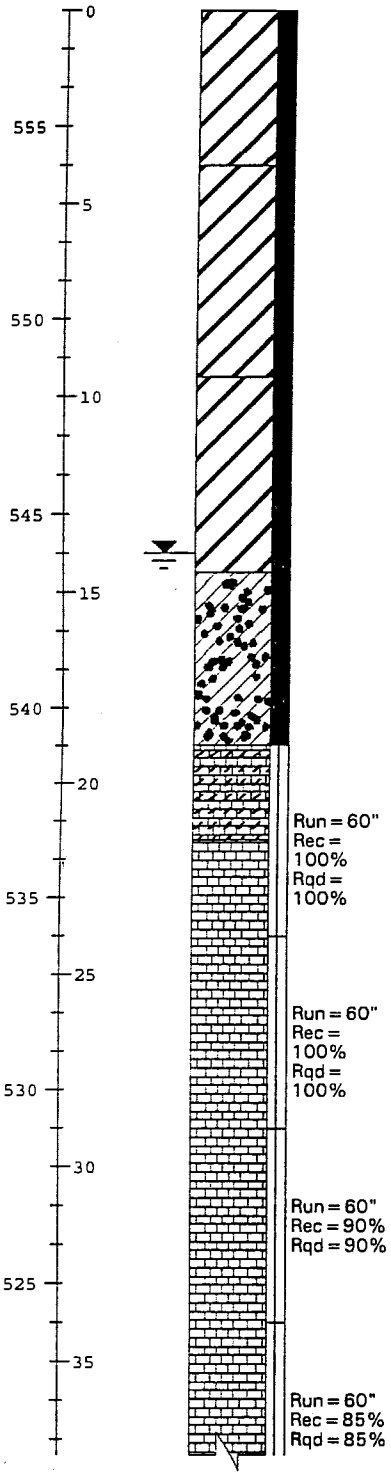
Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

Date: **3-4-98** Elev.: **558.0 +/-** Location: **See Figure 1**

Depth to water at completion of boring: **N/A (Bailed)**

Depth to water when checked: **3-5-98** was: **14.0'**

Depth to caving when checked: **3-5-98** was: **39.0'**



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
		Very stiff dark grayish brown lean CLAY w/ some limestone gravel (Residual Taylor) (CL)								
		Hard grayish brown fat CLAY w/ some limestone gravel (Residual Taylor) (CH)	16	70	20	50				6.6
		Hard pale yellow-brown fat CLAY w/ some limestone gravel (Residual Taylor) (CH)	18	68	20	48				5.2
		Dense yellow-brown clayey GRAVEL (Residual Taylor) (GC)								
		Soft weathered tan LIMESTONE w/fracture @ 21.5' w/ iron deposition (Austin Chalk)								
		Moderately hard light gray to medium light gray LIMESTONE w/ calcite crystals, amorphous calcite, finely crystalline calcareous material, foraminiferan shells and fragments, debris of inoceramus, pelecypods, gastropods, echinoids, and other organic fragments. Glauconite and pyrite, both nodules and disseminated, are common. The chalky, impure limestone (containing 85% or more of calcium carbonate) is very fine-grained with few fossil fragments and no secondary diagenetic changes. - dark gray & medium dark gray mottled & burrowed zones @ 22', & 23'-24' - medium light gray mottled & burrowed zones @ 25', 28', 31', 35', 36', & 38'.								

Notes: **Began drilling at 8:45 a.m. on 3-4-98. Set hollow-stem auger at 9', lost circulation. Reset at 14', lost circulation. Reset at 19', began coring. Completed drilling at 6:20 p.m. on 3-4.Completion Depth: 44.0'**

FIGURE NO.: 5

LOG OF BORING
BORING B-3

Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
520										
40										
	Run=60" Rec=100% Rqd=100%									
515		- medium light gray mottled & burrowed zones @ 42' (Austin Chalk)								
45		Note: Approximate tunnel invert elevation is 524'.								
510										
50										
505										
55										
500										
60										
495										
65										
490										
70										
485										
75										
480										
80										

FIGURE NO.: 6

LOG OF BORING
BORING B-4

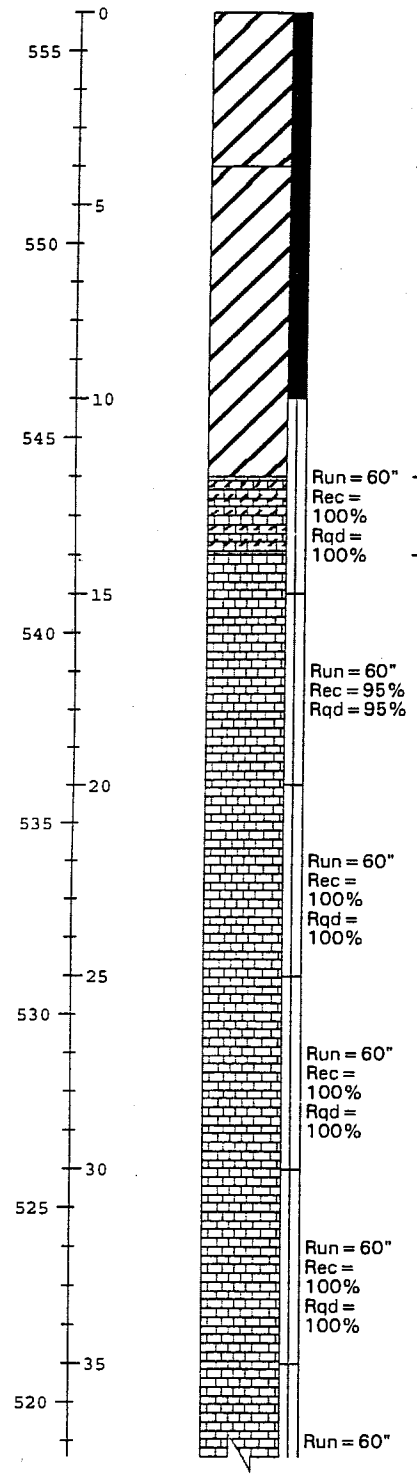
Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

Date: **3-3-98** Elev.: **556.0 +/-** Location: **See Figure 1**

Depth to water at completion of boring: **Dry (Prior to Coring)**

Depth to water when checked: **N/A** was: **N/A**

Depth to caving when checked: **N/A** was: **N/A**



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
		Very stiff dark yellowish brown lean CLAY w/ some limestone gravel (Residual Taylor) (CL)								
		Very stiff grayish brown lean CLAY w/ some limestone gravel	21	49	18	31				3.6
		- pale yellow brown & gray below 10' (Residual Taylor) (CL)	20	48	20	28				
		Very soft tan severely weathered LIMESTONE w/ clay layers - dark gray & medium dark gray mottled & burrowed zones @ 13' & 16' (Austin Chalk)								
		Moderately hard light gray to medium light gray LIMESTONE w/ calcite crystals, amorphous calcite, finely crystalline calcareous material, foraminiferan shells and fragments, debris of inoceramus, pelecypods, gastropods, echinoids, and other organic fragmentes. Glauconite and pyrite, both nodules and disseminated, are common. The chalky, impure limestone (containing 85% or more of calcium carbonate) is very fine-grained with few fossil fragments and no secondary diagenetic changes. - medium gray & medium dark gray mottled & burrowed zones @ 19', 21' & 23' - fracture @ 20.5' with iron stains extending 2-3" into matrix. Fracture showed no displacement. - medium gray & medium dark gray mottled & burrowed zones @ 27', 28', 30', and 32-35'.								

Notes: **Drilling began at 10:45 a.m. on 3-3-98, completed at 6:30 p.m. on 3-3-98. Set hollow-stem auger at 10'. Completion Depth: 45.0'**

FIGURE NO.: 7

LOG OF BORING
BORING B-4

Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

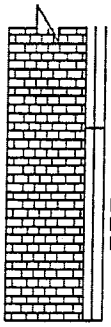
ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
40 515 45 510 50 505 55 500 60 495 65 490 70 485 75 480 80	 Rec = 100% Rqd = 100% Run = 60" Rec = 90% Rqd = 90%	- medium gray & medium dark gray mottled & burrowed zones @ 37'-38' - medium gray & medium dark gray mottled & burrowed zones @ 43' (Austin Chalk) Note: Approximate tunnel invert elevation is 526'.								

FIGURE NO.: 8

LOG OF BORING
BORING B-5

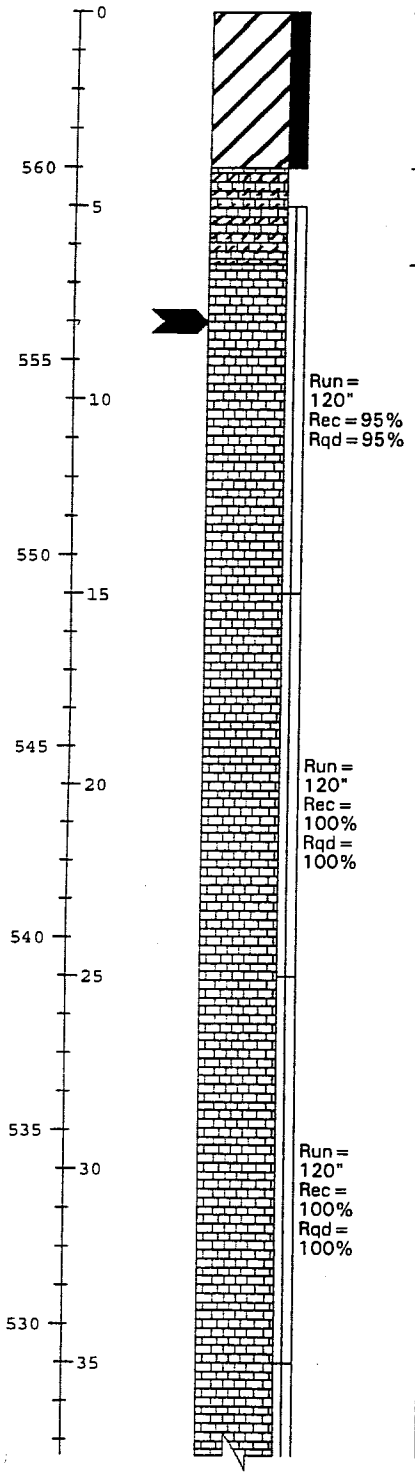
Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

Date: **2-6-98** Elev.: **564.0 +/-** Location: **See Figure 1**

Depth to water at completion of boring: **Dry (Bailed)**

Depth to water when checked: **2-7-98** was: **Dry**

Depth to caving when checked: **2-7-98** was: **8.0'**



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
		Very stiff to hard dark brown lean CLAY w/ some limestone gravel							3.5	
			14	34	19	15			3.0	
									3.0	
		(Residual Austin) (CL)	15	31	17	14			4.5+	
		Very soft severely weathered tan LIMESTONE w/ clay layers - fracture w/ iron stains @ 6.5' (Austin Chalk)								
		Moderately hard light gray to medium light gray LIMESTONE w/ calcite crystals, amorphous calcite, finely crystalline calcareous material, foraminiferan shells and fragments, debris of inoceramus, pelecypods, gastropods, echinoids, and other organic fragments. Glauconite and pyrite, both nodules and disseminated, are common. The chalky, impure limestone (containing 85% or more of calcium carbonate) is very fine-grained with few fossil fragments and no secondary diagenetic changes. - dark gray & medium dark gray mottled & burrowed zones @ 7 & 12' - possible slickenside @ 19'	7					141.1		179.3
		- medium gray & medium dark gray mottled & burrowed zone @ 22'	6					144.2		223.3
			7					143.0		163.5
		- medium gray & medium dark gray mottled & burrowed zone @ 26'-28'								
			8					136.9		193.3

Notes: **Drilling began at 4:15 p.m. on 2-6-98. Completed drilling at 6:30 p.m.on 2-6-98. Completion Depth: 45.0'**

FIGURE NO.: 9

LOG OF BORING
BORING B-5

Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

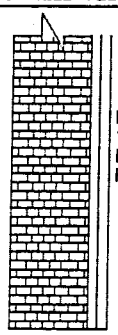
ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
525	 Run = 120" Rec = 85% Rqd = 85%									
40										
520		- possible slickenside @ 43.5' w/ secondary calcite filling @ 45 degrees								
45		Note: Approximate tunnel invert elevation is 529'.								
515										
50										
510										
55										
505										
60										
500										
65										
495										
70										
490										
75										
485										
80										

FIGURE NO.: 10

LOG OF BORING
BORING B-6

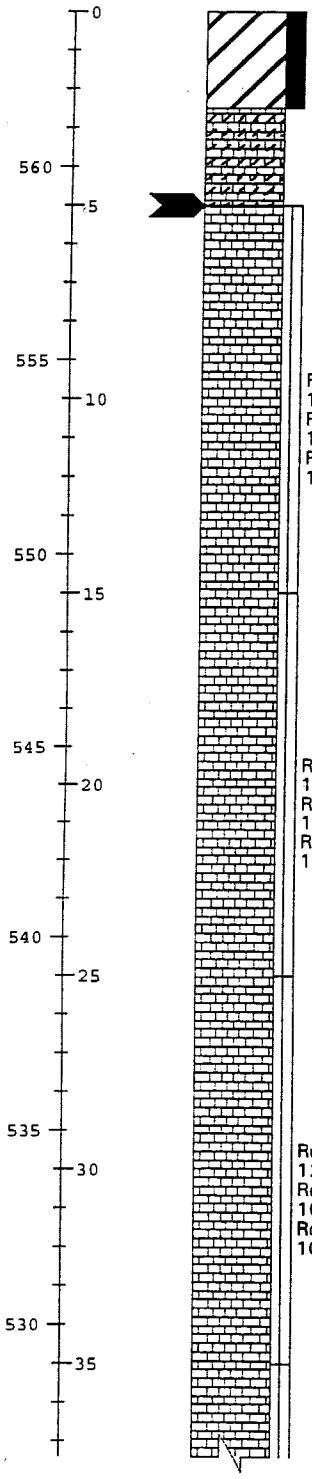
Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

Date: **2-6-98** Elev.: **564.0 +/-** Location: **See Figure 1**

Depth to water at completion of boring: **Dry (Bailed)**

Depth to water when checked: **2-7-98** was: **Dry**

Depth to caving when checked: **2-7-98** was: **5.0'**



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
		Very stiff to hard dark brown to light gray & yellow brown lean CLAY w/ some limestone gravel (Residual Austin) (CL)	15	33	15	18			2.5 1.5 4.5+	0.8
		Very soft tan severely weathered LIMESTONE w/ clay layers - fracture w/ iron stains @ 6.5'	14	26	14	12				
		Moderately hard light gray to medium light gray LIMESTONE w/ calcite crystals, amorphous calcite, finely crystalline calcareous material, foraminiferan shells and fragments, debris of inoceramus, pelecypods, gastropods, echinoids, and other organic fragments. Glauconite and pyrite, both nodules and disseminated, are common. The chalky, impure limestone (containing 85% or more of calcium carbonate) is very fine-grained with few fossil fragments and no secondary diagenetic changes. - fractures with iron stains extending 2" into matrix @ 6.5' - fractures with iron stains extending 10" into matrix @ 8' - medium gray & medium dark gray mottled & burrowed zone @ 11'-13'	5					147.7		210.3
		- medium gray & medium dark gray mottled & burrowed zone @ 22'-23'	6					145.0		208.2
			5					147.5		247.4
		- medium gray & medium dark gray mottled & burrowed zone @ 36'-37'	11					130.9		47.2

Notes: **Drilling began at 11 a.m. on 2-6-98. Completed at 2 p.m. Set hollow-stem auger at 5'. Completion Depth: 41.0'**

FIGURE NO.: 11

LOG OF BORING
BORING B-6

Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
525	Run=72" Rec=90% Rqd=90%	(Austin Chalk)								
40										
		Note: Approximate tunnel invert elevation is 533'.								
520										
45										
515										
50										
510										
55										
505										
60										
500										
65										
495										
70										
490										
75										
485										
80										

