Leon Creek Watershed Master Drainage Plan

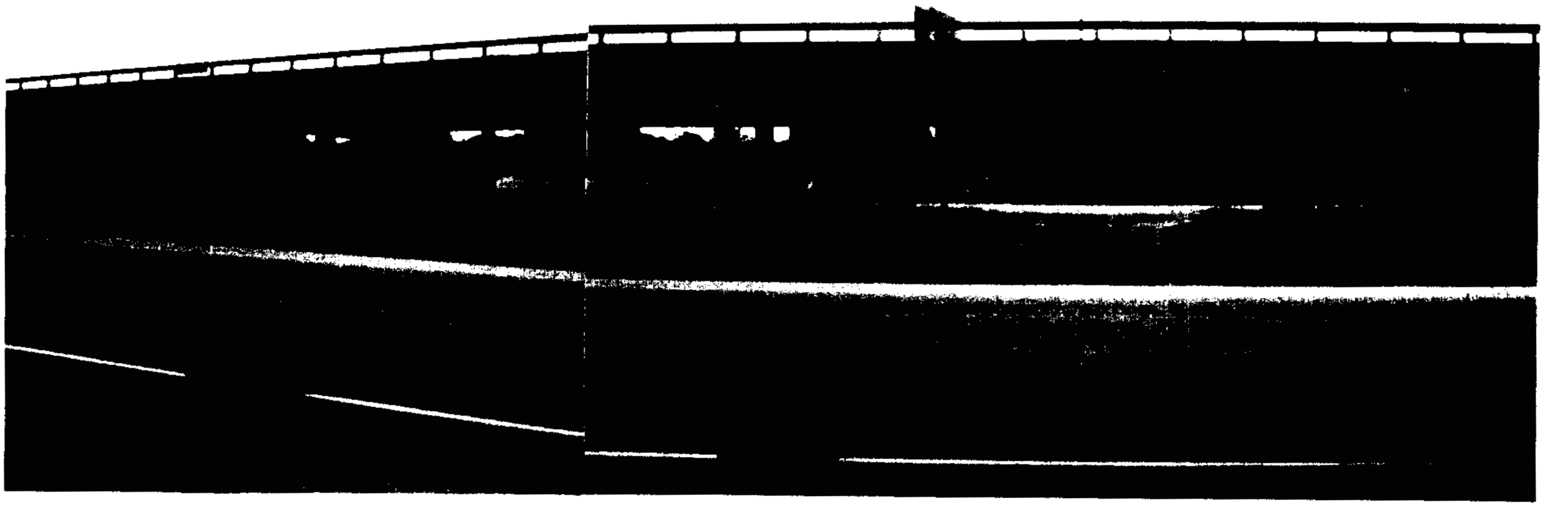
City of San Antonio
Public Works Department

November 1996

APPENDICES

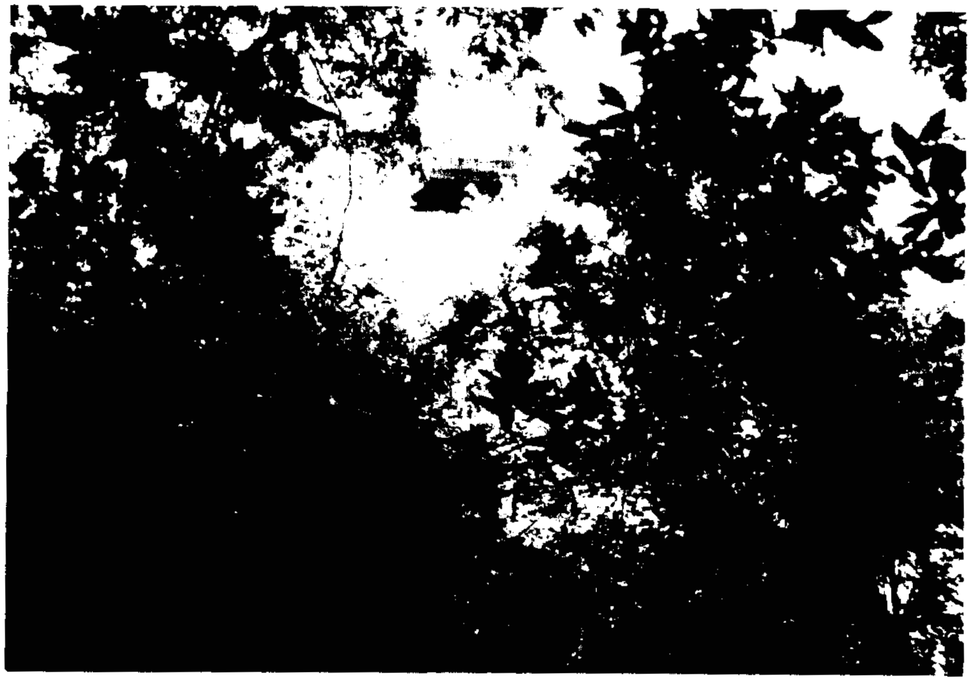
Pape-Dawson Consulting Engineers, Inc.
HNTB Corporation
Maestas & Bailey, Inc.
E.L. Fly & Associates, Inc.

APPENDIX A
Photographs

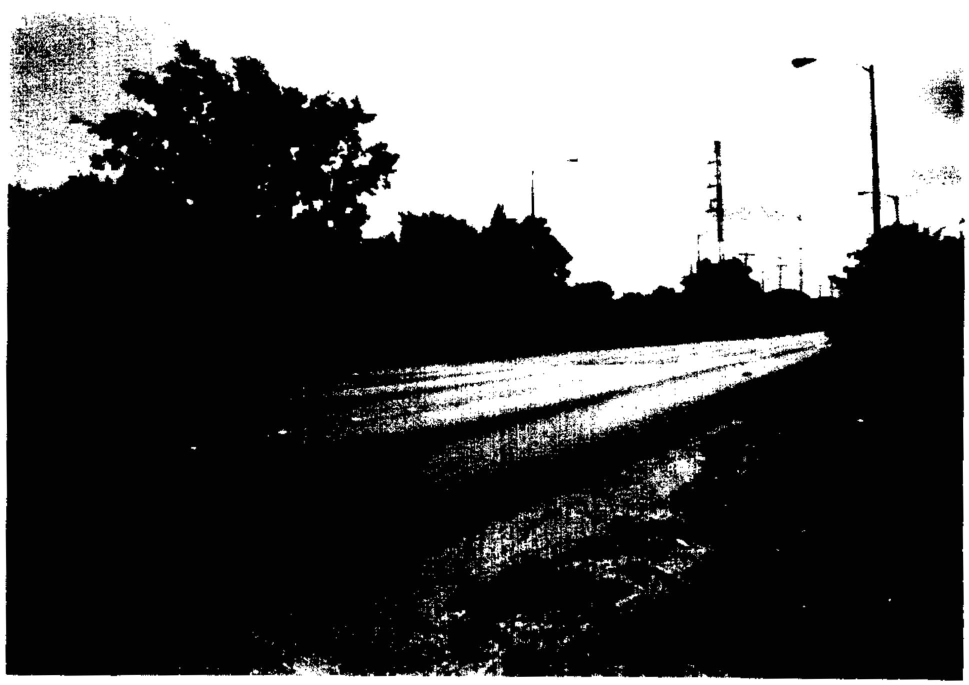


SITE 1
Highway 90 at Leon Creek
Upstream (East Bank to Channel Center)

SITE 1
Highway 90 at Leon Creek
Upstream (Channel Center to West Bank)



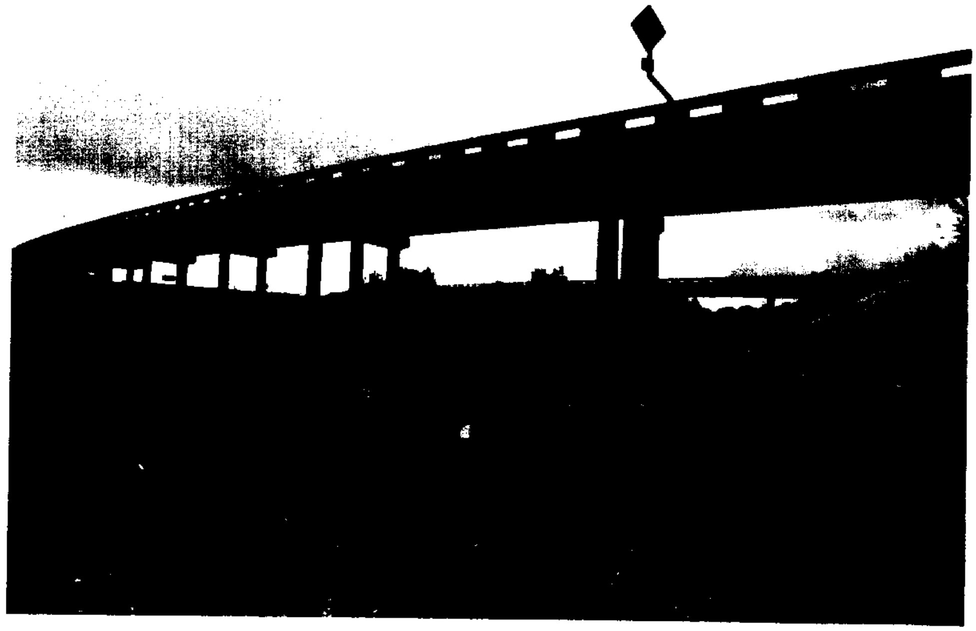
SITE 2
Old Highway 90 at Leon Creek
Upstream showing Heavy Growth



SITE 2
Old Highway 90 at Leon Creek
Along Road Looking Upstream

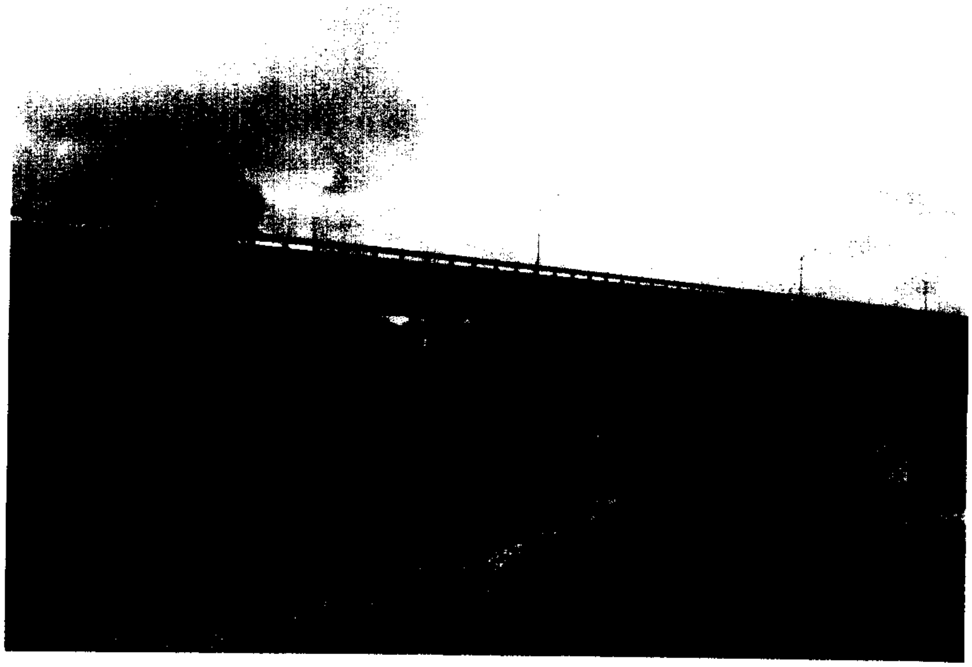
SITE 3
Arvil at Leon Creek
Entrance to Rodriguez Park

SITE 5
Pinn Road at Leon Creek
Downstream



SITE 6
Highway 151 at Leon Creek
Upstream

SITE 7
Commerce at Leon Creek
Downstream



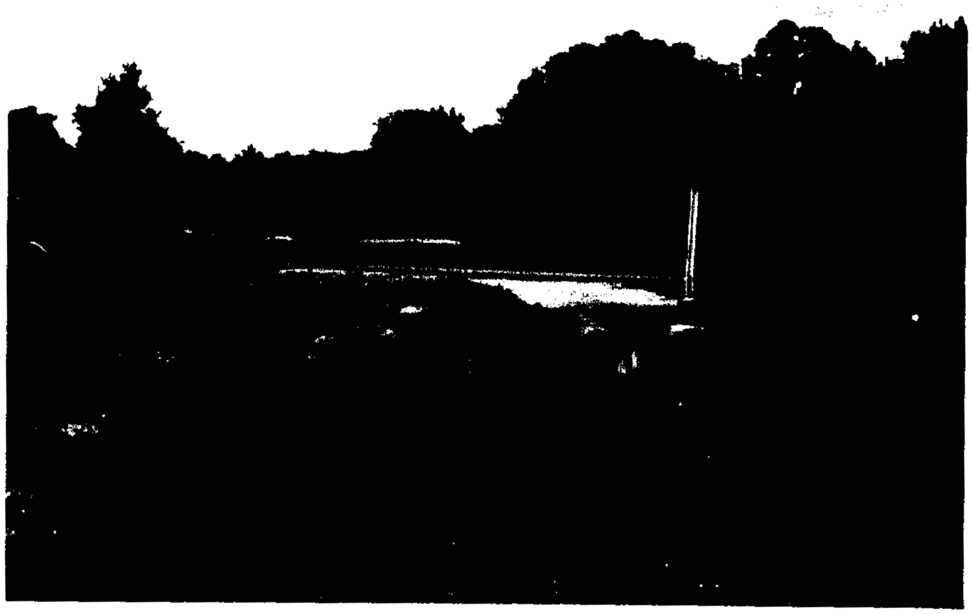
SITE 8
Loop 410 at Leon Creek
Upstream

SITE 9
Culebra Road at Leon Creek
Downstream

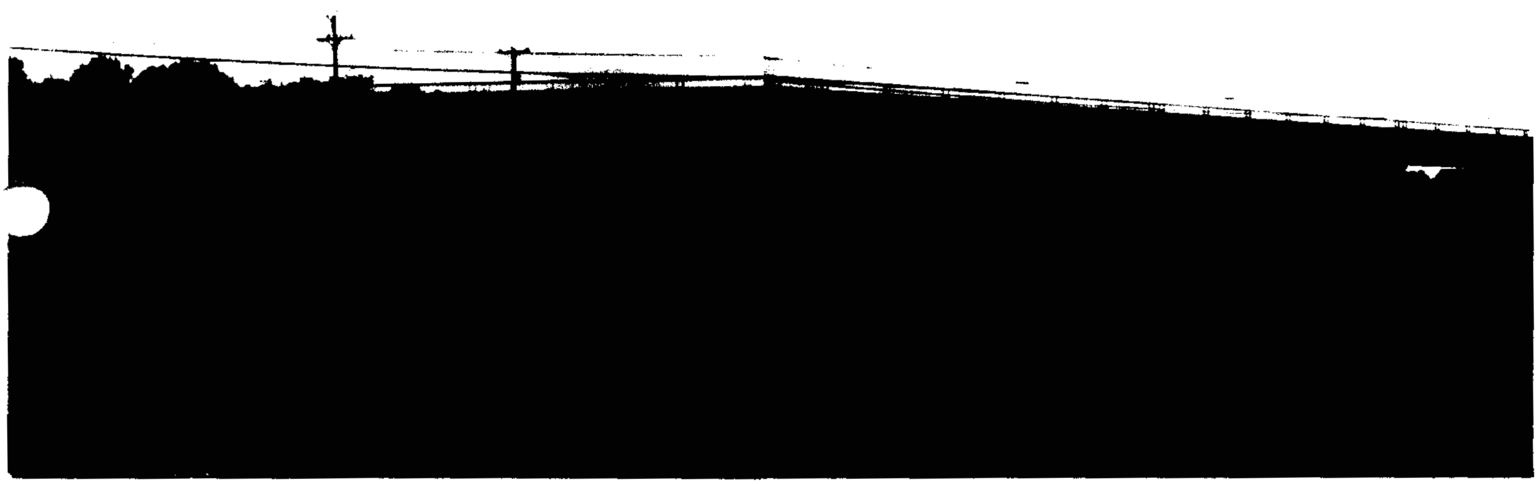
SITE 10
Ingram Road at Leon Creek
Downstream

SITE 11
Grissom Road at Leon Creek
Downstream

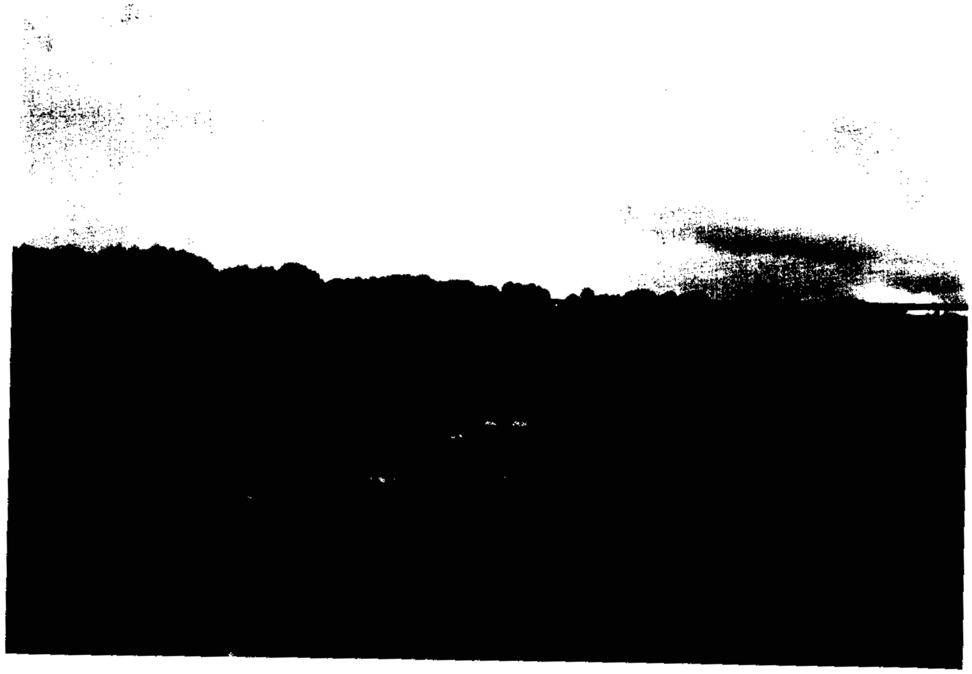
SITE 12
Bandera Road at Leon Creek
Downstream



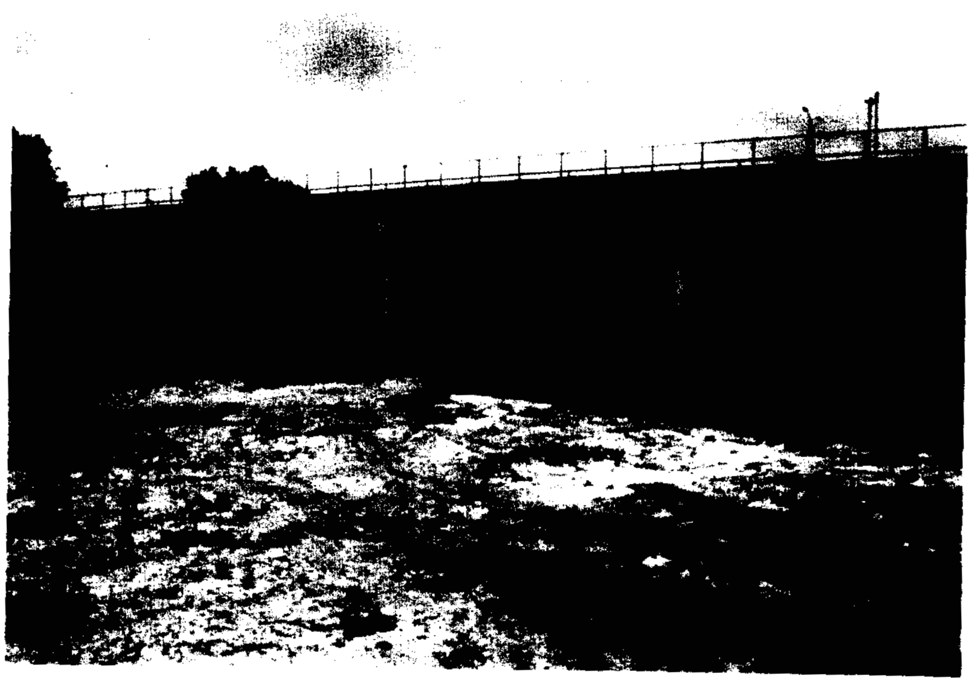
SITE 13
Babcock Road at Leon Creek
Upstream



SITE 14
Hausman Road at Leon Creek
Downstream

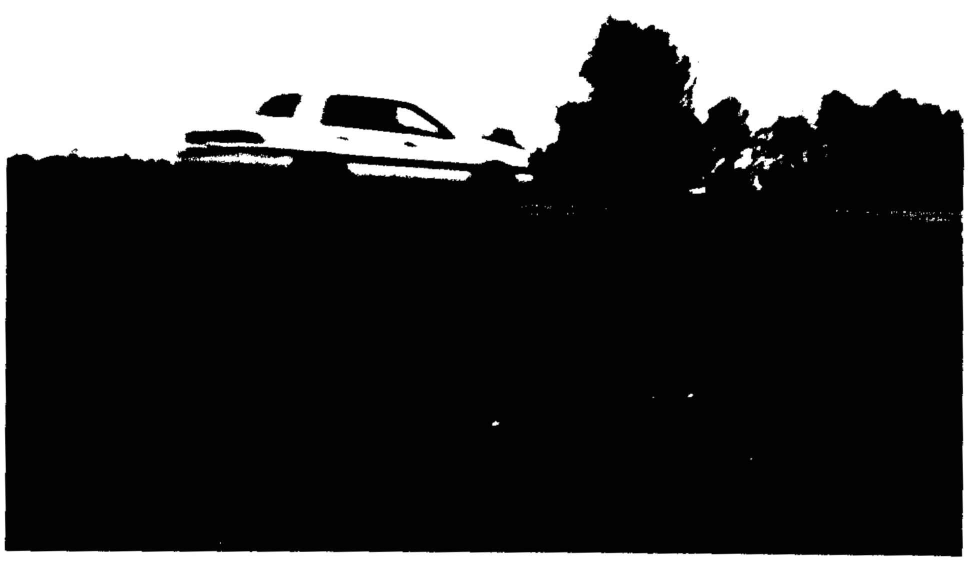


SITE 15
Vista Blvd. at Leon Creek
Downstream



SITE 16
Loop 1604 at Leon Creek
Downstream (4 Bridges)

SITE 17
Old Grissom Road at Leon Creek
Downstream



SITE 18
Timberpath at Culebra Creek
Downstream

SITE 19
Culebra Road at Culebra Creek
Downstream

SITE 20
Culebra Road at Culebra Creek
Upstream

SITE 21
Loop 1604 at Culebra Creek
Upstream

SITE 22
F.M. 1560 at Culebra Creek
Downstream

SITE 24
Mainland at French Creek
Upstream

SITE 25
Guilbeau Road at French Creek
Upstream

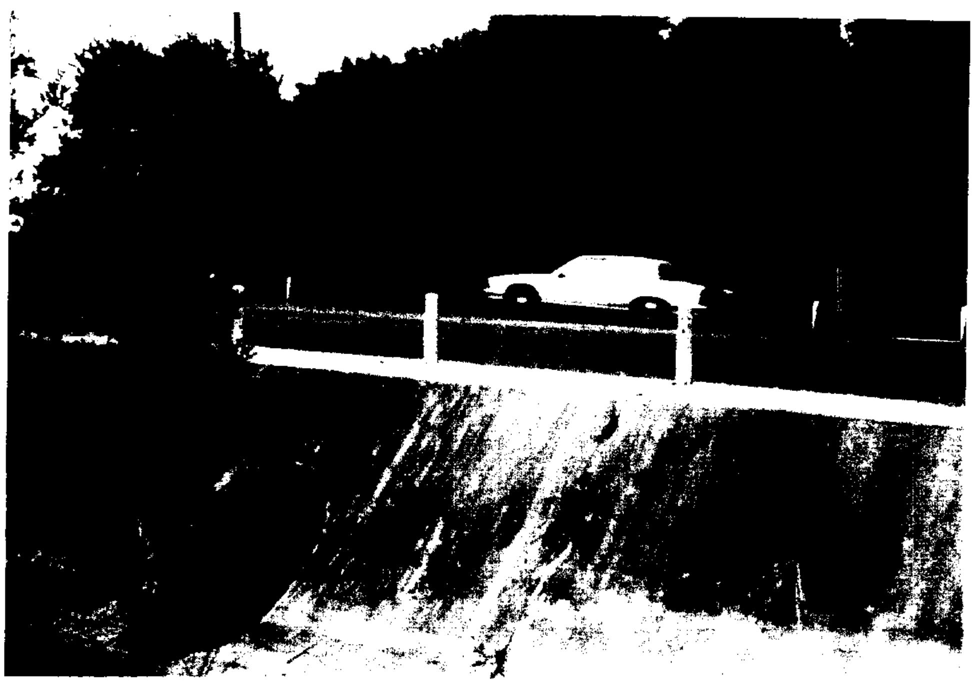
SITE 26
Bandera Road at French Creek
Downstream

SITE 27
Prue Road at French Creek
Upstream

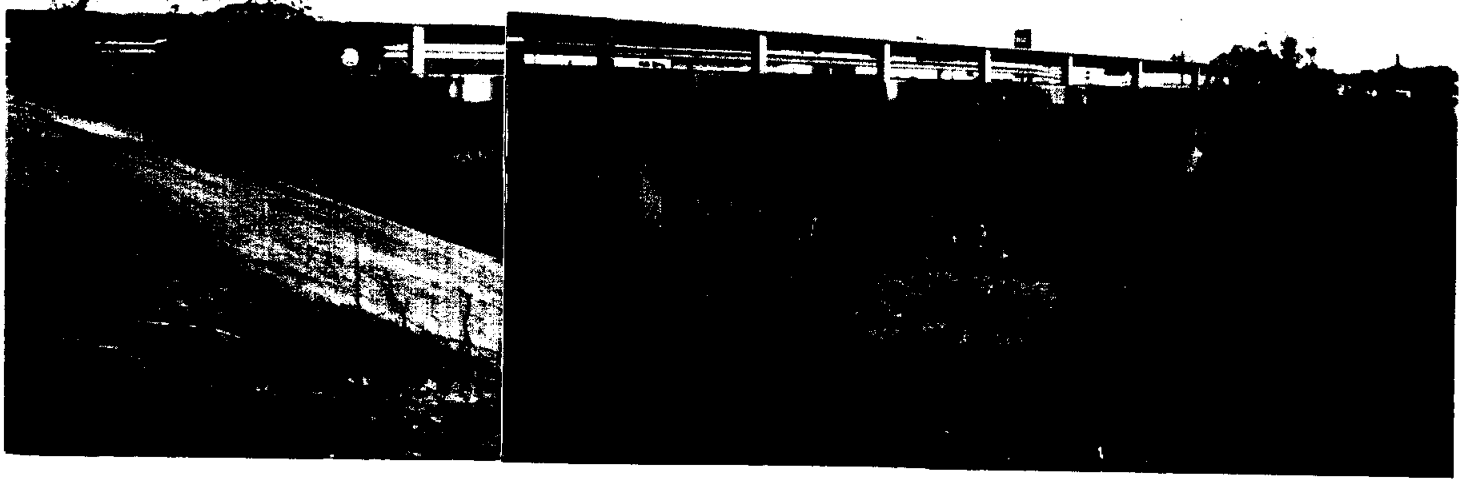
SITE 29
Loop 1604 at French Creek
Upstream

SITE 30
Leslie Road at French Creek
Downstream

SITE 31
Ingram Road at Huebner Creek
Upstream

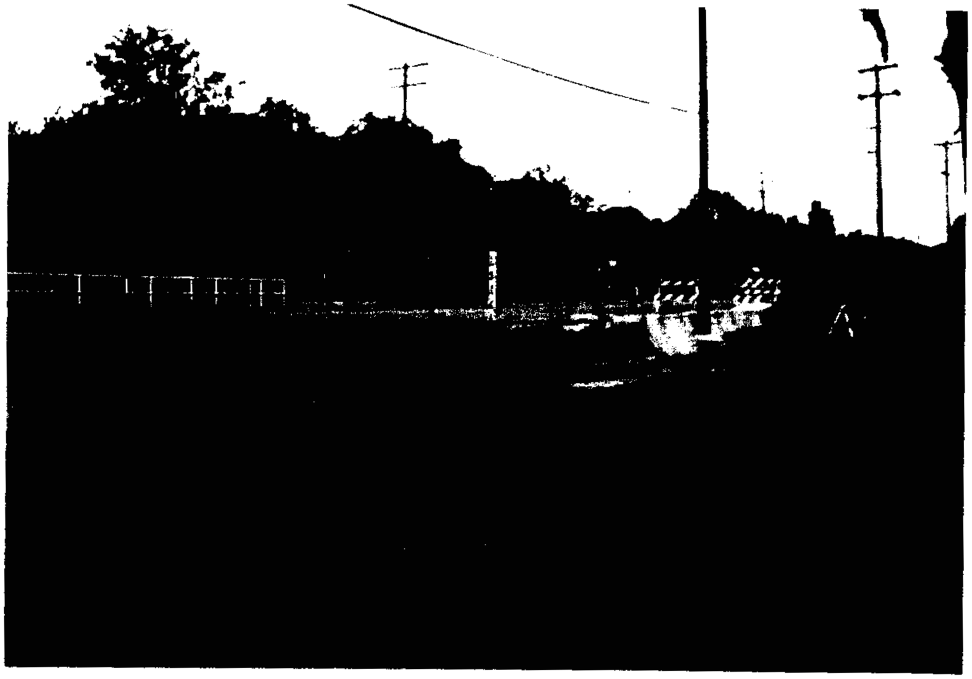


SITE 32
Timberhill at Huebner Creek
Downstream



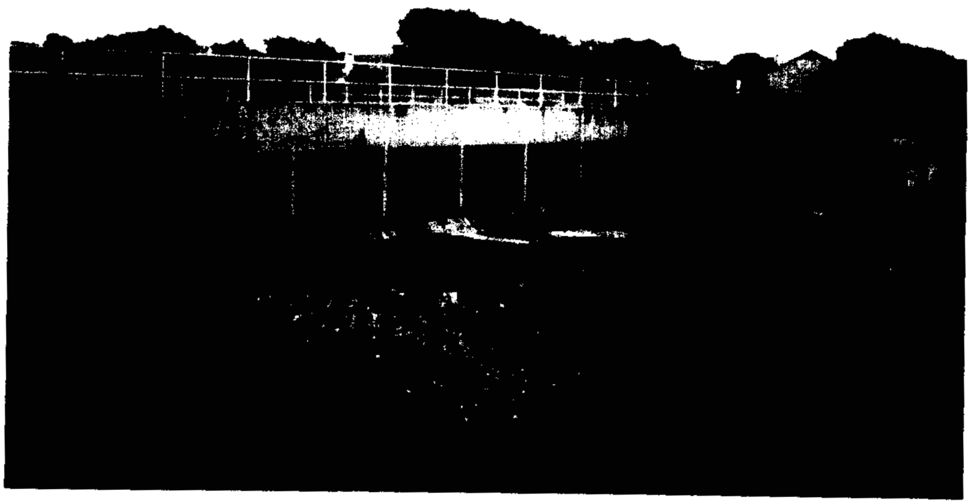
SITE 33
Bandera Road at Huebner Creek
Upstream

SITE 33
Bandera Road at Huebner Creek
Median



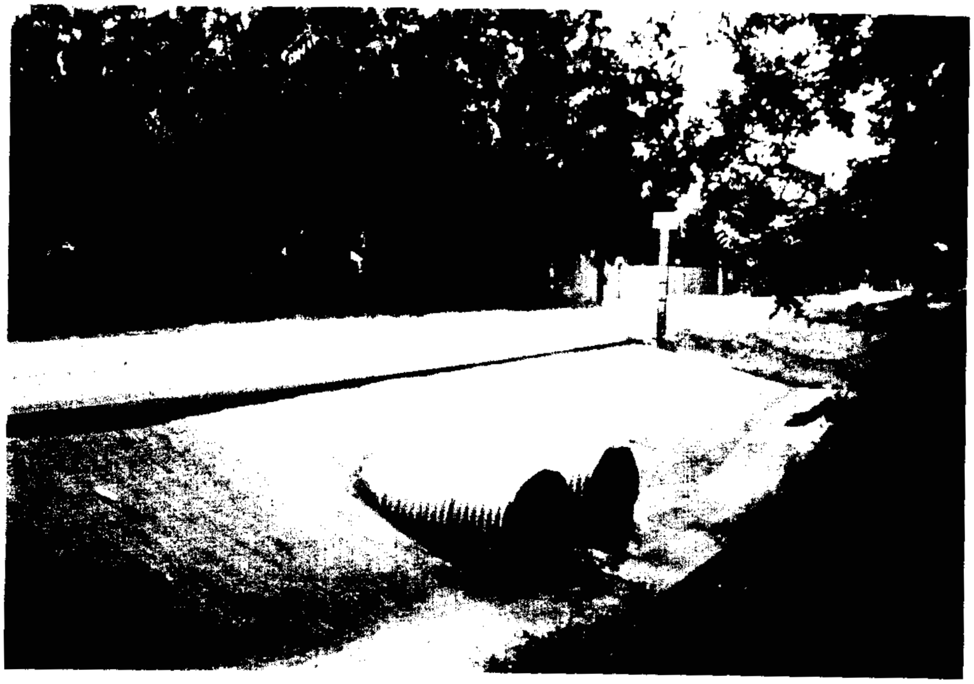
SITE 34
Evers at Huebner Creek
Upstream

SITE 35
Huebner Road at Huebner Creek
Downstream



SITE 36
Eckhert Road at Huebner Creek
Downstream

SITE 37
Babcock Road at Huebner Creek
Downstream



SITE 37.5
Babcock Road at West Huebner Creek
Downstream

SITE 38
Hollyhock at West Huebner Creek
Upstream

SITE 39
Babcock Road at West Huebner Creek
Upstream

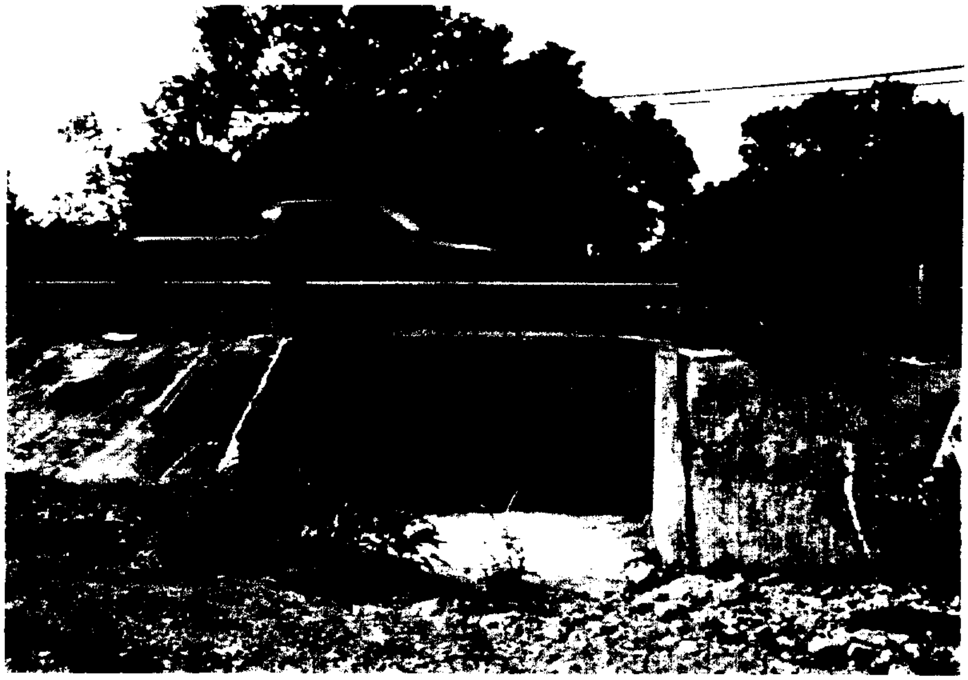
SITE 40
Lockhill Selma Road at West Huebner Creek
Upstream

SITE 41
White Bonnetl at West Huebner Creek
Downstream

SITE 42
Prue Road at West Huebner Creek
Upstream

SITE 43
Babcock Road at Huesta Creek
From North, Low Water Crossing, No Culvert

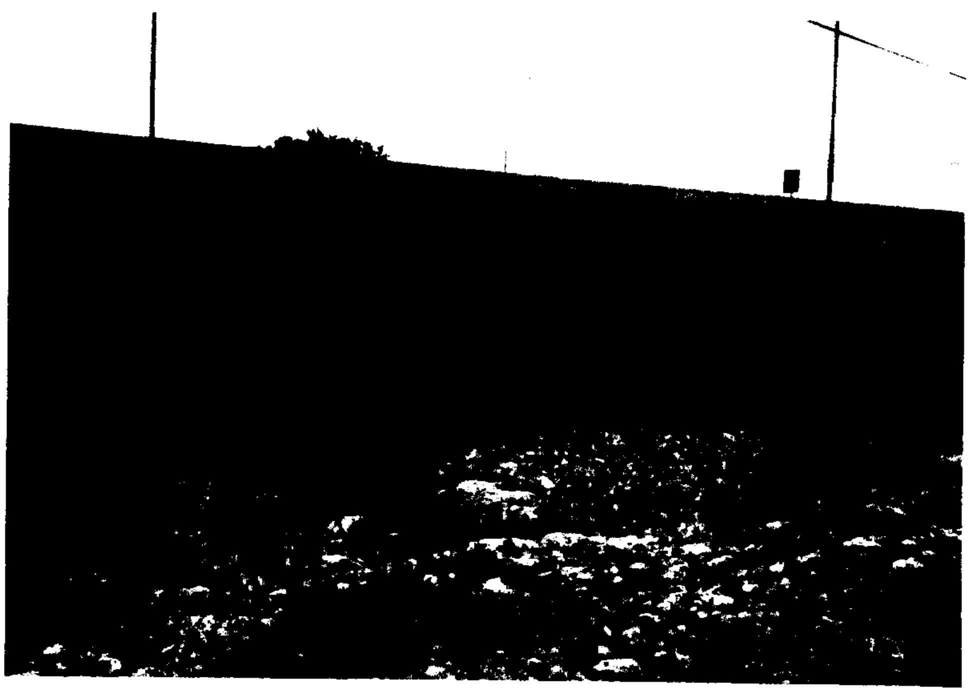
SITE 44
Danvers at Huesta Creek
Upstream



SITE 45
Hausman Road at Huesta Creek
Upstream

SITE 46
Loop 1604 at Huesta Creek
Upstream, Under Construction

SITE 47
UTSA Blvd. at Maverick Creek
Upstream



SITE 49
Loop 1604 at Maverick Creek
Upstream

SITE 50
Loop 1604 at Helotes Creek
Upstream

SITE 53
Braun Road at Helotes Creek
Downstream

APPENDIX B
Mitigation Projects

Table 1.1 - Leon Creek Problem Areas

Problem Number	Problem
LC-1	Split flow over Hausman Road creates low water crossing
LC-2	Building on edge of floodplain at section 150400- south of Hausman
LC-3	Low water crossing entire - Babcock Road under water
LC-4	Private drive low water crossing just south of Bandera
LC-5	Timber Creek Estates Subdivision 90+ houses in floodplain (Remove illegal dumping across Leon Creek from Timber Creek Estates: Old P-4)
LC-6	Flooded road - Heath Lane
LC-7	Grissom Road - Bridge flooding
LC-8	1 structure in floodplain and 4 on the edge just south of Grissom
LC-9	3 houses near floodplain in Pipers Meadow Subdivision -OUT
LC-10	Ingram Road - Low water crossing
LC-11	Frontage on Loop 410 - 2 Retail structures on edge of floodplain (car dealerships)
LC-12A	Culebra Road bridge causing problems
LC-12B	2 structures adjacent to Culebra Road bridge near floodplain - OUT
LC-13	1 apartment near edge of floodplain - OUT
LC-14	W. Commerce low water crossing
LC-15A	Pinn Road low water crossing
LC-15B	5 structures on edge of floodplain - OUT
LC-15C	1 commercial structure in floodplain
LC-16A	3 Structures in floodplain
LC-16B	8 Structures near floodplain - OUT
LC-17	Rodriquez Park in floodplain

Table 1.2 - Helotes Creek Problem Areas

Problem Number	Problem
HEL-1	Galm Road low water crossing
HEL-2A	Braun Road low water crossing
HEL-2B	2 structures near floodplain
HEL-3A	Leslie Road inundated
HEL-3B	Leslie Road inundated
HEL-3C	Leslie Road inundated
HEL-3D	7 structures in floodplain
HEL-4	FM 1604 overtopped - TxDOT project to mitigate
HEL-5	New Territories Subdivision; 48 houses near floodplain - OUT
HEL-6	Split flow down Wood Trail

Table 1.3 - Culebra Creek Problem Areas

Problem Number	Problem
C-1	Galm Road low water crossing
C-2	Galm Road low water crossing
C-3A	2 structures near floodplain
C-3B	1 structure near floodplain
C-4A	Stuebing low water crossing
C-4B	1 structure in floodplain
C-5A	FM 1604 inundated - TxDOT project to mitigate
C-5B	Culebra inundated
C-5C	10 structures near floodplain
C-6A	Culebra bridge inundated
C-6B	7 structures in floodplain, 3 structures OUT
C-7A	Easterling Road low water crossing; remove fill D.S. of low water crossing
C-7B	2 structures near floodplain
C-8A	Culebra Road in floodplain
C-8B	Timber Path in floodplain
C-8C	Old Grissom to New Grissom in floodplain
C-8D	7 structures near floodplain - OUT
C-8E	1 structure in floodplain

Table 1.4 - Huesta Creek Problem Areas

Problem Number	Problem
HUE-1	Split flow - no project required
HUE-2	4 structures on edge of floodplain - OUT
HUE-3	Inadequate drainage structure under Hausman Rd.
HUE-4	Alley View Mobile Park 12 trailers in floodplain, 2 others OUT
HUE-5	Babcock low water crossing inundated

Table 1.5 - Maverick Creek Problem Areas

Problem Number	Problem
M-1A	Babcock in floodplain
M-1B	Babcock in floodplain
M-1C	Babcock in floodplain
M-1D	7 structures near floodplain
M-2	Babcock in floodplain
M-3	Babcock too low
M-4	Babcock too low; UTSA Blvd. drainage structure too small
M-5	Hausman Rd. drainage structure too small (low water crossing)

Table 1.6 - Huebner Creek Problem Areas

Problem Number	Problem
HB-1	DeZavala Road low water crossing
HB-2	Section 90; 1 building in the floodplain
HB-3	Section 84; 3 houses near floodplain, Babcock Place Subdivision - OUT
HB-4	Prue Road low water crossing
HB-5A	Lockhill low water crossing
HB-5B	White Bonnet low water crossing
HB-5C	1 building in the floodplain, 1 other OUT
HB-5D	4 buildings in the floodplain
HB-6A	Hollyhock low water crossing (currently under design - 1994 bonds) Bridge cost shown is the additional amount required to bring bridge up to a 100 year design
HB-6B	5 structures near floodplain in Wellesly Manor Subdivision - OUT
HB-6C	5 structures in floodplain, 4 others OUT
HB-7A	Whitley low water crossing(currently under design - 1994 bonds) Bridge cost shown is the additional amount required to bring the bridge up to a 100 year design
HB-7B	1 structure near edge of floodplain - OUT
HB-8	Eckert Road low water crossing
HB-9A	Leon Valley in floodplain - approximately 167 structures flooded
HB-9B	9 houses near floodplain Win Creek Subdivision - OUT
HB-10	Timberhill low water crossing

Table 1.7 - French Creek Problem Areas

Problem Number	Problem
F-1	2 structures in floodplain, 13 others OUT or N/A
F-2A	Hausman Road low water crossing
F-2B	7 structures near floodplain - OUT
F-2C	4 structures near floodplain - OUT
F-3	Prue Road overtopped
F-4A	N Verde low water crossing
F-4B	S Verde low water crossing
F-4C	11 structures in floodplain
F-5A	Inadequate drainage structure at Bandera Rd.
F-5B	3 structures in floodplain, 2 others OUT
F-5C	1 structures in floodplain, 5 others OUT
F-5D	1 structures in floodplain, 3 others OUT
F-6	Inadequate drainage structures at Guilbeau
F-7	Wildwood Subdivision 9 houses near floodplain - OUT
F-8A	Inadequate drainage structure at Mainland
F-8B	2 structures near floodplain - No mitigation req'd if bridge is improved
F-8C	2 structures near floodplain - OUT
F-9A	Low Bid Lane in floodplain
F-9B	Heath in floodplain
F-9C	Clyde Dent Drive in floodplain

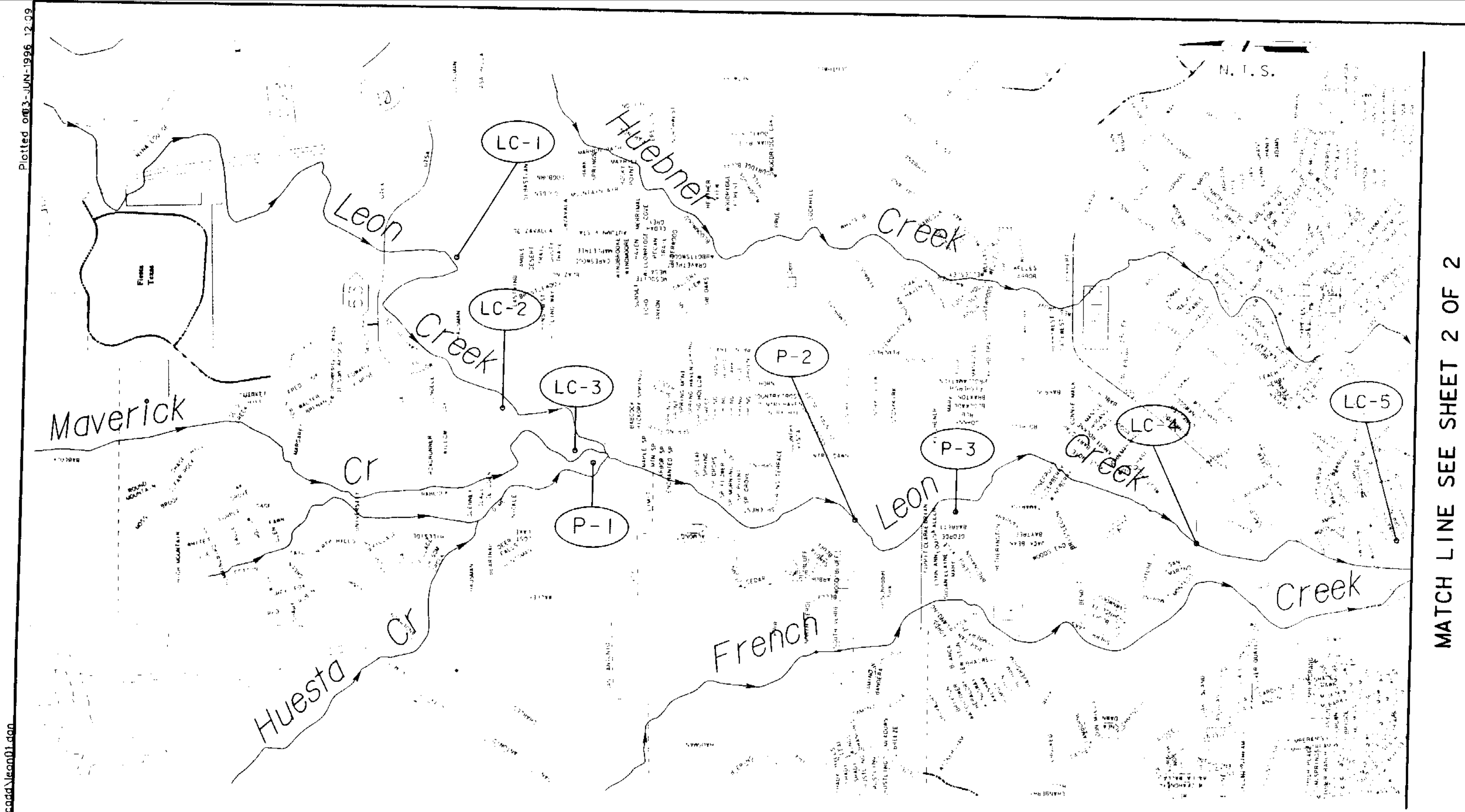


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FIGURE 1.1

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

LEON CREEK WATERSHED
PROJECT LOCATION MAP
LEON CREEK
SHEET 1 OF 2

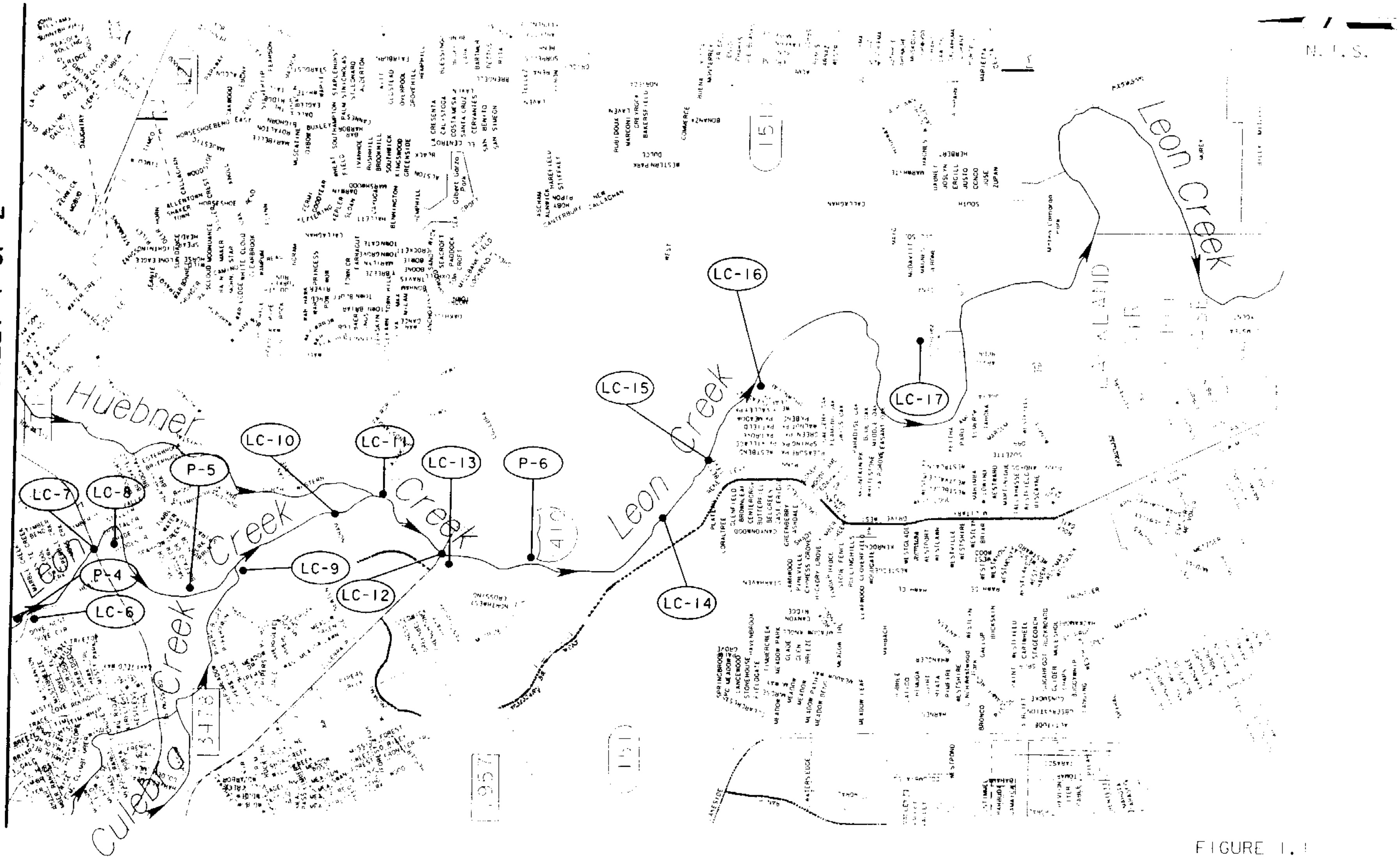

FIGURE 1.1

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

LEON CREEK WATERSHED
PROJECT LOCATION MAP
LEON CREEK

SHEET 2 OF 2

Plotted on 13-JUN-1996 12:10

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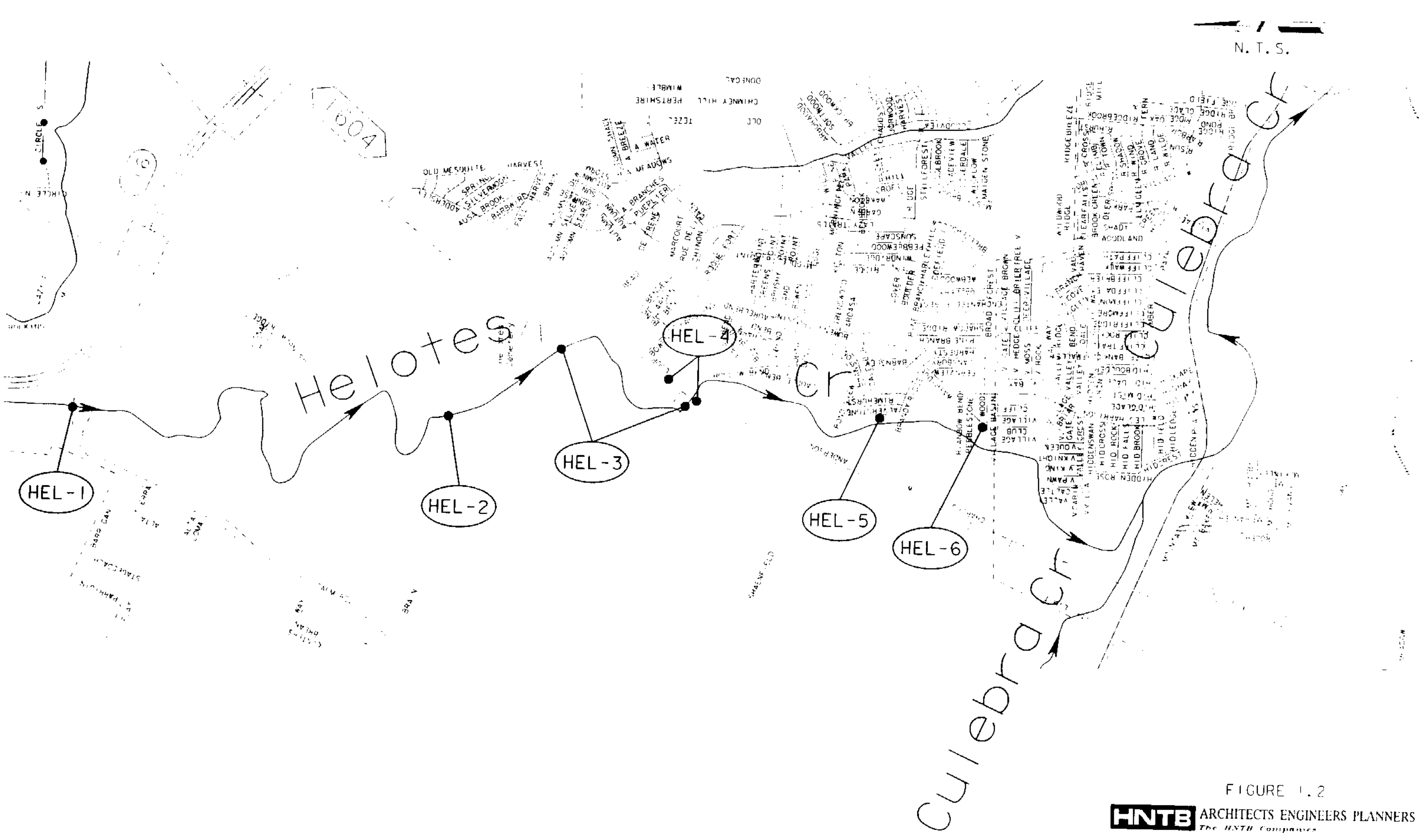

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Plotted ...JUN-1996

FIGURE 1.2

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

LEON CREEK WATERSHED
PROJECT LOCATION MAP
HELOTES CREEK

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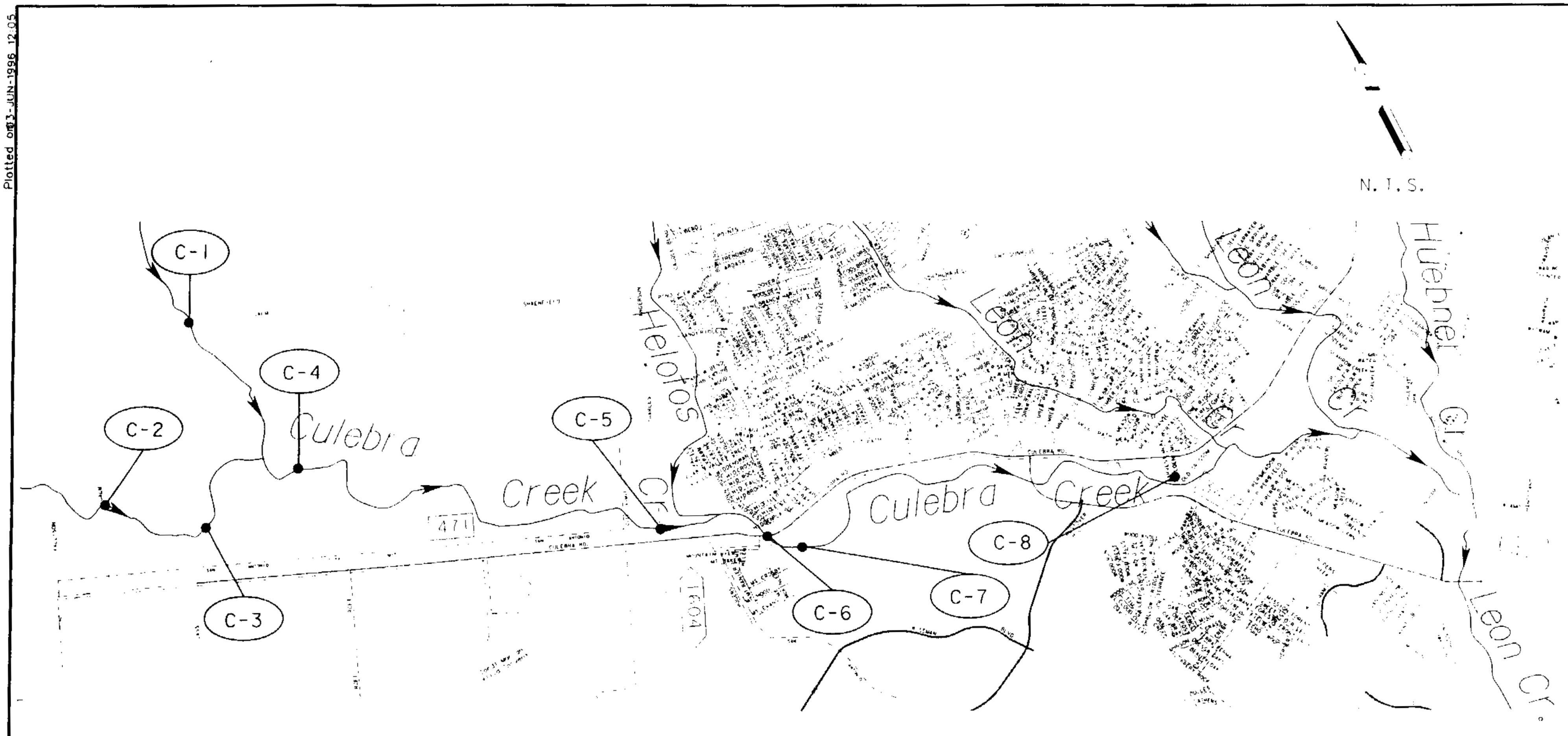

FIGURE 1.3

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

LEON CREEK WATERSHED
PROJECT LOCATION MAP
CULEBRA CREEK

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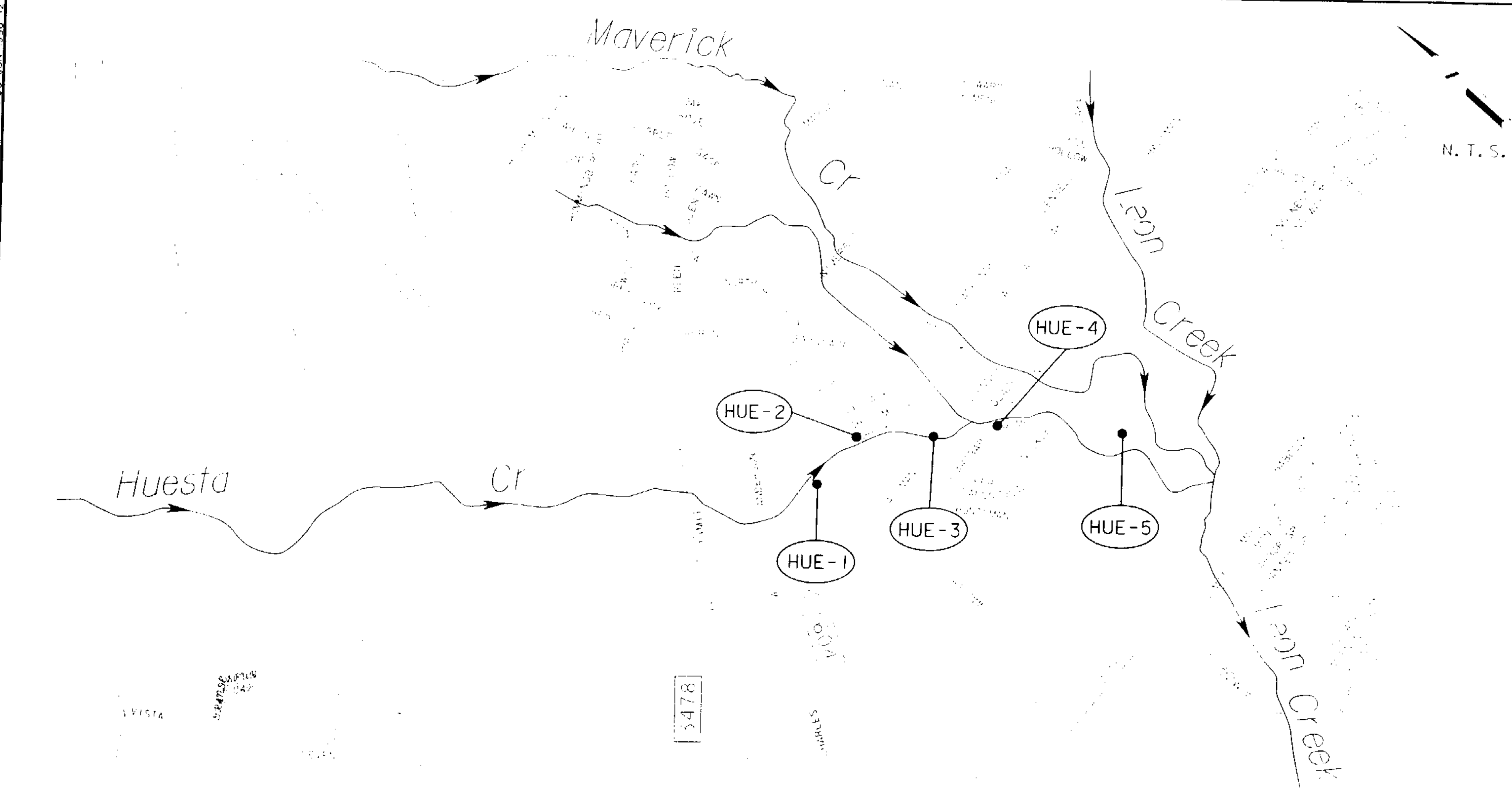

FIGURE 1.4

HNTB ARCHITECTS ENGINEERS PLANNERS

The HNTB Companies

LEON CREEK WATERSHED
PROJECT LOCATION MAP
HUESTA CREEK

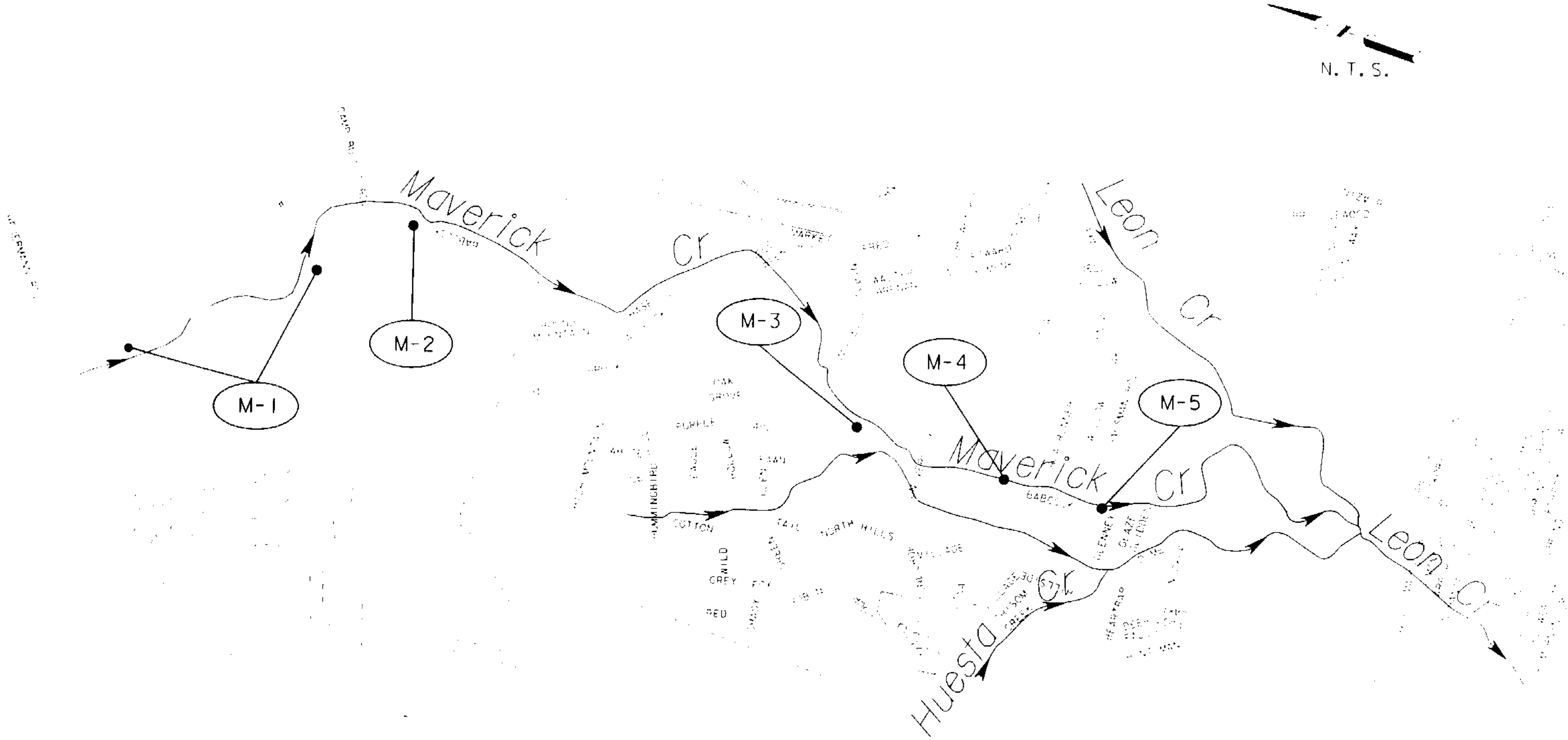

FIGURE 1.5

HNTB ARCHITECTS ENGINEERS PLANNERS

LEON CREEK WATERSHED
PROJECT LOCATION MAP
MAVERICK CREEK

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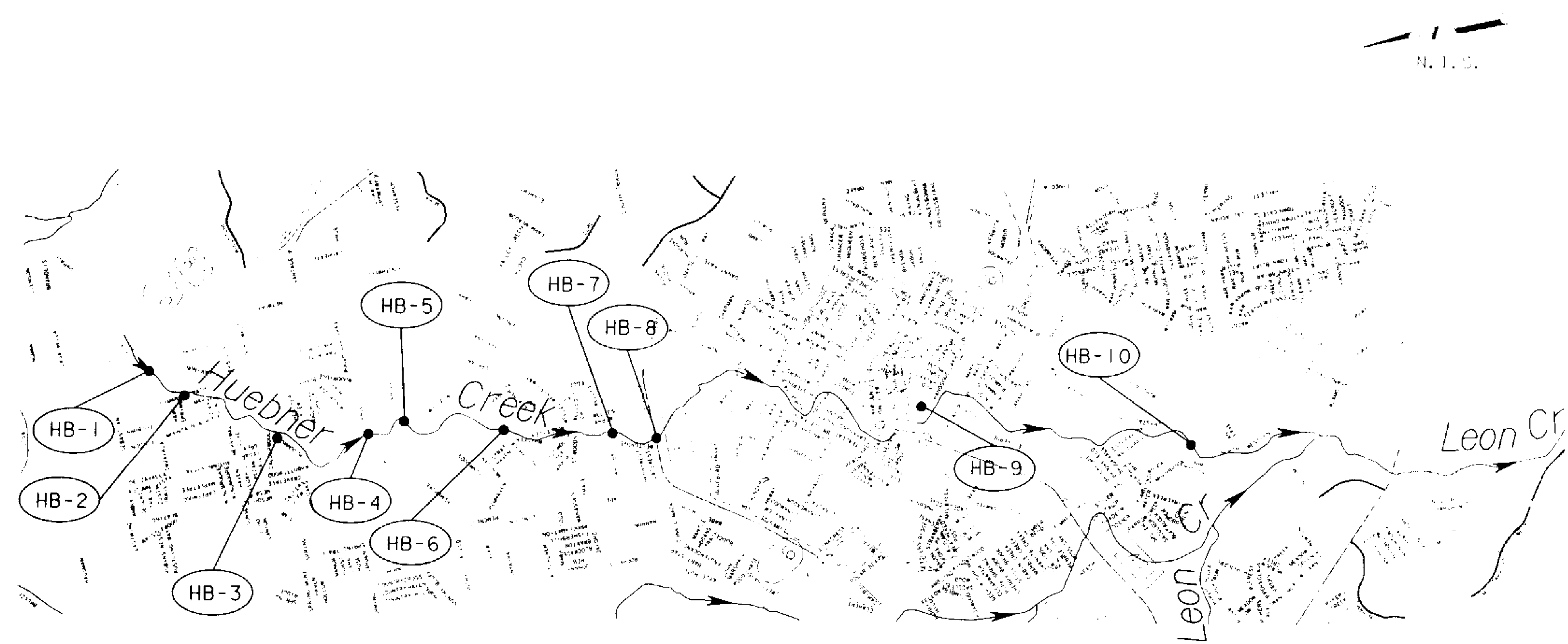

FIGURE 1.6

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

LEON CREEK WATERSHED
PROJECT LOCATION MAP
HUEBNER CREEK

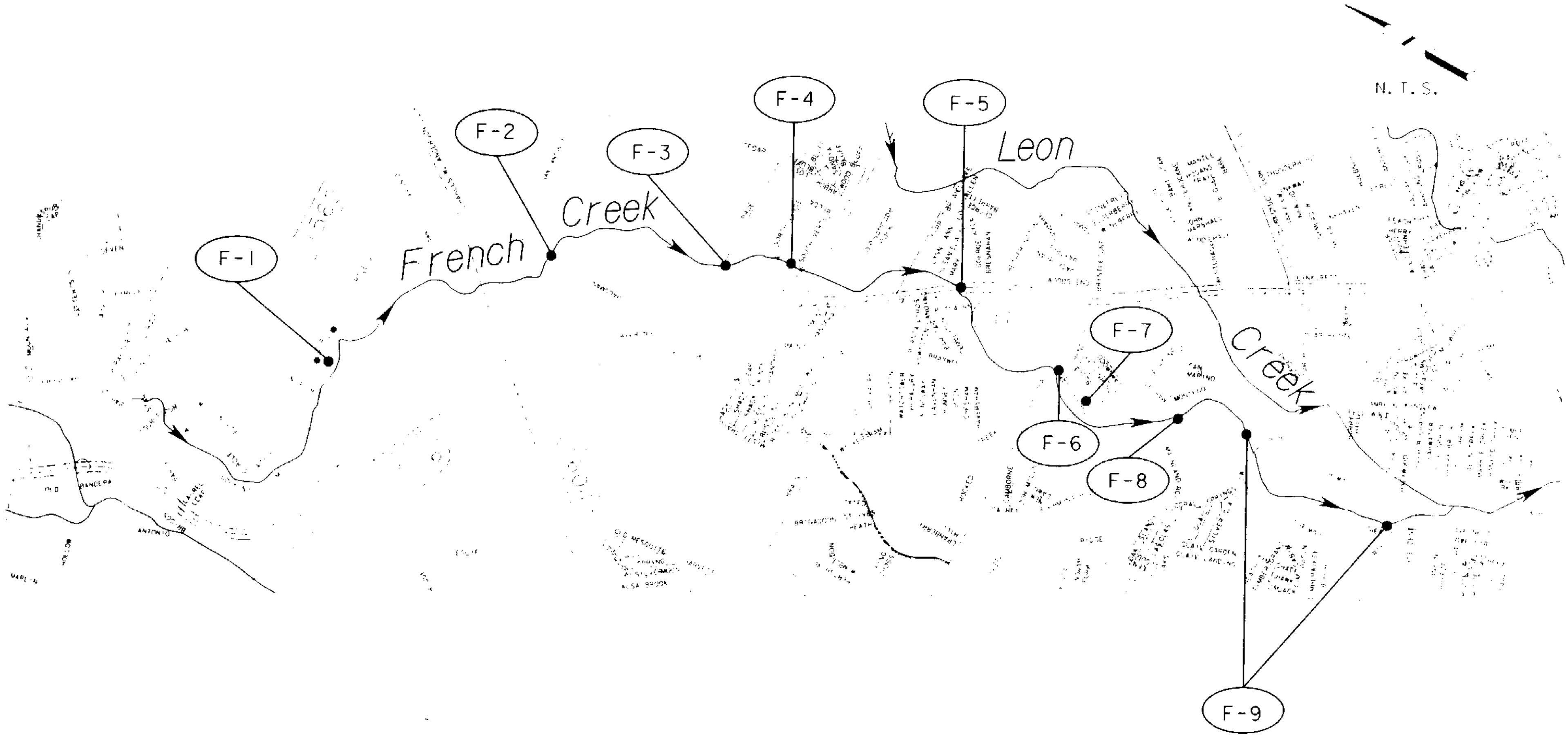

FIGURE 1.7

HNTB ARCHITECTS ENGINEERS PLANNERS

The HNTB Companies

LEON CREEK WATERSHED
PROJECT LOCATION MAP
FRENCH CREEK

CREEK	NUMBER	PROJECT	PROJECT TYPE	NO. OF STRUCTURES	PRIORITY	TYPE OF SOLUTION	[illegible]	[illegible]	Assumed Velocity (f.p.s.)	Required Length (l.f.)	Road Width (ft.)	Bridge Area (sq. ft.)	Unit Cost	Bridge Cost	Bridge/Roadway: Floodplain Width (ft.)	Length of Approach or Roadway (l.f.)	Fill Depth (ft.)	Fill Volume (c.y.)
Leon	LC-1	Split flow over Hausman Road creates low water crossing	Base	0	High	Levee									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-2	Building on edge of floodplain at section 150400- south of Hausman	Fringe	1	Low	Verify FF/Levee									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-3	Low water crossing entire - Babcock Road under water	Base	0	High	Bridge	[illegible]	[illegible]	10	350	30	10,500	$40	$420,000	[illegible]	[illegible]	[illegible]	[illegible]
Leon	LC-4	Private drive low water crossing just south of Bandera	Base	0	Low	Bridge	[illegible]	9	10	410	30	12,300	$40	$492,000	1,000	[illegible]	[illegible]	[illegible]
Leon	LC-5	Timber Creek Estates Subdivision 90+ houses in floodplain (Remove illegal dumping across Leon Creek from Timber Creek Estates: Old P-4)	Base	90	High	Channel Improvements									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-6	Flooded road - Heath Lane	Base	0	Low	Raise Roadway	[illegible]	[illegible]	-	20	30	600	$40	$24,000	[illegible]			
Leon	LC-7	Grissom Road - Bridge flooding	Base	0	Low	Bridge	[illegible]	[illegible]	10	280	75	21,000	$40	$840,000	[illegible]	[illegible]	[illegible]	[illegible]
Leon	LC-8	1 structure in floodplain and 4 on the edge just south of Grissom	Fringe	5	Low	Verify FF/Levee									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-9	3 houses near floodplain in Pipers Meadow Subdivision -OUT	N/A	0	High	Verify FF/Floodwall									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-10	Ingram Road - Low water crossing	Base	0	High	Bridge	[illegible]	[illegible]	10	690	50	34,500	$40	$1,380,000	1,500	[illegible]		[illegible]
Leon	LC-11	Frontage on Loop 410 - 2 Retail structures on edge of floodplain (car dealerships)	N/A	0	Moderate	Verify FF/Floodwall									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-12A	Culebra Road bridge causing problems	Base	0	High	Bridge	[illegible]	[illegible]	10	490	110	53,900	$40	$2,156,000	1,400	[illegible]	[illegible]	
Leon	LC-12B	2 structures adjacent to Culebra Road bridge near floodplain - OUT	N/A	0	Moderate	Purchase									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-13	1 apartment near edge of floodplain - OUT	N/A	0	Moderate	Verify FF/Floodwall									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-14	W. Commerce low water crossing	Base	0	Low	Bridge	[illegible]	20	10	460	60	27,600	$40	$1,104,000	2,500	2,000	[illegible]	[illegible]
Leon	LC-15A	Pinn Road low water crossing	Base	0	Low	Bridge	[illegible]	[illegible]	10	420	30	12,600	$40	$504,000	1,600	[illegible]		[illegible]
Leon	LC-15B	5 structures on edge of floodplain - OUT	N/A	0	Moderate	Verify FF/Channel Improvements									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-15C	1 commercial structure in floodplain	Base	1	Moderate	Purchase									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-16A	3 Structures in floodplain	Base	3	Low	Verify FF/Floodwall									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-16B	8 Structures near floodplain - OUT	N/A	0	Low	Verify FF/Floodwall									No Bridge, Roadway, or Fill Improvements Required			
Leon	LC-17	Rodriquez Park in floodplain	Base	Public Park	High	Signs and Gates									No Bridge, Roadway, or Fill Improvements Required			
Helotes	HEL-1	Galm Road low water crossing	Base	0	Moderate	Bridge	[illegible]	[illegible]	10	290	40	11,600	$40	$464,000	650	[illegible]		[illegible]
Helotes	HEL-2A	Braun Road low water crossing	Base	0	Moderate	Bridge	[illegible]	[illegible]	10	210	30	6,300	$40	$252,000	[illegible]	[illegible]	[illegible]	[illegible]
Helotes	HEL-2B	2 structures near floodplain	Fringe	2	Low	Verify FF/Levee									No Bridge, Roadway, or Fill Improvements Required			
Helotes	HEL-3A	Leslie Road inundated	Base	0	Low	Bridge	[illegible]	[illegible]	10	250	25	6,250	$40	$250,000	1,400	[illegible]		[illegible]
Helotes	HEL-3B	Leslie Road inundated	Base	0	Low	Bridge	[illegible]	[illegible]	10	240	25	6,000	$40	$240,000	1,400	[illegible]	[illegible]	[illegible]
Helotes	HEL-3C	Leslie Road inundated	Base	0	Low	Bridge	[illegible]	[illegible]	10	240	25	6,000	$40	$240,000	1,400	1,160	[illegible]	[illegible]
Helotes	HEL-3D	7 structures in floodplain	Base	7	Moderate	Purchase									No Bridge, Roadway, or Fill Improvements Required			
Helotes	HEL-4	FM 1604 overtopped - TxDOT project to mitigate	No	0	No	Bridge									TxDOT Funded			
Helotes	HEL-5	New Territories Subdivision; 48 houses near floodplain - OUT	N/A	0	High	Verify FF/Channel Improvements									No Bridge, Roadway, or Fill Improvements Required			
Helotes	HEL-6	Split flow down Wond Trail	Base	0	High	Channel Improvements									No Bridge, Roadway, or Fill Improvements Required			
Culebra	C-1	Galm Road low water crossing	Base	0	High	Bridge	[illegible]	6	10	380	60	22,800	$40	$912,000	2,400	[illegible]		[illegible]
Culebra	C-2	Galm Road low water crossing	Base	0	High	Bridge	[illegible]	5	10	250	60	15,000	$40	$600,000	650	400	[illegible]	[illegible]
Culebra	C-3A	2 structures near floodplain	Fringe	2	Low	Verify FF/Levee									No Bridge, Roadway, or Fill Improvements Required			
Culebra	C-3B	1 structure near floodplain	Fringe	1	Low	Verify FF/Floodwall									No Bridge, Roadway, or Fill Improvements Required			
Culebra	C-4A	Stuebing low water crossing	Base	0	High	Bridge	[illegible]	18	10	200	40	8,000	$40	$320,000	1,000	600		[illegible]
Culebra	C-4B	1 structure in floodplain	Base	1	Moderate	Verify FF/Levee									No Bridge, Roadway, or Fill Improvements Required			
Culebra	C-5A	FM 1604 inundated - TxDOT project to mitigate	No	0	No	Bridge									TxDOT Funded			
Culebra	C-5B	Culebra inundated	Base	0	High	Raise Roadway				30	40	1,200	$40	$48,000	[illegible]	[illegible]		
Culebra	C-5C	10 structures near floodplain	Fringe	10	High	Verify FF/Purchase									No Bridge, Roadway, or Fill Improvements Required			
Culebra	C-6A	Culebra bridge inundated	Base	0	High	Bridge	[illegible]	23	10	300	80	24,000	$40	$960,000	1,300	1,000		[illegible]
Culebra	C-6B	7 structures in floodplain. 3 structures OUT	Base	7	High	Verify FF/Purchase									No Bridge, Roadway, or Fill Improvements Required			
Culebra	C-7A	Easterling Road low water crossing; remove fill D.S. of low water crossing	Base	0	Low	Channel Improvements									No Bridge, Roadway, or Fill Improvements Required			
Culebra	C-7B	2 structures near floodplain	Fringe	2	Moderate	Verify FF/Purchase									No Bridge, Roadway, or Fill Improvements Required			
Culebra	C-8A	Culebra Road in floodplain	Base	0	Moderate	Bridge	[illegible]	20	10	330	65	21,450	$40	$858,000	5,500	[illegible]		[illegible]
Culebra	C-8B	Timber Path in floodplain	Base	0	Low	Bridge	[illegible]	20	10	400	35	14,000	$40	$560,000	2,300	[illegible]		[illegible]
Culebra	C-8C	Old Grissom to New Grissom in floodplain	Base	0	Low	Bridge	[illegible]	20	10	600	25	15,000	$40	$600,000	2,000	[illegible]		[illegible]
Culebra	C-8D	7 structures near floodplain - OUT	N/A	0	Moderate	Verify FF/Floodwall									No Bridge, Roadway, or Fill Improvements Required			
Culebra	C-8E	1 structure in floodplain	Base	1	Moderate	Purchase									No Bridge, Roadway, or Fill Improvements Required			
Huesta	HUE-1	Split flow - no project required	No	0	No	None									No Bridge, Roadway, or Fill Improvements Required			
Huesta	HUE-2	4 structures on edge of floodplain - OUT	N/A	0	Moderate	Verify FF/Floodwall									No Bridge, Roadway, or Fill Improvements Required			
Huesta	HUE-3	Inadequate drainage structure under Hausman Rd	Base	0	High	Bridge	[illegible]	4	10	150	30	4,500	$40	$180,000	[illegible]	[illegible]	[illegible]	[illegible]
Huesta	HUE-4	Alley View Mobile Park 12 trailers in floodplain, 2 others OUT	Base	12	Moderate	Verify FF/Levee									No Bridge, Roadway, or Fill Improvements Required			
Huesta	HUE-5	Babcock low water crossing inundated	Base	0	High	Bridge	[illegible]	9	10	120	30	3,600	$40	$144,000	[illegible]	[illegible]		[illegible]
Maverick	M-1A	Babcock in floodplain	Base	0	High	Bridge	[illegible]	11	10	100	30	3,000	$40	$120,000	1,200	1,100		[illegible]
Maverick	M-1B	Babcock in floodplain	Base	0	High	Bridge	[illegible]	11	10	100	30	3,000	$40	$120,000	1,200	1,100		[illegible]
Maverick	M-1C	Babcock in floodplain	Base	0	High	Bridge	[illegible]	11	10	100	30	3,000	$40	$120,000	1,200	[illegible]		[illegible]
Maverick	M-1D	7 structures near floodplain	Fringe	7	Low	Verify FF/Levee									No Bridge, Roadway, or Fill Improvements Required			
Maverick	M-2	Babcock in floodplain	Base	0	High	Levee									No Bridge, Roadway, or Fill Improvements Required			
Maverick	M-3	Babcock too low	Base	0	High	Levee									No Bridge, Roadway, or Fill Improvements Required			
Maverick	M-4	Babcock too low. UTSA Blvd. drainage structure too small	Base	0	High	Bridge	[illegible]	[illegible]	10	120	40	4,800	$40	$192,000	[illegible]	[illegible]		[illegible]
Maverick	M-5	Hausman Rd. drainage structure too small (low water crossing)	Base	0	High	Bridge	[illegible]	9	10	130	30	3,900	$40	$155,000	[illegible]	[illegible]		[illegible]
Huebner	HB-1	DeZavala Road low water crossing	Base	0	High	Bridge	2,775	5	10	70	80	5,600	$40	$224,000	[illegible]	[illegible]		[illegible]
Huebner	HB-2	Section 90: 1 building in the floodplain	Base	1	Moderate	Verify FF/Floodwall									No Bridge, Roadway, or Fill Improvements Required			
Huebner	HB-3	Section 84: 3 houses near floodplain. Babcock Place Subdivision - OUT	N/A	0	Moderate	Verify FF/Floodwall									No Bridge, Roadway, or Fill Improvements Required			
Huebner	HB-4	Prue Road low water crossing	Base	0	High	Bridge	5,529	2	10	[illegible]	40	11,600	$40	$464,000	[illegible]			
Huebner	HB-5A	Lockhill low water crossing	Base	0	Low	Bridge	5,529	4	10	150	30	4,500	$40	$180,000	[illegible]			[illegible]
Huebner	HB-5B	White Bonnet low water crossing	Base	0	Low	Bridge	5,529	4	10	150	30	4,500	$40	$180,000	[illegible]			[illegible]
Huebner	HB-5C	1 building in the floodplain. 1 other OUT	Base	1	Moderate	Verify FF/Floodwall									No Bridge, Roadway, or Fill Improvements Required			

CREEK	NUMBER	PROJECT	PROJECT TYPE	NO. OF STRUCTURES	PRIORITY	TYPE OF SOLUTION	Bridge/Roadway: Discharge (c.f.s.)	Depth (ft.)	Assumed Velocity (f.p.s.)	Required Length (l.f.)	Road Width (ft.)	Bridge Area (sq. ft.)	Unit Cost	Bridge Cost	Floodplain Width (ft.)	Length of Approach or Roadway (l.f.)	Fill Depth (ft.)	Fill Volume (c.y.)	Fill Unit Cost	Fill Cost	Paving Area
Leon	LC-1	Split flow over Hausman Road creates low water crossing	Base	0	High	Levee	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-2	Building on edge of floodplain at section 150400- south of Hausman	Fringe	1	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-3	Low water crossing entire - Babcock Road under water	Base	0	High	Bridge	35,081	11	10	350	30	10,500	$40	$420,000	2,100	1,750	6	26,500	$8	$212,000	5,900
Leon	LC-4	Private drive low water crossing just south of Bandera	Base	0	Low	Bridge	35,081	9	10	410	30	12,300	$40	$492,000	1,000	590	5	7,200	$8	$57,600	2,000
Leon	LC-5	Timber Creek Estates Subdivision 90+ houses in floodplain (Remove illegal dumping across Leon Creek from Timber Creek Estates: Old P-4)	Base	90	High	Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-6	Flooded road - Heath Lane	Base	0	Low	Raise Roadway				20	30	600	$40	$24,000	1,200	1,180	5	14,300	$8	$114,400	4,000
Leon	LC-7	Grissom Road - Bridge flooding	Base	0	Low	Bridge	43,634	18	10	280	75	21,000	$40	$840,000	700	420	19	45,000	$8	$360,000	3,600
Leon	LC-8	1 structure in floodplain and 4 on the edge just south of Grissom	Fringe	5	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-9	3 houses near floodplain in Pipers Meadow Subdivision -OUT	N/A	0	High	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-10	Ingram Road - Low water crossing	Base	0	High	Bridge	97,495	15	10	690	50	34,500	$40	$1,380,000	1,500	810	13	42,600	$8	$340,800	4,600
Leon	LC-11	Frontage on Loop 410 - 2 Retail structures on edge of floodplain (car dealerships)	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-12A	Culebra Road bridge causing problems	Base	0	High	Bridge	97,495	22	10	490	110	53,900	$40	$2,156,000	1,400	910	8	41,600	$8	$332,800	11,200
Leon	LC-12B	2 structures adjacent to Culebra Road bridge near floodplain - OUT	N/A	0	Moderate	Purchase	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-13	1 apartment near edge of floodplain - OUT	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-14	W. Commerce low water crossing	Base	0	Low	Bridge	97,495	24	10	460	60	27,600	$40	$1,104,000	2,500	2,040	16	154,800	$8	$1,238,400	13,700
Leon	LC-15A	Pinn Road low water crossing	Base	0	Low	Bridge	97,495	27	10	420	30	12,600	$40	$504,000	1,600	1,180	13	50,600	$8	$404,800	4,000
Leon	LC-15B	5 structures on edge of floodplain - OUT	N/A	0	Moderate	Verify FF/Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-15C	1 commercial structure in floodplain	Base	1	Moderate	Purchase	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-16A	3 Structures in floodplain	Base	3	Low	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-16B	8 Structures near floodplain - OUT	N/A	0	Low	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-17	Rodriguez Park in floodplain	Base	Public Park	High	Signs and Gates	No Bridge, Roadway, or Fill Improvements Required														
Helotes	HEL-1	Galm Road low water crossing	Base	0	Moderate	Bridge	30,780	12	10	290	40	11,600	$40	$464,000	650	360	2	1,800	$8	$14,400	1,700
Helotes	HEL-2A	Braun Road low water crossing	Base	0	Moderate	Bridge	30,780	18	10	210	30	6,300	$40	$252,000	1,000	790	4	7,300	$8	$58,400	1,700
Helotes	HEL-2B	2 structures near floodplain	Fringe	2	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Helotes	HEL-3A	Leslie Road inundated	Base	0	Low	Bridge	30,780	14	10	250	25	6,250	$40	$250,000	1,400	1,150	2	4,400	$8	$35,200	3,300
Helotes	HEL-3B	Leslie Road inundated	Base	0	Low	Bridge	30,780	15	10	240	25	6,000	$40	$240,000	1,400	1,160	3	7,000	$8	$56,000	3,300
Helotes	HEL-3C	Leslie Road inundated	Base	0	Low	Bridge	30,780	15	10	240	25	6,000	$40	$240,000	1,400	1,160	3	7,000	$8	$56,000	3,300
Helotes	HEL-3D	7 structures in floodplain	Base	7	Moderate	Purchase	No Bridge, Roadway, or Fill Improvements Required														
Helotes	HEL-4	FM 1604 overtopped - TxDOT project to mitigate	No	0	No	Bridge	TxDOT Funded														
Helotes	HEL-5	New Territories Subdivision. 48 houses near floodplain - OUT	N/A	0	High	Verify FF/Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Helotes	HEL-6	Split flow down Wood Trail	Base	0	High	Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-1	Galm Road low water crossing	Base	0	High	Bridge	21,779	6	10	380	60	22,800	$40	$912,000	2,400	2,020	10	82,400	$8	$659,200	3,500
Culebra	C-2	Galm Road low water crossing	Base	0	High	Bridge	11,837	5	10	250	60	15,000	$40	$600,000	650	400	4	5,500	$8	$44,000	2,700
Culebra	C-3A	2 structures near floodplain	Fringe	2	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-3B	1 structure near floodplain	Fringe	1	Low	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-4A	Stuebing low water crossing	Base	0	High	Bridge	28,301	18	10	200	40	8,000	$40	$320,000	1,000	800	3	6,200	$8	$49,600	3,600
Culebra	C-4B	1 structure in floodplain	Base	1	Moderate	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-5A	FM 1604 inundated - TxDOT project to mitigate	No	0	No	Bridge	TxDOT Funded														
Culebra	C-5B	Culebra inundated	Base	0	High	Raise Roadway				30	40	1,200	$40	$48,000	2,500	2,470	2	12,100	$8	$96,800	11,000
Culebra	C-5C	10 structures near floodplain	Fringe	10	High	Verify FF/Purchase	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-6A	Culebra bridge inundated	Base	0	High	Bridge	56,891	21	10	300	80	24,000	$40	$960,000	1,300	1,000	5	21,400	$8	$171,200	8,900
Culebra	C-6B	7 structures in floodplain. 3 structures OUT	Base	7	High	Verify FF/Purchase	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-7A	Eastering Road low water crossing: remove fill D.S. of low water crossing	Base	0	Low	Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-7B	2 structures near floodplain	Fringe	2	Moderate	Verify FF/Purchase	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-8A	Culebra Road in floodplain	Base	0	Moderate	Bridge	55,800	20	10	330	65	21,450	$40	$858,000	5,500	5,170	3	54,100	$8	$432,800	37,400
Culebra	C-8B	Timber Path in floodplain	Base	0	Low	Bridge	56,891	20	10	400	35	14,000	$40	$560,000	2,300	1,900	3	13,600	$8	$108,800	7,400
Culebra	C-8C	Old Grissom to New Grissom in floodplain	Base	0	Low	Bridge	56,891	20	10	600	25	15,000	$40	$600,000	2,000	1,400	7	24,000	$8	$192,000	3,900
Culebra	C-8D	7 structures near floodplain - OUT	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-8E	1 structure in floodplain	Base	1	Moderate	Purchase	No Bridge, Roadway, or Fill Improvements Required														
Huesta	HUE-1	Split flow - no project required	No	0	No	None	No Bridge, Roadway, or Fill Improvements Required														
Huesta	HUE-2	4 structures on edge of floodplain - OUT	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Huesta	HUE-3	Inadequate drainage structure under Hausman Rd.	Base	0	High	Bridge	7,655	6	10	150	30	4,500	$40	$180,000	1,100	950	4	8,800	$8	$70,400	3,200
Huesta	HUE-4	Alley View Mobile Park 12 trailers in floodplain, 2 others OUT	Base	12	Moderate	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Huesta	HUE-5	Babcock low water crossing inundated	Base	0	High	Bridge	9,069	9	10	120	30	3,600	$40	$144,000	3,800	3,680	3	24,200	$8	$193,600	12,300
Maverick	M-1A	Babcock in floodplain	Base	0	High	Bridge	8,071	11	10	100	30	3,000	$40	$120,000	1,200	1,100	5	13,300	$8	$106,400	3,700
Maverick	M-1B	Babcock in floodplain	Base	0	High	Bridge	8,071	11	10	100	30	3,000	$40	$120,000	1,200	1,100	5	13,300	$8	$106,400	3,700
Maverick	M-1C	Babcock in floodplain	Base	0	High	Bridge	8,071	11	10	100	30	3,000	$40	$120,000	1,200	1,100	5	13,300	$8	$106,400	3,700
Maverick	M-1D	7 structures near floodplain	Fringe	7	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Maverick	M-2	Babcock in floodplain	Base	0	High	Levee	No Bridge, Roadway, or Fill Improvements Required														
Maverick	M-3	Babcock too low	Base	0	High	Levee	No Bridge, Roadway, or Fill Improvements Required														
Maverick	M-4	Babcock too low. UTSA Blvd. drainage structure too small	Base	0	High	Bridge	9,708	10	10	120	40	4,800	$40	$192,000	2,100	1,980	2	9,700	$8	$77,600	8,900
Maverick	M-5	Hausman Rd. drainage structure too small (low water crossing)	Base	0	High	Bridge	9,483	9	10	130	30	3,900	$40	$156,000	800	670	3	4,500	$8	$36,000	2,300
Huebner	HB-1	DeZavala Road low water crossing	Base	0	High	Bridge	2,775	5	10	70	80	5,600	$40	$224,000	1,300	1,230	4	20,500	$8	$164,000	11,000
Huebner	HB-2	Section 90: 1 building in the floodplain	Base	1	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Huebner	HB-3	Section 84: 3 houses near floodplain, Babcock Place Subdivision - OUT	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Huebner	HB-4	Prue Road low water crossing	Base	0	High	Bridge	5,529	2	10	290	40	11,600	$40	$464,000	500	210	2	1,100	$8	$8,800	1,000
Huebner	HB-5A	Lockhill low water crossing	Base	0	Low	Bridge	5,529	4	10	150	30	4,500	$40	$180,000	900	750	4	6,900	$8	$55,200	2,600
Huebner	HB-5B	White Bonnet low water crossing	Base	0	Low	Bridge	5,529	4	10	150	30	4,500	$40	$180,000	900	750	4	6,900	$8	$55,700	2,600
Huebner	HB-5C	1 building in the floodplain. 1 other OUT	Base	1	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														

Table 2 - Project Details

Unit Cost	Paving Cost	Total Cost	Floodwall Length (l.f.)	Conc. (c.y.)	Unit Cost	Total Cost	Levee Length (l.f.)	Fill (c.y.)	Unit Cost	Stabilize (ac.)	Unit Cost	Total Cost	Channel Improvements Length (l.f.)	Excavation (c.y.)	Unit Cost	Disposal (c.y.)	Unit Cost	Riprap (s.y.)	Unit Cost	Total Cost	Purchase Structures No. Units	Total Area (s.f.)	Unit Cost	Total Cost
			Not economical when levee possible				700	1,500	$10	2	$5,500	$26,000	Channel Improvements not economical								No Structures			
			Not economical when levee possible				450	2,000	$10	2	$5,500	$31,000	Channel Improvements not economical								Structure can be protected			
$20	$118,000	$751,000	No Structures				No Structures						No Structures								No Structures			
$20	$40,000	$590,000	No Structures				No Structures						No Structures								No Structures			
			Floodwall would obstruct drainage from neighborhood				Levee would obstruct drainage from neighborhood						2,200	160,000	$8	160,000	$3	86,000	$30	$4,340,000	Structures can be protected			
$20	$80,000	$219,000	Floodplain not steep enough to provide drainage behind floodwall				Floodplain not steep enough to provide drainage behind levee						Channel Improvements not economical								No Structures			
$20	$72,000	$1,273,000	No Structures				No Structures						No Structures								No Structures			
			Not economical when levee possible				900	1,500	$10	2	$5,500	$26,000	Channel Improvements not economical								Structures can be protected			
			Finished floors surveyed -- NOT IN FLOODPLAIN				Side slope too steep for levee construction						Channel Improvements not economical								Structures can be protected			
$20	$92,000	$1,813,000	No Structures				No Structures						No Structures								No Structures			
			Finished floors surveyed -- NOT IN FLOODPLAIN				Side slope too steep for levee construction						Channel Improvements not economical								Structures can be protected			
$20	$224,000	$2,713,000	No Structures				No Structures						No Structures								No Structures			
			Culebra Road obstructs drainage of floodwall				Culebra Road obstructs drainage of levee						Channel Improvements (as shown for Project LC-12A) could lower the W.S.E., and is considered an option								Finished floors surveyed -- NOT IN FLOODPLAIN			
			Finished floors surveyed -- NOT IN FLOODPLAIN				Side slope too steep for levee construction						Channel Improvements not economical								Structure can be protected			
$20	$274,000	$2,617,000	No Structures				No Structures						No Structures								No Structures			
$20	$80,000	$989,000	No Structures				No Structures						No Structures								No Structures			
			Finished floors surveyed -- NOT IN FLOODPLAIN				Floodplain not steep enough to provide drainage behind levee						Finished floors surveyed -- NOT IN FLOODPLAIN								Structures can be protected			
			Floodplain not steep enough to provide drainage behind floodwall				Floodplain not steep enough to provide drainage behind levee						Floodplain is flat, channel improvements would need to lower flood elevations 10 feet								1	3,200	$75	$240,000
			1,600	1,800	$400	$720,000	Side slope too steep for levee construction						Channel Improvements not economical								Structures can be protected			
			Finished floors surveyed -- NOT IN FLOODPLAIN				Side slope too steep for levee construction						Channel Improvements not economical								Structures can be protected			
			Public Park				Public Park						Public Park								Public Park			
$20	$34,000	$513,000	No Structures				No Structures						No Structures								No Structures			
$20	$54,000	$365,000	No Structures				No Structures						No Structures								No Structures			
			Not economical when levee possible				700	2,500	$10	2	$5,500	$36,000	Channel Improvements not economical								Structures can be protected			
$20	$66,000	$352,000	No Structures				No Structures						No Structures								No Structures			
$20	$66,000	$363,000	No Structures				No Structures						No Structures								No Structures			
$20	$66,000	$363,000	No Structures				No Structures						No Structures								No Structures			
			Water surface too deep for floodwall				Side slope too steep for levee construction						Channel Improvements not economical								7	16,800	$75	$1,260,000
			No Structures				No Structures						No Structures								No Structures			
			Floodwall would obstruct drainage from neighborhood				Levee would obstruct drainage from neighborhood						Finished floors surveyed -- NOT IN FLOODPLAIN								Structures can be protected			
			Floodwall would obstruct drainage from neighborhood				Levee would obstruct drainage from neighborhood						2,000	100,000	$8	100,000	$3	10,000	$30	$1,400,000	Structures can be protected			
$20	$270,000	$1,842,000	No Structures				No Structures						No Structures								No Structures			
$20	$54,000	$699,000	No Structures				No Structures						No Structures								No Structures			
			Not economical when levee possible				1,600	4,500	$10	2	$5,500	$56,000	Channel Improvements not economical								Structures can be protected			
			500	380	$400	$152,000	Side slope too steep for levee construction						Channel Improvements not economical								Structure can be protected			
$20	$72,000	$442,000	No Structures				No Structures						No Structures								No Structures			
			Not economical when levee possible				500	2,050	$10	1	$5,500	$26,000	Channel Improvements not economical								Structures can be protected			
			No Structures				No Structures						No Structures								No Structures			
$20	$220,000	$365,000	No Structures				No Structures						No Structures								No Structures			
			Leslie Road obstructs drainage of floodwall				Leslie Road obstructs drainage of levee						Channel Improvements not economical								10	13,000	$75	$975,000
$20	$178,000	$1,310,000	No Structures				No Structures						No Structures								No Structures			
			Floodplain not steep enough to provide drainage behind floodwall				Floodplain not steep enough to provide drainage behind levee						Channel already improved in the adjacent reach								7	15,400	$75	$1,155,000
			No Structures				No Structures						300	13,000	$8	13,000	$3	0	$30	$143,000	No Structures			
			Easterling Road would obstruct drainage behind floodwall				Easterling Road would obstruct drainage behind levee						Channel Improvements not economical								2	2,800	$75	$210,000
$20	$748,000	$2,039,000	No Structures				No Structures						No Structures								No Structures			
$20	$148,000	$817,000	No Structures				No Structures						No Structures								No Structures			
$20	$78,000	$871,000	No Structures				No Structures						No Structures								No Structures			
			Finished floors surveyed -- NOT IN FLOODPLAIN				Side slope too steep for levee construction						Existing channel is wide, channel improvements may not lower W.S.E. sufficiently								Structures can be protected			
			Floodplain not steep enough to provide drainage behind floodwall				Floodplain not steep enough to provide drainage behind levee						Existing floodplain is flat, channel improvements would have to lower flood elevations by 4' - 5'								1	1,600	$75	$120,000
			No Structures				No Structures						No Structures								No Structures			
			Finished floors surveyed -- NOT IN FLOODPLAIN				Side slope too steep for levee construction						Channel Improvements not economical								Structures can be protected			
$20	$64,000	$315,000	No Structures				No Structures						No Structures								No Structures			
			Not economical when levee possible				1,000	2,500	$10	2	$5,500	$36,000	Channel Improvements not economical								Structures can be protected			
$20	$246,000	$584,000	No Structures				No Structures						No Structures								No Structures			
$20	$74,000	$301,000	No Structures				No Structures						No Structures								No Structures			
$20	$74,000	$301,000	No Structures				No Structures						No Structures								No Structures			
$20	$74,000	$301,000	No Structures				No Structures						No Structures								No Structures			
			Not economical when levee possible				1,500	4,500	$10	2	$5,500	$56,000	Channel Improvements not economical								Structures can be protected			
			Not economical when levee possible				1,200	7,000	$10	4	$5,500	$92,000	Channel Improvements not economical								No Structures			
			Not economical when levee possible				500	2,500	$10	2	$5,500	$36,000	Channel Improvements not economical								No Structures			
$20	$178,000	$448,000	No Structures				No Structures						No Structures								No Structures			
$20	$46,000	$239,000	No Structures				No Structures						No Structures								No Structures			
$20	$220,000	$609,000	No Structures				No Structures						No Structures								No Structures			
			800	250	$400	$100,000	Side slope too steep for levee construction						Channel Improvements not economical								Structure can be protected			
			Finished floors surveyed -- NOT IN FLOODPLAIN				Side slope too steep for levee construction						Channel Improvements not economical								Structures can be protected			
$20	$20,000	$393,000	No Structures				No Structures						No Structures								No Structures			
$20	$52,000	$288,000	No Structures				No Structures						No Structures								No Structures			
$20	$52,000	$288,000	No Structures				No Structures						No Structures								No Structures			
			600	430	$400	$172,000	Side slope too steep for levee construction						Channel Improvements not economical								Structures can be protected			

					Detention Pond Improvements							Signs and Gates				
(ac.)	Unit Cost	Excavation (c.y.)	Unit Cost	Disposal (c.y.)	Unit Cost	Fill (c.y.)	Unit Cost	Riprap (c.y.)	Unit Cost	Misc. Cost (Pipe, Seeding)	Total Cost	Total Cost	Total Project Cost	Remarks	Benefits	Benefit/Cost Ratio
													$26,000		$1,000,000	38.5
													$31,000		$250,000	8.1
													$791,000	Project included in MPO's Long Range Plan	$1,000,000	1.3
													$590,000	Install "Subject to Flooding" Sign	$1,000,000	1.7
													$4,340,000	Remove illegal fill (no solid waste or hazardous waste assumed - dirt only). City should seek funds from property owner who placed fill	$10,000,000	2.3
													$219,000	Raise roadway (culvert size estimated at 20 feet). Channel improvements from LC-5 may sufficiently reduce the W.S.E. such that no improvements would be necessary	$1,000,000	4.6
													$1,273,000		$2,000,000	1.6
													$26,000		$750,000	28.8
													$0			
													$1,813,000		$2,000,000	1.1
													[illegible]			
													$2,713,000	Improvements could also benefit Project LC-12B	$2,000,000	0.7
													[illegible]	Improvements for Project LC-12A may reclaim structures from floodplain. If floodplain cannot be sufficiently lowered with LC-12A improvements, structures may have to be purchased		
													$0			
													$2,617,000	Improve in future with main lane construction (Alternate route available)	$1,500,000	0.6
													$989,000		$1,500,000	1.5
													$0			
													$240,000	Structure is 10' below flood elevation and would have to be purchased	$1,120,000	4.7
													$720,000	Potential to use CDBG funding	$200,000	0.3
													$0	Potential to use CDBG funding		
												$50,000	$50,000	Use Flood Warning Signs and Gates (Potential for CDBG funding)	$1,000,000	20.0
													$513,000		$1,000,000	1.9
													$365,000		$1,000,000	2.7
													$36,000		$550,000	15.3
													$352,000	Alternate route available	$1,000,000	2.8
													$363,000	Alternate route available	$1,000,000	2.8
													$363,000	Alternate route available	$1,000,000	2.8
													$1,260,000		$1,500,000	1.2
													$0	Project included in MPO's Long Range Plan - TxDOT funded		
													$0			
													$1,400,000		$10,000,000	7.1
													$1,842,000		$2,000,000	1.1
													$699,000		$2,000,000	2.9
													$56,000		$200,000	3.6
													$152,000		$150,000	1.0
													$442,000		$1,500,000	3.4
													$26,000		$250,000	9.6
													$0	Project included in MPO's Long Range Plan - TxDOT funded		
													$365,000	Raise roadway (culvert size estimated at 30 feet)	$1,500,000	4.1
													$975,000		$1,500,000	1.5
													$1,310,000		$2,000,000	1.5
													$1,155,000		$1,500,000	1.3
													$143,000	Remove illegal fill (no solid waste or hazardous waste assumed - dirt only). City should seek funds from property owner who placed fill. A developer funded bridge will be required in the future	$250,000	1.7
													$210,000		$200,000	1.0
													$2,039,000		$1,500,000	0.7
													$817,000	Bridge is on skew	$1,500,000	1.8
													$871,000	Bridge is on skew	$1,500,000	1.7
													$0			
													$120,000		$150,000	1.3
													$0	Split flow is not a significant problem		
													$0			
													$315,000		$1,500,000	4.8
													$36,000	Mobile home park, finish floor elevations should be higher than existing ground elevations	$1,000,000	27.8
													$584,000	Project included in MPO's Long Range Plan	$1,500,000	2.6
													$301,000	Project included in MPO's Long Range Plan	$1,000,000	3.3
													$301,000	Project included in MPO's Long Range Plan	$1,000,000	3.3
													$301,000	Project included in MPO's Long Range Plan	$1,000,000	3.3
													$56,000		$900,000	16.1
													$92,000	Project is included as roadway improvements in MPO's Long Range Plan. Levee is recommended as a more efficient use of funds.	$1,500,000	16.3
													$36,000	Project is included as roadway improvements in MPO's Long Range Plan. Levee is recommended as a more efficient use of funds	$1,500,000	41.7
													$448,000	Project included in MPO's Long Range Plan	$1,500,000	3.3
													$239,000		$1,500,000	6.3
													$609,000		$1,000,000	1.6
													$100,000		$2,000,000	20.0
													$0			
													$493,000	Project included in MPO's Long Range Plan	$1,000,000	2.0
													$288,000		$1,000,000	3.5
													$288,000		$1,000,000	3.5
													$172,000		$250,000	1.5

CREEK	NUMBER	PROJECT	PROJECT TYPE	NO. OF STRUCTURES	PRIORITY	TYPE OF SOLUTION	Bridge/Roadway: Discharge (c.f.s.)	Depth (ft.)	Assumed Velocity (f.p.s.)	Required Length (l.f.)	Road Width (ft.)	Bridge Area (sq. ft.)	Unit Cost	Bridge Cost	Floodplain Width (ft.)	Length of Approach or Roadway (l.f.)	Fill Depth (ft.)	Fill Volume (c.y.)	Fill Unit Cost	Fill Cost	Paving Area (s.y.)
Leon	LC-1	Split flow over Hausman Road creates low water crossing	Base	0	High	Levee	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-2	Building on edge of floodplain at section 150400- south of Hausman	Fringe	1	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-3	Low water crossing entire - Babcock Road under water	Base	0	High	Bridge	35,000	11	10	350	30	10,500	$40	$420,000	2,100	1,750	6	26,500	$8	$212,000	5,900
Leon	LC-4	Private drive low water crossing just south of Bandera	Base	0	Low	Bridge	35,000	9	10	410	30	12,300	$40	$492,000	1,000	590	5	7,200	$8	$57,600	2,000
Leon	LC-5	Timber Creek Estates Subdivision 90+ houses in floodplain (Remove illegal dumping across Leon Creek from Timber Creek Estates: Old P-4)	Base	90	High	Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-6	Flooded road - Heath Lane	Base	0	Low	Raise Roadway			-	20	30	600	$40	$24,000	1,200	1,180	5	14,300	$8	$114,400	4,000
Leon	LC-7	Grissom Road - Bridge flooding	Base	0	Low	Bridge	43,634	18	10	280	75	21,000	$40	$840,000	700	420	19	45,000	$8	$360,000	3,600
Leon	LC-8	1 structure in floodplain and 4 on the edge just south of Grissom	Fringe	5	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-9	3 houses near floodplain in Pipers Meadow Subdivision -OUT	N/A	0	High	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-10	Ingram Road - Low water crossing	Base	0	High	Bridge	97,495	15	10	690	50	34,500	$40	$1,380,000	1,500	810	13	42,600	$8	$340,800	4,600
Leon	LC-11	Frontage on Loop 410 - 2 Retail structures on edge of floodplain (car dealerships)	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-12A	Culebra Road bridge causing problems	Base	0	High	Bridge	97,495	22	10	490	110	53,900	$40	$2,156,000	1,400	910	8	41,600	$8	$332,800	11,200
Leon	LC-12B	2 structures adjacent to Culebra Road bridge near floodplain - OUT	N/A	0	Moderate	Purchase	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-13	1 apartment near edge of floodplain - OUT	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-14	W. Commerce low water crossing	Base	0	Low	Bridge	97,495	24	10	460	60	27,600	$40	$1,104,000	2,500	2,040	16	154,800	$8	$1,238,400	13,700
Leon	LC-15A	Pinn Road low water crossing	Base	0	Low	Bridge	97,495	27	10	420	30	12,600	$40	$504,000	1,600	1,180	13	50,600	$8	$404,800	4,000
Leon	LC-15B	5 structures on edge of floodplain - OUT	N/A	0	Moderate	Verify FF/Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-15C	1 commercial structure in floodplain	Base	1	Moderate	Purchase	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-16A	3 Structures in floodplain	Base	3	Low	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-16B	8 Structures near floodplain - OUT	N/A	0	Low	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Leon	LC-17	Rodriguez Park in floodplain	Base	Public Park	High	Signs and Gates	No Bridge, Roadway, or Fill Improvements Required														
Helotes	HEL-1	Galm Road low water crossing	Base	0	Moderate	Bridge	30,780	12	10	290	40	11,600	$40	$464,000	650	360	2	1,800	$8	$14,400	1,700
Helotes	HEL-2A	Braun Road low water crossing	Base	0	Moderate	Bridge	30,780	18	10	210	30	6,300	$40	$252,000	1,000	790	4	7,300	$8	$58,400	2,700
Helotes	HEL-2B	2 structures near floodplain	Fringe	2	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Helotes	HEL-3A	Leslie Road inundated	Base	0	Low	Bridge	30,780	14	10	250	25	6,250	$40	$250,000	1,400	1,150	2	4,400	$8	$35,200	3,300
Helotes	HEL-3B	Leslie Road inundated	Base	0	Low	Bridge	30,780	15	10	240	25	6,000	$40	$240,000	1,400	1,160	3	7,000	$8	$56,000	3,300
Helotes	HEL-3C	Leslie Road inundated	Base	0	Low	Bridge	30,780	15	10	240	25	6,000	$40	$240,000	1,400	1,160	3	7,000	$8	$56,000	3,300
Helotes	HEL-3D	7 structures in floodplain	Base	7	Moderate	Purchase	No Bridge, Roadway, or Fill Improvements Required														
Helotes	HEL-4	FM 1604 overtopped - TxDOT project to mitigate	No	0	No	Bridge	TxDOT Funded														
Helotes	HEL-5	New Territories Subdivision; 48 houses near floodplain - OUT	N/A	0	High	Verify FF/Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Helotes	HEL-6	Split flow down Wood Trail	Base	0	High	Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-1	Galm Road low water crossing	Base	0	High	Bridge	21,779	6	10	380	60	22,800	$40	$912,000	2,400	2,020	10	82,400	$8	$659,200	3,500
Culebra	C-2	Galm Road low water crossing	Base	0	High	Bridge	11,837	5	10	250	60	15,000	$40	$600,000	650	400	4	5,500	$8	$44,000	2,700
Culebra	C-3A	2 structures near floodplain	Fringe	2	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-3B	1 structure near floodplain	Fringe	1	Low	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-4A	Stuebing low water crossing	Base	0	High	Bridge	28,301	11	10	200	40	8,000	$40	$320,000	1,000	800	3	6,200	$8	$49,600	3,600
Culebra	C-4B	1 structure in floodplain	Base	1	Moderate	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-5A	FM 1604 inundated - TxDOT project to mitigate	No	0	No	Bridge	TxDOT Funded														
Culebra	C-5B	Culebra inundated	Base	0	High	Raise Roadway				30	40	1,200	$40	$48,000	2,500	2,470	2	12,100	$8	$96,800	11,000
Culebra	C-5C	10 structures near floodplain	Fringe	10	High	Verify FF/Purchase	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-6A	Culebra bridge inundated	Base	0	High	Bridge	56,891	23	10	300	80	24,000	$40	$960,000	1,300	1,000	5	21,400	$8	$171,200	8,900
Culebra	C-6B	7 structures in floodplain, 3 structures OUT	Base	7	High	Verify FF/Purchase	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-7A	Easterling Road low water crossing; remove fill D.S. of low water crossing	Base	0	Low	Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-7B	2 structures near floodplain	Fringe	2	Moderate	Verify FF/Purchase	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-8A	Culebra Road in floodplain	Base	0	Moderate	Bridge	55,800	20	10	330	65	21,450	$40	$858,000	5,500	5,170	3	54,100	$8	$432,800	37,400
Culebra	C-8B	Timber Path in floodplain	Base	0	Low	Bridge	56,891	20	10	400	35	14,000	$40	$560,000	2,300	1,900	3	13,600	$8	$108,800	7,400
Culebra	C-8C	Old Grissom to New Grissom in floodplain	Base	0	Low	Bridge	56,891	20	10	600	25	15,000	$40	$600,000	2,000	1,400	7	24,000	$8	$192,000	3,900
Culebra	C-8D	7 structures near floodplain - OUT	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Culebra	C-8E	1 structure in floodplain	Base	1	Moderate	Purchase	No Bridge, Roadway, or Fill Improvements Required														
Huesta	HUE-1	Split flow - no project required	No	0	No	None	No Bridge, Roadway, or Fill Improvements Required														
Huesta	HUE-2	4 structures on edge of floodplain - OUT	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Huesta	HUE-3	Inadequate drainage structure under Hausman Rd	Base	0	High	Bridge	7,655	6	10	150	30	4,500	$40	$180,000	1,100	950	4	8,800	$8	$70,400	3,200
Huesta	HUE-4	Alley View Mobile Park 12 trailers in floodplain, 2 others OUT	Base	12	Moderate	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Huesta	HUE-5	Babcock low water crossing inundated	Base	0	High	Bridge	9,069	9	10	120	30	3,600	$40	$144,000	3,800	3,680	3	24,200	$8	$193,600	12,300
Maverick	M-1A	Babcock in floodplain	Base	0	High	Bridge	8,071	11	10	100	30	3,000	$40	$120,000	1,200	1,100	5	13,300	$8	$106,400	3,700
Maverick	M-1B	Babcock in floodplain	Base	0	High	Bridge	8,071	11	10	100	30	3,000	$40	$120,000	1,200	1,100	5	13,300	$8	$106,400	3,700
Maverick	M-1C	Babcock in floodplain	Base	0	High	Bridge	8,071	11	10	100	30	3,000	$40	$120,000	1,200	1,100	5	13,300	$8	$106,400	3,700
Maverick	M-1D	7 structures near floodplain	Fringe	7	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
Maverick	M-2	Babcock in floodplain	Base	0	High	Levee	No Bridge, Roadway, or Fill Improvements Required														
Maverick	M-3	Babcock too low	Base	0	High	Levee	No Bridge, Roadway, or Fill Improvements Required														
Maverick	M-4	Babcock too low; UTSA Blvd. drainage structure too small	Base	0	High	Bridge	9,708	10	10	120	40	4,800	$40	$192,000	2,100	1,980	2	9,700	$8	$77,600	8,900
Maverick	M-5	Hausman Rd. drainage structure too small (low water crossing)	Base	0	High	Bridge	9,483	9	10	130	30	3,900	$40	$156,000	800	670	3	4,500	$8	$36,000	2,300
Huebner	HB-1	DeZavala Road low water crossing	Base	0	High	Bridge	2,775	5	10	70	80	5,600	$40	$224,000	1,300	1,230	4	20,500	$8	$164,000	11,000
Huebner	HB-2	Section 90: 1 building in the floodplain	Base	1	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Huebner	HB-3	Section 84: 3 houses near floodplain, Babcock Place Subdivision - OUT	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Huebner	HB-4	Prue Road low water crossing	Base	0	High	Bridge	5,529	2	10	290	40	11,600	$40	$464,000	500	210	2	1,100	$8	$8,800	1,000
Huebner	HB-5A	Lockhill low water crossing	Base	0	Low	Bridge	5,529	4	10	150	30	4,500	$40	$180,000	900	750	4	6,900	$8	$55,200	2,600
Huebner	HB-5B	White Bonnet low water crossing	Base	0	Low	Bridge	5,529	4	10	150	30	4,500	$40	$180,000	900	750	4	6,900	$8	$55,200	2,600
Huebner	HB-5C	1 building in the floodplain, 1 other OUT	Base	1	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														

CREEK	NUMBER	PROJECT	PROJECT TYPE	NO. OF STRUCTURES	PRIORITY	TYPE OF SOLUTION	Bridge/Roadway Discharge (c.f.s.)	Depth (ft.)	Assumed Velocity (f.p.s.)	Required Length (l.f.)	Road Width (ft.)	Bridge Area (sq. ft.)	Unit Cost	Bridge Cost	Floodplain Width (ft.)	Length of Approach or Roadway (l.f.)	Fill Depth (ft.)	Fill Volume (c.y.)	Fill Unit Cost	Fill Cost	Paving (s.y.)
Huebner	HB-5D	4 buildings in the floodplain	Base	4	Moderate	Purchase	No Bridge, Roadway, or Fill Improvements Required														
Huebner	HB-6A	Hollyhock low water crossing (currently under design - 1994 bonds) Bridge cost shown is the additional amount required to bring bridge up to a 100 year design	Base	0	High	Bridge	5,529	5	10	130	40	5,200	$40	$208,000	1,200	1,070	5	14,900	$8	$119,200	4,8
Huebner	HB-6B	5 structures near floodplain in Wellesly Manor Subdivision - OUT	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Huebner	HB 6C	5 structures in floodplain, 4 others OUT	Base	5	Moderate	Verify FF/Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Huebner	HB-7A	Whitley low water crossing(currently under design - 1994 bonds) Bridge cost shown is the additional amount required to bring the bridge up to a 100 year design	Base	0	High	Bridge	5,529	4	10	150	30	4,500	$40	$180,000	500	350	4	3,300	$8	$26,400	1,2
Huebner	HB-7B	1 structure near edge of floodplain - OUT	N/A	0	Moderate	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
Huebner	HB 8	Eckert Road low water crossing	Base	0	High	Bridge	5,529	2	10	290	35	10,150	$40	$406,000	600	310	4	3,100	$8	$24,800	1,3
Huebner	HB-9A	Leon Valley in floodplain - approximately 167 structures flooded	Base	167	Low	Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Huebner	HB 9B	9 houses near floodplain Win Creek Subdivision - OUT	N/A	0	Moderate	Verify FF/Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
Huebner	HB-10	Timberhill low water crossing	Base	0	High	Bridge	17,262	6	10	310	60	18,600	$40	$744,000	900	590	6	12,900	$8	$103,200	4,0
French	F-1	2 structures in floodplain, 13 others OUT or N/A	Base	2	Low	Verify FF/Channel Improvements	No Bridge, Roadway, or Fill Improvements Required														
French	F 2A	Hausman Road low water crossing	Base	0	Moderate	Bridge	13,112	3	10	450	30	13,500	$40	$540,000	900	450	3	3,000	$8	$24,000	1,6
French	F-2B	7 structures near floodplain - OUT	N/A	0	Low	Verify FF/Purchase	No Bridge, Roadway, or Fill Improvements Required														
French	F-2C	4 structures near floodplain - OUT	N/A	0	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
French	F-3	Prue Road overtopped	Base	0	High	Bridge	15,857	8	10	220	50	11,000	$40	$440,000	600	380	3	3,400	$8	$27,200	2,2
French	F-4A	N Verde low water crossing	Base	0	Low	Bridge	15,857	6	10	280	50	14,000	$40	$560,000	800	520	3	4,600	$8	$36,800	2,9
French	F-4B	S Verde low water crossing	Base	0	Low	Bridge	15,857	5	10	330	50	16,500	$40	$660,000	900	570	2	3,300	$8	$26,400	3,2
French	F-4C	11 structures in floodplain	Base	11	Low	Verify FF/Purchase	No Bridge, Roadway, or Fill Improvements Required														
French	F 5A	Inadequate drainage structure at Bandera Rd	Base	0	Low	Bridge	15,857	8	10	220	100	22,000	$40	$880,000	2,300	2,080	3	29,900	$8	$239,200	23,
French	F 5B	3 structures in floodplain, 2 others OUT	Base	3	Low	Verify FF/Levee	No Bridge, Roadway, or Fill Improvements Required														
French	F 5C	1 structures in floodplain, 5 others OUT	Base	1	Low	Verify FF/Floodwall	No Bridge, Roadway, or Fill Improvements Required														
French	F 5D	1 structures in floodplain, 3 others OUT	Base	1	Low	Verify FF/Purchase	No Bridge, Roadway, or Fill Improvements Required														
French	F 6	Inadequate drainage structures at Guilbeau	Base	0	Moderate	Bridge	15,857	9	10	200	50	10,000	$40	$400,000	1,000	800	3	7,100	$8	$56,800	4,5
French	F-7	Wildwood Subdivision 9 houses near floodplain - OUT	N/A	0	Moderate	Verify FF/Levee															
French	F 8A	Inadequate drainage structure at Mainland	Base	0	Low	Bridge	16,696	7	See Remarks	70	65	4,550	$40	$182,000	600	350	2	2,400	$8	$19,200	2,6
French	F 8B	2 structures near floodplain - No mitigation req'd if bridge is improved	No	0	No	Verify FF/Out with Bridge	No Bridge, Roadway, or Fill Improvements Required														
French	F 8C	2 structures near floodplain - OUT	N/A	0	Moderate	Verify FF/Purchase	No Bridge, Roadway, or Fill Improvements Required														
French	F-9A	Low Bid Lane in floodplain	Base	0	Low	Bridge	1,951	6	10	50	30	1,500	$40	$60,000	850	800	2	3,400	$8	$27,200	2,7
French	F 9B	Heath in floodplain	Base	0	Low	Raise Roadway				25	30	750	$40	$30,000	350	325	2	1,400	$8	$11,200	1,1
French	F-9C	Clyde Dent Drive in floodplain	Base	0	Low	Bridge	1,957	12	10	50	30	1,500	$40	$60,000	600	550	4	5,100	$8	$40,800	1,9
Leon	P 1	Babcock Road detention, Retention & park at Huesta Maverick & Leon Creek	Special	0	High	Detention Pond															
Leon	P 2	Detention & park south of Prue, gravel pit reclamation	Special	0	High	Detention Pond															
Leon	P-3	Detention & park in French Creek Village area, gravel pit reclamation	Special	0	High	Detention Pond															
Leon	P 5	Detention, park & gravel pit reclamation at Culebra and Leon Creek behind Twin Creek Subdivision	Special	0	High	Detention Pond															
Leon	P 6	Detention, park & gravel pit reclamation at Southwest Research	Special	0	High	Detention Pond															
Helotes	P 7	Vulcan Quarry detention, Retention & park site	Potential Spec	0	Low	Retention Pond															
Culebra	P 9	Government Canyon Retention Pond	Potential Spec	0	High	Retention Pond															
Culebra	P-10	Culebra Retention Pond	Potential Spec.	0	High	Retention Pond															
							Bridge Notes: Required Length, Fill Volume, and Paving Area were rounded up to the next even 10 foot, 100 cubic yard, and 100 square yard interval, respectively. The Total Cost was rounded up to the next even $1,000														
		Base Projects	**70**	**318**	**Structures included in Base Projects (167 in Leon Valley)**															**46**	**Ba**
		Fringe Projects	**8**	**30**	**Structures included in Fringe Projects**															**0**	**F**
		Special Projects	**5**	**0**	**Structures included in Special Projects (Development of Special projects may reduce the number and/or magnitude of Base projects)**															**0**	**S**
		Total Projects (excluding Special Projects)	**78**	**348**	**Structures included in Total Projects**															**46**	**To**

(1) $400,000 of this
Two TxDOT fun

Paving Unit Cost	Paving Cost	Total Cost	Floodwall: Length (l.f.)	Floodwall: Conc. (c.y.)	Floodwall: Unit Cost	Floodwall: Total Cost	Levee: Length (l.f.)	Levee: Fill (c.y.)	Levee: Unit Cost	Levee: Stabilize (ac.)	Levee: Unit Cost	Levee: Total Cost	Channel Improvements: Length (l.f.)	Channel Improvements: Excavation (c.y.)	Channel Improvements: Unit Cost	Channel Improvements: Disposal (c.y.)	Channel Improvements: Unit Cost	Channel Improvements: Riprap (s.y.)	Channel Improvements: Unit Cost	Channel Improvements: Total Cost	Purchase Structures: No. Units	Purchase Structures: Total Area (s.f.)	Purchase Structures: Unit Cost	Purchase Structures: Total Cost
			Floodplain not steep enough to provide drainage behind floodwall				Floodplain not steep enough to provide drainage behind levee						Existing floodplain is flat, channel improvements would have to lower flood elevations by 3' - 4'								4	5,640	$75	$423,000
$20	$96,000	$424,000	No Structures				No Structures						No Structures								No Structures			
			Finished floors surveyed -- NOT IN FLOODPLAIN				Side slope too steep for levee construction						Channel Improvements not economical								Structures can be protected			
			Floodplain not steep enough to provide drainage behind floodwall				Floodplain not steep enough to provide drainage behind levee						3,000	30,000	$8	30,000	$3	30,000	$30	$1,230,000	Structure can be protected			
$20	$24,000	$231,000	No Structures				No Structures						No Structures								No Structures			
			Finished floors surveyed -- NOT IN FLOODPLAIN				Side slope too steep for levee construction						Channel Improvements not economical								Structure can be protected			
$20	$26,000	$457,000	No Structures				No Structures						No Structures								No Structures			
			Floodwall not feasible				Levee not feasible						12,000	322,000	$8	322,000	$3	231,000	$30	$10,472,000	Most structures can be protected			
			Floodplain not steep enough to provide drainage behind floodwall				Floodplain not steep enough to provide drainage behind levee						Finished floors surveyed -- NOT IN FLOODPLAIN								Structures can be protected			
$20	$80,000	$928,000	No Structures				No Structures						No Structures								No Structures			
			Floodwall not possible, blocks drainage from stream confluence				Levee not possible, blocks drainage from stream confluence						2,500	65,000	$8	65,000	$3	3,000	$30	$805,000	Structures can be protected			
$20	$32,000	$597,000	No Structures				No Structures						No Structures								No Structures			
			Floodwall not possible, blocks drainage from stream confluence				Levee not possible, blocks drainage from stream confluence						Channel Improvements not economical								Finished floors surveyed -- NOT IN FLOODPLAIN			
			Floodwall not economical when levee possible				Finished floors surveyed -- NOT IN FLOODPLAIN						Channel Improvements not economical								Structures can be protected			
$20	$44,000	$512,000	No Structures				No Structures						No Structures								No Structures			
$20	$58,000	$655,000	No Structures				No Structures						No Structures								No Structures			
$20	$64,000	$751,000	No Structures				No Structures						No Structures								No Structures			
			Floodplain not steep enough to provide drainage behind floodwall				Floodplain not steep enough to provide drainage behind levee						Channel Improvements not economical								11	16,000	$75	$1,200,000
$20	$464,000	$1,584,000	No Structures				No Structures						No Structures								No Structures			
			Floodwall not economical when levee possible				1,600	15,600	$10	10	$5,500	$211,000	Channel Improvements not economical								Structure can be protected			
			1,200	820	$400	$328,000	Side slope too steep for levee construction						Channel Improvements not economical								Structure can be protected			
			Floodplain not steep enough to provide drainage behind floodwall				Floodplain not steep enough to provide drainage behind levee						Channel Improvements not economical								1	2,600	$75	$195,000
$20	$90,000	$547,000	No Structures				No Structures						No Structures								No Structures			
			Floodwall not economical when levee possible				Finished floors surveyed -- NOT IN FLOODPLAIN						Channel Improvements not economical								Structure can be protected			
$20	$52,000	$254,000	No Structures				No Structures						No Structures								No Structures			
			Mainland Drive would obstruct drainage behind floodwall				Mainland Drive would obstruct drainage behind levee						Channel Improvements not required if bridge improved								Structures can be protected			
			Floodplain not steep enough to provide drainage behind floodwall				Floodplain not steep enough to provide drainage behind levee						Channel Improvements not economical								Finished floors surveyed -- NOT IN FLOODPLAIN			
$20	$54,000	$142,000	No Structures				No Structures						No Structures								No Structures			
$20	$22,000	$64,000	No Structures				No Structures						No Structures								No Structures			
$20	$38,000	$139,000	No Structures				No Structures						No Structures								No Structures			

dge/Roadway Projects (1)	$32,758,000	4	Base Floodwall Projects	$1,320,000	6	Base Levee Projects	$427,000	6	Base Channel Projects (2)	$18,390,000	7	Base Purchase Projects	$4,593,000
ridge/Roadway Projects	$0	1	Fringe Floodwall Projects	$152,000	5	Fringe Levee Projects	$205,000	0	Fringe Channel Projects	$0	2	Fringe Purchase Projects	$1,185,000
Bridge/Roadway Projects	$0	0	Special Floodwall Projects	$0	0	Special Levee Projects	$0	0	Special Channel Projects	$0	0	Special Purchase Projects	$0
idge/Roadway Projects (1)	$32,758,000	5	Total Floodwall Projects	$1,472,000	11	Total Levee Projects	$632,000	6	Total Channel Projects (2)	$18,390,000	9	Total Purchase Projects	$5,778,000

een funded already on projects HB-8A and HB-7A
rojects (HEL-4 and C-5A) not included in this total

(2) $1,760,000 of this should be property owner funded (project LC-5),
$143,000 of this should be property owner funded (project C-7A), and
$10,472,000 of this is within the city limits of Leon Valley (project HB-9A)

Area (ac.)	Unit Cost	Excavation (c.y.)	Unit Cost	Disposal (c.y.)	Unit Cost	Fill (c.y.)	Unit Cost	Riprap (c.y.)	Unit Cost	Misc. Cost (Pipe, Seeding)	Total Cost	Signs and Gates Total Cost	Total Project Cost	Remarks	Benefits	Benefit/Cost Ratio
Detention Pond Improvements																
													$423,000		$600,000	1.4
													$424,000	Approximately $400,000 is already available for projects HB-6A and HB-7A (1994 bonds).	$1,500,000	3.5
													$0			
													$1,230,000	A concrete lined channel is required due to width constraints.	$1,500,000	1.2
													$231,000	Approximately $400,000 is already available for projects HB-6A and HB 7A (1994 bonds).	$1,000,000	4.3
													$0			
													$457,000		$1,500,000	3.3
													$10,472,000	City of Leon Valley project. Some structures will be lost to channel improvements	$35,000,000	3.3
													$0			
													$928,000		$2,000,000	2.2
													$805,000		$1,500,000	1.9
													$597,000		$1,000,000	1.7
													$0			
													$0			
													$512,000		$1,000,000	2.0
													$655,000		$1,000,000	1.5
													$751,000		$1,000,000	1.3
													$1,200,000		$1,700,000	1.4
													$1,584,000		$2,000,000	1.3
													$211,000	Note that the levee may excessively encroach the floodplain	$1,500,000	7.1
													$328,000		$1,000,000	3.0
													$195,000		$600,000	3.1
													$547,000		$1,500,000	2.7
													$0			
													$254,000	Lengthened bridge (not replaced)	$1,500,000	5.9
													$0	F-8A improvements should lower W.S.E. sufficiently to remove building from floodplain, if not above flood elevation already.		
													$0			
													$142,000		$1,000,000	7.0
													$64,000	Raise roadway (culvert size estimated at 25 feet)	$1,000,000	15.6
													$139,000		$1,000,000	7.2
150	$2,000	350,000	$6	295,000	$3	55,000	$3	60,000	$30	$1,000,000	$6,250,000		$6,250,000	Reduce by 500 cfs (Total benefit of all five detention ponds is approximately $3,238,000)	$647,600	0.1
25	$2,000	355,000	$6	300,500	$3	54,500	$3	5,600	$30	$56,000	$3,469,000		$3,469,000	Reduce by 2,000 cfs (Total benefit of all five detention ponds is approximately $3,238,000)	$647,600	0.2
30	$2,000	120,000	$6	20,000	$3	100,000	$3	5,600	$30	$46,000	$1,334,000		$1,334,000	Reduce by 2,000 cfs (Total benefit of all five detention ponds is approximately $3,238,000)	$647,600	0.5
35	$2,000	175,000	$6	500	$3	174,500	$3	5,600	$30	$62,000	$1,855,000		$1,855,000	Reduce by 4,000 cfs (Total benefit of all five detention ponds is approximately $3,238,000)	$647,600	0.3
170	$2,000	1,260,000	$6	930,000	$3	330,000	$3	11,000	$30	$220,000	$12,230,000		$12,230,000	Reduce by 15,000 cfs (Total benefit of all five detention ponds is approximately $3,238,000)	$647,600	0.1
														No costs were estimated for Retention Ponds. No benefit from this project. Volume is too small. Pond fills before peak arrives	$0	
														No costs were estimated for Retention Ponds. Total benefit of both Retention ponds (P-9 and P-10) is approximately $5,465,000.	$2,732,500	
														No costs were estimated for Retention Ponds. Total benefit of both Retention ponds (P-9 and P-10) is approximately $5,465,000.	$2,732,500	
							0	Base Detention Pond Projects			$0	$50,000	$57,538,000	Base Total Projects (1)(2)(3)	70	
							0	Fringe Detention Pond Projects			$0	$0	$1,542,000	Fringe Total Projects	8	
							5	Special Detention Pond Projects			$25,138,000	$0	$25,138,000	Special Total Projects	5	
							0	Total Det. Ponds(excluding special projects)			$0	$50,000	$59,080,000	Total Projects (1)(3)(4) (excluding special projects)	78	

(3)Project LC-17 (Signs and Gates) included as a base project

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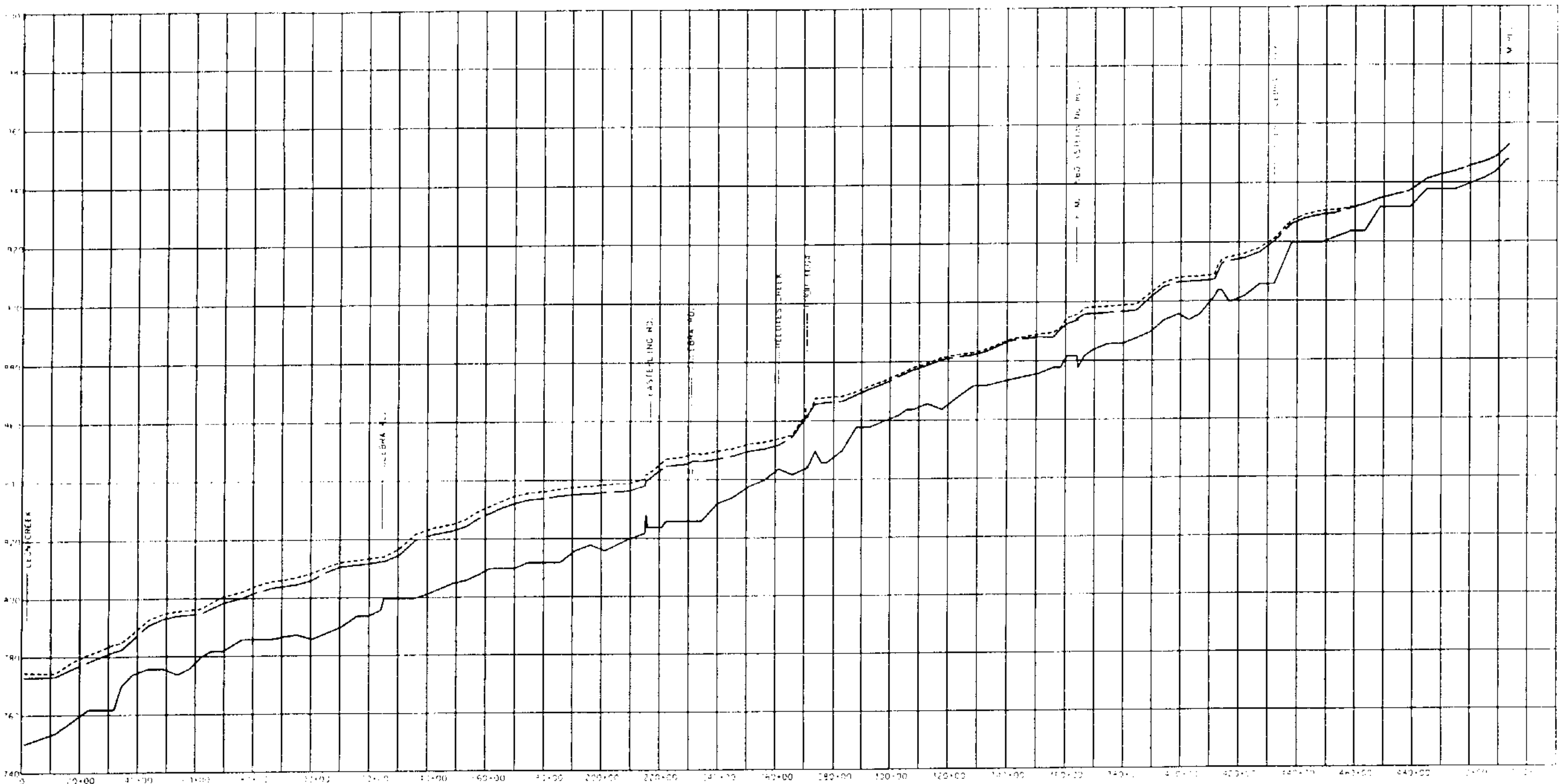
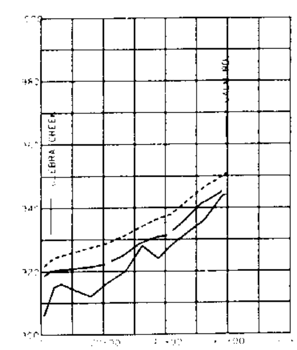


PROFILE
SCALE: HORIZ. 1" = 2000'
SCALE: VERT. 1" = 20'

CULEBRA CREEK
100 YEAR FLOOD

LEGEND



UPPER CULEBRA CREEK
100 YEAR FLOOD

PRELIMINARY - FOR REVIEW ONLY
THESE DOCUMENTS ARE FOR DESIGN REVIEW AND NOT INTENTED FOR CONSTRUCTION, BIDDING OR PERMIT PURPOSES. THEY WERE PREPARED BY, OR UNDER SUPERVISION OF:

JAMES P. AMICK	65295	3/96
Type or Print Name	PE #	Date

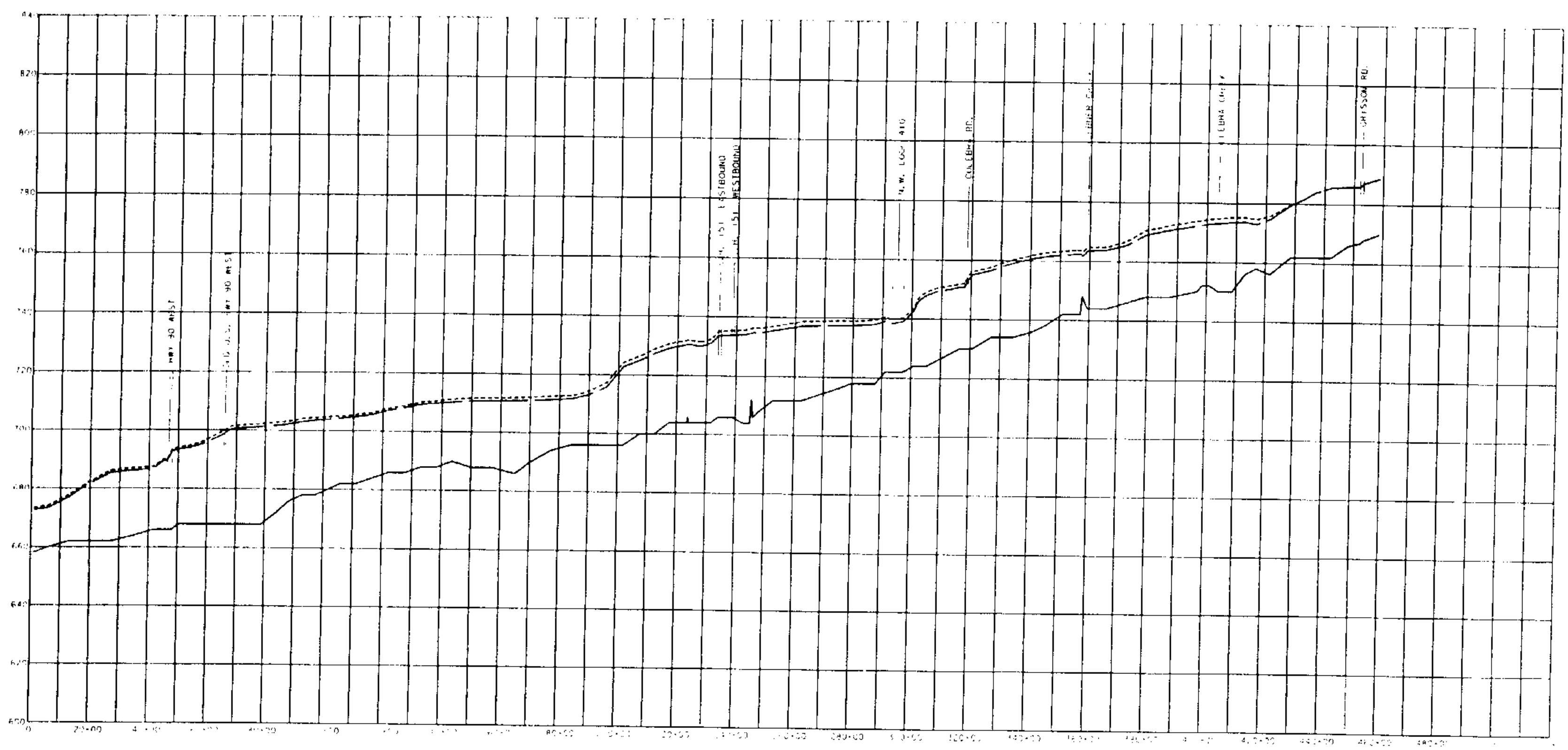
CULEBRA CREEK
AND
UPPER CULEBRA CREEK

RETENTION POND FLOOD PROFILES

HNTB
HNTB CORPORATION
ARCHITECTS ENGINEERS PLANNERS
(817) 870-1808 512 MAIN STREET, SUITE 711
FORT WORTH, TEXAS 76102

SUB. CONSULTANT ENGINEER		REC. CHIEF DESIGN ENGINEER	
REV. ENGINEER		FILE	DATE	SHEET
DESIGNED: J. P. A. CHECKED:	DRAWN: S. M. D. CHECKED:	X-####	3/1/96	

Filename: \21749\cadd\ccprof.dgn



PROFILE
SCALE HORIZ 1" = 2000'
SCALE VERT 1" = 20'

LEON CREEK

100 YEAR FLOOD

LEGEND

CHANNEL INVERT

COMPUTED WATER SURFACE

COMPUTED WATER SURFACE

PRELIMINARY - FOR REVIEW ONLY
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JAMES P. AMICK	65295	3/96
Type or Print Name	PE #	Date

LEON CREEK

DETENTION POND FLOOD PROFILES

HNTB

HNTB CORPORATION
ARCHITECTS ENGINEERS PLANNERS
(817) 870-1808 512 MAIN STREET, SUITE 717
FORT WORTH, TEXAS 76102

SUB. CONSULTANT ENGINEER
REC. CHIEF DESIGN ENGINEER
REV. ENGINEER

DESIGNED: J. P. A.	DRAWN: S. M. D.	FILE	DATE	SHEET
CHECKED:	CHECKED:	X-####	3-?/96	

Filename: \21749\cadd\lcprof.dgn

TABLE 3

LEON CREEK WATERSHED
MITIGATION COSTS BY ADMINISTRATION

(IN MILLIONS OF DOLLARS)

ADMINISTRATION	BASE PROJECTS		FRINGE* PROJECTS		SPECIAL PROJECTS	
	NUMBER	COSTS	NUMBER	COSTS	NUMBER	COSTS
CITY OF SAN ANTONIO DRAINAGE PROJECTS (levees, channels, flood walls, buyouts)	23	$ 14.2	8	$ 1.5	0	$ -
CITY OF SAN ANTONIO MULTI-USE/BENEFIT PROJECTS (detention ponds)	0	$ -	0	$ -	5	$ 25.1
ROAD & BRIDGE PROJECTS	46	$ 32.8	0	$ -	0	$ -
OTHER CITIES / MUNICIPALITIES (Leon Valley)	1	$ 10.5	0	$ -	0	$ -
TOTALS	70	$ 57.5	8	$ 1.5	5	$ 25.1

* Fringe projects include those projects near the edge of the flood plain which require detailed survey information to determine if they in fact are affected by the 100-yr event.

HNTB - TEXAS
e:\21749\admin\costdis.xls

Table 4.1a - Leon Creek High Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
LC-1	Levee	$26,000	$1,000,000	38.5
LC-3	Bridge	$751,000	$1,000,000	1.3
LC-5	Channel Improvements	$4,340,000	$10,000,000	2.3
LC-10	Bridge	$1,813,000	$2,000,000	1.1
LC-12A	Bridge	$2,713,000	$2,000,000	0.7
LC-17	Signs and Gates	$50,000	$1,000,000	20.0
Total		**$9,693,000**		

Note that $1,760,000 for project LC-5 should be property owner funded.

Table 4.1b - Leon Creek Moderate Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
LC-15C	Purchase	$240,000	$1,120,000	4.7
Total		**$240,000**		

Table 4.1c - Leon Creek Low Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
LC-4	Bridge	$590,000	$1,000,000	1.7
LC-6	Raise Roadway	$219,000	$1,000,000	4.6
LC-7	Bridge	$1,273,000	$2,000,000	1.6
LC-14	Bridge	$2,617,000	$1,500,000	0.6
LC-15A	Bridge	$989,000	$1,500,000	1.5
LC-16A	Verify FF/Floodwall	$720,000	$200,000	0.3
Total		**$6,408,000**		

Table 4.2a - Helotes Creek High Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
HEL-6	Channel Improvements	$1,400,000	$10,000,000	7.1
Total		**$1,400,000**		

Table 4.2b - Helotes Creek Moderate Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
HEL-1	Bridge	$513,000	$1,000,000	1.9
HEL-2A	Bridge	$365,000	$1,000,000	2.7
HEL-3D	Purchase	$1,260,000	$1,500,000	1.2
Total		**$2,138,000**		

Table 4.2c - Helotes Creek Low Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
HEL-3A	Bridge	$352,000	$1,000,000	2.8
HEL-3B	Bridge	$363,000	$1,000,000	2.8
HEL-3C	Bridge	$363,000	$1,000,000	2.8
Total		**$1,078,000**		

Table 4.3a - Culebra Creek High Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
C-1	Bridge	$1,842,000	$2,000,000	1.1
C-2	Bridge	$699,000	$2,000,000	2.9
C-4A	Bridge	$442,000	$1,500,000	3.4
C-5B	Raise Roadway	$365,000	$1,500,000	4.1
C-6A	Bridge	$1,310,000	$2,000,000	1.5
C-6B	Verify FF/Purchase	$1,155,000	$1,500,000	1.3
Total		**$5,813,000**		

Table 4.3b - Culebra Creek Moderate Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
C-4B	Verify FF/Levee	$26,000	$250,000	9.6
C-8A	Bridge	$2,039,000	$1,500,000	0.7
C-8E	Purchase	$120,000	$150,000	1.3
Total		**$2,185,000**		

Table 4.3c - Culebra Creek Low Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
C-7A	Channel Improvements	$143,000	$250,000	1.7
C-8B	Bridge	$817,000	$1,500,000	1.8
C-8C	Bridge	$871,000	$1,500,000	1.7
Total		**$1,831,000**		

Note that $143,000 for project C-7A should be property owner funded.

Table 4.4a - Huesta Creek High Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
HUE-3	Bridge	$315,000	$1,500,000	4.8
HUE-5	Bridge	$584,000	$1,500,000	2.6
Total		**$899,000**		

Table 4.4b - Huesta Creek Moderate Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
HUE-4	Verify FF/Levee	$36,000	$1,000,000	27.8
Total		**$36,000**		

Table 4.4c - Huesta Creek Low Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		**$0**		

Table 4.5a - Maverick Creek High Priority Projects
(Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
M-1A	Bridge	$301,000	$1,000,000	3.3
M-1B	Bridge	$301,000	$1,000,000	3.3
M-1C	Bridge	$301,000	$1,000,000	3.3
M-2	Levee	$92,000	$1,500,000	16.3
M-3	Levee	$36,000	$1,500,000	41.7
M-4	Bridge	$448,000	$1,500,000	3.3
M-5	Bridge	$239,000	$1,500,000	6.3
Total		**$1,718,000**		

Table 4.5b - Maverick Creek Moderate Priority Projects
(Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		**$0**		

Table 4.5c - Maverick Creek Low Priority Projects
(Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		**$0**		

Table 4.6a - Huebner Creek High Priority Projects
(Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
HB-1	Bridge	$609,000	$1,000,000	1.6
HB-4	Bridge	$493,000	$1,000,000	2.0
HB-6A	Bridge	$424,000	$1,500,000	3.5
HB-7A	Bridge	$231,000	$1,000,000	4.3
HB-8	Bridge	$457,000	$1,500,000	3.3
HB-10	Bridge	$928,000	$2,000,000	2.2
Total		**$3,142,000**		

Note that $400,000 has already been funded on projects HB-6A and HB-7A.

Table 4.6b - Huebner Creek Moderate Priority Projects
(Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
HB-2	Verify FF/Floodwall	$100,000	$2,000,000	20.0
HB-5C	Verify FF/Floodwall	$172,000	$250,000	1.5
HB-5D	Purchase	$423,000	$600,000	1.4
HB-6C	Verify FF/Channel Improvements	$1,230,000	$1,500,000	1.2
Total		**$1,925,000**		

Table 4.6c - Huebner Creek Low Priority Projects
(Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
HB-5A	Bridge	$288,000	$1,000,000	3.5
HB-5B	Bridge	$288,000	$1,000,000	3.5
HB-9A	Channel Improvements	$10,472,000	$35,000,000	3.3
Total		**$11,048,000**		

Note that $10,472,000 for project HB-9A should be Leon Valley funded.