FIGURE NO.: 12

LOG OF BORING
BORING B-7

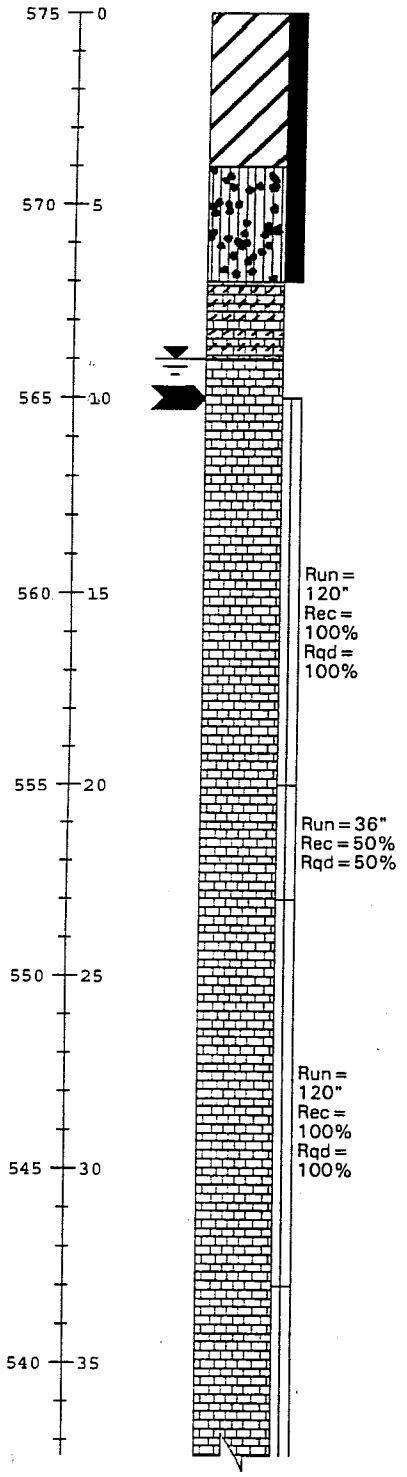
Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

Date: **2-4-98** Elev.: **575.0 +/-** Location: **See Figure 1**

Depth to water at completion of boring: **N/A**

Depth to water when checked: **2-5-98** was: **9.0'**

Depth to caving when checked: **2-5-98** was: **10.0'**



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
		Very stiff to hard dark brown to light gray & yellow brown lean CLAY w/ some limestone gravel (Terrace Deposit) (CL)	15	37	17	20			3.0 2.5 3.5 4.5+	
		Dense light brown silty GRAVEL (Terrace Deposit) (GM)								
		Very soft tan severely weathered LIMESTONE w/ clay layers (Austin Chalk)								
		Moderately hard light gray to medium light gray LIMESTONE w/ calcite crystals, amorphous calcite, finely crystalline calcareous material, foraminiferan shells and fragments, debris of inoceramus, pelecypods, gastropods, echinoids, and other organic fragments. Glauconite and pyrte, both nodules and disseminated, are common. The chalky,impure limestone (containing 85% or more of calcium carbonate) is very fine-grained with few fossil fragments and no seconary diagenetic changes. - fossil fragments @ 10' - reworked zones @ 13'-15' - fossil hash @ 19'-20'	6					144.7		234.1
		- filled worm burrows w/ finely disseminated glauconite @ 23'-33'	7					141.6		227.0
			7					141.4		168.8

Notes: **Drilling began at 8:30 a.m. on 2-4-98. Completed at 3 p.m. on 2-4-98. Hollow-stem auger set to 10'. Completion Depth: 52.0'**

FIGURE NO.: 13

LOG OF BORING
BORING B-7

Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON tsf
535 — 40	Run = 120" Rec = 100% Rqd = 100%		5					146.7		260.0
		- pale green shell hash @ 43.5'								
530 — 45										
	Run = 108" Rec = 100% Rqd = 100%									
525 — 50		(Austin Chalk)								
		Note: Approximate tunnel invert elevation is 535'.								
520 — 55										
515 — 60										
510 — 65										
505 — 70										
500 — 75										
495 — 80										

FIGURE NO.: 14

LOG OF BORING
BORING B-8

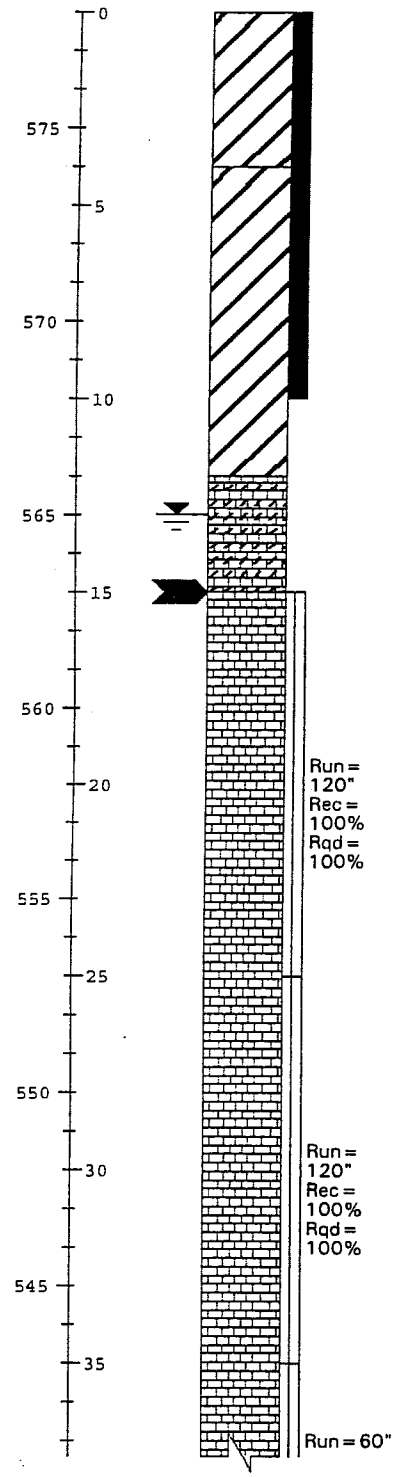
Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

Date: **2-5-98** Elev.: **578.0 +/-** Location: **See Figure 1**

Depth to water at completion of boring: **N/A**

Depth to water when checked: **2-6-98** was: **13'**

Depth to caving when checked: **2-6-98** was: **15'**



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
		Very stiff to hard brown & light brown lean CLAY - silty sand @ surface w/ trace clay (FILL) (CL)	14	35	18	17			2.5 4.5+ 4.5+ 4.5+	
		Medium stiff to stiff light gray & pale yellow brown fat CLAY w/ gravel (FILL) (CH)	24 27	57 56	14 21	43 35			3.0	1.1
		Very soft tan severely weathered LIMESTONE w/ clay layers (Austin Chalk)								
		Moderately hard light gray to medium light gray LIMESTONE w/ calcite crystals, amorphous calcite, finely crystalline calcareous material, foraminiferan shells and fragments, debris of inoceramus, pelecypods, gastropods, echinoids, and other organic fragments. Glauconite and pyrite, both nodules and disseminated, are common. The chalky, impure limestone (containing 85% or more of calcium carbonate) is very fine-grained with few fossil fragments and no secondary diagenetic changes. - fracture zone @ 15'-15.5' w/ iron stain penetrating matrix - fracture @ 17' w/ iron stain penetration 3" into matrix - medium gray to dark medium gray mottled & burrowed zone @ 19', 23', 27' & 28'.	7 8					141.1 140.1		122.5 195.0
		- medium gray & dark medium gray mottled & burrowed zone @ 36'	5					146.3		263.9

Notes: **Drilling began at 1:25 p.m. on 2-5-98. Completed at 4:30 p.m. Set hollow-stem auger at 15'. Completion Depth: 40.0'**

FIGURE NO.: 15

LOG OF BORING
BORING B-8

Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
540	Rec = 100% Rqd = 100%	(Austin Chalk)	8					138.9		92.6
40		Note: Approximate tunnel invert elevation is 539'.								
535										
45										
530										
50										
525										
55										
520										
60										
515										
65										
510										
70										
505										
75										
500										
80										

FIGURE NO.: 16

LOG OF BORING
BORING B-9

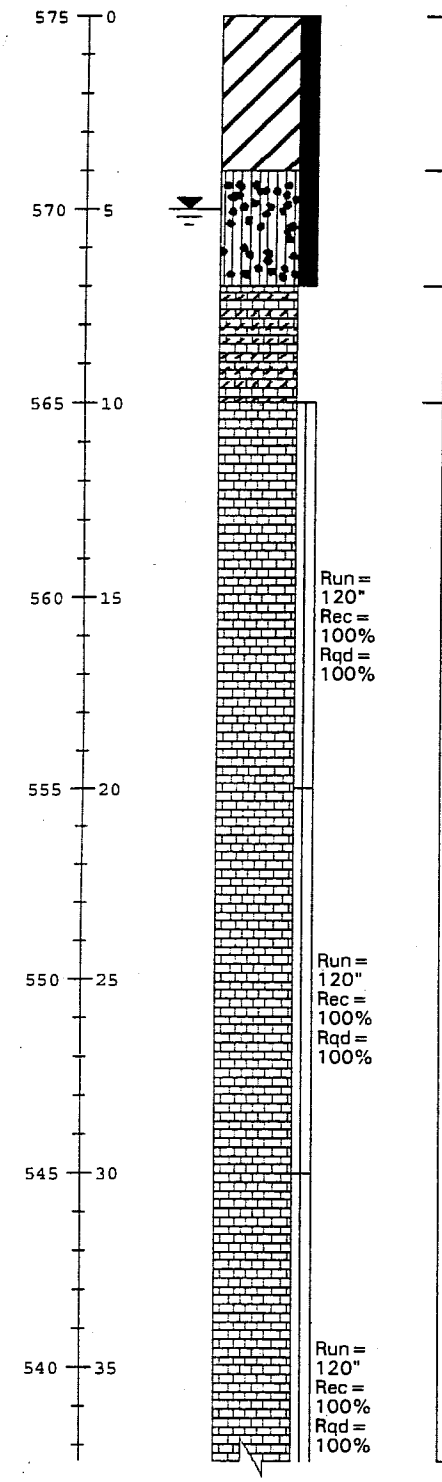
Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

Date: **2-4-98** Elev.: **575.0 +/-** Location: **See Figure 1**

Depth to water at completion of boring: **N/A (Bailed)**

Depth to water when checked: **2-5-98** was: **5'**

Depth to caving when checked: **N/A** was: **N/A**



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
		Stiff to very stiff dark brown lean CLAY	19	46	19	27			1.0 4.5+ 4.5+	
		(Terrace Deposit) (CL)	17	41	17	24			3.0	
		Medium dense to dense light brown silty GRAVEL (Terrace Deposit) (GM)								
		Very soft tan severely weathered LIMESTONE (Austin Chalk)	17							
		Moderately hard light gray to medium light gray LIMESTONE w/ calcite crystals, amorphous calcite, finely crystalline calcareous material, foraminiferan shells and fragments, debris of inoceramus, pelecypods, gastropods, echinoids, and other organic fragments. Glauconite and pyrite, both nodules and disseminated, are common. The chalky, impure limestone (containing 85% or more of calcium carbonate) is very fine-grained with few fossil fragments and no secondary diagenetic changes.	6					146.0		200.2
		- worm burrows & shell hash @ 10'-12' - mottled zone @ 15' - medium gray & dark medium gray mottled & burrowed zone @ 23'	7					141.0		190.8
		- medium gray & dark medium gray mottled & burrowed zones @ 26' & 28'	7					142.5		154.7
		- medium gray & dark medium gray mottled & burrowed zones @ 32'	9					137.1		199.2

Notes: **Drilling began at 3:30 p.m. on 2-4-98 and was completed at 6 p.m.**
Hollow-stem auger set to 10'. Completion Depth: 44.0'

FIGURE NO.: 17

LOG OF BORING
BORING B-9

Project: **Lower Williamson Creek Relief Interceptor** Project No.: **AE97-033**

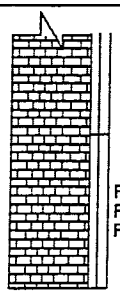
ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI	-200 %	D.D. pcf	P.PEN tsf	UNCON. tsf
535 — 40	 Run=48" Rec=90% Rqd=90%	- medium gray & dark medium gray mottled & burrowed zones @ 38'								
		- medium gray & dark medium gray mottled & burrowed zones @ 41' (Austin Chalk)								
530 — 45		Note: Approximate tunnel invert elevation is 543'.								
525 — 50										
520 — 55										
515 — 60										
510 — 65										
505 — 70										
500 — 75										
495 — 80										

FIGURE NO.: 18

BORING LOG SYMBOLS AND TERMS FOR SOIL

SOIL AND MATERIAL TYPES*

- Clay (CH, CL)
- Shaley Clay (CH, CL)*
- Silt (MH, ML)
- Silty Clay (CL, CH)*
- Sand (SP, SW)
- Sandy Clay (CL, CH)*
- Gravel (GP, GW)
- Clayey Sand (SC)*
- Asphalt Paving
- Concrete Paving

SAMPLER AND INDICATOR TYPES

- Thin Walled Shelby Tube
- Rock Core
- Standard Penetration
- THD Cone Penetrometer
- Bulk/Grab Sample
- Solid or Hollow Stem Auger
- Water level at Completion
- Water Level on the Date Indicated
- Depth to Caving at Completion
- Boring Continues

*Note: Dual symbols are used to indicate borderline and mixed soil classifications

STRENGTH OF COHESIVE SOILS

Soil Consistency	Pocket Penetrometer Reading, tsf
Very Soft	< 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very Stiff	2.00 to 4.00
Hard	> 4.00

DENSITY OF GRANULAR SOILS

SPT Blows per foot, N	Relative Density
0 to 4	Very Loose
4 to 10	Loose
10 to 30	Medium Dense
30 to 50	Dense
> 50	Very Dense

DESCRIPTIVE TERMS FOR SOIL

Description	Criteria
Stratified	Alternating layers of material or color with layers at least 1/4" thick
Laminated	Alternating layers of material or color with layers at least 1/4" thick
Fissured	Breaks along definite planes of fracture with little resistance
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small hard angular lumps
Calcareous	Containing appreciable quantities of calcium carbonate. (Reactive with HCl)
Ferrous	Containing appreciable quantities of ferrous oxide/iron nodules and/or stains

Inclusions

Parting	Inclusion < 1/8" thick
Seam	Inclusion 1/8" to 3" thick
Layer	Inclusion > 3" thick
Trace	< 5% of sample
Few	5% to 10% of sample
Some	10% to 25% of sample
Numerous	25% to 50% of sample

Soil Moisture

Dry	No obvious water in sample
Moist	Sample is damp without excess water
Very Moist	Sample is damp and water is visible
Wet	Sample bears free water

SOIL GRAIN SIZE

Soil Grain Size in Millimeters

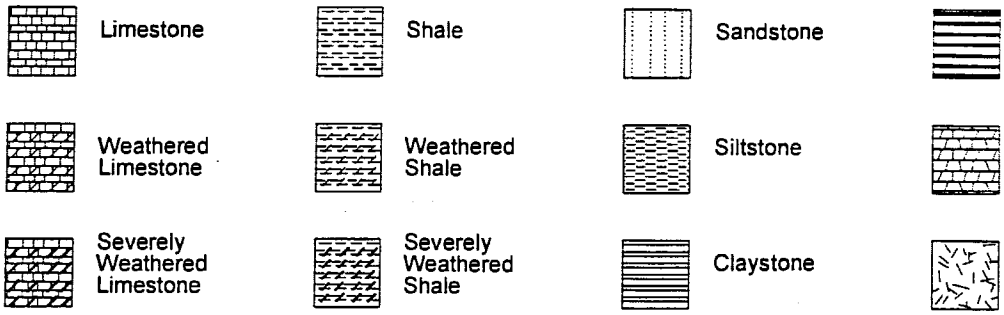
152	76.2	19.1	4.76	2.00	0.420	0.074	0.002
Boulders	Cobbles	Gravel		Sand		Silt	Clay
		Coarse	Fine	Coarse	Medium	Fine	

6"	3"	3/4"	4	10	40	200	270

U.S. Standard Sieve Size

BORING LOG SYMBOLS AND TERMS FOR ROCK

ROCK TYPES



ROCK HARDNESS

Friable	Crumbles under hard pressure
Very Soft	Dented w/ moderate finger pressure
Soft	Scratched easily with fingernail
Moderately hard	Can be scratched easily with knife but not fingernail
Hard	Can be scratched with knife with some difficulty and broken with light to moderate hammer blow
Very Hard	Cannot be scratched with knife and can be broken by one to several hard hammer blows

ROCK HARDNESS BY THD CONE

Description	Penetration per 100 Blows
Very Soft	> 6"
Soft	5" to 6"
Soft to Hard	4" to 5"
Hard	2" to 4"
Hard to Very Hard	1" to 2"
Very Hard	0" to 1"

WEATHERING GRADES OF ROCK MASS

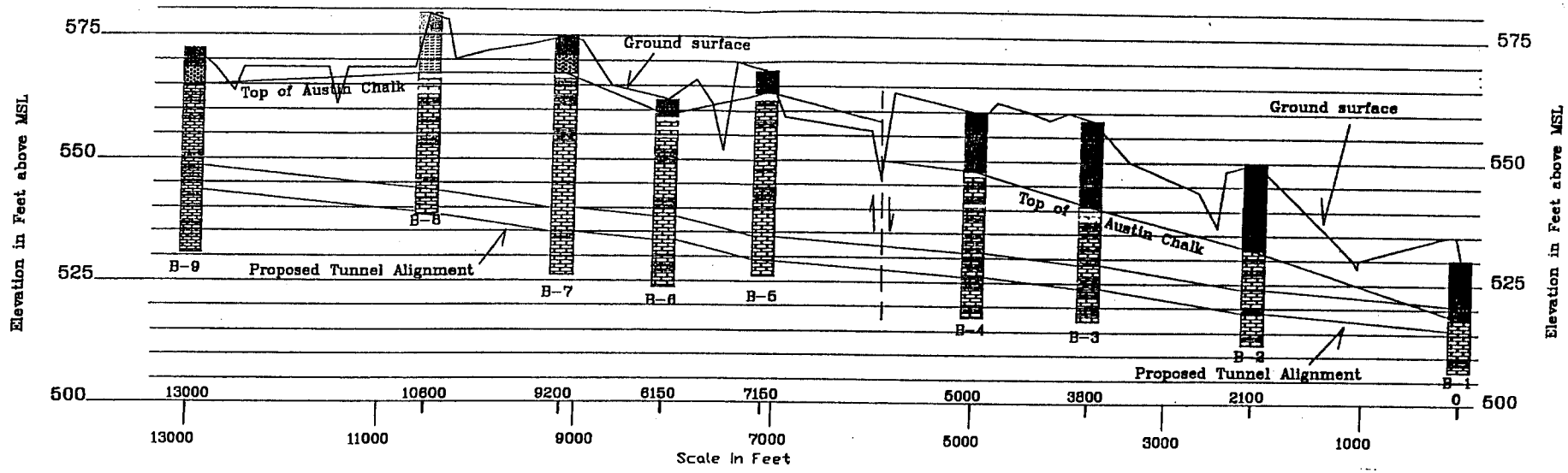
Slightly Weathered	Some discoloration indicates weathering of rock, may be some small decomposed rock pockets, layers and/or seams
Weathered	Discoloration of majority to all of the rock mass, with trace to some of decomposed rock pockets, layers and/or seams
Severely Weathered	All of the rock material is discolored and most of the rock mass has decomposed into a soil with the original mass structure still largely intact

JOINT DESCRIPTION

Spacing		Inclination (degrees)		Surfaces	
Very Close	<2"	Horizontal	0-5	Slickensided	Polished
Close	2"-12"	Shallow	5-35	Smooth	Planar
Close to Wide	12"-3'	Moderate	35-65	Irregular	Granular
Wide	>3'	Steep	65-85	Rough	Jagged
		Vertical	85-90		

BEDDING THICKNESS

Very Thick	>4'
Thick	2' to 4'
Thin	2" to 2'
Very Thin	0.5" to 2"
Laminated	0.08" to 0.5"
Thinly Laminated	<0.08"



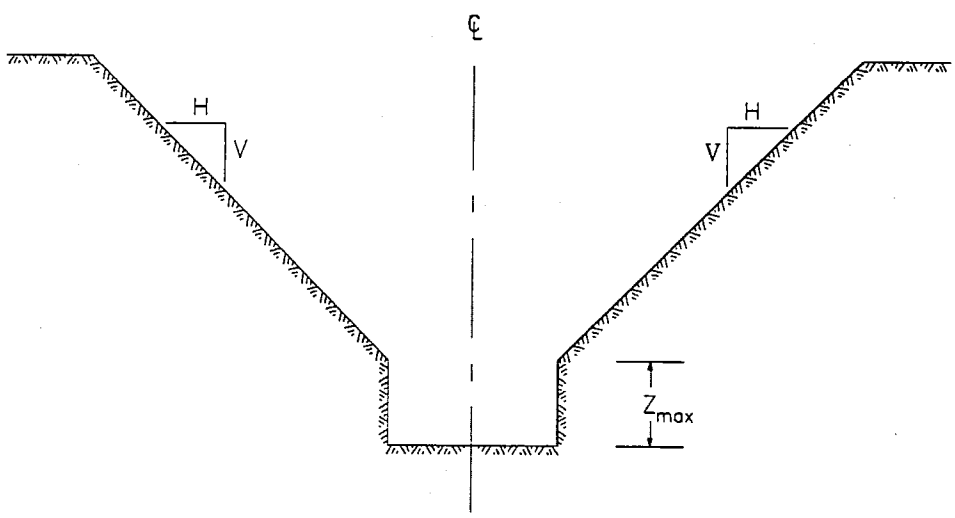
Refer to Report and Boring Logs for more detail.
A weathered zone approximately 2 feet thick exists at the top of the Austin Chalk.

Not to Scale

Preliminary Geotechnical Investigation Williamson Creek Relief Interceptor Austin, Texas	TERRA-MAR, INC. Boring Log Profile Figure 21
Terra-Mar No. AE97-033	

SUGGESTEDED SLOPE RATIOS BASED ON OSHA SOIL CLASSIFICATION

	Short Term (under 8 hours)		Long Term (over 8 hours)		Bedding Cut*
SOIL / ROCK	H	V	H	V	Z (ft.)
Type A 1) Cohesive soils with an unconfined compressive strength >1.5 tsf, 2) caliche and/or hardpan	¾	1	1	1	4
Type B 1) Cohesive soils with an unconfined compressive strength >0.5 tsf but <1.5 tsf, 2) granular cohesionless soils such as angular gravel, silt, silt loam, sandy loam, silty/sandy clay loam, 3) Type A soils that are fissured or subject to vibration, 4) previously disturbed soils except those that would be classified as Type C, 5) dry rock that is not stable, 6) material that is part of a sloped layered system where layers dip into the excavation at a slope flatter than 4H:1V.	1	1	1½	1	3½
Type C 1) Cohesive soils with an unconfined compressive strength <0.5 tsf, 2) granular soils such as gravel, sand, and loamy sand, 3) submerged soil/soil from which water is seeping, 4) submerged rock that is not stable, 5) material that is part of a sloped layered system where layers dip into the excavation at a slope steeper than 4H:1V.	1½	1	2	1	N/A
Stable Rock 1) A natural solid mineral matter that can be excavated with vertical sides and remain intact while exposed	¼	1	¼	1	4

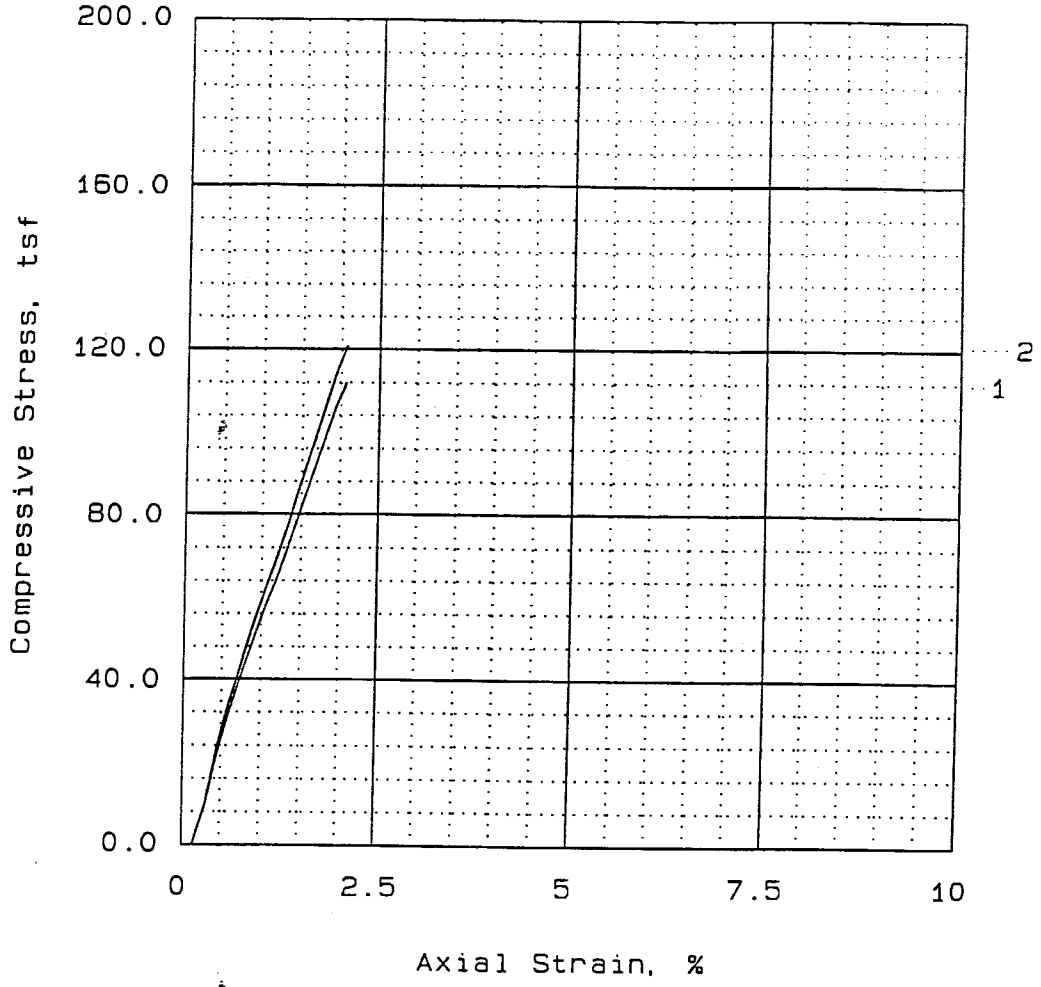


* Bedding cuts are only to be used in trench excavations that are less than 12 feet deep in dry soil and/or rock which is open less than 8 hours.

NOTE:
1) The design of the actual slope ratios is the sole responsibility of the Contractor.
2) Recommended slope ratios will be subject to reduced stability under saturation by rain.
3) Excavation sideslopes are required for all un-shored excavations regardless of depth.
4) Slopes are excavations less than 20 feet. Excavations greater than 20 feet shall be designed by a registered professional engineer.

TERRA-MAR, INC.	Preliminary Geotechnical Investigation Williamson Creek Relief Interceptor Austin, Texas			Recommended Slope Ratios		FIGURE
	DRAWN: MTM	DATE: 07-22-98	REVISED: 2-12-98	SCALE:	JOB NO.: AE97-033	22

UNCONFINED COMPRESSION TEST



Sample number:	1	2		
Unconfined strength, tsf	111.8	120.9		
Undrained shear strength, tsf				
Rate of strain, %/min				
Water content, %	9.9	9.6		
Void ratio				
Saturation, %				
Dry density, pcf	132.9	134.0		
Specimen diameter, in	2.04	1.96		
Specimen height, in	4.08	4.06		

Description: Gray Limestone

LL = N/A	PL = N/A			Type: Rock Core

Project No.: AE97-033
Date: 2-19-98

Remarks:
1 - B-1 @ 15.3'-16.1'
2 - B-1 @ 19.4'-20.0'

Client: Espey, Huston & Associates, Inc.

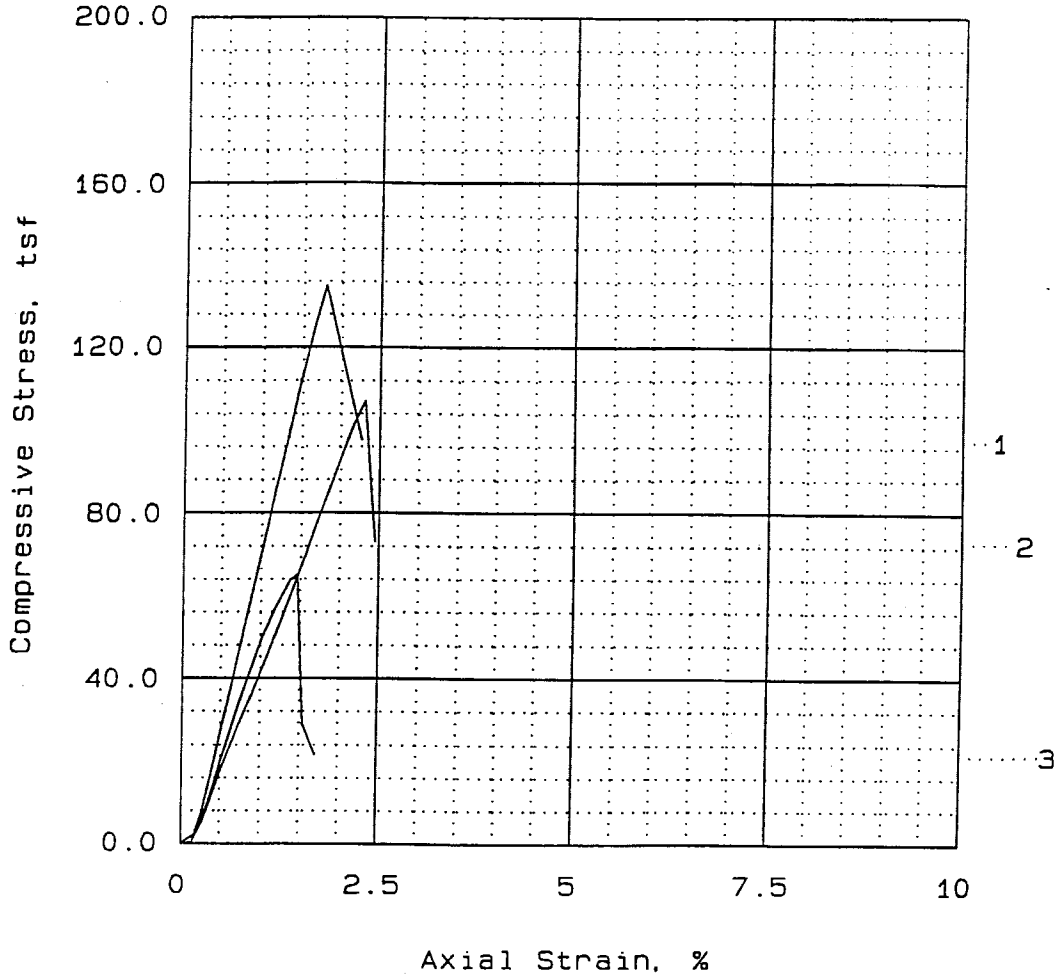
Project: Lower Williamson Creek
Relief Interceptor
Location: See Remarks

Fig No. 23

UNCONFINED COMPRESSION TEST

TERRA-MAR, INC.