Table 4.7a - French Creek High Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
F-3	Bridge	$512,000	$1,000,000	2.0
Total		**$512,000**		

Table 4.7b - French Creek Moderate Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
F-2A	Bridge	$597,000	$1,000,000	1.7
F-6	Bridge	$547,000	$1,500,000	2.7
Total		**$1,144,000**		

Table 4.7c - French Creek Low Priority Projects (Base Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
F-1	Verify FF/Channel Improvements	$805,000	$1,500,000	1.9
F-4A	Bridge	$655,000	$1,000,000	1.5
F-4B	Bridge	$751,000	$1,000,000	1.3
F-4C	Verify FF/Purchase	$1,200,000	$1,700,000	1.4
F-5A	Bridge	$1,584,000	$2,000,000	1.3
F-5B	Verify FF/Levee	$211,000	$1,500,000	7.1
F-5C	Verify FF/Floodwall	$328,000	$1,000,000	3.0
F-5D	Verify FF/Purchase	$195,000	$600,000	3.1
F-8A	Bridge	$254,000	$1,500,000	5.9
F-9A	Bridge	$142,000	$1,000,000	7.0
F-9B	Raise Roadway	$64,000	$1,000,000	15.6
F-9C	Bridge	$139,000	$1,000,000	7.2
Total		**$6,328,000**		

Table 5.1a - Leon Creek High Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		**$0**		

Table 5.1b - Leon Creek Moderate Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		**$0**		

Table 5.1c - Leon Creek Low Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
LC-2	Verify FF/Levee	$31,000	$250,000	8.1
LC-8	Verify FF/Levee	$26,000	$750,000	28.8
Total		**$57,000**		

Table 5.2a - Helotes Creek High Priority Projects (Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		**$0**		

Table 5.2b - Helotes Creek Moderate Priority Projects (Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		**$0**		

Table 5.2c - Helotes Creek Low Priority Projects (Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
HEL-2B	Verify FF/Levee	$36,000	$550,000	15.3
Total		**$36,000**		

Table 5.3a - Culebra Creek High Priority Projects (Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
C-5C	Verify FF/Purchase	$975,000	$1,500,000	1.5
Total		**$975,000**		

Table 5.3b - Culebra Creek Moderate Priority Projects (Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
C-7B	Verify FF/Purchase	$210,000	$200,000	1.0
Total		**$210,000**		

Table 5.3c - Culebra Creek Low Priority Projects (Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
C-3A	Verify FF/Levee	$56,000	$200,000	3.6
C-3B	Verify FF/Floodwall	$152,000	$150,000	1.0
Total		**$208,000**		

Table 5.4a - Huesta Creek High Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		$0		

Table 5.4b - Huesta Creek Moderate Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		$0		

Table 5.4c - Huesta Creek Low Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		$0		

Table 5.5a - Maverick Creek High Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		**$0**		

Table 5.5b - Maverick Creek Moderate Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		**$0**		

Table 5.5c - Maverick Creek Low Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
M-1D	Verify FF/Levee	$56,000	$900,000	16.1
Total		**$56,000**		

Table 5.6a - Huebner Creek High Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		$0		

Table 5.6b - Huebner Creek Moderate Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		$0		

Table 5.6c - Huebner Creek Low Priority Projects
(Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		$0		

Table 5.7a - French Creek High Priority Projects (Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		$0		

Table 5.7b - French Creek Moderate Priority Projects (Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		$0		

Table 5.7c - French Creek Low Priority Projects (Fringe Projects Only)

Project Number	Type	Cost	Benefit	Benefit/Cost Ratio
Total		$0		

Table 6.1 - Summary by Stream of High Priority Projects

Stream	Cost (Base Projects Only)	Cost (Fringe Projects Only)	Total Cost
Leon Creek	$9,693,000	$0	$9,693,000
Helotes Creek	$1,400,000	$0	$1,400,000
Culebra Creek	$5,813,000	$975,000	$6,788,000
Huesta Creek	$899,000	$0	$899,000
Maverick Creek	$1,718,000	$0	$1,718,000
Huebner Creek	$3,142,000	$0	$3,142,000
French Creek	$512,000	$0	$512,000
Total	**$23,177,000**	**$975,000**	**$24,152,000**

Note that $4,340,000 for a project on Leon Creek should be property owner funded, and that $400,000 has already been funded for two projects on Huebner Creek. These three projects are base projects.

Table 6.2 - Summary by Stream of Moderate Priority Projects

Stream	Cost (Base Projects Only)	Cost (Fringe Projects Only)	Total Cost
Leon Creek	$240,000	$0	$240,000
Helotes Creek	$2,138,000	$0	$2,138,000
Culebra Creek	$2,185,000	$210,000	$2,395,000
Huesta Creek	$36,000	$0	$36,000
Maverick Creek	$0	$0	$0
Huebner Creek	$1,925,000	$0	$1,925,000
French Creek	$1,144,000	$0	$1,144,000
Total	**$7,668,000**	**$210,000**	**$7,878,000**

Table 6.3 - Summary by Stream of Low Priority Projects

Stream	Cost (Base Projects Only)	Cost (Fringe Projects Only)	Total Cost
Leon Creek	$6,408,000	$57,000	$6,465,000
Helotes Creek	$1,078,000	$36,000	$1,114,000
Culebra Creek	$1,831,000	$208,000	$2,039,000
Huesta Creek	$0	$0	$0
Maverick Creek	$0	$56,000	$56,000
Huebner Creek	$11,048,000	$0	$11,048,000
French Creek	$6,328,000	$0	$6,328,000
Total	**$26,693,000**	**$357,000**	**$27,050,000**

Note that $143,000 for a project on Culebra Creek should be property owner funded, and that $10,472,000 for a project on Huebner Creek should be Leon Valley funded. Both of these projects are base projects.

Table 7.1 - Cost Summary by Streams and Priority
(Base Projects Only)

Stream	High	Moderate	Low	Total
Leon Creek	$9,693,000	$240,000	$6,408,000	$16,341,000
Helotes Creek	$1,400,000	$2,138,000	$1,078,000	$4,616,000
Culebra Creek	$5,813,000	$2,185,000	$1,831,000	$9,829,000
Huesta Creek	$899,000	$36,000	$0	$935,000
Maverick Cree	$1,718,000	$0	$0	$1,718,000
Huebner Creek	$3,142,000	$1,925,000	$11,048,000	$16,115,000
French Creek	$512,000	$1,144,000	$6,328,000	$7,984,000
Total	$23,177,000	$7,668,000	$26,693,000	$57,538,000

Note that $4,340,000 for a project on Leon Creek and $143,000 for a project o Culebra Creek should be property owner funded, that $400,000 has already been funded for two projects on Huebner Creek, and that $10,472,000 for a project on Huebner Creek should be Leon Valley funded.

Table 7.2 - Cost Summary by Streams and Priority
(Fringe Projects Only)

Stream	High	Moderate	Low	Total
Leon Creek	$0	$0	$57,000	$57,000
Helotes Creek	$0	$0	$36,000	$36,000
Culebra Creek	$975,000	$210,000	$208,000	$1,393,000
Huesta Creek	$0	$0	$0	$0
Maverick Cree	$0	$0	$56,000	$56,000
Huebner Creek	$0	$0	$0	$0
French Creek	$0	$0	$0	$0
Total	$975,000	$210,000	$357,000	$1,542,000

Table 7.3 - Cost Summary by Streams and Priority

Stream	High	Moderate	Low	Total
Leon Creek	$9,693,000	$240,000	$6,465,000	$16,398,000
Helotes Creek	$1,400,000	$2,138,000	$1,114,000	$4,652,000
Culebra Creek	$6,788,000	$2,395,000	$2,039,000	$11,222,000
Huesta Creek	$899,000	$36,000	$0	$935,000
Maverick Cree	$1,718,000	$0	$56,000	$1,774,000
Huebner Creek	$3,142,000	$1,925,000	$11,048,000	$16,115,000
French Creek	$512,000	$1,144,000	$6,328,000	$7,984,000
Total	$24,152,000	$7,878,000	$27,050,000	$59,080,000

Note that $4,340,000 for a project on Leon Creek and $143,000 for a project o Culebra Creek should be property owner funded, that $400,000 has already been funded for two projects on Huebner Creek, and that $10,472,000 for a project on Huebner Creek should be Leon Valley funded.

Figure 7.1 - Cost Summary by Stream and Priority (Base Projects Only)

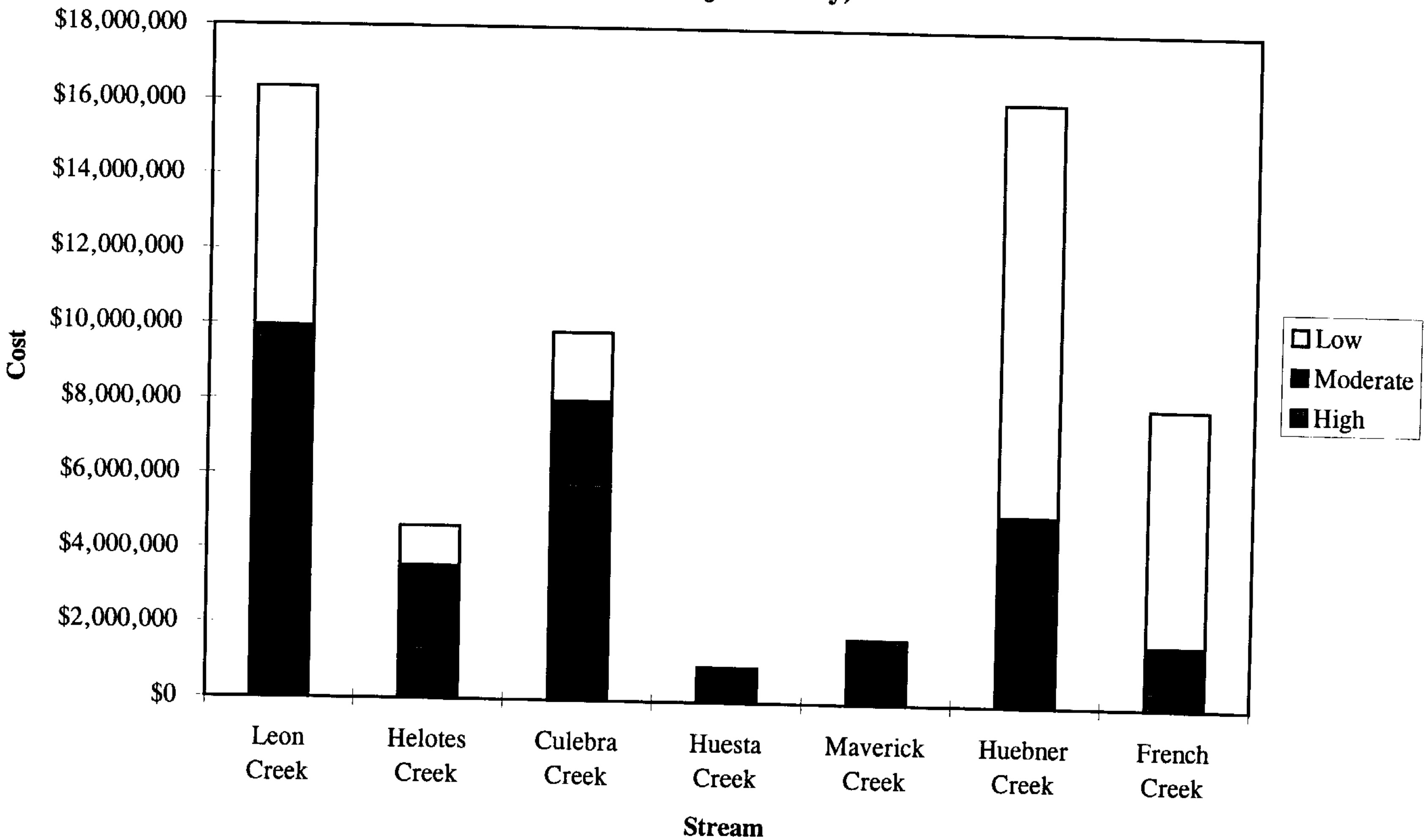


Figure 7.2 - Cost Summary by Stream and Priority (Fringe Projects Only)

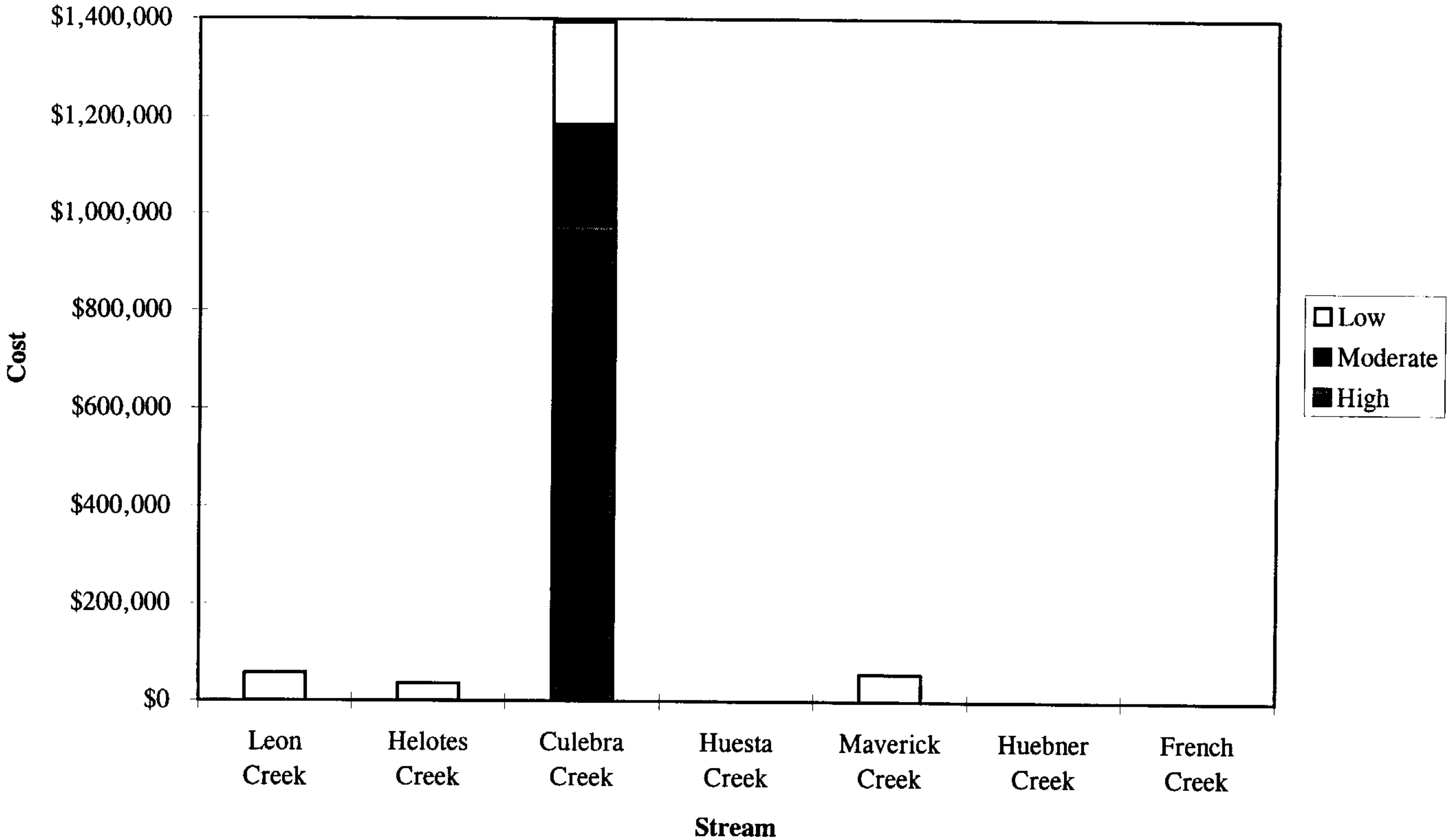
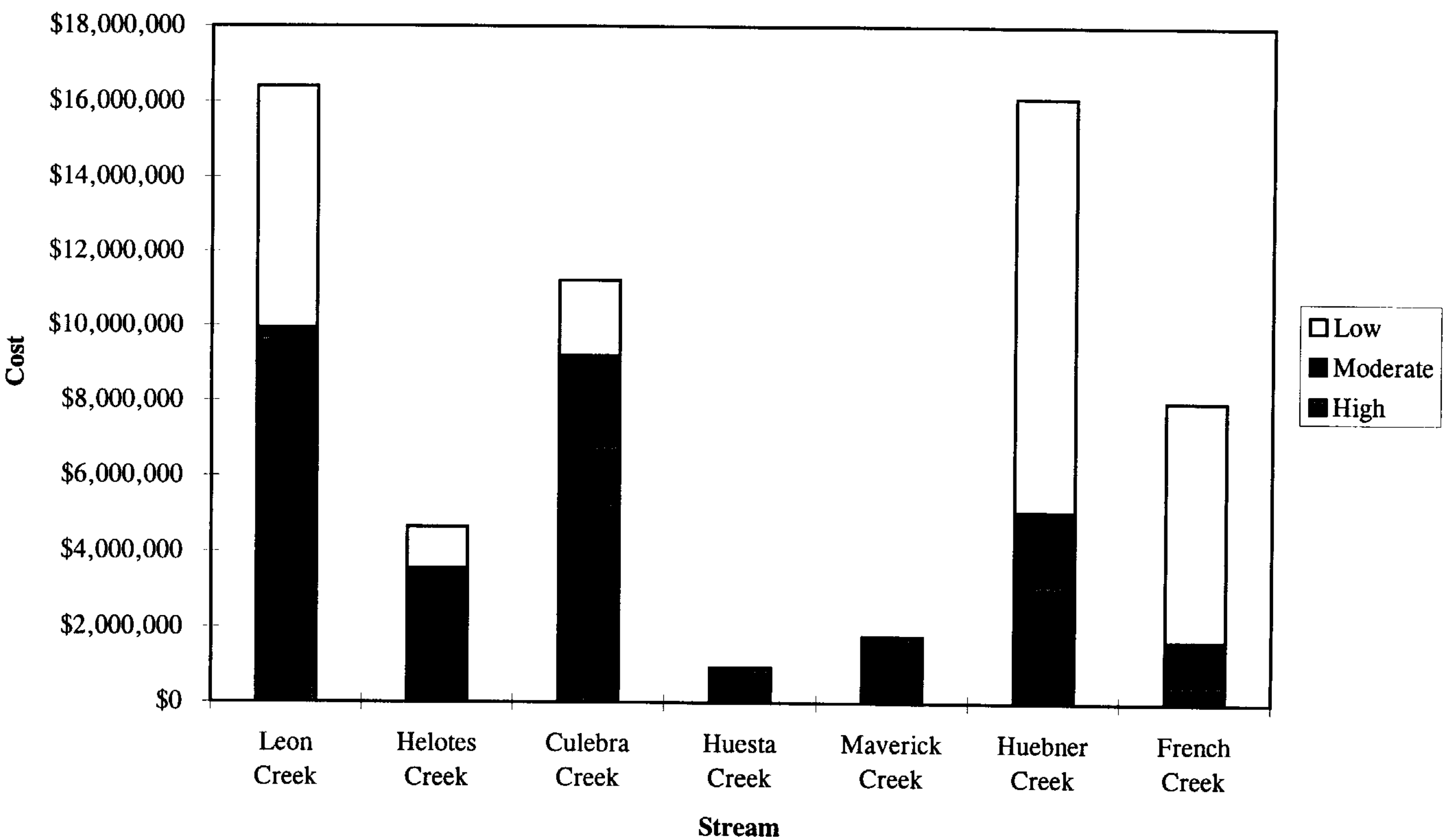


Figure 7.3 - Cost Summary by Stream and Priority



Leon Creek Watershed
Master Drainage Plan

City of San Antonio
Public Works Department

November 1996

Pape-Dawson Consulting Engineers, Inc.
HNTB Corporation
Maestas & Bailey, Inc.
E.L. Fly & Associates, Inc.

PAPE-DAWSON ENGINEERS
CIVIL & ENVIRONMENTAL
9310 BROADWAY, BUILDING II
SAN ANTONIO, TEXAS 78217-5987
(210) 824-9494
FAX 824-3491

November 21, 1996

Mr. John Kight, P.E.
City of San Antonio Public Works
114 W. Commerce, 7th Floor
San Antonio, Texas 78205

Re: Leon Creek Watershed Master Drainage Plan
Final Report

Dear Mr. Kight:

Attached are ten (10) copies of our final report for the Leon Creek Watershed Master Drainage Plan.

We look forward to supporting your staff in presenting the findings of our report to City Council. Please call if you need additional copies or if you have any questions.

Sincerely,
Pape-Dawson Consulting Engineers, Inc.

Rick Wood

Rick Wood, P.E.
Senior Project Manager

STATE OF TEXAS
W.R. WOOD
66364
REGISTERED
PROFESSIONAL ENGINEER

RW 3370-00

cc: Charlie Dodge - HNTB Corporation
Tom Bailey - Maestas & Bailey, Inc.
Everett Fly - E.L. Fly & Associates, Inc.

M961120A1.RW

Leon Creek Watershed Master Drainage Plan

City of San Antonio
Public Works Department

November 1996

Pape-Dawson Consulting Engineers, Inc.
HNTB Corporation
Maestas & Bailey, Inc.
E.L. Fly & Associates, Inc.

List of Figures

List of Tables

SECTION I. PROJECT SCOPE & OBJECTIVES

STUDY SPONSOR & ADVISORS

The Leon Creek Watershed Master Drainage Plan project was developed by the City of San Antonio Public Works Department. This project is being funded and administered through the same department. The Public Works Department is coordinating with the San Antonio Water System, Bexar County, Texas Department of Transportation, CSA Planning Department, CSA Parks and Recreation Department, Edward's Underground Water District and other local entities to coordinate the common interest of all parties.

A citizens advisory committee was created by San Antonio's City Council to seek citizen input and insure their representation in the formulation of the Drainage Master Plan. This committee is chaired by Councilman Howard Peak and has been named the Drainage Regulation and Review Committee. Members of this committee are listed in the Table I-1 below.

Table I-1
DRAINAGE REGULATIONS & REVIEW COMMITTEE

Committee Member	Representing
Howard Peak (Chair)	City Council
Bob Ross	City Council
Linda Billa Burke	City Council
Ed Cross	Planning Commissioner
Mike Cude	Professional Engineers in Private Practice
Norm Dugas	Real Estate Council
Dan Kossl	Greater S.A. Homebuilders Assoc.
Mike Gonzales	San Antonio River Authority
June Kachtik	Open Space Advisory Board
Charlie Connors	NODD
Unknown	Near Westside neighborhood representative
Larry DeMartino	Southeast neighborhood representative
John German	CSA Department of Public Works
Ray Rendon	Bexar County Department of Public Works
staff	SAWS
Steve Ramsey	SARA
Gayle Kipp	EUWD
John Kight	CSA Project Manager

PURPOSE

The City of San Antonio has authorized this study with the intent of developing a Master Drainage Plan for the Leon Creek Basin including the Leon Creek and its major tributaries from U.S. Hwy 90 to north of Loop 1604. Flood plain limits based on existing conditions will be determined for the 10, 25, 50, 100 and 500 year storm events. Ultimate development flood plain limits will be determined for the 25 and 100 year storm events. From the existing and ultimate development flood plain analysis, projects and watershed management practices will be identified to reduce existing and potential flood hazards. A ten year plan to implement the projects, identified to reduce flood hazards, will be developed and will include an estimated cost, priority and implementation schedule.

SCOPE OF SERVICES

This project consist of developing a Master Plan for drainage improvements in the Leon Creek Watershed in the southwest, west and northwest areas of the City of San Antonio and its ETJ. Other tributaries to be included in the study are Huebner Creek, French Creek, Helotes Creek, Culebra Creek, Huesta Creek and Maverick Creek. There are approximately 58.4 miles of related flood plains included in this study.

Limits of Detailed Study

Although this study addresses the entire Leon Creek Watershed, detailed flood plain delineation, site specific analysis and project development are limited to the segments of Leon Creek described in Table I-2 below.

Table I-2
LIMITS OF FLOOD PLAIN DELINEATION STUDY

Creek	Limits of Detailed Study	Length
Leon Creek	U.S. 90 to Loop 1604	17.8 miles
Culebra Creek	Leon Creek to Galm Road	9.1 miles
Helotes Creek	Culebra Creek to Helotes city Limits	5.7 miles
Huebner Creek	Leon Creek to IH 10	8.7 miles
French Creek	Leon Creek to Helotes city Limits	7.6 miles
Huesta Creek	Leon Creek to fork in creek north of Loop 1604	3.8 miles
Maverick Creek	Leon Creek to Heuermann Road	5.7 miles
Study Total		58.4 miles

Specific Task

The study is divided into a preliminary phase and a design phase. The preliminary phase is a research or discovery effort to determine what information has been developed in the past and to generally develop background data for the design phase. After completion of the preliminary phase, design efforts will begin to develop the detailed delineation of the existing and ultimate development flood plain. Specific projects will be developed and included in a ten year master drainage plan to reduce flood hazards within the Leon Creek Watershed.

During the preliminary phase, watershed maps were developed illustrating the full limits of the Leon Creek Watershed. All available drainage studies prepared for public or private use were identified through file searches and interviews and an index of these studies was prepared. These studies were then analyzed to determine their usefulness for purposes of this watershed study. This report is a summary of the preliminary phase effort.

The design phase will encompass development of a hydrologic model of Leon Creek and its major tributaries. This model will include quantitative hydrology and hydraulic calculations for the 10, 25, 50, 100, and 500 year storm events based on existing conditions of the watershed. In areas where private property is found to be inundated by the 100 year rainfall event, projects will be developed to mitigate the flooding in each location. A map depicting the existing flood plain overlaid on the City's Block Maps will be produced in conjunction with the study. A model will also be developed for the 25 and 100 year storm event and overlaid on the City's Block Maps based on ultimate development conditions in the watershed to determine potential flood mitigation practices or identify improvement projects to offset the effects of development and prevent future development from creating flooding problems. Consideration will be given to water quality issues, potential reuse and recharge projects and proposed by SAWS and other environmental concerns. A cost estimate and ten year plan to implement the specific projects identified in the design phase will be prepared along with project priorities.

Throughout this process, all efforts will be coordinated through the City's designated watershed study manager to insure that all interested parties are represented. This may include being present at citizen group meetings and coordination meetings with other governmental agencies. Upon completion of the study, a final report will be issued to present the results and recommendations to the City.

SECTION II. DISCOVERY

INTRODUCTION

San Antonio is located in the south-central portion of Texas, approximately 150 miles from the Gulf of Mexico and 100 miles from the geographical center of Texas. Situated in Bexar County on the San Antonio River, the terrain to the northwest slopes upward to the Edwards Plateau and to the southeast it slopes downward to the Gulf Coastal Plains. These two distinct geological regions are divided by the Balcones Escarpment, a critical recharge zone for the Edwards aquifer. The rolling hills of the area account for the range in elevation from 600 feet MSL (feet above mean sea level) in southern San Antonio to 1000 feet MSL just below the Balcones Escarpment to over 1600 feet MSL in the upper reaches of Bexar County. A location map of the project area is shown on Figure II-1.

Watershed Geographic Setting

The Leon Creek Watershed is located in the northwestern portion of Bexar County stretching from the confluence of Leon Creek with the Medina River, south of Loop 410 to the southwest of the City, to the northwest limits of Bexar County. Leon Creek's total watershed area is 237 square miles at the Medina River. The watershed limits are shown on Figure II-1.

The watershed area includes a portions of the cities of San Antonio, Leon Valley and Helotes. Kelly and Lackland Air Force Bases are located in the southern portion of the watershed adjacent to US Highway 90. Just upstream of the bases near the intersection of Commerce Street and Loop 410 is the Southwest Research Institute. All of these facilities were developed prior to the 1960's.

Development of the Leon Creek Watershed has been extensive in the last 30 years or so. The vast majority of the commercial and residential development outside Loop 410 has be since the late 60's. Aerial mapping flown in the early 60's from the Soil Conservation Service, Soil survey for Bexar County, shows very little development outside of Loop 410 . Major development since the early 60's include: the Medical Center, the University of Texas at San Antonio and the USAA campus. Since the early 80's the following areas have been developed: Sea World, Fiesta Texas and The Dominion.

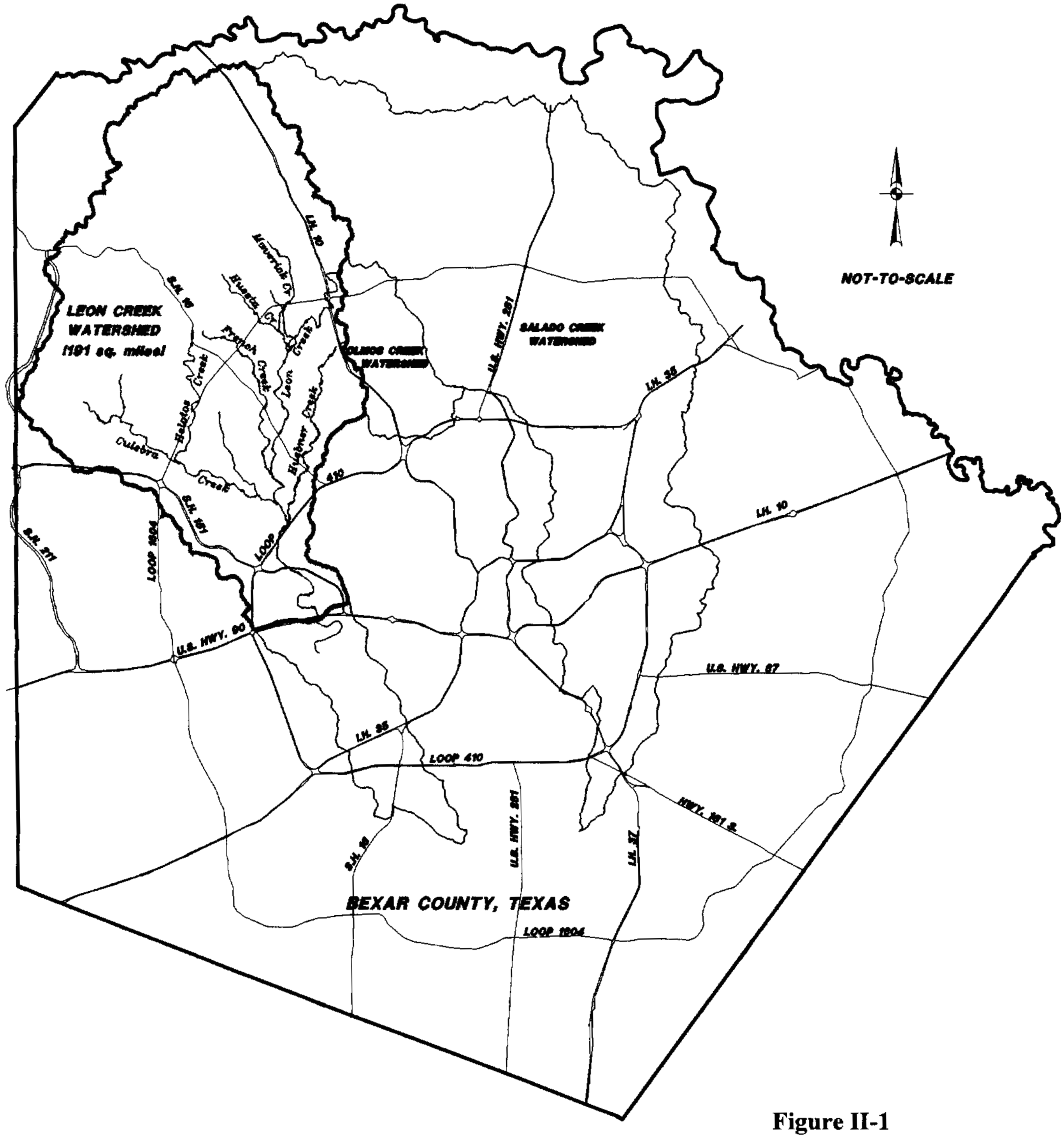


Figure II-1

WATERSHED LOCATION MAP

Leon Creek

Leon Creek originates in the northwestern portion of Bexar County. The stream flows in a southeasterly direction to its confluence with the Medina River. Within the Leon Creek Watershed are numerous other tributaries to the Leon Creek. Within The "Leon Creek Watershed Drainage Mater Plan" study area, only those segments or reaches of Leon Creek and its major tributaries shown in Table I-2 will receive specific analyses to determine the extent of the flood plain for design storm events.

HISTORIC RAINFALL & RUNOFF

The climate of San Antonio is best described as sub-tropical: continental during the winter months and hot during the summer. Due to its location between the semi-arid area to the west and the heavy rainfall area to the east and southeast, the annual rainfall of approximately 30 inches per year is sufficient for the normal production of most crops. Precipitation is reasonably distributed throughout the year, with the heaviest rains typically falling during May, in the spring, and September, in the fall. Similar to other Texas cities, rainfall in San Antonio varies greatly from year to year, ranging from approximately 10 inches in 1917 to approximately 50 inches in 1919. Recently, from December of 1992 To June of 1993, San Antonio received in the neighborhood of 50 inches of rain.

Rainfall from April through September usually occurs with thunderstorms. Large amounts falling in short periods of time create flash floods over some areas of the city. Winter precipitation occurs as light rain or drizzle, although thunderstorms and heavy rains have occurred in all months of the year. According to John Patton, of the National Weather Service, the average rain for San Antonio produces 1" to 1½" over a 50 square mile area and last for approximately 60 minutes, peaking in approximately 20 minutes. There are generally 40 to 45 of these storms each year that deposit rainfall over different parts of Bexar County.

Heavy rains over short periods of time cause flash flooding in certain sections of the city. Perhaps the worst flood of the century occurred in 1921 when 31.8 inches of rain fell in a 24 hour consecutive period of time[1]. This storm started as a hurricane along the Mexican coastline and moved inland and northeasterly across Texas. Five to nine feet of water stood in downtown San Antonio.

San Antonio's location on the Balcones Escarpment can be an intersection point for cold northern air to meet the warm moist prevailing southeast breezes of the coast. Frequently this condition results in rain, sometimes intense.

[1] *The amount of rain officially recorded for the month of September, 1921 is 8.27 inches. The 31.8 inches of rain occurred at a non-official localized rain gage.*

Throughout the "average" year measurable rain may be expected to fall on 80 days, with thunderstorms accounting for 36 of these. Rainfall lasts for only a brief period of time during the summer months as is characteristic of showers, except when the area comes under the influence of tropical storms. Longer periods of rainfall, drizzle and fog occur during the winter months when cool air stalls and is overrun by warm moist gulf air.

Rainfall Data

Official rainfall data was obtained from the National Climatic Weather Center in Ashville, North Carolina. Monthly and annual rainfall for San Antonio is presented in Table II-1. Figure II-2 illustrates the annual rainfall totals from 1900 to 1990.

During our research we observed that rainfall intensities typically can vary widely between different geographical area of the city. For example, on April 4 & 5, 1991, in Shavano Park 10.52 inches of rain was recorded in about two hours. However, small amounts of rainfall were measured at Loop 1604/IH 10 and at Vance Jackson; both areas adjacent to Shavano Park. Another example storm event happened on June 5, 1986 traveled from the southwest to the northeast parts of town. Rainfall along this line varied from about 6 inches to over 9 inches in Windcrest. Other areas of the city not directly within the path of the storm received less rain, within the range of 4 to 5 inches.

U. S. G. S. Stream Gage Recording Station

The United States Geological Survey (USGS) maintains a stream gaging station on the Leon Creek in the vicinity of Kelly and Lackland Air Force Bases. The station records the average daily flow in Leon Creek. Data from the USGS recording station provide daily mean flows and the maximum of the average daily discharge values in cubic feet per second (cfs) during each month. This recording station does not record the instantaneous peak flow, and therefore, does not provide any data to indicate what the peak flood flow from a storm event might have been. Table II-2 shows the monthly summaries of these values for the last 10 years.

Table II-1
SAN ANTONIO RAINFALL

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1900	5.42	0.34	4.35	9.11	4.47	0.78	2.24	4.05	0.97	2.94	1.82	0.70	**37.19**
1901	0.41	0.71	0.54	0.59	2.47	1.86	3.79	0.96	4.20	0.12	0.64	0.15	**16.44**
1902	0.70	0.55	0.12	2.31	3.14	0.02	3.85	0.00	5.52	2.54	3.53	2.51	**24.79**
1903	2.39	7.88	1.29	1.74	1.95	4.75	7.52	0.20	2.96	1.61	TR	0.82	**33.11**
1904	0.30	0.64	0.16	3.25	5.93	1.73	3.50	1.97	7.74	2.86	0.24	1.06	**29.38**
1905	0.88	1.62	2.74	6.08	4.11	6.01	2.82	0.51	1.80	1.83	2.63	1.56	**32.59**
1906	0.29	1.07	1.29	3.94	0.86	0.62	4.34	2.25	1.74	1.09	1.33	1.60	**20.42**
1907	0.80	0.78	1.88	3.77	4.64	0.18	2.68	0.80	1.11	3.54	6.79	0.80	**27.77**
1908	1.01	2.42	1.31	2.87	6.07	0.30	0.66	4.27	3.92	1.47	2.61	1.61	**28.52**
1909	0.10	0.71	0.88	0.82	1.77	1.65	3.27	1.70	0.56	1.55	0.53	1.38	**14.92**
1910	0.88	0.78	0.42	3.31	1.56	0.55	1.37	0.37	0.56	3.35	1.38	1.69	**16.22**
1911	0.02	1.66	2.72	3.41	2.01	0.30	1.03	0.48	0.12	3.57	2.01	1.35	**18.68**
1912	0.28	5.12	1.86	1.78	1.49	3.22	1.27	0.29	1.47	2.74	1.45	2.76	**23.73**
1913	0.90	1.91	1.36	1.32	2.88	2.90	0.03	1.29	7.21	8.86	4.55	4.47	**37.68**
1914	0.09	1.38	0.83	5.26	5.59	0.01	0.02	7.80	2.24	5.78	3.24	1.43	**33.67**
1915	0.53	1.81	1.20	11.64	1.89	0.03	0.92	3.90	2.39	1.11	0.29	1.57	**27.28**
1916	2.25	0.01	0.79	1.85	3.85	0.49	4.53	5.07	3.78	2.57	2.14	0.33	**27.66**
1917	0.95	0.49	0.16	0.28	3.30	0.02	2.19	0.10	1.39	0.48	0.75	TR	**10.11**
1918	0.10	1.10	1.45	5.14	2.80	3.35	1.68	2.61	1.49	4.05	2.53	3.61	**29.91**
1919	3.78	1.56	1.39	3.60	3.06	7.01	7.88	2.14	7.61	8.66	1.56	2.05	**50.30**
1920	3.36	0.27	0.83	1.09	2.42	2.83	0.39	2.26	0.15	2.85	2.95	0.16	**19.56**
1921	1.40	0.23	5.91	2.78	2.01	4.59	0.48	0.45	8.27	1.02	1.16	0.23	**28.53**
1922	1.23	1.26	3.29	5.46	3.46	3.92	0.10	0.27	0.97	3.55	0.98	0.10	**24.59**
1923	0.46	5.47	3.07	3.24	1.33	0.79	2.54	2.94	2.98	1.39	4.21	4.29	**32.71**
1924	0.97	3.02	1.29	3.36	4.71	4.66	0.05	TR	2.52	0.52	0.24	2.31	**23.65**
1925	0.36	0.09	0.24	0.18	2.85	0.48	1.24	1.72	2.87	2.23	1.44	1.29	**14.99**
1926	3.42	0.08	4.77	7.06	3.33	3.57	1.37	0.31	0.43	1.82	1.99	2.24	**30.39**
1927	0.65	1.96	2.02	2.05	2.04	7.91	0.49	0.15	1.52	1.44	0.03	2.49	**22.75**
1928	0.65	2.85	2.34	1.70	3.90	3.29	1.03	1.21	6.30	1.69	2.29	2.95	**30.20**
1929	2.21	0.16	3.12	2.37	7.73	2.19	2.58	0.01	2.02	1.60	3.17	2.08	**29.24**
1930	1.25	0.94	1.76	2.20	0.89	4.03	1.99	0.41	1.74	4.01	2.69	0.88	**22.79**
1931	5.86	2.68	2.06	2.28	1.36	3.10	3.09	0.30	0.01	0.75	0.72	2.79	**25.00**
1932	3.30	1.86	1.05	2.61	2.10	1.94	5.52	6.71	8.77	0.60	0.10	1.01	**35.57**
1933	0.66	1.92	0.54	1.30	2.23	1.74	1.92	2.78	3.18	0.27	0.65	0.39	**17.58**
1934	4.88	0.43	2.05	4.56	1.65	0.18	3.83	0.88	1.95	0.19	2.88	4.17	**27.65**
1935	0.31	1.87	2.31	3.52	14.07	8.41	1.61	0.98	5.61	1.94	0.44	1.86	**42.93**
1936	0.43	0.40	2.66	2.77	6.13	6.43	2.68	2.23	4.07	1.89	2.17	1.75	**33.61**
1937	0.96	0.13	2.10	0.84	7.68	2.19	1.82	0.14	0.04	3.09	0.86	6.22	**26.07**
1938	3.35	0.33	3.82	6.06	3.88	0.65	0.91	0.44	1.82	0.13	0.63	1.24	**23.26**
1939	2.08	0.95	0.65	0.78	3.22	0.10	2.12	5.08	1.90	0.07	0.99	0.89	**18.83**
1940	0.64	1.86	0.94	2.50	4.19	7.47	0.64	1.22	1.42	4.66	2.40	2.85	**30.79**
1941	2.14	1.86	2.95	4.56	2.50	2.03	0.62	0.23	4.88	3.13	0.47	0.97	**26.34**
1942	0.13	2.01	0.29	3.48	2.19	1.95	8.19	1.88	7.67	9.56	0.47	0.64	**38.46**
1943	0.73	0.09	1.58	1.48	2.56	1.91	3.72	0.78	4.34	0.17	1.95	1.20	**20.51**
1944	3.49	1.68	3.72	0.94	6.76	1.64	TR	4.32	1.30	1.52	3.66	4.16	**33.19**
1945	2.97	3.90	2.73	2.91	1.24	5.31	1.19	1.19	3.00	3.49	1.35	1.18	**30.46**
1946	3.64	2.24	1.75	5.54	3.47	2.92	0.20	4.03	15.78	1.31	1.86	2.43	**45.17**

Table II-1 (continued)
SAN ANTONIO RAINFALL

1947	2.14	0.29	1.46	0.30	3.32	0.31	1.00	5.34	0.06	0.19	1.01	1.90	**17.32**
1948	0.61	1.86	0.59	1.40	1.59	2.96	2.35	5.83	1.98	3.24	1.00	0.23	**23.64**
1949	2.91	2.98	2.27	8.99	0.85	8.26	2.24	1.03	0.78	7.58	0.13	2.79	**40.81**
1950	0.32	1.43	0.24	3.42	2.41	1.03	1.60	6.15	3.02	0.08	0.13	0.03	**19.86**
1951	0.25	2.43	2.76	0.93	4.44	7.07	0.51	0.06	3.75	1.44	0.67	0.13	**24.44**
1952	0.81	2.01	2.34	3.40	1.91	1.80	2.75	0.00	3.02	TR	4.47	3.67	**26.18**
1953	0.41	0.90	0.53	2.08	1.00	2.19	0.01	3.12	2.48	3.06	0.34	1.44	**17.56**
1954	1.51	0.03	0.03	1.94	1.46	2.71	1.25	1.05	0.52	1.98	2.02	0.20	**14.70**
1955	1.45	2.33	1.40	0.14	4.44	2.88	1.32	0.81	0.79	0.39	1.57	0.66	**18.18**
1956	0.81	0.85	0.27	0.49	3.07	0.27	1.53	3.94	0.62	1.23	1.13	1.10	**15.31**
1957	0.51	2.53	4.19	9.32	8.22	3.49	0.73	0.21	11.10	4.71	2.90	0.92	**48.83**
1958	4.57	3.88	1.08	1.32	1.98	3.39	7.39	0.45	8.36	5.43	0.77	1.07	**39.69**
1959	0.52	2.50	0.13	2.55	2.43	1.32	1.48	3.05	1.72	5.11	2.17	1.52	**24.50**
1960	0.76	1.22	1.65	2.08	1.21	2.70	1.31	5.96	0.76	7.84	1.30	2.97	**29.76**
1961	0.68	1.79	0.03	0.32	0.17	7.87	7.04	0.15	2.24	3.39	2.09	0.70	**26.47**
1962	0.48	0.90	0.91	4.02	1.31	2.44	0.13	1.57	2.69	2.19	4.97	2.29	**23.90**
1963	0.27	3.59	0.21	1.88	3.03	2.28	0.03	0.63	1.11	2.75	1.93	0.94	**18.65**
1964	3.40	1.88	1.73	1.16	1.79	4.88	0.02	5.19	4.15	0.86	4.81	1.22	**31.09**
1965	2.40	6.43	2.30	1.97	8.18	2.42	0.08	1.65	3.13	2.69	0.91	4.58	**36.74**
1966	1.47	2.30	1.13	3.20	3.53	1.78	0.06	4.28	2.13	1.11	TR	0.42	**21.41**
1967	0.18	0.48	2.18	0.94	2.22	0.01	2.12	3.16	11.16	2.00	3.42	1.38	**29.25**
1968	8.25	1.85	1.27	1.92	2.82	2.63	1.53	0.94	2.98	0.69	4.58	0.66	**30.12**
1969	1.76	2.90	2.36	2.46	4.61	2.32	0.36	4.19	1.32	5.83	1.02	2.28	**31.41**
1970	1.10	2.66	1.98	1.13	7.30	0.89	0.91	0.95	4.35	1.31	0.01	0.01	**22.60**
1971	0.04	0.81	0.04	1.39	1.52	2.74	1.05	9.42	4.75	4.62	2.74	2.86	**31.98**
1972	1.35	0.40	0.13	1.94	11.24	2.86	3.13	4.24	1.40	1.99	2.37	0.44	**31.49**
1973	2.77	2.76	1.58	5.41	2.73	10.44	6.91	1.29	13.09	4.85	0.29	0.16	**52.28**
1974	1.36	0.04	0.94	2.18	4.28	1.02	1.28	11.14	3.85	4.09	5.39	1.43	**37.00**
1975	1.04	3.30	0.52	2.69	6.91	4.60	1.06	1.28	0.51	2.25	0.03	1.48	**25.67**
1976	0.56	0.13	1.20	5.67	5.80	1.61	5.39	2.09	3.79	8.48	2.46	1.95	**39.13**
1977	3.10	0.91	0.88	8.80	1.62	2.26	0.10	0.06	2.11	3.47	6.01	0.32	**29.64**
1978	0.68	1.76	1.71	3.62	2.45	3.96	1.43	4.97	8.86	0.55	4.91	1.09	**35.99**
1979	4.07	1.38	3.55	5.34	1.98	5.59	7.38	2.09	0.84	0.11	1.43	2.86	**36.62**
1980	0.72	0.74	0.98	1.67	6.42	0.52	0.26	2.64	5.05	1.09	3.53	0.61	**24.23**
1981	2.06	0.96	1.96	2.21	6.43	8.71	0.25	2.41	1.36	8.61	0.72	0.69	**36.37**
1982	0.72	1.28	0.69	1.23	6.42	1.37	0.14	0.55	0.87	2.84	4.54	2.31	**22.96**
1983	1.48	1.54	3.89	0.13	4.37	1.27	2.43	2.00	3.86	1.64	3.06	0.39	**26.06**
1984	1.87	0.54	1.91	0.11	3.76	1.40	TR	2.99	1.06	5.94	2.91	3.41	**25.90**
1985	2.68	1.91	2.85	3.27	2.47	8.20	5.80	0.45	4.80	3.91	3.93	0.00	**40.27**
1986	0.76	2.52	0.35	0.60	6.29	11.95	0.05	1.89	2.83	6.58	1.83	7.11	**42.76**
1987	1.13	4.78	1.10	1.48	12.85	7.69	1.21	0.33	2.24	0.44	2.53	2.18	**37.96**
1988	0.39	0.92	0.86	1.23	0.41	5.50	5.58	1.98	0.83	0.62	0.02	0.67	**19.01**
1989	2.96	0.29	1.24	2.55	0.33	3.96	0.69	0.48	1.54	5.81	1.93	0.36	**22.14**
1990	1.17	2.68	5.17	4.52	3.28	1.18	8.29	1.30	3.70	3.71	3.11	0.20	**38.31**
***AVE**	**1.56**	**1.68**	**1.66**	**2.89**	**3.61**	**3.04**	**2.20**	**2.19**	**3.25**	**2.80**	**2.02**	**1.64**	**28.34**

* For period of record shown (1900-1990).

Figure II-2
ANNUAL RAINFALL

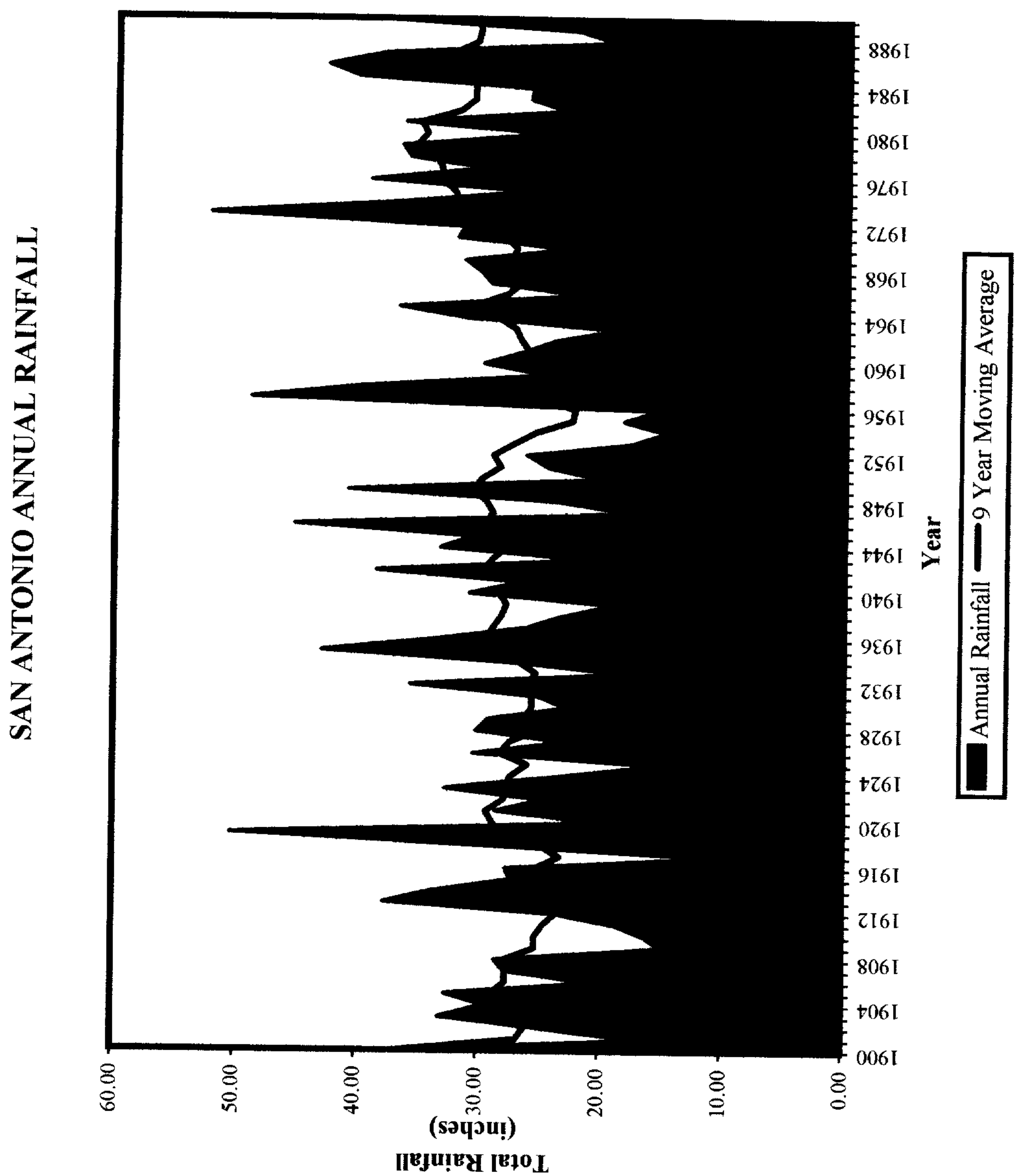


Table II-2
USGS MONTHLY STREAM FLOWS

	1985		1986		1987		1988		1989		1990		1991		1992		1993	
Month	Mean	Max	Mean	Max	Mean	Max	Mean	Max	Mean	Max	Mean	Max	Mean	Max	Mean	Max	Mean	Max
Jan	14.3	72	11.6	29	21	60	9.71	12	18.1	237	5.36	11	28.1	517	116	1320	26.1	116
Feb	15.3	193	16.1	107	45.9	310	12	102	5.93	19	9.4	58	23.7	287	355	5020	56.9	355
Mar	39.5	468	9.01	14	23.5	57	12.4	150	6.46	20	21.9	186	10.4	21	192	2630	37.4	192
Apr	16.6	64	6.92	13	18.8	36	6.94	15	29.2	471	18	283	82.6	1430	46.3	218	26.5	82.6
May	16.9	150	49.3	1040	181	1150	7.61	38	5.89	11	12.4	110	37.3	348	356	3400	96.1	356
Jun	115	1660	324	4540	824	5580	10.9	100	25	361	4.96	46	50.9	804	168	1220	174	824
Jul	18.6	147	12.9	23	25.3	65	10.6	142	2.56	4.2	144	2260	23.2	182	17.4	25	29.5	144
Aug	13.9	69	8.21	21	13.8	19	11.7	222	1.94	7.1	7.03	36	4.64	17	23	156	10.1	23
Sep	25.2	111	36.1	365	11.9	39	38.3	879	1.97	3.3	7.04	27	7.12	23	20	199	17.2	38.3
Oct	56	633	69.6	426	7.3	22	6.14	11	7.18	73	6.83	42	4.94	20	21.6	69.6		
Nov	21.8	78	12.6	56	10.7	74	5.16	21	4.07	22	9.2	98	3.95	7	12.7	37.9		
Dec	11.1	16	30.3	250	12.8	41	5.05	7.4	4.62	11	5.73	6.2	575	6190	74.6	575		

Flood Events

San Antonio has experienced a number of significant floods as shown in Table II-3. This information was gathered from newspaper articles and other sources. Consequently, the duration of some of these rainfall events was not available. The most significant flood occurred in 1921. Another major flood event took place in September, 1946 when over 6 inches of rain fell in an 8 hour period and more than 10 inches of rain fell during the storm. Development in the Leon Creek Watershed has occurred primarily since the late 50's, and consequently, little flood damage has been documented.

Table II-3
FLOODS OF RECORD

Date	Description
September, 1921	up to 17" in two hours
May, 1937	6.21" in 8 hours
September, 1946	6.05" in 8 hours, 10.43" for the total storm event
May, 1965	6" prompting congressional action by Henry B. Gonzalez
September 23, 1969	6" downtown
August 8, 1974	4" in brief time with wet preceding conditions
June 13, 1981	3.2" in one hour at Kelly, 5" at Woodlawn Lake,
September 19, 1983	4.2"
June 5, 1986	9.61" reported in Windcrest
May - June, 1987	12.85" in May
June 11, 1987	7.21" in Helotes, 6.48" in 26 hours at Trailwood
May 6, 1993	7.25"

GOVERNING AND CONTROLLING AGENCIES

There are numerous agencies that have interest in the Leon Creek Watershed. During this project many of these agencies and individuals were contacted to obtain information relevant to the drainage conditions in the Leon Creek Watershed. On the following pages are summaries of the agencies and individuals contacted, reports that were reviewed and studies that were analyzed.

Agencies Contacted

During the investigation for this project many agencies were contacted for information that could be beneficial to completing this study. We have listed below the agencies contacted and the individual(s) we talked with.

Table II-4
AGENCY INTERVIEWS

Agency	Person(s) Interviewed
City of San Antonio Drainage Department	Roy Akiona, Tom Carrasco & Mendi Littman
San Antonio Water System	Jay Aldean, Tom Fox & Chris Powers
Bexar County Public Works	Ron Pena
Edwards Underground Water District	Bobby Bader
Texas Department of Transportation	Julia Brown, Preston Streicher & Judy Freisenhahn
City of Leon Valley	Jim Malone
San Antonio River Authority	Steve Ramsey
San Antonio Police Department	Desk Officer & Human Resources
San Antonio Fire Department	Lt. Jim Collins
City of San Antonio Information Services	Steve Bishop
City of San Antonio Traffic Department	Andy Ballard
Kelly Air Force Base	William Ryan
Lackland Air Force Base	Eric Staph & Gabe Gonzalez
City of San Antonio Mapping	Abner Martinez
UTSA Center for Archeological Research	Robert Hard & Ann Fox
City of San Antonio Department of Parks & Recreation	Dale Bransford
Soil Conservation Service	Dale Mengers
U.S. Army Corp of Engineers	Brian Rowe

City of San Antonio Drainage Department

<u>Drainage Assessment for the Middle Leon Creek</u>

Drainage Engineering of the City of San Antonio Department of Public Works prepared a Drainage Assessment for the Middle Leon Creek and Huebner Creek in October of 1993. The area included in "Middle Leon Creek" study consisted of Huebner and Leon Creeks from Huebner Road upstream to Loop 1604. Presented in this report are known problem areas within the Leon and Huebner Creek area along with proposed improvements that will address these problems. Table II-5, shown below, is a summary of these problem areas, proposed improvements and estimated costs for construction, right-of-way and engineering.

Table II-5
CSA - MIDDLE LEON CREEK DRAINAGE ASSESSMENT

Problem Area	Proposed Improvement				Cost Estimate		
	Structure	Channel	Pipe System	Realignment	Construction	ROW	Engineering
Hills & Dales Subd.	Y	Y	Y		$684,000	$16,000	$86,000
Dell Oak Subd. - Lake Breeze St.	Y			Y	$1,026,000	$243,000	$129,000
Hausman Rd. - W. of Babcock @ Huesta Creek	Y				$492,000	NA	$62,000
Hausman Rd. - E. of Babcock @ Maverick Creek	Y	Y			$650,000	$81,000	$82,000
Valley View Subd. - Nickle & Dime area Phase I	Y	Y			$1,154,000	$185,000	$145,000
Valley View Subd. - Nickle & Dime area Phase II	Y	Y			$1,963,000	$308,000	$247,000
Babcock Rd. crossing Huesta, Maverick	Y				$7,985,000	$297,000	$1,005,000
Babcock Rd. crossing Leon Creek (East)	Y			Y	$3,678,000	$306,000	$463,000
Babcock Rd. crossing Leon Creek (West)				Y	$4,259,000	$378,000	$536,000
DeZavala Rd. - North of Babcock	Y	Y			$85,000	$4,000	$11,000
Babcock Rd. - West of DeZavala Rd.	Y	Y			$419,000	$42,000	$53,000
Spring Forest Drive	Y				$635,000	$41,000	$80,000
Prue @ Huebner Creek	Y	Y			$743,000	$56,000	$60,000
White Bonnet at Lockhill Selma	Y	Y			$992,000	$112,000	$125,000
Hollyhock - West of Babcock	Y	Y			$671,000	$143,000	$85,000
Strathaven - North of Hollyhock	Y				$689,000	$63,000	$87,000
Abe Lincoln and Hollyhock			Y		$829,000	$23,000	$104,000
Whitby @ Huebner Creek	Y	Y			$527,000	$59,000	$66,000
Total					$27,481,000	$2,098,000	$3,426,000

Low Water Crossings

Many locations where streets cross creeks have little or no drainage structures. This condition is commonly known as a low water crossing. A list of low water crossings in the Leon Creek Basin was obtained from the San Antonio Fire Department and is shown in Table II-6 below.

Table II-6
LOW WATER CROSSINGS

Barricade	District	Location	Creek	Page No.	Block	Street	Block #
38	6	2000 block of Pinn Rd.	Leon, branch	613	F7	Pinn	2000
39	6	Arvil btw Keitha & Elmer	Leon, branch	614	B7		
40	6	Rodriquez	Leon	614	B7		
41	6	Military & Westbriar	Leon, branch	613	E7		
42	6	Martinique btw Barbados & Andros	Leon, branch	613	F7		
43	6	Tallahasse btw Barbados & Andros	Leon, branch	613	F7		
44	6	Westfield btw Barbados & Andros	Leon, branch	613	F7		
45	6	Biscayne btw Barbados & Andros	Leon, branch	613	F7		
68	7	W. Commerce btw Pinn & Military	Leon	613	F3	W. Commerce	
69	7	Pinn, 2500' s. of W. Commerce	Leon	613	F4	Pinn	100 - 500
70	7,8	Timber Path, 500' se of Grissom	Culebra	579	B7	Timber Path	9000-9100
72	8	Hausman, 200' e of Babcock	Huesta	513	E8	Hausman	7500
73	8	Hausman @ Roadrunner	Huesta	513	F8	Hausman	7000-7100
74	8	Hausman	Leon	514	A8	Hausman	6700
75	8	Old Fredericksburg, n of 1604	Leon	514	C5	Old Fred	15800
76	8	Hausman, 4800' w of IH10	Leon	514	B7	Hausman	6000-6100
77	8	Danvers btw Glidden & Dime	Huesta	513	E8	Danvers	short
78	8	Babcock, 100' n of Nickle	Huesta	513	E8	Babcock	12500
79	8	Babcock, 500' s of Nickle	Huesta	513	E8	Babcock	to
80	8	Babcock, 2300' s of Nickle	Huesta	547	E1	Babcock	
81	8	Babcock, 3700' s of Nickle	Leon	547	F1	Babcock	13500
89	8	Prue Rd, 1600' e of Babcock	Huebner	548	C4	6300-7000	
90	8	Lockhill, 250' e of White Bonnet	Huebner	548	C4	Lockhill @ White Bonnet	
91	8	White Bonnet, s of Lockhill	Huebner	548	C5	same	
92	8	Hollyhock, 600' w of Babcock	Huebner	548	B7	Hollyhock	6100-6500
93	8	Whitby, 200' n of Wellesly Manor	Huebner	548	B8	Whitby & Wellesly Manor	
96	8	Huebner, 400' s of Apple Green	Huebner	548	B8	Huebner @ Wade Lane	
112	7	Wurzbach, 750' s of Seville	Huebner, branch	580	B5	Wurzbach	4700-5000
113	6,7	Timberhill, n of Wurzbach	Huebner	579	F6	Timberhill	4000-4200
114	7	Ingram, 2500' e of Culebra	Leon	579	E7	Ingram btw Mabe & Northwestern Dr.	
115	8	Easterling, s of Culebra	Culebra	578	D4	Easterling	
116	8	Old Grissom, 500' e of Culebra	Culebra	579	C5	Old Grissom	Culebra

During a moderate storm event the roadway at the low water crossing is overtopped by the creek flow. Fire, Police and Public Works personnel typically put up barricades at the low water crossings to warn the public of the danger. Problems can arise when a motorist drives a vehicle into water that reaches the floorboard. The combination of the force of the water splashing on the upstream side of the vehicle coupled with the vehicle's poor traction caused by the wet conditions and the vehicle's tendency to float can push the vehicle off the road and into the creek bottom. Many cases of motorist being stranded in a low water crossing have been documented. The Fire Department keeps records of high water rescues. Table II-7 below is a listing of recent rescues.

Table II-7
RECENT LOW WATER CROSSING RESCUES

Incident No.	Location	Date
92002144	IH 35 S @ Leon creek	1/26/92
92002151	Hwy 151 @ Pinn Rd.	1/26/92
92002149	Ingram Rd. @ Potranco	1/26/92
92002169	Ingram @ Wurzbach	1/27/92
92002141	Ingram @ Wurzbach	1/26/92
92002757	Military Dr/Pearsall Rd.	2/4/92
92002740	Babcock/Hausman Rd. W	2/3/94
92002809	Babcock/Hausman Rd. W	2/4/92
92005068	Babcock/Southpoint	3/3/92
92005145	Babcock	3/4/92
92005135	Babcock/Hollyhock	3/4/92
92010234	Babcock/Nickle	5/4/92
92011159	Gen. McMullen S/EB New Hwy 90	5/14/92
92011580	Hwy 151/Pinn Rd	5/19/92
92011616	Babcock/Hausman Rd W	5/20/92
92012275	Babcock/Louis Pasteur	5/27/92
92012286	Culebra Ave	5/27/92
92012294	Culebra /Loop 1604	5/27/92
92012293	Hwy 151/Loop 410 SW	5/27/92
92012289	Culebra/Laven Dr.	5/27/92
92012371	Leon Creek/Prue Rd	5/27/92
92013405	Hwy 151/Potranco rd	6/9/92
92028521	Babcock/Hollyhock	11/19/92
93011942	Floyd Curl St./Huebner Rd	5/5/93
93011841	Eckhert/John Marshall	5/5/93
93011937	Babcock, 5700	5/5/93
93011967	Eckhert/Huebner	5/5/93
93011927	Babcock, 5700	5/5/93
93015952	Gen. McMullen S/EB New Hwy 90	6/12/93
93029135	Babcock, 2626	10/5/93

An example of a typical low water crossing incident was found in the March 4th and 5th, 1992 issue of the San Antonio Express News. Excerpts from those two issues are shown below.

"Violent thunderstorms with occasional hail and winds of more than 50 mph bore down from the west late Tuesday night, March 3rd, and early Wednesday, March 4, 1992. The storm dumped an average of 3 inches of rain across the city. Soon after the first storm began, low water crossings flooded across the Northwest Side, keeping police and firefighters hopping to respond to reports of trapped cars. In San Antonio, 26 calls for vehicles trapped in water were reported.

Among the locations where vehicles were reported trapped in high waters were the intersections of Callaghan Road below Interstate 10, Interstate 410 at Bandera Road, Babcock and Vance Jackson roads, Babcock and Huebner Roads, Hillcrest and Midcrest Drives, the 300 block of Cherry Ridge, and at Dreamland Drive and Vance Jackson Road.

Jian Ke, a student at the University of Texas at San Antonio, had to be rescued about noon Wednesday, March 4, when his car was pushed off Babcock Road into Leon Creek. The water floated his vehicle off the road and lodged it between a couple of trees. Firefighters had a difficult time getting to him because the water, about 5 feet deep, was moving fast and his electric windows would not open. A rear window had to be smashed to free the man. The rescue took about 45 minutes. Fire Caption Dennis O'Neill said: "He's lucky to be alive. If the car would have turned over, he would have been gone".

San Antonio Water System

Reuse Plan

SAWS has developed a water plan for the City of San Antonio that has many elements. The reuse of treated effluent from the City's wastewater treatment plants for non-potable uses could be a significant source of water that now is not appreciably used by the City.

Integral to the reuse program will be a need for storage facilities for seasonal and temporary storage. There could be locations within the Leon Creek Watershed that could serve a dual purpose of detention for flood abatement and storage for reuse water. Again, the amount of flood abatement achieved depends on the storage capacity of the impoundment facility. If a facility is to be shared with reuse storage, determination of a balance of storage capacity for reuse and flood abatement would be critical.

Water Quality

Although water quality is not a direct charge of this report, we did discover information on this subject. The Environmental Management staff at Kelly Air Force Base has and is developing extensive baseline data on water quality in the Leon Creek as it crosses their base. When complete, this information will be very useful for the SAWS stormwater department.

The SAWS stormwater department is also developing water quality data through a contract with the USGS.

Edwards Underground Water District

Recharge

The Edwards Underground Water District has sponsored a study to investigate recharge enhancement in the San Antonio and Guadalupe River Basins. In this study three potential recharge site were identified in the Leon Creek Watershed. These three location are:

1.	Culebra Creek	Government Canyon
2.	Helotes Creek	North of Helotes
3.	Leon Creek	Near IH10 Loop 1604 interchange

These locations were identified during the phase 1 study in a general manner. A fourth site located along Helotes Creek in the Vulcan Materials Quarry has been discussed as a potential recharge site since the study was released. During the on-going phase 2 study, field surveys of the potential recharge enhancement sites will be performed. The site evaluations should be completed by the end of 1994.

Recharge enhancement impoundment facilities may also assist in flood abatement by detaining a portion of the watershed runoff. The amount of flood abatement achieved depends on the storage capacity of the impoundment facility.

EXISTING REPORTS AND STUDIES

During this project, numerous agencies and individuals were contacted to obtain information relevant to the drainage conditions in the Leon Creek Watershed. On the following pages are summaries of the agencies and individuals contacted, reports that were reviewed and studies that were analyzed. The following paragraphs contain a synopsis of the information we collected from these interviews, reports and studies. Table II-8 below is an index of drainage reports sponsored by Public Agencies.

Table II-8
EXISTING DRAINAGE REPORTS

Report	Author	Date
Flood Insurance Study	FEMA	July 2, 1991
Flood Plain Information, Leon Creek	Corp of Engineers	April, 1971
Flood Plain Information, Huebner Creek	Corp of Engineers	June, 1973
Issues & Impacts of Stormwater Drainage, Bexar County, TX	UTSA	Summer, 1993
The Edwards Aquifer; S.A. mandates for Water Quality Protection	SAWS	April 1, 1994
Drainage Assessment for the middle Leon Creek & Huebner Creek	CSA	October 1, 1993
Recharge Enhancement Study, Guadalupe - San Antonio River Basins	HDR	Summer, 1993
Lake Travis Non-point source Pollution Control Ordinance	LCRA	January 1, 1991
Hydrologic Data for Urban Studies in San Antonio, TX metro area	USGS	May, 1976
Hydrologic Data for Urban Studies in San Antonio, TX metro area	USGS	February, 1982
Flood Protection Plan for Portions of Salado, Cibolo & Leon Creeks	CH2MHill	August, 1989
Soil Survey, Bexar County Texas	SCS	1962

Review of Reports

In reviewing the existing reports and studies we where interested in information that would be relevant for use in this study. Below is a description of the relevant portions of the reports.

Flood Insurance Study

This study includes a complete analysis of the Leon Creek. The water surface profiles for the design storm events have been used to define the floodplain limits. Although this study gave a complete picture of the Leon Creek, the base survey information of the existing ground contours was based on course data.

The study was performed in the late 70's. Portions of the study have been updated by private developers who modified the existing creek system to accommodate their developments. The resulting 1991 update of this report is a mosaic of the original analysis along with a number of updates.

***Flood Plain Information, Leon Creek* and *Huebner Creek*(2 separate reports)**
Both of these reports provide the same types of historical information for the respective creeks. Information presented includes: Background information, flood information (past, current and future) and guidelines and suggestions for floodplain management.

Issues & Impacts of Stormwater Drainage, Bexar County, TX
A product of the Environmental Sciences and Engineering Programs at the University of Texas at San Antonio, "the intent of this study is to develop a clear definition of the nature and extent of existing drainage problem"

The Edwards Aquifer; San Antonio Mandates for Water Quality Protection
This SAWS report presents regulatory requirements, organizational programming and potential activities. These items consist of:

- Regulatory Requirements
  - The Unified Development Code
  - Stormwater
  - Water Code
- Organizational Programming
  - Texas Natural Resource conservation Commission rules & regulations
  - Technical Improvements
  - Emergency Measures
- Potential Activities
  - Future Studies

Drainage Assessment for the Middle Leon Creek & Huebner Creek
This assessment presents known problem areas, projected projects to solve these problem areas and projects that are all ready funded to solve problem area.

Recharge Enhancement Study, Guadalupe - San Antonio River Basins
The Edwards Underground Water District sponsored this study to find potential recharge enhancement projects. Three potential recharge enhancement sites were listed in this report. The recharge dams may also assist in flood abatement.

Lake Travis Non-point Source Pollution Control Ordinance
This manual provides developers with guidance on the LCRA review requirements and procedures. Also outlined are best management practices to meet the LCRA standards.

Hydrologic Data for Urban Studies in San Antonio, TX metro area
Presented in these reports is a compilation of hydrologic data for various water years.

Flood Protection Plan for Portions of Salado, Cibolo & Leon Creeks
This report was sponsored by the Bexar County Public Works with a matching grant from the Texas Water Development Board. The purpose of this report was to develop a flood protection plan for segments of the Leon, Cibolo and Salado Creeks.

Review of Existing Studies

The studies generally were engineering backwater analyses of stretches of a particular creek. These studies where mostly calculations with very little text and were completed to support floodplain improvements or development activities.

The methodology used in the reviewed studies varied. Studies performed from the early 80's on were performed on a computer system, typically using HEC II (the industry standard backwater stream analysis program). Prior to the early 80's, some studies were performed on computer, some by hand and some a combination of both. Most of the studies are small stretches of the creek.

Many of the studies had historical significance in that they gave a "snapshot" of a particular reach of a creek at a point in time. Some of the information in these studies is no longer relevant due to changes in the development of the watershed and/or changes in the creek morphology.

In our review we found that the reports all used the same hydrologic parameters to base the analysis on. The Rational method is used to calculate discharges for drainage area that are less than 2000 acres. The Rational method is based upon drainage area, a cover factor and the rainfall intensities (in inches per hour). The rainfall intensities were developed by the City's drainage department in the early 70's. For areas large than 2000 acres a graph relating drainage area to discharge (DA vs Q) is used. The DA vs Q graph was also developed in the early 70's by the City's drainage department.

A listing of the existing studies reviewed is on the following page in Table II-9 and illustrated on Figure II-3. The index numbers shown on this table correspond to those shown on Figure II-3.

Table II-9
INDEX OF EXISTING STUDIES

Index	Waterway	Subdivision or Project	Engineer	Date
1	Leon	Pablo Grove, CSA landfill	Jay Aldean	90
2	Leon	Pablo Grove, CSA landfill	Jay Aldean	72-74
3	Leon	Brown Leaf	P D	87
4	Leon	Pin Oak	MBC	73
5	Leon	West Wood Park	P D	69
6	Leon	Hwy 151	TxDOT	
7	Leon	SW Research	P D	85
8	Leon	West Park	P D	83
9	Leon	Twin Creek	Vickrey	72
10	Leon	Ingram Square	Bob Opitz	79
11	Leon	Timber Creek Estates	Vickrey	79
12	Leon	Ingram Plaza	Brown	80
13	Leon	Parkwood	WF Castella	85
14	Leon	One North Place	Bain	73
15	Leon	Babcock Place		early 70's
16	Leon	Alamo Farmstead	WF Castella	82
17	Leon	French Creek Village	PD	74
18	Leon	Wildwood	WF Castella	76-85
19	Leon	Prue Road Bridge	Mike Cude	91
20	Leon	Quail Creek	Mike Cude	84
21	Leon	Heath Road	CEC	87
22	Leon	Fiesta Tx	P D	
23	Leon	Dominion	P D	83
24	Leon	IH10 Boerne Stage Road	Overby Descamps	
25	Culebra	Pipers Meadow	D R Frazier	74, 80
26	Culebra	Village	Brown	87
27	Culebra	Great Northwest unit 2	Vickrey	77-on
28	Culebra	Culebra Bridge	TxDOT	
29	Culebra	Culebra Bridge	TxDOT	
30	Culebra	Hidden Meadows	Glen Galbraith/ Cude	83
31	Culebra	Loop 1604	TxDOT	
32	Helotes	NW Crossing	MBC	86, 87
33	Helotes	New Territories	MBC	78
34	Helotes	Loop 1604	TxDOT	
35	Helotes	Hidden Meadows	Glen Galbraith	
36	French	Quail Creek	Mike Cude	
37	French	Wildwood		
38	French	Concord	Mike Cude	
39	French	Loop 1604	TxDOT	

Table II-9 (continued)
INDEX OF EXISTING STUDIES

Index	Waterway	Subdivision or Project	Engineer	Date
40	French	N. of Loop 1604	MBI	
41	French	NW Bus Park	Tom Flores	88
42	French	Cedar Springs	SEDA	87
43	Huesta	Hunters Chase	Rosin Kroesche	83 - 86
44	Huesta	North Hills Village	Brown	
45	Huesta	N. of Loop 1604	TxDOT	
46	Maverick	Loop 1604	P D	
47	Maverick	North Hills Village	Brown	

NOTE: The index number corresponds to those shown on Figure II-3.

Watershed Mapping

The Mapping Section of the City of San Antonio Department of Public Works has developed extensive mapping of the city on the Intergraph computer system. The work performed in this study will be in the Intergraph format and will be compatible in layers, colors and other program parameters.

The existing files that are referenced include:

Bexar County limits
Watershed limits (developed and labeled by SAWS)
City Streets
Street names
Railroads
State and Federal Highways
Creeks
Edwards Aquifer Recharge Limits

Site Reconnaissance

During the initial site reconnaissance, all street crossings of the creeks within the detailed study area were visited and photographs were taken. A list of these sites is shown in Table II-10 and illustrated on Figure II-4. The site numbers shown on Table II-10 correspond to those shown on Figure II-4.

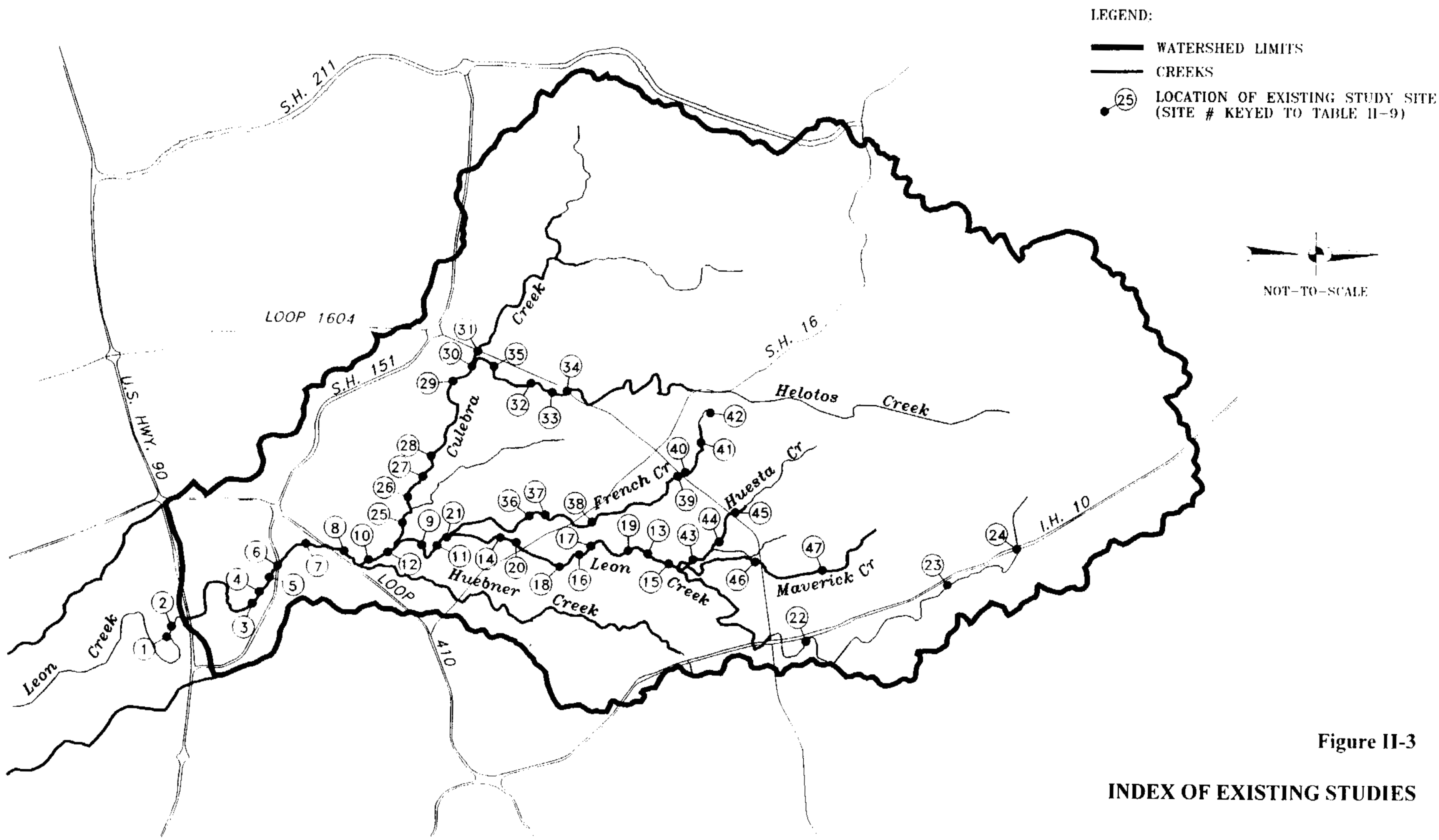


Figure II-3

INDEX OF EXISTING STUDIES

Table II-10
STREET CROSSINGS WITHIN THE DETAILED STUDY AREA

Site #	Creek	Location
1	Leon	Highway 90
2	Leon	Old Highway 90 St
3	Leon	Arvil Avenue
4	Leon (Proposed Crossing)	Shady Grove Drive
5	Leon	Pinn Road
6	Leon	Highway 151
7	Leon	Commerce Street
8	Leon	Loop 410 NW
9	Leon	Culebra Road
10	Leon	Ingram Road
11	Leon	Grissom Road
12	Leon	Bandera Road
13	Leon	Babcock
14	Leon	Hausman
15	Leon	UTSA BLVD.
16	Leon	Loop 1604
17	Culebra	Old Grissom Road
18	Culebra	Timber Path
19	Culebra	Culebra
20	Culebra	Culebra
21	Culebra	Loop 1604
22	Culebra	Stuebing
23	Culebra	Galm
24	French	Mainland
25	French	Guilbeau
26	French	Bandera
27	French	Prue road
28	French	Hausman
29	French	Loop 1604
30	French	Leslie Road
31	Huebner	Ingram Road
32	Huebner	Timber Hill
33	Huebner	Bandera
34	Huebner	Evers
35	Huebner	Huebner Road
36	Huebner	Eckhert Road
37	Huebner	Babcock
37.5	W. Huebner	Eckhert Road
38	W. Huebner	Hollyhock

Table II-10 (continued)
STREET CROSSINGS WITHIN THE DETAILED STUDY AREA

39	W. Huebner	Babcock
40	W. Huebner	Lockhill road
41	W. Huebner	White Bonnet
42	W. Huebner	Prue Road
43	Huesta	Babcock
44	Huesta	Danvers Road
45	Huesta	Hausman
46	Huesta	Loop 1604
47	Maverick	UTSA Blvd.
48	Maverick	Bartlett Cocke
49	Maverick	Loop 1604
50	Helotes	Loop 1604
51	Helotes	Leslie Road
52	Helotes	Leslie Road
53	Helotes	Braun Road

NOTE: The Site #'s correspond to those shown on Figure II-4 and to the photographs in the Appendix. There is no photograph for site #28.

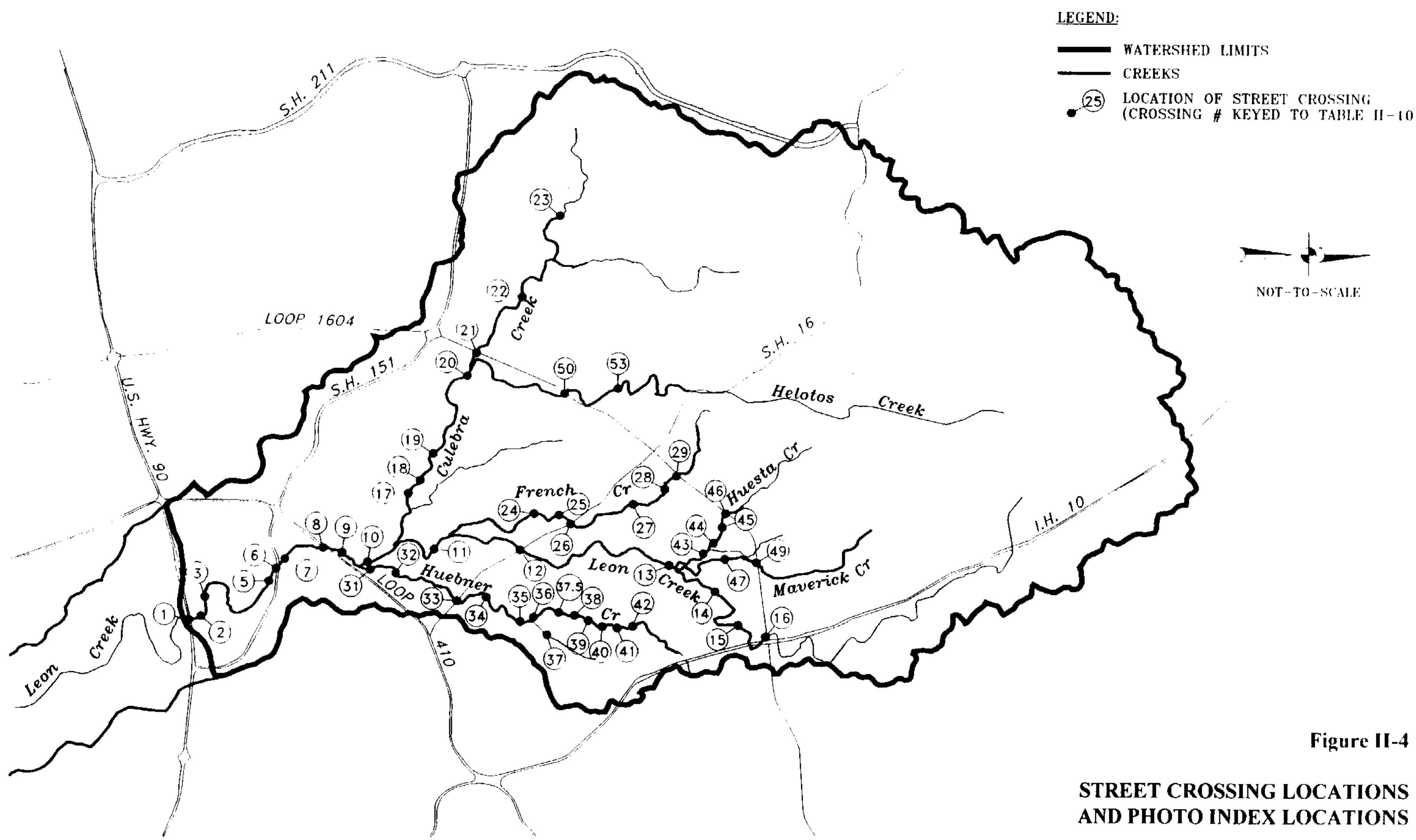


Figure II-4

STREET CROSSING LOCATIONS AND PHOTO INDEX LOCATIONS

HYDROLOGIC FORECASTING ISSUES

One of the objectives of this study is to produce a drainage master plan that establishes standards for design procedures to be followed in the future. In order to accomplish this goal, careful attention must be given to the hydrologic modeling techniques or procedures used to develop the detailed flood plain delineation. Therefore, it is appropriate to review the existing requirements and practices used in San Antonio and explore the options available for use as future design standards. This information can then be considered by the City of San Antonio and used to develop and implement design standards for future drainage projects and development. The procedures used to develop the detailed flood plain study included in the three watershed studies should also satisfy the requirements established by the Corps of Engineers for the FEMA flood study program.

The hydrologic forecasting issues addressed in this report focus on quantitative hydrology methodologies and modeling rather than hydraulic modeling. Methods of hydraulic computations and modeling are much more standardized and better understood by the engineering community. The FEMA Flood Insurance Program, administered by the Corps of Engineers, recognizes the HEC-2 Water Surface Profiles computer program as the standard tool for calculating water surface profiles. There is no reason to consider changing the methodology used in calculating water surface profiles except in specific cases where the hydraulic parameters being modeled are to complex for HEC-2.

Hydrology - Existing Practice

For subdivisions and bond projects, the Rational Method is used for watershed areas up to 2,000 acres. The SCS unit and storm hydrographs with City of San Antonio hytetographs derived from City Intensity Curves are used for watershed areas exceeding 2,000 acres. For some large streams, the U.S. Corps of Engineers Snyder's Synthetic unit hydrograph is used with the City's hytetographs to develop storm hydrographs at various points on stream. SCS routing methods are used through existing and proposed SCS dams in the area to be consistent with the design of these structures.

Rainfall Analysis

Rainfall values in the form of Intensity-Frequency-Duration Curves for San Antonio were first developed in 1920 by Metcaf & Eddy Consulting Engineers. Terrell Bartlett Engineers of San Antonio updated the intensities in 1945. Robert B. Hahn, City Drainage Engineer updated the intensities with a Gumbles Analysis from rainfall records from 1903 through 1972 in February 1973. An additional modification to this update was accomplished in 1979 to apply the results of the NWS's Hydro35 publication to the first 2 hours of intensities of the TP-40 publication. This modification did not update rainfall records through 1979.

Research of rainfall data from the NWS indicates that annual rainfall has increased since records were kept beginning in the 1885. A straight line approximation of the nine year moving average of annual rainfall indicates a definite upward trend in total annual rainfall. Based on this information, the rainfall intensity-frequency-duration curves for San Antonio were updated to include the time period from 1972 to the present. Then the Hydro35 publication techniques were used to modify the first two hours of intensities. The updated rainfall intensity curves were submitted to the City in a separate report titled "Statistical Analysis of Rainfall Records for San Antonio, Bexar County, Texas", dated August, 1994. More discussions with the NWS should also be considered so that weather trends can be identified and used in the decision process for future revisions or updates to the City's intensity curves. Another point to consider is the regional setting of the Leon Watershed. When the watershed area is considered and not a small area within the watershed, it becomes important to consider the inclusion of rainfall data from other official NWS stations such as Boerne or Rio Medina.

Analysis of Runoff

Most analysis of runoff are based on a "design storm" approach with time of concentration, frequency, runoff coefficient or infiltration rates for the various methods described above. A history of the actual runoff from actual storm events on various watersheds have been performed through the years by the U.S. Geological Survey from data gathered at local gaging stations and can be obtained by interviewing people who have witnessed actual flood events. It would be prudent to calibrate or check the hydraulic and hydrology model to actual flood events where possible. This would provide a level of comfort to the flood forecasting effort.

Available Computer Simulation Models

The HEC-1 computer program can calculate various hydrograph models including the Clark, Snyders, time area and SCS or the user can input his or her own hydrograph. HEC-1 is also capable of flood routing with several methods and combining storm hydrographs. SCS Curve Numbers can also be used with HEC-1.

The SCS TR-20 curvilinear unit hydrograph method is almost universally accepted for most watershed analysis. The methodology used in this model allows for a very flexible and realistic method of predicting the ratio of runoff to total rainfall by means of the SCS Curve Number (CN) which takes into account land management or development, soil types, slopes and vegetative cover. TR-20 will allow the user to input any rainfall distribution for hydrograph development and rating curves for routing purposes. Flood routing is accomplished by the modified attenuation-kinematic procedure.

SCS TR-55 is a quick method obtaining the peak flow and hydrographs for small Urbanized Watersheds. This method is not as accurate as the TR-20 Method.

The HEC-1 or the TR-20 computer models are the most flexible, widely recognized, and powerful tools for estimating peak flows and volumes of storm runoff. Either of these models would be well suited to the watersheds found in Bexar and surrounding Counties. A less cumbersome method such as TR-55 or the Rational Formula should continue to be used for small watersheds. These models (TR-20, HEC-1 and TR-55) work with storm volumes as well as storm peaks. This is important since one of the flood mitigation methods that will likely become more prevalent in San Antonio is storm runoff detention and or retention.

The three watershed study teams met regularly under the direction of the City's Project Manager to discuss the various hydrologic forecasting methods and computer models. Each study team calculated storm runoff for various locations in their respective watershed using all of the methods described above. These methods and computer models were evaluated for accuracy by checking the results against observed high water marks, gauging station data, previous hydrology studies and against each other method to check the sensitivity of each respective method.

Once the analysis of computer models and methodology was completed, it was determined that the SCS TR-20 methodology would be combined with the HEC-1 computer program to calculate runoff from design rainfall events. The only variation from the TR-20 methodology was the selection of the Muskingham Routing formula for use is routing storm hydrographs through the watershed.

SECTION III. EXISTING AND ULTIMATE DEVELOPMENT CONDITIONS

INTRODUCTION

As described in the scope of work, computer models were completed to determine the design runoff and resulting water surface elevations for existing and ultimate development conditions of the watershed. Storm frequencies modeled were the 10, 25, 50, 100 and 500 year rainfall events for existing conditions and the 25 and 100 year rainfall events for ultimate development conditions. The calculated water surface elevations have been used to define accurate flood plain limits or boundaries that can be used by the City to update the current FEMA maps. These new flood plain boundaries can also be used with the City's block map database to facilitate management of the flood plains by various City and County Agencies.

The 100 year water surface elevations calculated for existing conditions have been used to identify flooded structures along these creeks. These flooded structures and potential mitigation projects to remove them from the flood plain are presented in Section IV.

HYDROLOGY

Design runoff for existing and ultimate development conditions were computed using the SCS TR-20 methods within the HEC-1 computer simulation model. Based on NWS rainfall and storm event data, antecedent moisture condition II was used in the runoff model. Curve numbers (CN's) were based on soil type and slope as shown below.

Hydrologic Soil Group	SCS Curve Number
A	25
B	55
C	70
D	77

The percent impervious cover was developed from typical impervious cover conditions for the various land use categories as shown in Table III-1. Existing and projected land use was provided by the City of San Antonio's Planning Department. A weighted average CN and percent impervious cover was calculated for each sub-watershed. All of these parameters and their application to each of the three watersheds were discussed and applied consistently by the three study teams. Separate reports were submitted to the City to document the selection of CN values and percent impervious cover. A calibration check was made using various gaging stations throughout the Leon Creek watershed to verify the selection of CN values.

Table III-1
PERCENT IMPERVIOUS COVER

Land Use Category	Average Percent Impervious Cover
Residential	
1/8 acre Garden or Townhouse	65 - 85%
¼ acre Residential Lot	38%
1/3 acre Residential Lot	30%
½ acre Residential Lot	25%
1 acre Residential Lot	20%
Industrial	72 - 85%
Business & Commercial	85- 95%
Densely Developed (apartments)	65 - 85%
Streets, Roads & Parking Areas	98%

The SCS standard 24 hour storm distribution was used with the City's updated rainfall intensity values to develop the storm hydrograph. Design rainfall values were reduced for large areas using the depth area rainfall reduction method in accordance with the SCS methodology. The time of concentration for each sub-watershed was calculated based on an overland flow time and a channel flow time. The lag time used for generation of storm hydrographs was calculated as 60% of the time of concentration in accordance with the methodology used.

Hydrograph routing through the watershed was accomplished using the Muskingum method in the HEC-1 computer model. This routing method takes into account the unique characteristics of each creek segment for which a storm hydrograph is routed downstream to the next flow calculation point. By routing the storm hydrograph from its calculation point to the next downstream calculation point, natural storage or detention in the creek channel is accounted for in determination of design flows. Natural channel storage in the Leon Creek basin was found to be insignificant. Therefore, the routing parameters or channel characteristics used for hydrograph routing under ultimate development conditions were the same as those used under existing conditions. The results of the hydrology model are shown in Table III-2.

Table III-2
100 YEAR FREQUENCY
DISCHARGE SUMMARY SHEET

CREEK	CALCULATION POINT NO.	LOCATION	DRAINAGE AREA (Sq. Mi.)	EXISTING CONDITION DISCHARGE (CFS)	ULTIMATE DEVELOPMENT DISCHARGE (CFS)
French	F10	Approximately 1800 L.F. downstream of FM 1560	1.48	3,853	4,414
	F20	Approximately 3800 L.F. downstream of FM 1604	7.46	14,447	16,921
	F30	Approximately 800 L.F. downstream of Guilbeau Road	11.77	17,299	20,193
	F40	Just above junction with Leon Creek	13.87	17,899	20,799
Helotes	HE10	Scenic Loop Road at Wagner Road	13.20	19,758	24,183
	HE20	Approximately 1000 L.F. downstream of S.H. 16	23.93	31,173	37,824
	HE30	At FM 1560	24.42	30,780	37,791
	HE40	AT FM 1604	29.02	30,598	37,243
	HE50	Just above junction with Culebra Creek	33.31	30,352	36,784
Upper	C10	Approximately 10,000 L.F. upstream of Galm Road along the westernmost draw of Upper Culebra Creek	11.51	16,475	19,839
Culebra	C20	Approximately 5500 L.F. upstream of Galm Road along the center draw of Upper Culebra Creek	1.45	3,335	4,003
	C30	Approximately 10,500 L.F. upstream of Galm Road along the easternmost draw of Upper Culebra Creek	1.87	3,893	4,609
Culebra	C40	At Galm Road	17.40	21,779	25,911
	C50	Approximately 7000 L.F. downstream of Galm Road	25.41	28,301	32,833
	C60	Approximately 2000 L.F. downstream of FM 1560	31.22	31,923	36,767
	C70	Approximately 4000 L.F. downstream of FM 1560	36.01	36,306	41,637
	C80	Just below junction with Helotes Creek	72.03	56,891	67,862
	C90	Approximately 4000 L.F. upstream of junction with Leon Creek	80.50	57,303	68,173
	C100	Just above junction with Leon Creek	81.07	57,153	68,005
Huebner	HB10	At Prue Road	2.52	5,529	6,191
	HB20	Approximately 1700 L.F. downstream of Huebner Road	8.20	15,188	17,199
	HB30	Just above junction with Leon Creek	12.20	17,253	19,484
Leon	L10	At FM 1604	39.37	33,162	37,166
	L20	Just below junction with Maverick & Huesta Creeks	54.88	35,394	39,596
	L30	Approximately 1200 L.F. downstream of Prue Road (below junction with Leon Creek Overflow Creek)	57.97	35,618	39,782
	L40	Approximately 2500 L.F. above FM 471 (below junction with French Creek)	75.71	43,219	49,717
	L50	Just below junction with Culebra Creek	157.59	93,198	109,415
	L60	Just below junction with Huebner Creek	170.42	97,780	114,704
	L70	Just below junction with Southwest Research Creek	187.99	99,692	116,669
	L80	At U.S. Highway 90 West (below junction with Southwest Research Creek)	190.23	99,714	116,574
Maverick	MC20	Just above junction with Leon Creek	6.04	11,067	11,961
Huesta	HU30	Just above junction with Leon Creek	5.45	10,457	11,516

HYDRAULICS

Hydraulic calculations were completed using the Corps of Engineers HEC-2 computer program. Cross section data input into the computer model were taken from an aerial topographic map provided by the City. Field elevations were taken at various locations throughout the study area to verify the elevations shown on the topographic maps. Contours on the topographic maps were shown at two (2) foot intervals and the maps were produced at a scale of 1 inch = 200 feet. Other input parameters such as bridges, culverts, low-water crossings and manning's roughness coefficient ("n" value) were determined by a combination of field reconnaissance, inspection of aerial photographs, construction plans and past experience on projects within the watershed. A complete set of hydraulic calculations has been submitted to the City under a separate report.

The Manning's roughness coefficients or n values were determined in accordance with the guidelines established by the three watershed study teams under the direction of the City's Project Manager. A separate report titled "Leon Creek N Value Analysis" was submitted to the City and served as a guide for the selection of N values. Selection of the appropriate N values were made by a combination of visual inspection of the creeks and aerial photographs. Typical N values used in this study are as follows:

Creek Segment Characteristics	Manning's N Value
Concrete lined channel	0.015
Clean, uniform vegetated channel	0.035
Large trees with little or no underbrush or deep flow depth over dense growth	0.050 - 0.055
Dense growth in overbank areas	0.060 - 0.090

Results of the 100 year existing condition water surface profiles indicated that the flow was generally confined to areas defined as being within the existing flood plain. There were isolated incidents of illegal fill encroachment into the flood plain that created wider flood plains than previously defined and areas in which development occurred outside the influence of the City's Flood Plain Ordinance. Exhibits of the existing condition flood plain for the 10, 25, 50, 100 and 500 year storm event can be found in the exhibits section of this report.

SECTION IV. RECOMMENDED MITIGATION PROJECTS

IDENTIFICATION OF PROBLEM AREAS

As part of the task of developing the Leon Creek Master Drainage Plan, this study identifies and prioritizes specific projects which will mitigate potential flood hazards. The project team utilized HEC-2 floodplain models to identify 78 specific areas where the 100 year flood presents a flooding hazard based on the existing watershed conditions. Several other potential problem areas were initially considered, but were eliminated based on more detailed analysis or are being addressed by TxDOT or other agency projects or programs. Exhibit MP-1 contained in the Exhibits section of this report shows the location of each of the 78 flood mitigation projects. For each of the problem areas a specific capital improvement project has been identified to mitigate the potentially dangerous flooding condition.

Generally, the problem areas can be categorized into three types: inundated roadways or bridges, areas where building structures flood, and a public park. Analysis and modeling of the floodplain shows that the 100 year flood peak discharge increases only very slightly under ultimate development conditions compared with that under existing conditions. Moreover, an element of the Master Plan provides for management practices which may require developers to take measures to accommodate their own discharge in future projects. Therefore, the project recommendation is based on models simulating only the existing extent of development. Appendix "B" contains Tables 1.1 - 1.7 summarizing these problem areas by stream. Figures 1.1 - 1.7 show the problem areas located on project location maps in the Appendix.

Definition of "Base" and "Fringe" Projects

Of the 78 flooded areas identified in the Leon Creek Watershed, 70 are definitely inundated by the 100 year flood. Projects in these areas are labeled as "base" projects and include all of the inundated roadway/bridge areas, approximately ninety percent of the building structures, and the park. The remaining eight sites, including the remainder of the structures, appear to be near the edge of the 100 year floodplain and may actually be outside the limits of it. Projects mitigating flooding of the inundated structures in these areas are labeled as "fringe" projects. Fringe projects will require a survey of finished floor elevation to determine their actual disposition. The fringe structures found to be in the 100 year floodplain would then be included as candidates for mitigation projects.

Project Selection

The criterion for the selection of sites for specific projects is that the 100 year flood presents a potential for damage to persons or property at the site. More specifically, the peak water surface elevation is at least as high as the pavement surface at roadways or the top of the foundations of structures. Floodwaters even a few inches above this critical elevation present safety concerns at low water crossings due to the possibility of a motorist being stranded within or swept away by flood waters. The potential for loss of life at these locations is a very real concern. Any flooding of structures presents concern for property damage and economic adversity, while more severe cases threaten the lives of inhabitants.

The problem areas are interrelated as parts of the overall watershed system; thus, in some cases one project may reclaim more than one problem area. Also, projects such as detention/retention ponds could lower peak water surface elevations, potentially decreasing flooding in multiple problem areas. Nevertheless, in the majority of cases, a single project has been selected for each problem area. Each area has been analyzed independently to arrive at the most economical method of solution for the specific site. Solutions for the problem areas employ several different strategies which are described in greater detail in the following paragraphs. 78 specific projects are recommended using selected strategies based on the characteristics of the area. Table IV-1 summarizes the recommended projects and their costs. A more detailed summary of the projects and estimated costs is included in Table 2 in the Appendix.

Funding of the projects may be borne in large part by the citizens of San Antonio in Bexar County. Additional funds may be sought from sources such as federal, state and local roadway and drainage programs, other municipalities, and in some instances, private property owners. Funding strategies are discussed in detail under a separate report entitled "Funding Strategies for Drainage Improvements" developed for the City of San Antonio Public Works Department. Table 3 in the Appendix gives a basic summary of how the cost of the 78 mitigation projects might be distributed among the responsible administrative agencies.

Priority System and Cost Benefit Ratio

Each project is given a high, moderate, or low priority based its potential to reduce flooding damages to the community. Tables 4.1a - 5.7c in the Appendix summarize projects by priority for each stream in the Leon Creek Watershed. The cost benefit ratio is one indicator of a project's value, but this ratio must be understood and applied appropriately. The benefit evaluation is estimated differently for roadway/bridge and structure protection projects. Therefore, cost benefit ratios can only be compared among roadway/bridge projects or among structure protection projects. Grouping cost benefit ratios for roadway/bridge and structure protection projects together would not be meaningful in this study.

TABLE IV-1
LEON CREEK WATERSHED
RECOMMENDED PROJECTS SCENARIO

MITIGATION PROJECTS	PROJECT DESCRIPTION	COST
70 BASE PROJECTS (318 - Structures)	1 - PUBLIC PARK (Signs and Gates)	$50,000
	46 - ROADWAYS/BRIDGES	$32,758,000
	4 - FLOODWALLS (6 - Structures)	$1,320,000
	6 - LEVEES (16 - Structures & 3 - Roadways)	$427,000
	7 - BUYOUTS (32 - Structures)	$4,593,000
	4 CHANNEL IMPROVEMENTS (264 - Structures & 2 - Roadways)	$18,390,000
	TOTAL BASE PROJECTS COST	**$57,538,000**
8 FRINGE* PROJECTS (30 - Structures)	5 - LEVEES (17 - Structures)	$205,000
	2 - BUYOUTS (12 - Structures)	$1,185,000
	1 - FLOODWALL (1 - Structure)	$152,000
	TOTAL FRINGE* PROJECTS COST	**$1,542,000**

TOTAL COST OF 78 RECOMMENDED PROJECTS **$59,080,000**

*Fringe projects include those projects near the edge of the flood plain which require detailed survey information to determine if they in fact are affected by the 100 year event.

At roadways, a project's real benefit involves public safety as well as tangible property. Quantifying such benefits requires subjective judgment. Therefore the estimation of benefits is based on the project's ability to protect the public, relative to the other roadway projects in the study. Benefits are assigned at $1 million, $1.5 million, or $2 million, depending on daily traffic using the crossing.

For projects protecting structures, the benefit associated with each project has been quantified based on the real value of the structures only. No evaluation has been made for the potential inconvenience, injury or loss of life associated with the flooding of structures.

DESCRIPTION OF MITIGATION PROJECTS

This section defines and describes the different types of solutions suggested to mitigate flooding in areas in the Leon Creek Watershed. Generally, the solutions may be grouped into two conceptual categories. One strategy is to relocate the facility away from the reaches of floodwaters. At roadways, this goal is accomplished through bridge improvements or through raising the roadway and providing a culvert for cross drainage as necessary. Occasionally the purchase and demolition of an inundated structure is the most economical means of removing such a hazard, in lieu of constructing significant infrastructure to protect it. The second strategy is to improve upon the capacity or direction of the floodwater conveyance. This method may employ channel improvements, levees, or floodwalls. At roadways, the improvement of bridges or culverts causing constrictions may accomplish the desired effect. A third strategy, which is explored in this chapter under the heading Special Projects, is to lower the discharge, and water surface elevation, using detention or recharge ponds.

Bridges

Among the inundated roadway/bridge areas, recommendations include 46 new or lengthened bridges or culverts. Two TxDOT funded bridges (Projects HEL-4 and C-5A) have been omitted from the scenario of projects because they are already programmed for construction by TxDOT.

The total estimated cost for each new bridge includes a concrete bridge structure and roadway approaches (fill and paving). Calculations have been performed to estimate the cost of construction for each bridge. First, the discharge and depth of flow are obtained under existing conditions from the HEC-2 models for all bridges. A velocity of 10 feet per second is assumed for the stream through the bridge. Dividing discharge by velocity yields an approximation of the required area for the bridge opening. Dividing the required bridge opening area by the depth yields an approximation of the required bridge length for a rectangular opening. Finally, adding twice the depth accounts for assumed 2:1 abutment slopes. The resulting calculated bridge length is increased to account for any skew to the channel, then is rounded up to the next even 10 foot interval. The bridge width is obtained by scaling the existing bridge widths from mapping or is based on known future improvements. Multiplying the bridge width by the bridge length yields the total bridge deck surface area. The bridge cost is estimated using a unit price of $40 per square foot of bridge deck surface. This unit price is based on past contracts and bid tabulations for standard pier supported, concrete bridges.

Roadway embankment cost is estimated using the roadway length, roadway width and depth of embankment. The roadway length is determined by subtracting the calculated bridge length from the overall floodplain width. The depth of embankment is ascertained from the mapping based on the average amount of fill required to elevate the roadway above the floodplain. The fill volume and area of approach pavement is then calculated and rounded up to the next even 100 cubic yard and 100 square yard intervals, respectively. Using unit prices of $8 per cubic yard for embankment and $20 per square yard for asphalt paving, the fill and paving costs are computed. The estimated total bridge construction cost is the sum of the bridge cost, approach paving cost and embankment fill cost rounded up to the next even $1,000 interval.

Three of the roadway/bridge projects identified consist of raising the roadway to prevent inundation of the roadway during the 100 year storm. All three projects require construction of a cross drain culvert as a part of the solution. The culvert size and cost is estimated similarly to that described for bridges, with the same unit price of $40 per square foot of deck surface. Project cost for raising the roadway is estimated similarly to that described for approaches to bridges.

The 46 base roadway/bridge projects recommended to provide safe passage on roadways during the 100 year storm range in project costs from $64,000 to $2,713,000. The total cost of the roadway/bridge improvements was estimated at $32,758,000. Federal, state, and local roadway and drainage funds could potentially be applied toward this total. In fact, 7 of these projects are already listed on the MPO Long Range Plan. Two additional projects are partially funded under the City's Capital Improvement Plan through the 1994 bond program. Thus, funding amounting to over $3,000,000 is already programmed. The remaining projects potentially could be included in these established roadway improvement programs.

Levees

A levee may best be defined as an earthen dam used to divert a channel without retaining the flows. Levees are best suited for those areas with wide, flat overbanks. They are not practical in areas with steep banks due to the large amount of fill required. Floodwalls are best suited for those areas with steep banks, where levees are not practical. Levee construction is generally less expensive than channel improvements or floodwalls if the proposed site is flat and the water surface profile has adequate slope to allow outfall behind the levee.

Recommendations include six base levee projects which mitigate flooding at three low water crossings and protect 16 building structures. Two of the projects (M-2 and M-3) are already listed as roadway improvements on the MPO Long Range Plan. Since the construction of levees is significantly less expensive than raising the roadway at these sites, consideration should be given to redirecting those MPO funds and incorporating levees into a more efficient solution for these two problem areas. Also identified are five additional fringe levee projects which may be required to protect 17 fringe structures if survey data proves these structures to be in the floodplain.

The levee project costs include the cost of fill and stabilization. Calculations have been performed to estimate the cost of construction for each levee. First, the length and height are estimated based on the existing conditions using HEC-2 models. The levees start upstream of the point where water flows to inundate a structure. They continue downstream to a point where the drainage behind the levee can outfall based on the water surface elevation computed in the model. The levees provide for three feet of freeboard in accordance with FEMA standards. The width is based on three to one side slopes and a 10 foot wide top. The fill volume is calculated and rounded up to the next even 100 cubic yard interval. The estimated area of stabilization is rounded up to the next even one acre increment. Using unit prices of $10 per cubic yard for embankment and $5,500 per acre for stabilization, the levee cost is computed.

The base levee projects range in cost from $26,000 to $211,000. The total cost of the six base levee projects is estimated at $427,000. The five fringe levee projects range in cost from $26,000 to $56,000. The total cost of the fringe levee projects is estimated at $205,000.

Floodwalls

A floodwall may best be defined as a reinforced concrete wall founded on a footing and used to divert a channel without retaining the flows. Improved aesthetic treatments to the wall such as construction of a top rail or colored stamped concrete is assumed in the total cost estimated. Adequate slope in the water surface profile is required to allow the drainage behind the floodwall to outfall.

Since floodwalls are generally more costly than levees per unit foot, they are proposed only in areas where the ground slope is too steep for levee construction. For example, in an area where the existing side slope is steeper than 3:1, a levee with a proposed side slope of 3:1 would not tie back into the existing slope until it reaches the bottom of the channel.

The estimated floodwall cost includes the cost of concrete. Calculations have been performed to estimate the cost of construction for each floodwall. First, the length and height are estimated based on the existing conditions using HEC-2 models. The floodwalls start upstream of the point where water flows to inundate a structure. They continue downstream to a point where the drainage behind the floodwall can outfall based on the water surface elevation computed in the model. The floodwalls provide for three feet of freeboard in accordance with FEMA standards. The wall width is assumed to be 1 foot. The footing is as wide as the wall is high. The calculated concrete volume is rounded up to the next even 10 cubic yard interval. Using a unit price of $400 per cubic yard for concrete, the floodwall cost is computed.

Four base floodwall projects are identified to protect six structures. Individual base floodwall project costs range from $100,000 to $720,000. Total cost of all four base floodwall projects is estimated to be $1,320,000.

One identified fringe floodwall project may be required to protect one fringe structure if survey data proves this structure to be in the floodplain. The total cost of this fringe floodwall project is estimated at $152,000.

Channel Improvements

Channel improvements are proposed in the areas where it is realistic to protect structures or roadways from inundation, but levees or floodwalls will not suffice. Grass lined channels with 3:1 side slopes are initially sized. However, in several areas the available width is inadequate or the velocity too high for a grass lined channel. A 2:1 side slope concrete lined channel is proposed in these areas. Channel areas where the flowline is lowered require a concrete lined drop structure. The concrete lined channelization projects are particularly expensive since only full concrete channelization of the stream is considered. The potential exists in some areas to use a relief or pilot channel rather than full concrete channelization.

The preliminary sizes of the proposed channels are based on Manning's equation using the existing discharge in the stream. Several sections taken at each site are used to estimate the approximate amount of excavation required to construct the channel.

The total channelization cost includes the cost of excavation, disposal, and concrete riprap (if required). Calculations have been performed to estimate the cost of construction for each channel. First, the length and depth are estimated based on the existing conditions using HEC-2 models. The calculated excavation volume is rounded up to the next even 1000 cubic yard interval. Using unit prices of $8 per cubic yard of excavation, $3 per cubic yard of disposal, and $30 per square yard of concrete riprap, the channel cost is computed.

Recommendations include six base channelization projects to protect 264 structures and two low-lying roadways. The base project costs range from $143,000 to $10,472,000. Project HB-9A, for which $10,472,000 is estimated to protect 167 structures, is under the jurisdiction of the City of Leon Valley. In addition, reimbursement of costs for Projects LC-5 and C-7A could be sought from the property owners who placed illegal fills in these areas. The total base channelization projects estimated cost of $18,390,000 could be substantially reduced if these other funding sources are considered.

Purchases

Structures are threatened by the 100 year flood in nine problem areas where either it is not reasonable to protect the structures or it would be less expensive to purchase the property than to make improvements to protect it. The cost of purchasing structures is estimated at $75 per square foot.

Of these nine areas, seven are base projects containing 32 structures of various sizes. The approximate costs of the base purchase projects range from $120,000 (for the single structure in Project C-8E), to $1,260,000 (for the seven structures in Project HEL-3D). The total cost of the base purchases is estimated at $4,593,000. The remaining two fringe project areas contain 12 structures of various sizes which may have to be purchased if survey data shows that they are in the floodplain. The approximate costs of the fringe purchase projects range from $210,000 (for the two structures in Project C-7B), to $975,000 (for the ten structures in Project C-5C). The total cost of the fringe purchases is estimated at $1,185,000.

Additional Projects

Project LC-17 involves installing flood warning signs and gates in Rodriquez Park to reduce the risk of loss when the park is flooded. The estimated cost of this base project is $50,000.

SPECIAL PROJECTS

The base and fringe projects identified in this study have been selected to target specific flood-prone sites. In addition to these point remedies, this comprehensive Master Plan also considers five regional detention facilities and four potential retention ponds to collect and manage flows. Locations of these five detention and four retention facilities are shown on Exhibit MP-1. Innovative use of these water features could also provide a focal point for recreational areas, or could be linked with other water resource management strategies, such as SAWS water reuse plans.

Although benefit of the detention/retention pond projects is that they may significantly reduce the number and/or magnitude of the base mitigation projects identified. These benefits are not included in the recommended project scenario. Further detailed analysis is required to determine the potential benefits of these ponds.

Detention Ponds

A detention pond may be described as a basin placed adjacent to a channel for the purpose of detaining excess flows. The advantage of using such facilities is twofold: it shaves off the peak water surface elevation at critical points along the drainage system, and it creates assets in the form of stormwater-filled basins. These projects could possibly serve as "runoff banks" for developers who prefer to pay an impact fee to support the projects in lieu of detaining runoff on their own site. Regional detention facilities are very beneficial for small high density properties where there is no practical method of detending runoff onsite. These off-channel detention basins would begin to fill when the channel water surface elevation exceeds the level of a spillway. The basin could be lined or unlined, depending its purpose within the overall stormwater management strategy. For example, a drained basin could begin to discharge slowly back into the channel immediately after the peak. This basin would be dry most of the time, creating an ideal setting for recreational land such as athletic fields. Alternatively, a lined basin could be used to contain the runoff for a longer period, allowing stormwater to be mixed with SAWS reuse water and distributed to users. Wet or dry, the basin could be used in conjunction with scenic parkland projects. Two of the ponds identified (Projects P-2 and P-3) are relatively close together and could be connected with a linear park and scenic hike and bike path. All of these detention sites are located in abandoned quarries which provides an opportunity to reclaim these unsightly areas in an aesthetically pleasing way.

The total project cost for each pond includes the cost of land acquisition at the unit cost of $2000 per acre, excavation at $6 per cubic yard, disposal and fill at $3 per cubic yard, and concrete riprap at $30 per square yard. Five potential detention pond projects are identified with costs ranging from $1,334,000 to $12,230,000. The total cost of the detention pond projects is $25,138,000. Without subsurface investigation, it is difficult to estimate the magnitude of rock excavation. Also, disposal costs could vary depending on the actual distance to the disposal site.

Leon Creek flood profiles are shown at the back of the Appendix. The preliminary hydraulic analysis of Leon Creek with all five detention ponds modeled shows that the water surface elevation at the downstream reach of Leon Creek is lowered by approximately two feet. This change does not remove any of the identified problem areas along Leon Creek, or its tributaries, from the floodplain. However, the floodplain limits for 11 sites would be reduced significantly enough to decrease the overall cost of the projects identified to protect or improve those sites.

These five detention ponds reduce the peak flow in Leon Creek by approximately 10,000 cfs or roughly 10%. Ultimate development flows calculated for this study show an average increase of approximately 15% over existing condition flows. These detention ponds would be best utilized to offset ultimate development flow increases on a regional basis should the City of San Antonio adopt a new flood plain ordinance that required detention. This would provide a facility that could reduce peak flows from properties being developed that are too small for onsite detention.

Retention Ponds

A retention pond may be described as a basin placed to interrupt a channel such that all of the channel flows are collected in the basin at that point. An outlet structure can allow for required minimum flows to be released to the downstream channel. By retaining the flows at a certain location, all downstream flooding problems are reduced to some extent. Retention ponds have potential additional benefits similar to those of detention ponds. They can be an appealing way to reclaim rock quarries and also have the potential to enhance recharging of the Edwards Aquifer if, of course, they are located over the recharge zone.

Four retention ponds were identified during the course of this study. Three of these retention ponds were modeled as a part of this study to gage the benefit of these retention facilities. The preliminary hydrologic analysis of the Leon Creek watershed with all three ponds modeled shows that only the Government Canyon and the Culebra Retention Ponds are sufficient in size to contain the peak of the 100 year storm. The Vulcan Quarry (Helotes) Retention Pond could be beneficial with more storage volume made available through future mining. Culebra Creek flood profiles are shown at the back of the appendix. With the Government Canyon and Culebra ponds in place, the water surface elevation at the downstream reach of Culebra Creek is lowered by approximately two feet. One other retention pond that should be considered is on Leon Creek in the Redland Quarry. There was not enough information available at the time of this study to assess the beneficial impact of the Redland Quarry site. Another benefit from these retention ponds is recharge to the Edward's Aquifer. All four of these potential retention sites are located over the recharge zone as shown on Exhibit MP-1..

Multi-Functional Concepts

Critical to the feasibility of the detention projects is the ability for these facilities to be multi-functional. Therefore, it is important to examine the other benefits of the five detention projects. One of these possible detention sites (Project P-1) is already being evaluated as a multi-use facility by the City and was not included our evaluation of multi-functional facilities. The basic goal of the Multi-Functional Projects is to design them to have more than one specialized use such as open space, wildlife habitat and/or recreation. There is also a need to increase the number of recreation facilities in the Leon Creek corridor where the growth has been tremendous over the past two decades. These types of muti-use facilities add to the variety of recreation and open space facilities currently available in the Leon Creek corridor as well as enhance the environmental quality and character of typical storm detention facilities.

Though each project will have its unique design, all must share common site planning goals. Each detention facility must be visually pleasing in as many conditions as possible and must be durable to withstand flood situations. Each site should include clear definition of hazardous areas and provide protection from public injury. These sites must also be accessible from more than one direction and every effort should be made to enhance natural features and materials.

Existing Recreational Facilities

The number and variety of existing recreational facilities in the Leon Creek corridor is limited. School properties and public parks with traditional group shelters and picnic sites are the only types of existing recreational areas. None of the recreational sites are linked with dedicated bicycle routes or hike/bike trails in the creek corridor. The following facilities exist within one mile east or west of Leon Creek between Highway 90 and Loop 1604

- Mateo Camargo Park [Highway 90 between Military Drive and South Callaghan Road]
- Rodriguez Park [Old Highway 90 between Military Drive and South
- Callaghan Road]
- Gustafson Stadium [N.W. Loop 410 between Culebra Road and Ingram Road]
- O.P. Schnabel Park [Bandera Road between Old Prue Road and Braun Road]

Proposed Multi-Functional Detention Projects

The proposed projects are distributed along a two and one half mile stretch of Leon Creek . The ultimate program and development of each should be tailored to the type and intensity of adjacent land use. These new projects should not duplicate nearby recreation facilities. The designs should be in harmony with hydraulic characteristics of the adjacent creek . Exhibits of these projects are shown in the exhibits section of this report.

Project P-2 and Project P-3

Project P-2 and Project P-3 are immediately adjacent to two well developed residential neighborhoods. Both sites are approximately one half mile south of O.P. Schnabel Park. Project P-2 includes approximately 140 acres. The existing topography divides the basin into two separate areas. The north is proposed as open space for storm detention area but also includes recreational trails and picnic facilities. Project P-3 covers approximately 140 acres. The northern portion is proposed for storm detention and informal recreation activities such as jogging. The 38 acres at the south are above the existing flood plain. The plan proposes that this area be purchased as part of the mitigation project. Structured recreation activities such as softball and soccer are proposed in this area.

Project P-5

Project P-5 is bordered by open land and a developing residential neighborhood. Project P-5 covers approximately 169 acres. Softball and multipurpose fields are proposed for the northern third of the site. Purchase of land for these uses will be necessary. The central third of the site is planned as storm detention and informal exercise trails. The land which composes the southern third would be acquired to serve as open space above the flood prone area.

Project P-6

Project P-6 is the largest of the proposed Multi-Functional sites at 340 acres. There are no residential neighborhoods in close or direct proximity. The limited access and coarse topography make this site a good candidate as an " Urban Wilderness". The basin area is proposed primarily as nature trails and storm detention. Picnic sites are suggested for the higher elevations. It will be necessary to purchase easements on the northeast and southwest for permanent vehicular access or arrange for access to the site from the City owned Public Works Maintenance Yard adjacent to the site.

Environmental Impact

Leon Creek is in one of the most rapidly developing sectors of San Antonio. Environmental management policies and practices have not kept pace with the intensity of urban growth. Most of the developed land along the corridor turns its back on the creek. The channel is viewed only as convenient place to discard local runoff. Without a master plan and practical conservation practices, the environmental impacts on the creek will affect larger areas of the city. These detention projects must be designed to be compatible with the ecological framework and environmental character of Leon Creek.

Design of these facilities must consider basin scour and slope erosion while providing some filtration of sediment laden stormwater. The filtration of stormwater may also be part of SAWS overall storm water pollution prevention plan for the City as part of the Environmental Protection Agency's mandated stormwater quality program (National Pollution Discharge Elimination System). In order to maintain the functional uses of these facilities, design consideration must be given to controlled release of stormwater, sediment storage and removal, and cleanup of debris deposited during extreme storm flow events.

Environmental enhancement of the Leon Creek corridor may also be achieved by the creation of wildlife habitat within designated areas of the detention pond sites. The presence of natural water flow and location within the flood plain of Leon Creek are factors critical to sustaining a variety of wildlife, especially birds in an urban setting. Careful attention to reclamation of these old quarry areas through planting with a diverse perennial native plant community and planting species that will be compatible with succession and evolution of the creek environment will insure a stable long term natural habitat with low maintenance cost.

Muti-Functional Detention Pond Cost

Cost for adding the multi-use benefits to the detention ponds were estimated based on some generalized assumptions of land use within the detention sites. Depictions of how these sites might be developed were submitted to the City under separate cover. Estimated construction cost include site infrastructure (slope stabilization, site grading, access roads and utilities), facilities (paths, trails, sports fields, shelters and restrooms), emergency and security communications and revegetation (ground cover and trees). The estimated construction cost to enhance the detention projects with muti-functional uses are shown below:

Project	Estimated Construction Cost
P-2	$ 4 million
P-3	$ 4 million
P-5	$ 8 million
P-6	$ 6 million

CONCLUSION

Table IV-1 summarizes 78 recommended projects as a single scenario for the purpose of flood mitigation in the Leon Creek watershed. In addition to the site specific projects, the Master Plan includes five regional detention facilities and four retention/recharge facilities which have immediate value in the role of peak flood abatement, plus multi-faceted advantages in providing for future flexibility in the comprehensive stormwater management scheme.

EXHIBITS

LEON CREEK WATERSHED MASTER DRAINAGE PLAN
CITY OF SAN ANTONIO
PUBLIC WORKS DEPARTMENT
CONTRACT # 95-483-080

Large Scale Maps located in the Official file, may be copied upon request.   November 1996

EXHIBIT MP-1

EXHIBIT LE-1 LEON CREEK – FLOOD PLAIN MAP

EXHIBIT LE- 2 LEON CREEK – FLOOD PLAIN MAP

EXHIBIT LE-3 LEON CREEK FLOOD PLAIN MAP

EXHIBIT LE-4 LEON CREEK – FLOOD PLAIN MAP

EXHIBIT LE-5 LEON CREEK FLOOD PLAIN MAP

EXHIBIT LE-6 LEON CREEK FLOOD PLAIN MAP

EXHIBIT LE-7 LEON CREEK FLOOD PLAIN MAP

EXHIBIT CU-1 CULEBRA CREEK FLOOD PLAIN

EXHIBIT CU-2 CULEBRA CREEK FLOOD PLAIN

EXHIBIT CU-3 CULEBRA CREEK FLOOD PLAIN

EXHIBIT CU-4 CULEBRA CREEK FLOOD PLAIN

EXHIBIT HE-1 HELOTES CREEK FLOOD PLAIN

EXHIBIT HE-2 HELOTES CREEK FLOOD PLAIN

EXHIBIT HE-3 HELOTES CREEK FLOOD PLAIN

EXHIBIT HS-1 HUESTA CREEK FLOOD PLAIN

EXHIBIT MA-1 MAVERICK CREEK FLOOD PLAIN

EXHIBIT MA-2 MAVERICK CREEK FLOOD PLAIN

EXHIBIT FRENCH CREEK FLOOD PLAIN

EXHIBIT FR-2 (1 OF 2) FRENCH CREEK

EXHIBIT FR-2 (2 OF 2) FRENCH CREEK

EXHIBIT HB-1(2 OF 2) HUEBNER CREEK

EXHIBIT HB-1 (1 OF 2) HUEBNER CREEK

EXHIBIT HB-2 (1 OF 2) HUEBNER CREEK

EXHIBIT HB-2 (2 OF 2) HUEBNER CREEK

Please Contact Research and Planning Fund Grants Management Division at (512) 463-7926

SALADO CREEK WATERSHED STUDY AND DRAINAGE MASTER PLAN

Final Report

March 1997

Table of Contents

CHAPTER 5

Summary Phase

List of Tables

List of Figures

List of Appendices

1

Executive Summary

A. Purpose

A drainage study has been performed on the Salado Creek and its major tributaries for the City of San Antonio. The purpose of this study is to provide a sound basis for the development of a master plan for future drainage improvements and development in this watershed. The study was performed in three phases which included the Preliminary, Design, and Summary Report Phases. In the Preliminary Phase existing models, precipitation and stream gage data, recharge zone development plans, dam analyses, and storm flow information gathered, reviewed and assembled. Meetings were held with the various governmental agencies which are affected or have jurisdiction on Salado Creek and its tributaries. A hydrologic model was also prepared which calculates stream flows resulting from rainfall events. The Design Phase of the study included the preparation of a hydraulic model which calculates water surface elevations and flow profiles. Water surface elevations generated by the hydraulic model were used to map the flood plains. In the Summary Report Phase of this study, various mitigation projects were identified which could remove existing structures and developable land from the flood plain and eliminate potentially dangerous flooded roadway crossings.

The Salado Creek Watershed contains an area of approximately one hundred ninety (190) square miles, that was used for the hydrologic analysis. The hydraulic analysis included 55 miles of creeks. The lengths of each creek is as follows:

Creek	Limits of Study	Length
Salado Creek	S.E. Loop 410 to N. Loop 1604	33.6 miles
Panther Springs Creek	Salado Creek to N. Loop 1604	6.0 miles
Mud Creek	Salado Creek to N. Loop 1604	5.5 miles
Elm Creek	Mud Creek to N. Loop 1604	1.5 miles
Elm Waterhole Creek	Elm Creek to N. Loop 1604	2.3 miles
Beitel Creek	Salado Creek to O'Connor Road	6.1 miles
		55 miles

The study limits started downstream of S.E. Loop 410 and extended upstream along Salado and its tributaries to Loop 1604 on the northside of San Antonio. The Watershed's boundaries cross the jurisdictions of Bexar County, The City of San the U.S. Military facilities at Fort Sam Houston, Camp Bullis and Camp and smaller suburban communities including Shavano Park, Hill Country Village, Hollywood Park, Windcrest, and Terrel Hills. Within the Watershed exist thirteen (13) flood control dams.

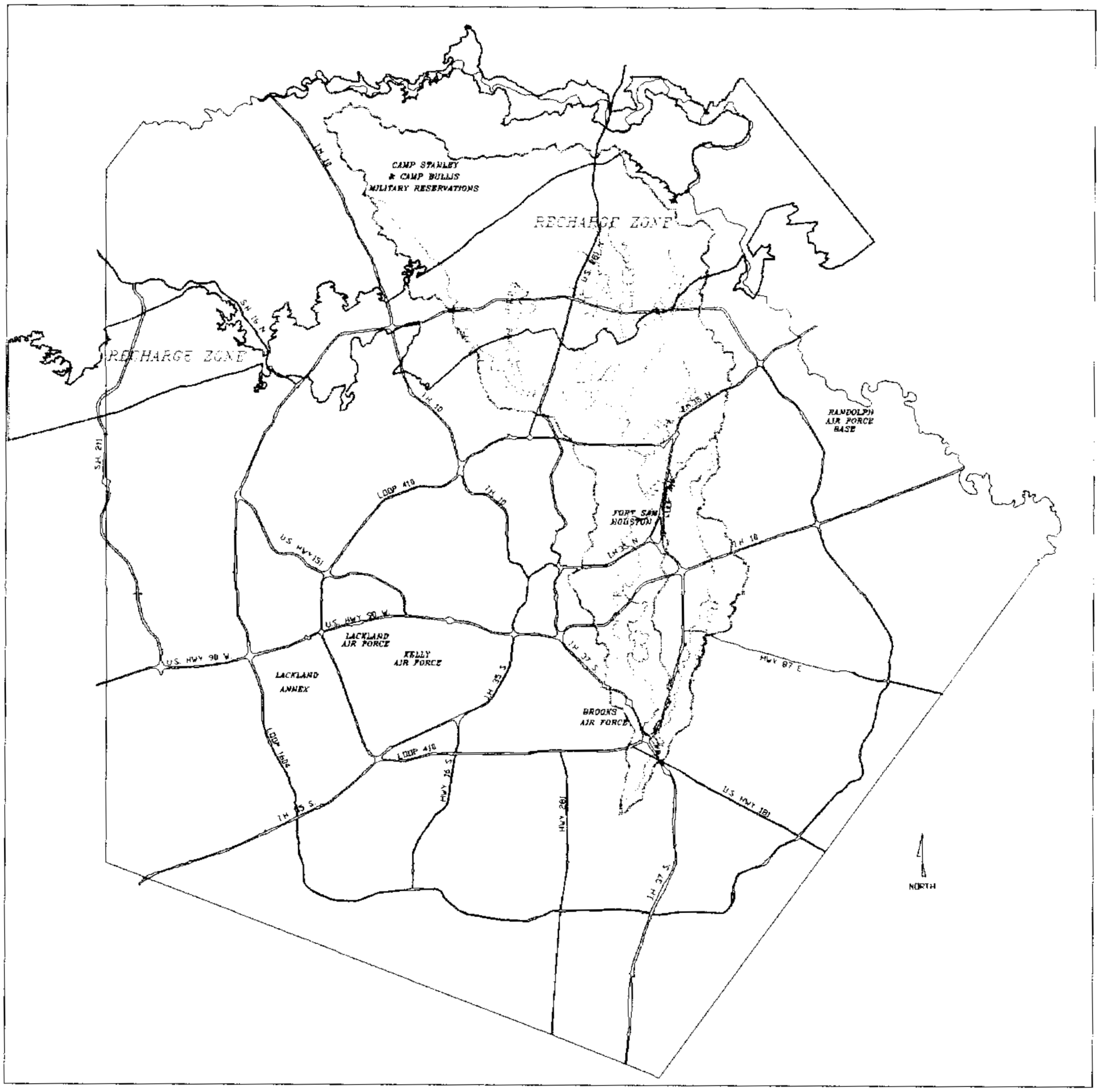


Figure 1 - "Salado Creek Watershed"

B. Preliminary Phase

Gathering data, reviewing existing hydraulic studies, and the hydrologic analyses were tasks performed in the Preliminary Phase. The hydrologic analysis is a process where rainfall data, ground surface conditions, various stream alignments and confluence rainfall data, ground surface conditions, various stream alignments and confluence locations are studied to determine stream flows which result from rainfall accumulations across the watershed. Storm water runoff generated by rainfall is affected by soil type, soil moisture conditions, vegetation, ground slope and impervious cover. Storm water flow within the various streams is also influenced by the existing flood control retarding dams. The Salado Creek Watershed is somewhat unique from the other major watersheds in San Antonio in that thirteen flood control dams exist within the upper watershed which are typically located north of Loop 1604 and within the Edwards Aquifer Recharge Zone. This study confirms that these existing dams provide significant reductions in flooding along the Salado Creek and its tributaries in the San Antonio area.

This drainage study also addressed the affect of current and future development within the Salado Creek Watershed. The source for ultimate development land use projections was the City of San Antonio, Planning Department. Information on land use indicated that approximately thirty eight percent (38%) of the land in the Salado Creek Watershed is vacant and available for development. The Planning Department projected approximately eight five percent (85%) of the undeveloped land area will actually be developed.

Storm water flows were computed for the 10, 25, 50, 100 and 500 year frequency storms within the Salado Creek study area for existing and ultimate development conditions. A comparison of the storm water flows at major road crossings is shown on Table 1. This table indicates the current Federal Emergency Management Agency (F.E.M.A) model, existing conditions model, and ultimate conditions model flows in cubic feet per second (cfs) for the 10, 50, and 100 year frequency storms.

Table 1 - "Comparison of Storm Water Flows"

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Loop 1604		15414	15379		23250	23243		26676	26667
West Ave.	12200	16570	16937	17300	25001	25336	19300	28664	28982
U. S. 281	16700	17209	17622	24000	25735	26123	27000	29441	29813
Wetmore Rd	28600	26873	29435	41600	39650	42132	46600	45227	47681
Nacogdoches Rd.	28600	27673	30383	41600	40793	43476	46600	46528	49204
N.E. Loop 410	30100	28189	31178	44300	41614	44602	49100	47504	50470
Austin Hwy.	36900	32310	35875	54200	47646	51236	60500	54365	57946
Rittiman Rd.	36900	31029	34274	54300	45675	48935	61000	52097	55337
I. H. 35	36900	21900	24089	54300	32147	34408	61000	36656	38922
Commerce St.	36900	20078	22123	54300	29415	31550	61000	33526	35674
Rigsby Ave.	36900	18247	20134	54300	26672	28661	61000	30382	32394
E. Southcross Blvd.	36900	14139	15567	54300	20512	21986	61000	23250	24723
S.E. Military Dr.	36900	14139	15567	54300	20512	21986	61000	23250	24723
S.E. Loop 410	36900	13292	14657	54300	19262	20673	61000	21822	23236

C. Design Phase

The hydraulic analysis performed in the Design Phase is a process where the stream shape or cross section and vegetated condition are considered to determine the depth of storm water flows and the resulting flooded area that is caused by rainfall events. Roadway crossings and other man made improvements tend to create restrictions within the stream bed area which also may impact the depth and the conditions of storm water flow within a stream. The cross-sections and channel slopes used in the study were based on aerial mapping prepared for the Leon, Upper Olmos, and Salado Creek watershed studies by United Aerial Mapping Company and provided by the City of San Antonio. The study also addressed the existing conditions within the creeks related to vegetation and other encroachments such as fill materials and structures. Previous flood study information and stream gage records maintained by the United States Geological Survey were also reviewed and incorporated into the study. Field investigation of the various creeks within the study area was included in the study. Many areas within the floodplains are not accessible because right-of-way or easements do not exist for access and the embankment areas are densely vegetated. The study results show that the Salado Creek between S.E. Loop 410 and N.E. of Loop 410 possesses a unique linear channel storage

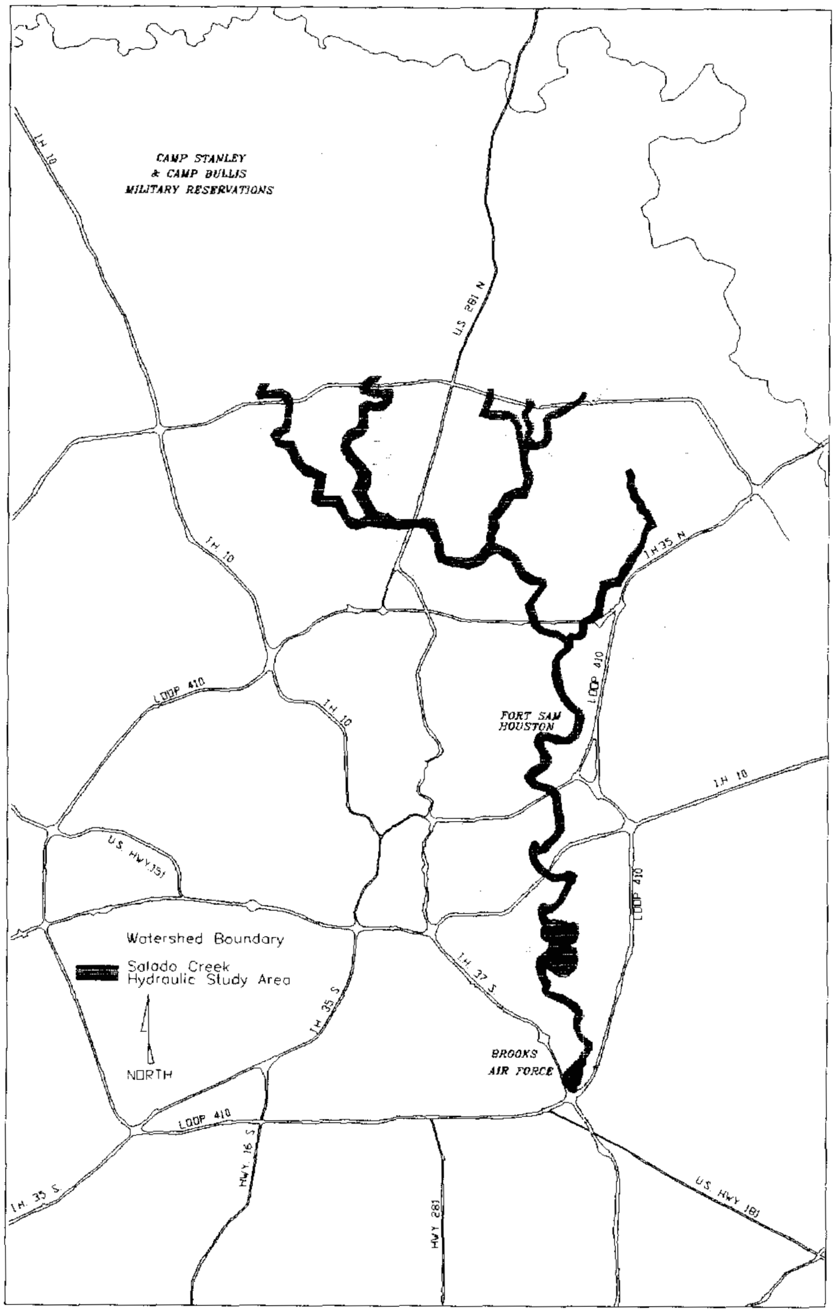


Figure 2 - "Hydraulic Study Area"

accessible because right-of-way or easements do not exist for access and the embankment areas are densely vegetated. The study results show that the Salado Creek between S.E. Loop 410 and N.E. of Loop 410 possesses a unique linear channel storage condition. Linear channel storage (detention) occurs when storm water flows along the banks and outside the banks is slowed down by dense vegetation and flatter slopes. Existing conditions along the lower 20 miles of Salado Creek consist of wide flat stream sections and relatively flat slopes. Storage conditions are increased within these areas by dense vegetation growth within the floodplain areas. This linear storage provides a significant reduction of storm water flows downstream.

D. Summary Phase

Upon completion of the hydrologic and hydraulic models for the Salado Creek Watershed, the floodplains for the 10, 25, 50, 100, and 500 year frequency storms were mapped. In the Summary Phase, mitigation projects were identified for reducing and eliminating flooding of structures and roadways.

Flood prone areas have been identified based on this study. The impact of the 100 year frequency storm and its resultant floodplain on existing structures has been identified. One hundred sixty nine (169) houses and ten (10) apartment buildings are located within the floodplain. Sixty five (65) commercial and industrial type structures are also located within the floodplain with an additional twenty three (23) structures identified as recreational use type facilities. Another sixty eight (68) structures have been identified as barns or sheds. Major areas of flooding for a 100 year storm event exist in the East Park Subdivision (Wheatley Heights) south of Martin Luther King Drive. There are approximately ninety nine (99) residential structures within this area. There are also forty four (44) homes in the Garden Court East and Fairfield Village North Subdivisions and Gemini Drive area. Ten (10) apartment buildings have been found to be in the floodplain within the Renaissance Village North and Villa Apartments. Eighteen (18) commercial and industrial buildings located in the Austin Highway Industrial Subdivision are in the floodplain. A list of the structures located in the floodplain is provided in Chapter 5 of the report. A field survey confirming the floor elevation of these structures has been obtained. Thus, all structures having finished floor elevations above the floodplain are not included in the floodplain. Numerous roadways have been identified in the floodplain. A complete list of roadways crossing the creeks in the study is included in Chapter 3. Roadways with low water crossings have been identified.

Ten (10) projects have been identified for mitigation of the flooding that occurs during the 100 year storm event and nine (9) additional projects have been identified that can eliminate existing flooded roadways. Projects developed for mitigation are listed in Table 2 with a description provided in Chapter 4 of the report. These projects will eliminate the majority of the residential and commercial structural flooding problems that occur during the 100 year storm event. Estimated construction costs are provided, but easement and right-of-way cost have not been included.

Table 2 - "Proposed Mitigation Projects"

Project No.	Project Description	Estimated Costs
1	Flood Control Dam at Site #15r	$ 6,000,000*
2	Remove 5000' of Weidner and 2500' of Old O'Connor Rds., Reroute 1200' of Lookout Rd and enlarge railroad bridge structure	$ 844,750
3	Channelize Beitel Creek, 4000' east of Garden Court East Subdivision (Esm't. Acquisition Cost Not Included)	$ 1,330,737
4	Reroute and raise 4600' of Holbrook Rd. to elevations equal to 25 Year Floodplain	$ 961,226
5	Construct a 4400' long levee from MLK Blvd. to the south between Salado Creek and East Park Subdivision (Wheatley Heights)	$ 458,857
6	Remove brush and small trees to height of 6' along lower 20 miles of Salado Creek (Esm't. Acquisition cost Not Included)	$ 7,418,075+
7	Channelize 600' of Beitel Creek from Vicar to Perrin Beitel and 2000' downstream of Perrin Beitel (Esm't. Acquisition Cost Not Included)	$ 685,726
8	Remove 1900' of Ira Lee from Austin Hwy. northward to limits of floodplain. Remove 600' roadway connection to Holbrook Rd. and reroute 600' of Holbrook Rd.	$ 345,900
9	Clear and channelize 5000' of Salado Creek south of Martin Luther King Drive (Not Recommended)	$ 3,490,725++
10	Clear and channelize 12900' of Salado Creek between Wetmore Road and Jones Maltsberger Road (Not Recommended)	$20,189,400++
TOTAL ESTIMATED COST		**$4,627,196.00**

* Cost not included in Total Estimated Cost (Federally Funded Project)
+ Cost not included in Total Estimated Cost (Project not Recommended)
++ Cost not included in Total Estimated Cost (Project not Recommended)

Several structures exist within the floodplain which appear to have no feasible or cost effective alternative for mitigation. Those properties remaining in the floodplain are listed in Table 3. The cost as provided are based on 1996 Bexar County Appraisal District property tax information.

The remaining mitigation projects described in this report address existing roadway flooding. Most of the roadways identified as being flooded have drainage structures that are to small for the storm water flows resulting from a 100 year storm event. Only one of the roadways, Jones Maltsberger Road, does not have any drainage structure and exists as a low water crossing at Mud Creek and Elm Creek. The street crossings identified for new drainage structures are listed in Table 4.

Table 3 "Flooded Properties"

Structures	Location	Appraised Value	Flood Depth
4 Houses	236 Holbrook Rd.	$56,100	6 feet
	243 Holbrook Rd.	$21,900	6 feet
	274 Holbrook Rd.	$36,200	6 feet
	Holbrook Rd.	$80,000	6 feet
1 Commercial Bldg	4354 Industrial Ctr	$680,000	4.5 feet
1 House	12522 Maltsberger Lane	$426,500	4 feet
2 Houses	205 Cresthill Rd.	$32,500	4 feet
	207 Cresthill Rd.	$85,200	3.5 feet
3 Buildings	11919 N. Weidner Rd.	$91,000	3-4 feet
	11609 N. Weidner Rd.	$21,800	3-4 feet
	11603 N. Weidner Rd.	$104,300	3-4 feet
1 Commercial Bldg	3400 Nacogdoches Rd.	$246,700	2-3 feet
1 House	3722 Bunche Rd.	$18,500	2 feet
2 Houses	12656 West Ave.	$80,000	2 feet
	12678 West Ave.	$30,980	2 feet
2 Houses	311 North Loop W.	$56,800	2 feet
	239 North Loop W.	$68,200	2 feet
TOTAL ESTIMATED COST		**$2,136,680**	

Table 4 - "Proposed Bridge and Culvert Projects"

Project No.	Project Description	Estimated Costs
1	New Bridge Structure at West Avenue and Salado Creek	$ 3,567,060
2	New Multiple Box Culverts at West Avenue and Panther Springs Creek	$ 332,500
3	New Bridge Structure at Vicar Rd. and Beitel Creek	$ 1,995,000
4	2 New Bridges Structures at Roland St.	$ 3,192,000
5	New Multiple Pipe Culverts at Jones Maltsberger and Mud Creek	$ 332,500
6	New Multiple Box Culverts at Jones Maltsberger and Elm Creek	$ 532,000
7	New Bridge Structure at Binz-Engleman Rd.	$ 4,309,200
8	New Bridges Structures for Frontage Roads at IH35 and Reroute Seguin Rd. (TxDOT)	$ 3,990,000
9	New Multiple Box Culverts and Raise 2700' of Bulverde Rd. at Redland Road	$ 665,000
TOTAL ESTIMATED COST		**$18,915,260**

The selection of the mitigation projects is based upon the results of this study which defines existing and ultimate development conditions within the watershed. Two projects

Brush clearing within the banks of Salado Creek should be avoided. Limited clearing along the outer banks should not have adverse effects on the linear detention benefits in Salado Creek. Project No. 9 which includes the channelization of Salado Creek south of Martin Luther King Drive would significantly change the aesthetics and wild life habitat features of the natural floodway. This project has a much greater cost than Project No. 5 which provides the same benefits. The environmental characteristics would significantly be changed by brush clearing or channelization of the creeks. Salado Creeks natural conditions provide erosion and sedimentation control along with the linear detention. A minor problem Salado Creek does have is debris that has either washed in or been dumped. Debris such as tires, lumber, and other trash should be removed. A clean natural Salado Creek provides an environment that is beneficial for all.

Benefit has also been gained from the Flood Control Program implemented by the U.S.D.A. Soil Conservation Service and San Antonio River Authority. Flood water reductions resulting from the thirteen Flood Retarding Dams has greatly reduced the number of properties that would be adversely effected. Thus requirements for mitigation have greatly been reduced and the cost estimated for eliminating flooding problems is less than would be anticipated otherwise. Total estimated costs for the recommended flood mitigation projects, flooded property, bridge and culvert projects is $25,679,135. Included are TxDOT costs associated with their highway system and the value of flooded properties. With these costs deducted the total cost is reduced to $19,552,455.

2

Introduction

A. Scope of Project

A study of Salado Creek and its major tributaries was authorized in April, 1994 by the City of San Antonio. The purpose of the study is to map the floodplains and develop projects that will mitigate the flooding identified by the study. Floodplains have been redrawn and mapped for the 10, 25, 50, 100, and 500 year frequency storms. Mitigation projects which can eliminate flooding problems caused by a 100 year frequency storm have been identified in this study. These projects form the basis for the Drainage Master Plan for the Salado Creek Watershed. These projects have been prioritized based benefits and costs. Presented with this report, are hydrologic and hydraulic models, new floodplain maps, and a definition of mitigation projects for a master plan.

The watershed study tasks were performed in three phases; a Preliminary Phase, Design Phase, and Summary Phase. Research, investigation, and hydrologic modeling were performed in the Preliminary Phase. Research efforts included gathering data on flooding complaints, previous flood studies, precipitation and stream flood gage records, aerial mapping, U.S.G.S. mapping, soil characteristics, plans for culverts, bridges, and dams, and land use information. Field investigation involved observing and photographing the creeks, bridges and culverts. Hydrologic models were created for the drainage areas above the Salado and Rosillo Creek confluence. Watershed subareas were networked along Salado Creek and its tributaries. Rainfall input in the form of precipitation hydrographs are used to compute runoff for each subarea. The runoff discharged into the creeks is routed down the stream network using unit hydrograph techniques. Runoff hydrographs are combined at the nodes along the network producing new hydrographs and peak discharges at each node. The hydrologic model computed discharges for the 10, 25, 50, 100, and 500 year frequency storms.

In the Design Phase, water surface profiles were computed using the hydrologic model storm water flows for the 10, 25, 50, 100, and 500 year frequency storms. Hydraulic modeling of Salado Creek along with the major tributaries: Beitel Creek, Mud Creek, Elm Creek, Elm Waterhole Creek, and Panther Springs Creeks was performed in the Design Phase. During the initial hydraulic analysis of the lower 20 miles of Salado Creek it became evident that a significant reduction of storm water flow was occurring. Reduction of the storm water flow could only be attributed to linear channel storage. This required that the study be expanded to include a storage analysis to accommodate this unexpected condition. Utilizing the hydrologic and hydraulic models a storage analysis was completed for existing conditions and ultimate development. The effect of storage on the water surface elevations is significant and lowered 100 year flood elevations approximately four and a half (4.5) feet in the southern reaches of Salado Creek. Water surface elevations derived from the hydraulic model were used to prepare floodplain maps showing the new floodplains for the 10, 25, 50, 100, and 500 year frequency storms under existing conditions. The new floodplains are shown on aerial maps produced by United Aerial Mapping for the City of San Antonio. These maps revealed the existing structures and roadways that are subject to flooding. Projects were identified and developed which could mitigate flooding where practical. Costs were developed for the mitigation projects and the projects prioritized for implementation based on benefits and costs.

The Summary Report Phase was the final phase and included the preparation of this report, compilation of data from the Preliminary and Design Phases, development of summary and recommendations, and presentation to the public. This Summary Report contains details of the investigations, criteria of the project, and details of the models and analyses. Included in the report are the appendices, research data, the model's inputs and summary outputs. Also provided are descriptions of the processes, results of the modeling, mitigation projects and alternatives with recommendations and estimated costs.

B. Salado Creek Watershed

The Salado Creek Watershed is a drainage basin of approximately 190 square miles. Storm runoff from the drainage basin as shown in Figure 1 is characterized by components of surface runoff (sheet flow), street flows (shallow concentrated flow), stream flows(channelized flows) and reservoirs(storage). These components are linked by a stream network that is used to create a HEC-1 Model. HEC-1 is an abbreviation for a computer program developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center. This program is widely used for developing hydrologic models.

The entire watershed is subdivided into smaller drainage areas that are identified as subareas. The Salado Creek Watershed was divided into eighty-five subareas as shown in Figure 3. Runoff from the subareas was computed using the sheet flow, shallow concentrated flow, and channelized flow. The computed runoff from each subarea was discharged into channels or creeks as storm water flow. Storm water flows routed in the stream network are combined with the runoff from adjacent subareas to compute the peak storm water flows in the creeks.

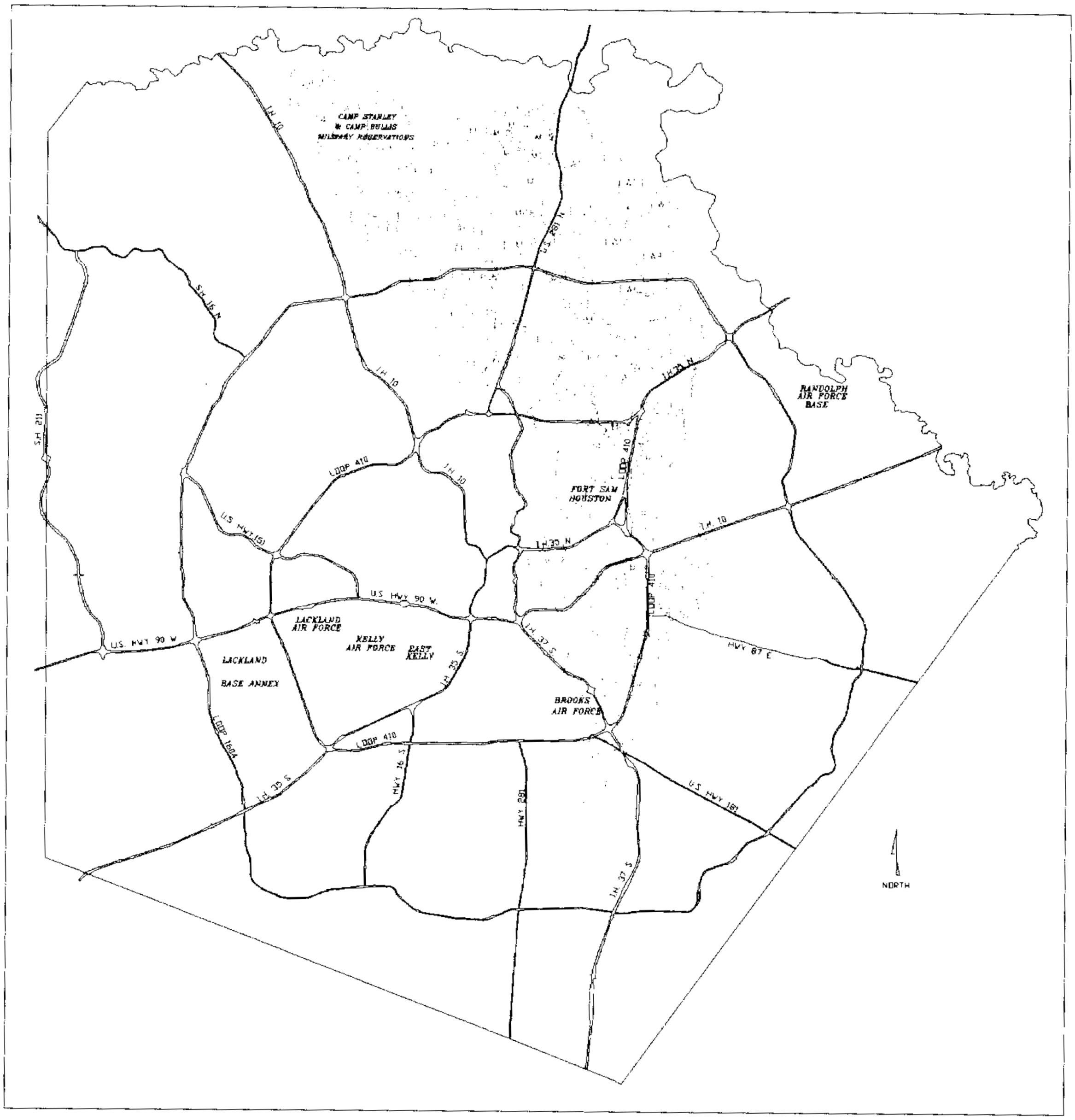


Figure 3 - "Salado Creek Watershed - Subareas"

I. Salado Creek and Tributaries

Salado Creek runs through eastern San Antonio and Bexar County. The Salado Creek ends in southeastern Bexar County as a tributary to the San Antonio River. Following Salado Creek upstream from its convergence with the San Antonio River, it travels in a northeasterly direction for approximately two to three miles. At the location where the Salado Creek crosses S.E. Loop 410 it turns northward and except for a slight east and west meandering, the creek follows a northerly direction to N.E. Loop 410. Continuing upstream, the Creek turns west to northwest prior to crossing Nacogdoches Road. From Nacogdoches Road, Salado Creek travels in a west northwesterly direction through northern San Antonio. After Salado Creek crosses West Avenue, it turns northward, traveling in a north, northwesterly direction towards Loop 1604. Upstream of Loop 1604, Salado Creek meanders in a northwesterly direction through a portion of the lower hill country. The upper reach of Salado Creek travels through the Leon Springs Military Reservation, but does not reach the northern limits of Bexar County or Interstate Highway 10. Salado Creek's upper limits and drainage area are defined by a ridge east of Interstate Highway 10 and south of the Bexar County line. Salado Creek lies solely within Bexar County and as shown in Figure 4 is approximately 43 miles in length.