UNCONFINED COMPRESSION TEST



Sample number:	1	2	3	
Unconfined strength, tsf	135.1	72.9	56.8	
Undrained shear strength, tsf				
Rate of strain, %/min				
Water content, %	9.9	10.3	9.5	
Void ratio				
Saturation, %				
Dry density, pcf	132.2	132.7	134.0	
Specimen diameter, in	1.96	2.03	1.97	
Specimen height, in	3.93	4.06	3.35	

Description: Gray Limestone

LL = N/A	PL = N/A			Type: Rock Core

Project No.: AE97-033

Date: 2-19-98

Remarks:

1 - B-2 @ 27.2'-28.0'

2 - B-2 @ 31.6'-32.5'

3 - B-2 @ 34.0'-34.8'

Client: Espey, Huston & Associates, Inc.

Project: Lower Williamson Creek Relief Interceptor

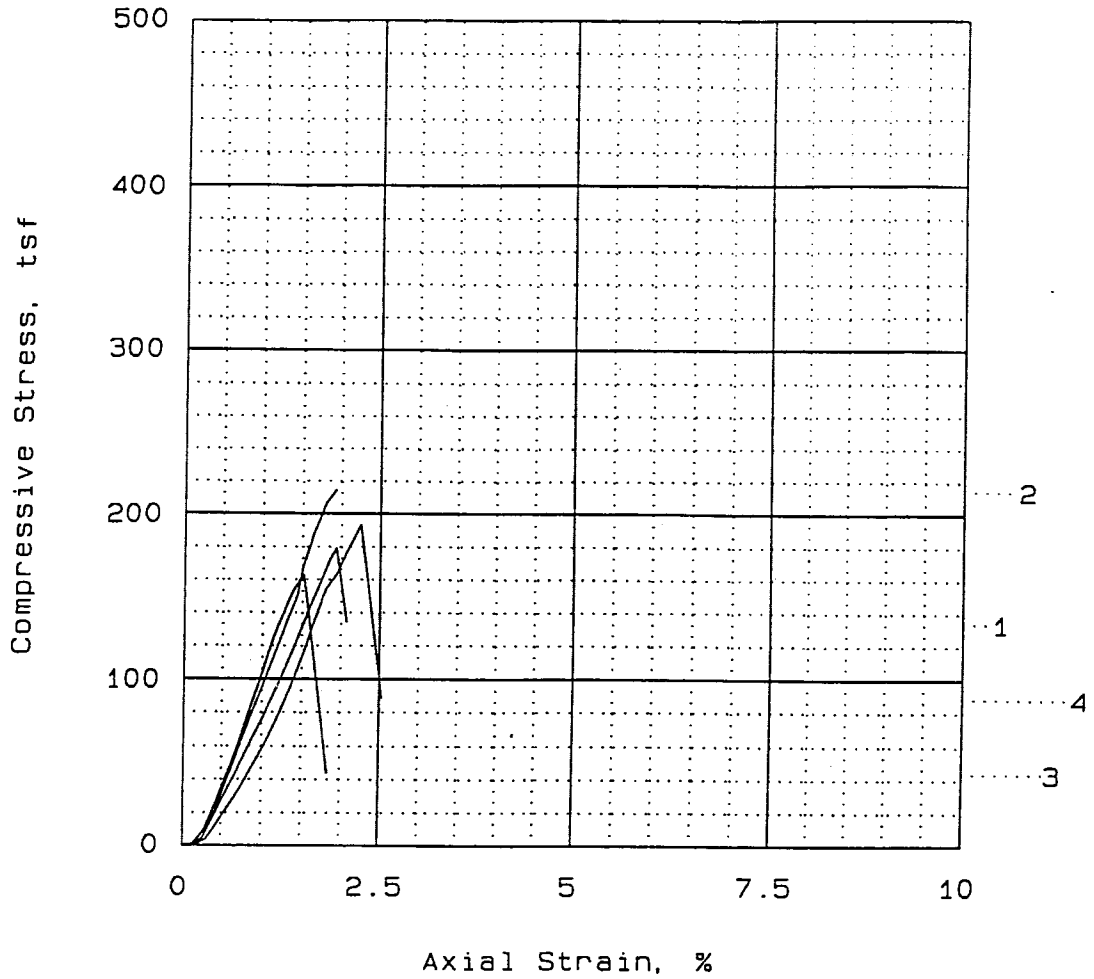
Location: See Remarks

UNCONFINED COMPRESSION TEST

TERRA-MAR, INC.

Fig No. 24

UNCONFINED COMPRESSION TEST



Sample number:	1	2	3	4
Unconfined strength, tsf	179	134	163	193
Undrained shear strength, tsf				
Rate of strain, %/min				
Water content, %	6.9	6.0	6.6	8.2
Void ratio				
Saturation, %				
Dry density, pcf	141.2	144.3	143.0	137.0
Specimen diameter, in	2.05	2.04	1.96	2.00
Specimen height, in	4.10	4.10	3.90	3.91

Description: Gray Limestone

LL = N/A | PL = N/A | | | Type: Rock Core

Project No.: AE97-033
Date: 2-19-98

Remarks:
1 - B-5 @ 11.4'-11.9'
2 - B-5 @ 22.6'-23.2'
3 - B-5 @ 25.0'-25.7'
4 - B-5 @ 31.4'-32.5'

Fig No. 25

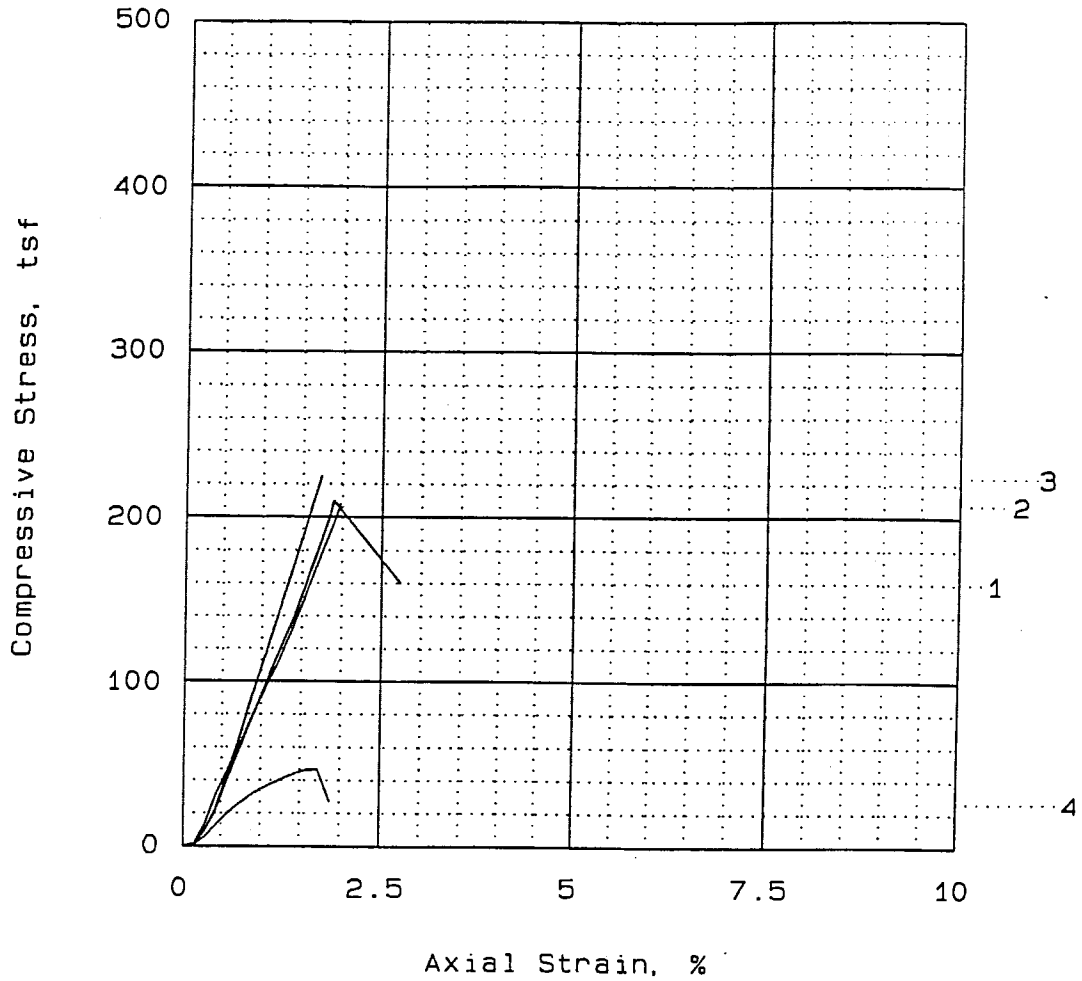
Client: Espey, Huston & Associates, Inc.

Project: Lower Williamson Creek
Relief Interceptor
Location: See Remarks

UNCONFINED COMPRESSION TEST

TERRA-MAR, INC.

UNCONFINED COMPRESSION TEST



Sample number:	1	2	3	4
Unconfined strength, tsf	210	208	225	47
Undrained shear strength, tsf				
Rate of strain, %/min				
Water content, %	5.4	5.7	5.2	10.5
Void ratio				
Saturation, %				
Dry density, pcf	147.7	145.0	147.6	131.0
Specimen diameter, in	2.04	2.03	2.01	1.95
Specimen height, in	4.16	3.97	4.06	3.95

Description: Gray Limestone

LL = N/A | PL = N/A | | | Type: Rock Core

Project No.: AE97-033
Date: 2-19-98

Remarks:
1 - B-6 @ 8.2'-9.0'
2 - B-6 @ 20.7'-21.6'
3 - B-6 @ 26.6'-27.0'
4 - B-6 @ 31.5'-31.8'

Fig No. 26

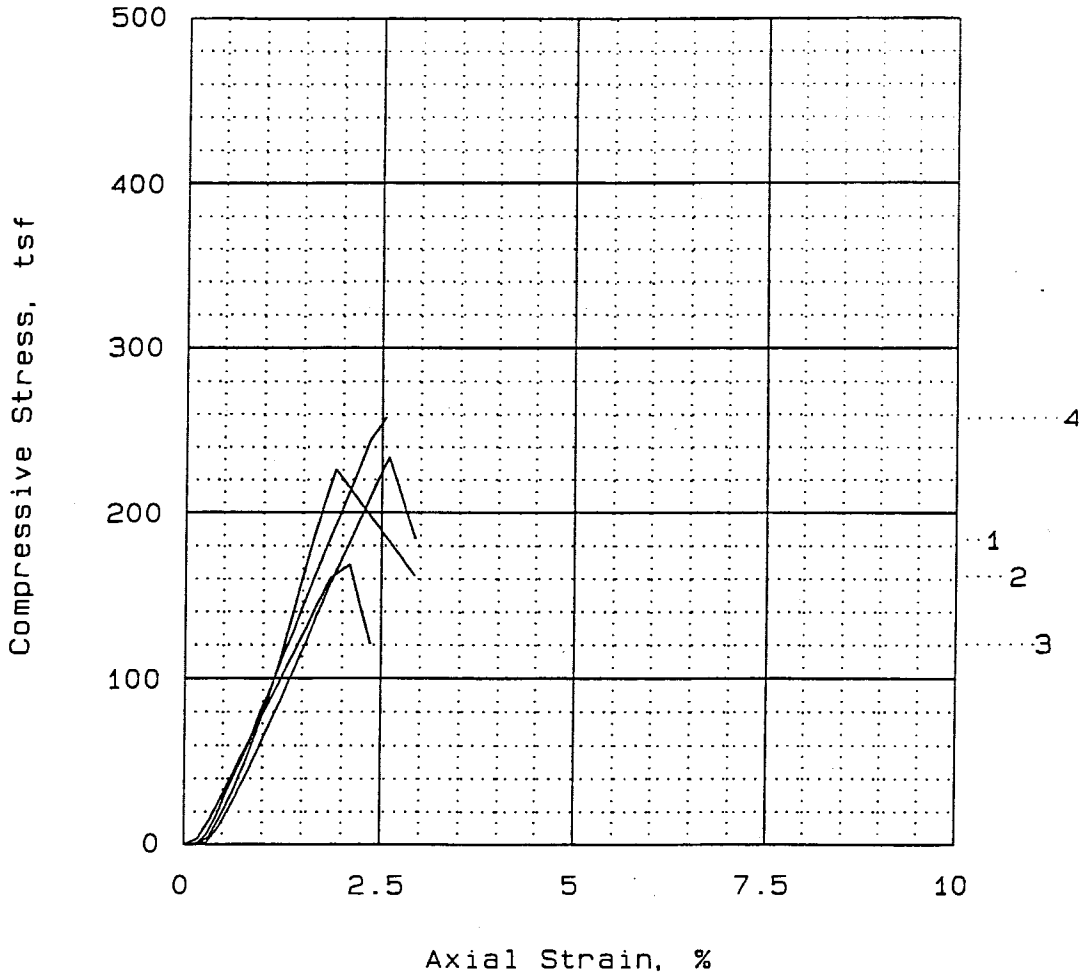
Client: Espey, Huston & Associates, Inc.

Project: Lower Williamson Creek
Releif Interceptor
Location: See Remarks

UNCONFINED COMPRESSION TEST

TERRA-MAR, INC.

UNCONFINED COMPRESSION TEST



Sample number:	1	2	3	4
Unconfined strength, tsf	234	226	169	258
Undrained shear strength, tsf				
Rate of strain, %/min				
Water content, %	5.9	6.7	7.0	5.3
Void ratio				
Saturation, %				
Dry density, pcf	144.7	141.6	141.4	146.7
Specimen diameter, in	1.96	1.94	2.00	2.02
Specimen height, in	3.82	3.84	3.78	4.03

Description: Gray Limestone

LL = N/A	PL = N/A			Type: Rock Core

Project No.: AE97-033

Date: 2-19-98

Remarks:

1 - B-7 @ 13.6'-14.2'

2 - B-7 @ 25.0'-25.8'

3 - B-7 @ 32.7'-33.8'

4 - B-7 @ 40.2'-40.8'

Fig No. 27

Client: Espey, Huston & Associates, Inc.

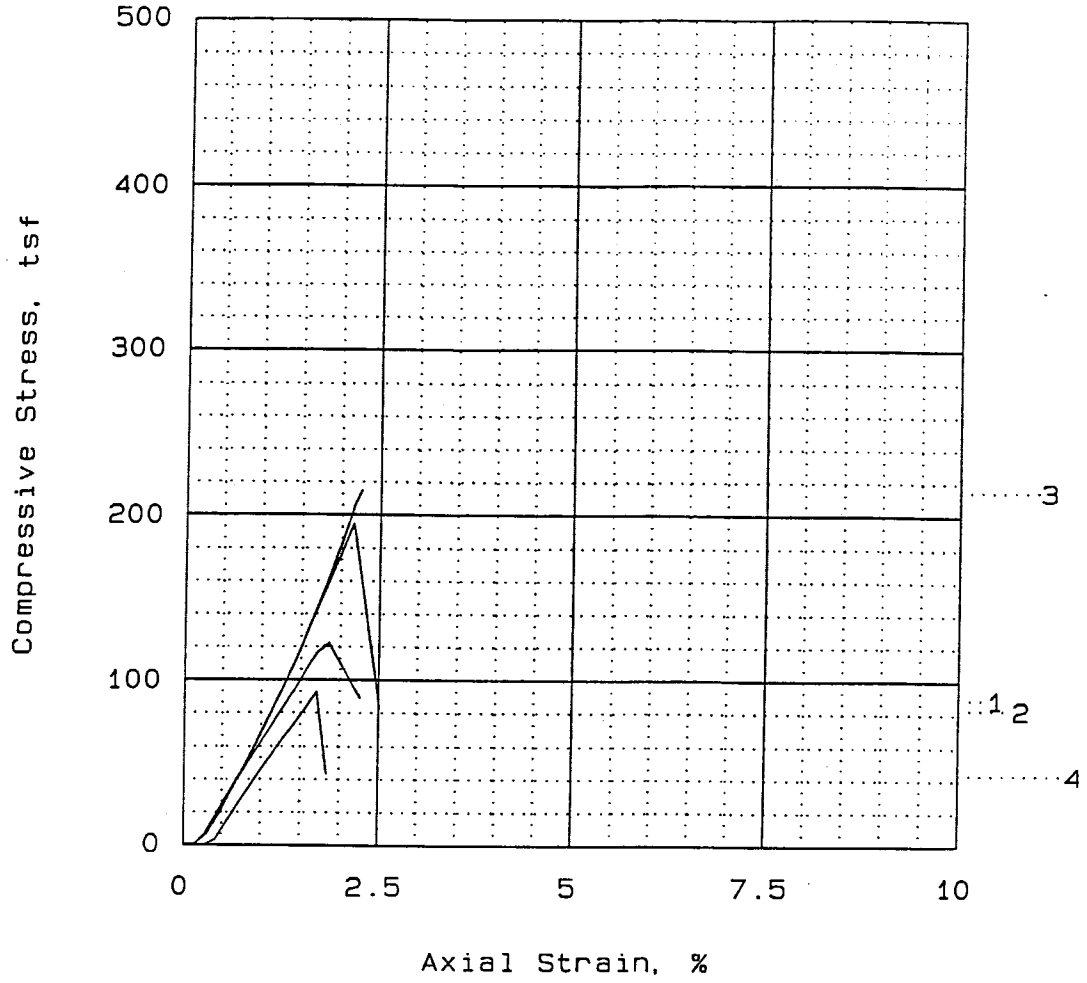
Project: Lower Williamson Creek
Releif Interceptor

Location: See Remarks

UNCONFINED COMPRESSION TEST

TERRA-MAR, INC.