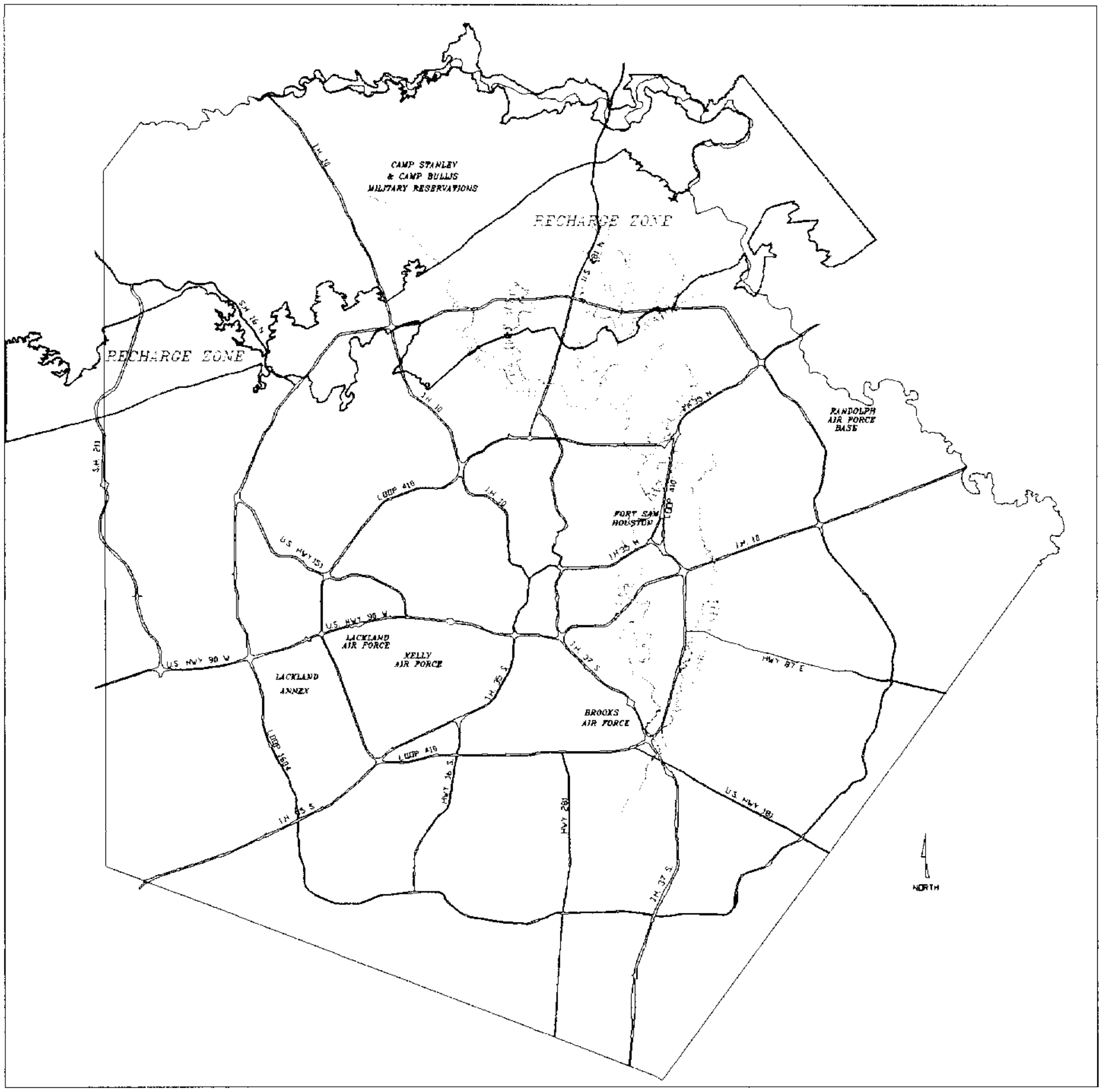


Figure 4 - "Salado Creek and Tributaries"

There are several tributaries that exist within the Salado Creek watershed, including Panther Springs Creek, Lorence Creek, Mud Creek, Beitel Creek, Walzem Creek, Rosillo

Creek, Quail Creek, and several unnamed creeks. Elm Creek and Elm Waterhole Creek are tributaries of Mud Creek.

II. Drainage Basin

Salado Creek and each of its tributaries has a drainage basin. The subareas have been identified according to the drainage basin wherein they lie. SC signifies Salado Creek and likewise PS for Panther Springs Creek, LC for Lorence Creek, MC for Mud Creek, EC for Elm Creek, EW for Elm Water Hole Creek, BC for Beitel Creek, WC for Walzem Creek, and RC for Rosillo Creek. SR signifies Stahl Road because the tributary in that drainage basin was unnamed.

Rosillo Creeks drainage basin has been included for the purpose of evaluating backwater effects. Rosillo creek is outside the limits of the hydraulic study area, however, backwater created at the Salado and Rosillo Creek was analyzed.

Topography

Topography within the Salado Creek Watershed varies in the upper and lower areas of the watershed. The upper area is in the Edwards Plateau and is hilly with steeper slopes. In this area, the Salado Creek and tributary creeks have cut steep valleys through the land and because this area is the larger portion of the watershed it contributes a large amount to the total stream flow. A combination of rocky and clay soils also contribute to the larger runoff. Rock, clays, and steep slopes create nearly impervious conditions and this reduces the effect of development and its associated impervious cover on storm water flows. Salado Creek as it runs from West Avenue across north San Antonio to N.E. Loop 410, has a milder slope, however, the drainage basins around the creek still have steeper slopes. The southern or lower areas of the watershed are located in the Blackland Prairies. Slopes across the drainage basins and along the creek in the lower area south of N.E. Loop 410 are even more mild. Elevations in the watershed range from 500 feet above mean sea level to over 1500 feet. Upper watershed areas, having the steeper slopes, vary in elevation from 700 feet to 1500 feet above mean sea level. This variation in elevation occurs from N.E. Loop 410 to the upper limits of the watershed. The lower watershed varies from 500 feet at S.E. Loop 410 to 700 feet at N.E. Loop 410.

Soils

To evaluate the rainfall and runoff relationship for the drainage basin it is necessary to assess the characteristics of the existing soils. Data was obtained from the United States Department of Agriculture, Soil Conservation Service now identified as U.S.D.A. Natural Resources Conservation Service. Soil data was obtained in database files (Soil Survey Geographic Data Base) which is the same data published in the "Soil Survey for Bexar County, Texas". The database contains characteristics for the various soil types located in Bexar County. Included with the database was a digitized graphic file showing the location of the various soils. The Salado Creek Watershed and graphic file of the soils were overlain and the soil types within the watershed were identified. Soil types are classified by Hydrologic Soil Groups. The four Hydrologic Soil Groups are A, B, C, and

D. The definition or soil characteristics of the four Hydrologic Soil Groups are provided in Table 5. A list of soil types found in the Salado Creek Watershed is provided in Table 6. The soil types within the Salado Creek Watershed were grouped according to the Hydrologic Soil Groups and mapped accordingly as shown in Figure 5. A single small area of Eufalia sand (Hydrologic Soil Group A) was found in the watershed. This area was used as Hydrologic Soil Group B to simplify the computation of land use and soil groups.

Table 5

Definition of the SCS Hydrologic Soil Groups

A	These Soils have a high infiltration rate. They are chiefly deep, well drained sands or gravels. (Low Runoff Potential)
B	These Soils have a moderate infiltration rate when thoroughly wet. They are moderately deep, well drained soils of moderately fine to moderately course texture.
C	These Soils have a slow infiltration rate when wet. They are soils with a layer that impedes downward movement of water and soils of Moderately fine to fine texture.
D	These Soils have a slow infiltration rate. They are chiefly clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan at or near the surface, and shallow soils over nearly impervious material. (High Runoff Potential)

Table 6 - "Soil Types in Salado Creek Watershed"-

SOIL GROUP	SOIL TYPE	SOIL NAME
A	EuC	EUFAULA SAND (ALUF)
B	DmC	DUVAL LOAMY FINE SAND
	DnB	DUVAL FINE SANDY LOAM
	DnC	DUVAL FINE SANDY LOAM
	DsC2	DUVAL SOILS
	Fr	FRIO CLAY LOAM (SUNEV)
	Go	GOWEN CLAY LOAM
	Gu	GUILLED LAND (SUNEV)
	KaB	KARNES LOAM (ATCO)
	KaC	KARNES LOAM (ATCO)
	KcC2	KARNES CLAY LOAM (ATCO)
	LvA	LEWISVILLE SILTY CLAY
	LvB	LEWISVILLE SILTY CLAY
	LvC	LEWISVILLE SILTY CLAY
	PaA	PATRICK SOILS
	PaB	PATRICK SOILS
	PaC	PATRICK SOILS
	VaA	VENUS LOAM (SUNEV)
	VaB	VENUS LOAM (SUNEV)
	VcA	VENUS CLAY LOAM (SUNEV)
	VcB	VENUS CLAY LOAM (SUNEV)
	VcC	VENUS CLAY LOAM (SUNEV)
	WmA	WILLACY LOAM
	WmB	WILLACY LOAM
	Za	ZAVALA FINE SANDY LOAM
	Zg	ZAVALA AND GOWEN SOILS
C	AuB	AUSTIN SILTY CLAY
	AuC	AUSTIN SILTY CLAY
	BpC	BRACKETT CLAY LOAM (WHITEWRIGHT)
	BrD	BRACKETT SOILS (KERRVILLE)
	BrE	BRACKETT SOILS (KERRVILLE)
	BsC	BRACKETT-AUSTIN COMPLEX (WHITEWRIGHT)
	BtE	BRACKETT-TARRANT ASSOC. (KERRVILLE)
	HgD	OLMOS, HILLY GRAVELLY LAND
	HkB	HOCKLEY LOAMY FINE SAND (WILCO)
	HkC	HOCKLEY LOAMY FINE SAND (WILCO)
	HkC2	HOCKLEY LOAMY FINE SAND (WILCO)
	LfB	LEMING LOAMY FINE SAND
	SaB	SAN ANTONIO CLAY LOAM
	SaC	SAN ANTONIO CLAY LOAM
	SaC2	SAN ANTONIO CLAY LOAM
	ScB	STEPHEN SILTY CLAY
	ScC	STEPHEN SILTY CLAY
	Tb	TARRANT SOILS (EDDY)
	WbB	WEBB FINE SANDY LOAM (FLORESVILLE)
	WbC	WEBB FINE SANDY LOAM (FLORESVILLE)
	WeC2	WEBB SOILS (FLORESVILLE)
	WeC3	WEBB SOILS (FLORESVILLE)
D	Ca	CRAWFORD CLAY (ANHALT)
	Cb	CRAWFORD AND BEXAR STONY SOILS (ANHALT)
	CfA	CROCKETT FINE SANDY LOAM (MIGUEL)
	CfB	CROCKETT FINE SANDY LOAM (MIGUEL)
	CkC2	CROCKETT SOILS (MIGUEL)
	HnB	HOUSTON CLAY (HEIDEN)
	HnC2	HOUSTON CLAY (HEIDEN)
	HnC3	HOUSTON CLAY (HEIDEN)
	HoD3	HOUSTON-SUMTER CLAYS (HEIDEN)
	HsA	HOUSTON BLACK CLAY
	HsB	HOUSTON BLACK CLAY
	HsC	HOUSTON BLACK CLAY
	HtA	HOUSTON BLACK CLAY (BRANYON)
	HtB	HOUSTON BLACK CLAY (BRANYON)
	HuB	HOUSTON BLACK GRAVELLY CLAY
	HuC	HOUSTON BLACK GRAVELLY CLAY
	HuD	HOUSTON BLACK GRAVELLY CLAY
	Kr	KRUM COMPLEX
	OrA	ORELIA SANDY CLAY LOAM
	OrB	ORELIA SANDY CLAY LOAM
	Pt	PITS AND QUARRIES
	TaB	TARRANT ASSOC. (ECKRANT)
	TaC	TARRANT ASSOC. (ECKRANT)
	TaD	TARRANT ASSOC. (ECKRANT)
	Tc	TRINITY CLAY (TINN)
	Tf	TRINITY AND FRIO SOILS (TINN)

3

Preliminary Phase

The preliminary phase included research, investigation, and hydrologic modeling. The tasks and efforts are detailed as follows.

A. Research

I. Existing Data

Research performed for this study included visiting and interviewing representatives of various City, County, State, and Federal agencies to locate, identify, and subsequently analyze available data on Salado Creek and its tributaries. Several tables presented in Appendix A list the agencies and data reviewed. Data analyzed included several previous studies of Salado Creek including an analysis by the U.S. Army Corps of Engineers in 1969, the F.E.M.A. floodplain analysis, and a watershed study completed by the U.S. Department of Agriculture Soil Conservation Service in 1994. The methodologies, assumed conditions, and floodway characteristics used in these studies were also evaluated. Other hydraulic studies identified in the City of San Antonio files were for land development projects performed by other engineering consultants.

Evaluation of the studies included review of the techniques, modeling softwares, and objectives. The F.E.M.A. floodplain analysis and studies performed for land development were the only studies which specifically defined floodplains. Most of the studies reviewed were performed for analysis and simulation of previous floods and flood control projects.

II. Historical Storms

The initial task required to develop the hydrologic model involved research of historical rain fall and creek flow data. Historical data dates to the early 1900's, but accurate records of creek flow depths and storm water flows did not begin until the 1960's. The United States Geological Survey (U.S.G.S.) began installing stream gaging stations on the creeks in Bexar County, in the 1960's. Continuous recording gages that measure creek flow depth and precipitation have been utilized for the past twenty six years.

Two gages have been maintained by the U.S.G.S. on Salado Creek; one at N.E. Loop 410 and the other at S.E. Military Drive. Other gaging sites were utilized in the 1970's but have been removed. In 1990, the City of San Antonio established an Early Flood

Warning System which included the installation of precipitation and stream gages. A stream gage is maintained at Interstate Highway 10 and Salado Creek and Precipitation gages have been installed at numerous locations within San Antonio. Other sources of precipitation data are the U.S.G.S. and the National Weather Service (N.W.S.). A precipitation gage is maintained by the U.S.G.S. at N.E. Loop 410 and a gage is maintained by the N.W.S. at the San Antonio International Airport. These agencies have provided data from their gages that was recorded during past storms.

Stream and Watershed conditions were evaluated for each of the largest storm events recorded in the past twenty five years. Conditions such as existing land development, construction of dams and other structures along Salado Creek were the main criteria used to narrow the selection of storms to those that occurred in the 1990's. The land use data had been updated by the City of San Antonio in 1991 and twelve flood control dams were complete with the thirteenth dam under construction. The largest storms that have occurred since 1990 were on April 4-5, 1991 and May 5-6, 1993. Precipitation and stream gage data pertaining to these storms is presented in Appendix B. Descriptions of the storms were provided by the N.W.S. along with isohyetals of the storm rainfall totals. The isohyetals shown in Figures 6 & 7 represent rainfall distribution patterns of the two storms. The rainfall data shown represents approximate rainfall totals for the duration of the storm. The rainfall patterns are interpolated from numerous gage reports which are scattered over the City.

The largest rainfall totals for each storm occurred in different areas. Rainfall during the April 4-5, 1991 storm had higher concentrations west of the watershed and produced larger storm water flows in those areas. Although the storm was centered outside the Salado Creek Watershed, the storm water flows produced in Salado Creek are the second largest recorded since 1990. The largest storm water flows recorded in the Salado Watershed occurred during the May 5-6, 1993 storm. The highest rainfall totals were in the mid region of the watershed. Storm water flows produced in Salado Creek were measured at the three stream gaging stations described previously. The stream gages at Interstate Highway 10 and N.E. Loop 410 malfunctioned in May 1993 and did not record the peak storm water flows in Salado Creek. A manual field measured depth of the storm water flow at the approximate time of the peak flow was taken at N.E. Loop 410. All three stream gaging stations shown on Figure 8 were operating in April 1991 and recorded continuously through the storm.

Although the May 5-6, 1993 storm produced larger runoff and discharges in Salado Creek, the recorded data was incomplete. Data recorded during the April 4-5, 1991 storm was utilized in the HEC-1 and HEC-2 models for comparison and verification of the models.

Figure 8
Stream Flow Gage Stations

conditions observed are contained in Volume II, Appendix C. Field investigation did identify several channelized sections within Salado Creek and it's tributaries. Channelization was identified along Salado Creek between Nacogdoches and Wetmore Road. This area of Salado Creek is all that was observed that has been channelized, except for roadway crossings. Beitel Creek upstream and downstream of N.E. Loop 410 has been channelized by the development process. Additional channelization has occurred in the upper reach of Beitel Creek at the O'Connor Road and Nacogdoches Road crossings. Channelization has also occurred on Mud Creek, Elm, and Elm Waterhole Creek around Thousand Oaks and Redland Oaks Road. The channelization that has occurred primarily consists of clearing and reshaping of the earthen channel sections. In two locations, however, the channel has been lined with concrete. Concrete channels have been built on Beitel Creek between Vicar Drive and N.E. Loop 410 and on Salado Creek under the IH-35 bridge.

Fill and debris deposits within the flood plain of Salado Creek on the north side of San Antonio International Airport were observed on properties owned by the City of San Antonio. Fill Materials were stock piled adjacent to the floodplain at Arion Parkway and U.S. Hwy. 281.

II. Structures

Field investigation revealed that a variety of drainage structures exist within the banks and floodway of the Salado Creek and its tributaries. These structures include pipe culverts, box culverts, bridges and dams. A list of existing structures and their locations is provided in Table 7. These structures have been examined in the field and documented with photographs. Available as-built plans were obtained for these structures and utilized in the hydrologic and hydraulic analyses.

Bridges and Culverts

The majority of the bridges at road crossings that were observed were designed and constructed by the Texas Department of Transportation. As-built plans for these bridges were obtained from the Texas Department of Transportation and were utilized in development of the hydraulic model. Culverts exist in several locations including Interstate Highway 35, Interstate Highway 10, N.E. Loop 410, and Loop 1604. Several other culverts are located across Salado Creek and the tributaries that were constructed by developers or the City of San Antonio. Culvert crossings on Salado Creek flood on a regular basis. Other small Creek culverts that flood are located at Vicar Drive on Beitel Creek and West Avenue on Panther Springs Creek. Flooded roadway crossings are identified by * in Table 7.

Table 7 - "Existing Structures"

CREEK	CROSSING	STRUCTURE	DOWNSTREAM STATION	UPSTREAM STATION
Salado	S.E. Loop 410	Bridge	20440	20729
	S.E. Military Dr.	Bridge	33188	33294
	E. Southcross	Bridge	43166	43308
	* Roland	Culverts	50191	50255

Continue Table 7 - "Existing Structures"

CREEK		CROSSING	STRUCTURE	DOWNSTREAM STATION	UPSTREAM STATION
		Rigsby	Bridge	54551	54608
		Rice	Bridge	61634	61680
		Martin Luther King	Bridge	63552	63615
	*	MLK Park Rd.	Culverts	66969	67031
		I.H. 10	Bridge	69770	69937
		Commerce St.	Bridge	72015	72092
		Houston St.	Bridge	73040	73098
		Gembler	Bridge	81369	81444
		S. Pac. R.R.	Trestle	86460	86482
	*	I.H. 35	Bridge & Culverts	87081	87445
	*	Seguin Rd.	Culverts	87570	87609
		Mis-Kan-Tex R.R.	Trestle	90489	90507
	*	Binz-Engleman	Culverts	92110	92176
	*	W.W. White Rd.	Culverts	96242	96336
	*	Rittiman Rd.	Bridge	110026	110103
	*	Eisenhauer	Bridge	114557	114620
	*	Austin Hwy.	Bridge & Culverts	115915	116126
	*	N. Loop 410	Bridge & Culverts	125239	125541
	*	Nacogdoches	Bridge	132303	132365
		Mis-Pac R.R.	Bridge	138032	138061
		Wetmore Rd.	Bridge	138121	138194
	*	Entrance Ave.	Culverts	141965	142019
	*	Bitters Rd.	Culverts	144266	144420
	*	Bitters Rd.	Culverts	145362	145424
		Jones Maltsberger	Bridge	151236	151311
		U.S. Hwy 281	Bridge	157091	157442
	*	West Ave.	Culverts	161964	162051
		Vista Del Norte	Bridge	168226	168291
		Blanco Rd.	Bridge	170905	170967
	*	Old Blanco Rd.	None	171621	
		Huebner Rd.	Bridge	181787	181924
		Loop 1604	Bridge	192321	192471
Panther Springs	*	North Loop Rd	None	433	
	*	West Ave.	Culverts	1182	1272
		SCS Dam #7	Spillway	3955	4347
		Bitters Rd	Bridge	11248	11323
		Mission Ridge Dr	Bridge	15658	15750
		SCS Dam #6	Spillway	16921	17234
		Loop 1604	Bridge	30251	30655
Mud	*	Starcrest	None	1104	
	*	Buckhorn	Culverts	4990	5046
		Thousand Oaks	Culverts	11103	11201
	*	Jones Maltsberger	None	19633	
		SCS Dam #10	Spillway	20351	20776
		Loop 1604	Bridge	28182	28489

Continue **Table 7 - "Existing Structures"**

CREEK	CROSSING	STRUCTURE	DOWNSTREAM STATION	UPSTREAM STATION
Elm Waterhole	Redland Rd.	Culverts	5549	5628
	* Bulverde Rd	None	6822	
	Classen Rd.	Culverts	9807	9863
	Loop 1604	Bridge	11091	11576
Elm	Redland Rd.	Culverts	3198	3320
	* Jones Maltsberger	None	5075	
	Loop 1604	Culvert	6878	7316
Beitel	Perrin Beitel	Bridge	2802	2870
	* Vicar Dr.	Culverts	3370	3416
	N.E. Loop 410	Bridge	4839	5321
	Mis-Pac R.R.	Trestle	15592	15620
	Mis-Pac R.R.	Trestle	18842	18877
	* Shertz Rd.	Culverts	19067	19112
	* Weidner Rd.	Culverts	21854	21888
	O'Connor Rd.	Bridge	23842	23919
	* Old O'Connor	Culverts	24641	24674
	* Lookout Rd.	Culverts	25123	25172
	Mis-Kan-Tex R.R.	Bridge	25205	25217
	O'Connor Rd.	Culverts	26903	26975
	Nacogdoches Rd.	Culverts	29995	30087

* Flooded Crossing

Floodwater Retarding Dams

Within the upper Salado Creek watershed, are thirteen (13) floodwater retarding dams (see Figure 9). Over fifty percent of the total area within the watershed or 74,989 acres of land is located above the dams. These dams were designed and constructed under a Flood Control Program that resulted from the "Small Watershed Protection and Flood Prevention Act, Public Law 566" passed in 1954. The Salado Creek Flood Control Program was started in the late 1960's after being approved by Congress in 1962 and amended in 1968 and 1971. The U.S.D.A. Natural Resources Conservation Service and the San Antonio River Authority worked in cooperation in planning and constructing the dams. Sixteen dams were originally planned for the Flood Control Program. In 1964 the McAllister Park Proposed Master Land Use Plan was completed and included the fourteenth dam (15r). See Appendix F. The City of San Antonio is an additional sponsor of this dam as owner of the site. The dam in McAllister Park is expected to cost approximately $6,000,000. This estimate was provided by Mr. Trent Street, Design Engineer for the U.S.D.A. Natural Resources Conservation Service.

The Salado Creek Flood Control Program (Table 8) began with the design and construction of the first Floodwater Retarding Dam at Site No. 2. To date, thirteen (13) dams have been completed with the thirteenth having been completed in mid 1996.

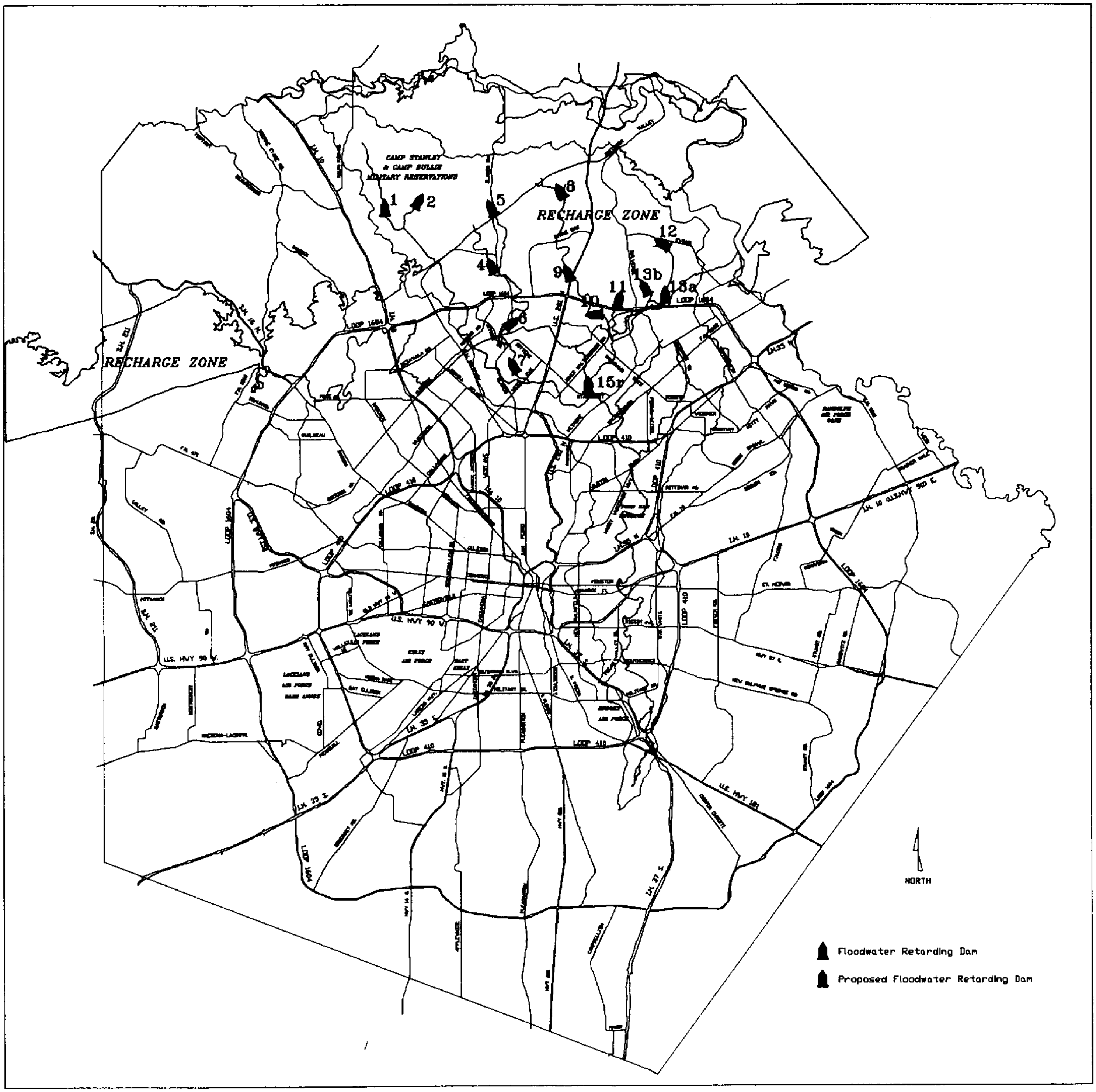


Figure 9
Floodwater Retarding Dams

Figure 9 shows the thirteen existing dams and proposed dam in McAllister Park. The first twelve dams were constructed at a cost of approximately $17,000,000. The thirteenth dam at Site No. 10 cost approximately $5,000,000. The fourteenth and final dam planned in the Salado Creek Flood Control Program at site #15r, is designated to be constructed under the Federally Funded Program.

Other benefits have been gained from these floodwater retarding dams, including, recharge of the Edwards Aquifer, water conservation, and erosion control. Several of the dams were built over the recharge zone and make significant contributions to recharge of the Edwards Aquifer.

Table 8

SALADO CREEK FLOOD CONTROL PROGRAM				
Site No.	Completion Date	Drainage Area Acres	Storage Area Acre-Feet	Dam Height Feet
1	11-25-75	7,232	4,189	75
2	03-05-71	3,674	2,293	55
4	10-31-72	3,526	1,982	55
5	10-18-76	5,670	3,293	58
6	03-09-82	2,928	1,490	62
7	04-25-87	3,710	2,340	47
8	05-16-73	7,154	4,178	62
9	03-09-82	1,517	1,026	49
10	1996	3,061	1,846	66
11	04-07-80	4,198	2,596	65
12	06-06-74	8,128	4,875	70
13A	08-13-76	2,099	1,441	43
13B	08-22-75	1,619	1,093	46
15R	Proposed	6,440	3,405	44

III. Land Use

Existing Development

The City of San Antonio Planning Department provided the land use categories and location database used in this study. Land uses included eight primary use categories described as follows: (10) Residential, (20) Commercial, (30) Industrial, (40) Services, (50) Open Space, (60) Agricultural, (70) Transportation, and (80) Vacant. Descriptions of the different land uses are presented in Appendix D. All land uses were divided and regrouped into seven categories according to average percentage of impervious cover. The seven categories that resulted are dispersed residential; residential; densely developed residential, such as apartments; business and commercial; industrial and institutional; open space and parks; and streets, roads, and parking areas. Table 9 lists the categories, land uses, and the average percent impervious cover used in this study. The seven different land uses were mapped over the Salado Creek Watershed and Figure 10 presents the resulting land uses in the Salado Creek Watershed. The areas of each land use within the subareas and their corresponding category characteristics were used as parameters in the HEC-1 modeling to compute runoffs. The landuses in the Salado Creek Watershed show that 46,340 acres which is 38 percent of the land is undeveloped or open space.

Table 9 - "Land Use Categories"

CATEGORY	LAND USE	AVERAGE % IMPERVIOUS
11	Dispersed Residential	20
12	Residential	38
13	Densely Developed (Apartments)	75
21	Business and Commercial	90
31	Industrial	78
51	Open Space, Range Land, Parks, and Agricultural	0
71	Streets, Roads, and Parking Areas	98

Ultimate Development

The majority of undeveloped land is in the upper watershed as shown on Figure 10. The City of San Antonio, Planning Department provided projections for ultimate development for the 46,340 acres of available, undeveloped land. The Development projections show 55% to be developed as residential, 5% to be developed as dense residential, 15% to be developed as commercial, 5% to be developed as industrial, 5% to be developed as roads, streets or parking areas, and 15% to be retained as open space or park land. In areas within and above the Recharge Zone, residential development is projected to be dispersed residential. All other areas below the recharge zone are projected to be residential.

C. Hydrologic Modeling

I. Theoretical Assumptions

There are certain assumptions that must be made in the application of all simulations and models. Hydrologic modeling requires that several assumptions be made to compute

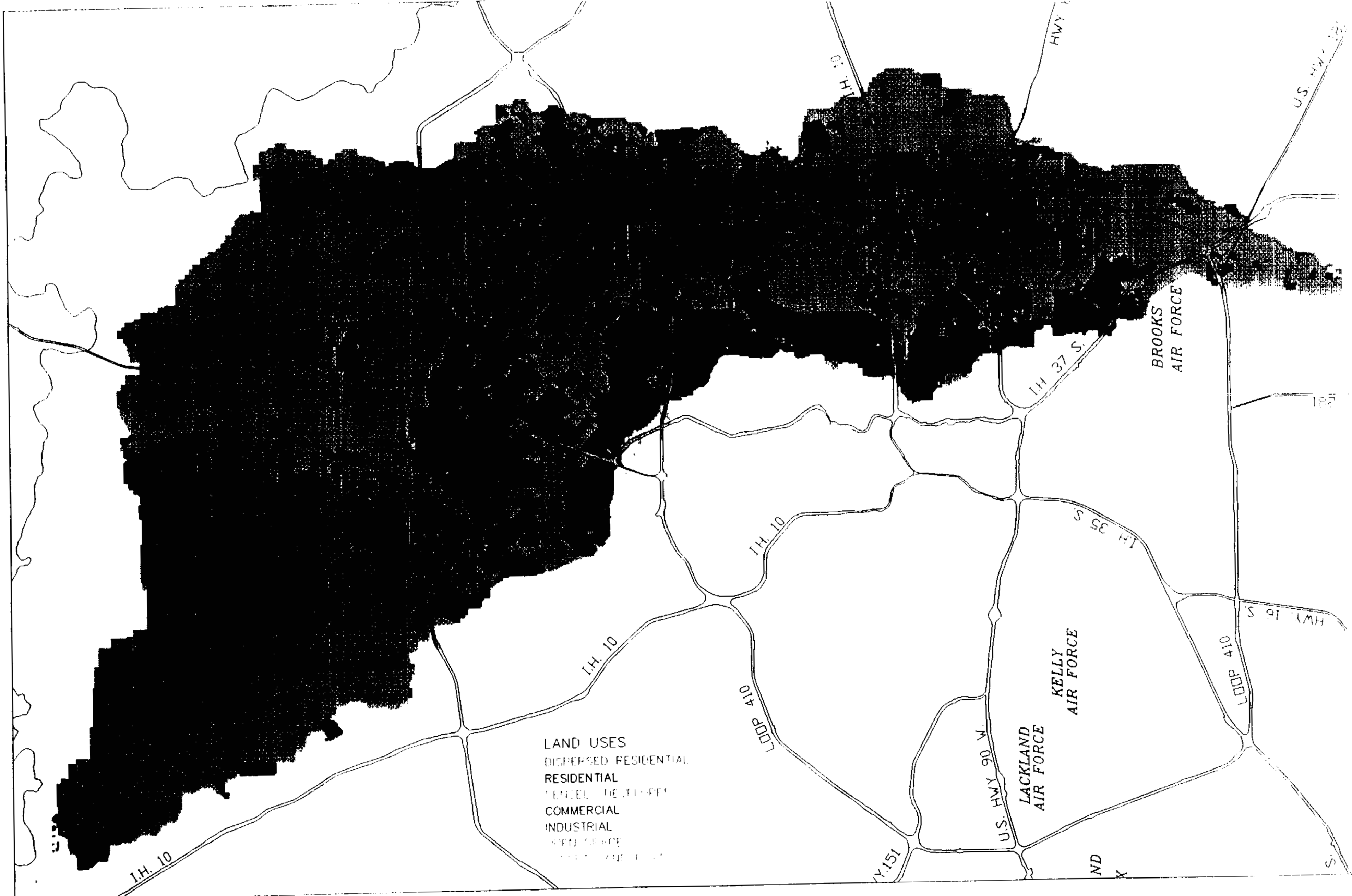


Figure 10 – Landuses in Salado Creek Watershed

runoff and losses. Included in a hydrologic model are initial losses and uniform losses that are associated with rainfall. Initial and uniform losses result from infiltration, interception, and depressions. After the initial loss of rainfall is determined, then uniform losses of rainfall runoff are determined based upon the assumption that they occur at a constant rate. Several variables are used to determine the initial and uniform losses, including soil type, slope, land use, and antecedent soil moisture condition.

During the Preliminary Phase, meetings were held with the City of San Antonio and the Consultants performing the Olmos Creek and Leon Creek Studies to review and discuss methodology. By a consensus it was determined that the Soil Conservation Service Methodology as outlined in SCS National Engineering Handbook, Section 4, Hydrology (NEH-4) was to be used for the hydrologic model.

Therefore, the Soil Conservation Service Methods were used for establishing rainfall runoff losses. As specified by the City of San Antonio, the initial rainfall abstraction (Ia) in the HEC-1 runoff simulation process was determined for all events using the standard SCS equation, which is a function of runoff curve number (CN), as follows:

$$Ia = 0.2 * [(1000 - 10 * CN) / CN]$$

The hydrologic soil group and land use are combined to create a hydrologic soil - cover complex. Runoff curve numbers have been assigned to the hydrologic soil cover complexes by the Soil Conservation Service.

The City of San Antonio selected the CN values with agreement by all consultants so that this study and others would be uniform. Presented below are the CN values and their associated hydrologic soil groups.

HYDROLOGIC SOIL GROUPS	A	B	C	D
SCS RUNOFF CURVE NUMBER (CN)	25	55	70	77

An average CN value for each subarea was calculated using the above CN values and the area of each hydrologic soil group. Tables 10 and 11 present the weighted average CN values for each subarea. Average CN values for existing conditions are presented in Table 10 and Table 11 presents values obtained for ultimate development. Likewise, the weighted average percent impervious cover for each subarea was obtained by averaging the area by land use category and applying the average percent impervious values presented in Table 9.

For all simulations of storm events using the HEC-1 model of the Salado Creek Watershed, a five-minute computational time step has been used. This time step provides sufficient temporal resolution to describe typical variations in rainfall and runoff patterns as they have been observed within the Salado Creek Basin and is consistent with time step requirements for the SCS unit hydrograph method. The five minute time step also provides a convenient time frame for distributing the reported historical measured rainfall

Table 10 - SCS RUNOFF CURVE NUMBERS FOR EXISTING CONDITIONS

SOIL TYPES	B							C							D								
LANDUSE	11	12	13	21	31	51	71	11	12	13	21	31	51	71	11	12	13	21	31	51	71		
% IMPERVIOUS	20	38	75	90	78	0	98	20	38	75	90	78	0	98	20	38	75	90	78	0	98		
SCS CN VALUE	55	55	55	55	55	55	55	70	70	70	70	70	70	70	77	77	77	77	77	77	77	AVERAGE	AVERAGE
SUBAREAS	AREA (acres)							AREA (acres)							AREA (acres)							CN	% IMPERVIOUS
SC1									219.2				170			144.7				332.4		74	15.962
SC2						33			114.9		1.8		332.5			28.9				671.5		74	4.758
SC2													1005							639.8		73	0.000
SC4								3.4					535.2		6.2					819.5		74	0.141
SC5						0.7							1962.8							1665		73	0.000
SC6								68					864.8		9.2					1288		74	0.692
SC7						25							316.4							783.8		75	0.000
SC8								17.4					236.4							751.6		75	0.346
SC9													160.5							567.5		75	0.000
SC10																				917.8		77	0.000
SC11																				340.1		77	0.000
SC12																		77.2		3374.5	11	77	2.318
SC13		42.3				136.2									188.6	219.9				746	52.4	74	13.621
SC14	0.92	301.8	52	24.8	155.9	627.4			198.7	66.9	2.8	200.8	113.2	0.92		357	32	35	95.9	394.7		66	31.926
SC15	24.8	13.8	2.1	42.3	6.2	116.8	25.8					3.7			45.1	341	8	103.9	24.2	68	12.9	71	42.032
SC16	86.5		7.6	20.2	22.8	66.1	26.7	24.8	24.8	22.5	20.2	67.7	57.1	20.2	346.9	128.8	14.2	3.7	42.8	69.9	27.6	71	36.319
SC17	75.4		3.2	837.4	9.7	273.3	19.3	3.7	92	12.2	109.5	36.6	51.6	41.4	95.7	72.7	14	335	42.1	247.5	25.8	65	59.507
SC18	1.8	193	20.2	331	60.8	241.8	47.8	4.6		1.4	21.2	4.1	148.1	9.2	6.4	1.8	4.1	126	12.5	104.9	15.6	61	48.497
SC19						7.4		16.6	132.6	33.6	173	100.8	177.6	18.4	6.2	46.9	20.2	104	60.8	102.2	8.3	72	51.025
SC20	62.6	142.7	28.8	9.2	86.3	97.5	15.6								70.8	81	5.5		16.6	66.3	8.3	63	36.079
SC21	8.3	193.2	29.7	3.7	89	18.4	47.8		282.4	5.3		15.9	2.8	2.8	46	532.4	52.9	182.2	158.6	77.3	106.7	71	53.186
SC22	0.92	114.9	19.5	5.5	58.7	39.5	18.4		46.9	13.8	15.6	41.4	35.9		6.4	158.2	47.6	80	142.8	232.7	17.5	71	45.554
SC23		46	10.6	4.6	31.7	35	20.2									3.7	2.1		6.2	71.7		63	35.336
SC24	35	270.6	45.5	13.8	136.7	333.9	11		277	28.7	8.3	86.3	100.3	30.4	12.9	288.1	18.6		55.9	164.7	2.8	65	35.277
SC25	9.2	74.5	10.8		32.4	937.9	9.2						34		12	212.4	43.2	92	129.8	383.3	86.5	65	22.101
SC26			2.5		7.6	172			646	9	1.8	26.9	43.2	1.8		199.4	70.8		212.6	119.6	3.7	71	38.416
SC27	9.2	240.1	35.9	579.6	107.6	1212.7	228.2	5.5	2.8		35	3.7	108.3	39.6	0.92	33.1	92.2	183.1	276.7	457.8	188.6	63	43.533
SC28		5.5	5.3	2.8	15.9	41.4	9.2						11	0.92		807.4	188.3	135.2	564.9	163.7	76.4	76	55.037
SC29	5.5	522.1	29.7	100.3	89	457.7	88.3		26.7	5.5	5.5	16.6	99.3	2.8	0.92	87.4	6.7	0.92	20	30.4	6.4	59	35.214
SC30		102.1	3.7	29.4	11	14.7	14.7		262	24.8	3.7	74.6	64.4	9.2		340.2	42.1	54.3	126.3	149.5	46	72	46.116
SC31	135.2	645.2	29	52.4	86.9	1626.2	9.2	7.4	483	27.8	31.3	83.5	195	41.4	2.8	824.3	34.3	83.7	102.8	369.9	40.5	64	26.312
SC32	18.4	2.8	1.4	12	4.1	1169	199.6	42.3	318.2	14.7	14.7	44.2	226.3	51.5	47.8	5.5	3.2	21.2	9.7	470.1	72.7	64	20.583
SC33	17.5	12	1.1	41.4	3.5	1171.7	40.5	11		4.4	10.1	13.1	254.9	9.2	5.5			1.8	0.92	7.4	2.8	58	7.992
PS1	4.6					115.9		239.1					917.2		105.8	9.2				1766.1		74	2.324
PS2								152.7	16.6				86.5		149	174.8				1897		76	5.373
PS3																		3.7		407.6		77	0.810
PS4													330.1							888.3		75	0.000
PS5													40.1					26.7		2201.6		77	1.059
PS6																6.4	2.1		6.2	403.9		77	2.113
PS7															58.9	290.7	6	1.8	17.9	915.4		77	11.027
PS8																162.8	8	1.8	24.2	512.4	48.8	77	17.966
PS9						35.9									1.8	455.3	6.4	32.2	19.4	1124.9	72.7	77	16.787
PS10	4.6	109.5	5.7		17.3	253	19.3								235.4	752.2	20.5	5.5	61.4	541.7	9.2	73	24.044
LC1	3.7	47.8	3.7		11	2.7									748	1473.9	81.9	41.4	245.6	1029.4	176.6	77	31.127
LC2		50.6		196.9	2.8	11		15.6	221.6	14.2	12.9	42.8	94.7		64.4	780.9	10.8	9.2	32.4	433.3		73	34.746
MC1								350.5	46				82.8		424.1	69				149		74	17.712
MC2								239	1.8				8.3		289.6	0.92				34.1	4.6	74	19.239
MC3								105.8	5.5				90.2		81					380		75	5.955
MC4								149	9.2				225.4		144.4					426.9	1.8	74	6.683
MC5								0.92				0.92	168.2		25.8					447.6		75	0.942
MC6																		1.92		472.9	3.7	77	1.119
MC7															97.5			4.6	3.7	2226.1	22.1	77	2.047
MC8																11				1486.4		77	0.279
MC9															64.4	103	3.4	151.7	10.4	1548.7	58.9	77	13.241
MC10															5.5	89.2	6		17.9	337.6	32.2	77	17.407
MC11	0.92	90.2		8.3	0.92	69									59.8	266.9	16.3	3.7	49	300	0.92	73	24.332

Table 10 - SCS RUNOFF CURVE NUMBERS FOR EXISTING CONDITIONS

SOIL TYPES	B							C							D								
LANDUSE	11	12	13	21	31	51	71	11	12	13	21	31	51	71	11	12	13	21	31	51	71		
% IMPERVIOUS	20	38	75	90	78	0	98	20	38	75	90	78	0	98	20	38	75	90	78	0	98		
SCS CN VALUE	55	55	55	55	55	55	55	70	70	70	70	70	70	70	77	77	77	77	77	77	77	AVERAGE	AVERAGE
SUBAREAS	AREA (acres)							AREA (acres)							AREA (acres)							CN	% IMPERVIOUS
MC12		76.4			1.8	83.7		11	99.4		38.6		250.4		0.92	266.9		4.6	3.7	179.4	0.92	71	21.094
MC13	0.92	28.5				116.7		3.7	78.2			1.8	54.2			12.9			2.8	122.2		67	11.841
EC1															92.7	358.7	3.7	39.6	11	1679.6	47.8	77	11.136
EC2															50.6	36.8		26.7		1902.1		77	2.387
EC3	19.3					11									6.4	33.1	5	14.7	15.2	348.6	40.5	76	17.465
EW1								42.3				0.92	204.2		358.8		7.1	20.2	21.4	4143.2	44.2	77	3.396
EW2																				1686.3		77	0.000
EW3																		158.2		1979.3		77	6.661
EW4						7.4			17.5		115		29.5	6.4		0.92		298.9		1406.1	28.5	76	21.658
EW5															87.4	8.3		219.9		1494.8		77	12.072
EW6		39.6		0.92		454.5	39.6								13.8	160.1		158.2	3.7	1214.2	47.8	71	14.556
SR1						54.3		2.8	251.2	9.4		28.3	493			58.9	1.4		4.1	262		71	13.023
SR2						1.8	0.92	22.1	430.4	3.7	14.7	11	390.9	6.4	34	71.8	1.4	13.8	4.1	248.2	7.4	72	20.413
SR3	1.8	41.4		12	0.92	43.3		27.6	65.3		268.6	1.8	102.1	8.3	6.4	14.7	1.6	246.6	4.8	99.4	24.8	71	58.397
BC1								40.5	534.4	37.5	11	112.5	557.3	5.5	3.7	94.8	3.2	1.8	9.7	139.8	0.92	71	25.221
BC2								98.4	125.1	6.2	13.8	18.6	120.5	17.5	49.7	479.2	9.9	48.8	29.7	605.1	11	75	24.075
BC3								81.9	664.3	28.7	345.9	86.3	632	95.7	1.8	341.3	6.7	281.5	20	286.1	0.92	72	40.642
BC4								22.1	199.6	22.3	54.3	66.9	294.4	82.8	21.2	218.9	1.8	13.8	5.6	313.6	3.7	73	29.400
BC5		22.1	5.5	1.8	16.6	0.92	16.6	0.92	51.5	16.5	13.8	49.7	42.3	48.8	3.7	115.9	15.4	31.3	46.2	124.2	27.6	72	49.449
WC1		91.1	5.7	1.8	17.3	18.4			162.8	5.7	4.6	17.3	10.1		1.8	702.3	68.1	34	204.2	208	76.4	74	44.234
RC1						0.92	11		66.2	4.6	0.92	13.8	74.6	3.7	8.3	652.4	31	64.4	93.2	1129.8	32.2	76	22.388
RC2															14.7	1140.8	92	567.2	276	2462.9	165.6	77	29.528
RC3								19.3		1.7	9.2	4.8	55.2		35.9			14.7		1089.5	6.4	76	3.545
RC4								69			4.6		460.9	3.7		49.7	1.6	18.4	4.8	563.9	4.6	74	5.627
RC5	0.92		12.6	69	38	293.3	54.3		60.7						134.3	171.1	20.5	236.1	61.4	836.5	189.5	72	33.559
RC6						2.8	1.8		5.5		1.8	0.92	376.8			1.8		1.8	1.8	122.3	12.9	72	4.251
RC7		96.8	1.4	36.8	4.1	146.5	51.5		12		11		18.4	1.8	0.92	244.2	1.1	20.2	3.5	505.7	20.2	70	23.412
RC8	33.1	820.1	40.5	35	121.4	333.1	69	0.92	5.5		28.5	0.92	79.1	13.8		248.4	3.9	13.8	11.7	55.3	2.8	60	36.843
RC9	24.8	5.5	2.7	91.1	8.3	1411.3	100.3	4.6	4.6		115.9		189.5	3.7				44.2	1.8	281.8	1.8	60	15.244

\+ SOIL GROUP DEFINITIONS - TABLE 5

* LANDUSE LISTED IN TABLE 4

Table 11 - SCS RUNOFF CURVE NUMBERS FOR ULTIMATE DEVELOPMENT

SOIL TYPES	B							C							D								
LANDUSE	11	12	13	21	31	51	71	11	12	13	21	31	51	71	11	12	13	21	31	51	71		
% IMPERVIOUS	20	38	75	90	78	0	98	20	38	75	90	78	0	98	20	38	75	90	78	0	98	AVERAGE	AVERAGE
SCS CN VALUE	55	55	55	55	55	55	55	70	70	70	70	70	70	70	77	77	77	77	77	77	77	CN	% IMPERVIOUS
SUBAREAS	AREA (acres)							AREA (acres)							AREA (acres)								
SC1									219.2				170			144.7				332.4		74	15.962
SC2						33			114.9		1.8		332.5			28.9				671.5		74	4.758
SC2													1005							639.8		73	0.000
SC4								3.4					535.2		6.2					819.5		74	0.141
SC5						0.7							1962.8							1665		73	0.000
SC6								68					864.8		9.2					1288		74	0.692
SC7						25							316.4							783.8		75	0.000
SC8								17.4					236.4							751.6		75	0.346
SC9													160.5							567.5		75	0.000
SC10																				917.8		77	0.000
SC11																				340.1		77	0.000
SC12																		77.2		3374.5	11	77	2.318
SC13	68.8	42.3	6.3	18.8	6.3	29.7	6.3								377.1	408.4	34.3	102.8	34.3	163.3	86.7	74	37.761
SC14	318	301.8	80.8	111.3	184.7	137.4	28.8		255.9	72.1	18.4	206	24.8	6.1		556.5	50.1	89.4	114	86.5	18.1	66	48.184
SC15	83.8	13.8	7.5	58.4	11.6	25.5	31.2					3.7			45.1	375.4	11.1	113.3	27.3	14.9	16	71	50.279
SC16	88.5	33.4	10.6	29.3	25.8	14.6	29.7	24.8	53.7	25.1	28.1	70.3	12.5	22.8	346.9	164.1	17.4	13.3	46	15.4	30.8	71	43.867
SC17	75.4	138.1	15.8	875.1	22.3	59.7	31.9	3.7	118.1	14.6	116.6	39	11.2	43.8	95.7	197.8	25.4	369.1	53.5	54.1	37.2	65	69.815
SC18	1.8	315.2	31.3	364.3	71.9	53	58.9	4.6	74.9	8.2	41.6	10.9	32.4	16	6.4	54.8	8.9	140.5	17.3	23	20.4	61	64.230
SC19						7.4		16.6	222.4	41.8	197.5	109	38.7	26.6	6.2	98.6	24.9	118.1	65.5	22.3	13	72	63.010
SC20	62.6	192	33.3	22.6	90.8	21.3	20.1								70.8	114.5	8.5	9.1	19.6	14.7	11.3	63	46.284
SC21	17.6	193.2	29.7	3.7	89	9.1	47.8		282.4	5.3		15.9	2.8	2.8	46	571.5	56.5	192.9	162.2	16.7	110.3	71	55.093
SC22	0.92	134.9	21.3	10.9	60.5	8.7	20.2		65	15.4	20.5	43	8.1	1.6	6.4	275.8	58.3	112.1	153.5	50.9	28.2	71	57.661
SC23		63.7	12.2	9.4	33.3	7.7	21.8									39.9	5.4	9.9	9.5	15.7	3.3	63	55.186
SC24	35	439.4	60.8	59.8	152	73.2	26.3		323.1	33.3	22.1	90.9	26.6	35	12.9	371.3	26.2	22.7	63.5	36	10.4	65	48.636
SC25	9.2	548.6	53.9	129.3	75.5	205.2	52.3		17.2	1.6	4.7	1.6	7.3	1.6	12	407.1	60.9	145.1	147.5	82.4	104.2	65	50.433
SC26		86.9	10.4	23.7	15.5	37.7	7.9		667.8	11	7.8	28.9	9.4	3.8		259.9	76.3	16.5	218.1	26.1	9.2	71	47.944
SC27	622.2	240.1	91.8	746.8	163.3	265.4	283.9	5.5	57.5	4.8	49.9	8.7	23.9	44.6	0.92	264.5	113.2	246.2	297.7	100.3	209.6	63	60.637
SC28		26.4	7.2	8.5	17.8	9.1	11.1		5.6				5.4	0.92		890.1	195.8	157.8	572.4	35.9	83.9	76	59.503
SC29	5.5	732.4	50.2	161.7	109.5	124.5	108.8		76.9	10.1	19.2	21.2	21.6	7.4	0.92	102.8	8.1	5.1	21.4	6.6	7.8	59	50.367
SC30		102.1	3.7	29.4	11	14.7	14.7		294.6	27.8	12.6	77.6	13.9	12.2		415.8	49	74.9	133.2	32.6	52.9	72	52.855
SC31	135.2	1468.2	103.8	276.8	161.7	354.4	84	7.4	584.2	37	58.9	92.7	38.6	50.6	2.8	1011.3	51.3	134.7	119.8	80.9	57.5	64	45.622
SC32	18.4	2.8	1.4	12	4.1	1169	199.6	42.3	318.2	14.7	14.7	44.2	226.3	51.5	47.8	5.5	3.2	21.2	9.7	470.1	72.7	64	20.583
SC33	17.5	12	1.1	41.4	3.5	1171.7	40.5	11		4.4	10.1	13.1	254.9	9.2	5.5			1.8	0.92	7.4	2.8	58	7.992
PS1	4.6					115.9		239.1					917.2		105.8	9.2				1766.1		74	2.324
PS2								152.7	60.3	4	11.9	4	18.9	4	1107.8	174.8	87.2	261.5	87.2	415.1	87.2	76	32.965
PS3															206		18.7	59.9	18.7	89.3	18.7	77	34.536
PS4													330.1							888.3		75	0.000
PS5													40.1					26.7		2201.6		77	1.059
PS6																210.6	20.7	55.7	24.8	88.2	18.6	77	43.778
PS7															58.9	753.4	48.1	128	60	200.2	42.1	77	41.637
PS8																421.8	31.5	72.4	47.7	112.3	72.3	77	47.115
PS9		18.1	1.6	4.9	1.6	8.1	1.6								1.8	1023.9	58.1	187.3	71.1	246.1	124.4	77	45.423
PS10	4.6	237.4	17.3	34.9	28.9	55.4	30.9								235.4	1026	45.4	80.2	86.3	118.5	34.1	73	40.891
LC1	3.7	47.8	3.7		11	2.7									748	1994.2	129.2	183.3	292.9	225.3	223.9	77	42.616
LC2		56.2		196.9	2.8	5.4		15.6	269.5	18.6	26	47.2	20.5	4.4	64.4	999.9	30.7	68.9	52.3	94.9	19.9	73	46.283
MC1								392.4	46	3.8	11.4	3.8	18.1	3.8	424.1	144.3	6.8	20.5	6.8	32.8	6.8	74	25.943
MC2								239	6				4.1		289.6	18.2	1.6	4.7	1.6	7.3	6.2	74	22.077
MC3								151.4	5.5	4.1	12.4	4.1	19.8	4.1	273.1		17.5	52.4	17.5	83	17.5	75	30.122
MC4								262.9	9.2	10.4	31.1	10.4	49.2	10.4	360.2		19.6	58.8	19.6	93.5	21.4	74	29.904
MC5								85.9		7.7	23.2	8.6	36.9	7.7	252		20.6	61.7	20.6	97.9	20.6	75	33.529
MC6															239		21.7	67.1	21.7	103.6	25.4	77	34.751
MC7															1222.7		102.3	306.9	106	491.7	124.4	77	34.073
MC8																762.3	68.3	204.9	68.3	325.3	68.3	77	43.109
MC9															64.4	885.8	74.6	365.2	81.6	338.8	130.1	77	47.682
MC10															5.5	259.8	21.5	46.5	33.4	74	47.7	77	47.215
MC11	0.92	125.1	3.2	17.8	4.1	15	3.2								59.8	418.5	30.1	45.1	62.8	65.6	14.7	73	42.732

Table 11 - SCS RUNOFF CURVE NUMBERS FOR ULTIMATE DEVELOPMENT

SOIL TYPES	B							C							D								
LANDUSE	11	12	13	21	31	51	71	11	12	13	21	31	51	71	11	12	13	21	31	51	71		
% IMPERVIOUS	20	38	75	90	78	0	98	20	38	75	90	78	0	98	20	38	75	90	78	0	98		
SCS CN VALUE	55	55	55	55	55	55	55	70	70	70	70	70	70	70	77	77	77	77	77	77	77	AVERAGE	AVERAGE
SUBAREAS	AREA (acres)							AREA (acres)							AREA (acres)							CN	% IMPERVIOUS
MC12		118.7	3.8	11.5	5.6	18.5	3.8	11	226	11.5	73.1	11.5	54.8	11.5	0.92	357.6	8.2	29.3	11.9	39.4	9.1	71	42.833
MC13	0.92	87.5	5.4	16.1	5.4	25.4	5.4	3.7	105.6	2.5	7.5	4.3	11.8	2.5		74.7	5.6	16.8	8.4	26.8	5.6	67	41.838
EC1															92.7	1207.7	80.9	271.1	88.2	367.5	125	77	43.591
EC2															50.6	998.2	87.4	288.9	87.4	416.3	87.4	77	43.092
EC3	19.3	5.6				5.4									6.4	209.3	21	62.8	31.2	76.3	56.5	76	48.355
EW1								145.5		9.4	28.1	10.3	44.7	9.4	2453		197.5	591.3	211.8	906.7	234.6	77	33.965
EW2															852.3		77.5	232.5	77.5	369	77.5	77	34.053
EW3															1000.4		90.9	431	90.9	433.4	90.9	77	38.182
EW4	3.7					3.7			32.4	1.4	119.1	1.4	6.3	7.8		711.6	64.6	492.7	64.6	307.8	93.1	76	54.127
EW5															87.4	763.8	68.7	426	68.7	327.1	68.7	77	47.700
EW6		269.3	20.9	63.6	20.9	99.4	60.5								13.8	773.8	55.8	325.6	59.5	265.7	103.6	71	48.325
SR1		27.4	2.5	7.5	2.5	11.9	2.5	2.8	500.4	32.1	68	51	107.7	22.7		191.3	13.4	36.1	16.1	57.5	12	71	42.990
SR2						1.8	0.92	22.1	628	21.7	68.6	29	85.4	24.4	34	197.3	12.8	48	15.5	54.3	18.8	72	42.261
SR3	1.8	63.3	2	18	2.9	9.4	2	27.6	116.9	4.7	282.7	6.5	22.3	13	6.4	64.9	6.2	260.3	9.4	21.7	29.4	71	69.286
BC1								40.5	816.1	63.1	87.8	138.1	122	31.1	3.7	165.5	9.6	21.1	16.1	30.6	7.3	71	44.589
BC2								98.4	186	11.7	30.4	24.1	26.5	23	49.7	785	37.7	132.2	57.5	132.5	38.8	75	43.232
BC3								81.9	983.7	57.7	433	115.3	138.5	124.7	1.8	485.9	19.8	320.9	33.1	62.8	14	72	54.419
BC4								22.1	348.4	35.8	94.9	80.4	64.5	96.3	21.2	377.4	16.2	57	20	68.7	18.1	73	49.250
BC5		22.1	5.5	1.8	16.6	0.92	16.6	0.92	72.9	18.4	19.6	51.6	9.4	50.7	3.7	178.7	21.1	48.4	51.9	27.2	33.3	72	60.455
WC1		100.4	6.5	4.3	18.1	4.2	0.8		167.9	5.7	4.6	17.3	5		1.8	807.4	77.7	62.7	213.8	45.4	86	74	50.346
RC1						0.92	11		107.2	8.3	12.1	17.5	11.2	7.5	629.7	652.4	87.5	233.9	149.7	169.5	88.7	76	43.130
RC2															1369.3	1140.8	215.1	936.6	399	369.4	288.7	77	48.863
RC3								49.7		4.5	17.5	7.6	8.3	2.8	635.1		54.5	178.1	54.5	163.4	60.9	76	37.846
RC4								322.5		23	73.7	23	69.1	26.7	310.1	49.7	29.8	103	33	84.6	32.8	74	37.767
RC5	162.2		27.3	113	52.7	44	69		60.7						594.4	171.1	62.3	361.6	103.2	125.5	231.3	72	52.777
RC6						2.8	1.8	207.2	5.5	18.8	58.3	19.8	56.5	18.8	67.3	1.8	6.1	20.1	7.9	18.3	19	72	39.118
RC7	80.6	96.8	8.7	58.8	11.4	22	58.8	10.1	12	0.92	13.8	0.92	2.8	2.7	279	244.2	26.4	96	28.8	75.9	45.5	70	44.533
RC8	216.3	820.1	57.2	85	138	50	85.7	44.4	5.5	4	40.3	4.9	11.9	17.8	30.4	248.4	6.7	22.1	14.5	8.3	5.6	60	45.883
RC9	801	5.5	73.3	302.8	78.9	211.7	170.9	108.8	4.6	9.5	144.3	9.5	28.4	13.2	155		14.1	86.5	15.9	42.3	15.9	60	45.682

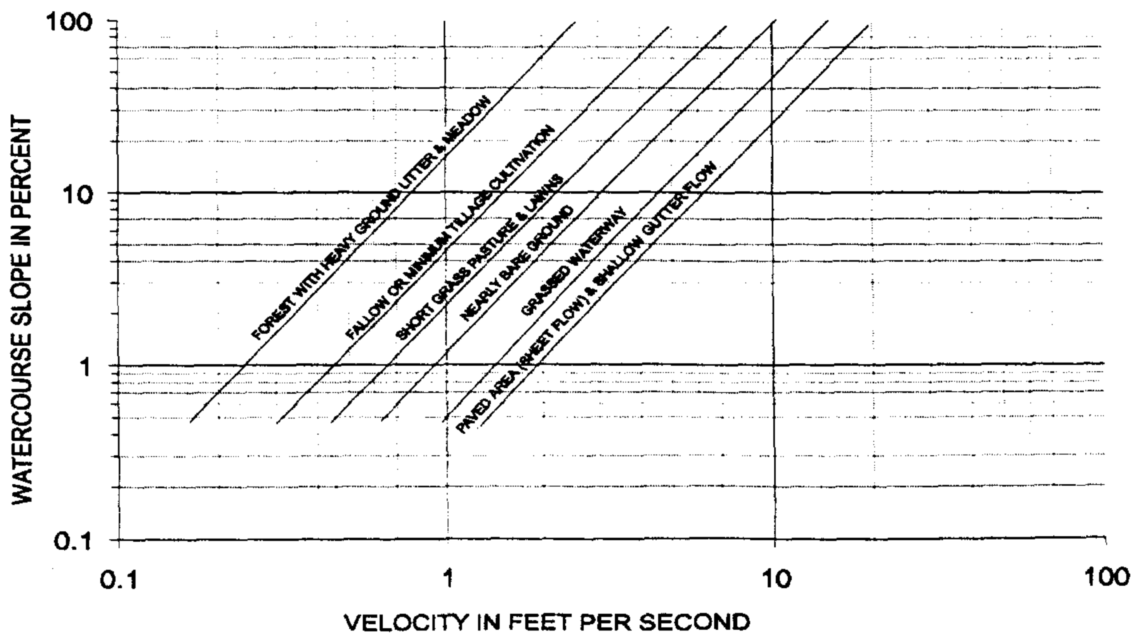
at recording precipitation gages located within the watershed and allows complete simulations of storms extending over a period of several days.

II. HEC-1 Model

Subareas

The Watershed was divided into eighty five (85) subareas. The upper watershed is defined by 57 subareas and covers about 139 square miles of area. The lower watershed has been divided into the lower Salado and Rosillo drainage areas. The lower Salado consists of 19 subareas and Rosillo consists of 9 subareas which cover about 51 square miles of area. Rainfall runoff was computed by determining the time of concentration of the overland flow within a subarea. Overland flows from each subarea are generated from sheet flow, shallow concentrated flow and channelized flows. Travel time is computed by dividing the travel distance by the average velocity of the overland storm water flow. Travel distances are established by determining a path for storm water flow through a subarea. Figure 11 - "Average Velocities for Estimating Travel Time for Overland Flow" was used in determining velocities for sheet flow and shallow concentrated flow. The average velocities for channel flow conditions have been estimated based on Manning's uniform flow equation. Travel times were computed for each of sheet flow, shallow concentrated flow and channelized flow. The Time of Concentration for each subarea is the sum of the three individual travel times. The SCS Lag Time, as required for use in the SCS unit hydrograph method, is equal to sixty percent of the Time of Concentration.

FIGURE 11. AVERAGE VELOCITIES FOR ESTIMATING TRAVEL TIME FOR OVERLAND FLOW.



"The Effects of Urbanization on Small Watersheds"

The reach routings that route the subarea runoffs from node to node along Salado Creek and the tributaries incorporate channelized flows. Computations are presented in Tables 12 and 13 - Summary of Time of Concentration and Reach Routing Calculations for existing conditions and ultimate development conditions.

Dams and Reservoirs

Within the Salado Creek Watershed, there are thirteen (13) existing Soil Conservation Service Floodwater Retarding Dams. These floodwater retarding structures were constructed for flood control, for the purpose of reducing flood flows and sediment loadings downstream. Included in the HEC-1 model analysis were floodwater storage capacities and outflow characteristics for each SCS structure using the Modified Puls method. Storage-Capacity-Discharge tables were developed from Engineering plans, reports, and previous hydraulic simulations prepared by the SCS and obtained from the San Antonio River Authority. These plans and reports are included in Appendix E.

Storm Simulation

As previously stated, two historical storms were selected for verification of the models. These storms occurred on April 4-5, 1991 and May 5-6, 1993. From the data for each storm, three precipitation recordings were used for interval distributions. Precipitation data for each of the two storm events was entered in the HEC-1 model as weighted precipitation gages. Total storm precipitation determined from the rainfall isohyetals were input as weighted averages for each subarea based upon the nearest precipitation gage. Rainfall patterns were based on three precipitation gages. These three gages recorded the rainfall in intervals used in the HEC-1 model. These gages are located at SCS Floodwater Retarding Dam No. 5, the U.S.G.S. Salado Creek (Upper Station), and at Spur 122 and Salado Creek. These gages were used for storm simulation of the April 4 - 5, 1991 event.

An antecedent soil moisture condition II was initially assumed for the storm of April 4-5, 1991. The results obtained from the HEC-1 model were larger than recorded data from April 4-5, 1991. Further review of rainfall records for the area indicated that the soil moisture conditions were drier than condition II. Re-running the HEC-1 model using antecedent moisture condition I, produced results that were lower than recorded data from April 4-5, 1991. It was thus determined that soil conditions prior to the April 4 - 5, 1991 storm were in between the two conditions. An average of the two conditions was used and the results of the hydrologic model compared very favorably to the recorded data of the April 4-5, 1991 storm.

Likewise, three precipitation gage intervals were used for the May 5-6, 1993 storm, however the locations of the precipitation gages were not evenly distributed. One gage is located at the San Antonio International Airport, the second at the U.S.G.S. Salado Creek (Upper Station) and the third at 3002 E. Southcross. Soil antecedent moisture conditions were reset to antecedent moisture condition II for the May 5-6, 1993 storm. The model

REVISED 7/9/96

Table 12 - SUMMARY OF TIME OF CONCENTRATION AND REACH ROUTING CALCULATIONS
Salado Creek Watershed Drainage Master Plan
Existing Conditions Land Use

Based on procedures described in "Urban Hydrology for Small Watersheds", TR-55, USDA Soil Conservation Service, June 1986.

SUB-WATER-SHED ID	SHEET FLOW					SHALLOW CONCENTRATED FLOW					CHANNEL/PIPE FLOW					TIME OF CONC. Minutes	SCS LAG TIME Hours	REACH ROUTING TIME				
	Length Feet	Mannings "n" -	Slope Ft/Ft	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Channel Type -	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Travel Time Minutes			Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Routing Time Hours
SC1	250	0.110	0.088	1.40	3.0	1500	0.047	unpaved	3.30	7.6	11000	0.012	0.050	10.1	18.2	29	0.288	-	-	-	-	-
SC2	300	0.110	0.007	0.40	12.5	1050	0.062	unpaved	3.80	4.6	11600	0.013	0.050	10.5	18.5	36	0.356	8600	0.007	0.045	7.6	0.312
SC3	200	0.110	0.100	1.50	2.2	1100	0.055	unpaved	3.50	5.2	14000	0.014	0.050	10.9	21.5	29	0.289	-	-	-	-	-
SC4	250	0.110	0.040	0.95	4.4	1400	0.036	unpaved	2.90	8.0	12600	0.011	0.050	9.6	21.8	34	0.342	7600	0.005	0.045	6.5	0.327
SC5	250	0.110	0.040	0.95	4.4	700	0.057	unpaved	3.60	3.2	10800	0.006	0.050	7.1	25.3	33	0.329	5400	0.007	0.045	6.8	0.219
SC6	300	0.110	0.033	0.85	5.9	1250	0.056	unpaved	3.55	5.9	14800	0.013	0.050	10.5	23.6	35	0.353	5900	0.006	0.045	6.3	0.258
SC7	200	0.110	0.100	1.50	2.2	950	0.079	unpaved	4.30	3.7	7200	0.018	0.050	12.3	9.7	16	0.157	7600	0.005	0.045	6.5	0.327
SC8	250	0.110	0.070	1.25	3.3	1200	0.108	unpaved	5.00	4.0	9500	0.015	0.050	11.2	14.1	21	0.214	-	-	-	-	-
SC9	250	0.110	0.008	0.42	9.9	200	0.350	unpaved	9.00	0.4	6800	0.024	0.050	15.3	7.4	30	0.303	2500	0.004	0.045	5.8	0.120
						800	0.106	unpaved	5.00	2.7	6000	0.012	0.050	10.1	9.9							
SC10	300	0.110	0.037	0.91	5.5	400	0.250	unpaved	7.60	0.9	800	0.050	0.050	22.0	0.6	30	0.301	-	-	-	-	-
						580	0.103	unpaved	4.90	2.0	4600	0.022	0.050	14.6	5.2							
											10000	0.013	0.050	10.5	15.9							
SC11	250	0.110	0.007	0.40	10.4	600	0.233	unpaved	7.50	1.3	3200	0.030	0.050	17.1	3.1	24	0.242	10700	0.005	0.05	5.8	0.511
						950	0.058	unpaved	3.70	4.3	2800	0.010	0.050	9.2	5.1							
SC12	150	0.110	0.013	0.55	4.5	850	0.082	unpaved	4.40	3.2	10200	0.010	0.050	9.2	18.5	59	0.593	13500	0.0054	0.055	7.8	0.480
						2150	0.033	unpaved	2.75	13.0	11000	0.010	0.050	9.2	20.0							
SC13	200	0.110	0.040	0.95	3.5	4100	0.025	unpaved	2.40	28.5	12400	0.005	0.060	5.4	38.2	70	0.702	19800	0.0044	0.055	6.8	0.808
SC14	300	0.080	0.010	0.70	7.1	2800	0.014	unpaved	1.80	25.9	19400	0.004	0.060	4.7	68.6	110	1.105	5000	0.0033	0.065	4.9	0.283
						850	0.012	unpaved	1.60	8.9												
SC15	250	0.080	0.012	0.76	5.5	1400	0.029	unpaved	2.55	9.2	4550	0.016	0.060	9.4	8.0	44	0.444	-	-	-	-	-
						1400	0.029	paved	3.45	6.8	5200	0.006	0.060	5.8	15.0							
SC16	200	0.080	0.073	1.90	1.8	1200	0.035	unpaved	2.80	7.1	11300	0.014	0.060	8.8	21.3	30	0.302	14200	0.003	0.06	4.4	0.900
SC17	300	0.080	0.010	0.70	7.1	1100	0.038	unpaved	3.00	6.1	11600	0.005	0.060	5.3	36.7	97	0.974	9700	0.0018	0.045	6.1	0.444
						3500	0.019	paved	2.80	20.8												
						2800	0.013	unpaved	1.75	26.7												
SC18	300	0.080	0.013	0.80	6.3	6600	0.003	unpaved	1.00	110.0	6900	0.004	0.065	4.4	26.4	170	1.699	-	-	-	-	-
						1100	0.027	paved	3.25	5.6												
						2200	0.012	unpaved	1.70	21.6												
SC19	200	0.080	0.040	1.42	2.3	5200	0.019	paved	2.80	31.0	8300	0.008	0.065	6.2	22.5	56	0.558	7900	0.0025	0.08	3.8	0.576
SC20	250	0.080	0.008	0.64	6.5	3350	0.018	paved	2.70	20.7	3300	0.003	0.065	3.8	14.6	44	0.439	-	-	-	-	-
						500	0.070	unpaved	4.00	2.1												
SC21	300	0.110	0.007	0.40	12.5	1300	0.025	unpaved	2.45	8.8	9000	0.004	0.065	4.4	34.5	85	0.851	4300	0.002	0.075	2.2	0.553
						4050	0.013	paved	2.30	29.3												
SC22	400	0.110	0.013	0.55	12.1	1600	0.038	paved	3.80	7.0	10800	0.008	0.065	6.1	29.6	49	0.488	-	-		-	-
SC23	200	0.110	0.040	0.95	3.5	2300	0.014	paved	2.40	16.0	300	0.017	0.065	8.9	0.6	63	0.630	7700	0.0023	0.065	2.5	0.846
						1300	0.019	unpaved	2.20	9.8	3700	0.001	0.075	1.9	33.1							
SC24	300	0.160	0.020	0.67	7.5	4100	0.010	paved	2.00	34.2	2100	0.028	0.075	9.9	3.6	89	0.892	17400	0.0015	0.07	2.2	2.188
											11000	0.005	0.075	4.2	44.0							
SC25	400	0.160	0.008	0.42	15.9	3500	0.026	paved	3.20	18.2	17100	0.002	0.075	2.6	108.2	168	1.677	-	-	-	-	-
						3050	0.010	paved	2.00	25.4												

Table 12 - SUMMARY OF TIME OF CONCENTRATION AND REACH ROUTING CALCULATIONS
Salado Creek Watershed Drainage Master Plan
Existing Conditions Land Use

Based on procedures described in "Urban Hydrology for Small Watersheds", TR-55, USDA Soil Conservation Service, June 1986.

SUB-WATER-SHED	SHEET FLOW					SHALLOW CONCENTRATED FLOW					CHANNEL/PIPE FLOW					TIME OF CONC.	SCS LAG TIME	REACH ROUTING TIME				
ID	Length Feet	Mannings "n" -	Slope Ft/Ft	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Channel Type -	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Travel Time Minutes	Minutes	Hours	Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Routing Time Hours
SC26	250	0.160	0.008	0.42	9.9	550	0.027	unpaved	1.70	5.4	10200	0.009	0.075	5.6	30.4	74	0.736	19500	0.0014	0.065	2.2	2.471
						4350	0.018	paved	2.60	27.9												
SC27	250	0.160	0.003	0.28	14.9	5300	0.003	paved	1.00	88.3	450	0.011	0.075	6.2	1.2	220	2.196	-	-	-	-	-
						2750	0.013	paved	2.30	19.9	350	0.014	0.075	7.0	0.8							
						1100	0.005	unpaved	0.70	26.2	13200	0.003	0.075	3.2	68.2							
SC28	350	0.160	0.003	0.28	20.8	3300	0.006	paved	1.55	35.5	3000	0.007	0.075	4.9	10.1	127	1.265	17600	0.0013	0.065	2.2	2.208
						1500	0.009	unpaved	0.95	26.3	400	0.025	0.075	9.3	0.7							
						1150	0.020	paved	2.80	6.8	9700	0.011	0.075	6.2	26.2							
SC29	300	0.200	0.003	0.28	17.9	4800	0.001	paved	1.00	80.0	2400	0.027	0.080	9.1	4.4	228	2.276	-	-	-	-	-
						1800	0.006	unpaved	0.80	37.5	9200	0.001	0.080	1.7	87.8							
SC30	200	0.200	0.005	0.34	9.8	1400	0.009	paved	1.90	12.3	11400	0.007	0.080	4.6	41.1	118	1.183	22900	0.0018	0.07	2.3	2.802
						4300	0.016	unpaved	1.30	55.1												
SC31	300	0.200	0.007	0.21	23.8	1300	0.012	unpaved	1.10	19.7	9000	0.013	0.080	6.3	23.8	163	1.632	18000	0.002	0.075	2.4	2.100
											14200	0.002	0.080	2.5	95.8							
SC32	300	0.200	0.004	0.16	31.3	4400	0.014	unpaved	1.20	61.1	15400	0.003	0.085	2.8	90.7	183	1.830					
SC33	300	0.200	0.010	0.25	20.0	2300	0.016	unpaved	1.30	29.5	7200	0.004	0.080	3.5	34.4	136	1.357	-	-	-	-	-
											9400	0.003	0.080	3.0	51.8							
PANTHER SPRINGS CREEK																						
PS1	200	0.110	0.117	1.60	2.1	1000	0.057	unpaved	3.60	4.6	5400	0.024	0.050	14.2	6.3	44	0.435	5200	0.005	0.045	6.1	0.238
											15200	0.008	0.050	8.3	30.5							
PS2	300	0.110	0.050	1.10	4.5	1400	0.100	unpaved	4.90	4.8	13400	0.011	0.050	9.6	23.2	33	0.325	6900	0.008	0.045	8.2	0.234
PS3	300	0.110	0.033	0.85	5.9	2700	0.024	unpaved	2.40	18.8	6200	0.007	0.050	7.7	13.5	38	0.381	-	-	-	-	-
PS4	200	0.110	0.133	1.75	1.9	1000	0.078	unpaved	4.30	3.9	9300	0.016	0.050	11.6	13.4	19	0.191	11400	0.01	0.045	9.1	0.347
PS5	300	0.110	0.050	1.10	4.5	1200	0.087	unpaved	4.50	4.4	8600	0.020	0.050	13.9	10.3	19	0.193	8100	0.003	0.045	5.0	0.450
PS6	300	0.110	0.033	0.85	5.9	3300	0.060	unpaved	3.70	14.9	5100	0.006	0.050	7.1	12.0	33	0.327	-	-	-	-	-
PS7	150	0.110	0.007	0.40	6.3	2300	0.052	paved	4.50	8.5	12850	0.012	0.050	10.1	21.3	36	0.361	14200	0.0029	0.05	4.9	0.810
PS8	300	0.110	0.017	0.62	8.1	1600	0.029	unpaved	2.50	10.7	8200	0.005	0.050	6.5	21.1	46	0.460	7300	0.0039	0.05	3.9	0.525
						1200	0.026	paved	3.20	6.3												
PS9	300	0.110	0.043	0.98	5.1	1600	0.051	unpaved	3.40	7.8	4000	0.020	0.050	12.7	5.3	38	0.384	4200	0.006	0.045	6.7	0.175
											400	0.010	0.050	8.9	0.7			5500	0.004	0.05	5.4	0.282
											11200	0.011	0.050	9.6	19.4							
PS10	300	0.080	0.007	0.60	8.3	3600	0.013	unpaved	1.80	33.3	8800	0.013	0.050	10.2	14.4	56	0.560	4000	0.0013	0.06	2.1	0.540
LORENCE CREEK																						
LC1	300	0.110	0.010	0.47	10.6	1000	0.035	unpaved	2.80	6.0	12200	0.014	0.060	8.8	23.0	70	0.704	15100	0.004	0.055	4.7	0.887
											1600	0.006	0.060	5.8	4.6							
											9800	0.007	0.060	6.2	26.2							
LC2	300	0.080	0.027	1.20	4.2	2400	0.027	unpaved	2.50	16.0	14000	0.004	0.060	4.7	49.5	83	0.827					
						2500	0.027	paved	3.20	13.0												
MUD CREEK																						
MC1	300	0.110	0.067	1.20	4.2	1400	0.036	unpaved	2.90	8.0	11000	0.011	0.050	9.6	19.0	31	0.313	-	-	-	-	-
MC2	200	0.110	0.125	1.70	2.0	1800	0.044	unpaved	3.10	9.7	3200	0.017	0.050	12.0	4.5	23	0.229	9000	0.008	0.045	8.4	0.297

Table 12 - SUMMARY OF TIME OF CONCENTRATION AND REACH ROUTING CALCULATIONS REVISED 7/9/96
Salado Creek Watershed Drainage Master Plan
Existing Conditions Land Use

Based on procedures described in "Urban Hydrology for Small Watersheds", TR-55, USDA Soil Conservation Service, June 1986.

SUB-WATER-SHED ID	SHEET FLOW					SHALLOW CONCENTRATED FLOW					CHANNEL/PIPE FLOW					TIME OF CONC.	SCS LAG TIME	REACH ROUTING TIME				
	Length Feet	Mannings "n" -	Slope Ft/Ft	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Channel Type -	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Travel Time Minutes	Minutes	Hours	Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Routing Time Hours
											4600	0.015	0.050	11.2	6.8							
MC3	200	0.110	0.100	1.50	2.2	1500	0.067	unpaved	3.90	6.4	2800	0.014	0.050	10.9	4.3	28	0.284	-	-	-	-	-
											6600	0.006	0.050	7.1	15.5							
MC4	200	0.110	0.075	1.30	2.6	2500	0.050	unpaved	3.40	12.3	7400	0.014	0.050	10.9	11.4	26	0.262	-	-	-	-	-
MC5	150	0.110	0.067	1.25	2.0	900	0.044	unpaved	3.10	4.8	10500	0.016	0.050	11.6	15.1	22	0.219	3500	0.006	0.045	7.3	0.134
MC6	250	0.110	0.020	0.67	6.2	1600	0.053	unpaved	3.50	7.6	5800	0.016	0.050	11.6	8.3	22	0.222	5800	0.006	0.045	7.3	0.221
MC7	250	0.110	0.016	0.60	6.9	1700	0.040	unpaved	3.00	9.4	9200	0.014	0.050	10.9	14.1	31	0.305	11500	0.007	0.045	7.9	0.406
MC8	200	0.110	0.050	1.10	3.0	1500	0.080	unpaved	4.40	5.7	11800	0.013	0.050	10.5	18.8	28	0.275	12700	0.0054	0.05	6.2	0.567
MC9	200	0.110	0.050	1.10	3.0	3400	0.040	unpaved	3.00	18.9	15700	0.008	0.050	8.0	32.7	55	0.546	12500	0.0056	0.055	5.8	0.603
MC10	250	0.110	0.016	0.60	6.9	350	0.017	unpaved	2.00	2.9	4800	0.006	0.060	5.8	13.8	32	0.321	9600	0.0035	0.055	5.1	0.523
						1300	0.050	paved	4.50	4.8												
						750	0.053	unpaved	3.50	3.6												
MC11	200	0.035	0.025	1.60	2.1	700	0.043	unpaved	3.10	3.8	12600	0.005	0.060	5.3	39.8	49	0.493	-	-	-	-	-
						900	0.044	paved	4.10	3.7												
MC12	300	0.035	0.067	2.55	2.0	850	0.029	paved	3.50	4.0	14400	0.009	0.060	7.1	33.9	45	0.446	6800	0.0037	0.06	4.5	0.418
						650	0.023	unpaved	2.30	4.7												
MC13	250	0.035	0.008	0.90	4.6	3200	0.023	paved	3.10	17.2	8200	0.005	0.060	5.3	25.9	52	0.519	-	-	-	-	-
						600	0.025	unpaved	2.40	4.2												
ELM CREEK																						
EC1	300	0.110	0.020	0.67	7.5	1900	0.052	unpaved	3.60	8.8	20000	0.008	0.050	8.0	41.6	58	0.579	-	-	-	-	-
EC2	200	0.110	0.067	1.20	2.8	2300	0.045	unpaved	3.40	11.3	22800	0.009	0.050	8.5	44.8	59	0.588	8100	0.0045	0.045	4.3	0.524
EC3	300	0.110	0.007	0.40	12.5	1100	0.027	unpaved	2.60	7.1	9800	0.013	0.050	10.2	16.0	36	0.356	-	-	-	-	-
ELM WATERHOLE CREEK																						
EW1	200	0.110	0.150	1.80	1.9	800	0.088	unpaved	4.50	3.0	8200	0.017	0.050	12.0	11.4	91	0.912	-	-	-	-	-
											31200	0.006	0.050	6.9	75.0							
EW2	300	0.110	0.007	0.40	12.5	1800	0.047	unpaved	3.30	9.1	5200	0.018	0.050	12.3	7.0	52	0.515	7100	0.008	0.045	8.2	0.241
											12600	0.010	0.050	9.2	22.9							
EW3	300	0.110	0.020	0.67	7.5	2800	0.023	unpaved	2.30	20.3	4800	0.007	0.050	7.5	10.7	38	0.384	2200	0.014	0.045	10.8	0.057
EW4	200	0.110	0.060	1.20	2.8	400	0.050	unpaved	3.40	2.0	9000	0.013	0.050	10.5	14.3	19	0.191	8000	0.003	0.05	4.0	0.560
EW5	300	0.110	0.007	0.40	12.5	1700	0.042	unpaved	3.10	9.1	11600	0.015	0.050	11.2	17.2	39	0.388	12400	0.003	0.05	4.0	0.869
EW6	250	0.110	0.008	0.42	9.9	1400	0.036	unpaved	2.80	8.3	17500	0.007	0.050	7.5	39.0	57	0.572	-	-		-	-
STAHL ROAD																						
SR1	300	0.080	0.017	0.90	5.6	3000	0.027	paved	3.30	15.2	9200	0.010	0.060	7.5	20.6	41	0.413	10100	0.006	0.045	6.7	0.421
SR2	300	0.080	0.017	0.90	5.6	1200	0.026	unpaved	2.50	8.0	10000	0.006	0.060	5.8	28.9	46	0.462	12200	0.006	0.045	6.7	0.509
						500	0.012	paved	2.20	3.8												
SR3	300	0.080	0.017	0.90	5.6	2800	0.025	unpaved	2.40	19.4	10200	0.006	0.065	5.3	31.9	57	0.569	-	-	-	-	-
BIETEL CREEK																						
BC1	300	0.110	0.023	0.71	7.0	2300	0.032	unpaved	2.80	13.7	12800	0.009	0.060	7.1	30.2	51	0.509	-	-	-	-	-
BC2	300	0.110	0.057	1.20	4.2	1400	0.041	unpaved	3.00	7.8	11200	0.010	0.060	7.5	25.0	37	0.370	16400	0.0043	0.045	6.0	0.760
BC3	300	0.080	0.013	0.80	6.3	1400	0.029	unpaved	2.60	9.0	17900	0.008	0.060	6.7	44.7	60	0.599	-	-	-	-	-
BC4	300	0.080	0.010	0.70	7.1	2800	0.045	unpaved	3.20	14.6	11800	0.011	0.070	6.7	29.3	61	0.609	8600	0.0035	0.045	6.8	0.351

Table 12 - SUMMARY OF TIME OF CONCENTRATION AND REACH ROUTING CALCULATIONS
Salado Creek Watershed Drainage Master Plan
Existing Conditions Land Use

Based on procedures described in "Urban Hydrology for Small Watersheds", TR-55, USDA Soil Conservation Service, June 1986.

SUB-	SHEET FLOW					SHALLOW CONCENTRATED FLOW					CHANNEL/PIPE FLOW					TIME	SCS	REACH ROUTING TIME				
WATER-SHED ID	Length Feet	Mannings "n" -	Slope Ft/Ft	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Channel Type -	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Travel Time Minutes	OF CONC. Minutes	LAG TIME Hours	Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Routing Time Hours
											2600	0.007	0.070	5.3	8.2							
											900	0.022	0.070	9.5	1.6							
BC5	300	0.080	0.010	0.70	7.1	500	0.018	unpaved	2.10	4.0	10850	0.009	0.070	6.0	30.2	61	0.606	-	-	-	-	-
						2550	0.013	paved	2.20	19.3												
WALZEM CREEK																						
WC1	300	0.110	0.073	1.30	3.8	5300	0.040	unpaved	3.00	29.4	19000	0.007	0.075	4.9	64.3	98	0.976	-	-	-	-	-
ROSILLO CREEK																						
RC1	300	0.110	0.050	1.10	4.5	1200	0.021	paved	2.80	7.1	8600	0.008	0.065	6.1	23.6	84	0.840	20900	0.003	0.05	4.1	1.419
						1150	0.009	paved	1.90	10.1	9400	0.005	0.065	4.8	32.6							
						1300	0.032	paved	3.60	6.0												
RC2	300	0.110	0.004	0.30	16.7	3200	0.004	paved	1.10	48.5	6600	0.003	0.065	3.7	29.6	214	2.136	-	-	-	-	-
											5000	0.006	0.065	5.3	15.8							
											18800	0.002	0.065	3.0	103.1							
RC3	400	0.110	0.003	0.24	27.8	2600	0.015	unpaved	1.90	22.8	8800	0.003	0.065	3.7	39.4	120	1.199	9700	0.003	0.05	4.1	0.659
						1200	0.004	unpaved	1.10	18.2												
						400	0.005	paved	1.40	4.8												
						500	0.006	unpaved	1.20	6.9												
RC4	300	0.110	0.003	0.17	29.4	2800	0.012	unpaved	1.60	29.2	5100	0.008	0.070	5.6	15.1	156	1.560	-	-	-	-	-
											5400	0.000	0.070	1.1	82.3							
RC5	300	0.110	0.003	0.17	29.4	650	0.005	unpaved	1.10	9.8	15400	0.002	0.070	2.8	91.0	158	1.582	-	-	-	-	-
											8200	0.006	0.070	4.9	28.0							
RC6	300	0.110	0.012	0.51	9.8	2050	0.020	unpaved	2.20	15.5	7400	0.008	0.075	5.3	23.4	49	0.487	14900	0.002	0.055	3.0	1.363
RC7	300	0.110	0.003	0.25	20.0	1400	0.019	unpaved	2.10	11.1	14400	0.004	0.075	3.7	64.4	96	0.955	-	-	-	-	-
RC8	450	0.110	0.003	0.25	30.0	16500	0.003	paved	1.30	211.5	3600	0.008	0.080	4.9	12.1	256	2.565	25600	0.003	0.055	3.7	1.912
											400	0.025	0.080	8.7	0.8							
											600	0.008	0.080	4.9	2.0							
RC9	400	0.160	0.003	0.18	37.0	1500	0.006	unpaved	1.20	20.8	24800	0.002	0.080	2.5	167.4	242	2.425	-	-	-	-	-
						1050	0.009	paved	1.90	9.2												
						1300	0.031	unpaved	2.70	8.0												

Table 13 - SUMMARY OF TIME OF CONCENTRATION AND REACH ROUTING CALCULATIONS
Salado Creek Watershed Drainage Master Plan
Ultimate Development Land Use

Based on procedures described in "Urban Hydrology for Small Watersheds", TR-55, USDA Soil Conservation Service, June 1986.

SUB-WATER-SHED	SHEET FLOW					SHALLOW CONCENTRATED FLOW					CHANNEL/PIPE FLOW					TIME OF CONC.	SCS LAG TIME	REACH ROUTING TIME				
ID	Length Feet	Mannings "n" -	Slope Ft/Ft	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Channel Type -	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Travel Time Minutes	Minutes	Hours	Length Feet	Slope Ft/Ft	Mannings "n" -	Vel. Ft/Sec	Routing Time Hours
SC1	250	0.110	0.088	1.40	3.0	1500	0.047	unpaved	3.30	7.6	11000	0.012	0.050	10.1	18.2	29	0.288	-	-	-	-	-
SC2	300	0.110	0.007	0.40	12.5	1050	0.062	unpaved	3.80	4.6	11600	0.013	0.050	10.5	18.5	36	0.356	8600	0.007	0.045	7.6	0.312
SC3	200	0.110	0.100	1.50	2.2	1100	0.055	unpaved	3.50	5.2	14000	0.014	0.050	10.9	21.5	29	0.289	-	-	-	-	-
SC4	250	0.110	0.040	0.95	4.4	1400	0.036	unpaved	2.90	8.0	12600	0.011	0.050	9.6	21.8	34	0.342	7600	0.005	0.045	6.5	0.327
SC5	250	0.110	0.040	0.95	4.4	700	0.057	unpaved	3.60	3.2	10800	0.006	0.050	7.1	25.3	33	0.329	5400	0.007	0.045	6.8	0.219
SC6	300	0.110	0.033	0.85	5.9	1250	0.056	unpaved	3.55	5.9	14800	0.013	0.050	10.5	23.6	35	0.353	5900	0.006	0.045	6.3	0.258
SC7	200	0.110	0.100	1.50	2.2	950	0.079	unpaved	4.30	3.7	7200	0.018	0.050	12.3	9.7	16	0.157	7600	0.005	0.045	6.5	0.327
SC8	250	0.110	0.070	1.25	3.3	1200	0.108	unpaved	5.00	4.0	9500	0.015	0.050	11.2	14.1	21	0.214	-	-	-	-	-
SC9	250	0.110	0.008	0.42	9.9	200	0.350	unpaved	9.00	0.4	6800	0.024	0.050	15.3	7.4	30	0.303	2500	0.004	0.045	5.8	0.120
						800	0.106	unpaved	5.00	2.7	6000	0.012	0.050	10.1	9.9							
SC10	300	0.110	0.037	0.91	5.5	400	0.250	unpaved	7.60	0.9	800	0.050	0.050	22.0	0.6	30	0.301	-	-	-	-	-
						580	0.103	unpaved	4.90	2.0	4600	0.022	0.050	14.6	5.2							
											10000	0.013	0.050	10.5	15.9							
SC11	250	0.110	0.007	0.40	10.4	600	0.233	unpaved	7.50	1.3	3200	0.030	0.050	17.1	3.1	24	0.242	10700	0.005	0.05	5.8	0.511
						950	0.058	unpaved	3.70	4.3	2800	0.010	0.050	9.2	5.1							
SC12	150	0.110	0.013	0.55	4.5	850	0.082	unpaved	4.40	3.2	10200	0.010	0.050	9.2	18.5	58	0.582	13500	0.0054	0.055	7.8	0.480
						1900	0.033	unpaved	2.75	11.5	11250	0.010	0.050	9.2	20.4							
SC13	200	0.110	0.040	0.95	3.5	3600	0.025	unpaved	2.40	25.0	13000	0.005	0.060	5.4	40.1	69	0.686	19800	0.0044	0.055	6.8	0.808
SC14	300	0.080	0.010	0.70	7.1	2500	0.014	unpaved	1.80	23.1	19700	0.004	0.060	4.7	69.6	109	1.088	5000	0.0033	0.065	4.9	0.283
						850	0.012	unpaved	1.60	8.9												
SC15	250	0.080	0.012	0.76	5.5	1400	0.029	unpaved	2.55	9.2	4550	0.016	0.060	9.4	8.0	44	0.444	-	-	-	-	-
						1400	0.029	paved	3.45	6.8	5200	0.006	0.060	5.8	15.0							
SC16	200	0.080	0.073	1.90	1.8	1200	0.035	unpaved	2.80	7.1	11300	0.014	0.060	8.8	21.3	30	0.302	14200	0.003	0.06	4.4	0.900
SC17	300	0.080	0.010	0.70	7.1	1100	0.038	unpaved	3.00	6.1	12400	0.005	0.060	5.3	39.2	94	0.941	9700	0.0018	0.045	6.1	0.444
						3000	0.019	paved	2.80	17.9												
						2500	0.013	unpaved	1.75	23.8												
SC18	300	0.080	0.013	0.80	6.3	6000	0.003	unpaved	1.00	100.0	7500	0.004	0.065	4.4	28.7	162	1.622	-	-	-	-	-
						1100	0.027	paved	3.25	5.6												
						2200	0.012	unpaved	1.70	21.6												
SC19	200	0.080	0.040	1.42	2.3	4600	0.019	paved	2.80	27.4	8900	0.008	0.065	6.2	24.1	54	0.538	7900	0.0025	0.08	3.8	0.576
SC20	250	0.080	0.008	0.64	6.5	3000	0.018	paved	2.70	18.5	3650	0.003	0.065	3.8	16.1	43	0.432	-	-	-	-	-
						500	0.070	unpaved	4.00	2.1												
SC21	300	0.110	0.007	0.40	12.5	1300	0.025	unpaved	2.45	8.8	9450	0.004	0.065	4.4	36.2	84	0.836	4300	0.002	0.075	2.2	0.553
						3600	0.013	paved	2.30	26.1												
SC22	400	0.110	0.013	0.55	12.1	1600	0.038	paved	3.80	7.0	10800	0.008	0.065	6.1	29.6	49	0.488	-	-	-	-	-
SC23	200	0.110	0.040	0.95	3.5	2000	0.014	paved	2.40	13.9	600	0.017	0.065	8.9	1.1	61	0.615	7700	0.0023	0.065	2.5	0.846
						1300	0.019	unpaved	2.20	9.8	3700	0.001	0.075	1.9	33.1							
SC24	300	0.160	0.020	0.67	7.5	3600	0.010	paved	2.00	30.0	2600	0.028	0.075	9.9	4.4	86	0.859	17400	0.0015	0.07	2.2	2.188
											11000	0.005	0.075	4.2	44.0							
SC25	400	0.160	0.008	0.42	15.9	3100	0.026	paved	3.20	16.1	17800	0.002	0.075	2.6	112.6	168	1.676	-	-	-	-	-
						2750	0.010	paved	2.00	22.9												

Table 13 - SUMMARY OF TIME OF CONCENTRATION AND REACH ROUTING CALCULATIONS
Salado Creek Watershed Drainage Master Plan
Ultimate Development Land Use

Based on procedures described in "Urban Hydrology for Small Watersheds", TR-55, USDA Soil Conservation Service, June 1986.

SUB-WATER-SHED ID	SHEET FLOW					SHALLOW CONCENTRATED FLOW					CHANNEL/PIPE FLOW					TIME OF CONC.	SCS LAG TIME	REACH ROUTING TIME				
	Length Feet	Mannings "n" -	Slope Ft/Ft	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Channel Type -	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Travel Time Minutes	Minutes	Hours	Length Feet	Slope Ft/Ft	Mannings "n" -	Vel. Ft/Sec	Routing Time Hours
SC26	250	0.160	0.008	0.42	9.9	550	0.027	unpaved	1.70	5.4	10700	0.009	0.075	5.6	31.9	72	0.719	19500	0.0014	0.065	2.2	2.471
						3850	0.018	paved	2.60	24.7												
SC27	250	0.160	0.003	0.28	14.9	4800	0.003	paved	1.00	80.0	950	0.011	0.075	6.2	2.6	212	2.116	-	-	-	-	-
						2550	0.013	paved	2.30	18.5	550	0.014	0.075	7.0	1.3							
						1100	0.005	unpaved	0.70	26.2	13200	0.003	0.075	3.2	68.2							
SC28	350	0.160	0.003	0.28	20.8	3000	0.006	paved	1.55	32.3	3300	0.007	0.075	4.9	11.2	124	1.243	17600	0.0013	0.065	2.2	2.208
						1500	0.009	unpaved	0.95	26.3	400	0.025	0.075	9.3	0.7							
						1150	0.020	paved	2.80	6.8	9700	0.011	0.075	6.2	26.2							
SC29	300	0.200	0.003	0.28	17.9	4300	0.001	paved	1.00	71.7	2900	0.027	0.080	9.1	5.3	220	2.202	-	-	-	-	-
						1800	0.006	unpaved	0.80	37.5	9200	0.001	0.080	1.7	87.8							
SC30	200	0.200	0.005	0.34	9.8	1400	0.009	paved	1.90	12.3	11900	0.007	0.080	4.6	42.9	114	1.137	22900	0.0018	0.07	2.3	2.802
						3800	0.016	unpaved	1.30	48.7												
SC31	300	0.200	0.007	0.21	23.8	1300	0.012	unpaved	1.10	19.7	9000	0.013	0.080	6.3	23.8	163	1.632	18000	0.002	0.075	2.4	2.100
											14200	0.002	0.080	2.5	95.8							
SC32	300	0.200	0.004	0.16	31.3	4400	0.014	unpaved	1.20	61.1	15400	0.003	0.085	2.8	90.7	183	1.830					
PANTHER SPRINGS CREEK																						
PS1	200	0.110	0.117	1.60	2.1	1000	0.057	unpaved	3.60	4.6	5400	0.024	0.050	15.3	5.9	43	0.435	5200	0.005	0.045	6.1	0.238
											15200	0.008	0.050	8.2	30.9							
PS2	300	0.110	0.050	1.10	4.5	1400	0.100	unpaved	4.90	4.8	13400	0.011	0.050	9.6	23.2	33	0.325	6900	0.008	0.045	8.2	0.234
PS3	300	0.110	0.033	0.85	5.9	2400	0.024	unpaved	2.40	16.7	6500	0.007	0.050	7.7	14.1	37	0.367	-	-	-	-	-
PS4	200	0.110	0.133	1.75	1.9	1000	0.078	unpaved	4.30	3.9	9300	0.016	0.050	11.6	13.4	19	0.191	11400	0.01	0.045	9.1	0.347
PS5	300	0.110	0.050	1.10	4.5	1200	0.087	unpaved	4.50	4.4	8600	0.020	0.050	13.9	10.3	19	0.193	8100	0.003	0.045	5.0	0.450
PS6	300	0.110	0.033	0.85	5.9	3000	0.060	unpaved	3.70	13.5	5400	0.006	0.050	7.1	12.7	32	0.321	-	-	-	-	-
PS7	150	0.110	0.007	0.40	6.3	2000	0.052	paved	4.50	7.4	13150	0.012	0.050	10.1	21.8	35	0.355	14200	0.0029	0.05	4.9	0.810
PS8	300	0.110	0.017	0.62	8.1	1600	0.029	unpaved	2.50	10.7	8200	0.005	0.050	6.5	21.1	46	0.460	7300	0.0039	0.05	3.9	0.525
						1200	0.026	paved	3.20	6.3												
PS9	300	0.110	0.043	0.98	5.1	1600	0.051	unpaved	3.40	7.8	4000	0.020	0.050	13.9	4.8	38	0.384	4200	0.006	0.045	6.7	0.175
											400	0.010	0.050	8.9	0.7			5500	0.004	0.05	5.4	0.282
											11200	0.011	0.050	9.4	19.9							
PS10	300	0.080	0.007	0.60	8.3	3200	0.013	unpaved	1.80	29.6	9200	0.013	0.050	10.2	15.0	53	0.530	4000	0.0013	0.06	2.1	0.540
LORENCE CREEK																						
LC1	300	0.110	0.010	0.47	10.6	1000	0.035	unpaved	2.80	6.0	12200	0.014	0.060	8.8	23.0	70	0.704	15100	0.004	0.055	4.7	0.887
											1600	0.006	0.060	5.8	4.6							
											9800	0.007	0.060	6.2	26.2							
LC2	300	0.080	0.027	1.20	4.2	2100	0.027	unpaved	2.50	14.0	14600	0.004	0.060	4.7	51.6	81	0.812					
						2200	0.027	paved	3.20	11.5												
MUD CREEK																						
MC1	300	0.110	0.067	1.20	4.2	1400	0.036	unpaved	2.90	8.0	11000	0.011	0.050	9.6	19.0	31	0.313	-	-	-	-	-
MC2	200	0.110	0.125	1.70	2.0	1800	0.044	unpaved	3.10	9.7	3200	0.017	0.050	12.0	4.5	23	0.229	9000	0.008	0.045	8.4	0.297
											4600	0.015	0.050	11.2	6.8							
MC3	200	0.110	0.100	1.50	2.2	1500	0.067	unpaved	3.90	6.4	2800	0.014	0.050	10.9	4.3	28	0.284	-	-	-	-	-

Table 13 - SUMMARY OF TIME OF CONCENTRATION AND REACH ROUTING CALCULATIONS
Salado Creek Watershed Drainage Master Plan
Ultimate Development Land Use

Based on procedures described in "Urban Hydrology for Small Watersheds", TR-55, USDA Soil Conservation Service, June 1986.

SUB-WATER-SHED ID	SHEET FLOW					SHALLOW CONCENTRATED FLOW					CHANNEL/PIPE FLOW					TIME OF CONC.	SCS LAG TIME	REACH ROUTING TIME				
	Length Feet	Mannings "n" -	Slope Ft/Ft	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Channel Type -	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Travel Time Minutes	Minutes	Hours	Length Feet	Slope Ft/Ft	Mannings "n" -	Vel. Ft/Sec	Routing Time Hours
											6600	0.006	0.050	7.1	15.5							
MC4	200	0.110	0.075	1.30	2.6	2200	0.050	unpaved	3.40	10.8	7700	0.014	0.050	10.9	11.8	25	0.252	-	-	-	-	-
MC5	150	0.110	0.067	1.25	2.0	900	0.044	unpaved	3.10	4.8	10500	0.016	0.050	11.6	15.1	22	0.219	3500	0.006	0.045	7.3	0.134
MC6	250	0.110	0.020	0.67	6.2	1600	0.053	unpaved	3.50	7.6	5800	0.016	0.050	11.6	8.3	22	0.222	5800	0.006	0.045	7.3	0.221
MC7	250	0.110	0.016	0.60	6.9	1700	0.040	unpaved	3.00	9.4	9200	0.014	0.050	10.9	14.1	31	0.305	11500	0.007	0.045	7.9	0.406
MC8	200	0.110	0.050	1.10	3.0	1500	0.080	unpaved	4.40	5.7	11800	0.013	0.050	10.5	18.8	28	0.275	12700	0.0054	0.05	6.2	0.567
MC9	200	0.110	0.050	1.10	3.0	3000	0.040	unpaved	3.00	16.7	16100	0.008	0.050	8.0	33.5	53	0.532	12500	0.0056	0.055	5.8	0.603
MC10	250	0.110	0.016	0.60	6.9	350	0.017	unpaved	2.00	2.9	4800	0.006	0.060	5.8	13.8	32	0.321	9600	0.0035	0.055	5.1	0.523
						1300	0.050	paved	4.50	4.8												
						750	0.053	unpaved	3.50	3.6												
MC11	200	0.035	0.025	1.60	2.1	700	0.043	unpaved	3.10	3.8	12600	0.005	0.060	5.3	39.8	49	0.493	-	-	-	-	-
						900	0.044	paved	4.10	3.7												
MC12	300	0.035	0.067	2.55	2.0	850	0.029	paved	3.50	4.0	14400	0.009	0.060	7.1	33.9	45	0.446	6800	0.0037	0.06	4.5	0.418
						650	0.023	unpaved	2.30	4.7												
MC13	250	0.035	0.008	0.90	4.6	2800	0.023	paved	3.10	15.1	8600	0.005	0.060	5.3	27.2	51	0.510	-	-	-	-	-
						600	0.025	unpaved	2.40	4.2												
ELM CREEK																						
EC1	300	0.110	0.020	0.67	7.5	1900	0.052	unpaved	3.60	8.8	20000	0.008	0.050	8.0	41.6	58	0.579	-	-	-	-	-
EC2	200	0.110	0.067	1.20	2.8	2000	0.045	unpaved	3.40	9.8	23100	0.009	0.050	8.3	46.2	59	0.588	8100	0.0045	0.045	4.3	0.524
EC3	300	0.110	0.007	0.40	12.5	1100	0.027	unpaved	2.60	7.1	9800	0.013	0.050	10.2	16.0	36	0.356	-	-	-	-	-
ELM WATERHOLE CREEK																						
EW1	200	0.110	0.150	1.80	1.9	800	0.088	unpaved	4.50	3.0	8200	0.017	0.050	12.0	11.4	91	0.912	-	-	-	-	-
											31200	0.006	0.050	6.9	75.0							
EW2	300	0.110	0.007	0.40	12.5	1800	0.047	unpaved	3.30	9.1	5200	0.018	0.050	12.3	7.0	52	0.515	7100	0.008	0.045	8.2	0.241
											12600	0.010	0.050	9.2	22.9							
EW3	300	0.110	0.020	0.67	7.5	2400	0.023	unpaved	2.30	17.4	5200	0.007	0.050	7.5	11.6	36	0.364	2200	0.014	0.045	10.8	0.057
EW4	200	0.110	0.060	1.20	2.8	400	0.050	unpaved	3.40	2.0	9000	0.013	0.050	10.5	14.3	19	0.191	8000	0.003	0.05	4.0	0.560
EW5	300	0.110	0.007	0.40	12.5	1700	0.042	unpaved	3.10	9.1	11600	0.015	0.050	11.2	17.2	39	0.388	12400	0.003	0.05	4.0	0.869
EW6	250	0.110	0.008	0.42	9.9	1400	0.036	unpaved	2.80	8.3	17500	0.007	0.050	7.5	39.0	57	0.572	-	-		-	-
STAHL ROAD																						
SR1	300	0.080	0.017	0.90	5.6	2500	0.027	paved	3.30	12.6	9700	0.010	0.060	7.5	21.7	40	0.399	10100	0.006	0.045	6.7	0.421
SR2	300	0.080	0.017	0.90	5.6	1200	0.026	unpaved	2.50	8.0	10000	0.006	0.060	5.8	28.9	46	0.462	12200	0.006	0.045	6.7	0.509
						500	0.012	paved	2.20	3.8												
SR3	300	0.080	0.017	0.90	5.6	2500	0.025	unpaved	2.40	17.4	10500	0.006	0.065	5.3	32.8	56	0.557	-	-	-	-	-
BIETEL CREEK																						
BC1	300	0.110	0.023	0.71	7.0	2000	0.032	unpaved	2.80	11.9	13100	0.009	0.060	7.1	30.9	50	0.498	-	-	-	-	-
BC2	300	0.110	0.057	1.20	4.2	1400	0.041	unpaved	3.00	7.8	11200	0.010	0.060	7.5	25.0	37	0.370	16400	0.0043	0.045	6.0	0.760
BC3	300	0.080	0.013	0.80	6.3	1400	0.029	unpaved	2.60	9.0	17900	0.008	0.060	6.7	44.7	60	0.599	-	-	-	-	-
BC4	300	0.080	0.010	0.70	7.1	2500	0.045	unpaved	3.20	13.0	12100	0.011	0.070	6.7	30.1	60	0.600	8600	0.0035	0.045	6.8	0.351
											2600	0.007	0.070	5.3	8.2							
											900	0.022	0.070	9.5	1.6							

Revised 7/9/96

Table 13 - SUMMARY OF TIME OF CONCENTRATION AND REACH ROUTING CALCULATIONS
Salado Creek Watershed Drainage Master Plan
Ultimate Development Land Use

Based on procedures described in "Urban Hydrology for Small Watersheds", TR-55, USDA Soil Conservation Service, June 1986.

SUB-WATER-SHED ID	SHEET FLOW					SHALLOW CONCENTRATED FLOW					CHANNEL/PIPE FLOW					TIME OF CONC.	SCS LAG TIME	REACH ROUTING TIME				
	Length Feet	Mannings "n" -	Slope Ft/Ft	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Channel Type -	Velocity Ft/Sec	Travel Time Minutes	Length Feet	Slope Ft/Ft	Mannings "n" -	Velocity Ft/Sec	Travel Time Minutes	Minutes	Hours	Length Feet	Slope Ft/Ft	Mannings "n" -	Vel. Ft/Sec	Routing Time Hours
BC5	300	0.080	0.010	0.70	7.1	500	0.018	unpaved	2.10	4.0	11200	0.009	0.070	6.0	31.2	59	0.590	-	-	-	-	-
						2200	0.013	paved	2.20	16.7												
WALZEM CREEK																						
WC1	300	0.110	0.073	1.30	3.8	4800	0.040	unpaved	3.00	26.7	19500	0.007	0.075	4.9	66.0	96	0.965	-	-	-	-	-
ROSILLO CREEK																						
RC1	300	0.110	0.050	1.10	4.5	1000	0.021	paved	2.80	6.0	9000	0.008	0.065	6.1	24.7	83	0.830	20900	0.003	0.05	4.1	1.419
						1150	0.009	paved	1.90	10.1	9400	0.005	0.065	4.8	32.6							
						1100	0.032	paved	3.60	5.1												
RC2	300	0.110	0.004	0.30	16.7	2500	0.004	paved	1.10	37.9	6900	0.003	0.065	3.7	30.9	206	2.056	-	-	-	-	-
											5400	0.006	0.065	5.3	17.1							
											18800	0.002	0.065	3.0	103.1							
RC3	400	0.110	0.003	0.24	27.8	1800	0.015	unpaved	1.90	15.8	9800	0.003	0.065	3.7	43.9	114	1.143	9700	0.003	0.05	4.1	0.659
						1000	0.004	unpaved	1.10	15.2												
						400	0.005	paved	1.40	4.8												
						500	0.006	unpaved	1.20	6.9												
RC4	300	0.110	0.003	0.17	29.4	2100	0.012	unpaved	1.60	21.9	5800	0.008	0.070	5.6	17.1	132	1.322	-	-	-	-	-
											5400	0.001	0.070	1.4	63.8							
RC5	300	0.110	0.003	0.17	29.4	650	0.005	unpaved	1.10	9.8	15400	0.002	0.070	2.8	91.0	158	1.582	-	-	-	-	-
											8200	0.006	0.070	4.9	28.0							
RC6	300	0.110	0.012	0.51	9.8	1600	0.020	unpaved	2.20	12.1	7850	0.008	0.075	5.3	24.8	47	0.468	14900	0.002	0.055	3.0	1.363
RC7	300	0.110	0.003	0.25	20.0	1200	0.019	unpaved	2.10	9.5	14600	0.004	0.075	3.7	65.3	95	0.949	-	-	-	-	-
RC8	450	0.110	0.003	0.25	30.0	8000	0.003	paved	1.30	102.6	11600	0.008	0.080	4.9	39.1	176	1.765	25600	0.003	0.055	3.7	1.912
											400	0.025	0.080	8.7	0.8							
											1100	0.008	0.080	4.6	4.0							
RC9	400	0.160	0.003	0.18	37.0	1300	0.006	unpaved	1.20	18.1	25200	0.002	0.080	2.5	170.1	241	2.412	-	-	-	-	-
						1050	0.009	paved	1.90	9.2												
						1100	0.031	unpaved	2.70	6.8												

results compared very favorable to the recorded data. Both historical storms were simulated with the HEC-1 model providing verification of the hydrologic modeling.

The final step of the hydrologic analysis involved applying the theoretical storms to the watershed. Rainfall intensities for the City of San Antonio were analyzed and updated during the Preliminary Phase. These updated rainfall intensities were used with the understanding that the City of San Antonio will incorporate them into a future update of it's Unified Development Code, Chapter 35 of the City Code. Rainfall data for the 10, 25, 50, 100, and 500 year frequency storms was incorporated into the study using a storm duration period of twenty four (24) hours with a SCS twenty four (24) hour Type-II rainfall distribution.

Table 14 - "Comparison of Storm Water Flows"

	10 Year			50 Year			100 Year			500 year		
RIVER CROSSINGS	FEMA Model	HEC-1 Existing	HEC-1 Ultimate	FEMA Model	HEC-1 Existing	HEC-1 Ultimate	FEMA Model	HEC-1 Existing	HEC-1 Ultimate	FEMA Model	HEC-1 Existing	HEC-1 Ultimate
Loop 1604		15414	15379		23250	23243		26676	26867		34607	34591
West Ave.	12200	16570	16934	17300	25001	25336	19300	28664	28962	58000	37164	37483
U. S. 281	16700	17209	17622	24000	25735	26123	27000	29441	29813	81000	38040	38373
Wetmore Rd	28600	26873	29435	41600	39650	42132	46600	45227	47681	130000	58282	60652
Nacogdoches Rd.	28600	27673	30383	41600	40793	43476	46600	46528	49204	130000	59954	62599
N.E. Loop 410	30100	28189	31178	44300	41614	44602	49100	47504	50470	140000	61287	64191
Austin Hwy.	36900	32310	35875	54200	47646	51236	60500	54365	57946	150000	70095	73634
Rittiman Rd.	36900	31029	34274	54300	45675	48935	61000	52097	55337	160000	67125	70302
I. H. 35	36900	21900	24089	54300	32147	34408	61000	36656	38922	170000	47250	49651
Commerce St.	36900	20078	22123	54300	29415	31550	61000	33526	35674	170000	43251	45725
Rigsby Ave.	36900	18247	20134	54300	26672	28661	61000	30382	32394	170000	39296	41847
E. Southcross Blvd.	36900	14139	15567	54300	20512	21986	61000	23250	24723	170000	30063	31979
S.E. Military Dr.	36900	14139	15567	54300	20512	21986	61000	23250	24723	170000	30063	31979
S.E. Loop 410	36900	13292	14657	54300	19262	20673	61000	21822	23236	170000	28285	30198

Ultimate development projections, as provided by the City of San Antonio Planning Department, were used to compute storm water flows for ultimate development conditions. All subareas except for those within the Camp Bullis area were adjusted for ultimate development using these projections. The time of concentration was adjusted for subareas that contained shallow concentrated flow travel lengths greater than 2000 feet by converting twenty(20) percent of the length to channelized flow. In subarea SC17, the reach routing from SC16 was modified to model the channelization of the creek that is being considered along the north side of the San Antonio Airport. The results of the HEC-1 (hydrologic) Model for ultimate development conditions are compared with those of existing conditions in Table 14 - Comparison of Storm Water Flows. Included in the comparison are the storm water flows obtained from FEMA. The storm water flows are presented in cubic feet per second (cfs).

A final modification of the HEC-1 model was made which removed storage routing for the thirteen (13) SCS Floodwater Retarding Dams. The 100 year theoretical storm was then applied. The storm water flows obtained by this Model run are compared to the previous existing condition results shown in Table 14. The SCS Floodwater Retarding

Dams produce more than a fifty (50) percent reduction in stream flows in the areas south of Loop 1604. Most of the dams are located on the Recharge Zone of the Edwards Aquifer and provide substantial recharge to the Aquifer, however, this study does not quantify the recharge effects of those structures. Table 15 shows the comparison of the existing conditions model with and without the thirteen floodwater retarding dams at several locations along the creek.

Table 15 - "Comparison of 100 Year Frequency Storm Water Flows with and without the Floodwater Retarding Dams"

RIVER CROSSING	HEC-1 Model	Minus Dams
Loop 1604	26676	42147
Loop 1604 (Panther Sp.)	8411	30044
Bitters Rd. (Panther Sp.)	7752	30014
West Ave. (Panther Sp.)	513	30538
Loop 1604 (Mud Ck.)	7222	26333
Thousand Oaks (Mud Ck.)	12901	62283
Loop 1604 (Elm Ck.)	136	15207
Loop 1604 (Elm Waterhole)	284	28268
West Ave.	28664	71588
U. S. 281	29441	71437
Wetmore Rd	45229	116782
Nacogdoches Rd.	46528	117316
N.E. Loop 410	47504	113686
Perrin Beitel (Beitel Ck.)	22050	22050
Austin Hwy.	54365	111891
Rittiman Rd.	52097	105444
I. H. 35	36656	70397
Commerce St.	33526	62580
I. H. 10	33526	62580
E. Southcross Blvd.	23250	55204
S.E. Military Dr.	23250	55204
S.E. Loop 410	21822	48987

4

Design Phase

A. Hydraulic Modeling

I. HEC-2 Model

Aerial Mapping was prepared by United Aerial Mapping and furnished by the City of San Antonio. Stream cross sections were produced for the Salado Creek and tributaries based upon the aerial mapping. Cross section characteristics were defined with Manning's roughness coefficients that represent the vegetation and varied floodway conditions observed along Salado Creek. Sections were placed at approximate intervals of 500 feet with variations depending upon the influence of curvature of the creek and structures that cross the creek. Each section was located perpendicular to the flow and extended to the limits of the mapping. Adjustments were made in placement of the cross-sections when bridge structures, culverts and cutbacks were encountered. Table 7 indicates the sections located at bridge and culvert crossings. Modeling of culvert and bridge structures was based upon plans obtained from the City of San Antonio and the Texas Department of Transportation. When plans were not available, the structure was measured and detailed by field survey. Along Salado Creek and the tributaries exist several low water crossings. These crossings are individually addressed as mitigation projects in this report.

The original F.E.M.A. models use roughness coefficients ranging from 0.035 to 0.075. Investigation and analysis of the Salado Creek suggest that these coefficients are not adequate to define the existing conditions of the Salado Creek. Since very thick vegetation exists along lower Salado Creek, stream cross sections along the lower regions of Salado Creek have been defined with coefficients ranging from 0.030 to 0.11. The coefficients were adjusted downward in areas where less vegetation is present. In several areas clearing has been done to create parks, golf courses and other similar use sites. In these areas where brush has been removed and the area is being maintained, roughness coefficients were adjusted downward. Higher roughness coefficients were used in the very dense to extremely dense vegetated areas along the Creek.

Five water surface profiles are produced by the HEC-2 model representing the 500, 100, 50, 25, and 10 year frequency storms. Storm water flows derived from the HEC-1 model are entered at sections representative of HEC-1 node locations. The HEC-1 nodes and HEC-2 sections with approximate locations are presented in Table 16.

Table 16 - "HEC-1 Node Locations"

WATERSHED	HEC-1 NODE	DESCRIPTION OF LOCATION	HEC-2 SECTION
Salado Creek	07S	Loop 1604	192321
	08S	Approximately 2000' Downstream of Huebner Road	178997
	08S2	Above the Confluence with Panther Springs Creek	160212
	09S	Below the Confluence with Panther Springs Creek Downstream of West Avenue	158339
	10S	U.S. Highway 281	154764
	10S2	Above the Confluence with Lorence Creek and Mud Creek	142679
	10S3	Above the Confluence with Mud Creek and Below the Confluence with Lorence Creek	140634
	11S	Below the Confluence with Mud Creek Upstream of Wetmore Road	138339
	11S2	Wetmore Road	132303
	12S	Nacogdoches Road	129765
	13S	N.E. Loop 410	125025
	14S	Approximately 2100' Downstream of N.E. Loop 410	121219
	15S	Above the Confluence with Beitel Creek	119314
	16S	Below the Confluence with Beitel Creek at Austin Highway	116016
	17S	Approximately 1000' Upstream of Rittiman Road Above the Confluence with Walzem Creek	111094
	18S	Approximately 640' Downstream of Rittiman Road	109387
	19S	Approximately 1000' Upstream of Binz-Engleman Road	93170
	20S	Houston Street	73661
	21S	Rigsby Avenue	56041
	22S	S.E. Military Drive	33188
	23S	S.E. Loop 410	19500
	24S	Confluence with Rosillo Creek	15140
Panther Springs	SCS6	Approximately 1400' Upstream of Bitters Road	12704
	07P	Approximately 2000' Downstream of Bitters Road	9891
	08P	Above SCS Dam No. 7	4347
	SCS7	Mouth of Panther Springs Creek	433
Mud Creek	SCS10	Above SCS Dam No. 10	16365
	08M	Above the Confluence of Mud, Elm, and Elm Waterhole Creeks	15865
	09M	Approximately 1700' Upstream of Buckhorn Road inside McAllister Park	6767
	10M	Mouth of Mud Creek	620
Elm Creek	01E	Approximately 440' Upstream of Jones Maltsberger	5541
	02E	Mouth of Elm Creek	184
Elm Waterhole	04W	Approximately 900' Downstream of Classen Road	8918
Creek	05W	Mouth of Elm Waterhole Creek	32
Beitel Creek	00B	Approximately 920' Downstream of Nacogdoches Road	24953
	01B	Old O'Connor Road	19112
	02B	Approximately 5100' Upstream of N.E. Loop 410	10435
	03B	Approximately 3200' Upstream of N.E. Loop 410	8557
	04B	Mouth of Beitel Creek	210

II. Depth-Discharge Rating Curves

Data collected on the storms that occurred on April 4-5, 1991 and May 5-6, 1993 was analyzed using the HEC-2 hydraulic model and the results were compared to the depth-discharge data obtained from the U.S.G.S. Comparison of this data, showed significant variation in the Depth-Discharge relationship. Using the Hydraulic Model, new Depth

Discharge Rating Curves were computed for the U.S.G.S. Upper Station and Lower Station. The Depth-Discharge Rating Curves that resulted are presented in Figures 12 and 13 along with a display of the U.S.G.S. rating curves. Using the new Depth-Discharge Rating Curves, the two historical storms were plotted and rated. The frequency of the April 1991 storm was determined to be approximately a 2 year storm event for the entire watershed with higher frequency being experienced in localized areas. The May 1993 storm was determined to be the equivalent of a twelve year storm for the entire watershed.

B. Storage Analysis

Observed conditions and recorded data from the United States Geological Survey, Water Resources Division in San Antonio, Texas indicated that linear storage occurs along lower Salado Creek. During the preliminary phase it was determined that additional analysis of the lower regions of Salado Creek was necessary to verify this condition. Thick vegetation located in this region of the creek increase the uniform losses assumed as a constant in the HEC-1 model. Stream flow gages maintained at the upper and lower limits of the lower Salado Creek indicate storage losses in excess of what is considered normal. Careful review of recorded data, throughout the 25 year history of the gages, substantiated considerable storage losses occur. While the HEC-1 model of the upper watershed generated data that was very comparable with gage records, data pertaining to the lower watershed produced results that substantially exceeded gage records.

A separate analysis of the lower Salado Creek was performed using the Hydrologic and Hydraulic Models to compute channel storage. A storage analysis was performed using storage-outflow data from the HEC-2 model as input data for the HEC-1 model. The first step in the process was adding the storage-outflow option to the HEC-2 model. Using the storage outflow option, sections corresponding to the reach routings were entered in the HEC-2 model. Storage and discharge values were generated by use of the HEC-2 model incorporating the Modified Puls method. Basic storage-outflow data was produced by the HEC-2 model. The data produced corresponds to storage records and discharge records. Records are generated for each profile of the HEC-2 model. The HEC-1 model generated new storm water flows for each of the reach routings along the lower Salado Creek. The storm water flows were then updated in the HEC-2 model. The updated HEC-2 model generated a new set of storage and discharge records. Following this process produces a set of storage and discharge records in the HEC-1 model, which is used to generate new storm water flows. This iteration process continued with the HEC-1 and HEC-2 models until the storm water flows generated by the HEC-1 model and the storage-discharge values generated by the HEC-2 model were repeated.

Computed storm water flows at the lower end of Salado Creek were consistently higher than the recorded data at Salado Creek (Lower Station). A split flow analysis was performed on Salado Creek at section 225. The split that occurs north of Roland Street results in approximately sixty five (65) percent of discharge flowing down the west fork stream and the other thirty five (35) percent flowing down the east fork. Travel distance

Figure 12 - Depth-Discharge Rating Curve

Upper Salado Creek Gaging Station

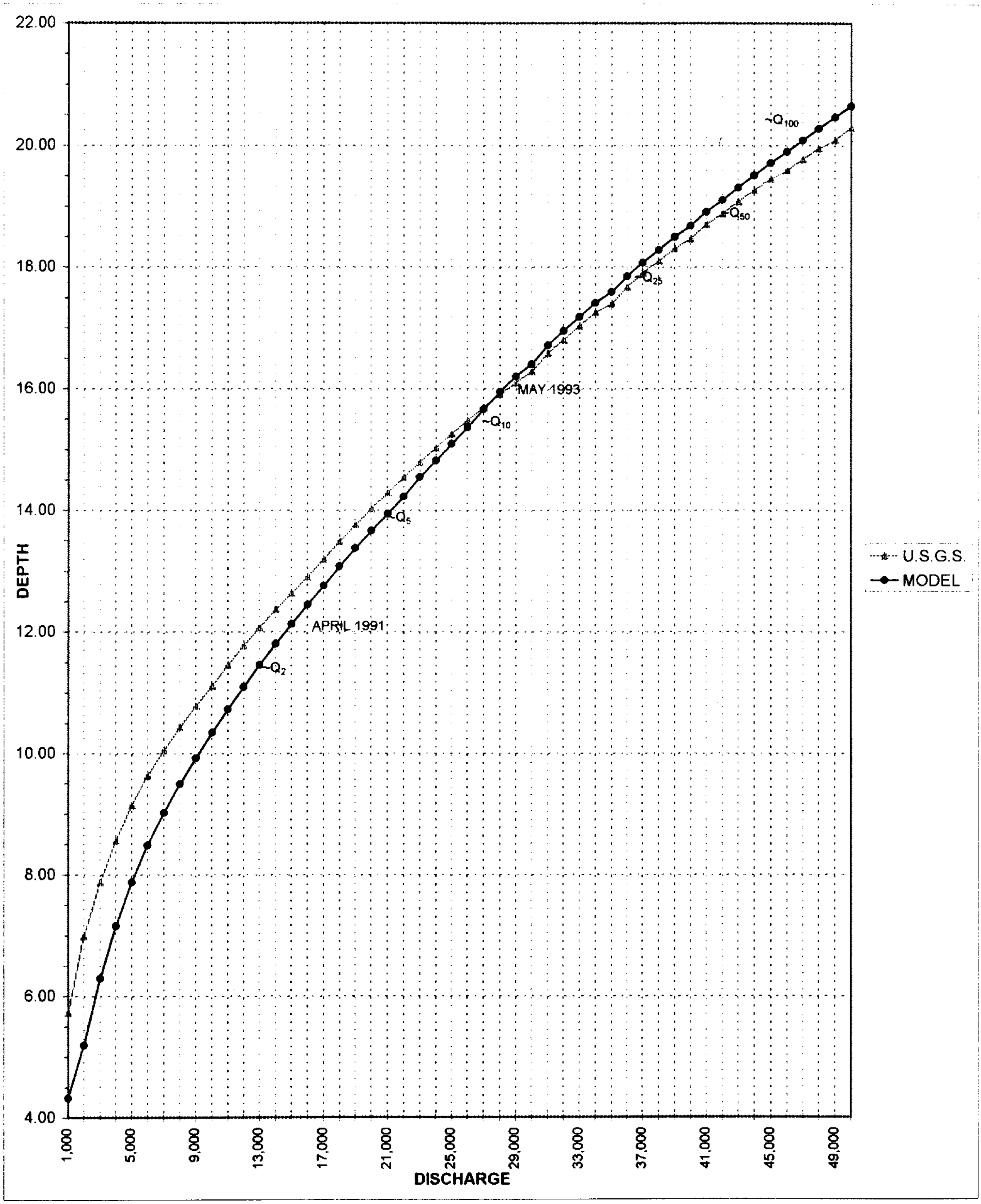
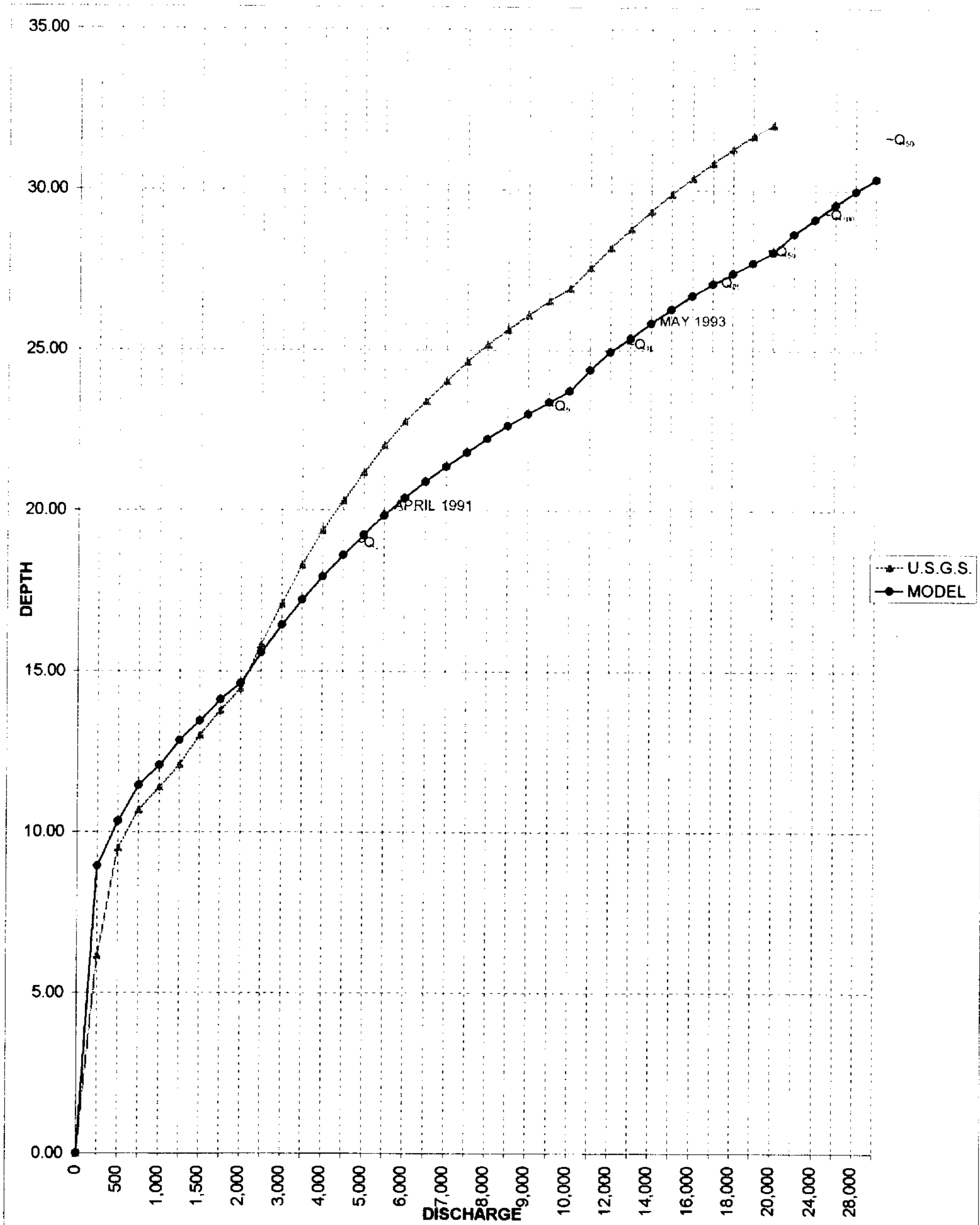


Figure 13 - Depth-Discharge Rating Curve
Lower Salado Creek Gaging Station



down the west side is approximately 6,980 feet requiring a travel time of approximately twenty three (23) minutes. Distance of travel on the east fork is approximately 18,900 feet requiring a travel time of approximately seventy (70) minutes. Distance and travel time are approximately three times larger for the east fork. The greater travel time along the east fork causes a delay in the peak storm water flow. Approximately sixty five (65) percent of the storm water flow in the east fork is returned to the storm water flow in the west fork at the confluence. The additional loss of storm water flow resulting from the split flow produced comparable storm water flows and depths to gage records at Salado Creek (Lower Station).

After completion of the storage analysis, the HEC-2 model for the April 4-5, 1991 storm (1991SAL.DAT) generated water surface elevations comparable to gage records. The HEC-2 model produced depths that compared very closely with stream gage recordings of the two U. S. Geological Survey stream gages and the City of San Antonio stream gage at the Interstate Highway 10 crossing of Salado Creek. The difference between the output of the model and the actual gage recording was less than half a foot at each of the three gage stations. At the upper gaging station a peak of 12.04 feet was recorded and at the lower gaging station a peak of 20.98 feet was recorded. The USGS Expanded Rating Tables show a datum difference of 2.5 feet at the Upper Gaging Station and 6.35 feet at the Lower Gaging Station. Thus the measured depth of flow at the Upper Station is 12.04 - 2.5 or 9.54 feet and the measured depth at the Lower Station is 20.98 - 6.35 or 14.63 feet. The City of San Antonio's stream flow gage station at IH 10, identified as Sensor #4764 was recording during the April 1991 storm. The recorded peak gage height during the storm was 603.36.

The HEC-2 model simulation produces a depth of 9.35 feet at the upper gaging station at section 403 and a depth of 14.79 feet at the lower gaging station at section 178. The Model produces a water surface elevation of 603.36 at the IH 10 gage that is located at section 260. Direct comparison of the depths at the stream flow gaging stations to the depths generated by the model provided verification of the model. Comparisons of the depths are as follows.

GAGE	MEASURED DEPTH	SIMULATED DEPTH	DIFFERENCE
USGS at Loop 410	9.54	9.35	0.19
COSA at IH 10	16.46	16.46	0.00
USGS at Loop 13	14.63	14.79	0.16

C. Floodplain Delineation

The final step in the hydraulic analysis involved applying theoretical storm water flows of the 500, 100, 50, 25 and 10 year frequency rainfall events to the hydraulic model.

Application of these storm water flows generated water surface elevations for each of these storms at each cross section. The resulting water surface elevations were plotted at each cross section. Interpolation of elevations between the sections establishes the limits for the floodplains. However, floodplain limits interpolated through or adjacent to existing structures have been adjusted. The determination of whether or not these structures are flooded was verified with foundation elevations.

Maps generated from the hydraulic modeling represent the 100 year floodplain under existing conditions. The HEC-2 model water surface elevations were compared with the water surface elevations provided on the Flood Insurance Rate Maps (FIRM) prepared by F.E.M.A. Previous comparisons of the discharges with the F.E.M.A. model had shown variations from slight in the upper reaches to great in the lower reaches and water surface elevation comparisons show varied differences. In areas along the lower reaches where the new storm water flows are much smaller, the new water surface elevations compare in a range from lower to higher than the F.E.M.A. water surface elevations.

The HEC-2 modeling based upon ultimate development is approximately one half foot to one foot higher than existing conditions water surface elevations. Increases which would normally be expected as a result of ultimate development, are largely being mitigated by the presence of the existing floodwater retarding dams. Floodplain Maps were not produced for ultimate development conditions, however, comparisons of existing conditions and ultimate development water surface elevations for a 100 year frequency storm are provided in Table 17.

Table 17 - "Comparison of Water Surface Elevations"
Existing Conditions vs. Ultimate Development

LOCATION	SECTION	EXISTING ELEVATION	ULTIMATE ELEVATION
S.E. Loop 410	20729	537.91	538.18
S.E. Military Dr.	33294	555.23	555.75
E. Southcross	43308	565.64	566.07
Rigsby	54608	591.56	591.92
Rice	61680	603.58	603.83
Martin Luther King	63615	605.64	605.99
I.H. 10	69937	612.23	612.84
Commerce St.	72092	615.90	616.43
Gembler Rd.	81444	624.78	625.15
I.H. 35	87445	635.55	636.20
Binz-Engleman Rd.	92176	646.73	647.78
W.W. White Rd.	96336	648.60	649.56
Rittiman Rd.	110103	672.42	672.83
Eisenhauer Rd.	114620	681.40	681.77
Austin Hwy.	116126	686.77	687.72
N.E. Loop 410	125541	705.81	706.35
Nacogdoches Rd.	132365	721.49	721.92
Wetmore Rd.	138194	729.04	730.16
Jones Maltsberger Rd.	151311	768.99	769.07
U.S. Hwy. 281	157442	788.61	788.74
West Ave.	162051	807.42	807.49
Vista Del Norte	168291	830.76	830.86
Blanco Rd.	170967	842.66	842.74
Huebner Rd.	181924	891.55	891.57
Loop 1604	192471	951.47	951.48
West Ave.	1272	797.11	797.19
Thousand Oaks	11201	777.06	777.55
Redland Rd.	5628	821.09	820.52
Classen Rd.	9863	823.56	826.60
Loop 1604	11576	826.91	830.09
Redland Rd.	3320	817.59	818.59
Loop 1604	7316	832.33	833.74
Perrin Beitel	2870	706.47	706.77
Vicar Rd.	3416	707.10	707.46
N.E. Loop 410	5321	711.95	711.94
Weidner Rd.	21888	781.17	781.61
O'Connor Rd.	23919	783.16	783.42

D. Mitigation Projects

Flooding of buildings became evident in several locations as the floodplains were being mapped. The number of structures identified as being located in the floodplain is 335. The number of residential structures is 179 and commercial or industrial structures number 65. The remaining 91 structures are sheds, pavilions, barns, stables, etc. The greatest area of flooding occurs south of Martin Luther King Drive in East Park Subdivision where stream sections are broad and flat. Ninety-Nine residences, two churches, and four apartment buildings are located in the floodplain. Other locations where multiple structures are flooded are along Holbrook Road, North Loop Road west of U.S. Hwy. 281, Nacogdoches Road, Garden Court East subdivision along Beitel Creek, Austin Hwy. Industrial Subdivision and Fairfield Village North Subdivision. Singular structures are flooded along the lower regions of Salado Creek and Beitel Creek. A list of the structures identified in the floodplain is provided in Table 19.

Projects considered to mitigate flooding include construction of detention dams, performing localized channelization, clearing stream vegetation, construction of levees, re-routing of roadways, and property acquisition. The first project evaluated was dam site No. 15r, the final proposed Natural Resources Conservation Service Floodwater Retarding Dam. This structure will be located in McAllister Park north of Starcrest Drive and it includes a temporary storage reservoir as originally planned in the McAllister Park Proposed Master Land Use Plan. The land was purchased by the City of San Antonio for flood control use and the Master Land Use Plan was completed in 1964. The Master Land Use Plan is included in Appendix F. According to Mr. Trent Street, Design Engineer with the Natural Resources Conservation Service, the floodwater retarding dam is scheduled for design in October 1996. Construction of the project will depend upon future funding allocations. Allocation of funds for the construction of dam 15r do not appear very likely through 1998. Land for the reservoir is currently being utilized by the City Parks and Recreation Department as a portion of McAllister Park. McAllister Park has become very popular with residents in the northern area of the City of San Antonio. Concerns raised by patrons of the park have created an issue concerning the design of the dam. If these concerns are abated, the dam must be designed so that it will not interfere with the continued utilization of park facilities. Temporary storage will occur in the reservoir when floodwater accumulates and portions of the park will become flooded for short periods depending upon the severity of the storm event. However, water will be quickly released until the reservoir is drained. The dam structure for this project will have a height of 44 feet and the reservoir storage capacity will be 3400 acre-feet. The National Resources Conservation Service has estimated the cost of construction at $6,000,000.

The second project developed for mitigation is located on upper Beitel Creek in the area of Lookout Road, Weidner Road and Old O'Connor Road. All three roadways and approximately 400 feet of Leonhardt Street are within the floodplain. New Wurzbach Parkway is also planned for construction through this area. This project would include rerouting Leonhardt Street and raising it above the floodplain to intersect with Wurzbach

Parkway. Portions of Weidner and Old O'Connor located within the floodplain are to be closed. Five thousand feet of Weidner Road and two thousand five hundred feet of Old O'Conner will be removed. Lookout Road will be rerouted to the east outside the floodplain to intersect with Old O'Connor Road. A railroad crossing at this location will be widened decreasing the embankment encroachment on the floodway.

Project three of the mitigation projects includes channelization of a section of Beitel Creek. Beitel Creek has been channelized from N.E. Loop 410 upstream to an area just south of Garden Court East Subdivision. Constructing an earthen channel from the existing channel, upstream for 3500 to 4000 feet will lower the creek and water surface elevations and narrow the sections. The channel would be adjacent to Garden Court East Subdivision.

Raising and rerouting Holbrook Road between Eisenhauer and Rittiman Roads is the fourth mitigation project. The project involves moving the roadway to the east, away from Salado Creek and raising its elevation. This project was evaluated individually and in conjunction with other projects. Alignment for the relocated roadway was established adjacent to existing buildings so that the structures are not affected. Raising the roadway to an elevation higher than the 25 year frequency flood will provide future mitigation of the 100 year frequency flood when Dam No. 15r is constructed.

Project five was evaluated individually and in conjunction with other projects. This project consists of a levee that is sized to contain water within the floodway. The levee would be constructed south along Salado Creek from the embankment of Martin Luther King Drive. The length of the levee will be approximately 4400 feet extending around East Park Subdivision along the west side of Salado Creek. The height of the levee will vary from four feet to seven and one half feet and the sides of the levee will be graded at a four to one slope with sodding for erosion control. The top width of the levee is thirty feet to provide for paths for either pedestrian, bicycle, or vehicular traffic. The top width can be varied according to intended use.

The sixth mitigation project evaluated, consists of brush clearing along lower Salado Creek. As described in Chapter 3, dense vegetation was observed along the banks and overbank areas along lower Salado Creek. The project limits are the bridge structures at S.E. Loop 410 and at N.E. Loop 410. The total length of the project is approximately 20 miles. This project does not include modification of creek sections. The project involves only the removal of grass, weeds, brush, small trees, and the small lower branches of trees up to a height of five or six feet. The project would leave significant trees that are larger than 3 inches in diameter in place. Existing dense vegetation along with the broad sections of Salado Creek currently provide significant linear storage. Clearing of the underbrush will have the detrimental effect of decreasing the linear storage and increasing flood elevations downstream by a substantial amount.

The seventh project developed and evaluated for mitigation is located on the lower end of Beitel Creek. The project involves channelization. Upstream of Vicar Road is an existing concrete channel. The conditions downstream of Vicar Drive are natural with the west bank of Beitel having been cleared of vegetation except for grasses. This project extends the concrete channel underneath Vicar Drive and transitions the channel into an earthen trapezoid section. An earthen trapezoidal channel section would be constructed downstream of Vicar Drive and past Perrin Beitel. Approximate length of the channelization would be 2,600 linear feet. Vicar Drive will be reconstructed with a new bridge crossing Beitel Creek.

Rerouting Holbrook Road at Austin Highway is the eighth mitigation project. Included in the project is closure and removal of the access roadway connecting Ira Lee Road and Holbrook Road under Austin Highway. Holbrook Road would be rerouted to a higher elevation for intersection with Austin Highway.

Project nine was evaluated as an alternative to the levee adjacent to East Park Subdivision (Project five). This project involves clearing Salado Creek and channelizing for a length of 5000 feet. Channelization would be performed south of Martin Luther King Drive and would consist of the construction of an earthen trapezoidal channel.

A channelization project at the San Antonio International Airport(S.A.I.A.) was evaluated as project ten. This project includes channelization of the Salado Creek within the limits of the Airport property. The project reroutes the natural channel through this area, reducing the overall length by approximately 2,300 linear feet to follow the proposed Wurzbach Parkway. Modeling the project involved creating a trapezoidal channel within the HEC-2 model. The stream sections that would be affected by this rerouting were replaced with trapezoidal channel sections. Routing of the Salado Creek was adjusted to follow the alignment of the Wurzbach Parkway with a reduction in overall length of approximately 2300 feet. The affects on water surface elevations were evaluated under ultimate development with the mitigation projects in place. A new earthen channel along Wurzbach Parkway will lower water surface elevations and eliminate the flooding of ten buildings at the upper end of the project.

The eleventh project analyzed is a detention pond in the Longhorn Quarry. This project was evaluated as an alternative to project three. The detention pond would require a diversion of flow through an adjacent box culvert under Wurzbach Parkway into the Longhorn Quarry west of Beitel Creek. Using a split flow diversion on Beitel Creek at section 3050, reduced flows were computed for complete mitigation of flooding downstream of this location. The size of the detention pond required for the diverted flow is approximately 1300 acre-feet. After it was determined that the Quarry had the capacity for only 400 acre-feet of storage, the analysis focused on smaller diversions. The diversion of flows for a 400 acre-foot detention pond does reduce flooding. A detention pond at Longhorn Quarry does not provide the benefits necessary to justify the cost. The limitation of storage capacity eliminated the project from further consideration.

I. Project Costs

The proposed floodwater retarding dam 15r is a proposed federally funded project, however, a cost estimate is provided to compare with other proposed mitigation projects. Funding for the project has not been allocated and the Natural Resources Conservation Service cannot predict when the allocation may occur. It is suggested that either lobbying for project funding or the partial allocation of funds by local agencies could provide the necessary impetuous to secure speedy federal funding.

Estimated costs for the other mitigation projects and roadway structures are presented in Table 18. Included in Table 18 are proposed acquisitions. Properties that are not benefiting from the mitigation projects have been identified for acquisition. Estimated values of the properties are based upon Bexar District appraisals. The mitigation projects developed provide relief for the majority of flooding problems identified, but do not solve all flooding problems. Thus, acquisition is the most cost effective alternative for removing some properties with buildings from the hazard of flooding. Benefits of the recommended mitigation are addressed in Chapter 5.

TABLE 18
PRELIMINARY COST ESTIMATE - MITIGATION PROJECTS
SALADO CREEK

DESIGN COMPONENTS	UNIT	UNIT COST	NUMBER OF UNITS	CONSTRUCTION COSTS PER COMPONENT	TOTAL COST PER PROJECT
1. Floodwater Retarding Dam No. 15r	LS				$6,000,000 *
2. Reroute Lookout and Leonhardt Rds.					
Right of Way	AC	$10,000	8	$80,000	
Misc. (Utilities, Fences, etc.)				$500,000	
Total Construction Costs				*$500,000*	
Mobilization (11%)				$55,000	
Preparation of ROW (4%)				$20,000	
Subtotal				$575,000	
Contingencies (10%)				$57,500	
Engineering (11%)				$63,250	
Administration (7%)				$40,250	
Stormwater Pollution Control (5%)				$28,750	
TOTAL				$844,750	$844,750
3. Channelization Beitel Creek					
Channelization (Section 9933 to 13285)					
Excavation/Disposal of Material	CY	$6	103600	$621,600	
Right of Way	AC	$10,000	38	$380,000	
Total Construction Costs				*$621,600*	
Mobilization (11%)				$68,376	
Preparation of ROW (4%)				$24,864	
Subtotal				$714,840	
Contingencies (10%)				$71,484	
Engineering (11%)				$78,632	
Administration (7%)				$50,039	
Stormwater Pollution Control (5%)				$35,742	
TOTAL				$1,330,737	$1,330,737
4. Reroute Holbrook Rd.					
Right of Way	AC	$10,000	12	$120,000	
Misc. (Utilities, Fences, etc.)				$550,000	
Total Construction Costs				*$550,000*	
Mobilization (11%)				$60,500	
Preparation of ROW (4%)				$22,000	
Subtotal				$632,500	
Contingencies (10%)				$63,250	
Engineering (11%)				$69,575	
Administration (7%)				$44,275	
Stormwater Pollution Control (5%)				$31,625	
TOTAL				$961,225	$961,225
5. Levee					
Embankment	CY	$9	5300	$47,700	
Right of Way	AC	$10,000	8	$80,000	
Misc. (Utilities, Fences, etc.)				$200,000	
Total Construction Costs				*$247,700*	
Mobilization (11%)				$27,247	
Preparation of ROW (4%)				$9,908	
Subtotal				$284,855	
Contingencies (10%)				$28,486	
Engineering (11%)				$31,334	
Administration (7%)				$19,940	
Stormwater Pollution Control (5%)				$14,243	
TOTAL				$458,857	$458,857
6. Channel Clearing (Station 20729 to 125239)	AC	$2,500	1940	$4,850,000	
Total Construction Costs				*$4,850,000*	
Mobilization (11%)				$533,500	
Preparation of ROW (4%)				$194,000	
Subtotal				$5,577,500	
Contingencies (10%)				$557,750	
Engineering (11%)				$613,525	
Administration (7%)				$390,425	
Stormwater Pollution Control (5%)				$278,875	
TOTAL				$7,418,075	$7,418,075

TABLE 18
PRELIMINARY COST ESTIMATE - MITIGATION PROJECTS
SALADO CREEK

DESIGN COMPONENTS	UNIT	UNIT COST	NUMBER OF UNITS	CONSTRUCTION COSTS PER COMPONENT	TOTAL COST PER PROJECT
7. Channelization (Beitel Creek)					
Channelization (Section 210 to 3370)					
Excavation/Disposal of Material	CY	$6	42800	$256,800	
Right of Way	AC	$10,000	14	$140,000	
Misc. (Utilities, Fences, etc.)				$100,000	
Total Construction Costs				*$356,800*	
Mobilization (11%)				$39,248	
Preparation of ROW (4%)				$14,272	
Subtotal				$410,320	
Contingencies (10%)				$41,032	
Engineering (11%)				$45,135	
Administration (7%)				$28,722	
Stormwater Pollution Control (5%)				$20,516	
TOTAL				$685,726	$685,726
8. Reroute Holbrook Rd. at Austin Hwy.					
Right of Way	AC	$10,000	4	$40,000	
Misc. (Utilities, Fences, etc.)				$200,000	
Total Construction Costs				*$200,000*	
Mobilization (11%)				$22,000	
Preparation of ROW (4%)				$8,000	
Subtotal				$230,000	
Contingencies (10%)				$23,000	
Engineering (11%)				$25,300	
Administration (7%)				$16,100	
Stormwater Pollution Control (5%)				$11,500	
TOTAL				$345,900	$345,900
9. Channelization					
Channel Clearing (Station 54659 to 63552)	AC	$2,500	112	$280,000	
Excavation/Disposal of Material	CY	$6	170000	$1,020,000	
Right of Way	AC	$10,000	112	$1,120,000	
Misc. (Utilities, Fences, etc.)				$250,000	
Total Construction Costs				*$1,550,000*	
Mobilization (11%)				$170,500	
Preparation of ROW (4%)				$62,000	
Subtotal				$1,782,500	
Contingencies (10%)				$178,250	
Engineering (11%)				$196,075	
Administration (7%)				$124,775	
Stormwater Pollution Control (5%)				$89,125	++
TOTAL				$3,490,725	$3,490,725
10. Channelization (SAIA)					
Channel Clearing (Station 138339 to 151236)	AC	$2,500	180	$450,000	
Excavation/Disposal of Material	CY	$5	2400000	$12,000,000	
Misc. (Utilities, Fences, etc.)				$750,000	
Total Construction Costs				*$13,200,000*	
Mobilization (11%)				$1,452,000	
Preparation of ROW (4%)				$528,000	
Subtotal				$15,180,000	
Contingencies (10%)				$1,518,000	
Engineering (11%)				$1,669,800	
Administration (7%)				$1,062,600	
Stormwater Pollution Control (5%)				$759,000	++
TOTAL				$20,189,400	$20,189,400

TABLE 18
PRELIMINARY COST ESTIMATE - MITIGATION PROJECTS
SALADO CREEK

DESIGN COMPONENTS	UNIT	UNIT COST	NUMBER OF UNITS	CONSTRUCTION COSTS PER COMPONENT	TOTAL COST PER PROJECT
Structures (Bridges, Culverts)					
Bridges:					
West Ave. at Salado Creek				$2,682,000	
Vicar Rd. at Beitel Creek				$1,500,000	
Binz-Engleman Rd. at Salado Creek				$3,240,000	
IH 35 Frontage Roads at Salado Creek				$3,000,000	
Roland St. at Salado Creek				$2,400,000	
Culverts:					
West Ave. at Panther Springs Creek				$250,000	
Jones Maltsberger at Mud Creek				$250,000	
Jones Maltsberger at Elm Creek				$400,000	
Bulverde Rd. at Redland Rd.				$500,000	
Total Construction Costs				$14,222,000	
Mobilization (11%)					
Preparation of ROW (4%)					
Subtotal					
Contingencies (10%)				$1,422,200	
Engineering (11%)				$1,564,420	
Administration (7%)				$995,540	
Stormwater Pollution Control (5%)				$711,100	
TOTAL				$18,915,260	$18,915,260
Buy-out remaining Houses or Properties within100-year Floodplain					
Cresthill Rd.	EA	$58,850	2	$117,700	
East Park Subdivision	EA	$18,500	1	$18,500	
Holbrook Rd.	EA	$48,550	4	$194,200	
Nacogdoches Rd.	EA	$246,700	1	$246,700	
Maltsberger Lane	EA	$426,500	1	$426,500	
North Loop Rd.	EA	$62,500	2	$125,000	
West Ave.	EA	$55,490	2	$110,980	
N.E. Loop 410	EA	$680,000	1	$680,000	
Weidner Rd.	EA	$72,367	3	$217,100	
TOTAL				$2,136,680	$2,136,680
Grand Total					$25,679,135

* Cost not included in Grand Total (Federally Funded Project)
\+ Cost not included in Grand Total (Project not Recommended)
++ Cost not included in Grand Total (Project not Recommended)

5

Summary Phase

A. Floodplain Maps

Delineation of the floodplains has produced a set of new floodplain maps at a scale of 1"=200'. Maps generated are based upon the hydrologic and hydraulic modeling produced with this study. The floodplains produced and mapped are the 10, 25, 50, 100, and 500 year frequency storm limits under existing development conditions. The new maps are based on the aerial maps provided by the City of San Antonio and the new floodplains have been indicated on the aerial topographic maps.