UNCONFINED COMPRESSION TEST



Sample number:	1	2	3	4
Unconfined strength, tsf	122	195	215	92
Undrained shear strength, tsf				
Rate of strain, %/min				
Water content, %	7.2	7.8	5.3	7.7
Void ratio				
Saturation, %				
Dry density, pcf	141.1	140.1	146.3	138.7
Specimen diameter, in	2.04	2.02	2.04	2.00
Specimen height, in	4.06	3.95	3.64	4.14

Description: Gray Limestone

LL = N/A | PL = N/A | | | Type: Rock Core

Project No.: AE97-033
Date: 2-19-98

Remarks:
1 - B-8 @ 16.6'-17.1'
2 - B-8 @ 26.6'-27.2'
3 - B-8 @ 34.4'-35.0'
4 - B-8 @ 39.3'-40.0'

Fig No. 28

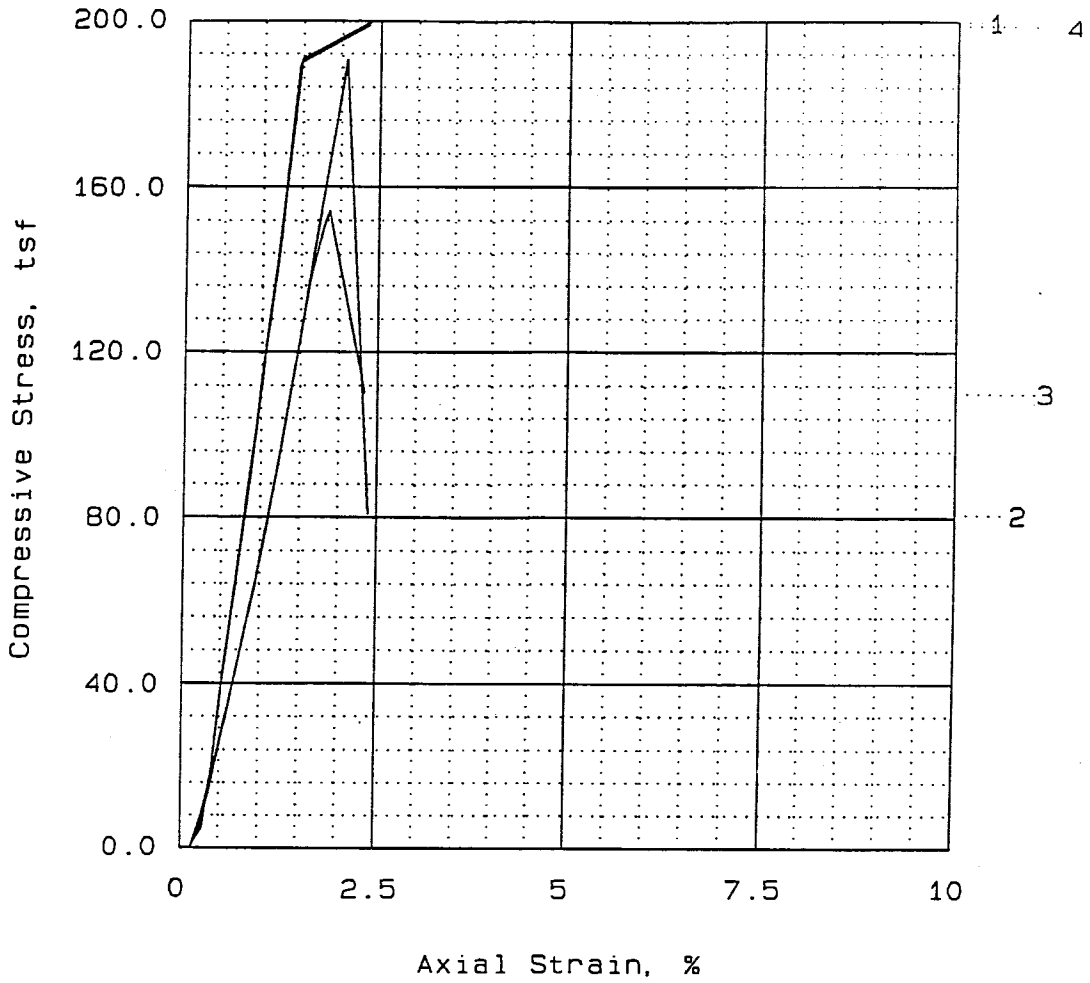
Client: Espey, Huston & Associates, Inc.

Project: Lower Williamson Creek
Relief Interceptor
Location: See Remarks

UNCONFINED COMPRESSION TEST

TERRA-MAR, INC.

UNCONFINED COMPRESSION TEST



Sample number:	1	2	3	4
Unconfined strength, tsf	199.8	190.6	154.3	198.8
Undrained shear strength, tsf				
Rate of strain, %/min				
Water content, %	5.6	7.3	6.7	8.8
Void ratio				
Saturation, %				
Dry density, pcf	146.0	141.0	142.4	137.1
Specimen diameter, in	2.05	2.05	2.06	2.05
Specimen height, in	4.10	4.13	4.11	4.16

Description: Gray Limestone

LL = N/A | PL = N/A | | | Type: Rock Core

Project No.: AE97-033
Date: 2-19-98

Remarks:
1 - B-9 @ 11.6'-12.2'
2 - B-9 @ 20.0'-20.5'
3 - B-9 @ 27.1'-27.9'
4 - B-9 @ 318.'-32.3'

Fig No. 29

Client: Espey, Huston & Associates, Inc.

Project: Lower Williamson Creek
Relief Interceptor
Location: See Remarks

UNCONFINED COMPRESSION TEST

TERRA-MAR, INC.

APPENDIX A

STANDARD FIELD AND LABORATORY INVESTIGATION PROCEDURES

FIELD INVESTIGATION PROCEDURES

The borings for this investigation were staked in the field by a Terra-Mar staff engineer using simple taping procedures from reference points noted on the site plan provided by the client. Ground surface elevations were estimated from topographical maps, when possible.

A field log of each boring was recorded during field drilling operations. These field logs and soil classifications were reviewed/verified by a geotechnical engineer in the laboratory through visual classification. Samples were visually classified according to color, texture, predominant material type, consistency and density. Results from the laboratory-testing program along with the visual classification of each soil stratum were used in the determination of the USCS classification for each soil stratum (ASTM D-2487). Finalized boring logs are provided in the report, as noted in the table of contents. Please note that the lines of soil demarcation represent the approximate soil boundary between differing material types, and that the actual transition between the soil strata in the field may be gradual.

Truck mounted drilling equipment mounted on an all-terrain vehicle was used to advance the borings to the completion depths using rotary wash or air drilling, or continuous hollow or solid stem auger procedures, unless otherwise noted. Soil samples were foil wrapped and sealed in plastic bags, for moisture control, and were marked to identify the boring number, sample depth, and job number. Unless notified to the contrary, soil/rock samples/cores will be stored for 90 days, and then discarded. Field soil/rock sampling procedures were performed in accordance with the following standards:

Standard Penetration Test (SPT) and Split Barrel Sampling of Soils (ASTM D-1586)

This sampling method consists of driving a 2-inch outside diameter split barrel sampler into the soil strata using a 140 pound hammer freely falling through a distance of 30 inches. The sampler is first seated 6 inches into the soil to be sampled, and is then driven an additional 12 inches in 6 inch increments. The number of blows required to drive the sampler the final 12 inches is then known as the Standard Penetration Resistance. The results of the SPT tests are given in blow counts per 6 inches on the boring logs. This sampling method is typically used for sampling non-cohesive or low cohesion soils, and gives an indication of the relative density of the in-situ soils, which can then be used to estimate allowable bearing capacities and potential elastic settlement within the soil mass.

Thin Walled "Shelby" Tube Sampling (ASTM D-1587)

This sampling method consists of pushing thin walled steel tubes, with an approximate 3 inch outside diameter, into the soil to be sampled using pressure provided hydraulically by the drilling rig, or manually by the field technicians. This sampling method is typically used on cohesive soil samples, and produces samples that are relatively undisturbed and suitable for shear strength testing, consolidation testing, permeability testing, and in-situ density approximation.

Sampling by Auger Boring Method "Grab Sample" (ASTM D-1452)

This sampling method consists of sampling the soil samples by removing representative soil samples from the continuous flight augers used to drill the boring, or by removing soil samples from the cuttings brought to the surface by the augers. This method provides highly disturbed samples that are appropriate for classification purposes only.

Diamond or Carbide Core Drilling for Site Investigation (ASTM D-2113)

This sampling method consists of sampling very hard strata (typically rock formations) by advancing a rotating double tube core barrel equipped with a diamond or carbide cutting bit. During this procedure, wash water or highly compressed air is used to remove excess material cuttings, and to cool the cutting bit. Typically, a 3 inch outside diameter by 2-1/8 inch inside diameter coring bit is used, unless otherwise noted. The recovered material is then examined in the field and stored according to the procedures mentioned at the beginning of this section. The soundness of the recovered cores were evaluated in two ways. These included calculating the Recovery Ratio (REC) and the Rock Quality Designation (RQD). These values are calculated according to the following formulas:

REC = (Total Length of Core Recovered / Length of Core Run) x 100

RQD = (Total Length of Cores Rec. Greater than 4 inches / Length of Core Run) x 100

Higher REC and RQD values are an indication of the cored material quality and its suitability as a bearing material. Generally speaking, the closer the REC and RQD values are to 100%, the higher the quality of the cored material. These values are recording on the boring logs next to the sample run when appropriate.

Please note that RQD values can sometimes be rather low due to the presence of hard layers in the rock strata and/or mechanical breakage in the coring tube. In these instances, the REC and RQD values are not recorded on the boring logs.

Groundwater Observations

Groundwater observations were made during drilling operations, and when appropriate, after a period of at least 24 hours. It should be noted that the use of water during coring operations can make the determination of groundwater conditions difficult. When necessary, wash water was bailed out of the boringto a depth below the depth of seepage observed during drilling operations.

LABORATORY INVESTIGATION PROCEDURES

The laboratory testing program was directed primarily toward the evaluation of the physical and engineering characteristics of the underlying site soils. Identification tests were performed to classify the soil samples according to the Unified Soil Classification System (ASTM D-2487) by a Geotechnical Engineer, unless otherwise noted. The results of this laboratory testing program are shown at the appropriate sample depths on the boring logs, or on the appropriate figures. Unless otherwise noted, laboratory procedures were performed according to the following standard procedures:

Moisture Content (ASTM D-2216)

Natural moisture contents of the soil samples (based on the dry weight of the soil) were determined for selected samples at selected depths. These moisture contents are useful in delineating the depth of the zone of moisture change with an increase in subsurface depth, and can be useful in locating the groundwater table. The moisture content can also be useful in analyzing and evaluating the expansion potential and/or shear strength of soil samples. The results of the test(s) are reported at the appropriate depth(s) on the boring log(s).

Atterberg Limits "Liquid and Plastic Limits" (ASTM D-4318)

The Atterberg Limits are given as the particular moisture contents of fine grained soil materials when they meet the requirements of a predefined test. In particular, the Plastic Limit (PL) is given as the approximate moisture content at which the soil material begins to behave more like a plastic material than a semi-solid material. Similarly, the Liquid Limit (LL) is given as the moisture content at which the soil material begins to behave more like a liquid material than a plastic material. The difference of these two test values can then be calculated, and is known as the Plasticity Index (PI). Larger PI values indicate an increased ability of the soil material to remain in a plastic state with changes in the moisture content. The results of the test(s) are reported at the appropriate depth(s) on the boring log(s).

The Atterberg Limits can then be used in conjunction with other parameters to classify the soil according to the Unified Soil Classification System (USCS). These parameters are also useful in evaluating the expansion potential of the fine grained soil materials with fluctuations in the moisture content.

Unconfined Compressive Strength of Soil Samples (ASTM D-2166)

The unconfined compressive strength testing of the soil sample(s) was performed in order to evaluate the undrained shear strength of the soil material(s). For this test, a sample with a minimum length to diameter ratio of 2:1 was used whenever possible in order to reduce end effects during testing. If the ratio was less than 2, a correction was applied according to standards listed in the ASTM manual. The results of the unconfined compressive strength test is useful in evaluating the shear strength of soil materials in undrained conditions and can also be used to determine the amounts of cohesion present. The results of the test(s) are reported at the appropriate depth(s) on the boring log(s).

Unconfined Compressive Strength of Rock Cores (ASTM D-2938)

The unconfined compressive strength of the rock material(s) was determined in order to evaluate the soundness of the rock material, and give a general indication of the ultimate bearing capacity of the bearing formation. For this test, the sample(s), with a minimum length to diameter ratio of 2:1, was used whenever possible in order to reduce end effects during testing. If the ratio was less than 2, a correction was applied according to standards listed in the ASTM manual. The results of the test(s) are reported at the appropriate depth(s) on the boring log(s).

Pocket Penetrometer Shear Strength Evaluation

A small hand held penetrometer device was used in the laboratory to evaluate the shear strength of relatively undisturbed cohesive soil samples. In this test, the flat tip of the device is placed on a flat portion of the undisturbed sample. A constant pressure is then applied until the device penetrates the sample a predetermined amount. The approximate shear strength developed is then read on the side of the device in tons per square foot, and is reported at the appropriate depth on the boring log(s). This test is valuable in developing relative correlations of the consistency of soils across the site, and can also be used as an indication of the in-situ moisture content relative to other samples.

APPENDIX B

SUMMARIES OF SIMILAR TUNNELING PROJECTS

Austin Tunnels in Austin Chalk:

1. Onion Creek Wastewater Interceptor (1984-1986)

   Length - 2700'

   Diameter - 9.2'

   Method -Fully shielded Lovat Model M-110 TBM, pick and teeth cutting tools

   Progress - 37, 26 and 16m/day in Claystone, tuffaceous and Chalk respectively

   Compressive strength – 2,050 to 3,800 PSI, RQD 46 to 98, w/ avg. of 75%

   Problems - Tuffaceous material was encountered in the claystone and chalk, creating mixed face problems for the contractor. The second problem was shallow river crossings. Plywood (2 layers, 9.5 mm thick) was used for temporary support. It was adequate when cover was limestone and at least one meter thick. However, at the second crossing the rock conditions deteriorated and terrace gravel was encountered, resulting in crown failure. This resulted in speed less than had been projected by the contractor.