B. Mitigation

The Salado Creek Watershed is similar to the other watersheds in Bexar County, yet it has unique features that provide the benefit of detention. The watersheds have similar soils, land uses, geologic features, vegetative habitats, and climates. The detention features that are unique to the Salado Creek Watershed provide flood control, erosion and sedimentation control, and recharge of the Edwards Aquifer. Results produced by these features are the same goals sought when considering and designing mitigation projects.

Mitigation projects were developed for the elimination of structural flooding. The mitigation projects have been analyzed and evaluated for benefit and cost. Seven projects of the ten developed will provide a significant reduction of flooding and are recommendations of this study. The other three projects do not provide cost effective or sufficient relief and/or create additional flooding downstream and are not recommended.

C. Recommendations for Master Drainage Plan

Projects proposed for mitigation of flooding were described in Chapter 4 and the benefits gained from construction of the recommended projects are presented in Table 19. Implementation of the proposed projects has been prioritized based on benefits gained. Prioritized implementation is also presented in Table 19. Description of the prioritization, benefit, and cost are provided as follows.

In the first two columns of the benefit and cost matrix is a list of the structures within the floodplain and their location. The first row of the matrix presents the projects by number as identified in Chapter 4. An example is project five shown in column three which represents the proposed levee project south of Martin Luther King Drive. Structures listed in that column benefit from this project with the estimated cost of the project provided at the bottom of the column. The remaining columns represent the other proposed projects identified by number in the first row. Projects were prioritized by greatest benefits produced.

Projects six and nine are not recommended based upon higher cost and negative dowstream effects associated with their construction. Properties that do not benefit from the proposed mitigation projects are proposed for acquisition and presented in the column titled Acquisition in Table 19. Mitigation for these properties is either cost prohibitive or unfeasible. Values for the individual properties were presented in Table 18, Chapter 4. The last column displays a project that the City of San Antonio has initiated at the San Antonio International Airport. Analysis of the project with the HEC-2 model revealed benefits for seven structures adjacent to the project.

New bridges and culverts were not included in Table 19, however, priority has been determined for new crossings. Priority for new bridge and culvert projects is based upon average daily traffic flows and utilization from area development. A new bridge at West Avenue and Salado Creek along with new box culverts at West Avenue and Panther Springs creek are placed first in priority. Second priority is placed on a new bridge for Vicar Drive at Beitel Creek. The bridges and culverts are prioritized as follows:

1. West Avenue at Salado Creek and Panther Springs Creek	$ 3,899,560
2. Vicar Road and Beitel Creek	$ 1,995,000
3. Roland Street at Salado Creek	$ 3,192,000
4. Jones Maltsburger and Mud Creek	$ 332,500
5. Jones Maltsburger and Elm Creek	$ 532,000
6. Binz-Engleman Road and Salado Creek	$ 4,309,200
7. I.H. 35 Frontage Road and Salado Creek	$ 3,990,000
8. Bulverde Road and Elm Waterhole Creek	$ 665,000
GRAND TOTAL	$18,516,260

Locations of the proposed projects and acquisitions are shown on Figure 14.

D. Summary

This Salado Creek Watershed study was performed for the purpose of preparing a Drainage Master Plan. The Drainage Master Plan consist of the flood plain maps and the projects identified for mitigation of flooding. Utilizing the flood plain maps for regulating future development can prevent additional flooding problems. An implementation of the projects recommended in this study can eliminate existing flooding problems.

An important feature that should be preserved is the natural condition of Salado Creek. Linear channel storage determined and verified with this study is natural detention that has reduced the storm water flows and water surface elevations along the lower Salado Creek. Alteration of the natural conditions will create an increase in flooding in downstream areas. Maintaining the linear channel storage can be done by retaining the existing conditions which include the dense vegetation. Debris and rubbish that has been dumped into the creeks should be cleaned up to preserve the environment.

In conclusion of this study, it has been determined that $25,679,135 can eliminate a majority of the flooding problems within the Salado Creek Watershed. Inclusion of federally funded project Dam #15 eliminates the remainder of the flooding problems. It is recommended that efforts be made to ensure the design and construction of the federally funded Floodwater Retarding Dam to be located in McAllister Park. The proposed dam will provide significant mitigation benefits that are worth the effort associated with implementation of this project. As with the existing thirteen dams, a large reduction in storm water flows and water surface elevations will result.

Table 19
Mitigation Benefit and Cost Matrix
Salado Creek Watershed
Drainage Master Plan

PRIORITIZED PROJECT IMPLEMENTATION		5	7	***1	3	4	8	2	**6	**9	**10	Acquisition
STRUCTURES	LOCATION	PROPERTIES AND STRUCTURES REMOVED FROM THE FLOODPLAIN										
6 Houses	Cresthill Rd.											6 Houses
99 Houses	East Park Subdivision	80 Houses		19 Houses								1 House
4 Apartment Bldgs.	East Park Subdivision	4 Bldgs.										
2 Churches	East Park Subdivision	2 Churches										
1 Houses	Holbrook Rd.			1 House								
3 Houses	Holbrook Rd.											3 Houses
2 Commercial Bldgs.	Holbrook Rd.			2 Bldgs.								
1 Office	Holbrook Rd.					1 Office						
1 Church Academy	Holbrook Rd.					1 Church						
Flea Market	Holbrook Rd.			1 Bldg.					1 Bldg.			
Trailer Park	Holbrook Rd.			1 Park								
3 Houses	Holbrook Rd.					3 Houses						
Flooded Roadway	Rittiman Rd.			1 Roadway								
15 Commercial Bldgs.	Eisenhauer Rd.											
Flooded Roadway	Eisenhauer Rd.			1 Roadway								
Flooded Roadway	Ira Lee Rd.			1 Roadway								
2 Houses	Ira Lee @ Loop 410			2 Houses					1 House			
5 Commercial Bldgs.	Los Patios Village			4 Bldgs.								
4 Commercial Bldgs.	Nacogdoches Rd.											6 Bldgs
Flooded Roadway	Nacogdoches Rd.			1 Roadway								
7 Houses	Gemini Dr.			8 Houses								
3 Commercial Bldgs.	Bitters Rd.										3 Bldgs.	
4 Commercial Bldgs.	Jones Maltsberger Rd.										4 Bldgs.	
1 House	Maltsberger Lane											1 House
2 Commercial Bldgs.	Beacon Circle Industrial Subd.											
1 House	North Loop Rd.											1 House
4 Houses	North Loop West											4 Houses
Flooded Roadways	West Avenue											
Flooded Roadway	Starcrest Rd.											
Flooded Roadways	Jones Maltsberger Rd.											
Flooded Roadway	Bulverde Rd. at Redland Rd.											
24 Houses	Fairfield Village North		24 Houses									
6 Apartment Bldgs.	Renaissance Village North		6 Bldg.									
1 Commercial Bldg.	Perrin Beitel Rd.		1 Bldg.									
1 Commercial Bldg.	Vicar Dr.		1 Bldg.									
1 Commercial Bldg.	Loop 410											1 Bldg.
13 Houses	Garden Court East Subd.				13 Houses							
18 Commercial Bldgs.	Austin Hwy. Industrial Subdivision				18 Bldgs.							
Flooded Roadway	Shertz Rd.											
5 Houses	Weidner Rd.											5 Houses
4 Commercial Bldgs.	Weidner Rd.											4 Bldgs
Flooded Roadways	Weidner, Old O'Conner, & Lookout							3 Roadways				
	Estimated Costs	**$ 458,857**	**$ 685,726**	**$ 6,000,000**	**$ 1,330,737**	**$ 961,225**	**$ 345,900**	**$ 844,750**	**$ 7,418,075**	**$ 3,490,725**	**$ 20,189,400**	**$ 2,136,680**

* Existing Preliminary Stage Project ** Construction not Recommended *** Federally Funded Project

TOTAL **$ 6,763,875**

SALADO CREEK WATERSHED STUDY AND DRAINAGE MASTER PLAN

Contract # 95-483-080

(1) Large Scale Map located in the official file, may be copied upon request. March 1997

Please Contact Research and Planning Fund Grants Management Division at (512)463-7926

SALADO CREEK WATERSHED STUDY AND DRAINAGE MASTER PLAN

VOLUME 2

Appendix A

Table A

Agency	Information Source Reviewed
City of San Antonio Engineering Division, Drainage Section	Subdivision Files Street Improvement Projects Drainage Complaints
City of San Antonio, Department of Parks and Recreation	Master Plan
San Antonio Water System	NA
San Antonio River Authority	Phase 1 Inspection Reports, National Dam Safety Program As Built Construction Plans Breech Analysis, DAMS-2 Dam Safety Inspection Report
Bexar County Public Works	Flood Plain Complaints Flood Plain Development Permit Applications "Flood Protection Plan for portions of Salado, Cibolo and Leon Creeks"
Fort Sam Houston, Public Works Division	Previous Studies Landuse Planning Maps
Edwards Underground Water District	Water Pollution Abatement Plans
Texas Department of Transportation	Federal Road Projects
TNRCC	Development Applications adjacent to Flood Plain
U.S. Agricultural Soil Conservation Service	Existing TR 20 model of Watershed Soil Survey Geographic Data Base
U.S. Army corps of Engineers	Existing FEMA model

Table C

City of San Antonio, Department of Parks and Recreation

Source of Information	Information Contained In Source	
	General Information	Information Found
Master Plan	McAllister Park plats and master plan report.	Plats of existing and proposed land use and a master plan.

Table D

San Antonio River Authority

Source of Information	Information Contained In Source	
	General Information	Locations
Phase 1 Inspection Reports, National Dam Safety Program	Location map, pertinent dam data, engineering data, drainage area map and may also include hydrographs.	For Dams 1, 2, 4, 5, 8, 12, 13A and 13B on Salado Creek Upper Watershed.
As built Construction Plans	Embankment plan, profiles and sections, and may also include a general plan.	For Dams 1, 2, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13A and 13B for Salado Creek Upper Watershed.
Breech Analysis, DAMS-2	Input data, output data, and computer generated hydrographs.	For Dams 4, 5, 6, 7, 8, 9, 10 and 11. No analysis was done for other dams in the Salado Creek Upper Watershed.
Dam Safety Inspection Reports	Size classification for dam, hazard classification, visual inspection, reservoir area and instrumentation.	For Dams 1, 2, 4, 5, 6, 7, 8, 9, 11, 12, 12, 13A and 13B on Salado Creek Upper Watershed. These reports were mostly yes or no questions, so the information was not pertinent to the Salado Creek Watershed.

Table G

Edwards Underground Water District

Source of Information	Information Contained In Source	
	General Information	Information Found
Water Pollution Abatement Plans	Total impervious cover, wastewater to be generated, size of the project, existing conditions and a geological assessment.	Locations were reviewed that were both in the Edwards Recharge Zone and in the Salado Creek Watershed:
		Sendero Ranch
		Blanco Bluffs Unit 1
		Blanco Woods
		Alzafar Shrine
		Big Spring/Evans Road
		Canyon Oaks Churchill Estates
		Club at Sonterra
		Comers at Deerfield(FARMCO #64)
		Cornerstone Church School Facility
		Cornerstone Church Parking Expansion
		Cornerstone Church-West Access Road
		Deerfield Units 6B, 11, 12 and 13
		Deerwood
		Diamond Shamrock No. 1020, 1038 and 1039
		Emerald Forest Units 1 and 2-8
		Encino Forest Unit 2
		Enclave at Hollywood Park
		Enclave at Sonterra
		Estates at Arrowhead
		Gates of Deerfield
		Fountains at Deerfield
		Greystone Country Estates
		Inwood Units 1F, 4 and 2G
		Inwood Hollow
		Inwood Heights
		Inwood Booster Station
		Inwood Fill Site Reclamation
		Inwood Village and Unit 6C
		Las Lomas
		Mission Ridge PUD II
		Northeast YMCA Athletic Fields
		Northside Funeral Chapel
		Northwoods Retail Center Oaklands
		Oakwood Units 1, 2 and 3
		Panther Springs Golf Driving Range
		Parktrail
		Redland Heights
		Redland Oaks Units 2A and 2B
		Redland Woods
		Shady Oaks
		Shavano Park Unit 15D
		67.947 Acre Jones Maltsberger Road Tract
		Sonic at Thousand Oaks
		St. Thomas Episcopal Church
		St. Andrew Lutheran Church
		Turkey Creek Unit 2
		Vistas of Encino Park Units 1 and 3
		Vistas at Sonterra
		West Shavano Development
		Woods at Sonterra Unit 4A

Table H

Texas Department of Transportation

Source of Information	**Information Contained In Source**	
	General Information	Information Found
Federal Road Projects	Plans, profiles, layout of connectors and may also include a drainage area map.	*Salado Creek* at: Southeast Loop 410 Southeast Military Drive (LP13) Rigsby IH 10 Gembler Road (MH 736) IH 35 Rittiman Road (MH 61) Austin Highway (Loop 368) Northeast Loop 410 U.S. 281 Blanco Road (FM 2696) Loop 1604
		Perrin Beitel Creek at: Northeast Loop 410 Nacogdoches Road (FM 2252)
		Panther Springs Creek at: Blanco Road (FM 2696) Loop 1604
		Mud Creek at: Loop 1604 U.S. 281
		Elm Creek at: Loop 1604
		Elm Waterhole Creek at: Loop 1604
		West Elm Creek at: U.S. 281

Appendix B

UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION   STATE 48 DIST 48F
PRIMARY COMPUTATIONS OF RAINFALL (INCHES)   RATINGS USED --
08178700   DATE PROCESSED: 01-24-1995 @ 10:44 BY JOPENA   INPUT 0001 10/01/85 (0001)
SALADO CREEK (UPPER STATION) AT SAN ANTONIO, TX
(00045) RAINFALL   STORE STATISTIC(S) 00006
PROVISIONAL DATA FOR WATER YEAR ENDING SEPT. 30, 1991   TEST DIFF: ******   PUNCH INTERVAL: 5 MIN

	DATE	RAINFALL TOTAL	DATUM CORR	<TIME>	RAINFALL AMOUNTS AT INDICATED TIMES IN INCHES											
P	04/04/91	.71		<1315>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<1615>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<1915>	.00	.00	.00	.25	.20	.00	.01	.00	.00	.14	.05	.02
				<2215>	.03	.00	.00	.00	.01	.00	.00	.00				
	04/05/91	1.03		<0015>	.02	.14	.14	.13	.34	.26	.00	.00	.00	.00	.00	.00
				<0315>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<0615>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<0915>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<1215>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<1515>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<1815>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<2115>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	04/06/91	.33		<0015>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<0315>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<0615>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<0915>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<1215>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<1515>	.00	.00	.00	.00	.02	.06	.03	.03	.00	.00	.00	.00
				<1815>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<2115>	.00	.00	.00	.00	.00	.00	.01	.08	.00	.05	.05	.00
	04/07/91	.00		<0015>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<0315>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<0615>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<0915>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<1215>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<1515>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<1815>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
				<2115>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00

NOTE. SYMBOLS USED ABOVE HAVE THE FOLLOWING MEANINGS --
P - DAILY SUMMARY IS FOR AN INCOMPLETE DAY

UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION STATE 48 DIST 48F
PRIMARY COMPUTATIONS OF GAGE HEIGHT AND DISCHARGE RATINGS USED --
78700 DATE PROCESSED: 01-15-1992 @ 10:22 BY FAWESSELS INPUT 10.0 01/01/01 (0001)
ADO CREEK (UPPER STATION) AT SAN ANTONIO, TX INPUT DD: FROM ADR STNRD 10.0 01/01/01 (0001)
PUT PARAMETER 00060 STORE STATISTIC(S) 00003
VISIONAL DATA FOR WATER YEAR ENDING SEPT. 30, 1991 TEST DIFF: 0.20 PUNCH INTERVAL: 30 MIN

DATE	MAX GH /DISCH <TIME>	MIN GH /DISCH <TIME>	MEAN GH	MEAN DISCH	SHIFT ADJ	DATUM CORR	0100 1300	0200 1400	0300 1500	0400 1600	0500 1700	0600 1800	0700 1900	0800 2000	0900 2100	1000 2200	1100 2300	1200 2400
							STAGE, IN HUNDREDTHS OF FEET, AT INDICATED HOURS											
03/30/91	2.65	2.57	2.59	37	0.04W	-0.08	259	259	259	259	259	259	259	259	259	259	259	259
	1.8	.18					258	258	258	258	258	258	258	258	257	260	260	259
	<2130>	<2100>																
03/31/91	2.63	2.59	2.61	84	0.05W	-0.09	259	259	259	259	259	259	260	262	262	263	262	262
	1.1	.40					263	263	263	263	263	263	263	263	262	262	262	262
	<1230>	<0030>																
04/01/91	2.61	2.59	2.60	53	0.04W	-0.09	261	261	261	261	261	261	260	260	260	260	260	260
	.63	.40					260	260	260	260	260	260	260	260	260	259	259	259
	<0030>	<2130>																
04/02/91	2.59	2.57	2.58	25	0.03W	-0.09	259	259	259	259	259	258	258	258	258	258	258	257
	.40	.18					257	257	257	257	257	257	257	257	257	257	257	257
	<0030>	<1200>																
04/03/91	2.57	2.57	2.57	18	0.03W	-0.09	257	257	257	257	257	257	257	257	257	257	257	257
	.18	.18					257	257	257	257	257	257	257	257	257	257	257	257
	<0030>	<0030>																
04/04/91	3.03	2.56	2.59	79	0.02W	-0.09	257	257	257	257	257	257	257	256	256	256	256	256
	20	.14					256	256	256	256	256	256	256	256	261	274	263	303
	<2400>	<0800>																
04/05/91	12.04	3.07	6.55	2750	0.00W	-0.09	316	405	479	674	1178	1204	1134	1005	880	772	698	647
	12900	25					612	581	563	549	539	530	524	515	507	508	502	497
	<0600>	<0030>																
04/06/91	4.94	3.70	4.28	328	[illegible]	-0.09	492	488	484	485	487	486	484	482	470	461	445	425
	569	137					407	389	376	373	372	371	372	373	377	375	375	382
	<0030>	<1730>																
04/07/91	4.66	3.17	3.74	162	[illegible]	-0.09	427	465	441	431	429	423	410	392	390	377	370	366
	441	37					355	352	348	343	340	336	332	329	326	323	320	317
	<0130>	<2400>																
04/08/91	3.16	2.93	3.02	20	[illegible]	-0.09	315	313	311	310	308	307	306	304	303	302	302	301
	36	12					301	299	297	297	297	297	297	296	296	293	293	293
	<0030>	<2200>																
04/09/91	2.93	2.89	2.90	9.9	[illegible]	-0.09	293	293	293	290	290	290	290	290	290	290	290	290
	12	9.2					290	290	290	290	290	290	289	289	289	289	289	289
	<0030>	<1900>																
04/10/91	2.89	2.87	2.88	8.4		-0.09	289	289	289	288	288	288	288	288	288	287	287	287
	9.2	8.1					287	287	287	287	287	287	287	287	287	287	287	287
	<0030>	<0930>																
04/11/91	2.90	2.87	2.88	8.5		-0.09	287	287	287	287	287	287	287	287	287	287	289	288
	9.7	8.1					288	288	288	288	288	288	288	288	288	288	288	288
	<1030>	<0030>																
04/12/91	2.88	2.87	2.88	8.4		-0.09	288	288	288	288	288	288	288	288	288	288	288	288
	8.6	8.1					288	287	287	287	287	287	287	287	287	287	287	287
	<0030>	<1400>																
04/13/91	2.92	2.87	2.89	9.2		-0.09	287	287	287	287	287	287	287	287	290	289	289	290
	11	8.1					291	292	292	290	290	290	290	290	290	290	290	290
	<1330>	<0030>																

UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION STATE 48 DIST 48F
PRIMARY COMPUTATIONS OF GAGE HEIGHT AND DISCHARGE RATINGS USED --
08178800 DATE PROCESSED: 02-13-1992 @ 09:55 BY JATOMLINSON INPUT 9.0 06/01/84 (1100)
SALADO CREEK (LOWER STATION) AT SAN ANTONIO, TX INPUT DD: FROM ADR STNRD 9.0 06/01/84 (1100)
OUTPUT PARAMETER 00060 STORE STATISTIC(S) 00003
PROVISIONAL DATA FOR WATER YEAR ENDING SEPT. 30, 1991 TEST DIFF: 0.20 PUNCH INTERVAL: 60 MIN

STAGE, IN HUNDREDTHS OF FEET, AT INDICATED HOURS

	DATE	MAX GH /DISCH <TIME>	MIN GH /DISCH <TIME>	MEAN GH	MEAN DISCH	SHIFT ADJ	DATUM CORR	0100 1300	0200 1400	0300 1500	0400 1600	0500 1700	0600 1800	0700 1900	0800 2000	0900 2100	1000 2200	1100 2300	1200 2400
	04/02/91	6.92	6.80	6.90	14P	0.37W	0.00												
		15	10						692	691	691	690	689	680	688	686	685	685	684
		<1400>	<1900>																
	04/03/91	6.90	6.86	6.88	13	0.37W	0.00	686	686	686	686	686	686	686	686	688	689	689	689
		14	12					689	690	690	690	690	690	690	690	690	690	690	690
		<1400>	<0100>																
	04/04/91	6.90	6.88	6.90	14	0.37W	0.00	690	690	690	690	690	690	690	690	690	690	690	690
		14	13					690	690	690	690	690	690	690	690	689	689	688	688
		<0100>	<2300>																
A	04/05/91	20.98	6.90	13.44	1610	0.09W	0.00	690	959	1137	1044	878	918	993	1035	1037	1038	1049	1055
		4770	14					1119	1220	2038	2098	2095	2047	1956	1852	1750	1646	1542	1456
		<1600>	<0100>																
A	04/06/91	13.80	10.05	11.10	575	0.09W	0.00	1380	1323	1271	1235	1198	1173	1142	1128	1098	1087	1069	1051
		1220	386					1043	1034	1031	1024	1016	1022	1027	1025	1019	1012	1009	1005
		<0100>	<2400>																
A	04/07/91	13.56	8.54	11.46	664	0.11W	0.00	1036	1066	1097	1149	1201	1232	1273	1305	1333	1353	1356	1345
		1140	192					1317	1275	1226	1178	1122	1068	1010	957	915	887	867	854
		<1100>	<2400>																
	04/08/91	8.42	7.56	7.90	107	0.44W	0.00	842	834	826	817	812	807	802	798	795	791	789	785
		173	68					783	780	778	773	773	769	766	764	762	760	758	756
		<0100>	<2400>																
	04/09/91	7.55	7.37	7.45	58	0.44W	0.00	755	753	752	750	749	749	748	748	747	746	744	744
		67	51					744	743	743	742	742	742	742	741	740	738	738	737
		<0100>	<2400>																
	04/10/91	7.37	7.29	7.32	48	0.44W	0.00	737	736	736	736	735	735	734	733	733	732	732	732
		51	45					732	731	731	731	731	731	730	730	730	729	729	729
		<0100>	<2200>																
	04/11/91	7.29	7.26	7.27	43	0.44W	0.00	729	728	727	727	727	727	727	727	727	726	726	726
		45	43					726	726	726	726	726	726	726	726	726	726	726	726
		<0100>	<1000>																
	04/12/91	7.26	7.24	7.25	42	0.44W	0.00	726	726	726	726	726	726	726	726	726	725	725	725
		43	41					725	725	725	725	725	725	725	725	725	724	724	724
		<0100>	<2200>																
	04/13/91	7.26	7.23	7.24	41	0.44W	0.00	724	724	724	724	724	724	724	723	723	723	723	723
		43	40					723	723	723	723	723	723	724	725	725	726	726	726
		<2200>	<0800>																
A	04/14/91	10.59	7.26	8.97	251	0.33W	0.00	726	727	727	728	731	812	820	824	870	933	990	1025
		455	43					1044	1055	1059	1052	1033	1008	976	940	910	885	866	849
		<1500>	<0100>																
	04/15/91	8.38	7.39	7.77	93	0.44W	0.00	838	830	820	814	805	801	796	790	783	778	773	770
		168	53					765	762	759	756	753	750	748	746	744	743	741	739
		<0100>	<2400>																
	04/16/91	7.37	7.24	7.30	46	0.44W	0.00	737	737	736	736	734	733	732	732	731	729	729	729
		51	41					729	728	728	728	727	727	726	726	725	724	724	724
		<0100>	<2200>																

Sensor # 4121 - Salado Ck. SCS Dam 5 Precip. Gage (inches)		
Date	Time	Precip.
4/5/91	1:00	5.83
4/5/91	12:45	5.83
4/5/91	12:30	5.75
4/5/91	12:15	5.75
4/5/91	12:00	5.75
4/5/91	11:45	5.75
4/5/91	11:30	5.75
4/5/91	11:15	5.75
4/5/91	11:00	5.67
4/5/91	10:45	5.67
4/5/91	10:30	5.67
4/5/91	10:15	5.67
4/5/91	10:00	5.59
4/5/91	9:45	5.55
4/5/91	9:30	5.55
4/5/91	9:15	5.52
4/5/91	9:00	5.52
4/5/91	8:45	5.52
4/5/91	8:30	5.52
4/5/91	8:15	5.52
4/5/91	8:00	5.48
4/5/91	7:45	5.40
4/5/91	7:30	5.40
4/5/91	7:15	5.36
4/5/91	7:00	5.28
4/5/91	6:45	5.20
4/5/91	6:30	5.20
4/5/91	6:15	5.20
4/5/91	6:00	5.20
4/5/91	5:45	5.00
4/5/91	5:30	5.00
4/5/91	5:15	4.93
4/5/91	5:00	4.85
4/5/91	4:45	4.81
4/5/91	4:30	4.69
4/5/91	4:15	4.61
4/5/91	4:00	4.53
4/5/91	3:45	4.45
4/5/91	3:30	4.33
4/5/91	3:15	4.29
4/5/91	3:00	4.29
4/5/91	2:45	4.29
4/5/91	2:30	4.29
4/5/91	2:15	4.29
4/5/91	2:00	3.47
4/5/91	1:45	3.31
4/5/91	1:30	3.00
4/5/91	1:15	2.60
4/5/91	1:00	1.89
4/5/91	0:45	1.15
4/5/91	0:30	0.59
4/5/91	0:15	0.44
4/5/91	0:00	0.36
4/4/91	23:45	0.32
4/4/91	23:30	0.28
4/4/91	23:15	0.20
4/4/91	23:00	0.16
4/4/91	22:45	0.08
4/4/91	22:30	0.00

Sensor # 4881 - Spur 122 @ Salado Ck. Precip. Gage			
Date	Time		Precip
4/6/91	102	31.38	0
4/5/91	13:02	31.38	0
4/5/91	9:46	31.38	0
4/5/91	8:46	31.3	0.08
4/5/91	8:26	31.26	0.04
4/5/91	8:12	31.22	0.04
4/5/91	8:00	31.18	0.04
4/5/91	7:41	31.1	0.08
4/5/91	7:30	31.06	0.04
4/5/91	7:19	31.02	0.04
4/5/91	7:11	30.98	0.04
4/5/91	6:55	30.94	0.04
4/5/91	6:45	30.91	0.03
4/5/91	6:40	30.87	0.04
4/5/91	6:21	30.79	0.08
4/5/91	6:15	30.75	0.04
4/5/91	6:11	30.71	0.04
4/5/91	6:09	30.67	0.04
4/5/91	6:07	30.63	0.04
4/5/91	6:04	30.59	0.04
4/5/91	5:54	30.51	0.08
4/5/91	5:24	30.47	0.04
4/5/91	5:18	30.39	0.08
4/5/91	4:55	30.35	0.04
4/5/91	4:45	30.31	0.04
4/5/91	416	30.28	0.03
4/5/91	317	30.24	0.04
4/5/91	314	30.2	0.04
4/5/91	311	30.16	0.04
4/5/91	310	30.12	0.04
4/5/91	307	30.08	0.04
4/5/91	306	30.04	0.04
4/5/91	305	30	0.04
4/5/91	304	29.96	0.04
4/5/91	303	29.92	0.04
4/5/91	301	29.84	0.08
4/5/91	300	29.8	0.04
4/5/91	257	29.68	0.12
4/5/91	257	29.65	0.03
4/5/91	256	29.61	0.04
4/5/91	255	29.57	0.04
4/5/91	254	29.49	0.08
4/5/91	253	29.37	0.12
4/5/91	251	29.25	0.12
4/5/91	250	29.17	0.08
4/5/91	249	29.13	0.04
4/5/91	249	29.09	0.04
4/5/91	248	29.06	0.03
4/5/91	247	28.98	0.08
4/5/91	247	28.94	0.04
4/5/91	244	28.7	0.24
4/5/91	243	28.54	0.16
4/5/91	242	28.5	0.04
4/5/91	242	28.43	0.07
4/5/91	241	28.35	0.08
4/5/91	241	28.31	0.04

4/5/91	240	28.23	0.08
4/5/91	239	28.11	0.12
4/5/91	239	28.03	0.08
4/5/91	237	27.91	0.12
4/5/91	237	27.87	0.04
4/5/91	236	27.8	0.07
4/5/91	236	27.76	0.04
4/5/91	235	27.72	0.04
4/5/91	234	27.56	0.16
4/5/91	234	27.52	0.04
4/5/91	224	27.28	0.24
4/5/91	206	27.24	0.04
4/5/91	157	27.2	0.04
4/5/91	1:42	27.17	0.03
4/5/91	0:22	27.13	0.04
4/4/91	23:55	27.09	0.04
4/4/91	13:02	27.05	0.04

NWS FORM E-5 (7-78) (PRES. BY WSOM E-41)

NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION U.S. DEPARTMENT OF COMMERCE NATIONAL WEATHER SERVICE

RIVER DISTRICT OFFICE: SAN ANTONIO WSFO

MONTHLY REPORT OF RIVER AND FLOOD CONDITIONS

REPORT FOR: MONTH APRIL | YEAR 1991

TO: Hydrologic Services Division, W22
National Weather Service
National Oceanic and Atmospheric Administration
Silver Spring, Maryland 20910

SIGNATURE: John Patton
In Charge of River District

DATE: MAY 6, 1991

When no flooding occurs, include miscellaneous river conditions, such as significant rises, record low stages, ice conditions, snow cover, droughts, and hydrologic products issued (WSOM E-41).

☐ No flood stages were reached in this river district for the month indicated above.

On April 5th, South Texas got a jump start on the flood season. Flooding was widespread and it was disastrous. A long wave was moving very slowly across Arizona and New Mexico extending down into Mexico. There was practically no pressure gradient above 500 millibars over Texas. The atmospheric sounding was very unstable with precipitable water at 1.70 inches in Brownsville, and biased very heavily in the lower layers.

A cluster of very slow moving impulses around the front of this long wave created extremely intense rain centers in the Brownsville-Harlingen area, San Antonio, and the interior mid-Coastal Bend area between Victoria and Angleton. The Harlingen area received over 9 inches of rain in a three to four hour period of the early morning hours. Harlingen would go on to receive 17.10 inches for a two day storm total, San Benito 13.96, and Brownsville 10.32 inches (see attached isohyetal).

Large rainfall amounts in this area are disastrous because it is in the historical flood plain of the Rio Grande River. Numerous old channels (resacas) meander through the area. Three days after the flood (4/5), 60% of the originally flooded 2500 homes in Harlingen and 90% of the 2000 homes in San Benito remained flooded. Some portions of Highway 77 were still flooded.

In April, Cameron County, (Brownsville) qualified as a federal flood disaster area, and Nueces County, (Corpus Christi) qualified as a federal drought disaster area.

San Antonio received over nine inches of rain between 11:30 PM 4/4 and 1:30 AM 4/5 in Shavano Park and Woods of Shavano. Numerous reports over 9.50 inches were received in the headwaters of Olmos Creek and Salado Creek. The upper Leon Creek drainage also received large totals but not that much (see attached isohyetal).

The headwater tables indicated a crest on ~~Leon~~ Olmos Creek at ~~Dresden Drive~~ near 15 feet. The USGS reported an observed crest of 14.38 feet. The record stage is 14.82 feet, Sept. 13, 1978. The USGS feels this is undoubtedly a record flow (19,670 cfs) since the channel has been greatly enlarged and concrete lined since the 1978 flood. The drainage is 21.2 mi**2 so this is a runoff of 928 cfs/mi**2.

The tables indicated 14 feet at Salado Creek at Northeast Loop 410 and it crested at 12.13 feet at 5:30 AM that morning. The headwater tables consistently work very well if realistic rainfall areal averages are input.

That sometimes can be a problem at 1:30 AM in 5 inch per hour rainfall. The observer at Woods of Shavano was awakened at 1:30 AM and he waded through the flood and lightening and gave us a 9.10 inch rainfall total. Hydrology is probably out on the ragged edge of liability sometimes, with obserbers dodging lightening bolts to get rainfall reports, and river observers dodging semi's on narrow bridges to get river reports.

Leon Creek was forecasted to reach 14 to 16 feet at the staff gauge on Loop 13, just south of Kelly AFB. A crest was never verified.

Major flooding occurred along the upper reach of Olmos Creek. Ten homes had major flood damage and another six, minor damage. An HEB supermarket had a few inches of water going through it at the crest.

There was the obligatory low water crossing fatality. A young man drowned near Culebra Road on Leon Creek as his car was swept downstream off a low water crossing. He and a companion had just come out of a nightclub and were the second car to go downstream at this site in a very short period. His foot became intangled in the safety belot and between the door post and seat of the drivers side. His companion tried to drag him out to no avail.

If history holds, almost three years to the day, his family will be in court for a multi-million dollar settlement against the city.

The flood drift that awaited viewers with the daylight was composed of twigs, grass, limbs, trees, and cars; lots of cars.

The above traffic fatality and one other in the Austin area were the only ones reported in South and Southeast Texas with all the disastrous flooding. This is a silent testimony to a lot of good work by the Weather Service and emergency officials all over South Texas. This was deadly major flooding.

Victoria received 9.87 inches between 2 and 9 AM of the 5th (see the attached mass rainfall curve). This exceeded the previous 24 hour record amount for the station of 9.30 inches in June 1977.

Note the period between five and six AM when 4.77 inches fell in an hour. Five inch per hour rainfall in South Texas is not uncommon. Other reports in Victoria were 7.02 inches at Ball Airport in the northside to 11.68 inches in the Fleetwood subdivision.

Edna reported 5.85 and 9.00 inches, Yorktown 4.25, Cordele 8.16, Thomaston 5.03 and 5.50, Goliad 4.25 and Ganado 8.55 inches.

The heavy rainfall caused an uncharacteristically sharp rise in the Guadalupe River at Victoria. It rose from 6.91 feet the morning of the 5th to a 27.84 foot crest at 0200 AM on the 6th, almost 7 feet above flood stage. The zoo in Riverside Park begins flooding at 28.5 feet, and homes in Victoria flood at 29.5 feet.

The city park, Riverside, was closed to the public and 12 people had to be rescued from a recreational vehicle park near Riverside Park. Many homes were flooded due to the local rainfall, not river flooding.

Farther up the coastal plain, localized rainfall flooding and clogged drainage ditches flooded 50 homes and busineses in West Columbia also on 4/5. Many subdivisions in Angleton, Bay City, and Palacios were isolated by flooding of streets and roads. Bay City had received 9 inches by 2 PM, Palacios almost 9, and Brazosport over 5 inches.

On 4/13, Granger residents in the Pecos Apartment complex and a few homes on the west side had to be evacuated as Willis Creek and other minor drainages in the area flooded.

Granger only received 2 inches but over 5 inches fell north and west of town. Bartlett had over 5 inches.

The Trinity River again flooded Liberty County, cresting at 26.50 feet on 4/25. Two homes had to be evacuated near the South Liberty Oil Field. When a levee protecting a large ranch broke last spring, it greatly alleviated flooding in Liberty County. Two years ago, people would have been evacuated from three or four communities in the flood plain with this flow.

Now the flood plain cross sectional area has been greatly enlarged and flooding problems are much less.

The Sabine River also experienced major flooding from a three to five inch rainfall event above Toledo Bend Dam in Texas and Louisiana on 4/18. The largest amunt reported was 5.50 inches in Hemphill.

The Newton County Sheriff's Office was evacuating people from the River Bend subdivision below Toledo Bend Dam the morning of 4/19. Many homes flooded again (last time was 1989).

Major flooding breaks the Burkeville and Bon Weir river gauges. This event was no exception. The LARC at Burkeville was almost dead the morning of the 19th, due to low battery voltage. If too many people get the phone number to a LARC, they'll all call the LARC frequently during a flood and drain the battery. If this happens again, we may consider changing the phone number of the Burkeville LARC.

The only thing working flawlessly during this period was Murphy's law. Hank Hughes, the ET at Port Arthur, was in San Antonio on scheduled annual leave. AFOS was down in Houston WSO, an even higher priority than the flooding, the ET in Galveston was working at the airport and in-communicado. The last easy chance was the ET at Lake Charles. He went up and changed the battery managing to keep the internal program intact.

This enabled readings from the gauge until it hung at 41 feet on the way up to a near record 45.5 foot forecast. The record was May 20, 1989 of 47.45 feet. Readings were'nt available from the Burkeville gauge during the flood. The observed crest was near 44.66 feet according to the USGS.

Downstream, Bon Weir's orifice evidently silted in, again standard operatig procedure in major flooding. The gauge was fluctuating a half to a foot per fifteen minute poll. The USGS recorded a crest near 36.10 feet.

Deweyville crested at 26.53 feet. This closed the only road into a subdivision of 30 to 40 homes beside the Sabine River on the Texas side. At 27 feet, the first homes flood in this area.

April rainfall	% of 30 yr normal (51-80)	Apr deficit	Calendar year rainfall	% of 30 yr normal (51-80)	Calendar year 1991 deficit	Deficit 1/1/88 to 5/1/91

Del Rio	1.88 in	102%	+0.03	3.24 in	84%	-0.64 in	-2.31 in
Brownsville	10.35	659	+8.78	13.34	274	+8.47	-5.59
Corpus Christi	4.00	201	+2.01	9.34	155	+3.33	-27.98
Victoria	11.09	425	+8.48	25.13	312	+17.07	-16.17
San Antonio	4.91	180	+2.18	13.39	179	+5.92	-2.01
Austin	4.91	158	+1.80	18.01	206	+9.26	-12.11
Houston	8.06	190	+3.82	25.39	190	+12.01	-6.25
Beaumont	6.78	167	+2.73	28.65	193	+13.78	+37.97

APR 25 1991

RECEIVED

EXTRAORDINARY DEEP SOUTH TEXAS FLOOD...APRIL 5-7, 1991

Rains of extraordinary magnitude fell over portions of deep South Texas during the period of April 5-7, 1991. Three day storm totals included 17.07 inches in Harlingen...13.96 inches in San Benito...and 10.32 inches at the National Weather Service Office located on the Brownsville Airport. Over 14 inches of the Harlingen rain total fell on April 5th...mostly between midnight and noon.

Clockwise circulation around a surface high pressure area centered off the southeast U.S. coast brought moisture laden air into deep South Texas that had traveled over water for over 1000 miles. Surface dewpoints were in the 70s. An upper level disturbance...centered in northeast Mexico south of the Big Bend area of Texas...provided a trigger to generate strong thunderstorms over deep South Texas in the very moist and unstable air mass. At the surface...a weak wind shift line (the remnants of a weak cool front) was stationary from portions of northeast Mexico across the San Benito and Harlingen area northeast into the Gulf.

Thunderstorms with very heavy rainfall begin developing along the windshift line just after midnight on April 5th. The thunderstorms moved toward the northeast along the line...but the windshift boundary remained stationary. At least 3 periods of thunderstorms with very heavy rain moved acroes the Harlingen and San Benito areas between midnight and 9 AM on April 5th. The first 2 periods saturated the ground...setting the stage for fast and heavy runoff of any additional rain. The 3rd period saw the development of a storm with rainfall rates near or in excess of 6 inches per hour. This storm inundated the Harlingen/San Benito and surrounding areas. It also produced significant wind damage...including 1 and possibly 2 small tornadoes.

The wind shift line then began moving southeast...and again became stationary over the Brownsville area. The National Weather Service Office in Brownsville received 9.15 inches between about 10 AM and 3 PM on April 5th.

Detailed rainfall records exist for the Brownsville area for a little over 100 years. Such records are not available for Harlingen...but the close proximity of the cities implies similar records. The 24 hour calendar day total of 14.76 inches of rain in Harlingen on April 5th is in the range of the 100 year record rainfall of 12.09 inches for Brownsville. However... rains of such magnitude in the past have always been associated with hurricanes in the month of September. The previous 24 hour record rainfall in Brownsville for April 5th was 1.96 inches prior to the 1991 event. And... to emphasize how dry this time of year normally is...the total of all rain that has ever fallen on March 30th in the 111 years of record at Brownsvllle is 0.41 inches. Thus the 14.76 inches in Harlingen on April 5, 1991, was an event with probably a 500 year or more return frequency for that time of year!

RICHARD R. HAGAN
Meteorologist in Charge
Brownsville Weather Office

UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION STATE 48 DIST 48F
PRIMARY COMPUTATIONS OF RAINFALL (INCHES) RATINGS USED --
08178700 DATE PROCESSED: 09-12-1994 @ 16:19 BY JOPENA INPUT 0001 10/01/85 (0001)
SALADO CREEK (UPPER STATION) AT SAN ANTONIO, TX
(00045) RAINFALL STORE STATISTIC(S) 00006
PROVISIONAL DATA FOR WATER YEAR ENDING SEPT. 30, 1993 TEST DIFF: ****** PUNCH INTERVAL 5 MIN

DATE	RAINFALL TOTAL	DATUM CORR	<TIME>	RAINFALL AMOUNTS AT INDICATED TIMES IN INCHES											
05/01/93	.00		<0015>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0315>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0615>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0915>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1215>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1515>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1815>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<2115>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
05/02/93	.05		<0015>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0315>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.03	.00	.01
			<0615>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0915>	.00	.01	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1215>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1515>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1815>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<2115>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
05/03/93	.00		<0015>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0315>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0615>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0915>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1215>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1515>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1815>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<2115>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
05/04/93	.00		<0015>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0315>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0615>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0915>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1215>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1515>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<1815>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<2115>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
05/05/93	5.78		<0015>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0315>	.00	.00	.00	.00	.00	.00	.13	.07	.02	.00	.00	.07
			<0615>	.00	.00	.00	.00	.00	.03	.15	.03	.10	.11	.00	.00
			<0915>	.00	.00	.00	.00	.00	.00	.06	.84	.60	.76	.46	.16
			<1215>	.00	.00	.26	.01	.00	.02	.01	.06	.21	.00	.00	[illegible]
			<1515>	.02	.09	.00	.00	.00	.18	.33	.30	.00	.07	.00	.00
			<1815>	.01	.04	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<2115>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
05/06/93	.02		<0015>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0315>	.00	.00	.00	.00	.00	.00	.00	.05	.00	.00	.00	.01
			<0615>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
			<0915>	.00	.00	.00	.00	.00	.00	.00	.00	.01	.00	.00	.00
			<1215>	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00

UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION — STATE 48 DIST 465
PRIMARY COMPUTATIONS OF GAGE HEIGHT AND DISCHARGE — RATINGS USED --
08178800 — DATE PROCESSED: 10-15-1993 @ 11:30 BY JFWOJCIK — STNRD 11.0 10/30/92 (1000)
SALADO CREEK (LOWER STATION) AT SAN ANTONIO, TX — INPUT DD: FROM THE DCP
OUTPUT PARAMETER 00060 STORE STATISTIC(S) 00003
PROVISIONAL DATA FOR WATER YEAR ENDING SEPT. 30, 1993 — TEST DIFF: ****** PUNCH INTERVAL: 60 MIN

DATE	MAX GH /DISCH <TIME>	MIN GH /DISCH <TIME>	MEAN GH	MEAN DISCH	SHIFT ADJ	DATUM CORR	STAGE, IN HUNDREDTHS OF FEET, AT INDICATED HOURS: 0100 1300	0200 1400	0300 1500	0400 1600	0500 1700	0600 1800	0700 1900	0800 2000	0900 2100	1000 2200	1100 2300	1200 2400
04/28/93	6.64 18 <0000>	6.64 18 <0000>	6.64	18	0.28W	-0.12	664 664	664 664	664 664	664 664	664 664	664 664	664 664	664 664	664 664	664 664	664 664	664 664
04/29/93	7.15 62 <2400>	6.63 17 <0400>	6.76	27	0.28W	-0.13	664 670	664 672	664 675	663 675	663 678	663 687	663 696	663 702	663 705	664 707	665 711	668 715
04/30/93	7.36 93 <0400>	7.05 50 <1900>	7.18	67	0.28W	-0.14	722 710	727 706	732 709	736 717	736 717	735 709	734 705	729 705	726 705	722 705	717 706	713 705
05/01/93	7.06 51 <0000>	6.87 33 <2300>	6.96	42	0.28W	-0.14	706 697	706 696	705 695	702 694	701 693	699 692	698 690	698 689	698 688	698 688	698 687	698 687
05/02/93	6.87 33 <0000>	6.73 23 <2200>	6.79	27	0.28W	-0.15	686 677	685 677	684 676	684 675	683 675	682 675	682 675	681 674	681 674	680 673	680 673	679 673
05/03/93	6.73 23 <0000>	6.68 20 <2100>	6.70	21	0.28W	-0.16	672 670	672 670	671 670	671 670	671 670	671 670	670 670	670 669	670 668	670 669	670 668	670 669
05/04/93	6.68 20 <0000>	6.59 15 <2300>	6.64	17	0.28W	-0.17	668 663	667 662	667 662	667 662	666 662	666 662	666 662	666 662	665 660	664 660	663 659	663 659
05/05/93	23.92 6900 <2400>	6.59 15 <0000>	11.94	1620	0.12W	-0.17	659 996	659 1040	659 1199	659 1474	661 1609	665 1720	675 1842	733 1961	848 2065	764 2172	819 2280	963 2392
MAX DISCHARGE 05/06/93	27.04 * 10200 <0400>	12.02 1190 <2400>	18.61	4480	0.00W	-0.19	2499 1557	2597 1468	2672 1406	2704 1358	2690 1320	2620 1292	2506 1269	2352 1247	2176 1235	1982 1222	1811 1210	1670 1202
05/07/93	12.02 1190 <0000>	8.88 370 <2400>	10.66	806	0.01W	-0.23	1190 1071	1183 1063	1173 1054	1159 1043	1148 1026	1134 1005	1123 984	1113 961	1101 941	1091 921	1085 904	1077 889
05/08/93	8.88 370 <0000>	7.75 154 <2400>	8.18	240	0.17W	-0.20	876 813	865 809	858 803	851 799	846 793	840 789	836 785	831 784	828 781	825 781	820 778	817 775
05/09/93	7.75 154 <0000>	7.29 73 <1900>	7.45	100	0.22W	-0.16	773 739	770 736	767 734	764 732	762 732	759 730	756 729	752 729	749 730	746 731	745 731	741 729
05/10/93	7.29 73 <0000>	7.16 56 <2300>	7.20	62	0.22W	-0.12	726 721	724 720	723 720	722 719	721 719	721 720	721 719	721 718	721 717	720 717	720 716	721 716
05/11/93	7.16 56 <0000>	7.11 50 <2400>	7.13	52	0.22W	-0.09	716 713	716 712	715 712	714 712	714 712	713 712	714 712	714 713	714 712	713 712	713 712	713 711
05/12/93	7.12 51 <0200>	7.07 46 <2200>	7.10	49	0.22W	-0.05	711 711	712 711	712 711	711 711	711 710	711 710	710 710	711 709	711 708	711 707	711 707	711 707

16/28

* STAGE VERIFIED BY EX SEEPLINE, 27.18 FT

Sensor # 2621 - 3002 E. Southcross		
5/5/93	1638	35.83
5/5/93	1616	35.67
5/5/93	1523	35.59
5/5/93	1507	35.47
5/5/93	1459	35.39
5/5/93	1447	35.16
5/5/93	1445	35.08
5/5/93	1439	35
5/5/93	1432	34.8
5/5/93	1431	34.72
5/5/93	1427	34.45
5/5/93	1418	34.17
5/5/93	1410	33.94
5/5/93	1410	33.9
5/5/93	1407	33.78
5/5/93	1405	33.74
5/5/93	1359	33.66
5/5/93	1357	33.58
5/5/93	1352	33.46
5/5/93	1156	33.35
5/5/93	1156	33.31
5/5/93	1108	33.23
5/5/93	1107	33.19
5/5/93	1103	33.11
5/5/93	1103	33.07
5/5/93	1102	33.03
5/5/93	1101	32.99
5/5/93	1101	32.95
5/5/93	1054	32.91
5/5/93	1048	32.83
5/5/93	1046	32.8
5/5/93	1042	32.68
5/5/93	1041	32.6
5/5/93	1007	31.89
5/5/93	951	31.73
5/5/93	949	31.61
5/5/93	948	31.57
5/5/93	736	31.06
5/5/93	734	30.98
5/5/93	734	30.94
5/5/93	727	30.35
5/5/93	726	30.31
5/5/93	720	29.88
5/5/93	433	29.45
5/5/93	323	29.25
5/4/93	1523	29.25

NWS FORM E-5
(7-76)
(PRES. BY WSOM E-41)

U.S. DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
NATIONAL WEATHER SERVICE

MONTHLY REPORT OF RIVER AND FLOOD CONDITIONS

RIVER DISTRICT OFFICE: SAN ANTONIO WSFO

REPORT FOR:

MONTH: MAY | YEAR: 1993

SIGNATURE: John Patton
In Charge of River District

DATE: JUNE 1, 1993

TO: Hydrologic Services Division, W22
National Weather Service
National Oceanic and Atmospheric Administration
Silver Spring, Maryland 20910

When no flooding occurs, include miscellaneous river conditions, such as significant rises, record low stages, ice conditions, snow cover, droughts, and hydrologic products issued (WSOM E-41).

☐ No flood stages were reached in this river district for the month indicated above.

IF THERE WEREN'T ENOUGH HEAVY RAINFALL PRODUCING SYSTEMS THAT IMPACT SOUTH TEXAS, (TRAIN EFFECT, CORE RAINS, UPPER LOWS AHEAD OF AND MOVING INTO LOWER BAROCLINIC ZONES, MERGING OUTFLOWS) WE MAY HAVE SEEN A DIFFERENT SORT OF BEAST.

THE EVENING OF MAY 4, 1993, SATELLITE IMAGERY INDICATED AN AREA OF CLOUD ENHANCEMENT MOVING NORTHEAST OUT OF THE SIERRA MADRE MOUNTAIN RANGE SOUTHWEST OF EAGLE PASS - NOT AN UNUSUAL PHENOMENON. THE AREA OF ENHANCEMENT SEEMED TO BE AN UPPER LOW MOVING AROUND A LONG WAVE TROUGH OVER THE SOUTHWESTERN UNITED STATES AND UP INTO THE GREAT BASIN.

BETWEEN MIDNIGHT AND TWO AM OF MAY 5, RADAR RETURNS INDICATED A VERY SMALL BUT INTENSE RAINFALL AREA NORTHEAST OF CHRYSTAL CITY. POLLING OBSERVERS IN THE AREA TURNED UP RAINFALL AMOUNTS OF .50 TO 1.00 INCHES ALTHOUGH THERE WERE NO OBSERVERS IN THE EXACT AREA - APPROXIMATELY 8 MILES NE OF CHRYSTAL CITY. THERE WERE PROBABLY 4 TO 5 INCHES IN A VERY TIGHT CENTER INDICATED BY THE VIP 5 AND 6 RADAR IMAGERY.

A LOW LEVEL JET ORIENTED FROM BROWNSVILLE NORTH INTO CENTRAL TEXAS WITH 850 MILLIBAR WINDS OF 40 TO 45 KNOTS WAS PROVIDING WARM MOIST LOW LEVEL ADVECTION OVER SOUTH TEXAS.

BY MID MORNING (7 TO 10 AM) THIS "DISTURBANCE" THAT MOVED OUT OF MEXICO HAD FORMED A CLOSED LOW ALOFT IN THE NORTH EDGE OF SAN ANTONIO WITH TWO SPIRAL BANDS (IN THE CONFIGURATION OF A HURRICANE) STREAMING OUT OF IT, LOOPING EAST, SOUTH, AND SOUTHWESTWARD.

THE LOW AT FIRST DEFINITELY WAS COLD CORE, BUT APPEARED TO "METAMORPHISIZE" INTO WARM CORE OR TROPICAL. THE VERY INTENSE RAINFALL RELEASED TREMENDOUS LATENT HEAT INTO THE ENVIRONMENT, INCREASING THE THICKNESS SIGNIFICANTLY, AND KICKING OUT THE SPIRAL BANDS.

THE HEAVIEST MORNING RAINFALL WAS IN THE SOUTH AND EASTERN PART OF BEXAR COUNTY AND IN WESTERN WILSON AND GUADALUPE COUNTIES BETWEEN FLORESVILLE AND SEGUIN. THE AREA IN AND WEST OF SEGUIN REPORTED UP TO 3.5 INCHES OF RAIN IN ABOUT TWO HOURS, (830-1030 AM) AND THE ABOVE BEXAR COUNTY AREA HAD UP TO OVER FOUR INCHES IN THE SAME TIME PERIOD. THIS WAS THE RAINFALL THAT FATALLY WASHED AN ELDERLY GENTLEMAN INTO THE SAN ANTONIO RIVER DRAINAGE FROM BROADWAY STREET NEAR AUSTIN HIGHWAY.

THIS WAS RAINFALL FROM THE INNER "SPIRAL BAND" (SEE SKETCH) WHICH WAS ORIENTED NNE BY SSW OVER THE AREA. RAINFALL DECREASED OVER BEXAR COUNTY UNTIL ANOTHER SEIGE OF INTENSE RAINFALL BEGINNING SHORTLY AFTER NOON IN NORTHERN BEXAR COUNTY. THE UPPER OLMOS AND SALADO CREEK DRAINAGES ABOVE LOOP 410 RECEIVED OVER 4 INCHES BETWEEN 1 AND 4 PM, RAISING TOTALS TO OVER 8 INCHES IN AN EAST WEST BAND JUST NORTH OF LOOP 410.

THIS SECOND, EARLY AFTERNOON SEIGE SEEMED TO BE PRODUCED BY THE CONVECTIVE BAND INTENSIFYING OVER THE AREA AND THEN THE EAST-WEST PORTION OF THE BAND MOVING SOUTH TO NORTH ACROSS NORTHERN BEXAR COUNTY DURING MID AFTERNOON.

BY 2 PM, THE LOWER GUADALUPE RIVER DRAINAGE NEAR AND BELOW CUERO TO BELOW VICTORIA BEGAN RECEIVING VERY INTENSE RAINFALL. VICTORIA WSO RECEIVED 4.21 INCHES BETWEEN 2 AND 5 PM CDT, AND ANOTHER 3.09 INCHES IN THE NEXT 6 HOURS -

FOR A 9 HOUR STORM TOTAL OF 7.30 INCHES. THE 24 HOUR TOTAL WAS 7.65 INCHES.

THE GUADALUPE RIVER AT GONZALES ROSE TO 23.4 FEET THE NEXT DAY (5/6), FS 20' AND VICTORIA SAW A FIRST CREST OF 25.2 FEET (FS 21') AT NOON ON THE 6TH - MODERATE LOWLAND FLOODING.

THE MOST SEVERE FLOODING WAS IN METROPOLITAN SAN ANTONIO. SALADO AND MEDIO CREEKS SAW RECORD LEVELS. SALADO CREEK AT NE LOOP 410 CRESTED AT 15.49 FEET NEAR 8 PM 5/5/93 - PREVIOUS RECORD - 15.22 FEET MAY 12, 1972. THE USGS ESTABLISHED THEIR READING FROM A HIGH WATER MARK AS THE MANOMETER WAS DEFECTIVE. THERE IS A LARGE RECREATIONAL VEHICLE CAMP ABOUT SIX MILES DOWNSTREAM WHICH WAS OVER HALF UNDERWATER. ALL THE TRAILERS (SOME PERMANENT) WERE EVACUATED SUCCESFULLY.

THE LOS PATIOS SHOPPING CENTER JUST ABOVE NE LOOP 410 ON SALADO CREEK HAD A GREENHOUSE FLOODED AND ALL THEIR PLANTS WASHED DOWNSTREAM.

IN ADDITION TO THE ABOVE GENTLEMAN WHO DROWNED IN THE SAN ANTONIO RIVER, AN ELDERLY COUPLE ALSO WASHED DOWNSTREAM IN THEIR CAR WHERE ELM CREEK (A TRIBUTARY OF OLMOS CREEK) CROSSES VERY HEAVILY TRAVELED LOCKHILL-SELMA STREET. AFTER THEY WERE LOST, WATER QUICKLY ROSE TO OVER 7 FEET OF VERY HIGH VELOCITY WATER. SEVERAL HOMES WERE FLOODED ALONG OLMOS CREEK AND TRIBUTARIES FOR THE SECOND TIME IN TWO YEARS. ON APRIL 4TH, 1991 9.25" FELL IN 2 HOURS IN THE SAME AREA.

OLMOS CREEK IS EXTREMELY DANGEROUS FOR TRAFFIC IN MAJOR FLOODS SUCH AS THIS. DREAMLAND STREET HAD SIX FEET OF WATER OVER THE LOW WATER CROSSING OF OLMOS CREEK. THERE ARE SEVERAL CROSSINGS WHERE CARS CROSS THE CREEK, WHICH CAN RISE VERY RAPIDLY. IF YOUR CAR STALLS AS THE WATER IS RISING, YOU'RE DEAD.

THE HEB SUPERMARKET IN THE SOUTHEAST QUADRANT OF WEST AVENUE AND LOOP 410 IN THE OLMOS CREEK FLOOD PLAIN HAD 1 TO 2 INCHES OF WATER THROUGH IT, (2' IN THE 91 FLOOD).

THE McALLISTER FREEWAY ABOVE OLMOS DAM HAD 3 OF 4 LANES EACH WAY CLOSED AS OLMOS CREEK PUT 33 FEET OF WATER BEHIND THE RETENTION DAM. THE FREEWAY FLOODS AT A POOL ELEVATION OF 713 FEET - DATUM 680' AND THE OBSERVED USGS READING WAS 713.09'.

JUST BELOW OLMOS DAM, THE SOUTHWESTERN BELL PARKING LOT JUST SOUTH OF INCARNATE WORD COLLEGE HAD UP TO 4 FEET OF WATER IN THE PARKING LOT AND 4 CARS COULD NOT BE MOVED QUICKLY ENOUGH AND WERE FLOODED. THE SAN ANTONIO RIVER DID NOT DIRECTLY FLOOD THE LOT (CREST 8.5', BF 9'). BUT THE RIVER WAS HIGH ENOUGH TO BACK UP THE PARKING LOT RUNOFF DRAINS AND PREVENT THE RUNOFF FROM ESCAPING. THE HIGH WATER AND FLOODING CAME OUT OF INCARNATE WORD COLLEGE AND THE SOUTHWESTERN BELL PARKING LOT ITSELF BECAUSE IT COULDN'T RUN OFF. MUCH OF THE PICNIC AREA OF BRACKENRIDGE PARK FLOODED BUT INCARNATE WORD COLLEGE DID NOT.

LEON CREEK ROSE TO JUST OVER 13 FEET AND FLOODED WESTOVER DRIVE ACROSS KELLY AFB.

MEDIO CREEK ALSO RECEIVED A FLOOD OF RECORD; 10.25 FEET, PREVIOUS RECORD

10.09 FEET JUN 13, 1987. THE PERIOD OF RECORD IS ONLY DEC 1986 TO CURRENT.

THERE IS ALSO A RECREATION CAMPGROUND AT THE LOWER END OF MEDIO CREEK WHERE IT IS CONFLUENT WITH THE MEDINA RIVER. THE GROCERY STORE HAD WATER JUST TO THEIR SLAB, BUT DIDN'T ACTUALLY FLOOD. ALL THEIR CAMPING AREA FLOODED AND THEY LOST SOME PICNIC TABLES DOWNSTREAM. THE MEDINA RIVER WAS ONLY 7.4 FEET AT THE TIME. WHEN THE MEDINA RIVER IS OVER 20 FEET, IT TAKES VERY LITTLE RUNOFF DOWN MEDIO CREEK TO FLOOD THIS AREA.

A SECOND SERIOUS FLOOD DEVELOPED DURING THE EARLY DAYLIGHT HOURS OF MAY 23RD (SEE SKETCHES). A LINE OF THUNDERSHOWERS ORIENTED SOUTHWEST TO NORTHEAST MOVED THROUGH BEXAR COUNTY BETWEEN 5 AND 6 AM VERY RAPIDLY, DROPPING GENERALLY .50 TO 1.00 INCHES OF RAIN. THIS SYSTEM WAS ASSOCIATED WITH AN UPPER LOW MOVING DOWN THE BACK SIDE OF A LONG WAVE MOVING THROUGH TEXAS. AS THIS UPPER LOW REACHED THE BOTTOM OF THE TROUGH (GEOGRAPHICALLY OVER THE AREA JUST SOUTH OF SAN ANTONIO IN ATASCOSA, WILSON, AND GONZALES COUNTIES) IT STALLED AND BEGAN RAINING VERY HEAVILY.

AS A SECOND SHORT WAVE WHICH HAD SPLIT FROM THE FIRST NEAR BIG BEND PARK AGAIN CAUGHT THE FIRST, A CONVECTIVE BOUNDARY ON THE LEADING EDGE OF IT RODE AROUND THE SOUTHERN PERIPHERY OF THE FIRST ONE AND MERGED OVER FLORESVILLE, PROLINGING AND INTENSIFYING THE RAINFALL. HONDO RADAR WENT DOWN TWICE DURING THE EXPLOSIVE PERIOD SO IT IS HARD TO KNOW THE EXACT DURATION. THE BEST ESTIMATE IS BETWEEN 7 AM AND NOON. SOME GAUGES IN AND NEAR FLORESVILLE HAD UP TO 8 INCHES OF RAIN. BETWEEN 10 AM AND NOON SEVERAL HOMES IN FLORESVILLE FLOODED FROM THE LOCAL RUNOFF, NOT FROM THE SAN ANTONIO RIVER. THE OFFICIAL COOP OBSERVER (CITY OF FLORESVILLE) REPORTED 7.55.
....SEE THE ISOHYETAL...

INTENSITIES IN SOME THUNDERSTORMS WERE UNDOUBTEDLY OVER 5 INCHES PER HOUR FOR SHORT DURATIONS. THE SAN ANTONIO NEWSPAPER REPORTED AN UNOFFICIAL 11.75 INCH OBSERVATION EAST OF FLORESVILLE BY A FARMER. THE ISOHYETAL PATTERN AND DIURNAL TIMING ARE VERY SIMILAR TO THE FLOOD OF NOVEMBER OF LAST YEAR (1992), A 10.50 INCH CENTER IN THE SAME AREA DURING THE SAME DIURNAL PERIOD.

THE HEAVY RAINFALL EXTENDED IN A BAND FROM PLEASANTON (3.73 INCHES) TO GONZALES (3.33 INCHES). AS THE UPPER LOW KICKED OUT OF THE AREA SOUTH AND EAST OF SAN ANTONIO IN THE EARLY AFTERNOON, IT ALSO RAINED AT RATES OVER TWO INCHES PER HOUR FOR ONE TO TWO HOURS IN THE LAVACA AND NAVIDAD RIVER DRAINAGES AND SANDIES CREEK WHICH FLOWS FROM THE WEST INTO THE GUADALUPE RIVER JUST ABOVE CUERO.

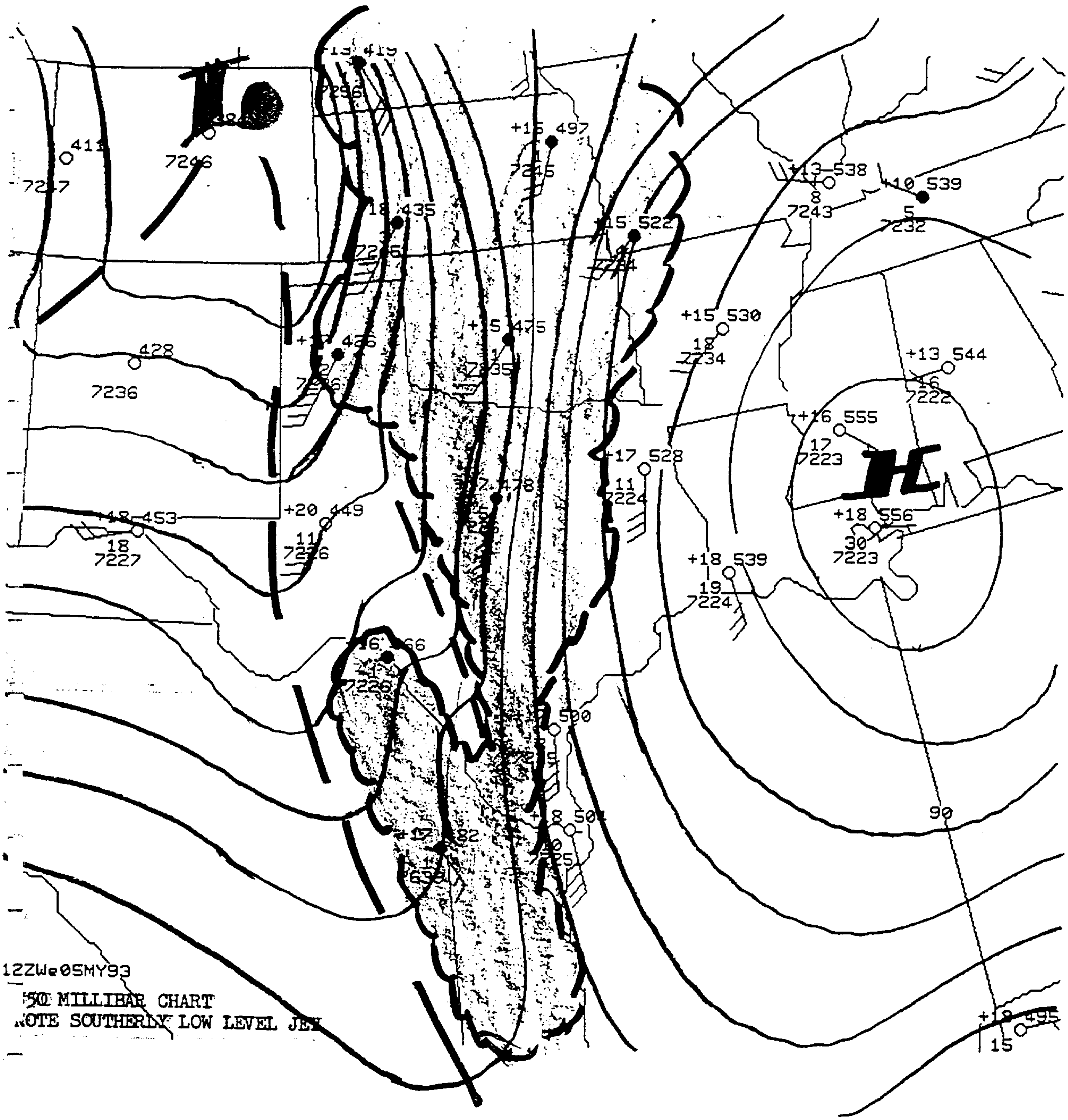
THE EL NINO-SOUTHERN OSCILLATION PHENOMENON OF 1990/1991 IS STILL WITH US AND THE SEA SURFACE TEMPERATURES FROM PERU WESTWARD TO HAWAII OVER THE EQUATORIAL REGION HAVE RISEN MARKEDLY SINCE NOVEMBER 1992. THIS IS THE LONGEST LIVED ENSO EVENT SINCE BEFORE THE PRE WORLD WAR 2 ERA OF 1939 INTO 1942 - 33 MONTHS. THIS ONE HAS LASTED 37 MONTHS.

SEA SURFACE TEMPERATURE RISES OVER THE "NINO 4" AREA FROM HAWAII TO THE WESTERN PACIFIC LAG A FEW MONTHS BEHIND THE PERU - HAWAII AREA BUT CORRELATE BEST WITH RAINFALL ANOMALIES OVER SOUTH TEXAS. THE NINO 4 AREA OBVIOUSLY IS HEATING UP ALTHOUGH DATA THROUGH APRIL 1993 DIDN'T INDICATE WARMING.

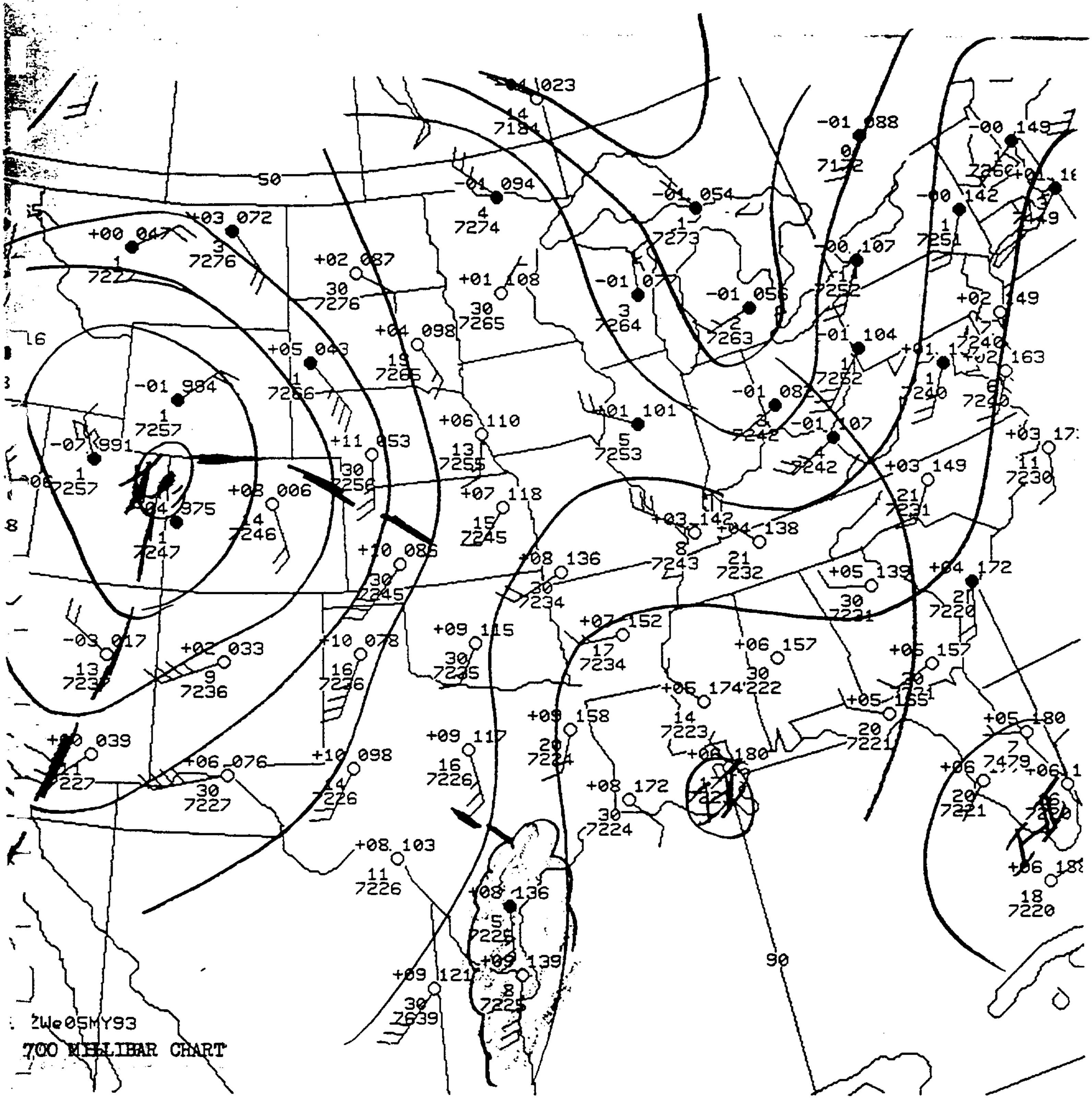
THIS ENSO EVENT WAS MANIFESTED PROMINENTLY IN MAY OVER SOUTH TEXAS. MAY 1993 WAS THE 6TH WETTEST ON RECORD IN CORPUS CHRISTI; SAN ANTONIO THE 3RD WETTEST WITH RECORDS GOING BACK TO 1885; AND VICTORIA SAW THE WETTEST MAY ON RECORD, WITH RECORDS BACK TO 1871. ALL THIS IN LIGHT OF MAY BEING THE WETTEST MONTH OF THE YEAR.

Rainfall percentages of normal at weather offices in South Texas:

station	CALENDAR YEAR 1993 RAINFALL TO DATE	JAN - MAY NORMAL RAINFALL	PERCENTAGE OF NORMAL	DEFICIT
DEL RIO	3.85 INCHES	6.21 INCHES	62 %	-2.36 INCHES
BROWNSVILLE	10.33"	7.65"	135%	+2.68"
CORPUS CHRISTI	16.33"	9.54"	171%	+6.79"
VICTORIA	28.71"	12.62"	227%	+16.09"
SAN ANTONIO	20.87"	11.76"	177%	+9.11"
AUSTIN	16.86"	13.09"	129%	+3.77"

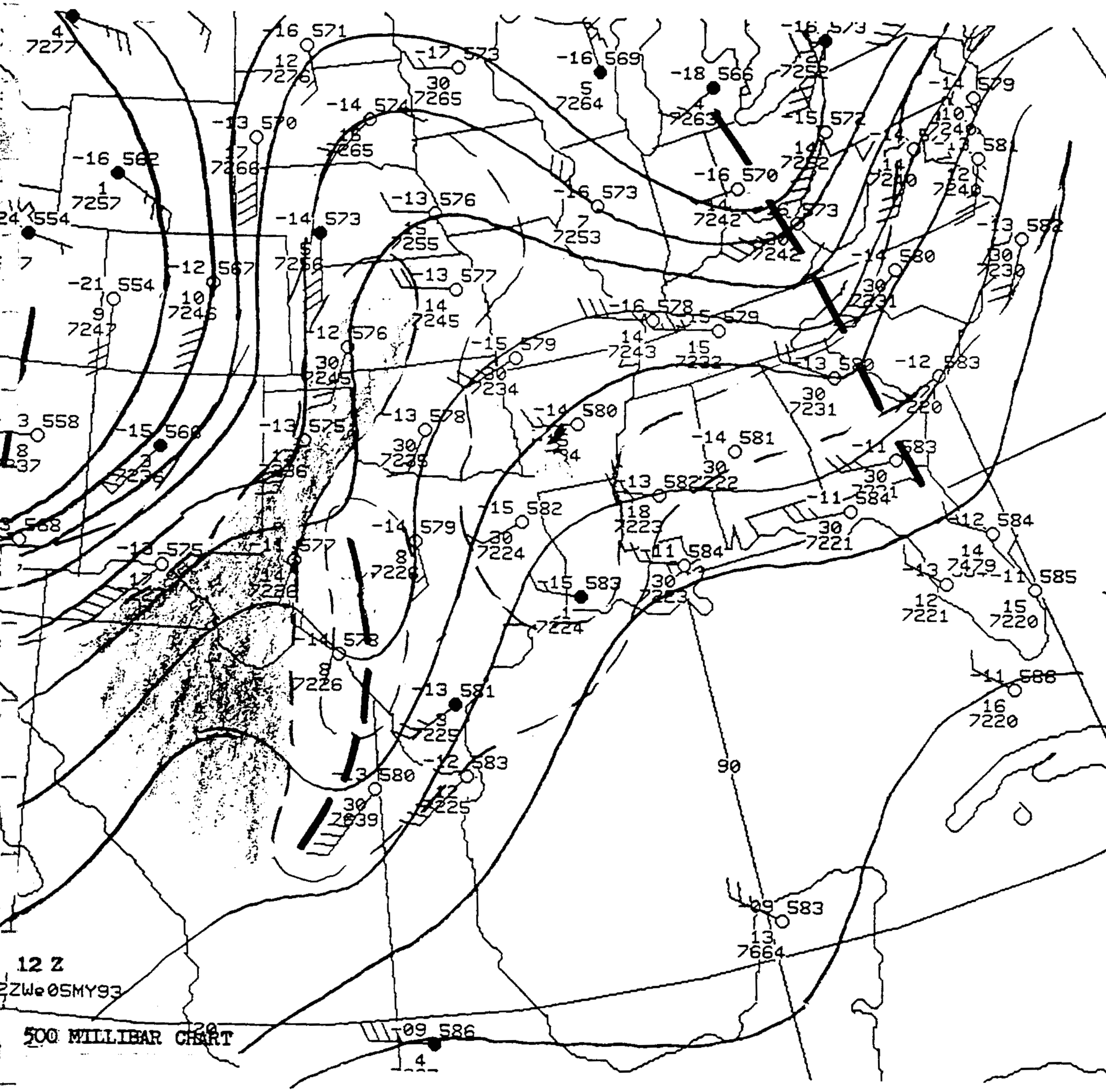


12ZWe05MY93
850 MILLIBAR CHART
NOTE SOUTHERLY LOW LEVEL JET

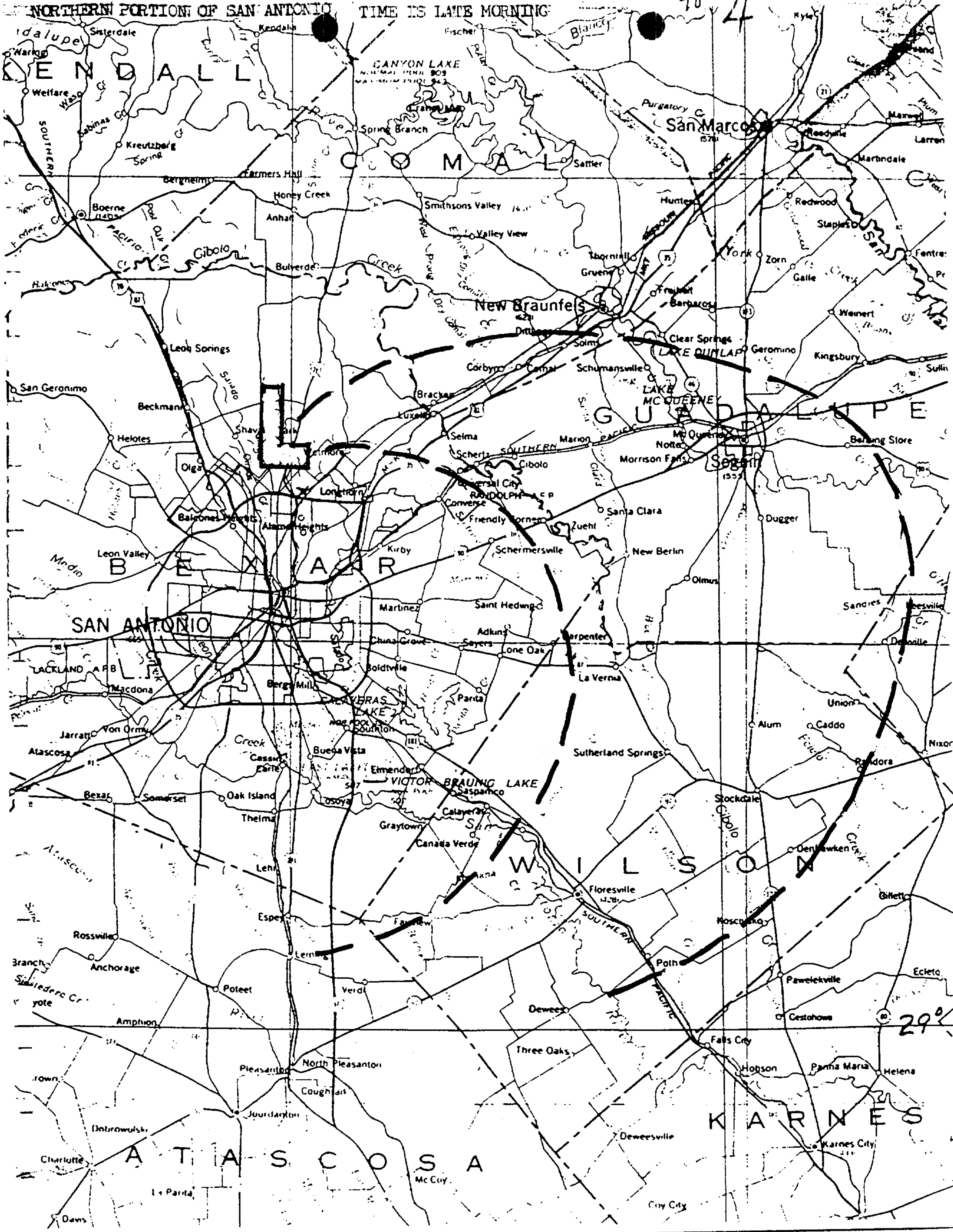


2We05MY93

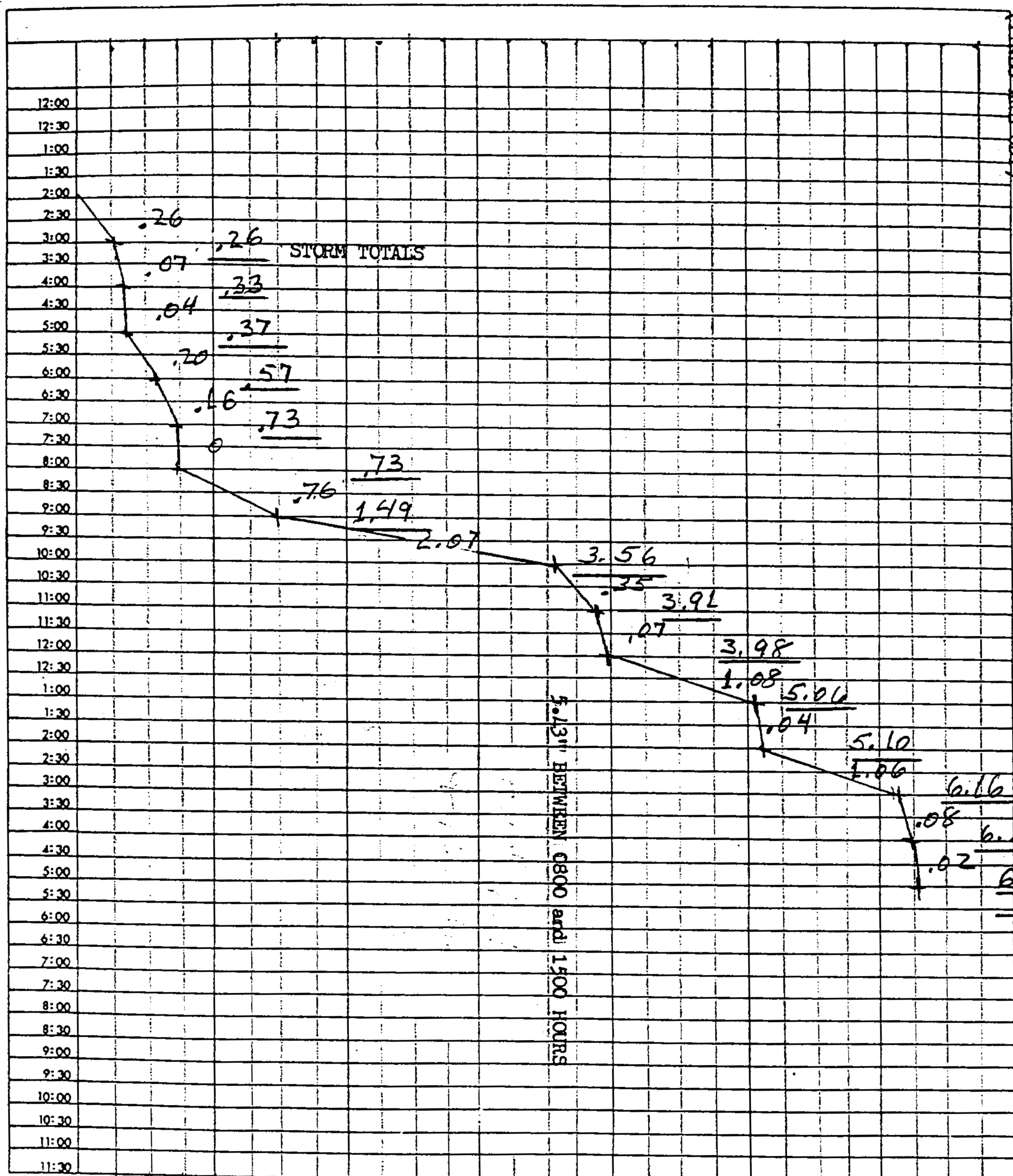
700 MILLIBAR CHART


12 Z
ZWe05MY93
500 MILLIBAR CHART

"SPIRAL BANDS" OR LINES OF CONVECTION WITH THE MAY 5, 1993 STORM – 8.00 to 8.5 INCH CENTER OVER NORTHERN PORTION OF SAN ANTONIO. TIME IS LATE MORNING



SAN ANTONIO WSFO
MASS CURVE FOR STORM OF MAY 5, 1993
(TIMES ARE CDT)



WS FORM E-3
(5-71)
(PRES. BY WSOM E-41)

U.S. DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
NATIONAL WEATHER SERVICE

RIVER DISTRICT OFFICE
SAN ANTONIO WSFO

FLOOD STAGE REPORT

REPORT FOR:

MONTH	YEAR
MAY	1993

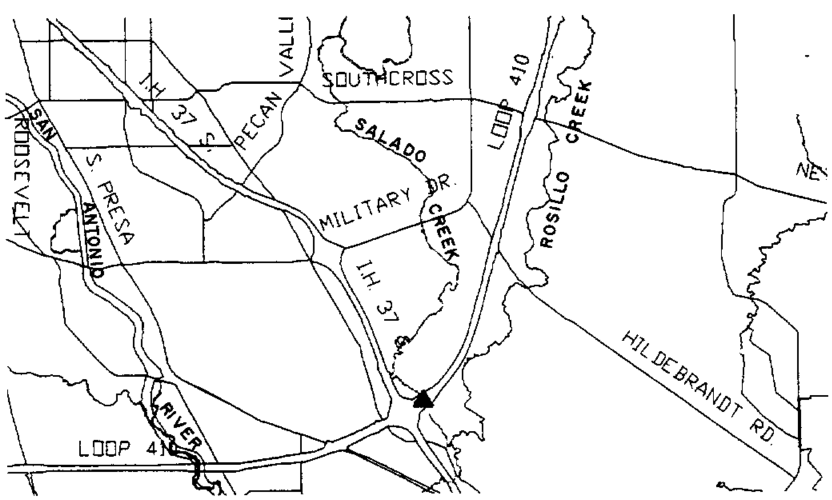
RIVER AND STATION	FLOOD STAGE (Feet)	ABOVE FLOOD STAGES (Dates) FROM	TO	CREST STAGE (Feet)	CREST DATE
NAVIDAD RIVER					
SUBLIME	BF 20	-	-	21.12	5/24 0400
LAVACA RIVER					
EDNA	21	5/25	5/26	24.0	5/25
GUADALUPE RIVER					
GONZALES	20	5/6	5/6	23.4	5/6 0900
CUERO	20	5/8	5/10	21.3	5/9
		5/24	5/28	27.10	5/27 1000
VICTORIA	21	5/6	5/12	25.2	5/6 1200
		5/24	5/28	27.10	5/27 1000
DUPONT	20	5/6	5/18	26.5	5/7
		5/24	EOM	26.5	5/7
CIBOLO CREEK	BF 6	-	-	19.8	5/5 2400
		-	-	26.05	5/26
SALADO CREEK					
NE LOOP 410 (UPR STN)	BF 12	-	-	15.49 (FOR)	5/5
LOOP 13 (LWR STN)	BF 16	-	-	27.22	5/6
OLMOS CREEK					
OLMOS DAM				713.05	5/5 1700
SAN ANTONIO RIVER					
@ LOOP 410	-	-	-	22.65	5/5
FALLS CITY	12	5/8	5/9	13.7	5/8 0600
		5/24	5/25	14.69	5/24 0600
GOLIAD	BF 23	-	-	28.2	5/10
		-	-	32.40	5/27 0600
LEON CREEK					
LOOP 13 (KELLY AFB)	13	5/5	5/5	13.2	5/5
MEDIO CREEK					
PEARSALL ROAD	BF 8	-	-	10.25(FOR)	5/5 2400
MEDINA RIVER					
SAN ANTONIO	BF 20	-	-	22.59	5/7
ATASCOSA RIVER					
WHITSETT	20	5/25	5/27	26.34	5/25 1600
NUECES RIVER					
THREE RIVERS	25	5/26	5/27	26.05	5/27 0400

WS FORM E-3 SUPERSEDES WB FORM E-3 WHICH MAY BE USED

* U.S. GOVERNMENT PRINTING OFFICE: 1978—765-092/1034

Appendix C

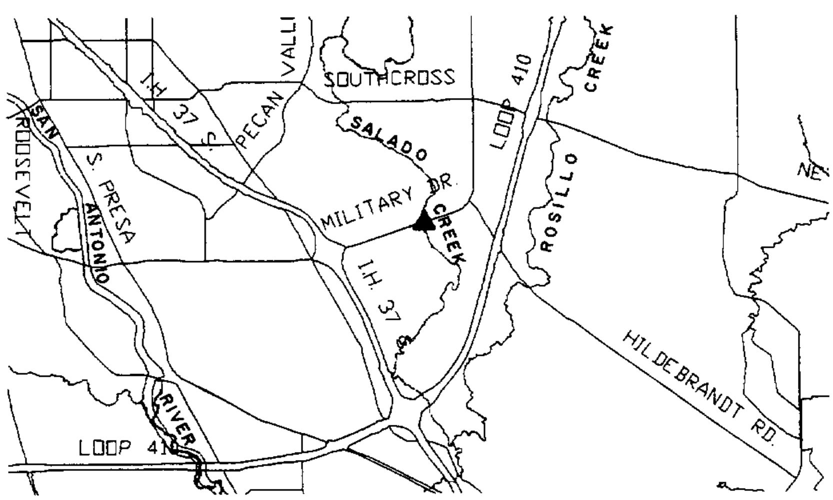
S.E. LOOP 410


SOUTHCROSS
LOOP 410
I.H. 37 S.
PECAN VALLI
SALADO CREEK
ROSILLO CREEK
MILITARY DR.
SAN ANTONIO RIVER
S. PRESA
ROOSEVELT
HILDEBRANDT RD.

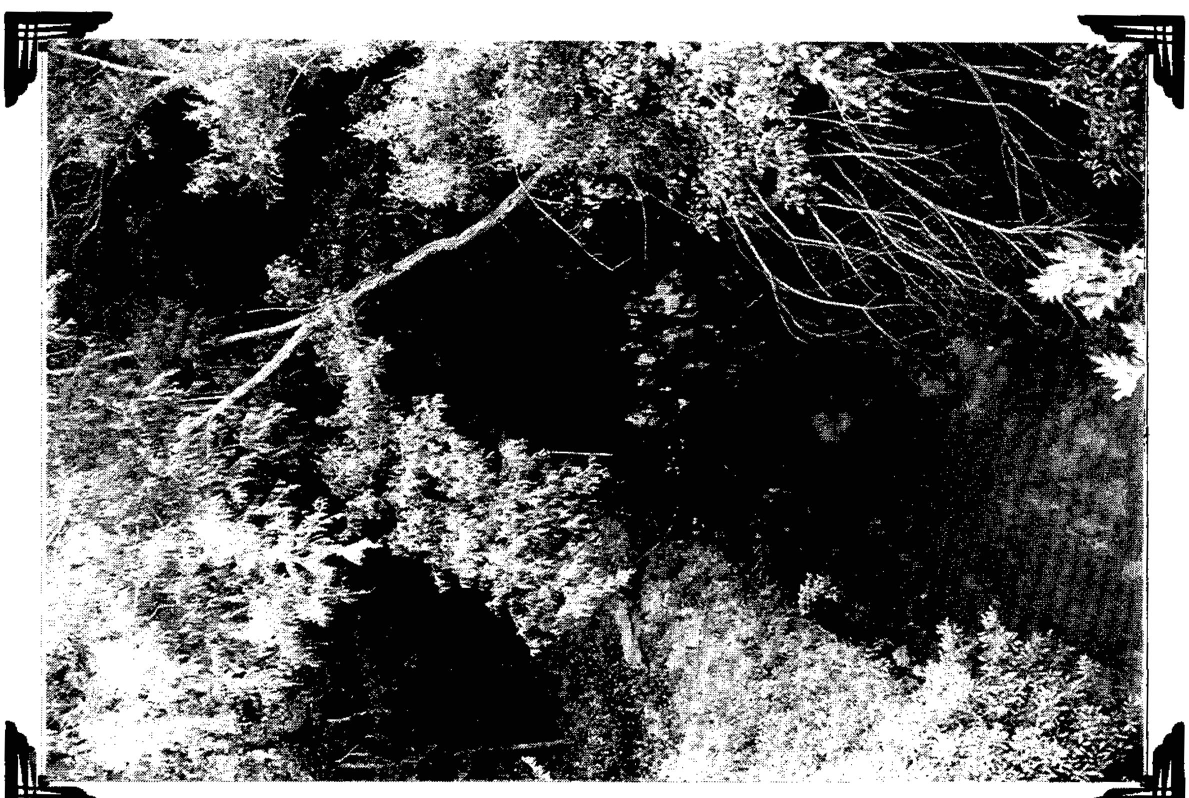
S.E. LOOP 410

UPSTREAM

S. E. MILITARY DRIVE

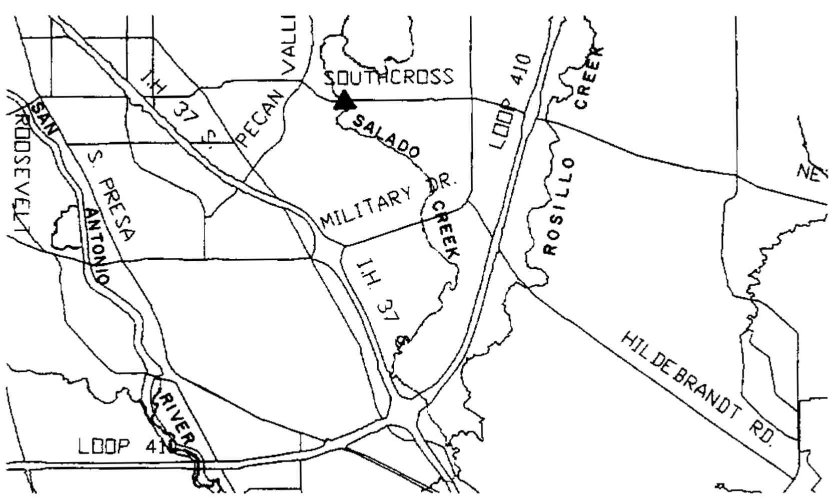


S.E. MILITARY DOWNSTREAM

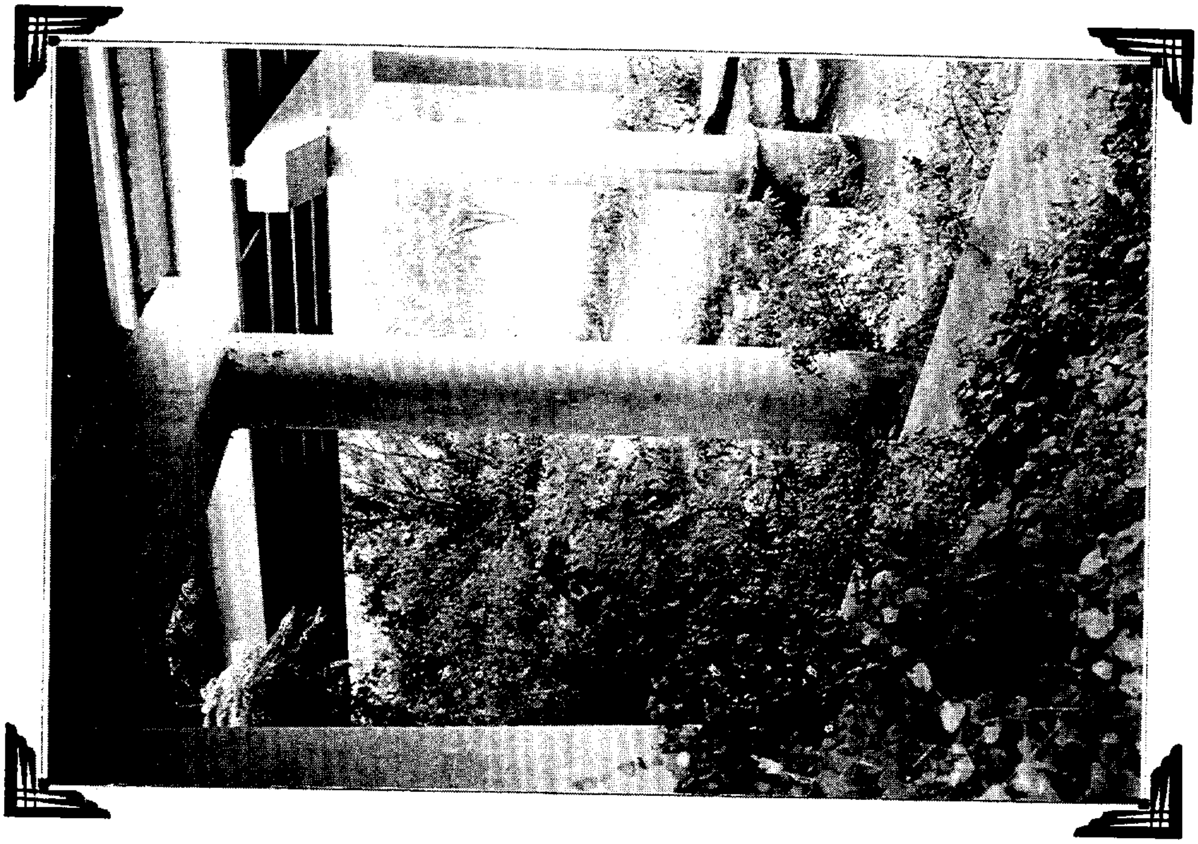


S.E. MILITARY UPSTREAM

SOUTHCROSS

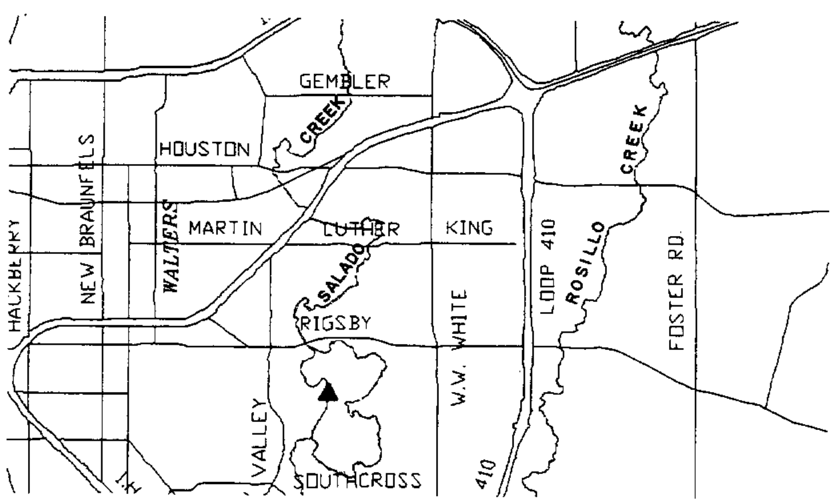


SOUTHCROSS
LOOP 410
CREEK
I.H. 37 S.
PECAN VALLI
SAN
ROOSEVELT
S. PRESA
SALADO
MILITARY DR.
CREEK
ROSILLO
ANTONIO
I.H. 37
HILDEBRANDT RD.
RIVER
LOOP 410

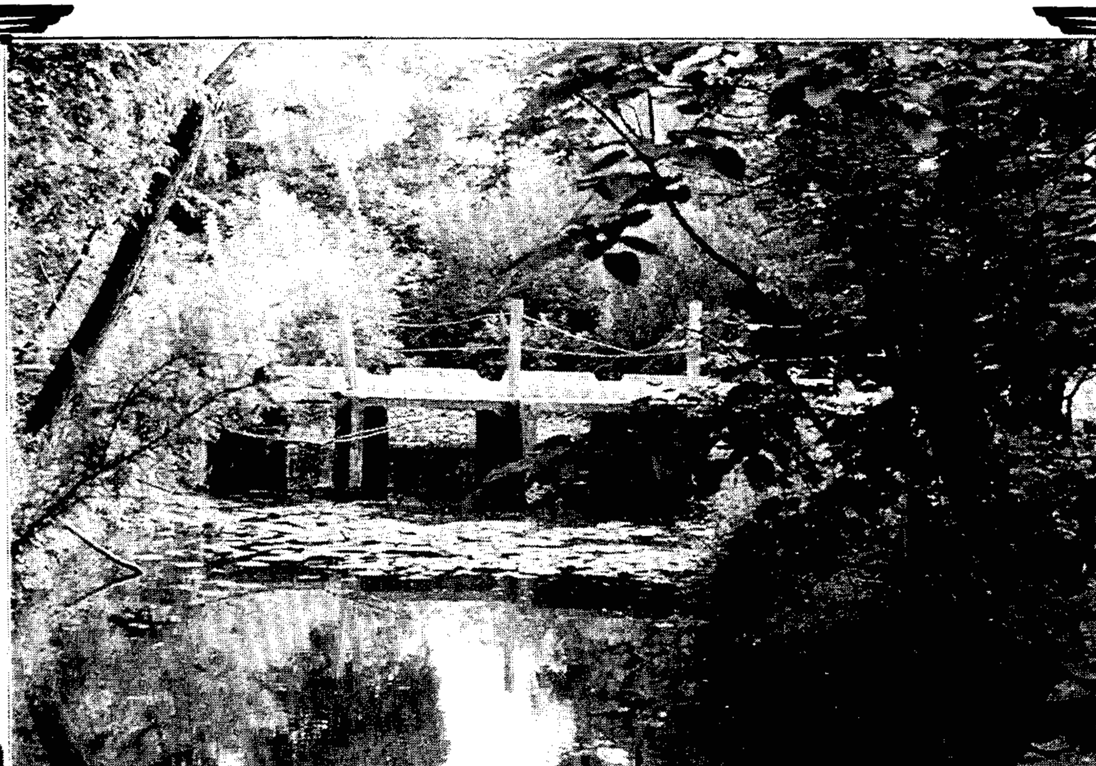


SOUTHCROSS UPSTREAM

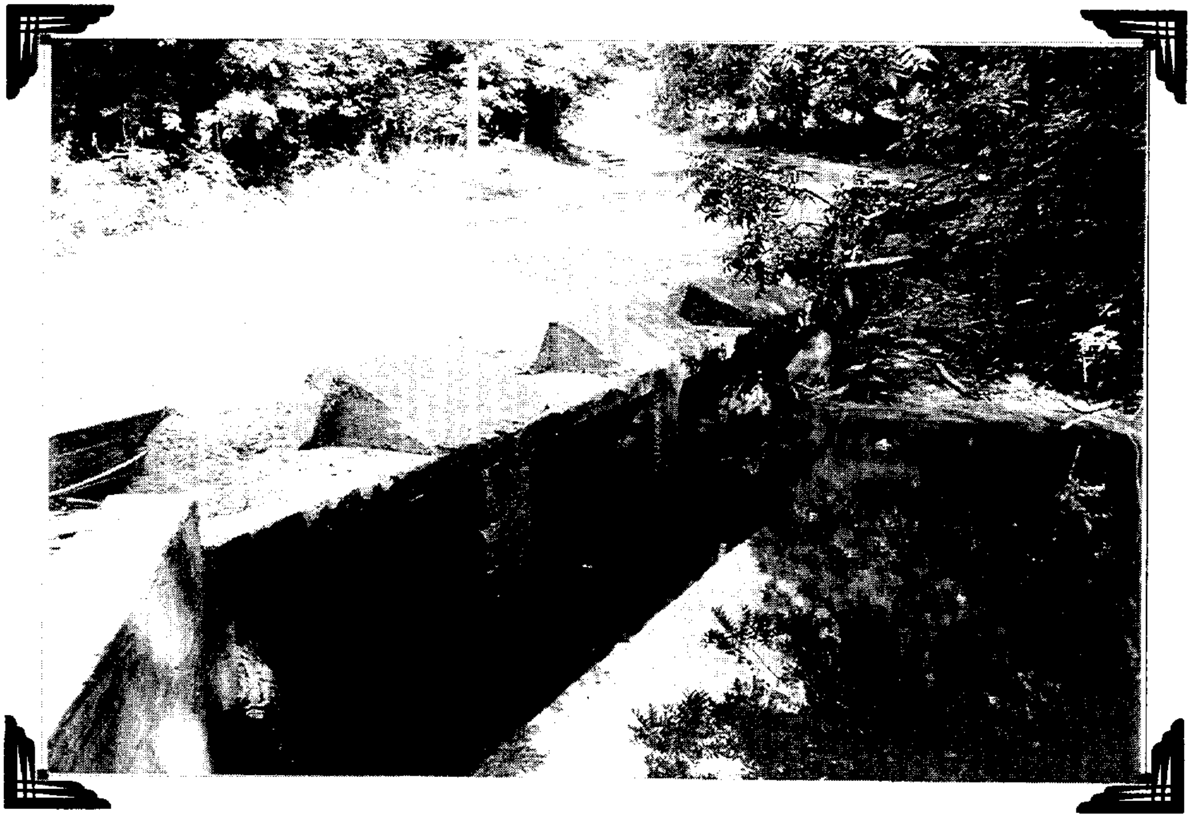
ROLAND ST.


GEMBLER
CREEK
HOUSTON
NEW BRAUNFELS
WALTERS
MARTIN
LUTHER
KING
HACKBERRY
SALADO
RIGSBY
W.W. WHITE
LOOP 410
ROSILLO
CREEK
FOSTER RD.
VALLEY
SOUTHCROSS
410

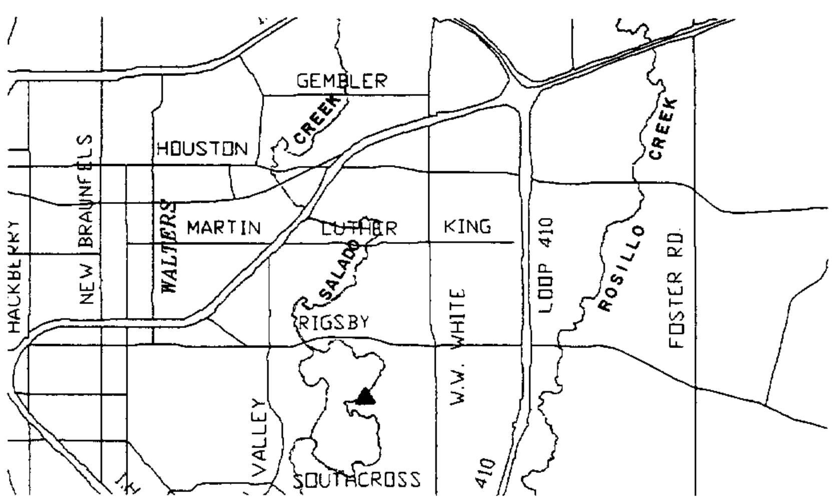
ROLAND ST. UPSTREAM (WEST CROSSING)



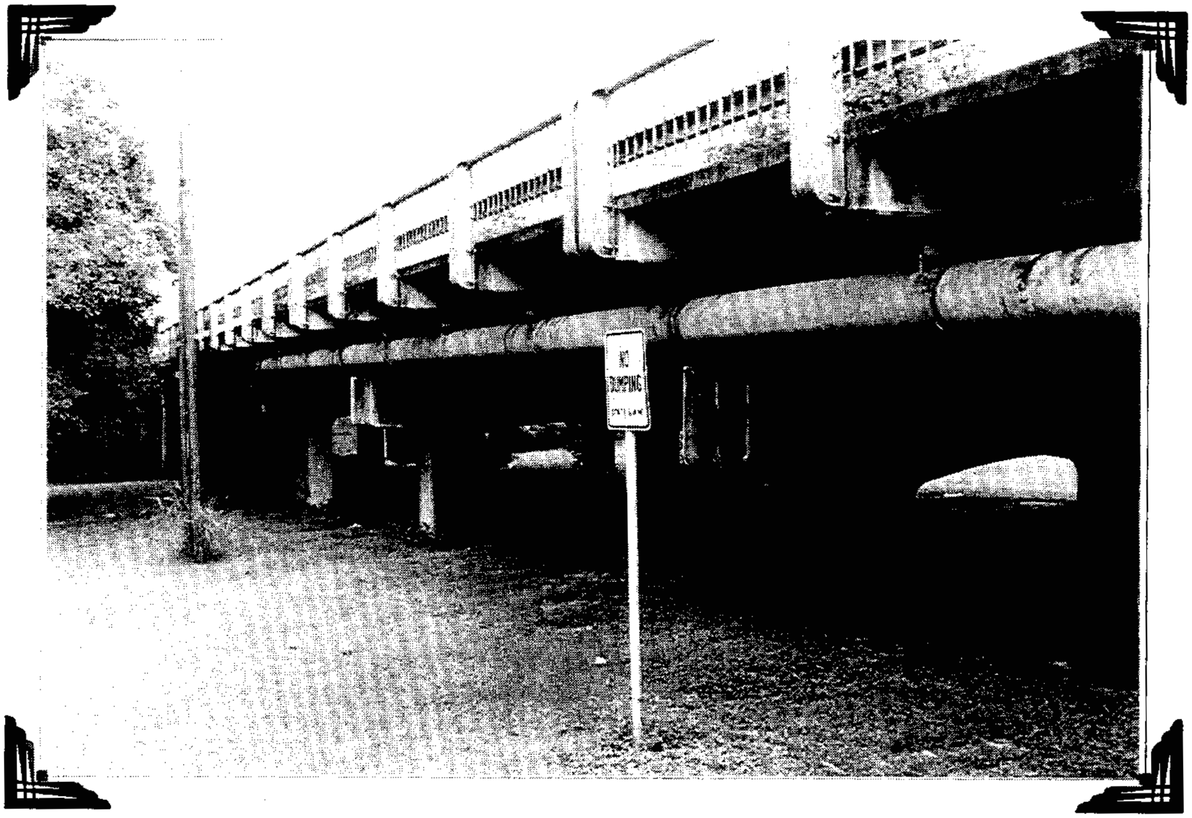
ROLAND ST. UPSTREAM (WEST CROSSING)



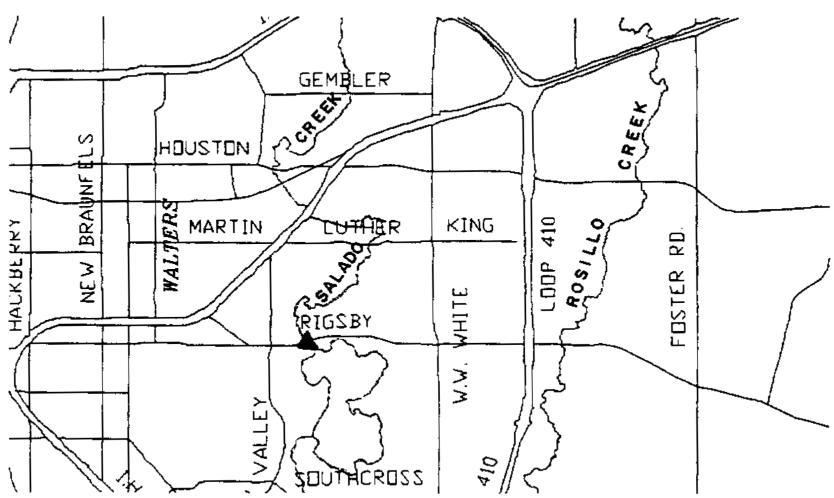
ROLAND ST.


GEMBLER
CREEK
HOUSTON
CREEK
NEW BRAUNFELS
WALTERS
MARTIN
LUTHER
KING
SALADO
HACKBERRY
RIGSBY
W.W. WHITE
LOOP 410
ROSILLO
FOSTER RD.
VALLEY
SOUTHCROSS
410

ROLAND ST. UPSTREAM (EAST CROSSING)



RIGSBY ST.



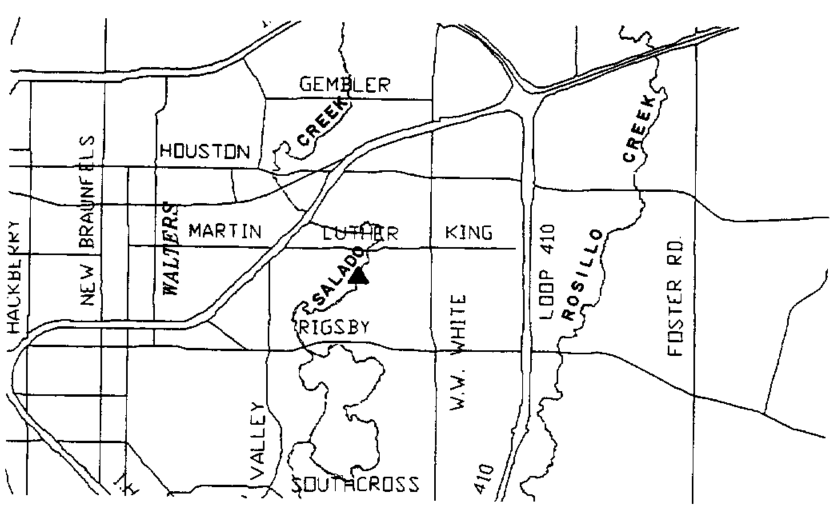
GEMBLER
CREEK
HOUSTON
HACKBERRY
NEW BRAUNFELS
WALTERS
MARTIN
LUTHER
KING
SALADO
RIGSBY
W.W. WHITE
LOOP 410
ROSILLO
CREEK
FOSTER RD.
VALLEY
SOUTHCROSS
410

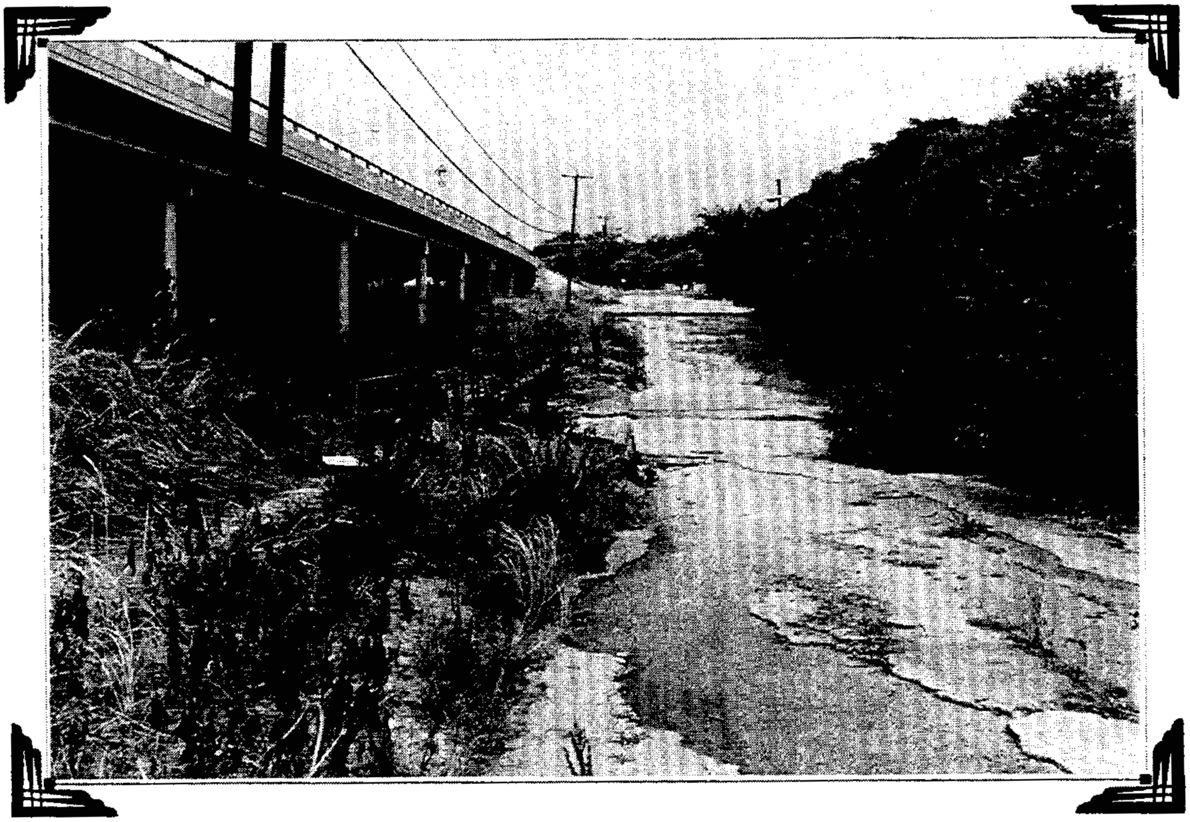
RIGSBY ST. UPSTREAM

RIGSBY ST. UPSTREAM

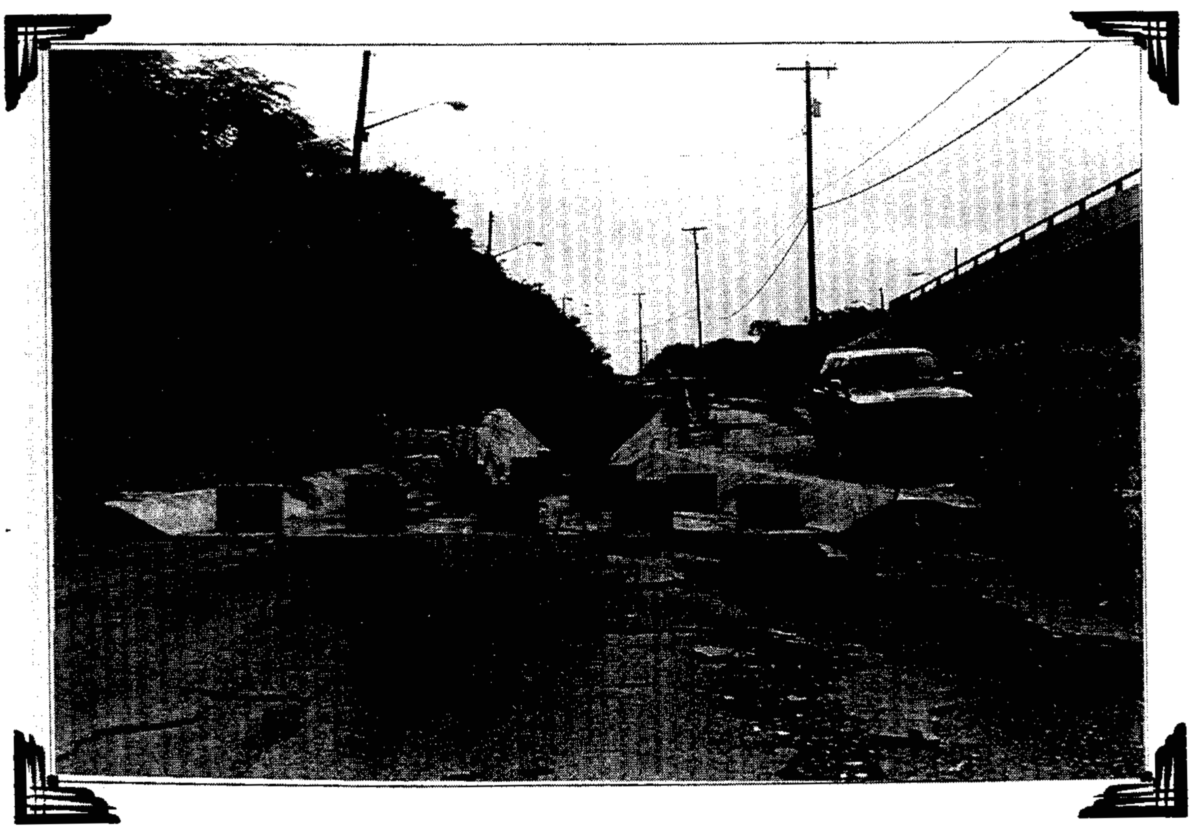
RIGSBY ST. DOWNSTREAM

RICE RD.

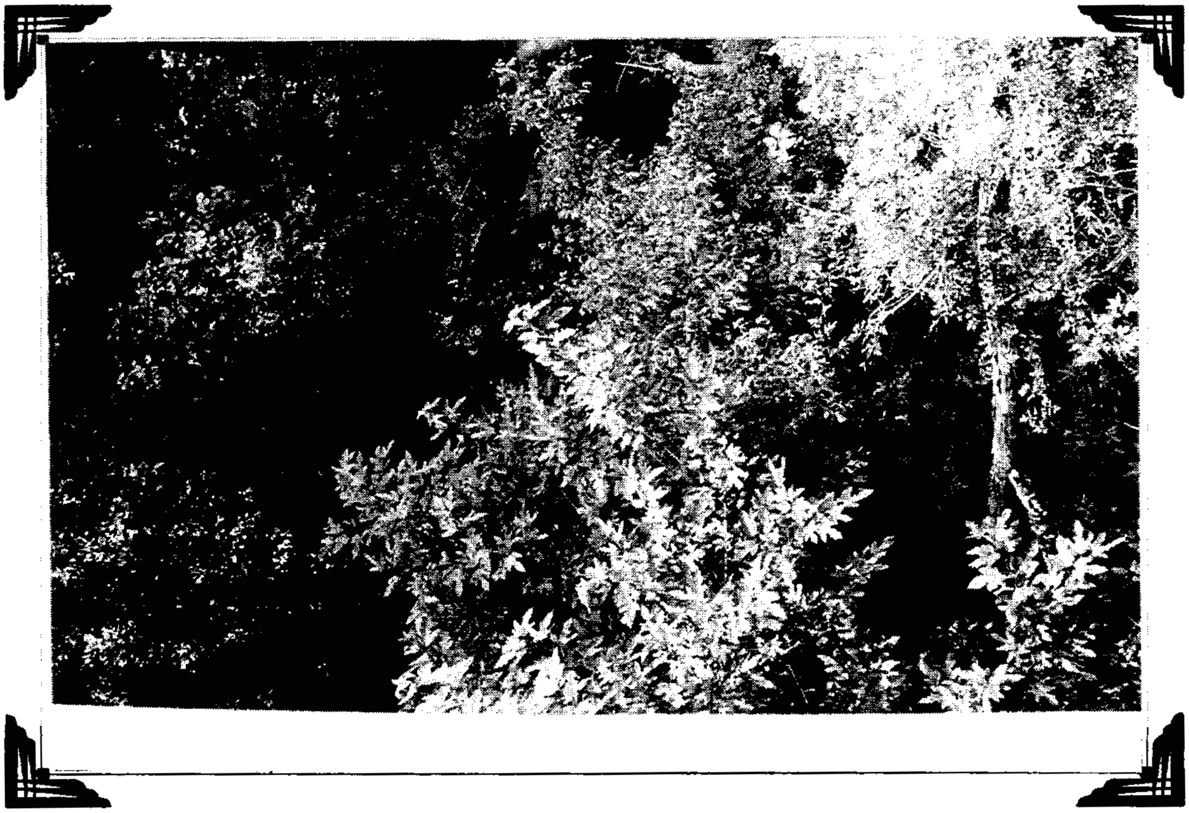

GEMBLER
CREEK
HOUSTON
NEW BRAUNFELS
WALTERS
MARTIN
LUTHER
KING
CREEK
LOOP 410
ROSILLO
FOSTER RD.
HACKBERRY
SALADO
RIGSBY
W.W. WHITE
VALLEY
SOUTHCROSS
410



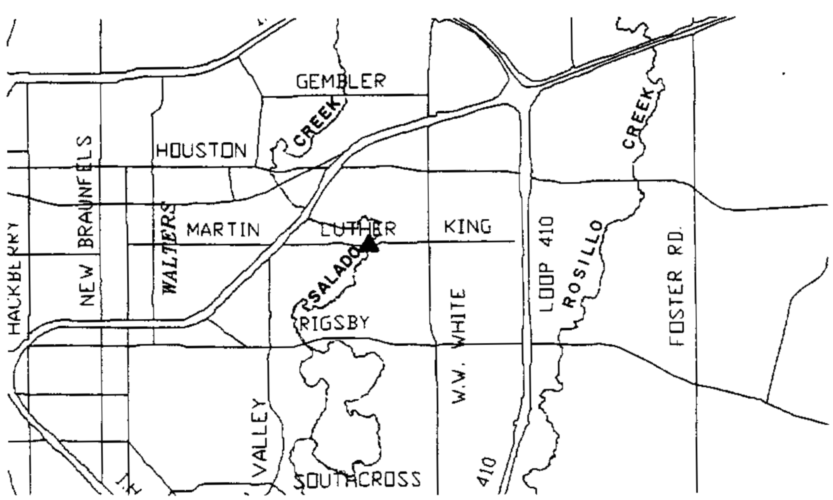
RICE RD. (ABANDONED)



RICE RD. (ABANDONED) EAST SIDE

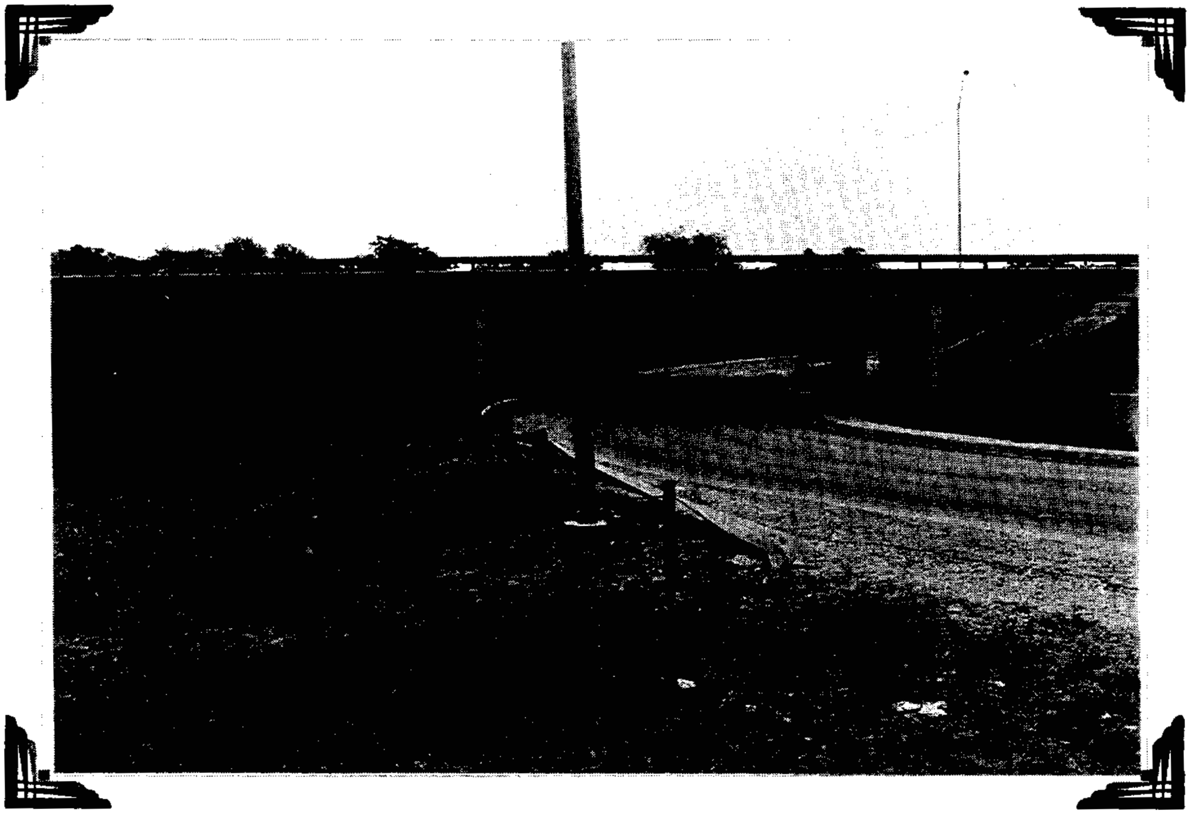


MARTIN LUTHER KING
UPSTREAM

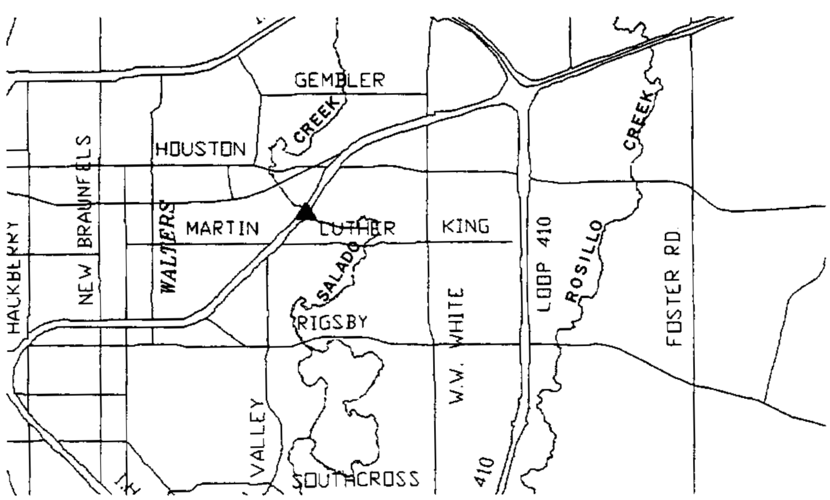


MARTIN LUTHER KING

DOWNSTREAM



I H 10


GEMBLER
CREEK
HOUSTON
NEW BRAUNFELS
HACKBERRY
WALTERS
MARTIN
LUTHER
KING
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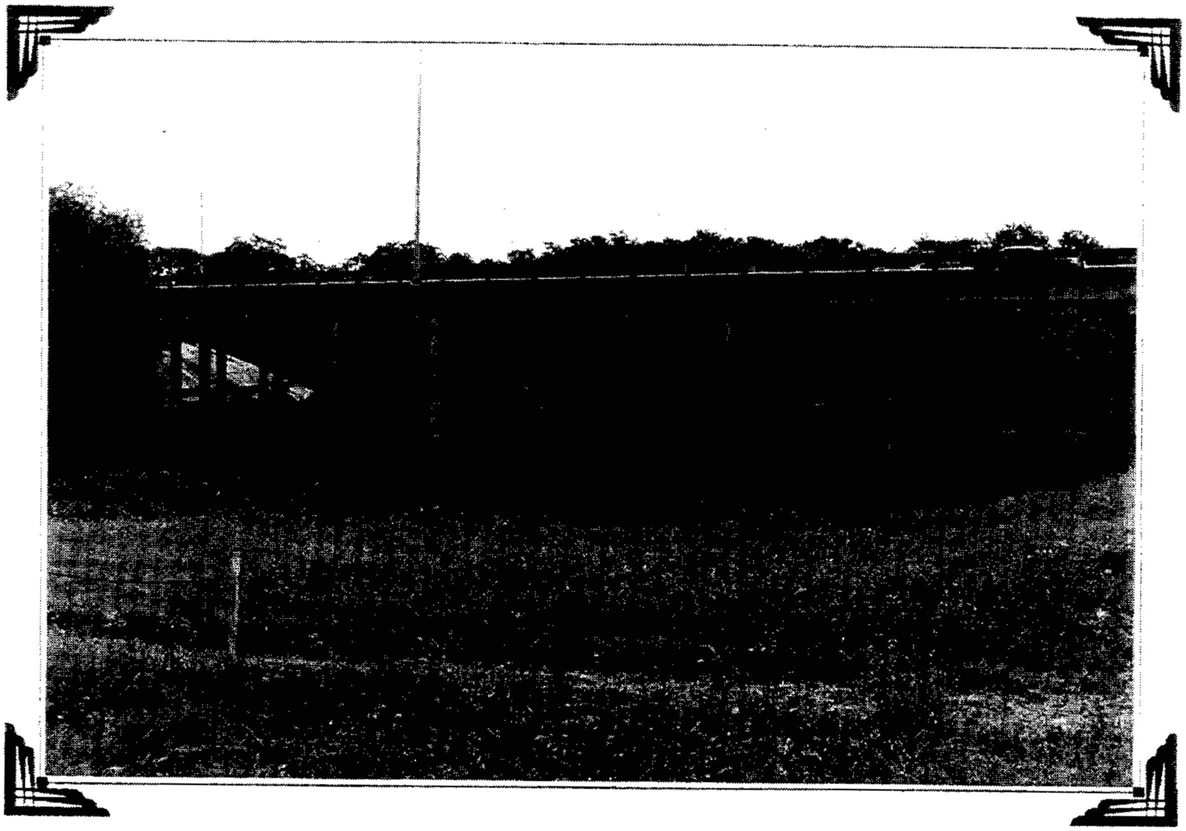
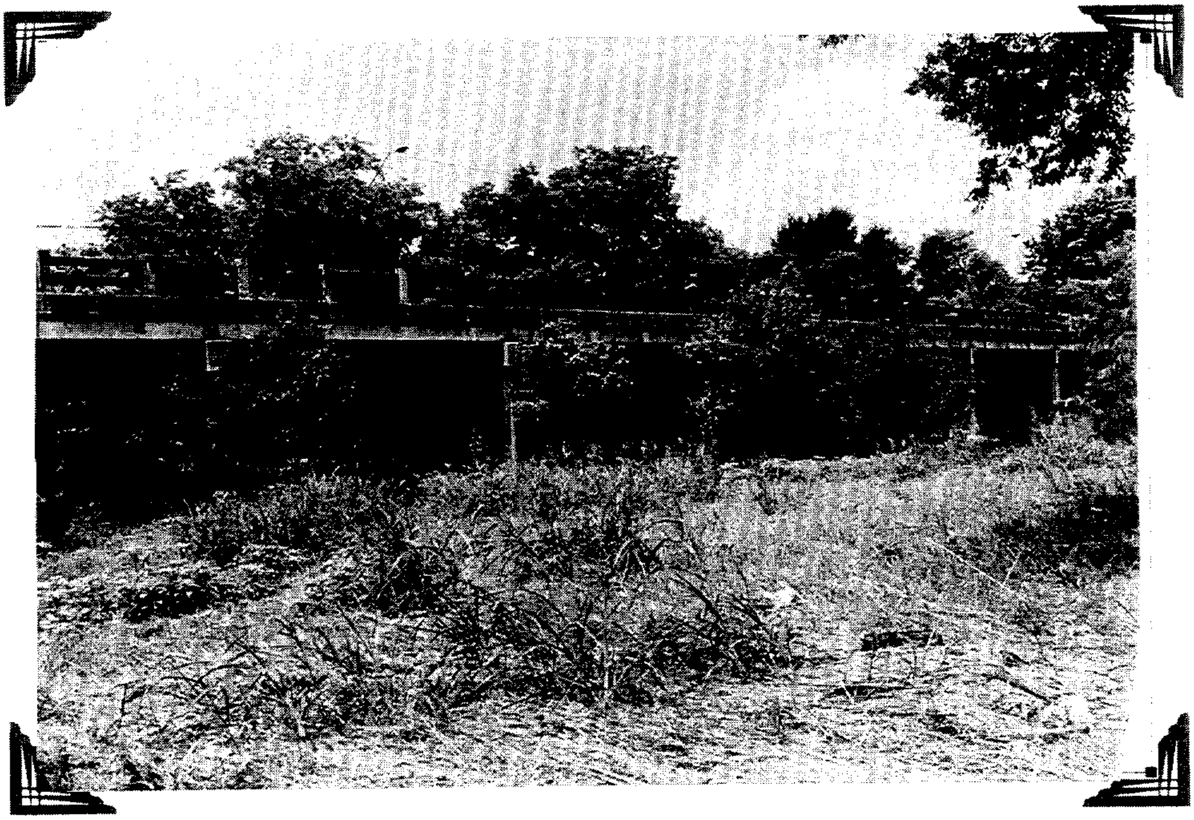
RIGSBY
W.W. WHITE
LOOP 410
ROSILLO
CREEK
FOSTER RD.
VALLEY
SOUTHCROSS
410

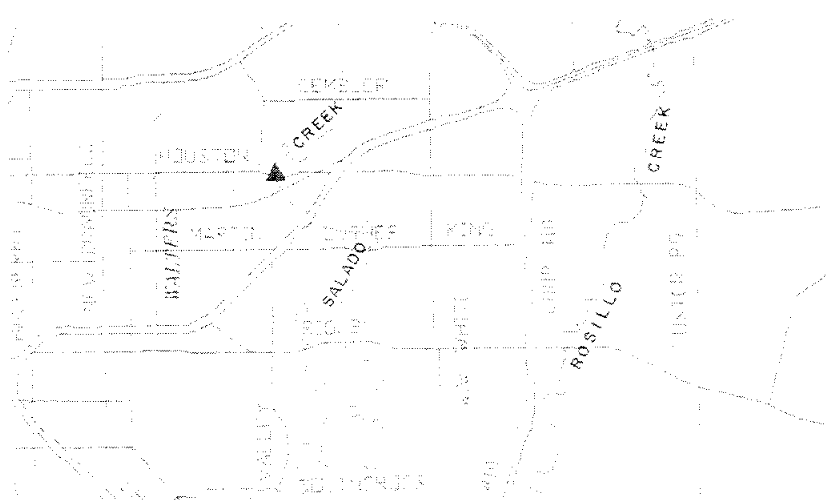


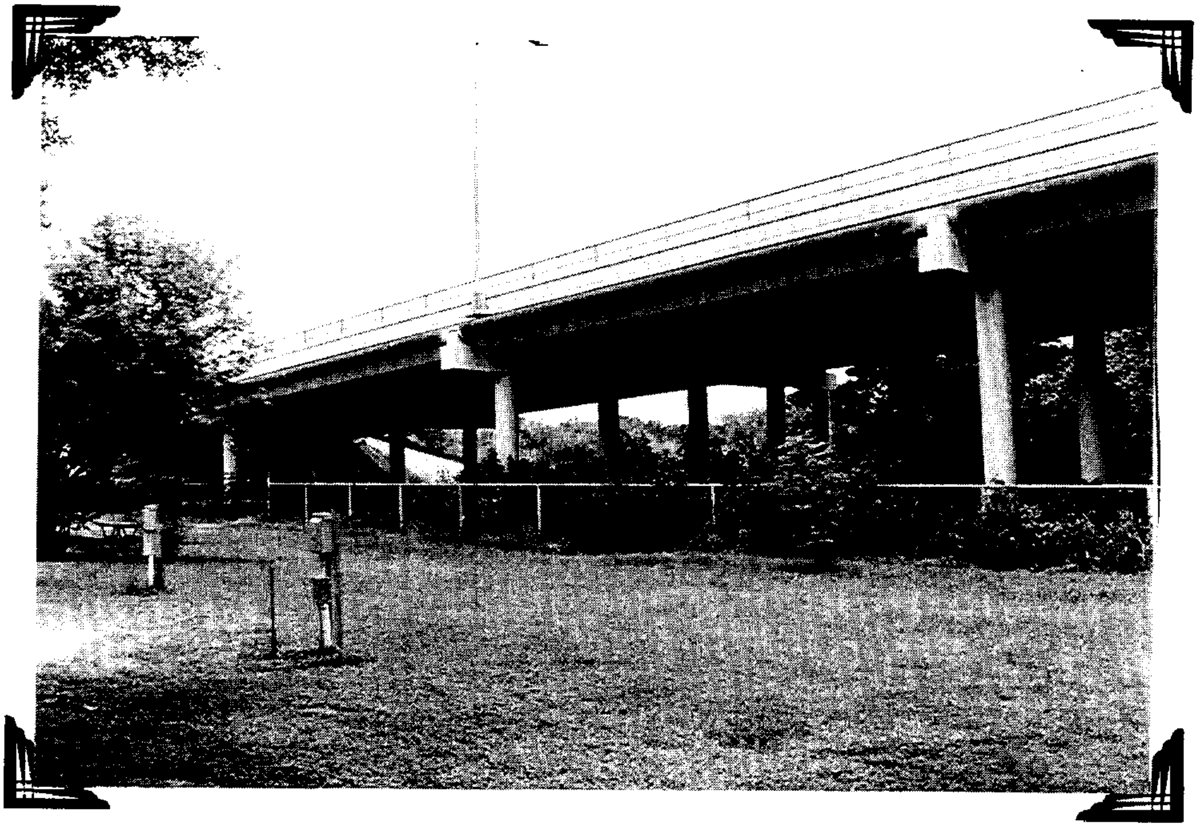
HWY.90/I H 10

UPSTREAM

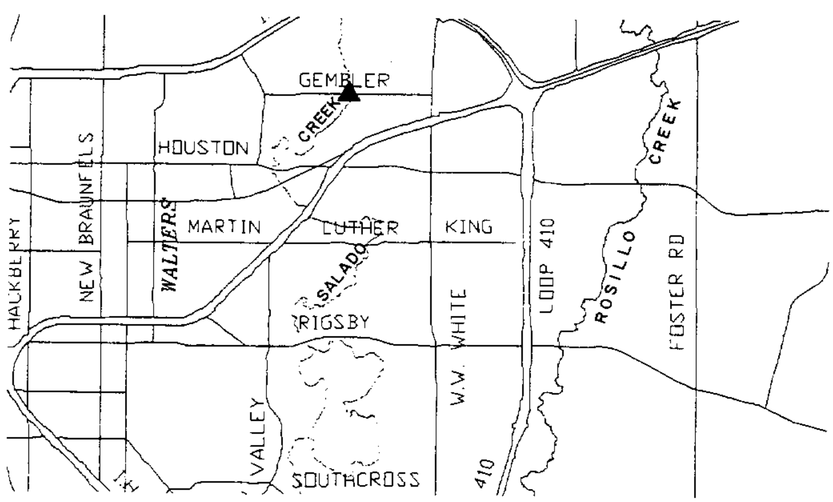


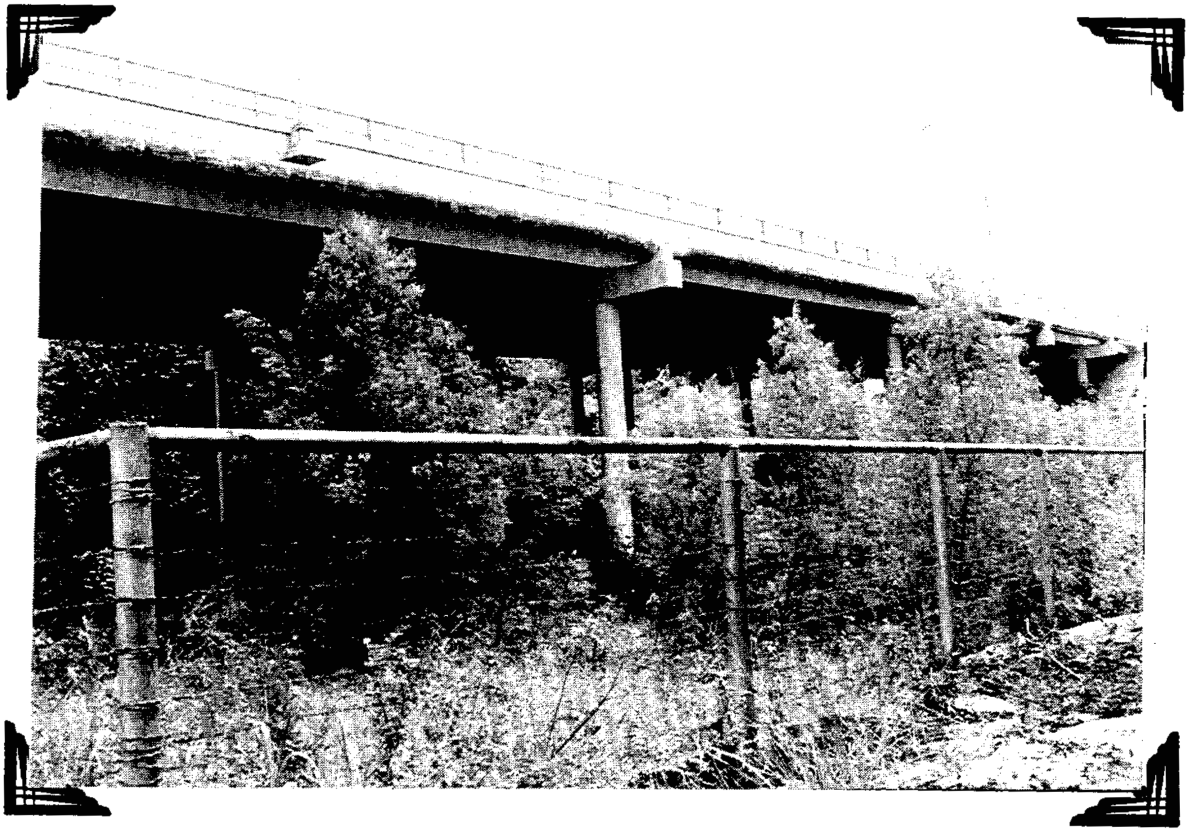
HOUSTON ST.





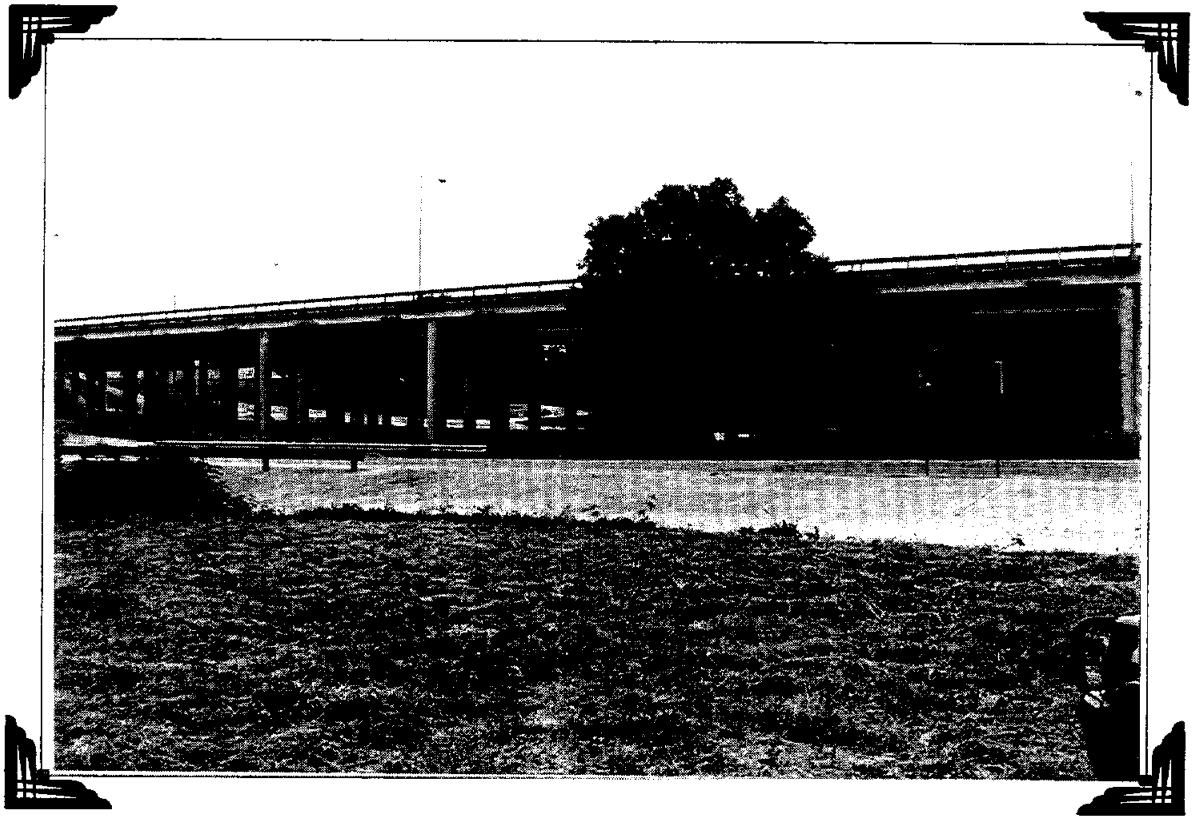
GEMBLER ST.