2. Govalle Wastewater Interceptor (1986)

   Length – 15,000'

   Diameter - 10.5'

   Method - Full face Robbins TBM, reportedly equipped with two twin center disc cutters and 19 additional disc cutters. Mucking planned was by shuttletrain system with special chain conveyor wagon drawn by a locomotive.

   Compressive strength – 1,000 to 3,000 PSI

   Progress - 60 to 65 m/day (profited from experience from Onion Creek)

3. Austin Cross-Town Wastewater Interceptor (1973-1974)

   Length – 11,000'

   Diameter - 10.5'

   Compressive strength – 1,050 to 2,650 PSI

   Method - Fully shielded Calweld TBM equipped with double edge disc cutters.

   Progress - Max. 680'/week (5 or 6 days), average 70'/day (10 hr/day, excluding startup and equipment assembly time)

   Problems - Primary support consisting of steel sets, rock bolts and mining pans were required in a fault area where shale was exposed above the springline. Some slippage of grippers were noted in the same area. Some problems with TBM.

4. Lake Travis Project (1988)

Length - over 15 km

Rock formation is believed to be harder limestone and marl and possibly dolomite. Progress is unknown, and because of its complexity, it is believed that variety of equipment was used.

Dallas and Ft. Worth projects

1. West Fork Relief Project in Ft. Worth:

Length - Almost 11,000 feet

Diameter - 132" cut, 96" finished

Total cost 11.1 Million dollars includes 6 junctions and one river crossing

Tunneling cost approximately $900/ft.

Rock - Shale with unconfined compressive strength generally between 400 and 1200 PSI

Method of excavation TBM

2. West Fork related section WF-3

Length - 360 '

Diameter - 108" (believed to be cut diam., 96" finished)

Cost - $1100/ft

Formation - Clays and sandy clays

Excavation - Backhoe

Progress - 6 to 9 feet per day

3. West Fork related section WF-2

Length 320'

Diameter - 108" (believed to be cut diameter, 96" finished)

Cost - $1182/ft.

Formation - Clays and sandy clays

Excavation - Backhoe

Progress - Unknown, probably the same as in 2

4. West Bank Relief Interceptor, Phase 1, Part 1, Dallas

   Length - 3970'

   Diam. - 140" cut, 120" finished

   Bid price - $5.957 million including 2 manholes ($220,000)

   Formation - 75% Austin Chalk limestone, 25% soft ground

   Equipment - Lovat TBM with rippers. Direct jacking.

5. White Rock Lake Relief Interceptor, Phase 2

   Length - 6220', including lake crossing

   Diameter - 91" cut, 72 " finished

   Bid price - $5.5 million including shafts ($480,000) and manholes ($30,000)

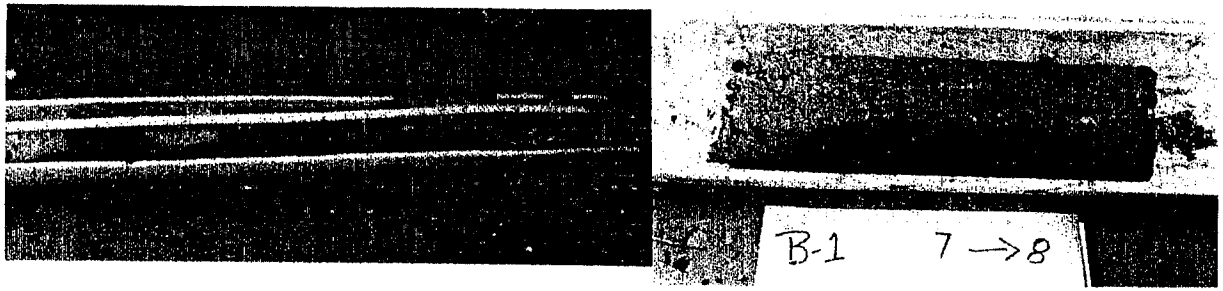
   Formation - 50% Austin Chalk limestone, 50% soft ground

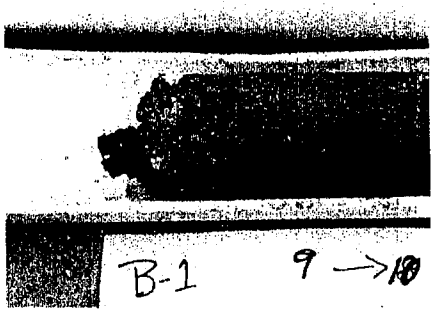
   Equipment - Lovat TBM with rippers. Direct jacking.

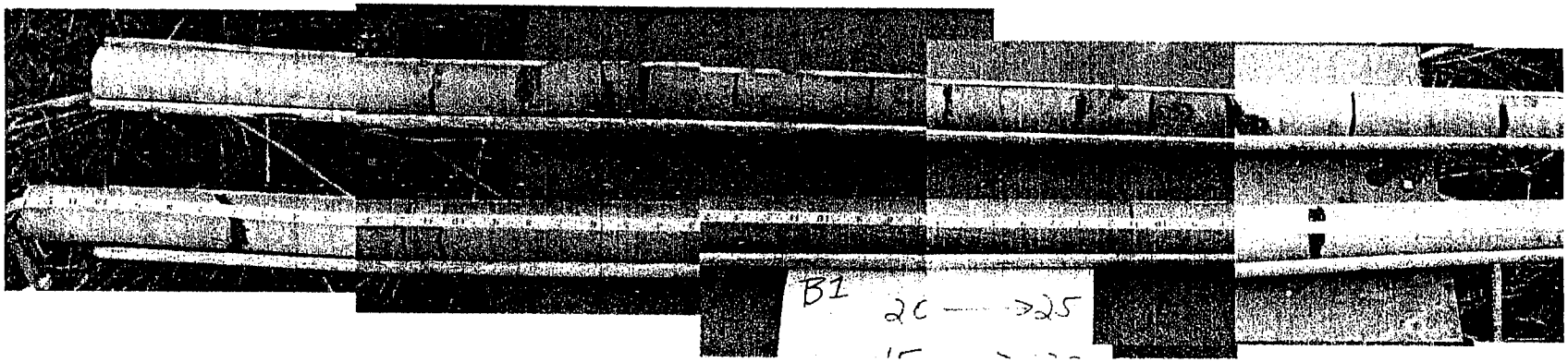
   Problems - Approximately 700 feet from end of lake crossing, encountered considerable amount of water.

APPENDIX C

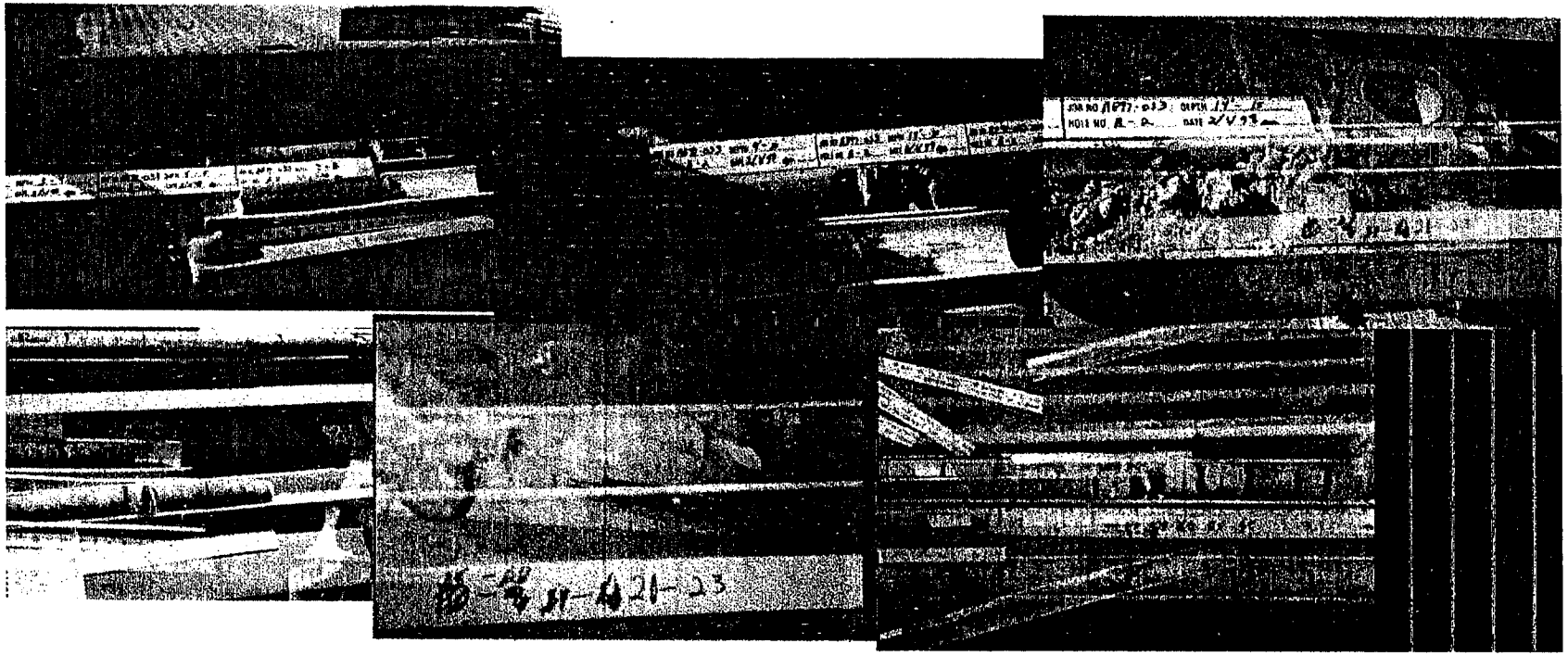
ROCK CORE SAMPLE PHOTOGRAPHS



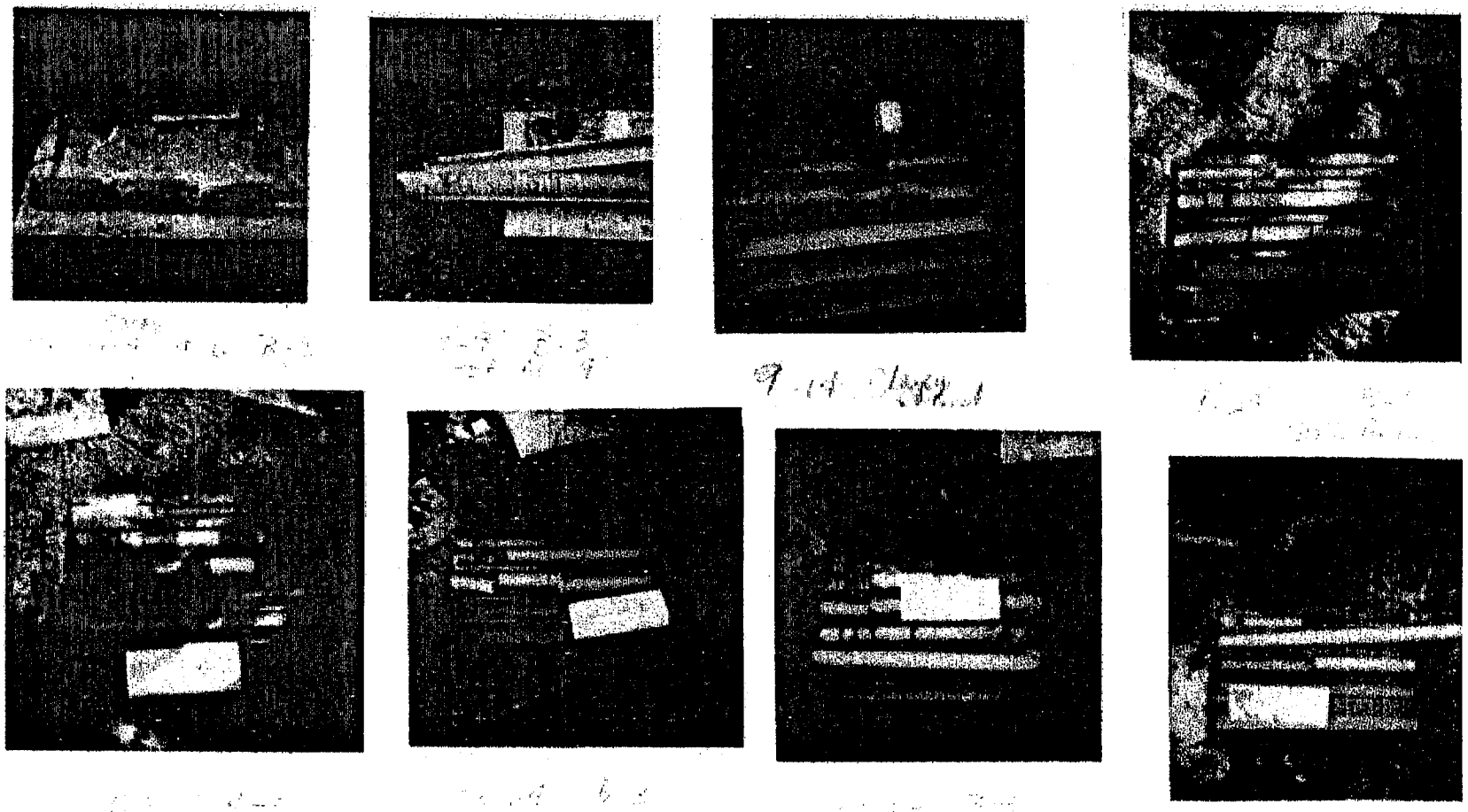




Borehole B-1
Surface -- 25 ft
Rock descriptions on
Lithologic logs

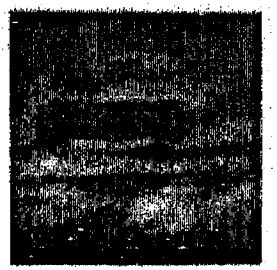


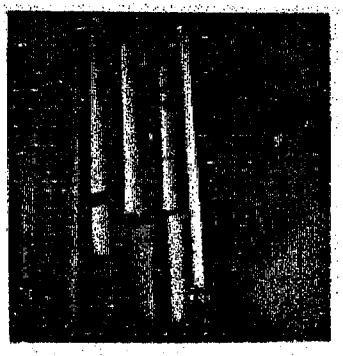
Borehole B-2
Photographs were
double exposed and therefore, content
was lost



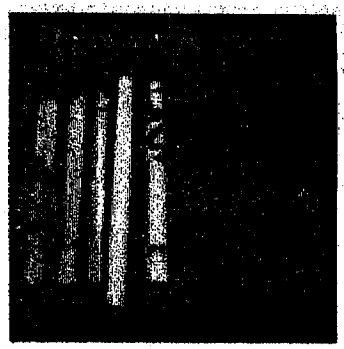
Borehole B-3

Surface - 44 ft





10-15 B-4

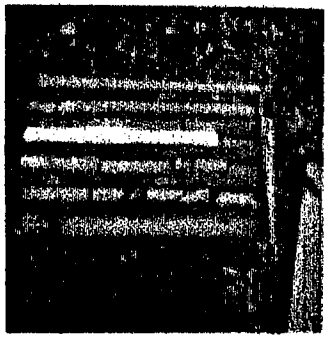


15-20' B-4

20-25' B-4

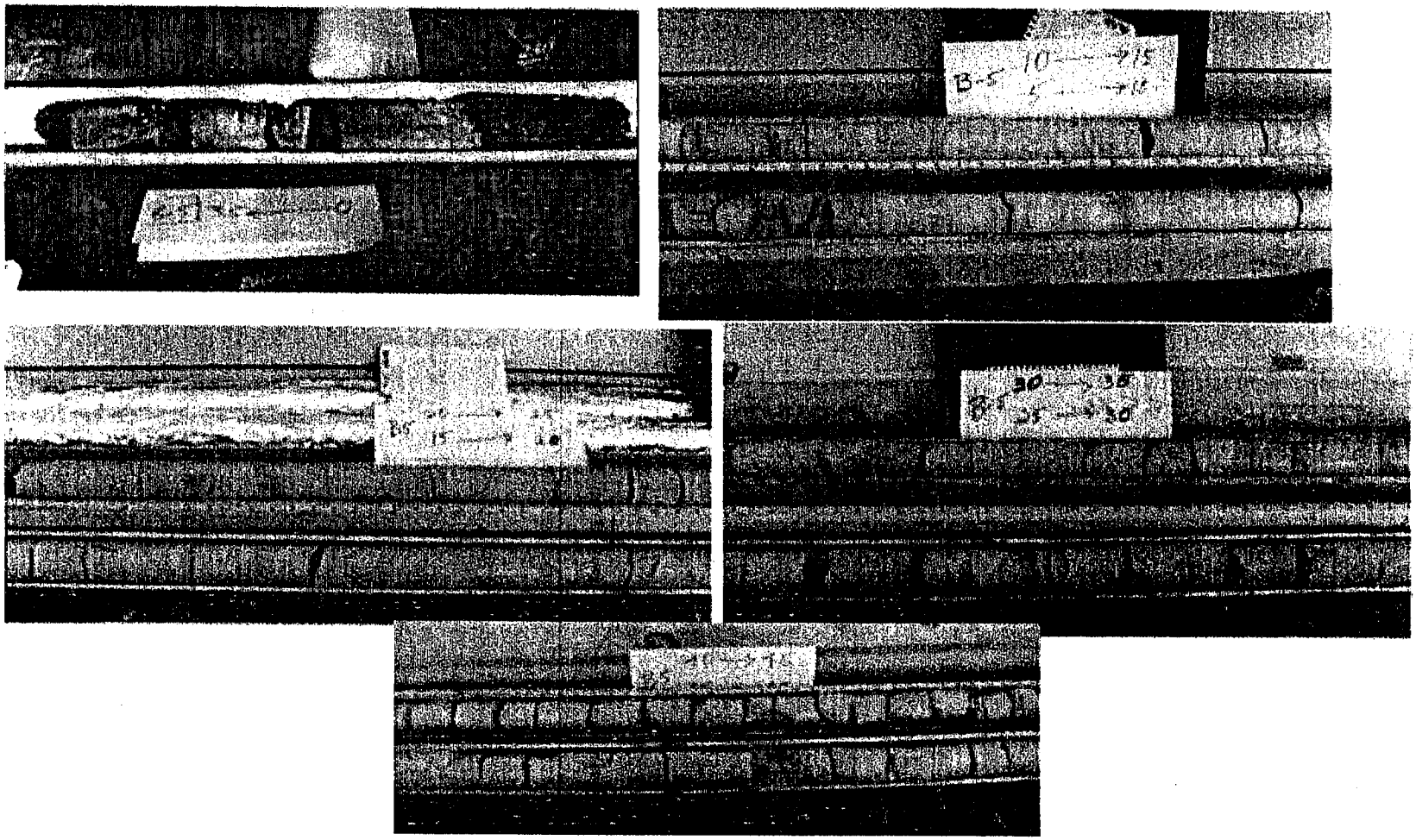
30-35 B-4

35-40' B-4



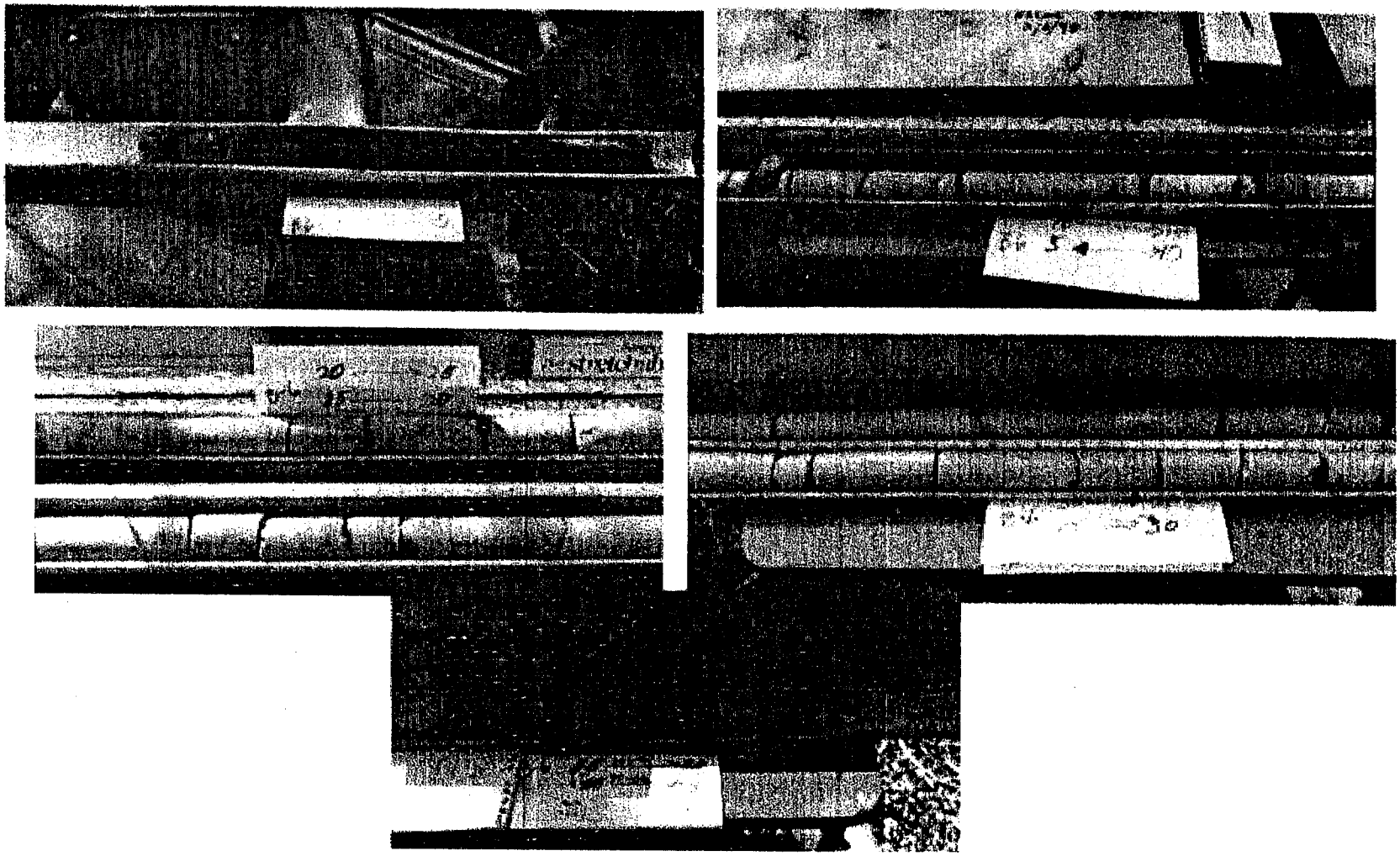
40-45 B-4

Borehole B-4
Surface - 45 ft



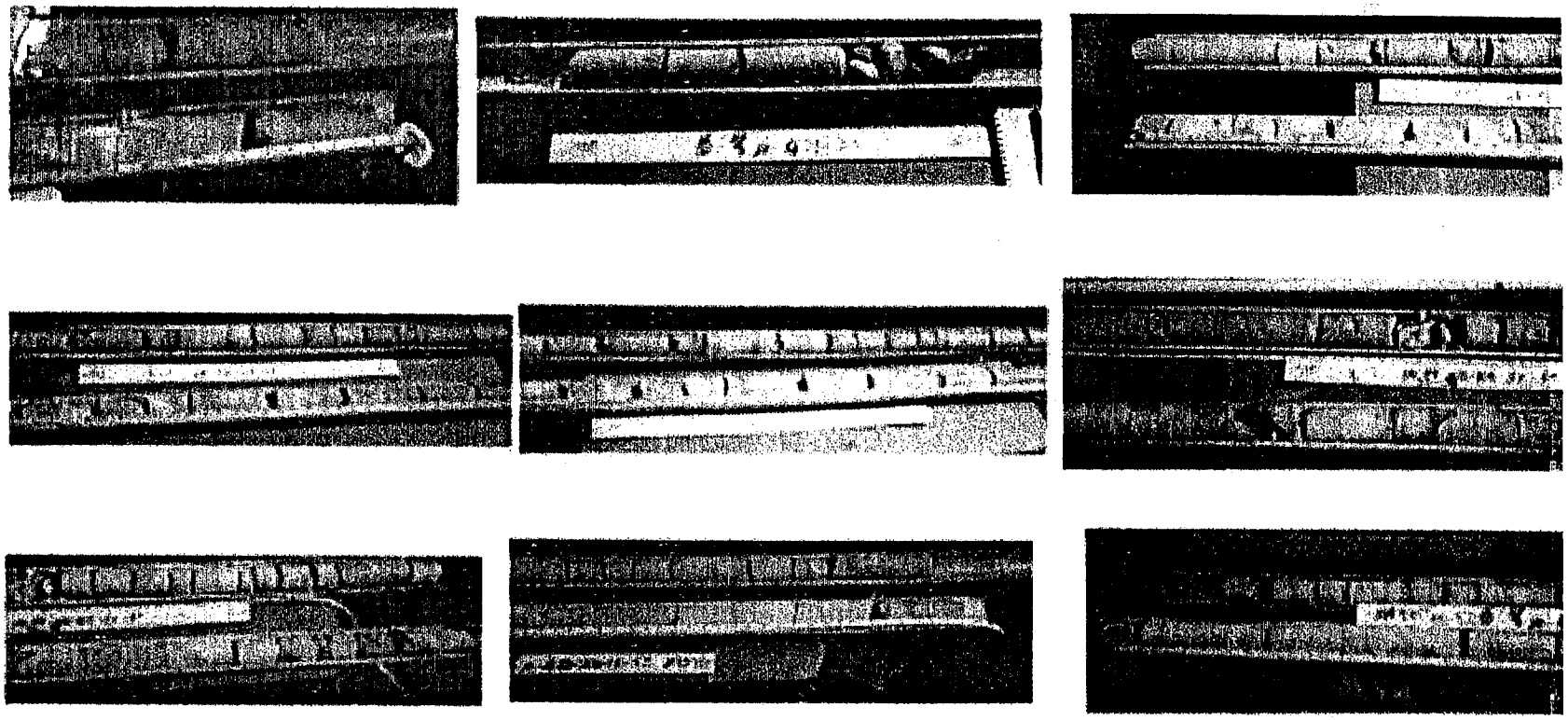
Borehole B-5

Surface - 45 ft



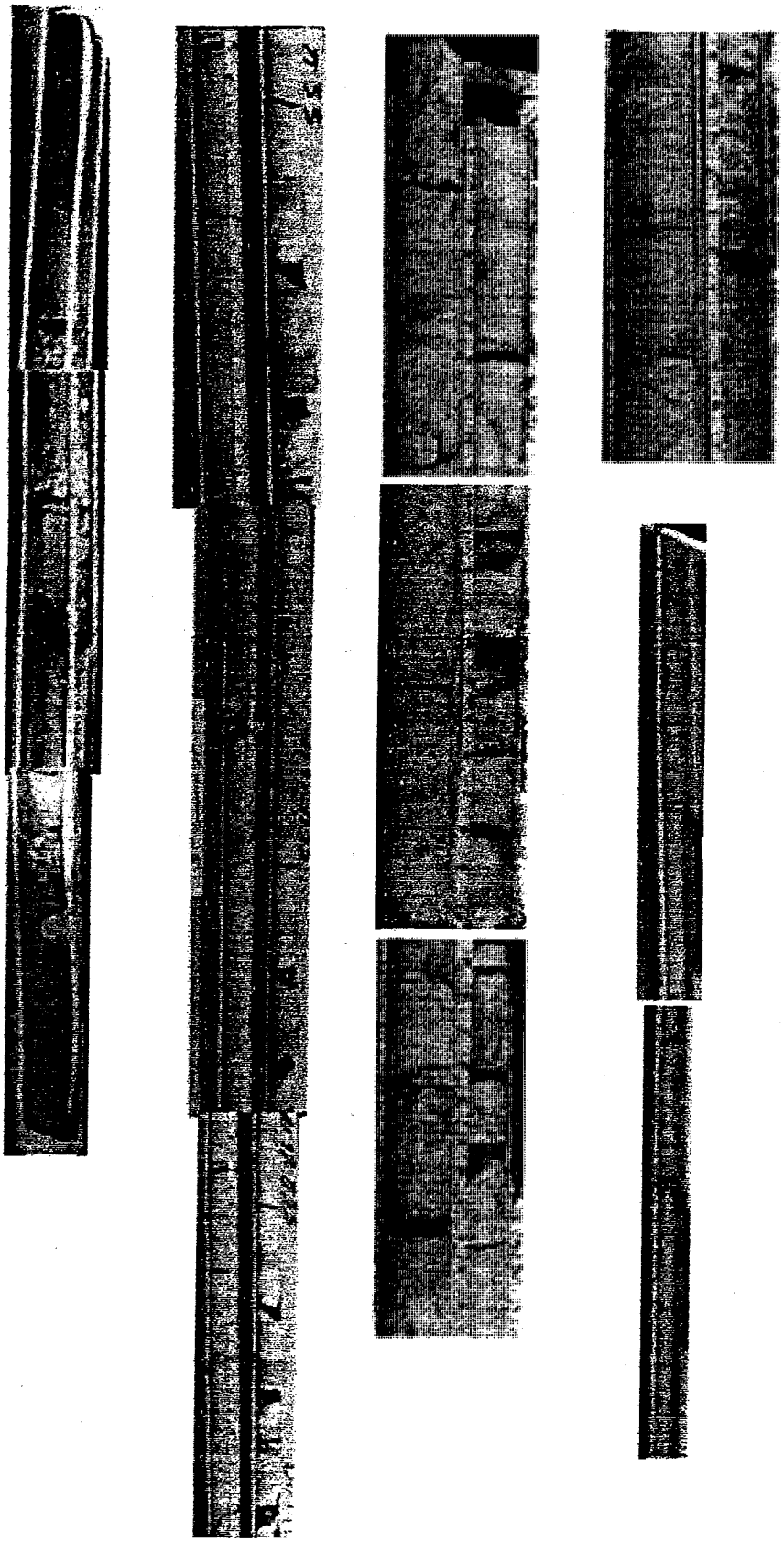
Borehole B-6

Surface - 41 ft

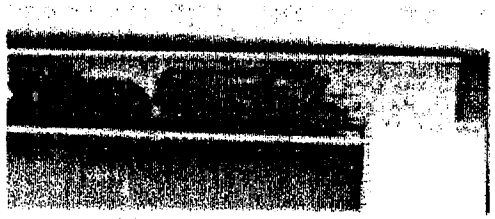


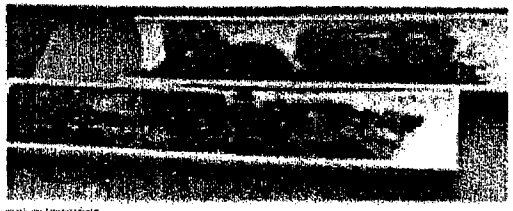
Borehole B-7

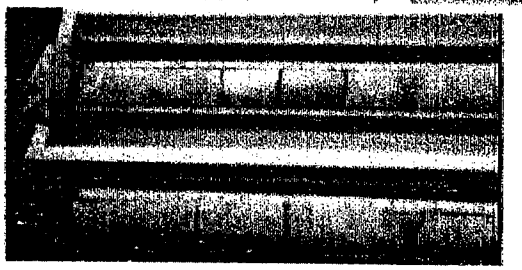
20 ft - 52 ft

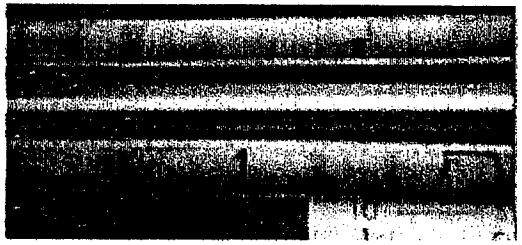


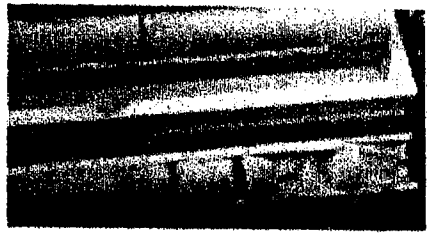
Borehole B-8
Surface - 40 ft.