GEMBLER
CREEK
HOUSTON
HACKBERRY
NEW BRAUNFELS
WALTERS
MARTIN
LUTHER
KING
SALADO
RIGSBY
W.W. WHITE
LOOP 410
ROSILLO
CREEK
FOSTER RD
VALLEY
SOUTHCROSS
410

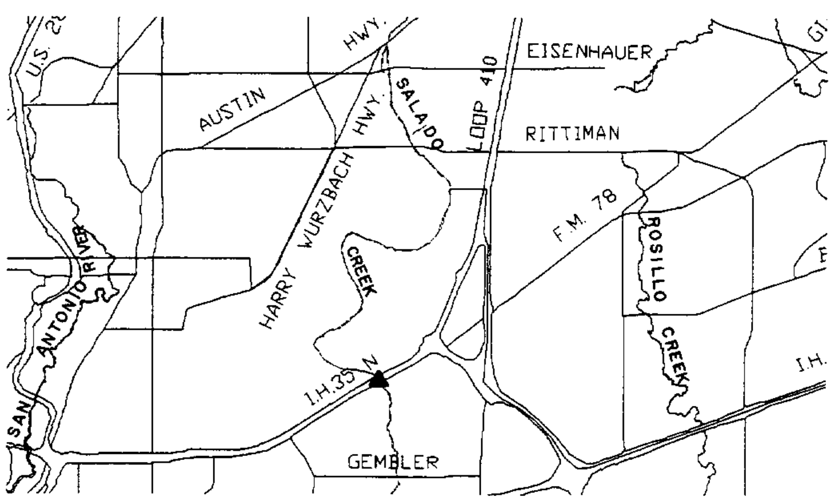


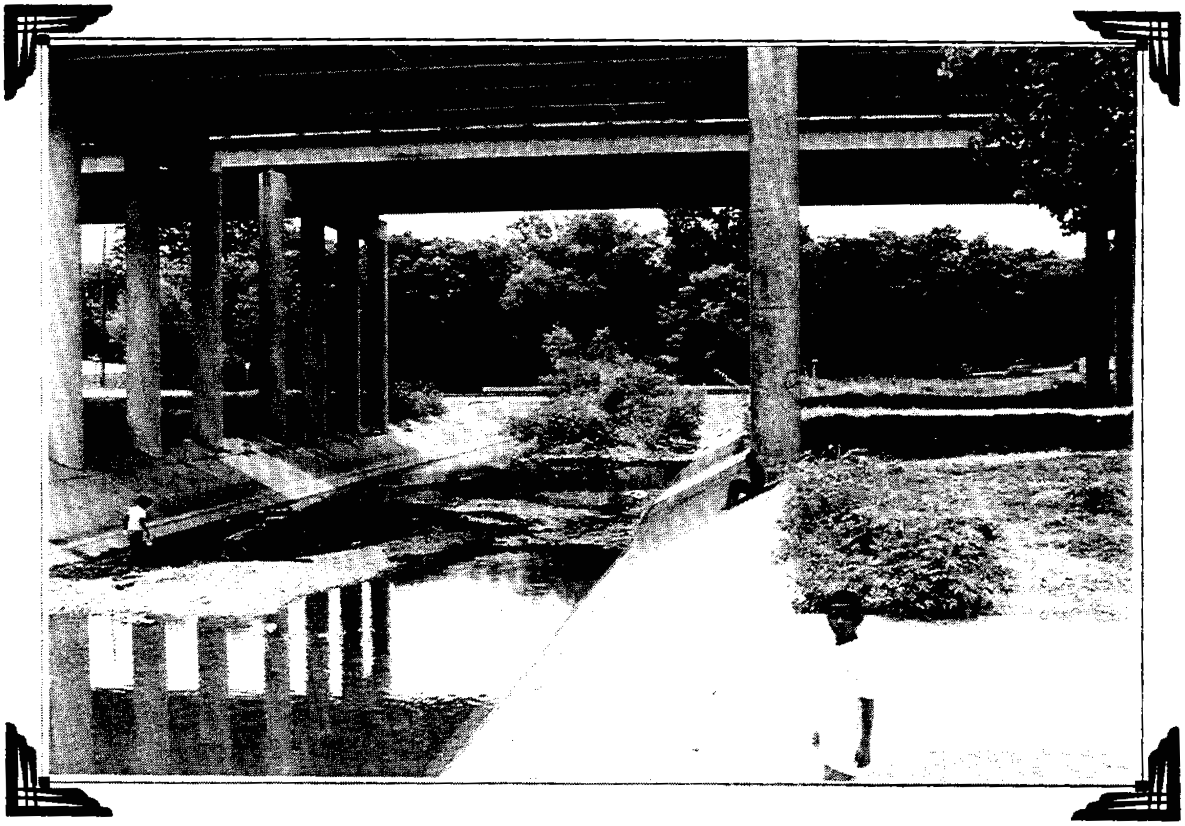
GEMBLER ST. UPSTREAM

GEMBLER ST. UPSTREAM

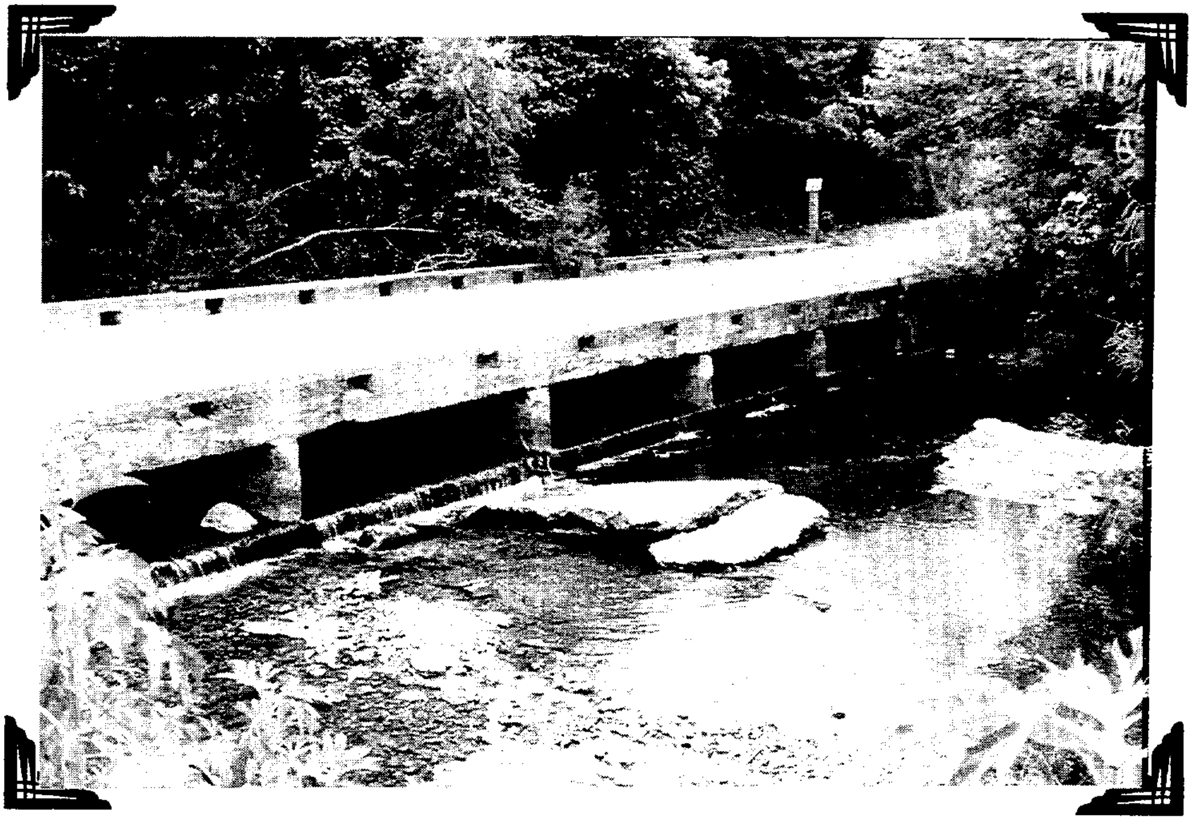


I H 35

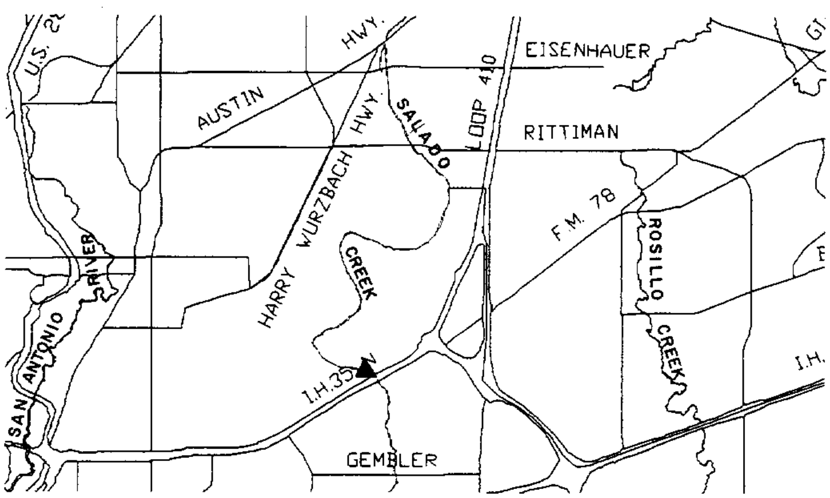

EISENHAUER
AUSTIN
HWY.
SALADO
LOOP 410
RITTIMAN
HARRY WURZBACH HWY.
CREEK
F.M. 78
ROSILLO
CREEK
SAN ANTONIO RIVER
I.H.35 N
GEMBLER



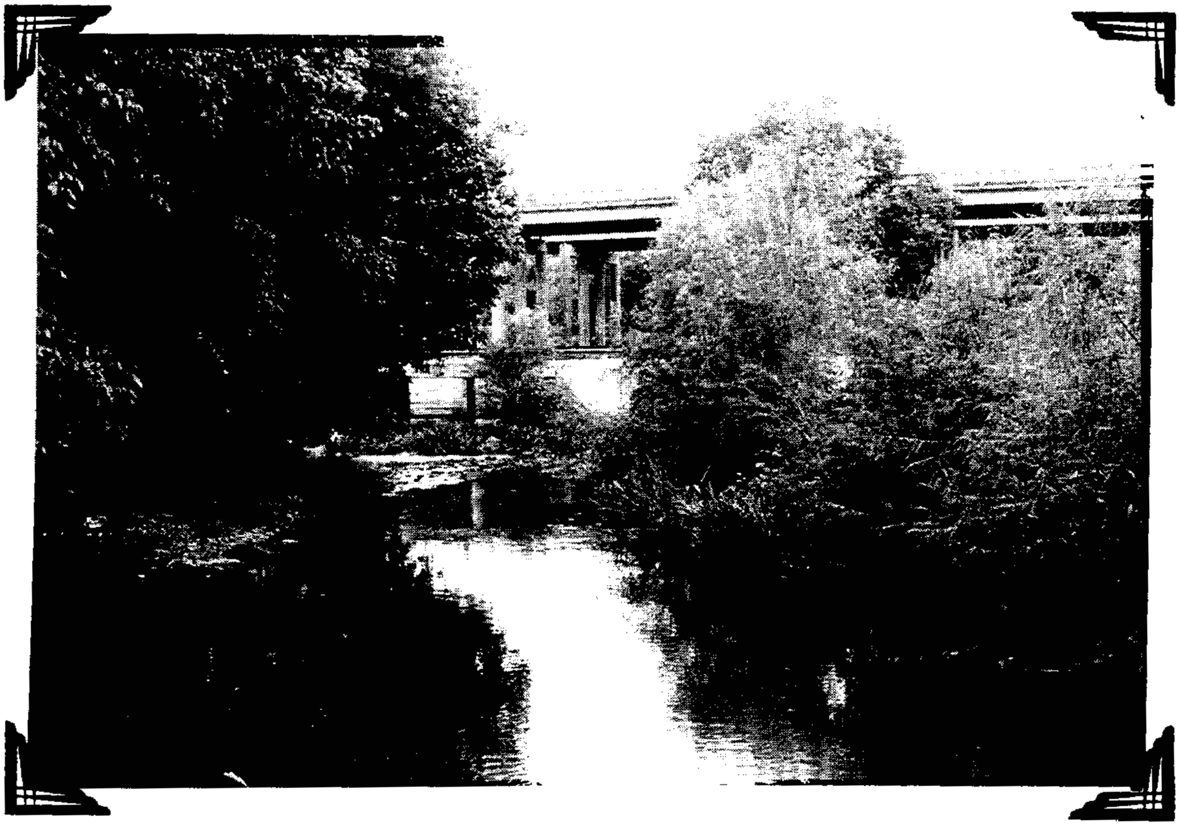
I H 35 DOWNSTREAM



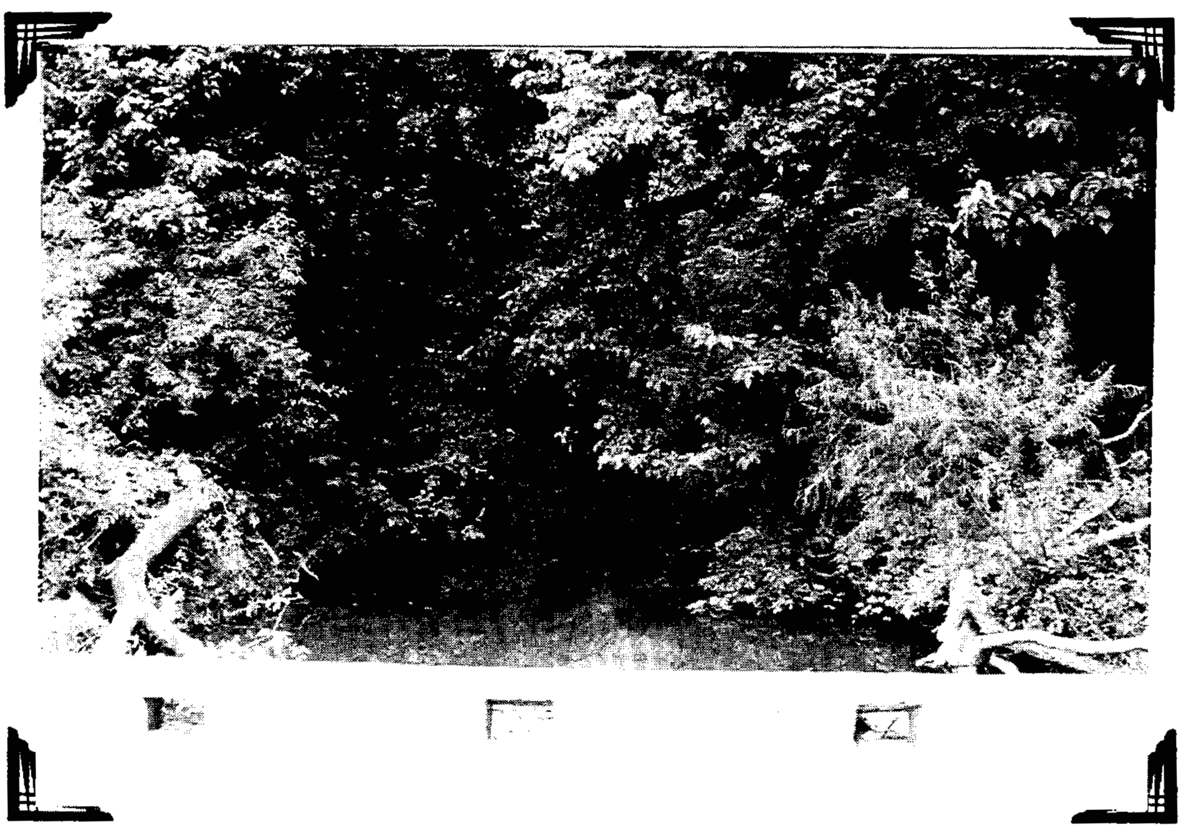
SEGUIN RD.



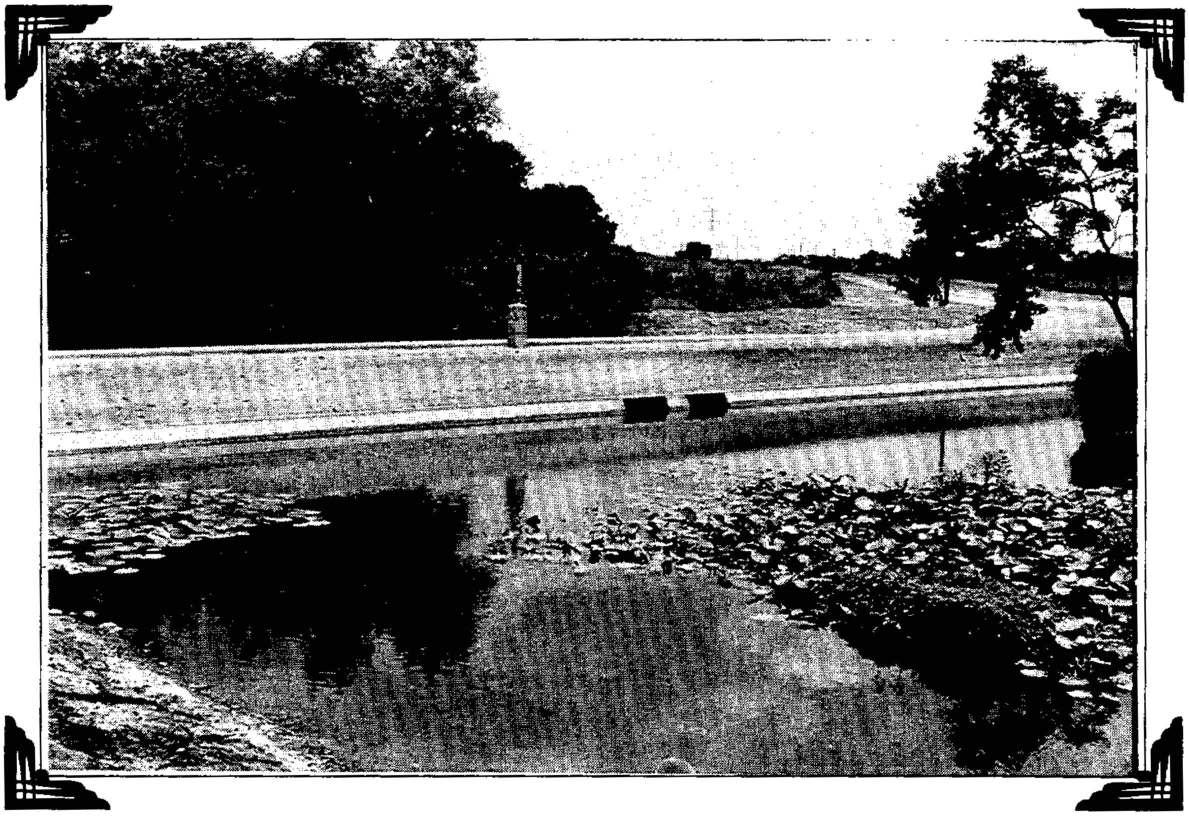
U.S. 281
HWY.
EISENHAUER
AUSTIN
HWY.
SALADO
LOOP 410
RITTIMAN
HARRY WURZBACH
CREEK
F.M. 78
ROSILLO
CREEK
RIVER
SAN ANTONIO
I.H.35
GEMBLER



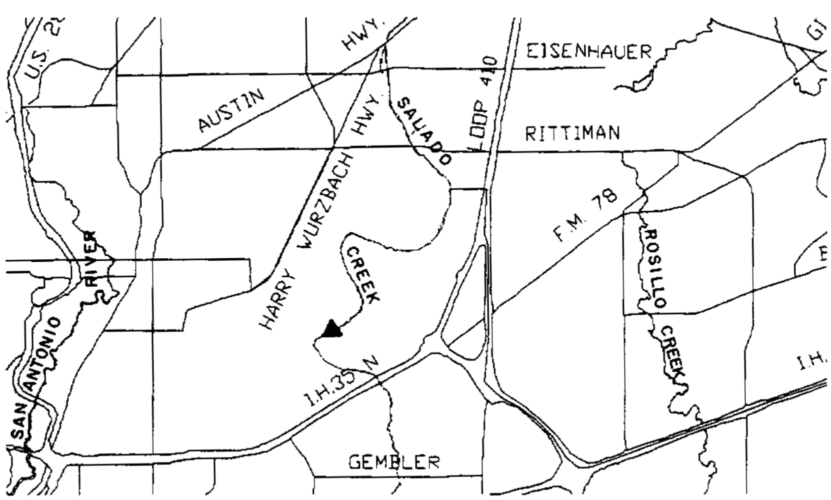
SEGUIN RD. DOWNSTREAM TO I H 35

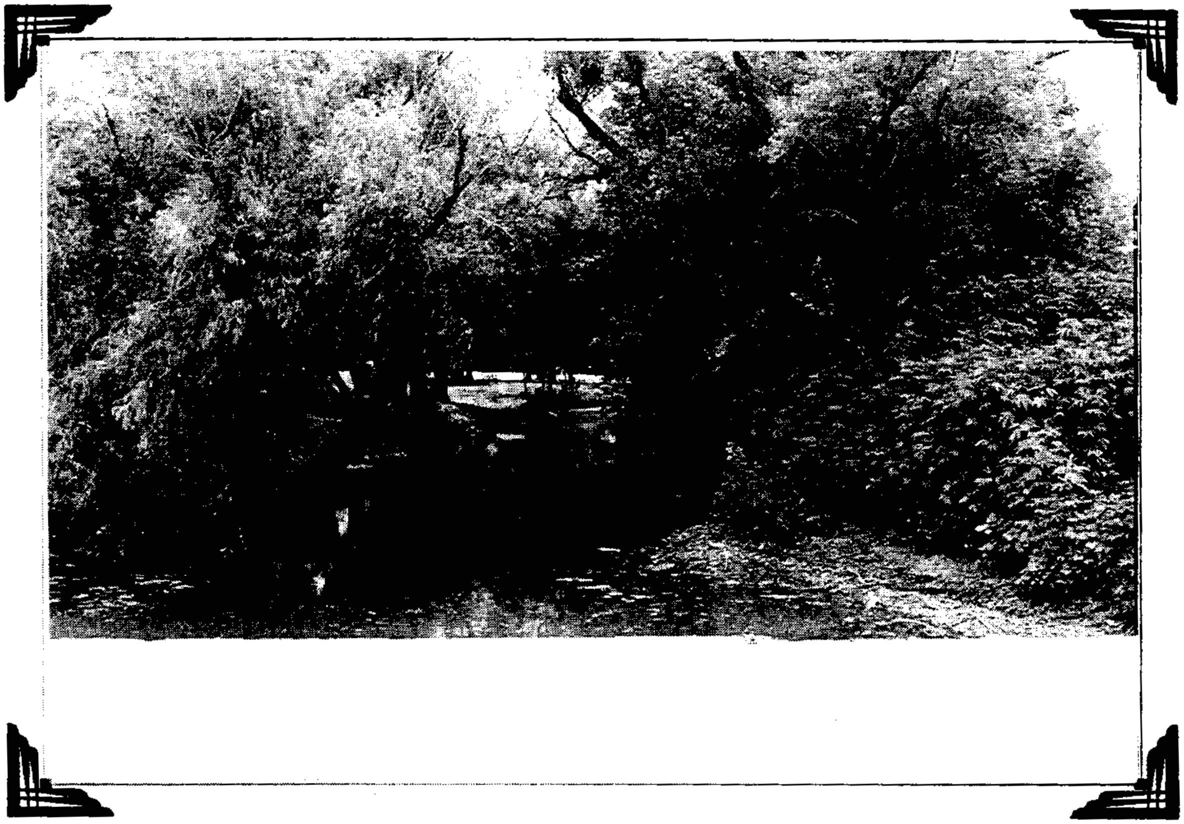


SEGUIN RD. UPSTREAM

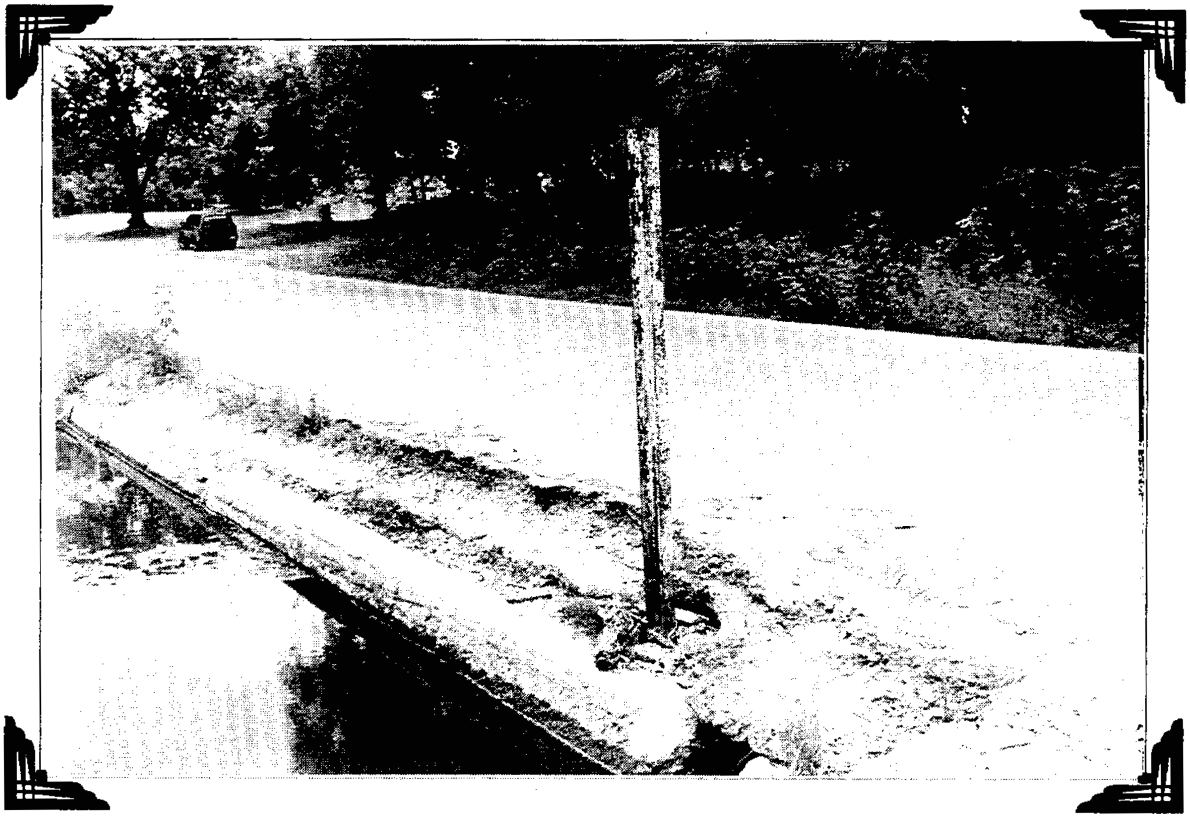


BINZ ENGLEMAN RD.

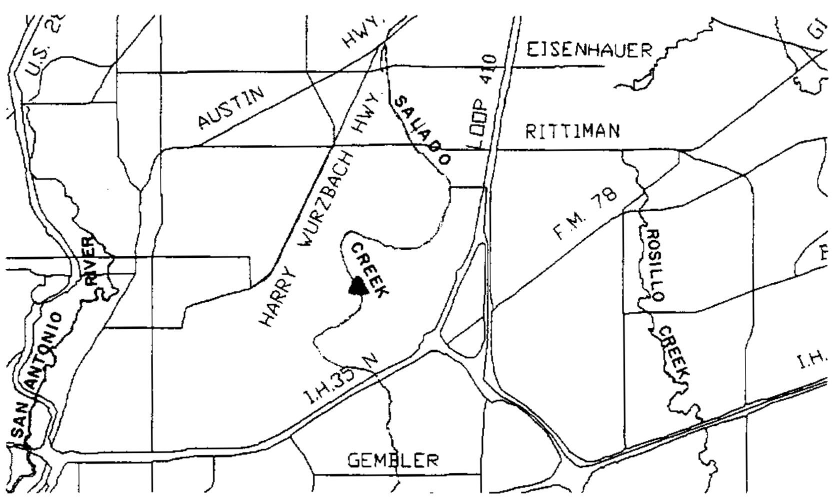




BINZ ENGLEMAN RD. UPSTREAM



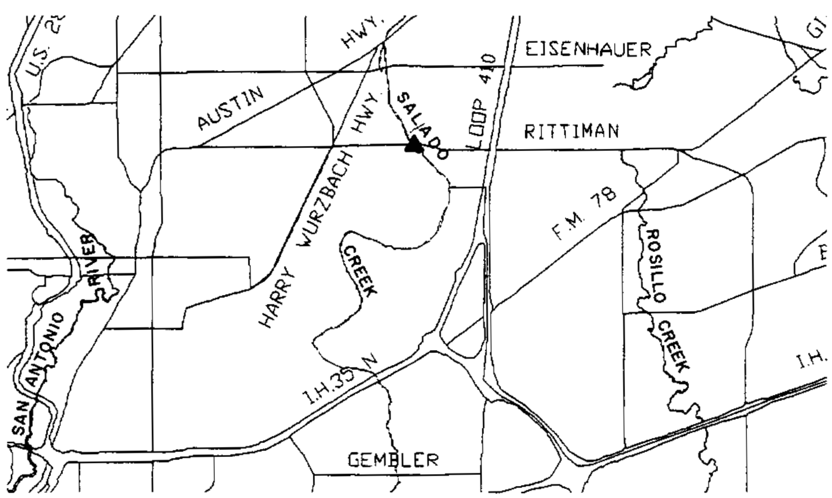
W. W. WHITE RD.

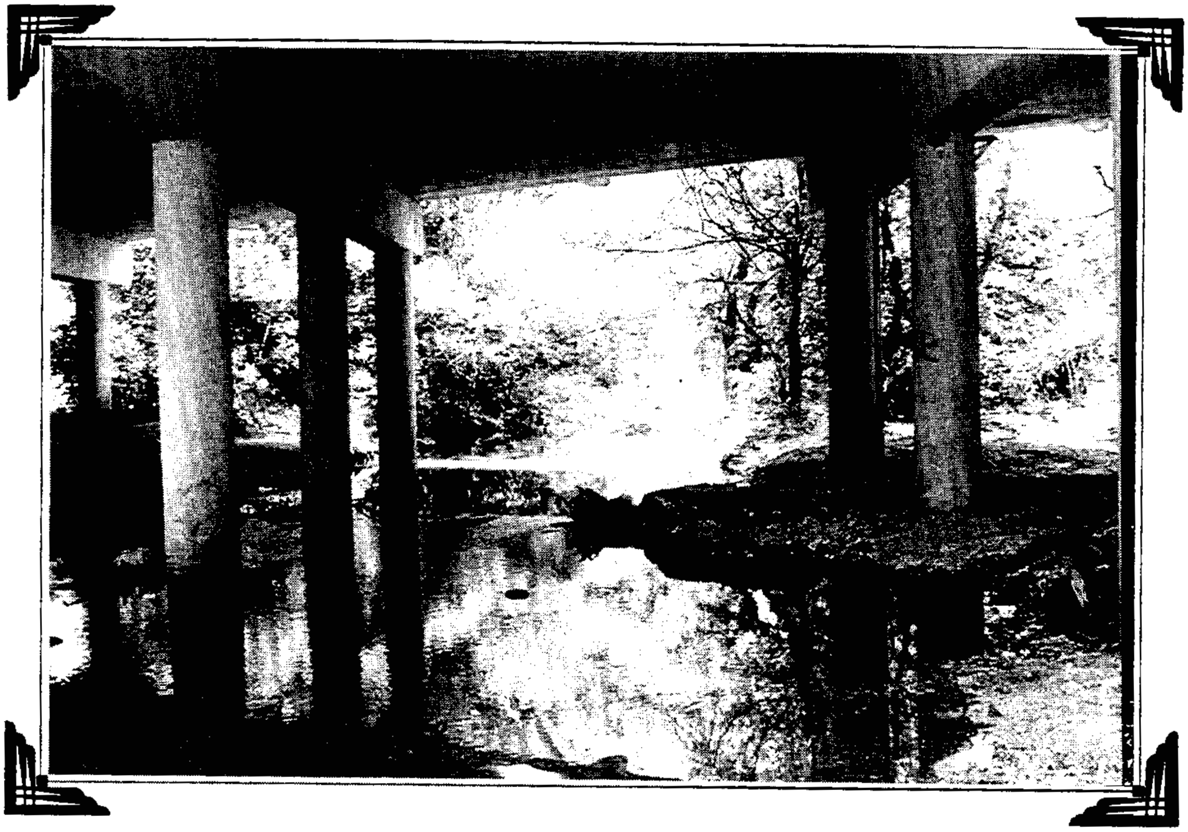


U.S. 2
HWY.
EISENHAUER
AUSTIN
SALADO
LOOP 410
RITTIMAN
HARRY WURZBACH HWY
F.M. 78
ROSILLO
RIVER
CREEK
SAN ANTONIO
CREEK
I.H.35 N
I.H.
GEMBLER

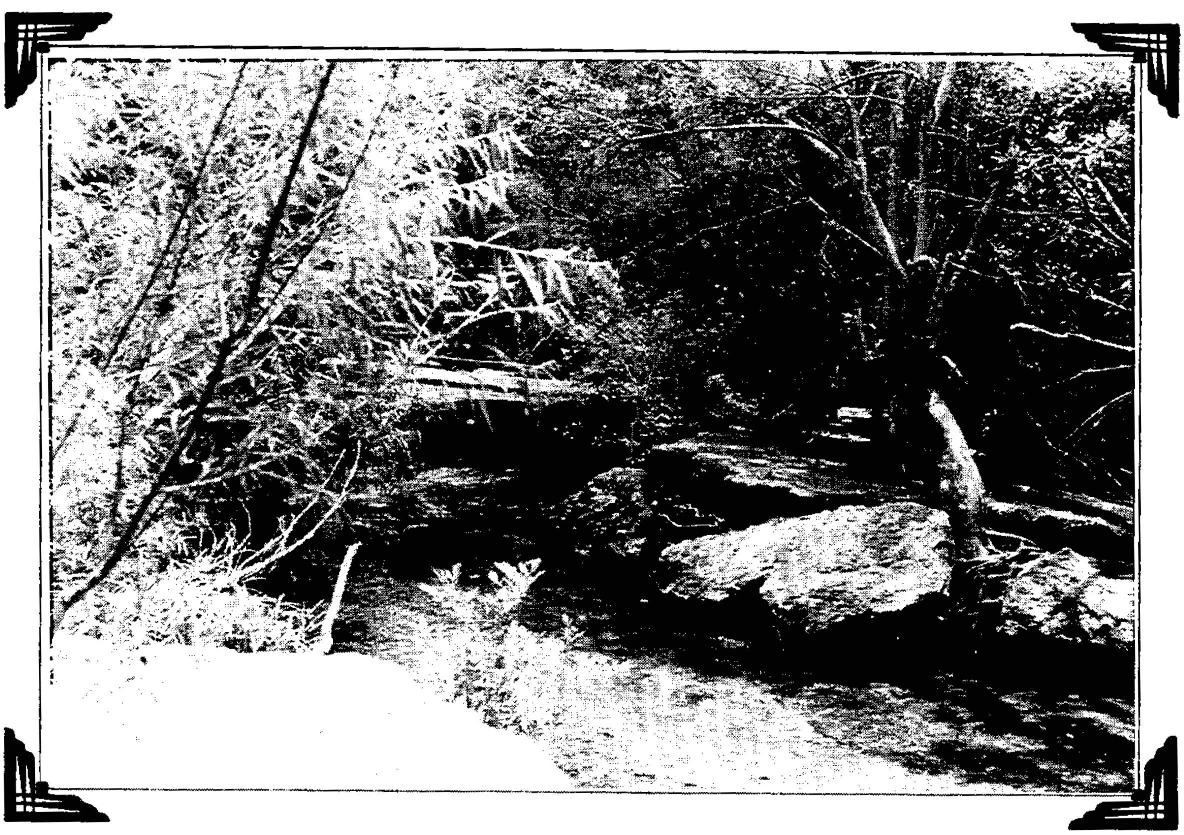
W.W. WHITE RD. UPSTREAM

RITTIMAN RD.


EISENHAUER
RITTIMAN
AUSTIN
HWY.
HWY
SALADO
LOOP 410
HARRY WURZBACH
CREEK
F.M. 78
ROSILLO
CREEK
SAN ANTONIO
RIVER
I.H.35 N
GEMBLER
I.H.

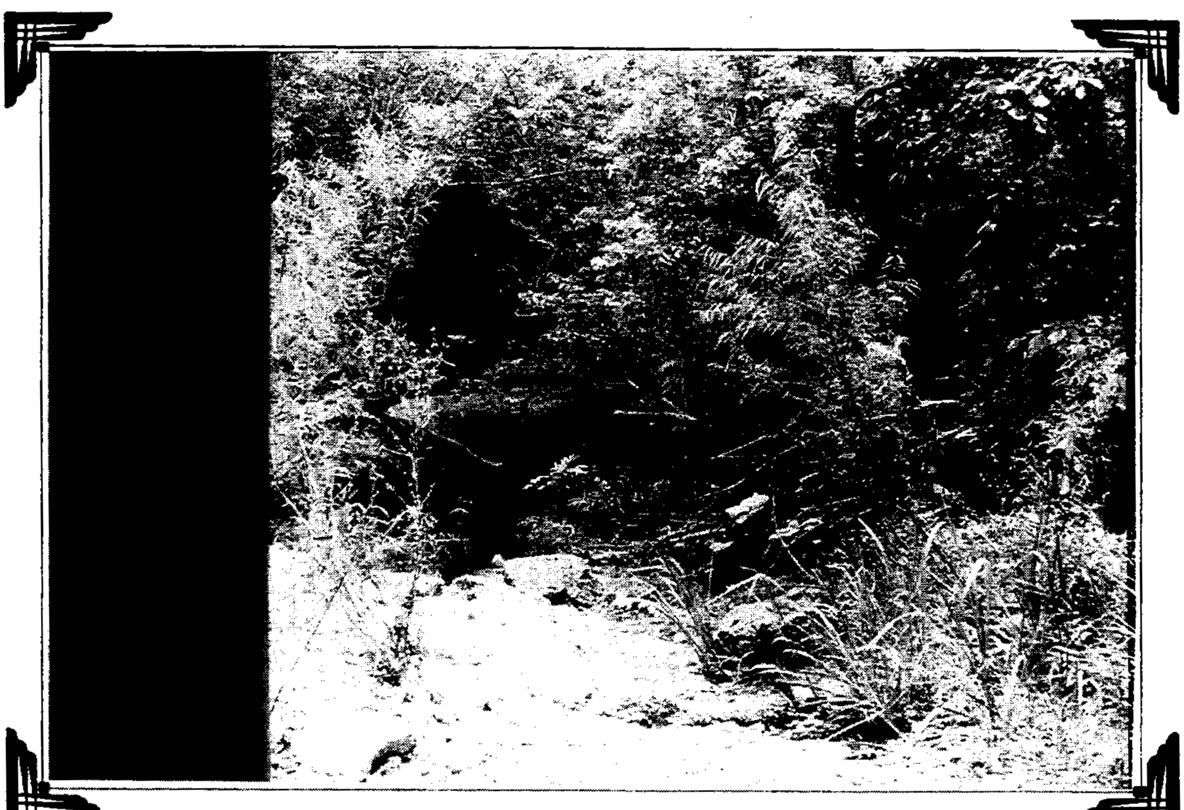


RITTIMAN RD. UPSTREAM



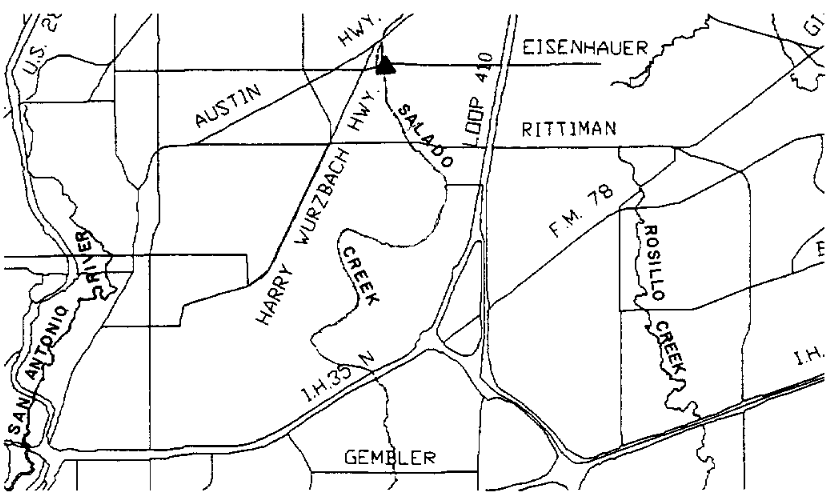
RITTIMAN RD. DOWNSTREAM

RITTIMAN RD.



RITTIMAN RD.

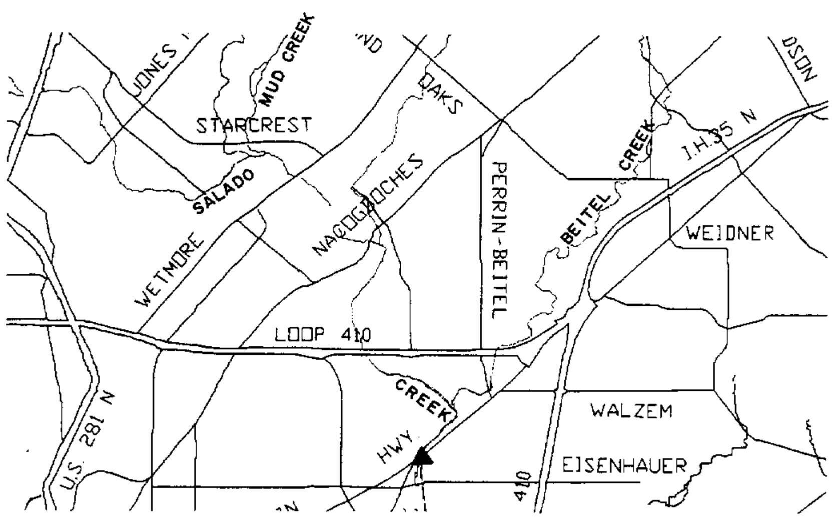
EISENHAUER RD.

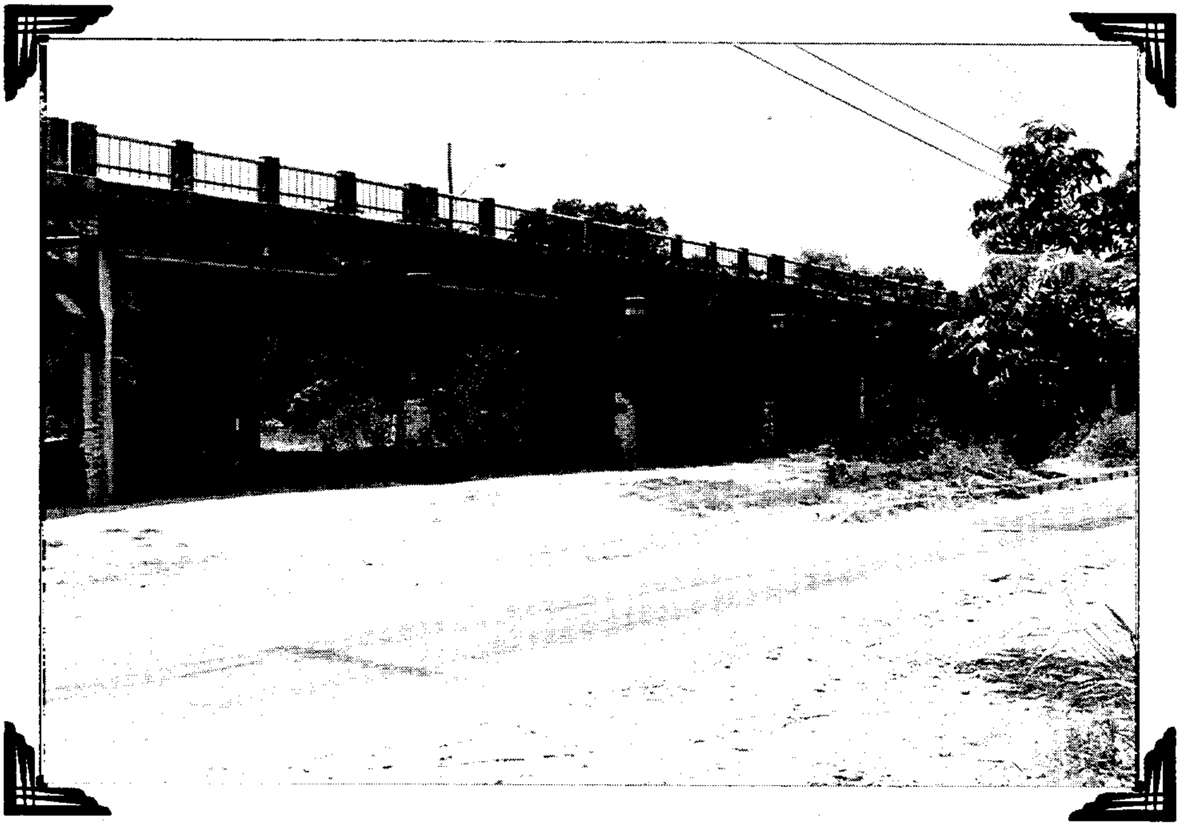


EISENHAUER RD.

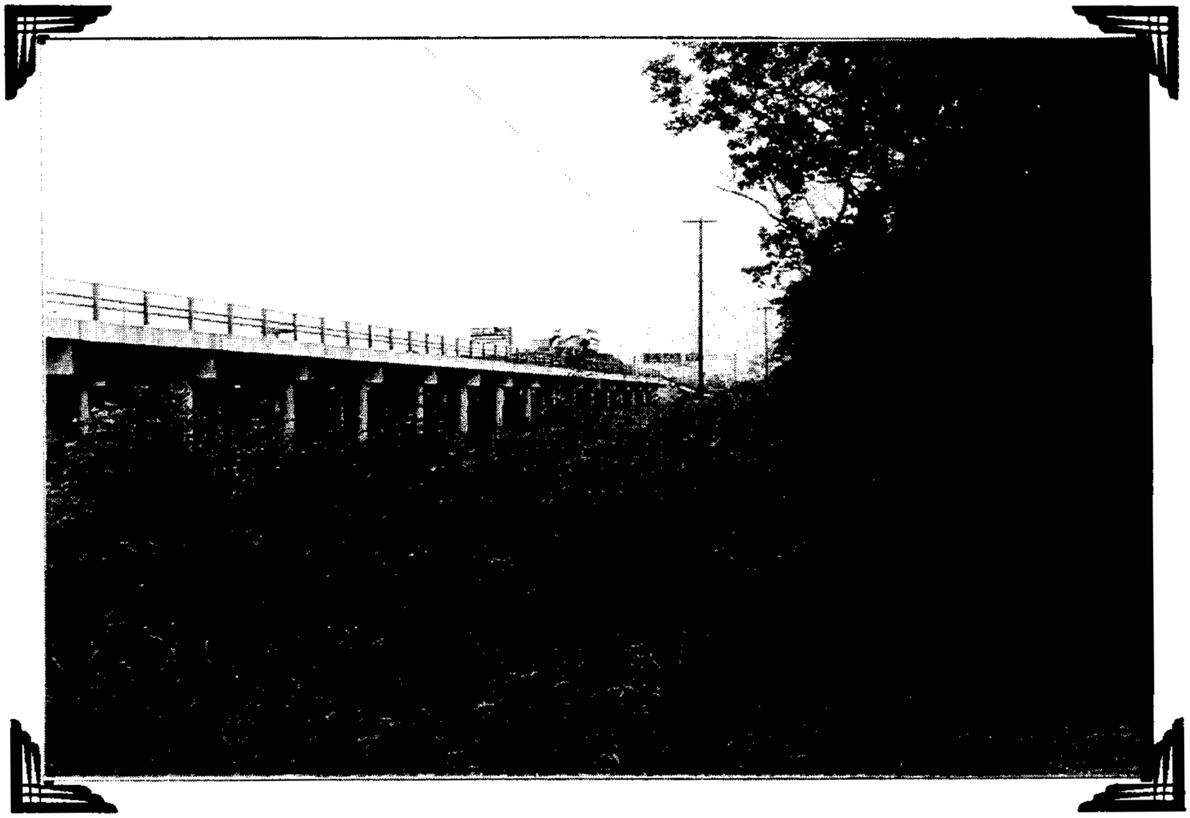
EISENHAUER RD. UPSTREAM

AUSTIN HWY.

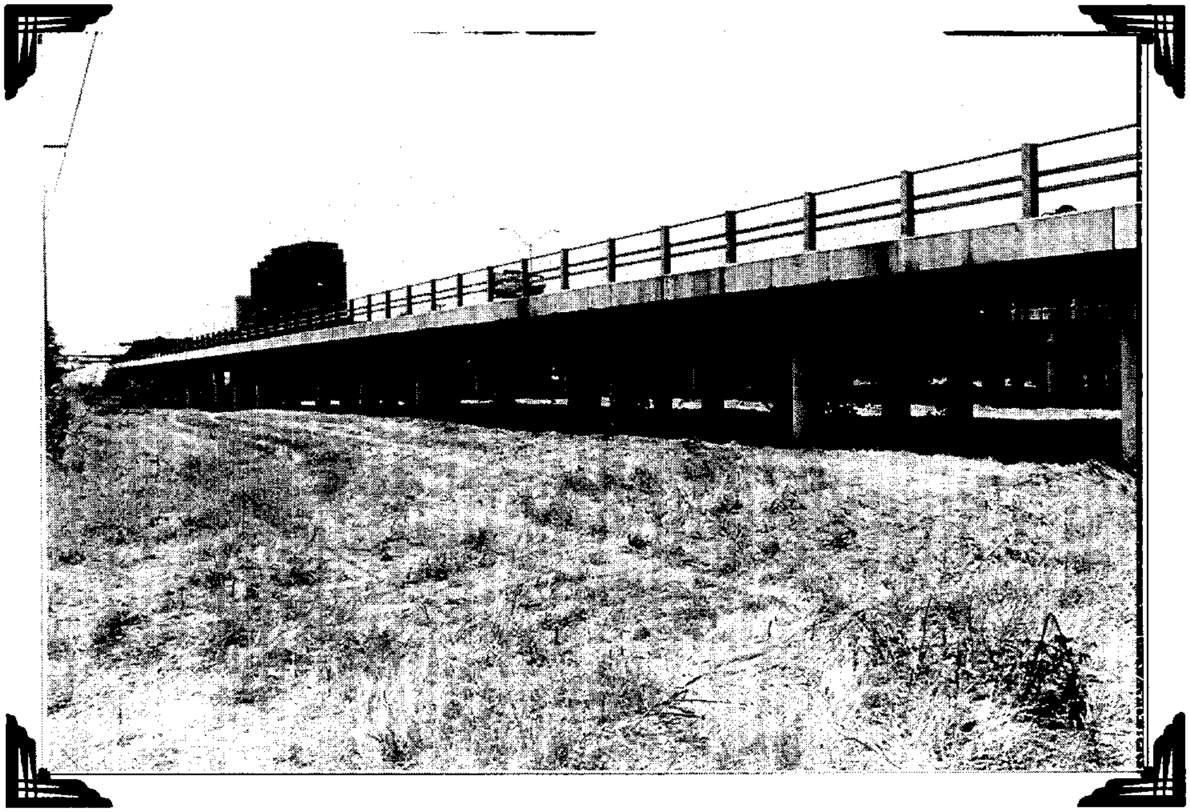

JONES
MUD CREEK
STARCREST
SALADO
WETMORE
OAKS
NACOGDOCHES
PERRIN-BEITEL
BEITEL
CREEK
I.H.35 N
WEIDNER
LOOP 410
CREEK
HWY.
WALZEM
EISENHAUER
U.S. 281 N
410



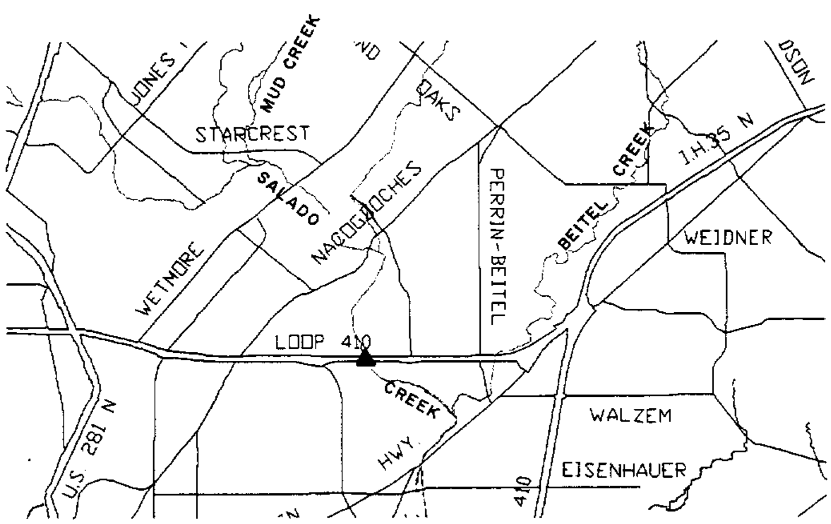
AUSTIN HWY. UPSTREAM

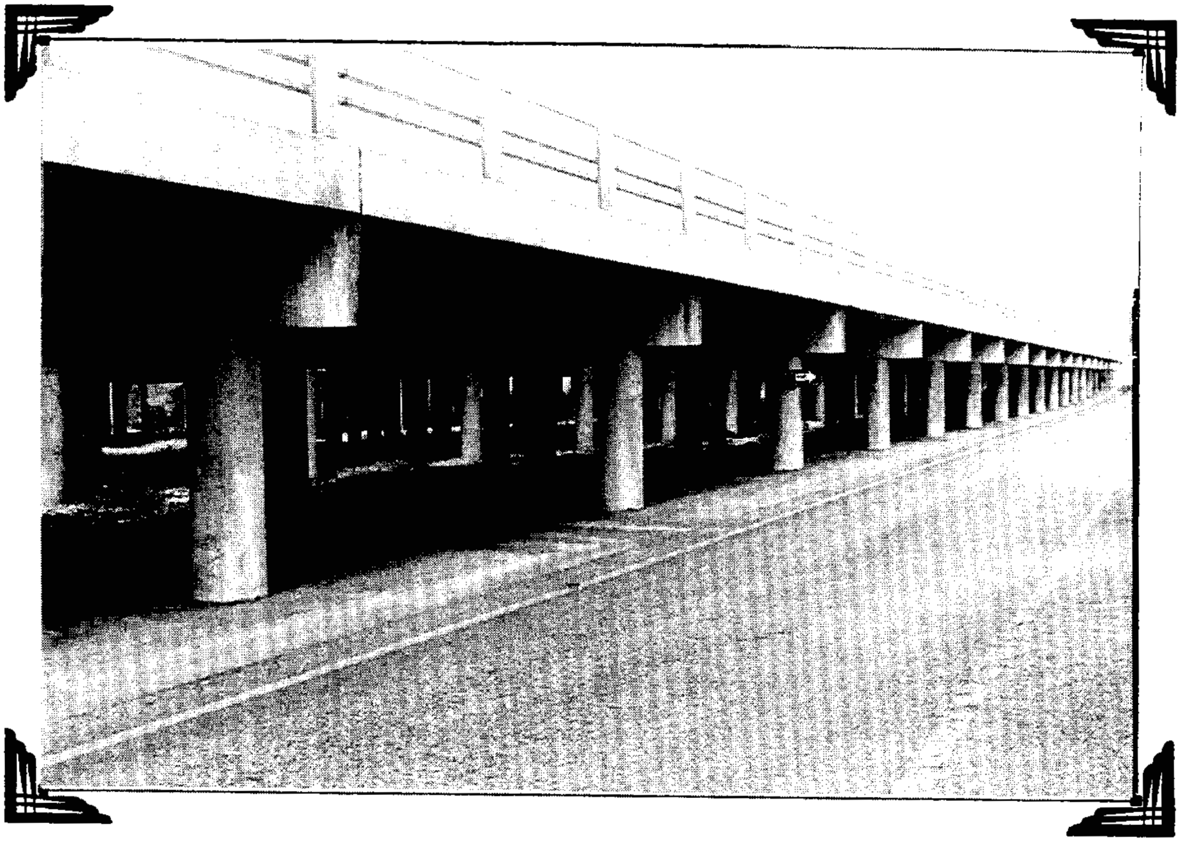


N.E. LOOP 410 DOWNSTREAM



N. E. LOOP 410


JONES
MUD CREEK
OAKS
STARCREST
SALADO
NACOGDOCHES
PERRIN-BEITEL
BEITEL CREEK
I.H.35 N
WEIDNER
WETMORE
LOOP 410
CREEK
WALZEM
HWY.
EISENHAUER
U.S. 281 N
410



N.E. LOOP 410

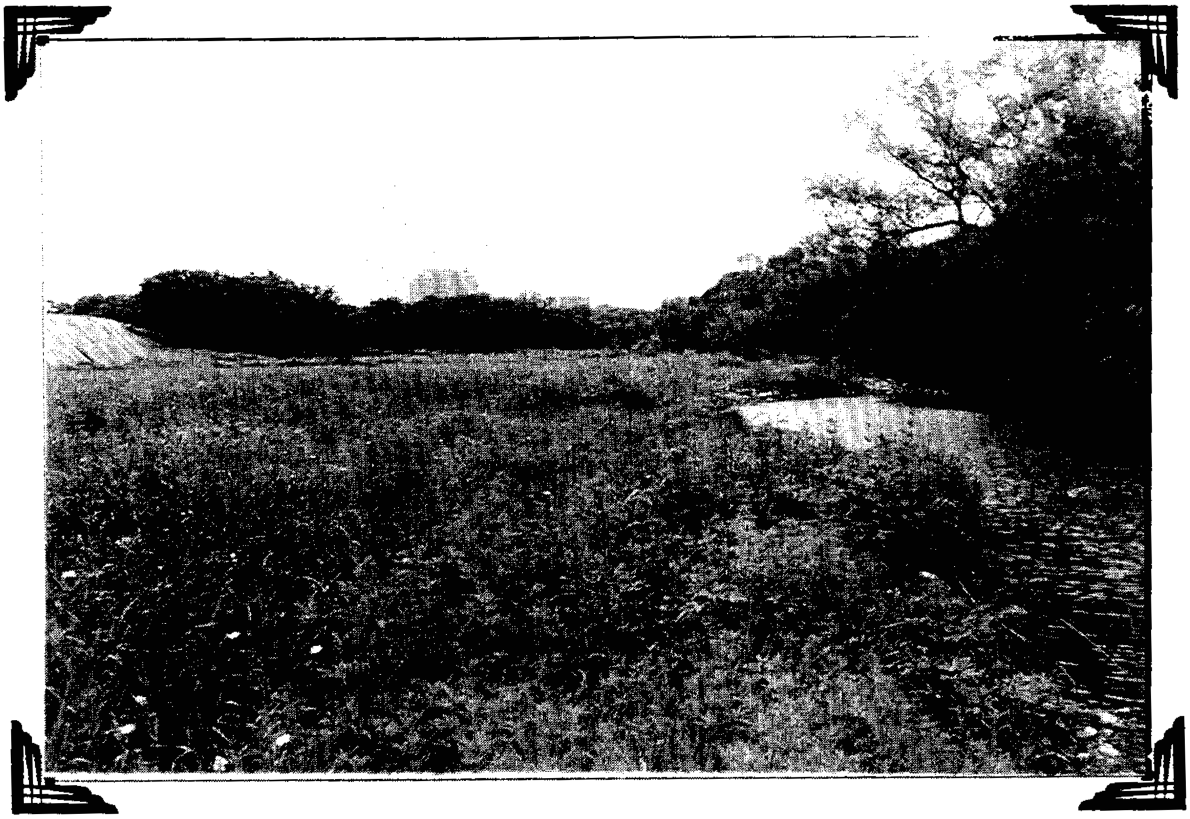
N.E. LOOP 410 DOWNSTREAM

N.E. LOOP 410 UPSTREAM ADJACENT TO LOS PATIOS

N.E. LOOP 410 ADJACENT TO LOS PATIOS

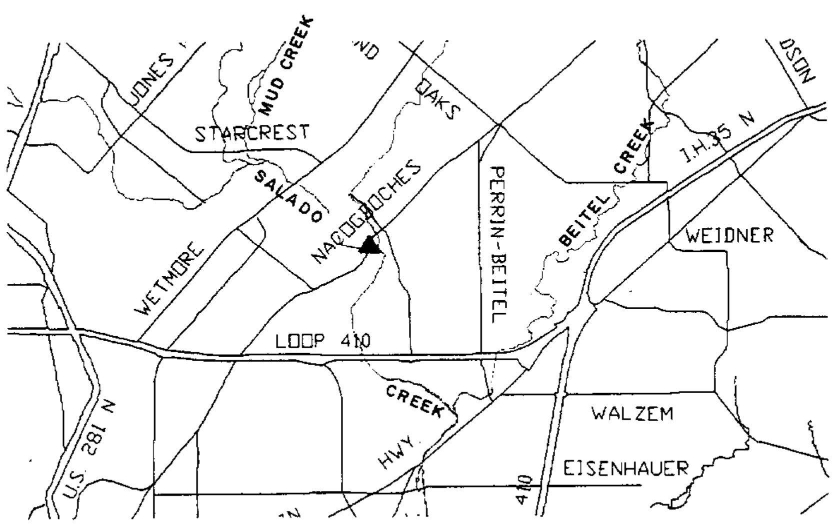
ADJACENT TO LOS PATIOS

ESTATES OF MARYMONT

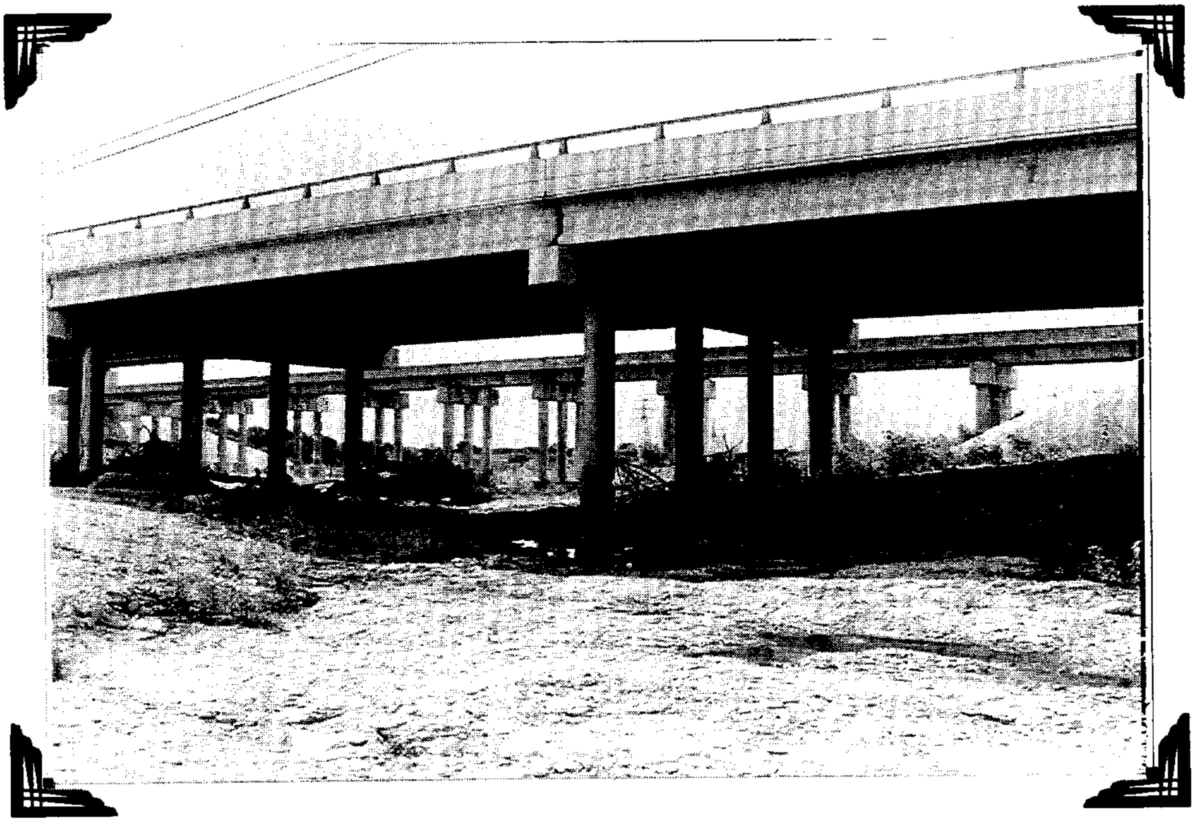


ADJACENT TO ESTATES
OF MARYMONT

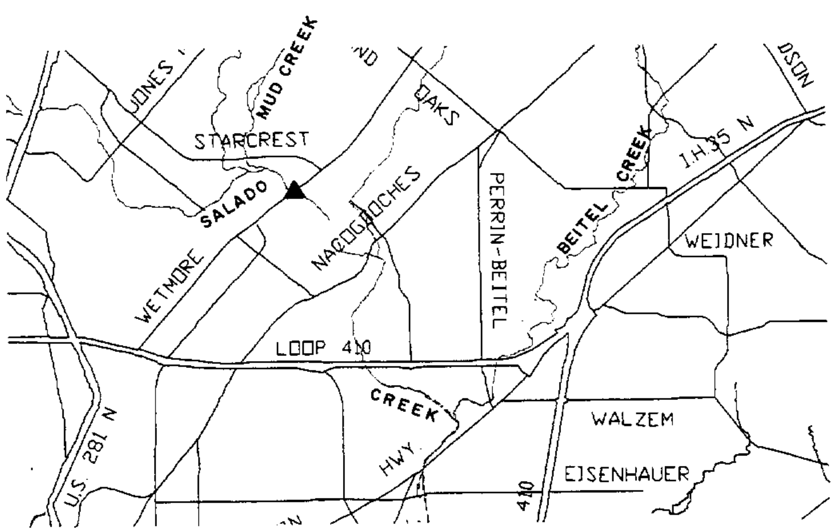
NACOGDOCHES RD.



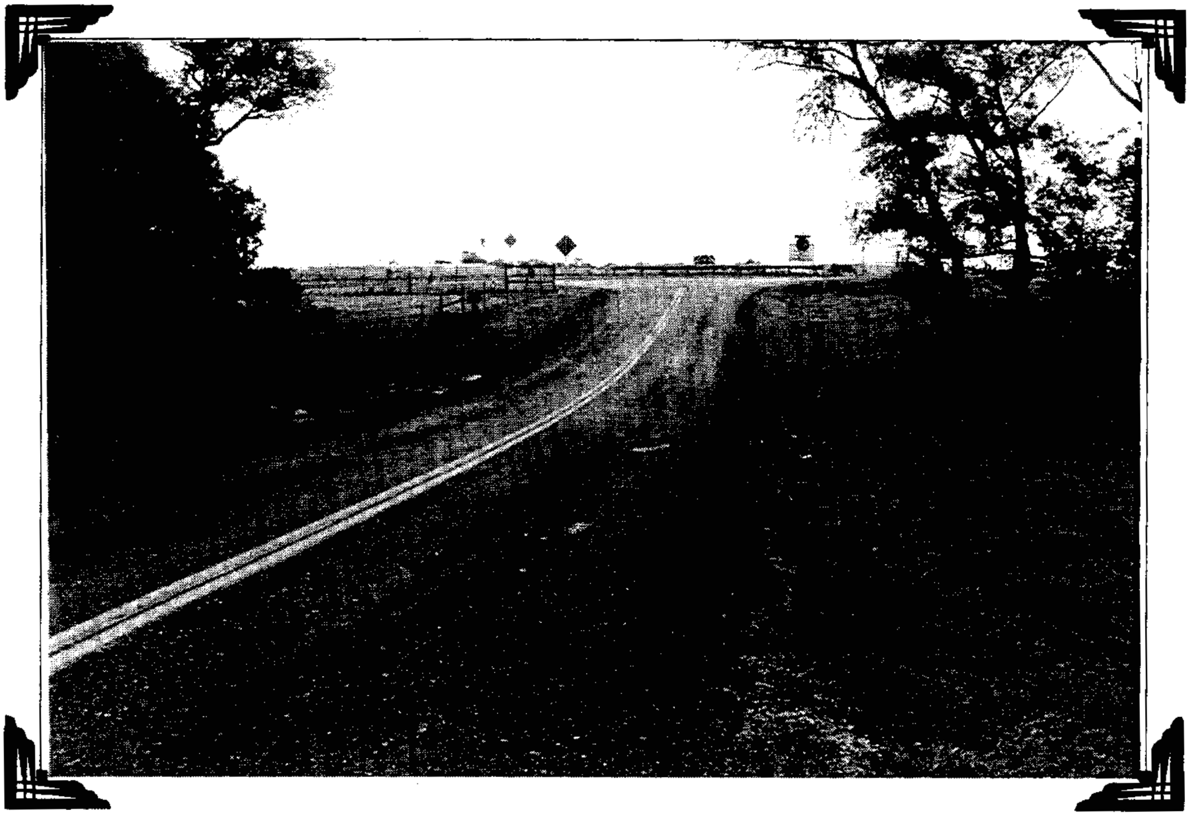
NACOGDOCHES RD. UPSTREAM



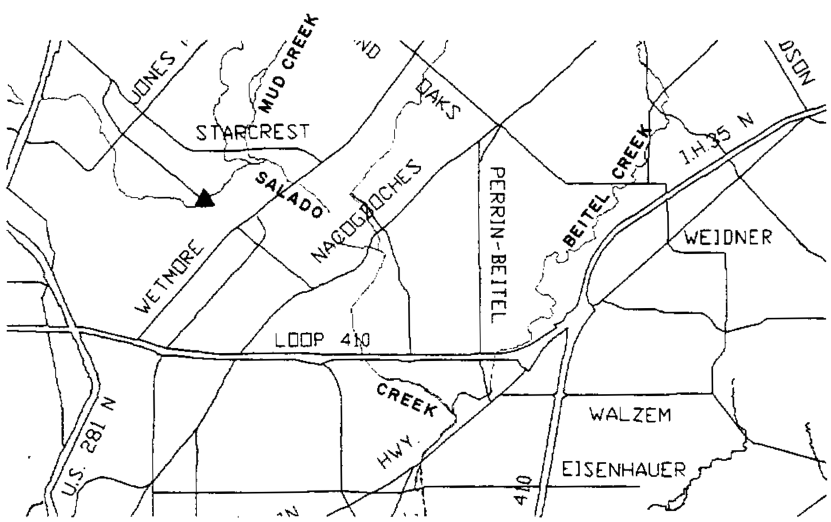
WETMORE RD.

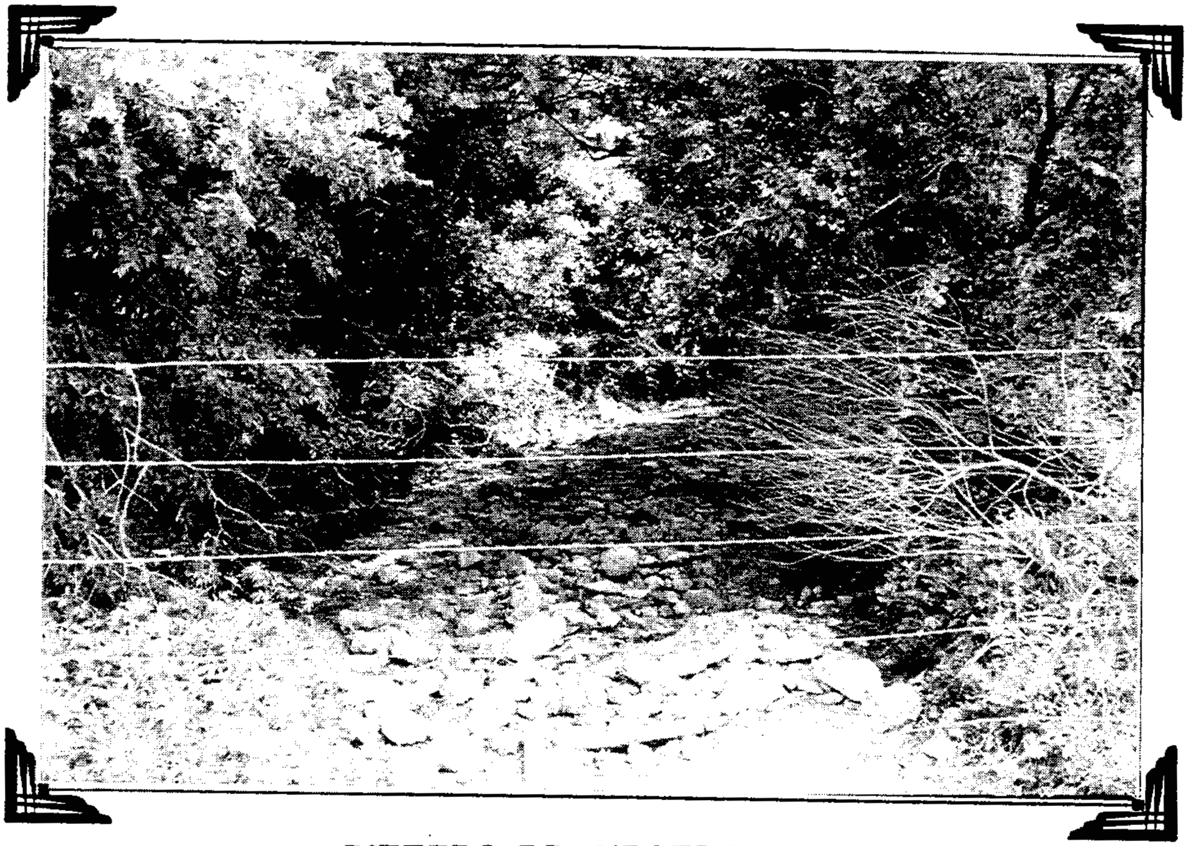

JONES
MUD CREEK
OAKS
STARCREST
SALADO
NACOGDOCHES
WETMORE
PERRIN-BEITEL
BEITEL CREEK
I.H. 35 N
WEIDNER
LOOP 410
CREEK
WALZEM
EISENHAUER
HWY.
U.S. 281 N
410

WETMORE RD. DOWNSTREAM



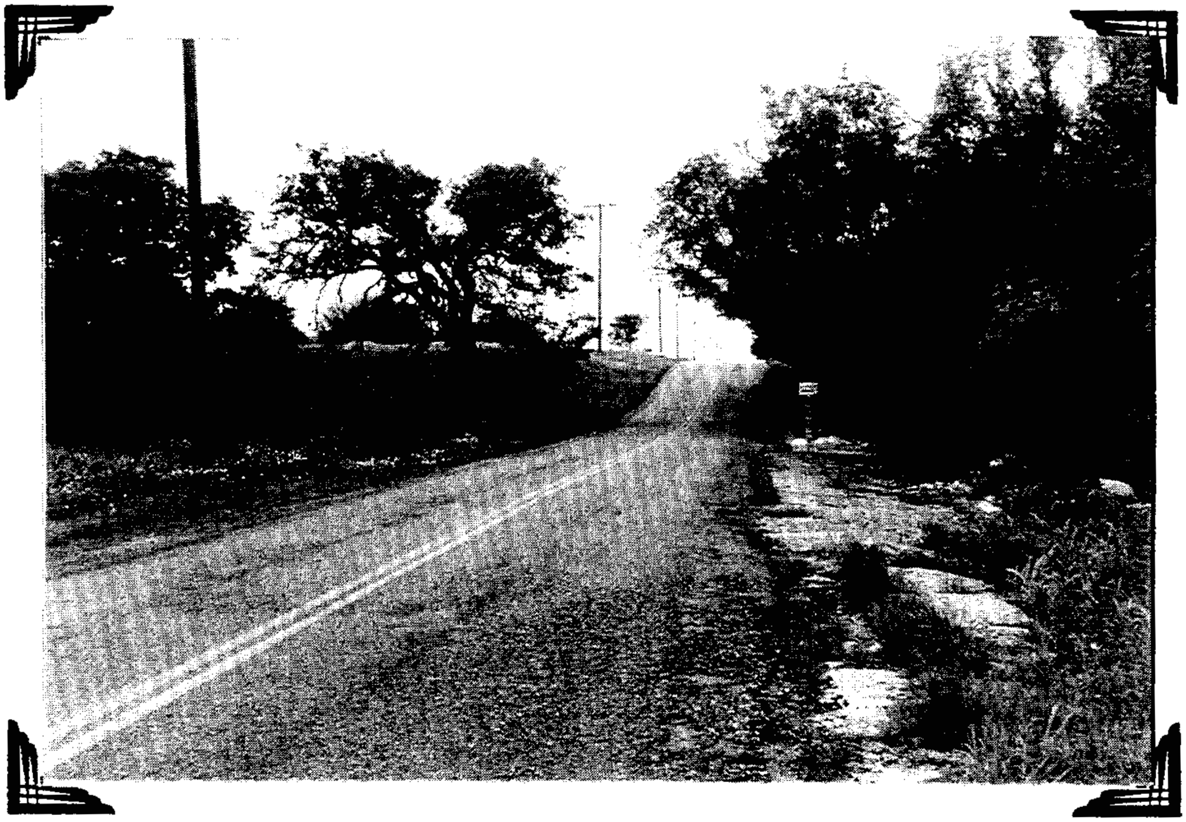
BITTERS RD.

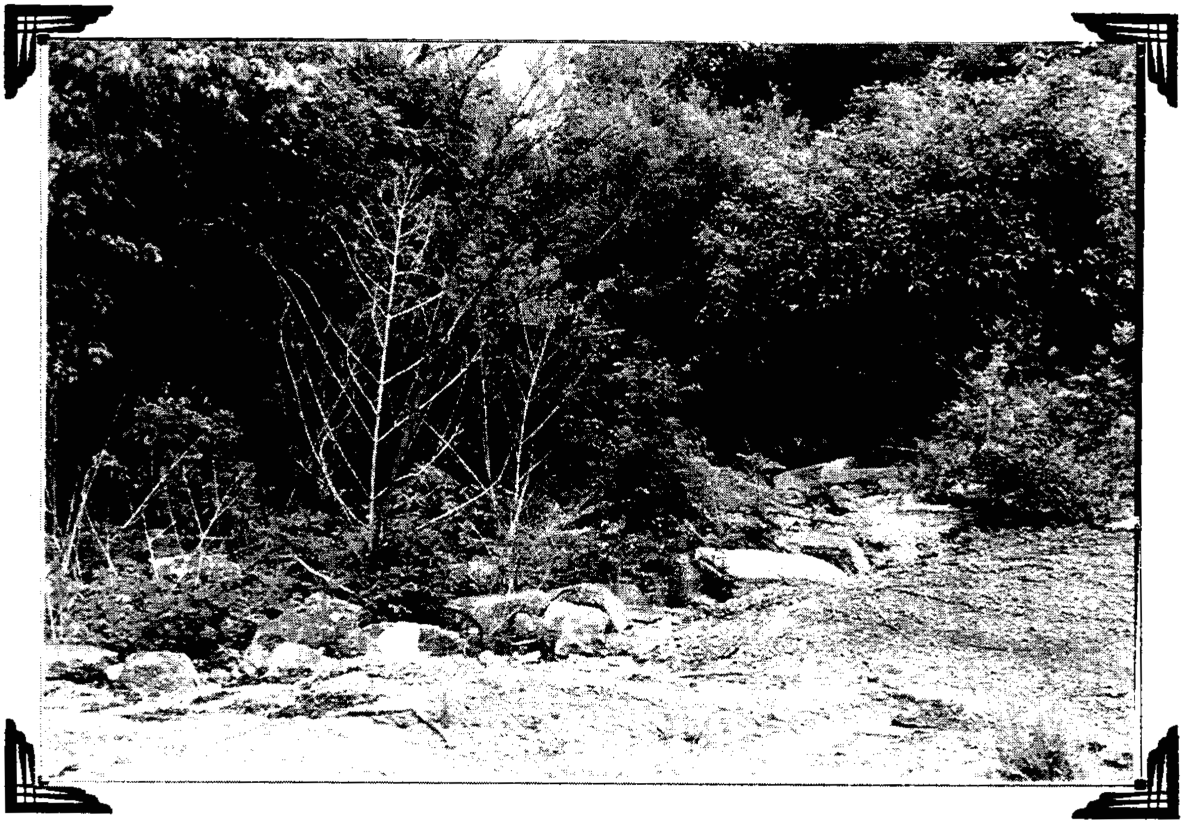

JONES
MUD CREEK
STARCREST
OAKS
SALADO
NACOGDOCHES
PERRIN-BEITEL
BEITEL CREEK
I.H.35 N
WEIDNER
WETMORE
LOOP 410
CREEK
WALZEM
U.S. 281 N
HWY.
410
EISENHAUER



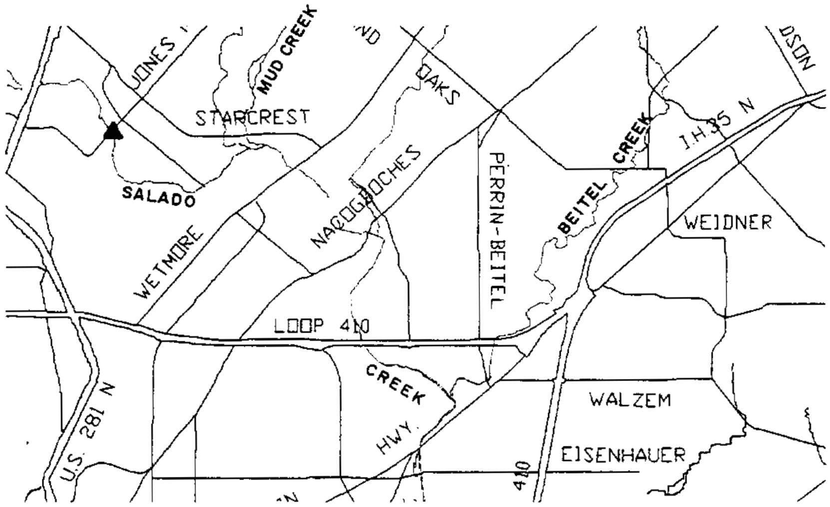
BITTERS RD. UPSTREAM

BITTERS RD. DOWNSTREAM



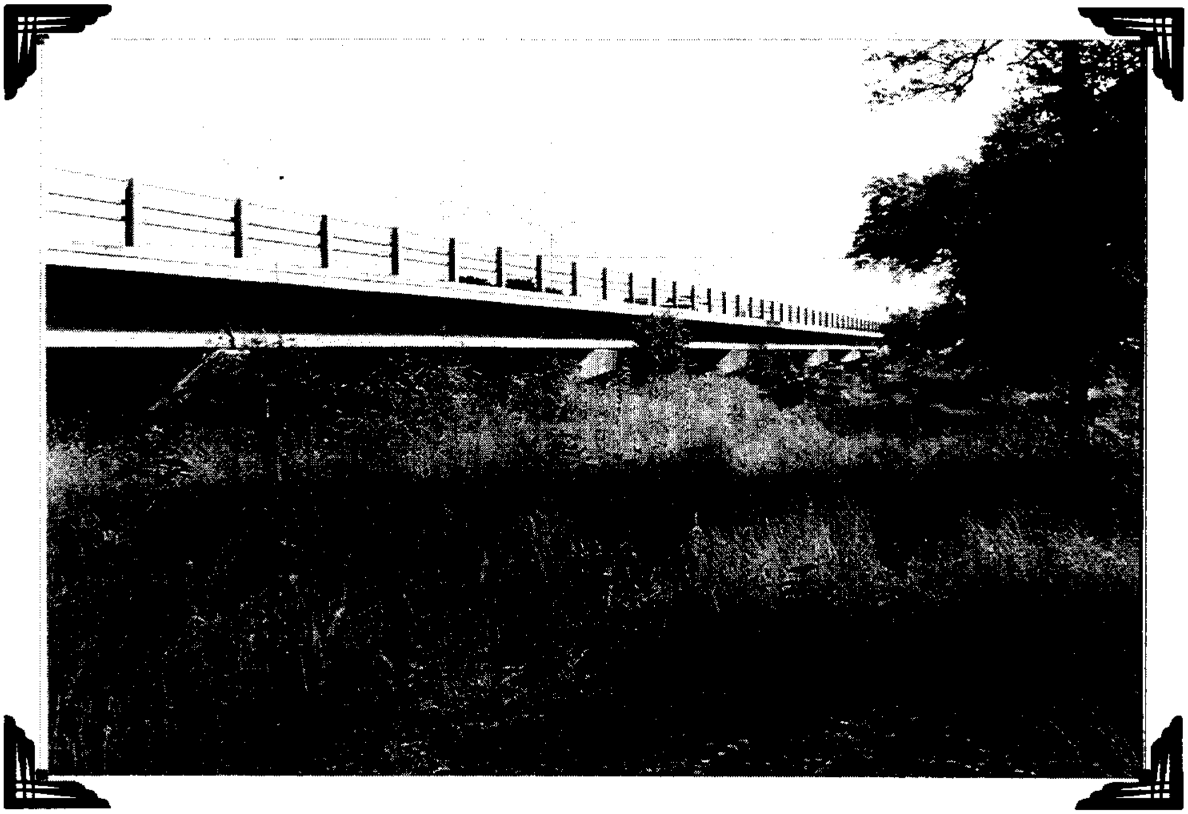


JONES MALTSBERGER RD.

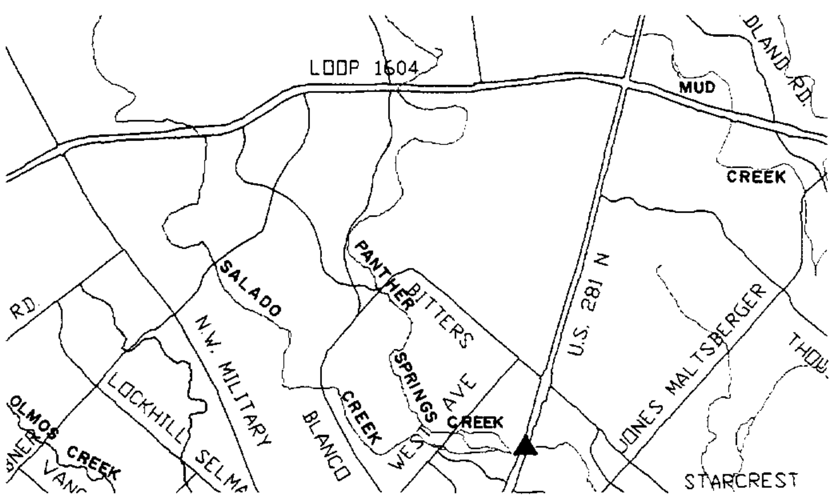


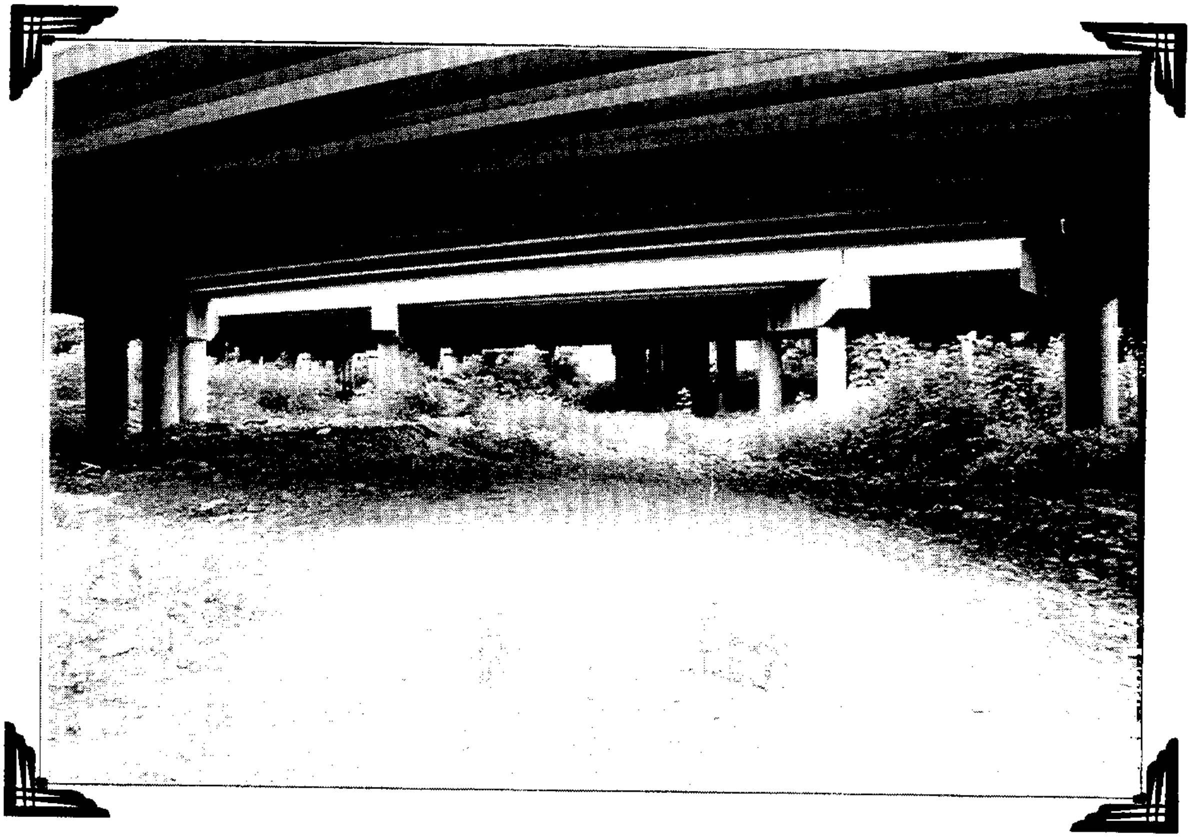
JONES MALTSBERGER RD.

JONES MALTSBERGER RD. UPSTREAM



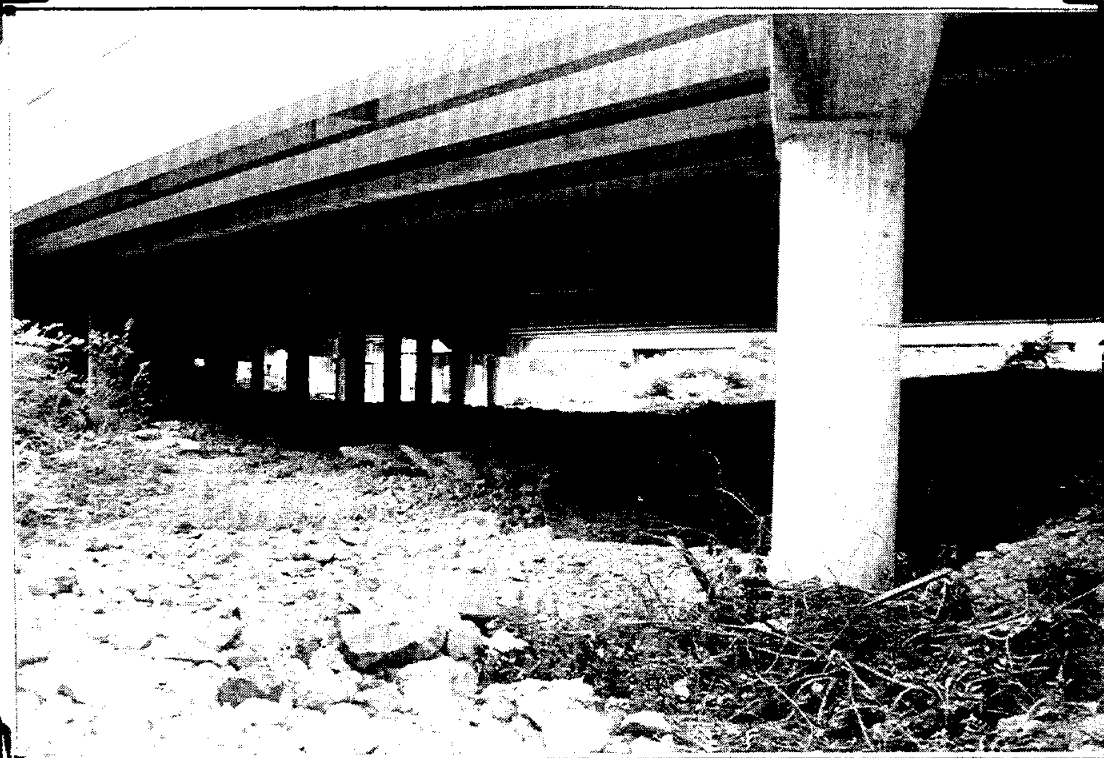
U. S. HWY. 281 N.


LOOP 1604
MUD
CREEK
U.S. 281 N
JONES MALTSBERGER
STARCREST
BITTERS
PANTHER
SPRINGS
CREEK
AVE
WEST
BLANCO
CREEK
SALADO
N.W. MILITARY
LOCKHILL
OLMOS
CREEK
RD.



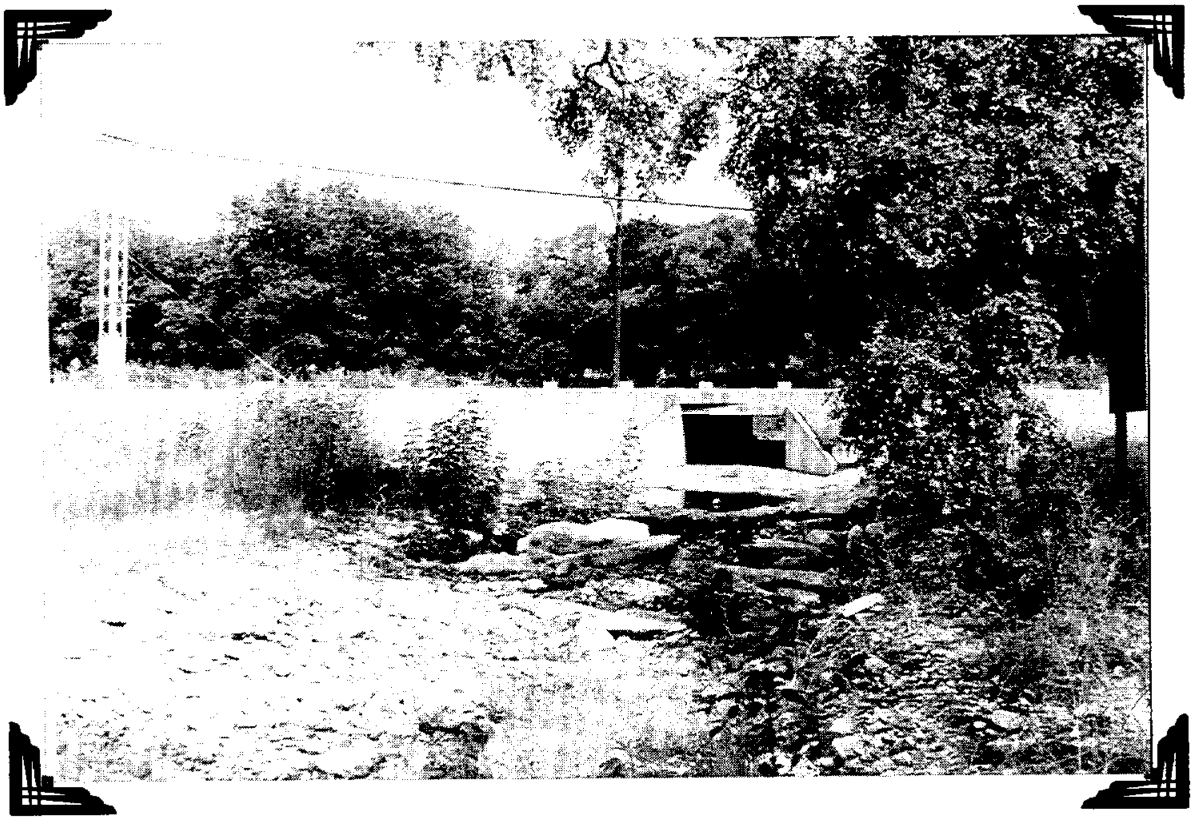
U.S. HWY. 281 N. UPSTREAM

U.S. HWY. 281 N. UPSTREAM

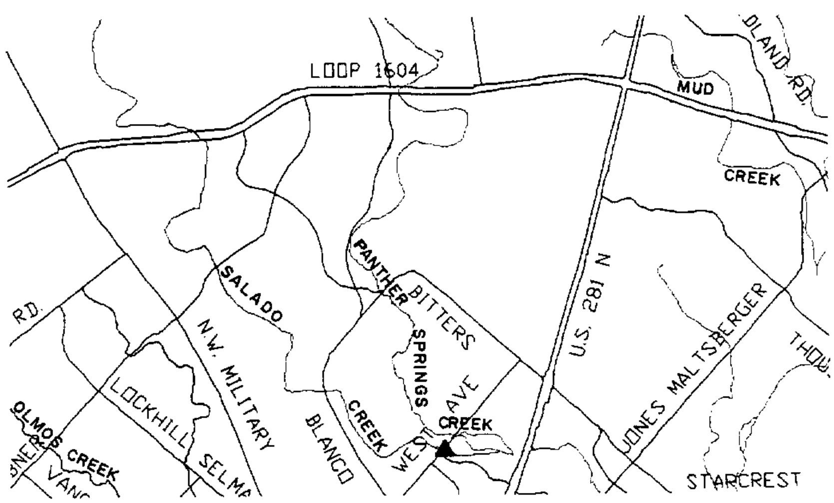


U.S. HWY. 281 N. DOWNSTREAM

U.S. HWY. N. 281 N. DOWNSTREAM

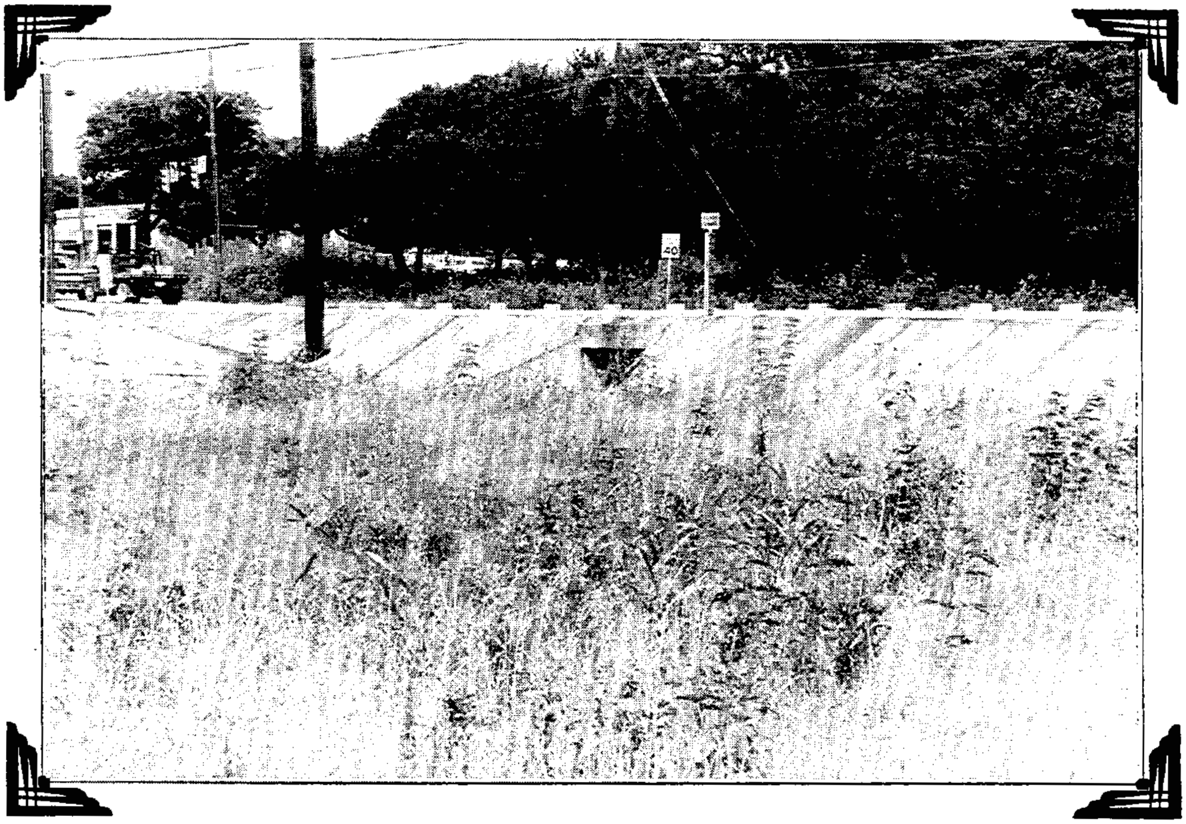


WEST AVE.

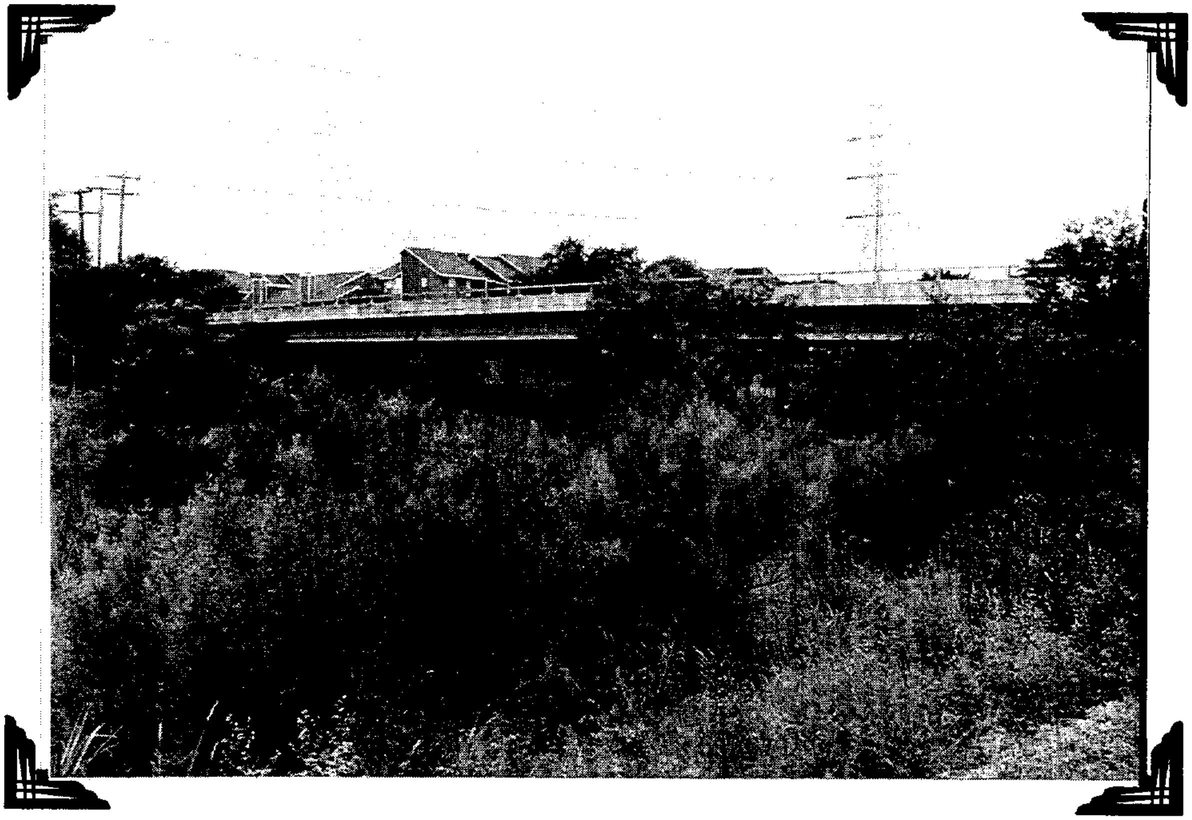

LOOP 1604
MUD
CREEK
SALADO
PANTHER
SPRINGS
CREEK
BITTERS
U.S. 281 N
JONES MALTSBERGER
RD.
N.W. MILITARY
LOCKHILL SELMA
BLANCO
WEST AVE
CREEK
OLMOS CREEK
STARCREST

WEST AVE. UPSTREAM

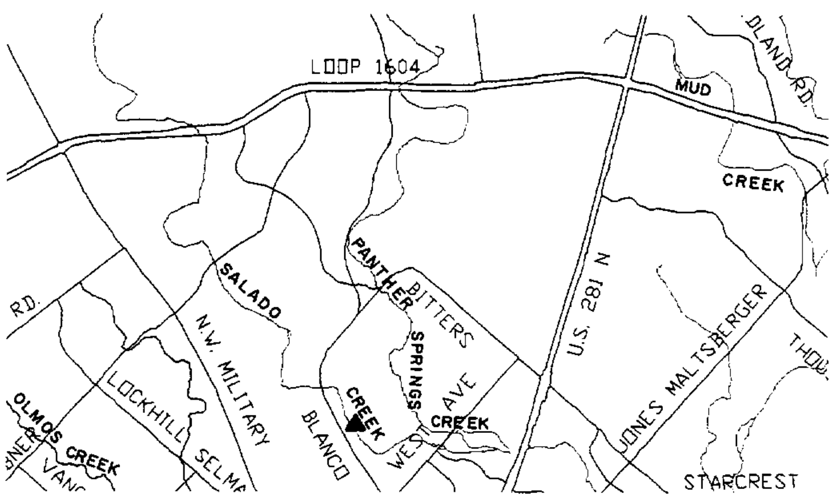
WEST AVE. DOWNSTREAM



WEST AVE. 2nd BOX CULVERT

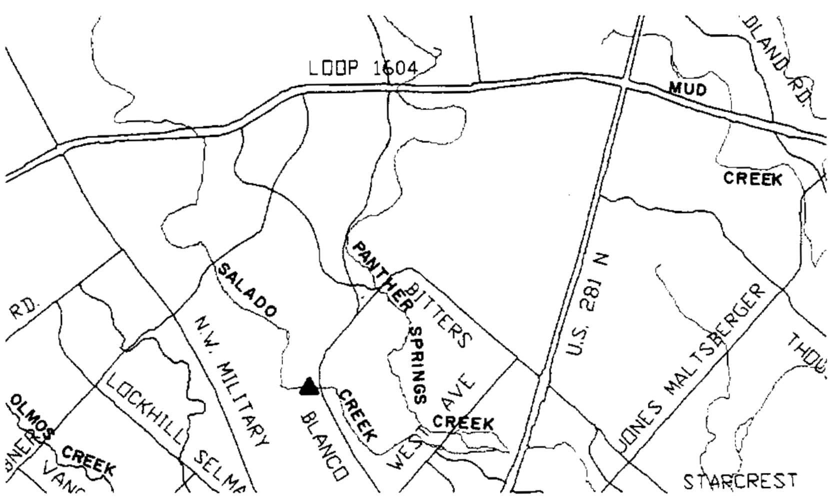


VISTA DEL NORTE

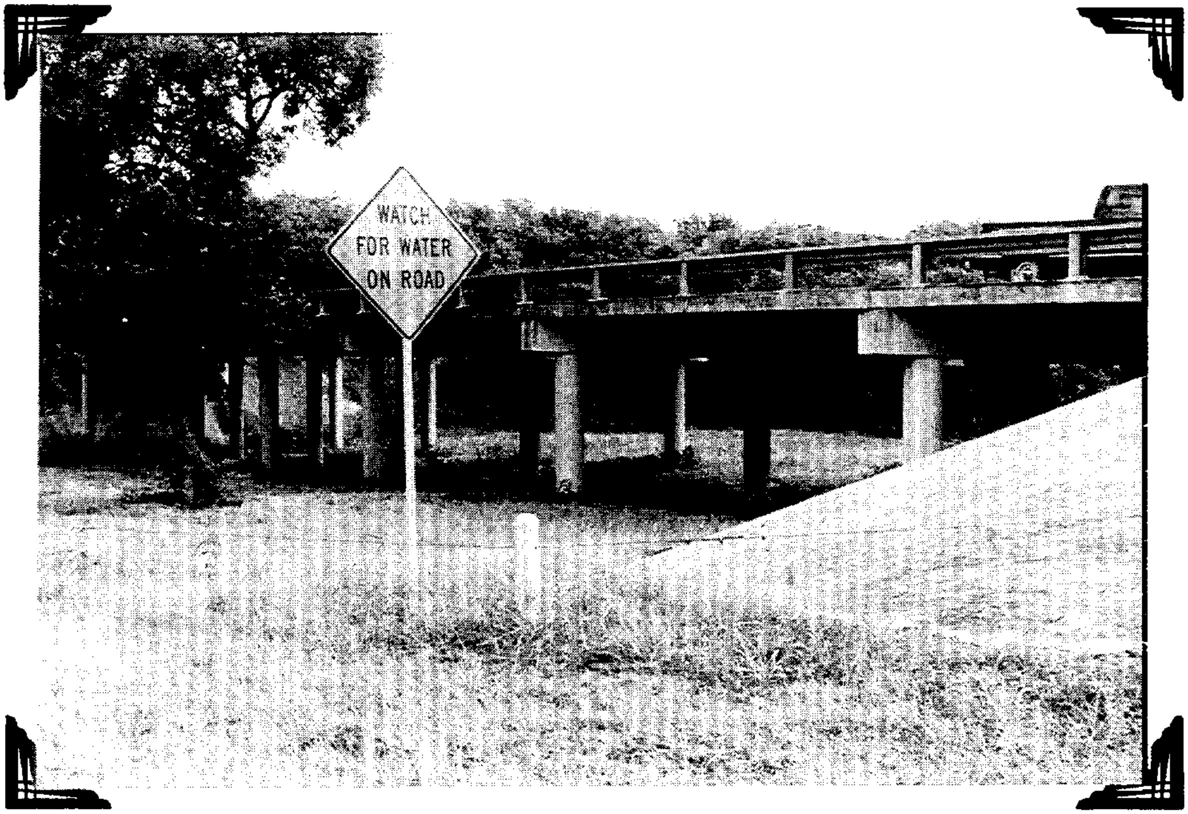


VISTA DEL NORTE UPSTREAM

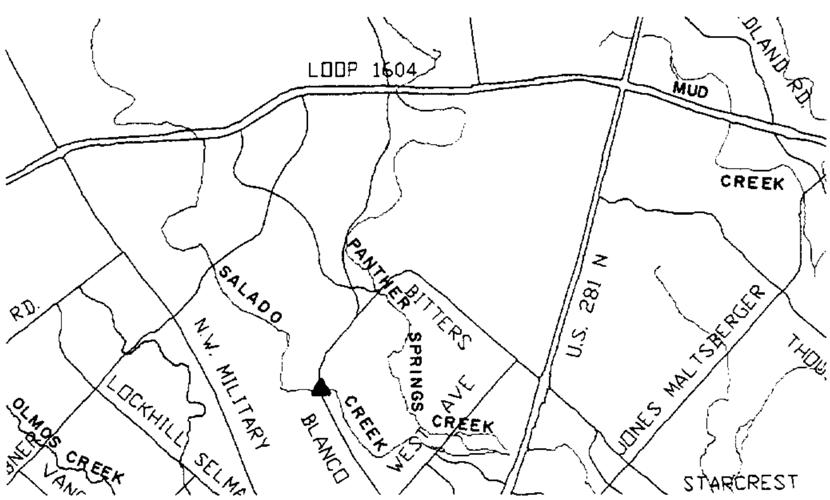
OLD BLANCO RD.

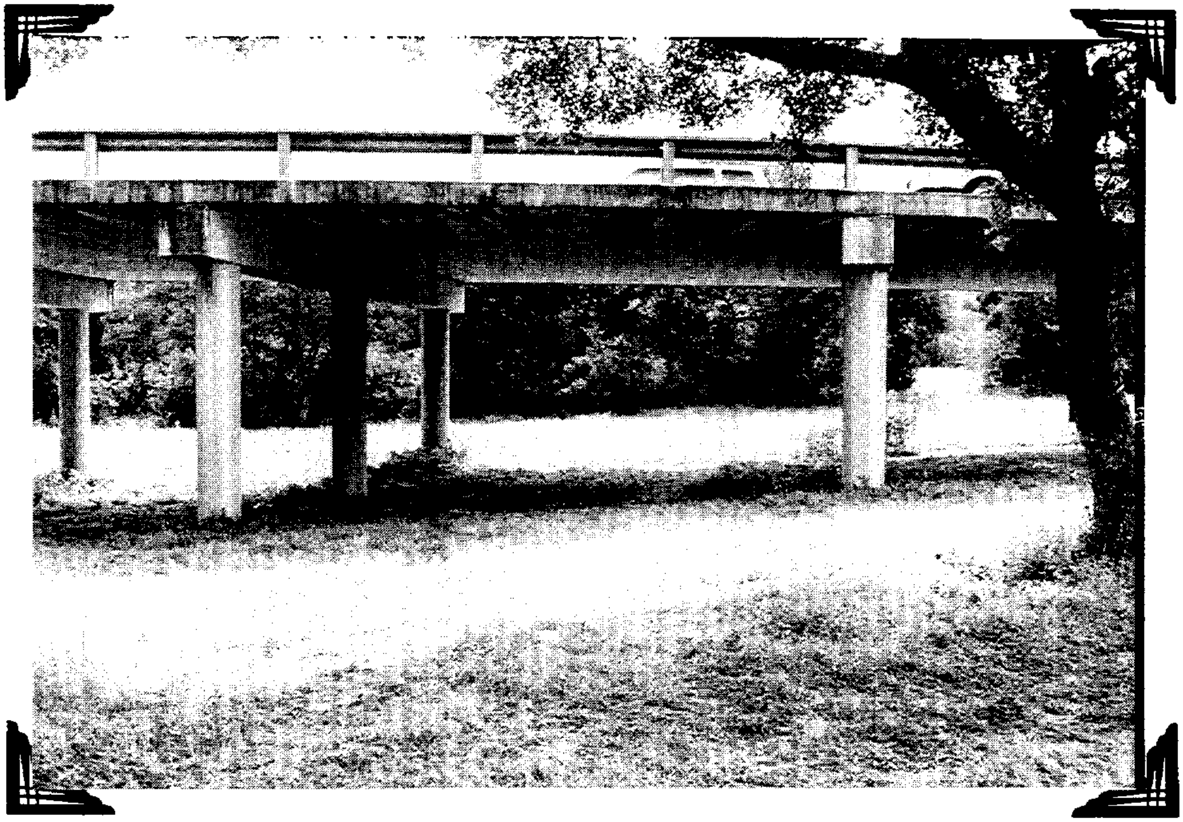


OLD BLANCO RD. UPSTREAM


WATCH
FOR WATER
ON ROAD

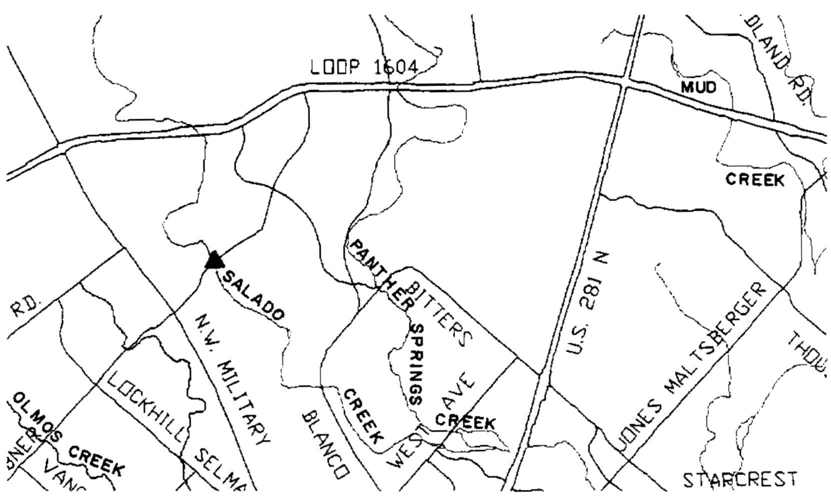
BLANCO RD.

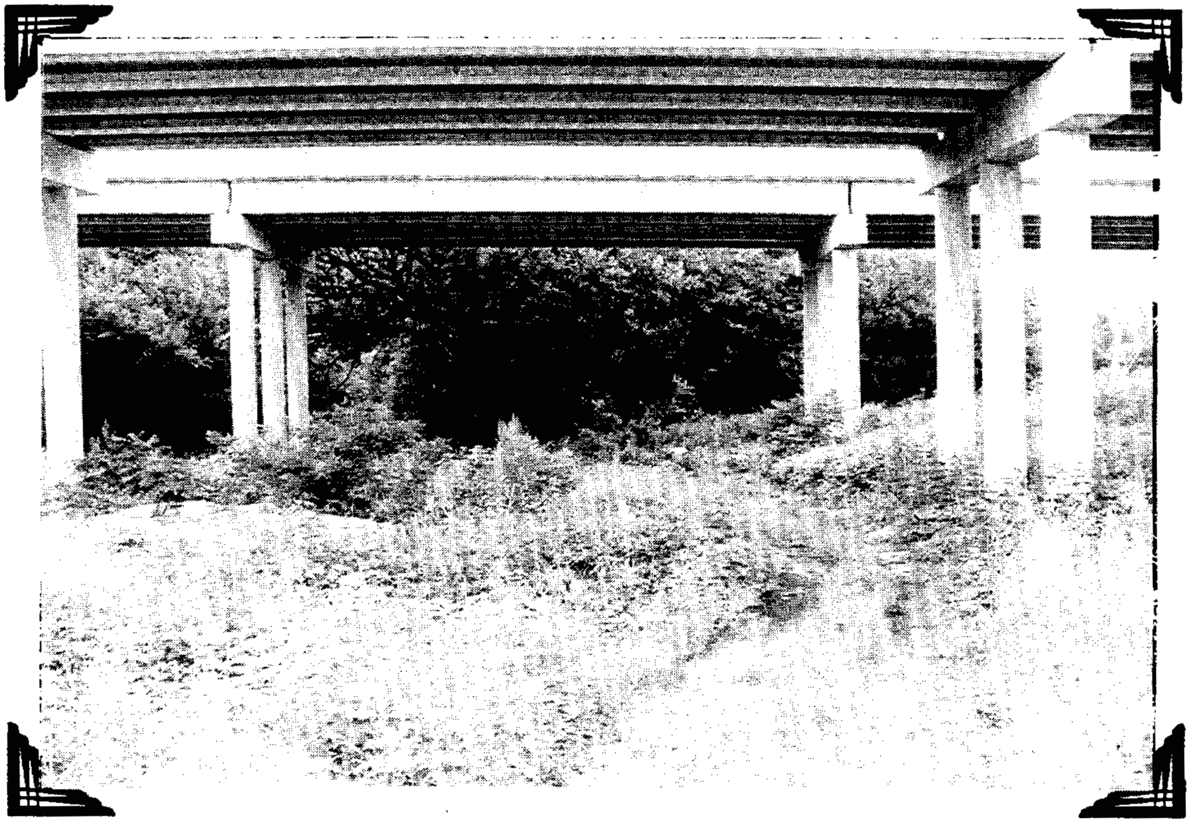

LOOP 1604
MUD
CREEK
SALADO
PANTHER
SPRINGS
CREEK
BITTERS
U.S. 281 N
JONES MALTSBERGER
RD.
N.W. MILITARY
LOCKHILL
BLANCO
CREEK
WEST AVE
CREEK
OLMOS CREEK
STARCREST



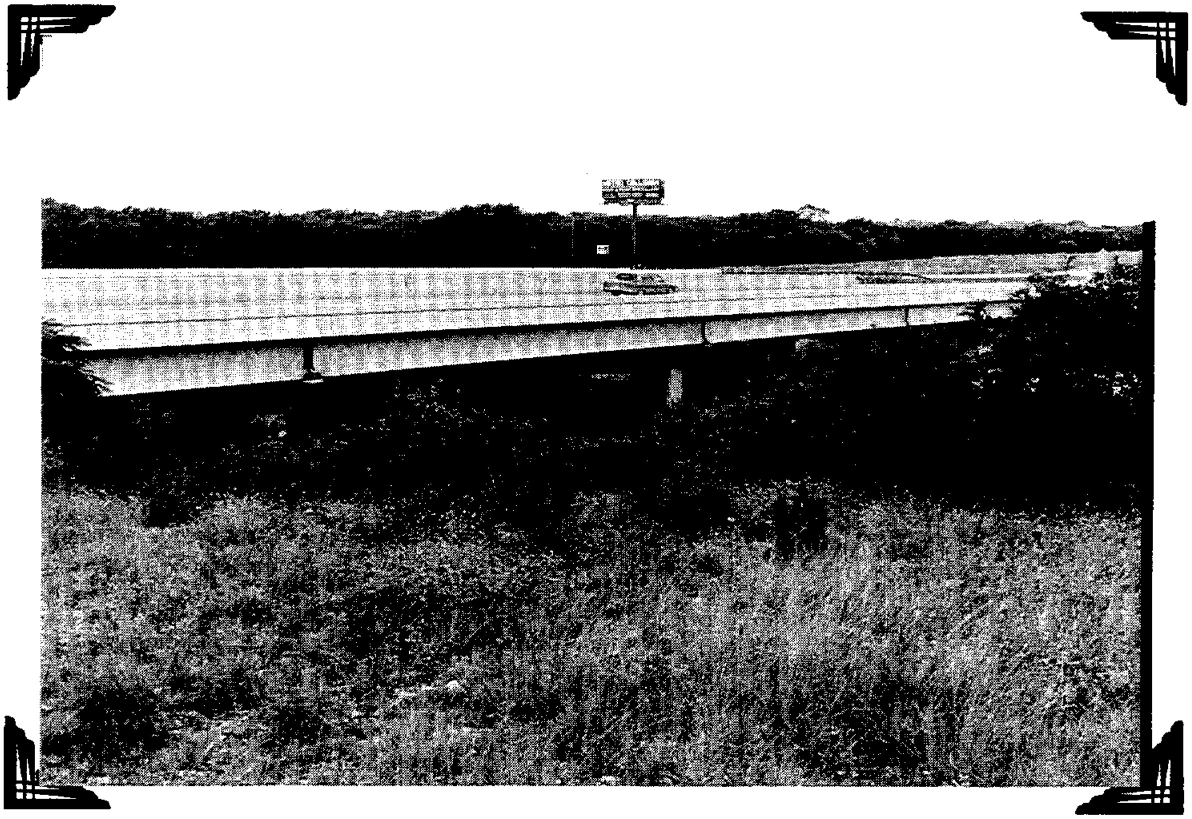
BLANCO RD. DOWNSTREAM

HUEBNER RD.

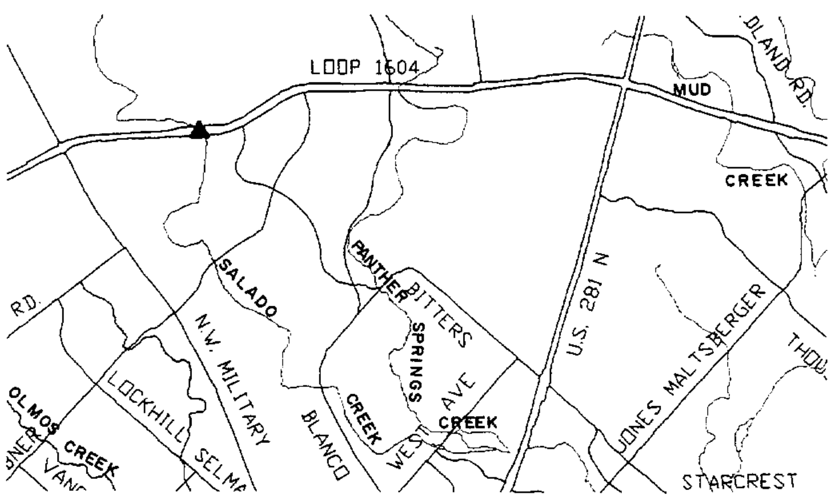

LOOP 1604
MUD
CREEK
SALADO
PANTHER
SPRINGS
CREEK
BITTERS
N.W. MILITARY
LOCKHILL SELMA
BLANCO
WEST AVE
CREEK
U.S. 281 N
JONES MALTSBERGER
STARCREST
OLMOS CREEK
RD.



HUEBNER RD. DOWNSTREAM



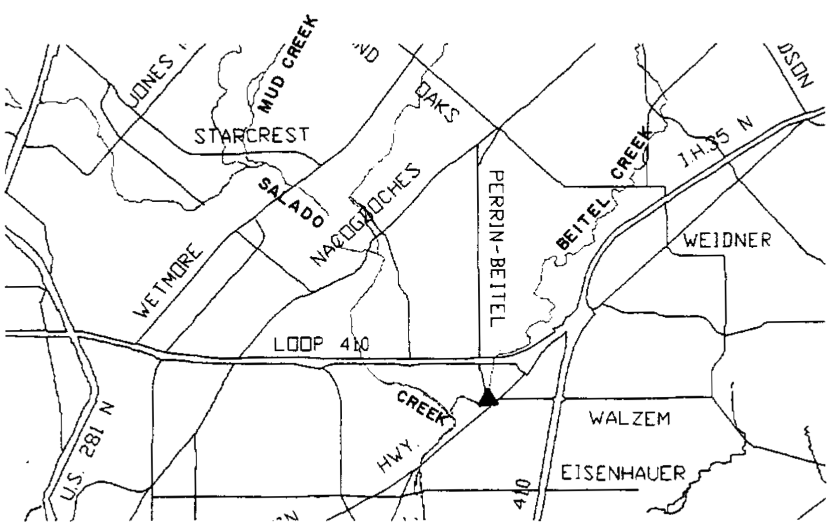
LOOP 1604


LOOP 1604
MUD
CREEK
RD.
SALADO
PANTHER
BITTERS
SPRINGS
U.S. 281 N
N.W. MILITARY
LOCKHILL
OLMOS
CREEK
BLANCO
CREEK
WEST AVE
CREEK
JONES MALTSBERGER
STARCREST

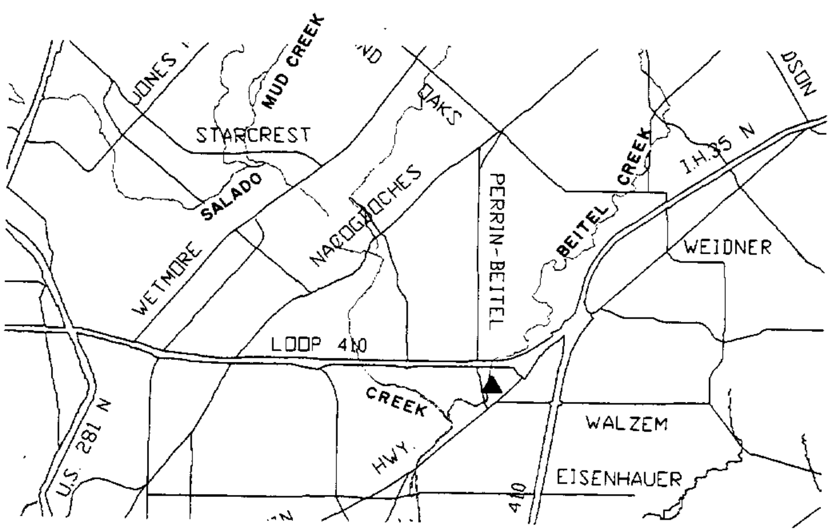
LOOP 1604 DOWNSTREAM

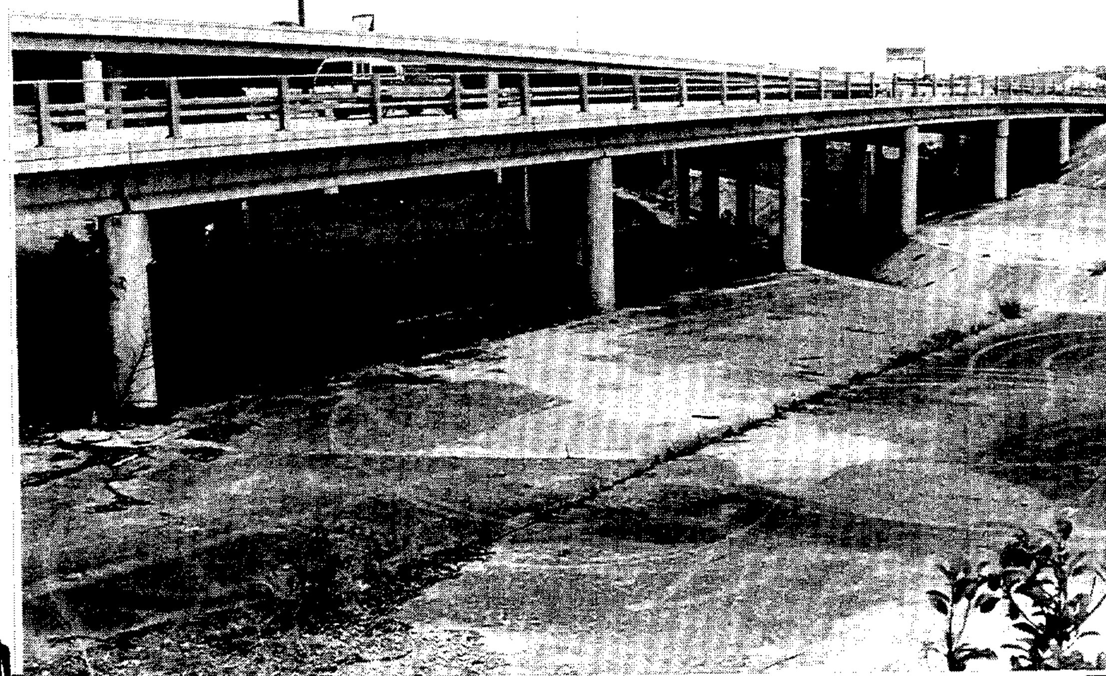
LOOP 1604 UPSTREAM

PERRIN-BEITEL RD.

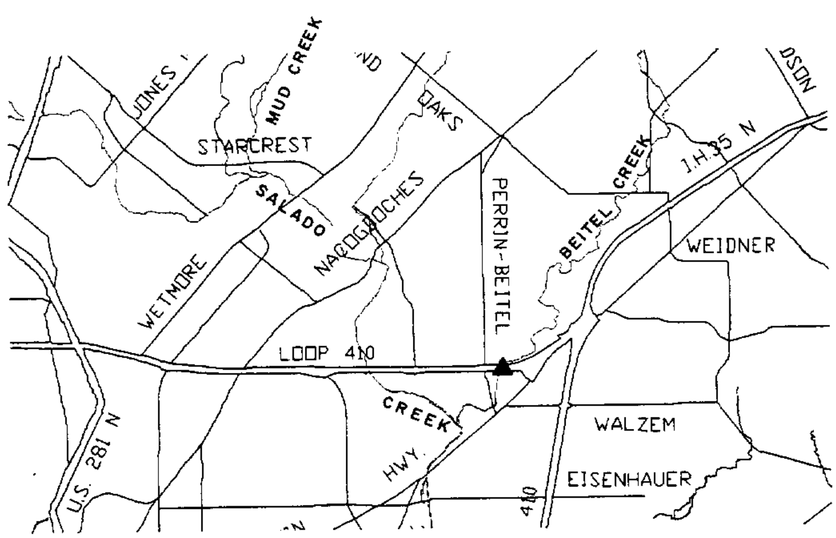


VICAR DR.


JONES
MUD CREEK
OAKS
STARCREST
SALADO
NACOGDOCHES
PERRIN-BEITEL
BEITEL
CREEK
I.H.35 N
WEIDNER
WETMORE
LOOP 410
CREEK
WALZEM
HWY.
U.S. 281 N
EISENHAUER
410



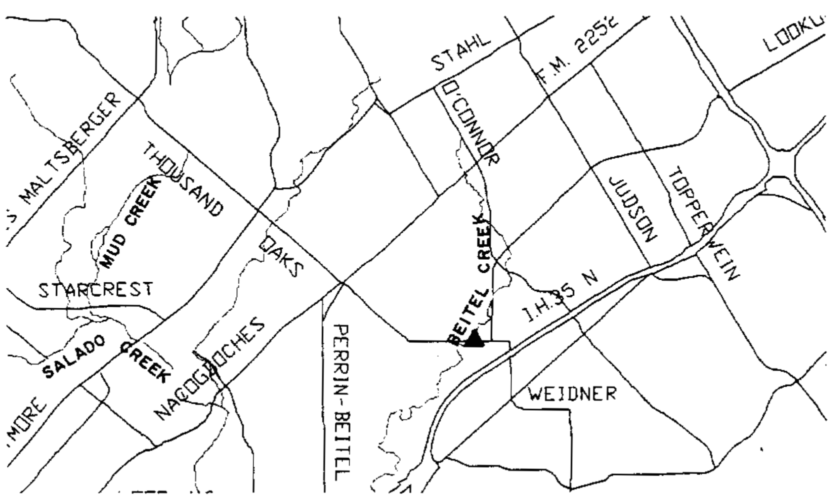
N. E. LOOP 410



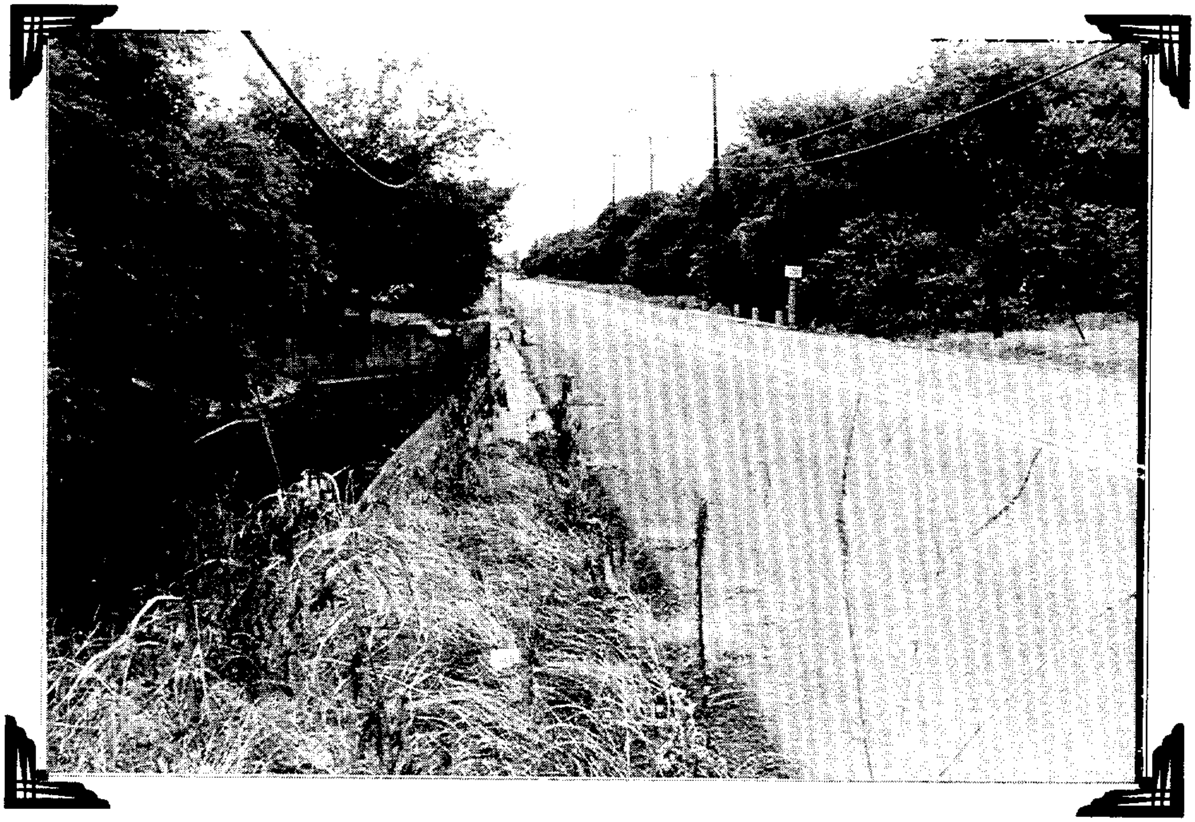
MUD CREEK
STARCREST
OAKS
SALADO
NACOGDOCHES
PERRIN-BEITEL
BEITEL CREEK
I.H. 35 N
WEIDNER
WETMORE
LOOP 410
CREEK
WALZEM
HWY.
EISENHAUER
U.S. 281 N
410

N.E. LOOP 410 UPSTREAM

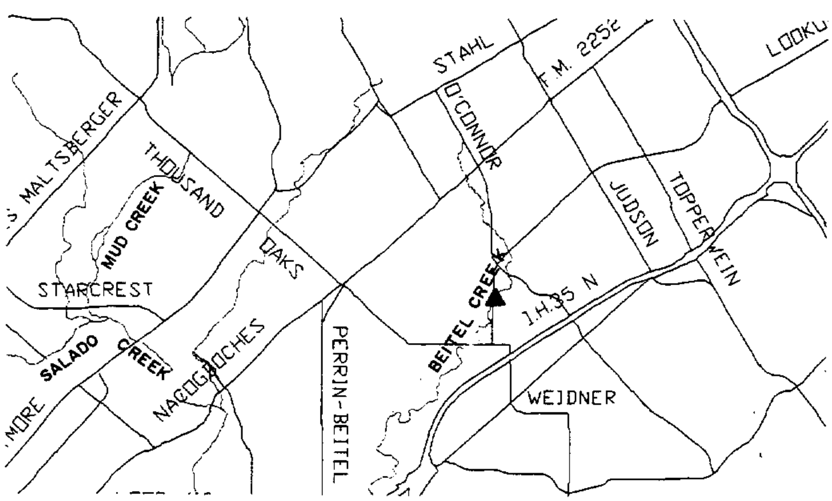
SHERTZ RD.


STAHL
O'CONNOR
MALTSBERGER
THOUSAND
MUD CREEK
OAKS
JUDSON
TOPPERWEIN
BEITEL CREEK
I.H.35 N
STARCREST
SALADO CREEK
NACOGDOCHES
PERRIN-BEITEL
WEIDNER

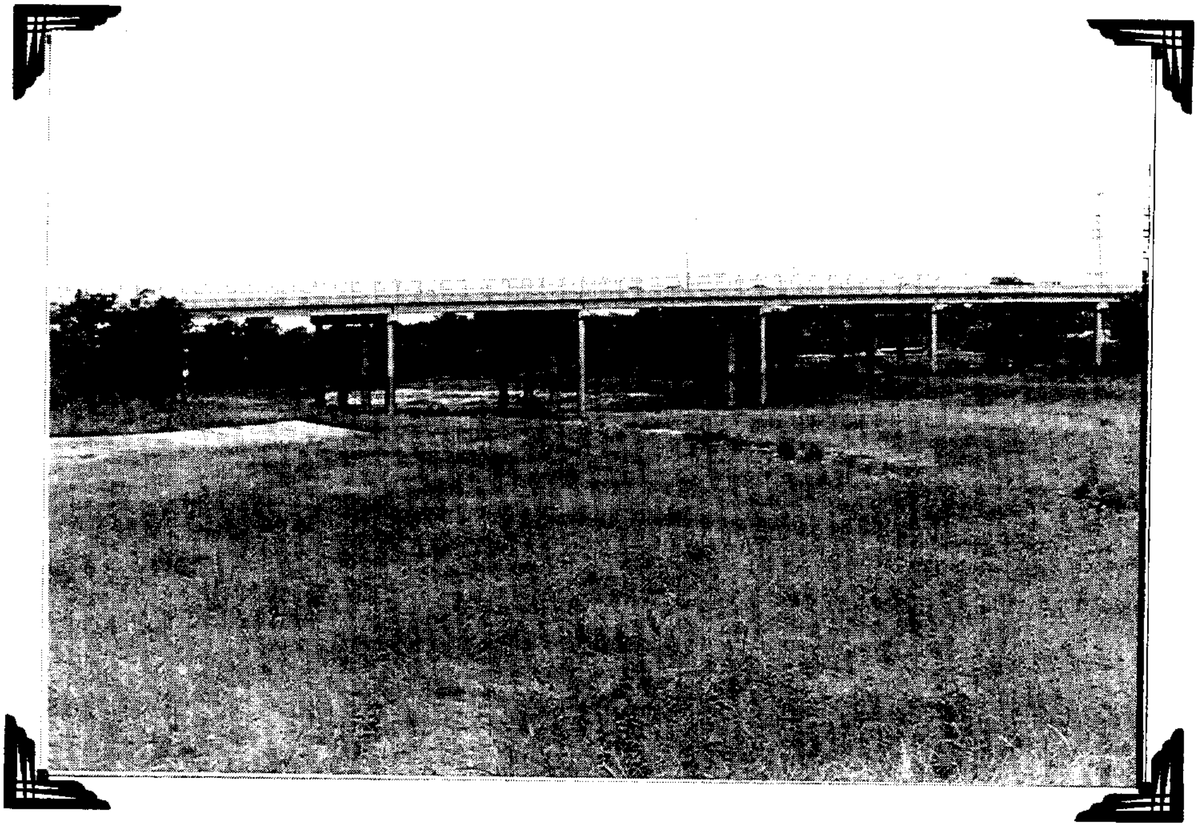
SCHERTZ RD. UPSTREAM



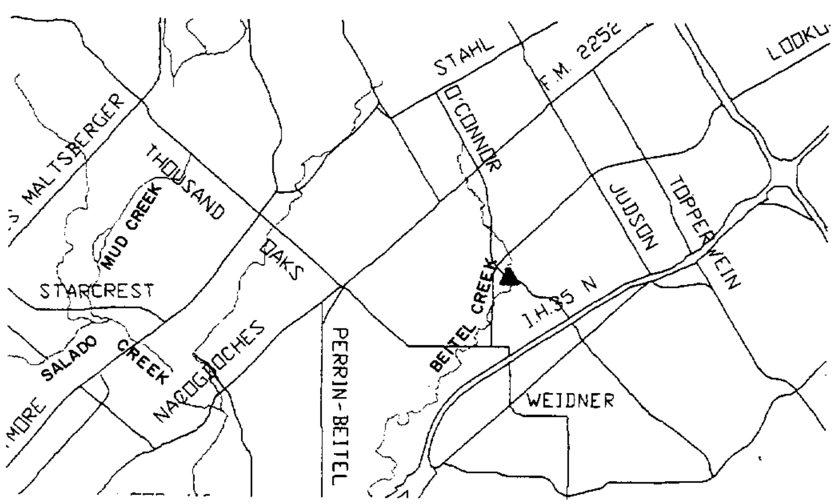
WEIDNER RD.



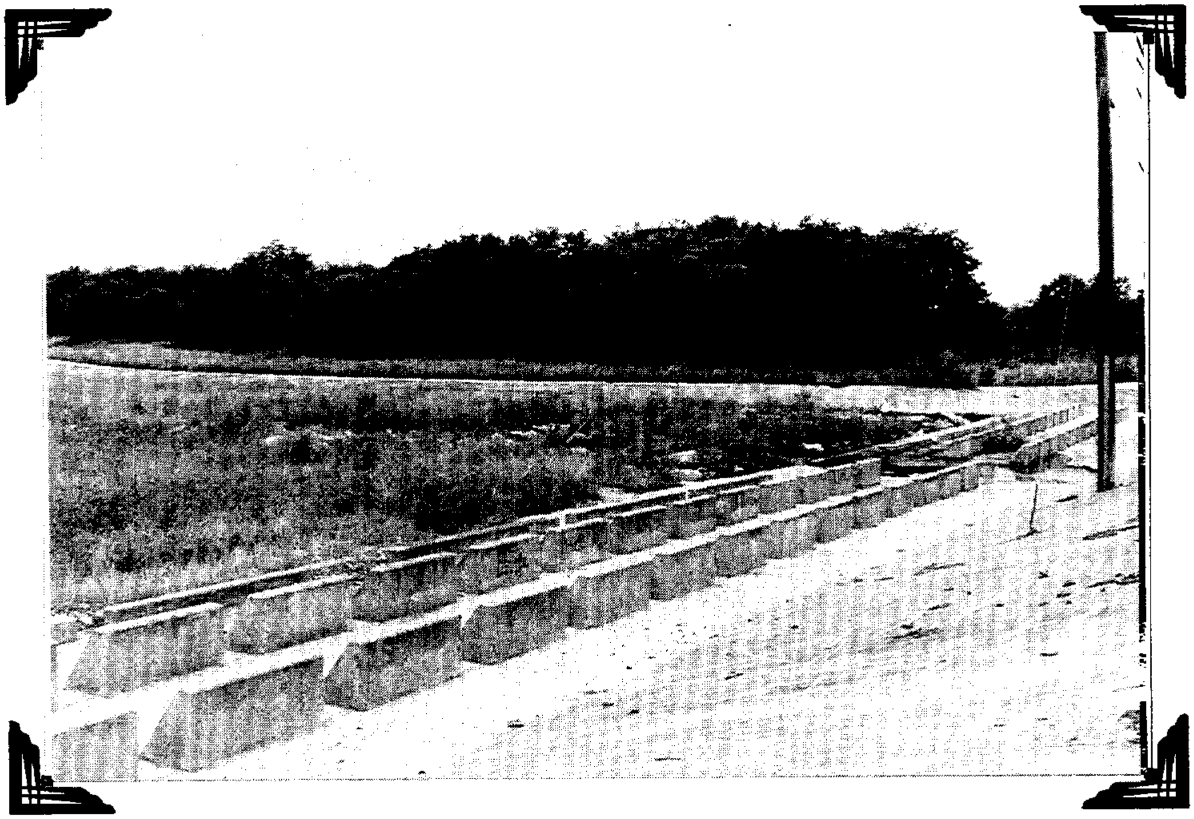
STAHL
F.M. 2252
O'CONNOR
S MALTSBERGER
THOUSAND
MUD CREEK
OAKS
JUDSON
TOPPERWEIN
STARCREST
BEITEL CREEK
I.H.35 N
SALADO CREEK
NACOGDOCHES
PERRIN-BEITEL
WEIDNER



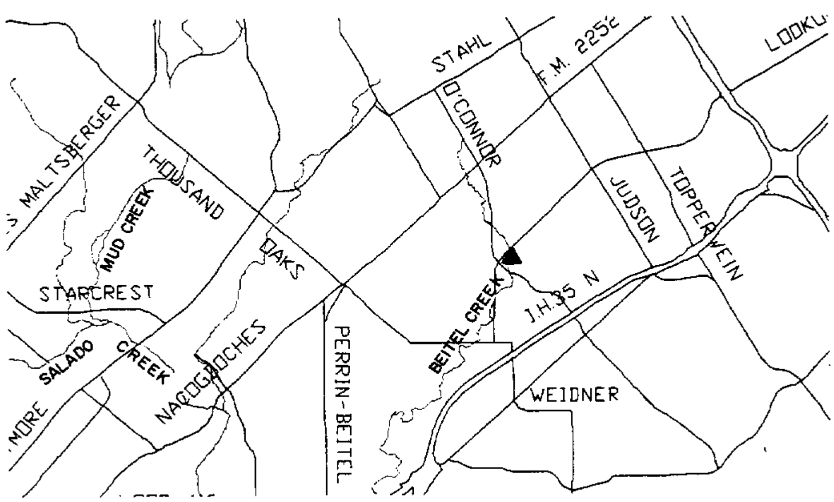
O'CONNOR RD.

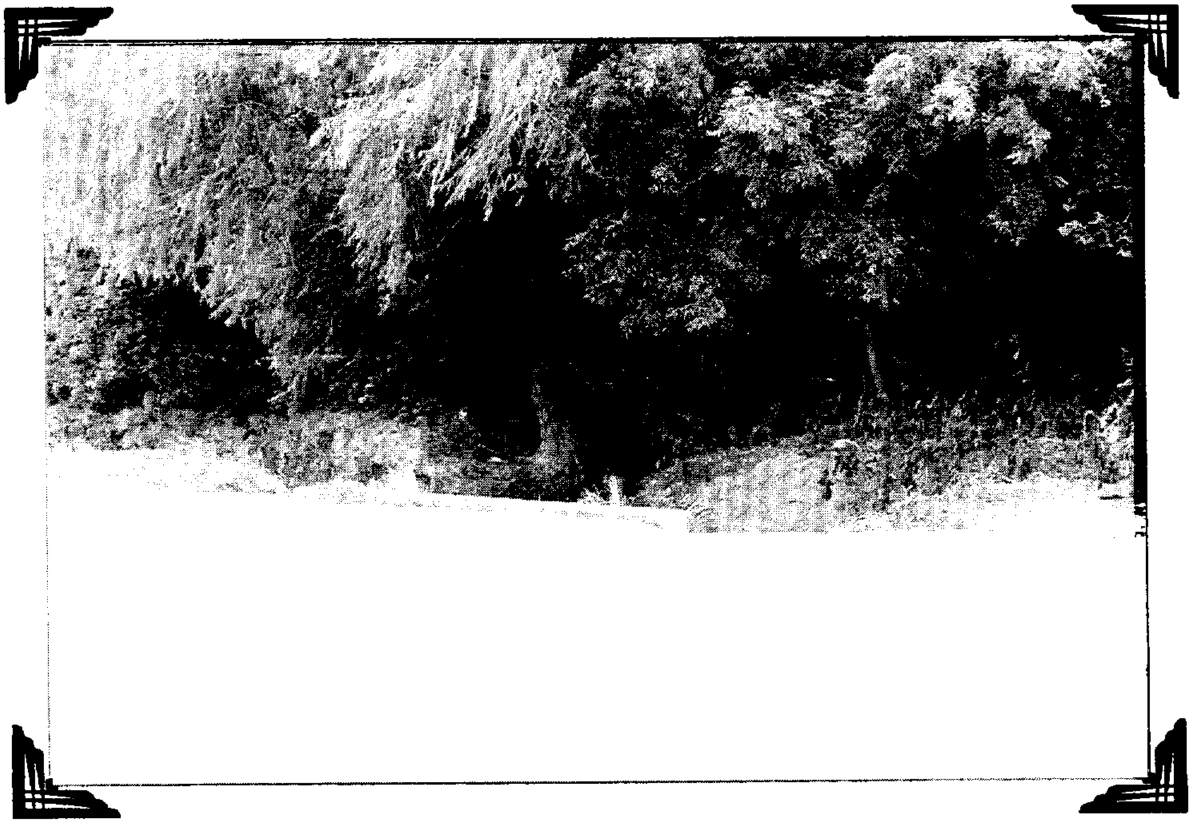


S MALTSBERGER
THOUSAND
OAKS
MUD CREEK
STARCREST
SALADO
CREEK
NACOGDOCHES
PERRIN-BEITEL
STAHL
O'CONNOR
F.M. 2252
JUDSON
TOPPERWEIN
BEITEL CREEK
I.H. 35 N
WEIDNER

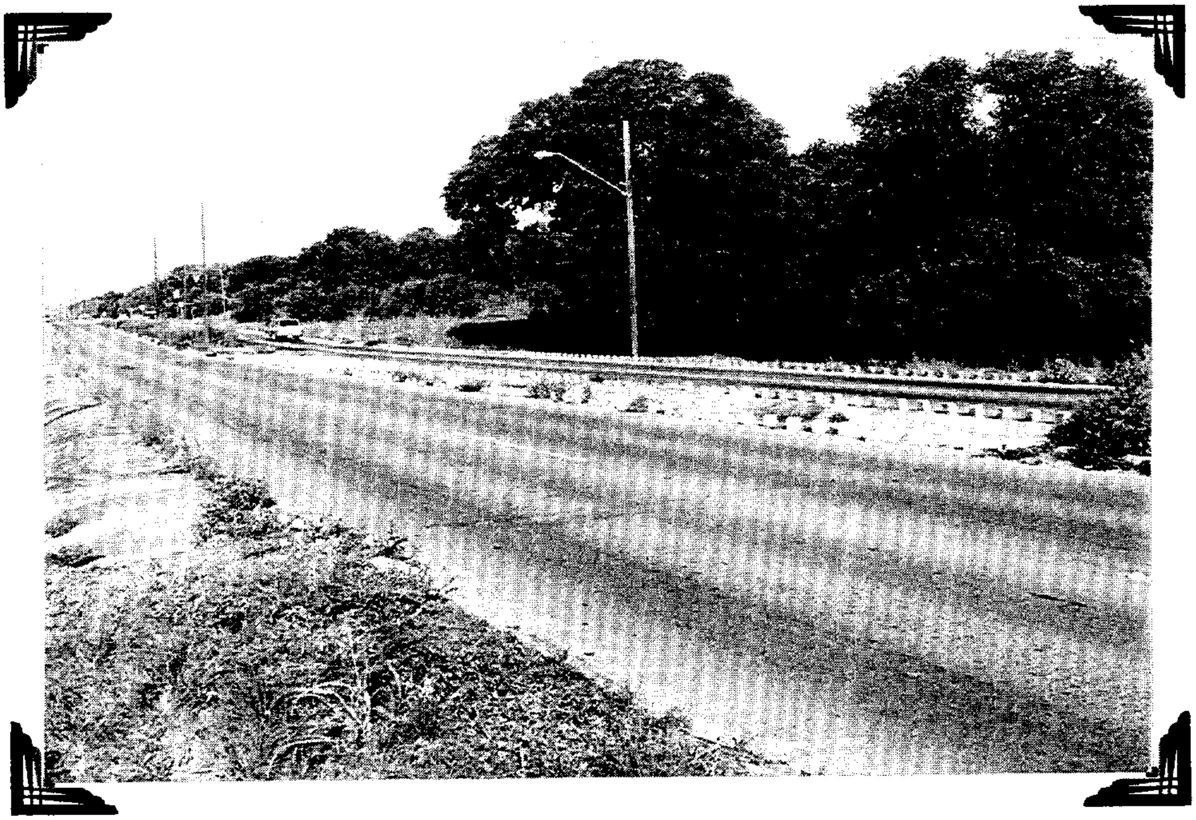


OLD O'CONNOR RD.



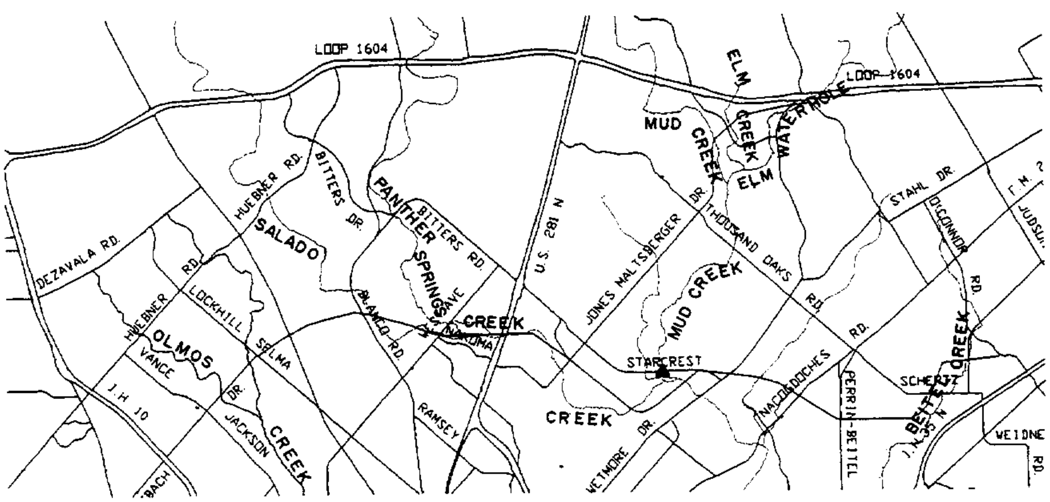


OLD O'CONNOR RD. UPSTREAM

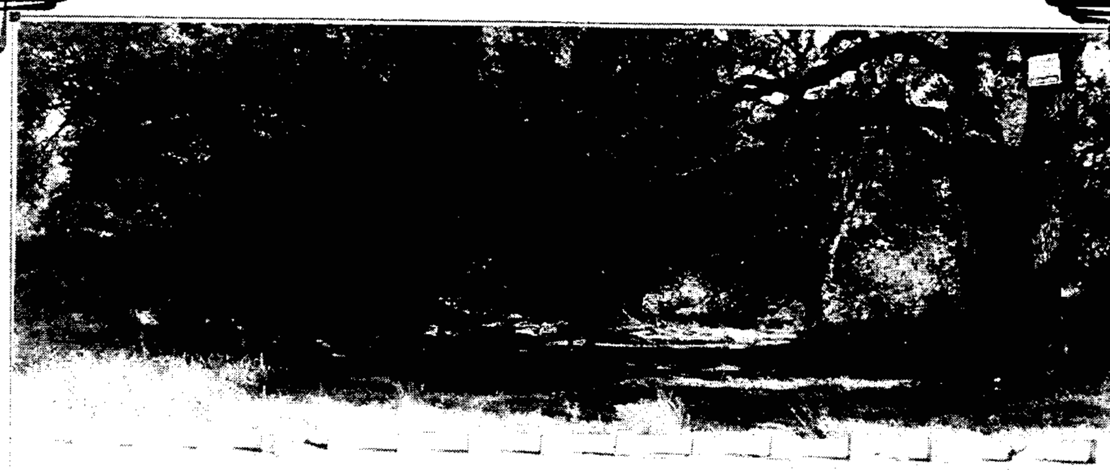


STARCREST DR.