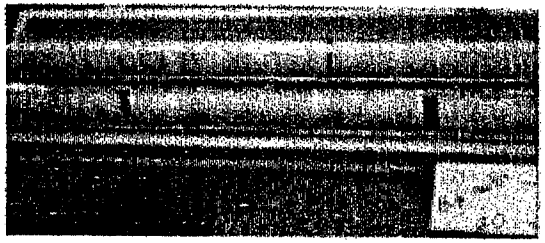


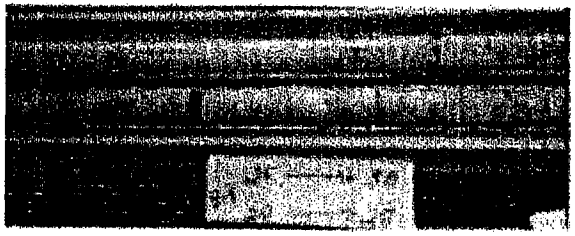


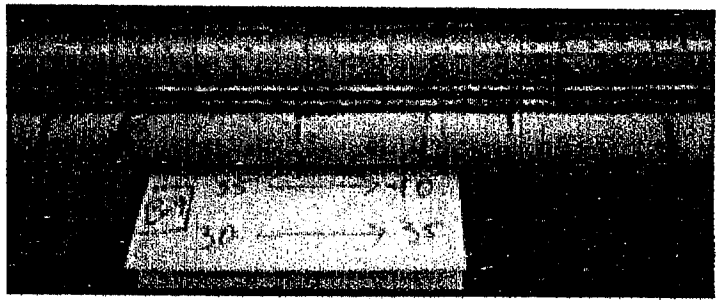


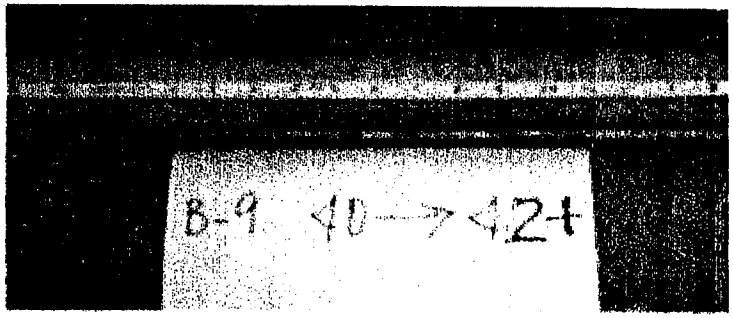












Borehole B-9 Surface - 42+ ft

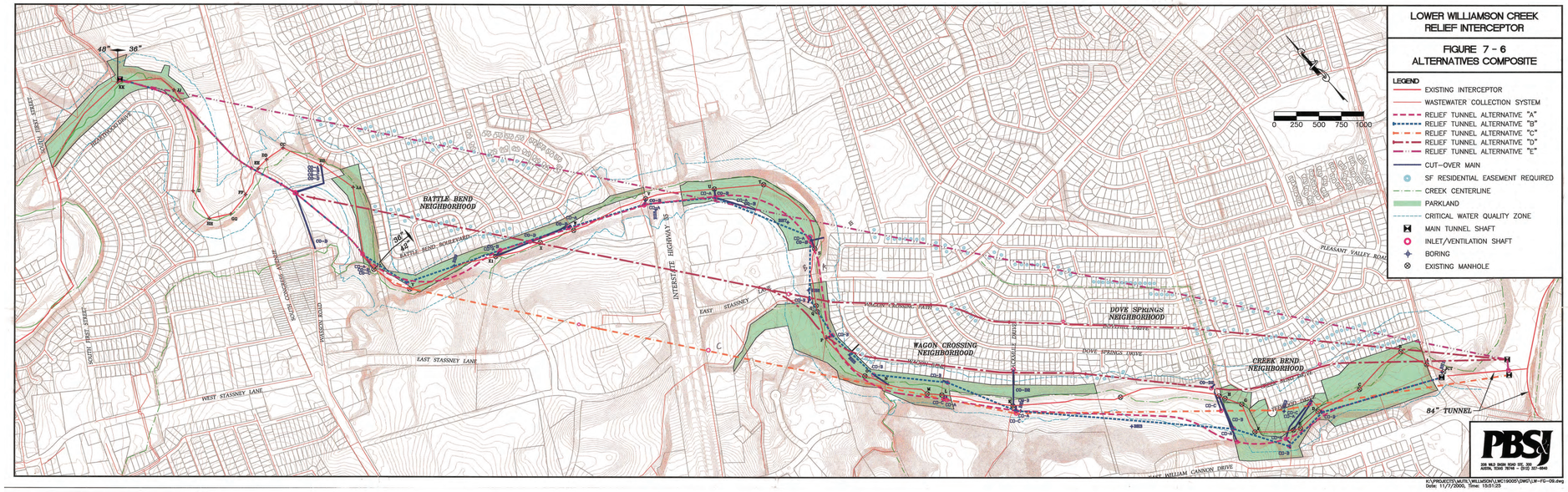
Although the OCWI Section IV contract was excavated entirely in the Taylor Group, the project encountered some problems which might be instructive for a Williamson Creek relief interceptor tunnel. Section IV was a joint venture of Mole Constructors and S&M Construction which used two 9-ft-diameter shielded TBMs with an expansion ring for thrust and torque reaction. The tunneled distance was 29,368 ft. The contractor selected double-layered plywood for primary support based on the high quality of most of the cores from the subsurface exploration and the contractor's experience with marly rock. The claystone was expected to be generally self-supporting, and the plywood was intended to retard rock deterioration from change in moisture content and to provide nominal support. However, in many locations the tunnel elevation was near top of rock, and critical problems resulted when softened claystone, groundwater, and alluvium (up to 6 inch cobbles) were encountered at the tunnel face. The contractor eventually constructed an extra shaft, installed a tail shield, and changed to conventional ribs and lagging for primary support. Better rock conditions toward the end of the tunnel drive allowed use of the plywood support system with occasional channel reinforcement.

Austin Crosstown Wastewater Interceptor. Tunneling for the ACWI was conducted in two contracts between January 1973 and December 1974. Contract 5029-1 tunneled a distance of 30,270 ft to produce a 10.5 ft diameter tunnel bore for a 8 ft diameter wastewater interceptor. Contract 5029-1 began in Taylor Group rock near the Walnut Creek treatment plant and extended westward through the Austin Group into the Balcones Fault zone. Contract 5029-3 began in the fault zone and extended westward 25,800 ft in Glen Rose limestone to Bull Creek.

In Contract 5029-1, Peter Kiewit Son's Company used a fully shielded TBM with double disk cutters and the capability to thrust off of primary rib and lagging support or to thrust with sidewall grippers extending through the shield. The contractor expected a relatively short zone of weathered rock at the start of Contract 5029-1 but encountered approximately 1,000 ft of softened claystone and alluvial material with water inflow. This material plugged the muck removal system and caused line and grade problems. A rib and lagging support and thrust system was used in this zone in place of the intended rockbolts and shotcrete for rock support and sidewall grippers for thrust. Once in the Austin Group, primary support was not used except at one fault location, where rockbolts and mining pans were installed. No significant water inflow was encountered at the fault, but slippage of the grippers was a problem at the transition back into full face chalk. The major construction difficulties on Contract 5029-1 were TBM repairs, which took 40 percent of the shift time in this tunnel section. Line and grade problems affected more than 25 percent of all shifts due to machine problems. According to the P. Nelson article, the line and grade problems were unrelated to any observed variability in ground conditions or curves in the tunnel alignment.

Davis WTP Transmission Main. The Davis WTP-Phase VII project excavated a 8.0 ft diameter tunnel for construction of a 66-inch water transmission main in the vicinity of Loop 360 in west Austin. Phase VII tunneled a distance of 6,332 ft with 55% of the alignment in curves with 850 ft to

1,050 ft radii. The tunnel was excavated in March-April 1986 by Seven K Construction in the Glen Rose and Walnut Formations (limestone, marl, dolomitic limestone, and clay shale with compressive strength of 700 psi to 5,400 psi). No rock support was installed, and the TBM used sidewall grippers for thrust reaction. No significant machine problems or line and grade problems were reported. TBM utilization was about 70 percent for this project. Downtime was associated with surveying for the alignment curves and the muck removal system (waiting for a muck train and track repair on account of the track working its way into the soft rock invert).


LOWER WILLIAMSON CREEK
RELIEF INTERCEPTOR
FIGURE 7 - 6
ALTERNATIVES COMPOSITE
LEGEND
EXISTING INTERCEPTOR
WASTEWATER COLLECTION SYSTEM
RELIEF TUNNEL ALTERNATIVE "A"
RELIEF TUNNEL ALTERNATIVE "B"
RELIEF TUNNEL ALTERNATIVE "C"
RELIEF TUNNEL ALTERNATIVE "D"
RELIEF TUNNEL ALTERNATIVE "E"
CUT-OVER MAIN
SF RESIDENTIAL EASEMENT REQUIRED
CREEK CENTERLINE
PARKLAND
CRITICAL WATER QUALITY ZONE
MAIN TUNNEL SHAFT
INLET/VENTILATION SHAFT
BORING
EXISTING MANHOLE
0 250 500 750 1000
BATTLE BEND NEIGHBORHOOD
WAGON CROSSING NEIGHBORHOOD
DOVE SPRINGS NEIGHBORHOOD
CREEK BEND NEIGHBORHOOD
INTERSTATE HIGHWAY 35
EAST STASSNEY LANE
WEST STASSNEY LANE
EAST STASSNEY
WAGON CROSSING TRAIL
DOVE SPRINGS DRIVE
PLEASANT VALLEY ROAD
EAST WILLIAM CANNON DRIVE
84" TUNNEL
48"
36"
PBSJ
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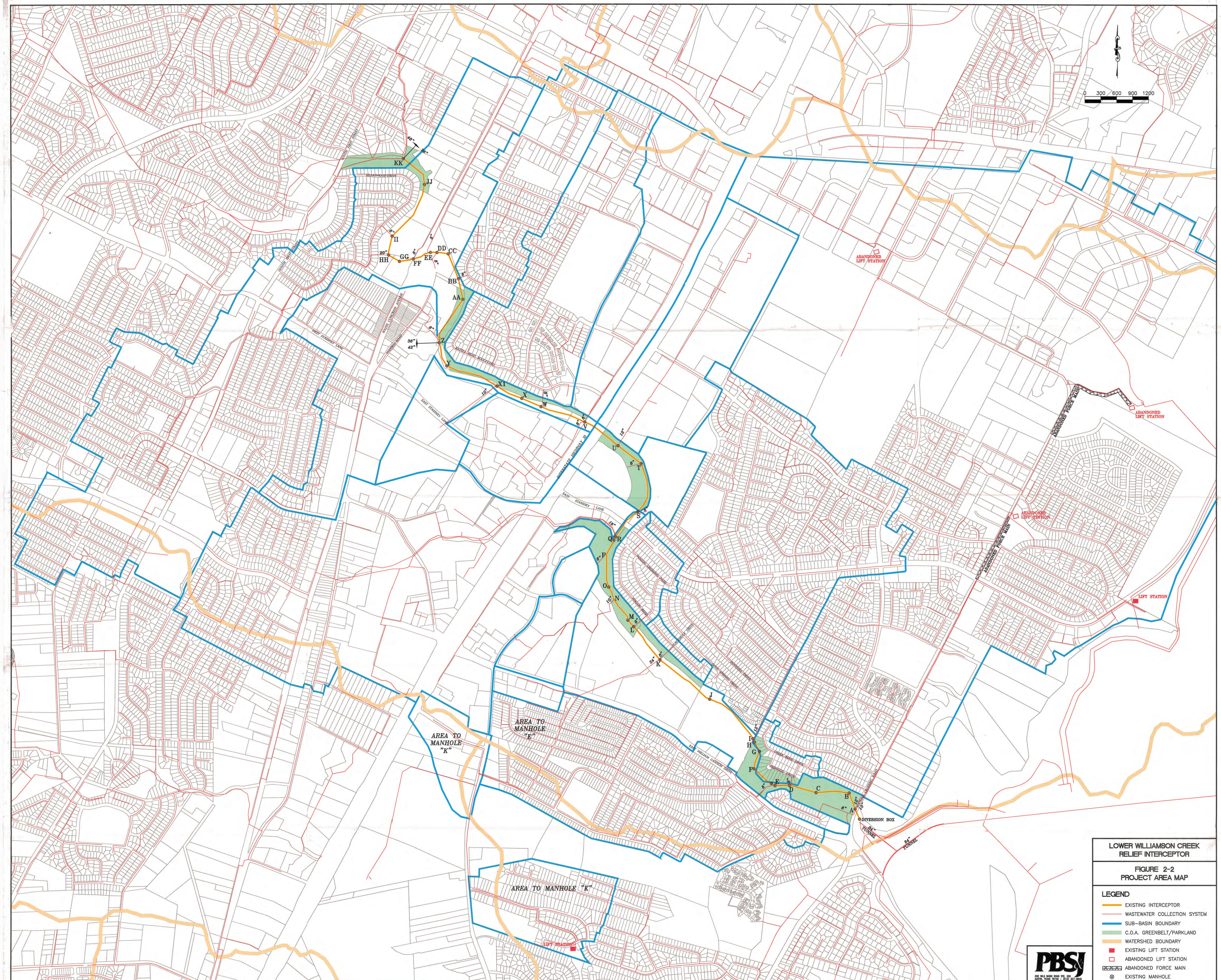
Date: 11/7/2006, Time: 10:51:25



LOWER WILLIAMSON CREEK
RELIEF INTERCEPTOR
FIGURE 2-2
PROJECT AREA MAP
LEGEND
EXISTING INTERCEPTOR
WASTEWATER COLLECTION SYSTEM
SUB-BASIN BOUNDARY
C.O.A. GREENBELT/PARKLAND
WATERSHED BOUNDARY
EXISTING LIFT STATION
ABANDONED LIFT STATION
ABANDONED FORCE MAIN
EXISTING MANHOLE
0 300 600 900 1200
PBSJ
206 WILD BASIN ROAD STE. 300
AUSTIN, TEXAS 78746 - (512) 327-6840
K:\PROJECTS\MUTIL\WILLMSON\LWC19005\DWG\Lw-fg-07.dwg
TR14037. Date: 12/6/2000. Time: 16:36:58
AREA TO MANHOLE "K"
AREA TO MANHOLE "E"
AREA TO MANHOLE "K"
LIFT STATION
LIFT STATION
ABANDONED LIFT STATION
ABANDONED LIFT STATION
ABANDONED LIFT STATION
ABANDONED FORCE MAIN
ABANDONED FORCE MAIN
DIVERSION BOX
84" TUNNEL
84" TUNNEL
SOUTH FIRST STREET
HEARTWOOD DRIVE
SOUTH CONGRESS AVENUE
WASSON ROAD
WEST STASSNEY LANE
EAST STASSNEY LANE
RATTLE BEND BOULEVARD
INTERSTATE HIGHWAY 35
WAGON CROSSING PATH
WAGON BEND
BLACKMULE DRIVE
DOVE SPRINGS DRIVE
DOVEHILL DRIVE
EAST WILLIAM CANNON DRIVE
CREEK BEND DRIVE
TEDWOOD DRIVE
NUCKOLS CROSSING ROAD
KK
JJ
II
HH
GG
FF
EE
DD
CC
BB
AA
Z
Y
X1
X
W
V
U
T
S
Q
R
P
O
N
M
L
K
J
I
H
G
F
E
D
C
B
A
48"
36"
42"
20"
24"
18"
12"
10"
8"
6"