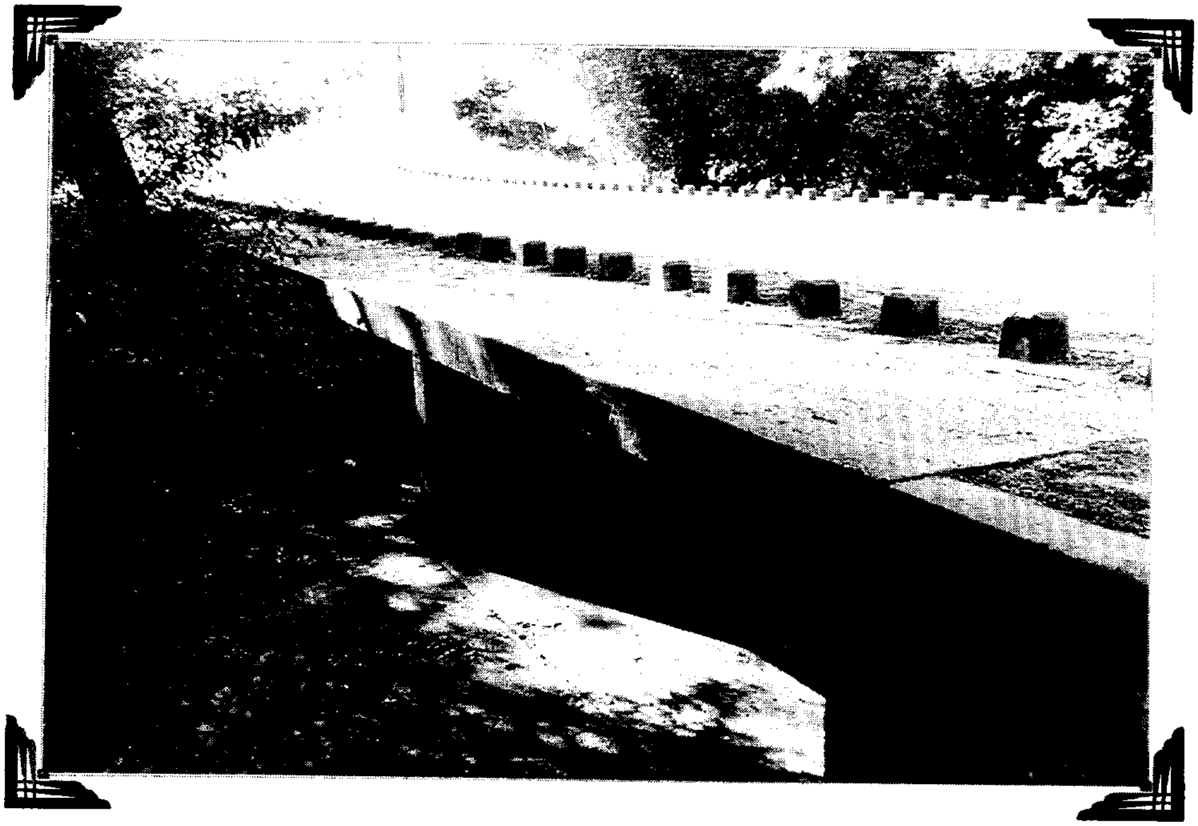
(WEST)



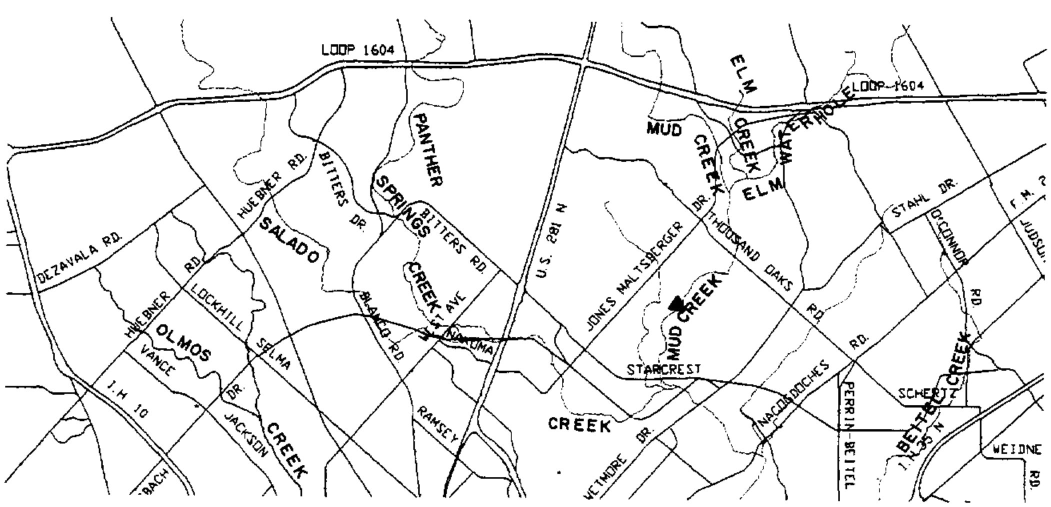
STARCREST UPSTREAM

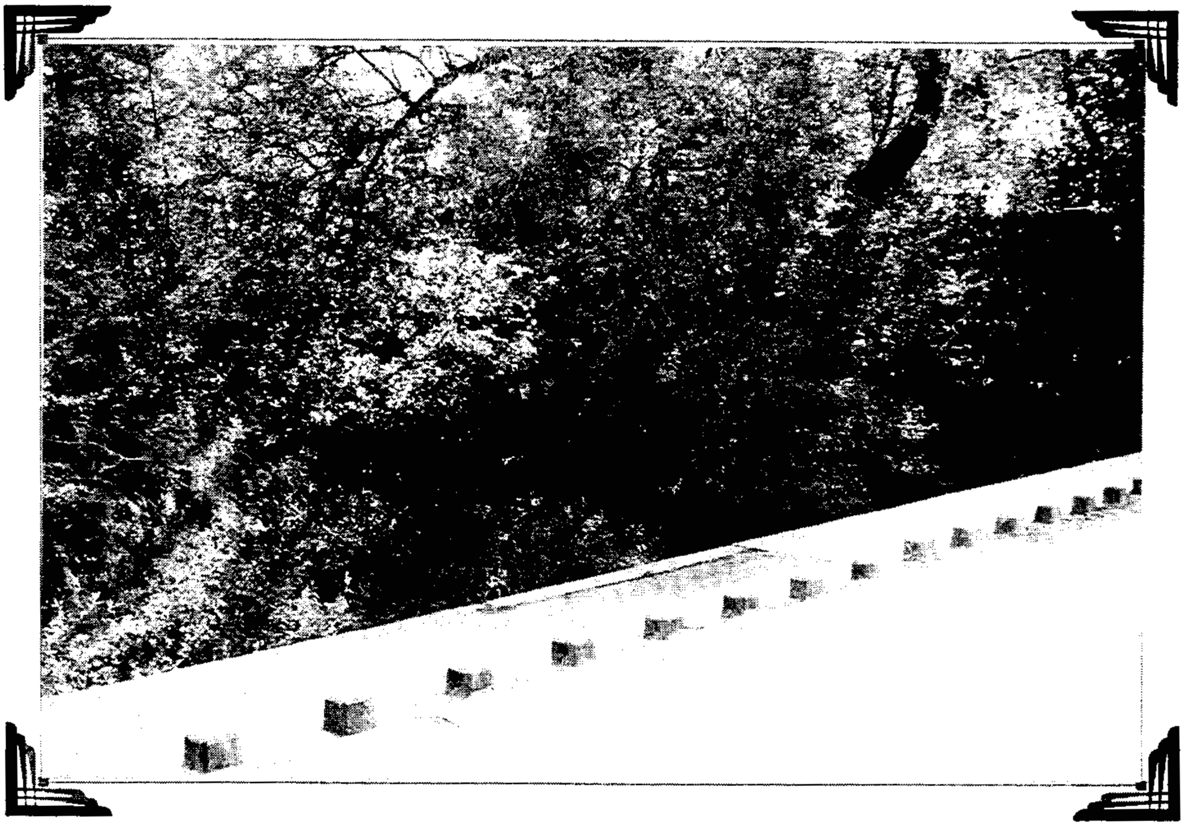


STARCREST DOWNSTREAM

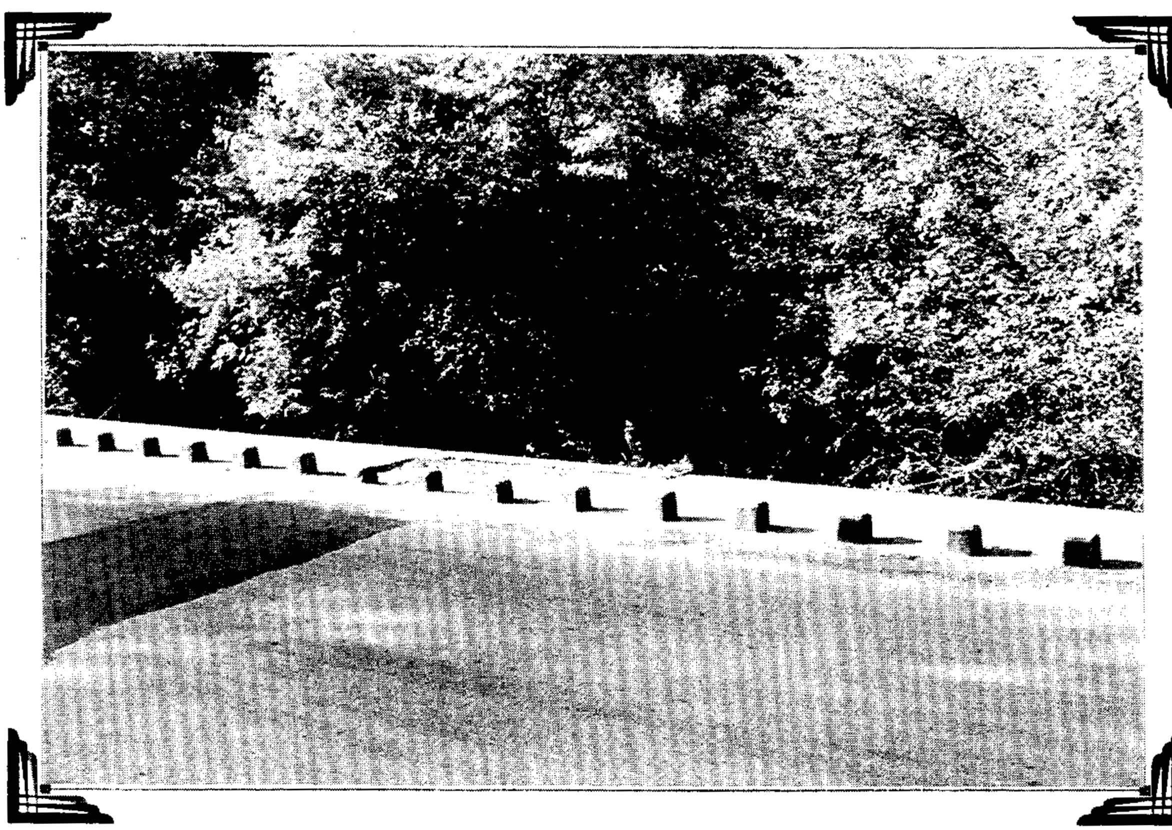


BUCKHORN RD.

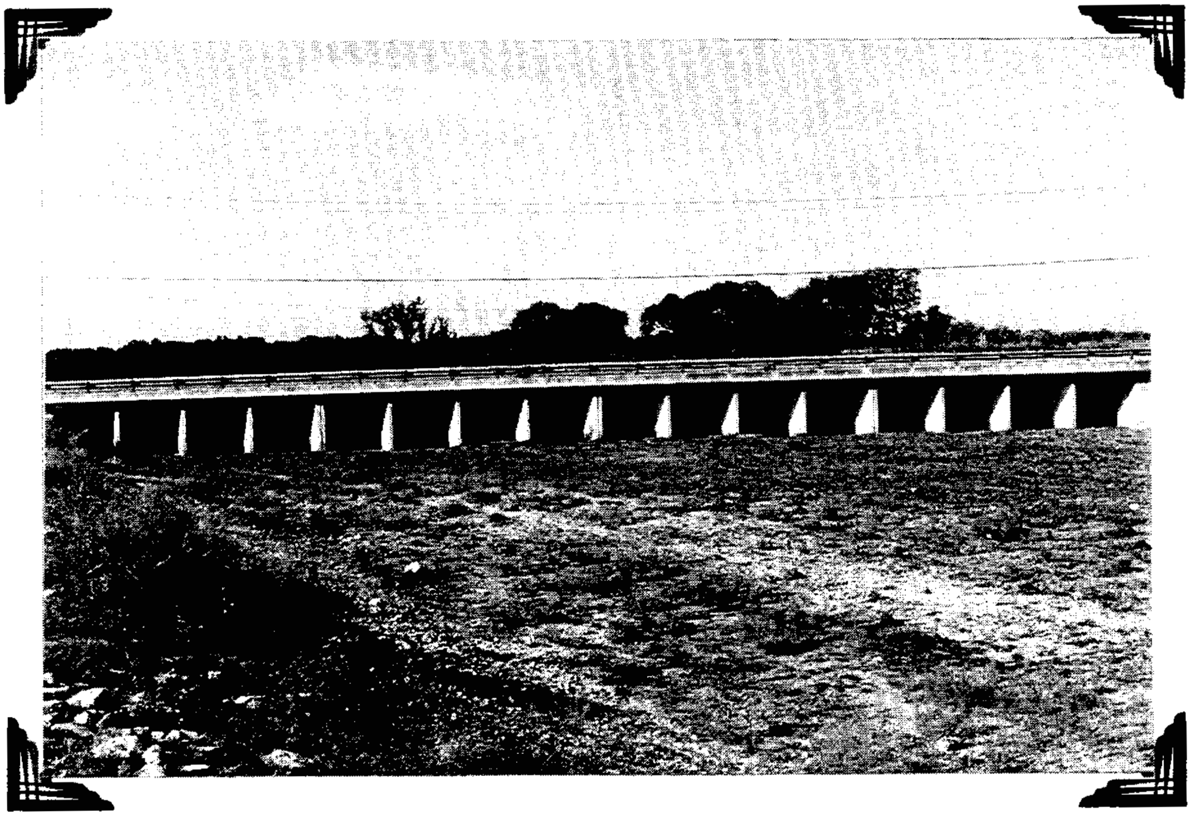

LOOP 1604
LOOP 1604
ELM
MUD
CREEK
CREEK
WATERHOLE
ELM
PANTHER
SPRINGS
CREEK
HUEBNER RD.
BITTERS RD.
BITTERS DR.
BITTERS RD.
SALADO
DEZAVALA RD.
U.S. 281 N
JONES MALTSBERGER DR.
THOUSAND OAKS RD.
STAHL DR.
O'CONNOR RD.
JUDSON
MUD CREEK
HUEBNER RD.
LOCKHILL SELMA
OLMOS
VANCE DR.
BLANCO RD.
AVE.
NAKOMA
STARCREST
NACOGDOCHES RD.
PERRIN-BEITEL
SCHERTZ
BEITEL CREEK
I.H. 35 N
WEIDNER RD.
I.H. 10
JACKSON
CREEK
RAMSEY
CREEK
WETMORE DR.



BUCKHORN UPSTREAM

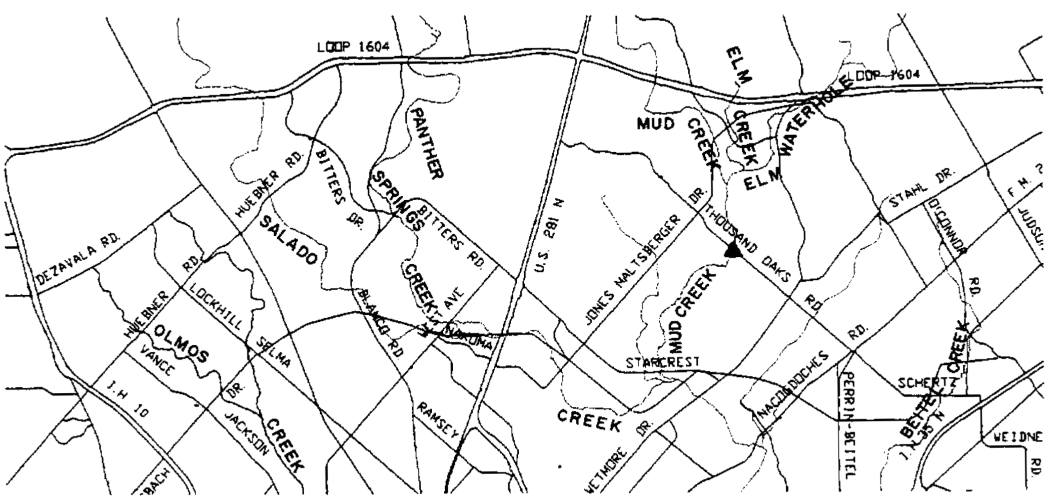


BUCKHORN DOWNSTREAM



THOUSAND OAKS

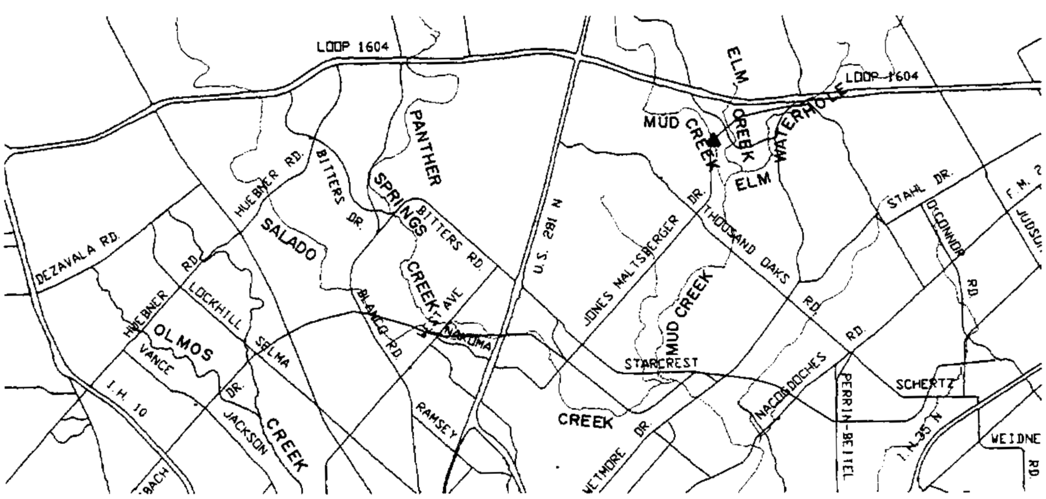
DOWNSTREAM



THOUSAND OAKS UPSTREAM

JONES MALTSBERGER

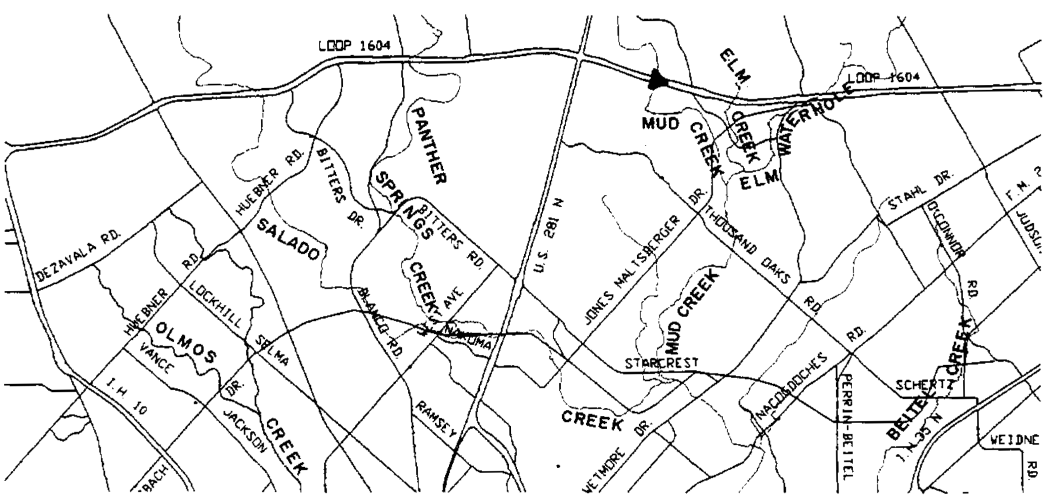
UPSTREAM



JONES MALTSBERGER DOWNSTREAM

LOOP 1604

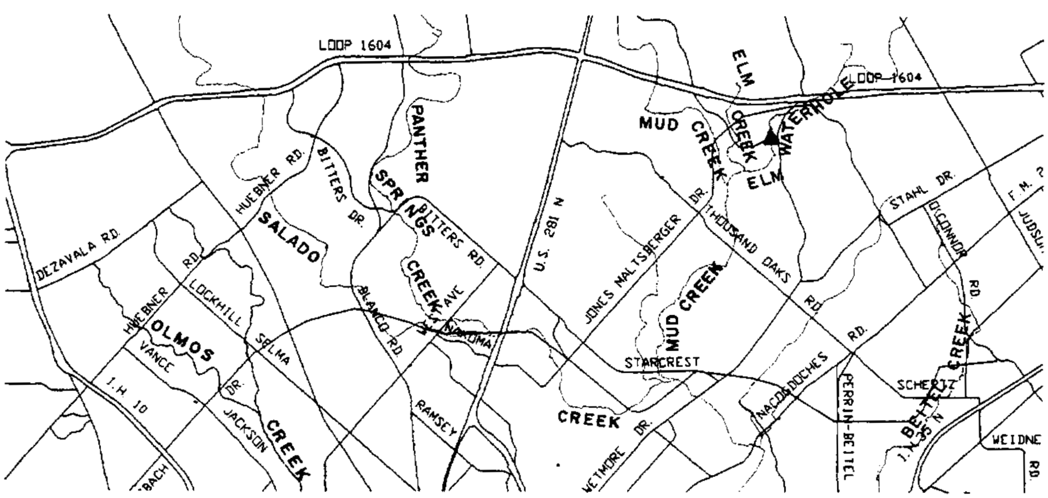
DOWNSTREAM



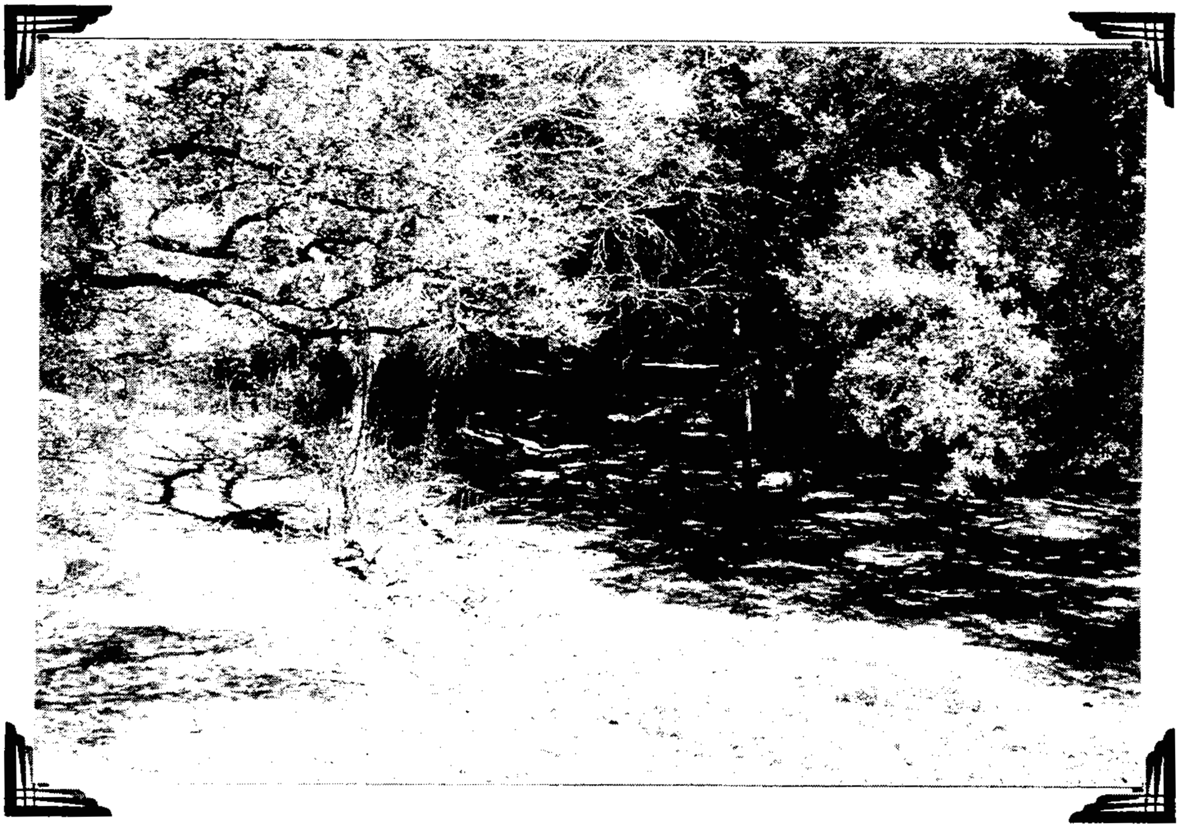
LOOP 1604 UPSTREAM

LOOP 1604 DOWNSTREAM

REDLAND RD.



LOOP 1604
LOOP 1604
ELM
MUD
CREEK
CREEK
WATERHOLE
ELM
PANTHER
SPRINGS
CREEK
HUEBNER RD.
BITTERS DR.
BITTERS RD.
SALADO
DEZAVALA RD.
HUEBNER RD.
LOCKHILL
SELMA
OLMOS
VANCE
DR.
I.H. 10
JACKSON
CREEK
BLANCO RD.
AVE
NAKOMA
RAMSEY
U.S. 281 N
JONES MALTSBERGER DR.
THOUSAND OAKS
RD.
MUD CREEK
STARCREST
CREEK
DR.
WETMORE
NACOGDOCHES
RD.
PERRIN-BEITEL
STAHL DR.
O'CONNOR
RD.
SCHERTZ
BEITEL CREEK
WEIDNER
RD.

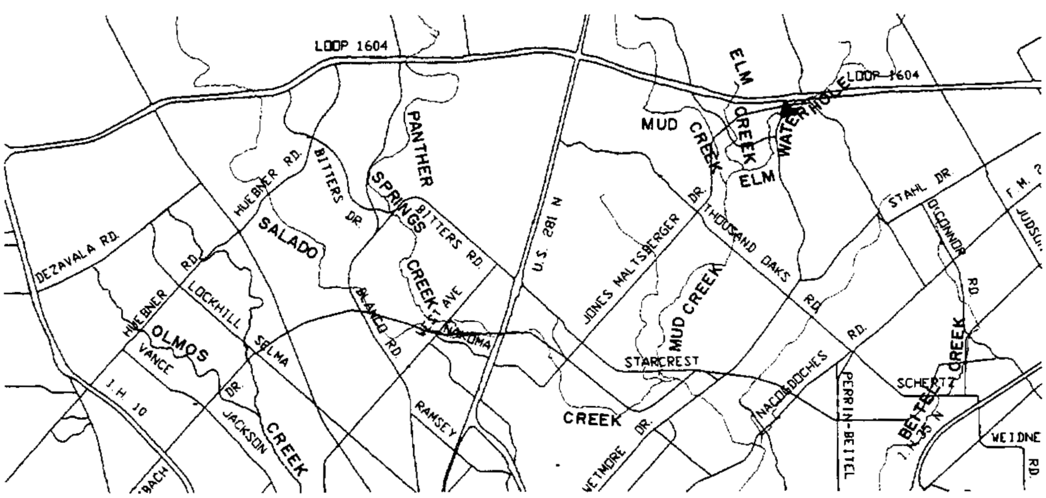


REDLAND RD. UPSTREAM

REDLAND RD. DOWNSTREAM

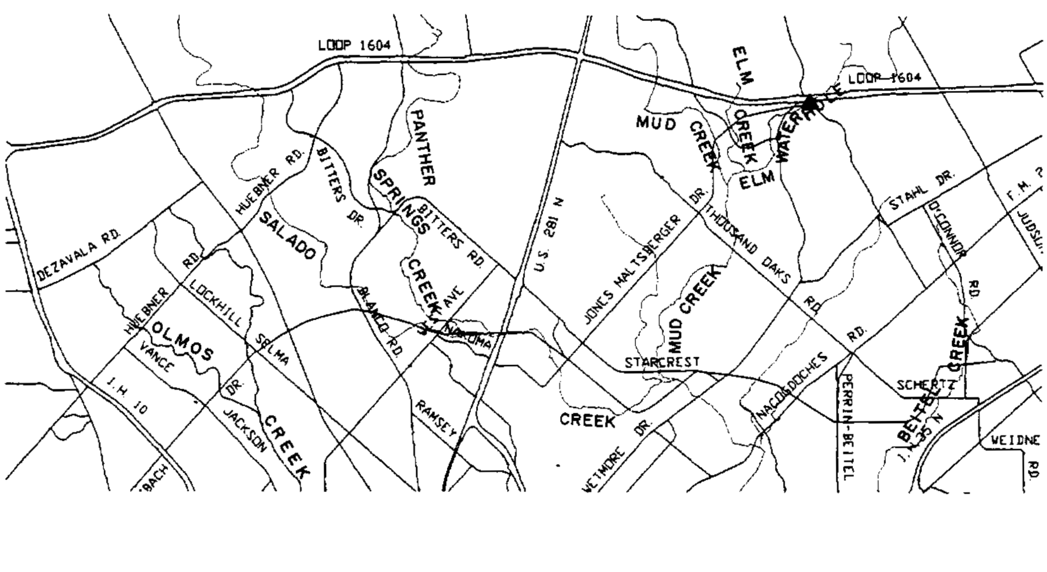
BULVERDE RD.

UPSTREAM

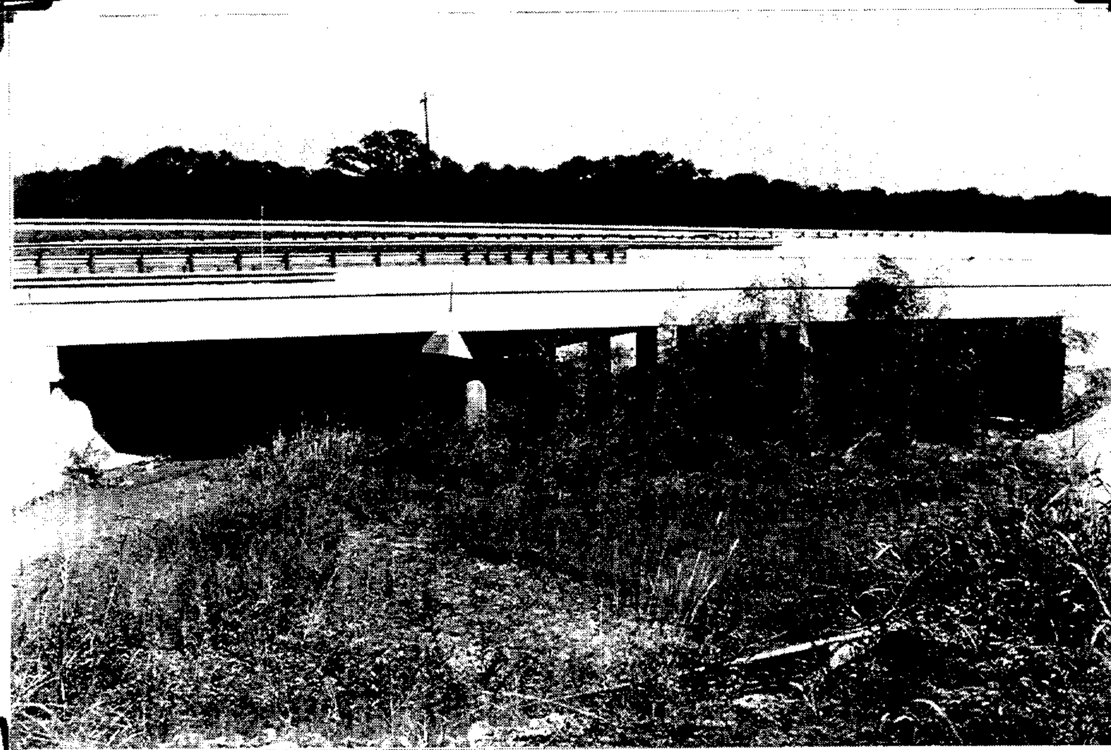


BULVERDE RD. DOWNSTREAM

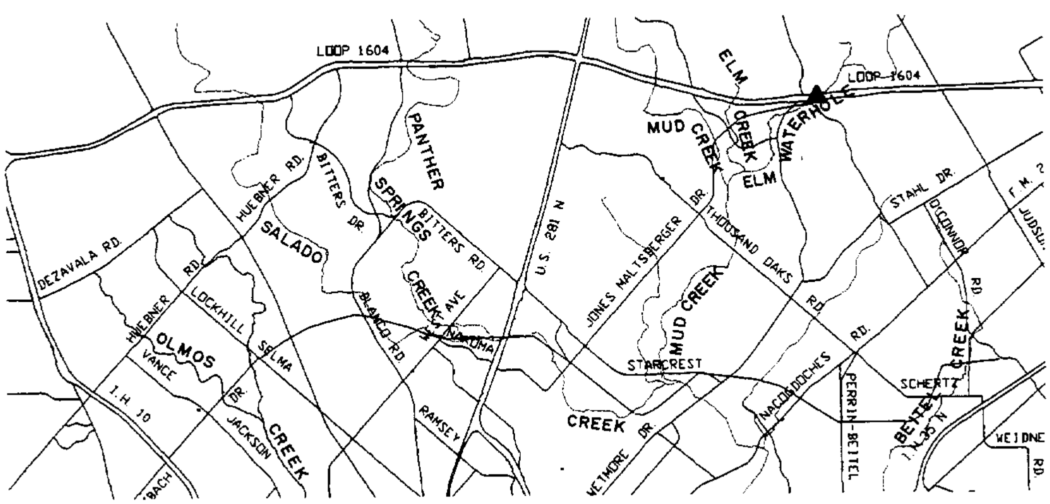
CLASSEN RD.


LOOP 1604
LOOP 1604
ELM
MUD
CREEK
CREEK
ELM
WATERHOLE
PANTHER
SPRINGS
CREEK
HUEBNER RD.
BITTERS DR.
BITTERS RD.
SALADO
DEZAVALA RD.
U.S. 281 N
JONES MALTSBERGER
DR.
THOUSAND OAKS
RD.
STAHL DR.
O'CONNOR
RD.
MUD CREEK
HUEBNER
RD.
LOCKHILL
SELMA
OLMOS
VANCE
BLANCO RD.
AVE.
NAKOMA
STARCREST
NACOGDOCHES
RD.
PERRIN-BEITEL
SCHERTZ
BELTEL
CREEK
I.H. 10
DR.
JACKSON
CREEK
RAMSEY
CREEK
WETMORE
DR.
I.H. 35 N
WEIDNE
RD.

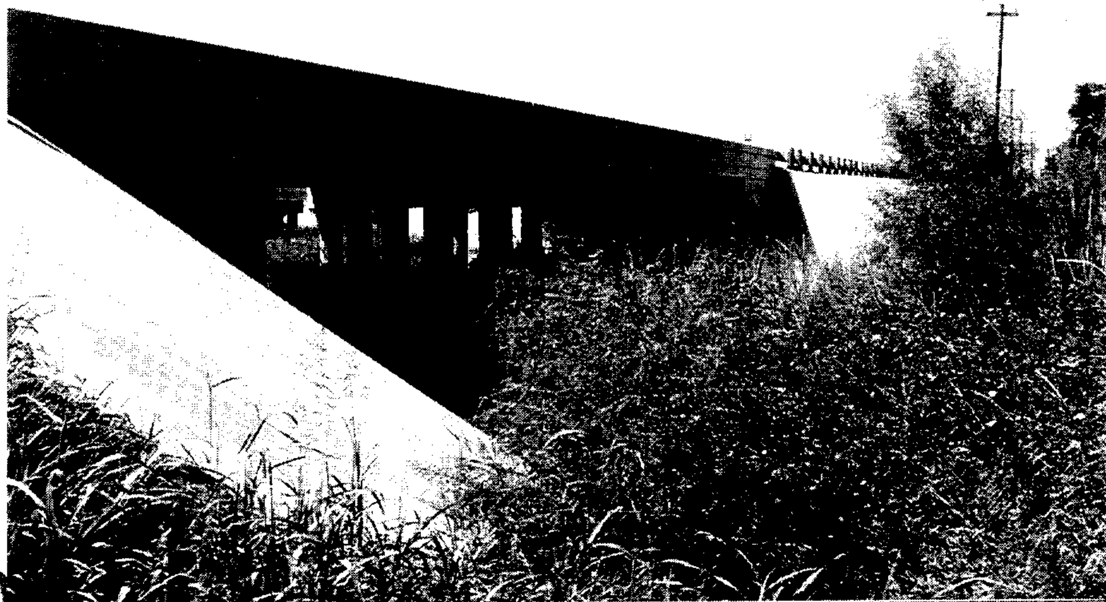
CLASSEN RD. UPSTREAM



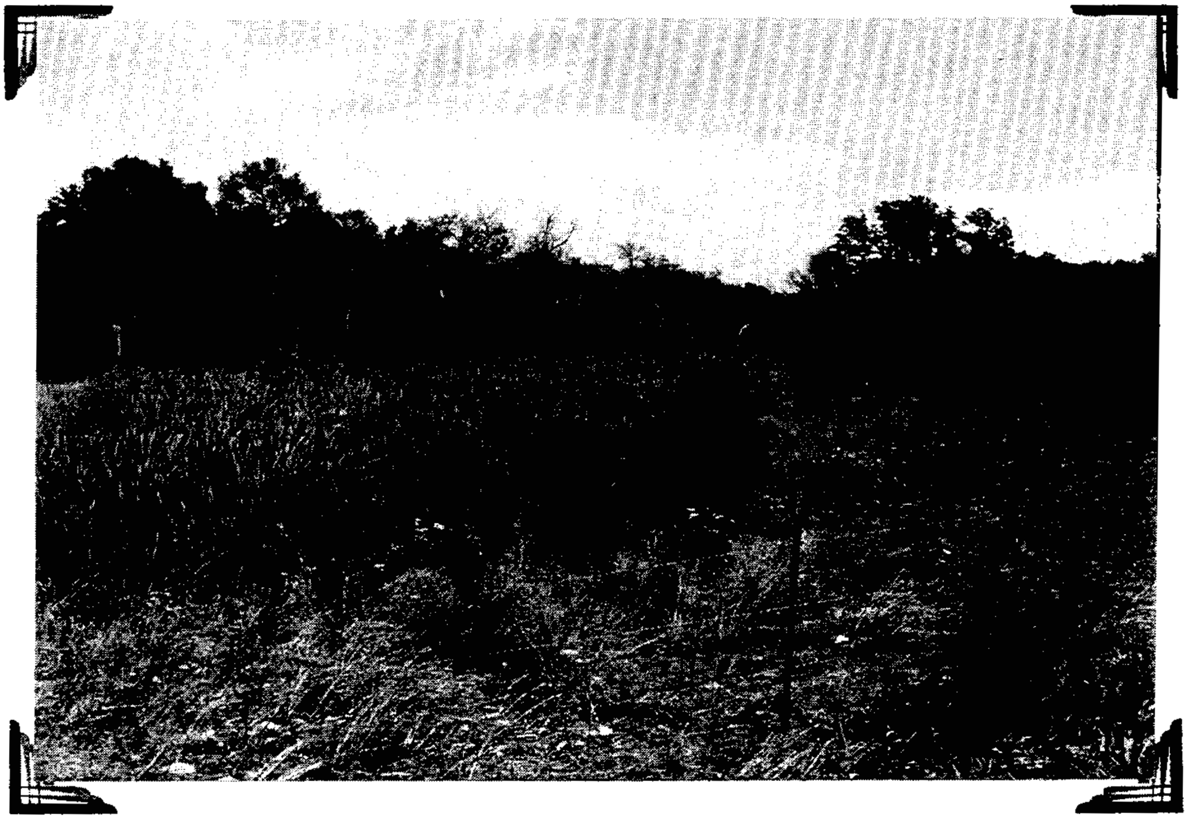
LOOP 1604



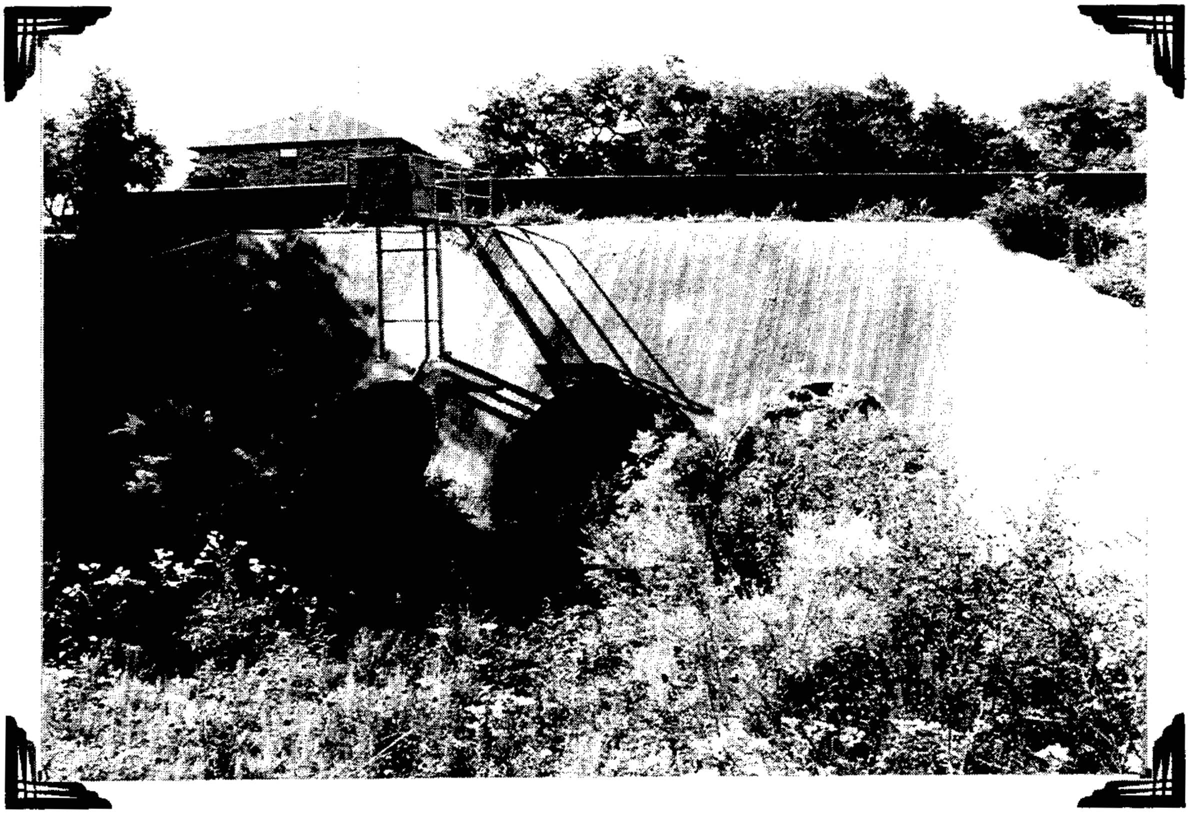
LOOP 1604
LOOP 1604
PANTHER
SPRINGS
CREEK
BITTERS DR.
BITTERS RD.
HUEBNER RD.
SALADO
DEZAVALA RD.
HUEBNER
RD.
LOCKHILL
SELMA
OLMOS
VANCE
I.H. 10
JACKSON
DR.
CREEK
BLANCO RD.
AVE
NAKOMA
RAMSEY
U.S. 281 N
JONES MALTSBERGER
DR.
MUD CREEK
ELM
CREEK
ELM
WATERHOLE
MUD CREEK
THOUSAND OAKS
RD.
STAHL DR.
O'CONNOR
RD.
STARCREST
CREEK
WETMORE DR.
NACOGDOCHES
RD.
PERRIN-BEITEL
SCHERTZ
BEITEL
CREEK
I.H. 35 N



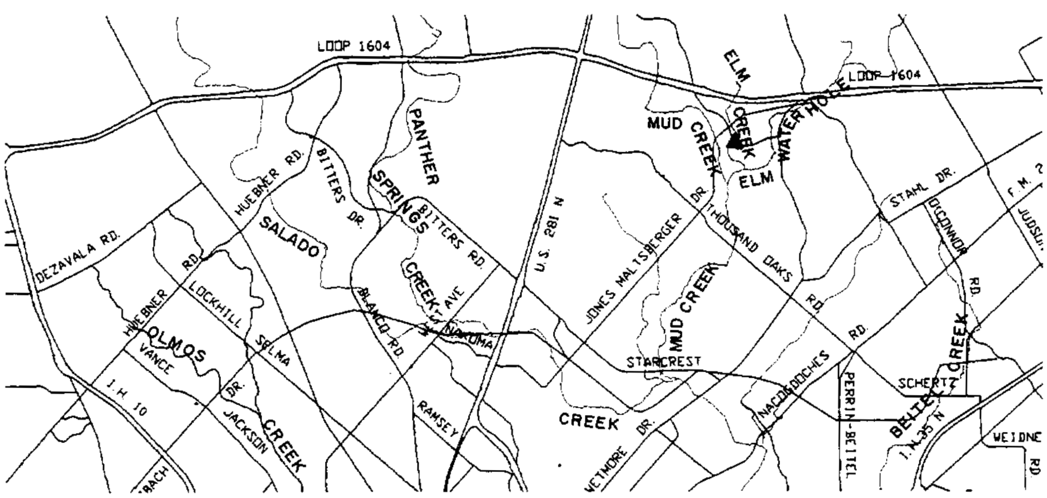
LOOP 1604 (NORTH)



LOOP 1604 UPSTREAM

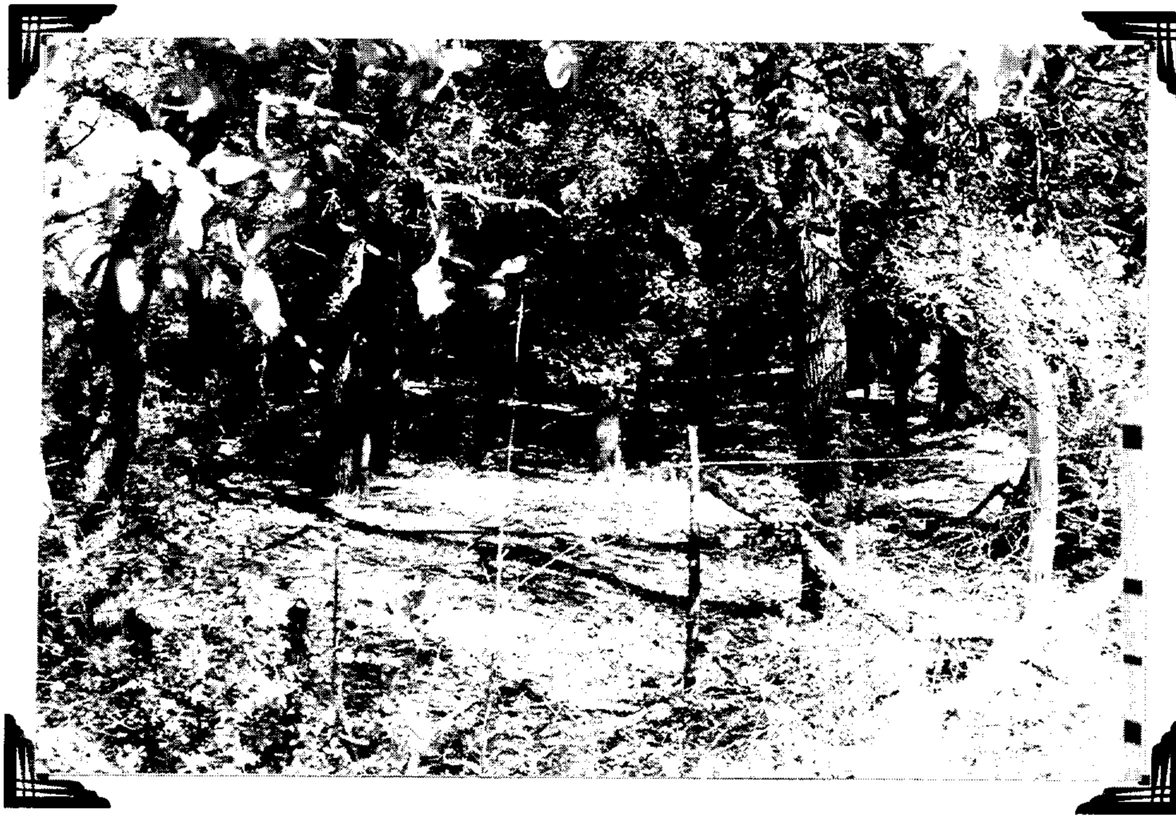


REDLAND RD.


LOOP 1604
LOOP 1604
ELM
CREEK
MUD
CREEK
ELM
WATERHOLE
PANTHER
SPRINGS
CREEK
HUEBNER RD.
BITTERS DR.
BITTERS RD.
SALADO
DEZAVALA RD.
U.S. 281 N
JONES MALTSBERGER DR.
THOUSAND OAKS
RD.
STAHL DR.
O'CONNOR RD.
F.M.
JUDSON
HUEBNER RD.
LOCKHILL SELMA
BLANCO RD.
AVE.
NAKOMA
MUD CREEK
OLMOS
VANCE
I.H. 10
JACKSON
DR.
CREEK
RAMSEY
STARCREST
CREEK
WETMORE DR.
NACOGDOCHES RD.
PERRIN-BEITEL
SCHERTZ
BEITEL CREEK
I.H. 35 N
WEIDNER RD.

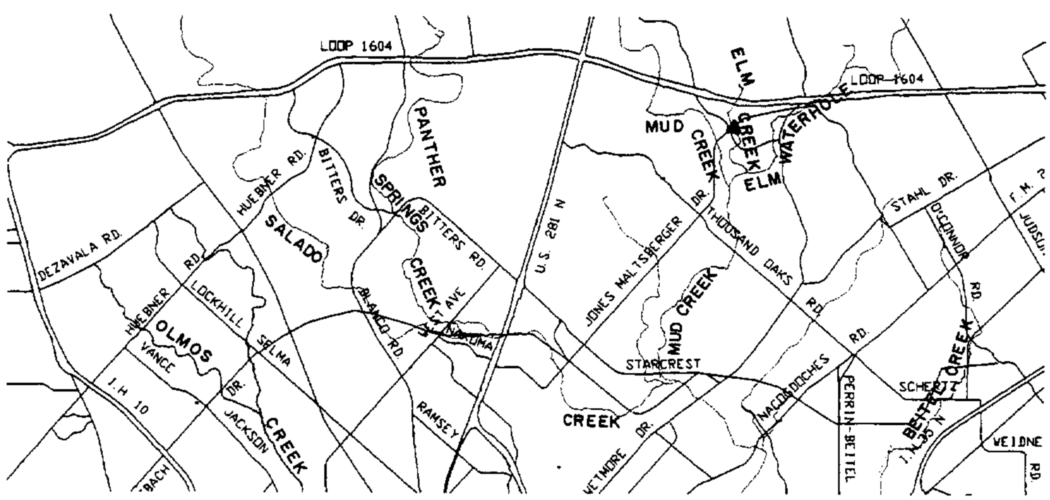
REDLAND RD. UPSTREAM

REDLAND RD. DOWNSTREAM

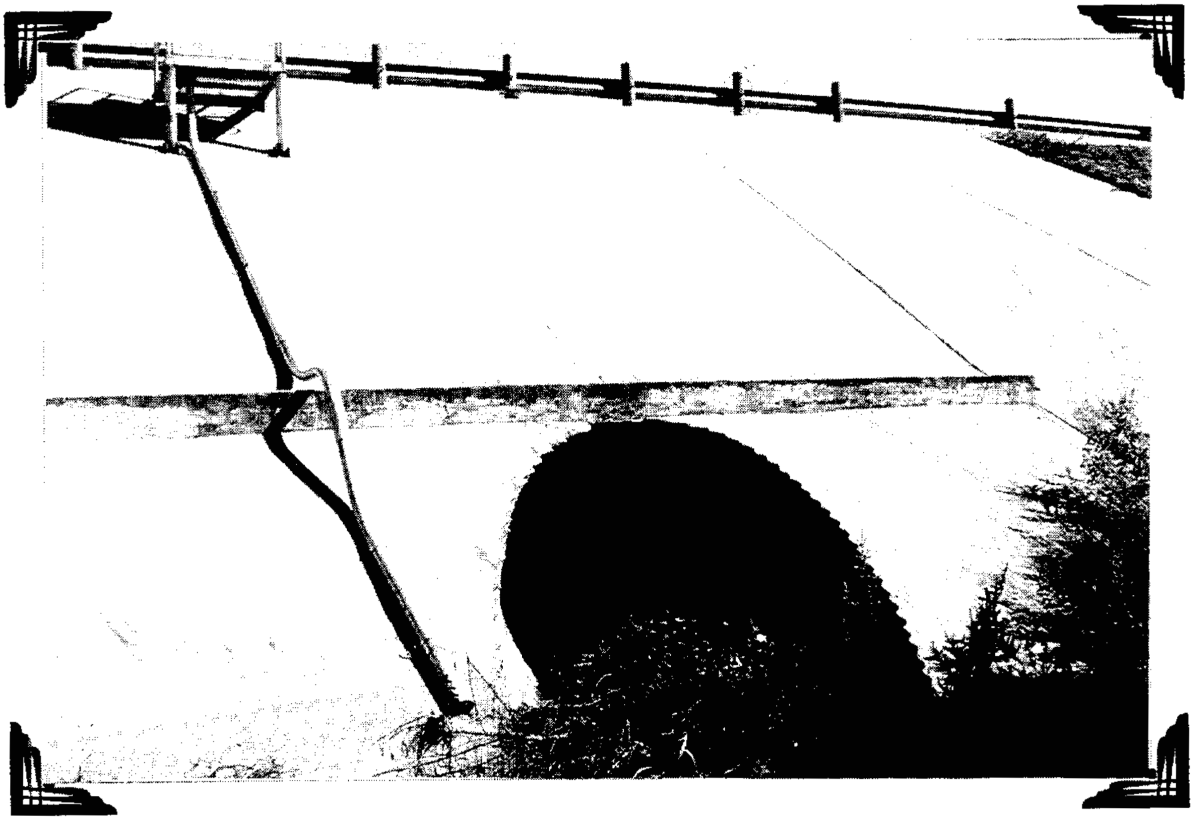


JONES MALTSBERGER

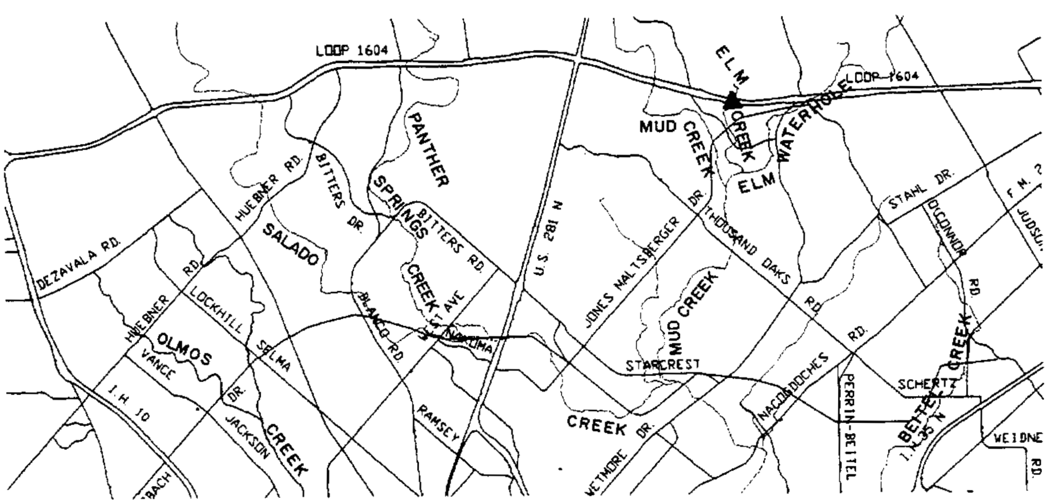
UPSTREAM



JONES MALTSBERGER DOWNSTREAM



LOOP 1604



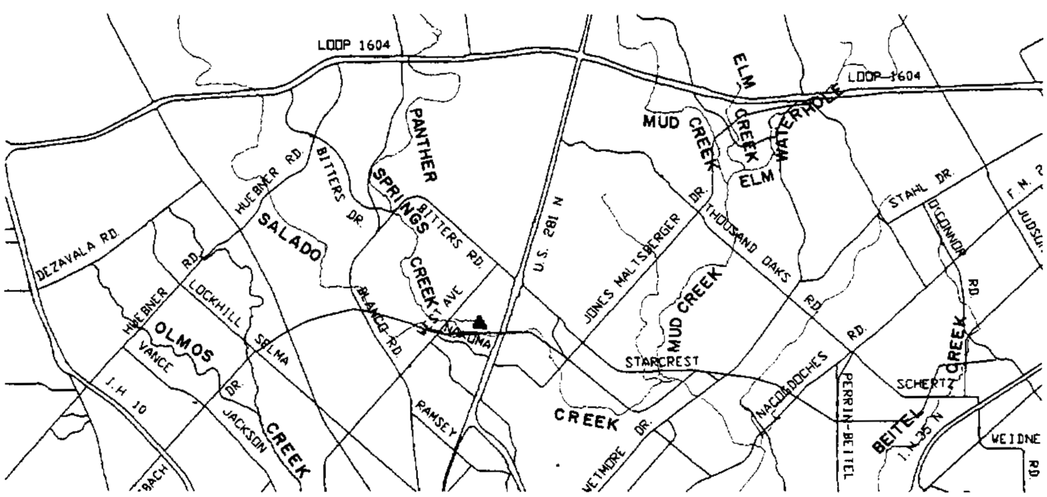
LOOP 1604
ELM
LOOP 1604
MUD CREEK
CREEK
WATERHOLE
ELM
PANTHER
SPRINGS
CREEK
BITTERS RD.
HUEBNER RD.
BITTERS DR.
SALADO
U.S. 281 N
JONES MALTSBERGER DR.
THOUSAND OAKS RD.
STAHL DR.
DEZAVALA RD.
HUEBNER RD.
LOCKHILL
SELMA
OLMOS
VANCE
DR.
JACKSON
I.H. 10
CREEK
BLANCO RD.
AVE.
NAKOMA
RAMSEY
MUD CREEK
STARCREST
CREEK DR.
WETMORE
NACOGDOCHES RD.
PERRIN-BEITEL
SCHERTZ
BEITEL CREEK
I.H. 35 N
WEIDNER RD.

LOOP 1604 DOWNSTREAM

LOOP 1604 (SOUTH)

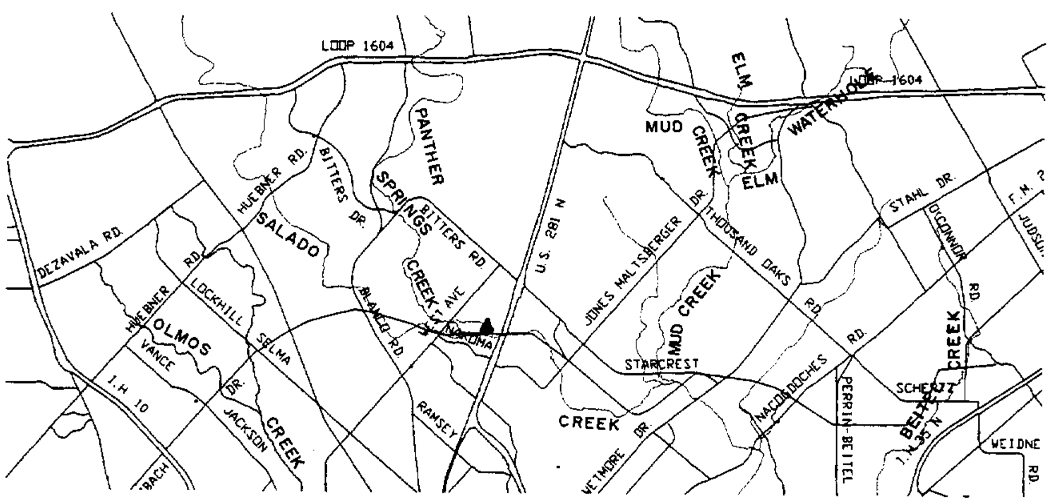
W. NORTH LOOP RD.

UPSTREAM



N. NORTH LOOP RD.

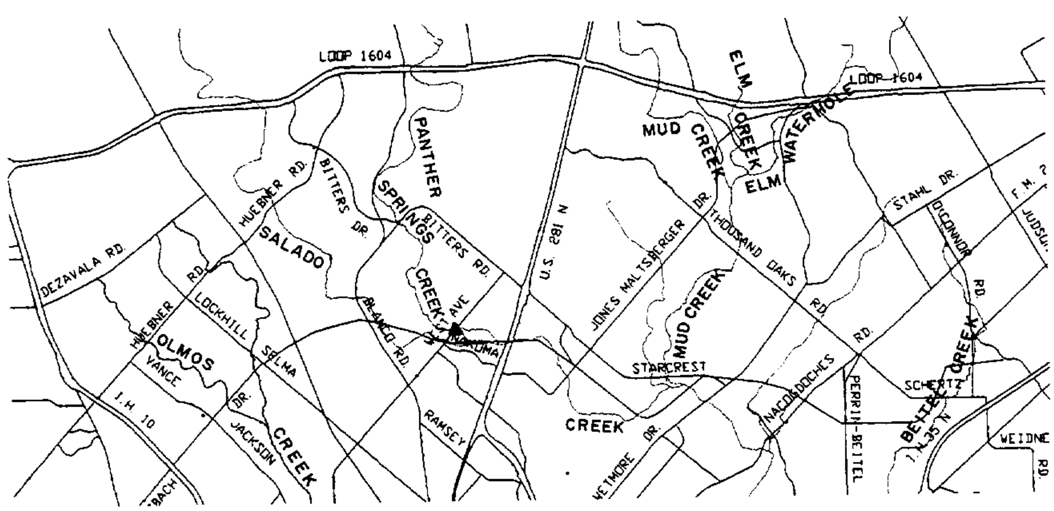
UPSTREAM

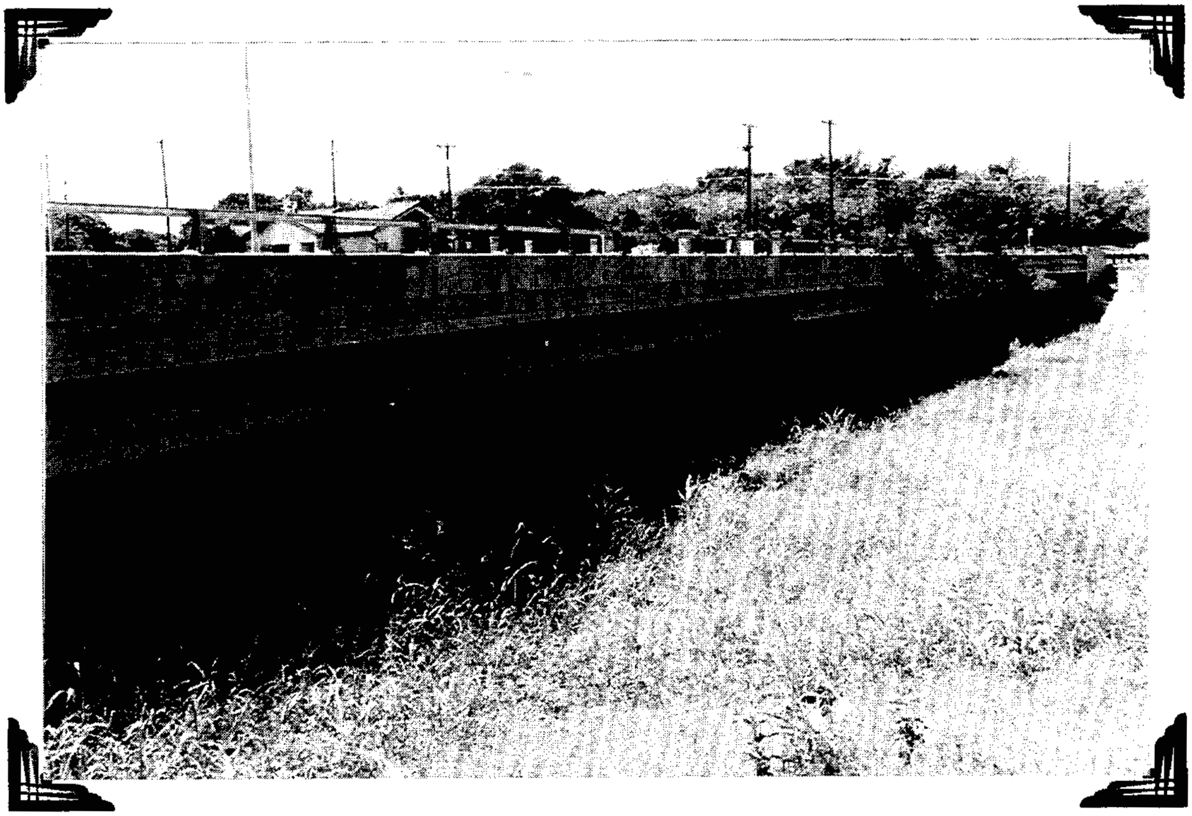


N. NORTH LOOP RD. DOWNSTREAM

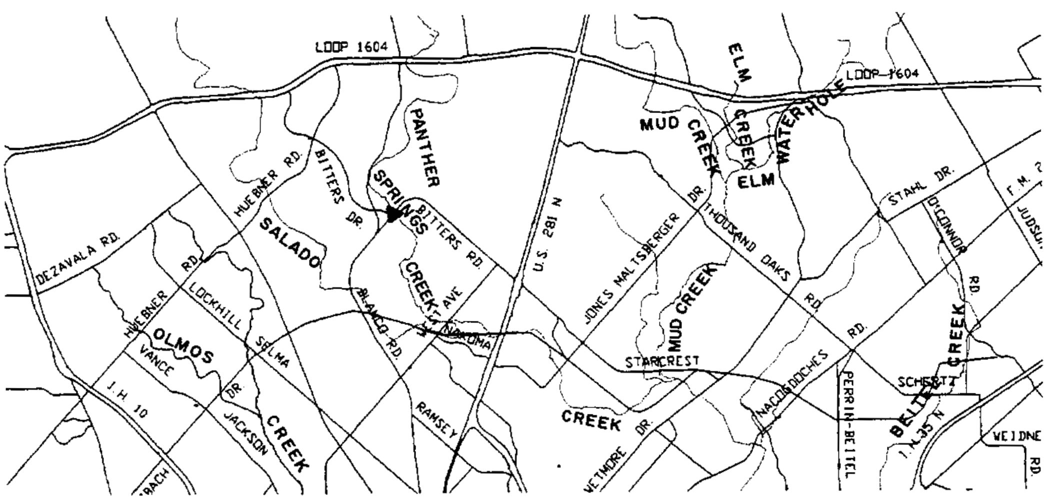
WEST AVE.

UPSTREAM

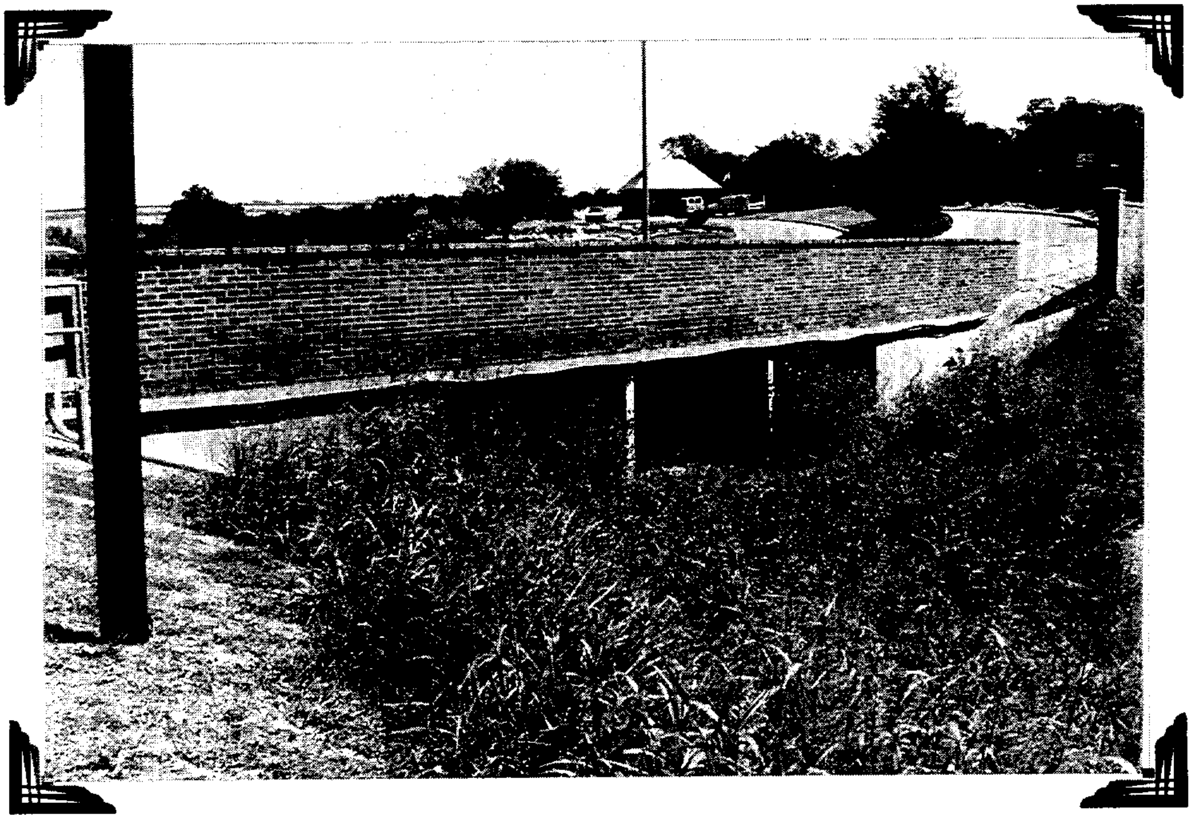




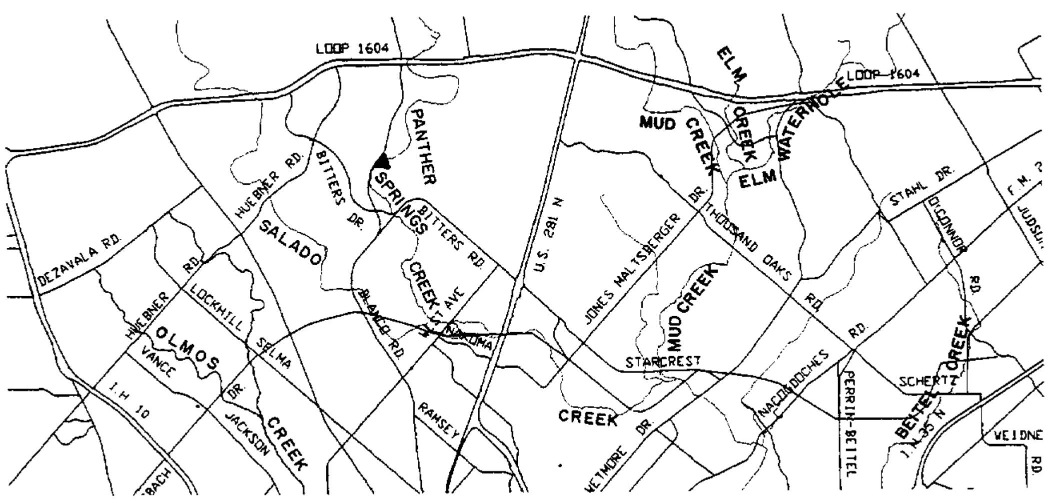
BITTERS RD.

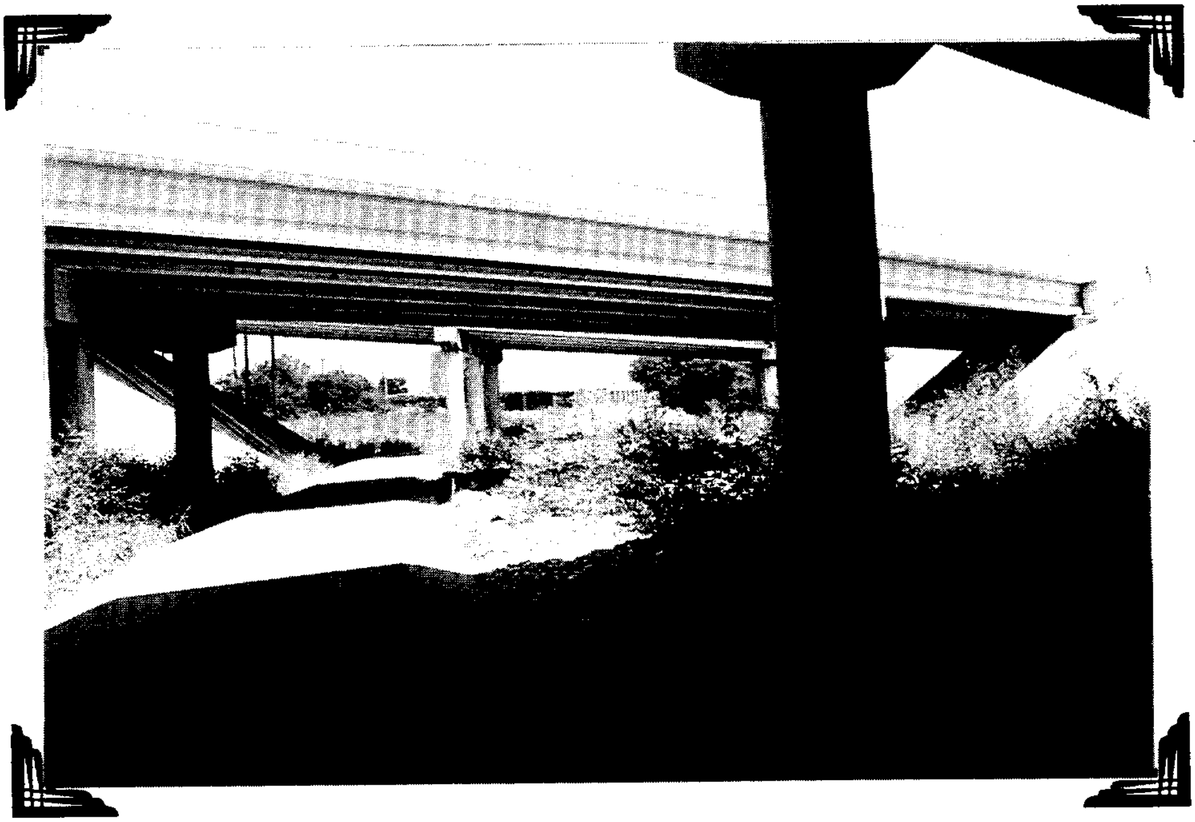

LOOP 1604
LOOP 1604
ELM
MUD
CREEK
CREEK
WATERHOLE
ELM
PANTHER
HUEBNER RD.
BITTERS DR.
SPRINGS
BITTERS RD.
U.S. 281 N
STAHL DR.
DEZAVALA RD.
SALADO
CREEK
AVE
JONES MALTSBERGER
DR.
THOUSAND OAKS
RD.
O'CONNOR
RD.
RD
HUEBNER
LOCKHILL
BLANCO RD.
NAKOMA
MUD CREEK
OLMOS
VANCE
SELMA
STARCREST
NACOGDOCHES
RD.
CREEK
DR
I.H. 10
JACKSON
CREEK
RAMSEY
CREEK
DR.
PERRIN-BEITEL
SCHERTZ
BEITEL
I.H. 35 N
RD.

BITTERS RD. UPSTREAM



MISSION RIDGE





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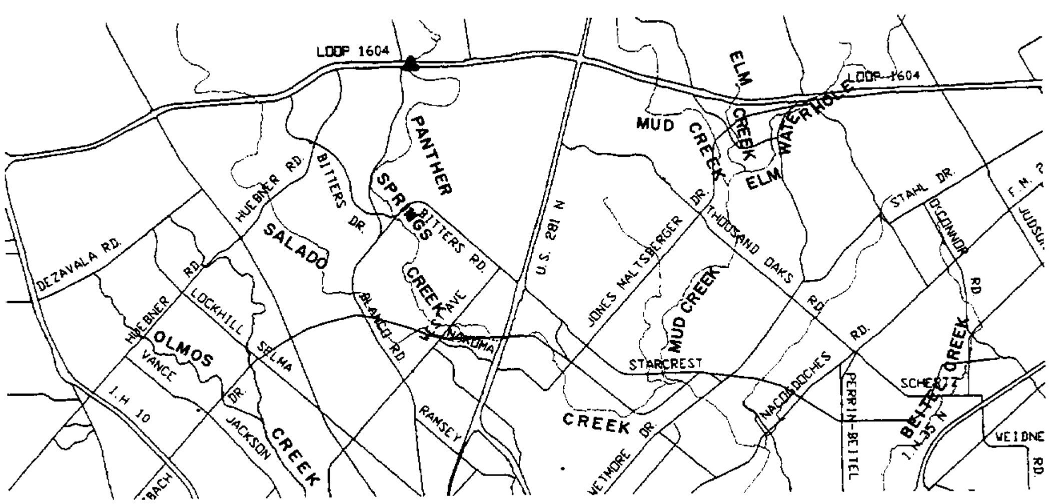
LOOP 1604
LOOP 1604
ELM
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BITTERS DR.
SALADO
U.S. 281 N
JONES MALTSBERGER DR.
THOUSAND OAKS
RD.
MUD CREEK
STAHL DR.
O'CONNOR
RD
DEZAVALA RD.
HUEBNER
RD.
LOCKHILL
SELMA
OLMOS
VANCE
DR.
I.H 10
JACKSON
CREEK
BLANCO RD
AVE
NAKOMA
RAMSEY
STARCREST
CREEK
WETMORE DR
NACOGDOCHES
PERRIN-BEITEL
SCHERTZ
BEITEL CREEK
I.H. 35 N
WEIDNER
RD

Appendix D

EXHIBIT 3

LAND USE CATEGORIES

The following categories are to be used with the 1981 land use maps prepared by the Department of Planning. There are a total of 392 maps at a scale of 1" = 500' which cover all of Bexar County. The examples represent illustrative uses within each category they are not comprehensive.

CATEGORY	EXAMPLE USES
10. RESIDENTIAL	
11. Dispersed (DI)	Clusters of rural dwellings, large lot (one acre or larger developments, may include isolated trailer mixed with houses; excludes most farm/ranch houses.
12. Subdivision (SD)	Conventional single-family residential development, may include duplexes.
13. Multifamily (MF)	Apartments, convents, orphanages, nursing homes, retirement complexes, boarding/rooming houses, condominiums.
14. Mobile Homes (MH)	Permanent and transient trailer parks.
15. Hotels/Motels (HM)	Hotels, motels, tourist courts.
20. COMMERCIAL	
21. Commercial (CO)	Retail and wholesale stores, radio/TV stations, malls, funeral homes, auto repair/sales, retail nurseries, ice houses, gas stations.
22. Office/Financial (OF)	Financial institutions, office buildings, medical/dental offices, veterinarians, governmental facilities.
23. Mixed Use (MX)	Strip commercial which includes significant residential and/or other non-commercial uses; structures which are used for two or more different uses (except for ground floor uses if rest of structures is single uses).

30.	<u>INDUSTRIAL</u>	
	31. Light (LT)	Electrical substations, water storage towers, mini-warehouses, telephone equipment buildings, outside storage yards, warehouse complexes.
	32. Heavy (HV)	Sewer treatment plants, airports, railroad yards, breweries, landfills.
	33. Extractive (EX)	Sand and gravel pits, quarries, earth moving/storage/filling operations.
40	<u>SERVICES</u>	
	41. Military (ML)	Military reservations.
	42. Institutional (IN)	Churches, schools sites (public and private), hospitals, clinics, post offices, fire and police centers, libraries, day-care centers.
	43. Cultural/Recreational (CR)	Park buildings, drive-in movies, rodeo arenas, racquetball courts, bowling alleys, miniature golf courses, fraternal organizations, tennis courts, theaters, swimming pools, party houses, dance halls, museums, sports complexes.
50.	<u>OPEN SPACE</u>	
	51. Parks (PK	Golf courses, developed or undeveloped park land (except extensively built up areas), ball parks.
	52. Restricted (RS)	Cemeteries, TV/radio antenna fields, undevelopable areas between rights-of-way, drainage channels.
	53. Incidental (IC)	Open space surrounding other land uses (e.g., open area around USAA building).
	54. Water (WT)	Bodies of water over one-half acres in size.
60.	<u>AGRICULTURAL</u> (AG)	Range, pastureland, cultivated fields, large nursery farms; includes farm/ranch houses.
70.	<u>TRANSPORTATION</u>	
	71. Rights-of-Way (RW)	Major road and railroad rights-of-way.
	72. Parking Lots (PL)	Parking lots and garages, vehicles storage areas.
80.	<u>VACANT</u> (VAC)	Land not used for any other purposes.

EXHIBIT 4

WATERSHED STUDY FOR OLMOS, LEON, AND SALADO CREEK

EXISTING VACANT/AGRICULTURE ACREAGE : 1991 LAND USE

ACRES	OLMOS	LEON	SALADO	TOTAL
TOTAL ACRES	12,900	150,800	121,000	284,700
VACANT ACRES	4,140	103,250	48,910	156,300
--VAC/DEVEL. *	4,050	88,600	46,340	138,990
% OF TOTAL	31%	59%	38%	49%

* VACANT DEVELOPABLE ACRES REPRESENTS THE VACANT ACRES LESS THE FLOODPLAINS & STEEP SLOPES

BASED UPON ULTIMATE DEVELOPMENT

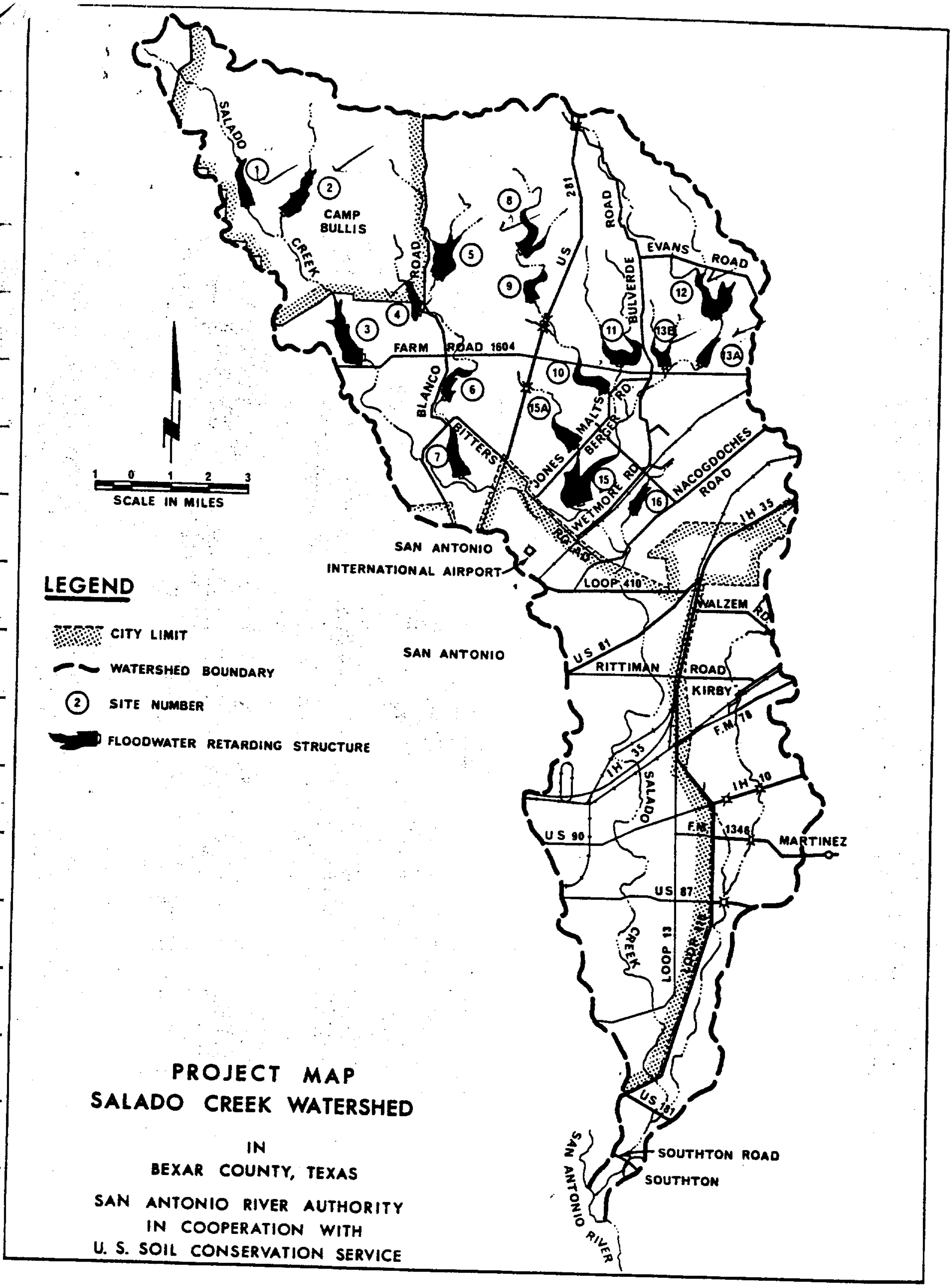
PROJECTED LAND USE ACREAGE: 1991 LAND USE TRENDS

LAND USE ACRES	OLMOS	LEON	SALADO	TOTAL
SINGLE FAMILY	2,228	48,730	25,487	76,445
MULTI FAMILY	203	4,430	2,317	6,950
COMMERCIAL/IND.	608	13,290	6,951	20,849
INSTITUTIONAL	203	4,430	2,317	6,950
TRANSPORTATION	203	4,430	2,317	6,950
OPEN SPACE/VACANT	608	13,290	6,951	20,849
TOTALS	4,050	88,600	46,340	138,990

PREPARED BY SAN ANTONIO PLANNING DEPARTMENT, NOVEMBER 1994

W_SHED5.WK3

Appendix E



PROJECT MAP
SALADO CREEK WATERSHED
IN
BEXAR COUNTY, TEXAS
SAN ANTONIO RIVER AUTHORITY
IN COOPERATION WITH
U. S. SOIL CONSERVATION SERVICE

Includes Flow over spillways

Drainage Study For SALADO CREEK- 12 DAMS IN (1,2,4,5,6,7,8,9,11,12,13A,13B)

7 – Panther Springs @ West Ave

REV. 6-23-87

NO.	LOCATION	Increment length (ft.)	TOTAL LENGTH = L Feet	Miles	Length to CG = LCA Feet	Miles	Drainage Area Acre	Sq.Mi.	Ct.	Cp.	25 Yr. Freq. Discharge = Q	100 Yr. Freq. Discharge = Q
1a	Salado Ck. Rosillo Ck. Dnstrm.	0		39.2				141.3		0.68	45,528	(⊛) 69,000
1b	Salado & Rosillo Cks. Upstream of Junction	0		39.2				111.4		0.68	36,094	(⊛) 58,500
1	Southeast Military Dr. (Loop 13)	16,400		36.1				108.1		0.68	37,642	(⊛) 58,000
2	Rigsby Ave.	20,000		32.3				100.7		0.68	38,620	(⊛) 53,000
3	Commerce	16,800		29.1				96.1		0.68	40,088	52,962
4	Gembler	8,800		27.4				90.39		0.68	39,717	52,428
5	Southern Pacific Railroad	4,800		26.5				88.5		0.68	40,012	52,789
6	Rittiman	24,000		22.1				80.7		0.68	42,362	55,691
7	Austin Hyw.	6,000		20.85				76.8		0.68	42,277	55,540
8	Salado Ck. & Beitel Above Confluence	3,400		20.21				62.7		0.68	35,551	46,732
9	Northeast Loop 410	6,000	100,700	19.07				55.8		0.68	33,229	43,663
10	Nacogdoches	7,200	93,500	17.71				48.3		0.68	30,592	40,179
11	Wetmore	5,700	87,800	16.63				47.0		0.68	31,222	40,953
12	Salado Ck. (Above Join. Mud Ck.)	3,000	84,800	16.06				25.2		0.68	17,101	22,684
13	San Pedro	18,000	66,800	12.65				19.1		0.68	15,422	20,391
14	Salado, 2,000' West of West Ave	6,600	60,200	11.40				17.5		0.68	14,993	19,405
15	Salado Ck. & F.M. 1604	27,300	32,400	6.14				12.2		0.68	15,711	20,045

*D.I. Refers to No. of Dams In.

⊛ U.S.G.S. min.

SAN ANTONIO RIVER BASIN

SCS FLOODWATER RETARDING DAM

SALADO CREEK WS SITE No.1

BEXAR COUNTY, TEXAS

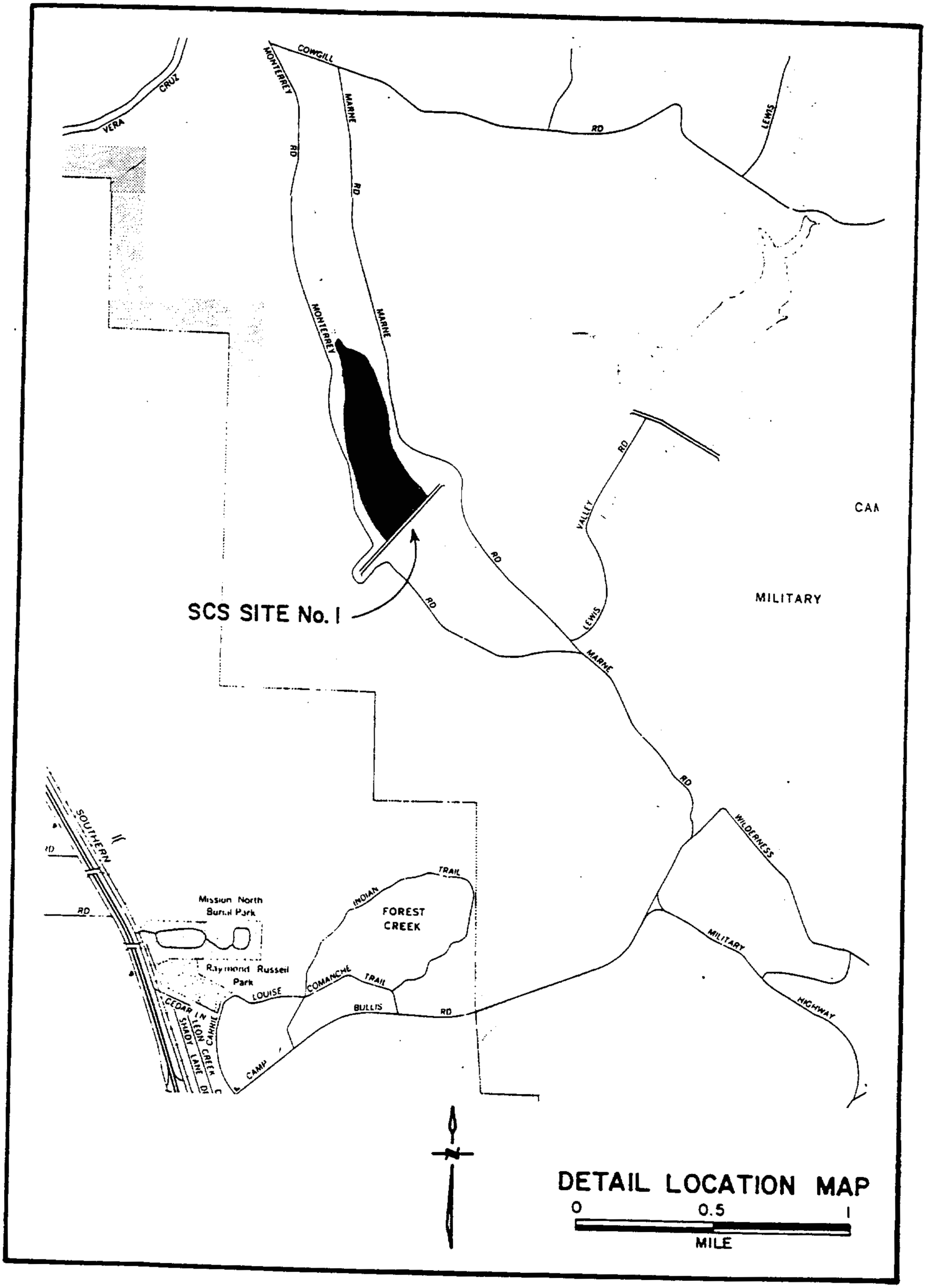
INVENTORY NUMBER TX04716

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

U. S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

MARCH 1981

REPORT PREPARED BY
TEXAS DEPARTMENT OF WATER RESOURCES
AUSTIN, TEXAS



COWGILL
MONTERREY
MARNE
RD
VERA
CRUZ
LEWIS
VALLEY
CAM
MILITARY
SCS SITE No. 1
WILDERNESS
SOUTHERN
Mission North
Burial Park
FOREST
CREEK
INDIAN
TRAIL
Raymond Russell
Park
LOUISE
COMANCHE
BULLIS
HIGHWAY
CEDAR LN
SHADY LANE
LEON CREEK
CAMP
DETAIL LOCATION MAP
0
0.5
1
MILE

f. *Purpose of dam.* SCS Site No.1 was designed as a floodwater-retarding structure.

g. *Design and construction history.* The dam was completed in November 1975 and was designed by the United States Department of Agriculture, Soil Conservation Service. The geological investigations, soil testing, and construction supervision were all conducted by the SCS. The contractor was Lawrence D. Krause.

h. *Normal operational procedures.* The spillways at SCS Site No.1 are uncontrolled; therefore, no operational procedures exist for periods of flooding. The operational procedures for the maintenance of the facility are discussed in Section 4.

1.3 *Pertinent Data.*

a. *Drainage area.*	11.3 square miles
b. *Discharge at dam site (CFS).*	
Maximum flood at dam site	Unknown
Combined spillway discharge at the effective crest elevation	37,355
c. *Elevation (feet above mean sea level).*	
Top of dam (effective crest)	1162.1
Maximum pool (PMF)	1163.7
Emergency spillway crest	1146.7
Service spillway crest	1117.1
Normal pool (ports)	1113.0
d. *Reservoir (length in miles).*	
Top of dam (effective crest)	1.5
Emergency spillway crest	1.0
Service spillway crest	0.3
Normal pool (ports)	0.2
e. *Storage (acre-feet).*	
Top of dam (effective crest)	9600
Maximum pool (PMF)	10,279
Emergency spillway crest	4189
Service spillway crest	313
Normal pool (ports)	199
f. *Reservoir surface (acres).*	
Top of dam (effective crest)	404
Maximum pool (PMF)	418
Emergency spillway crest	251
Service spillway crest	37
Normal pool (ports)	25
g. *Dam.*	
Type	Earthfill
Length	2320 feet

g. *Dam.* (cont'd)

Height	79.5 feet
Top width	14 feet
Upstream slope	2.5H:1V
Downstream slope	2.5H:1V with a 15-foot berm at elevation 1111 feet MSL, having the same slope below this elevation
Impervious core and cutoff	The cutoff trench for this structure has a 20-foot bottom width and 1H:1V side slopes. It is backfilled with compacted silty clay.
Zoning	SCS Site No.1 is a zoned earth embankment with an upstream zone of compacted silty clay and a downstream zone of clay, limestone, and shale. Both zones are protected by a 4-foot-thick outer blanket of gravelly clay.

h. *Diversion and regulating tunnel.* None

i. *Spillways.*

(1) *Service spillway.*

Type	The service spillway is uncontrolled and consists of 360 feet of 30-inch ID, prestressed concrete-lined, steel cylinder pipe through the embankment. A 7.5-foot-long by 2.5-foot-wide concrete drop inlet serves as an intake. There are also four 11-inch by 12-inch ports in the intake.
Crest elevation	1117.1 feet MSL
Port elevation	1113.0 feet MSL
Conduit invert at bottom of inlet	1102.1 feet MSL
Conduit invert at outlet	1092.2 feet MSL

(2) *Emergency spillway.*

Type	An uncontrolled earthcut channel approximately 200 feet in width
Crest elevation	1146.7 feet MSL

j. *Regulating outlets.* An 8-inch gate valve located in the upstream side of the drop inlet is used as a low-flow outlet. The invert is at elevation 1102.4 feet MSL.

(4) *Probable maximum flood outflow.* The probable maximum flood inflow hydrograph was routed through the structure beginning at elevation 1142.8 feet MSL. The outflow hydrograph is tabulated as follows:

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH

Time (hours)	*Instantaneous outflow (CFS)*	*Time (hours)*	*Instantaneous outflow (CFS)*
0.5	136	15.0	11,376
1.0	136	15.5	19,435
1.5	137	16.0	34,779
2.0	137	16.5	42,993
2.5	138	17.0	42,121
3.0	138	17.5	36,975
3.5	139	18.0	30,820
4.0	139	18.5	25,054
4.5	140	19.0	20,143
5.0	140	19.5	16,147
5.5	141	20.0	12,880
6.0	142	20.5	10,523
6.5	149	21.0	8,952
7.0	265	21.5	7,753
7.5	589	22.0	6,816
8.0	1,130	22.5	6,085
8.5	1,764	23.0	5,517
9.0	2,397	23.5	5,073
9.5	2,997	24.0	4,727
10.0	3,591	24.5	4,429
10.5	4,068	25.0	3,934
11.0	4,456	25.5	3,304
11.5	4,759	26.0	2,726
12.0	4,995	26.5	2,288
12.5	5,194	27.0	1,932
13.0	5,512	27.5	1,654
13.5	6,090	28.0	1,415
14.0	7,012	28.5	1,227
14.5	8,494	29.0	1,068

Note: Peak outflow occurs at 16.75 hours and is 43,386 CFS.

(5) *Discharge-frequency.* No data are available.

(6) *Reservoir area and storage capacity table.* An area-capacity table for SCS Site No.1 is presented following. These data were computed by the Soil Conservation Service.

RESERVOIR AREA AND STORAGE CAPACITY TABLE

Elevation (feet MSL)	*Reservoir area (acres)*	*Storage capacity (acre-feet)*
1089.0	0.0	0
1092.0	1.1	2
1096.0	3.3	11
1100.0	5.7	29
1104.0	8.2	57
1108.0	12.4	99
1112.0	21.5	168
1113.0	25.0	199
1116.0	34.3	280

(table cont'd)

RESERVOIR AREA AND STORAGE CAPACITY TABLE (cont'd)

Elevation (feet MSL)	*Reservoir area (acres)*	*Storage capacity (acre-feet)*
1117.1	37.0	313
1120.0	51.8	452
1124.0	75.5	707
1128.0	97.7	1053
1132.0	123.4	1495
1136.0	153.7	2049
1140.0	182.0	2720
1144.0	224.5	3533
1146.7	251.0	4189
1147.0		4278
1148.0	276.0	4534
1149.0		4828
1150.0		5122
1152.0	312.2	5710
1154.0		6420
1156.0		7100
1158.0		7900
1162.1		9600

(7) *Spillway rating table.* A spillway rating table for SCS Site No.1 is presented following. Data above elevation 1117.1 feet MSL were computed by the Soil Conservation Service. Data at and below this elevation were computed assuming orifice flow through the ports.

SPILLWAY RATING TABLE

Elevation (feet MSL)	*Combined spillway discharge (CFS)*
1113.0	0.0
1115.0	22.0
1117.1	33.7
1120.0	99.0
1128.0	114.0
1136.0	126.0
1144.0	138.0
1146.7	142.0
1147.0	214.0
1148.0	747.0
1149.0	1,732.0
1150.0	3,049.0
1152.0	6,578.0
1154.0	10,873.0
1156.0	16,143.0
1158.0	22,250.0
1162.1	37,355.0

(8) *Tailwater rating table.* A tailwater rating table for SCS Site No.1 is presented following. These data were computed using the Manning equation based on an idealized valley cross section and an average valley slope immediately downstream of the dam.

TAILWATER RATING TABLE

Elevation (feet MSL)	*Discharge (CFS)*
1082.6	0
1086.1	460
1089.7	2,280
1093.0	6,102
1096.5	12,482
1100.0	21,919
1106.0	53,374

(9) *Hydrologic network.* A map of the drainage area of SCS Site No.1 is presented in Exhibit A-3.

(10) *Breach analysis.* A breach analysis was performed to investigate the effects on the downstream area under five different conditions. These were:

1) Breaching of the dam with reservoir at normal operating level with no inflow.
2) Breaching due to overtopping by the PMF.
3) Overtopping without breaching by the PMF.
4) Breaching due to a barely overtopping flood that represents 89% of the PMF.
5) Overtopping without breaching due to a barely overtopping flood that represents 89% of the PMF.

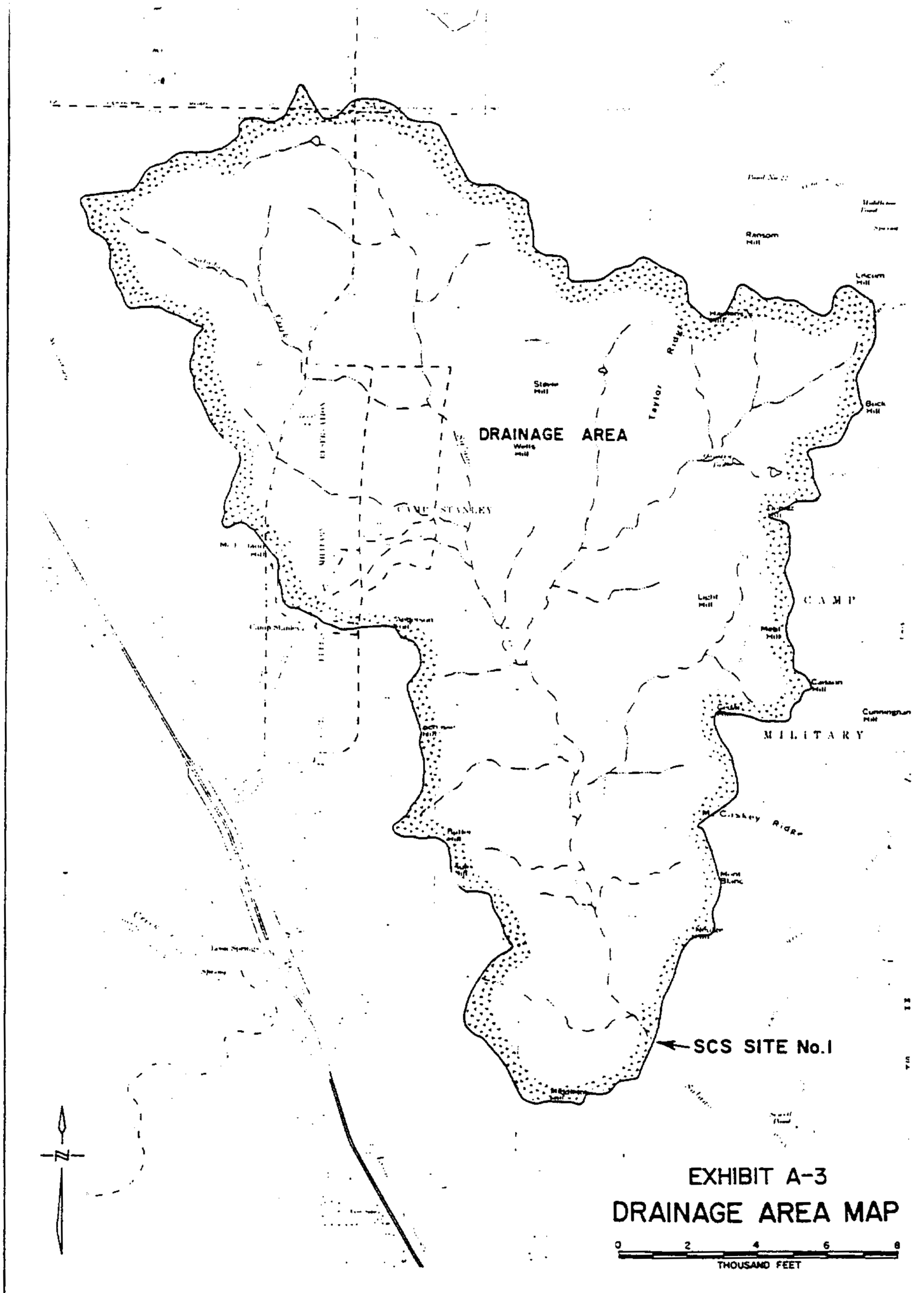
Results of the breach analysis for selected locations are graphically illustrated by computer plots, which appear as Exhibit A-4.

The dam is classified as a high-hazard structure due to the presence of scattered dwellings at various points downstream of the dam. A point of interest was chosen approximately 2.1 miles downstream from the dam at which point the appropriate USGS 7.5-minute topographic quadrangle, published in 1965 and photo-revised in 1973, shows several dwellings, the lowest of which appears to be between elevations 1059 and 1069 feet MSL. The following table shows the maximum water surface elevations and discharges at the point of interest for the five conditions investigated.

Condition	*Maximum water surface elevation (feet MSL)*	*Maximum discharge (CFS)*
Breaching at conservation pool with no inflow	1042.9	2,087
Breaching due to overtopping by PMF	1062.6	159,601
Overtopping without breaching by PMF	1053.6	49,319
Breaching due to overtopping by 89% PMF	1060.3	126,138
Overtopping without breaching by 89% PMF	1052.4	38,517

The breach analysis indicates that a significant increase in risk potential to loss of life would result downstream if the dam is breached by the PMF or by a flood that barely overtops the embankment. Therefore, should the dam be breached, an increase in the severity of flooding would occur downstream.

b. *Stability and stress analysis.* A stability analysis for the structure was performed using the Courtney method of analysis. A minimum factor of safety of 1.53 was found, which was for the downstream slope under steady seepage conditions.


DRAINAGE AREA
Taylor Ridge
CAMP STANLEY
CAMP
MILITARY
SCS SITE No.1
N
EXHIBIT A-3
DRAINAGE AREA MAP
0 2 4 6 8
THOUSAND FEET

SAN ANTONIO RIVER BASIN

SCS FLOODWATER RETARDING DAM

SALADO CREEK WS SITE No.2

BEXAR COUNTY, TEXAS

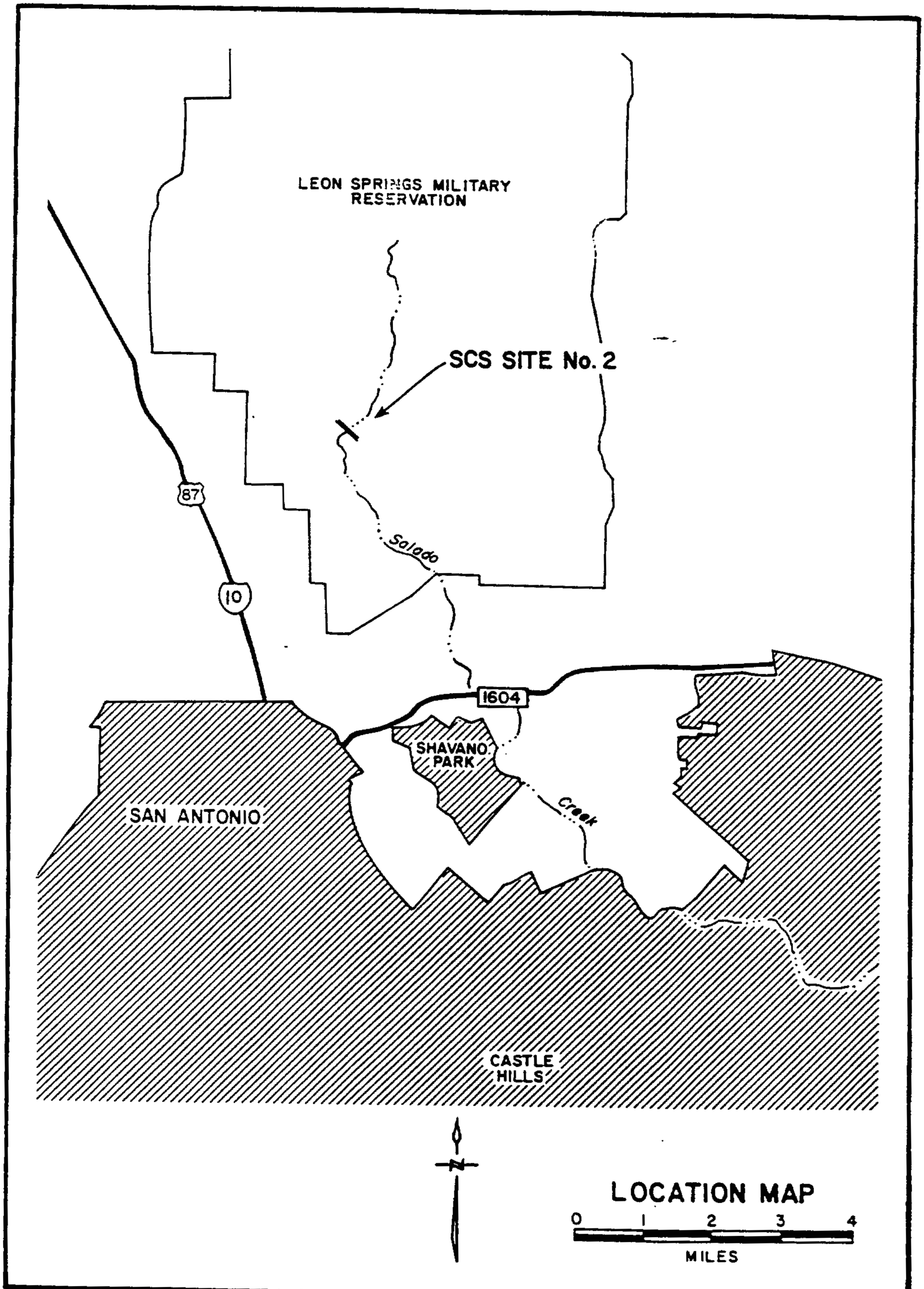
INVENTORY NUMBER TX01469

PHASE I INSPECTION REPORT NATIONAL DAM SAFETY PROGRAM

U. S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

MARCH 1981

REPORT PREPARED BY
TEXAS DEPARTMENT OF WATER RESOURCES
AUSTIN, TEXAS


LEON SPRINGS MILITARY RESERVATION
SCS SITE No. 2
87
10
Salado
1604
SHAVANO PARK
Creek
SAN ANTONIO
CASTLE HILLS
N
LOCATION MAP
0 1 2 3 4
MILES

f. *Purpose of dam.* SCS Site No.2 was designed as a floodwater-retarding structure.

g. *Design and construction history.* The dam was completed in March 1971 and was designed by the United States Department of Agriculture, Soil Conservation Service. The geological investigations, soils testing, and construction supervision were all conducted by the SCS. The contractor was William A. Pfeuffer.

h. *Normal operational procedures.* The spillways at SCS Site No.2 are uncontrolled; therefore, no operational procedures exist for periods of flooding. The operational procedures for the maintenance of the facility are discussed in Section 4.

1.3 *Pertinent Data.*

a. *Drainage area.*	5.7 square miles
b. *Discharge at dam site (CFS).*	
Maximum flood at dam site	Unknown
Combined spillway discharge at the effective crest elevation	18,988
c. *Elevation (feet above mean sea level).*	
Top of dam (effective crest)	1162.3
Maximum pool (PMF)	1165.9
Emergency spillway crest	1151.2
Service spillway crest	1130.3
Normal pool (ports)	1128.3
Low flow (slide gate invert)	1118.7
d. *Reservoir (length in miles).*	
Top of dam (effective crest)	1.1
Maximum pool (PMF)	Unknown
Emergency spillway crest	1.0
Normal pool (ports)	0.6
e. *Storage (acre-feet).*	
Top of dam (effective crest)	4317
Maximum pool (PMF)	5168
Emergency spillway crest	2293
Service spillway crest	269
Pool (ports)	199
f. *Reservoir surface (acres).*	
Top of dam (effective crest)	221
Maximum pool (PMF)	246
Emergency spillway crest	149
Service spillway crest	38
Normal pool (ports)	31
g. *Dam.*	
Type	Earthfill

g. *Dam.* (cont'd)	
Length	1910 feet
Height	65.3 feet
Top width	14 feet
Upstream slope	2.5H:1V
Downstream slope	2.5H:1V
Impervious core and cutoff	The cutoff trench for this structure has a 20-foot bottom width and 1H:1V side slopes. It is backfilled with compacted clayey silt.
Zoning	SCS Site No.2 is a zoned earth embankment with a thin outer shell of clayey gravel and an interior impervious zone of clayey silt.
h. *Diversion and regulating tunnel.*	None
i. *Spillways.*	
(1) *Service spillway.*	
Type	The service spillway is uncontrolled and consists of 280 feet of 30-inch ID, prestressed concrete-lined, steel cylinder pipe through the embankment. A 100-inch-long by 30-inch-wide concrete drop inlet serves as an intake. Four (two each side) 12-inch by 12-inch ports are also located in the inlet.
Crest elevation	1130.3 feet MSL
Port elevation (normal pool)	1128.3 feet MSL
Conduit invert at bottom of inlet	1118.2 feet MSL
Conduit invert at outlet	1102.9 feet MSL
(2) *Emergency spillway.*	
Type	An uncontrolled earthcut channel 210 feet in width
Crest elevation	1151.2 feet MSL
j. *Regulating outlets.*	A 12-inch by 12-inch slide gate, located on the upstream side of the drop inlet at an invert elevation of 1118.7 feet MSL, is used as a low-flow outlet.

PROBABLE MAXIMUM FLOOD INFLOW HYDROGRAPH (cont'd)

Time (hours)	*¼-hour average inflow (CFS)*	*Time (hours)*	*¼-hour average inflow (CFS)*
6.0	1,037	17.0	12,962
6.5	1,433	17.5	8,355
7.0	2,509	18.0	6,090
7.5	2,856	18.5	4,288
8.0	2,937	19.0	2,485
8.5	2,957	19.5	1,945
9.0	2,962	20.0	1,819
9.5	2,963	20.5	1,788
10.0	2,963	21.0	1,780
10.5	2,963	21.5	1,778
11.0	2,963	22.0	1,778
11.5	2,963	22.5	1,778
12.0	2,963	23.0	1,778
12.5	3,211	23.5	1,778
13.0	4,156	24.0	1,778
13.5	5,249	24.5	1,412
14.0	6,779	25.0	420
14.5	9,682	25.5	99
15.0	17,047	26.0	25
15.5	38,916	26.5	6
16.0	40,745	27.0	1
16.5	22,798		

Note: Peak ¼-hour average inflow occurs at 15.75 hours and is 47,048 CFS.

(4) *Probable maximum flood outflow.* The probable maximum flood inflow hydrograph was routed through the structure beginning at elevation 1147.6 feet MSL. The outflow hydrograph is tabulated as follows:

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH

Time (hours)	*Instantaneous outflow (CFS)*	*Time (hours)*	*Instantaneous outflow (CFS)*
0.5	74	11.5	2,610
1.0	74	12.0	2,702
1.5	74	12.5	2,807
2.0	75	13.0	3,099
2.5	75	13.5	3,591
3.0	76	14.0	4,460
3.5	76	14.5	6,010
4.0	76	15.0	9,290
4.5	76	15.5	19,101
5.0	76	16.0	28,767
5.5	76	16.5	27,836
6.0	78	17.0	22,651
6.5	88	17.5	17,117
7.0	142	18.0	12,507
7.5	323	18.5	9,536
8.0	652	19.0	7,048
8.5	1,034	19.5	5,262
9.0	1,400	20.0	4,114
9.5	1,796	20.5	3,448
10.0	2,092	21.0	3,014
10.5	2,317	21.5	2,692
11.0	2,485		

(table cont'd)

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH (cont'd)

Time (hours)	*Instantaneous outflow (CFS)*	*Time (hours)*	*Instantaneous outflow (CFS)*
22.0	2,453	25.0	1,558
22.5	2,277	25.5	1,253
23.0	2,148	26.0	1,022
23.5	2,054	26.5	842
24.0	1,984	27.0	713
24.5	1,880		

Note: Peak outflow occurs at 16.25 hours and is 29,246 CFS.

(5) *Discharge-frequency.* No data are available.

(6) *Reservoir area and storage capacity.* An area-capacity table for SCS Site No.2 is presented following. These data were computed by the Soil Conservation Service.

RESERVOIR AREA AND STORAGE CAPACITY TABLE

Elevation (feet MSL)	*Reservoir area (acres)*	*Storage capacity (acre-feet)*
1114.0	2.5	3
1118.0	6.0	20
1122.0	13.5	59
1126.0	24.0	134
1128.3	31.0	199
1130.0	37.0	256
1130.3	38.0	269
1132.0		320
1138.0	82.5	741
1142.0	105.5	1117
1146.0	125.5	1579
1150.0	142.0	2114
1151.2	149.0	2293
1152.0		2417
1153.0		2573
1153.2	161.0	2602
1154.0	165.5	2729
1156.0		3084
1158.0	189.5	3439
1160.0		3847
1162.0	219.0	4256
1162.3		4317
1166.0	247.0	5188
1170.0	279.5	6241
1174.0	367.4	7535

(7) *Spillway rating table.* A spillway rating table for SCS Site No.2 is presented following. Above elevation 1130.3 feet MSL, these data were computed by the Soil Conservation Service. Discharges at and below elevation 1130.3 feet MSL were computed assuming orifice flow through the service spillway ports.

SPILLWAY RATING TABLE

Elevation (feet MSL)	*Combined spillway discharge (CFS)*
1128.3	0
1129.3	14
1130.3	24
1132.0	50
1142.0	66
1150.0	77
1151.2	78
1152.0	209
1153.0	651
1154.0	1,459
1156.0	4,048
1158.0	7,804
1160.0	12,446
1162.3	18,988

(8) *Tailwater rating table.* A tailwater rating table for SCS Site No.2 is presented following. These data were computed using the Manning equation based on an idealized valley cross section and an average valley slope immediately downstream of the dam.

TAILWATER RATING TABLE

Elevation (feet MSL)	*Discharge (CFS)*
1097.0	0
1101.6	2,009
1106.2	11,762
1110.8	33,708
1115.4	71,553

(9) *Hydrologic network.* A map of the drainage area of SCS Site No.2 is presented in Exhibit A-3.

(10) *Breach analysis.* A breach analysis was performed to investigate the effects on the downstream area under five different conditions. These were:

1) Breaching of the dam with reservoir at normal operating level with no inflow.

2) Breaching due to overtopping by the PMF.

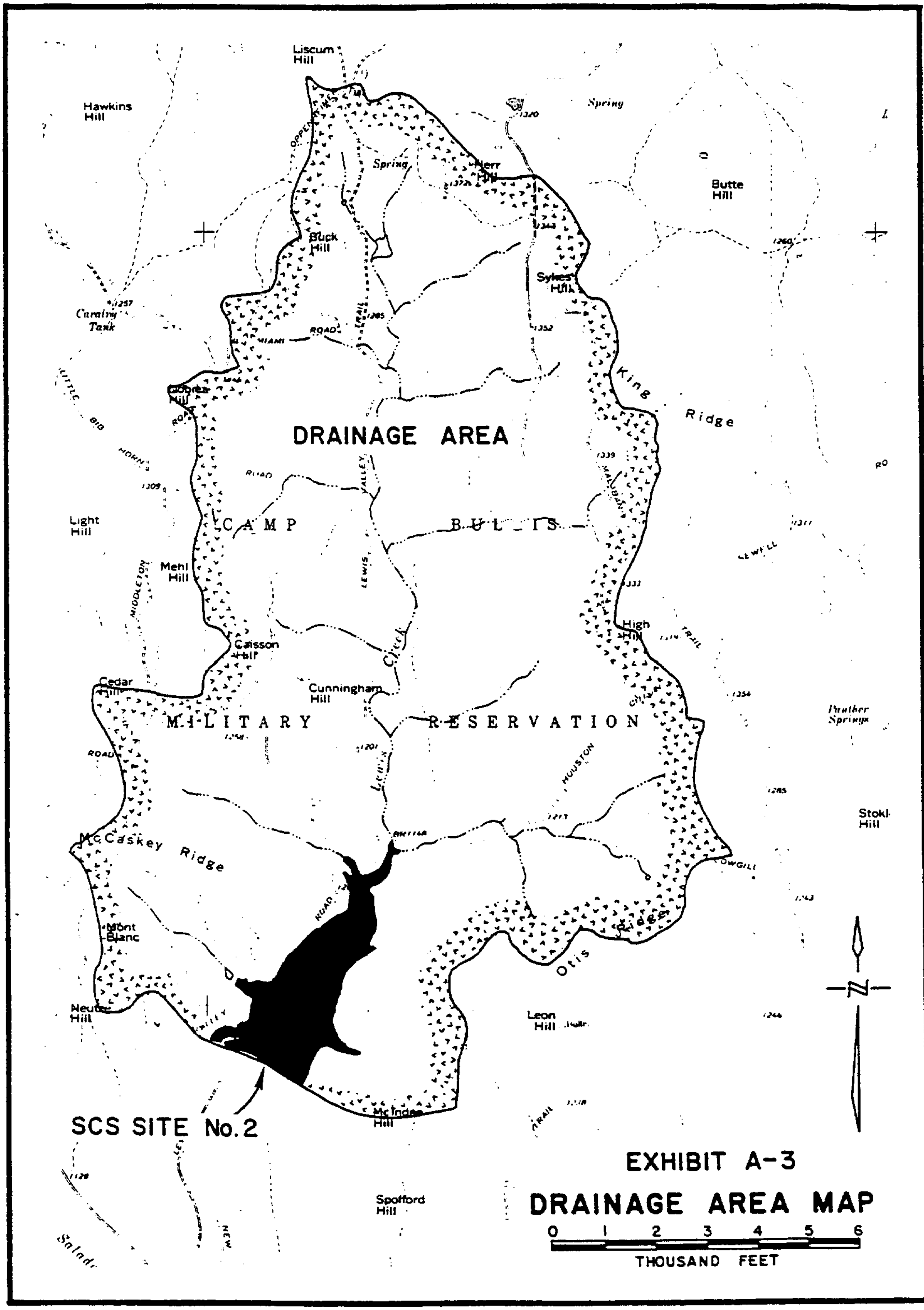
3) Overtopping without breaching by the PMF.

4) Breaching due to a barely overtopping flood that represents 68.3% of the PMF.

5) Overtopping without breaching due to a barely overtopping flood that represents 68.3% of the PMF.

Results of the breach analysis for selected locations are graphically illustrated by computer plots, which appear as Exhibit A-4.

The dam is classified as a high-hazard structure due to the presence of scattered dwellings at various points downstream of the dam. A point of interest was chosen approximately 1.55 miles downstream from the dam at which point the appropriate USGS 7.5-minute topographic quadrangle, published in 1965 and photo-revised in 1973, shows several dwellings, the lowest of which appears to be between elevations 1059 and 1069 feet MSL. The following table shows the maximum water surface elevations and discharges at the point of interest for the five conditions investigated.



DRAINAGE AREA
CAMP BULLIS
MILITARY RESERVATION
SCS SITE No. 2
EXHIBIT A-3
DRAINAGE AREA MAP
0 1 2 3 4 5 6
THOUSAND FEET
Liscum Hill
Hawkins Hill
Butte Hill
Herr Hill
Buck Hill
Sykes Hill
King Ridge
Light Hill
Mehl Hill
Caisson Hill
Cedar Hill
Cunningham Hill
High Hill
Panther Springs
Stokl Hill
McCaskey Ridge
Mont Blanc
Neutze Hill
Leon Hill
McIndoe Hill
Spofford Hill
Otis Ridge
Houston
Lewis Creek
Spring

SAN ANTONIO RIVER BASIN

SCS FLOODWATER RETARDING DAM

SALADO CREEK WS SITE No.4

BEXAR COUNTY, TEXAS

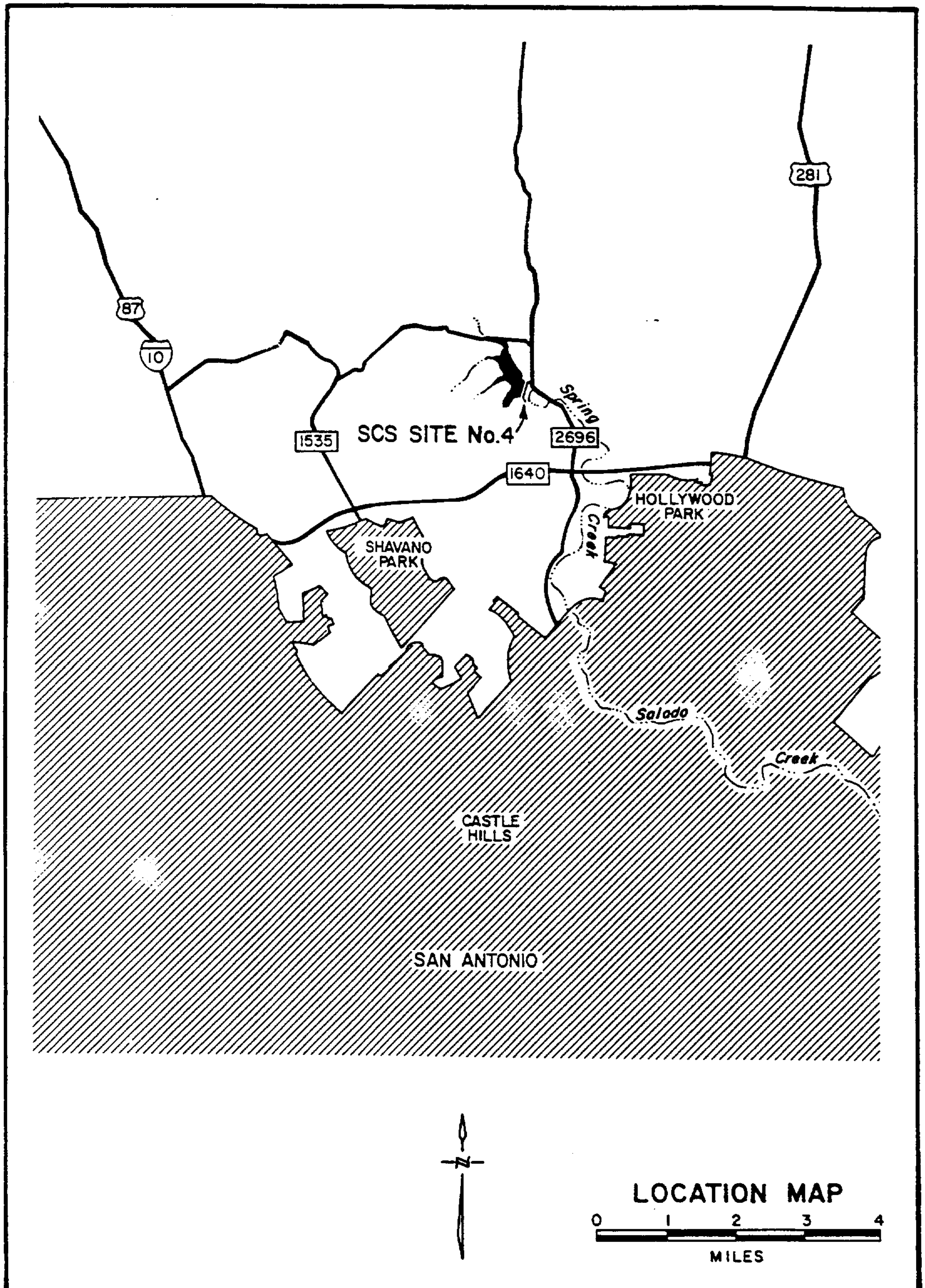
INVENTORY NUMBER TX01468

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

U. S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

MARCH 1981

REPORT PREPARED BY
TEXAS DEPARTMENT OF WATER RESOURCES
AUSTIN, TEXAS


281
87
10
1535
SCS SITE No.4
2696
Spring
1640
HOLLYWOOD PARK
Creek
SHAVANO PARK
Salado
Creek
CASTLE HILLS
SAN ANTONIO
N
LOCATION MAP
0 1 2 3 4
MILES

f. *Purpose of dam.* SCS Site No.4 was designed as a floodwater-retarding structure. It is also used for recharge purposes.

g. *Design and construction history.* The collection of the data necessary for the design of this structure was begun in July 1969 with a core boring program (geological study) to determine the geological sections and physical parameters for the materials in the area on which the structure was to be built. The interpretation of the data and resulting recommendations were completed on December 10, 1969. The final construction drawings were completed in December 1969 and approved by the Soil Conservation Service on October 26, 1970. The construction of the structure was completed on October 31, 1972. The contractor was William A. Pfeuffer.

h. *Normal operational procedures.* The spillways at SCS Site No.4 are uncontrolled; therefore, no operational procedures exist for periods of flooding. The operational procedures for the maintenance of the facility are discussed in Section 4.

1.3 *Pertinent Data.*

a. *Drainage area.*	5.5 square miles
b. *Discharge at dam site (CFS).*	
Maximum flood at dam site	Unknown
Combined spillway discharge at the effective crest elevation	18,802
c. *Elevation (feet above mean sea level).*	
Top of dam (effective crest)	1053.0
Maximum pool (PMF)	1056.5
Emergency spillway crest	1041.7
Service spillway crest	1013.0
Low flow (slide gate invert)	1008.08
Streambed (centerline of dam)	995.9
d. *Reservoir (length in feet).*	
Top of dam (effective crest)	6000
Emergency spillway crest	4200
Service spillway crest	2750
e. *Storage (acre-feet).*	
Top of dam (effective crest)	3957
Maximum pool (PMF)	4646
Emergency spillway crest	1982
Service spillway crest	85
f. *Reservoir surface (acres).*	
Top of dam (effective crest)	209
Maximum pool (PMF)	231
Emergency spillway crest	138
Service spillway crest	16
g. *Dam.*	
Type	Zoned earthfill
Length	1451 feet

g. *Dam.* (cont'd)

Height	57.1 feet
Top width	14 feet
Upstream slope	2.5H:1V
Downstream slope	2.5H:1V
Impervious core and cutoff	The excavated cutoff trench has 1H:1V side slopes. The bottom width of the cutoff trench is 20 feet. The fill materials are described in Exhibit C-3.
Zoning	The approximate zoning for the embankment is shown in Exhibit C-3. The zoning involved placement of fine-grained, high plasticity borrow materials in the center Zone 1 and gravelly borrow materials in the outer sections of Zone 2.
Grout curtain	None
Foundation drain	A rock toe drain is located at the downstream toe of the embankment. Refer to Exhibit C-6 for specific details.

h. *Diversion and regulating tunnel.* None

i. *Spillways.*

(1) *Service spillway.*

Type	The service spillway consists of a 30-inch by 100-inch by 5-foot 5-inch uncontrolled, reinforced concrete, drop-inlet structure and 270 feet of 30-inch ID, prestressed concrete-lined, steel cylinder pipe
Crest elevation	1013 feet MSL
Ports	None
Slide gate	A 12-inch slide gate is located in the upstream wall of the drop inlet and has an invert elevation of 1008.08 feet MSL.
Foundation	The pipe cradle and back fill are set on underlying bedrock of limestone. See Exhibits C-4 and C-5 for limits of the excavation.
Antiseep collars	There are six antiseep collars located on 20-foot centers with the upstream most collar being 35 feet upstream of the centerline of the dam.
Downstream channel	The downstream channel is an earth cut with a 12-foot wide bottom and 3H:1V side slopes. See Exhibit C-5 for details.

(2) *Emergency spillway.*

Type	An uncontrolled earthcut channel approximately 1330 feet in length with a 50-foot-long flat crest section
Width	200 feet

i. *Spillways.* (cont'd)

Crest elevation	1041.7 feet MSL
Upstream channel	An earthcut channel having side slopes of 1H:1V in rock and 3H:1V in earth fill and a 0.20 percent grade
Downstream channel	An earthcut channel having 3H:1V side slopes and a 5.04 percent grade for 450 feet, a 4.40 percent grade for 100 feet, a 2.55 percent grade for 300 feet and finally a 0.63 percent grade for approximately 60 feet until it ties into the natural ground. See Exhibit C-3.

j. *Regulating outlets.* The only regulating outlet is a 12-inch slide gate located on the upstream wall of the service spillway inlet. The slide gate invert elevation is 1008.08 feet MSL.

(4) *Probable maximum flood outflow.* The probable maximum flood inflow hydrograph was routed through the structure beginning at elevation 1038.9 feet MSL. The outflow hydrograph is tabulated as follows:

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH

Time (hours)	*Instantaneous outflow (CFS)*	*Time (hours)*	*Instantaneous outflow (CFS)*
0.5	52	14.5	5,546
1.0	52	15.0	8,200
1.5	53	15.5	15,930
2.0	53	16.0	26,558
2.5	53	16.5	27,259
3.0	54	17.0	22,621
3.5	54	17.5	17,278
4.0	54	18.0	12,862
4.5	64	18.5	9,762
5.0	85	19.0	7,364
5.5	151	19.5	5,508
6.0	217	20.0	4,214
6.5	294	20.5	3,385
7.0	519	21.0	2,926
7.5	859	21.5	2,615
8.0	1,265	22.0	2,389
8.5	1,651	22.5	2,219
9.0	1,946	23.0	2,092
9.5	2,168	23.5	1,996
10.0	2,336	24.0	1,924
10.5	2,462	24.5	1,837
11.0	2,557	25.0	1,571
11.5	2,628	25.5	1,240
12.0	2,682	26.0	973
12.5	2,744	26.5	812
13.0	2,971	27.0	676
13.5	3,409	27.5	570
14.0	4,231		

Note: Peak outflow occurs at 16.25 hours and is 28,007 CFS.

(5) *Discharge-frequency.* No data are available.

(6) *Reservoir area and storage capacity.* An area-capacity table for SCS Site No.4 is presented follo[illegible]g. These data were determined by the Soil Conservation Service.

RESERVOIR AREA AND STORAGE CAPACITY TABLE

Elevation (feet MSL)	*Reservoir area (acres)*	*Storage capacity (acre-feet)*
1000.0	0.4	1
1004.0	2.5	7
1008.0	8.3	28
1012.0	13.6	72
1013.0	16.0	85
1016.0	24.1	147
1020.0	34.6	265
1024.0	46.0	426
1028.0	61.7	641
1032.0	82.9	931

(table cont'd)

RESERVOIR AREA AND STORAGE CAPACITY TABLE (cont'd)

Elevation (feet MSL)	*Reservoir area (acres)*	*Storage capacity (acre-feet)*
1036.0	103.0	1302
1040.0	127.0	1762
1041.7	138.0	1982
1044.0	151.5	2320
1048.0	177.0	2977
1050.0		3369
1053.0		3957

(7) *Spillway rating table.* A spillway rating table for SCS Site No.4 is presented following. These data were determined by the Soil Conservation Service.

SPILLWAY RATING TABLE

Elevation (feet MSL)	*Combined spillway discharge (CFS)*
1013.0	0.0
1016.0	25.0
1024.0	37.0
1032.0	46.0
1040.0	53.0
1041.7	55.0
1042.0	91.0
1043.0	365.7
1044.0	1,022.4
1046.0	3,290.1
1048.0	6,720.1
1050.0	10,940.7
1053.0	18,801.7

(8) *Tailwater rating table.* A tailwater rating table for SCS Site No.4 is presented following. These data were computed using the Manning equation based on an idealized valley cross section and an average slope immediately downstream of the dam.

TAILWATER RATING TABLE

Elevation (feet MSL)	*Discharge (CFS)*
995.9	0
998.7	666
1001.5	3,304
1004.4	8,852
1007.2	18,117
1010.0	31,823

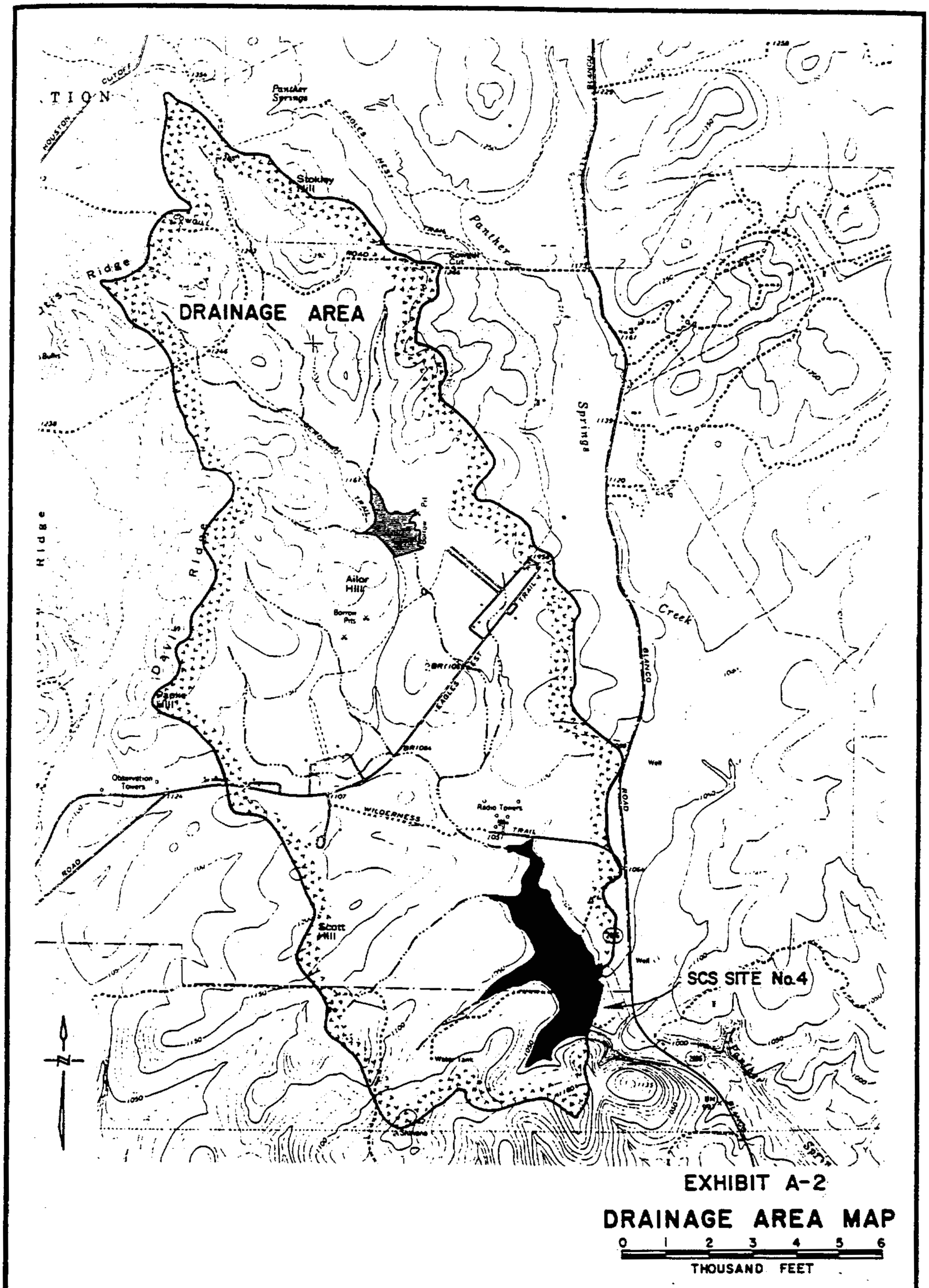
(9) *Hydrologic network.* A map of the drainage area of SCS Site No.4 is presented in Exhibit A-2.

(10) *Breach analysis.* A breach analysis was performed to investigate the effects on the downstream area under five different conditions. These were:

1) Breaching of the dam with reservoir at normal operating level with no inflow.

2) Breaching due to overtopping by the PMF.

3) Overtopping without breaching by the PMF.


DRAINAGE AREA
Stokley Hill
Ailor Hill
Scott Hill
Panther Springs
Panther Springs Creek
Davis Ridge
Radio Towers
Observation Towers
WILDERNESS TRAIL
EAGLES NEST TRAIL
BLANCO ROAD
SCS SITE No.4
N
EXHIBIT A-2
DRAINAGE AREA MAP
0 1 2 3 4 5 6
THOUSAND FEET

INPUT DATA FOR DAMS2 -TEXAS DESIGN SECTION

1-10	11-20	21-30	31-40	41-50	51-60	61-70
DAMS2	12/03/82			DATE	FILE NAME /DAMS2.	
STRUCTURE		WATERSHED AND SITE Salado Creek WS		Site 4	McGill	

	ELEV	SURF AC	PS DIS	ES DIS	AC FT
	1013.0		2		85.
	1016.		40.		147.
	1020.		48		265.
	1024.		56.		426.
	1028.		63.		641.
	1032.		69.		931.
	1036.		74.		1302.
	1040.		79.		1742.
	1041.7		81.		1982.
	1044.		84.		2320.
	1048.		79.		2977.
END TABLE					

WS DATA	class 4 C 2	cn 76	sq.mi. 5.51	Tc 2.3		base>csm
AREACRCT	4	ac 1 day 1.	ac 10 day 1.	1.0		
STORM	clim.index —	p 1da-in —	p 10da-in 24.	ac.esh-in —	ac.fbh-in —	p 100-10d —
POOL DATA ELEV		perm.pool 1013.	crest ps 1013.	sec el. 1013.		
~~PS DATA~~	~~1~~	ps length	size (in)		manning"n"	tw-outlet
PS ~~INLET ELEV~~		k 1	wier length			
ES CREST ELEV		1041.7	es crest elevations			
ES DATA	41	ent+crest 400.	"n" .040	z 1.	exit "n" .040	% exit 5.04
STM WIDTH FEET TRAPW		200.				
ESPROFILE	41	crest l 50. 10		ent+crest 400.	depth .7	
~~ESPROFILE~~						
	~~dist~~	~~elev~~	~~dist~~	~~elev~~	~~dist~~	~~elev~~
GO, STORM	2PC	—	-9.8			
GO, REACH	0 C6	—	22710	.165		
~~ENDJOB~~						
~~ENDRUN~~						

PERM POOL	1013.00 FT	35.0 ACFT	0.0 AC	0.0 CFS
CREST PS	1013.00 FT	35.0 ACFT	0.0 AC	0.0 CFS
SED ACCUM	1015.00 FT	85.0 ACFT	0.0 AC	0.0 CFS
ES CREST	1041.70 FT	1982.0 ACFT	0.0 AC	81.0 CFS

PS STORAGE 1897.0 ACFT BETWEEN ES CREST AND SED ACCUM ELEVATIONS

STARTING E	1013.00 FT	85.0 ACFT	0.0 AC	0.0 CFS

STORM HYD D= 24.00 HR P= 9.80 IN Q= 6.82 IN
TC= 2.30 HR CN= 76.00 VOL= 2004.8 ACFT
PEAK 3921.4 CFS AT 10.75 HRS

****
RATING TABLE DEVELOPED
WITH PS GIVEN AND EMG COMPUTED BY PROGRAM
EMG. RATING USED TRAPW METHOD

RATING TABLE NUMBER 2

	ELEV FEET	Q-TOTAL CFS	Q-PS CFS	VOLUME AC-FT	AREA ACRE
1	1013.00	0.0	0.0	85.00	0.0
2	1015.39	31.89	31.89	134.43	0.0
3	1017.78	43.57	43.57	199.60	0.0
4	1020.17	48.35	48.35	272.04	0.0
5	1022.57	53.13	53.13	368.30	0.0
6	1024.96	57.68	57.68	477.49	0.0
7	1027.35	61.86	61.86	606.04	0.0
8	1029.74	65.61	65.61	767.24	0.0
9	1032.13	69.17	69.17	943.32	0.0
10	1034.52	72.16	72.16	1165.14	0.0
11	1036.92	75.15	75.15	1407.34	0.0
12	1039.31	78.13	78.13	1682.38	0.0
13	1041.70	81.00	81.00	1981.91	0.0
14	1042.01	131.07	81.41	2028.17	0.0
15	1042.33	181.25	81.82	2074.46	0.0
16	1042.90	412.92	82.56	2157.80	0.0
17	1043.59	849.12	83.46	2259.65	0.0
18	1044.85	2046.86	85.06	2459.55	0.0
19	1046.42	4247.27	87.03	2718.27	0.0
20	1048.00	6993.78	89.00	2977.00	0.0

TYPE	BW	EMAX	VOL-MAX	AMAX	HP	VOL-ES	Q-PS	Q-ES	Q-TOT	D/C	V/C	S/C	S/C.25	Q-ES	QE/B
STORM HYD	200.0	1041.59	1967.8	0.0	0.0	0.0	80.9	0.0	80.9	0.0	0.0	0.0	0.0	0.0	0.0

PLOT 1 IN = 1000. CFS EXIT SLOPE = 0.050

0. 1000. 2000. 3000. 4000. 5000. 6000. 7000.

T	I	O	E	VOL	A	I I I I I I I I	EXIT VEL
0.0	0	0	1013.0	85.0	0.0	.	0.0
0.32	0	0	1013.0	85.0	0.0	.	0.0
0.63	0	0	1013.0	85.0	0.0	.	0.0
0.95	0	0	1013.0	85.0	0.0	.	0.0
1.26	0	0	1013.0	85.0	0.0	.	0.0
1.58	0	0	1013.0	85.0	0.0	.	0.0
1.90	0	0	1013.0	85.0	0.0	.	0.0
2.21	0	0	1013.0	35.0	0.0	.	

SAN ANTONIO RIVER BASIN

SCS FLOODWATER RETARDING DAM

SALADO CREEK WS SITE No.5

BEXAR COUNTY, TEXAS

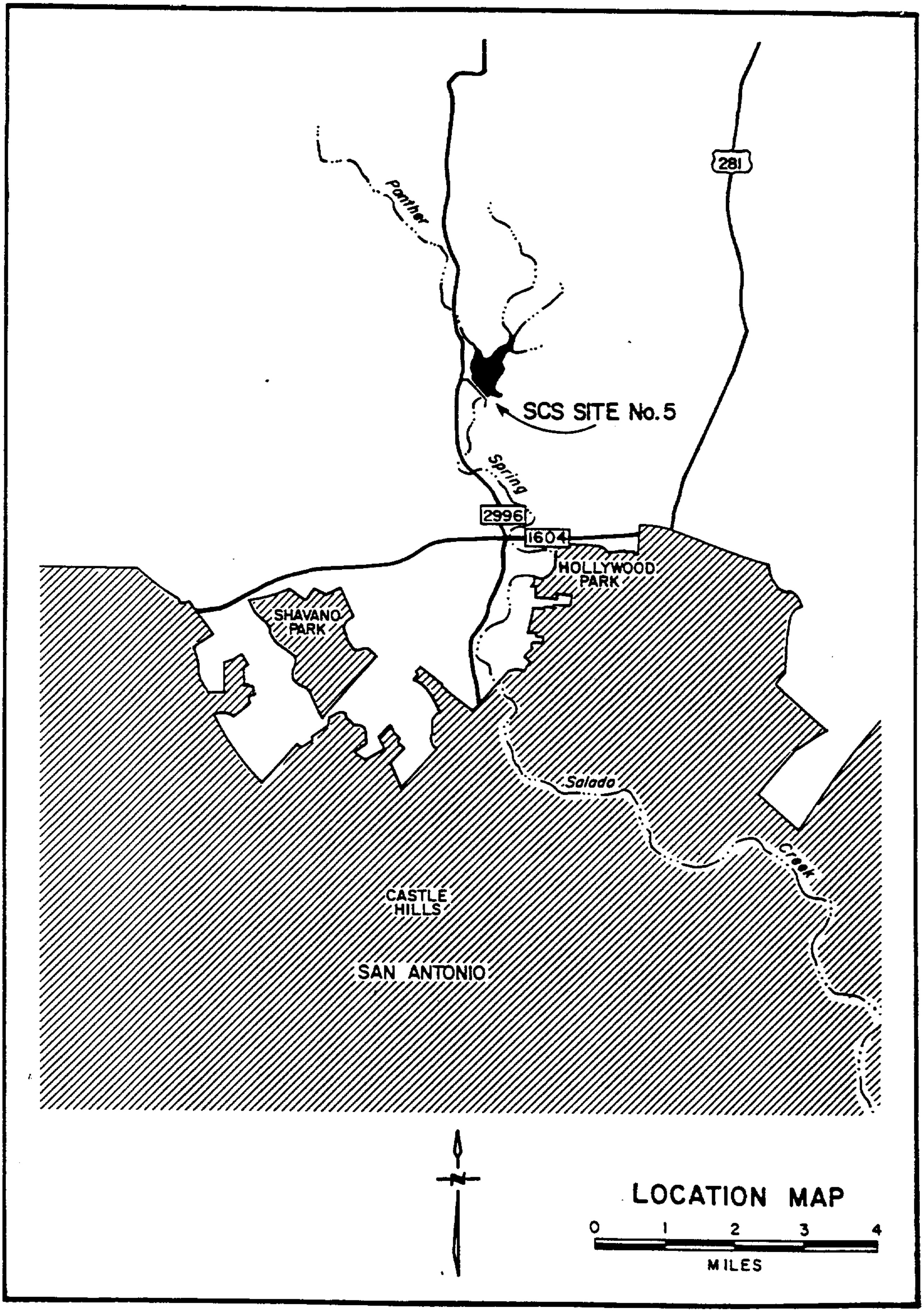
INVENTORY NUMBER TX04717

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

U.S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

MARCH 1981

REPORT PREPARED BY
TEXAS DEPARTMENT OF WATER RESOURCES
AUSTIN, TEXAS


281
Panther
SCS SITE No. 5
Spring
2996
1604
HOLLYWOOD PARK
SHAVANO PARK
Salado
Creek
CASTLE HILLS
SAN ANTONIO
N
LOCATION MAP
0 1 2 3 4
MILES

f. *Purpose of dam.* SCS Site No.5 was designed as a floodwater-retarding structure.

g. *Design and construction history.* The dam was completed on October 18, 1976, and was designed by the United States Department of Agriculture, Soil Conservation Service. The geologic investigation, soils testing, and construction supervision were all conducted by the SCS. The contractor was Leo P. Cloud, Jr. & Son.

h. *Normal operational procedures.* The spillways at SCS Site No.5 are uncontrolled; therefore, no operational procedures exist for periods of flooding. The operational procedures for the maintenance of the facility are discussed in Section 4.

1.3 *Pertinent Data.*

a. *Drainage area.*	8.9 square miles
b. *Discharge at dam site (CFS).*	
Maximum flood at dam site	Unknown
Combined spillway discharge at the effective crest elevation	32,049
c. *Elevation (feet above mean sea level).*	
Top of dam (effective crest)	1098.5
Maximum pool (PMF)	1101.6
Emergency spillway crest	1089.1
Service spillway crest	1061.0
Sediment pool (weirs)	1060.0
Low flow (slide gate invert)	1051.5
Maximum tailwater	Unknown
Streambed (centerline of dam)	1035.5
d. *Reservoir (length in feet).*	
Top of dam (effective crest)	7200
Emergency spillway crest	6400
Service spillway crest	2100
e. *Storage (acre-feet).*	
Top of dam (effective crest)	5807
Maximum pool (PMF)	6706
Emergency spillway crest	3293
Service spillway crest	231
Sediment pool (weirs)	198
f. *Reservoir surface (acres).*	
Top of dam (effective crest)	315
Maximum pool (PMF)	347
Emergency spillway crest	218
Service spillway crest	28
Sediment pool (weirs)	25
g. *Dam.*	
Type	Zoned earth-and-rock fill embankment

g. *Dam.* (cont'd)

Length	2673 feet
Height	63.5 feet
Top width	14.0 feet
Upstream slope	2.5H:1V
Downstream slope	2.5H:1V
Impervious core and cutoff	The entire zoned embankment was placed on the natural rock foundation after the overburden was removed. Therefore, there is no cutoff. See Exhibit C-2.
Zoning	SCS Site No.5 is a zoned embankment having a compacted core of clay and/or clayey gravel and an outer rockfill shell. An upstream and downstream transition zone separate the core from the rockfill zone. A blanket of the transition zone material of gravel and clayey gravel also extends upstream from the core to the toe of the rockfill section.
Grout curtain	All bedrock was to be treated with dental grout.
Foundation drain	None

h. *Diversion and regulating tunnel.* None

i. *Spillways.*

(1) *Service spillway.*

Type	The service spillway is uncontrolled and consists of a 30-inch by 90-inch by 10-foot drop-inlet structure and 280 feet of 30-inch ID, concrete-lined, steel cylinder pipe. See Exhibits C-4 and C-5.
Crest elevation	1061.0 feet MSL
Weirs	There are two 1-foot-high by 2-foot-long weirs in the drop inlet, one on each side. The elevation of the weirs is 1060.0 feet MSL.
Slide gate	A 12-inch by 12-inch slide gate is located on the upstream wall of the drop inlet and has an invert elevation of 1051.5 feet MSL.
Foundation	The conduit cradle and backfill are set on the natural bedrock. The backfilled trench is filled with compacted Zone 1 material. See Exhibits C-4 and C-5 for limits of excavation.
Antiseep collars	There are two antiseep collars located on 16-foot centers with the upstream most collar being 10 feet upstream of the centerline of the dam.
Plunge basin	The plunge basin is a rock-lined earthcut with a 20-foot-wide bottom and 2.5H:1V side slopes.

(2) *Emergency spillway.*

Type	An uncontrolled earthcut channel approx-

i. *Spillways.* (cont'd)

imately 1300 feet in length with variable side slopes

Width	400 feet
Crest elevation	1089.1 feet MSL

j. *Regulating outlets.*

The only regulating outlet is a 12-inch by 12-inch manually operated slide gate located on the upstream wall of the service spillway drop inlet. The gate invert elevation is 1051.5 feet MSL.

PROBABLE MAXIMUM FLOOD INFLOW HYDROGRAPH (cont'd)

Time (hours)	*¼-hour average inflow (CFS)*	*Time (hours)*	*¼-hour average inflow (CFS)*
10.00	4,572	19.50	3,592
10.50	4,574	20.00	3,021
11.00	4,574	20.50	2,837
11.50	4,574	21.00	2,776
12.00	4,574	21.50	2,754
12.50	4,745	22.00	2,747
13.00	5,831	22.50	2,744
13.50	7,350	23.00	2,744
14.00	9,414	23.50	2,744
14.50	13,001	24.00	2,744
15.00	21,233	24.50	2,490
15.50	44,818	25.00	1,210
16.00	63,055	25.50	427
16.50	44,456	26.00	144
17.00	26,944	26.50	50
17.50	16,903	27.00	18
18.00	11,595	27.50	6
18.50	8,317	28.00	2
19.00	5,176	28.50	0

Note: Peak inflow occurs at 16.0 hours and is 63,055 CFS.

(4) *Probable maximum flood outflow.* The probable maximum flood inflow hydrograph was routed through the structure beginning at elevation 1083.7 feet MSL. The outflow hydrograph is tabulated as follows:

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH

Time (hours)	*Instantaneous outflow (CFS)*	*Time (hours)*	*Instantaneous outflow (CFS)*
0.50	122	12.50	4,457
1.00	122	13.00	4,890
1.50	123	13.50	5,737
2.00	124	14.00	7,038
2.50	124	14.50	9,156
3.00	125	15.00	13,424
3.50	126	15.50	25,510
4.00	126	16.00	44,398
4.50	127	16.50	46,701
5.00	127	17.00	38,140
5.50	127	17.50	28,076
6.00	128	18.00	20,105
6.50	128	18.50	15,041
7.00	129	19.00	10,970
7.50	271	19.50	7,979
8.00	685	20.00	5,967
8.50	1,528	20.50	4,723
9.00	2,330	21.00	4,049
9.50	2,950	21.50	3,623
10.00	3,474	22.00	3,335
10.50	3,837	22.50	3,140
11.00	4,080	23.00	3,010
11.50	4,242	23.50	2,936
12.00	4,352		

(table cont'd)

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH (cont'd)

Time (hours)	*Instantaneous outflow (CFS)*	*Time (hours)*	*Instantaneous outflow (CFS)*
24.00	2,882	26.50	1,149
24.50	2,804	27.00	878
25.00	2,446	27.50	717
25.50	1,924	28.00	637
26.00	1,491	28.50	565

Note: Peak outflow occurs at 16.25 hours and is 47,684 CFS.

(5) *Discharge-frequency.* No data are available.

(6) *Reservoir area and storage capacity.* An area-capacity table for SCS Site No.5 is presented following. These data were determined by the Soil Conservation Service.

RESERVOIR AREA AND STORAGE CAPACITY TABLE

Elevation (feet MSL)	*Reservoir area (acres)*	*Storage capacity (acre-feet)*
1042.0	1.5	3
1046.0	4.0	14
1050.0	8.0	38
1054.0	15.0	84
1058.0	21.0	156
1060.0	25.0	198
1061.0	28.0	231
1062.0	30.0	258
1066.0	47.0	412
1070.0	67.0	640
1074.0	97.0	968
1078.0	124.0	1410
1082.0	156.0	1970
1086.0	193.0	2668
1089.1	218.0	3293
1090.0	228.0	3510
1094.0	269.0	4504
1098.0	310.0	5662

(7) *Spillway rating table.* A spillway rating table for SCS Site No.5 is presented following. Discharge values at and above elevation 1062 feet MSL were computed by the Soil Conservation Service. Discharge at elevation 1061 feet MSL was computed as weir flow through two notches in the crest of the service spillway inlet tower.

SPILLWAY RATING TABLE

Elevation (feet MSL)	*Combined spillway discharge (CFS)*
1060.0	0.0
1061.0	12.0
1062.0	84.0
1070.0	100.0
1078.0	113.0
1086.0	126.0
1089.1	130.0
1090.0	756.8
1091.0	2,407.4

(table cont'd)

SPILLWAY RATING TABLE (cont'd)

Elevation (feet MSL)	*Combined spillway discharge (CFS)*
1092.0	4,804.8
1094.0	11,271.7
1096.0	19,467.7
1098.5	32,049.2

(8) *Tailwater rating table.* A tailwater rating table for SCS Site No.5 is presented following. These data were computed using the Manning equation based on an idealized valley cross section and an average valley slope immediately downstream of the dam.

TAILWATER RATING TABLE

Elevation (feet MSL)	*Discharge (CFS)*
1035.0	0
1038.4	714
1041.8	3,032
1045.1	7,509
1048.5	14,657
1051.9	24,943
1055.5	44,220
1059.1	72,718

(9) *Hydrologic network.* A map of the drainage area of SCS Site No.5 is presented in Exhibit A-3.

(10) *Breach analysis.* A breach analysis was performed to investigate the effects on the downstream area under five different conditions. These were:

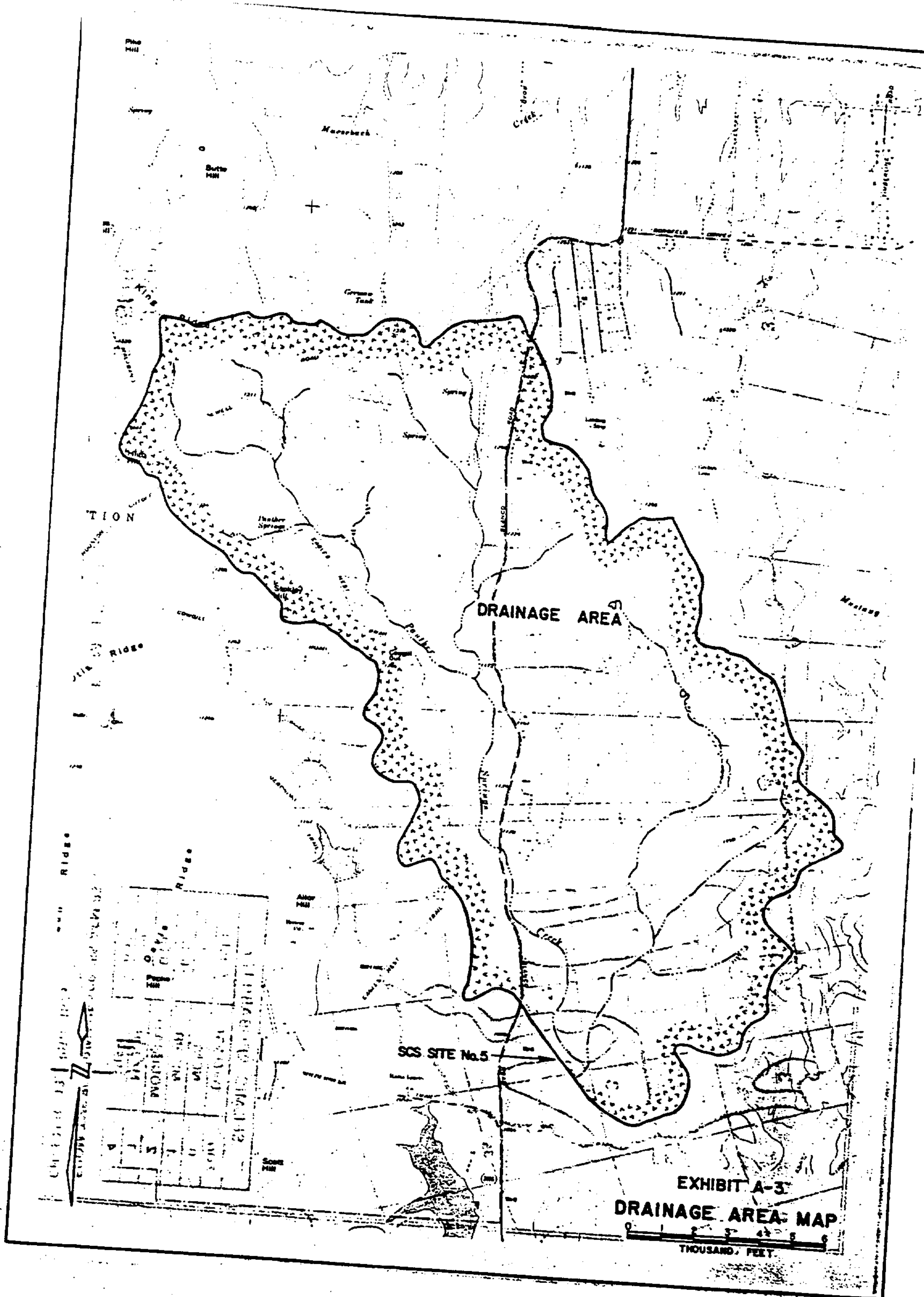
1) Breaching of the dam with reservoir at normal operating level with no inflow.
2) Breaching due to overtopping by the PMF.
3) Overtopping without breaching by the PMF.
4) Breaching due to a barely overtopping flood that represents 70% or the PMF.
5) Overtopping without breaching due to a barely overtopping flood that represents 70% of the PMF.

Results of the breach analysis for selected locations are graphically illustrated by computer plots, which appear as Exhibit A-4.

The dam is classified as a high-hazard structure due to the presence of scattered dwellings at various points downstream of the structure and to the presence of urban development at the urban outskirts of the City of San Antonio, approximately 8.3 miles downstream of the structure. A point of interest was chosen 8.35 miles downstream from the dam at which point the appropriate USGS 7.5-minute topographic quadrangle, photo-revised in 1973, shows dense development, the lowest of which appears to be between elevations 802 and 812 feet MSL. The following table shows the maximum water surface elevations and discharges at the point of interest for the five conditions investigated.

Condition	*Maximum water surface elevation (feet MSL)*	*Maximum discharge (CFS)*
Breaching at conservation pool with no inflow	790.9	955
Breaching due to overtopping by PMF	807.8	105,676

(table cont'd)


DRAINAGE AREA
SCS SITE No.5
EXHIBIT A-3
DRAINAGE AREA MAP
0 1 2 3 4 5 6
THOUSAND FEET

INPUT DATA FOR DAMS2 -TEXAS DESIGN SECTION

1 -10	11-20	21-30	31-40	41-50	51-60	61-70
	12/03/82			DATE	(FILE NAME	)
DAMS2	~~[illegible]~~				(/DAMS2.	)
STRUCTURE		WATERSHED AND SITE Salado Cr S, 6 5			McGill	
		ELEV	SURF AC	PS DIS	ES DIS	AC FT
		1061.				231.
		1062.				258.
		1066.				412.
		1070.				640.
		1074.				968.
		1078.				1410.
		1082.				1970.
		1086.				2668.
		1089.1				3293.
		1090.				3510.
		1094.				4504.
		1098.				5662.
END TABLE						
	class/16	cn	sq.mi.	Tc		base>csm
WS DATA	4C 2	76	8.86	3.0		
		ac 1 day	ac 10 day			
AREACRCT	4	1.	1.	1.0		
	clim.index	p 1da-in	p 10da-in	ac.esh-in	ac.fbh-in	p 100-10c
STORM	—	—	24.	—	—	—
		perm.pool	crest ps	sec el.		
POOL DATA	ELEV	1061.	1061.	1061.		
		ps length	size (in)		manning"n"	tw-outlet
PS DATA	1	280.	30.		.012	1042.
		k	wier length			
PS INLET	ELEV	1.	15.			
			es crest elevations			
ES CREST	ELEV	1089.1				
		ent+crest	"n"	z	exit "n"	% exit
ES DATA	41	700.	.040	3.	.040	7.67
BTM WIDTH	FEET	400.				
TRAPW						
		crest l		ent+crest	depth	
ESPROFILE	41	50.	10	700.	2.0	
ESPROFILE						
	~~dist~~	~~elev~~	~~dist~~	~~elev~~	dist	elev
GO, STORM	2PC	—	9.8			
GO, REACH	2 C6	—	28905	.65		
~~ENDJOB~~						
~~ENDRUN~~						

```
PERM POOL      1061.00 FT     231.0 ACFT     0.0 AC      0.0 CFS
CREST PS       1061.00 FT     231.0 ACFT     0.0 AC      0.0 CFS
SED ACCUM      1061.00 FT     231.0 ACFT     0.0 AC      9.0 CFS
ES CREST       1089.10 FT    3293.0 ACFT     0.0 AC    131.9 CFS
PS STORAGE     3062.0 ACFT    BETWEEN ES CREST AND SED ACCUM ELEVATIONS
STARTING E     1061.00 FT     231.0 ACFT     0.0 AC      0.0 CFS
STORM HYD      D=  24.00 HR    P=  9.80 IN     Q=  6.82 IN
              TC=   3.00 HR   CN=  76.00     VOL=  3222.4 ACFT
     PEAK      5644.7 CFS AT   11.12 HRS

************************************************************************
RATING TABLE DEVELOPED
 BY PROGRAM FOR PS AND EMG SPILLWAYS
 EMG. RATING USED TRAPW METHOD
```

RATING TABLE NUMBER 2

	ELEV FEET	Q-TOTAL CFS	Q-PS CFS	VOLUME AC-FT	AREA ACRE
1	1061.00	0.0	0.0	231.00	0.0
2	1061.38	10.87	10.87	241.24	0.0
3	1061.76	30.74	30.74	251.49	0.0
4	1062.14	56.46	56.46	263.32	0.0
5	1062.52	86.93	86.93	277.93	0.0
6	1065.84	93.82	93.82	405.85	0.0
7	1069.16	100.14	100.14	592.30	0.0
8	1072.49	106.09	106.09	843.84	0.0
9	1075.81	111.72	111.72	1167.85	0.0
10	1079.13	117.08	117.08	1568.39	0.0
11	1082.45	122.21	122.21	2049.24	0.0
12	1085.78	127.13	127.13	2629.06	0.0
13	1089.10	131.87	131.87	3292.95	0.0
14	1089.54	279.29	132.49	3400.18	0.0
15	1089.99	550.11	133.11	3507.47	0.0
16	1090.79	1399.94	134.21	3706.45	0.0
17	1091.77	3023.24	135.55	3949.73	0.0
18	1093.55	7251.57	137.95	4392.12	0.0
19	1095.77	14758.68	140.90	5017.83	0.0
20	1098.00	24708.05	143.79	5662.00	0.0

FULL CONDUIT FLOW, ELEV = 1062.52 FT

TYPE	BW	EMAX	VOL-MAX	AMAX	HP	VOL-ES	Q-PS	Q-ES	Q-TOT	D/C	V/C	S/C	S/C.25	D-ES	QE/B
STORM HYD	400.0	1088.81	3235.1	0.0	0.0	0.0	131.5	0.0	131.5	0.0	0.0	0.0	0.0	0.0	0.0

```
PLOT                                1 IN =   1000. CFS
                                                                      EXIT SLOPE = 0.077
                                    0.     1000.    2000.    3000.    4000.    5000.    6000.    7000. EXIT
  T       I      O      E      VOL      A  I        I        I        I        I        I        I        I   VEL
 0.0      0      0 1061.0   231.0   0.0 .                                                                0.0
 0.34     0      0 1061.0   231.0   0.0 .                                                                0.0
 0.67     0      0 1061.0   231.0   0.0 .                                                                0.0
 1.01     0      0 1061.0   231.0   0.0 .                                                                0.0
 1.35     0      0 1061.0   231.0   0.0 .                                                                0.0
 1.69     0      0 1061.0   231.0   0.0 .                                                                0.0
 2.02     0      0 1061.0   231.0   0.0 .                                                                0.0
 2.35     0      0 1061.0   231.0   0.0 .                                                                0.0
                                                                                                         0.0
```

UNCONTROLLED AREA HYD STORM PEAK = 3452.85 AT 10.73 HRS. LOCATION POINT 6

ADDHYD RESULTS STORM HYD - PEAK = 3571.82 AT 10.73 HRS. LOCATION POINT 6

PERM POOL	881.60 FT	100.0 ACFT	0.0 AC	0.0 CFS
CREST PS	881.60 FT	100.0 ACFT	0.0 AC	0.0 CFS
SED ACCUM	881.60 FT	100.0 ACFT	0.0 AC	0.0 CFS
ES CREST	910.90 FT	1490.0 ACFT	0.0 AC	278.4 CFS
PS STORAGE	1390.0 ACFT	BETWEEN ES CREST AND SED ACCUM ELEVATIONS		
STARTING E	881.60 FT	100.0 ACFT	0.0 AC	0.0 CFS

STORM HYD D= 24.00 HR P= 9.90 IN Q= 6.82 IN
TC= 2.10 HR CN= 76.00 VOL= 1665.7 ACFT
PEAK 3571.8 CFS AT 10.73 HRS

****

RATING TABLE DEVELOPED
BY PROGRAM FOR PS AND EMG SPILLWAYS
EMG. RATING USED TRAPW METHOD

RATING TABLE NUMBER 2

	ELEV FEET	Q-TOTAL CFS	Q-PS CFS	VOLUME AC-FT	AREA ACRE
1	881.60	0.0	0.0	100.00	0.0
2	882.09	22.43	22.43	107.99	0.0
3	882.58	63.44	63.44	115.97	0.0
4	883.07	116.54	116.54	123.96	0.0
5	883.57	179.43	179.43	131.94	0.0
6	886.98	194.85	194.85	202.38	0.0
7	890.40	208.84	208.84	294.18	0.0
8	893.82	221.94	221.94	411.37	0.0
9	897.23	234.31	234.31	556.09	0.0
10	900.65	246.06	246.06	732.24	0.0
11	904.07	257.28	257.28	949.95	0.0
12	907.48	268.02	268.02	1205.35	0.0
13	910.90	278.35	278.35	1489.98	0.0
14	911.53	380.99	280.22	1550.86	0.0
15	912.16	577.37	282.07	1611.77	0.0
16	913.29	1364.48	285.37	1721.40	0.0
17	914.68	3071.63	289.35	1855.37	0.0
18	917.20	8059.52	296.46	2119.47	0.0
19	920.35	17472.11	305.11	2477.18	0.0
20	923.50	29720.61	313.52	2830.00	0.0

FULL CONDUIT FLOW, ELEV = 883.57 FT

TYPE	BW	EMAX	VOL-MAX	AMAX	HP	VOL-ES	Q-PS	Q-ES	Q-TOT	D/C	V/C	S/C	S/C.25	D-ES	DE/B
STORM HYD	300.0	911.80	1577.0	0.0	0.90	97.0	281.0	184.3	465.3	0.23	2.71	0.038	0.052	12.9	0.3

PLOT 1 IN = 1000. CFS EXIT SLOPE = 0.018

0. 1000. 2000. 3000. 4000. 5000. 6000. 7000. EXIT

T	I	O	E	VOL	A	I I I I I I I I	VEL
0.0	0	0	881.6	100.0	0.0	.	0.0
0.31	0	0	881.6	100.0	0.0	.	0.0
0.61	0	0	881.6	100.0	0.0	.	0.0
0.92	0	0	881.6	100.0	0.0	.	0.0
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.0	.	0.0

INPUT DATA FOR DAMS2 -TEXAS DESIGN SECTION

1-10	11-20	21-30	31-40	41-50	51-60	61-70
	12/03/82			DATE	(FILE NAME	
DAMS2	~~[illegible]~~				(	/DAMS2.
STRUCTURE		WATERSHED AND SITE Salado Creek WS Site 7 McGill				
		ELEV	SURF AC	PS DIS	ES DIS	AC FT
		818.				94.
		822.				208.
		826.				457.
		830.				881.
		834.				1478.
		838.				2252.
		838.4				2340.
		842.				3255.
		846				4479.
		850.				5886.
		852.6				6864.
END TABLE						
	class (IL)	cn	sq.mi.	Tc		base>csr
WS DATA	4 5 2	82	5.80	2.4		
		ac 1 day	ac 10 day			
AREACRCT	4	1.	1.	1.0		
	clim.index	p 1da-in	p 10da-in	ac.esh-in	ac.fbh-in	p 100-10c
STORM	—	—	24.	—	—	—
		perm.pool	crest ps	sed el.		
POOL DATA	ELEV	818.	818.	818.		
		ps length	size (in)		manning"n"	tw-outlet
PS DATA	1	368.	60.		.012	804.
		k	wier length			
PS INLET	ELEV	1.	24.			
			es crest elevations			
ES CREST	ELEV	838.4				
		ent+crest	"n"	z	exit "n"	z exit
ES DATA	41	600.	.040	3.	.040	3.0
BTM WIDTH	FEET	350				
TRAPW						
		crest l		ent+crest	depth	
ESPROFILE	41	600. 10		700.	1.0	
~~ESPROFILE~~						
	~~dist~~	elev	~~dist~~	~~elev~~	dist	elev
GO, STORM	2PC	—	9.8			
~~GO, REACH~~		—				
ENDJOB						
END RUN						

SAN ANTONIO RIVER BASIN

SCS FLOODWATER RETARDING DAM

SALADO CREEK WS SITE No.8

BEXAR COUNTY, TEXAS

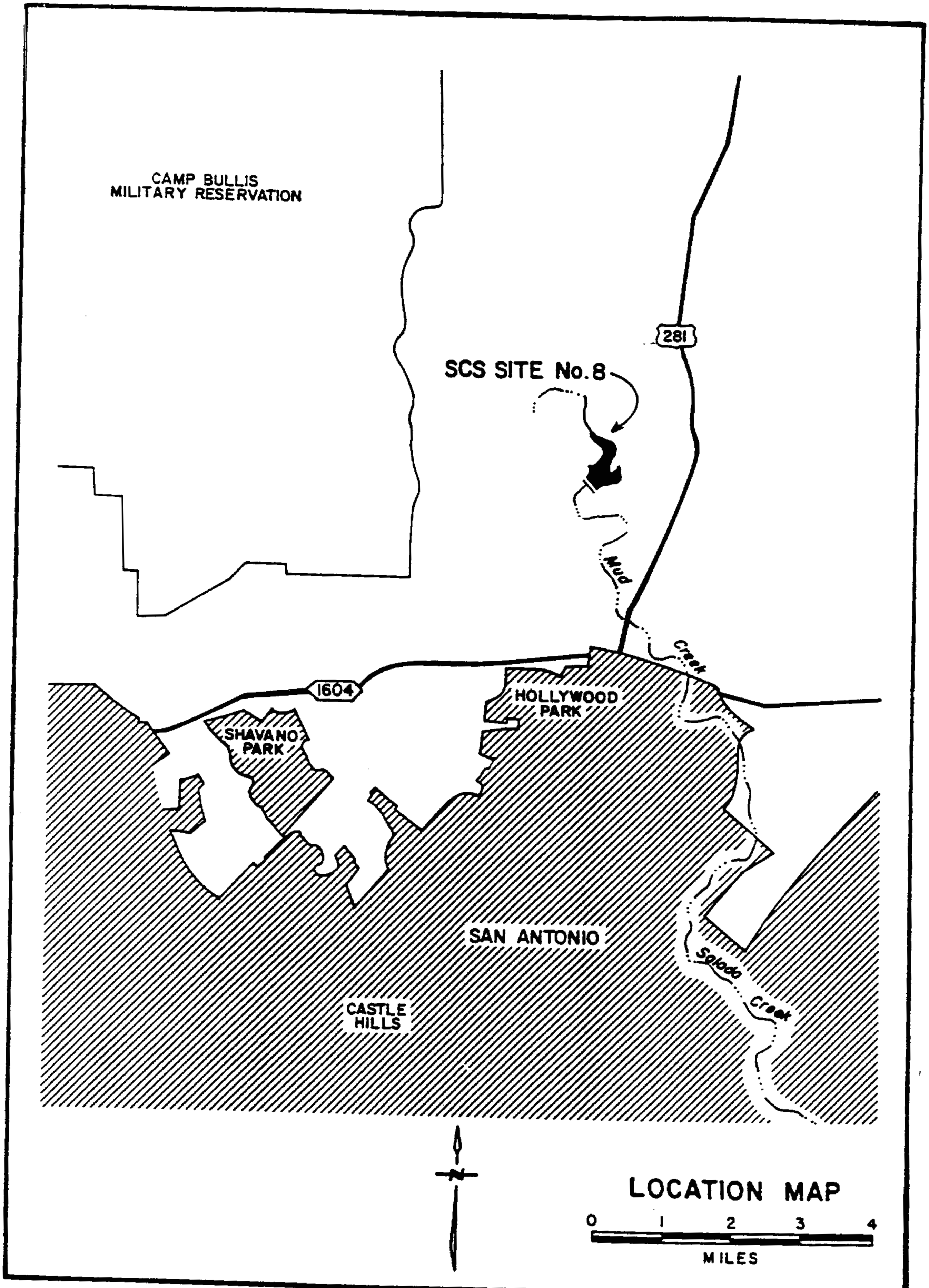
INVENTORY NUMBER TX01467

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

U. S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

MARCH 1981

REPORT PREPARED BY
TEXAS DEPARTMENT OF WATER RESOURCES
AUSTIN, TEXAS


CAMP BULLIS
MILITARY RESERVATION
281
SCS SITE No. 8
Mud
Creek
1604
HOLLYWOOD
PARK
SHAVANO
PARK
SAN ANTONIO
Salado
Creek
CASTLE
HILLS
N
LOCATION MAP
0 1 2 3 4
MILES

Mr. Fred H. P
San Antonio R
c/o Mr. Ervin
District Cons
Soil Conserv
San Antonio
630 South Ma
San Antonio,

Dear Mr. Pfe:

Re: Salado
SCS Sit
Nationa

Enclosed is
SCS Site No.
the National
report with
Colonel Dona
a copy of Ge
Under The Na

The letter o
this report,
letter to Co

Please feel
is 512/475-

Very truly

John R. Cla
Head, Dam S

Enclosures

f. *Purpose of dam.* SCS Site No.8 was designed as a floodwater-retarding structure.

g. *Design and construction history.* The dam was completed in May 1973 and was designed by the United States Department of Agriculture, Soil Conservation Service. The geological investigations, soil testing, and construction supervision were all conducted by the SCS. The contractor was William A. Pfeuffer.

h. *Normal operational procedures.* The spillways at SCS Site No.8 are uncontrolled; therefore, no operational procedures exist for periods of flooding. The operational procedures for the maintenance of the facility are discussed in Section 4.

1.3 *Pertinent Data.*

a. *Drainage area.*	11.2 square miles
b. *Discharge at dam site (CFS).*	
Maximum flood at dam site	Unknown
Combined spillway discharge at the effective crest elevation	45,528
c. *Elevation (feet above mean sea level).*	
Top of dam (effective crest)	1077.1
Maximum pool (PMF)	1079.8
Emergency spillway crest	1065.6
Service spillway crest	1035.5
Normal pool (ports)	1032.8
Low flow (slide gate invert)	1021.0
d. *Reservoir (length in miles).*	
Top of dam (effective crest)	1.4
Emergency spillway crest	1.1
Normal pool (ports)	0.5
e. *Storage (acre-feet).*	
Top of dam (effective crest)	7100
Maximum pool (PMF)	7808
Emergency spillway crest	4178
Service spillway crest	297
Normal pool (ports)	196
f. *Reservoir surface (acres).*	
Top of dam (effective crest)	377
Maximum pool (PMF)	441
Emergency spillway crest	232
Service spillway crest	43
Normal pool (ports)	32
g. *Dam.*	
Type	Earthfill
Length	1675 feet

g. *Dam.* (cont'd)	
Height	61.1 feet
Top width	14 feet
Upstream slope	2H:1V
Downstream slope	2H:1V
Cutoff	There is no cutoff; the dam is founded on bedrock.
Zoning	SCS Site No 8 is a zoned embankment having a small core of compacted clay and an outer shell of gravelly clay. Both the upstream and downstream slopes have an additional rock-fill zone.
Grout curtain	All open fractures in the bedrock under the core were dental grouted.
h. *Diversion and regulating tunnel.*	None
i. *Spillways.*	
(1) *Service spillway.*	
Type	The service spillway is uncontrolled and consists of 272 feet of 30-inch ID prestressed concrete-lined, steel cylinder pipe through the embankment. A 7.5-foot-long by 2.5-foot-wide drop inlet serves as an intake. There are also six (three each side) 9-inch by 9-inch ports in the inlet.
Crest elevation	1035.5 feet MSL
Ports	1032.8 feet MSL
Conduit invert at bottom of inlet	1020.5 feet MSL
Conduit invert at outlet	1018.0 feet MSL
(2) *Emergency spillway.*	
Type	An uncontrolled earthcut channel approximately 400 feet in width
Crest elevation	1065.6 feet MSL
j. *Regulating outlets.*	A 12-inch by 12-inch slide gate located in the upstream side of the drop inlet at an invert elevation of 1021 feet MSL is used as a low-flow outlet.

TEXAS WATER DEVE...
Louis A. Beechol...
John H. Garrett, ...
George W. McCle...
Glen E. Roney...
W. O. Bankston...
Lonnie A. "Bo" P...

Mr. Fred N. P...
San Antonio R...
c/o Mr. Ervin...
District Cons...
Soil Conservat...
San Antonio F...
630 South Mair...
San Antonio, T...

Dear Mr. Pfeif...

Re: Salado Cr...
SCS Site...
National...

Enclosed is a ...
SCS Site No. 8 ...
the National D...
report with an...
Colonel Donald...
a copy of Gene...
Under The Natio...

The letter of ...
this report, is...
letter to Cover...

Please feel fre...
is 512/475-8220

Very truly your...

John R. Clarke,
Head, Dam Safety

Enclosures

PROBABLE MAXIMUM PRECIPITATION (cont'd)

Time (hours)	Critical arrangement (inches)	Rainfall excess (inches)	Time (hours)	Critical arrangement (inches)	Rainfall excess (inches)
15.50	2.18	2.16	17.25	0.41	0.39
15.75	1.52	1.50	17.50	0.37	0.36
16.00	1.05	1.03	17.75	0.33	0.32
16.25	0.80	0.78	18.00	0.30	0.28
16.50	0.65	0.64	18.25	0.14	0.13
16.75	0.54	0.53	18.50	0.14	0.13
17.00	0.47	0.45	24.00	0.14	0.13
			TOTALS	35.43	33.86

(2) *Unit hydrograph.* The ¼-hour Snyder synthetic unit hydrograph computed for the drainage area of SCS Site No.8 is shown following:

UNIT HYDROGRAPH

L = 6.52 miles Lca = 3.37 miles Ct = 0.28 Cp640 = 418

Time (hours)	¼-hour average discharge (CFS)	Time (hours)	¼-hour average discharge (CFS)
0.25	70	3.00	238
0.50	1659	3.25	151
0.75	5102	3.50	97
1.00	6341	3.75	63
1.25	5327	4.00	41
1.50	3770	4.25	27
1.75	2468	4.50	18
2.00	1560	4.75	12
2.25	973	5.00	8
2.50	605	5.25	3
2.75	378		

(3) *Probable maximum flood inflow.* Rainfall excess values derived for the SCS Site No.8 drainage area were applied to the unit hydrograph to compute the PMF. Base flow was ignored since its magnitude is very small compared to the peak flow. To simulate saturated conditions in the watershed, a flood approximately one-half the magnitude of the PMF was initially routed through the reservoir. The PMF was routed 5 days (120 hours) after the beginning of the antecedent storm by which time the reservoir level had receded to elevation 1061.3 feet MSL, which is 28.5 feet above the top of the conservation pool. The PMF used in the routing to determine the maximum water surface elevation is tabulated as follows:

PROBABLE MAXIMUM FLOOD INFLOW HYDROGRAPH

Time (hours)	½-hour average inflow (CFS)	Time (hours)	½-hour average inflow (CFS)
0.5	138	5.0	2,313
1.0	1,054	5.5	2,313
1.5	1,782	6.0	2,313
2.0	2,104	6.5	2,520
2.5	2,230	7.0	3,894
3.0	2,279	7.5	4,985
3.5	2,299	8.0	5,469
4.0	2,307	8.5	5,658
4.5	2,311		

(table cont'd)

PROBABLE MAXIMUM FLOOD INFLOW HYDROGRAPH (cont'd)

Time (hours)	½-hour average inflow (CFS)	Time (hours)	½-hour average inflow (CFS)
9.0	5,732	19.0	7,883
9.5	5,761	19.5	5,475
10.0	5,774	20.0	4,440
10.5	5,779	20.5	4,011
11.0	5,782	21.0	3,855
11.5	5,782	21.5	3,796
12.0	5,782	22.0	3,772
12.5	5,922	22.5	3,763
13.0	7,045	23.0	3,759
13.5	8,808	23.5	3,758
14.0	11,164	24.0	3,758
14.5	15,096	24.5	3,534
15.0	23,872	25.0	2,046
15.5	48,380	25.5	863
16.0	75,124	26.0	340
16.5	59,852	26.5	135
17.0	38,683	27.0	55
17.5	24,762	27.5	22
18.0	16,803	28.0	9
18.5	12,007	28.5	3

Note: Peak inflow occurs at 16 hours.

(4) *Probable maximum flood outflow.* The probable maximum flood inflow hydrograph was routed through the structure beginning at elevation 1061.3 feet MSL. The outflow hydrograph is tabulated as follows:

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH

Time (hours)	Instantaneous outflow (CFS)	Time (hours)	Instantaneous outflow (CFS)
0.5	130	12.0	5,673
1.0	130	12.5	5,741
1.5	130	13.0	6,100
2.0	131	13.5	6,922
2.5	132	14.0	8,253
3.0	132	14.5	10,678
3.5	133	15.0	15,708
4.0	134	15.5	31,110
4.5	134	16.0	56,996
5.0	135	16.5	61,300
5.5	136	17.0	49,393
6.0	137	17.5	35,336
6.5	202	18.0	24,799
7.0	592	18.5	17,946
7.5	1,321	19.0	13,551
8.0	2,469	19.5	10,016
8.5	3,437	20.0	7,918
9.0	4,250	20.5	6,543
9.5	4,786	21.0	5,589
10.0	5,140	21.5	4,950
10.5	5,369	22.0	4,531
11.0	5,517	22.5	4,258
11.5	5,612		

(table cont'd)

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH (cont'd)

Time (hours)	*Instantaneous outflow (CFS)*	*Time (hours)*	*Instantaneous outflow (CFS)*
23.0	4,080	26.0	2,002
23.5	3,966	26.5	1,492
24.0	3,892	27.0	1,154
24.5	3,798	27.5	962
25.0	3,305	28.0	800
25.5	2,640	28.5	668

Note: Peak outflow occurs at 16.25 hours and is 62,234 CFS.

(5) *Discharge-frequency.* No data are available.

(6) *Reservoir area and storage capacity.* An area-capacity table for SCS Site No.8 is presented following. These data were determined by the Soil Conservation Service.

RESERVOIR AREA AND STORAGE CAPACITY TABLE

Elevation (feet MSL)	*Reservoir area (acres)*	*Storage capacity (acre-feet)*
1020.0	3	7
1024.0	8	30
1028.0	17	77
1032.0	29	173
1032.8	32	196
1035.5	43	297
1036.0	44	321
1040.0	62	536
1044.0	84	826
1048.0	109	1208
1052.0	130	1690
1056.0	158	2268
1060.0	190	2964
1064.0	220	3786
1065.6	232	4178
1068.0	257	4738
1072.0	300	5851
1076.0	363	6810
1077.1	377	7100

(7) *Spillway rating table.* A spillway rating table for SCS Site No.8 is presented following. Discharge values at and below elevation 1035.5 feet MSL were computed assuming orifice flow through the ports in the service spillway inlet tower. Discharge values above 1035.5 feet MSL were determined by the Soil Conservation Service.

SPILLWAY RATING TABLE

Elevation (feet MSL)	*Combined spillway discharge (CFS)*
1032.80	0
1033.55	10
1035.50	25
1041.00	93
1048.00	107
1056.00	121

(table cont'd)

SPILLWAY RATING TABLE (cont'd)

Elevation (feet MSL)	*Combined spillway discharge (CFS)*
1064.00	134
1065.60	137
1066.00	265
1067.00	1,319
1068.00	3,290
1070.00	9,245
1072.00	17,508
1074.00	27,482
1076.00	38,684
1077.10	45,528

(8) *Tailwater rating table.* A tailwater rating table for SCS Site No.8 is presented following. These data were computed using the Manning equation based on an idealized valley cross section and an average valley slope immediately downstream of the dam.

TAILWATER RATING TABLE

Elevation (feet MSL)	*Discharge (CFS)*
1016.0	0
1017.0	80
1018.0	320
1019.0	767
1020.0	1,462
1021.0	2,449
1023.6	7,683
1026.2	16,928
1028.8	31,300
1031.4	51,766
1034.0	79,209

(9) *Hydrologic network.* A map of the drainage area of SCS Site No.8 is presented in Exhibit A-3.

(10) *Breach analysis.* A breach analysis was performed to investigate the effects on the downstream area under five different conditions. These were:

1) Breaching of the dam with reservoir at normal operating level with no inflow.

2) Breaching due to overtopping by the PMF.

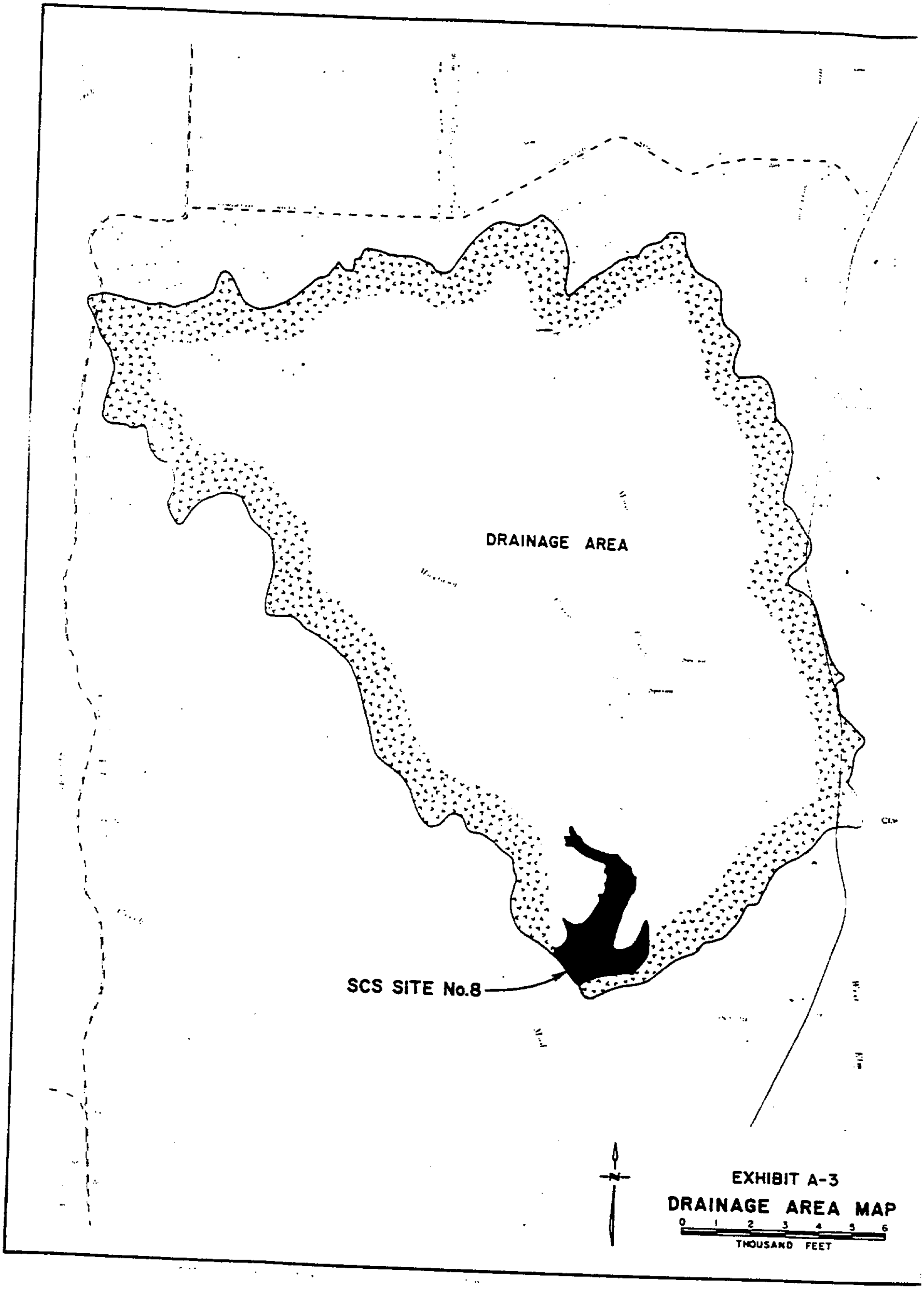
3) Overtopping without breaching by the PMF.

4) Breaching due to a barely overtopping flood that represents 76% of the PMF.

5) Overtopping without breaching due to a barely overtopping flood that represents 76% of the PMF.

Results of the breach analysis for selected locations are graphically illustrated by computer plots that appear as Exhibit A-4.

The dam is classified as a high-hazard structure due to the presence of a number of dwellings at various points near Mud Creek downstream of the structure. A point of interest was chosen 10.9 miles downstream from the dam on the outskirts of the City of San Antonio at which point the appropriate USGS 7.5-minute topographical quadrangle, photo-revised in 1973, shows development, within which several dwellings appear to be between elevations 715 and 725 feet MSL. The following table shows the maximum water surface elevations and discharges at the point of interest for the five conditions investigated.


DRAINAGE AREA
SCS SITE No.8
N
EXHIBIT A-3
DRAINAGE AREA MAP
0 1 2 3 4 5 6
THOUSAND FEET

INPUT DATA FOR DAMS2 -TEXAS DESIGN SECTION

1 -10	11-20	21-30	31-40	41-50	51-60	61-70
DAMS2	12/03/82 ~~[illegible]~~			DATE	(FILE NAME (Baxgovca/DAMS2.DE	
STRUCTURE		WATERSHED AND SITE Salado Creek DS		Site 8		McGill
		ELEV	SURF AC	PS DIS	ES DIS	AC FT
		1035.5				297.
		1036.				321.
		1040.				536.
		1044.				826.
		1048.				1208.
		1052.				1690.
		1056.				2268.
		1060				2964.
		1064				3786.
		1065.6				4178.
		1068.				4738.
		1072.				5851.
		1076.				6810.
		1077.1				7100.
END TABLE						
WS DATA	4 class C (16) 2	cn 77	sq.mi. 11.18	Tc 3.3		base>csm
AREACRCT	4	ac 1 day 1.	ac 10 day 1.	1.0		
STORM	clim.index —	p 1da-in —	p 10da-in 24.	ac.esh-in —	ac.fbh-in —	p 100-10c —
POOL DATA ELEV		perm.pool 1035.5	crest ps 1035.5	sec el. 1035.5		
PS DATA	1	ps length 272.	size (in) 30.		manning"n" .012	tw-outlet 1018.
PS INLET	ELEV	k 1.	wier length 15.			
ES CREST	ELEV	1065.6	es crest elevations			
ES DATA	41	ent+crest 230	"n" .040	z 2.0	exit "n" 040	% exit 3.0
BTM WIDTH FEET		400.				
TRAPW						
ESPROFILE	41	crest l 50	10	ent+crest 100.	depth 5.0	
ESPROFILE	230.	5.6				
GO, STORM	~~dist~~ 2PC	~~elev~~ —	~~dist~~ 9.8	~~elev~~	~~dist~~	~~elev~~
GO, REACH2	16 17 C9	—	10310.	.60		
~~ENDJOB~~						
~~ENDRUN~~						

PERM POOL	1035.50 FT	297.0 ACFT	0.0 AC	0.0 CFS
CREST PS	1035.50 FT	297.0 ACFT	0.0 AC	0.0 CFS
SED ACCUM	1035.50 FT	297.0 ACFT	0.0 AC	0.0 CFS
ES CREST	1065.60 FT	4179.0 ACFT	0.0 AC	133.6 CFS
FS STORAGE	3881.0 ACFT	BETWEEN ES CREST AND SED ACCUM ELEVATIONS		
STARTING E	1035.50 FT	297.0 ACFT	0.0 AC	0.0 CFS

STORM HYD   D= 24.00 HR   P= 9.80 IN   Q= 6.95 IN
TC= 3.30 HR   CN= 77.00   VCL= 4142.5 ACFT
PEAK   6952.3 CFS AT 11.42 HRS

****

RATING TABLE DEVELOPED
BY PROGRAM FOR PS AND EMG SPILLWAYS
EMG. RATING USED TRAPW METHOD

RATING TABLE NUMBER 2

	ELEV FEET	Q-TOTAL CFS	Q-PS CFS	VOLUME AC-FT	AREA ACRE
1	1035.50	0.0	0.0	297.00	0.0
2	1035.87	10.53	10.53	314.84	0.0
3	1036.24	29.79	29.79	334.07	0.0
4	1036.61	54.73	54.73	354.04	0.0
5	1036.99	84.26	84.26	374.02	0.0
6	1040.56	91.96	91.96	576.82	0.0
7	1044.14	98.98	98.98	839.34	0.0
8	1047.72	105.53	105.53	1180.91	0.0
9	1051.29	111.70	111.70	1604.80	0.0
10	1054.87	117.55	117.55	2104.66	0.0
11	1058.45	123.12	123.12	2693.65	0.0
12	1062.02	128.45	128.45	3379.72	0.0
13	1065.60	133.57	133.57	4177.94	0.0
14	1066.17	397.30	134.37	4312.09	0.0
15	1066.75	1108.73	135.17	4446.23	0.0
16	1067.78	3155.58	136.60	4687.70	0.0
17	1069.05	6604.16	138.32	5030.04	0.0
18	1071.35	15278.96	141.40	5670.03	0.0
19	1074.22	29419.71	145.16	6384.35	0.0
20	1077.10	46431.47	148.83	7100.00	0.0

FULL CONDUIT FLOW, ELEV = 1036.99 FT

TYPE	BW	EMAX	VOL-MAX	AMAX	HP	VOL-ES	Q-PS	Q-ES	Q-TOT	D/C	V/C	S/C	S/C.25	D-ES	DE/B
STORM HYD	400.0	1065.73	4207.5	0.0	0.13	29.5	133.7	57.9	191.6	0.09	1.68	0.052	0.071	5.5	0.0

PLOT   1 IN = 1000. CFS   EXIT SLOPE = 0.030
0. 1000. 2000. 3000. 4000. 5000. 6000. 7000. EXIT

T	I	O	E	VOL	A	EXIT VEL
0.0	0	0	1035.5	297.0	0.0	0.0
0.35	0	0	1035.5	297.0	0.0	0.0
0.69	0	0	1035.5	297.0	0.0	0.0
1.04	0	0	1035.5	297.0	0.0	0.0
1.38	0	0	1035.5	297.0	0.0	0.0
1.73	0	0	1035.5	297.0	0.0	0.0
2.08	0	0	1035.5	297.0	0.0	0.0
2.42	0	0	1035.5	297.0	0.0	0.0
2.77	0	0	1035.5	297.0	0.0	

INPUT DATA FOR DAMS2 -TEXAS DESIGN SECTION

1-10	11-20	21-30	31-40	41-50	51-60	61-70
				DATE	FILE NAME	
DAMS2	12/03/82 ~~00/00/00~~				(/DAMS2.	
STRUCTURE		WATERSHED AND SITE Salado Creek WS Site 9 MC5.11				
		ELEV	SURF AC	PS DIS	ES DIS	AC FT
		945.3				94.
		946.				104.
		950				203.
		954				353.
		958				558.
		962				826.
		964.5				1026.
		966.				1155.
		970.				1546.
		974.				2002.
		978.6				2612
END TABLE						
WS DATA	class(16) 4 C 2	cn 77	sq.mi. 2.37	Tc 1.5		base>csm
AREACRCT	4	ac 1 day 1.	ac 10 day 1.	1.0		
STORM	clim.index —	p 1da-in —	p 10da-in 24	ac.esh-in —	ac.fbh-in —	p 100-10 —
POOL DATA ELEV		perm.pool 945.3	crest ps 945.3	sed el. 945.3		
PS DATA	1	ps length 270.	size (in) 36.		manning"n" .012	tw-outle 930.
PS INLET	ELEV	k 1.	wier length 18.			
ES CREST	ELEV	964.5	es crest elevations			
ES DATA	41	ent+crest 500.	"n" .040	z 2.5	exit "n" .040	% exit 8.68
BTM WIDTH FEET		300				
TRAPW						
ESPROFILE	41	crest l 50	10	ent+crest 500	depth 2.5	
ESPROFILE						
	~~dist~~	~~elev~~	~~dist~~	~~elev~~	dist	~~elev~~
GO, STORM	2PC	—	9.8			
GO, REACH	2 C10	—	18430	.60		
~~END JOB~~						
~~END RUN~~						

UNCONTROLLED AREA HYD STORM PEAK = 2006.23 AT 10.39 HRS. LOCATION POINT 9

ADDHYD RESULTS STORM HYD - PEAK = 2095.04 AT 10.39 HRS. LOCATION POINT 9

PERM POOL	945.30 FT	94.0 ACFT	0.0 AC	0.0 CFS
CREST PS	945.30 FT	94.0 ACFT	0.0 AC	0.0 CFS
SED ACCUM	945.30 FT	94.0 ACFT	0.0 AC	0.0 CFS
ES CREST	964.50 FT	1026.0 ACFT	0.0 AC	174.0 CFS
PS STORAGE	932.0 ACFT	BETWEEN ES CREST AND SED ACCUM ELEVATIONS		
STARTING E	945.30 FT	94.0 ACFT	0.0 AC	0.0 CFS

STORM HYD D= 24.00 HR P= 9.80 IN Q= 6.95 IN
TC= 1.50 HR CN= 77.00 VOL= 878.1 ACFT
PEAK 2095.0 CFS AT 10.39 HRS

****
RATING TABLE DEVELOPED
BY PROGRAM FOR PS AND EMG SPILLWAYS
EMG. RATING USED TRAPW METHOD

RATING TABLE NUMBER 2

	ELEV FEET	Q-TOTAL CFS	Q-PS CFS	VOLUME AC-FT	AREA ACRE
1	945.30	0.0	0.0	94.00	0.0
2	945.72	15.14	15.14	99.99	0.0
3	946.14	42.83	42.83	107.42	0.0
4	946.56	78.69	78.69	117.79	0.0
5	946.98	121.15	121.15	128.17	0.0
6	949.17	129.69	129.69	182.38	0.0
7	951.36	136.90	136.90	253.90	0.0
8	953.55	143.75	143.75	336.04	0.0
9	955.74	150.29	150.29	442.09	0.0
10	957.93	156.55	156.55	554.35	0.0
11	960.12	162.58	162.58	699.98	0.0
12	962.31	168.39	168.39	850.77	0.0
13	964.50	174.00	174.00	1026.00	0.0
14	965.20	411.11	175.77	1086.62	0.0
15	965.91	983.67	177.52	1147.25	0.0
16	967.18	2665.93	180.63	1270.24	0.0
17	968.73	5675.54	184.36	1421.83	0.0
18	971.55	13402.90	190.95	1722.68	0.0
19	975.07	26771.11	198.89	2144.52	0.0
20	978.60	43862.64	206.52	2612.00	0.0

FULL CONDUIT FLOW, ELEV = 946.98 FT

TYPE	BW	EMAX	VOL-MAX	AMAX	HP	VOL-ES	Q-PS	Q-ES	Q-TOT	D/C	V/C	S/C	S/C.25	D-ES	O:
STORM HYD	300.0	962.90	898.0	0.0	0.0	0.0	169.9	0.0	169.9	0.0	0.0	0.0	0.0	0.0	

PLOT I IN = 500. CFS EXIT SLOPE = 0.087

T	I	O	E	VOL	A	0. I	500. I	1000. I	1500. I	2000. I	2500. I	3000. I	3500. I	EXIT VEL
0.0	0	0	945.3	94.0	0.0	.								0.0
0.29	0	0	945.3	94.0	0.0	.								0.0
0.59	0	0	945.3	94.0	0.0	.								0.0
0.87	0	0	945.3	94.0	0.0	.								0.0

INPUT DATA FOR DAMS2 -TEXAS DESIGN SECTION

1 -10	11-20	21-30	31-40	41-50	51-60	61-70
	12/03/82			DATE	(FILE NAME	
DAMS2	~~[illegible]~~				(/DAMS2.	
STRUCTURE		WATERSHED AND SITE Salado Cr WS site 10 McGill				
		ELEV	SURF AC	PS DIS	ES DIS	AC FT
		823.4				77.
		824.				108.
		828.				186.
		832.				296.
		836.				444.
		840.				634.
		844.				872.
		848.				1176.
		852.				1532.
		856.				1962.
		860.				2470.
		864.				3060.
		868.				3724.
		872.				4400.
		876.				5120.
		880.				5850.
		884.				6520
END TABLE						
WS DATA	4 C class 2	cn 81	sq.mi. 4.78	Tc 1.6		base>csm
AREACRCT	4	ac 1 day 1.	ac 10 day 1.	1.0		
STORM	clim.index —	p 1da-in —	p 10da-in 24.	ac.esh-in —	ac.fbh-in —	p 100-10c —
POOL DATA	ELEV	perm.pool 823.4	crest ps 823.4	sed el. 823.4		
PS DATA	1	ps length 290.	size (in) 42		manning"n" .012	tw-outlet 808.
PS INLET	ELEV	k 1.	wier length 21.			
ES CREST	ELEV	855.9	es crest elevations			
ES DATA	41	ent+crest 75	"n" .012	z 0	exit "n" .012	% exit 57.14
STM WIDTH	FEET	120				
TRAPW						
ESPROFILE	41	crest l 70.	10	ent+crest 75.	depth 2	
~~ESPROFILE~~						
GO, STORM	~~dist~~ 2PC	~~elev~~ —	~~dist~~ 9.8	~~elev~~	~~dist~~	~~elev~~
~~GO, REACH~~		—				
ENDJOB						
END RUN						

UNCONTROLLED AREA HYD STORM PEAK = 4305.25 AT 10.20 HRS. LOCATION POINT 10

ADDHYD RESULTS STORM HYD - PEAK = 4408.32 AT 10.20 HRS. LOCATION POINT 10

PERM POOL	823.40 FT	97.0 ACFT	0.0 AC	0.0 CFS
CREST PS	823.40 FT	97.0 ACFT	0.0 AC	0.0 CFS
SED ACCUM	823.40 FT	97.0 ACFT	0.0 AC	0.0 CFS
ES CREST	855.90 FT	1951.2 ACFT	0.0 AC	287.4 CFS

PS STORAGE 1854.2 ACFT BETWEEN ES CREST AND SED ACCUM ELEVATIONS

STARTING E	823.40 FT	97.0 ACFT	0.0 AC	0.0 CFS

STORM HYD D= 24.00 HR P= 9.80 IN Q= 7.46 IN
TC= 1.60 HR CN= 81.00 VOL= 1900.9 ACFT
PEAK 4408.3 CFS AT 10.20 HRS

****

RATING TABLE DEVELOPED
BY PROGRAM FOR PS AND EMG SPILLWAYS
EMG. RATING USED TRAPW METHOD

RATING TABLE NUMBER 2

	ELEV FEET	Q-TOTAL CFS	Q-PS CFS	VOLUME AC-FT	AREA ACRE	
1	823.40	0.0	0.0	97.00	0.0	
2	823.88	21.43	21.43	105.74	0.0	
3	824.35	60.62	60.62	114.89	0.0	
4	824.83	111.37	111.37	124.19	0.0	
5	825.31	171.47	171.47	133.49	0.0	FULL CONDUIT FLOW, ELEV = 825.31 F
6	829.13	190.87	190.87	217.11	0.0	
7	832.96	207.42	207.42	331.34	0.0	
8	836.78	222.74	222.74	481.00	0.0	
9	840.60	237.08	237.08	669.88	0.0	
10	844.43	250.60	250.60	903.81	0.0	
11	848.25	263.42	263.42	1192.71	0.0	
12	852.07	275.65	275.65	1540.06	0.0	
13	855.90	287.36	287.36	1951.13	0.0	
14	857.30	858.40	291.55	2127.60	0.0	
15	858.71	1973.10	295.67	2306.04	0.0	
16	861.24	4803.20	302.95	2652.61	0.0	
17	864.33	9329.27	311.62	3114.63	0.0	
18	868.95	19756.48	326.80	4053.46	0.0	
19	876.97	35421.09	344.83	5297.86	0.0	
20	884.00	51308.84	361.97	6520.00	0.0	

TYPE	BW	EMAX	VOL-MAX	AMAX	HP	VOL-ES	Q-PS	Q-ES	Q-TOT	D/C	V/C	S/C	S/C.25	D-ES
STORM HYD	120.0	854.42	1792.0	0.0	0.0	0.0	282.8	0.0	282.8	0.0	0.0	0.0	0.0	0.0

PLOT 1 IN = 1000. CFS EXIT SLOPE = 0.571

T	I	O	E	VOL	A	0. I	1000. I	2000. I	3000. I	4000. I	5000. I	6000. I	7000. I	EXIT VEL
0.0	0	0	823.4	97.0	0.0	.								0.0
0.29	0	0	823.4	97.0	0.0	.								0.0
0.58	0	0	823.4	97.0	0.0	.								0.0
0.97	0	0	823.4	97.0	0.0	.								0.0

INPUT DATA FOR DAMS2 -TEXAS DESIGN SECTION

1 -10	11-20	21-30	31-40	41-50	51-60	61-70
	12/03/82			DATE	(FILE NAME	
DAMS2	~~[illegible]~~			[illegible]	(_______	/DAMS2.CF
STRUCTURE		WATERSHED AND SITE				
		Salado Creek WS Site 11			McGill	
		ELEV	SURF AC	PS DIS	ES DIS	AC FT
		845.3				24.
		848.				129.
		852.				240.
		856.				394.
		860.				603.
		864.				878.
		868.				1237.
		872.				1701.
		876.				2287.
		877.8				2596.
		880.				3010.
		884.				3874.
		888.				4881.
		893.				6318.
END TABLE						
	class (1/2)	cn	sq.mi.	Tc		base>csr
WS DATA	4 C 2	81	6.56	2.6		
		ac 1 day	ac 10 day			
AREACRCT	4	1.	1.	1.0		
	clim.index	p 1da-in	p 10da-in	ac.esh-in	ac.fbh-in	p 100-10c
STORM	—	—	24.	—	—	—
		perm.pool	crest ps	sec el.		
POOL DATA	ELEV	845.3	845.3	845.3		
		ps length	size (in)		manning "n"	tw-outlet
PS DATA	1	200.	30.		.012	834.
		k	wier length			
PS INLET	ELEV	1.	15.			
			es crest elevations			
ES CREST	ELEV	877.8				
		ent+crest	"n"	z	exit "n"	% exit
ES DATA	41	350	.040	3.	.040	5.5
BTM WIDTH	FEET	75				
TRAPW						
		crest l		ent+crest	depth	
ESPROFILE	41	50.	10	350.	1.2	
~~ESPROFILE~~						
	~~dist~~	~~elev~~	~~dist~~	~~elev~~	~~dist~~	~~elev~~
GO, STORM	2PC	—	9.8			
~~GO, REACH~~		—				
ENDJOB						
END RUN						

PERM POOL	845.30 FT	84.0 ACFT	0.0 AC	0.0 CFS
CREST PS	845.30 FT	84.0 ACFT	0.0 AC	0.0 CFS
SED ACCUM	845.30 FT	84.0 ACFT	0.0 AC	0.0 CFS
ES CREST	877.80 FT	2596.0 ACFT	0.0 AC	137.9 CFS

ES STORAGE 2512.0 ACFT BETWEEN ES CREST AND SED ACCUM ELEVATIONS

STARTING E 845.30 FT 84.0 ACFT 0.0 AC 0.0 CFS

STORM HYD D= 24.00 HR P= 9.80 IN Q= 7.46 IN
TC= 2.60 HR CN= 81.00 VOL= 2609.0 ACFT
PEAK 4899.3 CFS AT 10.73 HRS

****

TING TABLE DEVELOPED
Y PROGRAM FOR PS AND EMG SPILLWAYS
EMG. RATING USED TRAPW METHOD

RATING TABLE NUMBER 2

	ELEV FEET	Q-TOTAL CFS	Q-PS CFS	VOLUME AC-FT	AREA ACRE
1	845.30	0.0	0.0	84.00	0.0
2	845.64	9.19	9.19	89.66	0.0
3	845.98	26.00	26.00	95.31	0.0
4	846.32	47.77	47.77	100.97	0.0
5	846.66	73.54	73.54	106.62	0.0
6	850.55	84.76	84.76	199.76	0.0
7	854.44	94.21	94.21	334.05	0.0
8	858.34	102.79	102.79	516.04	0.0
9	862.23	110.70	110.70	756.21	0.0
10	866.12	118.09	118.09	1068.39	0.0
11	870.01	125.04	125.04	1470.64	0.0
12	873.91	131.62	131.62	1980.37	0.0
13	877.80	137.89	137.89	2596.00	0.0
14	878.56	204.05	139.09	2739.01	0.0
15	879.32	364.75	140.27	2882.01	0.0
16	880.69	868.65	142.37	3158.55	0.0
17	882.36	1851.17	144.89	3519.73	0.0
18	885.40	4550.08	149.38	4226.42	0.0
19	889.20	9559.97	154.80	5225.86	0.0
20	893.00	16341.54	160.04	6318.00	0.0

FULL CONDUIT FLOW, ELEV = 846.66 FT

TYPE	BW	EMAX	VOL-MAX	AMAX	HP	VOL-ES	Q-PS	Q-ES	Q-TOT	D/C	V/C	S/C	S/C.25	D-ES	OE/B
TORM HYD	75.0	877.07	2470.6	0.0	0.0	0.0	136.6	0.0	136.6	0.0	0.0	0.0	0.0	0.0	0.:

PLOT 1 IN = 1000. CFS EXIT SLOPE = 0.055

0. 1000. 2000. 3000. 4000. 5000. 6000. 7000. EXIT

T	I	O	E	VOL	A	I I I I I I I I	VEL
0.0	0	0	845.3	84.0	0.0	.	0.0
0.33	0	0	845.3	84.0	0.0	.	0.0
0.65	0	0	845.3	84.0	0.0	.	0.0
0.98	0	0	845.3	84.0	0.0	.	0.0
1.30	0	0	845.3	84.0	0.0	.	0.0
1.63	0	0	845.3	84.0	0.0	.	0.0
1.95	0	0	845.3	84.0	0.0	.	0.0
2.28	0	0	845.3	84.0	0.0	.	0.0
2.60	0	0	845.3	84.0	0.0	.	0.0
							0.0

SAN ANTONIO RIVER BASIN

SCS FLOODWATER RETARDING DAM

SALADO CREEK WS SITE No.12

BEXAR COUNTY, TEXAS

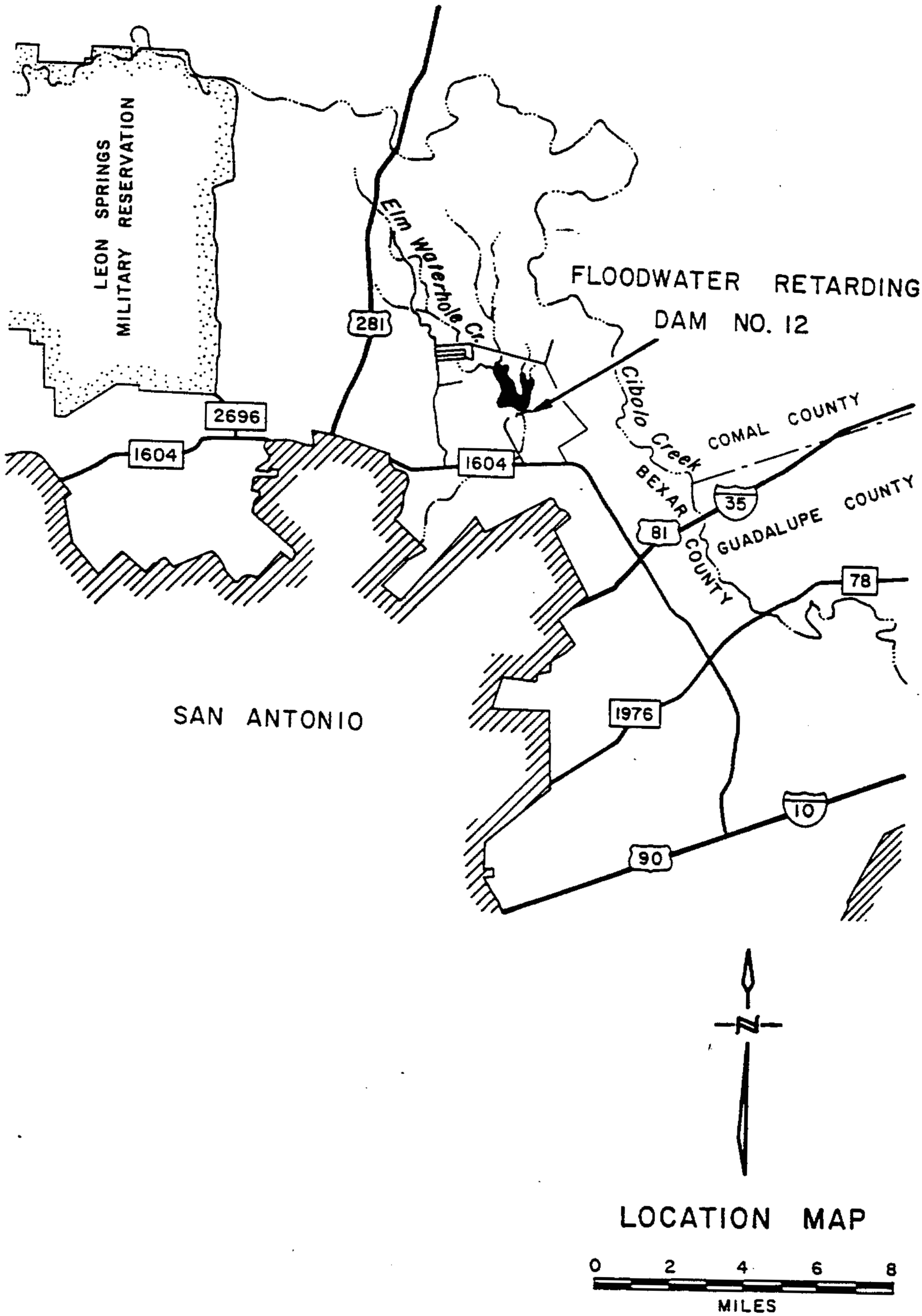
INVENTORY NUMBER TX 04208

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

U. S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

FEBRUARY 1979

REPORT PREPARED BY
TEXAS DEPARTMENT OF WATER RESOURCES
AUSTIN, TEXAS



LOCATION MAP

f. *Purpose of dam.* The dam is used as a floodwater retarding structure.

g. *Design and construction history.* The dam was completed in June of 1974 and was designed by the U.S. Department of Agriculture, Soil Conservation Service. Soils testing was conducted by the SCS Materials Testing Section, Fort Worth, Texas. The contractor for the dam was Mr. Lawrence D. Krause of New Braunfels. Construction of the dam was supervised by the SCS. A quarry now exists in the floodpool area of the structure.

h. *Normal operational procedure.* SCS Dam No. 12 is a floodwater retarding structure. Except for a 12-inch low-flow gate in the drop inlet, flow through both the service (principal) and emergency spillways is uncontrolled. The 12-inch low-flow gate is kept open, however, and water is impounded only temporarily. The structure is inspected yearly by personnel from the SCS and the sponsors. The sponsors are responsible for performing any necessary maintenance. The structure is also inspected for damage after a flood, and if possible, during any period of flooding. No formal warning plan is in effect.

1.3 *Pertinent Data.*

a.	*Drainage area.*	12.7 square miles
b.	*Discharge at dam site (CFS).*	
	Maximum flood at dam site	Unknown
	Service and emergency spillways at top-of-dam elevation	45,000
c.	*Elevation (feet above MSL).*	
	Maximum pool (PMF)	944.8
	Top of dam (effective)	945.2
	Emergency spillway crest	936.2
	Service spillway crest	902.8
	Low-flow invert	889.3
d.	*Reservoir (miles).*	
	Length at top of dam	1.8 miles
	Length at emergency spillway crest	1.4 miles
	Length at service spillway crest	0.38 miles
e.	*Storage (acre-feet).*	
	Maximum pool (PMF)	7305
	Top of dam	7425
	Emergency spillway crest	4735
	Service spillway crest	169
	Sediment storage (includes borrow areas)	323
f.	*Reservoir surface (acres).*	
	Maximum pool (PMF)	Unknown
	Top of dam	Unknown

f. *Reservoir surface (acres).* (cont'd)	
Emergency spillway crest	275
Service spillway crest	28
g. *Dam.*	
Type	Earth and rockfill
Length	2600 feet
Height	70 feet
Top width	14 feet
Side slopes (upstream)	1.5H:1V
Side slopes (downstream)	1.5H:1V
Zoning	SCS Site No. 12 is a zoned embankment with a compacted core of plastic, silty clay and moderately plastic clayey gravel with 1H:8V side slopes to an elevation of 936.2 feet MSL (emergency spillway crest). The core has a top width of 8 feet at this elevation. There is also a 10-foot-wide transition zone having 1H:8V side slope on each side of the core. The transition zones are composed of gravel and clayey gravel and also end at an elevation of 936.2 feet MSL. The core and both transition zones are capped with a section of the same material as used in the core. The dimensions of this section are variable. The outer shell of the embankment is composed of rockfill having a 1.5H:1V slope. The top 18 inches of the dam is gravelly clay.
Cutoff	None
Grout curtain	No grouting was used. However, deeper localized excavations were used where necessary.
h. *Diversion and regulating tunnel.*	None

i. *Spillways.*

(1) *Service spillway.*

Type	Uncontrolled-196 feet of 30-inch ID prestressed, concrete-lined, steel cylinder pipe through the embankment with a concrete box drop inlet and rock-lined plunge basin at the outlet
Crest elevation (drop inlet)	902.8 feet MSL
Pipe invert at bottom of inlet	888.8 feet MSL
Pipe invert at outlet	884.0 feet MSL
Drop inlet	30-inch by 90-inch opening at the top of the inlet with a 14-foot drop from the crest to the floor of the inlet and one 12-inch low-flow slide gate, invert elevation 889.0 feet MSL
Plunge basin	2-foot thickness of dumped rock lining a circular plunge basin with 1.5H:1V side slopes and 8- to 12-foot bottom width
Upstream channel	Lake
Downstream channel	Natural channel of Long Creek

(2) *Emergency spillway.*

Type	Uncontrolled channel 596 feet in width formed by earth cuts with 1H:1V side slopes and earth dikes with 2H:1V side slopes
Crest elevation	936.2 feet MSL

j. *Regulating outlets.* A 12-inch low-flow gate in the drop inlet. See paragraph 1.3(i) above.

PROBABLE MAXIMUM FLOOD INFLOW HYDROGRAPH (cont'd)

Time (hours)	*Inflow (CFS)*	*Time (hours)*	*Inflow (CFS)*
16	13,351	30	534
18	41,478	32	176
20	24,598	34	49
22	12,445	36	16
24	6,245	38	5
26	3,854	40	1
28	1,569	TOTAL	126,133

(4) *Probable maximum flood outflow.* The probable maximum flood inflow hydrograph was routed through the structure beginning at elevation 932.66 feet MSL. The outflow hydrograph is tabulated following.

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH*

Time (hours)	*Outflow (CFS)*	*Time (hours)*	*Outflow (CFS)*
0	142	22	13,451
2	142	24	7,083
4	143	26	4,493
6	145	28	2,437
8	148	30	1,131
10	1,204	32	711
12	4,594	34	470
14	5,914	36	302
16	11,890	38	191
18	41,246	40	150**
20	28,799	TOTAL	124,786

*Includes some outflow from antecedent flood.

**Reservoir content at end of 40 hours is 4681 acre-feet. Reservoir content at beginning of PMF was 3702 acre-feet.

(5) *Discharge-frequency.* No data are available.

(6) *Reservoir area and storage capacity.* An area-capacity table for SCS Dam No. 12 is presented following. This table is based on data determined by the Soil Conservation Service.

RESERVOIR AREA AND STORAGE CAPACITY TABLE

Elevation (feet MSL)	*Reservoir area (acres)*	*Storage capacity (acre-feet)*
884	1	1
888	2	7
892	6.8	25
896	12.3	63
900	21	123
902.8	28	323

H (cont'd)

Inflow (CFS)
534
176
49
16
5
1
126,133

flood inflow hydro-
et MSL. The outflow

RAPH*

Outflow (CFS)
13,451
7,083
4,493
2,437
1,131
711
470
302
191
150**
124,786

at beginning of PMF

e for SCS Dam No. 12
e Soil Conservation

TABLE

rage capacity (acre-feet)
1
7
25
63
123
323

RESERVOIR AREA AND STORAGE CAPACITY TABLE (cont'd)

Elevation (feet MSL)	*Reservoir (acres)*	*Storage capacity (acre-feet)*
907.5	-	350
924	158	2153
936.2	-	4735
936.7	-	4884
937.2	-	5033
937.7	-	5183
938.2	-	5332
939	-	5572
940	336.5	5871
941	-	6169
942	-	6468
944	-	7066
946	-	7664

(7) *Spillway rating.* A spillway rating table for SCS Dam No. 12 is presented following. These data were computed by the Soil Conservation Service.

SPILLWAY RATING TABLE

Elevation (feet MSL)	*Combined spillway discharge (CFS)*
902.8	0
907.5	98
924.0	130
936.2	150
936.7	510.3
937.2	1,232.4
937.7	2,437.6
938.2	3,826.2
939.0	6,740.5
940.0	11,001.2
941.0	16,249.9
942.0	21,904.1
944.0	35,009.0
946.0	50,585.6

(8) *Tailwater rating curve.* No data are available.

(9) *Hydrologic network.* A map of the SCS Dam No. 12 watershed is presented in exhibit A-2.

b. *Stability and stress analysis.* The slope stability of the earthen and rockfill embankment was checked by the Courtney method of analysis and by a wedge-type failure surface. The analysis concluded that the failure surfaces in the core and transition zones are less critical than the failure surfaces in the rockfill and that the controlling elements are almost solely the slope of the rockfill and shear strength of the fill. The Corps of Engineers in their publication EM-1110-2-1902, December 1960, page 45, affirms this conclusion as summarized following:

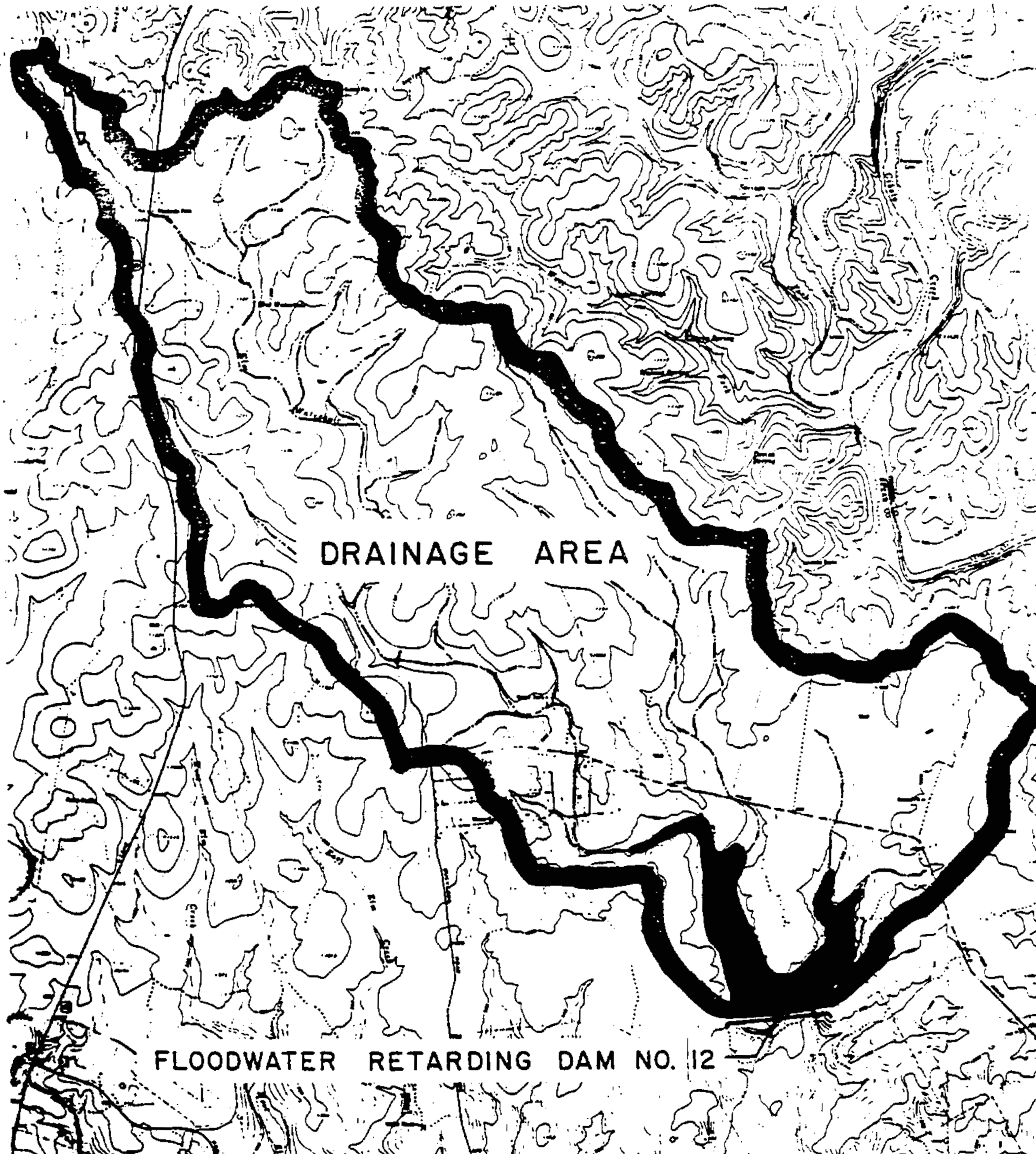


EXHIBIT A-2

DRAINAGE AREA M

SAN ANTONIO RIVER BASIN

SCS FLOODWATER RETARDING DAM

SALADO CREEK WS SITE No.13A

BEXAR COUNTY, TEXAS

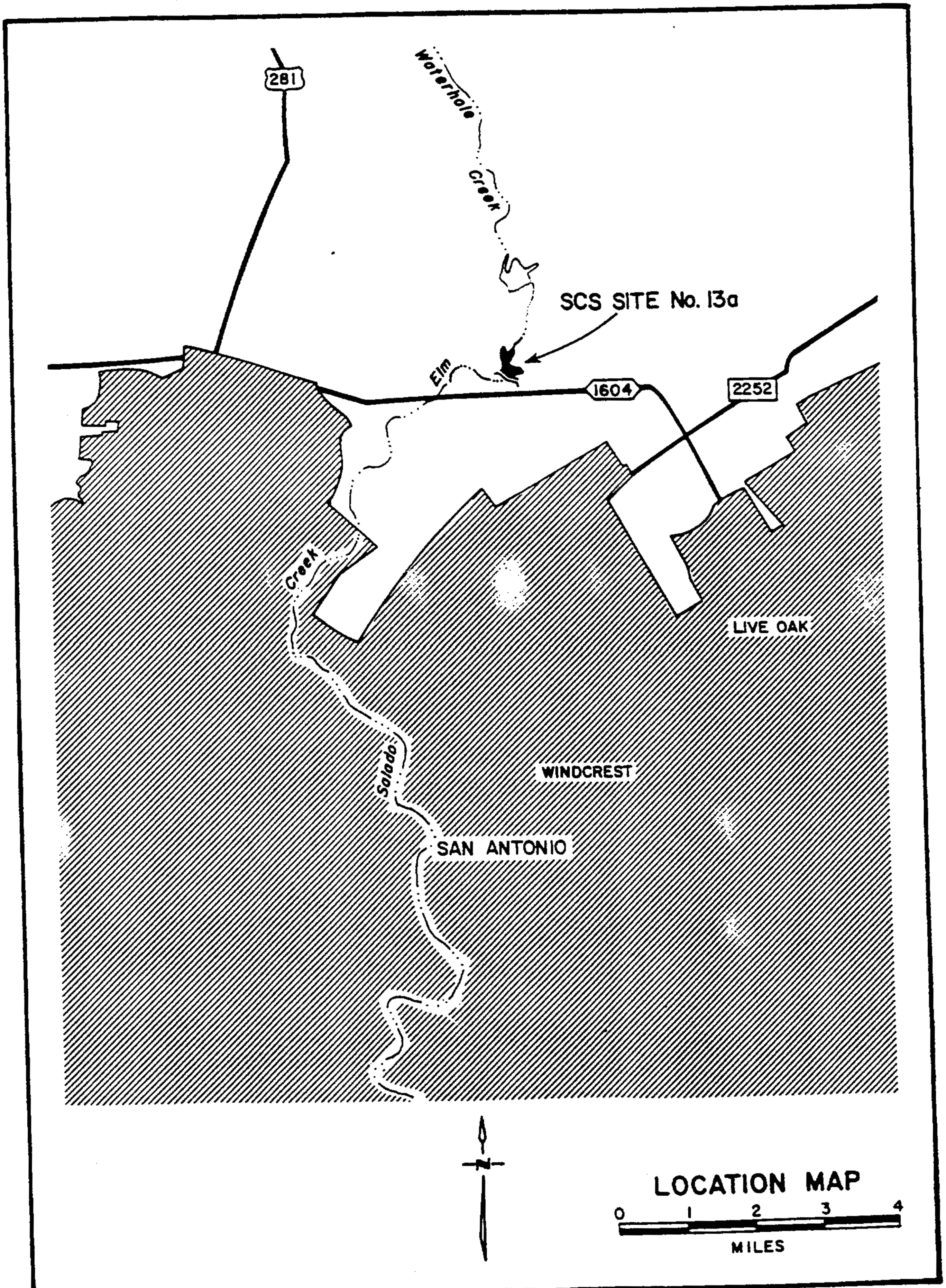
INVENTORY NUMBER TX04364

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

U. S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

MARCH 1981

REPORT PREPARED BY
TEXAS DEPARTMENT OF WATER RESOURCES
AUSTIN, TEXAS


281
Waterhole
Creek
SCS SITE No. 13a
Elm
1604
2252
Creek
LIVE OAK
WINDCREST
Salado
SAN ANTONIO
N
LOCATION MAP
0
1
2
3
4
MILES

Mr. Fred N. Pfeiffer, General Manager
San Antonio River Authority
c/o Mr. Ervin L. Willard, District Conservationist
Soil Conservation Service
San Antonio Field Office
630 South Main Street
San Antonio, Texas 78204

f. *Purpose of dam.* SCS Site No.13A was designed as a floodwater-retarding structure. It also has the additional benefit of recharging into the underground formation.

g. *Design and construction history.* SCS Site No.13A was designed by the Soil Conservation Service after extensive field investigation of the proposed site. Results of these investigations are included in Appendix A of this report. All preliminary investigation, quality control testing, and other procedures during the construction were under the general supervision of the SCS. The actual construction was carred out by House - Braswell Company. The date of completion of the structure was August 13, 1976. The local records for the structure indicate that only normal maintenance has been required.

h. *Normal operational procedures.* The spillways at SCS Site No.13A are uncontrolled; therefore, no operational procedures exist for periods of flooding. The operational procedures for the maintenance of the facility are discussed in Section 4.

1.3 *Pertinent Data.*

a. *Drainage area.*	16 square miles, of which 12.7 square miles are controlled by SCS Site No.12, located upstream
b. *Discharge at dam site (CFS).*	
Maximum flood at dam site	Unknown
Combined spillway discharge at the effective crest elevation	30,848
c. *Elevation (feet above mean sea level).*	
Top of dam (effective crest)	884.5
Maximum pool (PMF)	888.7
Emergency spillway crest	877.0
Service spillway crest	861.8
Low flow (slide gate invert)	852.3
Streambed (centerline of dam)	841.5
d. *Reservoir (length in miles).*	
Top of dam (effective crest)	1.71
Maximum pool (PMF)	1.81
Emergency spillway crest	1.37
Service spillway crest	0.68
e. *Storage (acre-feet).*	
Top of dam (effective crest)	3026
Maximum pool (PMF)	4126
Emergency spillway crest	1441
Service spillway crest	128
f. *Reservoir surface (acres).*	
Top of dam (effective crest)	267.5

f. *Reservoir surface (acres).* (cont'd)

Maximum pool (PMF)	346.0
Emergency spillway crest	161.0
Service spillway crest	26.0

g. *Dam.*

Type	Rolled. zoned, earthfill embankment with an impervious core
Length	1690 feet
Height	43 feet
Top width	18 feet
Upstream slope	2.5H:1V with a 20-foot-wide sloping berm having a centerline elevation of 861.8 feet MSL
Downstream slope	2.5H:1V
Cutoff	Cutoff trench with a 20-foot bottom width and 2H:1V side slopes beneath the embankment; maximum depth 34 feet depicted in Exhibit C-3
Impervious core	The impervious core is depicted in Exhibit C-4.
Zoning	The zoning of the embankment consists of two zones. Materials were placed in accordance with the table depicted in Exhibit C-4.
Grout curtain	None
Foundation drain	The trench foundation drain and the blanket drain are depicted in Exhibits C-7 and C-8.

h. *Diversion and regulating tunnel.* None

i. *Spillways.*

(1) *Service spillway.*

Type	Concrete drop inlet, 10 feet high with inside dimensions of 9 feet by 3 feet, discharging through a 36-inch ID, prestressed concrete-lined, steel cylinder pipe 230 feet long
Crest elevation	861.8 feet MSL
Ports	None
Slide gate	12-inch by 12-inch slide gate, invert elevation 852.3 feet MSL, manually controlled by a handwheel at the top of the drop inlet
Foundation	The bottom of the concrete cradle is the lower limit of conduit foundation excavation from station 3+30 to station 4+10.
Antiseep collars	There are seven antiseep collars located on 20-foot centers with the upstream most collar being 75 feet upstream of the centerline of the dam.
Upstream channel	30-foot-wide channel filled to elevation 851.8 feet MSL from upstream embankment toe to limits shown on Exhibit C-5

i. *Spillways.* (cont'd)

Downstream channel	Plunge basin, 10 feet deep and 16 feet wide at the bottom with 3H:1V side slopes; flow discharges into a 10-foot-wide channel with 3H:1V side slopes.
(2) *Emergency spillway.*	
Type	A detached and uncontrolled earthen channel approximately 550 feet in length; the spillway is 640 feet wide (300 feet on both sides of a 40-foot splitter dike) and is located approximately 1000 feet north of the right abutment.
Crest elevation	877.0 feet MSL
Upstream channel	Gently sloping natural ground with a short earthfill approach with a 10-percent grade
Downstream channel	A 250-foot-long earthfill channel with a 4-percent grade
j. *Regulating outlets.*	The only regulating outlet is the 12-inch by 12-inch slide gate in the upstream face of the service spillway inlet.

[illegible] *Probable maximum flood outflow.* The probable maximum flood inflow hydrograph w routed through the structure beginning at elevation 876.5 feet MSL. The outflow hydrograph tabulated as follows:

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH

Time (hours)	*Instantaneous outflow (CFS)*	*Time (hours)*	*Instantaneous outflow (CFS)*
0.5	21	25.5	5,116
1.0	159	26.0	4,502
1.5	159	26.5	3,891
2.0	159	27.0	3,335
2.5	160	27.5	2,833
3.0	225	28.0	2,470
3.5	290	28.5	2,092
4.0	345	29.0	1,729
4.5	394	29.5	1,419
5.0	468	30.0	1,185
5.5	532	30.5	1,004
6.0	585	31.0	859
6.5	631	31.5	785
7.0	720	32.0	730
7.5	922	32.5	675
8.0	1,287	33.0	619
8.5	1,728	33.5	566
9.0	2,361	34.0	514
9.5	3,275	34.5	466
10.0	4,418	35.0	420
10.5	5,434	35.5	388
11.0	6,264	36.0	364
11.5	6,910	36.5	342
12.0	7,351	37.0	322
12.5	7,657	37.5	304
13.0	7,977	38.0	288
13.5	8,448	38.5	274
14.0	9,168	39.0	260
14.5	10,410	39.5	248
15.0	12,706	40.0	237
15.5	18,546	40.5	228
16.0	30,259	41.0	219
16.5	39,641	41.5	211
17.0	48,984	42.0	205
17.5	56,223	42.5	198
18.0	55,800	43.0	193
18.5	50,240	43.5	188
19.0	43,196	44.0	184
19.5	35,388	44.5	180
20.0	28,126	45.0	177
20.5	22,241	45.5	174
21.0	17,907	46.0	171
21.5	14,644	46.5	169
22.0	12,340	47.0	167
22.5	10,346	47.5	165
23.0	8,793	48.0	163
23.5	7,680	48.5	161
24.0	6,878	49.0	160
24.5	6,291	77.0	160
25.0	5,730		

(table cont'd)

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH (cont'd)

Time (hours)	*Instantaneous outflow (CFS)*	*Time (hours)*	*Instantaneous outflow (CFS)*
77.5	159	123.5	144
120.5	159	124.0	37
121.0	144		

Note: Peak outflow occurs at 17.75 hours and is 56,973 CFS.

(5) *Discharge-frequency.* No data are available.

(6) *Reservoir area and storage capacity.* An area-capacity table for SCS Site No.13A is presented following. These data were computed by the Soil Conservation Service.

RESERVOIR AREA AND STORAGE CAPACITY TABLE

Elevation (feet MSL)	*Reservoir area (acres)*	*Storage capacity (acre-feet)*
848.0	1.0	1
852.0	3.0	9
856.0	8.0	31
858.8	14.0	62
860.0	18.0	83
861.8	26.0	128
864.0	38.0	195
868.0	70.0	411
872.0	107.0	765
876.0	149.0	1277
877.0	161.0	1441
877.5		1529
878.0		1617
879.0		1794
880.0	199.0	1973
882.0		2441
884.0	262.0	2895
884.5	267.5	3026

(7) *Spillway rating table.* A spillway rating table for SCS Site No.13A is presented following. These data were computed by the Soil Conservation Service.

SPILLWAY RATING TABLE

Elevation (feet MSL)	*Combined spillway discharge (CFS)*
861.8	0
864.0	117
868.0	131
872.0	145
877.0	160
877.5	400
878.0	822
879.0	2,878
880.0	6,038
882.0	15,288
884.5	30,848

(8) *Tailwater rating table.* A tailwater rating table for SCS Site No.13A is presented following. These data were computed using the Manning equation based on an idealized valley cross section and an average valley slope immediately downstream of the dam.

TAILWATER RATING TABLE

Elevation (feet MSL)	*Discharge (CFS)*
843.0	0
844.1	44
845.2	133
846.2	254
847.3	400
848.4	568
850.7	1,282
853.0	2,964
855.4	6,120
857.7	11,167
860.0	18,479
863.4	37,098
866.8	63,428

(9) *Hydrologic network.* A map of the drainage area of SCS Site No.13A is presented in Exhibit. A-2.

(10) *Breach analysis.* A breach analysis was performed to investigate the effects on the downstream area under five different conditions. These were:

1) Breaching of the dam with reservoir at normal operating level with no inflow.
2) Breaching due to overtopping by the PMF.
3) Overtopping without breaching by the PMF.
4) Breaching due to a barely overtopping flood that represents 57% of the PMF.
5) Overtopping without breaching due to a barely overtopping flood that represents 57% of the PMF.

Results of the breach analysis for selected locations are graphically illustrated by computer plots, which appear as Exhibit A-3. SCS Site No.13A is classified as a high-hazard structure due to the presence of scattered dwellings at various downstream points and to the presence of a suburban development approximately 3 miles downstream of the dam. A point of interest was chosen approximately 2.8 miles downstream from the dam at which point the results of a recent field inspection indicate dense development with dwellings ranging in elevation from approximately 800 to 880 feet MSL. The following table shows the maximum water surface elevations and discharges at the point of interest for the five conditions investigated.

Condition	*Maximum water surface elevation (feet MSL)*	*Maximum discharge (CFS)*
Breaching at conservation pool with no inflow	793.3	2,329
Breaching due to overtopping by PMF	808.4	76,083
Overtopping without breaching by PMF	806.2	58,447
Breaching due to overtopping by 57% PMF	805.1	51,505
Overtopping without breaching by 57% PMF	802.0	32,166

SAN ANTONIO RIVER BASIN

SCS FLOODWATER RETARDING DAM

SALADO CREEK WS SITE No.13B

BEXAR COUNTY, TEXAS

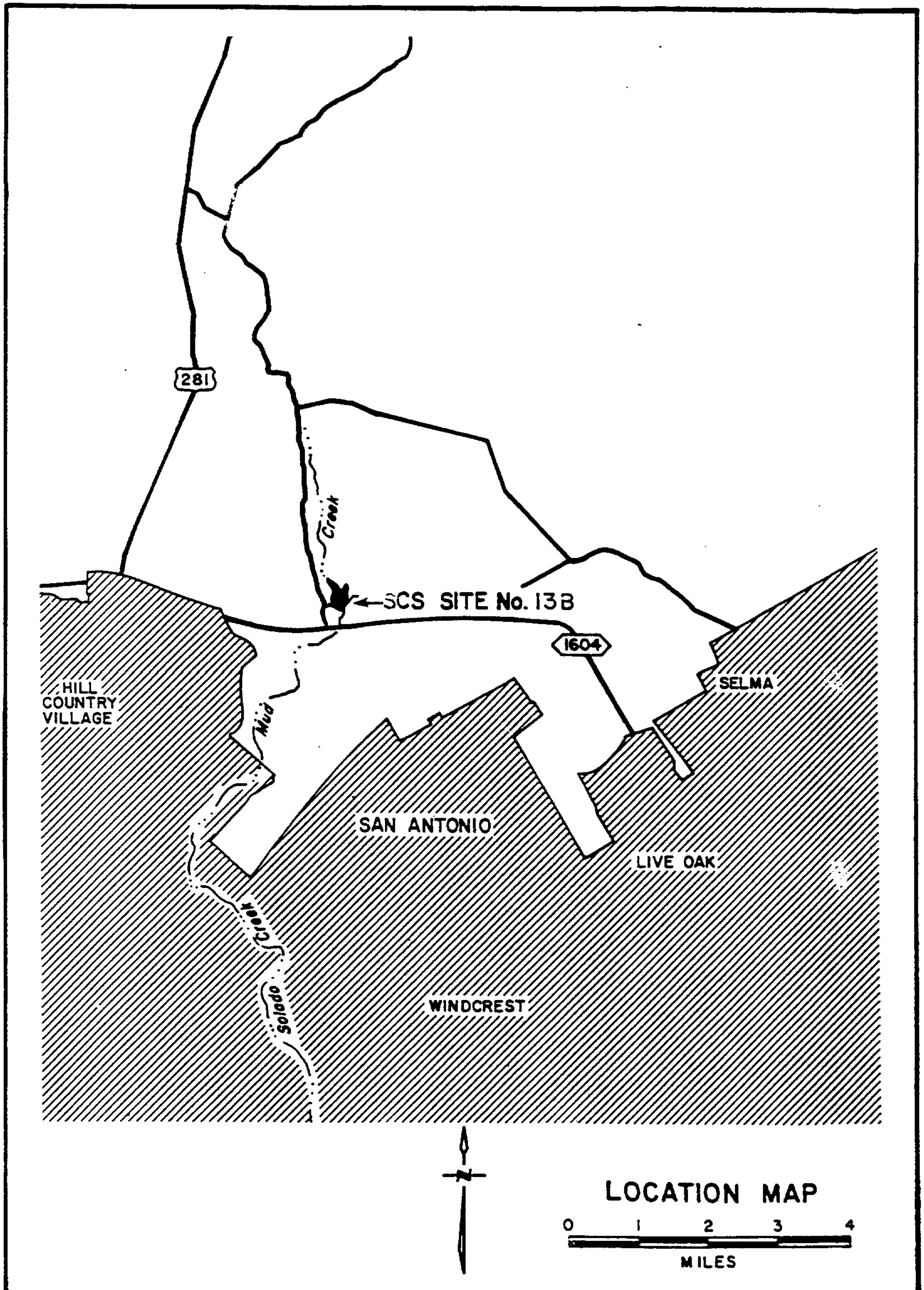
INVENTORY NUMBER TX04718

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

U. S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

MARCH 1981

REPORT PREPARED BY
TEXAS DEPARTMENT OF WATER RESOURCES
AUSTIN, TEXAS


281
Creek
SCS SITE No. 13B
1604
SELMA
HILL
COUNTRY
VILLAGE
Mud
SAN ANTONIO
LIVE OAK
Creek
Salado
WINDCREST
N
LOCATION MAP
0 1 2 3 4
MILES

included in Appendix A of this report. All preliminary investigations, quality control testing, and other procedures during the construction were under the general supervision of the SCS. The actual construction was carried out by Lawrence D. Krause. The date of completion of the structure was August 22, 1975. The local records for the structure indicate that no maintenance has been required.

h. *Normal operational procedures.* The spillways at SCS Site No.13B are uncontrolled; therefore, no operational procedures exist for periods of flooding. The operational procedures for the maintenance of the facility are discussed in Section 4.

1.3 *Pertinent Data.*

a. *Drainage area.*	2.5 square miles
b. *Discharge at dam site (CFS).*	
Maximum flood at dam site	Unknown
Combined spillway discharge at the effective crest elevation	11,849
c. *Elevation (feet above mean sea level).*	
Top of dam (effective crest)	886.0
Maximum pool (PMF)	888.5
Emergency spillway crest	878.4
Service spillway crest	857.5
Low flow (slide gate invert)	853.0
Streambed (centerline of dam)	840.0
d. *Reservoir (length in miles).*	
Top of dam (effective crest)	0.90
Maximum pool (PMF)	0.94
Emergency spillway crest	0.78
Service spillway crest	0.17
e. *Storage (acre-feet).*	
Top of dam (effective crest)	1828
Maximum pool (PMF)	2014
Emergency spillway crest	1093
Service spillway crest	72
f. *Reservoir surface (acres).*	
Top of dam (effective crest)	131.0
Maximum pool (PMF)	143.0
Emergency spillway crest	92.0
Service spillway crest	15.3
g. *Dam.*	
Type	Rolled, zoned, earthfill embankment with an impervious core
Length	2802 feet
Height	46 feet
Top width	14 feet

g. *Dam.* (cont'd)	
Upstream slope	2.5H:1V with a 2-foot rock cover for slope protection
Downstream slope	2.5H:1V with a 2-foot rock cover for slope protection
Impervious core and cutoff	Cutoff trench with a 20-foot bottom width and 1H:1V side slopes (Exhibits C-1 and C-2)
Zoning	Highly plastic clay cutoff trench and core section with shale in the core above the service spillway elevation, clayey gravel outer embankment section, and a 2-foot rock blanket on the upstream slope and the downstream slope (Exhibit C-2)
Grout curtain	None
Foundation drain	A rock blanket drain extends from station 21+00 to station 27+50 with an overall thickness of 42-inches and a 25-foot length constructed as shown in Exhibit C-4.
h. *Diversion and regulating tunnel.*	None
i. *Spillways.*	
(1) *Service spillway.*	
Type	Concrete drop inlet, 5 feet high with inside dimensions of 7.5 feet by 2.5 feet; discharge is through a 30-inch ID, prestressed concrete-lined, steel cylinder pipe 210 feet long.
Crest elevation	857.5 feet MSL
Ports	None
Slide gate	12-inch by 12-inch slide gate, invert elevation 853.0 feet MSL, manually controlled by a handwheel at the top of the drop inlet
Foundation	The conduit foundation excavation bottoms on limestone bedrock; backfilled to line and grade at the bottom of the concrete cradle
Antiseep collars	Five antiseep collars located on 20-foot centers with the upstream most collar being 20 feet upstream of the centerline of the dam
Upstream channel	40-foot-wide by 40-foot-long channel with 4H:1V slopes backfilled to elevation 852.5 feet MSL
Downstream channel	Outlet channel has a 16-foot-wide bottom with 3H:1V side slopes. A 3-foot thickness of rock lining covers the slopes above sound bedrock.
(2) *Emergency spillway.*	
Type	An uncontrolled earthcut channel approximately 400 feet in length

i. *Spillways.* (cont'd)

Width	200 feet
Crest elevation	878.4 feet MSL
Upstream channel	160-foot-long approach channel
Downstream channel	Flow returns to original downstream channel
j. *Regulating outlets.*	12-inch by 12-inch slide gate on the upstream face of the drop inlet

(3) *Probable maximum flood inflow.* Rainfall excess values derived for the SCS Site No.13B drainage area were applied to the unit hydrograph to compute the PMF. Base flow was ignored since its magnitude is very small compared to the peak flow. To simulate saturated conditions in the watershed, a flood approximately one-half the magnitude of the PMF was initially routed through the reservoir. The PMF was routed 5 days (120 hours) after the beginning of the antecedent storm by which time the reservoir level had receded to elevation 864.5 feet MSL, which is 7 feet above the top of the conservation pool. The PMF used in the routing to determine the maximum water surface elevation is tabulated as follows:

PROBABLE MAXIMUM FLOOD INFLOW HYDROGRAPH

Time (hours)	*¼-hour average inflow (CFS)*	*Time (hours)*	*¼-hour average inflow (CFS)*
0.5	169	15.0	8,662
1.0	454	15.5	20,508
1.5	511	16.0	16,768
2.0	520	16.5	8,458
2.5	522	17.0	4,837
6.0	522	17.5	3,244
6.5	776	18.0	2,431
7.0	1,203	18.5	1,657
7.5	1,289	19.0	950
8.0	1,303	19.5	812
8.5	1,306	20.0	789
12.0	1,306	20.5	784
12.5	1,460	24.0	784
13.0	1,908	24.5	530
13.5	2,406	25.0	103
14.0	3,202	25.5	17
14.5	4,693	26.0	3

Note: Peak inflow occurs at 15.75 hours and is 21,884 CFS.

(4) *Probable maximum flood outflow.* The probable maximum flood inflow hydrograph was routed through the structure beginning at elevation 864.5 feet MSL. The outflow hydrograph is tabulated as follows:

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH

Time (hours)	*Instantaneous outflow (CFS)*	*Time (hours)*	*Instantaneous outflow (CFS)*
0.5	92	9.0	116
1.0	93	9.5	118
1.5	94	10.0	119
2.0	96	10.5	121
2.5	97	11.0	122
3.0	98	11.5	123
3.5	100	12.0	124
4.0	101	12.5	125
4.5	102	13.0	181
5.0	103	13.5	570
5.5	104	14.0	1,301
6.0	104	14.5	2,408
6.5	106	15.0	4,815
7.0	108	15.5	13,536
7.5	110	16.0	17,169
8.0	112	16.5	11,673
8.5	114		

(table cont'd)

PROBABLE MAXIMUM FLOOD OUTFLOW HYDROGRAPH (cont'd)

Time (hours)	*Instantaneous outflow (CFS)*	*Time (hours)*	*Instantaneous outflow (CFS)*
17.0	7,128	22.0	909
17.5	5,299	22.5	868
18.0	3,938	23.0	843
18.5	2,997	23.5	827
19.0	2,205	24.0	816
19.5	1,695	24.5	768
20.0	1,380	25.0	608
20.5	1,186	25.5	463
21.0	1,056	26.0	368
21.5	968	26.5	292

Note: Peak outflow occurs at 15.75 hours and is 17,540 CFS.

(5) *Discharge-frequency.* Records of daily or peak inflow into SCS Site No.13B are not available. The Soil Conservation Service has published a document entitled *A Method for Estimating the Volume and Rate of Runoff in Small Watersheds,* SCS TP-149, Revised April 1973. This publication, used in conjuction with the United States Weather Bureau's TP-40, allows the computation of peak flows for 24-hour storms on small watersheds (less than 2000 acres). Based on this publication, the estimated discharge-frequency curve for the area of SCS Site No.13B was drawn for recurrence intervals up to and including 100 years. The discharge-frequency curve is presented as Exhibit A-2.

(6) *Reservoir area and storage capacity.* An area-capacity table for SCS Site No.13B is presented following. These data were computed by the Soil Conservation Service.

RESERVOIR AREA AND STORAGE CAPACITY TABLE

Elevation (feet MSL)	*Reservoir area (acres)*	*Storage capacity (acre-feet)*
844.0	0.3	0.3
848.0	1.6	4.0
852.0	5.0	17.0
854.5	9.0	35.0
856.0	12.3	52.0
857.5	15.3	72.0
860.0	21.0	118.0
864.0	32.0	224.0
868.0	45.2	379.0
872.0	61.3	592.0
876.0	81.4	877.0
878.4	92.0	1093.0
879.0		1147.0
880.0	99.5	1239.0
881.0		1348.0
882.0		1457.0
884.0	119.9	1678.0
886.0		1828.0

(7) *Spillway rating table.* A spillway rating table for SCS Site No.13B is presented following. These data were computed by the Soil Conservation Service.

SPILLWAY RATING TABLE

Elevation (feet MSL)	*Combined spillway discharge (CFS)*
857.5	0
860.0	78
868.0	102
876.0	122
878.4	126
879.0	294
880.0	987
881.0	2,113
882.0	3,540
884.0	7,269
886.0	11,849

(8) *Tailwater rating table.* A tailwater rating table for SCS Site No.13B is presented following. These data were computed using the Manning equation based on an idealized valley cross section and an average valley slope immediately downstream of the dam.

TAILWATER RATING TABLE

Elevation (feet MSL)	*Discharge (CFS)*
840.9	0
841.4	18
841.9	63
842.4	134
842.9	235
843.4	367
844.7	996
846.0	1,995
847.4	3,441
848.7	5,402
850.0	7,942
852.0	13,722
854.0	21,697

(9) *Hydrologic network.* A map of the drainage area of SCS Site No.13B is presented in Exhibit A-3.

(10) *Breach analysis.* A breach analysis was performed to investigate the effects on the downstream area under five different conditions. These were:

1) Breaching of the dam with reservoir at normal operating level with no inflow.
2) Breaching due to overtopping by the PMF.
3) Overtopping without breaching by the PMF.
4) Breaching due to a barely overtopping flood which represents 78% of the PMF.
5) Overtopping without breaching due to a barely overtopping flood which represents 78% of the PMF.

Results of the breach analysis for selected locations are graphically illustrated by computer plots, which appear as Exhibit A-4.

The dam is classified as a high-hazard structure due to the presence of scattered dwellings at various points downstream of the dam. A point of interest was chosen approximately 1.5 miles downstream of the dam at which point the appropriate USGS 7.5-minute topographic quadrangle published in 1967 and photo-revised in 1973 shows a dwelling, the elevation of which appears to be

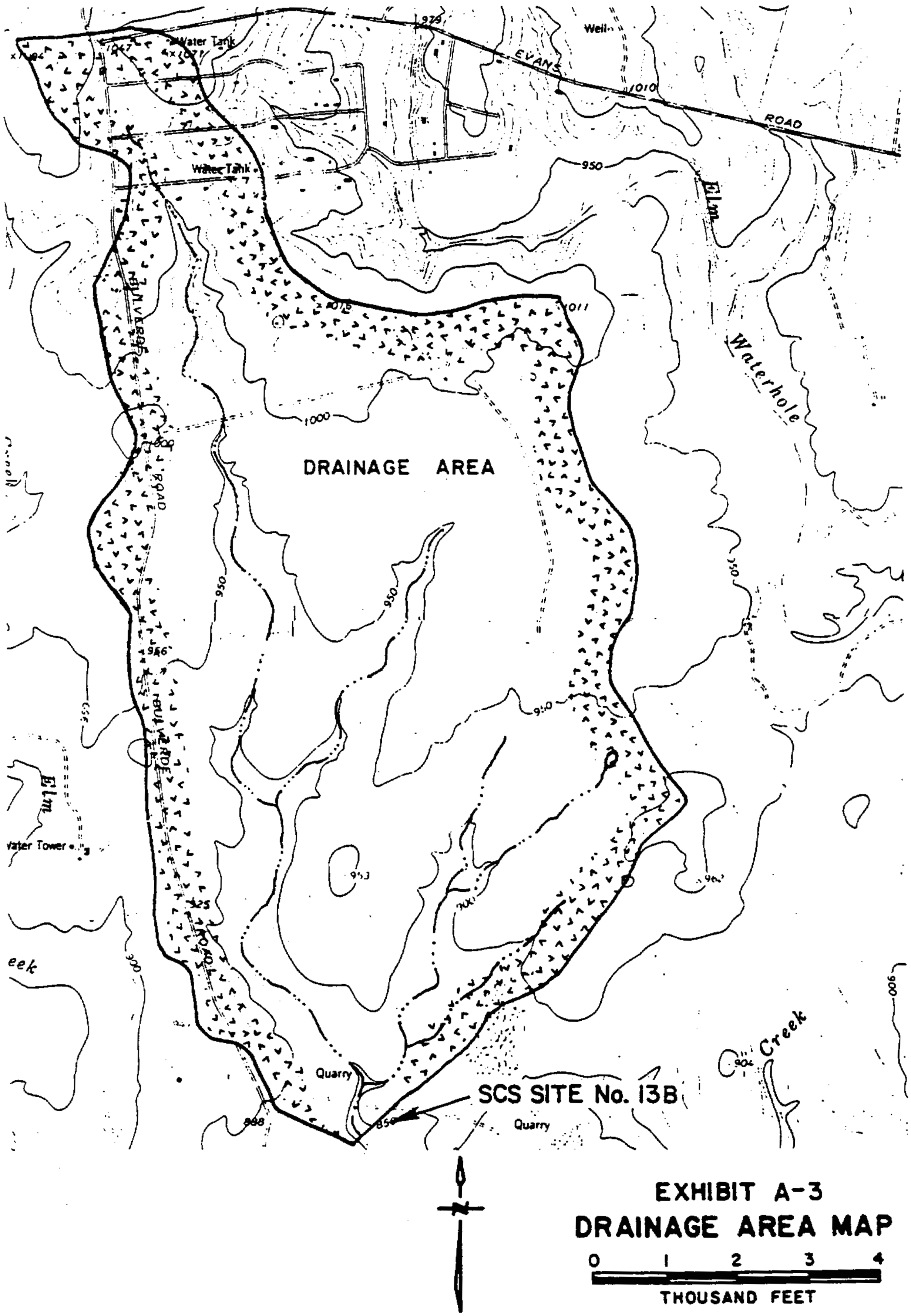


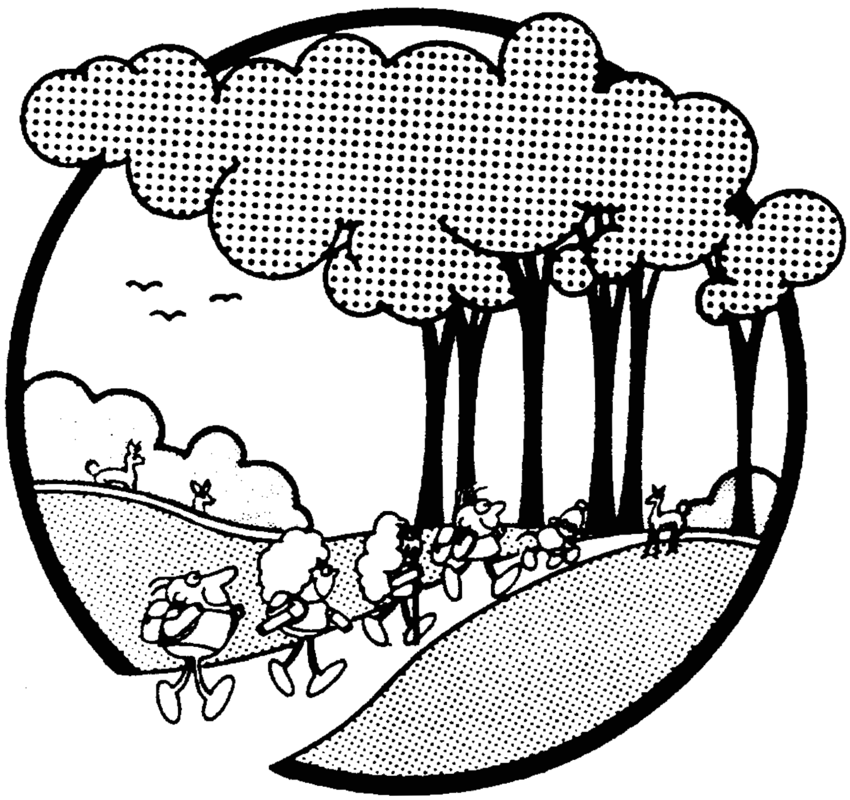
EXHIBIT A-3

DRAINAGE AREA MAP

0 1 2 3 4

THOUSAND FEET

Appendix F



MCALLISTER PARK MASTER PLAN

PREPARED BY THE

MC ALLISTER PARK ADVISORY COMMITTEE

AND THE

CITY OF SAN ANTONIO
PARKS AND RECREATION DEPARTMENT

AUGUST 1988 . UPDATED JANUARY 1991

August 9, 1988

Parks and Recreation Advisory Board
City of San Antonio
P. O. Box 9066
San Antonio, Texas 78285

Subject: McAllister Park Master Plan

Dear Board Members:

The McAllister Park Advisory Committee was chartered by the Advisory Board on February 16, 1986 to work with the City Parks Department in the development and implementation of a master plan for McAllister Park.

Since that time, the McAllister Park Advisory Committee has met on a monthly basis discussing various issues related to the park and working on the development of a master plan.

Included herein is a Master Plan that reflects the many months of work by this committee. This document consists of sections related to the major issues of the plan, its objectives, and the plan itself.

Our objective was to eventually produce a formal document that would be endorsed by you and the City Council.

We recommend this plan be used as a guide when the City considers improvements within the park. When implemented, this plan will ensure proper planning and management of this very important recreational resource of San Antonio.

Sincerely,

Members of the McAllister Park Advisory Committee

CONTENTS

MAPS

OBJECTIVES OF THE McALLISTER PARK MASTER PLAN

I. McAllister Park shall remain an open space and/or recreation area accessible to the public for all time.

II. No land within the McAllister Park boundaries shall be utilized in any manner other than for public open space and recreational purposes.

III. All lands that comprise McAllister Park shall be dedicated as Park land, in perpetuity, by the City Council of San Antonio.

IV. To maintain the native flora and fauna to which the Park is presently host, and to protect the existing wildlife habitats.

V. Make every effort to maintain an appropriate balance between recreational facilities for use by the public, and preservation of the Park's open space character, allowing for an enjoyable, quality, individual, experience.

VI. Monitor the use of the land area surrounding the Park to ensure that it will not be developed in a manner detrimental to the present natural setting of the Park. Conversely, Park uses which present a detriment to the surrounding private property shall not be considered.

McALLISTER PARK ADVISORY COMMITTEE
MAJOR ISSUES OF THE McALLISTER PARK MASTER PLAN

BACKGROUND

The Master Plan shall emphasize the importance of McAllister Park as a major public open space and recreation resource.

A. McAllister Park is over 800 acres of park land, and is one of the largest parks in the City of San Antonio.

B. McAllister Park is one of only five city parks North of Loop 410 from Interstate 10 to Interstate 35.

C. McAllister Park will become increasingly more valuable to the City of San Antonio providing major leisure opportunities to the public as the City's population grows, and the majority of development takes place in the Northern sector of the City.

PLAN GUIDELINES

I. The Master Plan shall emphasize the goal of providing a recreational area for the public, with limited vehicular activity.

A. No roads for "through" vehicular traffic shall be provided.

B. Vehicular "cruising" in the Park shall be discouraged by having roads end in designated parking areas.

C. Established speed limits shall be posted and enforced.

D. Pedestrian crosswalks shall be established and marked at all points where hike and bike trails cross vehicular roads. Pedestrian safety shall be a primary consideration.

E. Adequate parking, and its impact, shall be considered in developing recreational facilities.

II. The Master Plan shall maintain an appropriate balance, providing recreational facilities for use by the public, and preserving the Park's open space character.

A. A major priority of the Master Plan shall be to preserve the natural beauty of the Park, and this consideration shall be the governing factor in determining new development or redevelopment of the Park.

B. Emphasis shall be placed on designing and locating future facilities so as not to adversely impact the habitats of the wildlife in the Park.

C. The Master Plan shall place emphasis on activities that limit motor vehicle access to the Park.

D. Future facilities shall be designed and located to cause a minimum of interference with surrounding activities.

III. The Master Plan shall emphasize improved access and traffic circulation.

A. Access from the major streets that border the Park shall be created and located to serve the planned improvements.

1. Roads associated with future entrances shall not connect with existing Park roads to provide vehicular traffic flow from one major City street to another.

2. Short side roads may be provided to allow access to picnic and pavilion areas.

B. Access for maintenance and emergency vehicles shall be considered.

IV. The Master Plan shall emphasize flood control solutions that do the least environmental damage to the Park, and enhance its attractiveness.

V. The Master Plan shall emphasize the relocation of those activities not suited to the Park, and which conflict with other goals.

VI. The Master Plan shall emphasize the expansion of the Park where possible.

A. Consideration shall be given to securing property along creeks or flood zones outside of the Park to establish recreational corridors that connect the Park with nearby subdivisions.

B. Coordination shall take place between the City's Parks and Recreation Department, and San Antonio International Airport regarding possible use of adjoining airport property for public use for recreational and open space purposes.

VII. The Master Plan shall emphasize that the Park shall have a favorable environmental impact on surrounding areas and neighborhoods, and that improvements such as athletic fields, trails, and picnic areas shall not be located so close to a neighborhood as to disturb that neighborhood.

VIII. The Master Plan shall emphasize the importance of maintaining compatible development adjacent to and near the Park.

A. The zoning department of the City of San Antonio shall consult with the Parks and Recreation Department, and the McAllister Park Advisory Committee, on zoning requests for property adjacent to the Park.

B. City Departments and Agencies shall consider the effects of public works projects on the Park such as: airport expansion, sewage treatment plants, surrounding road improvements or expansions, flood control projects.

McALLISTER PARK MASTER PLAN

The Master Plan divides McAllister Park into four sections as delineated on the attached map. The Sections are defined as follows:

CENTRAL SECTION

The Central Section is that portion of the park which presently includes established recreational facilities, and which is currently most utilized.

WETMORE SECTION

The Wetmore Section is the large rectangular portion of the park that is primarily undeveloped, and which includes the Texas Transportation Museum.

MUD CREEK SECTION

The Mud Creek Section is the linear shaped parcel extending from Thousand Oaks to the Central Section of the park.

STARCREST SECTION

The Starcrest Section is that portion of the park that is undeveloped and abuts the Blossom Park Subdivision and Starcrest Road.

The Master Plan proposes the following:

CENTRAL SECTION

Retain existing pedestrian or hike and bike trails.

Increase pavilion areas.

Increase family picnic areas to possibly include the conversion of existing camping area.

De-emphasize the use of this section for athletic fields.

Retain existing playground facilities.

Expand family picnic facilities, when justified, in the flood control area.

WETMORE SECTION

Locate all future athletic fields in this section and provide sufficient parking.

Accommodate the Texas Transportation Museum.

Establish Park access off Wetmore Road, North of the Texas Transportation Museum.

Establish pedestrian or hike and bike trails.

Establish family picnic areas.

MUD CREEK SECTION

Establish pedestrian or hike and bike trails.

Establish small picnic areas with limited parking and access off Thousand Oaks Road.

NOTE: All improvements will be governed by the fact that Mud Creek is a flood plan.

STARCREST SECTION

Establish a pedestrian or hike and bike trail.

Establish small picnic areas with limited parking and access off Starcrest Drive.

McALLISTER PARK ADVISORY COMMITTEE

SAN ANTONIO, TEXAS

CHARTER

CHARACTERISTICS:

The McAllister Park Advisory Committee shall be composed of community and user group representatives who will meet at scheduled intervals of their choosing to discuss matters affecting McAllister Park. McAllister Park is a relatively undisturbed ecosystem that exhibits outstanding geological, floral and faunal features.

The committee shall submit recommendations to and coordinate its activities with the San Antonio Department of Parks and Recreation and the Parks and Recreation Advisory Board. In turn, these organizations will cooperate with the committee and consider it to be a focal point for the promotion of harmony and good public relations with the community in the Northeast and North Central sections of San Antonio.

FUNCTIONS:

Work with the San Antonio Parks Department in the development and implementation of a Master Plan for McAllister Park.

Submit recommendations and take appropriate actions to protect and preserve the Park plant and animal life and their life support environment.

Encourage the private ball associations, civic clubs, etc. as well as the commercial sector to provide Park facilities.

Submit recommendations and take action to maximize the resources available and the recreational experience of the user.

Submit recommendations and take action to provide a uniform distribution of recreational attractions.

Provide a community focal point for matters which may have an impact on the Park or activities within the Park.

Perform any other functions which the committee deems appropriate to preserve the Park ecosystem and in promoting the recreational facilities for the users.

ORGANIZATION:

The committee shall be composed of community volunteers and civic organization representatives. Every effort should be made to attain approximately 12 to 15 active working members for effective decision making.

The committee shall elect a chairman at the first opportunity in order that the activities of the committee may be conducted in an orderly manner.

The committee may choose to meet once a month or as often as necessary depending upon pertinent issues, urgency factors and the desire of members.

APPROVED BY:

For the Department of Parks and Recreation — Date

For the Parks and Recreation Advisory Board — Date

CHARTER MEMBERS:

SALADO CREEK WATERSHED STUDYAND DRAINAGE MASTER PLAN VOLUME 2

Contract #95-483-080

(1) Large Scale Map located in the official file, may be copied upon request.

McAllister Park Proposed Master Land Use Plan

Please Contact Research and Planning Fund Grants Management Division at (512) 463-7926

Upper Olmos Creek Watershed Master Drainage Plan for the City of San Antonio

Job No. 67187
December 1995
Revised February 1996

RUST LICHLITER/JAMESON
Environment & Infrastructure
Consulting Engineers, Scientists and Planners
2929 Briarpark Drive, Suite 600, Houston, Texas 77042-3203

UPPER OLMOS CREEK WATERSHED MASTER DRAINAGE PLAN

FOR THE

CITY OF SAN ANTONIO

Rust Lichliter/Jameson
2929 Briarpark, Suite 600
Houston, Texas 77042
Telephone: (713) 785-9800
Telefax: (713) 785-9779

Project No. 67187

February, 1996

I hereby certify these engineering documents were prepared by me or under my direct personal supervision and that I am a duly registered Professional Engineer under the laws of the State of Texas.

Signature: *Martha F Juch*

Name (typed or printed): Martha F. Juch, P.E.

Date: *2/15/96* Reg. No. *65416*

My registration renewal date is *December*, 19*96*.

STATE OF TEXAS
MARTHA F. JUCH, P.E.
65416
REGISTERED
PROFESSIONAL ENGINEER

TABLE OF CONTENTS

LIST OF TABLES

TABLE OF CONTENTS - Continued

LIST OF EXHIBITS

LIST OF APPENDICES

1.0 PROJECT DESCRIPTION

1.1 Project Authorization

In November of 1993, the City of San Antonio issued a request for Statements of Interest and Qualifications for the performance of Drainage Master Plans for three watersheds. The three watersheds include the Upper Olmos Creek Watershed, the Salado Creek Watershed and the Leon Creek Watershed. The latter two watershed studies were authorized to begin in the Spring of 1994 while the Upper Olmos Creek Watershed study was authorized to begin with City Council action on June 23, 1994. The City of San Antonio Public Works Department developed the project scope and objectives as discussed below and guided the progress of the projects. This report details the completed engineering services for the Upper Olmos Creek Watershed Drainage Master Plan project.

1.2 Purpose and Objectives

The Upper Olmos Creek Watershed Drainage Master Plan study was designed to provide the City of San Antonio and its citizens with a comprehensive plan with which to manage storm water runoff and minimize recurrent flooding of roads and structures. The limits of the study include the Olmos Creek watershed and main channel from the intersection of Loop 410 and West Avenue to a point upstream of Dreamland Road. From this point, the study includes both West Olmos Creek and East Olmos Creek (also known as Elm Creek) upstream to their limits in the watershed to the north of Anderson Loop 1604. Approximately 11 miles of drainage ways are included in the study effort.

1.3 Scope of Services

The Scope of Services for the Upper Olmos Creek Watershed Drainage Master Plan is divided into four engineering tasks. These are listed below along with a brief description of each:

A. <u>Preliminary Phase</u>

The Preliminary Phase of the Scope of Services for the Olmos Creek project involved the development of a watershed map illustrating the full limits of the watershed from Loop 410 at West Avenue to the headwaters north of Anderson Loop 1604. This Phase also involved the collection of all previous drainage studies including submittals to FEMA, Corps of Engineers studies, San Antonio River Authority studies, City and County studies, and studies for development purposes or street projects. These studies were analyzed with respect to their individual and collective contribution to the hydraulic and hydrologic understanding of the watershed.

The Preliminary Phase also involved extensive field reconnaissance and data collection with regard to recurrent flooding locations, drainage problems, low water crossings and watershed/subarea drainage boundaries. A photographic log of the significant hydraulic features was prepared as part of the field work. In addition, several informal information exchange meetings were held with citizens and other interested parties to discuss drainage issues in the watershed. Section 2.0 of this report presents the details of the services completed as part of the Preliminary Phase.

B. Design Phase

The Design Phase of the project involves all services relative to the development of the recommended Master Drainage Plan for Upper Olmos Creek. Specifically, this phase includes the development of hydrological models for the watershed based upon existing conditions and future full development of the watershed using land use projections provided by the City of San Antonio. The 10, 25, 50, 100 and 500 year flows for both conditions of development were predicted for the watershed. In addition, hydraulic models were developed for existing and future development conditions and analyzed to determine the 10, 25, 50, 100 and 500 year flood plains for existing development and the 100 and 500 year flood plains for future development conditions. Areas were identified where private property is inundated as a result of the 100 year rainfall event and a project was designed to mitigate the flooding for both existing and future development conditions. The Design Phase of the study is presented in Section 3.0.

C. Financial Plan

The Financial Plan portion of the Scope of Services for the Upper Olmos Creek Watershed project involved the development of a financing plan to fully implement the recommended Drainage Master Plan over a ten-year period, including a proposed funding source, proforma and schedule. An implementation plan is included within the Financial Plan presented in Section 4.0 of this report.

D. Development Criteria - Phase 1

As part of the Scope of Services for the Upper Olmos Creek Watershed Drainage Master Plan, a specific task has been identified to research and define new development criteria for the City of San Antonio which would address the drainage issues identified in the three watershed studies. This task encompasses the entire City and is closely tied in to the work being performed by the Drainage Regulation and Review Committee established by the City Council and SAWS Water Quality Task Force. The City of San Antonio established this Committee to insure a venue for interagency discussion and cooperation and for citizen input into the development of the drainage criteria. This committee has also reviewed the development of the three Drainage Master Plans and has had input into their design.

Phase 1 of the Development Criteria task is covered under this contract and is presented under separate cover in Appendix C. Phase 2 is projected to be performed under an Additional Services contract in 1996. The portions covered under this contract include a determination of San Antonio's goals for managing stormwater drainage as well as a comprehensive survey of ten other large cities with respect to drainage/stormwater management practices. The task also includes the identification of specific options for the City of San Antonio to implement in order to effectively manage stormwater drainage in both flood-prone and environmentally sensitive areas. Phase 2 of the Development Criteria task will involve the development of actual methods to implement the proposed criteria, including development of City ordinances and other political avenues.

2.0 PRELIMINARY PHASE

2.1 Introduction and Background

The City of San Antonio is located in Bexar County in south central Texas as shown on the Location Map in Exhibit 2-1. The City is one of the most rapidly growing urban centers in the State and is also the home to unique ecological, hydrologic and environmental features. This combination of urban growth and environmental sensitivity comes into potential conflict when addressing drainage issues in the City and provides the basis for the urgent need for comprehensive Drainage Master Plans for the City's watersheds and new Drainage Criteria to guide future growth.

The Olmos Creek Watershed is located in the north central part of the City of San Antonio and originates in the area between Leon Creek on the west and Salado Creek on the east in the southern edge of the Edwards Plateau. The stream flows in a southeasterly direction to its confluence with the San Antonio River near the fault zone in Brackenridge Park. The creek traverses the geological regions of the Edwards Plateau, the Balcones Escarpment, the Blacklands and the Rio Grande (Gulf Coastal) Plains[8] and ranges in elevation from 1180 feet NGVD to about 670 feet NGVD[1]. A large portion of the upper watershed is located in the recharge zone for the Edwards Aquifer as shown on Exhibit 2-2 and therefore requires special drainage considerations. Olmos Dam is located less than a mile upstream from the creek's confluence with the San Antonio River and was constructed to control flood flows in Olmos Creek. The limits of the study presented herein are confined to the upper reaches of the watershed north of the intersection of Loop 410 with West Avenue as shown in the highlighted area on Exhibit 2-2.

The study area north of Loop IH 410 addressed by this report and shown on Exhibit 2-2 encompasses about 16.6 square miles. The two main tributaries, West Olmos Creek and East Olmos Creek (Elm Creek), are included in the study and have drainage areas of 5.3 square miles and 7.8 square miles, respectively. Most of the study watershed is located within the corporate limits of the City of San Antonio, although a small portion to the north is in unincorporated Bexar County. Portions of the watershed are also located within the corporate limits of the Cities of Castle Hills and Shavano Park. The majority of the watershed is developed, with most of the growth occurring since about 1965.

2.2 Historical Hydrologic Data

The location of the City of San Antonio in south central Texas places it in a modified subtropical climatic zone in the Gulf Coast Region of the United States. The City has an average annual rainfall of 30.98 inches[8] and a prevailing south wind. Thunderstorms with characteristic brief, intense rainfall periods occur frequently in the spring and summer, while long-duration low-intensity storms resulting from southward moving cold fronts occur during the fall and winter. Historically, some of the heaviest rainfall has occurred in late summer and early fall as a result of hurricanes moving inland from the Gulf of Mexico[2].

The National Weather Service (NWS) operates a recording rain gage at the airport, which is approximately 3 miles east of Olmos Creek at Loop 410. No other NWS rain gages are located in or near the Olmos Creek Watershed. The City of San Antonio operates several rain gages in the areas as does the U.S. Geological Survey (U.S.G.S.). The locations of the gages located in or near the watershed are shown on Exhibit 2-3.

The U.S. Army Corps of Engineers prepared a report in 1972 which summarized flooding events in the Olmos Creek watershed prior to that year[1]. According to the historical sources referenced in this report, large floods were observed and recorded in the Olmos Creek watershed in 1893, 1921, and twice in 1946. From data collected on other watersheds, the report concluded that flooding probably also occurred in 1819, 1865, 1880, 1899, and twice in 1913, 1935 and 1957. The largest flood event this century was probably the one that occurred on September 9-10, 1921. A tropical storm moved inland from the Gulf of Mexico and produced rainfall ranging from 17 inches in the upper watershed to about 11 inches near San Pedro Avenue. This flood is reported to have been about six feet higher at Blanco Road than the 1946 flood discussed below. The City of San Antonio reported 49 persons killed in the flood with an additional 14 missing.

Other major flood events occurred on September 26-27, 1946 and November 10, 1946. The most devastating of these was the September flood, which resulted from a cold front stalling over the City, colliding with warm air from the Gulf, and producing rainfall of 10 inches at Olmos Dam and six inches in upper reaches of the watershed. Ten persons were reported dead in San Antonio as a result of this flood[1].

The United States Geological Survey installed a water-stage recorder on Olmos Creek at Dresden Drive in June of 1968. The maximum flood stage in the first two decades following the installation of the gage was 14.82 feet on September 13, 1978 (observed from floodmarks). This compares to an estimated stage of 8.5 feet at the location of the gage for the 1946 flood event[2]. The 1978 flood resulted from 3.5 inches of rain occurring in 3 hours and caused severe flooding in low lying areas throughout the watershed. Not until April 4-5, 1991 did severe weather again produce short periods of intense rainfall which resulted in major flooding in the watershed. A rainfall map of this storm prepared by Mr. John Patton of the National Weather Service River Forecast Center is replicated as Exhibit 2-3. A maximum rainfall of 10 inches was concentrated in the Upper Olmos Creek Watershed. Of the City's area network, gage 2201 (located at Vance Jackson and Wurzbach) recorded the rainfall at the 6.2 inch isohyetal line. A plot of the accumulated rainfall is presented as Exhibit 2-4. The estimated peak of 19,700 cubic feet per second at the U.S.G.S. gage on Olmos Creek at Dresden Drive produced the maximum discharge for the period of record. A stage of 14.4 feet was interpreted from floodmarks.[4] Severe flooding occurred along most of the length of Upper Olmos Creek and numerous areas of localized flooding outside of the flood plain were also reported.

On May 5 of 1993, a severe thunderstorm produced 4.5 inches of rain in 4 hours across the Olmos Creek watershed and again caused widespread flooding. The peak at the Dresden Drive gage was recorded at 13,860 cubic feet per second with a stage of 12.30 feet.[6] This flood and the 1991 flood both exceeded the 100 year recurrence frequency design flood estimated by the Federal Emergency Management Agent (FEMA). FEMA predicts a 100-year stage of 9.3 feet at Dresden Drive.[5]

Using NWS, U.S.G.S. and the City of San Antonio rainfall data compared to the U.S.G.S. flow data at Dresden Drive, an evaluation was made to relate the peak discharges to the maximum one-, two- and three-hour rainfall depths. Based on a comparison of the relationships, the two- and three-hour rainfall depths result in the best correlation to peak discharge in Olmos Creek. The three-hour rainfall vs. peak discharge at Dresden Drive relationship is shown as Exhibit 2-5. The scatter of data points may be attributable to antecedent soil moisture, distribution of the rainfall across the watershed, impact of runoff losses within the Edwards Aquifer recharge zone, or other hydrologic factors which have not been accounted for in this investigation.

2.3 Hydraulic Problems and Concerns

The flooding potential of Olmos Creek and its tributaries has been recognized for many years. Although structural damages from major flooding on Upper Olmos Creek, West Olmos Creek and East Olmos Creek were rare prior to the 1960's due to sparse development in the upper watershed, deaths from floods did occur in the area and economical hardships from flooded homes and properties were not uncommon. Since the advent of large residential developments in the upper watershed in the past thirty years, the damages from flooding have become more severe and the loss of life more dramatic. Concerned citizens have become more outspoken as their own and their neighbors' properties have been repeatedly flooded since 1990.

The recent flooding in the Olmos Creek Watershed has been concentrated in several critical residential areas: (1) Homes in the low areas near George Road and Lockhill Selma Road on East Olmos Creek and near Orsinger Road on West Olmos Creek, (2) Whispering Oaks Subdivision downstream of Wurzbach Road, and (3) Dreamland Oaks Subdivision downstream of Dreamland Drive. Reports of near-flooding have also indicated possible problems along the lengths of both forks. In addition, flood waters at the low water crossings in the watershed cause dangerous road conditions, including impaired vehicular and emergency access and occasional loss of life. The low water crossings on Upper Olmos Creek and both East and West Olmos Creeks are listed on Table 2-1. During severe storm events (and, in some cases, even minor storm events) these crossings become inundated with flows from the creeks. Photographs were taken at each observed hydraulic feature (i.e. bridges, low water crossings, etc.) and are presented as Exhibits 2-6 thru 2-16. Table 2-2 lists recent low water crossing rescues reported by the City of San Antonio Fire Department.[12] Reports of deaths or near-drownings at these crossings are not uncommon.

TABLE 2-1
LOW WATER CROSSINGS ON UPPER OLMOS CREEK MAIN CHANNELS

Roadway	Channel
Dreamland Drive	Olmos Creek
Lockhill Selma Road	East Olmos Creek
George Road	East Olmos Creek
Orsinger Road	West Olmos Creek
Sleepy Hollow	West Olmos Creek

Additional crossings which are overtopped during severe flood events:

Bridge/Roadway	Channel
Loop 410 Frontage Roads	Olmos Creek
West Avenue	Olmos Creek
Five-Southern Pacific Railroad Bridges	West Olmos Creek

TABLE 2-2
HIGH WATER RESCUES FROM JANUARY 1992 TO JULY 1994

Date	Location
01/26/92	Lockhill Selma / Wurzbach Road
01/26/92	Military Drive NW / George Road
02/04/92	Dreamland Drive / Vance Jackson
03/03/92	Dreamland Drive / Vance Jackson
03/03/92	George Road / Lockhill Selma
03/04/92	Garden View Drive / Lockhill Selma
03/04/92	Dreamland Drive / Vance Jackson
03/29/92	George Road / Lockhill Selma
03/29/92	North 410 Loop / West Avenue
03/29/92	North 410 Loop / West Avenue
05/16/92	Cherry Ridge / West Avenue
05/20/92	Lockhill Selma / Wurzbach Road
05/20/92	Cherry Ridge / Vance Jackson
05/21/93	Dreamland Drive / Lockhill Selma
05/05/93	North 410 Loop / West Avenue
05/05/93	Jackson Keller / West Avenue
05/05/93	George Road / Lockhill Selma
05/05/93	Janet Lee Street / Mary Knoll Lane
05/05/93	200 Quill Drive
05/05/93	Lockhill Selma / Wurzbach Road
05/05/93	North 410 Loop / West Avenue
05/06/93	North 410 Loop / West Avenue

Flooding conditions in the last few years have led to an increased awareness of the potentially threatening hydraulic characteristics of the drainage system by the City of San Antonio and other government agencies. The following paragraphs describe the entities which have addressed the flooding situation on Olmos Creek either directly or indirectly:

A. City of San Antonio

Most of the severe flooding which has been observed along Olmos Creek north of Loop 410 has been limited to properties within or very near the regulatory 100-year flood plain boundary as shown on the Federal Emergency Management Agency Flood Insurance Rate Maps. In the early development of the watershed, the attraction for building homes near to the natural creek bed in the desirable wooded flood plain outweighed the potential flooding risks. Flood plains were not well understood or identified until the advent of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. Although the City of San Antonio is a participant in the National Flood Insurance Program, many homes subjected to flooding along Olmos Creek were built prior to the publication of the Flood Insurance Rate Maps for San Antonio in 1983 and their owners are therefore unaware of or are not required to buy National Flood Insurance. As a result, most of the homes flooded in the recent 1991 and 1993 storm events were not insured for their damages. This increased economical strain, when combined with the emotional stress of potentially life-threatening flooding in neighborhoods and along roadways, catalyzed the residents and the political representatives of the areas in and around Upper Olmos Creek to request a review of the drainage problems in the area and the development of a drainage improvement plan to address the potential for future flooding.

The City of San Antonio became a participant in the National Flood Insurance Program in the early 1980's. Maps were produced by the Federal Emergency Management Agency which showed flood prone areas along major streams and tributaries and which were used to set flood insurance rates. The most recent maps were published in 1992 and show the regulatory flood plains for the 100 year and 500 year flood events for all areas, both incorporated cities and unincorporated areas, within Bexar County. The report which accompanies the maps is titled "Flood Insurance Study: City of San Antonio, Texas" and includes flow data and water surface profiles for Olmos Creek.[5] The limitations of the accuracy of these maps is discussed later in this report (Section 3.0).

In June of 1993 the Public Works Department of the City of San Antonio produced a "Drainage Assessment of Upper Olmos Creek" which identified critical elements of the Olmos Creek watershed with respect to potential flooding.[7] Ten channel improvement projects were described which, when totally constructed, would mitigate structural flooding from the 100 year design storm along Olmos Creek from San Pedro Avenue to upstream of Loop 1604. Two of these projects have either been completed or are in the advanced planning process. The first is referenced as the Olmos Creek Drainage Project Number 87-88 and was completed in 1984 by the City of San Antonio. The project extends from just downstream of San Pedro Avenue to upstream of Jackson Keller Road and consists of a combination of a fully lined concrete channel and an earthen channel with partial concrete lining and a pilot channel. This project was designed to contain the 100 year frequency design runoff within the banks of the channel through a heavily developed area of the City.

The second project referred to in the Drainage Assessment is a channel improvement project planned by the Texas Department of Transportation from Jackson Keller to a point upstream of West Avenue. The project is designed to improve flooding conditions at the Loop 410 frontage roads and West Avenue intersection and is in the last stages of planning.

The remaining eight projects described in the Drainage Assessment are possible channel improvement projects which would serve to increase the hydraulic capacity of Olmos Creek, West Olmos Creek and East Olmos Creek and alleviate flooding conditions in critical reaches of the streams. Most of the projects involve extensive channel excavation and the replacement of low-water road crossings with all-weather bridge crossings. Two of the projects involve detention/retention storage of flood waters in excavated areas adjacent to the channel. The projects would provide protection from the 100 year design flood along the entire length of Olmos Creek, but would disrupt the existing natural channel from the limits of the Olmos Creek Drainage Project Number 87-88 at Jackson Keller Road to George Road on East Olmos Creek. The extensiveness of the channel improvement projects described in the report is not widely accepted by the residential community in the watershed due to this disruption.

The City of San Antonio is currently pursuing an alternative to channel improvements which involves the diversion of flows from West Olmos Creek into the Vulcan Materials Company quarry upstream of Huebner Road. In the plan being considered by the City, the City and other governmental entities would pay Vulcan Materials Company to relocate their operations to another quarry site. In return, Vulcan would transfer ownership of the majority of the Huebner road site to the City for use as a regional retention facility. Vulcan would remove all surface improvements (i.e. buildings, parking lots, roadways, etc.) from the site, dredge the channel of West Olmos Creek adjacent to the site to remove a buildup of sediments and return it to its original capacity, and excavate a diversion channel from the creek to the quarry excavated pits. The plans for this detention alternative will be discussed in more detail in the Design Phase of this study.

B. <u>City of Shavano Park</u>

The City of Shavano Park is located in the northeast portion of the Olmos Creek watershed and extends into the Salado Creek watershed to the east. The City is composed entirely of residential developments, including both average-sized and estate-sized residential lots. Flooding occurs within the flood plain of Olmos Creek in the City and also within the flood plain of a tributary, Turkey Creek. Localized flooding also occurs in poorly drained areas developed prior to the City's participation in the National Flood Insurance Program in the early 1980's. No drainage easements were provided in the early planning of the City and, as a result, the natural drainage in some areas causes structural flooding. In addition, drainage swales have been altered by development or blocked by privacy fences causing repetitive flooding in isolated areas. The City commissioned a drainage study in 1993 which identified existing flooding problems and created a Master Drainage Plan to guide mitigation projects to relieve flooding.[9] Funding is not currently available for the City of Shavano Park to construct most of the identified projects.

C. City of Castle Hills

The City of Castle Hills is located in the south and southeast portions of the watershed and extends outside of the watershed to the east. Major flooding occurs along Olmos Creek within the Castle Hills city limits just upstream of Loop 410. This flooding is caused by a bottleneck of the flood plain through Loop 410 and West Avenue as well as the constriction on flows caused by the Southern Pacific Railroad just upstream of this intersection. In addition, localized flooding of structures occurs at isolated locations in the City and along tributary channels due to inadequate drainage structures. In 1983 the City commissioned a drainage study which identified flooding problems in the City and recommended improvements to mitigate this flooding.[3] The hydraulic calculations in the report were updated in 1991 in order to model the existing flood plain using the HEC-2 program and to predict the impact on the water surface profiles from proposed channel improvement projects. Projects will be completed by the City of Castle Hills as right-of-way and funding are obtained.

D. Other Governmental Agencies

Several other agencies have indirectly been concerned with drainage on Olmos Creek due to involvement in adjacent watersheds or water quality issues. The San Antonio River Authority is a political subdivision of the State of Texas which mainly targets the development of rural watershed protection and flood prevention projects within six watersheds in the San Antonio River basin. The SARA worked with the U.S. Army Corps of Engineers, the City of San Antonio and Bexar County to develop the San Antonio Channel Improvement Project. This project, to be completed in 1995, involved drainage improvements designed to provide the City of San Antonio with improved drainage and prevent severe flooding. The scope of these improvements does not extend to the Upper Olmos Creek watershed, however.

Another entity concerned with drainage on Olmos Creek as it relates to stormwater management is the San Antonio Water System (SAWS). This agency is heavily involved with issues related to implementation of the National Pollution Discharge and Elimination System (NPDES) requirements for the development of comprehensive stormwater management programs for the City of San Antonio. In addition, SAWS is concerned with water supply and, therefore, recharge of the Edwards Aquifer. The agency published "The Edwards Aquifer: San Antonio Mandates for Water Quality Protection" in 1994 which summarizes regulatory requirements, organizational programming and potential activities involving the Edwards Aquifer and its recharge zone, including those areas located within the Upper Olmos Creek basin.[10]

E. Subdivisions

Approximately 17 residential subdivisions are located adjacent to the main channel of Olmos Creek, West Olmos Creek or East Olmos Creek. These are listed on Table 2-3. As many of these subdivisions were developed, drainage studies were submitted to the City of San Antonio for review prior to plan approval. An examination of the files at the City of San Antonio Drainage Department showed that most of these studies contained limited hand calculations or abbreviated hydraulic computer model output files with little or no explanation in text form. These studies are of little value in the development of watershed hydrologic and hydraulic models for Upper Olmos Creek.

Residents of the Elm Creek subdivision at the northwest corner of Wurzbach Road and Lockhill-Selma Road have met with the Public Works Department of the City of San Antonio and expressed their concern over localized flooding across Lockhill-Selma Road and in reaches of East Olmos Creek (Elm Creek) downstream of Wurzbach Road. While the residents are concerned with flooding problems, they are also protective of the natural beauty of the Creek through their subdivision and are reluctant to endorse possible plans to either create excavated detention in the area or construct channel improvements upstream of Wurzbach Road. The engineering consultants and the Public Works Department have met with the residents' representatives, walked the channel and flood plain within the subdivision boundaries and recorded the natural features for consideration in the Design Phase of the Master Plan.

In addition, the Northside Neighborhoods for Organized Development (NNOD) has been involved in recognizing flooding problems in the area and in working with the City and other agencies to address critical concerns of the residents. NNOD hosted a public meeting on April 10, 1995 to review the progress of the study and will host future meetings to discuss the Master Drainage Plan for Upper Olmos Creek.

TABLE 2-3
RESIDENTIAL SUBDIVISIONS ADJACENT TO UPPER OLMOS CREEK MAIN CHANNELS

Subdivision	Adjacent to	Access Road
Colonial Oaks	Olmos Creek	Vance Jackson
Kings Grant Forest	Olmos Creek	Vance Jackson
Colonies North	West Olmos Creek	Vance Jackson
Colonies Village	West Olmos Creek	Vance Jackson
Mission Trace	West Olmos Creek	Vance Jackson
Woodland Manor	West Olmos Creek	Vance Jackson
Village Green	West Olmos Creek	DeZavala
University Oaks	West Olmos Creek	DeZavala
The Woods of Shavano	West and East Olmos Creek	DeZavala \ N.W. Military
Park Forest	West Olmos Creek	Lockhill Selma
Elm Creek	West and East Olmos Creek	Wurzbach
Whispering Oaks	West and East Olmos Creek	Wurzbach
Dreamland Oaks	Olmos Creek	Lockhill Selma
Hunter Creek	East Olmos Creek	Lockhill Selma
Castle Hills Forest	East Olmos Creek	N.W. Military
Shavano Creek	East Olmos Creek	N.W. Military

2.4 Preliminary Phase Chronological Bibliography

[1] "Flood Plain Information, Olmos Creek, San Antonio, Texas," prepared for The City of San Antonio and The San Antonio River Authority by Corps of Engineers, U.S. Army, Fort Worth, Texas, District, February 1972.

[2] "Hydrologic Data for Urban Studies in the San Antonio, Texas Metropolitan Area, 1975-1981," U.S. Geological Survey Open File Reports (multiple), prepared in cooperation with the Texas Department of Water Resources, 1977 through 1983.

[3] "City of Castle Hills Master Drainage Study," Garcia & Wright Consulting Engineers, Inc., March 1983.

[4] "Water Resources Data, Texas, Water Year 1991," U.S. Geological Survey Water Data Report TX-91-3, prepared in cooperation with the State of Texas, 1991.

[5] "Flood Insurance Study, City of San Antonio, Texas, Bexar County," Federal Emergency Management Agency, Revised July 2, 1991.

[6] "Water Resources Data, Texas, Water Year 1993," U.S. Geological Survey Water Data Report TX-93-3, prepared in cooperation with the State of Texas, 1993.

[7] "Drainage Assessment for Upper Olmos Creek," Drainage Engineering Section, Department of Public Works, City of San Antonio, June 1993.

[8] "Issues and Impacts of Stormwater Drainage in Bexar County," Environmental Sciences and Engineering Programs, The University of Texas at San Antonio, Summer 1993.

[9] "City of Shavano Park Master Drainage Plan," Vickrey & Associates, Inc., December 20, 1993.

[10] "The Edwards Aquifer: San Antonio Mandates for Water Quality Protection," San Antonio Water System, April 1994.

[11] "Site Analysis for Storage Detention Feasibility, East Fork of Olmos Creek Upstream of Lockhill-Selma Road, San Antonio, Bexar County, Texas," Robert B. Hahn, P.E., Pape-Dawson Consulting Engineers, Inc., June 1994.

[12] "High water rescue data - January 1992 through July 1994, provided by Lt. Jim Collins, San Antonio Fire Department, August 1994.

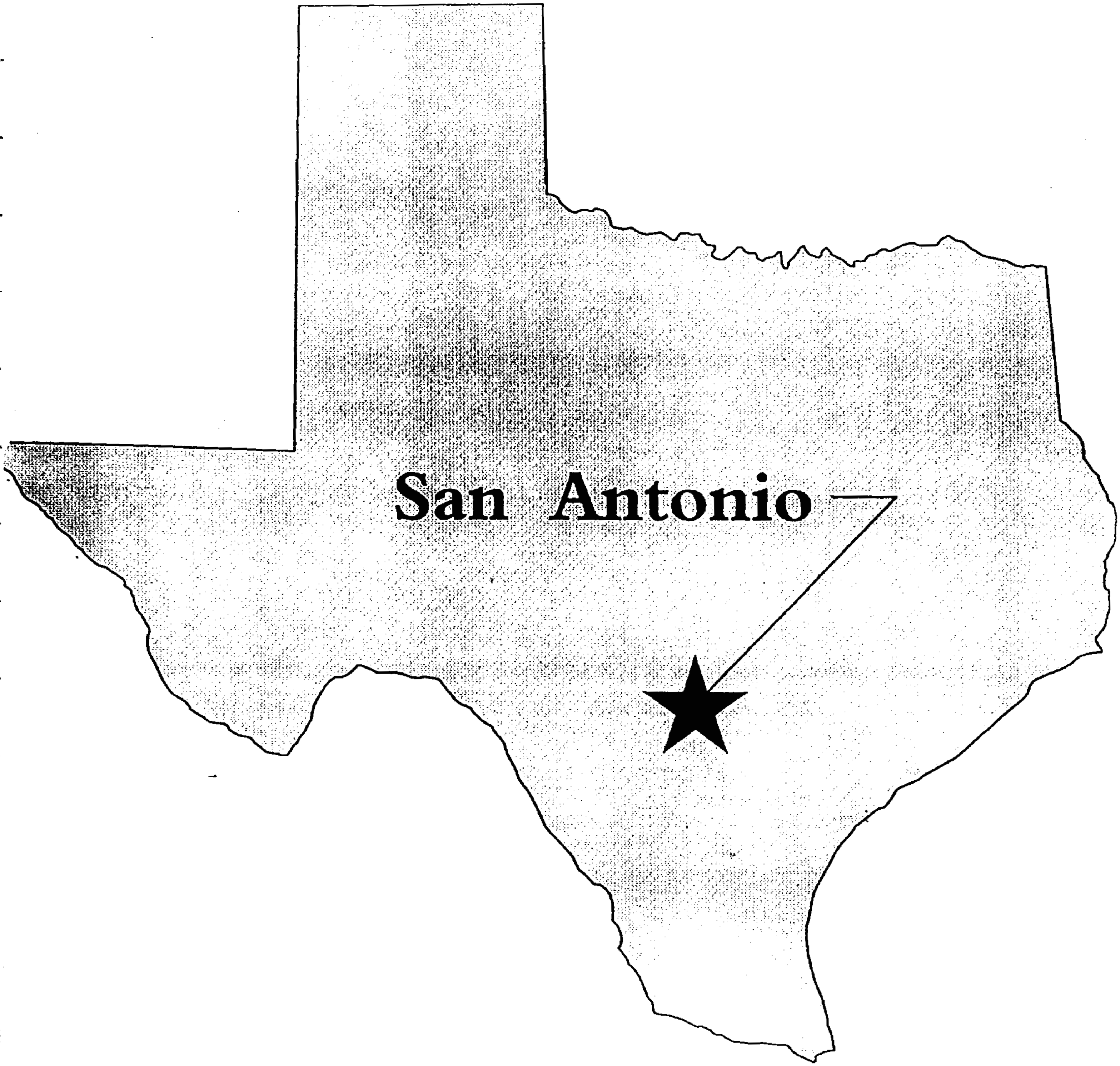
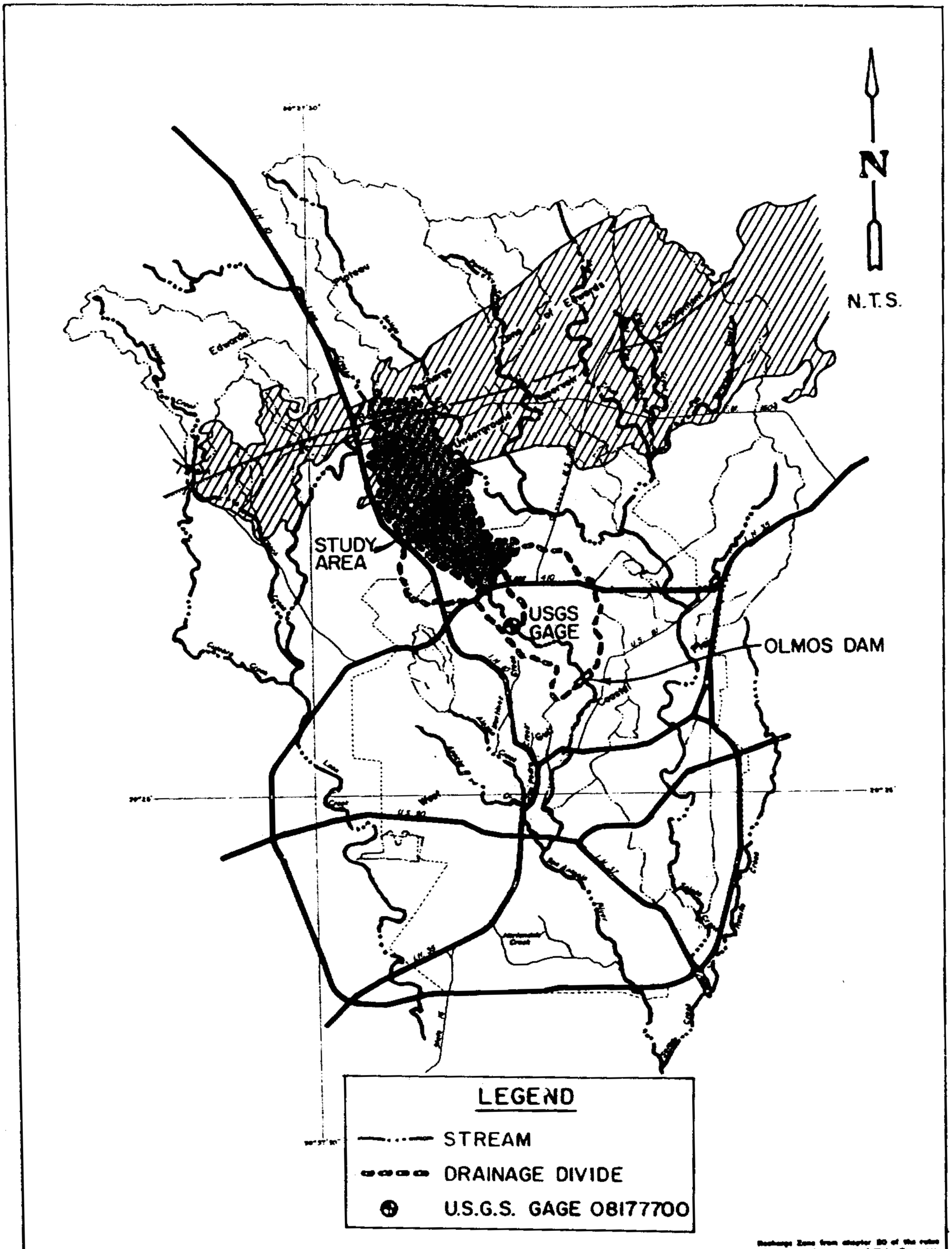


EXHIBIT 2-1

VICINITY MAP

SEPTEMBER, 1995

RUST LICHLITER/JAMESON
Environment & Infrastructure
Consulting Engineers, Scientists and Planners
2929 Briarpark, Suite 600, Houston, Texas 77042-3703



BASED ON FIGURE 1 - "HYDROLOGIC DATA FOR URBAN STUDIES IN THE SAN ANTONIO, TEXAS, METROPOLITAN AREA, 1981", U.S. GEOLOGICAL SURVEY OPEN FILE REPORT 83-35, AUSTIN, TEXAS, 1983.

EXHIBIT 2-2

WATERSHED LOCATION MAP

SEPTEMBER, 1995

RUST LICHLITER/JAMESON

Environment & Infrastructure
Consulting Engineers, Scientists and Planners
2929 Briarpark, Suite 600, Houston, Texas 77042-3703

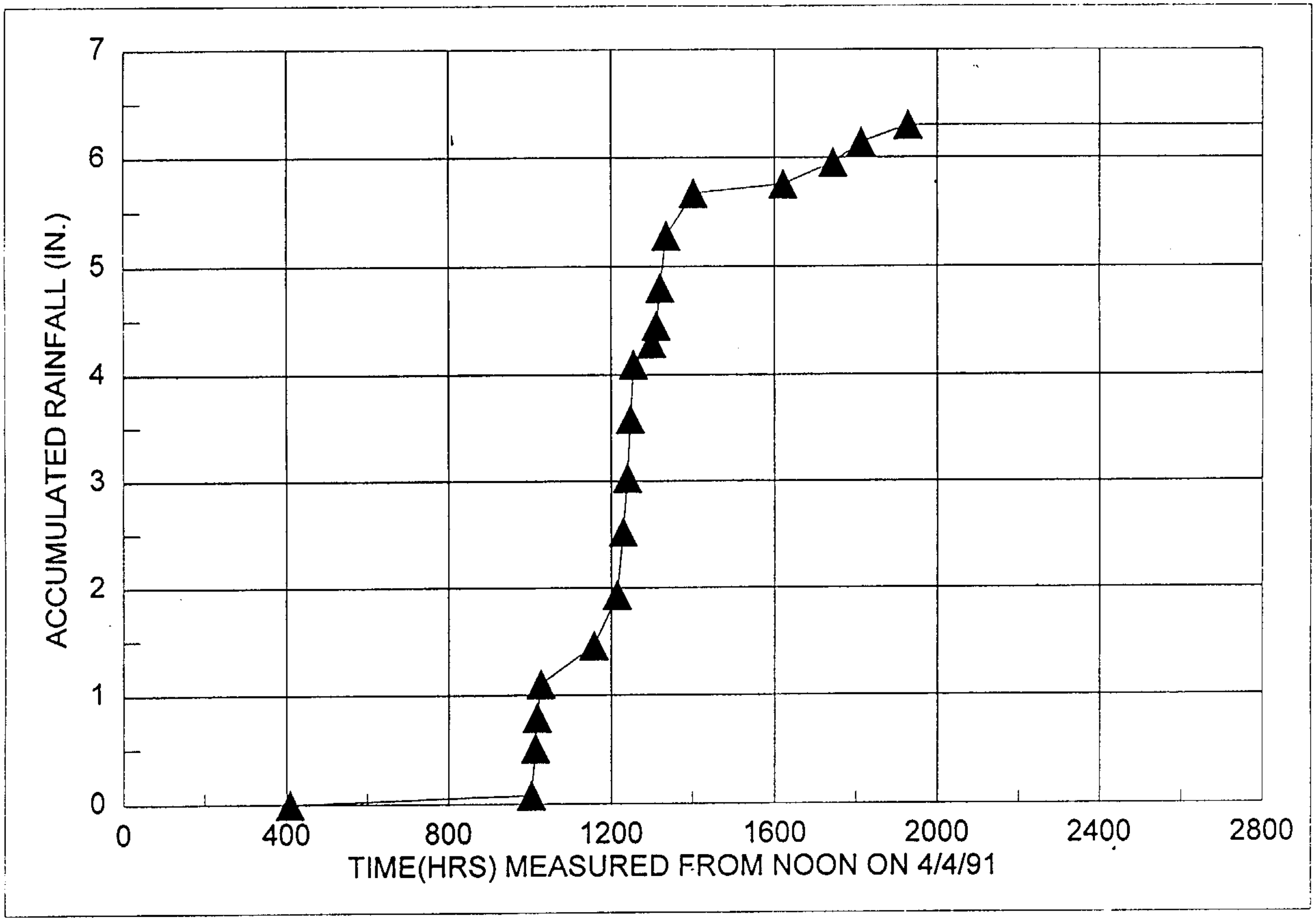


EXHIBIT 2-4

ACCUMULATED RAINFALL AT GAGE 2201 APRIL 4-5 ,1991

RUST LICHLITER/JAMESON

Environment & Infrastructure
Consulting Engineers, Scientists and Planners
2929 Briarpark, Suite 600, Houston, Texas 77042-3703

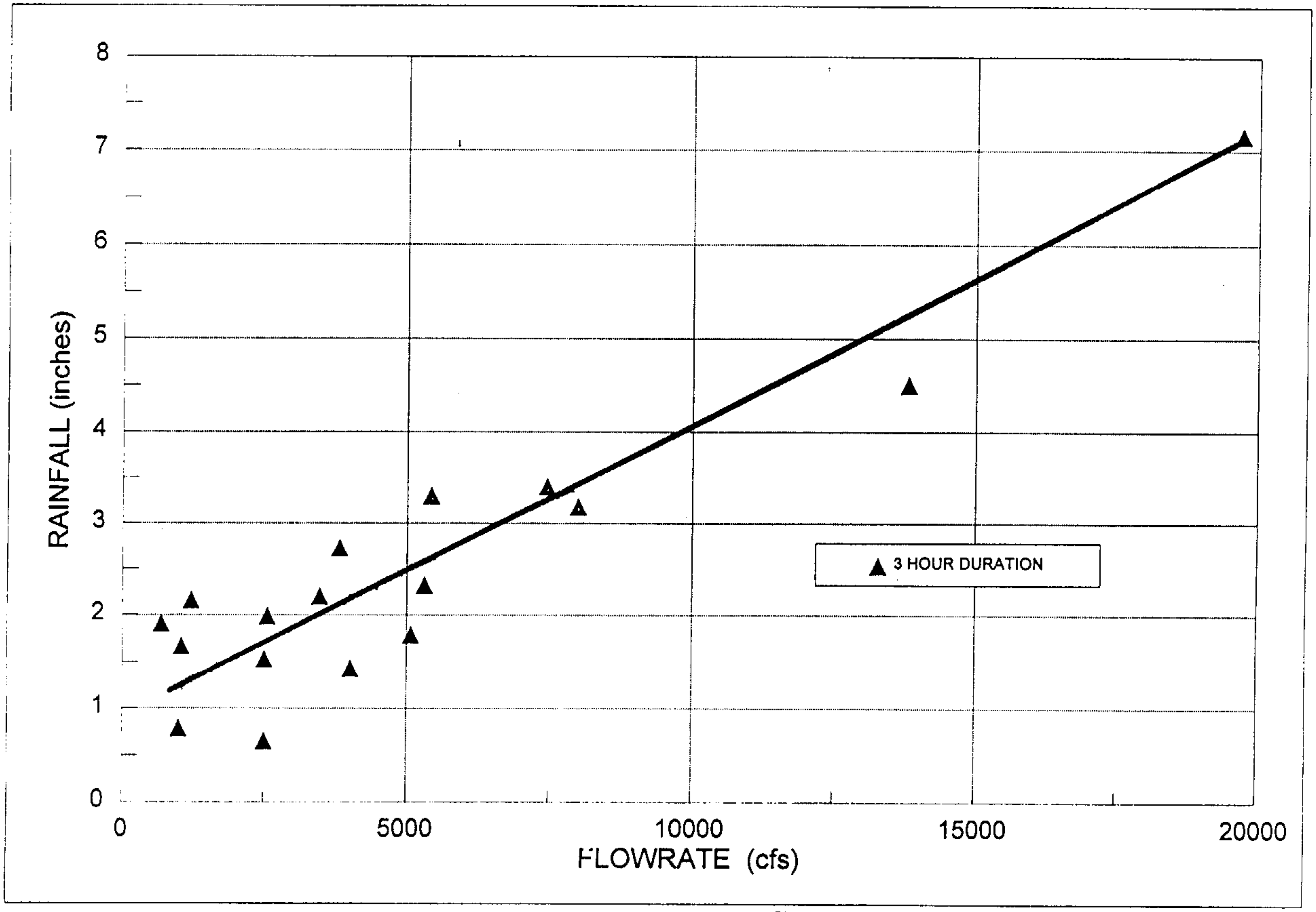


EXHIBIT 2-5

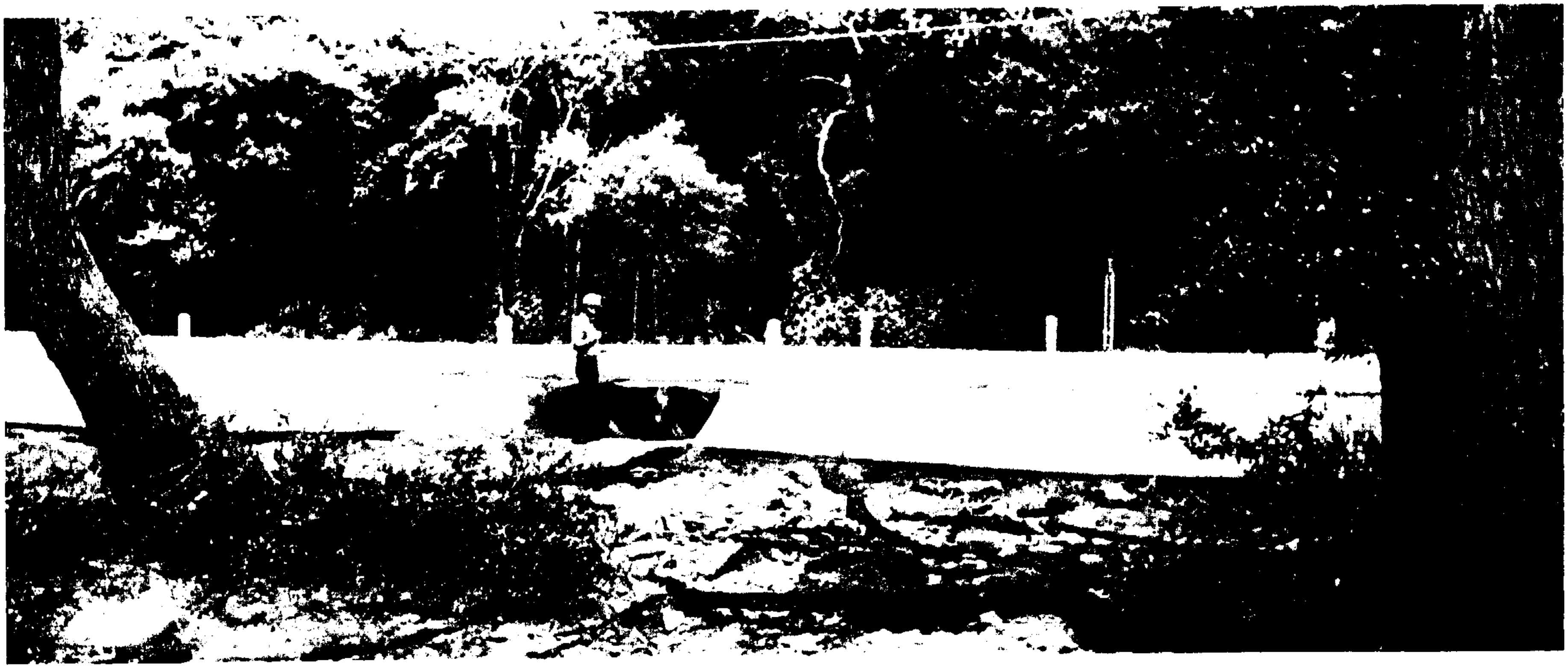
THREE-HOUR RAINFALL VS. PEAK DISCHARGE AT DRESDEN DRIVE

RUST LICHLITER/JAMESON

Environment & Infrastructure
Consulting Engineers, Scientists and Planners

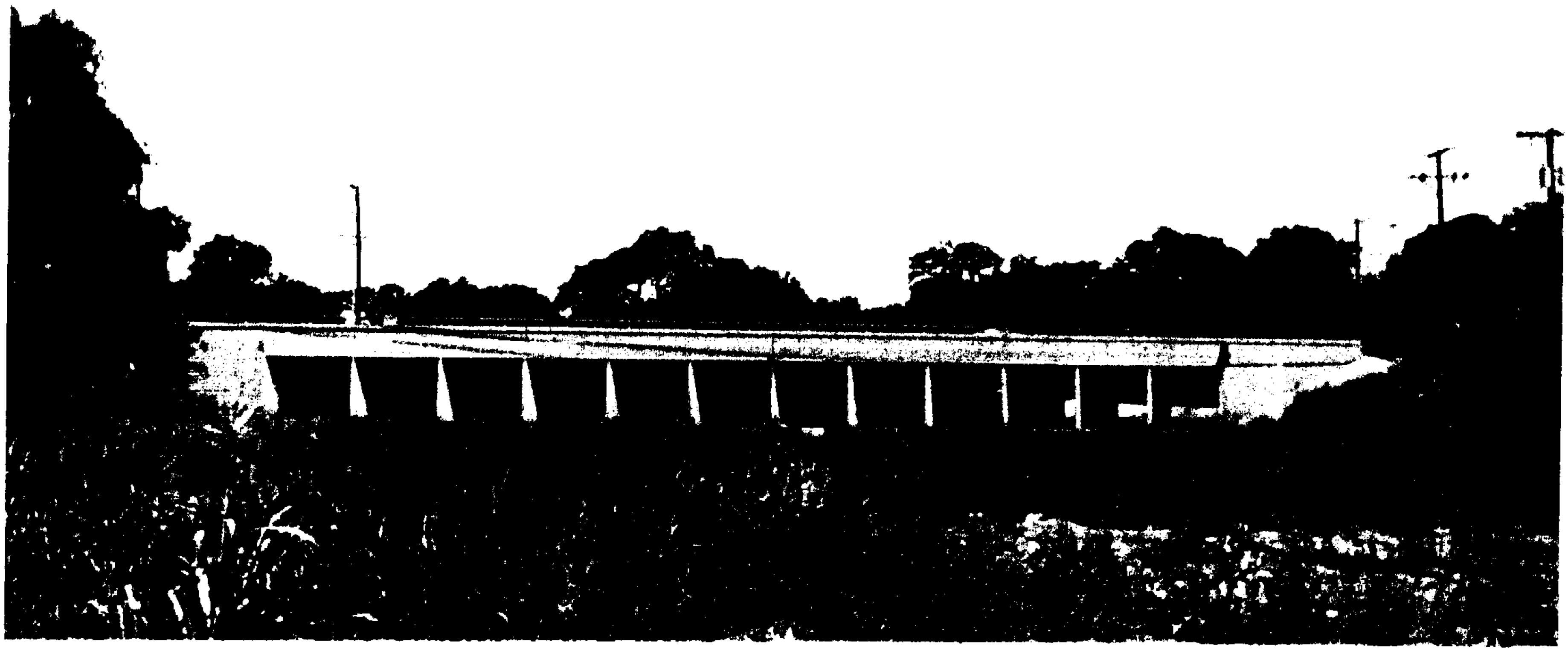
Olmos Creek
Upstream of West Road at Loop IH410
6-8'x4' Box Culverts
Looking Downstream

EXHIBIT 2-6



Olmos Creek
Downstream of Dreamland
3-24" Circular Culverts
Looking Upstream

EXHIBIT 2-7



East Olmos Creek
Downstream of Wurzbach Road
12-10'x10' Box Culverts
Looking Upstream

EXHIBIT 2-8

East Olmos Creek
at Lockhill Selma
Looking Downstream

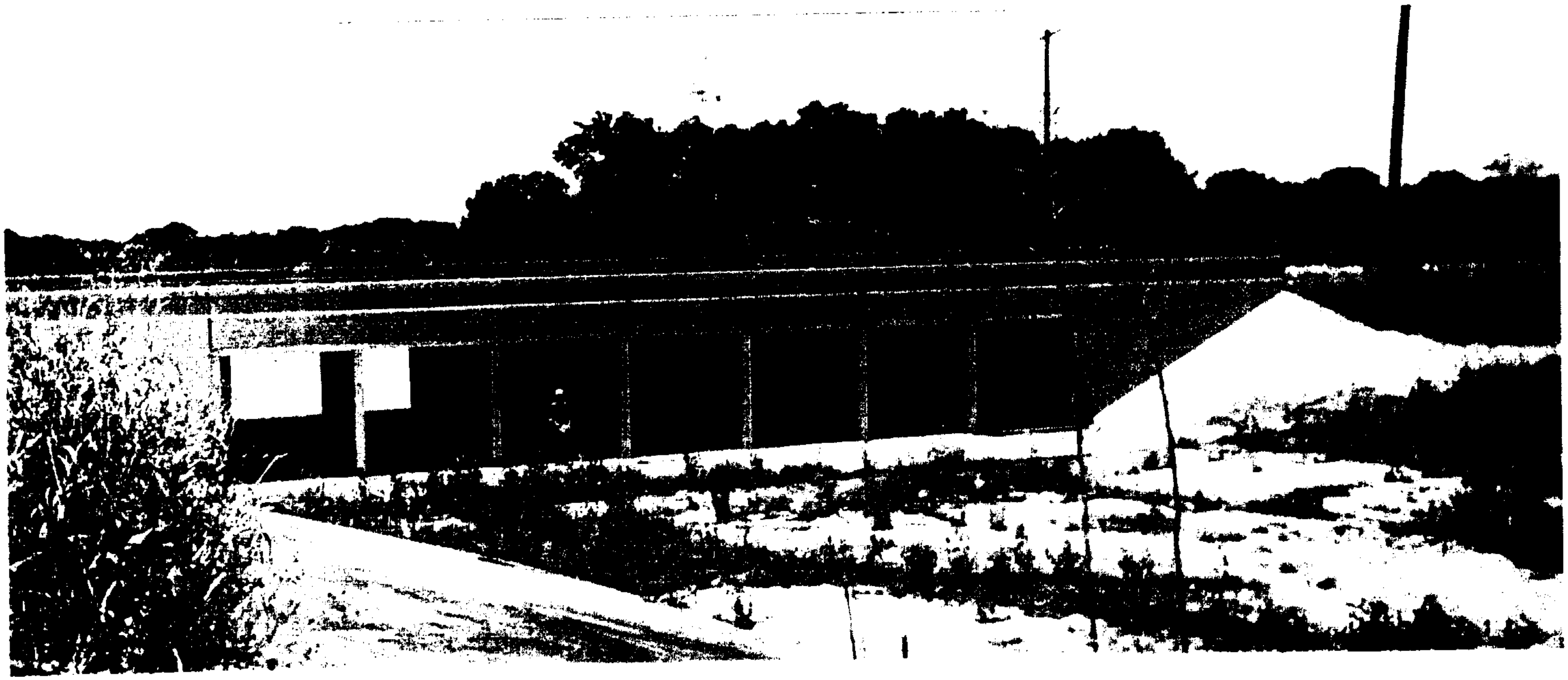
EXHIBIT 2-9

East Olmos Creek
at George Road
Looking Upstream

EXHIBIT 2-10

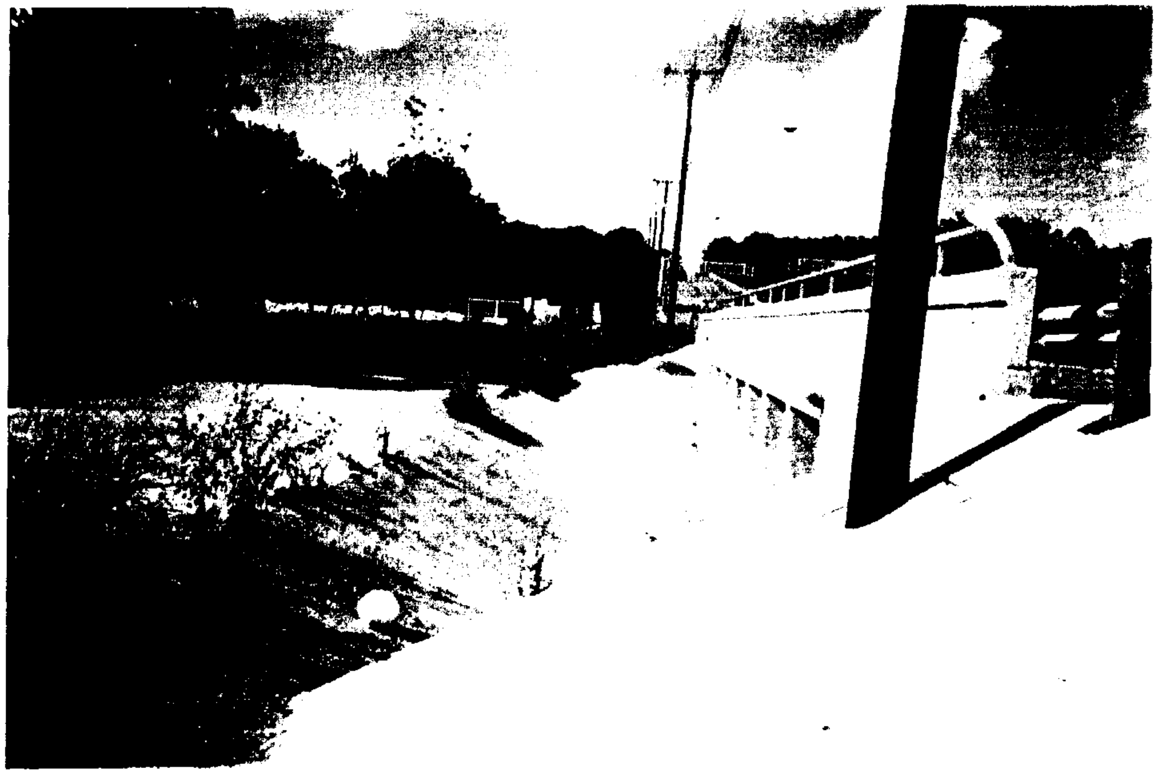
East Olmos Creek
Downstream of De Zavala Road
6-8'x8' Box Culverts
Looking Upstream

EXHIBIT 2-11



West Olmos Creek
Downstream of Wurzbach Road
7-10'x10' Box Culverts
Looking Upstream

EXHIBIT 2-12



West Olmos Creek
at Wurzbach Road
7-10'x10' Box Culverts with Drop
Looking East along upstream face

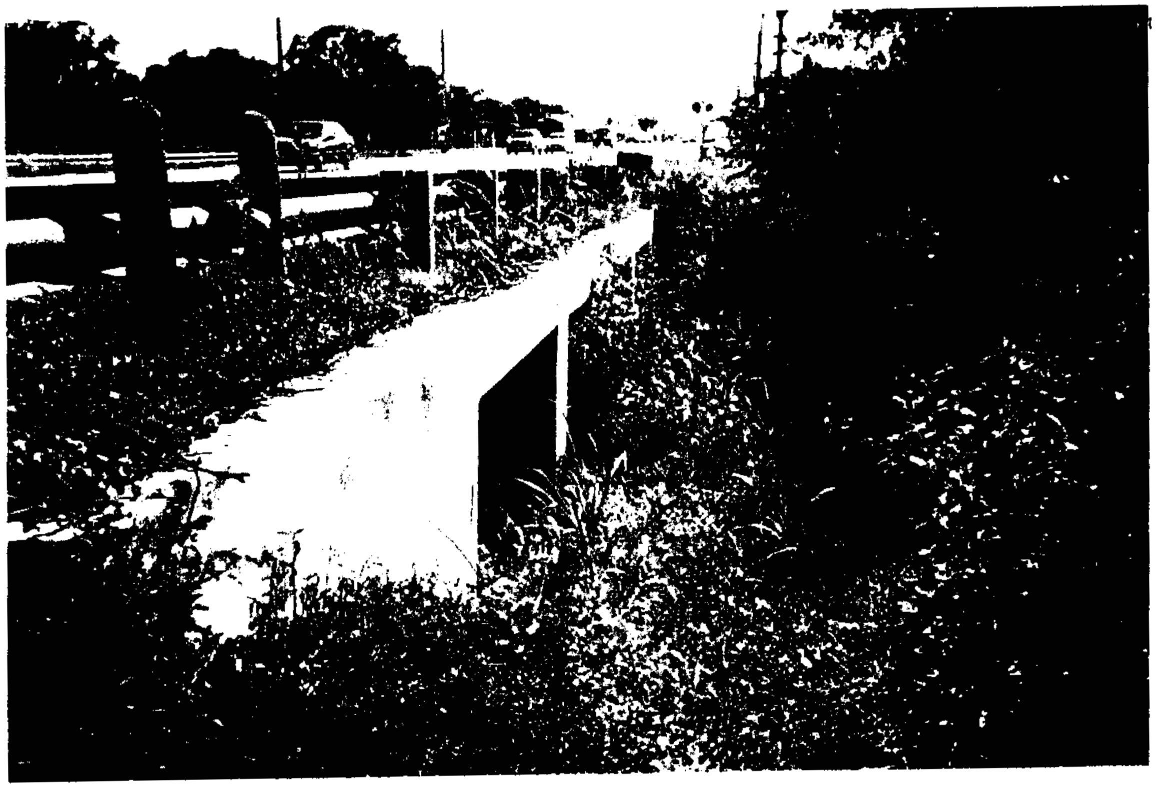
EXHIBIT 2-13

Typical Railroad Bridge

EXHIBIT 2-14

West Olmos Creek
at Orsinger
Looking Southwest

EXHIBIT 2-15



West Olmos Creek
Upstream of De Zavala
4-10'x5' Culverts
Looking Downstream

EXHIBIT 2-16

3.0 DESIGN PHASE

3.1 Introduction

The Design Phase of the Upper Olmos Creek Watershed Master Drainage Plan involved the development of new computer models to simulate the hydrology and hydraulics of the Upper Olmos Creek watershed, the determination of accurate flood plain boundaries for the 10-, 25-, 50-, 100- and 500-year frequency design storm events, and the development of a Master Drainage Plan which will reduce or eliminate the flooding of structures and property.

Historically, the regulatory flood plain for Olmos Creek was determined and mapped by the Federal Emergency Management Agency (FEMA) using generalized regional hydrologic equations and hydraulic conditions simulated by the Corps of Engineers. In order to more accurately determine the flood plain boundaries for existing and ultimate conditions in the watershed, a set of computer models using the most recent available topographic and hydrologic data was necessary. The development and verification of these models, as well as their application to the development of a Master Drainage Plan, is described below.

3.2 Analysis of Existing Conditions

The U.S. Army Corps of Engineers Hydrologic Engineering Center (HEC) in Davis, California, has developed a series of hydrologic and hydraulic computer models which enable engineers and scientists across the United States to select appropriate methodologies for their given regional parameters and then simulate rainfall, runoff and channel flow under various conditions. These models are considered the industry standard and are used extensively in Texas to provide consistent and reproducible results when analyzing watershed drainage systems. The models used in the San Antonio watershed studies are the HEC-1 "Flood Hydrography Package" and the HEC-2 "Water Surface Profiles" computer models.

A. <u>Hydrology</u>

The model used to simulate the rainfall-runoff characteristics in the Upper Olmos Creek study is the HEC-1 model. This model allows the user to select the methodology used to compute the runoff hydrography based on the physical characteristics of the watershed, the level of detail of data available, and the degree of sophistication required for the analysis. Printouts of all of the HEC-1 models described in this report are included under separate cover as Appendix A. Each component of the hydrologic analysis performed by the model is described in the following paragraphs.

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1. Unit Hydrograph Methodology

The Upper Olmos Creek watershed is defined as the 16.55 square mile contributing drainage area above the Loop 410 crossing. The area was divided into nine subareas in order to determine the drainage characteristics under existing conditions. A subarea drainage map is shown as Exhibit 3-1. Table 3-1 shows the physical parameters determined for the Upper Olmos Creek watershed subareas which were used as input data for the HEC-1 model.

The Project Team, composed of representatives from the City of San Antonio staff, Rust Lichliter/Jameson, Inc., Pape-Dawson Engineers and Vickery & Associates, determined a consistent methodology for analyzing the hydrology of the three watersheds being studied. The HEC-1 SCS Method was selected to determine the loss rates and define the unit hydrograph in the runoff computations. The SCS Curve Numbers (CN) used in the analyses were determined from standard tables relating CN values to soil types, and the ranges of values to be applied to each watershed were developed jointly by the Project Team. The raw CN values were input directly into the HEC-1 model, and the impervious cover parameter (RIMP) was used to represent variations in land use.

Land use values for each subarea shown on Exhibit 3-2 were determined from a 1994 aerial photograph of the watershed and were verified by land use data supplied by the City. These values are also shown on Table 3-1. The percentages of each land use type in a subarea were weighted to give the effective impervious cover for the subarea as shown on the table as the RIMP value. The effective impervious cover is defined as that area which directly carries 100% of the runoff through the subarea to the outfall channel. All of the impervious cover in commercial/industrial land use areas (average impervious cover of 80%) was assumed to be effective. Half of the impervious cover in residential/low density areas was assumed to be effective.

The initial surface moisture storage capacity (initial abstraction) is computed by HEC-1 from the SCS curve number and produces results consistent with a wet antecedent soil moisture condition. The lag for each subarea was determined from an investigation of overland flow paths, lengths and travel times estimated from aerial photographs and field data. The lag values were calculated by taking the length of the longest water course and dividing it by the average velocity in the channel (5 ft/s).

TABLE 3-1
HYDROLOGIC RUNOFF PARAMETERS FOR EXISTING CONDITIONS
UPPER OLMOS CREEK WATERSHED

Subarea	Area (sq mi)	Land Use Type (Percent of Total Area)				RIMP** (%)	Lag (hours)	Raw CN
		Residential	Commercial*	Low Density	Undeveloped			
W1	1.49	85	5	0	10	18.9	0.50	74.2
W2	1.64	50	10	0	40	16.8	0.25	76.7
W3	2.26	20	10	0	70	11.5	0.50	74.8
E1	0.82	60	5	0	35	14.5	0.50	76.7
E2	1.64	60	5	0	35	14.5	0.50	76.7
E3	1.75	35	0	40	25	10.1	0.50	77.0
E4	1.28	20	0	50	30	8.5	0.50	77.0
E5	2.76	0	0	10	90	1.0	0.50	77.0
M1	2.91	45	35	0	20	35.9	0.50	74.9
Total (Study Area)	16.55							

* "Commercial" includes commercial, industrial and high density residential (apartments)

** RIMP = 0.5[35% imperviousness(residential %) + 20% imperviousness(low density %)] + 80% imperviousness(commercial %)

Precipitation data for the design storms used in the analysis was provided by the City of San Antonio. The 10-, 25-, 50-, 100-, and 500-year frequency events were simulated for a 24-hour duration. All three watersheds studied use the same rainfall distributions shown in Table 3-2 to simulate the design storms and develop flood plain maps.

TABLE 3-2
DESIGN RAINFALL VALUES

Duration	Frequency				
	10-Year	25-Year	50-Year	100-Year	500-Year
5 minute	0.64	0.73	0.80	0.87	1.03
15 minute	1.39	1.59	1.75	1.91	2.25
60 minute	2.90	3.43	3.84	4.25	5.20
2 hour	3.66	4.42	4.99	5.57	6.95
3 hour	4.23	5.04	5.64	6.23	7.60
6hour	4.99	5.89	6.52	7.13	8.47
12 hour	5.55	6.58	7.32	8.05	9.68
24 hour	6.55	7.78	8.78	9.91	12.75

2. Hydrograph Routing

The normal-depth channel routing option was selected for use in the HEC-1 model by the Project Team. This method uses Manning's equation to determine outflows for normal depth conditions. Storage is computed from the cross-sectional area and the reach length of the routing reach. Input values for the overbank and channel Manning's coefficient ("n"), the reach length, the energy grade line slope and a representative channel cross-section configuration are used in the calculation to route the hydrograph through the subarea. The average cross-section for each routing reach was determined by visually inspecting each cross section within the reach and picking the most representative cross section. The HEC-1 model allows eight elevation/station pairs of data (RX-RY points); therefore, the representative cross sections were slightly modified from the more detailed HEC-2 model input data. The physical parameter values used for each routing reach in the study area are shown on Table 3-3.

TABLE 3-3
HYDROLOGIC ROUTING PARAMETERS FOR EXISTING CONDITIONS
UPPER OLMOS CREEK

Reach From Node - To Node	Manning's n Value		Reach Length (ft)	Energy Grade Line Slope (ft/ft)	Representative Cross Section
	Overbank	Channel			
ROUT1 (OLW1 to OLW2)	0.09	0.06	4750	0.01	121500
ROUT2 (OLW2 to OL1)	0.09	0.06	9000	0.005	116500
ROUT3 (OLE1 to OLE2)	0.09	0.09	5500	0.006	34000
ROUT4 (OLE2 to OLE3)	0.09	0.06	7400	0.006	30500
ROUT5 (OLE3 to OLE4)	0.09	0.06	7000	0.004	21000
ROUT6 (OLE4 to OL1)	0.09	0.06	5000	0.004	16500
ROUT7 (OL1 to OL2)	0.08	0.05	9450	0.005	9000

The hydrologic modeling described above produced results which are consistent with observations of runoff and flooding in the area as reported by City staff and residents in the watershed. Table 3-4 compares the peak flow rates predicted by the model at Loop 410 to those published by FEMA. The peak flowrates reported by FEMA for use in the Flood Insurance Study differ from the results of the hydrologic modeling due to the differences in methodologies used in the two studies. FEMA used general regionalized hydrologic equations to predict flows in the channel. This method produces results which are reasonable and accurate when averaged over a large region; however, the precision of the results are reduced when applied to a specific portion of a watershed subarea or channel reach. The hydrologic modeling used in this study utilized detailed physical parameters specific to the Upper Olmos Creek watershed. In addition, the rainfall intensities used in this study were developed for the City of San Antonio and reflect pattens consistent with historical rainfall records in the City, and the routing in the Upper Omos Creek channel reflects data based on a recent aerial topographic survey of the area.

TABLE 3-4
COMPARISON OF PEAK FLOW RATES AT LOOP 410
UPPER OLMOS CREEK

Rainfall Return Frequency	HEC-1 Peak Flow Rate (cfs)	FEMA Peak Flow Rate (cfs)
10-year	12,068	4,100
50-year	18,767	11,250
100-year	22,233	20,500
500-year	29,917	34,200

B. Hydraulics

The hydraulics of the Upper Olmos Creek drainage system were modeled using the HEC-2 backwater program. Input data for the cross sections coded into the HEC-2 were developed from three-dimensional topographic data derived from aerial survey data taken along the channel in 1994. Extensive field reconnaissance was performed in order to observe the hydraulic characteristics of the main channel and tributaries. Photographs were taken at each observed hydraulic feature (i.e. bridges, low water crossings, culverts, tributary confluences, and other abrupt changes in channel configuration) and are shown in Section 2 of this report. Printouts of all of the HEC-2 models described within this report are included under separate cover as Appendix B.

1. Manning's Roughness Coefficients

Manning's roughness coefficients ("n") used in the model were developed from field observations from the 1994 aerial photograph and from earlier engineering studies performed for large developments in the watershed. The values were also revised to reflect preliminary calibration analyses performed by the engineering teams. For example, very dense natural growth and trees in the channel were modeled using an "n" value of 0.09; natural but less dense growth and trees in the channel were modeled using 0.075; and, natural with grass undergrowth or past clearing (but little or no maintenance) were modeled using 0.05. Channel reaches which had been rectified and cleared as well as maintained were simulated with an "n" value of 0.035. Overbank areas with buildings, other structures, or dense trees and undergrowth blocking the natural flow of the water were modeled with 0.09.

The topographic data was supplemented in several key locations by field surveys of slab elevations of selected structures. This information was used to verify historical flood elevations and to refine the floodplain maps.

2. Bridge Analysis

Bridge plans were collected where available and measurements of hydraulic features were taken in the field. A listing of all the bridge crossings for the Upper Olmos Creek watershed is shown on Table 3-5.

TABLE 3-5
BRIDGE CROSSINGS UPPER OLMOS CREEK

Channel	Roadway	Type	Modeling Method	HEC-2 Station
Main	Loop IH 410	Culverts (6-8'x4')	SB-w/Plans and Photos	1570
Main	West Avenue	Culverts (6-8'x4')	SB-w/Plans and Photos	2000
Main	Railroad	Low Bridge	SB-w/Photos	2810
Main	Dreamland Drive	Low Water with Culverts	GR-w/Photos	10930
East	Railroad	Low Bridge	SB-w/Photos	11500
East	Wurzbach Road	Culverts (12-10'x10')w/Drop	SC-w/Plans and Photos	15600
East	Lockhill-Selma	Low Water without Culverts	GR-w/Photos	16900
East	George Road	Low Water with Culverts	GR-w/Photos	21200
East	Huebner Road	High Bridge	SB-w/Plans	28000
East	De Zavala Road	Culverts (6-8'x8')	SC-w/Photos	32800
West	Wurzbach Road	Culverts (7-10'x10') w/Drop	SC-w/Plans and Photos	114600
West	Railroad No. 1	Low Bridge	SB-w/Photos	115030
West	Railroad No. 2	Low Bridge	SB-w/Photos	117050
West	Railroad No. 3	Low Bridge	SB-w/Photos	118300
West	Railroad No. 4	Low Bridge	SB-w/Photos	119800
West	Orsinger Road	Low Water with Culverts	GR-w/Photos	122100
West	Railroad No. 5	Low Bridge	SB-w/Photos	123200
West	Huebner Road	Culverts (11-9'x9') w/Drop	SC-w/Plans and Photos	123800
West	De Zavala Road	Culverts (4-10'x5')	SC-w/Plans and Photos	130600
West	De Zavala Road	Culverts (4-10'x5')	SC-w/Plans and Photos	130600
West	Red Maples Road	Culverts (4-10'x5')	SC-w/Plans and Photos	131400

SB = Special Bridge Method
SC = Special Culvert Method
GR = Modeled with one cross section defining the low water crossing while not modeling the culverts.
NOTE: *Confluence of East and West Olmos Creek* = HEC-2 Station 11200

3. Split Flow Analysis at IH 410

Through discussions with the City of San Antonio staff and several residents in the area, the flow at the IH 410 / Olmos Creek intersection has been observed to be partially diverted to the west at the Jackson-Keller / IH 410 intersection during high rainfall events. The split flow hydraulic condition was analyzed by creating two separate HEC-2 models. The first model contains four cross sections and begins just downstream of the Jackson-Keller / IH 410 intersection and proceeds upstream to match the main Olmos Creek existing model at cross section 3100. This cross section is 200 feet upstream of the first railroad crossing upstream of West Road. Critical depth was used to determine the starting water surface elevation for this model. The second model is a truncated version of the Upper Olmos Creek existing conditions model which contains cross sections 100 to 3100. The starting water surface elevation for this model is determined by the slope area method.

It was determined that the flow begins to be diverted when the flow in the Olmos Creek channel reaches 11,000 cfs. When the flow in the channel is above 11,000 cfs, the railroad crossing backs up the water so that it becomes diverted at the Jackson-Keller / IH 410 intersection. The amount of diversion above 11,000 cfs was determined by running a combination of flows in the two models until the calculated water surface elevations in both models were equal. Table 3-6 is a rating table of the diversion. The diversion reduces flows in the existing HEC-2 model between cross section 100 and 3100. The total flow given by the HEC-1 model at the combination point at IH 410 is used upstream of cross section 3100.

TABLE 3-6
RATING TABLE OF THE DIVERSION AT IH 410
UPPER OLMOS CREEK

Total Combined Flow (cfs)	Flow in Channel (cfs)	Diverted Flow (cfs)
11,000	11,000	0
15,000	12,500	2500
17,000	13,500	3500
20,000	15,200	4800
25,000	17,500	7500
30,000	20,200	9800
35,000	23,000	12,000

4. Flow Distribution

Shown as Tables 3-7 and 3-8 are the calculations of the flow distribution process for the existing model. Table 3-7 is the raw flow values taken from the combination points from the HEC-1 printout. Table 3-8 shows the flows distributed for input into the HEC-1 model using semi-log interpolation to determine intermediate flow values between combination points. The flows were also incremented at the confluence of large tributaries in order to reflect the changes in runoff based on the contributing drainage area. A graphical representation of the 10-year and 500-year frequency flow distribution is shown on Exhibit 3-3.

TABLE 3-7
PEAK FLOW RATES COMPUTED BY THE HEC-1 PROGRAM
EXISTING CONDITIONS - UPPER OLMOS CREEK

Location	HEC-1 Computation Point	HEC-2 Section	10-Year Peak Flow (cfs)	25-Year Peak Flow (cfs)	50-Year Peak Flow (cfs)	100-Year Peak Flow (cfs)	500-Year Peak Flow (cfs)
Loop 410	OL2	1800	11598	15528	18322	21729	29502
Dnstr Confluence	OL1 D/S	11200	10974	14645	17323	20079	26844
East Fork:							
Upstream -East	OL1 U/S	11500	6370	8553	10088	11715	15638
Lockhill Selma	OLE4	17000	6013	8147	9605	11212	15020
Upstr. George	OLE3	24000	5322	7110	8512	10015	13608
Dnstr. DeZavala	OLE2	32000	4280	5624	6594	7710	10293
Trib. Confluence	OLE1	37500	3497	4478	5209	5982	7809
West Fork:							
Upstream-West	OL1 U/S	112500	5079	6730	7925	9178	12654
Dnstr Huebner	OLW2	121000	4397	5634	6564	7551	9970
Vulcan Quarry	OLW1	126000	2850	3645	4240	4870	6360

TABLE 3-8
DISTRIBUTION OF FLOWS FOR INPUT INTO HEC-2 MODEL
EXISTING CONDITIONS - UPPER OLMOS CREEK

HEC-1 Computation Point	Channel Fork	HEC-2 Section	10-Year Flow (cfs)	25-Year Flow (cfs)	50-Year Flow (cfs)	100-Year Flow (cfs)	500-Year Flow (cfs)
OL2	Main	1800	11600	15530	18320	21730	29500
+	Main	8500	11290	15090	17820	20910	28170
+++	Main	9000	11130	14870	17570	20500	27500
OL1 D/S	Main	11200	10970	14650	17320	20080	26840
OL1 U/S	East	11500	6370	8550	10090	11720	15640
+	East	16900	6020	8160	9620	11220	15030
OLE4	East	17000	6010	8150	9610	11210	15020
+++	East	17500	5730	7730	9170	10730	14460
+	East	21000	5510	7390	8810	10340	14000
OLE3	East	24000	5320	7110	8510	10020	13610
+	East	25500	5110	6800	8110	9540	12910
+	East	27000	4900	6510	7730	9080	12260
+	East	28500	4710	6250	7350	8650	11650
+	East	30000	4520	5960	7030	8230	11040
+	East	31000	4400	5790	6800	7970	10660
OLE2	East	32000	4280	5620	6590	7710	10290
+	East	33000	4130	5390	6310	7360	9790
+	East	34000	3980	5180	6050	7030	9310
+	East	35000	3840	4970	5800	6710	8850
+	East	36000	3700	4770	5550	6410	8420
+	East	37000	3560	4570	5320	6120	8010
OLE1	East	37500	3500	4480	5210	5980	7810
OL1 U/S	West	112500	5080	6730	7930	9180	12650
+	West	114500	4910	6450	7580	8770	11960
+	West	116000	4790	6250	7350	8450	11450

TABLE 3-8
DISTRIBUTION OF FLOWS FOR INPUT INTO HEC-2 MODEL
EXISTING CONDITIONS - UPPER OLMOS CREEK
(continued)

HEC-1 Computation Point	Channel Fork	HEC-2 Section	10-Year Flow (cfs)	25-Year Flow (cfs)	50-Year Flow (cfs)	100-Year Flow (cfs)	500-Year Flow (cfs)
+	West	118000	4630	6000	7010	8090	10840
+++	West	118500	4460	5730	6670	7680	10170
OLW2	West	121000	4400	5630	6560	7550	9970
+	West	121500	4210	5390	6280	7230	9530
+	West	122000	4030	5160	6010	6920	9110
+++	West	122100	3640	4670	5430	6250	8210
+	West	122725	3500	4490	5220	6010	7880
+	West	123500	3330	4270	4970	5710	7490
+	West	124500	3130	4010	4660	5360	7020
+	West	125000	3030	3890	4520	5190	6790
+	West	125500	2940	3770	4380	5030	6570
OLW1	West	126000	2850	3650	4240	4870	6360
+	West	12800	2250	2780	3260	3860	4880
++	West	130000	1780	2120	2500	3060	3740
++	West	130500	1410	1690	1980	2430	2980
+	West	132000	1170	1390	1640	2000	2460
++	West	134500	850	1010	1190	1450	1780

\+ Indicates confluence of large tributary
++ Flow value from D.A. vs Q graph
+++ Flow value from semi-log interpolation

Since the HEC-2 cross sections extend upstream of the most upstream combination point on the West Fork, a drainage area vs. flow graph was developed to estimate the flows upstream of the combination point. The drainage area - flow graph for the 10-year and 100-year events is included as Exhibit 3-4. This graph was developed by plotting the flows for each frequency from each subarea and combination point verses the respective drainage areas.

5. Flood Plain Mapping

The water surface elevations generated by the HEC-2 program using the above mentioned flow rates, techniques and parameters are listed in Table 3-9. These water surface elevations were used to develop maps which show the flood plain boundaries for the 10-, 25-, 50-, 100- and 500-year frequency design storm events. Exhibits 3-5 through 3-13 show the existing conditions flood plain boundaries at 1 inch equals 200 feet scale. The base maps are developed from aerial surveys of the area and were developed by United Aerial Mapping during the Fall of 1994.

TABLE 3-9
EXISTING CONDITIONS PEAK FLOWS AND WATER SURFACE ELEVATIONS AT SELECTED LOCATIONS ALONG CHANNEL SYSTEM
UPPER OLMOS CREEK

Section	10-Year		25-Year		50-Year		100-Year		500-Year	
	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)
1800	11200	780.38	12700	780.82	14200	781.24	15800	781.67	20000	782.72
4000	11600	788.27	15530	788.97	18320	789.42	21730	789.90	29500	790.94
6000	11600	798.17	15530	799.07	18320	799.65	21730	800.36	29500	801.60
9000	11130	815.06	14870	815.89	17570	81604.	20500	816.92	27500	817.96
10900	11130	824.45	14870	825.44	17570	826.05	20500	826.64	27500	827.89
11200	10970	825.13	14650	826.12	17320	826.73	20080	827.31	26840	828.54
East Fork Olmos Creek (Elm Creek):										
15500	6370	842.11	8550	843.57	10090	844.47	11720	845.38	15640	846.30
17000	6010	850.78	8150	851.71	9610	852.27	11210	852.82	15020	853.97
19000	5730	859.26	7730	860.27	9170	860.89	10730	861.50	14460	862.74
22000	5510	870.68	7390	871.53	8810	872.09	10340	872.60	14000	873.71
24000	5320	879.00	7110	880.03	8510	880.67	10020	881.29	13610	882.58
26000	5110	885.71	6800	886.72	8110	887.42	9540	888.10	12910	889.42
28000	4900	894.62	6510	895.78	7730	896.53	9080	897.26	12260	898.68
30000	4520	912.01	5960	912.74	7030	913.19	8230	913.65	11040	914.53
32000	4280	920.65	5620	921.65	6590	922.27	7710	922.89	10290	924.13
34000	3980	930.91	5180	931.61	6050	932.10	7030	932.65	9310	933.77
36000	3700	940.69	4770	941.51	5550	942.05	6410	942.57	8420	943.65
37500	3500	949.57	4480	950.25	5210	950.94	5980	951.37	7810	952.26

TABLE 3-9
EXISTING CONDITIONS PEAK FLOWS AND WATER SURFACE ELEVATIONS AT SELECTED LOCATIONS ALONG CHANNEL SYSTEM
UPPER OLMOS CREEK
(continued)

Section	10-Year		25-Year		50-Year		100-Year		500-Year	
	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)
West Fork Olmos Creek:										
112500	5080	828.84	6730	829.59	7930	830.09	9180	830.58	12650	831.74
114600	4910	839.31	6450	840.35	7580	841.03	8770	841.76	11960	843.47
116000	4790	851.89	6250	852.56	7350	852.99	8450	853.40	11450	854.37
118000	4630	861.20	6000	861.82	7010	862.22	8090	862.58	10840	863.43
120000	4460	870.92	5730	871.37	6670	871.68	7680	871.99	10170	872.67
122100	3640	882.36	4670	882.87	5430	883.18	6250	883.50	8210	884.17
123800	3330	889.60	4270	890.03	4970	890.32	5710	890.57	7490	891.05
126000	2850	918.81	3650	919.24	4240	919.52	4870	919.79	6360	920.35
128000	2250	928.10	2780	928.55	3260	928.85	3860	929.17	4880	929.77
130600	1410	931.56	1690	931.89	1980	932.31	2430	932.93	2980	933.59
132000	1170	944.16	1390	944.42	1640	944.66	2000	944.91	2460	945.20
134000	1170	960.76	1390	961.07	1640	961.38	2000	961.74	2460	962.19

C. <u>Verification of New Models</u>

To check the validity of the existing conditions models, verification models of historical flood events were run. The results of the models were then compared to recorded data to determine the accuracy and precision of the developed models.

1. Extraneous Subarea Downstream of Loop 410

In order to compare the HEC-1 model to the U.S.G.S. gage records at Dresden Drive, a modification was incorporated into the model. One subarea was modeled to represent the area contributing to the watershed downstream of Loop 410 and above Dresden Drive (site of the U.S.G.S. gage). This subarea is referenced as L1. This subarea is outside of the limits of the study presented in the project scope and therefore was not studied in detail. The tributary serving this area and the main channel downstream of Loop 410 were not included in the hydraulic modeling described previously; therefore, the channel routing was performed using the kinematic wave method to represent the channel between Loop 410 and Dresden Drive. This method utilizes the channel length, slope, roughness, shape, bottom width and side slopes to perform routing calculations through the reach. The final HEC-1 and HEC-2 models submitted to FEMA for a Physical Map Revision and used by the City of San Antonio for future planning purposes will not include the subarea or channel downstream of Loop IH 410.

2. Hydrologic Conditions

The verification HEC-1 model is identical to the existing condition HEC-1 model with the exception of the rainfall values and the addition of subarea L1 described above. The rainfall values for the verification model are from the April 4, 1991 historical storm. The rainfall information throughout the watershed was collected from the National Weather Service as described in Section 2.0 and a hyetograph was created. Using the rainfall hyetograph, rainfall distributions for each subarea were determined. The hydrograph generated by the verification model at Dresden Drive was very similar in magnitude, timing and volume when compared the historical hydrograph. Table 3-10 lists the comparison between the verification model and the historical data.

TABLE 3-10
APRIL 4, 1991 STORM HYDROGRAPH COMPARISON
UPPER OLMOS CREEK AT DRESDEN DRIVE

Condition	Peak Flowrate (cfs)	Time to Peak	Volume (ac-ft)
U.S.G.S. Gage Measurement	19,400	2:30 a.m.	5157
HEC-1 Model	20,340	2:30 a.m.	5540

3. Hydraulic Conditions

The verification HEC-2 model is identical to the existing condition HEC-2 with the exception of the flowrate used. The verification hydraulic analysis uses flowrates simulated by the HEC-1 model described above for the April 1991 storm. This flowrate was distributed along the channel using the same technique as discussed in the earlier section. The HEC-2 verification model produced an elevation one foot lower relative to the measured high water mark for the same storm at Dreamland Drive (Section number 10000).

3.3 Analysis of Ultimate Conditions

In order to evaluate the effects of full development of the watershed on the recommended Master Drainage Plan, a set of models which simulates ultimate development conditions in the watershed were prepared. Full development was defined using an assumption that current development types in each subarea would expand uniformly into undeveloped areas. Thus, if land use in a subarea currently was distributed as 20% commercial, 20% large-lot residential, and 20% average-lot residential, the remaining 40% undeveloped property was assumed to develop as one-third of each land use type. The final ultimate distribution of land use in the watershed would therefore be 33% commercial, 33% large-lot residential, and 33% average-lot residential.

In reality, each subarea in the Upper Olmos Creek watershed has several exceptions to this general assumption:

- Open areas owned by schools, churches, parks or other permanent inclusive facilities as well as platted green space were assumed to remain undeveloped;
- Corridors along Olmos Creek and its major tributaries defined by the regulatory 100-year flood plain boundary were assumed to remain undeveloped;
- Undeveloped areas within incorporated cities (such as Shavano Park and Castle Hills) were assumed to develop with a land use consistent with other developments in these cities; and,
- Undeveloped areas within the recharge zone of the Edwards Aquifer were limited to future development consistent with SAWS guidelines.

Table 3-11 lists the percentages of each land use type used to define ultimate development conditions in each subarea. Table 3-12 lists the peak flow values predicted by the ultimate conditions HEC-1 model for each design storm frequency. Table 3-13 compares the resulting 100-year and 500-year peak flows and water surface elevations for existing and ultimate development conditions at representative locations along Olmos Creek and the East and West Forks.

As can be seen from the modeling comparisons presented in Table 3-13, the 100 year peak flowrates increased under ultimate conditions less than one percent near Loop 410 to less than three percent in upstream areas. The 500 year peak flowrates for ultimate conditions increased slightly over eight percent near Loop 410, about one-and-a-half percent downstream of the confluence, and about two percent in the two tributaries.

Similarly, the 100 year water surface elevations under ultimate development conditions increase less than one tenth of a foot along most of the channel system. The 500 year water surface elevations for ultimate conditions responded in the same magnitude, with only one reach showing an increase of greater than a tenth of a foot. This minimal response to full development is due to the comparitively high imperviousness of the soils in the watershed, the quick response of the drainage system, and the relatively low percentages of new development projected in the already developed subareas of the watershed.

TABLE 3-11
LAND USE DISTRIBUTION FOR ULTIMATE CONDITIONS
UPPER OLMOS CREEK WATERSHED

Subarea	Area (sq mi)	Land Use Type (Percent of Total Area)				RIMP (%)	Lag (hr)	Raw CN
		Residential	Commercial*	Low Density	Undeveloped			
W1	1.49	90	5	0	5	19.8	0.50	74.2
W2	1.64	60	15	10	15	23.5	0.25	76.7
W3	2.26	50	20	25	5	27.3	0.50	74.8
E1	0.82	85	5	0	10	18.9	0.50	76.7
E2	1.64	70	5	0	15	16.3	0.50	76.7
E3	1.75	40	0	60	0	13.0	0.50	77.0
E4	1.28	25	0	70	5	11.4	0.50	77.0
E5	2.76	0	5	70	25	11.0	0.50	77.0
M1	2.91	45	40	0	15	39.9	0.50	74.9

* "Commercial" includes commercial, industrial and high density residential (apartments, etc)

TABLE 3-12
PEAK FLOW RATES COMPUTED BY THE HEC-1 PROGRAM
ULTIMATE CONDITIONS - UPPER OLMOS CREEK

Location	HEC-1 Computation Point	HEC-2 Section	10-Year Peak Flow (cfs)	25-Year Peak Flow (cfs)	50-Year Peak Flow (cfs)	100-Year Peak Flow (cfs)	500-Year Peak Flow (cfs)
Loop 410	OL2	1800	12068	15968	18767	22233	29917
Dnstr Confluence	OL1 D/S	11200	11444	15077	17749	20479	27239
East Fork:							
Upstream -East	OL1 U/S	11500	6585	8742	10269	11887	15799
Lockhill Selma	OLE4	17000	6206	8327	9791	11384	15189
Upstr. George	OLE3	24000	5517	7310	8704	10207	13776
Dnstr. DeZavala	OLE2	32000	4454	5775	6745	7855	10446
Trib. Confluence	OLE1	37500	3634	4611	5349	6128	7935
West Fork:							
Upstream-West	OL1 U/S	112500	5381	6987	8180	9422	12900
Dnstr Huebner	OLW2	121000	4648	5885	6810	7800	10214
Vulcan Quarry	OLW1	126000	3045	3851	4459	5089	6552

TABLE 3-13
COMPARISON OF EXISTING VS ULTIMATE CONDITIONS
UPPER OLMOS CREEK

Section	Existing 100-Year		Ultimate 100-Year		Existing 500-Year		Ultimate 500-Year	
	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)
1800	15800	781.67	15950	781.70	20000	782.72	21750	783.15
4000	21730	789.90	22230	789.96	29500	790.94	29920	791.09
9000	20500	816.92	20740	816.96	27500	817.96	27640	817.97
10900	20500	826.64	20740	826.68	27500	827.89	27640	827.91
11200	20080	827.31	20480	827.36	26840	828.54	27240	828.56
East Fork Olmos Creek (Elm Creek):								
15500	11720	845.38	11890	845.47	15640	846.30	15800	846.33
17000	11210	852.82	11380	852.88	15020	853.97	15190	854.02
19000	10730	861.50	10910	861.57	14460	862.74	14630	862.79
22000	10340	872.60	10530	872.66	14000	873.71	14170	873.75
26000	9540	888.10	9720	888.17	12910	889.42	13080	889.48
28000	9080	897.26	9260	897.35	12260	898.68	12420	898.74
30000	8230	913.65	8390	913.70	11040	914.53	11200	914.57
32000	7710	922.89	7860	922.96	10290	924.13	10450	924.19
34000	7030	932.65	7180	932.72	9310	933.77	9460	933.82
37500	5980	951.37	6130	951.45	7810	952.26	7940	952.31
West Fork Olmos Creek:								
11250	9180	830.58	9420	830.65	12650	831.74	12900	831.79
11460	8770	841.76	9010	841.90	11960	843.47	12210	843.60
11600	8450	853.40	8700	853.49	11450	854.37	11700	854.44
120000	7680	871.99	7940	872.07	10170	872.67	10420	872.74
122100	6250	883.50	6480	883.57	8210	884.17	8430	884.23
123800	5710	890.57	5940	890.64	7490	891.05	7700	891.10
126000	4870	919.79	5090	919.88	6360	920.35	6550	920.42
128000	3860	929.17	3950	929.25	4880	929.77	4950	929.83
130600	2430	932.93	2430	932.94	2980	933.59	2980	933.61
134000	2000	961.74	2000	961.74	2460	962.19	2460	962.19

3.4 Potential Flooding Areas

As can be seen on the maps shown as Exhibits 3-5 through 3-13, the flood plain boundaries for the less frequent flood events (i.e. 100- and 500-year) show many structures adjacent to the channels as being threatened by potential flood conditions. Approximately 75 structures are mapped within the 100-year flood plain boundary. In some cases, the potential for flooding is less than a foot of water under 100-year flood conditions and may be considered an economic hardship; however, many homes in the Dreamland Oaks subdivision and along Orsinger Road could be inundated by as much as two to seven feet of water.

Flooding of structures greater than a foot deep will not only cause severe economic losses to property, but also may threaten the lives of people either trapped inside the structure or trying to evacuate flooded property. In addition, potential flooding of streets in residential neighborhoods limits evacuation and may prevent emergency rescue personnel from reaching citizens in need of assistance. Flooding of low-water crossings also presents life-threatening conditions as discussed in Section 2.0 of this report. Table 3-14 lists the identified flooding potential for each reach of Upper Olmos Creek and the East and West Forks.

In order to define the flood plain boundary as accurately as possible, field surveys of selected slab elevations of homes identified as potentially in the flood plain were performed as discussed previously. These elevations were used to identify potential structural flooding so that homeowners could be aware of the status of their property. If the maps developed for this study are submitted and adopted as a Physical Map Revision to the FEMA Flood Insurance Study (FIS) for the City of San Antonio and Bexar County, they would become the new FIS Flood Insurance Rate Maps (FIRM) for the area.

Homeowners with structures shown within the 100-year regulatory flood plain would be eligible for federally supplemented flood insurance. Perhaps more importantly, the City of San Antonio, Bexar County, and other jurisdictional municipalities or agencies will have a more complete and accurate understanding of the nature of potential flooding along Upper Olmos Creek. This increased knowledge will allow them to make planning decisions which will mitigate or eliminate the potential for flooding in this area. To that end, a Master Drainage Plan for Upper Omos Creek was developed as described in the remainder of this report.

TABLE 3-14
EXISTING CONDITIONS 100-YEAR FLOOD PLAIN
UPPER OLMOS CREEK

Reach	General Location	Description of Flooding
Main Channel:		
0-1800	DS Loop 410	Not in Limits of Detailed Study
1800-9000	Castle Hills/ San Antonio	2 commercial structures near Loop 410 and 3 houses at intersection Old Brook and Oak Downs (S.A.) in flood plain
9000-10900	Dreamland Oaks	38 houses > 1ft to 7ft deep in flood plain.
10900-11500	Confluence	No apparent structures in flood plain.
East Fork:		
11500-15500	Confluence to Wurzbach Road	3 apartment buildings and 2 houses in flood plain. Whisper Willow (11200 block) subject to street flooding.
15500-22000	Wurzbach to George	One house near Sect. 21000 (Downstream of George Rd.) in flood plain.
22000-27000	Hunters Creek - George to Huebner	2 trailers/storage units and 2 houses in flood plain. Hunters Circle and Hunters View subject to street flooding.
27000-31000	Shavano Park	No apparent structures in flood plain.
31000-37000	Shavano Park	9 houses in flood plain < 2 ft deep.
West Fork:		
11200-114600	Confluence to Wurzbach Rd	5 houses in flood plain < 2 ft deep. Mossbank, Oakbank and Quail Meadow subject to street flooding
114600-122000	Wurzbach to Orsinger	No apparent structures in flood plain.
122000-123800	Orsinger to Huebner	9 structures (houses/outbuildings) in flood plain.
123800-134500	US Huebner	No apparent structures in flood plain.

3.5 Selection of Mitigation Alternatives

In order to address the potential for flooding the areas identified in Exhibits 3-5 through 3-13, a number of structural and nonstructural projects were identified which could lessen flows in the channel, provide more capacity in the channel, or remove the structures themselves from the flood plain. Each project was evaluated independently in order to determine the sensitivity of the Olmos Creek drainage system to the project. Several of the most beneficial projects were then selected as components for four alternative flood protection plans. The evaluation of the components and the development of the alternatives are described in more detail in the following paragraphs.

A. Evaluation of Flood Control Components

A wide variety of potential flood control projects were initially identified as possible components of a comprehensive flood mitigation plan for Upper Olmos Creek. Projects considered included traditional flood control structural projects, such as detention facilities and channel improvements, to less traditional non-structural alternatives such as local buy-out of properties within the 100-year flood plain. Each component was considered on its own merit and was evaluated with the computer models to determine its effectiveness in the Upper Olmos Creek drainage system.

1. Texas Department of Transportation (TxDOT) Project at Loop 410: Phase 1 and 2

The two phased TxDOT project at Loop 410 involves the downstream enlargement of the Olmos Creek channel from Loop 410 to the existing lined channel near Jackson Keller road (Phase 1) and the enlargemnt of the culverts under the West Avenue - Loop 410 interchange (Phase 2). This project was described earlier in Section 2.

2. Vulcan Quarry

The Vulcan Materials Company quarry adjacent to the West Fork of the Olmos Creek upstream of Huebner Road was modeled as a regional detention facility during the study. All flows in the West Fork channel were diverted into the excavated portion of the quarry via a weir/channel system located approximately 2000 feet upstream of Huebner Road. If selected as part of the Master Drainage Plan, an engineering design would be developed to channelize the flows to enter the southern, older quarry site first and then be allowed to overflow into the second, newer quarry site. The runoff water will then be allowed to recharge into the underlying aquifer.

3. Channel Clearing

Several variations on clearing the main channels of vegetation were analyzed. One method involves clearing the channel of all vegetation and replanting with grass. A second less severe method involves clearing the channel of brush, debris and trees less than three to four inches in diameter and trimming the lower branches of larger trees to a height of five to six feet above the ground. This method can be visualized as an area with trees but enough clearing to allow the throwing of a frisbee in an unobstructed understory space.

4. Levee Systems

Construction of flood protection levees were evaluated at several critical locations along the channel system. The largest of these projects was analysed for the Dreamland Oaks subdivision. A levee was designed to protect the homes in the subdivision from the 100-year design storm event. The levee was located between the homes and the creek and was designed to be ten to twelve feet high. The levee was configured with an interior swale and storm sewer drainage system which would gravity outfall approximately 2000 feet downstream of the levee. Approximately 16 homes would have to be bought and removed in order to accomodate the levee structure, swale and drainage system.

5. Regional Detention Facility near Shavano Park

A detention area was evaluated at the confluence of several tributaries on the East Fork downstream of Loop 1604 within the City of Shavano Park and Bexar County. Three configurations of the facility were analysed as potential drainage plan components. All of the designs involve the construction of a dam across the East Fork immediately downstream of the confluence of the tributaries apprioximately 3000 feet downstream of Loop 1604. The first two configurations provide approximately 400 acre-feet of detention storage during the 100-year design storm event on 55 acres of undeveloped property.

The first configuration utilizes the existing topography to provide the detention storage. The dam is supplemented by a levee which protects homes in Shavano Park while providing additional capacity in the detention facility. Two to three properties in the City would be required to provide land from their existing backyards to accomodate the levee structure. Vegetation is left intact in its existing condition except immediately adjacent to the control structure. Although not currently identified, aquifer recharge features may be naturally present on the site.

The second design for this facility involves the excavation of the detention site to accomodate the 400 acre-feet of storage without the levee structure. Most of the excavation would occur around the perimeter of the site, with much of the existing vegetation remaining intact. Recharge features discovered during excavation could be protected and enhanced.

A third option for the facility was analyzed which expanded the excavation to its maximum configuration on the 55-acre site. Approximately 900 acre-feet of storage were designed into the reservoir through the use of extensive excavation. No levee was involved in this design, although vegetation would have to be removed or replanted after construction was completed. The leveling of the bottom of the reservoir could encourage the use of the site as a multi-use recreational facility (i.e.

ball fields, tennis courts, parks, etc.). Recharge features identified during construction could be protected and enhanced.

6. Regional Detention Facility near Lockhill Selma Road

A second site on the East Fork was identified for regional stormwater detention immediately upstream of Lockhill Selma Road. Two options were analyzed for this 45 acre site, with both including the construction of an all-weather crossing at Lockhill Selma which would function as the control structure for the facility.

The first configuration of the detention facility was designed to store approximately 350 acre-feet of runoff within the existing topography. The road was raised and contained multiple culverts which allow the more frequent flood events to pass through while constricting the larger events and providing detention storage. Existing vegetation would remain intact or be groomed/managed by adjacent subdivisions.

The second configuration of the facility provided for the maximum volume of storage capacity of over 800 acre-feet by excavating the site into a reservoir with a level bottom and 3:1 side slopes. Vegetation or grass would be planted after construction was complete. The detention reservoir could be developed into a multi-use facility with the addition of recreational amenities (i.e., ball fields, tennis courts, parks, etc.).

7. Channel Realignment

A portion of the West Fork meanders back and forth across a railroad line between Wurzbach and Orsinger Roads. A design for the realignment of the channel between stations 114600 and 122000 was modeled in order to determine the effectiveness of confining the majority of the flow down the channel to the east of the railroad. Two crossings of the channel under the railroad were eliminated, although flows still overtopped the railroad during large rainfall events.

8. Channel Enlargement

The Olmos Creek channel downstream of Dreamland Drive was enlarged to accomodate the 100-year flows within the banks. Sideslopes were designed with a 3:1 ratio and would be planted with grass. Regular mowing was assumed for the analysis. Limited channel enlargements or improvements were also considered in several reaches on the East Fork.

9. Elimination of Low Water Crossings

The low-water crossings at Dreamland Drive and Lockhill Selma would be eliminated by raising the roads to the level of the 100-year flood plain. This was evaluated through both fill/culvert structures and bridge structures.

10. Buy-out of Structures in Flood Plain

The potential for a buy-out by the City and other entities of the structures which are within the design 100-year flood plain was considered as a nonstructural project which eliminates the flooding problem under most severe storm conditions. The structures would be bought, demolished, and the sites either converted into open-space (parks or green space) or allowed to be purchased and developed with raised structures (the latter alternative would only be feasible in areas with shallow flooding).

Each component described above was analyzed utilizing the models developed for the study. The results of the change in water surface for each of the components throughout the channel reach are shown schematically on Exhibits 3-14 and 3-15.

Several components were eliminated from further study because they were ineffective in reducing flooding or aggravated flooding conditions in adjacent areas. The TxDOT channel improvements eliminate flooding of the Loop 410 frontage roads and West Avenue interchange and remove adjacent commercial properties from the flood plain. While having little impact on flooding upstream of West Avenue, this project became a component of every alternative included in the study.

Both the Vulcan Detention Facility and the "frisbee" type channel clearing were included in one or more of the alternatives described below. Implementation of the Vulcan Detention Facility removes all structures on the West Fork from the 100-year flood plain except nine homes and/or outbuildings near Orsinger Road. It also may have an added benefit of reducing flow velocities during all runoff events so that erosion is slowed in critical areas along the West Fork.

None of the independent levee systems studied were deemed appropriate for consideration as a component of an alternative. The levee system considered for Dreamland Oaks only removed 21 homes from the flood plain while destroying 16 others. The structure was not considered cost-beneficial since it would be costly to construct, would adversely impact the aesthetics of the neighborhood, and would require significant ongoing maintenance by the City and the homeowners association. In addition, the levee constricted the Olmos Creek channel and caused the water surface elevation to rise directly across from and upstream of Dreamland Oaks, forcing at least five more homes into the flood plain.

Both regional detention facilities on the East Fork were modeled as components of several alternatives described below. The first two configurations of the Shavano Park facility were analyzed in the alternatives as well as the first configuration of the Lockhill Selma facility (an adaptation of the excavation configuration was also included in an alternative).

The channel realignment did not prove effective in reducing the 100-year flood plain and was not included in an alternative. The channel improvement projects were only effective in the main channel of Olmos Creek downstream of Dreamland Drive. The channel enlargement project in this reach was included as a component in an alternative. In addition, all four alternatives included a component which involved the buy-out of structures in the flood plain.

B. Development of Mitigation Alternatives

Four independent flood mitigation alternatives were developed and evaluated with respect to the following selection criteria:

- Effectiveness in reducing the potential for structural flooding,
- Benefit to the community in terms of reduction of dollar damages during flood events,
- Less tangible benefits such as the reduction of street flooding and channel erosion,
- Elimination of flood hazards such as low-water crossings,
- Ability to provide a ""buffer zone" along the major channels which increases the level of comfort of residents,
- Enhancement of the watershed through multiple-use facilities, and
- Preservation of natural habitat.

Although as many as ten variations on the alternative packages were initially considered, four scenarios were selected for detailed modeling and cost comparisons.

1. Scenario A

Scenario A was developed with provisions for all three regional detention facilities described previously and limited channel clearing. This alternative includes both phases of the TxDOT channel improvements downstream of West Avenue, the Vulcan Quarry detention facility, channel clearing using the "frisbee" option from the end of the TxDOT improvements to Dreamland Drive, the Shavano Park detention facility, the Lockhill Selma detention facility (with the raising of Lockhill Selma Road), the raising of Dreamland Drive, and the buy-out of all structures remaining in the residual 100-year flood plain. Table 3-15 lists the components included in the scenario and provides a generalized cost estimate for the alternative.

The Shavano Park detention facility was evaluated for two options: Option 1 - the facility is constructed through excavating the perimeter of the 55 acre site to

TABLE 3-15
PRELIMINARY COST ESTIMATE - SCENARIO A
UPPER OLMOS CREEK

DESIGN COMPONENTS	UNIT	UNIT COST	NUMBER OF UNITS	CONSTRUCTION COST PER COMPONENT	LAND COST PER COMPONENT	TOTAL COST PER SUBCOMPONENT
1. TxDOT Phase 1 and 2 (Paid by TxDOT)						$0
2. Vulcan Quarry						
Land					$850,000	
Channel Diversion/Misc.				$750,000		
Total Costruction Costs				$750,000		
Mobilization (11%)				$82,500		
Preparation of ROW (4%)				$30,000		
Subtotal				$862,500		
Contingencies (10%)				$86,250		
Engineering (11%)				$94,875		
Administration (7%)				$60,375		
Stormwater Pollution Control (5%)				$43,125		
TOTAL				$1,147,125	$850,000	$1,997,125
3. Channel Clearing, Station 2800 to 11200 (L = 8400 FT, W = 300 FT)	AC	$2,500	58	$145,000		
Total Costruction Costs				*$145,000*		
Mobilization (11%)				$15,950		
Preparation of ROW (4%)				$5,800		
Subtotal				$166,750		
Contingencies (10%)				$16,675		
Engineering (11%)				$18,343		
Administration (7%)				$11,673		
Stormwater Pollution Control (5%)				$8,338		
TOTAL				$221,778	$0	$221,778
4. Shavano Park Detention (400 ac-ft)						
Land	AC	$27,000	55		$1,500,000	
Fully Excavate	CY	$5	500,000	$2,500,000		
Minimum Structure (Max Elev = 956 ft)	EA	$50,000	1	$50,000		
Minimum Levee	EA	$20,000	1	$20,000		
Topsoil (D=3 in.)	AC	$3,250	30	$97,500		
Seeding	AC	$1,200	30	$36,000		
Misc.				$50,000		
Total Costruction Costs				*$2,753,500*		
Mobilization (11%)				$302,885		
Preparation of ROW (4%)				$110,140		
Subtotal				$3,166,525		
Contingencies (10%)				$316,653		
Engineering (11%)				$348,318		
Administration (7%)				$221,657		
Stormwater Pollution Control (5%)				$158,326		
TOTAL				$4,211,478	$1,500,000	$5,711,478
5. Lockhill Selma Detention						
Land	AC	$20,000	45		$900,000	
Road (L = 2000', W = 60')	SF	$5	120,000	$600,000		
Road Fill (Ave D = 10', W = 60')	CY	$4	44,500	$178,000		
Culverts (5 - 11'x11', L = 60')	FT	$655	300	$196,500		
Misc. (including downsream energy dissipators)				$75,000		
Total Costruction Costs				*$1,049,500*		
Mobilization (11%)				$115,445		
Preparation of ROW (4%)				$41,980		
Subtotal				$1,206,925		
Contingencies (10%)				$120,693		
Engineering (11%)				$132,762		
Administration (7%)				$84,485		
Stormwater Pollution Control (5%)				$60,346		
TOTAL				$1,605,210	$900,000	$2,505,210
6. Dreamland Bridge (L = 700', W 50')	SF	$50	35,000	$1,750,000		
Total Costruction Costs				*$1,750,000*		
Mobilization (11%)				$192,500		
Preparation of ROW (4%)				$70,000		
Subtotal				$2,012,500		
Contingencies (10%)				$201,250		
Engineering (11%)				$221,375		
Administration (7%)				$140,875		
Stormwater Pollution Control (5%)				$100,625		
TOTAL				$2,676,625	$0	$2,676,625
7. Buy-out Remaining Houses in 100-year Floodplain						
Main Channel	EA	$100,000	29		$2,900,000	
West Fork	EA	$50,000	9		$450,000	
East Fork	EA	$50,000	2		$100,000	
East Fork	EA	$100,000	1		$100,000	
East Fork	EA	$275,000	1		$275,000	
TOTAL				$0	$3,825,000	$3,825,000
TOTAL SCENARIO A						$16,937,216

accommodate approximately 400 acre-feet of storage (the dam structure is minimized); Option 2 - the facility is constructed with a larger control structure and levee system, with minimal excavation (restricted to amount necessary to provide fill for the levee). The Lockhill Selma detention facility was also analyzed for two options: Option 1 - if constructed in conjunction with or after the Shavano Park facility, the Lockhill Selma facility can provide storage without excavation by using the existing topography and detaining flows through the use of a constriction under the raised Lockhill Selma Road; Option 2 - if constructed prior to the Shavano Park facility, the limited available freeboard between the existing 100-year flood plain and houses along the channel does not allow for additional backwater behind Lockhill Selma Road; therefore, culverts under the raised road would have to be added and excavation of the site north of and along the channel bottom would be necessary in order to accommodate the flows. Based on discussions with the City of San Antonio staff, Option 2 did not appear to be feasible and was not included in the cost estimates presented in Table 3-15.

Dreamland Drive was included in this alternative as a bridge which would span the channel and railroad tracks and provide an all-weather crossing. A bridge structure was necessary because of the configuration of the railroad crossing and the desire to eliminate as much head loss as possible through the structure.

Approximately 42 structures would remain in the 100-year flood plain after construction of the components of Scenario A. Most of these structures are concentrated in the Orsinger Road area on the West Fork and in the Dreamland Oaks subdivision on Olmos Creek. Both of these areas are characterized by homes and outbuildings located within the high banks of the channel. Although further detailed surveying of slab elevations would verify the elevations of structures located close to the flood plain boundary, for study purposes all 42 structures were assumed to be purchased by the City.

2. Scenario B

Scenario B was developed with many of the same components as Scenario A and is presented in Table 3-16. The TxDOT improvements, the Vulcan Quarry detention facility, the Shavano Park detention facility (Option 1), the Lockhill Selma detention facility (Option 1), and the Dreamland Drive bridge are included in Scenario B. In addition, a channel enlargement component is included in Scenario B in order to remove all of the homes in Dreamland Oaks subdivision from the 100-year flood plain. This project would involve construction of approximately 8400 feet of enlarged channel from the upstream end of the TxDOT channel improvements near West Avenue to just upstream of Dreamland Drive. The top width of the new channel would average approximately 200 feet. The channel would be designed with 3:1 side slopes and grass lining (dependent upon geological testing of the soils). Erosion controls may be included in the final design but were not included in the cost estimate prepared for the study. Approximately 13 structures remaining in the residual flood plain on the East and West Forks are included as a buy-out component in Scenario B.

TABLE 3-16
PRELIMINARY COST ESTIMATE - SCENARIO B
UPPER OLMOS CREEK

DESIGN COMPONENTS	UNIT	UNIT COST	NUMBER OF UNITS	CONSTRUCTION COST PER COMPONENT	LAND COST PER COMPONENT	TOTAL COST PER COMPONENT
1. TxDOT Phase 1 and 2 (Paid by TxDOT)						$0
2. Vulcan Quarry (Same as Scenario A)						$1,997,125
3. Channel Enlargement, Sta. 2800 to 11200						
(L = 8400 FT, W = 200 FT)						
Excavation/Disposal of Material	CY	$6	850,000	$5,100,000		
Right of Way	AC	$10,000	39		$390,000	
Replace RR Bridge Sta 2800	EA	$100,000	1	$100,000		
Replace RR Bridge Sta 11500	EA	$100,000	1	$100,000		
Misc. (Utilities, Fences, etc.)				$200,000		
Total Costruction Costs				*$5,500,000*		
Mobilization (11%)				$605,000		
Preparation of ROW (4%)				$220,000		
Subtotal				$6,325,000		
Contingencies (10%)				$632,500		
Engineering (11%)				$695,750		
Administration (7%)				$442,750		
Stormwater Pollution Control (5%)				$316,250		
TOTAL				$8,412,250	$390,000	$8,802,250
4. Shavano Park Det. (Same as Scenario A)						$5,711,478
5. Lockhill Selma Det. (Same as Scenario A)						$2,505,210
6. Dreamland Bridge (Same as Scenario A)						$2,676,625
7. Buy-out Remaining Houses in 100-year						
Floodplain						
Main Channel	EA	$100,000	0		$0	
West Fork	EA	$50,000	9		$450,000	
East Fork	EA	$50,000	2		$100,000	
East Fork	EA	$100,000	1		$100,000	
East Fork	EA	$275,000	1		$275,000	
TOTAL				$0	$925,000	$925,000
TOTAL SCENARIO B						$22,617,688

3. Scenario C

Scenario C is shown in Table 3-17 and is an adaptation of Scenario A which includes the TxDOT channel improvements, the Vulcan Quarry detention facility, the channel clearing in Olmos Creek and the Dreamland Drive bridge. The Shavano Park detention facility is expanded to include approximately 900 acre-feet of storage through extensive excavation of the 55 acre site. The Lockhill Selma detention facility is also expanded to include approximately 850 acre-feet of storage through extensive excavation of the 45 acre site. Neither of these detention facilities maintains natural habitat, although either could be replanted as a wildlife preserve or used as a recreational park facility. Approximately 41 structures remain in the residual flood plain and for study purposes would be assumed to be purchased the City. Only one house which was left in the flood plain under Scenario A is removed from the flood plain by Scenario C.

TABLE 3-17
PRELIMINARY COST ESTIMATE - SCENARIO C
UPPER OLMOS CREEK

DESIGN COMPONENTS	UNIT	UNIT COST	NUMBER OF UNITS	CONSTRUCTION COST PER COMPONENT	LAND COST PER COMPONENT	TOTAL COST PER COMPONENT
1. TxDOT Phase 1 and 2 (Paid by TxDOT)						$0
2. Vulcan Quarry (Same as Scenario A)						$1,997,125
3. Channel Clearing (Same as Scenario A)						$221,778
4. Shavano Park Detention (900 ac-ft)						
Land	AC	$27,000	55		$1,500,000	
Excavation	CY	$5	937,500	$4,687,500		
Structure	EA	$60,000	1	$60,000		
(Max Elev = 966 ft)						
Topsoil (D=3 in.)	AC	$3,250	55	$178,750		
Seeding	AC	$1,200	55	$66,000		
Dam	EA	$120,000	1	$120,000		
Misc.				$50,000		
Total Costruction Costs				*$5,162,250*		
Mobilization (11%)				$567,848		
Preparation of ROW (4%)				$206,490		
Subtotal				$5,936,588		
Contingencies (10%)				$593,659		
Engineering (11%)				$653,025		
Administration (7%)				$415,561		
Stormwater Pollution Control (5%)				$296,829		
TOTAL				$7,895,661	$1,500,000	**$9,395,661**
5. Lockhill Selma Detention (840 ac-ft)						
Land	AC	$20,000	45		$900,000	
Excavation	CY	$5	820,000	$4,100,000		
Road (L = 2000', W = 60')	SF	$5	120,000	$600,000		
Road Fill (Ave D = 10', W = 100')	CY	$2	74,000	$111,000		
Topsoil (D=3 in.)	AC	$3,250	45	$146,250		
Seeding	AC	$1,200	45	$54,000		
Culverts (3 - 11'x11', L = 100')	FT	$500	300	$150,000		
Misc. (incl. energy dissipators)				$75,000		
Total Costruction Costs				*$5,236,250*		
Mobilization (11%)				$575,988		
Preparation of ROW (4%)				$209,450		
Subtotal				$6,021,688		
Contingencies (10%)				$602,169		
Engineering (11%)				$662,386		
Administration (7%)				$421,518		
Stormwater Pollution Control (5%)				$301,084		
				$8,008,844	$900,000	**$8,908,844**
6. Dreamland Bridge (Same as Scenario A)						**$2,676,625**
7. Buy-out Remaining Houses in 100-year Floodplain						
Main Channel	EA	$100,000	29		$2,900,000	
West Fork	EA	$50,000	9		$450,000	
East Fork	EA	$50,000	2		$100,000	
East Fork	EA	$100,000	1		$100,000	
TOTAL				$0	$3,550,000	**$3,550,000**
TOTAL SCENARIO C						**$26,750,034**

4. Scenario D

Scenario D closely resembles a "do nothing" alternative which minimizes the engineering components of the scenario. This alternative includes the TxDOT channel improvements and the Dreamland Bridge and adds a bridge at Lockhill Selma Road in order to eliminate the hazardous low-water crossings at these locations. No other structural improvements or detention facilities are included in Scenario D. The flood plain mitigation is achieved through a massive buy-out program which includes the purchase of all 73 structures remaining in the 100-year flood plain. Legal costs of condemnation or court challenges were not included in the cost estimate provided in Table 3-18 but are likely to be substantial. Participation by the City of Shavano Park in the buy-out of homes within its jurisdiction would be necessary. Less tangible watershed problems such as street flooding, channel erosion and the threat of flooding on property surrounding homes are not addressed by this scenario.

TABLE 3-18
PRELIMINARY COST ESTIMATE - SCENARIO D
UPPER OLMOS CREEK

DESIGN COMPONENTS	UNIT	UNIT COST	NUMBER OF UNITS	CONSTRUCTED COST PER COMPONENT	LAND COST PER COMPONENT	TOTAL COST PER COMPONENT
1. TxDOT Phase 1 and 2 (Paid by TxDOT)						$0
2. Lockhill Selma Road/Bridge						
Road (L = 850', W = 60')	SF	$5	51,000	$255,000		
Road Fill (Ave D = 10', W = 60')	CY	$4	19,000	$76,000		
Bridge (L = 1150', W = 50')	SF	$50	57,500	$2,875,000		
Misc.				$50,000		
Total Costruction Costs				*$3,256,000*		
Mobilization (11%)				$358,160		
Preparation of ROW (4%)				$130,240		
				$3,744,400		
Contingencies (10%)				$374,440		
Engineering (11%)				$411,884		
Administration (7%)				$262,108		
Stormwater Pollution Control (5%)				$187,220		
				$4,980,052	$0	$4,980,052
3. Dreamland Bridge (Same as Scenario A)						$2,676,625
4. Buy-out Remaining Houses in 100-year Floodplain						
Main Channel	EA	$100,000	40		$4,000,000	
West Fork	EA	$50,000	9		$450,000	
West Fork	EA	$100,000	5		$500,000	
East Fork	EA	$50,000	2		$100,000	
East Fork	EA	$200,000	2		$400,000	
East Fork	EA	$100,000	3		$300,000	
East Fork	EA	$100,000	2		$200,000	
East Fork	EA	$100,000	1		$100,000	
East Fork	EA	$275,000	9		$2,475,000	
Subtotal					*$8,525,000*	
Estimated Legal Fees (20%)					$1,705,000	
				$0	$10,230,000	$10,230,000
TOTAL SCENARIO D						$17,886,677

3.6 Recommended Master Drainage Plan

A presentation of the four scenarios was made to the City staff and the Drainage Criteria Review Committee on August 28 and 29, 1995. At this time, a recommendation was made to adopt Scenario A as the Master Drainage Plan for Upper Olmos Creek for the reasons outlined below. Exhibit 16 shows the locations of each component contained in Scenario A.

Scenario A is estimated to cost about $5.7 million less than Scenario B. This indicates that the channel project along Olmos Creek included in Scenario B would only add the removal of 29 houses in Dreamland Oaks (estimated value of $2,900,000) as a benefit over Scenario A and therefore Scenario B does not appear to be cost effective. Similarly, Scenario A costs about $9.8 million less than Scenario C. Even with maximum use of the detention reservoir sites, Scenario C only removes one additional house in Shavano Park when compared to Scenario A. Again, Scenario C does not appear to be cost effective.

Scenario D, which includes the buy-out of all of the structures shown within the 100-year flood plain boundary on the existing conditions flood plain maps (with the exception of the commercial properties near Loop 410), is estimated to cost about $1 million more than Scenario A. However, certain costs, ie. legal fees associated with condemnation and buy-out, have been estimated from much smaller scale voluntary buy-out projects and may be significantly more than the 20 percent estimated in Table 3-18. As discussed previously, this scenario does not address related issues such as street flooding, property flooding, erosion, etc.

In addition to the cost analysis described above, Scenario A satisfies the selection criteria more completely than any of the other alternatives developed for the study:

- Effectiveness in reducing structural flooding: Scenario A removes 33 structures from the effective 100-year flood. Under Scenario A it is assumed that the 42 structures remaining in the 100-year flood plain would be bought out by the City.
- Reduction of dollar damages from flood events: Scenario A eliminates damages to structures during the 100-year and lesser storm events. The scenario also eliminates most street flooding and confines most of the flood waters to the main channels, thus reducing damages to streets, landscaping, automobiles parked along streets, and other previously threatened property. Implementation of this scenario would also reduce current community costs associated with high water rescues from low-water street crossings and from flooded homes.
- Reduction of street flooding and channel erosion: Access to homes and businesses along the channel would be enhanced by Scenario A. The scenario reduces the over bank flood plain and therefore eliminates long-term street flooding adjacent to the channel in most areas. By using detention to reduce the amount of runoff reaching the channel and to slow flood waters in the channel, the progression of ongoing erosion problems along the West Fork would be retarded.

- Elimination of flood hazards: The most serious flood hazard in the Upper Olmos Creek watershed occurs at low-water crossings of major streets. Both Dreamland Drive and Lockhill Selma Road are raised and converted to all weather crossings in Scenario A.
- Provision of a "buffer" along the main channels: Scenario A greatly increases the level of comfort experienced by residents along Olmos Creek and the West and East Forks by lowering the 100-year flood water surface elevation. Under current conditions, in many locations homes are actually surrounded by flood waters on adjacent property and streets and are only raised above the flood waters by the thickness of the structure's slab. In other locations, the flood plain boundary was mapped around a structure based on the highest elevation adjacent to the slab (the assumption was made that the slab would be constructed level with the highest ground surface on the pad site - the possibility of split-level homes following the ground elevation downward was not considered in the mapping procedure). With the components of Scenario A in place, the 100-year water surface is lowered and the flood plain boundary would be moved away from most of these structures. thereby providing a buffer zone around many structures.
- Enhancement of the watershed through multiple-use facilities: All three of the detention facilities included in Scenario A could provide multiple uses to the community. The Vulcan Quarry site is a significant recharge feature for the underlying aquifer. In addition, unexcavated areas which currently are used as roads and office/parking areas will be cleared and converted to City park facilities. The Shavano Park detention area is within the Edwards Aquifer Recharge Zone and may have naturally occurring recharge features within the detention site. By minimizing the amount of excavation used in the design of the facility (either Option 1 or 2), most of this area can be left undisturbed and preserved as wildlife habitat. Similarly, the Lockhill Selma detention area can function as a detention facility as well as either a natural preserve or a park area. If maintenance is assumed by adjacent neighborhoods, the vegetation may be manicured and recreational facilities such as park benches/tables, jogging paths, etc. may be added. In addition, bicycle or hiking trails could possibly be incorporated into the main channel from Loop 410 to Dreamland Drive during the channel clearing project to maximize use of the Olmos Creek stream corridor. The purchase and demolition of groups of threatened homes in Dreamland Oaks and near Orsinger Road would allow the conversion of the flood plain property to park facilities or open space.
- Preservation of natural habitat: As discussed above, the natural vegetation in both the Shavano Park and Lockhill Selma detention facilities can be left mostly intact. In addition, the upstream detention facilities included in Scenario A detain enough flood runoff so that channel improvement projects on the East and West Forks are not necessary. By limiting channel clearing to the Olmos Creek channel downstream of Dreamland Drive, most of the stream corridors along the East and West Forks are left in their natural state.

The Committee and the City of San Antonio engineering staff agreed to the recommendation of Scenario A as the Master Drainage Plan for Upper Olmos Creek, pending the solicitation of public comment. A public meeting was held on November 15, 1995, during which the study and the selection of the Master Drainage Plan were reviewed in detail. Based on the comments received during the meeting, Scenario A is this study's recommended Master Drainage Plan for Upper Olmos Creek.

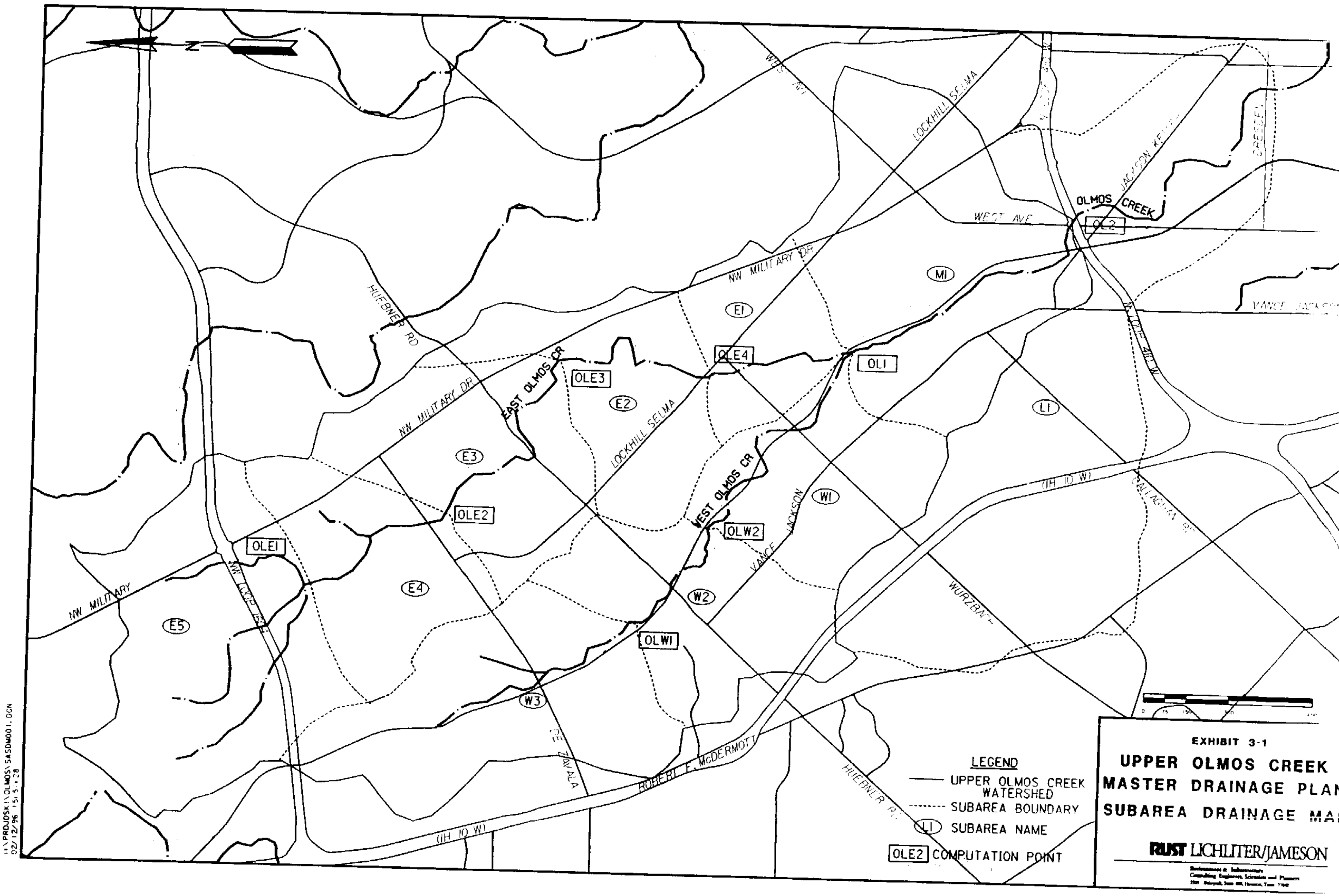
EXHIBIT 3-1
UPPER OLMOS CREEK
MASTER DRAINAGE PLAN
SUBAREA DRAINAGE MAP
RUST LICHLITER/JAMESON
LEGEND
UPPER OLMOS CREEK WATERSHED
SUBAREA BOUNDARY
L1 SUBAREA NAME
OLE2 COMPUTATION POINT
OLMOS CREEK
OL2
WEST AVE
NW MILITARY DR
LOCKHILL SELMA
HUEBNER RD
EAST OLMOS CR
WEST OLMOS CR
VANCE JACKSON
WURZBACH
(IH 10 W)
ROBERT E. McDERMOTT
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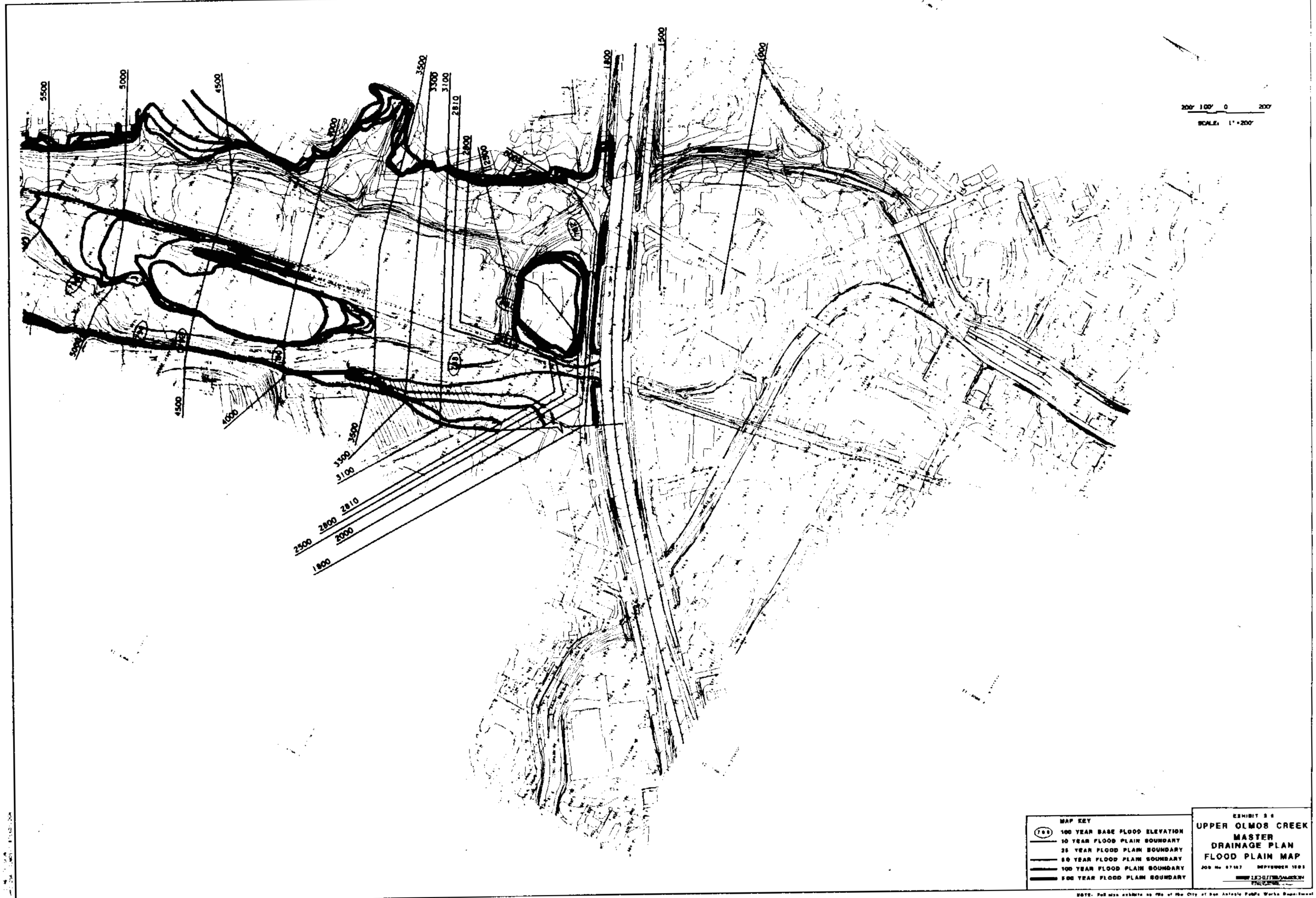
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10 YEAR FLOOD PLAIN BOUNDARY
25 YEAR FLOOD PLAIN BOUNDARY
50 YEAR FLOOD PLAIN BOUNDARY
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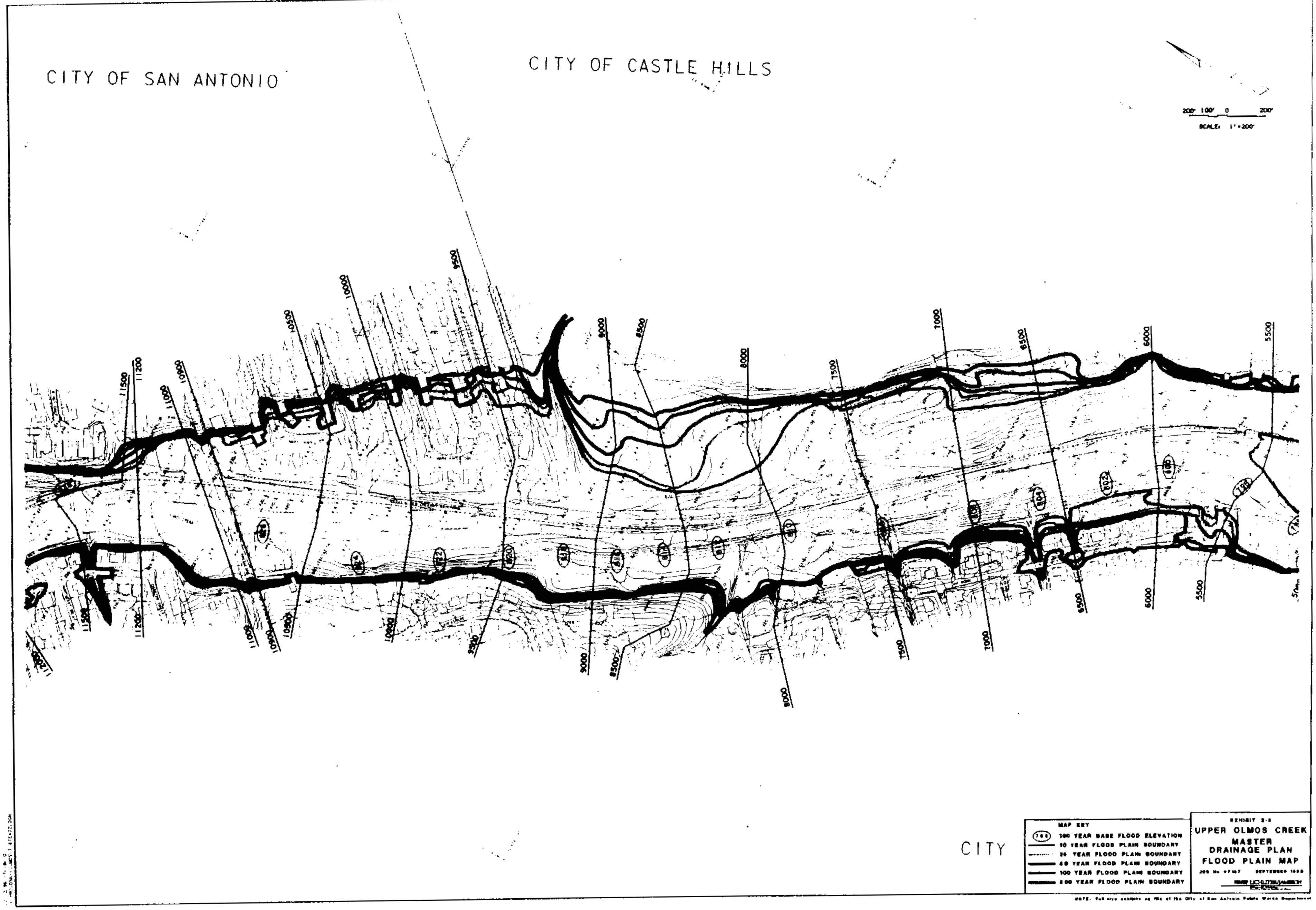
500 YEAR FLOOD PLAIN BOUNDARY
UPPER OLMOS CREEK
MASTER
DRAINAGE PLAN
FLOOD PLAIN MAP



CITY OF SAN ANTONIO
CITY OF CASTLE HILLS
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UPPER OLMOS CREEK
MASTER
DRAINAGE PLAN
FLOOD PLAIN MAP

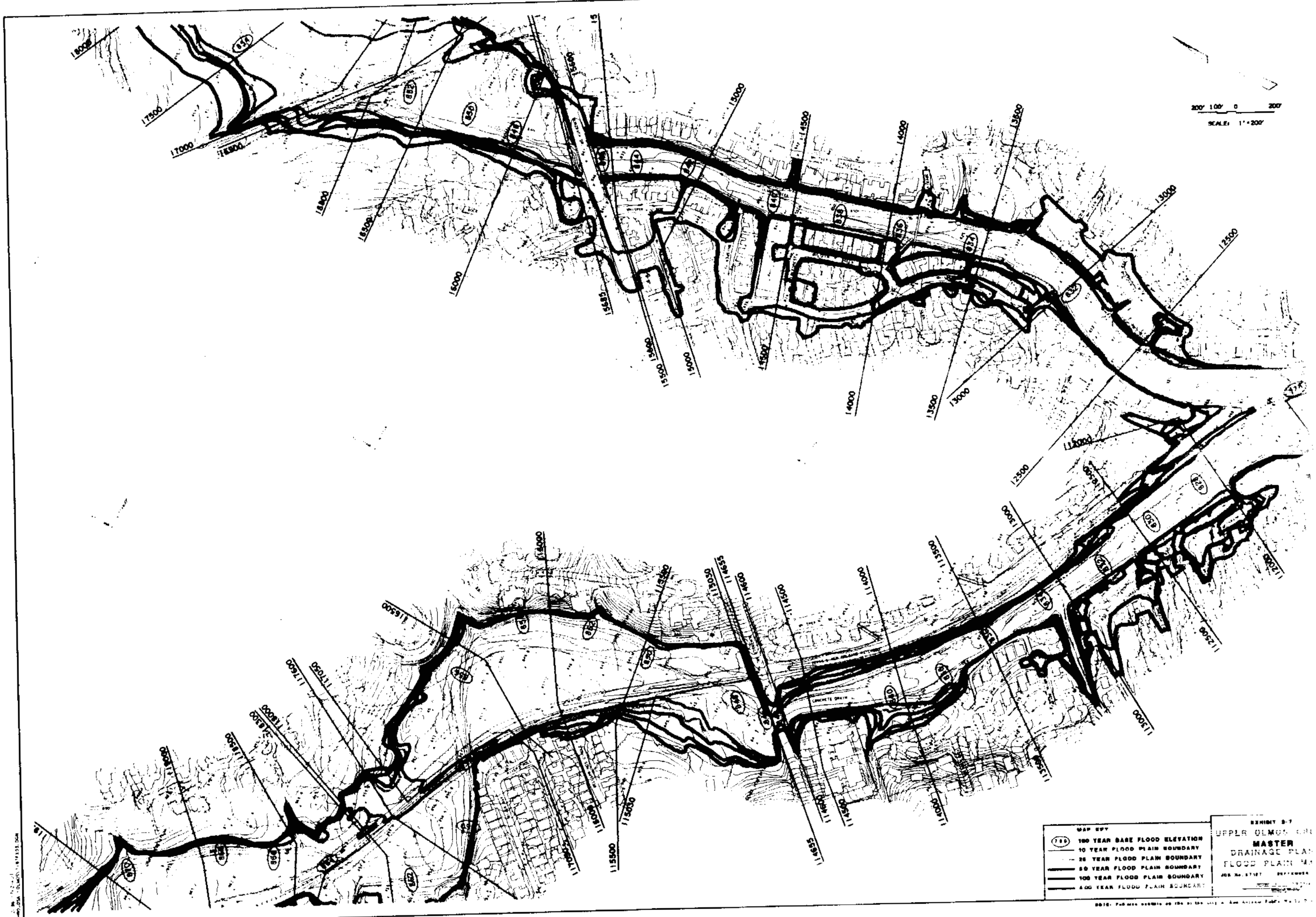
EXHIBIT 3-7
MASTER
DRAINAGE PLAN
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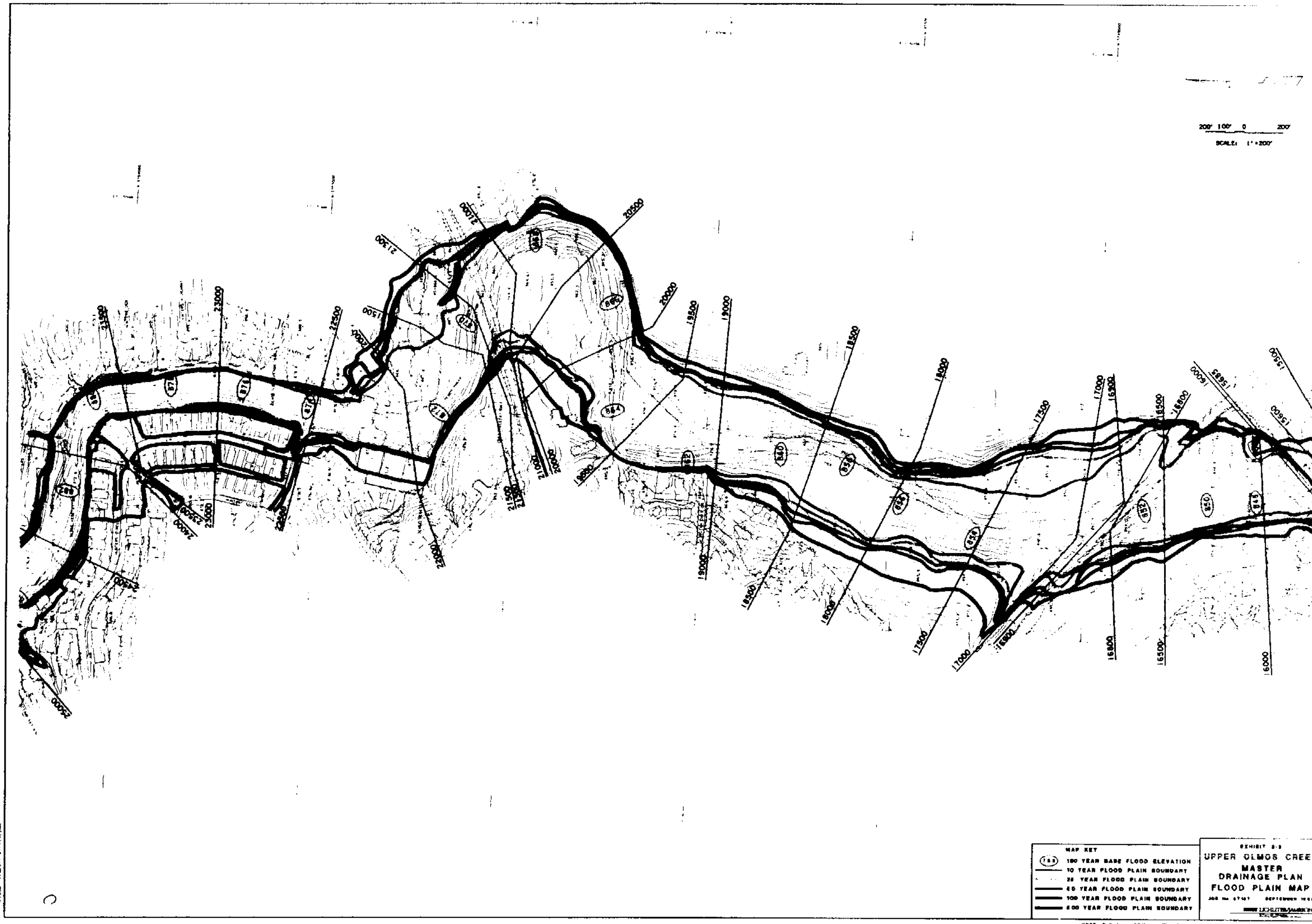
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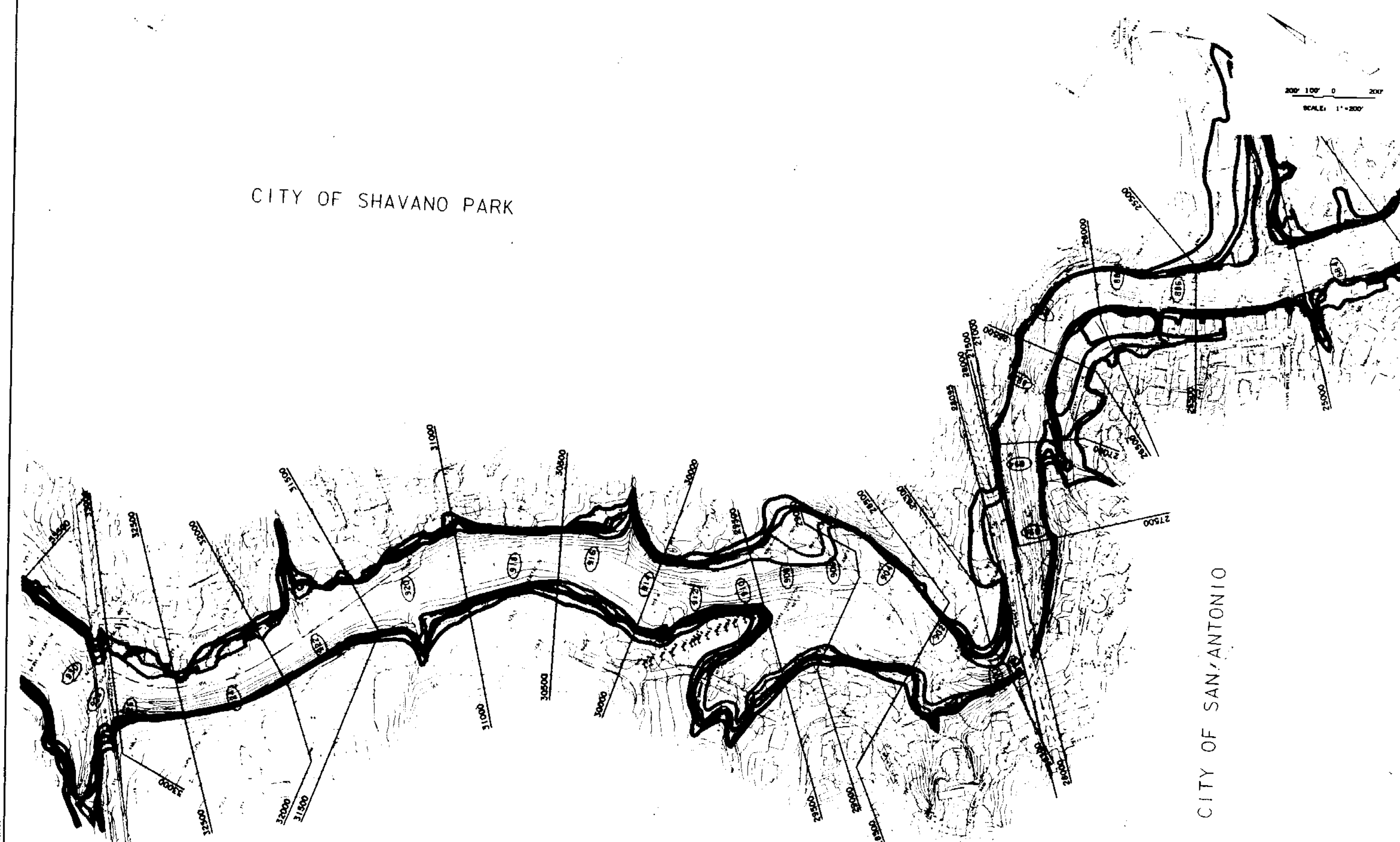
UPPER OLMOS CREEK
MASTER
DRAINAGE PLAN
FLOOD PLAIN MAP



MAP KEY

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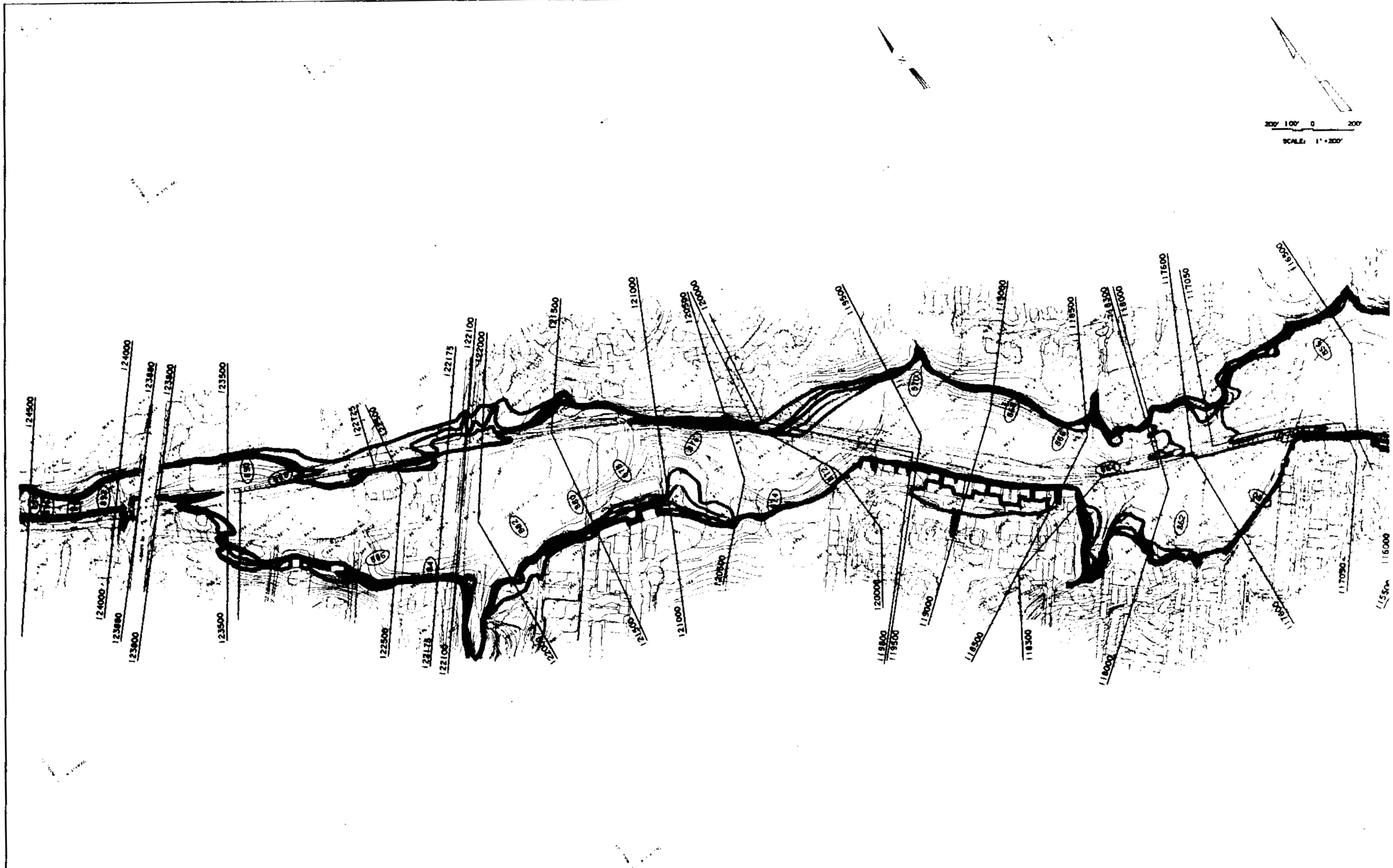
- 10 YEAR FLOOD PLAIN BOUNDARY
- 25 YEAR FLOOD PLAIN BOUNDARY
- 50 YEAR FLOOD PLAIN BOUNDARY
- 100 YEAR FLOOD PLAIN BOUNDARY
- 500 YEAR FLOOD PLAIN BOUNDARY

EXHIBIT 5-9

UPPER OLMOS CREEK MASTER DRAINAGE PLAN FLOOD PLAIN MAP

JOB No. 87147 FEBRUARY 1989

NOTE: Full size exhibits on file at the City of San Antonio Public Works Department

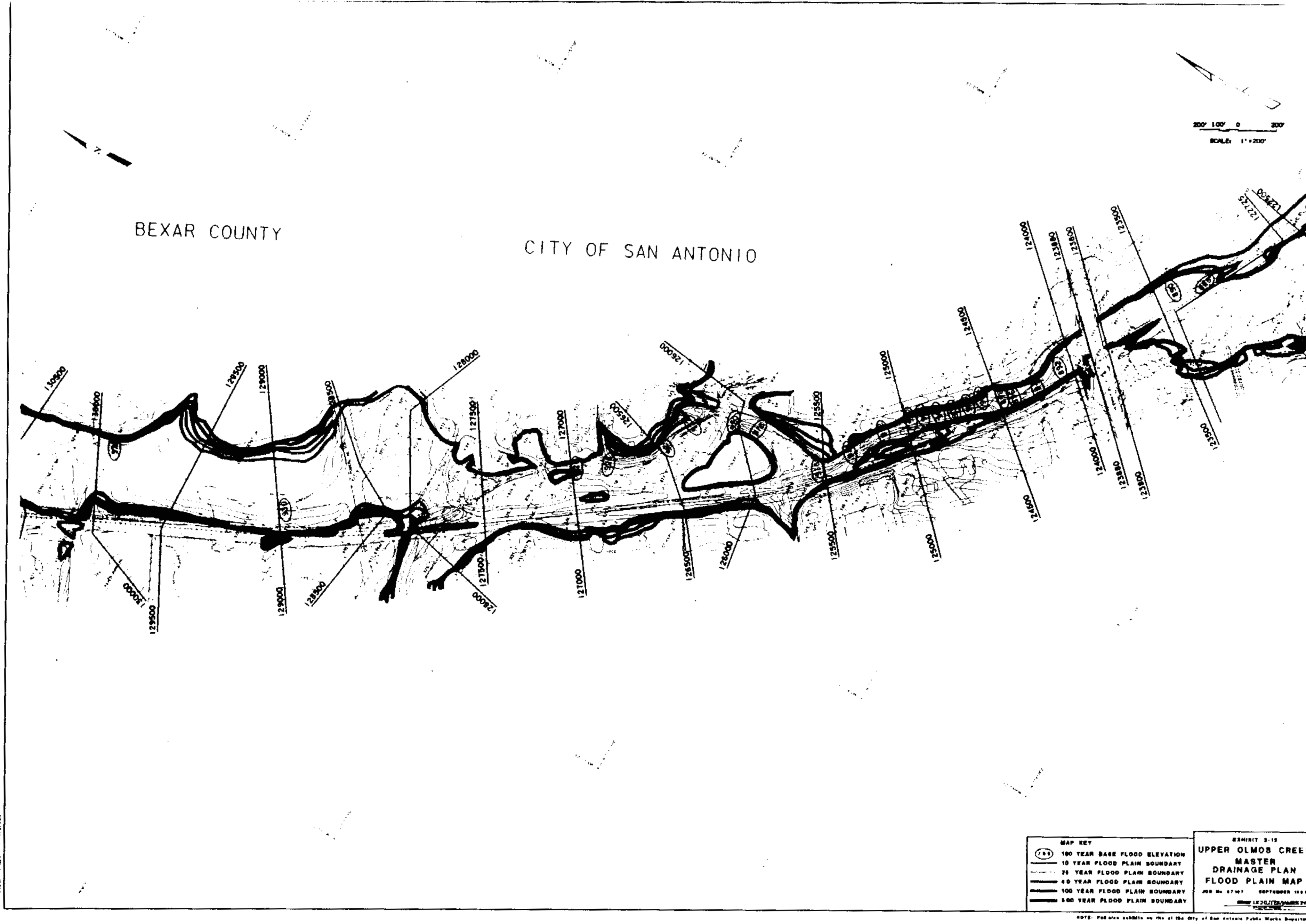


MAP KEY
- 100 YEAR BASE FLOOD ELEVATION
- 10 YEAR FLOOD PLAIN BOUNDARY
- 25 YEAR FLOOD PLAIN BOUNDARY
- 50 YEAR FLOOD PLAIN BOUNDARY
- 100 YEAR FLOOD PLAIN BOUNDARY
- 500 YEAR FLOOD PLAIN BOUNDARY

EXHIBIT 2-11

UPPER OLMOS CREEK
MASTER
DRAINAGE PLAN
FLOOD PLAIN MAP

NOTE: Full-size exhibits on file at the City of San Antonio Public Works Department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CITY OF SAN ANTONIO
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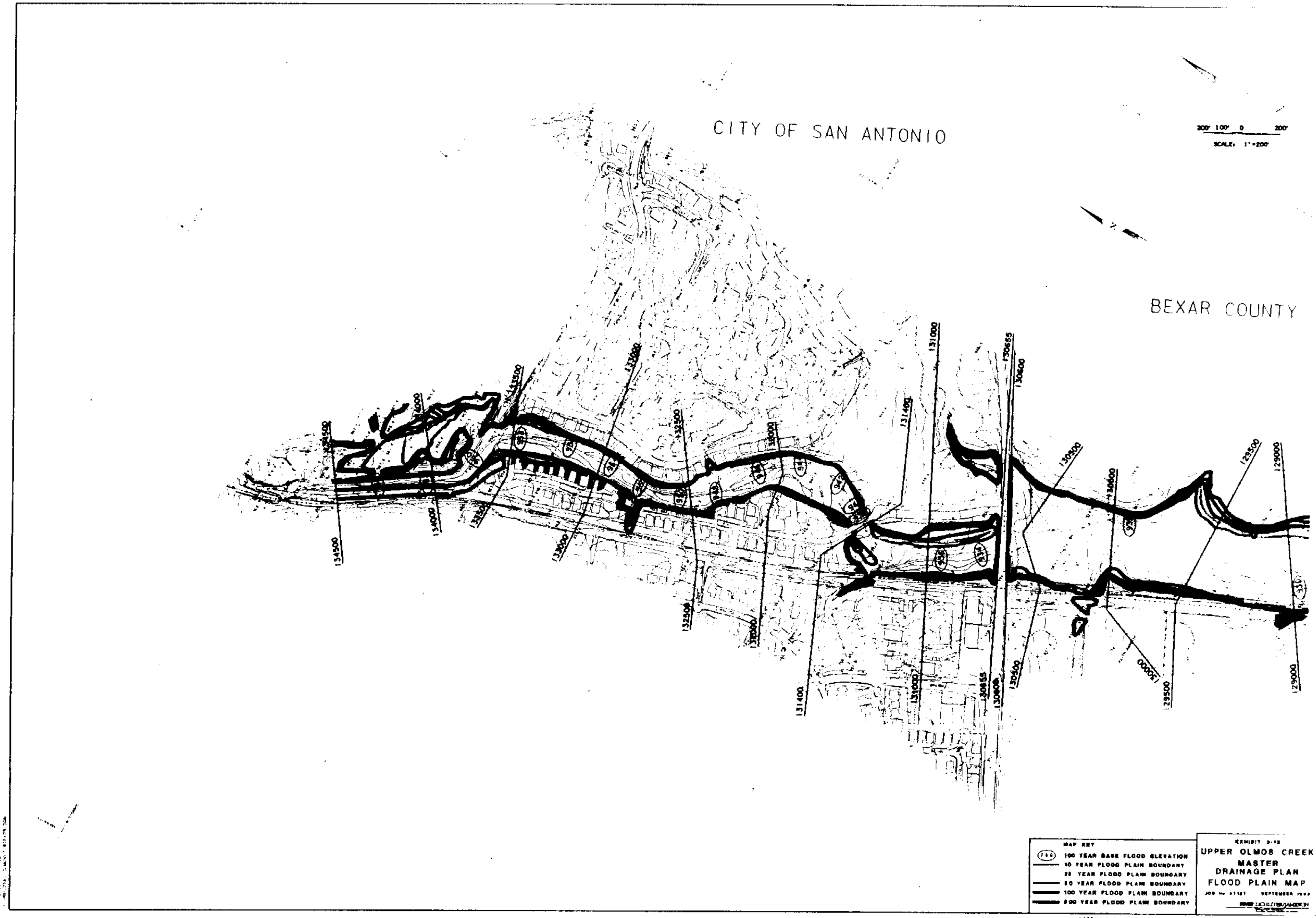
EXHIBIT 3-12
UPPER OLMOS CREEK
MASTER
DRAINAGE PLAN
FLOOD PLAIN MAP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MAP KEY
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10 YEAR FLOOD PLAIN BOUNDARY
25 YEAR FLOOD PLAIN BOUNDARY
50 YEAR FLOOD PLAIN BOUNDARY
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UPPER OLMOS CREEK
MASTER
DRAINAGE PLAN
FLOOD PLAIN MAP

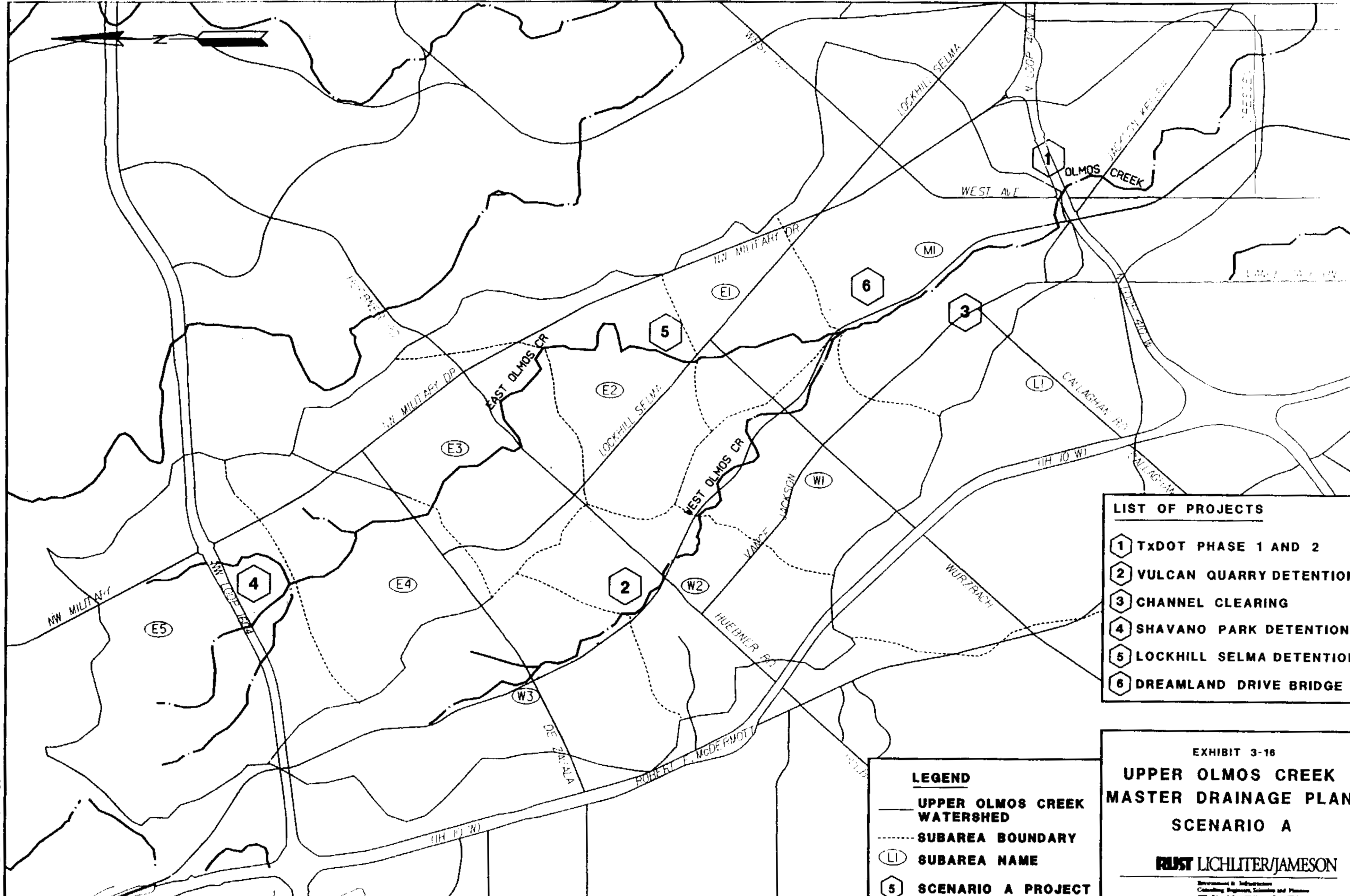

EXHIBIT 3-16

UPPER OLMOS CREEK MASTER DRAINAGE PLAN SCENARIO A

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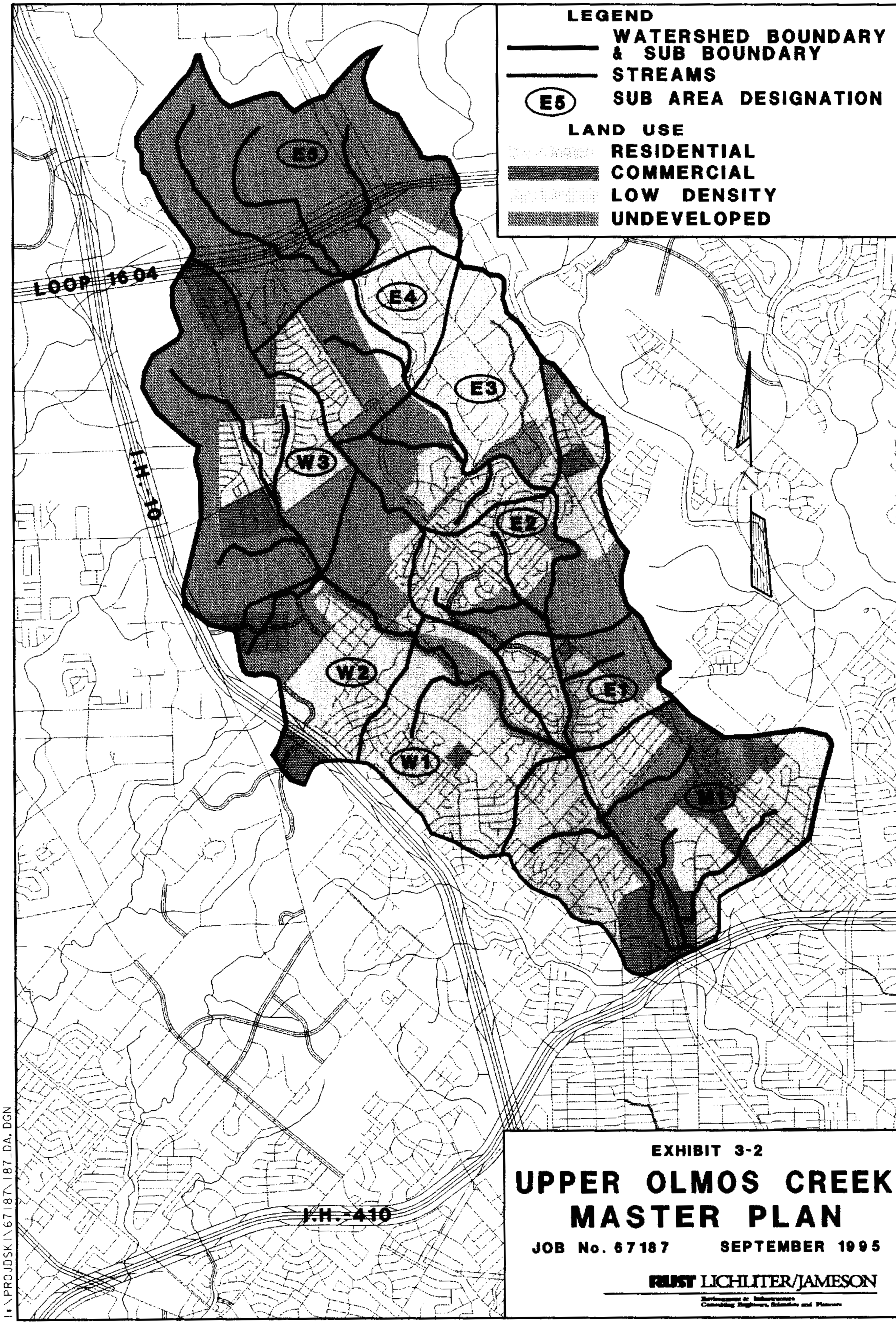

EXHIBIT 3-2

UPPER OLMOS CREEK MASTER PLAN

JOB No. 67187 SEPTEMBER 1995

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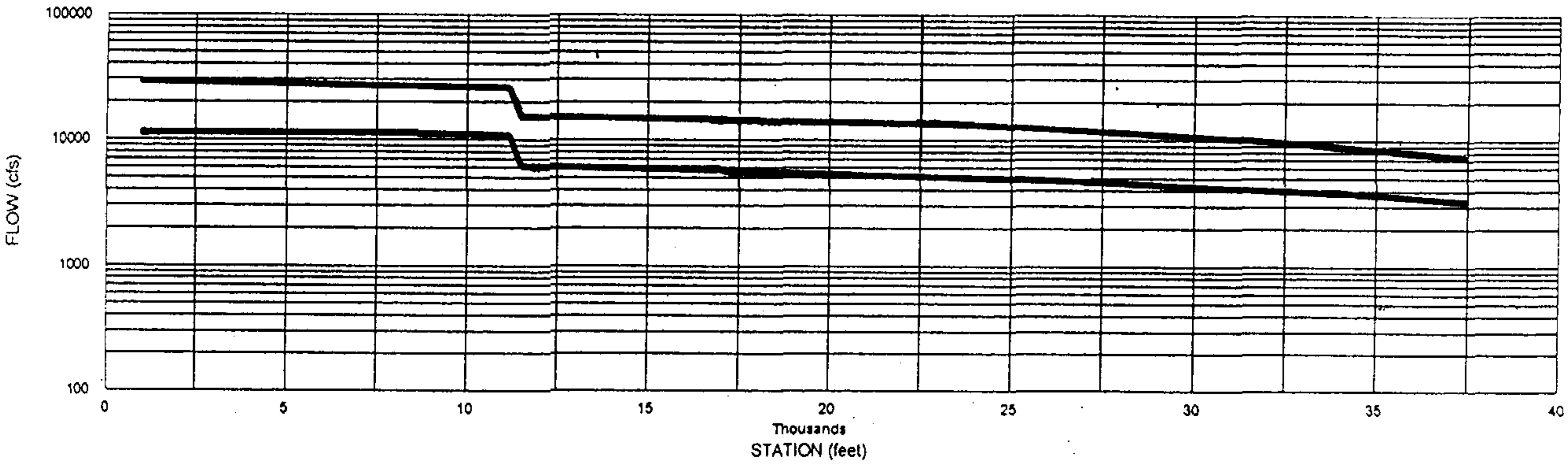
STATION DISCHARGE CURVE

Main and East Fork Channel



STATION DISCHARGE CURVE

WEST FORK CHANNEL

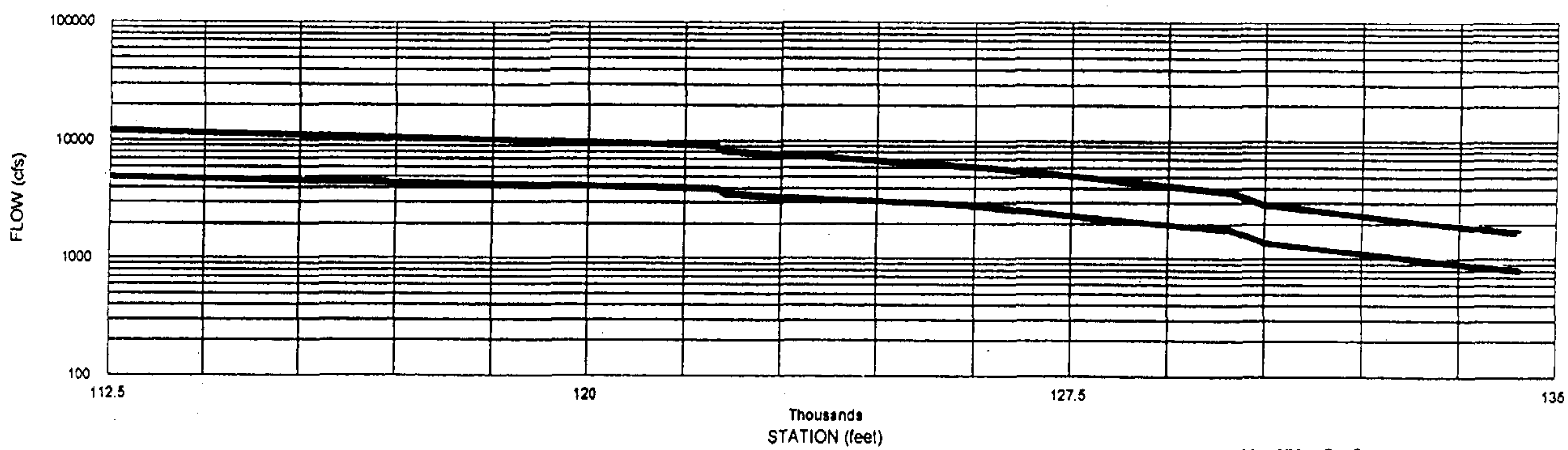


EXHIBIT 3-3

10-YEAR AND 500-YEAR FLOW DISTRIBUTION

SEPTEMBER, 1995

RUST LICHLITER/JAMESON

Environment & Infrastructure

Consulting Engineers, Scientists and Planners

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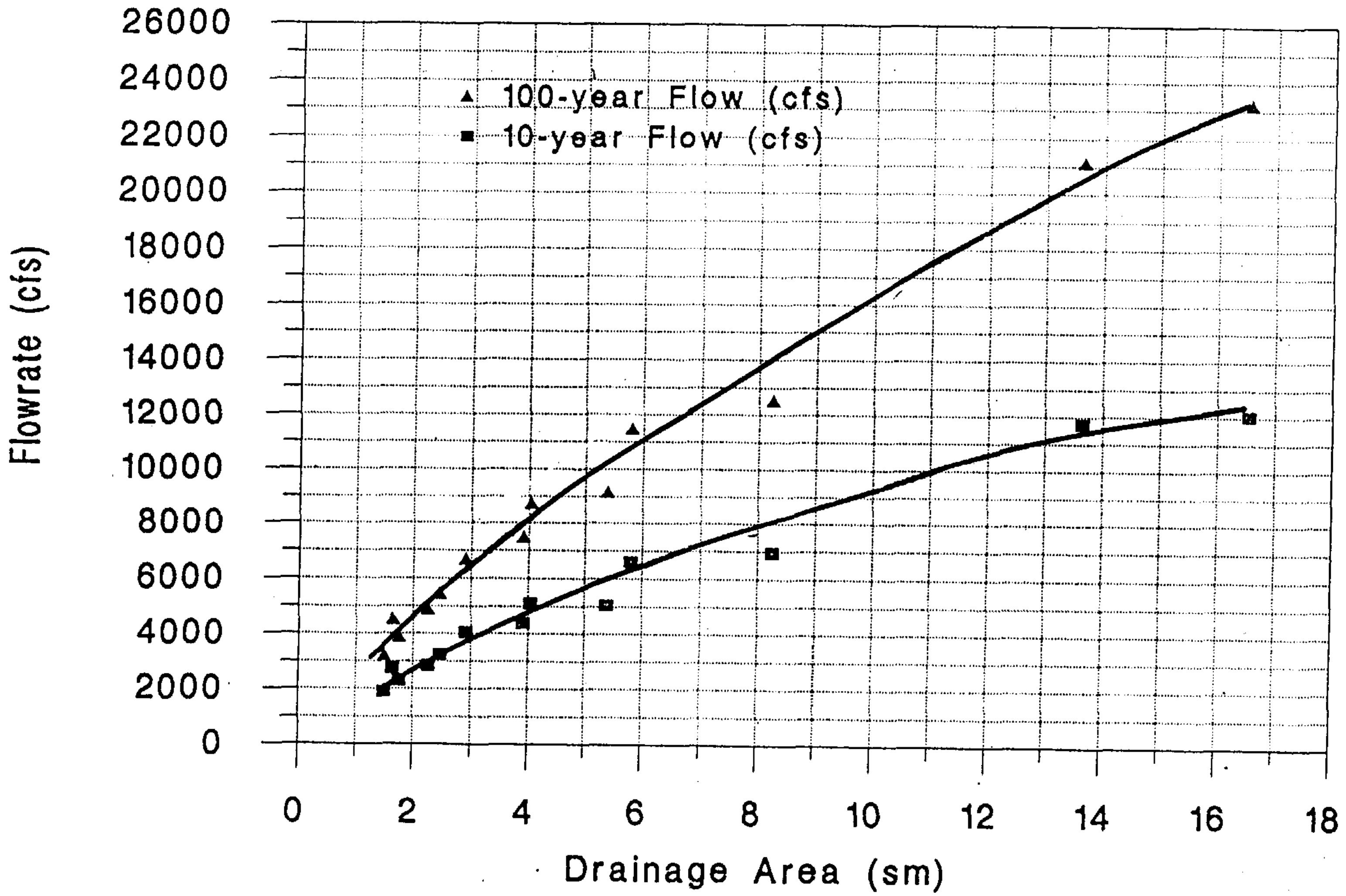

EXHIBIT 3-4

DRAINAGE AREA VS. FLOWRATE GRAPH

SEPTEMBER, 1995

RUST LICHLITER/JAMESON

Environment & Infrastructure
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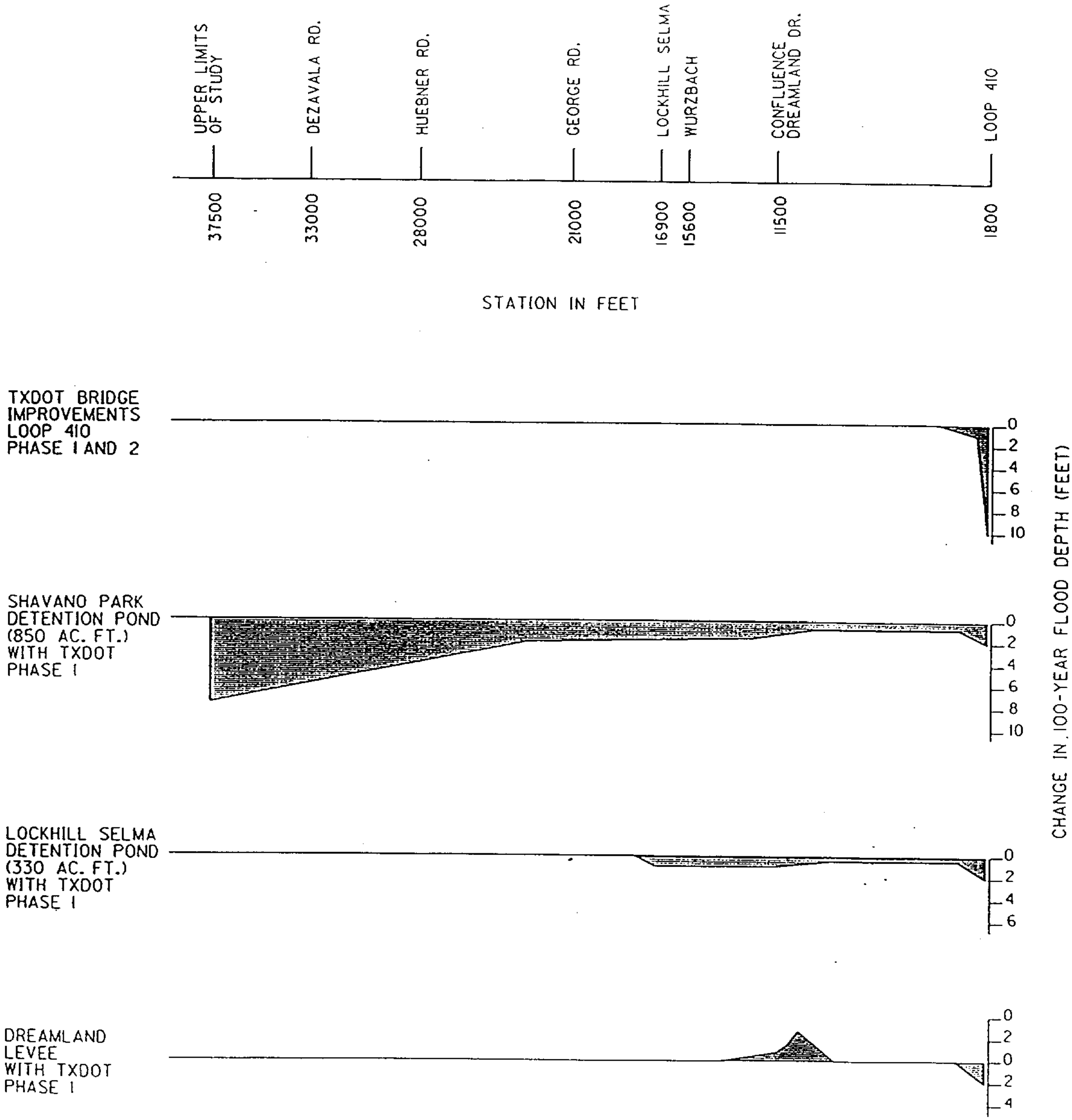

EXHIBIT 3-14

UPPER OLMOS CREEK MASTER DRAINAGE PLAN

COMPONENT ANALYSIS - EAST FORK

RUST LICHLITER/JAMESON

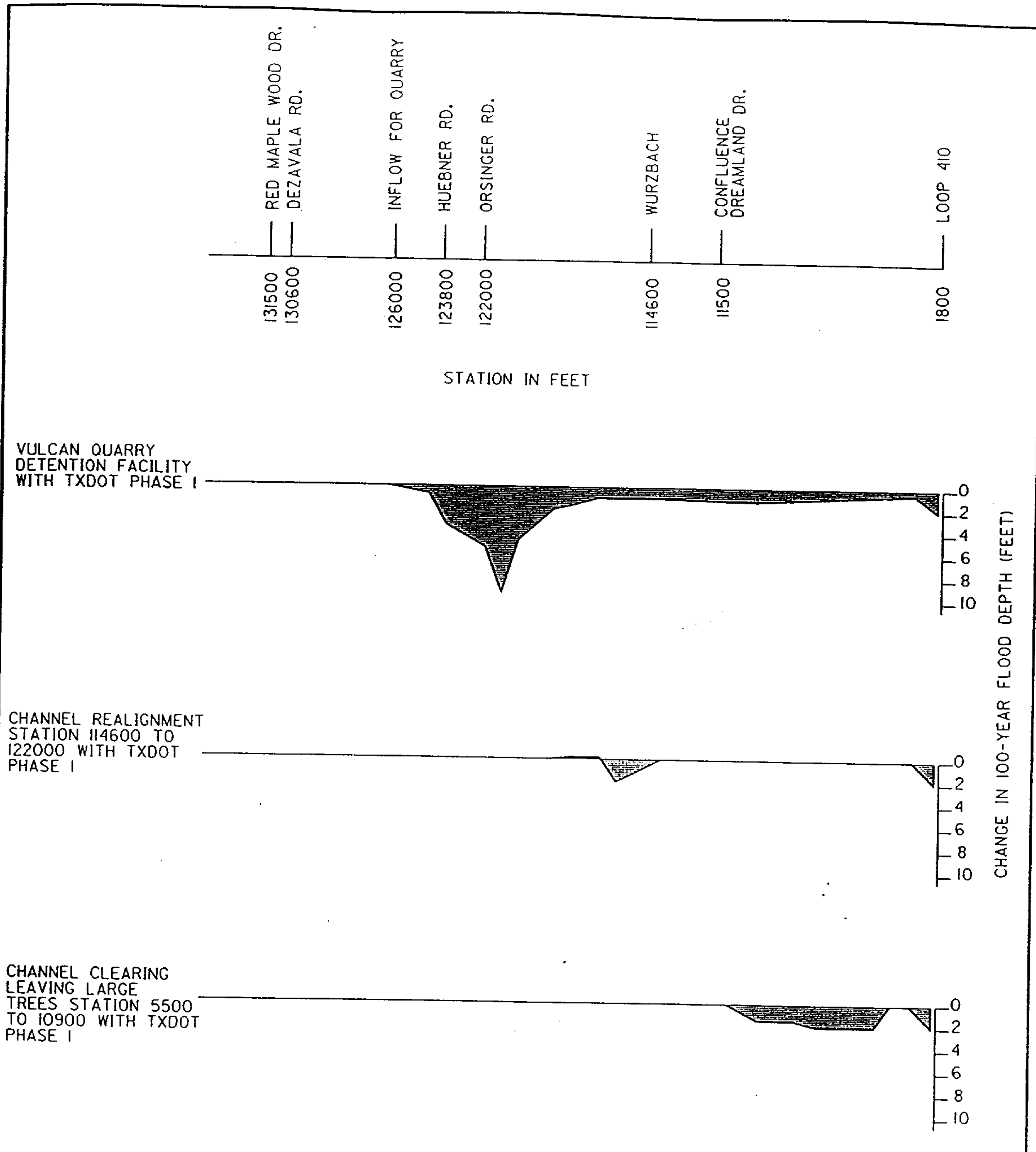

EXHIBIT 3-16

UPPER OLMOS CREEK MASTER DRAINAGE PLAN

COMPONENT ANALYSIS - WEST FORK

RUST LICHLITER/JAMESON

4.0 FINANCIAL PLAN

4.1 Project Cost

As a result of the Drainage Regulations Review Committee presentation, City of San Antonio engineering staff review and the public meetings, Scenario A, as described in Section 3.0 of this report, is the recommended Master Drainage Plan for Upper Olmos Creek. During the Design Phase of the project, a preliminary cost estimate was developed for the Plan as presented in Table 3-15. The individual component costs were estimated using information supplied by the City, construction costs derived from similar projects and the experiences of engineering firms providing similar services in Texas. The costs are based on 1995 construction costs and have not been adjusted to reflect annual inflationary influences for construction at some time in the future, or to reflect the costs associated with interest on bond funds or loans necessary to fund the projects. The costs shown also do not include the cost for future maintenance of the projects. The costs listed in detail in Table 3-15 are summarized by component in Table 4-1.

TABLE 4-1
SUMMARY OF PRELIMINARY COST ESTIMATE
MASTER DRAINAGE PLAN FOR UPPER OLMOS CREEK

DESIGN COMPONENTS	CONSTRUCTION COST	LAND COST	TOTAL COST
1. TxDOT Phase 1 & 2	Paid by TxDOT	$0	$0
2. Vulcan Quarry Detention	$1,147,125	$850,000	$1,997,125
3. Channel Clearing Lower Reach	$221,778	$0	$221,778
4. Shavano Park Detention	$4,211,478	$1,500,000	$5,711,478
5. Lockhill Selma Detention	$1,605,210	$900,000	$2,505,210
6. Dreamland Bridge	$2,676,625	$0	$2,676,625
SUBTOTAL INFRASTRUCTURE			**$13,112,216**
7. Buy-out Remaining Houses	$0	$3,825,000	$3,825,000
TOTAL COST OF PLAN			**$16,937,216**

4.2 Funding Sources

Traditional sources of funding for flood plain management projects in Texas include the use of general obligation bonds and development impact fees. In San Antonio, general obligation bonds approved by the citizens are used to fund capital improvement projects. Bond funds have already been approved for use on several of the Upper Olmos Creek flood control projects identified in the Master Drainage Plan.

In the early 1980's, the Texas Legislature authorized the use of development impact fees to fund drainage improvement projects. The concept is based on the premise that public funds are allocated for projects which solve existing flooding problems and that new developments contribute a per-acre impact fee for regional facilities (channel improvements and detention reservoirs) required to offset the impact of the new development. The use of impact fees to fund the projects identified in the Master Drainage Plan for Upper Olmos Creek is not practical due to several considerations:

1. The watershed is heavily urbanized. The limited amount of new development which will occur in the watershed is small in comparison to the existing development, limiting the amount of fees which could be collected.

2. The impact from new developments on total stormwater runoff in the watershed is very small. In Section 3.3 an analysis of ultimate development is described which concludes that full development of the watershed would only increase the 100-year water surface elevations in the channel less than a tenth of a foot. This is due to the relatively limited amount of new development possible in the watershed as well as the high runoff rates already associated with the soil conditions in the watershed.

3. The infrastructure projects recommended for construction in the Master Drainage Plan are required to mitigate the existing flood plain and their respective mitigation capacities are not impacted significantly by increased runoff from new development in the watershed.

Other possible funding sources in San Antonio include Federal cost-sharing through the United States Army Corps of Engineers and state and local funds from agencies such as the San Antonio River Authority, the San Antonio Water Supply, the Texas Water Development Board Fund, and various transportation related agencies (for funding of drainage projects associated with roadways). Cooperative funding or in-kind services may also be provided from Bexar County, the City of Shavano Park, the City of Castle Hills and state and/or local parks and recreation agencies.

4.3 Implementation Plan

The recommended Master Drainage Plan for Upper Olmos Creek may be implemented by the City of San Antonio in a series of phases. The Texas Department of Transportation initiated construction of the Phase 1 channel improvement project downstream of Loop 410 in the Fall of 1995. Phase 2 of the TxDOT project is not scheduled to be implemented within the next five-year construction period; however, the City of San Antonio is discussing with TxDOT the possibility of moving this project up to an earlier schedule.

The City of San Antonio also began negotiations in 1995 to obtain the Vulcan Materials Quarry on the West Fork of Olmos Creek, as well as the 55 acre site required to implement the detention facility on the East Fork near the City of Shavano Park. In addition, replacement of Lockhill Selma with an all-weather crossing has already been approved for funding through an earlier City of San Antonio bond election, and the project is scheduled for design and construction in 1996.

Table 4-2 shows a possible implementation schedule for the recommended Master Drainage Plan assuming completion of the infrastructure projects within a ten year time frame. This schedule assumes the funding is available for each year's scheduled projects and that the City of San Antonio receives necessary support from adjacent municipalities and Bexar County as necessary prior to the initiation of a scheduled project. The entire Master Drainage Plan can be implemented within the ten year time frame shown in Table 4-2 at a capital expenditure in 1995 dollars of between $0.2 and $2 million per year. The typical annual expenditure on construction projects is in the range of $1.0 to $2.0 million. If additional funding is available, the schedule can be accelerated to fit into a shorter time period of approximately six years without violating the constraints of the critical path.

TABLE 4-2
IMPLEMENTATION SCHEDULE FOR INFRASTRUCTURE
MASTER DRAINAGE PLAN FOR UPPER OLMOS CREEK

YEAR OF CONSTRUCTION (YEAR 1 = 1996)	1	2	3	4	5	6	7	8	9	10
1. TxDOT Phase 1 and 2	■						■	■		
2. Vulcan Quarry Detention		░	░							
3. Shavano Park Detention				■	■	■				
4. Lockhill Selma Road and Detention	░						■			
5. Channel Clearing										░
6. Dreamland Bridge								░	░	
Annual Construction Cost ($ Millions 1995)	1.6	1.0	1.0	1.7	2.0	2.0	0.9	1.3	1.4	0.2

The critical path for construction of the projects is shown by highlighting the controlling projects in bold type. These projects must be implemented in a specific order to avoid hydraulic problems and adverse impacts in the drainage system. For example, the Shavano Park detention basin must be constructed prior to constructing a constriction at Lockhill Selma which would back-up water behind the road during the 100-year storm event. This is due to the very limited amount of freeboard between the existing 100-year flood plain and the houses upstream of Lockhill Selma. By detaining water in the Shavano Park facility, the 100 year water surface elevation upstream of Lockhill Selma is lowered, thereby allowing the flow of water under the road to be constricted with culverts in order to create a ponding area upstream of the road.

Without the Shavano Park detention facility in place, the 100 year flood must be allowed to flow freely under the Lockhill Selma all-weather crossing in order to maintain the current flood plain level upstream. If the road is reconstructed as an all-weather crossing in 1996 as anticipated, the culverts under the roadway can be designed and constructed to allow full passage of the 100 year flood event, while also allowing for modifications to be made to constrict the flow when reconfigured as a detention facility (shown on Table 4-2 as occurring in Year 6 following construction of the Shavano Park facility).

Similarly, the TxDOT Phase 1 and 2 channel improvements must be completed before any channel clearing project is constructed. A large flood plain currently exists immediately upstream of Loop 410, indicating that structural flooding in this area would be aggravated by any increase in flows resulting from channel clearing upstream.

The approximately $13.1 million total cost for the infrastructure projects identified in the recommended plan may be reduced to $12.9 million if the City of San Antonio elects not to include the channel clearing project from Station 2800 to 11200 (lower reach of Olmos Creek below the confluence of the East and West Forks). The clearing project lowers the water surface elevation of the 100 year frequency event by as much as two feet downstream of the confluence; however, according to the map shown in Exhibit 3-6, this component only removes one additional house from the 100 year flood plain. If the slab elevation of this house on Old Brook near channel Station 5500 is verified by the City to be above elevation 796.6 feet, it is out of the 100 year flood plain as a result of the rest of the Scenario A components without the channel clearing project. In this case, the channel clearing project could be eliminated from the recommended Master Drainage Plan for Upper Olmos Creek. If the house slab is below elevation 796.6 feet, a second option would be for the city to buy-out this house instead of implementing the channel clearing project. Although the channel clearing project appears to be a relatively low-cost project, it should be remembered that the costs for the project estimated in this study do not account for the continued maintenance of the cleared channel on a regular basis.